Tension-Field Design of Tapered Webs

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THIS PAPER deals with the formulation of design guides for tapered webs based on post-buckling strength. Answers are sought to two specific problems:

1. Formulation of necessary modifications to the current AISC "tension-field formula" of prismatic girders for the application to tapered webs
2. The adequacy of using the AISC tension-field formula for tapered roof beams in frames of the gable type presently encountered in structural engineering practice

Following the basic format and approach used in earlier studies[1,2,3,4] and Basler's tension-field model,[5] allowable stress expressions are derived for tapered webs. They are compared with the current AISC Specification. It is shown that for small or even moderately large tapering ratios, the average depth of the tapered segment can be used in the AISC prismatic girder design formula for tapered members.

The results of this study can be used in the design of tapered roof girders of the gable frames commonly encountered.

DESCRIPTION OF THE PROBLEM

The design criteria for thin-web girders of uniform depth have been established based on the post-buckling strength of plate elements. For thin-web panels buckled in shear between vertical stiffeners, the stress at the onset of buckling is easily obtained by classical methods. The post-buckling stress can be accounted for by the well-known "tension field concept" of Basler,[5] in which the web with the neighboring sections of flanges and the stiffeners are conceived to behave like a Pratt truss, with the stipulation that no additional compression stress may be taken by the web in the buckled state. In this circumstance a diagonal band of the web transmitting tension force in that direction is assumed to support additional web shear. The allowable shear stress formulas of the AISC Specification find their genesis in this tension-field concept.

In determining the slope of the tension field,[5] the assumption was made that only an effective band width, b_e, takes part in transmitting the additional tension force. However, when the ultimate shear force is determined, a complete tension field across the depth of the web is assumed. This inconsistency will be referred to again when formulas for allowable stress in tapered webs are being derived.

In this study, formulas for the allowable stress for tapered webs will be derived from the same proposition as has been employed for the AISC design formulas for shear stress in thin webs. It is to be anticipated that the formulas so obtained will bear some resemblance and relationship to the corresponding AISC formulas.

The procedure for obtaining the post-buckling stress is as follows: (1) the contribution of the tension force in the tension field to the vertical shear force is determined for a field of width b_e; (2) the slope of the tension field is calculated such that the vertical force is maximized; (3) the post-buckling tension stress, f_t, which is prescribed by the direct yield stress, is applied to the complete web to obtain the shear force derived from this source; (4) finally, the total shear stress is obtained from the sum of the initial shear buckling stress and the post-buckling shear stress.

DERIVATION OF TENSION FIELD FORMULAS

It will be found to be useful at this juncture to introduce AISC Formula (1.10-2) for the allowable shear stress of thin webs of uniform depth and to define the symbols. The formula may be compared with those to be derived subsequently. It is:

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right] < 0.4F_y \quad (1)$$

where

F_y = direct yield stress, ksi
t = web thickness, in.
a = clear distance between transverse stiffeners, in.
h = clear distance between flanges, in.

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$$C_v = \frac{45{,}000k}{F_y(h/t)^2} \text{ when } C_v < 0.8 \text{ (for steel)}$$
$$= \frac{190}{h/t}\sqrt{\frac{k}{F_y}} \text{ when } C_v > 0.8 \text{ (for steel)}$$
$$k = 4.00 + \frac{5.34}{(a/h)^2} \text{ when } a/h < 1.0$$
$$= 5.34 + \frac{4.00}{(a/h)^2} \text{ when } a/h > 1.0$$

Equation (1) may be adapted to tapered beams with small angle of taper by substituting the term h_{av} in place of h, where h_{av} is the average clear distance between flanges:

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h_{av})}}\right] < 0.4F_y \quad \text{(1a)}$$

A somewhat similar expression may be derived for beams with more severe angle of taper. Figure 1 represents a section of tapered web bounded by vertical stiffeners, a horizontal top flange, and a sloping bottom flange. For this case, the following nomenclature is introduced:

h_0 = clear distance between flanges at shallower end of the tapered segment, in.
h_1 = clear distance between flanges at deeper end of the tapered segment, in.
α = angle between tapered and horizontal flanges
ϕ = slope angle of tension field
θ = slope angle of web diagonal
b_e = width of tension field for calculating slope, in.
a = clear distance between transverse stiffeners, in.
β = $\tan \alpha = (h_1 - h_0)/a$
f_t = uniform tension stress in tension field

