Behavior of Prestressed AASHTO Girders Under Static Loading

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Recently FHWA restricted the use of certain sizes of seven-wire prestressing strand in the fabrication of highway bridge beams. The research reported is the result of the prestressing industry's interest in evaluating the effects of development length and transfer length on the behavior of AASHTO Type I beams. Additionally, variations in transfer and development length were investigated for different diameters of reinforcing and different wire manufacturers. Twenty-two full-scale AASHTO Type I beams were static-tested to failure; the results of these tests are reported. Factors that affect both transfer and development length are evaluated and discussed. The computed and measured moment capacities are compared, and reasons for variations are evaluated. Three distinct failure modes were observed, depending on the location of the load in relation to the development length of the prestressing strand. The relationship between failure modes and the ultimate moment capacity is evaluated.

Recently FHWA limited the use of certain sizes and spacings of seven-wire prestressing strand in AASHTO bridge girders. This restriction was primarily based on the results of the experimental research in which both transfer and development lengths significantly larger than those predicted by the American Concrete Institute (ACI) and AASHTO equations were obtained (*1*). Since the FHWA mandate severely restricted the ability of many AASHTO prestress producers to satisfy the needs of the construction programs of several state departments of transportation, the Precast/Prestressed Concrete Institute in conjunction with the Regional Transportation Center program funded the University of Tennessee to perform additional research to confirm or refute these results (*1*). The results of the University of Tennessee research were reported in 1991 (*2*).

The purpose of the research at Tennessee was twofold:

1. To determine whether or not the existing AASHTO design criteria for transfer lengths and development lengths in prestressed beams were satisfactory or should be modified to reflect changes in technology and methodology of production since the code criteria were established.
2. To identify the factors affecting both transfer and development lengths and to quantify their effects.

Consideration of these factors is the subject of this paper. Twenty-two full-scale beams were tested to failure to evaluate their performance when loaded at or near the expected development length of the strand. Since the loads were applied at a position closer to the reaction than to the centerline of the beam, each beam allowed for two static tests; therefore, 44 static tests were performed, and results were obtained on 43 of the tests.

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DESCRIPTION OF TEST PROGRAM

Each of the 22 beams was 31 ft long with a cross section conforming to the AASHTO Type I configuration and having varying strand configurations depending on the diameter of the strand. Figure 1 describes the shear and confinement reinforcing used for each beam, and Figure 2 shows the various strand configurations used. Prestress reinforcement manufactured by four manufacturers was evaluated. A three-part designation is used to identify an end of a beam, as illustrated by 5S-1-EXT. The first part refers to the diameter of the strands used in a beam for which

- 5 = $\frac{1}{2}$-in. regular strand, nominal diameter of 0.500 in.
- 5S = $\frac{1}{2}$-in. special strand, nominal diameter of 0.5224 in.
- 916 = $\frac{9}{16}$-in. strand, nominal diameter of 0.5625 in.
- 6 = $\frac{6}{10}$-in. strand, nominal diameter of 0.600 in.
- 5E = $\frac{1}{2}$-in. epoxy-coated strand.

For the beams from the strand-diameter groups or with the epoxy-coated strands, the second part of the designation refers to one of the beams prestressed with the strands, and the third part refers to a specific end of this beam. INT or EXT refers to the interior or exterior end of a beam, because two beams were stressed with strands between the same two abutments. For the beams from the strand manufacturer's group, the second part of the designation refers to the name of the manufacturer, and the third part refers to an end of the beam. A summary of the beam properties is given in Table 1.

TRANSFER LENGTH TESTS

Transfer lengths of the strands were measured at prestress transfer on 26 Type I AASHTO girders. These results were initially reported by Smith (*3*). Table 2 gives the average transfer bond strengths for all milled surface strands. Confinement reinforcement was placed over a distance of about 40 in. from each end of the beams, a distance that is longer than almost all the measured transfer lengths. In some exploratory tests with spiral smooth-wire confinement reinforcement around each strand, Kaar et al. found a decrease of 15 percent in transfer length as compared with those without the reinforcement (*4*). Accordingly, transfer bond strengths for the beam tests should be higher than for strands with no

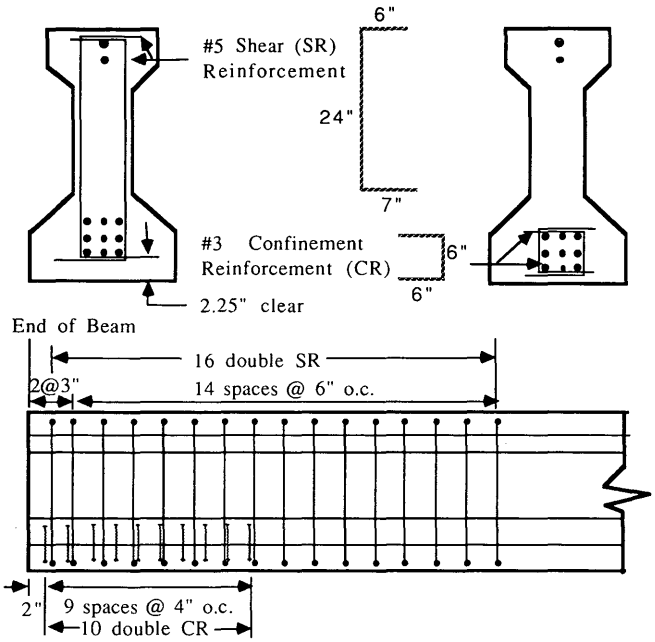

FIGURE 1 Shear and confinement reinforcement in each end of beam.

