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Bond Performance of 1-1/8 Inch Diameter Prestressing Strands

FINAL REPORT
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16. Abstract Experimental research addressing the bond performance, transfer and development length of 1-1/8 in. diameter grade 250 strands to concrete is presented in this report. Three different bond testing methodologies were employed to appraise the pullout force values of the strands in concrete and mortar, namely the Large Block Pullout Test (LBPT), the ASTM A1081 test, and non-prestressed square concrete prisms. The results revealed that the 1 1/8" diameter strands exhibit good bonding quality to normal weight concrete and mortar, though there is limited information for comparison. Eight full-scale beams were tested to evaluate the development length, prestress loses, and flexural performance of large diameter strands. The transfer length measurements were determined by analyzing the concrete surface strain readings and their corresponding strain profiles employing the 95% AMS method. It was found that current code equations overestimate transfer length for the 1-1/8 in. diameter strands. The beams were also employed to assess the development length of the strands and it was determined that current code may overestimate 1-1/8 in. diameter strands in the concretes tested.		
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Executive Summary

Large diameter prestressing strands have been investigated in recent years as potential options for increasing span lengths of prestresses members. Common strand diameters in use today for bridge girder applications and range from 0.5 in. and 0.6 in. to the newer 0.7 in. diameter strands available in the United States. Larger diameter strands and higher wire count strands exist on the international market but have largely remained unstudied in the United States and elsewhere. Furthermore, it is unclear how a 19-wire strand will perform embedded in concrete and the applicability of current code expressions which are widely regarded as conservative, but based on 7-wire strands of diameters under 0.7 in.

In this project, Grade 250, 19-wire, 1-1/8 in. diameter strands were procured and investigated for use in pretensioning applications in a largely preliminary study. Bond testing was performed using several standard and popular research methods for benchmarking purposes. Bond behavior of the large diameter strands exhibited higher pullout values, due to their larger diameter with direct comparison to 0.5 in. diameter control strands.

Eight total large-scale T-beams were pretensioned and tested in the lab. Transfer lengths were monitored for various concrete strengths and found to be over predicted by the AASHTO LRFD provisions. The average transfer lengths were 60, 42, 32, and 29 inches for concrete strengths of 6.2, 8.3, 9.4 and 10.2 ksi, respectively, which were shorter than was predicted by the AASHTO LRFD equation ($60d_b$). The beams were tested in flexure to evaluate their bond performance as related to their development length. Half of the tests resulted in bond failure and the other half resulted in the section reaching its predicted strength. Large scale embedment length testing of 1-1/8 in. diameter strand in 6500 psi class concrete indicated development length is greater than 135 in. Large scale embedment length testing of 1-1/8 in. diameter strand

in 8500 psi class concrete indicated development length is between 66 in. and 78 in. Finally, large scale embedment length testing of 1-1/8 in. diameter strand in 9500 psi class concrete indicated development length is between 66 in. and 99 in. Generally, the AASHTO predicted development lengths (between 161 in. and 184 in. depending on transfer lengths observed) were much higher than those observed.

Generally, the 1.125 in. diameter strands performed as expected though there was some evidence that higher strength concretes would benefit end-zone performance and bond performance. Some differential slip between outer wires and inner wires was noted during the NASP testing, though this was not noticed during large-scale flexural testing. This preliminary study indicates that 1-1/8 in. strands may be a viable option – with additional study – in prestressing applications if their longer transfer and development lengths are acceptable for the member.

CHAPTER 1 Introduction

1.1. Problem Statement

In recent years, concrete bridges have benefited from larger diameter prestressing strands, which have gone from 0.5 in. to 0.6 in., mostly adopted as the standard in the 1990s. The largest available strands in the United States are 7-wire 0.7 in. diameter strands but are not used as frequently in the country despite several researchers finding that they can provide longer spans and more efficient structures with their use. Even large strands exist in the international market. This research looks into Grade 250 19-wire 1-1/8 in. diameter strands as another option in pretensioned applications to increase span lengths and learn more about the performance of stranded wire bonded to concrete. Because there is little known about their performance, this research seeks to characterize bond and performance in pretensioned applications. Since the overarching goal of this research is to investigate the use of 19-wire 1-1/8 in. diameter prestressing strand purchased internationally; it is crucial to understand their mechanical behavior, bond to concrete, and their fundamental behavior when employed in flexural members.

The Grade 250 19-wire 1-1/8 in. diameter strand procured for this research has a nominal area of 0.825 in^2 and conforms to Japanese Industrial Standard (JIS) G3536:2014 and is compared to other popular prestressing strands in Table 1-1. Anecdotally, from the experience of the researchers on this project, handling the 19-wire strand comparable to past experience with 0.7 in. diameter 7-wire strands, though it is obviously heavier, in spite of the fact that the moment of inertia is much larger.

Table 1-1 Nominal Strand Diameters Areas, and Moments of Inertia

Diameter (in.)	**Area (in^2)**	**Moment of Inertia ($in.^4$)**
0.5	0.153	2.65E-04
0.6	0.217	5.38E-04
0.7	0.294	9.84E-04
1.125	0.825	35.0E-04

A photograph of this strand is in Figure 1-1 along with the single use barrel-and-wedge gripping devices (chucks) in Figure 1-2 and in the 6.5 ft diameter (average outside diameter) spool in Figure 1-3.

(a) (b)

Figure 1-1 1-1/8 in. diameter strand (a) Cross Section (b) Measurement of Cross Section

(a) (b)

Figure 1-2 1-1/8 Diameter Strand Single Use Chuck (a) Barrel and Wedge (b) Transverse dimension.

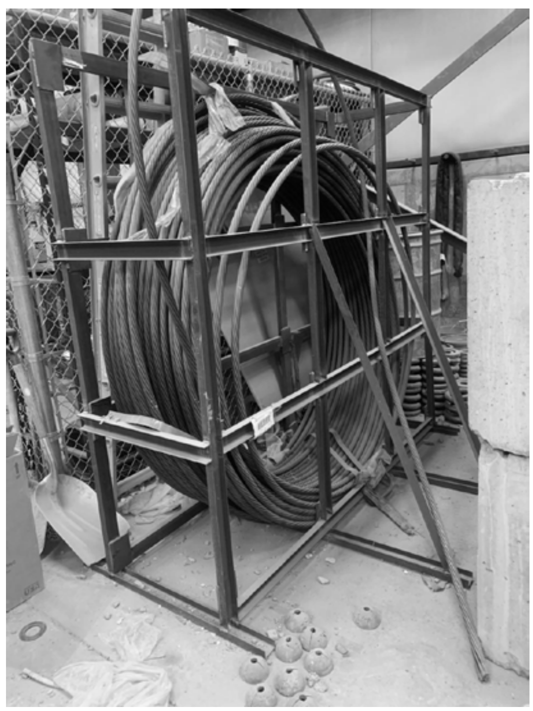


Figure 1-3 1-1/8 in. diameter strand in 6.5 ft diameter spool.

The 1-1/8 in. diameter strands contain the same area as 5.4 total 0.5 in. diameter strands and 3.8 total 0.6 in. strands indicating that even if strand spacing must be extended considerably beyond currently typical 2 in. by 2 in. spacing more prestressing force may be applied per unit area, resulting in higher precompression forces. Table 1-2 shows a comparison between 1-1/8 in. strand force per area ratio as compared to the steel area per concrete area for different spacing, allowing a comparison between strand precompression capability if the same grade steel was used. Clearly, the 1-1/8 in. strand greatly exceeds the available strands at standard 2 in. by 2 in. spacing, but it is unlikely to be functional at this spacing. However, at 3 in. by 3 in. spacing – the minimum spacing possible given the available chuck diameters – the strand becomes more efficient as compared to 0.7 in. diameter strands. The work contained in this research investigates the bond behavior and the transfer and development lengths for 1-1/8 in. diameter strands, but because it is the first effort of its kind cannot answer the question of strand spacing.

Table 1-2. Comparison of 1-1/8 in. strand at different spacings to other strands at 2 in. by 2 in. spacing.

Square Strand Spacing (in.)	0.5in. Strand (0.153 in.^{2})	0.6in. Strand (0.217 in.^{2})	0.7in. Strand (0.294 in.^{2})
2	+439%	+280%	+181%
2.5	+245%	+143%	+80%
3	+140%	+69%	+25%
3.5	+76%	+24%	-8%
4	+35%	-5%	-30%

1.1 Objective and Scope

Because the existing testing methodologies to evaluate bond of strands have shown to be more sensitive to the handling of the specimen, the strand surface condition, strand diameter, embedment length of strand and concrete strength, it is hypothesized that the 1-1/8 in. diameter strands can bond appropriately to concrete if the proper embedment length is provided and that different testing methodologies can provide an accurate estimate of the bond of 1-1/8 in. diameter grade 250 strands to concrete. It is further hypothesized that higher concrete strengths will increase the pullout force of the strand due to an increase in mechanical bonding of the strand to the surrounding concrete; thus, decreasing the end-slip of the strand, transfer and development length. This study seeks to evaluate the above-mentioned hypothesis employing three different testing methodologies. Furthermore, full-scale specimens are used to examine the flexural response, transmission and development length of 1-1/8” diameter strands. To that end transfer and development length, and concrete strength are compared to characterize the bond of the strand and determine whether the studied strand size is appropriate for pre-tensioning applications.

1.2 Report Organization

This report is organized as follows:

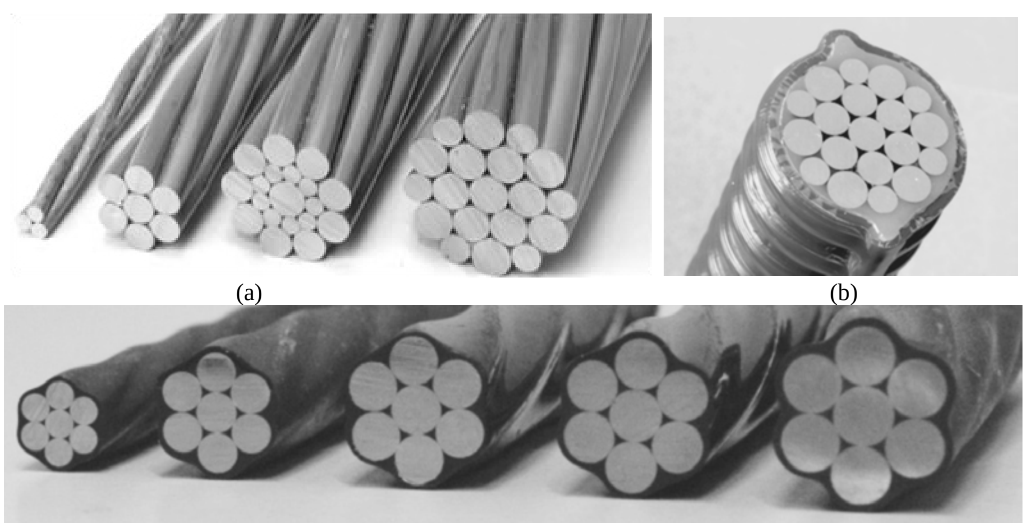
- Chapter 2 provides a brief literature review on the methodologies for bond performance evaluation, transfer and development length historical perspective and equations to predict them.
- Chapter 3 summarizes the experimental program conducted to appraise the bond of prestressing strands to concrete, the method utilized to evaluate the transfer length of large diameter strands, and testing layout for development length and flexural performance determination.
- Chapter 4 presents material testing results of all concrete employed in this research, along with results of bond evaluation using three different methodologies, and the transfer and development length testing for eight different beams.
- Chapter 5 summarizes research findings and highlights differences between the code and the testing results.

CHAPTER 2 Literature Review

2.1 Introduction

In recent years, concrete bridges have benefited from larger diameter prestressing strands, which have gone from 0.5 in. to 0.6 in., mostly adopted as the standard in the 1990s (Maguire et al., 2018; Pettigrew et al., 2016; Six et al., 2019; Tawadrous et al., 2019). In the mid-2000s, many researchers investigated the performance of larger strands sizes and high strength strands, namely 0.6 in. special, 0.7 in. and Grade 300. The largest available strands in the United States are 7-wire 0.7 in. diameter strands and are used sparingly in some early adoption states, despite several researchers finding that they can provide longer spans and more efficient structures with their use. Since the overarching goal of this research is to investigate the use of 19-wire 1.125 in. diameter prestressing strand purchased internationally; it is crucial to understand their mechanical behavior, bondability to concrete, and their fundamental behavior when employed in flexural members. This chapter summarizes the current methodologies used to evaluate the bond quality, transfer and development length, and ultimate behavior of prestressed concrete flexural members. Other aspects, such as concrete composition, thermal loading and multi-span behavior influence the behavior of the precast/prestressed members but are not addressed herein (George Morcous et al., 2009; F. F. Pozo-Lora & Maguire, 2019; F. Pozo-Lora & Maguire, 2020; Tavakoli et al., 2017).

The first accountable application of prestressing to concrete dates to 1886, when an engineer from California obtained a U.S. Patent for using tensioned steel rods in concrete arches and slabs, (Jackson as cited in Naaman, 2012). In near parallel, a German engineer, in 1888, patented a method for prestressing concrete using steel wires (Doehring as cited in Naaman, 2012). Both prestressing examples did not perform adequately due to the lack of understanding of the interaction between the concrete and steel, and the enormous losses due to creep, shrinkage, and relaxation. Some researchers suggested re-tensioning to recover the loss, whereas others tried to determine losses. Later, in the mid-1930s, Hoyer (Edwards, 1978) managed to prestress concrete slabs with piano wires, which was not successful due to the economic impact of World War II and because only small wire could be used to ensure bond. These issues were overcome by Freyssinet (1936)when the researcher finally understood the significance of losses and implemented different approaches to overcome them. After beating that obstacle, many buildings and bridges have been built and put to the service of society, but as cities grow, infrastructure needs to become more efficient, and spending needs to be reduced; thus, better materials need to be studied and developed to satisfy these needs (Maguire et al., 2012, 2015, 2016, 2017; McKinney et al., 2019; Torres et al., 2019). In the case of prestressed concrete reinforcement, strand sizes have gone from as small as 0.08 in. diameter wire, to the current and most efficient 7-wire 18 mm. diameter, see Figure 2-1. Although other sizes exist, such as the 19-wire 7/8 in. and 1-1/8 in. diameter strands, they are mostly used for soil nailing due to the gap in the knowledge for those sizes, and the expectation that the transfer length would be too large (Gilbert et al., 2016).



(c)

Figure 2-1 Examples of strands used in prestressed concrete : (a) Bare 3-, 7-, and 19-wire strands, (b)19-wire grouted strand, and (c) different sizes of coated strands. ***(Oshima et al., 2016)***

2.2 Bond of prestressing strands to concrete

The bond mechanism of strands in pre-tensioned concrete is a complex phenomenon in reinforced concrete. In conventionally reinforced concrete, which uses ribbed bars in most cases, the forces are transferred from the steel to the surrounding uncracked concrete through chemical adhesion, but as the concrete cracks, that adhesion is lost. Following concrete cracking, the ribs on the rebar surface, the friction between concrete and the reinforcement and the degree of confinement (stirrups) are the responsible of preventing a bond failure (Fédération Internationale du Béton, 2000). The prestressing steel and the concrete are in constant interaction in pre-tensioned concrete due to the pre-applied force that is introduced to the members. Guyon (1953), described this phenomenon as steel in compression prevented from returning to zero stress by the concrete in compression, regarded as the opposite phenomenon that happens in reinforced concrete, in which the steel and surrounding concrete usually are in tension. In other words, prestressed concrete was depicted as a problem of bond in compression, whereas reinforced concrete as a problem of bond in tension.

2.2.1 Research Prior to 1990

Quantifying the bond characteristics of prestressing wire and strands have always been a complicated and controversial matter, which many researchers have intensively worked on since the late 1940s. Marshall (1949), studied the bond of 0.08 in. and 0.2 in. high-grade wire to concrete, tensioned to 100 ton/in^2 and 70 ton/in^2, respectively. The specimens were 4 in. x 4 in. x 4 in. – 72 in. and the reinforcement were greased and ungreased. This researcher found that larger diameter wire tends to lose tension over time and suggested using a locking device to prevent it, or to switch to the 0.08 in. wire to ensure proper bonding.

This author also identified the components of the bond as adhesion, frictional resistance due to the pressure between the wire and the concrete generated by concrete shrinkage, and friction resistance due to the wedge effect.

Janney (1954) studied the nature of the bond of both clean and lubricated 0.100 in. to 0.276 in. wire, and 5/16 in. in pre-tensioned prestressed concrete. This researcher found the different wires and the strand to have satisfactory bond qualities and enough strength to bond efficiently to concrete and rusting the strands to have a positive impact on transfer length, i.e., the transfer length decreases when rusting strands, whereas greasing them ameliorated the bond capacity with respect to clean wire and strands. The main components of the bond defined by Janney (1954a)were three, namely chemical adhesion, friction and mechanical interlocking, but until nowadays, no consensus has been reached on what is the quantitative contribution of each. Other researchers (Hanson & Kaar, 1959) investigated the flexural and transfer bond of 7-wire 0.25 in., 0.385 in., and 0.500 in. strands. Results have shown the strand size and embedment length to have a considerable influence on the value of the average bond stresses at which general bond-slip occurs; rusting the strands increased bonding, whereas an increase in steel percentage decreased bonding; and increasing the jacking stress to increase embedment length.

Moustafa (1974), developed the first pullout test method to evaluate the bond quality and strength of 7-wire strands and lifting loops. This test consisted of 12 in. wide by 26 in. deep by 12 ft. long concrete block with strands embedded into them. The strands were both clean and light rusted, unstressed 0.385 in., 0.438 in. and 0.50 in. diameter, grade 270 strands, embedded 12 to 30 inches concrete with a straight, broom, or 90° bend at the end of the embedment. The compressive strength of the concrete ranged from 4 to 8 ksi. The strands were pulled out using two hydraulic jacks.

This researcher found the pullout results to be qualitatively correlated with the transfer length, i.e., the transfer length decreases when the pullout force is “high”, and the surface condition of the strands which showed smaller transfer length and larger pullout forces with respect to the clean strands.

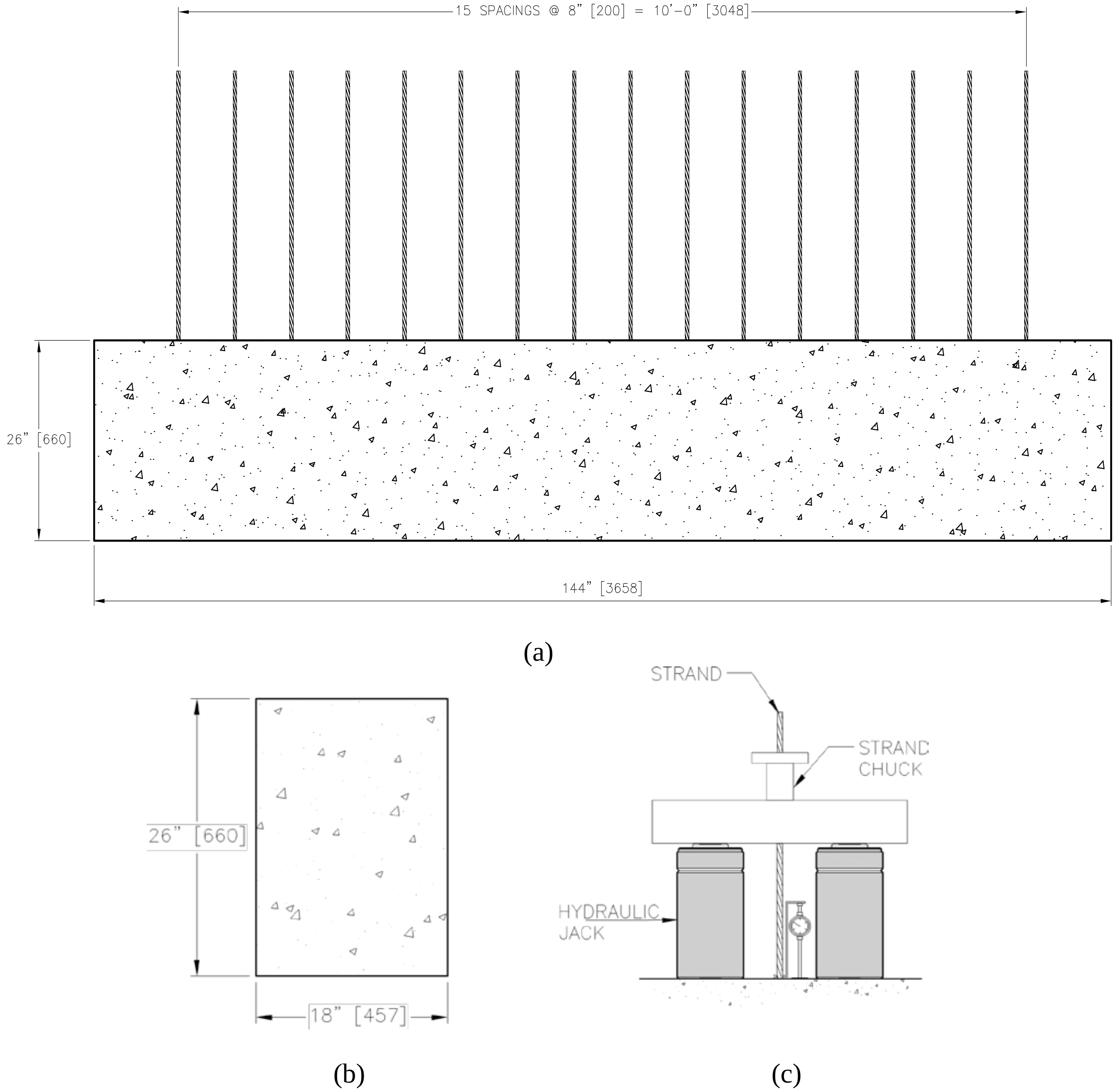


Figure 2-2 Illustration showing the elevation view of the big block and spacing of strands (a), cross-sectional dimensions of the block (b), testing layout (c).

Jokela & Tepfers (1982) investigated the bond of 0.5 in. bundled prestressing strands to normal weight 5 to 6.6 ksi concrete using rectangular prisms. This research included two types of specimens, one simulating the pre-tensioning of the strands and another using an unstressed strand. The testing results showed that the use of bundled strands or groups closely spaced do not affect bond; however, specimens with pretensioned strands had slightly better performance than the unstressed ones. It was also pointed out that the release method affected the slip of strands.

2.2.2 Research from 1990-2000

Tertea, Magureanu, & Onet (1992), studied the effect of long term and cyclic loading on the bonding properties of pre-tensioned concrete. Their main focus was to investigate the effect of prestressing degree, the type of loading, and the age of the specimens on the performance of prestressed concrete beams. The dimensions of the specimens were 4.75 in. x 10 in. x 126 in., reinforced with 3/8 in. grade 260 strands, PC60 (87 ksi ultimate strength) deformed bars, and the concrete *f'c* was 4.35 ksi. These researchers found out that Closed spaced stirrups lead to a shortening of the bonding length, that the force corresponding to the critical slippage increases with the prestressing degree, and that with partially prestressed members, the long-term loading leads to a decrease of the critical bending moment.

While most researchers prior to 1992 agreed on that surface condition, concrete compressive strength, strand size, and steel grade affected bond of prestressing, none of them established clear acceptance criteria on bond quality assessment. Rose & Russell (1997) conducted several tests to evaluate the bond performance of 0.5 in diameter, Grade 270 low-relaxation strands with varying surface conditions. The surface condition included as received, cleaned, silane treated, and weathered. The study also considered strands from three different manufacturers in as-received condition. Their experimental program considered two different testing procedures to investigate the bond of prestressing strands: the large block pullout test (LBPT), and tensioned pullout test specimens. Transfer length specimens were also fabricated, and the end slip and surface concrete strain were measured. These researchers found that the pullout test did not represent a reasonable measure of the bond of prestressed pretensioned concrete members because individual strands with the same surface condition exhibited large pullout force values and considerable transfer lengths. On the other hand, strand end slip measurements showed the best correlation to transfer length, that is, 0.95. They also indicated that the strand end slip measurement results were independent of the strand exterior, size, cutting location and the strength of the concrete.

Since the study conducted by (Rose & Russell, 1997) was inconclusive regarding the relationship between different pullout test results and transfer and development length, (Logan, 1997) performed a similar study gathering experienced researchers on bond testing. The objectives of the research were to establish minimum acceptance criteria for pullout test; to determine whether the variations on strand exterior conditions or dimensions have influence on the bond of strands; to evaluate feasibility of using the strand end slip at release as bond quality indicator; and to find whether pullout test results correlate with strand transfer length. To accomplish those tasks, strands from different manufacturer across the United States were chosen to test the pullout capacity and transfer length of 0.5in strands in concrete. This research concluded that strands that exhibit “high bond quality” also have a shorter transfer length, whereas strands with “poor bond quality” showed more significant transfer length and exhibit poor performance when texted in flexure. The LBPT results indicated that for strands with pullout capacities over 36 kips, the ACI requirement for transfer length was conservative, whereas strands with pullout capacities of 12 kips or under exhibited transfer length in accordance with the code, however, these values increased after 21 days, indicating poor bond quality. On the other hand, the end slip at release, strand surface coloration, and elimination of surface residuals did not considerably affect the transfer length of strands. This researcher also indicated that rusting the strands was not a necessary treatment for strands with excellent bonding characteristics, because they will perform similarly to the weathered ones. However, Southworth (1997) argued that it was premature and potentially overreaching to adopt the testing procedure and benchmarking values reported by Logan (1997) because the repeatability of the LBPT had not been verified in other locations and its variability was considered high within the sample size. Following this situation, Russell and Paulsgrove (1999) compared the three most common pullout testing methodologies proposed by different researchers to examine bond of prestressing strands, namely, the LBPT (Moustafa, 1974), the PTI pullout test in grout (Post-Tensioning Institute, 1998), and the friction bond pullout test. The researchers found out that the LBPT and PTI pullout test results benchmarked bond at different force values across different laboratories and precast plants, whereas the friction bond pullout test did not show consistent results within the same testing facility. As a result, the authors' recommendation was to refine further the above methods to eliminate the inconsistencies in testing variables, such as loading rate, number of specimens, and the type of mortar used, and eliminate the latter from the recommended testing methods. These recommendations became the basis for the "North American Strand Producers (NASP) test Round Two".

To eliminate the inconsistencies listed above, Russell and Paulsgrove (1999b) proposed a new testing method based on the PTI bond test and compared it to the PTI bond test and the LBPT. In this test, the cement mortar was replaced by a sand-cement mortar, the strand slip was measured at different values, and the geometry of the specimens was modified to limit the variability of results. These researchers reported that the LBPT failed to yield consistent results across multiple locations, but good correlation at the same facility, which indicates that the testing is operator-sensitive, whereas the PTI test showed a direct relationship to pullout force, but the R^2 values varied depending on the location. The NASP test showed comparable pullout forces to the PTI bond test for the different strand slip values, and the R^2 values were almost entirely correlated to the forces. These results were repeatable and reproducible across multiple facilities and users, which indicated the high reliability of the method. Following these findings, multiple rounds of the NASP test were performed to ensure its reproducibility and to establish quality control measures, such as handling, testing duration, curing time, temperature, and relative humidity of the environment. This eventually transformed the method into the ASTM Standard A1081 (ASTM International, 2015)

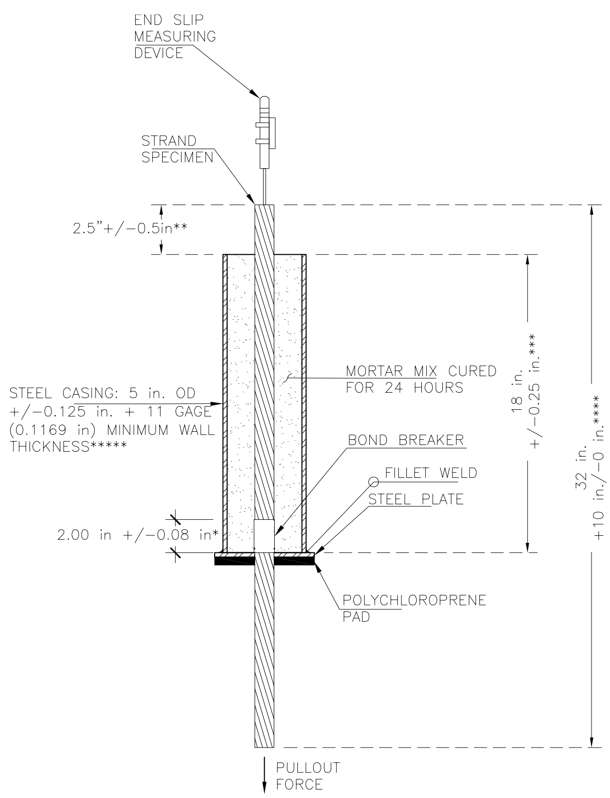


Figure 2-3 NASP/ASTM A1081 Specimen

2.2.3 2001-2015

Martí, Serna, Fernández, & Miguel (2002), created a new methodology to evaluate the bond capacity of steel strands, known as the ECADA (test for the characterization of pretensioned steel bond). The method consists of six stages, each of them emulating a precast prestressed concrete member fabrication, as shown on Figure 2-4. The first stage consists of hand-tightening the strands on the prestressing bed, the second step corresponds to the steel tensioning; the third step consists of anchoring the steel at the ends. After that, a concrete prism of arbitrary dimensions is poured. Lastly, the specimen is stripped from the forms, then the strand is released after the concrete reaches the strength (usually on the second day), and the strand is pulled out from the concrete. The pullout force is usually correlated to transfer and development length results to determine whether this force is adequate.

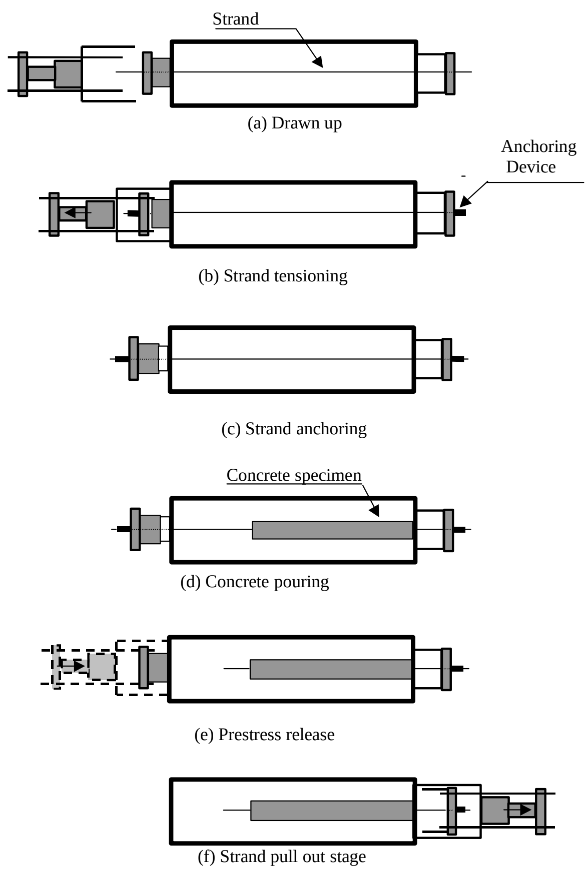


Figure 2-4 ECADA Test procedure

Peterman (2009), also developed a new quality assurance methodology to evaluate prestressing strand bond to concrete. This testing procedure consists of an 8 in. wide x 6 in. depth x 11.5 ft prestressed beam, reinforced with a single 1/2 in. strand tested in flexure using a four-point bending load arrangement, see Figure 2-5. The beam is gradually loaded to 85% of the nominal bending moment capacity and then this load is sustained for a minimum of 24 hrs. to evaluate uncharacteristic signs of distress, that can be linked to poor bond behavior. If the beam can hold the load for at least one day, then it is loaded to the maximum capacity for at least 10 minutes. If it does not collapse, then the bond of the strand is adequate, and the beam has passed the quality assurance test. This method is particularly intriguing because, unlike all the other testing procedures, it explores the significance of the bond for a sustained load.

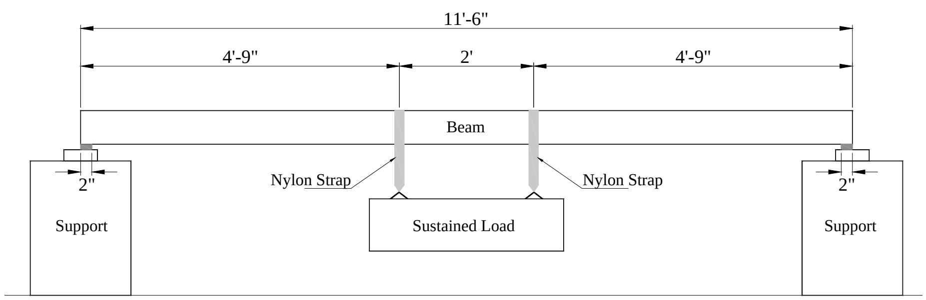


Figure 2-5 Peterman Bond Test layout (recreated from (Peterman, 2009)*)*

The Pennsylvania State Department of Transportation (PENN DOT) has also developed and implemented its own method to assess bond, regarded as the direct-tension pullout test, which is akin to the ECADA test but employs a more straightforward procedure and layout. This testing procedure was developed by Naito, Cetisli, & Tate (2015), and compared to the three major bond assessment tests, namely the ASTM A1081, the Large Block Pullout Test, the flexural beam test, and the PCI beam test. This research found out that the direct pull out test is more sensitive to the changes in the concrete type and surface condition and provides a closer emulation of the precast prestressed concrete member fabrication. However, since there has not been an interlaboratory test to evaluate variability and reproducibility, it cannot be widely adopted.

