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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.



Bracing Success with Delegated Connection Design

June 27, 2019



Carol Drucker, SE, PE, PEng
Principal

dzse
Drucker Zajdel Structural Engineers, Inc.
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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

Advantage

- Rigid
- Typically more cost effective than moment frames

Disadvantage

- May interfere with openings

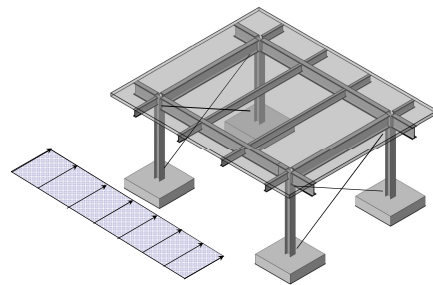


Photo by Jon Miller



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Load Path For Wind

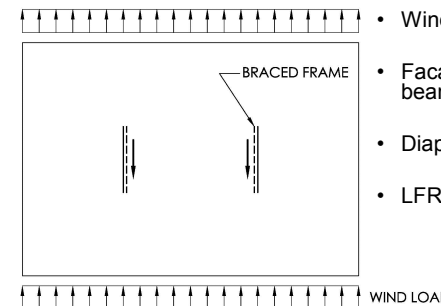


- Wind on facade
- Facade to structure (e.g. beams or diaphragms)
- Diaphragms to lateral force resisting system (LFRS)
- LFRS to foundations



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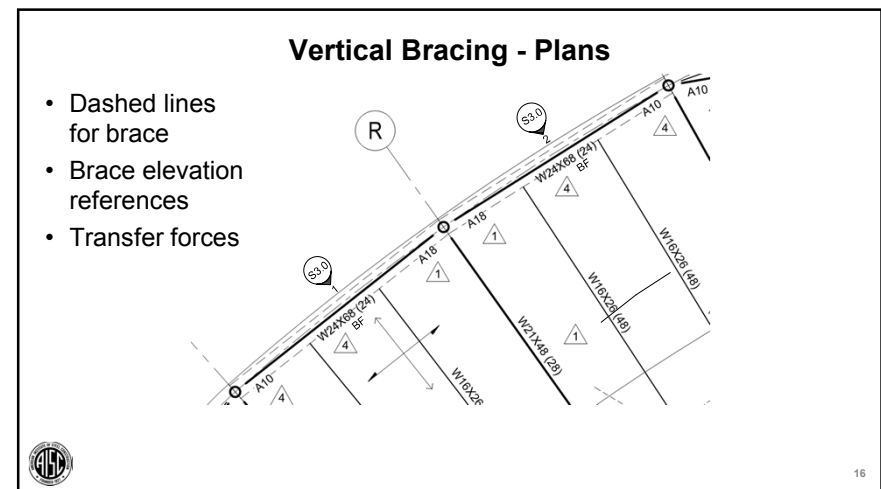
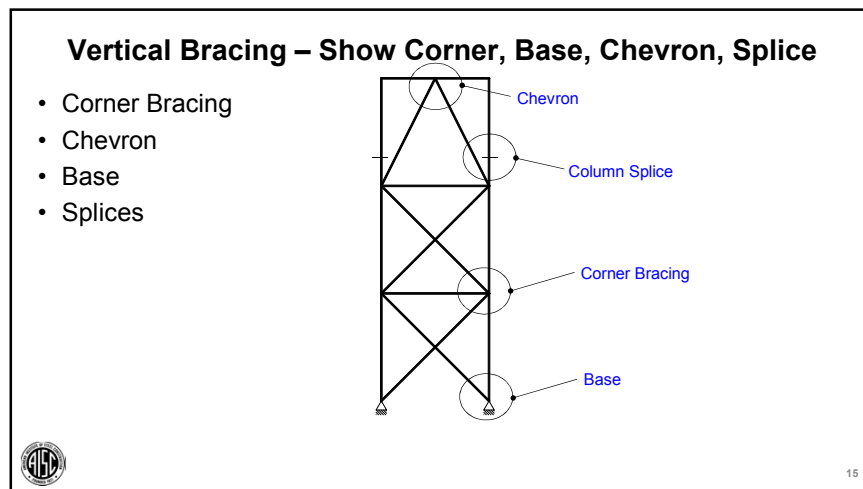
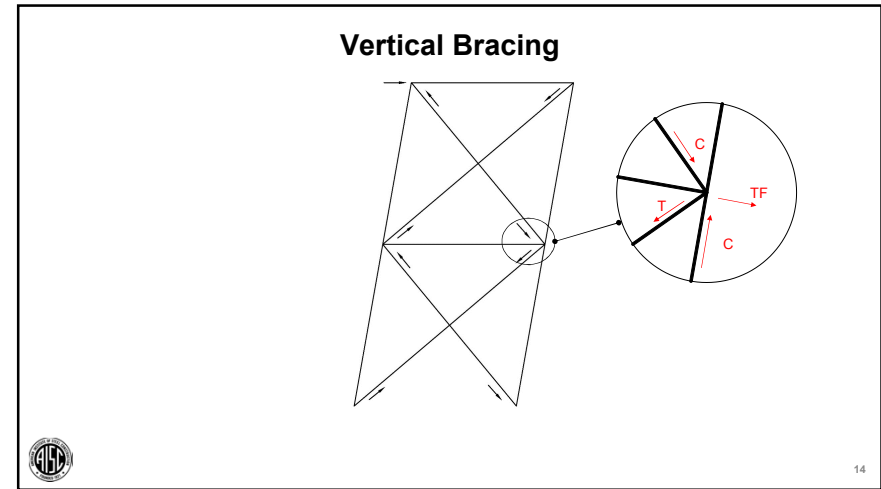
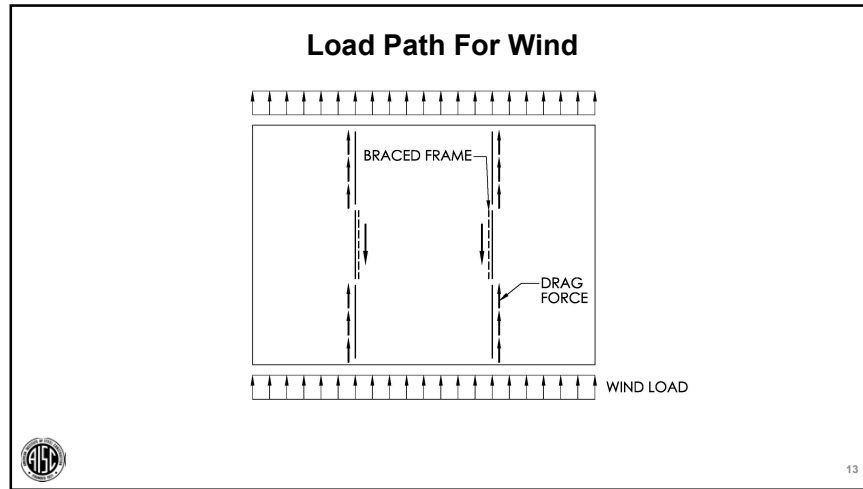
Load Path For Wind



