One Engineer's Opinion

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Q. *A frequently encountered design problem is the stability of tee sections so loaded as to put the stem into compression. Figure 1 illustrates two common cases of such loading: first, where a truss or joist having tees for chords must be used to support loads such as hoists, heaters, process piping, etc., and second, where a tee is used as a cantilever section to support a canopy or roof overhang.*

These members constitute beams which are flangeless on the compression side. Both Formulas (4) and (5) of AISC Specification Sect. 1.5.1.4.5 contain terms which are dependent upon compression flange dimensions.

What is the proper method for determining the allowable bending stress for tee sections such as those in Fig. 1?

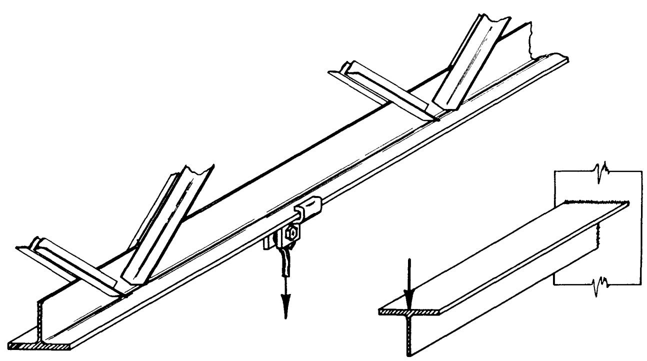

Figure 1

A. Formula (4) is intended for sections with a compression flange having considerable lateral bending stiffness. Formula (5) is intended for use with members having generally an I-shape. These formulas are simplifications which result from the substitution of the properties of an I-shape of usual proportion into the more complex theoretical formulas for the critical stress due to St. Venant's torsion and the critical stress due to warping torsion. They are inappropriate for tee sections; thus, for such sections a more rigorous formula must be used. Tee sections, which at first glance may appear to be unstable with regard to lateral buckling, prove to be quite satisfactory under the usual conditions of use.

Equation (4.26) of *Guide to Design Criteria for Metal Compression Members*[1] gives safe approximations of the results obtained by an exact solution for lateral buckling critical stress for members symmetrical about the *y-y* axis but unsymmetrical about the *x-x* axis:

$$\sigma_{cr} = \frac{\pi^2 Ed}{2S_c(KL)^2}\left[I_c - I_t + I_y\sqrt{1 + \frac{4GJ(KL)^2}{\pi^2 I_y Ed^2}}\right] \tag{4.26}$$

where, for the tee sections under consideration,

S_c = section modulus referred to the compression area

I_c = moment of inertia of compression area about *y-y* axis

I_t = moment of inertia of tension area about *y-y* axis

$I_y = I_t + I_c \qquad I_c \cong 0 \qquad I_t = I_y$

K = 1 (for a simple beam condition)

J = torsion constant

G = shear modulus of elasticity = 12×10^6

$E = 29 \times 10^6$

Substituting and rearranging,

$$\sigma_{cr} = \frac{143 I_y d}{S_c L^2}\left[\sqrt{1 + \frac{JL^2}{5.95 I_y d^2}} - 1\right] \times 10^6$$

A simple, reasonable approximation of J is

$$J = \sum \frac{bt^3}{3} = \frac{bt_f^3 + dt_w^3}{3}$$

where t_w = thickness of web

t_f = thickness of flange

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1. *Column Research Council,* Guide to Design Criteria for Metal Compression Members, *1960.*

SECTION	SPAN	σ_{cr}
ST 12 WF 38	240 in.	53,600
ST 10 WF 31	200 in.	56,600
ST 8 WF 18	160 in.	47,800
ST 7 WF 15	140 in.	65,300
ST 6 WF 13.5	120 in.	108,000

Table 1

Table 1 shows values of σ_{cr} for a representative group of standard tees with a span equal to $20d$, determined from Equation (4.26). Note that in each case σ_{cr} is far above the yield point of A36 steel. This indicates that failure of the member would occur by some other mode than general lateral buckling such as yielding or local buckling. These results might be expected, since load is applied to these members in the plane of symmetry virtually at the shear center and through the shear center; therefore, there is negligible tendency toward lateral torsional buckling.

The allowable bending stress, F_b, to be recommended in the cases under consideration, assuming any reasonable method of application of load to the section, would be $0.60F_y$ for sections meeting the requirements of Sect. 1.9.1 of the AISC Specification.

Q. *When a composite beam receives its significant loading by concentrated loads at the third points, the question is frequently raised as to the need for shear connectors in the center third of the span. If one assumes a relatively weightless beam, shear connectors could be divided equally in each outer third of the span. However, another possible interpretation is to equally space all connectors throughout the entire span length, since the point of maximum moment could be chosen at midspan as well.*

Is a minimum number of connectors needed to resist slab pull-out and deflection in the center third where no appreciable horizontal shear transfer exists?

A. The Specification *requires* that the number of shear connectors provided shall be adequate to fully develop the cross section (Formulas 18 and 19) and is *permissive* as far as the use of uniform spacing between point of maximum moment and zero moment. For a beam supporting two superimposed loads at the one-third points of the span, no shear connectors are required between the load points, provided the number of shear connectors used between the load points and the ends of the span are adequate to resist shear forces calculated by Formulas 18 and 19. Judgment would indicate that some provision should be made to insure that the concrete slab does not separate from the steel beam in the center one-third even though there is no requirement for shear transfer in this area. Therefore, an arbitrary rule frequently recommended by engineers is to use a 24 in. maximum spacing regardless of the requirement for shear transfer. Slutter and Driscoll[1] suggest a maximum spacing of six times the slab thickness, but this is not a requirement of the AISC Specification. The shear connectors provided for hold down in low shear areas are not in addition to those required by Formulas 18 and 19.

1. *Driscoll, George C. and Slutter, Roger G.* Research on Composite Design at Lehigh University, p. 22, 1961 Proceedings of the AISC National Engineering Conference, *AISC, New York, N. Y.*