specwise

SHEER IMPROVEMENT TO SHEAR DESIGN

A look at member shear strength in the 2016 AISC *Specification*.

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CHAPTER G of the 2016 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-16)—Design of Members for Shear—has seen significant improvements thanks to research conducted over the last decade.

The improvements specifically pertain to built-up I-shaped members with webs that are thin enough to undergo shear buckling. Such members are used as columns and rafters in metal building systems and as transfer girders and other heavy members in conventional buildings.

Web shear buckling.

Thanks to the inclusion of post-buckling strength, the web shear strength of unstiffened built-up I-shapes with moderate to high web slenderness is much higher in the 2016 *Specification*. Additionally, the shear strength provisions for stiffened webs have been improved by including an equation that applies to members with small flanges, thus eliminating two of the applicability limits from previous specifications. Finally, shear stiffener design provisions have been consolidated and clarified.

Strength of Webs without Tension Field Action

Let's take a closer look at some of the changes in the 2016 *Specification*. Section G2.1, Shear Strength of Webs without Tension Field Action, can be applied to any web panel, regardless of stiffener spacing, flange size and whether or not the panel is at the end of the member. Note that this section *does* include post-buckling strength, but not through the traditional tension field action. It is the go-to section for unstiffened members such as metal building system moment frame rafters, plate girders without closely-spaced shear stiffeners and end panels in plate girders with closely spaced stiffeners.

This section is significantly different from the 2010 *Specification*, which is based on the paper *Strength of Plate Girders in Shear* by Basler (1961). In Basler's model, web panels without fairly closely spaced transverse stiffeners have no post-buckling strength. However, this assumption is very conservative for webs with moderate to high web slenderness, h/t_w.

The Swedish researcher, Höglund, developed the Rotated Stress Field Theory, which predicts significant post-buckling strength regardless of the presence of stiffeners. His 1997 paper is the basis of the shear strength equations in *Eurocode 3* (CEN, 2006). Lee et al. (2008) also developed a method that includes post-buckling strength for members with widely-spaced stiffeners.

During MBMA- (Metal Buildings Manufacturers Association) and AISC-sponsored research, Daley et al. (2017) investigated the accuracy and feasibility of various shear strength prediction methods and determined that the simple equations in Höglund (1997) provided the best combination of accuracy, slight conservatism (allowing $\phi = 0.9$ to match most other parts of Chapter G), consistency and simplicity.

They converted Höglund's equations to the familiar product of the shear yield strength and web shear strength coefficient, C_v, and made slight adjustments, resulting in the 2016 *Specification* Section G2.1(b) provisions. The shear strength of members with low h/t_w is the shear yield strength. For members with moderate to high h/t_w, it is the buckling plus post-buckling strength. The main equations are repeated on the next page.

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$$V_n = 0.6F_y t_w d C_{v1} \qquad (Spec.\ \text{Eq. G2-1})$$

where

$$C_{v1} = 1.0 \text{ if } h/t_w \leq 1.1\sqrt{k_v E/F_{yw}} \qquad (Spec.\ \text{Eq. G2-3})$$

$$C_{v1} = \frac{1.1\sqrt{k_v E/F_{yw}}}{h/t_w} \text{ if } h/t_w > 1.1\sqrt{k_v E/F_{yw}} \qquad (Spec.\ \text{Eq. G2-4})$$

The web shear strength coefficient has been named C_{v1} to distinguish it from the traditional Basler-based C_v, called C_{v2} in the 2016 *Specification*, that is used in the rest of Chapter G.

Figure 1, a summary of comparisons of measured and predicted shear strengths, indicates that the 2016 method is much more accurate than the 2010 method.

Figure 2 is a comparison of 2010 and 2016 web shear strength coefficients for F_y = 50 ksi. It indicates that the methods provide equal strengths at low h/t_w, so the strength of standard hot-rolled shapes is unchanged. Daley et al. (2017) did not include shapes with low h/t_w, so no change was justified for those. Note that the 2016 *Specification* also retains the special case with ϕ = 1.0 that applies to almost all hot-rolled shapes. The plot also indicates that the strength of webs with moderate to high h/t_w is much higher in the 2016 *Specification*.

