Specification for Allowable Stress Design of Single Angle Members

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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:

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In addition, the full AISC Committee on Specifications has reviewed and endorsed this Specification.

1. SCOPE

This document contains allowable stress design criteria for hot-rolled, single-angle members with equal and unequal legs in tension, shear, compression, flexure and for combined stresses. It is intended to be compatible with, and a supplement to, the

1989 AISC Specification for Structural Steel Buildings—Allowable Stress Design (AISC ASD) and repeats some common criteria for ease of reference. For design purposes, several conservative simplifications and approximations were made in the Specification provisions for single-angles which can be refined through a more precise analysis.

The Specification for single-angle design supersedes any comparable but more general requirements of the AISC ASD. All other design, fabrication and erection provisions not directly covered by this document shall be in compliance with the AISC ASD. For design of slender, cold-formed steel angles, the current AISI *Specification for the Design of Cold-formed Steel Structural Members* is applicable.

2. TENSION

The allowable tension stress F_t shall not exceed $0.6F_y$ on the gross area A_g, nor $0.50F_u$ on the effective net area A_e.

a. For members connected by bolting, the net area and effective net area shall be determined from AISC ASD Specification Sects. B1 to B3 inclusive.
b. When the load is transmitted by longitudinal or a combination of longitudinal and transverse welds through just one leg of the angle, the effective net area shall be

$$A_e = 0.85A_g \tag{2-1}$$

c. When load is transmitted by transverse weld through just one leg of the angle, A_e is the area of the connected leg.

For members whose design is based on tensile force, the slenderness ratio L/r preferably should not exceed 300. Members which have been designed to perform as tension members in a structural system, but may experience some compression, need not satisfy the compression slenderness limits.

3. SHEAR

The allowable shear stress due to flexure and torsion shall be:

$$F_v = 0.4F_y \tag{3-1}$$

4. COMPRESSION

The allowable compressive stress on the gross area of axially compressed members shall be:

when $Kl/r < C_c'$

$$F_a = \frac{Q\left[1 - \dfrac{(Kl/r)^2}{2C_c'^2}\right]F_y}{\dfrac{5}{3} + \dfrac{3}{8}\left(\dfrac{Kl/r}{C_c'}\right) - \dfrac{(Kl/r)^3}{8C_c'^3}} \tag{4-1}$$

when $Kl/r > C_c'$

$$F_a = \frac{12\,\pi^2 E}{23\,(Kl/r)^2} \tag{4-2}$$

where

Kl/r = largest effective slenderness ratio of any unbraced length as defined in AISC ASD Specification Sect. E1

$$C_c' = \sqrt{\frac{2\pi^2 E}{QF_y}}$$

The reduction factor Q shall be:

when $b/t \leq 76/\sqrt{F_y}$

$$Q = 1 \tag{4-3a}$$

when $76/\sqrt{F_y} < b/t < 155/\sqrt{F_y}$

$$Q = 1.340 - 0.00447\,(b/t)\sqrt{F_y} \tag{4-3b}$$

when $b/t \geq 155/\sqrt{F_y}$

$$Q = 15{,}500/[F_y\,(b/t)^2] \tag{4-3c}$$

where

b = full width of the longest angle leg
t = thickness of angle

For short, thin or unequal leg angles, flexural-torsional buckling may produce a significant reduction in strength. In such cases, the allowable stress shall be determined by the previous equations substituting an equivalent slenderness ratio $(Kl/r)_{equiv}$ for Kl/r

$$(Kl/r)_{equiv} = \pi\sqrt{E/F_e} \tag{4-4}$$

where F_e is the elastic buckling strength for the flexural-torsional mode.

For members whose design is based on compressive force, the largest effective slenderness ratio preferably should not exceed 200.

5. FLEXURE

The allowable bending stress limits of Sect. 5.1 shall be used as indicated in Sects. 5.2. and 5.3.

5.1. Allowable Bending Stress

The bending stress is limited to the minimum allowable value F_b determined from Sects. 5.1.1, 5.1.2 and 5.1.3, as applicable.

