There's always a solution in steel.

AISC Live Webinars

Thank you for joining our live webinar today.
We will begin shortly. Please stand by.

Thank you.

Need Help?
Call ReadyTalk Support: 800.843.9166

structural STEEL

AISC Live Webinars

Today's audio will be broadcast through the internet.

Alternatively, to hear the audio through the phone, dial:

(888) 378-4398   Passcode: 694105

AISC Live Webinars

Today's live webinar will begin shortly. Please stand by.

As a reminder, all lines have been muted. Please type any questions or comments through the Chat feature on the left portion of your screen.

Today's audio will be broadcast through the internet. Alternatively, to hear the audio through the phone, dial:

(888) 378-4398 Passcode: 694105

AISC Live Webinars

AISC is a Registered Provider with The American Institute of Architects Continuing Education Systems (AIA/CES). Credit(s) earned on completion of this program will be reported to AIA/CES for AIA members. Certificates of Completion for both AIA members and non-AIA members are available upon request.

This program is registered with AIA/CES for continuing professional education. As such, it does not include content that may be deemed or construed to be an approval or endorsement by the AIA of any material of construction or any method or manner of handling, using, distributing, or dealing in any material or product.

Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.

AISC Live Webinars

Copyright Materials

The information presented herein is based on recognized engineering principles and is for general information only. While it is believed to be accurate, this information should not be applied to any specific application without competent professional examination and verification by a licensed professional engineer. Anyone making use of this information assumes all liability arising from such use.

Course Description

Stiffeners and Doublers – Oh My!

July 12, 2018

This webinar will present tips for avoiding costly doubler and stiffener detailing, along with good engineering practices that can make a big difference in the success of projects. The webinar will present a relative cost comparison between using stiffeners and doublers versus increasing member sizes. The latest requirements regarding doubler and stiffener design responsibilities will also be presented.

Learning Objectives

- List locations within a building structure where stiffener and doubler plates are utilized.
- Describe the cost implications of using stiffener and doubler plate details versus using larger member sizes.
- Describe how the responsibility of stiffener and doubler detailing is assigned on the project using guidelines provided in the 2016 AISC *Code of Standard Practice for Steel Buildings and Bridges*. Explain how to communicate design responsibility in the contract documents.
- Explain how to properly design stiffener and doubler plates according to the 2016 AISC *Specification for Structural Steel Buildings*.

There's always a solution in steel.

Stiffeners and Doublers Oh My!

Carol Drucker, S.E., P.E., P.Eng.
Drucker Zajdel Structural Engineers, Inc.

structural STEEL

Stiffeners and Doublers Summary

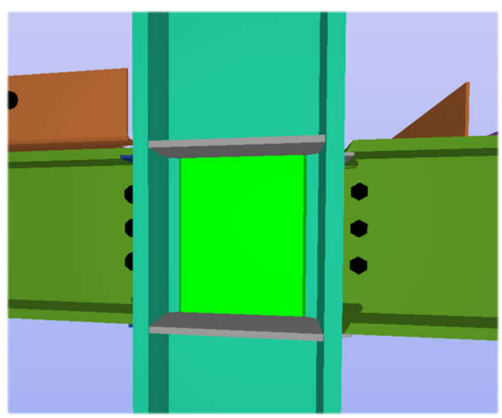
- What are doublers?
- When are they required?
- Doubler Cost
- COSP and Doublers
- Doubler Design
- Stiffener Applications
- Stiffener Design

Stiffeners and Doublers 9

What is a Doubler?

- A doubler is a plate at a member web used to reinforce a section for the required load

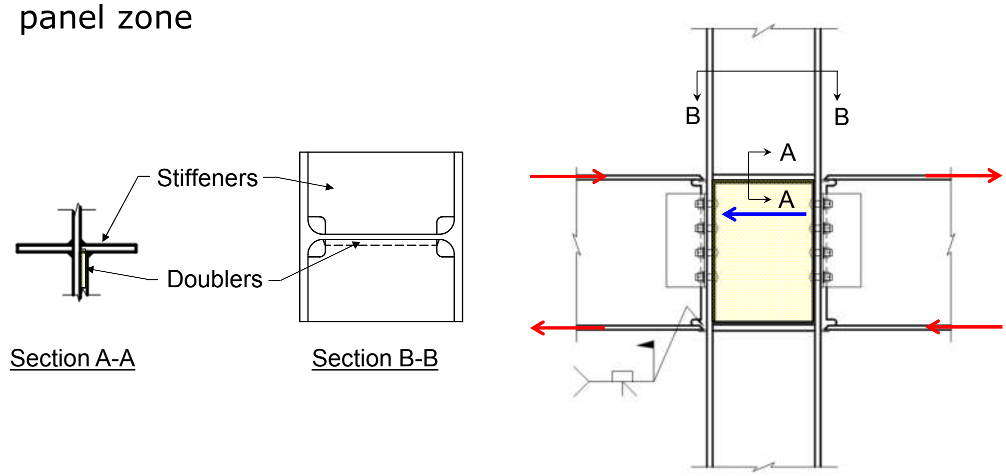
Courtesy of Lyndon Steel Company



Stiffeners and Doublers 10

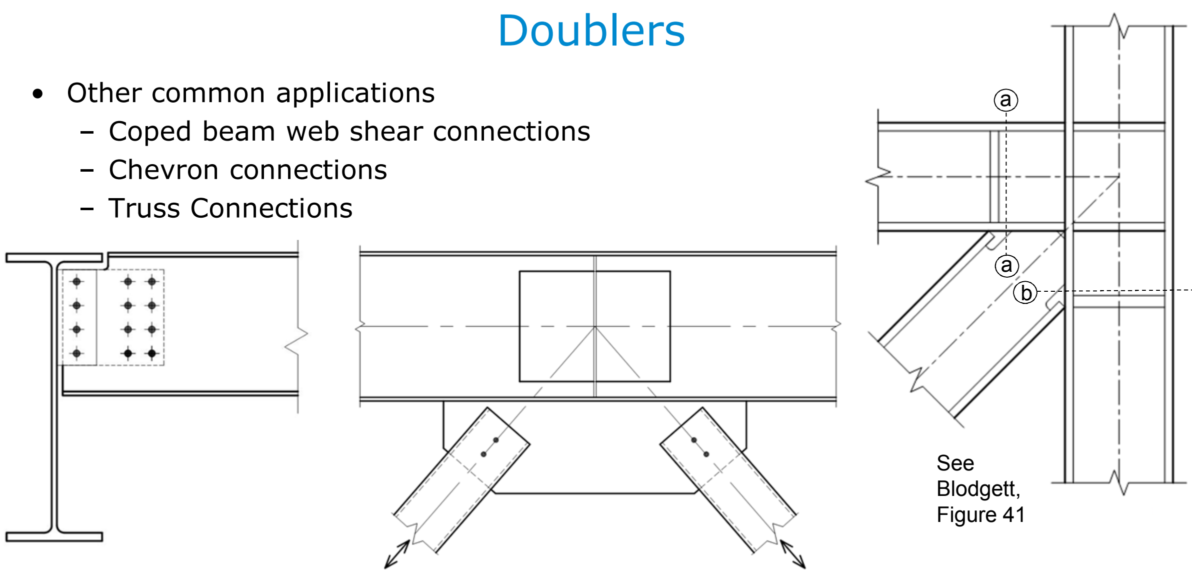
What is a Doubler?

- A doubler plate is typically used to reinforce a column web panel zone



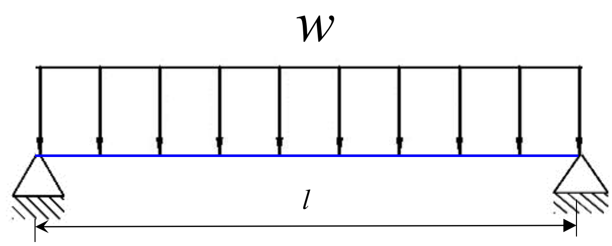
Doublers

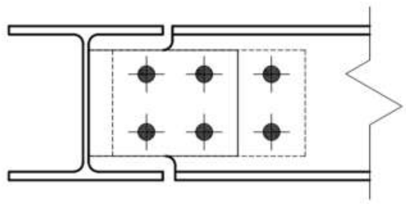
- Other common applications
  - Coped beam web shear connections
  - Chevron connections
  - Truss Connections



Why Doublers?

- Coped Beam to Girder
  - Shear Yielding, Shear Rupture
  - Block Shear
  - Flexural Yielding, Flexural Rupture
  - Flexural Buckling
  - See *Spec* Part J and Manual Chapter 9

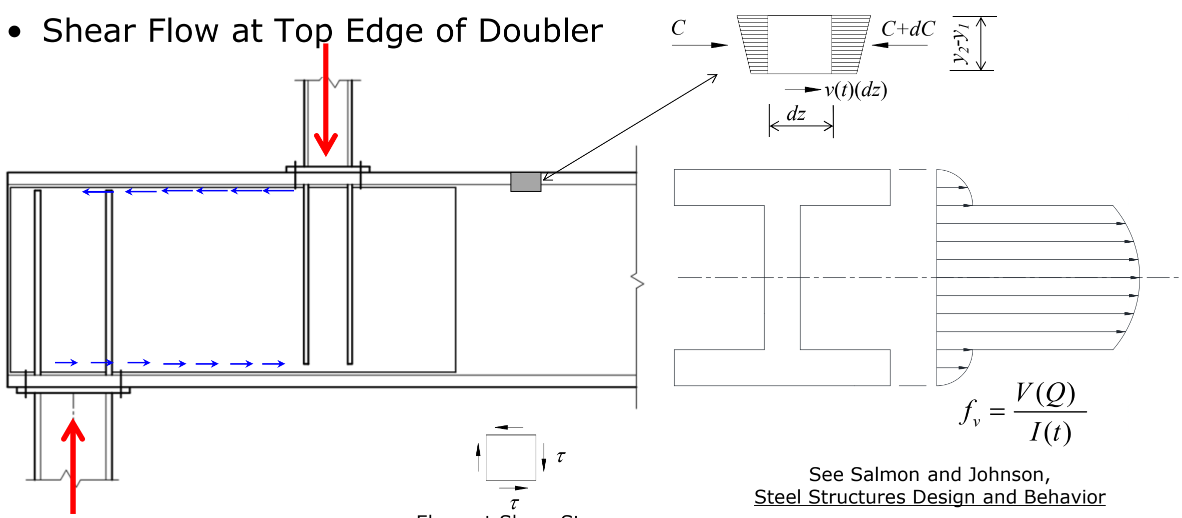




Doubler Force = Required Force – Strength of Section

Shear Force and Stress

- Shear Flow at Top Edge of Doubler

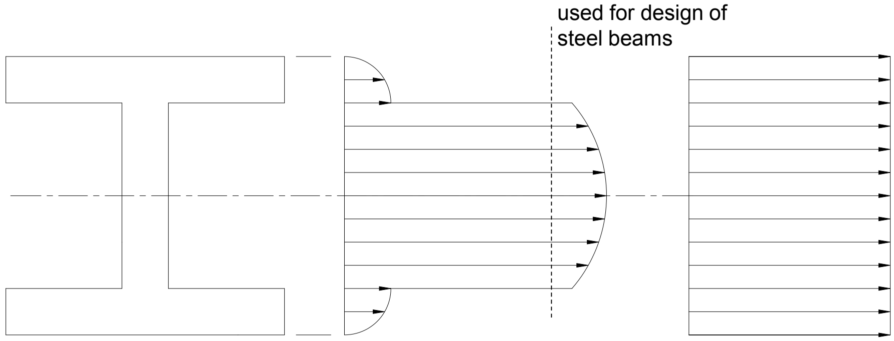


$$f_v = \frac{V(Q)}{I(t)}$$

See Salmon and Johnson,
Steel Structures Design and Behavior

Shear Force and Stress

- Shear Flow at Top Edge of Doubler

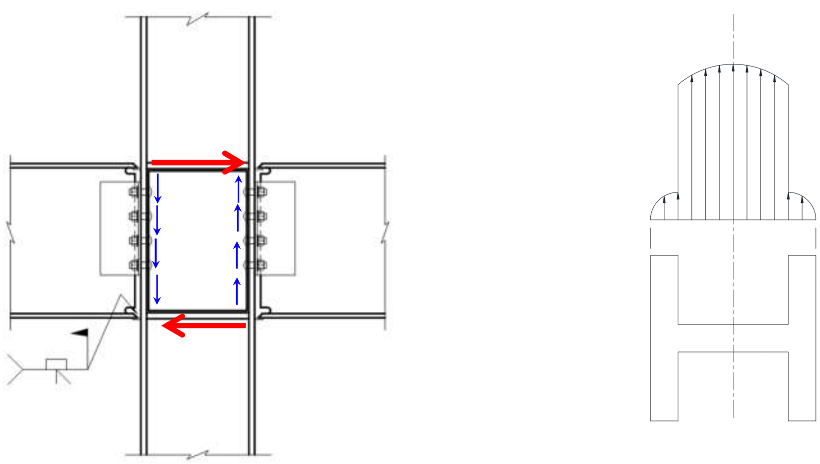


$$f_v = \frac{V(Q)}{I(t)}$$

$$f_{avg} = \frac{V}{(d)(t_w)}$$

Shear In a Member

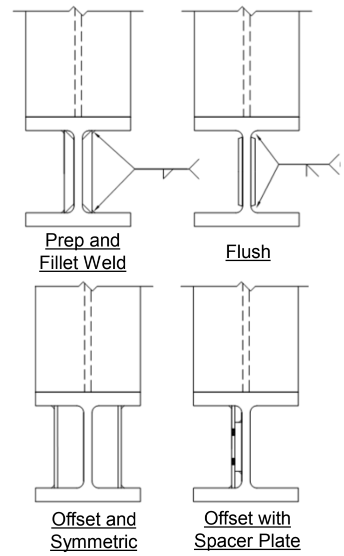
- Shear Flow at Top Edge of Doubler



$$f_v = \frac{V(Q)}{I(t)}$$

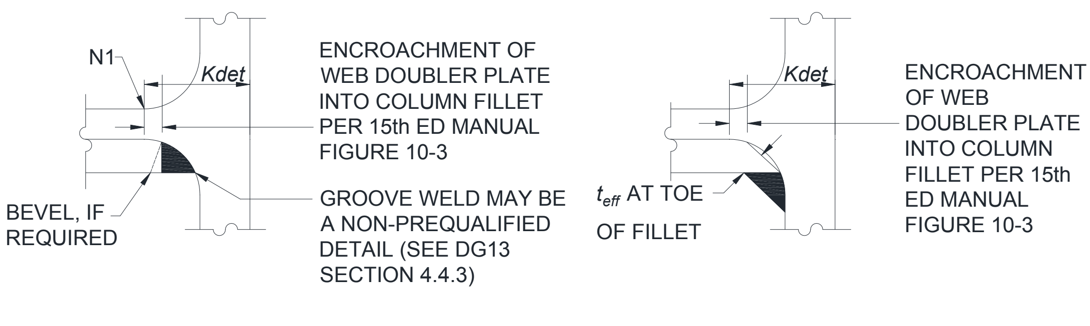
Doubler Configurations

- Doubler configurations commonly used: Prepped, Flush, and Offset
- For flush doublers: use (1) doubler if thickness ≤ 1/2" to 5/8".
- Doubler hierarchy (flush and prepped configurations):
  1. Place on side without shear connection
  2. Place on side with smaller beam framing into column web
- Consider shear beam framing into doubler



Doublers

- Design Guide 13



(A) GROOVE-WELDED DETAIL
N1. RIDING THE *K* CAUSES A GAP BETWEEN THE DOUBLER AND FACE OF WEB

(B) FILLET-WELDED DETAIL WITH PLATE BEVEL EQUAL TO PLATE THICKNESS

Doubler Prep

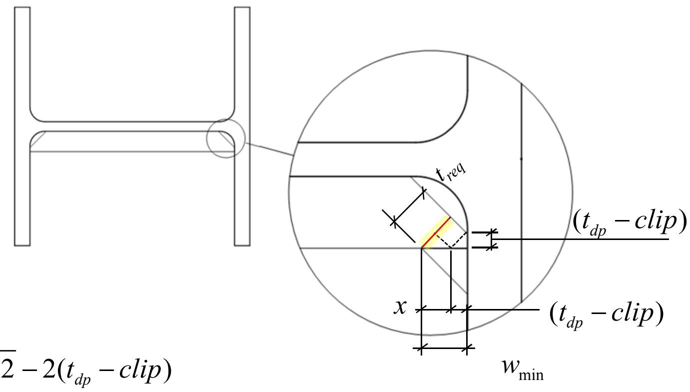
- Design Guide 13

$$D_{\min} = \frac{0.6F_y(t_{req})}{1.392}$$

$$w_{\min} \geq t_{req}\sqrt{2} - (t_{dp} - clip)$$

$$D_{req} = \max(D_{min}, w_{min}(16))$$

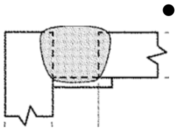
$$x = \left(t_{req} - (t_{dp} - clip)\sqrt{2}\right)\sqrt{2} = t_{req}\sqrt{2} - 2(t_{dp} - clip)$$



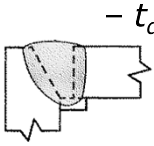
Stiffeners and Doublers 19

Flush Doublers: DG13

- Not a prequalified CJP weld
- Design Guide 13
  - t_{dp} =1/4" to 3/8" max:

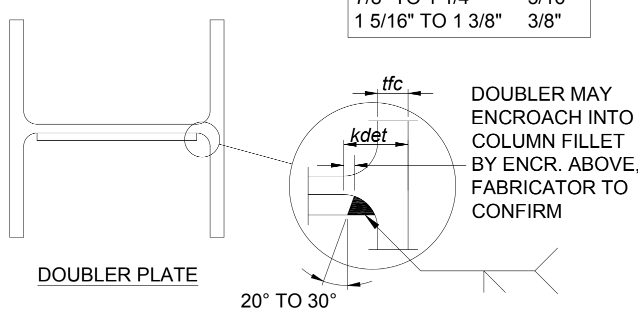


    - C-L1a or C-L1a-GF with square cut edges (or beveled)
  - t_{dp} > 1/4" to 3/8":



    - TC-U4a with beveled plates edges

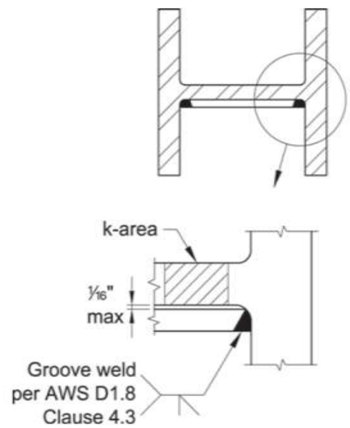
kdet - tfc	ENCR.
5/16"	1/8"
3/8" TO 1/2"	3/16"
9/16" TO 13/16"	1/4"
7/8" TO 1 1/4"	5/16"
1 5/16" TO 1 3/8"	3/8"



Stiffeners and Doublers 20

Flush Doubler: Seismic Provisions

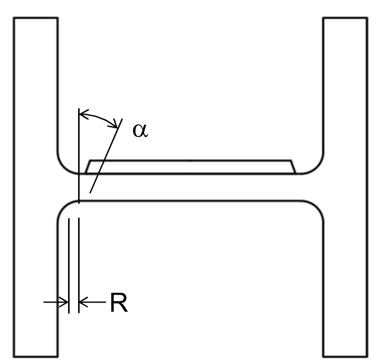
- Prequalified doubler weld added to 2016 AWS D1.8/D1.8M Clause 4.3
- AISC 341-16 Seismic Provisions designate this weld as a "full strength" PJP Groove Weld (See Comm E3)
  - Routine ultrasonic testing is not required
  - 1/16-in. gap permitted between doubler and column web
  - Required strength of weld = available shear yielding strength of doubler



(a) Doubler plate groove welded to column flanges

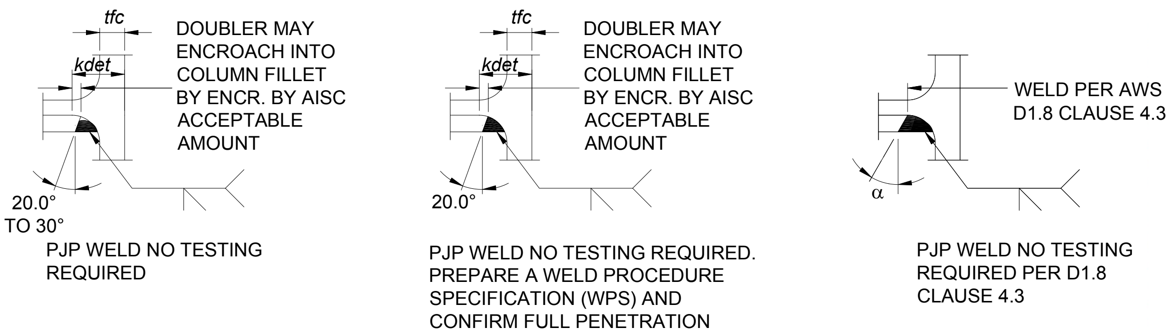
Flush Doubler: AWS D1.8/D1.8M :2016

Doubler to Column Flange Joint Full Shear Yielding Strength PJP Detail Per Figure 4.3 AWS D1.8/D1.8M			
Welding Process	**Joint Designation**	**Root R, in.**	**α Degrees**
SMAW	Dblr	0	20
GMAW FCAW	Dblr-GF	0	30
SAW	Dblr-S	0	30



*AWS D1.8/D1.8M Clause 4.3: Decreased groove angle (α) and decreased root (R) require 3 macroetch test

Flush Doubler Welds at Column Radius



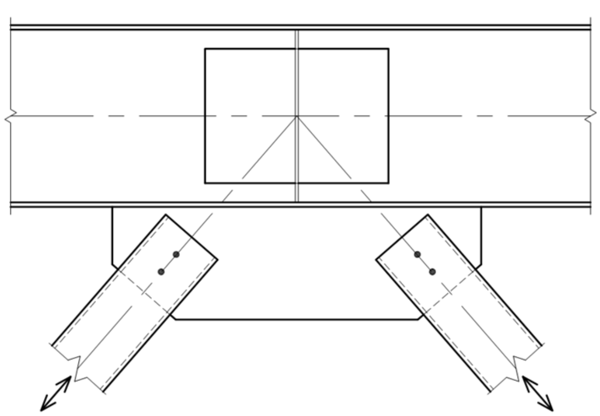
ϕRweld=(0.75)(0.6)(F_{exx})(E)

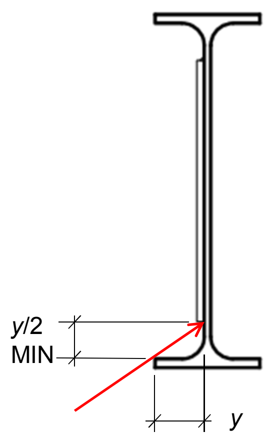
ϕRweld= FULL SHEAR STRENGTH

ϕRweld= FULL SHEAR STRENGTH

Shear In a Member

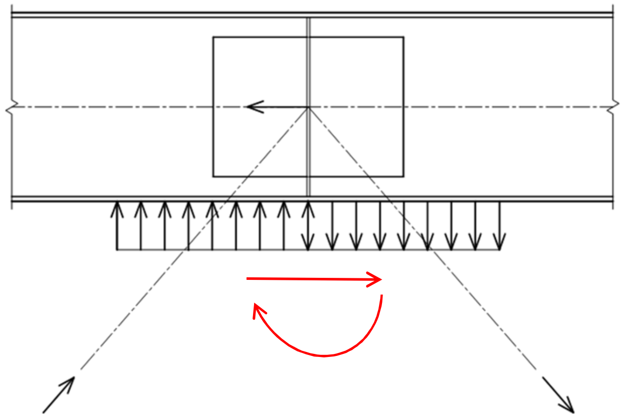
- Exceptions at Chevron





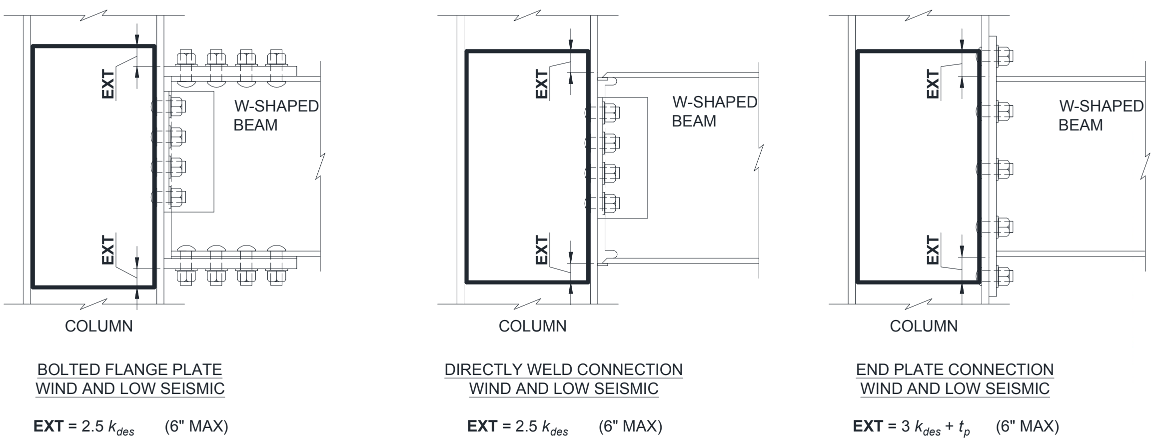
Shear In a Member

- Exceptions at Chevron



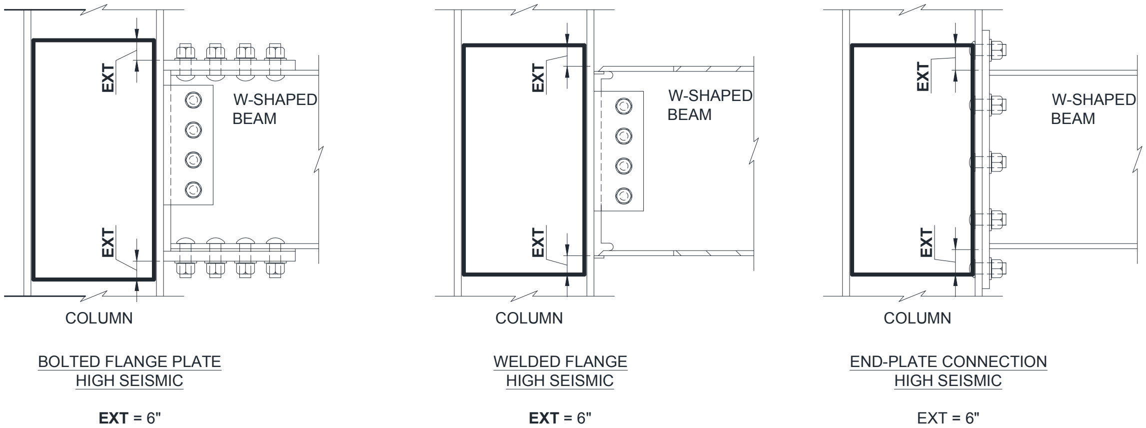
Doubler Extension

- Doubler Extension



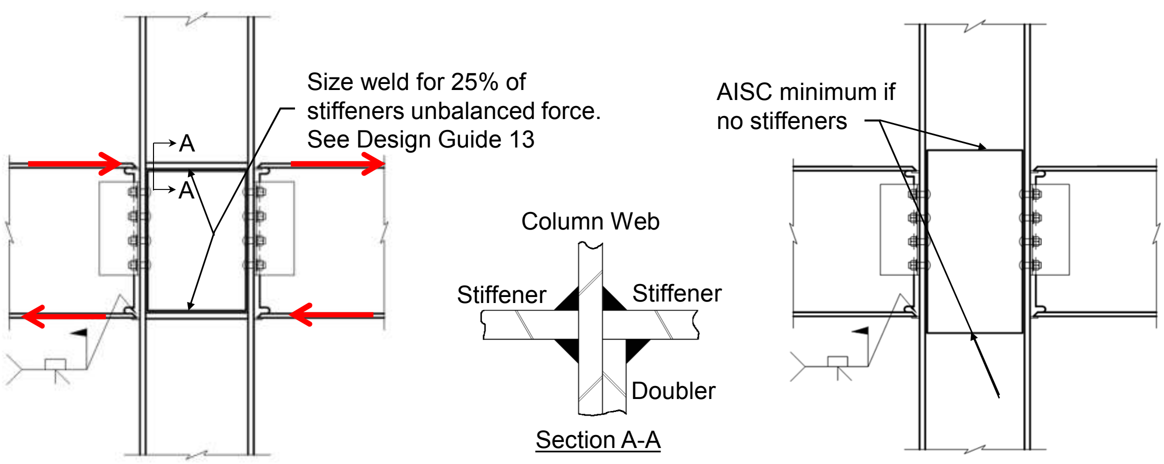
Doubler Extension Seismic

- Doubler Extension



Wind and Low Seismic

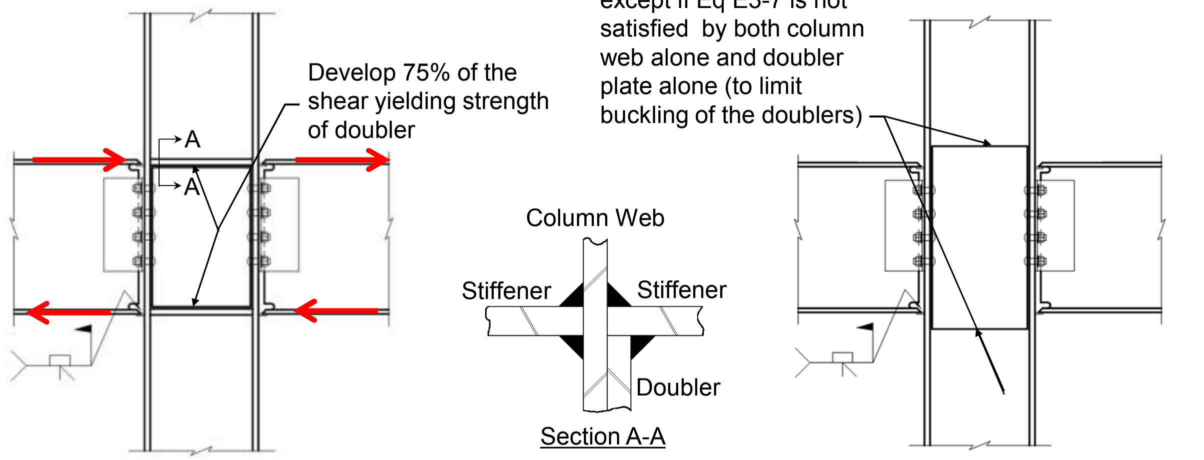
- Doubler Termination



High Seismic

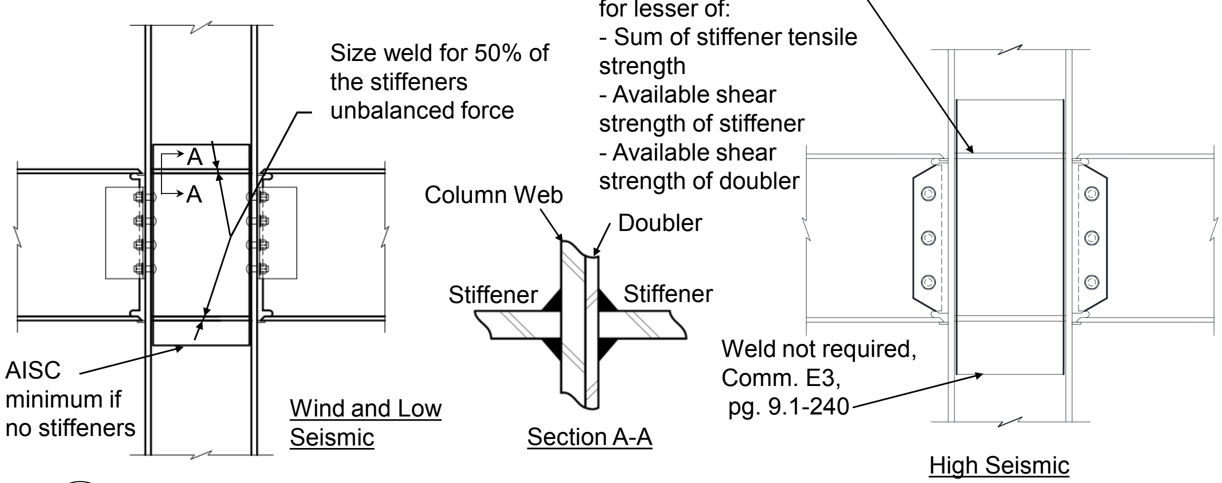
- Doubler Termination

$$t \geq \frac{(d_z + w_z)}{90} \qquad \text{Eq. E3-7}$$



Continuous Doublers

- Doubler Termination



Cost of Doublers

- Rule of Thumb (Per Seismic Design Manual): Upsizing columns between 50 to 100 lb/ft might still be more cost effective than installing doubler plates and continuity plates.
- Cost of Steel = \$800/ton average (FOB mill) (\$881 for plate) http://stld-cci.com/
- Platts S&P Global is also a good source for price of steel

PRICE LIST

Steel Dynamics®
Steel Dynamics Sales North America, Inc.
Long Products Group
Structural and Rail Division
Columbia City, Indiana USA

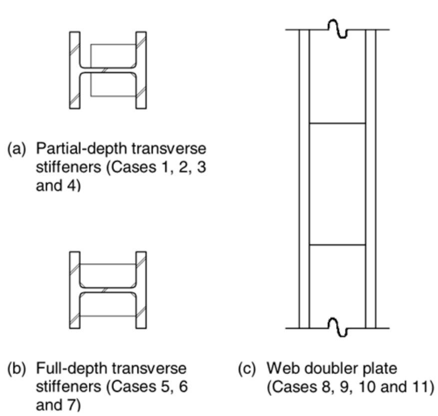
January 9, 2018
Truck Fuel Surcharge January = 16%
Truck Fuel Surcharge February = 17%
Rail Fuel Surcharge January & February = 0%

Wide Flange Sections

Size	WT/FT	Price/cwt*
W4X4	13	\$36.00
W5X5	16-19	\$46.25
W6X4	8.5	Inquire
W6X4	9-16	\$36.00
W6X6	15-25	\$36.00
W8X4	10-15	\$36.00
W8X5 1/4	18-21	\$36.00
W8X6 1/2	24-28	\$36.00
W8X8	31-67	\$36.00
W10X4	12-19	\$36.00
W10X5 3/4	22-30	\$36.00
W10X8	33-45	\$36.00
W10X10	49-112	\$36.00
W12X4	14-22	\$36.00
W12X6 1/2	26-35	\$36.00
W12X8	40-50	\$36.00
W12X10	53-58	\$36.00
W12X12	65-136	\$39.50
W12X12	152-210	\$44.50
W12X12	230, 252	\$49.50
W14X5	22-26	\$36.00
W14X6 3/4	30-38	\$36.00
W14X8	43-53	\$36.00
W14X10	61-82	\$36.00
W14X14 1/2	90-132	\$40.00
W14X16	145-283	\$51.75
W16X5 1/2	26-31	\$36.00
W16X7	36-57	\$36.00
W16X10 1/4	67-100	\$36.00
W18X6	35-46	\$36.00
W18X7 1/2	50-71	\$36.00
W18X11	76-211	\$41.75

*cwt=hundredweight: 100 lbs
Multiply by 20 for \$/ton

Cost of Doublers – DG13 (1999)



Note: dimensions and edge connections for the above column stiffening elements are as given in Table 3.1, based upon a W14 column.