2.3 Transfer and Development Length

Research on transfer and development length of strands in precast prestressed concrete has always been a complex and controversial matter; therefore, this section does not intend to cover all of it, but the most relevant experimental and theoretical programs to this date. In order to fully understand the development of this section, it is necessary to define two key concepts, namely transfer and development length. The transfer or transmission length, in pretensioned concrete, is the distance needed for the prestressing steel to develop the effective prestress (f_{pe}), see Figure 2-6. Over that length, the assumption that plane sections remain plane does not hold due to high-stress concentrations and a non-linear bond-slip relationship. Beyond that length, the effective prestress is nearly constant and plane sections remain plane after deformations due to the pre-compression state. On the other hand, the development length is the length, in addition to the transfer length, that pre-tensioned member needs to fully develop the nominal moment capacity of the cross-section while reaching the design stress in the strand (fps). Both parameters are highly dependent on several variables such as the concrete compressive strength, the degree of confinement of concrete, type of concrete (SCC, lightweight, etc.), surface condition, type of prestress release, and the structural member's dimensions, which the ACI code conservatively does not consider, see equation Eq. (2-1) (Maguire et al., 2012; George Morcous et al., 2011; Quezada et al., 2018).

$$L\_d = (fpe/3000)\ d\_b + ((fps - fpe)/1000)\ db \qquad \textit{Eq. (2-1)}$$

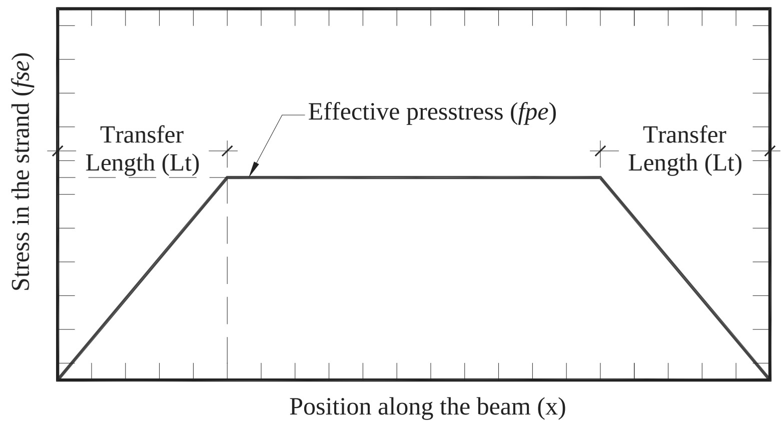


Figure 2-6 illustration of the transfer length

2.3.1 Research Prior to 1990

Research on transfer length can be traced back to the work of Hanson & Kaar (1959). This pair of researchers tested 47 beams in flexure to study the nature of the bond, transfer bond and mode of failure. The scope of this work included the influence of grade 250 strand sizes, which were 1/4, 3/8, and ½-in.; reinforcement ratio variation from 0.274 to 0.991; embedment length, which varied from 27 to 174 in.; and different concrete strengths, *f'c* = 3700 to 7800 psi. These researchers found out that strand size and embedment length have a significant effect on the value of the average bond stress at which bond-slip occurs. They additionally found that the increase in reinforcing ration or reduction in concrete strength reduces the general bond-slip and that the rusting of the strands increased the service and ultimate moment. However, it was found in another study (Kaar et al., 1963), that the transfer length slightly decreases with a concrete strength increase, but they were not correlated.

In response to the confusion among designers and producers of precast pretensioned concrete on the development length required in the code and the failure of a hollow core slab during construction, Martin & Scott (1976) tested a similar slab to the one failed, and it resulted on a bond failure at the 85% of the calculated ultimate strength.

These researchers also evaluated the existing research data that led to the creation of Eq. (2-1) and proposed Eq. (2-) and Eq. (2-3), which set a limit on *fps*:

For $l_x < 80d_b$": "

$$f_{ps} \leq \frac{l_x}{80d_b}\left(\frac{135}{d_b^{1/6}}\right) + 31 \quad \text{Eq. (2-2)}$$

$$f_{ps} \leq \left(\frac{135}{d_b^{1/6}}\right) + 0.39\frac{lx}{d_b} \quad \text{Eq. (2-3)}$$

The results presented above (Martin & Scott, 1976) lead to the reliability evaluation of the Eq. (2-1) by Zia & Mostafa (1977). These researchers reviewed the extensively available data on transfer and development length at the time, the AASHTO and ACI code transfer and development length equations and proposed a new set of equations to include the concrete strength in them, see Eq. (2-4) and Eq. (2-5). The range of applicability was 2000 to 8000 psi for concrete strength, and the prestressing steel grade was grade 250.

1

$$l_t = 1.5\frac{f_{pi}}{f_{ci}} - 4.6 \quad \text{Eq. (2-4)}$$

$$l_d = \left(1.5\frac{f_{pi}}{f_{ci}} - 4.6\right) + 1.25(f_{ps} - f_{pe}) \quad \text{Eq. (2-5)}$$

2.3.2 Research from 1990-2000

As precast prestressed concrete construction advanced and new materials were developed, the corrosion in pre-tensioned bridges raised a significant concern throughout the industry drawing more interest in epoxy-coated strands. However, the development length of coated reinforcement was known to be longer according to the concrete fundamentals; therefore, questioning of the applicability of those strands regarding good bonding came into discussion. In addition, the FHWA memorandum (1988) prohibited the use of 0.6 in. strands in pre-tensioned applications, among other restrictions, in response to the findings by the PCI and PCA.

In response to the needs for an equation considering the use of high-strength epoxy-coated strands, Cousins, Johnston, & Zia (1990) studied the effect of those parameters in the transfer and development length. These researchers tested concrete prisms reinforced with grade 270 uncoated and epoxy-coated 3/8 in., 1/2 in. and 0.6 in. strands. Concrete strength ranged from 4100 psi to 6900 psi for the specimens tested. These researchers concluded that the ACI and Zia & Mostafa (1977) equations underestimate the transfer and development length of bare strands, but overestimate the ones for 3/8" coated strands, but underestimate the 1/2 in. and 0.6 in. diameter medium coated strands. They also proposed Eq. (2-6)-(2.8) to determine the transfer length and development length for strands

$$l_t = 0.5\left(\frac{U'_t\sqrt{f'_{ci}}}{B}\right) + \frac{f_{se} A_s}{\pi d U'_t \sqrt{f'_{ci}}} \qquad \textit{Eq. (2-6)}$$

$$l_{fb} = \frac{(f_{ps} - f_{pe}) A_s}{\pi d U'_d \sqrt{f'_c}} \qquad \textit{Eq. (2-7)}$$

$$l_d = l_t + l_{fb} \qquad \textit{Eq. (2-8)}$$

Where:

U'_t	Plastic transfer bond stress coefficient
U'_d	Plastic bond stress coefficient of development
B	Bond Modulus
$f'ci$	Compressive strength of concrete at release
$f'c$	Compressive strength of concrete at 28-days
d	Diameter of strands
fse	Effective prestress after transfer
fps	Stress in prestressing strand at nominal strength
l_{fb}	Length of flexural bond

Shahawy, Issa, & Batchelor (1992) conducted experimental and analytical research on AASHTO Type II girders to determine the adequacy of the equations presented in the ACI Code and the AASHTO specification. The girders were 41 ft. long precast pre-tensioned concrete, all of them designed for the same nominal strength by a local contractor. These researchers studied the influence of 0.5 in. and 0.6 in. uncoated strands, shear reinforcement and strand shielding on the transfer and development lengths. They found out that the provisions at the time provided inaccurate predictions for transfer and development length, and that strand shielding increased the transfer length to a distance equal to the shielding distance plus the transfer length of the strand. They also encounter that spacing equal to four times the diameter of the strands used as the spacing was too conservative because they implemented a spacing of two inches in specific beams and did not detect any sign of distress nor cracking in the girders.

Mitchell, Cook, & Tham (1993), tested 22 precast pre-tensioned girders to assess the impact of compressive strength of concrete on the transmission and development length of strands. This parameter varied from 4500 psi to 12,900 psi, whereas the strand diameters were 3/8 in., 1/2 in., and 0.62 in. These researchers found out that concrete strength at release is negatively correlated with the transfer and flexural bond lengths. Hence, an increase in concrete strength at release results in a decrease in development length. They suggested substituting *fse* with f_{pi} in the code equation Eq. (2-1), and to introduce a factor to include the concrete strength in such an equation, see Eq. (2-9):

$$L\_d = (f\_pe/3000)\ d\_b\ \sqrt{(3000/f'\_ci\ )} + ((f\_ps - f\_pe)/1000)\ d\_b\ \sqrt{(4{,}500/f'\_ci\ )} \qquad \textit{Eq. (2-9)}$$

To respond to the FHWA 1988 memorandum that tightened the requirements for several strand sizes and forbade the use of 0.6 strands for pre-tensioned applications, Deatherage, Burdette, & Chew (1994), studied the transmission length for 1/2 in., 1/2 in. special, 9/16 in., and 0.6 in. strands. They also studied the spacing requirements for 1/2 in. strand. The sixteen beams were 31 ft. long I-shaped girders, divided into four groups and the spacing for the strands was 2 in. for all beams but one set, in which the spacing was 1.75 in. The prestressing steel was grade 270, low-relaxation strand. These researchers determined that the performance of girders is not affected by strand spacing or the use of 0.6 in. strands; thus, large-diameter strands should be allowed. They also encounter that the increase in 60% of the development length by the AASHTO was not justified by the results. In an effort to clarify the discrepancies in research done after the FHWA 1988 memorandum, Buckner (1995), reviewed the literature on the topic, analyzed the data on transfer and development length, and recommended new equations for transfer and development length that were "more consistent" with the state of the knowledge and practice, see Eq. (2-10) and Eq. (2-11).

$$L\_t = (f\_si/3000)\ d\_b \qquad \textit{Eq. (2-10)}$$

$$l\_d = (f\_si/3000)\ d\_b + (\lambda(f\_ps - f\_se)d\_b)/1000 \qquad \textit{Eq. (2-11)}$$

Russell & Burns (1997) studied the transfer and development length of pre-tensioned concrete beams reinforced with grade 270 low relaxation strands. The specimens were concentrically prestressed prisms with 4 in. x 6 in. x 12 ft. dimensions. These researchers found out that the expressions used to compute transfer and development length underpredicted the real values for members with debonded strands and that 0.6 in strands properly bond to the concrete of moderate and high compressive strength.

Finally, in May 1998, the pre-tensioned concrete industry and research community received the news from the FHWA that 0.6 in. spaced at 2 in. were appropriate for their use in precast concrete applications (Lane & Rekenthaler Jr., 1998). The FHWA also accepted the introduction of 0.5 in. spaced at 1.75 in. on center.

2.3.3 Research from 2001-2010

Barnes, Grove, & Burns (2003) investigated the effect of concrete strength, release method and strands' surface condition on the transfer and development length of 0.6 in. strands. These researchers fabricated 36 AASHTO Type I girders, all of them prestressed with the same strand diameter, spaced 2 in. on center, and their surface condition was either bright or rusted. The concrete used in this research varied from 5700 to 14,700 psi. The results of this research showed that a negative correlation exists between concrete strength and bond length; that rusted strands exhibit shorter transfer length vs. clean strands up to strengths of 7,000 psi; and a positive correlation between transfer length and time during the first 28 days from casting, that is, the transfer length increases with time.

Girgis & Tuan (2005), researched the effect of using self-consolidating concrete (SCC) on a transfer length of 0.6 in. strands. These researchers examined three different projects in which the combination of SCC, 0.6 in. diameter strands and NU I-Girders was implemented. The first project comprised a 72.5 ft. long girder, the second a 90 ft., and the third a 124 ft. The results revealed that SCC adversely affects the transfer length of 0.6 in. strands at release due to the low early age bond strength of this type of concrete. Kose & Burkett (2005), reevaluated the data and equations for transfer and development length to that date and developed new expressions to estimate those lengths more accurately, see Eq. (2-12) - Eq. (2-14).

$$l\_t = 95\ (f\_pi\ (1 - d\_b\ )\text{^}2)/\sqrt{}(f'\_c\ ) \qquad \textit{Eq. (2-12)}$$

$$l\_fb = 1.7 + 328.6(f\_ps - f\_se)\ \ (1 - d\_b\ )\text{^}2/\sqrt{}(f'\_c\ ) \qquad \textit{Eq. (2-13)}$$

$$l\_d = [95\ (f\_pi\ (1 - d\_b\ )\text{^}2)/\sqrt{}(f'\_c\ )] + [8 + 400(f\_pu - f\_pi)\ (1 - d\_b\ )\text{^}2/\sqrt{}(f'\_c\ )] \qquad \textit{Eq. (2-14)}$$

Ramirez & Russell (2008), tested forty-three 6.5 in. x 12 in. x 17 ft. and eight 10 in. x 24 in. x 24 ft. I-shaped beams with concrete strengths (*f'c*) varying from 6,000 psi to 16,000 psi, reinforced with 0.5 in., or 0.6 in. strands. They found out that code equations were conservative for high-strength concrete and proposed the following equations:

$$L_t = (120/\sqrt{f'_{ci}})\, d_b \leq 40 d_b \qquad \textit{Eq. (2-15)}$$

$$l_d = (120/\sqrt{f'_{ci}})\, d_b + (225/\sqrt{f'_c})\, d_b \geq 100 d_b \qquad \textit{Eq. (2-16)}$$

2.3.4 Research from 2011-2019

Morcous, Hatami, Maguire, Hanna, & Tadros (2012) studied mechanical properties and bond, and Maguire, Morcous, & Tadros (2012) researched the transfer and development length of grade 270 0.7 in. diameter, low-relaxation strands spaced at 2 in. on high-strength bridge girders. The concrete was self-consolidating and had a strength of 12 ksi and 15 ksi. These researchers found out that transfer length of these strands was shorter than the ones predicted by the equation in both the ACI and AASHTO LFRD codes for harped strands.

Vázquez-Herrero, Martínez-Lage, & Martínez-Abella (2013), studied the effect of lightweight concrete vs. normal-weight concrete on the transfer length of 0.6 in. strands. To accomplish this task, the researchers fabricated rectangular beams reinforced with grade 270, low relaxation strands spaced 2 in. on center. These researchers found out that the Eurocode 2 equations provide an unsafe prediction of the transfer length for the studied conditions.

Bai & Davidson (2016), applied the composite beam theory to the study of prestressed concrete girders. In this novel application, the authors derived continuous functions to determine the strain profile of the prestressing steel and identified a new equation for transfer length, which is similar to the one by Guyon (1953). This equation is particularly interesting because it was mathematically derived using the theory of elasticity and the beam on elastic foundation principle, unlike most equations to the date. According to these researchers, the transfer length is dependent on the modulus of elasticity of concrete and prestressing steel, the area of the concrete section and strands, the eccentricity of the strands relative to the centroid of the cross-section, the moment of inertia of the section, and the tolerance of axial force at mid-span (see Eq. (2-17)).

$$L\_t = \varphi\_pe/\varepsilon\_is \;\; ln(1/(1-\gamma)) \qquad \textit{Eq. (2-17)}$$

Where:

φ_{pe} =	$\varphi_p(l/2)$, slip at the end of the beam due to prestressing force
ε_is	Strain resulting from prestressing, before the transfer, in the prestressing tendon
γ	Tolerance of axial force at mid-span

Salazar et al. (2017), parametrically studied the benefits of using 0.7 in. versus 0.5 in. and 0.6 in. strands diameter strands in pre-tensioned bridge girders. The study comprised thousands of design cases for AAHTO I-beams, and bulb tees, as well as TxDOT Bulb tees, spread box beams, and U beams. These researchers found out that 0.7 in. diameter strands are the most efficient among the studied sizes in terms of steel quantity needed to achieve a required precompression force, span length, and slenderness of the section. However, the authors pointed out that a release strength of 10 ksi is often required to achieve the most benefits.

Carroll, Cousins, & Roberts-Wollmann (2017), explored the use of grade 300 in pre-tensioned concrete members. The study consisted of a series of 18 transfer length specimens and 35 flexural bond tests. The results of this investigation showed increases of transfer lengths for grade 300 versus grade 250 strands, whereas the flexural tests displayed an increase in nominal capacity, but less ductility.

Greene & Graybeal (2019), studied the effect of high-strength lightweight concrete on the transfer and development length of 18 I-beams prestressed with 0.5 in. and 0.6 in. diameter strands. The concrete strength was between 6-10 ksi, and the length of the specimens was 45 ft. These researchers found that the expressions to calculate transfer length from the AASHTO LRFD Bridge Design Specification and the ACI Building Code overestimate the transmission length but underestimate the flexural bond length for the type of concrete studied.

CHAPTER 3 Experimental Program

3.1 Introduction

There are numerous methods to evaluate the bond of prestressing strands to concrete; however, some of them can be complex and difficult to replicate for researchers precasters routinely. Pullout tests give a direct measurement of the mechanical bond of steel reinforcement to concrete, which is one of the primary mechanisms of force transfer in prestressed concrete (Maguire et al., 2017). The Standard Test Method for Evaluating Bond of Seven-Wire Steel Prestressing Strand (ASTM International, 2015) results for untensioned strands and the ECADA test for tensioned strands have proven to be the most reliable, reproducible and repeatable testing among all test; however, developing a full study employing the latter can be strenuous and time consuming if the proper equipment is not available. Other tests, such as the LBPT (Ramirez & Russell, 2008) and untensioned prisms (Jiang et al., 2017), can reliably assess bond performance but have not been standardized for prestressing applications. In this study, an experimental program to characterize the bond performance of 1 1/8" diameter strands have been performed employing the ASTM A1081 standard, the LBPT, and concentrically reinforced pullout untensioned prims.

3.2 Big Block Pullout test (LBPT)

The LBPT, also known as the Mostafa Pull Out Test, consists of pulling out prestressing steel strands embedded into a concrete block. This testing comprised strands in the "as received" condition of 1/2 in. and 1-1/8 in. diameter in high strength concrete and rusted 1-1/8 in. strands in normal strength concrete.

3.2.1 *Specimen Fabrication*

In this research, a series of 18 in. x 24 in. x 56 in. or 116 in. reinforced concrete blocks with six prestressing strands embedded into them, as shown in Figure 3-2 and Figure 3-1, was built to evaluate the pullout capacity of the strands in normal-weight concrete. The strands were debonded from both ends using a 2 in. PVC pipe bond breaker around the strand at the live-end and 4 in. on the free-end to avoid stress concentration and the abrupt failure of concrete near the surface.

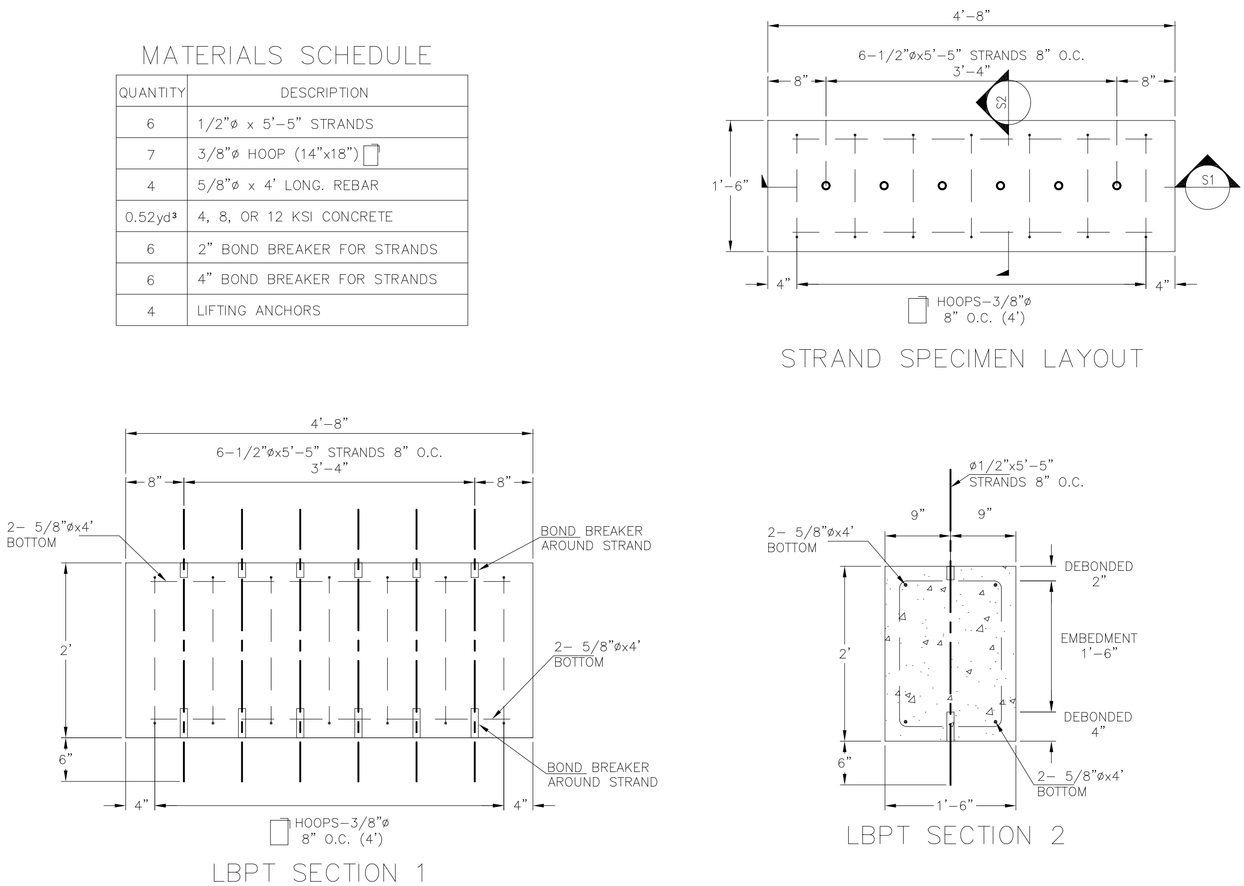
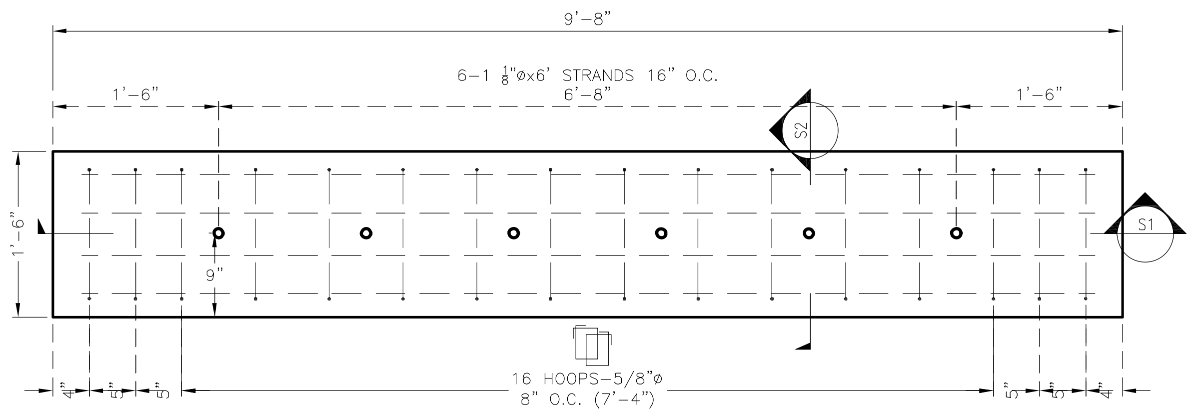
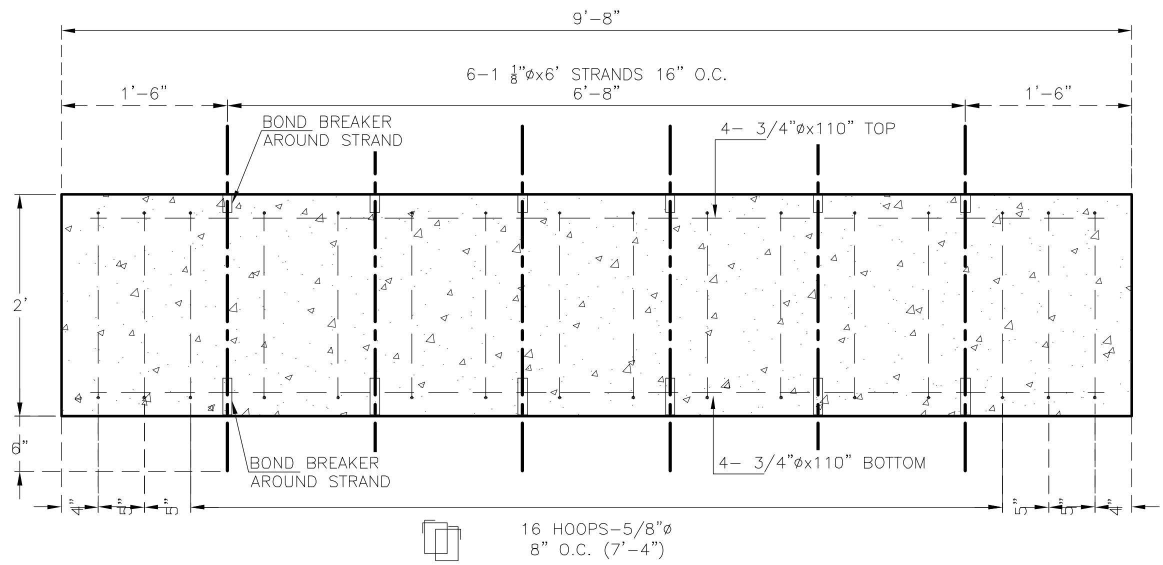


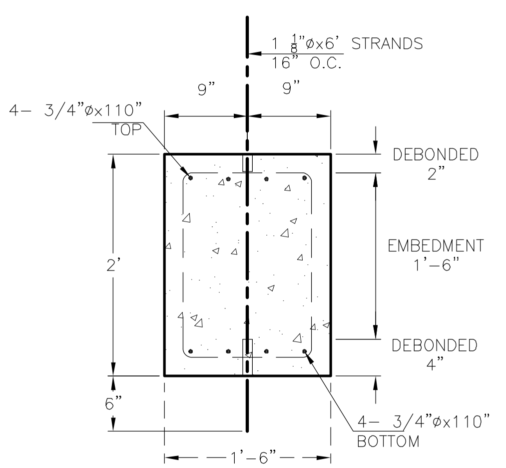
Figure 3-1 1/2 in. LBPT Specimen dimensions and strand layout



LBPT PLAN VIEW



LBPT SECTION 1



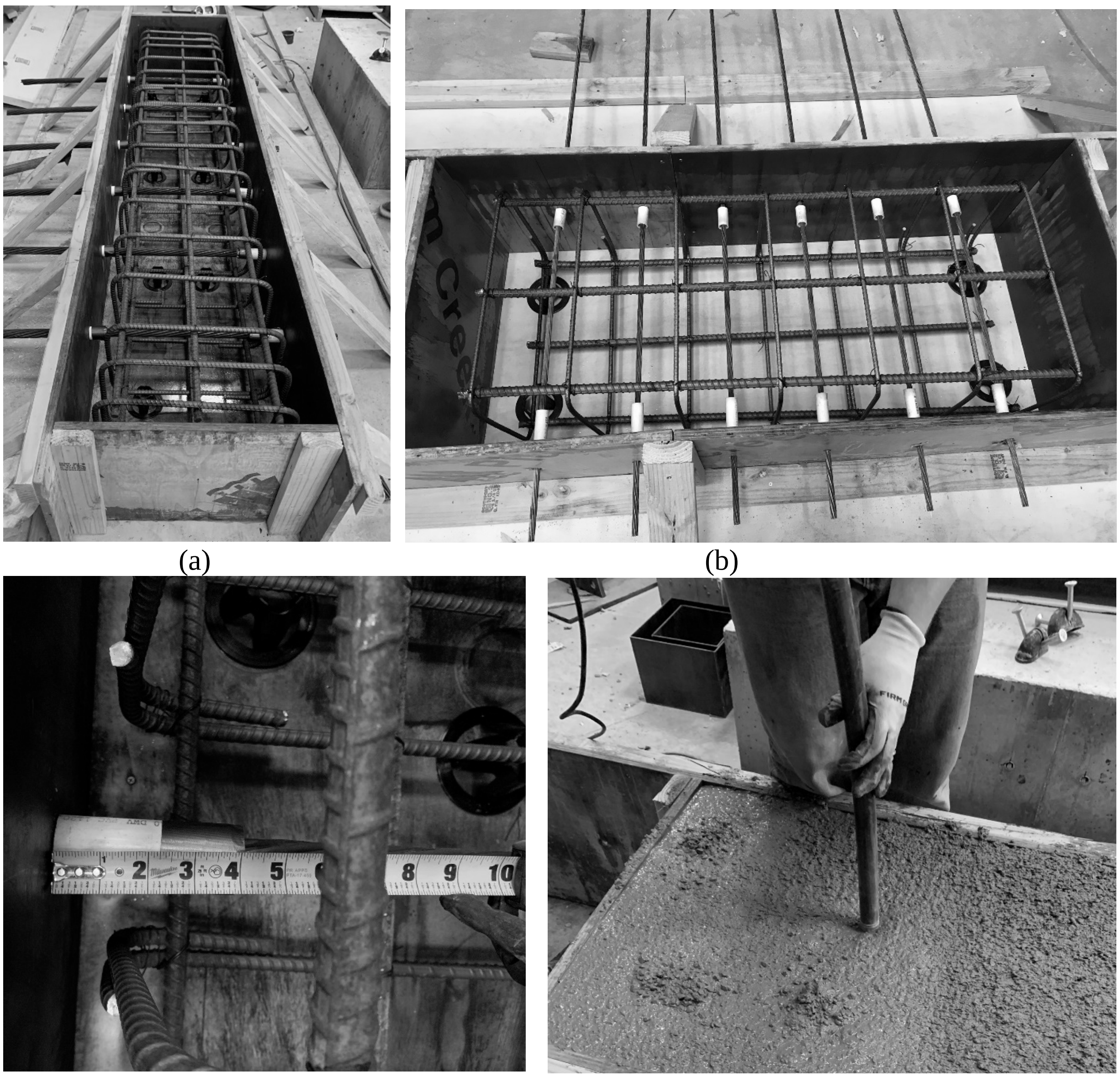
LBPT SECTION 2

MATERIALS SCHEDULE

QUANTITY	DESCRIPTION
6	1 1/8"ø x 6' STRANDS
16	5/8"ø HOOP (10"x20")
8	3/4"ø x 9'-2" LONG. REBAR
1.25 yd³	5 KSI CONCRETE
6	2" BOND BREAKER FOR STRANDS
6	4" BOND BREAKER FOR STRANDS
12	PLASTIC CYLINDERS (4"ø x 8")

Figure 3-2 1-1/8 in. LBPT Specimen dimensions and strand layout

Large block specimens were cast horizontally in layers of no more than 12 in. to ensure compaction of concrete properly, and to ease the testing process. All forms were built using 2 in by 4 in. dimensional lumber and high-density overlay (HDO) board joined with wood screws. All the steel used for these specimens was bent by a local rebar fabricator and the block anchors were purchased from a certified provider. All concrete was normal weight concrete purchased from a locally available ready-mix producer.