5.1.1. To prevent local buckling when the tip of an angle leg is in compression,

when $b/t \leq 65/\sqrt{F_y}$:

$$F_b = 0.66F_y \tag{5-1a}$$

when $65/\sqrt{F_y} < b/t \leq 76/\sqrt{F_y}$:

$$F_b = 0.60F_y \tag{5-1b}$$

when $b/t > 76/\sqrt{F_y}$:

$$F_b = 0.60Q\,F_y \tag{5-1c}$$

where

b = full width of angle leg in compression
Q = stress reduction factor per Eq. (4-3a), (b) and (c)

An angle leg shall be considered to be in compression if the tip of the angle leg is in compression, in which case the calculated stress f_b at the tip of this leg is used.

5.1.2. For the tip of an angle leg in tension

$$F_b = 0.66\ F_y \tag{5-2}$$

5.1.3. To prevent lateral-torsional buckling, the maximum compression stress shall not exceed:

when $F_{ob} \leq F_y$

$$F_b = [0.55 - 0.10\ F_{ob}/F_y]\ F_{ob} \tag{5-3a}$$

when $F_{ob} > F_y$

$$F_b = [0.95 - 0.50\ \sqrt{F_y/F_{ob}}]\ F_y \leq 0.66F_y \tag{5-3b}$$

where

F_b = allowable bending stress at leg tip, ksi
F_{ob} = elastic lateral-torsional buckling stress, from Sect. 5.2 or 5.3 as applicable, ksi
F_y = yield stress, ksi

5.2. Bending About Geometric Axes

5.2.1.

a. Angle bending members with lateral-torsion restraint along the length may be designed on the basis of geometric axis bending with stress limited by the provisions of Sects. 5.1.1 and 5.1.2.
b. For equal leg angles if the lateral-torsional restraint is only at the point of maximum moment, the stress, f_b, is calculated on the basis of geometric axis bending limited by F_b in Sect. 5.2.2b.

5.2.2. Equal leg angle members without lateral-torsional restraint subjected to flexure applied about one of the geometric axes may be designed considering only geometric axis bending provided:

a. The calculated compressive stress f_b, using the geometric axis section modulus, is increased by 25%.
b. For the angle leg tips in compression, the allowable bending stress F_b is determined according to Sect. 5.1.3, where

$$F_{ob} = \frac{85{,}900}{(\ell/b)^2}\ C_b\ [\sqrt{1 + 0.78\ (\ell t/b^2)^2} - 1] \tag{5-4}$$

and by b/t provisions in Sect. 5.1.1. When the leg tips are in tension, F_b is determined only by Sect. 5.1.2.

ℓ =unbraced length, in.
C_b =$1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2 \leq 1.5$ where M_1 is the smaller and M_2 the larger end moment in the unbraced segment of the beam; (M_1/M_2) is positive when the moments cause reverse curvature and negative when bent in single curvature. C_b shall be taken as unity when the bending moment at any point within an unbraced length is larger than at both ends of its length.

5.2.3. Unequal leg angle members without lateral-torsional restraint subjected to bending about one of the geometric axes shall be designed using 5.3.

5.3. Bending about Principal Axes

Angles without lateral-torsional restraint shall be designed considering principal-axis bending except for cases covered by Sect. 5.2.2. Bending about both of the principal axes shall be evaluated using the interaction equations in AISC ASD Specification Sect. H1.

5.3.1. Equal leg angles

a. Major axis bending

The principal bending compression stress f_{bw} shall be limited by F_b in Sect. 5.1.3, where

$$F_{ob} = C_b \frac{28{,}250}{(\ell/t)} \tag{5-5}$$

and by b/t provisions in Sect. 5.1.1.

b. Minor axis bending

The principal bending stress f_{bz} shall be limited by F_b in Sect. 5.1.1 when the leg tips are in compression, and by Sect. 5.1.2 when the leg tips are in tension.

5.3.2. Unequal leg angles

a. Major axis bending

The principal bending compression stress f_{bw} shall be limited by F_b in Sect. 5.1.3, where

$$F_{ob} = \frac{143{,}100 I_z}{S_w \ell^2} C_b \left[\sqrt{\beta_w^2 + 0.052\,(\ell t/r_z)^2} + \beta_w\right] \tag{5-6}$$

and by b/t provisions in Sect. 5.1.1 for the compression leg.

S_w = section modulus to tip of leg in compression, in.^{3}
I_z = minor principal axis moment of inertia, in.^{4}
r_z = radius of gyration for minor principal axis, in.

