Blackwell
Structural Engineers

PLAN REVIEW ACCEPTANCE

FOR COMPLIANCE WITH THE APPLICABLE CONSTRUCTION CODES IDENTIFIED BELOW.

☐ BUILDING	☒ STRUCTURAL
☐ MECHANICAL	☐ PLUMBING
☐ ELECTRICAL	☐ ENERGY
☐ ACCESSIBILITY	☐ FIRE

PLAN REVIEW ACCEPTANCE OF DOCUMENTS DOES NOT AUTHORIZE CONSTRUCTION TO PROCEED IN VIOLATION OF ANY FEDERAL, STATE, OR LOCAL REGULATIONS.

BY: MEM DATE: 03/22/18

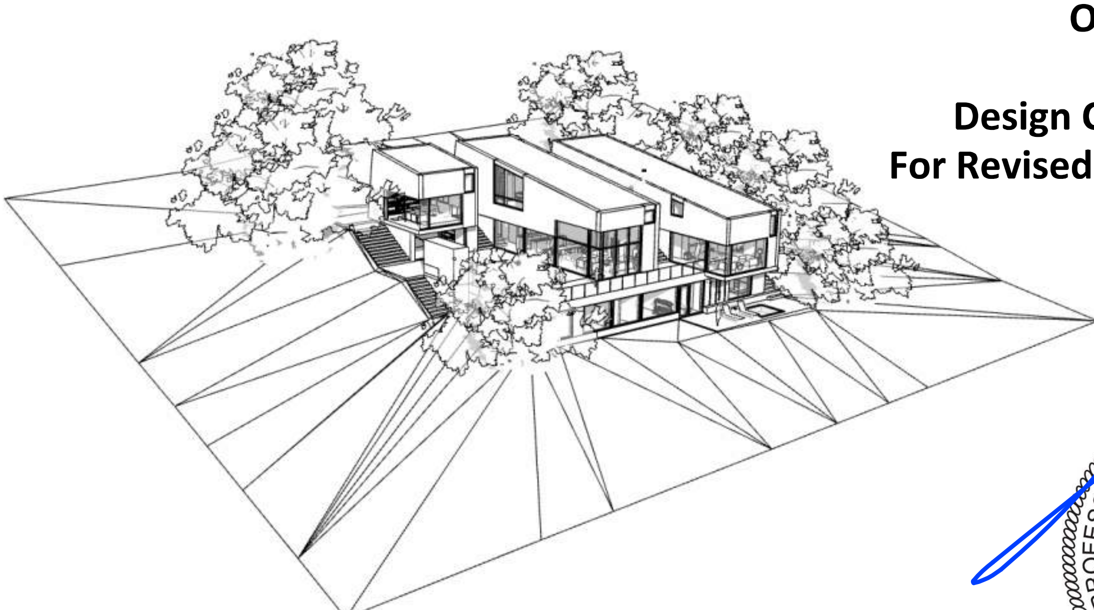
WEST COAST CODE CONSULTANTS, INC.

KIMMELMAN MAY RESIDENCE

Our Project - 170266

Design Calculation Package For Revised Permit Submission

February 15, 2018



PROFESSIONAL STRUCTURAL ENGINEER
No. 6985627-2203
MICHAEL F SULLAWAY
STATE OF UTAH

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MATERIAL DEFINITION

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm (\1E5 F)	Density[k/ft^3]	Yield[ksi]	Ry	Fu[ksi]	Rt
1	A992	29000	11154	.3	.65	.49	50	1.1	65	1.1
2	A36 Gr.36	29000	11154	.3	.65	.49	36	1.5	58	1.2
3	A572 Gr.50	29000	11154	.3	.65	.49	50	1.1	65	1.1
4	A500 Gr.B RND	29000	11154	.3	.65	.527	42	1.4	58	1.3
5	A500 Gr.B Rect	29000	11154	.3	.65	.527	46	1.4	58	1.3
6	A53 Gr.B	29000	11154	.3	.65	.49	35	1.6	60	1.2
7	A1085	29000	11154	.3	.65	.49	50	1.4	65	1.3

Wood Material Properties

	Label	Type	Database	Species	Grade	Cm	Emod	Nu	Ther...	Dens[k...
1	Spruce-Pine-Fir	Solid Sawn	Visually Graded	Spruce-Pine-fir	No.2		1	.3	.3	.026
2	2.0E Microllam LVL	Custom	N/A	2.0E Microllam LVL	na		1	.3	.3	.035

Custom Wood Properties

	Label	Fb [ksi]	Ft [ksi]	Fv [ksi]	Fc [ksi]	E [ksi]	E05 [ksi]	Type	CF
1	LVL_PRL_1.5E_2250F	2.25	1.5	.22	1.95	1500	1005	SCL	
2	LVL_PRL_2.0E_2900F	2.9	1.9	.285	2.75	2000	1340	SCL	
3	LVL_Microllam_1.9E_2600F	2.6	1.555	.285	2.51	1900	1273	SCL	
4	PSL_Parallam_2.0E_2900F	2.9	2.025	.29	2.9	2000	1340	SCL	
5	PSL_Parallam_1.8E	2.4	1.755	.18	2.5	1800	1206	SCL	
6	LSL_TimberStrand_1.55E_2325F	2.325	1.07	.31	2.05	1550	1038.5	SCL	
7	LSL_TimberStrand_1.3E_1700F	1.7	1.075	.4	1.4	1300	871	SCL	
8	2.0E Microllam LVL	2.6	1.555	.285	2.51	2000	184.6	SCL	0

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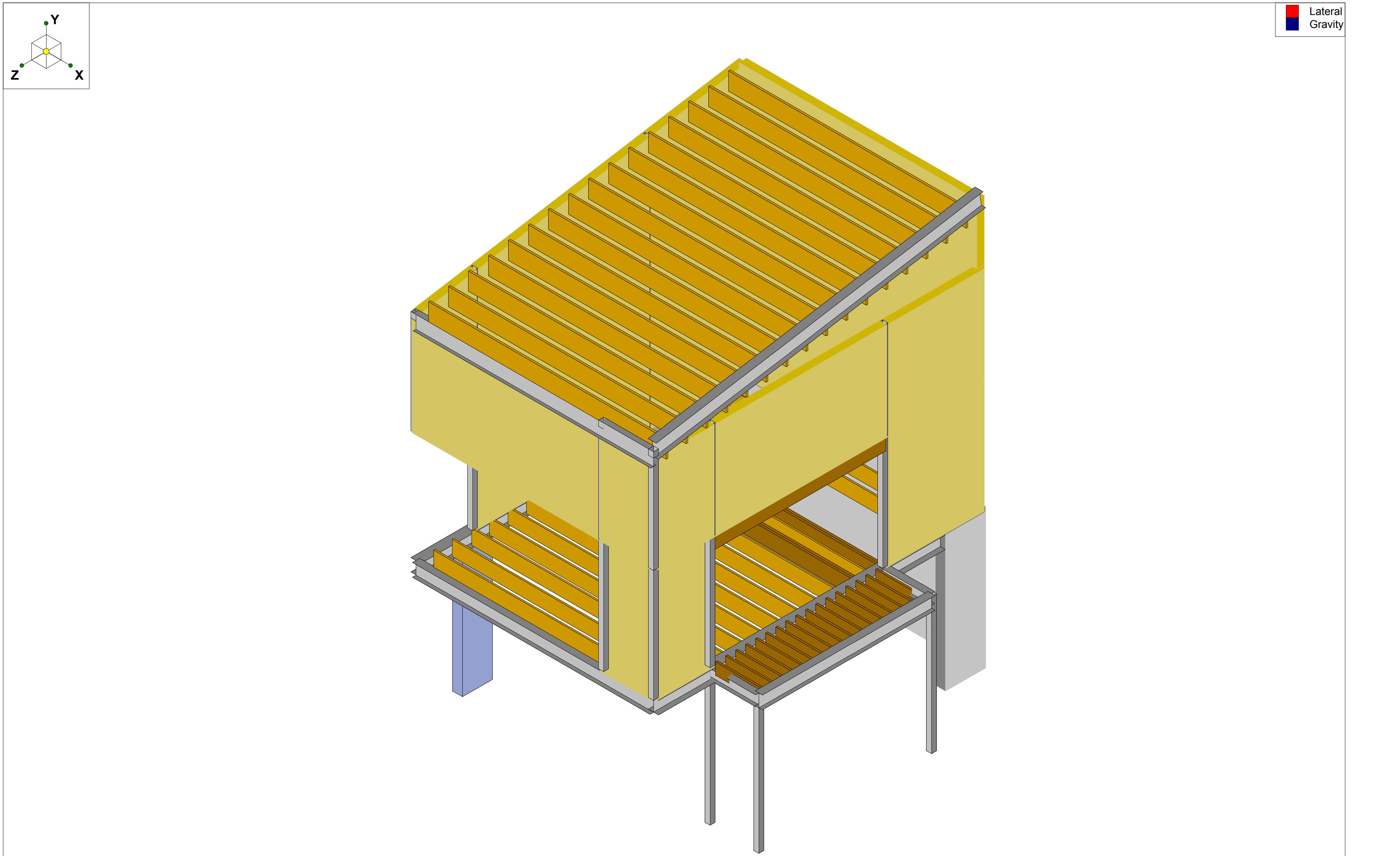
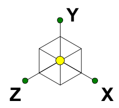
VOLUME 1 (Library)



*RENDERED VIEW SHOWN FOR CONTEXT ONLY. REFER TO MEMBER PROPERTIES AND STRUCTURAL DRAWINGS FOR DETAILS.

Blackwell Structural Engineers	Full Model	GENERAL 3D RENDER
BG	Kimmelman May Residence Volume 1	
170266		



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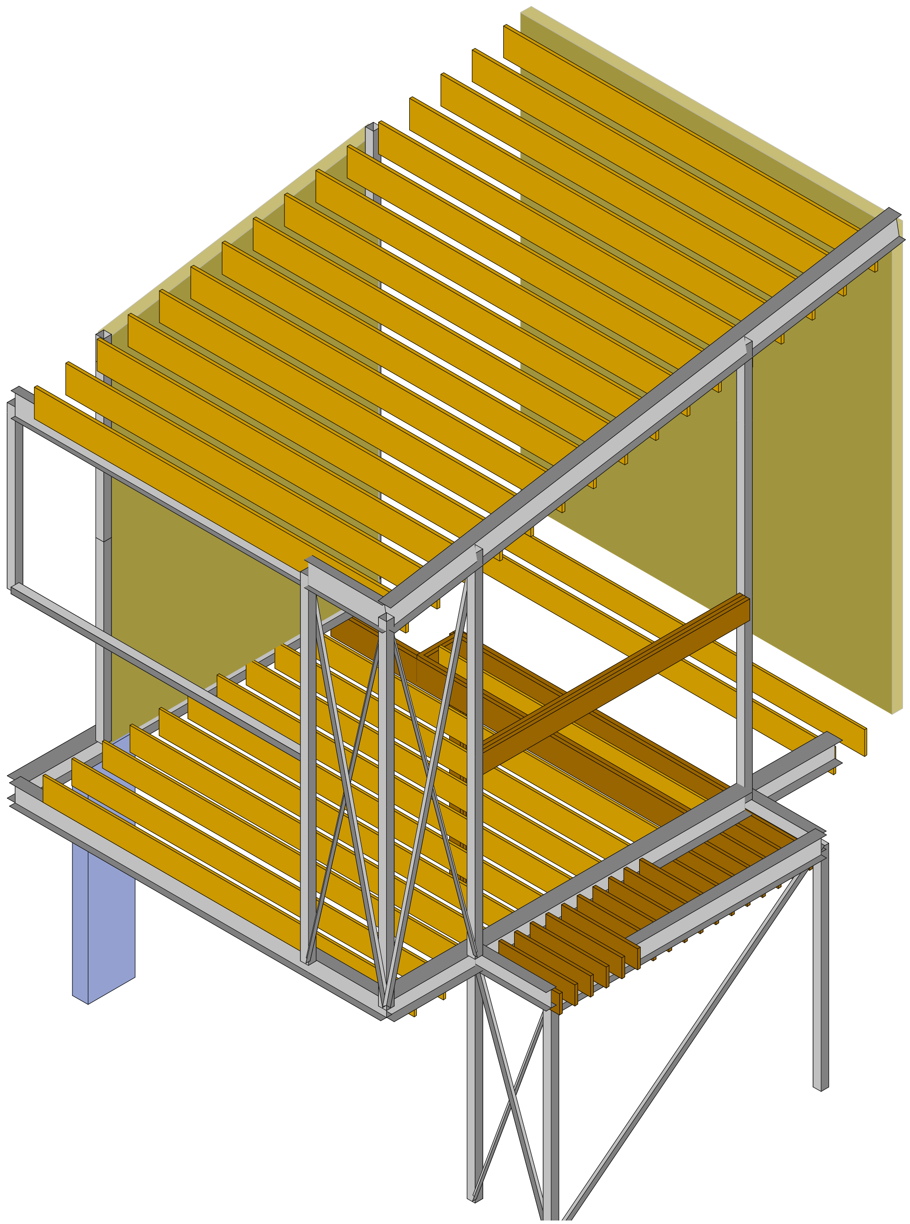

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Blackwell Structural Engineers		GENERAL 3D RENDER
BG	Kimmelman May Residence Volume 1	
170266	Shows bracing. Some walls not shown for clarity.	

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GRAVITY SYSTEM
Designed using RISAFloor

Gravity Geometry and Shapes Definition

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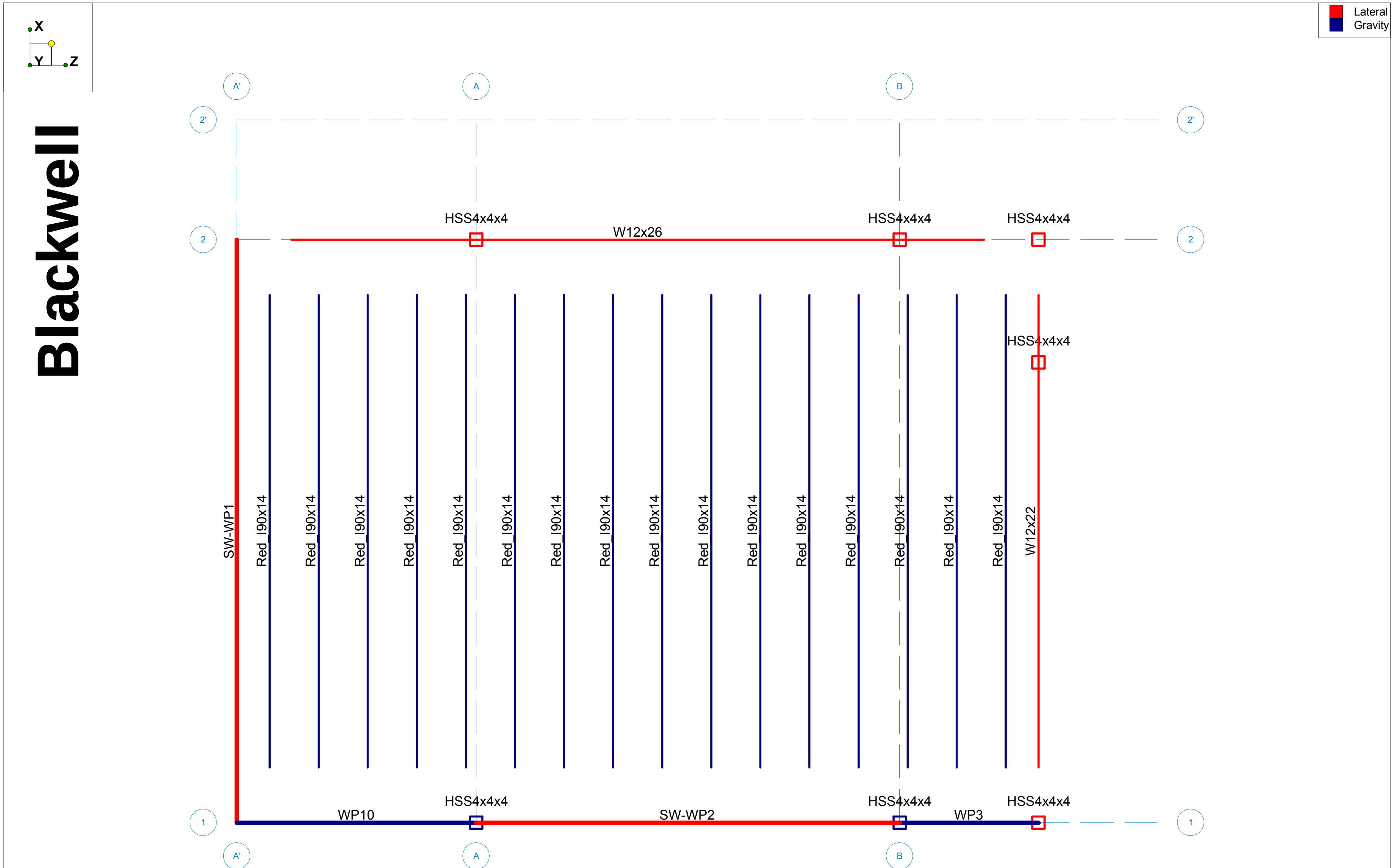
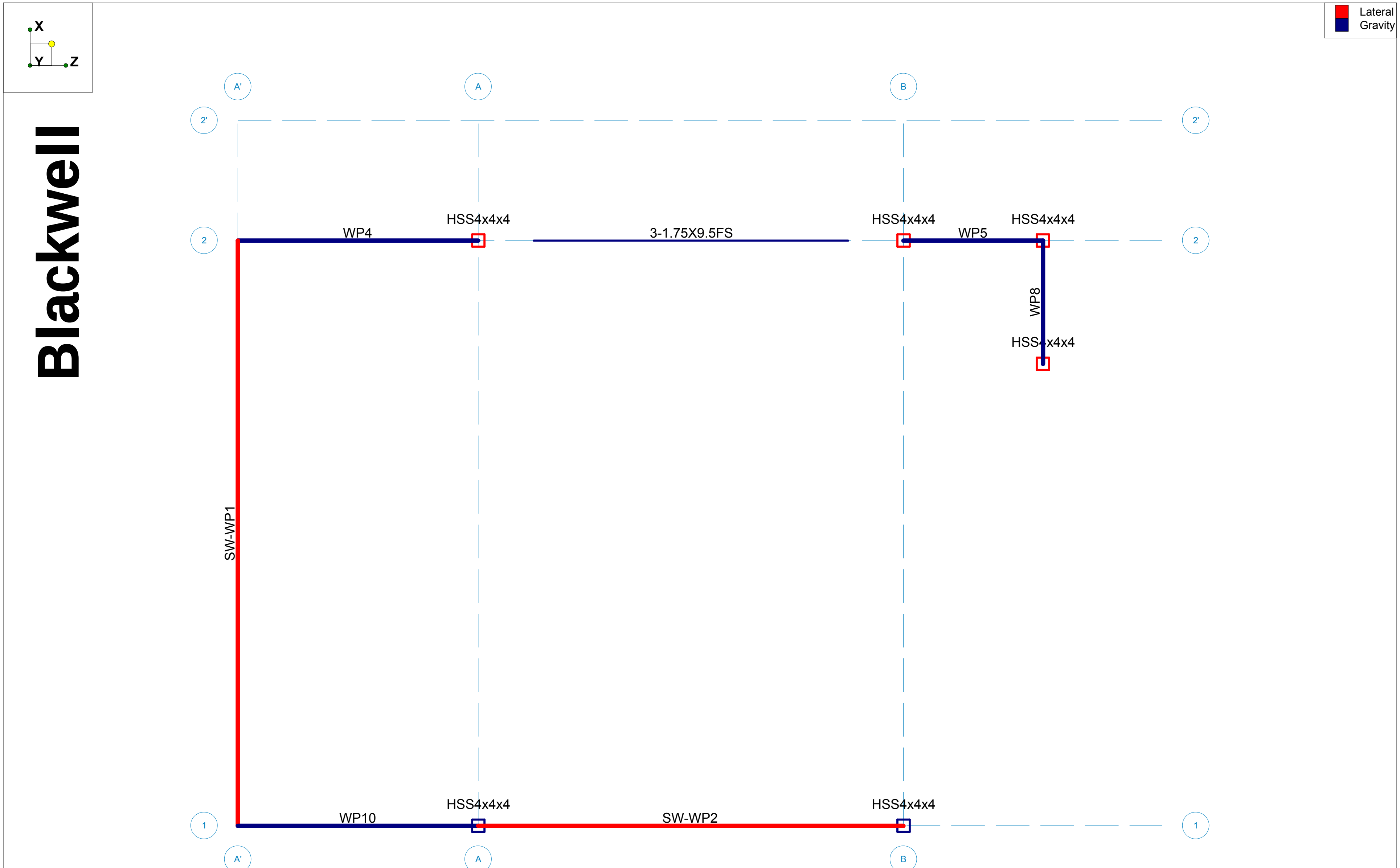


Blackwell Structural Engineers	Roof	ROOF SHAPES
BG	Kimmelman May Residence Volume 1	
170266		



