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# Shear Strength of Unstiffened Steel Plate Girders

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**Abstract**
Ultimate shear strengths of twenty unstiffened and long web panel I-shaped plate girder specimens from the literature and seven new specimens are compared to predictions from models recommended by Basler, Höglund, and Lee and colleagues. Basler's method is shown to be accurate for members with stocky webs and very conservative for members with typical plate girder web slenderness ratios. Höglund's methods are slightly conservative. The method by Lee and colleagues is accurate on average, but significantly over-predicts the strength of several specimens. A proposed method, based on Höglund's 1997 research, produces slightly conservative predictions. Resistance factors are computed to facilitate potential inclusion in modern specifications. Basler's method and Höglund's 1997 method, which is a basis of the Eurocode provisions, can be used with a resistance factor of 1.0. Höglund's 1973 method and the proposed method can be used with a resistance factor of 0.9. The method by Lee and colleagues is less conservative; it can be used with a resistance factor of 0.75.

## 1. Introduction
Built-up I-section steel plate girders are used as metal building system rafters because they can be precisely optimized for least weight. They are also used commonly in bridges and as building girders. Unstiffened webs are used when they are less expensive than stiffened webs or when stiffeners are aesthetically unacceptable. Shear strength is a critical limit state for these members because they usually have thin webs.

Basler (1960, 1961) and Höglund (1971, 1973, 1997) proposed shear strength prediction methods for unstiffened webs, and Lee et al. (2008) proposed a method for webs with widely spaced stiffeners. However, these methods are mostly unverified. Thus, the primary objective of this research is to evaluate their accuracies. Another objective is to develop resistance factors to facilitate potential inclusion in modern design specifications. These objectives are accomplished by identifying prediction methods from the literature, collecting experimental data from the literature, testing additional specimens, and comparing measured and predicted ultimate strengths.

## 2. Strength Prediction Methods
Numerous authors have researched shear strength of plate girders with thin webs. The vast majority of the research has focused on the ultimate strength of short web panels bounded by transverse stiffeners at a spacing not exceeding three times the web depth. Plate girders with unstiffened webs are of primary interest of this study. The following web shear strength prediction methods are evaluated experimentally.

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*2.1 Basler's Method*

Basler's research (1960, 1961) is the basis of the plate girder shear strength evaluation methods in the American Institute of Steel Construction (AISC) *Specification for Structural Steel Buildings*, AISC 360-10 (AISC 2010) and American Association of State and Highway Transportation Officials *LRFD Bridge Design Specifications* (AASHTO 2014). Basler's fundamental assumptions are: (i) the web is subjected to a pure shear stress state up to the buckling shear and (ii) the principal compressive stress does not increase after shear buckling. Shear above the buckling strength is attained via a diagonal tension stress field that is equilibrated by transverse stiffeners in compression. Based on assumption (ii), it follows that unstiffened plate girders would have little to no post-buckling strength. Therefore, the shear strength of unstiffened webs is taken simply as the shear buckling strength. The elastic buckling stress, $\tau_e$, is computed using the classical plate buckling equation

$$\tau_e = \frac{k_v \pi^2 E}{12(1-\nu^2)(h/t_w)^2} \tag{1}$$

and the inelastic buckling stress, $\tau_{cr}$, is computed using

$$\tau_{cr} = \sqrt{0.8\tau_y \tau_e} \tag{2}$$

where:

- $k_v$ = plate buckling coefficient for shear
- $E$ = elastic modulus of steel
- $\nu$ = Poisson's ratio of steel
- $h$ = web depth
- $t_w$ = web thickness
- $\tau_y$ = shear yield stress, $F_y/\sqrt{3}$
- $F_y$ = uniaxial yield stress

The following plate buckling coefficient for a web with simply-supported flange-to-web connections was used originally in Basler method:

$$k_v = 4 + \frac{5.34}{(a/h)^2} \text{ if } a/h \le 1 \tag{3a}$$

$$k_v = 5.34 + \frac{4}{(a/h)^2} \text{ if } a/h > 1 \tag{3b}$$

The nominal strength, $V_n$, can be written as

$$V_n = (F_y/\sqrt{3})A_w C_v \tag{4}$$

where $A_w$ is $t_w h$ and $C_v$ is the ratio of buckling stress to shear stress, $\tau_e / \tau_y$, for elastic buckling and $\tau_{cr} / \tau_y$ for inelastic buckling. After simplification, and with $\nu$ = 0.3 included, $C_v$ is given by the following equations for stocky webs that fail by yielding, webs with intermediate slenderness which fail by inelastic buckling, and slender webs which fail by elastic buckling, respectively:

$$C_v = 1.0 \text{ if } h/t_w \le 1.12\sqrt{k_v E / F_y} \tag{5a}$$

$$C_v = \frac{1.12\sqrt{k_v E / F_y}}{h/t_w} \text{ if } 1.12\sqrt{k_v E / F_y} < h/t_w \le 1.40\sqrt{k_v E / F_y} \tag{5b}$$

$$C_v = \frac{1.57 k_v \mathrm{E}}{F_y (h/t_w)^2} \text{ if } h/t_w > 1.40\sqrt{k_v E / F_y} \tag{5c}$$

*2.2 Basler's Method with Plate Buckling Coefficient by Lee et al.*

Another method selected for this study is Basler's method with the following plate buckling coefficient developed by Lee et al. (1996).

$$k_v = k_{ss} + 0.8(k_{sf} - k_{ss})\left[1 - \frac{2}{3}\left(2 - \frac{t_f}{t_w}\right)\right] \text{ if } 0.5 < t_f / t_w \le 2 \tag{6a}$$

$$k_v = k_{ss} + 0.8(k_{sf} - k_{ss}) \text{ if } t_f / t_w > 2 \tag{6b}$$

where $k_{ss}$ is computed using Eqs. (3) and $k_{sf}$ is given by

$$k_{sf} = \frac{5.34}{(a/h)^2} + \frac{2.31}{a/h} - 3.44 + 8.39(a/h) \text{ if } a/h < 1 \tag{7a}$$

$$k_{sf} = 8.98 + \frac{5.61}{(a/h)^2} + \frac{1.99}{(a/h)^3} \text{ if } a/h > 1 \tag{7b}$$