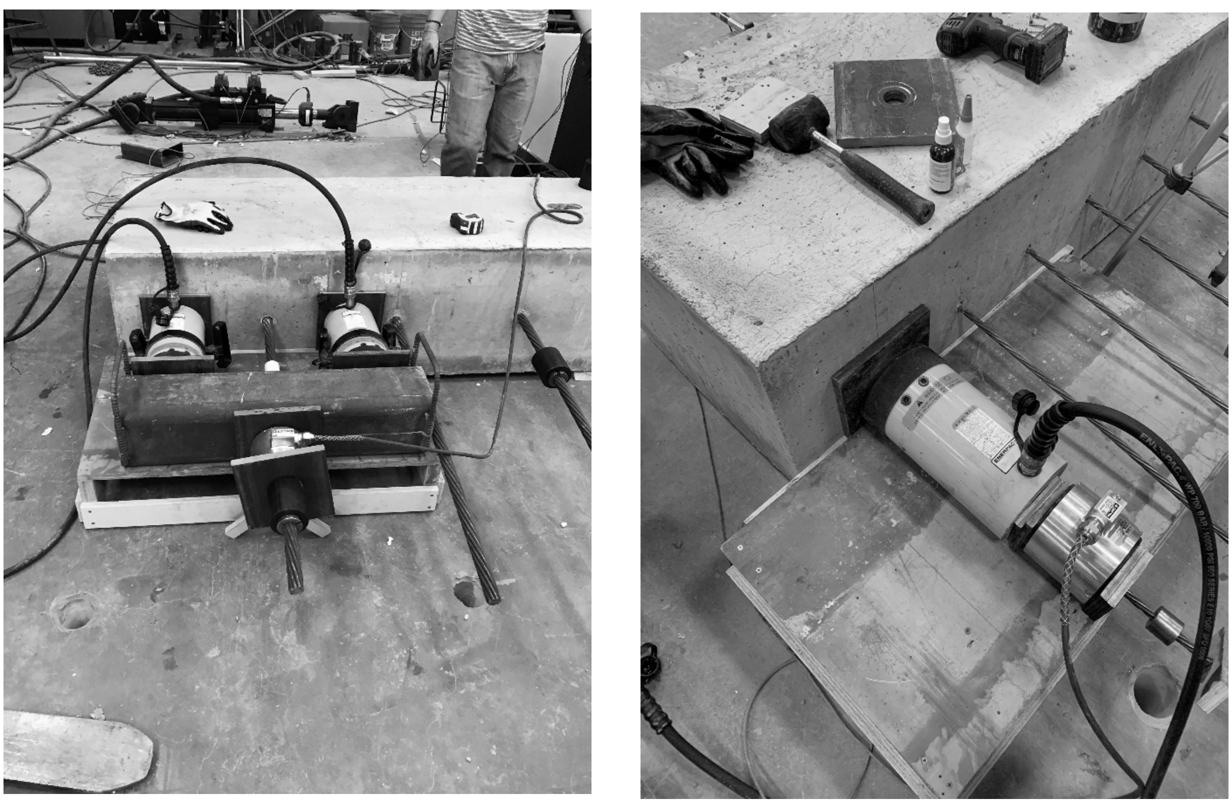


(a) (b) (c) (d)

Figure 3-3 LBPT Specimen fabrication: (a) Big strand specimen ready to be poured, (b) control specimen, (c) 4-inch debonding length on the dead-end verification, (d) vibration of concrete.

3.2.2 Experimental setup

The experimental testing described in this section consisted of a pair of hydraulic jacks assembled in parallel to pull out the strands from the big blocks. The load is applied at an approximate rate of 20 kip/min for the 0.5 in. diameter strands, used as a control-specimens, and 40 kip/min for the 1-1/8 in. diameter strands. The load was recorded using a calibrated and verified load cell, whereas strand slip was measured using Linear Variable Differential Transformer (LVDT) sensors with an accuracy of 0.001 in. The hydraulic jacks were shimmed in all cases to prevent deterioration of their bases and to help the load cell read the load more uniformly. A spreader beam was used for the larger strand pull out test to overcome the pressure constraint generated by the pump pressure controller. This beam was filled with a 4 ksi concrete mix to ensure the shape would locally not buckle under the imposed load.



(a) (b)

Figure 3-4 LBPT Specimen test setup: (a) 1.125 in. [28.6 mm] strand test, (b) 0.5 in. [12.7 mm.] strand test

3.3 ASTM 1081: Standard Test Method for Evaluating Bond of Seven-Wire Steel Prestressing Strand

The testing procedure consisted of 5 in. outside diameter (O.D.) by 16 in. long steel pipe, with a welded plate on one end to encase the mortar and to ensure no water from the mix was lost and to provide a flat finish on one end. A 1.5 in. hole is made on the plate for passing the 0.5 in. or the 1.125 in. diameter strand, and a 2 in. PVC pipe bond breaker is used around the strand to reduce stress concentration on the mortar at the endplate, see Figure 3-5.

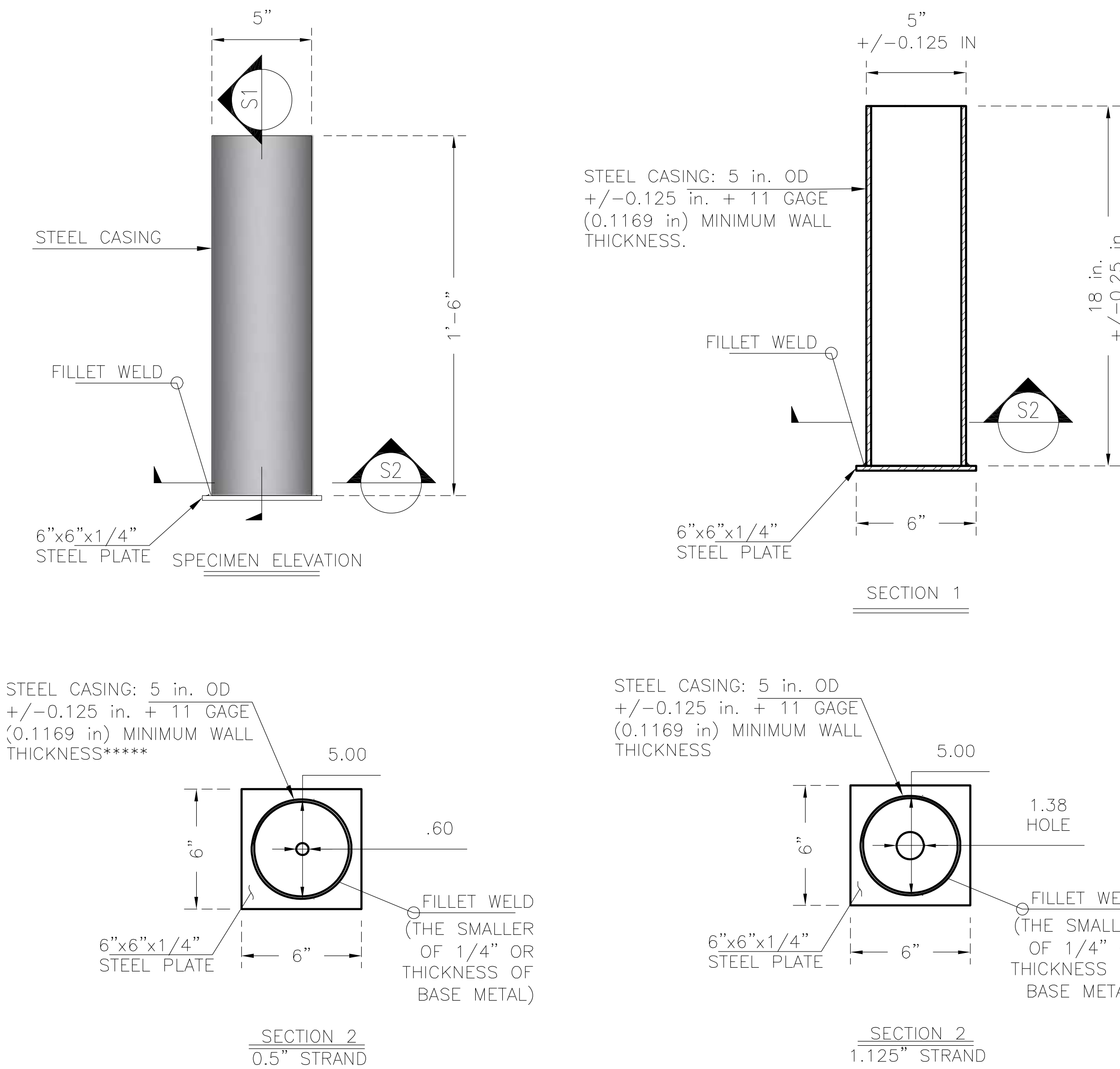
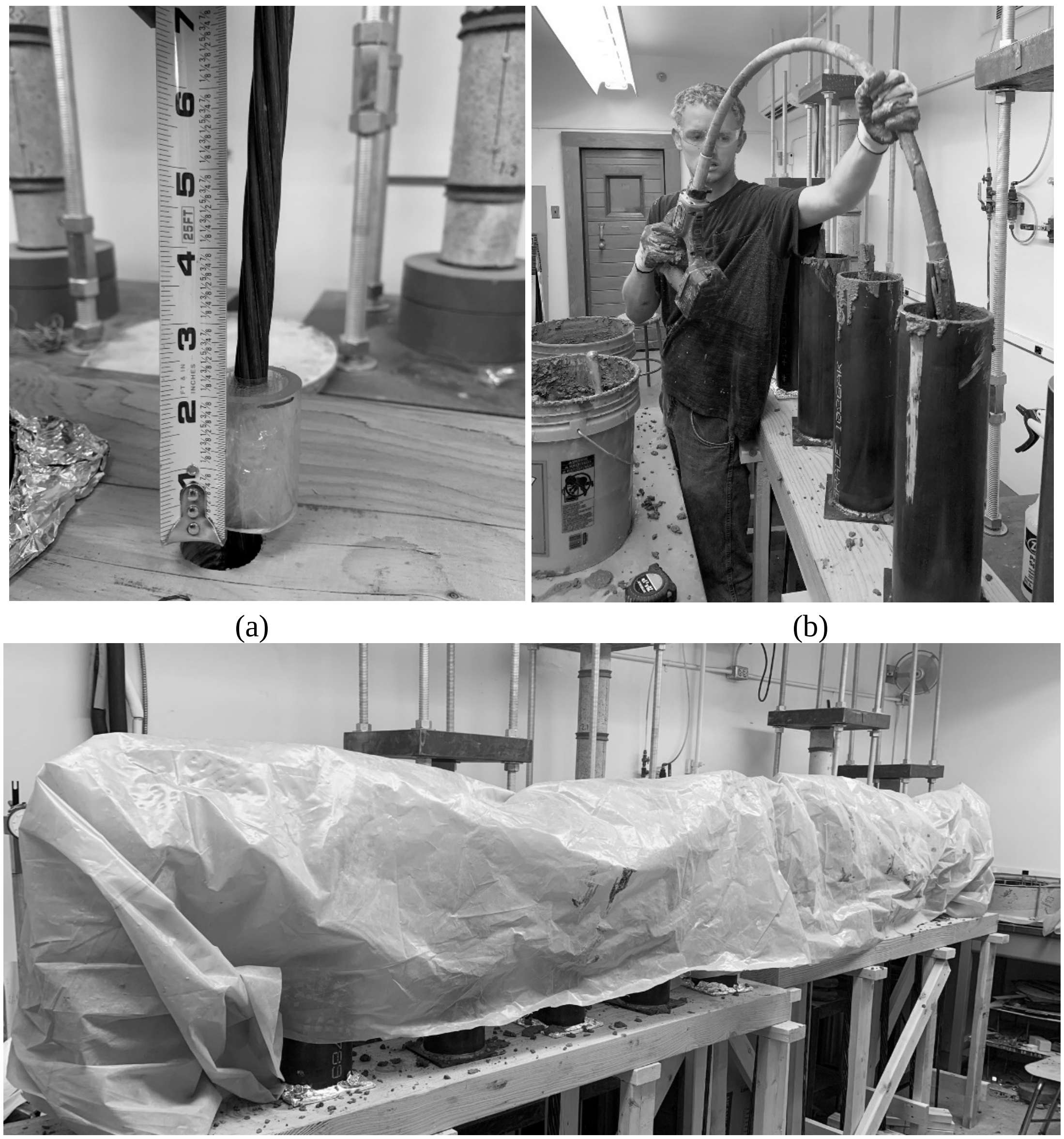


Figure 3-5 Illustration of the ASTM A1081 casing dimensions.

3.3.1 Specimen Fabrication

The specimens were cast in vertical position and vibrated, as required by the corresponding standard, as shown in Figure 3-6. The mix designed used in this test was comprised of one part of cement, one part of sand and 0.425 parts of water measured by weight. Cement was type III per ASTM C150, and the sand was purchased from a local concrete aggregate seller complying with ASTM C33.



(a) (b)

(c)

Figure 3-6 Specimen pouring: (a) debonded length of the strand, (b) mortar placing, (c) specimens covered and curing.

The specimens were cured in a controlled environment for 24 hours +/-2 hours from casting the specimens, with a temperature of 23°C +/-2°C and relative humidity above 90% all the time. The compressive strength of the mortar is evaluated using 2 in. by 2 in. cubes, as indicated in the ASTM C109 MPa (ASTM, 2016), with a target mean compressive strength ranging from 4500 to 5000 psi. The flow of the mortar employed in the specimens shall be in the 100-125% range, measured according to the ASTM C1437 (ASTM, 2014).

3.3.2 Experimental setup

The specimens are mounted on a table to allow the specimen to rotate during the testing, and the strand is pulled out at a displacement rate of 0.1 in./min., and the load was recorded using a calibrated and verified load cell, whereas strand slip was measured on both ends using Linear Variable Differential Transformer (LVDT) sensors with an accuracy of 0.001 in. Figure 3-7 illustrates the specimen dimensions and the testing layout. After the process was completed, the strand was flame cut in order to remove all the parts of the assembly because the chucks used for 1.125 in. diameter strands were not reusable.

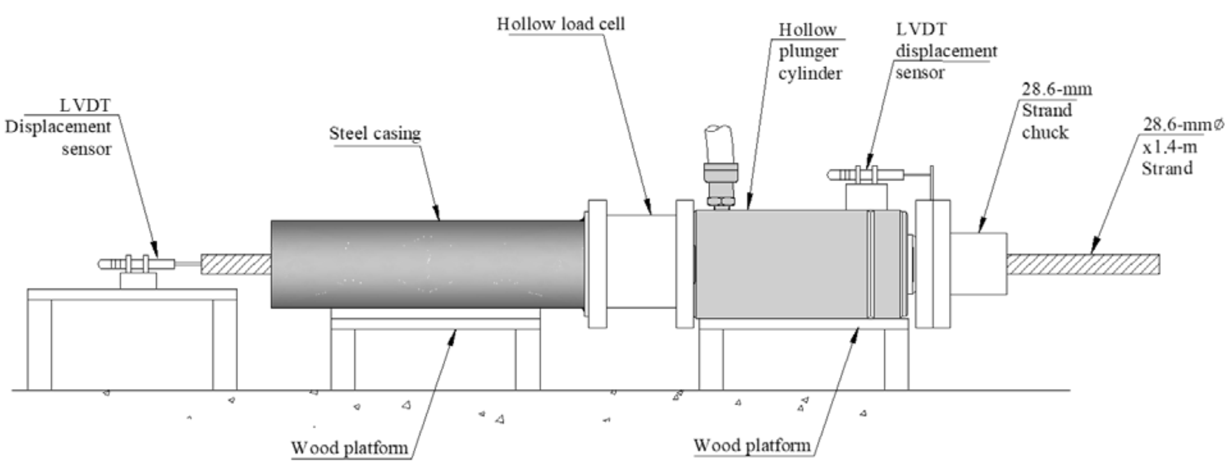


Figure 3-7 ASTM A1081 Test setup

3.4 Concentrically reinforced untensioned prisms (CRUP)

The objective of this test was to examine the effect of embedment length on the pullout force of untensioned strands. This test consisted of one series of 8 in. x 8 in., concentrically reinforced prisms with lengths varying from 4 to 8 ft.

3.4.1 Experimental setup

The specimens were tested on the floor using an assembly, as shown in Figure 3-8. The strands were not debonded from any of the ends, as recommended in (Jiang et al., 2017), and they were pulled out in increments of 5-10 kips per step. The load was recorded using a load cell, whereas strand slip was measured on both ends using Linear Variable Differential Transformer (LVDT) sensors with an accuracy of 0.001 in. After the process was completed, the strand was flame cut in order to remove all the parts of the assembly because the chucks used for 1.125 in. diameter strands were not reusable.

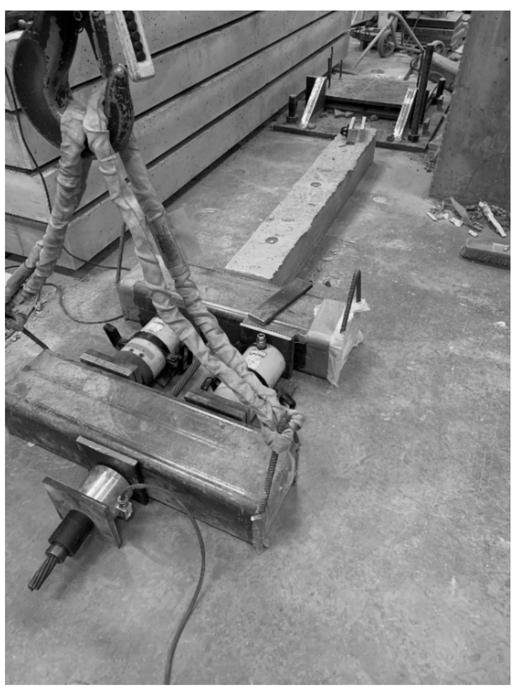


Figure 3-8 CRUP testing layout

3.5 Large Scale test

The large-scale testing of this research comprised six precast pre-tensioned T-beam specimens/ The objectives of these test were the following:

- Verify whether the prestressing force applied to the strand could be adequately transferred to the concrete beams.
- Determine the transfer and development length of the 1-1/8 in. strands on different concrete strengths.
- Investigate the flexural capacity of pre-tensioned beams reinforced with 1-1/8 in. strands.

3.5.1 Large-scale test specimen fabrication

A 22.5 ft. long T-beam was designed to avoid shear failure before flexural failure due to concentrated loads using both, the self-weight and the external load resulting from the member achieving its maximum flexural capacity. The beam was reinforced with one 1-1/8 in. diameter Grade 250 strand. The shear and additional longitudinal reinforcement were Grade 60 deformed steel, which size varied depending on the specimen. The web width was limited by the ability of local rebar fabricators to achieve the desired bending radius.

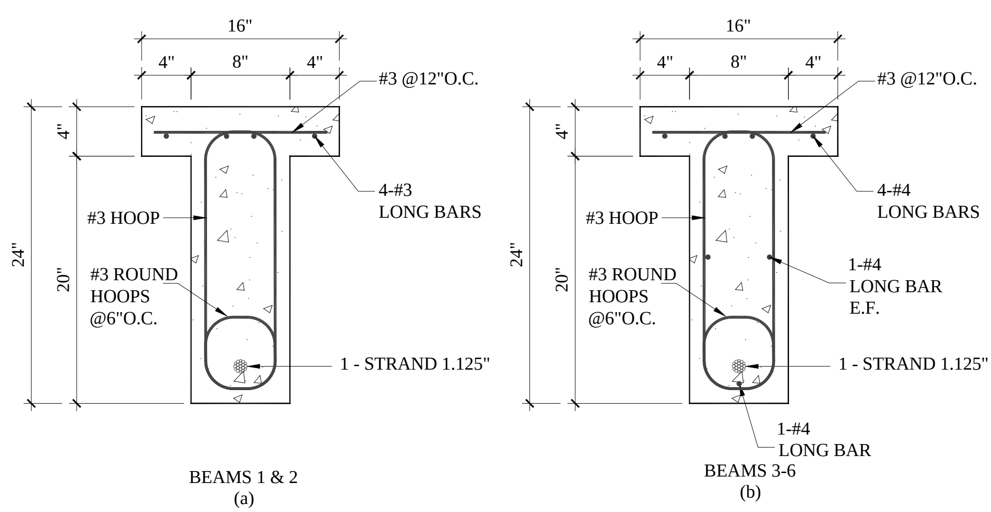


Figure 3-9 Cross section dimensions and reinforcing

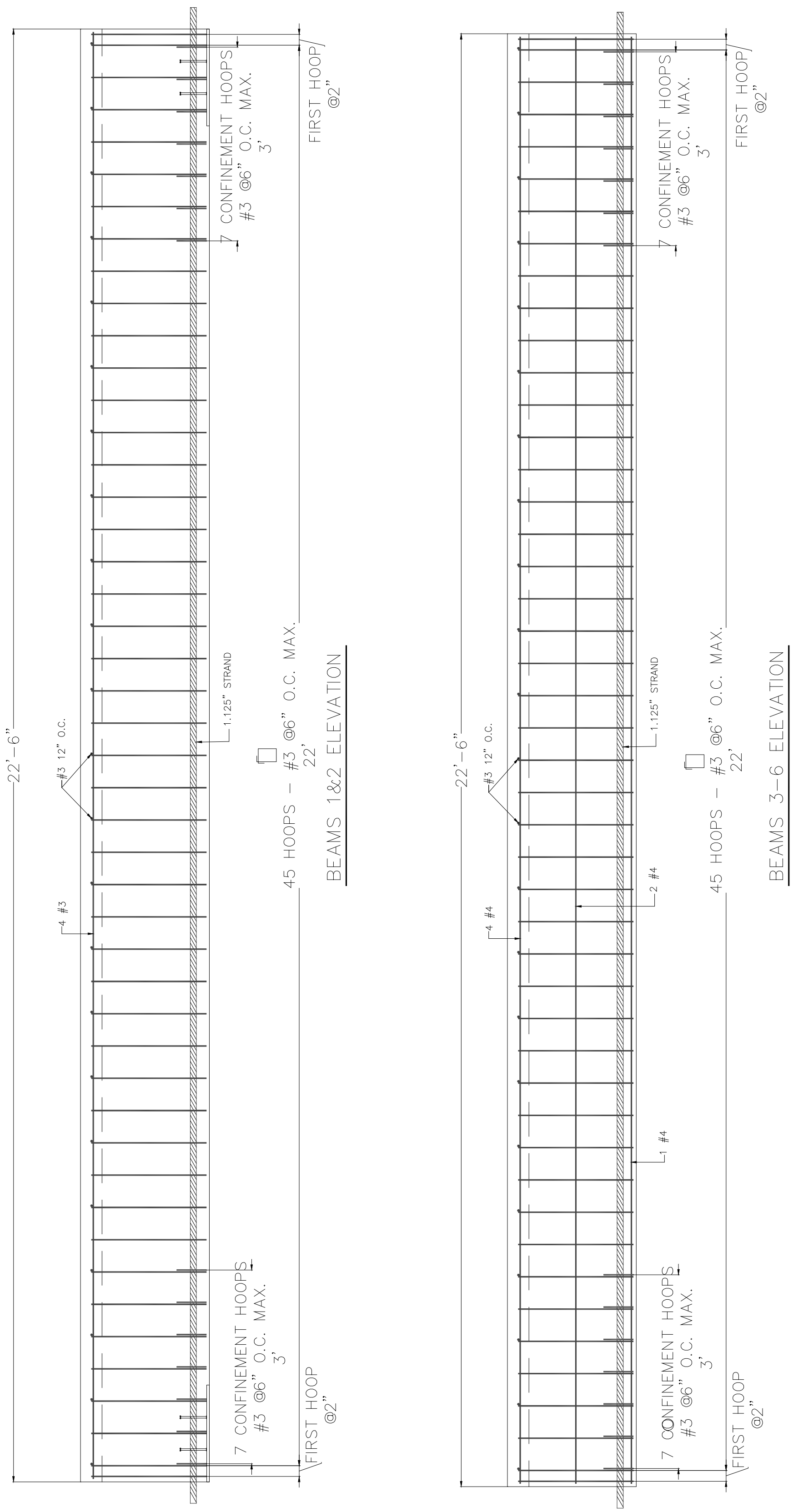


Figure 3-10 Elevation view of reinforcement for beams 1-8

Large-scale specimens were cast in layers of no more than 12 in. to ensure compaction of concrete properly, and to ease the testing process. Figure 3-11 shows the typical fabrication sequence of the beams, which forms were built using 2 x 4 wood and high-density overlay (HDO) board joined with wood screws. All the steel used for these specimens was bent by a local rebar fabricator, whereas all concrete was normal weight concrete purchased from the same local ready-mix producer.

(a)

(b)

(c)

(d)

Figure 3-11 Typical construction of the prestressed beams

3.5.2 Transfer length test set-up

Six full-scale beams were instrumented using Detachable Mechanical (DEMEC) strain gauge disks spaced six inches on center, attached to both sides along the beams, as shown in Figure 3-12. Strain readings were taken using an electronic gauge reader, fabricated by MASTRAD Limited, in the UK. The strain readings were taken before the release of pre-compression force, immediately after and before the flexural testing of the specimens. These readings were used to determine the transfer length of the strands for each beam that was fabricated.

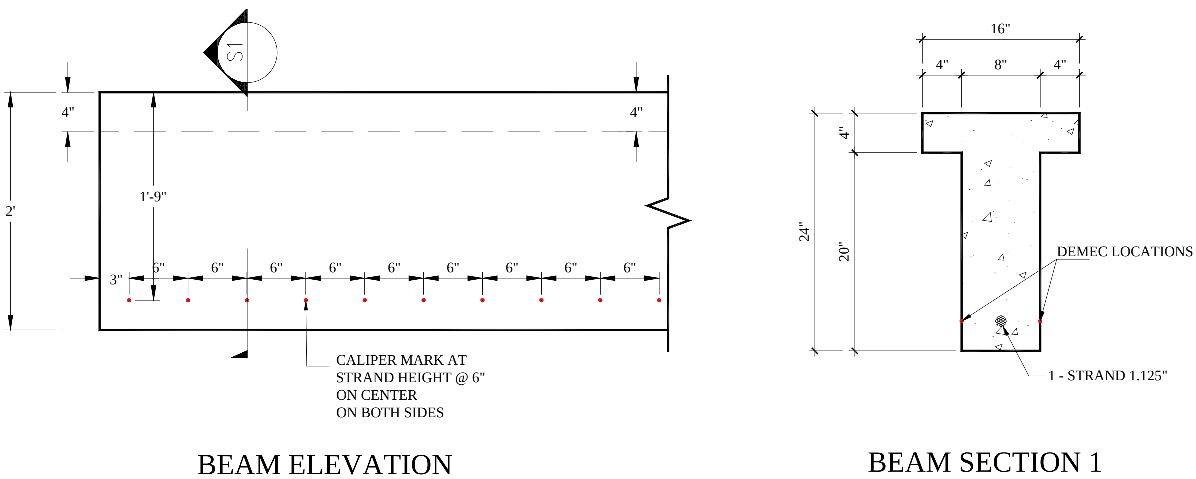


Figure 3-12 DEMEC locations for strain reading

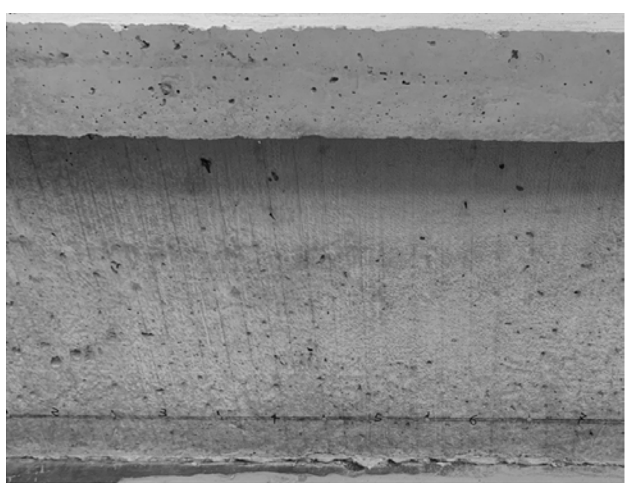
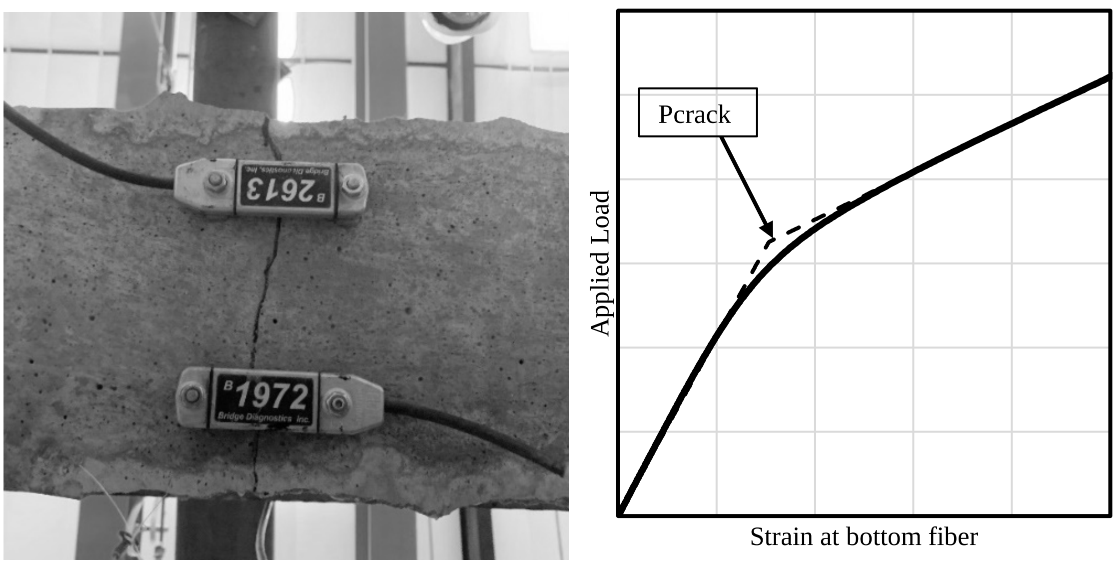


Figure 3-13 Caliper marks mounted on beam at the level of the strand

3.5.3 *Effective prestress and flexural test*

This portion of the experimental program comprised two different tests. The first one consisted of applying a load to the beam until the first visible crack opened. After that, two surface strain gauges were mounted on the bottom surface of the beam at the formed crack. Given the crack and the necessary load to open it, three cycles were performed, and the load and surface strains were measured at the bottom of the beam. These results were plotted, and the cracking load was determined by estimating the intersection between two tangent lines to the linear and non-linear portions of the test, see Figure 3-14. The location of the load is shown in Figure 3-15 through Figure 3-17.



(a) (b)

Figure 3-14 Illustration of the possible cracking load on a prestressed beam

After finding the load, the effective prestress was computed by finding the stress in the strand using the average cracking load, the self-weight, and assuming the stress at the bottom fiber is zero because the crack is open. The equation used in this process was Eq. (2-1).

$$f_{pe} = \frac{\frac{M_{tot}\, y_b}{I_n}}{\frac{A_p}{A_n} + \frac{A_p\, e_n\, y_b}{I_n}} \qquad \textit{Eq. (3-1)}$$

Where:

A_n	*Gross area of the cross-section*
A_p	*Area of prestressing steel*
e	Eccentricity of the strand with respect of centroid of the cross-section
f_{pe}	Effective prestress
I_n	Gross moment of inertia of the section
M_{tot}	Total flexural moment at the load application point, including self-weight
y_b	Centroid of the cross-section measured from the bottom fiber

The second test involved applying a load to the beam until failure. However, the surface strain gauges were removed during this portion of the experimental program to avoid damaging them and because significant distress was expected. The load was recorded using a load cell, the slip on the strand was monitored at the ends using Linear Variable Differential Transformer (LVDT) sensors with an accuracy of 0.001 in. and the deflection at the point of load application was recorded using a string potentiometer with an accuracy of 0.01 in.