- 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

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Vertical Bracing – Items to Show

- Architectural requirements
 - 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 $P_a = 0.75 (0.6F_y)$ 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

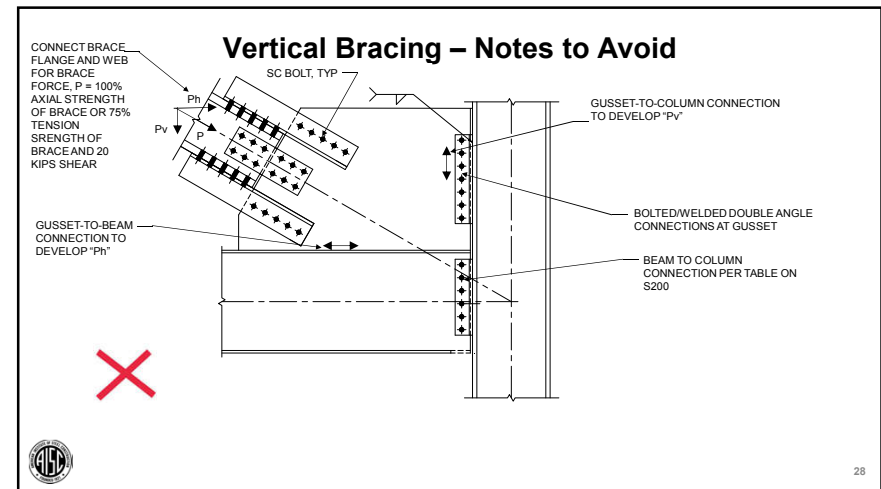
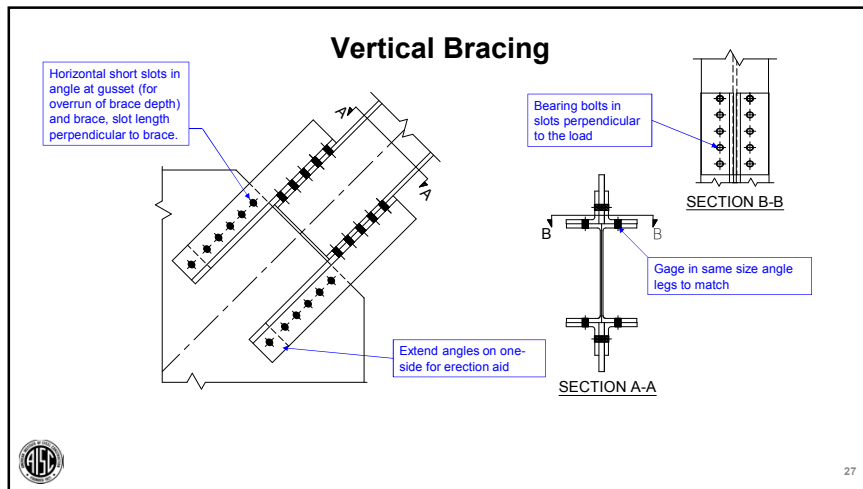
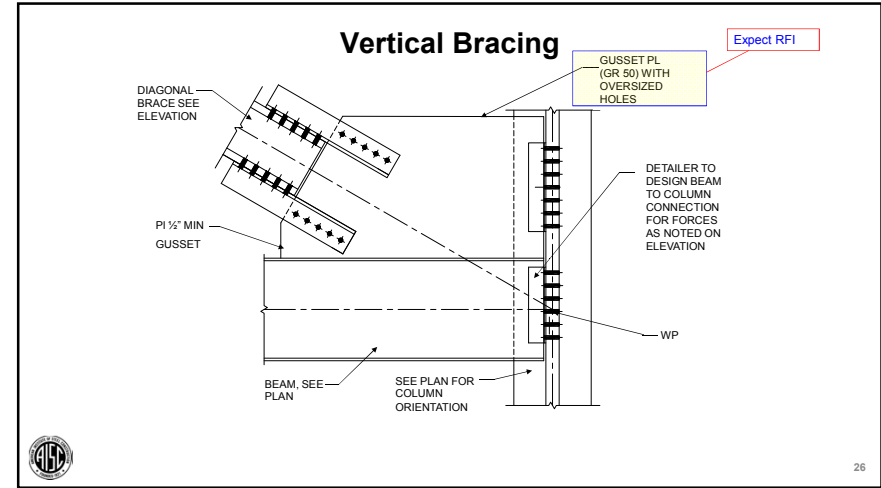
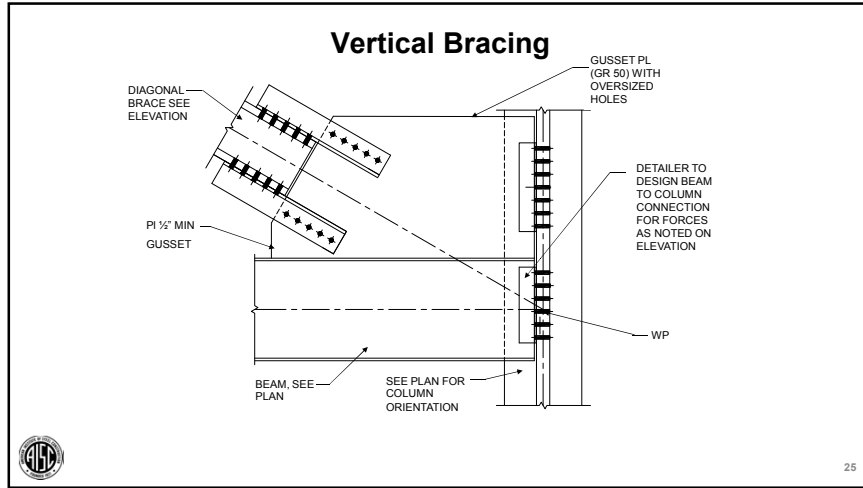
- SC Bolt Requirements given in RCSC Specification
 - 4.3. Slip-Critical Joints

