TRANSFER AND DEVELOPMENT LENGTH OF PRETENSIONED GIRDERS CONSTRUCTED WITH SLATE HIGH STRENGTH LIGHTWEIGHT CONCRETE

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ABSTRACT

This paper describes the results of transfer and development length testing on pretensioned AASHTO Type II composite girders constructed using slate high strength lightweight concrete with f_c' between 8910 psi (61 MPa) and 10980 psi (76 MPa). Each was prestressed using 10 0.6-in, (15 mm) LOLAX strands tensioned to 75% of strand ultimate stress. Transfer length measurements were taken from time of release until the beams reached an age of 14 days. Test results indicated the following: both AASHTO Standard Specification 17th Edition and ACI 318 transfer and development length equations are conservative for use with slate lightweight concrete having compressive strengths below 11,000 psi (76 MPa); shear cracking in the transfer region across the bottom strands did not induce significant strand slip if stirrup density were doubled over the current AASHTO specified density.

KEYWORDS: Prestressed Concrete, Lightweight Concrete, High-Strength Concrete, Transfer Length, Development Length, Direct Pull-out, Pretensioned Bridge Girders, Prestressing Strand Slip, Silica Fume

INTRODUCTION

The purpose of this research was to determine if the transfer and development lengths of 0.6-in. (15 mm) diameter prestressing strands in high strength lightweight concrete (HSLC) girders was less than the values predicted in current code specifications. Assuring that girders made with HSLC satisfy existing standards would permit the utilization of this high performance concrete for construction of longer span girders and, thus, provide for more economical transportation and erection. Previous research conducted relating to lightweight concrete (LWC) used in prestressed applications involved concrete with compressive strengths less than 7,500 psi (52 MPa). An earlier analytical study showed that the use of HSLC would be beneficial for extending the lengths of bridge girders[1].

BACKGROUND

TRANSFER LENGTH

Transfer length is defined as the distance required to transfer the effective prestressing force from the strand to the surrounding concrete. Many experimental programs have focused on factors like concrete strength and strand diameter in determining an expression to predict transfer length[2-12].

Initial transfer length testing by Janney[2] in 1954 concluded that transfer length was attributable to diameter and surface condition of the prestressing wire and concrete strength. Hanson and Kaar's[3] work in 1959 found an average transfer bond stress of 400 psi (2.75 MPa). In 1962, Mattock[4] assumed an average transfer bond stress of 400 psi to derive the current ACI[13] equation, Eq. (1), for transfer length.

$$l_t = \frac{f_{se} d_b}{3} \tag{1}$$

Mattock[4] assumed the effective stress, f_{se}, for 250 ksi (1.72 GPa) strand was 150 ksi (1.38 GPa) and further simplified Eq. (1) to Eq. (2), which was adopted by ACI in 1963 and AASHTO[14] in 1973.

$$l_t = 50 d_b \tag{2}$$

To date, no equations have been suggested to predict transfer length that specifically address HSLC. AASHTO LRFD[15] has increased l_t to $60d_b$.

DEVELOPMENT LENGTH

Development length of prestressing strands is the sum of the transfer length and the flexural bond length. Development length can be defined as the minimum distance from the end of the member beyond which the application of a point load will result in a flexural failure rather than a bond failure. As in transfer length, many factors are thought to affect development length. Many experimental programs have addressed development length resulting in suggested equations for its prediction[6-8, 10-12, 16].

In 1959, Hanson and Kaar[3] reported a study in involving 47 small-scale concrete beams reinforced with various sizes of Grade 250 stress-relieved strand. The tests focused on five factors including strand diameter, embedment length, concrete strength, percentage of reinforcement, and strand surface condition. All beams were loaded to failure. Hanson and Kaar reported observations similar to those of Janney[2]. The results of Hanson and Kaar were the basis for the current AASHTO development length expression developed by Mattock[4] and shown in Eq. (3) which was based on Grade 250 prestressing strand.

