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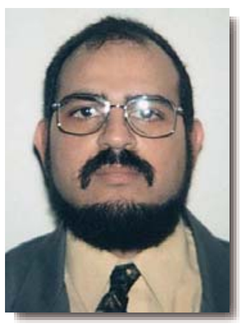

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In this study, a literature review was conducted to establish the background of current specifications and to evaluate the applicability of various theories and methods for design of end zone reinforcement. Analytical methods reviewed in this paper include finite element analysis, strut-and-tie modeling, and the Gergely-Sozen equivalent beam method. Previous experimental work combined with work conducted as part of this study was used to correlate between various theoretical and experimental results. This paper illustrates that no single theoretical method adequately represents the complex behavior at the end of a pretensioned concrete member. A general semi-empirical design procedure is proposed here. It is based on theoretical behavior and experimental observation. Standard reinforcement details are given. The proposed procedure could result in significant reduction in the amount of reinforcement while maintaining acceptable crack control at the member end. Application of the proposed procedure to highly pretensioned bridge girders is demonstrated.

Cracks are frequently observed at the ends of pretensioned concrete members at the time of prestress transfer, especially in narrow-stemmed members such as I-girders and inverted-tee beams. These cracks are caused primarily by the concentration of prestressing forces at the time of prestress release. They are commonly horizontal and occur near the junction of the bottom flange and web. Some diagonal cracks are also observed higher up on the web. The AASHTO LRFD Specifications[1] require that 4 percent of the total prestressing force be used as the tensile force in the vertical reinforcement at the end zone of a girder. AASHTO LRFD further stipulates that this vertical reinforcement be designed for a stress of 20 ksi (138 MPa) and placed within a distance from the end equal to one-fourth of the girder depth, $h/4$. With the use of high strength concrete increasing, relatively large prestressing forces are specified for pretensioned girders. Also, 0.6 in. (15 mm) diameter strands are being used at the standard 2 in. (51 mm) spacing in place of conventional 0.5 in. (13 mm) diameter strands, which can increase the prestressing force by as much as 40 percent. With greater prestressing forces, it has become impractical to place a large amount of reinforcement within the short distance required by AASHTO. Designers are faced with the dilemma of either violating the AASHTO requirements or using less efficient girder designs. Some designers question the validity of AASHTO because the girders do not undergo significant end cracking. They do not see any negative effects of these cracks, which are often enclosed in cast-in-place end diaphragms.

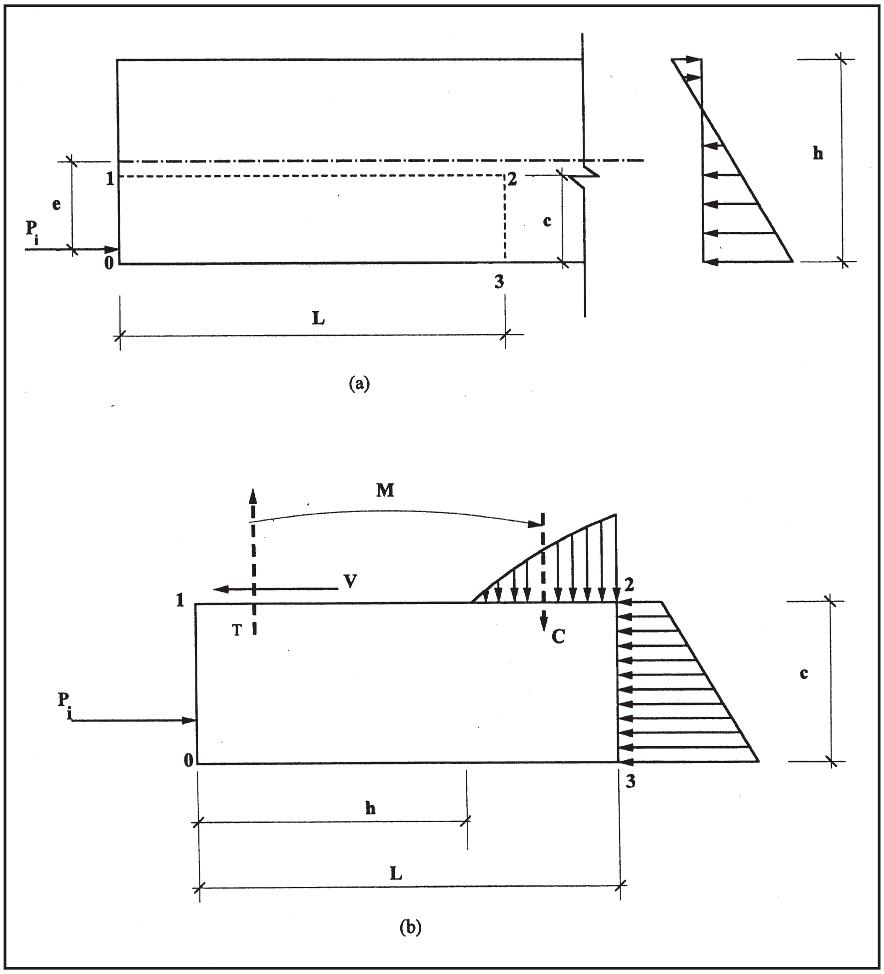


Fig. 1. Equilibrium analysis model – Gergely and Sozen.[5]

BACKGROUND

A literary review was conducted to document the background of end reinforcement requirements in AASHTO and to survey various design methods. An experimental investigation was also conducted to evaluate the strains and stresses in end zone reinforcement designed according to current specifications. Based on this investigation, a design procedure is proposed. Reinforcement details based on the design procedure were used in several full-scale bridge girders, which exhibited less end zone congestion and improved crack control.

CURRENT SPECIFICATIONS

In the early 1960s, Marshall and Mattock[2] developed a simple design equation for the required area end zone reinforcement. The semi-empirical equation was based on testing of 14 pretensioned girders whose depths ranged from 22.5 to 25.0 in. (572 to 635 mm). The splitting reinforcement area A_s is given by the following equation:

$$A_s = 0.021\left(\frac{P_i}{f_{sm}}\frac{h}{l_t}\right) \qquad (1)$$

where P_i is the total prestressing force, f_{sm} is the maximum allowable stress in area A_s, h is the total girder depth, and l_t is the transfer length of the strand.

This equation was deemed accurate for girders whose geometries satisfied $h/l_t \leq 2$ and yielded conservative designs for $h/l_t > 2$. A transfer length equal to 50 strand diameters was recommended unless experimental evidence dictated otherwise. Marshall and Mattock recommended that the end zone stirrups be distributed uniformly over a length equal to one-fifth of the girder depth. Their work was based on relatively small laboratory beams. It was difficult to extend its use to girders in current practice without additional experimental work. In some cases it is common to have girders as deep as 100 in. (2540 mm). Direct application of this method would imply the need to provide end zone reinforcement for a splitting force as great as 8 percent of the prestressing force.

Article 9.22.1 of the AASHTO Standard Specifications[3] appears to be a simplified form of the recommendations of Marshall and Mattock. The following statement regarding the end zone reinforcement requirements for pretensioned concrete girders first appeared in the 1961 AASHTO Interim Specifications: "In pretensioned beams, vertical stirrups acting at a unit stress of 20,000 psi [138 MPa] 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."

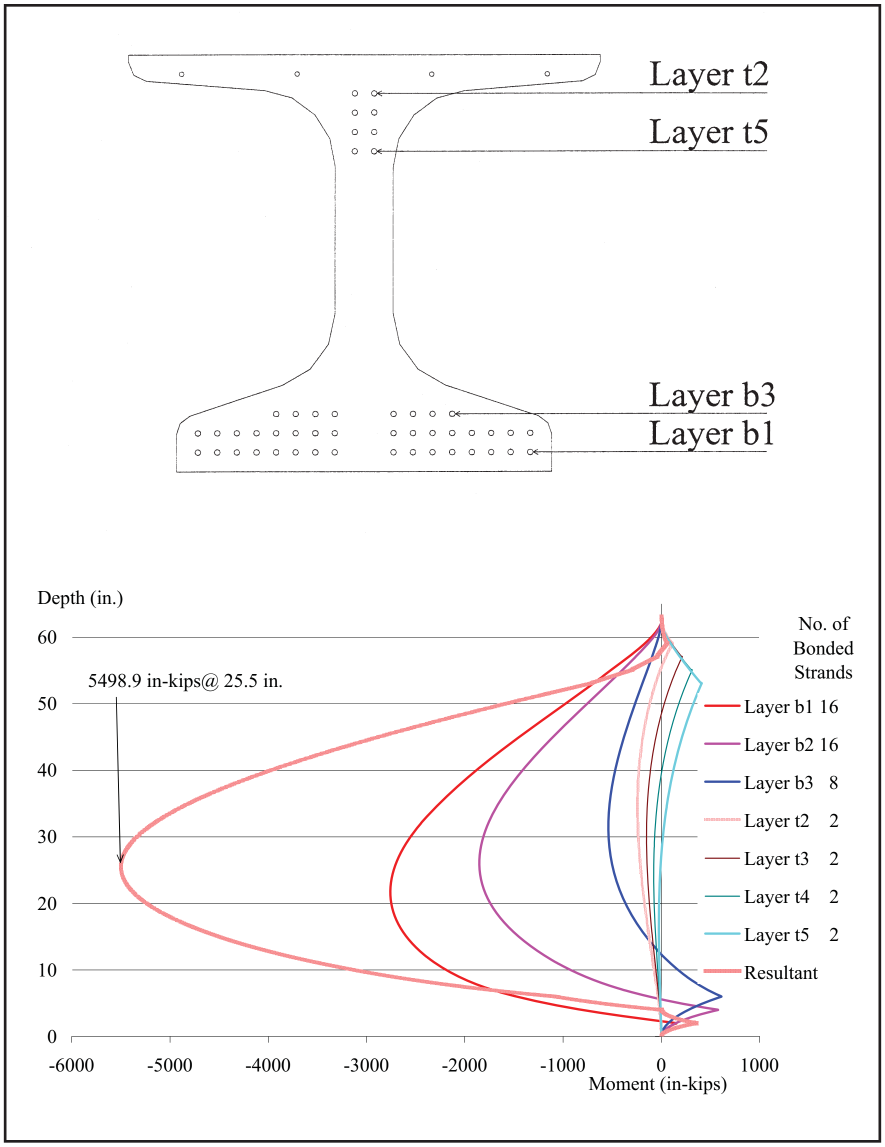


Fig. 2. Moment diagram at end zone of NU1600 I-girder.