The width of the tension field b_e is:

$$b_e = h_1 \cos \phi - a(\tan \alpha + \tan \phi) \cos \phi$$
$$= h_1 \cos \phi - a \sin(\alpha + \phi) \sec \alpha \quad (2)$$

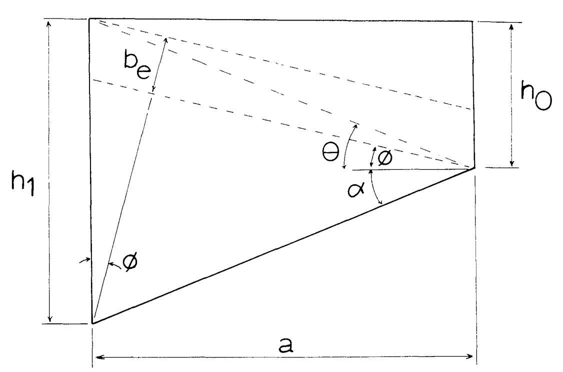

Fig. 1. Tapered web between stiffeners

The contribution to the vertical shear from the total tension force in the tension field is obtained by assuming that the tension stress, f_t, across b_e is uniform and is:

$$V_t = f_t\, b_e\, t \sin \phi \quad (3)$$

Substituting Eq. (2) into Eq. (3):

$$V_t = f_t\, t\left[\frac{h_1}{2}\sin 2\phi - \frac{a}{2}[\cos \alpha - \cos(2\phi + \alpha)] \sec \alpha\right] \quad (4)$$

The slope of the tension field is obtained by invoking the condition for $V_{t(max)}$:

$$\frac{dV_t}{d\phi} = 0 \quad (5)$$

When Eq. (5) is simplified, the expression for the slope is found to be:

$$\tan 2\phi = \frac{h_1}{a} - \tan \alpha \quad (6)$$

But,

$$\tan \alpha = (h_1 - h_0)/a \quad (7)$$

Therefore,

$$\tan 2\phi = \frac{h_0}{a} = \tan \theta \quad (8)$$

Thus, as in the case of uniform depth,[5]

$$2\phi = \theta \quad (9)$$

i.e., the slope angle of the diagonal is twice that of the tension field.

At this point, following Basler's treatment, it is assumed that the tension field develops down the complete depth of the stiffener. Therefore, the shear force at the deeper end (see Fig. 1) is:

$$V_{t_1} = f_t\, h_1\, t \cos \phi \sin \phi \quad (10)$$

From Eqs. (9) and (10):

$$V_{t_1} = \tfrac{1}{2} f_{t_1} h_1\, t \sin \theta \quad (11)$$

Noting that $\sin \theta = 1/\sqrt{1 + (a/h_0)^2}$ and representing the shear stress at initial buckling by τ_{cr}, the total shear stress is:

$$f_{v_1} = \tau_{cr_1} + \tfrac{1}{2} f_{t_1}(1/\sqrt{1 + (a/h_0)^2}) \quad (12)$$

It may be shown that for a web of uniform depth h_1:

$$\tau_{cr_1} = \frac{k\pi^2 E}{12(1 - \mu^2)(h_1/t)^2}$$
$$= \frac{1}{(1 + \gamma)^2\,(12)}\;\frac{k\pi^2 E}{(1 - \mu^2)(h_0/t)^2} \quad (13)$$

$$f_t = F_y - \sqrt{3}\,\tau_{cr_1} \quad (14)$$

and the yield shear stress is assumed to be

$$F_{vy} = F_y/\sqrt{3} \quad (15)$$

In Eq. (13), k is a dimensionless parameter.

