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**Connection Design**  
Tips, Tricks, and Lessons Learned



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### Course Description

#### 19.5 Vertical Bracing Connections March 11, 2019

Don't waste time showing too much information that isn't used or that can unnecessarily complicate your design. This session will include tips for successful delegated vertical bracing design and what information should be included on drawings, which will help you limit RFIs and resubmittals.



### Learning Objectives

- Identify detailing notes, for vertical bracing connections, that can lead to challenges for the connection engineer.
- Explain how to analyze and account for transfer forces in braced-frame buildings.
- Describe the various analysis methods that can be applied to the design of corner bracing connections.
- List issues to look for when reviewing vertical bracing connection designs.



## Night School 19 Connection Design: Tips, Tricks, and Lessons Learned

Session 5: Vertical Bracing Connections  
March 11, 2019



Carol Drucker, SE, PE, PEng  
Principal

**dzse**  
Drucker Zajdel Structural Engineers, Inc.  
Chicago, IL



### Items Covered

- Load path
- Information needed for delegated design
- Information not necessarily needed
- When to expect RFIs
- First steps to design
- Corner, Chevron, Base, and Column Splices
- Seismic design
- Reviewing calculations



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### Systems to Resist Lateral Load

- Concrete Shear Walls
- **Braced Frames**
- Moment Frames
- Dual Systems

#### Advantage

- Rigid
- Typically more cost effective than moment frames

#### Disadvantage

- May interfere with openings



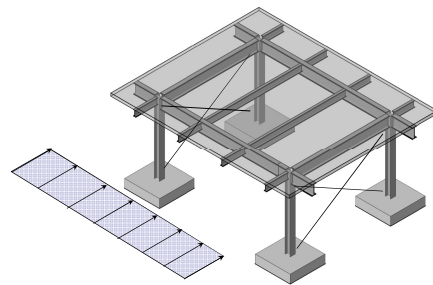
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### Load Path For Wind



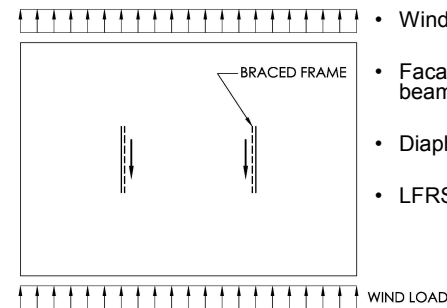
- Wind on facade
- Facade to structure (e.g. beams or diaphragms)
- Diaphragms to lateral force resisting system (LFRS)
- LFRS to foundations



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### Load Path For Wind



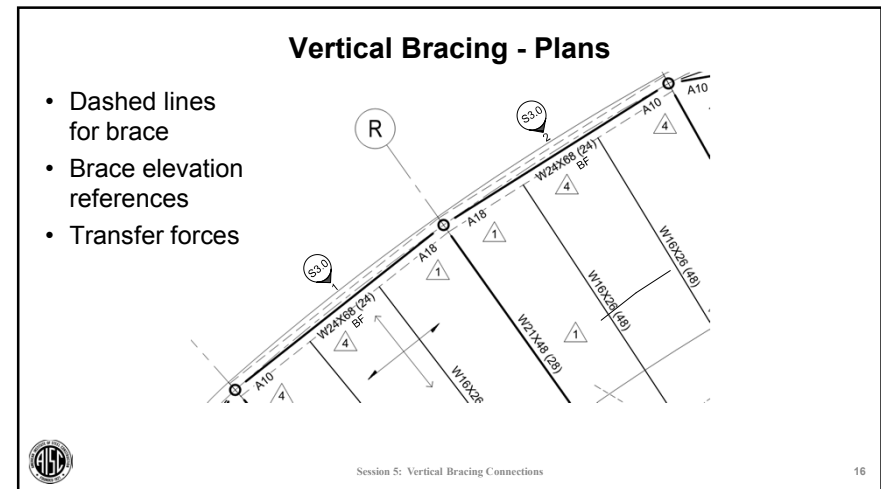
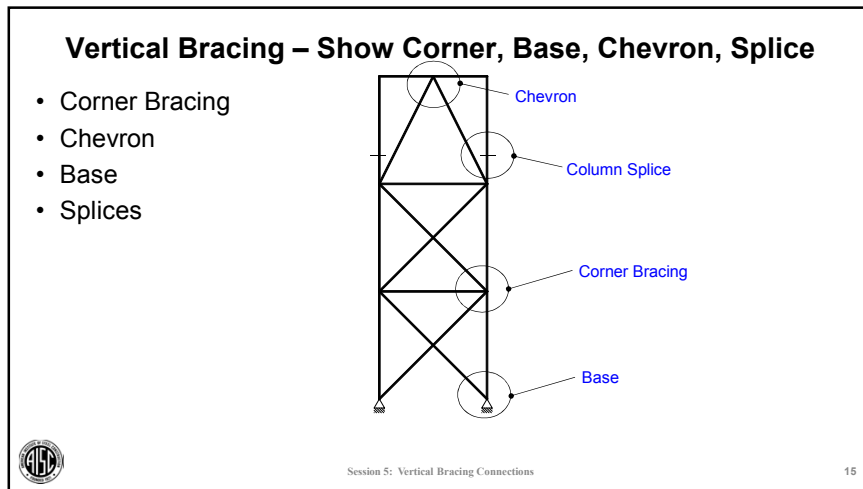
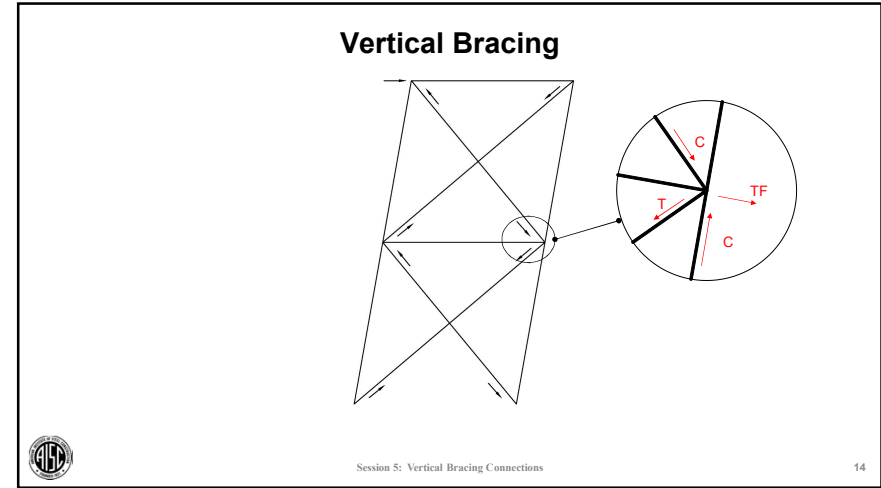
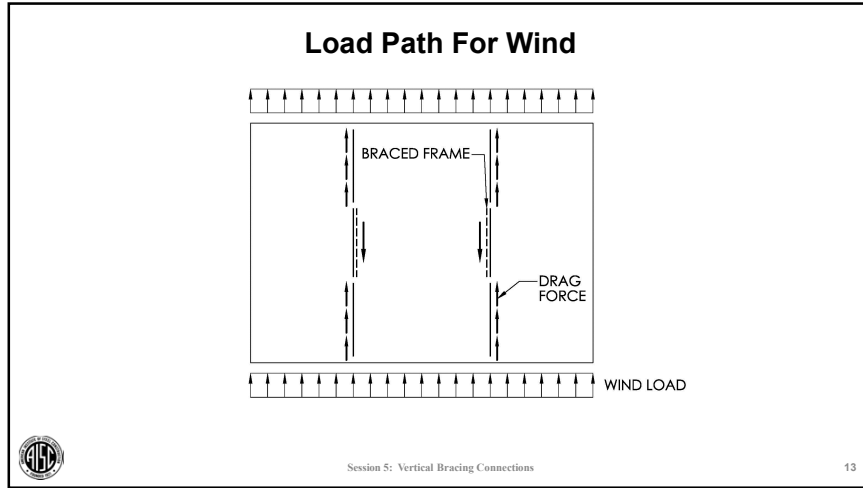
- Wind on facade
- Facade to structure (e.g. beams or diaphragms)
- Diaphragms to LFRS
- LFRS to foundations



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### Vertical Bracing Elevations

- Typically shown
  - Elevations
  - Brace size
  - Brace force
  - Grid locations
- Helpful information
  - Beam size
  - Column size
  - Transfer forces
  - Bay length
  - Column orientation

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### Vertical Bracing Elevations

- Corner Bracing
  - Map transfer force
  - Map gravity loads
  - Map beam sizes
  - Map column sizes

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### Vertical Bracing – Items to Show

- Notes
  - All connections, unless specifically designated as being completely designed on the structural drawings, shall be designed and detailed by a structural engineer licensed in the state where the project is located.
  - Schematic detail. Fabricator to design actual connection based on required loading

5 TYP. BRACING DETAIL AT HSS COL  
 SCALE: NOT TO SCALE

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### Vertical Bracing – Items to Show

- Architectural requirements

5 TYP. BRACING DETAIL AT HSS COL  
 SCALE: NOT TO SCALE

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### Vertical Bracing – Items to Show

- Information needed for design
  - SINGLE SHEAR PLATE SLOT THROUGH HSS IF REQUIRED BY CONNECTION ENGINEER CALCULATIONS
  - FOR CHECKING HSS WALL, USE  $P_a = 0.75 (0.6F_y)$  ASD

5 TYP. BRACING DETAIL AT HSS COL  
SCALE NOT TO SCALE

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### Code of Standard Practice

– 2016 COSP (303-16) Section 3.1.1 and Section 3.1.2

- Option 3: in the structural *design documents* or *specifications*, the *connection* shall be designated to be designed by a licensed engineer working for the *fabricator*
- Options 3A and 3B: More applicable to moment connection design but could apply at vertical bracing (mainly at chevron connections)
  - Option 3A: Member reinforcement at connections shall be designed by owners' designated representative for design and shown in the structural design documents
  - 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.