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. (13 mm) diameter strands, this ratio represented a girder depth of 50 in. (1270 mm). At the time of their introductions in the 1960s, the provisions conservatively covered most of the girder sizes used at that time, and the constant ratio of 2 was believed to be conservative. Article 9.22.1 in the AASHTO Standard Specifications remains unchanged to this day. Article 5.10.10.1 in the AASHTO LRFD Specifications 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 percent of the effective depth, *d*.

ANALYTICAL MODELS

This section summarizes the analytical models that have been used in previous research on end zone behavior in pretensioned concrete members. The end zone stress distribution in a pretensioned concrete girder is a function of the location and magnitude of the prestressing strands, the degree of bond between strands and the surrounding concrete, the amount of strand draping in the end zone, the section geometry, and the concrete material properties. Once the concrete cracks, modeling the cracked end zone becomes more complicated.

Finite Element Modeling

Elastic stress analysis methods, particularly the finite element analysis, are useful for predicting the probable locations and orientations of cracks. Kannel et al.[4] developed a finite element model to predict the formation of end zone cracks under the influence of three-dimensional stresses created by draped prestressing strands and the order of strand release. Their models were used to investigate field measurement results and to manage the order of strand release. However, they were not easy to use in design of reinforcement for crack control.

Gergely-Sozen Model

Due to the presence of cracking, elastic analysis methods become invalid for determining end zone stress distribution caused by prestressing forces. In addition, the vertical stirrups in the end zone are not effective until horizontal cracks have been initiated. Due to these facts, Gergely and Sozen[5] developed a method of analysis based on the equilibrium conditions of the cracked end zone which was a further development of the work of Lenschow and Sozen.[6] This equilibrium analysis procedure is equally applicable to pretensioned and post-tensioned members.

Fig. 1 shows a representation of the end zone stresses. In Fig. 1a, the compressive stress distribution in the concrete due to the applied prestressing force becomes linear at some distance *L* from the member end. If a horizontal crack occurs along Face 1-2 at a height *c* from the bottom of the girder, the equilibrium conditions can be established using the free body diagram defined by Points 0, 1, 2, and 3 illustrated in Fig. 1b. A moment and shear force will generally exist on Face 1-2 due to the prestressing forces and the compressive concrete stress distribution. The linear stress distribution along Face 2-3 is calculated based on the geometry of girder cross section, the prestressing forces, and the strand pattern. The Face 1-2 location is determined such that the internal moment,

M, is the maximum value to occur on any horizontal face between the bottom and top faces. This is the face where cracking is likely to initiate. The resisting moment is provided by the tensile force, T, in the end zone reinforcement and the compressive resultant force, C, in the concrete, as shown in Fig. 1b.

Fig. 2 shows a typical moment diagram for an NU1600 I-girder. Additional graphs have been developed in Reference 7 for the maximum moments of several sections, including standard AASHTO bulb tees, rectangular girders, and box beams as well as Nebraska inverted-tee and I-girders shapes.[8] These graphs serve as valuable guides for locating the initiation of splitting cracks. The authors believe that the Gergely-Sozen Method is the most practical solution for analysis of prestressed member end zones.

Strut-and-Tie Model

The strut-and-tie model (STM) is a strength limit state analysis method. The model is useful in assessing the stress flow in the end zone and locating zones of concrete tension. Its use in determining the amount of splitting reinforcement, however, may produce overly conservative estimates. Because some researchers[9] have applied this method to the design of pretensioned member end zones, the method is briefly discussed here.

In the STM method, the forces internal to a member are confined in a series of straight-line compression struts and tension ties that are connected at discrete nodal points, thus forming a truss. The compressive forces are carried by the concrete struts, and the tensile forces are carried by the conventional and/or prestressed reinforcement. Because of the nonlinear stress distribution in the end zone due to concentrated prestressing forces, it is called a disturbed region, or D-region.

A strut-and-tie model must satisfy force equilibrium in this region. It does not require that compatibility of deformations, or strains, be satisfied. It is generally analyzed at the strength limit state, where it is assumed that concrete tension is non-existent and concrete compression has a uniform intensity. Because compatibility is not required to be satisfied and because a large number of truss models can be generated for the same problem, the results of the STM method give upper-bound solutions that can be much more conservative than those given by other methods.

In applying the STM to a prestressed girder end zone analysis, designers must keep the steel stress below the limit that would create undesirable crack widths, typically 20 to 24 ksi (140 to 160 MPa). The inconsistency between the intent and application of the STM in the design end zone reinforcement for serviceability leads to the conclusion that the STM method should be used only qualitatively, rather than quantitatively, for the design of pretensioned girder end zones.

Sanders and Breen[10] conducted extensive research on the anchorage zone reinforcement for post-tensioned concrete girders using STM at the strength limit state. An STM was developed for the design of reinforcement to resist splitting in the end zone of post-tensioned concrete members. They indicated that their model could not be directly applied to pretensioned members because the primary objective in pretensioned members is to control cracks at service load, and the force transfer between pretensioned strands and the surrounding concrete is gradual.

References 7 and 9 indicate that using the STM in pretensioned end zones could yield a splitting force between 4 to 13 percent of the pretensioning force. Because other methods yield 4 percent or less, the results of the STM analysis lead to significantly overestimating the end zone reinforcement required.

Table 1. Geometric and material properties and reinforcement details (Phase I).

Properties	NU1800	NU1600	IT600	IT400
Number of specimens	3	3	3	3
Girder length (ft)	127	118	50	48
Drape length (ft)	51	55	—	—
Number of draped strands	8	6	—	—
Number of straight strands	46 (4 debonded)	38 (12 debonded)	16	12
Cross-sectional area (sq in.)	857.3	810.8	246	196
Moment of inertia (in.^{4})	611,328	458,482	11,938	3568
Strand diameter (in.)	0.5	0.5	0.5	0.5
Reinforcement in end zone	2 No. 5 @ 2 in.	2 D18 @ 2 in.	2 D20 @ 2 in.	2 D18 @ 2 in.
Amount of steel (within h/5 distance)	14 No. 5	12 D18	4 D20	4 D18
Concrete strength, f'_{ci} (psi)	7021	6890	7343	6112
Concrete modulus, E_{ci} (ksi)	5003	5658	5148	5116

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 sq in. = 645.2 mm^2; 1 in.^{4} = 416,231 mm^4; 1 psi = 0.0069 MPa; 1 ksi = 6.9 MPa.

EXPERIMENTAL INVESTIGATION

The experimental program in this study consisted of two phases. In Phase I, the stresses and strains in the end zone vertical reinforcement of various girders designed in accordance with the current AASHTO LRFD Specifications[1] were measured and analyzed. In Phase II, new end zone reinforcement details were proposed, tested, and evaluated.

