Evaluation and Modifications of the AASHTO Procedures for Flexural Strength of Prestressed Concrete Flanged Sections

Eray Baran
University of Minnesota
Civil Engineering Department
Minneapolis, MN
bara0045@umn.edu

Arturo E. Schultz
University of Minnesota
Civil Engineering Department
Minneapolis, MN
schul088@umn.edu

Catherine E. French
University of Minnesota
Civil Engineering Department
Minneapolis, MN
cfrench@umn.edu

ABSTRACT

Different interpretations of the equivalent rectangular stress block approximation used by the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) and AASHTO Standard Specifications may lead to inconsistencies in the sectional response of the prestressed concrete flanged sections predicted by the two specifications. According to the LRFD specifications, the limits on the contribution of the compression flange overhangs causes the neutral axis to be located lower in the web of the section to satisfy the internal force equilibrium. This artificial overestimation of the neutral axis depth according to the LRFD Specifications as compared to the Standard Specifications, leads to inconsistencies in the provisions that depend on neutral axis depth, such as whether or not the section is over-reinforced, as well as differences in nominal moment capacity.

In this paper, provisions of the AASHTO LRFD and Standard Specifications are compared to the results from the nonlinear strain compatibility analyses of prestressed concrete nonrectangular sections. The purposes of this research are to illustrate the discrepancies that exist in the sectional response predicted by these specifications and to identify design procedures that more accurately represent the actual behavior of such members.

Modifications to the AASHTO LRFD procedure are proposed to correct for errors in determining the contribution of compression flange overhangs. Improvements in the accuracy of predicted sectional response and maximum reinforcement limits are demonstrated through a set of examples. Flexural strengths predicted by the specifications, the proposed modified LRFD procedure, and the strain compatibility analyses were also compared to measured flexural strengths of prestressed concrete I-beams found in the literature to validate the proposed modifications.

Key words: equivalent stress block—load and resistance factor design—prestressed concrete—standard specifications—T-section

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PROBLEM STATEMENT

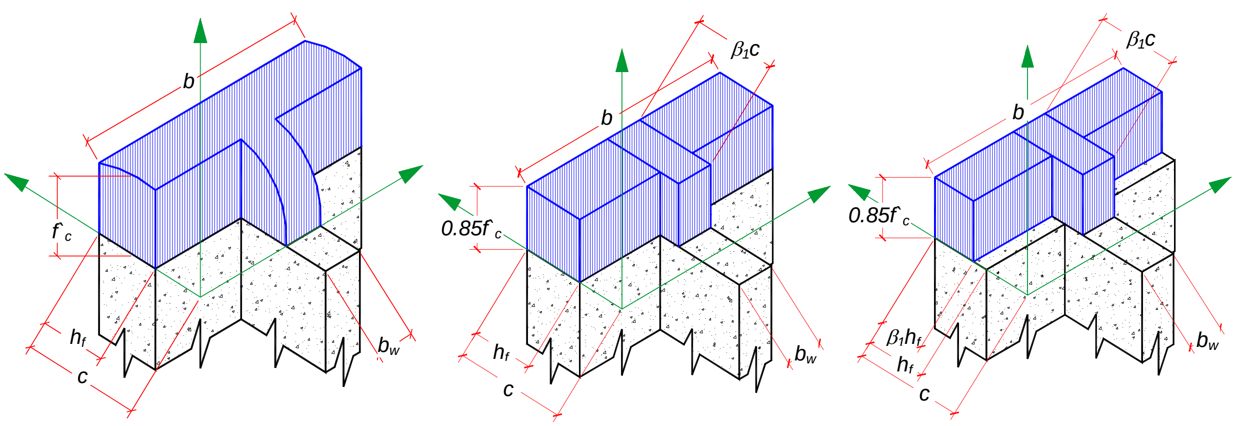
Inconsistencies in the sectional response of prestressed concrete flanged sections, as predicted by the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications (2002) and the AASHTO Load and Resistance Factor Design (LRFD) Specifications (1998), may arise due to different interpretations of the equivalent compressive stress block idealization (Badie and Tadros 1999; Seguirant 2002; Naaman 2002; Naaman 2002a; Rabb 2003; Girgis, Sun, and Tadros 2002; Weigel, Seguirant, Brice, and Khaleghi 2005; Seguirant, Brice, and Khaleghi 2005). For sections with T-section behavior at ultimate capacity, an artificial overestimation of the neutral axis depth according to the LRFD Specifications, as compared to the Standard Specifications, leads to inconsistencies in the provisions that depend on neutral axis depth, such as whether or not the section is over-reinforced, as well as differences in nominal moment capacity.

