A Critical Evaluation of the AASHTO Provisions for Strand Development Length of Prestressed Concrete Members

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This paper presents a critical evaluation of recent proposals for calculating the development length of prestressing strands. The results of an extensive testing program on a variety of prestressed concrete members are discussed. An evaluation of the results indicates that there is a direct interaction between shear and bond at the ends of prestressed concrete members. The effect of this interaction on the development length is important yet ignored in all existing proposals. This paper examines the relevance of recently proposed modifications of methods for calculating transfer and development lengths presented in the AASHTO Specifications. It presents proposed modifications based on a number of investigations conducted by the author over the past ten years. The present study offers a new rational approach for estimating the development length and compares it against other proposals. Based on the results, it is recommended that the current AASHTO equation [Eq. (2)] be used to estimate the development length of strands for prestressed concrete members with a depth of 24 in. (610 mm) or less. For deeper members, the simplified equation presented in this paper offers the best correlation with test results.

Prior to presenting the author's proposal for determining the strand development length of prestressed concrete members, the work of other investigators and background information is first reviewed.

Cousins et al.[1] generated considerable discussion on the development length for prestressing strands when, in 1986, they reported that the existing AASHTO[2] provisions for development length were inadequate, based on the findings of their test program. Because of their investigation, the Federal Highway Administration (FHWA) imposed a multiplier of 1.6 on the AASHTO equation for development length of strands up to $^9/_{16}$ in.

(14 mm) in diameter, and a minimum strand spacing of four times the nominal strand diameter.

Although these two provisions may not have a significant effect on the design of prestressed concrete bridge girders, they have a marked influence on the design of prestressed concrete piles. In many cases, the depth of a footer or pile cap is 4 to 5 ft (1.2 to 1.5 m), which does not provide sufficient embedment for piles to meet the longer development length requirement. This will often result in larger and deeper pile caps with a significant increase in construction cost.

To overcome this problem, one option that is being used by the Florida Department of Transportation (FDOT) is to provide mild steel reinforcement at the ends of the piles to resist the nominal moment at the interface of the pile and pile cap. This option is costly, however, because of the significant amount of mild steel reinforcement and labor required to cut the piles to size in the field.

Many bridge engineers have questioned the validity of the 1.6 multiplier, and this has spurred research by many investigators. The FDOT has questioned whether the multiplier is necessary, particularly in the design of prestressed concrete piles and short slab bridges. Consequently, the FDOT Structures Research Center (FSRC) has conducted extensive research on the transfer and development lengths of strands in prestressed concrete girders, slabs, and piles. The findings of these studies, and recommendations arising from them, are summarized in this paper.

In 1995, Buckner[3] presented a detailed review of proposals for calculating transfer and development lengths. He recommended the acceptance of the FSRC proposals for calculating transfer length of strand, and prescribed yet another modification to the AASHTO Specifications for calculating development length.

In 1998, the FHWA[4] published new proposals for transfer and development lengths in bridge beams and piles. Their objective was to provide more conservative predictions of transfer and development lengths for all concrete strengths, including high strength concrete. They argued that existing proposals reflect average values of normal strength concrete and are, therefore, unconservative.

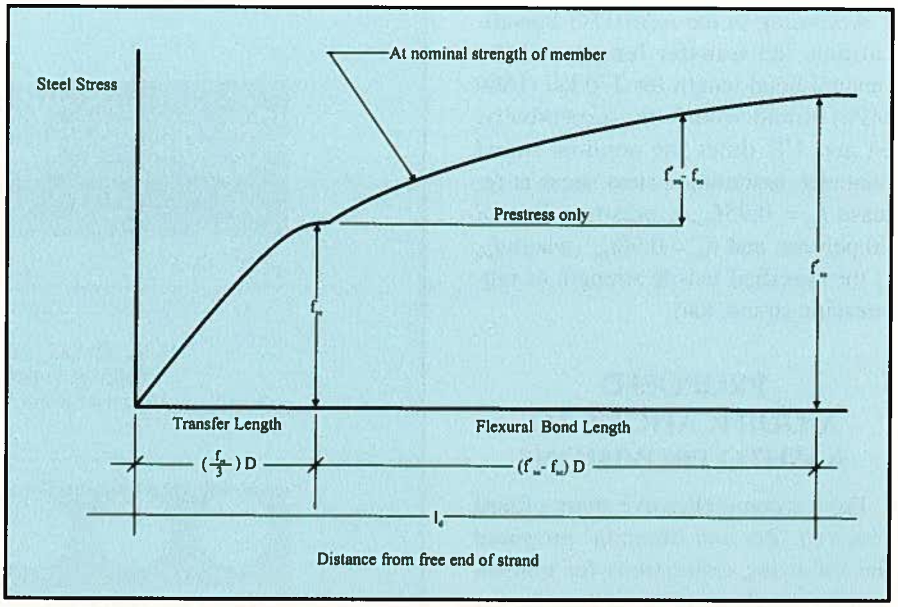


Fig. 1. Variation of steel stress with distance from free end of strand.

The objectives of the present study are to:

1. Compare and contrast the development length equations given by the AASHTO Specifications, those proposed by other researchers based on their studies, and those recommended by the FHWA in its 1998 report.
2. Compare the FHWA results with findings from the FSRC tests.
3. Present a rational method for calculating development length of prestressing strands.

CURRENT AASHTO PROVISIONS

The current AASHTO Specifications (16th Edition with 1999 Interim Revisions) require that three- and seven-wire prestressing strands be bonded beyond the critical section for a development length, L_d, in inches, not less than that given by:

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

where D is the nominal diameter of the strand (in.), f_{su}^* is the stress in the prestressed reinforcement at nominal strength (ksi), and f_{se} is the effective stress in the prestressed strand after all losses (ksi). The parenthetical expression is considered to be unitless.

AASHTO also provides that the 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, AASHTO provides that the development length required by Eq. (1) shall be doubled.

Eq. (1) can be rewritten in the form:

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

Eq. (2) is shown graphically in Fig. 1. The first term in the equation is 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, or 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 the prestressing steel at transfer, and the amount of prestress loss. Zia and Mostafa[5] pointed out that the "3" in the denominator of the transfer length term represents a conservative average concrete strength in ksi.

Similarly, in the flexural bond length term, a denominator of 1 ksi (6.9 MPa) is implied, which is a factored value of an average bond stress of 250 psi (1.72 MPa) over the flexural bond length.

According to the AASHTO Specifications, the transfer length and the flexural bond length for 270 ksi (1860 MPa) strand would be, respectively, 54 and 103 times the nominal strand diameter, assuming a steel stress at release $f_{si} = 0.75f_{pu}$, a prestress loss of 20 percent, and $f^*_{su} = 0.98f_{pu}$ (where f_{pu} is the specified tensile strength of prestressing strand, ksi).

PROPOSED MODIFICATIONS TO AASHTO PROVISIONS

From a comprehensive study of past research, Zia and Mostafa[5] proposed the following expressions for transfer length, L_t, flexural bond length, L_b, and development length, 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 the time of initial prestress release, ksi.

Eq. (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_{ave} = 250 psi (1.72 MPa). In Eq. (4), however, a value of u_{ave} = 200 psi (1.38 MPa) is assumed.

The expression for transfer length, Eq. (3), is assumed to be applicable for concrete strengths ranging from 2 to 8 ksi (14 to 55 MPa), and accounts for the effects of strand size, initial prestress, and concrete strength at transfer. This equation gives transfer lengths comparable to those specified in the AASHTO Specifications, particularly in cases where the concrete strength at transfer is low. However, a comparison of Eq. (2) and Eq. (4) indicates that the AASHTO provisions for L_b, and hence L_d, would be inadequate.

Based on the work of Cousins et al.,[1] the FHWA initially required the application of a 2.5 multiplier to

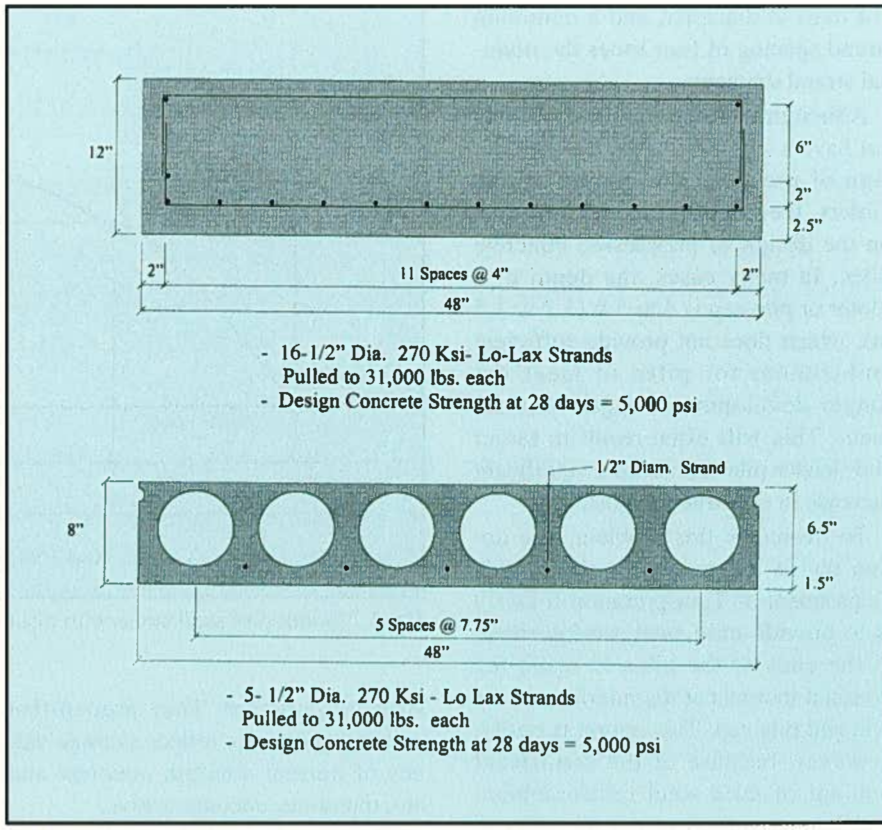


Fig. 2. Solid and voided slab cross sections.

AASHTO Eq. (1), but the FDOT opposed the use of any multiplier. After further deliberation, the FDOT recommended to FHWA that the value of the multiplier be reduced to 1.6.

In 1988, at a joint meeting between the AASHTO Technical Committee for Prestressed Concrete and the PCI Bridge Committee, the recommendation for a multiplier value of 1.6 was formally presented for use with strands up to $^9/_{16}$ in. (14 mm) in diameter. The FHWA accepted this recommendation, but retained the provision for minimum strand spacing.

As a result of an extensive study by FSRC[6] in 1992, it was recommended that the strand transfer length be calculated as:

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

where f_{si} replaces f_{se} in Eq. (2).

It should be noted, however, that the transfer length $(f_{se}/3)D$ in Eq. (2) is based on experimental studies conducted on stress-relieved 250 ksi (1720 MPa) strand. Since that time, the industry standard has changed to Grade 270 (1860 MPa) low-relaxation strand, which has a cross-sectional area about 6 percent greater than Grade 250 (1720 MPa) strand of the same nominal diameter. Furthermore, the use of low-relaxation strand over stress-relieved strand results in higher transfer stresses, and hence a requirement for a higher transfer length.

Buckner[3] 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 [$f'_{ci} \geq$ 3500 psi (24 MPa)] should be taken as the expression for transfer length proposed by FSRC[6,7] in Eq. (6). This equation was shown to be representative of test results. Eq. (6) can be rationalized 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.

In general, the transfer length calculated from Eq. (6) is about 20 percent greater than that resulting from the use of the current AASHTO provisions. An approximate transfer length of 50D is allowed. Buckner[3] suggested that

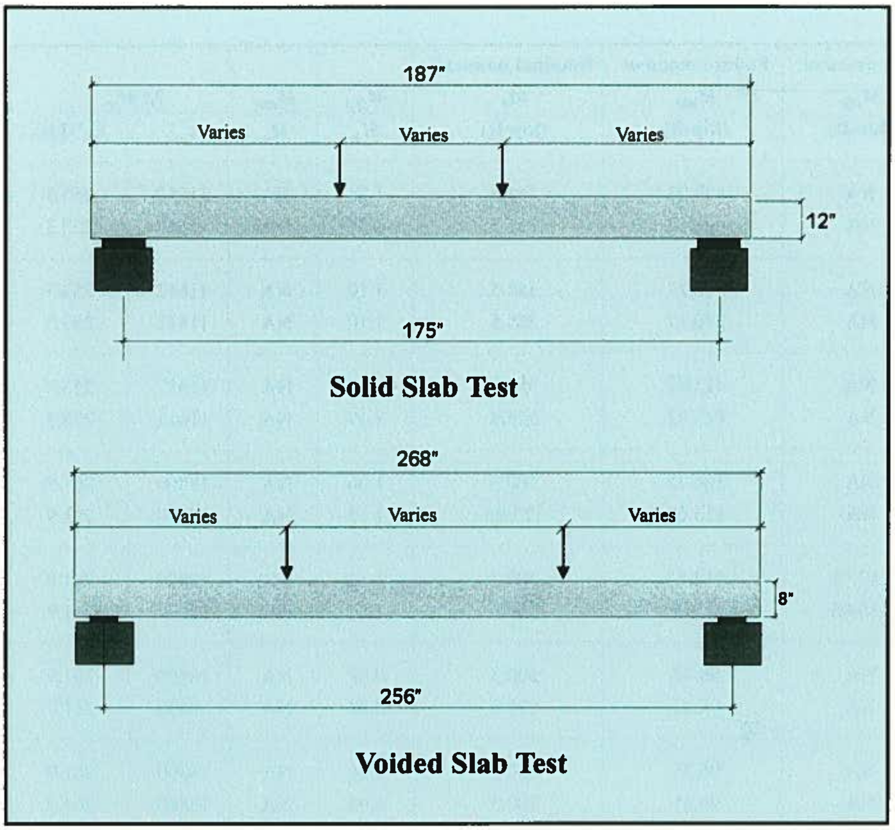


Fig. 3. Slab test setup.

this value should be increased to $60D$ to account for the longer transfer length of Grade 270 (1860 MPa) strands and to reflect the general findings of recent studies.

The author pointed out in Reference 7 that the AASHTO provisions are inadequate for strand development length. In particular, the requirement for the development of nominal flexural strength close to the support is somewhat unrealistic in the case of straight strands. Since a shear mode of failure is likely in girders in which the shear span-to-depth ratio, 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, based on the experimental results, that a development length of 130 in. (3.30 m) ($260D$) for $^1/_2$ in. (13 mm) and $^1/_2$ in. (13 mm) special strands, and 140 in. (3.56 m) ($233D$) for 0.6 in. (15 mm) diameter strands, would allow the development of the normal design moment. In addition, the FSRC study indicated a direct interaction between shear and bond at the ends of girders, and stated that the initial slippage of strand occurs immediately or shortly after the appearance of the first diagonal crack.

