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Design of Built-up Plate Girder based on AISC 360-16

Lesson 2: Shear

July 29, 2021

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Course Description

Design of Built-up Plate Girders based on AISC 360-16 *Specification for Structural Steel Buildings*, Part 2 – Shear

There are many situations in flexural member design where the geometrical or loading conditions require the engineer to look beyond standard rolled wide-flange shapes. One solution for such cases is built-up plate girders, which introduce their own design challenges related to section slenderness that engineers rarely encounter when working with rolled shapes. The second session in this two-part webinar will focus on designing built-up plate girders for shear. We'll compare the two approaches that the *Specification* uses to determine shear strength: rotated stress field theory and tension field action theory. The lesson will also cover the design of both transverse and bearing stiffeners as well as the connection between web and flange plate elements.

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Learning Objectives

- Identify the two theories that provide the basis for the AISC *Specification*'s treatment of shear design for built-up members.
- Explain how the web shear strength coefficient varies as a function of stiffener spacing.
- Compare the shear strength considering tension field action to a scenario where post-buckling stress of the web is neglected.
- Describe the role of stiffeners in built-up plate girders and list the steps to design transverse and bearing stiffeners.

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Design of Built-up Plate Girders based on AISC 360-16 *Specification for Structural Steel Buildings*

Summer Webinar 2021

Lesson 2

Shear

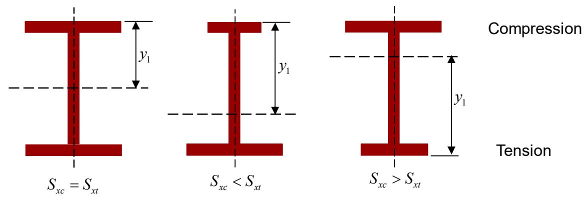
Plate Girders

- Plate girders as a term has not been used in AISC 360 since the 2005 Specification
- The previous ASD and LRFD Specifications had a separate chapter, Chapter G, that dealt with them by that name
- Currently the provisions are found in Chapter F for bending and Chapter G for shear

8

Plate Girders

- A member made (built-up) from plates in the form of a singly or doubly symmetric I-shape is what we will be referring to as plate girders



These shape differences do not matter when it comes to shear.

9

Plate Girders

- For proportioning of plate girders, F13;
  - I-shaped members with slender webs

When $\frac{a}{h} \leq 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = 12.0\sqrt{\frac{E}{F_y}}$$

When $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = \frac{0.40E}{F_y}$$

For unstiffened girders $h/t_w \leq 260$

Although this is a flexural limitation, *a*, the spacing of stiffeners, will have an impact on shear strength.

10

Plate Girders

- Chapter G addresses shear strength.
- For all shapes the limit states to be considered are *shear yielding* and *shear buckling*.
- Post-buckling strength increases are also accounted for but are not treated as a separate limit state but are built into the strength equations.

11

Plate Girders

- G2. I-shaped Members and Channels
  - G2.1 Shear Strength of Webs without Tension Field Action
    - Accounts for post-buckling strength of web through rotated stress field theory. (Höglund, 1997)
  - G2.2 Shear Strength of Interior Web Panels with $a/h \leq 3$ Considering Tension Field Action
    - Accounts for post-buckling strength through tension field action theory. (Basler, 1961)

12

Plate Girders

- G2.1 Shear Strength of Webs without Tension Field Action

$$V_n = 0.6 F_y A_w C_{v1} \quad \text{G2-1}$$

$\boxed{A_w = dt_w}$

- G2.1(b) For all other I-shaped members

(1)The web shear strength coefficient, C_{v1}

For $h/t_w \le 1.10\sqrt{k_v E/F_y}$

This represents shear yielding

$$C_{v1} = 1.0 \quad \text{G2-3}$$

For $h/t_w > 1.10\sqrt{k_v E/F_y}$

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \quad \text{G2-4}$$

G2.1(a) is for rolled W-shapes

13

Plate Girders

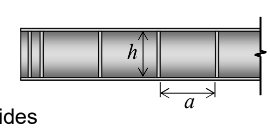
- G2.1(b)(2) The web plate shear buckling coefficient, k_v
  - For webs without transverse stiffeners

$$k_v = 5.34$$

  - For webs with transverse stiffeners

$$k_v = 5 + \frac{5}{(a/h)^2} \quad \text{G2-5}$$

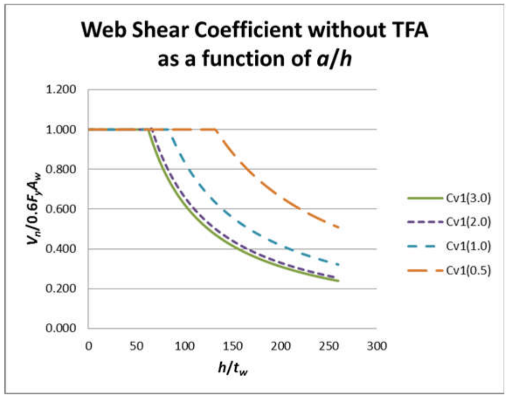
$$= 5.34 \text{ when } a/h > 3.0$$



There is a discontinuity at $a/h = 3$ that provides for additional conservativeness.

14

Plate Girders



$$V_n = 0.6F_yA_wC_{v1}$$

$$F_y = 50 \text{ ksi}$$

$$C_{v1} = \frac{1.10\sqrt{k_vE/F_y}}{h/t_w}$$

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

$$= 5.34 \text{ when } a/h > 3.0$$

For stiffened panels with $a/h = (\ )$

Section G2.1 uses Rotated Stress Field Theory to include post-buckling strength.

15

Plate Girders

- G2.2 Shear Strength of Interior Web Panels with $a/h \le 3$ considering Tension Field Action
  (a) when $h/t_w \le 1.10\sqrt{k_vE/F_y}$

$$V_n = 0.6F_yA_w \qquad \text{G2-6}$$

This is shear yielding, the same as $C_{v1} = 1$, so buckling does not occur. Thus, there is no post-buckling strength.

16

Plate Girders

(b) When $h/t_w > 1.10\sqrt{k_v E/F_y} = \lambda_{pwv}$

(1) When $2A_w/(A_{fc}+A_{ft}) \le 2.5$, h/b_{fc} and $h/b_{ft} \le 6$

$$V_n = 0.6F_yA_w\left(C_{v2} + \frac{1-C_{v2}}{1.15\sqrt{1+(a/h)^2}}\right) \qquad \text{G2-7}$$

(2) Otherwise (essentially small flanges)

$$V_n = 0.6F_yA_w\left(C_{v2} + \frac{1-C_{v2}}{1.15\left(a/h+\sqrt{1+(a/h)^2}\right)}\right) \qquad \text{G2-8}$$

17

Plate Girders

- The web shear buckling coefficient, C_{v2}

For $h/t_w \le 1.10\sqrt{k_v E/F_y} = \lambda_{pwv}$

This represents shear yielding

$$C_{v2} = 1.0 \qquad \text{G2-9}$$

For $1.10\sqrt{k_v E/F_y} < h/t_w \le 1.37\sqrt{k_v E/F_y}$

$$C_{v2} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \qquad \text{G2-10}$$

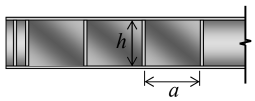
For $h/t_w > 1.37\sqrt{k_v E/F_y} = \lambda_{rwv}$

$$C_{v2} = \frac{1.51k_vE}{(h/t_w)^2 F_y} \qquad \text{G2-11}$$

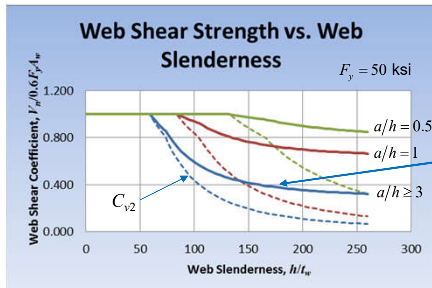
18

Plate Girders

- With stiffeners and tension field action



C_{v2} is what you would be using if there was no accounting for post-buckling strength through rotated stress field theory.



