IMPACT OF LARGER DIAMETER STRANDS ON AASHTO/PCI Bulb Tees

Jayaprakash Vadivelu, Dept. of Civil Engineering, University of Tennessee, Knoxville, TN
Zhongguo (John) Ma, Ph.D., P.E., Dept. of Civil Engineering, University of Tennessee, Knoxville, TN

ABSTRACT

The main purpose of this analytical study was to verify that the 2 inch minimum spacing of ACI 318-05 and AASHTO (2004) 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 a 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 finite element analysis was carried out with two full-scale three dimensional 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. Design of two full-scale specimens was also discussed in the paper.

Keywords: Prestressed Concrete, Bulb Tees, FE Analysis, 0.7-inch strands, Strut-and-Tie Model, Full-scale testing, Confinement reinforcement.

INTRODUCTION

Pretensioned, prestressed members such as I-girders are widely used in the construction of bridges. The strand diameters in these members are predominantly 0.5 and 0.6 inches. 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. Further, an increased roadway clearance can possibly be achieved by using shallower members.

To investigate the maximum usable concrete strength in the application of bridge I-girders, Ma (2000)[1] performed an analytical study. In his study, the following assumptions were made:

- Design was based on a typical interior girder that was simply supported.
- Cross sectional shapes studied included AASHTO-PCI BTs and NUs.
- Girder spacings were 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**.

Take the example of a simple span with NU1100 I-girders and a girder spacing of 8 ft. The concrete strength of the cast-in-place deck is $f'_{c,\text{ deck}}$ = 4000 psi with a 7.5 in depth. Table 1 shows the impact of the 0.7-inch strand and girder concrete strength on the maximum span capacity of bridge I-girders. The maximum usable concrete strength level was in the range of 9000 to 12000 psi, 13000 to 16000 psi, 17000 to 20000 psi and 24000 to 29000 psi for 0.5 inch strand pattern, 0.6 inch strand pattern, 0.6 inch strand with bundling pattern and 0.7 inch strand with bundling pattern respectively. When 0.7-inch strands at 2-inch spacing are used, the span capacity can be increased by **178%**. For the NU section shape, the bottom flange can accommodate a total of 54 strands, comparing 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 thus longer spans can be produced since the maximum shipping length of I-girders has an upper limit.

Table 1 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

In order to fully realize the benefits shown in Table 1, it is important to study the feasibility of placing large 0.7 inch strands at 2 inch center-center spacing and to develop the quality control and design criteria, which is the objective of this research.

To achieve this objective, a FE analysis will be performed evaluating the potential impacts of 2 inch spacing for 0.7 inch strands and comparing it with 0.6 inch strands placed at 2 inch spacing[2,3,4] and possibly other reinforcement details at the girder end regions. In the second phase of the project two AASHTO Type I girders with 0.7 inch strands with the selected spacing will be produced to evaluate the transfer and development lengths of the two AASHTO Type I girders.

FINITE ELEMENT ANALYSIS

A finite element analysis was carried out in ABAQUS CAE[5] to evaluate the effects of 0.7 inch strands at 2 inch spacing and compared 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 only.

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. The prestressing force was the only force which was considered for the analysis, and was introduced by applying an initial compressive stress to the tendon elements.

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,

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

Where w_c = unit weight of concrete (kcf)

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.

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.

CONSTRAIN 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)].

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 Figures 1a and 1b.

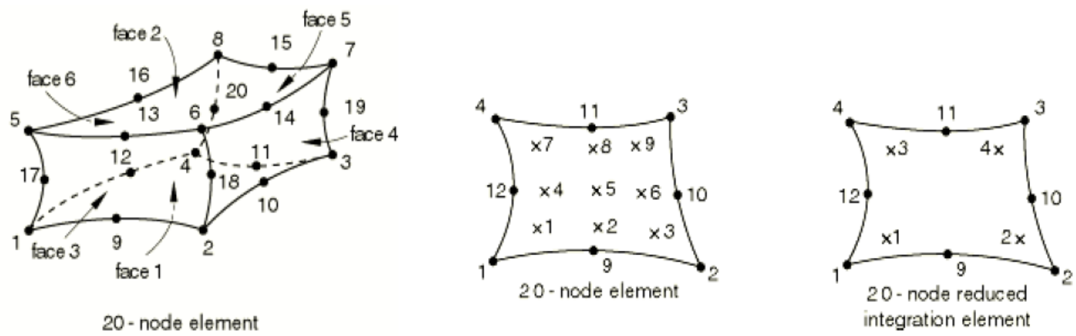


Fig. 1a 20-noded quadratic brick element with the integration points

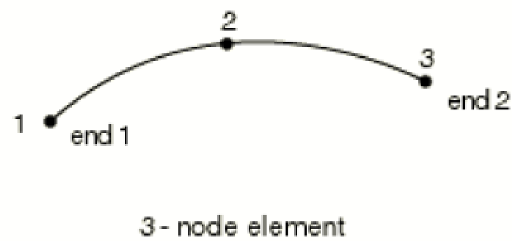


Fig. 1b 3-node quadratic 3D truss elements [ABAQUS/Standard User's manual (Version 6.7-5)]

RESULTS

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 below.

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 2, the girder with the 0.7 inch strand diameter reaches a maximum tensile stress of 1.74 ksi. Figure 2a 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 1.54 ksi as shown in Table 2. Figure 2b 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)[6], 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 Figures 3a and 3b, respectively. The same stress contoured along the central vertical plane for 0.7 inch and 0.6 inch strands are shown in Figure 4a and 4b, respectively. These figures show the cracking potential in the end zone of the girder.