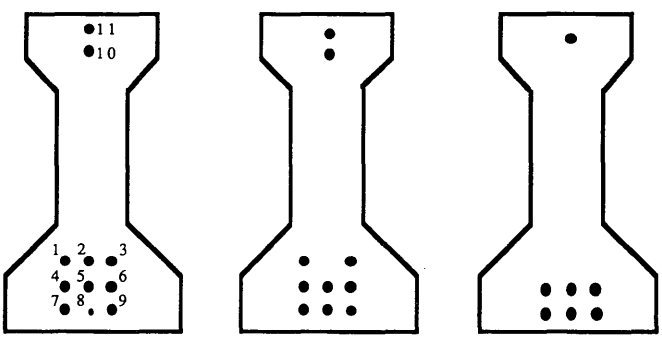

FIGURE 2 Strand configurations: *left*, Pattern A; *middle*, Pattern B; *right*, Pattern C.

TABLE 1 Measured Transfer Bond Strength for Beams

Milled Surface Strand Diameter (in.)	Bond Strength (kips/inch)	Number of Tests
1/2" Regular	0.889	4
1/2" Special	0.982	4
9/16"	1.050	4
6/10"	1.700	8

TABLE 2 Average Measured Transfer Lengths of Beams

Strand	Average Transfer Length (d_b)		
	Milled	Weathered 1-day	Weathered 3-day
1/2" Regular	64.5	46.5	39.0 (8)
1/2" Special	62.2	--	59.3 (4)
9/16"	62.2	--	48.9 (4)
6/10"	40.6	--	--

confining reinforcing. However, other factors such as the multiple strands in beams may have a negative influence on the bond strength. Multiple strands in beams were flame-cut one at a time, and strands cut earlier might have been affected by the transfer force of strands cut later. This effect could degrade the transfer bond strength in beams, perhaps more so for beams with more strands. From Table 2, the decrease in the measured bond strengths for $\frac{1}{2}$-in. regular and $\frac{6}{10}$-in. strands were about 25 and 10 percent, respectively. Greater reduction in bond strengths of beams with the $\frac{1}{2}$-in. regular strands may be due to the greater number of strands in the beams.

Effects of Strand Diameter

The perimeter of seven-wire prestressing strand is approximately equal to $4/3\pi d_b$. Adhesion force, which is directly proportional to the amount of adhered surface, is therefore directly proportional to strand diameter. Friction may be affected by strand diameter because of the difference in normal force from different wire sizes. Because the grooves between the outer wires of a strand get larger with increasing strand diameter, mechanical bond strength would increase with strand diameter. Table 3 illustrates that the average transfer lengths for the $\frac{1}{2}$-in. special, $\frac{1}{2}$-in. regular, and $\frac{9}{16}$-in. strands of milled surface condition are approximately proportional to the strand diameter, but this relationship is not true for the $\frac{6}{10}$-in. strands. The shorter transfer length for $\frac{6}{10}$-in. strands may be attributed to the increase in mechanical bond. The numbers in parentheses are the number of tests.

Effects of Strand Surface Condition

The wires of all stress-relieved and low-relaxation strands have residual surface oil from the wire drawing process. The residuals prevent good adhesion and lower the coefficient of friction at shear interface, thereby decreasing friction bonds. Weathered strands have their residual surface oil washed away and have roughened rusted surfaces with higher coefficients of friction, which are also better for concrete adhesion. From Table 3, the average improvement in the transfer length for 1-day weathering of $\frac{1}{2}$-in. regular strands was about 27.9 percent, and that for 3-day weathering for the same strand appears to be about 40 percent. The average improvement for all the 3-day weathered strands was about 22 percent. This reduction in transfer length may be due to an increase in friction for the slightly roughened surface.

DEVELOPMENT LENGTH TESTS

Development length of strand in a pretensioned member consists partly of transfer length and an additional bonded length called flexural bond length. Development length, unlike transfer length, cannot be measured directly: it can be measured only with an indirect approach in which the behavior and strength of beams loaded at different locations from the ends of beams are studied to determine whether the ultimate stress of the strands could be developed at the load points. This procedure