Figure 3-1 Column stiffening arrangements for cost estimates in Table 3.1.

Table 3.1
Estimated Cost of Various Column Stiffening Details (as illustrated in Figure 3-1)

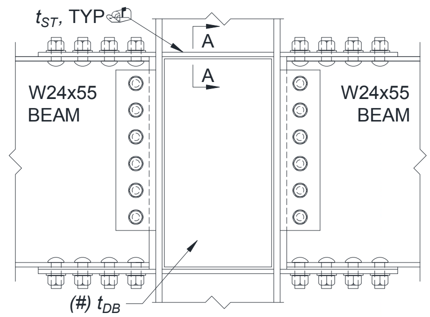
Case	Thickness	Attachment to Column Flange	Attachment to Column Web	Estimated Cost	Equivalent Column Weight (lb/ft) if Wide-Flange Steel Costs \$425 per Ton from Rolling Mill[3]
Partial-Depth Transverse Stiffeners (Two Pairs) 4 PL 4½ × 0'-10 (ASTM A36) with one ¾ × ¾ corner clip each					
1	½ in.	fitted to bear	3/16-in. fillet welds	\$80	27
2	1 in.	fitted to bear	5/16-in. fillet welds	\$120	40
3	½ in.	¼-in. fillet welds	3/16-in. fillet welds	\$90	30
4	1 in.	½-in. fillet welds[1]	5/16-in. fillet welds	\$140	47
Full-Depth Transverse Stiffeners (Two Pairs) 4 PL 4½ × 1'-0 9/16 (ASTM A36) with two ¾ × ¾ corner clips each					
5	½ in.	¼-in. fillet welds	3/16-in. fillet welds	\$120	40
6	1 in.	½-in. fillet welds	5/16-in. fillet welds	\$210	71
7	1½ in.	CJP groove weld	½-in. fillet welds[1]	\$470	158
Web Doubler Plate (One) 1 PL 12⅝ × 2'-0 (ASTM A36)					
8	½ in.	CJP groove weld	3/16-in. fillet welds	\$245	82
9	¾ in.	CJP groove weld	5/16-in. fillet welds	\$370	124
10	¾ in.	⅝-in. fillet weld[2]	5/16-in. fillet welds	\$215	72
11	1 in.	⅞-in. fillet weld[2]	5/16-in. fillet welds	\$305	103

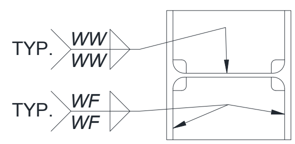
[1] The consulted fabricators were asked if they would instead prefer a CJP-groove-welded detail in place of this larger-size fillet-welded detail. In all cases, the answer was no.
[2] A ¾-in. by ¾-in. bevel on the column-flange edges of the web doubler plate is used to clear the column flange-to-web fillet. It should be noted that the fillet-welded web doubler plate detail in Case 10 is not suitable for high seismic applications because the weld size does not develop the strength of the full thickness of the web doubler plate.
[3] A floor-to-floor height of 14 ft has been used in this tabulation.

Cost of Doublers

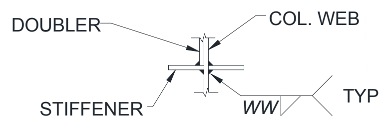
- LRFD, Grade 50 Plates
- W14x90 Column
- W24x55 Moment Connected Beams
- M = 350 kip-ft
  - 1/2" stiffeners required
  - 1/2" doubler required
- Upsize to W14x109 Column
  - Stiffeners not required
  - 3/8" doubler required
- Upsize to W14x193
  - Stiffeners not required
  - Doubler not required

NOTES:
1. GR50 STEEL
2. 14 FT FLOOR-TO-FLOOR





STIFFENERS

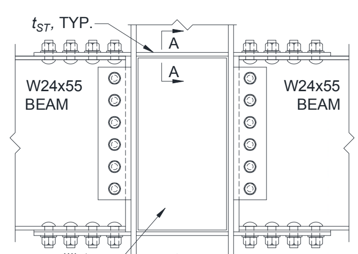


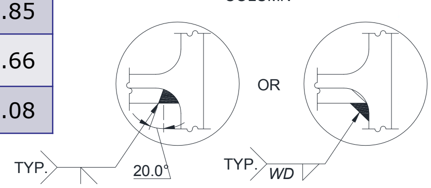
SECTION A

Cost of Doublers

Cost Comparison for Different Column Sizes								
Column	Stiffeners			Doublers				Cost Ratio
	t_{ST}	Flange Weld ***WF***	Web Weld ***WW***	#	t_{DB}	Doubler Weld ***WD***	Weld Det.	
W14x90	1/2"	5/16"	5/16"	1	1/2"	-	A	1.00
W14x90	1/2"	5/16"	5/16"	1	1-1/2"	3/8"	B	0.75
W14x109	-	-	-	1	3/8"	-	A	0.85
W14x109	-	-	-	1	1-1/2"	5/16"	B	0.66
W14x193	-	-	-	-	-	-	-	1.08

NOTES:
1. GR50 STEEL
2. 14 FT FLOOR-TO-FLOOR





DETAIL (**A**)
GROOVE WELD

DETAIL (**B**)
FILLET WELD

Who Checks for Doublers?

- Code of Standard Practice
  - 2005 and 2010 COSP Section 3.1.1

3.1.1 Permanent bracing, column stiffeners, **column web doubler plates**, bearing stiffeners in beams and girders, web reinforcement, openings for other trades and other special details, where required, **shall be shown in sufficient detail in the Structural Design Drawings** so that the quantity, detailing and fabrication requirements for these items can be readily understood.

Stiffeners and Doublers 35

Who Checks for Doublers?

- Code of Standard Practice
  - 2005 and 2010 COSP Section 1.1

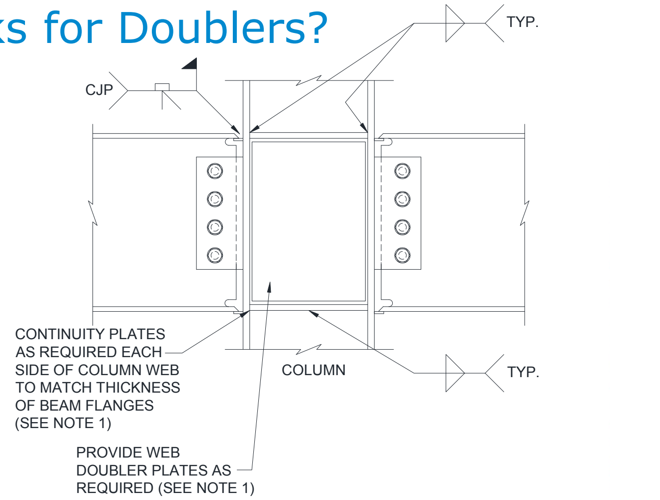
1.1 **In the absence of specific instructions to the contrary** in the Contract Documents, the trade practices that are defined in this Code shall govern the fabrication and erection of the Structural Steel.

Stiffeners and Doublers 36

Who Checks for Doublers?

- Specific instructions to the contrary:
  - Contract documents can modify the COSP. For example, the Specification can indicate:

"AISC's COSP applies with the following modifications: Where reaction forces are given on the design drawings, Fabricator is responsible for designing stiffeners and/or doublers if necessary."

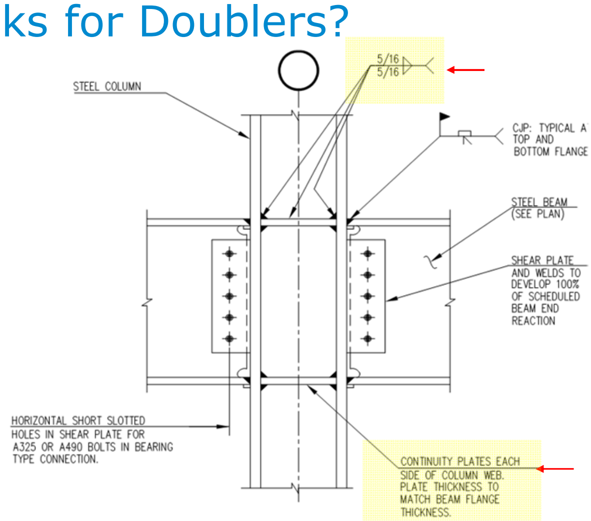


NOTES:
1. PER THE PROVISIONS OF CHAPTER J OF THE AISC SPECIFICATIONS, PROVIDE COLUMN FLANGE STIFFENERS WHERE REQUIRED TO RESIST FORCES FROM MOMENT CONNECTIONS. PROVIDE COLUMN WEB DOUBLER PLATES AS REQUIRED FOR UNBALANCED MOMENT. ASSUME 20 KSI AXIAL STRESS IN COLUMNS FOR THE PANEL ZONE CHECKS

Stiffeners and Doublers 37

Who Checks for Doublers?

- Misleading Information:
  - Detail indicates stiffeners to be checked but not doubler
  - No loads given
  - No indication that Fabricator is to check for doublers on drawings or in Spec



1 TYPICAL BEAM TO COLUMN FLANGE MOMENT CONNECTIO
SCALE: NOT TO SCALE

Stiffeners and Doublers 38

Who Checks for Doublers?

- Current Code of Standard Practice
  - 2016 COSP (303-16) Section 3.1.2
    - » Option 3A: Member reinforcement (e.g. stiffeners, doubler, etc.) shall be designed by the **owner's designated representative** and shown in the structural design documents issued for bid
    - » Option 3B: Member reinforcement at connections is delegated design, **but the quantities and conceptual configurations shall be provided and relied upon for bidding purposes**. If no quantities or conceptual configurations are shown, member reinforcement at *connections* will not be included in the bid.

Note: For Option 1 and 2, EOR designs stiffeners and doublers

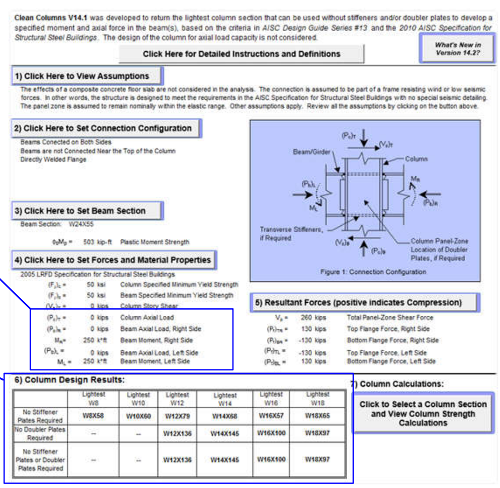
Check for Doublers
Determine Column Panel Zone Shear Strength

- AISC Clean Column: www.aisc.org/cleancolumn

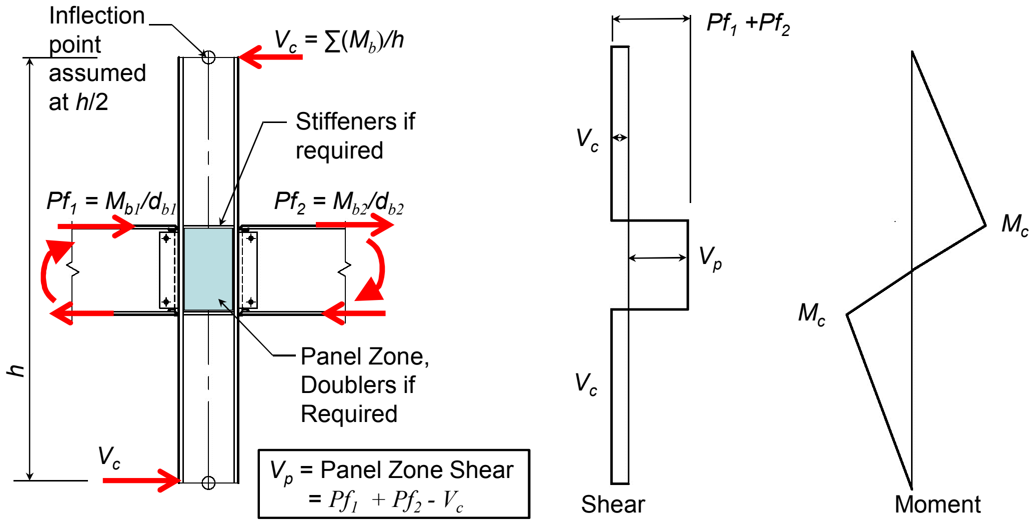
$(P_c)_T$ =	0 kips	Column Axial Load
$(P_b)_R$ =	0 kips	Beam Axial Load, Right Side
M_R=	250 k*ft	Beam Moment, Right Side
$(P_b)_L$ =	0 kips	Beam Axial Load, Left Side
M_L =	250 k*ft	Beam Moment, Left Side

6) Column Design Results:

	Lightest W8	Lightest W10	Lightest W12	Lightest W14	Lightest W16	Lightest W18
No Stiffener Plates Required	W8X58	W10X60	W12X79	W14X68	W16X57	W18X65
No Doubler Plates Required	--	--	W12X136	W14X145	W16X100	W18X97
No Stiffener Plates or Doubler Plates Required	--	--	W12X136	W14X145	W16X100	W18X97

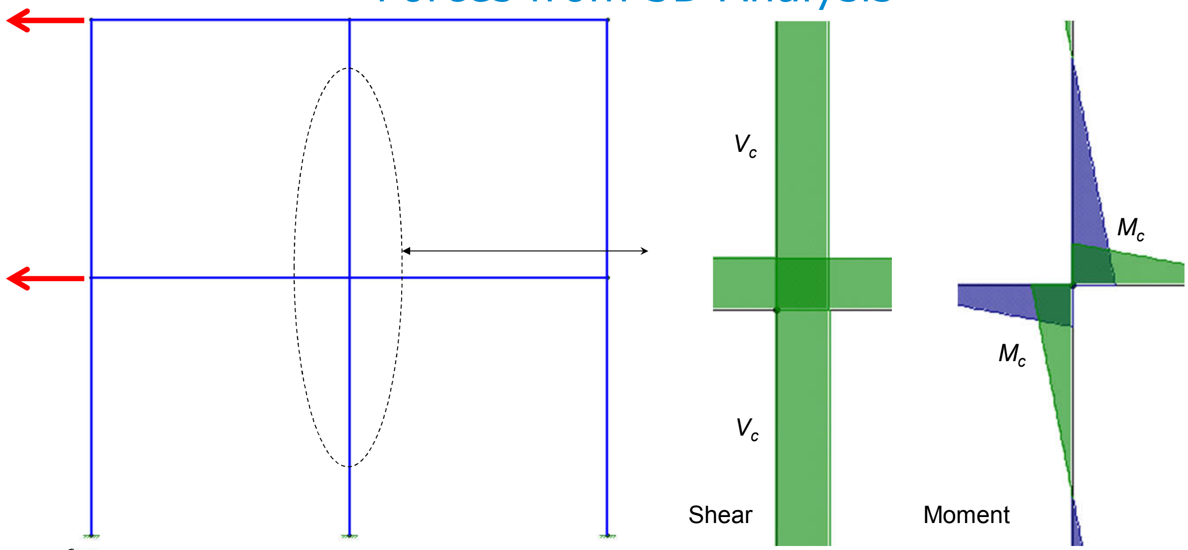


Doublers at Lateral Moment Connections - FBD



Stiffeners and Doublers 41

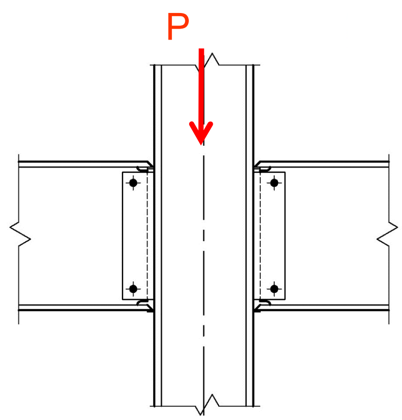
Forces from 3D Analysis



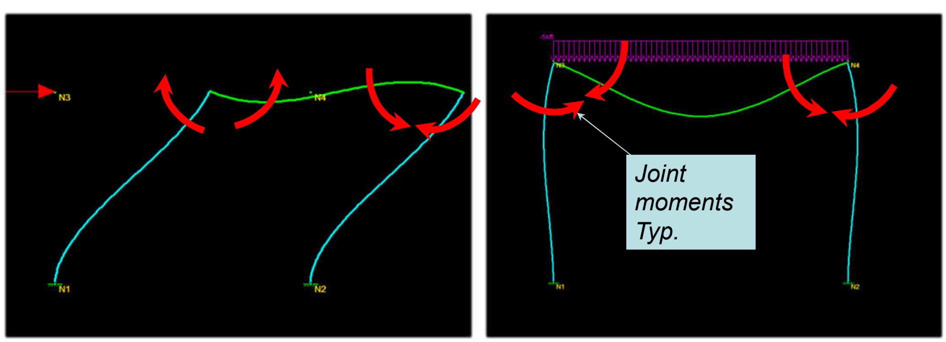
Stiffeners and Doublers 42

Check for Doublers
Determine Column Panel Zone Shear Strength

- Must Have
  - Beam Moments
- Best to Have
  - Break Down of Beam DL, LL, and Lateral Moments
  - Column Axial Load



