DRAFT REPORT

CRITICAL EVALUATION OF THE DESIGN CODE REQUIREMENTS FOR DEVELOPMENT LENGTH OF PRESTRESSING TENDONS

by

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ABSTRACT

This report deals with a review of tests conducted at the Structural Research Center of the Florida Department of Transportation (FDOT) on the development length of low relaxation prestressing strand. The main variables in the tests were strand size and embedment length as determined by the location of a test load from the end of a member. Concrete strength was not intentionally varied. Tests were conducted on solid and voided prestressed slabs, AASHTO Type II girders and piles.

The development lengths determined in the tests are compared with results based on a formula proposed by FDOT as well as other proposals, including one by the Federal Highway Administration (FHWA).

The study indicates that the FDOT proposal gives quite satisfactory results for all the types of members tested. It is concluded that the FHWA proposal is too conservative and requires further refinement.

The use of a 1.6 multiplier with the basic AASHTO equation for development length, yields satisfactory results for the girders, while the basic AASHTO equation is more appropriate with slender members such as slabs and piles.

The study confirms a recent finding by Buckner that the strain in the strand at ultimate is an important factor affecting flexural development length. However, indications are that Buckner's proposal requires some refinement.

A proposal by FDOT for calculating the transfer length of strand is recommended for general adoption. It is also recommended that a minimum spacing of 2 in. should be adopted for strands up to 0.6 in. diameter.

ACKNOWLEDGEMENTS

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Mr. Tom Beitelman and Mr. Adnan El Saad were responsible for the test setup, assembling the data and presenting them in tabular or plotted form as requested by the investigator. They worked tirelessly to produce the information required. It was a pleasure to work with both of them and their contribution is

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NOTATION

a	=	Shear span length
D	=	Nominal diameter of prestressing strand
d	=	Effective depth from compression face to center of gravity of prestressed reinforcement in tension zone
f_c'	=	Specified compressive strength of concrete
f_{pu}	=	Specified tensile strength of prestressing strand
f_{se}	=	Effective stress in prestressed reinforcement after all losses
f_{si}	=	Stress in prestressed reinforcement at time of initial prestress, i.e. immediately after release
f_{su}^*	=	Stress in prestressed reinforcement at nominal strength
h	=	Overall thickness of member
k_b	=	Constant used in FDOT expression for development length
L	=	Span of member
L_b	=	Flexural bond length
L_d	=	Development length
L_t	=	Transfer length
M_a, M_{app}	=	Applied moment
M_n	=	Nominal Moment strength
M_u	=	Design moment
μ_{ave}	=	Average flexural bond stress
β_1	=	Ratio of depth of equivalent rectangular stress block to depth of neutral axis
ε_{ps}	=	Strain in prestressed reinforcement at nominal strength
ρ_p	=	Ratio of prestressed reinforcement to effective depth times width of compression face
λ	=	Factor applied to flexural bond length
ω_n	=	Reinforcement index $\frac{\rho_p f_{su}^*}{\quad}$

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1. INTRODUCTION AND BACKGROUND

The development length provisions for prestressing strands in the AASHTO Code have generated much discussion since 1986, when Cousins, Johnson and Zia reported the findings of their tests[1]. They indicated that the existing AASHTO provisions for development length of prestressing strands were inadequate. As a result of this investigation, the Federal Highway Administration (FHWA) imposed a multiplier of 1.6 on the AASHTO equation for development length of strands up to 9/16 inch diameter, and restricted minimum spacing of strands to four times their nominal diameter.

The application of such a large multiplier has raised questions regarding its validity, and has spurred research by many investigators. The Florida Department of Transportation (FDOT) has questioned whether the use of a multiplier is necessary, particularly in the design of prestressed concrete piles. Consequently, extensive research has been conducted by the FDOT Structures Research Center (FSRC) on transfer and development lengths of strands in prestressed concrete girders, slabs and piles. The findings of these studies, and recommendations arising there from, are summarized in this report.

Buckner presented a detailed review of proposals for calculating transfer and development lengths[2] in 1995. The study recommended the acceptance of the FDOT proposals for calculating transfer length of strand, and prescribed yet another modification of the AASHTO method for calculating development length.

In 1998 the FHWA published new proposals for transfer and development lengths for use in bridge beams and piles[3]. The objective, as stated in this latter study, was to provide more conservative predictions of transfer and development lengths for all concrete strengths, including high strength concrete. It was argued that existing proposals reflect average values for concrete of normal strength, and are unconservative.

This report examines the relevance of recently proposed modifications of methods presented in the AASHTO design specifications for calculating transfer and development lengths. It presents proposed modifications based on a number of investigations conducted at FSRC.

1.1 CURRENT AASHTO PROVISIONS

The current AASHTO provisions for development length of prestressing strand require that three or seven-wire strands be bonded beyond the critical section for a development length, L_d, in inches, not less than:

$$L_d = (f^*_{su} - 2/3 f_{se}) D \tag{1}$$

The term, D, in Eqn. (1), is the nominal diameter of the strand in inches; f^*_{su} is the stress in prestressed reinforcement at nominal strength, and f_{se} is the effective stress in prestressed reinforcement after all losses. Both stresses are expressed in units of ksi. The parenthetical expression in Eqn. (1) is considered to be without units.

It is also specified that investigation may be limited to sections nearest the end of the member that are required to develop their full ultimate capacity.

Where a strand is debonded at the end of a member and tension at service load is allowed in the prestressed tensile zone, it is specified that development length required above shall be doubled.

Eqn. (1) can be written in the form:

$$L_d = (f_{se}/3)D + (f^*_{su} - f_{se})D$$

Equation (2) is shown plotted in Figure 1. It can be seen that the first term in Eqn. (2) represents the transfer length, which is the distance over which the strand must be bonded to the concrete to develop the effective prestress, f_{se}, in the strand. The second term represents the additional length, termed the flexural bond length, required to develop the nominal strength, f^*_{su}, of the strand. The value of f_{se} obviously depends on the stress, f_{si}, in prestressing steel at transfer, and the amount of prestress loss. Zia and Mostafa[4] have pointed out that the denominator "3" in the expression for transfer length represents a conservative average concrete strength in ksi.

Similarly, in the expression for flexural bond length in Eqn. (2), a denominator of 1.0 ksi is implied, which is a factored value of an average bond stress of 250 psi over the flexural bond length.

According to the AASHTO Specifications, the transfer length and the flexural bond length for 270 ksi strand would be respectively, 54 and 103 times the nominal strand diameter, assuming that $f_{si} = 0.75\ f_{pu}$, a prestress loss of 20%, and $f^*_{su} = 0.98\ f_{pu}$ (where f_{pu} is the specified tensile strength of prestressing strand, ksi).

1.2 MODIFICATIONS PROPOSED TO AASHTO PROVISIONS

From a comprehensive study of past research, Zia and Mostafa[4], proposed the following expressions for transfer length, L_t, flexural bond length, L_b, and development, L_d:

$$L_t = (1.5 f_{si} D / f'_{ci}) - 4.6 \qquad (3)$$

$$L_b = 1.25 (f^*_{su} - f_{se}) D \qquad (4)$$

$$L_d = L_t + L_b \qquad (5)$$

where f'_{ci} is the compressive strength of concrete at time of initial prestress, ksi.

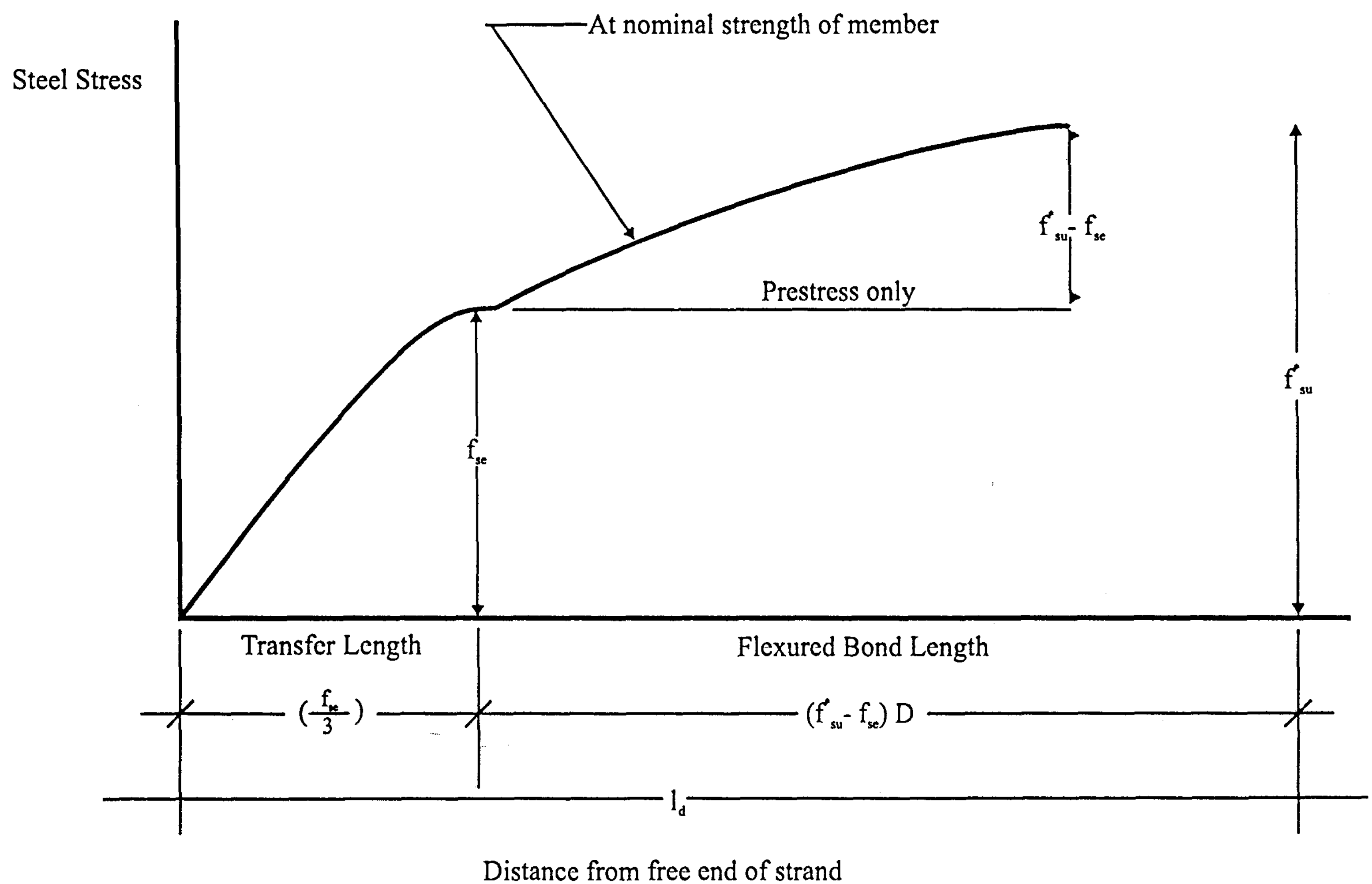


Figure1- Variation Of Steel Stress With Distance From Free End Of Strand

Equation (4) is based on the theoretically derived expression:

$$L_b = (f_{su}^* - f_{se}) D / 4u_{ave}$$

where u_{ave} is the average bond stress within the flexural bond length. Note that, as stated before, it is implied in the current AASHTO Specifications, that u_{av} = 250 psi; however in Eqn. (4) a value of u_{av} = 200 psi is assumed.

The expression for transfer length in Eqn. (3) is assumed to be applicable for concrete strength ranging from 2 ksi to 8 ksi, and accounts for the effects of strand size, initial prestress and concrete strength at transfer. This equation gives transfer length comparable to those specified in the AASHTO Specifications, particularly in cases where the concrete strength at transfer is low. However, a comparison of Eqn. (2) and Eqn. (4) indicates that the AASHTO provisions for L_b, and hence L_d, would be inadequate.

Based on Cousins, Johnston, and Zia's work[1], the FHWA initially required the application of a 2.5 multiplier to AASHTO Eqn. (1), but FDOT initially opposed the use of any multiplier. After further deliberation, a recommendation was made by FDOT to FHWA that the value of the multiplier be reduced to 1.6. At a joint meeting in Philadelphia in 1988, between the AASHTO Technical Committee for Prestressed Concrete and the PCI Committee, the recommendation for a multiplier value of 1.6 was formally presented for use with strands up to and including 9/16-inch diameter strand. The FHWA accepted this recommendation, but retained the restriction on minimum strand spacing.

As a result of an extensive study by FDOT[5] on transfer and development length of strands, it was recommended that the transfer length be calculated as

$$L_t = \left(\frac{f_{si}}{3}\right) D \qquad \textbf{(6)}$$

in which the terra, f_{se}, in Eqn. (2) is replaced by the term, f_{si}.

It should be noted, however, that the transfer length, $(f_{se}/3)D$, in Eqn. (2) specified by AASHTO, is based on experimental studies conducted on stress-relieved 250 ksi strand. Since then, the industry standard has changed to Grade 270 low-relaxation strand, which has a cross sectional area about 6 percent greater than Grade 250 strand of the same nominal diameter. In addition, the use of low-relaxation strand instead of stress-relieved strand, results in higher transfer stresses, and hence a requirement for higher transfer length.

In a recent study by Buckner[2] it was concluded that although there is a wide variation in measured values of transfer length, the value for seven wire low-relaxation strand in normal weight concrete (f'_{ci} = 3500 psi or higher), should be taken as the expression for transfer length proposed by FDOT[5,6] in Eqn. (6). This latter equation was shown to be representative of test results.

Equation (6) can be rationalized[2,5,6] by the fact that the transfer length, which is established at release of prestress, does not exhibit significant change over time. Also, the expression of L_t in terms of f_{si}, rather than f_{se}, is convenient for design purposes.

Generally, the transfer length calculated from Eqn. (3) is about 20 percent longer than that resulting from the use of the current AASHTO prescription. An approximate transfer length of 50D is allowed. Buckner[2] has suggested that this should be increased to 60D to account for the longer transfer length of Grade 270 strands, and to reflect the general findings of recent studies.

In dealing with development length of strand, FDOT[6] has pointed out that the AASHTO requirements are inadequate. In particular, the requirement for the development of nominal flexural strength close to the support is somewhat unrealistic in the case of straight

Since a shear mode of failure is likely in girders in which the shear span/depth ration (a/h) is below 2.5, it was argued that the effect of the a/h ratio should be taken into account in any general expression for development length of strand. It was also stated that a development length of 130 inches. (260D), (for 0.5 in. and 0.5 in. special) to 140 (233D) for 0.6 inch diameter strands would allow the development of the normal design moment. The FDOT study also indicated a direct interaction between shear and bond at the ends of girders, and stated that initial slip of strand occurs immediately or shortly after the appearance of the first diagonal crack.

It was suggested by FDOT that the provisions for development of strand should be cast in the following form:

$$L_d = (f_{si}/3)D + \frac{(f_{su}^* - f_{su})D}{k_b U_{av}} \tag{7}$$

in which

kb is a dimensionless factor, which reflects the actual value of average flexural bond stress that can be developed in particular cases.
U_{av} represents a basic average value of bond stress of 0.25 ksi

Recommended values of k_b are as follows:
k_b= 8, for piles embedded in a footing or a pier cap
k_b= 4, for slabs and slender members (i.e., the original AASHTO prescription applies)

It is recommended that if the ratio of (I,d/h) calculated using k_b= 4, is equal to or less than *3*, the value of kb should be taken as 2.

In a report on his study, Buckner[2] proposed a development length equation for pretensioned strands as follows:

$$L_d = \frac{f_{siD}}{3} + \lambda(f_{su}^* - f_{se})D$$

where $\lambda = (0.6 + 40\varepsilon_{ps})$ is a multiplying factor applied to flexural bond length.

ε_{ps} = strain in prestressed reinforcement at nominal strength corresponding to f^*_{su}.

Buckner states that if the Eqn. [18.3] of the ACI Building Code is used to calculate design stress (in terms of the reinforcement index, $\omega_p = \rho_p f^*_{su} / f'_c$), the equivalent expression for λ is $(0.72 + 0.102\beta_1 / \omega_p)$. Buckner also states that it is reasonable to set an upper limit of 2.0 for λ, when ε_{ps} is well beyond yield, and a lower limit of 1.0 when design strains are below yield.

Thus:

$$1.0 \leq (0.6 + 40\varepsilon_{ps}) \leq 2.0 \qquad \textbf{(9)}$$

In the ACI expressions:

ρ_p = ratio of prestressed reinforcement to effective depth times width at compression face.
β_1 = ratio of depth of equivalent rectangular stress block to depth of neutral axis.

As stated above, the transfer length $\left(\frac{f_{si}}{3}\right)D$ in Eqn. (8) is exactly the same as originally proposed by FDOT.

After extensive statistical manipulation of results from several studies, FHWA has presented[3] the following equations for transfer and development of prestressing strand.

Transfer Length:

$$L_t = \frac{4 f_{pt} D}{f'} - 5 \qquad \textbf{(10)}$$

Flexural Bond Length:

$$L_b = \frac{6.4(f^*_{su} - f_{sc})D}{f'_c} + 15 \qquad \textbf{(11)}$$

Development Length:

$$L_d = \left[\frac{4f_{pt}}{f'_c}(D) - 5\right] + \left[\frac{6.4(f^*_{su} - f_{se})(D)}{f'_c} + 15\right] \qquad \textbf{(12)}$$

in which f_{pt} = stress in prestressing steel prior to transfer of prestress.

Equations (10) - (12) are cast on a format that reflects the effect of f'_c in light of the use of concrete strength higher than usual. FHWA has suggested that for $f'_c > 10{,}000$ psi, a value of $f'_c = 10{,}000$ psi should be substituted in the above equations.

It is also recommended that Eqns. (10) and (12) be used for piles, and that a 1.3 multiplier be applied to any strands (straight or draped) in any member (beam or pile) that has 12 inches or more of concrete cast beneath them. This latter recommendation, which was also proposed by Buckner[2], is intended to address the lower bond strengths that can be developed in so-called "top reinforcement."

The development of Equations (10) - (12) was based on statistical analysis of reported test results. However, the physical significance of the transfer length equation of Eqn. (10) is open to question, based on the fact that for a value of $f_{pt} = 0$ (non-prestressed concrete), a value of $L_t = -5$ inches would result. The format of Eqn. (10) should preferably be such that it passes through the origin. A similar comment applies to the format of Eqn. (4) proposed by Zia and Mostafa.

1.3 FDOT TESTS

Since 1990 the FDOT Structures Research Center has conducted an extensive study 5,6 of the behavior of prestressed concrete members namely:

- Prestressed Voided and Solid Slabs
- Prestressed Concrete Girders
- Prestressed Concrete Piles

In this report, comparisons will be made between FDOT test results and the transfer and development lengths of strands predicted by the expressions presented in this section.

1.4 OBJECTIVES OF STUDY

The objectives of the present study are as follows:

1) Review and compare development length requirements specified by the AASHTO Code, and the recommendation by FHWA in its 1998 report.

2) Comparison of FHWA results with findings from tests conducted by FDOT.

3) Review and evaluation of the FDOT proposals for development length.

2. SOLID AND VOIDED PRESTRESSED SLABS

2.1 TEST SPECIMENS AND PROCEDURES

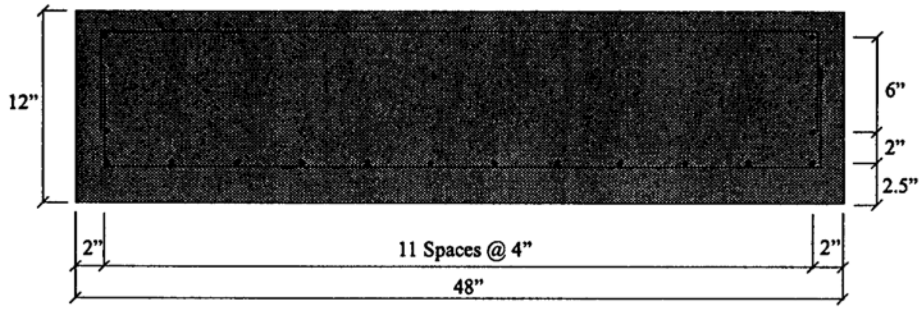
Seven full-scale prestressed voided and solid slabs were tested to failure. Details and dimensions of the cross sections are shown in Figures 2 and 3.

The main objective of the test program was to determine the minimum development length of unshielded $^1/_2$' dia, 270 ksi, low-relaxation prestressing strand.

The prestressed concrete slabs were tested under static loads applied incrementally to failure, with the location of the loading varied as shown in Figures 4 and 5. It should be pointed out that the development length based on AASHTO provisions would be approximately 77 inches (154 D). Application of the FHWA multiplier of 1.6 would increase this length to 123 inches (246 D), which renders these types of members unusable over short spans.

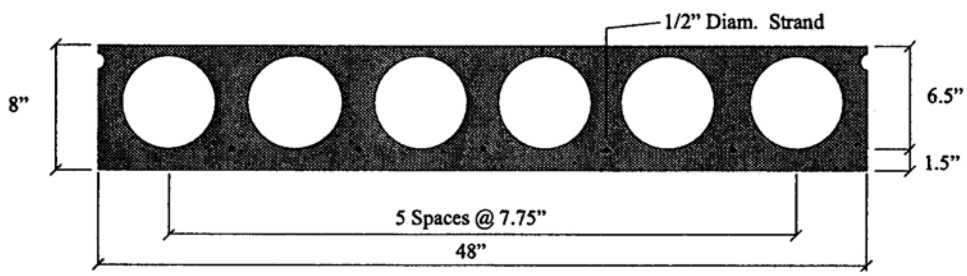
Figure 4 shows that the solid slabs were loaded with two symmetrically applied point loads, and that three sets of tests were conducted on the solid slabs.

Figure 5 shows that the first two voided slabs (VS1 and VS2) were loaded with a single point load applied at varying distances from the south end of the member. The member was loaded beyond its nominal moment capacity, or until significant strand slip occurred. The loading assembly was then removed and placed at a set distance from the other end. The remaining two voided slabs (VS3 and VS4) were loaded with symmetrical two-point loading. Thus, each voided slab provided two test points for a total of eight points. The load points were such that the a/d ratios were greater than 5.0 for all specimens, and therefore well above the range that would result in shear failures, i.e. the tests were essentially flexural tests.



- 16-1/2" Dia. 270 Ksi- Lo-Lax Strands
  Pulled to 31,000 lbs. each
- Design Concrete Strength at 28 day = 5,000 psi

Figure 2 - Solid Slab Cross Section (SS)



- 5- 1/2" Dia. 270 Ksi - Lo Lax Strands
  Pulled to 31,000 lbs. each
- Design Concrete Strength at 28 days = 5,000 psi

Figure 3 - Voided Slab Cross Section (VS)

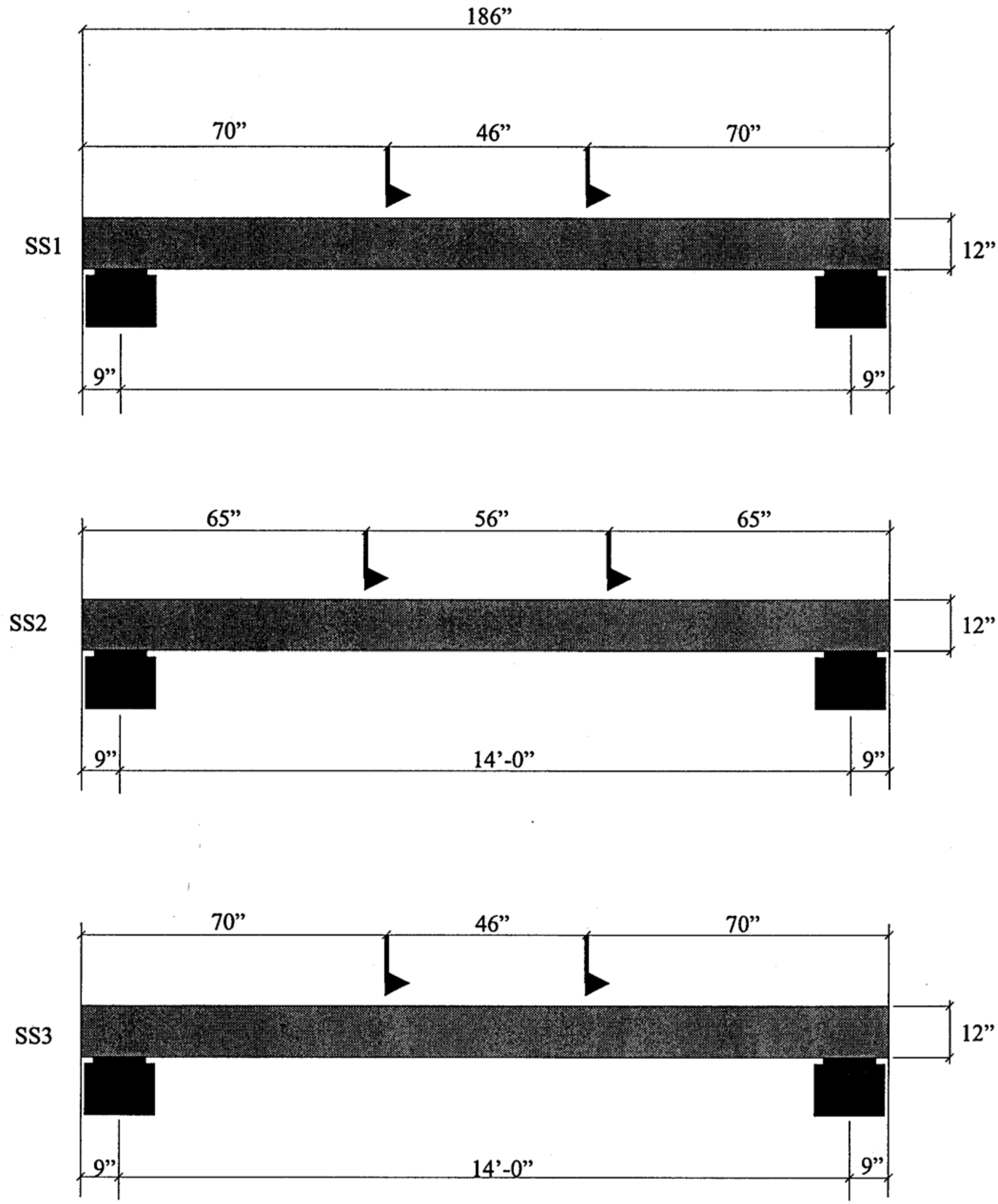
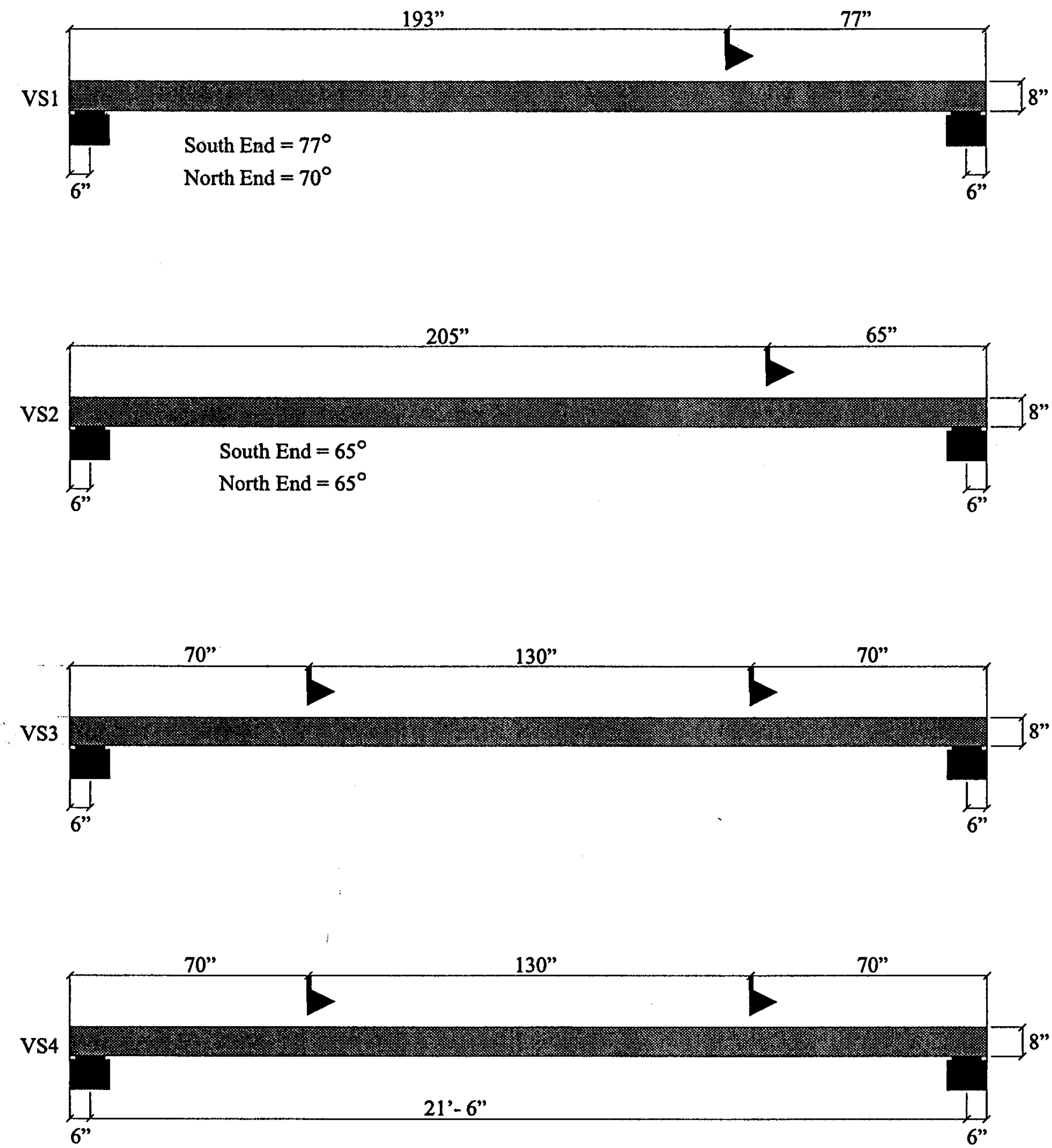


Figure 4 - Solid Slab Test Setup



Note: For VS1 and VS2 the south end was tested first, then the north end was tested using the same span length.

Figure 5 - Voided Slab Test Setup

Prior to the tests, the slabs were instrumented with surface strain gauges placed along the member at the level of the bottom strands. Deflection was monitored at support, quarter and mid-span as well as the load point. The ends of the prestressing strands were instrumented with LVDT's to monitor strand slip throughout loading.

The gauge measurements were recorded at every load stage, and crack development was marked during testing. Upon the completion of each test, a complete crack chart was developed and filed with the other test information.

2.2 RESULTS

A summary of the prestressed slab test results is presented in Table 1. The applied test moment, M_{app}, includes the effect of the dead load moment of the member. The embedment lengths in the tests were 65 in., 70 in. and 77 in., respectively.

In the SS1 test, the member was loaded to complete failure. In order to avoid damage to the instrumentation and to ensure safety of personnel, the specimens SS2 and SS3 were loaded to a lower level, which was well above the nominal moment.

Except for the two tests on specimen VS2, the observed strand slips were negligible (<0.001 in). In the VS2 tests, end slip of only one strand (0.003 in. and 0.004 in.) was observed at the maximum applied load, but the slip was negligible at nominal moment value. Even so, the maximum applied moments for the north and south tests were, respectively, 12 and 15 percent greater than the nominal moment values.

All failures were flexural, as indicated by specimen behavior and crack patterns. In only one test (SS2) was a diagonal crack observed (between a load point and support).

Slab Number*	End	Span, L (in.)	Embedment (in)	Slip Moment, M_{slip} (kip-ft)	Failure Moment $M_{applied}$ (kip-ft)	Nominal Moment, M_n, (kip-ft)	$M_{applied}/M_n$	M_{slip}/M_n	At M_n	
									ε_{ps}** (microstrain)	f_{su}^{*}** (ksi)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
SS1	N	175	70	NA	428.92	355.5	1.21	NA	11612	253.3
SS1	S	175	70	NA	428.92	355.5	1.21	NA	11612	253.3
SS2	N	175	65	NA	390.17	355.5	1.10	NA	11612	253.3
SS2	S	175	65	NA	390.17	355.5	1.10	NA	11612	253.3
SS3	N	175	70	NA	415.92	355.5	1.17	NA	11612	253.3
SS3	S	175	70	NA	415.92	355.5	1.17	NA	11612	253.3
VS1	N	256	70	NA	106.75	100.3	1.06	NA	18800	261.9
VS1	S	256	77	NA	113.67	100.3	1.13	NA	18800	261.9
VS2	N	256	65	112.42	112.42	100.3	1.12	1.12	18800	261.9
VS2	S	256	65	115.08	115.08	100.3	1.15	1.15	18800	261.9
VS3	N	256	70	NA	98.42	100.3	0.98	NA	18800	261.9
VS3	S	256	70	NA	98.42	100.3	0.98	NA	18800	261.9
VS4	N	256	70	NA	96.25	100.3	0.96	NA	18800	261.9
VS4	S	256	70	NA	96.25	100.3	0.96	NA	18800	261.9

* SS – Solid Slab, VS – Voided Slab
(N) – North End, (S) – South End
** Determined by strain compatibility analysis

Table 1 – Results of Slab Tests

Table 1 shows that the value of f^*_{su} for solid slabs was just below the value of the yield stress f_{py} ($\approx$ 256 ksi) of the strand. The value of f^*_{su} in the voided slabs was 261.9 ksi which exceeded the yield stress of the strand.

Except for voided slab specimens VS3 and VS4, the applied moment exceeded the nominal moment. In these two specimens, the applied moments were, respectively, 98 and 96 percent of the nominal value; however, strand slip was very small at ultimate.

It can be inferred from Table 1, that, an embedment as low as 65 inches is adequate to develop the nominal moment of both solid and voided slabs containing ½" diameter 270 ksi strand. This represents about 85 percent of the prescribed AASHTO value for development length (77 inches).

2.3 GENERAL DISCUSSION OF SLAB TEST RESULTS

The slab test results for development length are compared with AASHTO predictions as well as with values calculated from other sources. In Figure 6, actual embedment lengths are compared with AASHTO provisions for development length, and Figures 7, 8 and 8(a) show comparisons with recommendations by Buckner, FHWA and FDOT, respectively.

Figure 6 shows that the AASHTO provision for development length in Eqn. (1) gives good comparison with actual values in both solid and voided slabs. Predictions based on 1.6 AASHTO are far too conservative. (Points below the line are conservative).

In Figure 7, it is apparent that Buckner's proposal (Eqn. 8) also gives good comparisons with actual test values, although its predictions are somewhat conservative. For this plot the values of ε_{ps} and f^*_{su}, listed in Table 1, were obtained using strain-compatibility analysis.