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### Vertical Bracing

- Expect RFI
  - HIGH STRENGTH BOLTS IN A SLIP-CRITICAL TYPE CONNECTION
  - DOUBLE ANGLES

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### Vertical Bracing

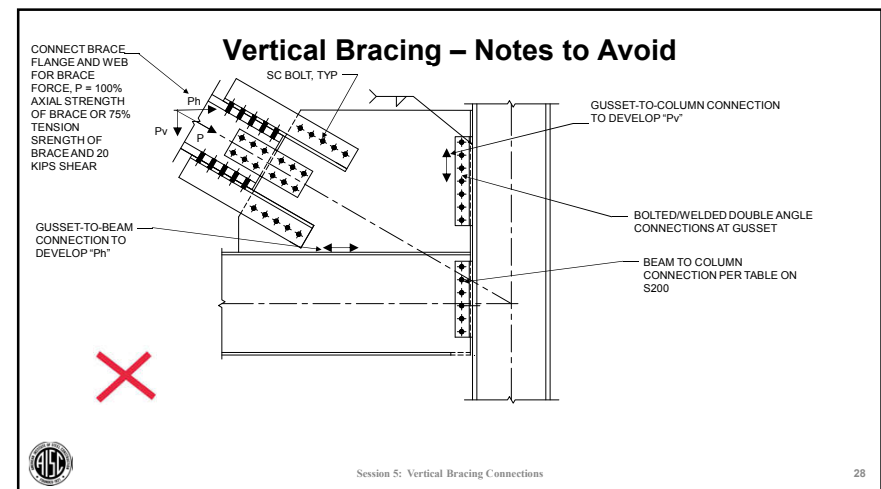
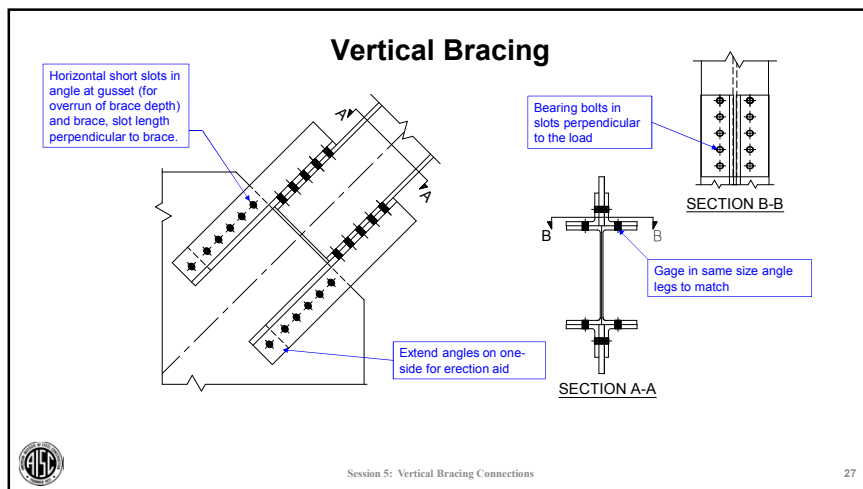
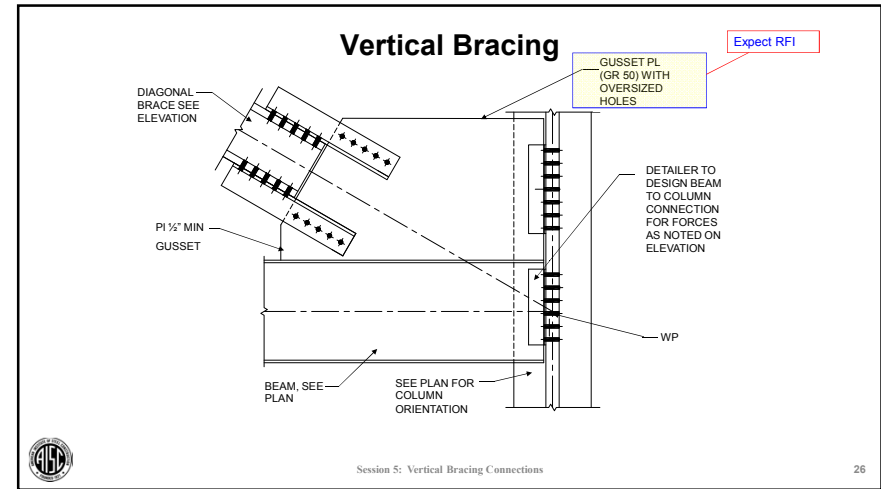
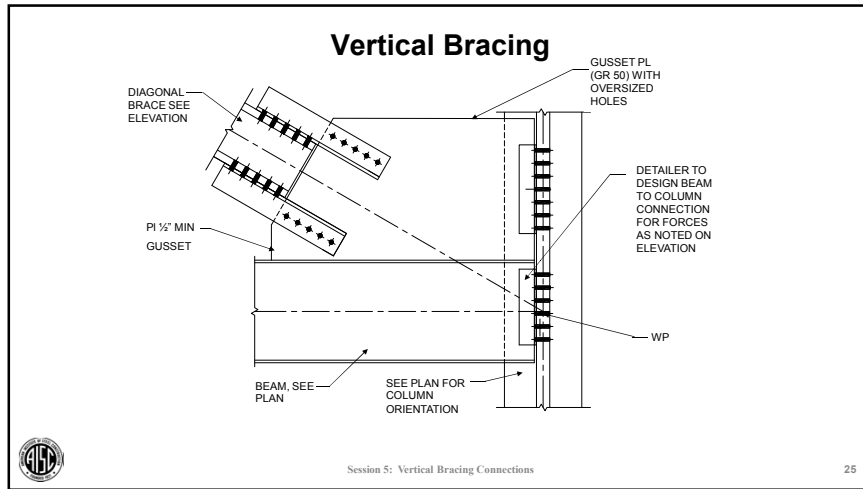
- SC Bolt Requirements given in RCSC Specification
  - 4.3. Slip-Critical Joints
 

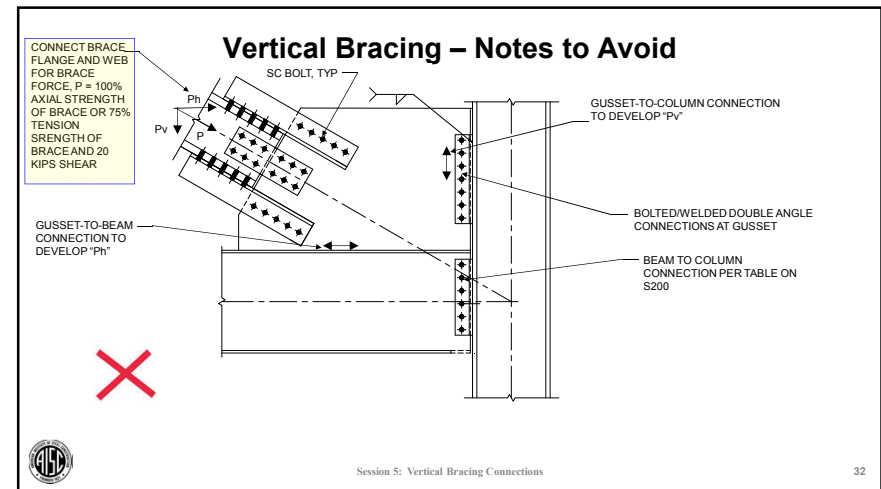
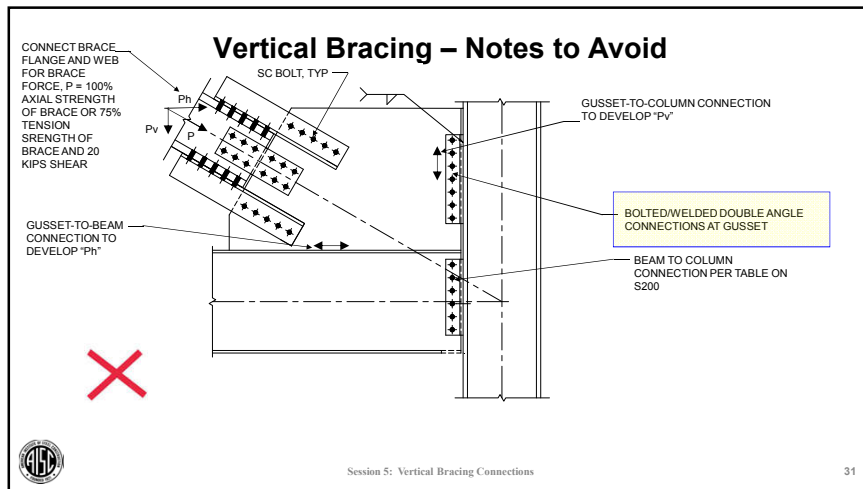
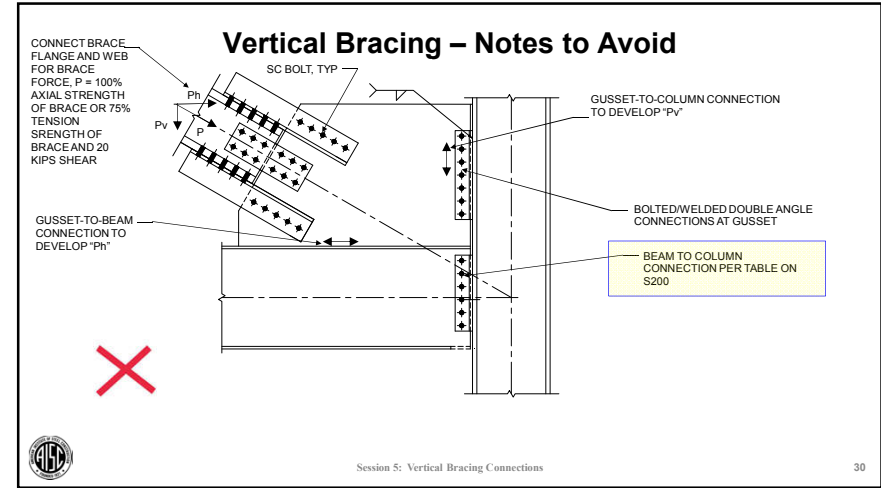
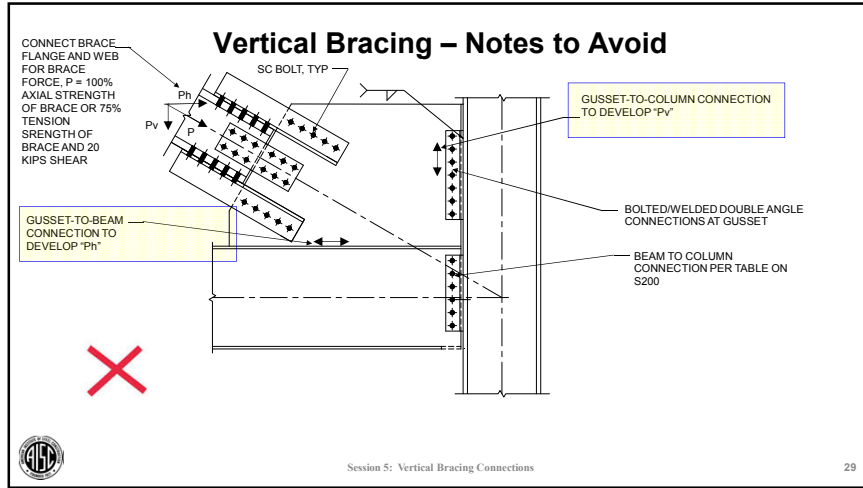
*Slip-critical joints* are required in the following applications involving shear or combined shear and tension:

    - Joints* that are subject to fatigue load with reversal of the loading direction;
    - Joints* that utilize oversized holes;
    - Joints* that utilize slotted holes, except those with applied load approximately normal (within 80 to 100 degrees) to the direction of the long dimension of the slot; and,
    - Joints* in which slip at the *faying surfaces* would be detrimental to the performance of the structure.

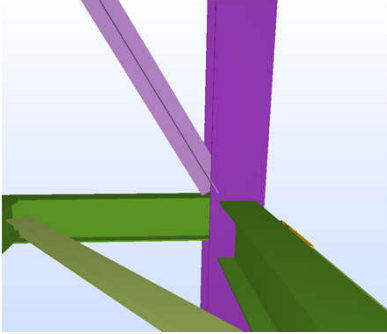
Bolts in *slip-critical joints* shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2, 5.3, 5.4 and 5.5, installed in accordance with Section 8.2 and inspected in accordance with Section 9.3.

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### Vertical Bracing – Model Transfer



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### Vertical Bracing – Model Transfer

	A	B	C	D	E	F	G	H	I	J
	memid	section	risa_memb_e _Type	section_id	start_max_a xial_transfer	start_min_a xial_transfer	start_max_s hear_transfe r	start_min_s hear_transfe r	Start_Max_ axial	Start_Min_axi al
1										
2	Column	Column2	Column	Column	Column	Column	Column	Column	Column	Column
84	ROS_172	LL5x5x8x3	VBrace	LL5x5x8x3					191.201841	-0.44519998
85	ROS_173	LL5x5x8x3	VBrace	LL5x5x8x3					229.61399	-0.29399999
86	ROS_174	LL5x5x8x3	VBrace	LL5x5x8x3					239.931289	-0.31814999
88	ROS_190	LL5x5x8x3	VBrace	LL5x5x8x3					257.088288	-0.49674998
89	ROS_191	LL5x5x8x3	VBrace	LL5x5x8x3					232.498298	0
90	ROS_192	LL5x5x8x3	VBrace	LL5x5x8x3					266.979288	0
01	ROS_193	LL5x5x8x3	VBrace	LL5x5x8x3					241.310889	-0.40844998
02	ROS_194	LL5x5x8x3	VBrace	LL5x5x8x3					254.741538	-0.43784998
03	ROS_195	LL5x5x8x3	VBrace	LL5x5x8x3					240.974889	-0.38219998
04	ROS_196	LL5x5x8x3	VBrace	LL5x5x8x3					253.032139	-0.50504998
05	ROS_197	LL5x5x8x3	VBrace	LL5x5x8x3					265.292988	0
06	ROS_198	LL5x5x8x3	VBrace	LL5x5x8x3					229.29584	-0.46829998
07	ROS_199	LL5x5x8x3	VBrace	LL5x5x8x3					242.536339	-0.49064993
10	ROS_200	LL5x5x8x3	VBrace	LL5x5x8x3					255.835638	-0.48028993
14	ROS_205	LL5x5x8x3	VBrace	LL5x5x8x3					343.513784	-0.21442849
15	ROS_206	LL5x5x8x3	VBrace	LL5x5x8x3					332.024685	-0.24964789
16	ROS_207	LL5x5x8x3	VBrace	LL5x5x8x3					341.390684	-0.23204999
17	ROS_208	LL5x5x8x3	VBrace	LL5x5x8x3					328.254135	0
18	ROS_209	LL5x5x8x3	VBrace	LL5x5x8x3					348.604184	-0.30764999
20	ROS_210	LL5x5x8x3	VBrace	LL5x5x8x3					331.561635	-0.30764999
21	ROS_211	LL5x5x8x3	VBrace	LL5x5x8x3					354.462134	-0.32234999
22	ROS_212	LL5x5x8x3	VBrace	LL5x5x8x3					337.188635	-0.21209999

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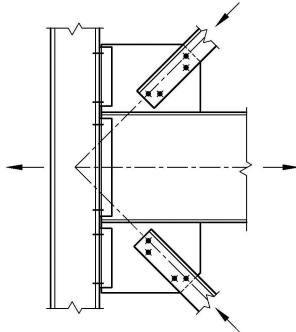
### Vertical Bracing – Load Case Break Down

1	Member	Joint	Load		P	V2	V3	T	M2	M3
2	ID	ID	Combo		Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
6808	3365	31	COMB46		-232	0	0	0	0	0
6809	3365	31	COMB47		21	0	0	0	0	0
6810	3365	31	COMB48		-19	0	0	0	0	0
6811	3365	31	COMB49		-237	0	0	0	0	0
6812	3365	31	COMB50		-249	0	0	0	0	0
6813	3365	31	COMB51		11	0	0	0	0	0
6814	3365	31	COMB52		-37	0	0	0	0	0
6815	3366	31	COMB01		-143	0	0	0	0	0
6816	3366	31	COMB02		-229	0	0	0	0	0
6817	3366	31	COMB03		-230	0	0	0	0	0
6818	3366	31	COMB04		-228	0	0	0	0	0
6819	3366	31	COMB05		-218	0	0	0	0	0
6820	3366	31	COMB06		-237	0	0	0	0	0
6821	3366	31	COMB07		-74	0	0	0	0	0
6822	3366	31	COMB08		-82	0	0	0	0	0
6823	3366	31	COMB09		-261	0	0	0	0	0
6824	3366	31	COMB10		-265	0	0	0	0	0

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### Vertical Bracing



- Indicate if both top and bottom braces in compression


Note: Do not use force from one brace to offset force from opposite brace

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### Common General Notes

- Notes:
  - ASD or LRFD **Avoid mixing on the same project** ✓
  - Load criteria ✓
  - Bolt pre-tensioning and/or slip critical requirements ✓
  - Shear loads for members in the braced frame ✓
  - Connection work points at member centerlines ✓
  - Transfer force notes ✓
  - Seismic response modification coefficient,  $R$  ✓




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### Design Notes

- Good Notes
  - Seismic response modification coefficient,  $R$  (Table 12.2-1 ASCE 7-16)
    - $R = 3$ : Structural steel systems not specifically detailed for seismic resistance. Seismic Design Category A, B or C. Use when possible
  - $R > 3$ 
    - Ordinary Concentrically Braced Frame (OCBF):  $R = 3.25$
    - Special Concentrically Braced Frame (SCBF): Strength of member:  $R = 6$
    - Eccentrically Braced Frame (EBF) :  $R = 8$
    - Buckling-Restrained Braced Frames (BRBF) :  $R = 8$




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### GENERAL NOTES

- Notes that could cause RFI
  - All plate and angles are A36
  - Hole type requirements
  - Specific bolt requirements
  - Spacing and edge distance requirements




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### Most Common RFI Questions

- ASD or LRFD
- Use of Slip Critical bolts
- Alternate Connections
- Full Strength
- HSS Beam Loading
- Transfer forces (TF)
- Column splice loading (Min. Axial)
- Material grade
- Restrictions
- Stitch Plates
- Shear load in beams (especially if a percentage of the Maximum Total Uniform Load, UDL, is used)
- Work Point



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### Getting Started with Design

- Forces
- Sizes
- System requirements

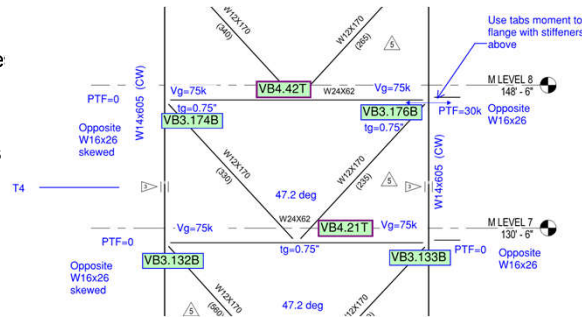


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### Getting Started with Design

