Proposed Equal Leg Single Angle Flexural Design Provisions for Consideration in the Development of Future AISC Specification Editions

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Currently, the design of single angle members is governed by the *LRFD Specification for Design of Single Angle Members* (AISC, 2000). The provisions in this document are meant to augment the more general design provisions contained in the *LRFD Specification for Structural Steel Buildings* (AISC, 1999) for use with single angle members. The design provisions promulgated by the *LRFD Specification for Design of Single Angle Members* are the result of the best information available at the time of the preparation of the document in the early 1990s. Over the past decade, new information about single angle flexural response has accumulated from experimental and analytical research efforts aimed at more accurately quantifying the behavior of single angle beams. This new information is summarized herein and recommendations are made for rational single angle design provisions for consideration and possible inclusion in Chapter F of future AISC specification editions. An appendix of proposed Specification language is provided herein.

CURRENT PRACTICE

The current *Load and Resistance Factor Design Specification for Single-Angle Members* (AISC, 2000), hereafter referred to as the Specification, focuses on the five most common flexural orientations of the angle cross section as encountered in practice. These orientations are displayed in Figure 1. The five orientations consist of both senses of minor principal and geometric axis flexure as well as major principal axis flexure. The current Specification (AISC, 2000) addresses both equal and unequal leg angles. The Specification views the case of equal leg single angle cross sections as being a special case of the more general unequal leg scenario. However, since flexural applications (not involving continuous lateral bracing) most often involve equal leg angles, the present discussion focuses on equal leg angles only.

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The three general flexural limit states considered in the Specification are: 1) local buckling when the tip of an angle leg is in compression; 2) yielding when the tip of an angle leg is in tension; and 3) lateral-torsional buckling of the angle beam.

Local Buckling

In consideration of the local buckling limit state, within the context of non-slender elements, the Specification conservatively adopts a limiting plate slenderness criterion consistent with the case of plate buckling wherein the stresses acting on the loaded plate edges vary linearly from zero to some maximum value across a distance equal to $^2/_3$ of the plate width (i.e. such that the ratio of bending stress to uniform compression is 2). While such an approach is consistent with the stress distribution that might be encountered in elastic single angle geometric axis flexure, the same approach is quite conservative when applied to the case of minor principal axis flexure where ratio of bending stress to uniform compression is closer to 5.

The slender element capacity reduction strategy adopted for angle beams in the previous edition of the Specification (AISC, 1993) was simply that which was specified for use with angle columns composed of slender plate elements subjected to a uniform stress condition. Implied in this approach was that it was conservative to apply provisions formulated for use with the uniform stress field associated with column response to the less critical stress distribution developed during flexural response. This conservatism has been addressed in the most recent version of the Specification (AISC, 2000) wherein a multiplicative factor of 1.34 has been adopted to treat the stress gradient case. The provisions recommended herein seek to further liberalize this approach by increasing this factor to be 2.6.

Yielding

The Specification currently limits cross-sectional capacity to $1.5M_y$, an approach that implies a shape factor that is most appropriate for principal axis flexure; this value is somewhat conservative when applied to the case of geometric axis flexure where shape factors as high as 1.8 are possible. In the previous version of the Specification (AISC, 1993) more stringent cross-sectional capacity limits

(1.25 times the yield moment, M_y) were imposed out of concern that single angles were incapable of developing their full-plastic capacity and that large cross-sectional distortions might accompany the high stress levels associated with M_p. These concerns have been alleviated as a result of the analytical and experimental studies carried out in recent years (Earls and Galambos, 1997a; Madugula, Kojima, Kajita, and Ohama, 1995 and 1996).