To identify the inconsistencies, the sectional responses of several prestressed concrete nonrectangular sections were predicted by the AASHTO LRFD and Standard Specifications (Baran, Schultz, and French 2005). The sections were also analyzed using strain compatibility analysis that incorporated nonlinear material properties. This paper presents a comparison of the sectional response that the specifications predicted and that determined by the nonlinear strain compatibility analysis. A comparison of the measured flexural strengths of prestressed concrete I-beams found in the literature with those predicted by the specifications and the proposed procedure is also included.

BACKGROUND

Equivalent Stress Block Approximation

Both the AASHTO Standard Specifications and the AASHTO LRFD Specifications approximate the actual nonlinear concrete compressive stress distribution at nominal capacity (Figure 1) with the Whitney rectangular stress block, which has an average compressive stress of $0.85f'_c$ "uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a line parallel to the neutral axis at a distance $a=\beta_1 c$ from the extreme compression fiber." However, there are two differences in the implementation of the equivalent rectangular compression block according to the two specifications. As illustrated in Figure 1, the LRFD Specifications limit the depth of the compression flange overhangs contributing to the internal compressive force to $\beta_1 h_f$. In the case of AASHTO Standard Specifications, on the other hand, a full compression flange depth of h_f can contribute to the internal compressive force.



(a) Actual Nonlinear Stress Distribution (b) AASHTO Standard Specifications (c) AASHTO LRFD Specifications

Figure 1. Compressive stress distributions for a T-section

The second difference between the two specifications is in the limiting value of the neutral axis depth at which the transition from rectangular section behavior to T-section behavior occurs. According to the Standard Specifications, T-section behavior starts when $a=\beta_1 c$ becomes equal to the compression flange depth, h_f, while the LRFD Specifications consider the section as a T-section as soon as c reaches h_f.

The differences in the way the two specifications treat the overhanging portions of the top flange of nonrectangular sections result in overestimation of the neutral axis depth, according to the LRFD Specifications, which leads to inconsistencies in the determination of whether the section is considered over-reinforced, as well as in the resulting flexural capacity.

Reinforcement Limits

The amount of tensile reinforcement that can be placed in a prestressed concrete section is limited in both the LRFD and the Standard Specifications. The two specifications define the maximum reinforcement limits in terms of different parameters. AASHTO Standard Specifications limit the maximum value of reinforcement index (Equation 1-a), ω_w, while the LRFD Specifications limit the value of the ratio of neutral axis depth, c, to effective depth, d_e (Equation 1-b).

$$\omega_w = \frac{A_{sr} f_{ps}}{b_w d f'_c} \leq 0.36\beta_1 \qquad \text{(1-a)}$$

$$\frac{c}{d_e} \leq 0.42 \qquad \text{(1-b)}$$

Even though the Standard and the LRFD Specifications define the maximum reinforcement limit using different criteria, two parameters (the reinforcement index, ω_w, and the c/d_e ratio) are related to each other through the equilibrium of internal tensile and compressive forces.

Limiting the maximum tensile reinforcement in flexural members dates back to the 1971 edition of the ACI Code (1971), which placed an upper limit of 0.30 on the reinforcement index, ω, which is directly proportional to the amount of tensile reinforcement. The first appearance of a limit on the reinforcement index equal to 0.30 is found in the report titled “Tentative Recommendations for Prestressed Concrete” by the ACI-ASCE Joint Committee 323 (1958). The justification for such a limit was expressed as the need “to avoid approaching the condition of over-reinforced beams for which the ultimate flexural strength becomes dependent on the concrete strength.”

In both the LRFD and Standard Specifications, the sections with tensile reinforcement exceeding the maximum reinforcement limit are termed “over-reinforced.” By preventing the use of the full flexural capacity of over-reinforced sections, the specifications impose an additional safety margin to account for the limited ductility of those sections. In other words, the specifications permit the use of prestressed concrete sections with steel amounts exceeding the maximum limit, but with a usable flexural strength that is less than the actual strength of the section.

Even though the specifications penalize the use of over-reinforced sections by making a trade-off between ductility and strength, Provision 5.7.3.3.1 of the AASHTO LRFD Specifications states that “Over-reinforced sections may be used in prestressed and partially prestressed members only if it is shown by analysis and experimentation that sufficient ductility of the structure can be achieved.” This statement effectively penalizes the design and use of prestressed and partially prestressed over-reinforced sections more severely than provisions that simply limit the flexural resistance, as is done in the Standard Specifications.

