

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



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

This presentation is protected by US and International Copyright laws. Reproduction, distribution, display and use of the presentation without written permission of AISC is prohibited.

© The American Institute of Steel Construction 2018

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*.



Stiffeners and Doublers Oh My!

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



There's always a solution in steel.

Stiffeners and Doublers Summary

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

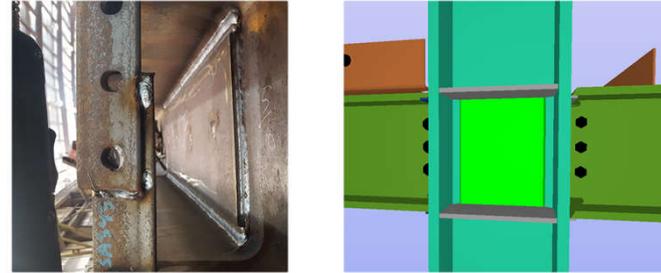


Stiffeners and Doublers

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

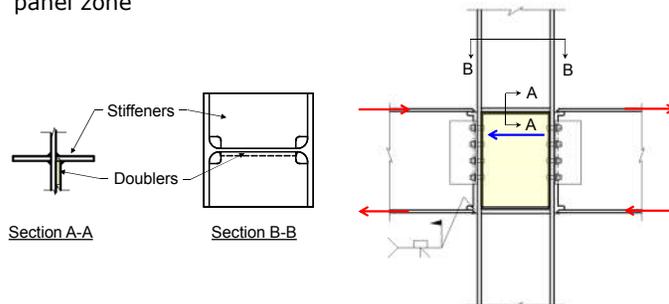


Stiffeners and Doublers

10

What is a Doubler?

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

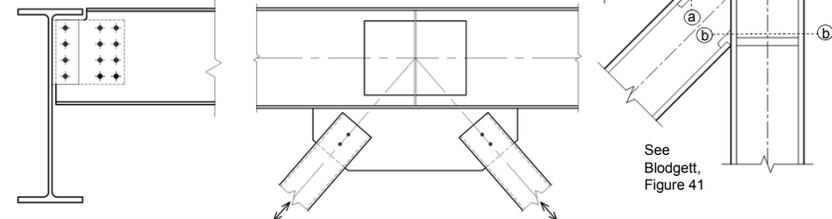


Stiffeners and Doublers

11

Doublers

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



Stiffeners and Doublers

12

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

Capacity

$V+$

W

l

Doubler Force = Required Force – Strength of Section

Stiffeners and Doublers
13

Shear Force and Stress

- Shear Flow at Top Edge of Doubler

C

$C+dC$

y_p

$v(t)(dz)$

dz

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

See Salmon and Johnson,
 Steel Structures Design and Behavior

Element Shear Stress

Stiffeners and Doublers
14

Shear Force and Stress

- Shear Flow at Top Edge of Doubler

Average web shear
 used for design of
 steel beams

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

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

Stiffeners and Doublers
15

Shear In a Member

- Shear Flow at Top Edge of Doubler

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

Stiffeners and Doublers
16

Doubler Configurations

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

Stiffeners and Doublers 17

Doublers

- Design Guide 13

Stiffeners and Doublers 18

Doubler Prep

- Design Guide 13

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

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

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

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

Stiffeners and Doublers 19

Flush Doublers: DG13

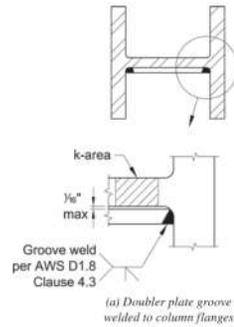
- Not a prequalified CJP weld
- Design Guide 13
 - $t_{dp} = 1/4"$ to $3/8"$ max:
 - C-L1a or C-L1a-GF with square cut edges (or beveled)
 - $t_{dp} > 1/4"$ to $3/8"$:
 - TC-U4a with beveled plates edges

$k_{det} - t_{fc}$	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

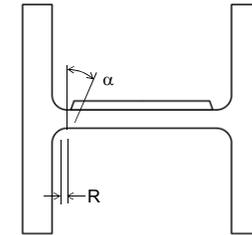


Stiffeners and Doublers

21

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	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



Stiffeners and Doublers

22

Flush Doubler Welds at Column Radius

DOUBLER MAY ENCR OACH INTO COLUMN FILLET BY ENCR. BY AISC ACCEPTABLE AMOUNT

PJP WELD NO TESTING REQUIRED

$\phi R_{weld} = (0.75)(0.6)(F_{exx})(E)$

DOUBLER MAY ENCR OACH INTO COLUMN FILLET BY ENCR. BY AISC ACCEPTABLE AMOUNT

PJP WELD NO TESTING REQUIRED. PREPARE A WELD PROCEDURE SPECIFICATION (WPS) AND CONFIRM FULL PENETRATION

$\phi R_{weld} = \text{FULL SHEAR STRENGTH}$

WELD PER AWS D1.8 CLAUSE 4.3

PJP WELD NO TESTING REQUIRED PER D1.8 CLAUSE 4.3

$\phi R_{weld} = \text{FULL SHEAR STRENGTH}$

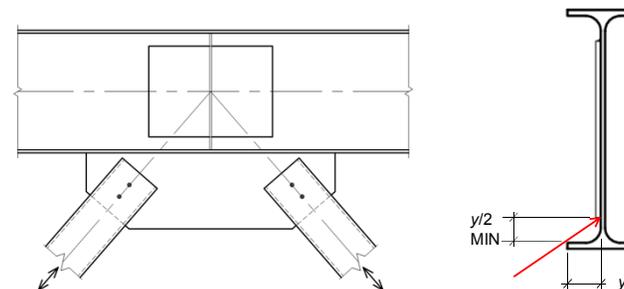


Stiffeners and Doublers

23

Shear In a Member

- Exceptions at Chevron



Stiffeners and Doublers

24

Shear In a Member

- Exceptions at Chevron

Stiffeners and Doublers

25

Doubler Extension

- Doubler Extension

Stiffeners and Doublers

26

Doubler Extension Seismic

- Doubler Extension

Stiffeners and Doublers

27

Wind and Low Seismic

- Doubler Termination

Stiffeners and Doublers

28



High Seismic

- Doubler Termination**

Weld not required except if Eq E3-7 is not satisfied by both column web alone and doubler plate alone (to limit buckling of the doublers)

$$t \geq \frac{(d_s + w_2)}{90} \quad \text{Eq. E3-7}$$

Develop 75% of the shear yielding strength of doubler

Stiffeners and Doublers 29

Continuous Doublers

- Doubler Termination**

E3.6f.2(c): Connect for lesser of:

- Sum of stiffener tensile strength
- Available shear strength of stiffener
- Available shear strength of doubler

Size weld for 50% of the stiffeners unbalanced force

Weld not required, Comm. E3, pg. 9.1-240

W12X10 (AISC minimum if no stiffeners)

Wind and Low Seismic

Stiffeners and Doublers 30

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

Long Products Group
Structural and Non-Structural
Products Only, subject to USG

PRICE LIST

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
W8X6	15-25	\$36.00
W8X4	10-15	\$36.00
W8X5 1/4	19-21	\$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	50-58	\$36.00
W12X12	65-136	\$39.50
W12X12	152-210	\$44.50
W12X12	230, 252	\$49.50
W14X5	22-28	\$36.00
W14X5 3/4	30-38	\$36.00
W14X5	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

Stiffeners and Doublers

Cost of Doublers – DG13 (1999)

(a) Partial-depth transverse stiffeners (Cases 1, 2, 3 and 4)

(b) Full-depth transverse stiffeners (Cases 5, 6 and 7)

(c) Web doubler plate (Cases 8, 9, 10 and 11)