In addressing the inelastic flexural behavior of single angles it is often useful to employ plastic analysis and design methodologies. These techniques are philosophically consistent with modern limit states design specifications and are quite easy to apply. However, in order to apply these techniques it is necessary that sufficient plastic hinge rotation capacity be available at the member level so as to allow for the development of a collapse mechanism at the structural system level. The *LRFD Specification for Structural Steel Buildings* (AISC, 1999) requires that a member section exhibit a minimum plastic hinge rotation capacity of three if that member is to be considered compact and hence suitable for proportioning with plastic design techniques. Cross-sectional compactness parameters are prescribed in Table B5.1 of the *LRFD Specification for Structural Steel Buildings* (AISC, 1999) on a case-by-case basis. Absent from this table, however, is any reference to single angle flexure. This omission is tantamount to prohibiting the use of single angle beams in designs based on plastic design methods.

Lateral-Torsional Buckling

At the heart of the Specification's lateral-torsional buckling limit state provisions are the theoretical elastic buckling solutions developed by Thomas and Leigh (Thomas and Leigh, 1970 and 1973). These elastic solutions are modified by the Specification for use in the inelastic range through the application of a linear inelastic transition zone (as outlined in Section 5.1.3 of the Specification.) A moment value of $0.75M_y$ is used in the Specification to delineate between the inelastic and elastic response regimes in lateral-torsional buckling behavior. Modifications to the elastic moment capacities are prescribed by the Specification such that a reduction of between 8 and 25 percent, of the theoretical solutions provided by Thomas and Leigh (Thomas and Leigh, 1970 and 1973), are used for design. It is further noted that for the case of unbraced geometric axis flexure, the calculation of the yield moment (as required by the provisions in Section 5.1.3 of the Specification) must be carried out with a reduction factor of 0.8 applied to the cross-sectional elastic section modulus to account for anticipated out-of-plane deflections and twisting of the angle cross section due to the nature of non-principal axis flexure and its resulting increase in compressive stresses in the toe of the angle beam.

RECENT DEVELOPMENTS

Research efforts carried out during the last five years have yielded much new information related to single angle flex-

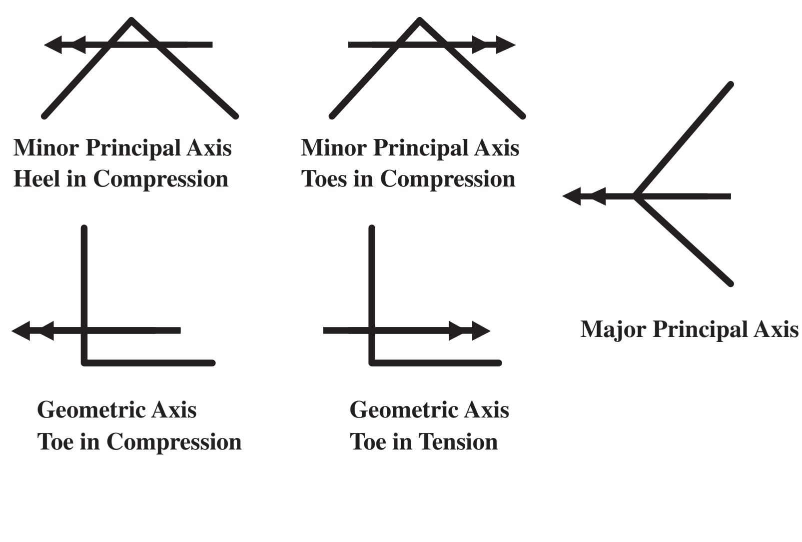

Fig. 1. Common single angle flexural orientations.

ural behavior. Cross-sectional compactness requirements, bracing requirements, and nominal moment capacity predictions have been explored in this work (Madugula et al., 1995 and 1996; Earls and Galambos, 1997a, 1997b, and 1998; Earls, 1995, 1999a, 1999b, 2001a, 2001b, 2001c, 2001d, and 2001e). From this recent work it has become clear that the approach of the Specification to single angle flexural design is, in most cases, unnecessarily conservative. However, it is also pointed out that this research has identified instances where the current Specification is unconservative in its predicted capacities for single angle beams (Earls and Galambos, 1997a and 1998).

