Proceedings of the
Annual Stability Conference
Structural Stability Research Council
San Antonio, Texas, March 21-24, 2017

The History of the Development of the Single Angle Design Provisions in the AISC Specification

LeRoy A. Lutz[1]

Abstract
There was very limited guidance on the design of single-angle members in 1978 in the AISC specification (AISC, 1978). Pressure for more guidance from industry and the government led to AISC forming a task committee to develop design provisions for single angles. This led to development of a separate specification dedicated to the design of single-angle members. The initial specification was an Allowable Stress Design version. This was followed by LRFD versions. Ultimately it was decided to assimilate single angle provisions into the main specification along with the provisions for hollow structural shapes.

Single Angle Design in 1978
In 1978 using the AISC specification (AISC, 1978) there was limited guidance on the design of single-angle members. Provisions followed the allowable stress approach. Single-angle members with its legs in axial compression or in compression due to bending were considered to be fully effective and have an allowable stress limit of $0.6F_y$ provided that the b/t ratio is not greater than $76/\sqrt{F_y}$ with F_y in ksi units or $0.45\sqrt{E/F_y}$. If b/t is greater, then one is sent to Appendix C of the specification to obtain an allowable stress $0.6F_yQ_s$ where for

$$76/\sqrt{F_y} < b/t < 155/\sqrt{F_y} \quad \text{or} \quad 0.45\sqrt{E/F_y} < b/t < 0.91\sqrt{E/F_y}$$

$$Q_s = 1.340 - 0.00447 b/t\sqrt{F_y} \quad \text{or} \quad Q_s = 1.34 - 0.76 b/t\sqrt{F_y/E} \tag{1}$$

$$\text{and for } b/t \geq 155/\sqrt{F_y} \quad \text{or} \quad \text{for } b/t \geq 0.91\sqrt{E/F_y}$$

$$Q_s = 15{,}500/\left[F_y(b/t)^2\right] \quad \text{or} \quad Q_s = 0.53E/\left[F_y(b/t)^2\right] \tag{2}$$

in U. S. Customary Units — independent of units used

where F_y is the specified minimum yield stress, b is the full width of the longest angle leg, t is the thickness of angle leg, and Q_s is a reduction factor. These Q_s expressions for axial compression of single angles remained unchanged until the 2016 AISC specification (AISC,

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[1] Vice President Emeritus, Computerized Structural Design, S. C. (CSD Structural Engineers), Milwaukee, WI 53217.

2016). The variable $\sqrt{F_y}$ was replaced in later specifications by $\sqrt{F_y / E}$ as shown above where E is the Modulus of Elasticity. An E =29,000 ksi was used in the expressions listed in U. S. Customary Units.

For tension and compression stresses in bending the specification permitted an allowable bending stress of $F_b = 0.66F_y$ for wide flange members bent about their strong axis if unstiffened compression elements provided that b/t is less or equal to $65/\sqrt{F_y}$, i.e., $0.38\sqrt{E/F_y}$. This limit applied only if the member was symmetrical about the minor axis and had limits on unbraced length of the compression flange. Certain rectangular tubular sections were also permitted to reach an allowable flexural stress $F_b = 0.66F_y$. Doubly symmetric wide flange shapes bent about their minor axis were permitted to reach an allowable stress of $0.75F_y$. However, no allowable stress on single angles in excess of $0.6F_y$ was explicitly permitted.

Impetus for Development of Single-Angle Design Provisions

The US Nuclear Regulatory Commission (NRC) initiated design criteria for single angles in 1985 in response to questions about the safety of angle hangers for cable trays in a California nuclear power plant. Of particular concern was the effect of unbraced length on the allowable bending stress as well as the treatment of combined stresses. In December of 1985 an ASME Task Force met to discuss establishing design rules for single angles. At that meeting it was mentioned that the AISC was considering adopting the Australian rules for single angles in flexure. In January of 1986 T. G. Longlais from Sargent & Lundy Engineers wrote a letter to Geehard Haaijer, Vice President of AISC, enclosing the minutes of the December 1985 meeting and indicating that ASME would defer development of a single-angle design criteria pending AISC initiative. This led to the formation of the AISC Ad hoc Committee on Design Criteria for Single Angle Members. In mid 1985, before the AISC ad hoc committee on single angles was formed, the SSRC formed Task Group 26 entitled *Stability of Angle Struts* to try to consolidate and advance information on angle analysis and design.

Design Issues Prior to AISC ad hoc Committee Involvement

Allowable stress upper limits were discussed. Should factor of safety for flexure be 1.67 while the factor of safety of 2.0 is applicable for columns? The shape factors for equal-leg angles bent about their principal axes are 1.5, however, should the maximum allowable stress limit be 0.6, 0.66 or $0.75F_y$?

