$$M_{nmb} = \left[\frac{\pi^2 E}{\left(\dfrac{L_b}{r_{ye}\sqrt{C_b}}\right)^2}\right]^{1/3} F_e^{2/3} S_{xc} \qquad \text{(F.4-13)}$$

F.5 SINGLE ANGLES

For single angles, the nominal flexural strength M_n shall be determined as follows.

a) For the limit state of local buckling:

(1) If a leg tip is a point of maximum compression (Figure F.5.1):

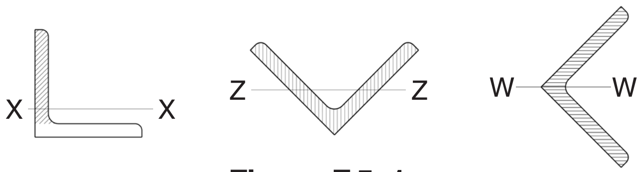


Figure F.5. 1

LIMIT STATE	M_n	b/t	Slenderness Limits
yielding	$1.5F_{cy}S_c$	$b/t \le \lambda_1$	$\lambda_1 = \dfrac{B_{br} - 1.5F_{cy}}{4.0D_{br}}$
inelastic buckling	$[B_p - 4.0D_p\,(b/t)]S_c$	$\lambda_1 < b/t < \lambda_2$	
elastic buckling	$\dfrac{\pi^2 ES_c}{(4.0\,b/t)^2}$	$b/t \ge \lambda_2$	$\lambda_2 = \dfrac{C_{br}}{4.0}$

Buckling constants B_{br}, D_{br}, and C_{br} are given in Tables B.4.1 and B.4.2.

(2) If a leg is in uniform compression (Figure F.5.2):

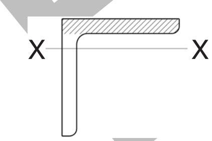


Figure F.5. 2

LIMIT STATE	M_n	b/t	Slenderness Limits
yielding	$F_{cy}S_c$	$b/t \le \lambda_1$	$\lambda_1 = \dfrac{B_p - F_{cy}}{5.0D_p}$
inelastic buckling	$[B_p - 5.0D_p\,(b/t)]S_c$	$\lambda_1 < b/t < \lambda_2$	
elastic buckling	$\dfrac{\pi^2 ES_c}{(5.0\,b/t)^2}$	$b/t \ge \lambda_2$	$\lambda_2 = \dfrac{C_p}{5.0}$

Buckling constants B_p, D_p, and C_p are given in Tables B.4.1 and B.4.2.

b) For the limit state of yielding (Figure F.5.3):

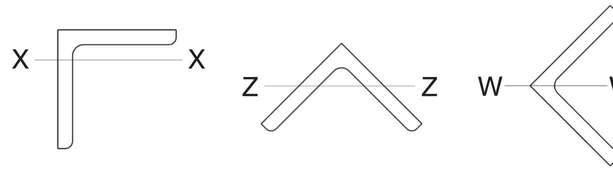


Figure F.5. 3

$$M_n = 1.5M_y \qquad \text{(F.5-1)}$$

where M_y = yield moment about the axis of bending.

c) For the limit state of lateral-torsional buckling:

(1) for $M_e \le M_y$, $M_n = (0.92 - 0.17M_e/M_y)M_e$ (F.5-2)

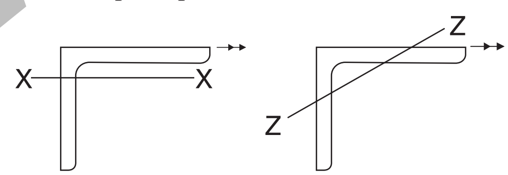
(2) for $M_e > M_y$, $M_n = (1.92 - 1.17\sqrt{M_y/M_e})M_y \le 1.3M_y$ (F.5-3)

where M_e = elastic lateral-torsional buckling moment from Section F.5.1 or F.5.2.

C_b between brace points shall be determined using Equation F.2-1 but shall not exceed 1.5.

F.5.1 Bending About Geometric Axes

Bending about a geometric axis is shown in Figure F.5.4. For combined axial compression and bending, resolve moments about principal axes and use Section F.5.2.



Subsections (a) and (b) Subsection (c)

Figure F.5. 4

a) *Angles with continuous lateral-torsional restraint:* M_n is the lesser of:

(1) local buckling strength determined by Section F.5a.
(2) yield strength determined by Section F.5b.

b) *Equal leg angles with lateral-torsional restraint only at the point of maximum moment*: Strengths shall be calculated with S_c being the geometric section modulus. M_n is the least of:

(1) local buckling strength determined by Section F.5a.
(2) yield strength determined by Section F.5b.
(3) If the leg tip is in compression, lateral-torsional buckling strength determined by Section F.5c with

$$M_e = \frac{0.73Eb^4tC_b}{L_b^2}\left[\sqrt{1 + 0.88(L_bt/b^2)^2} - 1\right] \qquad \text{(F.5-4)}$$

The requirements of this section apply to bracing of doubly- and singly symmetric I-shaped members subjected to flexure within a plane of symmetry and zero net axial force.