Phase I End Zone Details

The objective of Phase I was to evaluate stresses in the end zone vertical reinforcement at strand release in actual girders, designed in accordance with the AASHTO LRFD Specifications. Data were collected from 12 pretensioned concrete girders produced by two precast producers in the state of Nebraska, including six NU I-girders and six inverted-tee (IT) girders. Table 1 gives the geometric properties, material properties, and prestressing and reinforcement details

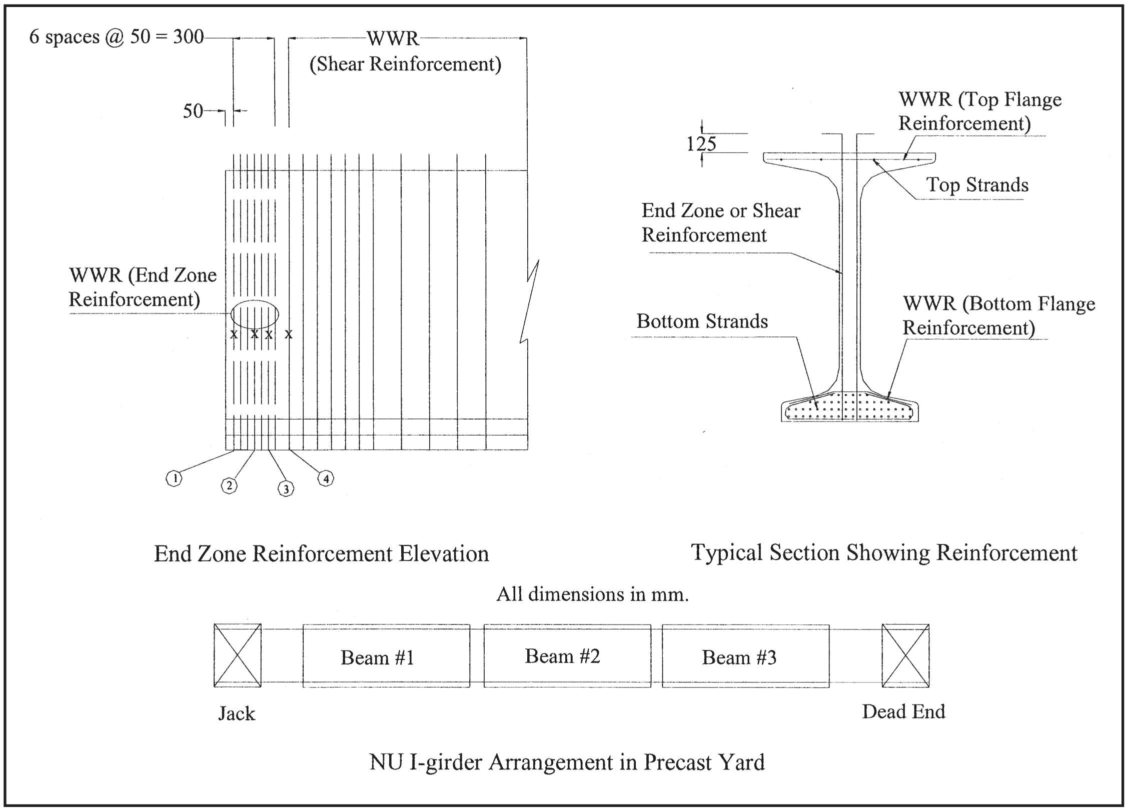


Fig. 3. End zone reinforcement detail and strain gauge location-NU1800.

of the Phase I girders. End zone reinforcement in these specimens was placed a distance equal to one-fourth of the girder height at each end. The reinforcement shown in Table 1 was spaced at 2 in. (51 mm) on center in each face of the web.

Figs. 3 and 4 show the girder cross section and locations of the strain gauges for the NU I-girders and IT girders, respectively. The Gergely-Sozen model[5] was used to predict the maximum moment locations where strain gauges were to be mounted. Strain gauges were mounted on the

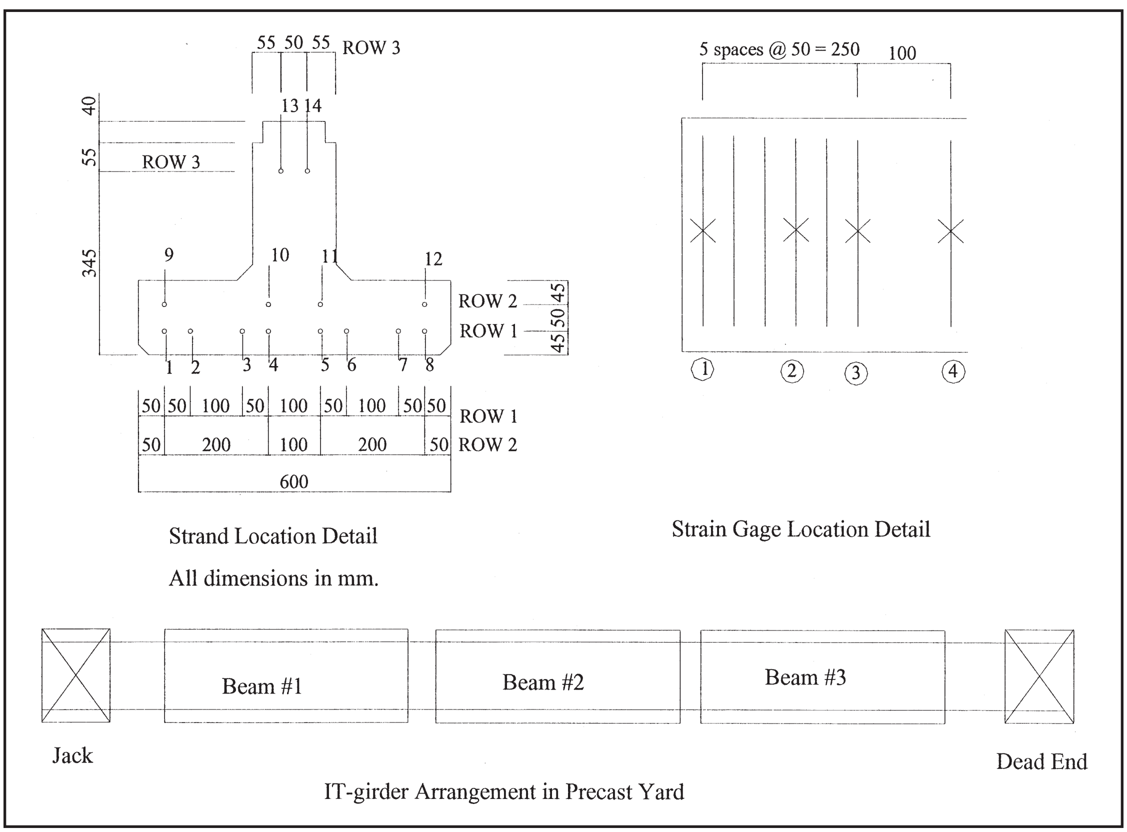


Fig. 4. End zone detail and strain gauge location-IT400.

vertical reinforcement of the girder end nearest the live end of the prestressing bed abutment. This location was selected to ensure that the highest prestressing force in the entire bed was utilized in the measurements. In some girders, strain gauges were installed at both ends to distinguish between each end, and to average the splitting force for both ends.

Samples of D18 and D20 deformed wires and No. 5 reinforcing bars were tested to obtain the stress-strain relationships needed to convert strain gauge readings into stresses. During release, strain readings of vertical reinforcement were recorded.

For all girders, the draped strands were released suddenly by flame cutting. Then, the bottom straight strands were released gradually with hydraulic jacks. Gradual release of all straight strands is the standard practice in Nebraska. Flame cutting of strands should be performed carefully to avoid dynamic and uneven distribution of release stresses. The authors recommend that precast concrete producers release the entire prestressing force gradually and simultaneously, especially for girders with high prestressing forces.

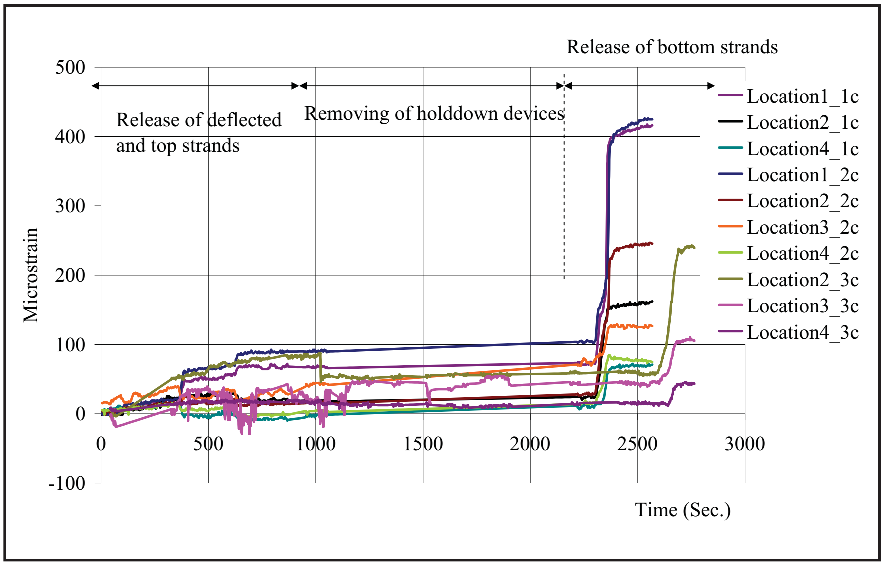


Fig. 5. Change of strain in the end zone reinforcement during release for NU1800_1c, NU1800_2c, and NU1800_3c I-girders.