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

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 ¹
4 PL 4 1/2 x 9-10 (ASTM A36) with one 1/2 x 1/2 corner clip each					
1	1/2 in.	fillet to bear	1/2-in. fillet welds	\$80	27
2	1 in.	fillet to bear	1/2-in. fillet welds	\$120	40
3	1/2 in.	1/2-in. fillet welds	1/2-in. fillet welds	\$90	30
4	1 in.	1/2-in. fillet welds	1/2-in. fillet welds	\$140	47
4 PL 4 1/2 x 11-10 1/2 (ASTM A36) with two 1/2 x 1/2 corner clips each					
5	1/2 in.	1/2-in. fillet welds	1/2-in. fillet welds	\$120	40
6	1 in.	1/2-in. fillet welds	1/2-in. fillet welds	\$210	71
7	1 1/2 in.	CJP groove weld	1/2-in. fillet welds	\$470	158
Web Doubler Plate (One) 1 PL 12 1/2 x 2-0 (ASTM A36)					
8	1/2 in.	CJP groove weld	1/2-in. fillet welds	\$245	82
9	3/4 in.	CJP groove weld	1/2-in. fillet welds	\$370	124
10	1 in.	1/2-in. fillet weld	1/2-in. fillet welds	\$215	72
11	1 in.	1/2-in. fillet weld	1/2-in. fillet welds	\$305	103

¹The 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.

²A 1/4-in. by 1/4-in. level 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.

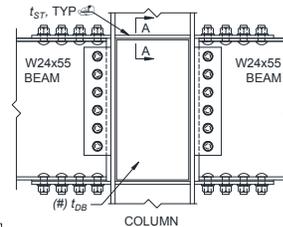
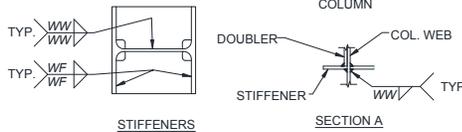
³A floor-to-floor height of 14 ft has been used in this tabulation.

Stiffeners and Doublers

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 and Doublers

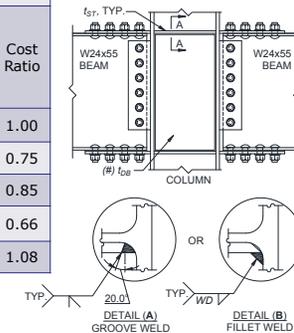
33

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



Stiffeners and Doublers

34

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.



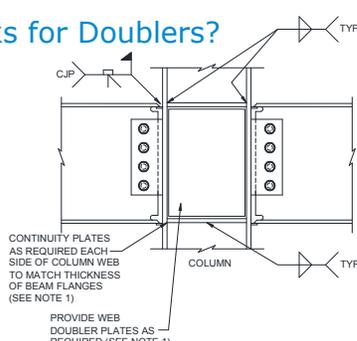
Stiffeners and Doublers

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.”



CONTINUITY PLATES AS REQUIRED EACH SIDE OF COLUMN WEB TO MATCH THICKNESS OF BEAM FLANGES (SEE NOTE 1)

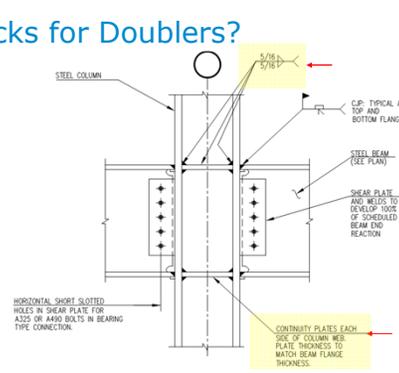
PROVIDE WEB DOUBLER PLATES AS REQUIRED (SEE NOTE 1)

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 CONNECTION
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

Stiffeners and Doublers 39

Check for Doublers Determine Column Panel Zone Shear Strength

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

(P_c)_r = 0 kips Column Axial Load

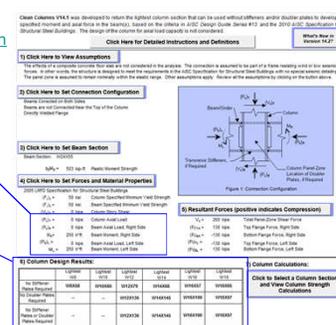
(P_c)_s = 0 kips Beam Axial Load, Right Side

M_q = 250 k-ft Beam Moment, Right Side

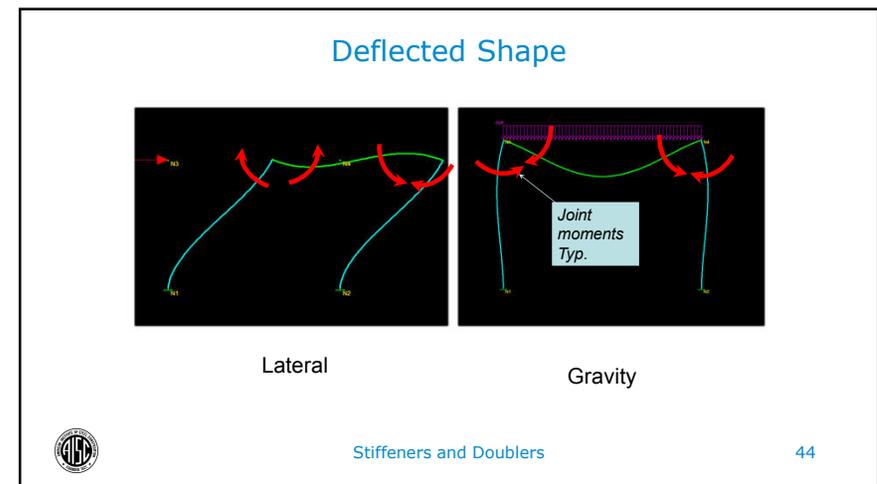
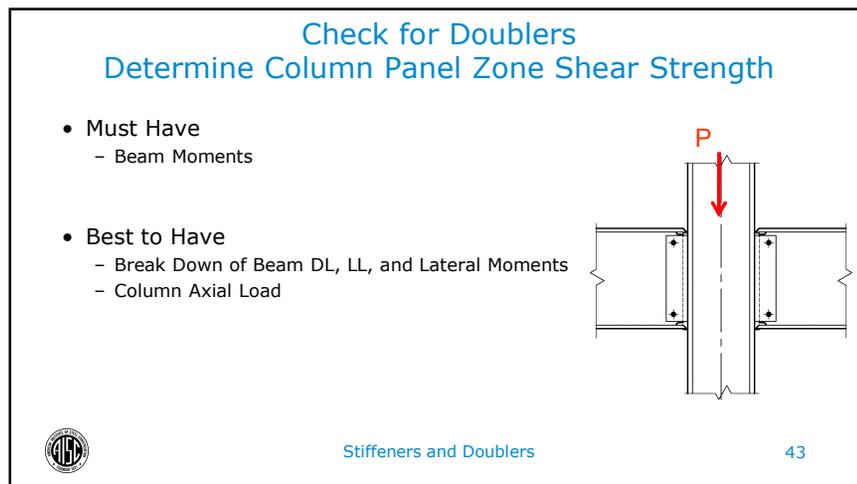
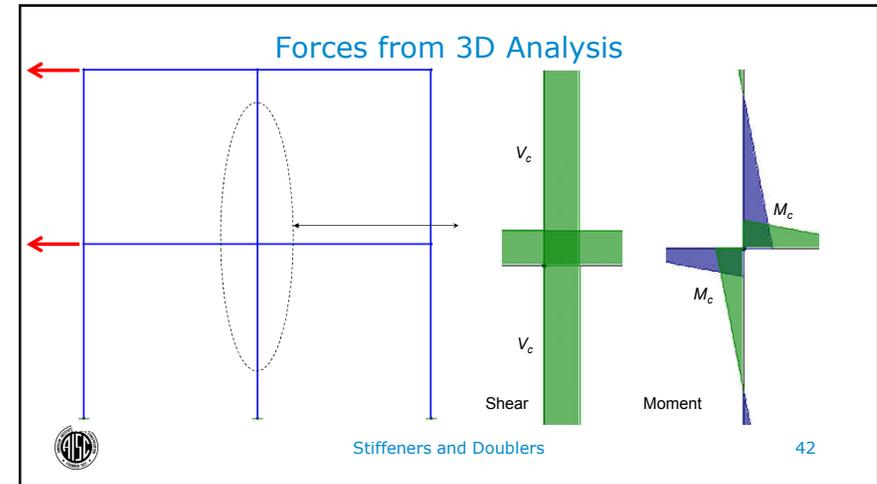
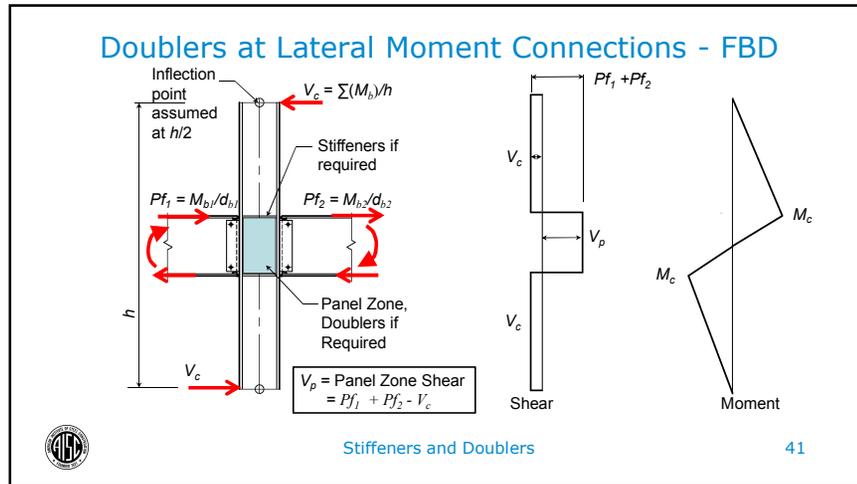
(P_b)_r = 0 kips Beam Axial Load, Left Side

