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Bracing Success with Delegated Connection Design
June 27, 2019

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Course Description

Bracing Success with Delegated Connection Design
June 27, 2019

This webinar will guide you toward the successful delegated design of vertical bracing connections. Don't waste time showing too much information that isn't used, or which unnecessarily complicates your design. Learn what information should be included on drawings, and avoid excessive RFIs and resubmittals. This webinar will cover a number of topics, including lateral load path to vertical braces, transfer forces, vertical bracing analysis methods, and what to consider at brace-to-base connections.

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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.

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Bracing Success with Delegated Connection Design

June 27, 2019

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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

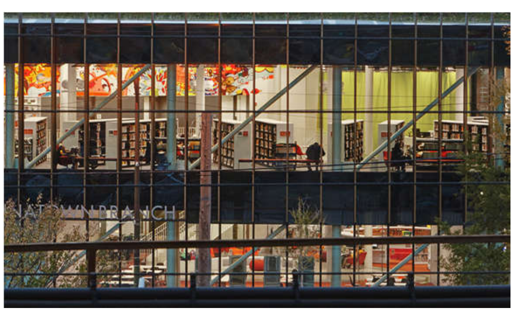

Photo by Jon Miller

Advantage

- Rigid
- Typically more cost effective than moment frames

Disadvantage

- May interfere with openings

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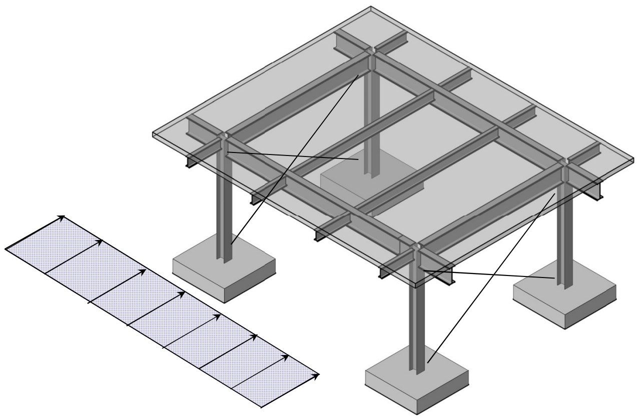
Load Path For Wind



- 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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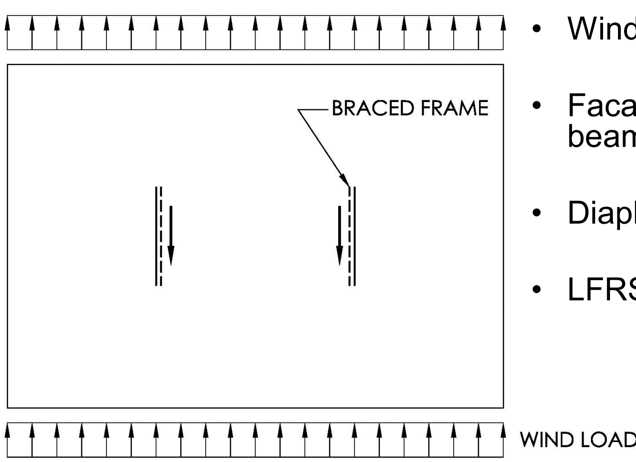
Load Path For Wind



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

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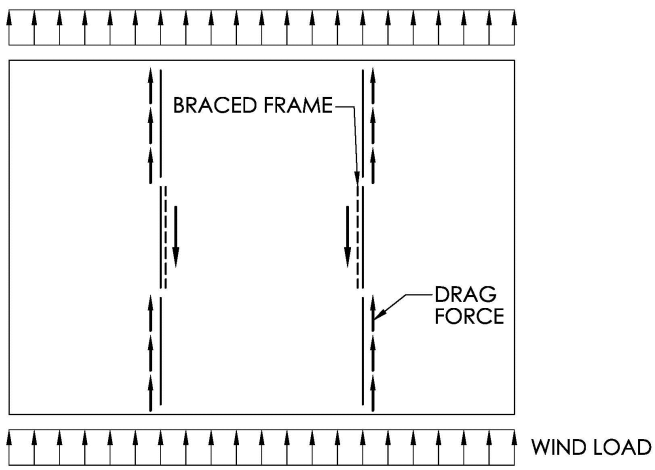
Load Path For Wind



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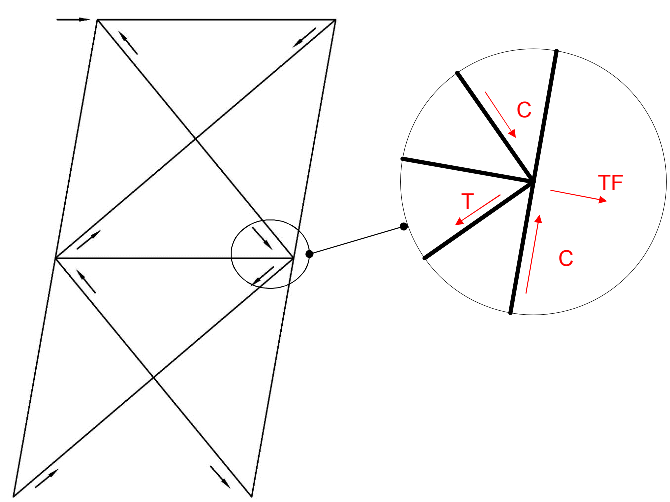
Vertical Bracing



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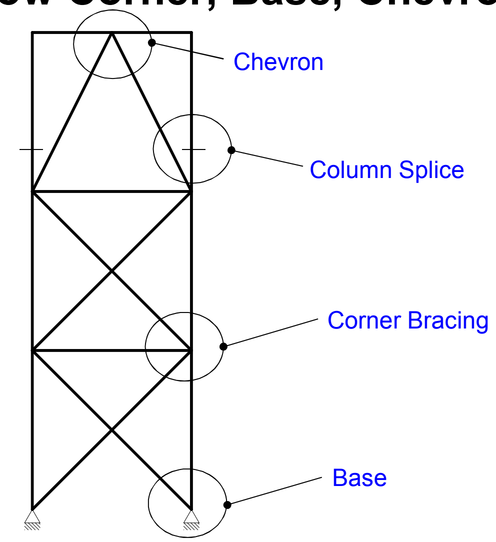
Vertical Bracing – Show Corner, Base, Chevron, Splice

- Corner Bracing
- Chevron
- Base
- Splices



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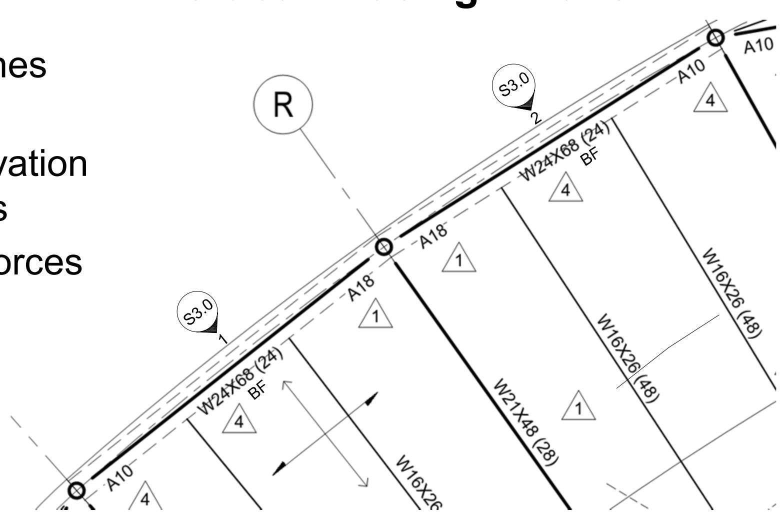
Vertical Bracing - Plans

- Dashed lines for brace
- Brace elevation references
- Transfer forces



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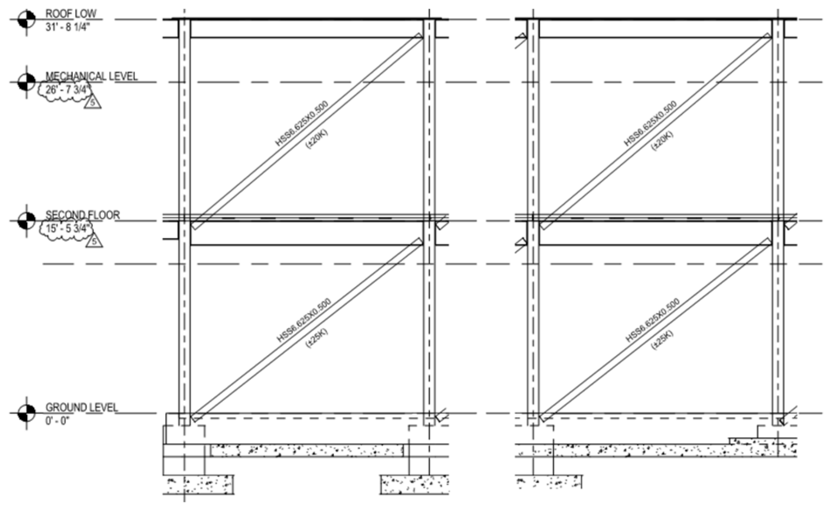
Vertical Bracing Elevations

- 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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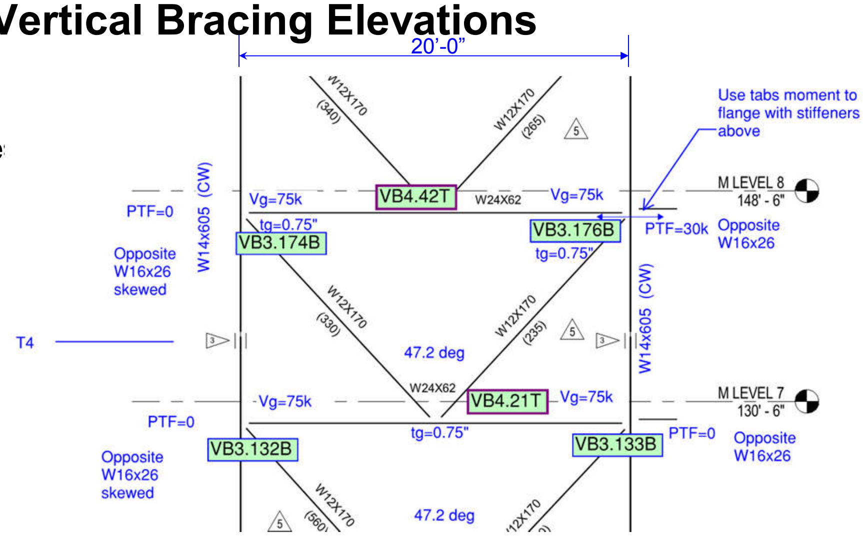
Vertical Bracing Elevations

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



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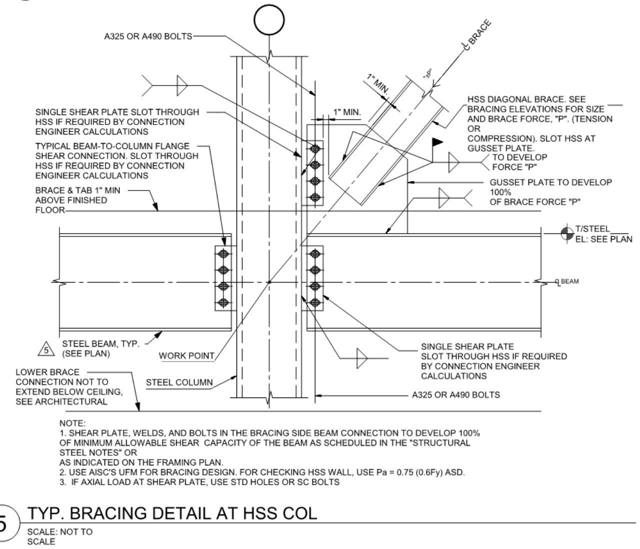
Vertical Bracing – Items to Show

- 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



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Vertical Bracing – Items to Show

- Architectural requirements

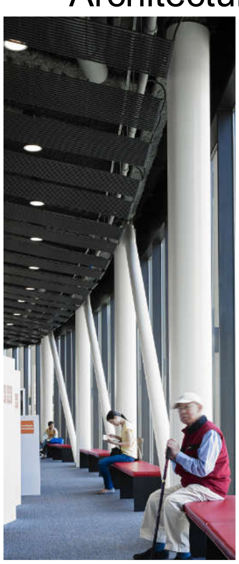
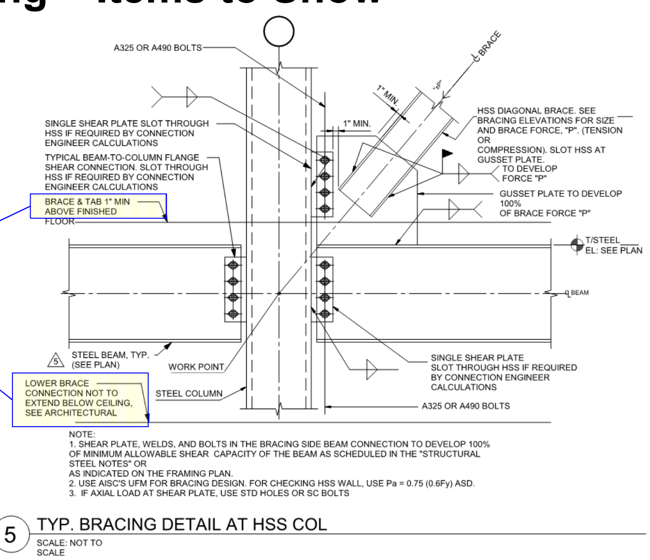


Photo by Jon Miller

BRACE & TAB 1" MIN ABOVE FINISHED FLOOR

LOWER BRACE CONNECTION NOT TO EXTEND BELOW CEILING, SEE ARCHITECTURAL



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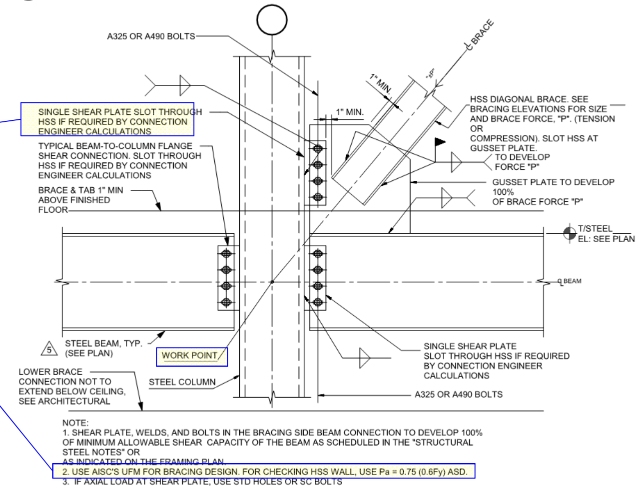
Vertical Bracing – Items to Show

- Information needed for design

SINGLE SHEAR PLATE SLOT THROUGH HSS IF REQUIRED BY CONNECTION ENGINEER CALCULATIONS

FOR CHECKING HSS WALL, USE Pa = 0.75 (0.6Fy) 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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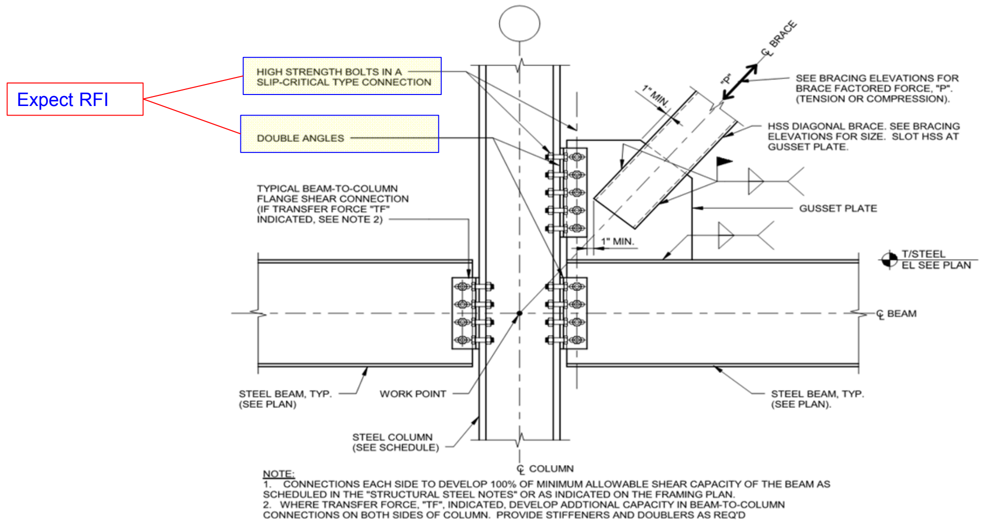
Vertical Bracing



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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:

(1) *Joints* that are subject to fatigue load with reversal of the loading direction;
(2) *Joints* that utilize oversized holes;
(3) *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,
(4) *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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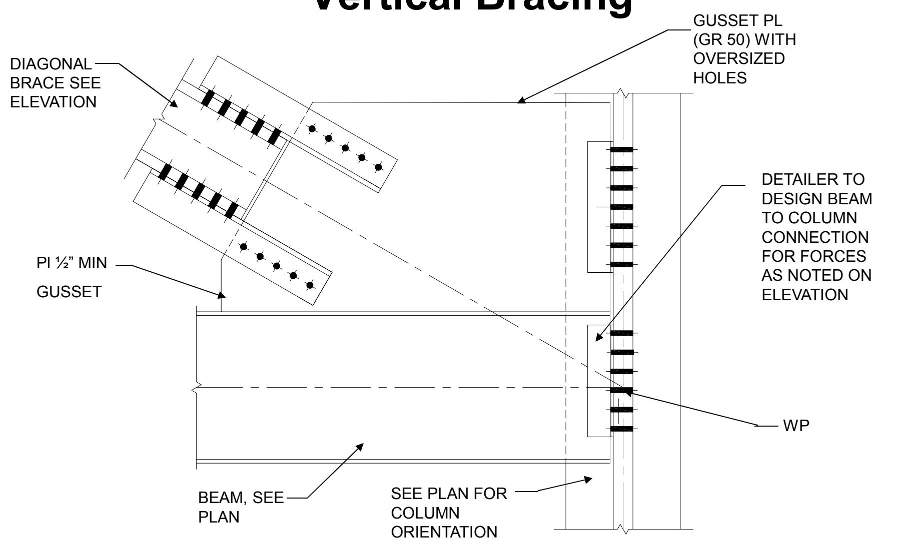
Vertical Bracing



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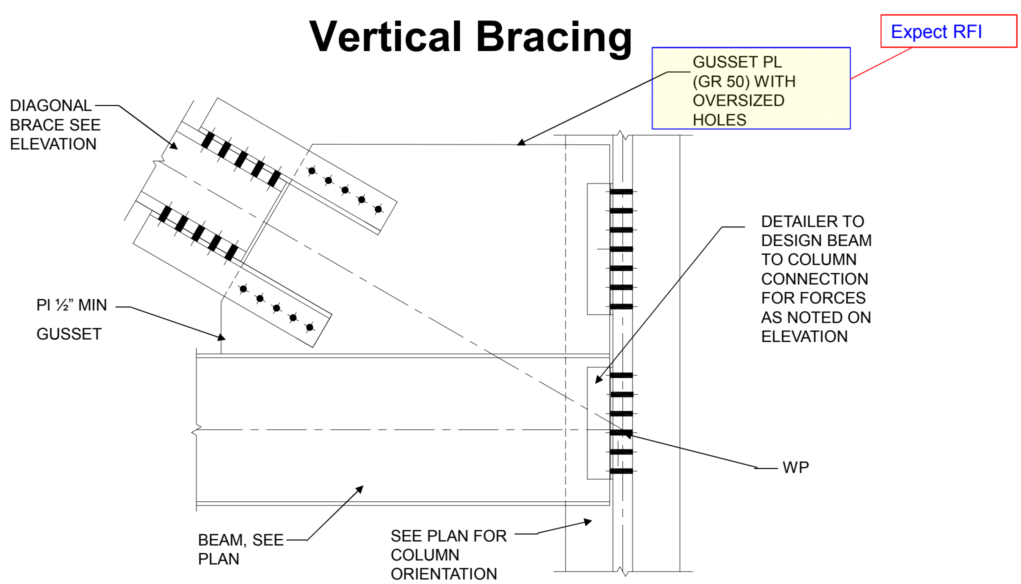
Vertical Bracing



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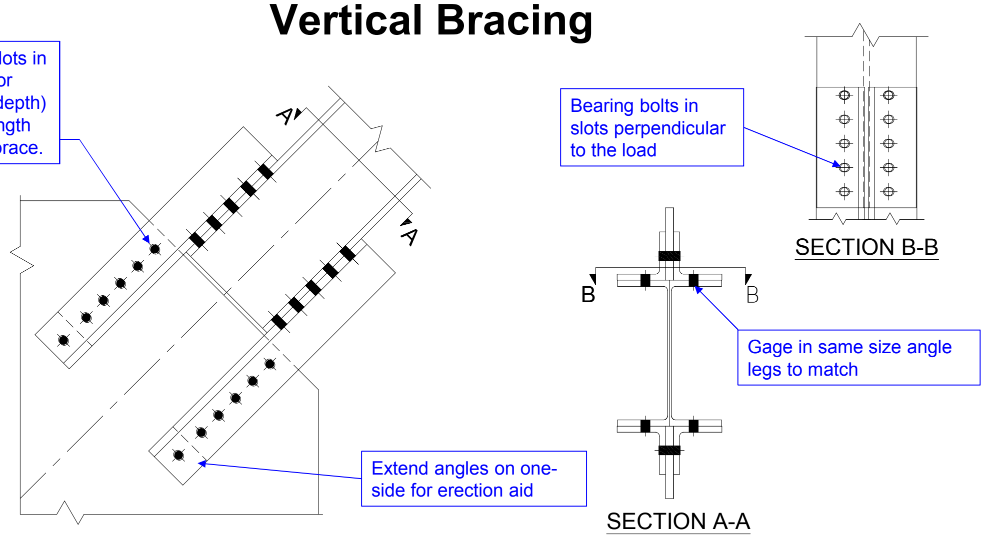
Vertical Bracing



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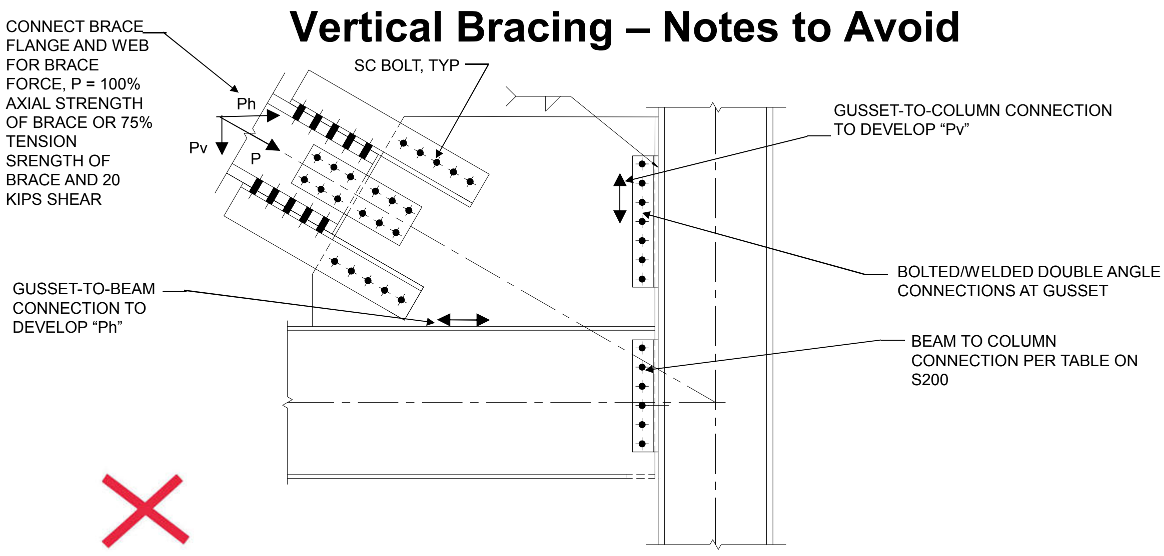
Vertical Bracing – Notes to Avoid



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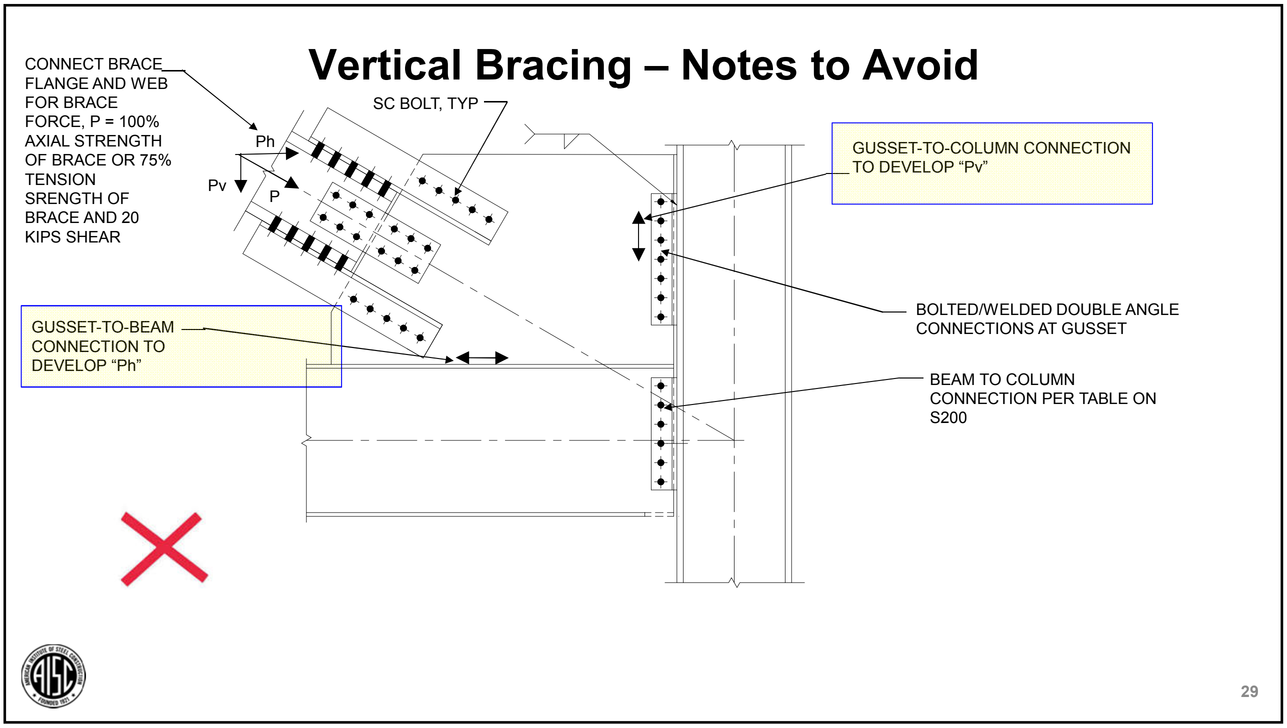
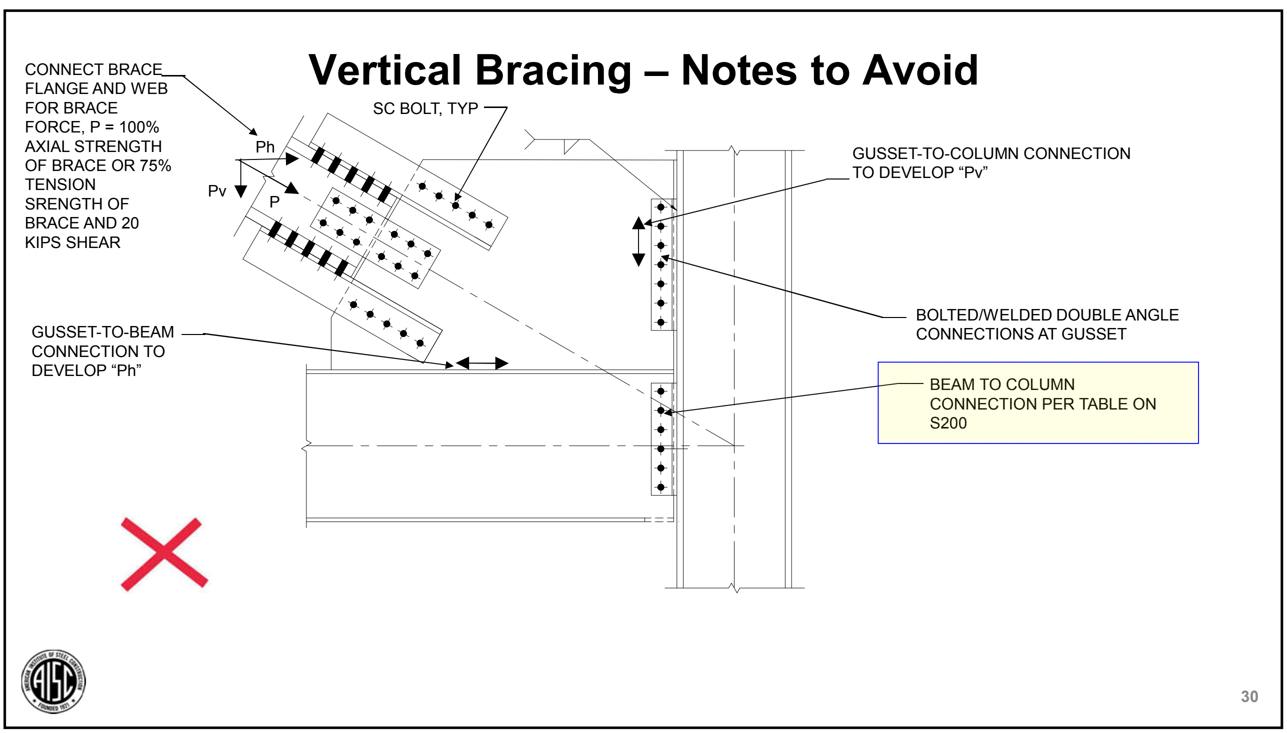
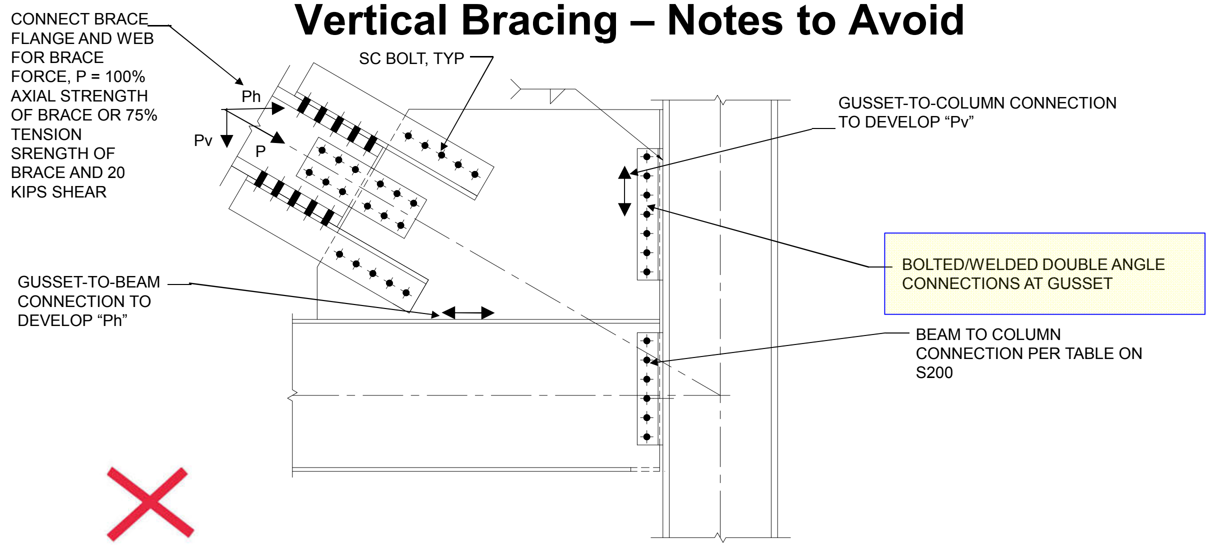

Vertical Bracing – Notes to Avoid
CONNECT BRACE FLANGE AND WEB FOR BRACE FORCE, P = 100% AXIAL STRENGTH OF BRACE OR 75% TENSION SRENGTH OF BRACE AND 20 KIPS SHEAR
SC BOLT, TYP
Ph
Pv
P
GUSSET-TO-COLUMN CONNECTION TO DEVELOP "Pv"
BOLTED/WELDED DOUBLE ANGLE CONNECTIONS AT GUSSET
GUSSET-TO-BEAM CONNECTION TO DEVELOP "Ph"
BEAM TO COLUMN CONNECTION PER TABLE ON S200
29


Vertical Bracing – Notes to Avoid
CONNECT BRACE FLANGE AND WEB FOR BRACE FORCE, P = 100% AXIAL STRENGTH OF BRACE OR 75% TENSION SRENGTH OF BRACE AND 20 KIPS SHEAR
SC BOLT, TYP
Ph
Pv
P
GUSSET-TO-COLUMN CONNECTION TO DEVELOP "Pv"
BOLTED/WELDED DOUBLE ANGLE CONNECTIONS AT GUSSET
GUSSET-TO-BEAM CONNECTION TO DEVELOP "Ph"
BEAM TO COLUMN CONNECTION PER TABLE ON S200
30