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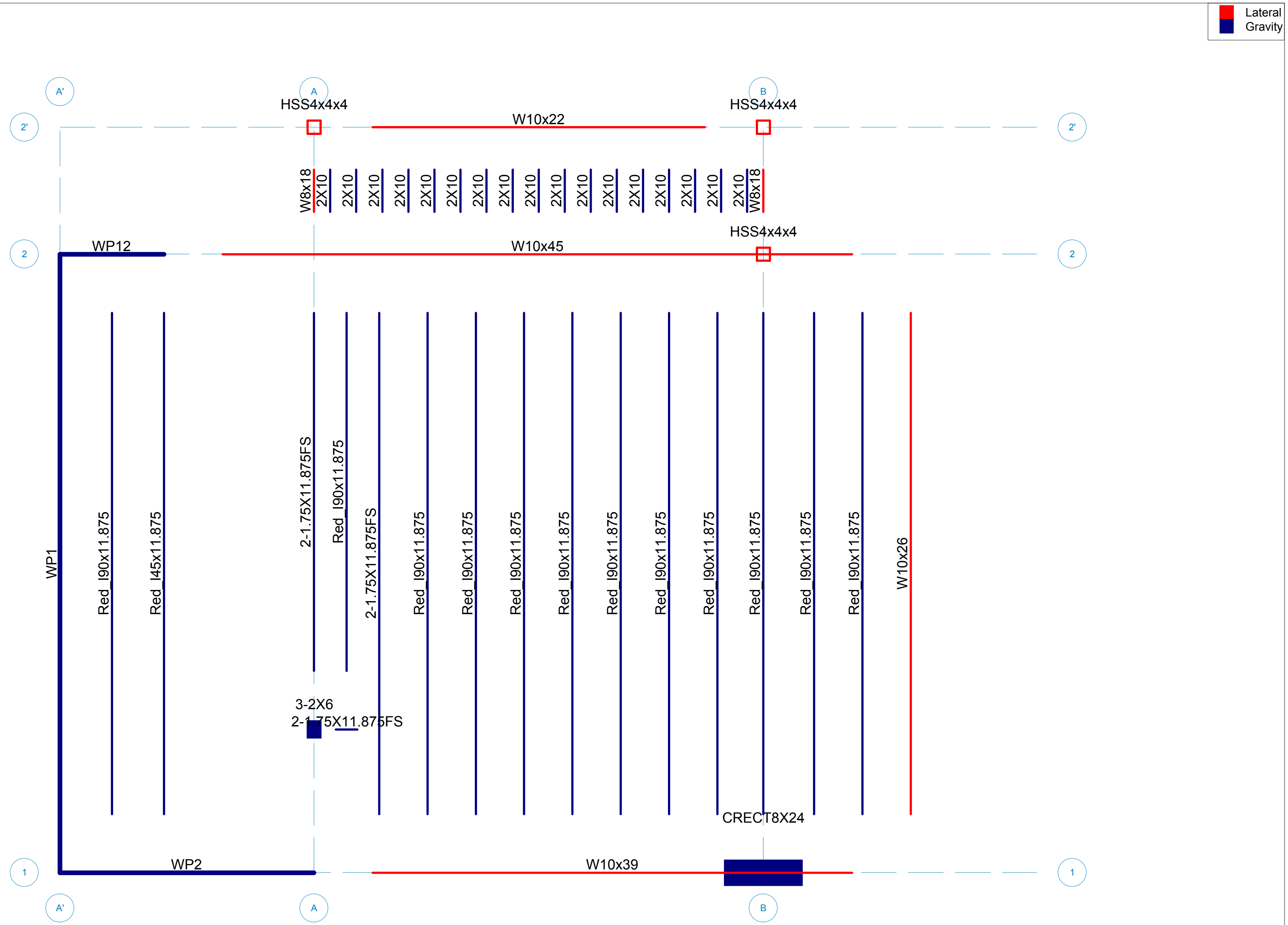
Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	26'-6"	26'-6" SHAPES
BG	Kimmelman May Residence Volume 1	
170266		

Blackwell



Blackwell Structural Engineers	19'	19' SHAPES
BG	Kimmelman May Residence Volume 1	
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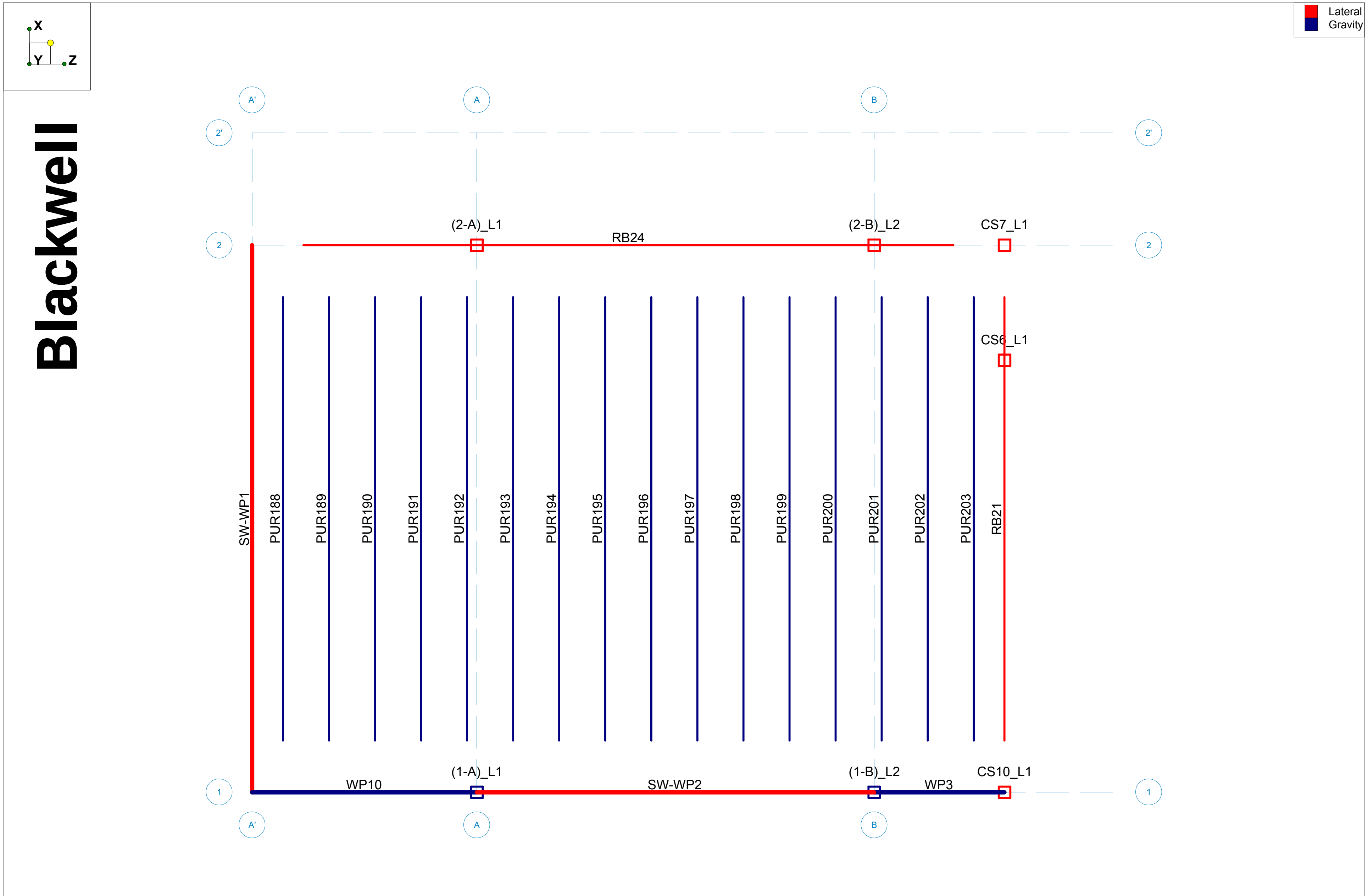
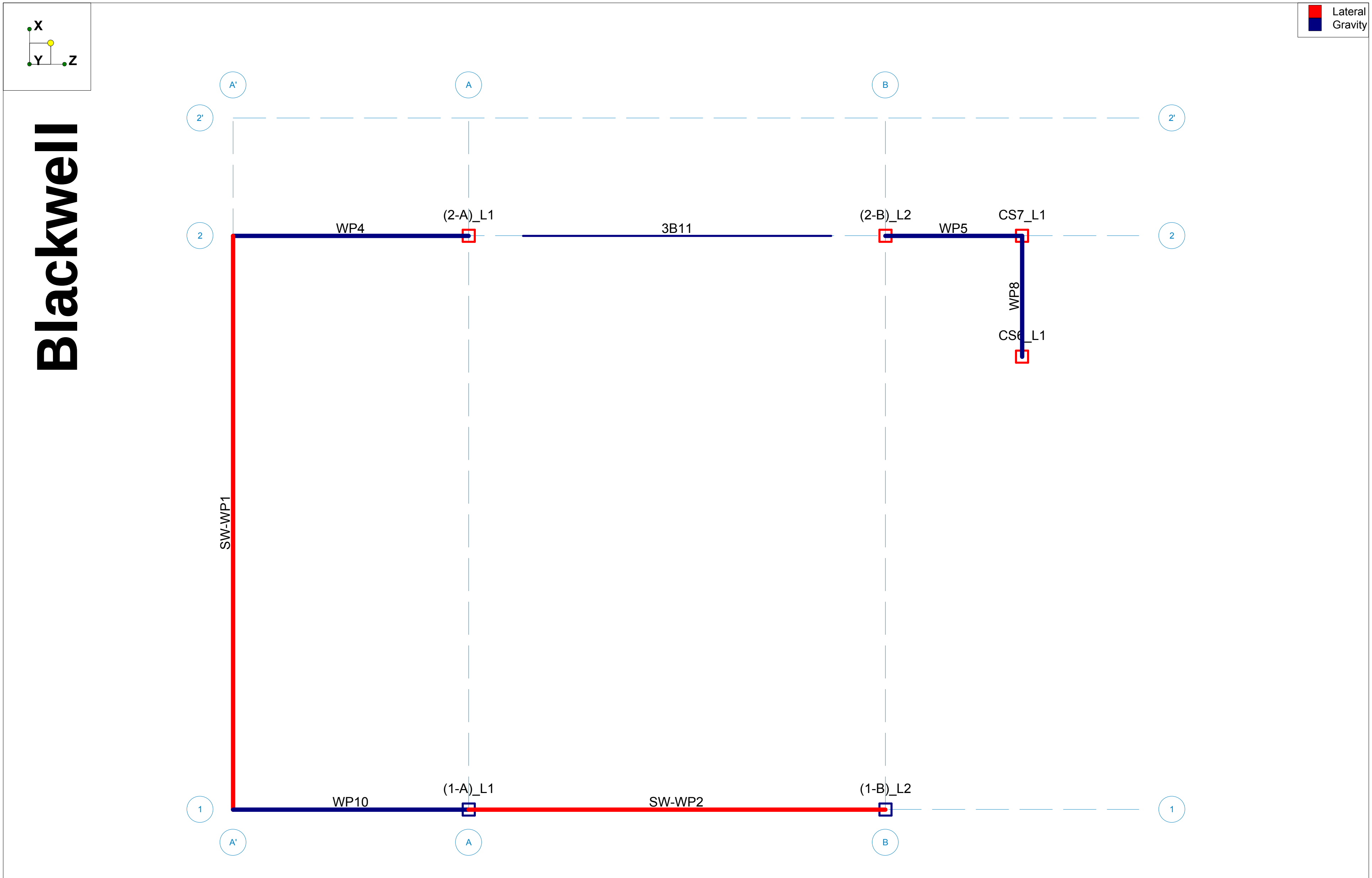
Gravity Wall and Member Designation



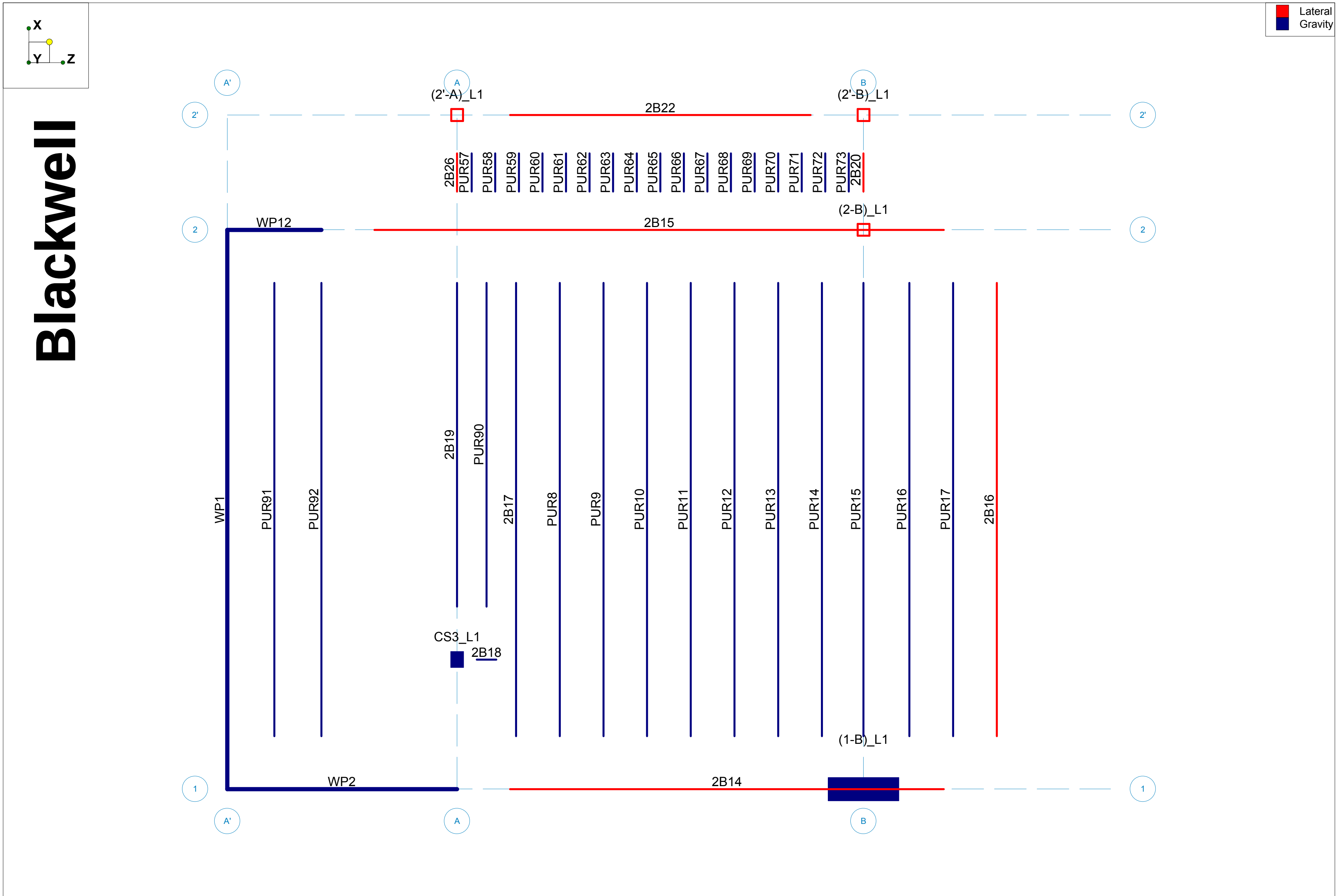
Loads: DL PreComp - PreComposite Dead Load

Blackwell Structural Engineers	Roof	ROOF MEMBER DESIGNATION
BG	Kimmelman May Residence Volume 1	
170266		



Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	26'-6"	26'-6" MEMBER DESIGNATION
BG	Kimmelman May Residence Volume 1	
170266		



Blackwell Structural Engineers	19'	19' MEMBER DESIGNATION
BG	Kimmelman May Residence Volume 1	
170266		

Column Stacks

	Stack L...	Project...	Z [ft]	X [ft]	Lift ...	Lengt...	Bot El..	Top E...	Shape	Material	Funct...	Design ...	Flexural La...	Shear Lay...
1	(2-B)	2-B	18	15.8...	1	9	10	19	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
2					2	14.991	19	33.991	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
3	CS3	-	6.5	3.667	1	9	10	19	3-2X6	Spruce-Pine-Fir	Gravity	Typical	N/A	N/A
4	(1-B)	1-B	18	0	1	9	10	19	CRECT8X24	Conc25NW	Gravity	Typical	Use Desig...	Use Desig...
5					2	14.991	19	33.991	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
6	(2'-A)	2'-A	6.5	19.0...	1	9	10	19	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
7	(2'-B)	2'-B	18	19.0...	1	9	10	19	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
8	(1-A)	1-A	6.5	0	1	16.907	19	35.907	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
9	CS6	-	21.7...	12.5	1	14.362	19	33.362	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
10	(2-A)	2-A	6.5	15.8...	1	16.907	19	35.907	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
11	CS7	-	21.7...	15.8...	1	14.362	19	33.362	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
12	CS10	-	21.7...	0	1	6.862	26.5	33.362	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A

Beam Primary Data : Roof

	Label	Start P...	End Po...	Shape	Material	Design ...	Function	Orientation	Start R...	End R...	Outrig...
1	RB21	N8	N9	W12x22	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
2	RB24	N2	N8	W12x26	A992	Typical	Lateral	Strong Axis	Fixed	Fixed	
3	PUR188	N364	N365	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
4	PUR189	N366	N367	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
5	PUR190	N368	N369	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
6	PUR191	N370	N371	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
7	PUR192	N372	N373	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
8	PUR193	N374	N375	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
9	PUR194	N376	N377	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
10	PUR195	N378	N379	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
11	PUR196	N380	N381	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
12	PUR197	N382	N383	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
13	PUR198	N384	N385	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
14	PUR199	N386	N387	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
15	PUR200	N388	N389	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
16	PUR201	N390	N391	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
17	PUR202	N392	N393	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
18	PUR203	N394	N395	Red_I90x14	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : Dummy Floor for Walls

Label	Start P...	End Po...	Shape	Material	Design ...	Function	Orientation	Start R...	End R...	Outrig...
No Data to Print ...										

Beam Primary Data : 26'-6"

	Label	Start P...	End Po...	Shape	Material	Design ...	Function	Orientation	Start R...	End R...	Outrig...
1	3B11	N7	N6	3-1.75X9.5FS	2.0E Microllam L...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 19'

	Label	Start P...	End Po...	Shape	Material	Design ...	Function	Orientation	Start R...	End R...	Outrig...
1	2B15	N42	N10	W10x45	A992	Typical	Lateral	Strong Axis	Pinned	Fixed	
2	2B14	N11	N13	W10x39	A992	Typical	Lateral	Strong Axis	Pinned	Fixed	
3	2B19	N7	N8	2-1.75X11.875FS	2.0E Microllam L...	Typical	Gravity	Strong Axis	Pinned	Pinned	
4	2B17	N14	N15	2-1.75X11.875FS	2.0E Microllam L...	Typical	Gravity	Strong Axis	Pinned	Pinned	
5	2B18	N8	N16	2-1.75X11.875FS	2.0E Microllam L...	Typical	Gravity	Strong Axis	Pinned	Pinned	
6	2B16	N13	N10	W10x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	

Beam Primary Data : 19' (Continued)

	Label	Start P...	End Po...	Shape	Material	Design ...	Function	Orientation	Start R...	End R...	Outrig...
7	PUR8	N17	N18	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
8	PUR9	N19	N20	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
9	PUR10	N21	N22	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
10	PUR11	N23	N24	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
11	PUR12	N25	N26	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
12	PUR13	N27	N28	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
13	PUR14	N29	N30	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
14	PUR15	N2	N6	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
15	PUR16	N33	N34	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
16	PUR17	N35	N36	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
17	PUR91	N39	N38	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
18	PUR92	N41	N42	Red_I45x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
19	PUR90	N43	N44	Red_I90x11.875	Wood Products ...	Typical	Gravity	Strong Axis	Pinned	Pinned	
20	2B22	N40	N45	W10x22	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
21	2B26	N7	N40	W8x18	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
22	2B20	N45	N6	W8x18	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
23	PUR57	N124	N125	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
24	PUR58	N126	N127	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
25	PUR59	N128	N129	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
26	PUR60	N130	N131	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
27	PUR61	N132	N133	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
28	PUR62	N134	N135	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
29	PUR63	N136	N137	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
30	PUR64	N138	N139	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
31	PUR65	N140	N141	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
32	PUR66	N142	N143	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
33	PUR67	N144	N145	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
34	PUR68	N146	N147	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
35	PUR69	N148	N149	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
36	PUR70	N28	N151	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
37	PUR71	N152	N153	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
38	PUR72	N154	N155	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
39	PUR73	N156	N157	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 10'

Label	Start P...	End Po...	Shape	Material	Design ...	Function	Orientation	Start R...	End R...	Outrig...
No Data to Print ...										