When should lateral instability begin reducing the allowable stress? An expression of the form $F_{cr} = F_y[A - (F_y / F_e)/B]$ was suggested where $F_e = \pi^2 E / (2\sqrt{2.6}L/t)$, the elastic flexural buckling stress of an equal-leg angle about its principal axis. Setting $F_{cr} = 1.5F_y$ at $L=0$ and $F_{cr} = 0.75F_y$ at $F_e = 0.75F_y$, one gets $A = 1.5$ and $B = 1.78$. Using $B=2$ it was shown that a factor of safety of 2 could be achieved all the way to $F_b = 0.66F_y$ as compared to expressions developed in Australia.

Should combined bending stresses be calculated using the principal axes or the geometrical axes?

These and other issues were addressed in a set of interim criteria developed by NRC and Sargent & Lundy with the technical assistance of Ted Galambos and AISC. These issues were reviewed and discussed at the first meeting of the AISC Ad Hoc meeting in November of 1986.

Initial work of the AISC Ad hoc Committee on Design Criteria for Single Angle Members
By 1988 the ad hoc Committee had developed a Specification for Allowable Stress Design of Single-Angle Members. Initially this document was to be Appendix F of the main specification, however, it was subsequently decided that it would be a stand-alone specification (AISC, 1989b). A view of part of the first page of AISC, 1989b is shown in Fig. 1.

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SPECIFICATION FOR ALLOWABLE STRESS

DESIGN OF SINGLE-ANGLE MEMBERS

PREFACE

The intention of the AISC Specification is to cover the common everyday design criteria in routine design office usage. It is not feasible to also cover the many special and unique problems encountered within the full range of structural design practice. This separate Specification and Commentary addresses one such topic—single-angle members—to provide needed design guidance for this more complex structural shape under various load and support conditions.

The single-angle Allowable Stress Design criteria were developed through a consensus process by a balanced ad-hoc Committee on Single Angle Members:

Donald R. Sherman, Chairman
Hansraj G. Ashar
Wai-Fah Chen
Raymond D. Ciatto
Mohammed Elgaaly
Theodore V. Galambos
Nestor R. Iwankiw
Thomas G. Longlais
Leroy A. Lutz
William A. Milek
Raymond H. R. Tide

The assistance of the Structural Stability Research Council Task Group on Single Angles in the preparation and review of this document is acknowledged.

Figure 1: View of Part of the First Page of AISC, 1989b

A summary of the content of the various sections of the specification follows:

Tension—Effective area of welded connections is given.

Shear—Allowable shear stress due to flexure and torsion is $0.4F_y$.

Compression—The provisions are the same as in the main specification including the provisions of Appendix C. The equivalent slenderness ratio Kl/r equal to $\pi\sqrt{E/F_e}$ is to be checked if flexural-torsional buckling controls where F_e is the elastic buckling stress in the flexural-torsional mode.

Flexure—The maximum allowable stress is $0.66F_y$ if the angle legs are in tension and for angle legs in compression when b/t is less or equal to $65/\sqrt{F_y}$, i.e., $0.38\sqrt{E/F_y}$. For larger b/t ratios the allowable stress follows the Appendix C values given in Eqs. 1 and 2 for axial compression.

Lateral buckling was based on expressions developed based on tests conducted in Australia (Leigh and Lay, 1984; Australian Institute of Steel Construction, 1975; Leigh and Lay, 1978). The expressions are a function of the elastic lateral-torsional buckling stress F_{ob}, and F_y.

When $F_{ob} > F_y$: $$F_b = [0.95 - 0.50\sqrt{F_y/F_{ob}}]F_y \le 0.66F_y \quad (3)$$

When $F_{ob} \le F_y$: $$F_b = [0.55 - 0.10F_{ob}/F_y]F_{ob} \quad (4)$$

F_{ob} expressions are given for both general major-axis bending, and geometric-axis bending of equal-leg angles. Bending about the geometric axis of equal-leg angles was addressed a separate case because it was felt to be a very common situation. The fact that the laterally unbraced equal-leg angle bent about the geometric axis has a stress that is 25% more than the same laterally braced angle was recognized in the specification. This is because the laterally unbraced angle deflects laterally as well as in the direction of the applied load.

Combined Stresses—Combined stress rules refer back to the equations in the main AISC specification (AISC, 1989a), but provide clarification on their use with single angles. One example is that "the maximum compression bending stresses due to each moment acting alone must be used even though they may occur at different cross sections of the member." Another example is when axial load occurs with geometric axis bending in the evaluation of F_e', the Euler stress divided by a factor of safety, which was employed in the determination of the moment magnification. In this case the geometric axis radius of gyration needs to be divided by 1.35 to obtain the slenderness for the appropriate F_e'.