Vertical Bracing – Notes to Avoid



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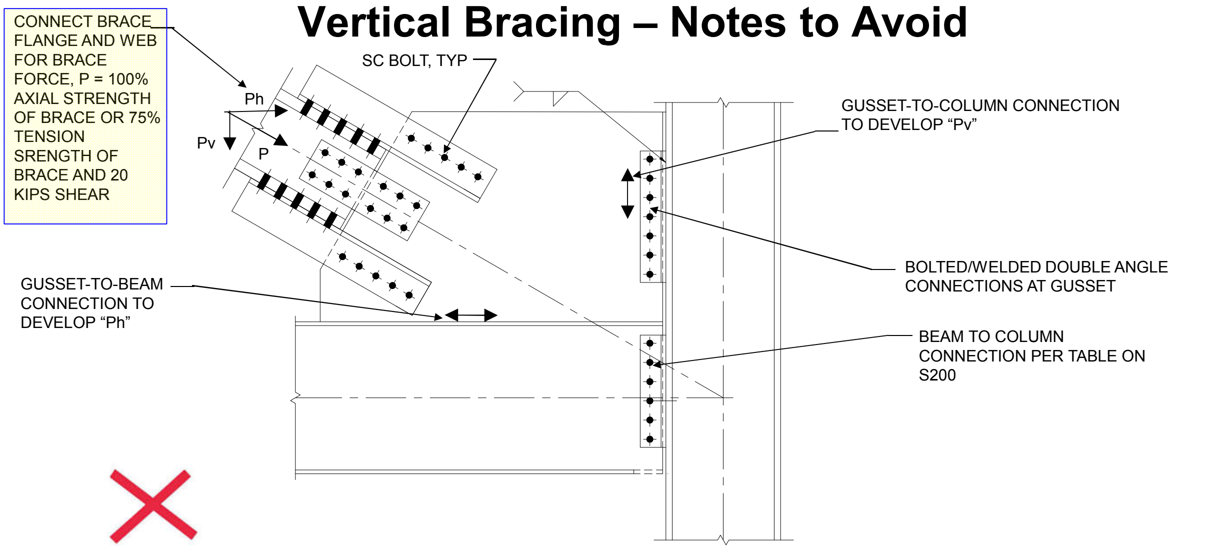
Vertical Bracing – Notes to Avoid



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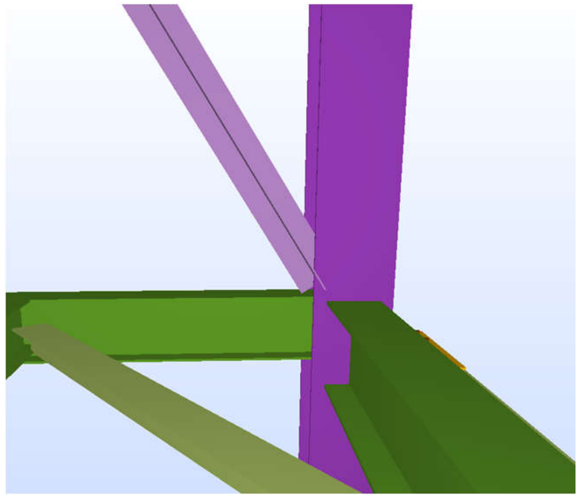
Vertical Bracing – Model Transfer



Vertical Bracing – Model Transfer

	A	B	C	D	E	F	G	H	I	J
1	memid	section	risa_member_type	section_id	start_max_axial_transfer	start_min_axial_transfer	start_max_shear_transfer	start_min_shear_transfer	Start_Max_axial	Start_min_axial
2	Column	Column2	Column	Column	Column	Column	Column	Column	Column	Column1
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
98	ROS_190	LL5x5x8x3	VBrace	LL5x5x8x3					257.088288	-0.49874998
99	ROS_191	LL5x5x8x3	VBrace	LL5x5x8x3					232.496239	0
00	ROS_192	LL5x5x8x3	VBrace	LL5x5x8x3					266.979288	0
01	ROS_193	LL5x5x8x3	VBrace	LL5x5x8x3					241.310989	-0.40844998
02	ROS_194	LL5x5x8x3	VBrace	LL5x5x8x3					254.741538	-0.43784998
03	ROS_195	LL5x5x8x3	VBrace	LL5x5x8x3					240.674689	-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	-14.9005493
10	ROS_200	LL5x5x8x3	VBrace	LL5x5x8x3					255.835638	-14.8028993
14	ROS_205	LL5x5x8x3	VBrace	LL5x5x8x3					343.513784	-21.442049
15	ROS_206	LL5x5x8x3	VBrace	LL5x5x8x3					332.024685	-24.9647989
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					337.168635	-0.21209999

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	-92	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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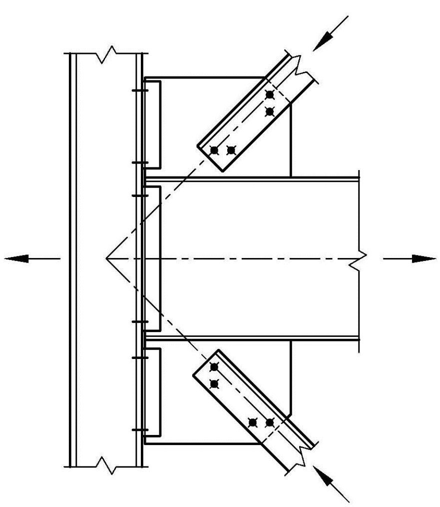
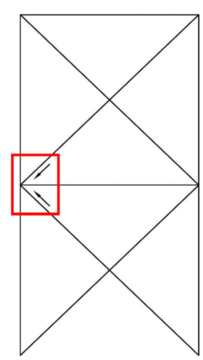
Vertical Bracing



- 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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COMPANY:
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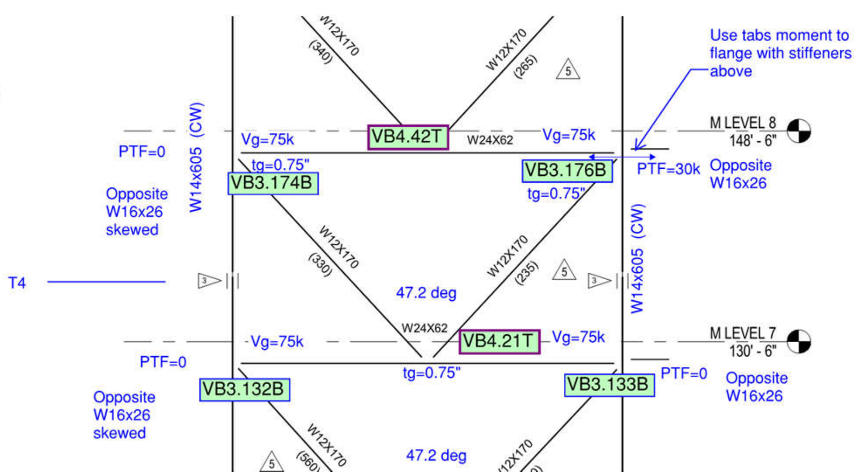
Getting Started with Design

- Forces
- Sizes
- System requirements

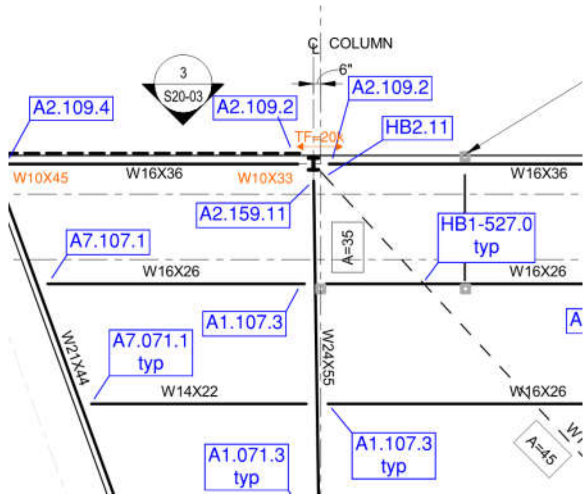
Getting Started with Design

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



Getting Started with Design

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



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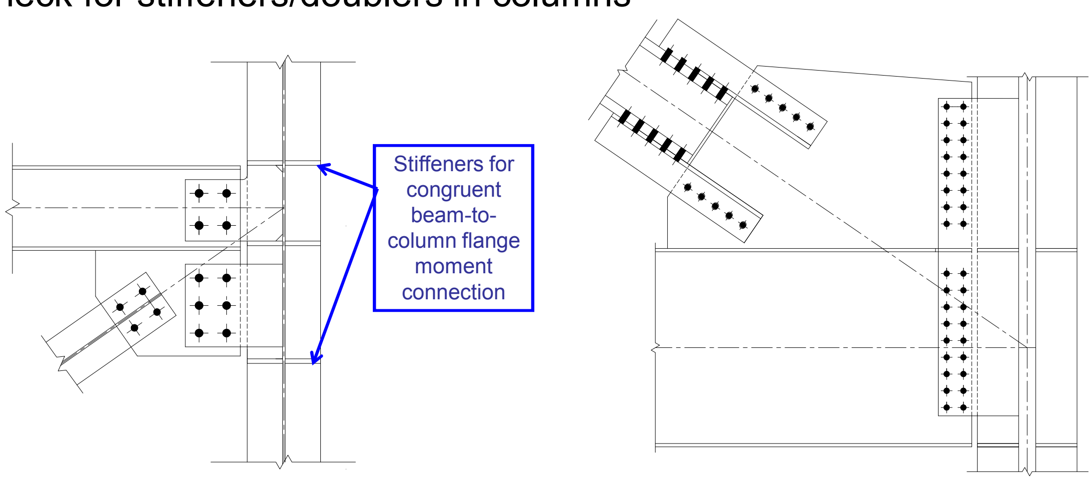
Getting Started with Design

- Check for stiffeners/doublers in columns



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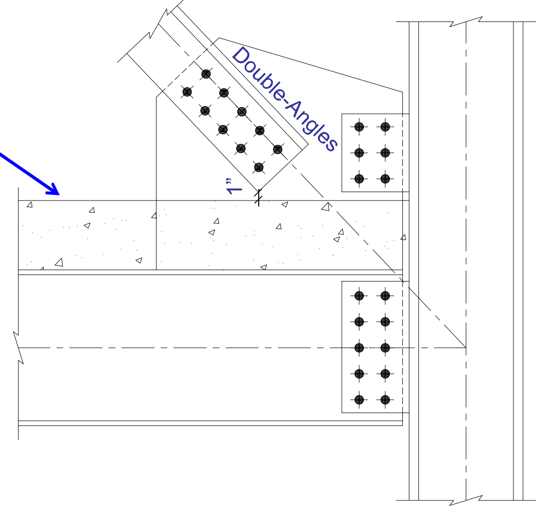
Getting Started with Design

- Flooring Type



Getting Started with Design

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

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

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

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

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

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad \text{(E6-2b)}$$

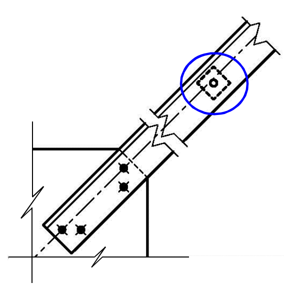
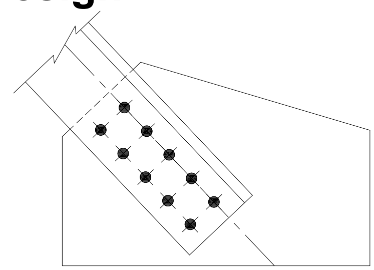


Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB
2L6 LLBB
F_y = 36 ksi

Shape		2L6×4× 7/8		2L6×4× 3/4		2L6×4× 5/8		2L6×4× 9/16		No. of connectors[a]
lb/ft		54.4		47.2		40.0		36.2		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, KL (ft), with respect to indicated axis — X-X Axis	0	345	518	300	450	252	379	229	343	
	4	333	501	290	435	244	366	221	332	
	6	319	479	277	417	234	351	212	318	
	8	300	451	261	393	220	331	200	300	
	10	277	416	242	363	204	307	185	278	
	12	252	378	220	331	186	279	169	254	
	14	224	337	197	296	166	250	151	228	
	16	197	296	173	260	146	220	133	201	b
	18	170	255	150	225	127	191	116	174	
	20	144	216	127	191	108	162	98.6	148	
	22	119	179	106	159	90.1	135	82.5	124	
	24	100	151	89.0	134	75.7	114	69.3	104	
	26	85.5	128	75.9	114	64.5	97.0	59.1	88.8	
	28	73.7	111	65.4	98.3	55.6	83.6	50.9	76.6	
	30	64.2	96.5	57.0	85.6	48.5	72.9	44.4	66.7	
Y-Y Axis	0	345	518	300	450	252	379	229	343	
	4	319	480	273	410	224	336	199	298	
	6	304	456	259	390	213	320	189	284	
	8	283	425	241	363	198	298	176	265	
	10	251	378	214	322	176	264	156	235	
	12	222	334	189	284	155	233	138	208	
	14	192	289	163	244	133	200	119	179	2
	16	162	244	137	205	112	168	99.8	150	

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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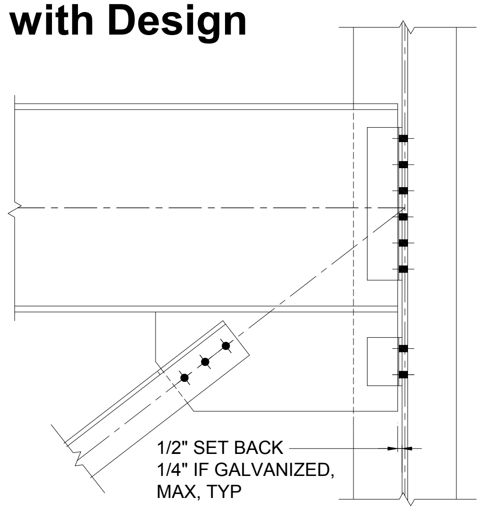
Getting Started with Design

- 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:

(1) End plate moment connections conforming to the requirements of Section E1, or ANSI/AISC 358

(2) 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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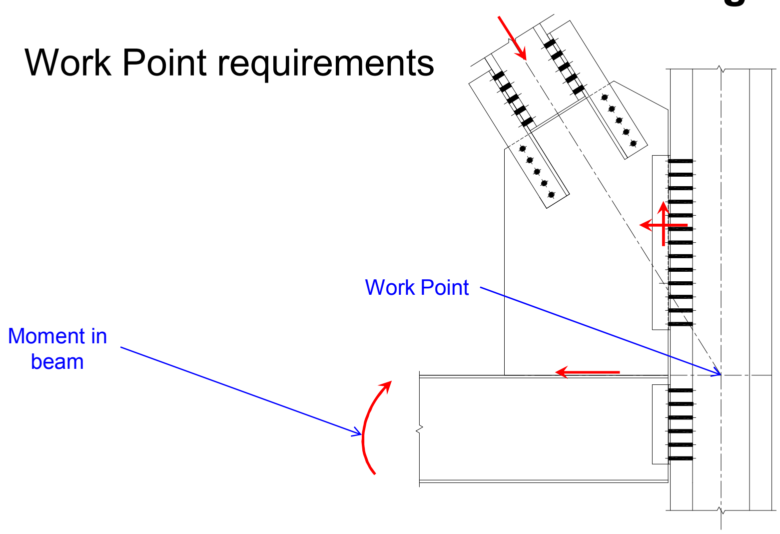
Vertical Bracing

- Work Point requirements



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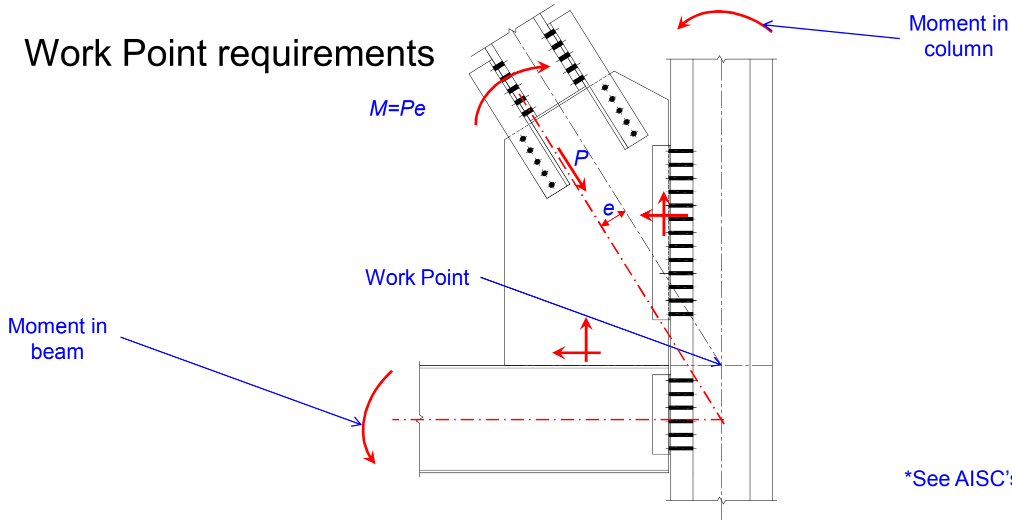
Getting Started with Design