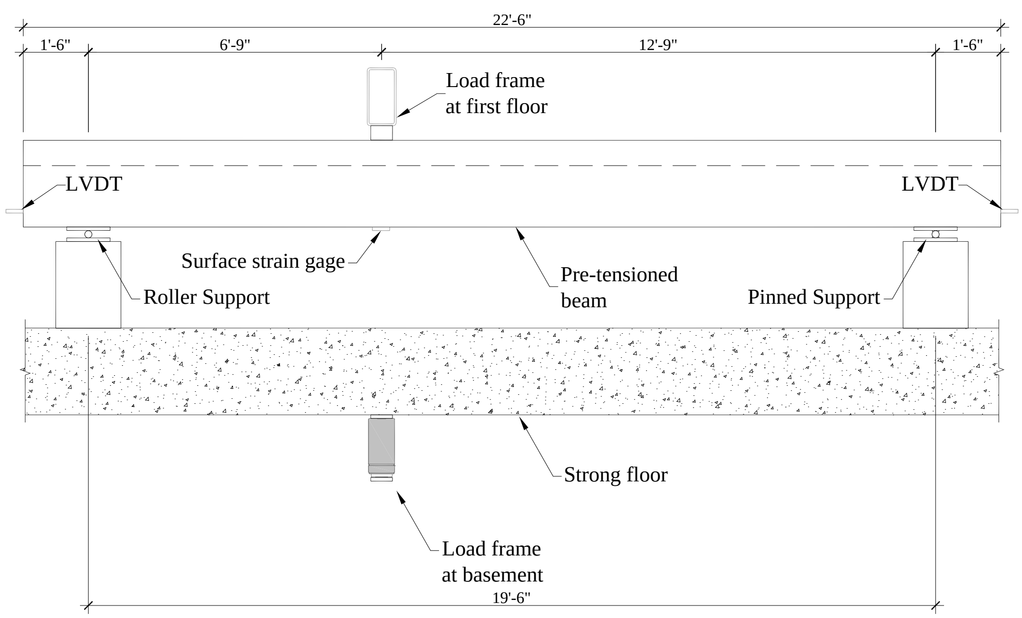


Figure 3-15 Test layout for load at 6’-9”

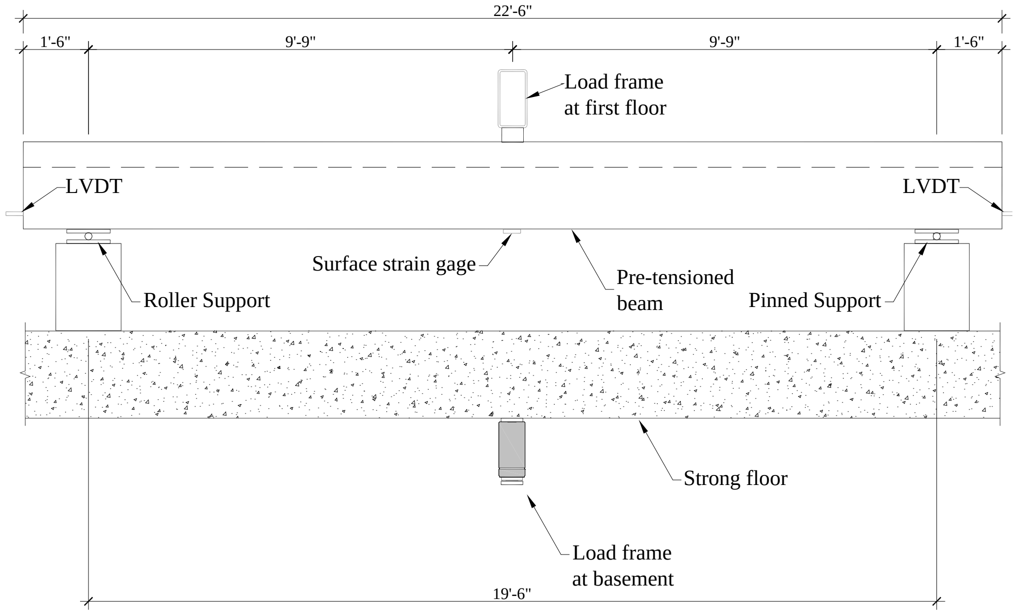


Figure 3-16 Cracking test for load at mid-span (9’-0”)

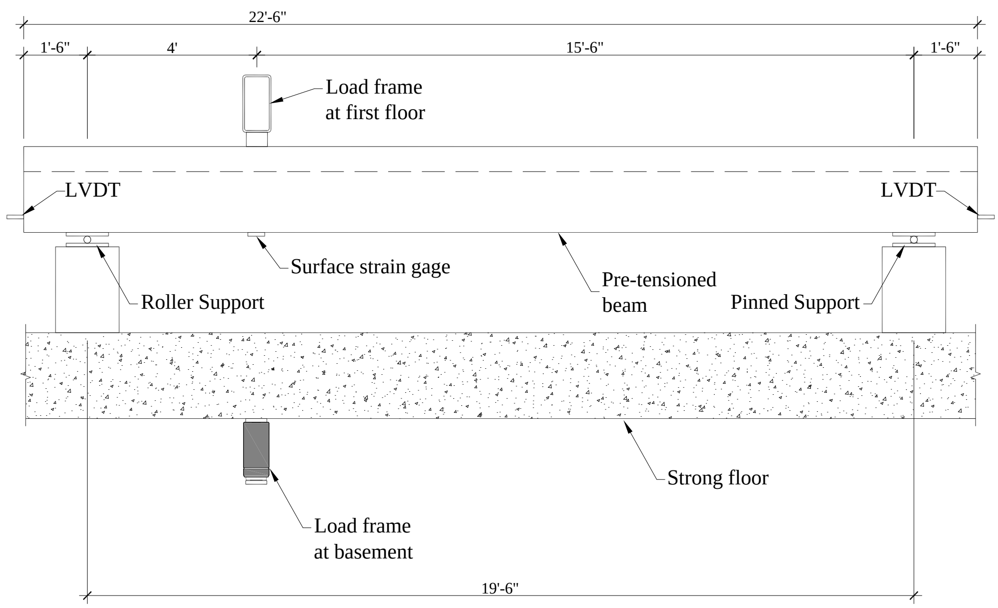


Figure 3-17 Test layout for load at 4'-0” from the support

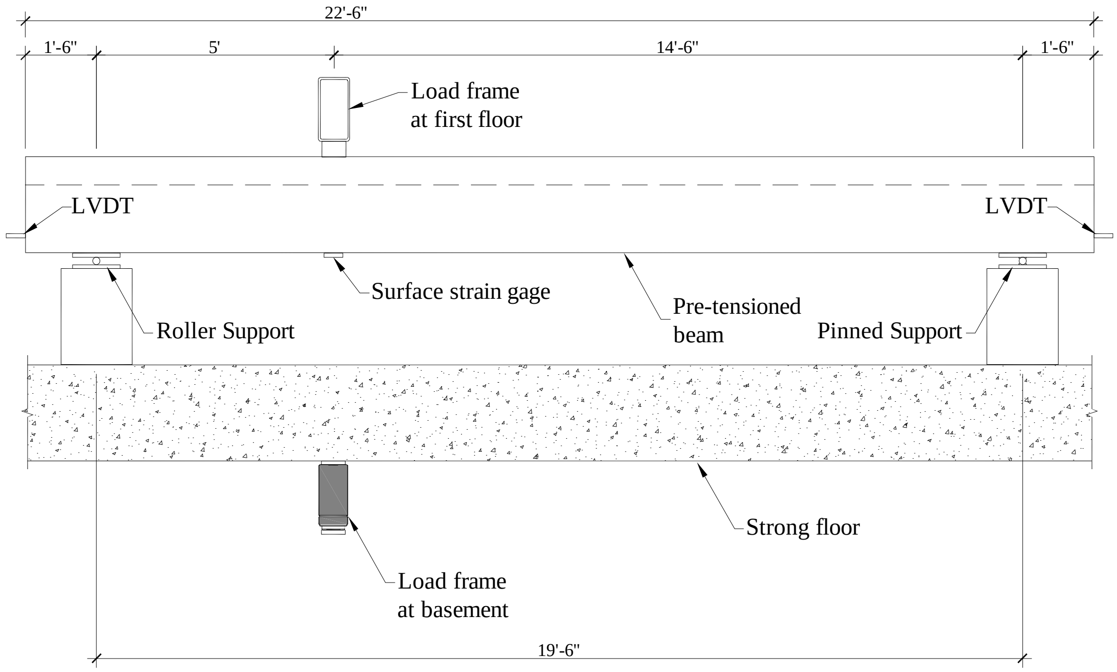


Figure 3-18 Test layout for load at 5'-0" from the support

3.6 Material Testing

The materials used in this test were sampled using ASTM standards. Concrete compressive strength, tensile stress capacity and modulus of elasticity were determined using the ASTM C39, C496, and C469 respectively (American Society for Testing and Materials, 2017; ASTM International, 2012, 2014). Mortar flow and compressive strength of cubes were sampled using ASTM C109 and ASTM C1437 (ASTM, 2014, 2016). Figure 3-19 shows a mortar cube and a concrete cylinder before being tested. The steel rebar modulus of elasticity, yield, and ultimate strength was determined from mill certificates as well as laboratory tests during the experimental program following the corresponding ASTM standard (ASTM, 2018; ASTM E8/E8M-16a, 2016). The rebar samples were tested for each group of bars that had a different heat number.

Moreover, strand samples were taken from the coil used to construct all specimens applying the testing procedure described in the standard test method for testing multi-wire prestressing strands and the JIS G3536:2014 (ASTM A1061 / A1061M-16, 2016; Kyōkai & Chōsakai, 2015).

(a) (b)

Figure 3-19 Compression testing of mortar cube (a) and concrete cylinder (b)

CHAPTER 4 Experimental Results

4.1 Material Testing Results

Concrete cubes and cylinder compressive strength test were performed for all grout and concrete specimens. For the large-scale test, the cylinders were prepared from the ready-mix truck at different intervals during the pour. The material testing results are presented herein according the section they belong to, instead of presenting them altogether. The main goal of this approach was to identify differences in the lapsed time between pour and test day, the type of test and significance of concrete strength and age.

4.1.1 Large Block Concrete Testing Results

Four large block pullout test (LBPT) specimens were poured, as part of the experimental program of this research, with the objective of benchmarking the pullout capacity of the strands with an 18 in. bond length and different concrete strengths. This test, as mentioned in the literature, does not replicate the typical precast pre-tensioned environment in which the strands are stressed to a certain jacking stress; therefore, the concrete strength at early age is not a significant parameter and was not considered in this test. Only compressive strength tests were performed for the concrete of this test, which are displayed in Table 4-1. Although the specimens were reinforced, testing them was not the goal of this test; thus, testing the rebar was not necessary.

Table 4-1 Concrete compressive strength for large blocks

Specimen	Age (days)	Average Compressive Strength (psi)	Target Strength
Control	36	7303	7000
Clean	36	7303	7000
Clean	123	11980	11000
Rusted	125	6115	7000

4.1.2 ASTM 1081 standard test material results

Mortar cubes were cast as part of the ASTM-1081 standard test procedure. This test also required the determination of the mortar flow following ASTM C1437 standard. The specimens were cured in a controlled environment for 24 hours +/-2 hours from casting the specimens, with temperature of 73°F +/-4°F and relative humidity above 90% all the time. The compressive strength of the mortar was evaluated using 2 in. x 2 in. cubes, as indicated in the ASTM C109 MPa (ASTM, 2016), with a target mean compressive strength ranging from 4,500 psi to 5,000 psi . The flow of the mortar used to fabricate the specimens was in the rage 100-125%, and it was measured according to the ASTM C1437 (ASTM, 2014). Table 4-2 shows the mortar flow readings during the pour of the specimens and the compressive strength of the cubes before and after finishing the test.

(a)

(b)

Figure 4-1 Determination of mortar flow (a), and compressive strength (b)

Table 4-2 Mortar flow and compressive strength results

	Mean	S.D.	COV
Average Flow (%)	111%	2%	2.12%
f'm start (psi)	4572	263	5.74%
f'm end (psi)	4847	356	7.34%

4.1.3 CRUP concrete compressive strength

Only one concrete strength was employed in the fabrication of the concentrically reinforced untensioned prisms. The concrete was delivered by a local ready-mix concrete supplier, which had a target compressive strength at 28 days of 8,000 psi. The actual compressive strength of the concrete was 8516 psi.

4.1.4 Concrete testing results of full-scale test

The large-scale testing program comprised three different concrete pours, two beams each pour. Concrete cylinder compressive test, modulus of elasticity and split tension test were performed for all concrete batches. Concrete cylinders were molded halfway through each of the pours following the ASTM C39 standard. Results for these variables are shown in Table 4-3.

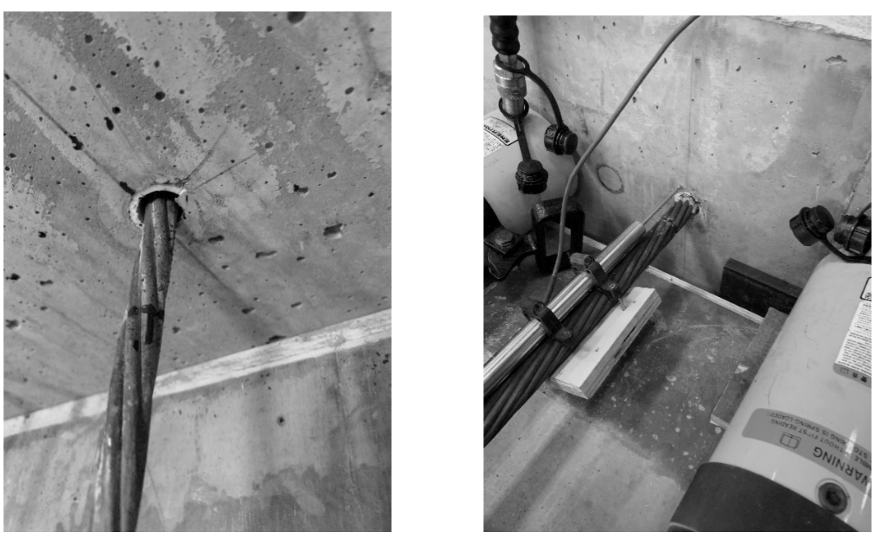
Table 4-3 Properties of concrete used for the large-scale specimens

Pour	**f'c at release (psi)**	**f'c at testing (psi)**	**Split Tension (psi)**	**Modulus of Elasticity (ksi)**
1	5111	6556	505	5656
2	7156	8458	497	6304
3	8203	9391	582	6445
4	8300	8551	491	5569

4.2 Bond benchmarking results

4.2.1 Big Block Test Results

Three different rounds of the LBPT were performed to evaluate the quality of bond of 1-1/8 in. strands to normal weight concrete. The first round comprised six 0.5 in. diameter strand samples embedded into a concrete block, used as the control sample, and six 1-1/8 in. diameter strands embedded into a different block. A black line was marked on the strands before conducting the test at the PVC pipe location to determine the pullout displacement. The strands from the control sample were pulled up to 4 in. displacement and subsequently ruptured either at the chuck or along the stressed length. Most 0.5 in. strands ruptured at the anchor because of the stress concentration at that location. The 1-1/8 in. diameter strands pulled from the concrete significantly less but did not break at any point because the bond length was not enough for the strand to develop the ultimate strength, see Figure 4-2. The average pullout force of the 0.5 in. strands was about 40 kips with a standard deviation of 0.5 kips, whereas the average force in the 1-1/8 in. was 88.9 kips with standard deviation of 3.9 kips. Figure 4-3 shows an example of testing results for 0.5 in. and 1-1/8 in. diameter strands.



(a) *(b)*

Figure 4-2 Control sample at the end of the test (a), and larger diameter strand (b)

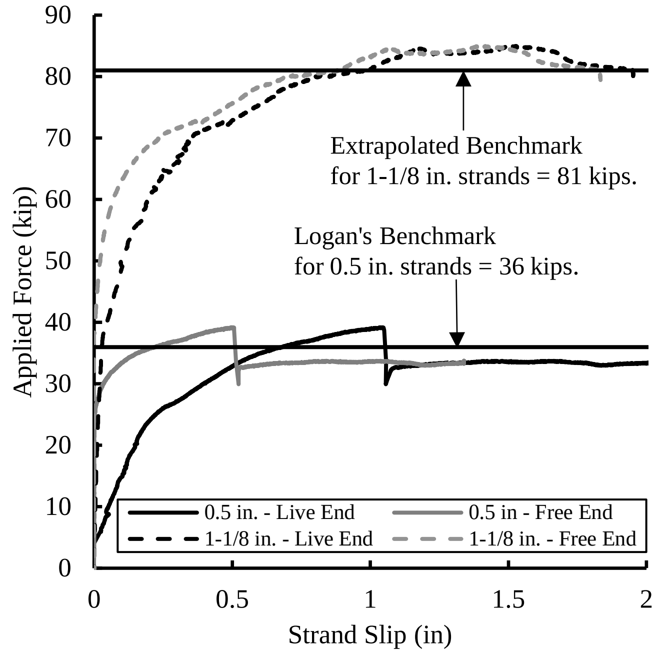
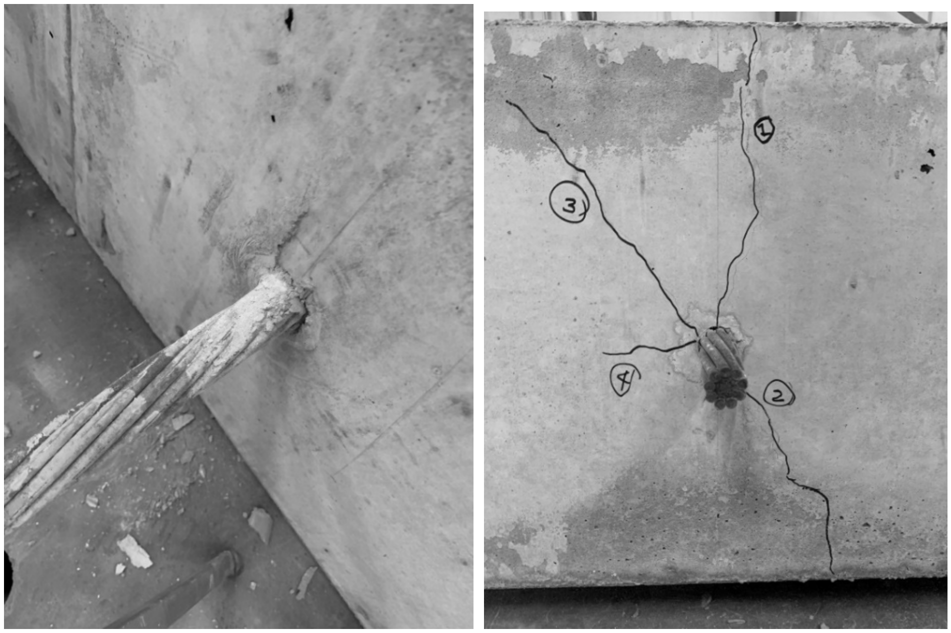


Figure 4-3 Example of LBPT results for 0.5 in. and 1-1/8 in. strands

The second round of testing consisted of performing the same test as the original, but in high-strength concrete. This group did not include 0.5 in. strands because these reach their rated capacity in lower concrete strengths. The average pullout force in this group was 108.4 kips with 4.6 kips standard deviation.

A final LBPT set was performed to evaluate whether rusting the strands would be beneficial to bond. The average pullout force was 110 kips in 6110 psi concrete, similar to the bright strands used in 11,980 psi concrete and higher than those in 7,303psi concrete. Along with the variability of pullout values, the concrete at the unstressed end exhibited cracking on the surface for the middle strands, whereas the live end showed more traveling than non-rusted strands, that is, approximately 0.5 inch with a lower applied load, see Figure 4-4.

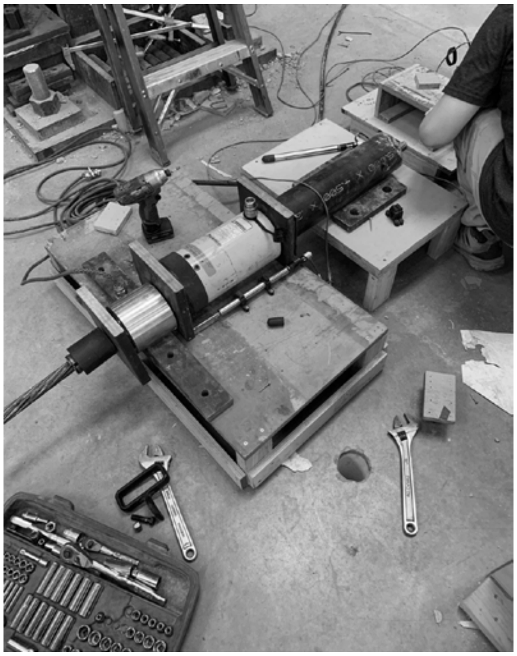


(a) *(b)*

Figure 4-4 Rusted strand after the test (a), noticeable cracking on dead end (b)

4.2.2 ASTM 1081 test results

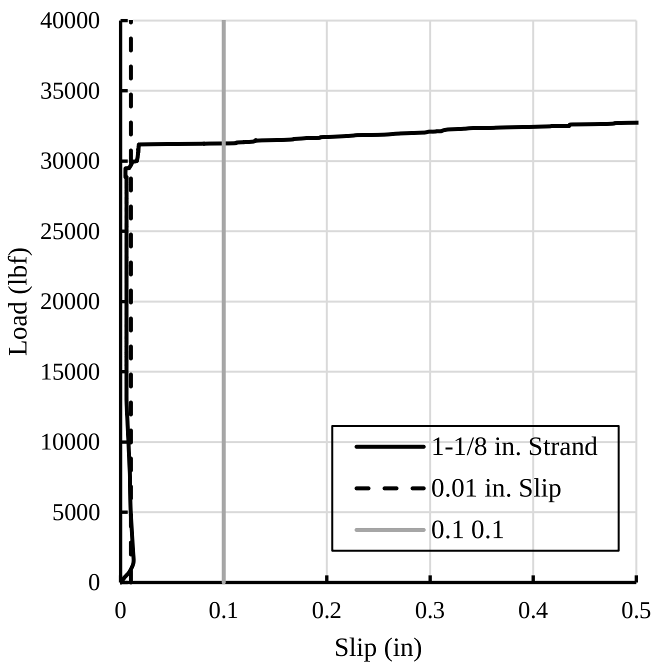
The objective of this test was to explore the possibility of applying the ASTM A1081 test to the large diameter strands. The specimens were mounted on a table to allow the specimen to rotate during the test, and the strand was pulled out at a displacement rate of 0.1 in./min. for the control specimen, and 0.2 in./min. for the large diameter strand specimens, see Figure 4-5a. The 0.5 in. strands had a pullout force of 12913 lbf with a standard deviation of 691 lbf for a 0.10 displacement at the free end, whereas the 1-1/8 in. strand had a pullout force of 32240 lbf with 2160 lbf standard deviation for a 0.1 in. displacement at the free end, see Figure 4-6. At the end of the testing, the large-diameter strands had pulled out from the inner strands in almost all cases, which was due to their 19-wire configuration. It is also noteworthy to mention that the 19-wire strand generates a substantially larger rotation because of their size and pullout capacity and this testing layout might not be appropriate for future use. Moreover, the strands must be cut at the end of the testing, which prolonged the duration of the test in comparison to the smaller strand size.



(a)

(b)

Figure 4-5 Steel casing prior to conducting the test (a), specimen at the end of testing (b)



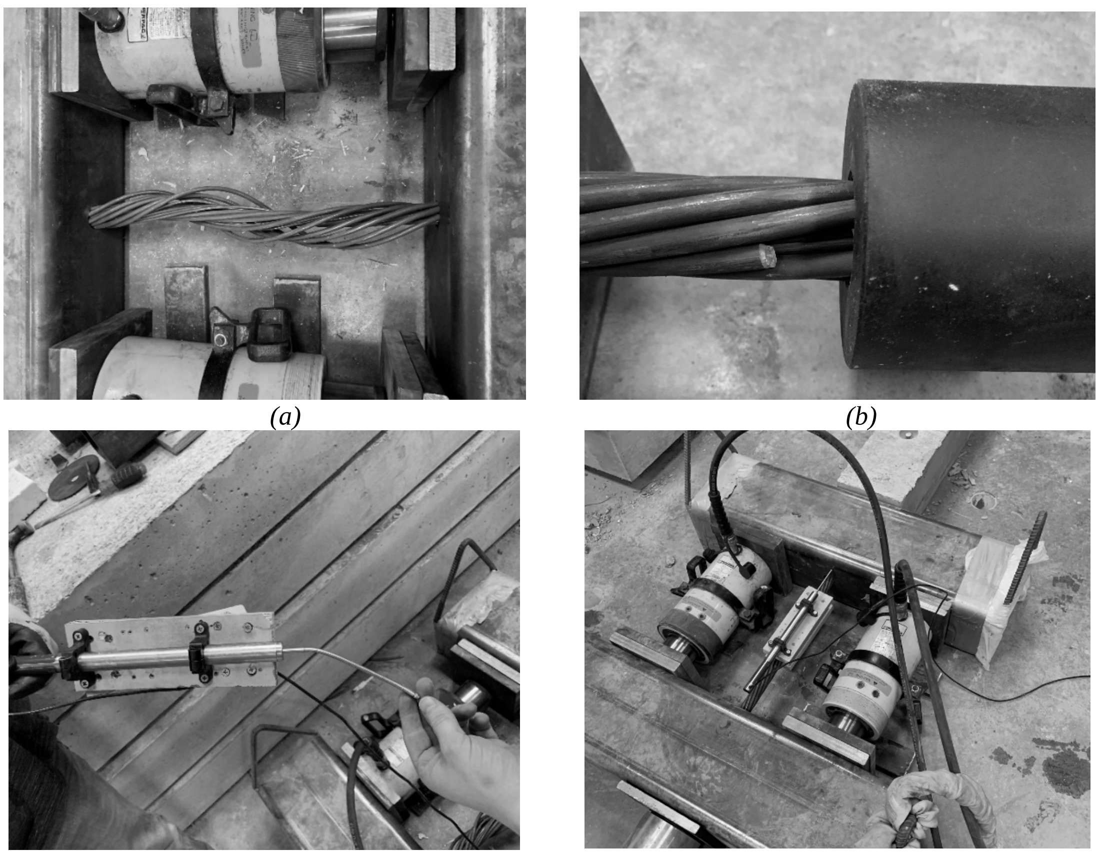
Strands	Force (lbf)	Average (lbf)	Std. Dev. (lbf)	COV (%)
0.5 in.	13133 12229 12657 14290 12290 12880	12913	691	5.35%
1-1/8 in.	31250 30460 30805 36710 33040 31170	32239	2159	6.70%

(a) (b)

Figure 4-6 Slip vs load curve for 1-1/8 in. diameter strand (a), summary of all test results for 0.10 in. slip (b)

4.2.3 Concentrically Reinforced Untensioned Prism (CRUP) test results

CRUPs are a practical and useful way to evaluate bond of prestressing strands to concrete developed at the University of Tennessee (Jiang et al. 2017). In this part of the experimental program, four different lengths were tested to assess the bond length at which the strands will develop their ultimate capacity. The strands were pulled out at a rate of 5-10 kips per step and the slip at the live (LE) and free end (FE) end was recorded using LVDT sensors. In several tests, the strands broke either at the chuck or abruptly within the body of the strands, as shown in Figure 4-7. The strands also slipped at the free end in all cases, but the slipping load was larger for the longest specimen, that is, the 8-ft. prisms.



(a) *(b)* *(c)* *(d)*

Figure 4-7 Abrupt failure of the strands (a), failure of strand at the chuck (b), and defective LVDT (c), and maximum ram travel during testing (d)

The ultimate strand force for the 1-1/8 in. strand is 206 kips assuming 250ksi strength, but the strand cannot reach this strength using the single-use chucks. The slipping load was about 155 kips for the 6-ft. specimens and 201 kips for the 8-ft. specimens, whereas the breaking loads were 194 kips and 202 kips for the 6 and 8-ft. specimens, respectively.

For the 4 ft prisms, results shown in Figure 4-8 the strand exhibited the characteristic free end slip characterized by Jiang et al. to be of less than the transfer length in Specimen 2, but close to the transfer length in Specimen 1. Furthermore, the 4 ft prisms exhibited a pullout force of only 167 kips and 145 kips, in Specimen 1 and Specimen 2, respectively. The average force is approximately 156 kips which is around the expected effective prestress which would be around 188 ksi (0.75*f_{pu}) or 155 kips. Each of the 4 ft long prisms finally failed where the strand split the concrete along its length as shown in Figure 4-9.

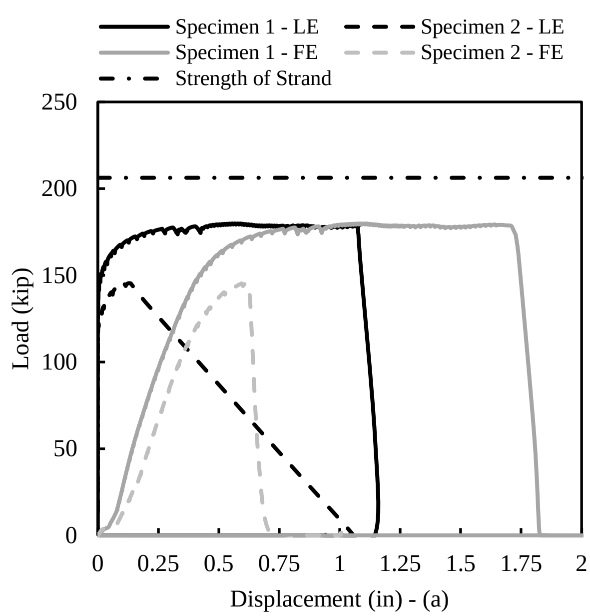
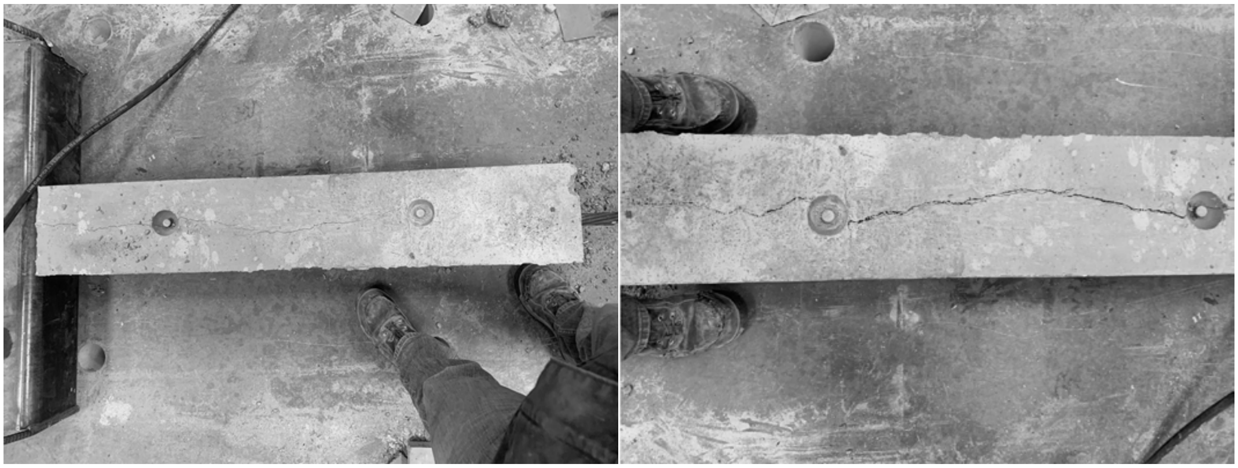


Figure 4-8 Pull out test results for 4 ft



(a) (b)

Figure 4-9. Splitting failure of 4 ft prisms (a) Specimen 1 (b) Specimen 2

Similar mixed results were found in the 5 ft long specimens shown in Figure 4-10. The breaking force of Specimen 1 was 206 kips (250 ksi) indicating that it was – or very near – fully developed at 5 ft. The maximum force for Specimen 2 was less at 186 kips (225ksi) and the free end slip behavior indicates that it was not fully developed for the 5 ft strand length. The reason for the mixed results on the 4 ft and 5 ft specimens is unclear but may indicate large artifacts associated with the CRUP testing as reported by Jiang et al. (2017).

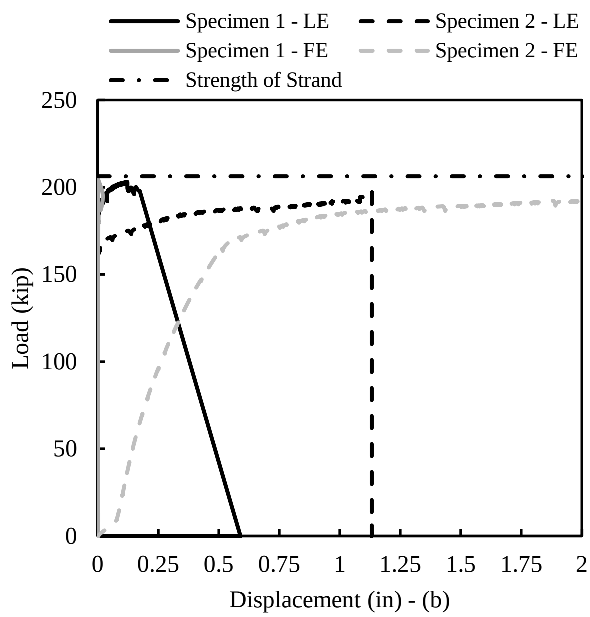


Figure 4-10 Pull out test results for 5-ft long specimens

(a) (b)

Figure 4-11 5 ft (a) Specimen 1 splitting (b) Specimen 2 No Splitting

For the 6 ft specimens, data presented in Figure 4-12, the strand nearly reached nominal breaking strength (206 kip) but significant slip was observed on both free ends, indicating development length is longer than 6 ft, which is consistent with the 8 ft results. No splitting was observed in the 6 ft specimens.

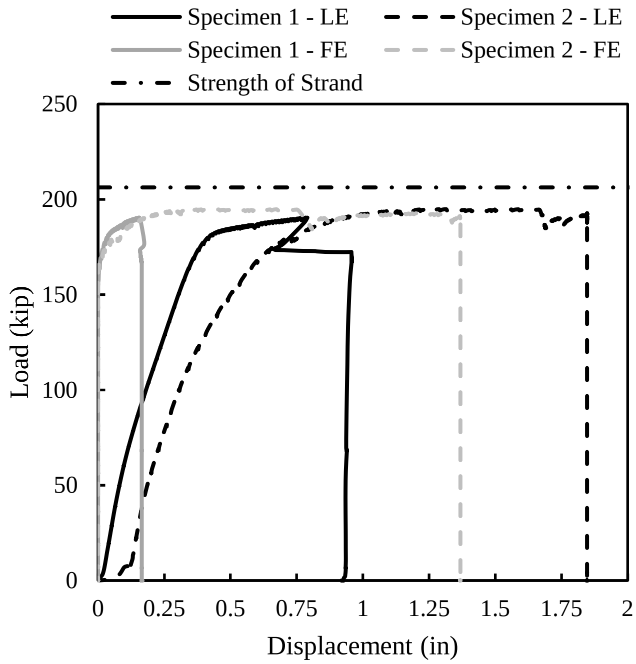


Figure 4-12 Pull out test results for 6-ft.

The 8 ft CRUP results indicate that the 1-1/8 in. strand is close to the development length at 8 ft of embedment length (see Figure 4-13) because the strand achieved very near nominal strand breaking force (and ruptured in the chuck) with minimal slipping for 8 ft Specimen 1 and no slipping in 8 ft Specimen 2. The 8 ft CRUPs did not exhibit splitting failures.