TABLE 3 Summary of *fcu/f'c* and Steel Stress Calculations

Specimen Designation	Concrete Strength f'c,(ksi)	Horizontal Reaction H, Kips	Measured Moment Mn, (k-ft)	Mode of Failure	With Strain Compatibility: Steel Stress fs,(ksi)	Depth to N.A. kd (in.)	Calculated fcu/f'c	Force Equilibrium: Adjusted fcu/f'c	Calculated kd (in.)	fsc (ksi)
5-1-EXT	5.48	12.7	645	B-F	277	6.20	1.05	1.20	5.13	270
5-1-INT	5.48	10.6	564	B-S	273	11.18	0.69	1.20	4.49	232
5-2-EXT	6.75	11.6	612	B-S	276	8.28	0.70	1.20	3.90	248
5-2-INT	6.75	12.3	639	B-F	277	6.57	0.81	1.20	4.04	260
5-3-EXT	6.86	12.9	675	F	278	4.44	1.13	1.20	4.17	277
5-3-INT	6.86	12.7	679	B-F	279	4.23	1.19	1.20	4.19	279
5-4-EXT	7.60	12.8	676	F	278	4.37	1.03	1.20	3.77	274
5-4-INT	7.60	12.5	665	B-F	278	4.99	0.91	1.20	3.72	269
5-SWAI-EAST	5.55	12.2	641	F	277	6.81	0.96	1.20	4.96	265
5-SWAI-WEST	5.55	12.2	660	F-B	277	5.58	1.12	1.20	5.12	275
5-UWR-EAST	5.99	12.7	679	B-F	278	4.57	1.26	1.20	4.81	280
5-UWR-WEST	5.99	20.7	707	B-F	279	4.01	1.47	1.20	5.00	286
5-FWC-EAST	5.34	11.9	641	B-F	277	6.81	1.00	1.20	5.19	267
5-FWC-WEST	5.34	12.4	660	F	277	5.65	1.15	1.20	5.37	276
5-ASW-EAST	5.40	11.9	633	F	277	7.27	0.95	1.20	5.06	263
5-ASW-WEST	5.40	11.8	638	F	277	6.93	0.98	1.20	5.11	265
5S-1-EXT	6.62	10.7	569	B-S	256	15.83	0.51	0.90	5.19	196
5S-1-INT	6.62	13.6	724	F-B	268	8.08	0.85	0.90	7.26	263
5S-2-EXT	-	-	-	-	-	-	-	-	-	-
5S-2-INT	-	-	-	F-B	-	-	-	0.90	5.95	223
5S-3-EXT	5.97	12.2	640	F	268	8.84	0.80	0.90	7.01	257
5S-3-INT	5.97	11.1	587	B-S	263	11.66	0.65	0.90	6.14	231
5S-4-EXT	6.18	10.7	574	B-S	262	12.32	0.60	0.90	5.67	223
5S-4-INT	6.18	12.3	666	B-F	269	7.19	0.89	0.90	7.03	268
916-1-EXT	5.53	14.0	688	B-F	261	10.63	0.85	0.90	9.56	254
916-1-INT	5.53	13.5	703	B-F	262	9.85	0.90	0.90	9.95	263
916-2-EXT	5.92	13.2	666	B-F	260	11.71	0.74	0.90	7.85	237
916-2-INT	5.92	13.1	682	B-F	261	10.86	0.78	0.90	8.19	245
916-3-EXT	6.12	13.1	663	B-F	274	2.85	1.76	2.20	2.31	271
916-3-INT	6.12	13.7	675	F	277	2.45	2.10	2.20	2.34	276
916-4-EXT	6.24	13.6	669	B-F	275	2.66	1.88	2.20	2.28	272
916-4-INT	6.24	14.0	684	F	278	2.20	2.33	2.20	2.34	281
6-1-EXT	5.13	13.9	658	F	264	7.22	0.98	1.00	6.91	262
6-1-INT	5.13	27.4	682	F	263	7.44	1.00	1.00	7.37	263
6-2-EXT	5.29	12.6	660	B-F	264	6.89	0.98	1.00	6.57	262
6-2-INT	5.29	12.3	657	B-F	264	7.06	0.96	1.00	6.51	261
6-3-EXT	7.46	13.3	701	B-F	268	4.72	0.94	1.00	4.42	266
6-3-INT	7.46	13.6	704	F	268	4.59	0.97	1.00	4.44	268
6-4-EXT	7.98	13.3	695	F	267	5.00	0.83	1.00	4.08	262
6-4-INT	7.98	13.6	711	F	269	4.29	0.97	1.00	4.16	269
5E-1-EXT	5.78	12.2	684	F	279	3.88	1.55	1.20	5.11	288
5E-1-INT	5.78	12.3	665	F	278	4.93	1.21	1.20	4.96	278
5E-2-EXT	6.57	10.9	620	B-S-F	277	7.65	0.75	1.20	4.04	253
5E-2-INT	6.57	11.5	652	F	277	5.63	0.94	1.20	4.22	268

was followed for different strand diameters, surface conditions, concrete strengths, and pretension levels. Different spacings of strands from different manufacturers were used. Blended in the results are other variables such as concrete strength, reinforcement ratio, and strand pretension level, all of which have some influence on both the behavior and the strength of the beam section.

The development length for a beam is interrelated with the beam behavior. The steel stress that a strand can develop at a beam section is affected by variables such as concrete strength, steel pretension level, reinforcement percentage, concrete confinement, and beam geometry. The corresponding force in the strand must also be adequately transmitted along the development length into the concrete. Transmission of the force over the available development length is dependent on the bond behavior and the beam behavior. For example, the deformational characteristics of the strand at the beam section when the steel force is transmitted over the development length affect the steel stress that can be developed at the section. Using the following procedure to obtain the steel stress developed, the effects on the steel stress of the variables that affect beam strength are minimized.