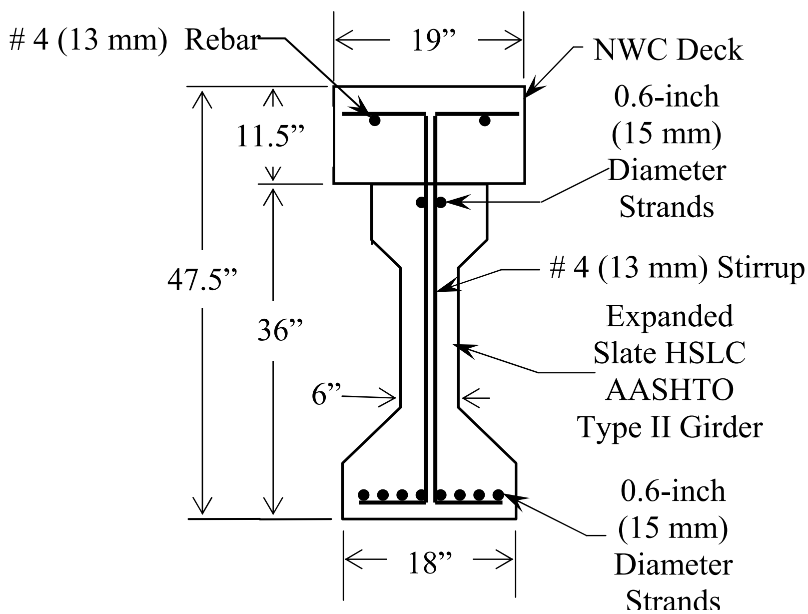
$$l_d = (f^*_{su} - \frac{2}{3} f_{se}) d_b \tag{3}$$

Researchers have also suggested other equations for predicting development length that include terms to address concrete strength and prestressing strand stress values at various times[6-8, 10-12, 16]. To date, no equations have been suggested to predict development length that specifically address HSLC.

EXPERIMENTAL PROGRAM

Six AASHTO Type II composite girders were constructed with ten 0.6-inch (15.2 mm) diameter 270 ksi (1.86 GPa) low relaxation strands as illustrated in Figs. 1 and 2.

Fig. 1. West End of Girder G2A Before Development Length Testing



Note: 1 inch = 25.4 mm

Fig. 2. Composite Girder Cross Section

Three girders were made with an 8,000 psi (55.2 MPa) design strength expanded slate HSLC and three girders were made with a 10,000 psi (68.9 MPa) design strength expanded slate HSLC mix. The 8,000 psi (55.2 MPa) HSLC was termed a Grade 1 mix while the 10,000 psi (68.9 MPa) HSLC was termed a Grade 2 mix. The eight bottom and two top strands were stressed to $0.75f_{pu}$ (203 ksi, 1.40 GPa) prior to release. The girders were cast in pairs on three different days as listed in Table 1.

Table 1 provides details on the mix components for each girder, the date of casting, and the 1-day, 56-day, and “on day of test” compressive strengths. The G1 and G2 series girders refer to girders constructed with the 8,000 psi (55.2 MPa) and 10,000 psi (68.9 MPa) design strengths, respectively. The coarse aggregate in both mix designs consisted of expanded slate lightweight aggregate from North Carolina.

The 39-foot (11.9-m) (type “A” and “B”) and 43-foot (13.1-m) (type “C”) long girders were designed so that point loads could be placed near each end for development length tests and so that the middle section could be subsequently tested to examine shear and flexural capacity. A composite deck with a thickness of 11.5 inches (292 mm) and width of 19 inches (483 mm) was placed on each girder. The composite deck was designed to give the same internal flexural moment arm “*jd*” as in a composite bridge with an 8-inch (20.3-cm) thick deck and an effective width of 93 inches (2.36 m), but to ease construction by having the deck with a width as close to the width of the top flange of the girder as possible. A 3,500 psi (24.1 MPa) normal weight bridge mix was used for the composite deck; its compressive strength at time of girder testing was 5,370 psi (37 MPa). The test configurations are given in Table 2. Fig. 3 illustrates the test set-up.

Table 1. Expanded Slate HSLC Mixes for Girder Placements (Quantities per Cubic Yard)

Material / Girder #	G1A, G1B	G2A, G2B	G1C	G2C
Design Strength (psi)	8,000	10,000	8,000	10,000
Date of Casting	9 Jul 01	12 Jul 01	17 Jul 01	17 Jul 01
Lightweight Aggregate (lbs)	989	979	974	998
Normal Weight Sand (lbs)	1098	1087	1078	1083
Class F Fly Ash (lbs)	142	150	142	150
Silica Fume (lbs)	19	100	19	100
Type III Portland Cement (lbs)	785	740	785	740
Water Reducer (WRDA 35) (fl oz)	57	60	57	60
Air Entrainer (Daravair 1000) (fl oz)	12	10	12	10
HR Water Reducer (ADVA Flow) (fl oz)	48	129	48	129
Water (lbs)	165	153	186	138
Water/Cementitious Materials Ratio	0.28	0.23	0.28	0.23
1-day f_c' (average, accelerated curing) (psi)	7,470	9,640	6,320	8,260
28-day f_c' (average, ASTM curing) (psi)	8,840	10,120	7,600	9,810
56-day f_c' (average, ASTM curing) (psi)	9,350	10,820	8,460	10,510
f_c' on day of girder test (psi)	10,230	11,010	9,120	10,870
Girder age at test (days)	103	123	110	144