M_l = 250 k-ft Beam Moment, Left Side

6) Column Design Results:						
	Lightest W10	Lightest W12	Lightest W14	Lightest W16	Lightest W18	Lightest W24
No Stiffener Plates Required	WBX58	W10X60	W12X79	W14X68	W16X57	W18X65
No Doubler Plates Required	--	--	W12X136	W14X145	W16X100	W18X97
No Stiffener Plates or Doubler Plates Required	--	--	W12X136	W14X145	W16X100	W18X97



Stiffeners and Doublers 40



Deflected Shape

Stiffeners and Doublers

45

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

$$V_{p \max} = \text{Panel Zone Shear}$$

$$= 2(\phi M_c)/d_b - 2(\phi M_c)/h$$

Stiffeners and Doublers

46

Moment Connections – Doublers

Top of Column

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

$$V_{c \max} = \text{Panel Zone Shear}$$

$$= \phi M_c / d_b$$

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%



Stiffeners and Doublers

49

Give Actual Loads



1 1/2" Stiffeners on a
W14x90 Column

Courtesy of Larry Muir and Bill Thornton



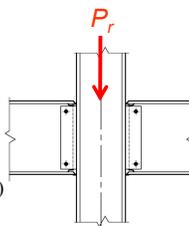
Stiffeners and Doublers

50

Check for Doublers

Determine Column Panel Zone Shear Strength, J10.6

- Elastic Range: $\phi = 0.9$
 $\alpha P_r \leq 0.4P_y$: $R_n = 0.6F_y d_c t_w$ $\Omega = 1.67$ (J10-9)
- $\alpha P_r > 0.4P_y$: $R_n = 0.6F_y d_c t_w \left(1.4 - \frac{\alpha P_r}{P_y}\right)$ (J10-10)
- Plastic Range:
 $\alpha P_r \leq 0.75P_y$: $R_n = 0.6F_y d_c t_w \left(1 + \frac{3b_c t_{cf}^2}{d_b d_c t_w}\right)$ (J10-11)
- $\alpha P_r > 0.75P_y$: $R_n = 0.6F_y d_c t_w \left(1 + \frac{3b_c t_{cf}^2}{d_b d_c t_w}\right) \left(1.9 - \frac{1.2\alpha P_r}{P_y}\right)$ (J10-12)



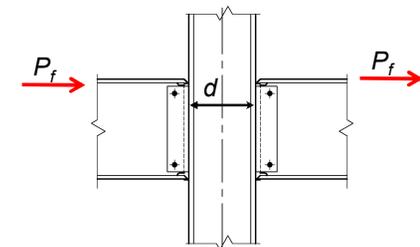
Stiffeners and Doublers

51

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



Stiffeners and Doublers

52



Check for Doublers Determine Column Panel Zone Shear Strength

- Consider beam shear when beam frames into doubler

Alternate 1 Distribution

Alternate 2 Distribution, DG13

Stiffeners and Doublers 53

Doubler Web Buckling

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

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

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

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

W-SHAPED BEAM
COLUMN

Stiffeners and Doublers 54

Doubler Web Buckling

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

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

$$t \geq \frac{127}{\sqrt{F_y}} \quad \text{Engineering for Steel Construction, 1984}$$

Based on 1978 *Spec* Section 1.18.2.3 for Compression of Built-up Members

W-SHAPED BEAM
COLUMN

Stiffeners and Doublers 55

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

W-SHAPED BEAM
COLUMN

Stiffeners and Doublers 56

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

Stiffeners and Doublers 57

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 and Doublers 58

Stiffeners

- Stiffener Plates

Stiffeners and Doublers 59

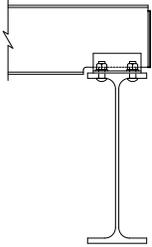
Stiffeners

- Other Common Applications

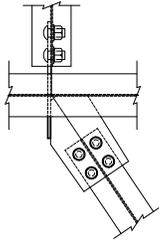
Stiffeners and Doublers 60

Stiffeners

- Other Common Applications



Stability
(J10.7)



To Direct Force





Stiffeners and Doublers

61

Stiffeners

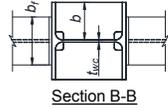
- Stiffeners, *Spec J10.8*
 - Minimum width, b :

$$b \geq \frac{b_f}{3} - \frac{t_{wc}}{2}$$
 - Minimum thickness, t :

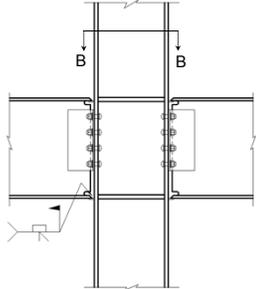
$$t \geq \frac{t_f}{2}$$

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

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



Section B-B





Stiffeners and Doublers

62

Stiffeners

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

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

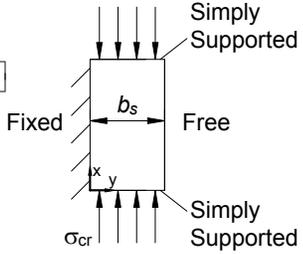
$D_x = 8,000 \text{ ksi}$ $D_{xy} = 16,000 \text{ ksi}$