$\beta_w = \left[\frac{1}{I_w}\int_A z(w^2 + z^2)dA\right] - 2z_o$, special section property for unequal leg angles, positive for short leg in compression and negative for long leg in compression, in. (see Commentary for values). If the long leg is in compression anywhere along the unbraced length of the member, use the negative values of β_w.

z_o = coordinate along z axis of the shear center with respect to centroid, in.
I_w = major principal axis moment of inertia, in.^{4}

b. Minor axis bending

The principal bending stress f_{bz} shall be limited by F_b in Sect. 5.1.1 when leg tips are in compression and by Sect. 5.1.2 when the leg tips are in tension.

6. COMBINED STRESSES

6.1. Axial Compression and Flexure

Members subjected to both axial compression and bending shall satisfy the requirements of AISC ASD Specification Sect. H1, subject to the following conditions:

6.1.1. In evaluating AISC ASD Specification Eqs. (H1-1) or (H1-2), 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.

6.1.2. AISC ASD Specification Eq. (H1-2) is to be evaluated at the critical member support cross section and need not be based on the maximum moments along the member length.

6.1.3. For members constrained to bend about a geometric axis with compressive stress and allowable stress determined per Sect. 5.2.1a, the radius of gyration r_b for F'_e shall be taken as the geometric axis value.

6.1.4. For equal leg angles without lateral-torsional restraint along the length and with bending applied about one of the geometric axes, the provisions of Sect. 5.2.2 shall apply for the calculated and allowable bending stresses. If Sect. 5.2.1b or 5.2.2 is used for F_b, the radius of gyration about the axis of bending r_b for F'_e should be taken as the geometric axis value of r divided by 1.35 in the absence of a more detailed analysis.

6.1.5. For members that do not meet the conditions of Sect. 6.1.3 or 6.1.4, the evaluation shall be based on principal axis bending according to Sect. 5.3 and the subscripts x and y in AISC ASD Specification Sect. H1 shall be interpreted as the principal axes, w and z, in this Specification when evaluating the length without lateral-torsional restraint.

6.2. Axial Tension and Bending

Members subjected to both axial tension and bending stresses due to transverse loading shall satisfy the requirements of AISC ASD Specification Sect. H2. Bending stress evaluation shall be as directed by Sects. 6.1.3, 6.1.4 and 6.1.5 for compressive stresses.

COMMENTARY

C2. TENSION

The criteria for the design of tension members in AISC ASD Specification Sect. D1 have been adopted for angles with bolted connections. However, recognizing the effect of shear lag when the connection is welded, the criteria in Sect. B3 of the AISC Allowable Stress Design Specification have been applied.

The advisory upper slenderness limits are not due to strength considerations, but are based on professional judgement and practical considerations of economics, ease of handling and transportability. The radius of gyration about the z-axis will produce the maximum ℓ/r and, except for very unusual support conditions, the maximum $K\ell/r$. Since the advisory slenderness limit for compression members is less than for tension members, an accommodation has been made for members with $K\ell/r > 200$ that are always in tension, except for unusual load conditions which produce a small compression force.

C3. SHEAR

Shear stresses in a single angle member are the result of the gradient in the bending moment along the length (flexural shear) and the torsional moment.

The elastic stress due to flexural shear may be computed by

$$f_v = 1.5V_b/bt \tag{C3-1}$$

where

V_b = component of the shear force parallel to the angle leg with length b, and thickness t, kips

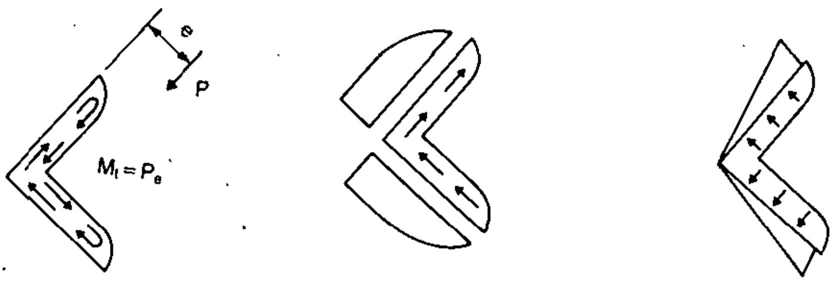
The stress, which is constant through the thickness, should be determined for both legs to determine the maximum.