Strength Considerations

As mentioned earlier, AASHTO Standard Specifications assume that T-section behavior begins when the depth of the equivalent rectangular compressive stress block, $a=\beta_1 c$, drops below the top flange of the section. In this case, Equation 2-a is the expression for the internal equilibrium of the compressive force in the concrete and tensile force in the prestressing steel. As seen in the second term of the left-hand side of Equation 2-a, the Standard Specifications allow the full flange depth of h_f to contribute to the total compressive force carried by the section. In this case, the contribution of the top flange overhangs to the total internal compressive force is $0.85 f'_c (b - b_w) h_f$.

$$(0.85 f'_c)(\beta_1 c) b_w + (0.85 f'_c) h_f (b - b_w) = A_{ps} f_{ps} \tag{2-a}$$

In the case of AASHTO LRFD Specifications, the expression for the internal force equilibrium of a T-section is the one given in Equation 2-b. As seen in the second term of the left-hand side of the equilibrium expression, the LRFD Specifications limit the depth of the equivalent rectangular stress block acting on the flange overhangs to $\beta_1 h_f$. This implicitly means that, in the case of a T-section, the full depth of the top flange never contributes to the total compressive force when the equivalent rectangular stress block assumption is used, regardless of the magnitude of c. This assumption results in an overestimation of the neutral axis depth, c, for the LRFD Specifications in comparison with the Standard Specifications, since the web contribution must increase to compensate for the portion of the top flange that is being neglected.

$$(0.85 f'_c)(\beta_1 c) b_w + (0.85 f'_c)(\beta_1 h_f)(b - b_w) = A_{ps} f_{pu} \left(1 - k \frac{c}{d_p}\right) \tag{2-b}$$

Strand Stress

The LRFD and Standard Specifications use different procedures to predict the stress in the prestressing steel at nominal capacity. In the procedure used by the Standard Specifications, the strand stress is predicted by Equation 3-a, which is independent of the neutral axis depth.

$$f_{ps} = f_{pu} \left[1 - \left(\frac{\gamma}{\beta_1} \right) \left(\rho \frac{f_{pu}}{f'_c} \right) \right] \tag{3-a}$$

In the LRFD procedure, on the other hand, the location of the neutral axis is determined first using Equation 2-b, which implicitly includes an assumed value for the strand stress. With this estimate of the neutral axis location, strand stress is computed using Equation 3-b.

$$f_{ps} = f_{pu} \left(1 - k \frac{c}{d_p} \right) \tag{3-b}$$

As seen, the equation used by the Standard Specifications (Equation 3-a) is independent of the neutral axis depth. At the ultimate capacity of a prestressed concrete section, any change in the neutral axis location will result in a change in the strand strain, and hence in the strand stress. The procedure used by the Standard Specifications to determine the strand stress cannot represent this behavior. The LRFD procedure, on the other hand, satisfactorily takes the changes in neutral axis location into account when determining the strand stress at ultimate capacity.

Nominal Moment Capacity

Both the AASHTO Standard and AASHTO LRFD Specifications use formulas for computing the flexural strength of over-reinforced sections that differ from those used for under-reinforced sections. The AASHTO Standard and LRFD Specifications use Equations 4-a and 4-b, respectively, for the calculation of moment capacity of under-reinforced prestressed concrete sections.

$$M_n = A_{sr} f_{ps} d \left[1 - 0.6 \left(\frac{A_{sr} f_{ps}}{b_w d f'_c} \right) \right] + 0.85 f'_c \left(b - b_w \right) h_f \left(d - 0.5 h_f \right) \quad \text{(4-a)}$$

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right) + 0.85 f'_c \left(b - b_w \right) \left(\beta_1 h_f \right) \left(\frac{a}{2} - \frac{h_f}{2} \right) \quad \text{(4-b)}$$

For the moment capacity of over-reinforced sections, on the other hand, the Standard and LRFD Specifications recommend using Equations 5-a and 5-b, respectively.

$$M_n = \left(0.36 \beta_1 - 0.08 \beta_1^{\,2} \right) f'_c b_w d^2 + 0.85 f'_c \left(b - b_w \right) h_f \left(d - 0.5 h_f \right) \quad \text{(5-a)}$$

$$M_n = \left(0.36 \beta_1 - 0.08 \beta_1^{\,2} \right) f'_c b_w d_e^{\,2} + 0.85 f'_c \left(b - b_w \right) \left(\beta_1 h_f \right) \left(d_e - 0.5 h_f \right) \quad \text{(5-b)}$$

The last two equations are obtained by substituting into Equations 4-a and 4-b, respectively, the maximum amount of tensile reinforcement allowed by Equations 1-a and 1-b. Through the use of Equations 5-a and 5-b, in effect, the flexural strength of over-reinforced sections is limited to the value of the moment capacity corresponding to the maximum limit of tensile reinforcement. Any additional capacity that may be provided by having more steel than allowed by the reinforcement limits is neglected. This limitation on moment capacity is intended to ensure that sections with limited ductility have reserve moment capacity.