Members of the Ad hoc Committee on Design Criteria for Single Angle Members and their affiliation are given in Table 1.

Table 1: Members of Ad hoc Committee on Single Angles & Task Committee 116

Member	Affiliation
Donald R. Sherman, Chairman	Univ. of Wisconsin-Milwaukee
Hansraj G. Ashar	US Nuclear Regulatory Commission
Wai-Fah Chen	Purdue University
Raymond D. Ciatto	Stone & Webster
Mohammed Elgaaly	University of Maine-Orono
Theodore V. Galambos	University of Minnesota
Nestor R. Iwankiw	AISC Director, Research & Codes &Secretary
Thomas G. Longlais	Sargent & Lundy
LeRoy A. Lutz	Computerized Structural Design
William A. Milek	Consultant
Raymond H. R. Tide	Wiss, Janney, Elstner & Associates

Load and Resistance Factor Design for Single Angles

In December of 1988 (before the ASD version was published) the Ad hoc Committee on Single Angles began to develop an LRFD version of a single angles specification.

A summary of the content of the various sections of the LRFD version of the specification (AISC, 1993b) follows:

Tension—The tensile strength expressions match those in the main specification. Specific conditions for angles were given relating to evaluation of effective area. Also the preferred maximum slenderness ratio of 300 was continued in this version.

Shear—The limit state of yielding for shear stress was $\phi 0.6F_y$ where $\phi = 0.9$.

Compression—The equations for compression were the same as in the main specification. The only difference is that local buckling expressions in the angle specification contained E, the modulus of elasticity, and Q was used instead of Q_s. However, the need to check flexural-torsional buckling was no longer required based on work by Galambos, 1991.

Flexure—The nominal moment capacity based on yielding when the angle tip is in compression was

$$M_n = 1.25F_yS_c \tag{5}$$

$$\text{when } b/t \le 0.38\sqrt{E/F_y}$$

The 1.25 represented the 25 % increase (0.75/0.6) permitted in allowable flexural stress in AISC, 1989a for members with a shape factor of 1.5. Even though the shape factor for angles at any orientation is greater than 1.5, the more conservative value of 1.25 was selected. S_c is the elastic section modulus to the tip in compression.

When $0.38\sqrt{E/F_y} < b/t < 0.45\sqrt{E/F_y}$, the nominal moment capacity transitions from 1.25 F_yS_c to F_yS_c. When $b/t > 0.45\sqrt{E/F_y}$ the M_n = QF_yS_c where Q is that employed for axial compression (Eqs. 1 and 2).

When the tip of the angle leg is in tension then

$$M_n = 1.25M_y \tag{6}$$

Lateral buckling expressions used for allowable stress design as shown by equations (3) and (4) were modified into nominal moment capacity expressions.

When $M_{ob} > M_y$: $$M_n = [1.58 - 0.83\sqrt{M_y/M_{ob}}]M_y \le 1.25M_y \tag{7}$$

When $M_{ob} \le M_y$: $$M_n = [0.92 - 0.17M_{ob}/M_y]M_{ob} \tag{8}$$

The elastic lateral-torsional buckling stress (F_{ob}) expressions given in the allowable stress specification (AISC, 1989b) were converted to M_{ob} moment expressions. Note that inelastic portion of M_n (Eq. 7) begins at $0.75M_y$ and that the bracketed portion of Eq. 8 provides a transition from 0.75 M_{ob} to 0.92 M_{ob}. The C_b expression used was the one used in AISC, 1993a, however, its maximum value was limited to 1.5.

Combined Forces—Flexure and axial compression was addressed using the same equations that were used in AISC, 1993a except that the principal axes were denoted as w and z rather than

x and y. Since single angle sections are not doubly symmetric, the M_{nw} and M_{nz} used were qualified by the sentence "Use section modulus for the specific location in the cross section and consider the type of stress." Even though one is calculating moment ratios (and load ratios), one should look at the type of stress at a particular location as one would do if one were evaluating elastic stresses at point.

In 1992 the Ad Hoc Committee on Design Criteria for Single Angle Members was designated as TC 116. The AISC, 1993b document was credited to TC 116 who are the same members as were credited with the AISC, 1989b document (See Table 1). The title page of AISC, 1993b is shown in Fig. 2.