- Corner Bracing
  - Map transfer force
  - Map gravity loads
  - Map beam sizes
  - Map column sizes

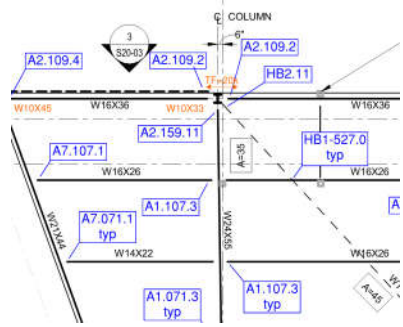


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### Getting Started with Design

- Corner Bracing
  - Map transfer forces
  - Map gravity loads
  - Map beam sizes
  - Map column sizes

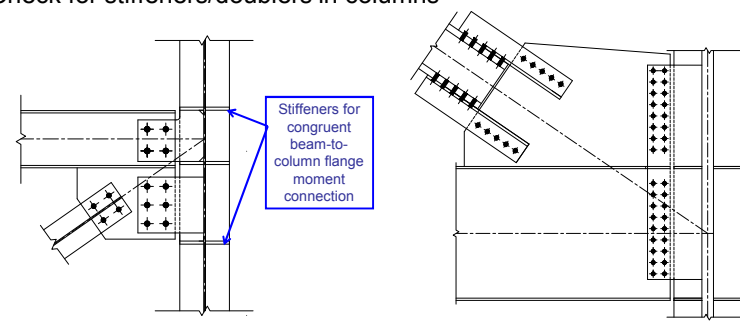


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### Getting Started with Design

- Check for stiffeners/doublers in columns



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### Getting Started with Design

- Flooring Type

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### Getting Started with Design

- Double-Angle Requirements
  - Stitch Plates
  - AISC Section E6
  - Table 4-8 to 4-10: Axial Compression Double Angles

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{a}{r_f}\right)^2} \quad (E6-1)$$

(1) When  $\frac{a}{r_f} \leq 40$

$$\left(\frac{L_c}{r}\right)_m = \left(\frac{L_c}{r}\right)_o \quad (E6-2a)$$

(2) When  $\frac{a}{r_f} > 40$

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{K_1 a}{r_f}\right)^2} \quad (E6-2b)$$

Table 4-9 (continued)  
 Available Strength in Axial Compression, kips  
 Double Angles—LLBB  $F_y = 36$  ksi

Shape	2L6 x 6				2L6 x 4			
	$t_b$	$t_b$	$t_b$	$t_b$	$t_b$	$t_b$	$t_b$	$t_b$
in/lb	54.4	47.2	40.0	36.2	30.0	26.2	22.2	18.2
Design	$P_n$	$P_n$	$P_n$	$P_n$	$P_n$	$P_n$	$P_n$	$P_n$
0	345	318	290	262	212	188	164	140
4	333	301	276	248	204	180	156	132
8	319	289	264	236	192	168	144	120
12	272	244	220	192	156	132	108	84
16	224	200	176	152	120	96	72	48
20	180	160	140	120	96	72	48	24
24	144	128	112	96	72	48	24	0
28	112	100	88	76	56	40	24	8
32	88	80	72	64	48	36	24	16
36	64	60	56	52	40	32	24	24
40	40	40	40	40	32	24	16	8
44	24	24	24	24	20	16	12	8
48	8	8	8	8	8	8	8	8

### Getting Started with Design

## E6. BUILT-UP MEMBERS

### 1. Compressive Strength

This section applies to built-up members composed of two shapes either (a) interconnected by bolts or welds or (b) with at least one open side interconnected by perforated cover plates or lacing with tie plates. The end connection shall be welded or connected by means of pretensioned bolts with Class A or B faying surfaces.

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### Getting Started with Design

- Surface Preparation Requirements
  - Class A Bolts
  - Class B Bolts
  - Galvanizing


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### Getting Started with Design

- Seismic Design,  $R > 3$  (AISC 341-16, Section D2.2(d))
  - (d) All bolts shall be installed as pretensioned high-strength bolts. Faying surfaces shall satisfy the requirements for slip-critical connections in accordance with *Specification* Section J3.8 with a faying surface with a Class A slip coefficient or higher.
 

Exceptions: Connection surfaces are permitted to have coatings with a slip coefficient less than that of a Class A faying surface for the following:

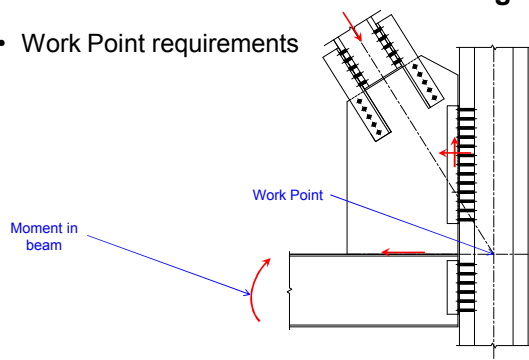

    - End plate moment connections conforming to the requirements of Section E1, or ANSI/AISC 358
    - Bolted joints where the seismic load effects are transferred either by tension in bolts or by compression bearing but not by shear in bolts



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### Vertical Bracing

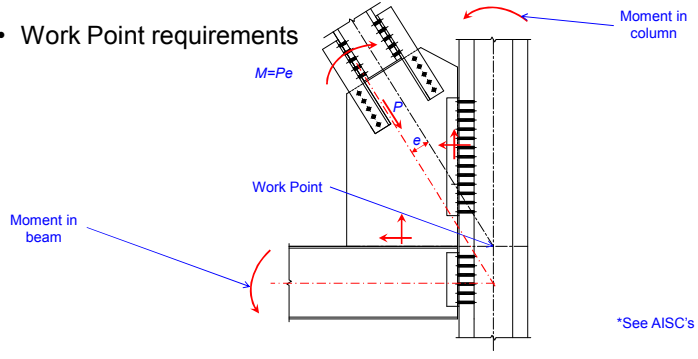
- Work Point requirements

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### Getting Started with Design

- Work Point requirements



*M=Pe*

\*See AISC's Design Guide 29



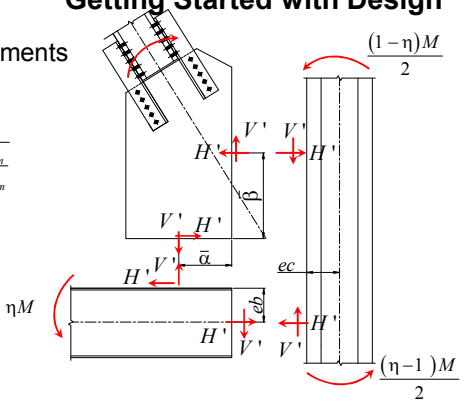
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### Getting Started with Design


- WP requirements

$$\eta = \frac{\frac{I_{beam}}{L_{beam}}}{\frac{I_{col}}{L_{col}} + \frac{I_{beam}}{L_{beam}}}$$

$$H' = \frac{(1-\eta)M}{e_b + \bar{\beta}}$$

$$V' = \frac{M - H'\bar{\beta}}{\bar{\alpha}}$$


\*Reference AISC's Night School and AISC's DG29



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### Getting Started with Design

- WP requirements

Work Point at Top of Beam

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### Frame Analysis

- WP requirements

Note: At least one member framing into joint needs to not be released to the maintain numerical stability of the analysis

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### Getting Started with Design

- Work line location
- Bolted on gage,  $g_1$
- Welded on  $\bar{y}$

$\theta$	Leg	12	10	8	7	6	5	4	3 1/2	3	2 1/2	2	1 1/2	1 1/4	1 1/8	1
$\theta$	$\theta$	6	5	4 1/2	4	3 3/4	3	2 3/4	2	1 3/4	1 1/4	1	3/4	3/4	3/4	3/4
$\theta$	$\theta$	3	3	3	3	2 1/2	2	2	2	1 1/2	1 1/4	1	3/4	3/4	3/4	3/4
$\theta$	$\theta$	2 1/2	2 1/2	2 1/2	2 1/2	2	2	2	2	1 1/2	1 1/4	1	3/4	3/4	3/4	3/4
$\theta$	$\theta$	2 1/2	2 1/2	2 1/2	2 1/2	2	2	2	2	1 1/2	1 1/4	1	3/4	3/4	3/4	3/4

Note: Other gages are permitted to suit specific requirements subject to clearances and edge distance limitations.