Theoretical Steel Stress at Ultimate Flexural Compression Failure

The ultimate moment strength of each beam was measured during the development length tests. Because of confinement vertically and laterally by the steel bearing plate at the load point of the top fibers of the concrete in compression and the sharp stress gradient beneath the single load point, a considerably higher concrete strength was apparently attainable at the critical load-point section. The moment strength calculated according to the ACI allowable rectangular stress block and strain-compatibility method was therefore somewhat lower than the actual moment strength for these beams. The steel stress is overestimated greatly if it is calculated from the measured moment strength using the ACI method. This overestimation leads to an error in the steel stress assumed to have been developed, which is then used in the derivation of the bond strength for the development length. Though the ACI method for the calculation of flexural strength is adequate for general design purposes, particularly for a flexural member without lateral confinement for the concrete in compression, it is not sufficiently accurate for the needs of the present research.

A fundamental approach somewhat similar to the analysis method found in the publication by Warwaruk et al. (5) is devised here to solve this problem. In their research, the compressive concrete in the beams loaded at mid-span appeared to have twice the effective strength of the concrete in beams with two-point loadings. Typically, a condition of strain and stress at failure is assumed for a beam as in Figure 3. The steel strain at ultimate is calculated by adding the steel strain at effective prestress (ε_{se}), the elastic shortening (ε_{ce}), and the additional steel strain beyond flexural cracking of the section (ε_{sa}). Assumptions for the present analysis include the following:

1. Plane sections remain plane under flexure;
2. Strain compatibility factor is known and is a constant 1.0;

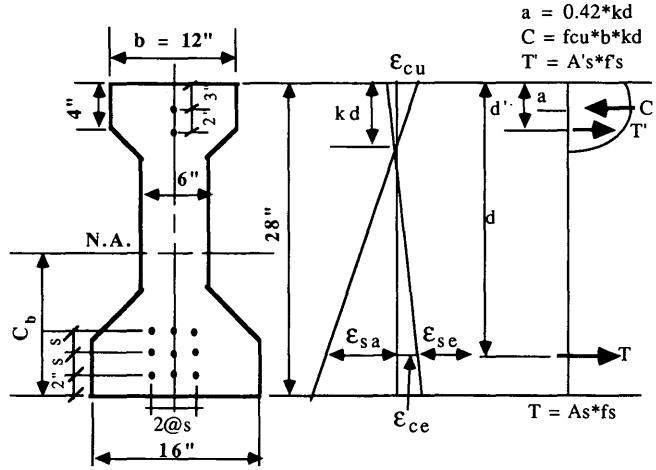

FIGURE 3 Conditions of strain and stress for AASHTO Type I beam, cross section; area = 276 in.^{2}, I = 22,750 in.^{4}, C_b = 12.59 in., s = strand spacing (in.).

3. Failure of concrete occurs when concrete strain in the extreme fibers reaches a useful limit of 0.005 (i.e., ε_{cu} = 0.005.4);
4. The location of the resultant compressive force of the concrete is at a distance from the extreme compressive fibers equal to 0.42 of the depth of the neutral axis (i.e., $a = 0.42 * kd$);
5. Concrete does not carry tension.
6. The stress-strain relationship for the reinforcement is known.
7. A horizontal inward reaction, H, is equal to a friction coefficient of 0.4 times the smaller of the two support reaction forces.

It is realized that the fourth assumption is likely to vary very slightly with concrete strength or when kd is greater than the flange thickness of 4 in. However, the calculated flexural strength is quite insensitive to such small variation. For example, for an actual value of $a = 0.35kd$ in comparison to the assumed $a = 0.42kd$, only an error of 0.35 in. in moment arm results for a typical kd value of 5 in. This translates into an error in the flexural strength of less than 1.5 percent.

It is necessary to determine the effective concrete strength, fcu, of the concrete subjected to the effect of confinement for the beams in this test program. A computer program was written that iteratively calculates the fcu for each of the beams from the measured ultimate moment in accordance with the strain compatibility condition for the steel strand and the assumptions stated previously.

The ratio of the effective concrete strength to the 6 × 12-in. concrete cylinder strength, $fcu/f'c$, is calculated. The results of the calculation are summarized in Table 4 under the heading "With Strain Compatibility." The value of $fcu/f'c$ is plotted against the depth to the neutral axis, kd, in Figure 4. From this figure, the ratio of $fcu/f'c$ is much larger at smaller values of kd and rapidly reduces to a more-stable lower value of approximately 0.85 as kd increases. The data points with the smaller values of kd correspond to the beams with the smaller reinforcement ratios. The larger $fcu/f'c$ with smaller kd values is consistent with the knowledge that the effect of