Specification for
Load and Resistance
Factor Design of
Single-Angle Members

December 1, 1993

Figure 2: Partial View of Title Page of AISC, 1993

Second Edition of Load and Resistance Factor Design for Single Angles

In June of 1997 task committee TC 116 became TC 12 with members listed in Table 2. The charge of TC 12 was to develop the second edition of the LRFD Design Specification for single-angle members which was issued in 2000 (AISC, 2000) in conjunction with the main AISC specification (AISC, 1999). The title page of AISC, 2000 is shown in Fig. 3. No modifications were made to the tension, shear, and compression provisions. However, significant changes were made to the flexural provisions.

Flexure—The nominal moment capacity based on yielding when the angle tip is in compression was increased when $b/t \le 0.54\sqrt{E/F_y}$ to

$$M_n = 1.5 F_y S_c \tag{9}$$

When $0.54\sqrt{E/F_y} < b/t \le 0.91\sqrt{E/F_y}$:

$$M_n = F_y S_c[1.50 - 0.93(\frac{b/t}{0.54\sqrt{E/F_y}} - 1)] \quad (10)$$

When $b/t > 0.91\sqrt{E/F_y}$: $\qquad M_n = \frac{0.71E/F_y}{(b/t)^2}$ (11)

It was acknowledged that a shape factor of 1.5, representing a lower bound for all angle orientations, was justified. A 1.6 factor is the maximum shape factor to prevent yielding at service load. A shape factor for an equal-leg angle bent about a geometric axis is approximately 1.8. Use of the 1.5 factor was backed by analytical work by Earls and Galambos, 1997 which employed ABAQUS to model inelastic behavior. Also tests conducted by Madugula et al, 1995 and 1996 showed that the 1.5 shape factor was appropriate. The nominal moment of a single angle with the leg tip in tension was also raised to the Eq. 9 level.

As a result of using the 1.5 factor, the lateral buckling expression given in Eq. 7 needed modification as follows:

When $M_{ob} > M_y$: $\qquad M_n = [1.92 - 1.17\sqrt{M_y / M_{ob}}]M_y \le 1.5M_y$ (12)

M_{ob} reaches a value of $7.7M_y$ in Eq. 12 when $M_n = 1.5M_y$.

Table 2: Members of TC 12

James M. Fisher, Chairman
LeRoy A. Lutz, Vice Chairman
Mohamed Elgaaly
Shu-Jin Fang
Theodore V. Galambos
Subhash Goel
Charlotte S. Harman
Todd Helwig
Donald W. White
Sergio Zoruba, Secretary

Load and Resistance Factor Design Specification for Single-Angle Members

November 10, 2000

Supersedes the *Specification for Load and Resistance Factor Design of Single-Angle Members* dated December 1, 1993

Prepared by the
American Institute of Steel Construction, Inc.
Under the Direction of the
AISC Committee on Specifications and approved by
the AISC Board of Directors

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
One East Wacker Drive, Suite 3100
Chicago, IL 60601-2001

Figure 3: Title Page of AISC, 2000

Simplified Approach to Design of Single-Angle Struts

Determining the strength of single-angle members subjected to axial compression is difficult for the typical situation where the load is applied to one leg of the angle. The applied load is eccentric about a non-principal axis. Some continuity invariably exists which tends to improve the axial capacity, but its benefit is very hard to evaluate.

In 1986 there existed a simplified procedure used for the design of single-angle struts in the electrical transmission tower industry. The single-angle strut slenderness ratio was modified so that the angle could be designed as a concentrically loaded member. When the minimum slenderness ratio is small such that the member is in the inelastic region a greater equivalent *KL/r* is computed to reflect the effect of the applied end moment. When the slenderness ratio is in the elastic range a smaller equivalent *KL/r* is computed to reflect the effect of the end moment trying to buckle the member about an axis other than the z-axis. For the most common case with normal framing eccentricities where the connection is to the same leg on both ends of equal-leg angles [ASCE (2000) or ASCE (2015)]:

$$\text{When} \quad 0 \le \frac{L}{r_z} \le 120 \qquad \frac{KL}{r} = 60 + 0.5\frac{L}{r_z} \tag{13a}$$

$$\text{When} \quad 120 < \frac{L}{r_z} \le 250 \qquad \frac{KL}{r} = 46.2 + 0.615\frac{L}{r_z} \tag{13b}$$

Although this approach was employed for transmission structures for a long time, the impetus for incorporating the effective slenderness into the AISC specification occurred only after research was conducted at the University of Texas (Mengelkoch and Yura, 2002). TC 12 developed a effective slenderness approach for TC 4 (Members) which became part of AISC, 2005.
Since the buckling occurs principally about an axis (x) parallel to the connected leg, the AISC expressions used the radius of gyration about the x-axis r_x as a variable. So for space truss structures:

$$\text{When} \quad 0 < \frac{L}{r_x} \le 75 \qquad \frac{KL}{r} = 60 + 0.8\frac{L}{r_x} \tag{14a}$$

$$\text{When} \quad 75 < \frac{L}{r_x} \le 155 \qquad \frac{KL}{r} = 45 + \frac{L}{r_x} \tag{14b}$$

Eqs. 14a and 14b are essentially identical to Eqs. 13a and 13b, respectively, for equal-leg angles.