For small tapering angles, since the initial buckling stress is small, Eq. (13) may be employed for the tapered web. Substituting Eqs. (13), (14) and (15) into (12) and simplifying:

$$f_{v1} = \frac{F_y}{\sqrt{3}}\left[C_{v1} + \frac{\sqrt{3}}{2}(1 - C_v)/\sqrt{1 + (a/h_0)^2}\right] \quad (16)$$

where

$$C_{v1} = \tau_{cr1}/F_{vy}$$

When the safety factor of 1.65 is incorporated into Eq. (16), an expression for the total allowable shear stress at the deep end of the panel comparable to Eq. (1) is obtained:

$$F_v = \frac{F_y}{2.89}\left[C_{v1} + \frac{1}{1.15}(1 - C_{v1})\frac{1}{\sqrt{1 + (a/h_0)^2}}\right] < 0.4F_y \quad (17)$$

The parameter C_{v1} for steel is a function of F_{y1}, h_1, and t may be taken conservatively as:

$$C_{v1} = \frac{45{,}000k}{F_y(h_1/t)^2} \quad (18)$$

The AISC Specification limits this value of C_{v1} to 0.8.

It can be shown that when the tension field is oriented along the other diagonal the allowable shear stress at the shallow end of the panel is:

$$F_v = \frac{F_y}{2.89}\left[C_{v0} + \frac{1}{1.15}(1 - C_{v0})\frac{1}{\sqrt{1 + (a/h_1)^2}}\right] < 0.4F_y \quad (19)$$

where

$$C_{v0} = \frac{45{,}000k}{F_y(h_0/t)^2} < 0.8 \quad (20)$$

Equations (17) and (19) are similar in general form to the AISC formula for allowable shear stress, given in Eq. (1).

The inconsistency regarding the width of the tension field, alluded to earlier, may be removed by assuming a stress distribution across the tension field. Thus, it may be assumed that the full uniform stress, f_t, extends across the width, b_e, and that the stress then decreases linearly to zero at the tapered flange as shown in Fig. 2. This assumption is conservative and purely intuitive, but it may be used both for the determination of slope[5] and shear force.[10] When it is employed in this manner, the allowable stress at the deep end of the tapered segment is derived. It may be written as:

$$F_{v1} = \frac{F_y}{2.89}\left[C_{v1} + \frac{1}{1.15}(1 - C_{v1})\frac{\tan\phi}{1 + \beta/2}\right] < 0.4F_y \quad (21)$$

where

$$\tan 2\phi = 2\,[(h_1 + h_0)/2]/a = 2h_{av}/a$$

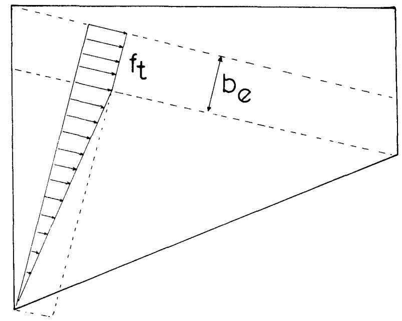

Fig. 2. Assumed tension field stress distributions

and

$$\beta = (h_1 - h_0)/a$$

The development of Eq. (21) is quite straightforward. It represents a lower bound to the allowable shear stress.

In summary, the formulas for allowable shear stress in a stiffened tapered web at a specified section may be expressed in the following general form:

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15}A\right]$$

where $C_v = F_{vcr}/F_{vy}$ at the section and A is as indicated in Table 1.

Table 1

AISC Spec. ($h = h_{av}$) (Based on Basler's assumptions)	Eq. (1a)	$A = 1/\sqrt{1 + (a/h_{av})^2}$
Modified AISC (Based on Basler's assumptions)	Eq. (17)	$A = 1/\sqrt{1 + (a/h_0)^2}$
	Eq. (19)	$A = 1/\sqrt{1 + (a/h_1)^2}$
Modified AISC (Lower bound)	Eq. (21)	$A = \tan\phi/(1 + \beta/2)$

DISCUSSION OF THE DESIGN PROVISIONS

The formula for allowable shear stress given in Eq. (21) is based upon the assumption of an extremely conservative stress distribution, and is not directly comparable to the other equations in Table 1. Equations (1a) and (17) will be compared directly by examining the corresponding representations of A in Table 1.