From Eq. G2-7

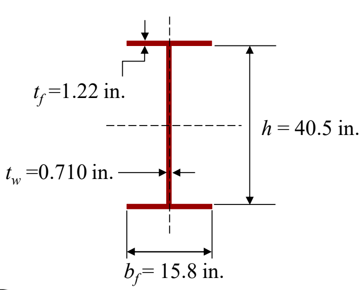
$$\left[C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1 + (a/h)^2}}\right]$$

Section G2.2 uses Tension Field Action Theory to include post-buckling strength.

19

Example 6

- Determine the shear strength of a plate girder similar to a W-shape. $F_y = 50$ ksi



(W44x230)

$A_w = 42.9(0.710) = 30.5$ in.^{2}

$h = 40.5$ in.

$t_w = 0.710$ in.

$h/t_w = 57.0$

For $k_v = 5.34$ (no stiffeners)

$$\lambda_{pwv} = 1.10\sqrt{k_v E/F_y} = 61.2$$

20

Example 6

- Since

$$h/t_w = 57.0 < \lambda_{pwv} = 61.2$$

the web will yield in shear and

$$C_{v1} = 1.0 \qquad \text{G2-3}$$

and the shear strength is

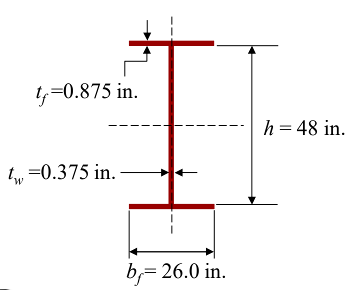
$$V_n = 0.6F_yA_wC_{v1} = 0.6(50)(30.5)(1.0) = 915 \text{ kips} \qquad \text{G2-1}$$

$\phi V_n = 0.9(915) = 824$ kips (LRFD) $V_n/\Omega = 915/1.67 = 548$ kips (ASD)

21

Example 1G

- Determine the shear strength of the plate girder from Example 1 without Tension Field Action.



$$A_w = 49.8(0.375) = 18.7 \text{ in.}^2$$

$$h = 48.0 \text{ in.}$$

$$t_w = 0.375 \text{ in.}$$

$$h/t_w = 128$$

$$F_y = 36 \text{ ksi}$$

For $k_v = 5.34$ (no stiffeners)

$$\lambda_{pwv} = 1.10\sqrt{k_v E/F_y} = 72.1$$

22

Example 1G

- Since

$$h/t_w = 128 > \lambda_{pwv} = 72.1$$

No λ_{rwv} to be checked since that only occurs when TFA is considered.

the web will buckle in shear and

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} = \frac{1.10\sqrt{5.34(29,000/36)}}{128} = 0.564 \quad \text{G2-4}$$

and the shear strength is

$$V_n = 0.6F_y A_w C_{v1} = 0.6(36)(18.7)(0.564) = 228 \text{ kips} \quad \text{G2-1}$$

23

Example 1G

- Assume this girder is on a 100 ft span with bracing every 20 ft. From Example 1 with $L_b = 20$ ft, the strength is controlled by flange local buckling, $M_n = 3240$ ft-kips
- For LRFD

$$\phi M_n = 0.9(3240) = 2920 \text{ ft-kips}$$

and for uniform load

$$w_{uM} = \frac{2920(8)}{(100)^2} = 2.34 \text{ kip/ft}$$

24

Example 1G

- Based on shear strength

$$\phi V_n = 0.9(228) = 205 \text{ kips}$$

and for uniform load

$$w_{uV} = \frac{205(2)}{100} = 4.10 \text{ kips/ft}$$

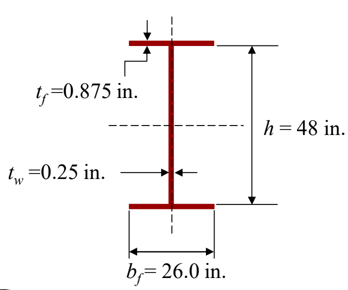
- So, the strength of this girder is controlled by bending since

$$w_{uV} = 4.01 \text{ kip/ft} > w_{uM} = 2.34 \text{ kip/ft}$$

25

Example 5G

- Determine the shear strength of the slender web plate girder from Example 5 without Tension Field Action.



$$A_w = 49.8(0.250) = 12.5 \text{ in.}^2$$

$$h = 48.0 \text{ in.}$$

$$t_w = 0.250 \text{ in.}$$

$$h/t_w = 192$$

$$F_y = 36 \text{ ksi}$$

For $k_v = 5.34$ (no stiffeners)

$$\lambda_{pwv} = 1.10\sqrt{k_v E/F_y} = 72.1$$

26

Example 5G

- Since

$$h/t_w = 192 > \lambda_{pwv} = 72.1$$

the web will buckle in shear and

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} = \frac{1.10\sqrt{5.34(29,000/36)}}{192} = 0.376 \quad \text{G2-4}$$

and the shear strength is

$$V_n = 0.6F_y A_w C_{v1} = 0.6(36)(12.5)(0.376) = 102 \text{ kips} \quad \text{G2-1}$$

27

Example 5G

- Assume this girder is on a 90 ft span with bracing every 15 ft. From Example 5 with $L_b = 15$ ft, the strength is controlled by flange local buckling, $M_n = 3010$ ft-kips
- For LRFD

$$\phi M_n = 0.9(3010) = 2710 \text{ ft-kips}$$

and for uniform load

$$w_{uM} = \frac{2710(8)}{(90)^2} = 2.68 \text{ kip/ft}$$

28

Example 5G

- Based on shear strength

$$\phi V_n = 0.9(102) = 91.8 \text{ kips}$$

and for uniform load

$$w_{uV} = \frac{91.8(2)}{90} = 2.04 \text{ kips/ft}$$

- So, the strength of this girder is controlled by shear since

$$w_{uV} = 2.04 \text{ kip/ft} < w_{uM} = 2.68 \text{ kip/ft}$$

29

Example 5G

- To increase shear strength, without changing the web dimensions, we must increase C_{v1}.
- Thus, add stiffeners with $a/h < 3$ so that k_v will be greater than 5.34.
- The required shear strength is

$$V_u = \frac{2.68(90)}{2} = 121 \text{ kips} \qquad V_n = \frac{V_u}{\phi} = \frac{121}{0.9} = 134 \text{ kips}$$

30

Example 5G

- Determine the minimum C_{v1} to provide the needed strength.
- Since

$$V_n = 0.6F_y A_w C_{v1} \qquad \text{G2-1}$$

- Then

$$C_{v1} = \frac{V_n}{0.6F_y A_w} = \frac{134}{0.6(36)(12.5)} = 0.496$$

31

Example 5G

- To increase C_{v1}, we must put in sufficient stiffeners to increase k_v.
- For the slenderness of the web that we have,