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### Transfer Forces

- Lateral force in a discontinuous bay needs to be transferred to the lateral system below

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### Transfer Forces - Discontinuous Braced Bays

Sum forces in "X" direction  
 $TF = -3.34k - 9.6k(0.707)$   
 $= -10k (T)$

Sum forces in "X" direction  
 $TF = -14.0k(0.707) - 0.115k$   
 $= -10.0k (T)$

- To determine transfer force:
  - Determine the transfer plane
  - sum forces either side of that plane

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### Transfer Forces - Discontinuous Braced Bays

Sum forces in "X" direction  
 $TF = -3.34k - 9.6k(0.707)$   
 $= -10k (T)$

Sum forces in "X" direction  
 $TF = -14.0k(0.707) - 0.115k$   
 $= -10.0k (T)$

- To determine transfer force:
  - Determine the transfer plane
  - sum forces either side of that plane

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### Getting the Load to the Lateral System

TRANSFER FORCE AT COLUMN C2

TRANSFER FORCE AT COLUMN C3

DIAPHRAGM SHEAR (FORCE/LENGTH)

$TF = 0k$     $TF = 20k$     $TF = 40k$

C1   C2   C3

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### Transfer Forces

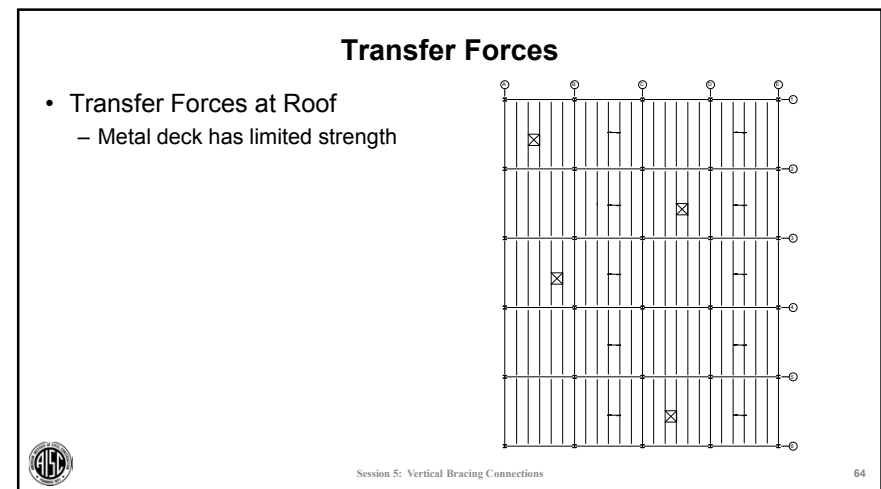
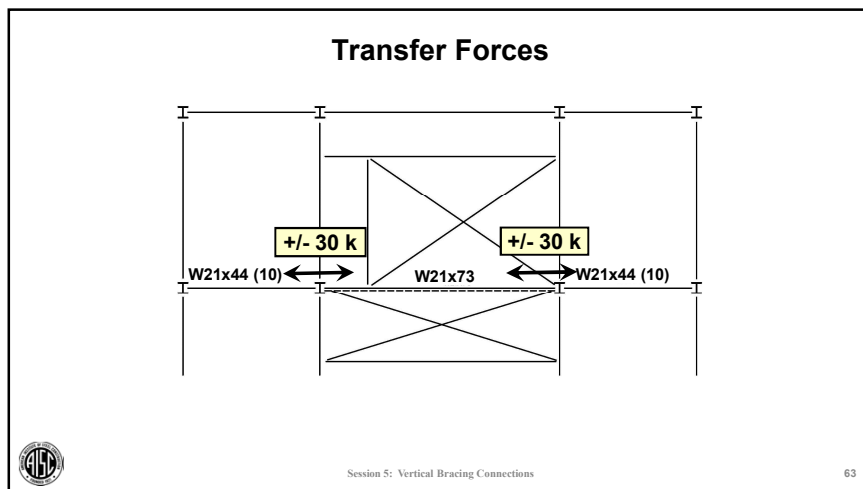
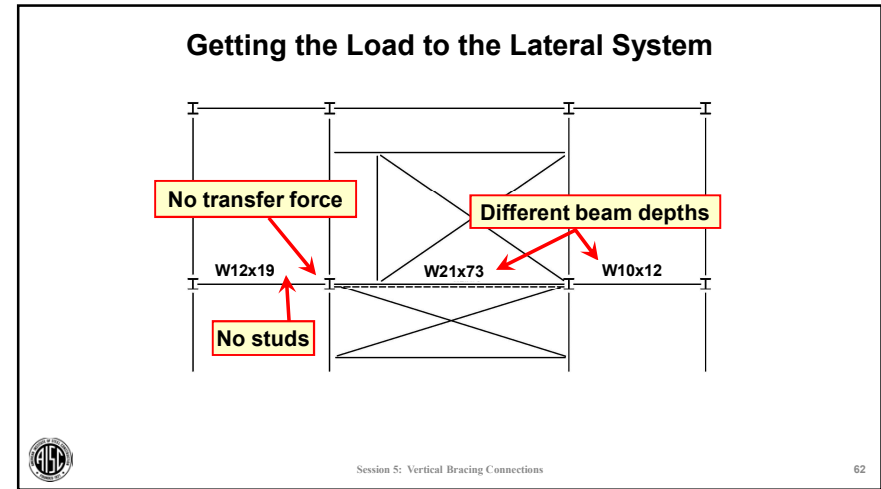
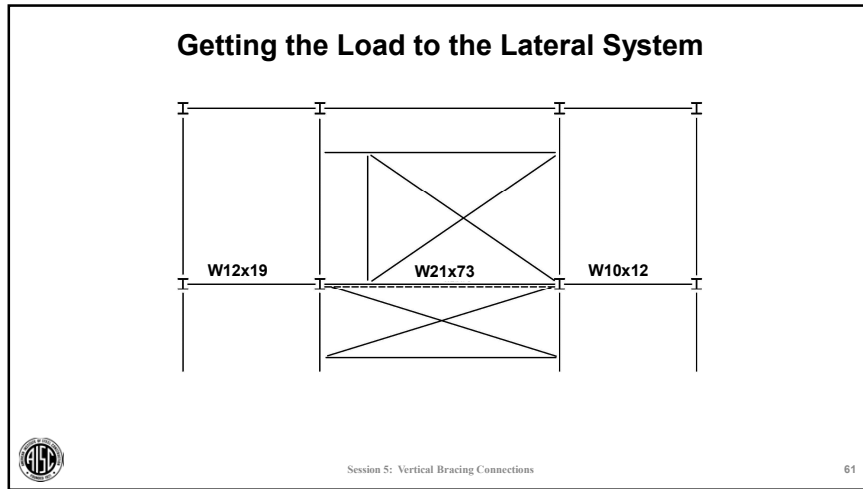
Transfer forces on opposite sides of the column should be equal

Horizontal component of brace force is internal to the braced frame and the beam-to-column connection does not need to resist the entire horizontal force component

$A = \pm 70k$

$TF = 14k$

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### Diaphragm Strength

#### B, BA 18 GA. Diaphragm Design

Design Thickness = 0.0474 in.      ϕ (EQ): 0.55      Ω (EQ): 3.00  
 Support Fasteners: 5/8" Puddle Welds      ϕ (Wind): 0.70      Ω (Wind): 2.35  
 Side Lap Fasteners: # 10 Screws      ϕ (Other): 0.60      Ω (Other): 2.65

Fib. Type	Support Fastener Pattern	Side Lap Conn. per Splice	Nominal Diaphragm Shear Strength (kip)													
			10'-0"	11'-0"	12'-0"	13'-0"	14'-0"	15'-0"	16'-0"	17'-0"	18'-0"	19'-0"	20'-0"			
3605 (B) D <sub>1</sub> = 281 D <sub>2</sub> = 286	No Fit (Bare Deck)	0	994	891	821	751	686	626	561	500	472	425	390	379	0.739	
		2	1347	1239	1147	1066	985	917	857	804	756	-	-	-	0.637	
		3	1504	1389	1296	1201	1121	1053	984	923	869	821	777	733	644	0.763
		4	1653	1530	1423	1329	1246	1172	1106	1043	982	928	879	795	729	0.810
		5	1791	1654	1552	1452	1364	1288	1214	1150	1092	1035	981	888	814	0.921
		6	1919	1769	1673	1570	1477	1393	1319	1250	1198	1152	1101	989	899	0.947
		7	2039	1907	1788	1681	1580	1498	1419	1348	1282	1223	1168	1072	983	0.971
		8	2149	2016	1896	1797	1698	1598	1510	1442	1373	1311	1253	1151	1064	1.038
		9	2250	2118	1997	1897	1796	1694	1603	1532	1461	1398	1336	1232	1143	1.060
		10	2344	2213	2092	1991	1891	1785	1693	1619	1546	1479	1417	1305	1220	1.100
		11	2430	2301	2181	2079	1977	1872	1781	1703	1628	1557	1489	1373	1289	1.134
3604 (B) D <sub>1</sub> = 399 (BA) D <sub>1</sub> = 408	No Fit (Bare Deck)	0	753	667	619	574	534	497	462	428	395	363	329	298	0.603	
		2	1105	1019	945	880	814	757	706	652	622	-	-	-	0.496	
		3	1253	1161	1090	1029	946	890	834	782	735	684	657	593	544	0.603
		4	1398	1292	1207	1131	1063	1002	947	896	847	801	759	686	629	0.712
		5	1531	1413	1324	1244	1179	1106	1049	996	948	904	860	779	713	0.803
		6	1622	1523	1432	1350	1278	1208	1140	1080	1038	992	948	871	796	0.927
		7	1721	1622	1532	1449	1372	1303	1230	1180	1136	1076	1030	953	876	0.970
		8	1810	1713	1623	1539	1462	1391	1326	1265	1209	1157	1109	1023	947	0.997
		9	1889	1795	1708	1623	1546	1474	1407	1346	1288	1235	1185	1090	1018	0.994
		10	1960	1869	1782	1700	1624	1552	1484	1422	1363	1308	1257	1162	1084	1.059
		11	2024	1938	1851	1771	1695	1624	1557	1493	1434	1379	1326	1230	1153	1.081

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### Diaphragm Strength

- Transfer Forces at Composite Deck  
– Consider topping slab

$$\phi V_n = \phi L_s f_s (2\lambda\sqrt{f'_c}) \quad \text{ACI 12.5.3.3}$$

$$\phi = 0.75$$

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### Diaphragm Strength

- Transfer Forces at Composite Deck

$$\phi V_n = \phi L_s f_s (2\lambda\sqrt{f'_c}) \quad \text{ACI 12.5.3.3}$$

$$\phi = 0.75$$

$$\Sigma TF = \max(\text{abs}(\Sigma F_{x2} - \Sigma F_{x1}) - \phi V_n, 0 \text{ kips})$$

Red Arrow =  $F_x$  = Brace horizontal component  
 Blue Arrow = Diaphragm strength  
 Green Arrow = Transfer Forces