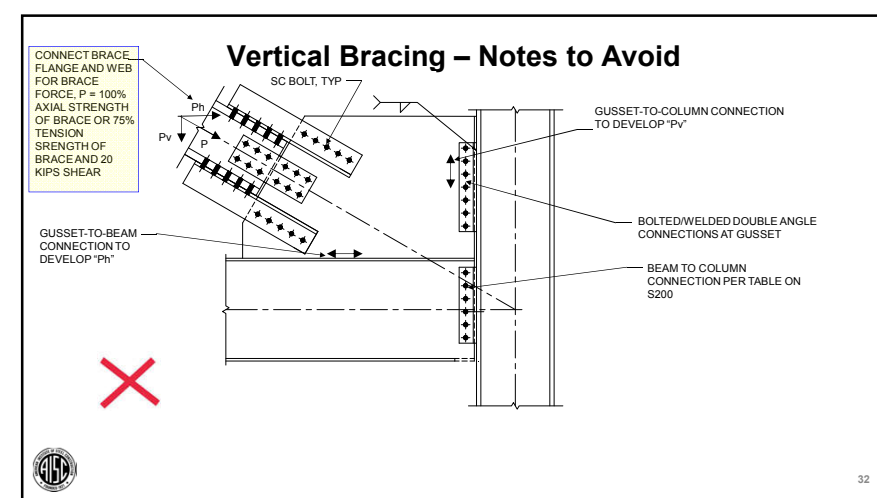
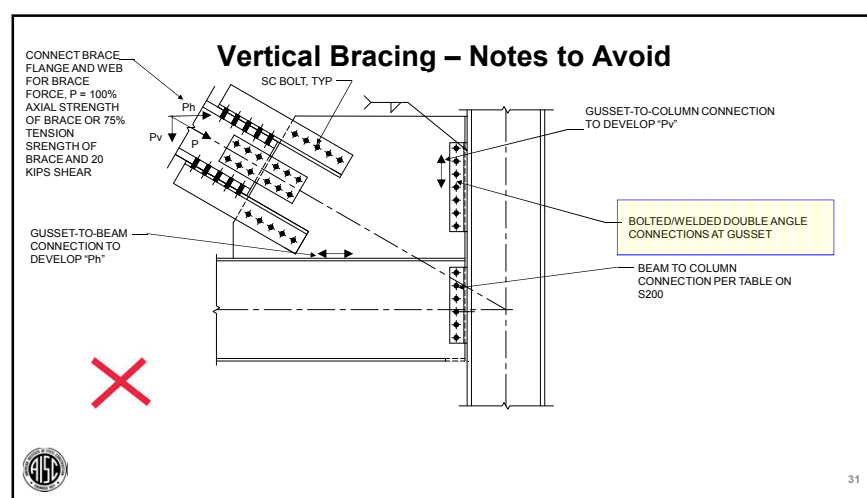
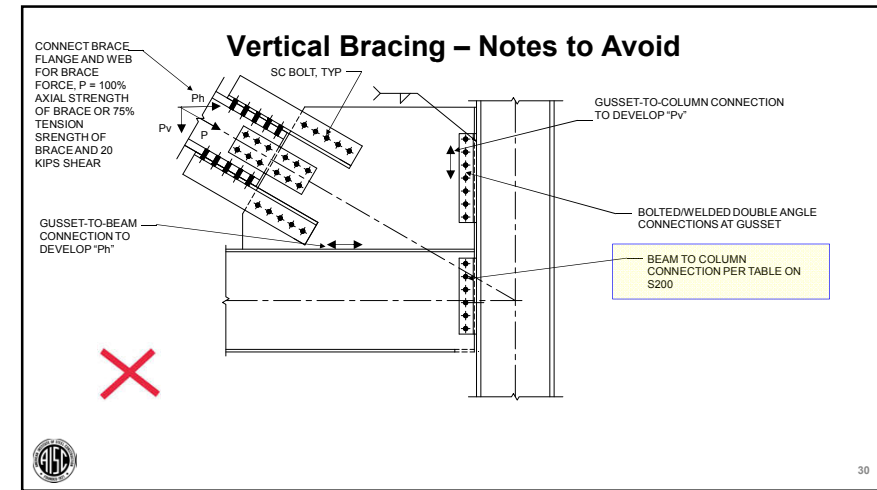
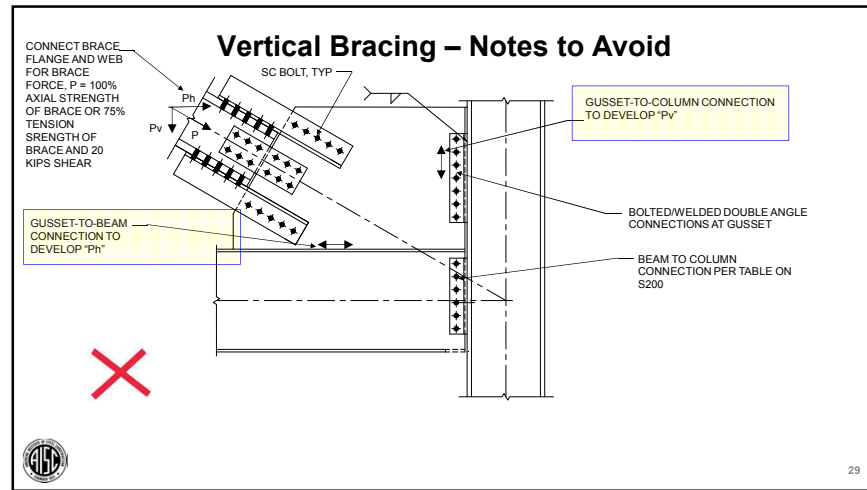
Slip-critical joints are required in the following applications involving shear or combined shear and tension:

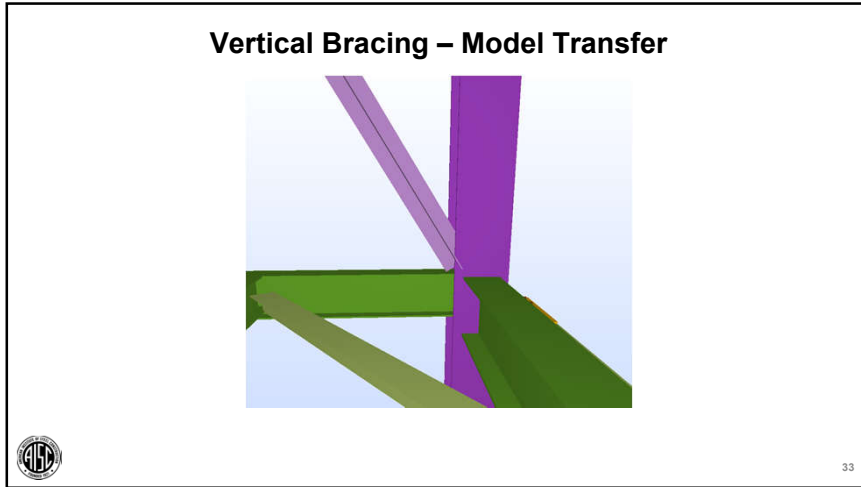
 - Joints* that are subject to fatigue load with reversal of the loading direction;
 - Joints* that utilize oversized holes;
 - Joints* that utilize slotted holes, except those with applied load approximately normal (within 80 to 100 degrees) to the direction of the long dimension of the slot; and,
 - 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