Similar expressions were derived for planar trusses. In planar trusses one cannot be assured of as much rotational restraint from the truss chord assembly. This led to the following expressions for planar trusses:

$$\text{When} \quad 0 < \frac{L}{r_x} \le 80 \qquad \frac{KL}{r} = 75 + 0.75\frac{L}{r_x} \tag{15a}$$

$$\text{When} \quad 80 < \frac{L}{r_x} \le 134.4 \qquad \frac{KL}{r} = 32 + 1.25\frac{L}{r_x} \tag{15b}$$

Eqs. 14b and 15b impose an L/r_x upper limit of 200. Since test results also existed for unequal-leg angles, these expressions can be used for unequal-leg angles although there are restrictions and modifications when the connection is to the shorter leg and the longer leg projects. The test results reported by Usami and Galambos, 1971 and by Mengelkoch and Yura, 2002 were compared with Eqs. 14 and 15 (Lutz, 2003 and 2006). Test end conditions were either pinned or fixed as illustrated in Fig. 4. For both end conditions the angle will deflect primarily in the direction of the projecting leg due to the eccentricity of the applied load and the significant rotational restraint provided about the other axis by the tee stem or gusset plate.

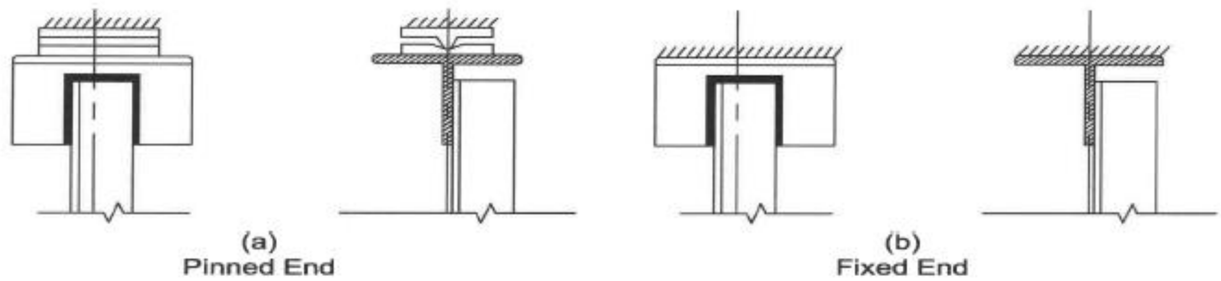

Figure 4: Test End Conditions for Single-Angle Struts

The fixed-end tests were compared with the space truss expression of Eq. 14 in Fig. 5. With less-than-fixed end conditions the test data would be lower, and the design load is also lower at ϕP_n.

