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受剪杆件设计

王立军
(华诚博远工程技术集团有限公司, 北京 100055)

摘 要:对 AISC 360-16《建筑钢结构标准》(简称《美国钢标》)受剪杆件设计方法进行了解读,并与 GB 50017—2017《钢结构设计标准》(简称《17 钢标》)受剪杆件的设计方法进行了比较。

《美国钢标》受剪杆件稳定的强度能力计算在 G 章。设计抗剪强度取 $\phi_v V_n$,一般情况下抗剪抗力系数 $\phi_v=0.9$。

1)对于工字形截面和槽形截面,梁腹板受剪屈曲后会产生屈曲后强度,使腹板的抗剪能力提高。屈曲后强度来源之一为内力重分布,之二为腹板形成的拉力带作用。对不设置加劲肋及加劲肋间距 $a>3h$ 的腹板,只有内力重分布的作用。对于加劲肋间距 $a\leqslant 3h$ 的腹板,会存在两种作用。

2)不考虑腹板拉力带作用的抗剪强度,当 $h/t_w \leqslant 1.10\sqrt{k_v E/F_y}$,抗剪强度由腹板剪切屈服提供,此时腹板抗剪承载力系数 $C_{v1}=1.0$;当 $h/t_w > 1.10\sqrt{k_v E/F_y}$ 时,抗剪强度由腹板屈曲及屈曲后强度提供,此时 $C_{v1}<1.0$。

3)无加劲肋,腹板剪切屈曲系数 $k_v=5.34$,剪切屈服与屈曲的高厚比分界点 $h/t_w=1.10\sqrt{k_v E/F_y}=74\sqrt{235/F_y}$,因此腹板高厚比 $74\varepsilon_k$ 是屈服与屈曲的界限高厚比。

4)当腹板设置加劲肋且 $a/h\leqslant 3$ 时,拉力带将起作用。

对于梁的抗剪屈曲承载力计算,《17 钢标》于第 6.3.3 条给出了不考虑屈曲后强度的腹板剪切屈曲应力的计算公式,并于第 6.4 节给出了腹板考虑屈曲后强度的计算公式。

1)当不考虑屈曲后强度时,《17 钢标》式(6.3.3-8-12)给出了腹板剪切临界应力 τ_{cr} 与正则化宽厚比 $\lambda_{n,s}$ 的关系式。简支梁,取 $\eta=1.11$,则 $h_0/t_w=76\varepsilon_k$,此值与《美国钢标》的 $74\varepsilon_k$ 一致。《17 钢标》取 $h_0/t_w=80\varepsilon_k$ 为屈服与屈曲分界点,在 6.3.1 条指出,当 $h_0/t_w>80\varepsilon_k$,应计算梁腹板的屈曲稳定性。

2)当考虑屈曲后强度时,对于梁抗剪,《17 钢标》考虑腹板屈曲后拉力带的作用。

由分析可知:《美国钢标》考虑腹板屈曲后强度给出抗剪计算公式,《17 钢标》按考虑及不考虑腹板屈曲后强度给出抗剪计算公式。

关键词:受剪杆件; 屈曲后强度; 拉力带

1 AISC 360-16《美国建筑钢结构标准》[1]

AISC 360-16《美国建筑钢结构标准》[1](简称《美国钢标》)受剪杆件稳定的强度能力计算在 G 章。其设计抗剪强度取 $\phi_v V_n$,一般情况下抗剪抗力系数 $\phi_v=0.9$。

1.1 G2 工字形截面和槽形截面

梁腹板受剪屈曲后会产生屈曲后强度,使腹板的抗剪能力提高。屈曲后强度来源之一为内力重分布,之二为腹板形成的拉力带作用。对不设置加劲肋及加劲肋间距 $a>3h$ 的腹板,只有内力重分布的作用。对于加劲肋间距 $a\leqslant 3h$ 的腹板,会存在两种作用。

1)作用一:不考虑腹板拉力场作用的抗剪强度。

$$V_n = 0.6F_y A_w C_{v1} \tag{1}$$

作 者:王立军,男,1963 年出生,教授级高级工程师。
Email:13901212966@sina.com
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其中 $A_w = ht_w$

式中：A_w 为腹板截面积；h 为腹板高度；t_w 为腹板厚度；F_y 为屈服强度；C_{v1} 为腹板抗剪承载力系数。

考虑屈曲后应力重分布，抗剪强度会高于屈曲强度，因而抗剪强度采用屈服强度 $0.6F_yA_w$ 乘以 C_{v1} 表示。腹板部分受拉对其屈曲有利，这时不考虑腹板拉区的有利作用偏于安全并使计算简化。

a. 轧制工字形截面，腹板高度与厚度之比 $h/t_w \leqslant 2.24\sqrt{E/F_y} = 65\sqrt{235/F_y}$，此时，可取 $\phi_v = 1.0$，$C_{v1} = 1.0$。

b. 其他工字形截面和槽形截面：

$$C_{v1} = 1.0 \qquad h/t_w \leqslant 1.10\sqrt{k_vE/F_y} \tag{2a}$$

$$C_{v1} = \frac{1.10\sqrt{k_vE/F_y}}{h/t_w} \qquad h/t_w > 1.10\sqrt{k_vE/F_y} \tag{2b}$$