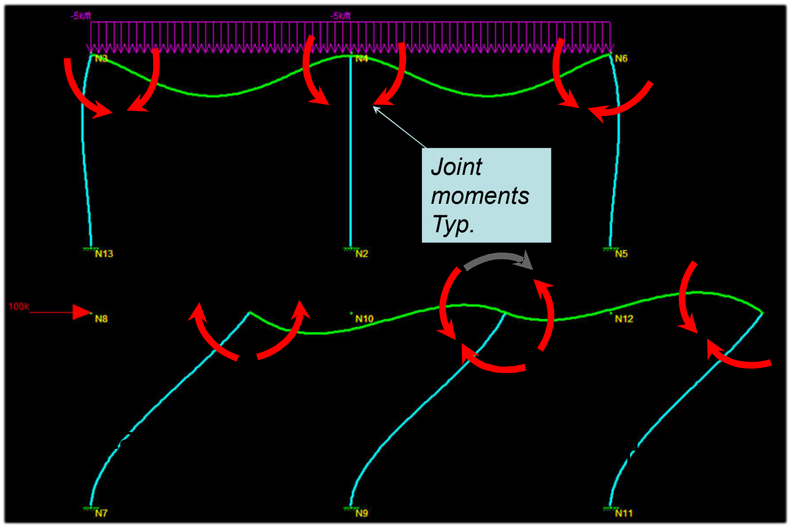
Deflected Shape



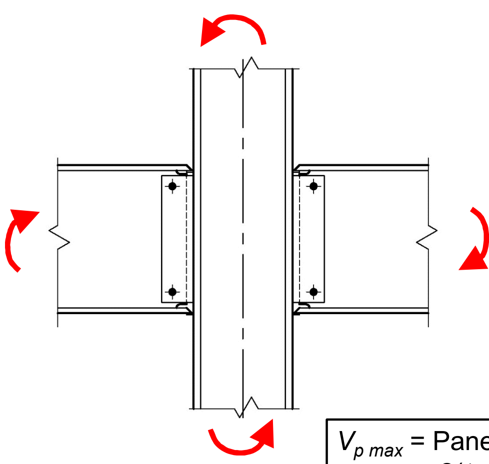
Lateral

Gravity

Deflected Shape



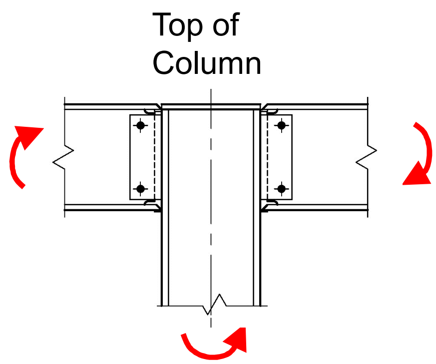
Moment Connections – Doublers



- $\Sigma M = 0$
  $\therefore \Sigma(M_b) \leq \Sigma(\phi M_c)$
- Sum of beam moments can not exceed sum of column moment strengths

$V_{p\ max}$ = Panel Zone Shear
= $2(\phi M_c)/d_b - 2(\phi M_c)/h$

Moment Connections – Doublers

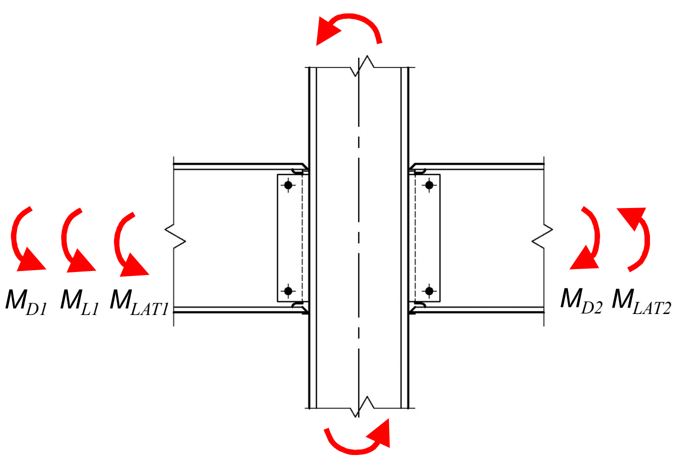


- $\Sigma M = 0$
  $\therefore \Sigma(M_b) \le \phi M_c$
- At top of column, sum of beam moments can not exceed column moment strength

$V_{c\,max}$ = Panel Zone Shear
= $\phi M_c/d_b$

Stiffeners and Doublers 47

Moment Connections – Doublers



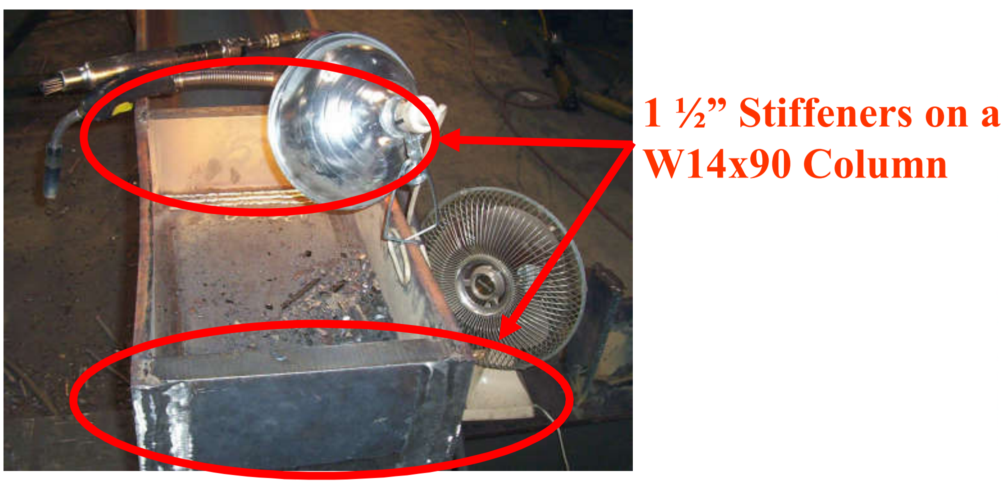
- Add dead and lateral loads on both sides
- Add live load on one side

Stiffeners and Doublers 48

Moment Connections – Doublers

Column Web Doublers	
Loading Criteria	% Columns Requiring Doublers
Full Moment Beam, Limited to Strength of Column	100%
Actual Moments Given with No Break Down	79%
Actual Moments Given with Break Down	26%

Give Actual Loads



Courtesy of Larry Muir and Bill Thornton

Check for Doublers
Determine Column Panel Zone Shear Strength, J10.6

$\phi = 0.9$

$\Omega = 1.67$

- Elastic Range:

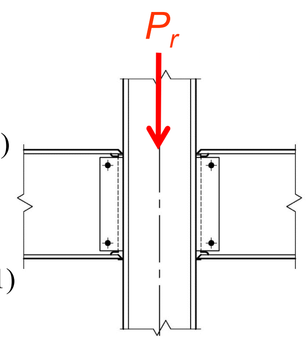
$$\alpha P_r \le 0.4P_y : \ R_n = 0.6F_y d_c t_w \qquad \text{(J10-9)}$$

$$\alpha P_r > 0.4P_y : \ R_n = 0.6F_y d_c t_w \left(1.4 - \frac{\alpha P_r}{P_y}\right) \qquad \text{(J10-10)}$$

- Plastic Range:

$$\alpha P_r \le 0.75P_y : \ R_n = 0.6F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w}\right) \qquad \text{(J10-11)}$$

$$\alpha P_r > .75P_y : \ R_n = 0.6F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2\alpha P_r}{P_y}\right) \qquad \text{(J10-12)}$$



- http://risa.com/news/code-requirements-for-panel-zone-shear-deformation/

Check for Doublers
Determine Column Panel Zone Shear Strength

- Force in Doubler

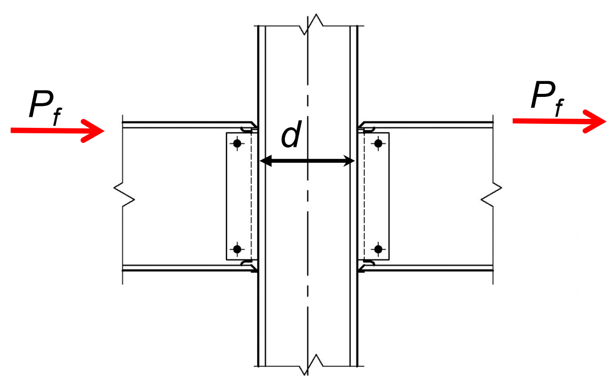
$V_{doubler} = \Sigma P_f - V_c - \phi R_n$ LRFD

$V_{doubler} = \Sigma P_f - V_c - R_n/\Omega$ ASD

- Size of Doubler
  - AISC *Specification* Eq. G2-1
    Use "*d*" of column

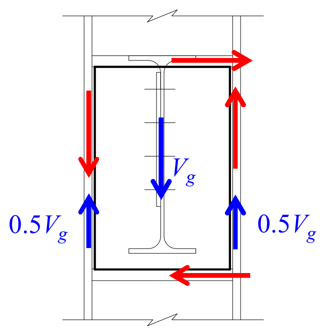
$$\phi V_n = \phi(0.6F_y A_w C_{v1})$$

$$V_n/\Omega = (0.6F_y A_w C_{v1})/\Omega$$

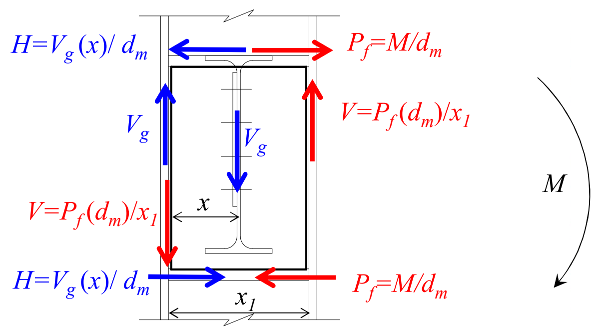


Check for Doublers
Determine Column Panel Zone Shear Strength

- Consider beam shear when beam frames into doubler



Alternate 1 Distribution



Alternate 2 Distribution, DG13

Doubler Web Buckling

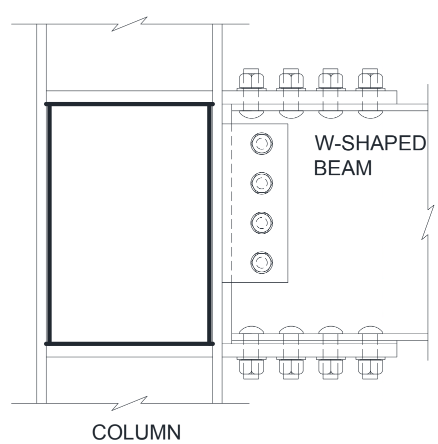
- Wind and Low Seismic Doublers: Minimum thickness to prevent shear buckling (*Spec* Section G2):

$$\frac{h}{t} \leq 2.24\sqrt{E/F_y} \quad \Rightarrow \quad t \geq = \frac{h\sqrt{F_y}}{381}$$

$$h = d_c - 2k_{des}$$

t = thickness of web doubler

Alternatively, use plug welds and the total thickness (t= thickness column web + thickness doublers) can be used

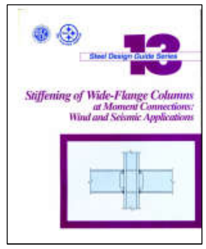


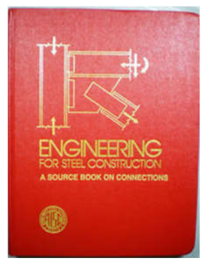
Doubler Web Buckling

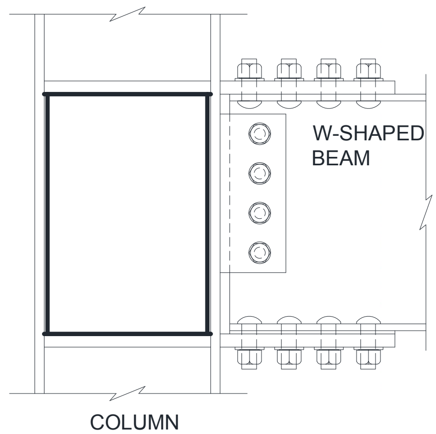
- DG13 is per 1993 *Spec* which ϕ = 0.9 versus ϕ = 1.0 in 2016 *Spec* for Shear

$$t \geq \frac{h\sqrt{F_y}}{418}$$ 1993 LRFD, Section F2

$$t \geq \frac{127}{\sqrt{F_y}}$$ Engineering for Steel Construction, 1984 Based on 1978 *Spec* Section 1.18.2.3 for Compression of Built-up Members







Doubler Web Buckling

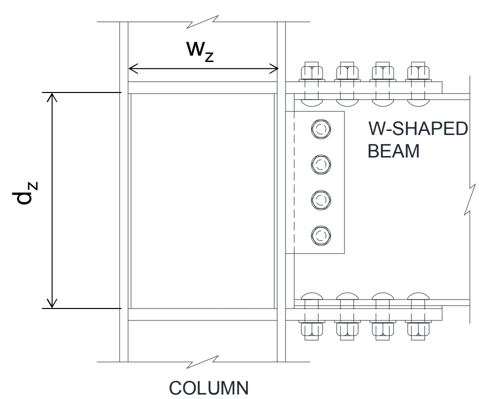
- High Seismic Doublers

$$t \geq \frac{(d_z + w_z)}{90}$$

$$d_z = d_b - 2t_{bf}$$

$$w_z = d_c - 2t_{cf}$$

Where:
t = thickness of column web or individual doubler

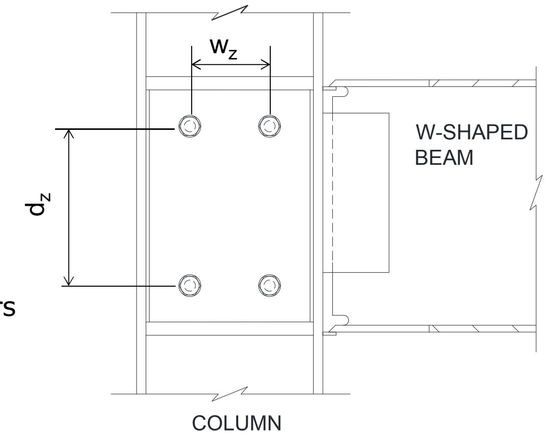


Doubler Web Buckling

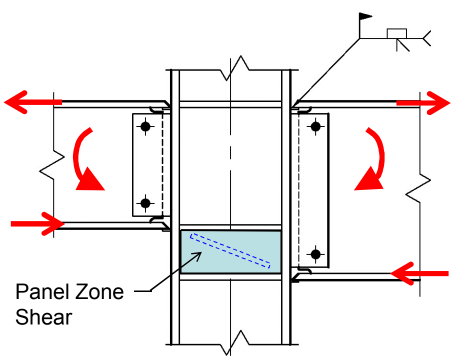
- High Seismic Doublers with plug welds

$$t \geq \frac{(d_z + w_z)}{90}$$

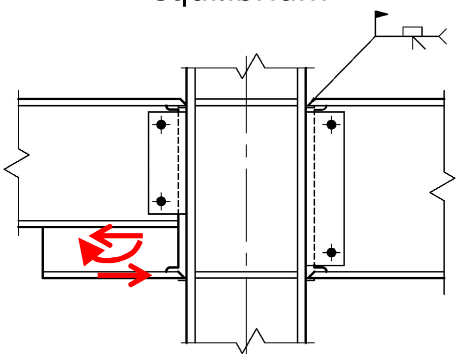
Where:
t = thickness of column web
+ thickness of individual doublers



Moment Connections



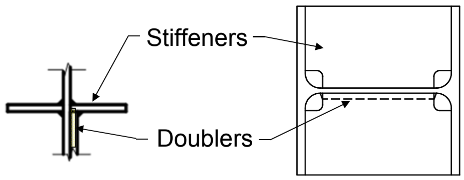
- Consider load path and equilibrium



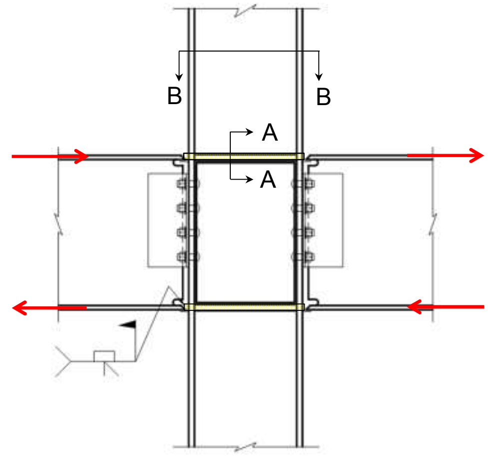
- Same depth beams at cantilevers with equal and opposite moments do not need to be checked for doublers