Note: 1000 psi = 1 ksi = 6.9 MPa, 1 lb = 4.45 N, 1 fluid oz = 29.6 ml

Table 2. Girder Test Configurations

Test Configuration	Stirrup Density	Stirrup Spacing (in)	Shear Span "a" (in)	Distance "L_1" (in)	Distance "L" (Girder Length – 6 in.) (in)
1	Single	7	90	456	456
2	Double	3.5	61	316	456
3	Double	3.5	75	456	456
4	Single	7	85	504	504
5	Single	7	61	321	456
6	Single	7	75	369	504

Note: 1 inch = 25.4 mm

Shear reinforcement in the development length region of each girder end was designed to satisfy AASHTO and ACI provisions and was termed "single density." In some girders, twice this required shear reinforcement was included, termed "double density," in order to study the effect of shear reinforcement on strand development. Stirrups were two "C" shaped No. 4 (13 mm) Grade 60 bars (f_y = 62 ksi, 427 MPa).

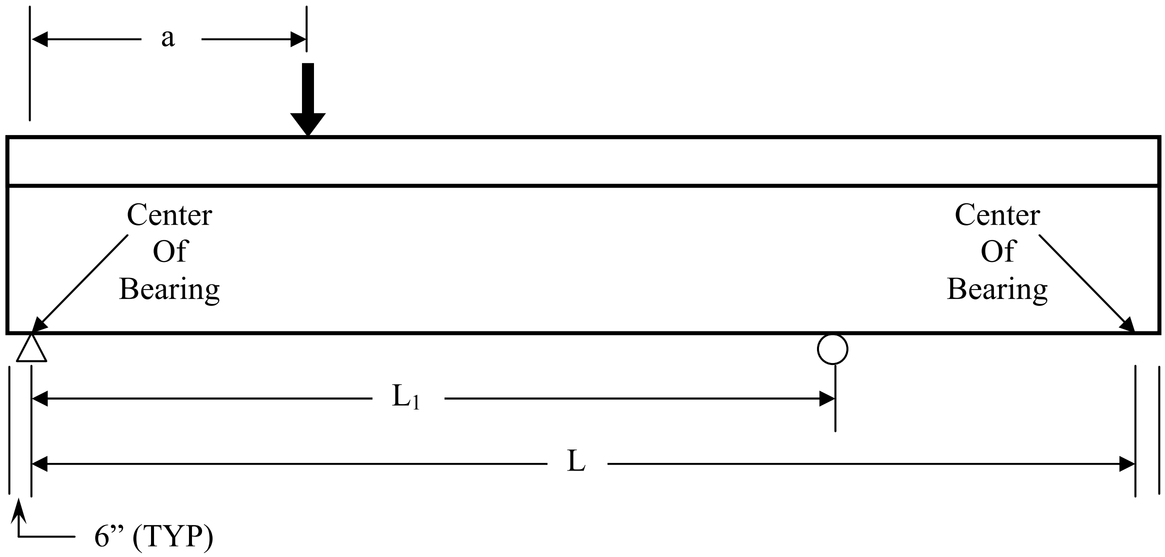


Fig. 3. Girder Layout Dimensions

TRANSFER LENGTH

Transfer length data for the bottom prestressing strands were measured using the Concrete Surface Strain (CSS) method[9]. As the prestressing strands transferred stress to the concrete, compressive stress and thus strain was induced in the concrete. Based on compatibility, the point at which the concrete strain reaches a maximum value indicates the transfer length.

To collect transfer length data for the bottom strands, detachable mechanical (DEMEC) gauge embedments were installed at 2-inch (50.8-mm) spacing at the level of the bottom strands over a 48-inch (1.22-m) distance from each girder end on each side of the girder. The 8-inch DEMEC gauge readings showed that surface strains stabilized 7 days after release.

