12-2009

Impact of Larger Diameter Strands on AASHTO/PCI Bulb-Tees

Jayaprakash Vadivelu
University of Tennessee - Knoxville

To the Graduate Council:

I am submitting herewith a thesis written by Jayaprakash Vadivelu entitled "Impact of Larger Diameter Strands on AASHTO/PCI Bulb-Tees." I have examined the final electronic copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

Z. John Ma, Major Professor

We have read this thesis and recommend its acceptance:

Edwin G. Burdette, Qiuhong Zhao

Accepted for the Council:
Carolyn R. Hodges

Vice Provost and Dean of the Graduate School

(Original signatures are on file with official student records.)

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A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Jayaprakash Vadivelu
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DEDICATION

This thesis is dedicated to all of those people who have made life a journey truly worth taking. My parents, Vadivelu and Vijaya, for always supporting, inspiring, and encouraging me in all walks of life. My siblings, Jaya Chandran and Nithya, for a lifetime of warm memories and continued moral support and my friends.

"Live as if you were to die tomorrow. Learn as if you were to live forever."
– Mahatma Gandhi

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I would like to thank Dr. John Ma for his continued support and useful guidance throughout the entire research process. I would like to acknowledge Rick Merritt, General Manager and David Miller, Project Manager at Ross prestress plant and the personals at both Knoxville plant and Bristol plant for helping us to make the girders for this experimental program. I would like to thank Coreslab Structures, MMI Strand Co., and Sumiden Wire Products Corporation for donating materials for this research program. I wish to thank Ken Thomas and Larry Roberts, Civil engineering department shop technicians for their assistance in this research program. I would also like to acknowledge all Dr. Ma's research team for their cooperation and help in various aspects. The financial support for this research was provided by National Science Foundation (NSF) and Tennessee Department of Transportation (TDOT). Finally I would like to thank Dr. Z. John Ma, Dr. Edwin G. Burdette and Dr. Qiuhong Zhao for serving on my committee.

ABSTRACT

This thesis consists of the analytical study and the experimental investigation of larger diameter strands in AASHTO Type I girders. The main purpose of this study was to verify that the 2 inch minimum spacing recommended by ACI 318-08 and AASHTO (2008) can be used for 0.7 inch diameter strands by comparing various effects in girders using 0.7 and 0.6 inch diameter strands. Based on the parametric analysis it was concluded that by using 0.7 inch strands there was a considerable saving in the material. For example, an AASHTO BT-72 with 0.6 inch strand could be replaced with AASHTO BT-54 with 0.7 inch strand for the same span capacity. In order to fully realize the benefits and to verify the adequacy of 2 inch spacing, a three dimensional finite element analysis was performed with two full-scale AASHTO Type I girders with 0.6 inch and 0.7 inch diameter strands. Only the effects due to the prestressing force at transfer were studied in the two models. The maximum principal stress and the axial stress in the concrete along the direction of the strands were determined. Based on the analytical results from the FE model it was found that the girder with the 0.7 inch diameter strand was more vulnerable to cracking at the transition zone between the bottom flange and the web. This defect could be overcome by placing the required amount of confinement reinforcement at the end zone of the girder. Based on the analytical study, two I-girder specimens, one with larger 0.7 in. strand and other with high strength 0.62 in. strand were cast. The transfer lengths of both the girders were measured and compared with the current AASHTO 2008 and ACI 318-08 equations. It was found that both strands exhibited a shorter transfer length than obtained in the equations. Based on these experimental results further studies are to be carried out for the implementation of these highly efficient strands.

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CHAPTER 1

INTRODUCTION

1.1 Overview

Pretensioned, prestressed members such as I-girders are widely used in the construction of bridges. The strand diameters used in these members are predominantly 0.5 in. and 0.6 in. at spacing of 2 inches in both horizontal and vertical directions. This research verifies whether this 2 inches spacing is suitable for the larger diameters such as 0.7 in. and 0.62 in. strands.

1.2 Need for larger diameter strands

In sections like I-girders, the area in the bottom flange to accommodate the strands is limited. Using the 0.7 inch diameter strands can decrease the required number of strands in a given section for an equivalent span capacity. Alternatively, an equal number of the larger 0.7 inch diameter strands can be used to accommodate longer spans for a given section with higher concrete strength. Further, an increased roadway clearance can possibly be achieved by using shallower members. The research conducted on 0.7 inch diameter strands is very limited.

States like Tennessee use AASHTO Bulb-Tee (BT) sections which have very limited room in the bottom flange when compared to Nebraska University (NU) sections. Thus, using larger diameter strands helps in increasing the span capacity of the girders without increasing the number of strands in the bottom flange of the section. Thus, these states which are using the Bulb-Tee sections can obtain longer sections without switching over to NU sections or changing their form work. This prevents them having to make extensive changes to the design and fabrication procedures.

1.3 Scope of the Research

Experience with using 0.7 in. and 0.62 in. strands at 2 inches spacing is very limited. Thus, the lack of research has limited its application in the real world. The main scope of this thesis is to provide design guidelines for 0.7 in. and 0.62 in. strands with 2 inches spacing by analytical and experimental studies.

1.4 Research Objective

The primary objectives of this research program were

- To determine the transfer length for both 0.7 inch and 0.62 inch diameter strands.
- To determine the prestress losses and observe the development of cracks on full scale specimens.
- To develop design criteria for larger diameter (0.7 in.) and high strength (0.62 in.) strands.

1.5 Parametric study

A parametric study was conducted to see the effects of higher diameter strand on the different cross sections such as NU sections and AASHTO Bulb-Tee sections. The design was based on the AASHTO LRFD 2008 design specification.

1.5.1 Design Assumptions

An example composite bridge with a single span was designed for this study using AASHTO Bulb Tee or Nebraska University sections topped with concrete deck. The following assumptions were made for this study:

- The superstructure consists of six beams spaced at 8 feet center to center.
- The bridge was designed with an 8 inch cast-in-place concrete deck to resist all the superimposed dead, live, and impact loads.
- A ½ inch wearing surface was considered as a part of the 8 inch thick deck.
- An additional 2 inches of wearing surface was considered to be the future wearing surface.
- Different concrete strengths were considered such as 10 ksi, 15 Ksi and 28 Ksi for the prestressed concrete girder.
- The live load considered was HL-93, which consists of a load combination of design truck or design tandem with a dynamic allowance and a design lane load of 0.64 kip/ft without a dynamic allowance.
- The concrete strength for the precast concrete deck was 4 ksi at service.

The design was accomplished in accordance with the AASHTO LRFD 2008 bridge design specification. The concrete strength at transfer was taken as 80 percent of the strength at service.

1.5.2 Section shape Vs. Strand size

1.5.2.1 ASSHTO Bulb -Tees

The cross section area of a 0.7 inch (0.294 in^2) diameter strand is about 35% more than the cross section area of a 0.6 inch (0.217 in^2) diameter strand as shown in Table 1. In this example the design was done for three different strand diameters (0.5 inch, 0.6 inch and 0.7 inch) with 2 in. spacing. The concrete strength considered in this design was 8 ksi at transfer and 12 ksi at service. A spread sheet was setup where the span was increased and the corresponding numbers of strands were noted. The capacity of the AASHTO Bulb-Tee (BT- 54) girder, with the maximum 40 strands which could be accommodated in the bottom flange, increased 16.7 percent as the diameter of the strands was increased from 0.6 in. to 0.7 in. as shown in the Table 2. A typical cross section of the AASHTO Bulb-Tee section is shown in the Figure 1 with 40 strands in the bottom flange.

When an AASHTO Bulb-Tee 72 with 0.6 inch strands was compared with an AASHTO Bulb-Tee 54 with 0.7 inch strands it could be seen that both sections had essentially the same span capacity. These comparisons showed a considerable reduction in the section size when 0.7 inch strands were used, as shown in Figure 2.

Table 1 Properties of different diameters of strand

Strand Diameter (in)	**Strand Area (in^2)**	**Ultimate Strength (ksi)**	**Jacking Force (JF), (kips)**	**Percentage increase, JF %**
0.5	0.153	270	31.0	
0.6	0.217	270	43.9	42.0
0.62	0.2227	330	57.5	31.0
0.7	0.294	270	59.5	35.5

Table 2 Increase in the span capacity with the stand diameter

Strands (No. – Type)	**Maximum Span Capacity (ft)**	**Girder Depth (in)**	**Span / Depth**	**Percent Increase in Span**
40 # - 0.5" Diameter	100	54	22.2	-
40 # - 0.6" Diameter	120	54	26.7	16.7
40 # - 0.7" Diameter	140	54	31.1	16.7
40 # - 0.6" Diameter	140	72	23.3	-

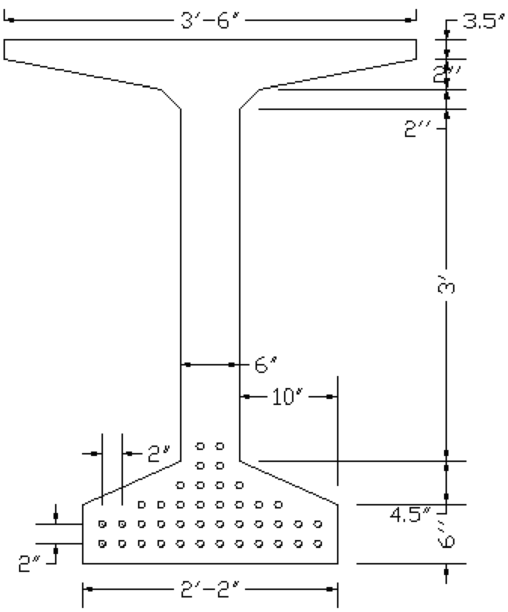


Figure 1 AASHTO Bulb-Tee 54 with 40 0.7-inch strands at 2” spacing

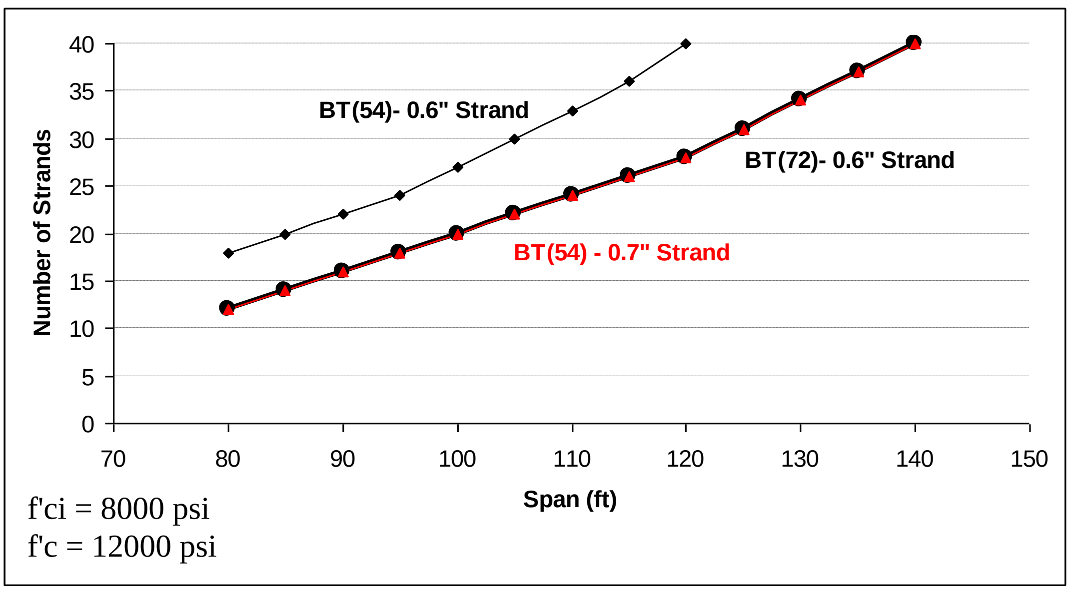


Figure 2 Different span capacities for varying diameter of strand and section

1.5.2.2 AASHTO Bulb Tees and Nebraska University Sections

NU 1350(54”), NU 1800(72”), BT 54 and BT 72 were designed for three different concrete strengths at service, such as 10 ksi, 15 ksi and 28 ksi with their strength at release to be equal to 80 percent of the strength at service. The girders were designed with a girder spacing of 8 feet. The NU sections could accommodate 58 strands in the bottom flange when compared to the BT sections which can only hold 40 numbers of strands in their bottom flange as shown in Figure 3.

In this example two sections with the same depths were considered: the NU 1350 (54”) which can accommodate 58 strands in the bottom flange having the same depth as BT 54 which can accommodate 40 strands in the bottom flange. Three different concrete strengths were considered such as 10 ksi, 15 ksi and 28 ksi.

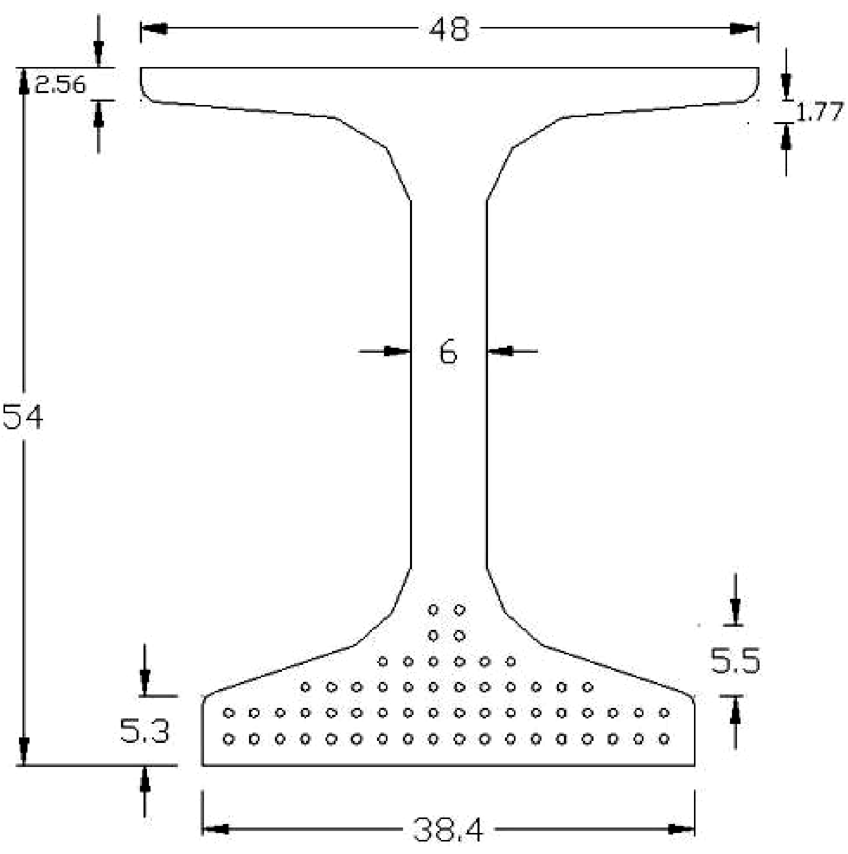


Figure 3 Nu 1350(54”) with 58 Nos. of strands at 2” spacing

When the concrete strength is 10 ksi, NU girders spanned up to 140 ft with 0.6 inch strand and 135 ft with 0.7 inch strand. BT girders spanned up to 120 ft with 0.6 inch strand and 115 ft with 0.7 inch strand as shown in Figure 4.

When the concrete strength was increased from 10 ksi to 15 ksi, the span capacities of both the sections increased. NU girders spanned up to 165 ft with 0.7 inch strand and BT girders spanned up to 140 ft with 0.7 inch strand as shown in Figure 5.

When the concrete strength was further increased to 28 ksi, the span capacities of both the sections remained the same compared to the span capacities at 15 ksi as shown in Figure 6. This shows that increasing the concrete strength beyond a certain limit had no significant effect on the span capacities for both the cross sections. This was due to the fact that the tensile stress limit in the top flange at transfer controlled.

When sections with larger depth such as NU 1800 (72") and BT 72 were considered for the analysis, it showed the same trend as the NU1350 (54") and BT 54. The span capacity of NU 1800 (72") with 0.7 inch strand increased by 8.5 % when the concrete strength was increased from 10 ksi to 15 ksi. There was a negligible increase in span when the concrete strength was increased from 15 ksi to 28 ksi.

BT 54 sections had minimal effect from varying the concrete strength. The span capacities variations were trivial for both 0.6 inch and 0.7 inch strands as shown in Figure 7, Figure 8 & Figure 9.

As shown in

Table 3, there is no increase in the span capacity as the strength of the concrete is increased for all the sections in case of the 0.6 inch diameter strands. In the case with 0.7 inch strands there is a considerable increase in the span when the concrete strength is increased from 10 ksi to 28 ksi. This shows that using 0.7 inch strands helps in talking full advantage of high strength concrete.

When BT sections were compared with NU sections, the impact of 0.7 inch strands was much higher on NU sections than BT sections. NU 1800 (72") could reach a maximum of 190 ft whereas BT 72 could reach only 160 ft. But the maximum transportable length is 160 ft. Thus, BT sections with 0.7 inch strand are more practicable than NU sections.

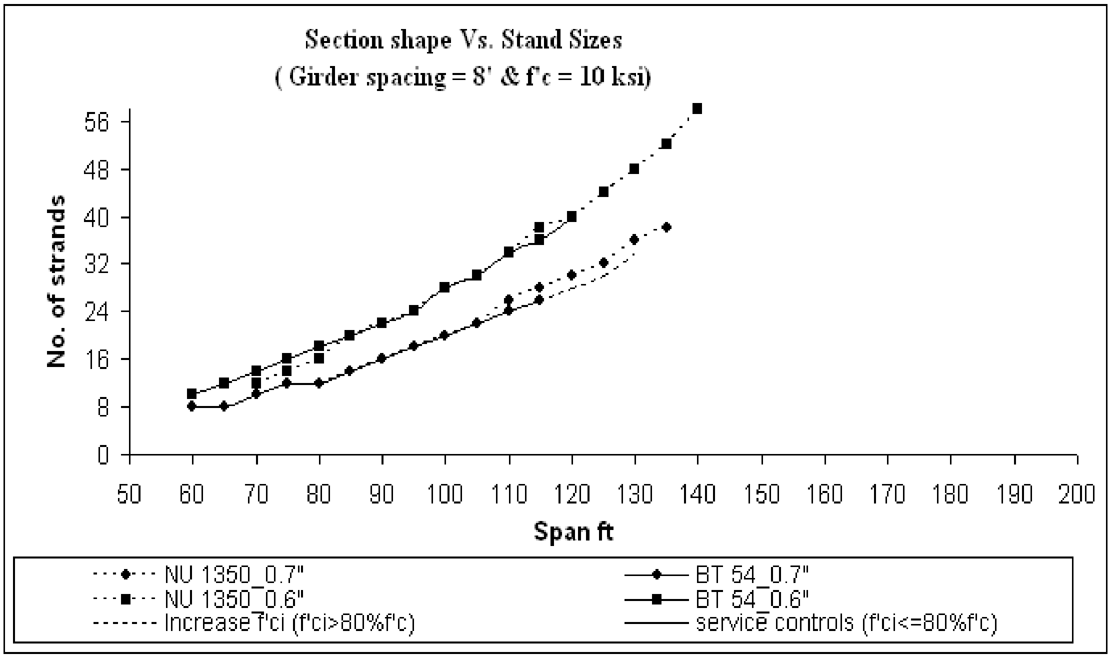


Figure 4 Section shape Vs. Strand Sizes (NU 1350 & BT 54) with f'c = 10 ksi

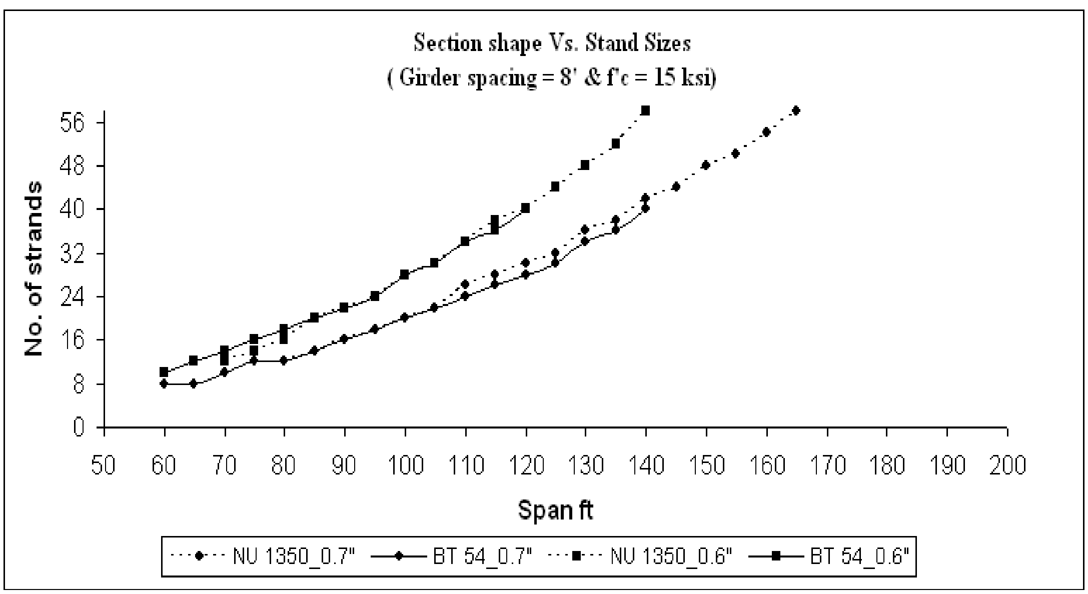


Figure 5 Section shape Vs. Strand Sizes (NU 1350 & BT 54) with f'c = 15 ksi

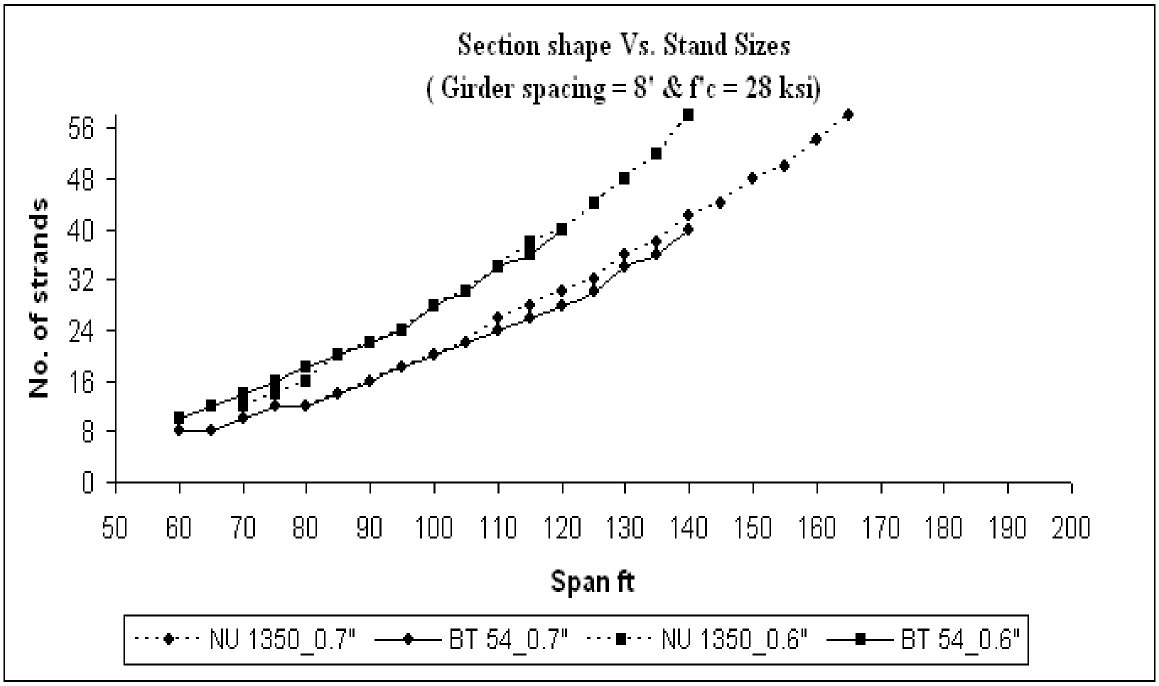


Figure 6 Section shape Vs. Strand Sizes (NU 1350 & BT 54) with f'c = 28 ksi

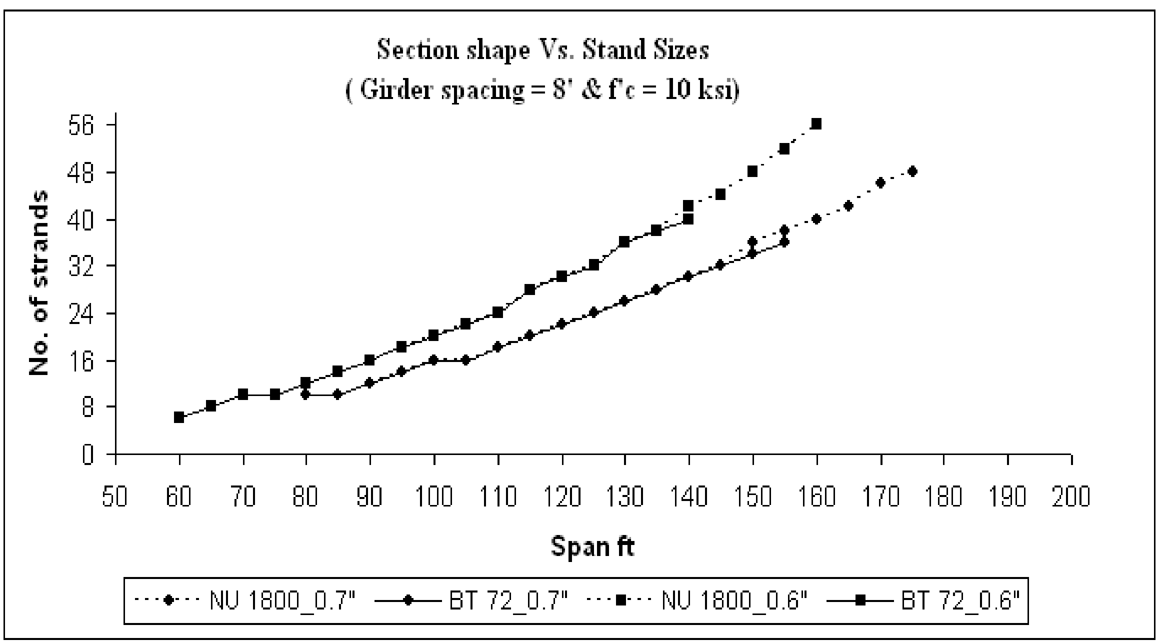


Figure 7 Section shape Vs. Strand Sizes (NU 1800 & BT 72) with f'c = 10 ksi

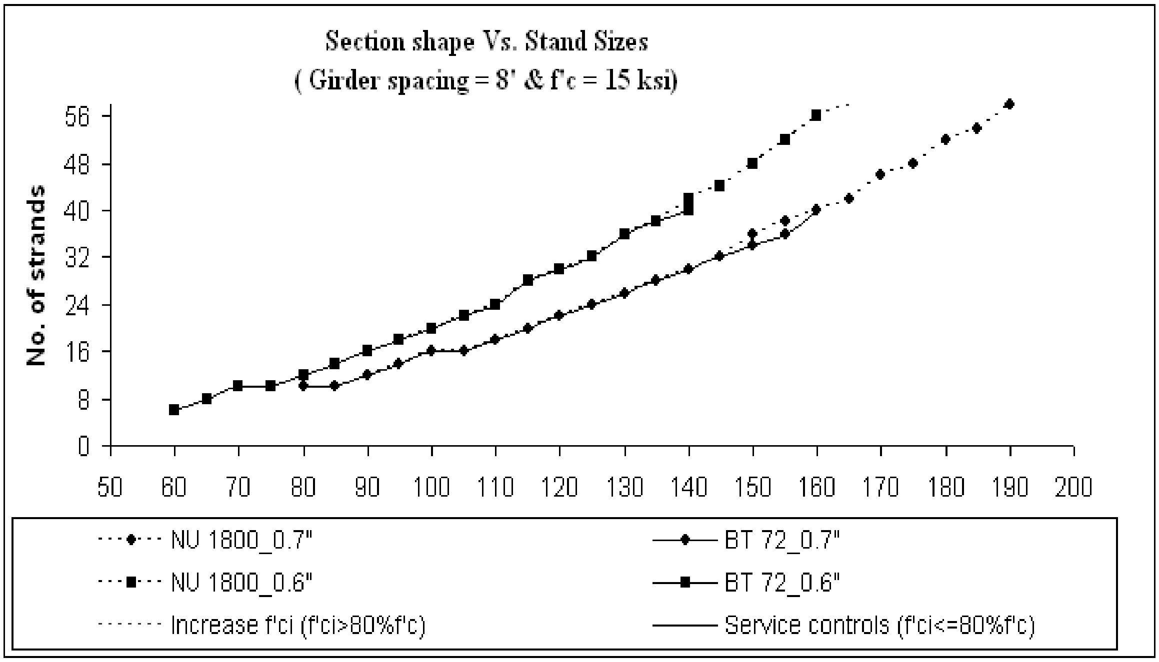


Figure 8 Section shape Vs. Strand Sizes (NU 1800 & BT 72) with f'c = 15 ksi

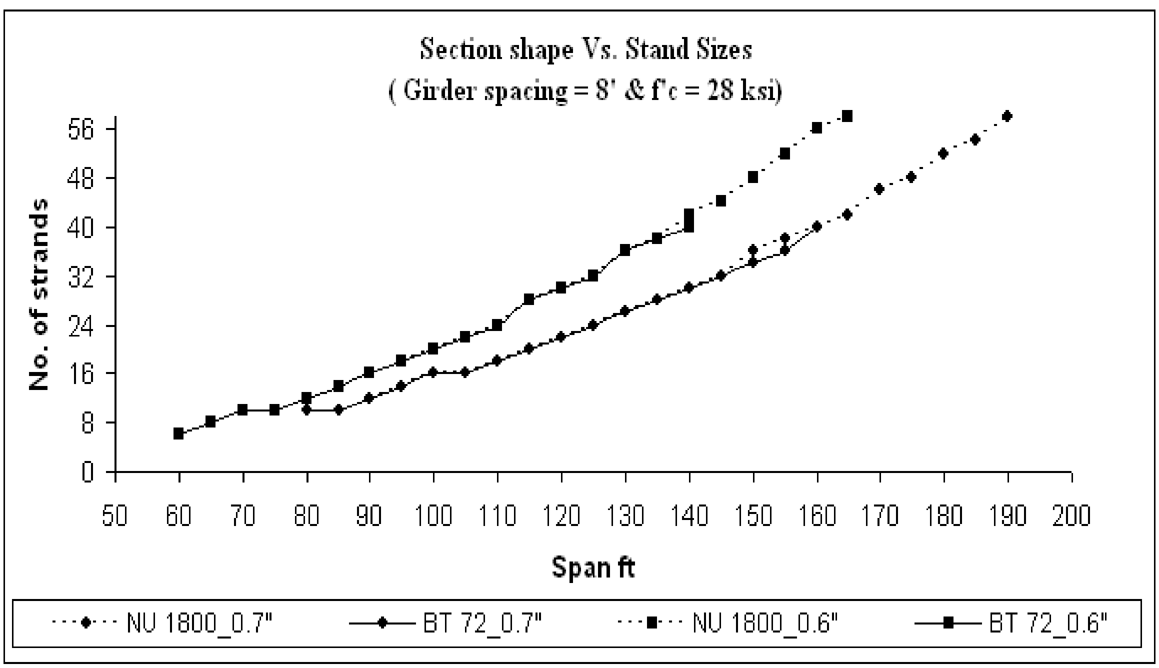


Figure 9 Section shape Vs. Strand Sizes (NU 1800 & BT 72) with f'c = 28 ksi

1.5.3 Strand size Vs. Strand Strength

1.5.3.1 AASHTO Bulb-Tees

The 0.7 inch (270 ksi) strand was compared with 0.62 inch (330 ksi) Ultra High Strength Strand (UHS). Both BT 54 and BT 72 were considered for the analysis. As shown in Figure 10, 0.7 inch strand (270 ksi) had a greater span capacity than 0.62 inch (330 Ksi) in a BT 54 section. The same trend was seen with the BT 72 section as shown in Figure 11 and Figure 12.