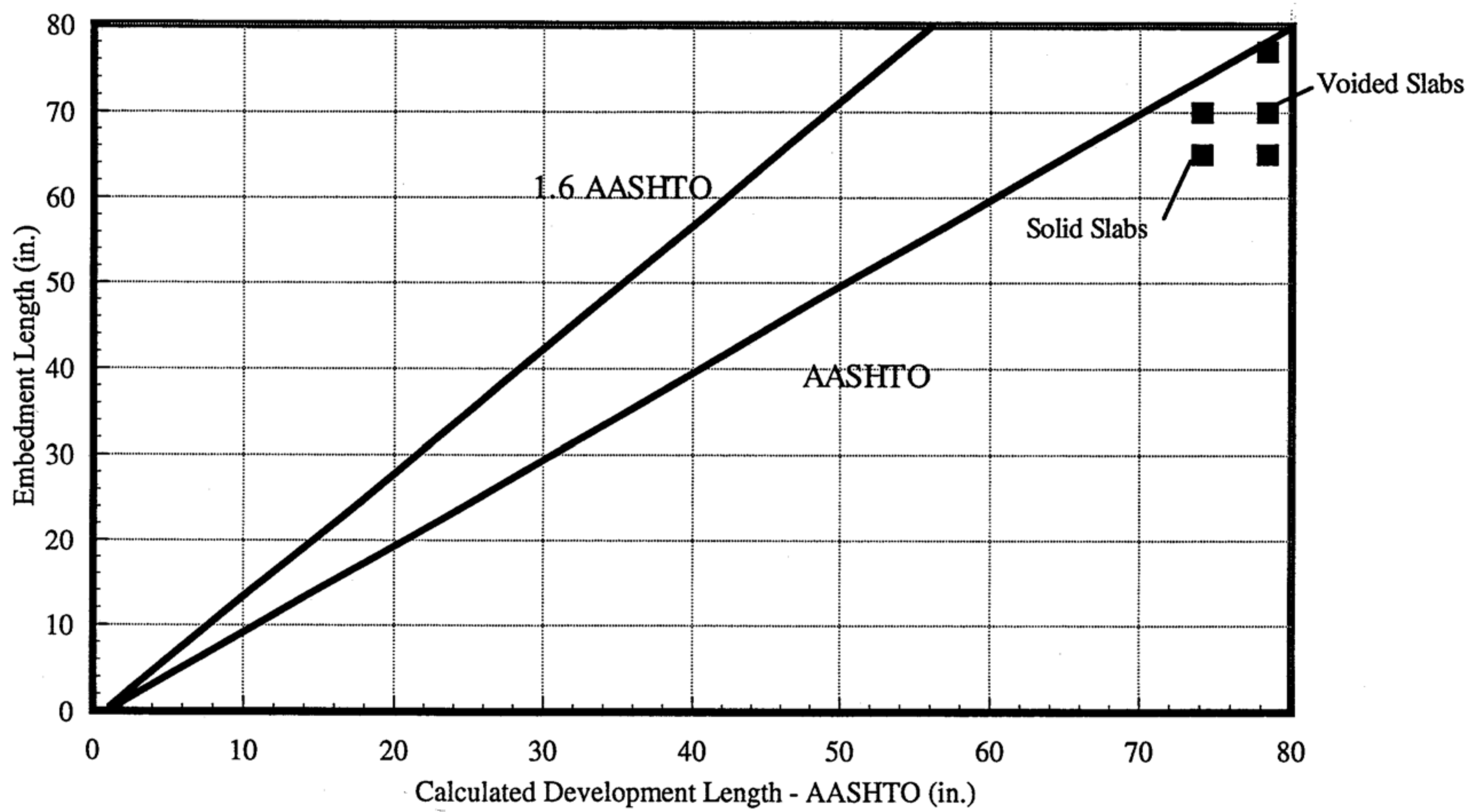


Figure 6 - Comparison of Embedment Length To AASHTO Calculated Development Length Slabs

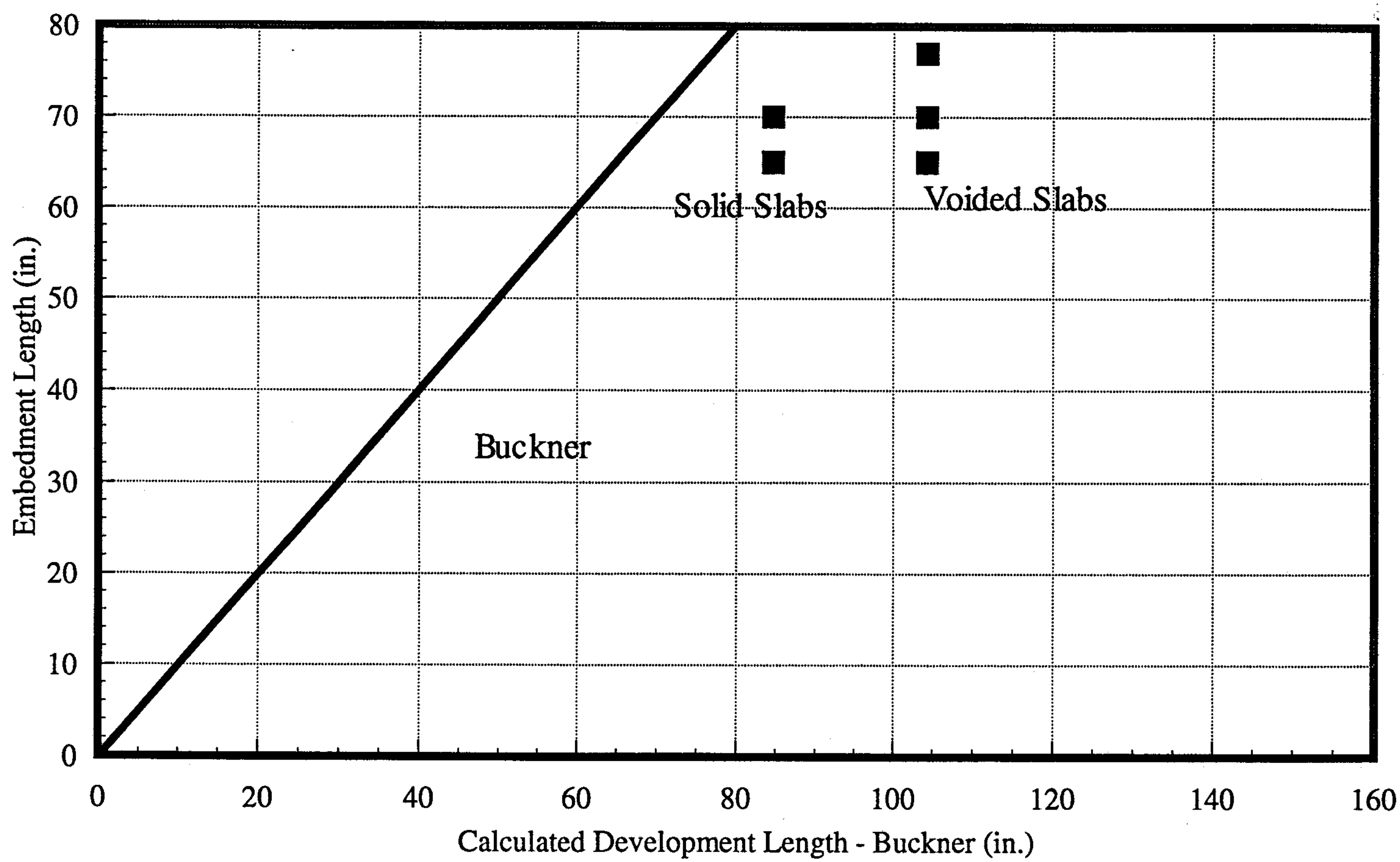


Figure 7 - Comparison of Embedment Length To Buckner Calculated Development Length Slabs

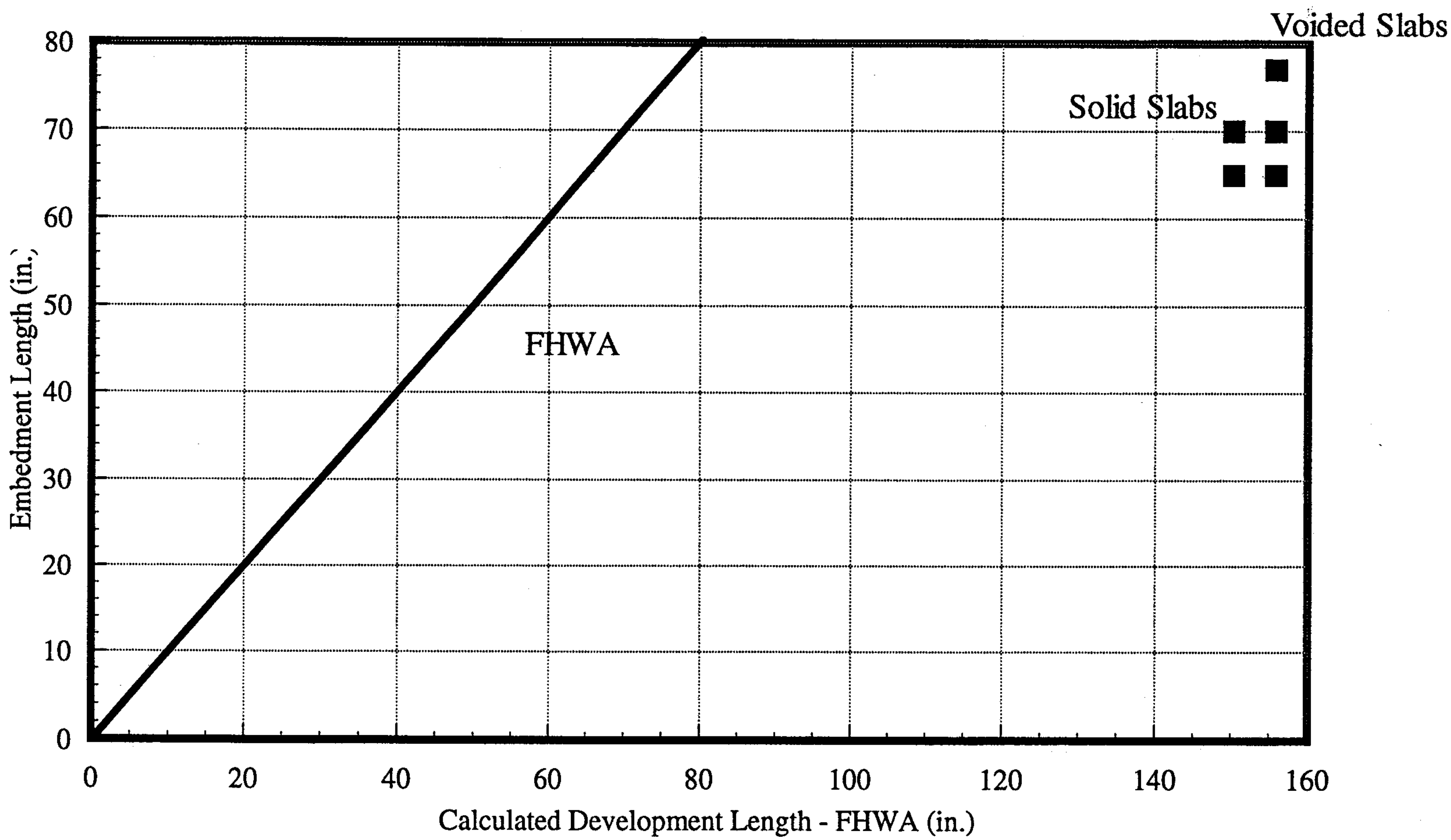


Figure 8 - Comparison of Embedment Length To FHWA Calculated Development Length Slabs

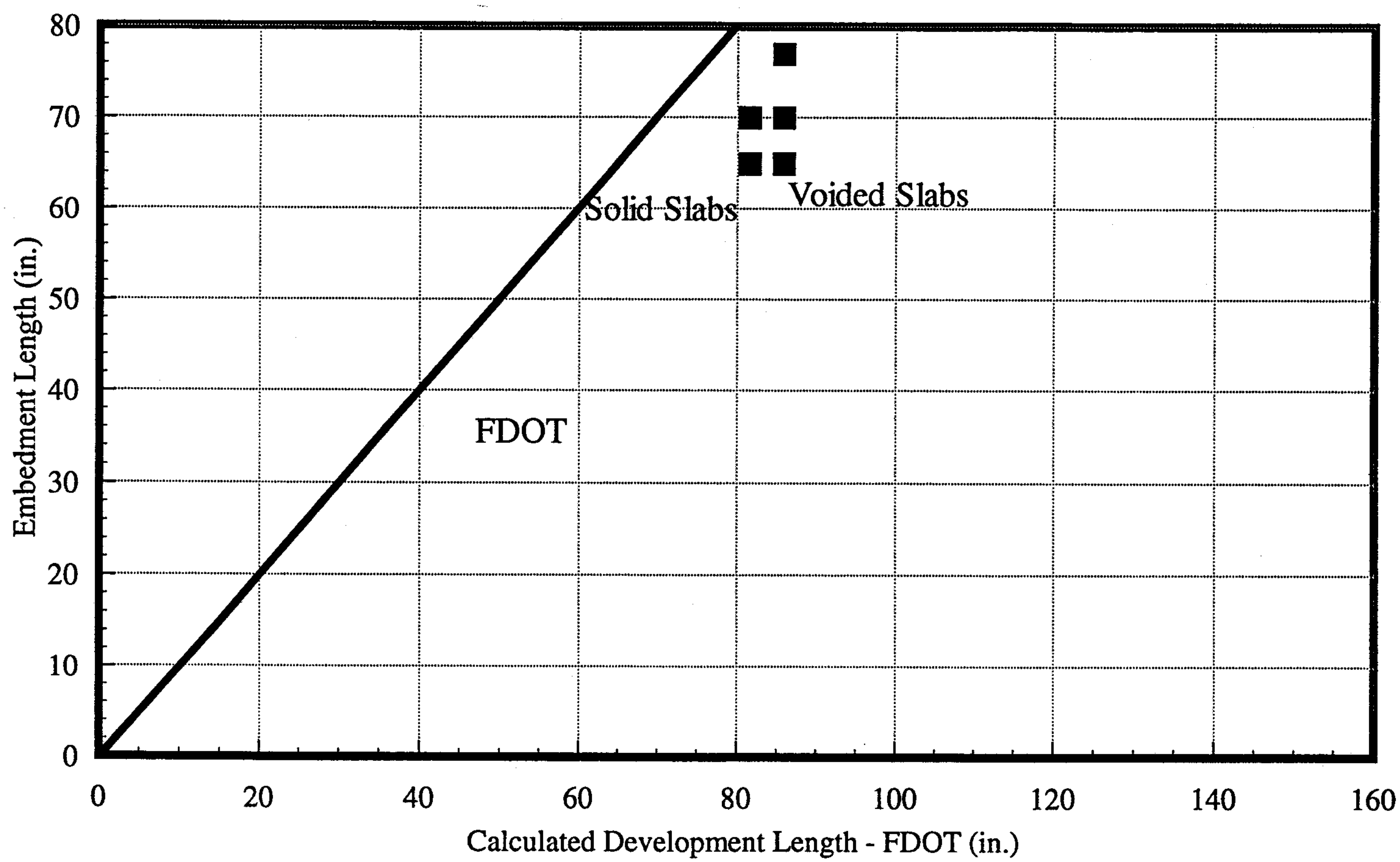


Figure 8(a) - Comparison of Embedment Length To FDOT Calculated Development Length Slabs

Figure 8 shows that the FHWA proposal (Eqn. 12) yields extremely conservative results, and is apparently not applicable to pretensioned slabs, while Figure 8(a) indicates that the FDOT proposal yields satisfactory results.

In Figure 9, all the slab test results are plotted for Applied Moment/Nominal Moment ratio versus Embedment Length. This plot shows that a development length of 70 inches (140 D) will definitely ensure that the nominal moment strength of $^{1}/_{2}$" diameter pretensioned strands can be fully developed.

It is interesting to examine the results based on simple FDOT recommendations for developed length (Eqn. (7)). The calculations presented in Appendix A show that the FDOT recommendations give development lengths of 64 inches for the solid slabs and 86.6 inches for the voided slab specimens, which compare quite favorably with the test results.

A summary of the calculated results in Appendix I is presented in Table 2 and plotted in Figure 10. It can be seen that the proposals by AASHTO, Buckner and FDOT give generally good predictions. In fact, AASHTO is the best of these three proposals, while Buckner is the most conservative. The FHWA proposal and 1.6 AASHTO give results that are far too conservative (by a factor of over 2.0).

Figure 11 shows the values of f^*_{su} and corresponding strains (ϵ_{ps}) at ultimate for all the prestressed slab specimens plotted on the standard stress-strain curve for the strand. As pointed out by Buckner, it can be seen that the strains in the slender slab specimens at ultimate are relatively low (near yield). This explains the relatively low values of development length of such specimens.

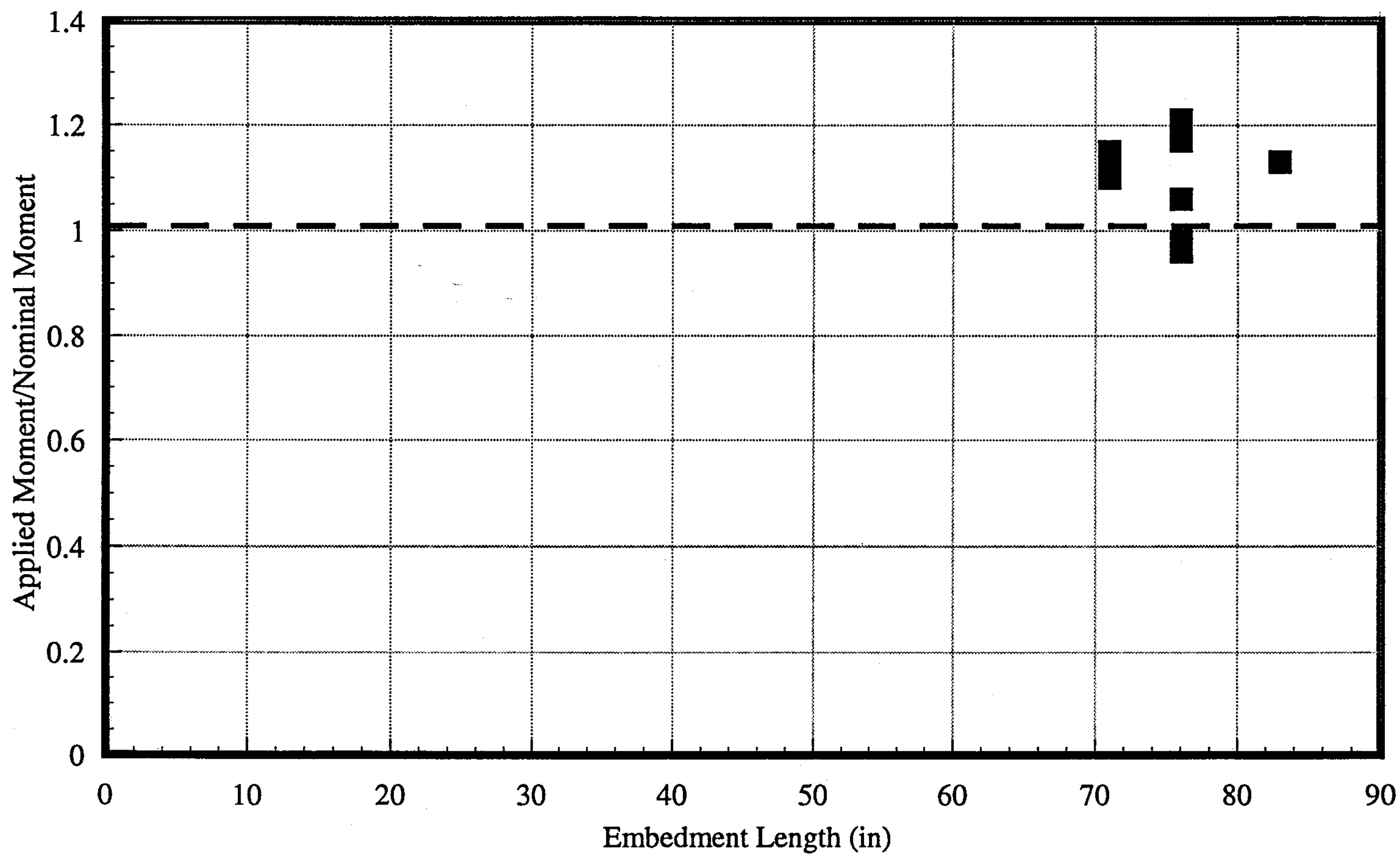


Figure 9 - Applied Moment/Nominal Moment vs. Embedment Length

Slabs

Member Type	AASHTO Eqn. (2)	1.6 AASHTO	Buckner Eqn. (8)	FHWA Eqn. (12)	FDOT Eqn. (7)	Measured L_d
Solid Slabs	74.2	118.6	84.5	148.4	81.6	65
Voided Slabs	78.5	125.5	104.2	155.8	85.9	65

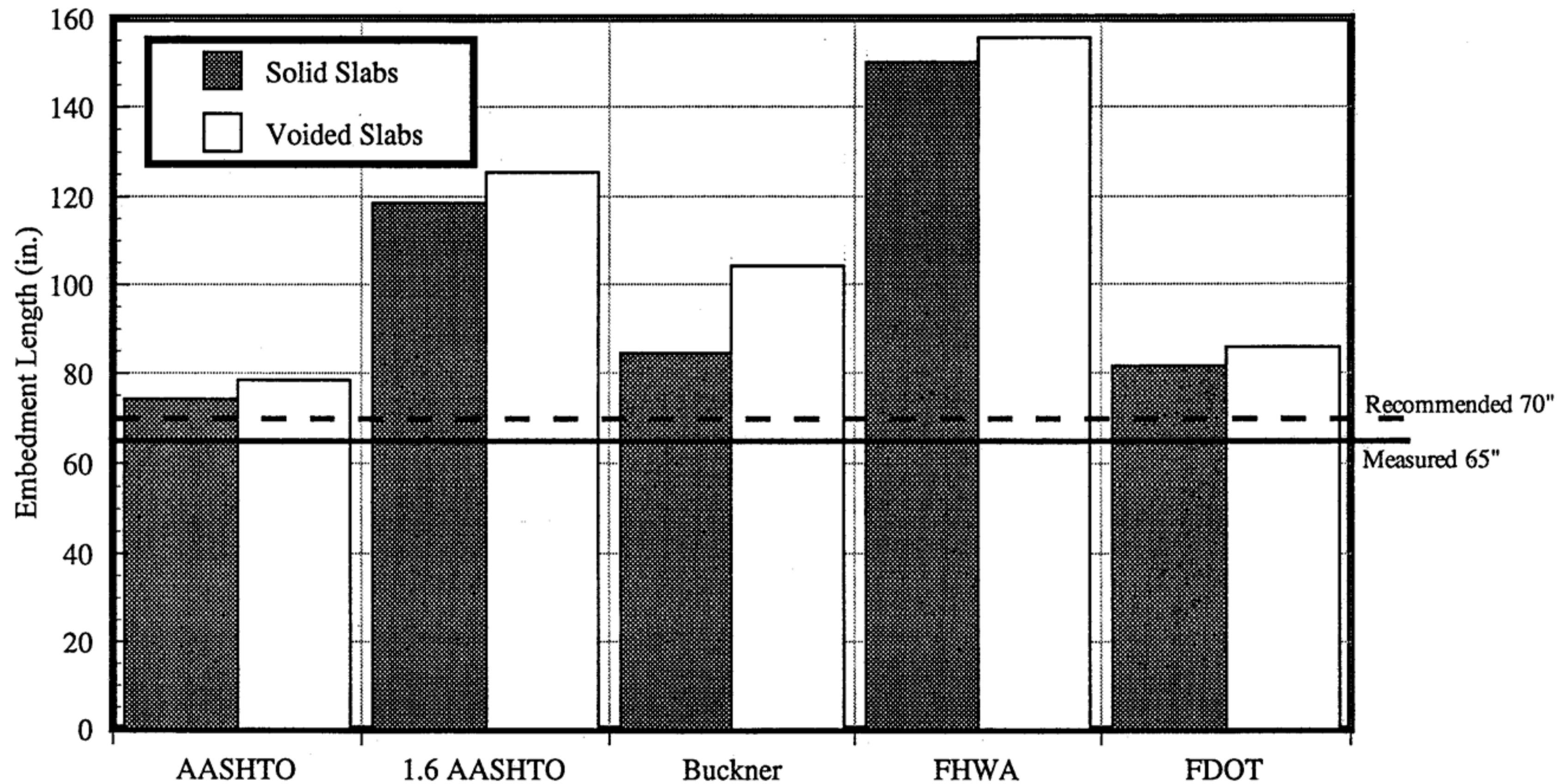


Figure 10 - Comparison of Development Lengths For Slabs

In the FHWA study, plots were developed for measured embedment length versus (f^*_{su} - f_{se}) in comparing results from other studies. Similar plots are presented in Appendix A, (Figs. A1 and A2), which confirm the general trends discussed above.

Figure A3 of Appendix A shows that the proposal by Zia and Johnston (Eqn. 5) is conservative for slabs.

2.4 CONCLUSIONS FROM SLAB STUDY

The following conclusions can be made from the study of development length of ½" diameter Grade 270 strand in prestressed slabs:

1. The FHWA recommendations give results that are far too conservative and are not considered acceptable for use in prestressed slabs. Cousins and Zia's proposal is less conservative than that of FHWA.
2. Both sets of recommendations by Buckner and FDOT give satisfactory results for solid slabs, but are somewhat conservative for voided slabs for which the FDOT equation yields results closer to those observed in the tests.
3. The AASHTO equation yields surprisingly good predictions, probably because, as pointed out by Buckner, the stress in prestressing steel is low at failure of slender members. The use of a multiplier of 1.6 is not warranted for such members.
4. As previously proposed by FDOT, a development length of 70 inches (140 D) is definitely appropriate for 0.5 inch diameter Grade 270 strand in prestressed slabs.

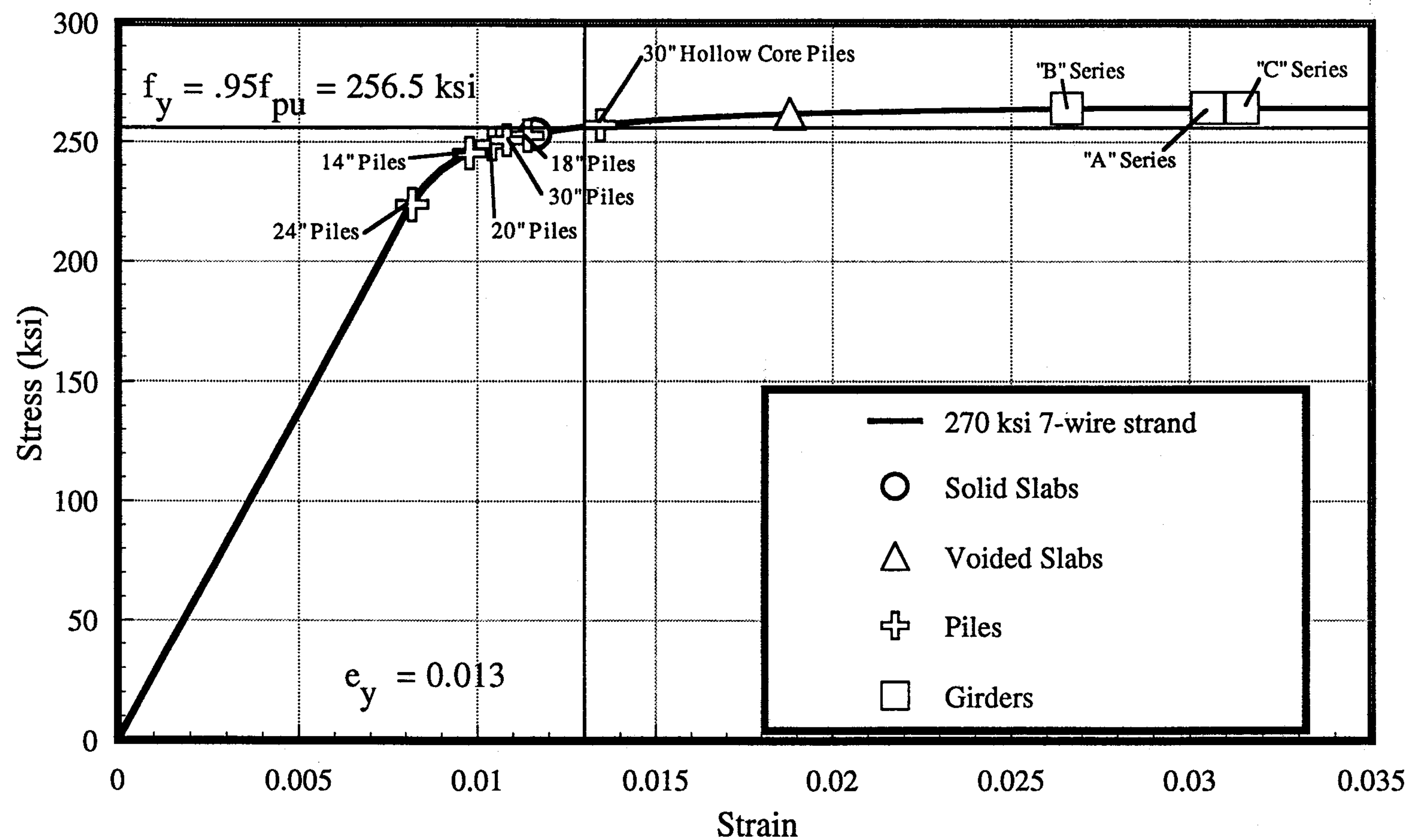


Figure 11 - Stress vs. Strain for 270 ksi 7-wire strand

3. GIRDERS

The Florida Structural Research Center has conducted a wide-ranging study" [6] of the behavior of AASHTO Type II girders with respect to transfer and development lengths of strand, flexure, shear, and fatigue behavior. Only those tests applicable to transfer and development of prestressing strand are covered in this section.

3.1 TRANSFER LENGTH

As stated earlier, the use of f_{si} instead of f_{se} for calculating transfer length of strand in the first part of Eqn. (2), closely predicted the transfer lengths observed for strands of $^1/_2$ in., $^1/_2$ in. special, and 0.6 in diameter. This results in Equation (6), which was first proposed by MOT for calculating transfer lengths, was later confirmed by Buckner. The average measured transfer lengths for 0.5 in and 0.6 in diameter unshielded strands were observed to be 30 in. (60 D) and 34 in. (57 D), respectively. In view of this finding, Eqn. (6) will be adopted for calculating transfer length of strand; and the topic of transfer length will not be discussed further in this report.

It was also stated in the MOT study, that the AASHTO Code requirement for a minimum spacing of four strand diameters for 0.6 in diameter strand appears to be too restrictive; and that a strand spacing of 2 inches did not produce any adverse results in tests. This was also confirmed by FHWA[3] and by Buckner[2].

3.2 TEST SPECIMENS AND PROCEDURE

The development length results are summarized in Tables 3 and 4, which contain data for only those girders designed according to the AASHTO Code. The specimens were of three types, designated A, B and C which, respectively, incorporated $^1/_2$ in. dia, $^1/_2$ in. special and 0.6 in. diameter low relaxation Grade 270 strands, as shown in Figure 12. The girders were labeled (A, B or C) according to strand size, degree of shielding (zero in this part of the study) and amount of shear reinforcement. Thus, the designation AO-00-RA is interpreted as follows:

A: strand size of 0.5 in. ("B" represents 0.5 in. dia. special strand, and "C" represents 0.6 in. dia. strand)

00 zero shielding (all girders in this part of the study were unshielded)

R shear reinforcement provided in accordance with the AASHTO Code. The term, RD, indicates that shear reinforcement was provided in accordance with AASHTO requirements, but confinement reinforcement was omitted in the tension flange. ("A" or "B" indicates that more than one set of a particular specimen was tested).

The main variables in this part of the study were:

(a) Strand size: $^1/_2$ in. dia, $^1/_2$ in. special and 0.6 in. dia. Grade 270

(b) Available embedment length as a result of varying the shear span length

(c) Confinement reinforcement to strands in tension flange (the D-bar shown in Figure 12).

Except for the four girders labeled "D" in Table 3, the test specimens were provided with confinement reinforcement.

Girder Number (1)	End (2)	Span L (in.) (3)	Shear Span* a (in) (4)	a/d (5)	Slip Moment M.,ip (kip-ft) (6)	Applied Moment MWuka (kip-ft) (7)	Design Moment M,, (kip-ft) (8)	Nominal Moment M_o (kip-ft) (9)	M,/M,, (10)	M_su^ (11)	M.$_W$/M,, (12)	M$_{S,;p}$/M. (13)
AO-00-R	N	480	85	2.1	1535	2237	1083	2058	2.07	1.42	1.09	0.75
AO-00-R	S	324	85	2.1	1403	1975	1083	2058	1.82	1.29	0.96	0.68
AO-00-RD	N	480	74	1.8	1289	1433	968	2058	1.48	1.33	0.70	0.63
AO-00-RD	S	306	85	2.1	1471	1634	1083	2058	1.67	1.36	0.89	0.71
Al-00-R	N	480	102	2.5	1250	1809	1236	2058	1.46	1.01	0.88	0.61
Al-00-R	S	378	124	3.1	1500	2191	1376	2058	1.59	1.09	1.07	0.73
Al-00-RD	N	480	102	2.5	1322	1551	1236	2058	1.26	1.07	0.75	0.64
Al-00-RD	S	378	124	3.1	1597	1997	1376	2058	1.45	1.16	0.97	0.78
A3-00-RA	N	480	102	2.5	1750	2334	1236	2058	1.89	1.42	1.13	0.85
A3-00-RA	S	378	124	3.1	2150	2441	1376	2058	1.77	1.56	1.19	1.04
A3-00-RB	N	480	85	2.1	1600	2120	1083	2058	1.96	1.48	1.03	0.78
A3-00-RB	S	424	85	2.1	1400	1969	1083	2058	1.82	1.29	0.96	0.68
BO-00-R	N	480	102	2.5	1600	1899	1236	2142	1.54	1.29	0.89	0.75
BO-00-R	S	378	124	3.1	2175	2175	1376	2142	1.58	1.58	1.01	1.02
B 1-00-R	N	240	60	1.5	1150	1235	822	1883	1.50	1.40	0.66	0.61
B 1-00-R	S	222	54	1.3	975	1052	759	1792	1.39	1.28	0.59	0.54
CO-00-R	N	336	142	3.5	2139	2139	1470	2036	1.45	1.45	1.05	1.05
CO-00-R	S	480	132	3.2	2021	2021	1431	2036	1.41	1.41	0.99	0.99
CO-00-RD	N	264	60	1.5	815	957	833	1673	1.15	0.98	0.57	0.40
CO-00-RD	S	480	148	3.6	2000	2011	1500	2036	1.34	1.19	0.99	0.98
C1-00-R	N	480	142	3.5	2148	2148	1470	2036	1.46	1.46	1.05	1.06
C1-00-R	S	378	132	3.2	2197	2197	1431	2036	1.54	1.54	1.08	1.08
C1-00-RD	N	480	149	3.7	2185	2185	1520	2036	1.44	1.44	1.07	1.07
CI-00-RD	S	378	149	3.7	1949	1949	1520	2036	1.28	1.28	0.96	0.96

Girder number (1)	End (2)	Embedment/ Bar Diameter (3)	Nominal Moment M_n (kip-ft) (4)	At M_n	
				ε_{ps} (microstrain) (5)	f_{su}^{*} (ksi) (6)
A0-00-R	N	182	2058	30490	264.6
A0-00-R	S	182	2058	30490	264.6
A0-00-RD	N	160	2058	30490	264.6
A0-00-RD	S	182	2058	30490	264.6
A1-00-R	N	216	2058	30490	264.6
A1-00-R	S	260	2058	30490	264.6
A1-00-RD	N	216	2058	30490	264.6
A1-00-RD	S	260	2058	30490	264.6
A3-00-RA	N	216	2058	30490	264.6
A3-00-RA	S	260	2058	30490	264.6
A3-00-RB	N	182	2058	30490	264.6
A3-00-RB	S	182	2058	30490	264.6
B0-00-R	N	216	2142	26567	264.3
B0-00-R	S	260	2142	26567	264.3
B1-00-R	N	132	1883	26567	264.3
B1-00-R	S	120	1792	26567	264.3
C0-00-R	N	247	2036	31440	264.6
C0-00-R	S	230	2036	31440	264.6
C0-00-RD	N	110	1673	31440	264.6
C0-00-RD	S	257	2036	31440	264.6
C1-00-R	N	247	2036	31440	264.6
C1-00-R	S	230	2036	31440	264.6
C1-00-RD	N	258	2036	31440	264.6
C1-00-RD	S	258	2036	31440	264.6

Table 3(b) – Results For AASHTO Girders
(Stresses and Strains in Strands at Nominal Strength)

Girder Number (1)	End (2)	Span, L (in.) (3)	Shear Span, a (in.) (4)	a/d (5)	Shear at First Slip, V_{slip} (6)	Applied Shear, V_a (7)	Nominal Shear, V_n (8)	V_a/V_n (9)	Failure Mode (10)
A0-00-R	N	480	85	2.1	259	313	221	1.41	shear/bond
A0-00-R	S	324	85	2.1	276	276	221	1.25	shear
A0-00-RD	N	480	74	1.8	207	230	217	1.06	shear/bond
A0-00-RD	S	306	85	2.1	NA	228	221	1.03	shear/bond
A1-00-R	N	480	102	2.5	144	210	193	1.09	shear/bond
A1-00-R	S	378	124	3.1	190	208	159	1.31	flexure/bond
A1-00-RD	N	480	102	2.5	152	179	193	0.93	shear/bond
A1-00-RD	S	378	124	3.1	151	189	159	1.19	shear/bond
A3-00-RA	N	480	102	2.5	203	271	169	1.60	shear/bond
A3-00-RA	S	378	124	3.1	204	232	165	1.41	shear/bond
A3-00-RB	N	480	85	2.1	223	297	221	1.34	flexure/bond
A3-00-RB	S	424	85	2.1	195	275	221	1.24	shear/bond
B0-00-R	N	480	102	2.5	185	220	194	1.13	shear/bond/flexure
B0-00-R	S	378	124	3.1	207	206	161	1.28	shear/flexure
B1-00-R	N	240	60	1.5	228	245	212	1.16	shear/bond
B1-00-R	S	222	54	1.3	215	232	210	1.10	shear/bond
C0-00-R	N	336	142	3.5	176	176	147	1.20	flexure
C0-00-R	S	480	132	3.2	180	180	160	1.13	flexure
C0-00-RD	N	264	60	1.5	161	189	213	0.89	shear/bond
C0-00-RD	S	480	148	3.6	NA	158	148	1.07	shear/flexure
C1-00-R	N	480	142	3.5	177	177	147	1.20	flexure
C1-00-R	S	378	132	3.2	196	196	160	1.23	flexure
C1-00-RD	N	480	149	3.7	NA	171	141	1.21	flexure
C1-00-RD	S	378	149	3.7	NA	152	141	1.08	flexure

Table 4 – Shear Test Results for AASHTO Girders

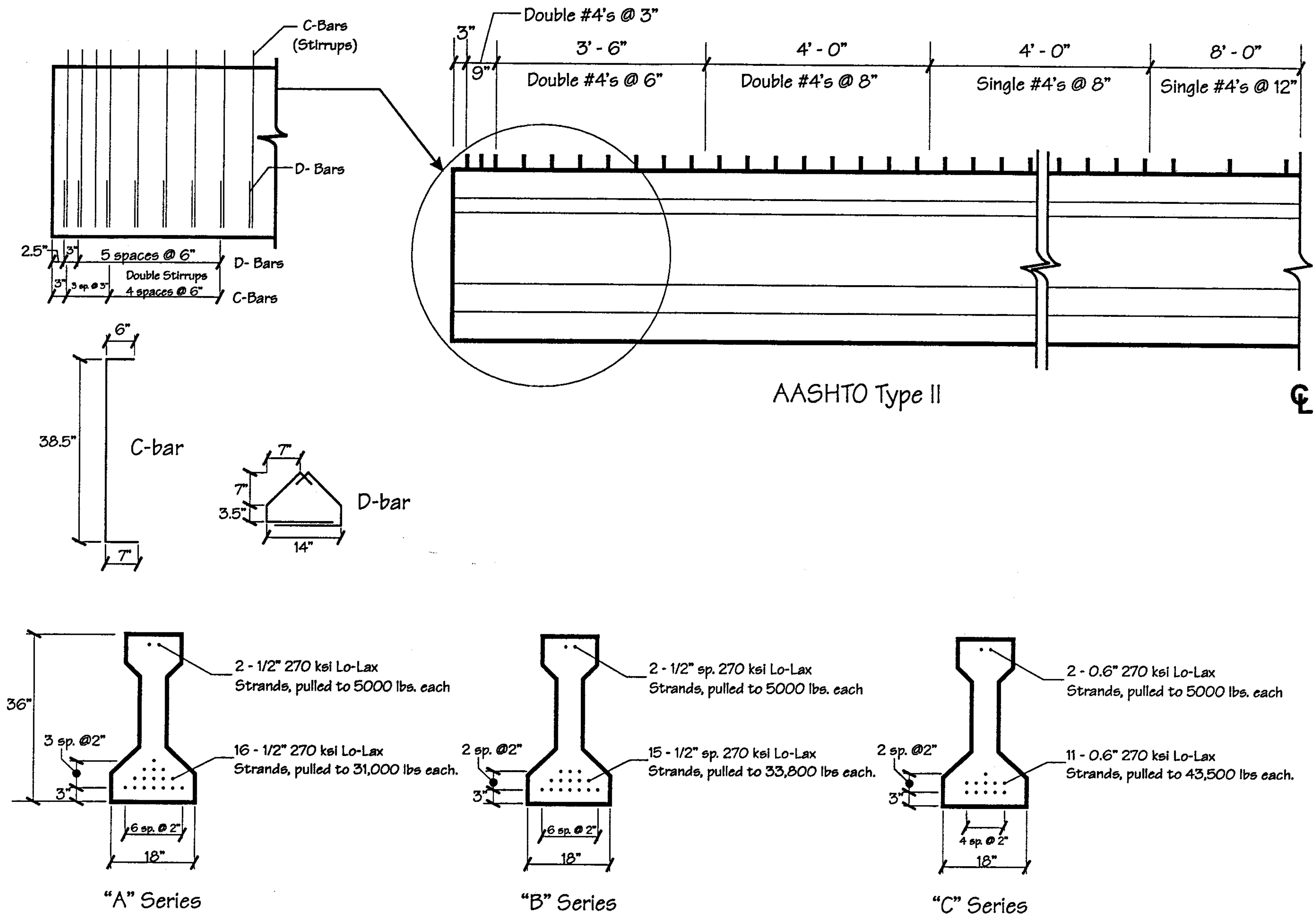


Figure 12 - Reinforcement Details of Test Girders

There was no intentional variation of other parameters. In order to check the FHWA strict requirements for the spacing of the strands, the spacing, this was kept constant at 2 inches center/center regardless of strand size (Fig. 12).