*2.3 Höglund's 1973 Method*

Höglund's (1971, 1973) method predicts post-buckling strength even when transverse stiffeners are not installed. He reasoned the web is subjected to a pure shear stress state up to the elastic buckling stress, $\tau_e$, from Eq. (1) computed using the plate buckling coefficient from Eqs. (3). At buckling, the tensile and compressive principal stresses are $\sigma_1 = \tau_e$ and $\sigma_2 = -\tau_e$. After buckling, as shown in Fig. 1, $\sigma_1$ increases beyond $\tau_e$ while $\sigma_2$ remains equal to $-\tau_e$. However, the normal membrane stress component in the vertical direction must remain zero because there is little to no transverse restraint from the flanges. The principal stress angle rotates to simultaneously allow zero vertical membrane stress and $\sigma_1 > -\sigma_2$, hence the name Rotated Stress Field Theory. As shear increases, $\sigma_1$ increases until yielding is predicted by the von Mises yield criterion. The stress state at yielding has the horizontal component, $\sigma_h$, shown in Fig. 1. For the full strength to be developed, a rigid end post—acting like a vertical beam—must resist $\sigma_h$. Substantial post-buckling strength still develops without a rigid end post, however (Höglund 1973).

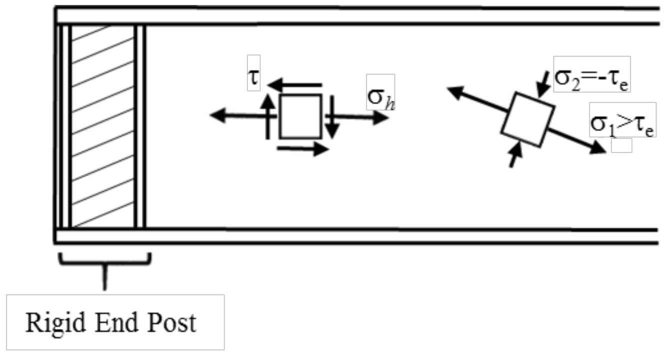


Fig. 1: Web stresses for rotated stress field theory

For members with non-rigid end posts, such as the ones in this study, the shear strength prediction equations recommended for stocky, intermediate, and slender webs, respectively, are:

$$V_n = \tau_y h t_w \text{ if } \lambda_w \le 0.8 \tag{8a}$$

$$V_n = \frac{1.8}{\lambda_w + 1}\tau_y h t_w \text{ if } 0.8 < \lambda_w \le 1.25 \tag{8b}$$

$$V_n = \frac{1}{\lambda_w}\tau_y h t_w \text{ if } \lambda_w > 1.25 \tag{8c}$$

where $\lambda_w = \sqrt{\tau_y / \tau_e}$

*2.4 Höglund's 1997 Method*

In 1997, Höglund recommended a modified version of his method, which is the basis of the Eurocode 3 (CEN 2006) shear strength provisions. This method predicts additional strength due to strain hardening of stocky webs with low-strength steel. The Höglund (1997) equations are slightly more conservative for non-stocky webs "to allow for scatter in test results as a result of initial imperfections and plastic buckling." For members with non-rigid end posts,

$$V_n = \eta F_{yw} h t_w \text{ if } \lambda_w < 0.48/\eta \tag{9a}$$

$$V_n = \frac{0.48}{\lambda_w} F_{yw} h t_w \text{ if } \lambda_w \ge 0.48/\eta \tag{9b}$$

where $\eta$ = 0.70 for steels with $F_{yw} \le 355$ MPa and 0.6 for higher strength steels.

*2.5 Lee et al. Method*

Lee and Yoo (1998) used FEA of hypothetical plate girders with $a/h \le 3$ to conclude post-buckling strength is approximately 40% of the difference between the buckling strength and plastic shear strength. They proposed the following nominal shear strength equation based on a synthesis of their FEA studies:

$$V_n = V_{cr} + V_{PB} = V_{cr} + 0.4(V_p - V_{cr}) = V_p(0.6C_v + 0.4) \tag{10}$$

where $V_{cr}$ is the shear buckling strength computed using the plate buckling coefficient from Eqs. (6). The plastic shear strength, $V_p = 0.58F_y t_w h$ and $C_v$ is computed using Eqs. (5).

Lee et al. (2008) extended this method to long web panels by comparing ultimate strengths from FEA test simulations on hypothetical plate girders with $3 \le a/h \le 6$ to predictions from their 1998 paper. They developed an adjustment factor, λ, and an initial imperfection adjustment factor, $R$ to bring the equations into agreement with the FEA. By their method, the predicted shear strength is

$$V_n = R\lambda V_p(0.6C_v + 0.4) \tag{11}$$

where

$$\lambda = 1.0 \text{ if } C_v \ge 0.3 \tag{12a}$$
$$\lambda = 1.35C_v + 0.6 \text{ if } 0.1 < C_v < 0.3 \tag{12b}$$
$$\lambda = 5.62C_v + 0.145 \text{ if } C_v \le 0.1 \tag{12c}$$

and

$$R = 1.0 - 0.2\frac{h/t_w\sqrt{F_y/(k_v E)}}{1.10} \text{ if } h/t_w < 1.1\sqrt{k_v E/F_y} \tag{13a}$$
$$R = 0.8 + 0.2\frac{h/t_w\sqrt{F_y/(k_v E)} - 1.10}{1.10} \text{ if } 1.1\sqrt{k_v E/F_y} \le h/t_w \le 2.2\sqrt{k_v E/F_y} \tag{13b}$$
$$R = 1 \text{ if } h/t_w > 2.2\sqrt{k_v E/F_y} \tag{13c}$$

## 3. Measured Strengths

*3.1 Specimens from the Literature*

Carksaddan (1968) reported experimental results for six hybrid plate girders with $a/h$ = 5.5 and $h/t_w$ ranging from 68.8 to 143. Fig. 2 shows the configuration of these four-point bending specimens.