Table 2 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

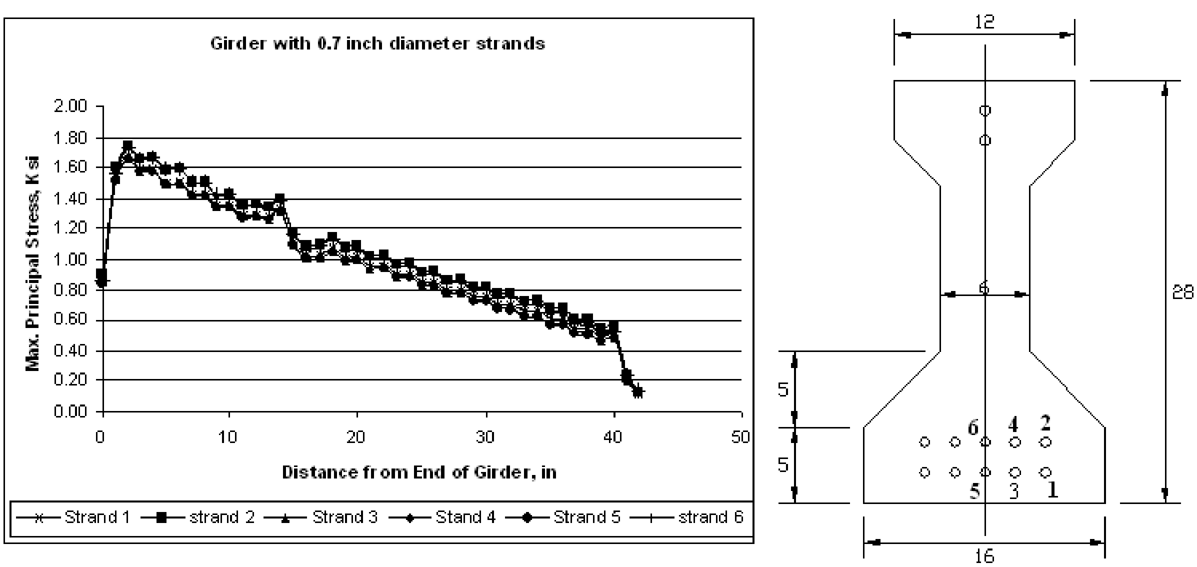


Fig. 2a Variation of maximum principal stress along the length of the girder from the end face at different locations of 0.7 inch diameter strand

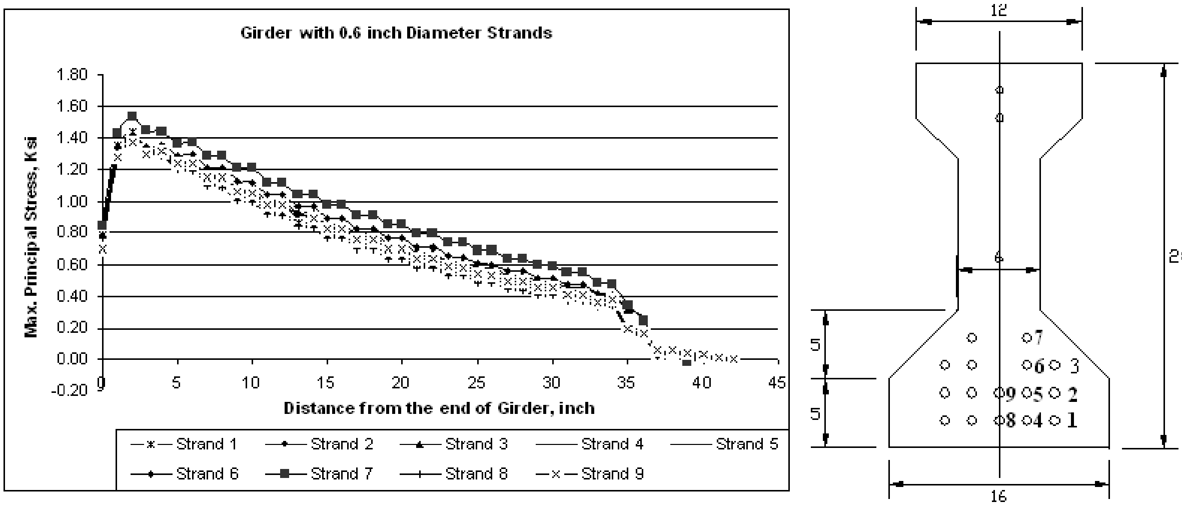


Fig. 2b Variation of maximum principal stress along the length of the girder from the end face at different locations of 0.6 inch diameter strand

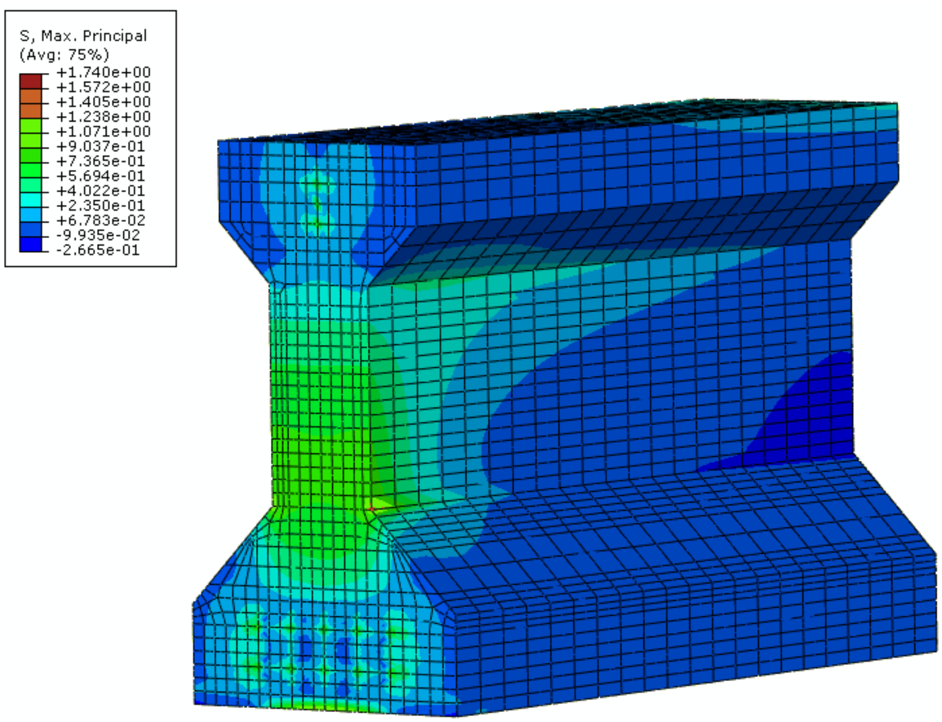


Fig. 3a Stress contour for the maximum principal stress for the end zone of a girder with 0.7 inch diameter strands