The author suggested that the provisions for strand development length be expressed by the following equation (hereafter referred to as Shahawy 93):

$$L_t = \left(\frac{f_{si}}{3}\right)D + \frac{\left(f_{su}^* - f_{se}\right)D}{k_b u_{ave}} \quad (7)$$

where k_b is a dimensionless factor that reflects the actual value of average flexural bond stress that can be developed in particular cases, and u_{ave} represents a basic average value of bond stress of 0.25 ksi (1.72 MPa). 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 current AASHTO provision applies)

$k_b = 2$, if the ratio of L_d/h, calculated using $k_b = 4$, is equal to or less than 3

Buckner[3] proposed the following development length equation for pretensioned strands:

$$L_d = \left(\frac{f_{si}}{3}\right)D + \lambda\left(f_{su}^* - f_{se}\right)D \quad (8)$$

where

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

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

Buckner stated that if Eq. (18-3) of the ACI Building Code[8] is used to calculate the 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 stated 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:

$$1.0 \le (0.6 + 40\varepsilon_{ps}) \le 2.0 \quad (9)$$

In the ACI Code 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

After extensive statistical manipulation of results from several studies, the FHWA[4] presented the following equations for transfer length, flexural bond length, and development length:

$$L_t = \frac{4 f_{pt} D}{f_c'} - 5 \quad (10)$$

$$L_b = \frac{6.4\left(f_{su}^* - f_{se}\right)D}{f_c'} + 15 \quad (11)$$

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

where f_{pt} is the stress in the prestressing steel prior to transfer of prestress.

Eqs. (10) to (12) reflect the effect of f_c' in light of the use of concrete strengths higher than usual. FHWA has suggested that for $f_c' > 10{,}000$ psi (69 MPa), a value of $f_c' = 10{,}000$ psi (69 MPa) should be substituted in the above equations.

It is also recommended that Eqs. (10) to (12) be used for piles, and that a 1.3 multiplier be applied to any

Table 1. Results of slab tests.

Slab number*	End	Span, L (in.)	Embedment (in.)	Slip moment M_{slip} (kip-ft)	Failure moment M_{app} (kip-ft)	Nominal moment M_n (kip-ft)	$\frac{M_{app}}{M_n}$	$\frac{M_{slip}}{M_n}$	At M_n ε_{ps}†	At M_n f_{su}^*† (ksi)
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	115.08	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 slabs; VS - Voided slabs; N - North end; S - South end
† Determined by strain-compatibility analysis.

Note: 1 in. = 25.4 mm; 1 kip-ft = 1.356 kN-m; 1 ksi = 6.895 MPa.

strands (straight or draped) in any member (beam or pile) that has 12 in. (305 mm) or more of concrete cast beneath them. This latter recommendation, which Buckner[3] also proposed, is intended to address the lower bond strengths that can be developed in so-called top reinforcement.

The development of Eqs. (10) to (12) was based on statistical analyses of reported test results. However, the physical significance of the transfer length equation, Eq. (10), is open to question, based on the fact that for a value of f_{pt} = 0 (nonprestressed concrete), a value of L_t = –5 in. (–127 mm) would result. Graphically, the curve for Eq. (10) should preferably pass through the origin, as should Eq. (3) proposed by Zia and Mostafa.[5]

One of the observations from the FHWA study is that there exists significant variation in the strand surface conditions from the different manufacturers. Contamination of the strand surface due to the oil or grease used in the strand production process has a negative effect on the measured development length. Eqs. (10) to (12) take this variation into account. Controlling the quality of the strand surface conditions is an important issue that will be discussed later in this paper.

FSRC TESTS

Since 1990, the FDOT Structures Research Center has conducted extensive studies[6,7] on the behavior of prestressed concrete members, including voided and solid slabs, piles, and girders. In this section, a summary of these tests, along with a new expression for calculating the development length, will be presented. Comparisons will be made between the FSRC test results and the transfer and development lengths of strands predicted by the expressions presented in this paper.

Solid and Voided Prestressed Concrete Slabs

The main objective of this test program was to determine the minimum development length of unshielded $^1/_2$ in. (13 mm) diameter, 270 ksi (1860 MPa) low-relaxation prestressing strand. Seven full-scale prestressed solid and voided slabs, with cross-sectional details and dimensions shown in Fig. 2, were tested to failure. They were tested under static loads applied incrementally to failure, with the location of the loading varied as shown in Fig. 3.

Note that the development length calculated by the AASHTO provisions for solid and voided slabs is approximately 74 and 78 in. (1.88 and 1.98 m) (148D and 156D), respectively. Applying the FHWA multiplier of 1.6 increases these lengths to 118 and 125 in. (3.00 and 3.18 m), which renders these types of members unusable over short spans.

The three solid slabs tested were loaded with two symmetrically applied point loads. 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. Each slab was loaded beyond its nominal moment capacity, or until significant strand slippage occurred. The loading assembly was then removed and placed at a fixed 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 ratio of shear span to depth, a/d, was 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.

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 the support, quarter and midspan points, and the load point. The ends of the prestressing strands were instrumented with LVDTs to monitor strand slippage throughout the loading. The gauge measurements were recorded at every load stage, and crack development was marked during testing. Upon completion of each test, a complete crack chart was developed and filed with the other test information.

Table 1 summarizes the test results. The applied test moment, M_{app}, includes the effect of the self-weight dead load. The embedment lengths in the tests were 65, 70, and 77 in. (1.65, 1.78, and 1.96 m).

Fig. 4. Encased pile test setup.

In the SS1 test, the member was loaded to complete failure. To avoid damage to the instrumentation and to ensure safety of personnel, 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, no strand slippage was observed. In the VS2 slab, end slippage of only one strand [0.003 in. (0.08 mm)] was detected at the maximum applied load, but the slippage was negligible at the nominal moment value. Even so, the

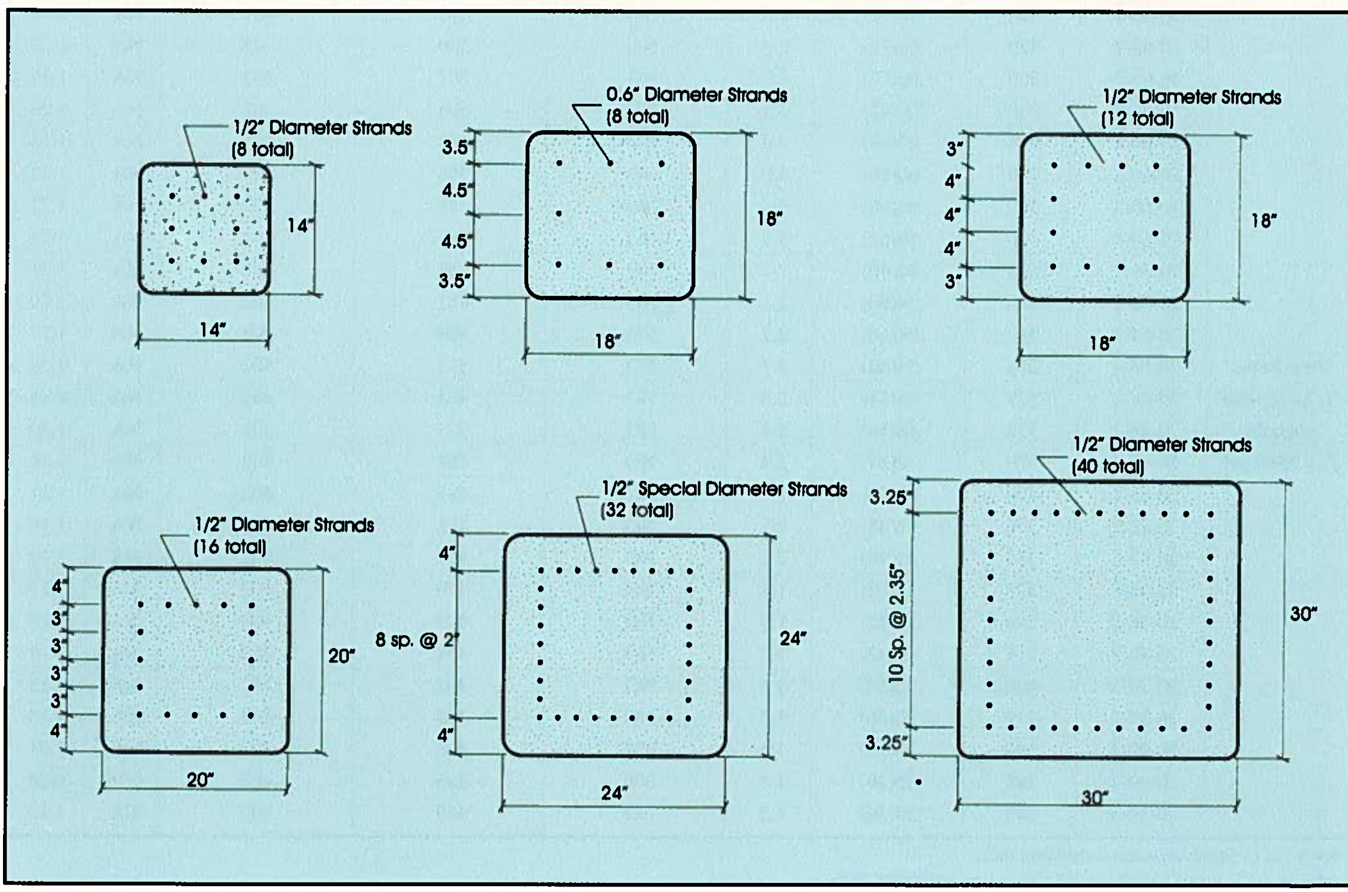


Fig. 5. Pile cross sections and strand configurations.

Table 2. Pile test results.

Group (1)	Specimen number (2)	Span (L) (in.) (3)	Shear span* (a) (in.) (4)	a/h (5)	Slip moment M_{slip} (kip-ft) (6)	Failure moment M_{app} (kip-ft) (7)	Nominal moment M_n (kip-ft) (8)	$\frac{M_{slip}}{M_n}$ (9)	$\frac{M_{app}}{M_n}$ (10)
14 x 14 in. $^1/_2$ in. strands $f_c' = 6500$ psi	14-49-1	130	49(55)	3.5	NO	134	120.3	NA	1.11
	14-42-1	156	42(48)	3.0	NO	129	120.3	NA	1.07
	14-42-2	182	42(48)	3.0	NO	143	120.3	NA	1.19
	14-35-1	156	35(41)	2.5	NO	139	120.3	NA	1.16
	14-28-1	130	28(34)	2.0	NO	137	120.3	NA	1.14
	14-28-2	130	28(34)	2.0	NO	132	120.3	NA	1.10
Group A 18 x 18 in. 0.6 in. strands $f_c' = 7500$ psi	A18-63-1	233	63(69)	3.5	NO	234	239.4	NA	0.98
	A18-63-2	233	63(69)	3.5	NO	234	239.4	NA	0.98
	A18-54-1	200	54(60)	3.0	184	210	239.4	0.77	0.88
	A18-54-2	200	54(60)	3.0	237	243	239.4	0.99	1.02
	A18-45-1	167	45(51)	2.5	192	233	239.4	0.80	0.97
	A18-45-2	167	45(51)	2.5	184	216	239.4	0.77	0.90
Group B 18 x 18 in. $^1/_2$ in. strands $f_c' = 7000$ psi	B18-54-1	492	54(60)	3.0	NO	305	263	NA	1.16
	B18-48-1	456	48(54)	2.67	NO	296	263	NA	1.12
	B18-48-2	432	48(54)	2.67	NO	301	263	NA	1.14
	B18-48-3	336	48(54)	2.67	NO	315	263	NA	1.20
	B18-42-1	324	42(48)	2.33	NO	287	263	NA	1.09
	B18-42-2	288	42(48)	2.33	NO	306	263	NA	1.16
	B18-42-3	264	42(48)	2.33	NO	283	263	NA	1.08
	B18-36-1	480	36(42)	2.0	NO	293	263	NA	1.11
	B18-36-2	408	36(42)	2.0	NO	312	263	NA	1.19
	B18-36-3	312	36(42)	2.0	NO	284	263	NA	1.08
	B18-36-4	292	36(42)	2.0	276.8	290	263	1.05	1.10
	B18-30-1	276	30(36)	1.67	NO	285	263	NA	1.08
	B18-30-2	204	30(36)	1.67	NO	332	263	NA	1.26
	B18-30-3	204	30(36)	1.67	NO	298	263	NA	1.13
20 x 20 in. $^1/_2$ in. special strands $f_c' = 7500$ psi	20-66-1	528	66(72)	3.3	NO	498	402	NA	1.24
	20-66-2	420	66(72)	3.3	NO	500	402	NA	1.24
	20-66-3	360	66(72)	3.3	NO	507	402	NA	1.26
	20-66-4	240	66(72)	3.3	NO	506	402	NA	1.26
	20-60-1	528	60(66)	3.0	NO	475	402	NA	1.18
	20-60-2	420	60(66)	3.0	NO	496	402	NA	1.23
	20-60-3	360	60(66)	3.0	NO	489	402	NA	1.22
	20-60-4	240	60(66)	3.0	NO	495	402	NA	1.23
	20-54-1	528	54(60)	2.7	NO	488	402	NA	1.21
	20-54-2	420	54(60)	2.7	NO	481	402	NA	1.20
	20-54-3	312	54(60)	2.7	NO	509	402	NA	1.27
	20-54-4	216	54(60)	2.7	NO	513	402	NA	1.28
	20-48-1	528	48(54)	2.4	NO	464	402	NA	1.15
	20-48-2	528	48(54)	2.4	NO	523	402	NA	1.30
	20-48-3	420	48(54)	2.4	NO	468	402	NA	1.16
	20-42-1	420	42(48)	2.1	NO	485	402	NA	1.21
	20-42-2	336	42(48)	2.1	NO	465	402	NA	1.16
	20-42-3	312	42(48)	2.1	NO	477	402	NA	1.19
	20-36-1	528	36(42)	1.8	NO	470	402	NA	1.17
	20-36-2	276	36(42)	1.8	NO	503	402	NA	1.25
	20-36-3	216	36(42)	1.8	NO	458	402	NA	1.14
	20-30-1	420	30(36)	1.5	NO	462	402	NA	1.15
	20-30-2	312	30(36)	1.5	NO	463	402	NA	1.15
	20-30-3	180	30(36)	1.5	373	406	402	0.93	1.01
	20-30-4	288	30(36)	1.5	306	336	402	0.76	0.84
	20-30-5	240	30(36)	1.5	NO	449	402	NA	1.12

* Bracketed ( ) figures represent embedment (in.).
Note: 1 in. = 25. 4 mm; 1 kip-ft = 1.356 kN-m; 1 psi = 0.006895 MPa.

Table 2 (cont.). Piles test results.