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

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

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

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





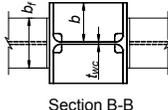
Stiffeners and Doublers

63

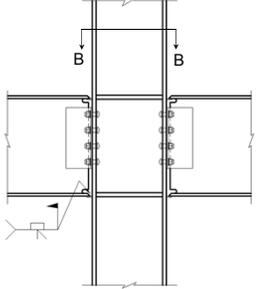
Stiffeners

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

$$b \geq \frac{b_f}{2} - \frac{t_{wc}}{2}$$
 - Minimum thickness, t :
 - One sided connections: $t \geq \frac{t_f}{2}$
 - Two sided connections: $t \geq \frac{3t_f}{4}$



Section B-B



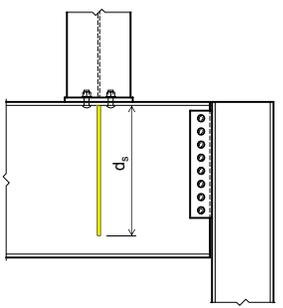


Stiffeners and Doublers

64

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**, **1/2-depth stiffener increased to 3/4-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

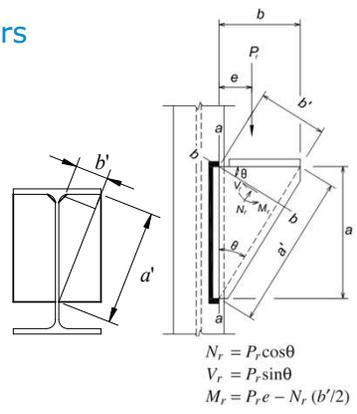


Stiffeners and Doublers

65

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

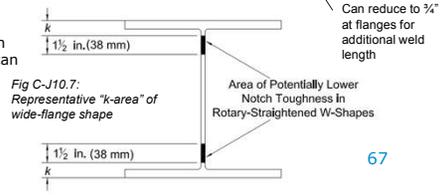
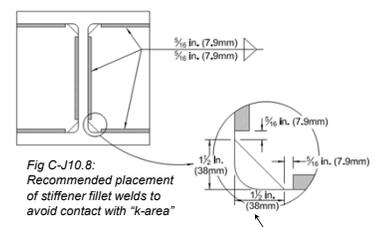


Stiffeners and Doublers

66

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

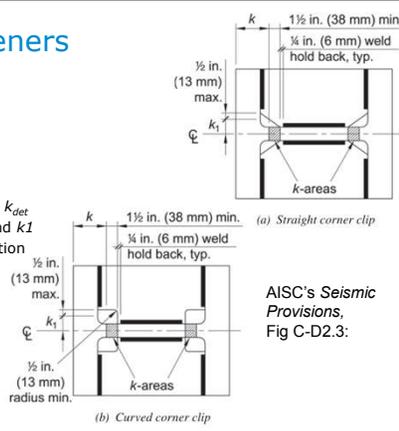


Stiffeners and Doublers

67

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



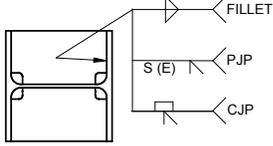
Stiffeners and Doublers

68



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)



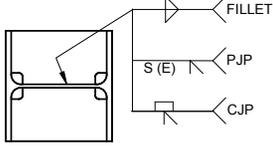


Stiffeners and Doublers

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)



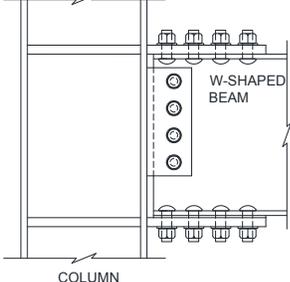


Stiffeners and Doublers

70

Stiffeners/Continuity Plates

- Stiffener Design
 - AISC Specification Chapter J
 - $P_{stiffener} = P_r$ - minimum ϕR_n LRFD
 - $P_{stiffener} = P_r$ - 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





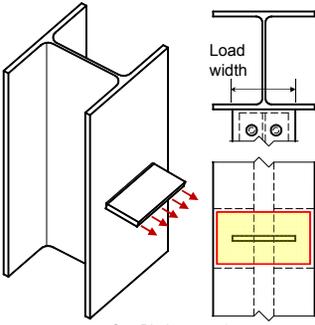
Stiffeners and Doublers

71

Stiffener Design

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

$$R_n = 6.25F_y t_f^2$$
 - $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (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





Stiffeners and Doublers

72

Stiffeners

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

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

$$R_n = F_{yw}t_w(2.5k_{des} + l_b) \quad \text{Spec Equation J10-3, Force } \leq d \text{ from end of column}$$

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

l_b = bearing length, in.

Stiffeners and Doublers 73

Stiffeners

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

$$R_n = 0.80t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f \quad \text{Spec Equation J10-4, Force applied distance } \geq d/2 \text{ from end of member}$$

$$R_n = 0.40t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f \quad \text{Spec Equation J10-5a, Force applied distance } < d/2 \text{ from end of member, } l_b/d \leq 0.2$$

$$R_n = 0.40t_w^2 \left[1 + \left(\frac{4l_b}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f \quad \text{Spec Equation J10-5b, Force applied distance } < d/2 \text{ from end of member, } l_b/d > 0.2$$

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

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

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (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 $l_b/d > \text{approximately } 1$, Use Chapter E
 - See Chapter C and Appendix 6 for Stability

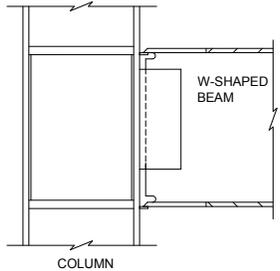
See Spec Appendix Section 6.2.2 for point bracing (nodal bracing) required strength and stiffness

Test Set-up from Chen and Oppenheim 1970 76

Stiffeners and Doublers

Stiffeners/Continuity Plates

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



COLUMN

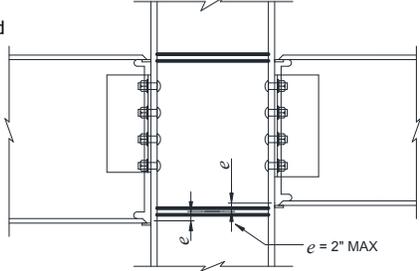
W-SHAPED BEAM


Stiffeners and Doublers
77

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




Stiffeners and Doublers
78

Web Sidesway Buckling – Beams

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

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

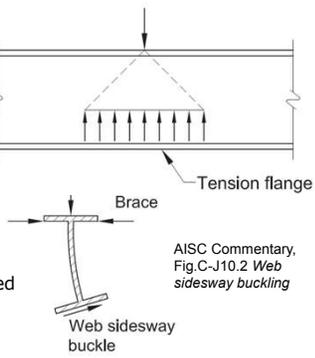
Equation J10-6, Compression Flange restrained, applicable if $(h/t_w)(L_b/b_f) \leq 2.3$

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

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

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

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



Tension flange

Brace

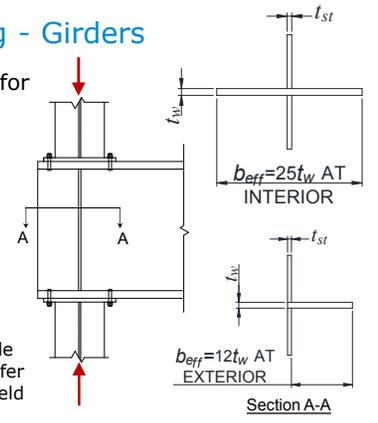
Web sidesway buckle

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


Stiffeners and Doublers
79

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



Section A-A

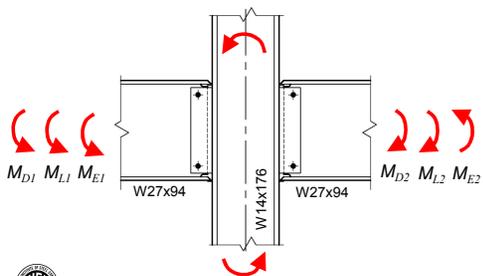
$b_{eff} = 25 t_w$ AT INTERIOR

$b_{eff} = 12 t_w$ AT EXTERIOR


Stiffeners and Doublers
80

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



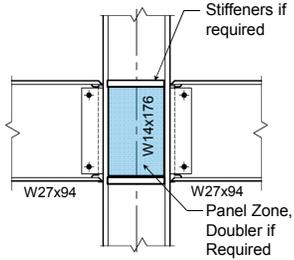
$M_{D1} = 130$ kip-ft
$M_{L1} = 220$ kip-ft
$M_{E1} = 690$ kip-ft
$M_{D2} = 130$ kip-ft
$M_{L2} = 220$ kip-ft
$M_{E2} = 690$ kip-ft