Table 3 shows the maximum spans with different strand sizes, sections and concrete strength.

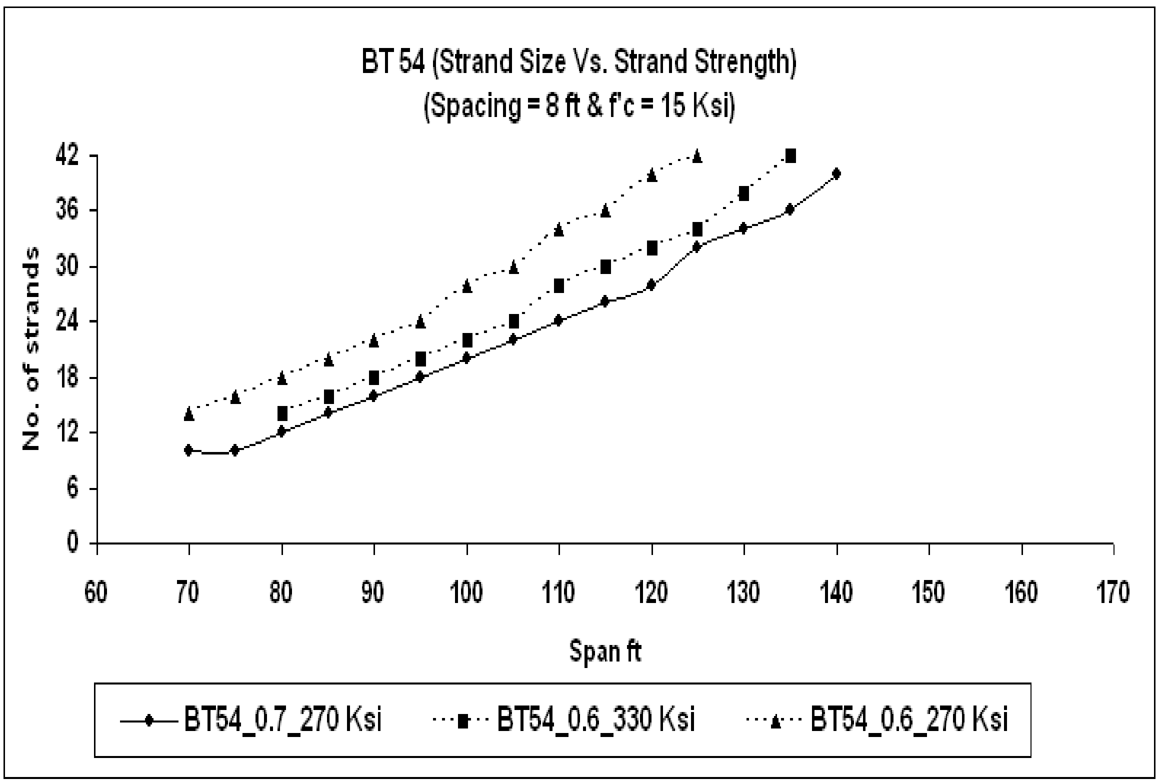


Figure 10 Strand size Vs. Strand Strength (BT 54)

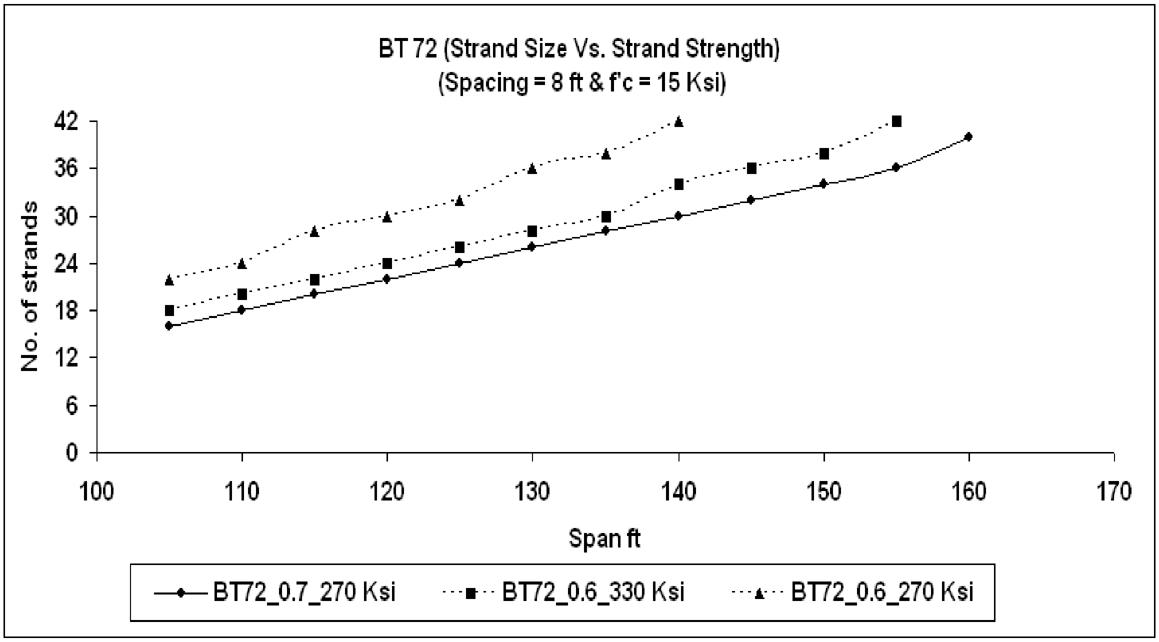


Figure 11 Strand size Vs. Strand Strength (BT 72)

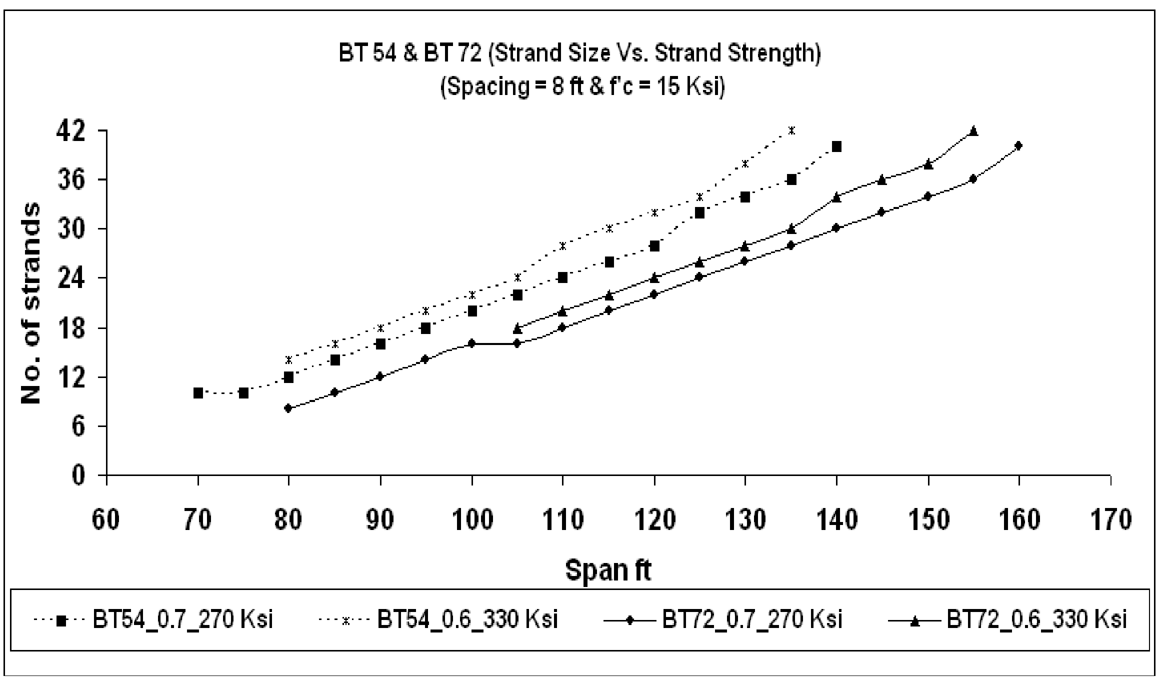


Figure 12 Strand size Vs. Strand Strength (BT 54 & BT 72)

Table 3 Maximum Spans obtained with different strand sizes, sections and concrete strength

Sections	Strand Diameter	Maximum Span, ft		
		10 ksi	**15 ksi**	**28 ksi**
BT 54	0.6”	120	120	120
	0.7”	115	140	140
NU1350	0.6”	140	140	140
	0.7”	135	165	165
BT72	0.6”	140	140	140
	0.7”	155	160	160
NU1800	0.6”	160	160	165
	0.7”	175	190	190

1.6 End Zone Reinforcement

Table 4 shows the depth of different sections and the maximum number of strands which could be accommodated in the bottom flange. The required amount of end zone reinforcement in each section is shown in Figure 13. It was designed based on AASHTO LRFD 2008 design specification. When AASHTO Type I section which has the same depth as that of NU 750 (29.5”) are compared, the amount of end zone reinforcement provided is more in NU section than the AASHTO section. Which shows that the end zone reinforcement is distributed based on the amount of prestressing force. Since the NU 750 can accommodate more strands than the AASHTO Type I section. Similarly when the grade and diameter of strands are increased for all the sections, it further increases the amount of vertical reinforcement to be provided at the end section of the girder.

The distributions of the splitting reinforcement in the end zone of the girder for the different diameters of strand are shown in Figure 14, Figure 15 & Figure 16. Three different details are compared such as AASHTO LRFD 2008 design specification, Nebraska University detailing (NU) and Tennessee department of transportation detailing. It could be seen that both AASHTO LRFD 2008 design specification and Tennessee department of transportation detailing were the same. In all these detailing the distribution of the splitting reinforcement is based on the overall height of the section.

For the distance from the very end of the girder to H/8, the NU detailing follows the same detailing as AASHTO LRFD 2008 design specification. For the distance from H/8 to H/4 the NU detailing has a closer spacing than that of AASHTO LRFD 2008 design specification. For the distance from H/4 to H/2 AASHTO LRFD 2008 design specification does not require any splitting reinforcement but NU detailing requires the same spacing provided in the zone H/8 to H/4.

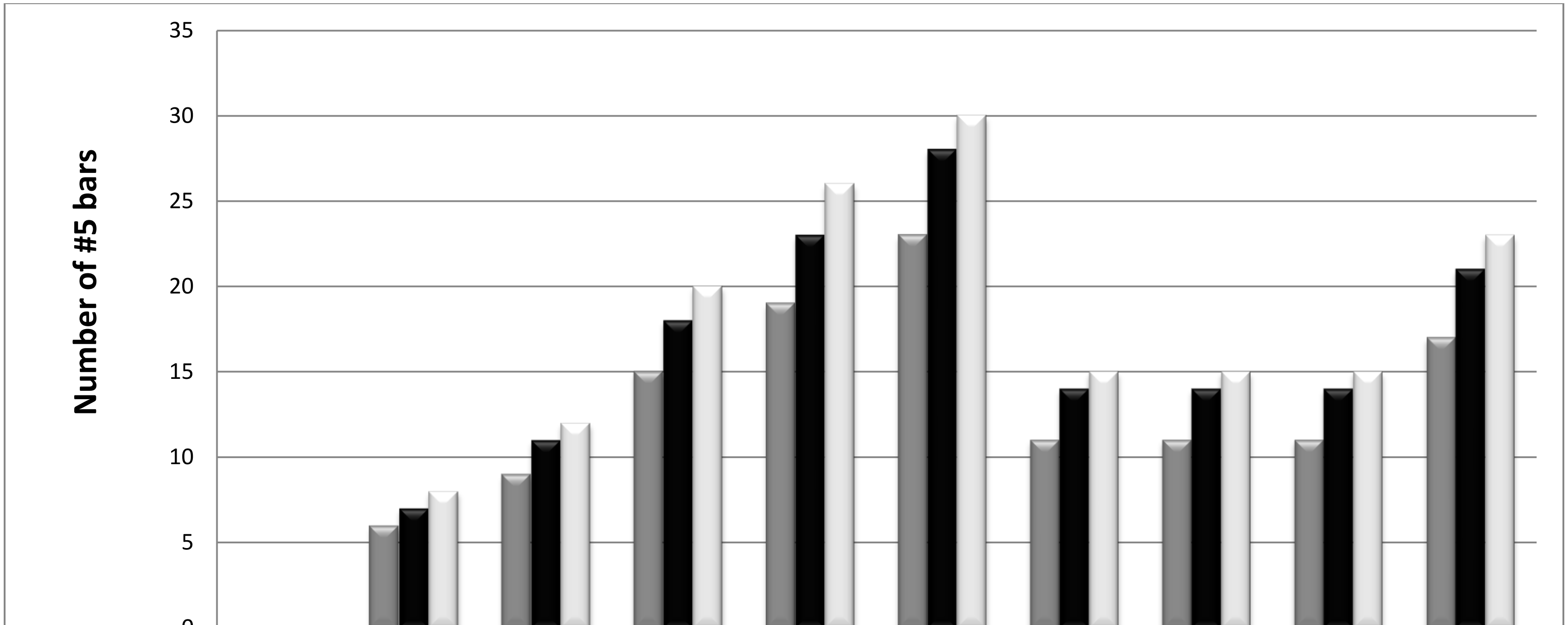
Since 0.7”, 270 ksi grade strands have the same tensioning force per strand as that of the 0.62”, 330 ksi grade strands, the splitting reinforcement details were the same in all the three details.

1.7 Conclusion

Using 0.7 inch diameter strands improves span length in all sections; for states such as Tennessee using Bulb-Tee sections, 0.7 inch strand can efficiently utilize high strength concrete to increase the span length up to transportable limits. Using 0.7 inch strands in the structurally efficient NU cross section can increase spans to lengths that are yet to be able to be transported.

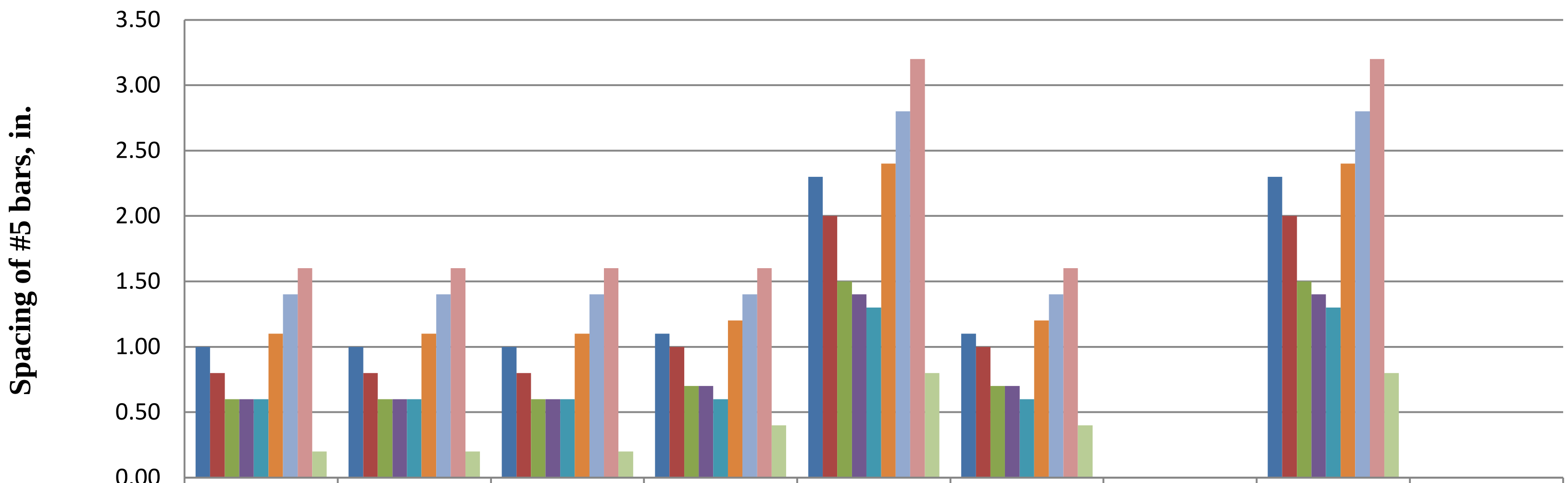
Table 4 Sections with their depth and No. of strands

Section	Depth, inches	No. of strands in the bottom flange
AASHTO Type I	28.0	26
AASHTO Type II	36.0	34
AASHTO Type III	45.0	52
AASHTO Type IV	54.0	66
AASHTO Type V	63.0	78
AASHTO BT-54	54.0	40
AASHTO BT-63	63.0	44
AASHTO BT-72	72.0	48
NU 750	29.5	58



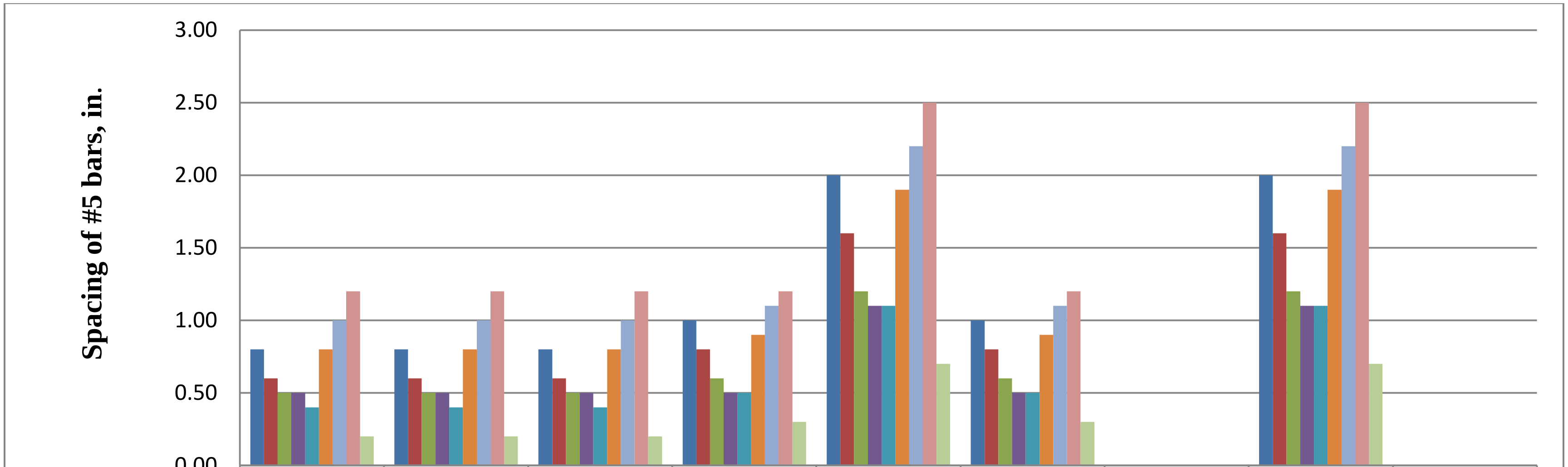
No. of strand	20	30	50	66	78	38	38	38	58
Depth H, in.	28	36	45	54	63	54	63	72	29.5
	AASHTO Type I	AASHTO Type II	AASHTO Type III	AASHTO Type IV	AASHTO Type V	AASHTO BT-54	AASHTO BT-63	AASHTO BT-72	NU 750
0.6" - 270 ksi	6	9	15	19	23	11	11	11	17
0.62"- 330 ksi	7	11	18	23	28	14	14	14	21
0.7" - 270 ksi	8	12	20	26	30	15	15	15	23

Figure 13 End zone reinforcement for sections with different strand



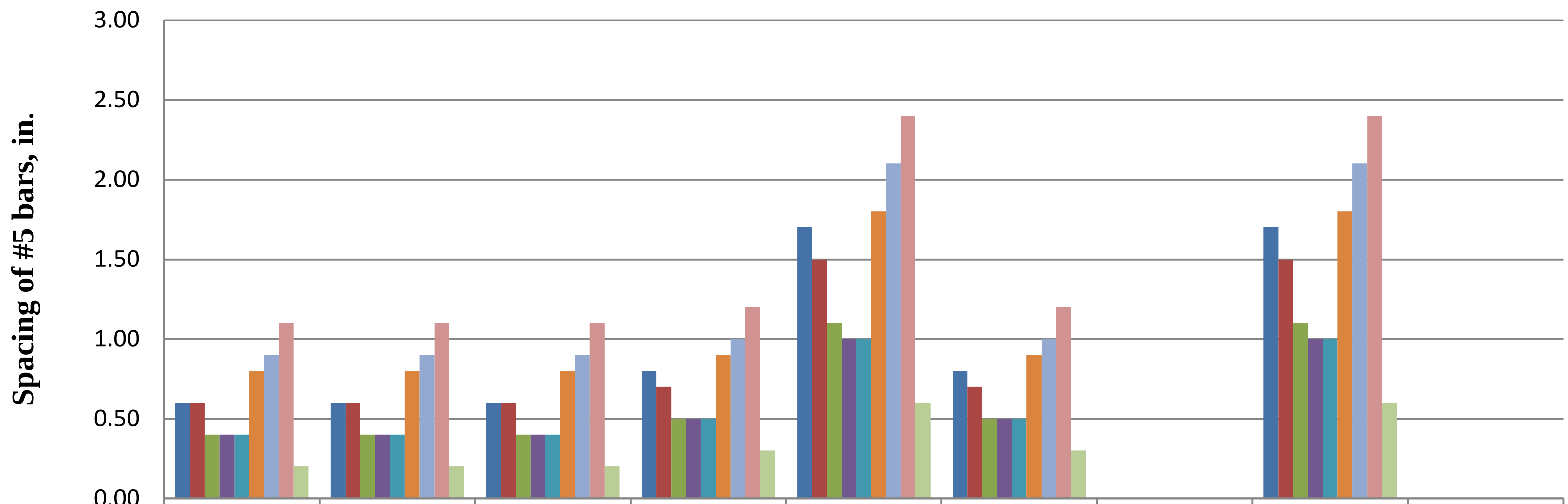
	AASHTO LRFD 2008(0 to H/8)	NU (0 to H/8)	TDOT (0 to H/8)	AASHTO LRFD 2008 (H/8 to H/4)	NU (H/8 to H/4)	TDOT (H/8 to H/4)	AASHTO LRFD 2008 (H/4 to H/2)	NU (H/4 to H/2)	TDOT (H/4 to H/2)
AASHTO Type I	1.00	1.00	1.00	1.10	2.30	1.10	0	2.30	0
AASHTO Type II	0.80	0.80	0.80	1.00	2.00	1.00	0	2.00	0
AASHTO Type III	0.60	0.60	0.60	0.70	1.50	0.70	0	1.50	0
AASHTO Type IV	0.60	0.60	0.60	0.70	1.40	0.70	0	1.40	0
AASHTO Type V	0.60	0.60	0.60	0.60	1.30	0.60	0	1.30	0
AASHTO BT-54	1.10	1.10	1.10	1.20	2.40	1.20	0	2.40	0
AASHTO BT-63	1.40	1.40	1.40	1.40	2.80	1.40	0	2.80	0
AASHTO BT-72	1.60	1.60	1.60	1.60	3.20	1.60	0	3.20	0
NU 750	0.20	0.20	0.20	0.40	0.80	0.40	0	0.80	0

Figure 14 End zone reinforcement for different sections with 0.6" strand



	AASHTO LRFD 2008(0 to H/8)	NU (0 to H/8)	TDOT (0 to H/8)	AASHTO LRFD 2008 (H/8 to H/4)	NU (H/8 to H/4)	TDOT (H/8 to H/4)	AASHTO LRFD 2008 (H/4 to H/2)	NU (H/4 to H/2)	TDOT (H/4 to H/2)
AASHTO Type I	0.80	0.80	0.80	1.00	2.00	1.00	0	2.00	0
AASHTO Type II	0.60	0.60	0.60	0.80	1.60	0.80	0	1.60	0
AASHTO Type III	0.50	0.50	0.50	0.60	1.20	0.60	0	1.20	0
AASHTO Type IV	0.50	0.50	0.50	0.50	1.10	0.50	0	1.10	0
AASHTO Type V	0.40	0.40	0.40	0.50	1.10	0.50	0	1.10	0
AASHTO BT-54	0.80	0.80	0.80	0.90	1.90	0.90	0	1.90	0
AASHTO BT-63	1.00	1.00	1.00	1.10	2.20	1.10	0	2.20	0
AASHTO BT-72	1.20	1.20	1.20	1.20	2.50	1.20	0	2.50	0
NU 750	0.20	0.20	0.20	0.30	0.70	0.30	0	0.70	0

Figure 15 End zone reinforcement for different sections with 0.62" strand



	AASHTO LRFD 2008(0 to H/8)	NU (0 to H/8)	TDOT (0 to H/8)	AASHTO LRFD 2008 (H/8 to H/4)	NU (H/8 to H/4)	TDOT (H/8 to H/4)	AASHTO LRFD 2008 (H/4 to H/2)	NU (H/4 to H/2)	TDOT (H/4 to H/2)
AASHTO Type I	0.60	0.60	0.60	0.80	1.70	0.80	0	1.70	0
AASHTO Type II	0.60	0.60	0.60	0.70	1.50	0.70	0	1.50	0
AASHTO Type III	0.40	0.40	0.40	0.50	1.10	0.50	0	1.10	0
AASHTO Type IV	0.40	0.40	0.40	0.50	1.00	0.50	0	1.00	0
AASHTO Type V	0.40	0.40	0.40	0.50	1.00	0.50	0	1.00	0
AASHTO BT-54	0.80	0.80	0.80	0.90	1.80	0.90	0	1.80	0
AASHTO BT-63	0.90	0.90	0.90	1.00	2.10	1.00	0	2.10	0
AASHTO BT-72	1.10	1.10	1.10	1.20	2.40	1.20	0	2.40	0
NU 750	0.20	0.20	0.20	0.30	0.60	0.30	0	0.60	0

Figure 16 End zone reinforcement for different sections with 0.7” strand

CHAPTER 2

BACKGROUND AND LITERATURE REVIEW

2.1 Definitions

2.1.1 Transfer Length

The distance from the end of a member over which the effective stress f_{se} is fully transferred from the strand to the concrete is called transfer length.

2.1.2 Flexural Bond length

Flexural bond length is the additional embedment length required to develop strand stress due to external load from the effective prestress f_{se} to the stress fps at the nominal flexural strength of the member.

2.1.3 Development Length

Development length is the length that is required to develop the strand stress, fps at the ultimate strength of the member under the application of the external loads. This length is equal to the sum of the transfer and flexural bond lengths.

2.1.4 Embedment Length

Embedment length is the length that starts from the beginning of the bond and extends to the location of the critical section. The critical section is located at the section where the strand stress is maximum which occurs at the location of the maximum moment. To prevent bond failure, the embedment length should always be greater than the development length.

The transfer length, flexural bond length and the development length are shown in Figure 17.

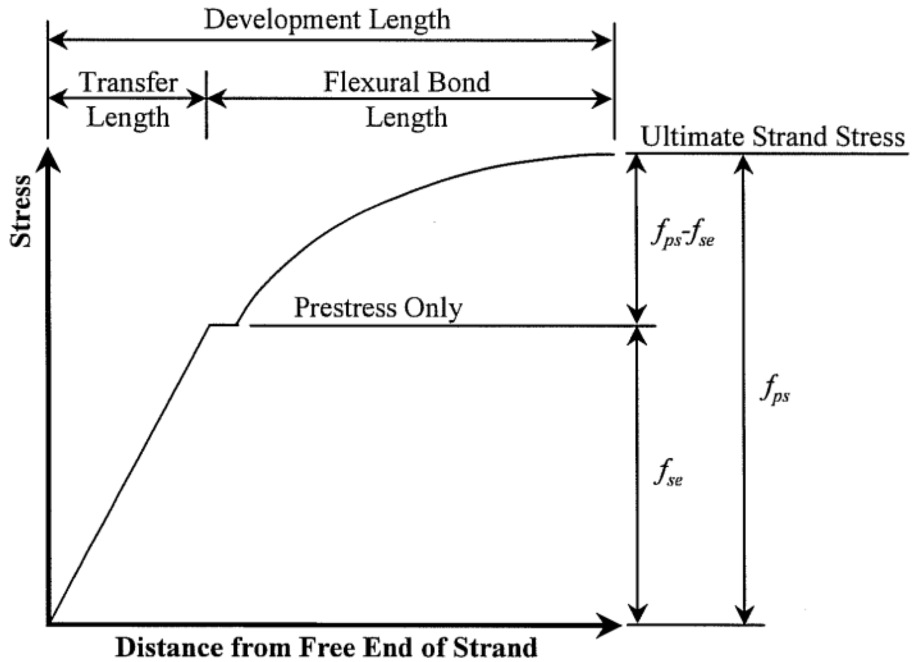


Figure 17 Variation of steel stress

There are various factors affecting the transfer and development length. The lists of factors which are considered by different researchers are

- Diameter of the strand
- Surface condition of strand
- Compressive strength of concrete at the time of release
- Method of release
- Amount of confining reinforcement
- Level of prestressing
- Strand spacing
- Time-dependent effects
- Concrete around the strand
- Type of loading
- Type of prestressing strand

2.2 Strand – Concrete bond

2.2.1 Elements of bond

There are three distinct and different elements of bond which are,

1. Hoyer Effect
2. Mechanical Interlocking
3. Adhesion

2.2.1.1 Hoyer Effect

The Hoyer effect is named after E. Hoyer who first investigated the mechanism, a consequence of the tensioning process of the strand. The Hoyer effect is the tendency of the prestressing strand to decrease in diameter by Poisson's ratio as it is elongated in pretension. Then the concrete is cast around the strand. Once the concrete reaches its initial strength the strand is cut and the strand is unstressed at the extreme end of the member. Due to this the strand tends to expand laterally to regain its original position. Since it is enclosed in the concrete this lateral expansion is resisted, thus creating a normal force in the boundary between the steel and concrete. Thus, the stress varies from zero at the very end of the strand to a constant stress at a distance along the strand. This constant stress is known as the effective prestress f_{se}. The varying degree of stress within the strand causes a variation in the strand diameter; the strand diameter at the member end is greater than the diameter of the strand further in the member. The variation of strand diameter creates a wedge effect. The concrete acts against this wedging effect, transferring the stress from the strand to the concrete. This mechanism is shown in Figure 18.

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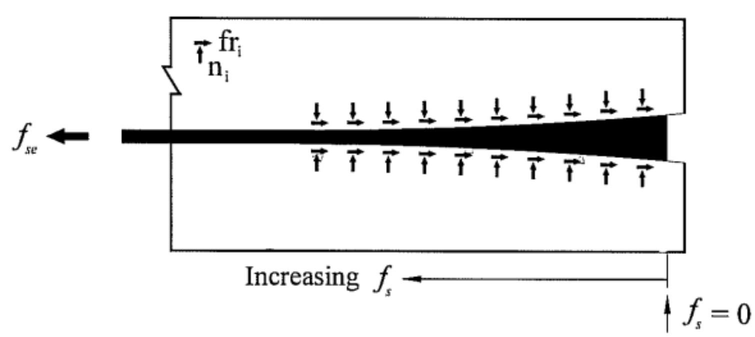


Figure 18 Hoyer's Effect (Russell and Burns, 1996)

2.2.1.2 Mechanical Interlocking

Mechanical interlock depends on the shape of the strand. When concrete hardens around the strand it takes the shape of the strand. This pattern provides a high resistance, thus increasing the amount of stress transferred to the concrete. This resistance is called mechanical interlocking. The strand is prevented from slipping as long as the strand does not twist. This effect is illustrated in Figure 19.

2.2.1.3 Adhesion

Adhesion is one of the mechanisms which helps in the transfer of prestressing stress from the strand to the concrete. This is a chemical bond which occurs between the strand and the surrounding concrete. This mechanism as shown in Figure 20, contributes the least in developing bond stress in the concrete compared with Hoyer effect and mechanical interlocking. Once slip occurs, this chemical bond is lost.