式中：k_v 为腹板剪切屈曲系数；E 为弹性模量。

腹板无横向加劲肋时：

$$k_v = 5.34 \tag{3a}$$

腹板有横向加劲肋时：

$$k_v = 5 + \frac{5}{(a/h)^2} \tag{3b}$$

式中：a 为加劲肋间距；当 $a/h>3.0$ 时，取 $k_v=5.34$。

2016 版《美国钢标》的抗剪承载力系数 C_{v1} 分成两段，见图 1 实线。2010 版《美国钢标》的抗剪承载力系数 C_{v1} 为三段式，见图 1 中虚线所示。

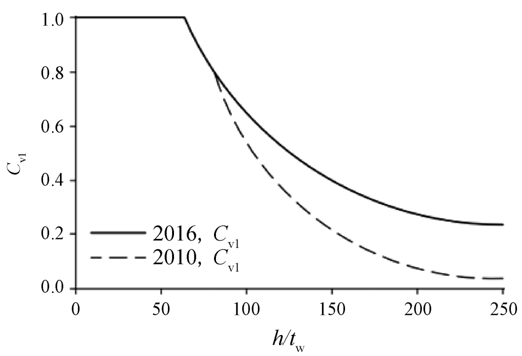


图 1 剪切屈曲系数（$F_y = 345$ MPa）

当 $h/t_w \leqslant 1.10\sqrt{k_vE/F_y}$ 时，抗剪强度由腹板剪切屈服提供，此时为式(2a) 即 $C_{v1} = 1.0$。$h/t_w > 1.10\sqrt{k_vE/F_y}$ 时，抗剪强度由腹板屈曲及屈曲后强度提供，此时为式(2b) 即 $C_{v1} < 1.0$。

式(3b)来自四边简支板受纯剪时的屈曲系数公式：

$$k_v = \begin{cases} 4 + \dfrac{5.34}{(a/h)^2} & \dfrac{a}{h} \leqslant 1 \\ 5.34 + \dfrac{4}{(a/h)^2} & \dfrac{a}{h} > 1 \end{cases} \tag{4}$$

式(4)可由式(3b)替代。

无加劲肋时，$k_v = 5.34$，剪切屈服与屈曲的高厚比分界点 $h/t_w = 1.10\sqrt{k_vE/F_y} = 74\sqrt{235/F_y}$，此时 $C_{v1} = 1.0$，抗剪强度由式(1) 确定，即腹板可达到剪应力屈服，因此腹板高厚比 $74\varepsilon_k$（$\varepsilon_k = \sqrt{235/F_y}$）是屈服与屈曲的界限高厚比。

2）作用二：腹板 $a/h \leqslant 3$ 时，考虑拉力带作用的腹板抗剪强度。

作用一为仅考虑腹板屈曲后内力重分布而不考虑其拉力场作用的抗剪强度的计算方法。当腹板设置加劲肋且 $a/h \leqslant 3$ 时，拉力带将起作用，此时腹板抗剪强度计算如下。

a. $h/t_w \leqslant 1.10\sqrt{k_vE/F_y}$ 时：

$$V_n = 0.6F_yA_w \tag{5a}$$

b. $h/t_w > 1.10\sqrt{k_vE/F_y}$ 时：

① $2A_w/(A_{fc} + A_{ft}) \leqslant 2.5$，$h/b_{fc} \leqslant 6.0$，$h/b_{ft} \leqslant 6.0$，则：

$$V_n = 0.6F_yA_w\left[C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1 + (a/h)^2}}\right] \tag{5b}$$

②其他：

$$V_n = 0.6F_yA_w\left\{C_{v2} + \frac{1 - C_{v2}}{1.15\left[\dfrac{a}{h} + \sqrt{1 + (a/h)^2}\right]}\right\} \tag{5c}$$

式中：A_{fc} 为受压翼缘面积；A_{ft} 为受拉翼缘面积；b_{fc} 为受压翼缘宽度；b_{ft} 为受拉翼缘宽度；C_{v2} 为腹板剪切屈服强度系数。

C_{v2} 由式(6)计算：

$h/t_w \leqslant 1.10\sqrt{k_vE/F_y}$ 时：

$$C_{v2} = 1.0 \tag{6a}$$

$1.10\sqrt{k_vE/F_y} < h/t_w \leqslant 1.37\sqrt{k_vE/F_y}$ 时：

$$C_{v2} = \frac{1.10\sqrt{k_vE/F_y}}{h/t_w} \tag{6b}$$

$h/t_w > 1.37\sqrt{k_vE/F_y}$ 时：

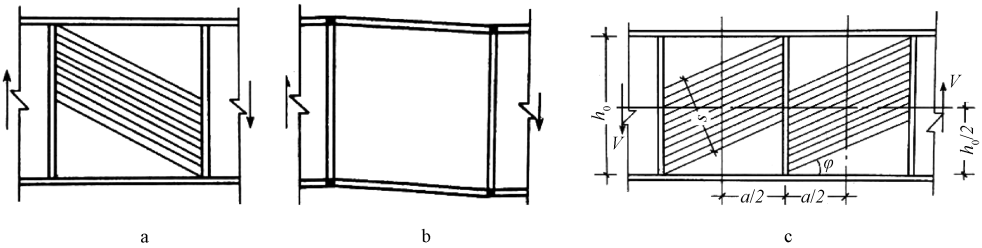
$$C_{v2} = \frac{1.51k_vE}{(h/t_w)^2F_y} \tag{6c}$$