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} = \frac{1.10\sqrt{k_v(29{,}000/36)}}{192} = 0.496 \qquad \text{G2-4}$$

Thus,

$$k_v = \frac{(0.496)^2(192)^2(36)}{(1.10)^2(29{,}000)} = 9.30$$

32

Example 5G

- Since

$$k_v = 5 + \frac{5}{(a/h)^2} = 9.30 \qquad \text{G2-5}$$

- a/h must be

$$a/h = \sqrt{\frac{5}{k_v - 5}} = \sqrt{\frac{5}{9.30 - 5}} = 1.08$$

- Thus,

$$a(\text{max}) = 1.08(48) = 51.8 \text{ in.}$$

33

Example 5G

- Try stiffeners at 48 in.

$$a/h = 48/48 = 1.0$$

$$k_v = 5 + \frac{5}{(1.0)^2} = 10.0 \qquad \text{G2-5}$$

For $k_v = 10.0$

$$\lambda_{pwv} = 1.10\sqrt{k_v\, E/F_y} = 98.7$$

$h/t_w = 192$ so the web will still buckle

34

Example 5G

- Determine the strength of this stiffened web

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} = \frac{1.10\sqrt{10.0(29{,}000/36)}}{192} = 0.514 \qquad \text{G2-4}$$

and

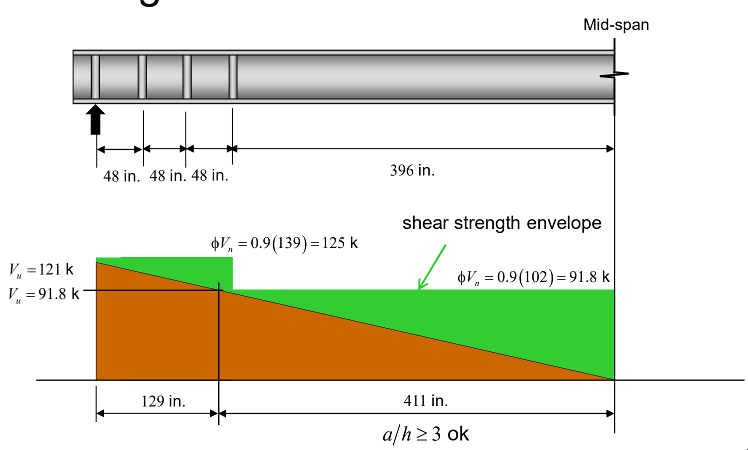
$$V_n = 0.6F_y A_w C_{v1} = 0.6(36)(12.5)(0.514) = 139 \text{ kips} \qquad \text{G2-1}$$

$$w_{uV} = \frac{0.9(139)(2)}{90} = 2.78 \text{ kips/ft} > w_{uM} = 2.68 \text{ kips/ft}$$

35

Example 5G

- Final design



36

Example 5G

- This solution requires stiffeners at 48 in. for 144 in. at each end of the girder.
- Tension field action is not permitted in end panels or with $a/h > 3$.
- Even for $a/h = 3$ tension field action might increase strength sufficiently to reduce the number of stiffeners.

37

Example 5G

- G2.2 has two equations for use of tension field action.

Eq. G2-7 may be used if:	Our example:
not end panels	not in end panels
$a/h \le 3.0$	$a/h \le 3.0$
$2A_w/(A_{fc}+A_{ft}) \le 2.5$*	$2A_w/(A_{fc}+A_{ft}) = 0.549$
h/b_{fc} and $h/b_{ft} \le 6.0$*	h/b_{fc} and $h/b_{ft} = 1.85$

All criteria are satisfied

* If these limits are exceeded, use Equation G2-8.

38

Example 5G

- Therefore, see if we can eliminate one stiffener if we use TFA with

$$a/h = 96/48 = 2.0$$

$$k_v = 5 + \frac{5}{(2.0)^2} = 6.25 \qquad \text{G2-5}$$

For $k_v = 6.25$

$$\lambda_{pwv} = 1.10\sqrt{k_v\, E/F_y} = 78.1$$

$$\lambda_{rwv} = 1.37\sqrt{k_v\, E/F_y} = 97.2$$

$h/t_w = 192$ so the web will still buckle

39

Example 5G

- Determine strength with tension field action

$$C_{v2} = \frac{1.51 k_v E}{(h/t_w)^2 F_y} = \frac{1.51(6.25)(29,000)}{(192)^2(36)} = 0.206 \qquad \text{G2-11}$$

$$V_n = 0.6 F_y A_w \left(C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1 + (a/h)^2}} \right) \qquad \text{G2-7}$$

$$= 0.6(36)(12.5)\left(0.206 + \frac{1 - 0.206}{1.15\sqrt{1 + (2.0)^2}} \right) = 139 \text{ kips}$$

40

Example 5G

- The available shear strength with tension field action with stiffeners at 96 in. is

$$\phi V_n = 0.9(139) = 125 \text{ kips}$$

- The required shear strength at the first stiffener is

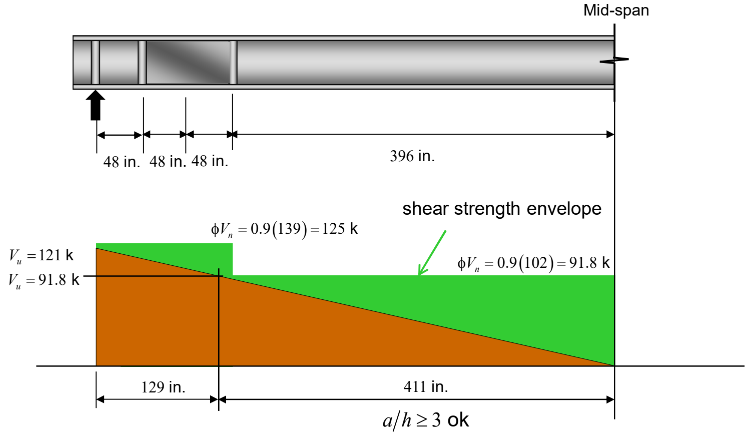
$$V_u = 121 - 2.68(4.0) = 110 \text{ kips} < \phi V_n = 125 \text{ kips}$$

- Thus, a second stiffener at 96 in. will be sufficient. (Can not use TFA in end panel)

41

Example 5G

- Final design with TFA



42

Example 5G

- Stiffeners must be sized according to G2.3
- Stiffeners are *bearing* stiffeners at the support and *transverse* stiffeners along the span.
- We will address stiffeners toward the end of the presentation.

43

Plate Girders

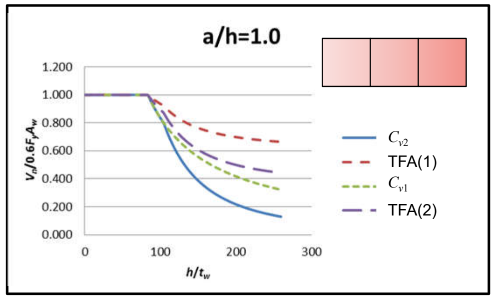
- Next, we will look at the second Tension Field Action equation and the limits that apply there.
- We will see that *tension field action* does not always give more strength than *rotated stress field* theory.
- We will also look at the design aids in the 15th edition *Manual*.

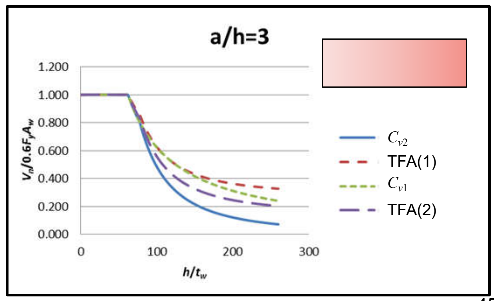
44

Comparison between Rotated Stress Field, Section G2.1 and Tension Field Action, Section G2.2.