Durastress in Leesburg, Florida produced the precast portions of the girders. After transportation to the FDOT Structural Research Center, a top flange, 42 inches wide and 6 in. thick was cast on all specimens (Fig. 13). Except for variation of prestressing steel in the bottom flange, as shown in Fig. 12, the cross sections of all specimens conformed to that shown in Fig. 13. The concrete for the precast sections and the cast-in-place top slab was designed for a 28day cylinder compressive strength, f'_c, of 5,000 psi. The compressive strength at transfer, f'_{ci}, was 4000 psi.

In general, each end of a girder was tested using a single concentrated load applied incrementally to failure. The location of the load (shear span) was varied from girder to girder, or from one end to the other, as shown in Figure 14. It can be seen that in order to eliminate the failed zone from the test span the test span was shortened after one end of the girder was tested.

LVDT's were used to monitor slip at the ends of all strands continuously during a test. Strains and deflections were also appropriately monitored[5].

3.3 RESULTS

3.4 General Observations

The detailed behavior of the test specimens is described in References (5) and (6). Typical observations related to development of strands are presented in this section.

Before discussing the development length results, it is helpful to explain the typical bond failure mechanism in the girders. An important observation was the value of the applied moment

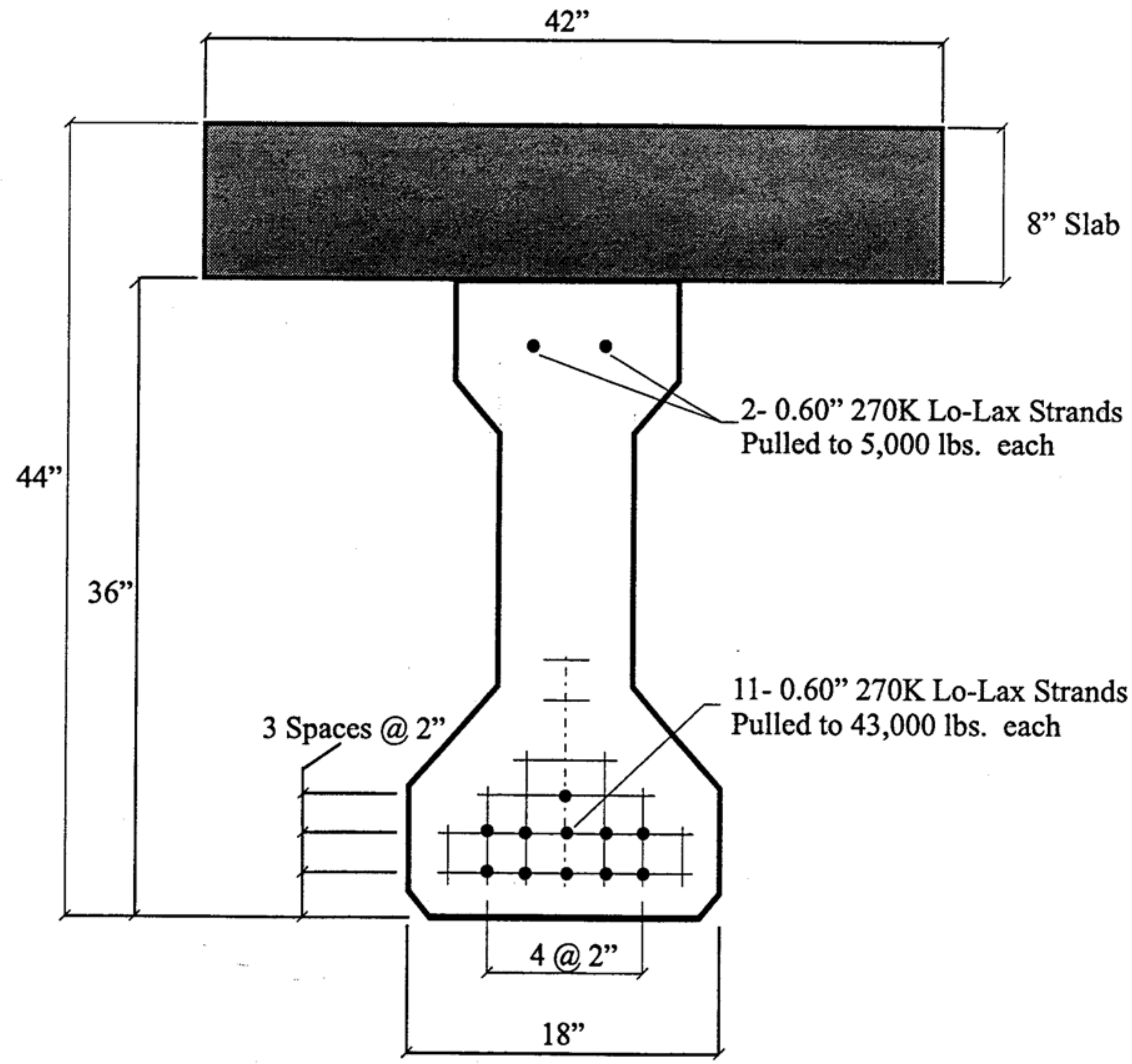
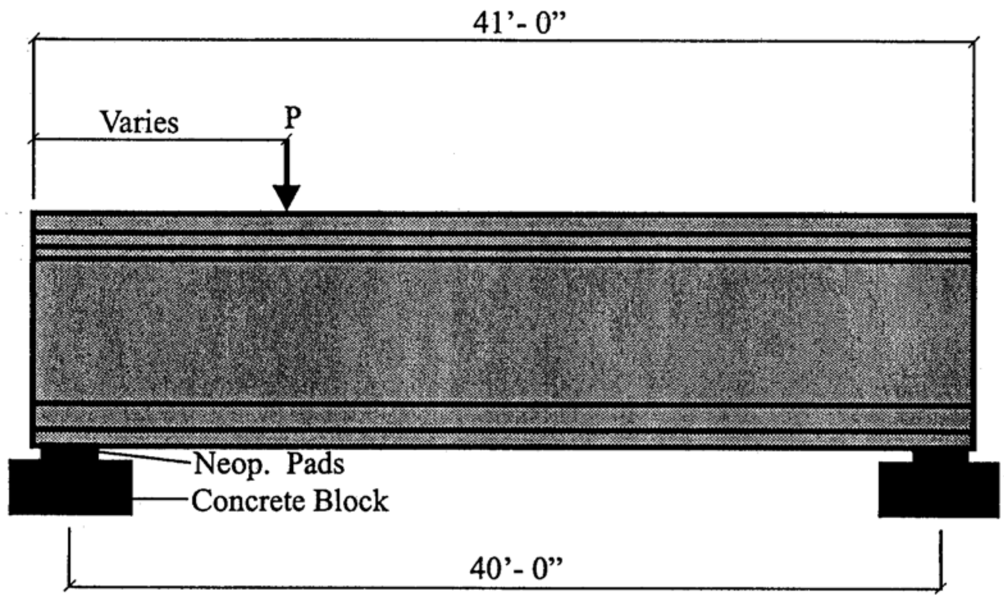
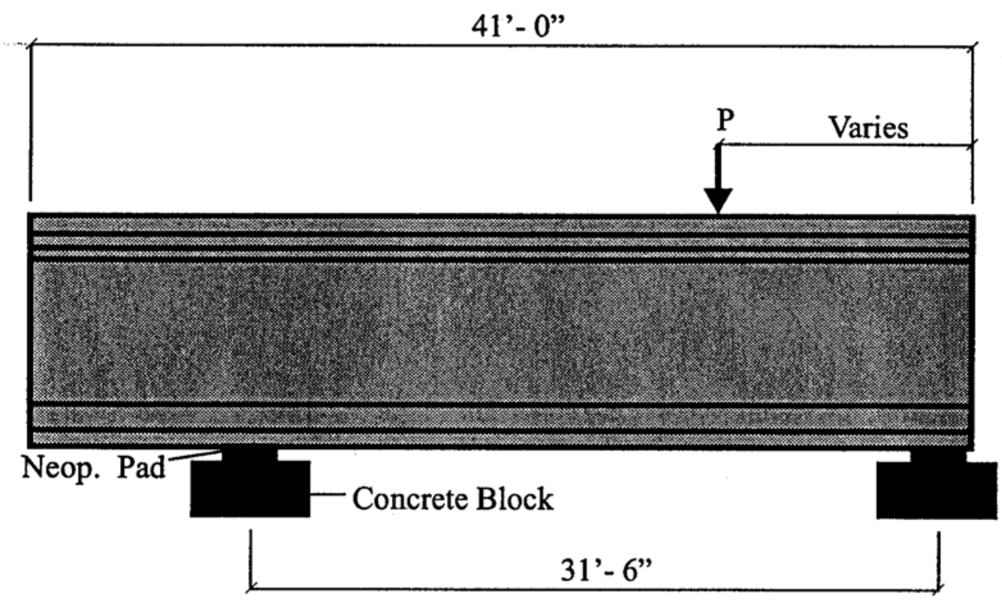


Figure 13 - Cross Section Details (Group C)



Test Setup - North End



Test Setup - South End

Figure 14 - Test Setup

at which initial strand slip occurred, as determined from continuous scanning of readings from LVDT's used to monitor strand slip. For example, Figure 15 shows the strand slip sequence, with increasing load, for girder A1-00-R (south end). The initial slip occurred at, or shortly after appearance of the first shear crack. The beam then continued to carry increased loading. As the load was increased, new cracks developed and additional strands started to slip. The ability of the girder to carry more loads continued, until complete bond slip of all strands occurred. This type of bond slip mechanism was typical for all girders.

Figure 16 shows a plot of test results, with the ratio of Slip Moment/Design Moment plotted against the shear span/depth ratio. From Figure 16 and Table 3(a), it is seen that except for the test CO-00-RD(N), which had no confinement reinforcement, the applied moment at initial slip, M_{slip} (Column 6), was greater than the design moment, M_u, (Column 8), although in most cases, the value of M_{slip} was less than the nominal moment, M_n. (Column 13). This is an important observation, which should be addressed in any formulation of a realistic development length equation.

From a study of crack development in the girders, together with the sequence of strand slip, it was obvious that there existed an interaction between shear and bond of the strands. Figure 17 presents a plot of the ratio of M_{slip}/M_n versus a/d for all the girders, which indicates that at a value of a/d = 3.5, the value of M_{slip}/M_n is close to 1.0. The results for girder A3-00-RA and A3-00-RB are omitted from Figure 17, since a wire mesh was used as shear reinforcement in these specimens.

The values of nominal and design moments and shears along the span were calculated and plotted along the length of each girder. The values of test moments and shears were also plotted on these diagrams. Typical plots are shown in Figures 18 and 10 for tests A1-00-R(N)

Girder A1-00R (South End)

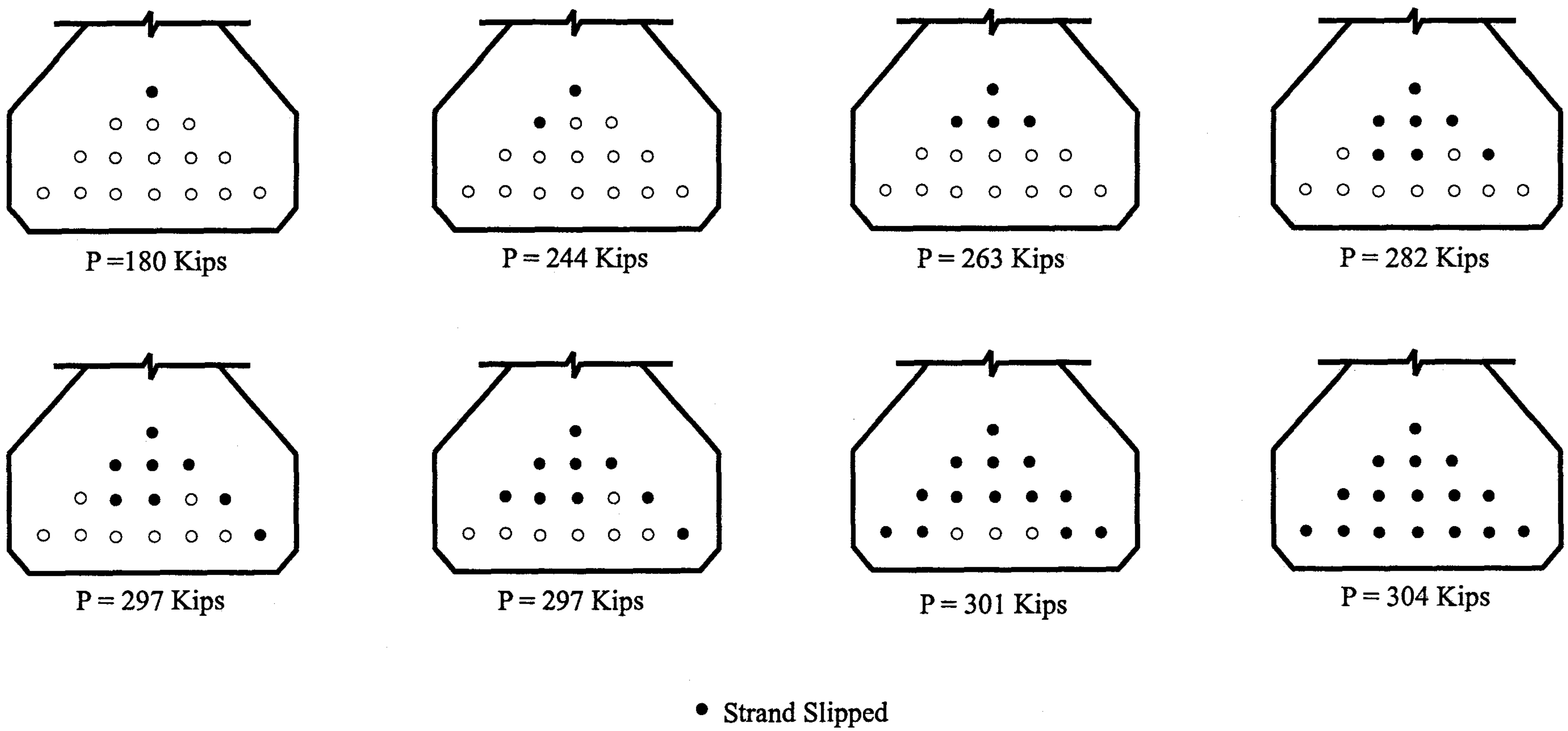


Figure 15 - Strand Slip Sequence

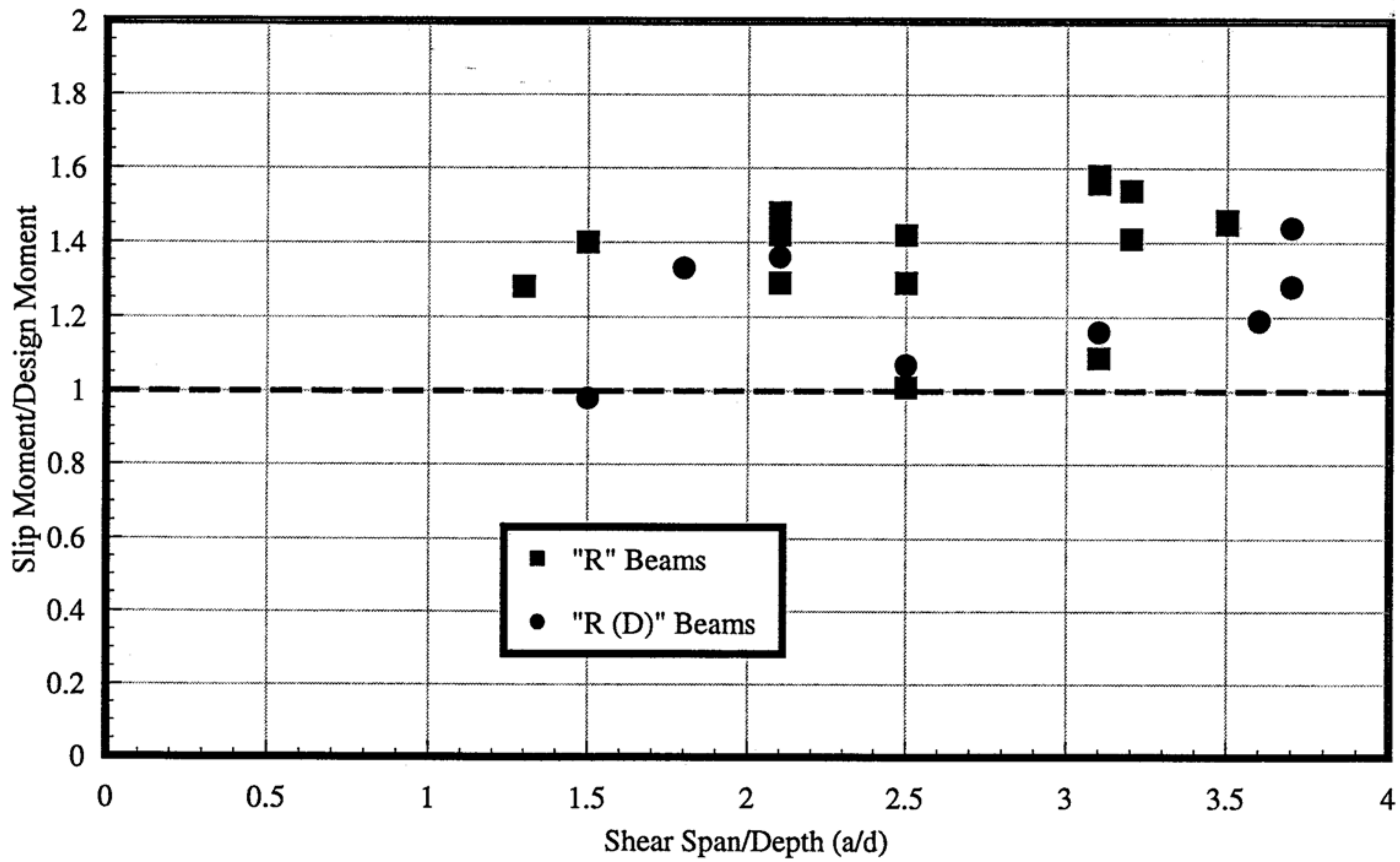


Figure 16 - Effect of Shear Span to Depth Ratio on Strand Slip

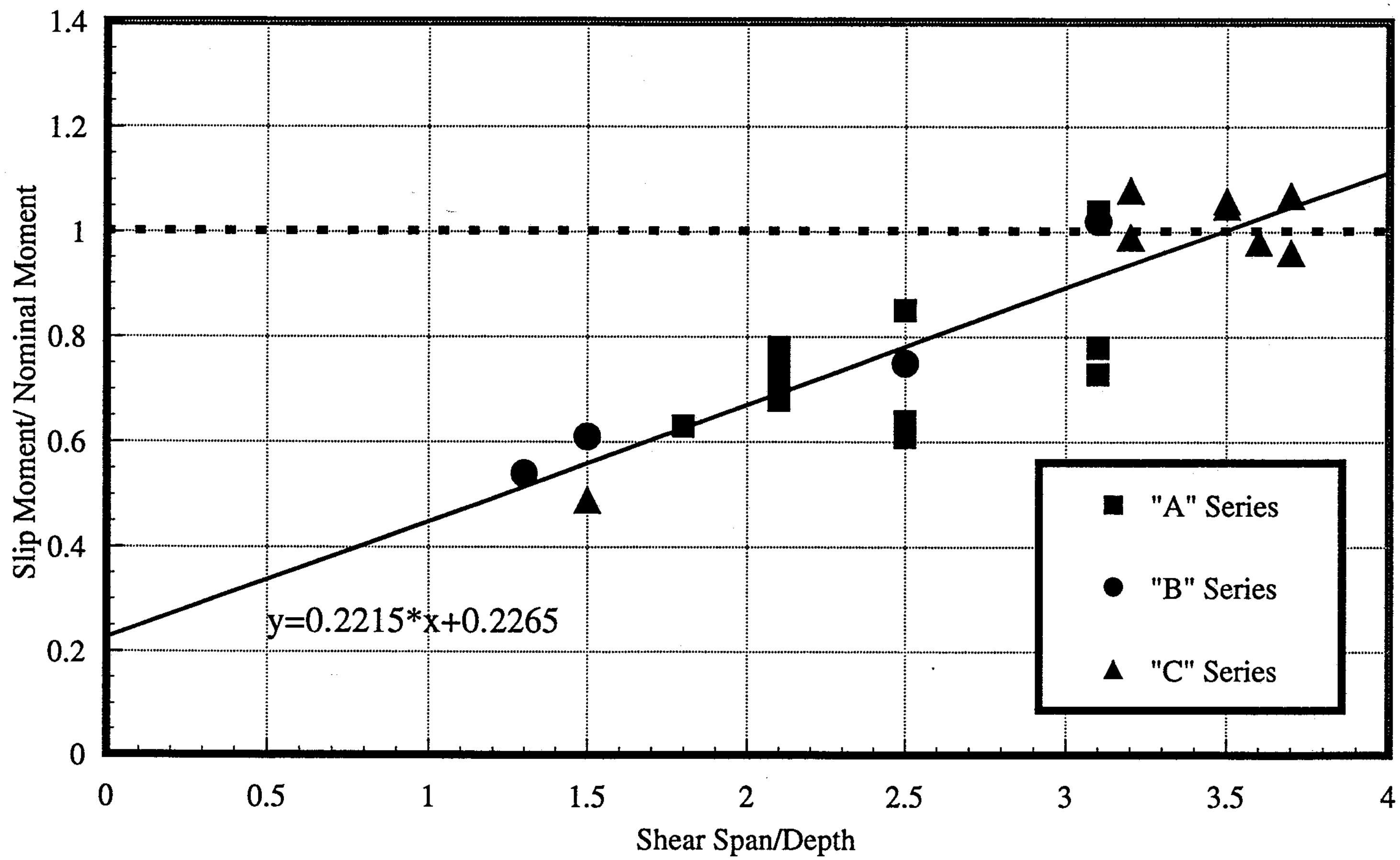


Figure 17 - Relationship Between Slip Moment and Shear Span

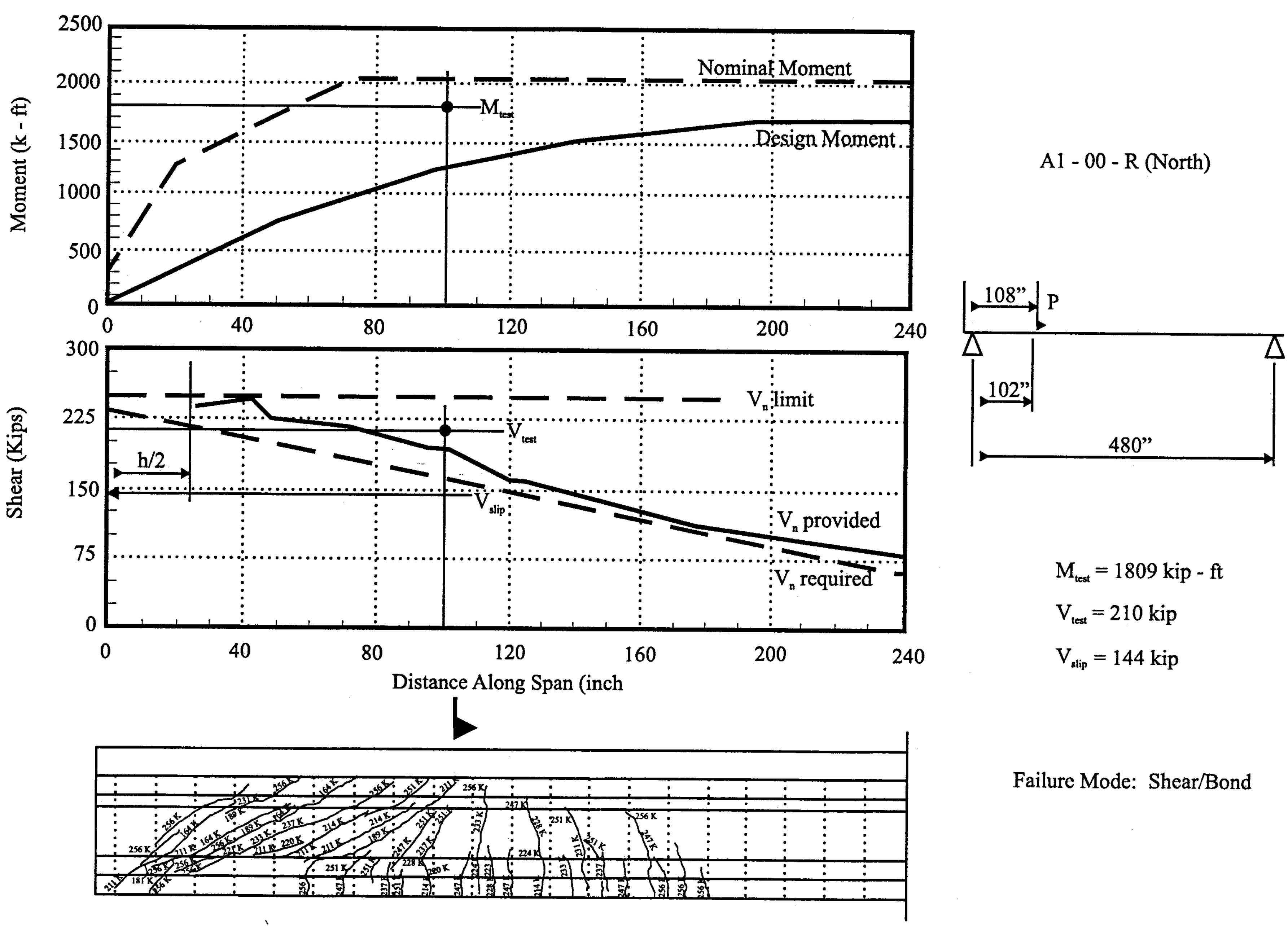
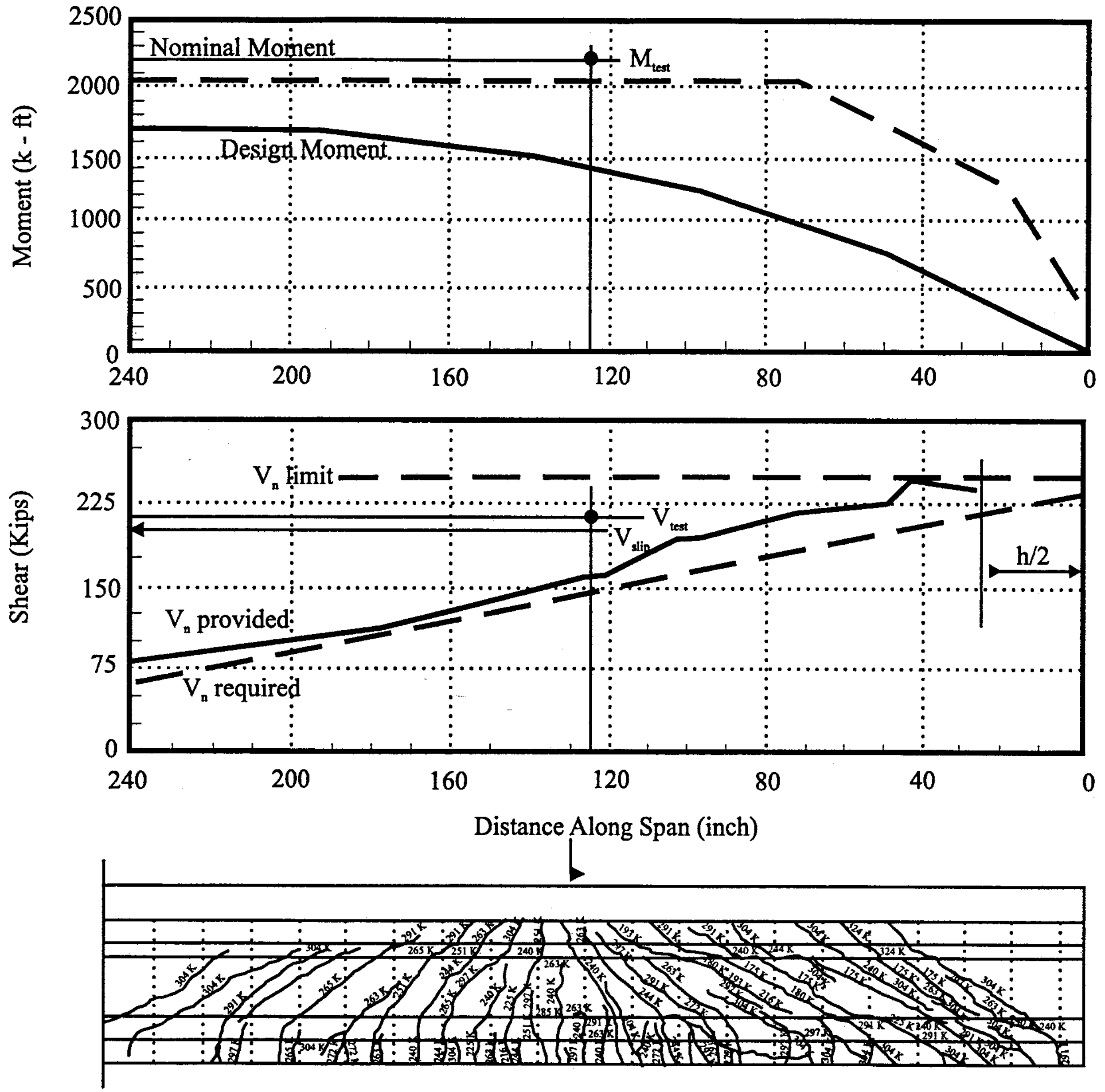
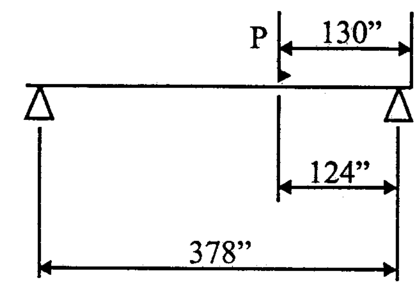


Figure 18 - Response Of A1 - 00 - R (N)



A1 - 00 - R (South)



M_{test} = 2191 kip - ft

V_{test} = 208 kip

V_{slip} = 190 kip

Failure Mode: Flexure/Bond

Figure 19 - Response of A1 - 00 - R (S)

and Al-00-R(S), respectively. These plots, together with Tables 3 and 4, show that the test moments and shears exceeded the design values, which raises some questions regarding the general applicability of the AASHTO development length equation when dealing with loads applied near an end support of a girder.

3.5 COMPARISON OF OBSERVED AND PREDICTED DEVELOPMENT LENGTHS

In Figures 20, 21 and 22, plots are presented of different moment ratios against embedment length/strand diameter for all the test results listed in Table 3. The corresponding plots for the different strand sizes are shown plotted in Appendix Figures B1-B9, inclusive. Generally, the trends discussed herein for all strands combined (Figs. 20-22) are also reflected in the Appendix figures.

The plot for M_a/M_n in Figure 20 shows that the nominal moment of a girder is fully developed when the embedment length is about 230 times strand diameter. Below this value, the types of failures are predominantly shear or shear/bond. The plot in Figure 21 shows that for embedment of about 230 times strand diameter, the moment at slip is at least equal to the nominal moment value. The two exceptions are A-00-RD(N) and Al-00-R(S). The former was not provided with confinement reinforcement, while the latter was the second test on a conventionally detailed girder. The fact that this latter test exhibited a flexure failure mode at an embedment length of 260 D is unexpected, and difficult to explain in the absence of any observed defects or previous cracking within the shear span.

The plot in Figure 22 for M_{slip}/M_n reveals some interesting and important points. Regardless of the mode of failure and the test embedment length, all test moments were above **the sectional design moment, (except for test C-00-RD(N) which did not contain confinement**

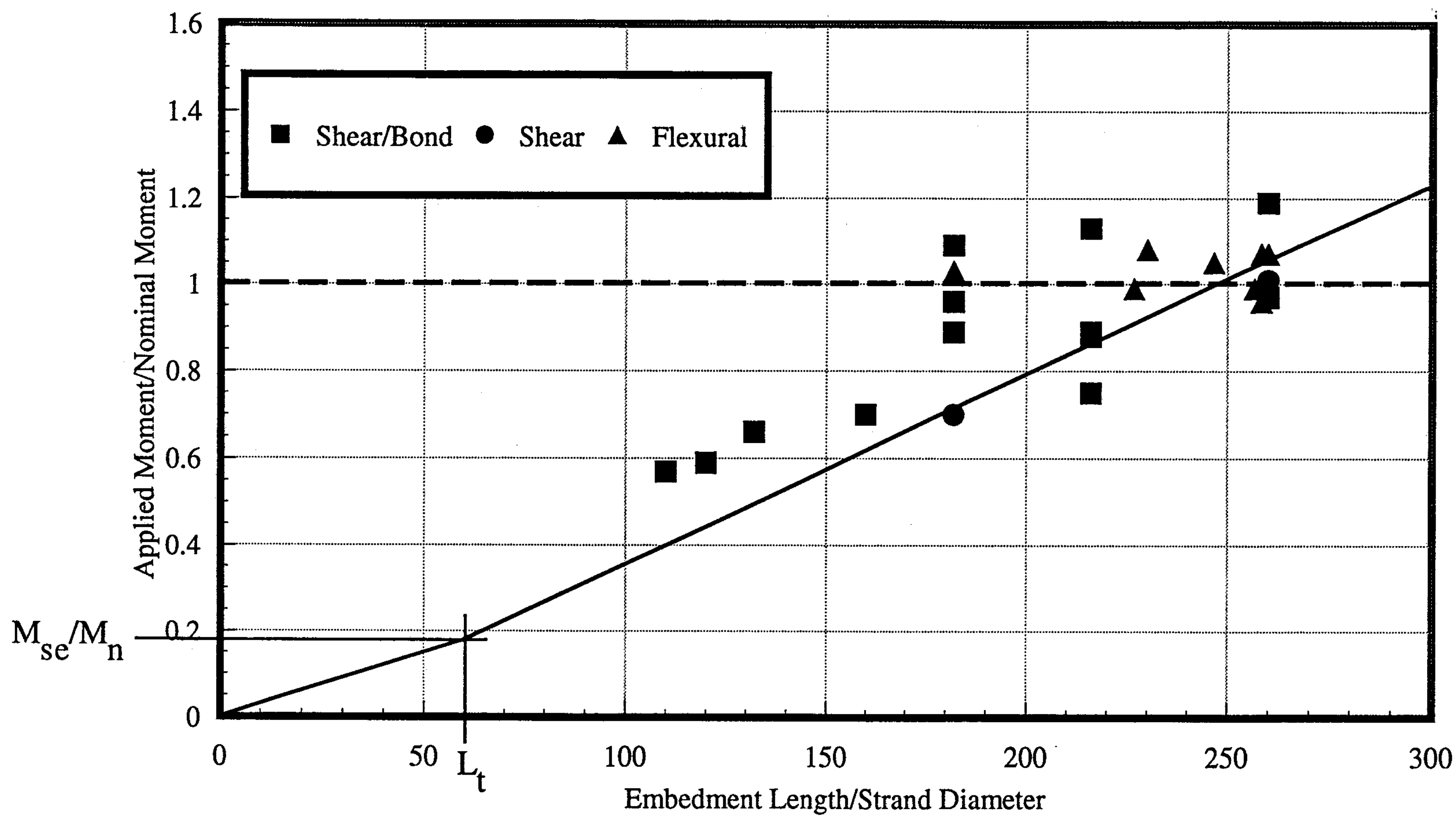


Figure 20 - Embedment Length/Strand Diameter vs. Applied Moment/Nominal Moment

AASHTO Type II Girders
(all strands combined)

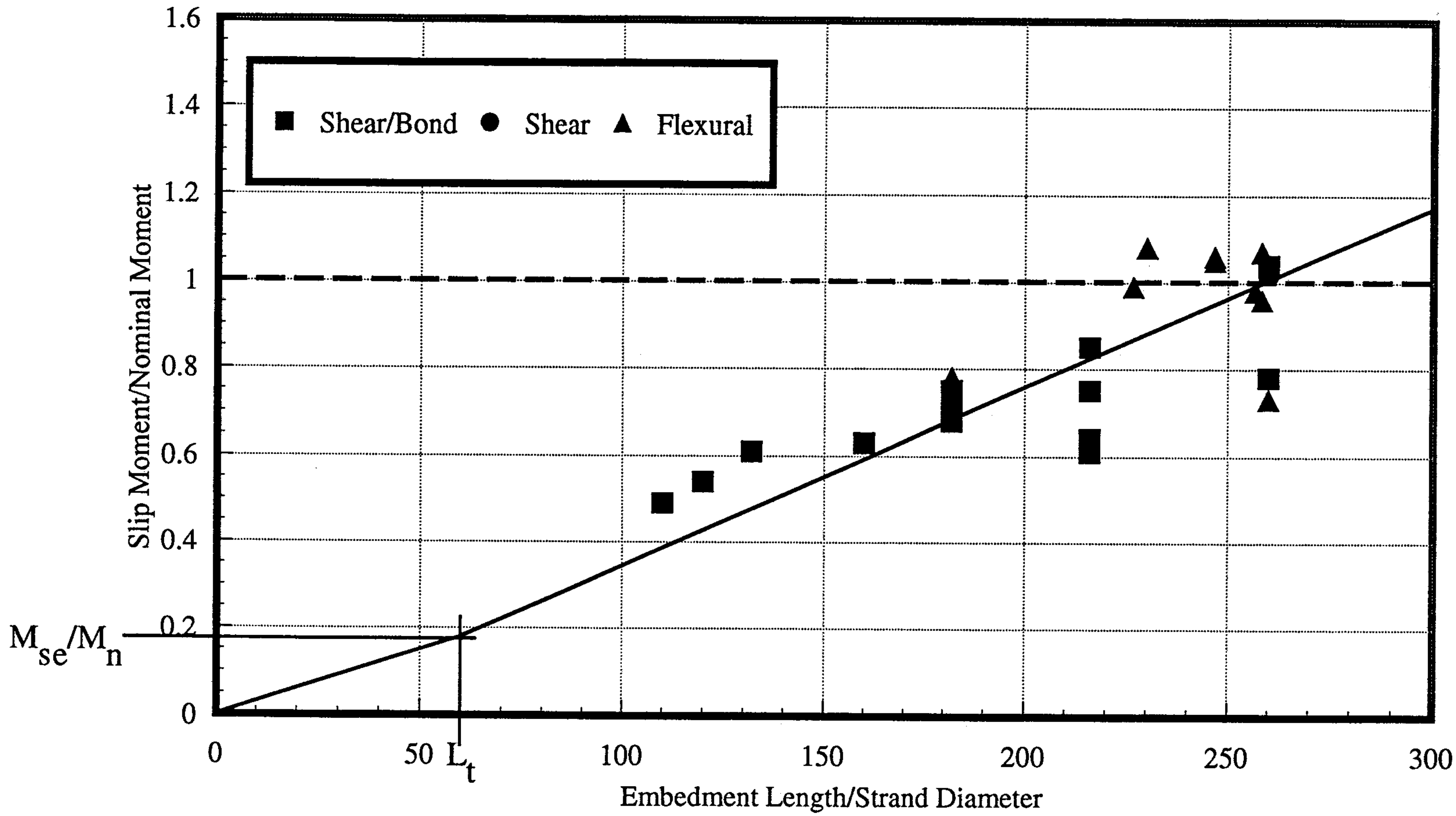


Figure 21 - Embedment Length/Strand Diameter vs. Slip Moment/Nominal Moment

AASHTO Type II Girders
(all strands combined)

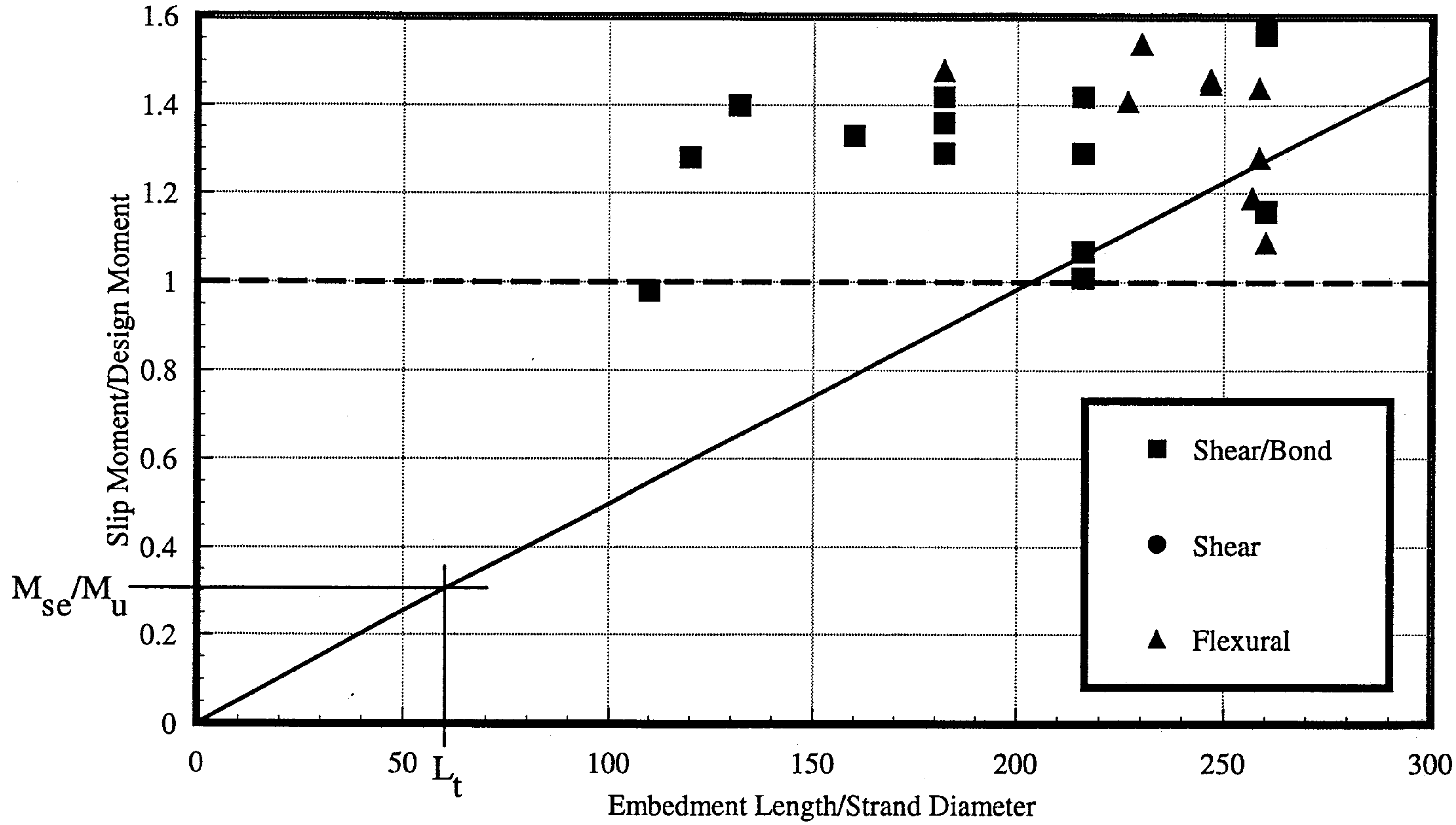


Figure 22 - Slip Moment/Design Moment vs. Embedment Length/Strand Diameter

AASHTO Type II Girders
(all strands combined)

reinforcement.) This was also found to be generally the case for the values of shear force at failure. Table 3(a) (Column 10) also reveals that in all tests, the failure moment, M.PP, exceeded the design moment value at the test section. These trends raise questions regarding the current philosophy on development length requirements in the end regions of a pretensioned girder. One of the most important observations from Figure 20 - 22 is that a development length of approximately 230 times strand diameter should ensure that full nominal moment can be developed in accordance with current code requirements. This contrasts with a value of approximately 154 D based on current AASHTO requirements.