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### Diaphragm Strength

- Use Table 3-21 for Stud Strength

**Table 3-21**  
 $F_u = 65 \text{ ksi}$       **Shear Stud Anchor**       $Q_n$   
 Nominal Horizontal Shear Strength  
 for One Steel Headed Stud Anchor,  $Q_n$ , kips

Deck Condition	Stud Anchor Diameter, in.	Normal Weight Concrete			
		$w_c = 145 \text{ pcf}$			
		$f'_c = 3 \text{ ksi}$	$f'_c = 4 \text{ ksi}$	$f'_c = 3 \text{ ksi}$	$f'_c = 4 \text{ ksi}$
1	3/8	4.31	4.31	4.28	4.31
	1/2	7.66	7.66	7.60	7.66
	5/8	12.0	12.0	11.9	12.0
	3/4	17.2	17.2	17.1	17.2
2	3/8	3.66	3.66	3.66	3.66
	1/2	6.51	6.51	6.51	6.51
	5/8	10.2	10.2	10.2	10.2
	3/4	14.6	14.6	14.6	14.6

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### Diaphragm Strength

- Stud Strength (*Specification* Eq. I8-1)
 
$$Q_n = 0.5 A_{sa} \sqrt{f'_c E_c} \leq R_g R_p A_{sa} F_u$$

$$A_{sa} = \text{area of stud, in}^2$$

$$E_c = \text{concrete modulus of elasticity} = w_c^{1.5} \sqrt{f'_c}, \text{ ksi}$$

$$F_u = \text{specified minimum tensile strength of stud} = 65 \text{ ksi}$$

$$Q_n = 0.5 (0.4418 \text{ in}^2) \sqrt{(4 \text{ ksi}) [(145 \text{ pcf})^{1.5} (\sqrt{4 \text{ ksi}})]}$$

$$= 26.1 \text{ kips/stud}$$

$$R_g R_p A_{sa} F_u = 1.0 (0.6) (0.4418 \text{ in}^2) (65 \text{ ksi})$$

$$= 17.2 \text{ kips/stud, controls}$$

User Note: The table below presents values for  $R_g$  and  $R_p$  for several cases. Available strengths for steel headed stud anchors can be found in the AISC *Steel Construction Manual*.

Condition	$R_g$	$R_p$
No decking	1.0	0.75
Decking oriented parallel to the steel shape		
$w_c \geq 1.5$ $h_f$	1.0	0.75
$w_c < 1.5$ $h_f$	0.85 <sup>(a)</sup>	0.75
Decking oriented perpendicular to the steel shape		
Number of steel headed stud anchors occupying the same decking rib:		
1	1.0	0.6 <sup>(b)</sup>
2	0.85	0.6 <sup>(b)</sup>
3 or more	0.7	0.6 <sup>(b)</sup>

(a)  $n_s$  = nominal rib height, in. (mm)  
 $w_c$  = average width of concrete rib or haunch (as defined in Section 13.2c), in. (mm)  
<sup>(b)</sup> For a single steel headed stud anchor  
 This value may be increased to 0.75 when  $d_{stud} \geq 2$  in. (50 mm).

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### Fabricator Preferences and Standards

- Plate Grade: A36 or Grade 50 plate.
  - Cost increase for Grade 50 plate
  - Recommended if single plate connections
- Angle Grade: A36 or Grade 50
  - Typically no cost differences
- Bolt preference
- Connection preferences

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### Fabricator Preferences

- Connection Preferences
  - Single Plate, Double-Angles, End Plates
  - Matching gusset thickness
  - Shipping limitations
- Detailing standards
  - Preferred angle sizes
  - HSS slot widths
  - Lap start/stops

Gusset thickness at opposite ends to match, increase as required, typ.

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### Vertical Bracing – References

- AISC's Design Guide 29 (DG29)
- Handbook of Structural Steel Connection Design and Details
- AISC's Night School 6
- Steel Tips
- 15<sup>th</sup> Ed. Manual, Part 13
- AISC's Design Examples

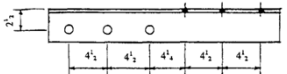
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### Vertical Bracing – Articles

- *Effective Length Factors for Gussets Plate Buckling*, EJ 2<sup>nd</sup> Qtr, 2006, Bo Dowswell
- *Effective Length Factors for Gussets Plates in Chevron Braced Frames*, EJ 3<sup>rd</sup> Qtr, 2012, Bo Dowswell
- *The Effect of Eccentricity on Brace-to-gusset Angles*, EJ 4<sup>th</sup> Qtr, 1996, W.A. Thornton

Gusset Configuration	Effective Length Factor	Buckling Length	$\frac{P_{app}}{P_{ult}}$
Compact corner	$\frac{b}{l_{br}}$	$\frac{b}{l_{br}}$	1.36
Noncompact corner	1.0	$l_{br}$	3.08
Extended corner	0.6	$l_1$	1.45
Single brace	0.7	$l_1$	1.45
Chevron	0.85	$l_1$	1.17

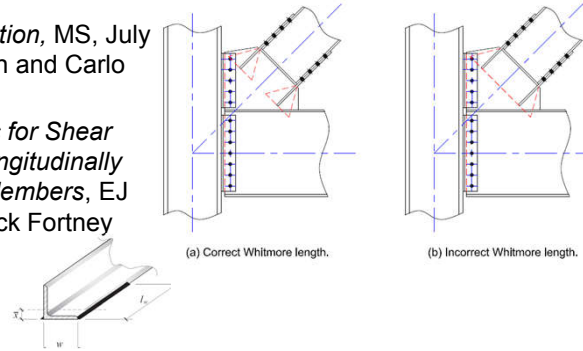
<sup>a</sup> Table 3 from Dowswell (2006) with revisions.  
<sup>b</sup> Yielding in the applicable limit state for compact corner gusset plates; therefore, the effective length factor and the buckling length are not applicable.





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### Vertical Bracing – Articles


- *The Whitmore Section*, MS, July 2011, WA Thornton and Carlo Lini
- *Recommendations for Shear Lag Factors for Longitudinally Welded Tension Members*, EJ 1<sup>st</sup> Qtr, 2012, Patrick Fortney and WA Thornton



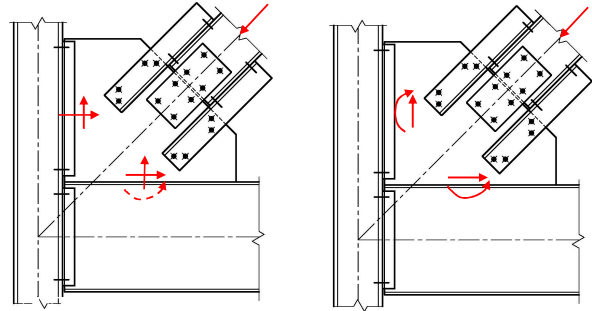

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### Vertical Bracing – Articles

- *Prying Action - a General Treatment*, EJ 2<sup>nd</sup> Qtr, 1985, WA Thornton
- *Designing Compact Gussets with the Uniform Force Method*, EJ 1<sup>st</sup> Qtr, 2008, Larry Muir
- *24 Tips for Simplifying Braced Frame Connections*, MS May 2006, Victor Shneur



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### Connections-Bracing



**Uniform Force Method (UFM)**

**KISS**


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### Connections-Bracing UFM

$$V_b = \frac{e_b}{r} P$$

$$H_b = \frac{\alpha}{r} P$$

$$V_c = \frac{\beta}{r} P$$

$$H_c = \frac{e_c}{r} P$$

\*Reference AISC's Night School 6 and AISC's DG29

$y = mx + b$

$y_{col\_line} = \frac{\beta}{e_c} x + e_b$

$y_{work\_line} = \frac{1}{\tan(\theta)} x$

$y_{beam\_line} = \frac{e_b}{\alpha} x - \frac{e_b e_c}{\alpha}$

\*Reference AISC's Night School 6 and AISC's DG29

### Connections-Bracing UFM

$$V_b = \frac{e_b}{r} P$$

$$H_b = \frac{\alpha}{r} P$$

$$V_c = \frac{\beta}{r} P$$

$$H_c = \frac{e_c}{r} P$$

\*Reference AISC's Night School 6 and AISC's DG29

$y = mx + b$

$y_{col\_line} = \frac{\beta}{e_c} x + e_b$

$y_{work\_line} = \frac{1}{\tan(\theta)} x$

$y_{beam\_line} = \frac{e_b}{\alpha} x - \frac{e_b e_c}{\alpha}$

\*Reference AISC's Night School 6 and AISC's DG29

### Connections-Bracing UFM

$$\frac{1}{\tan(\theta)} x = \frac{\beta}{e_c} x + e_b \quad x = \frac{e_b}{\left(\frac{1}{\tan(\theta)} - \frac{\beta}{e_c}\right)}$$

\*Reference AISC's Night School 6 and AISC's DG29

Work line = column line and solve for x

Work line = beam line and solve for x

$\frac{1}{\tan(\theta)} x = \frac{e_b}{\alpha} x - \frac{e_b e_c}{\alpha}$

$x = \frac{-e_b e_c}{\alpha \left(\frac{1}{\tan(\theta)} - \frac{e_b}{\alpha}\right)}$

\*Reference AISC's Night School 6 and AISC's DG29

### Connections-Bracing UFM

$$\frac{1}{\tan(\theta)} x = \frac{e_b}{\alpha} x - \frac{e_b e_c}{\alpha} \quad x = \frac{-e_b e_c}{\alpha \left(\frac{1}{\tan(\theta)} - \frac{e_b}{\alpha}\right)}$$

\*Reference AISC's Night School 6 and AISC's DG29

Set x = x

$\frac{-e_b e_c}{\alpha \left(\frac{1}{\tan(\theta)} - \frac{e_b}{\alpha}\right)} = \frac{e_b}{\left(\frac{1}{\tan(\theta)} - \frac{\beta}{e_c}\right)}$