TFA(1) = Eq. G2-7
TFA(2) = Eq. G2-8

Sometimes TFA does not increase strength over Rotated Stress Field.

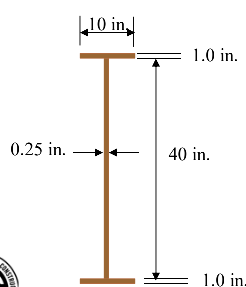




45

Example 7

- Determine the available shear strength without tension field action and with tension field action.



Use A572 Gr. 50 plates and stiffeners every 5.0 ft.

Web width-to-thickness ratio

$$h/t_w = 40/0.25 = 160$$

Web plate shear buckling coefficient

$$k_v = 5 + \frac{5}{(a/h)^2} = 5 + \frac{5}{(60/40)^2} = 7.22 \qquad \text{(G2-5)}$$

46

Example 7

- Without tension field action Section G2.1(b)

$$h/t_w = 40/0.25 = 160 > 1.10\sqrt{k_v E/F_y} = 1.10\sqrt{7.22E/50} = 71.2$$

- Therefore,

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} = \frac{71.2}{160} = 0.445 \qquad \text{(G2-4)}$$

- and

$$V_n = 0.6F_y A_w C_{v1} = 0.6(50)\big(42(0.25)\big)(0.445) = 140 \text{ kips}$$

$$\phi V_n = 0.9(140) = 126 \text{ kips}$$

47

Example 7

- With tension field action Section G2.2

$$a/h = 60/40 = 1.5 \le 3$$

$$h/t_w = 40/0.25 = 160 > 1.10\sqrt{k_v E/F_y} = 1.10\sqrt{7.22E/50} = 71.2$$

$$> 1.37\sqrt{k_v E/F_y} = 1.37\sqrt{7.22E/50} = 88.7$$

$$2A_w/\left(A_{fc} + A_{ft}\right) = 2\big(42(0.25)\big)/\big(1(10) + 1(10)\big) = 1.05 \le 2.5$$

$$h/b_{fc} = 40/10 = 4 \le 6$$

$$h/b_{ft} = 40/10 = 4 \le 6$$

- Therefore, use Section G2.2(b)(1)

This is the same Specification Section that we have used in all our previous TFA examples

48

Example 7

- Therefore,

$$C_{v2} = \frac{1.51k_v E}{(h/t_w)^2 F_y} = \frac{1.51(7.22)E}{(40/0.25)^2(50)} = 0.247 \qquad \text{(G2-11)}$$

$$V_n = 0.6F_y A_w \left[C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1+(a/h)^2}} \right] \qquad \text{(G2-7)}$$

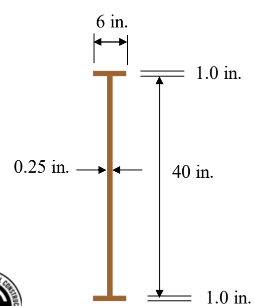
$$= 0.6(50)(42(0.25)) \left[0.247 + \frac{1 - 0.247}{1.15\sqrt{1+(60/40)^2}} \right] = 192 \text{ kips}$$

$$\phi V_n = 0.9(192) = 173 \text{ kips}$$

49

Example 7

- Consider how flange size impacts the shear strength.
- Reduce the flange width to 6.0 in.



Check criteria for which equation to use.

$$2A_w/(A_{fc}+A_{ft}) = 2(42(0.25))/(1(6.0)+1(6.0)) = 1.75 \le 2.5$$

$$h/b_{fc} = 40/6.0 = 6.67 > 6$$

$$h/b_{ft} = 40/6.0 = 6.67 > 6$$

These are now controlling.

Therefore, use Section G2.2(b)(2)

50

Example 7

- Therefore,

C_{v2} does not change

$$C_{v2} = \frac{1.51 k_v E}{(h/t_w)^2 F_y} = \frac{1.51(7.22)E}{(40/0.25)^2(50)} = 0.247 \qquad \text{(G2-11)}$$

This is what has been added to Eq. G2-7

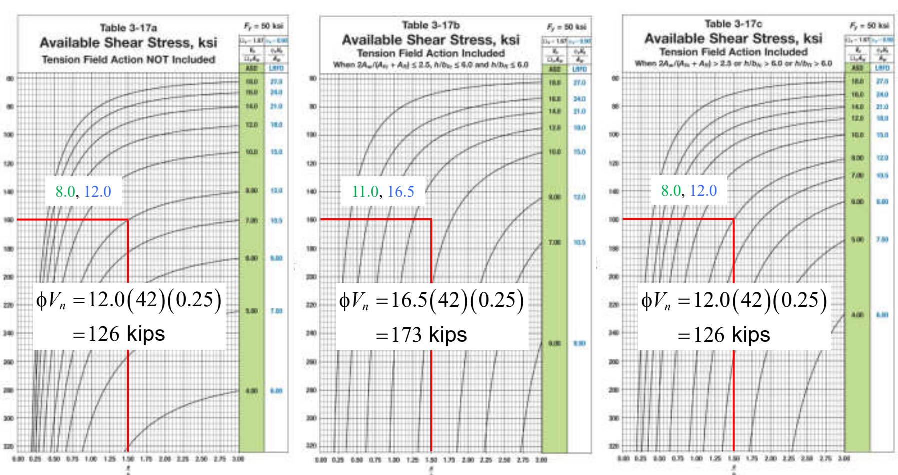
$$V_n = 0.6 F_y A_w \left[C_{v2} + \frac{1 - C_{v2}}{1.15\left[a/h + \sqrt{1+(a/h)^2} \right]} \right] \qquad \text{(G2-8)}$$

$$= 0.6(50)(42(0.25)) \left[0.247 + \frac{1-0.247}{1.15\left[60/40 + \sqrt{1+(60/40)^2} \right]} \right] = 140 \text{ kips}$$

$$\phi V_n = 0.9(140) = 126 \text{ kips}$$

51

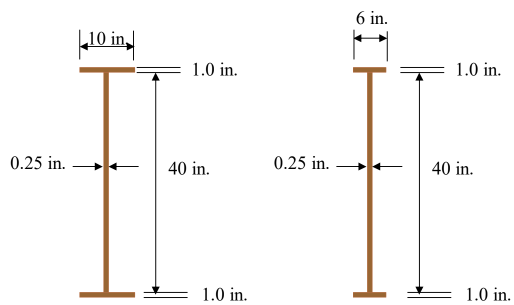
Example 7



52

Example 8

- Repeat Example 7 with stiffeners at 80 in.