Determination of the transfer length was a several step process involving the calculation of raw strains on each side of the girder, averaging the raw strain values between the sides, and then smoothing the averaged raw strain values using a floating three point average. From the smoothed strain profile, the transfer length was determined using the 95 Percent Average Maximum Strain Method (95% AMS). Both Buckner[11] and Russell[9] recommended this method as a conservative and objective technique for determining transfer length. Figure 4 shows the smoothed 14-day CSS profile for girder end G1C-East with the transfer length determined using the 95 percent AMS technique. Table 3 lists the measured transfer lengths from each girder end. The asterisk for three of the values indicates the girder ends which had over 25 feet (7.62 m) of free strand between the girder end and the abutment. Measured transfer lengths on those free ends were substantially longer in comparison to ends having only a small amount of free strand. This finding has been reported by others[9].

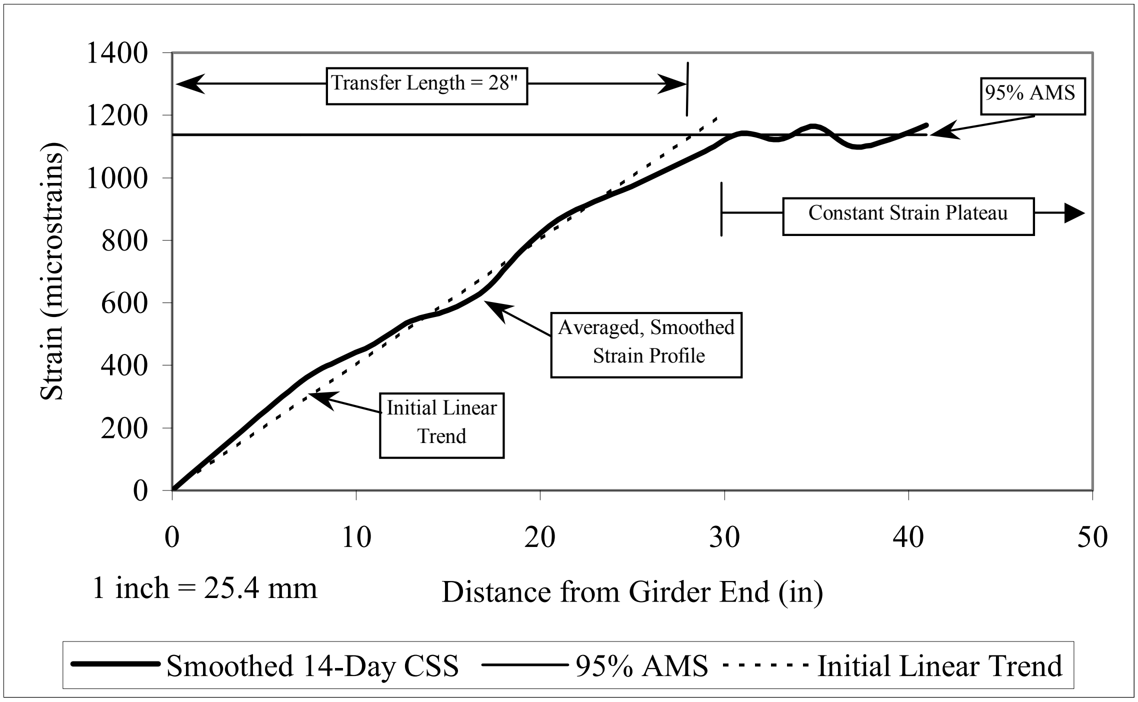


Fig. 4. Transfer Length Determined Using 95 Percent Average Maximum Strain Method for G1C-East

For the G1 girders with an initial concrete strength, f'_{ci}, of 7,080 psi (48.8 MPa), the mean transfer length, l_t, was 21.9 in. (556 mm). For the G2 girders with an f'_{ci} of 9,180 psi (63.3 MPa), the mean l_t was 15.6 in. (396 mm).

A comparison of current code provisions with the 12 HSLC transfer lengths obtained in this research showed the current ACI[13] and AASHTO[14] (Eqs. 1 and 2) to be conservative as presented in Table 3. The AASHTO equation overestimated transfer lengths by 37% on average and never underestimated transfer lengths. The ACI equation using VWSG data for determining actual strand stress overestimated transfer lengths by 41% on average and never underestimated transfer lengths. Use of either the ACI or AASHTO equations to predict transfer length for expanded slate HSLC was conservative.