These inconsistencies in conservatism have motivated further study as well as an effort to codify new design specification provisions that incorporate many of the recent research results. An effort has also been made to adhere to a Specification format that is consistent with the philosophical approach promulgated in the current version of the *LRFD Specification for Structural Steel Buildings*. This effort is important since it is anticipated that new single angle specification language related to flexure will be incorporated into Chapter F of new editions of the AISC Specification.

ORGANIZATION

The present paper is organized to be similar to a commentary on the design provisions contained in Appendix A herein. The individual sections of the paper will follow the order of the Appendix A provisions starting with a treatment of principal axis flexure, of both types and both senses, and ending with geometric axis flexure of both senses. Under the heading of principal axis flexure, the discussions will include issues related to compactness and nominal moment capacity while for geometric axis flexure the discussions will also include bracing requirements.

PRINCIPAL AXIS FLEXURE

The following section deals with compactness and cross-sectional capacity provisions contained in Appendix A. In addition, lateral-torsional buckling provisions for major principal axis flexure are also discussed. Principal axis flexure is used here to refer to flexure about either the major or minor principal centroidal axes of a single angle. In the case of minor principal axis flexure (referred to as the z-z axis by AISC), lateral-torsional buckling is not observed and hence this flexural orientation is omitted from any of the present discussions on lateral-torsional buckling. The effects of unbraced length on cross-sectional capacity are observed to be negligible for minor principal axis flexure (Earls and Galambos, 1998); the same is unfortunately not true for the case of geometric axis flexure as will be seen in later sections.

Compactness

Early research (Earls, 1995) on single angle compactness indicates that steel yield strength has an important effect on compactness requirements for a given single angle flexural orientation. For the case of minor principal axis flexure, this dependence on yield strength is easily handled. However, in the case of major principal axis flexure the situation becomes more complicated.

Minor Axis Bending with Toes in Compression

A parametric study performed using finite element models yields a large number of moment-rotation plots for minor principal axis flexure of this sense. Using these plots, and the definition of rotation capacity, *R*, adopted by ASCE (1971), trends in compactness can be observed when the data are plotted in a fashion consistent with Figure 2. The relationship between the compactness parameter, λ_p, and the material yield stress is of a linear nature as can be seen in Figure 2. This linearity can be expressed mathematically using an equation based on a regression analysis

$$\lambda_p = 0.756\sqrt{\frac{E}{F_y}} - 1.67 \tag{1}$$

Equation 1 results are plotted with the finite element data in Figure 2 where the close agreement between the two data sets can be observed. Equation 1 can be closely approximated by a simpler expression of the form:

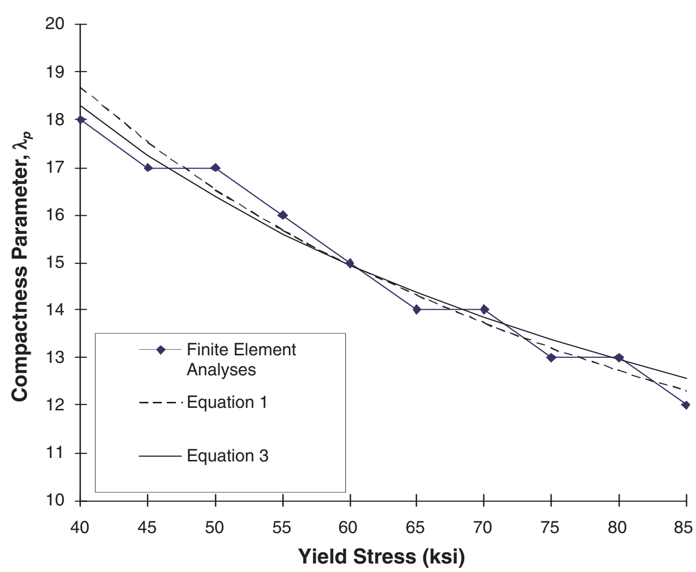

Fig. 2. Trend in compactness for minor principal axis flexure, with toes in compression, as steel yield strength varies.