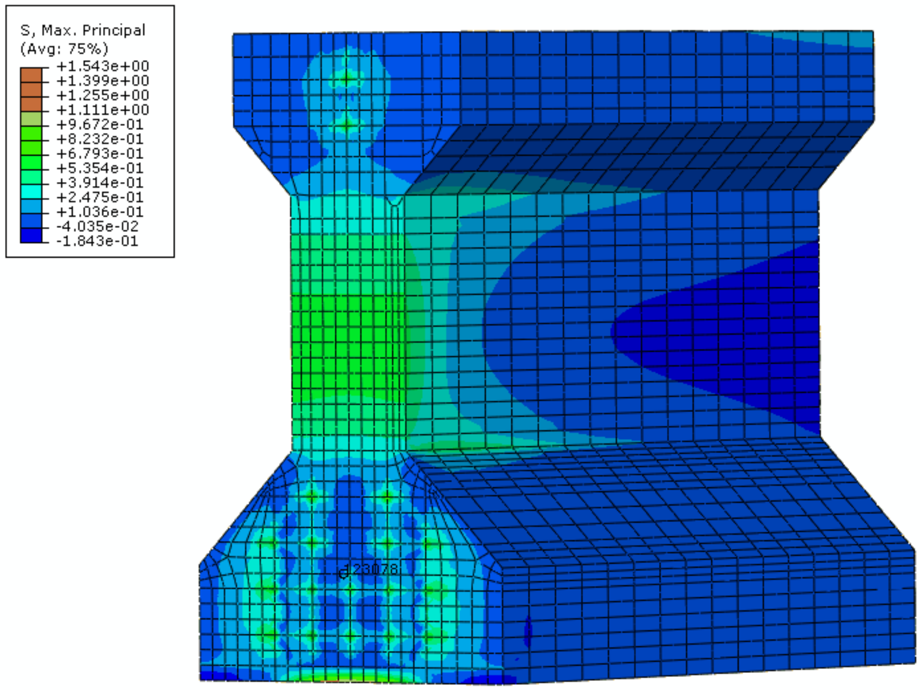


Fig. 3b Stress contour for the maximum principal stress for the end zone of a girder with 0.6 inch diameter strands

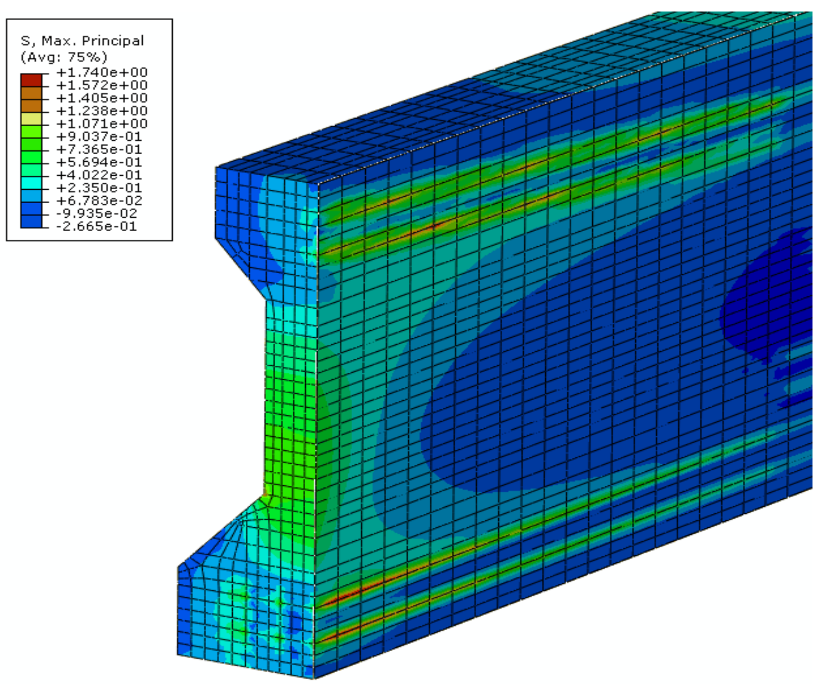


Fig. 4a Maximum principal stress distribution near the end zone of the girder with 0.7 inch diameter strands

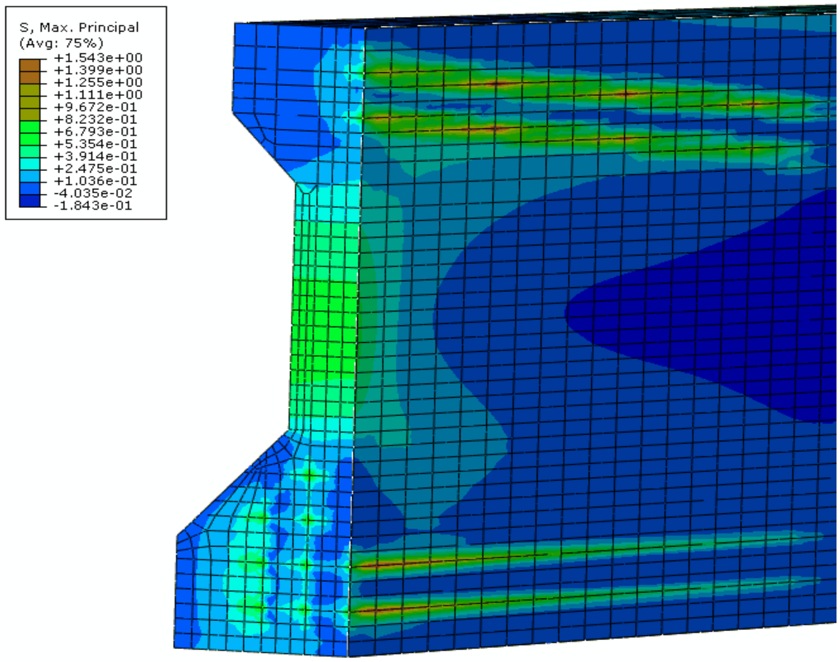


Fig. 4b Maximum principal stress distribution near the end zone of the girder with 0.6- inch diameter strands

The axial stress variation along the depth of the girder at the selected sections shown in Figure 5 for 0.7 and 0.6 inch strands are shown in Figures 6a and 6b, 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 point, which are 32 inches for 0.6 inch strands and 42 inches for 0.7 inch strands. The transfer point is calculated based on the equation in AASHTO LRFD-2004.

As shown in the Table 3, an axial stress of 1.09 ksi (Tension) is reached in the girder with 0.7 inch diameter strands were a stress of 0.43 Ksi (Tension) is reached in the girder with 0.6 inch diameter strands. Thus the girder with 0.7 inch strand exceeds the maximum concrete tensile strength 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 7a and 7b, respectively. It can be seen how the effective stress is reached from the end face to the transfer point of the girder.

Table 4 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	$0.07^{T}/\mathbf{1.09}^{T}$	$0.027^{T}/0.43^{T}$
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