Group (1)	Specimen number (2)	Span (L) (in.) (3)	Shear span* (a) (in.) (4)	a/h (5)	Slip moment M_{slip} (kip-ft) (6)	Failure moment M_{app} (kip-ft) (7)	Nominal moment M_n (kip-ft) (8)	$\frac{M_{slip}}{M_n}$ (9)	$\frac{M_{app}}{M_n}$ (10)
Group A 24 x 24 in. 1/2 in. strands $f'_c = 7000$ psi	A24-84-1	311	84(90)	3.5	NO	1005	776.8	NA	1.29
	A24-72-1	267	72(78)	3.0	NO	1055	776.8	NA	1.36
	A24-72-2	267	72(78)	3.0	NO	1035	776.8	NA	1.33
	A24-60-1	222	60(66)	2.5	NO	1047	776.8	NA	1.35
	A24-60-2	222	60(66)	2.5	NO	1098	776.8	NA	1.41
	A24-60-3	222	60(66)	2.5	NO	970	776.8	NA	1.25
	A24-48-1	234	48(56)	2.0	414	552	776.8	0.53	0.71
	A24-48-2	222	48(56)	2.0	586	737	776.8	0.75	0.95
Group B 24 x 24 in. 1/2 in. special strands $f'_c = 7500$ psi	B24-66-1	360	66(72)	2.75	NO	813	646	NA	1.26
	B24-66-2	288	66(72)	2.75	NO	805	646	NA	1.25
	B24-66-3	288	66(72)	2.75	NO	848	646	NA	1.31
	B24-60-1	576	60(66)	2.5	NO	789	646	NA	1.22
	B24-60-2	360	60(66)	2.5	NO	827	646	NA	1.28
	B24-54-1	552	54(60)	2.25	NO	733	646	NA	1.13
	B24-54-2	432	54(60)	2.25	NO	827	646	NA	1.28
	B24-54-3	372	54(60)	2.25	NO	835	646	NA	1.29
	B24-48-1	576	48(54)	2.00	639	714	646	0.99	1.10
	B24-48-2	576	48(54)	2.00	701	718	646	1.08	1.11
	B24-48-3	480	48(54)	2.00	NO	846	646	NA	1.31
	B24-48-4	312	48(54)	2.00	NO	858	646	NA	1.33
	B24-42-1	264	42(48)	1.75	700	735	646	1.08	1.14
	B24-42-2	192	42(48)	1.75	NO	662	646	NA	1.02
	B24-42-3	182	42(48)	1.75	674	696	646	1.04	1.04
30 x 30 in. 1/2 in. strands $f'_c = 6000$ psi	30-105-1	389	105(111)	3.5	NO	1387	1283.5	NA	1.08
	30-90-1	334	90(96)	3.0	NO	1473	1283.5	NA	1.15
	30-90-2	334	90(96)	3.0	NO	1443	1283.5	NA	1.12
	30-72-1	240	72(78)	3.0	NO	1485	1283.5	NA	1.16
	30-72-2	240	72(78)	3.0	1200	1550	1283.5	0.93	1.21
	30-60-1	222	60(66)	2.0	1152	1242	1283.5	0.90	0.96
	30-60-2	222	60(66)	2.0	NO	1485	1283.5	NA	1.16
	30-60-3	222	60(66)	2.0	876	1097	1283.5	0.68	0.85

* Bracketed ( ) figures represent embedment (in.).
Note: 1 in. = 25. 4 mm; 1 kip-ft = 1.356 kN-m; 1 psi = 0.006895 MPa.

maximum applied moments for the north and south tests were, respectively, 12 and 15 percent greater than the nominal moment values.

The specimen behavior and crack patterns indicated that all failures were flexural. In only one test specimen (SS2) was a diagonal crack observed (between a load point and support).

Table 1 shows that the value of f^*_{su} for solid slabs was just below the value of the yield stress f_{py} [≈ 256 ksi (1770 MPa)] of the strand. The value of f^*_{su} in the voided slabs was 261.9 ksi (1806 MPa), which exceeded the yield stress of the strand. Except for voided slab Specimens VS3 and VS4, the applied moment exceeded the nominal moment. In Specimens VS3 and VS4, the applied moments were, respectively, 98 and 96 percent of the nominal value.

It can be inferred from Table 1 that an embedment length of 65 in. (1.65 m) is adequate to develop the nominal moment of both solid and voided slabs containing 1/2 in. (13 mm) diameter, 270 ksi (1860 MPa) strand. This represents about 85 percent of the prescribed AASHTO value for development length [77 in. (1.96 m)].

A comparison of the slab test results with the current AASHTO provisions for development length [Eq. (1) or (2)] clearly indicates that the AASHTO provisions are both adequate and conservative. Provisions that include the 1.6 multiplier are far too conservative for these types of members. It should also be noted that the 65 in. (1.65 m) embedment length was the minimum used in the tests, so it is quite feasible that the strands can be developed at embedment lengths less than 65 in. (1.65 m).

All the expressions discussed earlier, with the exception of FSRC Eq. (7) and Buckner Eq. (8), yield results that are considerably higher than those obtained by AASHTO. The results using Eq. (7) for the solid and voided slabs are approximately 82 and 86 in. (2.08 and 2.18 m), respectively, which are slightly higher than the AASHTO results.

Prestressed Concrete Piles

The first study of piles conducted by the author[9] examined the effect of pile embedment on the development length of 1/2 in. (13 mm) diameter prestressing strand embedded in a pile cap or footing. Fig. 4 shows one of the test setups from the previous study. It was concluded that this type of end condition should result in lower development length requirements due to the enhancement of bond strength caused by concrete shrinkage and resulting confinement.

The measured bond strength exceeded 500 psi (3.4 MPa), which is twice the bond value used in the AASHTO development length equation [Eq. (2)]. As a result of the findings of the experimental and analytical study, it was recommended that a minimum embedment length of 50 in. (1.27 m) [100*D* for $^1/_2$ in. (13 mm) diameter strands] be adopted for piles embedded in pier caps. Complete details of that investigation and associated recommendations can be found in Reference 9.

In a subsequent study by FSRC, prestressed concrete piles were tested without simulation of the embedment conditions just referred to. The main objectives of this study were to experimentally evaluate the development length requirements and to study the effect of section depth on the measured values.

Tests were conducted on standard precast, prestressed piles having six different cross sections (see Fig. 5). The test specimens were cut from precast, prestressed members, and the prestressing strand nominal diameters were $^1/_2$ in., $^1/_2$ in. special, and 0.6 in. (13, 13, and 15 mm). Test span length and shear span were other variables in the study.

Table 2 shows a summary of all pile tests. Each group of piles representing a certain cross section was obtained from ongoing or completed bridge construction jobs to ensure correlation with actual conditions. These pile groups, which were produced by various prestressed concrete manufacturers in Florida, were transported to the FDOT Structural Research Center for testing.

Testing consisted of the application of load up to failure, using an incremental point load at various distances from the support. Columns 1 to 5 of Table 2 give details of the loading parameters and Fig. 6 shows the test

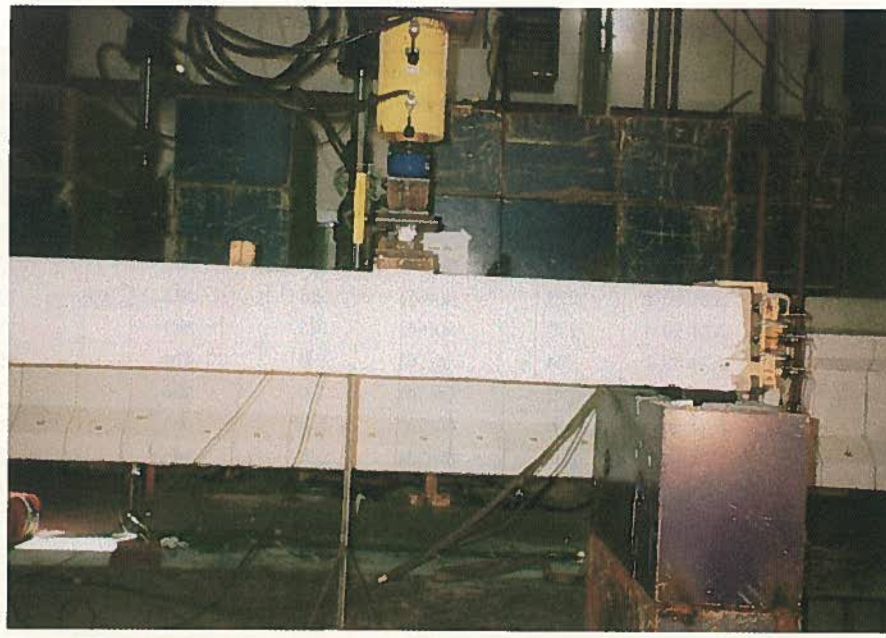


Fig. 6. Pile test setup.

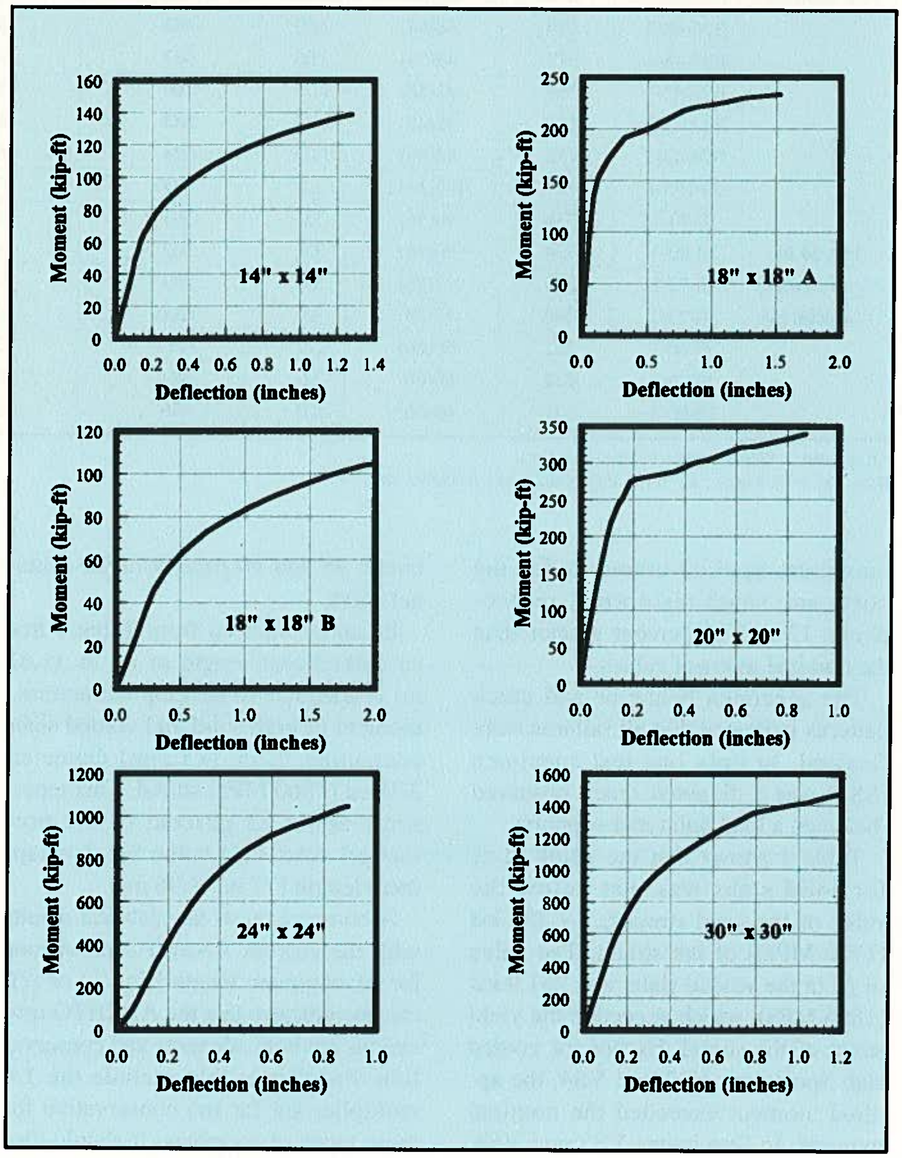


Fig. 7. Moment versus deflection (all piles).

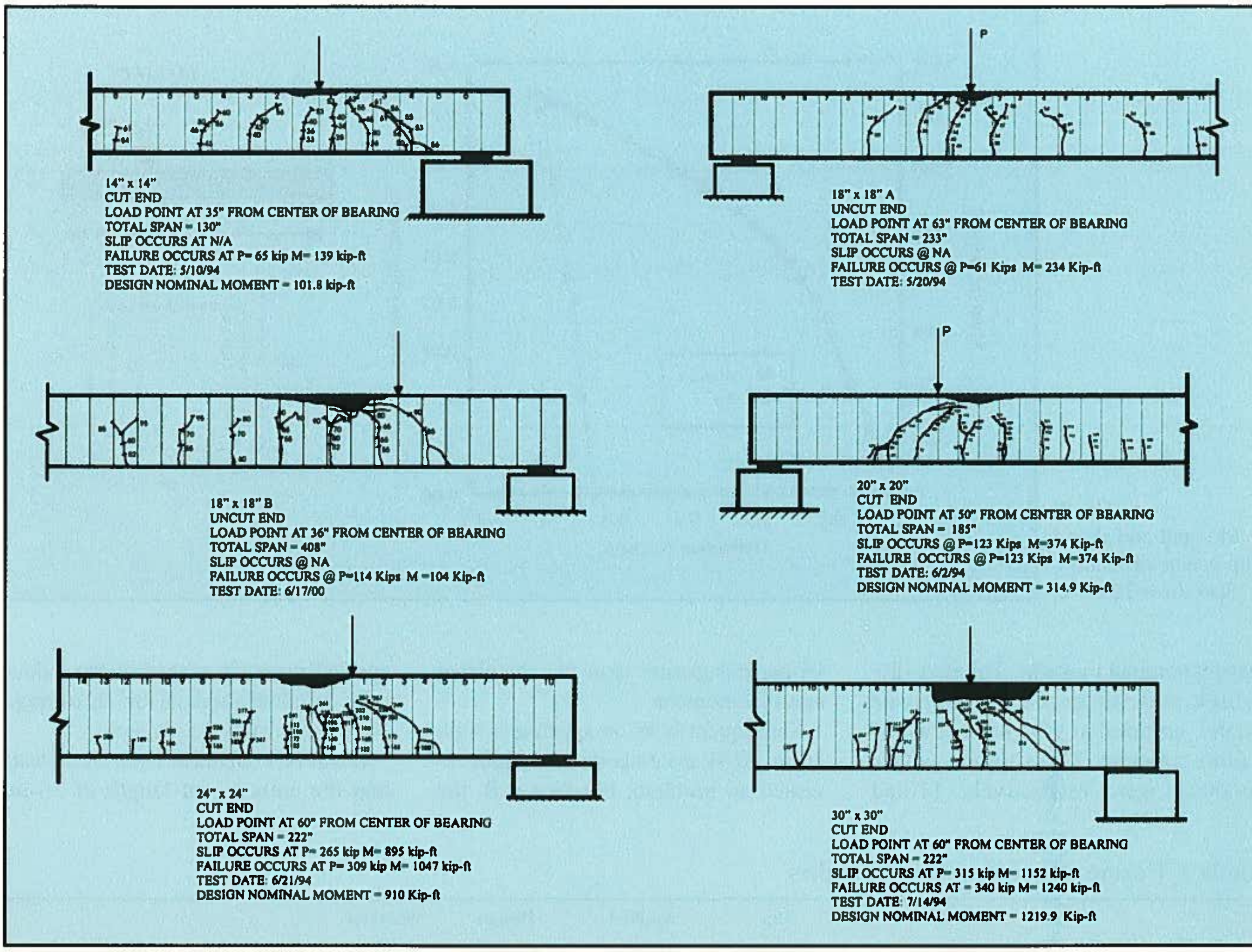


Fig. 8. Pile test crack charts.

setup. The test specimens were instrumented to continuously monitor deflections, strains, strand slippage, loading, and cracking up to failure. Crack patterns were recorded for all specimens.

The test results are presented in Columns 6 and 7 of Table 2. Columns 8 to 10 give the values of calculated nominal moment, M_n, for each section as well as comparisons to measured values.

Fig. 7 shows typical plots for the moment-deflection behavior, and Fig. 8 shows the typical crack patterns. Continuous monitoring of the strand slip gauges enabled the investigators to determine accurately the applied moment at initial strand slippage and the order in which strand slippage occurred.

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 and depth effects. Table 2 presents the data gathered for the testing program.