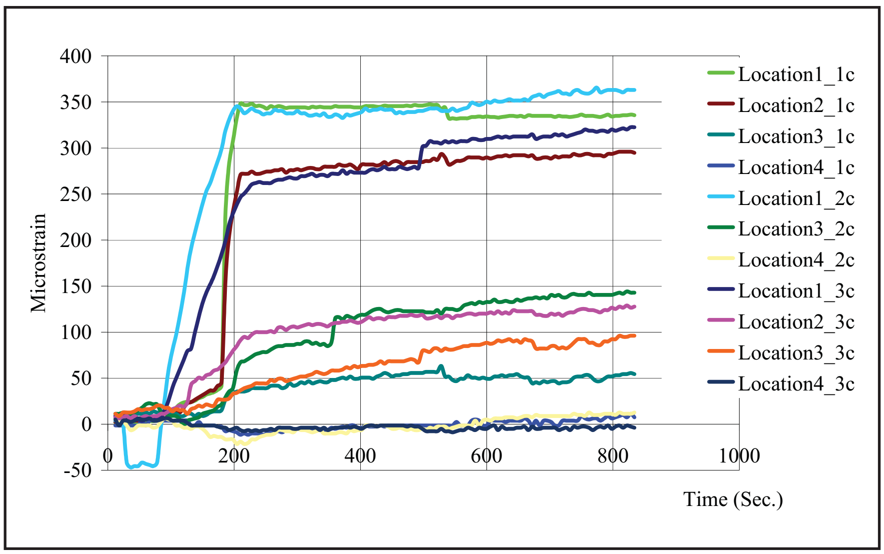


Fig. 6. Change of strain in the end zone reinforcement during release for IT600_1c, IT600_2c, and IT600_3c girders.

Analysis of Phase I Test Results

Figs. 5 and 6 show examples of differential strain in the end zone reinforcement due to prestress transfer to the concrete member. It was found that the first reinforcing bar had the highest strain. The strain in the reinforcing bars rapidly decreased as the distance from the end increased until it totally dissipated at approximately *h* from the girder end, where *h* is the total member depth. This behavior is consistent with the findings of Marshall and Mattock.[2]

The strain in the first reinforcing bar was instantaneously affected by flame cutting of the top strands. However, release of top the strands did not give the maximum strain values. The maximum strain values were reached when all strands in the girders were released. The maximum strain gauge readings after release are given in Table 2. Fig. 7 shows the distribution of the maximum stresses, which are the products of strains and modulus of elasticity, in the reinforcing bars. The maximum stress in the end zone reinforcement varied between 0.2 and 12.9 ksi (1.4 and 89.0 MPa). The maximum stresses in the end zone reinforcement were less than the allowable design stress of 20 ksi (138 MPa) provided by the AASHTO Specifications.

Fig. 8 shows the relationship between the prestressing forces and the splitting forces. The average splitting force was about 2 percent of the prestressing force for the girders tested in Phase I. The splitting force was calculated as the product of the measured steel stress and the provided steel area. If the design stress of 20 ksi (138 MPa) had been used in conjunction with the provided steel area, then the total theoretical splitting force would have been the 4 percent originally used in designing the specimens. For the IT sections, cracking was invisible. For a typical I-girder, several horizontal fine cracks were observed in the web near the theoretically anticipated locations. The cracks extended about 8 to 12 in. (203 to 305 mm) from the member ends (see Fig. 9). The information gained from this phase was utilized in the design of the end zone reinforcement in Phase II.

Table 2. Microstrain in the end zone reinforcement at release (Phase I).

Specimen	Location 1		Location 2		Location 3		Location 4	
	z (in.)	Microstrain	*z* (in.)	Microstrain	*z* (in.)	Microstrain	*z* (in.)	Microstrain
NU1800_1c	2	416	8	162	12	N/A	16	71
NU1800_2c	2	425	8	245	12	126	16	74
NU1800_3c	2	N/A	8	239	12	105	16	43
NU1600_1c	2	216	8	201	12	188	16	116
NU1600_2c	2	460	8	220	12	156	16	N/A
NU1600_3c	2	444	8	N/A	12	165	16	82
IT600_1c	2	348	6	201	12	54	18	8
IT600_2c	2	363	6	N/A	12	143	18	13
IT600_3c	2	323	6	128	12	96	18	6
IT400_1c	2	315	8	108	12	68	16	32
IT400_2c	2	302	8	N/A	12	70	16	16
IT400_3c	2	96	8	39	12	32	16	30

Note: 1 in. = 25.4 mm.

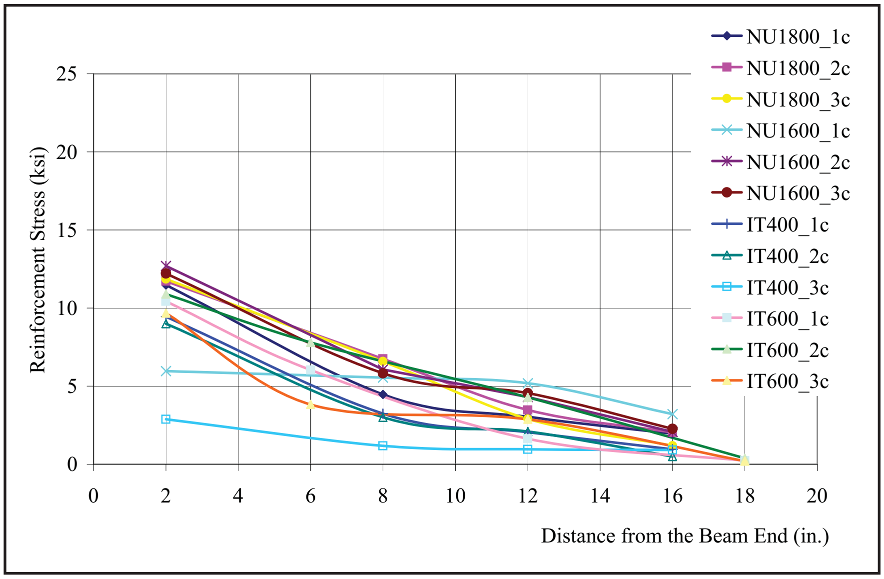


Fig. 7. Variation of maximum vertical reinforcement stress over end zone.

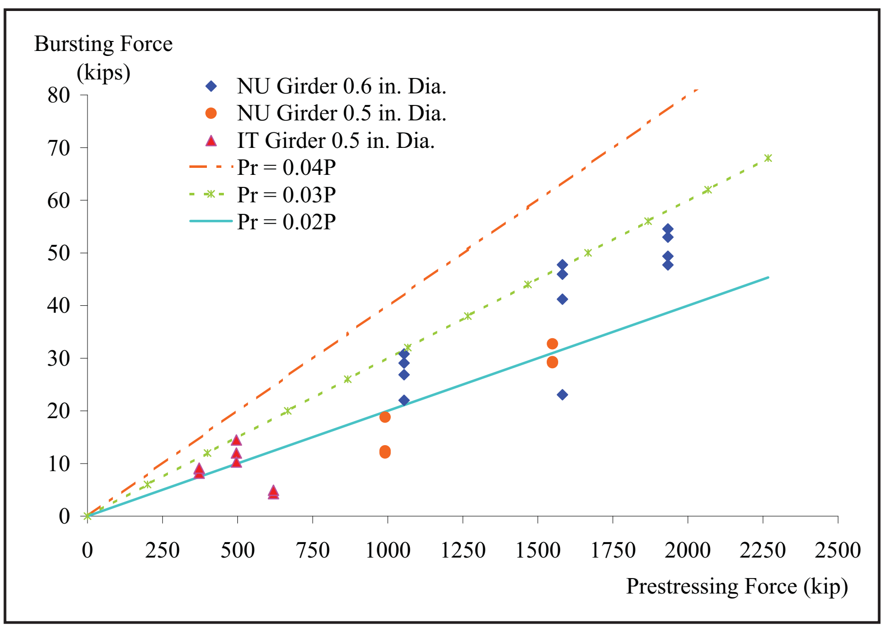


Fig. 8. Relationships between splitting force and prestressing force.

Phase II Proposed End Zone Details

Based on the observations and analysis from Phase I, new end zone details were developed. The design of the new details was based on the following criteria:

1. End zone reinforcement should be concentrated at the end of the girder such that the first bar is located as close as possible to the end. It was observed in Phase I that only the reinforcement located $h/8$ from the end of the member experienced significant stress. There was some stress applied to the reinforcement located $h/8$ to $h/2$ from the end. Beyond that zone, tensile stresses in the splitting reinforcement were very small, as seen in Fig. 7.

2. A steel stress of 20 ksi (138 MPa) was used to design the reinforcement for of the splitting force. This stress was expected to correlate well with observed steel stress near the very end of the member in Phase II, where the splitting reinforcement was reduced compared to that of Phase I.

3. The special vertical reinforcement at the end zone was designed for a splitting resistance of about 2 percent of the prestressing force and was placed within $0.125h$ of the end. The remainder was balanced with reinforcement corresponding to that calculated for the critical shear section. For example, if the critical shear section is $0.720h$ from the support face and if the support is 6 in. (152 mm) wide, then there are two reinforcement zones. The first zone contains the

splitting reinforcement at 2 in. (51 mm) spacings. It should be located to 0.125*h* from the member end. The second begins at 2 in. (51 mm) from the end of the first zone and continues as required for shear. (See Table 3 for illustration.) Note that some of the specimens of Phase II were reinforced with the details used in current practice. This was done for comparison purposes.