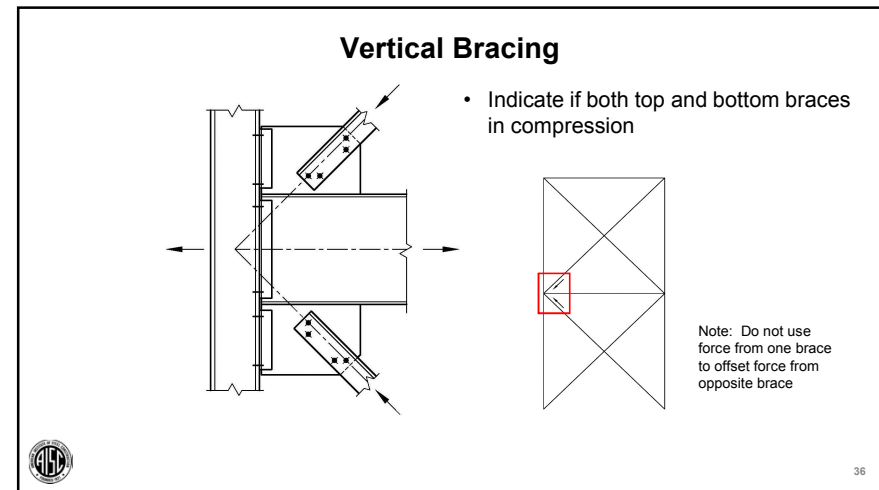
	A	B	C	D	E	F	G	H	I	J
	memid	section	risa_memb_ctype	section_id	start_max_axial_transfer	start_min_axial_transfer	start_max_shear_transfer	start_min_shear_transfer	Start_Max_axial	Start_min_axial
1										
2	Column1	Column2	Column	Column	Column	Column	Column	Column	Column1	Column1
84	ROS_172	LL5x5x8x3	VBrace	LL5x5x8x3					191.201841	-0.44519998
85	ROS_173	LL5x5x8x3	VBrace	LL5x5x8x3					229.613999	-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					232.498238	0
90	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					240.974689	-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	-0.21442049
15	ROS_206	LL5x5x8x3	VBrace	LL5x5x8x3					332.024685	-0.249647989
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					337.186635	-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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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

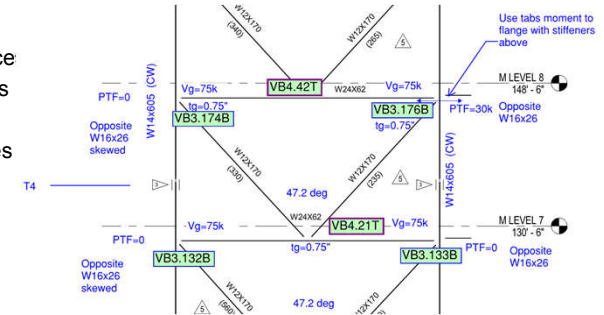
- Forces
- Sizes
- System requirements



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Getting Started with Design

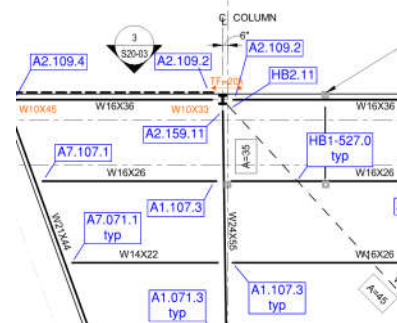
- Corner Bracing
 - Map transfer force
 - Map gravity loads
 - Map beam sizes
 - Map column sizes



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Getting Started with Design

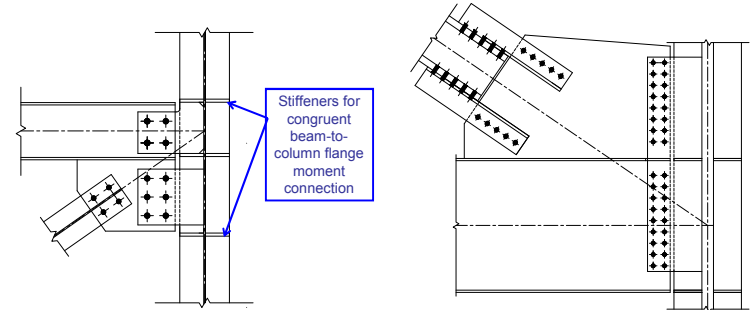
- Corner Bracing
 - Map transfer forces
 - Map gravity loads
 - Map beam sizes
 - Map column sizes



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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 5/4	t_b 47.2	t_b 48.8	t_b 36.2	t_b 5/4	t_b 47.2	t_b 48.8	t_b 36.2
Design	P_n	P_n	P_n	P_n	P_n	P_n	P_n	P_n
	430	430	430	430	430	430	430	430
	345	316	300	450	252	379	229	343
4	333	301	290	435	244	366	221	332
6	319	479	377	417	234	351	213	318
8	300	451	361	395	220	331	200	300
10	277	416	242	362	204	307	185	278
12	252	378	220	331	186	279	169	254
14	234	337	197	296	166	250	151	228
16	197	296	173	260	146	220	133	201
18	179	250	150	225	127	191	116	174
20	146	216	127	191	108	162	96.6	146
22	119	179	106	159	90.1	135	82.5	124
24	100	151	89.0	134	75.7	114	69.3	104
26	86.5	129	75.9	114	64.5	97.0	59.1	89.0
28	73.7	111	65.4	98.3	55.8	83.6	52.9	76.6
30	66.2	96.1	57.0	85.6	48.5	72.9	44.4	66.7
4	345	316	300	450	252	379	229	343
4	319	480	273	410	224	336	199	290
6	304	456	259	380	213	320	189	284
8	283	423	241	363	198	298	176	265
10	251	378	214	322	178	264	156	235
12	222	334	189	284	156	233	138	208
14	192	289	163	244	133	206	119	179
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

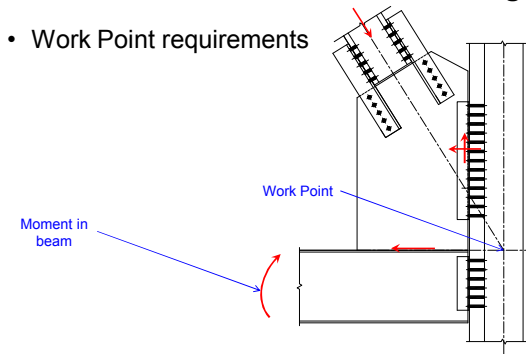
- Seismic Design, $R > 3$ (AISC 341-16, Section D2.2(d))
 - (d) All bolts shall be installed as pretensioned high-strength bolts. Faying surfaces shall satisfy the requirements for slip-critical connections in accordance with *Specification* Section J3.8 with a faying surface with a Class A slip coefficient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