Stiffeners

- Stiffener Plates

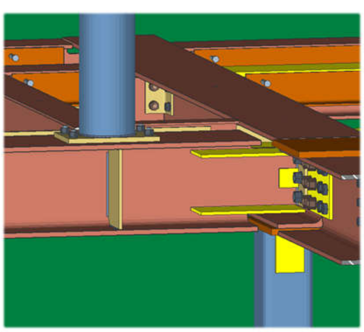


Section A-A   Section B-B

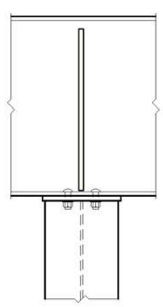


Stiffeners

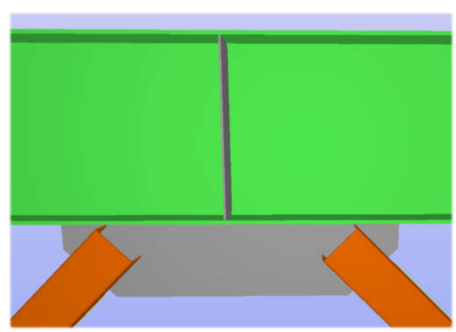
- Other Common Applications



Column Bearing on Beam



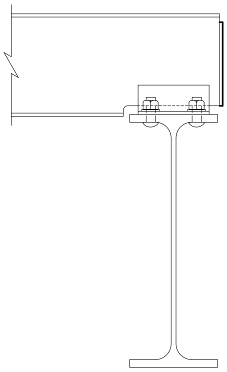
Beam over Top of Column



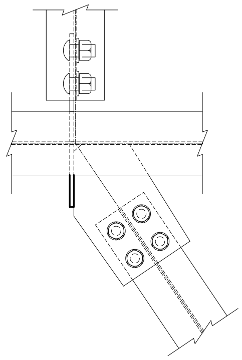
Chevron Connections

Stiffeners

- Other Common Applications



Stability
(J10.7)



To Direct Force

Stiffeners

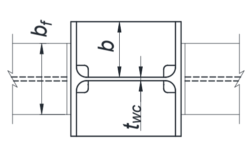
- Stiffeners, *Spec* J10.8
  - Minimum width, b:

$$b \geq \frac{b_f}{3} - \frac{t_{wc}}{2}$$

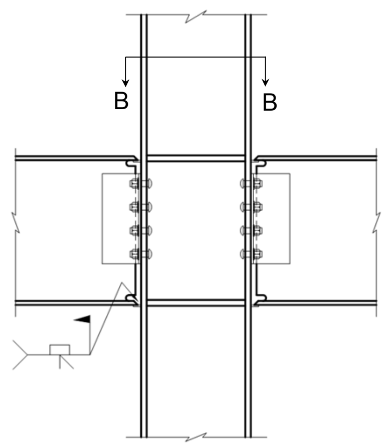
  - Minimum thickness, t:

$$t \geq \frac{t_f}{2}$$

$$t \geq \frac{b}{16}$$



Section B-B



Where: t_f (thickness) and b_f (width) are for the element delivering the force

Stiffeners

- Limiting Stiffener Slenderness
  - Minimum width, $t \geq b/16$:
    - For plates with one free edge:

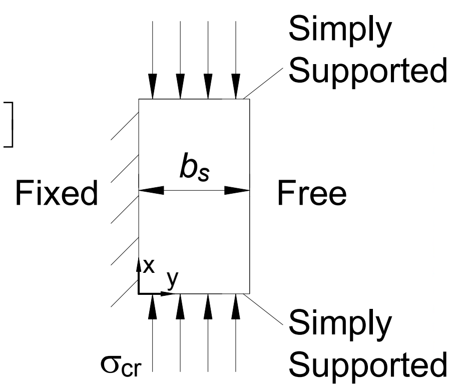
$$\sigma_{cr} = \left(\frac{t}{b}\right)^2 \left[0.769\sqrt{D_x D_y} - 0.270(D_{xy} + D_{yx}) + 1.712G_t\right]$$

$D_x = 8{,}000\,\text{ksi}$ $\quad D_{xy} = 16{,}000\,\text{ksi}$

$D_y = 31{,}000\,\text{ksi}$ $\quad G_t = 2{,}400\,\text{ksi}$

$\sigma_{cr} = 33\,\text{ksi}$

$$\frac{b_s}{t_s} = 15.2 \Rightarrow 16$$



Reference: *Welded Interior Beam-to-Column Connections,* AISC

Stiffeners

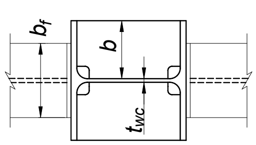
- Stiffeners, *Seismic Provisions*
  - Minimum width, b:

$$b \geq \frac{b_f}{2} - \frac{t_{wc}}{2}$$

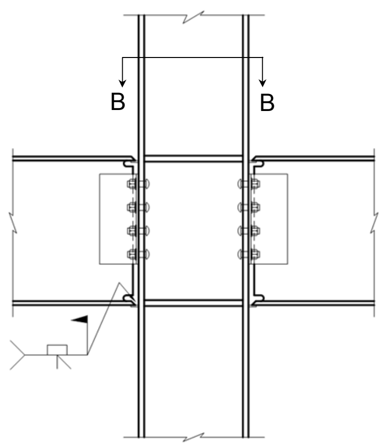
  - Minimum thickness, t:

$$t \geq \frac{t_f}{2} \qquad \text{One sided connections}$$

$$t \geq \frac{3t_f}{4} \qquad \text{Two sided connections}$$



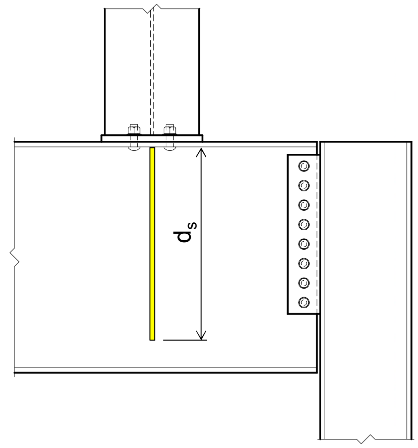
Section B-B



Stiffeners

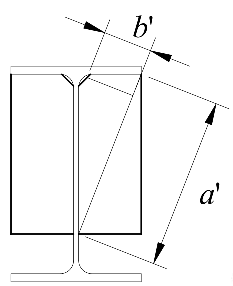
- Stiffeners
  - Minimum Length
    - AISC 360-16 *Specification* Section J10:
      - Transverse stiffeners shall extend a minimum of one-half the depth of the member except as required in Section J10.3 (WC)*, J10.5(WB), and J10.7(Unframed ends of beams not restrained against rotation about their longitudinal axis)

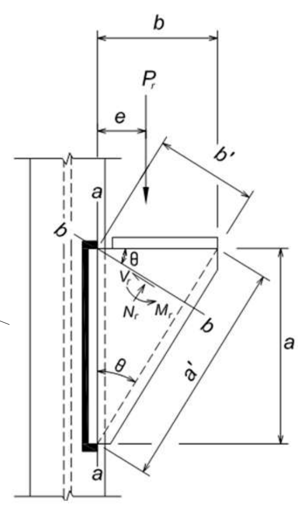
        *To satisfy **web crippling strength, ½-depth stiffener increased to ¾-depth** in the 15th Edition based on EJ 4th Qtr, 2015, *Crippling of Webs with Partial-Depth Stiffeners under Patch Loading,* by Salkar, et al
  - Avoid welding stiffeners on 3-sides if possible, terminate at k_{det} *for* fabrication cost savings



Stiffeners

- Stiffeners
  - Designing Partial Depth Stiffeners
    - Reference EJ 4th Qtr, 2015, *"Crippling of Webs with Partial-Depth Stiffeners under Patch Loading"* by Salkar, et al
    - Reference 15th Manual Ed, Fig 15-2 for brackets





$$N_r = P_r \cos\theta$$
$$V_r = P_r \sin\theta$$
$$M_r = P_r e - N_r\,(b'/2)$$

Stiffeners

- Stiffener Clips
  - Gravity, Wind, and Seismic R=3
    - Stiffeners should clear k_{det} and $k1$
    - *k-area* (1-1/2" from *k*) is area of potentially lower notch toughness in rotary-straightened W-Shapes, see Commentary J10.8
    - Tests show a corner clip of 1-1/2" with fillets stopped short by (1) weld size can avoid this problem

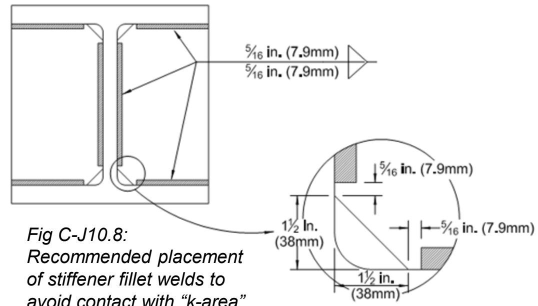


Fig C-J10.8: Recommended placement of stiffener fillet welds to avoid contact with "k-area"

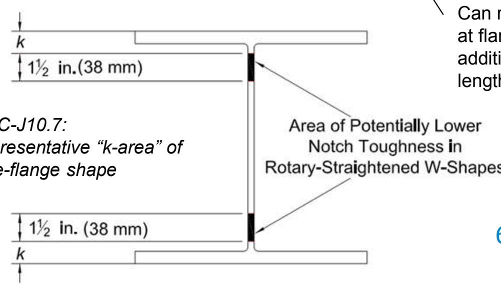
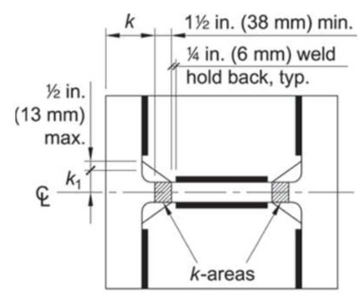


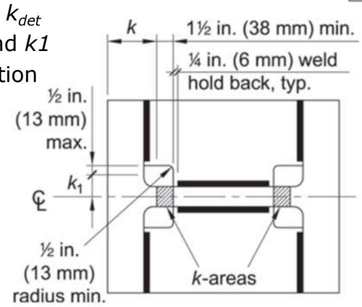
Fig C-J10.7: Representative "k-area" of wide-flange shape

Stiffeners

- Seismic Stiffener clips
  - Seismic Provisions:
    - Reference AWS D1.8/D1.8M, Clause 4.1
      - At web: Extend 1-1/2" min beyond k_{det}
      - At flange: Not to exceed 1/2" beyond $k1$
      - Shall facilitate suitable weld termination
      - If curved, r = 1/2" min
    - Welding in the *k-area* should be avoided



(a) Straight corner clip

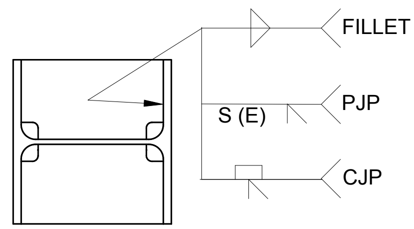


(b) Curved corner clip

AISC's *Seismic Provisions*, Fig C-D2.3:

Stiffeners/Continuity Plates

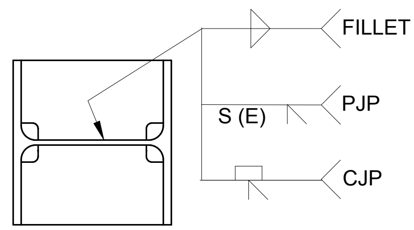
- Flange Welds
  - AISC *Specification* Section J10.8
    - Welds to be sized for the difference between the required strength and available strength.
  - AISC's Seismic Provision
    - Shall be CJP Welded per E3.6f2(c)



Stiffeners and Doublers 69

Stiffeners/Continuity Plates

- Web Welds
  - Design Guide 13
    - Weld for unbalanced stiffener force
  - AISC's Seismic Provision
    - E3.6f.2(c): Connect for lesser of:
      - Sum of stiffener tensile strength at column flanges
      - Available shear strength of stiffener at column web
      - Available shear strength of column web (or doubler if doubler extended)



Stiffeners and Doublers 70

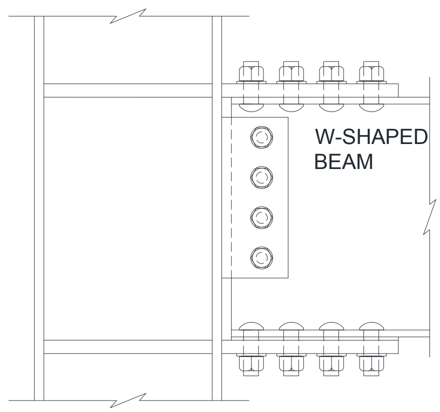
Stiffeners/Continuity Plates

- Stiffener Design
  - AISC *Specification* Chapter J

    $P_{stiffener} = P_f$ – minimum ϕR_n LRFD

    $P_{stiffener} = P_f$ – minimum R_n/Ω ASD

    R_n is the minimum nominal strength for the applicable limit states
  - If $kl/r \leq 25$, yielding (k = 0.75) (J4.4)
    If $kl/r > 25$, see *Spec* Chapter E



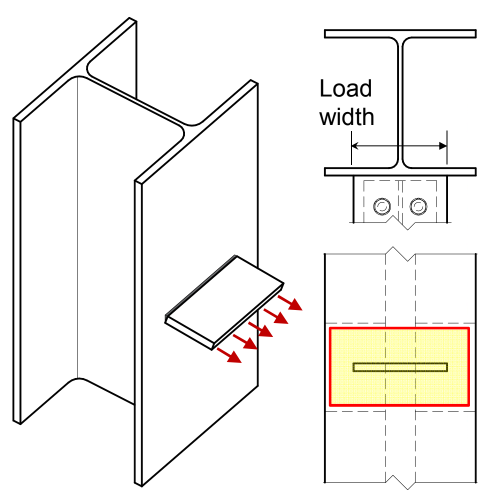
Stiffener Design

- Flange Local Bending
  - AISC *Specification* Section J10.1, Eq. J10-1

$$R_n = 6.25 F_{yf} t_f^2$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

  - Applicable to tension load only
  - Not applicable if load width is less than $0.15b_f$
  - Reduce R_n by 50% if force is applied less than $10t_{fc}$ from end of column



See Blodgett 5.7-8

Stiffeners

- Web Local Yielding
  - AISC *Specification* Section J10.2

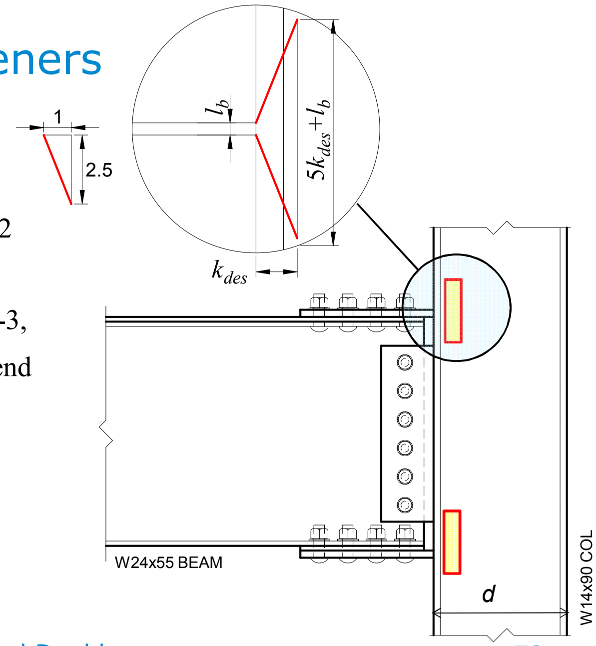
$$R_n = F_{yw} t_w \left(5k_{des} + l_b\right) \quad \textit{Spec} \text{ Equation J10-2}$$

$$R_n = F_{yw} t_w \left(2.5k_{des} + l_b\right) \quad \textit{Spec} \text{ Equation J10-3,}$$

Force $\le d$ from end of column

$\phi = 1.00$ (LRFD) $\quad \Omega = 1.50$ (ASD)

l_b = bearing length, in.