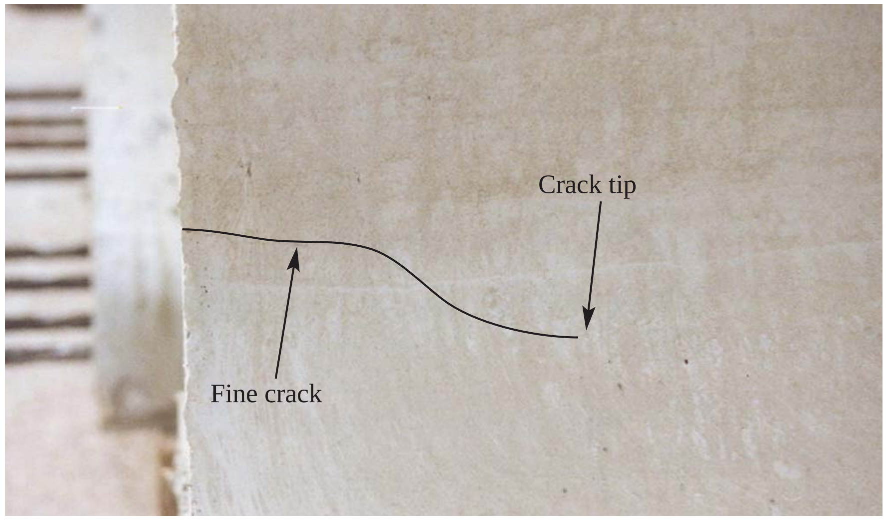
For IT400 girders, threaded rods were initially used as reinforcement for splitting resistance. They were later revised to Grade 60 reinforcing bars fabricated with a welded base plate. For the I-girder specimens the details used in the experiments were the ones recommended for implementation.

Fig. 10 shows the reinforcement used in the end zone of the IT400 specimen. Fig. 11 shows the inverted-tee reinforcement detail recommended for practice. The proposed end zone reinforcement details used in the Phase II experiments are shown in Figs. 12 and 13. The details shown in these two figures show small welded top plates used to provide adequate anchorage. Note that all shear reinforcement used in Nebraska consists of specially fabricated welded wire reinforcement. Chapter 3 of the PCI Bridge Design Manual[11] shows details of WWR used as shear reinforcement in I-girders. The current and proposed end zone details for the NU1100 I-girder are presented in Figs. 14a and 14b, respectively.

Because both the special end zone reinforcement and shear reinforcement is located within a distance of $h/4$, the splitting resistance within $h/4$ of the member may be significantly larger than 2 percent of the prestressing force. Strain measurements in this phase were taken similar to Phase I.

Phase II Test Results

The maximum strains in the end zone reinforcement are summarized in Table 4. Note that the specimen designations in the table include the letter "c" for current detail, and the letter "n" for new details. Concrete cracking and steel strains in this phase exhibited very similar patterns to those observed in Phase I. The strains found using the new details were smaller compared those found using the conventional details. This indicates improved efficiency of steel placement despite the lower quantities used.



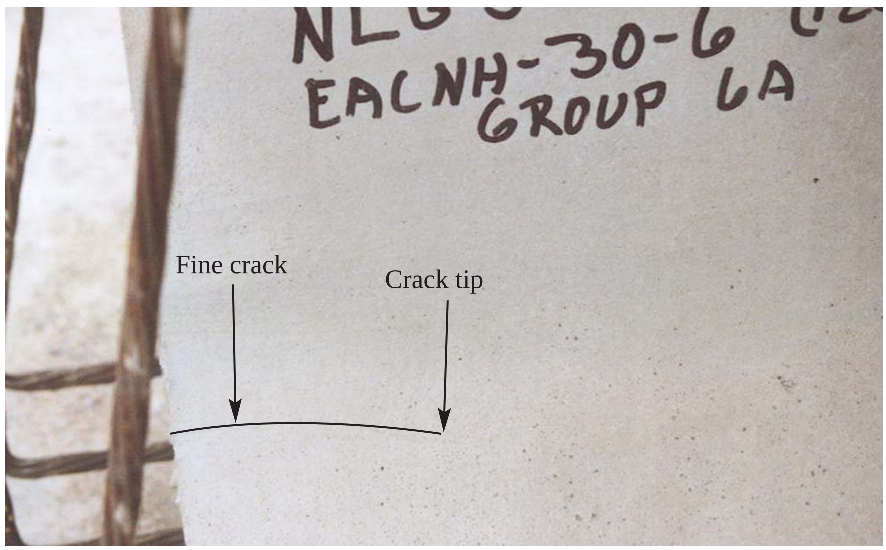


Fig. 9. Comparison on splitting cracks on the NU1600 I-girder web between current (top) and proposed (bottom) end zone reinforcement.

The inverted-tee specimen also demonstrated that even with the same amount of reinforcement as the conventional details, concrete cracking and steel strain were improved with the new reinforcement details. This confirms the need to place a relatively large area of reinforcement as close to the member end as possible. Higher prestressing force used in the inverted tee in Phase II [619.7 kips (2.75 MN)] compared to that used in Phase I [371.8 kips (1.65 MN)] did not cause additional cracking or steel strain as long as the design was done for a proportionate splitting force and good detailing was provided.

The IT400 is only 400 mm (15.74 in.) deep. In this case, one-eighth of the member depth is less than 2 in. (51 mm). It is important for most of the splitting resistance reinforcement to be placed within that limited length. One must keep in mind that most of bridge member ends eventually get enclosed in concrete diaphragms. Thus, concrete cover to that steel is less critical than in the exposed part of the girder. The confinement and anchorage provided by the end plates contributed to the improved performance.

Because of the relatively low strain in the inverted tee specimen and because of the relatively high cost of the threaded rod used in the test, it was decided to convert to the reinforcing

Table 3. Geometric and material properties and reinforcement details (Phase II).

Properties	NU1600		NU1100		NU1100	IT400	
Number of specimens	2	2	2	2	4	1	1
Girder length (ft)	135		78		10	51	
Draped length (ft)	60		35		—	—	
Number of draped strands	8		6		—	—	
Number of straight strands	40 (12 debonded)		18		44	18 + 2 (top)	
Cross-sectional area (sq in.)	810.8		694.6		694.6	196	
Moment of inertia (in.^{4})	458,482		182,279		182,279	3568	
Strand diameter (in.)	0.6		0.6		0.6	0.5	
Splitting reinforcement	14 D28	2 No. 5 + 2 No. 5 @ 2 in.	8 D28	2 No. 5 + 2 No. 5 @ 2 in.	2 No. 5 + 2 No. 5 @ 2 in.	4 TR1/2	4 TR3/4
Shear reinforcement	2 D18 @ 4 in.	2 D18 @ 4 in.	2 D18 @ 4 in.	2 D18 @ 4 in.	2 D18 @ 4 in.	2 D18 @ 4 in.	2 D18 @ 4 in.
Concrete strength, f'_{ci} (psi)	8669	6881	6003	5537	11,474	6821	
Concrete modulus, E_{ci} (ksi)	4385	4512	5102	4080	6213	4940	

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 sq in. = 645.2 mm^2; 1 in.^{4} = 416,231 mm^4; 1 psi = 0.0069 MPa; 1 ksi = 6.9 MPa.

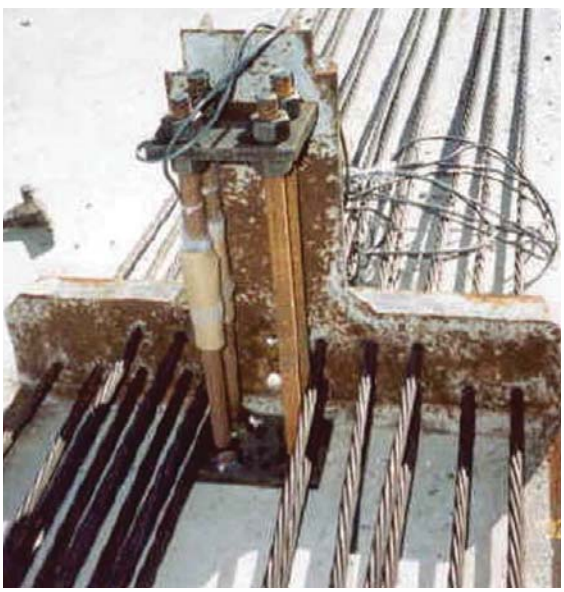


Fig. 10a. New end zone reinforcement detail – IT400.

Fig. 10b. Shear reinforcement placed after the end zone reinforcement – IT400.

bar shown in Fig. 11. It is still recommended that the inverted U-shaped bar be welded to the base plate for confinement and anchorage benefits.

This observation can also be made to the I-girders. The first reinforcing bar experienced the strain. The strain decreased rapidly from the girder end. The maximum stresses varied between 0.4 and 25.8 ksi (2.8 and 178 MPa). The crack width and length from the end were smaller than those observed using the current reinforcement details. Using a design steel stress of 20 ksi (138 MPa) works well with the end conditions in this phase. If the required reinforcement is spread over a relatively long distance, e.g. $h/2$ or h, the stress level as an average over the length would not be reached. On the other hand, it appears that the design is not optimum if wider cracks, represented by larger steel stress, can be tolerated. An extreme stress as high as 30 ksi is currently accepted in flexural design for crack control.

Again, the Gergely-Sozen method was an effective means of prediction of crack location. The cracks occurred around the maximum moment locations predicted using that method. The use of 0.6 in. prestressing strands to slightly increases the splitting force, as compared to that provided by 0.5 in. strands. This is illustrated in Fig. 8 and was factored into the recommended design procedure.