The 1.5 factor is the calculated elastic value for equal leg angles loaded along one of the principal axes. For equal leg angles loaded along one of the geometric axes (laterally braced or unbraced) the factor is 1.35. Constants between these limits may be calculated conservatively from V_bQ/It to determine the maximum stress at the neutral axis.

Alternatively, a uniform flexural shear stress in the leg of V_b/bt may be used due to inelastic material behavior and stress redistribution.

If the angle is not laterally braced against twist a torsional moment is produced equal to the applied transverse load times the perpendicular distance e to the shear center, which is at the heel of the angle cross section. Torsional moments are resisted by two types of shear behavior: pure torsion (St. Venant) and warping torsion (AISC, 1983). If the boundary conditions are such that the cross section is free to warp, the applied torsional moment M_T is resisted by pure shear stresses as shown in Fig. C3.1a. Except near the ends of the legs, these stresses are constant along the length of the leg and the maximum value can be approximated by

$$f_v = M_T t/J = 3M_T/At \tag{C3-2}$$



a. Pure torsion *b. In-plane warping* *c. Across-thickness warping*

Fig. C3.1. Shear stresses due to torsion

where

J = torsional constant, in.^{4} (approximated by $\Sigma\, bt^3/3$ when precomputed value unavailable.)
A = angle cross-sectional area, in.^{2}

At section where warping is restrained, the torsional moment is resisted by warping shear stresses of two types (Gjelsvik, 1981). One type is in-plane (contour) as shown in Fig. C3.1b, which varies from zero at the toe to a maximum at the heel of the angle. The other type is across the thickness and is sometimes referred to as secondary warping shear. As indicated in Fig. C3.1c, it varies from zero at the heel to a maximum at the toe.

In an angle with typical boundary conditions and unrestrained load point, the torsional moment produces all three types of shear stresses (pure, in-plane warping and secondary warping) in varying proportions along its length. The total applied moment is resisted by a combination of three types of internal moments that differ in relative proportions according to the distance from the boundary condition. Using typical angle dimensions, it can be shown that the two warping shears are approximately the same order of magnitude and are less than 20% of the pure shear stress for the same torsional moment. Therefore, it is conservative to compute the torsional shear stress using the pure shear equation and total applied torsional moment M_T as if no warping restraint were present. This stress is added directly to the flexural shear stress to produce a maximum surface shear stress near the mid-length of a leg. Since this sum is a local maximum that does not extend through the thickness, applying the limit of $0.4F_y$ adds another degree of conservatism relative to the design of other structural shapes.

In general, torsional moments from laterally unrestrained transverse loads also produce warping normal stresses that are superimposed on bending stresses. However, since the warping strength for a single angle is relatively small, this additional bending effect is negligible and often ignored in design practice.

C4. COMPRESSION

The provisions for the allowable compression stress account for three possible failure modes that may occur in an angle depending on its proportions: general column flexural buckling, local buckling of thin legs and flexural-torsional buckling of the member. The Q factor in the equations for allowable stress accounts for the local buckling and the provisions are extracted from AISC ASD Specification Appendix B5. The F_e used for modification of the slenderness ratio for flexural-torsional buckling may be based on the provisions of Appendix E of the AISC LRFD Specification (AISC, 1986) while conservatively neglecting warping resistance, which is approximately less than 3% at unbraced lengths of 5 ft or more. The angle-end restraint conditions should be considered in determining the appropriate member effective length.

The equations for the elastic flexural-torsional buckling stress from AISC LRFD Appendix E with no warping resistance are:

For equal leg angles with w as the axis of symmetry:

$$F_e = \frac{F_{ew} + F_{ej}}{2H}\left(1 - \sqrt{1 - \frac{4\, F_{ew}\, F_{ej}\, H}{(F_{ew} + F_{ej})^2}}\right) \tag{C4-1}$$