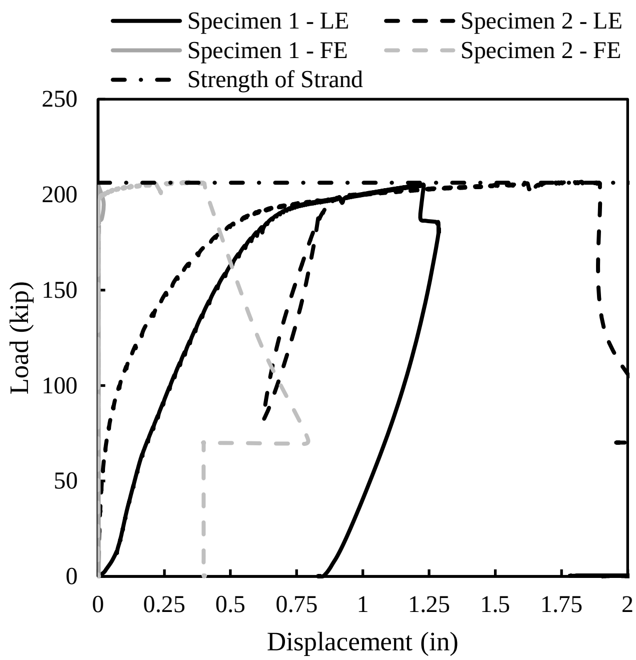


Figure 4-13 Pull out test results for 8-ft long specimens

The results from the 6ft and 8 ft CRUPs indicate that the development length should be approximately 96 in., perhaps shorter for the concrete strength of 8516 psi, which is later confirmed with the larger scale test results for development length. The results from the 4 ft and 5 ft CRUP indicate that the transfer length is likely near 4 ft. The CRUP gave conflicting results and may suffer from high scatter, and somewhat qualitative results, but seem to appropriately predict behavior (when compared to the results presented later in the paper) and if additional specimens were made, may perform well on average.

4.3 Full-scale Results

4.3.1 Pre-tensioned beams transfer length

Transfer length measurements were taken from all full-scale beams' caliper points using Demountable Mechanical (DEMEC) strain gauges at the live and dead ends. The live end was the end at which the strand was cut, whereas the dead end was the location at where the strand was tensioned and anchored. DEMEC readings were taken prior to the release of the force as reference readings and the strand was subsequently cut to have the release transfer length. The release of the force caused end zone cracking in most specimens, though it was arrested by the splitting reinforcement. Minor spalling at the interface concrete-strand of the live end only occurred in specimen 3, as shown in Figure 4-14a. These spalls were caused by stirrup movement/misplacement during fabrication and the close proximity (12 in.) of the flame cut to the beam live end and subsequent uncoiling of the strand.

(a) *(b)*

Figure 4-14 Live end surface cracking on beam 1 (a), and beam 3 (b)

DEMEC readings were plotted and analyzed using the 95% of the average maximum strains method, described in (Bruce W Russell & Burns, 1997). This method consists of identifying the location where the strain profile starts flattening or becomes nearly constant. The values beyond that location are known as the “maximums”; thus, the 95% of their average line is plotted to determine the point at which the ascending branch of the plot intersects it. The distance between that point and the end of the specimen is the transfer length, see Figure 4-15.

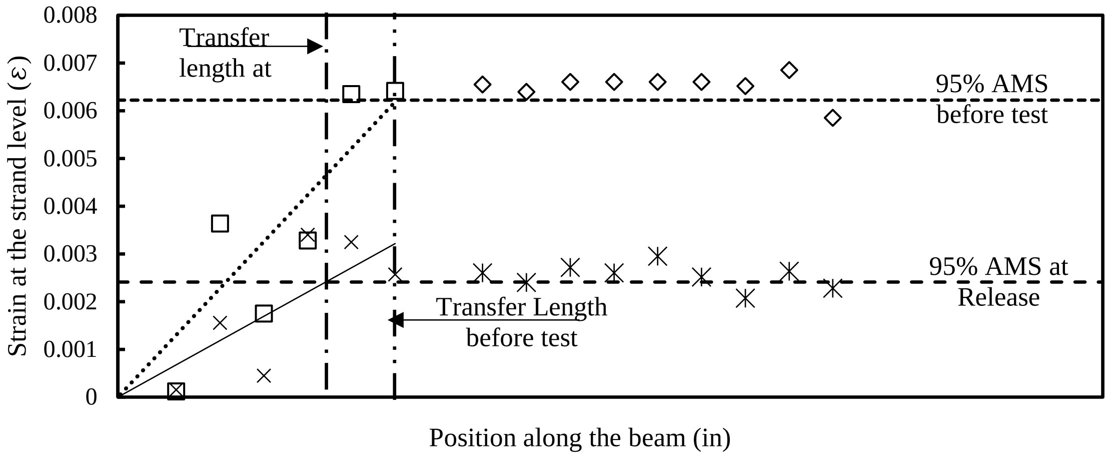


Figure 4-15 Determination of transfer length using the 95% AMS method

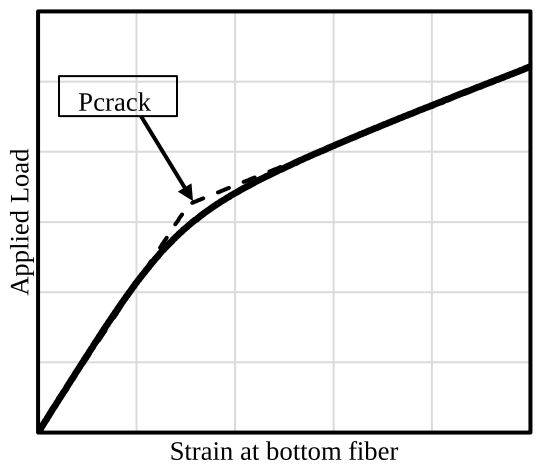
The procedure mentioned above was performed for all available data, except for locations where the gauge points fell off and data could not be retrieved. As Figure 4-15, the ascending branch of the DEMEC measurements was plotted using a linear regression forced through zero. Data was only recorded before release, after release, and before the testing of the specimens, which are displayed in Table 4-4.

Table 4-4 Transfer length results summary

			1	2	3	4	5	6	7	8
At release	Dead End	Lt 1 (in.)	45	45	33	27	21	25	19	22
	Dead End	Lt 2 (in.)	-	-	31	29	21	24	18	21
	Average (in)		45	45	32	28	21	24.5	18.5	21.5
	Live End	Lt 1 (in.)	58	60	30	37	30	30	24	27
	Live End	Lt 2 (in.)	-	-	30	29	27	30	22	26
	Average (in)		58	60	30	33	28.5	30	23	26.5
	f'c (psi)		5111	5111	7156	7156	8203	8203	8300	8300
At testing	Dead End	Lt 1 (in.)	-	-		32	24	29	24	30
	Dead End	Lt 2 (in.)	-	-	-	36	27	27	24	29
	Average (in)		-	-		34	25.5	28	24	30
	Live End	Lt 1 (in.)	-	-	44	40	33	33	24	32
	Live End	Lt 2 (in.)	-	-	-	37	27	34	23	31
	Average (in)		-	-	44	38.5	30	33.5	24	32
	f'c (psi)		6556	6556	8458	8458	9391	9391	8551	8551

4.3.2 *Pre-tensioned beams effective prestress*

The experimental determination of the effective prestress was done by loading the beams at the location of interest until the first crack formed at the bottom fiber. The beams were subsequently unloaded, and the strain gauges were mounted at the bottom fiber across the observed first crack. After that, the beams were loaded and unloaded three times to build the strain vs. load profile and determine the cracking load, see Figure 4-16a. The results of this test are summarized in Figure 4-16b.



Beam #	Testing location from support (ft)	P (kip)	Mp (kip-ft)	Msw (kip-ft)	fpe (ksi)
1	9.75	15.58	911	133	137
2	6.75	16.31	864	120	129
3	9.75	16.63	973	133	145
4	6.75	19.03	1008	120	148
5	4	27.61	1053	87	150
6	6.75	19.38	1026	120	151
7	5	24.99	1115	102	160
8	4	29.5	1126	87	159

(a) (b)

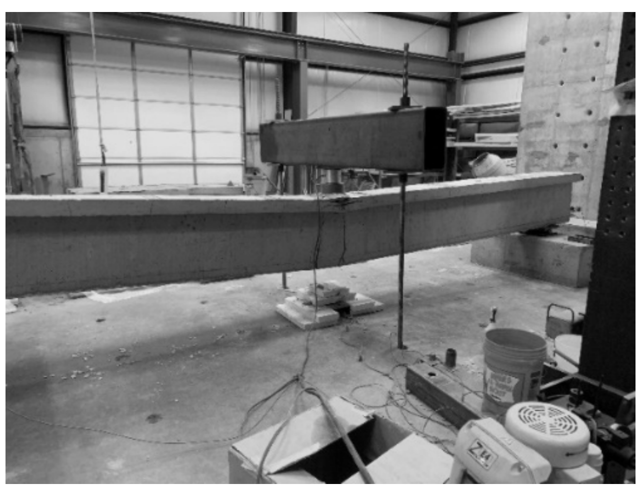
Figure 4-16 Surface strain gauges placed at the bottom fiber (a), effective prestress on the strand (fpe) and calculations (b)

4.3.3 *Flexural performance*

The development length of the strands was experimentally determined from the flexural performance of the beams that comprised this study. Embedment length was varied from 5.5ft to 9.5ft from the beam end. The location of load application was changed to apply maximum loading to the strand closer to the support for different concrete strengths and expected development lengths. Load, deflection, and strand slip were monitored during all tests in order to accomplish the task mentioned above.

The first set of beams was comprised of two beams with different load arrangement, one with a point load 6.75-ft. from the end and another one with a point load at midspan. The concrete compressive strength (f'c) was 6.5 ksi and the beams had only mild steel in the flange. Figure 4-17 shows Beam 1 and Beam 2 after failure. Note concrete crushing of top flange at failure. In both cases, the strand slipped at failure in excess of the 0.01 in. slip limit. Both Beam 1 and Beam 2 did not reach the expected nominal capacity, see Figure 4-18a through d.

(a)



(b)

Figure 4-17 Failed beam 1 (a), Failed beam 2 (b)

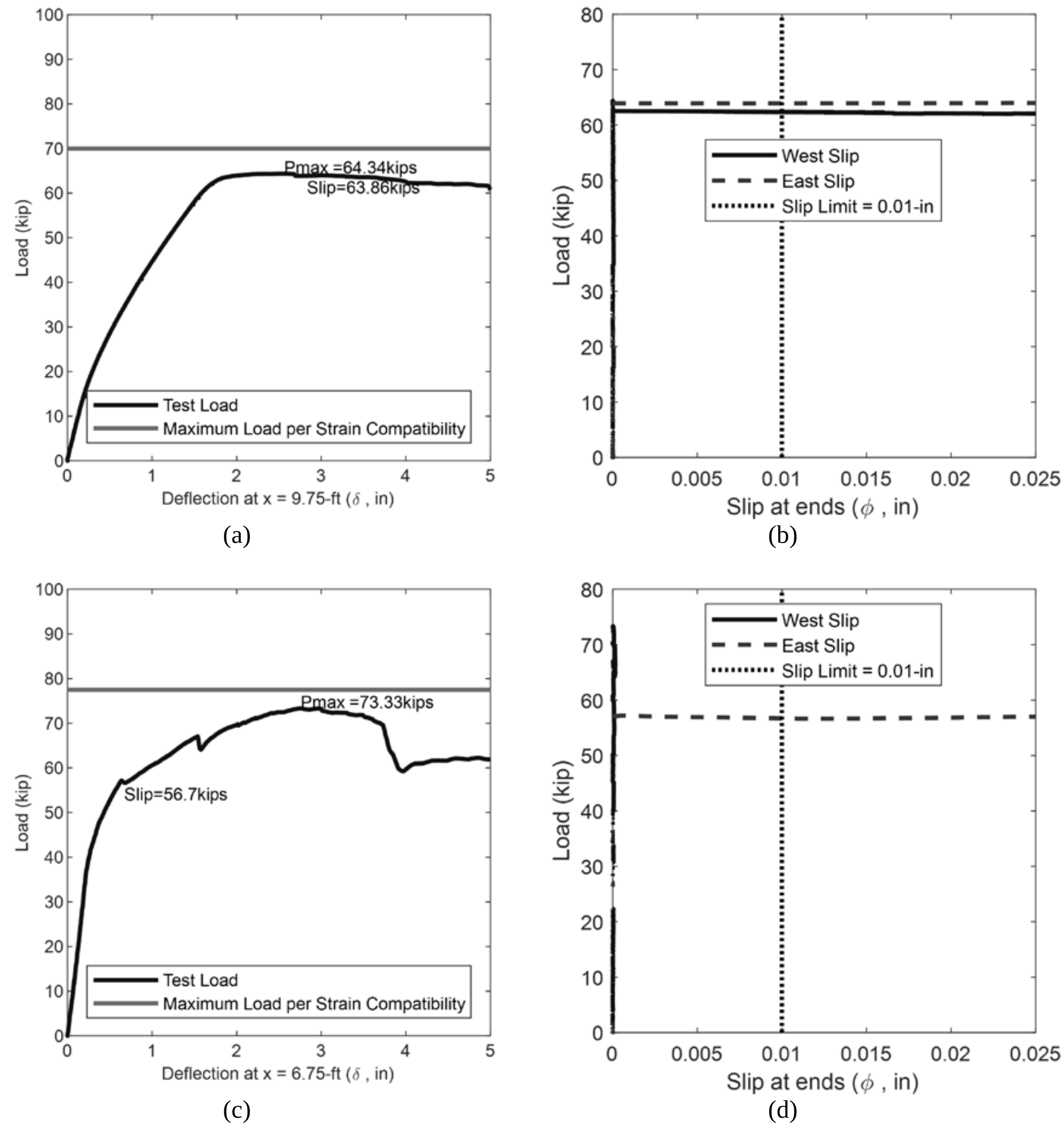


Figure 4-18 Load versus deflection curve for beam 1(a), load versus slip curve for beam 1 (b), Load versus deflection curve for beam 2 (c), and load versus slip curve for beam 2 (d)

The second set of beams was tested at concrete strength of 8.2 ksi, and mild reinforcement in the top flange, middle and bottom of the web. Beam 3 and Beam 4were test at the same locations that Beam 1 and Beam 2, respectively and some slip was measured. However, the measured end slip was less than the 0.01in. limit as presented in Figure 4-19 indicating that in both instances the strand was developed at the tested embedment lengths (10.75 ft and 7.75 ft) for up to 250 ksi per the sectional analysis. Both beams failed in flexure and reached or exceeded their nominal moment capacity.

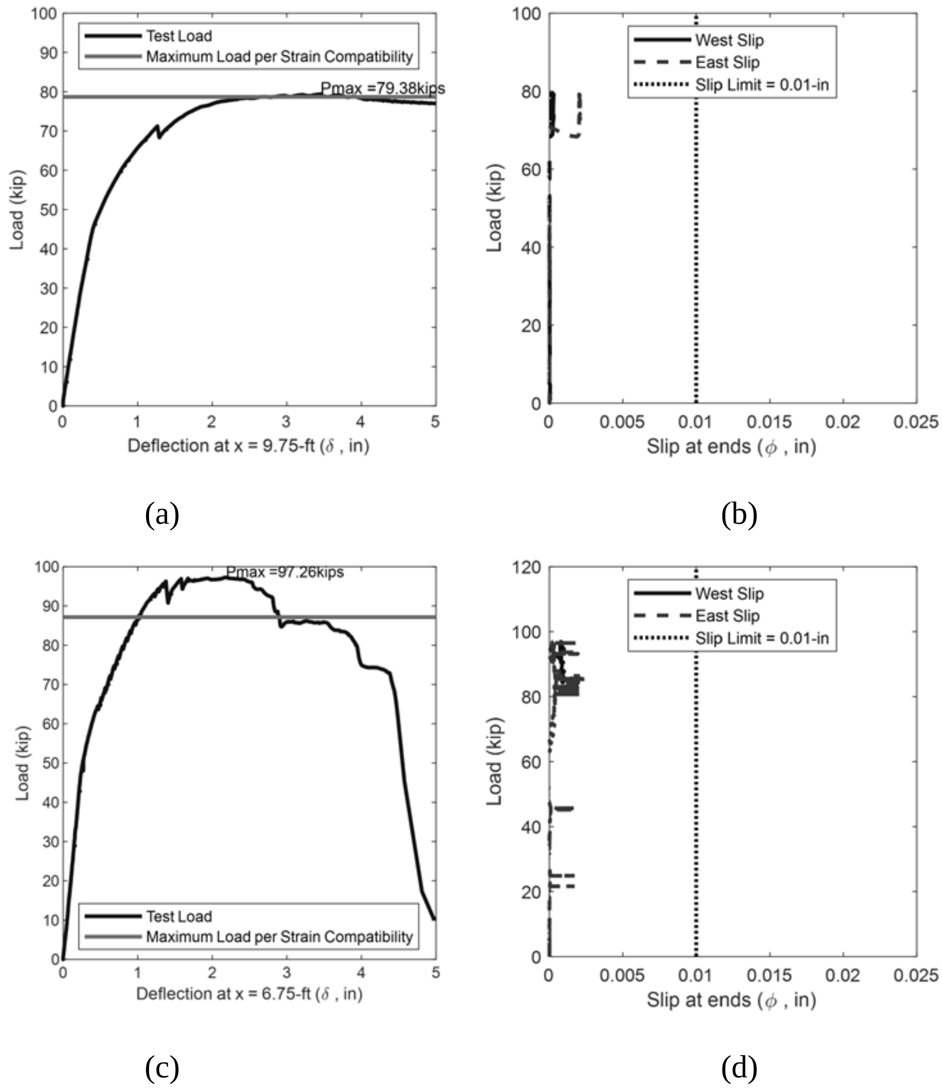


Figure 4-19 Load versus deflection curve for beam 3(a), load versus slip curve for beam 3 (b), Load versus deflection curve for beam 4 (c), and load versus slip curve for beam 4 (d)

The third set of beams consisted of two beams with concrete strength of 9.39 ksi, and mild reinforcement in the top flange, middle and bottom of the web. These beams were tested at 4 ft and 6.75 ft from the support, but only the former exhibited strand slip, see Figure 4-20. The beam tested at x = 4 ft yielded slip failure at 100.11 kips, whereas the beam tested at 6.75 ft. from the support failed in flexure.

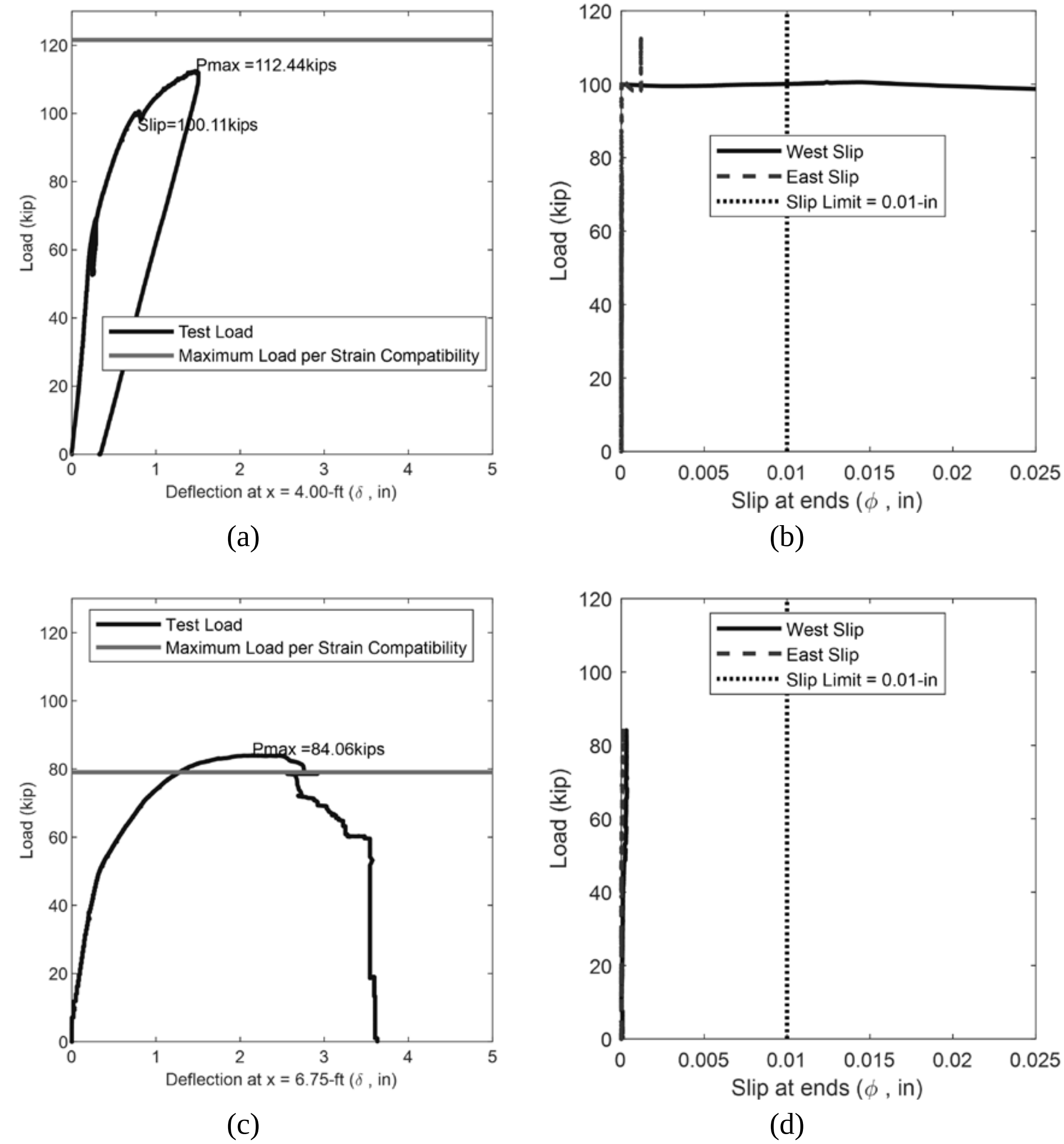


Figure 4-20 Load versus deflection curve for beam 5(a), load versus slip curve for beam 5 (b), Load versus deflection curve for beam 6 (c), and load versus slip curve for beam 6 (d)

The last set of beams consisted of two beams with concrete strength of 8.55 ksi, and mild reinforcement in the top flange, middle and bottom of the web. These beams were test at 4 and 5 ft. from the support, but only one of them exhibited a strand slip, see Figure 4-21. The beam tested at x = 4 ft yielded a peak force of 119.33 kip while exceeding the slip limit at 117 kip resulting in a hybrid failure, whereas the beam tested at 5-ft. from the support failed in flexure at 102 kips very close to the predicted load without observable slip.

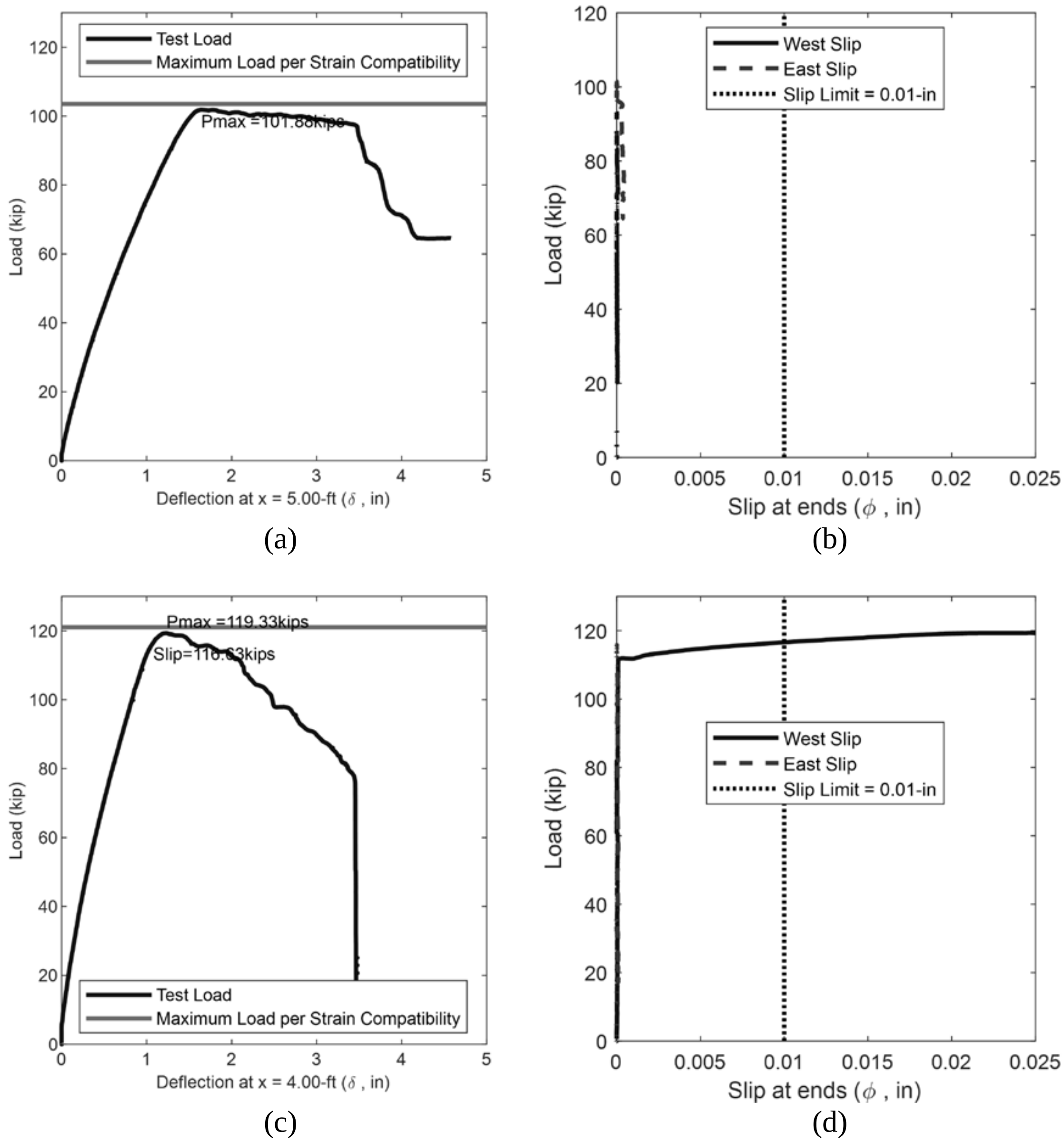


Figure 4-21 Load versus deflection curve for beam 7(a), load versus slip curve for beam 7 (b), Load versus deflection curve for beam 8 (c), and load versus slip curve for beam 8 (d)

A summary of all full-scale testing is presented in Table 4-5. The first three columns represent the beam number designation, the end of the beam tested, and the location of the point load applied using the hydraulic ram, measured from the center of load to the center of the support. Subsequent columns represent the load at which the strand slip exceeds the 0.01-in., which is a commonly accepted threshold for strand slip (Ramirez & Russell, 2008). Peak loads are also presented along with their respective flexural moments at failure.

Table 4-5 Full-scale loading results

Beam Number	End	Point Load Location[1] (ft)	Load at 0.01-in Slip (kip)	Peak Test Load (kip)	Moment at 0.01-in Slip (kip-ft)	Peak Test Moment (kip-ft)	Self-weight Moment (kip-ft)	Total Moment at 0.01-in Slip (kip-ft)	Total Moment at Failure (kip-ft)
1	NA	9.75	64.34	64.86	313.66	316.19	11.09	324.75	327.28
2	Live	6.75	56.7	73.33	250.24	323.64	10.04	260.28	333.68
3	NA	9.75	--	79.38	--	386.98	11.09	--	398.07
4	Live	6.75	--	97.26	--	429.25	10.04	--	439.29
5	Live	4	100.11	112.44	318.30	357.50	7.23	325.53	364.73
6	Live	6.75	--	84.06	--	371.00	10.04	--	381.04
7	Live	5	--	101.88	--	378.78	8.46	--	387.24
8	Live	4	116.63	119.33	370.82	379.41	7.23	378.06	386.64

4.4 Data Analysis

4.4.1 Bond performance

The bond of prestressing strands was evaluated using the three different methods, namely the ASTM A1081-15, the LBPT, and the CRUP. The number of specimens, dimensions, average concrete properties and pullout forces are shown in Table 4-6. As this table summarizes, the different bond benchmarking methodologies can be used to estimate the bond performance of the big strands to normal-weight concrete.

In the literature, recommended minimums for bond tests are not always linked to structural performance (like transfer length) but arrive at reasonable minimums through copious testing and experience. Furthermore, the nature of the 1-1/8 in. strands, considering the larger

[1] Measured from support.

diameter and higher strand count makes comparing to minimum acceptable bond strengths from LBPT and ASTM A1081-15 difficult. Ramirez and Russell (2008) recommend a minimum average NASP bond test value of 10.5 kips for 0.5 in. diameter strands as a minimum NASP value. All tested 0.5 in. and 1-1/8 in. strands met this criterion. When testing 0.5 in. and 0.5 in. super diameter strands Carroll et al. (2017) indicated a ratio of areas may provide an adequate benchmark, though some strands failed this benchmark yet performed adequately in transfer length, development and flexural capacity (Loflin, 2008). This ratio indicates the minimum pullout for 1-1/8 in. diameter strand is 56.6 kips (10.5 kip * 0.825 in^2 / 0.153 $in.^2$), but the strands failed this benchmark (compare to 32,238 lbs in Table 4-6). Perhaps a better conversion might be the ratio of the generalized circumference (i.e., ratio of diameters) at 23.6 kips (10.5 kip * 1.125 in. / 0.5 in.), which 1-1/8 in. diameter strand does pass.

For the LBPT, benchmark values for 0.5 in. diameter strands are similarly determined. Logan (1997) recommended a minimum pullout capacity of 36 kip for 0.5 in. diameter strands for concrete strengths between 3,500 psi and 5,900 psi, which were thought to be lower strengths than needed for the 1-1/8 in. strands. In all cases concrete strengths were higher for the LBPT and pull out strength exceeded this for 0.5 in. strand. If this strength is multiplied by the ratio of strand diameters, 81 kips (36 kip *1.125 in. / 0.5 in.), the 1-1/8 in. diameter strands passed.

The CRUP testing indicated that the transfer length in 8516 psi concrete will likely be around 48 inches based on the results from the 4 ft and 5 ft prisms and the development length will be less than 8 ft based on the 6 and 8 ft prisms.

Table 4-6 Summary of bond testing results

Test	Number of Specimens	Transverse Dimensions (in)	Diameter of Strand (in)	Length (in)	f'c or f'm (psi)	Load Rate	Average pullout force (lbf)	Std. Dev. (lbf)	COV (%)
ASTM A1081-15	6	5 (O.D.)	1/2	18	4572-4846†	0.1 in./min	12913	691	5.45%
ASTM A1081-15	6	5 (O.D.)	1 1/8	18	4572-4846†	0.2 in./min	32238	2160	6.70%
LBPT	6	18 x 24	1/2	55	7303	0.35 in./min	39987	558	1.39%
LBPT	6	18 x 24	1 1/8	116	7303	0.35 in./min‡	88913	3887	4.37%
LBPT	6	18 x 24	1 1/8	116	11980	40 kip/min	108395	4589	4.23%
LBPT	6 - Rusted	18 x 24	1 1/8	116	6116	40 kip/min	109988	3964	3.60%
CRUP	2	8 x 8	1 1/8	48	8516	5 to 10 kips/step	162,670	--	--
CRUP	2	8 x 8	1 1/8	60	8516	5 to 10 kips/step	200,060	--	--
CRUP	2	8 x 8	1 1/8	72	8516	5 to 10 kips/step	192,630	--	--
CRUP	2	8 x 8	1 1/8	96	8516	5 to 10 kips/step	206,040	--	--

4.4.2 Transfer length and development length results

The average transfer lengths were 59, 32, 29, and 25 in. for concrete strengths of 5.10 ksi, 7.16 ksi, 8.20 ksi, and 8.30 ksi at release, respectively. Referenced to strand diameters, this is 52, 28, 26, and 21 strand diameters, respectively, which compare favorably to the AASHTO LRFD prediction of 60 strand diameters. Prior to testing, transfer lengths were measured again, assumed to be final at 41 in., 32 in., 28 in., when the concrete had reached 8.46 ksi, 9.39 ksi, and 8.55ksi, respectively. Referenced to strand diameters, this is 37 in., 28 in., and 24.9 strand diameters. Transfer lengths increased an average of 17.5% prior to testing.

As expected, there is a general negative correlation of concrete strength and transfer length. Furthermore, the four sets of concrete beams tested in flexure exhibited good performance relative to the literature as would be expected from a 7-wire strand with no issues identified that indicate problems with the behavior related to having 19-wires.