ANALYSIS OF END ZONE BEHAVIOR

Linear regression analysis was used to develop the relationship between stress in the splitting reinforcement and distance from girder end. This analysis was used to predict the tensile stresses in the reinforcement that were not directly instrumented. The total splitting force equaled the sum of individual bar forces. Table 5 shows the significant parameters affecting the splitting force in the tested girders. The ratio of the splitting force to prestress force ranges from 0.69 to 3.02 percent. The splitting force is the total force not limited to steel in the end (h/4) of the member. Thus, although the current AASHTO requires a 4 percent splitting force, the current overdesign is somewhat greater than the ratio 3.02/4 would indicate. According to the data shown in Fig. 8, it is conservative to use a splitting force of 3 percent of the prestressing force, even with 0.6 in. diameter strands, if the steel is concentrated very near the end of the member. Thus, the total area of end zone vertical reinforcement required to resist the splitting force at release is given by Eq. (2):

$$A_s = 0.03\frac{P_i}{f_s} \qquad (2)$$

where

A_s = total area of the end zone vertical reinforcement

P_i = initial prestressing force ($0.75f_{pu}$)

f_{sa} = average stress in the vertical end zone reinforcement

The splitting force, P_r, distribution along the member end, up to a length equal to the member depth, h, is given in Fig. 15. The best-fit curve of the test results can be represented by the logarithmic function, Eq. (3):

$$\frac{P_r}{P_i} = -0.0031\,Ln\left[\frac{z}{h}\right] - 0.00003 \qquad (3)$$

where z is distance of those bars from member end. Table 6 provides the splitting force ratio in each zone of Fig. 15. It shows that almost 40 percent of the splitting force is resisted a distance h/8 from the end, and about 85 percent of the force is resisted a distance h/2 from the end.

Fig. 16 illustrates average steel stress variation with distance from member end. It may be represented by Eq. (4):

$$f_s = \frac{2.4\text{ ksi}}{\frac{z}{h}+0.1} \qquad (4)$$

where

f_s = stress in end zone reinforcement a distance z from member end.

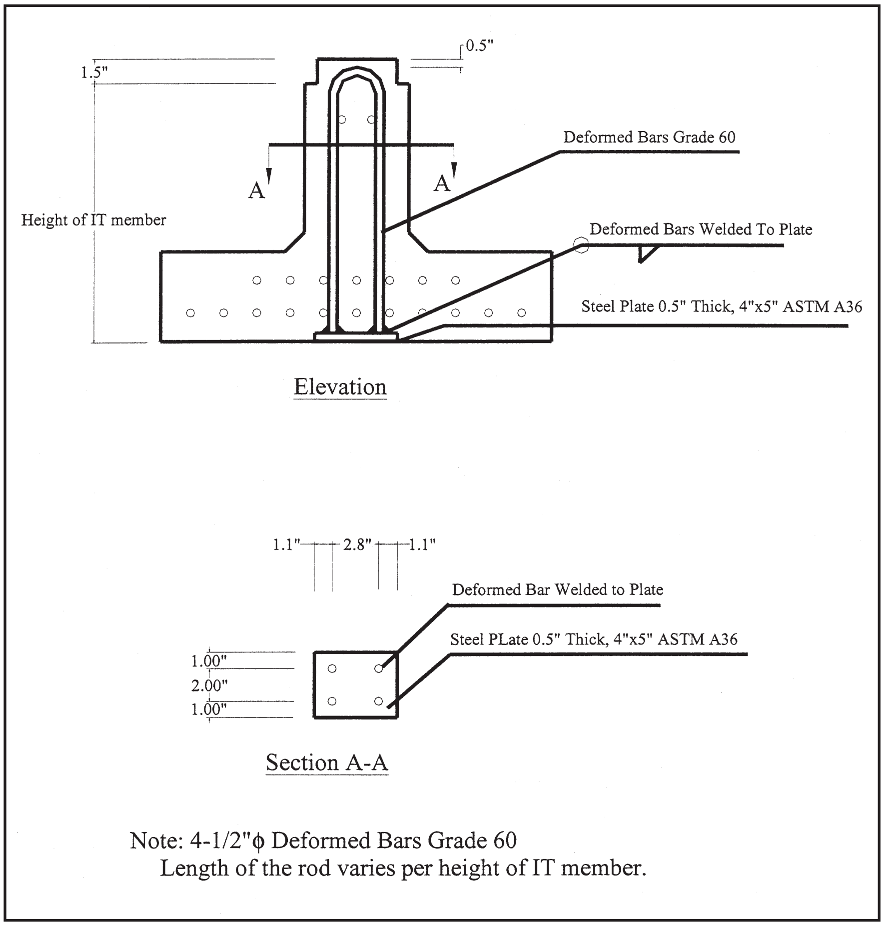


Fig. 11. Revised details of end zone reinforcement in IT member.

DESIGN OPTIONS

The designer has several options in designing the splitting reinforcement. The authors believe that it is most efficient to place the splitting reinforcement near the end of the member. A pair of very large bars or a structural steel shape designed for 2 percent of the prestress at a stress level of 20 ksi (138 MPa) can effectively control splitting cracks. The remaining balance of prestressing force can be resisted by the reinforcement placed in the member for shear resistance. However, this solution may be too expensive and/or too radical for many designers.

Alternatively, a splitting force of 4 percent of the prestress can be distributed a distance h/2 from the end, with at least 50 percent of that force placed a distance h/8 from the end. Since that zone resists about 85 percent of a splitting force of no more than 3 percent of the prestressing force, it is reasonable to apply a constant stress limit of 20 ksi (138 MPa) to the 4 percent force to obtain the required reinforcement. For the example given in Fig. 15, the equivalent splitting force, with 20 ksi (138 MPa) stress uniformly applied to the steel in Subzones A and B, is equal to:

$$20\left[\frac{0.39}{14.8}+\frac{0.21}{8.3}\right](0.03)P = 0.031P$$

Additional reinforcement in Subzones C and D due to shear design would likely produce a total splitting force resistance in the total zone greater than the 4 percent required in the current specification the measured splitting force of 3 percent.

RECOMMENDATIONS

1. The following procedure is recommended for designing the splitting reinforcement. Determine the total area of steel required to resist the splitting (bursting) force at member ends

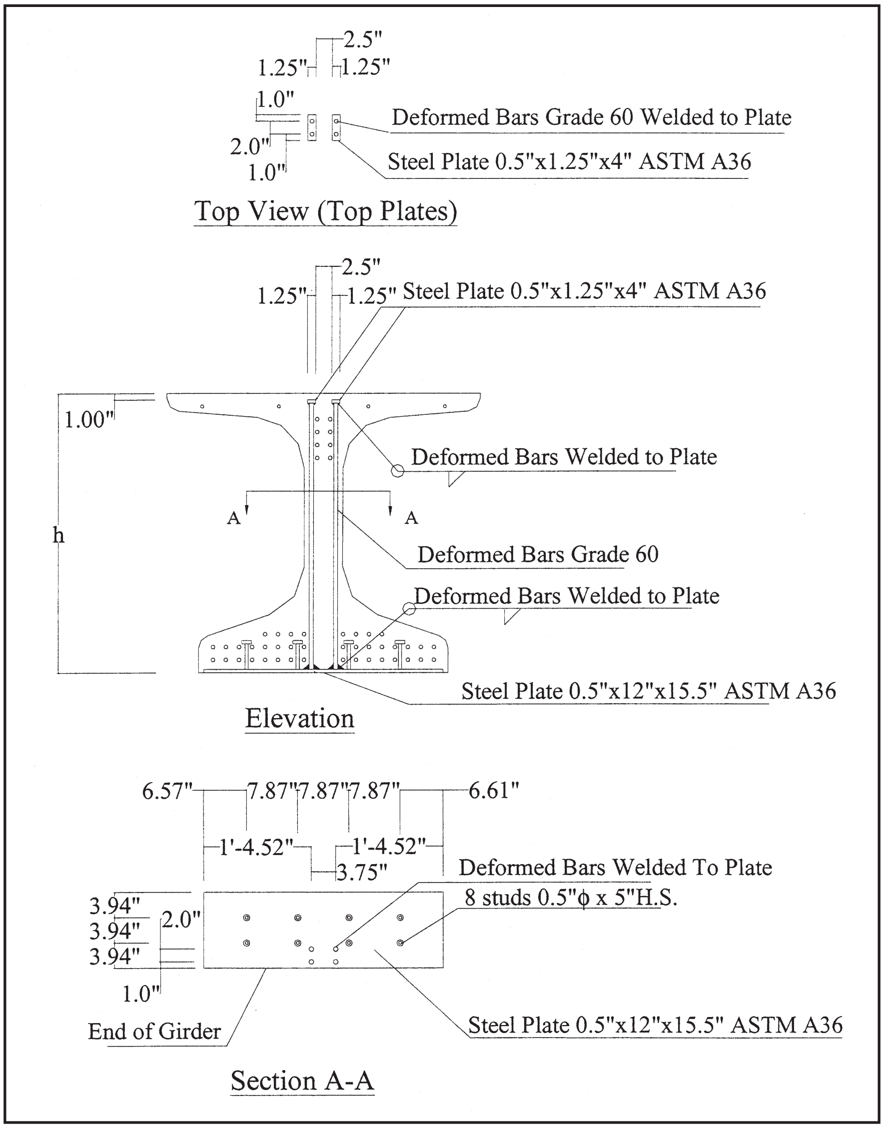


Fig. 12. Details of proposed end zone reinforcement.