Wood Wall Panel Parameters

	Label	Top Plate	Sill Plate	Studs	Min Stud Sp...	Max Stud Sp...	Green Lumb...	Header Size	Header Matl
1	Typical	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall

Additional Wood Wall Panel Parameters

	Label	Schedule	Min. Panel...	Max. Pane...	Double Sid...	Max. Nail ...	Min. Nail S...	HD Chords	HD Chord ...	Hold Down
1	Typical	0.469 (8d) Pan...	.295	.61	No	6-in.	2-in.	2-2X6	Same as ...	HDU_DF-SP

Wall Panel Parameters

	Label	Top Floor	Bottom Floor	Start...	End ...	Material ...	Material ...	Thickn...	Function	Design R...	Distance Belo...	Parape...
1	WP1	19'	10'	N3	N1	Concrete	Conc25N...	8	Gravity	Typical	9	NA
2	WP2	19'	10'	N11	N1	Concrete	Conc25N...	8	Gravity	Typical	9	NA
3	WP3	Roof	26'-6"	N4	N9	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	6.5	Default...
4	WP4	Dummy Floor for W...	19'	N3	N4	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	7.5	NA
5	WP5	Dummy Floor for W...	19'	N5	N6	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	7.5	NA
6	WP6	Dummy Floor for W...	26'-6"	N4	N5	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	6.5	NA
7	WP7	Dummy Floor for W...	26'-6"	N7	N9	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	6.5	NA
8	WP8	Dummy Floor for W...	19'	N9	N6	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	7.5	NA
9	SW-WP1	Roof	19'	N1	N2	Wood	Spruce-P...	5.5 (st...	Lateral	Typical	7.5	Default...
10	WP10	Roof	19'	N1	GRD...	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	7.5	Default...
11	SW-WP2	Roof	19'	GRD...	N4	Wood	Spruce-P...	5.5 (st...	Lateral	Typical	7.5	Default...
12	WP12	19'	10'	N3	N42	Concrete	Conc25N...	8	Gravity	Typical	9	NA

Wall Panel Advanced Parameters

	Label	Design Method	In-plane Icr Factor	Out-plane Icr Factor	K
1	WP1	N/A			
2	WP2	N/A			
3	WP3	Segmented	N/A	N/A	
4	WP4	Segmented	N/A	N/A	
5	WP5	Segmented	N/A	N/A	
6	WP6	Segmented	N/A	N/A	
7	WP7	Segmented	N/A	N/A	
8	WP8	Segmented	N/A	N/A	
9	SW-WP1	Segmented	N/A	N/A	
10	WP10	Segmented	N/A	N/A	
11	SW-WP2	Segmented	N/A	N/A	
12	WP12	N/A			

Gravity Loading

Floors

	Label	Elev...	Area Load...	Floor ...	Deck Default	Deck...	Parent Des...	Bot Rebar D...	Continu...	Top R...	Ina...	Sp...	Sp...	No ...	Par...
1	Roof	33.3...	Roof	Floor ...	Roof Deck	0	None	N/A	N/A		N/A		.083	Sh...		0
2	Dummy Floo...	33	Floor	Floor ...	Interior Wood ...	0	None	N/A	N/A		N/A		6.5	Sh...		0
3	26'-6"	26.5	Floor	Floor ...	Interior Wood ...	0	None	N/A	N/A		N/A		.083	Sh...		0
4	19'	19	Floor	Floor ...	Interior Wood ...	0	None	N/A	N/A		N/A		.083	Sh...		0
5	10'	10	Floor	Floor ...	Interior Wood ...	0	None	N/A	N/A		N/A		.083	Sh...		0

Deck General Properties

	Label	Material Type	Deck	Unbraced[ft]	Max Spa...
1	Interior Wood w/ Topping	Wood Deck	1 in	1	4
2	Roof Deck	Wood Deck	1 in	1	4

Deck Loads

	Label	Two Way	Self Wt [psf]	Super DL [psf]	Const DL [psf]	Const LL [psf]
1	Interior Wood w/ Topping		3	37	0	0
2	Roof Deck		3	22	0	0

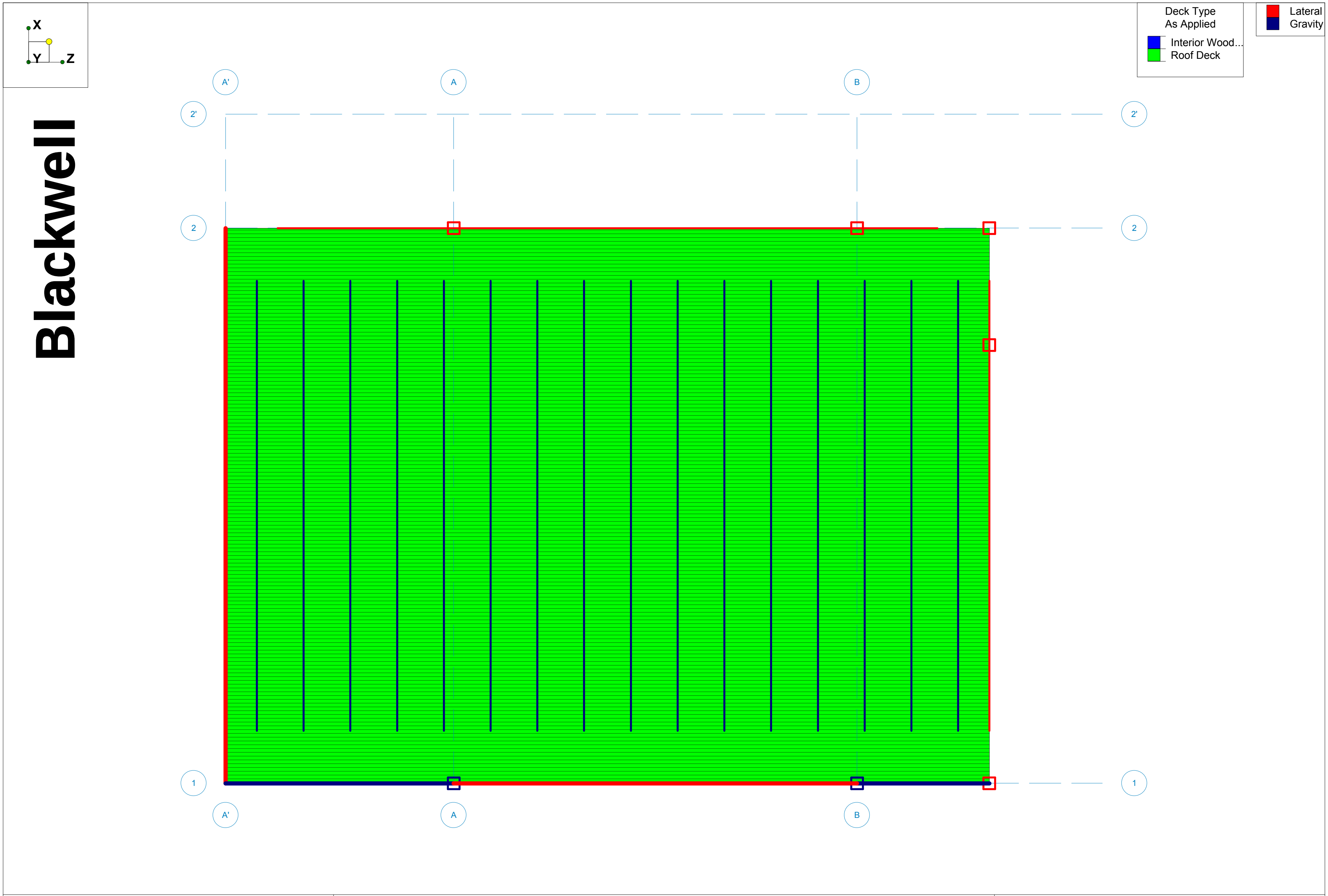
Uniform Area Loads

	Label	Additive	PreDL[psf]	PostDL[psf]	LL[psf]	LL Type	VL[psf]	Dyn Load[psf]
1	Floor				40	LL-Non		
2	Roof				192	SL		57.6

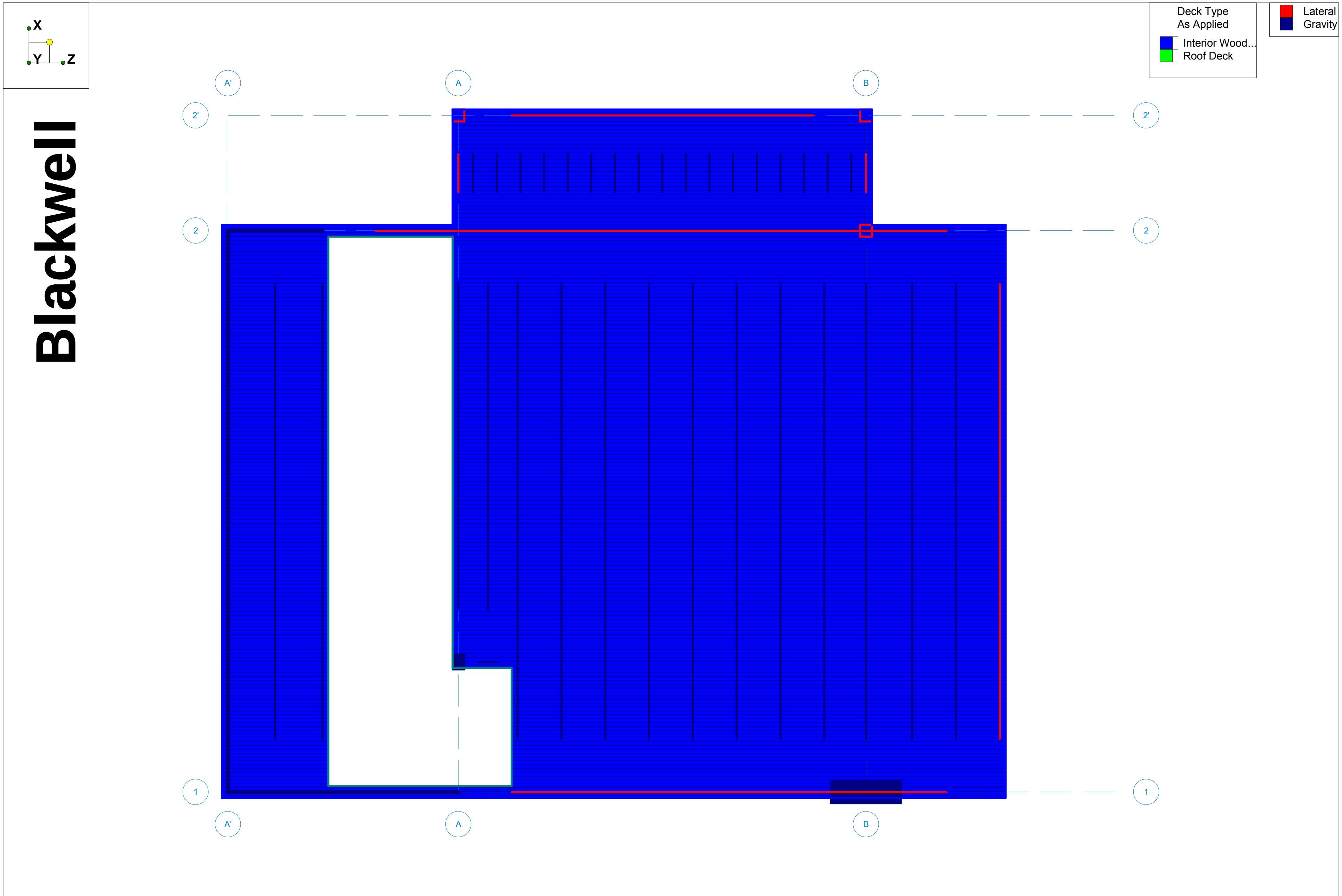
Combinations

	Label	Sol...	Cat...	Fa...	Cat...	Fa...	Cat...	Fa...	Cat...	Fa...	Cat...	Fac...	Cat...	Fac...	Cat...	Fac...	Cat...	Fac...	Cat...	Fac...	Cat...	Fac...
1	ASCE Strength 1 ...	Yes	DL	1.4																		
2	ASCE Strength 2 (...	Yes	DL	1.2	LL	1.6																
3	ASCE Strength 2 (...	Yes	DL	1.2	LL	1.6	SL	.5														
4	ASCE Strength 3 (...	Yes	DL	1.2	LL	.5																
5	ASCE Strength 3 (...	Yes	DL	1.2	SL	1.6	LL	.5														
6	ASCE ASD 1 Post	Yes	DL	1																		
7	ASCE ASD 2 Post	Yes	DL	1	LL	1																
8	ASCE ASD 3 (b) ...	Yes	DL	1	SL	1																
9	ASCE ASD 4 (a) ...	Yes	DL	1	LL	.75																
10	ASCE ASD 4 (b) ...	Yes	DL	1	LL	.75	SL	.75														

This load is 30% of the roof snow load and only contributes to the mass for seismic load calculations completed in RISA3D

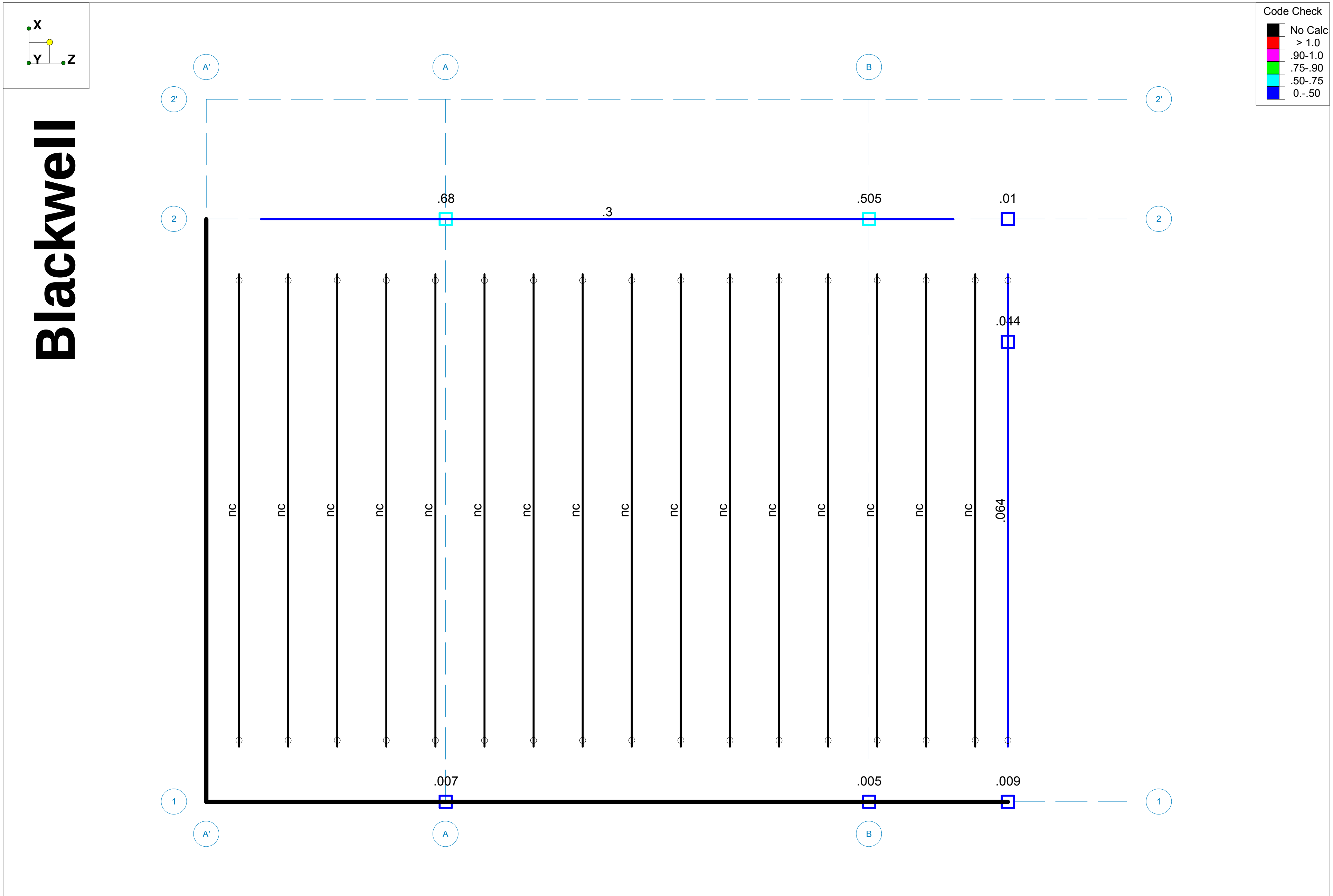


Blackwell Structural Engineers	Roof	ROOF DECK
BG	Kimmelman May Residence Volume 1	
170266		

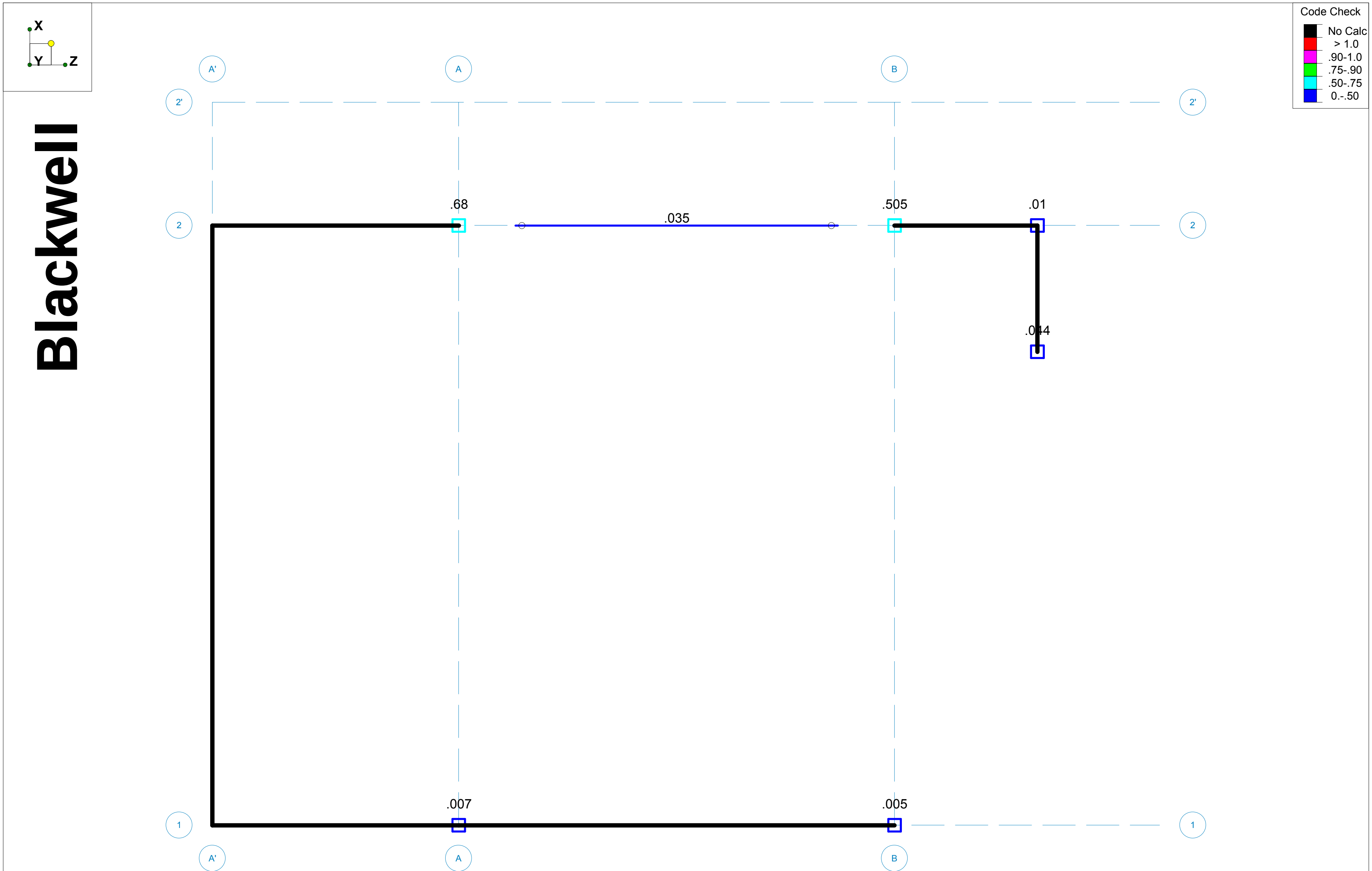


Blackwell Structural Engineers	19'	19' DECK
BG	Kimmelman May Residence Volume 1	
170266		

Gravity Steel and Wood Member Utilization



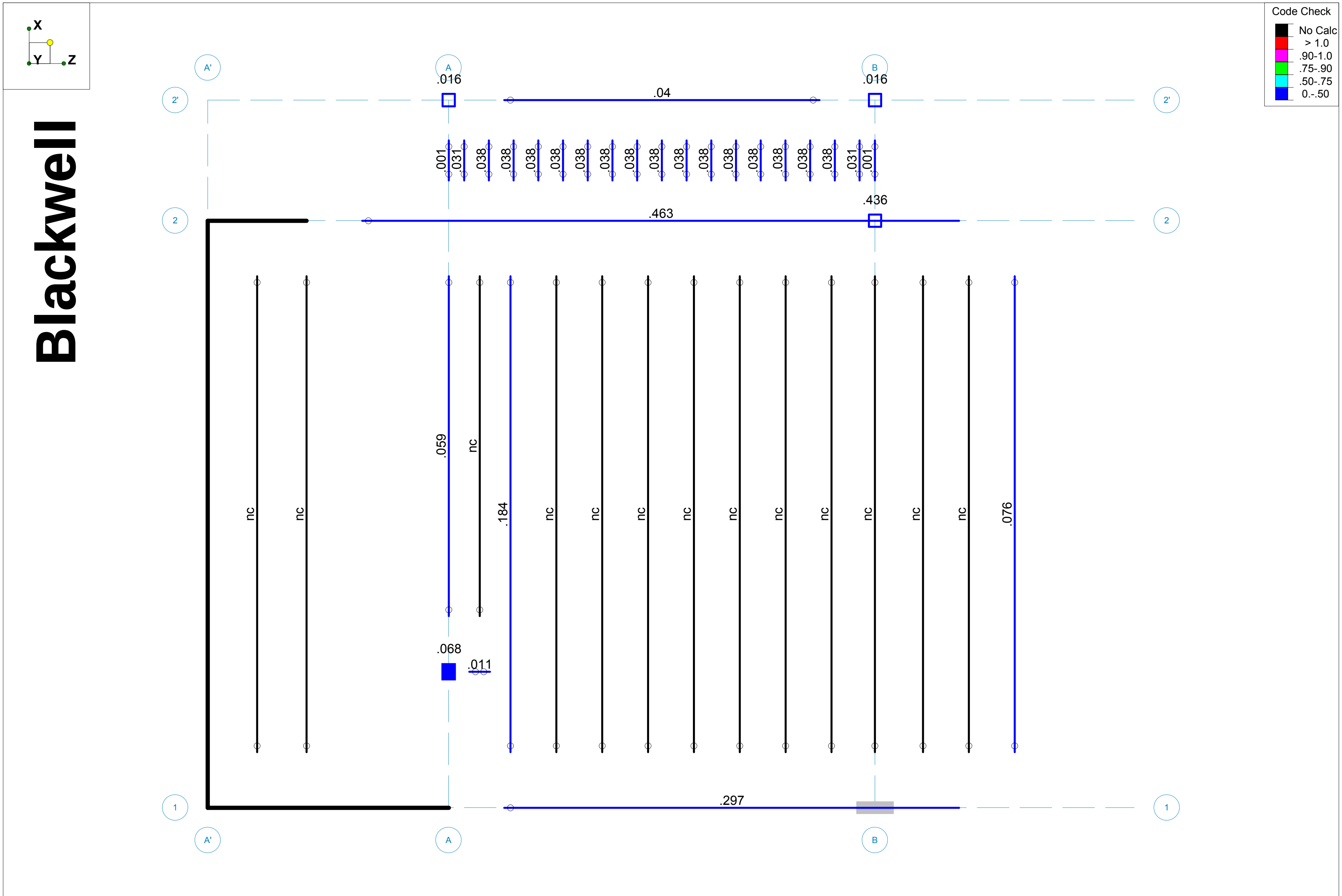
Blackwell Structural Engineers	Roof	ROOF BENDING CHECK
BG	Kimmelman May Residence Volume 1	
170266		



Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	26'-6"	26'-6" BENDING CHECK
BG	Kimmelman May Residence Volume 1	
170266		

Blackwell



Blackwell Structural Engineers	19'	19' BENDING CHECK
BG	Kimmelman May Residence Volume 1	
170266		

Beam Code Summary for Hot Rolled : Roof

	Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
1	RB21	W12x22	Yes		0	A992	.064	3.299	5	.036	0	LL	.013	3.463	5
2	RB24	W12x26	Yes		0	A992	.3	6.667	5	.117	12.415	LL	.174	6.667	5

Beam Code Summary for Hot Rolled : Dummy Floor for Walls

Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC

No Data to Print ...

Beam Code Summary for Hot Rolled : 26'-6"

Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC

No Data to Print ...