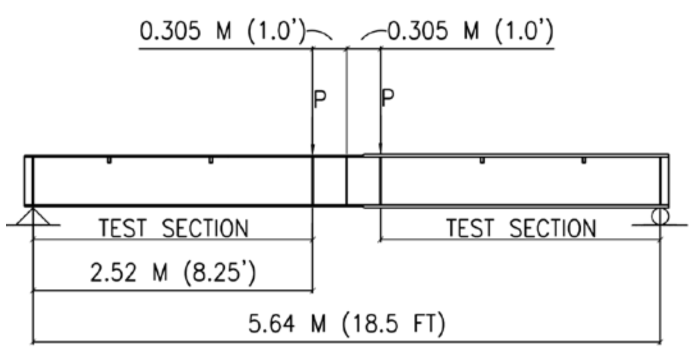


Fig. 2: Carskaddan specimen elevation

Höglund (1971) reported experimental results for three plate girders of the configuration shown in Fig. 3. Stiffeners were only installed at the ends. The web slenderness values $h/t_w$ were 209, 209, and 300.

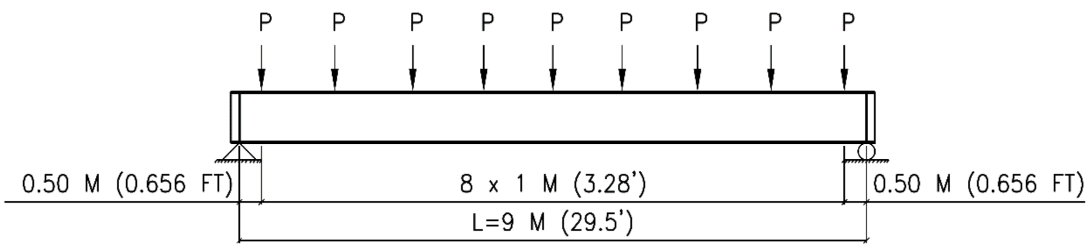


Fig. 3: Höglund specimen elevation

Frey and Anslijn (1977) tested four girders with $h/t_w$ equal to 200 and 300, in the configuration shown in Fig. 4. Stiffeners were only installed at the ends. Each girder was loaded to shear failure at the left end. Then, the failed end was reinforced, the girder was repositioned to load the other end in maximum shear, and the test was repeated, thus providing eight measured ultimate strengths.

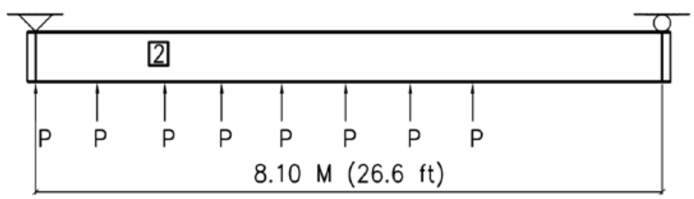


Fig. 4: Frey and Anslijn specimen elevation

Ravinger (1983) tested three unstiffened plate girders with $h/t_w$ = 224, of the configuration shown in Fig. 5.

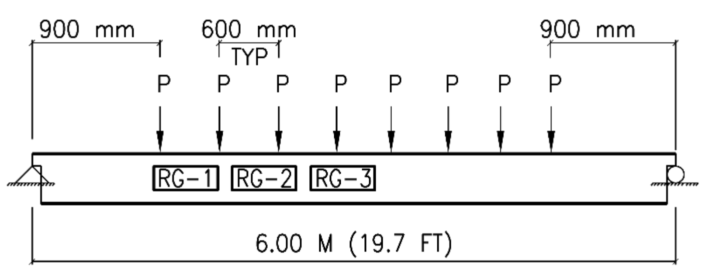


Fig. 5: Ravinger specimen elevation

Table 1 is a summary of these 20 specimens from the literature. The specimens cover the range of common plate girder web slenderness values, $h/t_w$, except there are no specimens with slenderness between 143 and 200. The Höglund, Frey and Anslijn, and Ravinger specimens have no interior stiffener, so $a/h$ is undefined and thus not listed in the table.

### *3.2 Specimens Tested at the University of Kentucky*

*Specimens and Test Setup*

Seven specimens of the configuration shown in Fig. 6 were tested at the University of Kentucky. Specimen dimensions, summarized in Table 2, were selected such that the specimens were expected to fail by shear buckling. Web depths and thicknesses were selected to obtain $h/t_w$ values between 150 and 200. The aspect ratio, $a/h$, exceeded the 3.0 demarcation between stiffened and unstiffened panels.

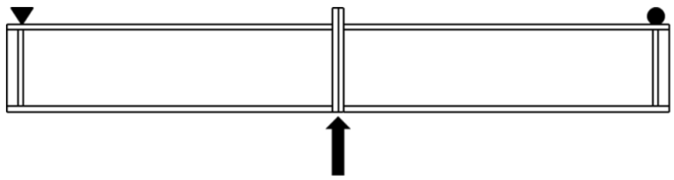


Fig. 6: University of Kentucky specimen elevation

Table 1: Specimens from the literature

| Specimen | $V_{meas}$ (kN) | $h$ (mm) | $t_w$ (mm) | $b_f$ (mm) | $t_f$ (mm) | $F_{yw}$ (MPa) | $F_{yf}$ (MPa) | $h/t_w$ | $a/h$ |
|---|---|---|---|---|---|---|---|---|---|
| Carskaddan C-AC1 | 178 | 454 | 4.47 | 128 | 7.92 | 232 | 786 | 102 | 5.50 |
| Carskaddan C-AC2 | 119 | 454 | 3.18 | 92.4 | 9.73 | 211 | 752 | 143 | 5.50 |
| Carskaddan C-AC3 | 397 | 455 | 6.45 | 140 | 13.0 | 252 | 745 | 70.6 | 5.50 |
| Carskaddan C-AC4 | 245 | 455 | 4.45 | 134 | 16.2 | 232 | 779 | 102 | 5.50 |
| Carskaddan C-AC5 | 233 | 456 | 4.45 | 132 | 19.0 | 232 | 786 | 103 | 5.50 |
| Carskaddan C-AH1 | 578 | 456 | 6.63 | 141 | 25.4 | 336 | 731 | 68.8 | 5.50 |
| Höglund B1 | 112 | 599 | 2.87 | 226 | 9.91 | 410 | 289 | 209 | - |
| Höglund K1 | 104 | 599 | 2.87 | 226 | 9.91 | 410 | 289 | 209 | - |
| Höglund B4 | 54.3 | 599 | 2.00 | 150 | 6.00 | 274 | 298 | 300 | - |
| Frey & Anslijn 1A | 145 | 599 | 3.00 | 225 | 10.0 | 239 | 246 | 200 | - |
| Frey & Anslijn 1B | 129 | 599 | 3.00 | 225 | 10.0 | 239 | 246 | 200 | - |
| Frey & Anslijn 2A | 129 | 599 | 3.00 | 225 | 10.0 | 239 | 246 | 200 | - |
| Frey & Anslijn 2B | 125 | 599 | 3.00 | 225 | 10.0 | 239 | 246 | 200 | - |
| Frey & Anslijn 3A | 57.4 | 599 | 2.00 | 150 | 5.99 | 286 | 281 | 300 | - |
| Frey & Anslijn 3B | 59.6 | 599 | 2.00 | 150 | 5.99 | 286 | 281 | 300 | - |
| Frey & Anslijn 4A | 69.4 | 599 | 2.00 | 150 | 5.99 | 286 | 281 | 300 | - |
| Frey & Anslijn 4B | 67.2 | 599 | 2.00 | 150 | 5.99 | 286 | 281 | 300 | - |
| Ravinger RG1 | 60.1 | 437 | 1.95 | 150 | 6.11 | 280 | 290 | 224 | - |
| Ravinger RG2 | 67.2 | 437 | 1.95 | 150 | 6.12 | 280 | 290 | 224 | - |
| Ravinger RG3 | 77.0 | 437 | 1.95 | 150 | 11.8 | 280 | 290 | 224 | - |