RESEARCH METHODOLOGY

This paper summarizes some of the findings of a study conducted at the University of Minnesota to investigate precast concrete T-section behavior at nominal strength. The study included a comparison of sectional analyses of several reinforced and prestressed concrete nonrectangular sections, following the procedures given in the AASHTO Standard Specifications, the AASHTO LRFD Specifications, a nonlinear strain compatibility analysis, and a modified LRFD procedure (Baran, Schultz, and French 2005).

In addition, analytical results were compared to the test results of prestressed concrete I-beams found in the literature, which were identified as over-reinforced and as having neutral axis depths within the web at nominal strength.

Strain compatibility analyses were conducted using RESPONSE-2000, a sectional analysis program by Bentz and Collins (2001) incorporating a nonlinear stress-strain material model. A computer code utilizing internal force equilibrium and strain compatibility between steel and concrete was also developed to verify the results obtained using RESPONSE-2000. For this paper, the results of sectional analyses performed using RESPONSE-2000 were used to make comparisons between the sectional responses predicted by the AASHTO Specifications. When the term “strain compatibility” is used, it refers to the strain compatibility analyses performed using RESPONSE-2000 with nonlinear material models for the concrete and the steel.

ANALYSIS OF SECTION BEHAVIOR

Limits of T-Section Behavior

Analyses were conducted to reproduce the results given in Figure C5.7.3.2.2-1 of the LRFD Specifications (Figure 2 in this paper). This chart, originally developed by Naaman (1992), is used to indicate the difference in neutral axis depth with the amount of steel for the AASHTO LRFD and AASHTO Standard (and ACI) Specifications for the T-section shown. Note that this figure as given in the LRFD Specifications contains an incorrect interpretation of the neutral axis depth for the ACI and AASHTO Standard Specifications.

To compare the neutral axis locations calculated according to the specifications with those from the strain compatibility study (superimposed on the chart with triangle symbols), the plot has been divided along the x-axis into three regions to compare the neutral axis depths. Region I represents those cases for which c is always smaller than h_f, and both the LRFD and Standard Specifications indicate rectangular section behavior. As shown, in this region the neutral axis depth values calculated according to both specifications with the equivalent rectangular stress block assumption agree with the values obtained from the nonlinear strain compatibility analyses.

Region II in Figure 2 covers those cases for which the neutral axis begins to drop below the flange. In this region, the LRFD Specifications indicate T-section behavior while the Standard Specifications still indicate that the section behaves as a rectangular section. In Region II, the Standard Specifications approximate the location of the neutral axis with acceptable accuracy compared to the strain compatibility analyses, while the procedure in the LRFD Specifications grossly overestimates the neutral axis depth.

In Region III, T-section behavior is predicted by both the LRFD and Standard Specifications. As shown in Figure 2, for a specific value of steel area, both specifications overestimate the neutral axis depth compared to the strain compatibility analyses, but with much larger errors for neutral axis depth calculations using the LRFD Specifications.

The dashed line in Figure 2 is used in provision C5.7.3.2.2 of AASHTO LRFD Specifications to show the inconsistency between the AASHTO LRFD and the AASHTO Standard and ACI Specifications. This dashed line is actually a misinterpretation of the neutral axis depth in the ACI and Standard Specifications. The plot is based on the assumption that T-section behavior initiates when $c \geq h_f$, taking the total depth of the flange as effective in compression at that point, and thereby indicating a negative neutral axis depth in the web for equilibrium. The correct assumption, denoted by the solid line in Figure 2, is that T-section behavior initiates when the depth of the equivalent rectangular stress block exceeds the flange depth (i.e., $a = \beta_1 c \geq h_f$).