- Work Point requirements



*See AISC's Design Guide 29

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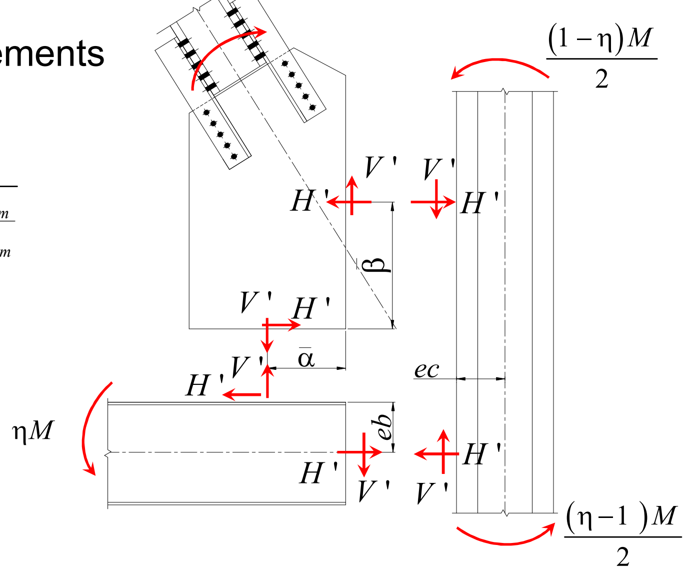
Getting Started with Design

- WP requirements

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



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

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

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

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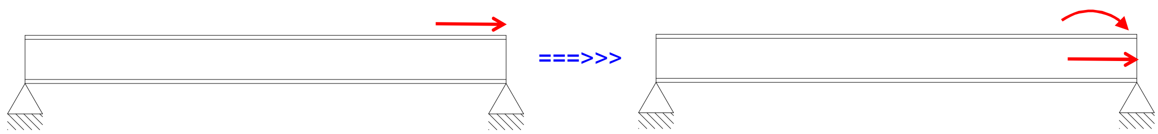
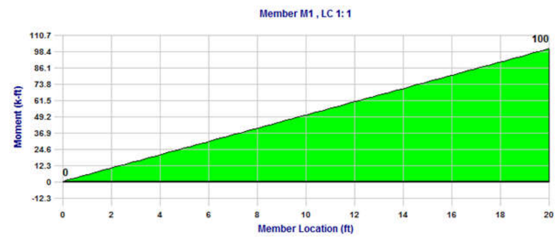
Getting Started with Design

- WP requirements



Work Point at Top of Beam



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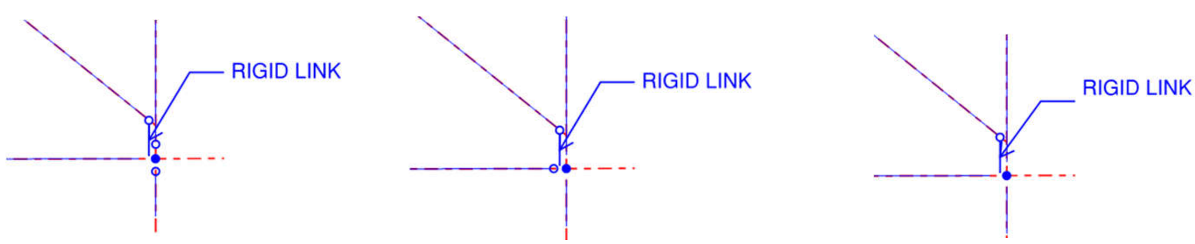
Frame Analysis

- WP requirements



Brace eccentricity moment induced in beam

Brace eccentricity moment induced in column

Brace eccentricity moment induced in column and beam

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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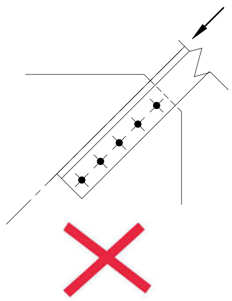
Getting Started with Design

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



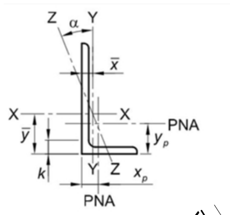
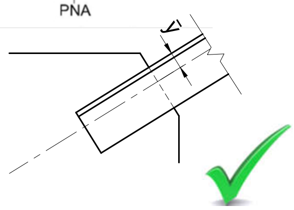


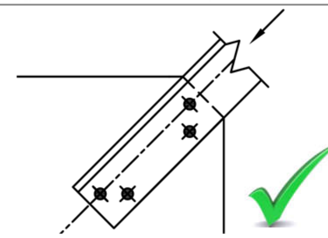
Table 1-7A

Workable Gages in Angle Legs, in.

Leg	12	10	8	7	6	5	4	3 1/2	3	2 1/2	2	1 3/4	1 1/2	1 3/8	1 1/4	1
g	6	5	4 1/2	4	3 1/2	3	2 1/2	2	1 3/4	1 3/8	1 1/8	1	7/8	7/8	3/4	5/8
g_1	3	3	3	2 1/2	2 1/4	2	–	–	–	–	–	–	–	–	–	–
g_2	2 1/2	2 1/2	3	3	2 1/2	2	–	–	–	–	–	–	–	–	–	–
g_3	2 1/2	2 1/2	–	–	–	–	–	–	–	–	–	–	–	–	–	–
g_4	2 1/2	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

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





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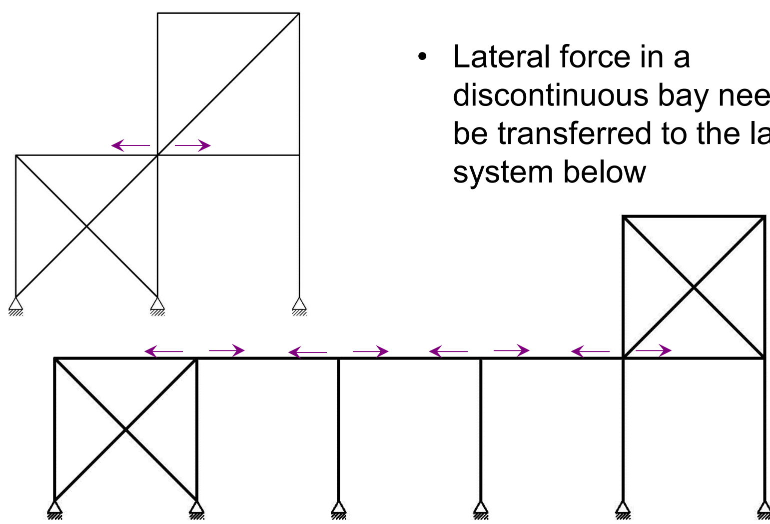
Transfer Forces



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

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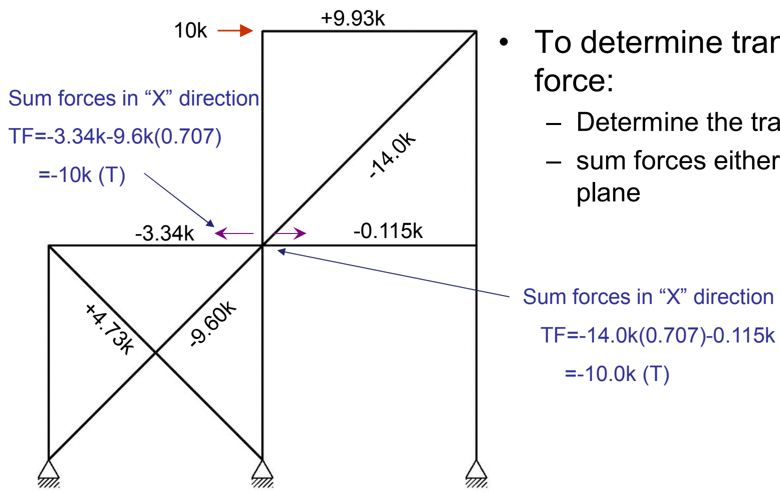
Transfer Forces - Discontinuous Braced Bays



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

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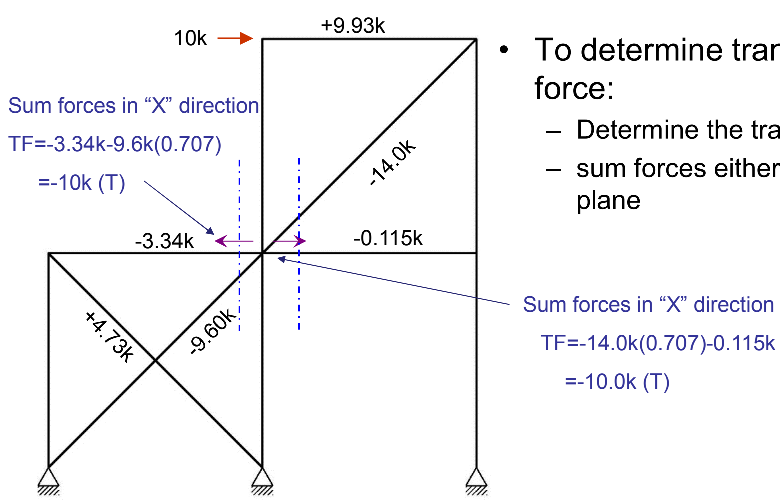
Transfer Forces - Discontinuous Braced Bays



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

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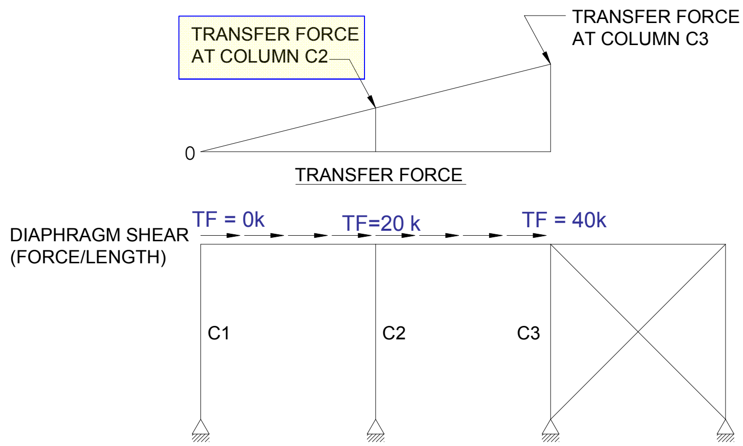
Getting the Load to the Lateral System



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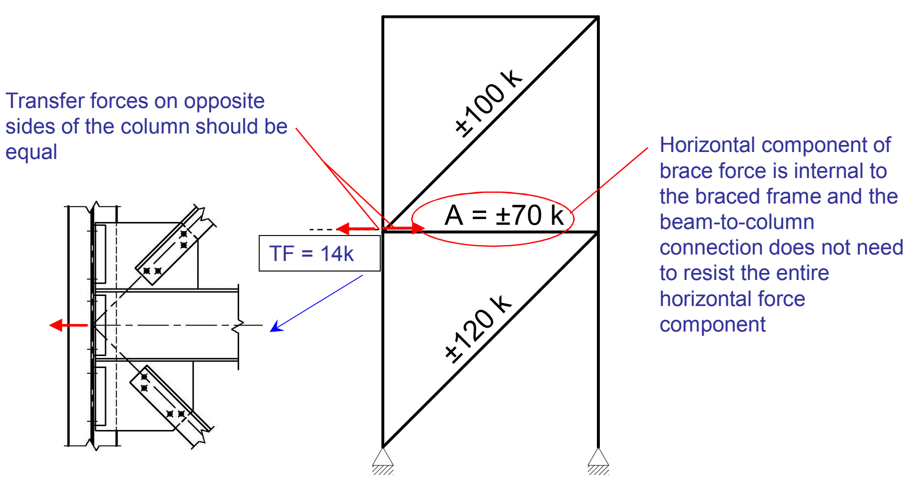
Transfer Forces



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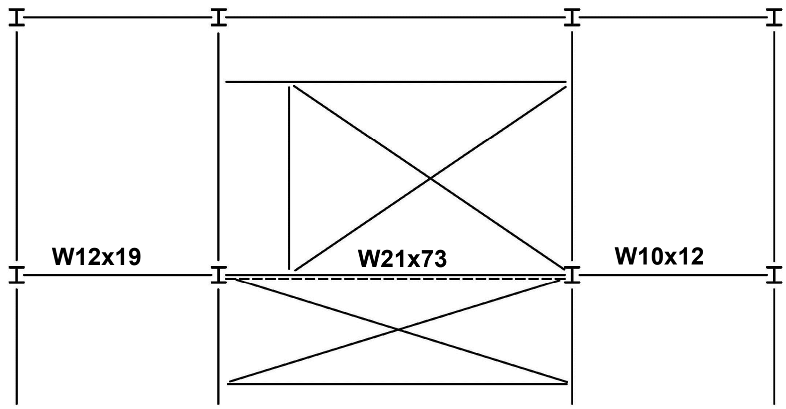
Getting the Load to the Lateral System



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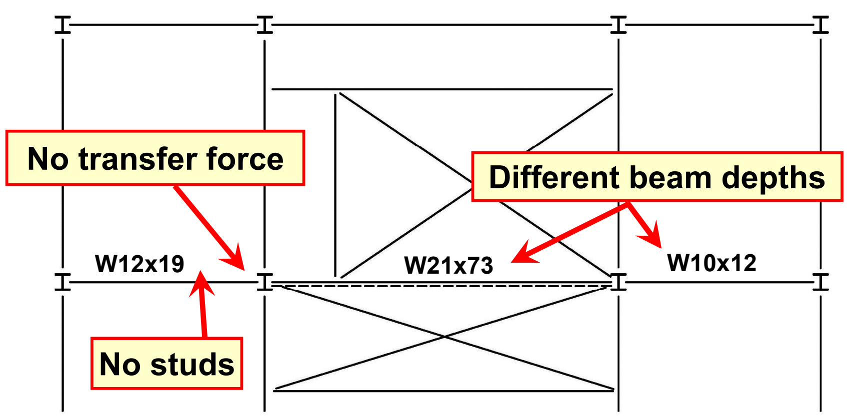
Getting the Load to the Lateral System



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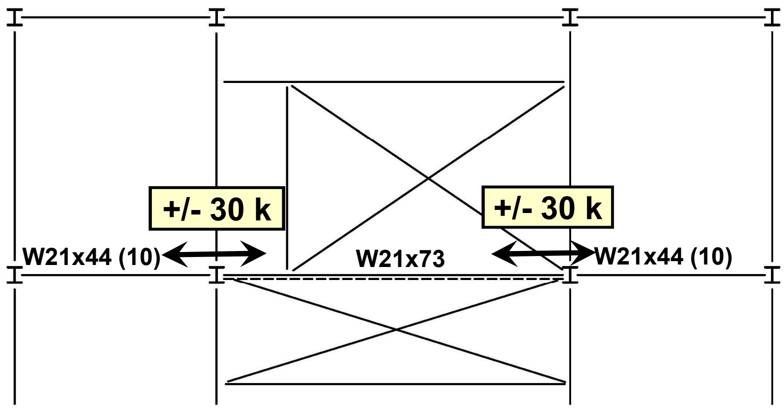
Transfer Forces



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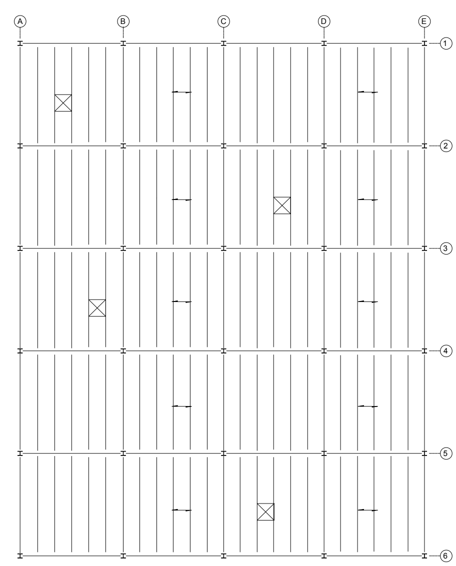
Transfer Forces

- Transfer Forces at Roof
  - Metal deck has limited strength



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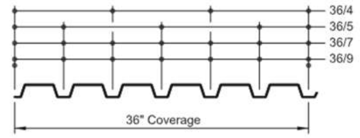
Diaphragm Strength