6.3.1 Lateral Bracing

Lateral bracing shall be attached at or near the beam compression flange, except as follows

a) At the free end of a cantilevered beam, lateral bracing shall be attached at or near the tension flange.

b) For braced beams subjected to double curvature bending, bracing shall be attached at or near both flanges at the braced point nearest the inflection point.

It is permitted to use either panel or point bracing to provide lateral bracing for beams.

6.3.1.1 Panel Bracing

Panel bracing systems shall have the strength and stiffness specified in this section. The connection of the bracing system to the member shall have the strength specified in Section 6.3.1.2 for a point brace at that location.

The required shear strength of the bracing system is

$$V_{br} = 0.01M_r\,C_d\,/h_o \tag{6-5}$$

The required stiffness of the bracing system is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{4M_rC_d}{L_{br}h_o}\right) \tag{6-6 LRFD}$$

$$\beta_{br} = \Omega\left(\frac{4M_rC_d}{L_{br}h_o}\right) \tag{6-6 ASD}$$

where

ϕ = 0.75 (LRFD)

Ω = 2.00 (ASD)

h_o = distance between flange centroids

C_d = 1.0 except C_d = 2.0 for the brace closest to the inflection point in a beam subjected to double curvature

L_{br} = unbraced length within the panel under consideration

M_r = required flexural strength of the beam in the panel under consideration

6.3.1.2 Point Bracing

The required strength of end and intermediate point braces in the direction perpendicular to the longitudinal axis of the beam is

$$P_{br} = 0.02M_r\,C_d\,/h_o \tag{6-7}$$

The required stiffness of the brace is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10M_rC_d}{L_{br}h_o}\right) \tag{6-8 LRFD}$$

$$\beta_{br} = \Omega\left(\frac{10M_rC_d}{L_{br}h_o}\right) \tag{6-8 ASD}$$

where

ϕ = 0.75 (LRFD)

Ω = 2.00 (ASD)

h_o = distance between flange centroids

C_d = 1.0 except C_d = 2.0 for the brace closest to the inflection point in a beam subject to double curvature

L_{br} = unbraced length adjacent to the point brace

M_r = largest of the required flexural strengths of the beam within the unbraced lengths adjacent to the point brace

When the unbraced lengths adjacent to a point brace have different M_r/L_{br} values, the larger value shall be used to determine the required brace stiffness.

For intermediate point bracing of an individual beam, L_{br} in Equation 6-8 need not be taken less than the maximum effective length, L_b, permitted for the beam based on the required flexural strength, M_r.

6.3.2 Torsional Bracing

Torsional bracing shall be attached at any cross-section location, and need not be attached near the compression flange.

6.3.2.1 Point Bracing

The required flexural strength of the brace about the longitudinal axis of the beam is

$$M_{br} = 0.02M_r \tag{6-9}$$

The required flexural stiffness of the brace is

$$\beta_{br} = \frac{\beta_T}{\left(1-\dfrac{\beta_T}{\beta_{sec}}\right)} \tag{6-10}$$

where

$$\beta_T = \frac{1}{\phi}\left(\frac{2.4LM_r^2}{nEI_{yeff}C_b^2}\right) \tag{6-11 LRFD}$$

$$\beta_T = \Omega\left(\frac{2.4LM_r^2}{nEI_{yeff}C_b^2}\right) \tag{6-11 ASD}$$

ACTUAL SAFETY FACTOR ON FLEXURAL RUPTURE FOR VARIOUS ALLOY-TEMPERS BASED ON THE STRESS-STRAIN CURVE, LIMITING THE MOMENT TO THE LESSER OF ZF_{ty}/Ω_y AND ZF_{tu}/Ω_u

alloy-temper	E ksi	F_{ty} ksi	F_{tu} ksi	F_{ty}/F_{tu}	ε_u	Ω
2014-T6	10,800	53	60	0.88	0.07	1.91
5052-H32	10,100	23	31	0.74	0.11	2.14
5083-O	10,300	18	40	0.45	0.16	3.35
5083-H116	10,300	31	44	0.70	0.12	2.24
6061-T6 plate	10,000	35	42	0.83	0.10	1.93
6061-T6 extrusion	10,000	35	38	0.92	0.08	1.92
6063-T5	10,000	16	22	0.73	0.08	2.17
6063-T6	10,000	25	30	0.83	0.10	1.93

The safety factors in the table above are great enough to justify using the plastic modulus. In the case of 2014-T6 above and other alloy-tempers not listed above for which k_t > 1, the available rupture moment would be further reduced by the tension coefficient k_t to account for notch sensitivity.