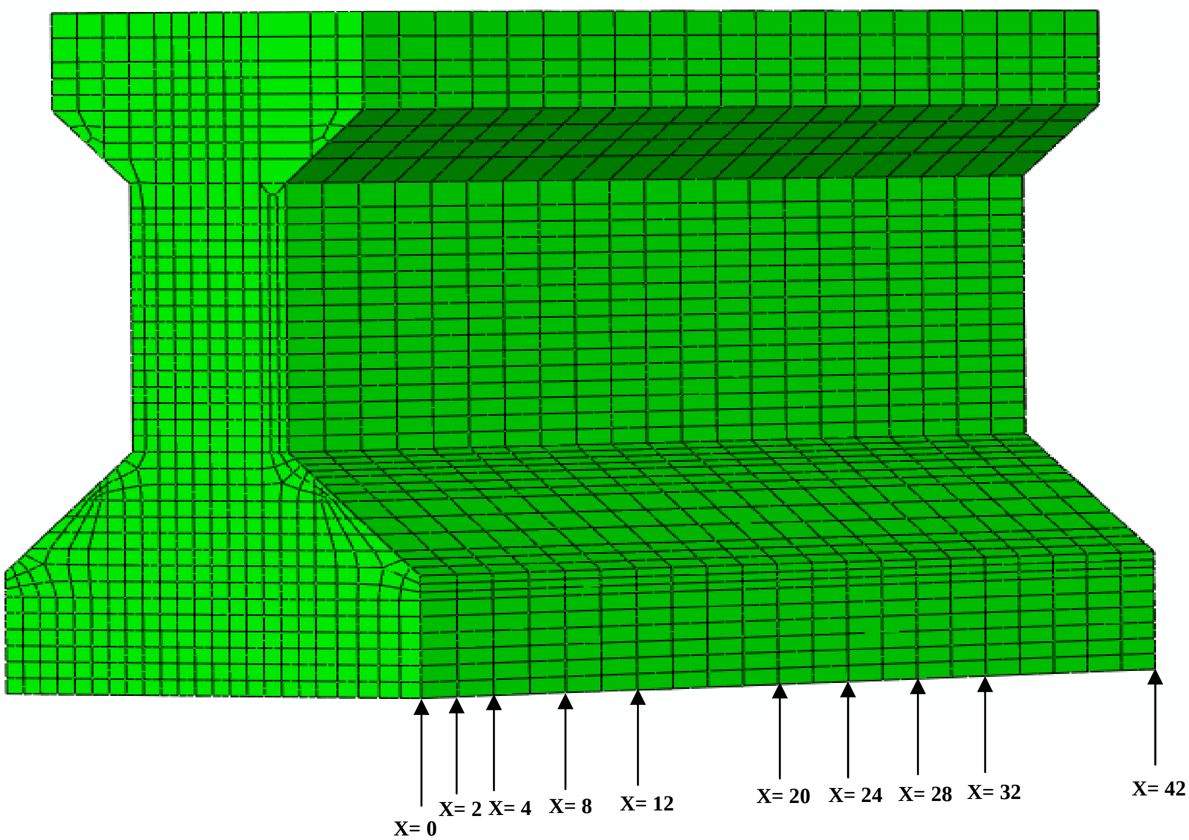


Fig. 5 Finite element model of a prestressed concrete I-girder

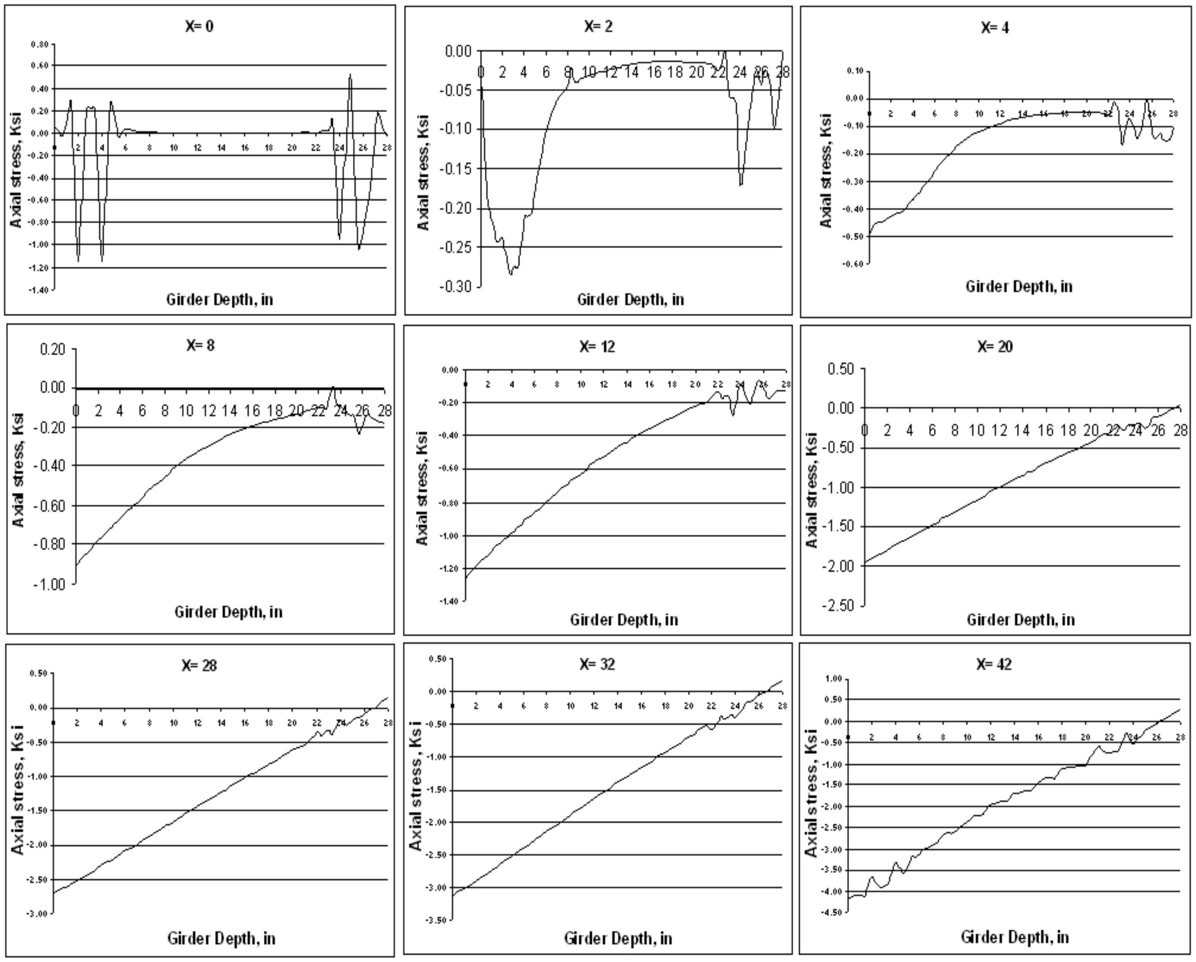


Fig. 6a Axial stress distribution at different sections near the end of the girder with 0.7 inch diameter strands