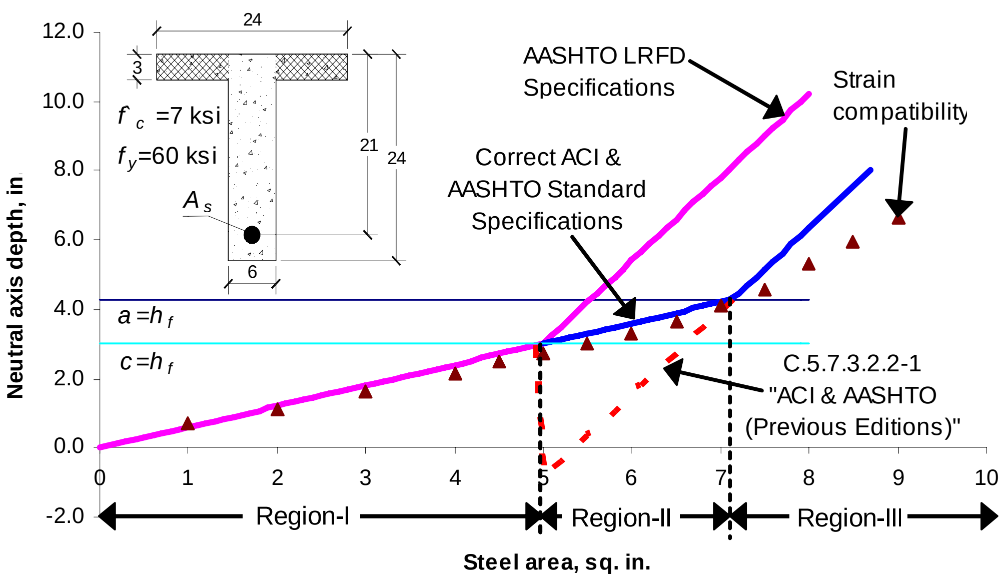


Figure 2. Change in neutral axis depth with amount of steel in a T-section

MODIFICATION OF LRFD PROCEDURE

Proposed Changes

As noted earlier, the LRFD Specifications overestimate the neutral axis depth for two reasons; the first reason is the use of $c=h_f$ as the limit for T-section behavior, and the second reason is the use of the $\beta_1 h_f$ limit for the maximum flange overhang contribution to the internal compressive force once T-section behavior begins. A modification, which overcomes both of these problems, was proposed to the procedure outlined in the LRFD Specifications to indicate that T-section behavior begins when $a = \beta_1 c = h_f$, rather than when $c = h_f$ (Baran, Schultz, and French 2005). This modification also fixes the second problem mentioned above by enabling the entire flange depth to become effective when $a \geq h_f$.

Because the neutral axis depth and moment capacity of under-reinforced sections and the moment capacity of over-reinforced sections depend on the amount of flange overhang contribution to the internal compressive force, the corresponding equations in the LRFD Specifications (Equations 5.7.3.1.1-3, 5.7.3.2.2-1, and C5.7.3.3.1-2) were also modified to remove the $\beta_1 h_f$ limit on the contribution of the flange overhangs. After the modification, Equations 5.7.3.1.1-3, 5.7.3.2.2-1, and C5.7.3.3.1-2 in the LRFD Specifications (Equations 2-b, 4-b, and 5-b, respectively, in this paper) take the forms of Equations 6, 7, and 8, respectively.

$$(0.85 f'_c)(\beta_1 c) b_w + (0.85 f'_c)(h_f)(b - b_w) = A_{ps} f_{pu} \left(1 - k \frac{c}{d_p}\right) \quad (6)$$

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2}\right) + 0.85 f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right) \quad (7)$$

$$M_n = \left(0.36\beta_1 - 0.08\beta_1^2\right) f'_c b_w d_e^2 + 0.85 f'_c (b - b_w) h_f (d_e - 0.5 h_f) \quad (8)$$

The procedure in the LRFD Specifications using Equations 6, 7, and 8 instead of Equations 5.7.3.1.1-3, 5.7.3.2.2-1, and C5.7.3.3.1-2 is referred to as the modified LRFD procedure in the rest of this paper.

Verification of Proposed Changes

The Minnesota Department of Transportation (Mn/DOT) Type 63 section shown in Figure 3 was analyzed for various amounts of prestressed steel to investigate the relationship between the sectional response predicted by the LRFD and Standard Specifications, strain compatibility analyses, and the modified LRFD procedure. This section is currently being used in Minnesota for prestressed concrete through-girder pedestrian bridge construction with typical spans on the order of 135 ft. Because of the through-girder–type construction, no composite deck exists on top of the girders, and there is interest in a more accurate evaluation of the strength and ductility of these girders.

The large span length requires the use of a large number of strands to control deflections. In addition, the section has a narrow top flange ($b_{flange}/b_{web} = 5$). Because neither a composite deck nor a wide top flange are provided to help carry the compressive part of the internal couple, the neutral axis is located within the web of the section, causing the section to be over-reinforced according to the LRFD Specifications, and consequently causing it to fail to meet the required strength. With the narrow top flange, the difference between the response of the section predicted by the LRFD Specifications and Standard Specifications is less significant than it would be for a section with a wider top flange. From this aspect, this section provides a lower bound for the difference between the sectional quantities predicted by the LRFD and Standard Specifications.

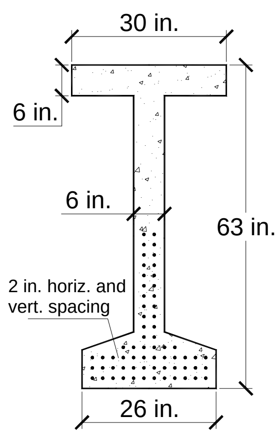


Figure 3. Mn/DOT type 63 section

The Mn/DOT Type 63 section was analyzed assuming an 8.2-ksi concrete strength and 0.5-in. diameter strands with an effective prestress of 162 ksi ($0.60f_{pu}$). The number of strands varied from 20 to 60, and the strands were placed in the typical pattern used for this type of section, that is, spaced two inches on-center in the horizontal and vertical directions. Thus, the depth to the center of gravity of strands was lowered as the number of strands increased. The results of the analyses are shown in Figure 4.