For unequal leg angles, F_e is the lowest root of the cubic equation:

$$\begin{aligned}(F_e - F_{ez})\,(F_e - F_{ew})\,(F_e - F_{ej}) - F_e^2 (F_e - F_{ew})\,(z_o/\bar{r}_o)^2 -\\ F_e^2 (F_e - F_{ez})\,(w_o/\bar{r}_o)^2 = 0\end{aligned} \tag{C4-2}$$

where

E = modulus of elasticity, ksi
G = shear modulus, ksi
J = torsional constant = $\Sigma\, bt^3/3 = t^2 A/3$, in.^{4}
I_z, I_w = moment of inertia about *principal* axes, in.^{4}
z_o, w_o = coordinates of the shear center with respect to the centroid, in.
$\bar{r}_o^2$ = $z_o^2 + w_o^2 + (I_z + I_w)/A$, in.^{2}
H = $1 - (z_o^2 + w_o^2)/\bar{r}_o^2$
F_{ez} = $\dfrac{\pi^2 E}{(K_z \ell / r_z)^2}$, ksi
F_{ew} = $\dfrac{\pi^2 E}{(K_w \ell / r_w)^2}$, ksi
F_{ej} = $\dfrac{GJ}{A \bar{r}_o^2}$, ksi
A = cross-sectional area of member, in.^{2}
ℓ = unbraced length, in.
K_z, K_w = effective length factors in z and w directions
r_z, r_w = radii of gyration about the principal axes, in.

The coordinate axes are defined in Fig. C5.3. For equal leg angles, the flexural-torsional buckling stress will not control if

$$(K\ell/r)_{max} > 5.4(b/t)/Q \tag{C4-3}$$

This limit can be derived for equal leg angles with $Q = 1$ by equating the equation for the elastic flexural-torsional stress to the Euler equation for column flexural buckling. For unequal leg angles, flexural-torsional buckling always controls though for higher slenderness ratios, it will be approximately equal to the minimum flexural buckling stress. Also, when $Q<1$, the limit cannot be derived because Q does not appear in the Euler equation. Numerical studies of the inelastic buckling strength of angles with a wide range of proportions indicate for members that exceed the $K\ell/r$ limit, the flexural-torsional buckling stress will be only a few percent less than the column buckling stress except when one leg is more than twice the length of the other. In the latter case, reductions as high as 10% may occur.

C5. FLEXURE

Flexural stress limits are established with consideration of local buckling and lateral-torsional buckling. In addition to addressing the general case of unequal leg single angles, the equal leg angle is treated as a special case. Furthermore, bending of equal leg angles about a geometric axis, an axis parallel to one of the legs, is addressed separately as it is a very common situation.

C5.1.1. These provisions follow typical AISC criteria for single angles under uniform compression. They are conservative when a leg is subjected to nonuniform compression stress if the maximum compression stress at the leg tip is used.

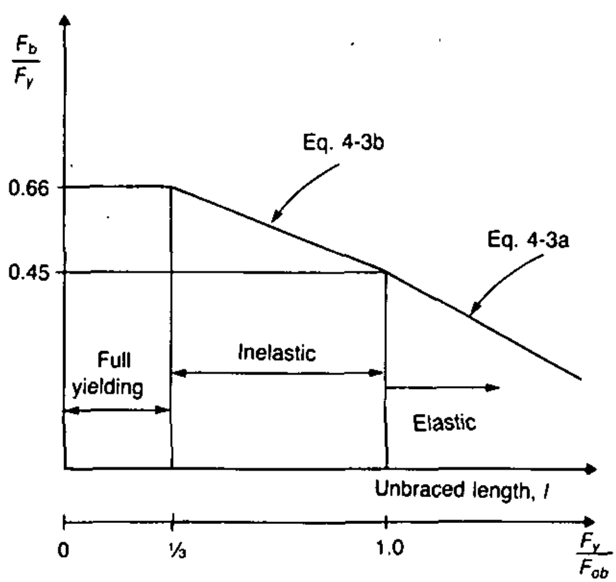


Fig. C5.1. Lateral-torsional buckling of a single-angle beam

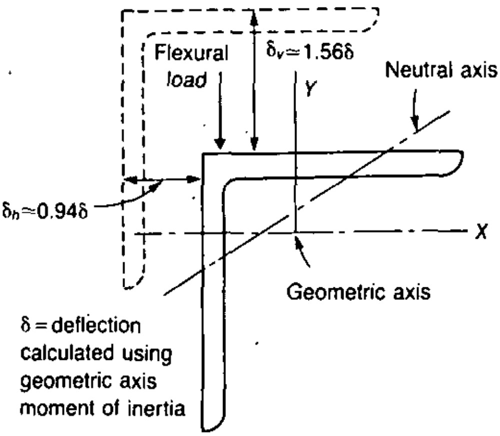


Fig. C5.2. Geometric axis bending of laterally unrestrained equal-leg angles

C5.1.2. Since the shape factor for angles is in excess of 1.5, the maximum allowable bending stress, $F_b = 0.66\ F_y$, for compact members is justified as long as instability does not control.