Simplify

$\alpha - \beta \tan(\theta) = e_b \tan(\theta) - e_c \quad (\text{eq. 13-1})$

\*Reference AISC's Night School 6 and AISC's DG29



### Connections-Bracing UFM

\*Reference AISC's Night School and AISC's DG29

$$\alpha - \beta \tan(\theta) = e_b \tan(\theta) - e_c \quad (\text{eq. 13-1})$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} \quad (\text{eq. 13-6})$$

$$V_b = \frac{e_b}{r} P \quad H_b = \frac{\alpha}{r} P$$

$$V_c = \frac{\beta}{r} P \quad H_c = \frac{e_c}{r} P$$

Can set  $\bar{\beta} = \beta$  and solve for  $\alpha$ .  
 If  $\bar{\alpha} = \alpha$ , no moment

Can set  $\bar{\alpha} = \alpha$  and solve for  $\beta$ .  
 If  $\bar{\beta} = \beta$ , no moment

To column flange, set  $\bar{\beta} = \beta$   
 To column web, set  $\bar{\alpha} = \alpha$

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### Connections-Bracing UFM

\*Reference AISC's Night School 6 and AISC's DG29

$$\alpha - \beta \tan(\theta) = e_b \tan(\theta) - e_c \quad (\text{eq. 13-1})$$

Can set  $\bar{\beta} = \beta$  and solve for  $\alpha$ .  
 If  $\bar{\alpha} = \alpha$ , no moment

Can set  $\bar{\alpha} = \alpha$  and solve for  $\beta$ .  
 If  $\bar{\beta} = \beta$ , no moment

To column flange, set  $\bar{\beta} = \beta$   
 To column web, set  $\bar{\alpha} = \alpha$

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### Connections-Bracing UFM

\*Reference AISC's Night School 6 and AISC's DG29

$$\alpha - \beta \tan(\theta) = e_b \tan(\theta) - e_c \quad (\text{eq. 13-1})$$

$$V_b = \frac{e_b}{r} P \quad V_c = \frac{\beta}{r} P$$

$$H_b = \frac{\alpha}{r} P \quad H_c = \frac{e_c}{r} P$$

$$M_b = V_b(\alpha - \bar{\alpha}) \quad M_c = H_c(\beta - \bar{\beta})$$

Note: If to column web,  $e_c = 0$  in.

Can set  $\bar{\beta} = \beta$  and solve for  $\alpha$ .  
 If  $\bar{\alpha} = \alpha$ , no moment

Can set  $\bar{\alpha} = \alpha$  and solve for  $\beta$ .  
 If  $\bar{\beta} = \beta$ , no moment

To column flange, set  $\bar{\beta} = \beta$   
 To column web, set  $\bar{\alpha} = \alpha$

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### Connections-Bracing UFM

\*Reference AISC's Night School 6 and AISC's DG29

$$V_b = \frac{e_b}{r} P - \Delta V_b$$

$$H_b = \frac{\alpha}{r} P$$

$$M_b = \frac{e_b}{r} P(\alpha - \bar{\alpha}) + \Delta V_b \bar{\alpha}$$

$$V_c = \frac{\beta}{r} P + \Delta V_b \quad H_c = \frac{e_c}{r} P$$

Can set  $\bar{\beta} = \beta$  and solve for  $\alpha$ .  
 If  $\bar{\alpha} = \alpha$ , no moment

Can set  $\bar{\alpha} = \alpha$  and solve for  $\beta$ .  
 If  $\bar{\beta} = \beta$ , no moment

To column flange, set  $\bar{\beta} = \beta$   
 To column web, set  $\bar{\alpha} = \alpha$

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### UFM Special Case – Compact Gussets

$$\text{ATAN} \left( \frac{e_c}{\beta} \right) > \theta$$

$$\text{ATAN} \left( \frac{\alpha'}{e_b} \right) < \theta$$

$$\text{ATAN} \left( \frac{e_c}{\beta} \right) = \theta$$

$$\text{ATAN} \left( \frac{\alpha'}{e_b} \right) = \theta$$

\*Reference AISC's AISC's DG29

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### UFM Special Case – Parallel Force Method (PFM)

$$\text{ATAN} \left( \frac{e_c}{\beta} \right) = \theta$$

$$\text{ATAN} \left( \frac{\alpha'}{e_b} \right) = \theta$$

$$\text{ATAN} \left( \frac{e_c}{\beta} \right) = \theta$$

$$\text{ATAN} \left( \frac{\alpha'}{e_b} \right) = \theta$$

\*Reference AISC's AISC's DG29

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### Connections-Bracing KISS

$$H_b = P \cos(\theta)$$

$$M_b = H_b(e_b)$$

$$V_c = P \sin(\theta)$$

$$M_c = V_c(e_c)$$

$$H_b = P \cos(\theta)$$

$$M_b = H_b(e_b)$$

$$V_c = P \sin(\theta)$$

$$M_c = V_c(e_c)$$

**KISS**

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### Connections-Bracing UFM

$$H_b = P \cos(\theta)$$

$$M_b = H_b(e_b)$$

$$V_c = P \sin(\theta)$$

$$M_c = V_c(e_c)$$

$$H_b = P \cos(\theta)$$

$$M_b = H_b(e_b)$$

$$V_c = P \sin(\theta)$$

$$M_c = V_c(e_c)$$

- $H_c$  from gusset-to-column to column-to-beam
- $H_b$  force is not resisted by column-to-beam connection
- $V_b$  from gusset-to-beam to beam-to-column
- Add gravity to  $V_b$
- Consider transfer forces and  $H_c$
- Allows for force redistribution

**UFM**

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### Connections-Bracing KISS

- No  $H_c$
- No  $V_b$
- Does not impact beam-to-column connection
- Need to consider moment at beam and at column

**UFM**

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### Connections-Bracing

**UFM Advantage**                      **Possible KISS Advantage**

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### Connections-UFM

**UFM Special Case IV**

$$F = \frac{R(e_c)}{\beta + e_b}$$

$$M_F = F\beta$$

**UFM**

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### Connections-UFM Gravity Distribution

**UFM Special Case IV**

$$F = \frac{R(e_c)}{\beta + e_b}$$

$$M_F = F\beta$$

**UFM**

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### Connections-UFM

**UFM Special Case IV**

- No eccentricity at bolt group if  $e_c = e$

$$H_c = H_c + F$$

$$M_b = M_b + M_F$$

$$M_F + F(e_b) = R(e)$$

$$H_b + H_c + TF$$

$$H_b = H_b + F$$

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### Details for High Axial Load

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### Details for High Axial Load and/or Moment

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### Chevron Connections

- Stability bracing
- Brace force directions

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### Chevron Connections

- Stability bracing
  - Beam framing into WP ✓
  - Tension brace
  - Stiffeners and studs on top

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### Chevron Connections

- Stability bracing
  - Beam framing into WP ✓
  - Tension brace
  - Stiffeners and studs

Reference AISC's 15 Ed. Manual, pp 2-16 and 2-17:

- When an infill beam frames into the continuous beam at the column top, the required stability normally can be provided by using connection element(s) for the infill beam that cover three-quarters or more of the T-dimension of the continuous beam. Alternatively, connection elements that cover less than three-quarters of the T-dimension of the continuous beam can be used in conjunction with partial-depth stiffeners in the beam web along with a moment connection between the column top and beam bottom to maintain alignment of the beam/column assembly. A cap plate of reasonable proportions and four bolts will normally suffice.

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### Chevron Connections

- Optimize connections

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### Chevron Connections

- Check for doublers

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### Single Brace Connections

- Stability Concern
  - See Chapter C and Appendix 6 for Stability
  - Web compression buckling: *Spec* Equation J10-8 developed from point loads and restrained flanges

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

Test Set-up from Chen and Oppenheim 1970

Session 5: Vertical Bracing Connections

### Brace to Base

- Work Point location: Typically top of plate or bottom of Plate
- See DG29
- Indicate base plate can or can not be extended
- Avoid welding washer plates unless anchor rods checked for shear and bending

Session 5: Vertical Bracing Connections

### Brace to Base

- Work Point location – Top of Plate

$$H = P \sin \theta$$

$$V = P \cos \theta$$

$$V_c = V$$

$$H_b = H$$

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### Brace to Base

- Work Point location – Bottom of Plate

Note: "e" is a negative if below the top of base plate

$$H = P \sin \theta$$

$$V = P \cos \theta$$

$$V_c = V$$

Sum Moments about  $\bar{y}$  to determine  $H_b$

$$H_b = \frac{H(\bar{\beta} - e)}{\beta}$$

$$H_c = H - H_b$$

\*Reference DG29

Session 5: Vertical Bracing Connections

### Brace to Base

- General Work Point FBD

Note:  $e_c = 0$  in. to column webs

$$H = P \sin \theta$$

$$V = P \cos \theta$$

$$V_c = V$$

$$H_b = \frac{H(\beta - e) - V e_c}{\beta}$$

$$H_c = H - H_b$$

\*Reference DG29

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### Brace to Base

- General Work Point FBD

$$H = P \sin \theta$$

$$V = P \cos \theta$$

$$V_c = V$$

$$H_b = \frac{H(\beta - e) - V e_c}{\beta}$$

$$H_c = H - H_b$$

Can also determine  $H_c$  by summing moments about face of column flange at base plate:

$$H_c = \frac{H(e) + V(e_c)}{\beta}$$

\*Reference DG29

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### Brace to Base

- General Work Point FBD

$$\beta = 0.5(L_{weld\_c}) + clip$$

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### Brace to Base

- Work Point location – Top of Plate

$$V_c = P \sin \theta_h$$

$$M_c = V_c (e_c)$$

$$H_b = P \cos \theta_h$$

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### Brace to Base

- Check anchor rods for shear and bending if washer plate is welded to base plate
- Base plate hole size per 15<sup>TH</sup> Ed. Manual Table 14-2
- Washer plates per 15<sup>TH</sup> Ed. Manual Table 14-2
- Consider lugs if high shear load and column has insufficient compression to resist the shear in friction

