Establishment of minimum acceptance criterion for strand bond as measured by ASTM A1081

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- ASTM International recently adopted ASTM A1081, *Standard Test Method for Evaluating Bond of Seven-Wire Steel Prestressing Strand*, a pull-out test procedure developed for verifying the ability of steel strands to bond to cementitious materials before their use as tensile reinforcement in prestressed concrete sections.

- PCI commissioned a study to determine a minimum ASTM A1081 bond threshold value for steel strand to be used in pretensioned applications.

- The results showed that it would take an ASTM A1081 pull-out value of 28,200 lb (125 kN) to have a 90% confidence that 95% of beams with a single strand would have a transfer length smaller than or equal to the ACI 318-14 calculated transfer length.

- The results also showed that an ASTM A1081 pull-out value of 14,600 lb (64.9 kN) would have a 90% confidence that 95% of beams containing at least six strands would have a moment capacity greater than or equal to the ACI 318-14 calculated nominal moment capacity.

Pretensioned concrete members rely on the bond between prestressing steel strands and concrete to transfer forces during the prestressing operation. This bond is provided by chemical adhesion before any slippage between the strand and concrete occurs, by friction between the strand and concrete after slippage occurs, and by the Hoyer effect. The Hoyer effect is the additional bond force that occurs because the strand wire diameters revert back to their original diameters from their lower stressed diameters after the prestressing strands are cut during manufacture. The concrete-strand bond alone cannot transfer all of the prestressing forces from the steel to the concrete at the end of the beam. It takes some distance from the beam end for the strand to fully transfer all of its forces from prestressing to the concrete. This distance is known as the transfer length L_t. The length from the end of the beam that is required for the steel to develop the full member design strength is called the development length L_d. **Figure 1** shows the idealized steel stress as a function of the distance from the end of the beam.

The American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*[1] gives predictive equations for prestressing steel transfer and development lengths, as shown in Eq. (1) and (2), respectively:

$$L_t = \left(\frac{f_{se}}{3}\right) d_b \qquad (1)$$

$$L_t = \left(\frac{f_{se}}{3}\right) d_b + \left(f_{ps} - f_{se}\right) d_b \qquad (2)$$

where

f_{se} = effective stress in prestressing strand

d_b = nominal strand diameter

f_{ps} = stress in prestressing strand at nominal strength

A structural engineer designs according to the code, assuming that the strand is fully anchored at a distance equal to the predicted development length from the beam end, with a reduced steel tensile capacity and consequent reduced moment capacity within the development length zone. The reduced prestressing force within the transfer-length zone also reduces aggregate interlock and shear capacity. Reduced shear-capacity contribution from the concrete due to reduced bond within the transfer length to the concrete can be compensated by the addition of stirrups to the member at the end. However, failures can occur when the actual transfer length is much longer than the predicted transfer length used in the member design, giving a lower shear capacity than was predicted.

The transfer length and development length depend not only on the prestressing forces but also on the concrete and steel properties. These properties can be variable and depend on the concrete strength, strand manufacturing process, and surface residue.[2] The prestressing industry wants to establish a minimum level of acceptable bond surface characteristics provided by the prestressing steel. A new index test method to measure prestressing steel bond characteristics was recently approved as ASTM A1081.[3]

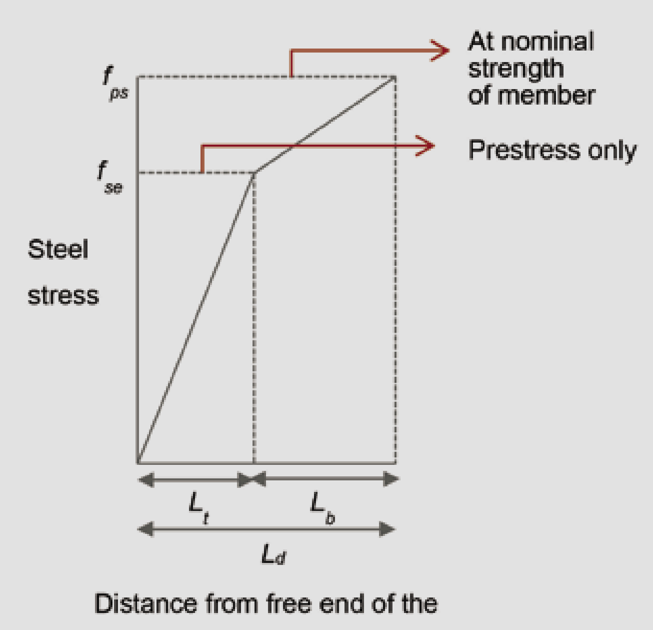

Figure 1. Variation of steel stress along development length. Note: f_{ps} = stress in prestressing strand at nominal strength; f_{se} = effective stress in prestressing strand; L_b = bond length; L_d = development length; L_t = transfer length.

This new method measures the pull-out strength of prestressing strand embedded in a 5 in. (130 mm) diameter mortar specimen encased in a steel tube. The pull-out strength is the force it takes to displace the strand 0.10 in. (2.5 mm) at the free end of the specimen by pulling on the strand at the opposite end. A study was initiated to correlate the pull-out strength, obtained from the ASTM A1081 method, to the transfer and development lengths measured in concrete beams manufactured with the same strands. Another goal of the study was to establish the steel bond characteristics required to achieve the transfer and development lengths predicted by Eq. (1) and (2). Three different strands with different levels of bond characteristics used previously in an interlaboratory study of ASTM 1081 were used in this study.[4] The beams described in this paper were made using the same dimensions, material sources, and equipment used in the bond study performed by Logan.[5] After the beams were fabricated, the transfer lengths were determined from concrete surface strains obtained using a noncontact method developed by Zhao et al.,[6] and also from measurements of strand end slip. The beam development lengths were not measured directly but inferred by systematically load testing the beams to failure with the applied load located within the development-length zone calculated according to ACI 318-14.

Materials

The three different prestressing strand sources used in this study were from the same strand packs that were also used in the interlaboratory study on ASTM A1081. The three strand-sample sources for this study were labeled A, G, and I. The letter corresponds to the order in which they were received at the laboratory. **Table 1** shows the average ASTM A1081 pull-out values found for each strand sample, along with the standard deviation and coefficient of variation found in the interlaboratory study.

Table 1. Average pull-out test result, standard deviation, and coefficient of variation for strands A, G, and I as measured using ASTM A1081[7] from 11 different complete ASTM A1081 tests

	Strand A	Strand G	Strand I
Average pull-out force, lb	13,500	17,700	11,600
Standard deviation	1903	2728	1543
Coefficient of variation	0.14	0.15	0.13

Note: 1 lb = 4.448 N.

The concrete mixture used during the beam study was designed to match the low-slump concrete mixture that was used by Logan.[5] The objective was to use a concrete mixture similar to what would be used in wet-cast hollow-core and double-tee production. The concrete mixture was designed for a concrete compressive strength at the time of detensioning f'_{ci} of 3500 psi (24 MPa) and a 28-day compressive strength f'_c of 6000 psi (41 MPa). Siliceous natural coarse and fine aggregate were used in the concrete mixture. The admixture used was an ASTM C494[8] Types B and D water-reducing and retarding admixture and was used to prevent slump loss during placement, consistent with the study by Logan. **Table 2** shows the mixture proportions.

Methodology

Beam specimen design and fabrication

The beams tested had nominal dimensions of 6½ × 12 in. (165 × 300 mm). The sections were reinforced with a single 0.5 in. (13 mm) diameter prestressing strand placed in the middle of the beam's width and at a distance of 10 in. (250 mm) from the top (as-cast) face of the section. The strand was initially tensioned to 75% of the ultimate tensile strength ($0.75f_{pu}$). Ten 18 ft (5.5 m) long beams were fabricated for each of the strand sources, A, G, and I.

Each of the pretensioned beams was load-tested to failure, in three-point bending, at both ends. Because each beam was tested twice, shear reinforcement was placed in the portion of each beam located in the tested span for both tests, but not between the loaded point and the adjacent end of the beam. The beams were loaded at one end, at a distance of 80% of the calculated ACI 318-14 development length L_d from its end (long end), and on the opposite side at a distance of 60% of the calculated development length from the end (short end).