Stiffeners and Doublers

81

Stiffener & Doubler Example

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



Stiffeners and Doublers

82

Stiffener & Doubler Example

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

1. 1.4D	= 182 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
2. 1.2D + 1.6L + 0.5(L or S or R)	= 508 kip-ft	
3. 1.2D + 1.6(L or S or R) + (L or 0.5W)	= 376 kip-ft	
4. 1.2D + 1.0W + L + 0.5(L 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	

Governing Load Combination: 5. 1.2D + 1.0E + L + 0.2S
 $M_1 = M_2 = 1.2(130 \text{ kip-ft}) + 1.0(690 \text{ kip-ft}) + 1.0(220 \text{ kip-ft}) = 1070 \text{ kip-ft}$

Stiffeners and Doublers

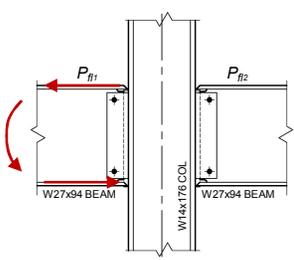
83

Stiffener & Doubler Example

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

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

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



Stiffeners and Doublers

84

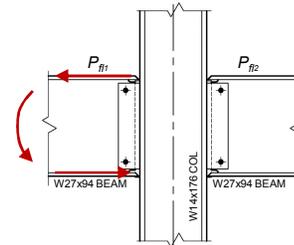
Stiffener & Doubler Example

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

$$P_{fl} = P_{fl1} = P_{fl2} = \frac{M_1}{d_b - t_{fb}}$$

$$= \frac{1070 \text{ kip-ft}}{26.9 \text{ in.} - 0.745 \text{ in.}}$$

$$= 491 \text{ kips}$$



Stiffeners and Doublers

85

Stiffener & Doubler Example

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

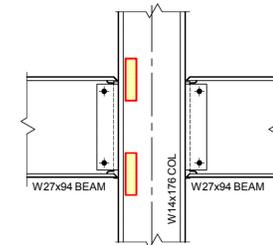
$$\phi R_{wy} = \phi F_{yw} t_{wc} (5k_{des} + l_b)$$

$$= 1.00(50 \text{ ksi})(0.830 \text{ in.})(5(1.91 \text{ in.}) + (0.745 \text{ in.}))$$

$$= 427 \text{ kips} < P_{fl} = 491 \text{ kips} \text{ n.g.}$$

Stiffeners Required

Note: $l_b = t_{fb}$



Stiffeners and Doublers

86

Stiffener & Doubler Example

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

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

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

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

Note: $l_b = t_{fb}$



Stiffeners and Doublers

87

Stiffener & Doubler Example

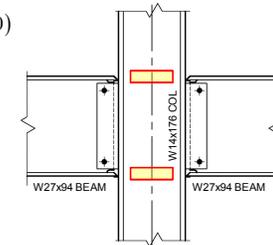
- Solution:
 - 3. Check Stiffener Limit States
 - Web Compression Buckling (J10.5) ($\phi = 0.90$)
 - AISC Specification Eq. J10-8

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

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

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

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



Stiffeners and Doublers

88

Stiffener & Doubler Example

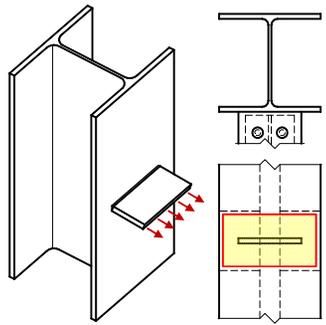
- Solution:
 - 3. Check Stiffener Limit States
 - Flange Local Bending (J10.1) ($\phi = 0.90$)
 - AISC Specification Eq. J10-1

$$\phi R_{fb} = \phi 6.25 F_{yc} l_f^2$$

$$= 0.90 (6.25) (50 \text{ ksi}) (1.31 \text{ in.})^2$$

$$= 483 \text{ kips} < P_{fi} = 491 \text{ kips} \text{ n.g.}$$

Stiffeners Required



See Blodgett 5.7-8



Use of Manual Tables

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

LRFD

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

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

$$\phi R_n = P_{wo} + P_{wi} l_b$$

$$= 396 \text{ kips} + \left(41.5 \frac{\text{kips}}{\text{in.}} \right) (0.745 \text{ in.})$$