14 x 14 in. (356 x 356 mm) Specimens [$^1/_2$ in. (13 mm) Diameter Strand]

Six tests were conducted, with four different values of embedment length varying from 55 to 34 in. (1.40 to 0.86 m). No strand slippage was observed in any of these tests, and the values of the maximum applied moment, M_{app}, were greater than the M_n values in all tests.

The failures were mainly flexural, with crushing of the concrete occurring before the reinforcement yielded (see Fig. 8). Embedment length as low as 34 in. (0.86 m) (68*D*) was adequate to ensure that no strand slippage occurred at failure (i.e., $M_{app} > M_n$). This value is approximately 50 percent of that calculated by the AASHTO equation [70 in. (1.78 m)].

18 x 18 in. (457 x 457 mm) Specimens (Groups A and B)

Group A consisted of six specimens containing 0.6 in. (15 mm) diameter strands. The embedment length varied from 69 to 51 in. (1.75 to 1.30 m) (115*D* to 85*D*), with three different values of embedment. Strand slippage was observed in specimens with embedment length less than 69 in. (1.75 m). It can be concluded that an embedment length of 69 in. (1.75 m) (115*D*) was adequate to ensure that no strand slippage occurred at failure. This value is approximately 78 percent of the calculated AASHTO value of 88.6 in. (2.25 m), indicating the sufficiency of the AASHTO equation.

Group B consisted of 14 specimens containing $^1/_2$ in. (13 mm) diameter strands. Embedment length varied from 60 to 36 in. (1.52 to 0.91 m) (120*D* to 72*D*), with five different values of embedment length. Strand slippage was observed in only one test at an applied moment 11 percent greater than the

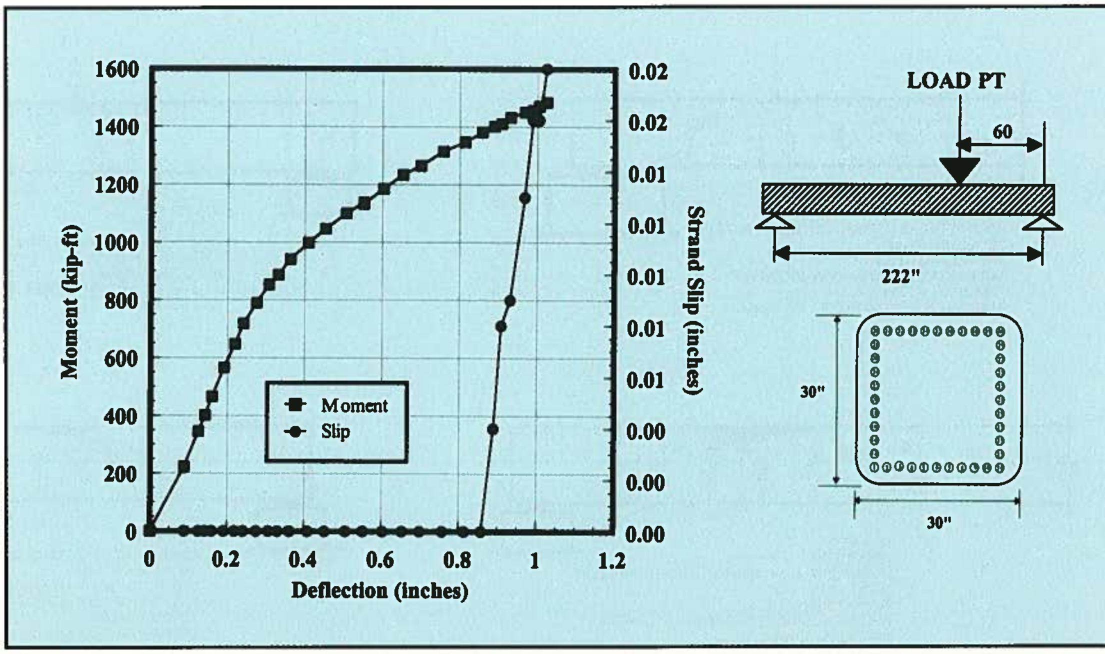


Fig. 9. Moment and strand slip versus deflection, Specimen 30-60-2.

design nominal moment. The first slip, which was observed in only one strand, occurred at 95.4 percent of the failure moment. The slip and failure moments were, respectively, 11 and 16 percent greater than the calculated nominal moment.

Subsequent tests on specimens with 36 in. (0.91 m) embedment length revealed no problem. For Group B, the applied moment exceeded the calculated nominal moment by an average of approximately 13 percent.

It can be concluded from these tests that the embedment length of 36 in.

Table 3. Flexure results for AASHTO girders.

Girder number (1)	End (2)	Span L (in.) (3)	Shear span a (in.) (4)	a/d (5)	Slip moment M_{slip} (kip-ft) (6)	Applied moment M_{app} (kip-ft) (7)	Design moment M_u (kip-ft) (8)	Nominal moment M_n (kip-ft) (9)	$\frac{M_{app}}{M_u}$ (10)	$\frac{M_{slip}}{M_u}$ (11)	$\frac{M_{app}}{M_n}$ (12)	$\frac{M_{slip}}{M_n}$ (13)
A0-00-R	N	480	85	2.1	1535	2237	1083	2058	2.07	1.42	1.09	0.75
A0-00-R	S	324	85	2.1	1403	1975	1083	2058	1.82	1.29	0.96	0.68
A0-00-RD	N	480	74	1.8	1289	1433	968	2058	1.48	1.33	0.70	0.63
A0-00-RD	S	306	85	2.1	1471	1634	1083	2058	1.67	1.36	0.89	0.71
A1-00-R	N	480	102	2.5	1250	1809	1236	2058	1.46	1.01	0.88	0.61
A1-00-R	S	378	124	3.1	1500	2191	1376	2058	1.59	1.09	1.07	0.73
A1-00-RD	N	480	102	2.5	1322	1551	1236	2058	1.26	1.07	0.75	0.64
A1-00-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
B0-00-R	N	480	102	2.5	1600	1899	1236	2142	1.54	1.29	0.89	0.75
B0-00-R	S	378	124	3.1	2175	2175	1376	2142	1.58	1.58	1.01	1.02
B1-00-R	N	240	60	1.5	1150	1235	822	1883	1.50	1.40	0.66	0.61
B1-00-R	S	222	54	1.3	975	1052	759	1792	1.39	1.28	0.59	0.54
C0-00-R	N	336	142	3.5	2139	2139	1470	2036	1.45	1.45	1.05	1.05
C0-00-R	S	480	132	3.2	2021	2021	1431	2036	1.41	1.41	0.99	0.99
C0-00-RD	N	264	60	1.5	815	957	833	1673	1.15	0.98	0.57	0.40
C0-00-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
C1-00-RD	S	378	149	3.7	1949	1949	1520	2036	1.28	1.28	0.96	0.96

Note: 1 in. = 25. 4 mm; 1 kip-ft = 1.356 kN-m.

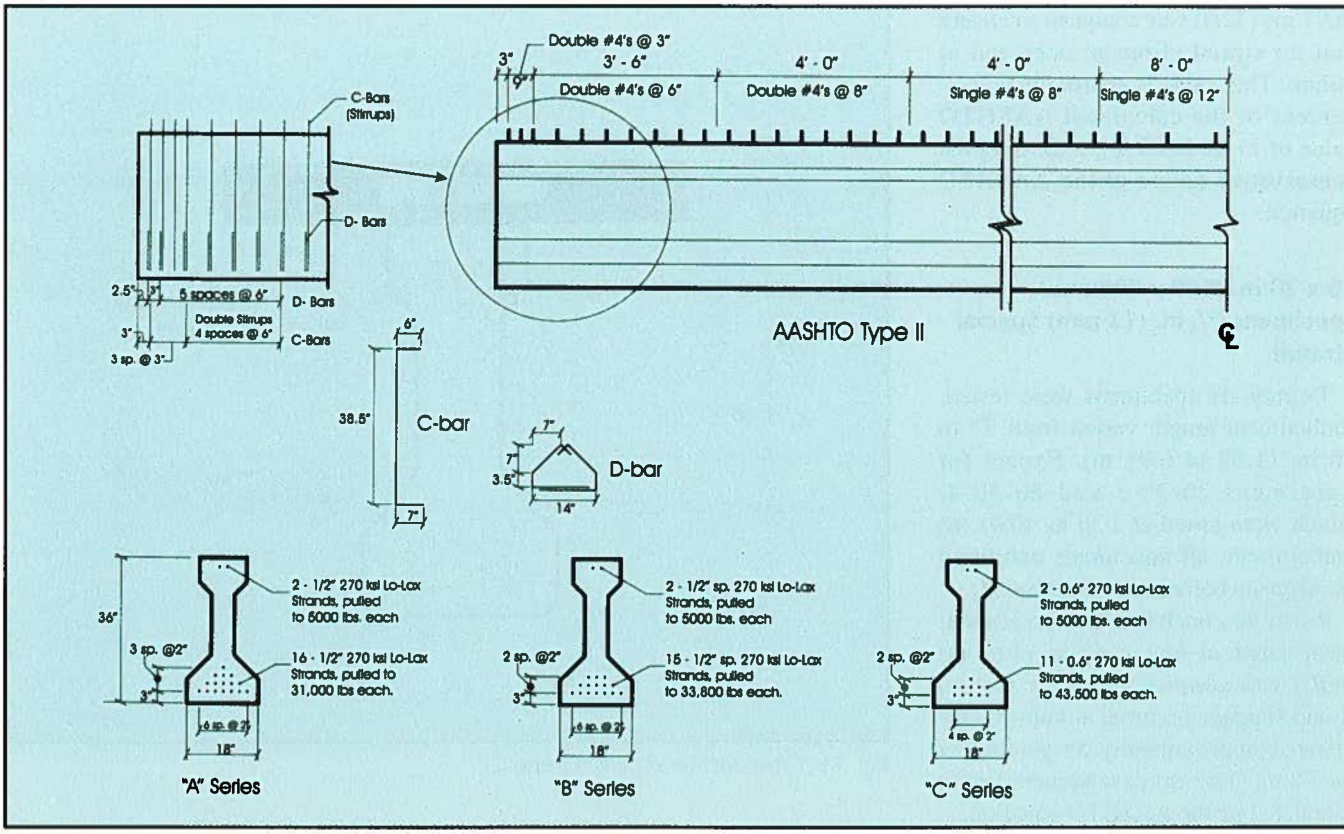


Fig. 10. Reinforcement details of test girders.

Table 4. Shear test results for AASHTO girders.

Girder number (1)	End (2)	Span L (in.) (3)	Shear span a (in.) (4)	a/d (5)	Shear at first slip V_{slip} (kips) (6)	Applied shear V_a (kips) (7)	Nominal shear V_n (kips) (8)	$\frac{V_a}{V_n}$ (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

Note: 1 in. = 25.4 mm; 1 kip = 4.44 kN.

(0.91 m) (72*D*) was adequate to ensure that no strand slippage occurred at failure. This value is approximately 50 percent of the calculated AASHTO value of 71 in. (1.80 m), indicating the conservative nature of the AASHTO equation.

20 x 20 in. (508 x 508 mm) Specimens [$^1/_2$ in. (13 mm) Special Strand]

Twenty-six specimens were tested. Embedment length varied from 72 to 36 in. (1.83 to 0.91 m). Except for Specimens 20-30-3 and 20-30-4, which were tested at a 36 in. (0.91 m) embedment, all specimens exhibited no slippage before attainment of M_n.

It can be concluded that an embedment length as low as 42 in. (1.07 m) (80*D*) was adequate to ensure that no strand slippage occurred at failure. This value is approximately 58 percent of the 72 in. (1.83 m) development length calculated by the AASHTO equation.

24 x 24 in. (610 x 610 mm) Specimens (Groups A and B)

Group A consisted of eight specimens with $^1/_2$ in. (13 mm) special diameter strands. Embedment length varied from 90 to 56 in. (2.29 to 1.42 m). Only two specimens, 24-48-1 and 24-48-2, tested at 56 in. (1.42 m) embedment, exhibited strand slippage before reaching their nominal moment, M_n. The failure mode for both specimens was that of shear/bond, whereas the failure modes for all other specimens were mainly flexure-compression.

It can be concluded that an embedment length as low as 66 in. (1.68 m) (125*D*) was adequate to ensure that no strand slippage occurred at failure. This value is approximately 96 percent of the 68.33 in. (1.736 m) development length calculated by the AASHTO equation.

Group B consisted of 15 specimens containing $^1/_2$ in. (13 mm) diameter strands. Embedment length varied from 72 to 48 in. (1.83 to 1.22 m). Strand slippage was observed in specimens with an embedment length less than 60 in. (1.52 m). These specimens failed at a moment higher than the nominal design moment.

It can be concluded from these tests

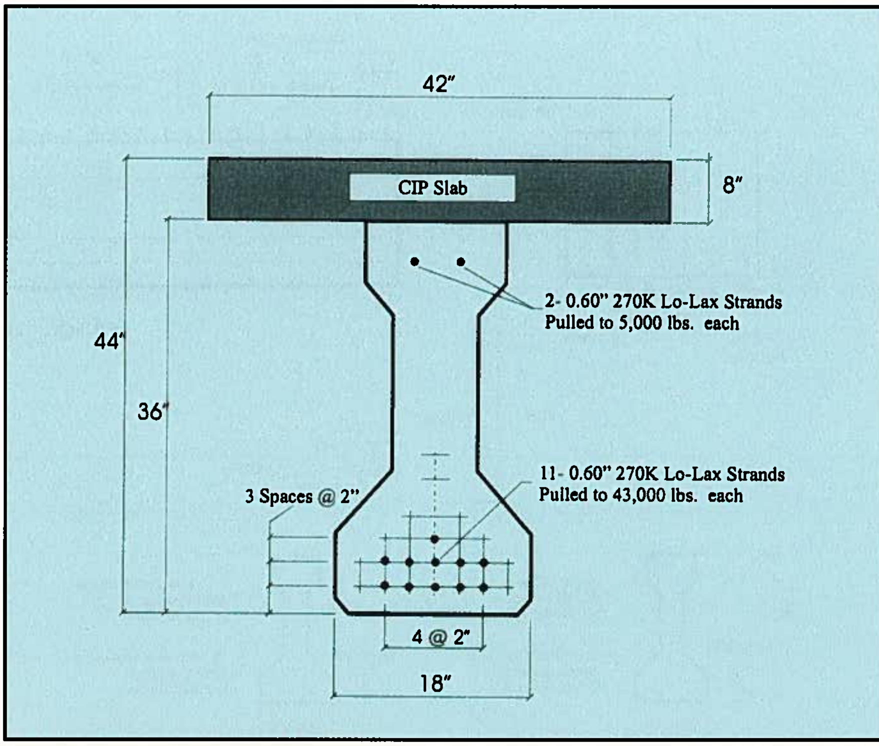


Fig. 11. Cross section details (Group C).