$V_{meas}$ = measured ultimate shear

$h$ = web depth

$t_w$ = web thickness

$b_f$ = flange width

$t_f$ = flange thickness

$F_{yw}$, $F_{yf}$ = web and flange yield stress, respectively

$a$ = clear distance between stiffeners

Table 2: University of Kentucky specimen properties

| Specimen | $L$ (m) | $h$ (mm) | $t_w$ (mm) | $b_f$ (mm) | $t_f$ (mm) | $F_{yw}$ (MPa) | $F_{yf}$ (MPa) | $h/t_w$ | $a/h$ |
|---|---|---|---|---|---|---|---|---|---|
| UK 1 | 3.66 | 476 | 3.18 | 152 | 15.9 | 423 | 385 | 150 | 5.50 |
| UK 2 | 5.49 | 546 | 3.40 | 203 | 15.9 | 448 | 378 | 160 | 4.97 |
| UK 3 | 5.49 | 610 | 3.40 | 203 | 12.7 | 448 | 367 | 179 | 4.45 |
| UK 4 | 7.32 | 686 | 3.40 | 203 | 15.9 | 461 | 379 | 201 | 5.29 |
| UK 5 | 8.84 | 559 | 3.15 | 203 | 19.1 | 416 | 404 | 177 | 7.85 |
| UK 6 | 8.23 | 991 | 4.37 | 203 | 19.1 | 434 | 406 | 227 | 4.12 |
| UK 7 | 8.23 | 508 | 3.10 | 203 | 19.1 | 441 | 379 | 164 | 8.04 |

$L$ = distance between supports
See Table 1 for other variable names.

*Experimental Results*

Each specimen was loaded incrementally until no additional load could be applied, and then additional displacement was induced to make the buckled shape more visible. Fig. 7 shows an example buckled shape, and the others are shown in Davis and Daley (2015). Table 3 summarizes the buckled web extents and measured ultimate shear strengths, and Fig. 8 is an example plot of shear versus mid-span deflection. Each specimen initially behaved nearly linearly and then gradually lost stiffness until the ultimate shear was reached. The web out-of-plane displacements were quite small until the shear approached the ultimate strength; however web out-of-plane movement was visible from the start of the loading. In each specimen, substantial shear strength in excess of the theoretical web buckling load was developed.

Table 3: University of Kentucky experiment summary

| Specimen | Extent of Web Buckling | $V_{meas}$ (kN) |
|---|---|---|
| UK1 | Almost all of half-span. | 148 |
| UK2 | 1.8 m long region near the support. | 217 |
| UK3 | 1.8 m long region, middle of half-span. | 219 |
| UK4 | 1.8 m long region near mid-span | 225 |
| UK5 | 1.8 m long region near mid-span | 182 |
| UK6 | Almost all of half-span | 327 |
| UK7 | 1.8 m long region near mid-span | 189 |

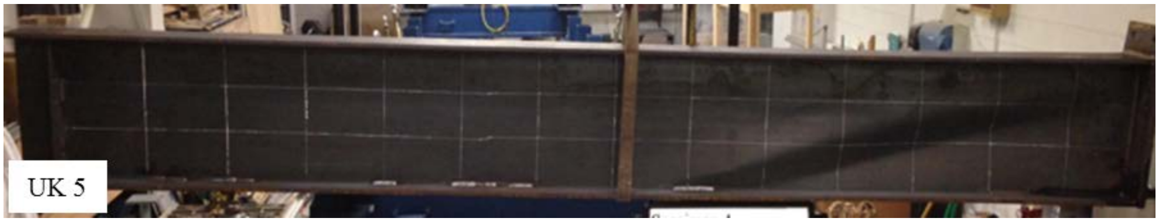


Fig. 7: Example failure mode (Specimen UK5)

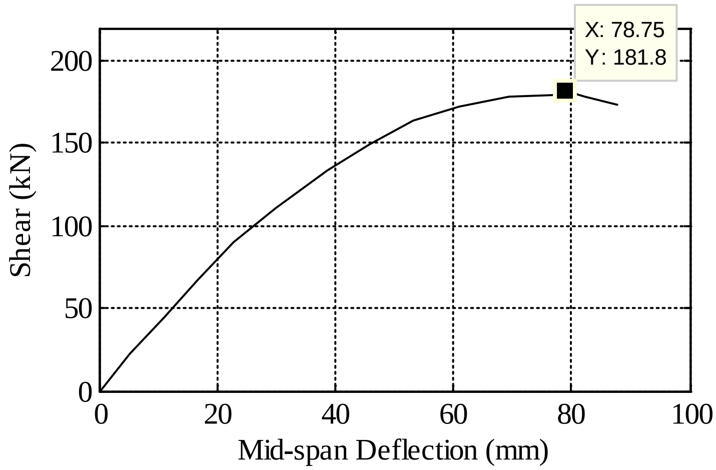


Fig. 8: Example shear-deflection plot (specimen UK5)

**4. Comparisons of Measurements and Predictions**

Table 4 lists the ratios of measured ultimate shear from Tables 1 and 3 to predicted shear strength from each method.