腹板受上下翼缘和左右加劲肋约束，剪力作用下的抗剪承载力会超过剪切屈曲应力。形成的拉压

应力带与加劲肋形成类似桁架的作用。$a/h<3$ 时，屈曲后效应明显。式(5b)第一项为考虑屈曲后内力重分布的部分，第二项为拉力带对强度的提高部分。

考虑屈曲后强度时，抗剪能力和抗弯能力存在相互影响。当腹板相对于翼缘面积不是很大时，剪力对抗弯能力影响小，可以采用式(5b)考虑全拉力场作用。否则，只考虑部分拉力场，按式(5c)计算。

下面对式(5b)加以说明。



a—拉力带仅锚固于横向加劲肋上；b—塑性铰分布；c—拉力带示意。

图 2　受剪屈曲板拉力带

关于腹板屈曲后斜向拉力带的强度计算问题，需考虑拉力带的分布和是否再考虑加劲肋的框架作用两个层面，进一步的介绍可见文献[2]。下面仅考虑拉力带锚固于横向加劲肋形成的拉力带，且不考虑加劲肋的框架作用。

假定腹板受剪屈曲时应力为 τ_{cr}，屈曲后增加的强度由拉力带提供。考虑图 2a 的情况，拉力带仅锚固于横向加劲肋上，可将梁的翼缘、横向加劲肋和拉力带分别看成桁架的弦杆、竖杆和斜腹杆，对应的塑性铰为图 2b，即在框架四角出现塑性铰。如图 2c 所示，拉力带的宽度 s 为：

$$s = h_0\cos\varphi - a\sin\varphi \tag{7}$$

式中：φ 为拉力带倾角；h_0 为腹板有效高度。

竖向剪力为：

$$V_t = \sigma_t s t_w \sin\varphi \tag{8}$$

式中：σ_t 为腹板受剪屈曲后拉力带的拉应力。将式(7)代入式(8)，并令 $\dfrac{dV_t}{d\varphi}=0$，得到最大剪力对应的倾角为：

$$\sin 2\varphi = \frac{1}{\sqrt{1+\left(\dfrac{a}{h_0}\right)^2}} \tag{9}$$

将式(9)代入式(7)、式(8)，并经简化得到：

$$V_t = \frac{1}{2}\sigma_t t_w \frac{h_0}{\sqrt{1+(a/h_0)^2}} \tag{10}$$

拉应力 σ_t 对应的剪应力为 $\sigma_t/\sqrt{3}$，此项剪应力最大值为 $f_{vy}-\tau_{cr}$，即：

$$\sigma_t = \sqrt{3}(f_{vy}-\tau_{cr}) \tag{11}$$

式中：f_{vy} 为屈服剪应力；τ_{cr} 为临界剪应力。

将式(11)代入式(10)，得到：

$$V_t = \frac{\sqrt{3}}{2}h_0 t_w \frac{f_{vy}-\tau_{cr}}{\sqrt{1+(a/h_0)^2}} \tag{12}$$

式(12)为腹板屈曲后由拉力带提供的附加剪力，屈曲时的剪力为 $\tau_{cr}h_0t_w$，则屈曲后总剪力为：

$$V_n = \tau_{cr}h_0t_w + \frac{\sqrt{3}}{2}h_0 t_w \frac{f_{vy}-\tau_{cr}}{\sqrt{1+(a/h_0)^2}} \tag{13}$$

令 $A_w=h_0t_w$，$f_{vy}=0.6F_y$，$h_0=h$，考虑屈曲后内力重分布作用，屈曲应力由 τ_{cr} 提高到 $C_{v2}f_{vy}$，则：

$$V_n = 0.6F_yA_w\left[C_{v2}+\frac{1-C_{v2}}{1.15\sqrt{1+(a/h)^2}}\right] \tag{14}$$

式(14)即为《美国钢标》的式(5b)。

1.2　G4 矩形管

$$V_n = 0.6F_yA_wC_{v2} \tag{15}$$

1.3　G5 圆管

$$V_n = F_{cr}A_g/2 \tag{16}$$

式中：A_g 为截面积。

F_{cr} 取下列两式较大值且不超过 $0.6F_y$：

$$F_{cr} = \frac{1.60E}{\sqrt{\dfrac{L_v}{D}}\left(\dfrac{D}{t}\right)^{\frac{5}{4}}} \tag{17a}$$

$$F_{cr} = \frac{0.78E}{(D/t)^{\frac{3}{2}}} \tag{17b}$$

式中：D 为外径；t 为壁厚；L_v 为自剪力最大至零剪力的长度。

一般来说，式(17)对于 $D/t\geqslant100$ 的钢管、高强钢钢管和长钢管起控制作用。对于通常截面，屈服剪应力起控制作用，这时 $F_{cr}=0.6F_y$。

1.4　G6 双轴对称和单轴对称截面弱轴抗剪

荷载作用在弱轴且无扭转，对于每一个抗剪单

元，抗剪强度为：

$$V_n = 0.6F_y b_f t_f C_{v2} \tag{18}$$

式中：C_{v2} 为腹板剪切屈曲强度系数，由 G2 计算。对于工字形截面和 T 形截面，取 $h/t_w = b_f/(2t_f)$；对于槽形截面，取 $h/t_w = b_f/t_f$；$k_v = 1.2$；b_f 为翼缘宽度；t_f 为翼缘厚度。