TABLE 4 Measured Moment and Mode of Failure

Specimen Designation	fc (psi)	Steel Reinforcement Ratio, (%)	Mode of Failure	Moment (kip-ft) Measured Mcr	Mb	Mu	ACI Mn	Mcr/Mu	Mb/Mn	Mu/Mn
5-1-EXT	5476	0.421	B-F	396	611	645	572	0.61	1.07	1.13
5-1-INT	5476	0.421	B-S	469	550	564	572	0.83	0.96	0.99
5-2-EXT	6746	0.421	B-S	444	612	612	596	0.73	1.03	1.03
5-2-INT	6746	0.421	B-F	415	623	639	596	0.65	1.05	1.07
5-3-EXT	6858	0.421	F	388	-	675	598	0.57	-	1.13
5-3-INT	6858	0.421	B-F	408	645	679	598	0.60	1.08	1.14
5-4-EXT	7600	0.421	F	410	-	676	607	0.61	-	1.11
5-4-INT	7600	0.421	B-F	453	644	664	607	0.68	1.06	1.09
5-SWAI-EAST	5553	0.417	F	430	-	641	579	0.67	-	1.11
5-SWAI-WEST	5553	0.417	F-B	485	660	660	579	0.73	1.14	1.14
5-UWR-EAST	5989	0.417	B-F	463	630	679	593	0.68	1.06	1.15
5-UWR-WEST(S)	5989	0.417	B-F	459	611	707	593	0.65	1.03	1.19
5-FWC-EAST	5341	0.417	B-F	419	624	640	572	0.65	1.09	1.12
5-FWC-WEST	5341	0.417	F	448	-	660	572	0.68	-	1.15
5-ASW-EAST	5400	0.417	F	429	-	633	574	0.68	-	1.10
5-ASW-WEST	5400	0.417	F	463	-	638	574	0.73	-	1.11
5S-1-EXT	6624	0.522	B-S	466	565	569	669	0.82	0.84	0.85
5S-1-INT(LC)	6624	0.522	F-B	449	719	724	669	0.62	1.07	1.08
5S-2-EXT	-	0.522	-	-	-	-	672	-	-	-
5S-2-INT(S)	6800	0.522	F-B	408	692	692	672	0.59	1.03	1.03
5S-3-EXT	5967	0.459	F-B	471	-	640	602	0.74	-	1.06
5S-3-INT	5967	0.459	B-S	457	587	587	602	0.78	0.98	0.98
5S-4-EXT	6181	0.459	B-S	442	530	574	606	0.77	0.87	0.95
5S-4-INT	6181	0.459	B-F	472	601	666	606	0.71	0.99	1.10
916-1-EXT	5533	0.528	B-F	488	682	688	641	0.71	1.06	1.07
916-1-INT	5533	0.528	B-F	476	633	703	641	0.68	0.99	1.10
916-2-EXT	5921	0.528	B-F	455	642	665	661	0.68	0.97	1.01
916-2-INT	5921	0.528	B-F	465	677	682	661	0.68	1.02	1.03
916-3-EXT	6119	0.384	B-F	419	628	663	557	0.63	1.13	1.19
916-3-INT	6119	0.384	F	390	-	675	557	0.58	-	1.21
916-4-EXT	6237	0.384	B-F	390	601	669	559	0.58	1.08	1.20
916-4-INT(LC)	6237	0.384	F	429	-	688	559	0.62	-	1.23
6-1-EXT	5126	0.434	F	412	-	658	595	0.63	-	1.11
6-1-INT(S)	5126	0.434	F	335	-	682	595	0.49	-	1.15
6-2-EXT	5285	0.434	B-F	398	660	660	600	0.60	1.10	1.10
6-2-INT	5285	0.434	B-F	462	609	657	600	0.70	1.02	1.10
6-3-EXT	7463	0.434	B-F	424	696	701	637	0.60	1.09	1.10
6-3-INT	7463	0.434	F	426	-	704	637	0.61	-	1.11
6-4-EXT	7984	0.434	F	406	-	695	642	0.58	-	1.08
6-4-INT	7984	0.434	F	428	-	711	642	0.60	-	1.11
5E-1-EXT	5783	0.421	F	436	-	683	579	0.64	-	1.18
5E-1-INT	5783	0.421	F	449	-	665	579	0.68	-	1.15
5E-2-EXT	6573	0.421	B-S-F	382	565	620	593	0.62	0.95	1.05
5E-2-INT	6573	0.421	F	417	-	652	593	0.64	-	1.10

NOTES: B - Bond Failure (Strands Slip), S - Shear Failure,
F-Flexural Compression Failure, (S) - Short Portion of Broken Beam

confinement is more significant right beneath the bearing plate and within a shallower *kd*. The figure also shows that for the beams with bond failure, the data points for which are plotted as BF, the *fcu*/*f'c* ratio is generally smaller than for the beams without bond failure (F). This is, however, an inherent result of the assumption for the analysis that strain compatibility holds and of the fact that the beams with bond failure generally have slightly lower moment strengths. For the beams with bond failures, deviation from the strain compatibility condition leads to a *kd* value greater than it would have been with no bond failure. This greater *kd* value consequently enters into the calculation of the resulting lower *fcu* value. Thus, only the beams that develop the full flexural strengths without any end slippages were used in establishing the *fcu* needed for the calculation of the steel stresses developed in the beams with bond failure.

For the beams with bond failure, the steel stresses can be calculated from the measured moment strengths with a chosen approximate *fcu* value for each beam group in reference to the *fcu*/*f'c* calculated earlier with strain compatibility for beams without bond failure. Because the bottom strands in the beams with bond failure slip, the assumption of perfect strain compatibility condition is not valid. Except for the strain compatibility condition for the bottom steel, the assumptions for the flexural strength analysis stated previously are used. The top steel does not slip; its stress can therefore be calculated from strain compatibility with the previously set $\varepsilon_{cu} = 0.005$ and a *kd* value. The concrete force, *C*, and position, $a = 0.42kd$, from which the moment, *Mn*, can be calculated, are only dependent on the *kd* value. There is a unique *kd* value that gives the *Mn* that is equal to the measured moment. A computer program was written to iteratively seek for this *kd* value. The steel stress was then calculated for this *kd* from the equilibrium of horizontal forces. The calculated steel stresses and the corresponding adjusted *fcu* values are presented in Table 4 under the heading "Force Equilibrium." The steel