Measured-to-predicted shear strength ratios are plotted for the predictions based on the theoretical shear buckling equations from Basler, with different shear buckling coefficients, $k_v$, in Fig. 9. The plot indicates that these estimates are reasonably accurate for low $h/t_w$ (Carskaddan specimens). However, the average ratio is 3.72 for these predictions when $h/t_w \geq 150$. The average ratio is 2.38 using the theoretical shear buckling strength with the plate buckling coefficient by Lee et al. (1996) when $h/t_w \geq 150$.

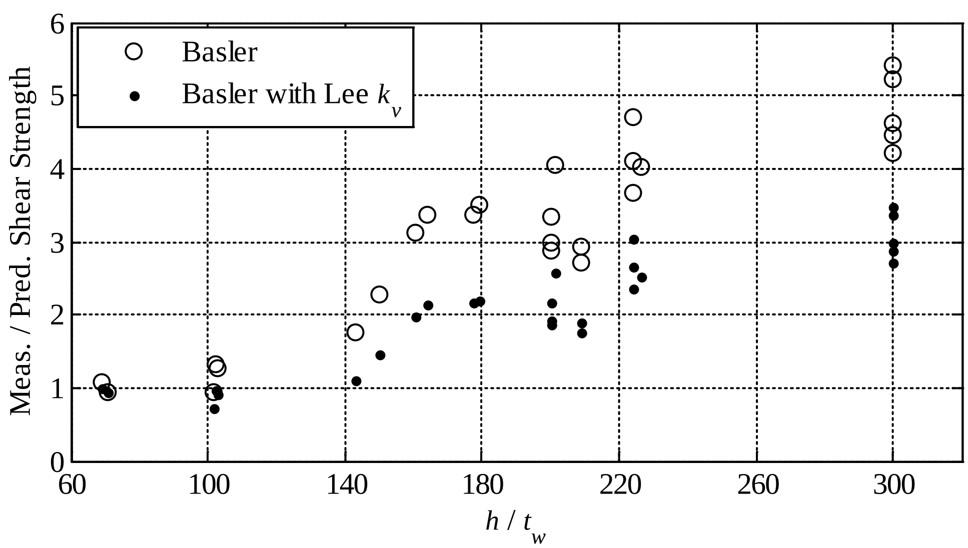


Fig. 9: Measured-to-predicted shear strength ratios using the theoretical web shear buckling equation from Basler with different shear buckling coefficients $k_v$.

Measured-to-predicted strength ratios are plotted for the Höglund methods in Fig. 10. Both variations are accurate and slightly conservative for almost all specimens and each has approximately the same level of conservatism for all $h/t_w$. The average ratio was 1.17 and 1.38 for the 1973 and 1997 method, respectively. It should be noted that none of the tests considered here evaluates the resistance of unstiffened webs that are sufficiently stocky such that the shear resistance is equal to or larger than the plastic shear resistance, $V_p$. However, several of Carskaddan's tests are close to the corresponding limit on $h/t_w$.

Table 4: Ratio of measured-to-predicted shear strength

| Specimen | Basler | Basler with Lee's $k_v$ | Höglund 1973 | Höglund 1997 | Lee et al. | Proposed |
|---|---|---|---|---|---|---|
| Carskaddan C-AC1 | 0.938 | 0.717 | 0.799 | 0.931 | 0.836 | 0.829 |
| Carskaddan C-AC2 | 1.75 | 1.11 | 1.09 | 1.29 | 0.984 | 1.14 |
| Carskaddan C-AC3 | 0.930 | 0.930 | 0.964 | 0.957 | 1.10 | 0.846 |
| Carskaddan C-AC4 | 1.32 | 0.970 | 1.11 | 1.30 | 1.15 | 1.11 |
| Carskaddan C-AC5 | 1.26 | 0.925 | 1.05 | 1.23 | 1.09 | 1.05 |
| Carskaddan C-AH1 | 1.07 | 0.985 | 1.08 | 1.14 | 1.19 | 0.946 |
| Höglund B1 | 2.94 | 1.89 | 0.897 | 1.08 | 0.704 | 0.951 |
| Höglund K1 | 2.73 | 1.76 | 0.833 | 1.00 | 0.654 | 0.882 |
| Höglund B4 | 4.22 | 2.73 | 1.10 | 1.32 | 0.828 | 1.18 |
| Frey & Anslijn 1A | 3.35 | 2.16 | 1.40 | 1.68 | 1.07 | 1.48 |
| Frey & Anslijn 1B | 2.99 | 1.93 | 1.25 | 1.50 | 0.955 | 1.32 |
| Frey & Anslijn 2A | 2.99 | 1.93 | 1.25 | 1.50 | 0.955 | 1.32 |
| Frey & Anslijn 2B | 2.88 | 1.85 | 1.20 | 1.44 | 0.919 | 1.27 |
| Frey & Anslijn 3A | 4.47 | 2.88 | 1.14 | 1.37 | 0.851 | 1.22 |
| Frey & Anslijn 3B | 4.64 | 2.99 | 1.18 | 1.42 | 0.884 | 1.27 |
| Frey & Anslijn 4A | 5.4 | 3.48 | 1.38 | 1.65 | 1.03 | 1.48 |
| Frey & Anslijn 4B | 5.23 | 3.37 | 1.33 | 1.60 | 0.996 | 1.43 |
| Ravinger RG1 | 3.67 | 2.37 | 1.26 | 1.52 | 1.00 | 1.35 |
| Ravinger RG2 | 4.10 | 2.65 | 1.41 | 1.70 | 1.12 | 1.51 |
| Ravinger RG3 | 4.70 | 3.03 | 1.62 | 1.95 | 1.28 | 1.68 |
| UK 1 | 2.29 | 1.45 | 0.959 | 1.14 | 0.724 | 0.984 |
| UK 2 | 3.12 | 1.97 | 1.19 | 1.41 | 0.918 | 1.23 |
| UK 3 | 3.51 | 2.21 | 1.20 | 1.41 | 0.933 | 1.26 |
| UK 4 | 4.05 | 2.57 | 1.21 | 1.44 | 0.938 | 1.27 |
| UK 5 | 3.37 | 2.16 | 1.20 | 1.44 | 0.946 | 1.23 |
| UK 6 | 4.03 | 2.52 | 1.10 | 1.30 | 0.834 | 1.16 |
| UK 7 | 3.35 | 2.15 | 1.26 | 1.50 | 0.983 | 1.28 |
| Average: | 3.16 | 2.06 | 1.17 | 1.38 | 0.958 | 1.21 |
| COV: | 0.412 | 0.378 | 0.157 | 0.174 | 0.153 | 0.176 |