The PCI advisory group chose loading-point locations to increase the potential for bond failure of the strand, thus allowing extrapolation to the actual development length of the strand. **Figure 2** shows the loading configuration details for the short and long ends of each beam. One side of each beam was loaded first, and then the beams were moved and prepared for their opposite end to be tested.

Table 2. Concrete mixture design

Material	Quantity
Type III cement, lb/yd^3	658
Water, lb/yd^3	322
Fine aggregate, lb/yd^3	1081
Coarse aggregate, lb/yd^3	1876
Water reducing and retarding admixture, oz/cwt	4

Note: 1 lb/yd^3 = 0.593 kg/m^3; 1 oz/cwt = 0.652 mL/kg cement.

Flexural beam sections were reinforced with welded-wire reinforcement (WWR) for shear between the two load application points to prevent failure in this region and thus allow the middle portion of the beam to be used for testing both ends. Similar to Logan's study, two layers of stem WWR were used, measuring 9 in. (230 mm) deep and 9 ft 7½ in. (2.93 m) long.[5]

Because the primary objective of the beam testing program was to evaluate the bond performance of the strands, care was taken not to have any lifting devices for the beam come in close proximity to the strands. Threaded ferrule loop inserts with a ¾ in. (19 mm) diameter were cast into the top surface, one per end, in the fresh concrete. The insert assemblies were only 6 in. (150 mm) in height, which ensured that they would not be near the strand that was located at a depth of 10 in. (250 mm). Eyebolts were then threaded into the inserts to allow for attachment of appropriate rigging.

All 30 beams were fabricated on a single prestressed concrete bed over three days (10 per day). Two wooden forms were used, and each was long enough to cast five 18 ft (5.5 m) long beam sections and two 4 ft (1.2 m) long dummy blocks, one on each end. The dummy blocks were cast to ensure that all beam ends experienced the same type of strand release mechanism during saw cutting.

On the first day of concrete placement, the first five strand A beams and the first five strand I beams were fabricated. On the second day of concrete placement, the second set of five strand A beams were cast along with the first five strand G beams. The second sets of strand G and strand I beams were fabricated on the third day of concrete placement.

The 30 beams were fabricated in groups of 10 on three different days, with concrete being placed at approximately 5:00 a.m. on placement days 1 and 2, and 9:00 a.m. on placement day 3. The concrete cylinders used to measure the concrete strength gain were match cured to the concrete beam temperature until the prestressing was released. **Table 3** summarizes the concrete mixture properties and placement conditions for each cast day. The beam specimens were covered and cured in their fabricating forms until their companion temperature-match-cured concrete cylinders reached the specified compressive strength of 3500 psi (24 MPa). Each individual beam section was saw cut from its strand line and moved to a nearby location, where the initial end slip and surface strain readings were measured.

The beams were allowed to cure outside for approximately 21 days before flexural testing, as previously done in Logan's study.[5] Because the specimens were fabricated on

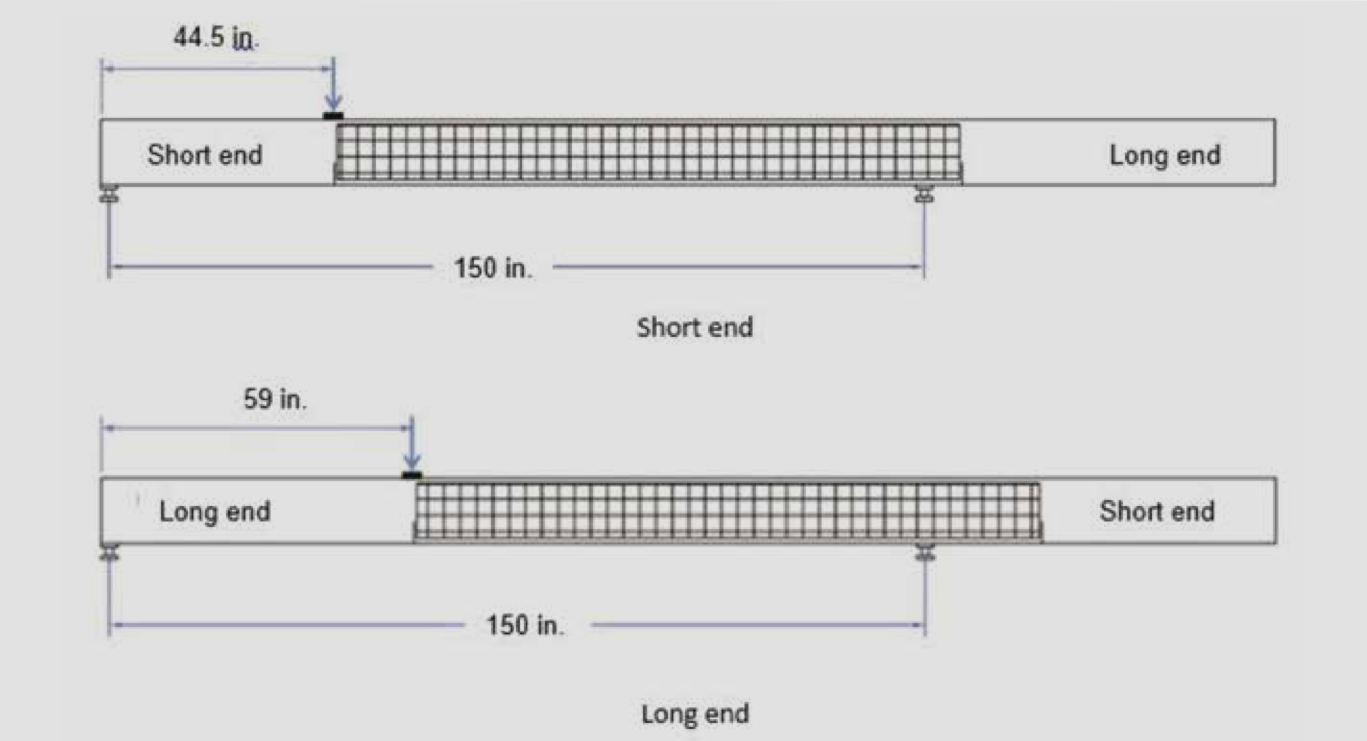

Figure 2. Loading configuration. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

three different days between July 16 and July 19, 2013, and tested between August 6 and August 10, 2013, the concrete age of the beam groups varied slightly, ranging from 19 to 23 days. The concrete beams were stored outside during this time. The specimens are named by the strand used to make the beam, the beam number, and L or S. The letter L is used to designate the beam end tested with the load placed at 80% of the ACI 318-14–calculated development length. The letter S is used to designate the beam end tested with the load placed at 60% of the ACI 318-14–calculated development length.

Table 3. Concrete placement conditions and mixture properties per cast day

Property	Day 1	Day 2	Day 3
Date	7/16/13	7/18/13	7/19/13
Air content, %	1.6	1.4	1.5
Slump, in.	3.25	3.25	3.5
Unit weight, lb/ft^3	145.6	145.6	146.0
Concrete temperature, °F	74	75	78
Ambient temperature, °F	53	57	69
Average release strength, psi	3860	3680	3880
Compressive strength at 21 days, psi	6690	6270	5800

Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 lb = 4.448 N; 1 psi = 6.895 kPa; °C = (°F – 32)/1.8.

Crack initiators

The PCI advisory group originally decided that 4 in. (100 mm) galvanized steel crack initiators should be provided at the lower part of every beam below both loading points to ensure consistent crack initiation locations for all beams. This also helps prevent high concrete strength from masking poor strand bond in the tests. The galvanized crack formers were covered with cloth tape to assist with debonding from the concrete material. Because of the relatively large cross section of the beams in this study and the fact that the beams were being loaded in three-point bending at an embedment length that was only 60% and 80% of the calculated development length, it would be unlikely that multiple flexural cracks would develop before reaching the nominal flexural capacity. The bilinear approximation in ACI 318-14 assumes that flexural cracking will occur in the flexural bond region. As stated by Russell and Burns, "The average flexural bond stress is lower because flexural cracking occurs within the development length and disturbs bonding between steel and concrete, thereby reducing bond strength."[9]

Unfortunately, after detensioning the beam specimens that were cast on day 1, higher surface-strain readings were observed near the vicinity of the crack formers. The likely reason for this is that the cloth tape allowed for additional localized compression of the pretensioned member. Because the surface-strain measurements were used to determine the initial transfer lengths of the beams, it was decided not to use crack initiators for the subsequent beam specimens cast on days 2 and 3.