B, BA 18 GA. Diaphragm Design

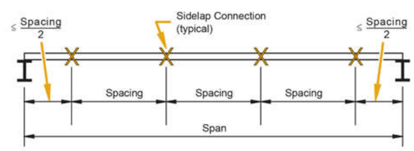
Design Thickness = 0.0474 in.
Support Fasteners: 5/8" Puddle Welds
Side Lap Fasteners: # 10 Screws

Φ (EQ): 0.55 Ω (EQ): 3.00
Φ (Wind): 0.70 Ω (Wind): 2.35
Φ (Other): 0.60 Ω (Other): 2.65

Fill Type	Support Fastener Pattern	Side Lap Conn. per Span	Nominal Diaphragm Shear Strength (plf) — Center to Center Span (ft. - in.) 5 - 0	5 - 6	6 - 0	6 - 6	7 - 0	7 - 6	8 - 0	8 - 6	9 - 0	9 - 6	10 - 0	11 - 0	12 - 0	12 - 4	K_1 (ft.$^{-1}$)
No Fill (Bare Deck)		11	2858	2663	2490	2335	2196	2072	1960	1858*	1766*	1683*	1606*	1471*	1356*	1320*	0.148
	36/5 (B) D_n = 281 (BA) D_n = 288	0	994	899	820	753	696	646	603	564	530	500	472	425	390	379	0.739
		2	1347	1239	1147	1066	986	917	857	804	756	-	-	-	-	-	0.437
		3	1504	1389	1288	1201	1123	1053	984	923	869	821	777	703	644	-	0.363
		4	1653	1530	1423	1329	1246	1172	1106	1043	982	928	879	795	729	709	0.310
		5	1791	1664	1552	1452	1364	1285	1214	1150	1092	1035	981	888	814	792	0.271
		6	1919	1789	1673	1570	1477	1393	1318	1250	1188	1132	1081	980	899	874*	0.241
		7	2039	1907	1788	1681	1585	1498	1419	1348	1282	1223	1168	1072	983*	957*	0.216
		8	2149	2016	1896	1787	1688	1598	1516	1442	1373	1311	1253	1151*	1064*	1038*	0.196
		9	2250	2118	1997	1887	1786	1694	1609	1532	1461	1396	1336*	1229*	1137*	1110*	0.180
		10	2344	2213	2092	1981	1879	1785	1699	1619	1546	1479*	1417*	1305*	1209*	1180*	0.166
		11	2430	2301	2181	2070	1967	1872	1784	1703	1628	1559*	1495*	1379*	1279*	1249*	0.154
	36/4 (B) D_n = 399 (BA) D_n = 408	0	753	680	619	568	524	485	452	422	396	373	352	316	289	282	0.923
		2	1105	1019	945	880	814	757	706	662	622	-	-	-	-	-	0.496
		3	1253	1161	1080	1009	946	890	834	782	735	694	657	593	544	-	0.403
		4	1389	1292	1207	1131	1063	1002	947	898	848	801	759	686	629	612	0.339
		5	1511	1413	1324	1244	1173	1108	1049	996	948	904	860	778	713	694	0.293
		6	1622	1523	1432	1350	1276	1208	1146	1090	1039	992	948	871	798	777	0.257
		7	1721	1622	1532	1449	1372	1303	1239	1180	1126	1076	1030	949	878	857	0.230
		8	1810	1713	1623	1539	1462	1391	1326	1265	1209	1157	1109	1023	949*	926*	0.207
		9	1889	1795	1706	1623	1546	1474	1407	1346	1288	1235	1185	1096*	1018*	994*	0.189
		10	1960	1869	1782	1700	1624	1552	1484	1422	1363	1308	1257	1165*	1084*	1059*	0.174
		11	2024	1936	1851	1771	1695	1624	1557	1493	1434	1379	1326*	1232*	1148*	1123*	0.161



B, BI, BV, BIV, BA, BIA, F, 1.5FDR, 1.5FDI, 1.5FDV, 1.5FDIV, 1.5CD, 1.5CDI



1511 plf/Ω(Wind)
1511/2.35 = 643 plf

NEW MILLENNIUM
BUILDING SYSTEMS

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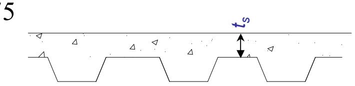
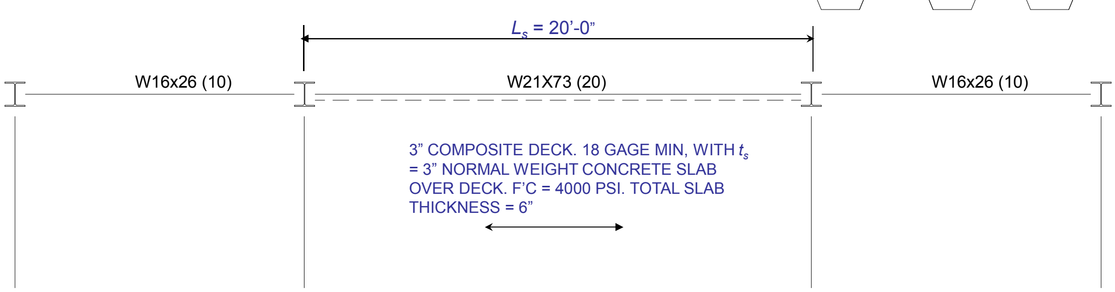
Diaphragm Strength

- Transfer Forces at Composite Deck
  - Consider topping slab

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

$$\phi = 0.75$$





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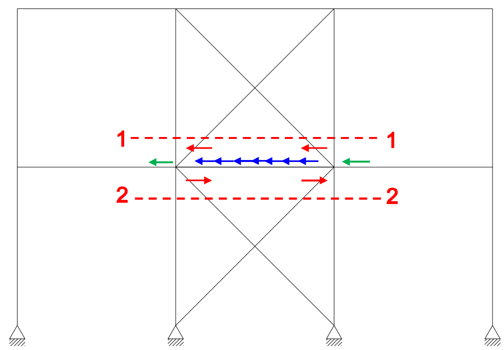
Diaphragm Strength

- Transfer Forces at Composite Deck

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

$$\phi = 0.75$$

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



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

Deck Perpendicular
Weak Studs per rib
(R_p = 0.60)

Table 3-21
Shear Stud Anchor Q_n

F_u = 65 ksi

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 pcf, f'_c = 3 ksi	Normal Weight Concrete w_c = 145 pcf, f'_c = 4 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 3 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 4 ksi
...uds per rib (R_p = 0.60)	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)

$$Q_n = 0.5A_{sa}\sqrt{f'_c E_c} \le R_g R_p A_{sa} F_u$$

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

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

F_u = specified minimum tensile strength of stud = 65 ksi

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

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

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

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

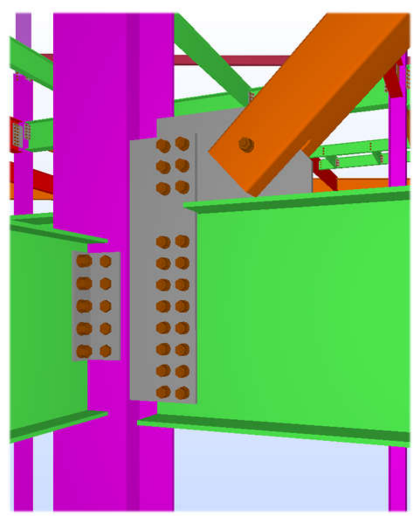
User Note: The table below presents values for R_g and R_p for several cases. Available strengths for steel headed stud anchors can be found in the AISC *Steel Construction Manual*.

Condition	R_g	R_p
No decking	1.0	0.75
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \ge 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85[a]	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[b]
2	0.85	0.6[b]
3 or more	0.7	0.6[b]

h_r = nominal rib height, in. (mm)
w_r = average width of concrete rib or haunch (as defined in Section I3.2c), in. (mm)
[a] For a single steel headed stud anchor
[b] This value may be increased to 0.75 when $e_{mid\text{-}ht} \ge 2$ in. (50 mm).

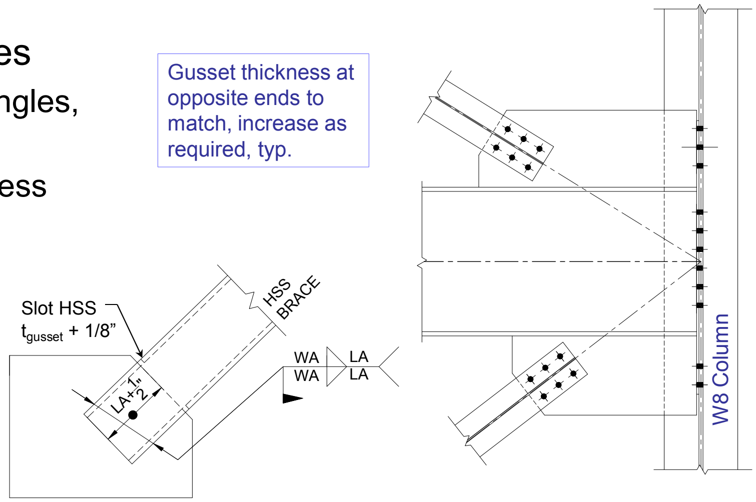
Fabricator Preferences and Standards

- 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



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



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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
- 15th Ed. Manual, Part 13
- AISC's Design Examples

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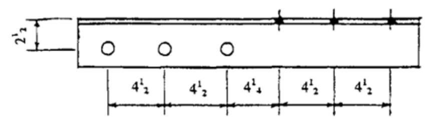
Vertical Bracing – Articles

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

Table 3. Summary of Proposed Effective Length Factors[a]

Gusset Configuration	Effective Length Factor	Buckling Length	$\frac{P_{exp}}{P_{calc}}$
Compact corner	_ [b]	_ [b]	1.36
Noncompact corner	1.0	l_{avg}	3.08
Extended corner	0.6	l_1	1.45
Single brace	0.7	l_1	1.45
Chevron	0.65	l_1	1.17

[a] Table 7 from Dowswell (2006) with revisions.
[b] Yielding is 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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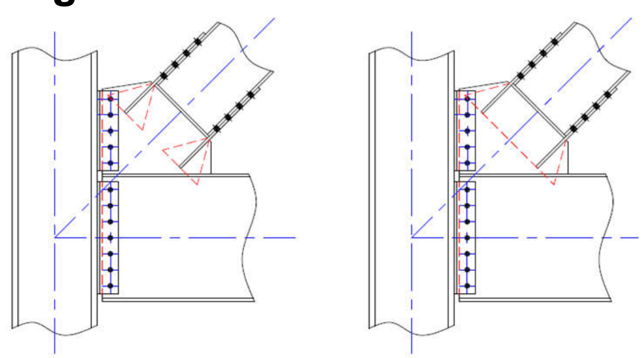
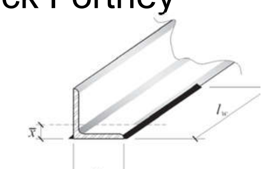
Vertical Bracing – Articles

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



(a) Correct Whitmore length. (b) Incorrect Whitmore length.



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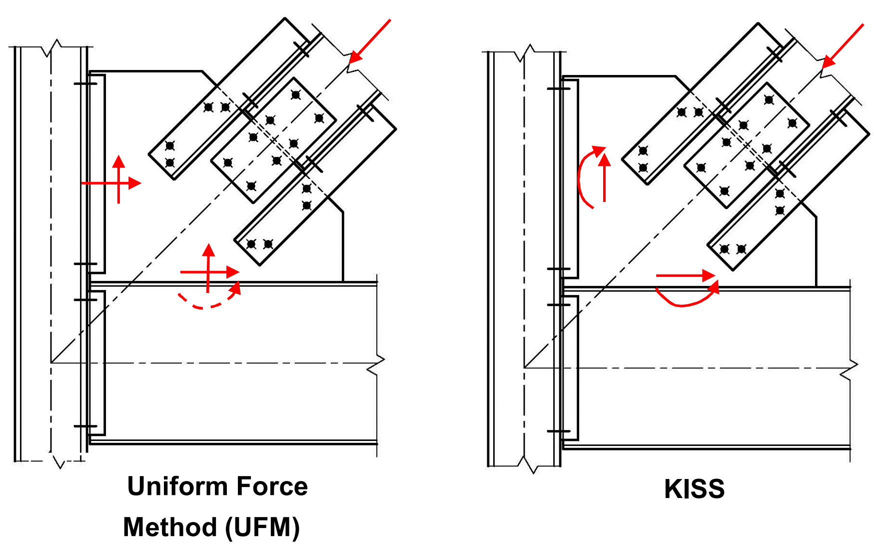
Vertical Bracing – Articles

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

75

Connections-Bracing



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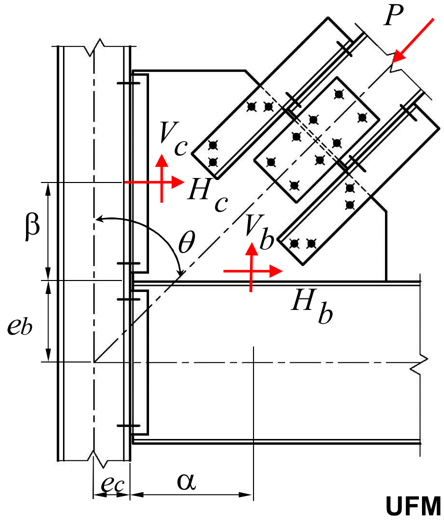
Connections-Bracing UFM



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

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

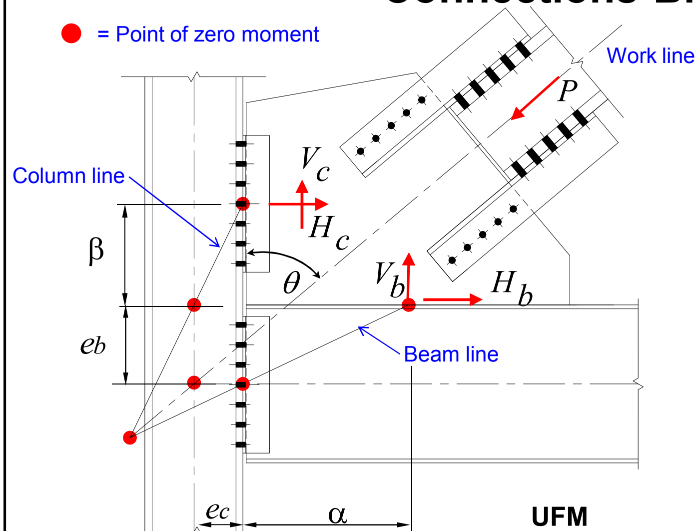
$$V_c = \frac{\beta}{r} P$$

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

77

Connections-Bracing UFM

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



$$y = mx + b$$

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

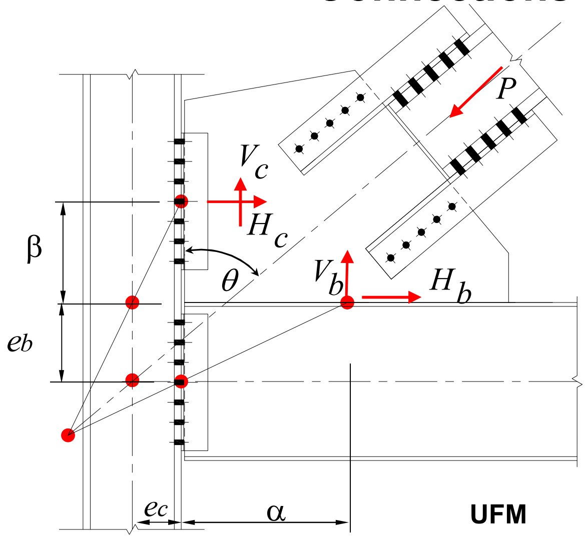
$$y_{work\_line} = \frac{1}{\tan(\theta)} x$$

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

78

Connections-Bracing UFM

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



Work line = column line and solve for x

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

Work line = beam line and solve for x

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

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

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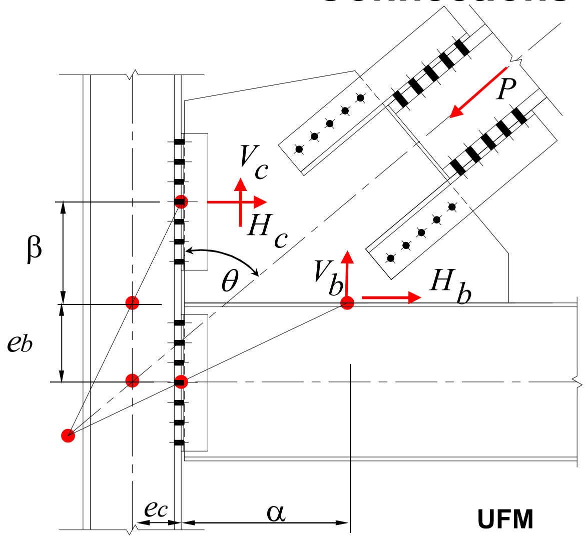
Connections-Bracing UFM