Beam Code Summary for Hot Rolled : 19'

	Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
1	2B15	W10x45	Yes		0	A992	.463	3.981	5	1.408	19.108	LL	.238	0	5
2	2B14	W10x39	Yes		0	A992	.297	5.249	5	.838	15.271	LL	.199	11.4...	5
3	2B16	W10x26	Yes		0	A992	.076	12.37	5	.089	8.576	DL+...	.036	15.8...	5
4	2B22	W10x22	Yes		0	A992	.04	5.75	3	.037	5.75	DL+...	.018	0	2
5	2B26	W8x18	Yes		0	A992	.001	1.625	2	.036	0	LL	.002	3.25	2
6	2B20	W8x18	Yes		0	A992	.001	1.625	2	.036	0	LL	.002	0	2

Beam Code Summary for Wood : Roof

Label	Size	Explicit	Material	Bending...	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear C..	Loc[ft]	LC

No Data to Print ...

Beam Code Summary for Wood : Dummy Floor for Walls

Label	Size	Explicit	Material	Bending...	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear C..	Loc[ft]	LC

No Data to Print ...

Beam Code Summary for Wood : 26'-6"

	Label	Size	Explicit	Material	Bending...	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear C..	Loc[ft]	LC
1	3B11	3-1.75X9.5FS	Yes	2.0E ...	.035	5.75	6	.036	0	LL	.022	0	6

Beam Code Summary for Wood : 19'

	Label	Size	Explicit	Material	Bending...	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear C..	Loc[ft]	LC
1	2B19	2-1.75X11.875FS	Yes	2.0E ...	.059	6.083	7	.047	6.083	DL+...	.044	12.1...	7
2	2B17	2-1.75X11.875FS	Yes	2.0E ...	.184	8.411	7	.194	8.246	DL+...	.108	15.8...	7
3	2B18	2-1.75X11.875FS	Yes	2.0E ...	.011	.833	7	.036	0	LL	.03	0	7
4	PUR57	2X10	Yes	Spruc...	.031	1.625	7	.036	0	LL	.06	3.25	7
5	PUR58	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
6	PUR59	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
7	PUR60	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
8	PUR61	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
9	PUR62	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
10	PUR63	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
11	PUR64	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
12	PUR65	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
13	PUR66	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
14	PUR67	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7

Beam Code Summary for Wood : 19' (Continued)

	Label	Size	Explicit	Material	Bending...	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear C..	Loc[ft]	LC
15	PUR68	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
16	PUR69	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
17	PUR70	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
18	PUR71	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
19	PUR72	2X10	Yes	Spruc...	.038	1.625	7	.036	0	LL	.073	3.25	7
20	PUR73	2X10	Yes	Spruc...	.031	1.625	7	.036	0	LL	.06	3.25	7

Beam Design for Wood Products : Roof

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-ft]	M'r[k-ft]	Max Start Re...	Max End R...	Min Start R...	Min End Rea...
1	PUR188	Red_I90x14	Yes	-1.95	2.921	-7.717	13.144	1.95	1.95	.262	.262
2	PUR189	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
3	PUR190	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
4	PUR191	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
5	PUR192	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
6	PUR193	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
7	PUR194	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
8	PUR195	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
9	PUR196	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
10	PUR197	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
11	PUR198	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
12	PUR199	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
13	PUR200	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
14	PUR201	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
15	PUR202	Red_I90x14	Yes	-2.332	2.921	-9.232	13.144	2.332	2.332	.306	.306
16	PUR203	Red_I90x14	Yes	-1.95	2.921	-7.717	13.144	1.95	1.95	.262	.262

Beam Design for Wood Products : Dummy Floor for Walls

Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-ft]	M'r[k-ft]	Max Start Re...	Max End R...	Min Start R...	Min End Rea...
No Data to Print ...										

Beam Design for Wood Products : 26'-6"

Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-ft]	M'r[k-ft]	Max Start Re...	Max End R...	Min Start R...	Min End Rea...
No Data to Print ...										

Beam Design for Wood Products : 19'

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-ft]	M'r[k-ft]	Max Start Re...	Max End R...	Min Start R...	Min End Rea...
1	PUR8	Red_I90x11....	Yes	-.82	2.255	-3.244	9.605	.82	.82	.428	.428
2	PUR9	Red_I90x11....	Yes	-.82	2.255	-3.244	9.605	.82	.82	.428	.428
3	PUR10	Red_I90x11....	Yes	-.82	2.255	-3.244	9.605	.82	.82	.428	.428
4	PUR11	Red_I90x11....	Yes	-.82	2.255	-3.244	9.605	.82	.82	.428	.428
5	PUR12	Red_I90x11....	Yes	-.82	2.255	-3.244	9.605	.82	.82	.428	.428
6	PUR13	Red_I90x11....	Yes	-.82	2.255	-3.244	9.605	.82	.82	.428	.428
7	PUR14	Red_I90x11....	Yes	-.8	2.255	-3.168	9.605	.8	.8	.418	.418
8	PUR15	Red_I90x11....	Yes	-.82	2.255	-3.244	9.605	.82	.82	.428	.428
9	PUR16	Red_I90x11....	Yes	-.839	2.255	-3.321	9.605	.839	.839	.438	.438
10	PUR17	Red_I90x11....	Yes	-.82	2.255	-3.244	9.605	.82	.82	.428	.428
11	PUR91	Red_I90x11....	Yes	-.88	2.255	-3.482	9.605	.88	.88	.458	.458
12	PUR92	Red_I45x11....	Yes	.546	1.785	-2.167	4.685	.546	.546	.283	.283
13	PUR90	Red_I90x11....	Yes	-.434	2.255	-1.319	9.605	.434	.434	.231	.231

Hot Rolled Steel Column Code Checks

	Stack	Lift	Shape	Code ...	Elev[ft]	LC	Shear ...	Elev[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y ...	phi*Mn z-z [k...	Cb	Eqn
1	(2-B)	1	HSS4x4...	.436	10	5	.000	10		5	99.405	139.518	16.181	16.181	1	H1-1a
2		2	HSS4x4...	.505	19	5	.000	19		5	54.475	139.518	16.181	16.181	1	H1-1a
3	(1-B)	2	HSS4x4...	.005	19	1	.000	19		5	54.475	139.518	16.181	16.181	1	H1-1...
4	(2'-A)	1	HSS4x4...	.016	10	3	.000	10		5	99.405	139.518	16.181	16.181	1	H1-1...
5	(2'-B)	1	HSS4x4...	.016	10	2	.000	10		5	99.405	139.518	16.181	16.181	1	H1-1...
6	(1-A)	1	HSS4x4...	.007	19	1	.000	19		5	42.809	139.518	16.181	16.181	1	H1-1...
7	CS6	1	HSS4x4...	.044	19	5	.000	19		5	58.847	139.518	16.181	16.181	1	H1-1...
8	(2-A)	1	HSS4x4...	.680	19	5	.000	19		5	42.809	139.518	16.181	16.181	1	H1-1a
9	CS7	1	HSS4x4...	.010	33.063	5	.000	19		5	58.847	139.518	16.181	16.181	1	H1-1b
10	CS10	1	HSS4x4...	.009	26.5	5	.000	26.5		5	114.564	139.518	16.181	16.181	1	H1-1...

Wood Column Code Checks

	Stack	Lift	Shape	Code C...	Elev[ft]	LC	Shear C...	Elev[ft]	Dir	LC	Fc' [ksi]	Ft' [ksi]	Fb1' [ksi]	Fb2' [ksi]	Fv' [ksi]	Eqn
1	CS3	1	3-2X6	.068	10	7	.000	10	z	10	.368	.585	1.134	1.308	.135	3.6.3

Blackwell

Structural Engineers

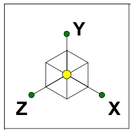
Gravity Wall Utilization

Wall Results, Wood Wall Panel

	Wall Panel	Region	Stud Size	Stud Spacing[in]	Axial Check	Gov LC
1	WP3	R1	2X6	16	.175	8
2	WP4	R1	2X6	16	.014	6
3	WP5	R1	2X6	16	.014	6
4	WP6	R1	2X6	16	.003	6
5	WP7	R1	2X6	16	.003	6
6	WP8	R1	2X6	16	.016	6
7	SW-WP1	R1	2X6	16	.106	8
8	WP10	R1	2X6	16	.562	8
9	SW-WP2	R1	2X6	16	.556	8

Blackwell
Structural Engineers

LATERAL SYSTEM
Designed using RISA3D integrated with RISAFloor



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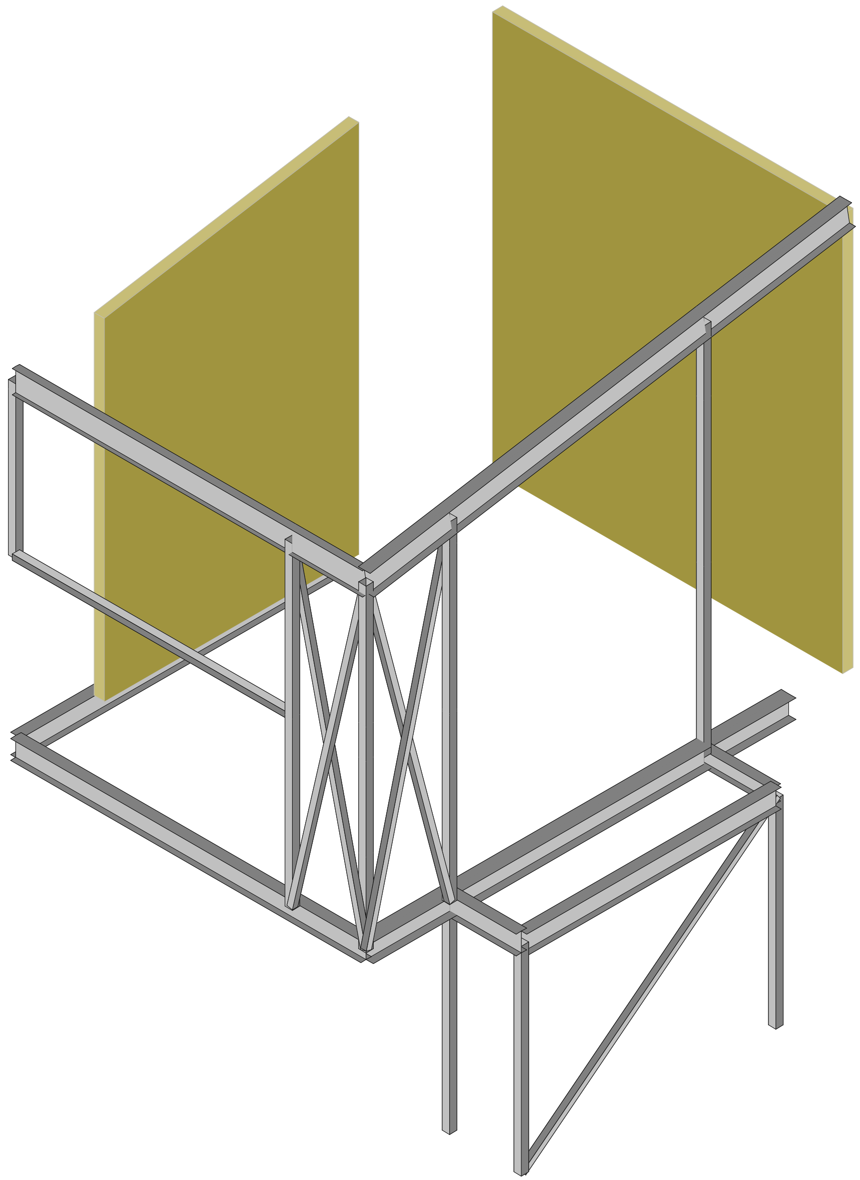
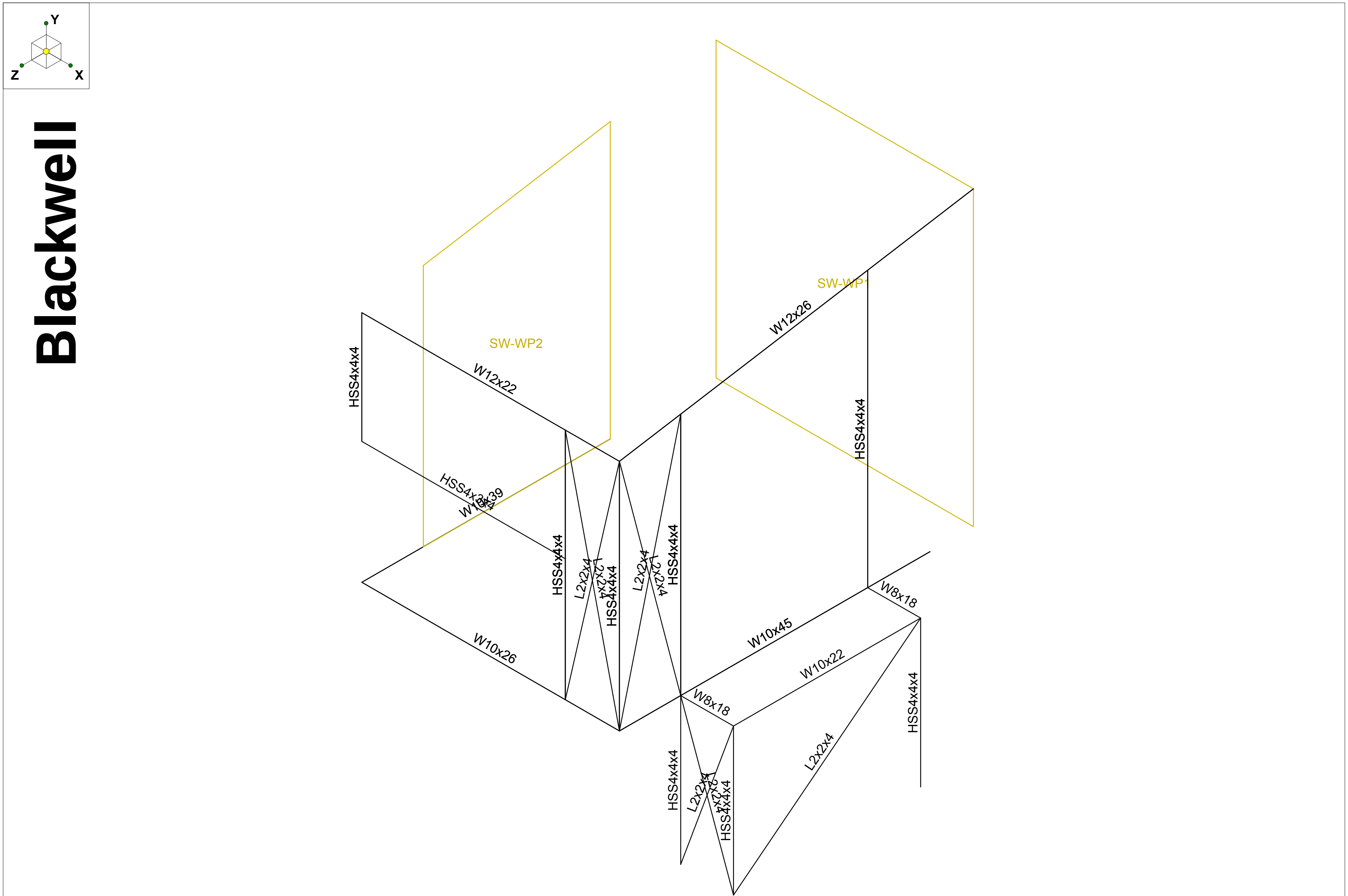


*RENDERED VIEW SHOWN FOR CONTEXT ONLY. REFER TO MEMBER PROPERTIES AND STRUCTURAL DRAWINGS FOR DETAILS.

Blackwell Structural Engineers	Kimmelman May Residence Volume 1	GENERAL LATERAL RENDER
BG		
170266		

Lateral Geometry Definition



Blackwell Structural Engineers	Kimmelman May Residence Volume 1	MEMBER SHAPES
BG		
170266		

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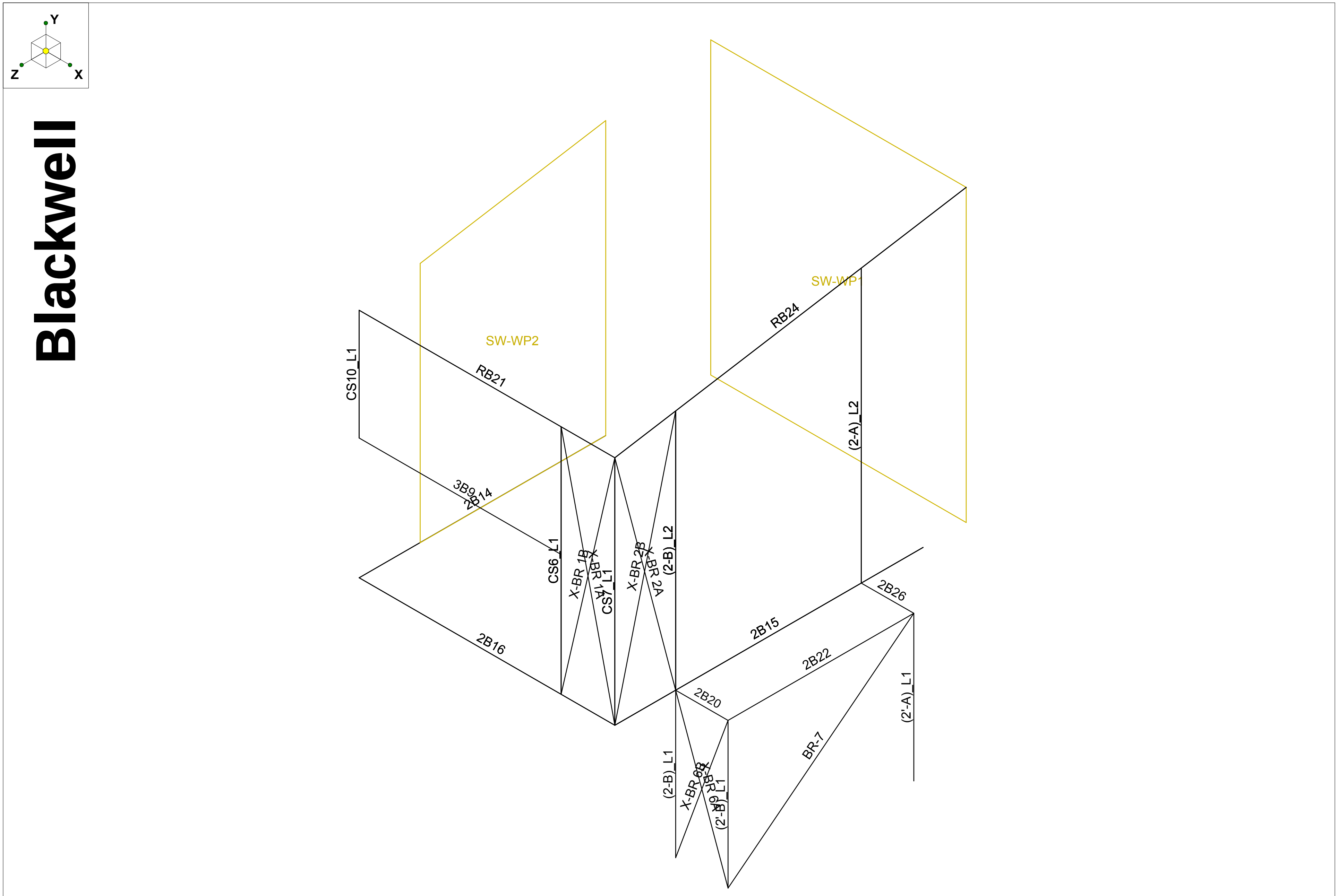
Lateral Wall and Member Designation
Linked to RISAFloor



Blackwell Structural Engineers	Kimmelman May Residence Volume 1	MEMBER DESIGNATIONS
BG		
170266		

Floor Diaphragms

	Elev..	Mas..	Mas..	Center ...	Plus ...	Minus...	Plus Z..	Minus...	Inac...	Diaph...	Type	Region	Design ...	SGAF	Material	Thick...	No Win...
1	33.3..	31.6..	199...	8.028, 1...	%5	%5	%5	%5		D3	Flexible		N/A	N/A	N/A	N/A	
2	19	30.1..	336...	8.085, 8...	%5	%5	%5	%5		D2	Flexible		N/A	N/A	N/A	N/A	

Member Primary Data

	Label	I Joint	J Joint	K Joint	Rotate(de...	Section/Shape	Type	Design List	Material	Design Rul...
1	RB21	N10	N1			W12x22	Beam	None	A992	Typical
2	2B15	F4_N42	N24			W10x45	Beam	None	A992	Typical
3	2B14	N25	N26			W10x39	Beam	None	A992	Typical
4	2B16	N26	N24			W10x26	Beam	None	A992	Typical
5	RB24	N5	N10			W12x26	Beam	None	A992	Typical
6	(2-A)_L2	N23	N9			HSS4x4x4	Column	None	A500 Gr....	Typical
7	(2'-A)_L1	N46	N42			HSS4x4x4	Column	None	A500 Gr....	Typical
8	(2'-B)_L1	N47	N43			HSS4x4x4	Column	None	A500 Gr....	Typical
9	CS6_L1	N32	N8			HSS4x4x4	Column	None	A500 Gr....	Typical
10	2B26	N23	N42			W8x18	Beam	None	A992	Typical
11	2B??	N43	N22			W8x18	Beam	None	A992	Typical
12	2B22	N42	N43			W10x22	Beam	None	A992	Typical
13	3B9	N2	N16		90	HSS4x3x4	Beam	Tube	A992	Typical
14	BR-7	N47	N42			Brace in X	VBrace	Single Angle	A36 Gr.36	Typical
15	CS10_L1	N16	N1			HSS4x4x4	Column	None	A500 Gr....	Typical
16	(2-B)_L1	N44	N22			HSS4x4x4	Column	None	A500 Gr....	Typical
17	(2-B)_L2	N22	N6			HSS4x4x4	Column	None	A500 Gr....	Typical
18	X-BR 1A	N8	N24			Brace in X	VBrace	Single Angle	A36 Gr.36	Typical
19	X-BR 1B	N32	N10			Brace in X	VBrace	Single Angle	A36 Gr.36	Typical
20	X-BR 2A	N10	N22			Brace in Z	VBrace	Single Angle	A36 Gr.36	Typical
21	X-BR 2B	N24	N6			Brace in Z	VBrace	Single Angle	A36 Gr.36	Typical
22	CS7_L1	N24	N10			HSS4x4x4	Column	None	A500 Gr....	Typical
23	X-BR 6A	N22	N47			Brace in X	VBrace	Single Angle	A36 Gr.36	Typical
24	X-BR 6B	N44	N43			Brace in X	VBrace	Single Angle	A36 Gr.36	Typical

Hot Rolled Steel Design Parameters

	Label	Shape	Lengt...	Lbyy[ft]	Lbzz[ft]	Lcomp t...	Lcomp b...	L-tor...	Kyy	Kzz	Cb	Function
1	RB21	W12x22	15.833	2.667	1.33	.5						Lateral
2	2B15	W10x45	19.108	1.333		.5						Lateral
3	2B14	W10x39	15.271	1.333	11.5	.5						Lateral
4	2B16	W10x26	15.833	2.667		.5						Lateral
5	RB24	W12x26	22.071	1.33	1.33	.5						Lateral
6	(2-A)_L2	HSS4x4x4	16.907	Floor	Floor	Floor	Floor	Floor				Lateral
7	(2'-A)_L1	HSS4x4x4	9	Floor	Floor	Floor	Floor	Floor				Lateral
8	(2'-B)_L1	HSS4x4x4	9	Floor	Floor	Floor	Floor	Floor				Lateral
9	CS6_L1	HSS4x4x4	14.362	Floor	Floor	Floor	Floor	Floor				Lateral
10	2B26	W8x18	3.25			Floor						Lateral
11	2B??	W8x18	3.25			Floor						Lateral
12	2B22	W10x22	11.5			Floor						Lateral
13	3B9	HSS4x3x4	12.5			Lbyy						Lateral
14	BR-7	Brace in X	14.603									Lateral
15	CS10_L1	HSS4x4x4	6.862	Floor	Floor	Floor	Floor	Floor				Lateral
16	(2-B)_L1	HSS4x4x4	9	Floor	Floor	Floor	Floor	Floor				Lateral

Hot Rolled Steel Design Parameters (Continued)

	Label	Shape	Lengt...	Lbyy[ft]	Lbzz[ft]	Lcomp t...	Lcomp b...	L-tor...	Kyy	Kzz	Cb	Function
17	(2-B)_L2	HSS4x4x4	14.991	Floor	Floor	Floor	Floor	Floor				Lateral
18	X-BR 1A	Brace in X	14.744									Lateral
19	X-BR 1B	Brace in X	14.744									Lateral
20	X-BR 2A	Brace in Z	14.849									Lateral
21	X-BR 2B	Brace in Z	15.458									Lateral
22	CS7_L1	HSS4x4x4	14.362	Floor	Floor	Floor	Floor	Floor				Lateral
23	X-BR 6A	Brace in X	9.569									Lateral
24	X-BR 6B	Brace in X	9.569									Lateral

Wall Panel Data

	Label	A Joint	B Joint	C Joint	D Joint	Material ...	Material ...	Thickn...	Design Rule	Panel/Spacing
1	SW-WP1	N4	N5	N21	N19	Wood	Spruce-...	5.5 (st...	Typical	S1_15/32_8d@6...
2	SW-WP2	N7	N33	N20	N25	Wood	Spruce-...	5.5 (st...	Typical	S1_15/32_8d@6...