Table 4. Crack-inducing techniques and cracking moments per beam end							
Crack-inducing technique	**A-L**	**A-S**	**G-L**	**G-S**	**I-L**	**I-S**	**Estimated cracking moment M_{cr}, kip-in.**
4 in. crack former at 43 in. from beam end	n/a	1 to 4	n/a	n/a	n/a	1 to 5	209
4 in. crack former at 57.5 in. from beam end	1 to 5	n/a	n/a	n/a	n/a	1 to 5	209
4 in. crack former at 43 in. from beam end and 1⅜ in. saw cuts on bottom + 1 in. deep side cuts at 33 in. from beam end	n/a	5	n/a	n/a	n/a	n/a	209 at point load, 228 at other saw cut locations
4 in. crack former at 57.5 in. from beam end, 1⅜ in. saw cuts on bottom + 1 in. deep side cuts at 47.5 in. from beam end, and 1⅜ in. saw cuts on bottom + 1 in. deep side cuts at 37.5 in. from beam end	n/a	n/a	n/a	n/a	5	n/a	209 at point load, 228 at other saw cut locations
One 1 in. saw cut at 43 in. from beam end	n/a	6	n/a	1 to 4, 6	n/a	n/a	250
One 1 in. saw cut at 57.5 in. from beam end	n/a	n/a	1 to 5	n/a	6	n/a	250
One ⅜ in. saw cut at 43 in. from beam end and one ⅜ in. saw cut at 33 in. from beam end on beam bottom	n/a	8 to 10	n/a	8 to 10	n/a	8 to 10	247
One ⅜ in. saw cut at 57.5 in. from beam end, one ⅜ in. saw cut at 47.5 in. from beam end, and 1⅜ in. saw cut at 37.5 in. from beam end on beam bottom	8 to 10	n/a	8-10	n/a	8 to 10	n/a	247
One ⅜ in. saw cut + 1 in. side cuts at 43 in. from beam end and 1⅜ in. saw cuts + 1 in. side cuts at 33 in. from beam end	n/a	n/a	n/a	5	n/a	6	228
One ⅜ in. saw cuts + 1 in. side cuts at 57.5 in. from beam end, one ⅜ in. saw cuts + 1 in. side cuts at 47.5 in. from beam end, and 1⅜ in. saw cuts + 1 in. side cuts at 37.5 in. from beam end	6	n/a	6	n/a	n/a	n/a	n/a
None	7	7	7	7	7	7	260

Note: n/a = not applicable. 1 in = 25.4 mm; 1 kip = 4.448 kN.

After the first day of flexural beam testing, when four of the 10 beams showed little difference in performance, saw-cut notches were added to the beams to induce cracking. Notches were made by saw cutting the bottoms of the beams in order to lower the cracking moment at these locations and hopefully allow additional flexural cracks to develop before the nominal moment capacity was reached. The concrete cross-sectional area after saw cutting does not affect the calculated moment capacity because the concrete beam ultimate capacity is determined assuming cracking in the tension zone.

The additional saw cuts were made at increments of 10 in. (250 mm) from the loading point toward the nearest end of the beam, but in no case were the saw cuts allowed to be made within the calculated ACI 318-14 transfer length of 30.7 in. (780 mm). A saw-cut spacing of 10 in. was selected because cracks tend to develop at a spacing equal to the distance *d*. When cracking within the transfer zone can be avoided, the strands should be expected to develop to their full tension capacity.[9] The saw cuts were made by placing the beams on large steel supports and using a concrete saw equipped with a diamond blade.

Table 4 lists the different crack initiation techniques used in this research program for each beam end and the estimated cracking moments for each beam crack configuration. The cracking moments were estimated using a modulus of rupture of 581 psi (4010 kPa), which corresponds to a compressive strength of 6000 psi (41 MPa). The table lists the numbered beam ends (1–10), the letter L indicates the ends that were loaded at a distance of 80% of the ACI 318-14 development length (long ends), and the letter S indicates the ends that were loaded at a distance of 60% of the ACI 318-14 development length (short ends). For some of the sections there was only one 1 in. (25 mm) deep saw cut implemented on the bottom face of the beam. These were the initial strand G beams that were replicating the steel crack initiators used in the strand A and strand I beams cast on day 1.

Transfer-length measurement

Transfer lengths were determined immediately after prestress release (initial transfer lengths) and also before load testing at 19 to 23 days after beam fabrication. Transfer lengths were determined by analyzing surface-strain readings combined with strand end-slip measurements. End-slip measurements were recorded immediately after the prestressing force was released and the beam was removed from the form and supported by wood blocks as well as before flexural testing of each beam. However, extreme abrasion on the saw-cut surface due to the non-uniform saw cutting (**Fig. 3**) rendered the initial end-slip measurements unreliable. Because of shear lag, it is expected that end-slip measurement would give slightly longer transfer lengths than surface-strain measurements. In this study, this effect is expected to be minimal.

A noncontact laser-speckle imaging device was used to measure concrete surface strains before and after detensioning to determine the transfer lengths immediately after strand release.[10] This device had an effective gauge length of 6 in. (150 mm) and relied on digital image correlation of the concrete surface before and after displacement. The accuracy of the laser-speckle imaging system was previously shown to be approximately ±20 microstrain in similar applications. To facilitate image correlation, a speckled paint was applied to the concrete surface before release to enhance the optical surface features of the concrete. Surface-strain measurement comparisons were made between laser-speckle imaging and measurements taken using a mechanical strain gauge in a previous study.[6] The laser-speckle imaging surface-strain measurements were found to be more accurate and more reliable than the measurements taken manually using the mechanical strain gauge.

Figure 3. Extreme abrasions due to saw cutting on beam end.

Because the beams were stored outdoors for three weeks before testing, the painted areas were covered with plastic to preserve the surface features. Unfortunately, despite these efforts, the paint features had changed due to trapped moisture and long-term image correlation was not possible in most cases.

Instead, the transfer-length values corresponding to the time of flexural beam testing were determined indirectly by adding the implied transfer-length increase (from strand end-slip growth) to the initial transfer-length values, which were determined from the laser-speckle imaging system at the time of prestress release. The increase in strand transfer length between the time of prestress release and the time of flexural beam testing was estimated using Mast's strand-slip theory as presented by Logan.[5] Strand end-slip readings were taken using a digital length indicator with the indicator needle placed on the flat portion of the strand center wire. The increase in transfer length can be calculated using Eq. (3).

$$\Delta L_t = \Delta S\left(\frac{2E_{ps}}{f_{si}}\right) = 288\Delta S \tag{3}$$

where

ΔL_t = increase in transfer length

ΔS = increase in end slip

E_{ps} = strand elastic modulus

f_{si} = initial stress in strand before long-term losses

To accommodate the laser-speckle imaging device, threaded inserts were cast into the side of the concrete surface and then used to secure aluminum mounting blocks. The aluminum blocks ensured that the laser-speckle imaging device could be consistently repositioned on the beam face so that images would be taken at exactly the same locations before and after detensioning. The same aluminum blocks were also used to mount a cardboard template for spraying the concrete surface with speckled paint, which enhanced the surface features of the concrete for better image correlation.

While the device moved along the concrete beam surface, two lasers illuminated the surface (and particles in the paint). The lasers and corresponding cameras were located 6 in. (150 mm) apart and digital images of the concrete surface were recorded before detensioning and compared with similar images occurring after detensioning.