*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

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

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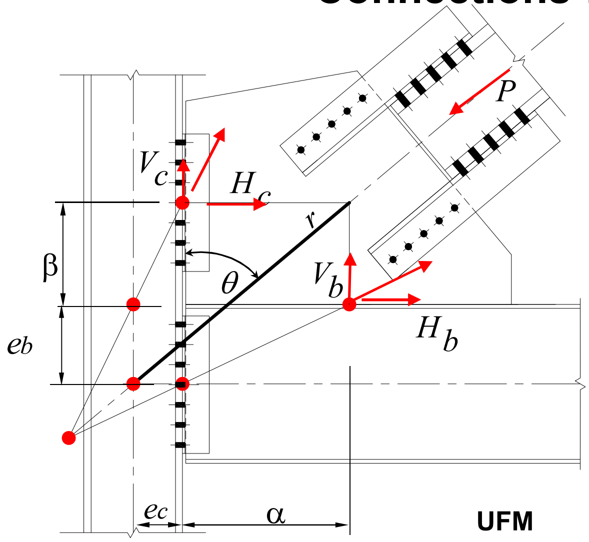
Connections-Bracing UFM

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



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

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

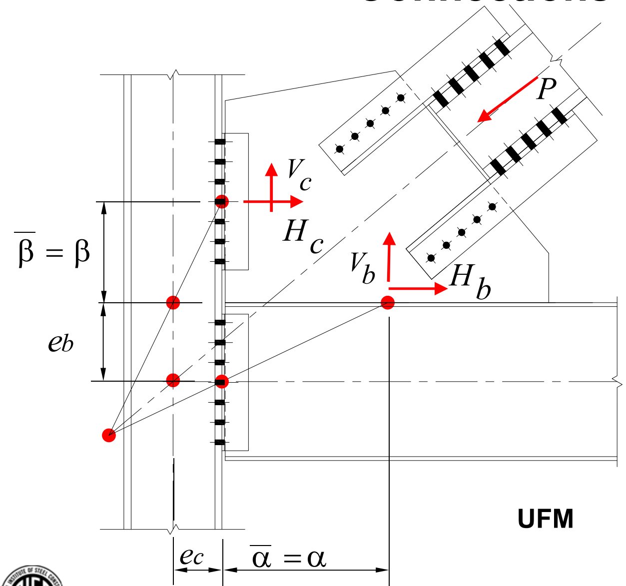
$$V_b = \frac{e_b}{r}P \qquad H_b = \frac{\alpha}{r}P$$

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

81

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 α.
If $\bar{\alpha} = \alpha$, no moment

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

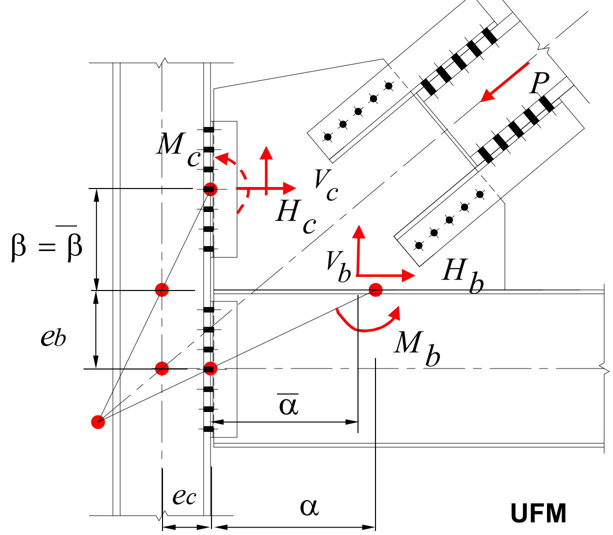
To column flange, set $\bar{\beta} = \beta$

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

82

Connections-Bracing UFM

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



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

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

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

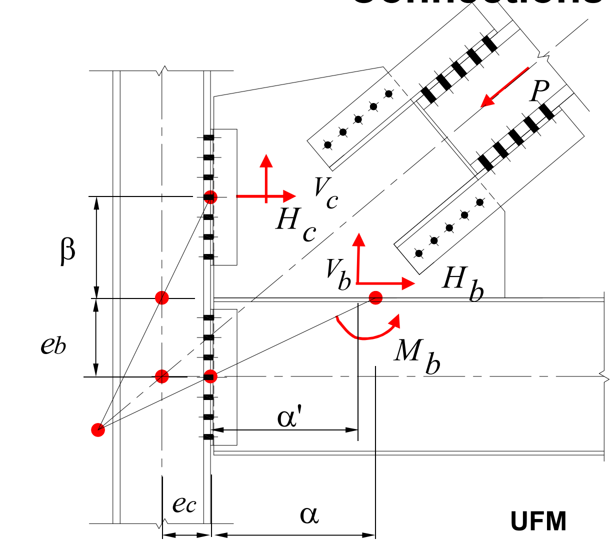
$$M_b = V_b(\alpha - \bar{\alpha}) \qquad M_c = H_c(\beta - \bar{\beta})$$

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

83

Connections-Bracing UFM

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



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

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

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

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

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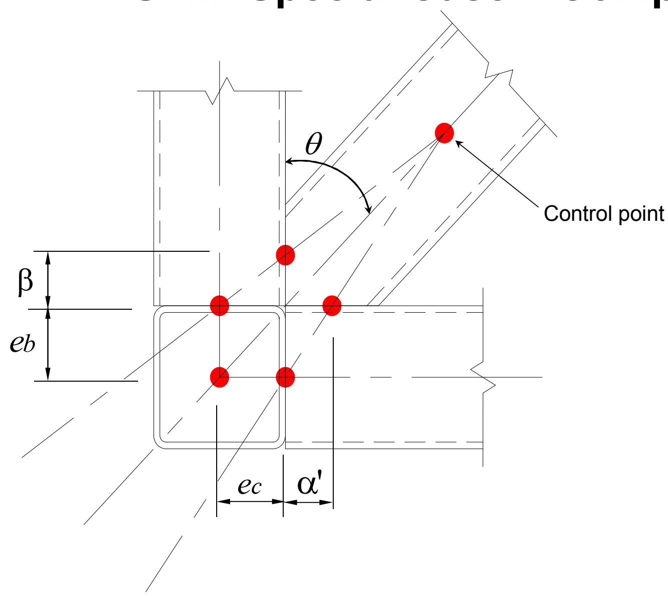
UFM Special Case – Compact Gussets

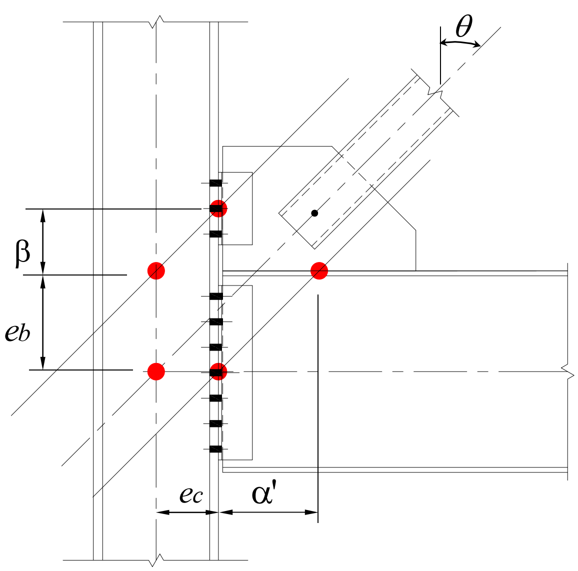


$$\mathrm{ATAN}\left(\frac{e_c}{\beta}\right) > \theta$$

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

85

UFM Special Case – Parallel Force Method (PFM)



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

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

*Reference AISC's
AISC's DG29

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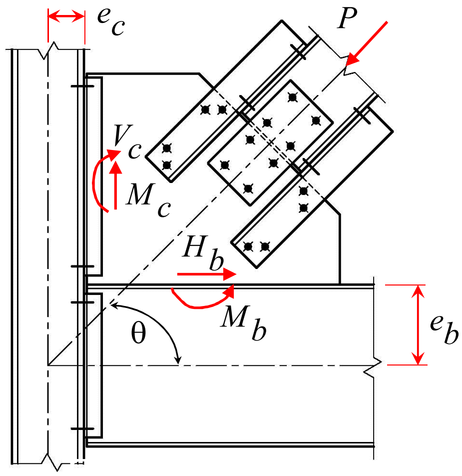
Connections-Bracing KISS



KISS

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

$$M_b = H_b(e_b)$$

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

$$M_c = V_c(e_c)$$

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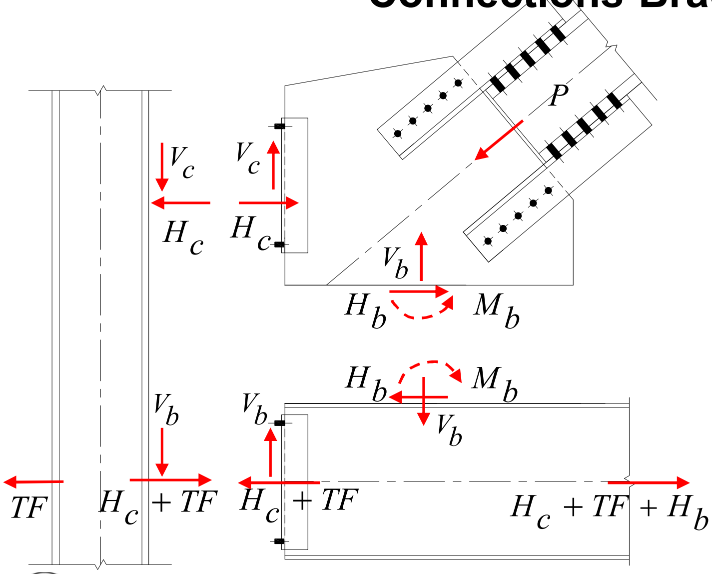
Connections-Bracing UFM



UFM

- Hc from gusset-to-column to column-to-beam
- Hb force is not resisted by column-to-beam connection
- Vb from gusset-to-beam to beam-to-column
- Add gravity to Vb
- Consider transfer forces and Hc
- Allows for force redistribution

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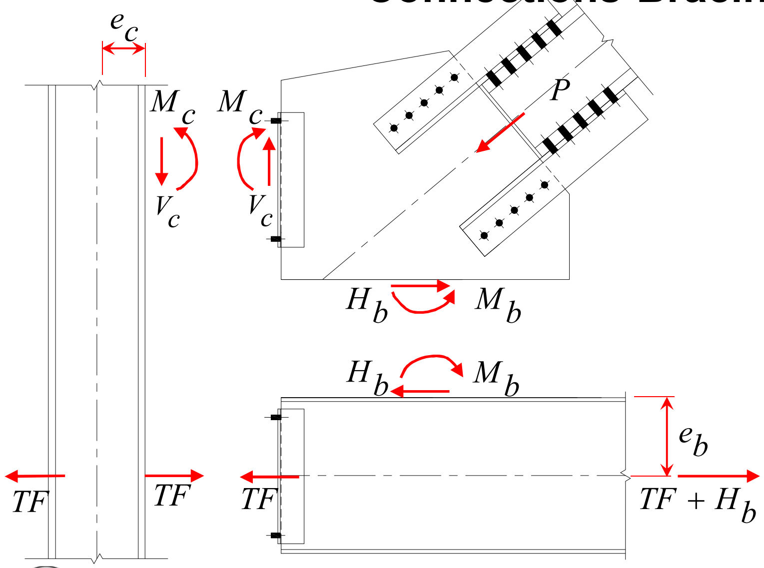
Connections-Bracing KISS



- 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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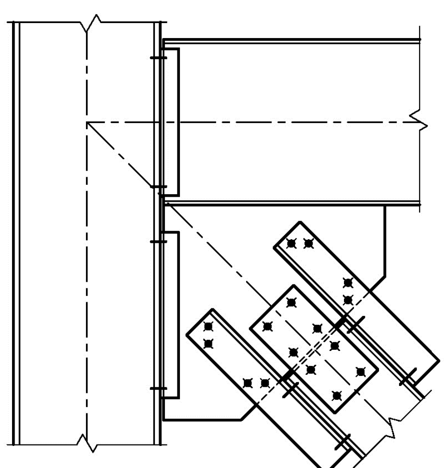
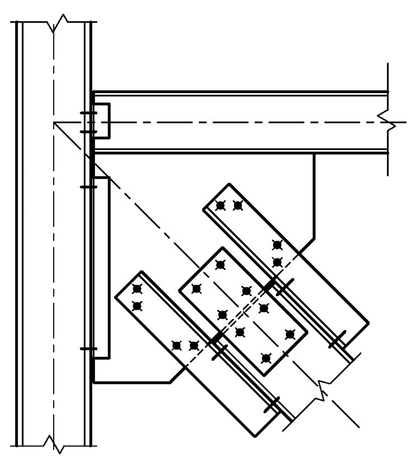
Connections-Bracing



UFM Advantage



Possible KISS Advantage

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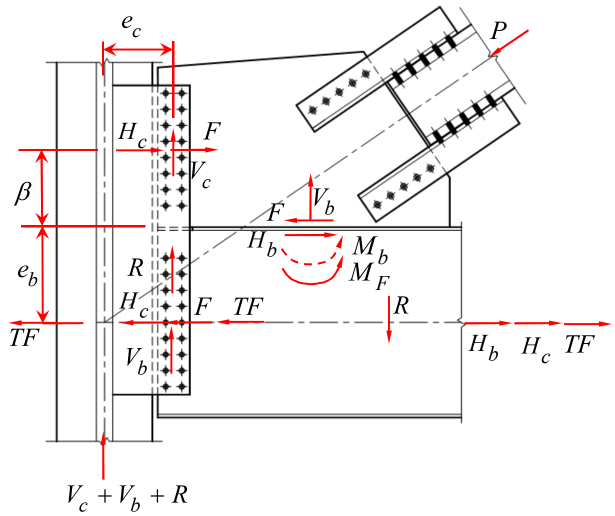
Connections-UFM



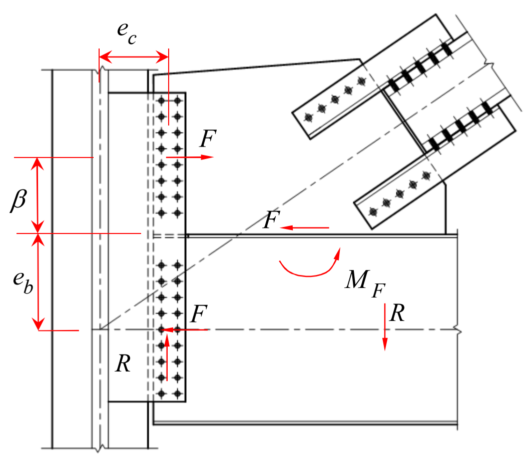
UFM Special Case IV

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

$$M_F = F\beta$$

91

Connections-UFM Gravity Distribution



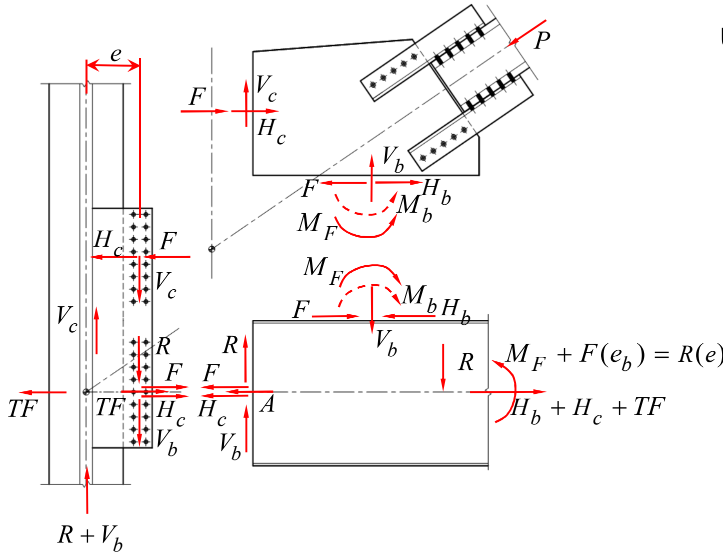
UFM Special Case IV

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

$$M_F = F\beta$$

92

Connections-UFM



UFM Special Case IV

- No eccentricity at bolt group if e_c= e

$$H_c = H_c + F$$

$$M_b = M_b + M_F$$

$$H_b = H_b + F$$

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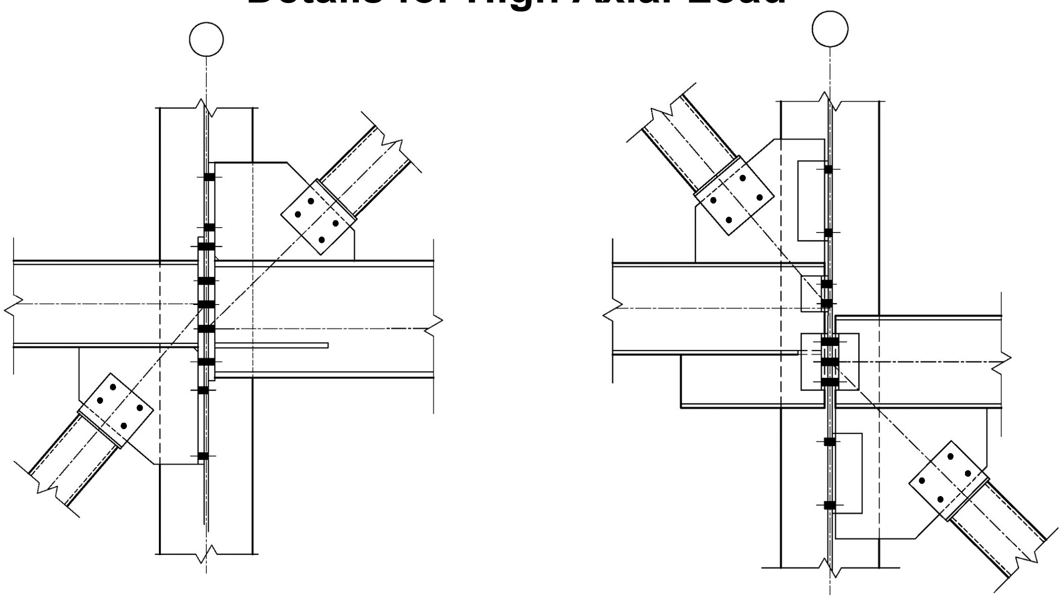
Details for High Axial Load