F.3 LOCAL BUCKLING

The *Specification* provides three methods for determining the local buckling strength.

F.3.1 Weighted Average Method

Using equation F.3-1, the weighted average method combines strengths determined separately for each element from the *Specification* Sections B.5.4.1 through B.5.4.6 and Sections B.5.5.1 through B.5.5.5. This method was originally based on tests by Jombock and Clark (1968) of formed sheet beams and used for the weighted average compression and tensile bending strengths in *Specification* editions prior to 2005. Kim (2003) improved the weighted average method accuracy for a variety of members, as illustrated in Figure CF.3.1. The distance c for a compression flange is the distance to its centerline because buckling is based on the flange's average stress.

For beams composed of a single element, the weighted average local buckling strength is the local buckling strength of the element. For example, the weighted average local buckling strength of a round tube is simply the local buckling strength of the tube determined using Section B.5.5.4.

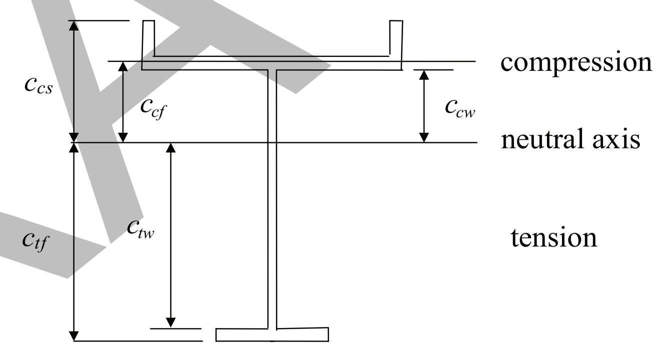


Figure CF.3.1

F.3.2 Direct Strength Method

In the direct strength method, the local buckling strength of the shape as a whole is determined by analysis such as the finite strip method which directly includes the interaction of the elements. This method is the most accurate of the three methods, and is based on the approach proposed by Kissell (2013). The advantage of this method is that the only section properties required to determine the local buckling strength are the plastic and elastic section moduli.

F.3.3 Limiting Element Method

This method limits the flexural stress in any element of the shape to the local buckling stress of that element. This method is less accurate because it does not account for interaction between elements.