C5.1.3. Lateral-torsional instability may limit the allowable flexural stress of an unbraced single angle beam. As illustrated in Fig. C5.1, Eq. (5-3a) represents the elastic buckling portion with a variable factor of safety ranging from 2.22 to 1.82. Equation (5-3b) represents the inelastic buckling transition expression between $0.45F_y$ and $0.66F_y$. At F_{ob} greater than about $3F_y$, the unbraced length is adequate to develop the maximum beam flexural strength of $F_b = 0.66F_y$. These formulas were based on Australian research on single angles in flexure and on an analytical model consisting of two rectangular elements of length equal to actual angle leg width minus one-half the thickness (Leigh and Lay, 1984; Australian Institute of Steel Construction, 1975; Leigh and Lay, 1978; Madugula and Kennedy, 1985).

The familiar C_b moment gradient formula based on doubly symmetric wide flanges is used to correct lateral-torsional stability equations from the assumed most severe case of uniform moment throughout the unbraced length (C_b = 1.0). However, in lieu of a more detailed analysis, the reduced maximum limit of 1.5 is imposed for single angle beams to represent conservatively the lower envelope of this cross-section's non-uniform bending response.

C5.2.1. An angle beam loaded parallel to one leg will deflect and bend about that leg only if the angle is restrained laterally along the length. In this case simple bending occurs without any torsional rotation or lateral deflection and the geometric axis section properties should be used in the evaluation of the flexural stresses and deflection. If only the point of maximum moment is laterally braced, lateral-torsional buckling of the unbraced length under simple bending must also be checked.

C5.2.2. When bending is applied about one leg of a laterally unrestrained single angle, it will deflect laterally as well as in the bending direction and twist. Its behavior can be evaluated by resolving the load and/or moments into principal axis components and determining the sum of these principal axis flexural effects while neglecting the relatively minor torsional response. In order to simplify and expedite the design calculations for this common situation with equal leg angles, an alternate method may be used.

For such unrestrained bending of an equal leg angle, the resulting maximum normal stress at the angle tip (in the direction of bending) will be approximately 25% greater than calculated using the geometric axis section modulus. The deflection calculated using the geometric axis moment of inertia has to be increased 82% to approximate the total deflection. Deflection has two components, a vertical component (in the direction of applied load) 1.56 times the calculated value and a horizontal component of 0.94 of the calculated value. The resultant total deflection is in the general direction of the weak principal axis bending of the angle (see Fig. C5.2). These unrestrained bending deflections should be considered in evaluating serviceability.

The horizontal component of deflection being approximately 60% of the vertical deflection means that the lateral restraining force required to achieve purely vertical deflection (Sect. 5.2.1) must be 60% of the applied load value (or produce a moment 60% of the applied value) which is very significant.

The lateral-torsional buckling is limited by F_{ob} (Leigh and Lay, 1984 and 1978) in Eq. 5-4, which is based on

$$M_{cr} = \frac{2.33Eb^4t}{(1 + 3\cos^2\Theta)(K\ell)^2} \times \left[\sqrt{\sin^2\Theta + \frac{0.162(1 + 3\cos^2\Theta)(K\ell)^2t^2}{b^4}} + \sin\Theta\right] \tag{C5-1}$$

(the general expression for the critical moment of an equal leg angle) with $\Theta = -45°$ which is the most severe condition with the angle heel (shear center) in tension. Flexural loading which produces angle heel compression can be conservatively designed by Eq. (5-4) or more exactly by using the above general M_{cr} Equation with $\Theta = 45°$ (see Fig. C5.3).