The only thing that changes is stiffener spacing. Thus,

$$\frac{h}{t_w} = \frac{40}{0.25} = 160$$

$$\frac{a}{h} = \frac{80}{40} = 2.0$$

$$k_v = 5 + \frac{5}{\left(a/h\right)^2} = 5 + \frac{5}{\left(80/40\right)^2} = 6.25 \qquad \text{(G2-5)}$$

53

Example 8

- Without tension field action Section G2.1(b)

$$h/t_w = 40/0.25 = 160 > 1.10\sqrt{k_v E/F_y} = 1.10\sqrt{6.25E/50} = 66.2$$

- Therefore,

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} = \frac{66.2}{160} = 0.414 \qquad \text{(G2-4)}$$

- and

$$V_n = 0.6F_y A_w C_{v1} = 0.6\left(50\right)\left(42\left(0.25\right)\right)\left(0.414\right) = 130 \text{ kips}$$

$$\phi V_n = 0.9\left(130\right) = 117 \text{ kips}$$

54

Example 8

- With tension field action Section G2.2

$$a/h = 80/40 = 2.0 \le 3$$

$$h/t_w = 40/0.25 = 160 > 1.10\sqrt{k_v E/F_y} = 1.10\sqrt{6.25E/50} = 66.2$$

$$> 1.37\sqrt{k_v E/F_y} = 1.37\sqrt{6.25E/50} = 82.5$$

$$2A_w/\left(A_{fc} + A_{ft}\right) = 2\left(42\left(0.25\right)\right)/\left(1\left(10\right) + 1\left(10\right)\right) = 1.05 \le 2.5$$

$$h/b_{fc} = 40/10 = 4 \le 6$$

$$h/b_{ft} = 40/10 = 4 \le 6$$

- Therefore, use Section G2.2(b)(1)

55

Example 8

- Therefore,

$$C_{v2} = \frac{1.51 k_v E}{\left(h/t_w\right)^2 F_y} = \frac{1.51\left(6.25\right)E}{\left(40/0.25\right)^2\left(50\right)} = 0.214 \qquad \text{(G2-11)}$$

$$V_n = 0.6F_y A_w\left[C_{v2} + \frac{1 - C_{v2}}{1.15\sqrt{1 + \left(a/h\right)^2}}\right] \qquad \text{(G2-7)}$$

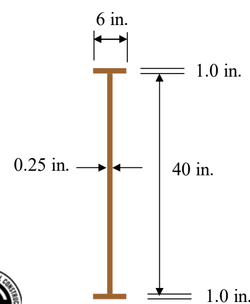
$$= 0.6\left(50\right)\left(42\left(0.25\right)\right)\left[0.214 + \frac{1 - 0.214}{1.15\sqrt{1 + \left(80/40\right)^2}}\right] = 164 \text{ kips}$$

$$\phi V_n = 0.9\left(164\right) = 147 \text{ kips}$$

56

Example 8

- Consider how flange size impacts the shear strength.
- Reduce the flange width to 6.0 in.



Check criteria for which equation to use.

$$2A_w/(A_{fc}+A_{ft}) = 2(42(0.25))/(1(6.0)+1(6.0)) = 1.75 \le 2.5$$

$$h/b_{fc} = 40/6.0 = 6.67 > 6$$
$$h/b_{ft} = 40/6.0 = 6.67 > 6$$

These are now controlling.

Therefore, use Section G2.2(b)(2)

57

Example 8

- Therefore,

C_{v2} does not change

$$C_{v2} = \frac{1.51k_vE}{(h/t_w)^2F_y} = \frac{1.51(6.25)E}{(40/0.25)^2(50)} = 0.214 \qquad \text{(G2-11)}$$

This is what has been added to Eq. G2-7

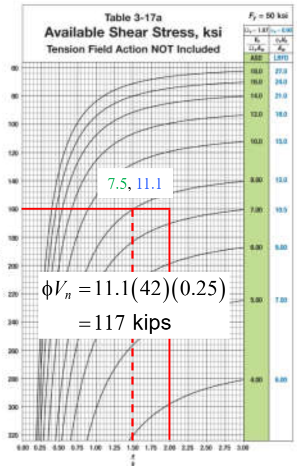
$$V_n = 0.6F_yA_w\left[C_{v2} + \frac{1-C_{v2}}{1.15\left[a/h+\sqrt{1+(a/h)^2}\right]}\right] \qquad \text{(G2-8)}$$

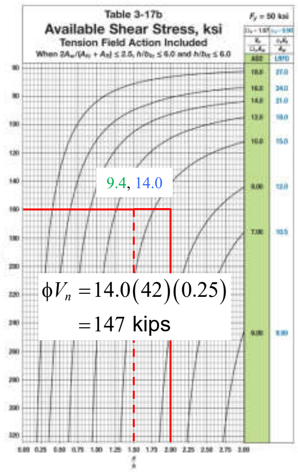
$$= 0.6(50)(42(0.25))\left[0.214 + \frac{1-0.214}{1.15\left[80/40+\sqrt{1+(80/40)^2}\right]}\right] = 118 \text{ kips}$$

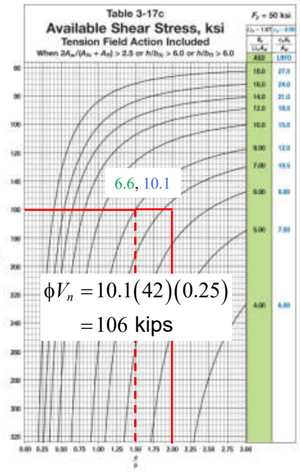
$$\phi V_n = 0.9(118) = 106 \text{ kips}$$

58

Example 8





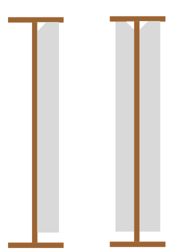


59

Plate Girders

- G2.3 Transverse Stiffeners

  For transverse stiffeners, the following shall apply (with and without TFA)



Single or double stiffeners

(a) Not required if

$$h/t_w \le 2.46\sqrt{E/F_{yst}}$$

or not needed for strength (can use $k_v = 5.34$)

(b) Stiffeners can be stopped short of tension flange

(c) Addresses welding or bolting of stiffener

(d)

$$(b/t)_{st} \le 0.56\sqrt{E/F_{yst}} \qquad \text{(G2-12)}$$

(e)

$$I_{st} \ge I_{st2} + (I_{st1} - I_{st2})\rho_w \qquad \text{(G2-13)}$$

60

Plate Girders

- G2.3 Transverse Stiffeners

$$I_{st} \geq I_{st2} + \left(I_{st1} - I_{st2}\right)\rho_w \qquad \text{(G2-13)}$$

$$I_{st1} = \frac{h^4 \rho_{st}^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5}$$ = minimum I for full shear post buckling strength of stiffened web, V_{c1} (G2-14)

$$I_{st2} = \left[\frac{2.5}{\left(a/h\right)^2} - 2\right] b_p t_w^3 \geq 0.5 b_p t_w^3$$ = minimum I for web shear buckling strength of stiffened web, V_{c2} (G2-15)

61

Plate Girders

- G2.3 Transverse Stiffeners

$$\rho_w = \left[\frac{V_r - V_{c2}}{V_{c1} - V_{c2}}\right] \geq 0$$ = maximum shear ratio

V_r = required shear strength in the panel being considered
V_{c1} = available shear strength calculated with V_n as defined in Section G2.1 or G2.2, as applicable
V_{c2} = available shear strength calculated with $V_n = 0.6 F_y A_w C_{v2}$

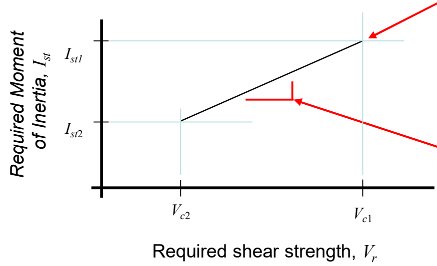
Note that this ratio includes the required strength so the entire calculation must be done in terms of LRFD or ASD (available strength).

62

Plate Girders

- Equation G2-13 allows interpolation between the minimum and maximum required stiffener inertia based on how much of the available strength we are using.