Comparisons of measured versus calculated development lengths based on AASHTO and other recommendations are presented on Figures 23 to 25, and 25(a), inclusive. Figure 23 indicates that the AASHTO provisions are unconservative, whereas the use of 1.6 AASHTO gives reasonable, but slightly unconservative comparisons with measured values.

Figure 25(a) indicates that the MOT proposal gives the closest, yet conservative, results. The comparison with Buckner's proposal in Figure 24 shows that the method also yields reasonable but slightly unconservative results. On the other hand the FHWA proposal (Figure 25) yields consistently conservative results.

The effect of the magnitude of ϵ_{pr} on strand development length is again apparent from an examination of Figure 11. It is obvious that the values of ϵ_{ps} for all three sets of girders containing 0.5 in., 0.5 in. special and 0.6 in. strands, respectively, are well beyond the value of the yield strain. Also the values of ϵ_{ps} are much greater in the girders than in the prestressed slabs.

This explains why the development length requirements are greater for girders than for slabs, and confirms the recommendation by Buckner (Eqn. 8), that the value of E_{ps} should **be**

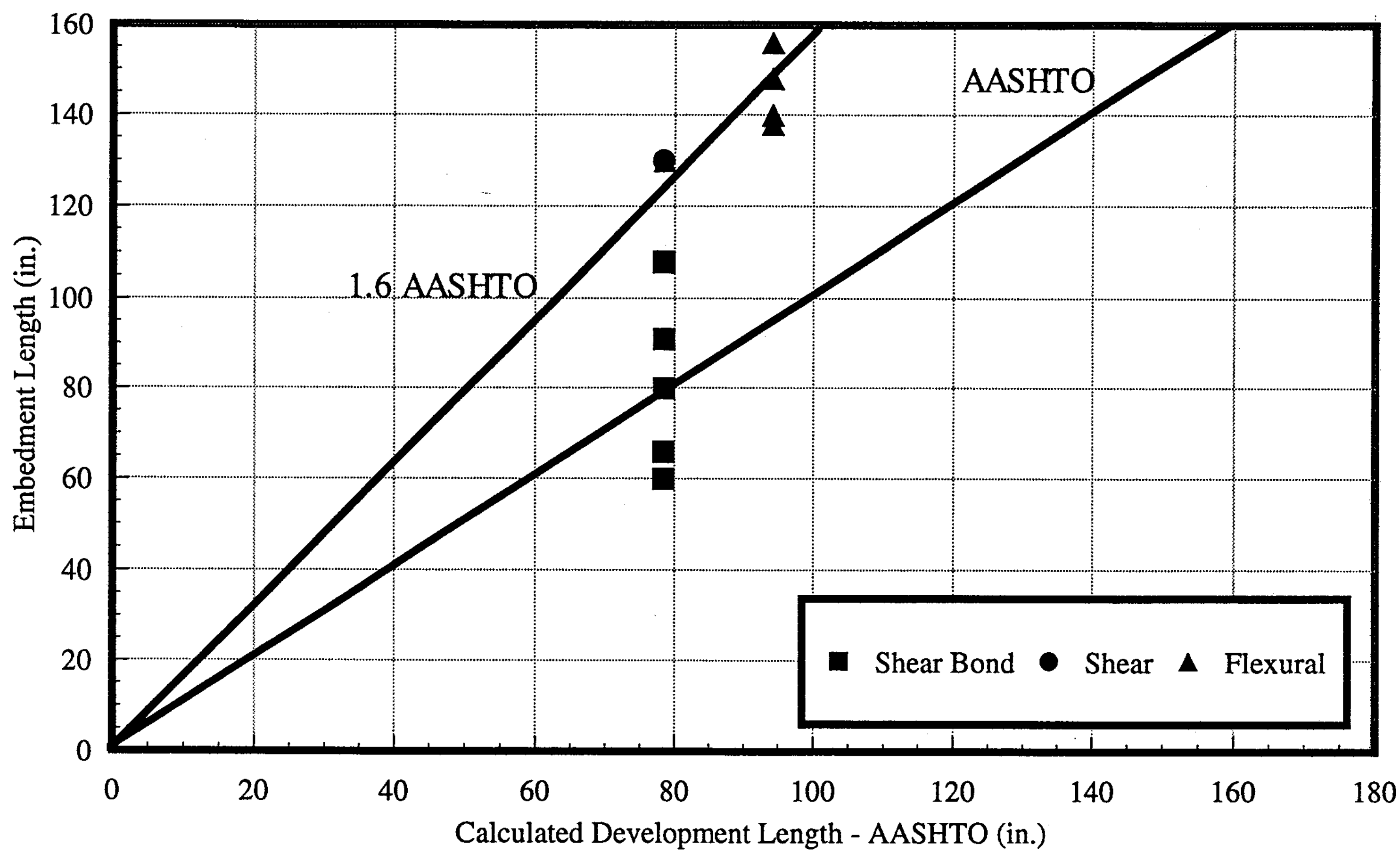


Figure 23 - Comparison of Embedment Length To AASHTO Calculated Development Length Girders

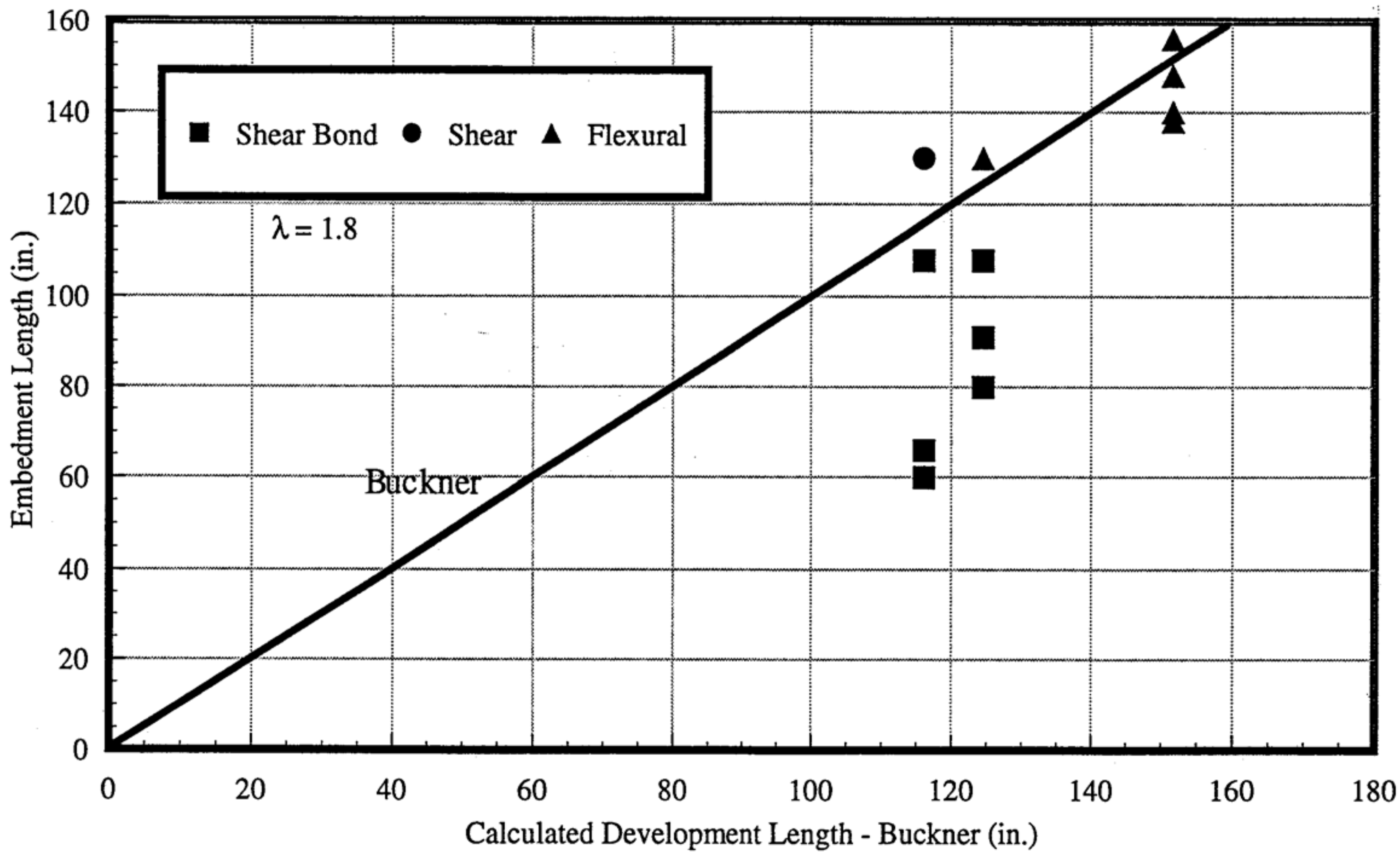


Figure 24 - Comparison of Embedment Length To Buckner Calculated Development Length Girders

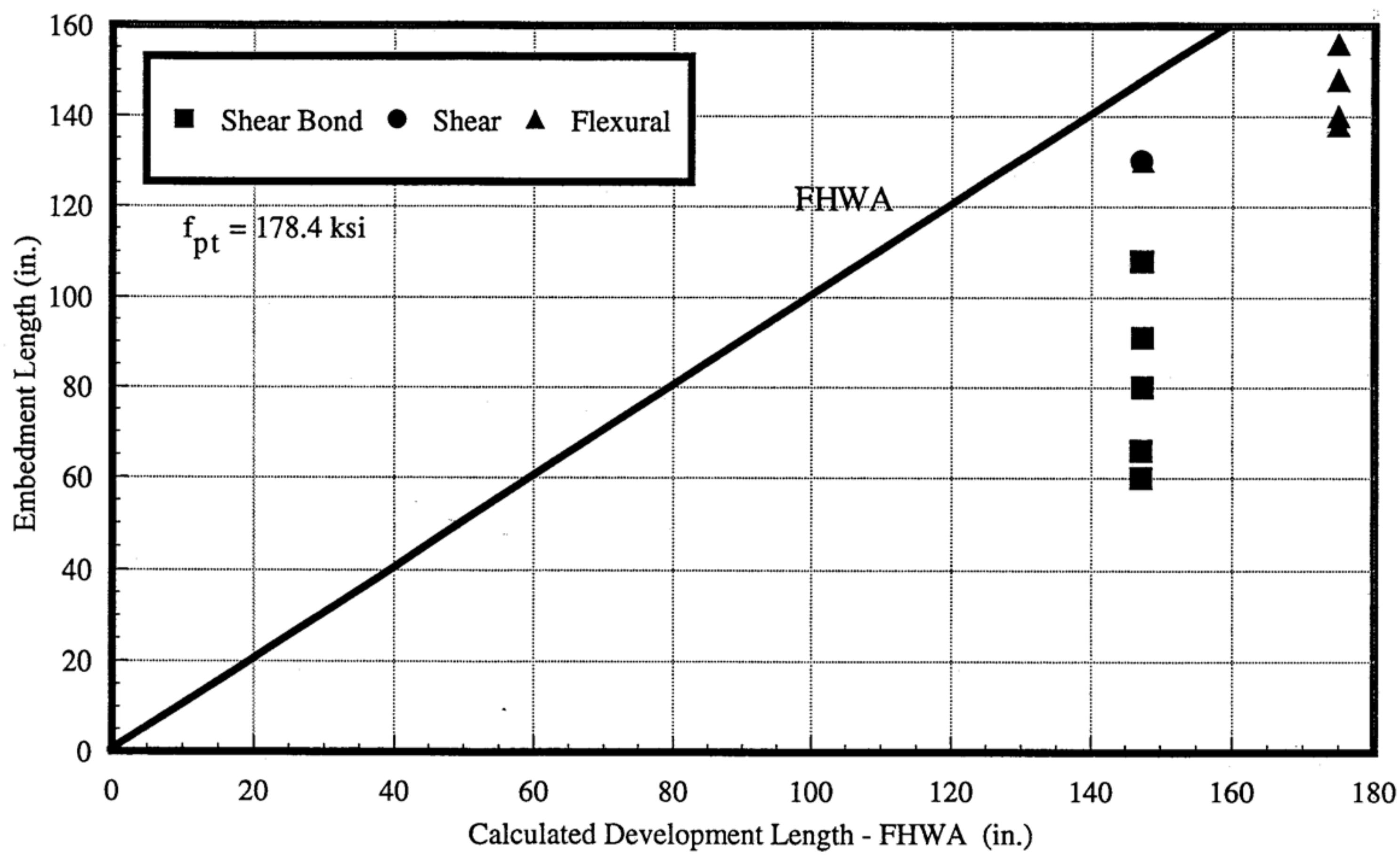


Figure 25 - Comparison of Embedment Length To FHWA Calculated Development Length Girders

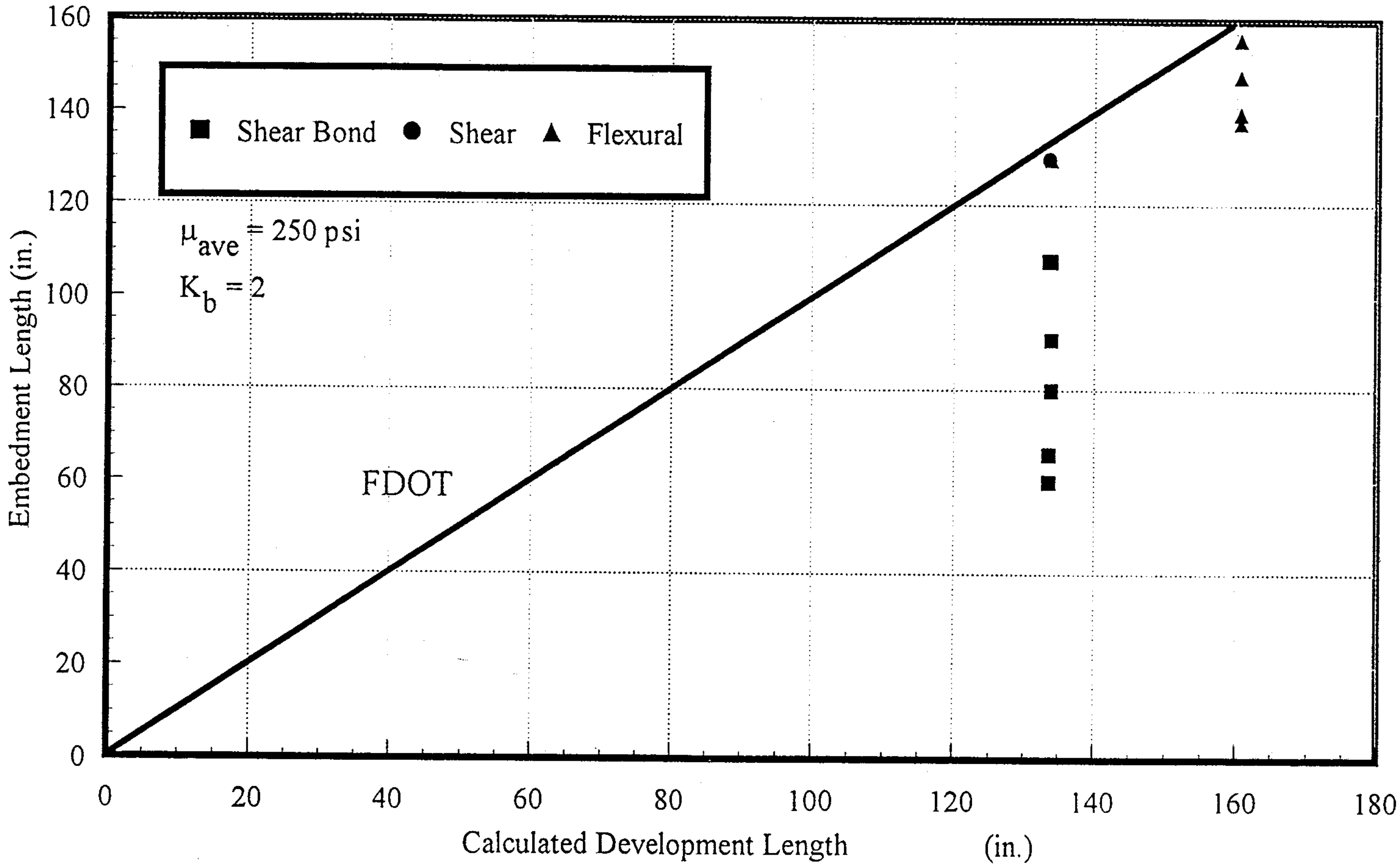


Figure 25(a) - Comparison of Embedment Length To FDOT Calculated Development Length Girders

taken into account in determining development length requirements. The use of kb in the FDOT expression (Eqn. 7) indirectly accounts for the effect of ep, on development length.

The foregoing points are clearly depicted in Appendix Figure B13, in which the other development length predictions are plotted against that of AASHTO. The girder test results shown in the plot indicate that both the Buckner and FDOT approaches yield acceptable results, while the AASHTO approach is unacceptable. However, the use of the 1.6 multiplier with the AASHTO prescription (1.6 AASHTO) is quite acceptable.

Figure B14 shows clearly that Zia and Mostafa's expression (Eqn. 5) is inadequate for use in.prestressed girders.

In the FHWA study, plots of measured embedment length versus (f_{su} - f_{se}) were presented. For completeness, such plots are presented for the AASHTO Type II girders in Appendix B (Figs. B 10 - B 12, inclusive) for the varying strand sizes, and with f'_c = 5000 psi.

With respect to the effect of confinement reinforcement, it was determined from the study that higher strength and higher ductility can be expected with the use of confinement reinforcement in the tension flange. The strength enhancement due to confinement can be inferred from Table 3(a) by comparing the strength ratios in Columns (10) - (12) for the specimens without confinement reinforcement (denoted by "D") with the other corresponding specimens.

3.6 CONCLUSIONS FROM GIRDER STUDY

The following conclusions are made from the study conducted on AASHTO Type II pretensioned girders.

1) Shear cracking in the end regions of girders affects bond behavior of strand.

2) The shear span/depth ratio has a marked effect on girder strength; however, all girders designed in accordance with AASHTO specifications can be expected to develop their design strengths in flexure and shear. At shear span/depth ratios (a/d), below 3.5, girder strength decreases linearly with a/d ratio.

3) If confinement reinforcement is provided, girders can be expected to develop in a ductile mode, strengths well beyond that at occurrence of initial slip of a strand.

4) Development length of 230 times bar diameter is appropriate for $^1/_2$ in. dia., $^1/_2$ in. special and 0.6 in. diameter strands. This justifies the 1.6 multiplier imposed by FHWA on development length calculations based on current AASHTO applications. However, the application of a factor to flexural development length, as recommended by Buckner and FDOT is more rational.

5) The FHWA proposal (Eqn. 12) yields consistently conservative results. On the other hand the Zia and Mostafa expression (Eqn. 5) is inadequate.

6) Spacing of strand at 2 in. center-to-center does not adversely affect girder behavior, with strand size up to 0.6 in diameter. The FHWA general prescription for a minimum strand spacing of four times diameter is, therefore, not warranted for strand diameter up to 0.6 in. diameter.

4. PRESTRESSED CONCRETE PILES

The first study of piles conducted by FDOT (Reference 6) examined the effect of pile embedment on the development length of $^1/_2$ in. diameter prestressing strand embedded in a pile cap or footing. It was considered that this type of end condition should result in lower development length requirements due to the enhancement of bond strength caused by shrinkage and by confinement of concrete. As a result of the findings of the experimental and analytical study, it was recommended that a minimum embedment length of 50 inches (100 D for $^1/_2$ in. dia. strands) be adopted for piles embedded in pier caps.

In subsequent studies by FDOT, prestressed piles were tested without simulation of the embedment conditions just referred to. Tests were conducted on piles having six different cross sections ranging from 14 in. x 14 in. to 30 in. x 30 in. The test specimens were cut from precast prestressed members, and the prestressing steel was $^1/z$ in. diameter or $^1/_2$ in. dia. special, except for the 18 in. x 18 in. piles in which 0.6 in. dia. strand was used. With the exception of three square hollow 30 in. x 30 in. piles, all the specimens had square solid cross sections. Test span length and shear span were other variables in the study.

The strand development lengths observed in this study are compared with those prescribed in the AASHTO specifications and with those recommended by others.

4.1 TEST SPECIMENS AND PROCEDURE

Figure 26 shows details of the six different cross sections of the test specimens. The piles, which were produced by various prestressed concrete manufacturers in Florida, were cut in the appropriate lengths, and transported to the FDOT Structural Research Center, where testing was conducted. The forty five tests pertaining to the subject of this report are summarized in

Table 5. Eight additional tests, in which a previously tested pile was tested after being repaired using carbon fiber sheet, are not discussed in this report. Testing consisted of the application of load up to failure, using an incremental point load at various distances from the support. Columns (1) to (6) of Table 5 give details of the loading parameters. As in the preceding tests, the specimens were instrumented to monitor deflections, strains, strand slip, loading and cracking continuously up to failure. Crack patterns were recorded for all specimens.

4.2 RESULTS

A summary of the test results is presented in Columns (7) and (8) of Table 5. Columns (9) - (11) give the values of calculated nominal moment, M_n, for each section as well as values of appropriate moment ratios.

Typical plots developed for the test specimens are shown in Figures 27 - 30 relating to specimen P18-45-1 in which strand slip occurred at a moment, M_{Slip}, less than M_n. Plots are presented in Figures 31 and 32 for specimen P18-63-1 in which strand slip was observed at approximately the point of failure. Crack patterns are shown in Figures 30 and 32. Slip and deflection plots similar to those shown in Figures 28 and 29 enabled a determination of the applied moment at initial strand slip, as well as the order in which strand slip occurred.

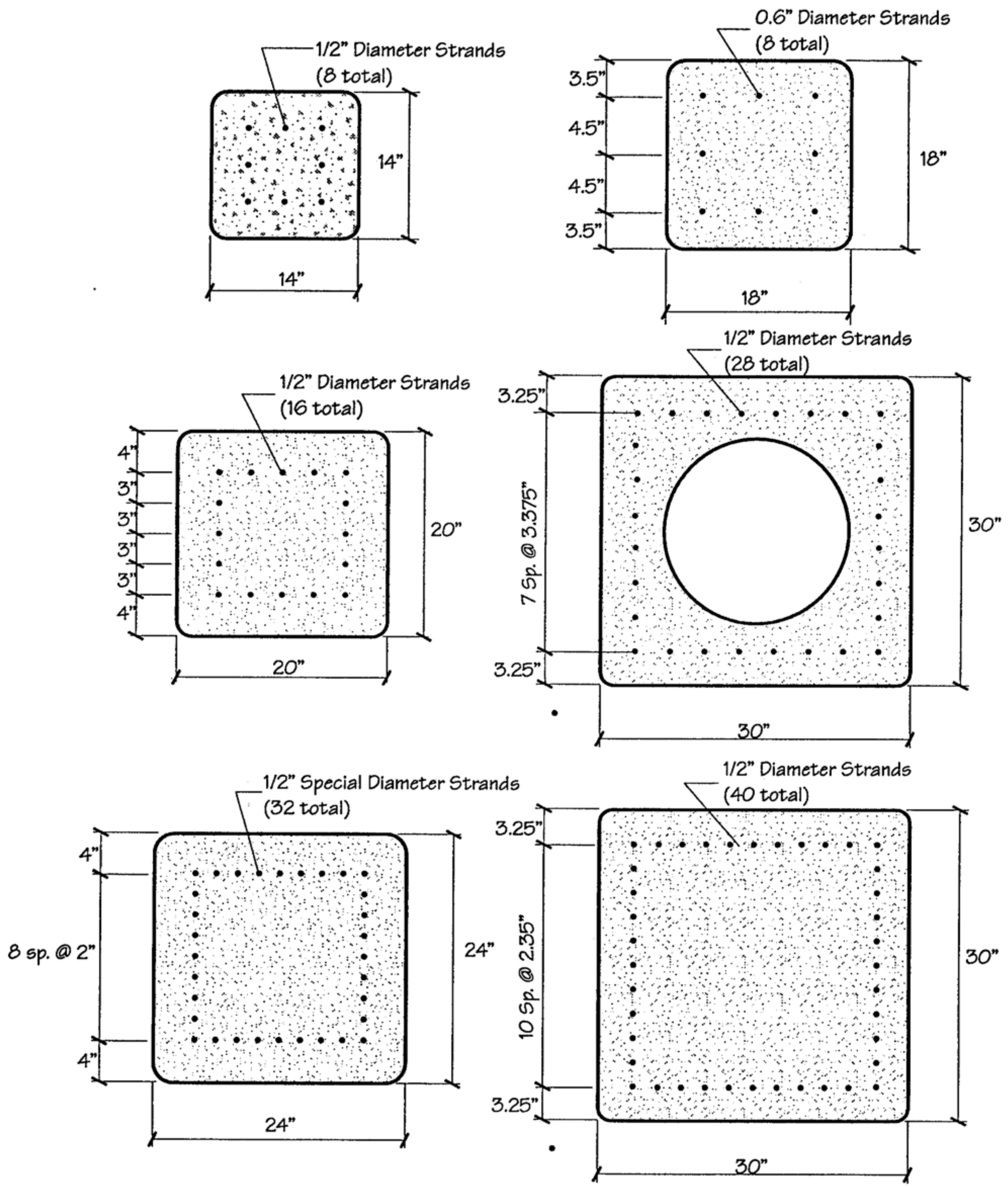


Figure 26 - Pile Cross Sections and Strand Configurations

Size (1)	Specimen (2)	End* (3)	Span (L) (4)	Shear Span** (a) (5)	a/h (6)	Slip Moment*** M_{slip} (7)	Failure Moment M_{app} (8)	Nominal Moment M_n (9)	M_{slip}/M_n (10)	M_{app}/M_n (11)	At M_n: ε_{ps} (micro) (12)	At M_n: f^*_{su} (ksi) (13)
14" x 14" (1/2" Strands)	14-28-1	C	130	28(34)	2.0	NA	137	120.3	NA	1.14	9782	245.1
	14-28-2	C	130	28(34)	2.5	NA	132	120.3	NA	1.10	9782	245.1
	14-35-1	C	156	35(41)	3.0	NA	139	120.3	NA	1.16	9782	245.1
	14-42-1	C	156	42(48)	3.0	NA	129	120.3	NA	1.07	9782	245.1
	14-42-2	U	182	42(48)	3.5	NA	143	120.3	NA	1.19	9782	245.1
	14-49-1	C	130	49(55)	2.0	NA	134	120.3	NA	1.11	9782	245.1
18" x 18" (0.6" Strands)	18-45-1	U	167	45(51)	2.5	192	233	239.4	0.80	0.97	11392	252.7
	18-45-2	U	167	45(51)	2.5	184	216	239.4	0.77	0.90	11392	252.7
	18-54-1	U	200	54(60)	3.0	184	210	239.4	0.77	0.88	11392	252.7
	18-54-2	U	200	54(60)	3.0	237	243	239.4	0.99	1.02	11392	252.7
	18-63-1	U	233	63(69)	3.5	NA	234	239.4	NA	0.98	11392	252.7
	18-63-2	U	233	63(69)	3.5	NA	234	239.4	NA	0.98	11392	252.7
20" x 20" (1/2" Strands)	20-50-1	C	185	50(56)	2.5	374	374	355.4	1.05	1.05	10410	248.8
	20-50-2	U	185	50(56)	2.5	216	307	355.4	0.61	0.86	10410	248.8
	20-50-3	U	185	50(56)	2.5	189	283	355.4	0.53	0.80	10410	248.8
	20-50-4	C	185	50(56)	2.5	240	359	355.4	0.66	1.01	10410	248.8
	20-50-5	U	185	50(56)	2.5	204	272	355.4	0.57	0.77	10410	248.8
	20-50-6	C	185	50(56)	2.5	261	307	355.4	0.73	0.86	10410	248.8
	20-50-7	C	185	50(56)	2.5	274	341	355.4	0.77	0.96	10410	248.8
	20-60-1	C	222	60(66)	3.0	390	467	355.4	1.10	1.31	10410	248.8
	20-60-2	U	222	60(66)	3.0	212	296	355.4	0.60	0.83	10410	248.8
	20-60-3	C	222	60(66)	3.0	343	390	355.4	0.97	1.10	10410	248.8
	20-70-1	C	259	70(76)	3.5	272	365	355.4	0.77	1.03	10410	248.8
	20-70-2	C	259	70(76)	3.5	215	383	355.4	0.60	1.08	10410	248.8
	20-70-3	C	259	70(76)	3.5	292	350	355.4	0.82	0.98	10410	248.8

* C = Cut end, U = Uncut End
** Brackted figures represent embedment (in.)
*** Moments in kip-ft

Table 5 – Pile Test Results

Size (1)	Specimen (2)	End* (3)	Span (L) (4)	Shear Span** (a) (5)	a/h (6)	Slip Moment*** M_{slip} (7)	Failure Moment M_{app} (8)	Nominal Moment M_n (9)	M_{slip}/M_n (10)	M_{app}/M_n (11)	At M_n: ε_{ps} (micro) (12)	At M_n: f^*_{su} (ksi) (13)
24" X 24" (1/2" SPECIAL STRANDS)	24-48-1	C	234	48(56)	2.0	414	552	776.8	0.53	0.71	8177	223.3
	24-48-4	C	222	48(56)	2.0	586	737	776.8	0.75	0.95	8177	223.3
	24-60-1	C	222	60(66)	2.5	895	1047	776.8	1.15	1.35	8177	223.3
	24-60-2	C	222	60(66)	2.5	NA	1098	776.8	na	1.41	8177	223.3
	24-60-3	C	222	60(66)	2.5	970	970	776.8	1.25	1.25	8177	223.3
	24-72-1	U	267	72(78)	3.0	992	1055	776.8	1.28	1.36	8177	223.3
	24-72-2	C	267	72(78)	3.0	1035	1035	776.8	1.33	1.33	8177	223.3
	24-84-1	U	311	84(90)	3.5	1005	1005	776.8	1.29	1.29	8177	223.3
	24-84-2	C	311	84(90)	3.5	710	895	776.8	1.01	1.15	8177	223.3
30" X 30" (1/2" STRANDS)	30-60-1	C	222	60(66)	2.0	1152	1242	1283.5	0.90	0.96	10820	250.6
	30-60-2	C	222	60(66)	2.0	1396	1485	1283.5	1.09	1.16	10820	250.6
	30-60-3	U	222	60(66)	2.0	876	1097	1283.5	0.68	0.85	10820	250.6
	30-75-1HC	C	240	75(81)	2.5	751	1002	1065.1	0.71	0.94	13437	257.2
	30-72-1	U	240	72(78)	3.0	1400	1485	1283.5	1.09	1.16	10820	250.6
	30-72-2	C	240	72(78)	3.0	1200	1550	1283.5	0.93	1.21	10820	250.6
	30-90-1	C	334	90(96)	3.0	NA	1473	1283.5	NA	1.15	10820	250.6
	30-90-2	U	334	90(96)	3.0	1197	1443	1283.5	0.93	1.12	10820	250.6
	30-105-1	C	389	105(111)	3.5	NA	1387	1283.5	NA	1.08	10820	250.6
	30-60-1HC	U	222	60(66)	3.0	438	825	1065.1	0.41	0.77	13437	257.2
	30-60-2HC	C	222	60(66)	2.0	514	859	1065.1	0.48	0.81	13437	257.2

* C = Cut end, U = Uncut End
** Brackted figures represent embedment (in.)
*** Moments in kip-ft