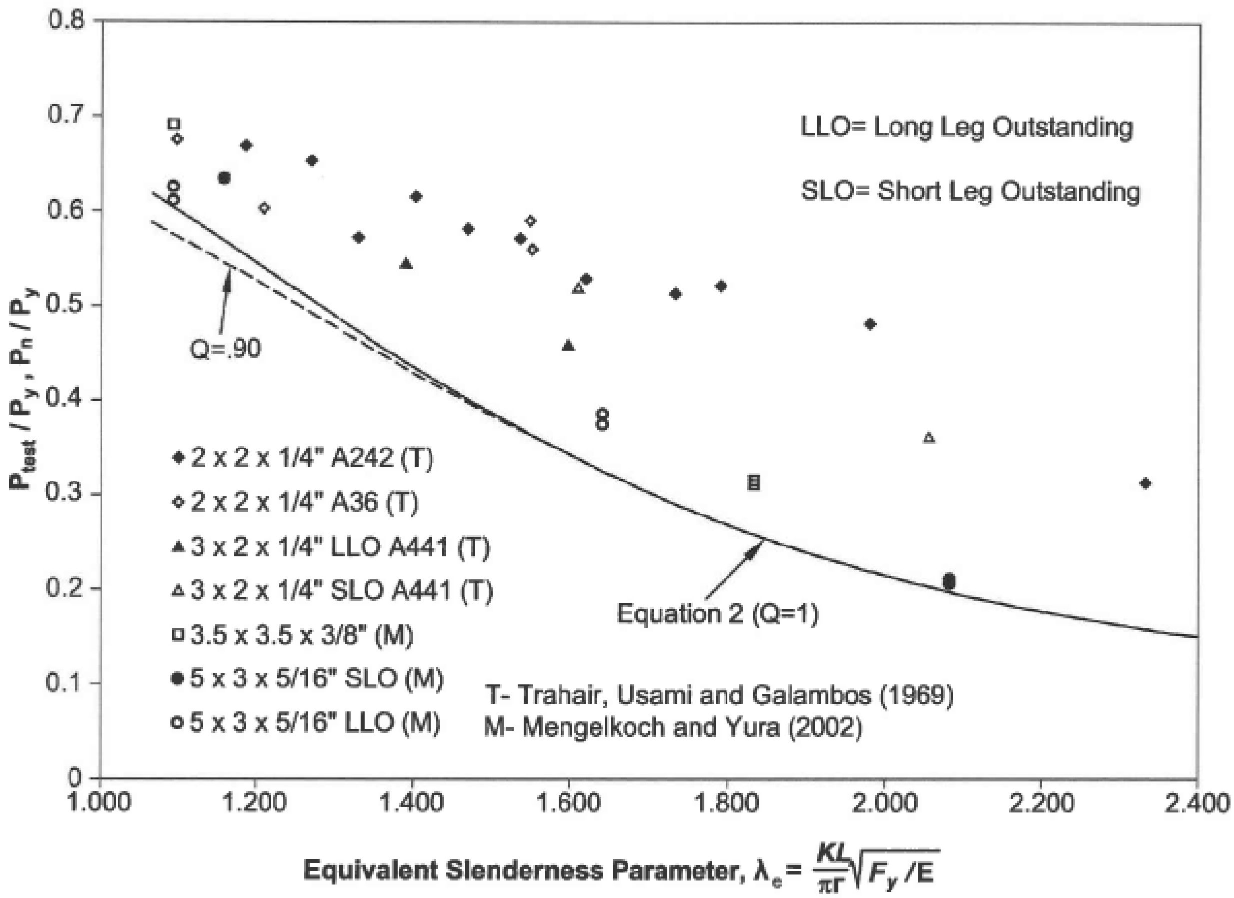

Figure 5: Equation 14 Shown as Equation 2 as Compared to Fixed-end Test Results

The pinned-end tests were compared with the planar truss expressions of Eq. 15 in Fig. 6. With end restraint the test data would be higher while the design load is lower at ϕP_n. Comparison of Eqs. 14 and 15 with equivalent slenderness procedures used in Europe are illustrated in Fig. 7.