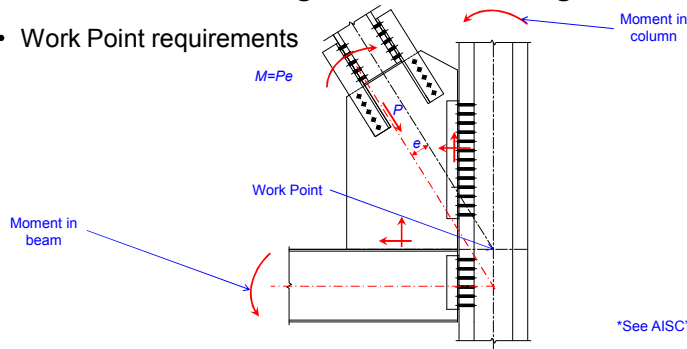
- Work Point requirements



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Getting Started with Design

- Work Point requirements



*See AISC's Design Guide 29

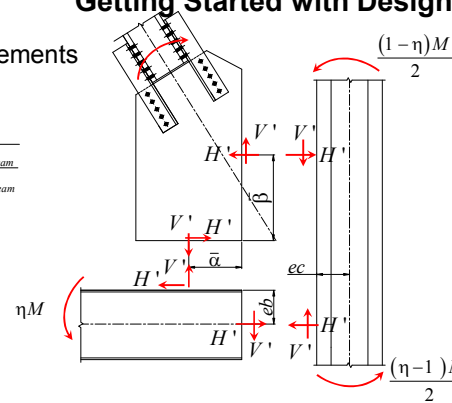


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Getting Started with Design

- WP requirements

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



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

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

*Reference AISC's Night School 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

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 1/2	1 1/4	1 1/4	1
g_1	6	5	4 1/2	4	3 1/2	3	2 1/2	2	1 1/2	1 1/4	1 1/4	1	7/8	3/4	3/4
g_2	3	3	3	2 1/2	2 1/4	2	2	2	1 1/2	1 1/4	1 1/4	1	7/8	3/4	3/4
g_3	2 1/2	2 1/2	3	3	2 1/2	2	2	2	1 1/2	1 1/4	1 1/4	1	7/8	3/4	3/4
g_4	2 1/2	2 1/2	3	3	2 1/2	2	2	2	1 1/2	1 1/4	1 1/4	1	7/8	3/4	3/4

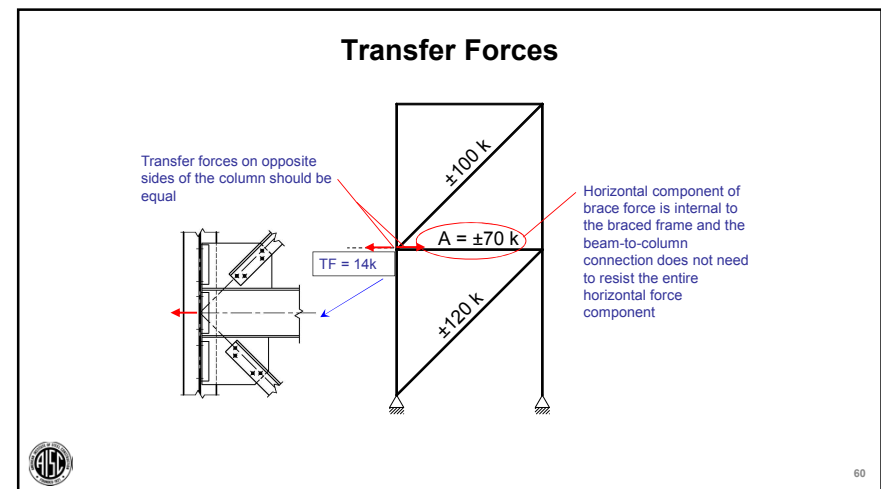
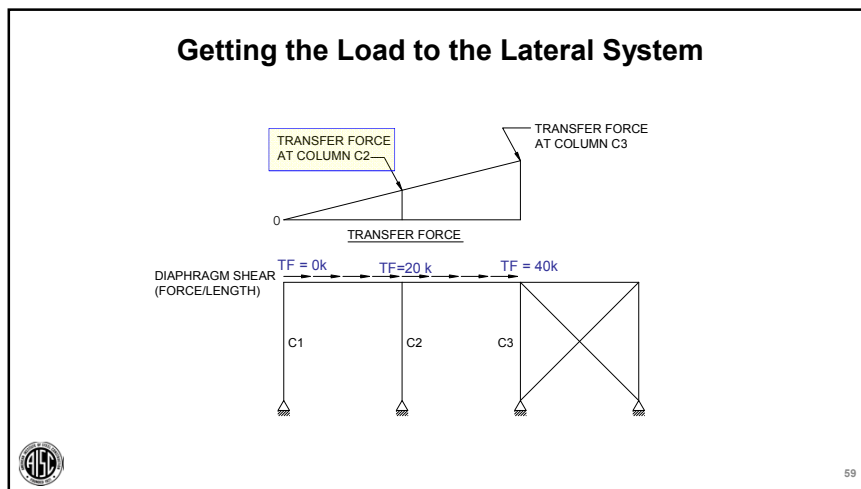
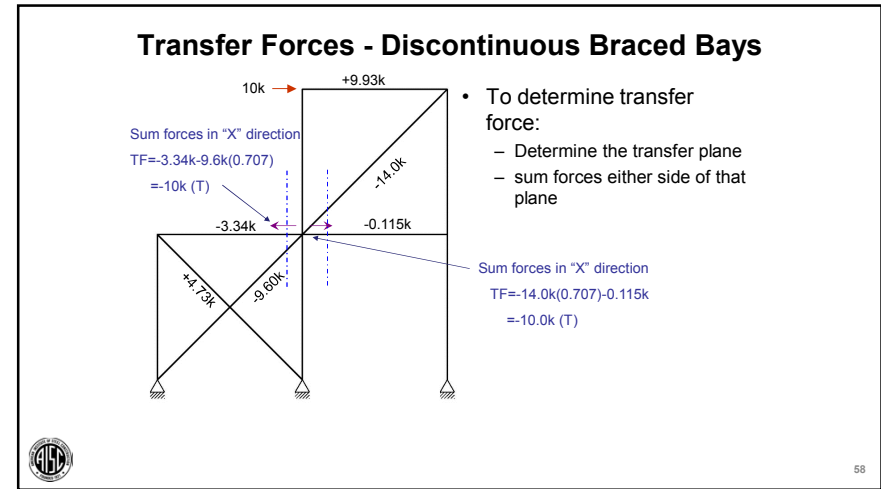
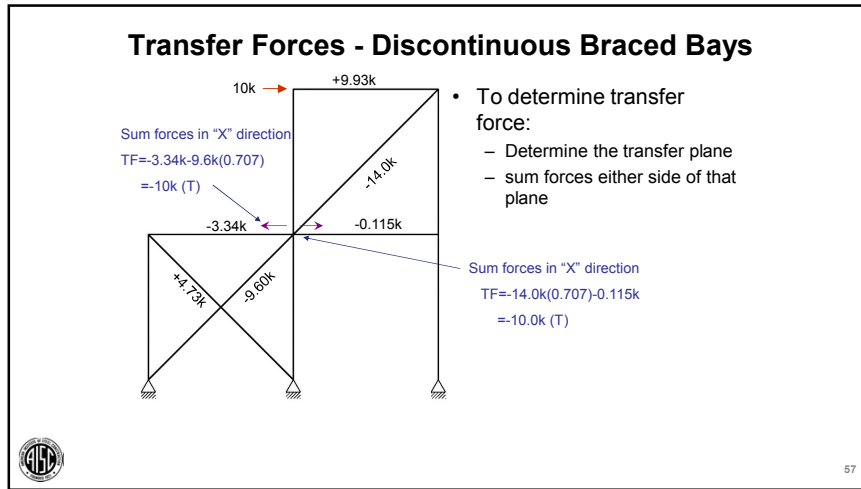
Note: Other gages are permitted to suit specific requirements subject to clearances and edge distance limitations.