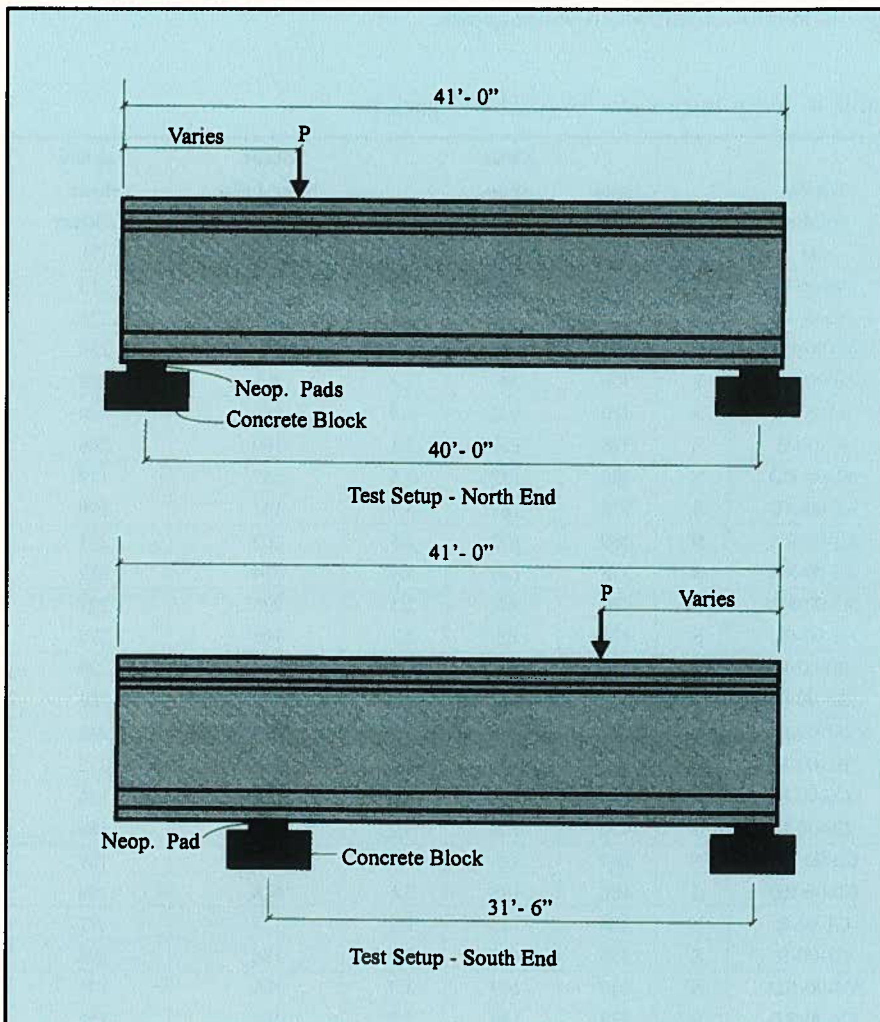


Fig. 12. Girder test setup.

that an embedment length of 60 in. (1.52 m) (120*D*) was adequate to ensure that no strand slippage occurred at failure. This value is approximately 93 percent of the calculated AASHTO value of 64.5 in. (1.64 m), indicating the conservative nature of the AASHTO equation.

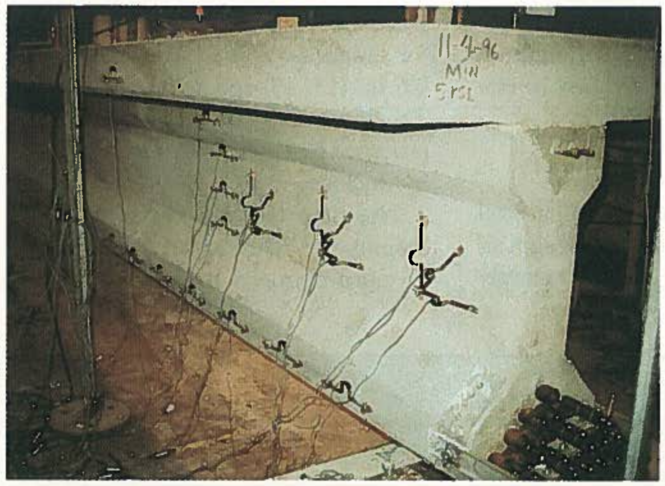
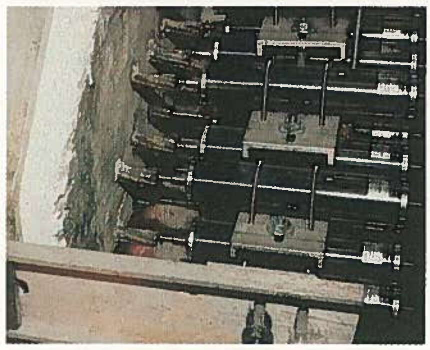
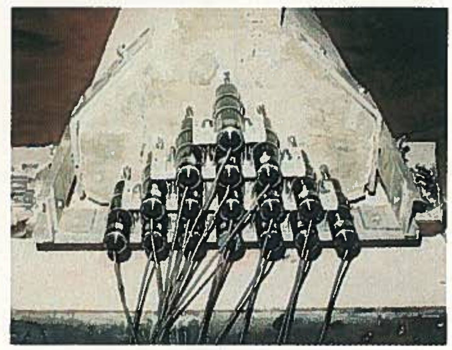


Fig. 13. Slip measurement.





30 x 30 in. (762 x 762 mm) Specimens [$^1/_2$ in. (13 mm) Strand]

Eight specimens were tested. Embedment length varied from 111 to 66 in. (2.82 to 1.68 m) (220*D* to 132*D*). Except for the specimens with lower *a/h* ratios [embedment length = 66 in. (1.68 m)], most of the piles failed at a higher moment than the calculated nominal value. However, strand slippage was observed in some specimens at approximately 93 percent of the nominal moment.

Fig. 9 shows the moment-deflection-slip relationship for Specimen 30-60-2. Failure modes were mainly flexure-compression, with the specimens having a 66 in. (1.68 m) embedment failing in a shear/bond mode.

The calculated AASHTO development length for this case is 73 in. (1.85 m). It is apparent from the results that this value is insufficient to develop the nominal capacity without strand slippage. The results suggest that a value of approximately 96 in. (2.44 m), which is 33 percent higher than the AASHTO value, is more appropriate.

Prestressed AASHTO Girders

The FSRC has conducted a wide-ranging study[6,7] of the behavior of AASHTO Type II girders with respect to transfer and development length of strands, flexure, shear, and fatigue be-

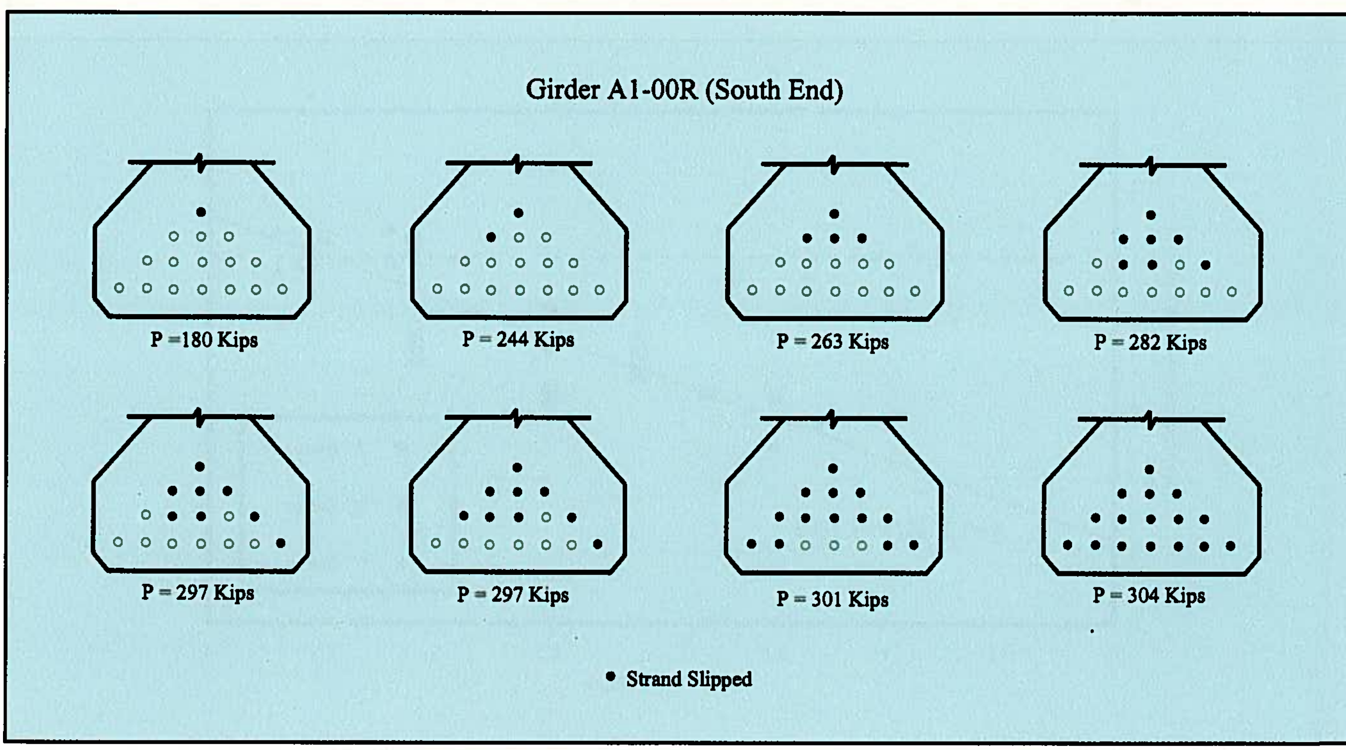


Fig. 14. Strand slip sequence.

havior. Only those tests applicable to transfer and development length of prestressing strand are covered in this section.

Transfer Length

As stated earlier, the use of f_{si} instead of f_{se} for calculating transfer length of strand in the first term of Eq. (2) closely predicted the transfer lengths observed for strands of $^1/_2$ in., $^1/_2$ in. special, and 0.6 in. (13, 13, and 15 mm) diameter. This resulted in Eq. (6), which was first proposed by FSRC, and later confirmed by Buckner.

The average measured transfer lengths for $^1/_2$ in. and 0.6 in. (13 and 15 mm) diameter unshielded strands were observed to be 30 and 34 in. (762 and 864 mm) (60*D* and 57*D*), respectively. In view of this finding, Eq. (6) will be adopted for calculating the transfer length of strand, and the topic of transfer length will not be discussed further in this paper.

Note that the FHWA study[4] indicated that the concrete strength has an

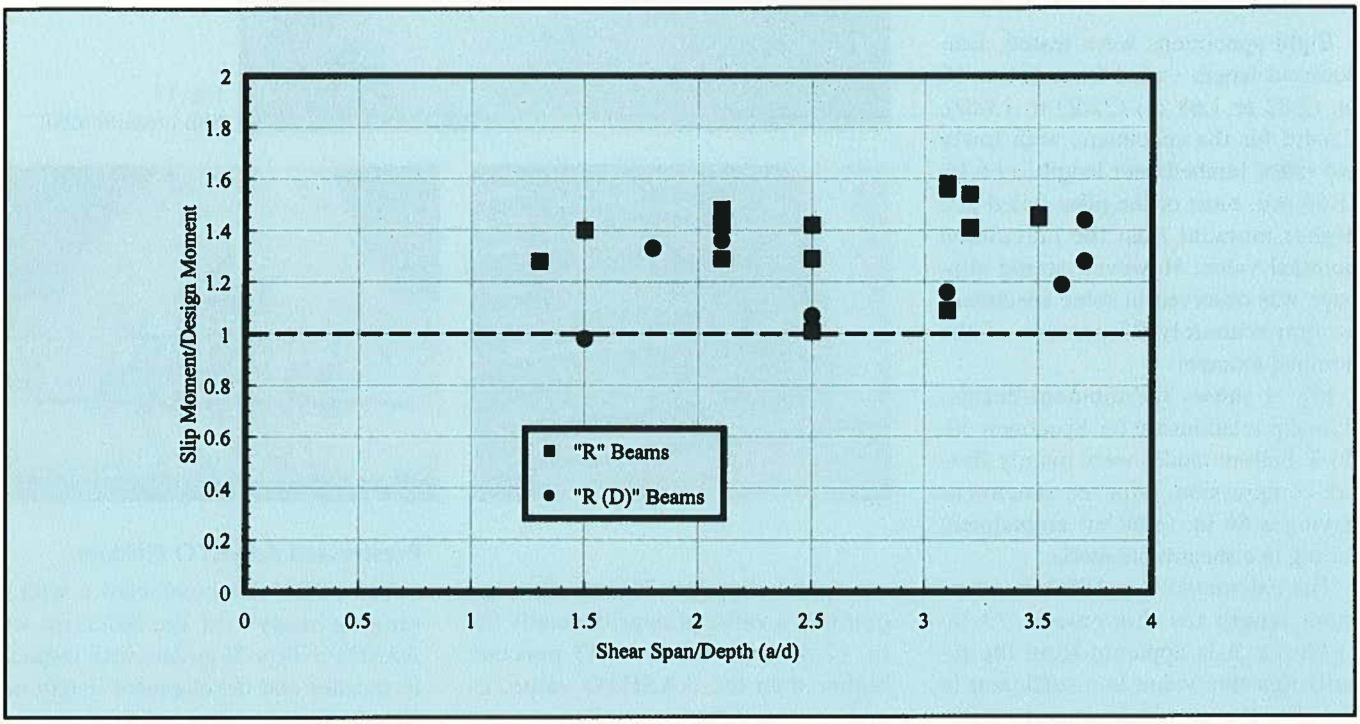


Fig. 15. Effect of shear span-to-depth ratio on strand slip.

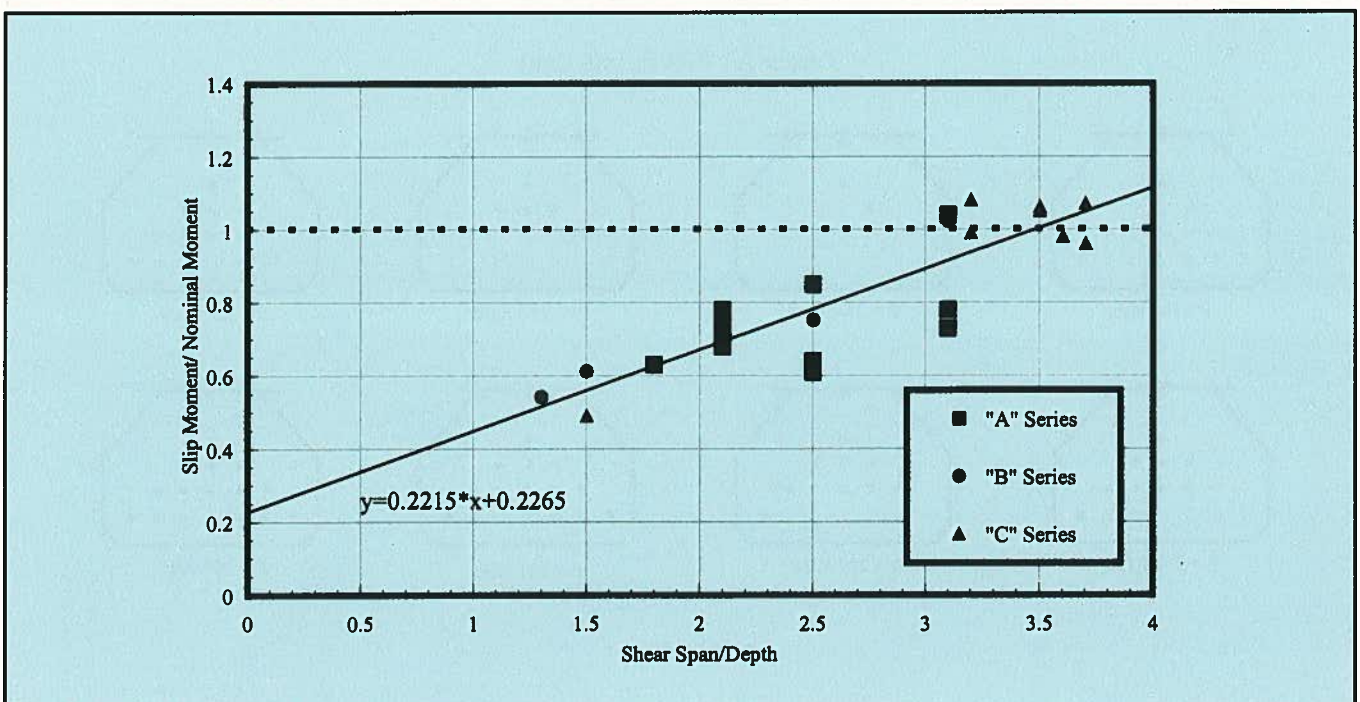


Fig. 16. Relationship between slip moment and shear span.