$$\lambda_p = 0.695\sqrt{\frac{E}{F_y}} \tag{2}$$

which may be conservatively recast in the form of Equation F6-1 in Appendix A reproduced below as Equation 3:

$$\lambda_p = 0.68\sqrt{\frac{E}{F_y}} \tag{3}$$

Minor Axis Bending with Heel in Compression

Earlier single angle compactness work (Earls, 1995) indicates that all currently hot-rolled angle sections (this includes sections with *b/t* as high as 20) are compact when made from steel with yield strengths as high as 50 ksi (345 MPa). Later research (Earls and Galambos, 1998) indicates that this idea can be extended to steel grades with yield strengths as high as 120 ksi (820 MPa) (i.e. ASTM A514 steel). Hence λ_p is set to be a constant value of 20 in Equation F6-2 in Appendix A as a result of this earlier work.

Major Axis Bending

In the case of major axis bending the coupling between local and global buckling effects is very pronounced (Earls, 1999a) and hence compactness is best quantified by an equation given in terms of plate and beam slenderness as well as material response (as indicated by Equation 4 below).

$$\frac{b}{t} = -0.075\left(\frac{L}{r_z}\right) - 2900\left(\frac{F_y}{E}\right) + 20 \tag{4}$$

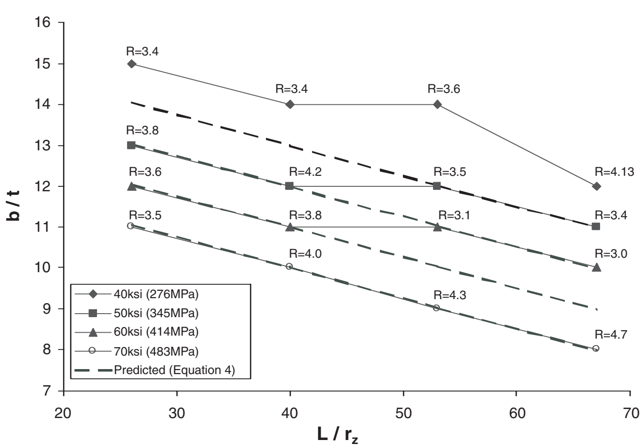

Fig. 3. Trends in major axis rotation capacity as affected by angle geometry and yield stress.

This equation emanates from a consideration of the relationship between the three parameters as displayed in Figure 3.

In keeping with the traditional AISC approach of separately treating local and global buckling effects, Equation 4 is simplified through the assumption that a conservative upper-bound limit on beam slenderness (L/r_z) for a member proportioned using plastic design techniques is 53; thus resulting in Equation 5:

$$\lambda_p = 16 - 2900\left(\frac{F_y}{E}\right) \tag{5}$$

This can be simplified even further if it is assumed that only commonly used constructional grades of steel are employed (i.e. those with yield strengths that are 70 ksi (483 MPa) and below); this results in Appendix A Equation F6-3:

$$\lambda_p = 0.44\sqrt{\frac{E}{F_y}} \tag{6}$$

Cross-Sectional Capacity

The specification language presented in Appendix A indicates that the maximum cross-sectional capacity is $1.5M_y$ and not M_p. While it may appear obvious that the theoretical shape factor for principal axis flexure ought to be 1.5, this is not necessarily the case practically speaking. If one were to perform an advanced analysis of a single angle section where actual physical dimensions are considered (i.e. fillets, etc.) as well as the effects of strain hardening, a shape factor in excess of 1.5 could be argued to be valid. However, it is conservative to use a shape factor of 1.5, as well as convenient. The convenience shows up in the formulation of an inelastic transition response between the fully yielded behavior and the elastic buckling behavior. If a definite capacity in the fully yielded state is not imposed then it becomes extremely cumbersome to try and define an inelastic transition response equation that works throughout the design space of all possible angle sections.

Minor Axis Bending with Toes in Compression

The proposed specification provisions in Appendix A are consistent with the current Specification limits on cross-sectional response in that at 0.75 M_y and a plate slenderness of approximately $2\lambda_p$, a transition from inelastic to elastic response is assumed to occur while the transition from inelastic to plastic response is assumed to occur at $1.5M_y$ with a plate slenderness of λ_p. Equation F6-5 in Appendix A is simply the equation of a line connecting the ordered pairs: (λ_p, $1.5M_y$); ($2\lambda_p$, 0.75 M_y).