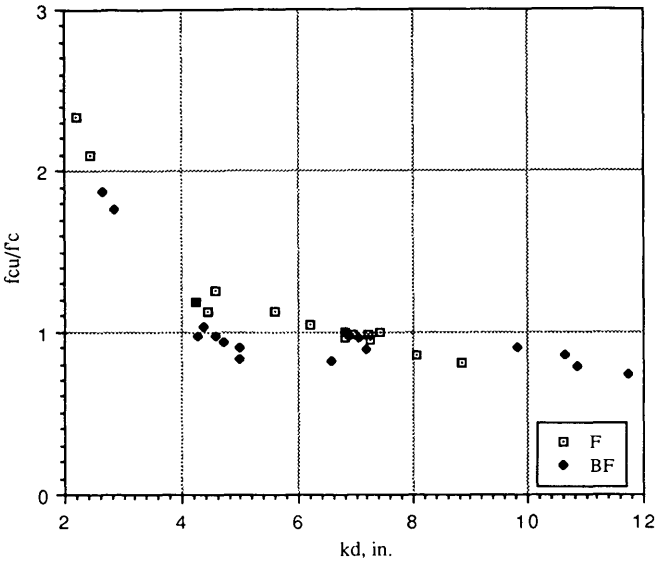

FIGURE 4 ***fcu/f'c*** **versus** ***kd*** **for all beams with flexural failure.**

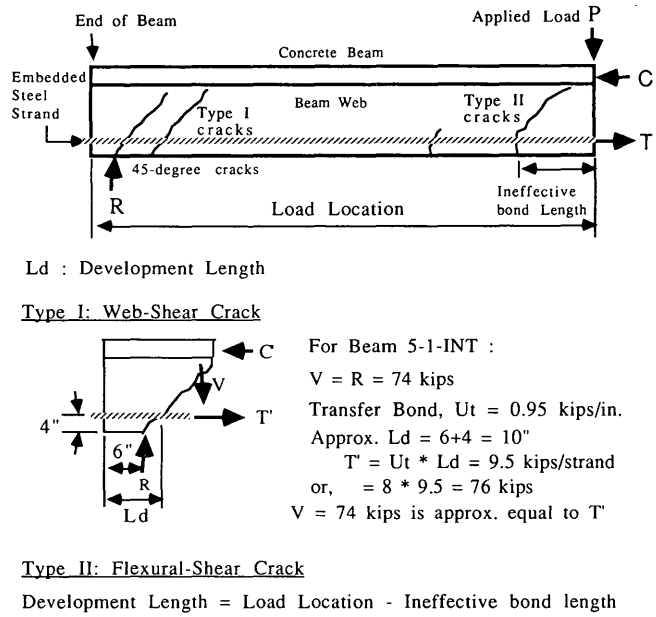

FIGURE 5 Interaction of shear crack with development length.

stress in the beams that failed by shear is not predictable with the flexural analysis. These beams are shaded in Table 4.

For beams with lower reinforcement ratios, slight slippage of some strands will only redistribute the stresses among the strands and do not immediately result in a lower flexural strength. Thus, a steel stress higher than that at first slippage may be achieved for an underreinforced beam. However, steel in underreinforced beams is generally stressed more than steel in highly reinforced beams. Although bond failure is more likely for an underreinforced beam, the flexural strength of the beam may not be affected by the bond failure. On the other hand, flexural strength of overreinforced beams is controlled by the available area of compressive concrete. The overreinforced beams, therefore, cannot tolerate slippage that reduces the compressive concrete area. These relationships are studied for a better understanding of the effects of strand slippage on beam behavior.

The different modes of failure observed in the destructive tests were examined in detail, particularly the interaction between the flexural compression failure or shear failure with a bond failure. As illustrated in Figure 5, a sudden shear failure at the initiation of diagonal shear cracking (Type I failure) may occur without an adequate development length from the end of a beam to the shear crack that starts at the interior face of support. The measured shear load, V, that beam 5-1-INT could sustain after the diagonal cracking was much reduced—only 74 kips. The effects of the diagonal shear cracks on the bond development seem to be adequately analyzed using the Truss analogy method. The total tensile force (T') that can be developed at the shear crack that initiates from the face of support is calculated from the transfer bond strength for the number of bottom strands. For a 45-degree angle of the shear crack, the shear force V at the cracked section that failed because of insufficient development length would be close to T'. Also illustrated in the figure is a flexural-shear failure (Type II failure) with crack that propagates from a flexural crack some distance closer to the end of beam. The cracking from Type I and Type II failures in essence move the critical section for the evaluation of development length to the cracks.

Effects of Bond Failure on Beam Behavior

The effects of bond failure on the behavior of all the bridge beams tested are examined in detail and discussed with illustrations. As has also been observed by previous investigators, many of the beams tested could develop additional beam strength even after end slips of some strands were detected. Bond failure of the strands apparently did not cause a significant loss in the steel stress of the strands, and the additional flexural strength gained was probably contributed by the other strands that did not slip. The plastic nature of the bond behavior permits the strands to keep slipping with virtually no loss of bond resistance. This is an extremely important characteristic of the strand performance in contrast to the plain wires. Slippage of the strands that have exhausted their "usable" bond strength is beneficial because it permits the other strands—those that are not yet slipping and have some reserve bond strength—to continue to participate in carrying the extra load. Ideally, for a beam that has less than the full development length, maximum flexural strength is developed only when all usable bond strength in all the strands has been exhausted. However, the inability of slipping strands to carry any additional tension reduces the stiffness of the beam. For a beam approaching its ultimate strength with the steel stressed past the yield point and the extreme fiber of compressive concrete stressed into the inelastic range, this loss of stiffness certainly would result in a flexural compression failure.