Stiffeners and Doublers 73

Stiffeners

- Web Local Crippling
  - AISC *Specification* Section J10.3

$$R_n = 0.80t_w^2\left(1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$$

Spec Equation J10-4, Force applied distance $\geq d/2$ from end of member

$$R_n = 0.40t_w^2\left(1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$$

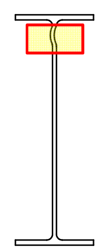
Spec Equation J10-5a, Force applied distance < $d/2$ from end of member, $l_b/d \leq 0.2$

$$R_n = 0.40t_w^2\left(1+\left(\frac{4l_b}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$$

Spec Equation J10-5b, Force applied distance < $d/2$ from end of member, $l_b/d > 0.2$

$\phi = 0.75$ (LRFD) $\quad \Omega = 2.00$ (ASD)

See Section J10.3 for Q_f definition



Stiffeners and Doublers 74

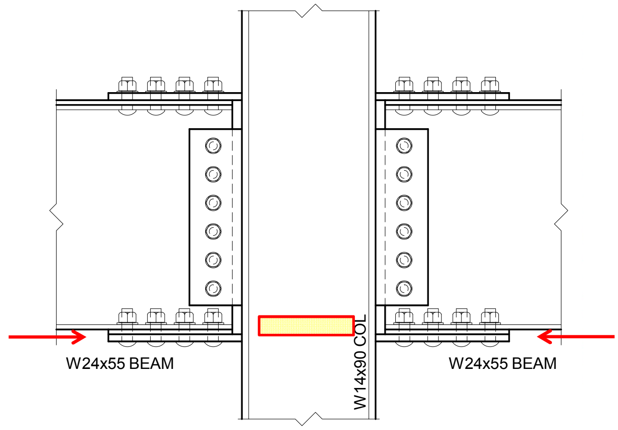
Stiffeners

- Web Compression Buckling
  - AISC *Specification* Section J10.5, Eq. J10-8

$$R_n = \frac{24t_w^3\sqrt{EF_{yw}}}{h}Q_f$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

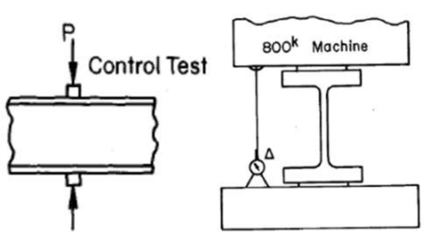
  - Applicable to double-concentrated forces at the same location
  - Reduce R_n by 50% if force is applied less than $d/2$ from end of column



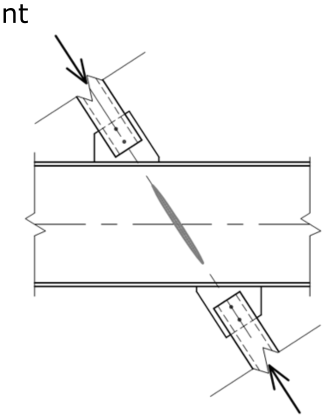
Stiffeners and Doublers 75

Stiffeners

- Web Compression Buckling
  - Equation J10.8 developed from point loads and restrained flanges
  - If l_b/d > *approximately* 1, Use Chapter E
  - See Chapter C and Appendix 6 for Stability



Test Set-up from Chen and Oppenheim 1970

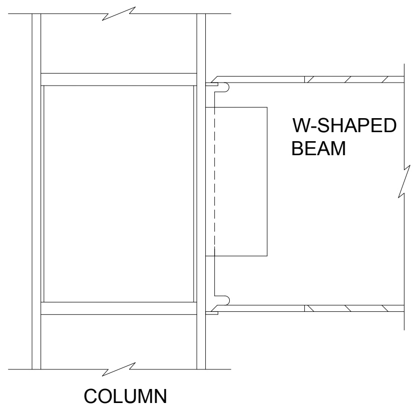


See *Spec* Appendix Section 6.2.2 for point bracing (nodal bracing) required strength and stiffness

Stiffeners and Doublers 76

Stiffeners/Continuity Plates

- Stiffener Design
  - AISC's Seismic Provision
    - Also Required if $t_{cf} < b_{bf}/6$ Eq (E3-8)

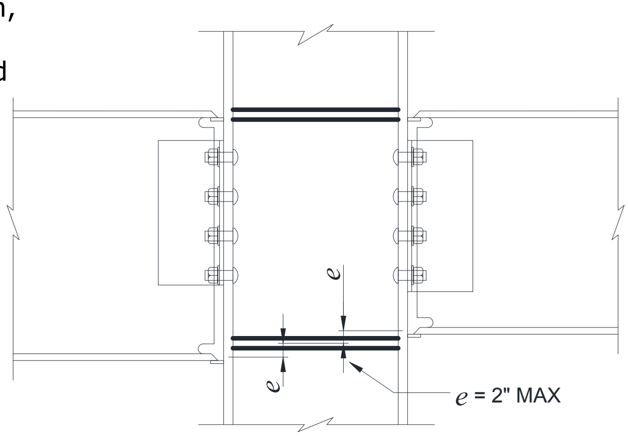


Stiffener Eccentricity

- DG13 which references testing by Graham, et al (1959), indicates stiffeners with 2" eccentricity, *e*, are 65% effective, reduced linearly:

$$R_{n\_emax} = 0.65\ R_n$$

- Can slope stiffener if stiffener not needed for beam-to-column web moment connection
- Further research discussed for eccentric stiffeners



Web Sidesway Buckling – Beams

- Web Sidesway Buckling (J10.4)
  - AISC *Specification* Eq. J10-6 and J10-7

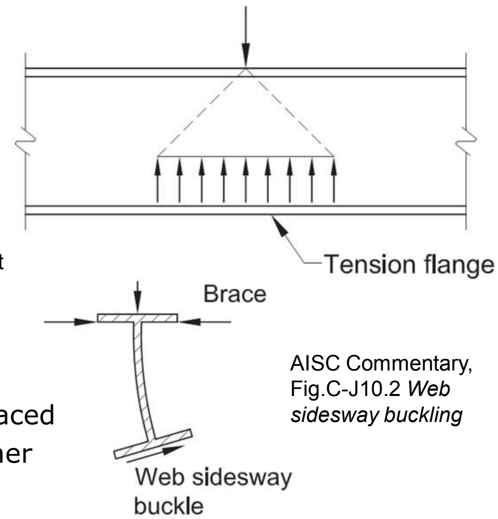
$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1 + 0.4\left(\frac{h/t_w}{L_b/b_f}\right)^3\right]$$

Equation J10-6, Compression Flange restrained, applicable if $(h/t_w)/(L_b/b_f) \le 2.3$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{L_b/b_f}\right)^3\right]$$

Equation J10-7, Compression Flange not restrained, applicable if $(h/t_w)/(L_b/b_f) \le 1.7$

$\phi = 0.85$ (LRFD) $\Omega = 1.76$ (ASD)

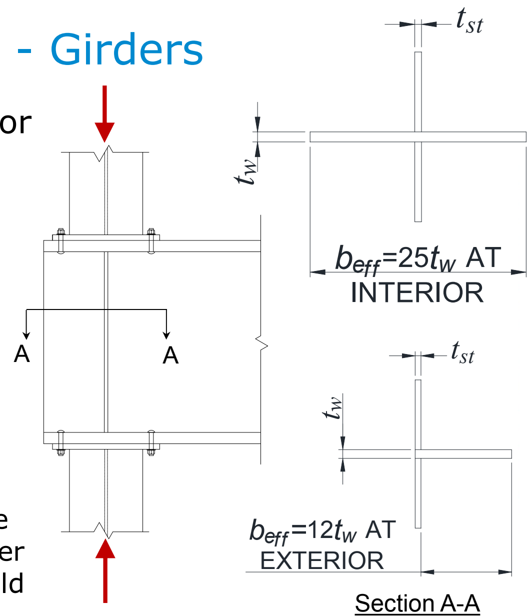


AISC Commentary, Fig.C-J10.2 *Web sidesway buckling*

- Compression flange braced, tension flange unbraced
- L_b = largest laterally unbraced section along either flange at the point of load.

Web Buckling - Girders

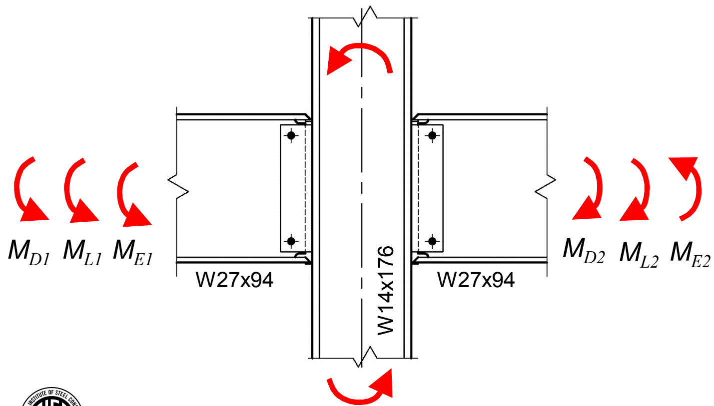
- Additional Stiffener Requirements for Beam and Girder Flange(s)
  - AISC *Specification* Section J10.8
    - Check web and stiffeners as column section with $kl = 0.75h$ (see *Spec* Sections E6.2, J4.4)
    - b_{eff} = 25 t_w at interior
      12 t_w at ends of members
  - For fabrication cost savings, fit one side of stiffener to bear at flange and transfer force at opposite end of stiffener by weld



Stiffener & Doubler Example

- Given:
  - Distance to end of column much larger than column depth
  - W Shapes ASTM A992, F_y = 50 ksi

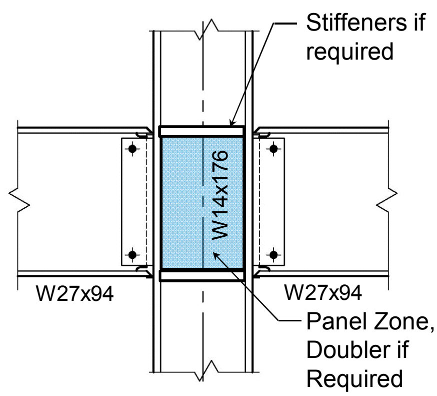
- Plates Grade 50, F_y = 50 ksi
- AISC Specification 360-16
- AISC 15th Edition, LRFD
- ASCE 7-10
- R= 3
- Wind determined not to control



M_{D1} = 130 kip-ft
M_{L1} = 220 kip-ft
M_{E1} = 690 kip-ft
M_{D2} = 130 kip-ft
M_{L2} = 220 kip-ft
M_{E2} = 690 kip-ft

Stiffener & Doubler Example

- Find:
  - Check if stiffeners are required. If required, size the stiffeners.
  - Check if doublers are required. If required, size the doublers.



Stiffener & Doubler Example

- Solution:
  - 1. ASCE 7-10, Load Combinations for Strength Design

1. 1.4D	= 182 kip-ft	
2. 1.2D + 1.6L + 0.5(L_r or S or R)	= 508 kip-ft	
3. 1.2D + 1.6(L_r or S or R) + (L or 0.5W)	= 376 kip-ft	
4. 1.2D + 1.0W + L + 0.5(L_r or S or R)	= 376 kip-ft	
5. 1.2D + 1.0E + L + 0.2S	= 1070 kip-ft	
6. 0.9D + 1.0W	= 117 kip-ft	
7. 0.9D + 1.0E	= 807 kip-ft	

M_{D1} = 130 kip-ft
M_{L1} = 220 kip-ft
M_{E1} = 690 kip-ft
M_{D2} = 130 kip-ft
M_{L2} = 220 kip-ft
M_{E2} = 690 kip-ft

Governing Load Combination: 5. 1.2D + 1.0E + L + 0.2S

$M_1 = M_2$ = 1.2(130 kip-ft) + 1.0 (690 kip-ft) + 1.0(220 kip-ft) = 1070 kip-ft

83

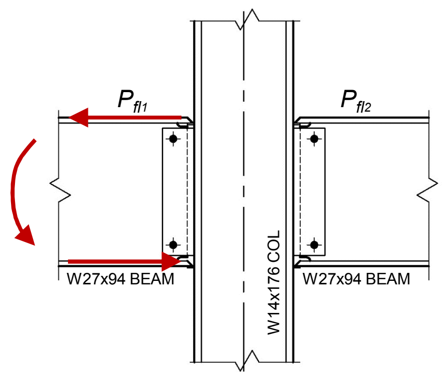
Stiffeners and Doublers

Stiffener & Doubler Example

- Solution:
  - 2. Determine Each Beam's Flange Force

$$P_{fl} = \frac{M_u}{d + t_{pl}}$$ Flange Plate

$$P_{fl} = \frac{M_u}{d - t_{fb}}$$ Directly welded



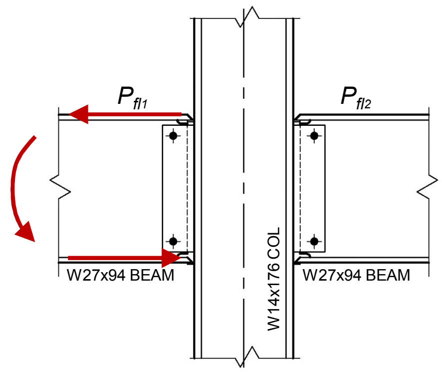
84

Stiffeners and Doublers

Stiffener & Doubler Example

- Solution:
  - 2. Determine Each Beam's Flange Force

$$P_{fl} = P_{fl1} = P_{fl2} = \frac{M_1}{d_b - t_{fb}}$$
$$= \frac{1070\,\text{kip-ft}}{26.9\,\text{in.} - 0.745\,\text{in.}}$$
$$= 491\,\text{kips}$$



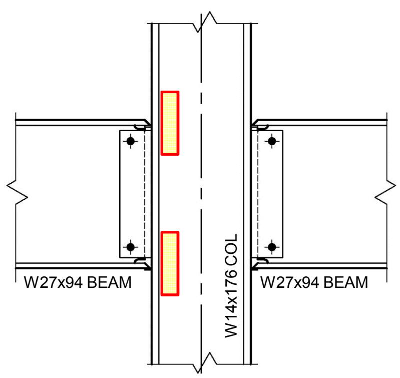
Stiffener & Doubler Example

- Solution:
  - 3. Check Stiffener Limit States
    - Web Local Yielding (J10.2) $(\phi = 1.00)$
      - AISC *Specification* Eq. J10-2

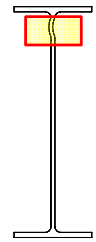
$$\phi R_{wy} = \phi F_{yw} t_{wc} \left(5k_{desc} + l_b\right)$$
$$= 1.00\left(50\,\text{ksi}\right)\left(0.830\,\text{in.}\right)\left(5\left(1.91\,\text{in.}\right) + \left(0.745\,\text{in.}\right)\right)$$
$$= 427\text{ kips} < P_{fl} = 491\,\text{kips} \quad \textbf{n.g.}$$

<ins>**Stiffeners Required**</ins>

Note: $l_b = t_{fb}$



Stiffener & Doubler Example



- Solution:
  - 3. Check Stiffener Limit States
    - Web Local Crippling (J10.3) $(\phi = 0.75)$
      - AISC *Specification* Eq. J10-4

$$\phi R_{wc} = \phi 0.80 t_{wc}^2 \left(1 + 3\left(\frac{l_b}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right)\sqrt{\frac{EF_{yc}t_{fc}}{t_{wc}}}$$

$$= 0.75(0.80)(0.830\,\text{in.})^2\left(1+3\left(\frac{0.745\,\text{in.}}{15.2\,\text{in.}}\right)\left(\frac{0.830\,\text{in.}}{1.31\,\text{in.}}\right)^{1.5}\right)\sqrt{\frac{(29000\,\text{ksi})(50\,\text{ksi})(1.31\,\text{in.})}{(0.830\,\text{in.})}}$$

$$= 672\,\text{kips} \geq P_{fl} = 491\,\text{kips}\ \ \textbf{o.k.}$$

Note: $l_b = t_{fb}$

Stiffener & Doubler Example

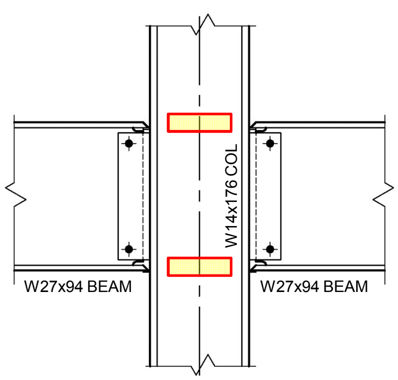
- Solution:
  - 3. Check Stiffener Limit States
    - Web Compression Buckling (J10.5) $(\phi = 0.90)$
      - AISC *Specification* Eq. J10-8

$$h = d_c - 2k_{desc} = 15.2\,\text{in.} - 2(1.91\,\text{in.}) = 11.4\,\text{in.}$$

$$\phi R_{wb} = \phi \frac{24 t_{wc}^3 \sqrt{EF_{yc}}}{h} Q_f$$

$$= (0.90)\frac{24(0.83\,\text{in.})^3\sqrt{(29000\,\text{ksi})(50\,\text{ksi})}}{11.4\,\text{in.}}(1.0)$$

$$= 1305\,\text{kips} \geq P_{fl} = 491\,\text{kips}\ \ \textbf{o.k.}$$

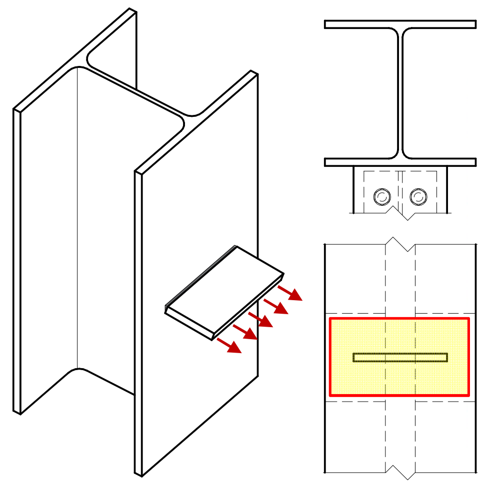


Stiffener & Doubler Example

- Solution:
  - 3. Check Stiffener Limit States
    - Flange Local Bending (J10.1) $(\phi = 0.90)$
      - AISC *Specification* Eq. J10-1

$$\phi R_{fb} = \phi 6.25 F_{yc} t_{fc}^2$$
$$= 0.90(6.25)(50\,\text{ksi})(1.31\,\text{in.})^2$$
$$= 483\,\text{kips} < P_{fl} = 491\,\text{kips} \quad \textbf{n.g.}$$

<ins>**Stiffeners Required**</ins>



See Blodgett 5.7-8

Use of *Manual* Tables

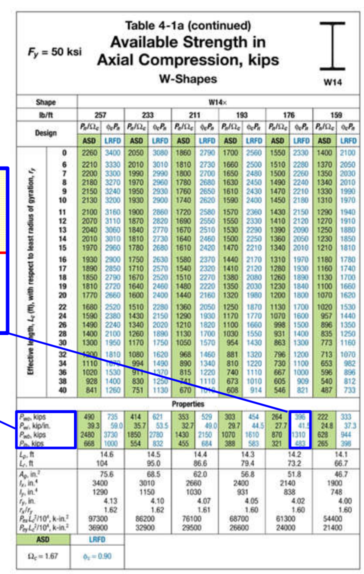
- Web Local Yielding $(\phi = 1.00)$
  - AISC 15th Edition *Manual* Eq. 4-2a

LRFD	
$\phi R_n = P_{wo} + P_{wi} l_b$	(4-2a)

P_{wo}, kips	396
P_{wi}, kips/in.	41.5
P_{wb}, kips	1310
P_{fb}, kips	483

$$\phi R_n = \boxed{P_{wo}} + \boxed{P_{wi} l_b}$$
$$= 396\,\text{kips} + \left(41.5\frac{\text{kips}}{\text{in.}}\right)(0.745\,\text{in.})$$
$$= 427\,\text{kips} < P_{fl} = 491\,\text{kips} \quad \textbf{n.g.}$$