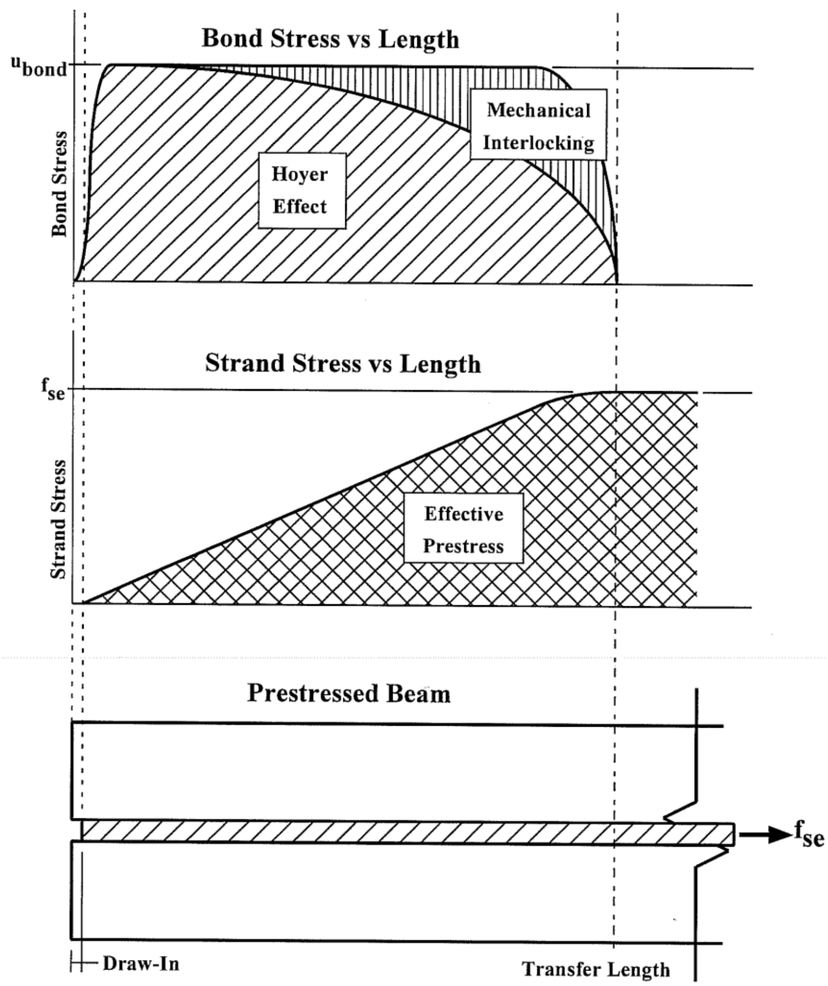


Figure 19 Mechanical Interlocking

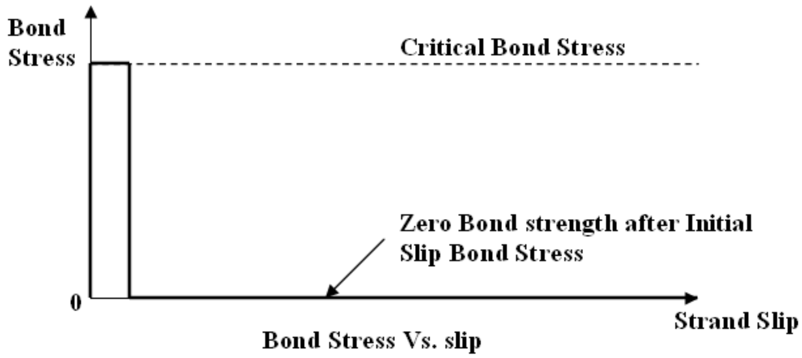


Figure 20 Adhesion: Rigid – Brittle Behavior (Russell and Burns, 1996)

2.3 Strand spacing

Burdette and Deatherage (1994)[5] conducted a research work initiated by the FHWA memorandum restricting the use of certain sizes of strand in prestressed concrete, girders and criteria were established for minimum strand spacing. In this study 20 full-scale AASHTO Type I beams with various strand diameters were tested to failure. The transfer and development lengths of four different diameter of strand such as 0.5 in., 0.5 in. (special), 9/16 in. and 0.6 in. were determined. Also, minimum strand spacing was investigated for 0.5 in. diameter strands. In addition to prestressing strands, mild steel shear and confinement reinforcement were placed in the girder ends. AASHTO Type I beams were used with a span of 31 ft. The initial prestress in all strands of the test beams was designed to be 203 Ksi.

The test data illustrate that the average transfer lengths for the 0.5 in., 0.5 in. (special), and 9/16 in. strands were approximately proportional to the strand diameter, but this relationship does not hold for 0.6 in. strands. The 0.6 inch diameter strand had shorter transfer length when compared to the other three diameter strands; this may be due to the increase in mechanical bond between the strand and concrete.

Based on the test results it was concluded that the use of 0.6 in. diameter strand should be accepted as strand practice and a center-to-center spacing of 1.75 in. should be allowed for 0.5 in. diameter strands.

The equation for the Transfer length for 0.5 in., 0.5 in. (special), and 9/16 in. strand diameter was given as

$$L_t = \left(f_{si} / 3\right) d_b \tag{2.1}$$

The equation for development length for all diameters of strands were given as

$$L_d = (f_{si}/3)d_b + 1.50(f_{ps} - f_{se})d_b \qquad (2.2)$$

Where,

f_{si} = Stress in prestressed reinforcement at transfer (ksi)
f_{ps} = Stress in prestressed reinforcement at nominal strength (ksi)
f_{se} = Effective stress in prestressed steel after losses (ksi)
d_b = Nominal diameter of the prestressed strand (in.)

It was recommended in the paper that future work should be done to develop an expression for transfer length of 0.6 in. diameter strand. The equation recommended for other sizes of strand was clearly conservative and may be used.

The research work conducted by Thomas E. Cousins, J. Michael Stallings, and Michael B. Simmons., at Auburn University, (1994)[6] have concluded after testing twelve specimens of T-shaped cross sections, six with 1.75-in spacing of 0.5-inch diameter strands and six with 2-in spacing of 0.5-in diameter strands, that decreasing the strand spacing in the pretensioned prestressed concrete members from 2 to 1.75 inch has no significant effect on the transfer length, development length, or nominal moment capacity. It does not result in splitting of members at transfer of prestressing force.

Two concrete mixes, a normal strength mix design (5000 psi at transfer and 6000 psi at service) and a high strength mix design (6000 psi at transfer and from 10000 to 12000 psi at service) were used in this study, so that the effect of concrete strength on strand spacing could be investigated. Thus it was found that an increase in the concrete strength from normal strength to high strength significantly reduced the transfer and the development lengths.

In the study conducted by Russell and Burns (1996)[7] at The University of Texas at Austin, AASHTO type girders and rectangular prisms were fabricated and tested for both transfer and development length. For all specimens with 0.5 inch diameter strands, a spacing of 2 inch was used which is the ACI minimum (four times the strand diameter) and for 0.6 inch diameter strands a spacing of 2 inch and 2.25 inches which is less than the ACI minimum spacing of 2.4 inches. The transfer lengths for both 0.5 inch and 0.6 inch diameter strands were measured.

AASHTO and ACI suggest the value of the transfer length and also recommend the assumption that the effective prestressing force varies from zero at the free end of the strand to the maximum over the transfer length. These suggestions are provided so that the designer can calculate the concrete contribution to the shear strength. One problem with this approach is that shear cracking has led to anchorage failure of the strands. Flexural tests demonstrate that when anchorage failure occurs, not only the concrete contribution in shear is lost, but the tension required from prestressing strand is also lost. Thus, transfer length is very important in accurately predicting strand development failures. In the ultimate limit state for highway girders, both the flexural capacity and shear capacity of pretensioned beams are affected by the transfer length.

Their testing consistently demonstrated that if a crack propagated through or near the transfer zone, then the crack can be expected to generate general bond slip. Because either flexural cracking or shear cracking can occur in the transfer zone of a strand, it is important to predict and prevent both types of cracks within the transfer zone. Therefore to prevent cracking in the transfer zone, a reliable transfer length is important to accurately predict cracking loads and location of cracking.

In this paper transfer lengths were measured on a wide variety of research variables such as

- Number of strands (1, 3, 4, 5, and 8)
- Size of strand (0.5 inch and 0.6 inch)
- Confining reinforcement (with or without)
- Size and shape of the cross section

Tests were conducted on 44 specimens to determine the transfer length. Of these specimens, 32 were constructed with concentric pre-prestressing in rectangular transfer length prisms and 12 specimens were built as scale model AASHTO type beams with four, five, or eight strands.

The test results in this paper indicate that the transfer bond characteristics for 0.6 inch strands are very similar to the bond behavior for 0.5 inch strands. The comparison of the transfer length data by strand size shows that 0.6 in. strands require longer transfer length than 0.5 in. strands. Furthermore, these data indicate that the relationship between transfer length may not vary linearly with strand diameter, d_b, but these data indicate that the expression, $L_t = K d_b^{\alpha}$ ($\alpha = 1.68$), would be more accurate. However the authors do not suggest that these data alone provide sufficient evidence to recommend adoption of an exponential equation. Therefore, a rational and safe expression for transfer length is determined:

$$L_t = \left(f_{si} d_b \right) 2 \qquad (2.3)$$

The transfer length data collected from different cross section size illustrate that larger specimens with multiple strand tend to possess significantly shorter transfer length.

In order to investigate the possibility of using 0.6 inch strand with 2 inch spacing (ACI minimum spacing for 0.5 inch diameter strands), tests were performed by Cousins, Stallings, and Simmons (1994)[6] to determine what effects strand spacing had on the transfer length. Tests showed that strands with 2.25 inch (40.9 inches) spacing had a longer transfer length than that of strands with 2 inch (44.2 inches) spacing. But the need for wider spacing of 0.6 inch strand was not demonstrated. Thus six specimens were fabricated, 5 inch wide and 13 inch deep with 5 strands (2 inch spacing) to check for splitting caused due to pretensioning release, because its larger size would cause larger bursting stresses. It was found that no signs of splitting were detected. It was recommended that the 0.6 inch diameter strands had a similar profile of transfer length and strain profile as that of 0.5 inch diameter strands. It could be used with the same spacing of 2 inches as 0.5 diameter strands.

The test results in this paper demonstrate that confining reinforcement did not contribute significantly to prestress transfer; specimens containing confinement reinforcement possessed slightly longer transfer length than specimens without the confinement reinforcement. These results indicate that confining, or transverse, reinforcement is not activated until splitting cracks occur at prestress transfer.

2.4 Transfer Length Research

There have been many studies conducted on the transfer length of pretensioned members. Attempts have been made by several researchers to revise the transfer and development length equations. The Table 5 shows the proposed equations by different researchers.

Table 5 Transfer Length of Prestressing Strands – Prediction

Source		Transfer Length Equations
Martin and Scott	(1976)	$L_t = 80 d_b$
Zia & Mostafa	(1977)	$L_t = 1.5 (f_{si} / f'_{ci}) d_b - 4.6$
Cousins, Johnston and Zia	(1990)	$L_t = \frac{U'_t \sqrt{f'_{ci}}}{2B} + \frac{f_{se}.A_{strand}}{\pi d_b U'_t \sqrt{f'_{ci}}}$
Russell & Burns	(1993)	$L_t = (f_{si} d_b) / 2$
Mitchell, Cook, Khan & Tham	(1993)	$L_t = (f_{si} d_b / 3) \sqrt{\frac{3}{f'_{ci}}}$
Buckner	(1994)	$L_t = (250 f_{si} / E_c) d_b \approx \frac{f_{si}}{3}.d_b$
Deatherage, Burdette & Chew	(1994)	$L_t = (f_{si} / 3) d_b$
Lane	(1998)	$L_t = (4 f_{pt} / f'_c) d_b - 5$

2.5 Concrete Strength

To investigate the maximum usable concrete strength in the application of bridge I-girders, Ma (2000)[15] performed an analytical study. In his study, the following assumptions were made:

- Design was based on a typical interior girder which was simply supported.
- Cross sectional shapes studied included AASHTO-PCI BTs and NUs.
- Girder spacing was 8 ft and 16 ft.
- Deck thickness was 7.5 in. for 8 ft girder spacing and 10 in. for 16 ft girder spacing.
- Concrete deck was cast-in-place and acted compositely with the girder.
- Concrete compressive strength of the deck was constant and equal to 4000 psi at 28 days.
- Live load consisted of HS-25 loading. Superimposed composite dead load was 40 psf.
- Prestress losses were constant and equal to 10% of initial prestress at release and 25% at service.
- The following prestressing strand diameters were used: 0.6-in. diameter Grade 270 ksi at 2-in. spacing and 0.7-in. diameter Grade 270 ksi at 2-in. spacing at midspan section.

Take the example of a simple span with NU1100 I-girders. The girder spacing was 8 ft. The concrete strength of cast-in-place deck $f'_{c,\,deck}$ = 4000 psi with a depth of 7.5 in. Table 6 shows the impact of the 0.7-inch strand and girder concrete strength on the maximum span capacity of bridge I-girders. When 0.7-inch strands at 2-inch spacing are used, the span capacity can be increased by **<u>178%</u>**. For the NU section shape, the bottom flange can accommodate a total of 54 strands, compared with 36 strands in the bottom flange of AASHTO-PCI BT shapes. When 0.7-inch strands are used, however, the disadvantage of accommodating less number of strands in BT shapes can be avoided because the maximum shipping length of I-girders has an upper limit.

Table 6 Impact of 0.7" strands and girder concrete strength

Strands (No. – Type)	Girder Concrete Strength (ksi)	Maximum Span Capacity (ft)	Span/Depth
26 – 0.6" strands	6	85	20
36 – 0.6" strands	8	100	24
54 – 0.7" strands	16	150	36

2.6 End confinement reinforcement research

Marshall and Mattock[8] in the early 1960s investigated the stresses which occur in the ends of pretensioned prestressed concrete girders at the time of transfer of prestress, which in turn result in the formation of horizontal cracks. A semi-empirical equation was recommended based on testing 14 specimens. The variables considered in the tests were size and the location of the prestressing strands and the magnitude of the prestressing force. The specimens had two basic cross sections (22.5 in and 25 in depth) and two sizes of vertical stirrup reinforcement. The area of reinforcement required for the splitting force, A_s is given by the following equation:

$$A_t = \frac{S}{(f_s/2)} = 0.021\frac{T}{f_s}\frac{h}{l_t} \tag{2.4}$$

Where,

T = Effective prestress force
f_s = Allowable stress in the stirrups
h = Total girder depth
l_t = Transfer length

A transfer length of 50 times the strand diameter was recommended unless experimental evidence dictated otherwise. This equation was justified experimentally only for values of $(h/l_t) \leq 2$, and for $(h/l_t) > 2$ this equation becomes conservative and the degree of conservatism increases with the increase in the (h/l_t) value.

It was also recommended that the amount of stirrup reinforcement be calculated using the equation should be distributed uniformly over a length equal to 1/5 of the girder depth, measured from the end face of the girder. For most efficient crack control the first stirrup is placed as close to the end face of the girder as possible.

2.7 End Zone splitting Reinforcement

Tuan, C.Y., Yehia, S.A., Jogpitaksseel, N., and Tadros, M.K. (2004)[12] conducted a study to evaluate the applicability of various theories and methods for the design of end zone reinforcement. The analytical methods reviewed in this paper include finite element analysis, strut and tie modeling, and Gergely–Sozen equivalent beam method. Experiments were conducted to correlate between the various analytical and the experimental results. Based on the theoretical behavior and the experimental observation a general semi-empirical design was proposed.

In the experimental program the authors considered two phases. In the first phase six NU I-girders and six inverted-tee I-girders were designed based on AASHTO LRFD specification, and in the second phase new end zone reinforcement was proposed based on the observations made with the data in the first phase. For example, the reinforcement located within the end h/8 of the member experienced significant stress. In the second phase four members of NU1600, eight numbers of NU1100 and two numbers of Inverted Tee (IT) 400 were designed based on the new end zone reinforcement.

New end zone reinforcement for the splitting force was proposed as follows; provide reinforcement equal to 4 percent of the total prestressing force and a uniform stress of 20 ksi. To allow for this high average stress to be used, at least 50% of that reinforcement should be placed at a distance h/8 from the end. The remainder should be placed between h/8 to h/2 from the end. Beyond h/2, splitting reinforcement should not be needed, and shear reinforcement, if needed, should be used.

2.7.1 Detailing proposed by University of Nebraska-Lincoln

The full-scale testing conducted at the University of Nebraska-Lincoln provided the following end zone reinforcement as shown in the Table 7 as a part of their ongoing NCHRP project (Evaluation and Repair Procedure for Precast/prestressed Concrete Girders with Longitudinal Cracking in the Web). In the AASHTO Type III girders were used in their testing program, neither girder appeared to have experienced visible end zone cracking. The research team suggested that the lack of end zone cracking was due to the limited amount of prestressing (Thirty 0.5 inch diameter strands stressed to 33.8 kips), the presence of end zone reinforcement, and the size and shape of the girders.

The proposed procedure states that the end zone reinforcement should be designed to resist 4 percent of the prestressing force at release with a uniform stress of 20 ksi., and 50 percent of this reinforcement should be placed h/8 (one-eighth of the depth of the girder) from the end of the beam. The remainder should be placed between h/8 and h/2 from the end.

According to the proposed procedure, the remainder of the end reinforcement that is provided between h/8 and h/2 from the end is not in addition to the vertical shear reinforcement. In this particular distance, i.e. between h/8 and h/2 from the end, the design engineer should compare the vertical shear reinforcement that is required through this distance with the end zone reinforcement and use whichever is greater.

Table 7 Detailing proposed by University of Nebraska-Lincoln

Tennessee Specimens (AASHTO Type III)	End Zone Reinforcement Detailing
TN1(left end)	AASHTO LRFD 2007 Splitting Force = 4% of Prestressing Force Allowable steel stress = 20 ksi 3 pairs of # 6 bars at 3" spacing placed within h/4 distance from the end of the girder
TN2(Left end)	Splitting Force = 4% of Prestressing Force Allowable steel stress = 20 ksi 3 horizontal #5 bars projected 5' into the web. 6 pairs of #6 bars spaced at 3in c/c starting at 3" from the end of the girder.
TN1 & TN2 (Right end)	Splitting Force = 4% of Prestressing Force Allowable steel stress = 20 ksi 2% of the splitting force placed within h/8 of the distance from the end of the girder Remaining 2% placed from h/8 to 3h/8 of the distance from the girder

2.8 Draping/Shielding of strands at the end of the girder

The research Noppakunwijai, P., Ma, Z., Yehia, S.A., Jogpitaksseel, N., and Tadros, M.K. (2002)[16] showed that the shear capacity of a pretensioned concrete simple span I-girder could be significantly increased by extending and bending strands that already exist in the bottom flange into the end diaphragms. In addition it could be a cost effective method of controlling creep and shrinkage effects in bridges. In this paper, the pullout capacity of 0.5 in. and 0.6 in. diameter strands were evaluated, and the authors gave recommendations for determining the required number and length of strands to be bent and embedded into the diaphragms. The number of strands and the bent length of the strand are determined by the equations developed based on the test results.

$$f_{ps} = 0.017 f_{pu} L_v / d_b \leq 0.8 f_{pu} \tag{2.5}$$

f_{ps} = developed strand stress, ksi
L_v = Vertical embedment length of non-prestressing bent strand, in.
f_{pu} = specified tensile strength of prestressing tendons
d_b = Nominal diameter of strand, in.

$$T \geq \left[\frac{V_u}{\Phi} - 0.5V_s - V_p \right] \cot\theta \tag{2.6}$$

T = Tension force in longitudinal reinforcement, kips
Φ = Capacity reduction factor for shear
V_u = Factored shear force at critical section, kips
V_s = Shear resistance provided by shear reinforcement at given section, kips
V_u = Components of effective prestressing force in the direction of applied shear, kips
ϴ = Angle of inclination of diagonal compressive stress.

$$T = nA_{ps1} f_{ps} \tag{2.7}$$

n = Number of bent strands
A_{ps1} = Cross-sectional area of one strand, sq. in.
f_{ps} = Development stress in strands, ksi

A minimum embedment length of 16 in. was recommended for crack control due to time dependent restraint positive moments at piers.

2.9 Current code specifications

2.9.1 Spacing of strands

The requirements for the spacing of strands and transfer and development length in a prestressed girder by both ACI 318-08 and AASHTO LRFD 2007 are given in the Table 8 and Table 9.

Table 8 Strand Spacing

Codes	Strand Spacing, in
ACI 318-2008	*Not less than 4db, Minimum strand spacing* *2 inch for 0.5 inch* *2.4 inch for 0.6 inch*
AASHTO LRFD 2007	*Not less than 1.33 times the maximum size of the aggregate nor less than the center-to-center distance specified as follows* *1.75 inch for 0.5 inch* *2 inch for 0.6 inch*

2.9.2 Transfer and Development Length

Table 9 Transfer and Development Length

	Transfer length Equation	**Development Length Equation**
ACI 318-2008	$L_t = (f_{se}/3000)d_b$	$L_d = (f_{se}/3000)d_b + ((f_{ps} - f_{se})/1000)d_b$
AASHTO LRFD 2008	$L_t = 60d_b$	$L_d \geq k(f_{ps} - (2/3)f_{ps})d_b$

2.9.3 Pretensioned anchorage zone reinforcement

The provision in article 9.22.1 of the AASHTO Standard specification appears to be a simplified form of the recommendations of Marshall and Mattock. The following statement regarding the end reinforcement requirements for pretensioned concrete girders first appeared in 1961 AASHTO interim specification:

> *"In pretensioned beams, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4 percent of the total prestressing force shall be placed within the distance of d/4 of the end of the beam, the end stirrup to be as close to the end of the beam as practicable"*

This provision is nearly identical to Marshall and Mattock's recommendation if h/l_t is taken as a constant of 2. For 0.5 in. diameter strands, this ratio represented a girder depth of 50 in. at the time of their introductions in the 1960's the provisions conservatively covered most of the girder sizes used at the time, and the constant ratio of 2 was believed to be conservative. Article 9.22.1 in the AASHTO standard specification remains unchanged to this day. Article 5.10.10.1 in the AASHTO LRFD Specification contains essentially the same provisions as those in the AASHTO Standard Specifications except that the reinforcement is placed within a distance equal to 25 percent of the member total depth (h), rather than 25% of the effective depth (d)[12].

5.10.10.1 Factored Bursting Resistance

The bursting resistance of pretensioned anchorage zone provided by vertical reinforcement in the ends of pretensioned beams at the service limit state shall be taken as:

$$P_r = f_s A_s$$

Where:

f_s = *stress in steel not exceeding 20 ksi*
A_s = *total area of vertical reinforcement located within the distance h/4 from the end of the beam (in.^{2})*
h = *overall depth of precast member (in.)*

The resistance shall not be less than 4 percent of the prestressing force at transfer.

The end vertical reinforcement shall be as close to the end of the beam as practicable.

5.10.10.2 Confinement Reinforcement

For the distance of 1.5d from the end of the beams other than box beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than No. 3 deformed bars, with spacing not exceeding 6.0 in. and shaped to enclose the strands.

For box beams, transverse reinforcement shall be provided and anchored by extending the leg of stirrup into the web of the girder.

CHAPTER 3

FINITE ELEMENT ANALYSIS

3.1 Introduction

A finite element analysis was carried out in ABAQUS CAE to evaluate the effects of 0.7 inch strands at 2 inch spacing and to compare it with 0.6 inch strands with the same 2 inch of spacing. The maximum principal stress in the concrete along the transfer length of the girder and the axial stress at selected sections of the girder end zone were obtained for the applied prestressing force.

A 3D model of the AASHTO Type I beam was considered for the analysis. Two girders were modeled, one with 0.7 inch and another with 0.6 inch strands. Prestressing force was the only external force considered for the analysis, and was introduced by applying an initial compressive stress to the tendon elements.

The modeling process consists of various stages in ABAQUS CAE such as:

1. Geometric Modeling
2. Material Modeling
3. Defining Section
4. Loading & Boundary Condition
5. Meshing
6. Analysis
7. Visualization & Results

3.2 Geometric modeling

There are several steps in the process of defining geometry.

3.2.1 Part Definition

The Part module is used to create each part of a structure, and the Assembly module is used to assemble each instance of the parts. Parts of different shape features can be created such as solid, shell, wire, cuts and blends. The I-girder was created using the solid part in three dimension as shown in Figure 21 and the prestressing strands were created using the wire part in two dimension. The cross section of the I-girder was first drawn in two dimensions and then extruded along its length to three dimensions.

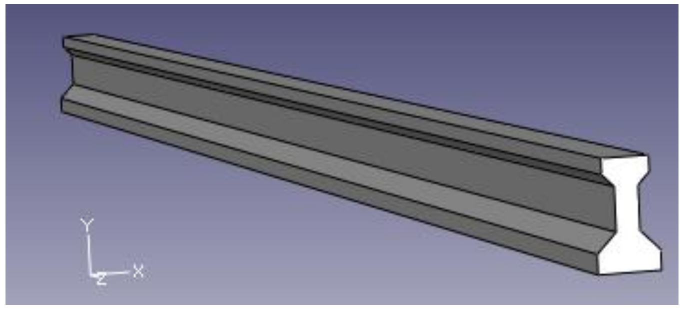


Figure 21 3D model of the AASHTO Type I girder in ABAQUS CAE

3.3 Material Properties

A linear material model was assumed for both the tendon and the concrete. The Poisson's ratio of the tendon was 0.27 and the modulus of elasticity was 28500 ksi. The Poisson's ratio of the concrete was 0.18. At release the concrete strength was 8000 psi and the modulus of elasticity was calculated using the equation,

$$E_c = 33{,}000\ w_c^{1.5}\sqrt{f'_c} \qquad (3.1)$$

Where,

w_c = unit weight of concrete (kcf)

3.4 Loading

Prestressing force was applied as a stress using the technique called the "Initial Condition". Initial conditions are specified for particular nodes or elements, as appropriate. The initial conditions can be set in the keywords editor or in some cases using a subroutine. In this analysis the stresses are applied using the keywords editor. An effective stress of 182 ksi was applied as the initial stress to the truss elements (tendon).The effective stress was obtained after considering the initial loss due to the elastic shortening of the beam. The time dependent losses such as creep and shrinkage were not considered since the stress at transfer of the prestressing force was only considered. This initial stress was applied to the elements of the tendon within the transfer length of the girder. The value of the effective stress was varied linearly from 0 ksi at the end face of the girder to 182 ksi at the transfer point of the girder.

3.5 Boundary condition

The boundary condition was assumed as pinned at one end and rollers at the other end resembling a simply supported beam. The whole model was restrained along the lateral direction of the girder.

3.6 Constraint between tendon and concrete

The contact between the concrete and the tendons were applied using a technique called the "embedded element technique". The embedded element technique is used to specify an element or a group of elements that lie embedded in a group of host elements whose response will be used to constrain the translational degree of freedom of the embedded nodes (i.e., nodes of the embedded elements). All the host elements can have only translational degrees of freedom, and the number of translational degrees of freedom at a node on the embedded element must be identical to the number of translational degrees of freedom at a node on the host element. ABAQUS searches for the geometric relationship between nodes of the embedded elements (Tendons) and the host elements (Concrete). If a node of an embedded element lies within the host element, the translational degree of freedom at the node is eliminated and the nodes become an embedded node. This model used a set of truss elements (tendon) that were embedded in a set of solid elements (concrete) [ABAQUS/Standard User's manual (Version 6.7-5)].

3.7 Meshing

The girder concrete was meshed with 20-noded quadratic brick elements and the tendons were modeled with 3-node quadratic 3D truss elements as shown in Figure 22 & Figure 23

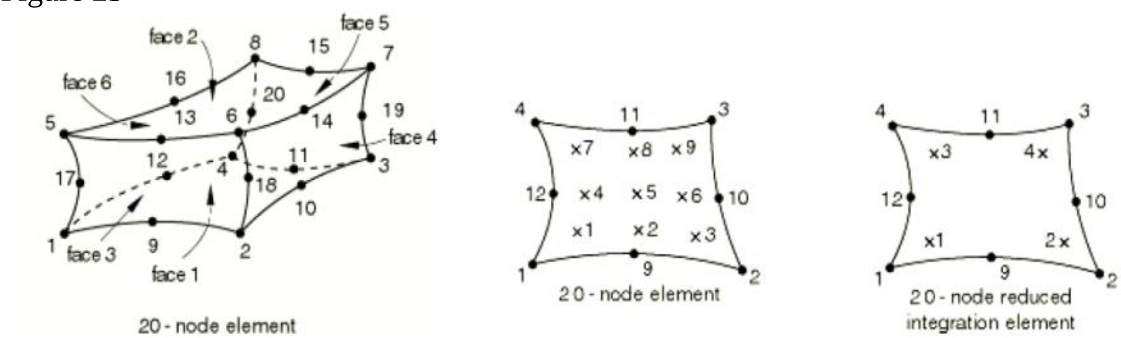


Figure 22 20-noded quadratic brick element with the integration points

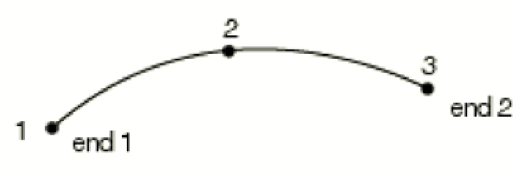


Figure 23 3-node quadratic 3D truss elements [ABAQUS/Standard User's manual (Version 6.7-5)]

3.8 Results and Discussion

Two AASHTO Type I girders were designed, one with 0.6 inch diameter strand and the other one with 0.7 inch diameter strand, with the same overall span capacity of 56 feet. The maximum principal and axial stresses in the concrete of the two 3D models are discussed in detail.

The deflection due to prestressing force at transfer was calculated based on the modulus of elasticity of concrete and the moment of inertia of the non-composite precast beam. A deflection of 2.42” (↑) and 2.32” (↑) was calculated for the girder with the 0.7” diameter strands and 0.6” diameter strands respectively. The maximum deflection values obtained from the FE model were 2.091” (↑) and 2.103” (↑) for the girder with the 0.7” diameter strands and 0.6” diameter strands respectively. The deflection due to the self weight of the beam was 0.515” (↓). Thus the expected camber values are 1.905” (↑) and 1.805” (↑) for the girder with 0.7” diameter strands and 0.6” diameter strands respectively.

As shown in the Table 10, the girder with the 0.7 inch strand reaches a maximum tensile stress of 1.74 ksi. The Figure 24 shows the maximum tensile stress occurs in the number 2 strand at a distance of 2 inches from the end face of the girder. A tensile stress of 1.43 ksi is reached at the transition zone between the bottom flange and the web, which results in a high probability of cracking.

The girder with the 0.6 inch diameter strand reaches a maximum tensile stress of 1.53 ksi as shown in Table 10. The Figure 25 shows the maximum tensile stress occurs in the number 7 strand at a distance of 2 inches from the end face of the girder. A tensile stress of 0.35 ksi is reached at the transition zone between the bottom flange and the web, which is less than the maximum tensile strength limit of concrete as specified in AASHTO LRFD (5.9.4.1.2), 0.68 ksi ($0.24\sqrt{f'_{ci}}$), which has the less probability of cracking.

The maximum principal stress contours at the end sections of the girder for 0.7 inch and 0.6 inch strands are shown in the Figure 26 & Figure 27 respectively. The same stress contoured along the central vertical plane for 0.7 inch and 0.6 inch strands are shown in Figure 28 & Figure 29 respectively. These figures show the cracking potential in the end zone of the girder.