Table 3. Comparison of Experimental Transfer Length with Code Predicted Values

Girder End	Experimental l_t (in)	AASHTO Predicted l_t (in)	Percent Diff. AASHTO vs. Exp.	f_{se} at 14-days + (ksi)	ACI Predicted l_t (in)	Percent Diff. ACI vs. Exp.
G1A-East	19.50	30	35	154.7	30.9	37
G1A-West	18.75	30	38	154.7	30.9	39
G1B-East	25.00 *	30	17	153.3	30.7	19
G1B-West	18.75	30	38	153.3	30.7	39
G1C-East	28.00 *	30	7	153.7	30.7	9
G1C-West	21.50	30	28	153.7	30.7	30
G2A-East	17.50 *	30	42	176.5	35.3	50
G2A-West	13.25	30	56	176.5	35.3	63
G2B-East	13.00	30	57	176.5	35.3	63
G2B-West	13.00	30	57	176.5	35.3	63
G2C-East	19.00	30	37	159.8	32.0	41
G2C-West	18.00	30	40	159.8	32.0	44
		Average	37		Average	41

Note: 1 inch = 25.4 mm, 1 ksi = 6.895 MPa
* Free end of strand
+ Based on measured strand force VWSG data

DEVELOPMENT LENGTH

Each end of the six Type II girders was tested for development length producing 12 sets of data. Girder end designations were the same as used in transfer length testing. In order to determine development length, the position of the point load was varied from girder to girder as detailed in Tables 2 and 4. The precise development length would be the location at which the point load resulted in concurrent flexural and bond failure. If the failure mode were purely flexural, the tested embedment length was greater than the development length. If the failure mode were bond or bond/shear, the tested embedment length was less than the development length. The load was placed at several locations between 70 and 100 percent of the predicted development length, l_d, to bracket the actual development length. The embedment length, l_e, was the shear span "*a*" plus the 6 inches (152 mm) from the center of bearing to the end of the girder. Table 4 shows the embedment lengths tested as a percentage of the estimated development length values and also shows the embedment lengths tested as a percentage of the development length calculated using Eq. 3 based on experimentally determined values of f_{su}^* and f_{se}, which are listed for each girder.

Table 4. Embedment Lengths Tested as Percentages of AASHTO Predicted Development Length Values

Girder End	Configuration and Stirrup Density	l_e (in)	l_e / l_d Based on Estimated Values of f_{su}^* and f_{se}	Actual Values f_{su}^* (ksi)	f_{se} (ksi)	l_d Based on Actual Values of f_{su}^* and f_{se} (in)	l_e / l_d Based on Actual Values of f_{su}^* and f_{se}
G1A-East	2 – Double	67	70 %	265.1	153.7	97.6	69 %
G1A-West	1 – Single	96	100 %	265.4	152.3	105.9	91 %
G1B-East	5 – Single	67	70 %	227.6	151.2	106.3	63 %
G1B-West	3 – Double	81	84 %	266.3	149.8	106.9	76 %
G1C-East	6 – Single	81	84 %	263.0	149.0	107.2	76 %
G1C-West	4 – Single	91	95 %	264.4	147.6	107.8	84 %
G2A-East	2 – Double	67	70 %	266.3	175.8	96.5	70 %
G2A-West	1 – Single	96	100 %	266.8	175.7	96.5	100 %
G2B-East	5 – Single	67	70 %	265.0	176.2	96.3	70 %
G2B-West	3 – Double	81	85 %	266.6	175.6	96.6	84 %
G2C-East	6 – Single	81	85 %	267.7	159.3	103.1	79 %
G2C-West	4 – Single	91	95 %	266.6	158.8	103.3	88 %

Note: 1 inch = 25.4 mm, 1 ksi = 6.9 MPa

External strain gauges were used to measure strain in the prestressing strand at the maximum moment location. Strand yielding was assumed to occur if the total strain in the strands exceeded 1 percent.

Strand slip was measured continually through each test using linear potentiometers. Failure from strand slipping was defined to occur if the average slip of the 8 bottom strands during the test exceeded 0.01 inches (0.25 mm) as defined by Russell[9] and Kahn et al.[18]. Typically the two exterior strands would slip more than interior strands if a slip failure occurred. This type of failure was termed "shear-slip" because it was believed that the formation of a shear crack was the trigger for strand slip to begin.

Table 5 provides the results of development length testing and includes failure modes. Shear cracking occurred in all tests. Since only selected embedment lengths were tested, it was not possible to exactly pinpoint the development length. It was possible, however, to identify the embedment length region in which the failure transitioned from flexure to shear-slip. The longer length of this region was conservatively identified as the development length. For G1 girders with single and double density stirrups, the development lengths were 91 and 81 inches (2.31 and 2.06 m), respectively. For G2 girders, all failed in flexure; so, the tested embedment lengths were all greater than the actual development length. The minimum embedment length of 67 inches (1.70 m) was identified as the experimental development length for the G2 girders, for both single and double density stirrups. Table 6 lists the experimentally determined development lengths.