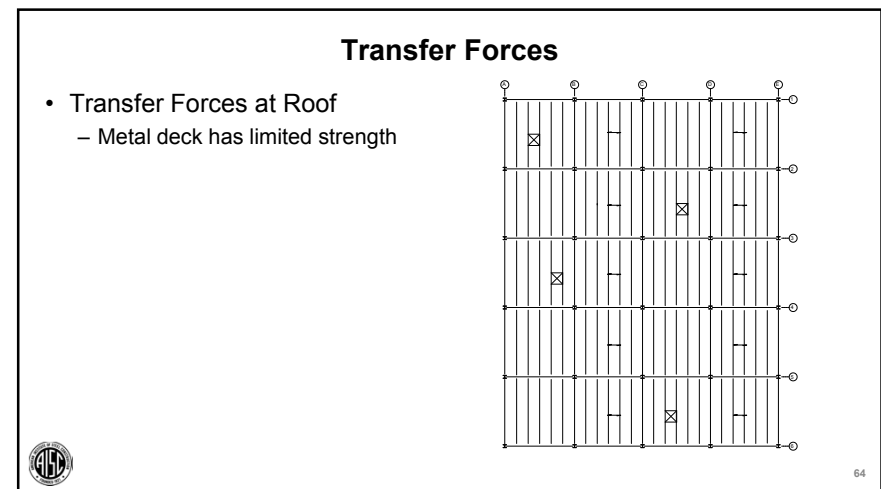
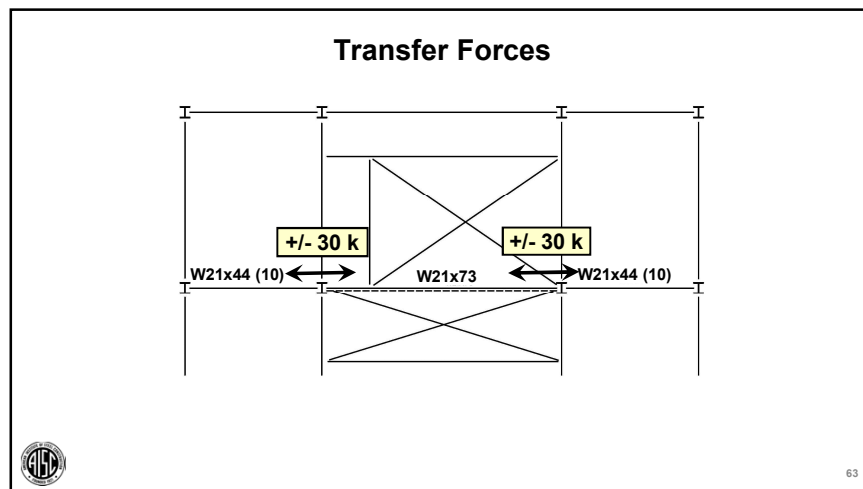
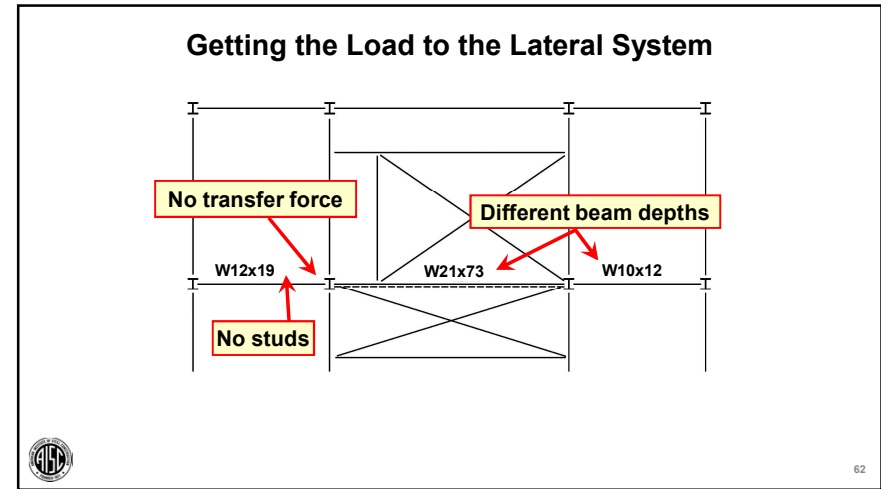
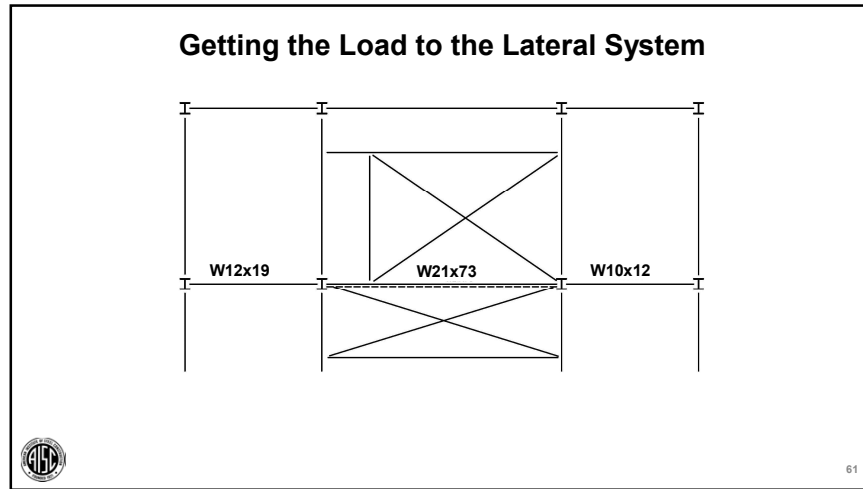
55