As Table 4-7 shows, 1-1/8 in. strands perform well in the higher strength concretes and can provide reasonable development lengths. For lower strength concretes of Beam 1 and Beam 2 (6556 psi) beams resulted in clear bond failures indicating development lengths are larger than the tested embedment lengths of 99 in. and 135 in. Beams 2, 3, 7, and 8 all had approximately the same strength concrete (8458psi and 8551psi). Embedment lengths of 78 in. (Beam 7), 99 in. (Beam 4), 135 in. (Beam 3) tests resulted in flexural failures indicating development length was less than 78 in. Beam 8 resulted in a bond type failure indicating the development length for this situation is between 66 in. and 78 in. For the higher strength concrete of Beams 5 and 6 (9391 psi), flexural failure at 99 in. and bond failure at 66in. indicates a development length between 66 in. and 99 in., similar to observed in the Beams 2, 3, 7, and 8. It is worth noting that the strand transfer length was protected from cracking by imposing the 18 in. overhang. AASHTO development lengths (presented in Table 4-7 as 161 in. to 184 in. assuming 250 ksi and the measured transfer length) seem to overpredict development lengths overall. These results are promising for the use and investigation of 1-1/8 in. strands in the future.

Table 4-7 Summary of flexural test and development length results

Beam #	**Live Lt. (in.)**	**f'c (psi)**	**fpe (ksi)**	**Failure Mode (in.)**	**Total Embedment Length**	**Ld. AASHTO (in.)**	**Estimated Development Length**	**Mtest/ Mpred**
1	58	6,556	137.32	Bond	135	178	> 135	93%
2	60	6,556	129.40	Bond	99	184	> 99	95%
3	44	8,458	145.40	Flexural	135	172	< 135	103%
4	39	8,458	148.34	Flexural	99	169	< 99	114%
5	30	9,391	149.90	Bond	66	168	> 66	94%
6	34	9,391	150.78	Flexural	99	168	< 99	99%
7	28	8,551	159.92	Flexural	78	161	< 78	101%
8	30	8,551	159.38	Bond	66	161	> 66	100%

CHAPTER 5 Conclusions

5.1 Summary

In this report, a literature review of bond, transfer and development length of prestressing strands was presented, and current code equations were discussed. Research was also conducted to investigate bond, transfer length, development length, and flexural behavior of fully bonded pretensioned members fabricated with normal-weight concrete and 1-1/8-in. strands. Current design specifications regarding these issues were explored, which yielded satisfactory results for design purposes if the results are taken in the proper context. Three different methodologies were employed to evaluate bond of 1-1/8-in. prestressing strands. These methods included the current ASTM A-1081 standard test, the Large Block Pullout Test (LBPT), and the Concentrically Reinforced Untensioned Prisms (CRUP) pullout test. Eight full-scale beams were fabricated at the Utah State University SMASH Lab, and transfer and development lengths were experimentally obtained from them. These beams were stored in the lab for several weeks after fabrication to monitor the increase in transfer length. All beams were tested after this period to determine the development length, effective prestress, prestress losses, and flexural performance data.

5.2 Bond performance

A total of 18 pullout specimens were fabricated to evaluate the bond performance of untensioned prestressing strands to concrete. There were no noticeable issues with the bond of the 19-wire strands as opposed to the 7-wire strands.

- While there are no bond criteria established for 19-wire 1-1/8 in. diameter strands, the large diameter strands performed acceptably in NASP and LBPT.

- For the NASP testing a generally accepted value for 0.5 in. strand is 10.5 kips and when multiplied by the ratio of diameters to the strands tested results in 23.6 kips, which is lower than the observed average pullout of 32.2 kips.
- Modification to the NASP test for 19-wire strands may be needed as there seemed to be considerable differential displacement between the outer and inner strands. More investigation is warranted.
- The pull-out strength from LBPTs exceeded the recommended limit of 36 kips and a limit multiplied by the ratio of the diameters of 81 kips. In all cases concrete strengths were higher for the LBPT than recommended for the LBPT because low strengths were not expected to perform well with the 1-1/8 in. strands.
- The CRUP tests indicated that transfer length would likely be approximately 48 inches for 8500psi concrete.
- The CRUP tests indicated that development length would likely be between 6 ft and 8 ft for 8500psi concrete.
- CRUP testing generally agreed with the large-scale measurements.

5.3 Transfer length

A total of eight large-scale beams were fabricated using 1-1/8 in. diameter prestressing strand. The following conclusions can be made regarding transfer lengths of 1-1/8 in. diameter prestressing strand in this study:

- All transfer lengths resulted in values below the AASHTO LRFD recommendation of 60 db. In certain circumstances, the AASHTO LRFD equations overestimated this value by 100% or more. These results indicate that 1-1/8 in. diameter strands may perform acceptably in prementioned members.

- A negative correlation of concrete strength and transfer length was observed.

5.4 Development length

A total of eight large-scale beams were fabricated using 1-1/8 in. diameter prestressing strand. AASHTO LRFD predicted development lengths ranged between 161 in. and 184 in., which were generally longer than those observed. The following conclusions can be made regarding development lengths of 1-1/8 in. diameter prestressing strand in this study:

- Large scale embedment length testing of 1-1/8 in. diameter strand in 6500 psi class concrete indicated development length is greater than 135 in.
- Large scale embedment length testing of 1-1/8 in. diameter strand in 8500 psi class concrete indicated development length is between 66 in. and 78 in.
- Large scale embedment length testing of 1-1/8 in. diameter strand in 9500 psi class concrete indicated development length is between 66 in. and 99 in.

5.5 Future Research

The results of the testing in this report are encouraging for the use of 1-1/8 in. diameter strands in pretension applications. Future research should investigate multiple strand concrete reinforcing strategies, specifically spacing. Additional studies are warranted on transfer and development length in different concrete strengths, especially considering the difficulty the ready-mix concrete in this report had in meeting strengths. Precasting at a plant will likely be difficult due to limitations on prestressing abutment capabilities, but if sufficient studies are performed that illustrate a benefit to using large diameter strands, this would be a necessary step. If successful in the above situations harping and debonding of strands will also be of interest.

This is the first known successful installation of 19-wire strands in pretensioned applications in the United States. As such, future research may be able to identify new phenomena associated with its bond and performance brought to light by 19-wire strand behavior that shed light on the bond performance of popular 7-wire strands and 3-wire strands.

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APPENDIX A

SUPLEMENTAL FIGURES

A.1. ASTM A1081 Results

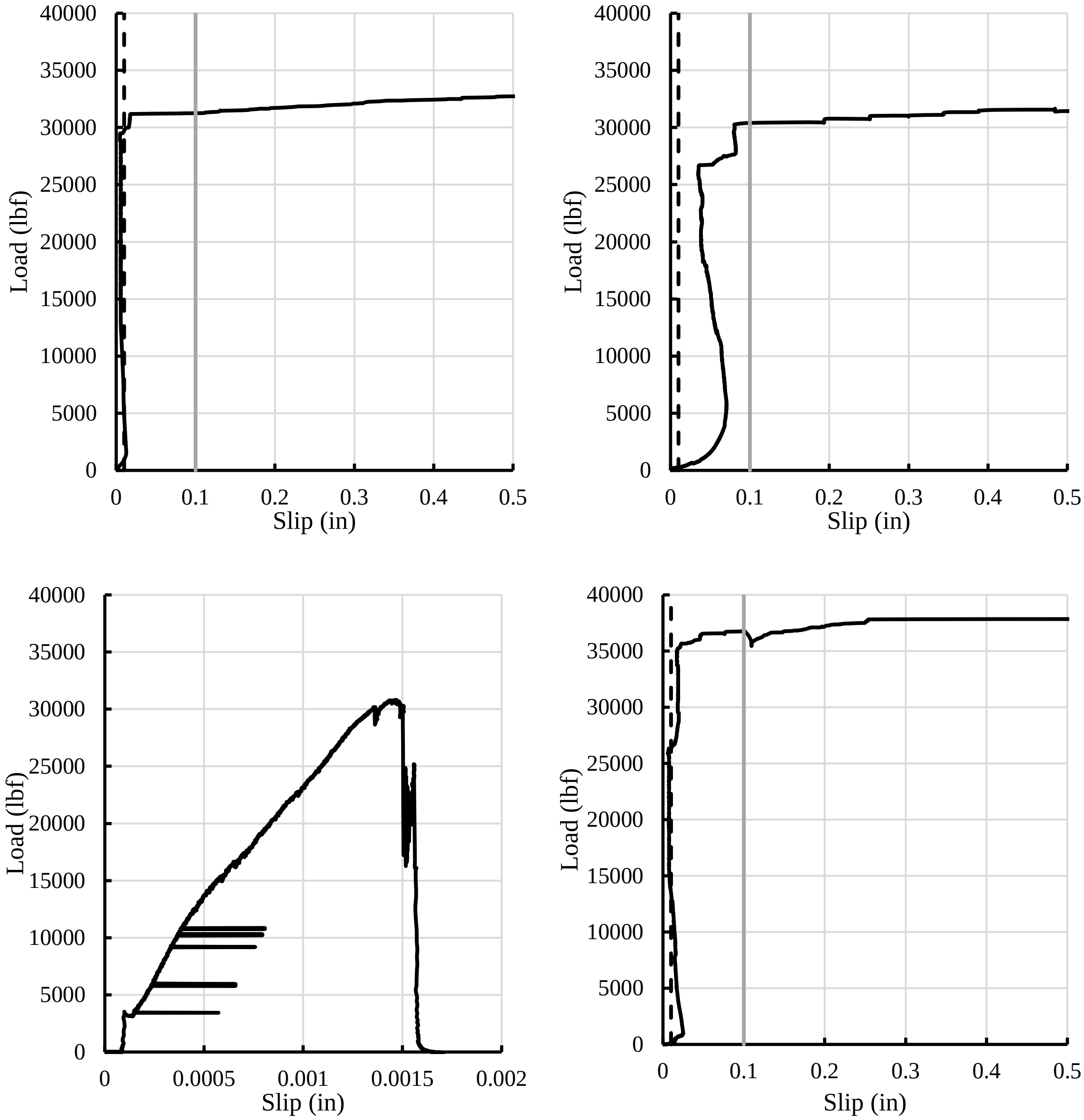

Figure A- 1 Slip versus load curves for 1-1/8 in. diameter strand specimens 1-4

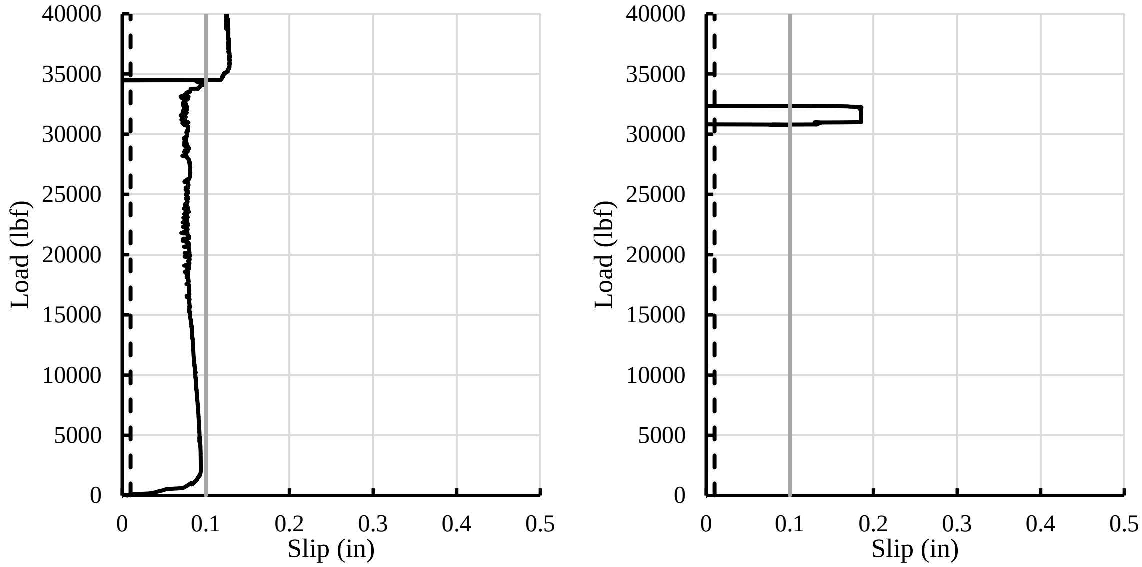

Figure A- 2 Slip versus load curves for 1-1/8 in. diameter strand specimens 5 and 6

A.2. Large Block Pullout Test

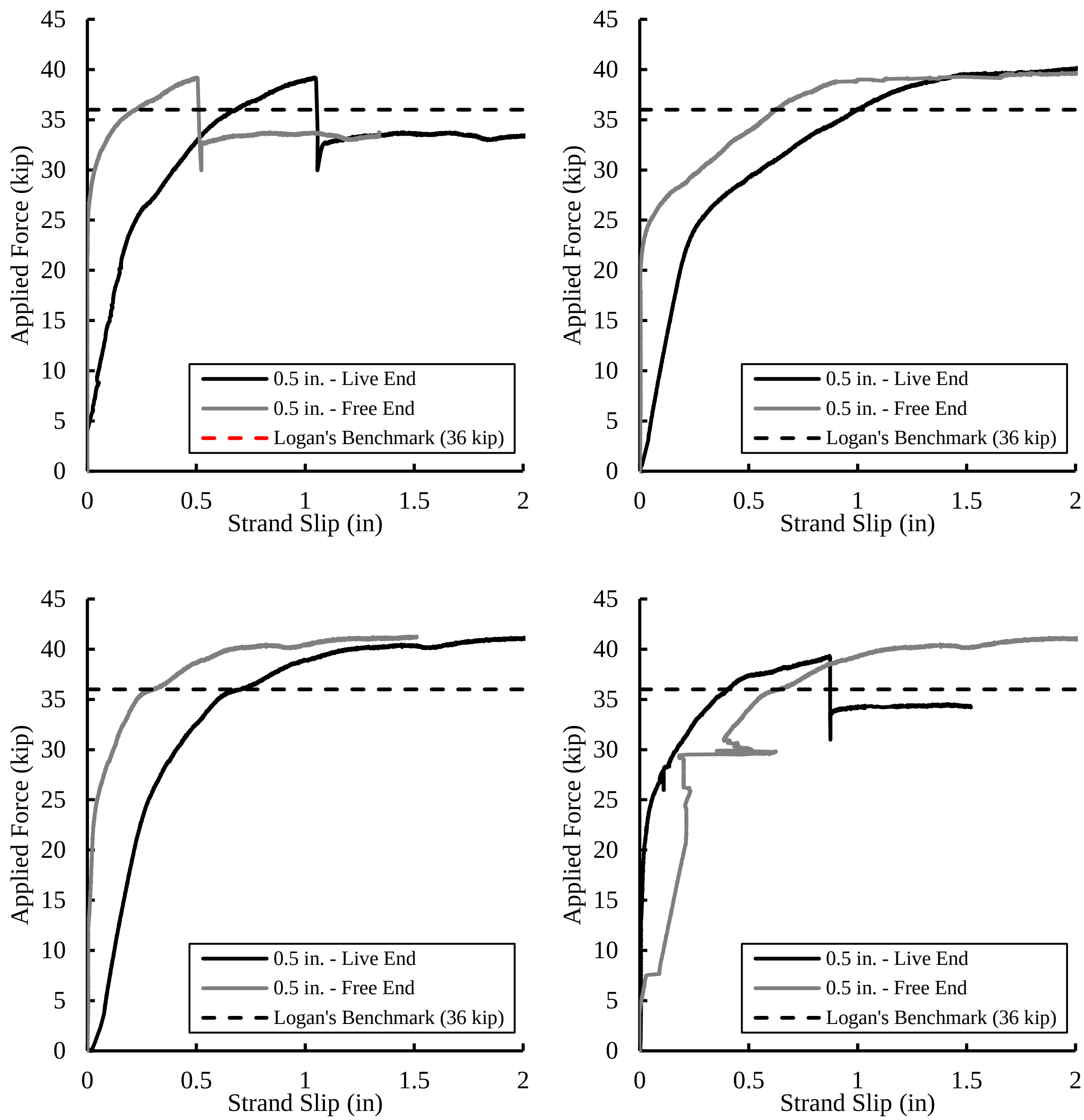

Figure A- 3 Slip versus load curves for 0.5 in. diameter strand (specimens 1-4 in 7303 psi)

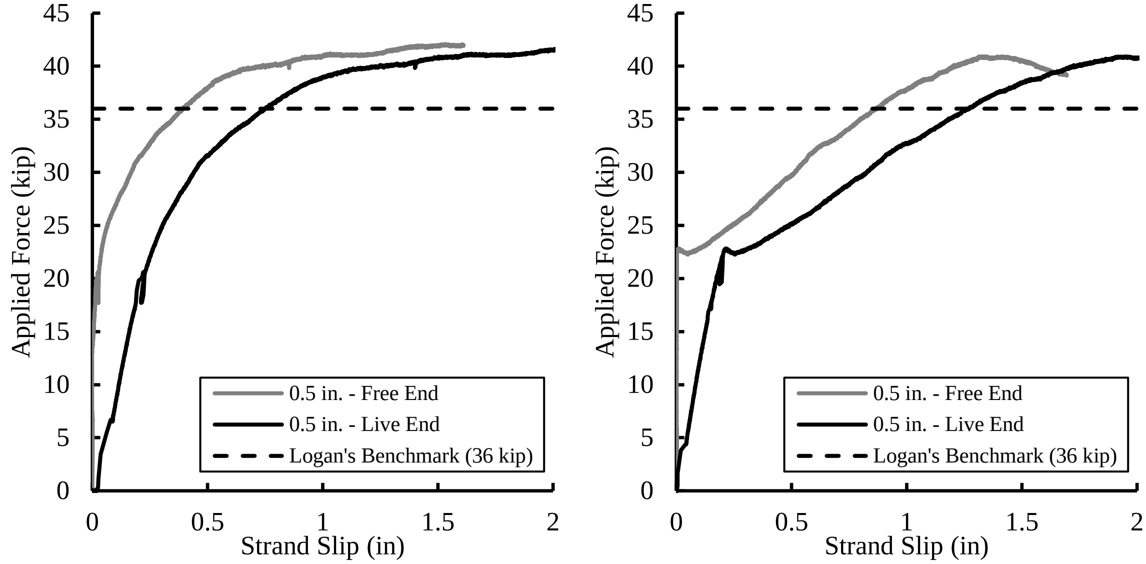

Figure A- 4 Slip versus load curves for 0.5 in. diameter strand (specimens 5 and 6 in 7303 psi)

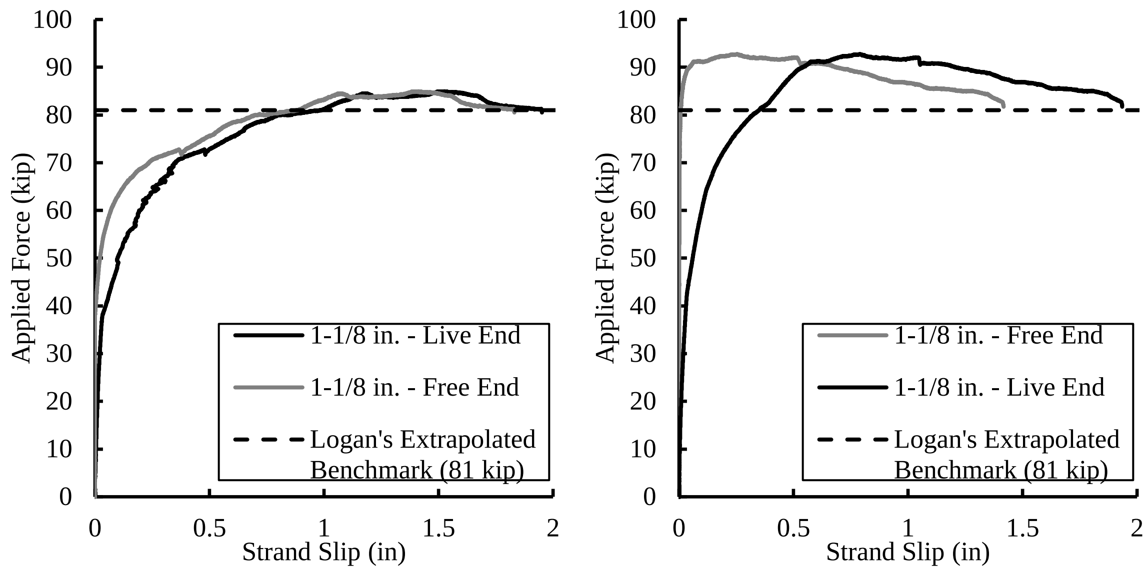

Figure A- 5 Slip versus load curves for 1-1/8 in. diameter strand (specimens 1 and 2 in 7303 psi concrete)

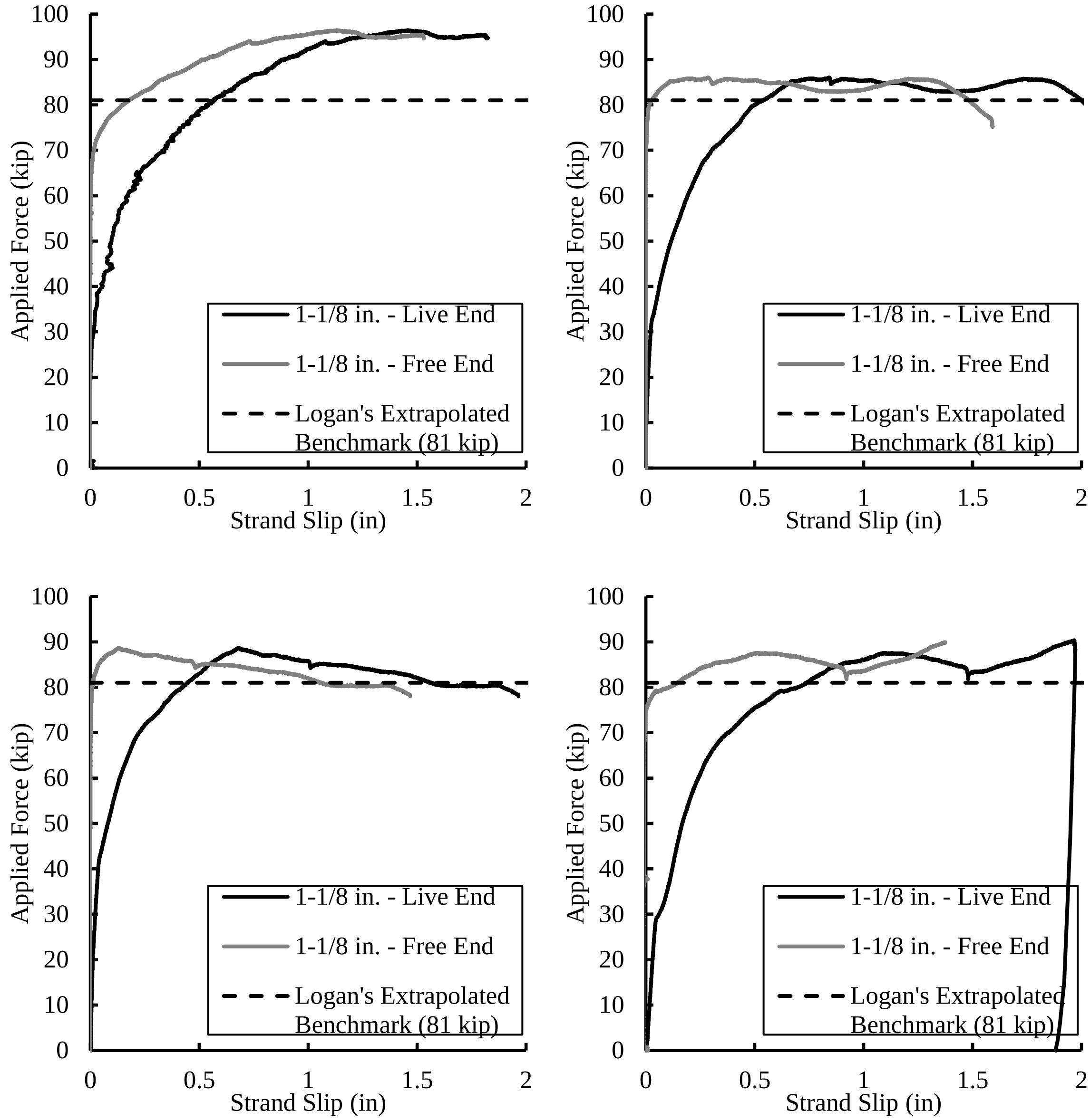

Figure A- 6 Slip versus load curves for 1-1/8 in. diameter strand (specimens 3-6 in 7303 psi concrete)

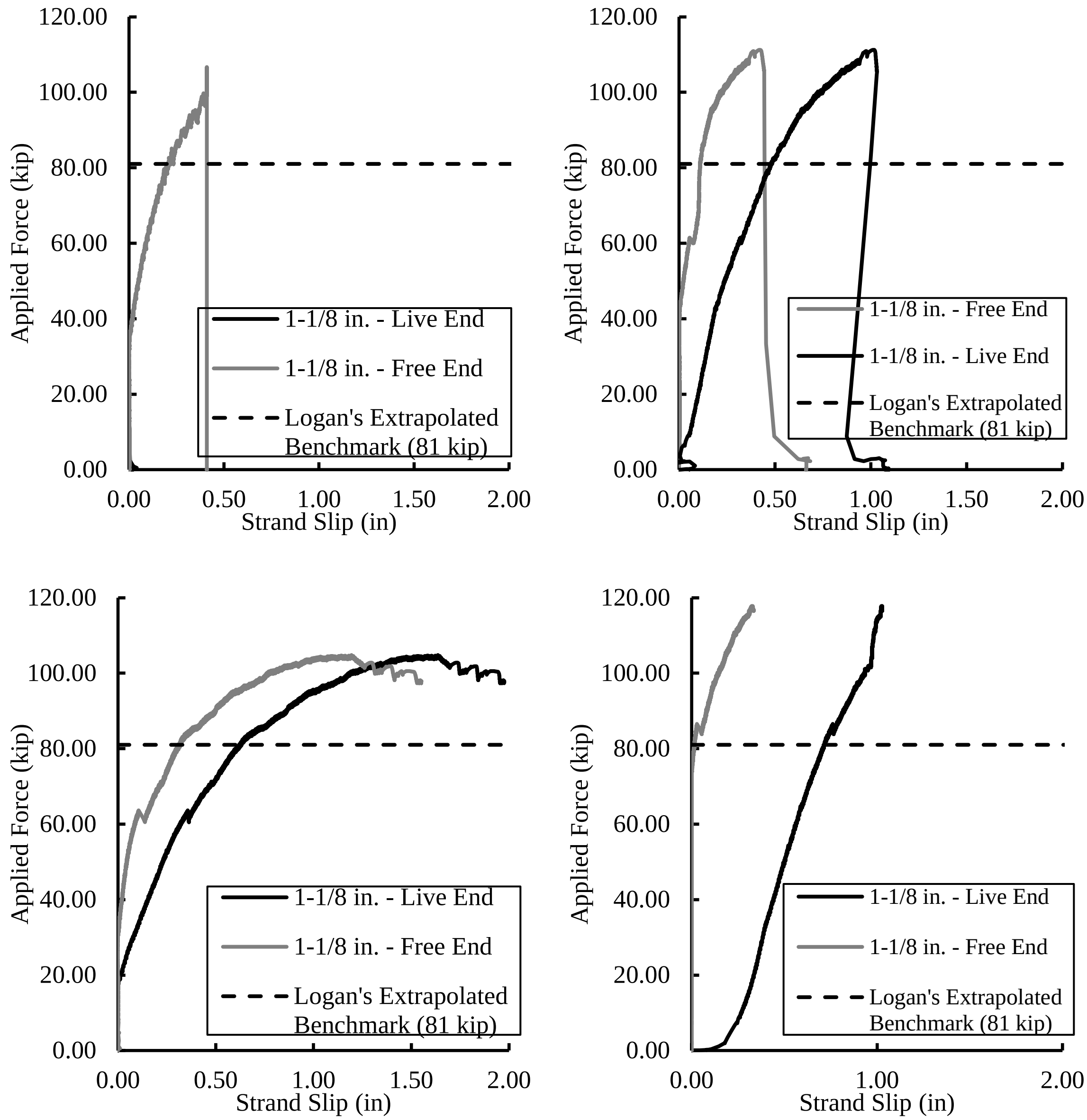

Figure A- 7 Slip versus load curves for 1-1/8 in. diameter strand (specimens 1-4 in 11980 psi concrete)

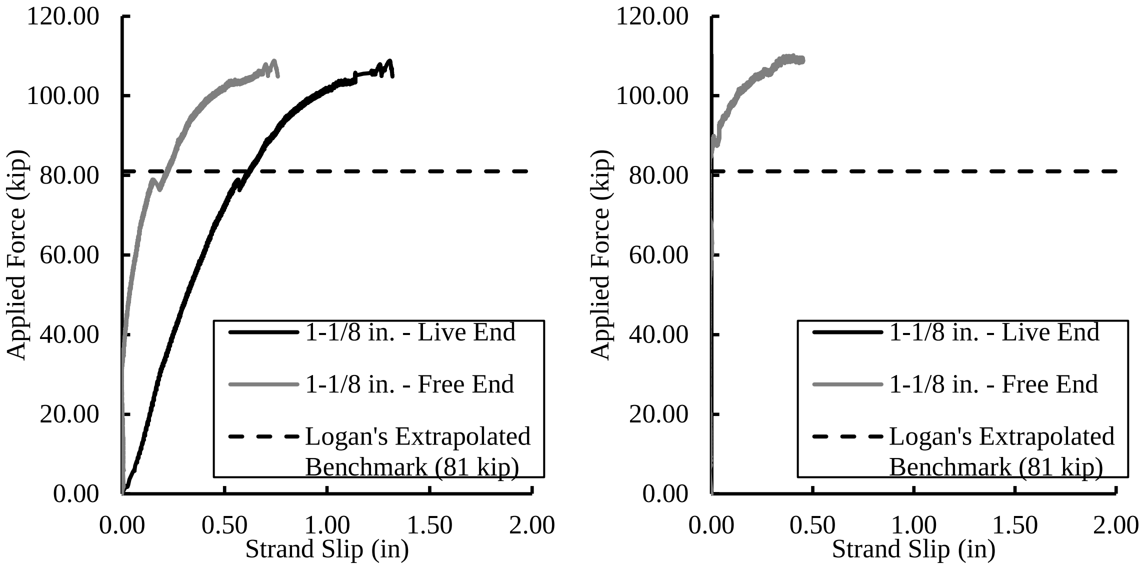

Figure A- 8 Slip versus load curves for 1-1/8 in. diameter strand (specimens 5 and 6 in 11980 psi concrete)

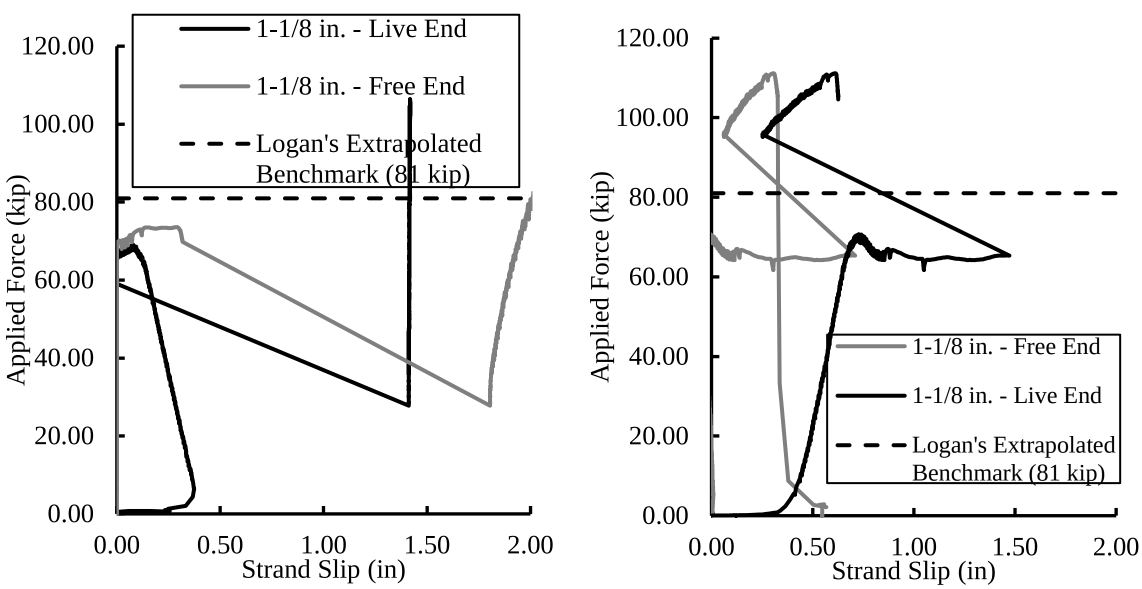

Figure A- 9 Slip versus load curves for rusted 1-1/8 in. diameter strand (specimens 1 and 2 in 6116 psi concrete)