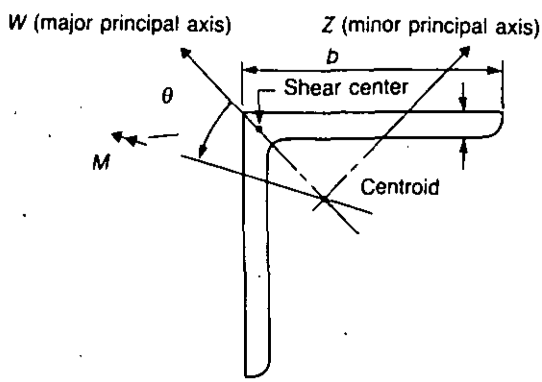


Fig. C5.3. Equal-leg angle with general moment loading

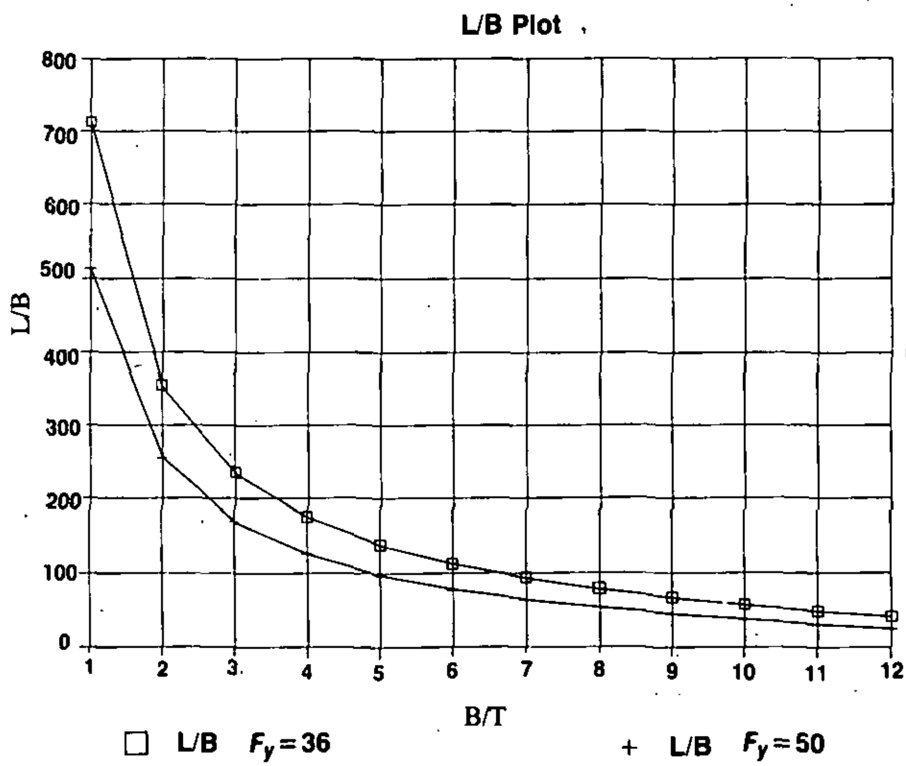


Fig. C5.4. Single-angle limits for $F_b = .66_y$

Lateral-torsional buckling will reduce the stress limit only when ℓ/b is relatively large. If the $\ell t/b^2$ parameter (which is a ratio of ℓ/b over b/t) is less than 2.43 (with $C_b = 1$), there is no need to check lateral-torsional stability inasmuch as local buckling provisions of Sect. 5.1.1 will control the allowable flexural stress.

Lateral-torsional buckling will produce $F_b < 0.66F_y$ for equal leg angles only if F_{ob} by Eq. (5-4) is less than about $3F_y$, for $C_b = 1.0$. Limits for ℓ/b as a function of b/t are shown graphically in Fig. C5.4. Local buckling must be checked separately.

Stress at the tip of the angle leg parallel to the applied bending axis is of the same sign as the maximum stress at the tip of the other leg when the single angle is unrestrained. For an equal-leg angle this stress is about one third of the maximum stress. It is only necessary to check the stress condition at the tip of the angle leg with the maximum stress when evaluating such an angle. Since this maximum applied compressive stress per Sect. 5.2.1a represents combined principal axis stresses and Eq. (5-4) represents the design limit for this combined flexural stress, only a single flexural term needs to be considered when evaluating combined flexural and axial effects.

C5.2.3. For unequal leg angles without lateral-torsional restraint the applied load or moment must be resolved into components along the two principal axis in all cases and designed for biaxial bending using the interaction equation.

C5.3.1. Under major axis bending of equal leg angles Eq. (5-5) in combination with (5-3a) or (5-3b) controls the flexural stress f_{bw} against overall lateral-torsional buckling of the angle. This is based on M_{cr}, given earlier with $\Theta = 0$.