Research with pullout bond tests has suggested that the bond resistance, although with significant variability, generally drops slightly when the last of the adhesive bond is broken at the initiation of strand end slip, and that higher bond resistance is attainable at greater slip when mechanical bond takes

effect (*6*). Therefore, bond length has an effect on the drop in load at the loaded end when end slip at the free end occurs. Stocker and Sozen observed from their test results that the mechanical bond may become effective immediately after slip or only after a relatively large slip of about 0.01 in. (*7*). The large slip cannot be accommodated at a flexural section without excessively reducing the compressive concrete area and causing a failure. When only a small proportion of the strands suffer "slight" slips, there may be just a minor drop in load with the stiffness provided by the other strands. Creep of the concrete surrounding a strand that experiences the high intensity of bond stress has been observed in pullout tests to gradually relieve bond resistance and permit slight slippage of the strand to occur until it becomes stabilized. This slight slippage probably occurred for the strands in the UTK beams. The load-versus-deflection plots in Figures 6 and 7 show that a slight drop in load occurred while the end slips of a small number of the strands were detected. The load capacity of the beams did not reduce with increasing applied deflection until flexural compression failure occurred. However, when most of the strands in a beam slip nearly at the same time, the load drops off significantly, to not regain strength and finally fail in a flexural compression failure at increasing deflection; Figure 8 shows the load-versus-deflection plot of Test 6-2-INT, which has five out of six strands slipping when load drops.

Increases in the steel stresses of strands that did not slip were measured by strain gauges, which confirmed that higher bond resistance was possible after slippage of some strands in the beam. As presented in Table 4, not all strands had slipped when the beam finally failed in flexural compression and lost most of its strength. Further increase in steel stress was also observed beyond flexural compression failure of some beams.

Some slight slippage of the strands during static testing did not significantly reduce a beam's flexural strength, but it did make the beam more susceptible to shear failure, especially when loaded at short shear span. Bond failure relieves the compressive strain in the lower beam web to permit greater participation of the concrete to carry tensile stresses until cracking of the concrete occurs. Shear cracks further reduce the available development length and precipitate a shear failure with a significantly reduced beam strength. Flexural compression failure of beams having web-shear cracks is often not reached.

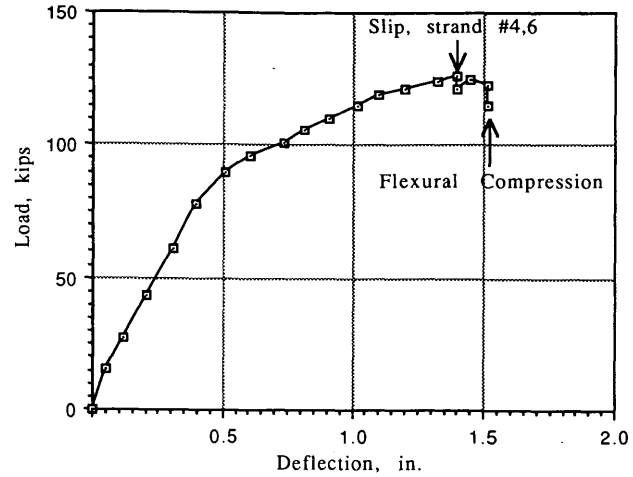

FIGURE 7 Load versus deflection for Beam 6-2-EXT.

Load Capacity and Mode of Failure

In accordance with the ACI equivalent concrete stress block and the strain compatibility condition with an ultimate compressive strain of 0.003 for the concrete extreme fiber, the theoretical moment strength is computed for each beam section and listed in Table 5 as *Mn*. The computed *Mn* values are compared with the measured *Mu* and *Mb* of the corresponding beams. As seen in the table, all beams without a bond failure had *Mu* exceeding *Mn*. The ratios of measured to computed ultimate moments of the beams, *Mu/Mn*, appear to be somewhat related to the reinforcement ratios—the lower the reinforcement ratio, the higher the *Mu/Mn*. The higher *Mu/Mn* ratios for the beams with lower reinforcement ratios

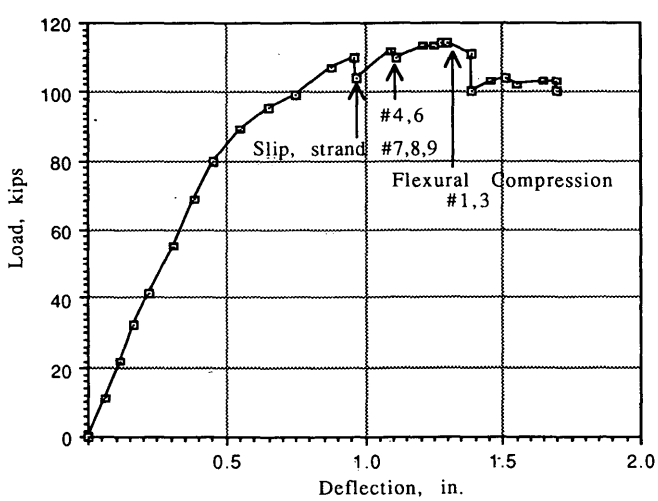

FIGURE 6 Load versus deflection for Beam 916-2-EXT.