The expression for A in Eq. (1a) is:

$$A = 1/\sqrt{1 + (a/h_{av})^2}$$

where the average depth of web is $h_{av} = (h_0 + h_1)/2$.

The corresponding expression for A in Eq. (17) is:

$$A = 1/\sqrt{1 + (a/h_0)^2}$$

The degree of taper β may be expressed in three ways:

$$\beta = (h_1 - h_0)/a \quad \text{(i)}$$

$$\beta = (h_1 - h_{av})/2a \quad \text{(ii)}$$

$$\beta = (h_{av} - h_0)/2a \quad \text{(iii)}$$

From Eq. (iii)

$$\frac{h_0}{a} = \frac{h_{av} - 2a\beta}{a}$$

or

$$\frac{a}{h_0} = \frac{a}{h_{av} - 2a\beta} \quad \text{(iv)}$$

Therefore,

$$A = 1\Big/\sqrt{1 + \left(\frac{a}{h_{av} - 2a\beta}\right)^2}$$

or

$$A = \left[1 + \left(\frac{a}{h_{av}}\right)^2\left(1 - \frac{2a\beta}{h_{av}}\right)^{-2}\right]^{-1/2} \quad \text{(v)}$$

When the term $(2a\beta/h_{av})$ is less than unity, the first two terms of a binomial expansion may be employed to simplify the equation, thus:

$$A = \left[1 + \left(\frac{a}{h_{av}}\right)^2\left(1 + \frac{4a\beta}{h_{av}}\right)\right]^{-1/2} \quad \text{(vi)}$$

$$A = \left[1 + \left(\frac{a}{h_{av}}\right)^2 + 4\beta\left(\frac{a}{h_{av}}\right)^3\right]^{-1/2} \quad \text{(vii)}$$

It must be noted that β must be always less than or equal to $[h_{av}/(a/2)]$. The equality obtains only when $h_0 = 0$. Some specific cases will be considered:

Case 1: a/h_{av} large, β small

Case 2: a/h_{av} small, β small

Case 3: a/h_{av} large, β moderate

Case 4: a/h_{av} small, β moderate

Case 1—When $a/h_{av} \approx 30$ and $\beta \approx 1/60$, the value of A is approximately zero both for Eq. (v) and Eq. (1a) in Table 1; therefore, these properties will not control the design in this Case.

Case 2—When $a/h_{av} \approx 1$ and $\beta \approx 1/40$, the term $[1 - (2a\beta/h_{av})] \approx 1$. Equation (v) then becomes identical to Eq. (1a) for this Case.

Case 3—This Case involves relative values of the parameters h_{av} that have no physical significance.

Case 4—When $a/h_{av} \approx 1$ and $\beta \approx 1/15$, Eq. (vii) becomes $A \approx [1 + (a/h_{av})^2]$, which is the same expression for A as in Eq. (1a) in Table 1.

DESIGN EXAMPLES

The purpose of this section is to demonstrate the application of the formulas previously derived for allowable shear stress in a stiffened web and to indicate, wherever possible, how existing formulas from the AISC Specification may be modified for the design of webs with moderately large tapering angle between vertical stiffeners. The first example is typical of Case 4 of the preceding section, while the second is typical of Case 2.

Example 1

For the A36 steel girder shown in Fig. 3, determine the allowable shear stress for third point loading by the following methods:

(a) AISC formula using $h = (h_0 + h_1)/2 = h_{av}$ [Eq. (1a)]
(b) Modified formula [Eq. (17) or (19)]
(c) Modified formula [Eq. (21)]

(a) Using Eq. (1a):

$$h_{av} = \tfrac{1}{2}(60 + 66) = 63 \text{ in.}$$

$$a/h_{av} = 84/63 = 1.33$$

$$h_{av}/t = 63/\tfrac{1}{4} = 252$$

$$k = 5.34 + \frac{4.0}{(a/h_{av})^2} \text{ for } a/h_{av} > 1.0$$

$$= 5.34 + \frac{4.0}{(1.33)^2} = 7.60$$

$$C_v = \frac{45{,}000k}{F_y(h_{av}/t)^2} = \frac{45{,}000(7.60)}{36.0\,(252)^2} = 0.15$$