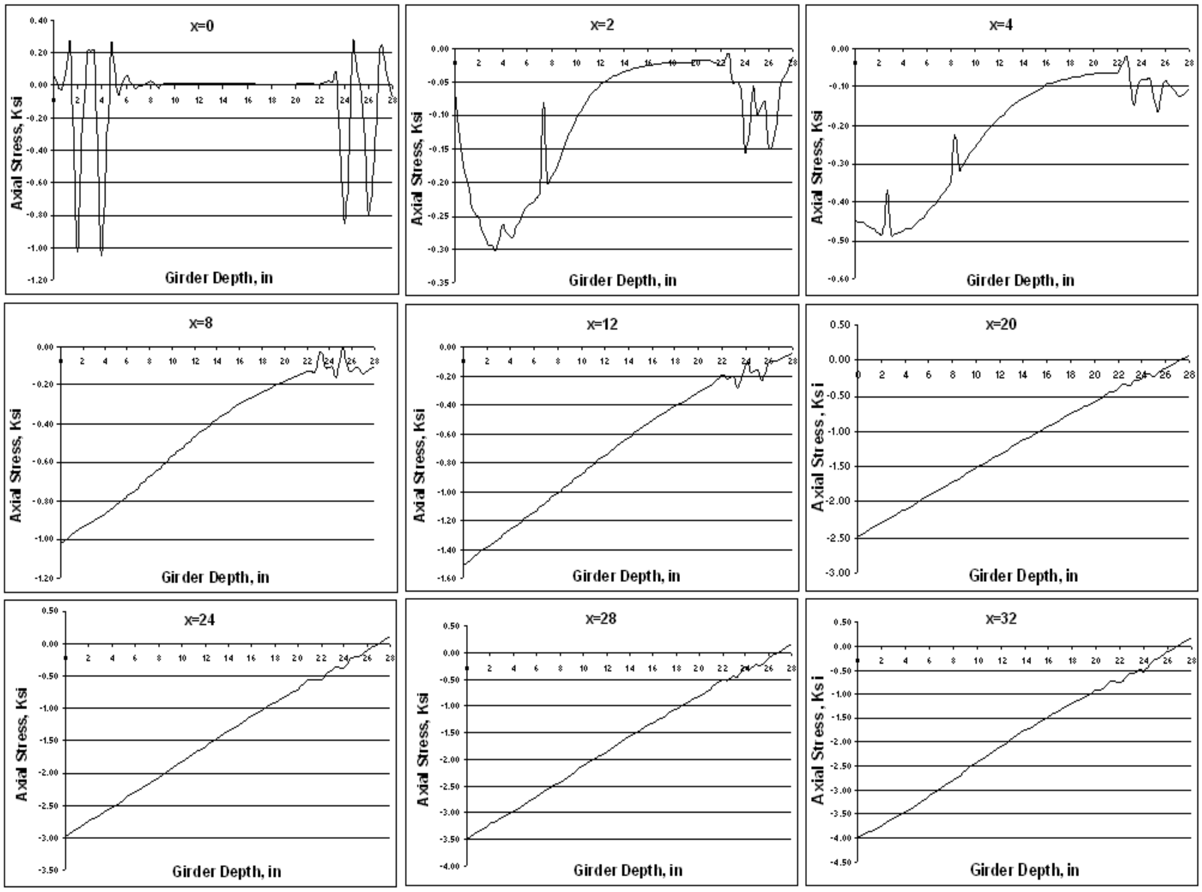


Fig. 6b Axial stress distribution at different sections near the end of the girder with 0.6 inch diameter strands

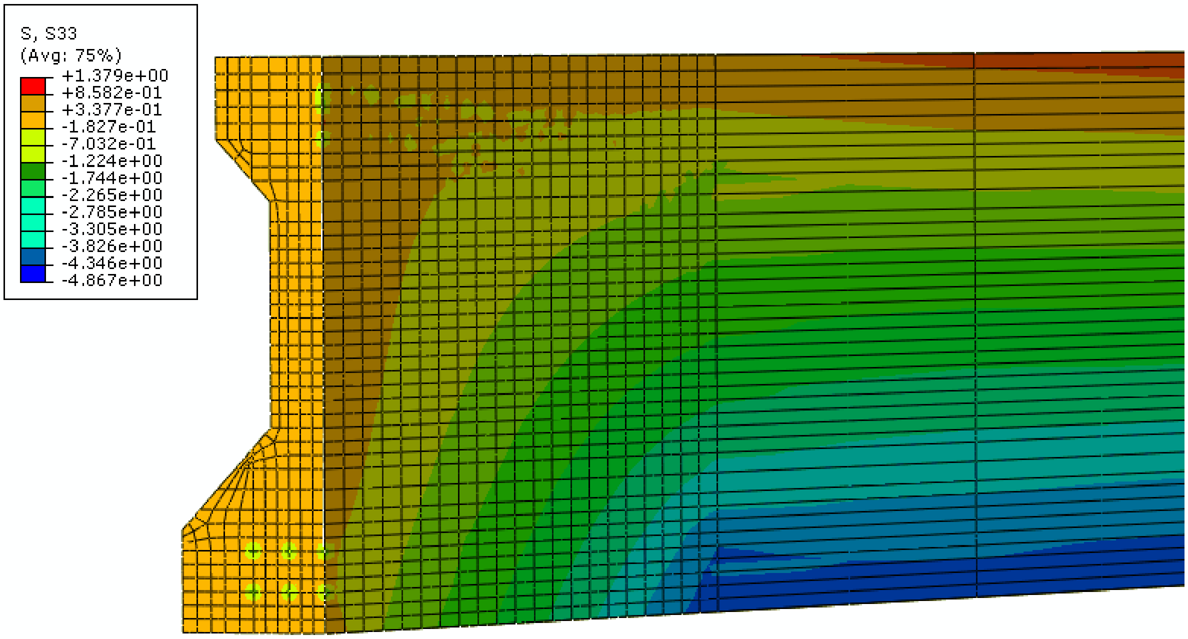


Fig. 7a Axial stress distribution in the direction parallel the direction of the tendons in the girder with 0.7-inch diameter strands.