Transfer-length values were obtained by two methods of analysis of the initial surface-strain readings. The first method is the 95% average maximum strain method,[9] which is commonly used in transfer-length studies. The transfer lengths calculated by the 95% average maximum strain method are determined when smoothed surface-strain data are plotted, and the operator determines the strain profile plateau by observation and judgment. The average maximum strain value is determined by taking the average of all the strain values that lie on the plateau. This value is then multiplied by 0.95 to determine the 95% average maximum strain. The point where the 95% average maximum strain line intersects the smoothed strain profile, which is a distance from the end of the beam, is then determined to be the transfer length.[9]

The second method used in this study is a statistics-based process, which uses a least-squares algorithm to minimize the difference between a best-fit bilinear strain profile and the raw data. This method is an unbiased process of strain profile analysis and was confirmed by previous researchers to provide faster, more accurate, and more reliable transfer-length values compared with the 95% average maximum strain method.[11] While the 95% average maximum strain method requires manual implementation by the operator in order to yield transfer-length values, the statistics-based process method software generates the values automatically, taking the random error due to the strain sensor into account and eliminating the potential bias or human error involved in the manual determination of the strain profile plateau.[11]

Figure 4 shows the analysis results for the initial surface-strain data (immediately after detensioning) obtained for specimen I9-S by the 95% average maximum strain and statistics-based process methods, respectively. Figure 4 shows the horizontal 95% average maximum strain line and the best-fit bilinear strain distribution determined using the statistics-based process method. The statistics-based process method assumes a localized bilinear strain profile. However, the idealized strain profile plotted in Fig. 4 has a small curved portion between the two linear segments because the raw data would also theoretically have a rounding effect in this region. This rounding is due to the 6 in. (150 mm)

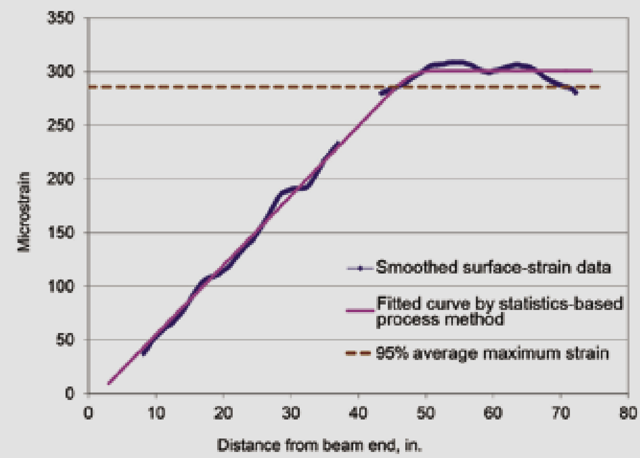

Figure 4. Analysis of specimen I9-S by statistics-based process method (transfer length = 47.9 in.). Note: 1 in. = 25.4 mm.

gauge length causing some averaging of the strain data in the ramp and plateau regions that is accounted for in the statistics-based process optimization procedure.

Load testing

A special load-testing frame was designed and fabricated for this study. The frame was fabricated by clamping a steel reaction beam onto two 7 × 7 × 2.5 ft (2.1 × 2.1 × 0.76 m) concrete blocks, which each weighed over 17,000 lb (76 kN) and provided enough self-weight to resist the required reaction load. The 21 in. (530 mm) deep steel beam was long enough to allow for six test specimens to be installed into the frame at one time with enough space between beams to provide the necessary working room. The specimens were loaded using a hydraulic cylinder. A special trolley-type support was designed and fabricated for the actuator, which allowed it to be quickly moved between the beams (**Fig. 5**).

A servo-hydraulic controller provided closed-loop control for all 30 beam tests. During the testing, the load, deflection of the beams at the loading point, and strand end slip were continuously recorded using a data acquisition

Figure 5. Beam load testing setup.

system. The servo-hydraulic controller and data acquisition system were housed in a mobile laboratory, which provided the required climate control and protection from the elements for the load control and data acquisition system during the week of outdoor testing. Linear variable differential transformers were used to measure beam deflection and strand end slip during the tests.

Steel plates measuring 3 × ¾ in. (75 × 19 mm) were secured to the top surface of each beam at the loading points using a gypsum product to distribute the load over the width of the beams and provide consistent loading conditions. The same plates extended laterally past the edges of the beam and were connected to the linear variable differential transformers to measure the deflection of the beam. Three-inch (75 mm) steel plates were also grouted to the bottom surface of each beam using the gypsum product at the support locations. The support that was located nearest the loading point was a roller, while the support at the other end of the loading span was a pin. The grouted pin and roller supports allowed for accurate measurements of the unrestrained beam deflection during testing.

Loading was applied to the beams at a rate of 1000 lb/min (4400 N/min) up to 7000 lb (31 kN). The loading rate was then reduced to 250 lb/min (1100 N/min) until failure occurred. During the loading, the researchers observed the beam for the initiation of cracks near the loading point.

During the first two days of testing, the loading procedure was paused and the applied load on the beam was held constant whenever strand end slip was detected (by the end-slip linear variable differential transformer). This was done to determine whether the loading rate has a significant effect on the observed strand-slip rate. This load level was then held constant until no additional end slip occurred (or less than the detectable amount of 0.001 in. [0.03 mm]) for one minute. If no additional end slip was detected, loading was increased at the aforementioned loading rates until additional slip occurred, at which point another load holding period was reinitiated. This procedure resulted in some long loading times (greater than 45 minutes) for some of the initial beams, especially those containing I strands because they tended to slip more than beams made with strands A or G. After the first two days of loading, the beams were loaded at the prescribed loading rates and without holding periods unless large end slip occurred, at which point the load was held until the end slip and deflections stabilized.

Data collected from the flexural beam testing included load, strand end slip, beam deflection, and the load at first visual detection of cracking. Each test was video recorded, and a description of each failure mode was also made.

Results

Transfer lengths

Table 5 shows the transfer lengths measured at release and at the time of beam testing as calculated using either the 95% average maximum strain method or the statistics-based process method. The two methods give similar transfer lengths. A considerable amount of transfer-length growth was seen in all three strands between release and the time of beam flexural testing. None of the strands showed an average transfer length less than or equal to the ACI 318-14–calculated transfer length.

Beam structural response

During the load testing of beams 1 through 4, a consistent trend was observed in which the beams with strand I did not develop as many cracks as the beams containing strands A and G. Typically, for the 60% L_d tests, the beams with strand I developed only one flexural crack under the loading point, which led to considerable strand end slip (sometimes more than 1 in. [25 mm] of additional slip with the resulting large crack width), and yet the beams ultimately failed by strand rupture. However, beams with strands A and G typically developed a second flexural crack at about 10 to 12 in. (250 to 300 mm) from the loading point toward the nearest end of the beam. As load was increased, this crack would then turn into a flexure-shear crack and often result in a shear-compression failure at a lower load than the beams with strand I.

Because strand I was the lowest bonding of the three strands, it was surmised that the bond of strand I was not sufficient to cause a second crack to develop. When

Table 5. Average transfer-length values at release for strands A, G, and I by 95% average maximum strain and statistics-based process method analysis

Strand source	Average transfer length at release, in.		Average transfer length at time of test by statistics-based process method and Mast's strand-slip theory, in.
	95% average maximum strain method	Statistics-based process method	
A	34.6	35.8	48.5
G	27.4	28.1	37.7
I	39.6	42.2	54.7

Note: 1 in = 25.4 mm.

flexural cracking of reinforced concrete occurs, tension in the concrete is transferred to the steel reinforcement, and at the crack location, nearly all of the tension in the member is carried by the steel. As the longitudinal distance from the initial crack increases, the tension in the concrete also increases as a function of the bond between the steel and concrete. At some distance from the initial crack, if the tension in the concrete reaches the cracking moment of the section, an additional flexural crack can initiate. However, because the experimental moment of beams (1S through 4S) exceeded the calculated nominal moment capacity of the section, in many cases for strands A and G a second crack did open as the peak moment increased beyond the theoretical nominal moment capacity. However, presumably because of the weaker bond of strand I, no additional cracks (other than the one under the loading point) opened in these initial beam tests, even though the experimental moment for beams I1-S through I4-S was at least 30% higher than the calculated ACI 318-14 nominal moment capacity of 308.3 kip-in. (34.83 kN-m).