2 GB 50017—2017《钢结构设计标准》[3]

对于梁的抗剪屈曲承载力计算，GB 50017—2017《钢结构设计标准》[3]（简称《17 钢标》）于第 6.3.3 条给出了不考虑屈曲后强度的腹板剪切屈曲应力的计算公式，并于第 6.4 节给出了腹板考虑屈曲后强度的计算公式。

2.1 不考虑屈曲后强度的腹板剪切屈曲计算

《17 钢标》式(6.3.3-8-12)给出了腹板剪切临界应力 τ_{cr} 与正则化宽厚比 $\lambda_{n,s}$ 的关系式，现列于如下。

$$\tau_{cr} = f_v \qquad \lambda_{n,s} \leqslant 0.8 \tag{19a}$$

$$\tau_{cr} = [1 - 0.59(\lambda_{n,s} - 0.8)]f_v \qquad 0.8 < \lambda_{n,s} \leqslant 1.2 \tag{19b}$$

$$\tau_{cr} = \frac{1.1f_v}{\lambda_{n,s}^2} \qquad \lambda_{n,s} > 1.2 \tag{19c}$$

式中：f_v 为抗剪屈服强度设计值。

正则化宽厚比 $\lambda_{n,s}$ 按下式计算：

$$\lambda_{n,s} = \frac{h_0/t_w}{37\eta\sqrt{4+5.34(h_0/a)^2}} \cdot \frac{1}{\varepsilon_k} \qquad a/h_0 \leqslant 1.0 \tag{20a}$$

$$\lambda_{n,s} = \frac{h_0/t_w}{37\eta\sqrt{5.34+4(h_0/a)^2}} \cdot \frac{1}{\varepsilon_k} \qquad a/h_0 > 1.0 \tag{20b}$$

式中：η 为参数，简支梁取 1.11，框架梁梁端最大应力区取 1；ε_k 为钢号修正系数。

可见剪切屈曲方程为三段式，分别对应塑性区、弹塑性区和弹性区。屈服与弹塑性屈曲分界点为 $\lambda_{n,b} = 0.8$，式(20b) 取 a/h_0 很大时，相当于无横向加劲肋情况，得到：

$$\lambda_{n,s} = \frac{h_0/t_w}{37\eta\sqrt{5.34}} \frac{1}{\varepsilon_k} \tag{21}$$

对于简支梁，取 $\eta = 1.11$，则 $h_0/t_w = 76\varepsilon_k$，此值与《美国钢标》的 $74\varepsilon_k$ 一致。

《17 钢标》取 $h_0/t_w = 80\varepsilon_k$ 为屈服与屈曲分界点，并在 6.3.1 条指出，当 $h_0/t_w > 80\varepsilon_k$ 时，应计算梁腹板的屈曲稳定性。

《17 钢标》式(19c) 为剪切屈曲临界应力计算公式，未考虑屈曲后强度。与仅考虑屈曲后内力重分布，不考虑拉力带作用的《美国钢标》式(2b)对比可知，《美国钢标》的公式也采用了临界应力的形式，只是将屈曲曲线抬高了。

2.2 考虑屈曲后强度的腹板剪切屈曲计算

对于梁抗剪，《17 钢标》考虑腹板屈曲后拉力带的作用，采用与 2.1 节类似的公式得到表达式如下：

$$V_u = h_w t_w f_v \qquad \lambda_{n,s} \leqslant 0.8 \tag{22a}$$

$$V_u = h_w t_w f_v[1 - 0.5(\lambda_{n,s} - 0.8)] \qquad 0.8 < \lambda_{n,s} \leqslant 1.2 \tag{22b}$$

$$V_u = h_w t_w f_v/\lambda_{n,s}^{1.2} \qquad \lambda_{n,s} > 1.2 \tag{22c}$$

图 3 为由式(22) 考虑屈曲后强度的极限剪应力与式(18)考虑屈曲临界剪应力的比较。可见，弹性段($\lambda_{n,s} > 1.2$) 两者差异大，进入塑性($\lambda_{n,s} \leqslant 1.2$) 后，屈曲后强度作用明显减小。

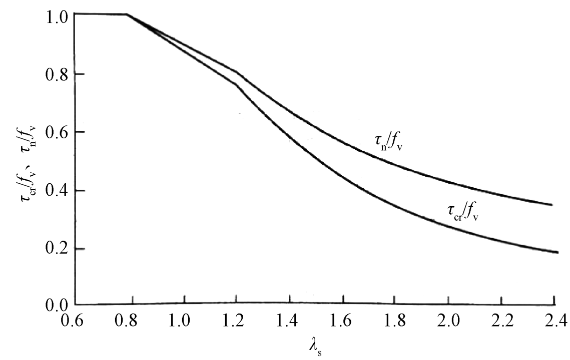


图 3 极限剪应力与临界剪应力比较

3 结束语

1)《美国钢标》杆件的抗剪计算考虑腹板的屈曲后强度。当腹板区隔 $a/h \leqslant 3$ 时，进而可以考虑拉力场对腹板抗剪强度的提高作用。

2)《17 钢标》第 6.3.3 条给出了腹板剪切临界应力的计算公式，第 6.4 节给出了考虑腹板屈曲后强度的抗剪计算公式。

3)《17 钢标》以梁腹板高厚比 $h_0/t_w = 80\varepsilon_k$ 为屈服与屈曲分界点，当 $h_0/t_w > 80\varepsilon_k$ 时需设置横向加劲肋以抵抗剪力屈曲。该分界点与《美国钢标》取值基本一致。

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Design of Members for Shear

Lijun Wang *

(*Huachengboyuan Engineering Technology Group Co.*, *Ltd.*, *Beijing* 100055, *China*)

Abstract:

The design method of members for shear in *Specification for Structural Steel Buildings* (AISC 360-16) is introduced and compared with that in *Standard for Design of Steel Structures* (GB 50017—2017).