Table 5 (continued) – Pile Test Results

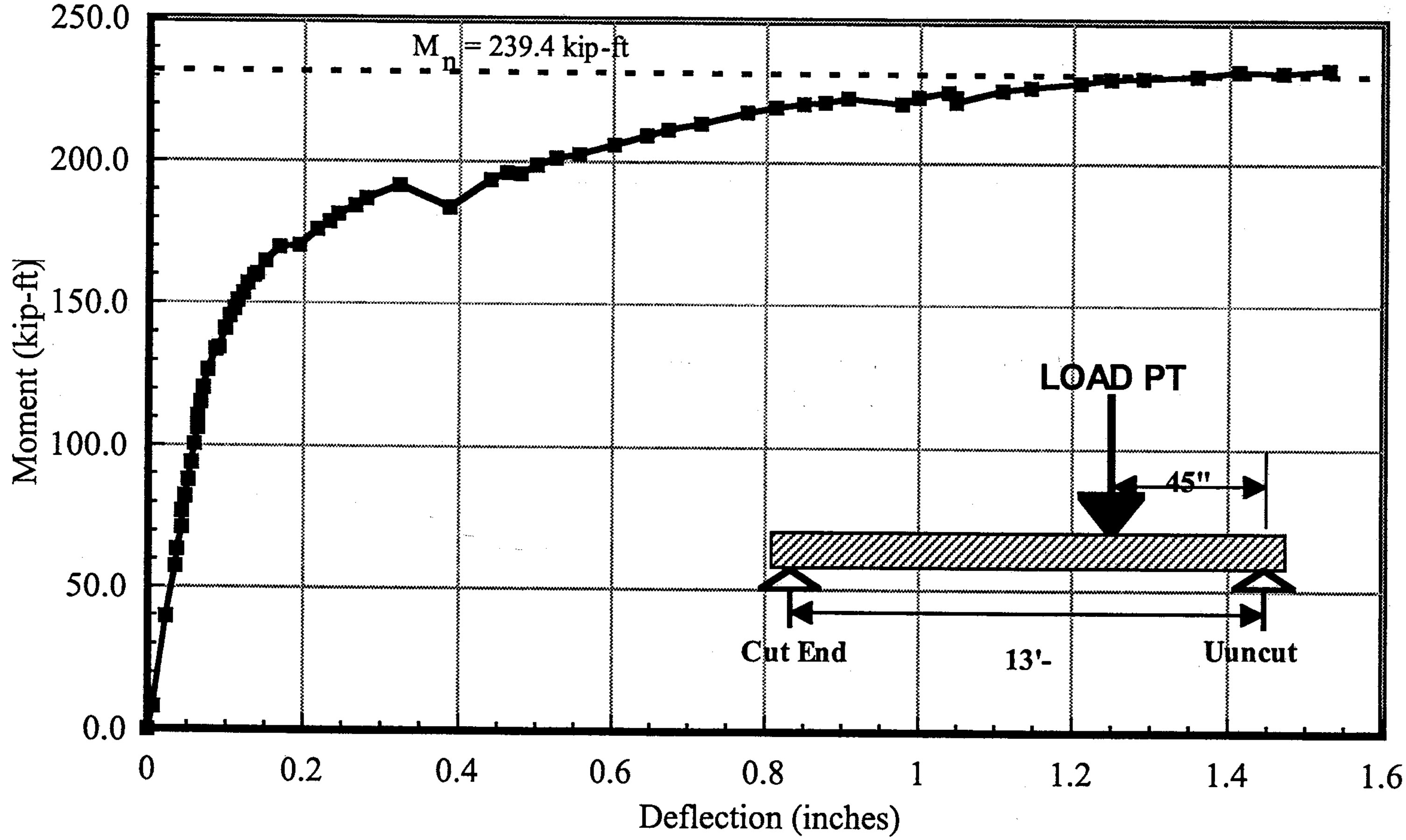


Figure 27 - Moment vs. Load Point Deflection
Pile #P18-45-1

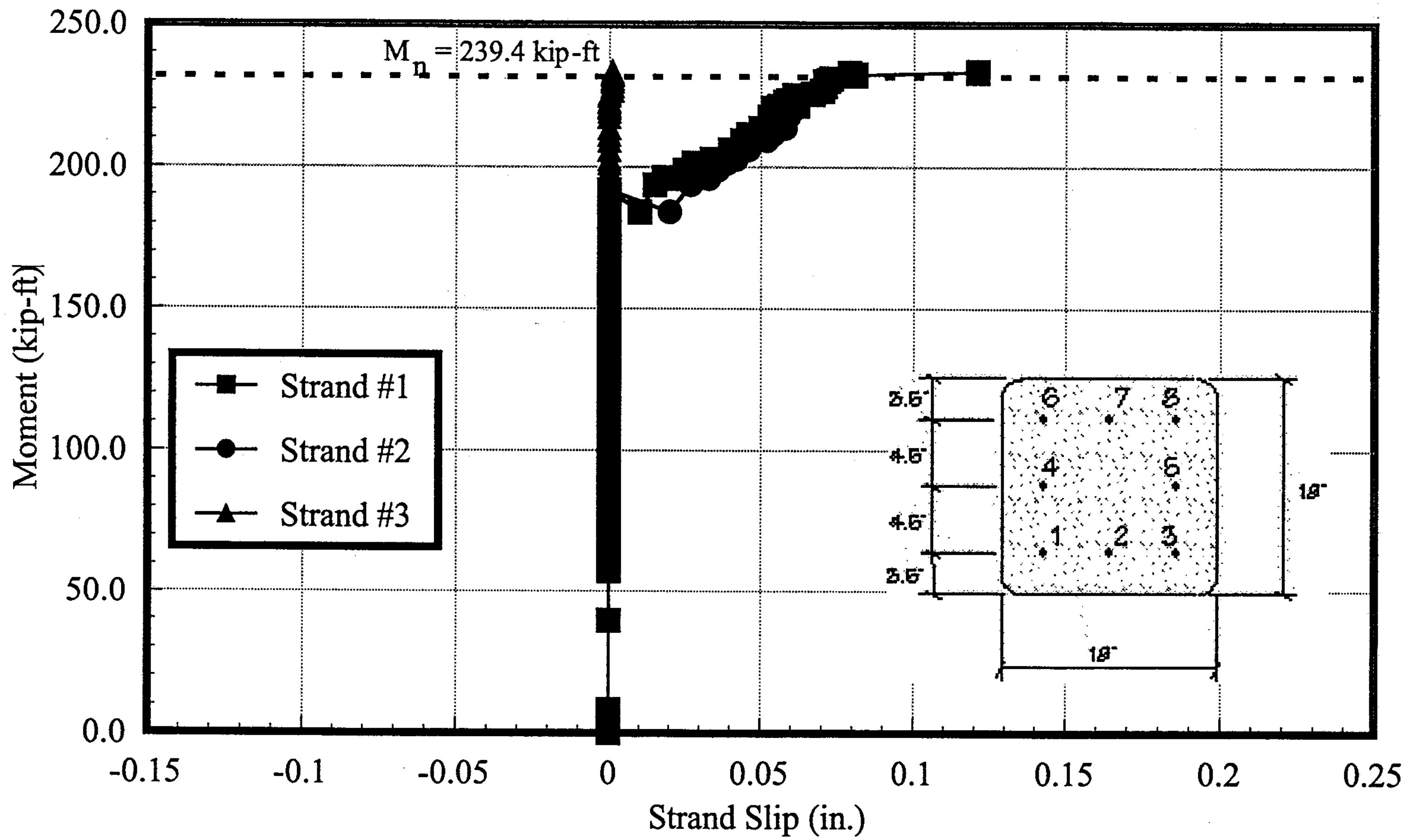


Figure 28 - Moment vs. Strand Slip
Pile #P18-45-1

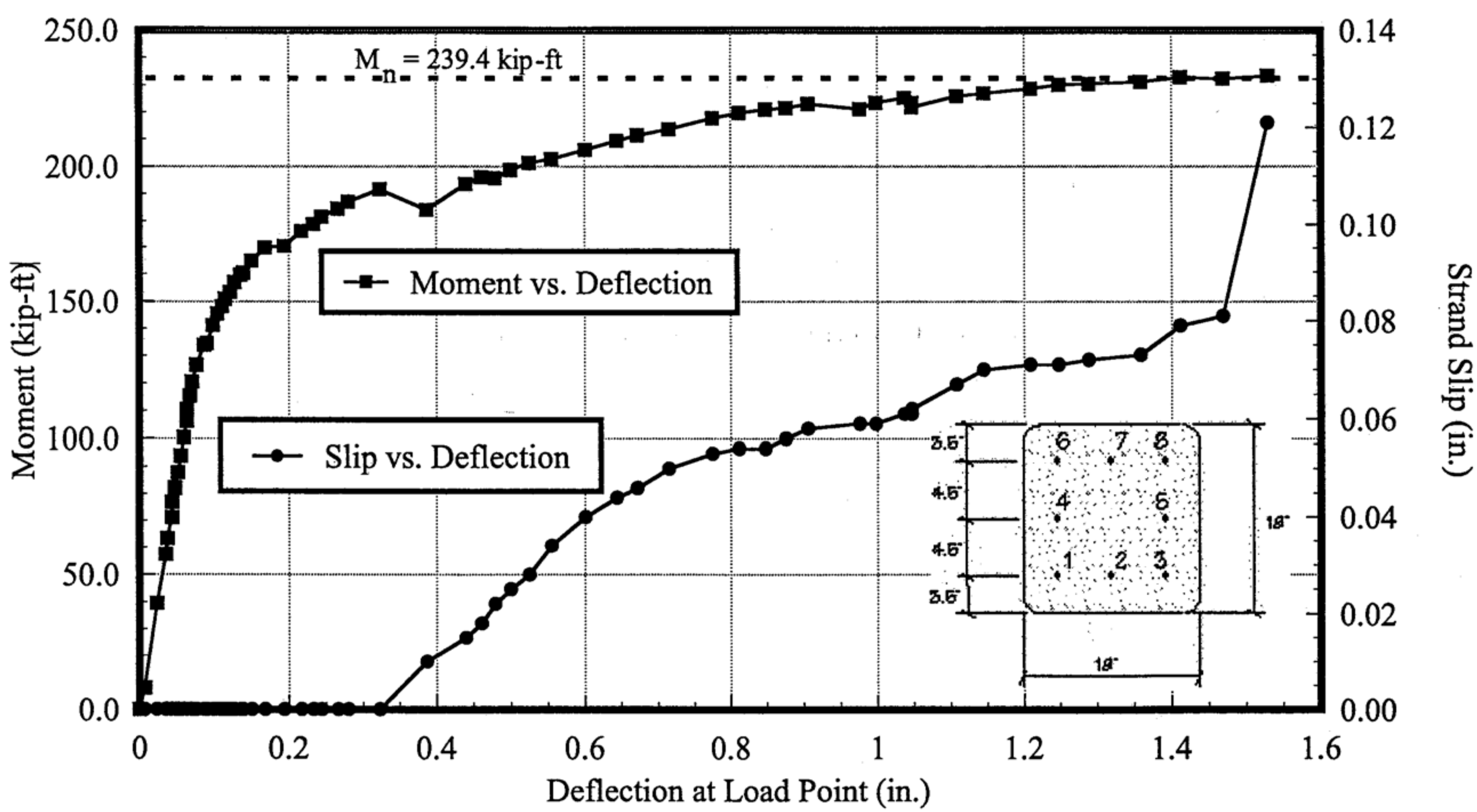
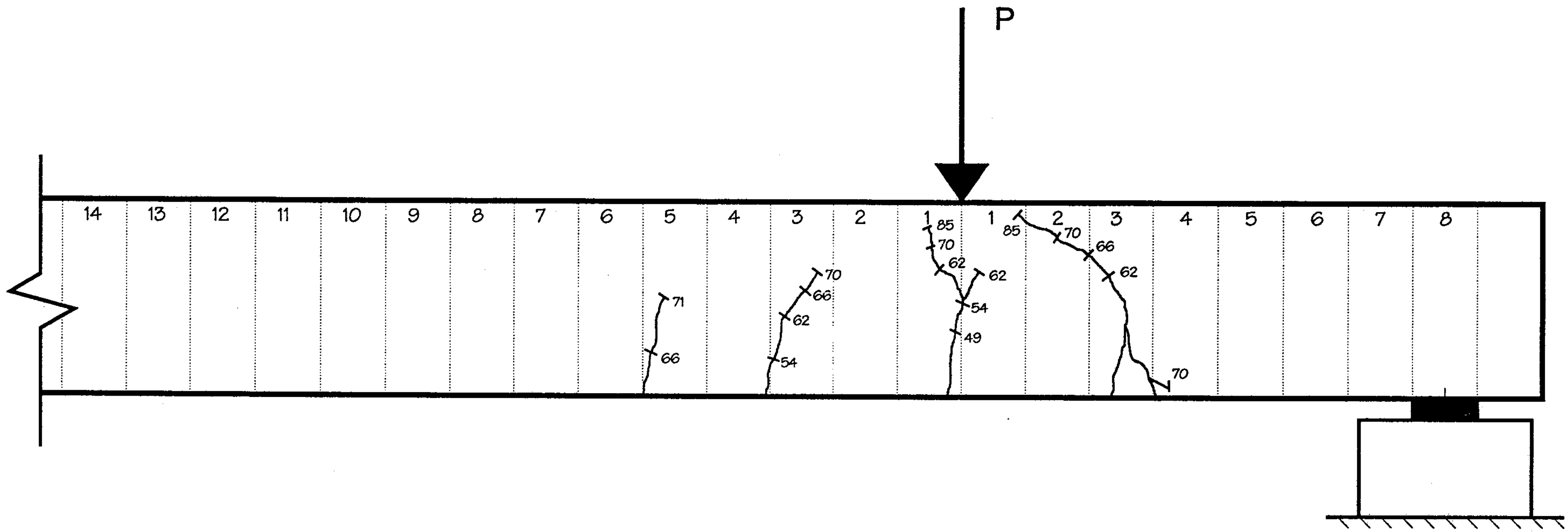


Figure 29 - Moment and Slip vs. Deflection
Pile #P18-45-1



UNCUT END
LOAD POINT AT 45" FROM CENTER OF BEARING
TOTAL SPAN = 167"
SLIP OCCURS @ P=70 Kips M= 192 Kip-ft
FAILURE OCCURS @ P=85 Kips M= 233 Kip-ft
TEST DATE: 5/23/94

Figure 30 - Pile #P18-45-1 Crack Pattern

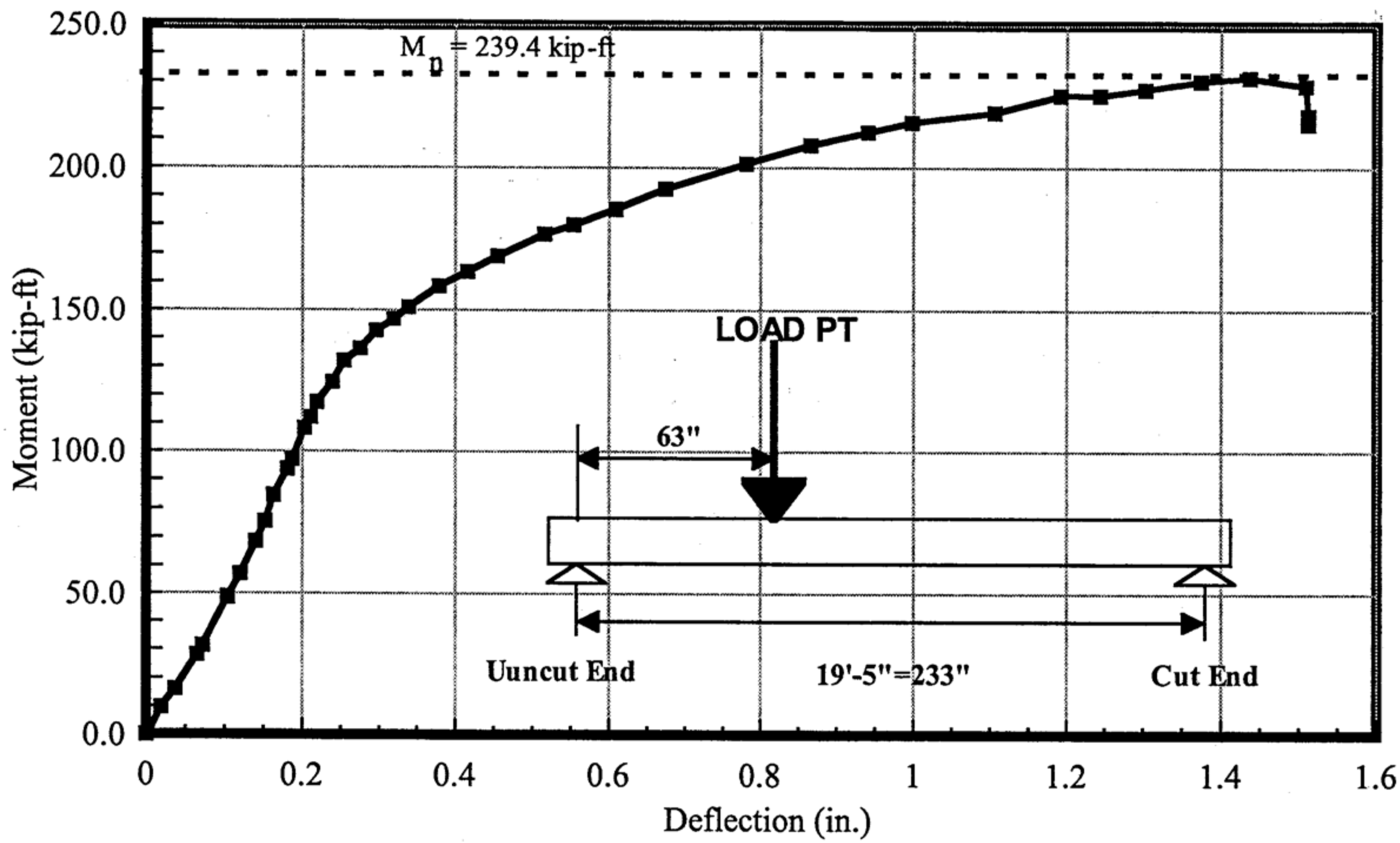
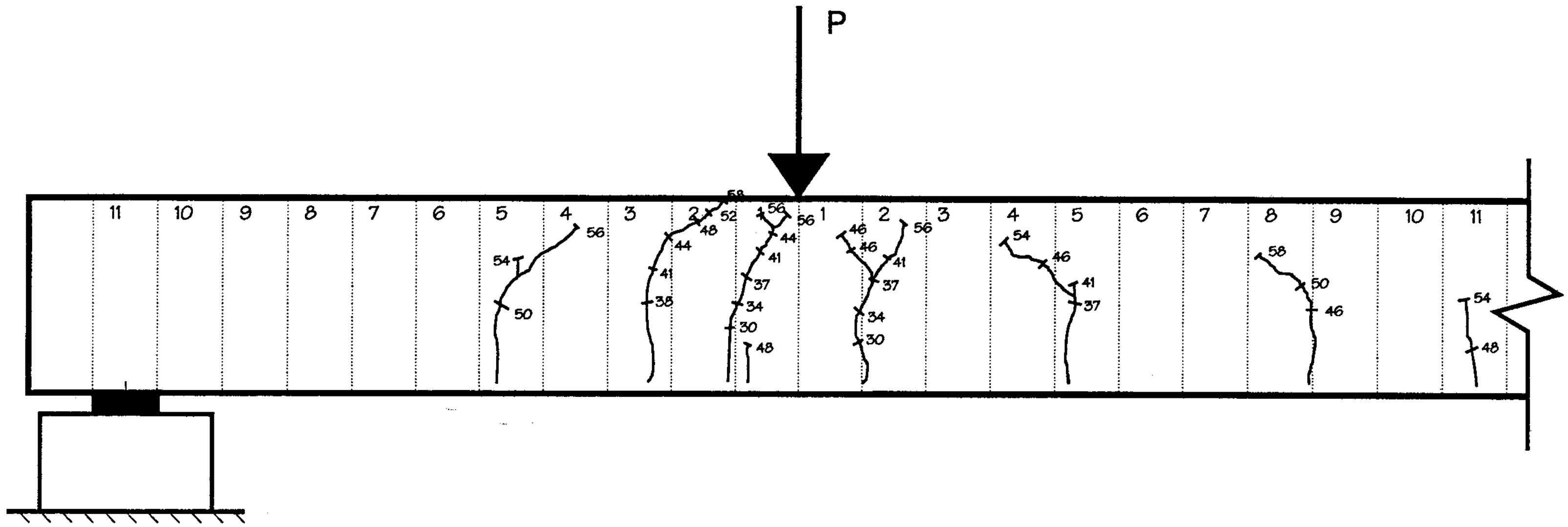


Figure 31 - Moment vs. Load Point Deflection
Pile #P18-63-1



UNCUT END
LOAD POINT AT 63" FROM CENTER OF BEARING
TOTAL SPAN = 233"
SLIP OCCURS @ NA
FAILURE OCCURS @ P=61 Kips M= 234 Kip-ft
TEST DATE: 5/20/94

Figure 32 - Pile #P18-63-1 Crack Pattern

4.2.1 General Observations

Specimens were prepared from longer lengths of piles; therefore, the end conditions of the specimens were not all identical. For example, the uncut ends had more closely spaced spiral reinforcement than the cut ends. The larger pitch of spiral could adversely affect transfer length at the cut ends; however, the higher concrete strength would also enhance transfer characteristics. Generally, the value of M_{app}/M_n (Col. 11, Table 5) was not significantly affected by whether the end was cut or uncut.

The specific trends for each pile set will now be briefly presented, and then the combined tests will be examined for general trends regarding development length.

4.2.1.1 14 in. x 14 in. Piles (½ in. dia strand)

Six tests were conducted, including only one uncut end (Table 5). Embedment length varied from 34 in. to 55 in. giving four different values of embedment. No slip of strand was observed in any of these tests, and the values of maximum applied moment, M_{app}, were greater than M_n values in all tests.

Examination of Fig. 11 and Table 5 reveals that the value of the strand stress, f^*_{su}, at nominal strength was well below the yield stress, f_{py}, thus resulting in lower strand development requirements.

The failures were mainly flexural, with crushing of concrete occurring before yield of reinforcement. Embedment length as low as 34 in. (68 D) was adequate to ensure that no strand slip occurred at failure, i.e., $M_{app} > M_n$.

Figure 33 shows plots of calculated development length versus embedment length for different predictions. This figure shows that even the unmodified AASHTO specification is conservative. The FDOT and Buckner's proposals yield identical and excellent predictions. The FHWA proposal yields the same results as the two latter methods, although it would tend to yield more conservative results at lower AASHTO calculated values.

4.2.1.2 18 in. x 18 in. Piles (0.6 in. dia. strand)

All six tests were conducted with uncut ends. Embedment of strand varied between 51 in. (98.3 D) and 69 in. (115 D) with three different values of embedment.

Strand slip was observed in three tests in which M_{app}/M. = 0.9; and embedment length was 60 in. or 51 in. Fig. 11 shows that the value of f*,,, = 252.7 ksi; was just lower than f_{py} (253 ksi). The predominant failure mode was observed to be that of flexure with concrete crushing.

Comparisons of the different proposals for development length are shown in Figure 34. It is seen that for 0.6 in. dia. strand, the basic AASHTO expression gives adequate, conservative values. All other models are more conservative than AASHTO, with Buckner and FDOT proposals giving slightly more conservative results. However the FHWA method is very conservative.

4.2.1.3 20 in. x 20 in. Piles (^{1}/z in. dia. strand)

Thirteen tests were conducted, four with uncut ends, and nine with cut ends. Embedment of strand was 56 in., 66 in., or 76 in.

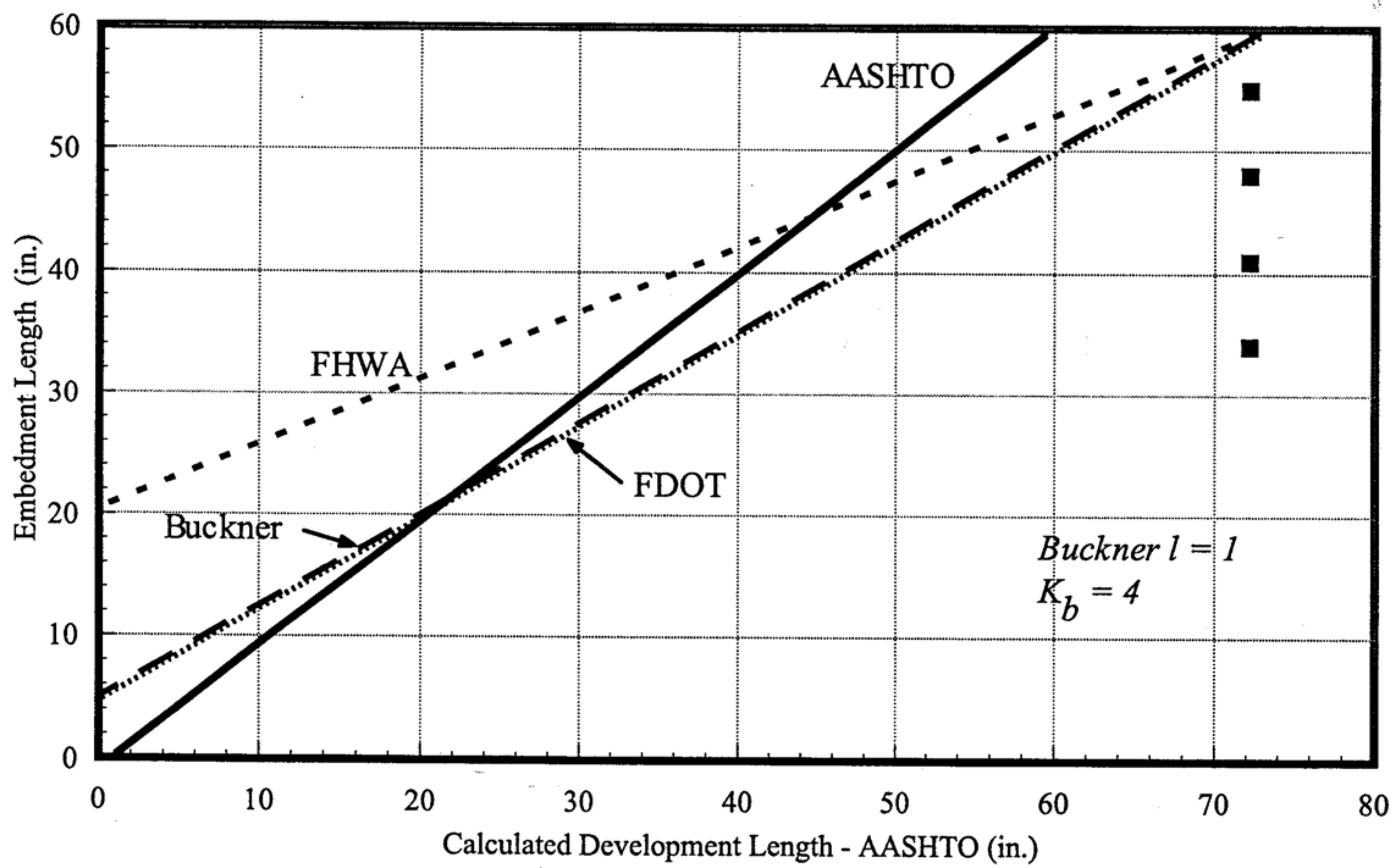


Figure 33 - Development Length, Comparison of AASHTO and Other Predictions

14 in. Piles

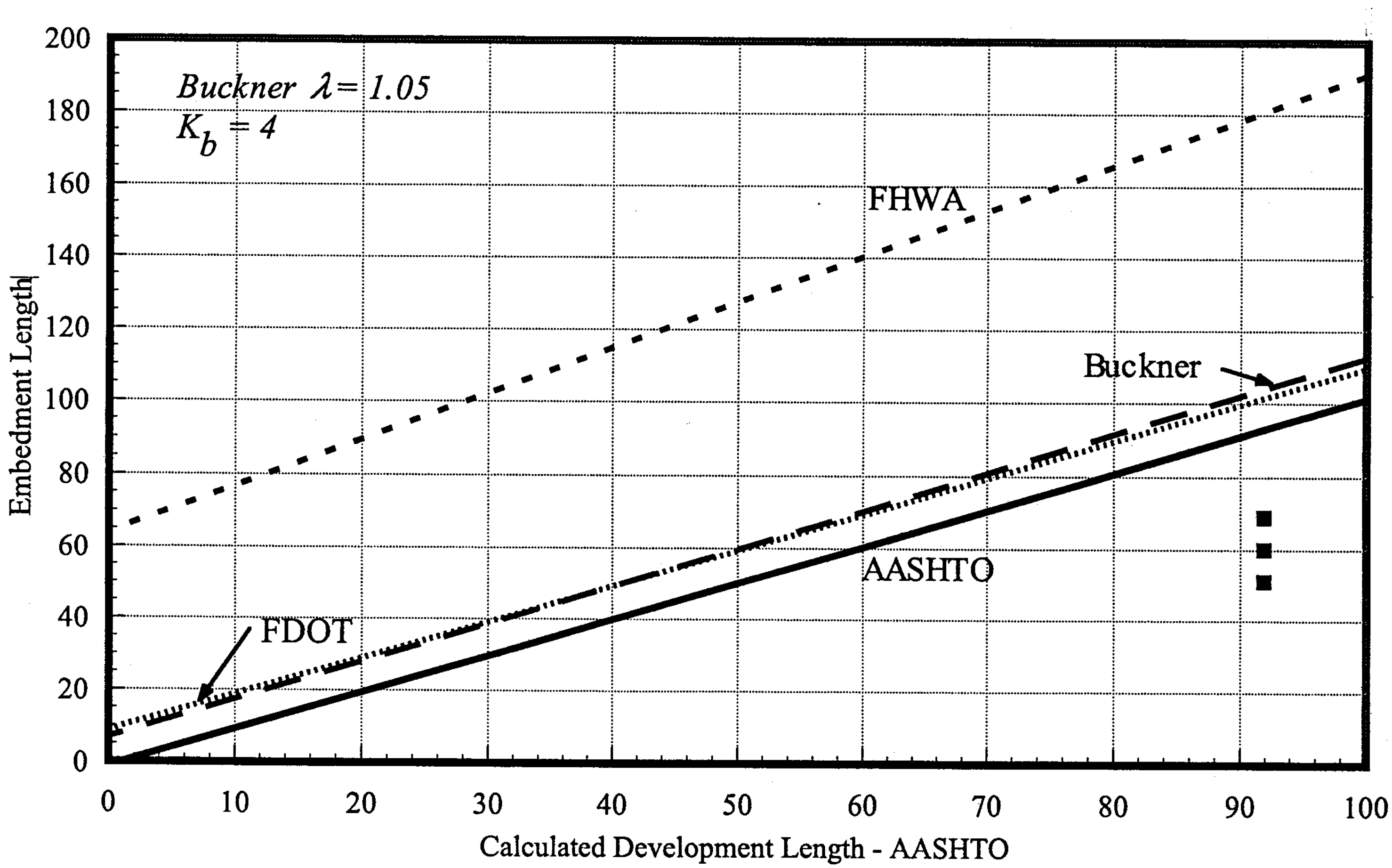


Figure 34 - Development Length Comparison of AASHTO and Other Predictions

18 in. Piles

Except for two specimens (20-50-1 and 20-60-1), all specimens exhibited some slip before attainment of M_n (Table 5-Column 10). Note that although slip occurred somewhat early in those specimens having a strand embedment of 76 in., it was observed that slip occurred in just one or two strands before M_n was developed. It is interesting that the cut ends, generally, have higher values of moments at slip and failure than the uncut ends.

Figure 11 shows that f^*_{su} was less than f_{py} for these specimens. Failure mode was observed to be either flexure-compression or shear bond.

The plots in Figure 35 indicate that the basic AASHTO provision for development is adequate (without the multiplier). The other methods yield conservative results with the FHWA method giving extremely conservative results.

The coincidence of the FDOT and Buckner's proposals up to the 20 in. x 20 in. piles, is due to the fact that substitution of $\lambda = 1.0$ in the Buckner equation gives almost identical results if values of kb = 4, and $u_{av} = 250$, are used in the FDOT equation.

4.2.1.4 24 in. x 24 in. Piles ($^1/_2$ in. dia. special strand)

A total of nine tests were performed, two involving uncut ends, and seven with cut ends. Embedment of strand varied in four stages from 56 ins. to 90 ins.

Only two specimens exhibited strand slip before the application of M_n. In specimen 24-84-2 C, a horizontal crack was observed to exist in the vicinity of the loaded end, which may explain the relatively low failure moment, which was still greater than M_n.

The value of f^*_{su} in these specimens was 222.3 ksi i.e. well below f_{py} (Fig. 11). Failure modes were mainly flexure-compression, with the specimens having lower a/h values failing in a shearlbond mode.

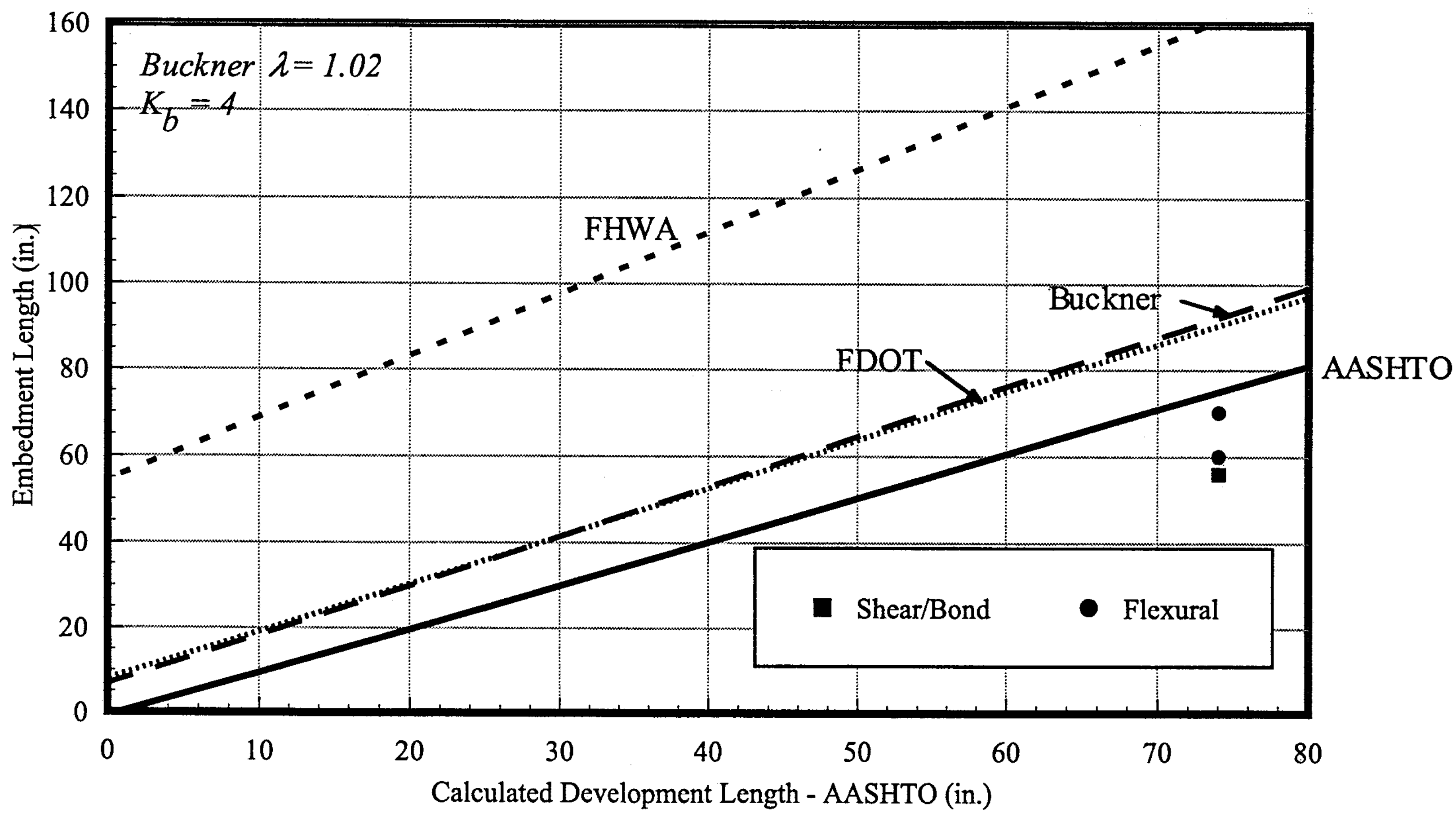


Figure 35 - Development Length Comparison of AASHTO and Other Predictions

20 (in.) Piles

From Figure 36, it can be seen that the provisions of 1.6 AASHTO are adequate, while the Buckner and the basic AASHTO predictions are unconservative. The FHWA and FDOT proposals are extremely conservative. An embedment length of 66 inches. (132 D) appears to be adequate for these specimens. The divergence between the MOT and Buckner plots is due to the fact that for these specimens the value of $k_b = 2$, instead of 4, in the MOT proposal for the piles greater than 20 in. x 20 in.

4.2.1.5 30 in. x 30 in. Piles ('/2 in. dia. strand)

Eleven tests were conducted. Eight of these tests involved solid piles (4 uncut, 4 cut ends), and three tests involved square hollow piles (2 cut ends, 1 uncut end). Embedment length was varied in five stages from 66 in. (132 D) to 111 in. (220 D).

Except for the specimens with lower a/h ratios (embedment = 66 in.), most of the solid piles attained M_n without strand slip. Slip was observed in all the specimens with hollow sections (HC). Fig. 11 and Table 5, indicate that the value of f^*_{su} for the solid sections was 250.6 ksi (< f_{py}), and the corresponding value for the hollow sections was 257.2 ksi (> f_{py}), hence the strain in prestressing steel at ultimate was nearly 20 percent higher in the hollow section.

The solid specimens failed mainly in flexure-compression, while the hollow sections exhibited shear/bond failure. Figure 37 shows that the provisions of 1.6 AASHTO are satisfactory, while Buckner's proposal is not. The FHWA and FDOT proposals yield conservative results.

In the FHWA study, plots of measured embedment length versus (f^*_{su} - f_{se}) were developed.