Table 10 Values of maximum principal stress for the two diameters of strands

	Maximum Principal Stress, ksi	
	0.7'' strands	0.6'' strands
Maximum Value at a section	1.74^T	1.53^T
Value at the transition zone (Bottom Flange and Web)	1.43^T	$0.35^T < 0.68^T$

T = Tensile Stress

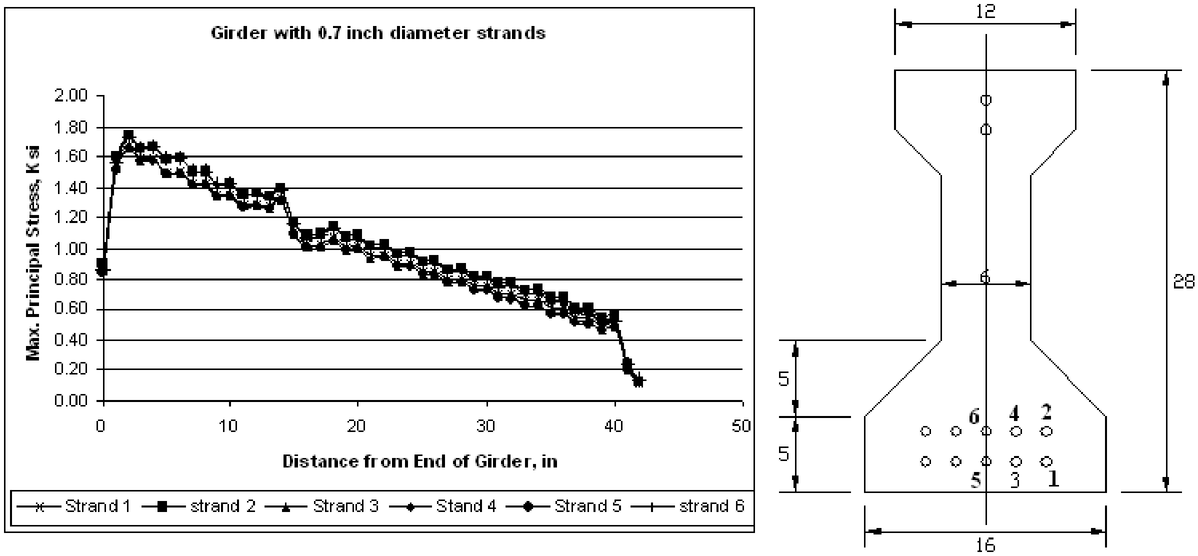


Figure 24 Maximum principal stress along the length of the girder from the end face at different locations of 0.7 in. strand

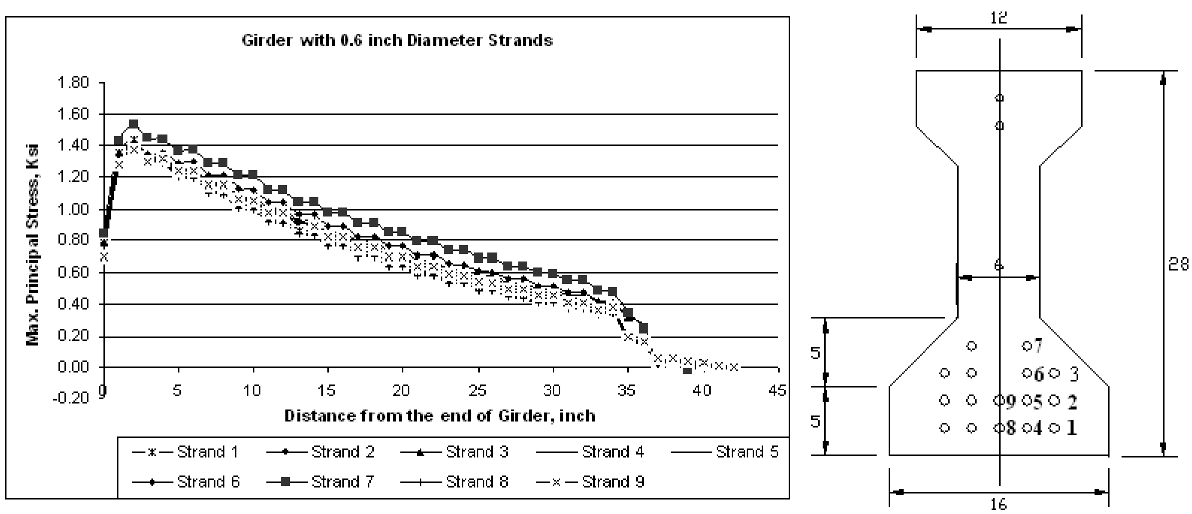


Figure 25 Maximum principal stress along the length of the girder from the end face at different locations of 0.6 in. strand

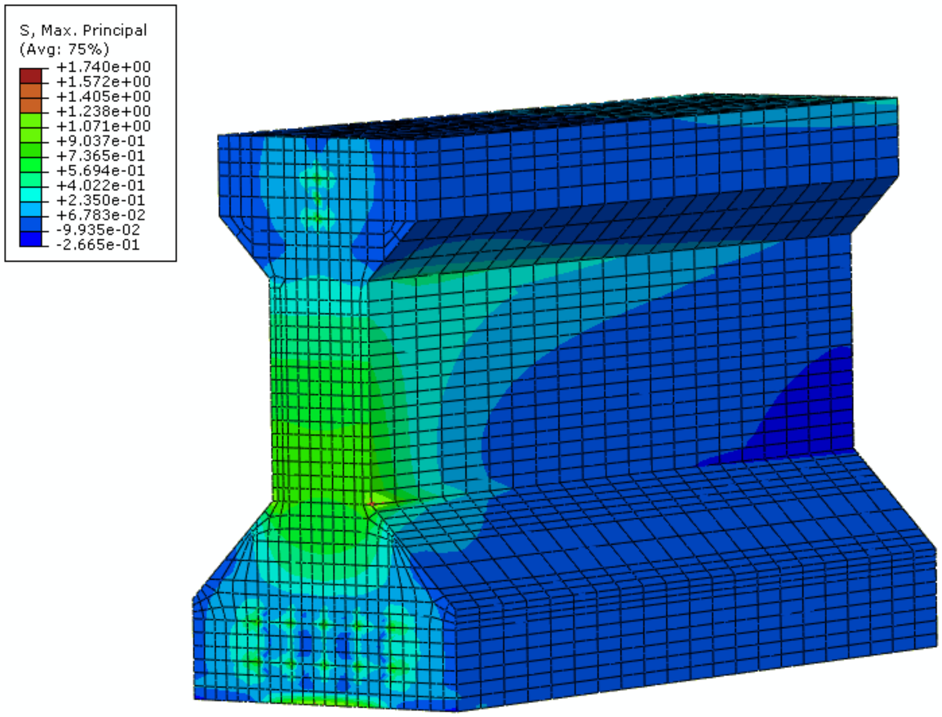


Figure 26 Maximum principal stress for the end zone of a girder with 0.7 in. strand.

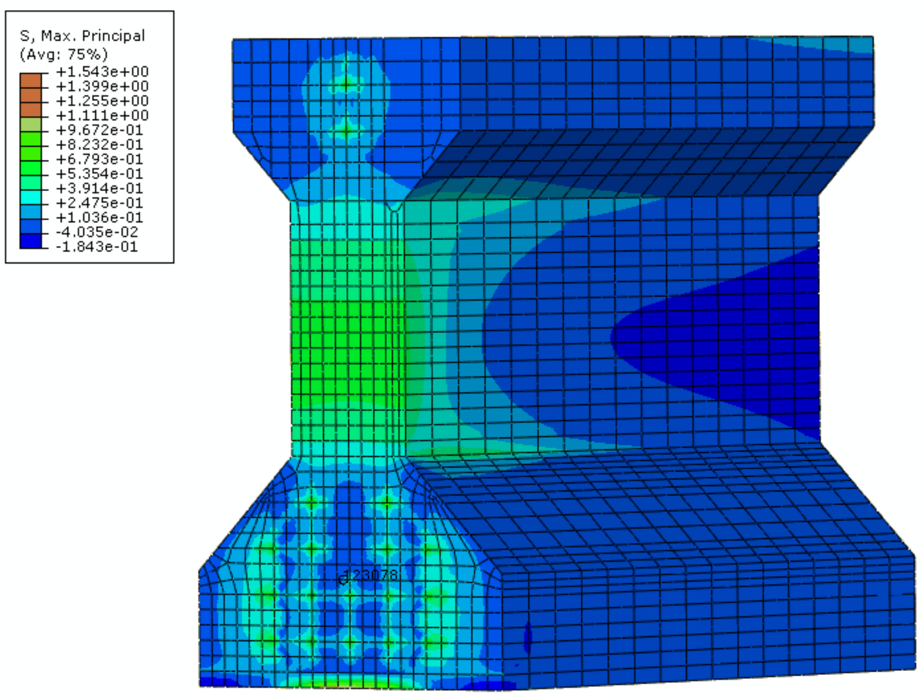


Figure 27 Maximum principal stress for the end zone of a girder with 0.6 in. strand.

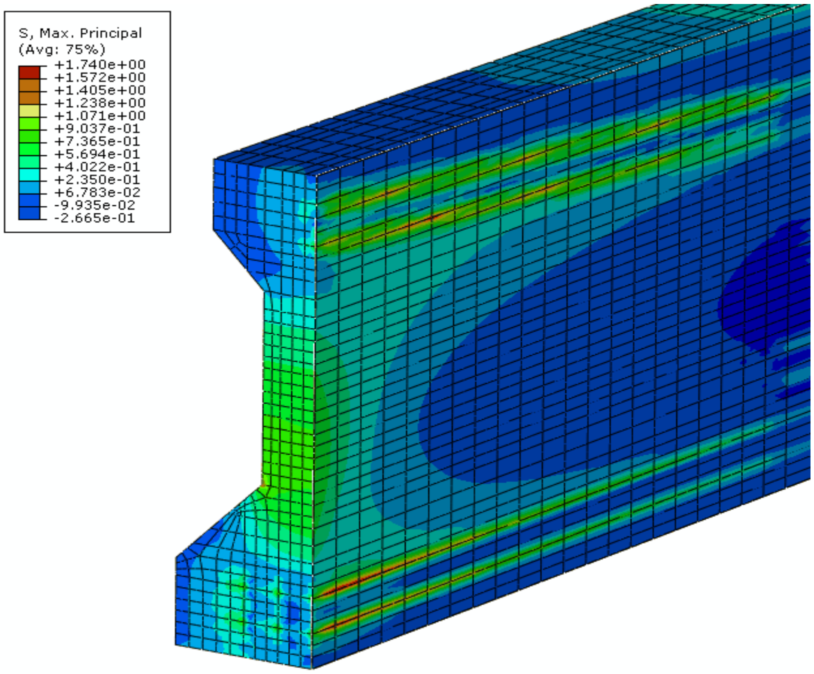


Figure 28 Maximum principal stress distribution near the end zone with 0.7 in. strand.

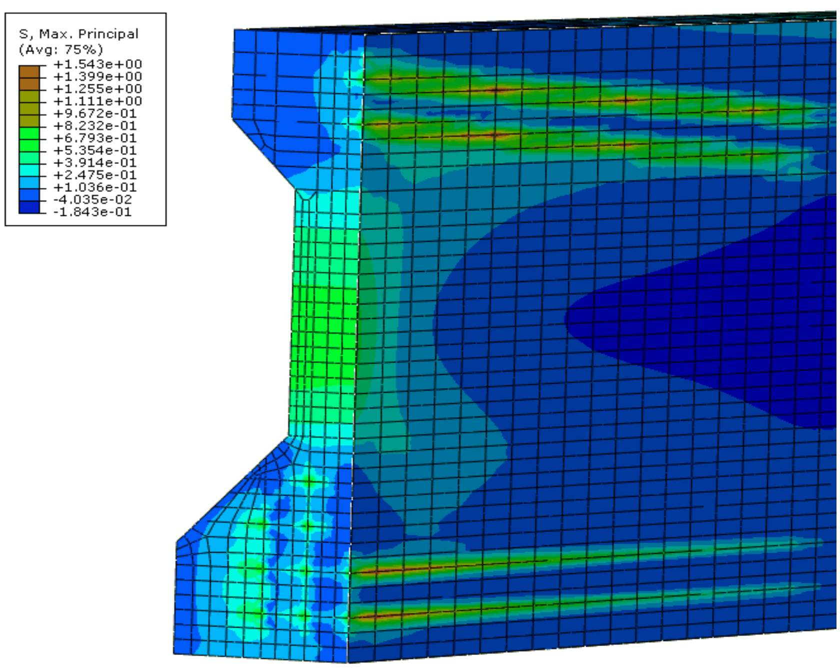


Figure 29 Maximum principal stress distribution near the end zone with 0.6 in. strand.

The axial stress variation along the depth of the girder at the selected sections shown in Figure 30 for 0.7 and 0.6 inch strands are shown in Figure 31 and Figure 32 respectively. The prestress force is transferred to the concrete and the axial stress variation becomes linear from the end face of the girder to the transfer points, which are 32 inches for 0.6 inch strands and 42 inches for 0.7 inch strands. The transfer point is calculated by the equation in AASHTO LRFD 2008.

As shown in Table 11, an axial stress of 1.09 ksi (Tension) is obtained in the girder with 0.7 inch diameter strands were as a stress of 0.43 Ksi (Tension) is obtained in the girder with 0.6 inch diameter strands. Thus the girder with 0.7 inch strand exceeds the maximum concrete tensile strength limit of 0.68 ksi.

At the transfer length, the girder with 0.7 inch diameter strands reached a compressive stress of 3.90 ksi at the bottom fiber and a tensile stress of 0.24 ksi at the top fiber which is below the maximum tensile strength limit of concrete. The girder with 0.6 inch diameter strand reached a compressive stress of 3.60 ksi at the bottom fiber and a tensile stress of 0.17 ksi at the top fiber which is also within the maximum tensile strength limit of concrete.

The axial stress contoured along the central vertical plane for 0.7 inch and 0.6 inch strands are shown in Figure 33 & Figure 34 respectively. It can be seen how the effective stress is reached from the end face to the transfer point of the girder.

Table 11 Values maximum axial stress for the two diameters of strands at different sections of the girder at the end zone

Distance from the End Face of Girder, inch	**Maximum Axial Stress, Ksi**	
	0.7'' strands (Top Fiber /Bottom fiber)	**0.6'' strands** (Top Fiber /Bottom fiber)
X= 0	$0.07^T/1.09^T$	$0.027^T/0.43^T$
Transfer Length (X=42'' for 0.7'' strands) (X=32'' for 0.6'' strands)	$0.24^T/3.40^C$	$0.17^T/3.61^C$

T = Tensile Stress, C= Compressive Stress

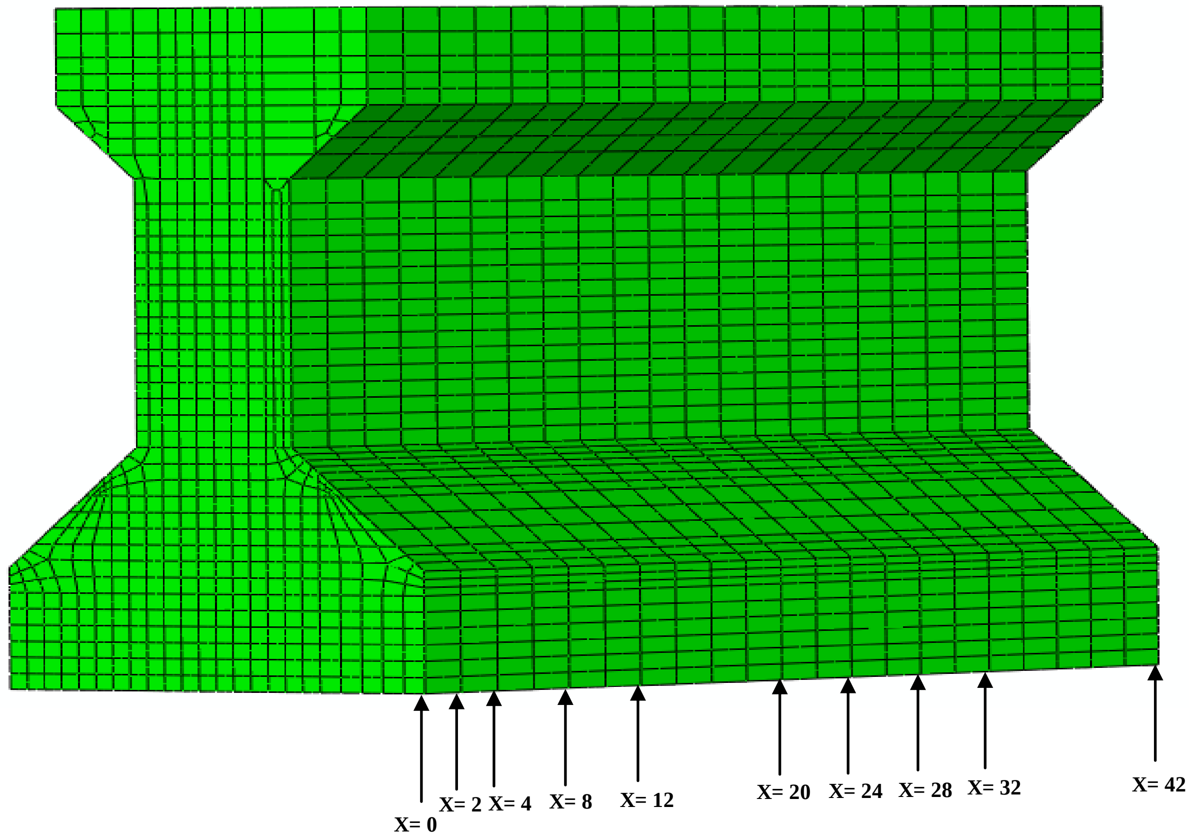


Figure 30 Finite element model of a prestressed concrete I-girder

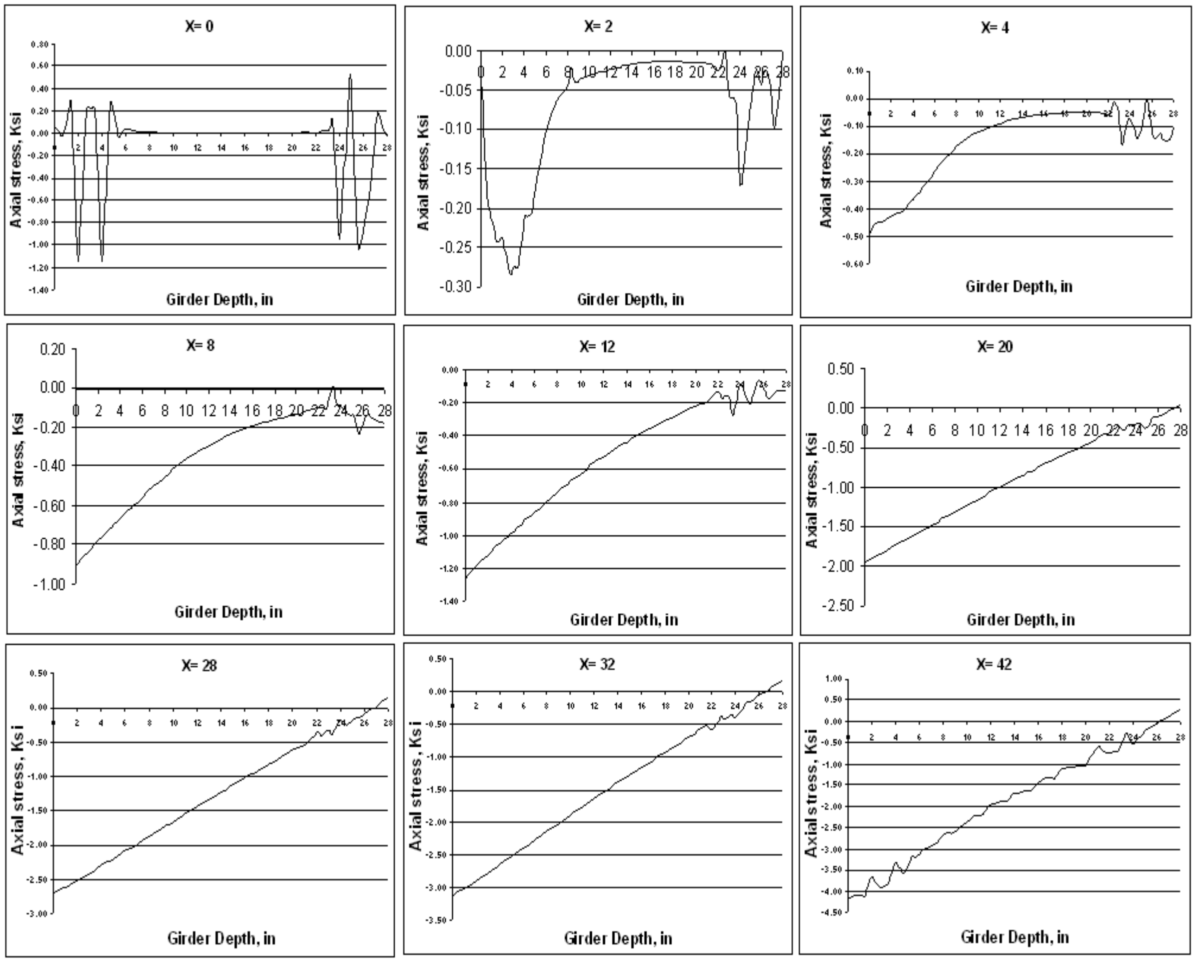


Figure 31 Axial stress distribution at different sections at the end zone with 0.7 in. strand.

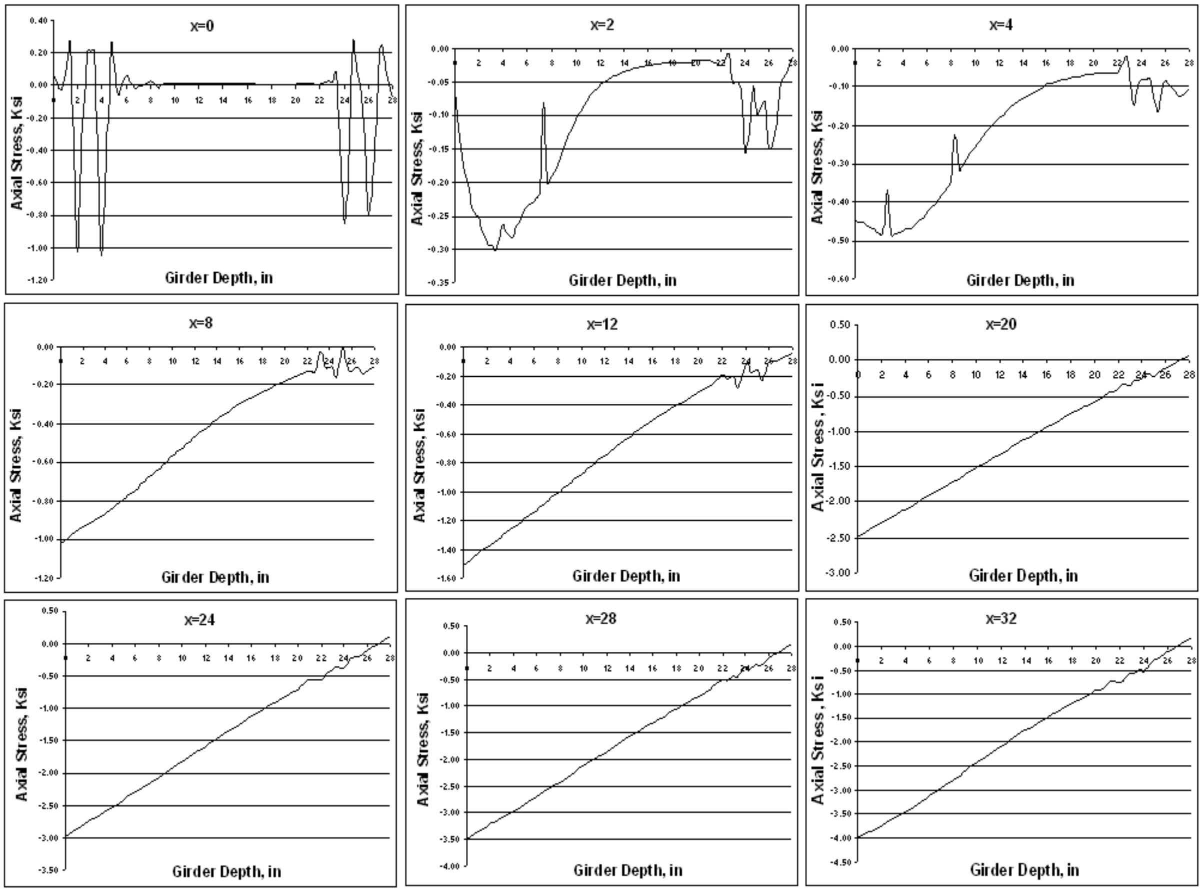


Figure 32 Axial stress distribution at different sections at the end zone with 0.6 in. strand.

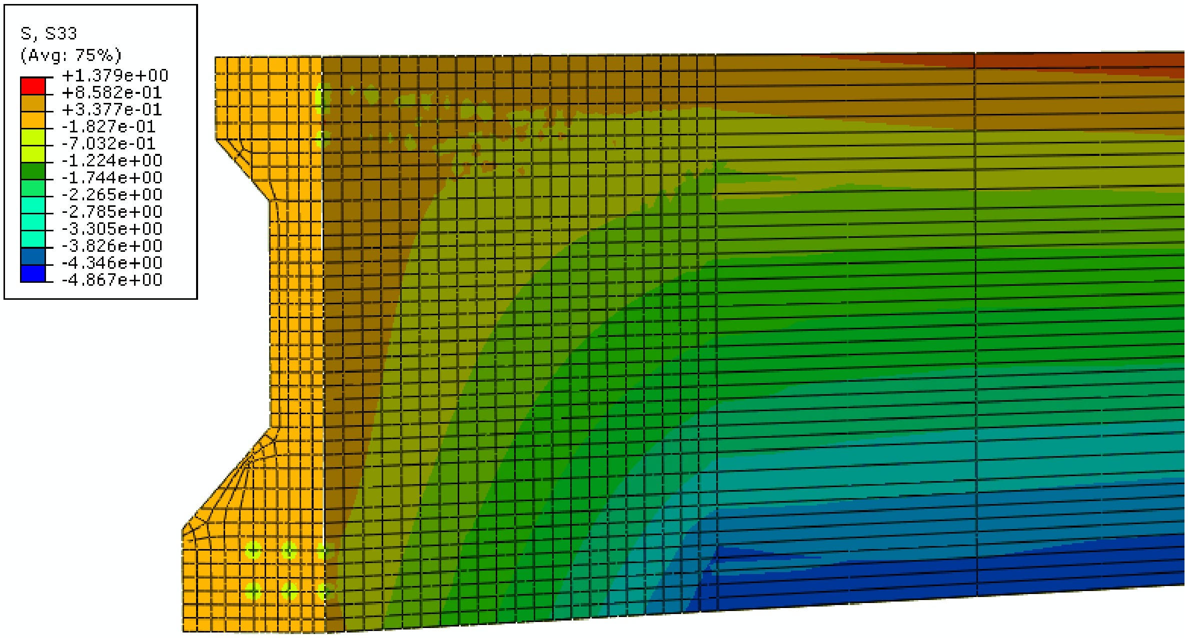


Figure 33 Axial stress distribution in the direction parallel to the direction of the 0.7 in. strand.

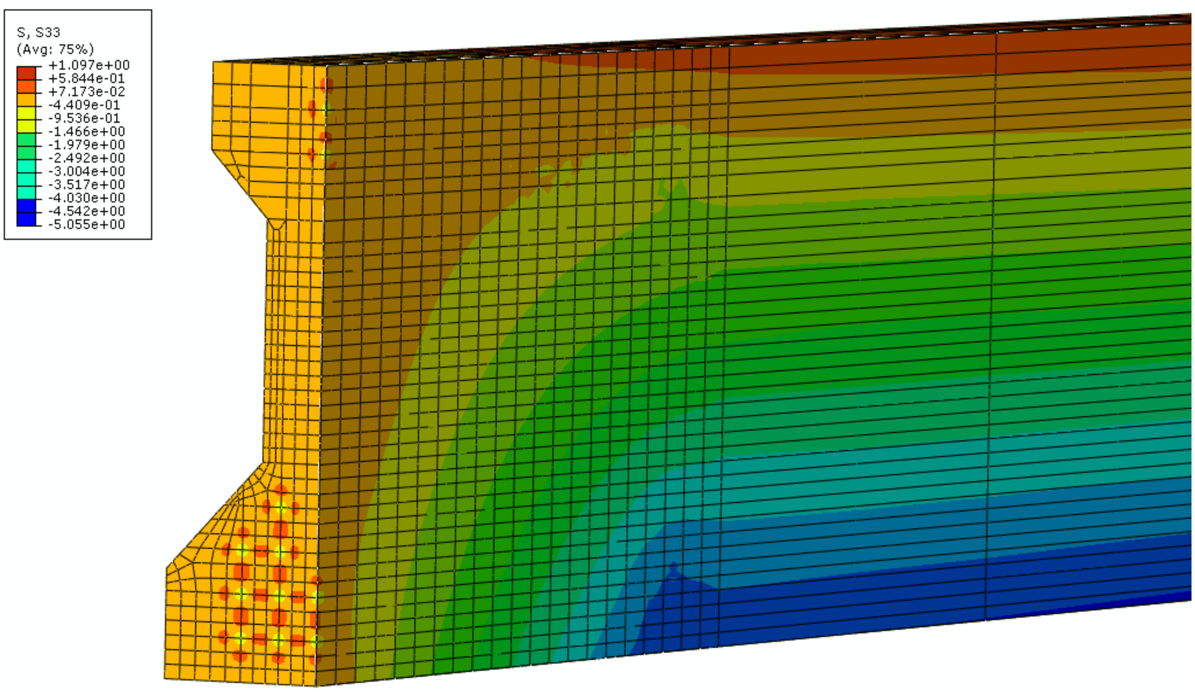


Figure 34 Axial stress distribution in the direction parallel to the direction of the 0.6 in. strand.

CHAPTER 4

STRUT AND TIE MODELLING

4.1 Introduction

Strut-and-Tie model of a structure is an idealized hypothesis truss that fits into the envelope of a structure and transmits forces from loading points to supports. The shape and geometry of the truss provide a visual representation of the flow of forces in the structure. Strut-and-Tie models are particularly useful in regions of the structure where stresses cannot be computed based on elastic bending theory[7,8,9]. In prestressed concrete girders the stresses acts non-linear in the anchorage zone. Thus using the strut and tie modeling these non-linear stresses can be determined and reinforcements are provided accordingly. In these members the prestressed force is considered as external load acting on the member.

The trusses in a strut and tie model consist of purely tension members (tie) and compression members (strut). The joints in the truss are pin joined which are defined as nodal zones. The two main criteria considered in a strut-and-Tie model are the strength of the elements and equilibrium of forces. Both ACI 318[10] and AASHTO LRFD Bridge Design Specifications[6] give provisions for the use of Strut-and-Tie modeling as a general design approach.

4.2 Assumptions

The basic assumptions used in the strut and tie modeling are

- Equilibrium of forces.
- External forces are applied at nodes.
- Forces in the strut and tie are uniaxial.
- Prestress force is considered as an external force.
- Struts must not cross or overlap each other.
- The angle between a strut and tie should not be less than 25°.
- Ties are permitted to cross struts or other ties.

4.3 Vertical splitting resistance reinforcement

Web splitting is developed at the end of the member due to the high prestressing force. This force is distributed in this region in a non-linear manner. This region of non-linear behavior where stresses cannot be computed based on beam theory is referred to as the D-region or disturbed zone. ACI defines a D-region as the portion of the member within a distance equal to the member height (h) from the force discontinuity or the geometric discontinuity. The bending theory and traditional design approach for shear and end zone reinforcement does not apply to D-region, because a major portion of the load is transferred directly to the supports by compressive concrete struts. Thus D-regions where shear and torsional forces can be controlling are more appropriately modeled by hypothetical trusses called the Strut-and-Tie models.

The stress distribution along the section at the boundary of the D-region caused due to the prestressing force and the self weight of the girder at transfer of prestressing force is determined based on the elastic analysis. The locations of the stress resultants are determined considering the triangular stress distribution and the girder cross section. The element forces and the stress distribution along the cross section are shown in Table 12 and Figure 35. The uniform self weight of the girder is resolved into equivalent concentrated loads applied at the joints of the truss in the strut and tie model. The stress diagram obtained using the bending theory is triangle with tensile stress at the top fiber and compressive stress in the bottom fiber of the girder. The equivalent tensile and compressive forces are determined based on the stress distribution and cross section of the girder, where Pt = Pc. The locations of both the compression and tension members in the D-region are determined, thus forming the truss. These members are analyzed for their respective forces using the method of joints. The required amount of reinforcement is provided based on the analyzed member forces.