The strength calculation of shear members is listed in Chapter G of AISC 360-16, in which the design shear strength takes $\phi_v V_n$, and the shear resistance coefficient $\phi_v = 0.9$.

1) For I-shape and channel sections, post-buckling strength after shear buckling can improve the shear capacity of the web, which is induced by the internal force redistribution and tension field in the web. For the web without stiffeners or with the spacing between stiffeners $a>3h$, post-buckling strength is only provided by internal force redistribution. However, for the web with the spacing between stiffeners $a\leqslant 3h$, both of them are contributors.

2) When shear strength without considering tension field of webs, $h/t_w \leqslant 1.10\sqrt{k_v E/F_y}$: shear strength is supported by shear yielding of the web, namely $C_{v1} = 1.0$; $h/t_w > 1.10\sqrt{k_v E/F_y}$: shear strength originates from both buckling of the web and its post-buckling strength, namely $C_{v1} < 1.0$.

3) If there is no stiffener, the shear buckling coefficient of the web $k_v = 5.34$, the cut-off point between height-to-thickness ratios under shear yielding and shear buckling is $h/t_w = 1.10\sqrt{k_v E/F_y} = 74\sqrt{235/F_y}$, so $74\varepsilon_k$ is the critical height-to-thickness ratios between yielding and buckling.

4) When the web has stiffeners and $a/h \leqslant 3$, tension field works and shear strength of the web should be calculated.

For the calculation of buckling bearing capacity of members under shear, GB 50017—2017 provides the formulas to calculate the shear buckling stress of the web in Article 6.3.3 without considering post-buckling strength, and presents the formulas considering post-buckling strength of the web in Chapter 6.4.

1) When shear buckling stress of the web without considering post-buckling strength, GB 50017—2017 (6.3.3-8-12) presents the relationship between critical shear stress τ_{cr} of web regardless of post-buckling strength and regularized width-to-thickness ratio $\lambda_{n,s}$. If $\eta = 1.11$ is adopted for simply supported beams, $h_0/t_w = 76\varepsilon_k$, consistent with the value $74\varepsilon_k$ in AISC 360-16. GB 50017—2017 takes $h_0/t_w = 80\varepsilon_k$ as the cut-off point between yielding and buckling, and points out in Article 6.3.1 that buckling stability of the web should be computed when $h_0/t_w > 80\varepsilon_k$.

2) When the shear buckling of the web is considered post-buckling strength, for the beam under shear, GB 50017—2017 takes into account the effect of tension field after buckling.

Analyses reveal that: AISC 360-16 presents the strength calculation of shear members considering the post-buckling strength of web plates, and GB 50017—2017 provides the formulas for the shear strength with and without post-buckling strength of the web respectively.

Key words: shear member; post-buckling strength; tension field

* Corresponding author
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1 AISC 360−16

The strength calculation of shear members is listed in Chapter G of *Specification for Structural and Building* (AISC 360−16)[1], in which the design shear strength is $\phi_v V_n$, and the shear resistance coefficient is $\phi_v = 0.9$ under normal circumstances.

1.1 G2 I-shape and channel sections

The post-buckling strength after shear buckling can improve the shear capacity of the web, which could be induced by internal force redistribution and tension field in the web. For the web without stiffeners or with the spacing between stiffeners $a>3h$, post-buckling strength is only provided by internal force redistribution. However, for the web with the spacing between stiffeners $a\leqslant 3h$, both internal force redistribution and tension field contribute to post-buckling strength.

1) Action 1: shear strength without considering tension field of the web.

$$V_n = 0.6F_y A_w C_{v1} \tag{1}$$

where $A_w = ht_w$ is the cross-sectional area of the web; h is the height of the web; t_w is the thickness of web; and F_y is the yielding strength; C_{v1} is web shear strength coefficient.

Considering the internal force redistribution after buckling, shear strength will be greater than buckling strength, so shear strength can be expressed by the product of buckling strength $0.6F_y A_w$ and web-shear strength coefficient C_{v1}. The tension field of a web is beneficial to its buckling strength, so it will be conservative and simple to ignore the beneficial effect of the tension field.

a. Rolled I-shape sections, the height-to-thickness ratio of the web is $h/t_w \leqslant 2.24\sqrt{E/F_y} = 65\sqrt{235/F_y}$, and $\phi_v = 1.0$, $C_{v1} = 1.0$.

b. Other I-shape and channel sections:

$$C_{v1} = 1.0 \qquad h/t_w \leqslant 1.10\sqrt{k_v E/F_y} \tag{2a}$$

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \qquad h/t_w > 1.10\sqrt{k_v E/F_y} \tag{2b}$$

where k_v is the shear buckling coefficient of the web; E is the elastic modulas.