Similar **plots for the pile tests are presented in Appendix Figs. C1 to C5, inclusive,**

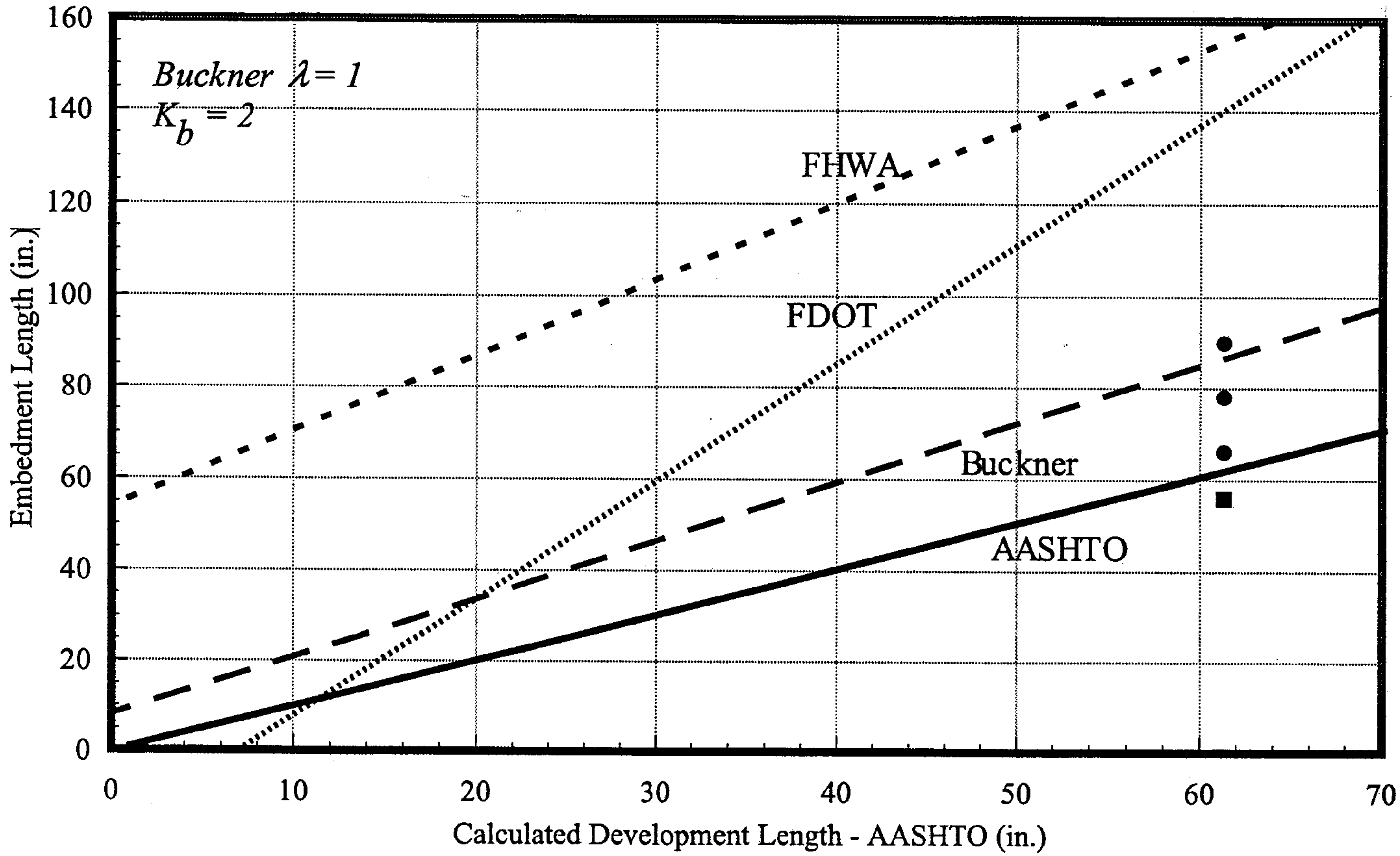


Figure 36 - Development Length Comparison of AASHTO and Other Predictions

24 in. Piles

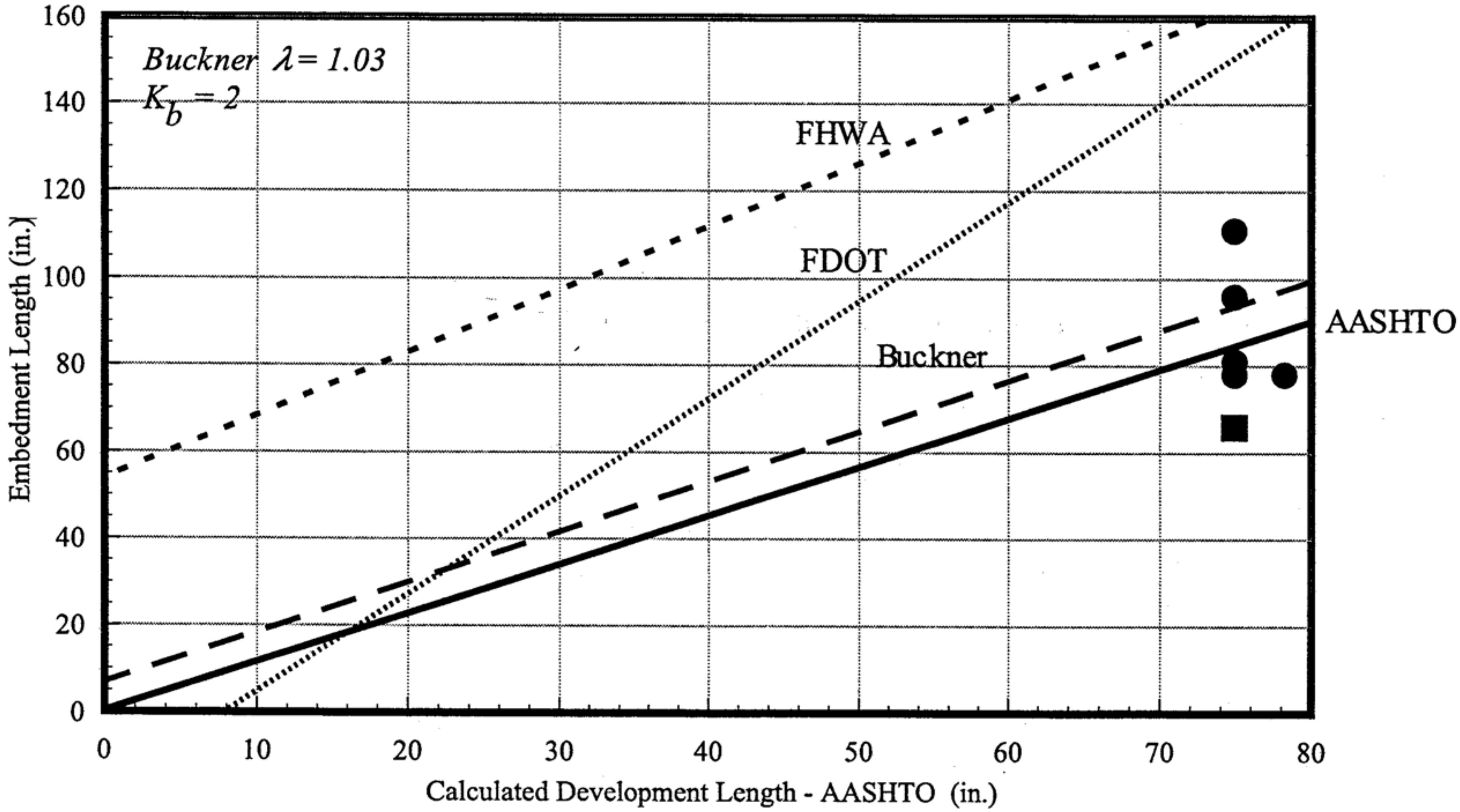


Figure 37 - Development Length Comparison of AASHTO and Other Predictions

30 in. Piles

for f'_c = 5,000 psi. These plats generally confirm the conclusions already reached with respect to the application of each proposal for calculating development length of strand in piles.

4.3 OBSERVATIONS FOR ALL PILES COMBINED

In Figure 38, the results of all the pile tests are shown plotted for Applied Moment ratio vs. Embedment Length ratio. This plot shows that at an embedment of as little as 140 times strand diameter, the undesirable shear-bond failure mode can be avoided. In Fig. 39, however, it can be seen that 1.6 AASHTO is appropriate, while the basic AASHTO equation is unconservative. This can be explained by an examination of Fig. 11, which indicates that the values of f^*_{su} for the larger piles are just below or just above the value of the yield stress of the strand. Thus, a development length based on 1.6 AASHTO would be satisfactory for all the piles tested, although the multiplier would be wasteful if applied to piles with smaller cross sections.

Examination of Fig. 40 indicates that Buckner's equation is unconservative, i.e. value of X in the equation may require more fine-tuning. Fig. 41 shows that the FHWA equation is too conservative for piles, while Fig. 42 indicates that the FDOT proposal yields excellent results for piles.

Typical FHWA plots of embedment length versus (f^*_{su} - f_{se}) for f'_c = 5 ksi are presented in Appendix C, confirm that 1.6 AASHTO is appropriate for all size of piles while the basic AASHTO equation is adequate for piles with sections 18" x 18" and lower. The latter figures also include the plots based on the FDOT (Shahawy) equation. It is clear that the FDOT proposal is adequate for all sizes of piles tested. The inadequacy of Zia and Mostafa's proposal is evident from Figure (C6).

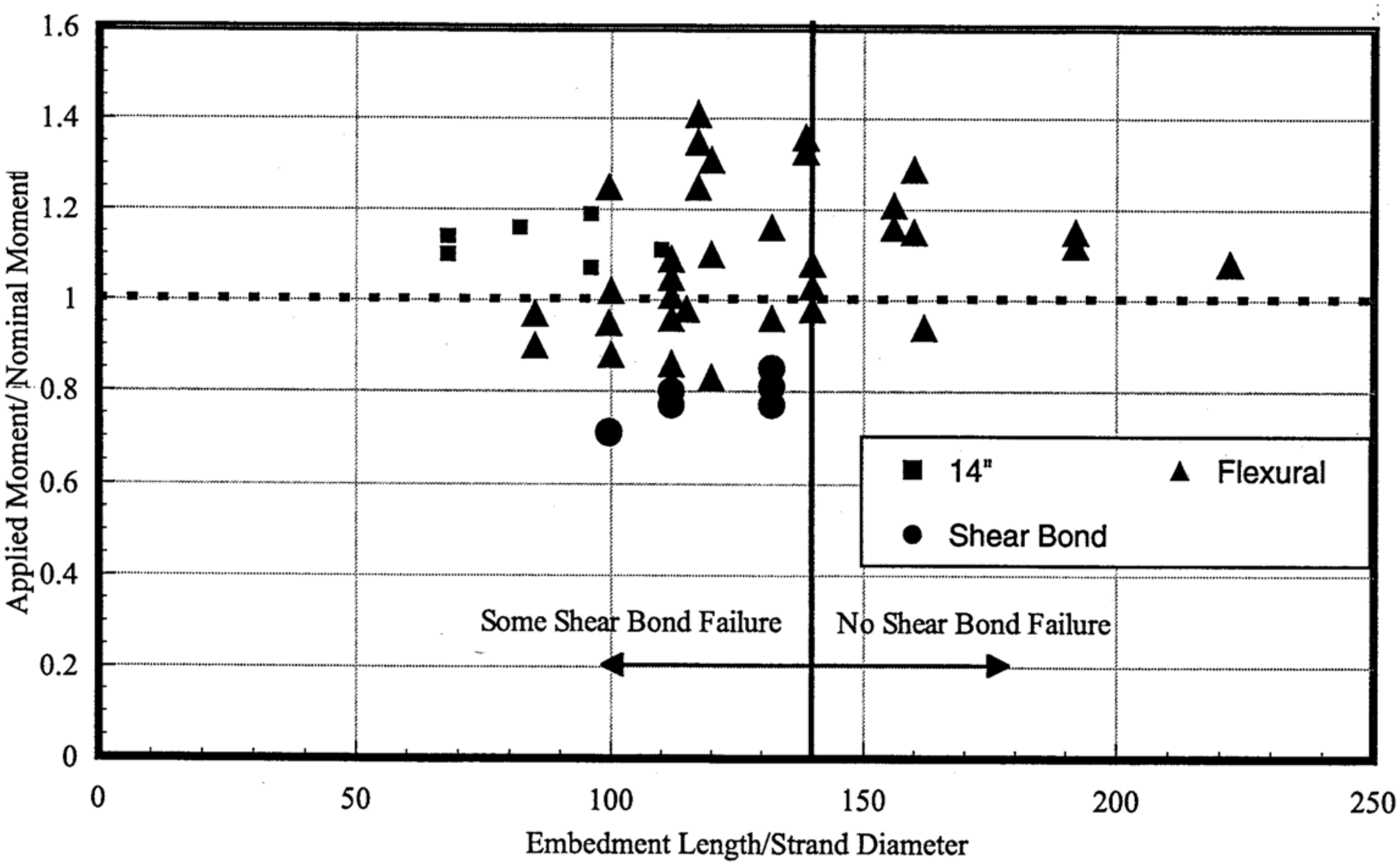


Figure 38 - Relationship Between Ma/Mn and Embedment Length/Diameter
All Piles

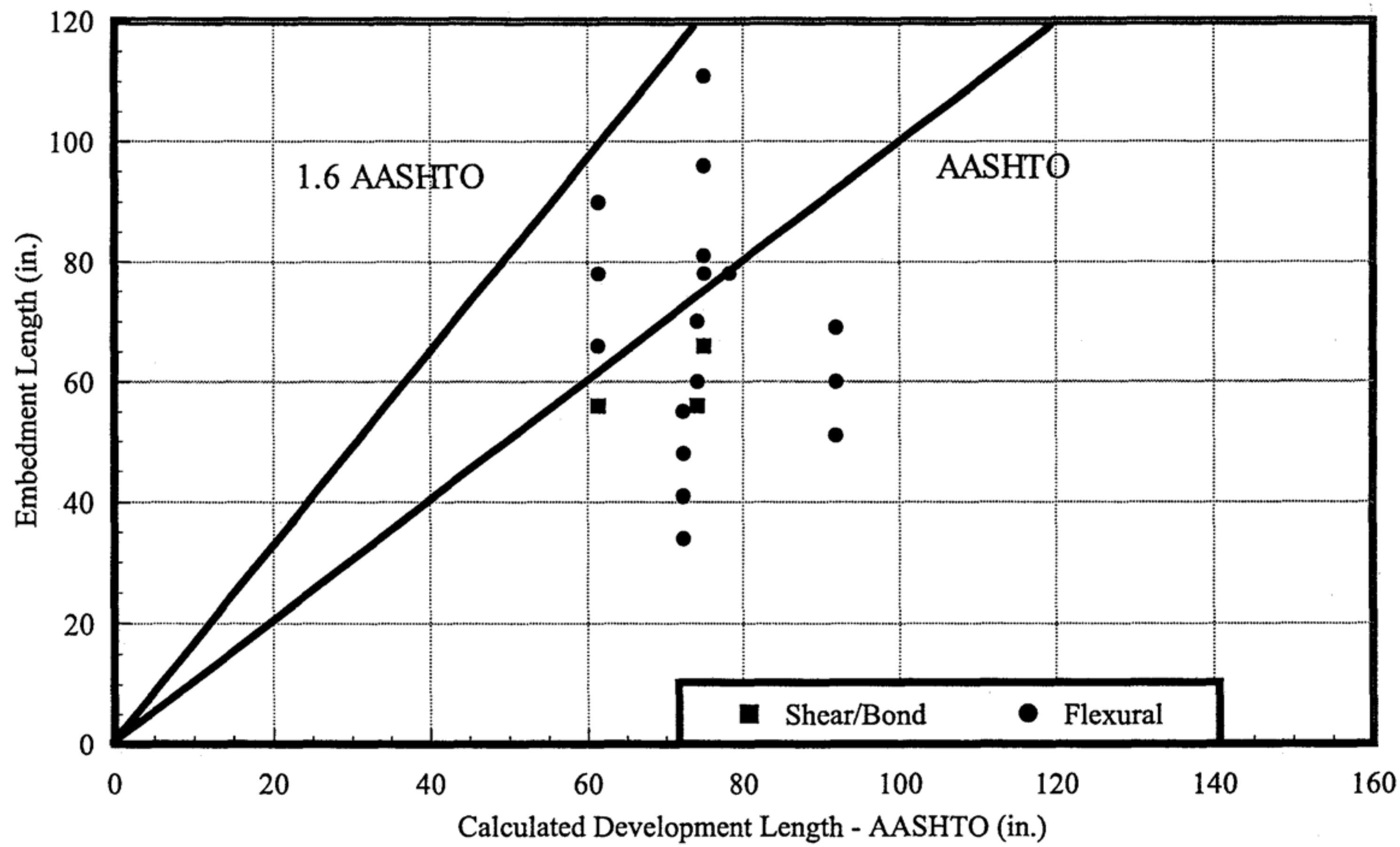


Figure 39 - Comparison of Embedment Length To AASHTO Calculated

All Piles

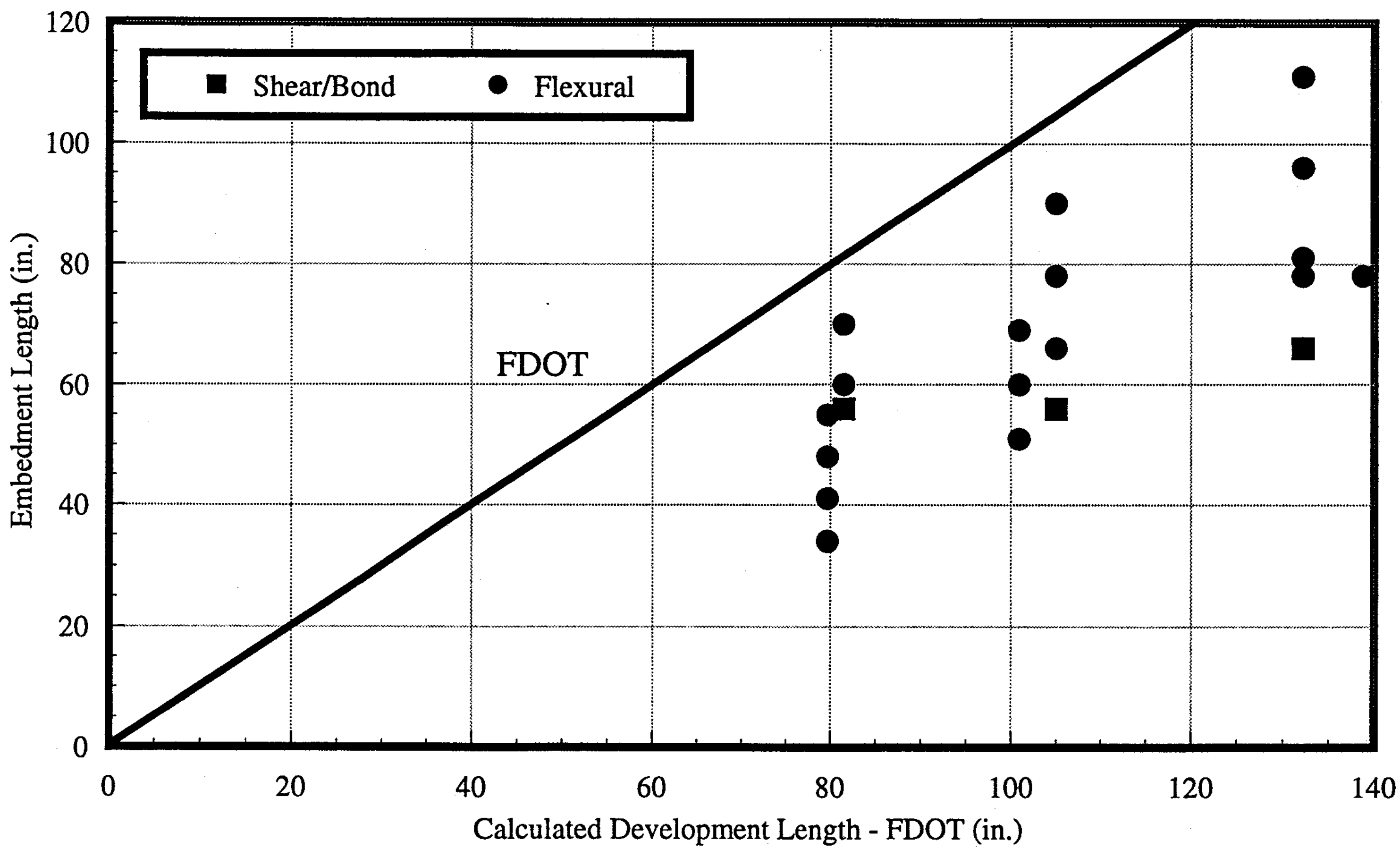


Figure 42 Comparison of Embedment Length To FDOT Calculated

All Piles

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Based on the study in this report, the following conclusions can be made:

TRANSFER LENGTH

The use of Eqn. (6) should be adopted for calculating the transfer length of strand up to 0.6 in. diameter i.e.

$$L_t = \left(\frac{f_{si}}{3}\right) D \qquad (6)$$

STRAND SPACING

A spacing of 2 in. is acceptable, irrespective of strand size, up to 0.6 in. dia.

DEVELOPMENT LENGTH

It is considered necessary to present the main conclusions for strand development length separately for the three types of prestressed members studied in this report. This highlights a number of common factors, which are useful in formulating the recommendations. The condensation of the main conclusions into a tabular format (Table 6) renders the comparison of the different methods more effective. The conclusions regarding slabs and piles are presented first, in view of the common factors that become apparent between these two types of members.

In view of the inadequacy of Zia and Mostafa's Equation (5), it is not included in the conclusions given below.

PROVISION	SLABS		PILES	GIRDERS
	Solid	Voided		
AASHTO Eqn. (2)	Satisfactory*	Satisfactory*	Unsatisfactory	Unsatisfactory
1.6 AASHTO	Conservative	Conservative	Satisfactory*	Satisfactory, but slightly unconservative
FDOT Eqn. (7)	Satisfactory	Satisfactory	Satisfactory, but somewhat conservative	Satisfactory*
Buckner Eqn. (8)	Satisfactory	Conservative	Unconservative, but Satisfactory for Piles up to 24 in. deep	Unsatisfactory (Unconservative)
FHWA Eqn. (12)	Too Conservative	Too Conservative	Too Conservative	Satisfactory Conservative
General	140 D (i.e. 70 in.) satisfactory for ½ in. dia. Strand		140 D should prevent shear/bond failure	260 D appropriate for strand sizes tested

***Indicates best predictor**

Table 6 Summary of Conclusions on Development Length

Slabs

1) The FDOT Equation (7) and Buckner's Equation (8) yield satisfactory results for solid slabs. The Buckner equation is somewhat conservative for voided slabs, for which the MOT equation yields better results.

2) The AASHTO Basic Equation (2) yields the closest predictions, and the use of a multiplier is not warranted. This applies to both solid and voided slabs.

3) The FHWA proposal (Eqn. 12) are far too conservative.

4) For general purposes, a development length of 70 in. (140 D) is appropriate for 0.5 in. dia. strand.

Piles

1) The existing AASHTO provisions (Eqn. 2) are unconservative. However, the use of the 1.6 multiplier to the AASHTO equation yields satisfactory results, although this would be conservative if applied to slender piles.

2) Buckner's Equation (8) yields satisfactory results for piles up to 24 in. deep, but yields unconservative results beyond. The FHWA proposal (Eqn. 12) is too conservative. On the other hand the MOT proposal (Eqn. 7) yields satisfactory results for all the piles tested.

3) An embedment length of 140 D should ensure that the undesirable shear-bond failure mode is avoided.

Girders

01) The application of a variable factor, such as λ or k_b, to flexural bond length is warranted in calculating strand development length in AASHTO girders.

2) The application of a multiplier of 1.6 to the basic AASHTO expression (Eqn. 2) yields slightly unconservative results. The application of a variable factor (Buckner, FDOT) to the flexural bond length is more appropriate to reflect the demonstrated effect of the magnitude of the strain in the strand at nominal strength of a member. The MOT proposal is the best prediction of all the expressions for development length in girders.

3) The FHWA proposal (Eqn. 12) yields satisfactory though conservative results.

4) A general assumption of 260 D for development length is satisfactory for $^{1}/_{2}$ in. and 0.6 in. diameter strands. Below this value, the moment at strand slip decreases linearly with embedment; and the undesirable shear/bond mode of failure is likely to result.

5) Shear cracking at end regions of girders affect strand slip and flexural strength. At a shear span/depth ratio below 3.5, flexural strength decreases linearly with this ratio.

6) The use of confinement reinforcement in the tension flange of a girder enhances ductile behavior, and is recommended.

5.2 RECOMMENDATIONS

Based on the findings in this report, the following recommendations are made:

1) A minimum spacing of 2 in. for strand up to 0.6 in. diameter should be adopted.
2) The formal adoption of Eqn. (6) is recommended to calculate the transfer length of prestressing strand.
3) With the acceptance of Recommendation (2), the use of FDOT Equation (7) or Buckner Equation (8), in that order, is recommended for calculating development length of strand. The simplicity and accuracy of the FDOT proposal render it preferable for general use.
4) An acceptable alternative to Recommendation (2) is the formal adoption of the 1.6 multiplier to be applied to the basis AASHTO Equation (1) in calculating the development length of strand in girders. In the case of slender members such as slabs and piles, the application of a multiplier is should not be required.
5) In the absence of further refinement, the FHWA proposals for calculating transfer and development lengths (Eqns. 11 and 12) should not be adopted without further study, since they yield results that are generally too conservative.
6) The use of 140 D for slabs and piles, and 260 D for girders, would ensure that the nominal strengths of the members would be developed without strand slip.

6. REFERENCES

1. Cousins, T., Johnston, D.W., and Zia, P., Publication No. FHWA/NC/87-005, Federal Highway Administration, Washington, DC, December 1986.
2. Buckner, C.O., "A Review of Strand Development Length of Pretensioned Concrete Members", PCI Journal, V40, No. 2, March-April 1995, pp. 84-105.
   Discussion, PCI Journal, V41, No. 2, March-April 1996, pp. 112-127.
3. Lane, S.N., "A New Development Length Equation for Pretensioned Strands in Bridge Beams and Piles", Federal Highway Administration, Report No. FHWA-RD-98-116, McLean, VA, December 1998, pp. 123.
4. Zia, P., and Mostafa, T., "Development Length of Prestressing Strands", PCI Journal, V22, No. 5, September-October 1977, pp. 54-65.
5. Shahawy, M.A., Issa, M., and Batchelor, B., "Strand Transfer Lengths in Full Scale AASHTO Prestressed Concrete Girders", PCI Journal, V37, No. 3, May-June 1992, pp. 84-96.
6. Shahawy, M., "An Investigation of Shear Strength of Prestressed Concrete AASHTO Type II Girders", Structural Research Centre, Florida Department of Transportation, May 1993, pp. 132.

APPENDIX A

ADDITIONAL DATA ON SLABS

APPENDIX A

Table A1 - Calculations of L_d in Slabs

Values required for calculation of L_d in Equations (2), (7), (8), and (9) are f_{si}, f_{se}, f_{su}^* and f_{pt}. The appropriate values are as follows:

Series	f_{si} (ksi)	f_{se} (ksi)	f_{pt} (ksi)	f_{su}^* (ksi)	ε_{ps} (microstrain)
Solid Slabs	202.5	157.6	197.6	250.3	11,612
Voided Slabs	202.5	157.6	197.6	261.9	18,860

The value of f_{si} was determined from FDOT standard design, for prestressed structures, and the value of f_{se} was calculated based on losses calculated by the lump sum AASHTO method.

AASHTO Method

Solid Slabs:

$$L_d = \frac{f_{se}}{3} D + (f_{su}^* - f_{se})\, D \qquad \text{Eqn. [2]}$$

$$L_d = \frac{157.6}{3}(0.5) + (23.3 - 157.6)(0.5)$$
$$= 26.3 + 47.9$$

$$L_d = 74.2\ inches\, (= 148.4D)$$

1.6 AASHTO = 1.6(74.2) = 118.7 inches

Voided Slabs:

$$L_d = \frac{f_{se}}{3} D + (f_{su}^* - f_{se})\, D \qquad \text{Eqn. [2]}$$

$$L_d = \frac{157.6}{3}(0.5) + (261.9 - 157.6)(0.5)$$
$$= 26.3 + 52.2$$

$$L_d = 78.5\ inches\ (= 148.4\, D)$$

1.6 AASHTO = 1.6(78.5) = 125.7 inches

FDOT Method

$$L_d = \frac{f_{si}}{3} D + \frac{(f^*_{su} - f_{se})}{0.25 k_b} D \qquad \text{Eqn. [7]}$$

Substituting $k_b = 4$ for slabs and slender beams gives:

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

Solid Slabs:

$$L_d = \left(\frac{202.5}{3} \right) (0.5) + (253.3 - 157.6)(0.5)$$

$$L_d = 33.75 + 47.85 = 81.6 \textit{ inches}$$

Voided Slabs:

$$L_d = \left(\frac{202.5}{3} \right) (0.5) + (261.9 - 157.6)(0.5)$$

$$L_d = 33.75 + 52.15 = 85.9 \textit{ inches}$$

Buckner's Method

$$L_d = \frac{f_{si}}{3} D + \lambda (f^*_{su} - f_{se}) D \qquad \text{Eqn. [8]}$$

where $\lambda = 1.0 \leq (0.6 + 40 \varepsilon_{ps}) D$

Solid Slabs:

$$\lambda = 0.6 + 40(0.0116) = 1.06$$

$$L_d = \frac{202.5}{3} (0.5) + 1.06(253.3 - 157.6)(0.5)$$

$$L_d = 33.75 + 50.72 = 84.5 \textit{ inches}$$

Voided Slabs:

$$\lambda = 0.6 + 40(.0188) = 1.35$$

$$L_d = \frac{202.5}{3}(0.5) + 1.35(261.9 - 1576)(0.5)$$

$$L_d = 33.75 + 70.40 = 104.2 \; inches$$

FHWA Method

$$L_d = \left(\frac{4 f_{pt} D}{f_c'} - 5\right) + \left(\frac{6.4(f_{su}^{*} - f_{se})\, D}{f_c'} + 15\right) \qquad \text{Eqn. [12]}$$

Solid Slabs:

$$L_d = \left(\frac{4 \times 197.6 \times 0.5}{5} - 5\right)\left(\frac{6.4(250.3 - 157.6)(0.5)}{5} + 15\right)$$

$$L_d = 74.04 + 74.33 = 148.37 \; inches$$

Voided Slabs:

$$L_d = \left(\frac{4 \times 197.6 \times 0.5}{5} - 5\right) + \left(\frac{6.4(261.9 - 157.6)(0.5)}{5} + 15\right)$$

$$L_d = 74.04 + 81.75 = 155.79 \; inches$$

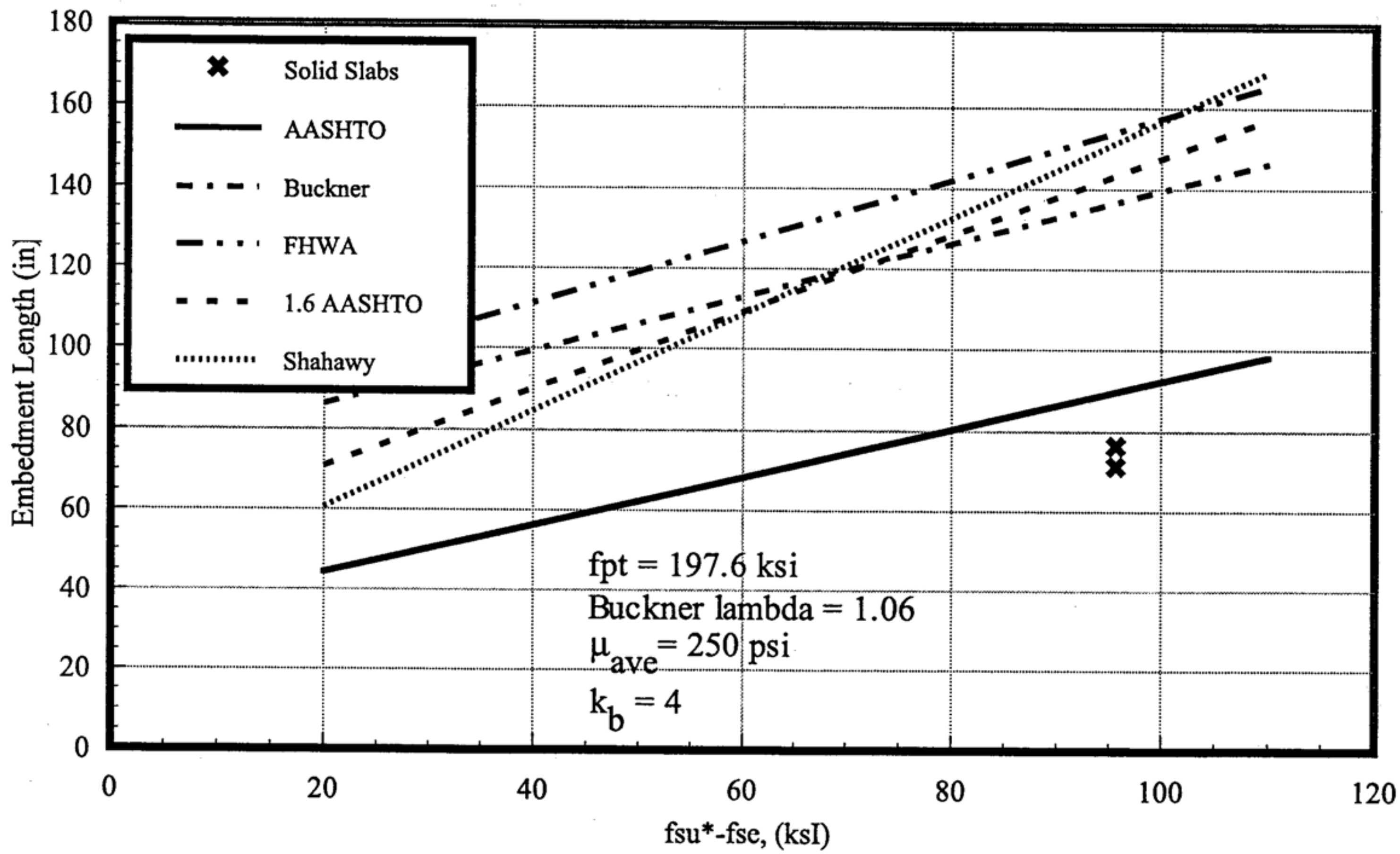


Figure A1 - Comparison of Development Length Equations
D= 0.5 in., and f'$_c$ = 5 ksi
Solid Slabs

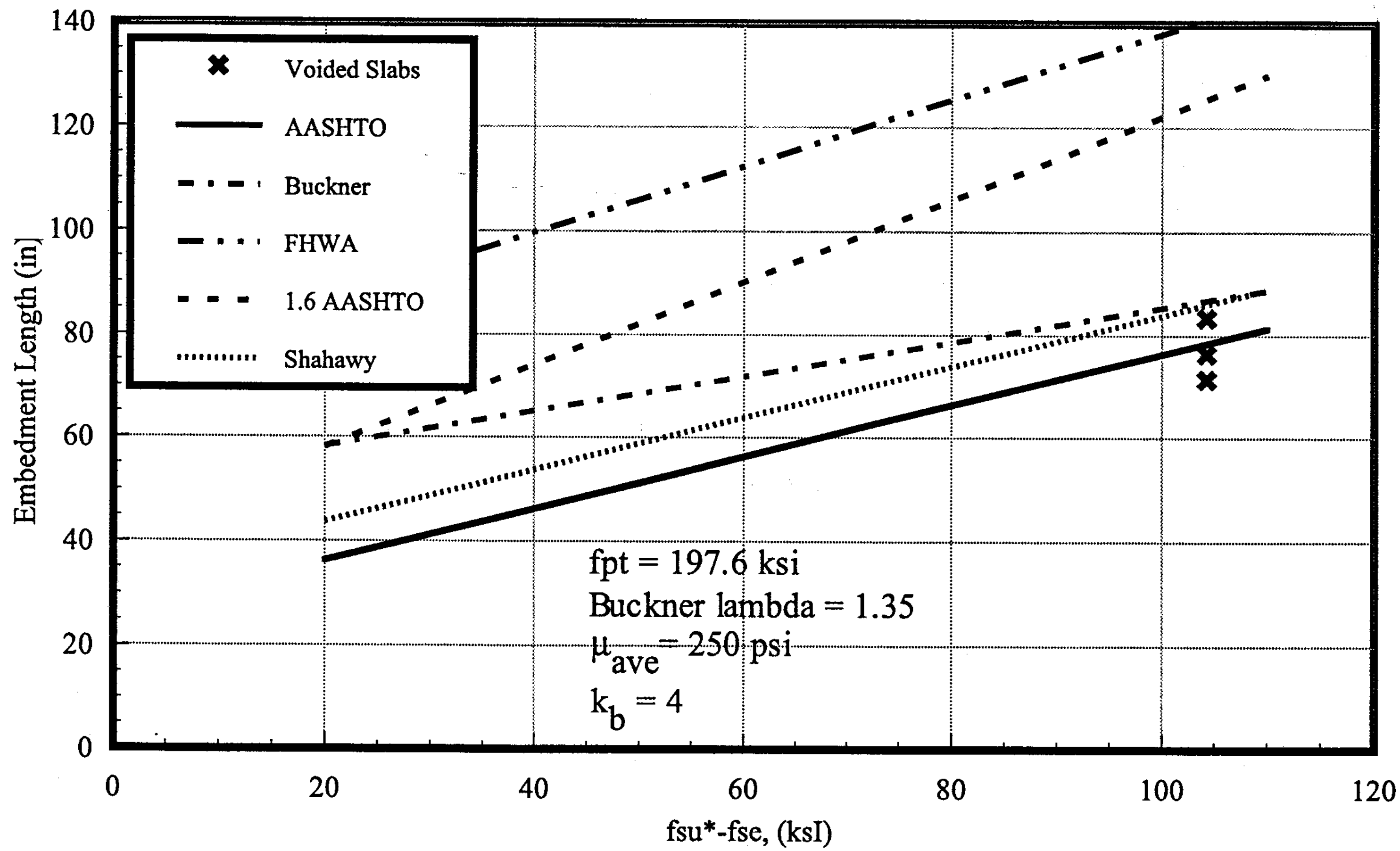


Figure A2 - Comparison of Development Length Equations
D = 0.5", and f'_c = 5 ksi
Voided Slabs

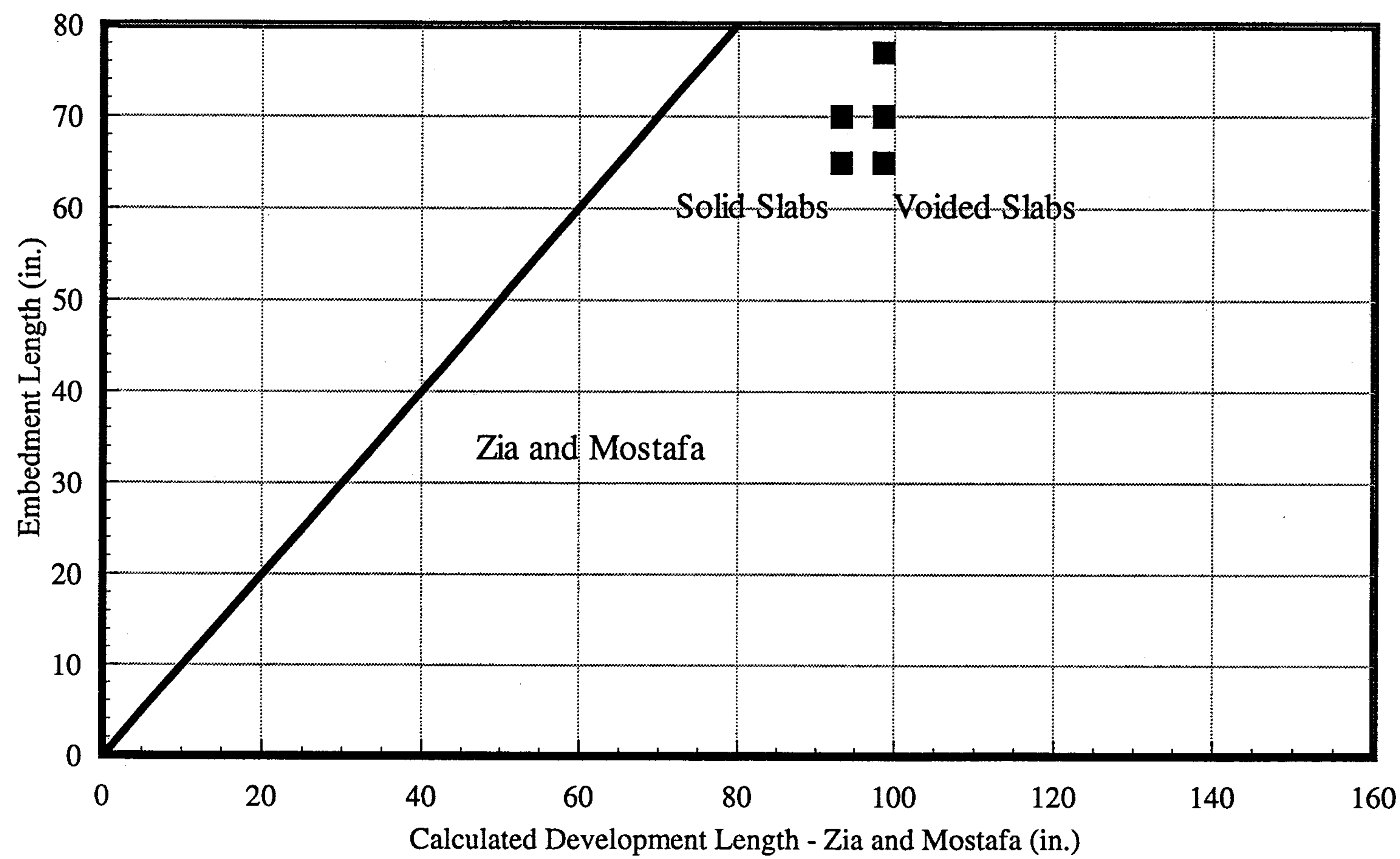


Figure A3 - Comparison of Embedment Length To Zia and Mostafa Calculated Development Length Slabs