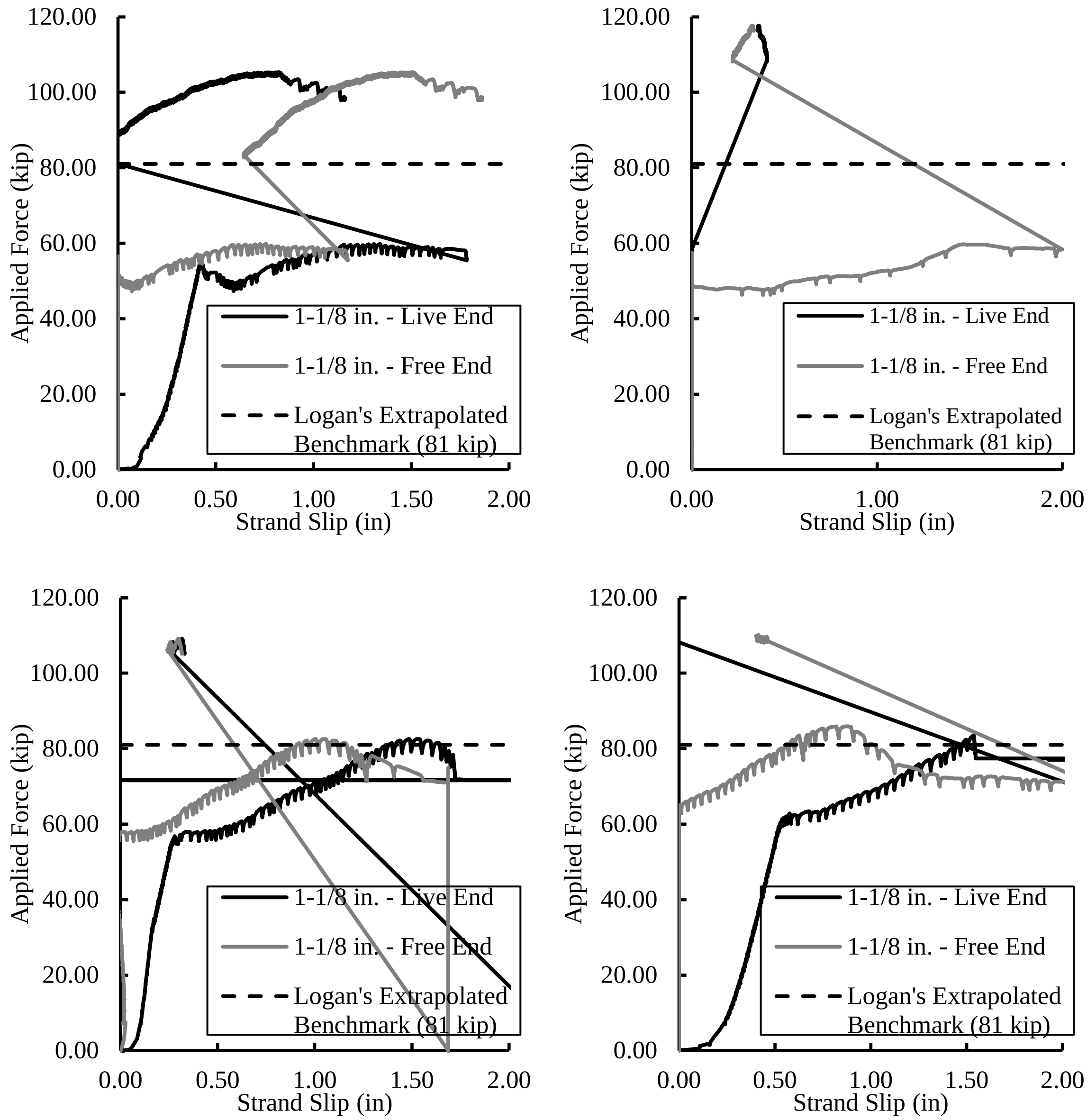

Figure A- 10 Slip versus load curves for rusted 1-1/8 in. diameter strand (specimens 3-6 in 6116 psi concrete)