4.4 Confinement reinforcement

The confinement reinforcement help in controlling the splitting cracks at the end section of the girder. A strut and tie model is developed in the transfer length portion of the girder in order to detail the splitting force due to the 12 prestressed straight strands in the bottom flange. The transfer length is assumed to be 42 inches ($60d_b$). The width of the model is taken as 3.5 inch based on the available width in both vertical and the horizontal directions in the bottom flange. The initial prestressing force is gradually introduced at different points in the truss model assuming a linear distribution along the transfer length as shown in Figure 36. Thus the required amount of splitting reinforcement is provided based on the tie forces determined after the analysis of the truss model.

Table 12 Element forces for Strut-and-Tie model in Figure 35

Member	Type	Force, kips
T1	Vertical Tie	106
T2	Horizontal Tie	157
S1	Inclined Strut	653
S2	Inclined Strut	189
S3	Vertical Strut	113

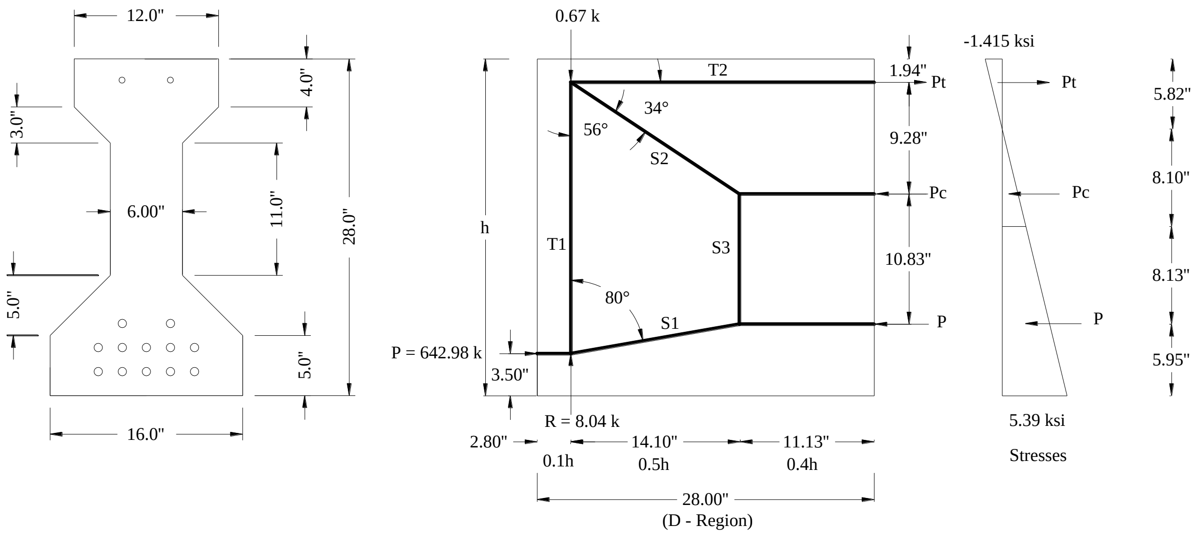


Figure 35 Strut-and-Tie for web splitting in the pretensioned girder

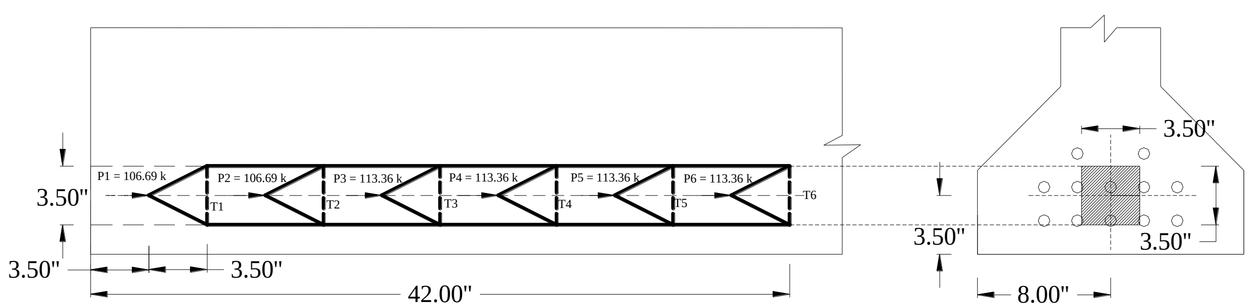


Figure 36 Strut-and-Tie model for the splitting force in the bottom flange

4.5 Shear Design Based on Strut-and-Tie Model

4.5.1 Vertical tie reinforcement

The shear reinforcement is provided based on the resultant force (due to the prestressing force as well as the factored dead and live load) in the vertical tie members obtained from the Strut-and-Tie model is as shown in Figure 37. The width of the horizontal tie in the bottom is 7 inch based on the centriod of the strands and 3.5 inch on top based on the horizontal reinforcement. The width of the bearing plate considered in the model is 12 inch. Using U-stirrups made with No. 4 bars, the total area of the vertical reinforcement for each tie is 0.4 in^2. Thus the spacing is determined based on the required area of steel obtained based on the equation below and the total design zone for a single vertical tie. As per the specifications of ACI code (section 11.5.5.1), a minimum spacing ($s \leq 0.75h \leq 24$ in) is provided for the vertical ties T6, T7 and T8, which is 20 inches. The values of the vertical tie forces are given in Table 13.

$$A_{St} = \frac{F_{tie}}{\emptyset\, f_y} \tag{4.1}$$

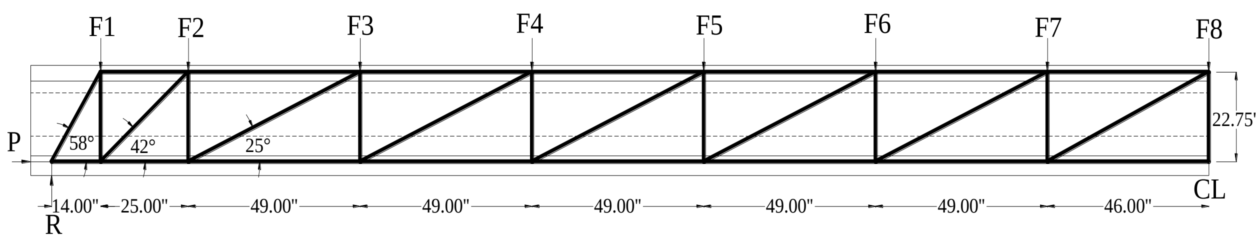


Figure 37 Truss Model for one half of the girder using Strut-and-Tie model

Table 13 Vertical tie forces for the Strut-and-Tie model in Figure 35

Vertical Tie	Force, kips	Design Zone Length, inches	A_{st} , in^2	Spacing, inches
T1	106.13	19.5	2.36	3
T2	99.68	37	2.21	6
T3	81.69	49	1.81	10
T4	63.91	49	1.42	10
T5	46.12	49	1.02	12
T6	28.30	49	0.63	20
T7	11.54	47.5	0.26	20
T8	3.06	46	0.068	20

4.5.2 Check for the capacity of inclined strut

The nominal compressive strength of a strut is determined using the effective compressive strength given by Eq. A-2 of the ACI-318(08).

$$F_{ns} = f_{ce} .A_{cs} \tag{4.2}$$

The effective compressive strength, f_{ce} is given by Eq. A-3 of the ACI-318(08), which is taken as smaller of the concrete strength in the strut and the nodal zone.

$$f_{ce} = 0.85\ \beta_s\ f_c' \tag{4.3}$$

The strength reduction factor, β_s for the node (C-C-T) is given as 0.8 and for the strut based on ACI 318 section A.3.2.1 is 0.6 which is considered since it is less than the factor for the node. The concrete strength of the girder at service is 12 ksi. Therefore the effective concrete strength for the inclined strut is

$$f_{ce} = 0.85\ \beta_s\ f_c' \tag{4.4}$$

$$f_{ce} = 0.85 \text{ x } 0.6 \text{ x } 12 = 6.12 \text{ ksi}$$

The width of the strut is calculated in order to determine the cross section area of the strut.

$$W_s = W_t cos\theta + W_b \sin\theta \tag{4.5}$$

Where W_t is the height of the horizontal tie which is 7 inch in the bottom flange and W_b is the bearing plate width which is 12 inch. Thus width of the strut S1 at the bottom as shown in Figure 38 is,

$$W_{s1b} = 7 \cos 58 + 12 \sin 58 = 13.89\text{”}$$

In order to determine the width of strut S1 at the top, the width of the vertical tie T1 is required and it is determined based on section RA.4.2 of the ACI code. Thus maximum tie width can be taken as the width corresponding to the width in a hydrostatic nodal zone, calculated as,

$$W_{t,max} = {}^{F_{nt}}/_{f_{ce} b_s} \tag{4.6}$$

$$W_{t1,max} = \frac{106.13}{0.75(0.85 \times 0.8 \times 12) \times 6} = 2.89"$$

$$W_{s1t} = 3.5 \cos 39.55 + 2.89 \sin 39.55 = 4.54\text{”}$$

Thus the capacity of the strut S1 is calculated at the section with the smallest width, which is at the top of the strut and is given as

$$\emptyset F_{s1} = 0.75 \times 6.12 \times 4.54 \times 12 = 250.1\ kips$$

The capacity of the strut (250.1 kips) is greater than the force in the strut (143.67 kips). Thus the strength of the strut is adequate. The strut forces for the Strut-and Tie model shown in Figure 38 are shown in Table 14.

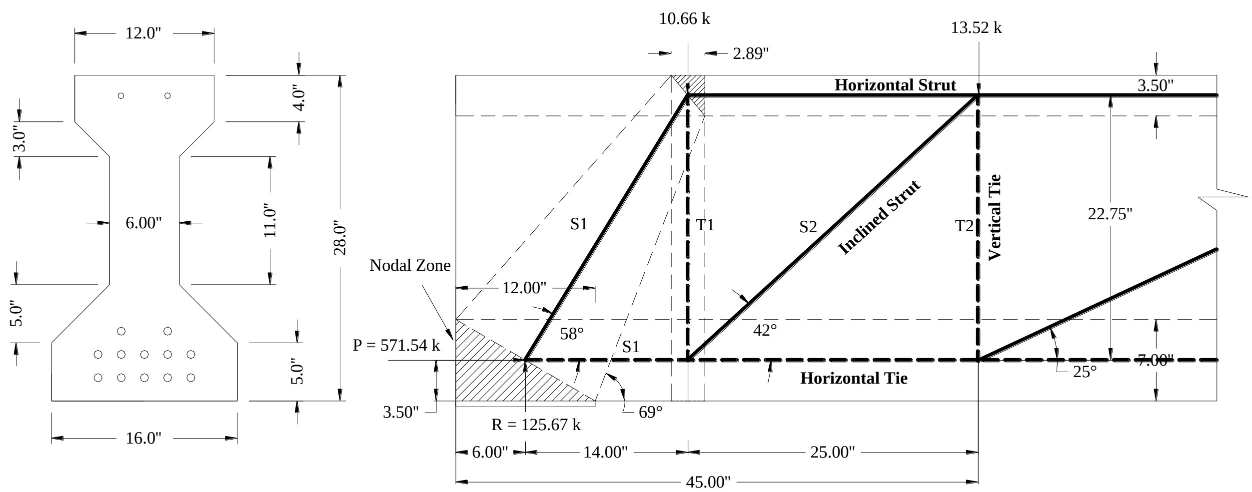


Figure 38 Strut-and-Tie model for the end region with horizontal strand pattern

Table 14 Strut forces for the truss in Figure 38.

Inclined Strut	Force, Kips
S1	143.67
S2	166.80
S3	235.91
S4	194.92
S5	153.17
S6	111.49
S7	69.48
S8	32.61

4.5.3 Check for bearing capacity

The bearing stress at the support location of the girder is given as

$$f_b = \frac{125.67}{12 \times 16} = 0.65\ ksi$$

The bearing strength limit based on ACI code for a C-C-T node is given as

$$\emptyset f_{cu} = 0.75 \times 0.85 \times 0.8 \times 12 = 6.12\ ksi$$

Thus the node at the support has adequate bearing capacity.

CHAPTER 5

MATERIALS AND TESTING

5.1 Introduction

This chapter gives the details of two specimens which were fabricated in this experimental investigation with two different types of strands. It includes the physical properties of both 0.7 in. and 0.62 in. strands used in this experimental investigation. It also includes the strength and mix of the high strength concrete used for the specimens.

5.2 Prestressing strands

Two different types of prestressing strand were used in this experimental investigation: 0.62 inch diameter (330 ksi) and 0.7 inch diameter (270 ksi). The 0.7 inch diameter strands were provided by MMI Strand Co. and 0.62 inch diameter strands were provided by Sumiden Wire Products Corporation. These strands were manufactured to meet ASTM A-416-05 specifications. The surface conditions of both types of strand were similar and were without any rust.

5.2.1 0.7 in. diameter strand

The strands were uncoated seven wire low- relaxation strands. All strands were grade 270 ksi. The physical properties of the strand provided in Table 15 are as reported by the strand manufacturer. Figure 39 shows a coil of 0.7 inch strand as provided by the manufacturer. They were wound similar to 0.5 inch and 0.6 inch diameter strands.

Table 15 Properties of 0.7 inch diameter, 270 grade strand

Grade	270 ksi
Nominal diameter	0.7”
Diameter tolerance	+0.026”, -0.006”
Nominal cross sectional area	0.294 in^2
Elastic modulus	28800 ksi
Minimum breaking strength	79400 lbs.
Minimum load at 1% extension	71500 lbs.
Minimum ultimate elongation in 24” gauge length	3.5%

Figure 39 0.7 inch, seven wire low-relaxation strands

5.2.2 0.62 in. diameter strand

The 0.62 inch diameter strands are made of high strength steel (330 Grade), uncoated, low-relaxation strand. The physical properties of the strand provided in Table 16 are as reported by the strand manufacturer.

Tension test was conducted on samples of strand in the structures lab at the University of Tennessee, Knoxville. The test did not show the values provided by the manufacturer, since in all the strand samples one of the seven wires failed at the anchorage location, where the strands were clamped by the jaws of the chuck as shown in Figure 40. This might be due to the stress concentration at the anchor points of the strand. The anchors used were similar to those used for post tensioning. The strands did not take any further load after the wire failed.

Table 16 Properties of 0.62 inch diameter, 330 grade strand

Grade	330 ksi
Nominal diameter	0.62”
Nominal cross sectional area	0.2227 in^2
Elastic modulus	28500 ksi
Breaking strength	76418 lbs.
Yield point	72576 lbs.
Minimum ultimate elongation	5.2%

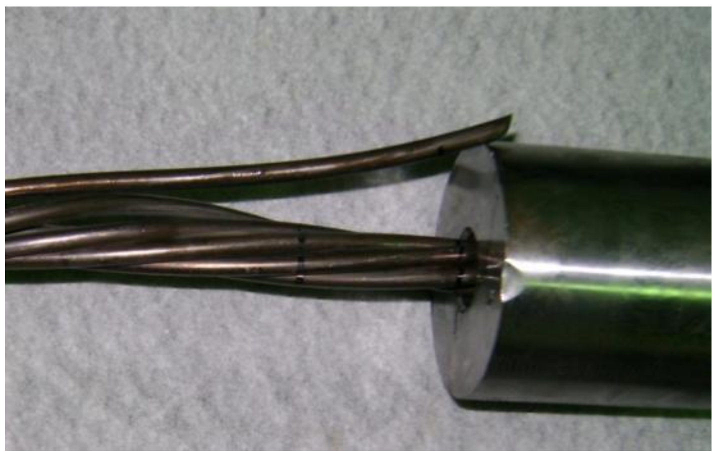


Figure 40 Failure of the wire in the tension test

5.3 Concrete

High strength concrete was used for both the girders with the two different strand diameters. The prestressed girders were designed for a concrete strength of 10 ksi at transfer and 12 ksi at service. About 39 cylinders were cast for each specimen to check the strength at transfer, 1 day, 3 days, 7 days, 14 days, 28 days and 58 days from detension and also to check the concrete strength to find the long term prestress losses, as shown in Figure 41.

5.3.1 Mix design

The concrete for the two specimens were mixed at the batching plant in Ross Prestress plant at Bristol, TN. The following mix design as listed in Table 17 was used for both girders. The slump of the concrete was 7 inches and the temperature was 75 ° F at the time of pour.

Figure 41 Concrete cylinders

Table 17 High strength concrete mix design

Materials	Quantity
Cement Type I	800 lbs.
Coarse aggregate (Lime Stone) 8P , ½”	1814 lbs
Fine aggregate (Sand)	1390 lbs
Silica fume	56 lbs
Water	28 Gallons
HRWR	125 oz
Water reducer	25 oz
w/c	0.292

5.3.2 Method of curing

Two methods of curing were done for the girders. The girder with 0.7 inch strand was cured using water as shown in the Figure 42. And the girder with 0.62 inch diameter strand was cured with steam. In order to obtain concrete strength of 10 ksi for the transfer of prestressing force as soon as possible the second specimen was steam cured.

Figure 42 Water curing of girder with 0.7 inch strand

5.3.3 Concrete strength

The concrete strength was tested every time before the concrete surface strain reading was taken on both the girders. The concrete cylinder test were performed at transfer of prestress force and 24 hrs, 3 days, 7 days, 14 days, 28 days from transfer. These tests were performed on 4x8 inch cylinders at Ross prestress plant. The test results of the cylinders are shown in Table 18. The typical failure of a cylinder in compression is shown in Figure 43.

Table 18 Average concrete strength at different days

Test Days	Cylinder Strength, psi	
	Specimen 1 (0.7")	**Specimen 2 (0.62")**
24 hrs from concrete pour	7,586	-
3 days from concrete pour	9,072	11,048
At Detension of prestress	10,252	11,592
24 hrs from Detension	10,929	11,618
3 day from Detension	10,438	11,724
7 day from Detension	11,791	11,877
14 day from Detension	12,374	12,255
28 day from Detension	14,191	12,295

Figure 43 Concrete compression test on 4x8 inch cylinder

5.4 Chucks

The chucks used in this experimental investigation had three components: the spring, the jaws and the barrel. The barrel dimensions were similar to those used for 0.5 in. and 0.6 in. diameter strands, 2 in. diameter and 4 in. long. The dimensions of the jaws varied for the different diameters of strand. 0.7 in. diameter strand had a larger jaw when compared to 0.6 in. strand as shown in Figure 44 and Figure 45. The jaws for both 0.6 in. and 0.62 in. strands were more or less similar.

Figure 44 Chuck used for 0.7 inch diameter strand

Figure 45 Jaws for 0.6”, 0.62” and 0.7” diameter strands

CHAPTER 6

GIRDER DESIGN AND CONSTRUCTION

6.1 Girder design: Design Parameters

AASHTO Type I girders were constructed for this experimental program. Two different methods of design were considered, AASHTO LRFD 2008 design specification and Strut-and-Tie modeling. The first specimen was designed using the 0.7 inch diameter, seven-wire, low-relaxation strand with the ultimate strength of 270 ksi and the second specimen was designed using the 0.62 inch diameter, seven-wire, low-relaxation strand with the ultimate strength of 330 ksi. In both the specimens, the left half was designed based on the AASHTO LRFD 2008 design specification and the right half was designed based on the Strut-and-Tie modeling.

The girders spanned 56 feet, determined based on the maximum span which could be tested at The University of Tennessee structures lab, with 12 numbers of strands provided in the bottom flange of the cross section. All 12 strands were straight and spaced at 2inch on both horizontal and vertical directions. The shear reinforcement, top flange reinforcement and the anchorage zone reinforcements are designed based on both AASHTO LRFD Design specification and Strut-and-Tie Modeling. The concrete strength was 10 Ksi at transfer and 12 Ksi at service. The design was consistent with the current TDOT practice. **Error! Reference source not found.** shows the cross-sectional roperties and dimensions of the AASHTO Type I I-girder.

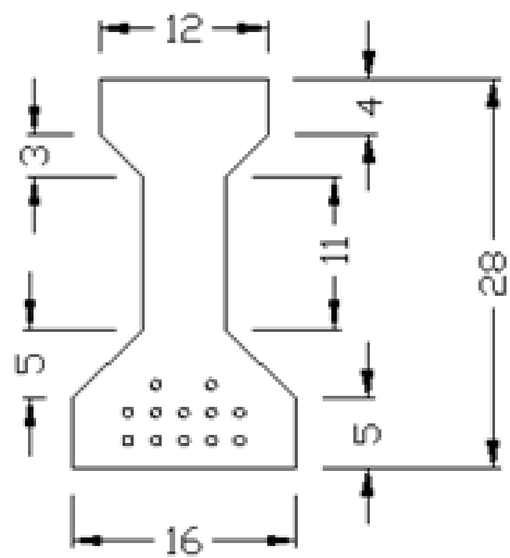


Figure 46 AASHTO Type I Girder with 12 Nos. of 0.7 in. or 0.62 in. strands

6.1.1 Specimen 1

The details of the reinforcement are show in Figure 47 and Figure 48 for specimen 1.

- AASHTO Type I Girder
- Design Live load is HL-93
- Span = 56 ft (Maximum span can be tested at UTK)
- Girder Concrete:
  - f'_{ci} (At Transfer) = 10 ksi
  - f'_c (At Service) = 12 ksi

- Number of Strand =12
- Diameter of the Strand = **0.7** inch
- Cross sectional area of the strand = **0.294** in^2
- Ultimate strength of strand, f_{pu} = **270** ksi
- Spacing of strands = 2" x 2"
- Force per strand = (1.0)(0.294)(0.75)(270) = 59.53 kips

6.1.1.2 Right Half of the Specimen (AASHTO LRFD 2008):

(Assumption: Details for 0.6" strands can be used here.)

Shear reinforcement:

- 15 Double legged #4 bar @ 8" spacing for 120"
- 14 Double legged #4 bar @ 10" spacing for 132"
- 6 Double legged #4 bar @ 12" spacing for 84"

Top flange reinforcement - 4 #6 bars for the entire length of the girder

Since the strands are not debonded or harped the tension in the top flange at the transfer length section of the girder exceeds the maximum allowable stress limit of $0.24\sqrt{f'_{ci}}$.

Temporary tensile stress limit in prestressed concrete before losses, fully prestressed components is $0.24\sqrt{f'_{ci}}$ with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5f_y$ not to exceed 30 ksi (AASHTO LRFD 2008).

Anchorage Zone Reinforcement:

Confinement reinforcement -7 #3 bars @ 6" spacing for a distance of 38".

Splitting resistance reinforcement - #4 double legged bars @ 1.5” spacing starting at 2”, for a distance of 7” from the end of the girder

Requirements in AASHTO LRFD 2008

5.10.10 Pretensioned Anchorage Zones

5.10.10.1 Splitting Resistance

The splitting resistance of pretensioned anchorage zones provided by reinforcement in the ends of pretensioned beams shall be taken as:

$$P_r = f_s A_s$$

Where:

f_s = stress in steel not exceed 20 ksi

A_s = total area of vertical reinforcement located within the distance h/4 from the end of the beam (in.^{2})

h =Overall dimension of precast member in the direction in which splitting resistance is being evaluated (in.)

For pretensioned I-girders or bulb tees, A_s shall be taken as the total area of the vertical reinforcement located within a distance of h/4 from the end of the member, where h is the overall height of the member (in.)

The resistance shall not be less than 4 percent of the total prestressing force at transfer.

The reinforcement shall be as close to the end of the beam as practicable.

For example, P_r = (20) (4*2*0.2) = 32 kips > 0.04{12*0.294[(0.75*270)-20.25]} = 25.72 kips

5.10.10.2 Confinement Reinforcement

For the distance of 1.5d from the end of the beams other than box beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than No. 3 deformed bars, with spacing not exceeding 6.0 in. and shaped to enclose the strands.

Where

d = distance from compression face to centriod of tension reinforcement (in.)

For example, 1.5(d) = 1.5(25) = 37.5 inches.

PCI Bridge Design Manual takes d as the overall depth of the girder (in.).For example, 1.5(d) = 1.5(28) = 42 inches.

The details of the reinforcement are shown in Figure 47.An example detail of the reinforcement used by the TDOT is shown in Appendix A.

6.1.1.3 Left Half of the Specimen: (Strut and Tie Modeling)

Shear reinforcement:

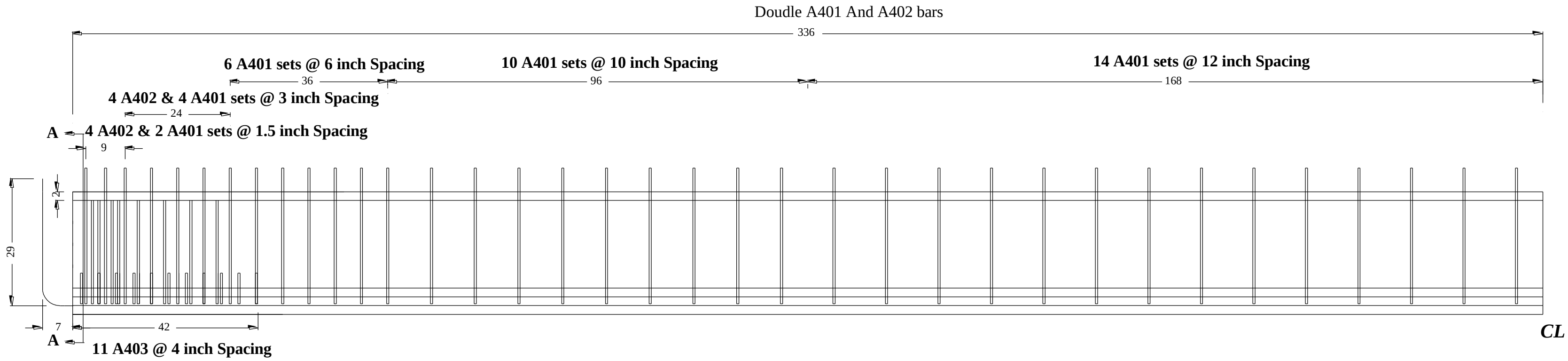
- 8Double legged #4 bar @ 3” spacing for 24”
- 6 Double legged #4 bar @ 6” spacing for 36”
- 10 Double legged #4 bar @ 10” spacing for 96”
- 14 Double legged #4 bar @ 12” spacing for 168”

Top flange reinforcement 4 #6 bars for the entire length of the girder

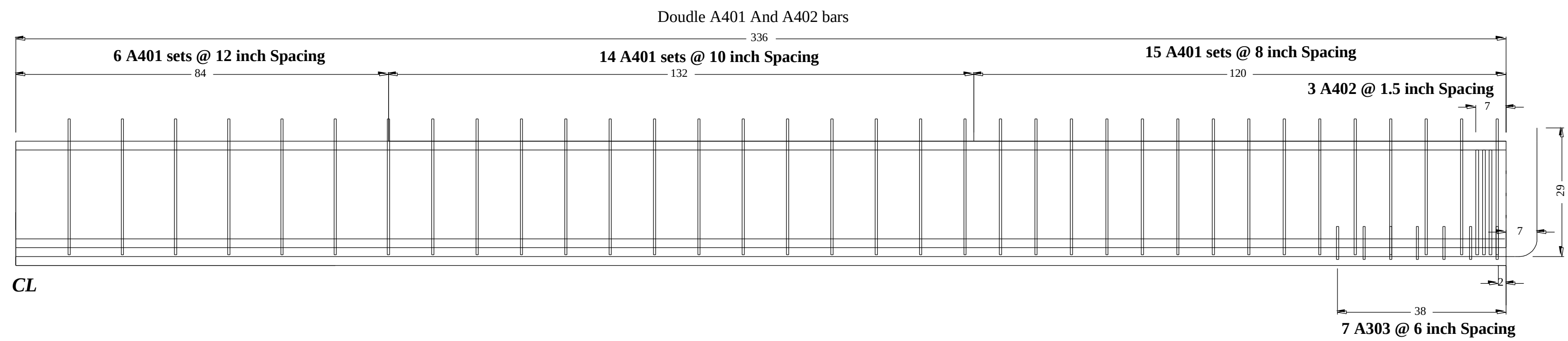
Anchorage Zone Reinforcement:

Confinement reinforcement	-11 #4 bars @ 4” spacing for a distance of 42”
Splitting resistance reinforcement	- #4 double legged bars @ 1.5” spacing starting at 2”, for a distance of 9” from the end of the girder

Left half design based on strut and tie modeling



Right half design based on AASHTO LRFD design specification



AASHTO Type I Beam 12 numbers of 0.7 inch diameter strands
All Dimensions are in inches

Figure 47 AASHTO Type I girder strand arrangement and shear reinforcement details

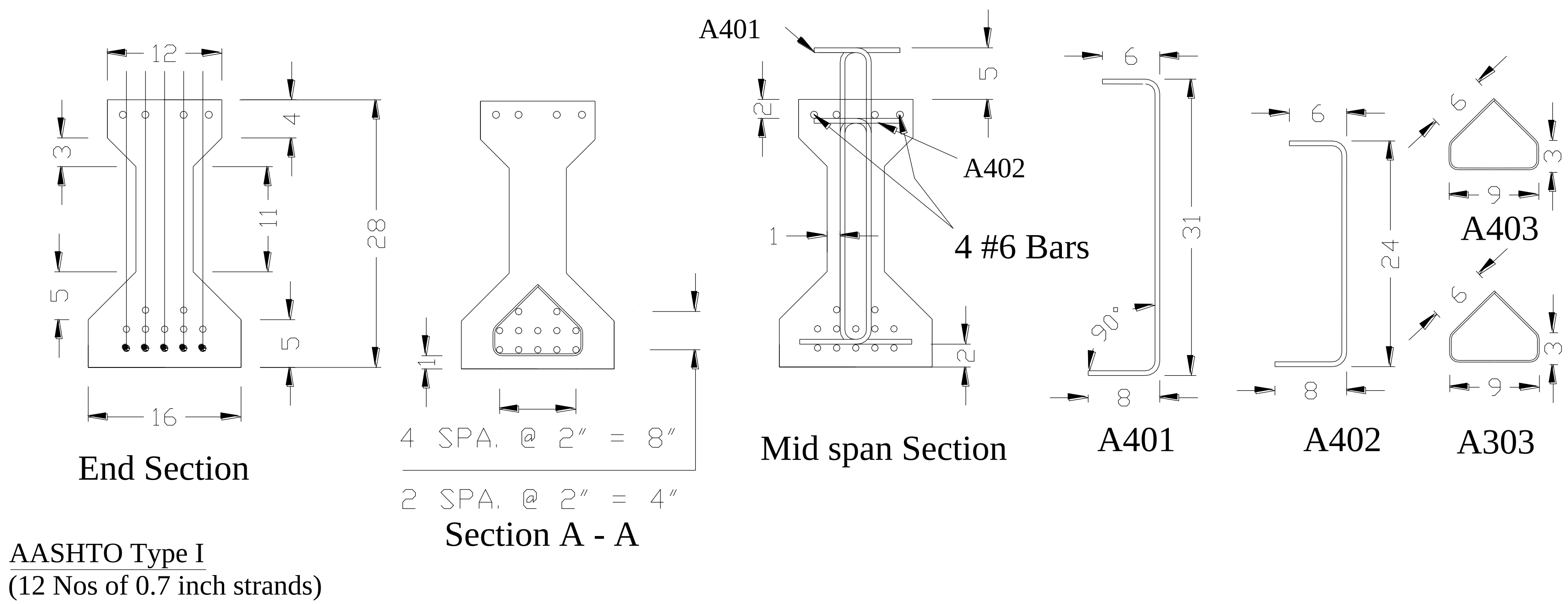


Figure 48 AASHTO Type I girder strand arrangement and shear reinforcement details

6.1.2 Specimen 2

The details of the reinforcement are show in Figure 47 specimen 2.