Fig. 13. Proposed end zone detail for NU-I girders.

due to prestress transfer in pretensioned members from the relationship in Eq. (5):

$$A_s = 0.04\frac{P_i}{f_s} \tag{5}$$

where P_i is initial prestress just before release, and f_s is the steel stress limit taken as 20 ksi. For members with fully tensioned Grade 270, low-relaxation strands, Eq. (5) may be simplified as Eq. (6):

$$A_s = 0.4A_{ps} \tag{6}$$

where A_{ps} is the total area of pretensioning steel.

2. Place at least 50 percent of the area of splitting reinforcement as close to the member end as possible, but not beyond $h/8$ from the member, where h is the total member depth. This reinforcement should consist of bars that are welded to the plate at the bottom of the member and to small plates at the top, as shown in Figs. 11 to 14, in order to ensure adequate anchorage of that reinforcement. The bars may be as large as needed with a clear end cover of 1 in. (25 mm) to allow for top anchorage space and a clear spacing of 1 in. (25 mm) to allow for $^3/_4$ in. (19 mm) aggregates. If the end $h/8$ of the girder is not embedded in a cast-in-place concrete diaphragm, or corrosion is otherwise a concern, the welded assembly of bars, bottom plate and top plates should be galvanized.

3. Distribute the remainder of the area of splitting reinforcement evenly within the next zone ($3h/8$) of the member end. This zone should also be checked for critical shear section reinforcement and use the larger area reinforcement required. The reinforcement in this zone may be detailed in a conventional manner with standard hooks and longitudinal (cross) wires.

EXAMPLES

The following examples are intended to illustrate rather extreme applications encountered in the states of Nebraska and Washington in recent years. These applications were made possible through the use of I-girders with relatively large bottom flanges,

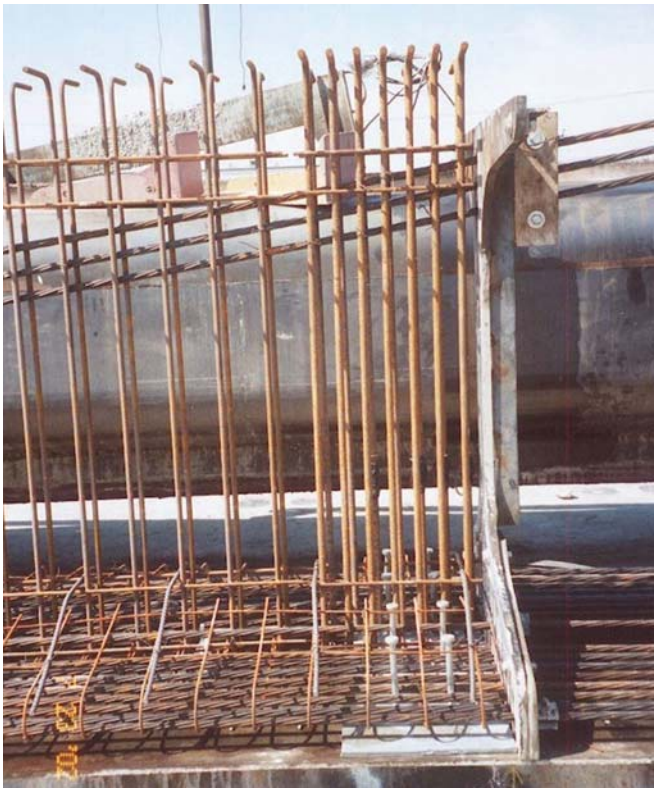


Fig. 14a. Current end zone detail.

Fig. 14b. Proposed end zone detail NU1100 I-girder NU1100 I-girder.

Table 4. Microstrain in the end zone reinforcement at release (Phase II).

	Location 1		Location 2		Location 3		Location 4	
Specimen	*z* (in.)	Microstrain	*z* (in.)	Microstrain	*z* (in.)	Microstrain	*z* (in.)	Microstrain
NU1600_1c	1	674	5	515	7	432	11	235
NU1600_2n	1	N/A	5	566	7.5	370	11.5	243
NU1600_3c	0.75	350	2.5	297	8.5	171	12.5	55
NU1600_4n	2.25	545	5.5	356	9.5	415	13.5	394
NU1100_1c	2	577	4	443	6	361	10	174
NU1100_2n	1	417	3	N/A	5	291	9	166
NU1100_3c	2	679	4	497	6	N/A	10	72
NU1100_4n	1	363	3	N/A	5	143	9	13
NU1100_5n	2	889	3.7	809	5.5	724	9.5	535
NU1100_6n	1.2	861	3	782	6	N/A	9.2	510
NU1100_7n	1.5	818	3.5	737	6.5	607	8.5	522
NU1100_8n	1.5	848	3.2	773	6.5	628	8.7	532
IT400_1n2	1	81	3	58	6	53	12	53
IT400_2n4	1	73	3	N/A	6	61	12	39

Note: 1 in. = 25.4 mm.

relatively large 0.6 in. (15 mm) diameter strands, and high strength concrete. These recent developments allow girders of limited depths to span farther than previously possible. As a result, however, large prestressing forces are encountered requiring large amounts of splitting reinforcement in a limited zone at member ends.

Example 1: Consider an NU1100 (43.3 in. deep) I-girder with 60–0.6 in. fully tensioned strands. The required splitting reinforcement = 0.4(60)(0.217) = 5.21 sq in. At least 5.21/2 = 2.60 sq in. should be placed within (43.3/8) = 5.41 in. from the member end. Use 2 No. 8 bars at 1.5 in. and 2 No. 8 bars at 3.5 in., for a total area of 4(0.79) = 3.16 sq in. The remaining area = 5.21 – 3.16 = 2.05 sq in. should be distributed between 3.5 in. and 43.3/2 = 21.65 in. measured from the end. If D18 welded wire reinforcement is selected, 2.05/0.18 = 12 vertical wires should be placed in that zone. The required maximum spacing of pairs of D18 is (21.65 – 3.5)/6, or 3 in. If at the critical section in shear, the shear stress required to be resisted

Table 5. Tensile forces in vertical reinforcement.

Phase	Specimen	P_r (kips)	P_i (kips)	L_t (in.)	h (in.)	y_b (in.)	M_{max} (ft-kips)	y_m (in.)	P_r/P_i (percent)
I	NU1800_1c	29.34	1549.1	30	70.9	32	372.9	30.25	1.89
	NU1800_2c	32.73	1549.1	30	70.9	32	372.9	30.25	2.11
	NU1800_3c	29.13	1549.1	30	70.9	32	372.9	30.25	1.88
	NU1600_1c	18.78	991.4	30	63.0	28.4	227.6	27.25	1.89
	NU1600_2c	12.41	991.4	30	63.0	28.4	227.6	27.25	1.25
	NU1600_3c	11.99	991.4	30	63.0	28.4	227.6	27.25	1.21
	IT600_1c	11.98	495.7	30	23.6	8.73	33.9	9.75	2.42
	IT600_2c	10.29	495.7	30	23.6	8.73	33.9	9.75	2.08
	IT600_3c	14.46	495.7	30	23.6	8.73	33.9	9.75	2.92
	IT400_1c	8.98	371.8	30	15.8	5.76	10.3	7.50	2.42
	IT400_2c	8.15	371.8	30	15.8	5.76	10.3	7.50	2.19
	IT400_3c	9.12	371.8	30	15.8	5.76	10.3	7.50	2.45
II	NU1600_1c	47.76	1581.9	36	63.0	28.4	222.7	26.00	3.02
	NU1600_2n	41.20	1581.9	36	63.0	28.4	222.7	26.00	2.60
	NU1600_3c	23.06	1581.9	36	63.0	28.4	222.7	26.00	1.46
	NU1600_4n	45.95	1581.9	36	63.0	28.4	222.7	26.00	2.90
	NU1100_1c	30.82	1054.6	36	43.3	19.6	182.7	15.20	2.92
	NU1100_2n	22.01	1054.6	36	43.3	19.6	182.7	15.20	2.09
	NU1100_3c	29.06	1054.6	36	43.3	19.6	182.7	15.20	2.76
	NU1100_4n	26.86	1054.6	36	43.3	19.6	182.7	15.20	2.55
	NU1100_5n	54.55	1933.5	36	43.3	19.6	253.9	17.50	2.82
	NU1100_6n	53.00	1933.5	36	43.3	19.6	253.9	17.50	2.74
	NU1100_7n	47.72	1933.5	36	43.3	19.6	253.9	17.50	2.47
	NU1100_8n	49.36	1933.5	36	43.3	19.6	253.9	17.50	2.55
	IT400_1n2	4.26	619.7	30	15.8	5.76	16.3	7.75	0.69
	IT400_2n4	4.92	619.7	30	15.8	5.76	16.3	7.75	0.79

Note: 1 in. = 25.4 mm; 1 kip = 4.45 kN; 1 ft-kip = 1.35 kN-m.