Figure 4a is similar to Figure 2, which was for a 24-in. deep reinforced concrete T-section. For the present case, the LRFD Specifications assume that T-section behavior starts when c exceeds h_f, and, as shown in Figure 4, the LRFD Specifications begin to overestimate the neutral axis depth when there are 20 strands in the section. The Standard Specifications, on the other hand, indicate almost the same neutral axis location as the strain compatibility analysis until the number of strands is increased to 32. For this amount of prestressed steel, the Standard Specifications begin to treat the section as a T-section, and overestimate *c* in comparison to the strain compatibility analysis.

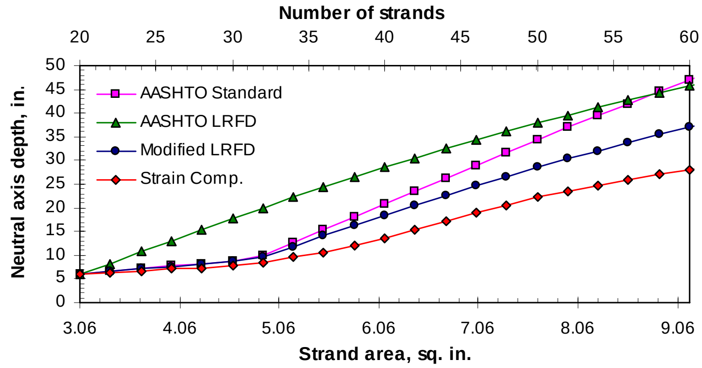
The neutral axis depth values computed with the modified LRFD procedure are also shown in Figure 4a. As for the Standard Specifications, T-section behavior starts at 32 strands for the modified LRFD procedure. As seen in Figure 4, there is better agreement between the neutral axis depth values computed with the modified LRFD approach and the strain compatibility results than there is with either the Standard or the LRFD Specifications.

The change in strand stress at the ultimate capacity of the section is shown in Figure 4b. As illustrated, the LRFD Specifications underestimate the strand stress compared to the strain compatibility analysis, while the Standard Specifications slightly overestimate it. When the LRFD Specifications are modified, as mentioned previously, the results fall into close agreement with those obtained by the strain compatibility analysis.

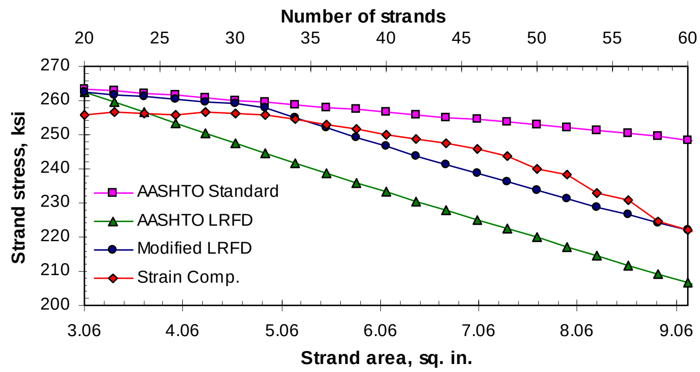
Figure 4c shows the variation among the nominal bending resistances calculated according to the LRFD, Standard Specifications, modified LRFD procedures, and strain compatibility analysis as the amount of prestressed steel is varied. Once T-section behavior begins (i.e., when there are 20 strands according to the LRFD Specifications) the LRFD Specifications begin to underestimate bending capacity. As the number of strands increases, the depth of the web participating in the internal compressive force increases until the section becomes over-reinforced at 38 strands.

After the section becomes over-reinforced (i.e., at 44 strands according to the Standard Specifications, at 38 strands according to the LRFD Specifications, and at 46 strands according to the modified LRFD procedure) Equations 5-a, 5-b, and 8 are used to compute the nominal bending resistance according to the Standard Specifications, the LRFD Specifications, and the modified LRFD procedure, respectively. These equations include only the geometric terms of the section and the concrete strength, and are independent of the amount of steel present in the section. As seen in Figure 4-c, the moment capacity of the section decreased after the section became over-reinforced according to the specifications. This occurred because the LRFD and Standard Specifications use the distance from the extreme compression fiber to the centroid of the strands to compute the moment capacity of over-reinforced sections, and this distance decreases with increasing number of strands as the strands were distributed through the depth of the section in this example.