APPENDIX B

ADDITIONAL DATA ON GIRDERS

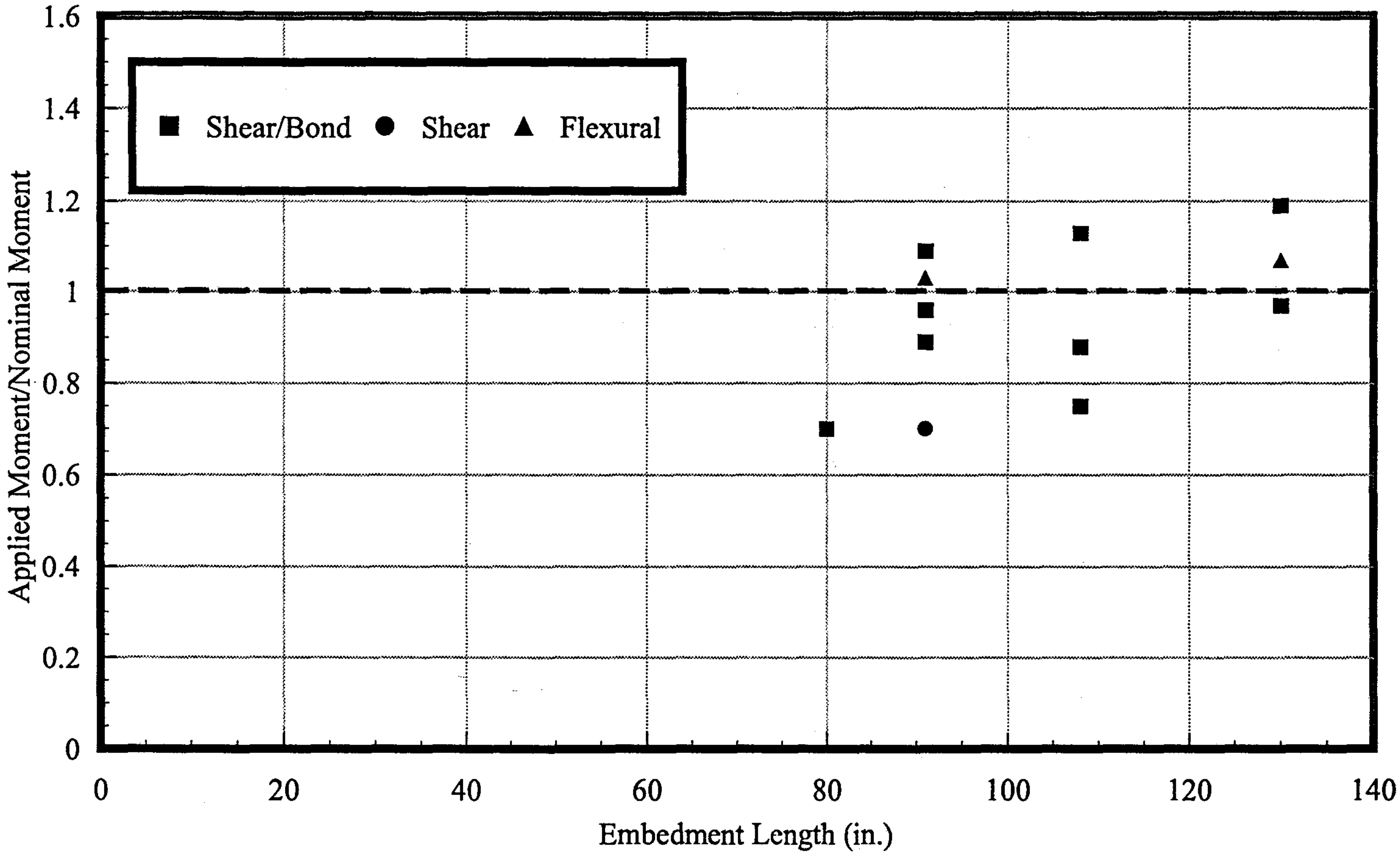


Figure B1 - Applied Moment/Nominal Moment vs. Embedment Length

AASHTO Girders
(1/2 in. diameter strands)

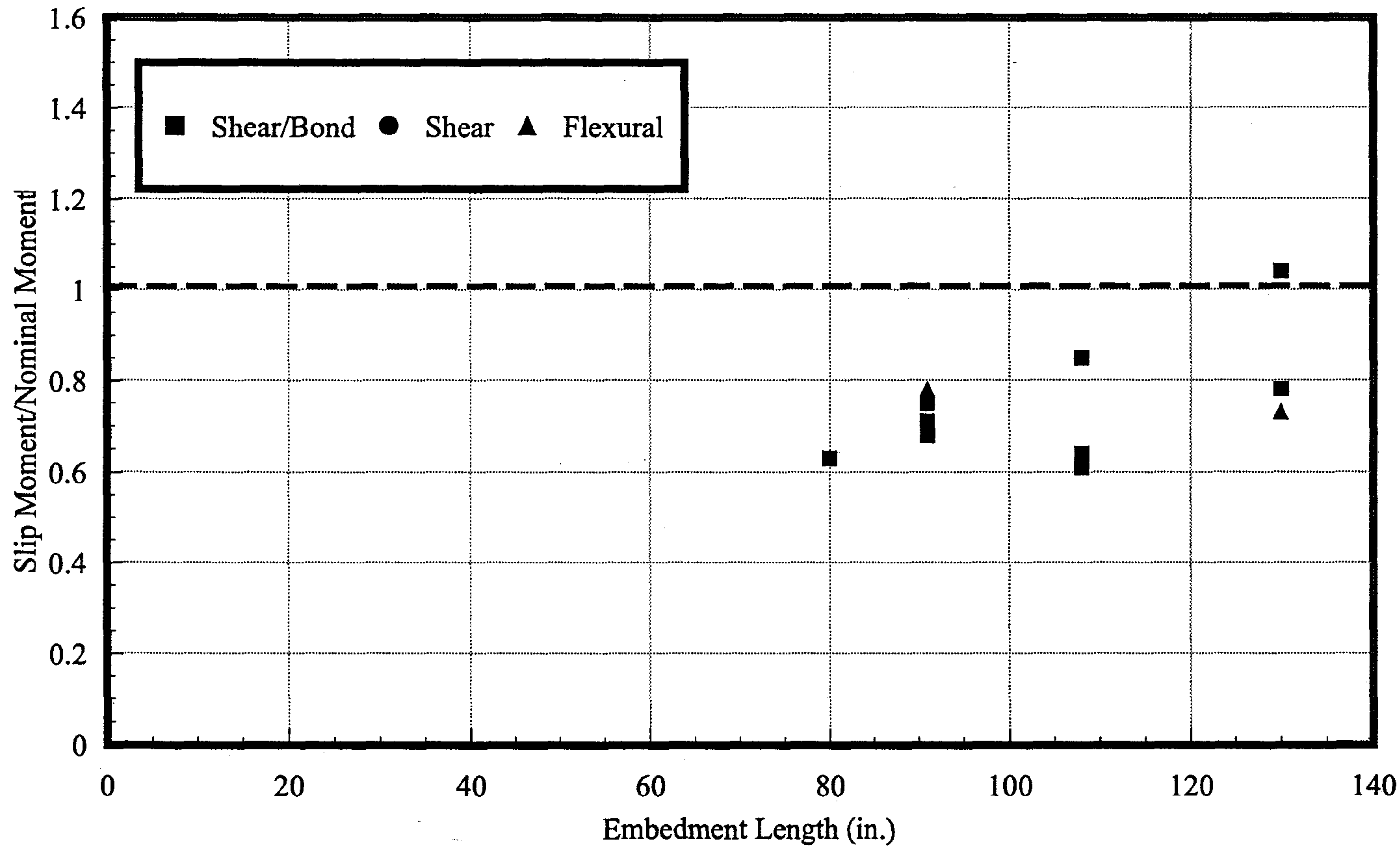


Figure B2 - Slip Moment/Nominal Moment vs. Embedment Length

AASHTO Girders
(1/2 in. diameter strands)

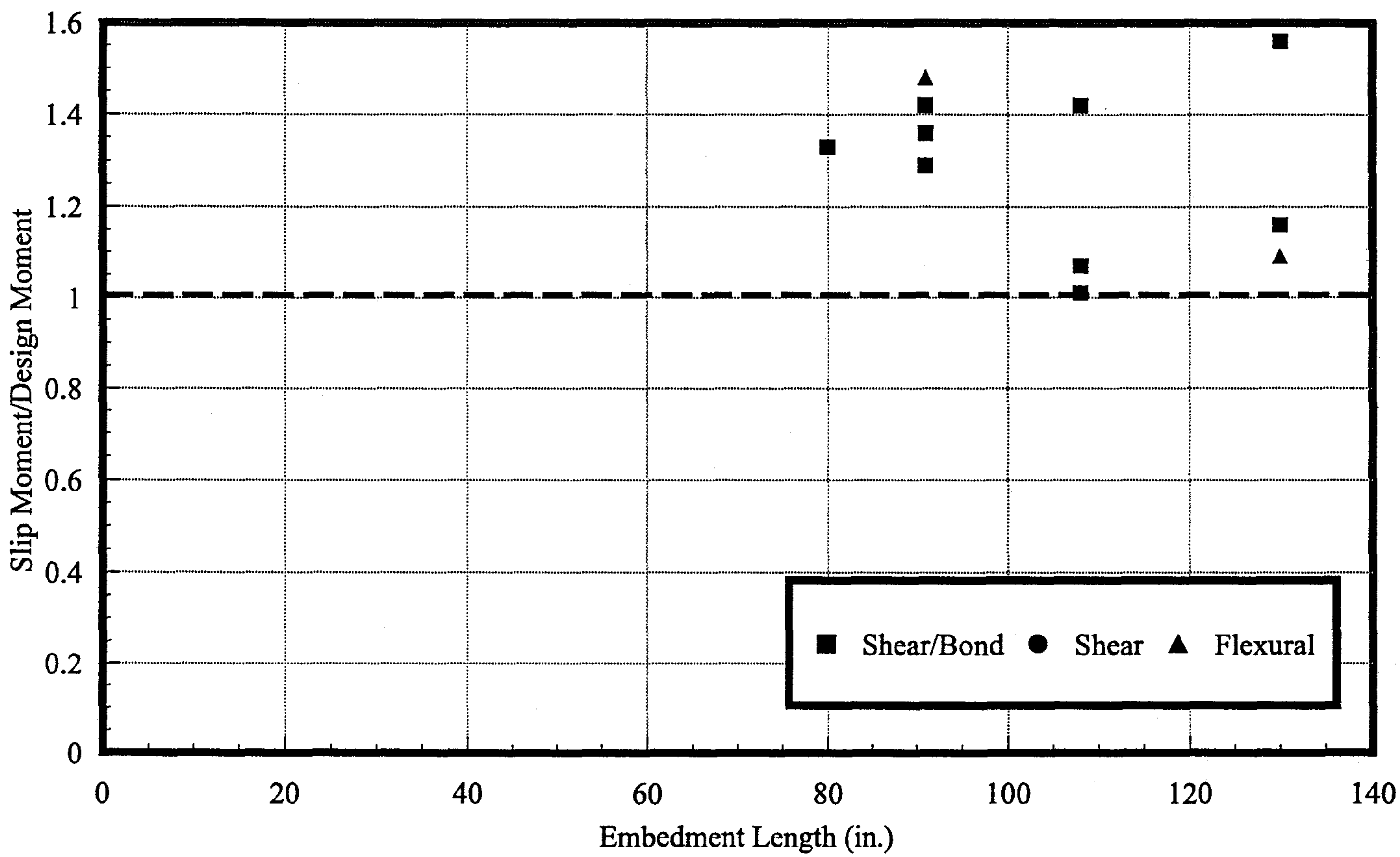


Figure B3 - Slip Moment/Design Moment vs. Embedment Length

AASHTO Girders
(1/2 in. diameter strands)

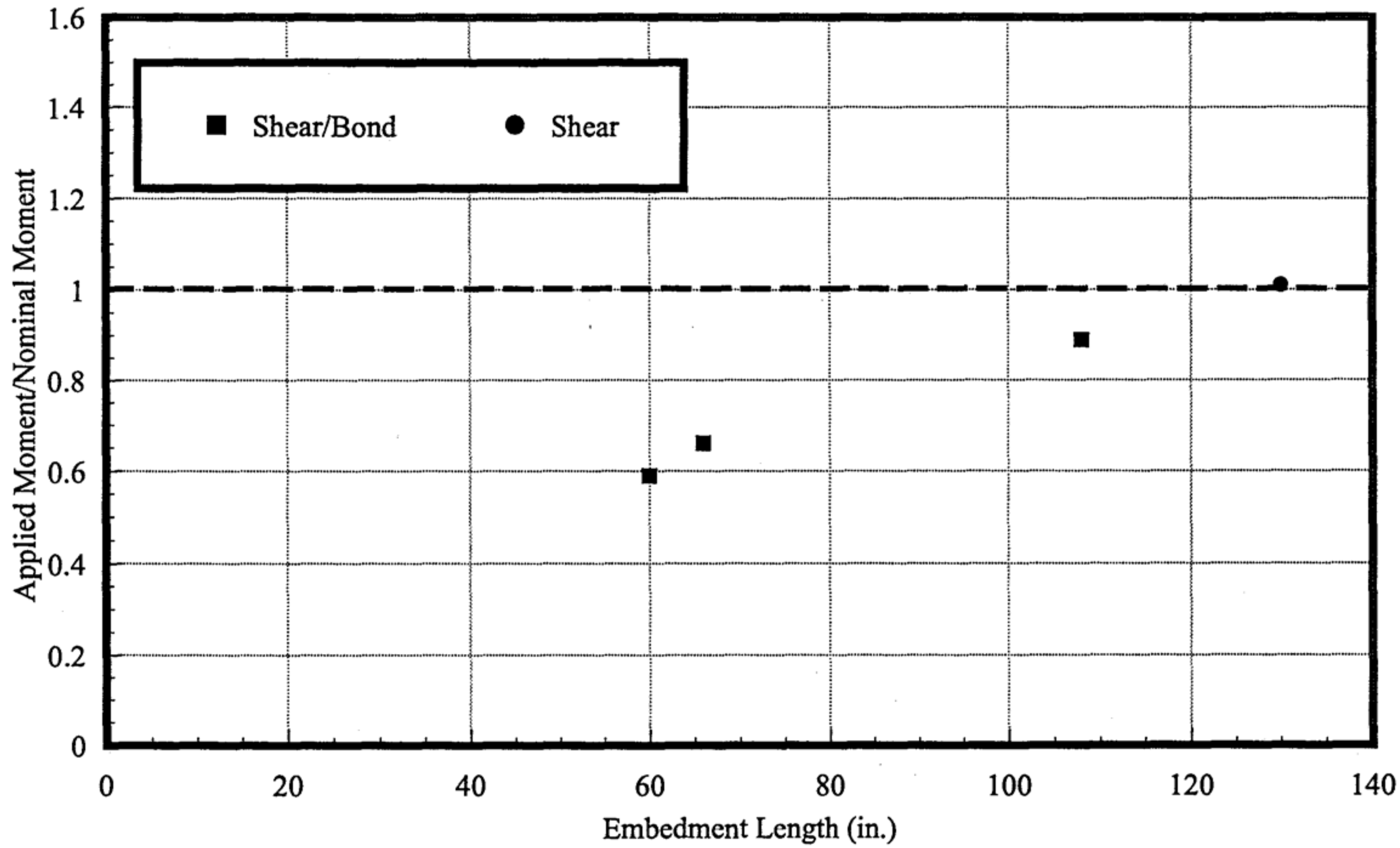


Figure B4 - Applied Moment/Nominal Moment vs. Embedment Length

AASHTO Girders
(0.5 in. special diameter strand)

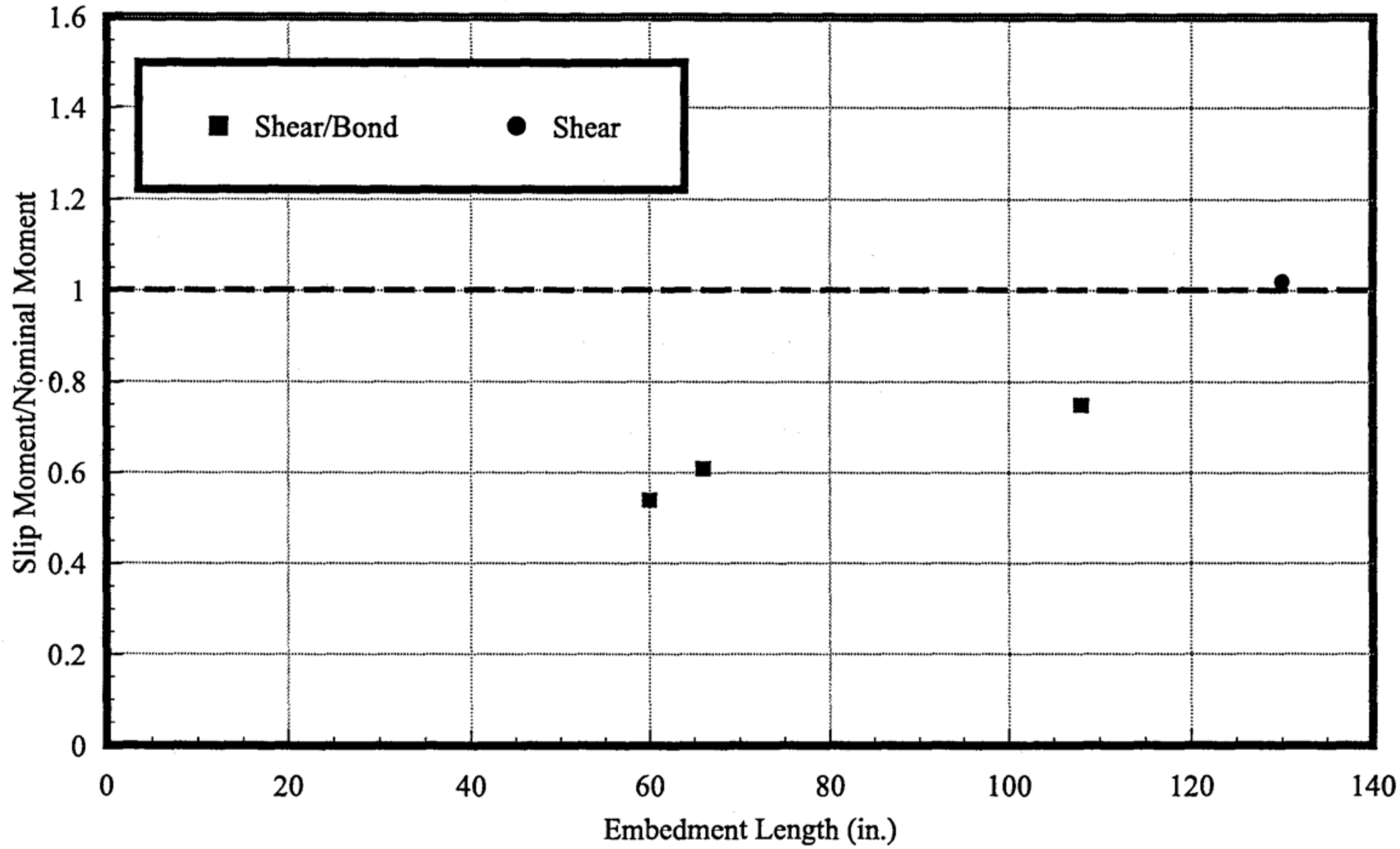


Figure B5 -Slip Moment/Nominal Moment vs. Embedment Length

AASHTO Girders
(0.5 in. special diameter strand)

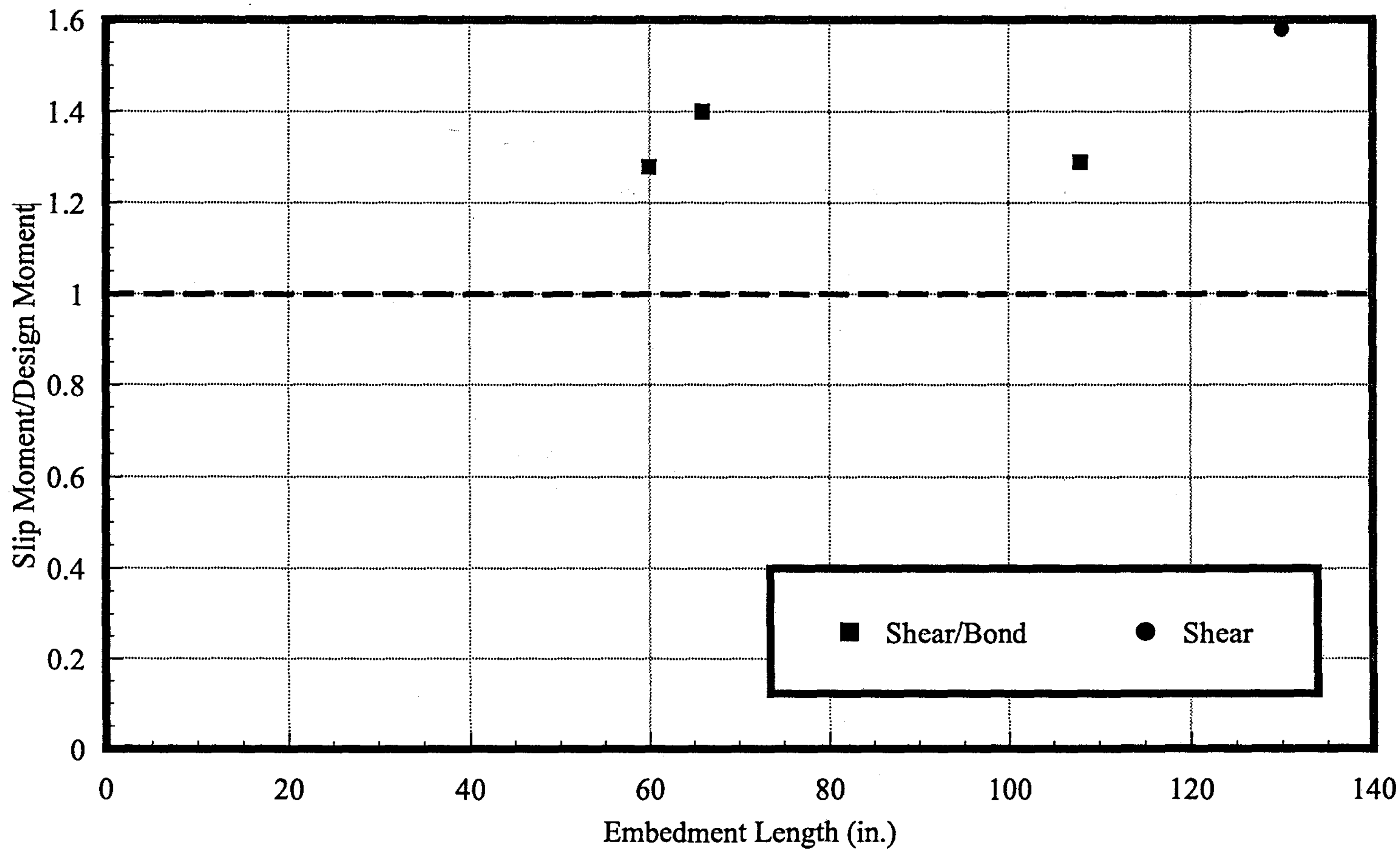


Figure B6 - Slip Moment/Design Moment vs. Embedment Length

AASHTO Girders
(0.5 in. special diamter strand)

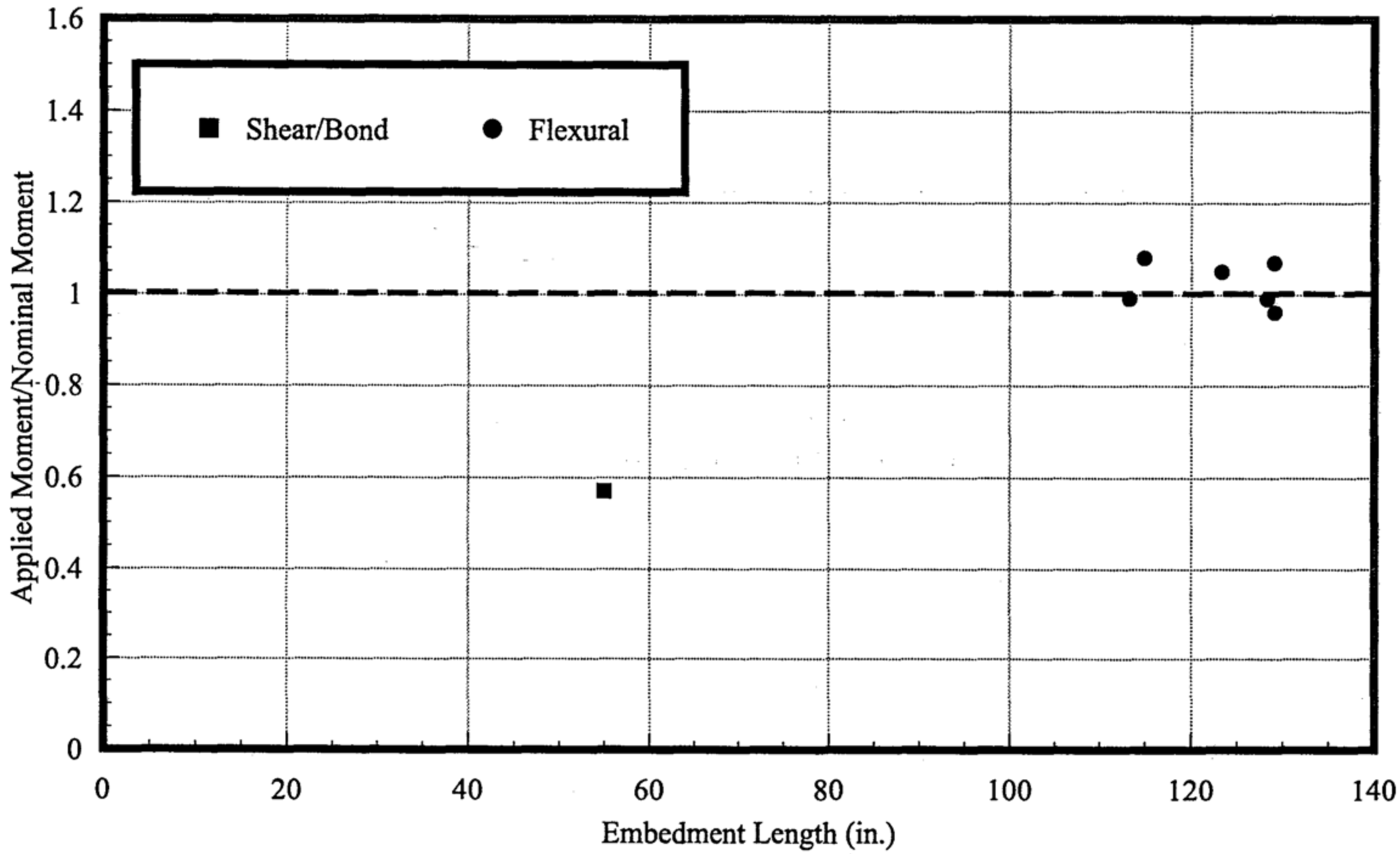


Figure B7 -Applied Moment/Nominal Moment vs. Embedment Length

AASHTO Girders
(0.6 in. diameter strand)

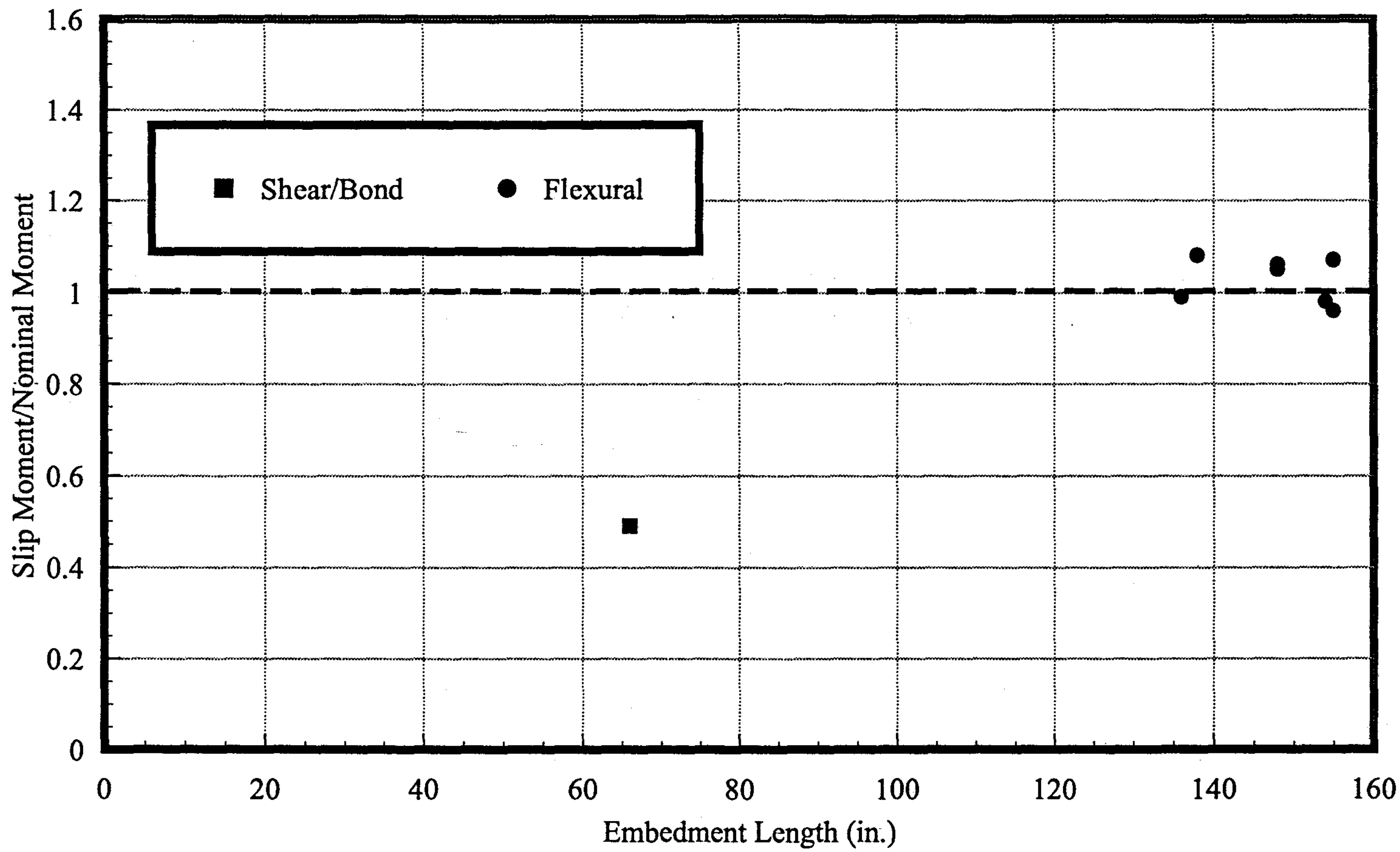


Figure B8 - Slip Moment/Nominal Moment vs. Embedment Length

AASHTO Girders
(0.6 in. diameter strand)

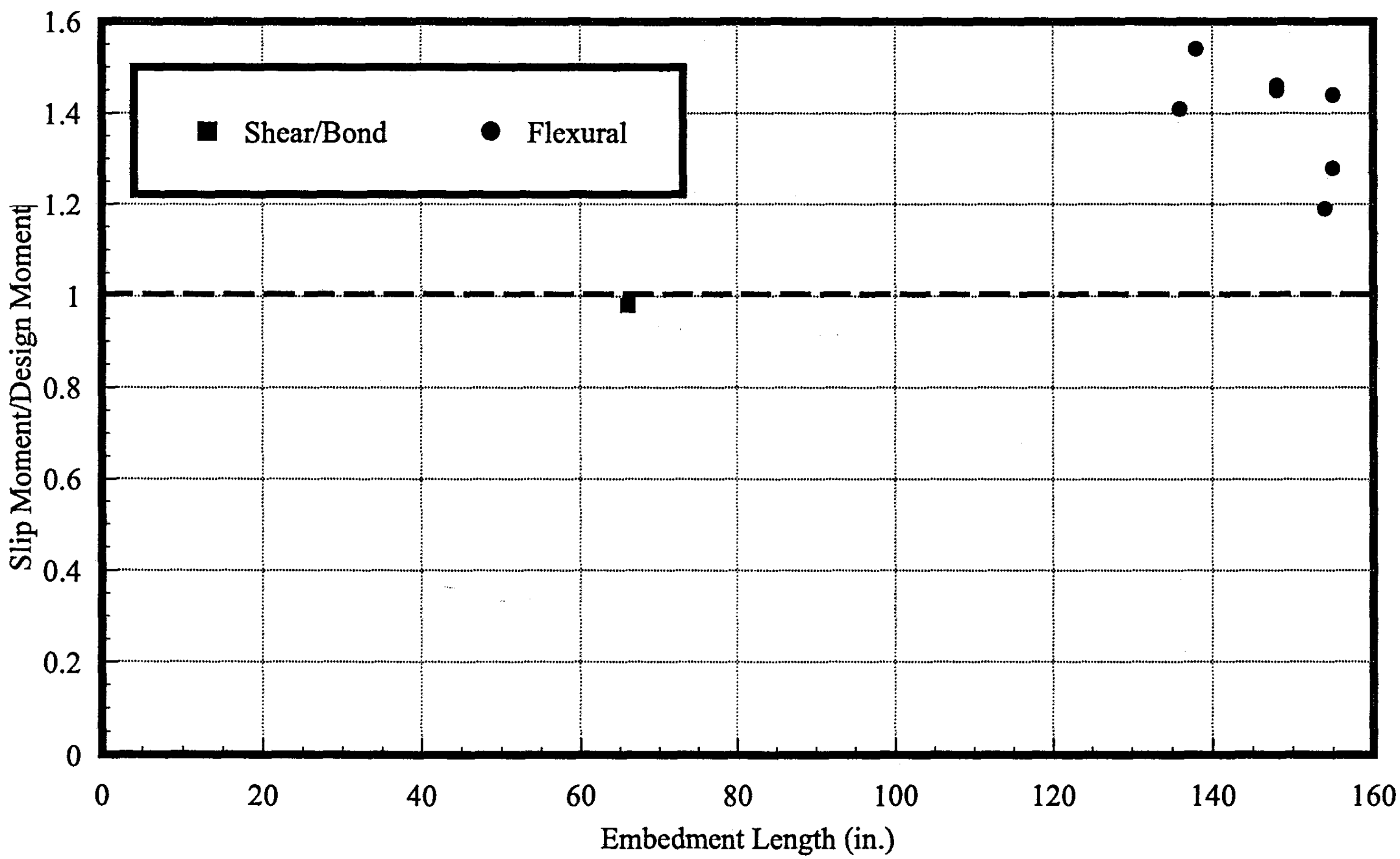


Figure B9 -Slip Moment/Design Moment vs. Embedment Length

AASHTO Girders
(0.6 in. diameter strand)

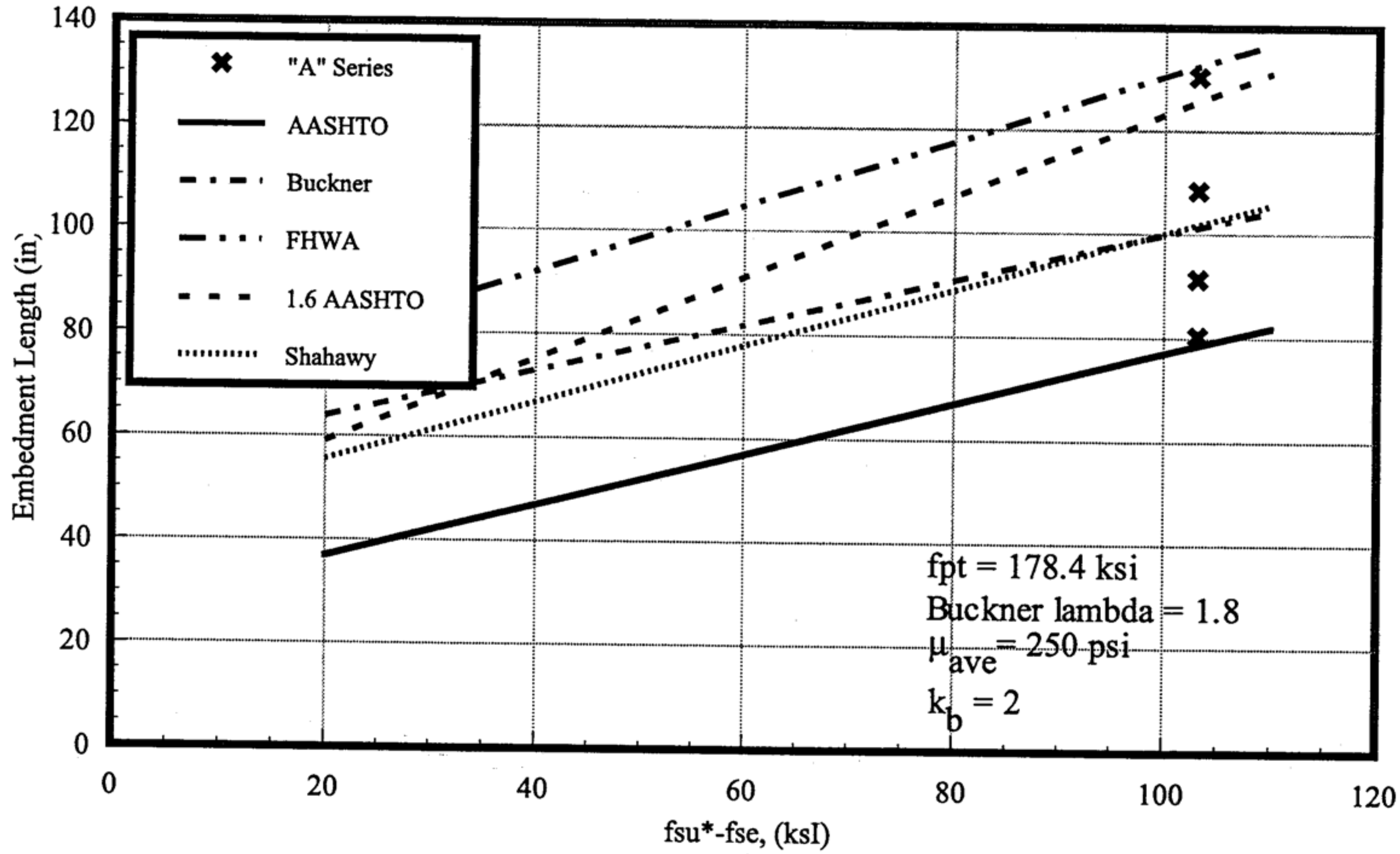


Figure B10 - Comparison of Development Length Equations
D = 0.5 in., f'_c 5 = ksi
"A" Series Girders