When the plate slenderness value of an individual angle component exceeds the slenderness limit of $2\lambda_p$, the slender element capacity is given by Equation 7 (Appendix A Equation F6-6)

$$M_n = 2.6 Q F_y S_c \tag{7}$$

The factor 2.6, acting on the reduction factor for slender compression elements, Q, is a correction that amplifies the compression-only Q factor to account for the more favorable stress distribution factor occurring in angle legs subjected to minor principal axis flexure. The value of 2.6 is obtained using the current Specification Equation 4-3c with substitutions made so as to be consistent with the transition from inelastic to elastic response ($2\lambda_p$, 0.75 M_y); Equation 8 details the calculation.

$$0.75 = F \frac{0.534 \, {}^{E}\!/_{F_y}}{\left(1.36\sqrt{\frac{E}{F_y}}\right)^2} \tag{8}$$

In Equation 8, the term "*F*" is the amplification factor previously discussed (in this case equaling 2.6). It is also noted that the denominator of Equation 8 is the equation for $2\lambda_p{}^2$.

Minor Axis Bending with Heel in Compression

In consideration of this sense of minor principal axis flexure, recent research indicates that all currently rolled equal leg single angles (*b/t* can be as high as 20) made from steel with yield strengths as high as 120 ksi (820 MPa) can easily achieve their full plastic capacity and rotation capacities in excess of 3. As previously mentioned, a shape factor of 1.5 is assumed for minor principal axis flexure and hence is retained here for consistency. At the opposite end of the plate slenderness spectrum, interesting comparisons can be made between the current Specification and recent research within the context of major principal axis flexure.

Major Axis Bending

The current Specification provisions related to slender element response in major principal axis flexure appear to be extremely conservative. In recent research (Earls, 1999a) it is observed that, "…the LRFD (Specification) classifies as *slender* those cross-sections that were able to not only exceed the full plastic capacity of the cross-section, but also exhibit rotation capacities well in excess of three." Based on this research it is concluded that the predicted slender element major axis moment capacities from the Specification are consistently lower than moment capacities obtained using experimentally verified nonlinear finite element models. A scaling factor of 1.5 applied to the current Specification slender element major axis moment capacities conservatively predicts the finite element results. Based on this fact, a factor of 1.5 is applied to the existing Specification Equation 5-1c resulting in the following:

$$M_n = 1.5\left(1.34 Q F_y S_c\right) \tag{9}$$

Simplification of Equation 9 results in the form of the equation presented in Appendix A as Equation F6-10 reproduced below as Equation 10.

$$M_n = 2.0\left(Q F_y S_c\right) \tag{10}$$

Considering Equation 10, a similar approach to that used in Equation 8, may now be employed to back-calculate a plate slenderness limit for the transition between elastic and inelastic flexural response (once again adopting the current Specification assumption that $0.75M_y$ represents the moment capacity at this transition point in response).

$$\frac{0.75}{2} = \frac{0.534 \, {}^{E}\!/_{F_y}}{\left(\frac{b}{t}\right)^2} \tag{11}$$

Solving for (*b/t*) results in Equation 12; which is used directly in Appendix A.

$$\frac{b}{t} = 1.193\sqrt{\frac{E}{F_y}} \tag{12}$$

For the inelastic transition response in major principal axis flexure, Equation F6-9 in Appendix A is developed by connecting the ordered pairs: (λ_p, $1.5M_y$) ; (Equation 12, 0.75 M_y). The result can be seen in Equation 13 below

$$M_n = \left[1.938 - \left(\frac{{}^{b}\!/_{t}}{\sqrt{\frac{E}{F_y}}}\right)\right] M_y \tag{13}$$

Lateral-Torsional Buckling Capacity (major axis bending only)

The lateral-torsional buckling provisions presented in Appendix A Equations F6-11 through F6-13 are verbatim from the current Specification.