Strength of Webs with Tension Field Action

As the name indicates, Section G2.2, Shear Strength of Interior Web Panels with $a/h \leq 3$ Considering Tension Field Action, applies to interior panels of members with closely-spaced stiffeners. Section G2.2 is a substantial improvement, in that it provides an equation for members with small flanges, thus eliminating two of the 2010 *Specification* Section G3.1 limits.

The shear strength of members with low h/t_w is the shear yield strength. For moderate-to-high h/t_w, it is the buckling strength plus the post-buckling strength provided by tension field action. By this model, the web is subjected to pure shear until shear buckling occurs. After that, the compressive stress component is constant while the tensile stress component increases to resist additional applied shear until the ultimate strength is attained. The net vertical stress and web out-of-plane displacement would cause the flanges to move toward each other if not for the presence of the vertical stiffeners that restrain out-of-plane displacement of the web in their vicinities. The resulting behavior is similar to that of a Pratt truss, with tensile stresses in the web between the stiffeners and compressive stresses in the web near the stiffeners.

Note that Höglund (1997) also provides equations for additional post-buckling strength (above the post-buckling strength utilized in Section G2.1) due to the presence of closely-spaced stiffeners. Considering that Höglund's research is the basis of Section G2.1, it seems reasonable to use his approach in Section G2.2 also. However, by his method, additional shear strength is achieved through flange bending, which is a huge departure from traditional AISC shear strength calculation methods. White and Barker (2008) showed that Höglund's method was not more accurate than Basler's for stiffened members. For these reasons, Section G2.2 is based on Basler's methods.

The strength of members with typical flange-web proportions is computed using Equation G2-7, repeated below, which is the full tension field action strength. The coefficient, C_{v2}, is identical to C_v in the 2010 *Specification*, and is plotted in Figure 2.

$$V_n = 0.6F_{yw} t_w d \left[C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1+(a/h)^2}} \right] \qquad (Spec.\ \text{Eq. G2-7})$$

In the 2010 *Specification*, tension field action was disallowed for members with large web-to-flange area and width ratios (Notes G3.1(c) and (d)). In the 2016 *Specification* Section G2.2, the slightly reduced tension field action strength in Equation 2-8, repeated below, is used for members with larger ratios. Equation 2-8 sometimes predicts lower strengths than the Höglund-based equations in Section G2.1. In such cases, the shear strength is the maximum of the two values.

Figure 1: Accuracy of Predictions – 2010 *Specification* Buckling Strength vs 2016 *Specification* Post-Buckling Strength

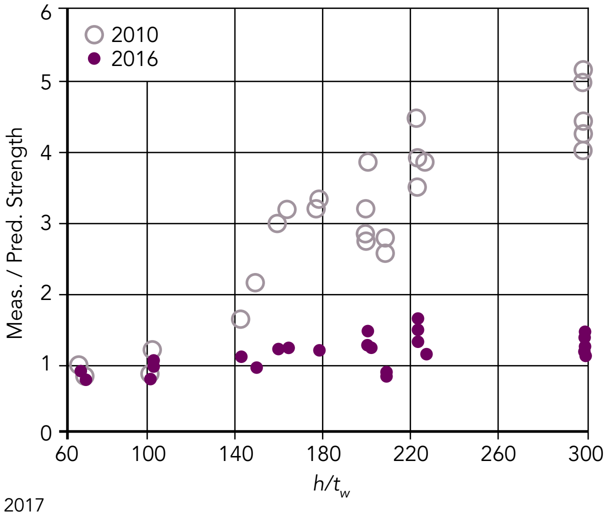
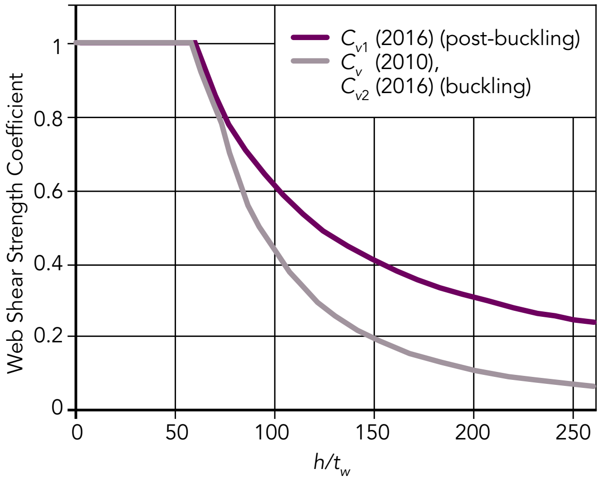