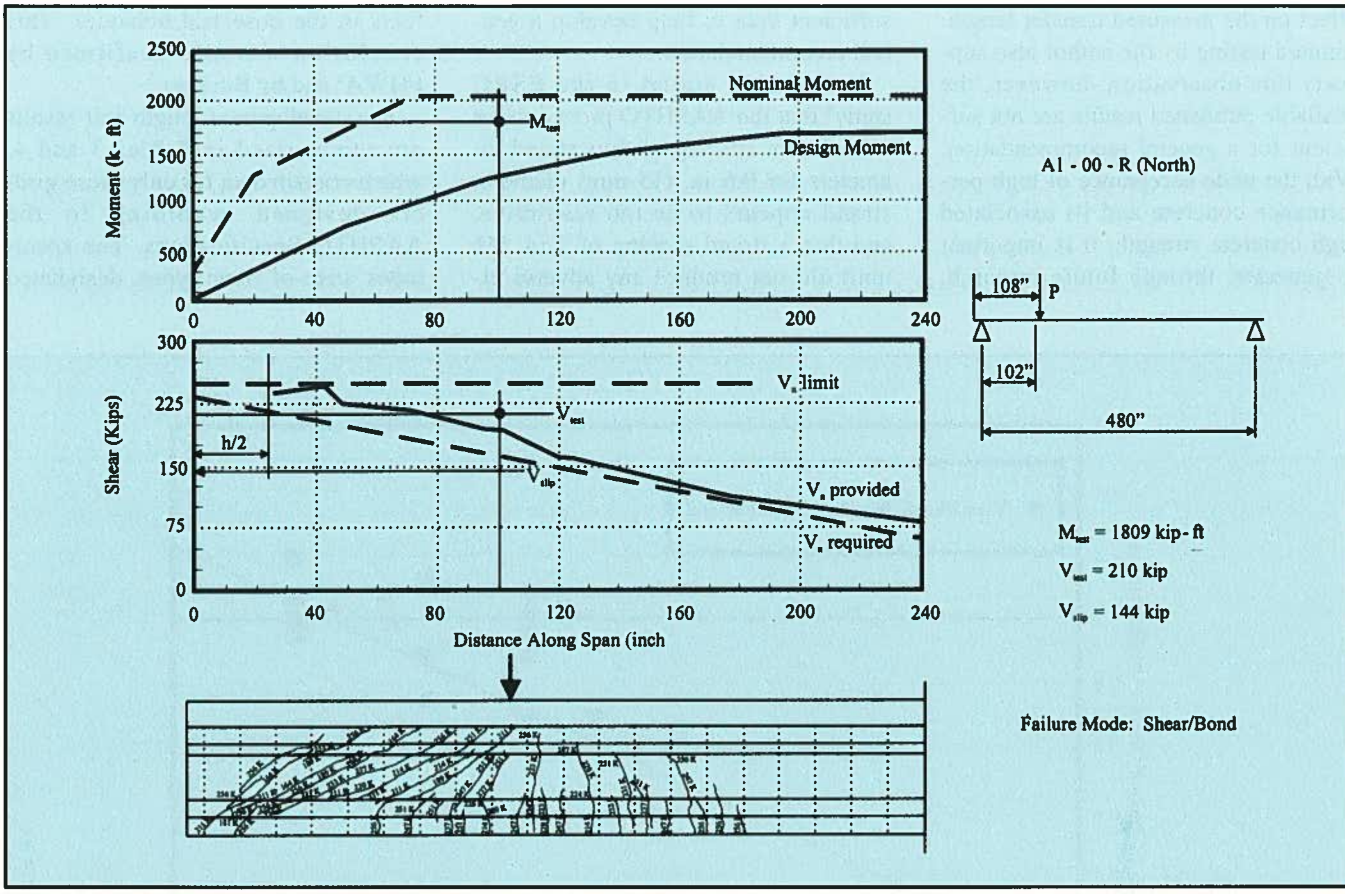


Fig. 17. Response of Girder A1-00-R(N).

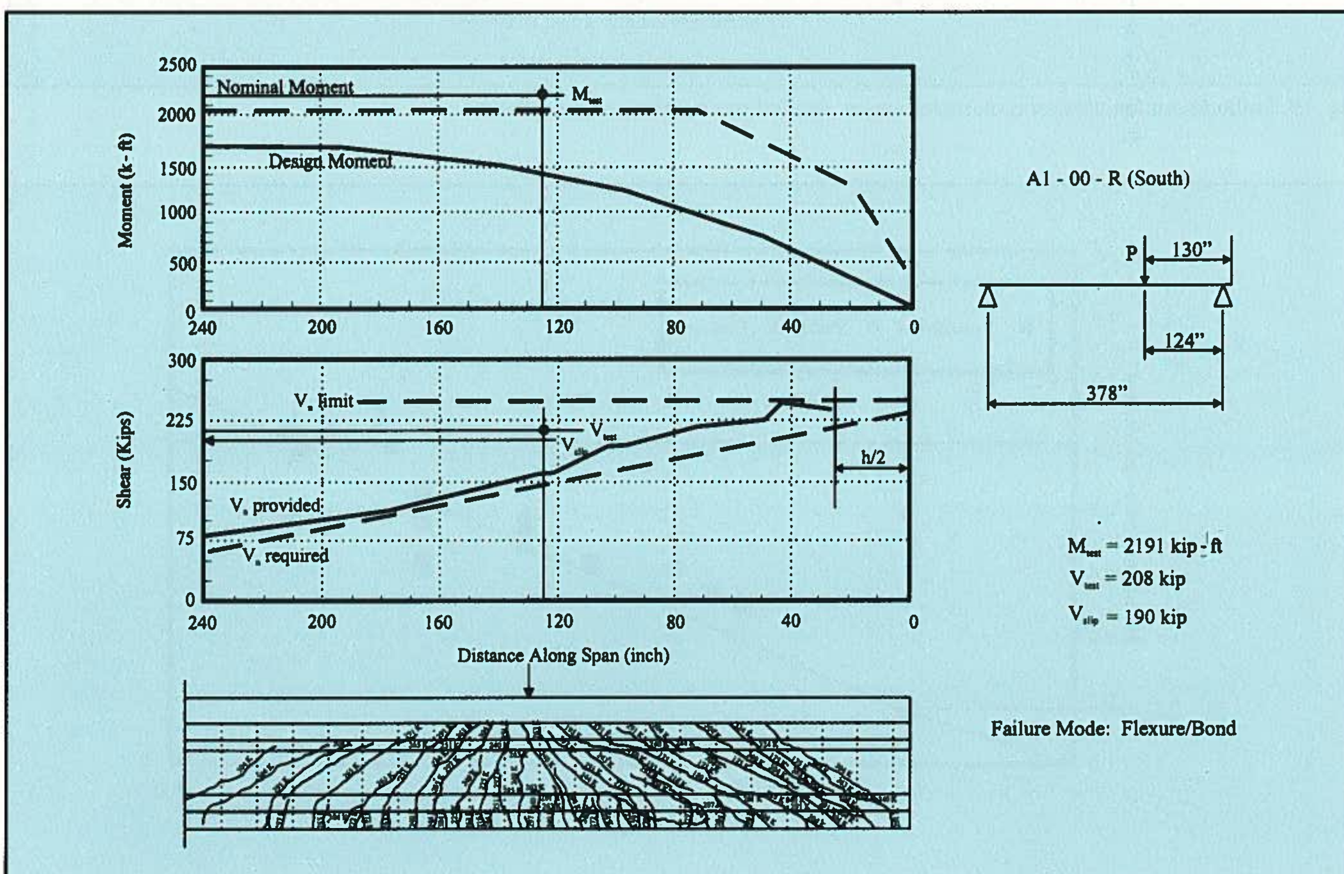


Fig. 18. Response of Girder A1-00-R(S).

effect on the measured transfer length. Limited testing by the author also supports this observation; however, the available published results are not sufficient for a general recommendation. With the wide acceptance of high performance concrete and its associated high concrete strength, it is important to generate, through future research, sufficient data to help develop a general recommendation.

It was also stated in the FSRC study[6] that the AASHTO provision for a minimum spacing of four strand diameters for 0.6 in. (15 mm) diameter strand appears to be too restrictive, and that a strand spacing of 2 in. (51 mm) did not produce any adverse effects in the observed behavior. This conclusion was also confirmed by FHWA[4] and by Buckner.[3]

The development length test results are summarized in Tables 3 and 4, which contain data for only those girders designed according to the AASHTO Specifications. The specimens were of three types, designated

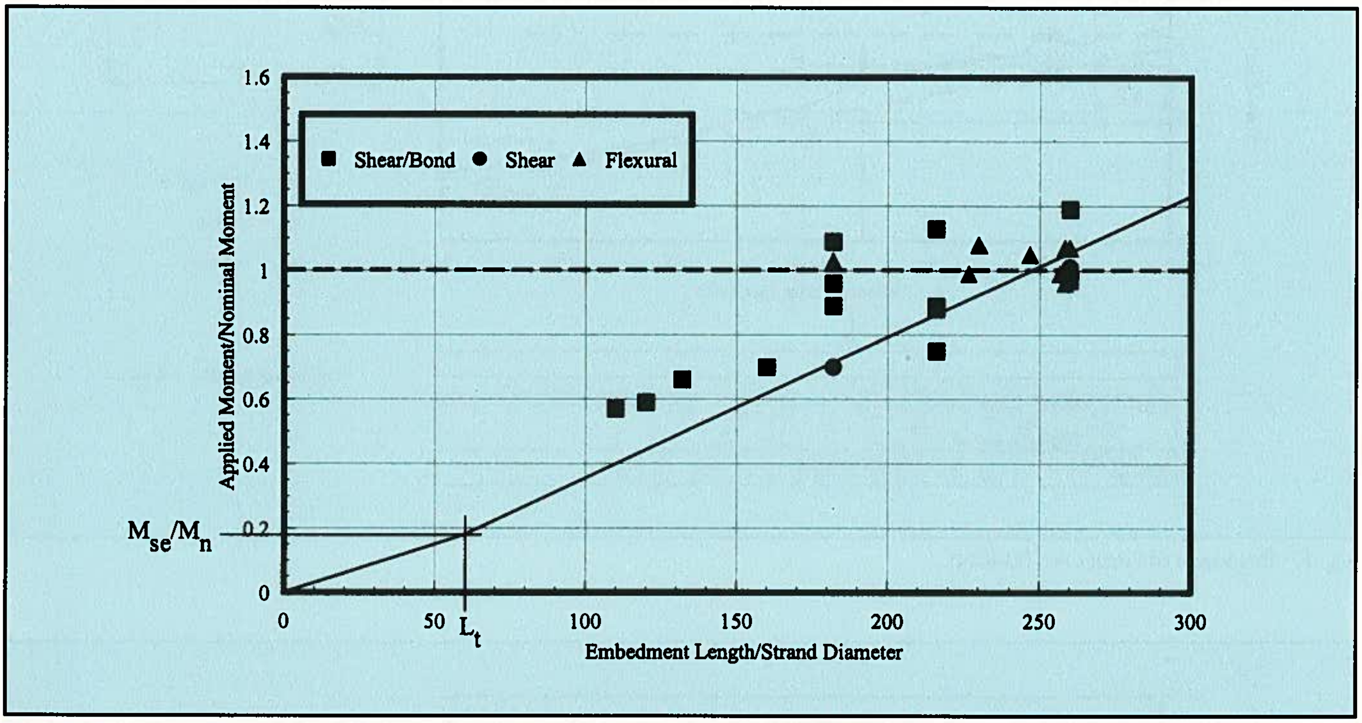


Fig. 19. Embedment length/strand diameter versus applied moment/nominal moment.

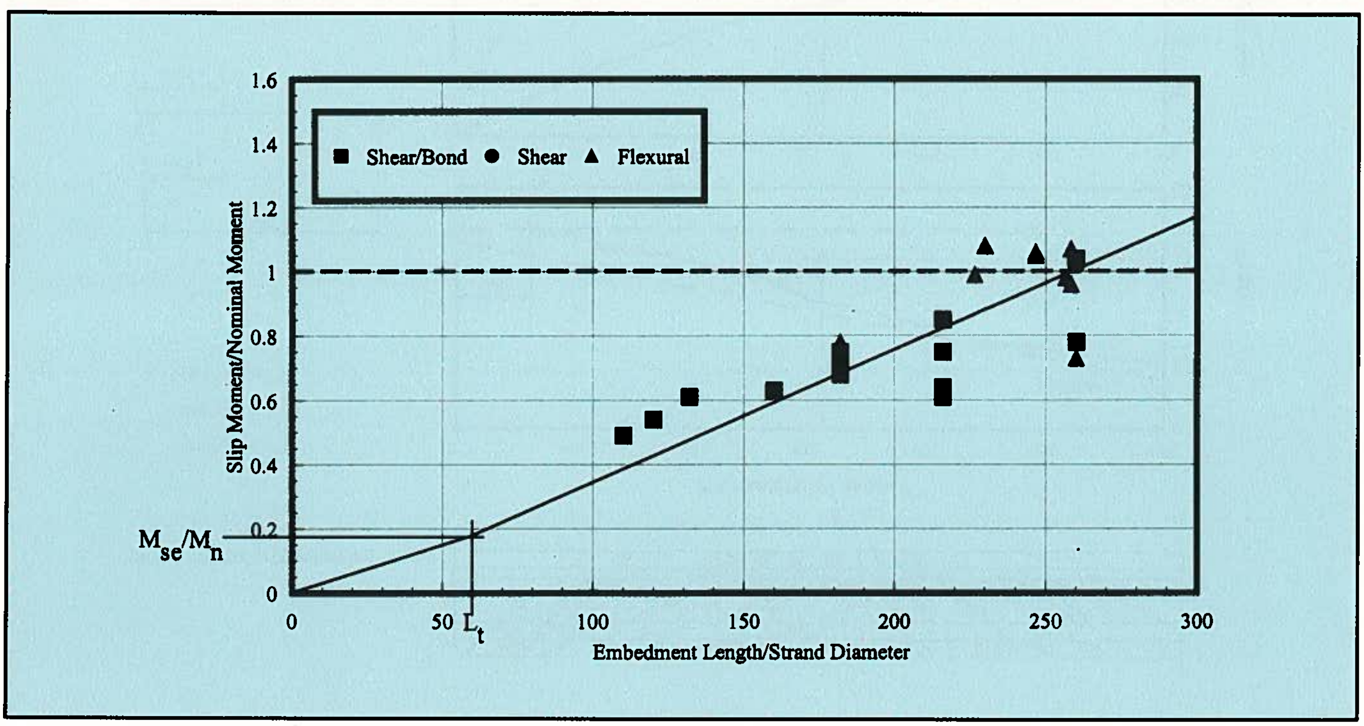


Fig. 20. Embedment length/strand diameter versus slip moment/nominal moment.