Wood Wall Panel Parameters

	Label	Top Plate	Sill Plate	Studs	Min Stud S...	Max Stud ...	Green Lumber?	Header Size	Header Matl
1	Typical	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall

Additional Wood Wall Panel Parameters

	Label	Schedule	Min. P...	Max. P...	Double Sided ...	Max. N...	Min. N...	HD Chor...	HD Chord M...	Hold Down	Ecce...
1	Typical	0.469 (8d) Panel Group	.295	.61	No	6-in.	2-in.	2-2X6	Same as Wall	HDU_DF-SP	Yes

Lateral Loading
Note: vertical loads applied via RISAFloor

Toronto 416.593.5300 | **Waterloo** 519.616.0895 | **Victoria** 778.817.1010 | **Halifax** 902.593.0125 | **blackwell.ca**

Seismic Generation Input

Seismic Code:	ASCE 7-10				
Ct_X:	.02	T_X (sec):	Not Entered	R_X:	3.25
Ct_Z:	.02	T_Z (sec):	Not Entered	R_Z:	3.25
Ct Exp. X:	.75	Ct Exp. Z:	.75		
Risk Cat	I or II	TL (sec):	8		
SD1 (g):	.363	SDS (g):	.591	S1 (g):	.304
Base Elev (ft):	19	Parapet Ht (ft):	0		

Seismic Generation Detail Results

T_X Used (sec):	.148	T_X Method A:	.148	T_X Upper Limit:	.207
T_Z Used (sec):	.148	T_Z Method A:	.148	T_Z Upper Limit:	.207
Importance Fac.:	1	Design Cat.:	D		
V_X (k):	5.756	Gov. Eqn.	ASCE Eqn 12.8-2	Cs_X:	0.182
V_Z (k):	5.756	Gov. Eqn.	ASCE Eqn 12.8-2	Cs_Z:	0.182

Seismic Generation Force Results

Floor Level	Height (ft)	Weight (k)	Force X (k)	Force Z (k)	CG X (ft)	CG Z (ft)
Roof	14.362	31.653	5.756	5.756	8.028	10.937
Totals		31.653	5.756	5.756		

Seismic Generation Diaphragm Results

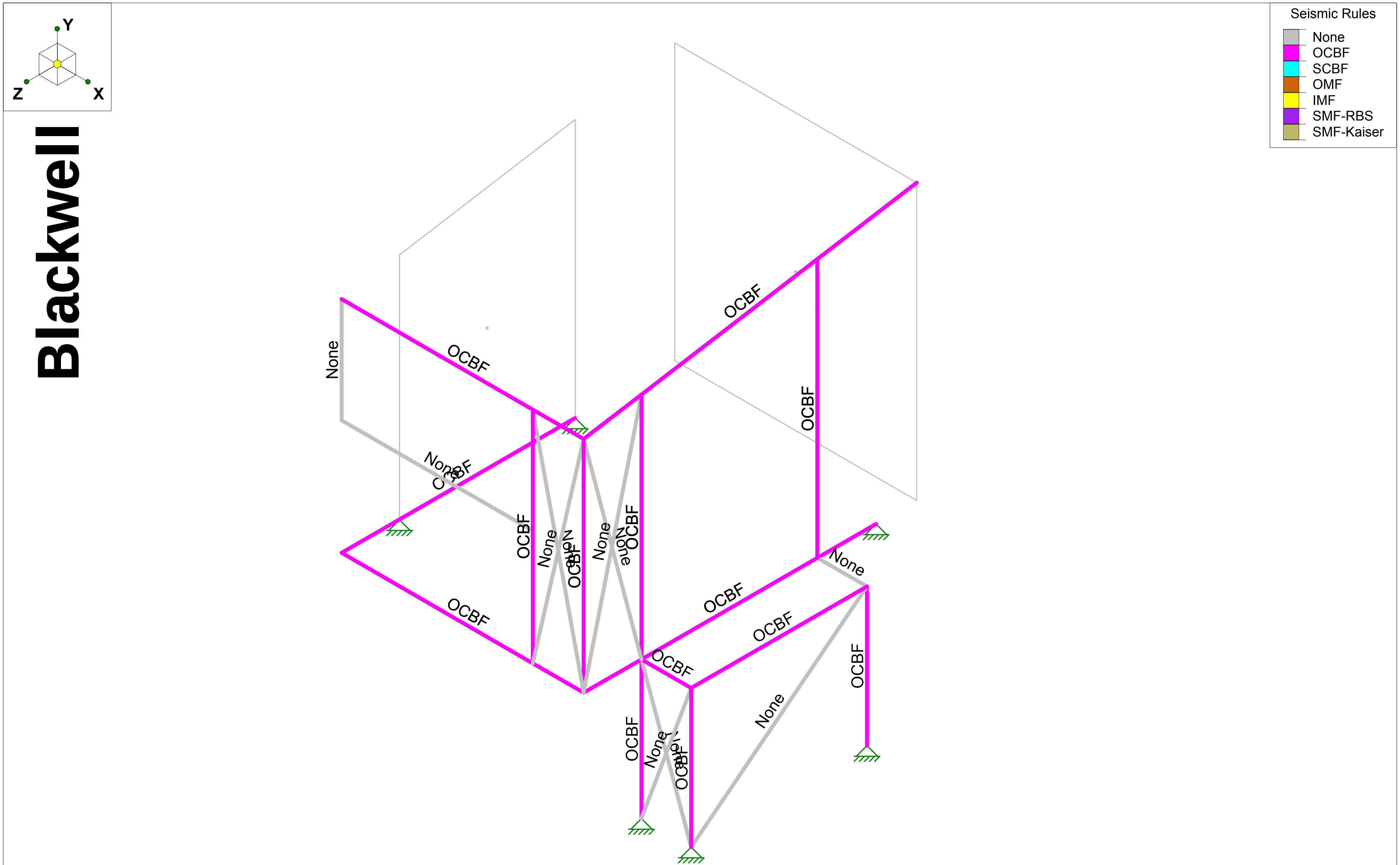
Floor Level	Width (X) (ft)	Length (Z) (ft)	X Plus (ft)	X Minus (ft)	Z Plus (ft)	Z Minus (ft)
Roof	15.833	21.771	.792	.792	1.089	1.089

Frame / HR Column Seismic Design Rule

	Label	Frame Ductility	Overstrength Reqd
1	OCBF	Minimal	Yes
2	SCBF	High	Yes
3	OMF	Minimal	Yes
4	IMF	Moderate	Yes
5	SMF-RBS	High	Yes
6	SMF-Kaiser	High	Yes

HR Beam Seismic Design Rule

	Label	Moment Connection	Overstrength Reqd	Z Factor	Hinge Location[in]
1	OCBF	Other/None	Yes		
2	SCBF	Other/None	Yes		
3	OMF	BUEEP			12
4	IMF	BFP			12
5	SMF-RBS	RBS		.685	14.625
6	SMF-Kaiser	KBB-B			12



Member Seismic Design Rule Displayed

Blackwell Structural Engineers	Kimmelman May Residence Volume 1	OCBF SEISMIC DESIGN MEMBERS
BG		
170266		

Wind Generation Input

Wind Code:	ASCE 7-10	Topographic Factor K1:	0
Wind Speed, V(mph):	119	Topographic Factor K2:	0
		Topographic Factor K3:	0
Exposure Category:	C	Directionality Factor Kd:	.85
Base Elevation(ft):	19	Parapet Height(ft):	0

Wind Generation Detail Results

		Kzt:	1
Exposure Constant Alpha:	9.5	h (ft):	16.176
Exposure Constant zg:	900	Kh:	.862
Gust Effect Factor, G:	.85	Windward Cp:	.8
		qh (psf):	26.577
		GCpn (windward):	+1.5
		GCpn (leeward):	-1.0

Wind Generation Floor Geometry Results

Floor Level	Height (ft)	Kz	Width (X) (ft)	Length (Z) (ft)	Leeward Cp(X)	Leeward Cp(Z)
Roof	14.362	.849	15.833	21.771	.5	.425
Sloped Roof	17.991	.882	39.498/39.498 (ft^2)	57.45/0 (ft^2)	.5	.425

Wind Generation Floor Force Results

Floor Level	qz (psf)	Windward Pres. (psf)	Leeward Pres. X (psf)	Leeward Pres. Z (psf)	Force X (k)	Force Z (k)
Roof	26.158	17.787	11.295	9.601	4.547	3.114
Sloped Roof	27.178	18.481	11.295	9.601	1.176	1.062
Total					5.723	4.176

Basic Load Cases

	BLC Description	Category	X Gravi..	Y Gravi..	Z Gravity	Joint	Point	Distributed	Area(M...	Surfac...
1	Dead Load	DL		-1		3	80	22		
2	Live Load	LL				1	56	17		
3	Live Load Special (public assemb	LLS								
4	Roof Live Load	RLL								
5	Snow Load	SL					24	2		
6	Snow Load Nonshedding	SLN								
7	Rain Load	RL								
8	Wind Load X	WLX				1				
9	Partial X Wind Load 1	WLXP1				1				
10	Partial X Wind Load 2	WLXP2				1				
11	Wind Load Z	WLZ				1				
12	Partial Z Wind Load 1	WLZP1				1				
13	Partial Z Wind Load 2	WLZP2				1				
14	Earthquake Load X	ELX				1				
15	Earthquake Load X Plus Z Ecce...	ELX+Z				1				
16	Earthquake Load X Minus Z Ec...	ELX-Z				1				
17	Earthquake Load Z	ELZ				1				
18	Earthquake Load Z Plus X Ecce...	ELZ+X				1				
19	Earthquake Load Z Minus X Ec...	ELZ-X				1				
20	Other Load 1	OL1								
21	Other Load 2	OL2								
22	Other Load 3	OL3								
23	Other Load 4	OL4								
24	Wind Load Roof +X	WLX+R								
25	Wind Load Roof -X	WLX-R								
26	Wind Load Roof +Z	WLZ+R							1	
27	Wind Load Roof -Z	WLZ-R							1	
28	BLC 8 Transient Area Loads	None						20		
29	BLC 11 Transient Area Loads	None						20		
30	BLC 14 Transient Area Loads	None						20		
31	BLC 17 Transient Area Loads	None						20		
32	BLC 26 Transient Area Loads	None						92		
33	BLC 27 Transient Area Loads	None						92		

Load Combinations

	Description	So...	PDelta	S...	BLC	Factor	BLC	Fa...	BLC	Fa...	BLC	Fa...	BLC	Fa...	B...	Fa...	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	Dead Load		Y		DL	1																		
2	Live Load		Y		LL	1																		
3	Snow Load		Y		SL	1																		
4	Wind Load X		Y		WLX	1																		
5	Partial X Wind Loa...		Y		WL...	1																		
6	Partial X Wind Loa...		Y		WL...	1																		
7	Wind Load Z		Y		WLZ	1																		
8	Partial Z Wind Loa...		Y		WL...	1																		
9	Partial Z Wind Loa...		Y		WL...	1																		
10	Earthquake Load X		Y		ELX	1																		
11	Earthquake Load ...		Y		EL...	1																		
12	Earthquake Load ...		Y		EL...	1																		
13	Earthquake Load Z		Y		ELZ	1																		

Load Combinations (Continued)

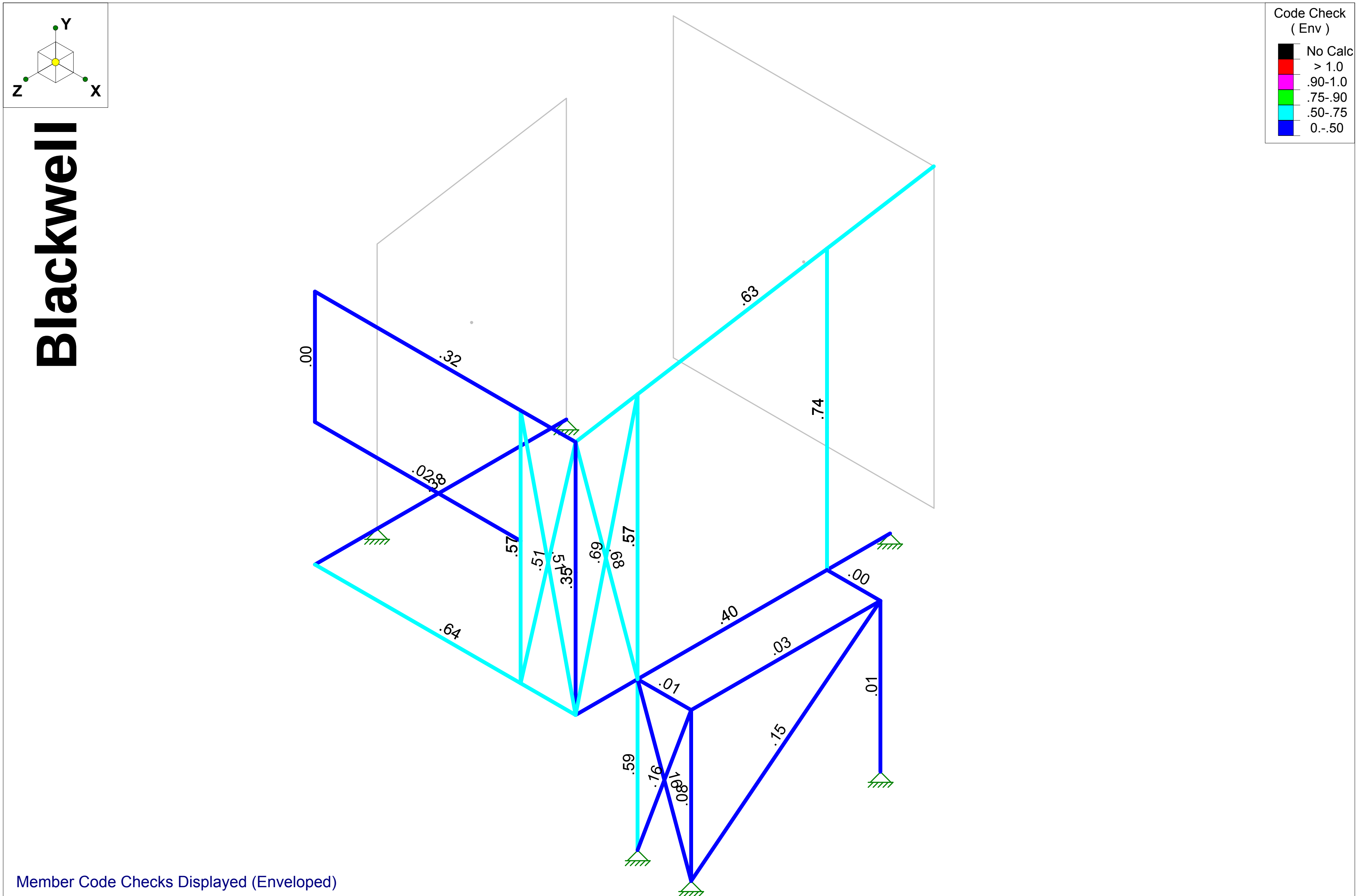
	Description	So...	PDelta	S...	BLC	Factor	BLC	Fa...	BLC	Fa...	BLC	Fa...	BLC	Fa...	B...	Fa...	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
14	Earthquake Load ...		Y		ELZ...	1																		
15	Earthquake Load ...		Y		ELZ...	1																		
16	Wind Load Roof +X		Y		WL...	1																		
17	Wind Load Roof -X		Y		WL...	1																		
18	Wind Load Roof +Z		Y		WL...	1																		
19	Wind Load Roof -Z		Y		WL...	1																		
20			Y																					
21	ASCE Strength 3 (...	Yes	Y		DL	1.2	WLX	.5	WL...	.5														
22	ASCE Strength 3 (...	Yes	Y		DL	1.2	WLZ	.5	WL...	.5														
23	ASCE Strength 3 (...	Yes	Y		DL	1.2	WLX	-.5	WL...	.5														
24	ASCE Strength 3 (...	Yes	Y		DL	1.2	WLZ	-.5	WL...	.5														
25	ASCE Strength 3 (...	Yes	Y		DL	1.2	SL	1.6	SLN	1.6	WLX	.5	WL...	.5										
26	ASCE Strength 3 (...	Yes	Y		DL	1.2	SL	1.6	SLN	1.6	WLZ	.5	WLZ...	.5										
27	ASCE Strength 3 (...	Yes	Y		DL	1.2	SL	1.6	SLN	1.6	WLX	-.5	WL...	.5										
28	ASCE Strength 3 (...	Yes	Y		DL	1.2	SL	1.6	SLN	1.6	WLZ	-.5	WLZ...	.5										
29	ASCE Strength 4 (...	Yes	Y		DL	1.2	WLX	1	LL	.5	LLS	1	WL...	1										
30	ASCE Strength 4 (...	Yes	Y		DL	1.2	WLZ	1	LL	.5	LLS	1	WLZ...	1										
31	ASCE Strength 4 (...	Yes	Y		DL	1.2	WLX	-1	LL	.5	LLS	1	WL...	1										
32	ASCE Strength 4 (...	Yes	Y		DL	1.2	WLZ	-1	LL	.5	LLS	1	WLZ...	1										
33	ASCE Strength 4 (...	Yes	Y		DL	1.2	WLX	1	LL	.5	LLS	1	SL	.5	S...	.5	WL...	1						
34	ASCE Strength 4 (...	Yes	Y		DL	1.2	WLZ	1	LL	.5	LLS	1	SL	.5	S...	.5	WL...	1						
35	ASCE Strength 4 (...	Yes	Y		DL	1.2	WLX	-1	LL	.5	LLS	1	SL	.5	S...	.5	WL...	1						
36	ASCE Strength 4 (...	Yes	Y		DL	1.2	WLZ	-1	LL	.5	LLS	1	SL	.5	S...	.5	WL...	1						
37	ASCE Strength 6 (...	Yes	Y		DL	.9	WLX	1	WL...	1														
38	ASCE Strength 6 (...	Yes	Y		DL	.9	WLZ	1	WL...	1														
39	ASCE Strength 6 (...	Yes	Y		DL	.9	WLX	-1	WL...	1														
40	ASCE Strength 6 (...	Yes	Y		DL	.9	WLZ	-1	WL...	1														
41	ASCE Strength 5 (...	Yes	Y		DL	1.2	Sds...	.2	Rho...	1	LL	.5	LLS	1	SL	.2	SLN	.2						
42	ASCE Strength 5 (...	Yes	Y		DL	1.2	Sds...	.2	Rho...	1	LL	.5	LLS	1	SL	.2	SLN	.2						
43	ASCE Strength 5 (...	Yes	Y		DL	1.2	Sds...	.2	Rho...	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
44	ASCE Strength 5 (...	Yes	Y		DL	1.2	Sds...	.2	Rho...	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
45	ASCE Strength 7 (...	Yes	Y		DL	.9	Sds...	-.2	Rho...	1														
46	ASCE Strength 7 (...	Yes	Y		DL	.9	Sds...	-.2	Rho...	1														
47	ASCE Strength 7 (...	Yes	Y		DL	.9	Sds...	-.2	Rho...	-1														
48	ASCE Strength 7 (...	Yes	Y		DL	.9	Sds...	-.2	Rho...	-1														
49	ASCE ASD 5 (a) (a)	Yes	Y		DL	1	WLX	.6	WL...	.6														
50	ASCE ASD 5 (a) (b)	Yes	Y		DL	1	WLZ	.6	WL...	.6														
51	ASCE ASD 5 (a) (c)	Yes	Y		DL	1	WLX	-.6	WL...	.6														
52	ASCE ASD 5 (a) (d)	Yes	Y		DL	1	WLZ	-.6	WL...	.6														
53	ASCE ASD 6 (a) (a)	Yes	Y		DL	1	WLX	.45	LL	.75	LLS	.75	WL...	.45										
54	ASCE ASD 6 (a) (b)	Yes	Y		DL	1	WLZ	.45	LL	.75	LLS	.75	WLZ...	.45										
55	ASCE ASD 6 (a) (c)	Yes	Y		DL	1	WLX	-.45	LL	.75	LLS	.75	WL...	.45										
56	ASCE ASD 6 (a) (d)	Yes	Y		DL	1	WLZ	-.45	LL	.75	LLS	.75	WLZ...	.45										
57	ASCE ASD 6 (c) (a)	Yes	Y		DL	1	WLX	.45	LL	.75	LLS	.75	SL	.75	S...	.75	WL...	.45						
58	ASCE ASD 6 (c) (b)	Yes	Y		DL	1	WLZ	.45	LL	.75	LLS	.75	SL	.75	S...	.75	WL...	.45						
59	ASCE ASD 6 (c) (c)	Yes	Y		DL	1	WLX	-.45	LL	.75	LLS	.75	SL	.75	S...	.75	WL...	.45						
60	ASCE ASD 6 (c) (d)	Yes	Y		DL	1	WLZ	-.45	LL	.75	LLS	.75	SL	.75	S...	.75	WL...	.45						
61	ASCE ASD 7 (a)	Yes	Y		DL	.6	WLX	.6	WL...	.6														
62	ASCE ASD 7 (b)	Yes	Y		DL	.6	WLZ	.6	WL...	.6														
63	ASCE ASD 7 (c)	Yes	Y		DL	.6	WLX	-.6	WL...	.6														
64	ASCE ASD 7 (d)	Yes	Y		DL	.6	WLZ	-.6	WL...	.6														
65	ASCE ASD 5 (b) (a)	Yes	Y		DL	1	Sds...	.14	Rho...	.7														