$$= 427 \text{ kips} < P_{fi} = 491 \text{ kips} \text{ n.g.}$$

Stiffeners Required

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

Shape	WT							
Depth	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	2200	2900	2200	2900	2200	2900	2200	2900
8	2700	3600	2700	3600	2700	3600	2700	3600
10	3300	4400	3300	4400	3300	4400	3300	4400
12	4000	5300	4000	5300	4000	5300	4000	5300
14	4800	6300	4800	6300	4800	6300	4800	6300
16	5700	7400	5700	7400	5700	7400	5700	7400
18	6700	8600	6700	8600	6700	8600	6700	8600
20	7800	9900	7800	9900	7800	9900	7800	9900
22	9000	11300	9000	11300	9000	11300	9000	11300
24	10300	12800	10300	12800	10300	12800	10300	12800
26	11700	14400	11700	14400	11700	14400	11700	14400
28	13200	16100	13200	16100	13200	16100	13200	16100
30	14800	17900	14800	17900	14800	17900	14800	17900
36	19000	23000	19000	23000	19000	23000	19000	23000
42	23000	28000	23000	28000	23000	28000	23000	28000
48	27000	33000	27000	33000	27000	33000	27000	33000
54	31000	38000	31000	38000	31000	38000	31000	38000
60	35000	43000	35000	43000	35000	43000	35000	43000
66	39000	48000	39000	48000	39000	48000	39000	48000
72	43000	53000	43000	53000	43000	53000	43000	53000
78	47000	58000	47000	58000	47000	58000	47000	58000
84	51000	63000	51000	63000	51000	63000	51000	63000
90	55000	68000	55000	68000	55000	68000	55000	68000
96	59000	73000	59000	73000	59000	73000	59000	73000
102	63000	78000	63000	78000	63000	78000	63000	78000
108	67000	83000	67000	83000	67000	83000	67000	83000
114	71000	88000	71000	88000	71000	88000	71000	88000
120	75000	93000	75000	93000	75000	93000	75000	93000
126	79000	98000	79000	98000	79000	98000	79000	98000
132	83000	103000	83000	103000	83000	103000	83000	103000
138	87000	108000	87000	108000	87000	108000	87000	108000
144	91000	113000	91000	113000	91000	113000	91000	113000
150	95000	118000	95000	118000	95000	118000	95000	118000
156	99000	123000	99000	123000	99000	123000	99000	123000
162	103000	128000	103000	128000	103000	128000	103000	128000
168	107000	133000	107000	133000	107000	133000	107000	133000
174	111000	138000	111000	138000	111000	138000	111000	138000
180	115000	143000	115000	143000	115000	143000	115000	143000
186	119000	148000	119000	148000	119000	148000	119000	148000
192	123000	153000	123000	153000	123000	153000	123000	153000
198	127000	158000	127000	158000	127000	158000	127000	158000
204	131000	163000	131000	163000	131000	163000	131000	163000
210	135000	168000	135000	168000	135000	168000	135000	168000
216	139000	173000	139000	173000	139000	173000	139000	173000
222	143000	178000	143000	178000	143000	178000	143000	178000
228	147000	183000	147000	183000	147000	183000	147000	183000
234	151000	188000	151000	188000	151000	188000	151000	188000
240	155000	193000	155000	193000	155000	193000	155000	193000
246	159000	198000	159000	198000	159000	198000	159000	198000
252	163000	203000	163000	203000	163000	203000	163000	203000
258	167000	208000	167000	208000	167000	208000	167000	208000
264	171000	213000	171000	213000	171000	213000	171000	213000
270	175000	218000	175000	218000	175000	218000	175000	218000
276	179000	223000	179000	223000	179000	223000	179000	223000
282	183000	228000	183000	228000	183000	228000	183000	228000
288	187000	233000	187000	233000	187000	233000	187000	233000
294	191000	238000	191000	238000	191000	238000	191000	238000
300	195000	243000	195000	243000	195000	243000	195000	243000
306	199000	248000	199000	248000	199000	248000	199000	248000
312	203000	253000	203000	253000	203000	253000	203000	253000
318	207000	258000	207000	258000	207000	258000	207000	258000
324	211000	263000	211000	263000	211000	263000	211000	263000
330	215000	268000	215000	268000	215000	268000	215000	268000
336	219000	273000	219000	273000	219000	273000	219000	273000
342	223000	278000	223000	278000	223000	278000	223000	278000
348	227000	283000	227000	283000	227000	283000	227000	283000
354	231000	288000	231000	288000	231000	288000	231000	288000
360	235000	293000	235000	293000	235000	293000	235000	293000
366	239000	298000	239000	298000	239000	298000	239000	298000
372	243000	303000	243000	303000	243000	303000	243000	303000
378	247000	308000	247000	308000	247000	308000	247000	308000
384	251000	313000	251000	313000	251000	313000	251000	313000
390	255000	318000	255000	318000	255000	318000	255000	318000
396	259000	323000	259000	323000	259000	323000	259000	323000
402	263000	328000	263000	328000	263000	328000	263000	328000
408	267000	333000	267000	333000	267000	333000	267000	333000
414	271000	338000	271000	338000	271000	338000	271000	338000
420	275000	343000	275000	343000	275000	343000	275000	343000
426	279000	348000	279000	348000	279000	348000	279000	348000
432	283000	353000	283000	353000	283000	353000	283000	353000
438	287000	358000	287000	358000	287000	358000	287000	358000
444	291000	363000	291000	363000	291000	363000	291000	363000
450	295000	368000	295000	368000	295000	368000	295000	368000
456	299000	373000	299000	373000	299000	373000	299000	373000
462	303000	378000	303000	378000	303000	378000	303000	378000
468	307000	383000	307000	383000	307000	383000	307000	383000
474	311000	388000	311000	388000	311000	388000	311000	388000
480	315000	393000	315000	393000	315000	393000	315000	393000
486	319000	398000	319000	398000	319000	398000	319000	398000
492	323000	403000	323000	403000	323000	403000	323000	403000
498	327000	408000	327000	408000	327000	408000	327000	408000
504	331000	413000	331000	413000	331000	413000	331000	413000
510	335000	418000	335000	418000	335000	418000	335000	418000
516	339000	423000	339000	423000	339000	423000	339000	423000
522	343000	428000	343000	428000	343000	428000	343000	428000
528	347000	433000	347000	433000	347000	433000	347000	433000
534	351000	438000	351000	438000	351000	438000	351000	438000
540	355000	443000	355000	443000	355000	443000	355000	443000
546	359000	448000	359000	448000	359000	448000	359000	448000
552	363000	453000	363000	453000	363000	453000	363000	4530

Use of Manual Tables

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

LRFD

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

$\phi R_n = P_{fb}$

= 483 kips < $P_{ft} = 491$ kips **n.g.**

Stiffeners Required

Shape	20F	24F	28F	30F	36F	42F	48F	60F	66F	72F	84F	96F	108F	120F	132F	144F	150F
W10	100	110	120	130	140	150	160	170	180	190	200	210	220	230	240	250	260
W12	120	130	140	150	160	170	180	190	200	210	220	230	240	250	260	270	280
W14	140	150	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300
W16	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320
W18	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340
W20	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360
W22	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380
W24	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380	390	400
W26	260	270	280	290	300	310	320	330	340	350	360	370	380	390	400	410	420
W28	280	290	300	310	320	330	340	350	360	370	380	390	400	410	420	430	440
W30	300	310	320	330	340	350	360	370	380	390	400	410	420	430	440	450	460
W36	360	370	380	390	400	410	420	430	440	450	460	470	480	490	500	510	520
W40	400	410	420	430	440	450	460	470	480	490	500	510	520	530	540	550	560
W44	440	450	460	470	480	490	500	510	520	530	540	550	560	570	580	590	600
W48	480	490	500	510	520	530	540	550	560	570	580	590	600	610	620	630	640
W54	540	550	560	570	580	590	600	610	620	630	640	650	660	670	680	690	700
W60	600	610	620	630	640	650	660	670	680	690	700	710	720	730	740	750	760
W66	660	670	680	690	700	710	720	730	740	750	760	770	780	790	800	810	820
W72	720	730	740	750	760	770	780	790	800	810	820	830	840	850	860	870	880
W78	780	790	800	810	820	830	840	850	860	870	880	890	900	910	920	930	940
W84	840	850	860	870	880	890	900	910	920	930	940	950	960	970	980	990	1000

93

Stiffener & Doubler Example

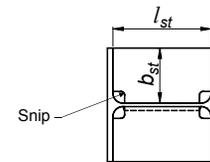
- Solution:**
 - 4. Column Stiffener Design
 - Determine Force in the Stiffeners

$$P_{st1tot} = P_{st2tot} = P_{ft} - \min(\phi R_{wp}, \phi R_{wc}, \phi R_{wb}, \phi R_{fb})$$

= 491 kips - 427 kips
= 64 kips
 - Determine Beam Force Per Each Stiffener

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

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



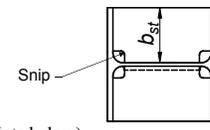
94

Stiffener & Doubler Example

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

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

$$b_{st \text{ max}} = \frac{b_{fc} - t_{wc}}{2} = \frac{15.7 \text{ in.} - 0.830 \text{ in.}}{2} = 7.44 \text{ in. Max}$$
 - Use $b_{st} = 7"$ Stiffener
 - *Note: Use b_{pl} in lieu of b_{fb} if moment connection plate delivering the load



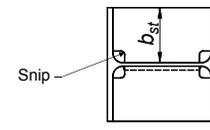
95

Stiffener & Doubler Example

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

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

$$t_{st \text{ min}} = \frac{t_{fb}}{2} = \frac{0.745 \text{ in.}}{2} = 0.373 \text{ in. (See Note below)}$$
 - Try a 1/2" Stiffener
 - *Note: Use t_{pl} in lieu of t_{fb} if moment connection plate delivering the load



96

Stiffener & Doubler Example

• Solution:

– 4. Column Stiffener Design

- Check Minimum Thickness Due to Tension Yielding
– AISC Specification Section J10.8

$$snip = \max(k_{des} - t_{fs}, k_{ts} - \frac{t_{wc}}{2}, 1.5 \text{ in.})$$

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

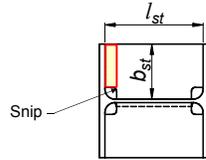
$$= 1.5 \text{ in.}$$

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

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

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

*Note: Can reduce snip at flange if needed.