A, B, and C, which, respectively, incorporated $^1/_2$ in., $^1/_2$ in. special, and 0.6 in. (13, 13, and 15 mm) low-relaxation Grade 270 strands, as shown in Fig. 10. 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 A0-00-RA is interpreted as follows:

A: Strand size of $^1/_2$ in. (13 mm). Note that "B" represents $^1/_2$ in. special strand, and "C" represents 0.6 in. (15 mm) strand.

00: Zero shielding (all girders in this part of the study were unshielded).

R: Shear reinforcement provided in accordance with the AASHTO Specifications. The term RD indicates that shear reinforcement was provided in accordance with the AASHTO Specifications, but that confinement reinforcement was omitted in the tension flange. Note that "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) Nominal strand diameter: $^1/_2$ in., $^1/_2$ in. special, and 0.6 in. (13, 13, and 15 mm).

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

(c) Confinement reinforcement to strands in tension flange (D-bar shown in Fig. 10).

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

There was no intentional variation of other parameters. In order to check the strict FHWA requirements for the spacing of strands, the spacing was kept constant at 2 in. (51 mm) on center regardless of strand size (see Fig. 10).

Florida Wire and Cable supplied all the prestressing strands, and Durastress in Leesburg, Florida produced the precast portions of the girders. After transporting the girders to the FDOT Structural Research Center, a top flange, 42 in. wide and 8 in. thick (1.07 m x 203 mm), was cast on all specimens. Except for variation of prestressing steel in the bottom flange (see Fig. 10), the cross sections of all specimens conformed to that shown in Fig. 11.

The concrete for the precast sections and the cast-in-place top slab was designed for a 28-day compressive strength, f'_c, of 5000 psi (35 MPa). The design compressive strength at transfer, f'_{ci}, was 4000 psi (28 MPa).

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 Fig. 12. To eliminate the failed zone from the test span, the test span was shortened after one end of the girder was tested.

Linear voltage differential transducers (LVDTs) were used continuously to monitor slippage at the ends of all strands during a test (see Fig. 13). Strains and deflections were also monitored. The detailed behavior of the test specimens is described in References 6 and 7. Typical observations related to the 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 at which initial strand slippage occurred, as determined by readings from the LVDTs. For example, Fig. 14 shows the strand slip sequence, with increasing load, for Girder A1-00-R (south end).

The initial slippage occurred at, or shortly after, appearance of the first shear crack. The beam then continued to carry increased loading. As the load

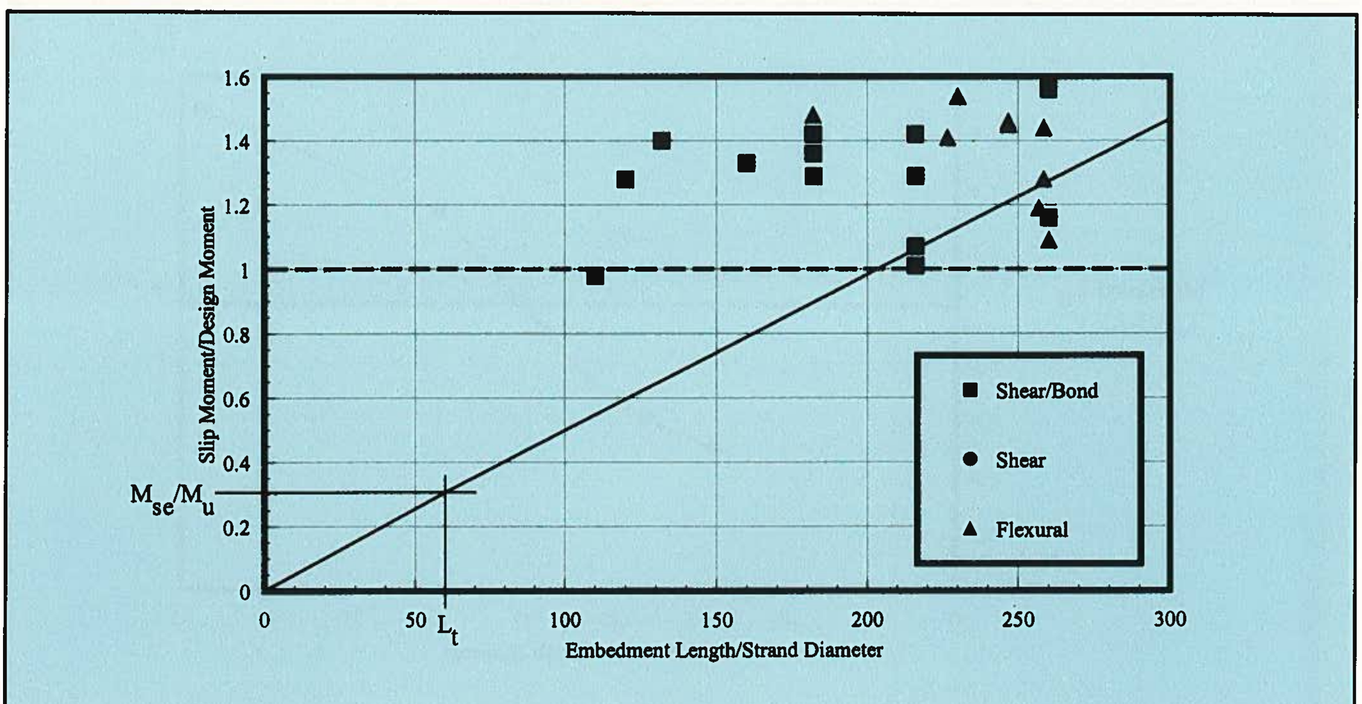


Fig. 21. Embedment length/strand diameter versus slip moment/design moment.

was increased, new cracks developed and additional strands started to slip. The ability of the girder to carry more load continued until complete bond slip of all strands occurred. This type of bond slip mechanism was typical for all the girders.

Fig. 15 shows a plot of test results, with the ratio of slip moment to design moment plotted against the shear span-to-depth ratio. From Fig. 15 and Table 3, it is seen that except for Test C0-00-RD(N), which had no confinement reinforcement, the applied moment at initial slippage, M_{slip} (Column 6), was greater than the design moment, M_u (Column 8), even though in most cases, the value of M_{slip} was less than the nominal moment, M_n (Column 13).

The crack development in the girders together with the sequence of strand slippage gave evidence that there was an interaction between the shear and bond of the strands. Fig. 16 presents a plot of the ratio M_{slip}/M_n versus a/d for all the girders, which indicates that at an a/d value of approximately 3.5, the value of M_{slip}/M_n is close to 1.0. The results for Girders A3-00-RA and A3-00-RB are omitted from Fig. 16, because 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 Figs. 17 and 18 for Tests A1-00-R(N) and A1-00-R(S), respectively. These plots, together with Tables 3 and 4, show that the test moments and shears exceeded the design values for both girders.

In both cases, strand slippage occurred and the failure moment for Test A1-00R(N) was 88 percent of the calculated nominal moment. This observation 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.

Figs. 19, 20, and 21 present plots of different moment ratios against embedment length/strand diameter for all the test results listed in Table 3. In general, the trends discussed herein for all strands combined are typical for all girders.

The plot for M_{app}/M_n in Fig. 19 shows that the nominal moment of a girder is fully developed when the embedment length is about 230*D*. Below this value, the types of failure are predominantly shear or shear/bond. The plot in Fig. 20 shows that for an embedment of about 230*D*, the moment at slippage is at least equal to the nominal moment value. The two exceptions are Girders A0-00-RD(N) and A1-00-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 these two girders exhibited low flexural strength 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. One possible explanation is that the surface of the prestressing strands could have been tainted by grease or other contaminant. Similar observations were made in the FHWA study.

In Fig. 21, the plot 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 Girder C0-00-RD(N), which did not contain confinement reinforcement. This was also found to be generally the case for the values of shear force at failure. Table 3, Column 10, also reveals that in all tests, the failure moment, M_{app}, exceeded the design moment value at the test section.

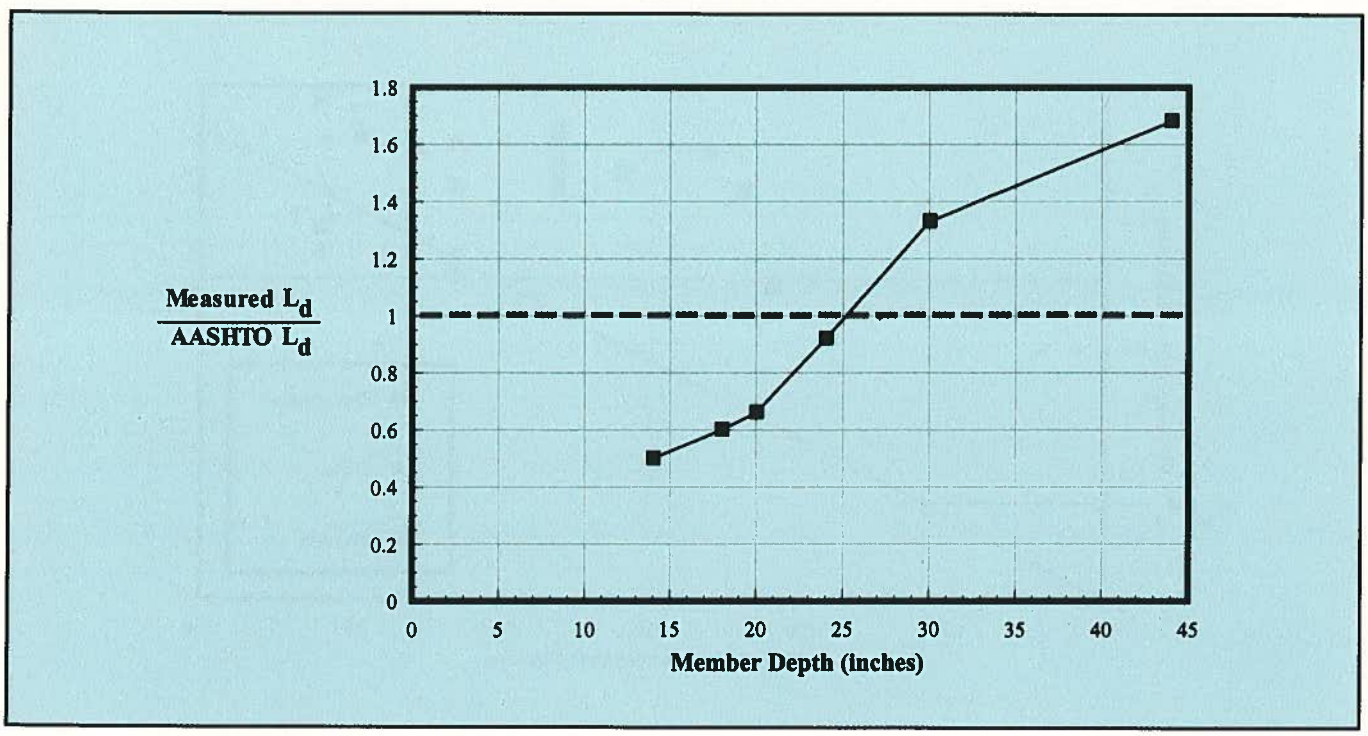


Fig. 22. Effect of member depth on development length.

These trends support the previous observation regarding the current philosophy on development length requirements in the end regions of a pretensioned girder. One of the most important observations from Figs. 19 to 21 is that a development length of approximately 230*D* should ensure that full nominal moment could be developed in accordance with current Code requirements. This contrasts with a value of approximately 154*D* based on current AASHTO provisions.

The test results indicate that the AASHTO provisions are unconservative, whereas the use of the 1.6 multiplier yields reasonable, but slightly unconservative, comparisons with measured values.

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 by comparing the strength ratios in Columns 10 to 12 for the specimens without confinement reinforcement (denoted by "D") with the other corresponding specimens.

SHEAR-BOND INTERACTION

The above results for slabs, piles, and girders strongly suggest that a direct interaction exists between shear and bond. This relationship is clearly seen in Fig. 22, which shows the effect of member depth on the measured development length for all tested piles and girders. It can be seen that for depths greater than 24 in. (610 mm), the AASHTO development length equation is unconservative.

The AASHTO equation, Eq. (1), depends on the level of effective and ultimate strand stress for a given strand size. In practical applications, these two parameters vary over a narrow range, resulting in a somewhat constant development length for a given strand diameter. In the case of the girders, the development length value from Eq. (1) is approximately 80 in. (2.03 m) for $^1/_2$ in. (13 mm), which is less than twice the depth of the girder.

The requirement for full strand development at a fixed distance from the ends without any consideration of the member geometry is questionable. In a relatively deep beam, at a distance less than twice the member depth, it is unrealistic to expect a flexural failure and complete strand development. The dominant mode of failure in regions close to the ends of such members is that of shear and not flexure.

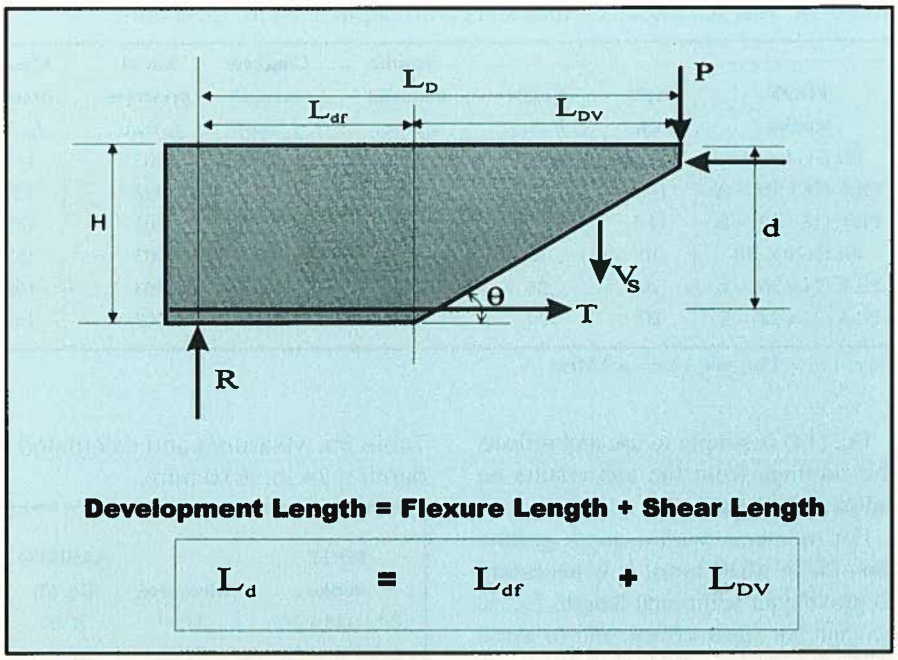


Fig. 23. Development length definition.

A logical conclusion from the above is that, for members deeper than 24 in. (610 mm), any development length requirements should reflect the interaction between shear and flexure at end regions. One can, therefore, conclude that the existing equations in their current forms are not applicable to bridge members with a depth exceeding 24 in. (610 mm).