- AASHTO Type I Girder
- Design Live load is HL-93
- Span = 56 ft
- Girder Concrete:
    - f'_{ci} = 10 ksi
    - f'_{c} = 12 ksi

- Number of Strand =12 (Designed based on the maximum testable span)
- Diameter of the Strand = **0.62** inch
- Cross sectional area of the strand = **0.2227** in^2
- Ultimate strength of strand, f_{pu} = **330** ksi
- Spacing of strands = 2” x 2”
- Force per strand = (1.0)(0.2227)(0.75)(330) = 55.12 kips

6.1.2.1 Right Half of the Specimen :(AASHTO LRFD 2008)

Shear reinforcement:

- 15 Double legged #4 bar @ 8” spacing for 120”
- 14 Double legged #4 bar @ 10” spacing for 132”
- 6 Double legged #4 bar @ 12” spacing for 84”

Top flange reinforcement	- 4 #6 bars for the entire length of the girder

Anchorage Zone Reinforcement:

Confinement reinforcement	- 7 #3 bars (A303) @ 6” spacing for a distance of 38” and 7 #3 (A304) bars @ 6” spacing for a distance of 38” as shown in Figure 50.
Splitting resistance reinforcement	- #4 double legged bars @ 1.5” spacing starting at 2”, for a distance of 7” from the end of the girder

6.1.2.2 Left Half of the Specimen: (Strut and Tie Modeling)

Shear reinforcement:

- 8Double legged #4 bar @ 3” spacing for 24”
- 6 Double legged #4 bar @ 6” spacing for 36”
- 10 Double legged #4 bar @ 10” spacing for 96”
- 14 Double legged #4 bar @ 12” spacing for 168”

Top flange reinforcement - 4 #6 bars for the entire length of the girder

Anchorage Zone Reinforcement:

Confinement reinforcement -10 #4 bars @ 4” spacing for a distance of 38”

Splitting resistance reinforcement - #4 double legged bars @ 1.5” spacing starting at 2”, for a distance of 9” from the end of the girder

6.2 Girder Fabrication

All the specimens were fabricated by a local producer at Bristol, TN. In the initial process of casting the girders, the span (56’) was set in the prestressing bed and they were lubricated in order to prevent the concrete from sticking to the bed. Then the strands were laid by passing them through the dead end and to the live end. The diameter of the holes on the very end walls of the bed was increased to accommodate the larger diameter strands as shown in Figure 51. The chucks were placed in the dead end to anchor the strands and tensioned in the live end. The Figure 52 and Figure 53 show the personnel’s at Ross prestress plant laying the 0.7” strands by passing them through the dead end of the bed. The two ends of the girder were marked as L (Design based on Strut and Tie modeling) and R (Design based on AASHTO LRFD 2008 specification). The two sides of the girder were marked as side 1 and side 2. Thus each end of the girder was marked as L1, R1, L2 and R2. The Figure 54 and Figure 55 shows the 12 strands before tensioning and the chucks placed at 2 in. spacing in the dead end of the bed.

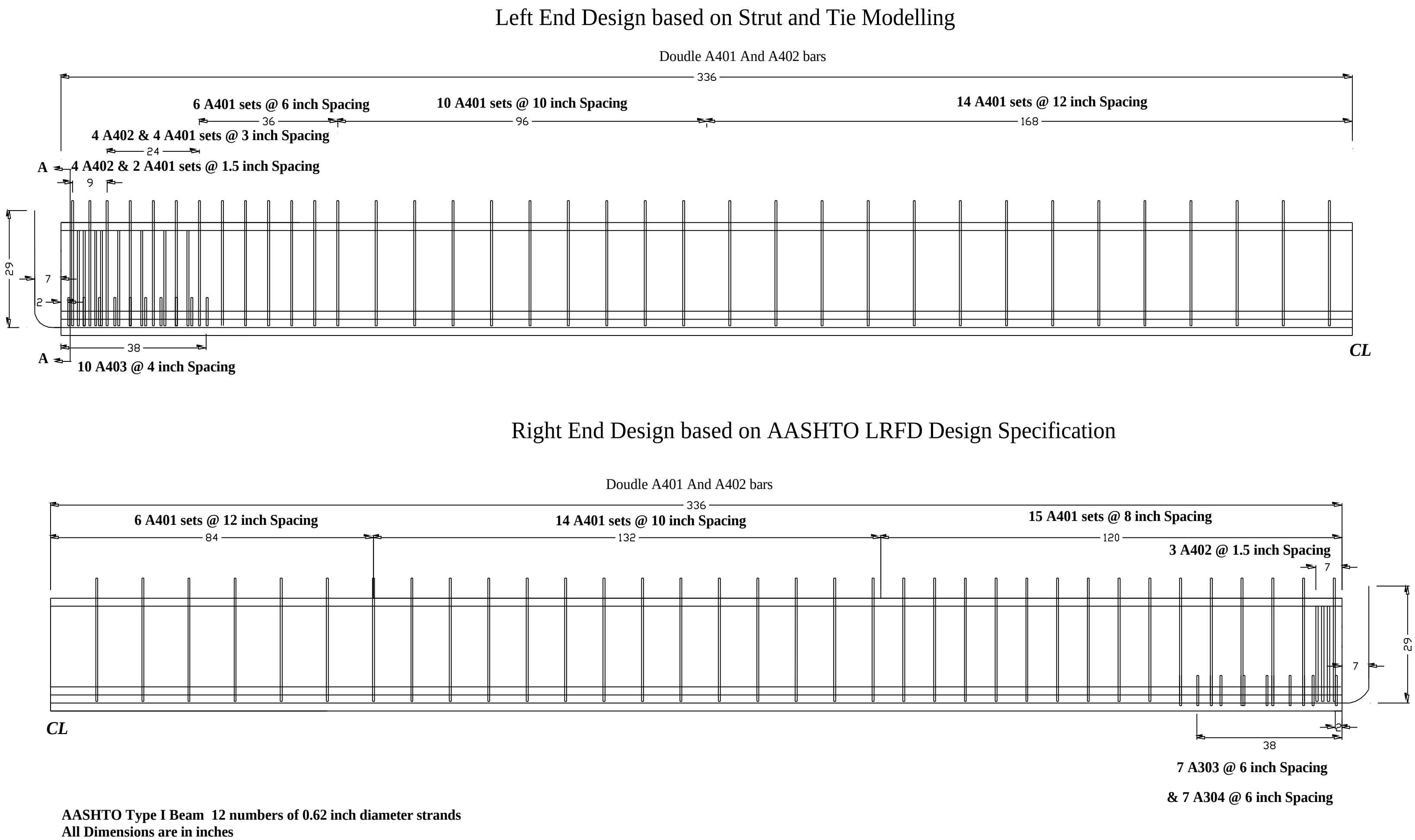


Figure 49 AASHTO Type I girder strand arrangement and shear reinforcement details

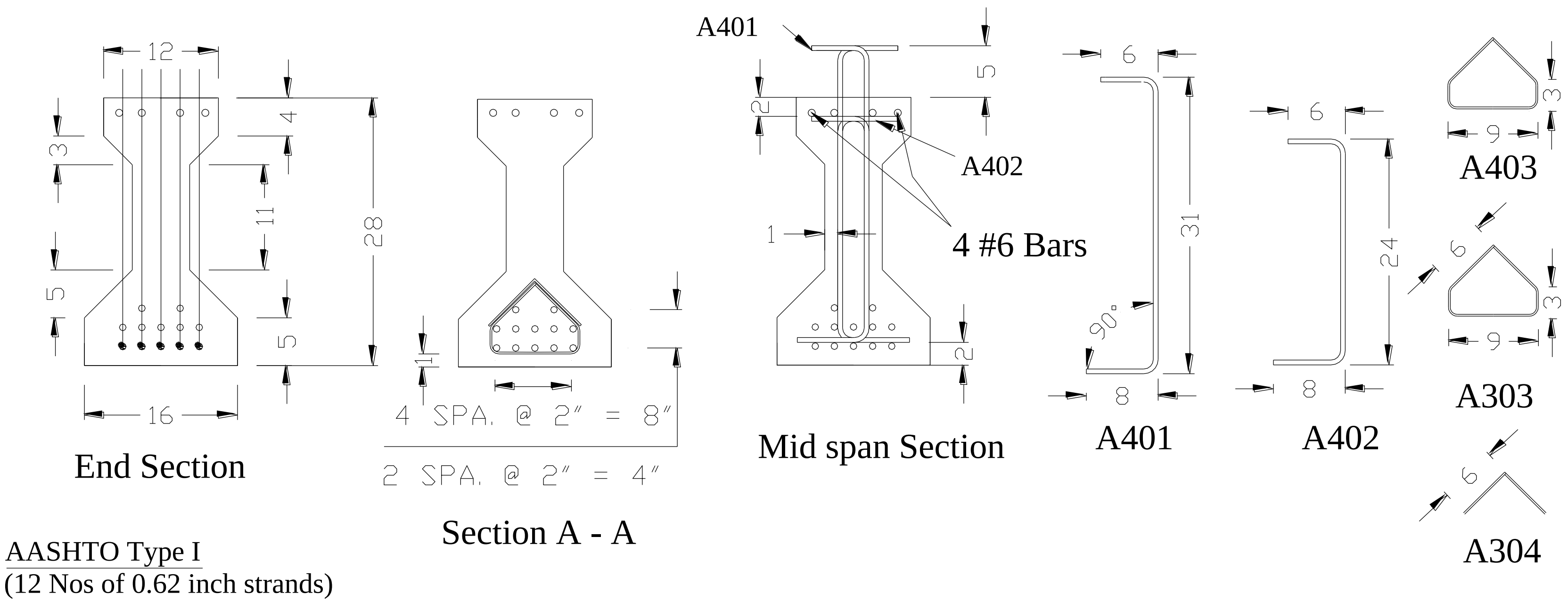


Figure 50 AASHTO Type I girder strand arrangement and shear reinforcement details

6.2.1 Strand Tensioning

In the strand tensioning process a 120 kip Hydraulic jack was used. Two chucks were placed, one at the dead end of the prestressing bed and other at the live end of the bed. The jack has a stroke length of 10.5 inches. Since the elongation was around 13 inches the strand was tensioned first to half of the load and then the pressure in the jack was released and the remaining half was reloaded. A 112 kip load cell was placed between the chuck and the piston to measure the amount of force applied to the strand. The entire process for tensioning a single strand took about 15 minutes and at the end the elongations were checked to the calculated values.

Figure 51 Increasing the diameter of the holes in the bed

Figure 52 Laying of 0.7 in. strand through the dead end of the prestressing bed

Figure 53 Ross prestress personals laying down the 0.7” strands

Figure 54 12 Nos. of 0.7 in. strands before tensioning

Figure 55 2”x 2” spaced chucks at the dead end of the girder bed

6.2.2 Measurement of tensioning force and elongation of the strands

A 112.4 kips (500 KN) load cell is attached to the end of the strand tensioning set up in order to determine the amount of force applied to each strand as shown in

Figure 56 and Figure 57. The load cell is attached to a data acquisition system to record the load applied to each strand. The load was monitored and the jack was stopped when the load reached the required value.

The elongation of the strand was measured by marking the strand at the end of the chuck after applying the initial force. An initial force of 7 kip was applied before marking the strand in order to account for the slag in the strand and the chuck slippage. Once the 7 kip was applied, the strand was marked as shown in

Figure 58 and the remaining load was applied. Once the strand was prestressed to its required load the jack was removed and the elongation of the strand was measured and checked with the calculated values. A tolerance of ±5 % was accepted for both the prestressing force and the elongation.

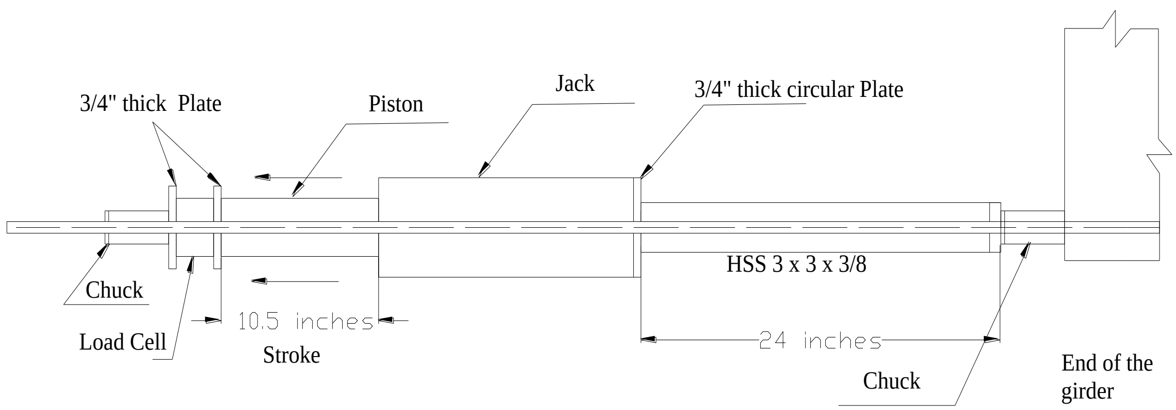


Figure 56 Setup for tensioning the strand

Figure 57 Strand Tensioning

Figure 58 Initial marking and the final measurement of elongation of the strand

6.2.3 Pre-Pour setup

Once the strands were tensioned, the positions for tying the rebar were marked on the strand and double checked. The rebar was tied firmly to the strands. Once all the rebar were tied and checked the side forms were placed. The vertical and horizontal levels were checked to make sure the forms were not inclined and bolted firmly to the prestressing bed. The Figure 59 and Figure 60 show the tying of rebar and placing of forms by Ross prestress personals.

Figure 59 Tying of mild steel reinforcement for the girder

Figure 60 Placing of the side forms for the girder with 0.7 in. strand

6.2.4 Placing the concrete

High strength concrete was used for the girders (0.7 in. and 0.62 in.), 10 ksi at transfer and 12 ksi at service. The concrete was mixed and placed at Ross Prestress Plant in Bristol, TN. The concrete was placed and consolidated with two electric vibrators moving side by side as shown in Figure 61. The slump was 7 in. and the temperature was 75°F for the concrete mix used for both the girders. The concrete for the two specimens were poured on two different days. The concrete for girder with 0.7 in. diameter strand was water cured and for 0.62 in. diameter strand was steam cured.

6.2.5 DEMEC gauge set up

After the forms were removed, DEMEC gauge points were affixed to both the sides of the girder using the appropriate adhesive. The instrumentation for the measurement of the camber for the girder was also set up by running a thin wire at the centroid location of the girder with one end tied and the other end going over a pulley and attached to weights. Hose clamps were setup at both ends of the girder to measure the strand drawn-in.

Figure 61 Placement of concrete

6.2.6 At Transfer of Prestress

The required concrete strength at detension was 10 ksi. Three cylinders were tested to check the concrete strength before detensioning the strands of the specimen. The detension was done in a symmetrical pattern as shown in the Figure 62.The concrete surface strain readings were taken after every step of the detensioning. The strands were cut using flaming cutting at the same time on both sides of the girder as shown in Figure 63. It was observed that the wires in the strands unwound as soon as the strands were flame cut. Once the strands were detensioned the concrete surface strain readings were taken and the girder was moved to a different location as shown in Figure 64 and the preparation for the second specimen was started. During the detensioning process the girder was displaced about 3 inches on the bed due to the large prestressing force.

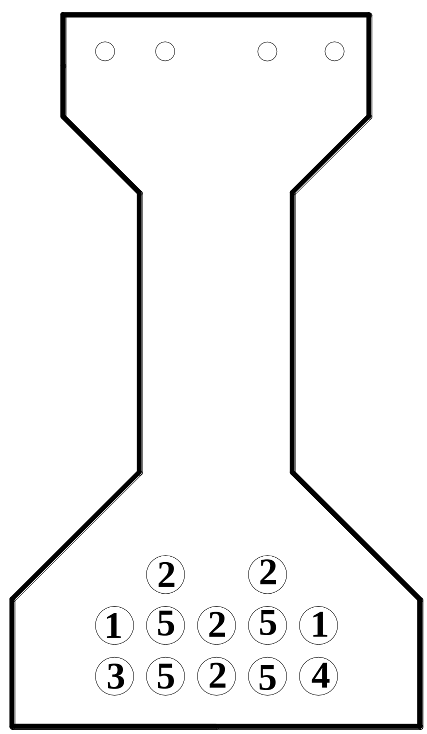


Figure 62 Detensioning steps

Figure 63 Release of prestress by flame cutting and unwinding of strand ends

Figure 64 Moving the girder from the prestressing bed

CHAPTER 7

TRANSFER LENGTH, GIRDER END CRACKING & PRESTRESS LOSSES

7.1 Introduction

An accurate estimate of the transfer length is important for several reasons: calculation of the concrete stresses at transfer and service loads, design of anchorage zone reinforcement for strut and tie models, and design of shear reinforcement which requires knowledge of the level of precompression in the concrete.

Two different types of instrumentation were used to determine the transfer length. One method was using the DEMEC strain measurement system, which involved the measurement of the surface strain of concrete. The other method for calculating transfer length was strand drawn-in, in which the distance slipped by the strand into the concrete is measured [27].

7.2 Transfer Length – Measurement, Data Reduction

The Transfer length measurements were made on both the AASHTO Type I I-girders, one with 0.7 in. diameter strands and other with 0.62 in. diameter strands.

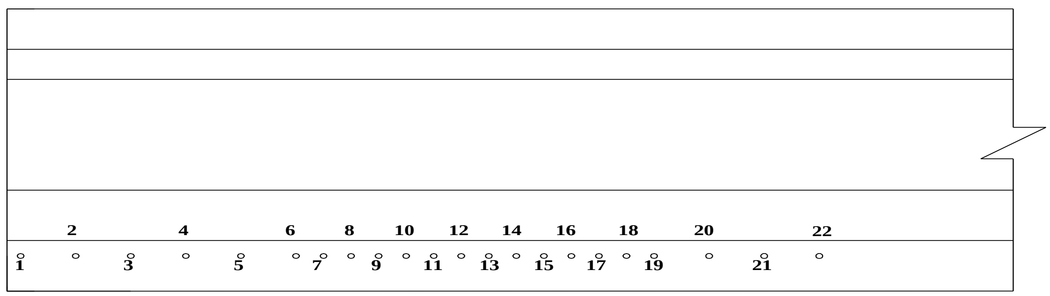
7.2.1 Measurement

The two most commonly used techniques for the measurement of the transfer length are DEMEC concrete surface strain and Strand drawn-in measurement.

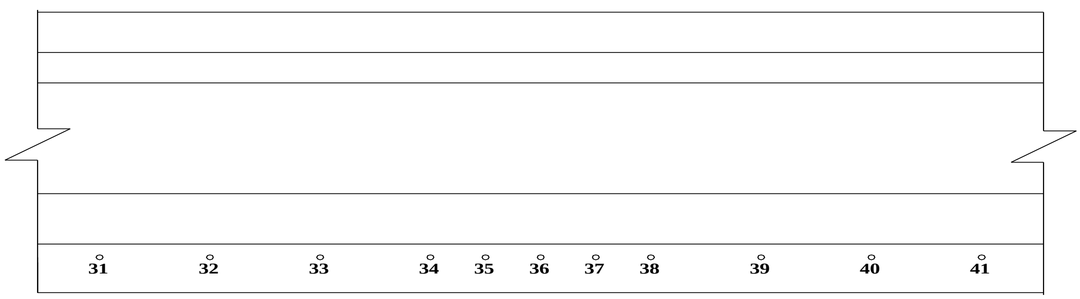
7.2.1.1 Concrete surface strain measurement

Transfer length is the distance required to transfer the effective prestressing force from the strand to the concrete. To determine the transfer length for the girders, a series of DEtachable MEChanical (DEMEC) strain measurement system is used to measure the concrete surface strain. DEMEC strain measurement system consists of a series of points placed on the surface of the concrete. These points have small metallic discs of 1/4th inch in diameter, which are placed at the centroid of the prestressing strands in the ends of the girder, on all four sides of the bottom flange. For our specimens the DEMEC points are placed at a spacing of 4 inches starting at 2 inch from the end of the girder for a distance of 20 inches, at a spacing of 2 inches for a distance of 26 inches and then at a spacing of 4 inches for a distance of 12 inches. These spacings were determined based on the gauge length (3.937 inches) of the DEMEC gauge and to increase the number of DEMEC points where it was required. Thus there are 22 points proving 20 readings on each side of the girder. Therefore each girder has four lines of DEMEC points on both sides and both ends of the girder. The set up of the DEMEC points are shown in Figure 65, Figure 66 and Figure 67.

DEMEC points were also set at the midspan section of the girder on both sides in order to determine the short term and long term losses of prestress in the girders. DEMEC points were also set on the web of the I-section at the end zones to determine the concrete strain in the vertical direction of the girder.



Left end with 22 nos of DEMEC points, 8 spaced at 4 in and 13 spaced at 2".



Mid section with 11 nos of DEMEC points, 6 spaced at 8" and 4 spaced at 4".

Figure 65 DEMEC points on both ends of the girder

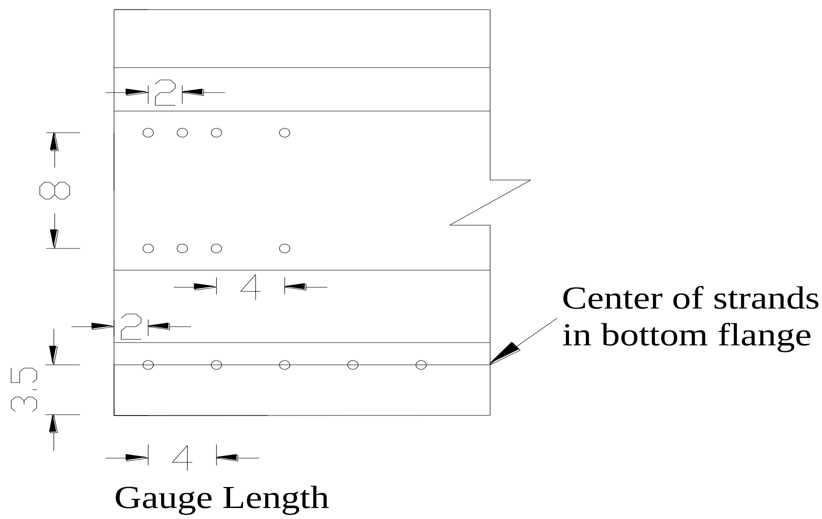


Figure 66 DEMEC points along the centroid of the strands and web of the girder

Figure 67 DEMEC points at the live end of the girder

7.2.1.2 Strand Drawn-in Measurement

Strand drawn-in is a measurement of how far the strand at the face of the concrete is pulled into the beam after the prestress is released. Strand drawn-in helps in determining the effectiveness of the bond between the concrete and prestressing strand after the prestress is released. To measure the drawn-in, 2” x 2” x 0.5” angle sections were attached at a distance of 3 ft from the end face of the girder. A typical set up is shown in Figure 68 and Figure 69. Two sets of reading of the distance between the end of the angle and the face of the beam end were taken to measure the drawn-in. The first reading was taken before the pretensioning was released and the second reading was taken after the release. These readings were measured using a digital caliper.

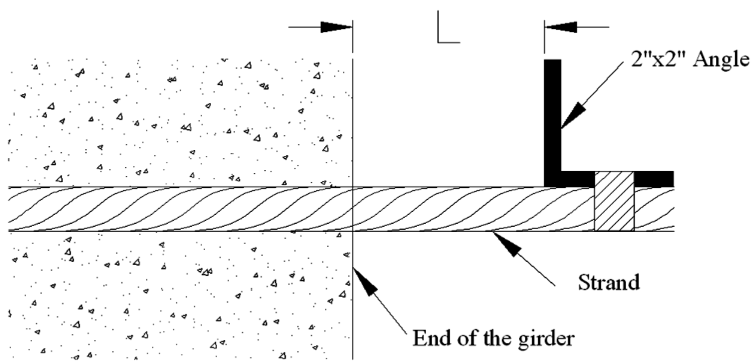


Figure 68 Strand drawn-in measurement setup

Figure 69 Strand drawn-in measurement setup

7.2.1.3 Camber Measurement

In order to determine the camber in the girder, a wire was anchored at one end and on the other end a pulley was set up. A set of weights was attached to the wire in order to ensure a constant tension. A ruler was affixed at the midspan of the girder directly behind the wire to read the camber. In order to prevent the parallax error during the reading process a mirror was attached directly behind the wire as shown in Figure 70 and Figure 71.The values of the camber measurements are shown in Table 19.

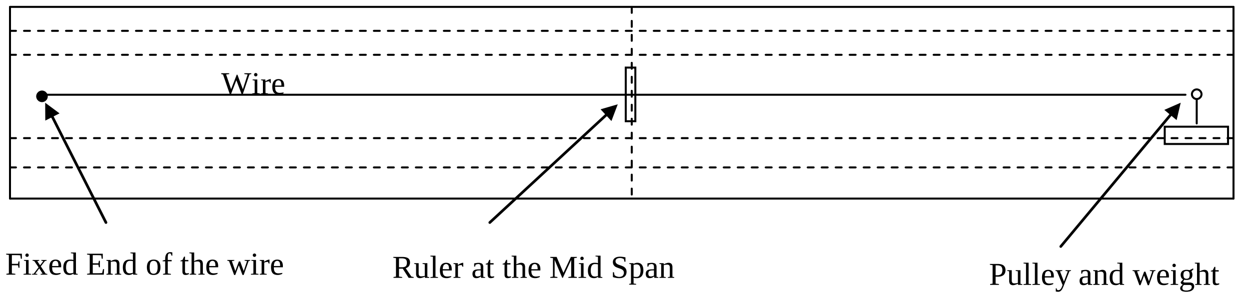


Figure 70 Deflection measurement setup at mid span

Figure 71 Deflection measurement setup at mid span

Table 19 Camber Measurements

	Camber, inches	
	0.7”(270 ksi) strand	**0.62”(330 ksi) strand**
Detension step 1	0.1	0.1
Detension step 2	0.5	0.6
Detension step 2	0.9	0.8
Detension step 2	1.1	1.0
At Transfer	1.6	1.5
24 Hrs from detension	2.4	1.9
3 Days from detension	2.8	2.2
7 Days from detension	2.8	2.2
14 Days from detension	2.8	2.2
28 Days from detension	2.8	2.2

7.2.2 Data reduction and determining transfer length

The measurements obtained using the DEMEC gauge were the actual distance between the insert points and not the strain. To convert these measurements into transfer length the following procedure is followed.

First, the strain values are obtained by subtracting the reading taken before detensioning from the reading taken after the prestress transfer and divided by the gauge length of the DEMEC gauge. Then the strains for corresponding sets of inserts on each side of a girder were averaged.

In the second step smoothed strain profile is obtained by using a floating 3-point average strain values for each girder ends as shown in Figure 72. The smoothing technique will lessen the scatter and reduce the effect of data points that have values higher or lower than the average value. By smoothing the data it is easier to define the plateau at which the constant strain in the girder is established. The floating 3-point average is obtained using the Eqn. given below,

$$\varepsilon_{i,smoothed} = \frac{\varepsilon_{i-1} + \varepsilon_i + \varepsilon_{i+1}}{3}$$

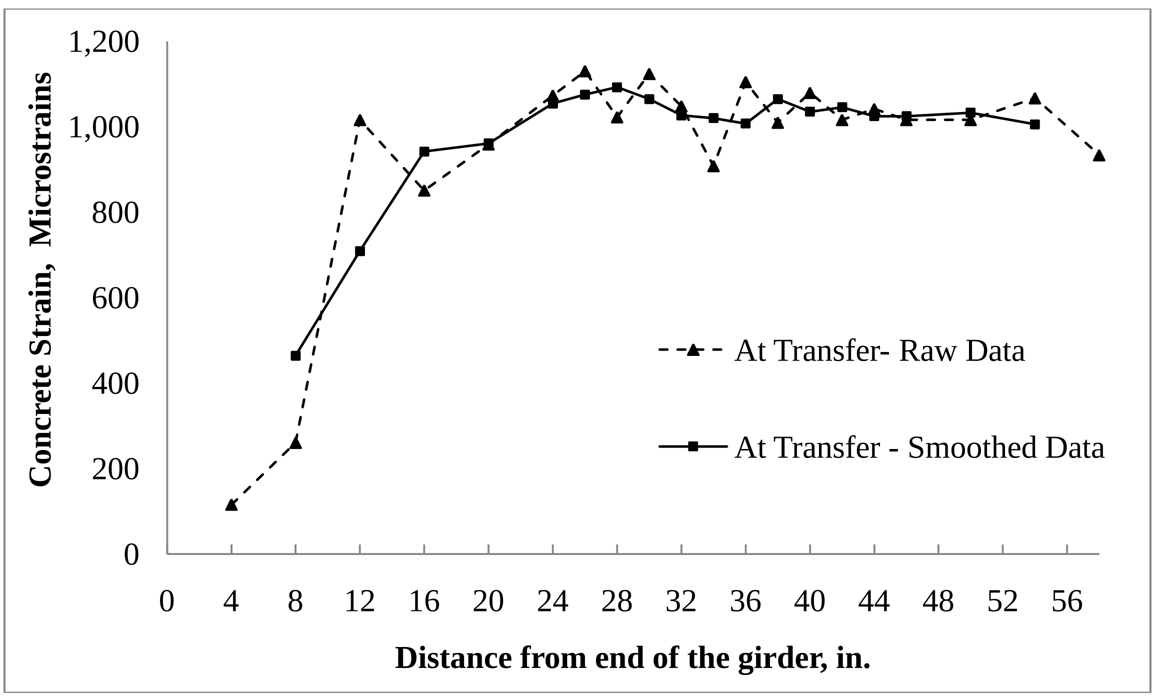


Figure 72 Raw and smoothed concrete surface strain data

In the third step the actual transfer length is determined based on different measurements such as the 95% Average maximum method, Slope Intercept, Strand Drawn-In, Final Average Method – Cousins, et al. (1993). In this experimental program 95% Average maximum method was used to determine the transfer length.

7.2.2.1 95% Average Maximum Strain Method – Russell and Burns (1993)

In this method to determine the transfer length, the point at which the strain is constant is determined. This is a subjective determination based on the strain profile. Once the initial point is determined the average maximum strain (AMS) is determined by taking the average of all the data points following the initial point. Then the 95% of the AMS value is taken and a horizontal line is plotted along with the strain profile graph. Thus the first intersection point of the horizontal line and the strain profile give the transfer length for the end of the girder as shown in Figure 73. The 95% AMS plot consists of separate regions, the initial linearly varying portion and the constant strain plateau and the resulting 95% AMS line.

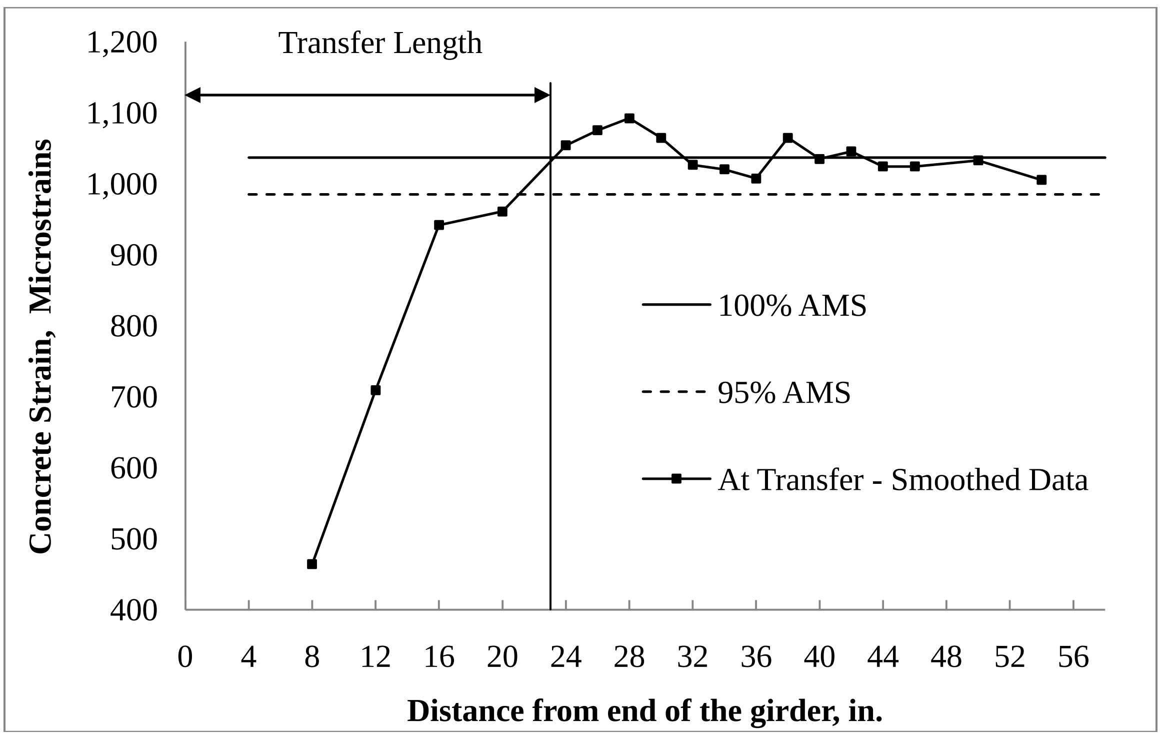


Figure 73 95% Average maximum strain method

7.2.2.2 Strand Drawn-In

The transfer length can be determined using the Drawn-in measurements from the strand. A theoretical relationship that relates the transfer length as a function of strand drawn-in was used. The equation was derived from a mechanistic relationship integrating steel strain over the transfer length and subtracting the concrete strains over that same length. Assuming linear variation of steel and concrete strains within the transfer zone yields the expression: [7]

$$L_t = \frac{2E_{ps}}{f_{si}}(L_{es})$$

In this equation L_t is the transfer length, E_{ps} is the elastic modulus of the steel strand, f_{si} is the strand stress immediately before transfer, and L_{es} is the measured strand drawn-in or end slip.