It is conservative to just use I_{st1}.



$$I_{st} \geq I_{st2} + \left(I_{st1} - I_{st2}\right)\left[\frac{V_r - V_{c2}}{V_{c1} - V_{c2}}\right]$$

$$I_{st} \geq I_{st2} + \frac{\left(I_{st1} - I_{st2}\right)}{\left(V_{c1} - V_{c2}\right)}\left(V_r - V_{c2}\right)$$

63

Example 9

- Determine the required stiffener moment of inertia for the three cases of nominal shear strength determined in Example 8.
  - a) $\phi V_n = 117$ kips without TFA
  - b) $\phi V_n = 147$ kips with TFA
  - c) $\phi V_n = 106$ kips with 6.0 in. flange width and TFA

Use A36 stiffeners, $F_y = 36$ ksi

64

Example 9

- For all three cases

b_p = smaller of a or h

$a = 80$

$h = 40$

$$I_{st2} = \left[\frac{2.5}{(a/h)^2} - 2\right] b_p t_w^3 \geq 0.5 b_p t_w^3 \qquad \text{(G2-15)}$$

$$= \left[\frac{2.5}{(80/40)^2} - 2\right](40)(0.25)^3 = -0.859 \geq 0.5(40)(0.25)^3 = 0.313 \text{ in.}^4$$

$$I_{st1} = \frac{h^4 \rho_{st}^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} \qquad \text{(G2-14)}$$

$$= \frac{(40)^4 (50/36)^{1.3}}{40}\left(\frac{50}{E}\right)^{1.5} = 7.02 \text{ in.}^4$$

ρ_{st} = larger of F_{yw}/F_{yst} and 1.0

$= 50/36 = 1.39$

65

Example 9

- For all three cases

$$I_{st2} = 0.313 \text{ in.}^4$$

$$I_{st1} = 7.02 \text{ in.}^4$$

$$\rho_w = \left[\frac{V_r - V_{c2}}{V_{c1} - V_{c2}}\right]$$

where

V_{c2} = available strength using

$$V_n = 0.6 F_y A_w C_{v2}$$

$$= 0.6(50)(42(0.25))(0.214) = 67.4 \text{ kips}$$

$$V_{c2} = \phi V_n = 0.9(67.4) = 60.7 \text{ kips}$$

66

Example 9

- Develop the equation for stiffener moment of inertia as a function of LRFD required strength, $V_r = 100$ kips.

$$I_{st} \geq I_{st2} + \left(I_{st1} - I_{st2}\right)\rho_w = 0.313 + \left(7.02 - 0.313\right)\rho_w$$
$$= 0.313 + 6.71\rho_w$$

Case a) without TFA, $V_{c1} = \phi V_n = 117 \text{ kips}$

$$\rho_w = \left[\frac{V_r - V_{c2}}{V_{c1} - V_{c2}}\right] = \frac{100 - 60.7}{117 - 60.7} = 0.698$$

$$I_{st} \geq 0.313 + 6.71\rho_w = 0.313 + 6.71\left(0.698\right) = 5.00 \text{ in.}^4$$

67

Example 9

- Develop the equation for stiffener moment of inertia as a function of LRFD required strength, $V_r = 100$ kips.

$$I_{st} \geq I_{st2} + \left(I_{st1} - I_{st2}\right)\rho_w = 0.313 + \left(7.02 - 0.313\right)\rho_w$$
$$= 0.313 + 6.71\rho_w$$

Case b) with TFA, $V_{c1} = \phi V_n = 147 \text{ kips}$

$$\rho_w = \left[\frac{V_r - V_{c2}}{V_{c1} - V_{c2}}\right] = \frac{100 - 60.7}{147 - 60.7} = 0.455$$

$$I_{st} \geq 0.313 + 6.71\rho_w = 0.313 + 6.71\left(0.455\right) = 3.37 \text{ in.}^4$$

68

Example 9

- Develop the equation for stiffener moment of inertia as a function of LRFD required strength, $V_r = 100$ kips.

$$I_{st} \geq I_{st2} + \left(I_{st1} - I_{st2}\right)\rho_w = 0.313 + \left(7.02 - 0.313\right)\rho_w$$
$$= 0.313 + 6.71\rho_w$$

Case c) with 6 in. Flange, extended TFA,

$$V_{c1} = \phi V_n = 106 \text{ kips}$$

$$\rho_w = \left[\frac{V_r - V_{c2}}{V_{c1} - V_{c2}}\right] = \frac{100 - 60.7}{106 - 60.7} = 0.868$$

$$I_{st} \geq 0.313 + 6.71\rho_w = 0.313 + 6.71\left(0.868\right) = 6.14 \text{ in.}^4$$

69

Example 9

- The required stiffener moment of inertia for the three cases of shear strength from Example 8 with $V_r = 100$ kips.
  - a) $\phi V_n = 117$ kips without TFA
    $I_{st} = 5.00$ in.^{4}
  - b) $\phi V_n = 147$ kips with TFA
    $I_{st} = 3.37$ in.^{4}
  - c) $\phi V_n = 106$ kips with 6.0 in. flange width and TFA
    $I_{st} = 6.14$ in.^{4}

70

Example 9

- Determine stiffener size.
  - Choose a stiffener with a thickness, $t_w = 0.375$ in.

$$\left(b/t\right)_{st} \le 0.56\sqrt{E/F_{yst}} \qquad \text{(G2-12)}$$

$$b \le 0.375\left(0.56\sqrt{E/36}\right) = 5.96 \text{ in.}$$

  - Therefore, try a 4.75 x 0.375 A36 stiffener

71

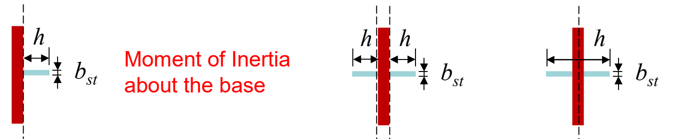
Example 9

- Consider a single 4.75 x 0.375 in. stiffener.
  - Confirm stiffener width-to-thickness ratio

$$\left(b/t\right)_{st} = 4.75/0.375 = 12.7 \le 0.56\sqrt{E/F_{yst}} = 0.56\sqrt{E/36} = 15.9$$

  - Determine stiffener moment of inertia

$$I_{st} = \frac{b_{st}h^3}{3} = \frac{0.375\left(4.75\right)^3}{3} = 13.4 \text{ in.}^4$$

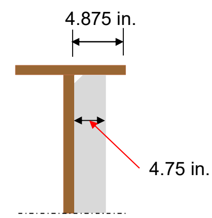


2 times $I_{st} = b_{st}h^3/3$ Or $I_{st} = b_{st}h^3/12$ as one

72

Example 9

- The 4.75 x 0.375 in. stiffener for the three cases of shear strength from Example 8 with $V_r = 100$ kips.
  - a) $\phi V_n = 117$ kips without TFA
    $I_{st} = 5.00$ in.$^4 < 13.4$ in.^{4}
  - b) $\phi V_n = 147$ kips with TFA
    $I_{st} = 3.37$ in.$^4 < 13.4$ in.^{4}
  - c) $\phi V_n = 106$ kips with 6.0 in. flange width and TFA
    $I_{st} = 6.14$ in.$^4 < 13.4$ in.^{4}



However, dimensionally the stiffener will not fit.