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### Tension Column Splices

- Typical order of preference:
  - Bolted
  - PJP welded
  - CJP welded
- SC Bolts or Bearing
- Avoid full strength, especially welded
- Show loads on Column Schedule or Braced Frame elevations

UPPER COLUMN	$E_{min}$
OVER $\frac{1}{2}''$ TO $\frac{3}{4}''$	$\frac{1}{4}''$
OVER $\frac{3}{4}''$ TO $1\frac{1}{2}''$	$\frac{3}{8}''$
OVER $1\frac{1}{2}''$ TO $2\frac{1}{2}''$	$\frac{1}{2}''$
OVER $2\frac{1}{2}''$ TO $6''$	$\frac{3}{4}''$
OVER $6''$	$\frac{3}{4}''$

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### Lateral Column Splices

- CJP versus PJP
  - Testing
  - Gap
- Land =  $\frac{1}{4}''$  Min. PJP
- Use similar column depths for alignment of flanges. W14 columns preferred. For column size W14:  
 $d - 2t_f = 12.6''$

Note: Weld prep should allow for flange tilt when  $d_{c1} - d_{c2} < 1/4''$

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### Welded Tension Column Splices

**Step 1: Check for Tension using S and Pmin:**

$$f_a = \frac{P_{min}}{A_g} - \frac{M_x}{S_x} - \frac{M_y}{S_y}$$

**Step 2: If tension, determine stresses in weld**

$$f_x = \frac{V_x}{(2)(L_{weld})} + \frac{Torsion}{(d)(L_{weld})}$$

$$f_y = \frac{P_{min, flange}}{(2)(L_{weld})} - \frac{M_x}{(d - \frac{t_f}{2})(L_{weld})} - \frac{4M_y}{(2)(L_{weld})^2}$$

**Step 3: Determine (E) required for shear and axial:**

$$(E) = \frac{f}{(\phi)(0.6F_{EXX})}$$

**Step 4: Sum squares and compare to AISC min. (Table J2.3)**

$$E_{min} = \sqrt{(E_{min}(shear)})^2 + (E_{min}(tension))^2}$$

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### Seismic Splices: AISC Seismic Provisions 341-16 OCBF, SCBF, BRBF (D2.5 and Chapter F)

Web Shear = maximum required per load combinations or

- $V = M_{pc}/H$  (OCBF)
- $V = \sum M_{pc}/H_c$  (Bolted SCBF/BRBF)  
 $F_y(Z_c + Z_b)(H - d_{col}/2 - d_{bf}/2 - t_{stab})$

Note: LFRD expressions shown

D2.5b: If tension using overstrength seismic loads:

- PJP welds develop 200% required strength
- Must develop  $0.5R_y F_y h_f$  of the smaller flange

Chapter F:  
 SCBF/BRBF: CJP or Bolted flanges with min strength =  $0.5R_y F_y Z_c, min$   
 OCBF: Load Combinations with amplified loads

- Demand critical welds
- Weld tabs to be removed, steel backing of groove welds need not be removed
- If tension >  $0.3F_y$  at smaller column, taper transitions of  $t_f$

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### Seismic Splices: 341-16 Column Not Part of the SFRS (D2.5)

$V = M_{pc}/H$   
 $M_{pc}$  = plastic flexural strength in direction of consideration. Consider both directions

Splice 4 ft or more above beam flange

Use shear equal to  $\frac{1}{2}$  of  $2M_{pc}/H$

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### Seismic Bracing

- SCBF – Expected strength
- OCBF – Expected tension strength need not exceed the overstrength seismic load
- BRBF – Buckling-Restrained Braced
- EBF - Eccentrically Braced Frame

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### OCBF

- Per AISC's 341-16 Section F1.6, the bracing connections shall be designed for the minimum of :
  - i. Force from ASCE7-16 Section 12.4.3.2 with Overstrength Factor,  $\Omega_o$ .
  - ii. The expected tension strength of the brace,  $R_y F_y A_g / \alpha_s$

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### OCBF

**6. Connections**

**6a. Brace Connections**

The required strength of diagonal brace connections shall be determined using the overstrength seismic load.

Exception: The required strength of the brace connection need not exceed the following.

- In tension, the expected yield strength divided by  $\alpha_s$ , which shall be determined as  $R_y F_y A_g / \alpha_s$ , where  $\alpha_s = \text{LRFD-ASD force level adjustment factor} = 1.0$  for LRFD and 1.5 for ASD.
- In compression, the expected brace strength in compression divided by  $\alpha_s$ , which is permitted to be taken as the lesser of  $R_y F_y A_g / \alpha_s$  and  $1.1 F_{cre} A_g / \alpha_s$ , where  $F_{cre}$  is determined from *Specification* Chapter E using the equations for  $F_{cr}$ , except that the expected yield stress,  $R_y F_y$ , is used in lieu of  $F_y$ . The brace length used for the determination of  $F_{cre}$  shall not exceed the distance from brace end to brace end.
- When oversized holes are used, the required strength for the limit state of bolt slip need not exceed the seismic load effect based upon the load combinations without overstrength as stipulated by the applicable building code.

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### OCBF

- ASCE 7-16 Section 12.4.3.1 Overstrength load combinations are (LRFD):

Where the seismic load effect with overstrength,  $E_{mh} = f(E_v, E_{mh})$ , defined in Section 12.4.3, is combined with the effects of other loads, the following seismic load combination for structures shall be used:

6.  $1.2D + E_v + E_{mh} + L + 0.2S$
7.  $0.9D - E_v + E_{mh}$

**12.4.3.1 Horizontal Seismic Load Effect Including Overstrength.** The effect of horizontal seismic forces including overstrength,  $E_{mh}$ , shall be determined in accordance with Eq. (12.4-7) as follows:

$$E_{mh} = \Omega_0 Q_E \quad (12.4-7)$$

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### OCBF

- ASCE 7-16, Table 12.2-1, Overstrength Factor,  $\Omega_0$

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, $R^a$	Overstrength Factor, $\Omega_0^b$	Deflection Amplification Factor, $C_d^c$
<b>B. BUILDING FRAME SYSTEMS</b>				
1. Steel eccentrically braced frames	14.1	8	2	4
2. Steel special concentrically braced frames	14.1	6	2	5
3. Steel ordinary concentrically braced frames	14.1	3/4	2	3/4
4. Special reinforced concrete shear walls <sup>d,h</sup>	14.2	6	2 1/2	5
5. Ordinary reinforced concrete shear walls <sup>d</sup>	14.2	5	2 1/2	4 1/2
6. Detailed plain concrete shear walls <sup>d</sup>	14.2 and 14.2.2.2	2	2 1/2	2

Session 5: Vertical Bracing Connections

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### Things to Look for in Review

- Notes:
- Correct Forces ✓
- Correct Member Sizes ✓
- Correct *Spec* and ASD/LRFD ✓
- Correct Work Point
- Confirm  $e_c$
- Confirm any special requirements met
- Hole type consistent with bolt type
- Hc force using the UFM applied at beam-to-column connection
- Transfer forces included in design

**Given:**

1. References: AISC Specification for Structural Steel Building, AISC 360-10  
AISC Manual 14th Edition
2. Design Basis: ASD design="ASD"
3. Materials:
 

a. W-Shape:	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$
b. HSS:	$F_y = 46 \text{ ksi}$	$F_u = 58 \text{ ksi}$
c. Plate:	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$
d. Angle:	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$
4. Bolts:
 

a. 3/4"-dia. A325-N bolts	$d_{nut} = 3/4 \text{ in}$
---------------------------	----------------------------

Session 5: Vertical Bracing Connections

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### Things to Look for in Review

- Notes:
  - Demand capacity ratio (DCR) < 1 ✓
  - Correct bolt values
  - Incomplete design

$$U_{wbr,c} := \text{abs} \left( \frac{P_c}{\phi P_{nc,c}} \right) = 0.627 \quad \text{if } (U_{wbr,c} > 1, "n.g.", "o.k.") = "o.k."$$

$$U_{wbr,j} := \text{abs} \left( \frac{P_j}{\phi P_{nc,j}} \right) = 0.573 \quad \text{if } (U_{wbr,j} > 1, "n.g.", "o.k.") = "o.k."$$

CONNECTION IS OK  
 (Strength equals or exceeds design loads)



### Conclusion

- Give actual forces on contract documents
- Avoid specifying unnecessary requirements that could penalize the contractor.
- Coordinate with other team members for most efficient design.
- Provide all information needed to properly complete connection design.



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Posted Tuesday mornings. [www.aisc.org/nightschool](http://www.aisc.org/nightschool) -- Click on Current Course Details.

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- REINFORCEMENT – Reinforce what you learn tonight. Get more out of the course.

*Note: If you attend the live presentation, you do not have to take the quizzes to receive CEUs/PDHs*



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#### Course Resources

Event	Start Date
108.14.8-Session Package-Night School 13 - Design of Industrial Buildings	3/10/2019 7:00:00 PM
108.14.8-Session Package-Night School 14 - Fundamentals of Seismic	6/5/2017 7:00:00 PM



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### Night School Resources



Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES


Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	Video	Pass	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Passcode: NS13ESN	Score: 80	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 02/13/2017 5pm EST	Available 02/13/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/05/2017 5pm EST	Available 03/05/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longspanne Brag Bracing Dn	3/27/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending



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- Weekly “quiz and recording” email.
- Weekly updates of the master quiz and attendance record, found at [www.aisc.org/nightschool19](http://www.aisc.org/nightschool19). Scroll down to Quiz and Attendance records.
  - Updated on Tuesday mornings.



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- Webinar connection information
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- Links to handouts also found here

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