Figure 6 plots the first observed crack data with the ratio of embedment length to transfer length at the time of testing. First, the observed cracking for members that are tested with embedment lengths that are greater than the implied transfer length at the time of testing typically occurs at a higher load than the calculated cracking moment (about 1.2 times the calculated cracking load). This is likely because the tensile strength of the concrete was greater than the assumed 581 psi (4010 kPa).There was also a lag time between the actual initiation of cracking and the visual observation of the cracking from a distance of several feet away, calling out of the cracking, and reading and recording the corresponding load value. Second, when the members were tested with an implied transfer length that was more than the embedment length, there was a reduction in the first observed cracking load. Furthermore, this was most prevalent for the beams with strand I and to a lesser degree for those with strand A. It is thought that these early first cracking loads are confirmation of the long transfer lengths that were noted at the time of testing (based on surface-strain readings plus additional strand end slip between detensioning and testing).

There were eight beam ends (out of 20 total) for each strand group that contained more than one crack initiation point. Of these, none of the strand G beam ends failed below the ACI 318-14 nominal moment capacity, one of the eight strand A beam ends failed below the ACI 318-14 nominal moment capacity (beam A10-L), and four of the eight strand I beams failed below the ACI 318-14 nominal moment capacity (beams I5-L, I9-S, I9-L, and I10-L).

Beams A10-L, I5-L, and I10-L were particularly noteworthy. All of them had transfer lengths at the time of testing that were significantly greater than the calculated ACI 318-14 value of 30.7 in. (780 mm), and all three failed by a flexure-shear crack that initiated at the third crack initiation point that was 20 in. (510 mm) from the loading point. **Figure 7** shows the measured moment-deflection relationship for beam I5-L and a picture of the cracks after failure seen for specimen I5-L. Several observations can be made regarding these beam tests. First, the fact that cracking did not fully materialize at the saw cut, which was only 10 in. (250 mm) from the loading point, suggests that the bond between the steel and concrete was not sufficient to produce a tensile stress in the concrete at this location that was in excess of the cracking moment. Second, the only way to calculate a cracking moment that is less than the applied moment at this location is if there is a significant reduction in the prestressing force, which is consistent with the long transfer lengths indicated for these beams at the time of testing.

The interpretation of flexural beam testing results was made by comparing the maximum moment that was with-

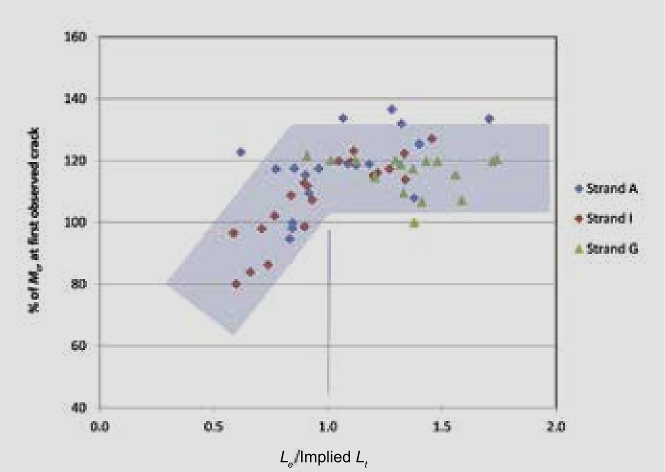

Figure 6. Comparison of transfer lengths and first observed cracking showing trend. Note: L_e = embedment length; L_t = transfer length; M_{cr} = cracking moment.

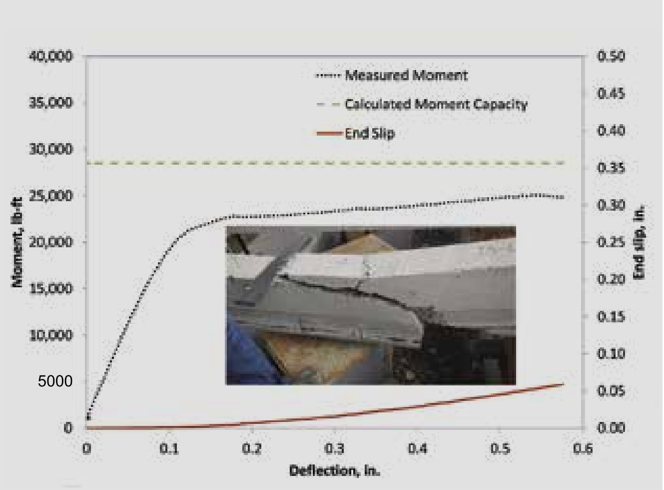

Figure 7. Beam end I5-L flexural test results. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 lb = 4.448 N.

Table 6. Summary of test results for strand A beams

Beam end	Experimental L_t at time of flexural test, in.	M_{exp}/M_n	Slip during test, in.	Crack-inducing technique	Ultimate failure mode
A1-S	40	1.37	0.214	Crack former	Rupture
A2-S	56	1.23	0.108	Crack former	Shear compression
A3-S	40	1.39	0.145	Crack former	Rupture
A4-S	48	1.41	0.197	Crack former	Rupture
A5-S	69	1.31	0.267	Crack former and one 1.375 in. saw cut on bottom and sides	Shear compression
A1-L	38	1.23	0.006	Crack former	Rupture
A2-L	43	1.20	0.033	Crack former	Shear compression
A3-L	42	1.04	0.066	Crack former	Shear compression
A4-L	35	1.12	0.069	Crack former	Shear compression
A5-L	45	1.14	0.032	Crack former	Shear compression
A6-S	47	1.38	0.130	One 1 in. saw cut	Rupture
A7-S	50	1.31	0.099	None	Rupture
A8-S	51	1.17	0.218	Two 1.375 in. saw cuts	Shear compression
A9-S	51	1.18	0.286	Two 1.375 in. saw cuts	Shear compression
A10-S	52	1.10	0.164	Two 1.375 in. saw cuts	Shear compression
A6-L	46	1.24	0.025	Three 1.375 in. saw cuts and 1 in. side cuts	Shear compression
A7-L	52	1.18	0.029	None	Shear compression
A8-L	50	1.21	0.082	Three 1.375 in. saw cuts	Shear compression
A9-L	54	1.21	0.091	Three 1.375 in. saw cuts	Shear compression
A10-L	61	0.95*	0.027	Three 1.375 in. saw cuts	Shear compression

*The value is less than 1, which indicates a failure.

Note: L_t = transfer length; M_{exp} = maximum moment calculated from experiment; M_n = nominal moment. 1 in. = 25.4 mm.

stood by each beam with the nominal moment capacity that was calculated for the section according to ACI 318-14. However, when comparing the performance of each of the 60 beam tests (30 specimens × 2 ends), the actual as-measured dimensions and the concrete strengths at release and testing shown were used in the calculation of the nominal moment M_n for each beam tested.

Tables 6 through **8** summarize the results from load testing of beams A, G, and I, respectively. Only beams with strand G had strand rupture failures after multiple crack initiation points were implemented. The ultimate failure mode of the strand I beams was affected the most by the multiple saw-cutting procedure. Before implementing the multiple saw-cutting procedure, nine of the 12 I-strand specimens failed by strand rupture. However, after additional saw-cutting, none of the I-strand beams failed by strand rupture and four of the eight failed before reaching the calculated ACI 318-14 nominal moment capacity. This shows that strand I was incapable of consistently meeting the ACI 318-14 development length requirements when cracking was present in the flexural bond region.

Statistical analysis

Analysis was conducted by plotting the average values of the transfer lengths at the time of testing against the average ASTM A1081 interlaboratory study results (Table 1). **Figure 8** shows how the ASTM A1081 pull-out strength required to achieve the ACI 318-14 predicted transfer length was determined through extrapolation.