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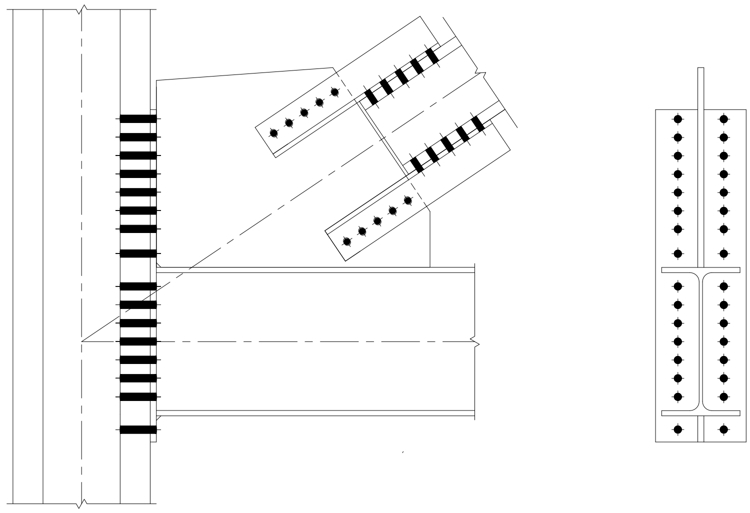
Details for High Axial Load and/or Moment



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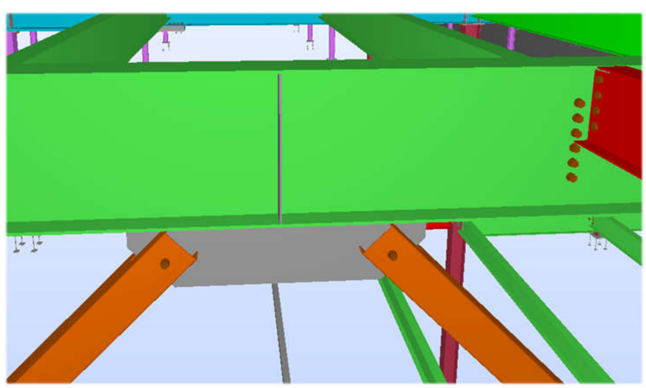
Chevron Connections

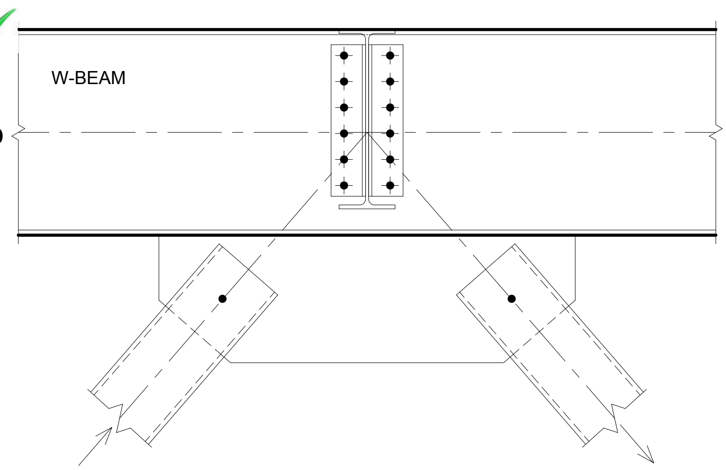
- Stability bracing
- Brace force directions



96

Chevron Connections

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



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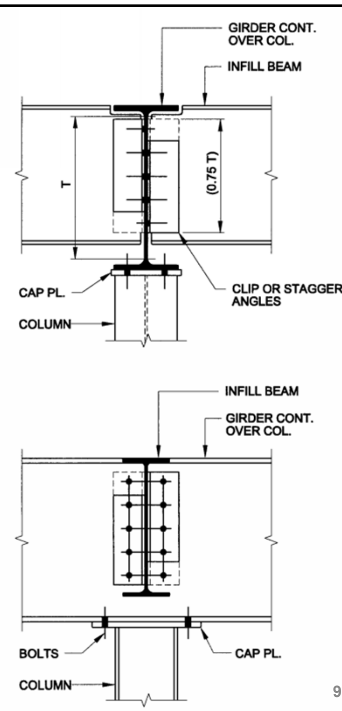
Chevron Connections

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

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

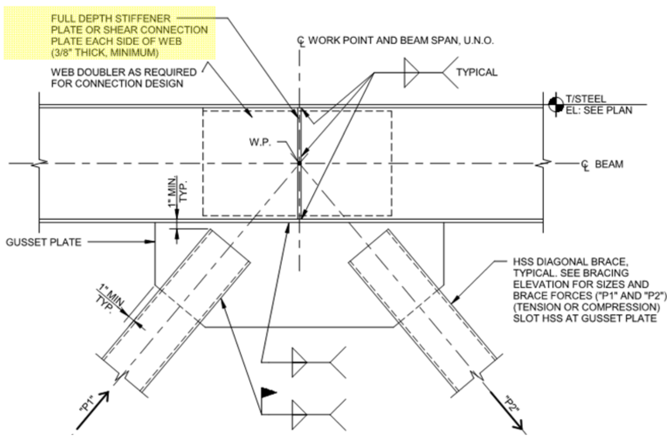
1. When an infill beam frames into the continuous beam at the column top, the required stability normally can be provided by using connection element(s) for the infill beam that cover three-quarters or more of the T-dimension of the continuous beam. Alternatively, connection elements that cover less 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.

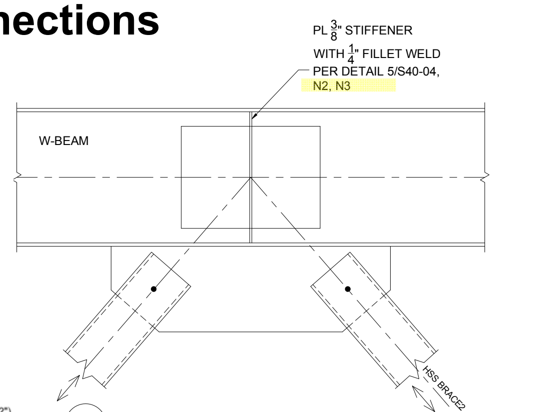


98

Chevron Connections

- Optimize connections





VB4-1 TYPICAL BRACE AT BEAM CENTER - CHEVRON

NOTE:

N1. WELDERS IN FIELD WILL ADD WELD FOR GAP PER AWS. DO NOT ADD WELD FOR GAP IN DETAILING

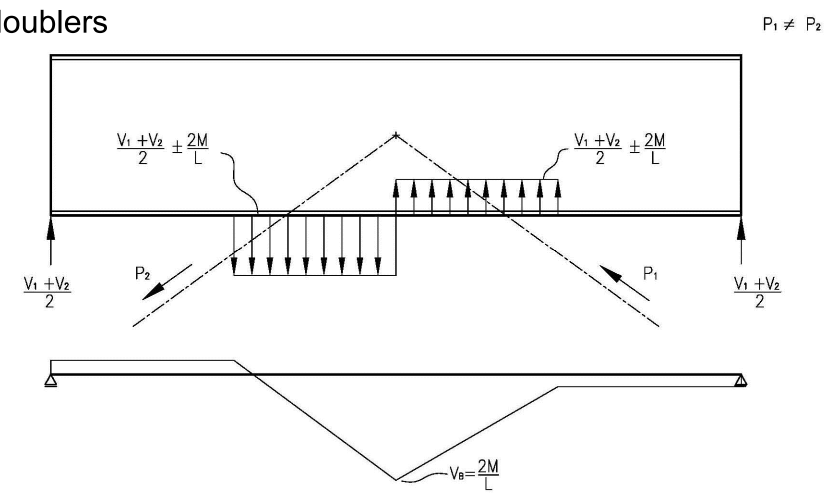
N2. STIFFENER ONLY NEEDED WHERE PERPENDICULAR BEAM THAT HAS A CONNECTION DEPTH = 3/4 CHEVRON BEAM DEPTH DOES NOT FRAME INTO CHEVRON BEAM AT WP. SEE RFI 20 RESPONSE

N3. AT LOCATIONS THAT REQUIRE DOUBLERS, DOUBLER TO BE CONTINUOUS. WELD STIFFENER TO DOUBLER AS NEEDED.

99

Chevron Connections

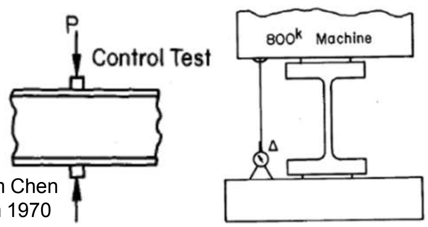
- Check for doublers



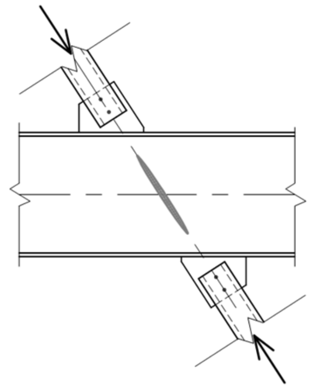
100

Single Brace Connections

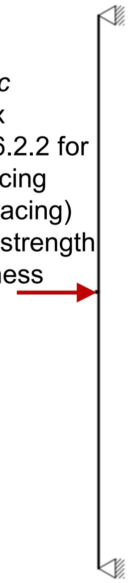
- Stability Concern
  - See Chapter C and Appendix 6 for Stability
  - Web compression buckling: *Spec* Equation J10-8 developed from point loads and restrained flanges



Test Set-up from Chen and Oppenheim 1970



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



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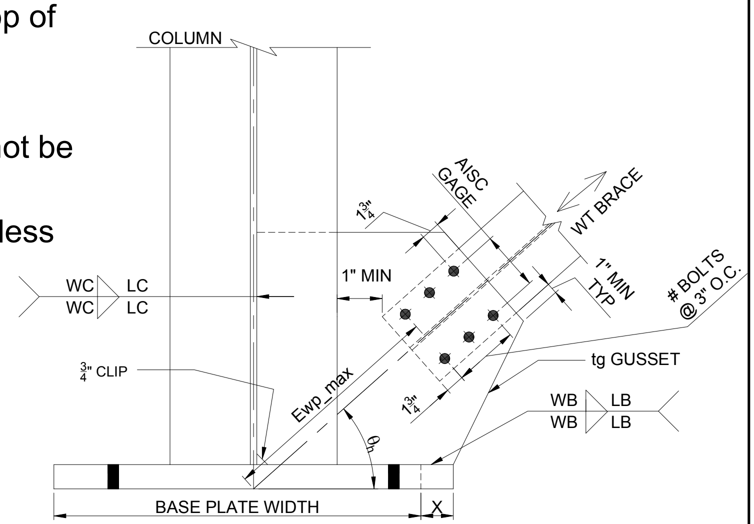
Brace to Base

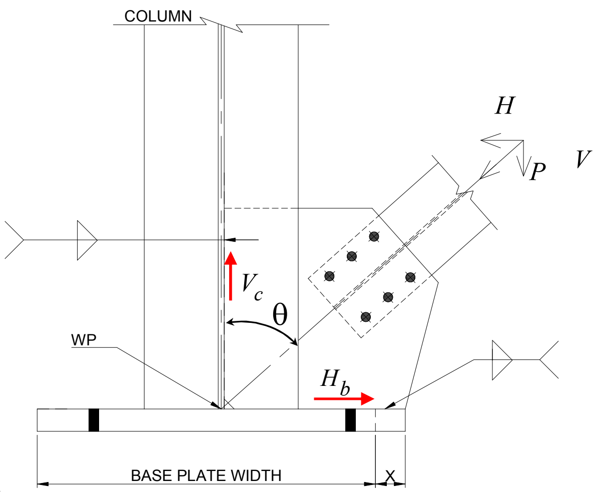
- Work Point location: Typically top of plate or bottom of Plate
- See DG29
- Indicate base plate can or can not be extended
- Avoid welding washer plates unless anchor rods checked for shear and bending



102

Brace to Base

- Work Point location – Top of Plate



$$H = P\sin\theta$$

$$V = P\cos\theta$$

$$V_c = V$$

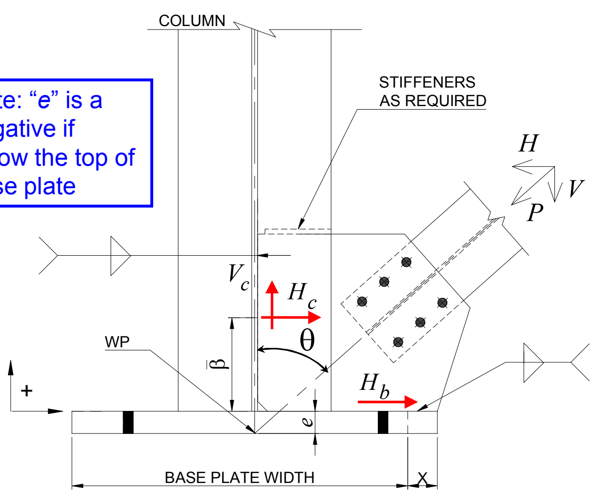
$$H_b = H$$

103

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 $\bar{\beta}$ to determine H_b

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

$$H_c = H - H_b$$

*Reference DG29

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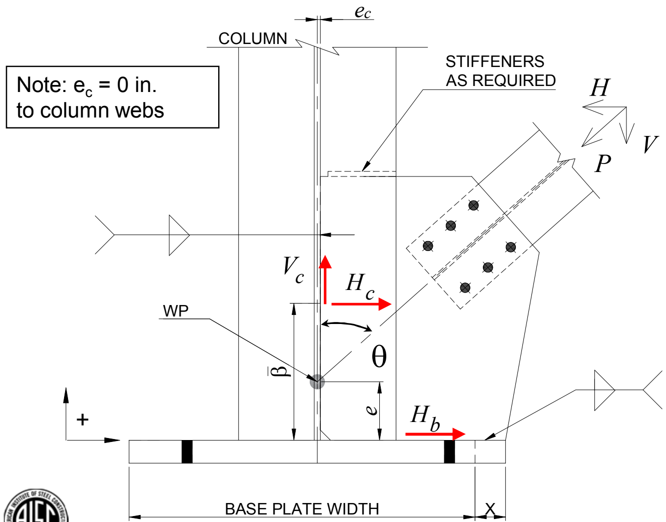
Brace to Base