Transfer Forces

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

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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

(EQ): 0.55 (EQ): 3.00
 (Wind): 0.70 (Wind): 2.35
 (Other): 0.60

R _n (kips)	Nominal Diaphragm Shear Strength (kips)												
	12'-0"	12'-6"	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"	16'-0"	16'-6"	17'-0"	17'-6"	
365	994	891	821	751	696	653	584	530	472	425	380	379	0.739
364	1347	1239	1147	1066	985	917	857	804	756	711	664	644	0.637
363	1504	1389	1296	1201	1121	1053	984	921	869	821	777	753	0.763
362	1663	1530	1423	1329	1246	1172	1106	1043	982	928	879	795	0.729
361	1791	1654	1552	1452	1364	1288	1214	1150	1092	1035	981	888	0.702
360	1919	1779	1673	1570	1477	1393	1319	1250	1198	1142	1089	994	0.684
359	2039	1907	1798	1691	1598	1498	1419	1348	1282	1223	1168	1072	0.671
358	2149	2016	1896	1787	1688	1588	1510	1442	1373	1311	1253	1157	0.657
357	2259	2118	1997	1887	1786	1684	1605	1532	1461	1396	1336	1237	0.643
356	2364	2215	2092	1981	1879	1785	1699	1619	1546	1478	1417	1320	0.629
355	2470	2311	2187	2075	1972	1877	1781	1703	1628	1557	1490	1390	0.614
354	2576	2407	2281	2168	2063	1966	1869	1782	1703	1628	1557	1457	0.600
353	2683	2504	2376	2262	2155	2056	1959	1872	1793	1714	1638	1537	0.585
352	2791	2601	2471	2356	2247	2146	2049	1962	1883	1804	1724	1623	0.571
351	2900	2700	2568	2452	2341	2238	2141	2054	1975	1896	1815	1714	0.557
350	3010	2800	2666	2549	2437	2333	2236	2149	2070	1991	1910	1809	0.543
349	3121	2901	2766	2649	2536	2431	2334	2247	2168	2089	2008	1907	0.529
348	3233	3003	2867	2749	2635	2529	2432	2345	2266	2187	2106	2005	0.515
347	3346	3116	2979	2860	2745	2638	2541	2454	2375	2296	2215	2114	0.501
346	3460	3230	3092	2972	2856	2748	2651	2564	2485	2406	2325	2224	0.487
345	3575	3345	3206	3085	2968	2859	2762	2675	2596	2517	2436	2335	0.473
344	3691	3461	3321	3200	3082	2972	2875	2788	2709	2630	2549	2448	0.459
343	3808	3578	3437	3315	3196	3085	2988	2901	2822	2743	2662	2561	0.445
342	3926	3696	3554	3432	3312	3201	3104	3017	2938	2859	2778	2677	0.431
341	4045	3815	3672	3549	3428	3316	3219	3132	3053	2974	2893	2792	0.417
340	4165	3935	3791	3667	3545	3432	3335	3248	3169	3090	3009	2908	0.403
339	4286	4056	3911	3786	3663	3549	3452	3365	3286	3207	3126	3025	0.389
338	4408	4178	4032	3906	3781	3666	3569	3482	3403	3324	3243	3142	0.375
337	4531	4301	4154	4028	3902	3786	3689	3602	3523	3444	3363	3262	0.361
336	4655	4425	4277	4151	4024	3907	3810	3723	3644	3565	3484	3383	0.347
335	4780	4550	4401	4274	4146	4029	3932	3845	3766	3687	3606	3505	0.333
334	4906	4676	4526	4398	4269	4151	4054	3967	3888	3809	3728	3627	0.319
333	5033	4803	4652	4523	4393	4274	4177	4090	4011	3932	3851	3750	0.305
332	5161	4931	4779	4648	4517	4397	4300	4213	4134	4055	3974	3873	0.291
331	5290	5060	4907	4775	4643	4522	4425	4338	4259	4180	4100	4000	0.277
330	5420	5190	5036	4903	4770	4648	4551	4464	4385	4306	4225	4125	0.263
329	5551	5321	5166	5032	4898	4775	4678	4591	4512	4433	4352	4252	0.249
328	5683	5453	5297	5162	5027	4903	4806	4719	4640	4561	4480	4380	0.235
327	5816	5586	5429	5292	5156	5031	4934	4847	4768	4689	4608	4508	0.221
326	5950	5720	5562	5424	5287	5161	5064	4977	4898	4819	4738	4638	0.207
325	6085	5855	5696	5557	5419	5292	5195	5108	5029	4950	4869	4769	0.193
324	6221	5991	5831	5691	5552	5424	5327	5240	5161	5082	4999	4900	0.179
323	6358	6128	5967	5826	5686	5557	5460	5373	5294	5215	5134	5035	0.165
322	6496	6266	6104	5962	5821	5691	5594	5507	5428	5349	5268	5169	0.151
321	6635	6405	6242	6100	5958	5827	5730	5643	5564	5485	5404	5305	0.137
320	6775	6545	6381	6238	6095	5963	5866	5779	5700	5621	5540	5441	0.123
319	6916	6686	6521	6377	6233	6100	6003	5916	5837	5758	5677	5578	0.109
318	7058	6828	6662	6517	6372	6238	6141	6054	5975	5896	5815	5716	0.095
317	7201	6971	6804	6658	6512	6377	6280	6193	6114	6035	5954	5855	0.081
316	7345	7115	6947	6800	6653	6517	6420	6333	6254	6175	6094	5995	0.067
315	7490	7260	7091	6943	6795	6658	6561	6474	6395	6316	6235	6136	0.053
314	7636	7406	7236	7087	6938	6800	6703	6616	6537	6458	6377	6278	0.039
313	7783	7553	7382	7232	7082	6943	6846	6759	6680	6601	6520	6421	0.025
312	7931	7701	7529	7378	7227	7087	6990	6903	6824	6745	6664	6565	0.011
311	8080	7850	7677	7525	7373	7232	7135	7048	6969	6890	6809	6710	0.000

NEW MILLENNIUM BUILDING SYSTEMS

Diaphragm Strength

- Transfer Forces at Composite Deck
– Consider topping slab

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

$$\phi = 0.75$$

Diaphragm Strength

- Transfer Forces at Composite Deck

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

$$\phi = 0.75$$

$$\Sigma TF = \max(\text{abs}(\Sigma F_{i2} - \Sigma F_{i1}) - \phi V_n, 0 \text{ kips})$$