The Shahawy 93 equation was an early attempt to consider some of these observations; however, it was purely empirical. In light of the extensive testing conducted since then, as reported in this paper, it is possible to develop a rational approach for predicting development length, taking into account the shear effects and end conditions. The following is a new rational proposal, which has been developed based on all the above observations.

New General Proposal

It has been shown that in regions near a support, the effects of shear on development length cannot be neglected in members deeper than 24 in. (610 mm). In such a case, interaction exists between shear and flexure, and slippage of strand is likely to occur before the flexural strength of the member can be developed. For members with depths of 24 in. (610 mm) or less, the existing AASHTO equation is conservative due to the assumed value of 250 psi (1.72 MPa) for bond strength. The general method proposed here is designed to take into account the effects of bond strength and shear on the prediction for development length.

For embedded piles and flexural members with a depth equal to or less than 24 in. (610 mm), it is proposed that the current AASHTO equation [Eq. (2)] be slightly modified to reflect the increased bond strength and the use of f_{si} instead of f_{se} in calculating the transfer length. The general form of the proposed equation is as follows:

$$L_{df} = \left(\frac{f_{si}}{3}\right)D + \frac{\left(f^*_{su} - f_{se}\right)D}{4u_{ave}} \quad (13)$$

where u_{ave} is an average value of bond stress of 0.300 ksi (2.07 MPa). Therefore, the above equation can be rewritten as:

$$L_{df} = \left(\frac{f_{si}}{3}\right)D + \frac{\left(f^*_{su} - f_{se}\right)D}{1.2} \quad (14)$$

Table 5a. Test parameters – members with depth ≤ 24 in. (610 mm).

FDOT studies	Depth d (in.)	Height h (in.)	Strand diameter d_b (in.)	Concrete strength f'_c (ksi)	Initial prestress f_{si} (ksi)	Effective prestress f_{se} (ksi)	Ultimate stress f_{su} (ksi)	Ultimate strain γ_{ps}	λ
PILE (14 x 14)	10.5	14	0.5	6.0	203	157.6	245.1	0.009775	1.0
PILE (18 x 18) – A	14.5	18	0.6	6.0	203	157.6	252.7	0.011402	1.056
PILE (18 x 18) – B	14.5	18	0.5	7.0	203	157.6	247	0.010071	1.0
PILE (20 x 20)	16	20	0.5	6.0	203	157.6	248.8	0.010406	1.017
PILE (24 x 24) – A	20	24	0.5	7.5	203	157.6	234	0.008706	1.0
PILE (24 x 24) – B	20	24	0.53	7.0	203	157.6	234	0.008706	1.0

Note: 1 in. = 25.4 mm; 1 ksi = 6.9 MPa.

Eq. (14) is simple to use and reflects the findings from the test results on full-scale specimens.

For members with a depth greater than 24 in. (610 mm), it is necessary to provide an additional length, L_{dv}, to account for shear effects and to avoid tendon slippage before failure, as shown in Fig. 23. The general form of the proposed equation is as follows:

$$L_d = L_{df} + L_{dv} \quad (15)$$

The quantity L_{dv} is defined as:

$$L_{dv} = d \cot\theta \quad (16)$$

where θ is the shear angle at failure.

For simplicity, the minimum value of θ for shear failure is assumed to be 30 degrees, and assuming that $d = 0.85h$, the following equation results:

$$L_{df} = \left(\frac{f_{si}}{3}\right)D + \frac{\left(f^*_{su} - f_{se}\right)D}{1.2} + 1.47h \quad (17)$$

This expression is extremely simple and in line with the current AASHTO equation. It also yields accurate results, as will be shown in the following section.

ANALYTICAL EVALUATION

The results from the following studies are used in the evaluation of the different proposals for calculating the development length:

(a) AASHTO Eq. (2)
(b) 1.6 AASHTO
(c) Buckner Eq. (8)
(d) FHWA Eq. (12)
(e) Proposed simplified approach Eq. (14) and Eq. (17)

Table 5b. Measured and calculated development lengths – members with depth ≤ 24 in. (610 mm).

FDOT studies	Measured	AASHTO Eq. (2)	1.6 AASHTO	Buckner Eq. (8)	FHWA Eq. (12)	New proposal Eq. (14)
PILE (14 x 14)	34.00	70.02	112.03	77.58	124.33	70.29
PILE (18 x 18) – A	69.00	88.58	141.73	100.86	152.06	88.15
PILE (18 x 18) – B	42.00	70.97	113.55	78.66	108.87	71.08
PILE (20 x 20)	42.00	71.87	114.99	80.17	126.31	71.83
PILE (24 x 24) – A	66.00	64.47	103.15	72.03	96.73	66.00
PILE (24 x 24) – B	60.00	68.33	109.34	76.36	108.50	69.61

Table 5c. Comparisons of measured and calculated development lengths – members with depth ≤ 24 in. (610 mm).

FDOT studies	AASHTO / Measured	1.6 AASHTO / Measured	Buckner / Measured	FHWA / Measured	New / Measured
PILE (14 x 14)	2.06	3.29	2.28	3.66	2.07
PILE (18 x 18) – A	1.28	2.05	1.46	2.20	1.28
PILE (18 x 18) – B	1.69	2.70	1.87	2.59	1.69
PILE (20 x 20)	1.71	2.74	1.91	3.01	1.71
PILE (24 x 24) – A	0.98	1.56	1.09	1.47	1.00
PILE (24 x 24) – B	1.14	1.82	1.27	1.81	1.16
Average	1.48	2.36	1.65	2.45	1.48
Maximum	2.06	3.29	2.28	3.66	2.07
Minimum	0.98	1.56	1.09	1.47	1.00
Standard deviation	0.37	0.60	0.41	0.73	0.37

The specimens used in the comparisons include appropriate FSRC tests on solid and voided slabs, piles, and AASHTO Type II girders with $^1/_2$ in., $^1/_2$ in. special, and 0.6 in. (13, 13, and 15 mm) diameter strands. Also included are the results from AASHTO Type I specimens tested at the University of Tennessee[9] and the results from FHWA[4] tests.

The test results are divided into two groups according to the member depth. Table 5 shows the results for members with depth equal to or less than 24 in. (610 mm). Table 5(a) shows the basic parameters needed for calculating the development length by the various proposals.

The values from Table 5(a) are used to calculate the development lengths shown in Table 5(b) along with the measured values. Table 5(c) shows the comparisons of the different predictions with test results. It can be seen from the comparisons in Table 5(c) that all proposals yield conservative values of development length.

The FHWA and the 1.6 AASHTO proposals yield very conservative values, averaging 2.45 and 2.36, respectively. Both the AASHTO and the new proposals yield almost identical re-

Table 6a. Test parameters – members with depth > 24 in. (610 mm).

Study	Depth d (in.)	Height h (in.)	Strand diameter d_b (in.)	Concrete strength f'_c (ksi)	Initial prestress f_{si} (ksi)	Effective prestress f_{se} (ksi)	Ultimate stress f_{su} (ksi)	Ultimate strain γ_{ps}	λ
FDOT (Type II girders)									
TYPE II ($^1/_2$ in.)	40	44	0.5	7	202.5	157.6	264.6	0.028559	1.742
TYPE II ($^1/_2$ in. S)	40	44	0.5224	7	202.5	157.6	264.6	0.028559	1.742
TYPE II (0.6 in.)	40	44	0.6	7	202.5	157.6	264.6	0.028559	1.742
PILE (30 x 30S)	26.75	30	0.5	6.0	203	157.6	250.6	0.010810	1.032
Tennessee									
5-2-EXT	24	28	0.5	6.746	203	191	260	0.015875	1.235
5S-3-EXT	24	28	0.5224	5.967	203	199	260	0.015875	1.235
916-2-INT	24	28	0.5625	5.921	203	186	260	0.015875	1.235
6-2-INT	24	28	0.6	5.285	203	187	260	0.015875	1.235
FHWA (Type II girders)									
5U5-2	34	36	0.5	6.47	204	170	262	0.019000	1.36
5U5-5	34	36	0.5	6.32	204	170	262	0.019000	1.36
6U5-1	34	36	0.6	6.13	204	170	262	0.019000	1.36
FHWA (Type II composite girders)									
5U5-3	42	44	0.5	6.22	204	170	262	0.019000	1.36
5U5-8	42	44	0.5	6.56	204	170	262	0.019000	1.36
6U5-4	42	44	0.6	6.44	204	170	262	0.019000	1.36

Note: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

sults, with average and standard deviations of 1.48 and 0.37, respectively. These nearly identical values are coincidental because the terms for the transfer and bond lengths are different in both equations.

Based on these results, one could question the need for a new approach over the AASHTO equation. However, there is considerable research supporting the use of f_{si} instead of f_{se} in calculating transfer length. It is possible to develop an approach that predicts the results more accurately, but this proposed approach is simple and provides continuity of a method that has been used for generations.

A comparison of the results for members with depths exceeding 24 in. (610 mm) is presented in Table 6(c) for the various proposals. It can be seen that the AASHTO and Buckner equations give unacceptably low values for development length in most cases. The FHWA, 1.6 AASHTO, and the new proposal yield conservative results. The new proposal, however, gives average values about 20 percent greater than measured values, with a standard deviation of 0.15; this is still conservative, but less so than the 1.6 AASHTO and the FHWA equations.

It should be emphasized that the majority of full-scale testing was conducted on AASHTO Type I and Type II girders, which is indicated by the close correlation of the various proposals with test results (see Table 6). The proposed approach is the only one that reflects the effect of shear, which is more pronounced in deeper members.

Table 6b. Measured and calculated development lengths – members with depth > 24 in. (610 mm).

Study	Measured (in.)	AASHTO Eq. (2)	1.6 AASHTO	Buckner Eq. (8)	FHWA Eq. (12)	New proposal Eq. (14)
FDOT (Type II girders)						
TYPE II ($^1/_2$ in.)	115.00	79.77	127.63	126.97	116.77	143.01
TYPE II ($^1/_2$ in. S)	115.00	83.34	133.34	132.65	121.55	146.52
TYPE II (0.6 in.)	138.00	95.72	153.15	152.36	138.13	158.68
PILE (30 x 30)	96.00	72.77	116.43	81.84	127.27	116.68
Tennessee						
5-2-EXT	77.40	66.33	106.13	76.44	102.91	103.74
5S-3-EXT	81.00	66.52	106.43	74.70	115.27	103.06
916-2-INT	87.00	76.50	122.40	89.47	132.13	113.91
6-2-INT	74.40	81.20	129.92	94.69	155.23	118.26
FHWA (Type II girders)						
5U5-2	106.00	74.33	118.93	96.56	118.56	125.25
5U5-5	116.00	74.33	118.93	96.56	121.14	125.25
6U5-1	143.00	89.20	142.72	115.87	147.50	139.72
FHWA (Type II composite girders)						
5U5-3	126.00	74.33	118.93	96.56	122.93	137.01
5U5-8	119.00	74.33	118.93	96.56	117.07	137.01
6U5-4	143.00	89.20	142.72	115.87	140.88	151.48

Note: 1 in. = 25.4 mm.

For members deeper than the AASHTO Type II girder, it is anticipated that the existing equations will result in unsafe values due to the neglect of the effect of shear-flexure interaction. The new proposal offers a rational, simple approach that ac-

Table 6c. Comparisons of measured and calculated development lengths – members with depth > 24 in. (610 mm).

Study	AASHTO [Eq. (2)] Measured	1.6 AASHTO Measured	Buckner [Eq. (8)] Measured	FHWA [Eq. (12)] Measured	New [Eq. (17)] Measured
FDOT					
TYPE II ($^1/_2$ in.)	0.69	1.11	1.10	1.02	1.24
TYPE II ($^1/_2$ in. S)	0.72	1.16	1.15	1.06	1.27
TYPE II (0.6 in.)	0.69	1.11	1.10	1.00	1.15
PILE (30 x 30)	0.76	1.21	0.85	1.33	1.22
Tennessee					
5-2-EXT	0.86	1.37	0.99	1.33	1.34
5S-3-EXT	0.82	1.31	0.92	1.42	1.27
916-2-INT	0.88	1.41	1.03	1.52	1.31
6-2-INT	1.09	1.75	1.27	2.09	1.59
FHWA (Type II girders)					
5U5-2	0.70	1.12	0.91	1.12	1.18
5U5-5	0.64	1.03	0.83	1.04	1.08
6U5-1	0.62	1.00	0.81	1.03	0.98
FHWA (Type II composite girders)					
5U5-3	0.59	0.94	0.77	0.98	1.09
5U5-8	0.62	1.00	0.81	0.98	1.15
6U5-4	0.62	1.00	0.81	0.99	1.06
Average	0.74	1.18	0.95	1.21	1.21
Maximum	1.09	1.75	1.27	2.09	1.59
Minimum	0.59	0.94	0.77	0.98	0.98
Standard deviation	0.13	0.21	0.15	0.30	0.15

counts for the actual behavior and the member geometry.

CONCLUSIONS

Based on the results of this investigation, the following conclusions can be drawn:

1. Shear-flexural interaction has a marked effect on the development length of prestressing strands and should be accounted for in any design recommendations.

2. The new proposal reflects the effect of shear, which is more pronounced in deeper members, and is strongly supported by test results.

3. For prestressed concrete members with a depth equal to or less than 24 in. (610 mm):

(a) The AASHTO equation [Eq. (2)] and the proposed approach [Eq. (14)] yield the closest predictions, and the use of a multiplier is not warranted.

(b) The FHWA proposal [Eq. (12)] consistently yields extremely conservative results.

4. For prestressed concrete members with a depth greater than 24 in. (610 mm):

(a) The AASHTO equation and Buckner equation [Eq. (8)] yield unacceptably low values for development length in most cases.

(b) The FHWA proposal, the 1.6 AASHTO proposal, and the new proposal [(Eq.17)] yield conservative results; however, Eq. (17) gives average values about 20 percent greater than measured values with a standard deviation of 0.15, which is still conservative, but less so than the 1.6 AASHTO and FHWA proposals.

RECOMMENDATIONS

1. The AASHTO provisions for strand should not be based on perfect conditions. Any oil residue or contamination existing on the surface of the strand should be removed before the strand is used. Basing AASHTO provisions on imperfect strand conditions does not represent good practice. Quality control should be implemented to ensure a level playing field for all strand manufacturers. The wide scatter observed in strand transfer and developments is likely due to the absence of adequate quality control of surface condition of strands.

2. The new proposal offers a rational, simple approach for predicting the strand development length and is recommended for adoption into the AASHTO Specifications.

3. The first study of piles conducted by the author[9] examined the effect of pile embedment on the development length of $^1/_2$ in. (13 mm) diameter prestressing strand embedded in a pile cap or footing. It was concluded that this type of end condition should result in lower development length requirements due to the enhancement of bond strength caused by concrete shrinkage and resulting confinement. It is conceivable that the development length requirements for these cases would be much lower than predicted by the new proposal. However, further research is needed in this area.

The conclusions and recommendations presented in this paper are those of the author and do not necessarily reflect the views of the Florida Department of Transportation.

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APPENDIX – 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'_{ci} = compressive strength of concrete at time of initial prestress release
f_{pt} = stress in the prestressing steel prior to transfer of prestress
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 of strand
L_d = development length of strand
L_t = transfer length of strand
M_{app} = applied moment
M_n = nominal moment strength
M_{slip} = applied moment at initial slippage
M_u = design moment
u_{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_p = reinforcement index, $\rho_p\, f^*_{su}/f'_c$