Load Combinations (Continued)

	Description	So...	PDelta	S...	BLC	Factor	BLC	Fa...	BLC	Fa...	BLC	Fa...	BLC	Fa...	B...	Fa...	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
66	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Rho...	.7														
67	ASCE ASD 5 (b) (c)	Yes	Y		DL	1	Sds...	.14	Rho...	-.7														
68	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Rho...	-.7														
69	ASCE ASD 6 (b) (a)	Yes	Y		DL	1	Sds...	.105	Rho...	.525	LL	.75	LLS	.75										
70	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Rho...	.525	LL	.75	LLS	.75										
71	ASCE ASD 6 (b) (c)	Yes	Y		DL	1	Sds...	.105	Rho...	-.5...	LL	.75	LLS	.75										
72	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Rho...	-.5...	LL	.75	LLS	.75										
73	ASCE ASD 6 (d) (a)	Yes	Y		DL	1	Sds...	.105	Rho...	.525	LL	.75	LLS	.75	SL	.75	SLN	.75						
74	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Rho...	.525	LL	.75	LLS	.75	SL	.75	SLN	.75						
75	ASCE ASD 6 (d) (c)	Yes	Y		DL	1	Sds...	.105	Rho...	-.5...	LL	.75	LLS	.75	SL	.75	SLN	.75						
76	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Rho...	-.5...	LL	.75	LLS	.75	SL	.75	SLN	.75						
77	ASCE ASD 8 (a)	Yes	Y		DL	.6	Sds...	-.14	Rho...	.7														
78	ASCE ASD 8 (b)	Yes	Y		DL	.6	Sds...	-.14	Rho...	.7														
79	ASCE ASD 8 (c)	Yes	Y		DL	.6	Sds...	-.14	Rho...	-.7														
80	ASCE ASD 8 (d)	Yes	Y		DL	.6	Sds...	-.14	Rho...	-.7														
81	ASCE Strength 5 (...	Yes	Y		DL	1.2	Sds...	.2	Om...	1	LL	.5	LLS	1	SL	.2	SLN	.2						
82	ASCE Strength 5 (...	Yes	Y		DL	1.2	Sds...	.2	Om...	1	LL	.5	LLS	1	SL	.2	SLN	.2						
83	ASCE Strength 5 (...	Yes	Y		DL	1.2	Sds...	.2	Om...	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
84	ASCE Strength 5 (...	Yes	Y		DL	1.2	Sds...	.2	Om...	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
85	ASCE Strength 7 (...	Yes	Y		DL	.9	Sds...	-.2	Om...	1														
86	ASCE Strength 7 (...	Yes	Y		DL	.9	Sds...	-.2	Om...	1														
87	ASCE Strength 7 (...	Yes	Y		DL	.9	Sds...	-.2	Om...	-1														
88	ASCE Strength 7 (...	Yes	Y		DL	.9	Sds...	-.2	Om...	-1														
89	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Om...	.7														
90	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Om...	.7														
91	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Om...	-.7														
92	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Om...	-.7														
93	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Om...	.525	LL	.75	LLS	.75										
94	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Om...	.525	LL	.75	LLS	.75										
95	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Om...	-.5...	LL	.75	LLS	.75										
96	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Om...	-.5...	LL	.75	LLS	.75										
97	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Om...	.525	LL	.75	LLS	.75	SL	.75	SLN	.75						
98	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Om...	.525	LL	.75	LLS	.75	SL	.75	SLN	.75						
99	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Om...	-.5...	LL	.75	LLS	.75	SL	.75	SLN	.75						
100	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Om...	-.5...	LL	.75	LLS	.75	SL	.75	SLN	.75						
101	ASCE ASD 8 (os-a)	Yes	Y		DL	.6	Sds...	-.14	Om...	.7														
102	ASCE ASD 8 (os-b)	Yes	Y		DL	.6	Sds...	-.14	Om...	.7														
103	ASCE ASD 8 (os-c)	Yes	Y		DL	.6	Sds...	-.14	Om...	-.7														
104	ASCE ASD 8 (os-d)	Yes	Y		DL	.6	Sds...	-.14	Om...	-.7														

Blackwell
Structural Engineers

Lateral Steel Member Utilization



Member Code Checks Displayed (Enveloped)

Blackwell Structural Engineers	Kimmelman May Residence Volume 1	UNITY CHECK
BG		
170266		

Envelope AISC 14th(360-10): LRFD Steel Code Checks

	Member	Shape	Code Che...	Loc[ft]	LC	Shear C...	Loc[ft]	Dir	LC	phi*Pnc [...	phi*Pnt [...	phi*Mn y...	phi*Mn z-...	Cb	Eqn
1	RB21	W12x22	.316	3.299	25	.053	3.299	y	25	250.738	291.6	13.725	52.062	1.69	H1-1b
2	2B15	W10x45	.403	3.782	28	.210	15.525	y	82*	486.999	598.5	76.125	205.875	1	H1-1b
3	2B14	W10x39	.376	11.612	83*	.091	11.453	y	27	393.145	517.5	64.5	175.5	1.8...	H1-1b
4	2B16	W10x26	.639	12.37	83*	.272	15.833	y	83*	297.865	342.45	28.125	117.375	1	H1-1b
5	RB24	W12x26	.626	18.163	86*	.195	18.393	y	82*	315.692	344.25	30.637	89.026	1.85	H1-1b
6	(2-A)_L2	HSS4x4x4	.744	7.397	28	.035	0	z	38	42.809	139.518	16.181	16.181	1.1...	H1-1a
7	(2'-A)_L1	HSS4x4x4	.012	0	84*	.000	0	z	28	99.405	139.518	16.181	16.181	1.6...	H1-1...
8	(2'-B)_L1	HSS4x4x4	.083	0	81*	.000	0	z	88*	99.405	139.518	16.181	16.181	1.6...	H1-1...
9	CS6_L1	HSS4x4x4	.571	0	83*	.092	0	z	34	58.847	139.518	16.181	16.181	1.0...	H1-1a
10	2B26	W8x18	.001	1.625	43	.021	0	z	28	219.949	236.7	17.475	63.75	1.0...	H1-1b
11	2B20	W8x18	.012	0	85*	.038	0	z	84*	219.949	236.7	17.475	63.75	1.5...	H1-1...
12	2B22	W10x22	.028	5.75	84*	.015	11.5	y	83*	132.186	292.05	22.875	97.5	1.0...	H1-1b
13	3B9	HSS4x3x4	.022	6.25	41	.005	12.5	z	44	39.259	130.95	11.7	14.287	1	H1-1b
14	BR-7	L2x2x4	.155	6.845	42	.003	0	y	44	1.04	30.586	.691	1.028	1.1...	H2-1
15	(2-B)_L1	HSS4x4x4	.594	9	28	.013	0	z	84*	99.405	139.518	16.181	16.181	1.6...	H1-1a
16	(2-B)_L2	HSS4x4x4	.571	7.339	26	.018	14.054	z	34	54.475	139.518	16.181	16.181	1.0...	H1-1a
17	X-BR 1A	L2x2x4	.508	0	43	.012	0	y	38	1.02	30.586	.691	1.023	1.1...	H2-1*
18	X-BR 1B	L2x2x4	.509	14.744	41	.002	14.744	y	41	1.02	30.586	.691	1.023	1.1...	H2-1*
19	X-BR 2A	L2x2x4	.681	0	42	.009	14.849	y	43	1.006	30.586	.691	1.02	1.1...	H2-1*
20	X-BR 2B	L2x2x4	.695	15.458	44	.009	0	y	47	.928	30.586	.691	1	1.1...	H2-1*
21	CS7_L1	HSS4x4x4	.355	0	86*	.015	14.063	z	38	58.847	139.518	16.181	16.181	1.1...	H1-1a
22	X-BR 6A	L2x2x4	.159	0	47	.003	9.569	y	47	2.422	30.586	.691	1.217	1.1...	H2-1*
23	X-BR 6B	L2x2x4	.161	9.569	45	.004	0	y	41	2.422	30.586	.691	1.217	1.1...	H2-1*
24	M24	HSS4x4x4	.001	6.862	44	.000	0	z	44	122.41	151.65	17.588	17.588	1	H1-1b

Shear Wall Utilization

Wood Wall Panel Axial Code Checks (AWC NDS-15: ASD)

	Wall Panel	Region	Stud Size	Stud Spaci...	Axial Check	Gov LC	Chord Size	Chord Axial Ch...	Gov LC
1	SW-WP1	R1	2X6	16	0	N/A	2-2X6	.766	73
2	SW-WP2	R1	2X6	16	0	N/A	2-2X6	.436	76

Wood Wall Panel In Plane Code Checks (AWC NDS-15: ASD)

	Wall Panel	Shear Panel Label	Region	Shear Check	Shear Forc...	Gov LC	Hold-Down ...	Tension Ch...	Tie-Dow...	Gov LC
1	SW-WP1	S1_15/32_8d@6	R1	.61	.171	65	HDU2-SDS...	.956	2.94	77
2	SW-WP2	S1_15/32_8d@6	R1	.44	.123	66	NC	NC	1.269	80

Envelope Wall Panel Forces

	Wall Label	Elevation ...		Axial [k]	LC	x Shear [k]	LC	z Shear [k]	LC	x-x Mo...	LC	z-z Mo...	LC
1	SW-WP1	19	max	6.725	99	4.151	91	.07	100	0	102	85.356	97
2		19	min	.04	102	-4.164	89	-.055	102	0	100	-63.685	103
3	SW-WP1	33.362	max	6.197	99	4.151	91	.07	100	.791	102	39.929	97
4		33.362	min	-.217	102	-4.164	89	-.055	102	-1.011	100	-4.165	103
5	SW-WP2	19	max	15.233	100	2.181	92	0	89	0	95	34.744	102
6		19	min	.935	62	-2.181	102	0	91	0	89	-34.966	92
7	SW-WP2	33.362	max	14.849	99	2.181	104	0	58	0	58	3.44	102
8		33.362	min	.718	62	-2.181	90	0	94	0	66	-3.609	92

Tie Down Anchorage Note:

See note on page 171 in the calculation package for volume 2, 3 and 4 in regards to the tie down force of SW-WP2.

Blackwell

Structural Engineers

Lateral Member Detailed Reports Are Included in Appendix C

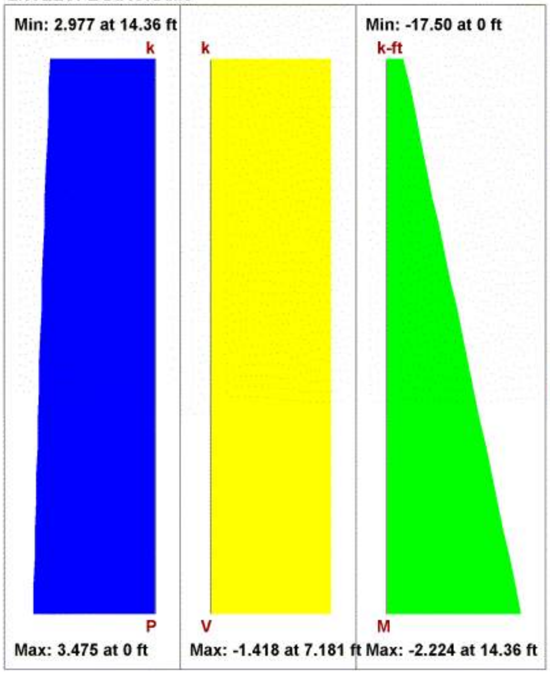
Blackwell

Structural Engineers

Lateral Shear Wall Detailed Reports

CRITERIA		MATERIALS		GEOMETRY	
Code	: **AWC NDS-15:ASD**	Wall Studs	: **Spruce-Pin...**	Total Height	: **14.362 ft**
		Stud Size	: **2X6**	Total Length	: **11.5 ft**
Wall Material	: **Spruce-Pine-Fir**	Chord Material	: **Spruce-Pin...**	Region H/W	: **1.25**
Panel Schedule	: **0.469 (8d) Panel G...**	Chord Size	: **2-2X6**	Cap. Adj. (2w/h)	: **1.00**
Optimize HD	: **Yes**	Top Pl & Sill	: **Spruce-Pin...**	Stud Spacing	: **16 in**
HD Manufacturer:	**SIMPSON**	Top Pl Size	: **2-2X6**	K	: **1.00**
		Sill Pl Size	: **2X6**		

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: **.123**	**k/ft**
Provided Cap	: **.28**	**k/ft**
Ratio	: **.44**	
Governing LC	: **66 (Seismic)**	

CHORDS

Max Comp Force	**3.079 k**
Comp Capacity	: **7.069 k**
Comp Ratio	: **.436**
Gov Comp LC	: **76**
Max Tens Force	: **1.269 k**
Tens Capacity	: **15.444 k**
Tens Ratio	: **.082**
Gov Tens LC	: **80**

STUDS

No gravity-only LC solved.

DEFLECTIONS

Flexure Comp	: **.132**	**in**
Shear Comp	: **.126**	**in**
HD Elong	: **0**	**in**
Tot Deflection	: **.258**	**in**
Governing LC	: **66**	

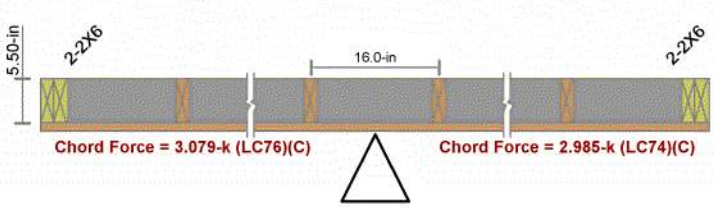
DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	: **St-I**	Nail Size	: **8d**	Num Sides	: **One**
Panel Thick	: **0.469 in**	Reqd Pen	: **1.375 in**	Over Gyp Brd.	: **No**
		Reqd. Spacing	: **6 in**	Shear Capacity	: **0.280 k/ft**
				Adjusted Cap	: **0.280 k/ft**

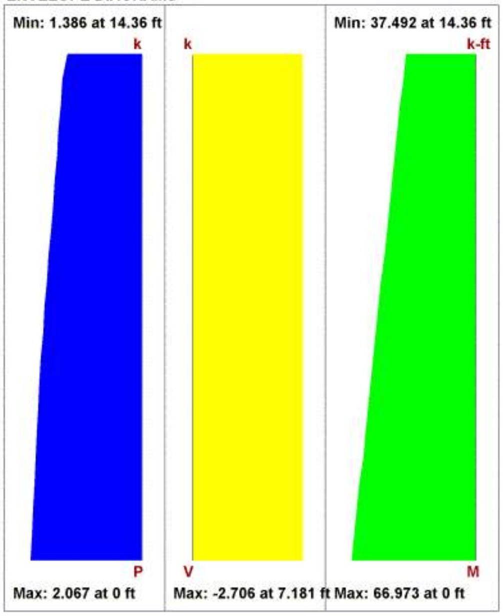
NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

CROSS SECTION DETAILING



CRITERIA		MATERIALS		GEOMETRY	
Code	: AWC NDS-15:ASD	Wall Studs	: Spruce-Pin...	Total Height	: 14.362 ft
		Stud Size	: 2X6	Total Length	: 15.833 ft
Wall Material	: Spruce-Pine-Fir	Chord Material	: Spruce-Pin...	Region H/W	: 0.91
Panel Schedule	: 0.469 (8d) Panel G...	Chord Size	: 2-2X6	Cap. Adj. (2w/h)	: 1.00
Optimize HD	: Yes	Top Pl & Sill	: Spruce-Pin...	Stud Spacing	: 16 in
HD Manufacturer:	SIMPSON	Top Pl Size	: 2-2X6	K	: 1.00
		Sill Pl Size	: 2X6	HD Eccentricity	: 4.313in

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: .171 k/ft
Provided Cap	: .28 k/ft
Ratio	: .61
Governing LC	: 65 (Seismic)

CHORDS

Max Comp Force	: 4.738 k
Comp Capacity	: 6.185 k
Comp Ratio	: .766
Gov Comp LC	: 73
Max Tens Force	: 2.895 k
Tens Capacity	: 13.514 k
Tens Ratio	: .214
Gov Tens LC	: 77

STUDS

No gravity-only LC solved.

HOLD-DOWNS

Required Cap	: 2.94 k
Provided Cap	: 3.075 k
Ratio	: .956
Governing LC	: 77

DEFLECTIONS

Flexure Comp	: .133 in
Shear Comp	: .175 in
HD Elong	: .07 in
Tot Deflection	: .378 in
Governing LC	: 65

DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@6

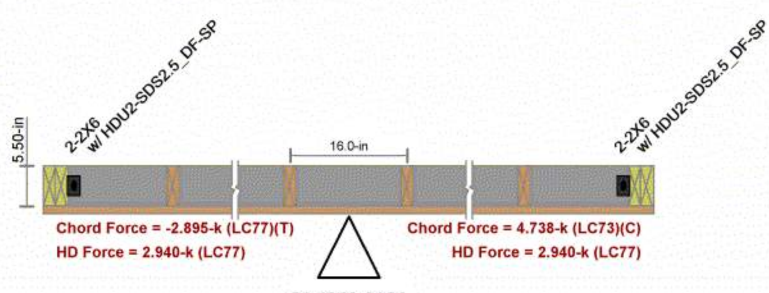
Panel Grade	: St-I	Nail Size	: 8d	Num Sides	: One
Panel Thick	: 0.469 in	Reqd Pen	: 1.375 in	Over Gyp Brd.	: No
		Reqd. Spacing	: 6 in	Shear Capacity	: 0.280 k/ft
				Adjusted Cap	: 0.280 k/ft

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or
2.5" x 0.113" galvanized box

SELECTED HOLD-DOWN : HDU2-SDS2.5_DF-SP

Min Chord Thk	: 3.00 in	Bolt Size:	: .625 in	Base Cap(CD=1):	1.922 k
Reqd Chord Mat	: Douglas Fir			CD factor	: 1.6
				Adjusted Cap	: 3.075 k