A.3. Transfer length readings

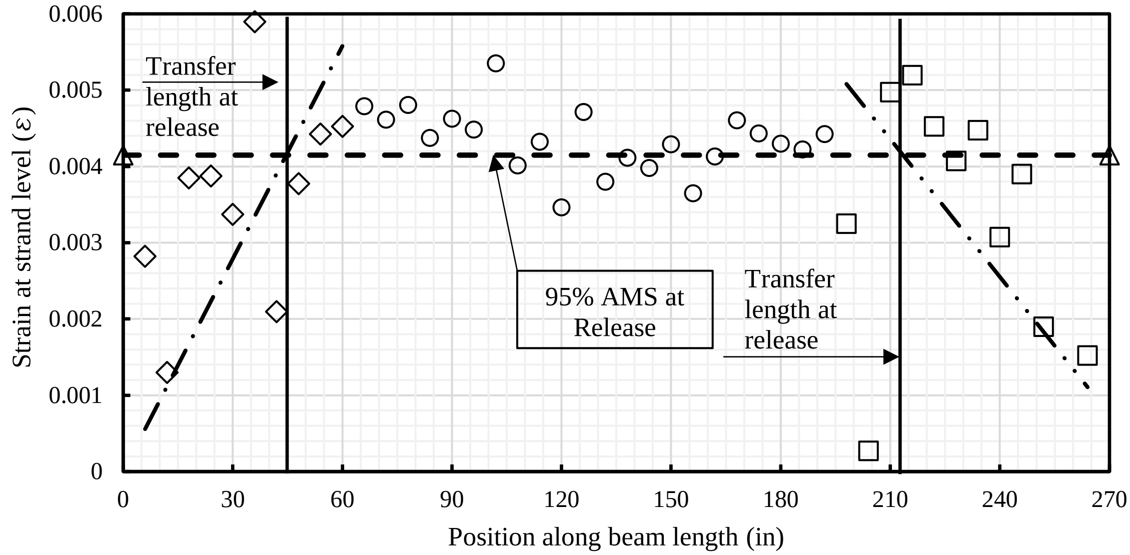

Figure A- 11 DEMEC readings for beam 1 after release

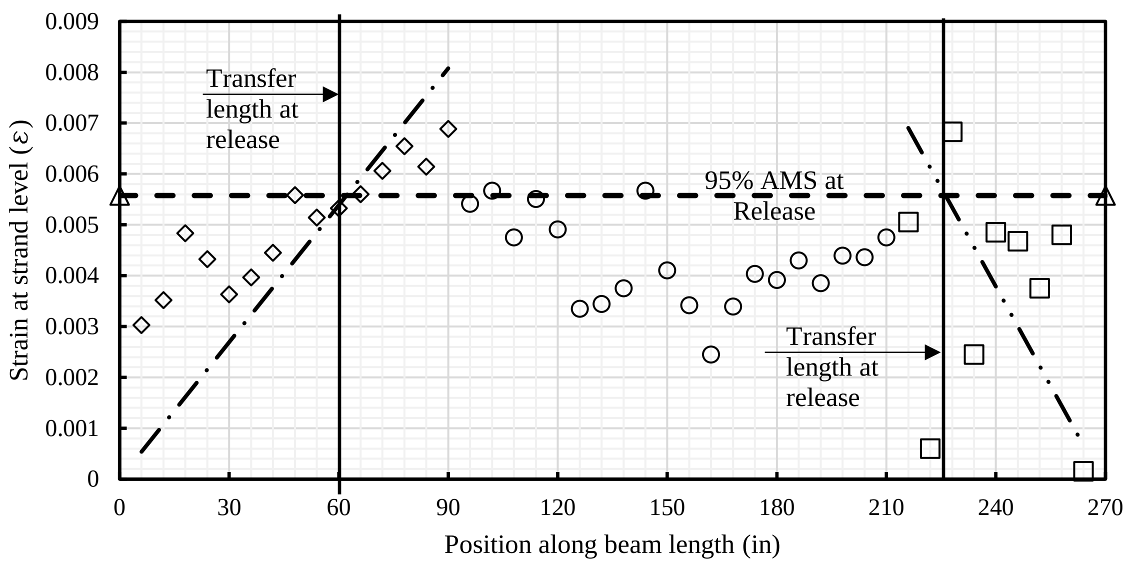

Figure A- 12 DEMEC readings for beam 2 after release

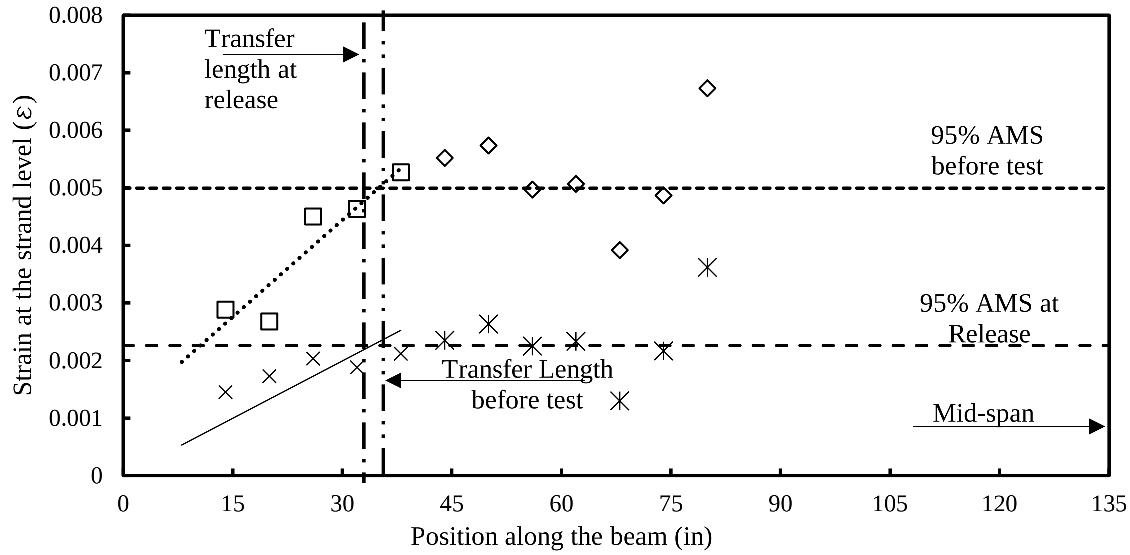

Figure A- 13 DEMEC readings of live end of beam 3 side 1

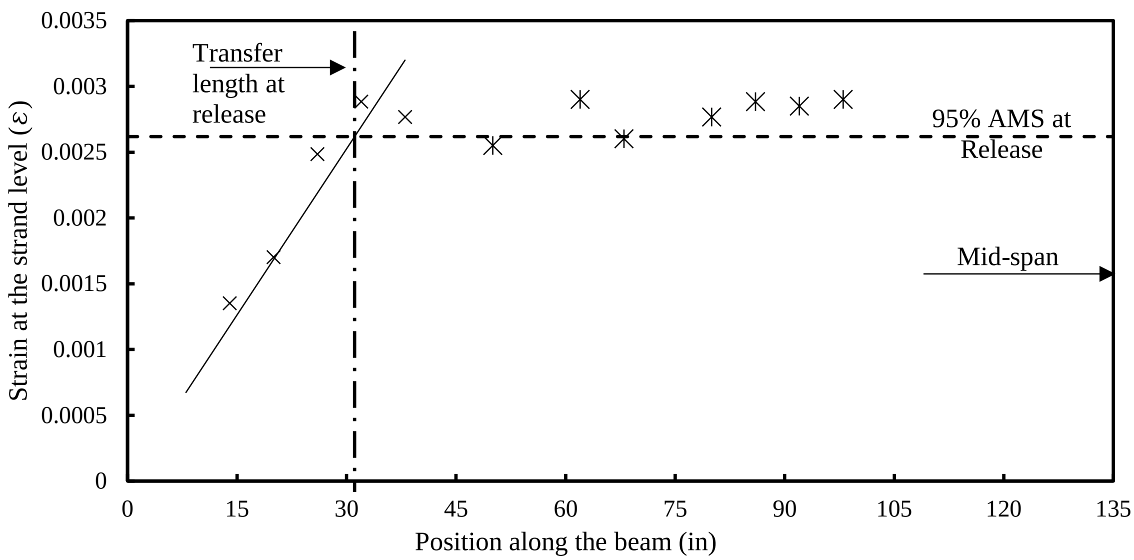

Figure A- 14 DEMEC readings of live end of beam 3 side 2

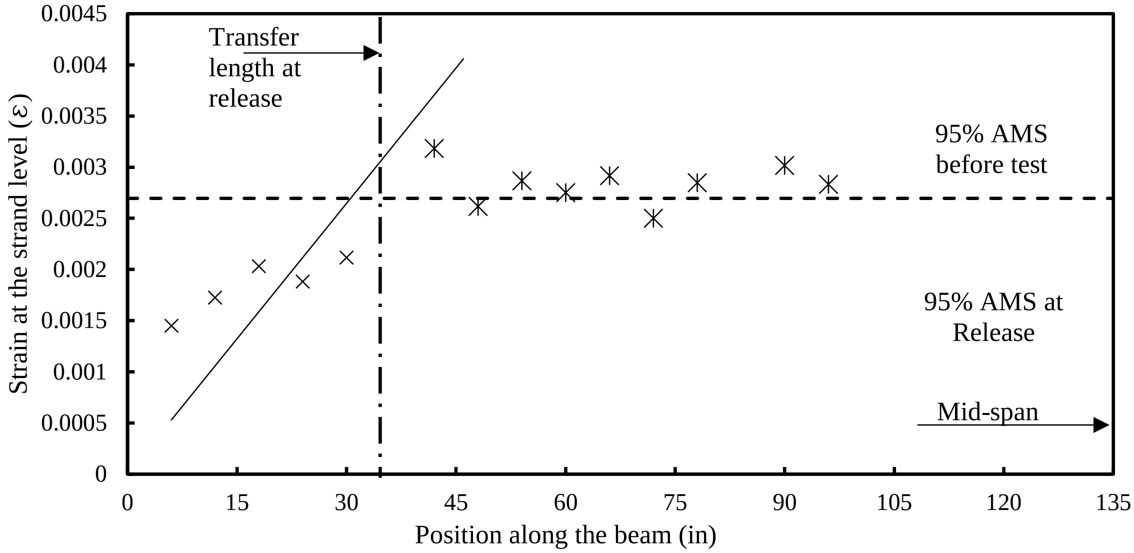

Figure A- 15 DEMEC readings of dead end of beam 3 side 1

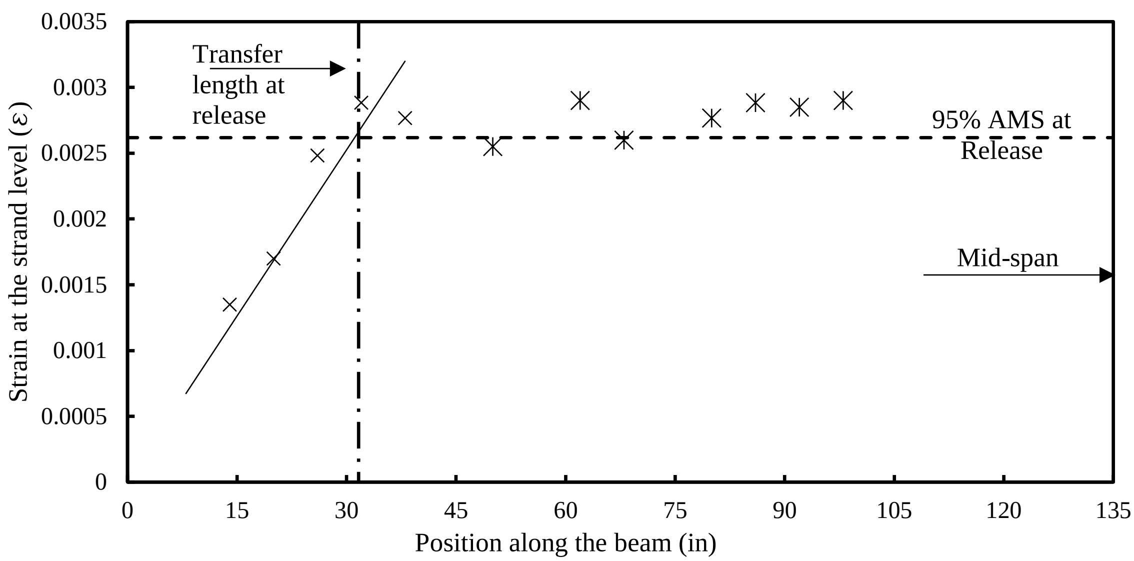

Figure A- 16 DEMEC readings of dead end of beam 3 side 2

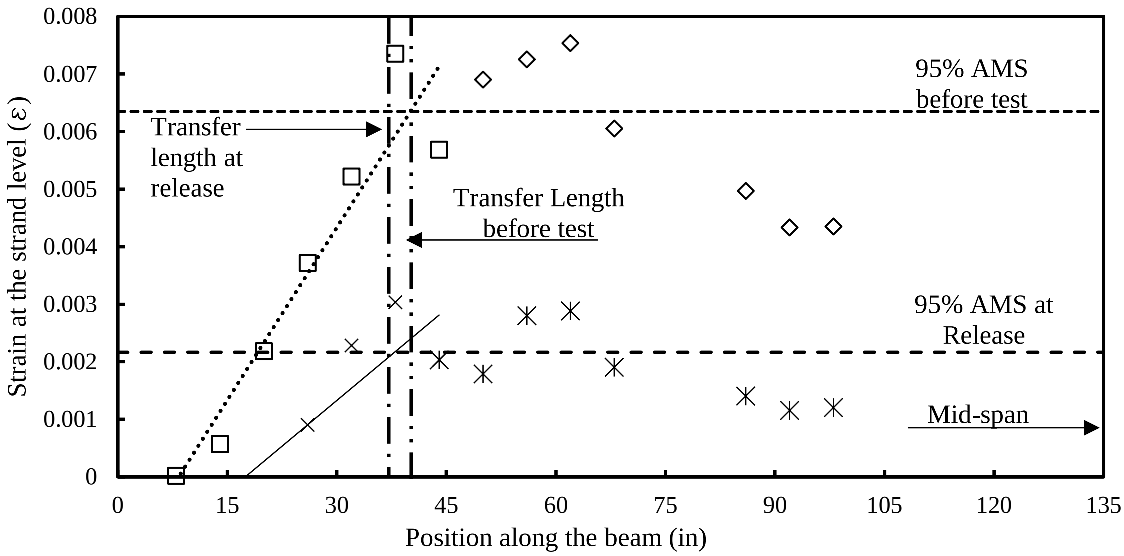

Figure A- 17 DEMEC readings of live end of beam 4 side 1

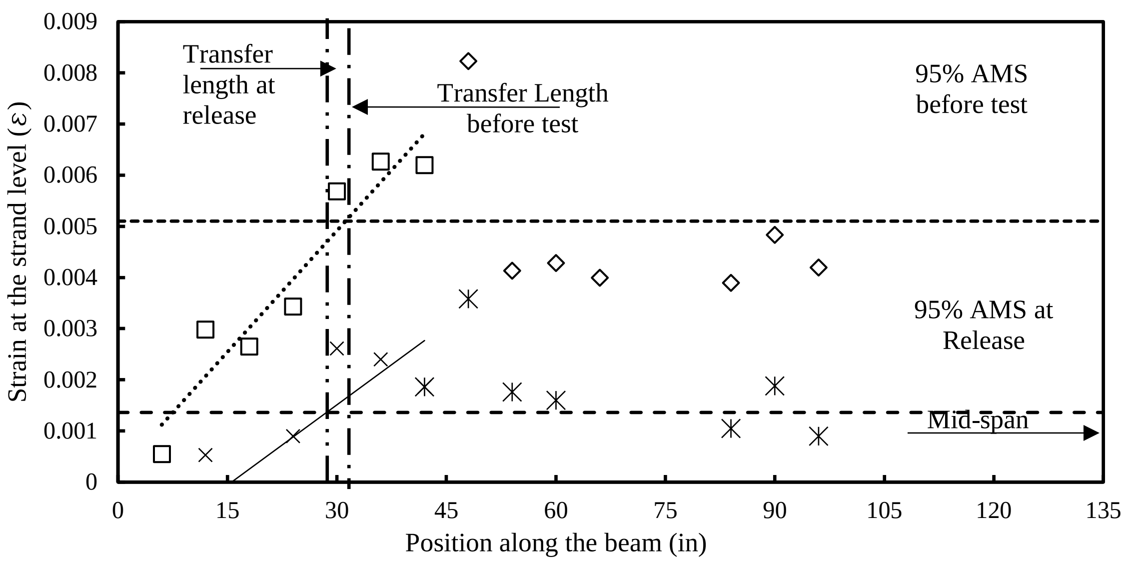

Figure A- 18 DEMEC readings of live end of beam 4 side 2

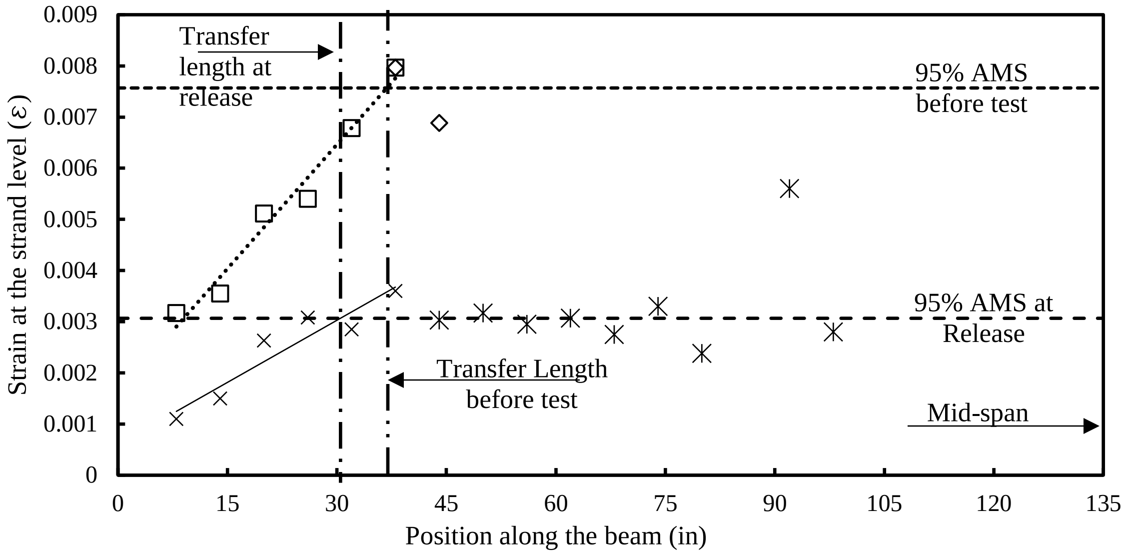

Figure A- 19 DEMEC readings of dead end of beam 4 side 1

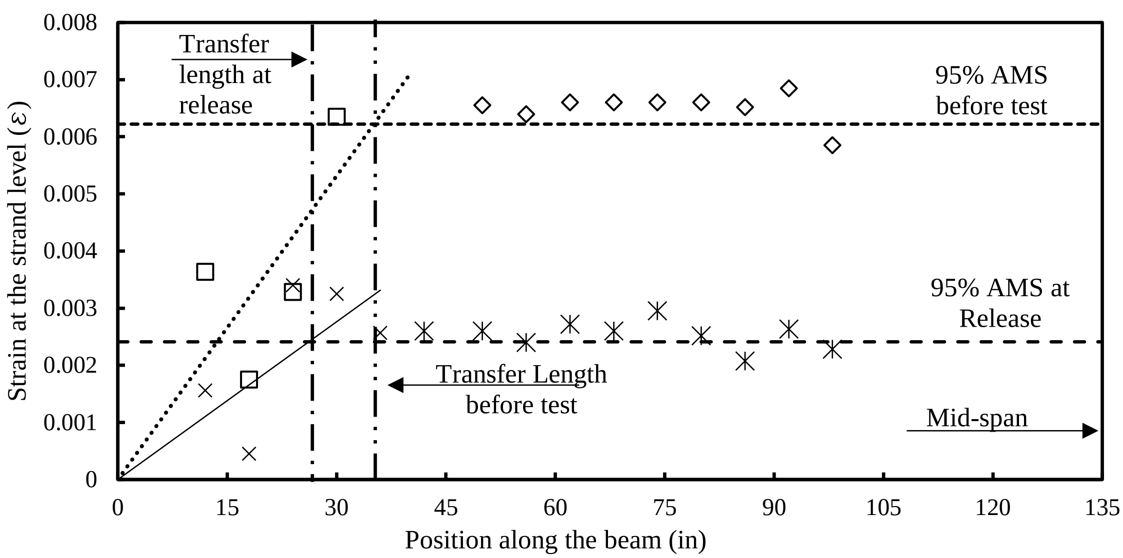

Figure A- 20 DEMEC readings of dead end of beam 4 side 2

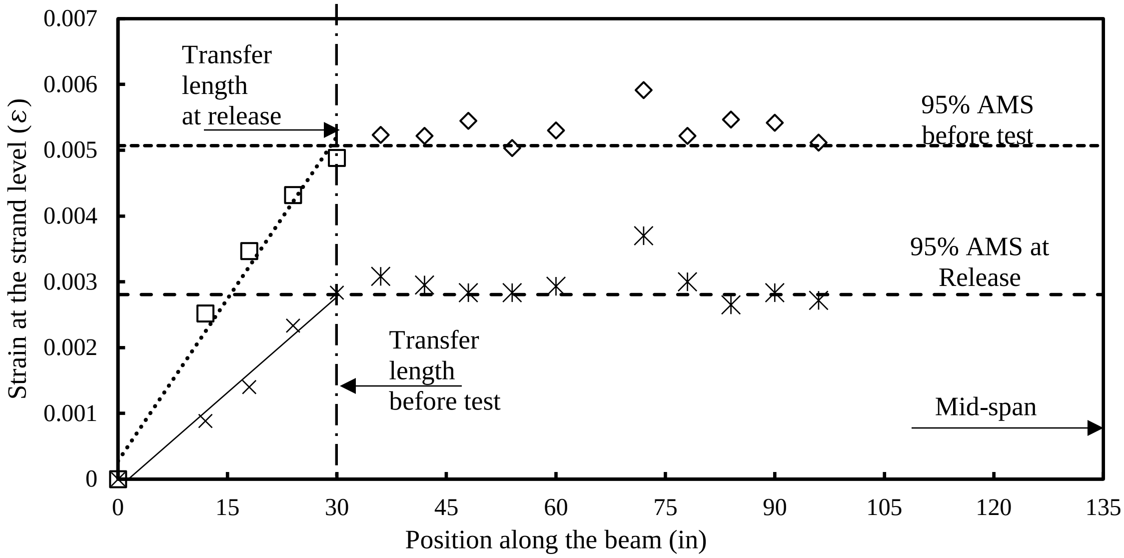

Figure A- 21 DEMEC readings of live end of beam 5 side 1

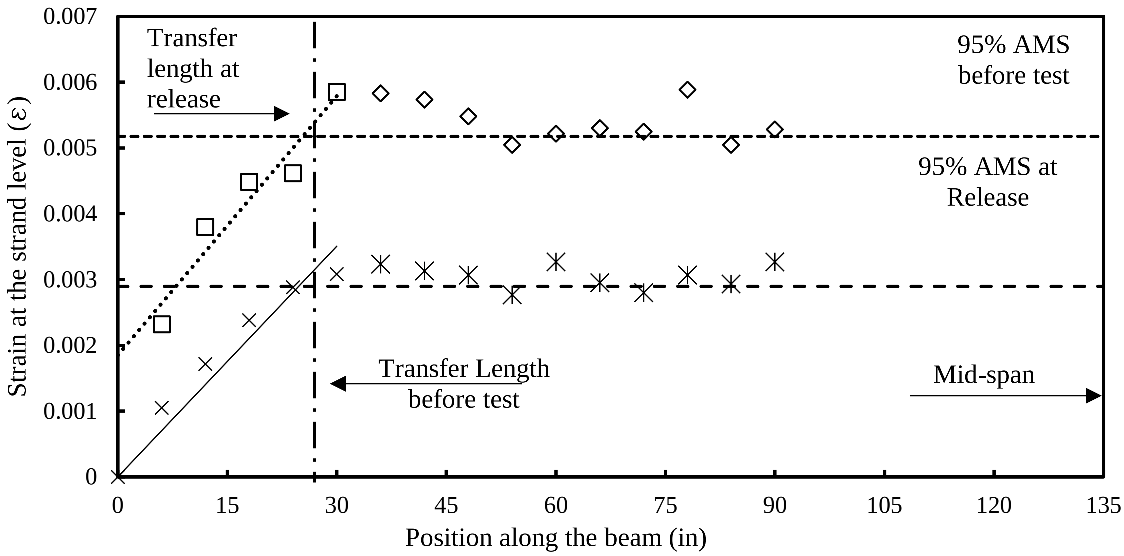

Figure A- 22 DEMEC readings of live end of beam 5 side 2

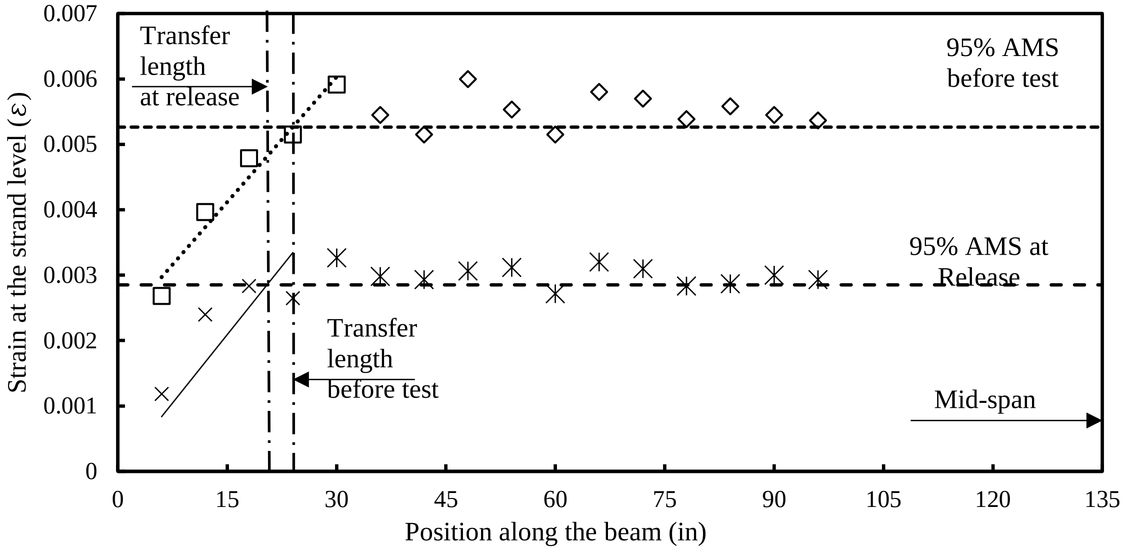

Figure A- 23 DEMEC readings of dead end of beam 5 side 1

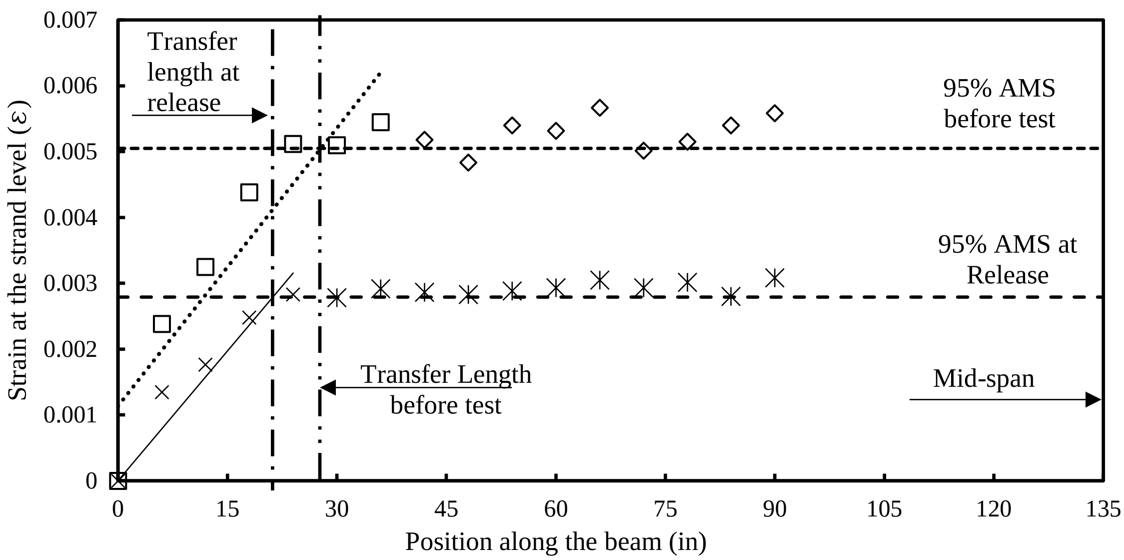

Figure A- 24 DEMEC readings of dead end of beam 5 side 2

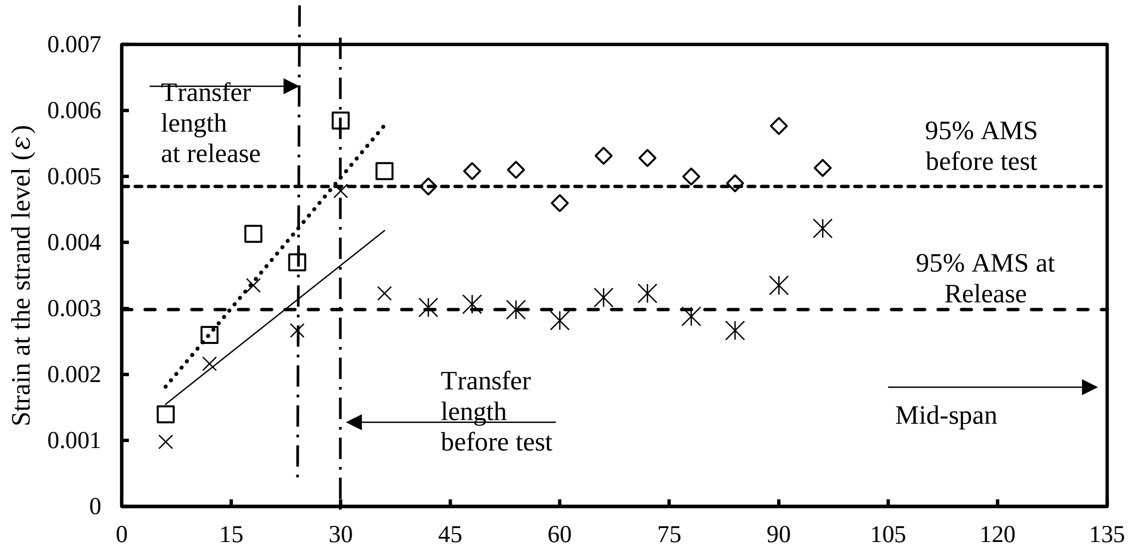

Figure A- 25 DEMEC readings of dead end of beam 6 side 1

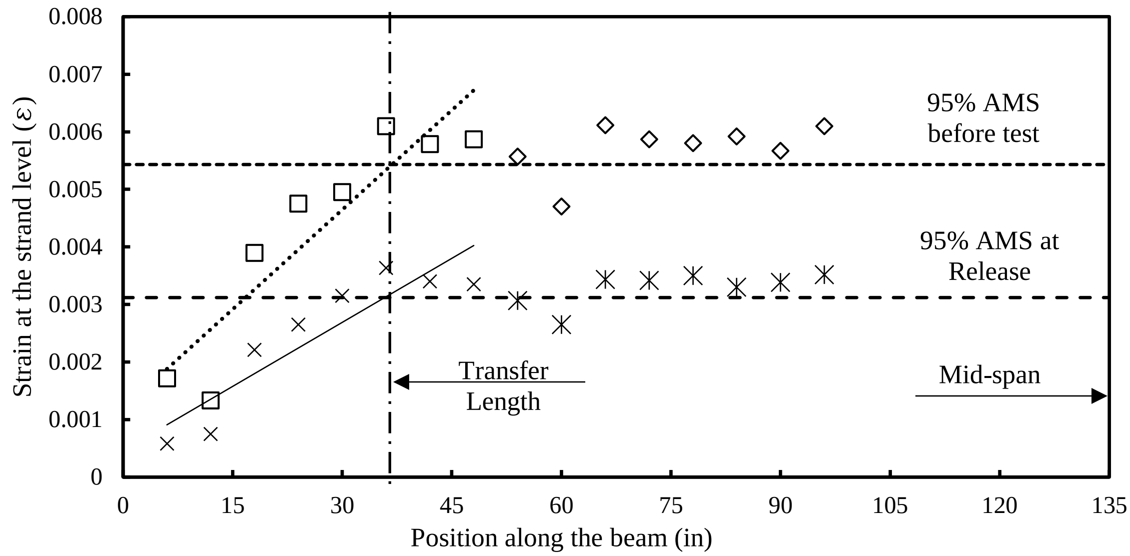

Figure A- 26 DEMEC readings of live end of beam 6 side 1

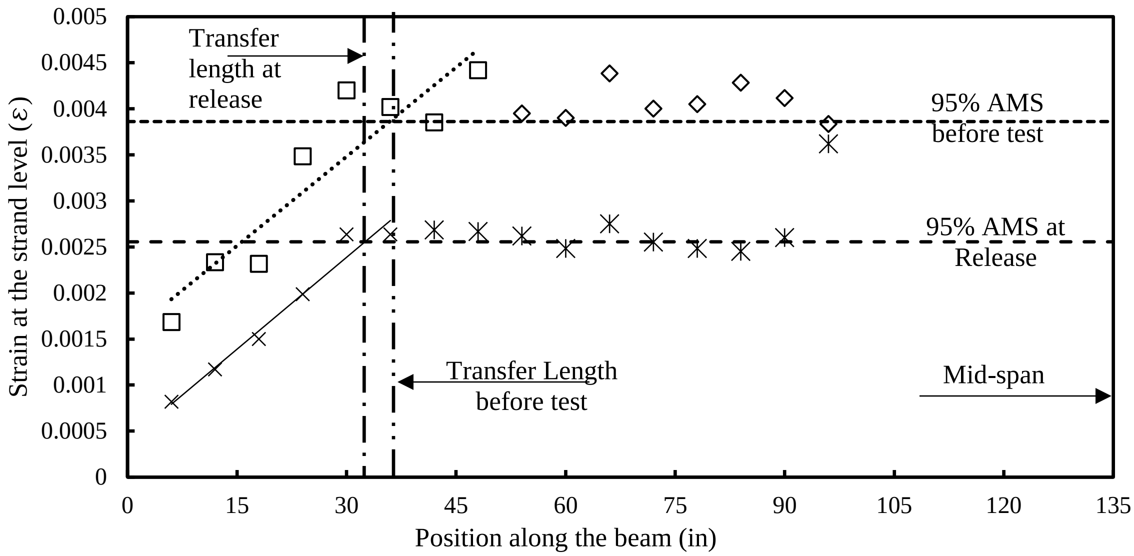

Figure A- 27 DEMEC readings of live end of beam 6 side 2

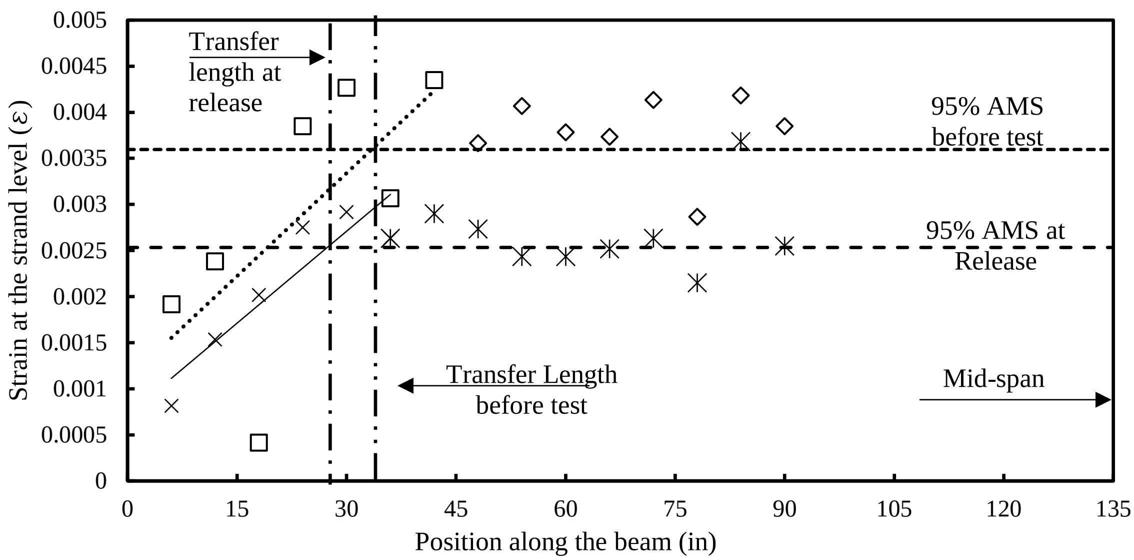

Figure A- 28 DEMEC readings of dead end of beam 6 side 2

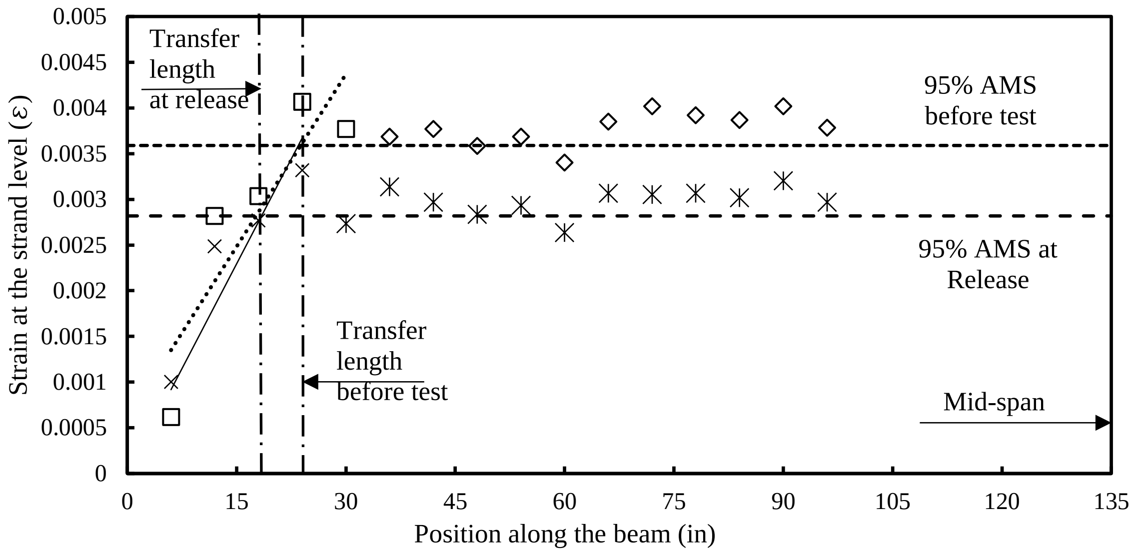

Figure A- 29 DEMEC readings of dead end of beam 7 side 1

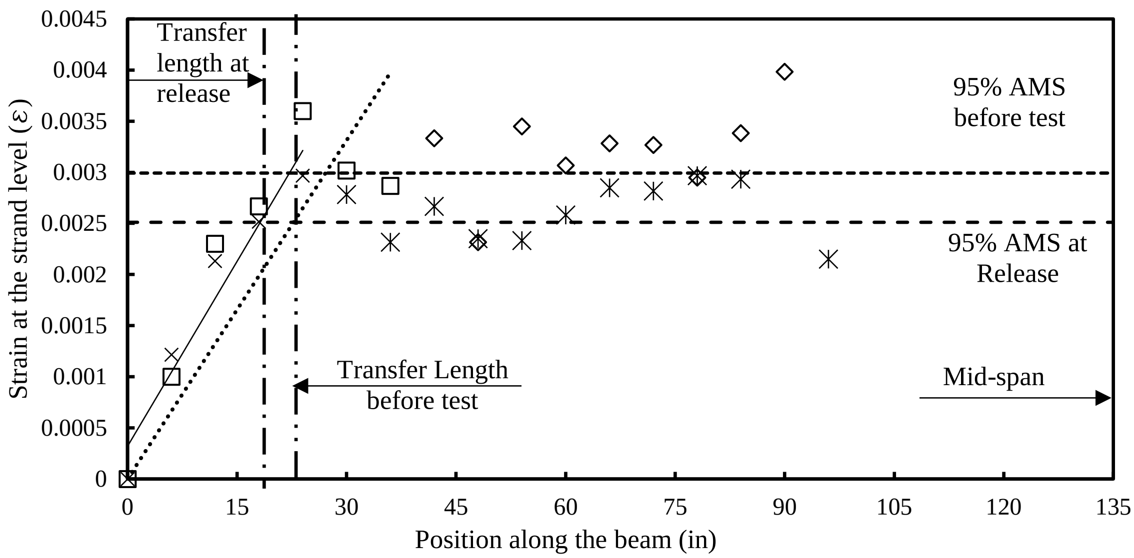

Figure A- 30 DEMEC readings of dead end of beam 7 side 2

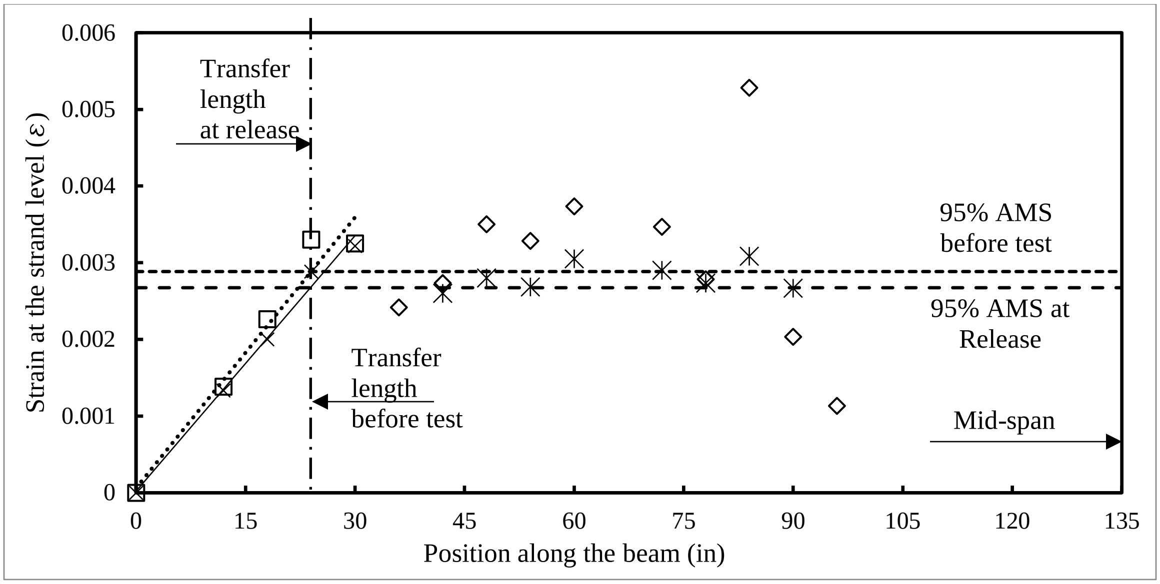

Figure A- 31 DEMEC readings of live end of beam 7 side 1

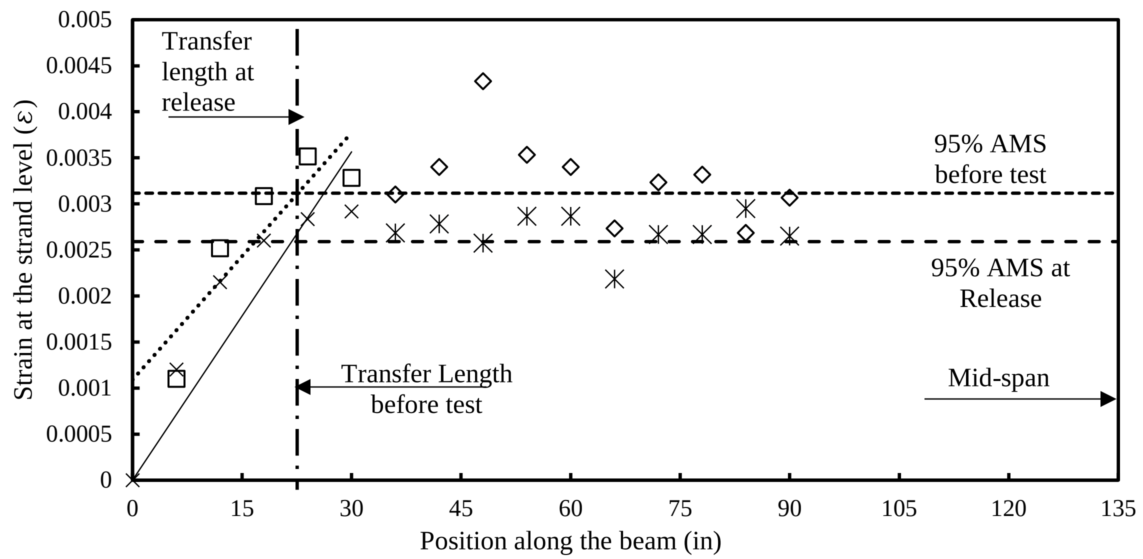

Figure A- 32 DEMEC readings of live end of beam 7 side 2

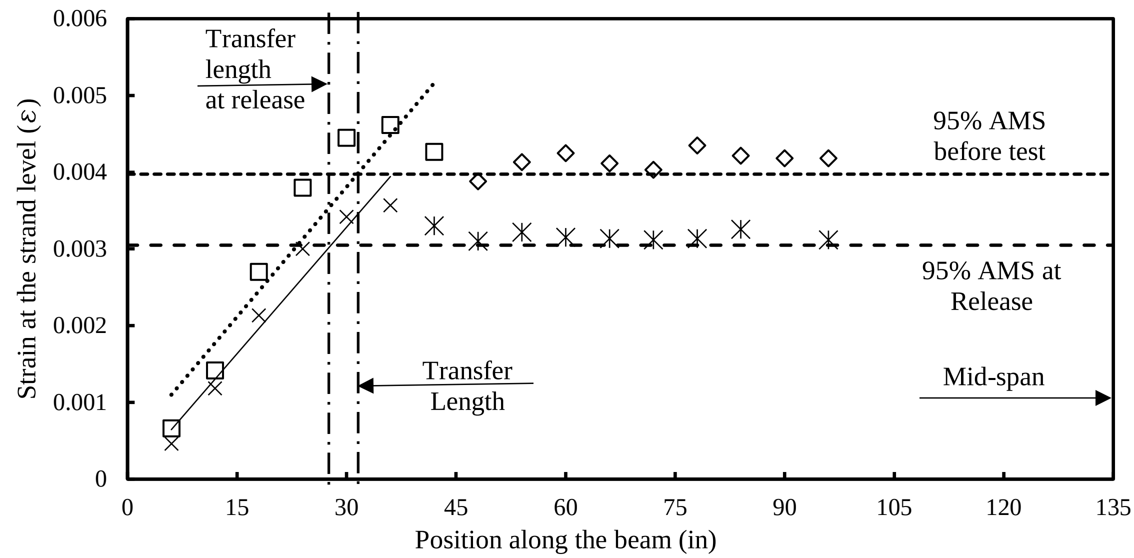

Figure A- 33 DEMEC readings of live end of beam 8 side 1

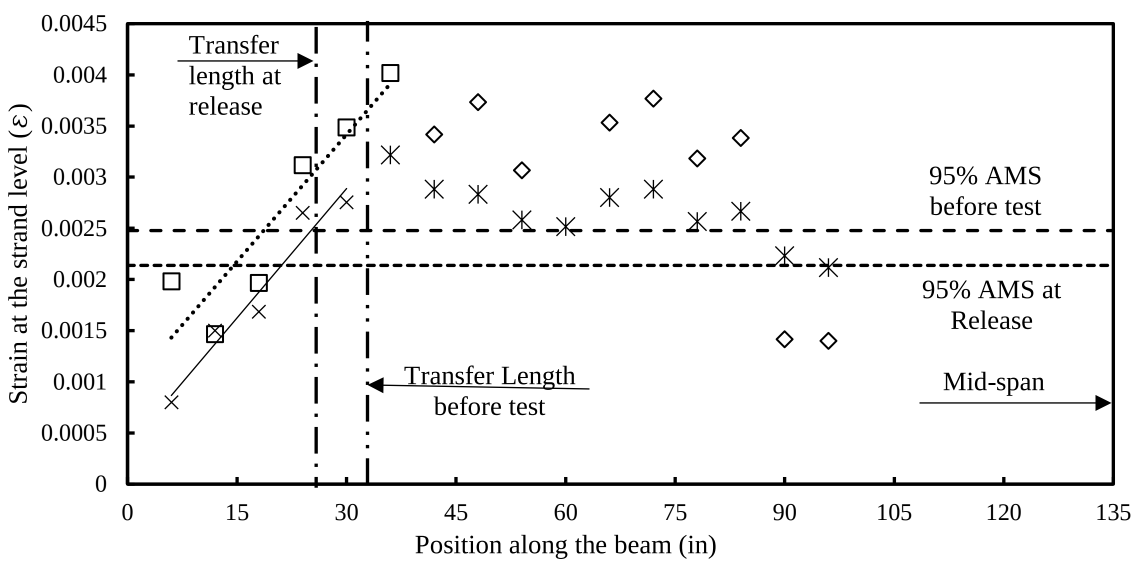

Figure A- 34 DEMEC readings of live end of beam 8 side 2

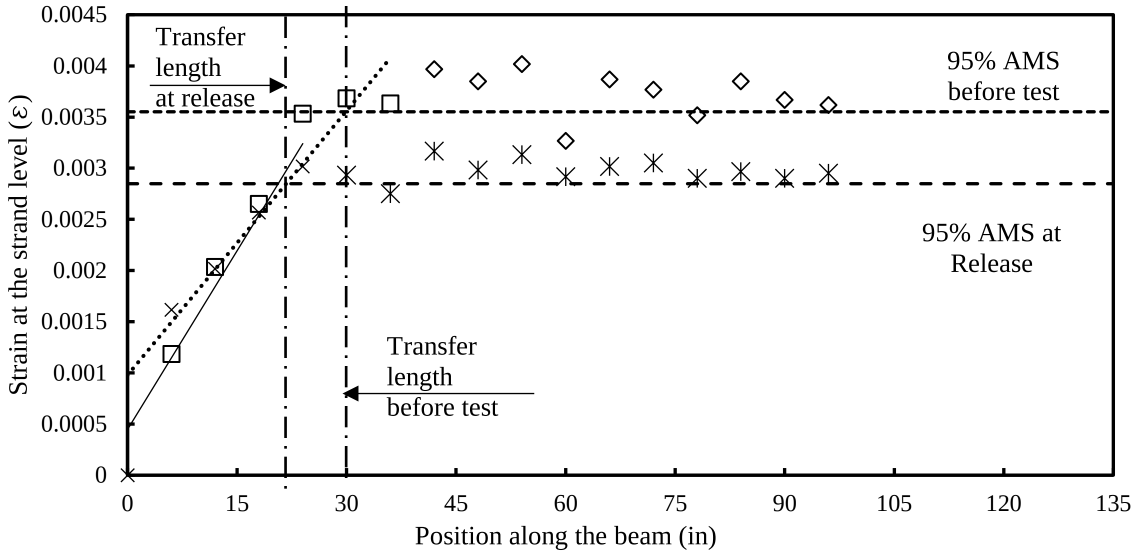

Figure A- 35 DEMEC readings of dead end of beam 8 side 1

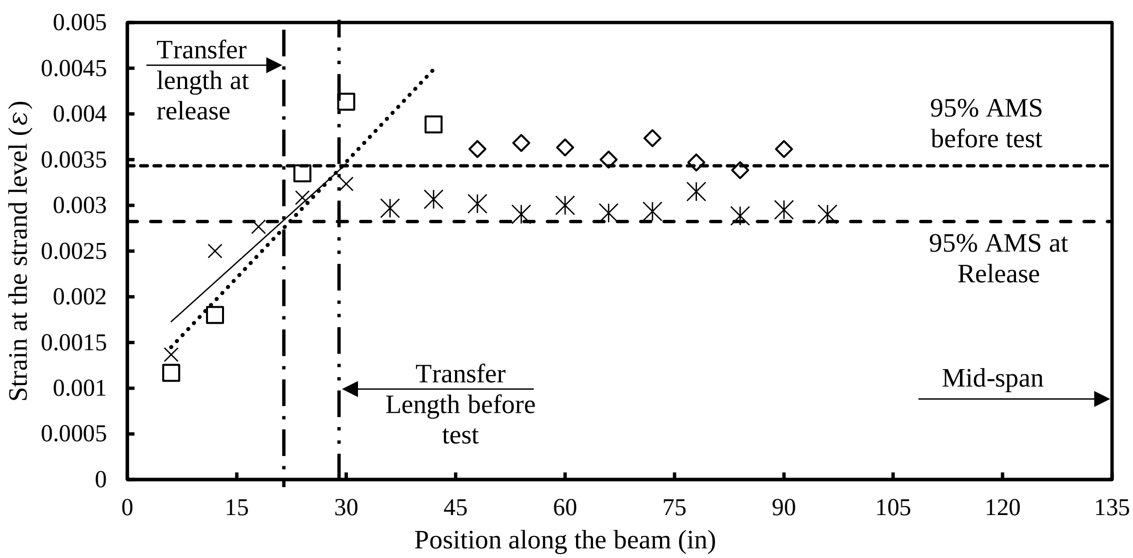

Figure A- 36 DEMEC readings of dead end of beam 8 side 2

A.4. Cracking test plots

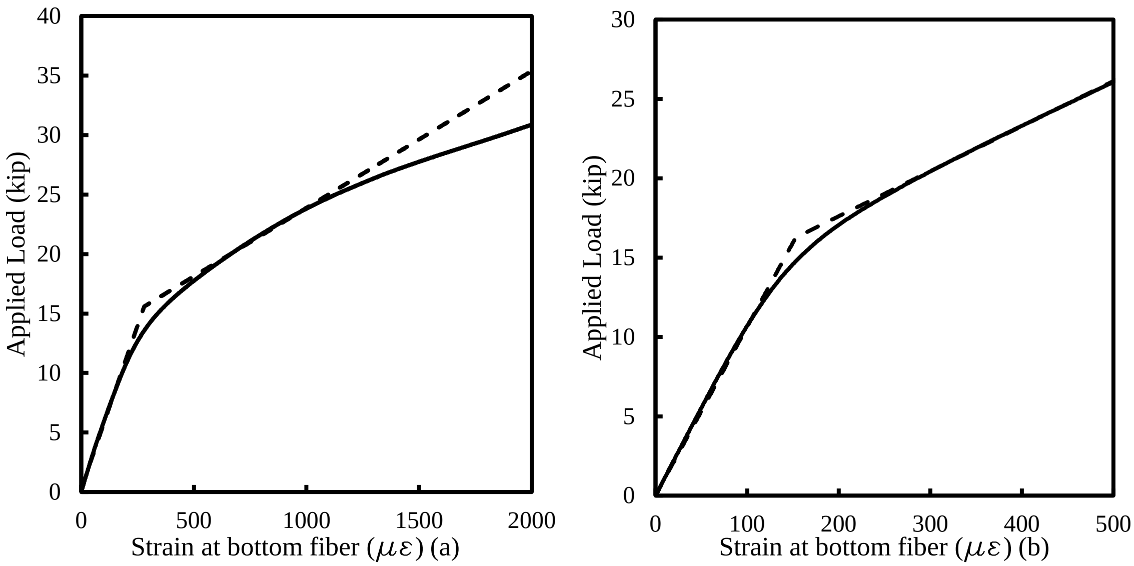

Figure A- 37 Strain vs applied load plot for beam 1 (a), and 2 (b)

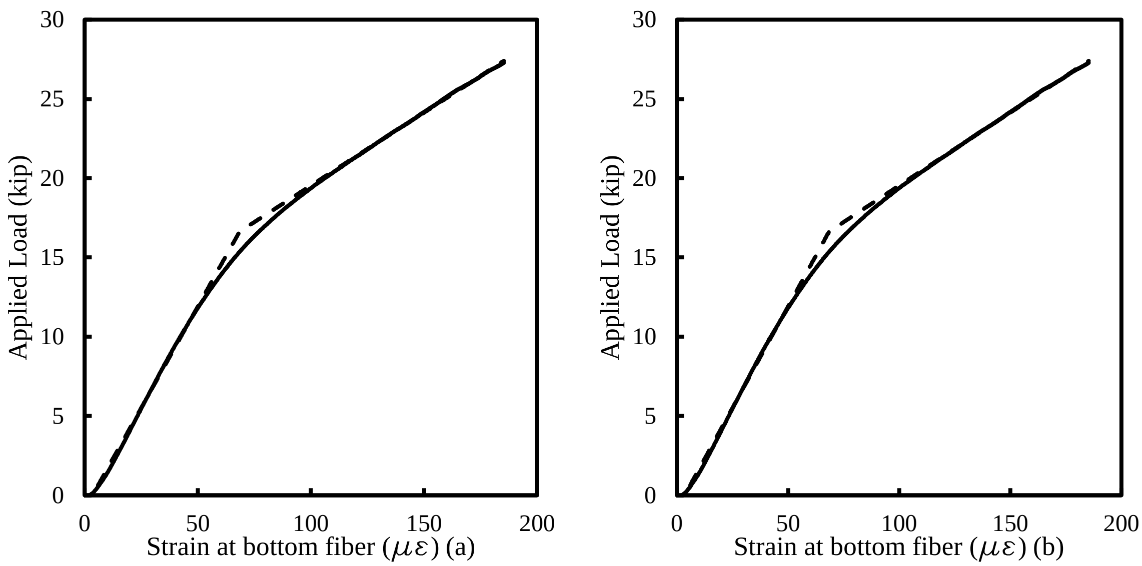

Figure A- 38 Strain vs applied load plot for beam 3 (a), and 4 (b)

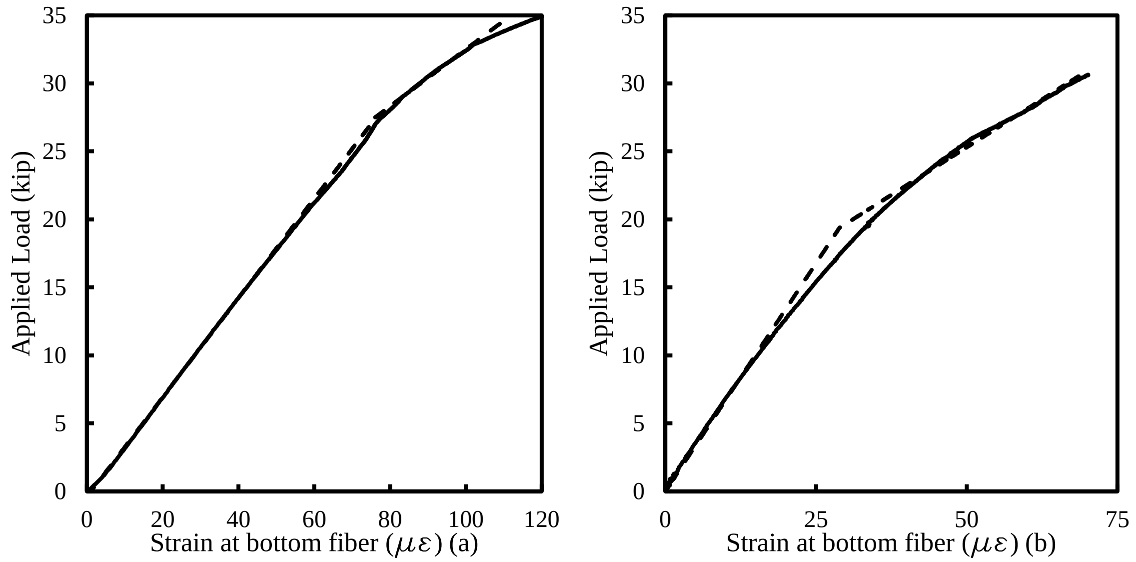

Figure A- 39 Strain vs applied load plot for beam 5 (a), and 6 (b)

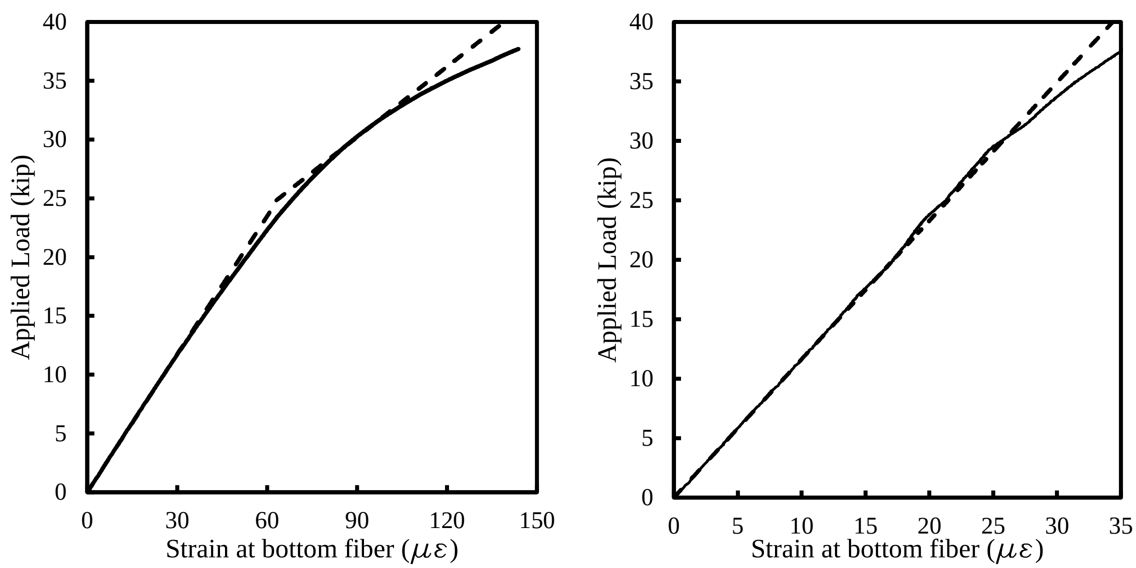

Figure A- 40 Strain vs applied load plot for beam 7 (a), and 8 (b)