Table 5. Development Length Test Results

Girder End	Stirrup Density	Strand Embedment (in)	Average Strand Slip at P_{max} (in)	Maximum Strand Strain ε_{ps} (in/in)	Maximum Deck Strain ε_{cu} (in/in)	Failure Mode FL-Flexure SH-Shear SL-Strand Slip
G1A-East	Double	67	0.0102	0.0158	0.0038	FL/SH-SL
G1A-West	Single	96	0.0000	0.0168	0.0042	FL
G1B-East	Single	67	0.7350	0.0078	0.0030	SH-SL
G1B-West	Double	81	0.0007	0.0190	0.0032	FL
G1C-East	Single	81	0.1988	0.0113	0.0032	SH-SL/FL
G1C-West	Single	91	0.0000	0.0140	0.0036	FL
G2A-East	Double	67	0.0033	0.0190	0.0036	FL
G2A-West	Single	96	0.0000	0.0204	0.0072	FL
G2B-East	Single	67	0.0065	0.0155	0.0045	FL
G2B-West	Double	81	0.0000	0.0199	0.0064	FL
G2C-East	Single	81	0.0041	0.0228	0.0045	FL
G2C-West	Single	91	0.0000	0.0198	0.0049	FL

Note: 1 inch = 25.4 mm

Table 6. Comparison of Experimental Development Length with Code Predicted Values

Girder End	Experimental l_d (in)	AASHTO, ACI Predicted l_d (in)	Percent Difference AASHTO, ACI vs. Experimental
G1A-East	81	98.0	17.3
G1A-West	91	98.8	7.9
G1B-East	91	99.2	8.3
G1B-West	81	99.6	18.7
G1C-East	91	100.0	9.0
G1C-West	91	100.4	9.4
G2A-East	67	89.2	24.9
G2A-West	67	89.2	24.9
G2B-East	67	89.2	24.9
G2B-West	67	89.2	24.9
G2C-East	67	96.0	30.2
G2C-West	67	96.0	30.2
		Average	19.2

Note: 1 inch = 25.4 mm

Examination of girder end tests having single stirrup density revealed that in every case a shear crack passing through the level of the bottom strands, within the transfer length region, initiated or dramatically increased strand slip. Tests incorporating double stirrup density showed the resulting slips were much less than for the single stirrup density tests. For the G1 series girders, doubling the stirrup density dramatically reduced slip values and increased the load at which significant slip occurred, especially in tests of shorter embedment lengths. For the G2 series girders, the effect was much less pronounced; however, the increased stirrup density did reduce end slip and cause slight increases in the load at which slip occurred.

The results clearly indicated that shear cracking across the bottom strands within the transfer length region initiated large increases in strand end slip in girders using only the AASHTO or ACI specified shear reinforcement. Russell[9] reported this phenomenon in normal weight concrete[9].

Concrete strength also played a significant role in the development length. The G1 series girders had compressive strengths on average approximately 1500 psi (10.3 MPa) less than the G2 series girders. A significant difference between the two mix designs was the amount of silica fume. The G1 series girders had 2 percent silica fume by weight of the total cementitious materials, and the G2 series girders had 10 percent. Past researchers have stated that the addition of silica fume will significantly improve the tensile strength and bond of lightweight concrete to prestressing strand[19-21]. The results of the present research confirm this observation.

COMPARISON OF DEVELOPMENT LENGTH RESULTS WITH VALUES PREDICTED BY CODE PROVISIONS

Experimental development length test results from this research were compared with development lengths predicted by ACI[13] and AASHTO[14] code provisions to determine their adequacy for design with expanded slate HSLC and are listed in Table 6. The ACI and AASHTO code provisions overestimated development lengths by 19 percent. Use of the current code provisions for design of development length for slate HSLC and 0.6-inch (15.2-mm) diameter strand was conservative. Based on the actual concrete strengths between 8,790 psi (60.6 MPa) and 11,010 psi (75.9 MPa) used in this research, modification of the current code specifications for development length was not necessary for expanded slate HSLC using 0.6-inch (15.2-mm) diameter strands.