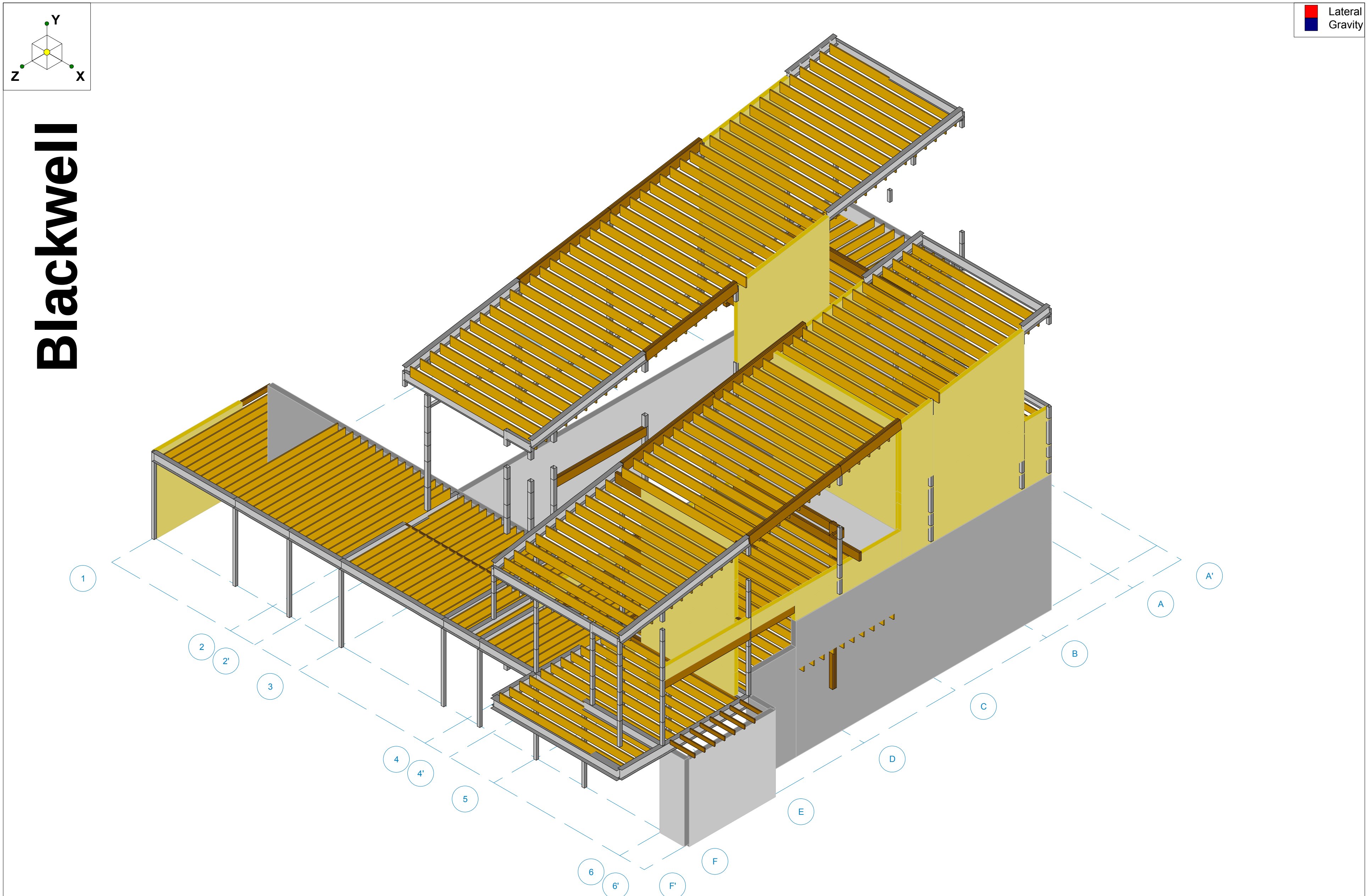
CROSS SECTION DETAILING



Blackwell

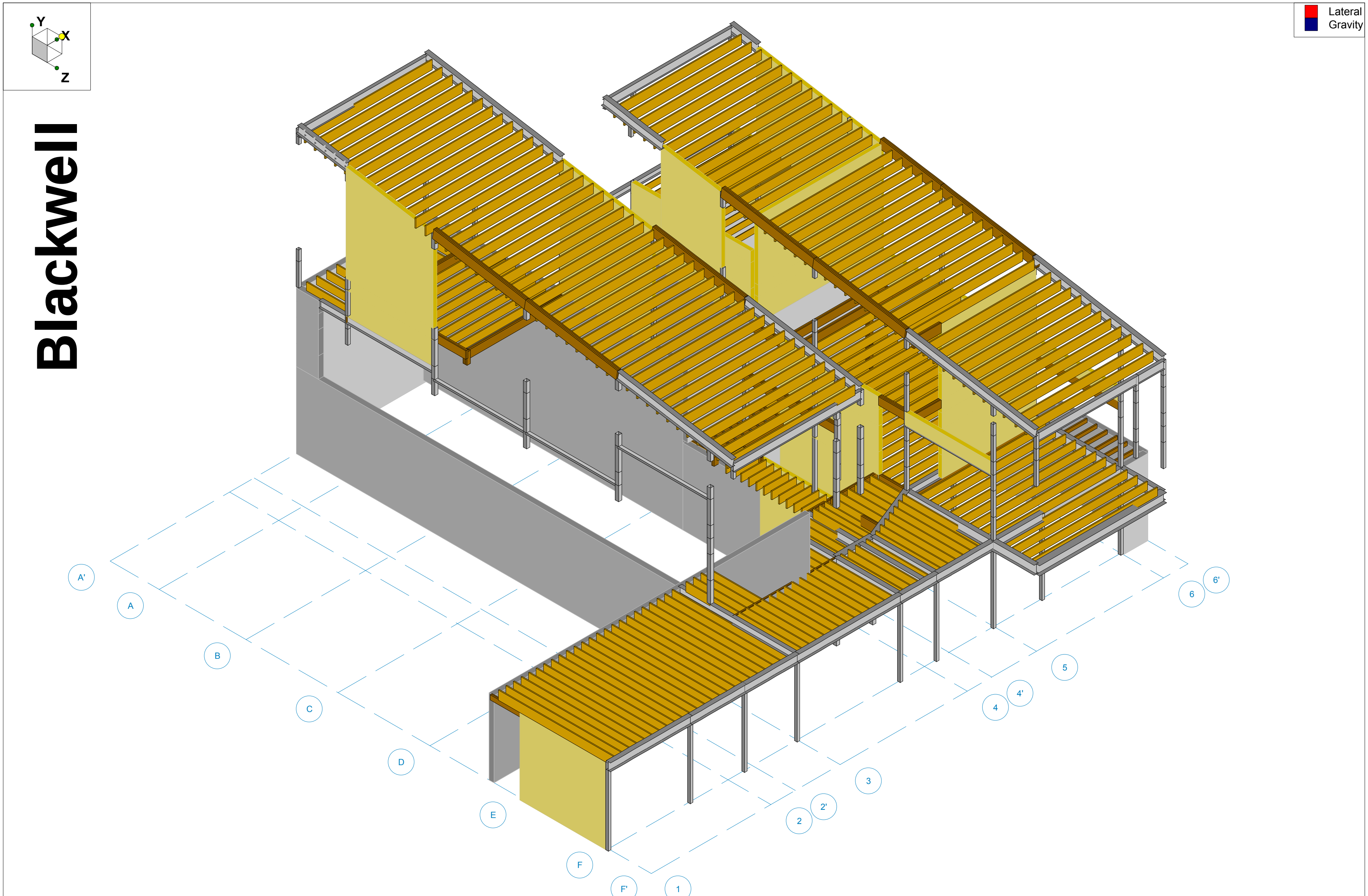
Structural Engineers

VOLUME 2, 3 & 4
(Dining, Master, Basement)



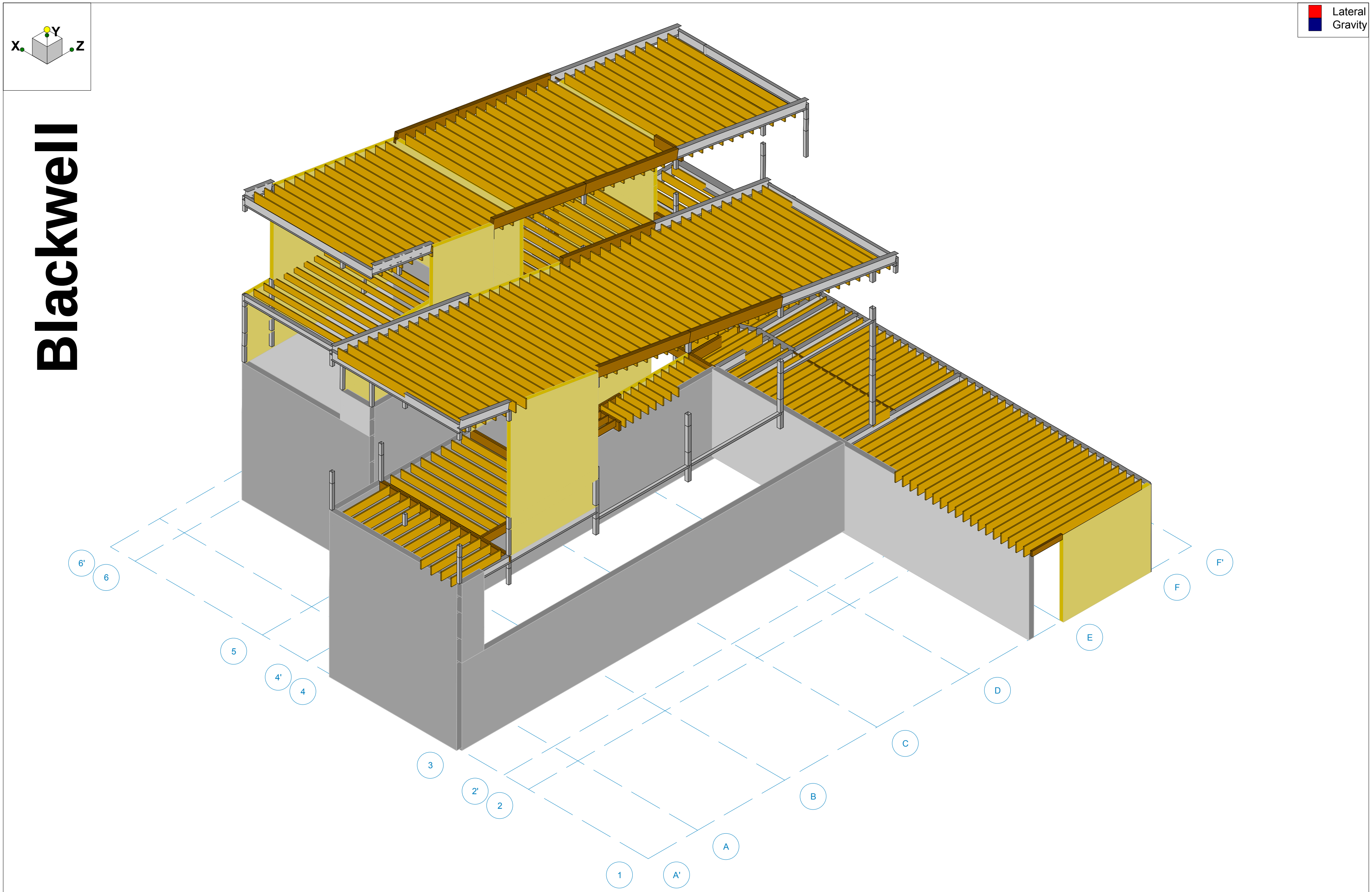
*RENDERED VIEW SHOWN FOR CONTEXT ONLY. REFER TO MEMBER PROPERTIES AND STRUCTURAL DRAWINGS FOR DETAILS.

Blackwell Structural Engineers	Full Model	GENERAL RENDER 1
BG	Kimmelman May Residence Volume 2, 3 and 4	
170266		



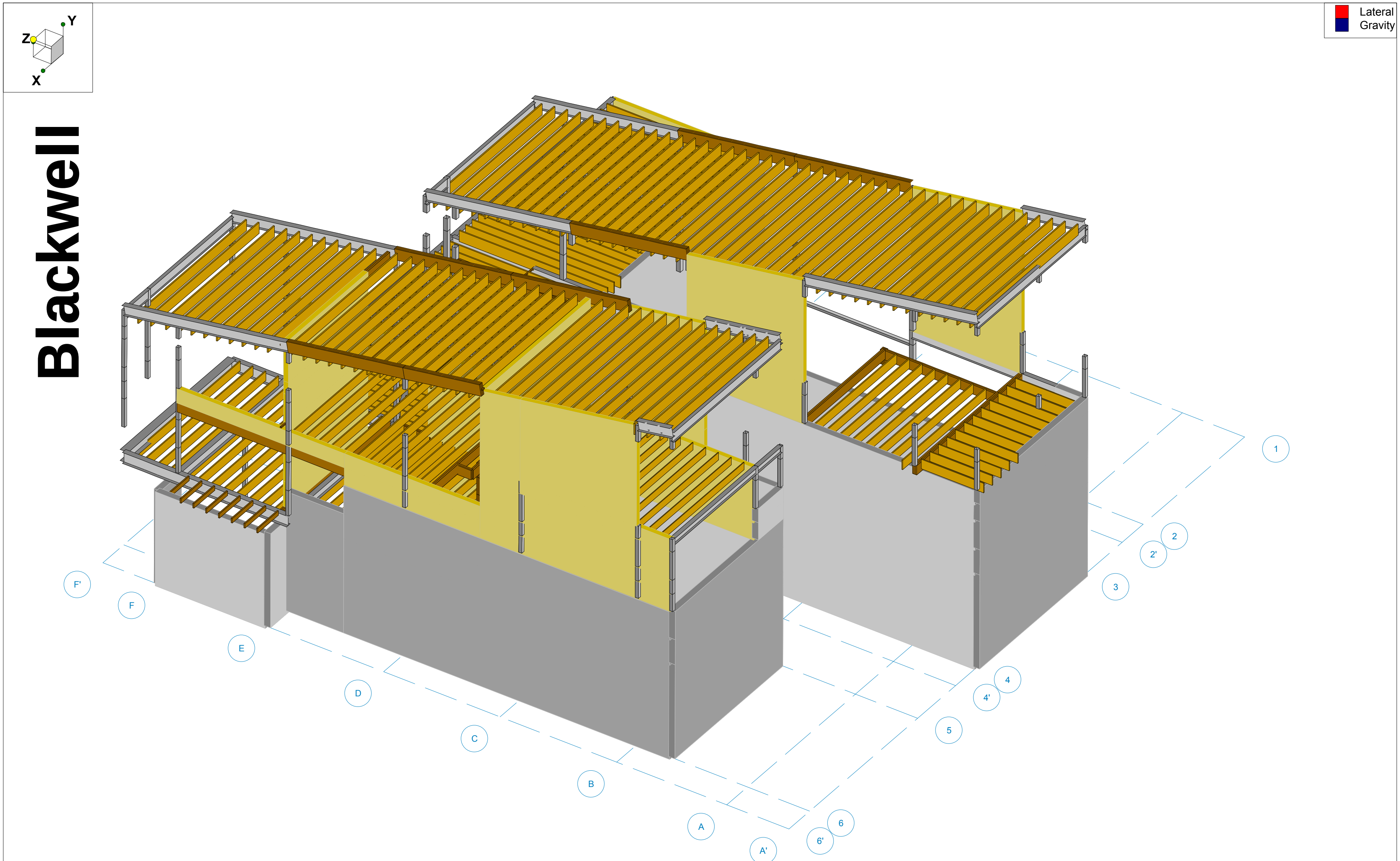
*RENDERED VIEW SHOWN FOR CONTEXT ONLY. REFER TO MEMBER PROPERTIES AND STRUCTURAL DRAWINGS FOR DETAILS.

Blackwell Structural Engineers	Full Model	GENERAL RENDER 2
BG	Kimmelman May Residence Volume 2, 3 and 4	
170266		



*RENDERED VIEW SHOWN FOR CONTEXT ONLY. REFER TO MEMBER PROPERTIES AND STRUCTURAL DRAWINGS FOR DETAILS.

Blackwell Structural Engineers	Full Model	GENERAL RENDER 3
BG	Kimmelman May Residence Volume 2, 3 and 4	
170266		

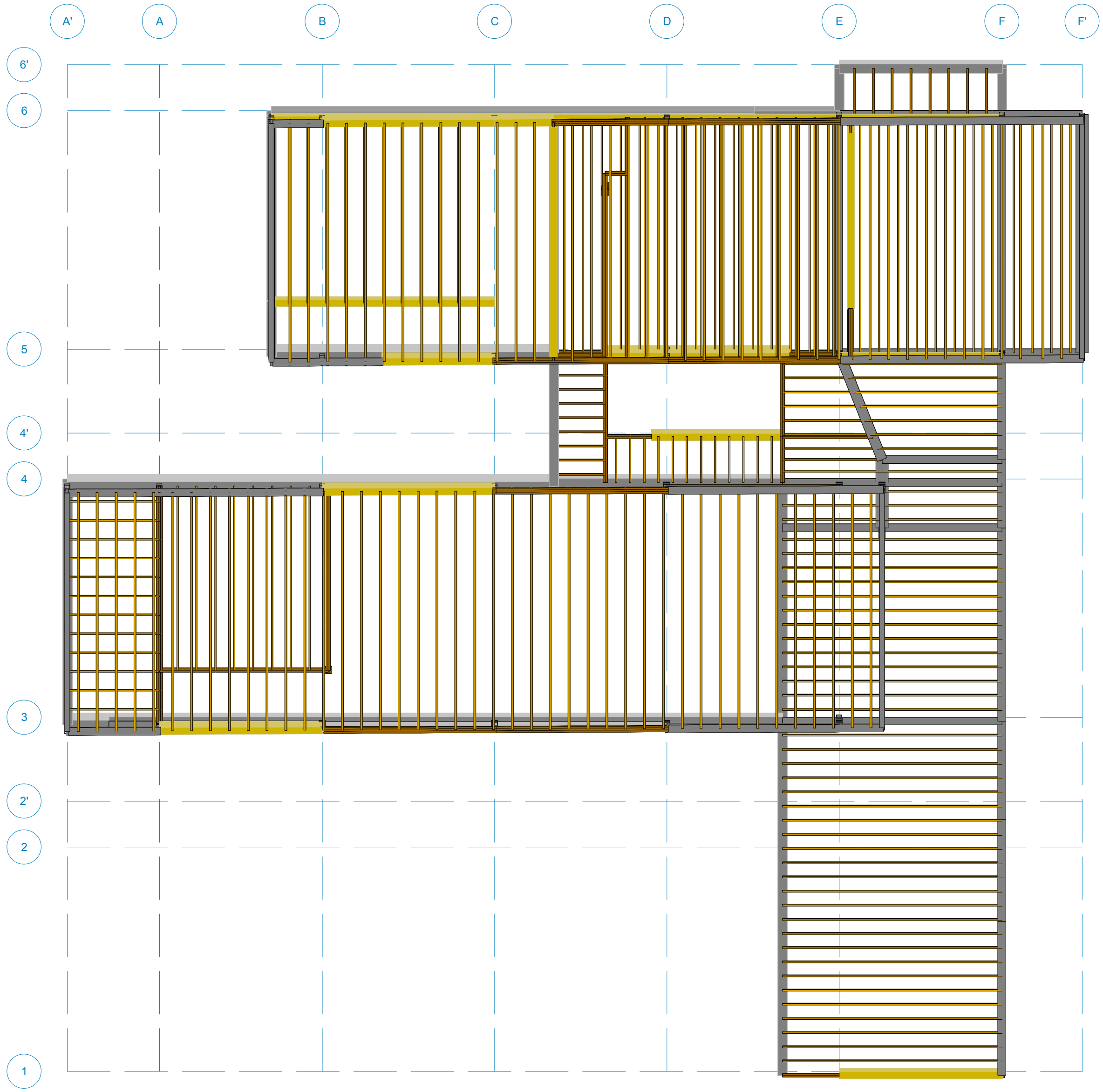


*RENDERED VIEW SHOWN FOR CONTEXT ONLY. REFER TO MEMBER PROPERTIES AND STRUCTURAL DRAWINGS FOR DETAILS.

Blackwell Structural Engineers	Full Model	GENERAL RENDER 4
BG	Kimmelman May Residence Volume 2, 3 and 4	
170266		

Lateral
Gravity

Blackwell



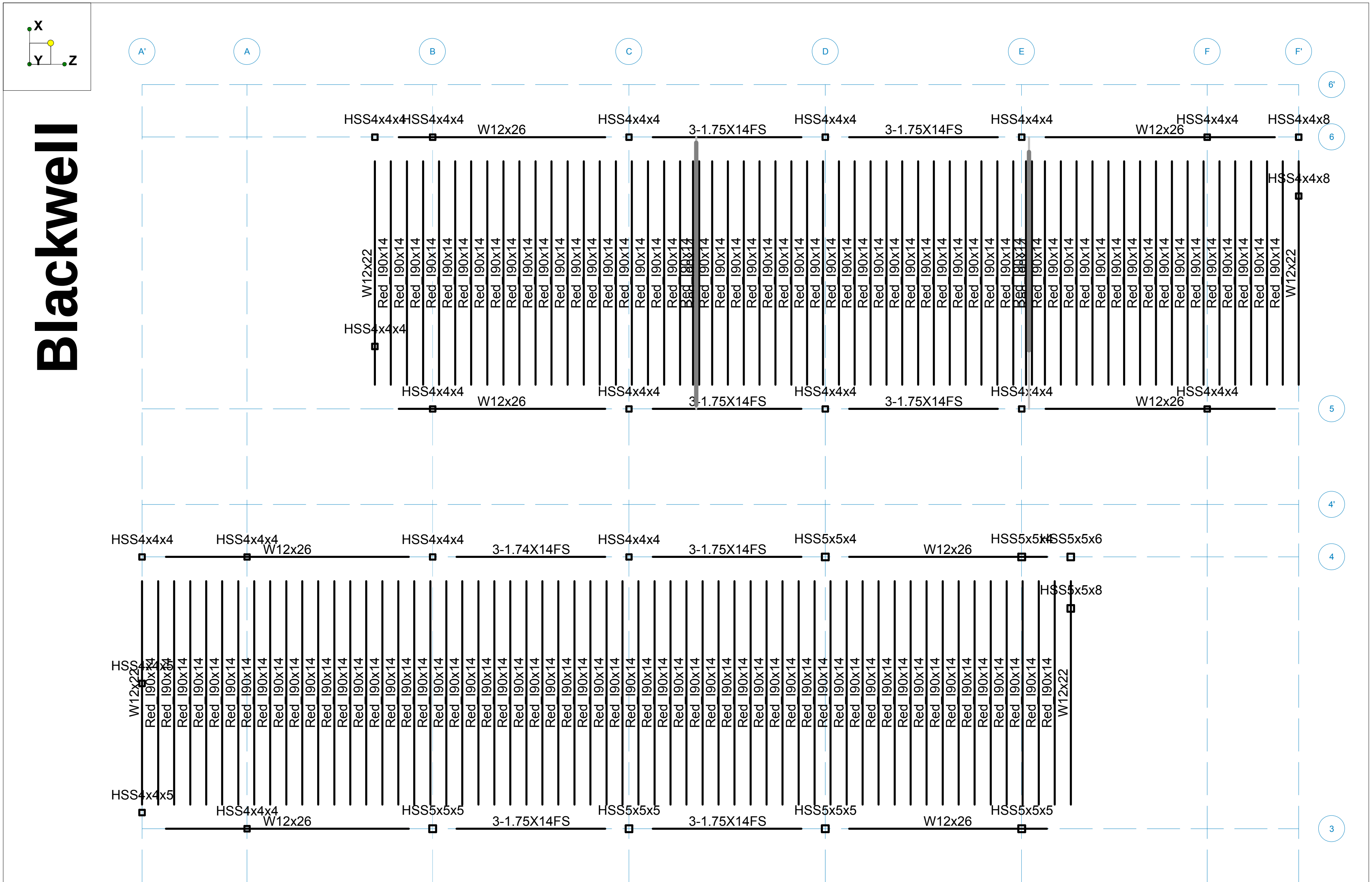
*RENDERED VIEW SHOWN FOR CONTEXT ONLY. REFER TO MEMBER PROPERTIES AND STRUCTURAL DRAWINGS FOR DETAILS.