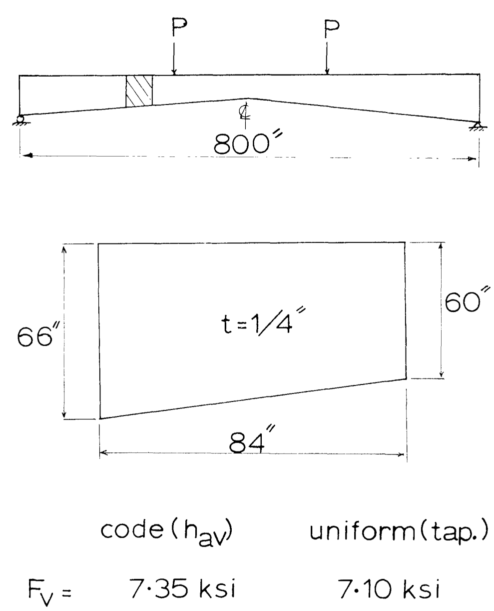

Fig. 3. Comparison of allowable stresses for tapered girder

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15}\,\frac{1}{\sqrt{1 + (a/h_{av})^2}}\right] \leq 0.4F_y \quad \text{Eq. (1a)}$$

$$= \frac{36}{2.89}\left[(0.15) + \frac{(1 - 0.15)}{1.15\sqrt{(1 + 1.33^2)}}\right]$$

$$= (12.46)(0.15 + 0.44) = 7.35 \text{ ksi}$$

(b) Using Eq. (17):

$$\beta = \frac{h_1 - h_0}{a} = \frac{66 - 60}{84} = \frac{1}{14}$$

$$k = 5.34 + \frac{4.0}{(a/h_1)^2} \text{ for } a/h_1 > 1.0$$

$$= 5.34 + \frac{4.0}{(84/66)^2} = 7.81$$

$$a/h_0 = 1.4$$

$$h_1/t = 66/\tfrac{1}{4} = 264$$

$$C_{v1} = \frac{45{,}000k}{F_y}\,[1/(h_1/t)^2]$$

$$= \frac{45{,}000\,(7.81)}{36.0}\,(1/264)^2 = 0.14$$

$$F_v = \frac{F_y}{2.89}\left[C_{v1} + \frac{1}{1.15}(1 - C_{v1})\frac{1}{\sqrt{1 + (a/h_0)^2}}\right] \quad \text{Eq. (17)}$$

$$= 12.46\left[0.14 + \frac{1}{1.15}(0.86)/(1.4)^2\right] = 7.10 \text{ ksi}$$

(c) Using Eq. (21):

$$\tan 2\phi = \frac{2(h_1 + h_0)}{2}\Big/ a = 2h_{av}/a$$

$$= 2(1/1.33) = 1.504$$

$$2\phi = 56.6^\circ$$

$$\phi = 28.3^\circ$$

From method (b), above:

$$C_{v1} = 0.14 \text{ and } \beta = 1/14$$

$$\beta/2 = (1/14)/2 = 0.036$$

$$F_{v1} = \frac{F_y}{2.89}\left[C_{v1} + \frac{1}{1.15}(1 - C_{v1})\frac{\tan\phi}{1 + \beta/2}\right] \quad \text{Eq. (21)}$$

$$= 12.46\left[0.14 + \left(\frac{0.86}{1.15}\right)\left(\frac{0.54}{1.036}\right)\right]$$

$$= 12.46\,(0.14 + 0.39)$$

$$= 6.6 \text{ ksi (lower bound solution)}$$

Example 2

The second example is taken from a recent paper[4] in which the allowable bending stress and calculated bending stress at specified sections of a tapered beam are compared. Figure 4 shows the beam as it appears in the reference, as well as additional details of problem data. The shear diagram is directly relevant and has been added for completeness.

The allowable shear stress will be determined for the two panels shown in Fig. 5, centered on sections **2** and **4**, assuming tension field action.