Due to the high prestressing force, the strands unwound during the detensioning process. Thus the readings obtained from some strands were distorted and could not be used for the calculation of the transfer length in both the specimens. In case of 0.7 in. strands due to the variation of the strand drawn in values the obtained transfer length was shorter than the value obtained using the concrete surface strain values. In case of 0.62 in. strands the values were very close to the values obtained using the concrete surface strain. The values of the transfer length based on the strand drawn-in measurement are shown in Table 20.

Table 20 Transfer Length based on strand Drawn-in

Specimen	**Transfer Length, in.**					
	Maximum		**Average**		**Minimum**	
	L End	R End	L End	R End	L End	R End
0.7 in.*	15.50	21.90	**12.87**	**15.66**	10.24	9.81
0.62 in.	34.39	34.71	**31.87**	**29.42**	28.21	21.32

**Certain strand readings were destroyed*

7.3 Results and Discussions

7.3.1 Transfer Length – 0.7" strand

The measured and calculated values of transfer length for each end of the specimen 1 at various days after release are shown in Table 21. Curves of individual girder end transfer lengths for different steps of detension are shown in Figure 74 and Figure 77.

The average value of transfer length obtained for the 0.7" diameter strand is about half the length obtained using the equations provided by AASHTO LRFD 2008 and ACI 318-08.The transfer lengths obtained from both the ends of the girder show very limited difference as shown in Figure 75and Figure 78. This shows that the two different designs had very minimum affect on the transfer length. Plots of individual girder end transfer lengths for different days after release are shown in Figure 76 and Figure 79.

Since there is very limited research on 0.7 in. diameter strand in the past the values obtained from this research program could not be compared with any other researcher's values.

Table 21 Transfer length for Specimen 1

	Transfer Length, inches	
	Specimen 1 (0.7")	
	Side L	**Side R**
At Transfer	21.04	21.50
AASHTO LRFD 2008	42	
ACI 318-08	38	

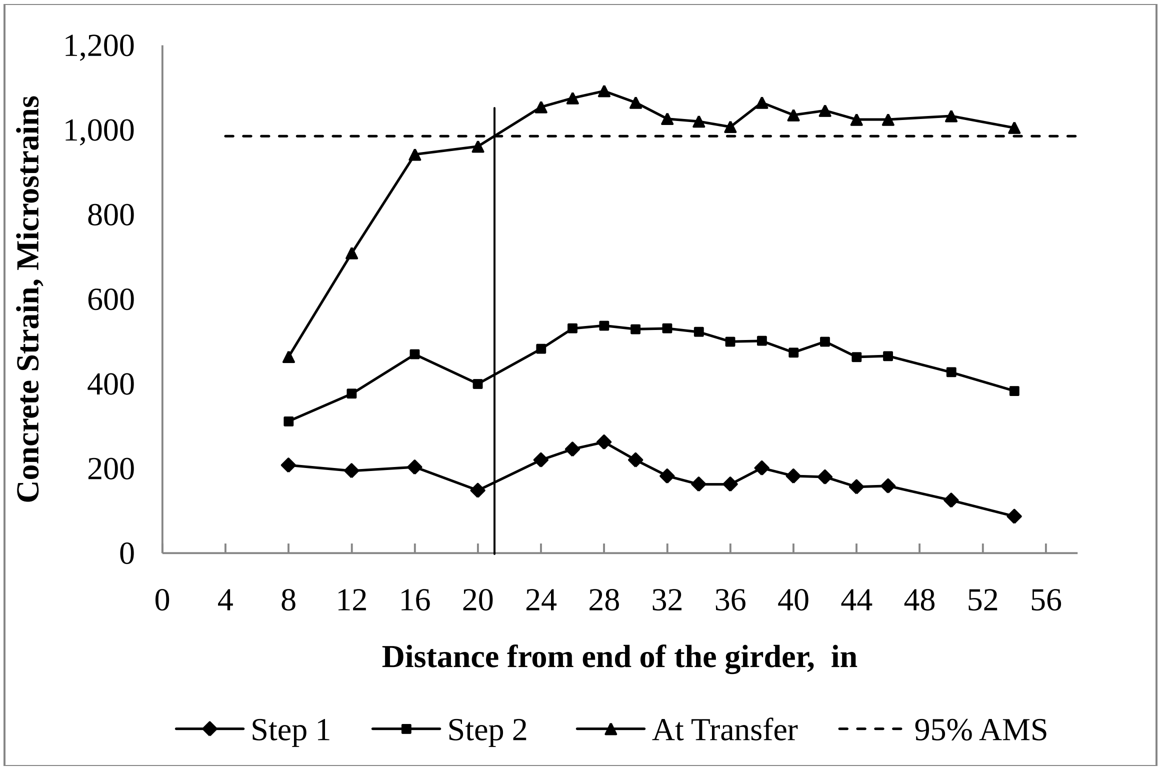


Figure 74 Development of prestressing force during detension in Specimen 1-End L

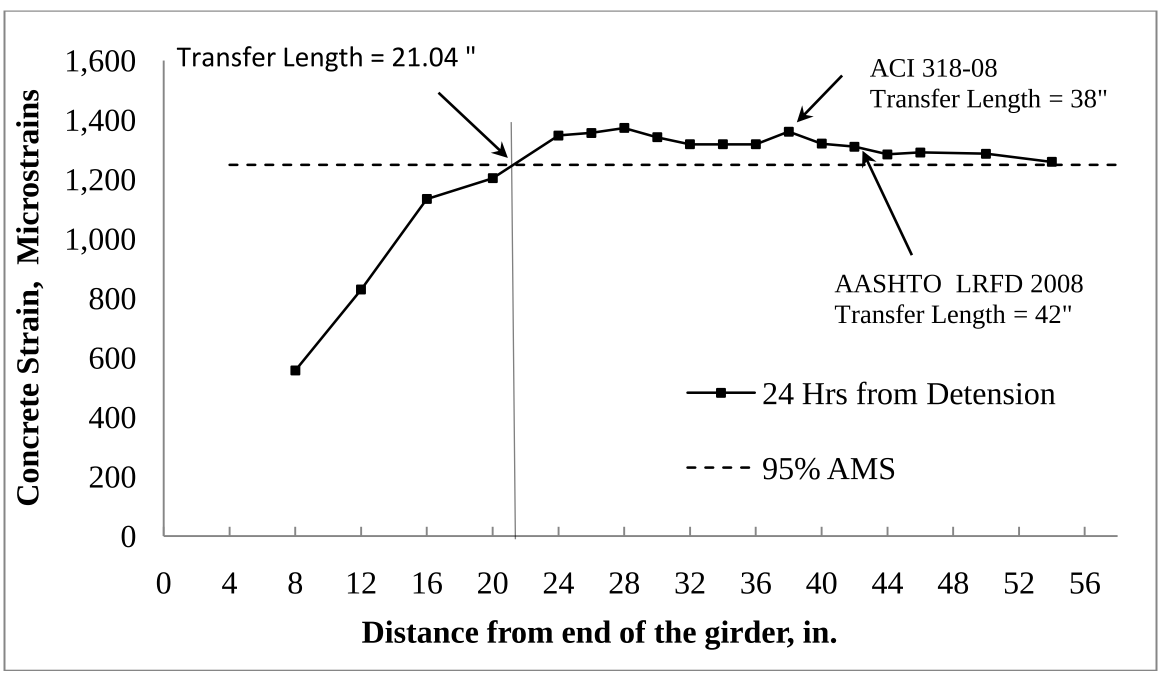


Figure 75 Strain distribution for specimen 1 – End L at transfer

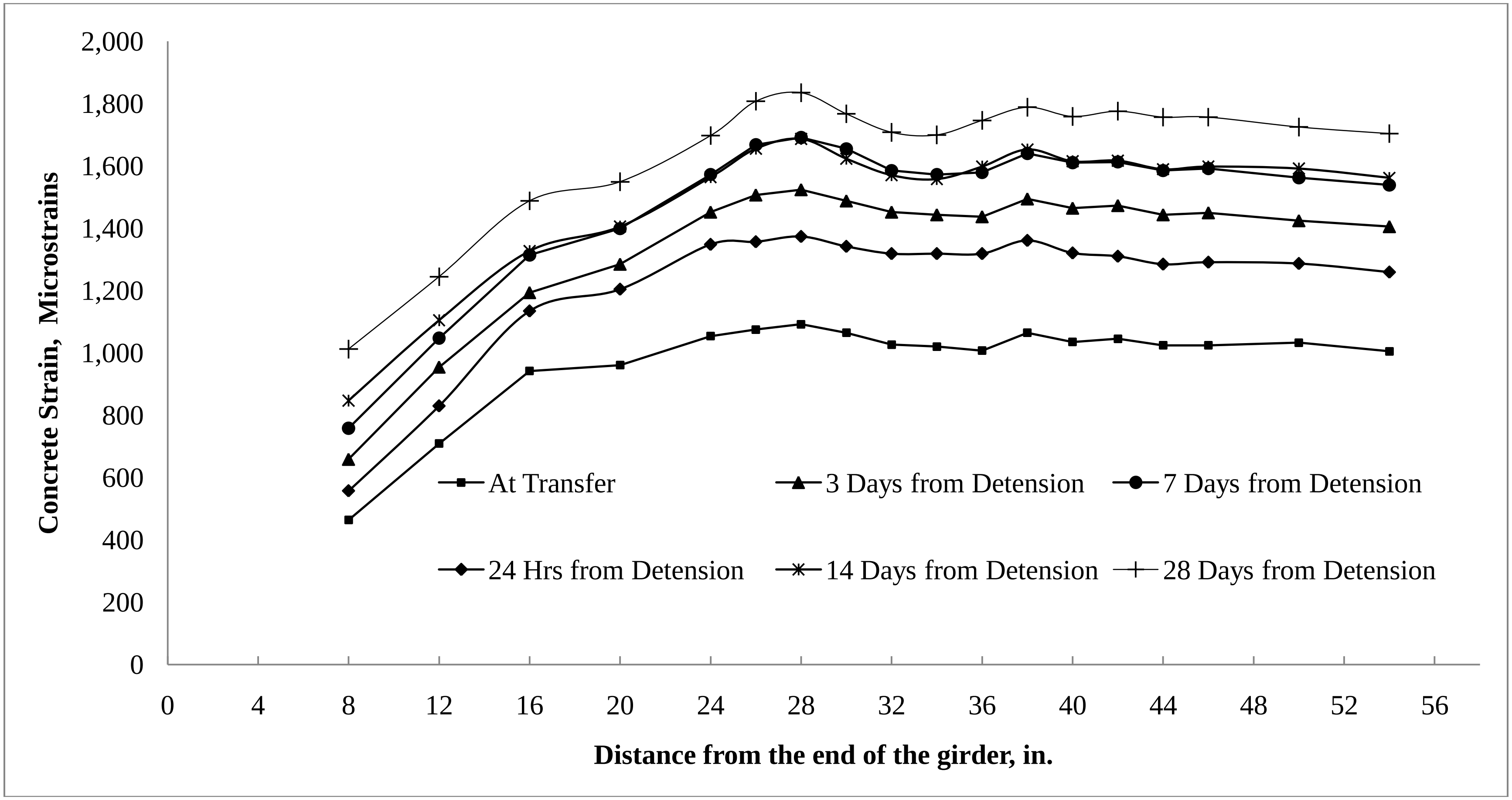


Figure 76 Strain distribution for specimen 1 – End L

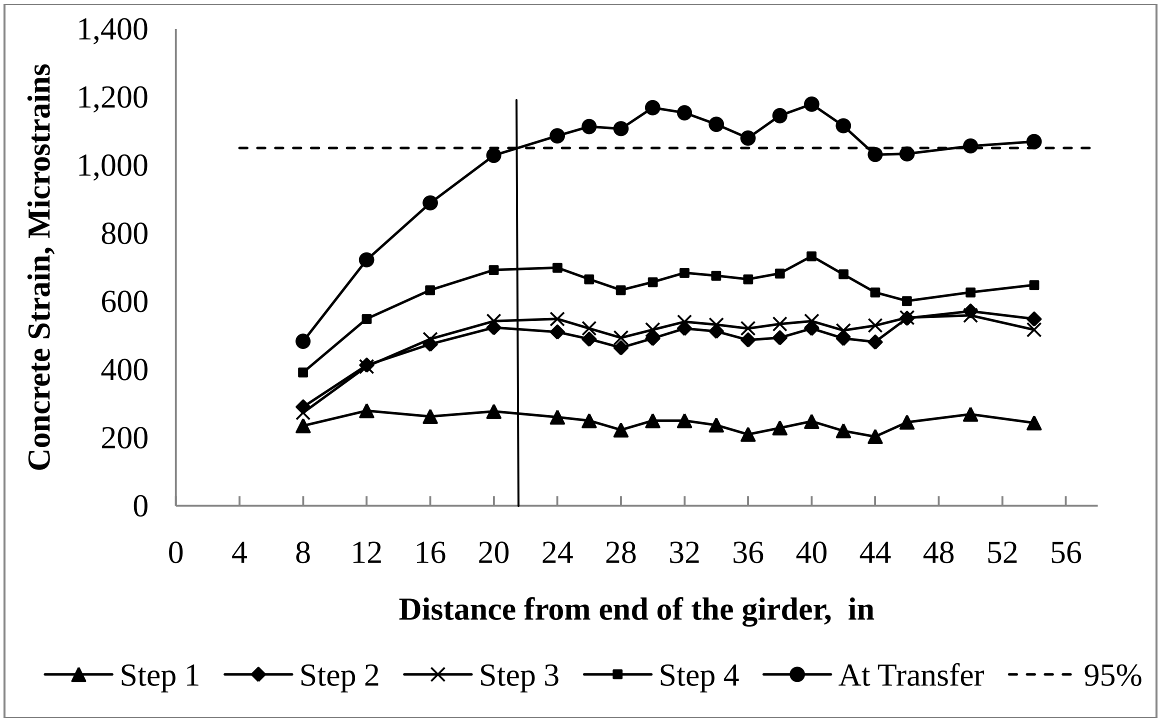


Figure 77 Development of prestressing force during detension in Specimen 1–End R

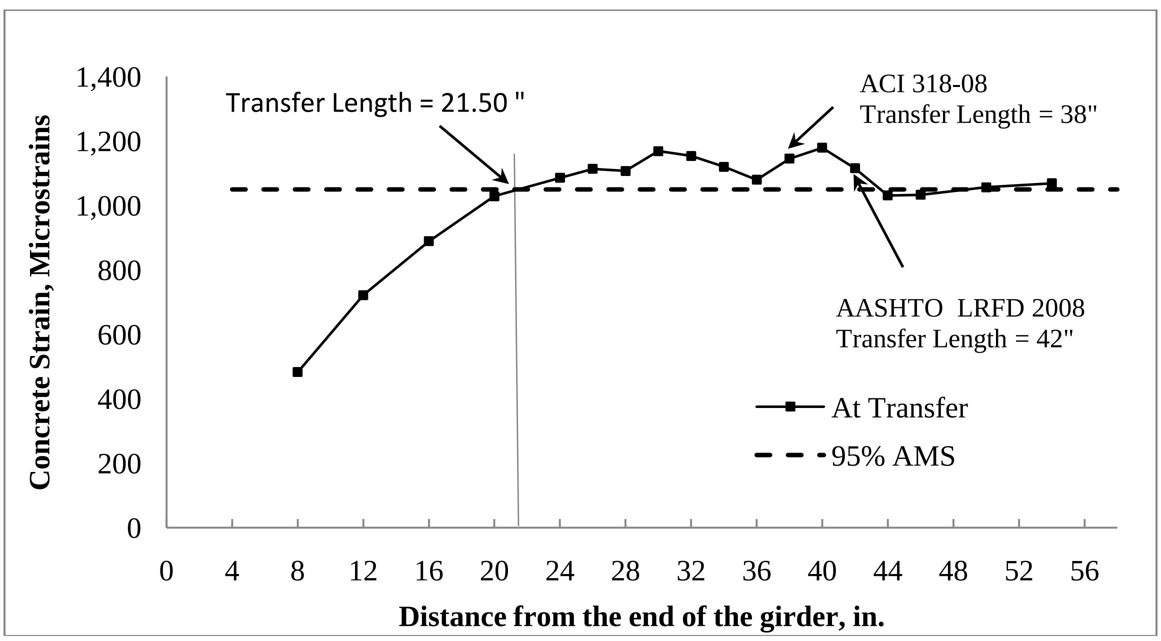


Figure 78 Strain distribution for specimen 1 – End R at transfer

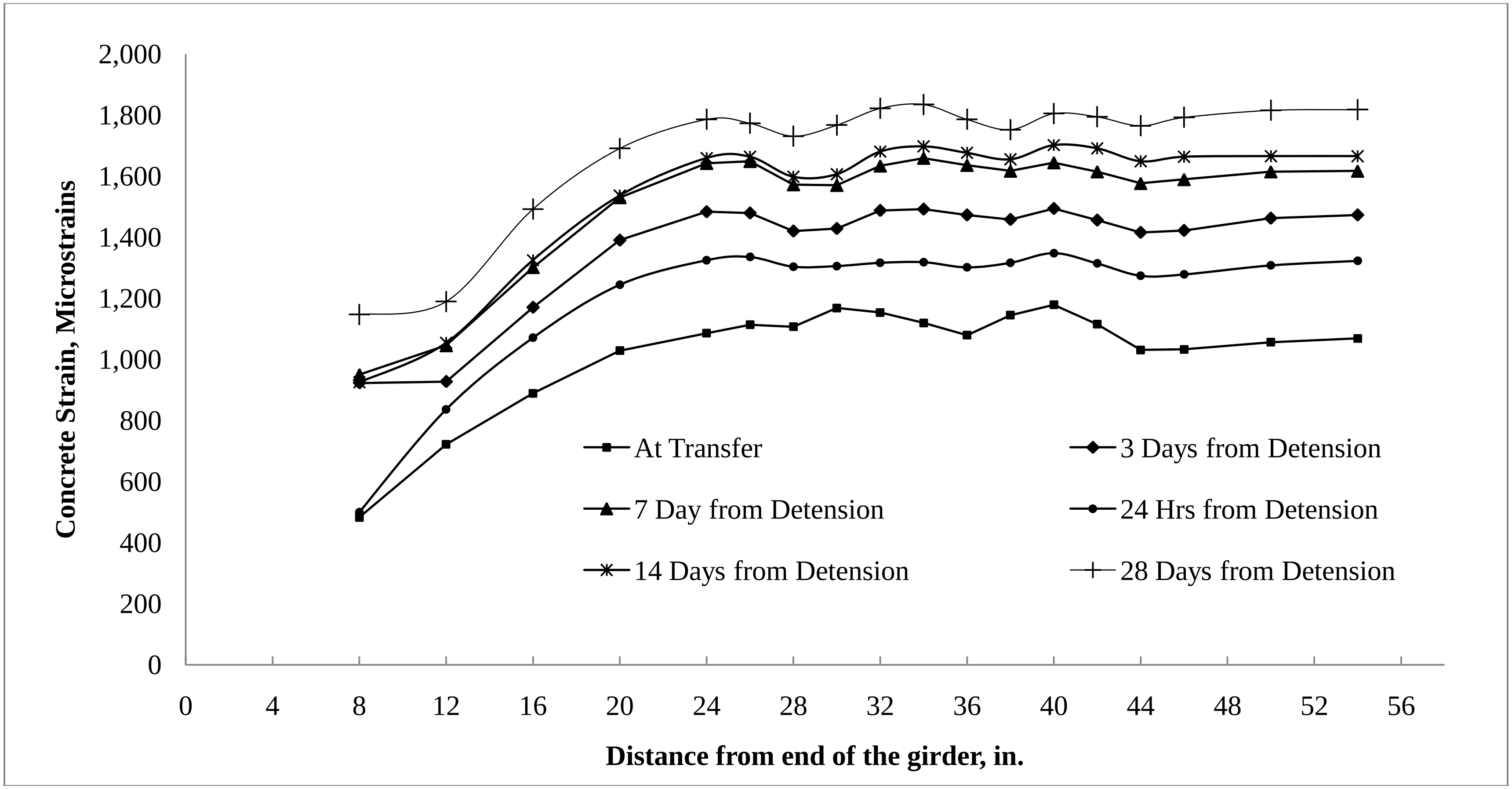


Figure 79 Strain distribution for specimen 1 – End R

7.3.2 Transfer Length – 0.62" strand

The measured and calculated values of transfer length for each end of the specimen 2 at various days after release are shown in Table 22. Curves of individual girder end transfer lengths for different steps of detension are shown in Figure 80 and Figure 83.

In case of 0.62 in. diameter strands the value of transfer length obtained using the ACI and AASHTO equations are conservative when compared to the experimental values. The average transfer length for 0.6 in. strand is 40.0 in. (Russell & Burns, 1996) where as the average value obtained in this research program is 29.14 in. Since the strand which was used had a grade of 330 ksi, as per Russell & Burns the average 40.0 in. is good for 0.6 in. strand with a grade of 270 ksi. The transfer lengths obtained from both the ends of the girder show very limited difference as shown in Figure 81and Figure 84. Plots of individual girder end transfer lengths for different days after release are shown in Figure 82 and Figure 85.

Table 22 Transfer Length for Specimen 2

	Transfer Length, inches	
	Specimen 1 (0.62")	
	Side L	**Side R**
At Transfer	28.10	28.19
AASHTO LRFD 2008	37.2	
ACI 318-08	41	