For the web without stiffeners,

$$k_v = 5.34 \tag{3a}$$

For the web with stiffeners,

$$k_v = 5 + \frac{5}{(a/h)^2} \quad \left(\frac{a}{h} > 3.0, \text{取 } k_v = 5.34\right) \tag{3b}$$

where a is the spacing between stiffeners.

The shear strength coefficient C_{v1} is divided into two segments, as the solid line shown in Fig. 1. C_{v1} in the 2010 edition of AISC 360−16 is divided into three segments, as the dashed line in Fig. 1.

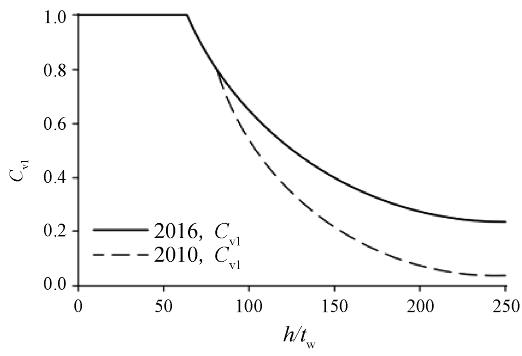


Fig. 1 Shear buckling coefficient (F_y = 345 MPa)

When $h/t_w \leqslant 1.10\sqrt{k_v E/F_y}$, shear strength is contributed by shear yielding of the web, and $C_{v1} = 1.0$ (Eq. (2a)). When $h/t_w > 1.10\sqrt{k_v E/F_y}$, it is supported by web buckling and its post-buckling strength, and $C_{v1} < 1.0$ (Eq. (2b)).

Eq. (3b) presents the buckling coefficient of four-edge simply-supported web under pure shear:

$$k_v=\begin{cases}4+\dfrac{5.34}{(a/h)^2} & \dfrac{a}{h}\leqslant 1\\ 5.34+\dfrac{4}{(a/h)^2} & \dfrac{a}{h}>1\end{cases} \tag{4}$$

The above two equations can be substituted by Eq. (3b).

When there is no stiffener, $k_v=5.34$, the cut-off point of height-to-thickness ratios between yielding and buckling of the web is $h/t_w=1.10\sqrt{k_vE/F_y}=74\sqrt{235/F_y}$ and shear strength can be determined by Eq. (1) with $C_{v1}=1.0$, namly that the web can reach shear stress yield. Thus $74\varepsilon_k$ is the critical height-to-thickness ratio between yielding and buckling of the web.

2) Action 2: when $a/h\leqslant 3$, the tension field should be considered in the calculation of shear strength of the web.

The above subsection 1) only discusses the internal force redistribution, while without the tension field, in the calculation of shear strength. When stiffeners are set on the web and the spacing between stiffeners $a\leqslant 3h$, tension field works and the shear strength of the web can be calculated by the following formulas:

a. $h/t_w\leqslant 1.10\sqrt{k_vE/F_y}$:

$$V_n=0.6F_yA_w \tag{5a}$$

b. $h/t_w>1.10\sqrt{k_vE/F_y}$:

① $2A_w/(A_{fc}+A_{ft})\leqslant 2.5, h/b_{fc}\leqslant 6.0, h/b_{ft}\leqslant 6.0$

$$V_n=0.6F_yA_w\left[C_{v2}+\frac{1-C_{v2}}{1.15\sqrt{1+\left(\dfrac{a}{h}\right)^2}}\right] \tag{5b}$$

②Others:

$$V_n=0.6F_yA_w\left\{C_{v2}+\frac{1-C_{v2}}{1.15\left[\dfrac{a}{h}+\sqrt{1+\left(\dfrac{a}{h}\right)^2}\right]}\right\} \tag{5c}$$

where A_{fc} is the area of compression flange; A_{ft} is the area of tension flange; b_{fc} is the width of compression flange; b_{ft} is the width of tension flange; C_{v2} is the shear buckling coefficient of the web.

C_{v2} can be determined by the following equations:

$h/t_w\leqslant 1.10\sqrt{k_vE/F_y}$

$$C_{v2}=1.0 \tag{6a}$$

$1.10\sqrt{k_vE/F_y}<h/t_w\leqslant 1.37\sqrt{k_vE/F_y}$

$$C_{v2}=\frac{1.10\sqrt{k_vE/F_y}}{h/t_w} \tag{6b}$$

$h/t_w>1.37\sqrt{k_vE/F_y}$

$$C_{v2}=\frac{1.51k_vE}{(h/t_w)^2F_y} \tag{6c}$$

Under the constraints of upper and lower flanges and left and right stiffeners, shear capacity of the web subject to shear will exceed the shear buckling stress. Tension and compression stress fields and stiffeners act like trusses. Post-buckling effect is obvious when $a/h<3$. The first term of Eq. (5b) indicates the internal force redistribution after buckling and the second term the contribution of tension field to the improvement of the strength.

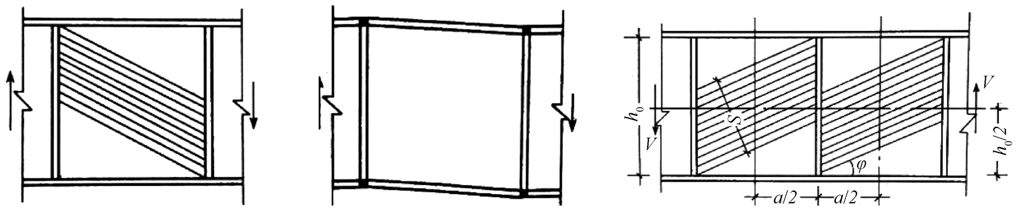
After the post-buckling strength is considered, shear strength and bending strength interact. When the web is not larger than the flange, shear force has little influence on bending capacity and Eq. (5b) can be used to explore the action of a full tension field. Otherwise, only part of the tension field should be considered, as Eq. (5c).