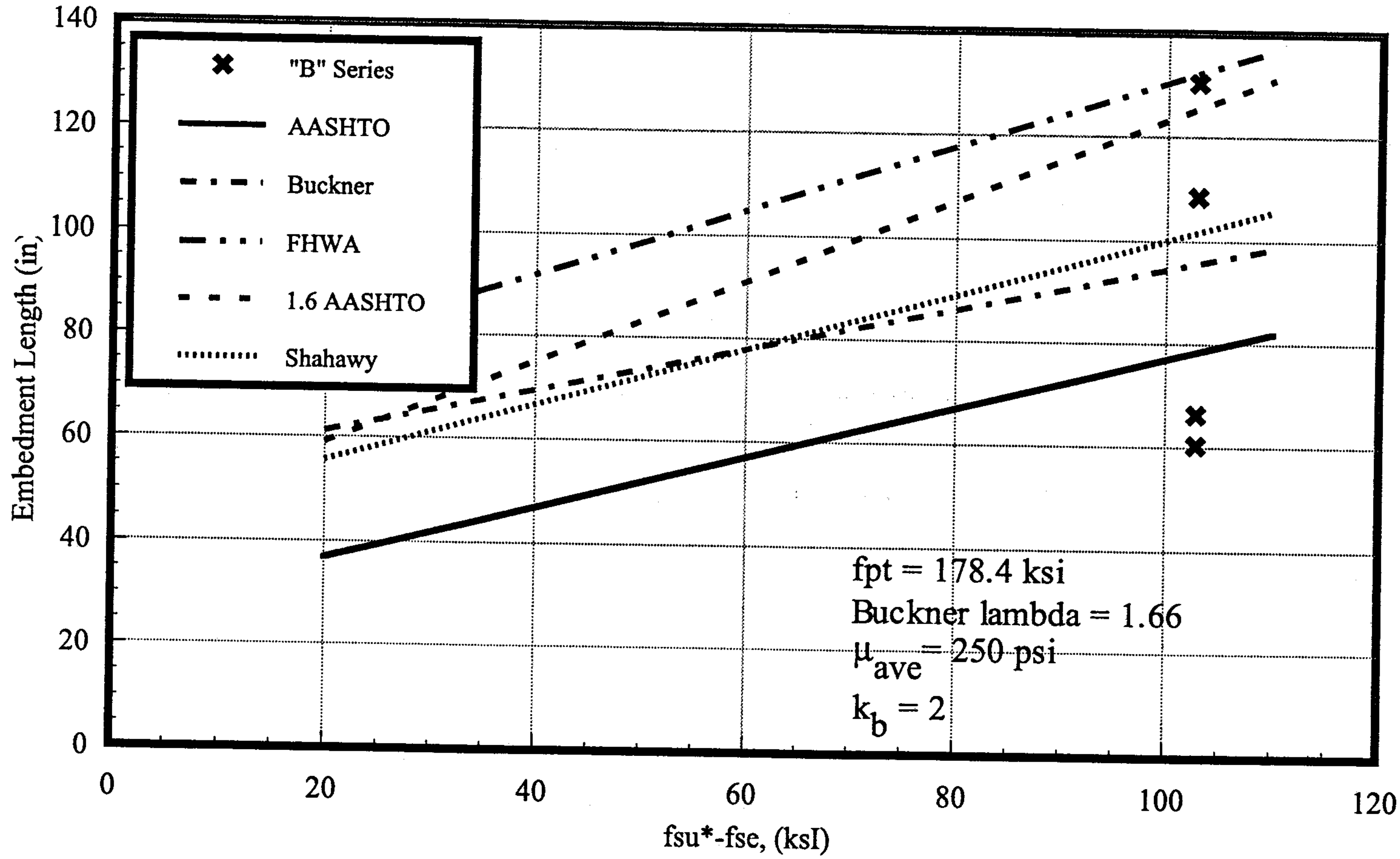


Figure B11 - Comparison of Development Length Equations
D = 0.5 in. special, f'_c = 5 ksi
"B" Series AASHTO Girders

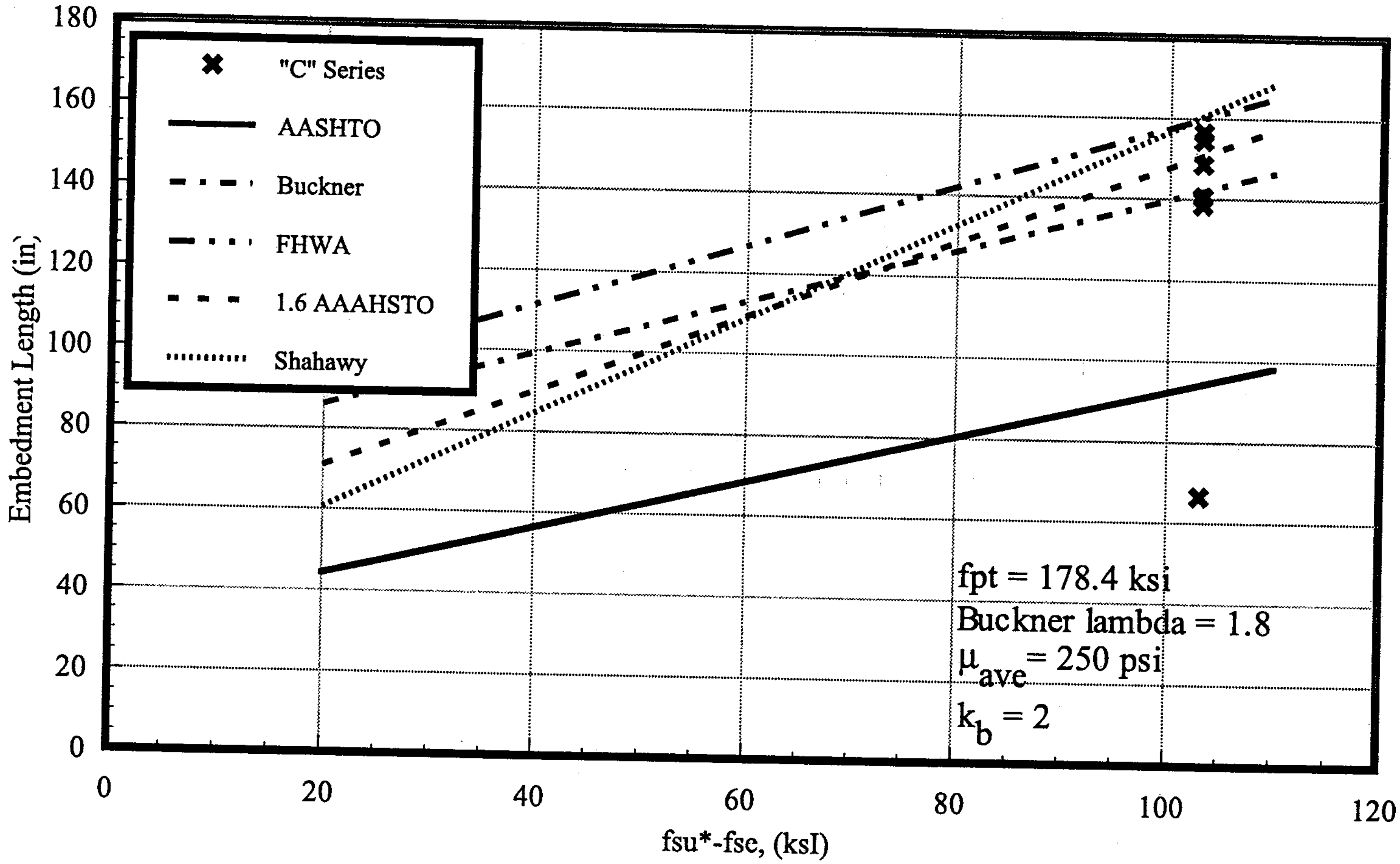


Figure B12 - Comparison of Development Length Equations
D = 0.6 in., f'_c = 5 ksi
"C" Series Girders

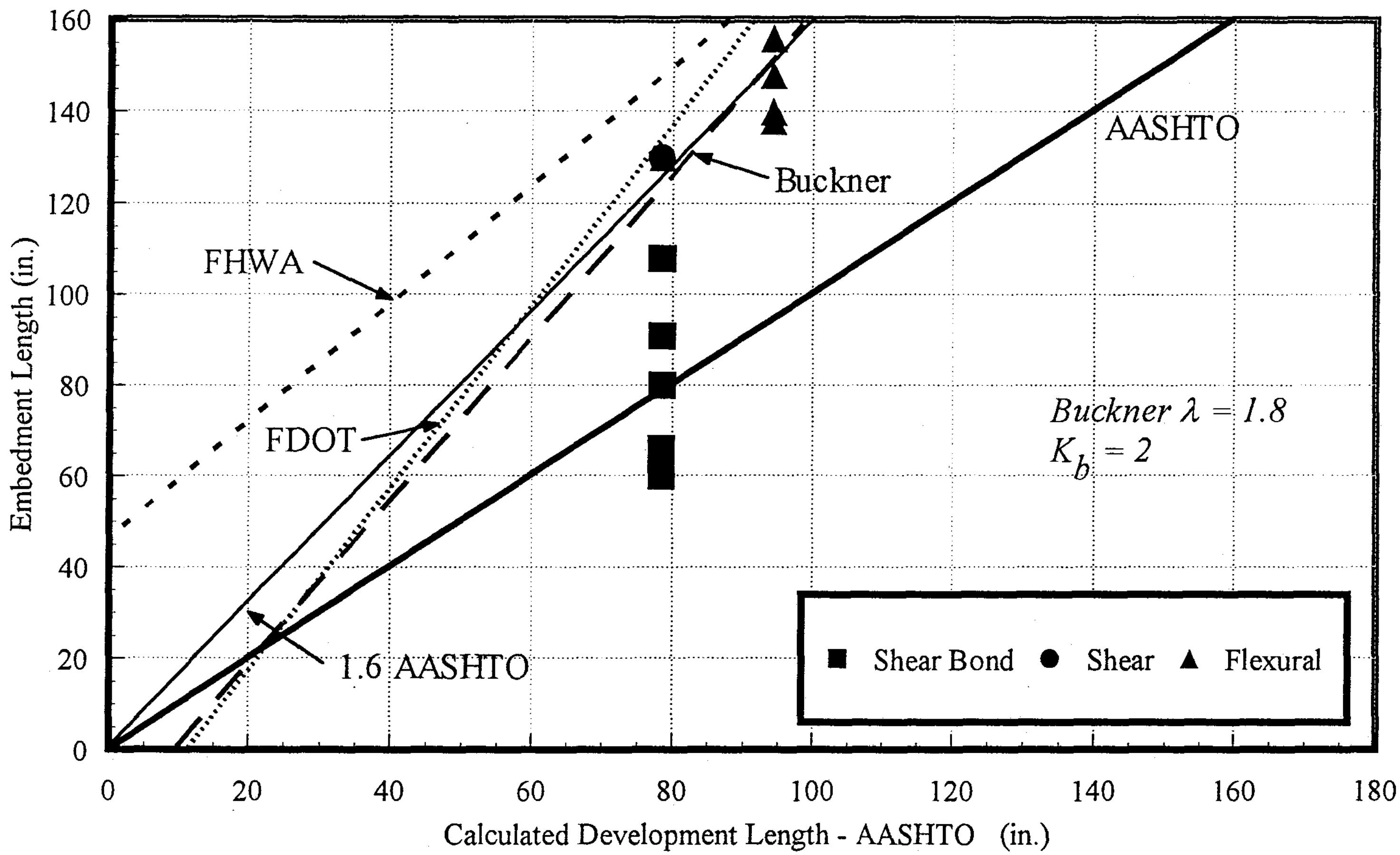


Figure B13 - Development Lengths - Comparison of AASHTO and Other Predictions

Girders

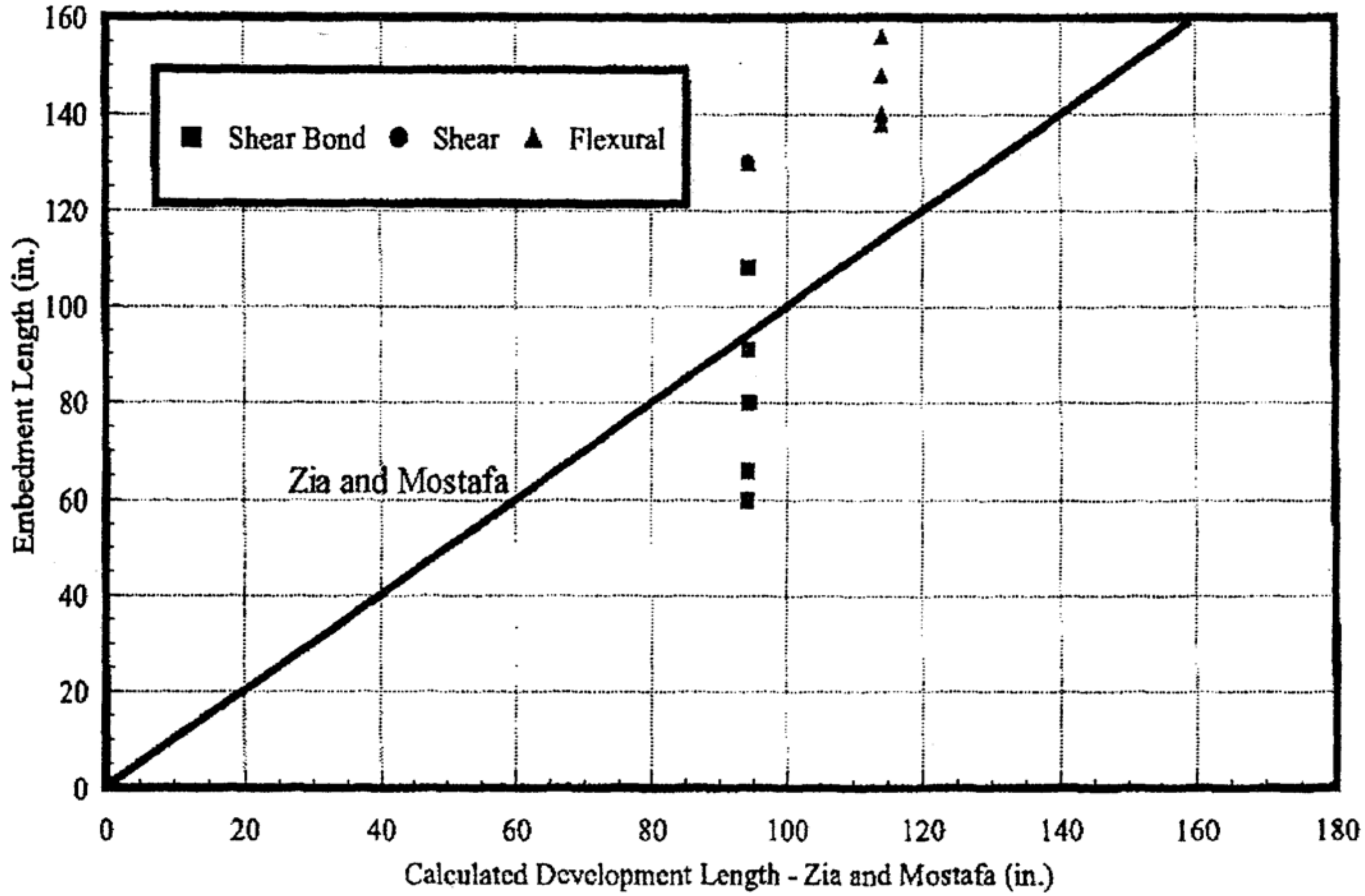


Figure B14 - Comparison of Embedment Length To Zia and Mostafa Calculated Development Length

Girders

APPENDIX C

ADDITIONAL DATA ON PILES

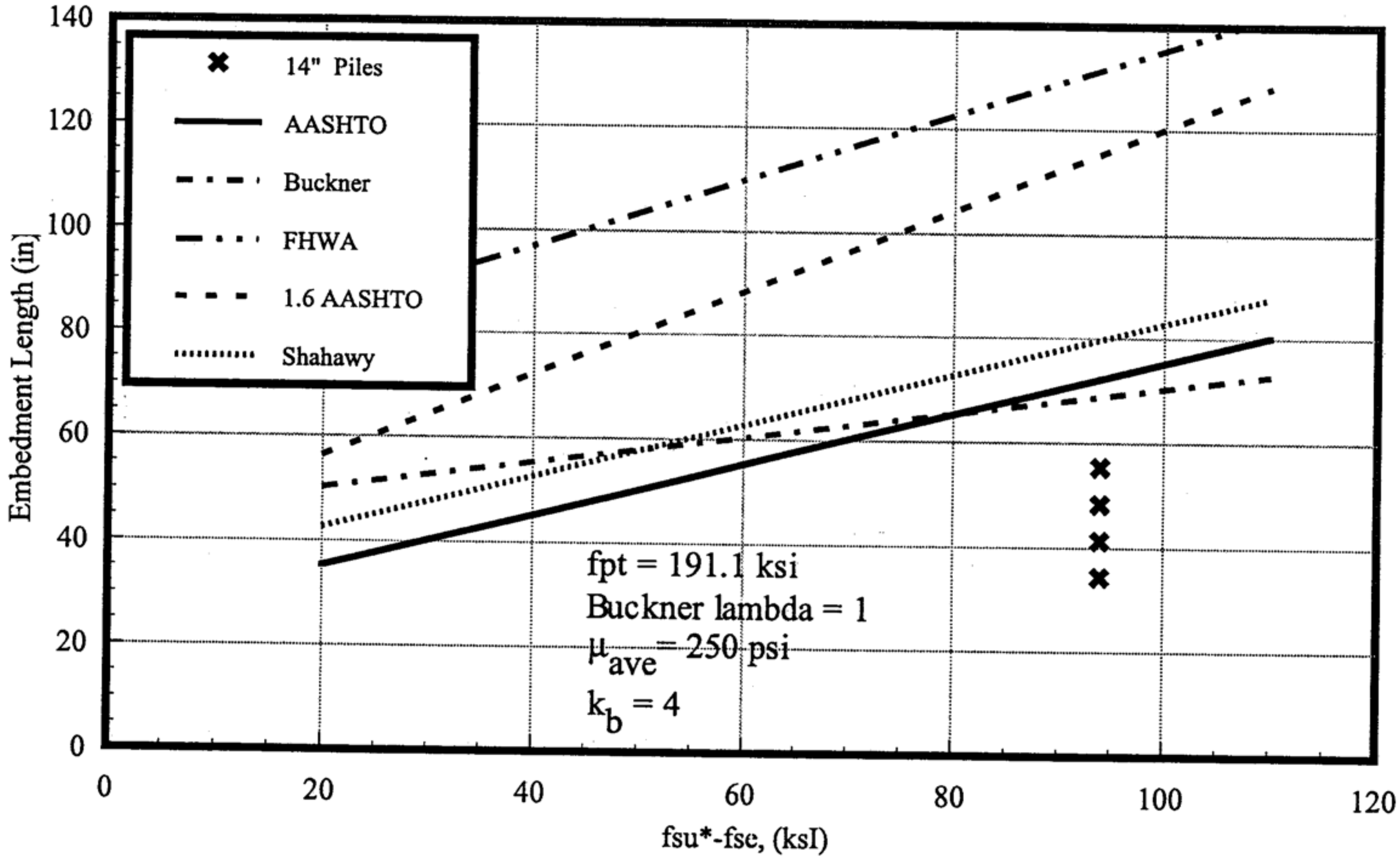


Figure C1 - Comparison of Development Length Equations
D = 0.5 in., f'_c = 5 ksi
14 in. Piles

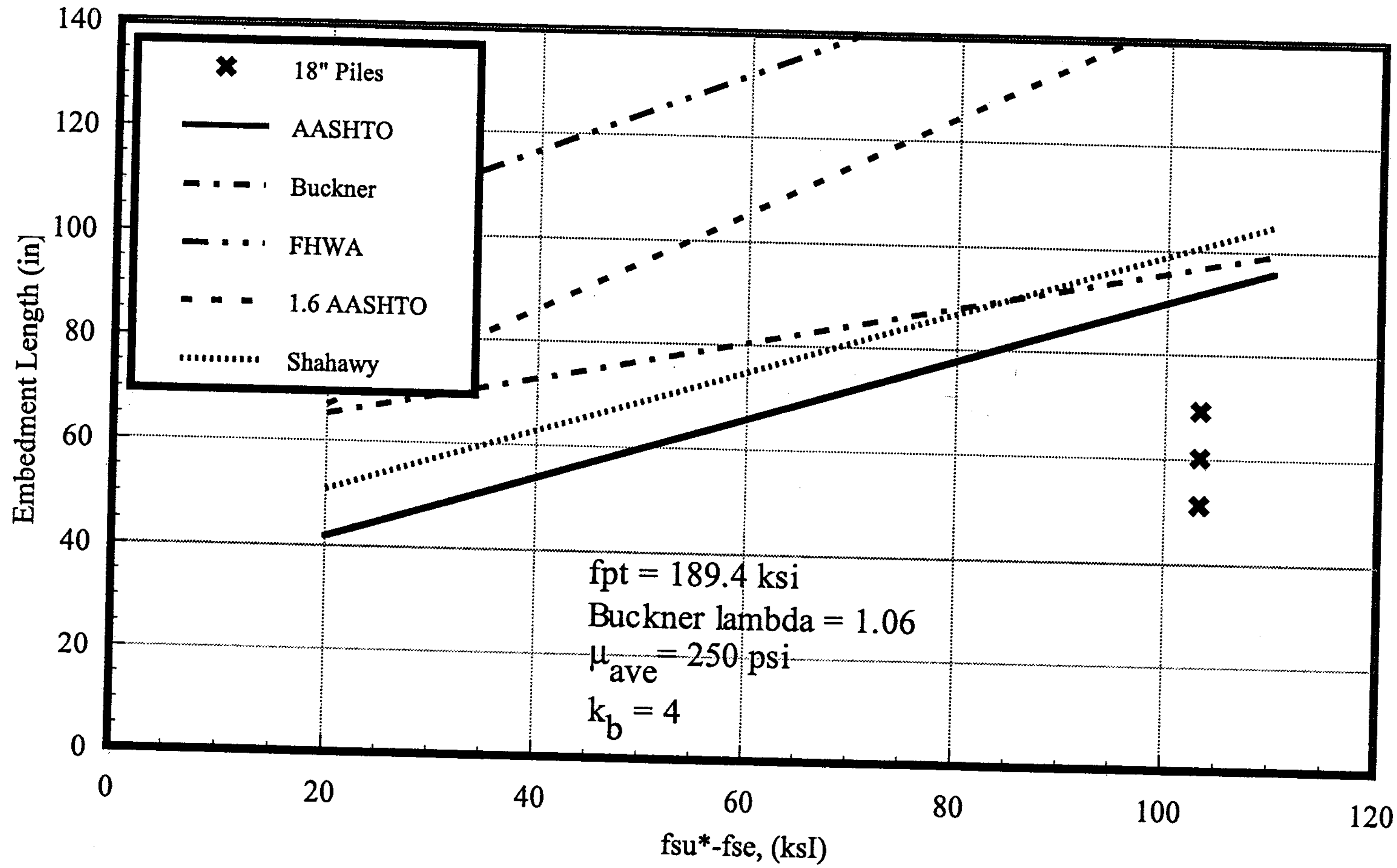


Figure C2 - Comparison of Development Length Equations
D = 0.6 in., and f'_c = 5 ksi
18 in. Piles

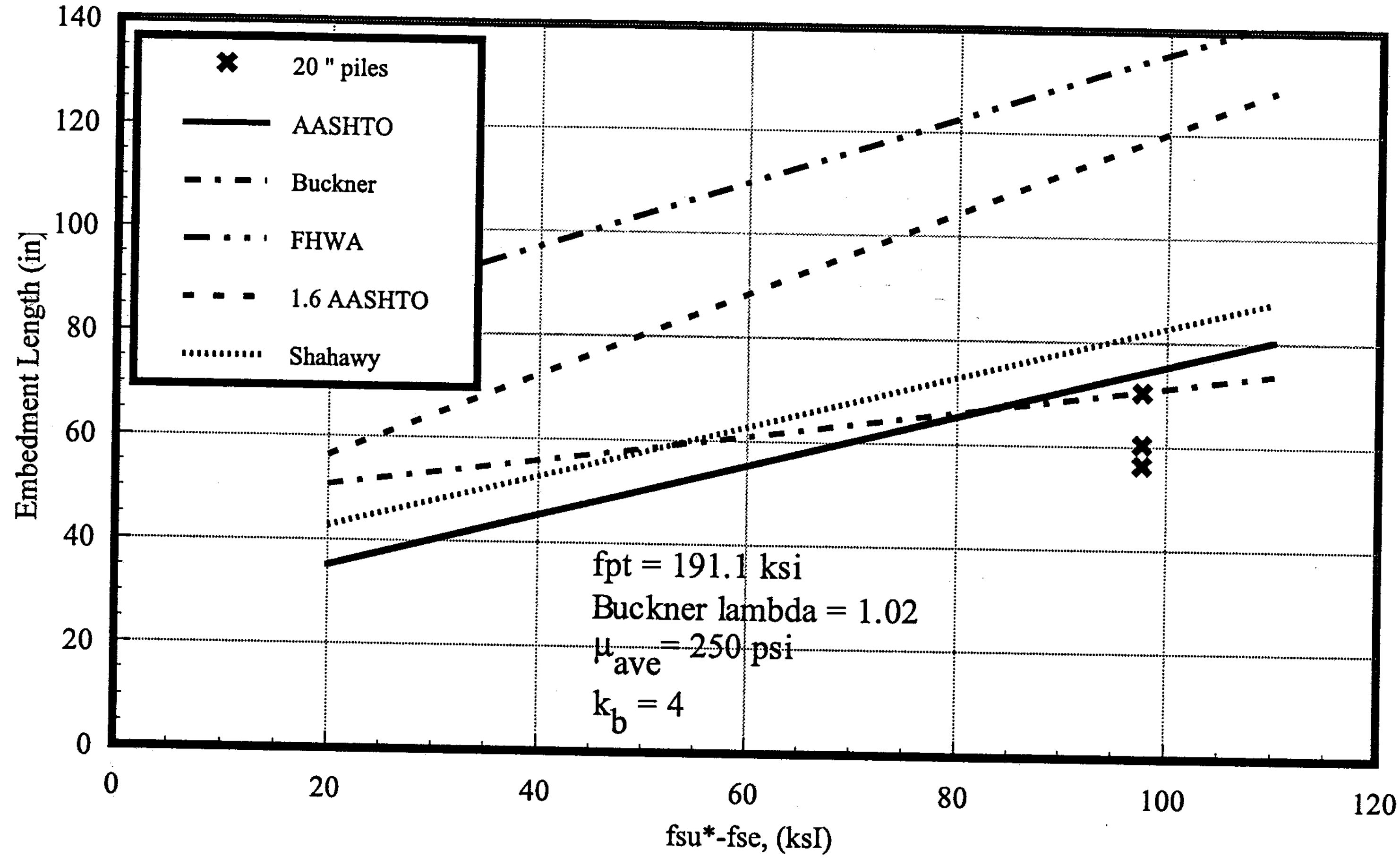


Figure C3 - Comparison of Development Length Equations
D = 0.5 in., f'_c = 5 ksi
20 in. Piles

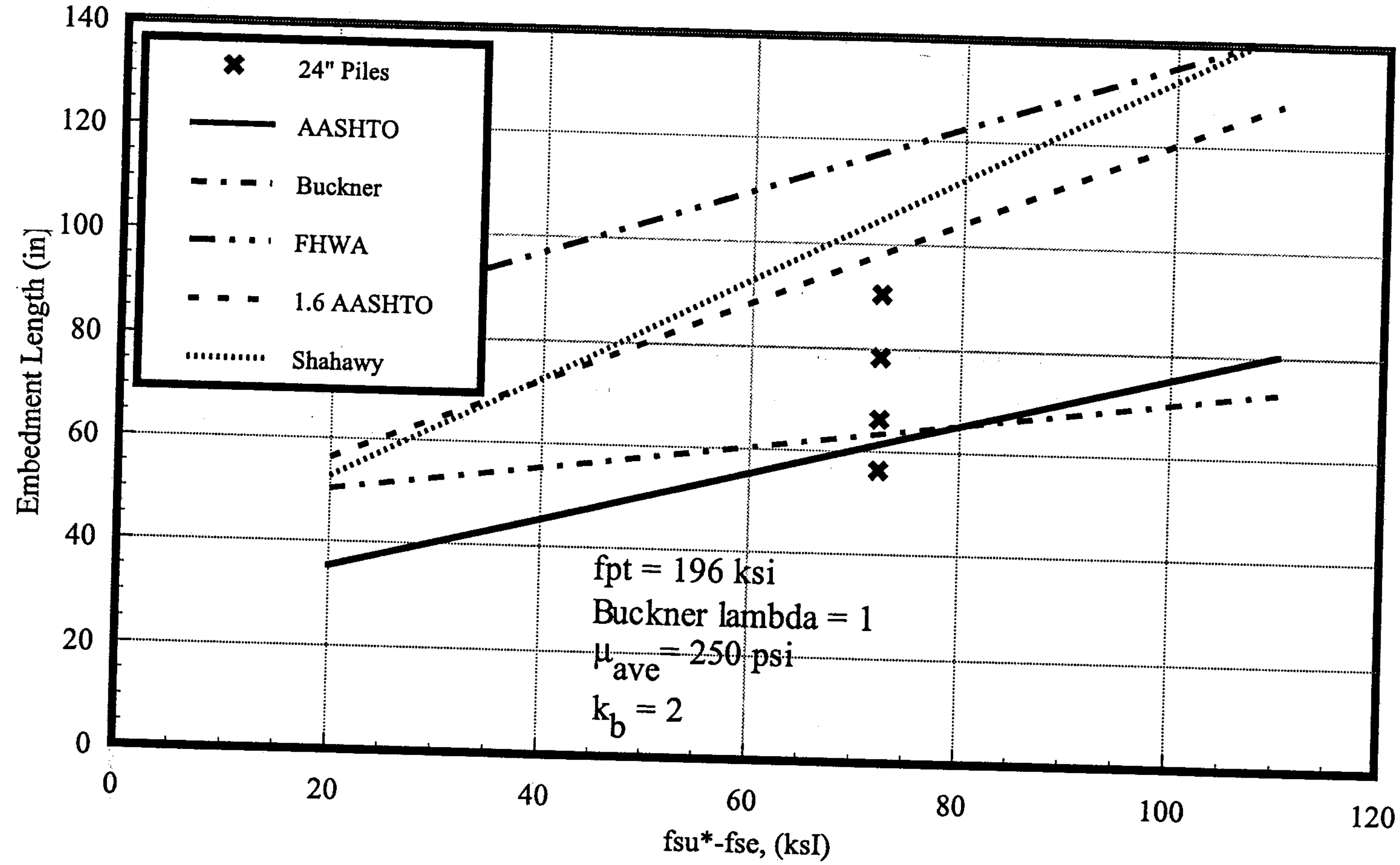


Figure C4 - Comparison of Development Length Equations
D = 0.5 in., f'_c = 5 ksi
24 in. Piles

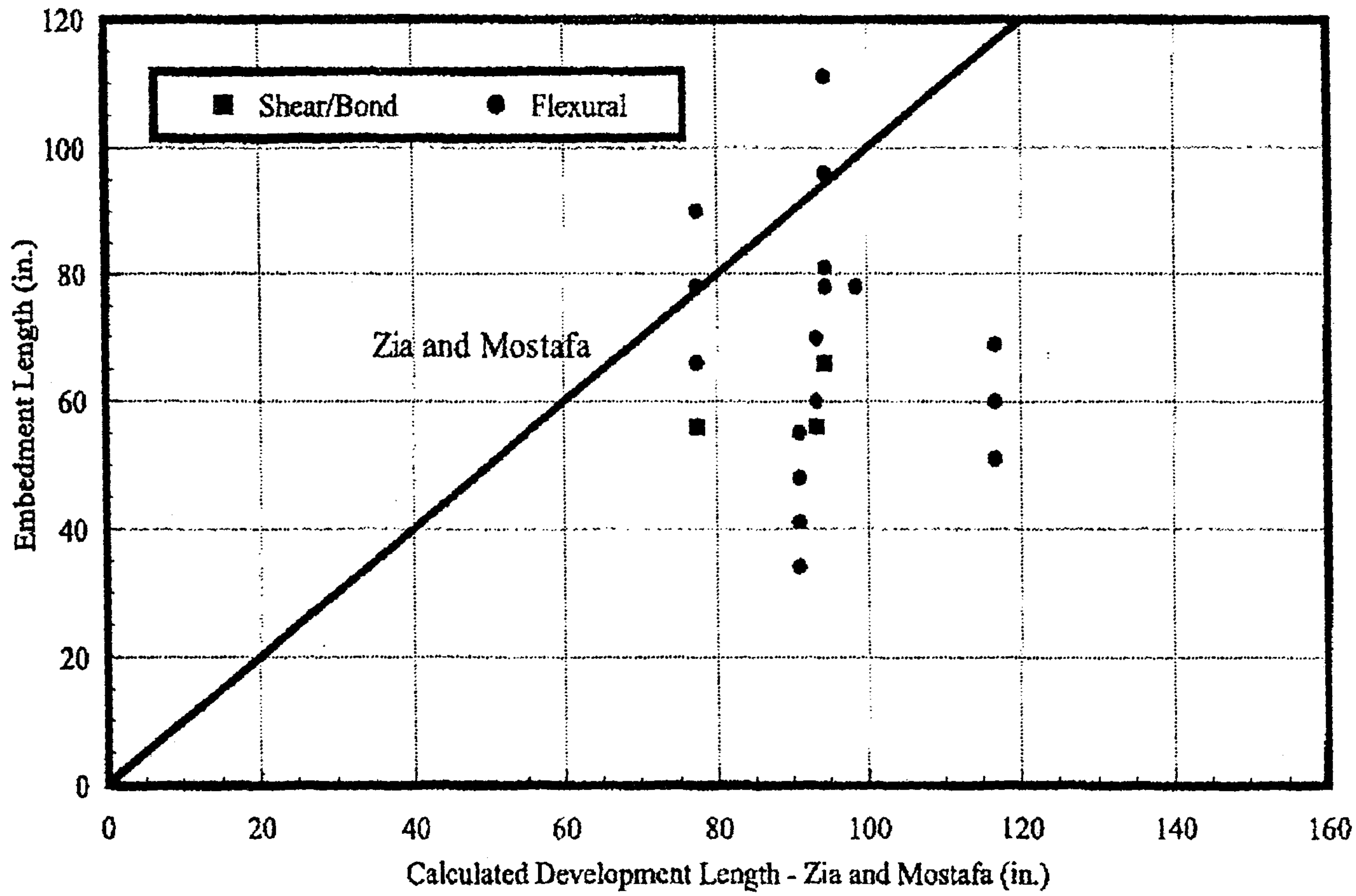


Figure C6 - Comparison of Embedment Length To Zia and Mostafa Calculated

All Piles

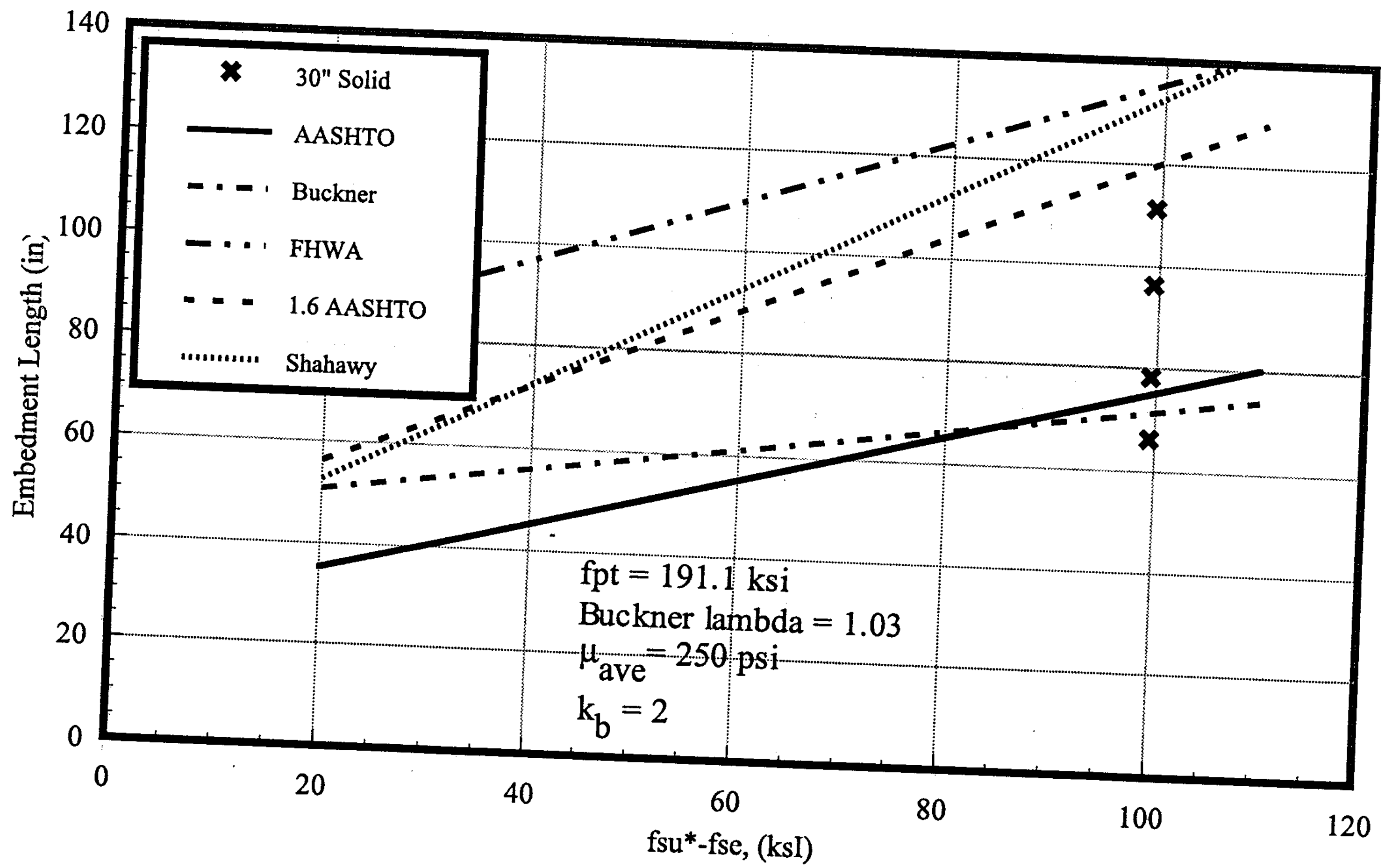


Figure C5 - Comparison of Development Length Equations
$D = 0.5"$, $f'_c = 5$ ksi
30" Solid Piles