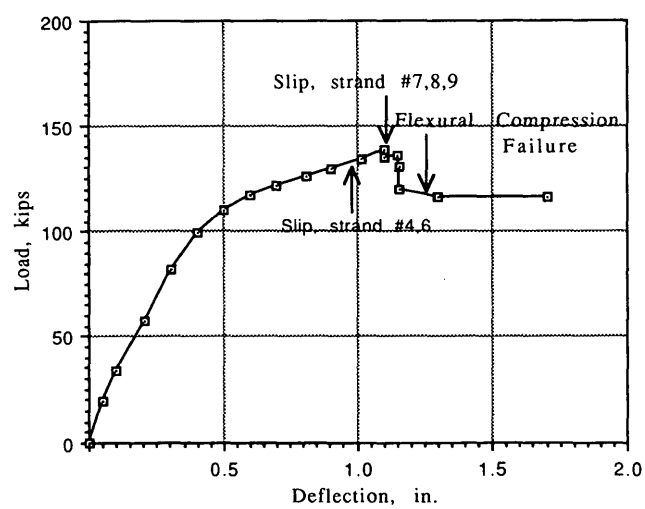

FIGURE 8 Load versus deflection for Beam 6-2-INT.

TABLE 5 Minimum and Full Development Lengths

Strand (Mill Condition)	Minimum Development Length (in.)	Full Development Length (in.)
1/2" Regular	77.4	above 92
1/2" Special	81	above 83
9/16"	87	above 106
6/10"	74.4	86

are due to the effects of greater confinement of the compressive concrete in these beams.

All the tests with bond failure also developed *Mu* exceeding the *Mn*. Bond failure of these beams occurred also at a moment above the *Mn*, except for four tests that developed the moments of slightly less than the *Mn* when bond failures occurred (i.e., tests 5S-4-INT, 916-1-INT, 916-2-EXT, and 5E-2-EXT). All the beams with bond failure were near to flexural compression failures when bond failures occurred, and the strengths of these beams did not seem to be affected significantly by the bond failures.

Only four tests did not ultimately develop the calculated *Mn*. In each of these tests, the beam experienced a shear failure soon after strand end slips were detected. One other test (5-2-EXT) reached an *Mu* slightly greater than *Mn* when shear cracking occurred simultaneously with a bond failure; it failed in shear immediately thereafter. Flexural compression failure of these five beams was not reached. The beams with shear failure apparently had load capacities after shear cracking that were below the load at bond failure. These beams are previously explained to have failed due to insufficient development length (or anchorage) at the shear cracks. According to the truss analogy for the Type I failure as illustrated in Figure 5, the load capacities of these beams after shear cracking are calculated and presented in Table 5.

The "minimum measured development lengths" of the strands to develop the computed ultimate moment, *Mn*, of the beams are 77.4, 81, 87, and 74.4 in., respectively, for $\frac{1}{2}$-in. regular, $\frac{1}{2}$-in. special, $\frac{9}{16}$-in., and $\frac{6}{10}$-in. strands of milled condition and pretension of 202.5 ksi. These minimum development lengths correspond to approximately 155 times the diameter of the strands ($155d_b$) for all the strands except the $\frac{6}{10}$-in. strand. Contrary to expectation, the $\frac{6}{10}$-in. strand requires only about 125 d_b or less than what is required for the $\frac{1}{2}$-in. regular strand. However, significantly longer development lengths are required to totally prevent any slippage of the strands at the ultimate flexural moments of the beams; in Table 5 these lengths are tabulated as the full development length. Strands exposed to weathering of up to 3 days in comparison with milled surface condition strands have better bonding characteristics and thus reduced development lengths. The epoxy-coated $\frac{1}{2}$-in. strands performed significantly better than all the uncoated strands with a full development length of less than 51 in. Reduction of strand spacing from 2 to 1.75 in. for $\frac{1}{2}$-in. strands did not appear to cause any adverse effects on the development length. In the eight tests with the smaller spacing, there was no evidence of splitting between the strands.

CONCLUSIONS

Information is presented herein regarding the bond strength of seven-wire strand in AASHTO-PCI beams and the effect of strand diameter and of weathering on bond strength and, in turn, on transfer length. Exposure to weathering has a significant effect on transfer length, as illustrated in Table 3.

An analysis is presented of the behavior of the beams tested in the research project on which this paper is based, and the effects of bond failure on this behavior are discussed. It was found that although bond failure and the resulting strand slip does not typically lead to a moment capacity less than that calculated by the ACI code, strand slippage did make the beam more susceptible to shear failure. Some of the most interesting information to emerge from the tests relates to the variation of average stress in the compressive stress block, *kd*, as illustrated in Figure 4. For beams with relatively low areas of reinforcement, *kd* is relatively small and the average concrete stress significantly larger than that assumed in conventional strength analysis.

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Publication of this paper sponsored by Committee on Concrete Bridges.