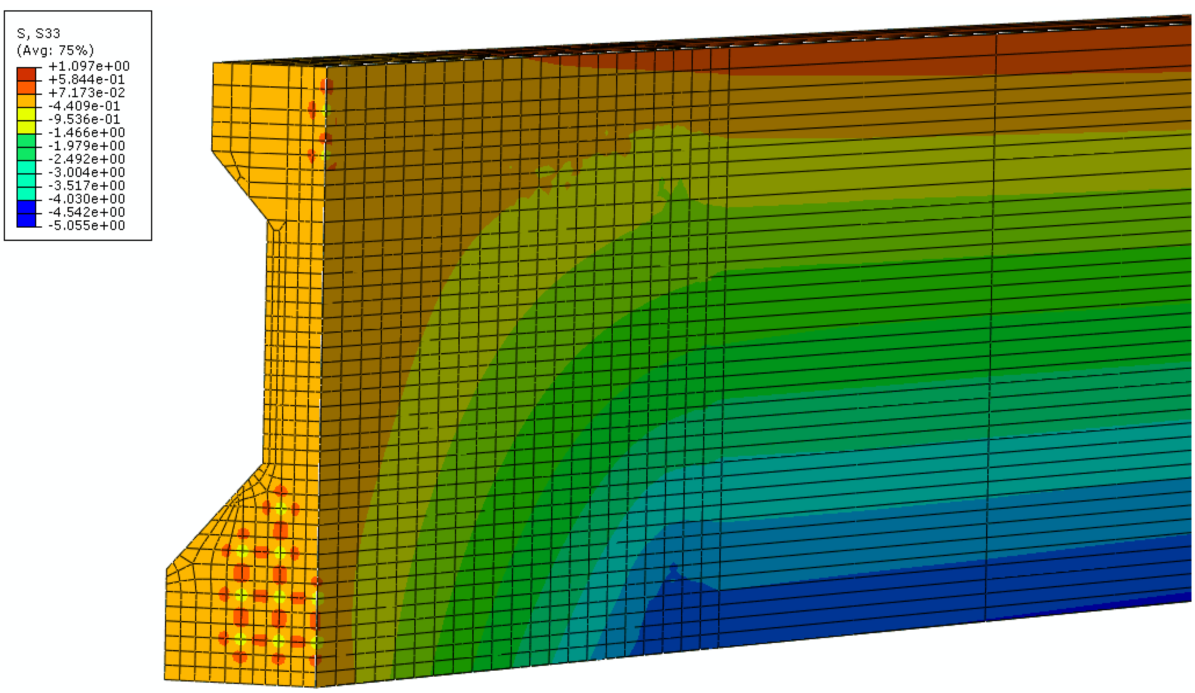


Fig. 7b Axial stress distribution in the direction parallel the direction of the tendons in the girder with 0.6-inch diameter strands.

STRUT AND TIE MODELLING

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 compute 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 consists 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.

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.

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 as shown in Figure 8. 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.

Table 4 Element forces for strut and tie model in Fig. 8

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

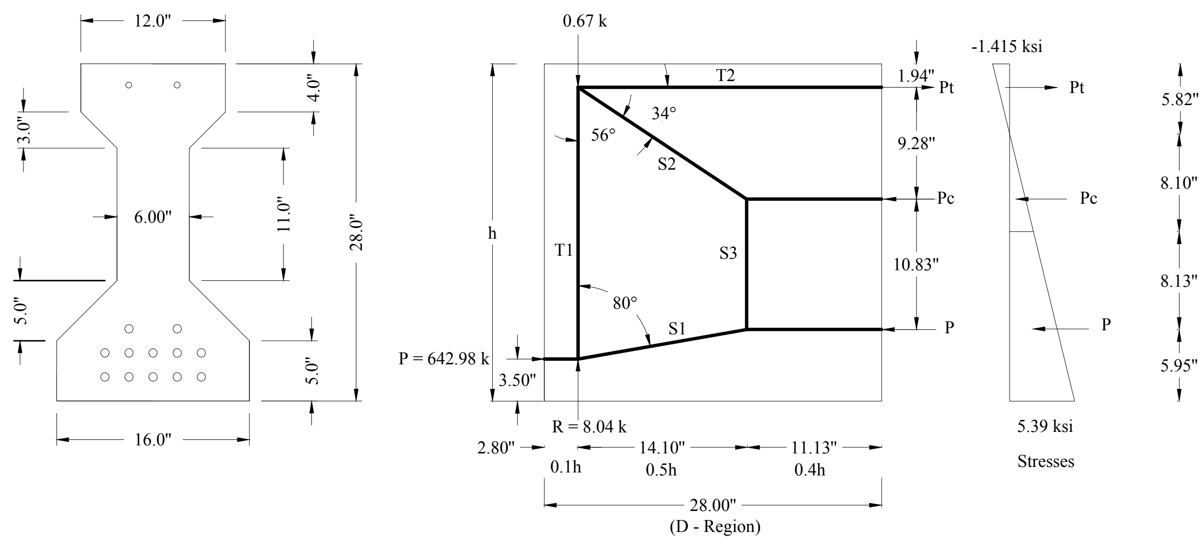


Fig. 8 Strut and Tie for web splitting in the pretensioned girder

CONFINEMENT REINFORCEMENT

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 9. Thus the required amount of splitting reinforcement is provided based on the tie forces determined after the analysis of the truss model. The confinement reinforcement help in controlling the splitting cracks at the end section of the girder.

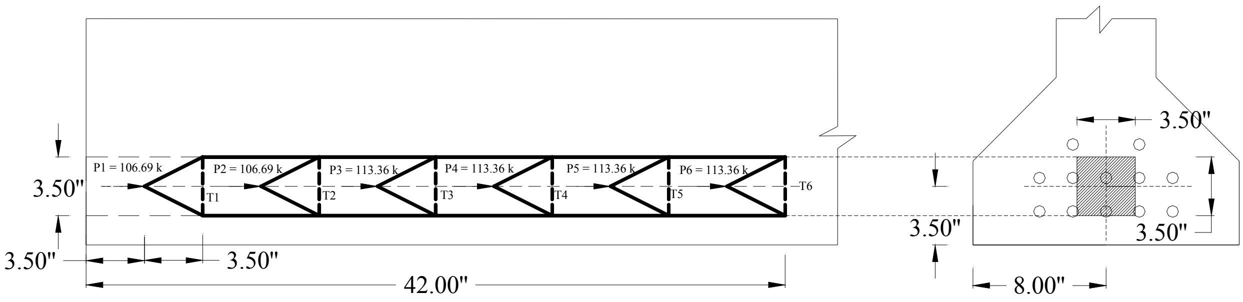


Fig. 9 Strut and Tie model for the splitting force in the bottom flange of the section

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 in the strut and tie model as shown in Figure 10. The width of the horizontal tie is 7inch based on the centriod of the strands and the horizontal strut is 3.5 inch on top. 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. Based on 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.

$$A_{st} = \frac{F_{tie}}{\emptyset f_y}$$

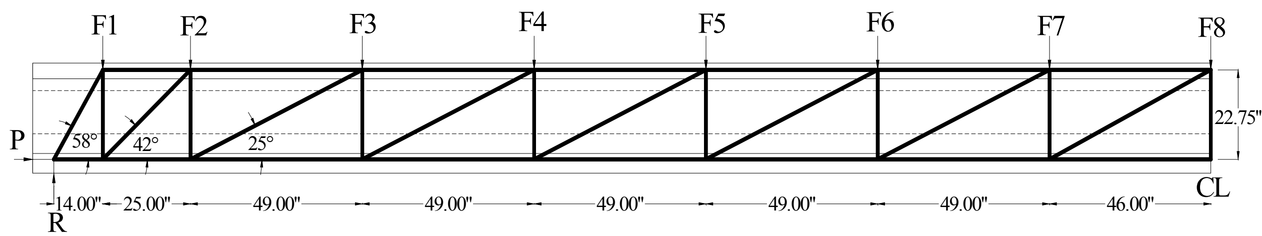


Fig. 10 Truss Model for one half of the girder using strut and tie model

Table 5 Vertical tie forces for the strut and tie model in Fig 10

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

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).