Figure 2: Comparison of Web Shear Coefficients



The bracketed parts of Equations G2-7 and G2-8 are plotted in Figure 3 for a/h = 3 and F_y = 50 ksi.

$$V_n = 0.6F_{yw}t_wd\left[C_{v2} + \frac{1-C_{v2}}{1.15\left[a/h+\sqrt{1+(a/h)^2}\right]}\right] \qquad (\textit{Spec.}\ \text{Eq. G2-8})$$

White and Barker (2008) studied the accuracy of various strength prediction model from the literature. Boxplots of the measured-to-predicted ratios are presented in Figure 4. The left-most entry is for the Basler method and the shear buckling coefficient from the 2010 and 2016 versions of the *Specification*. The Basler model performs better than most of the methods and approximately as well as the best performing ones.

Figure 3: Comparison of Equations G2-7 and G2-8

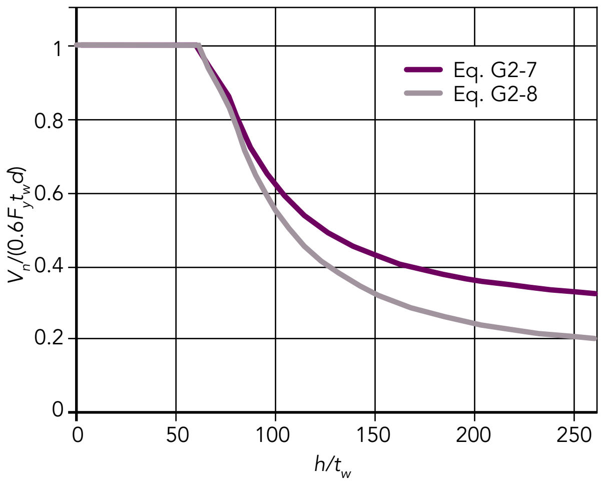
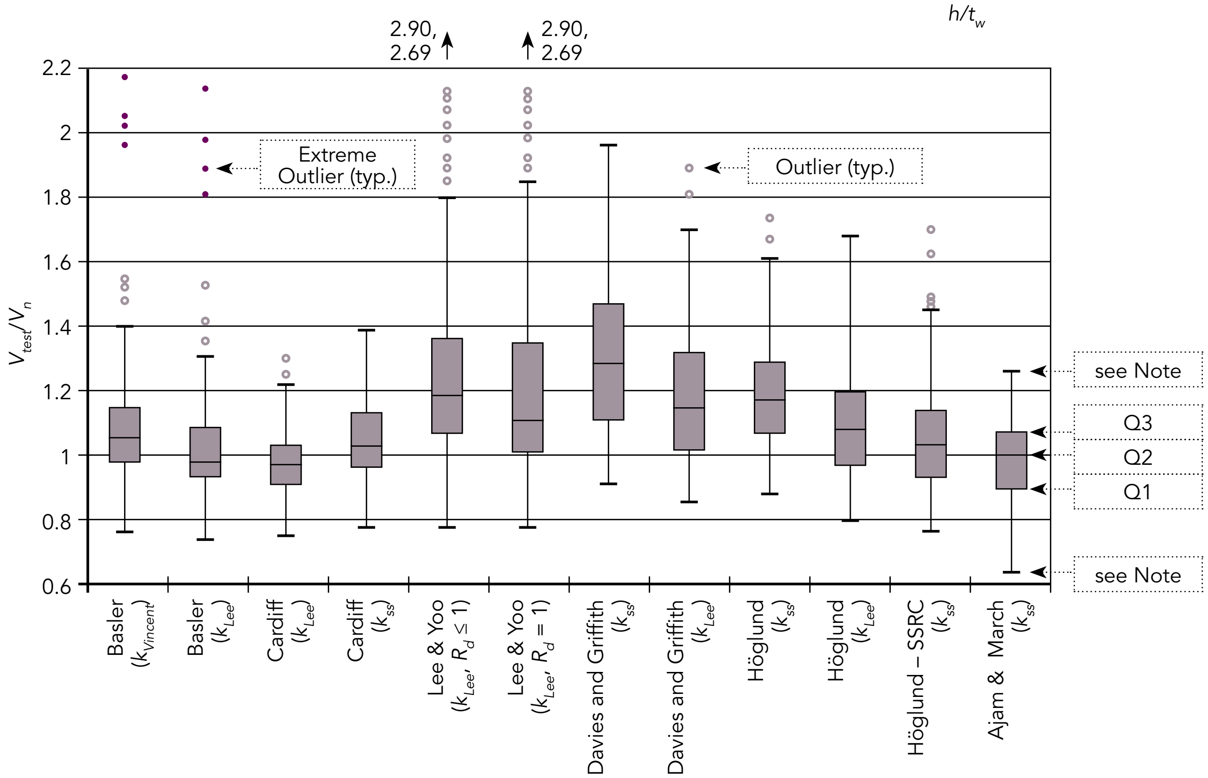