- General Work Point FBD



$$H = P\sin\theta$$

$$V = P\cos\theta$$

$$V_c = V$$

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

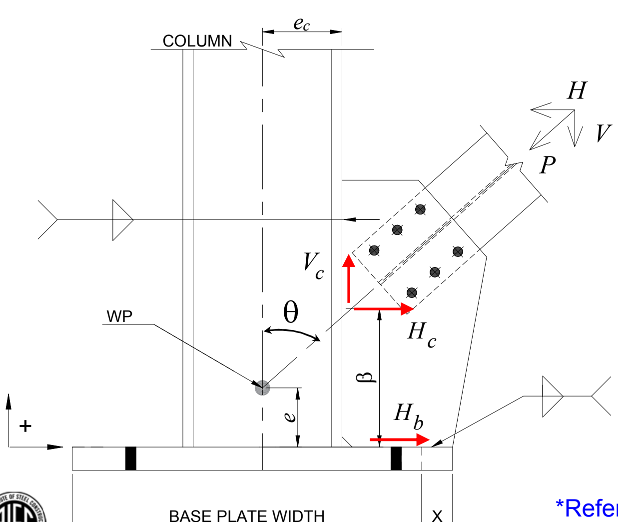
$$H_c = H - H_b$$

*Reference DG29

105

Brace to Base

- General Work Point FBD



*Reference DG29

$$H = P\sin\theta$$

$$V = P\cos\theta$$

$$V_c = V$$

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

$$H_c = H - H_b$$

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

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

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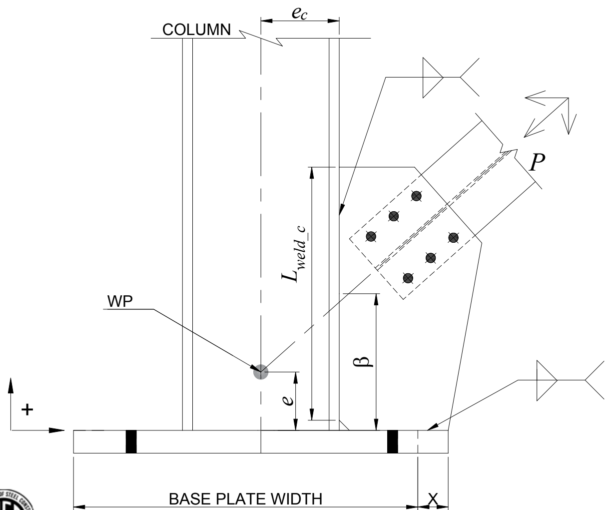
Brace to Base

- General Work Point FBD



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

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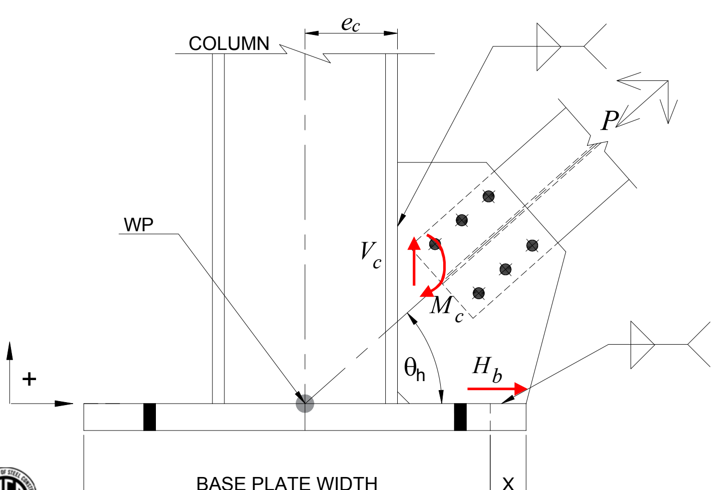
Brace to Base

- Work Point location – Top of Plate



$$V_c = P \sin \theta_h$$

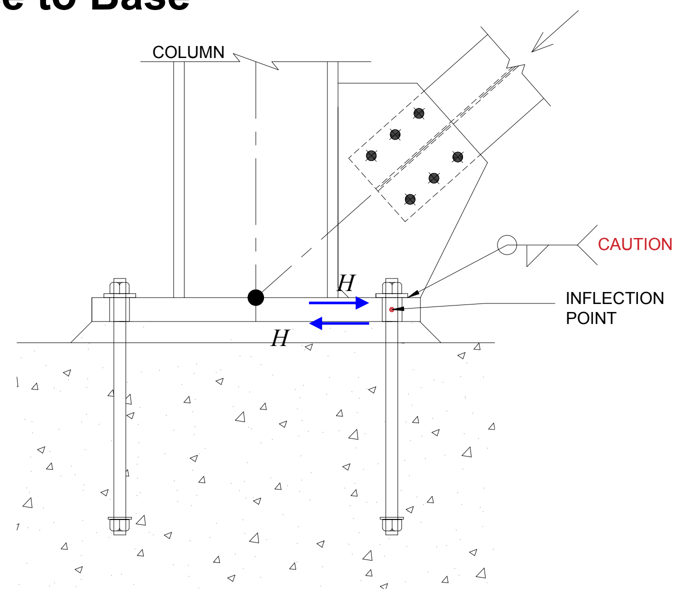
$$M_c = V_c(e_c)$$

$$H_b = P \cos \theta_h$$

108

Brace to Base

- Check anchor rods for shear and bending if washer plate is welded to base plate
- Base plate hole size per 15TH Ed. Manual Table 14-2
- Washer plates per 15TH 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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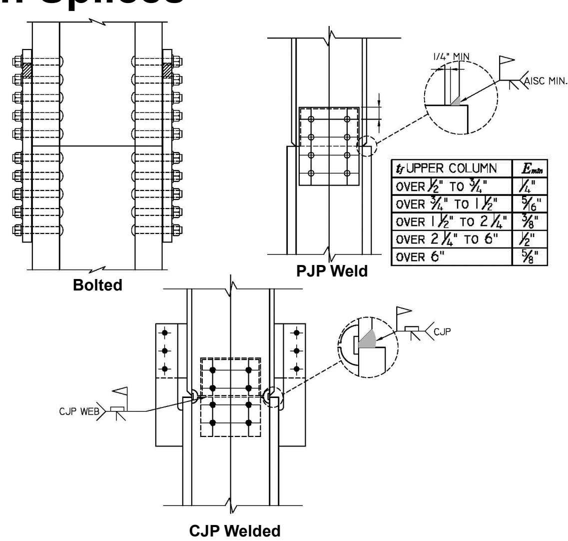
Tension Column Splices

- 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



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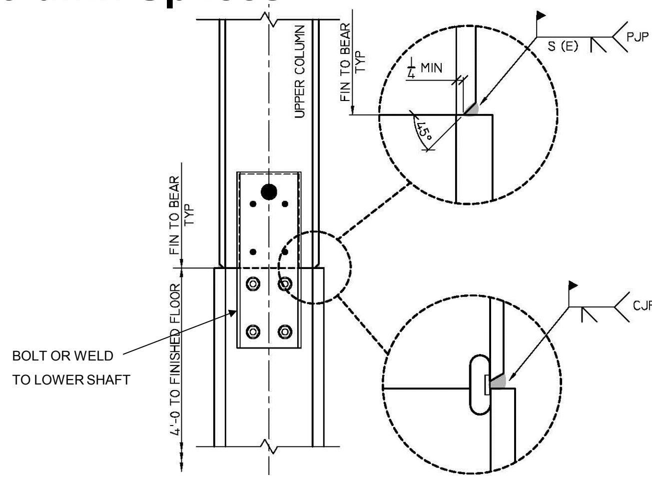
Lateral Column Splices

- CJP verses PJP
  - Testing
  - Gap
- Land = ¼” 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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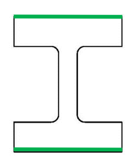
Welded Tension Column Splices



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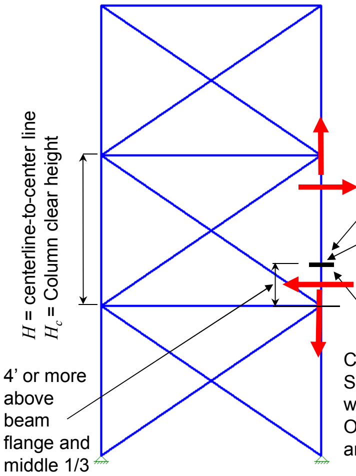
Seismic Splices: AISC Seismic Provisions 341-16 OCBF, SCBF, BRBF (D2.5 and Chapter F)



Web Shear = maximum required per load combinations or

1. $V = M_{pc}/H$ (OCBF)
2. $V = \Sigma M_{pc}/H_c$ (Bolted SCBF/BRBF)
   $F_y(Z_u + Z_l)/(H - d_{bu}/2 - d_{bl}/2 - t_{slab\_l})$

Note: LRFD expressions shown

D2.5b: If tension using overstrength seismic loads:
1. PJP welds develop 200% required strength
2. Must develop $0.5R_yF_yb_ft_f$ of the smaller flange

4' or more above beam flange and middle 1/3

Chapter F:
SCBF/BRBF: CJP or Bolted flanges with min strength = $0.5R_yF_yZ_{x\_min}$
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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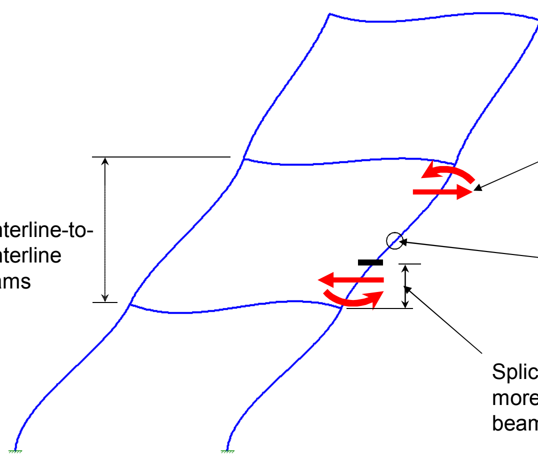
Seismic Splices: 341-16 Column Not Part of the SFRS (D2.5)



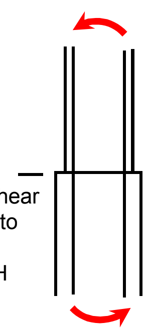
H = Centerline-to-Centerline Beams

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

Inflection point

Splice 4 ft or more above beam flange

Use shear equal to ½ of $2M_{pc}/H$



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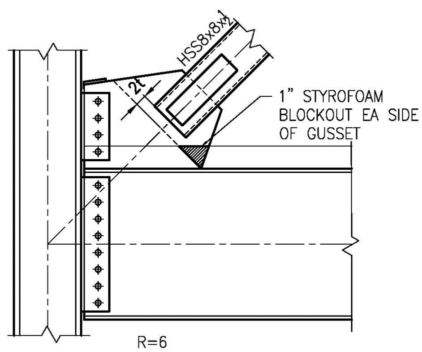
Seismic Bracing



- 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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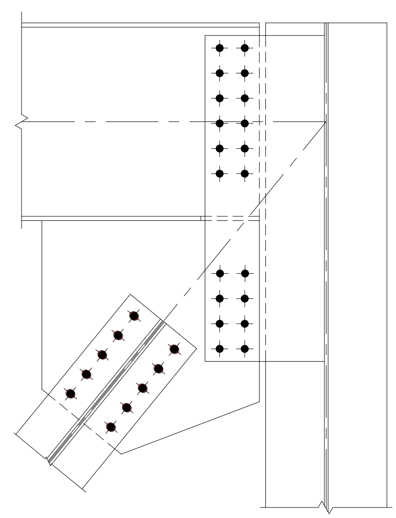
OCBF

- 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, Ω_o.
  - ii. The expected tension strength of the brace, $R_yF_yA_g/\alpha_s$



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OCBF

6. Connections

6a. Brace Connections

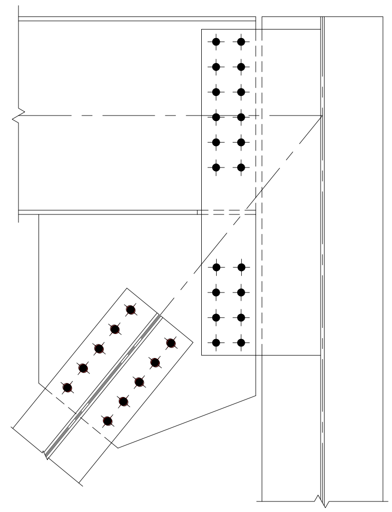
The required strength of diagonal brace connections shall be determined using the overstrength seismic load.

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

(a) In tension, the expected yield strength divided by α_s, which shall be determined as $R_yF_yA_g/\alpha_s$, where α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD.

(b) In compression, the expected brace strength in compression divided by α_s, which is permitted to be taken as the lesser of $R_yF_yA_g/\alpha_s$ and $1.1F_{cre}A_g/\alpha_s$, where F_{cre} is determined from *Specification* Chapter E using the equations for F_{cr}, except that the expected yield stress, R_yF_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.

(c) 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

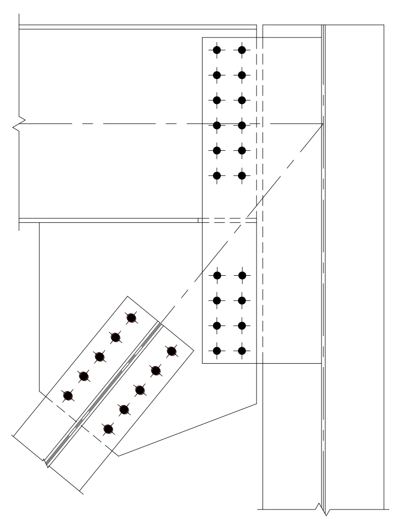
- ASCE 7-16 Section 12.4.3.1 Overstrength load combinations are (LRFD):

Where the seismic load effect with overstrength, $E_m = f(E_v, E_{mh})$, 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:

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



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OCBF

- ASCE 7-16, Table 12.2-1, Overstrength Factor, Ω_o

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, Ω_0^b	Deflection Amplification Factor, C_d^c
B. BUILDING FRAME SYSTEMS				
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	3¼	2	3¼
4. Special reinforced concrete shear walls [g,h]	14.2	6	2½	5
5. Ordinary reinforced concrete shear walls [g]	14.2	5	2½	4½
6. Detailed plain concrete shear walls [g]	14.2 and	2	2½	2

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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 — $design := \text{"ASD"}$
3. Materials:
   a. W-Shaped: $F_{y\_w} := 50\ ksi$ $F_{u\_w} := 65\ ksi$
   b. HSS: $F_{y\_hss} := 46\ ksi$ $F_{u\_hss} := 58\ ksi$
   c. Plate: $F_{y\_pl} := 36\ ksi$ $F_{u\_pl} := 58 \cdot ksi$
   d. Angle: $F_{y\_a} := 36\ ksi$ $F_{u\_a} := 58 \cdot ksi$
4. Bolts:
   a. 3/4"-dia. A325-N bolts $d_{bolt} := \frac{3}{4}\ in$

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Things to Look for in Review

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

$$U_{whit\_c} := \text{abs}\left(\frac{P_c}{\text{※}P_{n\_c}}\right) = 0.627 \qquad \textbf{if}\left(U_{whit\_c} > 1, \text{“n.g.”}, \text{“o.k.”}\right) = \text{“o.k.”}$$

$$U_{whit\_t} := \text{abs}\left(\frac{P_t}{\text{※}P_{n\_t}}\right) = 0.573 \qquad \textbf{if}\left(U_{whit\_t} > 1, \text{“n.g.”}, \text{“o.k.”}\right) = \text{“o.k.”}$$

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CONNECTION IS OK
   (Strength equals or exceeds design loads)
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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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- 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!

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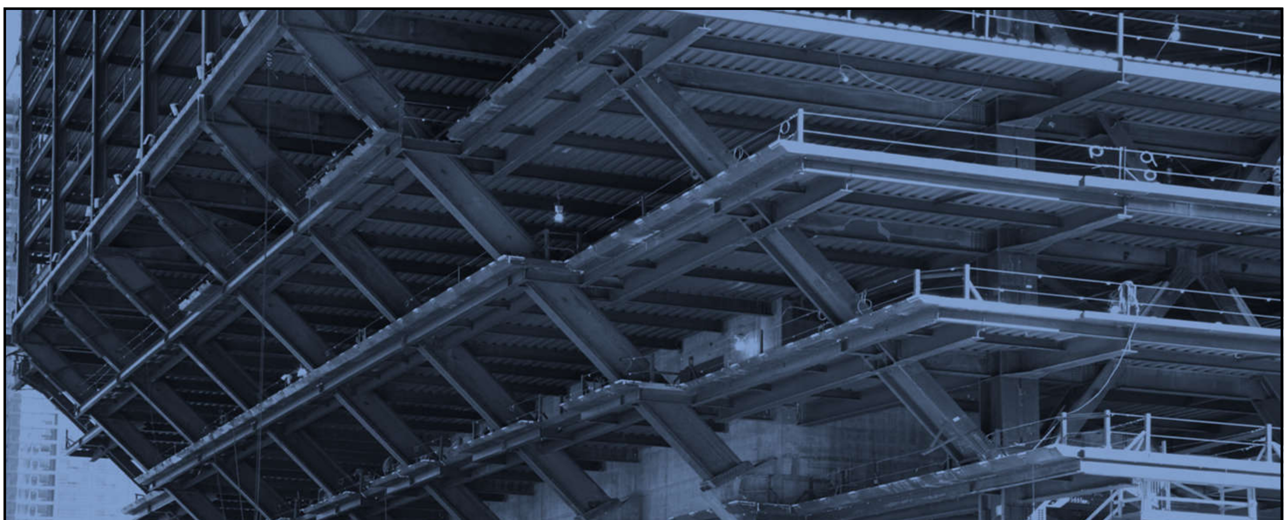

AISC | Thank you

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