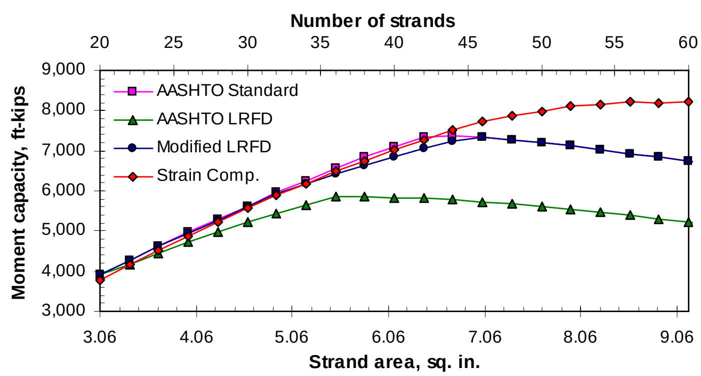
Note that once the section became over-reinforced according to the Standard Specifications and the modified LRFD procedure, both specifications (modified LRFD and Standard) indicated the same moment capacity values for an increasing number of strands. This is so because, according to both procedures, the moment capacity of over-reinforced sections is computed based solely on the compressive portion of the internal couple.



(a) Neutral axis depth, *c*.



(b) Strand stress at nominal capacity, f_{ps}.



(c) Moment capacity, M_n.

Figure 4. Variation in section response with amount of prestressed steel (63-in. deep section)

The inconsistency described above between the moment capacities of over-reinforced sections severely limits practitioners' choices, as the LRFD Specifications penalize the use of these so-called over-reinforced sections in two ways: (1) by placing a conservative limit on nominal bending resistance, and (2) by requiring additional analyses and experimentation to show that there is sufficient ductility. As shown in Figure 4, modifying the LRFD procedure as described earlier minimizes the inconsistency between the moment capacities calculated according to the LRFD and Standard Specifications.

Validation with Experimental Data

Measured flexural strengths of 38 12 in.-deep prestressed concrete I-beams tested in flexure by Hernandez (1958) were compared to the strengths predicted by the LRFD and Standard Specifications, the proposed modified LRFD procedure, and the strain compatibility analysis. Reported material properties were used for each beam and the tensile strength of the concrete was neglected. In the experimental study, two different reinforcement ratios were used with two different cross sections, as shown in Figure 5. Even though the nominal dimensions of all of the beams were the same, the web and flange dimensions varied slightly. In the analyses, the reported actual dimensions were used.

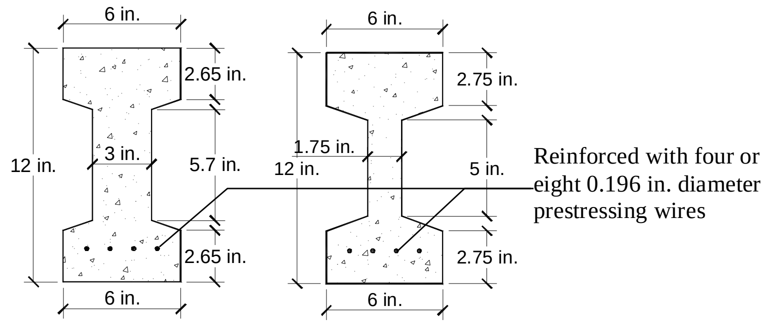


Figure 5. Nominal dimensions of beam sections tested by Hernandez (1958)