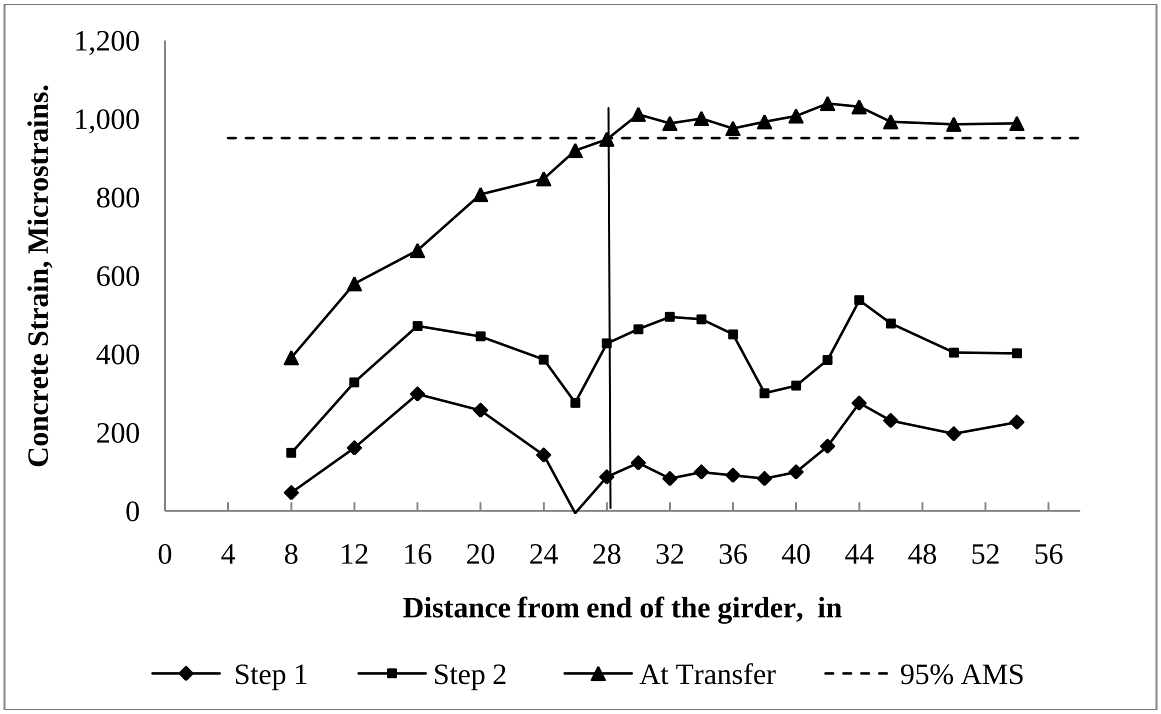


Figure 80 Development of prestressing force during detension in Specimen 2-End L

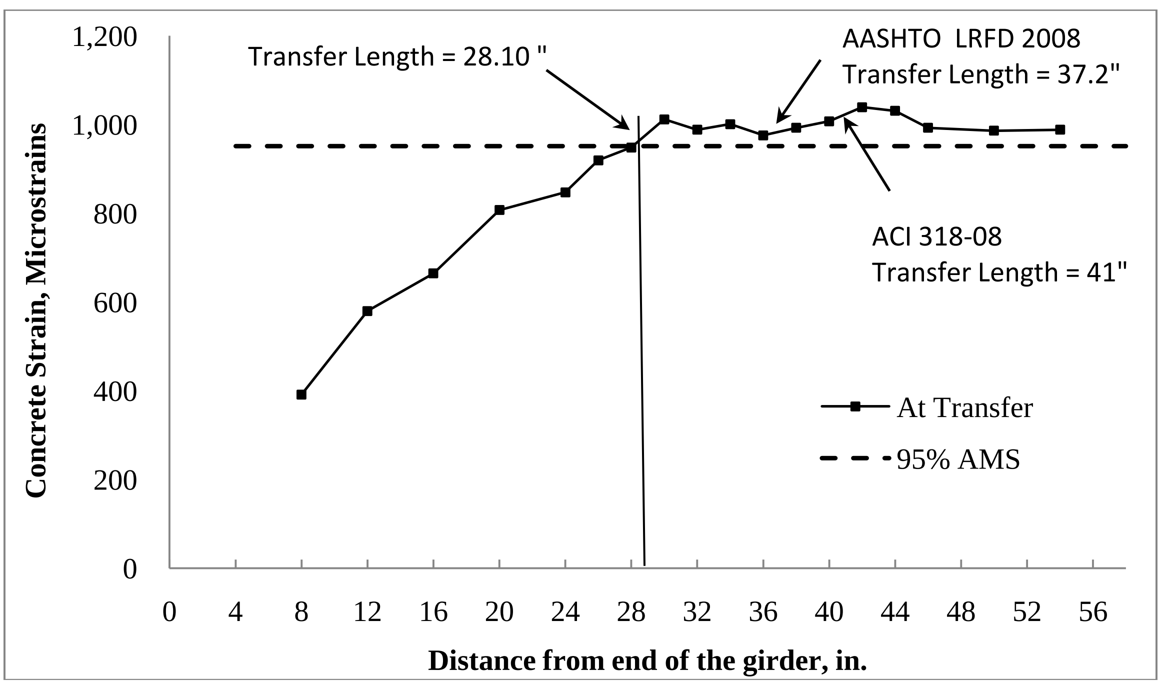


Figure 81 Strain distribution for specimen 2 – End L at transfer

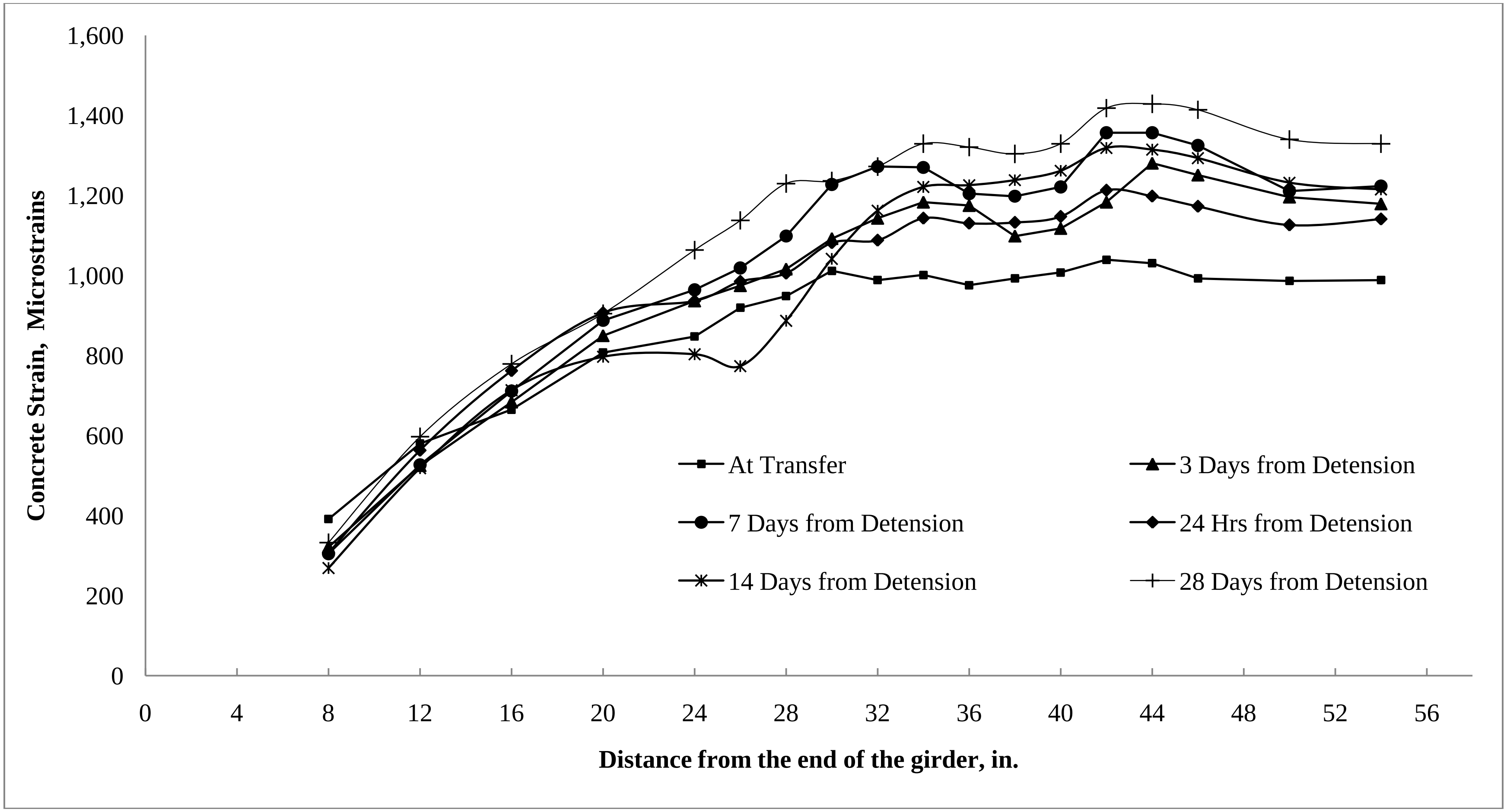


Figure 82 Strain distribution for specimen 2 – End L

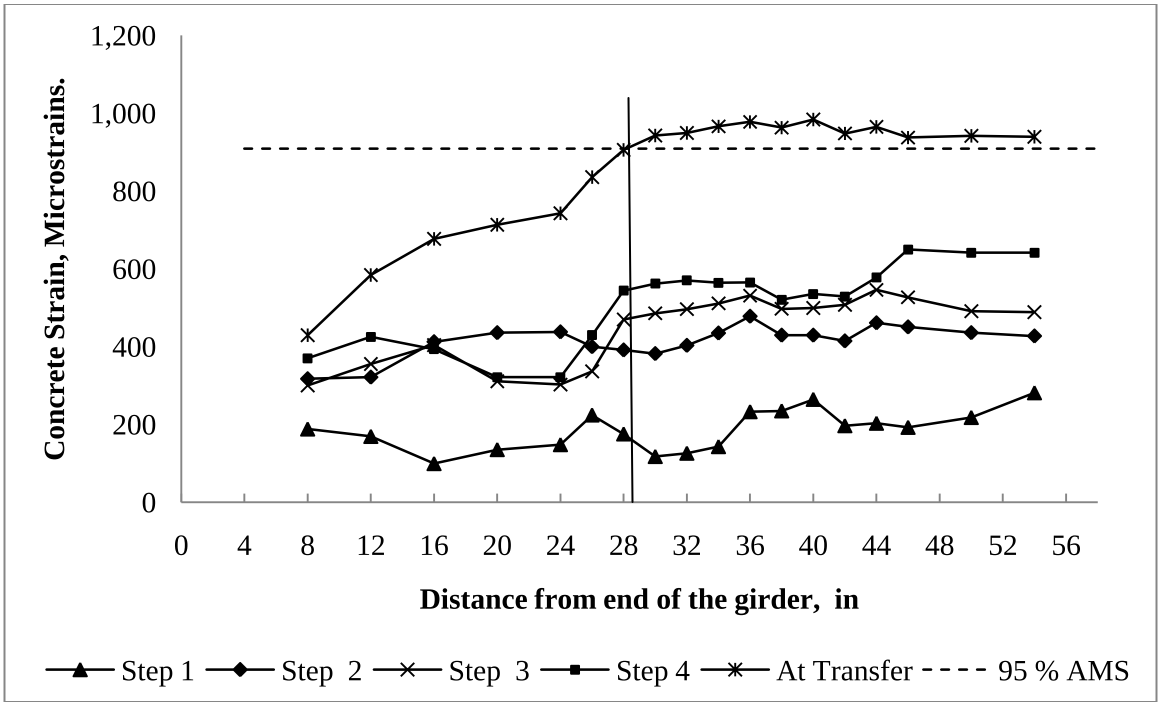


Figure 83 Development of prestressing force during detension in Specimen 2-End L

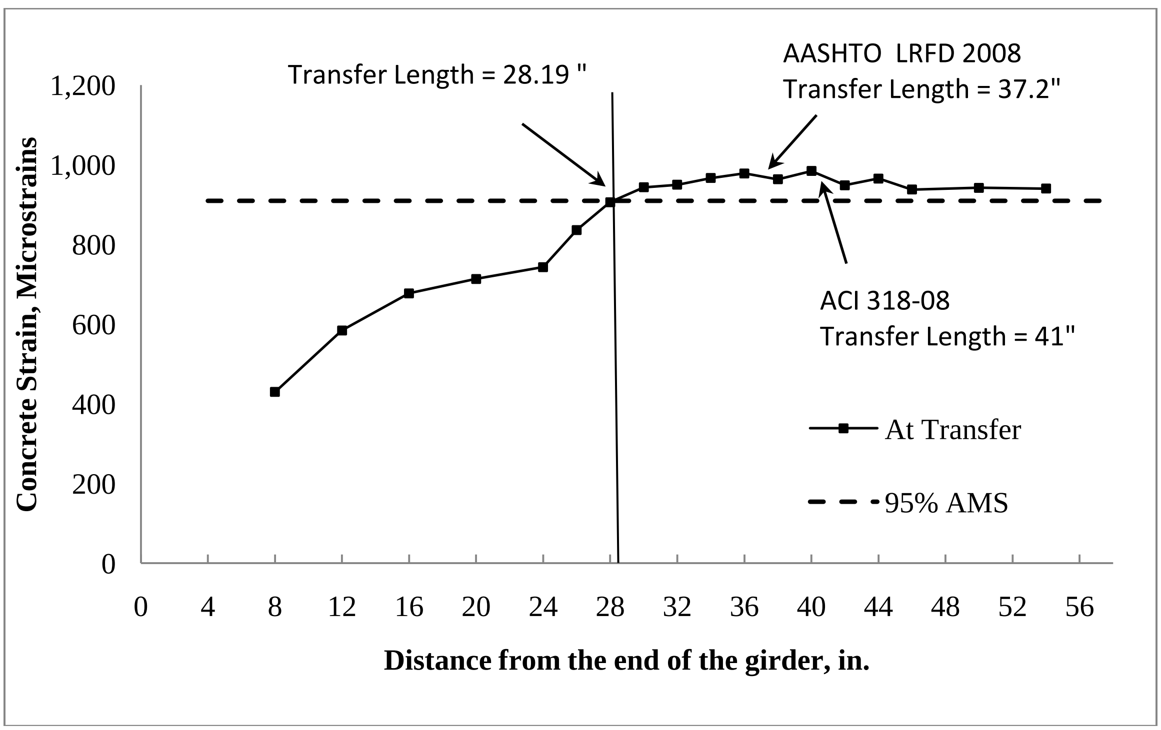


Figure 84 Strain distribution for specimen 2 – End R at transfer

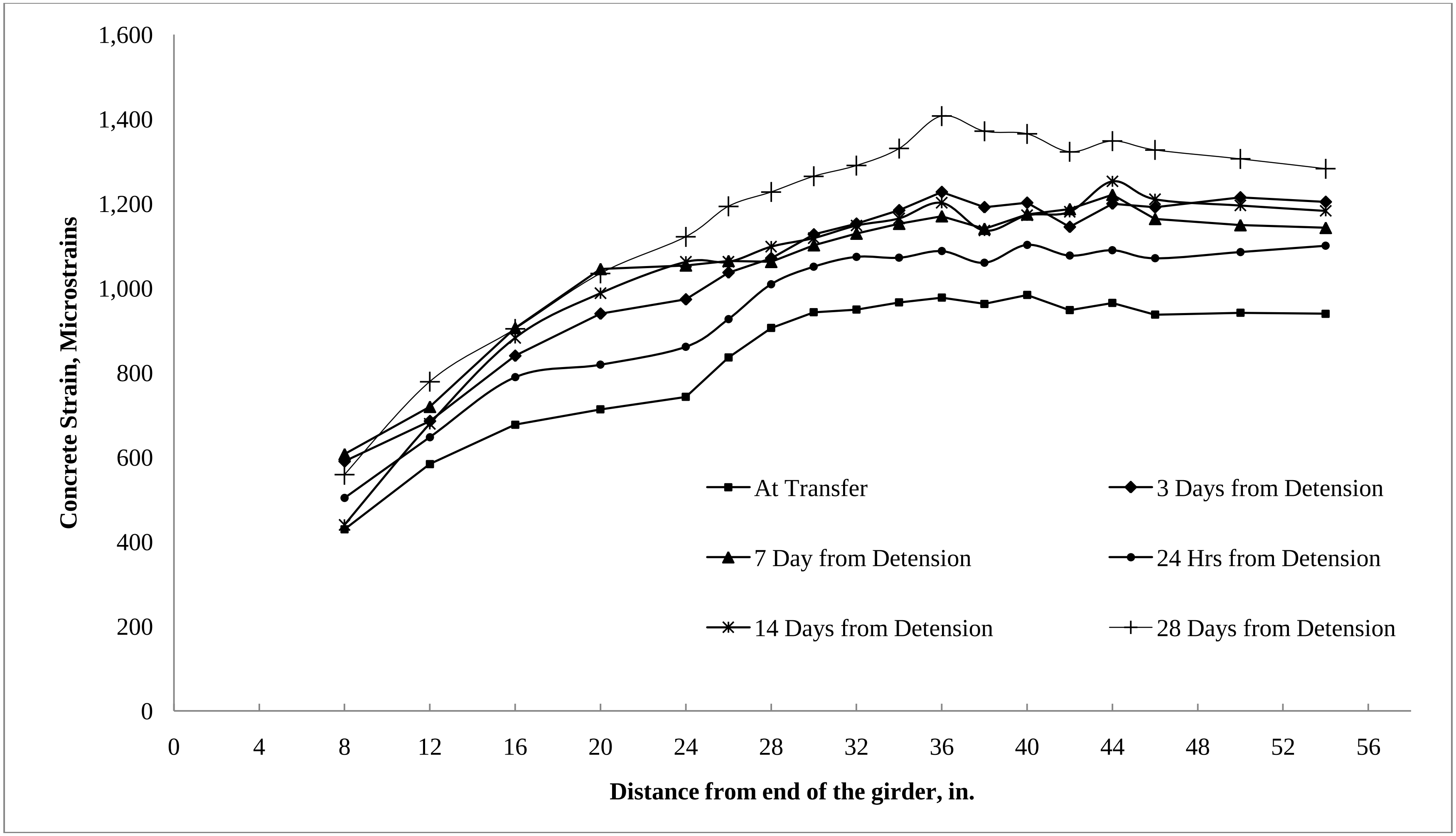


Figure 85 Strain distribution for specimen 2 – End R

7.3.3 Prestress Losses

DEMEC gauge points were placed at the mid span of the girder to measure the prestress loss due to the elastic shortening of the member and long term losses. The loss due to relaxation of the steel is not taken into account in this experimental investigation. The stress in the concrete at the various DEMEC point locations is obtained based on the measured concrete surface strain and the concrete strength at the time of measurement of this strain. Thus the following equation is used in the calculation of the elastic shortening loss of the girder:

$$\Delta \boldsymbol{f}_{ES} = \frac{\boldsymbol{E}_{ps}}{\boldsymbol{E}_{ci}} \boldsymbol{f}_{cgp}$$

Where $\Delta \boldsymbol{f}_{ES}$ loss due to elastic shortening, $\boldsymbol{E}_{ps}$ is the modulus of elasticity of prestressing strand, $\boldsymbol{E}_{ci}$ is the modulus of elasticity of concrete and $\boldsymbol{f}_{cgp}$ concrete stress at the centroid of the strand.

Table 23 and Table 24 show the development of prestress loss over time for both the girders. Table 25 shows the values of calculated prestress losses for both the specimens. The prestress loss was calculated by the AASHTO LRFD 2008 design specification for both the girders.

Table 23 Measured prestress loss at mid section for the specimen with 0.7" strand

	At transfer – Elastic loss	**24 hrs**	**3 day**	**7 day**	**14 day**	**28 day**
Concrete Strength, ksi	10.25	10.93	10.44	11.79	12.37	14.19
Elastic Modulus, ksi	6138	6338	6194	6583	6744	7222
Stress at mid span section, Ksi	5.44	6.64	7.30	8.59	9.12	10.41
Prestress loss, Ksi	25.51	30.18	33.92	37.58	38.95	41.51
Prestress loss, %	12.60	14.90	16.75	18.56	19.24	20.50

Table 24 Measured prestress loss at mid section for the specimen with 0.62" strand

	At transfer – Elastic loss	24 hrs	3 day	7 day	14 day	28 day
Concrete Strength, ksi	11.59	11.62	11.72	11.88	12.25	12.29
Elastic Modulus, ksi	6527	6535	6564	6607	6711	6722
Stress at mid span section, Ksi	5.04	5.50	5.79	5.89	6.16	6.66
Prestress loss, Ksi	21.99	23.98	25.16	25.43	26.15	28.23
Prestress loss, %	8.88	9.69	10.16	10.27	10.57	11.41

Table 25 Calculated prestress loss

Specimen 1 - 0.7"(270 ksi)				**Specimen 2 - 0.62" (330 ksi)**			
Elastic shortening loss		Total prestress loss		Elastic shortening loss		Total prestress loss	
ksi	%	ksi	%	ksi	%	ksi	%
19.59	10	39.19	19.35	21.99	8.88	36.73	14.84

The time-dependent prestress losses exhibit the same trend as elastic losses, with increase in losses from increased concrete stress. The measured elastic loss is underestimated over the predicted value in both specimens. The long-term losses are monitored in the future.

7.3.4 Cracking

The cracking in the concrete showed that end designed based on AASHTO requirements experienced more cracking than the end which was designed based on strut-and-tie modeling. The observed cracks were found within a distance of 2 in. from the very end of the girder.

7.3.4.1 Girder with 0.7 in. (270 ksi) diameter strand

There was significantly no cracking due to the splitting force at the transfer zone of the girder. The cracking which was observed was in the very bottom portion of the section which is influenced by the amount of confinement reinforcement close to the very end of the girder. The cracking of both the ends of the girder are shown in Figure 86 and Figure 87.

Figure 86 End R of specimen 1 designed based on AASHTO LRFD 2008

Figure 87 End L of specimen 1 designed based on Strut-and-Tie Modeling

7.3.4.2 Girder with 0.62 in. (330 ksi) diameter strand

The R end of specimen 2 had additional confinement reinforcement to enclose all the 12 strands as shown in Figure 88. There was a variation in the cracking pattern on both sides of the girder in the same end as shown in Figure 89 and Figure 90. This might be due to unsymmetrical detensioning of the strands, which was done to see the effect on the transfer length.

Figure 88 Reinforcement detailing for End R of specimen 2

Figure 89 Both sides of End R of specimen 2 designed based on AASHTO LRFD 2008

As per results of the finite element analysis it was concluded that there was a stress concentration at the transition zone between the web and the bottom flange of the I-girder. After the specimens were detensioned a hairline crack was observed in both the specimens where the stress concentration was found. This could be due to the high prestressing force and the larger eccentricity of the force.

Figure 90 End L of specimen 2 designed based on Strut-and-Tie Modeling

CHAPTER 8

Conclusion

Based on the analysis of the experimental results, the following conclusions for specimens prestressed with 0.7” and 0.62” diameter strand can be drawn:

8.1 Transfer length

The only variables which were considered in this research program were the diameter of the strands and the grade of the strand steel. All other variables such as spacing of the strands, concrete strength, confinement reinforcement, splitting reinforcement and surface condition of the strand were kept constant for both the specimens.

The current ACI and AASHTO equations overestimate the value of transfer length in case of 0.7 in. strands and cannot be used in case of larger diameter strands. In case of 0.62 in. strands the values obtained are less conservative than the values obtained using ACI and AASHTO equations.

8.2 Strand Spacing

There was no cracking of the girders due to the insufficient spacing of the strands found at transfer. It can also be said that due to the shorter transfer length obtained for the 0.7 in. strand, a lager splitting stress would be introduced at the transfer zone and cause a higher probability of cracking at transfer. Thus using 2 in. spacing in both directions for 0.7 in. and 0.62 in. strands did not show any signs of splitting of the members at the end zones.

8.3 Strand Diameter

The perimeter of seven-wire prestressing strand is approximately equal to $4/3\pi d_b$. Adhesion force, which is directly proportional to the amount of adhered surface, is therefore directly proportional to the strand diameter. Friction may be affected by the strand diameter due to the difference in the nominal force from different wire sizes. Because the grooves between the outer wires get larger with increasing strand diameter, mechanical bond strength would tend to increase with strand diameter[5]. As the diameter of the strand increases the value of transfer length tends to decrease. Thus for both 0.7 in. strand and 0.62 in. strand the obtained transfer length values, 21.6 in. and 29.8 in., are shorter than 40 in. for 0.6 in. strand as given by Russell & Burns.

8.4 Confinement reinforcement

Due to the high prestressing force in both cases, the confinement reinforcement design plays a vital role. It was observed that the ends which had the detailing of AASHTO required confinement had more cracks than the end designed based on the Strut-and-Tie modeling.

The cracks which were observed in both specimens occurred within a distance of 2 in. from the end of the girder. This shows a need to provide a large area of confinement reinforcement as close to the member end as possible. The ends with the details based on the Strut-and-Tie modeling were more efficient than the ends with the details based on the AASHTO recommendations.

Future Research

- The analytical study showed that there is a high probability of cracking at the transition zone between the bottom flange and the web for the 0.7 inch diameter strands when compared with the 0.6 inch diameter strands.
- Further analytical study should be performed in order to determine the effects of the confinement steel for both 0.7 inch diameter strands and 0.62 inch diameter strands.
- Development length should be determined for both 0.7 in. and 0.62 in. strands by applying transverse load.

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Appendix

Tennessee Department of Transportation Detailing

(Based on drawings provided by Ross Prestressed Concrete, Inc.)

- AASHTO Type II Girder
- Span = 60 ft
- Girder Concrete:
    - f'_{ci} (At Transfer) = 8 ksi
    - f'_c (At Service) = 8.5 ksi

- Number of Strand = 33
- Diameter of the Strand = **0.5** inch
- Cross sectional area of the strand = **0.153** in^2
- Ultimate strength of strand, f_{pu} = **270** ksi
- Spacing of strands = 2" x 2"
- Force per strand = (1.0)(0.153)(0.75)(270) = 30.98 kips

<u>Shear Reinforcement</u>

- 12 Double legged #5 bar @ 6" spacing for 66"
- 13 Double legged #5 bar @ 9" spacing for 108"
- 14 Double legged #5 bar @ 12" spacing for 156"

<u>Horizontal Reinforcement</u>

3 Nos. of #5 bars provides for a distance of 60 inches from the end of the girder.

<u>Top flange reinforcement</u>

2 Nos. of #7 bars provided for the entire length of the girder.

<u>Anchorage Zone Reinforcement</u>

Splitting resistance reinforcement

The splitting reinforcement consists of 6 pairs of #5 bars spaced at 3", provided for a distance of 18 inches from the end of the girder. Two bars are projected above the top flange of the girder and the four bars bent in the top flange of the girder.

Confinement reinforcement

The confinement reinforcement provided in the TDOT details consists of 5 #3 bars spaced at 6 inches starting at 4.5" from the end of the girder.

APPROVED AS NOTED

REV. NO.	DATE	BY	DESCRIPTION

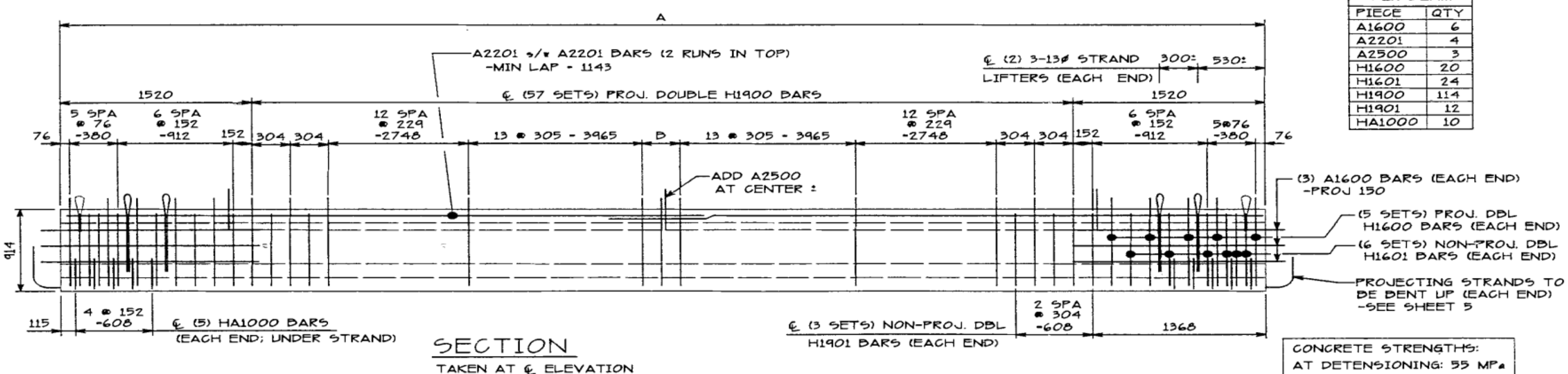
~~SEP 12 2008~~

Edward P. Wasserman
ENGR. DIRECTOR, STRUCTURES

Add I6A

DIMENSION TABLE		
BEAM	A	B
E3	18332	2 SPA @ 325 - 650
I5	18332	2 SPA @ 325 - 650
I6	18358	2 SPA @ 338 - 676
I7	18392	2 SPA @ 355 - 710
E4	18392	2 SPA @ 355 - 710

BILL OF MATERIAL PER BEAM	
PIECE	QTY
A1600	6
A2201	4
A2500	3
H1600	20
H1601	24
H1900	114
H1901	12
HA1000	10



SECTION
TAKEN AT ℄ ELEVATION

SEE SHEET 5 FOR A2500 BAR DETAIL

CONCRETE STRENGTHS:
AT DETENSIONING: 55 MPa
AT 28 DAYS: 59 MPa

SPAN 2

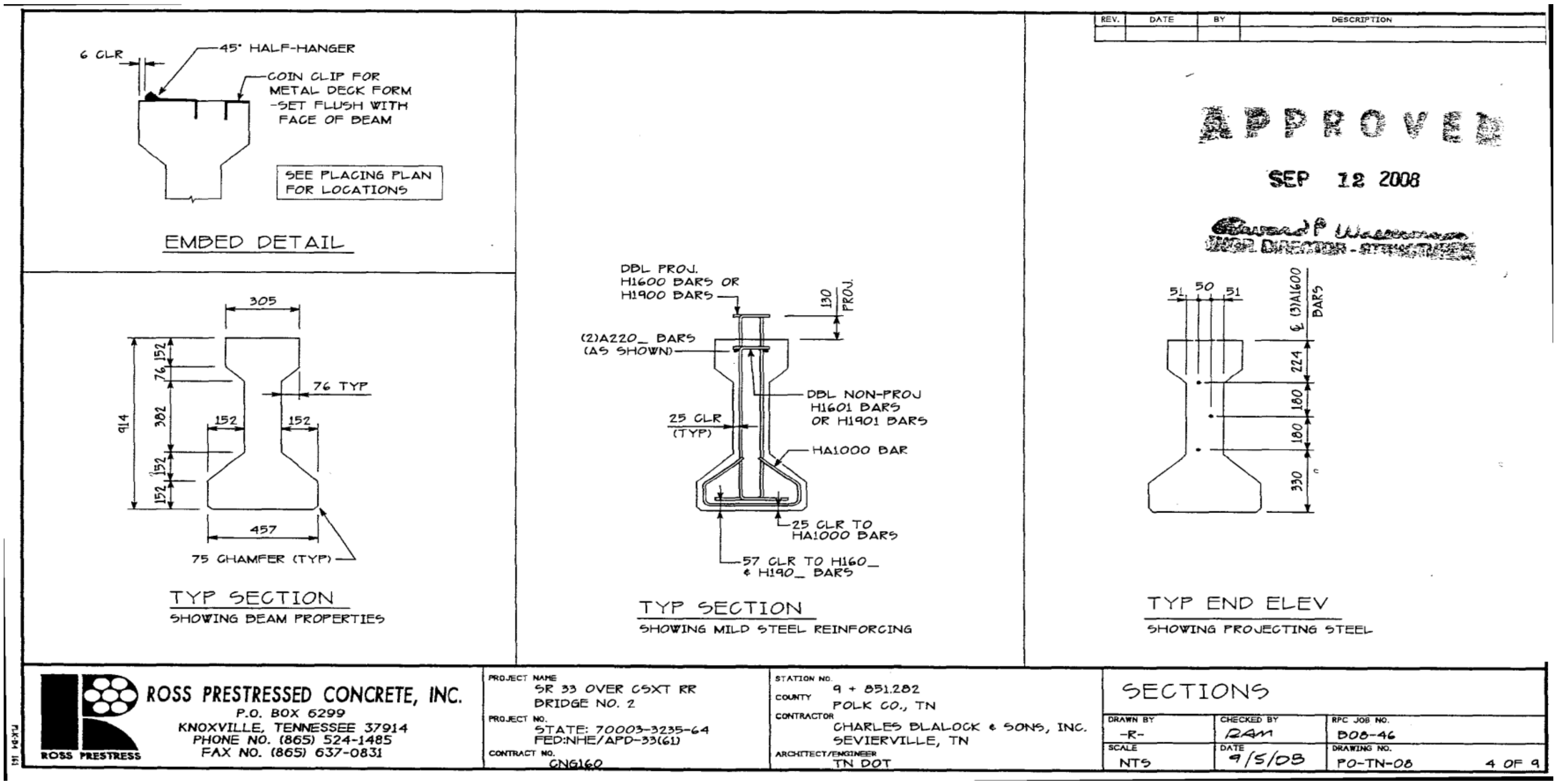
ROSS PRESTRESSED CONCRETE, INC.
P.O. BOX 6299
KNOXVILLE, TENNESSEE 37914
PHONE NO. (865) 524-1485
FAX NO. (865) 637-0831
ROSS PRESTRESS

PROJECT NAME: SR 33 OVER CSXT RR BRIDGE NO. 2
PROJECT NO.: STATE: 70003-3235-64 FED:NHE/APD-33(61)
CONTRACT NO.: CNG160

STATION NO.: 9 + 851.282
COUNTY: POLK CO., TN
CONTRACTOR: CHARLES BLALOCK & SONS, INC. SEVIERVILLE, TN
ARCHITECT/ENGINEER: TN DOT

BEAM DETAILS: E3, I5, I6, I6A, I7, E4

DRAWN BY	CHECKED BY	RPC JOB NO.
-R-	RAM	B08-46
SCALE	DATE	DRAWING NO.
NTS	9/5/08	PO-TN-08 8 OF 9


6 CLR
45° HALF-HANGER
COIN CLIP FOR METAL DECK FORM -SET FLUSH WITH FACE OF BEAM
SEE PLACING PLAN FOR LOCATIONS
EMBED DETAIL
305
76 TYP
152
152
914
382
457
75 CHAMFER (TYP)
TYP SECTION
SHOWING BEAM PROPERTIES
DBL PROJ. H1600 BARS OR H1900 BARS
130 PROJ.
(2)A220_ BARS (AS SHOWN)
DBL NON-PROJ H1601 BARS OR H1901 BARS
25 CLR (TYP)
HA1000 BAR
25 CLR TO HA1000 BARS
57 CLR TO H160_ & H190_ BARS
TYP SECTION
SHOWING MILD STEEL REINFORCING
REV.
DATE
BY
DESCRIPTION
APPROVED
SEP 12 2008
51
50
51
℄ (3)A1600 BARS
224
180
180
330
TYP END ELEV
SHOWING PROJECTING STEEL
ROSS PRESTRESSED CONCRETE, INC.
P.O. BOX 6299
KNOXVILLE, TENNESSEE 37914
PHONE NO. (865) 524-1485
FAX NO. (865) 637-0831
ROSS PRESTRESS
PROJECT NAME
SR 33 OVER CSXT RR BRIDGE NO. 2
PROJECT NO.
STATE: 70003-3235-64 FED:NHE/APD-33(61)
CONTRACT NO.
CNG160
STATION NO.
9 + 851.282
COUNTY
POLK CO., TN
CONTRACTOR
CHARLES BLALOCK & SONS, INC. SEVIERVILLE, TN
ARCHITECT/ENGINEER
TN DOT
SECTIONS
DRAWN BY
-R-
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RPC JOB NO.
B08-46
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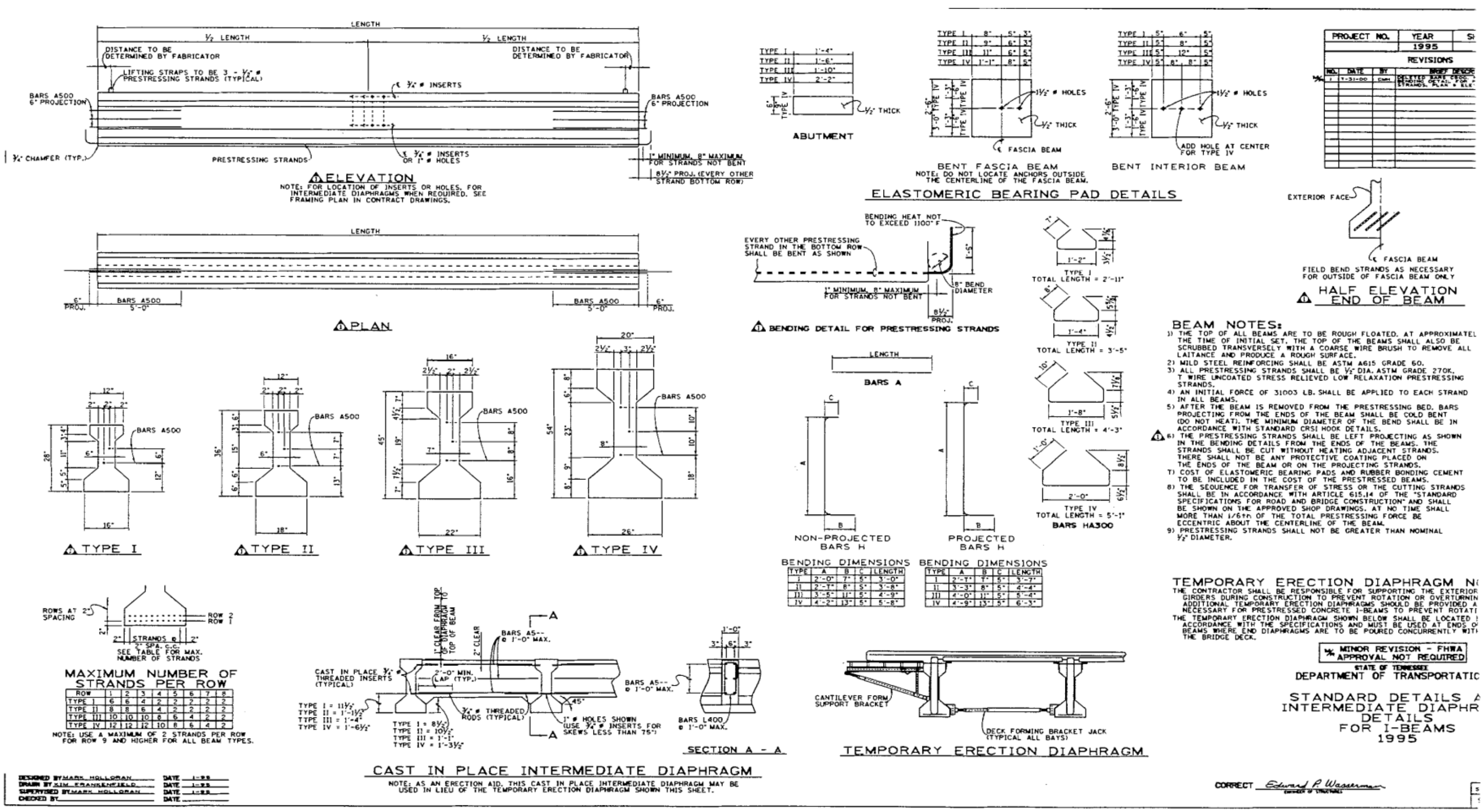
DATE
9/5/08
DRAWING NO.
PO-TN-08
4 OF 9


ELEVATION
NOTE: FOR LOCATION OF INSERTS OR HOLES, FOR INTERMEDIATE DIAPHRAGMS WHEN REQUIRED, SEE FRAMING PLAN IN CONTRACT DRAWINGS.
PLAN
TYPE I
TYPE II
TYPE III
TYPE IV
MAXIMUM NUMBER OF STRANDS PER ROW
CAST IN PLACE INTERMEDIATE DIAPHRAGM
SECTION A - A
ELASTOMERIC BEARING PAD DETAILS
ABUTMENT
BENT FASCIA BEAM
BENT INTERIOR BEAM
BENDING DETAIL FOR PRESTRESSING STRANDS
BARS A
NON-PROJECTED BARS H
PROJECTED BARS H
BENDING DIMENSIONS
BARS HA300
HALF ELEVATION END OF BEAM
BEAM NOTES:
TEMPORARY ERECTION DIAPHRAGM
MINOR REVISION - FHWA APPROVAL NOT REQUIRED
STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATIO
STANDARD DETAILS
INTERMEDIATE DIAPHR
DETAILS
FOR I-BEAMS
1995

Vita

Jayaprakash Vadivelu was born in Tamil Nadu, India on May 8, 1984. He was raised by his parents Vadivelu and Vijaya along with two siblings Jayachandran and Nithya in India. He did his schooling in India. In 2006, Jayaprakash graduated with his Bachelor of Engineering in Civil Engineering from Bannari Amman Institute of Technology (Affiliated to Anna University) in India. He is currently pursuing his Masters in Civil engineering with an emphasis in Structural Engineering at the University of Tennessee, Knoxville.