The Lee et al. (2008) method was derived for members with $3 \leq a/h \leq 6$, so it applies to the six Carskaddan specimens and UK1 through UK4, and UK6. For those eleven specimens, the average measured-to-predicted strength ratio was 0.972 with 15.0% COV. Two predictions were significantly unconservative (Carskaddan C-AC1 Meas./Pred. = 0.836 and UK1 Meas./Pred. = 0.724). The average ratio for the other specimens—with $a/h > 6$—was 0.948 with 15.9% COV. Thus, the method was almost as accurate for specimens with $a/h > 6$ as for specimens with $3 \leq a/h \leq 6$. The predictions were significantly unconservative for the three Höglund tests. Ratios for all specimens are plotted for the Lee et al. (2008) method in Fig. 11, indicating the method is accurate on average and has approximately the same level of conservatism for all $h/t_w$, but produced significantly unconservative predictions for a few specimens.

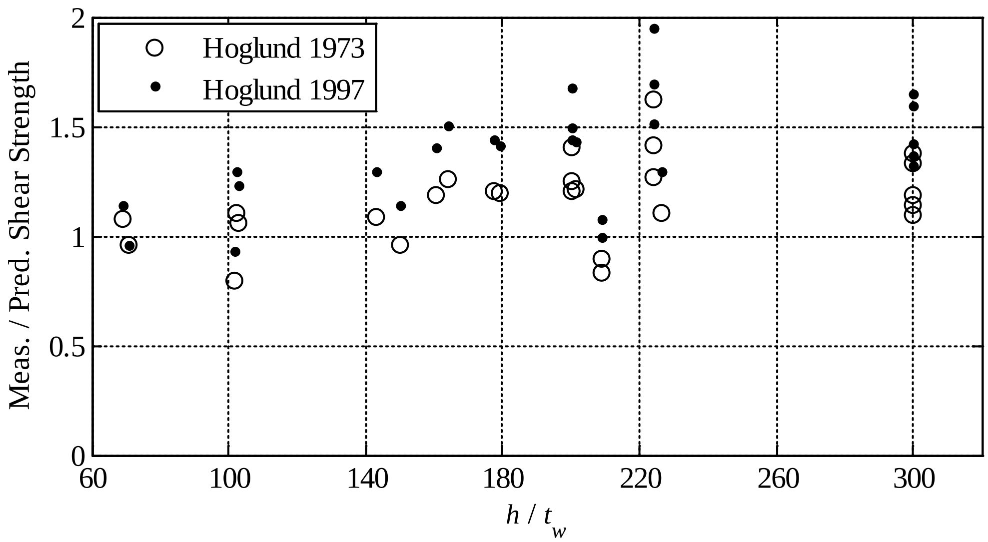


Fig. 10: Measured-to-predicted shear strength ratios for Höglund methods

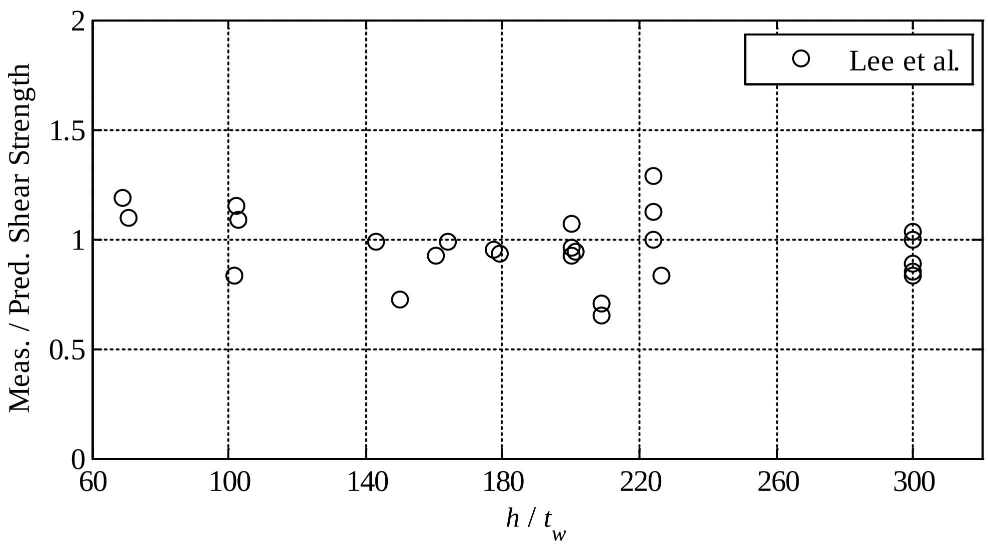


Fig. 11: Measured-to-predicted shear strength ratios for the Lee et al. method

## 5. Proposed Method

A strength prediction method, based on Höglund's 1997 method for non-rigid end posts, was proposed by the authors, vetted by the AISC Specification Committee, and adopted into the AISC 360-16 Specification (AISC, 2015). The method uses the familiar AISC $C_v$ formulation with slenderness expressed as $h/t_w$. It corresponds closely with the theoretical inelastic shear buckling strength prediction based on Basler's recommendations for I-section members with low $h/t_w$. In the proposed method, Eqs. (9) are simplified by using $\eta$ = 0.60 for all grades of steel and by approximating the plastic shear strength by $0.6F_y t_w d$ to match the AISC Specification within this range. The yielding limit, $\lambda_w \leq 0.48/\eta$, is converted to the equivalent limit, $h/t_w \leq 0.982\sqrt{k_v E/F_{yw}}$ . The ratio of the strength from Eqs. (9) to the plastic strength is

$$C_{vPB} = 1 \text{ if } h/t_w \leq 0.982\sqrt{k_v E/F_{yw}} \tag{14a}$$

$$C_{vPB} = \frac{\dfrac{0.48}{\lambda_w} F_{yw} d t_w}{0.6 F_{yw} d t_w} = \frac{0.982\sqrt{k_v E/F_{yw}}}{h/t_w} \text{ if } h/t_w > 0.982\sqrt{k_v E/F_{yw}} \tag{14b}$$

where the subscript *PB* is used to emphasize that Eqs. (14) characterize the post-buckling resistance of the unstiffened web. This equation is the same form as Basler's inelastic web shear buckling equation. Höglund (1997) uses this form to characterize shear post-buckling resistance in both the inelastic and elastic shear buckling ranges of Basler's equations.