Lateral-torsional buckling for this case will reduce the stress below 0.66 F_y only for $\ell/t \gtrsim 9400/F_y$ ($F_{obw} = 3F_y$). If the $\ell t/b^2$ parameter is less than $1.42C_b$ for this case, local buckling will control the allowable flexural stress and F_b based on lateral-torsional buckling need not be evaluated. Local buckling must be checked using 5.1.1.

C5.3.2. Lateral-torsional buckling about the major principal W axis of an unequal leg angle is controlled by M_{ow} in Eq. (5-6). Section property β_w reflects the location of the shear center relative to the principal axis of the section and the bending direction under uniform bending. Positive β_w and maximum M_{ow} occurs when the shear center is in flexural compression while negative β_w and minimum M_{ow} occurs when the shear center is in flexural tension (see Fig. C5.5). This β_w effect is consistent with behavior of singly symmetric I- shaped beams which are more stable when the compression flange is larger than the tension flange. For principal W axis bending of equal leg angles, β_w is equal to zero due to symmetry and Eq. (5-6) reduces to Eq. (5-5) for this special case.

For reverse curvature bending, part of the unbraced length has positive β_w, while the remainder negative β_w, and conservatively, the negative value is assigned for that entire unbraced segment.

β_w is essentially independent of angle thickness (less than 1% variation from mean value) and is primarily a function of the leg widths. The average values shown in Table C5.1 may be used for design.

C6. COMBINED STRESSES

The stability and strength interaction equations of AISC ASD Specification Chap. H have been adopted with modifications to account for various conditions of bending

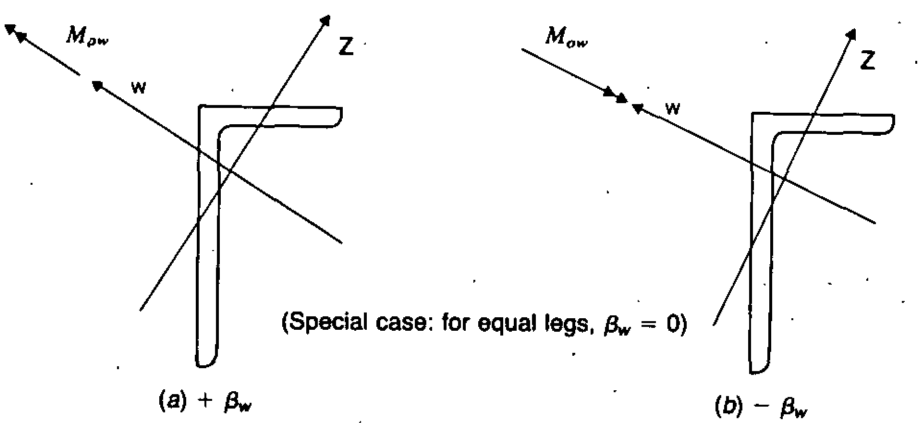


Fig. C5.5. Unequal-leg angle in bending

Table C5.1 β_w Values for Angles

Angle Size (in.)	β_w (in.)
9 × 4	6.54
8 × 6	3.31
8 × 4	5.48
7 × 4	4.37
6 × 4	3.14
6 × 3.5	3.69
5 × 3.5	2.40
5 × 3	2.99
4 × 3.5	0.87
4 × 3	1.65
3.5 × 3	0.87
3.5 × 2.5	1.62
3 × 2.5	0.86
3 × 2	1.56
2.5 × 2	0.85
Equal legs	0.00

that may be encountered. Bending will usually accompany axial loading in a single angle member since the axial load and connection along the legs are eccentric to the centroid of the cross section. Unless the situation conforms to Sect. 5.2.1 or 5.2.2 in that Sect. 6.1.3 or 6.1.4 may be used, the applied moment should be resolved about the principal axes for the interaction check.

C6.1.4. When the total maximum flexural stress is evaluated for a laterally unrestrained length of angle per Sect. 5.2, the bending axis is the inclined axis shown in Fig. C5.2. The radius of gyration modification for the moment amplification about this axis is equal to $\sqrt{1.82} = 1.35$ to account for the increased unrestrained bending deflection relative to that about the geometric axis for the laterally unrestrained length. The 1.35 factor is retained for angles braced only at the point of maximum moment to maintain a conservative calculation for this case. If the brace exhibits any flexibility permitting lateral movement of the angle, use of $r_b = r_x$ would not be conservative.

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