73

Example 9

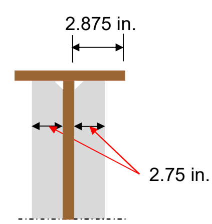
- For case (c), use the maximum width double stiffeners.
  - c) $\phi V_n = 106$ kips with 6.0 in. flange width and TFA

With double stiffeners, each must provide half of the required I_{st}

$I_{st} = 6.14/2 = 3.07$ in.^{4}

$$b = \frac{3.07(3)}{(2.75)^3} = 0.443 \text{ in.}$$

Therefore, use two 2.75 x 0.50 in. stiffeners.

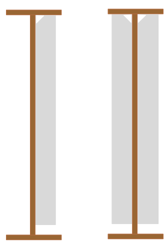


74

Plate Girders

- G2.3 Transverse Stiffeners

  For transverse stiffeners, the following shall apply (with and without TFA)



Single or double stiffeners

Additional requirements in (b) and (c):
- Stiffeners can be stopped short of tension flange provided bearing is not needed.
- Stiffener to web weld terminated not less than $4t_w$ nor more than $6t_w$ from toe of web to flange weld.
- For single stiffener, attach to compression flange.
- Clear distance between intermittent fillet welds between stiffener and web not more than $16t_w$ nor more than 10 in.

J2.2b(e) Intermittent fillet weld length $\geq 4w$ and 1.5 in.

75

Plate Girders

- Design of stiffener to web welds
  - There is no requirement for transfer of a specific force through this connection.
  - Historic recommendation from Basler, (1963) is

$$f_{nv} = 0.045h\sqrt{\frac{F_{yw}^3}{E}} \text{ kips/in.}$$

  - Strength of fillet weld is based on weld rupture, plate shear yield, or plate shear rupture.

76

Plate Girders

- Design of stiffener to web welds
  - J2.2 Strength of fillet welds

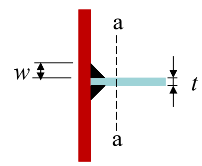
    $$\phi R_n = 2\phi\left(0.707w\right)\left(0.6F_{exx}\right)l$$

  - J4.2 Strength of stiffener at a-a
    - (a) Yield

      $$\phi R_n = \phi 0.6F_y tl \qquad \phi = 1.00$$

    - (b) Rupture

      $$\phi R_n = \phi 0.6F_u tl \qquad \phi = 0.75$$



77

Plate Girders

- Design of stiffener to web welds
  - Strength of welds

    $$\phi R_n = 2\phi\left(0.707w\right)\left(0.6F_{exx}\right)l$$

  - Strength of stiffener at a-a for A36 steel
    - Yield

      $$\phi R_n = \phi 0.6F_y tl = 1.00\left(0.6\left(36\right)\right)tl = 21.6tl$$

    - Rupture

      $$\phi R_n = \phi 0.6F_u tl = 0.75\left(0.6\left(58\right)\right)tl = 26.1tl$$

78

Plate Girders

- Design of stiffener to web welds
  - Strength of welds
    $$\phi R_n = 2\phi(0.707w)(0.6F_{exx})l$$
  - Strength of stiffener at a-a for A572 Gr. 50 steel
    - Yield
      $$\phi R_n = \phi 0.6F_y tl = 1.00(0.6(50))tl = 30.0tl$$
    - Rupture
      $$\phi R_n = \phi 0.6F_u tl = 0.75(0.6(65))tl = 29.3tl$$

79

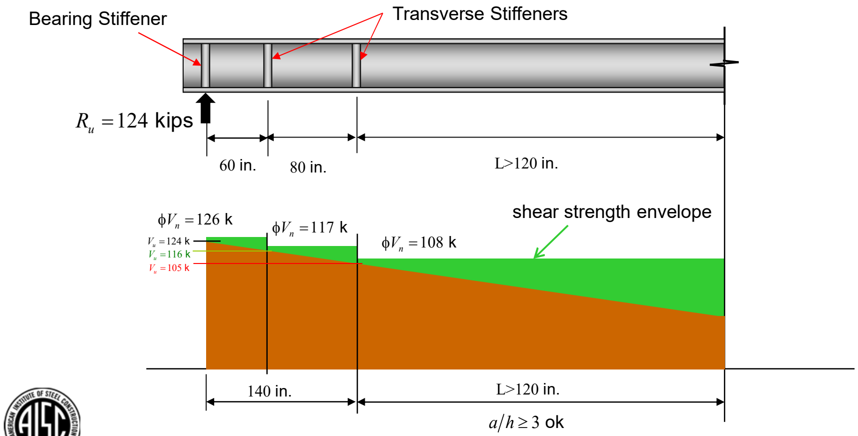
Plate Girders

- Design of stiffener to web welds
  - If we set the strength of welds equal to the strength of the stiffener, we can determine a maximum effective weld width.
    - Yield $2\phi(0.707w)(0.6F_{exx})l = \phi 0.6F_y tl$
      $$w = \frac{\phi 0.6F_y tl}{2\phi(0.707)(0.6F_{exx})l} = \frac{1.00(0.6F_y)t}{2(0.75)(0.707)(0.6F_{exx})} = 0.943\frac{F_y t}{F_{exx}}$$
    - Rupture $2\phi(0.707w)(0.6F_{exx})l = \phi 0.6F_u tl$
      $$w = \frac{\phi 0.6F_u tl}{2\phi(0.707)(0.6F_{exx})l} = \frac{0.75(0.6F_u)t}{2(0.75)(0.707)(0.6F_{exx})} = 0.707\frac{F_u t}{F_{exx}}$$

80

Example 10

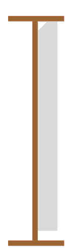
- Final design without TFA, Example 7a & 8a



81

Example 10

- Select welds for stiffener found in Ex. 9(a)



Our stiffener has $I_{st} > I_{st1}$ (independent of shear force.)

Determine the required weld strength

$$f_{nv} = 0.045h\sqrt{\frac{F_{yw}^3}{E}} = 0.045(40.0)\sqrt{\frac{(50)^3}{29,000}} = 3.74 \text{ kips/in.}$$

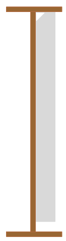
This can be reduced by the ratio of required/available strength (at first transverse stiffener)

$$f_{nv\ required} = \frac{V_u}{\phi V_n} f_{nv} = \frac{116}{117}(3.74) = 3.71 \text{ kips/in.}$$

82

Example 10

- Select welds for stiffener found in Ex. 9(a)



Determine minimum weld size based on thinner material, Table J2.4

$$w = \frac{1}{8} = 0.125 \text{ in.}$$

Determine maximum effective weld size based on yielding of stiffener (A36)

$$w_{\text{max eff}} = 0.943\frac{F_y t}{F_{exx}} = 0.943\frac{36(0.375)}{70} = 0.182 \text{ in.}$$

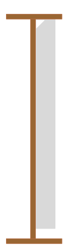
Try a $3/16$ in. weld, but only use a leg width of 0.182 in.

We are losing 0.006 in. of leg width.