Table 7. Summary of test results for strand G beams

Beam end	Experimental L_t at time of flexural test, in.	M_{exp}/M_n	Slip during test, in.	Crack-inducing technique	Ultimate failure mode
G1-S	33	1.28	0.289	One 1 in. saw cut	Shear compression
G2-S	36	1.30	0.126	One 1 in. saw cut	Shear compression
G3-S	31	1.40	0.038	One 1 in. saw cut	Rupture
G4-S	42	1.21	0.092	One 1 in. saw cut	Shear compression
G5-S	38	1.37	0.148	Two 1.375 in. saw cuts and 1 in. side cuts	Shear compression
G1-L	34	1.23	0.002	One 1 in. saw cut	Rupture
G2-L	37	1.22	0.003	One 1 in. saw cut	Rupture
G3-L	38	1.24	0.007	One 1 in. saw cut	Rupture
G4-L	43	1.16	0.057	One 1 in. saw cut	Shear compression
G5-L	40	1.23	0.022	One 1 in. saw cut	Rupture
G6-S	29	1.34	0.104	One 1 in. saw cut	Shear compression
G7-S	30	1.34	0.056	None	Shear compression
G8-S	47	1.16	0.304	Two 1.375 in. saw cuts	Shear compression
G9-S	49	1.38	0.787	Two 1.375 in. saw cuts	Rupture
G10-S	33	1.40	0.260	Two 1.375 in. saw cuts	Rupture
G6-L	31	1.23	0.002	Three 1.375 in. saw cuts and 1 in. side cuts	Rupture
G7-L	44	1.19	0.020	None	Rupture
G8-L	42	1.22	0.022	Three 1.375 in. saw cuts	Shear compression
G9-L	41	1.15	0.172	Three 1.375 in. saw cuts	Shear compression
G10-L	34	1.26	0.022	Three 1.375 in. saw cuts	Shear compression

Note: L_t = transfer length; M_{exp} = maximum moment calculated from experiment; M_n = nominal moment. 1 in. = 25.4 mm.

A comparison of the ASTM A1081 pull-out strength and transfer length at release is similar in nature to that found when using the transfer length at the time of testing; however, the values are shifted because of the lower transfer length at release. The transfer lengths at the time of testing were also plotted against the 90% confidence interval with 5% fractal applied to the average ASTM A1081 pull-out value, as well as the 90% interval with a 10% fractal applied to the average ASTM A1081 pull-out value. Determining the 90% confidence interval values for ASTM A1081 was performed following the "Experimental Statistics" portion of the *National Bureau of Standards Handbook*,[12] which lists the factors corresponding to one-sided tolerance limits per sample size per case of confidence interval targeted and the related fractal values.

For example, in our case, for a 90% confidence that 90% of our values will be above the average value obtained during the interlaboratory study, or a 90% confidence that no more than 10% will be lower than the interlaboratory study average value (10% fractal), the standard deviation of the interlaboratory study population was multiplied by a K factor of 2.012 as recommended by Natrella[12] and then added to the average value obtained by the interlaboratory study population.

The K factor is used to represent the width of the interval around a normal distribution that encompasses a one-sided tail interval that contains a given fractal (**Fig. 9**). The K factor used is also based on a sample size of 11 ASTM A1081 values and accounts for how well that sample size represents the entire data population. A larger sample size allows for a smaller K factor because the sample set would better represent the entire data population. Similarly, the 5% fractal values were determined, applying instead the recommended K factor of 2.503 for a sample size of 11.[12] The values calculated following Natrella's

Table 8. Summary of test results for strand I beams

Beam end	Experimental L_t at time of flexural test, in.	M_{exp}/M_n	Slip during test, in.	Crack-inducing technique	Ultimate failure mode
I1-S	51	1.30	1.030	Crack former	Rupture
I2-S	47	1.31	1.098	Crack former	Rupture
I3-S	56	1.38	0.702	Crack former	Rupture
I4-S	72	1.35	1.140	Crack former	Rupture
I5-S	73	1.29	> 0.5*	Crack former	Rupture
I1-L	53	1.17	0.295	Crack former	Rupture
I2-L	49	1.11	0.046	Crack former	Shear compression
I3-L	41	1.09	0.185	Crack former	Shear compression
I4-L	66	1.14	0.533	Crack former	Rupture
I5-L	44	0.88†	0.059	Crack former and two 1.375 in. saw cuts on bottom and sides	Shear compression
I6-S	34	1.12	0.257	Two 1.375 in. saw cuts and 1 in. side cuts	Shear compression
I7-S	46	1.28	0.957	None	Shear compression
I8-S	65	1.01	0.257	Two 1.375 in. saw cuts	Shear compression
I9-S	58	0.93†	0.216	Two 1.375 in. saw cuts	Shear compression
I10-S	32	1.16	0.525	Two 1.375 in. saw cuts	Shear compression
I6-L	48	1.22	0.256	One 1 in. saw cut	Rupture
I7-L	56	1.21	0.177	None	Rupture
I8-L	54	1.16	0.231	Three 1.375 in. saw cuts	Shear compression
I9-L	66	0.97†	0.213	Three 1.375 in. saw cuts	Shear compression
I10-L	83	0.94†	0.270	Three 1.375 in. saw cuts	Shear compression

*The final end-slip value was not recorded after the linear variable differential transformer went past the 0.5 in. range.

† The value is less than 1, which indicates a failure.

Note: L_t = transfer length; M_{exp} = maximum moment calculated from experiment; M_n = nominal moment. 1 in. = 25.4 mm.

recommendations were incorporated in the threshold determination analysis, obtaining potential acceptance criteria that will ensure adequate bond strength with high confidence. **Table 9** shows the ASTM A1081 pull-out strength value that corresponds to the ACI 318-14 predicted transfer length for a 0.5 in. diameter prestressing steel strand using fractal percentages. The ASTM A1081 values calculated to ensure that the beams meet the ACI 318-14 transfer lengths are high. An ASTM A1081 value of 28,200 lb (125 kN) would be difficult for many, if not all, manufacturers to achieve consistently. It was determined to then look at the ASTM A1081 value that would be required to ensure that the ACI 318-14 nominal moment capacity could be achieved for determining minimum acceptance criteria for ASTM A1081.

A method similar to the one used to determine the ASTM A1081 values required to achieve the ACI 318-14 predicted transfer length was used to determine the ASTM A1081 pull-out value required to achieve the ACI 318-14 nominal moment capacity. Instead of using a linear fit for the data, a second-order polynomial was used. This was done because once bond is sufficient to prevent a shear failure and cause a strand to have a rupture-type failure, the moment capacity of the beam no longer increases, even with better bond between the concrete and strand. There was some concern that the use of multiple strands reduces the considerable variability inherent in strand bond to some extent. Although having multiple strands in a beam would not alter the overall average capacity or transfer length expected for many beams, it would reduce

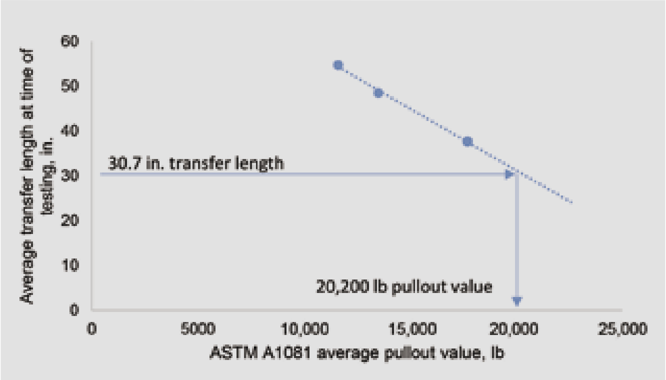

Figure 8. Average transfer length at time of test versus average ASTM A1081 pull-out value. Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

the expected standard deviation. It was decided to numerically investigate the effects of using multiple strands in a beam on the strand moment capacity. For this analysis, it was assumed that the moment capacity of a beam prestressed with two strands instead of one would be the average of the moment capacity of two separate beams prestressed with one strand each. This method assumed no interactions between the strands.

The PCI advisory group opined that a combination of six strands would be the likely minimum number of strands in a production beam, so that number was selected for use in the threshold determination. However, all combinations of strand samples from the same strand source up to 20 strands in a single beam were investigated. Considering combinations of multiple strands, therefore averaging their performance, implied lower standard deviations, resulting in decreasing threshold values with the increasing number of combined strands.