Stiffener & Doubler Example

• Solution:

– 4. Column Stiffener Design

- Check Minimum Thickness Due to Compression Buckling
– AISC Specification Section J10.8 and E3 ($\phi = 0.90$)

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

$$k = 0.75$$

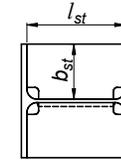
$$\frac{kl}{r} = \frac{kl_{st}\sqrt{12}}{t_{st}} = \frac{0.75(12.6 \text{ in.})\sqrt{12}}{(0.5 \text{ in.})} = 65.5$$

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

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

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

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



Stiffener & Doubler Example

• Solution:

– 4. Column Stiffener Design

- Check Minimum Thickness Due to Compression Buckling
– AISC Specification Section E3 ($\phi = 0.90$)

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

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

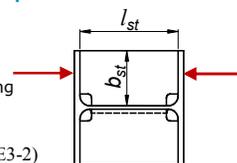
$$\text{Rearrange } \phi P_n = \phi F_{cr} A_g \text{ to Solve for } t_{st \text{ min}} \quad (\text{E3-1})$$

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

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

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

*Note: Buckling limit state included for example purposes.



Use of Manual Tables

– AISC Manual Table 4-14:
KL/r Table

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

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

$$F_{cr} = \left(0.658 \frac{F_y}{F_e}\right) F_y \quad (\text{E3-2})$$

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

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

$F_y = 50 \text{ ksi}$	
$\frac{KL}{r}$	F_{cr}
ksi	ksi
ASD	LRFD
65	22.0
66	21.8
67	33.0
68	32.7

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

F_y	$F_y = 50 \text{ ksi}$		$F_y = 60 \text{ ksi}$		$F_y = 70 \text{ ksi}$		$F_y = 80 \text{ ksi}$		$F_y = 90 \text{ ksi}$	
	ASD	LRFD								
41	19.2	20.1	19.7	20.7	20.6	21.7	21.5	22.8	23.3	24.8
42	19.2	20.1	19.7	20.7	20.6	21.7	21.5	22.8	23.3	24.8
43	19.1	20.1	19.6	20.6	20.5	21.6	20.4	21.7	22.2	23.7
44	19.0	20.0	19.5	20.5	20.4	21.5	20.3	21.6	22.1	23.6
45	18.9	20.0	19.4	20.4	20.3	21.4	20.2	21.5	22.0	23.5
46	18.8	20.0	19.3	20.3	20.2	21.3	20.1	21.4	21.9	23.4
47	18.7	20.0	19.2	20.2	20.1	21.2	20.0	21.3	21.8	23.3
48	18.6	20.0	19.1	20.1	20.0	21.1	19.9	21.2	21.7	23.2
49	18.5	20.0	19.0	20.0	19.9	21.0	19.8	21.1	21.6	23.1
50	18.4	20.0	18.9	20.0	19.8	21.0	19.7	21.1	21.6	23.1
51	18.3	20.0	18.8	20.0	19.7	21.0	19.6	21.1	21.6	23.1
52	18.2	20.0	18.7	20.0	19.6	21.0	19.5	21.1	21.6	23.1
53	18.1	20.0	18.6	20.0	19.5	21.0	19.4	21.1	21.6	23.1
54	18.0	20.0	18.5	20.0	19.4	21.0	19.3	21.1	21.6	23.1
55	17.9	20.0	18.4	20.0	19.3	21.0	19.2	21.1	21.6	23.1
56	17.8	20.0	18.3	20.0	19.2	21.0	19.1	21.1	21.6	23.1
57	17.7	20.0	18.2	20.0	19.1	21.0	19.0	21.1	21.6	23.1
58	17.6	20.0	18.1	20.0	19.0	21.0	18.9	21.1	21.6	23.1
59	17.5	20.0	18.0	20.0	18.9	21.0	18.8	21.1	21.6	23.1
60	17.4	20.0	17.9	20.0	18.8	21.0	18.7	21.1	21.6	23.1
61	17.3	20.0	17.8	20.0	18.7	21.0	18.6	21.1	21.6	23.1
62	17.2	20.0	17.7	20.0	18.6	21.0	18.5	21.1	21.6	23.1
63	17.1	20.0	17.6	20.0	18.5	21.0	18.4	21.1	21.6	23.1
64	17.0	20.0	17.5	20.0	18.4	21.0	18.3	21.1	21.6	23.1
65	16.9	20.0	17.4	20.0	18.3	21.0	18.2	21.1	21.6	23.1
66	16.8	20.0	17.3	20.0	18.2	21.0	18.1	21.1	21.6	23.1
67	16.7	20.0	17.2	20.0	18.1	21.0	18.0	21.1	21.6	23.1
68	16.6	20.0	17.1	20.0	18.0	21.0	17.9	21.1	21.6	23.1
69	16.5	20.0	17.0	20.0	17.9	21.0	17.8	21.1	21.6	23.1
70	16.4	20.0	16.9	20.0	17.8	21.0	17.7	21.1	21.6	23.1
71	16.3	20.0	16.8	20.0	17.7	21.0	17.6	21.1	21.6	23.1
72	16.2	20.0	16.7	20.0	17.6	21.0	17.5	21.1	21.6	23.1
73	16.1	20.0	16.6	20.0	17.5	21.0	17.4	21.1	21.6	23.1
74	16.0	20.0	16.5	20.0	17.4	21.0	17.3	21.1	21.6	23.1
75	15.9	20.0	16.4	20.0	17.3	21.0	17.2	21.1	21.6	23.1
76	15.8	20.0	16.3	20.0	17.2	21.0	17.1	21.1	21.6	23.1
77	15.7	20.0	16.2	20.0	17.1	21.0	17.0	21.1	21.6	23.1
78	15.6	20.0	16.1	20.0	17.0	21.0	16.9	21.1	21.6	23.1
79	15.5	20.0	16.0	20.0	16.9	21.0	16.8	21.1	21.6	23.1
80	15.4	20.0	15.9	20.0	16.8	21.0	16.7	21.1	21.6	23.1
81	15.3	20.0	15.8	20.0	16.7	21.0	16.6	21.1	21.6	23.1
82	15.2	20.0	15.7	20.0	16.6	21.0	16.5	21.1	21.6	23.1
83	15.1	20.0	15.6	20.0	16.5	21.0	16.4	21.1	21.6	23.1
84	15.0	20.0	15.5	20.0	16.4	21.0	16.3	21.1	21.6	23.1
85	14.9	20.0	15.4	20.0	16.3	21.0	16.2	21.1	21.6	23.1
86	14.8	20.0	15.3	20.0	16.2	21.0	16.1	21.1	21.6	23.1
87	14.7	20.0	15.2	20.0	16.1	21.0	16.0	21.1	21.6	23.1
88	14.6	20.0	15.1	20.0	16.0	21.0	15.9	21.1	21.6	23.1
89	14.5	20.0	15.0	20.0	15.9	21.0	15.8	21.1	21.6	23.1
90	14.4	20.0	14.9	20.0	15.8	21.0	15.7	21.1	21.6	23.1



Stiffener & Doubler Example

• Solution:

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

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

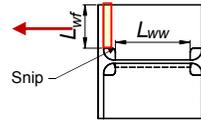
$$L_{wvf} = b_{st} - snip = 7 \text{ in.} - 1.5 \text{ in.} = 5.5 \text{ in.}$$

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

$$\epsilon = 1.392 \text{ LFRD}$$

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

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



Stiffeners and Doublers

101

Stiffener & Doubler Example

• Solution:

- 4. Column Stiffener Design

Note:

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

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

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

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

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



Stiffeners and Doublers

102

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

ASCE 7-10

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

$$M_{D1} = 130 \text{ kip-ft}$$

$$M_{L1} = 220 \text{ kip-ft}$$

$$M_{E1} = 690 \text{ kip-ft}$$

$$M_{D2} = 130 \text{ kip-ft}$$

$$M_{L2} = 220 \text{ kip-ft}$$

$$M_{E2} = 690 \text{ kip-ft}$$



Stiffeners and Doublers

103

Stiffener & Doubler Example

• Solution:

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

Governing Load Combination 5:

$$M_{1.5} = 1.2(130 \text{ k-ft}) + 1.0(690 \text{ k-ft}) + 1.0(220 \text{ k-ft}) = 1070 \text{ kip-ft}$$

$$M_{2.5} = 1.2(-130 \text{ k-ft}) + 1.0(690 \text{ k-ft}) = 534 \text{ kip-ft}$$

$$P_{\beta 1.5} = \frac{M_1}{d_{b1} - t_{f\beta 1}} = \frac{1070 \text{ kip-ft}}{26.9 \text{ in.} - 0.745 \text{ in.}} = 491 \text{ kip}$$

$$P_{\beta 2.5} = \frac{M_2}{d_{b2} - t_{f\beta 2}} = \frac{534 \text{ kip-ft}}{26.9 \text{ in.} - 0.745 \text{ in.}} = 245 \text{ kip (Stiffener not required)}$$

$$P_{fd} = P_{\beta 1.5} + P_{\beta 2.5} = 491 \text{ kips} + 245 \text{ kips} = 736 \text{ kips}$$

$$M_{D1} = 130 \text{ kip-ft}$$

$$M_{L1} = 220 \text{ kip-ft}$$

$$M_{E1} = 690 \text{ kip-ft}$$

$$M_{D2} = 130 \text{ kip-ft}$$

$$M_{L2} = 220 \text{ kip-ft}$$

$$M_{E2} = 690 \text{ kip-ft}$$



Stiffeners and Doublers

104

Stiffener & Doubler Example

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

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

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

$$\Sigma M_c = M_{c_t} + M_{c_b} = 1200 \text{ kip-ft} + 1200 \text{ kip-ft} = 2400 \text{ kip-ft}$$

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



Stiffeners and Doublers

105

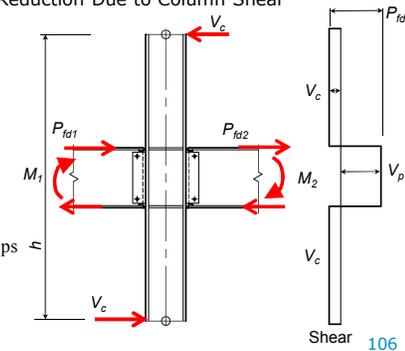
Stiffener & Doubler Example

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

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

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

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



Stiffeners and Doublers

106

Stiffener & Doubler Example

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

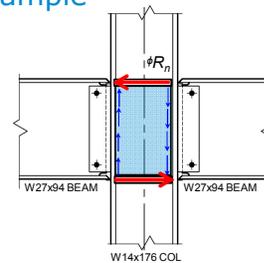
$$\alpha = 1.0 \text{ (LRFD)}; 1.6 \text{ (ASD)}$$

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

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

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

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



Stiffeners and Doublers

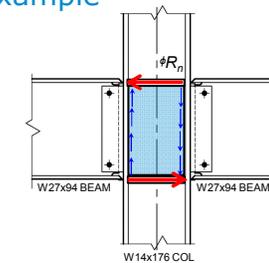
107

Stiffener & Doubler Example

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

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

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



Stiffeners and Doublers

108

Stiffener & Doubler Example

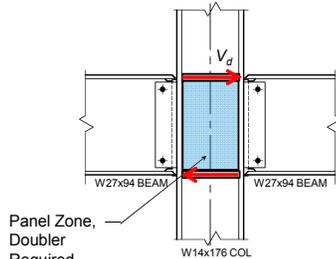
- Solution:
 - 7. Determine Doubler Shear Force

$$V_d = V_p - \phi R_n$$

$$= 629 \text{ kips} - 341 \text{ kips}$$

$$= 288 \text{ kips}$$

Number of Doublers, $n_d = 1$



109

Stiffeners and Doublers

Stiffener & Doubler Example

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

For, $C_v = 1.0$

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

110

Stiffeners and Doublers

Stiffener & Doubler Example

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

$$t_{d \text{ min } 2} = \frac{V_d(C_v)}{\phi(0.6)(F_{yp})(d_c)(n_d)}$$

$$= \frac{288 \text{ kips}(1.0)}{(1.00)(0.6)(50 \text{ ksi})(15.2 \text{ in.})(1)}$$

$$= 0.632 \text{ in.}$$

111

Stiffeners and Doublers

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

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

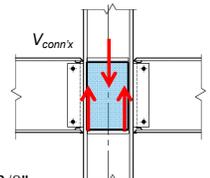
$$t_{d \text{ min } 3} = \frac{0.5(0 \text{ kips})}{1.00(0.6)(50 \text{ ksi})(26.9 \text{ in.} - 0.745 \text{ in.} - 0.5 \text{ in.})} = 0.0 \text{ in.}$$

Where: $L_d = \text{length of doubler} = d_b - t_b - t_c$

$$t_{d \text{ min}} = \max(t_{d \text{ min } 1}, t_{d \text{ min } 2} + t_{d \text{ min } 3})$$

$$= \max(0.211 \text{ in.}, 0.632 \text{ in.} + 0 \text{ in.}) = 0.632 \text{ in.}$$

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



112

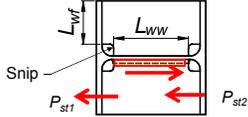
Stiffeners and Doublers

Stiffener & Doubler Example

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

$$t_{d \min} = \frac{P_{st1} + P_{st2}}{\phi(0.6)(F_{yp})(L_{ww})(2)} \quad \text{Ref Spec Eq (J4-3)}$$

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

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


Stiffeners and Doublers 113

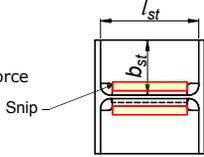
Stiffener & Doubler Example

- 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)

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

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

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


Stiffeners and Doublers 114

Stiffener & Doubler Example

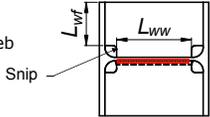
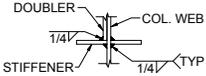
- Solution:**
 - 8. Column Stiffener Weld
 - Stiffener Weld to Doubler Horizontal Edge and Column Web for unbalanced stiffener force

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

$\epsilon = 0.928$ ASD
 $\epsilon = 1.392$ LRFD

$$D_{\min \text{ web}} = \frac{P_{st1} + P_{st2}}{\epsilon(2)(L_{vw})}$$

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

$$= 1.20 \text{ (16th of an inch)} \leq 4 \text{ (16th of an inch), use } 1/4" \text{ Fillet Weld o.k.}$$



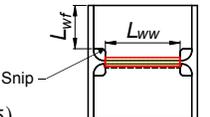
Stiffeners and Doublers 115

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 ($\phi = 0.75$)

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

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


Stiffeners and Doublers 116

Stiffener & Doubler Example

- Solution:
 - 9. Summary

DOUBLER
COL. WEB
STIFFENER
SECTION A
SECTION B
DOUBLER MAY ENCR OACH INTO COLUMN FILLET
PL 3/4" DOUBLER
W27x94
W14x176 COLUMN
1/2" STIFFENER PL NS/FS

Stiffeners and Doublers

117

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

Stiffeners and Doublers

118

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!

119

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.



Thank You

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



There's always a solution in steel.