83

Example 10

- Select welds for stiffener found in Ex. 9(a)



Strength of one line of welds on a double weld and stiffener

$$\begin{aligned} R_n &= (0.707w)(0.6F_{exx})l \\ &= (0.707(0.182))(0.6(70))(1.0 \text{ in.}) \\ &= 5.40 \text{ kips/in.} \end{aligned}$$

Required strength per line of weld

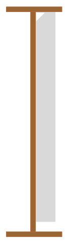
$$f_{nv\ required} = \frac{3.71}{2} = 1.86 \text{ kips/in.}$$

% of coninuous 3/16 in. weld required $= \frac{1.86}{5.40}(100) = 34.4\%$

84

Example 10

- Select welds for stiffener found in Ex. 9(a)



Minimum length of intermittent weld segment is 1.5 in. The strength of that segment is:

$$1.5R_n = 1.5(5.40) = 8.10 \text{ kips}$$

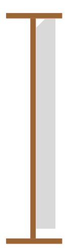
$$\text{Max pitch } = \frac{8.10}{1.86} = 4.35 \text{ in.}$$

However, the maximum clear spacing is $16t_w = 4.0$ in. which is a pitch of 5.5 in. Thus, the weld spacing is controlled by strength.

85

Example 10

- Consider a 1/8 in. weld



Strength of one line of welds on a double weld and stiffener

$$\begin{aligned} R_n &= (0.707w)(0.6F_{exx})l \\ &= (0.707(0.125))(0.6(70))(1.0 \text{ in.}) \\ &= 3.71 \text{ kips/in.} \end{aligned}$$

Required strength per line of weld (unchanged)

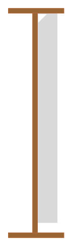
$$f_{nv\ required} = \frac{3.71}{2} = 1.86 \text{ kips/in.}$$

$$\text{\% of coninuous 3/16 in. weld required} = \frac{1.86}{3.71}(100) = 50.1\%$$

86

Example 10

- Consider a 1/8 in. weld



Minimum length of intermittent weld segment is 1.5 in. The strength of that segment is:

$$1.5R_n = 1.5(3.71) = 5.57 \text{ kips}$$

$$\text{Max pitch } = \frac{5.57}{1.86} = 2.99 \text{ in.}$$

The maximum clear spacing is $16t_w = 4.0$ in. which is a pitch of 5.5 in. Again, controlled by strength.
However, most fabricators would not use a weld smaller than 3/16 in.

87

Plate Girders

- Next consider the bearing stiffener at the unframed girder end.
  - J10. Flanges and Webs with Concentrated Forces
    2. Web Local Yielding
    3. Web Local Crippling
    4. Web Sidesway Buckling
    8. Additional Stiffener Requirements

88

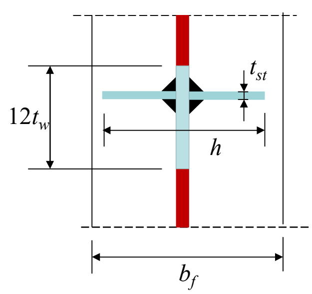
Plate Girders

- If the end of this plate girder is considered "an unframed end" then bearing stiffeners are required, regardless of the need for them based on the strength requirements of Sections J10.2, J10.3, and J10.4.
  - J10.7 Unframed Ends
    - pair of full depth stiffeners
  - J10.8 Additional Requirements
    - design as a column with $KL = 0.75h$
    - Include a portion of the web, $12t_w$ or $25t_w$
    - Stiffener dimensions (Not actually written for this use)
      - Stiffener width plus $t_w/2 > b_f/3$
      - Stiffener thickness $\geq t_f/2$ and $\geq b_f/16$

89

Example 11

- Determine the required Bearing Stiffener



Stiffener (an adaptation of J10.8a and b)

$h = 6.75 \geq 2b_f/3 = 2(10)/3 = 6.67$ in.

$t_{st} = 0.5 \geq t_f/2 = 1.0/2 = 0.5$ in.

$\geq h_{st}/16 = 6.75/16 = 0.42$ in.

Web

$12t_w = 12(.25) = 3.0$ in.^{2}

Column

$I = \dfrac{0.5(6.75)^3}{12} = 12.8$ in.^{4}

$A = 2(3.25)(0.5) + 3.0(0.25) = 4.0$ in.^{2}

$r = \sqrt{12.8/4.0} = 1.79$ in.

J4.4 Compression Strength

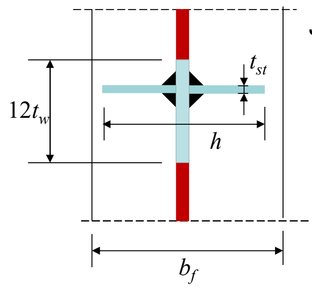
$\dfrac{KL}{r} = \dfrac{0.75(40)}{1.79} = 16.8 \leq 25$

$F_{cr} = F_y$

90

Example 11

- Determine the required Bearing Stiffener



J4.4 Compression Strength

Since the web and stiffener have different yield strengths, match area with strength.

$$P_n = F_y A_g$$
$$= 50(3.0)(0.25) + 36(2)(3.25)(0.5)$$
$$= 37.5 + 117 = 155 \text{ kips}$$
$$\phi P_n = 0.9(155) = 140 \text{ kips}$$

1.0 in. notch around corner weld

J7 Bearing Strength (without including web)

$$R_n = 1.8 F_y A_{pb} = 1.8(36)\left(2(3.25 - 1.0)(0.5)\right) = 146 \text{ kips}$$
$$\phi R_n = 0.75(146) = 110 \text{ kips}$$

91

Example 11

- From Example 10 the required bearing strength is 124 kips.
- The bearing stiffeners we have are more than adequate, without checking the limit states addressed in J10.2, J10.3, and J10.4.
- But we do need to include a bit of web for the bearing strength according to J7.

92

Example 11

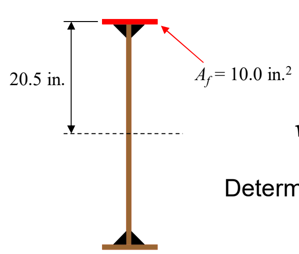
- Connection of web to flange
  - B4.1 (2022) requires that the web be continuously attached to the flange for the width-to-thickness limits in Table B4.1 to be applicable.
  - Effective weld for a 0.25 in. web to a 1.0 in. flange, A572 Gr 50, thus controlled by rupture:

$$w = 0.707\frac{F_u t}{F_{exx}} = 0.707\frac{65(0.25)}{70} = 0.164 \text{ in.}$$

93

Example 11

- Required horizontal shear strength at web flange intersection. V_u is a maximum 124 k.



$$Q = 10.0(20.5) = 205 \text{ in.}^3$$

$$I = 9740 \text{ in.}^4$$

$$v_u = \frac{V_u Q}{I} = \frac{124(205)}{9740} = 2.61 \text{ k/in.}$$

Determine the minimum required weld

Assuming that all the weld is required to carry is the shear force.

$$w = \frac{v_u}{2\phi(0.707)(0.6F_{exx})l}$$

$$= \frac{2.61}{2(0.75)(0.707)(0.6(70))(1.0)} = .059 \text{ in.}$$

Therefore, use a minimum 3/16 in. weld

94

Summary Lesson 2

- We looked at the shear strength of webs.
- Considered strength without TFA and strength with TFA.
- Selected intermediate stiffeners and the required welds
- Selected bearing stiffeners
- Determined web-flange weld requirements

95

Course Summary

- We have looked at the AISC 360-16 *Specification* requirements for flexural strength of built-up plate girders with webs that are not compact, Sections F3 and F4.
- We have looked at the AISC 360-16 *Specification* requirements for shear strength of built-up plate girders, Section G2.
- We have also looked at the requirements for welds.

96

Thank You

American Institute of Steel Construction
130 East Randolph St., Suite 2000
Chicago, IL 60601

97

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  - Password: Same as AISC website password.
- Once attendance has been reported, you will be receiving certificates via email from registration@aisc.org.

Smarter.
Stronger.
Steel.

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