Table 10 gives an example of the averaging procedure, which was as follows. Assume that the ratios of experimental moment to calculated nominal moment capacity from four single-strand tests are the following values:

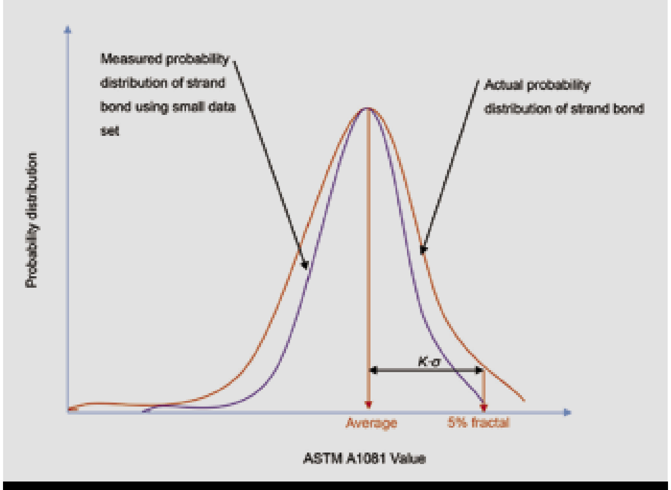

Figure 9. Graphical representation of how the K factor is used to calculate the fractal values used in statistical analysis. Note: K = K factor; σ = standard deviation.

- strand 1 = 1.300
- strand 2 = 1.310
- strand 3 = 1.380
- strand 4 = 1.351

The average of these four values is 1.335, and the standard deviation is 0.037. Table 10 illustrates this by considering the possibility that there would always be a minimum of two strands present in a given member, the standard deviation of combined values would drop to 0.020, while the average remains the same. For six strands, the averaging procedure reduced the coefficient of variation of each one of the strands by 66%.

Figure 10 is a plot of the threshold value variation as the number of strands in a single beam section increases, using the numbers generated during the first round of analysis, where the ratios of all 20 beam ends tested per strand source were considered. Figure 10 uses the 90% confidence interval on 5% fractal values for both the ASTM A1081 values and the ratio of the moment capacity to the nominal moment capacity for different numbers of strands used. The procedure was repeated for the average pull-out force and the 90% confidence interval on 10% fractal for the ASTM A1081 values. **Table 11** shows the minimum acceptance criteria for ASTM A1081 found using each fractal analyzed. Based on the results of the statistical analysis, a 14,600 lb ASTM A1081 pull-out value would give 90% confidence interval that only 5% of the values would have a capacity lower than the ACI 318-14 calculated nominal moment capacity and is recommended for adoption as a minimum acceptance criterion for ASTM A1081. This minimum acceptance criterion however does not ensure that the product will achieve the ACI 318-14–calculated transfer-length value.

Validation of proposed ASTM A1081 minimum acceptance criterion

In addition to the flexural beams tested, three additional beams with smaller cross sections for each strand were fabricated on cast day 3 and shipped to and tested by another laboratory. These beams were fabricated as described by Peterman.[13] The test method presented by Peterman provides a simple procedure for verifying bond for the concrete mixture, strand being used, placement conditions, and detensioning conditions used at a particular prestressed concrete plant.[13] The beam dimensions used in this test were 6 × 8 × 138 in. (150 × 200 × 3510 mm), with a single strand in the beam center 4.5 in. (110 mm) below the top surface. The Peterman test beams were cast with the same concrete mixture as the flexural beam test specimens. Three beams for each

Table 9. ASTM A1081 pull-out force values corresponding to ACI 318-14 transfer-length values at release and at time of test

Time of transfer-length measurement	Pull-out force that corresponds to the ACI 318-14–calculated transfer length for the ASTM A1081 interlaboratory study		
	Average pull-out force, lb	90% confidence interval on 10% fractal, lb	90% confidence interval on 5% fractal, lb
Strand release	16,400	21,300	22,500
Time of flexural testing	20,200	26,600	28,200

Note: 1 lb = 4.448 N.

strand source, A, G, and I, were fabricated. No shear reinforcement was provided for the specimens, as specified by the test protocol.

The beams were all fabricated in a single line, with splice chucks used to connect the three different strand sources. The nine beam sections were saw cut to their specified length to match the procedure used in the 6½ × 12 in. (165 × 310 mm) beams as soon as the companion concrete cylinder reached a compressive strength of 4040 psi (27.9 MPa), following the same detensioning procedures (saw cutting) as the flexural beam specimens. These beams were saw cut at a slightly higher release strength than the 6½ × 12 in. beams because the smaller cross-sectional area of these beams generated less heat from hydration. The maximum temperature reached in the smaller beams was lower than in the larger beams, giving a slower strength gain rate in these beams than in the larger beams. At the end of the work day, the strength in the smaller beams was not high enough to detension. The strength of the match-cured concrete cylinders for the smaller beams was measured early the next day and found to be 4040 psi. The beams were then saw cut immediately following the concrete cylinder strength tests.

Table 10. Strand averaging procedure example

Combination	Value 1, M_{exp}/M_n	Value 2, M_{exp}/M_n	Two-strand average, M_{exp}/M_n
Strands 1 and 2	1.300	1.310	1.305
Strands 1 and 3	1.300	1.380	1.340
Strands 1 and 4	1.300	1.351	1.326
Strands 2 and 3	1.310	1.380	1.345
Strands 2 and 4	1.310	1.351	1.331
Strands 3 and 4	1.380	1.351	1.366
Average			1.335
Standard deviation			0.020

Note: M_{exp} = moment calculated from experiment; M_n = nominal moment.

The Peterman beam test was conducted by setting each beam on roller supports and gradually loading each beam section to 85% of its calculated nominal moment capacity, or 4210 lb (18.7 kN), assuming a concrete compressive strength of 6000 psi (41 MPa).[13] The beam is loaded in four-point bending with the loads applied 4 ft 9 in. (1.45 m) from each beam end. Beam deflection and end-slip measurements were recorded with linear variable differential transformers.

The specimens were then inspected for cracks and strand end slip, and the details were documented. After this load was sustained for 24 hours, the beams were examined for additional signs of distress, such as increased end slip, concrete cracking, or crushing. After sustaining 85% of their nominal moment capacity for 24 hours, the beams were loaded to their full nominal capacity (applied load equal to 5010 lb [22.3 kN]) and allowed to hold that load for 10 minutes, unless they failed before that time. The beam specimens that were able to sustain their nominal moment capacity for 10 minutes passed the test and were later loaded to failure.

Many flexural cracks typically develop at both the 85% M_n and 100% M_n load levels in the Peterman beam test. Unlike the 60% L_d and 80% L_d tests on the 6½ × 12 in. (165 × 310 mm) members tested, the geometry and loading pro-

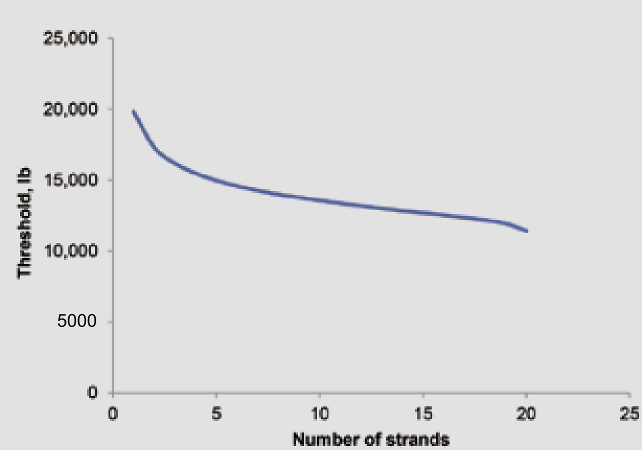

Figure 10. Comparison of threshold value and number of strands combined-polynomial analysis. Note: 1 lb = 4.448 N.

Type of analysis	Pull-out force that corresponds to achieving the ACI 318-14–calculated nominal moment capacity in 6 ½ × 12 in. single-strand beams for the ASTM A1081 interlaboratory study		
	Average pull-out force, lb	90% confidence interval on 10% fractal, lb	90% confidence interval on 5% fractal, lb
One strand, 20 beam ends	14,400	18,800	19,800
Six strands, 20 beam ends	10,900	13,900	14,600

Table 11. Recommended ASTM A1081 threshold values

Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

cedure used in the Peterman beam test results in significant cracking in the flexural bond region without the use of crack initiators.