Figure 4: Accuracy of Predictions for Stiffened Panels (White and Barker, 2008)



Transverse Stiffener Design

The 2010 *Specification* Section G2.2 provided stiffener design criteria for webs designed on the basis of buckling. This section requires that the stiffener moment of inertia be sufficient to develop the web shear strength computed without tension field action. Section G3.3 provided criteria for webs designed on the basis of tension field action. It requires that the width-to-thickness ratio not exceed a limiting value and that the moment of inertia be sufficient to develop the required shear force. Here are a few changes:

- The 2016 *Specification* consolidates all shear stiffener design criteria into Section G2.3.
- Local buckling is evaluated using Equation G2-12. The limit is identical to the Table B4.1a Case 1 limit for flanges of I-shapes and similar unstiffened elements.
- The stiffener moment of inertia required to develop the member required shear, V_r, is linearly interpolated between: (i) I_{st2}, the moment of inertia required to develop the buckling strength, V_{c2}; and (ii) I_{st1}, the moment of inertia required to develop the post-buckling strength,

V_{c1}, from Section G2.1 or G2.2. This is illustrated in Figure 5.

- The 2016 *Specification* clarifies that the interpolation is performed separately for the panels on both sides of the stiffener.

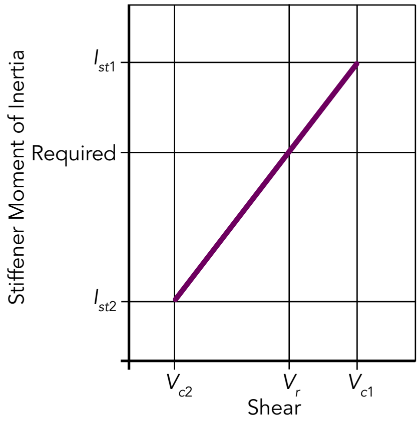


Figure 5:
Stiffener Required Moment of Inertia

Shear Strength of Angles and Other Members

The remaining sections, covering angles, tees, HSS and other members, are essentially identical to those in the 2010 *Specification*. Note that the Höglund-based provisions from Section G2.1 do not apply to these members, and the shear buckling strength is computed using the C_{v2} factor from Section G2.2.

References

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