F.4 LATERAL-TORSIONAL BUCKLING

Lateral-torsional buckling strengths are based on a beam

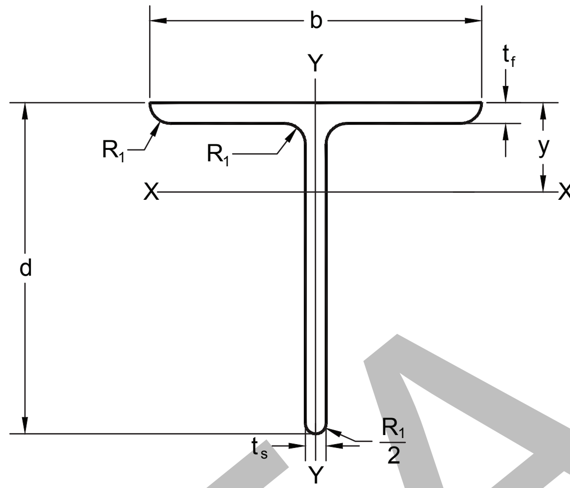


Table 14
TEES

Designation T $d \times b \times$ Wt in. in. lb/ft	Stem Thickness t_s in.	Flange Thickness t_f in.	Area A in²	R_1 in.	Axis X-X I_x in⁴	Axis X-X S_x in³	Axis X-X r_x in.	Axis X-X y in.	Axis Y-Y I_y in⁴	Axis Y-Y S_y in³	Axis Y-Y r_y in.
T 1.25 × 1.50 × 0.384	0.125	0.125	0.326	0.125	0.045	0.049	0.371	0.327	0.032	0.043	0.314
T 1.63 × 1.75 × 0.476	0.125	0.125	0.405	0.125	0.100	0.083	0.496	0.434	0.052	0.059	0.357
T 1.00 × 2.00 × 0.421	0.125	0.125	0.358	0.125	0.025	0.032	0.266	0.212	0.078	0.078	0.466
T 1.75 × 2.00 × 0.531	0.125	0.125	0.451	0.125	0.128	0.098	0.532	0.451	0.078	0.078	0.415
T 1.25 × 2.50 × 0.652	0.156	0.156	0.554	0.125	0.062	0.063	0.333	0.265	0.188	0.151	0.583
T 2.00 × 2.50 × 0.789	0.156	0.156	0.671	0.125	0.241	0.161	0.599	0.500	0.189	0.151	0.530
T 2.00 × 3.00 × 0.881	0.156	0.156	0.749	0.125	0.254	0.164	0.582	0.456	0.330	0.220	0.663
T 2.50 × 3.00 × 1.17	0.188	0.188	0.995	0.188	0.565	0.302	0.753	0.632	0.393	0.262	0.629
T 3.00 × 4.00 × 1.50	0.188	0.188	1.28	0.188	1.03	0.448	0.897	0.708	0.947	0.474	0.861
T 4.00 × 4.00 × 2.27	0.250	0.250	1.93	0.250	2.98	1.02	1.24	1.08	1.24	0.619	0.801
T 5.00 × 4.00 × 2.57	0.250	0.250	2.18	0.250	5.54	1.57	1.59	1.47	1.24	0.620	0.754
T 3.00 × 6.00 × 3.24	0.312[4]	0.312	2.75	0.312[4]	1.83	0.77	0.81	0.62	5.63	1.88	1.43
T 4.00 × 6.00 × 3.88	0.375[4]	0.313	3.30	0.313[4]	4.78	1.59	1.20	1.00	5.65	1.88	1.31
T 4.00 × 6.00 × 4.79	0.375[4]	0.450	4.07	0.312[4]	5.02	1.61	1.11	0.88	8.12	2.71	1.41
T 7.50 × 7.50 × 9.46	0. 500[4]	0.750	9.04	0.625[4]	42.7	7.51	2.17	1.81	26.5	7.07	1.71
T 7.50 × 7.50 × 14.4	1.13[4]	0.750	13.3	0.625[4]	74.8	15.1	2.37	2.55	27.1	7.22	1.43
T 6.00 × 8.00 × 11.2	0.500[4]	0.860	9.56	0.500[4]	22.9	4.82	1.55	1.24	36.8	9.19	1.96

1. Users are encouraged to check availability with suppliers.
2. Dimensional tolerances are given in *Aluminum Standards and Data*.
3. Weights are for a density of 0.098 lb/in³.
4. Both flange and stem of these shapes have square ends. Fillet radius R_1 applies only to juncture of stem and flange.

f) The inspector shall not direct or approve any deviation from the contract documents or approved fabrication or erection drawings without the written approval of the owner's designated representatives for design and construction.

8. Contracts

8.1 TYPES OF CONTRACTS

For contracts that stipulate a lump sum price, the contract documents shall specify the work required to be performed by the fabricator and erector.

For contracts that stipulate a price by weight, the work required to be performed by the fabricator and erector, the materials, and the character of fabrication and conditions of erection shall be specified in the contract documents.

For contracts that stipulate a price per item, the work required to be performed by the fabricator and the erector shall be based on the quantity and character of the items described in the contract documents.

8.2 CALCULATION OF WEIGHTS

For contracts stipulating a price by weight for fabricated structural aluminum that is delivered or erected, weights shall be calculated from the gross weight of materials shown on the fabrication drawings.

The unit weight of aluminum shall be taken as 0.1 lb/in^3 [2700 kg/m^3].

The weights of fabrication or erection weld metal and protective coatings shall not be included in calculating the weight for purposes of payment.

8.3 REVISIONS TO THE CONTRACT DOCUMENTS

Revisions to the contract documents shall be confirmed by change order. Unless otherwise noted, the issuance of a revision to the contract documents shall constitute authorization by the owner that the revision is released for construction. The contract price and schedule shall be adjusted in accordance with Sections 8.4 and 8.5.

8.4 CONTRACT PRICE ADJUSTMENT

When the scope of work of the fabricator or erector is changed from that previously established by the contract documents, the contract price shall be adjusted appropriately. To determine the adjustment, the fabricator and erector shall consider the quantity of work added or deleted, modifications in the character of the work, and timing of the change with respect to the status of material ordering, detailing, and fabrication and erection operations.

The fabricator and erector shall submit requests for contract price adjustments to the owner in a timely manner and shall provide a description of the change sufficient to permit evaluation.

Price-per item and price by weight contracts shall provide for additions or deletions to the quantity of work made before the work is released for construction. When changes are made to the character of the work at any time, or when additions or deletions are made to the quantity of the work after it is released for construction, the contract price shall be suitably adjusted.

8.5 SCHEDULE

If design drawings are not available at the time of bidding, the contract schedule shall state when design drawings will be released for construction. The contract schedule shall state when the job site will be ready and accessible to the erector.

The fabricator and erector shall inform the owner's designated representatives for design and construction in a timely manner of the effect any revision has on the contract schedule. If fabrication or erection is delayed due to revisions to the contract or the actions of others, the fabricator or erector shall be compensated for additional costs incurred.