The applied load, midspan deflection, and strand end slip during loading were plotted for each beam. While three beam sections were tested per strand source during the Peterman test program, all of the specimens that were prestressed with strands A and G passed the test and held 100% of the calculated nominal load, but all three strand I specimens failed before reaching the calculated nominal load.

Table 12 presents a summary of the test results, displaying the midspan deflection per beam after each load-sustaining period, as well as the maximum load sustained by each section. **Figure 11** illustrates the beam setup and shows the performance of the beams with a plot of their midspan deflection versus the load applied to them. The midspan deflection for the beams prestressed with strand I kept growing significantly along with strand end slip while the beams were sustaining 85% of their nominal capacity.

All strand A and strand G samples passed the Peterman beam test, but none of the three strand I specimens passed the test. The consistent outcome of these tests indicates that for the concrete mixture and release strength used, both strands A and G met the ACI 318-14 nominal moment design assumptions, while strand I did not. As with the Stresscon beams with saw cuts, the Peterman beam test showed the inability of strand I to meet the ACI 318-14 nominal moment requirements because of poor bond when cracking is present in the flexural bond region.

Conclusion

Based on the work performed, the following conclusions can be made:

- The average transfer lengths at release estimated using the 95% average maximum strain method from surface-strain readings measured using a laser-speckle interferometer imaging system for 6½ × 12 in. (165 × 310 mm) beams fabricated were 34.6 in. (879 mm) for strand A, 39.6 in. (1010 mm) for strand I, and 27.4 in. (696 mm) for strand G. The average transfer lengths at the time of flexural testing was considerably longer as measured using a combination of surface-strain and end-slip measurements and were 48.5 in. (1230 mm)

Table 12. Summary of Peterman beam test results

Test beam	Deflection after 24 hours at 85% M_n, in.	Deflection after 10 minutes at 100% M_n, in.	Maximum load, lb	$P_{exp}/P_{100\%}$
A-1	0.85	1.45	5740	1.15
A-S	0.70	1.16	5978	1.19
A-3	0.74	1.17	6106	1.22
G-1	0.68	1.08	6143	1.23
G-2	0.82	1.30	5802	1.16
G-3	0.79	1.31	5778	1.15
I-1	3.73	n/a	4659	0.93
I-2	2.38	n/a	4774	0.95
I-3	2.92	n/a	4607	0.92

Note: M_n = nominal moment; n/a = not applicable because beam failure occurred in less than 10 minutes; $P_{100\%}$ = load at 100% of ACI 318-14 nominal moment capacity; P_{exp} = maximum load measured. 1 in. = 25.4 mm; 1 lb = 4.448 N.

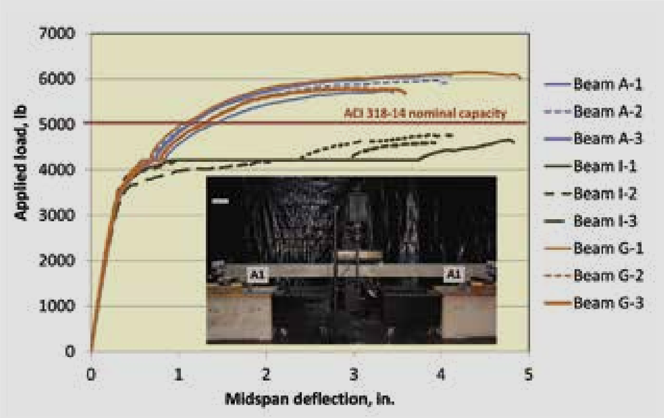

Figure 11. Comparison of midspan deflection and applied load for all specimens tested using the simple quality assurance test for strand bond. Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

for strand A, 54.7 in. (1390 mm) for strand I, and 37.7 in. (958 mm) for strand G. The average transfer lengths for beams made with each strand source were longer than predicted by the ACI 318-14 transfer-length equation. This is because of the combination of strand surface properties and the use of a low release strength near 3500 psi (24 MPa).

- The minimum ASTM A1081 acceptance criterion for a prestressed concrete beam released at 3500 psi (24 MPa) concrete strength to ensure that 90% of the time the transfer length is less than the ACI 318-14 predicted transfer length with a 90% confidence interval is 28,200 lb (125 kN). This ASTM A1081 value would preclude most strand produced in North America from being used in pretensioned applications and was judged by the research team to be impractical. More work is needed to reexamine the ACI 318-14 transfer length equation and determine if it should be updated to account for variables, such as release strength, that are not currently considered.

- The minimum ASTM A1081 acceptance criterion for a prestressed concrete beam released at 3500 psi (24 MPa) concrete strength to ensure that 95% of the time the moment capacity is greater than the calculated ACI 318-14 nominal moment capacity with a 90% confidence interval is 14,600 lb (64.9 kN).

- Small rectangular beams, fabricated and tested as described by Peterman,[13] were used to validate the recommended minimum acceptance criterion for ASTM A1081 and resulted in the following conclusions:

  – Each of the three beams made with strand I experienced large strand slip and deflection growth during the load 24-hour hold period at 85% of the calculated nominal moment capacity, and all failed prematurely before reaching 100% of the calculated beam nominal moment capacity.

  – Each of the three beams made with strand A and strand G had excellent performance during the 24-hour hold period (no end slip and small deflection increase) and all were able to withstand 100% of the calculated moment capacity for 10 minutes. After the 10-minute holding period, each of these beams withstood higher loading levels before eventually failing in flexure.

  – If the strand acceptance value is established based on the concrete mixture and release strengths used in this study, then the threshold value should be set such that strand I is excluded but strands A and G are allowed.

Recommendations

The structural load testing in this study revealed the susceptibility of pretensioned members fabricated with strand I to cracking that can occur in the flexural bond region of many member types. For this reason, the authors recommend that plants conduct regular flexural testing of their bond-critical products to verify the performance of their concrete and strand system used. In lieu of testing actual products, Peterman beam tests have been shown to provide a consistent means to verify the acceptable bond quality of concrete and strand combinations.

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Notation

d = distance

d_b = nominal strand diameter

E_{ps} = strand elastic modulus

f_{ps} = stress in prestressing strand at nominal strength

f_{pu} = ultimate tensile strength

f_{se} = effective stress in prestressing strand

f_{si} = initial stress in strand before long-term losses

f_c' = 28-day concrete compressive strength

f_{ci}' = concrete compressive strength at time of detensioning

K = K factor

L_b = bond length

L_d = development length

L_e = embedment length

L_t = transfer length

M_{cr} = cracking moment

M_{exp} = maximum moment calculated from experiment

M_n = nominal moment

$P_{100\%}$ = load at 100% of ACI 318-14 nominal moment capacity

P_{exp} = maximum load measured

ΔL_t = increase in transfer length

ΔS = increase in end slip

σ = standard deviation

About the authors

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Abstract

ASTM International recently adopted ASTM A1081, *Standard Test Method for Evaluating Bond of Seven-Wire Steel Prestressing Strand*, a pull-out test procedure to verify steel strands' ability to bond to cementitious materials before they are used as tensile reinforcement in pretensioned concrete members. PCI commissioned a study to determine a minimum ASTM A1081 bond threshold value for steel strand to be used in pretensioned applications. The minimum acceptance criteria for ASTM 1081 were determined by comparing ASTM A1081 pull-out strengths for three strands of different bond characteristics to their performance in pretensioned beams as measured by transfer lengths and moment capacity when loaded at a point at 60% or 80% of the ACI 318-14 calculated development length from the beam end. Statistical analysis of the flexural beam testing data and correlation with the prestressing strand sources' ASTM A1081 test results were performed. The results showed that it would take an ASTM A1081 pull-out value of 28,200 lb (125 kN) to have 90% confidence that 95% of beams with a single strand would have a transfer length smaller than or equal to the ACI 318-14–calculated transfer length. The results also showed that an ASTM A1081 pull-out value of 14,600 lb (64.9 kN) would have 90% confidence that 95% of beams containing at least six strands would have a moment capacity greater than or equal to the ACI 318-14–calculated nominal moment capacity.

Keywords

Beam, bond, moment capacity, strand, tensile reinforcement, transfer length.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

Reader comments

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