The allowable shear stresses will be calculated for Eq. (1a), Eq. (17) and Eq. (19). Equation (21) will not be checked here. For panel I the applicable formula is Eq. (19), while for panel II the applicable formula is Eq. (17).

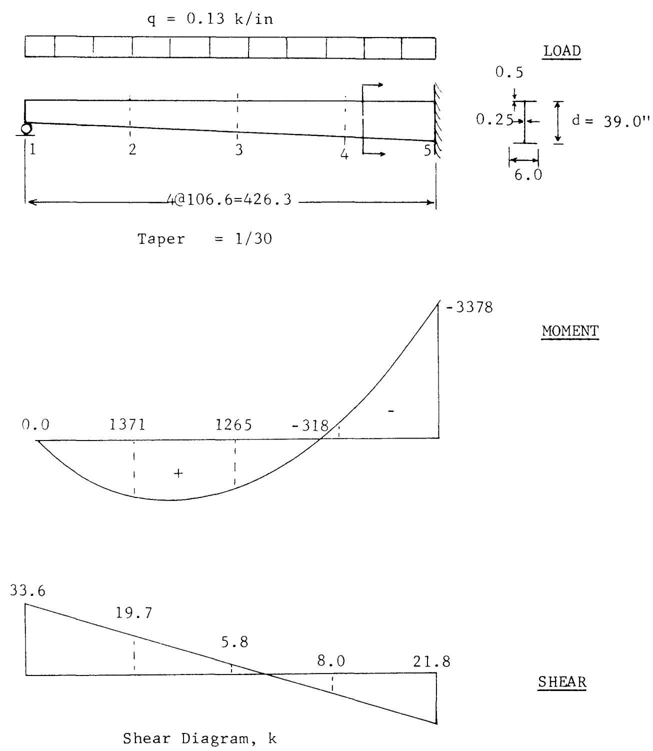

Fig. 4. Design of tapered propped cantilever

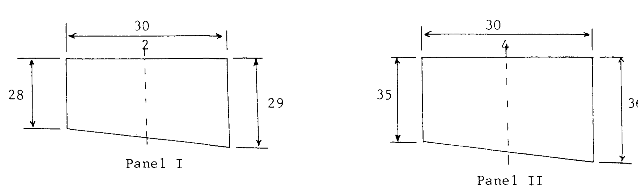

Fig. 5. Details of design panels

Panel I:

(a) Using Eq. (1a):

$h_{av} = \frac{1}{2}(28 + 29) = 28.5$ in.

$a/h_{av} = 30/28.5 = 1.05$

$h_{av}/t = 28.5/\frac{1}{4} = 114$

$$k = 5.34 + \frac{4.00}{(a/h_{av})^2} \text{ for } a/h > 1.0$$

$$= 5.34 + \frac{4.00}{(1.05)^2} = 8.97$$

$$C_v = \frac{45{,}000k}{F_y/(h_{av}/t)^2}$$

$$= \frac{(45{,}000)(8.97)}{(36)(114)^2} = 0.862 > 0.8 \quad \textbf{n.g.}$$

Use

$$C_v = \frac{190}{h_{av}/t}\sqrt{\frac{k}{F_y}}$$

$$= \frac{190}{114}\sqrt{\frac{8.97}{36}} = 0.832 > 0.8 \quad \textbf{o.k.}$$

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15}\frac{1}{\sqrt{1 + (a/h_{av})^2}}\right] \le 0.4F_y \quad \text{Eq. (1a)}$$

$$= \frac{36}{2.89}\left[0.83 + \frac{0.17}{1.15}\frac{1}{\sqrt{1 + 1.05^2}}\right]$$

$$= (12.46)(0.83 + 0.094)$$

$$= 11.51 \text{ ksi} < 14.4 \text{ ksi} \quad \textbf{o.k.}$$

(b) Using Eq. (19):