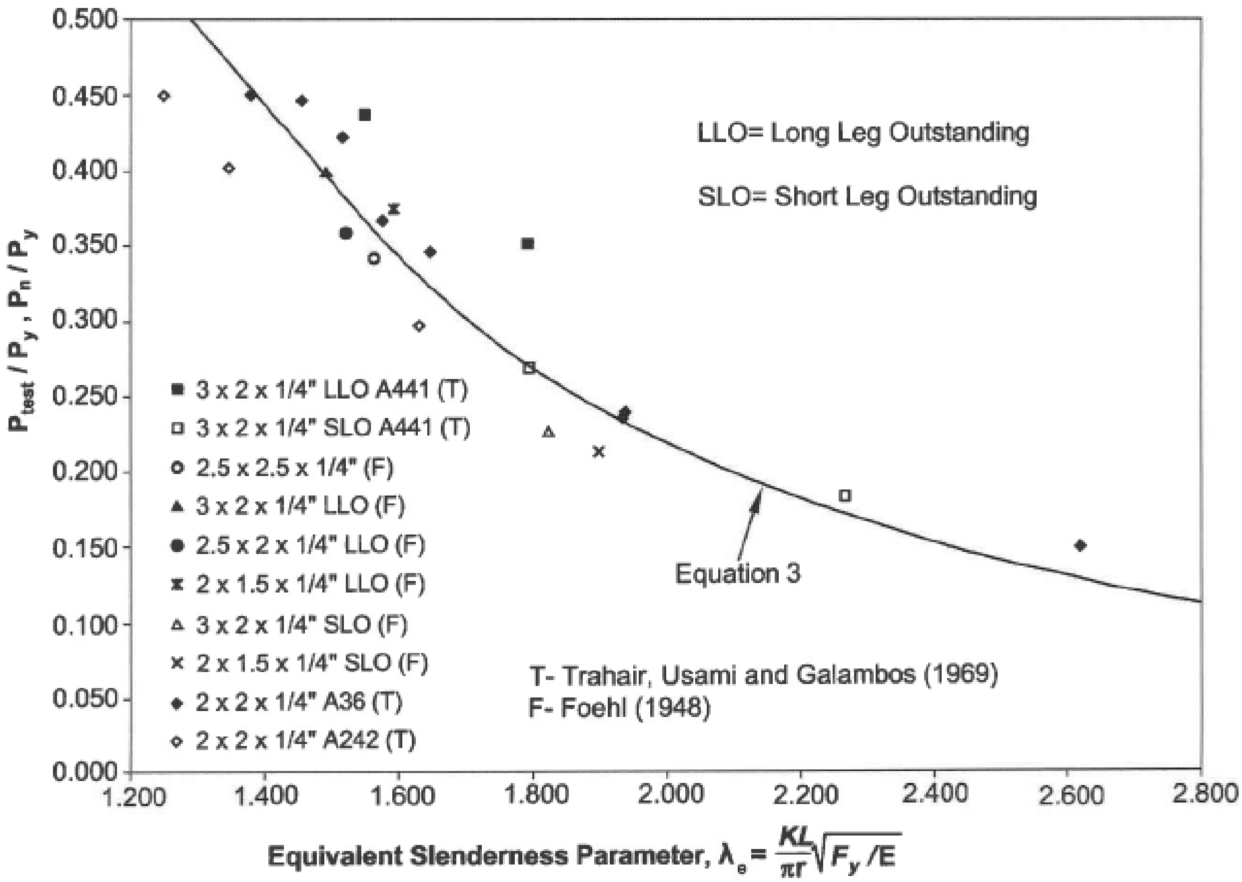

Figure 6: Equation 15 Shown as Equation 3 as Compared to Pinned-end Test Results

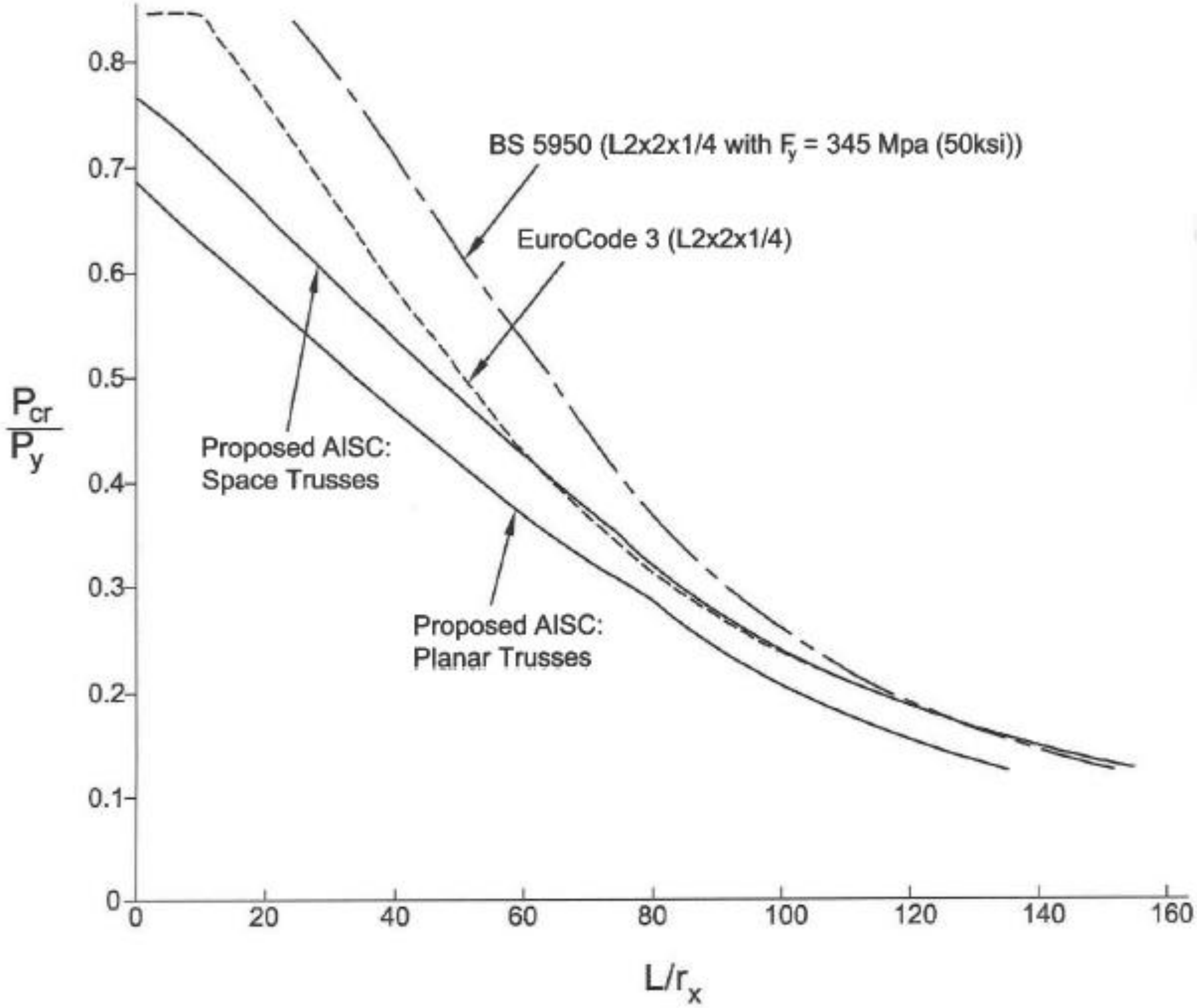

Figure 7: Comparison of Equivalent Slenderness Procedures for L2x2x1/4

Single Angle Provisions in the Main AISC Specification
Integration of the single angle provisions into the main AISC specification began in November of 1999. The TC 12 committee on Single Angles was eliminated in 2002 after the equivalent slenderness provisions were developed.

A. The single-angle provision changes in AISC, 2005 were:

1. Introduction of the equivalent slenderness provisions.
2. Change of the elastic *lateral-torsional buckling* moment M_{ob} to M_e.
3. Simplification of the *local buckling* transition express (Eq. 10) to
$$M_n = F_y S_c [2.43 - 1.72(b/t)\sqrt{F_y / E}] \tag{10a}$$
4. Use of Equation H2-1 can be used for combined stresses for unsymmetric members.

B. The single-angle provision changes in AISC, 2010 were:

1. An upper limit b/t = 20 was imposed for which flexural-torsional buckling would not need to be checked for axially loaded angles. This meant that none of the standard hot-rolled sections needed checking.
2. Axial compression was no longer permitted in combination with the special geometric axis flexural provisions. Axial compression must be combined with principal-axis flexure.

C. The single-angle provision changes in AISC, 2016 were:

1. The upper limit for which flexural-torsional buckling need not be checked was changed from 20 to $0.71\sqrt{E/F_y}$ in order to treat angles with F_y > 36 ksi the same as angles made from A36 steel.
2. Since the treatment of all slender unstiffened elements was changed to an effective area approach, the stress reduction factor Q (Eqs. 1 and 2) was no longer used to modify axial capacity.
3. The lateral-torsional buckling expressions were designated as M_{cr} rather than M_e and changes were made to the M_{cr} expressions.
   a. For equal-leg angles bent about the geometric axis:

   With maximum compression at the toe, $$M_{cr} = \frac{0.58Eb^4tC_b}{L_b^2}\left(\sqrt{1+0.88\left(\frac{L_bt}{b^2}\right)^2}-1\right) \qquad (16)$$

   With maximum tension at the toe one replaces the -1 with a +1. The coefficients in the equation were modified to improve the accuracy. The expression was developed by approximating the angle properties as two line elements of thickness t. However, the line element length is actually b-t/2. So the expression, which is approximate, was adjusted to be most accurate for b/t = 16 rather than b/t = 0.

   b. For major principal-axis bending an approximate M_{cr} expression was used previously for equal-leg angles and an exact expression was used for the unequal-leg case. So the unequal-leg expression was reconfigured so the exact expression for the equal-leg single angle could easily be obtained by setting β_w equal to zero in Eq. 17.

$$M_{cr} = \frac{9EAr_ztC_b}{8L_b}\left(\sqrt{1+\left(4.4\frac{\beta_w r_z}{L_bt}\right)^2}+4.4\frac{\beta_w r_z}{L_bt}\right) \qquad (17)$$

where

C_b is computed using Eq. F1-1 of AISC, 2016 with a maximum value of 1.5
A = cross-sectional area of angle, in.^{2} (mm^2)
L_b = laterally unbraced length of member, in. (mm)
r_z = radius of gyration about the minor principal axis, in. (mm)
t = thickness of angle leg, in. (mm)
β_w = section property for single angles about major principal axis, in. (mm). β_w is positive with short legs in compression and negative with long legs in compression for unequal-leg angles, and zero for equal-leg angles. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w shall be used.

Summary and Conclusions

A history of the development of the AISC single-angle design provisions is presented. Beginning only with local buckling expression for axial compression and an expressed need for more explicit information, a separate Allowable Stress Design document was developed. This transitioned into a separate LRFD document and finally into part of the main AISC specification. Background on the changes made over the years were presented along with reasons for the changes.

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