PROPOSED TRANSFER AND DEVELOPMENT LENGTH EQUATIONS

Straight forward equations were developed to better estimate and design the transfer length for the 0.6-in. (15 mm) strand. To insure applicability over a wider range of concrete strengths, including both normal and lightweight concrete, transfer length data from other researchers[3, 5, 8-10, 16, 22, 23] were included. Evaluations showed that the combination of prestressing strand diameter, d_b, and concrete compressive strength at time of strand release, f_{ci}', provided the best prediction of transfer length. Approximately 125 transfer length data points based on the use of 0.6-in diameter prestressing strand having concrete compressive strengths from approximately 2,200 psi (15.2 MPa) through almost 15,000 psi (103 MPa) were included in this evaluation. In most cases, the tests involved I-shaped girders; however, tests by Mitchell[10] and Russell[9] used rectangular cross sections.

In developing an equation, it was desirable to determine two equations, one that best fit the data and another that was more suitable for design. A “best fit” equation was defined as an equation that had the lowest average percent difference between experimental and predicted values. A “design” equation was defined as the equation having the lowest positive maximum underestimate value without being overly conservative. Eqs. (4) and (5) were developed and found to provide a better fit to the scatter of data points according to the “best fit” and “design” definitions, respectively.

$$50\, d_b \sqrt{\frac{4000}{f'_{ci}}} \tag{4}$$

$$50\, d_b \sqrt{\frac{6000}{f'_{ci}}} \tag{5}$$

Fig. 5 shows Eqs. (4) and (5) plotted, respectively, against the experimental transfer length data points. The rectangular shapes tested by Russell[9] were released simultaneously by flame cutting. Specimens by Mitchell[10] were released gradually which was reported to more closely match transfer lengths recorded under similar conditions in I-shaped girders. When evaluating Eqs. (5) and (6), the rectangular shapes from Russell’s testing were not included.

Improved development length equations incorporated d_b, f_c', the term $(f_{su}-f_{se})$, the term $(f_{ps}-f_{se})$, and the transfer length relation. Data from the current tests plus that reported by Kahn et al.[18] on 0.6-in. (15 mm) strand in normal weight HPC was used. All strand stress values were based on VWSG data. Eqs. (6) and (7) provided the most promising “best fit” and “design” equations, respectively.