$a/h_1 = 30/29.0 = 1.034$

$h_0/t = 28/\frac{1}{4} = 112$

$$k = 5.34 + \frac{4.0}{(1.034)^2} = 9.08$$

$$C_{v0} = \frac{190}{(h_0/t)}\sqrt{\frac{k}{F_y}}$$

$$= \frac{190}{112}\sqrt{\frac{9.08}{36}} = 0.852 > 0.8 \quad \textbf{o.k.}$$

$$F_v = \frac{F_y}{2.89}\left[C_{v0} + \frac{1}{1.15}(1 - C_{v0})\frac{1}{\sqrt{1 + (a/h_1)^2}}\right] \quad \text{Eq. (19)}$$

$$= 12.46\left[0.852 + \frac{1}{1.15}(0.148)\frac{1}{\sqrt{1 + 1.034^2}}\right]$$

$$= 12.46\,[0.852 + 0.089] = 11.72 \text{ ksi}$$

Panel II:

(a) Using Eq. (1a):

$h_{av} = \frac{1}{2}(35 + 36) = 35.5$ in.

$a/h_{av} = 30/35.5 = 0.845$

$h_{av}/t = 35.5/\frac{1}{4} = 142.0$

$$k = 4.00 + \frac{5.34}{(a/h_{av})^2} \text{ for } a/h < 1.0$$

$$= 4.00 + \frac{5.34}{(0.845)^2} = 11.48$$

$$C_v = \frac{(45{,}000)(11.48)}{(36)(142^2)} = 0.712 < 0.8 \quad \textbf{o.k.}$$

Eq. (1a):

$$F_v = \frac{36.0}{2.89}\left[0.712 + \frac{0.288}{1.15}\frac{1}{\sqrt{1 + (0.845)^2}}\right]$$

$$= (12.46)(0.712 + 0.191) = 11.25 \text{ ksi}$$

(b) Using Eq. (17):

$a/h_0 = 30/35 = 0.857$

$h_1/t = 36/(\frac{1}{4}) = 144.0$

$$k = 4.00 + \frac{5.34}{(a/h_0)^2} \text{ for } a/h < 1.0$$

$$= 4.00 + \frac{5.34}{(0.857)^2} = 11.27$$

$$C_{v1} = \frac{(45{,}000)(k)}{(F_y)(h_1/t)^2}$$

$$= \frac{(45{,}000)(11.27)}{(36)(144^2)} = 0.68 < 0.8 \quad \textbf{o.k.}$$

Eq. (17):

$$F_v = \frac{F_y}{2.89}\left[C_{v1} + \frac{1}{1.15}(1 - C_{v1})\frac{1}{\sqrt{1 + \left(\frac{a}{h_0}\right)^2}}\right]$$

$$= (12.46)\left[0.68 + \left(\frac{0.32}{1.15}\right)(0.759)\right] = 11.09 \text{ ksi}$$

The allowable shear stresses calculated in Examples 1 and 2 by Eqs. (1a), (17), and (19) are compared in Table 2.

Table 2. Summary of Allowable Stresses

Example	Taper	Eq. (1): F_v (ksi)	Eq. (17) or (19): F_v (ksi)	Error (%)
1	1/14	7.35	7.10	+3.5
2 (Panel I)	1/30	11.51	11.72	−1.8
2 (Panel II)	1/30	11.25	11.09	+1.4

CONCLUSIONS

Beams and girders having small or moderate tapers comprise over 90% of tapered beams used in practice. The allowable shear stress of such beams may be obtained either by employing the present AISC provision based on tension-field action, with the average depth replacing the uniform depth, Eq. (1a), or it may be obtained by employing Eq. (17) or (19). The latter expressions will give results that are somewhat more accurate.

When the taper is more severe, approaching $\beta = 1/15$, the discrepancy between the two expressions may approach 5%.

For tapers larger than $\beta = 1/15$, Eq. (17) or (19) should suffice. When the tapers approach 1/8 (for example, tapers at haunched connections), an expression such as Eq. (21) should be used. Equation (21) is quite conservative, because the stress distribution on which it is based is a safe lower bound distribution.

A task for the immediate future is the determination of a better approximation to this stress distribution and the development of an expression similar to Eq. (21) for severe tapers, but less conservative, for application in design of haunched connections.

Other problems remain to be investigated in this connection. Two of these are: the location of stiffeners and the effects of other changes in dimensions (for instance, cover plates) on post-buckling stress.

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