GEOMETRIC AXIS FLEXURE

In the proposed single angle provisions in Appendix A, as related to geometric axis flexure, it is not be possible to adhere to the current AISC philosophical approach wherein plate slenderness and beam slenderness are treated as separate and uncoupled from one another. The subsequent discussion outlines the steps taken when arriving at the proposed specification provisions presented in Appendix A related to geometric axis flexure.

Compactness

Recent research results (Earls, 2001a and 2001b) indicate that it is not possible to adopt an approach wherein plate slenderness and beam slenderness are treated as separate and un-coupled when addressing geometric axis compactness. As a result, the form of the compactness provisions presented in Appendix A Equations F6-14, F6-17, and F6-18 are cast in terms of beam and plate slenderness measures. Satisfaction of the proposed compactness criteria requires a consideration of bracing as well as plate slenderness due to the highly coupled local and global buckling modes that dominate at the time of failure in this particular single angle flexural orientation. It is noted that for both cases of geometric axis flexure, the full plastic capacity is permitted. This is different from the current Specification approach of conservatively assigning a shape factor of 1.5 to this flexural case. There is much research evidence pointing to the fact that a shape factor limit of 1.5 on single angle geometric axis flexure is grossly over-conservative.

Toes in the Plane of Bending in Compression

The compactness expression presented in Appendix A Equation F6-14 is identical to the Equation 14 below (Earls, 2001a and 2001e)

$$\frac{L_b}{r_z} = \frac{310.5}{F_y}\left[-0.1258\left(\frac{b}{t}\right)^3 + 6.46\left(\frac{b}{t}\right)^2 - 111.72\left(\frac{b}{t}\right) + 658.89\right] \tag{14}$$

except that it has been non-dimensionalized for material response and hence takes on the new form

$$\frac{L_b}{r_z} \le \frac{E}{644F_y}\left[-0.1258\left(\frac{b}{t}\right)^3 + 6.46\left(\frac{b}{t}\right)^2 - 111.72\left(\frac{b}{t}\right) + 658.89\right] \tag{15}$$

Heel in the Plane of Bending in Compression

The compactness expression presented in Appendix A Equations F6-17 and F6-18 are identical to those reported in the literature (Earls, 2001b and e). Appendix A Equation F6-17 applies to a plate slenderness range of: $10 \le b/t \le 20$; and Equation F6-18 applies to a plate slenderness range of $6 \le b/t < 10$. It is also noted that Appendix A Equation F6-18 is a conservative form of the compactness expression reported in the literature (Earls, 2001b and e) since the form of the equation is consistent with that given for use with steel having a minimum specified yield strength of 70 ksi (483 MPa).

Nominal Moment Capacity

As mentioned previously, in the case of single angle geometric axis flexure the failure mode at ultimate moment frequently involves a high degree of interaction between local and global buckling effects. The resulting failure does not bear any resemblance to classical lateral-torsional buckling. As a result, the remaining expressions for single angle geometric axis moment capacity, listed in Appendix A, are not presented under a heading of lateral-torsional buckling. The two geometric axis nominal moment capacity equations (Appendix A Equations F6-16 and F6-20) are reproduced in the form in which they appear in the literature (Earls, 2001c and d).

CONCLUSIONS

The existing single angle specification provisions in the *LRFD Specification for Design of Single Angle Members* (AISC, 2000) require updating in order to reflect the current state of knowledge as it relates to flexural response. The current paper proposes new specification provisions that would supersede those currently promulgated by the *LRFD Specification for Design of Single Angle Members* (AISC, 2000). The new specifications are presented for consideration and possible use in Chapter F of new editions of the AISC Specification. The paper has described the rationale behind these new provisions and in a sense serves as a commentary on them.

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APPENDIX A

SINGLE ANGLE FLEXURAL DESIGN STRENGTH

These provisions are applicable to equal leg single angles with a yield strength less than or equal to 483Mpa and $L_b / r_z \leq 200$.