<ins>**Stiffeners Required**</ins>



Use of *Manual* Tables

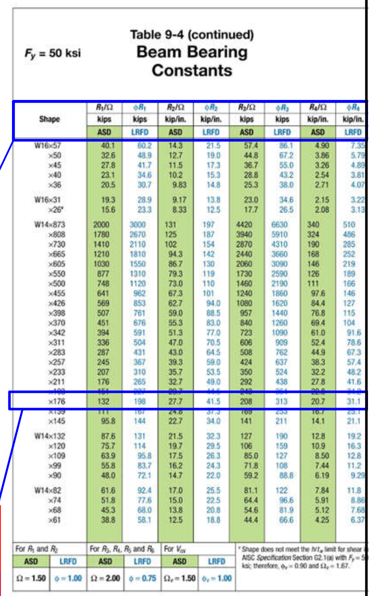
- Web Local Crippling $(\phi = 0.75)$
  - AISC 15th Edition *Manual* Eq. 9-50a

LRFD
$\phi R_n = 2[\phi R_3 + l_b(\phi R_4)]$ (9-50a)

$$\phi R_n = 2\left[\boxed{\phi R_3} + l_b\left(\boxed{\phi R_4}\right)\right]$$

$$= 2\left[313\,\text{kips} + (0.745\,\text{in.})\left(31.1\frac{\text{kips}}{\text{in.}}\right)\right]$$

$$= 672\,\text{kips} \geq P_{fl} = 491\,\text{kips} \quad \textbf{o.k.}$$



Shape	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
	kips	kips	kips/in.	kips/in.	kips	kips	kips/in.	kips/in.
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
×176	132	198	27.7	41.5	208	313	20.7	31.1

Stiffeners and Doublers

91

Use of *Manual* Tables

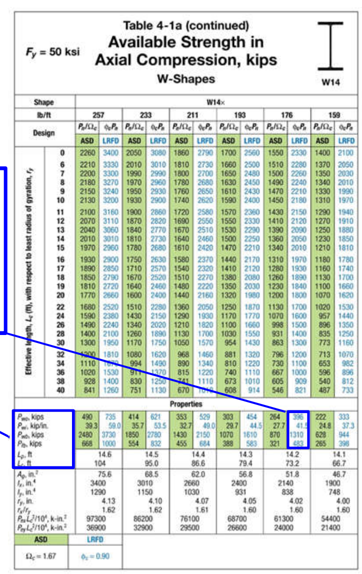
- Web Compression Buckling $(\phi = 0.90)$
  - AISC 15th Edition *Manual* Eq. 4-3a

LRFD
$\phi R_n = P_{wb}$ (4-3a)

$$\phi R_n = \boxed{P_{wb}}$$

$$= 1310\,\text{kips} \geq P_{fl} = 491\,\text{kips} \quad \textbf{o.k.}$$

P_{wo}, kips	396
P_{wi}, kips/in.	41.5
P_{wb}, kips	1310
P_{fb}, kips	483



Stiffeners and Doublers

92

Use of *Manual* Tables

- Flange Local Bending $(\phi = 0.90)$

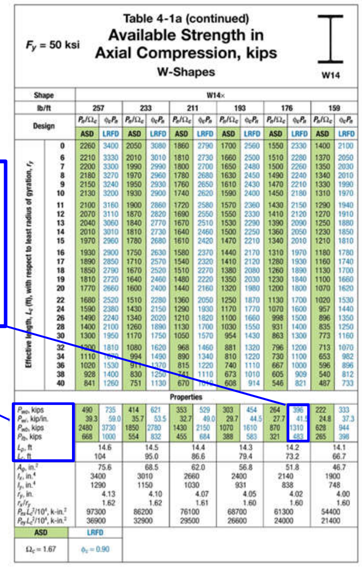
LRFD	
$\phi R_n = P_{fb}$	(4-4a)

$\phi R_n = \boxed{P_{fb}}$

$= 483\,\text{kips} < P_{fl} = 491\,\text{kips}$ **n.g.**

Stiffeners Required

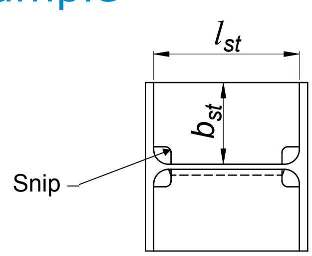
P_{wo}, kips	396
P_{wi}, kips/in.	41.5
P_{wb}, kips	1310
P_{fb}, kips	483



Stiffener & Doubler Example

- Solution:
  - 4. Column Stiffener Design
    - Determine Force in the Stiffeners

$$P_{st1tot} = P_{st2tot} = P_{fl} - \min(\phi R_{wy},\, \phi R_{wc},\, \phi R_{wb},\, \phi R_{fb})$$
$$= 491\text{ kips} - 427\text{ kips}$$
$$= 64\text{ kips}$$

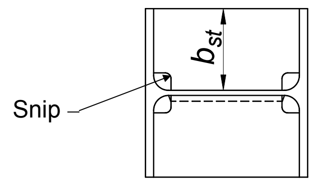


- Determine Beam Force Per Each Stiffener

$$P_{st1} = 0.5(P_{st1tot}) = 0.5(64\text{ kips}) = 32\text{ kips}$$
$$P_{st2} = 0.5(P_{st2tot}) = 0.5(64\text{ kips}) = 32\text{ kips}$$

Stiffener & Doubler Example

- Solution:
  - 4. Column Stiffener Design
    - Determine Minimum Stiffener Width per AISC's *Specification*, Section J10.8



$$b_{st\ \min} = \frac{b_{fb}}{3} - \frac{t_{wc}}{2} = \frac{10\text{ in.}}{3} - \frac{0.83\text{ in.}}{2} = 2.92\text{ in. Min. (See Note below)}$$

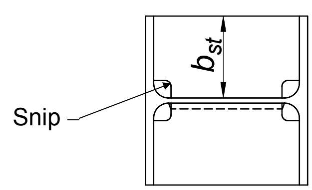
$$b_{st\ \max} = \frac{b_{fc} - t_{wc}}{2} = \frac{15.7\text{ in.} - 0.830\text{ in.}}{2} = 7.44\text{ in. Max}$$

Use b_{st} = 7" Stiffener

*Note: Use b_{pl} in lieu of b_{fb} if moment connection plate delivering the load

Stiffener & Doubler Example

- Solution:
  - 4. Column Stiffener Design
    - Determine Minimum Stiffener Thickness per AISC's *Specification*, Section J10.8



$$t_{st\ \min} = \frac{b_{st}}{16} = \frac{7\text{ in.}}{16} = 0.438\text{ in.} \quad \textbf{Controls}$$

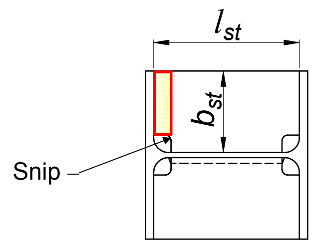
$$t_{st\ \min} = \frac{t_{fb}}{2} = \frac{0.745\text{ in.}}{2} = 0.373\text{ in.} \quad \text{(See Note below)}$$

- Try a 1/2" Stiffener

*Note: Use t_{pl} in lieu of t_{fb} if moment connection plate delivering the load

Stiffener & Doubler Example

- Solution:
  - 4. Column Stiffener Design
    - Check Minimum Thickness Due to Tension Yielding
      - AISC *Specification* Section J10.8



$$snip = \max\left(k_{\det c} - t_{fc},\ k_{1c} - \frac{t_{wc}}{2}, 1.5\text{ in.}\right)$$

$$= \max\left(2.625\text{ in.} - 1.31\text{ in.},\ 1.625\text{ in.} - \frac{0.83\text{ in.}}{2}, 1.5\text{ in.}\right)$$

$$= 1.5\text{ in.}$$

$$t_{st\ \min} = \frac{P_{st}}{\phi F_{yp}(b_{st} - snip)}$$

$$= \frac{32\text{ kips}}{0.90(50\text{ ksi})(7\text{ in.} - 1.5\text{ in.})}$$

$$= 0.129\text{ in.} \le 0.5\text{ in.}\ \textbf{o.k.}$$

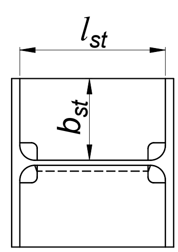
*Note: Can reduce snip at flange if needed.

97

Stiffeners and Doublers

Stiffener & Doubler Example

- Solution:
  - 4. Column Stiffener Design
    - Check Minimum Thickness Due to Compression Buckling
      - AISC *Specification* Section J10.8 and E3 $(\phi = 0.90)$



$$l_{st} = d_c - 2t_{fc} = 15.2\text{ in.} - 2(1.31\text{ in.}) = 12.6\text{ in.}$$

$$k = 0.75$$

$$\frac{kl}{r} = \frac{kl_{st}\sqrt{12}}{t_{st}} = \frac{0.75(12.6\text{ in.})\sqrt{12}}{(0.5\text{ in.})} = 65.5$$

$$Limit = 4.71\sqrt{\frac{E}{F_{yp}}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 113$$

$$F_e = \frac{\pi^2 E}{\left(kl/r\right)^2} = \frac{\pi^2\,(29{,}000\text{ ksi})}{(65.5)^2} = 66.6\text{ ksi}$$

When $\frac{L_c}{r} \le 4.71\sqrt{\frac{E}{F_y}}$ (or $\frac{F_y}{F_e} \le 2.25$)

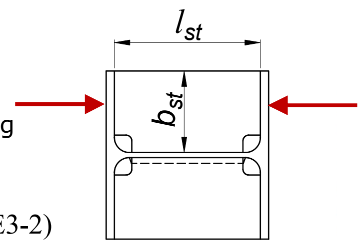
$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y$$

98

Stiffeners and Doublers

Stiffener & Doubler Example

- Solution:
  - 4. Column Stiffener Design
    - Check Minimum Thickness Due to Compression Buckling
      - AISC *Specification* Section E3 $(\phi = 0.90)$



$$F_{cr} = \left(0.658^{\frac{F_{yp}}{F_e}}\right)F_{yp} = \left(0.658^{\frac{50\text{ ksi}}{66.6\text{ ksi}}}\right)50\text{ ksi} = 36.5\text{ ksi} \qquad \text{(E3-2)}$$

$$\phi F_{cr} = 0.90(36.5\text{ ksi}) = 32.9\text{ ksi}$$

Rearrange $\phi P_n = \phi F_{cr} A_g$ to Solve for $t_{st\ \min}$ (E3-1)

$$t_{st\ \min} = \frac{P_{st}}{\phi F_{cr}(b_{st})}$$

$$= \frac{32\text{ kips}}{(32.9\text{ ksi})(7\text{ in.})}$$

$$= 0.139\text{ in.} \quad \leq \quad 0.5\text{ in. } \textbf{o.k.}$$

*Note: Buckling limit state included for example purposes.

99

Stiffeners and Doublers

Use of *Manual* Tables

– **AISC *Manual* Table 4-14: *KL/r* Table**

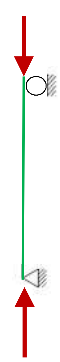
The *critical stress*, F_{cr}, is determined as follows:

(a) When $\dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} \leq 2.25$)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y \qquad \text{(E3-2)}$$

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} > 2.25$)

$$F_{cr} = 0.877F_e \qquad \text{(E3-3)}$$



F_y = 50 ksi		
$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
65	22.0	33.0
66	21.8	32.7

Table 4-14 (continued)
Available Critical Stress for Compression Members

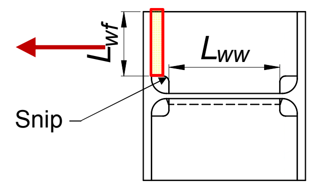
$\frac{L_c}{r}$	F_y = 35 ksi		F_y = 36 ksi		F_y = 46 ksi		F_y = 50 ksi		F_y = 65 ksi		F_y = 70 ksi	
	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
41	19.2	28.9	19.7	29.7	24.6	37.0	26.5	39.8	33.2	49.9	35.3	53.0
42	19.2	28.8	19.6	29.5	24.5	36.8	26.3	39.5	32.9	49.5	35.0	52.6
43	19.1	28.7	19.6	29.4	24.3	36.6	26.2	39.3	32.6	49.1	34.7	52.1
44	19.0	28.5	19.5	29.3	24.2	36.3	26.0	39.1	32.4	48.7	34.4	51.7
45	18.9	28.4	19.4	29.1	24.0	36.1	25.8	38.8	32.1	48.3	34.1	51.2
46	18.8	28.3	19.3	29.0	23.9	35.9	25.6	38.5	31.8	47.8	33.8	50.7
47	18.7	28.1	19.2	28.9	23.8	35.7	25.5	38.3	31.6	47.4	33.4	50.3
48	18.6	28.0	19.1	28.7	23.6	35.4	25.3	38.0	31.3	47.0	33.1	49.8
49	18.5	27.9	19.0	28.5	23.4	35.2	25.1	37.7	31.0	46.6	32.8	49.3
50	18.4	27.7	18.9	28.4	23.3	35.0	24.9	37.5	30.7	46.1	32.5	48.8
51	18.3	27.6	18.8	28.3	23.1	34.8	24.8	37.2	30.4	45.7	32.1	48.3
52	18.3	27.4	18.7	28.1	23.0	34.5	24.6	36.9	30.1	45.2	31.8	47.8
53	18.2	27.3	18.6	28.0	22.8	34.3	24.4	36.7	29.8	44.8	31.4	47.3
54	18.1	27.1	18.5	27.8	22.6	34.0	24.2	36.4	29.5	44.3	31.1	46.7
55	18.0	27.0	18.4	27.6	22.5	33.8	24.0	36.1	29.2	43.9	30.8	46.2
56	17.9	26.8	18.3	27.5	22.3	33.5	23.8	35.8	28.9	43.4	30.4	45.7
57	17.7	26.7	18.2	27.3	22.1	33.3	23.6	35.5	28.6	43.0	30.1	45.2
58	17.6	26.5	18.1	27.1	22.0	33.0	23.4	35.2	28.3	42.5	29.7	44.6
59	17.5	26.4	17.9	27.0	21.8	32.8	23.2	34.9	28.0	42.0	29.4	44.1
60	17.4	26.2	17.8	26.8	21.6	32.5	23.0	34.6	27.6	41.5	29.0	43.6
61	17.3	26.0	17.7	26.6	21.4	32.2	22.8	34.3	27.3	41.1	28.6	43.0
62	17.2	25.9	17.6	26.5	21.3	32.0	22.6	34.0	27.0	40.6	28.3	42.5
63	17.1	25.7	17.5	26.3	21.1	31.7	22.4	33.7	26.7	40.1	27.9	42.0
64	17.0	25.5	17.4	26.1	20.9	31.4	22.2	33.4	26.4	39.6	27.6	41.4
65	16.9	25.4	17.3	25.9	20.7	31.2	22.0	33.0	26.0	39.2	27.2	40.9
66	16.8	25.2	17.1	25.8	20.5	30.9	21.8	32.7	25.7	38.7	26.8	40.3
67	16.7	25.0	17.0	25.6	20.4	30.6	21.6	32.4	25.4	38.2	26.5	39.8
68	16.5	24.9	16.9	25.4	20.2	30.3	21.4	32.1	25.1	37.7	26.1	39.2
69	16.4	24.7	16.8	25.2	20.0	30.1	21.1	31.8	24.8	37.2	25.7	38.7
70	16.3	24.5	16.7	25.0	19.8	29.8	20.9	31.4	24.4	36.7	25.4	38.2
71	16.2	24.3	16.5	24.8	19.6	29.5	20.7	31.1	24.1	36.2	25.0	37.6
72	16.1	24.2	16.4	24.7	19.4	29.2	20.5	30.8	23.8	35.7	24.7	37.1
73	16.0	24.0	16.3	24.5	19.2	28.9	20.3	30.5	23.5	35.3	24.3	36.5
74	15.8	23.8	16.2	24.3	19.1	28.6	20.1	30.2	23.1	34.8	23.9	36.0
75	15.7	23.6	16.0	24.1	18.9	28.4	19.8	29.8	22.8	34.3	23.6	35.4
76	15.6	23.4	15.9	23.9	18.7	28.1	19.6	29.5	22.5	33.8	23.2	34.9
77	15.5	23.3	15.8	23.7	18.5	27.8	19.4	29.2	22.2	33.3	22.8	34.3
78	15.4	23.1	15.6	23.5	18.3	27.5	19.2	28.8	21.8	32.8	22.5	33.8
79	15.2	22.9	15.5	23.3	18.1	27.2	19.0	28.5	21.5	32.3	22.1	33.3
80	15.1	22.7	15.4	23.1	17.9	26.9	18.8	28.2	21.2	31.8	21.8	32.7

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Stiffeners and Doublers

Stiffener & Doubler Example

- Solution:
  - 4. Column Stiffener Design
    - Weld Design of Stiffener to Column
      - AISC Specification Section J2



$$L_{ww} = l_{st} - 2snip = 12.6 \text{ in. - (2)}1.5 \text{ in.} = 9.6 \text{ in.}$$

$$L_{wf} = b_{st} - snip = 7 \text{ in. - } 1.5 \text{ in.} = 5.5 \text{ in.}$$

$$\varepsilon = 0.928 \text{ ASD}$$

$$\varepsilon = 1.392 \text{ LRFD}$$

$$D_{\min fl} = \frac{P_{st}}{\varepsilon(2)L_{wf}(1.5)} = \frac{32 \text{ kips}}{1.392(2)(5.5 \text{ in.})(1.5)} = 1.39 \text{ (16th of an inch)}$$