Höglund's 1997 method is slightly conservative for non-stocky webs compared to the experimental results as well as to the AISC (2010) shear strength equations. As noted above, the form of Eq. (14b) is the same as the AISC inelastic shear buckling equation. Since there is limited justification to deviate from the AISC shear strength equations within the plastic and inelastic buckling shear strength ranges, the Eq. (14b) strength and yielding limit are increased by approximately 10%, such that the equations match exactly within these ranges, resulting in the following equation. The subscript "1" is adopted to distinguish this $C_v$ from the one used for stiffened webs.

$$V_n = 0.6 F_{yw} t_w d \, C_{v1} \tag{15}$$

where

$$C_{v1} = 1.0 \text{ if } h/t_w \leq 1.1\sqrt{k_v E/F_{yw}} \tag{16a}$$

$$C_{v1} = \frac{1.1\sqrt{k_v E/F_{yw}}}{h/t_w} \text{ if } h/t_w > 1.1\sqrt{k_v E/F_{yw}} \tag{16b}$$

The plate buckling coefficient is taken as

$$k_v = 5 + \frac{5}{(a/h)^2} \text{ if } a/h \leq 3 \tag{17a}$$

$$k_v = 5.34 \text{ if } a/h > 3 \tag{17b}$$

where the first form is the traditional AISC Specification coefficient from Vincent (1969). It is emphasized that Eqs. (16) are the same as the AISC (2010) shear resistance equations except that the application of Eq. (16b) is extended to all $h/t_w$ values larger than the limit shown. The third elastic shear buckling strength equation from AISC (2010) is no longer employed for I-Section members.

The accuracy of the proposed method is evaluated and summarized in Table 4. Measured-to-predicted shear strength ratios are plotted for the proposed method in Fig. 12. The proposed method predictions are slightly conservative, with an average ratio of 1.21 (COV = 0.176), and have approximately the same level of conservatism for all $h/t_w$. The proposed method has similar accuracy to several other the investigated methods, while providing a very simple nominal strength calculation. Furthermore, it matches the AISC (2010) Specification shear strength equations within the plastic and inelastic shear buckling range. The limited data for $h/t_w$ approximately equal to 100 and less than 100 suggests that possibly 0.58*h* should be employed instead of 0.6*d* within Eq. (15). However, the number of tests at these values of $h/t_w$ is limited. Eq. (15) has been well established for some time within the AISC specifications. In addition, Höglund's (1997) indicate that 0.577*h* can be conservative. This is due to the influence of strain hardening. Therefore, the authors defer to the established AISC Specification provisions for Eq. (15). This attribute is discussed further in the following section.

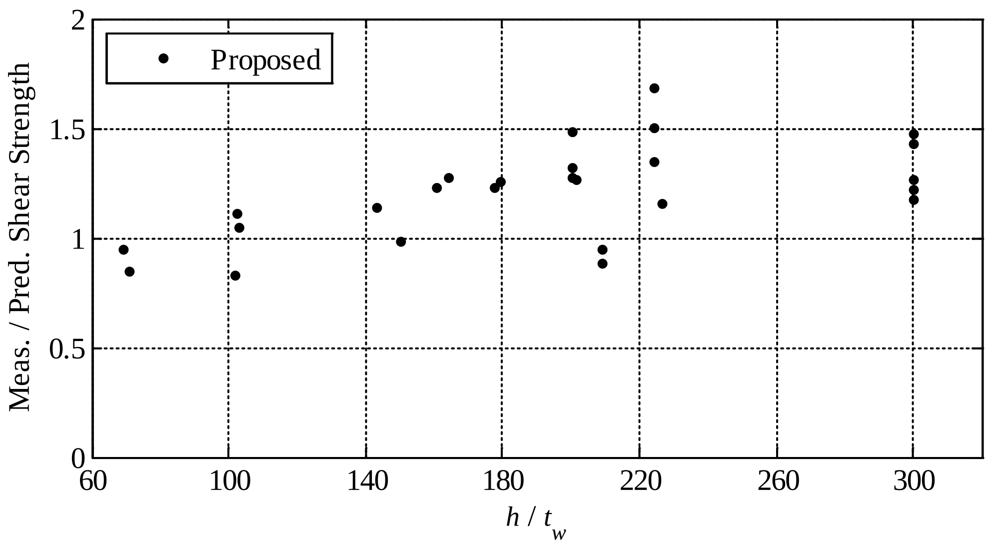


Fig. 12: Comparison of predicted-to-measured shear for the proposed method

## 6. Resistance Factors

Appropriate resistance factors for Load and Resistance Factor Design of buildings and building type structures are estimated below using the method provided in the *Guide to Stability Design Criteria for Metal Structures* (Ziemian 2010), which is summarized by Eqs. (18) through (20).

$$\rho_R = \rho_P \rho_G \rho_M \tag{18}$$

$$V_R = \sqrt{V_P^2 + V_M^2 + V_G^2} \tag{19}$$

$$\phi = \rho_R e^{-\beta \alpha_R V_R} \tag{20}$$

where

$\beta$ = 3

$\alpha_R$ = 0.55

$\rho_G$ = 1.015 (White and Barker 2008)

$V_G$ = 0.013 (White and Barker 2008)

$\rho_M$ = 1.10 (White and Barker 2008)

$V_M$ = 0.110 (White and Barker 2008)

$\rho_P$ = average measured-to-predicted strength ratio

$V_P$ = COV of measured-to-predicted strength ratio

Table 5 shows the estimated resistance factors. Basler's method (1961), Basler's method with $k_v$ by Lee et al. (1996), and Höglund's 1997 method are conservative enough to be used with $\phi$ = 1.0. Höglund's 1973 method and the proposed method can be used with $\phi$ = 0.9. The Lee et al. method can be used with $\phi$ = 0.75. The Lee et al. method gives the best prediction of the mean shear resistance of the equations considered; however, the coefficient of variation $V_P$ requires a lower $\phi$ to achieve the target reliability index of $\beta$ = 3.