PRINCIPAL AXIS FLEXURE:

Compactness:

Minor axis bending with toes in compression

$$\lambda_p = 0.68\sqrt{\frac{E}{F_y}} \tag{F6-1}$$

Minor axis bending with heel in compression

$$\lambda_p = 20 \tag{F6-2}$$

Major axis bending

$$\lambda_p = 0.44\sqrt{\frac{E}{F_y}} \tag{F6-3}$$

Cross-Sectional Capacity:

Minor axis bending with toes in compression

when $\frac{b}{t} \leq \lambda_p$,

$$M_n = 1.5M_y \tag{F6-4}$$

when $\lambda_p < \frac{b}{t} \leq 1.36\sqrt{\frac{E}{F_y}}$,

$$M_n = \left[2.25 - 0.75\left(\frac{b/t}{\lambda_p}\right)\right]M_y \tag{F6-5}$$

when $\frac{b}{t} > 1.36\sqrt{\frac{E}{F_y}}$,

$$M_n = 2.6QF_yS_c \tag{F6-6}$$

Minor axis bending with heel in compression

$$M_n = 1.5M_y \tag{F6-7}$$

Major axis bending

when $\frac{b}{t} \leq \lambda_p$,

$$M_n = 1.5M_y \tag{F6-8}$$

when $\lambda_p < \frac{b}{t} \leq 1.193\sqrt{\frac{E}{F_y}}$,

$$M_n = \left[1.938 - \left(\frac{b/t}{\sqrt{\frac{E}{F_y}}}\right)\right]M_y \tag{F6-9}$$

when $\frac{b}{t} > 1.193\sqrt{\frac{E}{F_y}}$,

$$M_n = 2QF_yS_c \tag{F6-10}$$

Lateral-Torsional-Buckling Capacity (Major axis bending only)

when $M_{cr} \leq M_y$

$$M_n = \left[0.92 - 0.17\left(M_{cr}/M_y\right)\right]M_{cr} \tag{F6-11}$$

when $M_{cr} > M_y$

$$M_n = \left[1.92 - 1.17\sqrt{M_y/M_{cr}}\right]M_y \leq 1.5\,M_y \tag{F6-12}$$

where,

$$M_{cr} = C_b\frac{0.46Eb^2t^2}{L_b} \tag{F6-13}$$

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GEOMETRIC AXIS FLEXURE:

Geometric axis flexure wherein the toe in the plane of bending is in compression

Satisfaction of the expression

$$\frac{L_b}{r_z} \le \frac{E}{644F_y}\left[-0.1258\left(\frac{b}{t}\right)^3 + 6.46\left(\frac{b}{t}\right)^2 - 111.72\left(\frac{b}{t}\right) + 658.89\right];$$

$$10 \le b/t \le 20 \qquad \text{(F6-14)}$$

Implies compactness and

$$M_n = M_p \qquad \text{(F6-15)}$$

Otherwise,

$$M_n\left[-\frac{L_b/r_z + 20\left(b/t\right)}{300} + 2.5\right]M_y \le M_p \qquad \text{(F6-16)}$$

Geometric axis flexure wherein the toe in the plane of bending is in tension

Satisfaction of

$$\frac{L_b}{r_z} \le -1.9\left(\frac{b}{t}\right) + 39;\ 10 \le b/t \le 20 \qquad \text{(F6-17)}$$

$$\frac{L_b}{r_z} \le -40\left(\frac{b}{t}\right) + 420;\ 6 \le b/t < 10 \qquad \text{(F6-18)}$$

Implies compactness and

$$M_n = M_p \qquad \text{(F6-19)}$$

Otherwise,

$$M_n = \left[-\frac{1}{133}\left(\frac{L_b}{r_z}\right) + 2\right]M_y \le M_p$$

subject to:

if $b/t \le 10$ then $F_y \le 483\,Mpa$

if $10 < b/t \le 16$ then $F_y \le 345\,Mpa$

if $16 < b/t \le 20$ then $F_y \le 276\,Mpa$