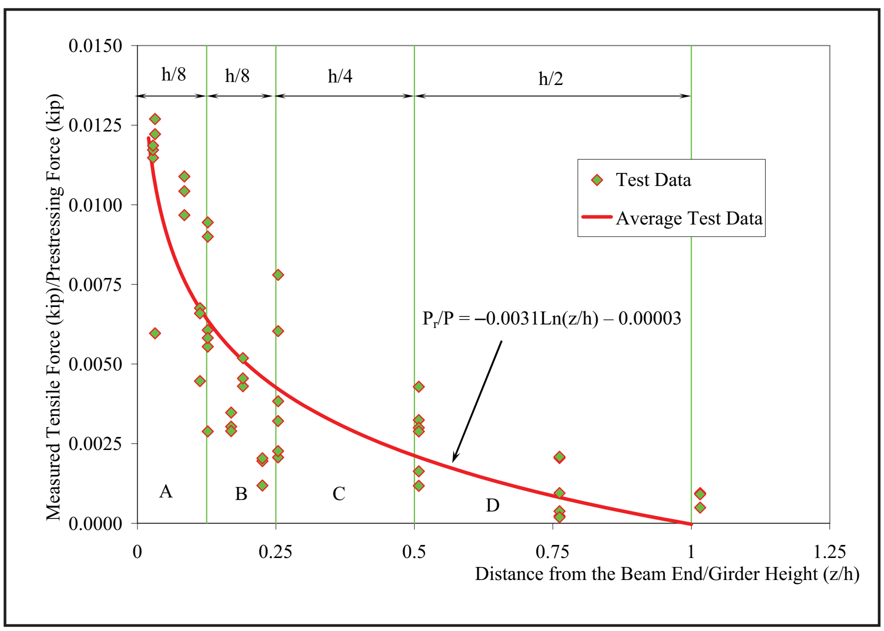


Fig. 15. Variation of force in vertical reinforcement due to prestressing force.

by the shear reinforcement, A_v, is 1.8 ksi. Considering the web width, b_w, of NU I-girder series of 150 mm (5.9 in.), the required spacing of D18 Grade 60 wire is $A_v f_y / v_s b_w$ = (2)(0.18)(60)/(1.8)(5.9) = 2 in. Thus, the spacing of 2 in. required for shear resistance controls the design of this zone. The end h/2 of the member is, therefore, to be reinforced with 4 No. 8 bars plus 16 D18 at a center-to-center spacing of 2 in. The total distance over which this reinforcement is provided is 1 in. plus 10 spaces at 2 in. = 21 in.

Example 2: Consider a W83MG Washington Super Girder. The total depth is 2100 mm (82.68 in.) and the web width is 155 mm (6.10 in.). The prestressing consists of 74–0.6 in. strands having a total area of 16.058 sq in. The required splitting reinforcement = 6.42 sq in. Provide at least 3.21 sq in. within the first 10.4 in. of the member. Use 4 No. 8 bars (3.16 sq in.) at 2 in. in pairs at 2 in. spacing. If a minimum cover of 1 in. must be satisfied, 8 No. 6 bars (3.52 sq in.) in pairs at 2 in. may be used. The remaining 2.9 sq in. may be distributed over a distance = 41.34 – (1.375 + 2 + 2 + 2) = 33.96 in. This would correspond to 16 No. 4 bars in pairs at 4 in. spacing. The shear capacity is assumed here not to control the design of the calculated reinforcement.

CONCLUSIONS

Based on the results in this investigation, the following conclusions are made:

1. Calculation of the amount and location of splitting reinforcement at the ends of pretensioned concrete members, to control splitting cracks at time of prestress transfer, is complicated due to the large number of contributing factors and the large random variability of the parameters involved. Unlike post-tensioning where the entire prestressing force is introduced at the member end, transfer of pretensioning force to concrete occurs gradually through bond between individual strands and the surrounding concrete. As the prestress spreads into the member cross section concrete material nonlinear behavior and cracking contribute to the inability to develop a closed form solution.

2. The Marshall-Mattock research in the 1960s gave a solid foundation for semi-empirical design of splitting reinforcement, although it was based on small laboratory experiments with very low prestressing forces. Left in its original form, it would give more accurate results than the simplified version included in the current AASHTO Specifications.

3. Current mathematical modeling techniques include the strut-and-tie method, finite element analysis and Gergely-Sozen equivalent girder analysis. The strut-and-tie modeling method produces an upper bound solution that is generally too conservative. Finite element analysis is too complex to use in conventional design and generally includes unrealistic simplifying assumptions. The Gergely-Sozen method is based on simple flexure analysis. It has been shown in this research to accurately predict cracking in the full-scale specimens.

4. An upper bound value of the splitting force, based on the experiments conducted in this research, is 3 percent of the prestressing force with 0.6 in. (15 mm) diameter strands. The splitting force is somewhat lower when 0.5 in. (13 mm) diameter strands are used. The stress in the splitting reinforcement is close to 25 ksi (172 MPa) in the bars nearest the member ends. It diminishes very sharply as the distance from the end increases. The steel stress may be assumed to practically disappear at a distance equal to the member depth. These findings are consistent with those of Marshall-Mattock. Initiation of cracking in the specimens was accurately predicted by the Gergely-Sozen method.

5. About 60 percent of the splitting force develops in the first $h/4$ of the member. About 85 percent develops in the first $h/2$. Because the effectiveness of the steel is largely a function of how close it is to the member end, design for a uniform stress of 20 ksi (138 MPa) would require inputting a splitting force larger than the measured 2 to 3 percent of the prestressing force.

6. The most effective reinforcement for control of end-zone splitting cracks is that placed at the very end of the member. Such special reinforcement may be too expensive to use.

7. A realistic solution is to design the splitting reinforcement for a force equal to 4 percent of the prestressing force and a uniform stress of 20 ksi (138 MPa). To allow for this high average stress to be used, at least 50 percent of that reinforcement should be placed 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.

8. The proposed procedure requires the same total splitting reinforcement area as the current AASHTO Specifications. Thus, it may be seen as a validation of the AASHTO provisions.

9. Calculation and detailing of the splitting reinforcement results in improved crack control due to the special anchorage requirements of the large end bars and in reduced end zone congestion when the steel is distributed over $h/2$ rather than the current $h/4$ requirement in AASHTO.

10. The proposed method may still be too conservative. Additional experiments are needed to investigate the possibility of reducing the splitting reinforcement with increased stress limit to 30 ksi (207 MPa) or even 36 ksi (248 MPa). These two values are sometimes accepted for crack control in flexural design. With adequate ex-

Table 6. Splitting force ratios.

Subzone	A	B	C	D
Length	$h/8$	$h/8$	$h/4$	$h/2$
Bursting force, percent	39	21	25	15

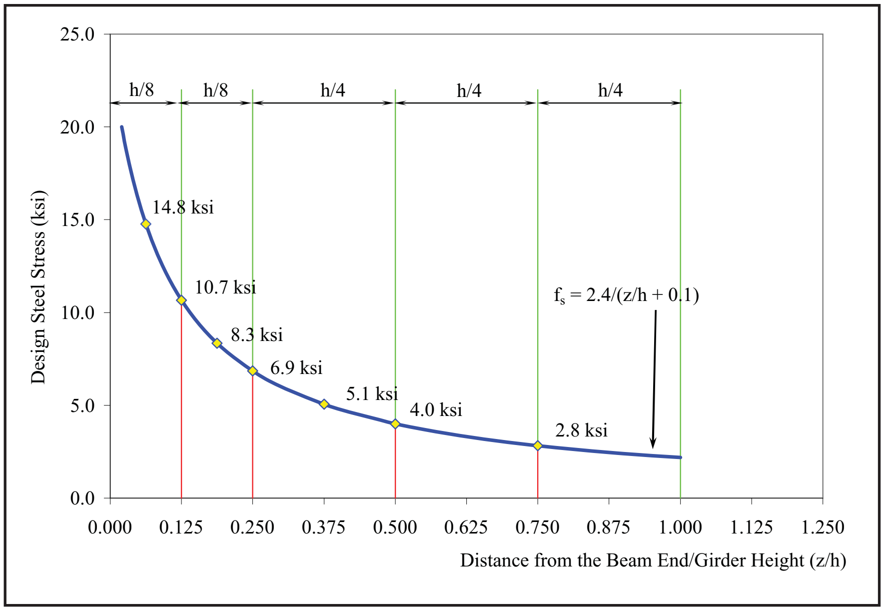


Fig. 16. Proposed design stress distribution in the end zone reinforcement.

perimental justification, it may be possible to raise the 20 ksi (138 MPa) limit.

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APPENDIX – NOTATION

A_{ps} = area of prestressing steel
A_s = area of the end zone vertical reinforcement
b_w = width of girder web
E_{ci} = modulus of elasticity of concrete at time of prestress release
f'_{ci} = concrete strength at release
f_{pu} = specified tensile strength of prestressing strand
f_s = stress in end zone reinforcement at distance z from member end
f_{sm} = maximum stress in end zone reinforcement
f_{sa} = average stress in the vertical end zone reinforcement
h = girder depth
l_t = transfer length of the strand
M_{max} = maximum moment in girder section
P_i = initial prestressing force, assumed $0.75f_{pu}$ for low relaxation strands
P_r = splitting force due to release of pretensioned strand to concrete
y_b = distance from centroid to bottom fiber of cross section
y_m = distance between location of maximum moment and bottom fiber of cross section
z = distance of vertical reinforcement from member end