Eq. (5b) is illustrated as below:

With regard to the strength calculation in the oblique tension field after web buckling, one way is to consider the distribution of tension field and the other is to discuss both the tension field and the frame of stiffeners, detailed introduction of which can be found in Reference [2]. The following only involves the tension field formed by anchoring to the transverse stiffeners, without the frame of stiffeners.

The shear buckling stress of the web is set as τ_{cr}, and the increased strength after buckling is provided by the tension field. Considering the situation in Fig. 2(a), the tension field is only anchored to the transverse stiffeners, and the flange, transverse stiffeners and the tension field can be regarded as the chord, vertical bars and the oblique bar of trusses respectively. The corresponding plastic hinges are shown in Fig. 2(b), which are at the four corners of the frame. As Fig. 2(c), the width of tension the field is

$$s = h_0 \cos\varphi - a\sin\varphi \tag{7}$$



(a) Tension field is anchored only to the transverse stiffener (b) Plastic hinge distribution (c) Schematic diagram of tension field

Fig. 2 Tension field after shear buckling

where φ is the inclination angle of the tension field; h_0 is the effective height of the web.

The vertical shear is

$$V_t = \sigma_t s t_w \sin\varphi \tag{8}$$

where σ_t is the tensile stress in the tension field of the web after buckling under shear. Substituting Eq. (7) into Eq. (8) and assuming $\frac{dV_t}{d\varphi} = 0$, we can get the inclination angle corresponding to the maximum shear:

$$\sin 2\varphi = \frac{1}{\sqrt{1 + (a/h_0)^2}} \tag{9}$$

After Eq. (9) is substituted into Eq. (7) and Eq. (8), the following equation can be obtained after simplification:

$$V_t = \frac{1}{2}\sigma_t t_w \frac{h_0}{\sqrt{1 + (a/h_0)^2}} \tag{10}$$

The shear stress corresponding to tensile stress σ_t is $\sigma_t/\sqrt{3}$, with the maximum as $f_{vy} - \tau_{cr}$, namely

$$\sigma_t = \sqrt{3}(f_{vy} - \tau_{cr}) \tag{11}$$

where f_{vy} is the yield shear stress; τ_{cr} is the critical shear stress.

Eq. (11) is substituted into Eq. (10):

$$V_t = \frac{\sqrt{3}}{2} h_0 t_w \frac{f_{vy} - \tau_{cr}}{\sqrt{1 + (a/h_0)^2}} \tag{12}$$

Eq. (12) demonstrates the additional shear provided by the tension field after the web buckling. The buckling shear is $\tau_{cr} h_0 t_w$, so the total shear after buckling is:

$$V_n = \tau_{cr} h_0 t_w + \frac{\sqrt{3}}{2} h_0 t_w \frac{f_{vy} - \tau_{cr}}{\sqrt{1 + (a/h_0)^2}} \tag{13}$$

It is set that $A_w = h_0 t_w$, $f_{vy} = 0.6F_y$, $h_0 = h$. In light of internal force redistribution after buckling, buckling stress should be increased from τ_{cr} to $C_{v2} f_{vy}$. Then

$$V_n = 0.6 F_y A_w \left[C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1 + (a/h)^2}} \right] \tag{14}$$

Eq. (14) is Eq. (5b) in AISC 360-16.

1.2 G4 rectangular hollow structural sections

$$V_n = 0.6 F_y A_w C_{v2} \tag{15}$$

1.3 G5 circular hollow structural sections

$$V_n = F_{cr} A_g / 2 \tag{16}$$

where A_g is the cross-sectional area.

F_{cr} should take the greater value of the following two equations, but less than $0.6F_y$.

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \tag{17a}$$

$$F_{cr} = \frac{0.78E}{(D/t)^{\frac{3}{2}}} \tag{17b}$$

where D is the external diameter; t is the wall thickness; L_v is the length from the maximum shear to zero.

In general, Eq. (17) control the steel pipe, high-strength steel pipe and long steel pipe with $D/t \geqslant 100$. For normal cross sections, yield shear stress is decisive and $F_{cr} = 0.6F_y$.

1.4 G6 weak-axis shear of doubly-symmetric and singly-symmetric sections

When the loading is applied to the weak axis without torsion, shear strength of each shear element is

$$V_n = 0.6F_y b_f t_f C_{v2} \tag{18}$$

where C_{v2} is the shear buckling coefficient of the web and computed as Chapter G2. $h/t_w = b_f/(2t_f)$ for I-shaped and T-shaped sections, and $h/t_w = b_f/t_f$ for channel section, $k_v = 1.2$; b_f is the width of flange; t_f is the thickness of flange.

2 GB 50017—2017

Regarding the calculation of buckling bearing capacity of beams under shear, *Standard for Design of Steel Structures* (GB 50017—2017)[3] provides the formulas to calculate the shear buckling stress of the web in Article 6.3.3 without considering post-buckling strength. It also presents the formulas considering post-buckling strength of the web in Chapter 6.4.