$$F_{ns} = f_{ce} .A_{cs}$$

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

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

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

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

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

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

$$W_s = W_t cos\theta + W_b \sin\theta$$

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 11 is

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

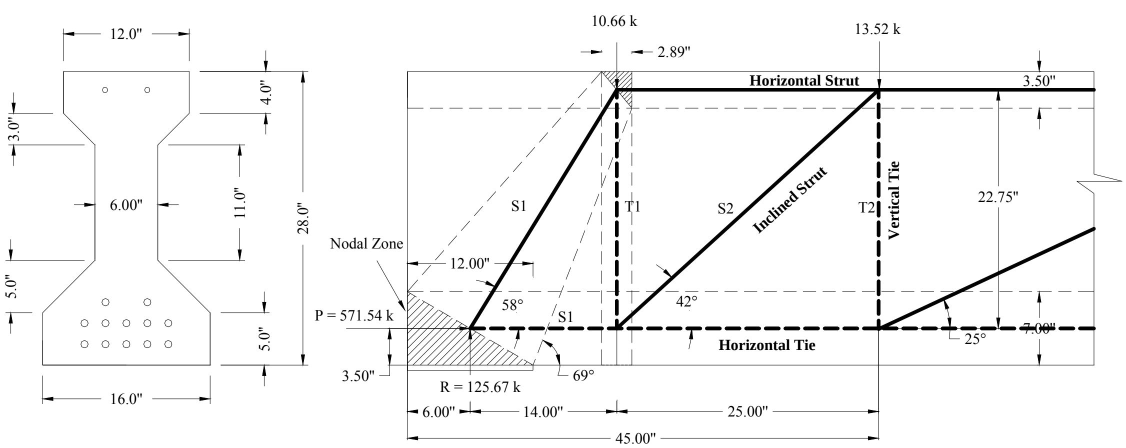


Fig. 11 Strut and tie model for the end region with horizontal strand pattern

In order to determine the width of strut S1 at the top, the width of the vertical tie T1 is required which 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,

$$W_{t,max} = {F_{nt}}/{f_{ce} b_s}$$

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

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

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.

Table 6 The strut forces for the strut and tie model

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

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.

EXPERIMENTAL INVESTIGATION

Two AASHTO Type I beams were designed with the following assumptions: (1) A bridge of six beams spaced at 8 feet centers. (2) The bridge was designed with cast-in-place concrete deck with a 8 inch actual thickness and 7.5 inch of structural thickness included in the 8 inch. (3) A haunch thickness of 0.5 inch was considered. (4) An additional 2 inches of wearing surface was considered to be the future wearing surface. (5) Prestress losses involved an initial loss of 10% at transfer and a total of 25% at service. (6) Span of each girder designed was 56 feet. The design was accomplished in accordance with the AASHTO LRFD bridge design specification.

Two specimens one with 0.7 inch diameter strand with an ultimate strength of 270 ksi and 0.62 inch diameter strand with an ultimate strength of 330 ksi are to be built.

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

<u>Right Half of the Specimen (AASHTO LRFD 2008):</u>
(Assumption: Details for 0.6” strands is 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 $0.24\sqrt{f'_{ci}}$ *. Detailed calculation is shown in Appendix A.*

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:

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

Confinement reinforcement - 7 #3 bars @ 6" spacing for a distance of 37.5".

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.

<u>Left Half of the Specimen (Strut and Tie Modeling):</u>

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"

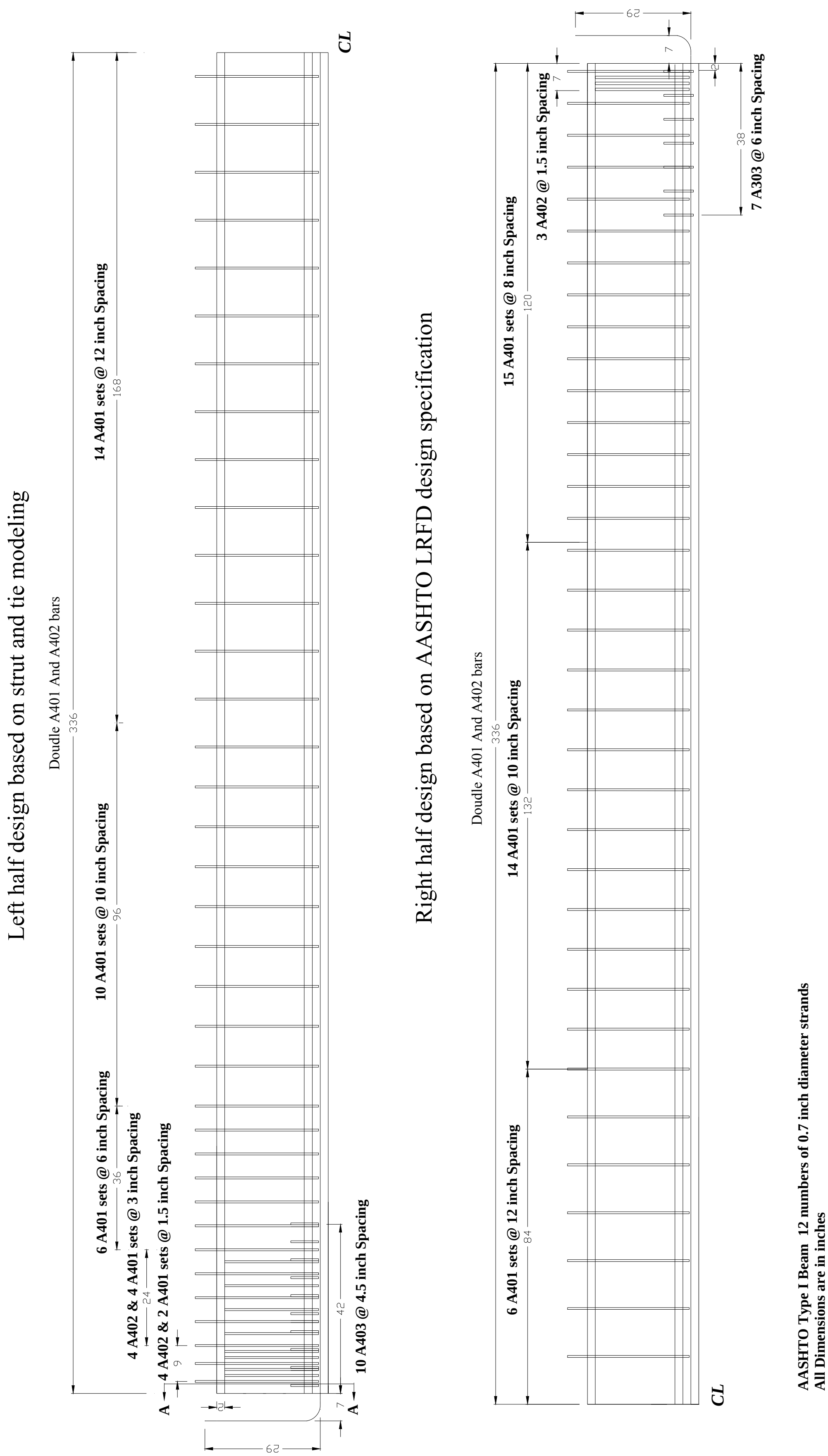
Top flange reinforcement:

- 4 #6 bars for the entire length of the girder

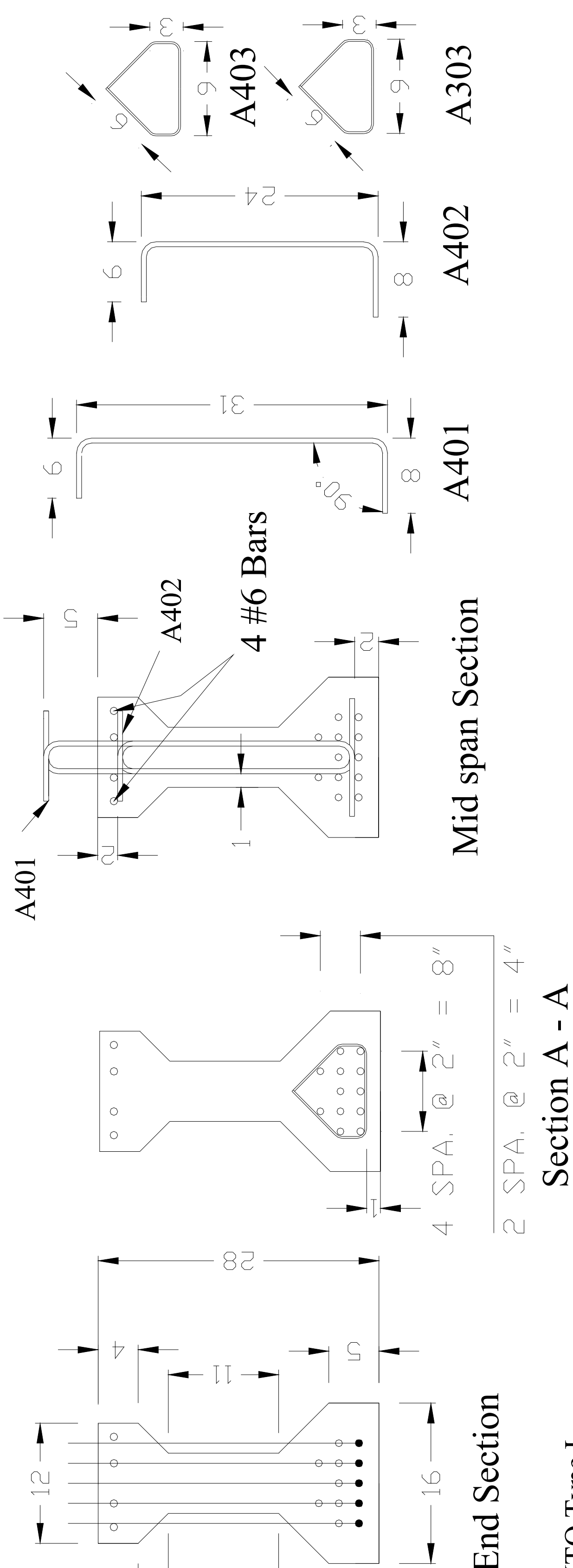
Anchorage Zone Reinforcement:

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

Confinement reinforcement -10 #4 bars @ 4.5" spacing for a distance of 42"



Left half design based on strut and tie modeling
Doudle A401 And A402 bars
336
14 A401 sets @ 12 inch Spacing
168
10 A401 sets @ 10 inch Spacing
96
6 A401 sets @ 6 inch Spacing
36
4 A402 & 4 A401 sets @ 3 inch Spacing
24
4 A402 & 2 A401 sets @ 1.5 inch Spacing
9
10 A403 @ 4.5 inch Spacing
42
CL
A
A
29
7
Right half design based on AASHTO LRFD design specification
Doudle A401 And A402 bars
336
15 A401 sets @ 8 inch Spacing
120
3 A402 @ 1.5 inch Spacing
7
7 A303 @ 6 inch Spacing
38
14 A401 sets @ 10 inch Spacing
132
6 A401 sets @ 12 inch Spacing
84
CL
29
7
AASHTO Type I Beam 12 numbers of 0.7 inch diameter strands
All Dimensions are in inches



AASHTO Type I
(12 Nos of 0.7 inch strands)

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 = **0.2325** in^2
- Ultimate strength of strand, f_{pu} = **330** ksi
- Spacing of strands = 2" x 2"
- Force per strand = (1.0)(0.2325)(0.75)(330) = 57.54 kips

<u>Right Half of the Specimen (AASHTO LRFD 2008):</u>

Shear reinforcement – Same as specimen 1

Top flange reinforcement - Same as specimen 1

Anchorage Zone Reinforcement

Splitting resistance reinforcement - Same as specimen 1
Confinement reinforcement - Same as specimen 1

<u>Left Half of the Specimen (Strut and Tie Modeling):</u>

Shear reinforcement – Same as specimen 1

Top flange reinforcement - Same as specimen 1

Anchorage Zone Reinforcement

Splitting resistance reinforcement - Same as specimen 1
Confinement reinforcement - Same as specimen 1

CONCLUSIONS

The analytical investigation presented in this paper considered only the effects due to the prestressing force at transfer. Based on the analytical investigation, the following conclusions were obtained.

- Using 0.7 inch diameter strands have potential advantages, such as higher span capacity, reduction in the section size and material saving and an increased roadway clearance can be achieved by using shallower members.

- Using 0.7 inch diameter strands improves span length in all sections. For states using Bulb Tee sections 0.7 inch strand can efficiently utilize high strength concrete to increase the span length to transportable limits. Using 0.7 inch strands in the structural efficient NU cross section can increase obtainable spans to lengths that have yet to be transported.
- The precast/prestressed I-girders are usually cracked at the end sections during the release of the prestressing force, the analytical study shows 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.
- This cracking at the end section can be minimized with the use of confinement steel at the girder ends.
- Further analytical study should be performed in order to determine the effects of the confinement steel for both 0.7 inch diameter strands and 0.6 inch diameter strands.

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