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

Diaphragm Strength

- Use Table 3-21 for Stud Strength

Table 3-21
Shear Stud Anchor
Nominal Horizontal Shear Strength
for One Steel Headed Stud Anchor, Q_n , kips

$F_u = 65 \text{ ksi}$

Deck Condition	Stud Anchor Diameter, in.	Normal Weight Concrete		Lightweight Concrete	
		$w_c = 145 \text{ pcf}$		$w_c = 110 \text{ pcf}$	
		$f'_c = 3 \text{ ksi}$	$f'_c = 4 \text{ ksi}$	$f'_c = 3 \text{ ksi}$	$f'_c = 4 \text{ ksi}$
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
2	3/4	17.2	17.2	17.1	17.2
	7/8	3.66	3.66	3.66	3.66
	1	6.51	6.51	6.51	6.51
3	1 1/8	10.2	10.2	10.2	10.2
	1 1/4	14.6	14.6	14.6	14.6

Deck Perpendicular Weak Studs per rib ($R_p = 0.60$)

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 ^(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)

(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)
^(b) 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

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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

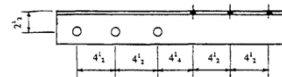
72

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

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

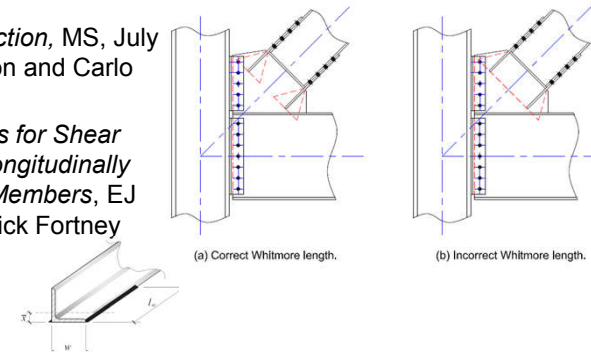
^a Table 3 from Dowswell (2006) with revisions.
^b Yielding is the applicable limit state for compact corner gusset plates; therefore, the effective length factor and the buckling length are not applicable.



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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



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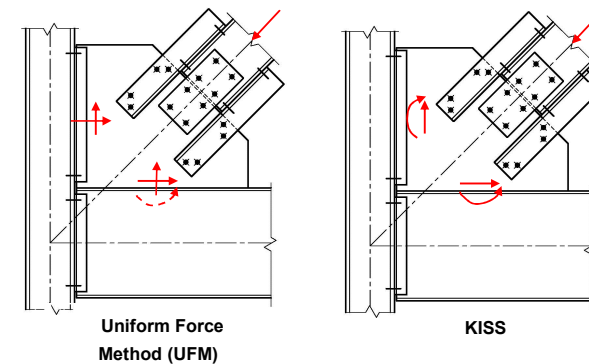
Vertical Bracing – Articles

- *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



76

Connections-Bracing UFM

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

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

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

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

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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

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

Work line = beam line and solve for x

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

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

79

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

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

80

Connections-Bracing UFM

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

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

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

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

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

81

Connections-Bracing UFM

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

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

82

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 $\bar{\alpha} = \alpha$

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.

83

Connections-Bracing UFM

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

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

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

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

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

84

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

85

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

86

Connections-Bracing KISS

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

$$M_b = H_b(e_b)$$

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

$$M_c = V_c(e_c)$$

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

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

KISS

87

Connections-Bracing UFM

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

$$M_b = H_b(e_b)$$

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

$$M_c = V_c(e_c)$$

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

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

- 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

89

Connections-Bracing

UFM Advantage **Possible KISS Advantage**

90

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 $e_c = e$

$$H_c = H_c + F$$

$$M_b = M_b + M_F$$

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

$$H_b = H_b + F$$

93

Details for High Axial Load

94

Details for High Axial Load and/or Moment

95

Chevron Connections

- Stability bracing
- Brace force directions

96

Chevron Connections

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

W-BEAM

97

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

W-BEAM

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Chevron Connections

- Check for doublers

$P_1 \neq P_2$

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

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

Control Test

900k Machine

Test Set-up from Chen and Oppenheim 1970

101

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{y} to determine H_b

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

$$H_c = H - H_b$$

*Reference DG29

104

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

105

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

106

Brace to Base

- General Work Point FBD

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

107

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

CAUTION
INFLECTION POINT

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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 3/4" TO 1 1/2"	1 1/2"
OVER 1 1/2" TO 2 1/2"	2"
OVER 2 1/2" TO 6"	2 1/2"
OVER 6"	3"

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Lateral Column Splices

- CJP verses PJP
 - Testing
 - Gap
- Land = 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)
- $V = \sum M_{pr}/H_c$ (Bolted SCBF/BRBF)
 $F_y(Z_e + Z_c)(H - d_{col}/2 - d_{col}/2 - t_{stab,c})$

Note: LFRD expressions shown

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

- PJP welds develop 200% required strength
- Must develop $0.5R_y F_y h_{tf}$ of the smaller flange

Chapter F:
SCBF/BRBF: CJP or Bolted flanges with min strength = $0.5R_y F_y Z_{e,beam}$
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

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, Ω_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

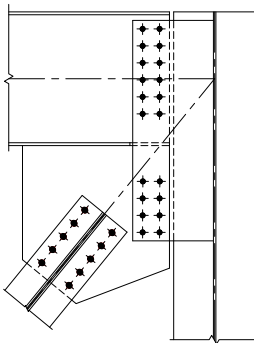
The required strength of diagonal brace connections shall be determined using the overstrength seismic load.


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

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

(b) In compression, the expected brace strength in compression divided by α_s , which is permitted to be taken as the lesser of $R_y F_y A_g / \alpha_s$ and $1.1 F_{cr} A_g / \alpha_s$, where F_{cr} 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_{cr} 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, $E_{mh} = f(E_v, E_{mh})$, defined in Section 12.4.3, is combined with the effects of other loads, the following seismic load combination for structures shall be used:

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

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, Ω_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, 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	3/4	2	3/4
4. Special reinforced concrete shear walls ^{a,h}	14.2	6	2 1/2	5
5. Ordinary reinforced concrete shear walls ^d	14.2	5	2 1/2	4 1/2
6. Detailed plain concrete shear walls ^d	14.2 and 14.2.2.2	2	2 1/2	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:

- References: AISC Specification for Structural Steel Building, AISC 360-10
AISC Manual 14th Edition
- Design Basis: ASD
- Materials:

a. W-Shaped:	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$
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:	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$
- Bolts:

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

• Notes:

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

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

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

CONNECTION IS OK
(Strength equals or exceeds design loads)



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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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AISC | Questions?



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- 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!



CEU / PDH Certificates

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