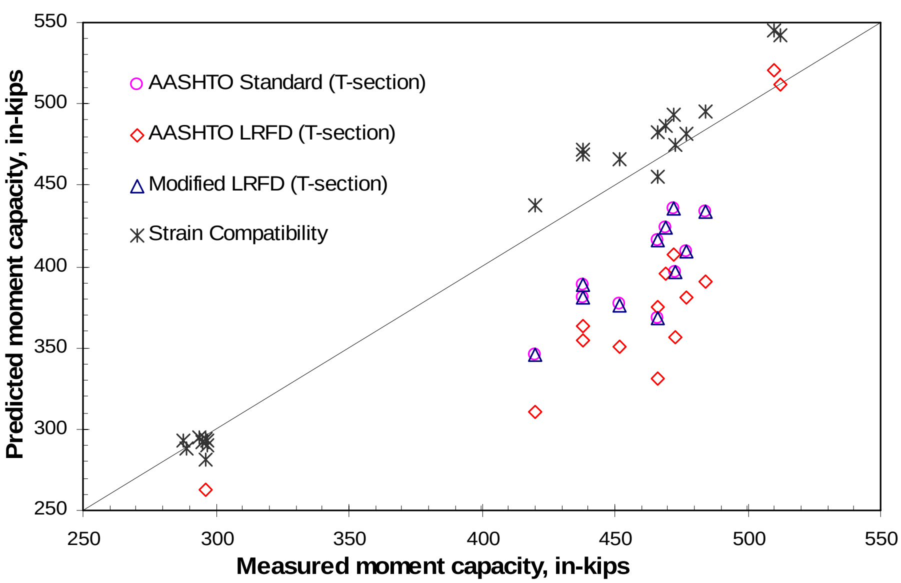


Figure 6. Comparison of predicted and measured moment capacities

Figure 6 provides a comparison of the predicted and measured moment capacities. For the sake of clarity, only the sections with T-section behavior at ultimate capacity according to the specifications are included in the figure. As shown in Figure 6, moment capacities predicted by the strain compatibility analysis were in agreement with the test results.

The reason that the Standard Specifications and the modified LRFD procedure predicted the same moment capacities for the beams is because all of the beams shown in the figure were considered over-reinforced according to the LRFD, Standard Specifications, and the modified LRFD procedure and because the Standard Specifications and the modified LRFD procedure use identical equations (Equations 5-a and 8, respectively) for the moment capacity of over-reinforced sections.

As evident in Figure 6, using a maximum tensile reinforcement limit and computing the moment capacity of over-reinforced sections with a different formula, which is based on the amount of tensile reinforcement corresponding to the maximum reinforcement limit, implicitly places a safety factor on the computed moment capacity of sections. Figure 6 also shows that the AASHTO LRFD Specifications grossly underestimated the nominal moment capacities, therefore resulting in a larger safety factor.

CONCLUSIONS

The following conclusions were drawn from this study:

1. Different interpretations of the equivalent rectangular stress block idealization used by the AASHTO LRFD and AASHTO Standard Specifications resulted in inconsistencies in the sectional response of nonrectangular prestressed concrete sections predicted according to these specifications.
2. Limiting the contribution of top flange overhangs to the internal compressive force caused an overestimation of the neutral axis depth of T-sections according to the LRFD Specifications, which in turn leads to the section being prematurely considered as over-reinforced. The tendency to classify some sections prematurely as over-reinforced results in large differences between the moment capacities predicted by the AASHTO LRFD Specifications and the other methods.
3. Limiting the maximum amount of tensile reinforcement to be used in determining the moment capacity, as used in the AASHTO LRFD and Standard Specifications, provides an additional safety margin to account for the poor flexural ductility of sections with large amounts of tensile reinforcement.
4. Modifying the procedure of the AASHTO LRFD Specifications by changing the T-section limit from $c = h_f$ to $a = h_f$ reduces the inconsistencies between the sectional response and the maximum reinforcement limits predicted by the AASHTO LRFD Specifications and the other methods (AASHTO Standard Specifications and the strain compatibility analyses). With this modification, the $\beta_1 h_f$ maximum limit for the depth of the top flange overhang contribution to the internal compressive force in the LRFD Specifications is automatically removed.
5. The AASHTO Standard Specifications does not take into account the effect of changes in the neutral axis location caused by changes in top flange depth when calculating the strand stress at ultimate moment capacity. In this respect, the LRFD procedure for strand stress provides a more realistic sectional response. Thus, it is proposed that the LRFD strand-stress relation be used with the modified procedure.

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NOTATION

a	=	depth of equivalent rectangular stress block, in.
A_{ps}	=	area of prestressing steel, sq.in.
A_s	=	area of nonprestressed tension reinforcement, sq.in.
A_{sr}	=	prestressing steel area required to develop the ultimate compressive strength of the web of the section, sq.in.
b	=	width of the compression face of the member, in.
b_w	=	web width, in.
c	=	distance from the extreme compression fiber to the neutral axis, in.
d	=	distance from extreme compression fiber to centroid of the prestressing force, in.
d_e	=	effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement, in.
d_p	=	distance from extreme compression fiber to the centroid of the prestressing tendons, in.
f'_c	=	specified compressive strength of concrete, ksi
f_{ps}	=	average stress in prestressing steel at ultimate load, ksi
f_{pu}	=	specified tensile strength of prestressing steel, ksi
h_f	=	compression flange depth, in.
k	=	factor for type of prestressing tendon
	=	0.28 for low-relaxation steel
	=	0.38 for stress-relieved steel
	=	0.48 for bars
M_n	=	nominal flexural resistance, in.-kips
β_1	=	ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone
γ, γ_p	=	factor for type of prestressing tendon
	=	0.28 for low-relaxation steel
	=	0.40 for stress-relieved steel
	=	0.55 for bars
ρ	=	prestressed reinforcement ratio
	=	A_{ps}/bd_p
ω_w	=	reinforcement index considering web of flanged sections
	=	$\dfrac{A_{sr} f_{ps}}{b_w d f'_c}$