2.1 Shear buckling of the web without considering post-buckling strength

GB 50017—2017 (6.3.3-8-12) presents the relation between critical shear stress τ_{cr} of the web without considering post-buckling strength and regularized width-to-thickness ratio $\lambda_{n,s}$, which is expressed as below:

$$\tau_{cf} = f_v \qquad \lambda_{n,s} \leqslant 0.8 \tag{19a}$$

$$\tau_{cr} = [1 - 0.59(\lambda_{n,s} - 0.8)]f_v \qquad 0.8 < \lambda_{n,s} \leqslant 1.2 \tag{19b}$$

$$\tau_{cr} = \frac{1.1f_v}{\lambda_{n,s}^2} \qquad \lambda_{n,s} > 1.2 \tag{19c}$$

Where f_v is the design value of shear yield strength.

The regularized width-to-thickness ratio $\lambda_{n,s}$ is determined by the following formulas:

$$\lambda_{n,s} = \frac{h_0/t_w}{37\eta\sqrt{4 + 5.34(h_0/a)^2}} \frac{1}{\varepsilon_k} \qquad a/h_0 \leqslant 1.0 \tag{20a}$$

$$\lambda_{n,s} = \frac{h_0/t_w}{37\eta\sqrt{5.34 + 4(h_0/a)^2}} \frac{1}{\varepsilon_k} \qquad a/h_0 > 1.0 \tag{20b}$$

Where η is the parameter, $\eta = 1.11$ for simply supported beams, $\eta = 1$ for maximum stress zone at beam end of frame beam; ε_k is the correction factor of the steel grade.

From the above, a shear buckling equation is divided into three segments, corresponding to a plastic zone, an elastoplastic zone and an elastic zone. The cut-off point between yielding and elastoplastic buckling is $\lambda_{n,b} = 0.8$, and if a/h_0 in Eq. (20b) is very large($a/h_0 > 3$), namely without transverse stiffeners, there is

$$\lambda_{n,s} = \frac{h_0/t_w}{37\eta\sqrt{5.34}} \frac{1}{\varepsilon_k} \tag{21}$$

If $\eta = 1.11$ is adopted for simply-supported beams, $h_0/t_w > 80\varepsilon_k$ is consistent with the value of $74\varepsilon_k$ in AISC 360-16.

GB 50017—2017 takes $h_0/t_w = 80\varepsilon_k$ as the cut-off point between yielding and buckling. Besides, it points out in Article 6.3.1 that buckling stability of the web should be computed when $h_0/t_w > 80\varepsilon_k$.

The critical shear buckling stress is calculated by Eq. (19c) in GB 50017—2017, without considering the post-buckling strength. Compared with Eq. (2b) in AISC 360-16, which only considers internal force redistribution after buckling, the formula in AISC 360-16 also adopts critical stress, but only raises the buckling curve.

2.2 Shear buckling of the web considering post-buckling strength

For the beam under shear, GB 50017—2017 takes into account the tension field after buckling and adopts the formulas similar to those in Section 2.1:

$$V_u = h_w t_w f_v \qquad \lambda_{n,s} \leqslant 0.8 \tag{22a}$$

$$V_u = h_w t_w f_v[1 - 0.5(\lambda_{n,s} - 0.8)] \qquad 0.8 < \lambda_{n,s} \leqslant 1.2 \tag{22b}$$

$$V_u = h_w t_w f_v / \lambda_{n,s}^{1.2} \qquad \lambda_{n,s} > 1.2 \quad (22c)$$

Fig. 3 presents the comparison between ultimate shear stress based on Eqs. (22) considering post-buckling strength and the critical shear stress based on Eqs. (19). The difference between them is significant in the elastic segment ($\lambda_{n,s}>1.2$), but post-buckling strength decreases obviously in the plastic segment ($\lambda_{n,s}\leqslant 1.2$).

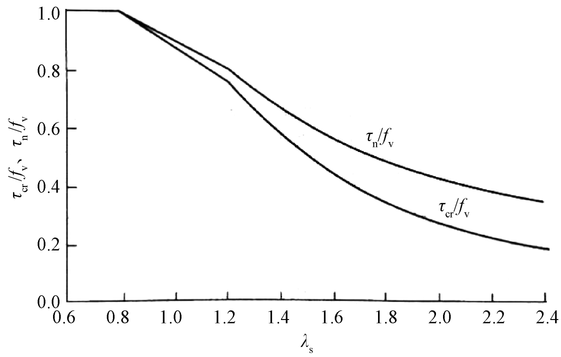


Fig. 3 Comparison between ultimate and critical shear stress

3 Conclusions

1) The shear strength calculation of members in AISC 360−16 involves the post-buckling strength of the web. When the spacing between stiffeners $a/h \leqslant 3$, the improvement of shear strength by the tension field can be discussed.

2) GB 50017—2017 provides the formulas for the critical shear stress of webs in Article 6.3.3 and those for shear strength considering post-buckling strength of the web in Chapter 6.4.

3) GB 50017—2017 takes the height-to-thickness ratio of webs $h_0/t_w = 80\varepsilon_k$ as the cut-off point between yielding and buckling, and when $h_0/t_w > 80\varepsilon_k$, transverse stiffeners are required to resist shear buckling. This cut-off point is consistent with that in AISC 360−16.

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