Table 5: Estimated resistance factors

| | $\rho_P$ | $V_P$ | $\phi$ |
|---|---|---|---|
| Basler | 3.16 | 0.412 | 1.72 |
| Basler with Lee's $k_v$ | 2.06 | 0.378 | 1.20 |
| Höglund 1973 | 1.17 | 0.157 | 0.95 |
| Höglund 1997 | 1.38 | 0.174 | 1.09 |
| Lee et al. | 0.958 | 0.153 | 0.78 |
| Proposed | 1.21 | 0.176 | 0.96 |

## 7. Summary and Conclusions

A research study was conducted to determine the accuracy of unstiffened prismatic plate girder shear strength prediction methods. Two variations of Basler's method, two variations of methods from Höglund, and the method by Lee et al. were identified. Twenty specimens were identified in the literature and seven additional specimens were tested at the University of Kentucky. The shear strength of each specimen was predicted using each method and compared with measured ultimate shears to evaluate the accuracy of the prediction methods.

The Basler (1961) method predictions were accurate for specimens with low $h/t_w$. For $h/t_w$ exceeding approximately 150, the Basler method's average measured-to-predicted strength ratio was 3.72. The Basler method with plate buckling coefficient by Lee et al. (1996) had an average ratio of 2.38. These high ratios indicate the strength far exceeded the buckling strength, thus indicating the presence of very significant post-buckling strength of unstiffened members and members with long web panels. Because they significantly under-predict the strength, both Basler methods can be used with a resistance factor, $\phi = 1.0$.

Both Höglund variations provided slightly conservative and consistent predictions. The average measured-to-predicted strength ratio was 1.17 for the Höglund (1973) equations, which can be used with $\phi = 0.9$. The Höglund (1997) method's average ratio was 1.38, and it can be used with $\phi = 1.0$.

The method by Lee et al. (2008) directly applies to the eleven specimens with $3 \leq a/h \leq 6$. For those, the average ratio of measured-to-predicted strength was 0.972. The average ratio for the other specimens was 0.948, so the method gives similar mean predictions for those. The method was accurate, on average, but significantly over-predicted the strength of several specimens. It can be used with $\phi = 0.75$.

A proposed method was developed by simplifying Höglund's (1997) method applicable for non-rigid end posts and converting it to the familiar AISC $C_v$ and $h/t_w$ format. Its average measured-to-predicted strength ratio was 1.21, and it can be used with $\phi = 0.9$. Because of its accuracy, appropriate level of conservatism, and simplicity, the proposed method is recommended for design usage.

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## References

AASHTO (2014). *LRFD Bridge Design Specifications*, American Association of Highway and Transportation Officials, Washington, D.C.

AISC (2010). *Specification for Structural Steel Buildings, AISC 360-10*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (2015). *Specification for Structural Steel Buildings, AISC 360-15*, American Institute of Steel Construction, Inc., Chicago, IL.

Basler, K. (1960). *Web Buckling Tests on Welded Plate Girders, Welded Plate Girders Report No. 251-11*, Lehigh University Libraries, Bethlehem, PA.

Basler, K. (1961). “Strength of Plate Girders in Shear.” *Journal of the Structural Division*, 87(ST7), 151-180.
Carskaddan, P.S. (1968). “Shear Buckling of Unstiffened Hybrid Beams.” *Journal of the Structural Division*, 94(ST8), 1965-1992.
CEN (2006). *Eurocode 3: Design of Steel Structures – Part 1-5: Plated Structural Elements. EN 1993-1-5*, European Committee for Standardization, Brussels, Belgium.
Daley, A. and Davis, B. (2015). *Shear Strength of Prismatic Steel I-Shaped Members*, Research Report, Metal Building Manufacturers Association, Cleveland, Ohio.
Frey, F., and Anslijn, R. (1977). “Shear Test on Unstiffened Plate Girders.” *Second International Colloquium on Stability of Steel Structures*, ECCS. Preliminary Report, Liege.
Höglund, T. (1971). *Behaviour and Strength of the Web of Thin Plate I-Girders, Bulletin No. 93, Building Statics and Structural Engineering*, The Royal Institute of Technology, Stockholm, Sweden.
Höglund, T. (1973). *Design of Thin Plate I-Girders in Shear and Bending, with Special Reference to Web Buckling, Bulletin No. 94*, Division of Building Statics and Structural Engineering, Royal Institute of Technology, Stockholm, Sweden.
Höglund, T. (1997). “Shear Buckling Resistance of Steel and Aluminum Plate Girders.” *Thin-Walled Structures*, 29(1-4), 13-30.
Lee, S. C., Lee, D. S., and Yoo, C. H. (2008). “Ultimate Shear Strength of Long Web Panels.” *Journal of Constructional Steel Research*, 64(12), 1357-1365.
Lee, S. C., Davidson, J. S., and Yoo, C. H. (1996). “Shear Buckling Coefficients of Plate Girder Web Panels.” *Journal of Computers and Structures*, 59(5), 789-795.
Lee, S. C., and Yoo, C. H. (1998). “Strength of Plate Girder Web Panels under Pure Shear.” *Journal of Structural Engineering*, 124(2), 184-194.
Ravinger, J. (1983). “Girders With Unstiffened Slender Webs.” *Journal of Constructional Steel Research*, 3(2), 14-22.
Vincent, G.S. (1969). “Tentative Criteria for Load Factor Design of Steel Highway Bridges.” *AISI Bulletin No. 15*, American Iron and Steel Institute, Washington, D.C.
White, D.W. and Barker, M.G. (2008). “Shear Resistance of Transversely Stiffened Steel I-Girders.” *Journal of Structural Engineering*, 134(9), 1425-1426.
Ziemian, R. D. (2010). *Guide to Stability Design Criteria for Metal Structures*, 6th ed. John Wiley & Sons, Hoboken, N.J.