1/4" Fillet Weld **o.k.**

Stiffener & Doubler Example

- Solution:
  - 4. Column Stiffener Design

Note:

$$0.928D = (0.6)(F_{exx})(0.707)(w)/\Omega_w$$

$$= (0.60)(70 \text{ ksi})(0.707)\left(\frac{D}{16}\right)/2 = 0.928D$$

$$1.392D = (0.6)(F_{exx})(0.707)(w)\left(\frac{D}{16}\right)\phi_w$$

$$= (0.60)(70 \text{ ksi})(0.707)\left(\frac{D}{16}\right)(0.75) = 1.392D$$

$$1.5 = 1.0 + 0.5(\sin(\theta))^{1.5} = 1 + 0.5(\sin(90°))^{1.5} = 1.5 \qquad \text{Reference } \textit{Spec} \text{ Eq (J2-5)}$$

Stiffener & Doubler Example

- Solution:
  - 5. Determine Flange Forces for Column Doubler Checks
    - Based on ASCE 7 Load Cases
    - Determine sum of flange forces for each load load combination. Consider symmetric gravity moments and pattern loading if applicable

M_{D1} = 130 kip-ft
M_{L1} = 220 kip-ft
M_{E1} = 690 kip-ft
M_{D2} = 130 kip-ft
M_{L2} = 220 kip-ft
M_{E2} = 690 kip-ft

ASCE 7-10
1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

Stiffeners and Doublers 103

Stiffener & Doubler Example

- Solution:
  - 5. Determine Flange Forces for Column Doubler Checks
    - Determine Critical Load Combination:

Governing Load Combination 5:

$$M_{1\_5} = 1.2(130\text{ k-ft}) + 1.0\,(690\text{ k-ft}) + 1.0(220\text{ k-ft}) = 1070\text{ kip-ft}$$

$$M_{2\_5} = 1.2(-130\text{ k-ft}) + 1.0\,(690\text{ k-ft}) = 534\text{ kip-ft}$$

$$P_{f1\_5} = \frac{M_1}{d_{b1} - t_{fb1}} = \frac{1070\text{ kip-ft}}{26.9\text{ in.} - 0.745\text{ in.}} = 491\text{ kip}$$

$$P_{f2\_5} = \frac{M_2}{d_{b2} - t_{fb2}} = \frac{534\text{ kip-ft}}{26.9\text{ in.} - 0.745\text{ in.}} = 245\text{ kip}$$ (Stiffener not required)

$$P_{fd} = P_{f1\_5} + P_{f2\_5} = 491\text{ kips} + 245\text{ kips} = 736\text{ kips}$$

M_{D1} = 130 kip-ft
M_{L1} = 220 kip-ft
M_{E1} = 690 kip-ft
M_{D2} = 130 kip-ft
M_{L2} = 220 kip-ft
M_{E2} = 690 kip-ft

Stiffeners and Doublers 104

Stiffener & Doubler Example

- Solution:
  - 5. Determine Flange Forces for Column Doubler Checks
    - Confirm column strength does not control:

$$\Sigma M_b = M_{1\_5} + M_{2\_5} = 1070 \text{ kip-ft} + 534 \text{ kip-ft} = 1604 \text{ kip-ft}$$

$$\phi M_{nc} = \phi(Z_{xc})(F_{yc}) = \frac{0.9(320 \text{ in.}^3)(50 \text{ ksi})}{12 \text{in./ft}} = 1200 \text{ kip-ft}$$

$$\Sigma M_c = M_{c\_t} + M_{c\_b} = 1200 \text{ kip-ft} + 1200 \text{ kip-ft} = 2400 \text{ kip-ft}$$

$$\Sigma M_b = 1604 \text{ kip-ft} < \Sigma M_c = 2400 \text{ kip-ft} \quad \textbf{Beams Control}$$

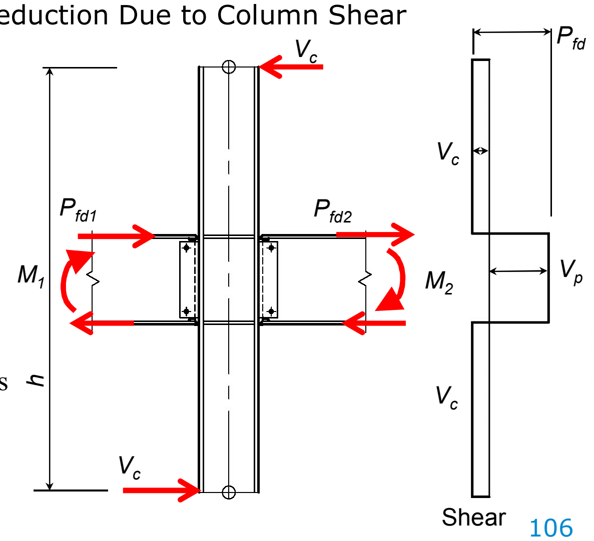
Stiffener & Doubler Example

- Solution:
  - 6. Determine Panel Zone Shear Reduction Due to Column Shear

$$h_{col} = 15 \text{ ft}$$

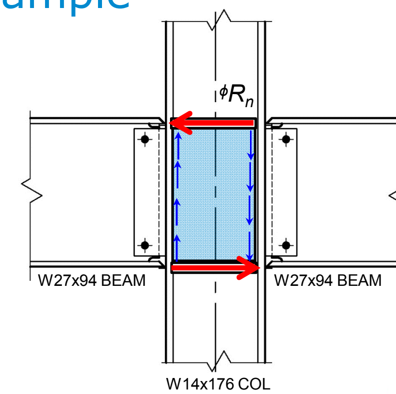
$$V_c = \frac{M_1 + M_2}{h_{col}} = \frac{1070 \text{ kip-ft} + 534 \text{ kip-ft}}{15 \text{ ft}} = 107 \text{ kips}$$

$$V_p = P_{fd} - V_c = 736 \text{ kips} - 107 \text{ kips} = 629 \text{ kips}$$



Stiffener & Doubler Example

- Solution:
  - 7. Column Doubler Check $(\phi = 0.90)$
    - Web Panel Zone Shear Strength
      - AISC *Specification* Section J10.6



$$\alpha = 1.0\ (\text{LRFD}); 1.6\ (\text{ASD})$$

$$\alpha P_r = (1.0)870 \text{ kips } = 870 \text{ kips LRFD}$$

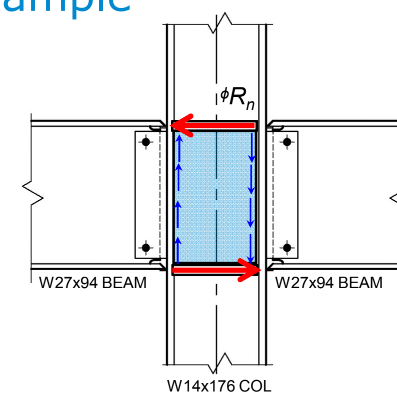
$$P_y = F_{yc} A_{gc} = (50 \text{ ksi})(51.8 \text{ in.}^2) = 2590 \text{ kips}$$

$$\frac{\alpha P_r}{P_y} = \frac{(1.0)870 \text{ kips}}{2590 \text{ kips}} = 0.336 \leq 0.4$$

$$\text{Since } \alpha P_r \leq 0.4 P_y:\ R_n = 0.6 F_y d_c t_w \qquad \text{(J10-9)}$$

Stiffener & Doubler Example

- Solution:
  - 7. Column Doubler Check $(\phi = 0.90)$
    - Web Panel Zone Shear Strength
      - AISC *Specification* Section J10.6



$$\begin{aligned} \phi R_n &= \phi 0.6 F_{yw} t_{wc} d_c \\ &= 0.90(0.6)(50 \text{ ksi})(0.830 \text{ in.})(15.2 \text{ in.}) \\ &= 341 \text{ kips} \end{aligned}$$

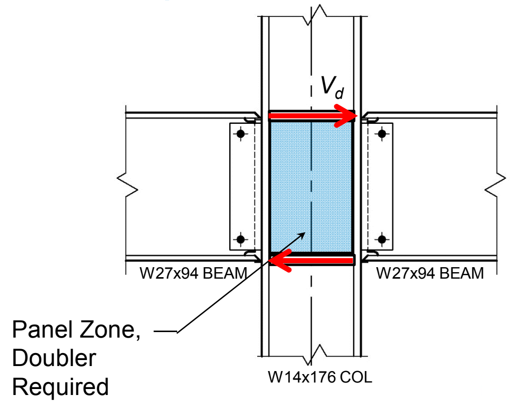
$V_p = 629 \text{ kips} > \phi R_n = 341 \text{ kips}$ **n.g.** **Doubler Required**

Stiffener & Doubler Example

- Solution:
  - 7. Determine Doubler Shear Force

$$V_d = V_p - \phi R_n$$
$$= 629 \text{ kips} - 341 \text{ kips}$$
$$= 288 \text{ kips}$$

Number of Doublers, $n_d = 1$



Stiffener & Doubler Example

- Solution:
  - 8. Column Doubler Design
    - Minimum Doubler Thickness to prevent Shear Buckling
      - AISC Specification Section G2.1

For, $C_v = 1.0$

$$t_{d\ \min 1} = \frac{h}{2.24}\sqrt{\frac{F_{yp}}{E}} = \frac{d_c - 2k_{desc}}{2.24}\sqrt{\frac{F_{yp}}{E}} = \frac{15.2 \text{ in.} - 2(1.91 \text{ in.})}{2.24}\sqrt{\frac{50 \text{ ksi}}{29,000 \text{ ksi}}} = 0.211 \text{ in.}$$

Stiffener & Doubler Example

- Solution:
  - 8. Column Doubler Design
    - Doubler Thickness Due to Panel Zone Shear $(\phi = 1.00)$
      - AISC Specification Equation G2-1

$$
\begin{aligned}
t_{d\ \min 2} &= \frac{V_d(C_v)}{\phi(0.6)(F_{yp})(d_c)(n_d)} \\
&= \frac{288 \text{ kips}(1.0)}{(1.00)(0.6)(50 \text{ ksi})(15.2 \text{ in.})(1)} \\
&= 0.632 \text{ in.}
\end{aligned}
$$

Stiffeners and Doublers

111

Stiffener & Doubler Example

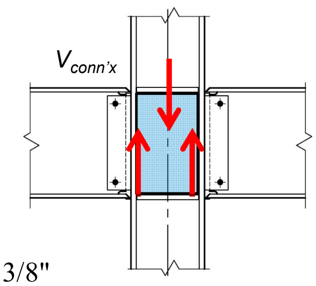
- Solution:
  - 8. Column Doubler Design
    - Doubler Vertical Edge Thickness Due to Beam-to-Column Web Shear Connection, Reference AISC's *Spec* Equation J4-3

$$t_{d\ \min 3} = \frac{0.5V_{conn'x}}{\phi(0.6)(F_{yp})(L_d)} \qquad (\phi = 1.00)$$

$$t_{d\ \min 3} = \frac{0.5(0 \text{ kips})}{1.00(0.6)(50 \text{ ksi})(26.9 \text{ in.} - 0.745 \text{ in.} - 0.5 \text{ in.})} = 0.0 \text{ in.}$$

Where: L_d = length of doubler $= d_b - t_{fb} - t_s$

$$
\begin{aligned}
t_{d\ \min} &= \max(t_{d\ \min 1}, t_{d\ \min 2} + t_{d\ \min 3}) \\
&= \max(0.211 \text{ in.},\ 0.632 \text{ in.} + 0 \text{ in.}) = 0.632 \text{ in.}
\end{aligned}
$$



Use (1) PL 3/4" or (2) PL 3/8"

Stiffeners and Doublers

112

Stiffener & Doubler Example

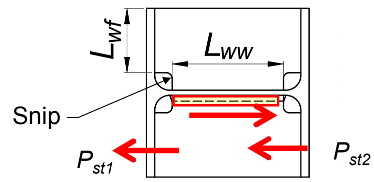
- Solution:
  - 8. Column Doubler Design $(\phi = 1.00)$
    - Doubler Horizontal Edge Thickness Due to Unbalanced Stiffener Force

$$t_{d\ \min 4} = \frac{P_{st1} + P_{st2}}{\phi(0.6)(F_{yp})(L_{ww})(2)}$$

Ref *Spec* Eq (J4-3)

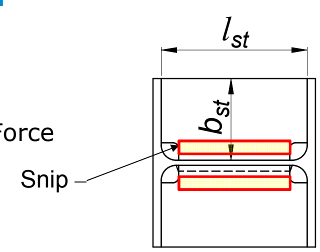
$$= \frac{32\text{ kips} + 0\text{ kips}}{(1.00)(0.6)(50\text{ ksi})(9.6\text{ in.})(2)}$$

$$= 0.056\text{ in.} \quad < 3/4\text{"}\ \textbf{o.k.}$$



Stiffener & Doubler Example

- Solution:
  - 8. Column Stiffener Thickness for Shear $(\phi = 1.00)$
    - Check Minimum Thickness Due to Unbalanced Stiffener Force



Rearrange $\phi R_n = 0.6 F_{yp} A_{gv}$ to Solve for $t_{st\ \min}$ Ref (J4-3)

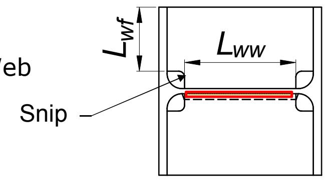
$$t_{st\ \min} = \frac{P_{st1} + P_{st2}}{\phi(0.6)F_{yp}(l_{st} - 2(snip))}$$

$$= \frac{32\text{ kips} + 0\text{ kips}}{1.00(0.6)(50\text{ ksi})(12.6\text{ in.} - 2(1.5\text{ in.}))}$$

$$= 0.111\text{ in.} \quad \leq \quad 0.5\text{ in.}\ \textbf{o.k.}$$

Stiffener & Doubler Example

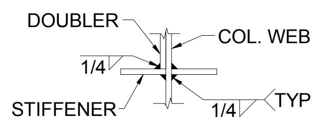
- Solution:
  - 8. Column Stiffener Weld
    - Stiffener Weld to Doubler Horizontal Edge and Column Web for unbalanced stiffener force



$$L_{ww} = l_{st} - 2snip = 12.6 \text{ in. - (2)1.5 in.} = 9.6 \text{ in.}$$

$$\varepsilon = 0.928 \text{ ASD}$$

$$\varepsilon = 1.392 \text{ LRFD}$$



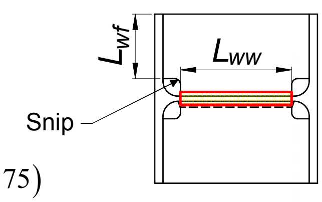
$$D_{\min\, web} = \frac{P_{st1} + P_{st2}}{\varepsilon(2)(L_{ww})}$$

$$= \frac{32 \text{ kips} + 0 \text{ kips}}{1.392(2)(9.6 \text{ in.})}$$

$= 1.20$ (16th of an inch) $\leq$ 4 (16th of an inch), use 1/4” Fillet Weld **o.k.**

Stiffener & Doubler Example

- Solution:
  - 8. Column Stiffener Thickness
  - Weld Design of Stiffener to Column
    - AISC Specification Section J2



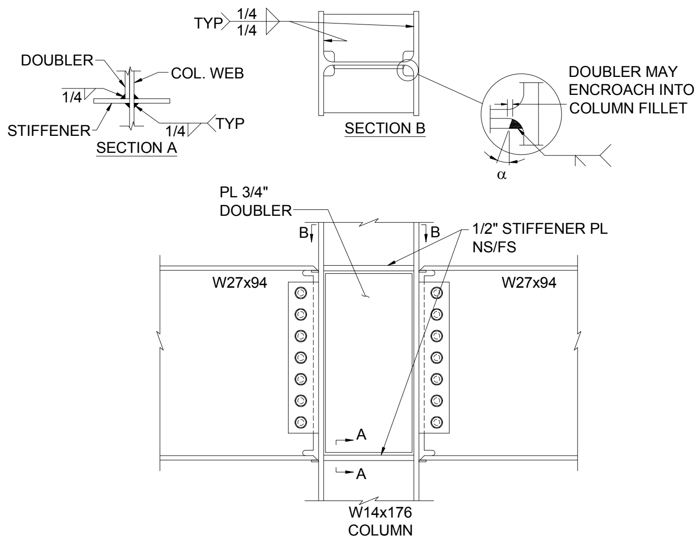
Check plate and column web thicknesses for weld size $(\phi = 0.75)$

$$t_{st\,\min} = \frac{\varepsilon(2)(D_{\min})}{\phi(0.6)F_{up}} = \frac{1.392(2)(1.20)}{0.75(0.6)(65\,\text{ksi})} = 0.114 \text{ in.}$$ $\leq t_{st} =$ 0.5 in. **o.k.**

$$t_{wc\,\min} = \frac{\varepsilon(2)(D_{\min})}{\phi(0.6)F_{uc}} = \frac{1.392(2)(1.20)}{0.75(0.6)(65\,\text{ksi})} = 0.114 \text{ in.}$$ $\leq t_{wc} =$ 0.83 in. **o.k.**

Stiffener & Doubler Example

- Solution:
  - 9. Summary



Conclusion

- Doublers are required to resist shear forces exceeding the panel zone shear strength
- AWS D1.8/D1.8M: 2016 has a prequalified doubler weld configuration
- Sufficient information is needed to check for web doublers
- Increasing the column size to eliminate stiffeners and doublers can be cost effective
- Stiffeners (also known as continuity plates) are required to resist applied loads greater than the local strength of the supporting member
- Stiffener applicable limit states are in *Specification* Section J10

Questions?

119

PDH Certificates

Within 2 business days…

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

PDH Certificates

Within 2 business days…

- Reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

There's always a solution in steel.

Thank You

Please give us your feedback!
Survey at conclusion of webinar.

structural STEEL