$$\left(50\sqrt{\frac{2500}{f_{ci}'}} + f_{ps} - f_{se}\right) d_b \tag{6}$$

$$\left(50\sqrt{\frac{5000}{f_{ci}'}} + f_{ps} - f_{se}\right) d_b \tag{7}$$

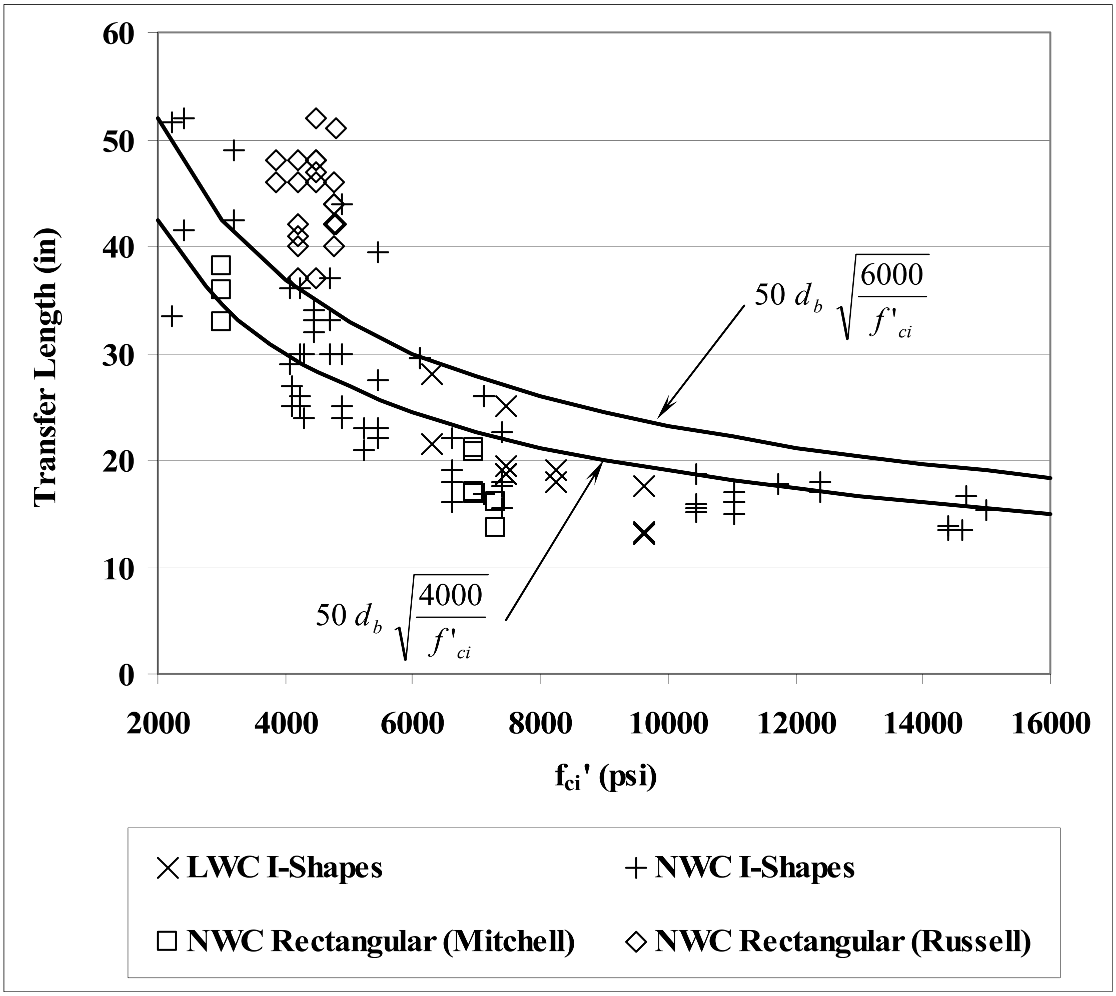


Figure 5 – Eqs. (5) "Best Fit" and (6) "Design" Plotted Against Transfer Length Data

Table 7 presents an evaluation of Eqs. (6) and (7) for the 12 development length tests on HSLC and the 8 tests on normal weight concrete. The "best-fit" equation has the smallest percent difference and the most promising "design" equation has the smallest positive maximum under length prediction. The initial term of each equation is very similar to the equations for transfer length. Comparison and evaluation indicates that the transfer length portion of the equation is apparently reduced in the development length equation. This trend was seen during this research; as girders were loaded, the strands tended to transfer more force in a region somewhat shorter than the specified transfer length.

Table 7. Results of Evaluation of Proposed Development Length Equations

Basis of Comparison	AASHTO, ACI Code $(f_{ps}-\frac{2}{3}f_{se})d_b$	Proposed Design Equation $\left(50\sqrt{\frac{5000}{f'_{ci}}}+f_{ps}-f_{se}\right)d_b$	Proposed Best Fit Equation $\left(50\sqrt{\frac{2500}{f'_{ci}}}+f_{ps}-f_{se}\right)d_b$
Average Diff (HSLC)	19%	13%	4%
Maximum Over (HSLC)	30%	31%	21%
Maximum Under (HSLC)	8%	2%	-6%
Average Diff (NWC)	18%	2%	-5%
Maximum Over (NWC)	18%	4%	-3%
Maximum Under (NWC)	17%	0%	-6%
Average Diff (Overall)	19%	9%	1%
Maximum Over (Overall)	30%	31%	21%
Maximum Under (Overall)	8%	0%	-6%

CONCLUSIONS AND RECOMMENDATIONS

Current ACI[13] and AASHTO[14] provisions for transfer and development length of 0.6-inch (15.2-mm) prestressing strand are conservative for expanded slate high strength lightweight concrete. The AASHTO equation overestimated transfer lengths by 37% on average and never underestimated transfer lengths. The ACI equation with f_{se} determined using VWSG data overestimated transfer lengths by 41% and never underestimated transfer lengths. The code equations overestimated development lengths by 19 percent and never underestimated them. Based on the HSLC strengths between 8,790 psi (60.6 MPa) and 11,010 psi (75.9 MPa) used in this research project, modification of the current code specifications for transfer and development length is not necessary for expanded slate HSLC using 0.6-inch (15.2-mm) diameter strands. There was no indication throughout this analysis that need existed to differentiate between slate HSLC and normal weight HPC.

Test results showed that shear cracking in the transfer length region across the bottom strands did not induce significant strand slip if stirrup density were doubled over the current AASHTO[14] specified stirrup spacing in the transfer length region. Further investigations should consider doubling the shear reinforcement requirement in the transfer length region in order to limit strand slip in the event of shear cracking.

The addition of silica fume into the mix design at a 10% replacement rate by weight of cementitious materials appeared to reduce development length. Further research should investigate the effectiveness of silica fume for increasing bond.

An evaluation of 32 possible equation forms showed that d_b and f_{ci}' were the best parameters for predicting transfer length. Eqs. (4) and (5) for predicting transfer length based on "best fit" and "design" produced more accurate results than current code equations.

A similar evaluation yielded Eqs. (6) and (7) for better predicting development length in high-strength concrete.

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