Blackwell Structural Engineers	Full Model Kimmelman May Residence Volume 2, 3 and 4	GENERAL RENDER 5
BG		
170266		

Blackwell
Structural Engineers

GRAVITY SYSTEM
Designed using RISAFloor

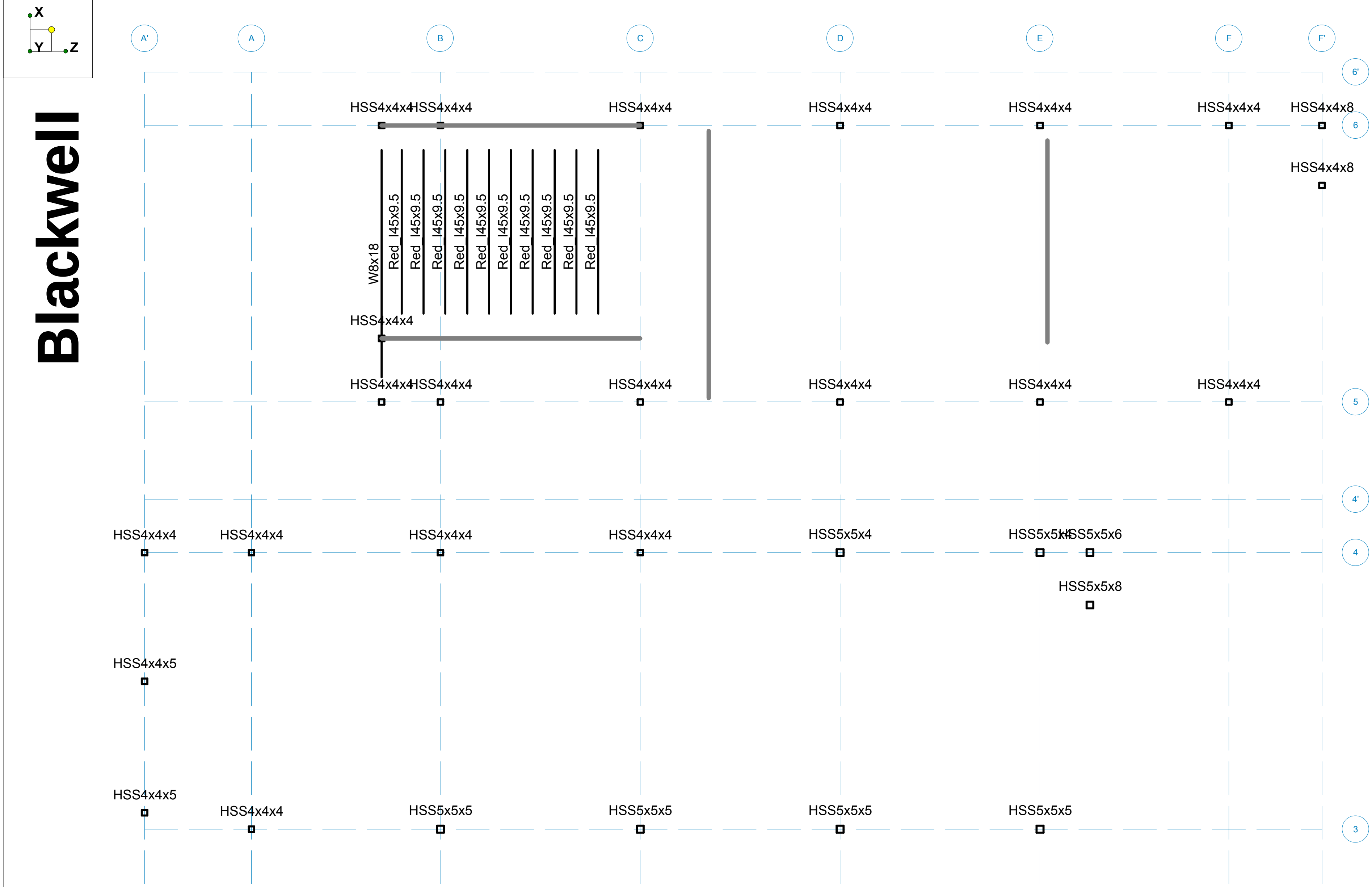
Gravity Geometry and Shapes Definition



Note due to modeling requirements some members are only modeled and designed in RISA3D

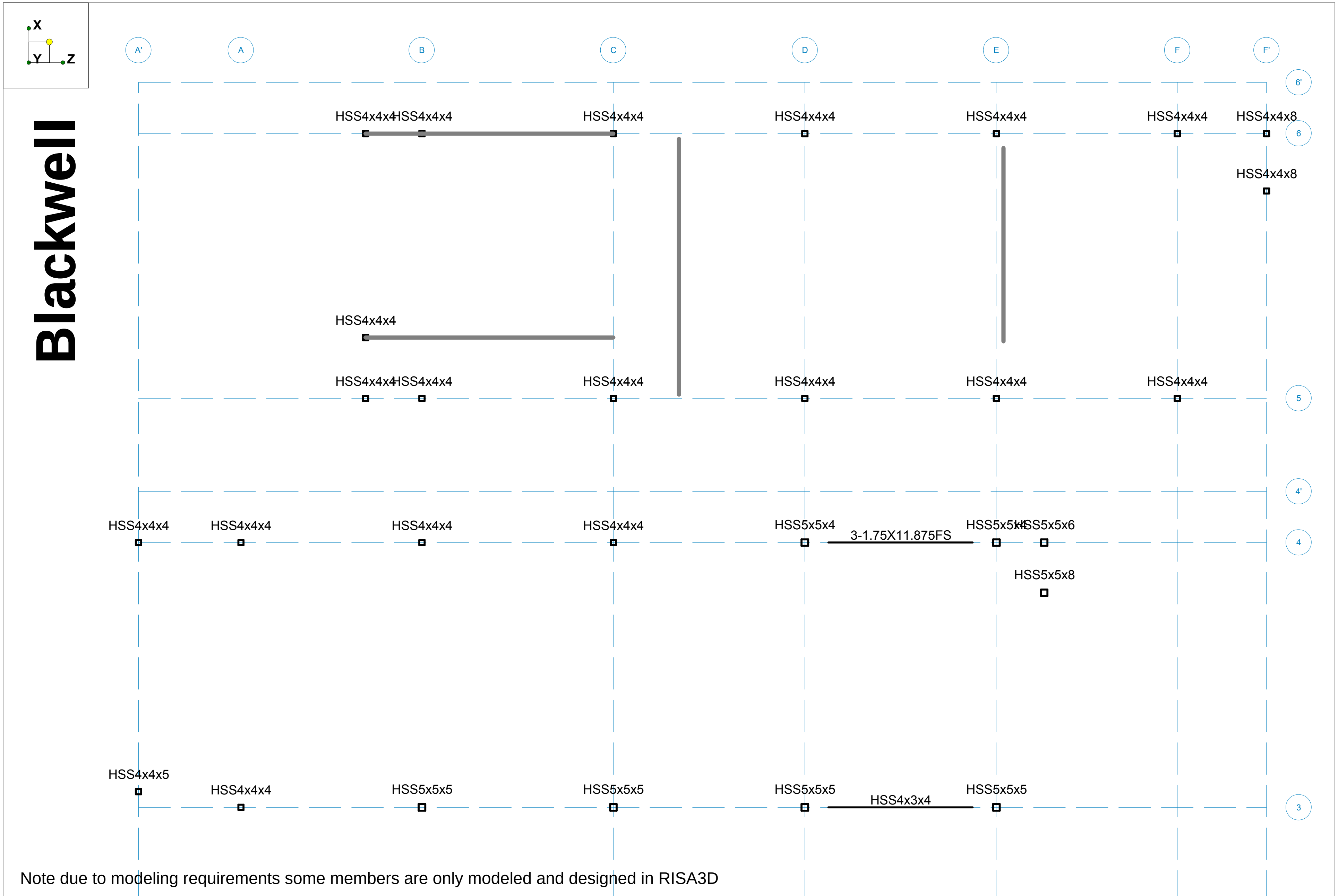
Blackwell Structural Engineers	25' 1/4" Roof	SHAPES
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:09 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.r3d

Blackwell



Note due to modeling requirements some members are only modeled and designed in RISA3D

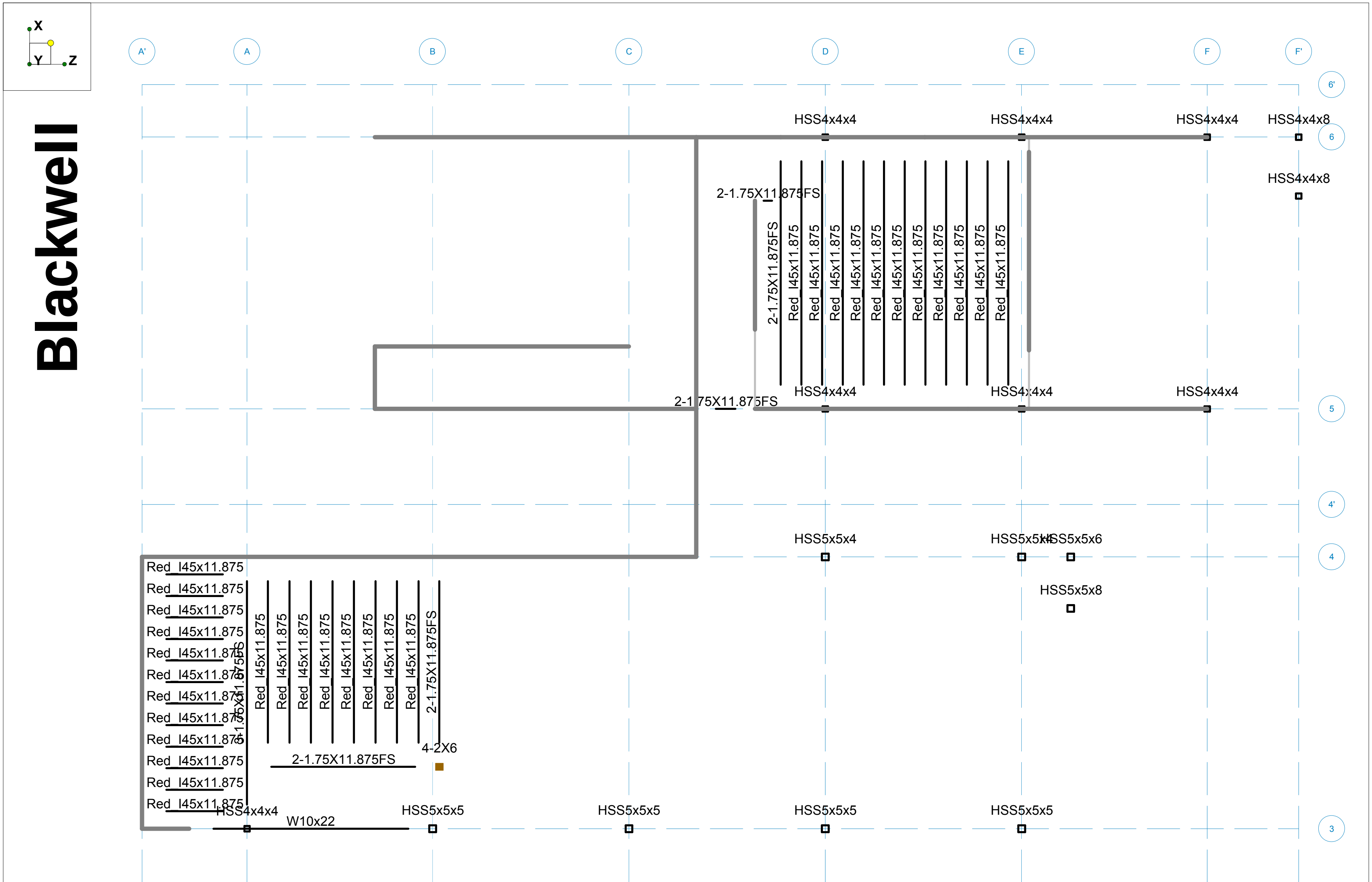
Blackwell Structural Engineers	23'-4" V3 Nanny	SHAPES
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:09 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	22' V2 Top of Window	SHAPES
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:09 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

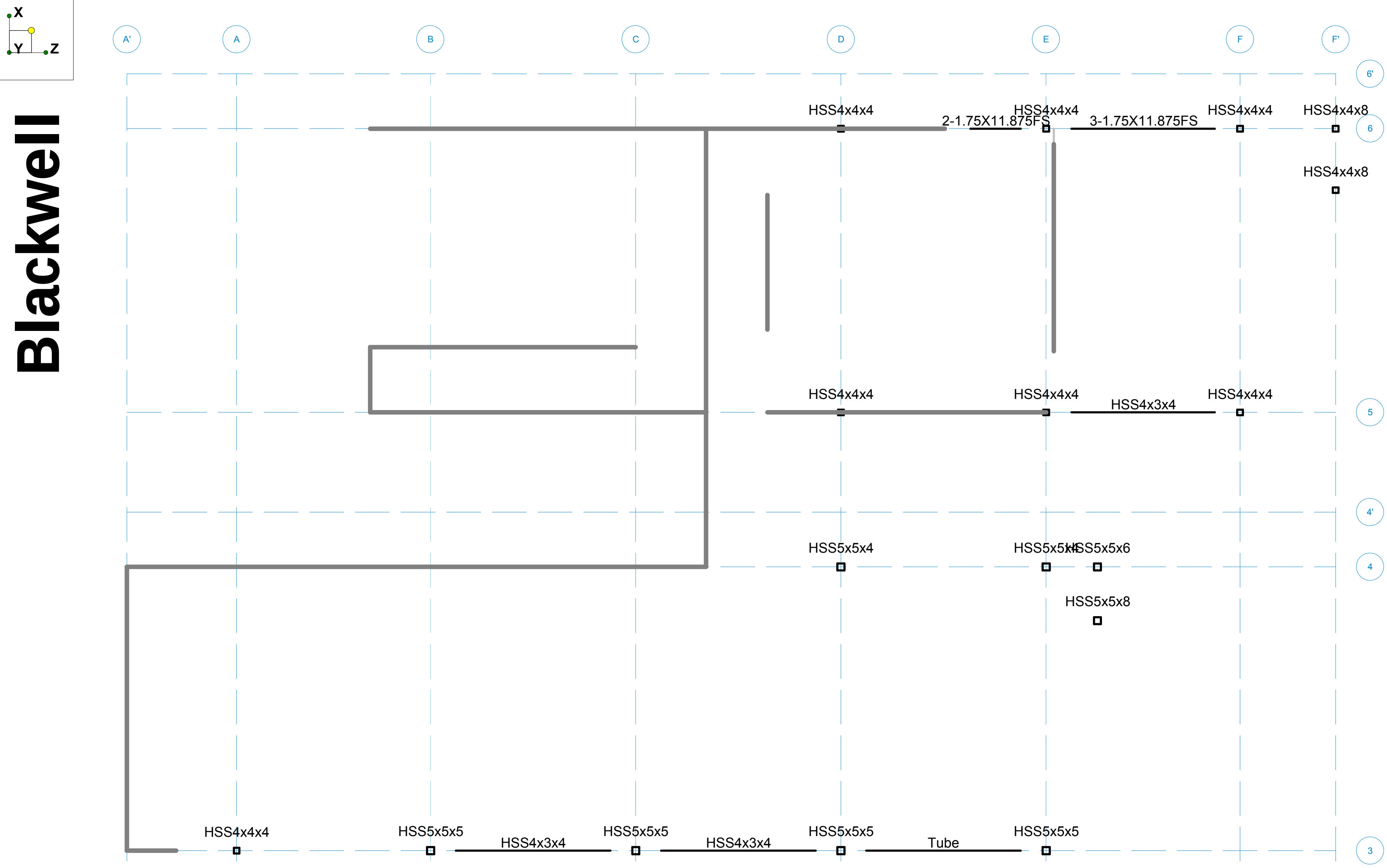
Blackwell



Note due to modeling requirements some members are only modeled and designed in RISA3D

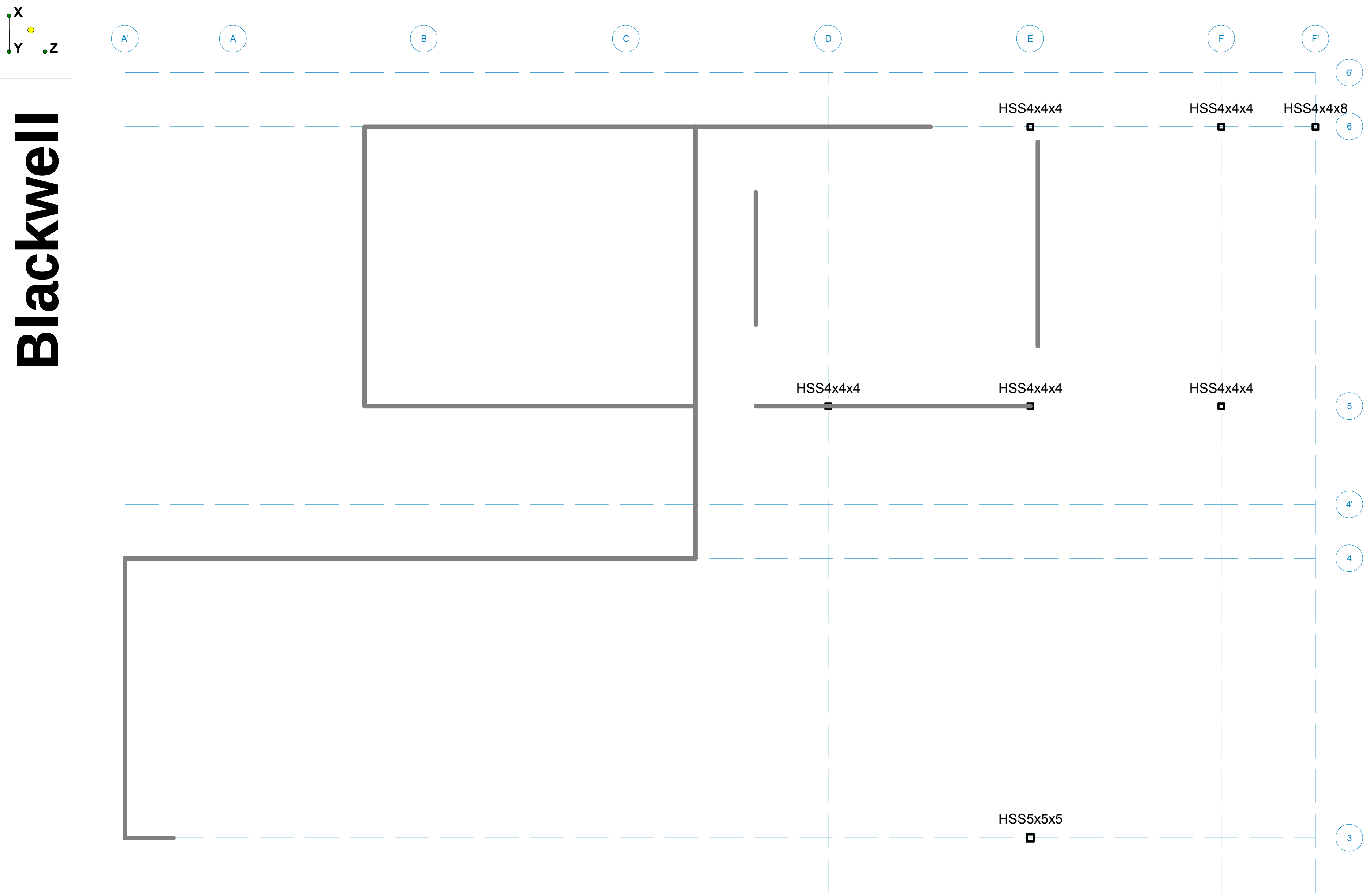
Blackwell Structural Engineers	19' V1 Mezz V3 Study	SHAPES
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:11 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

Blackwell



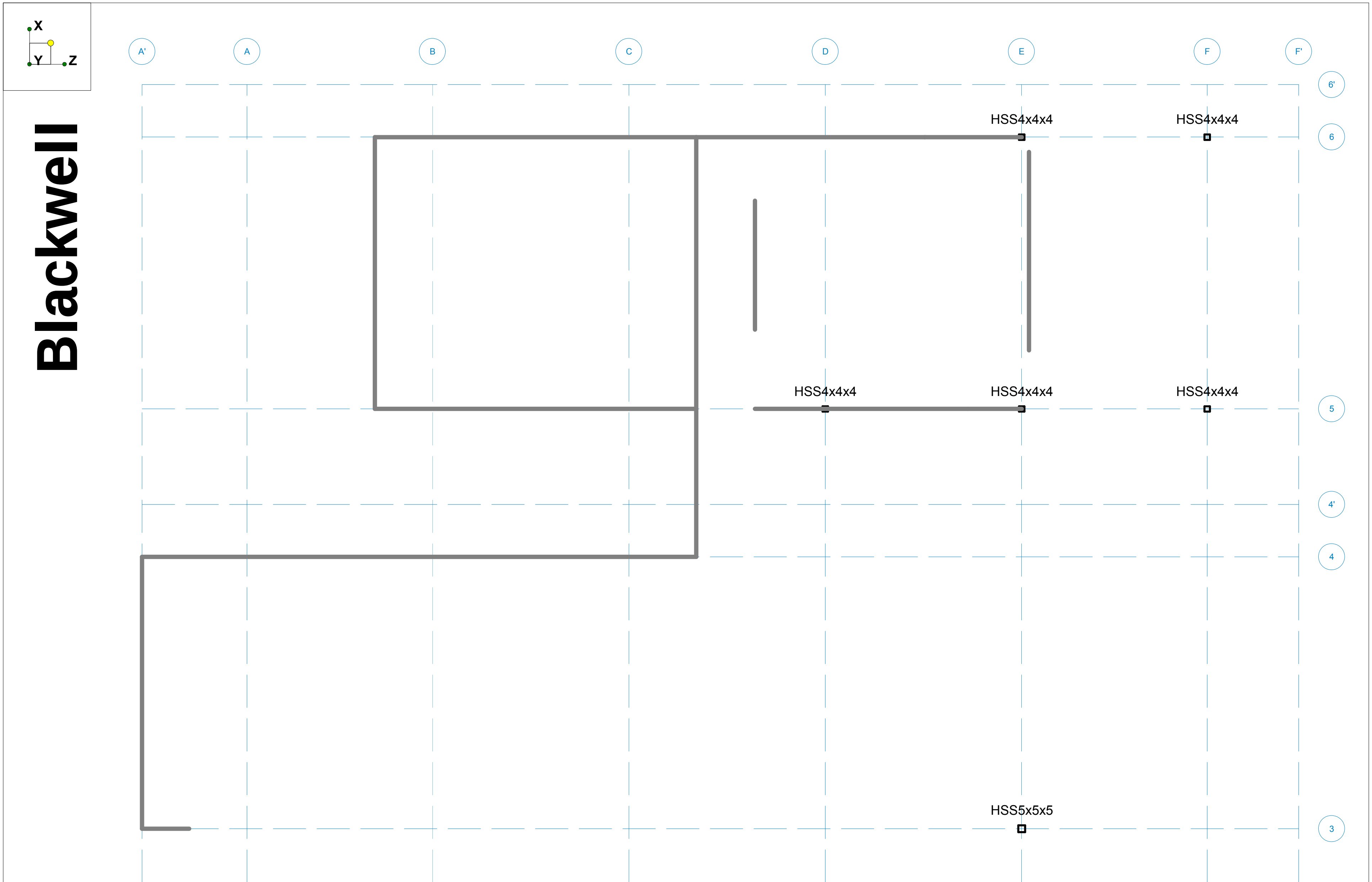
Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	17.5' V2 V3 Top of (low) windows Kimmelman May Residence Volume 2, 3 and 4	SHAPES
BG		Feb 15, 2018 at 4:10 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	15'-8"	SHAPES
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:10 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

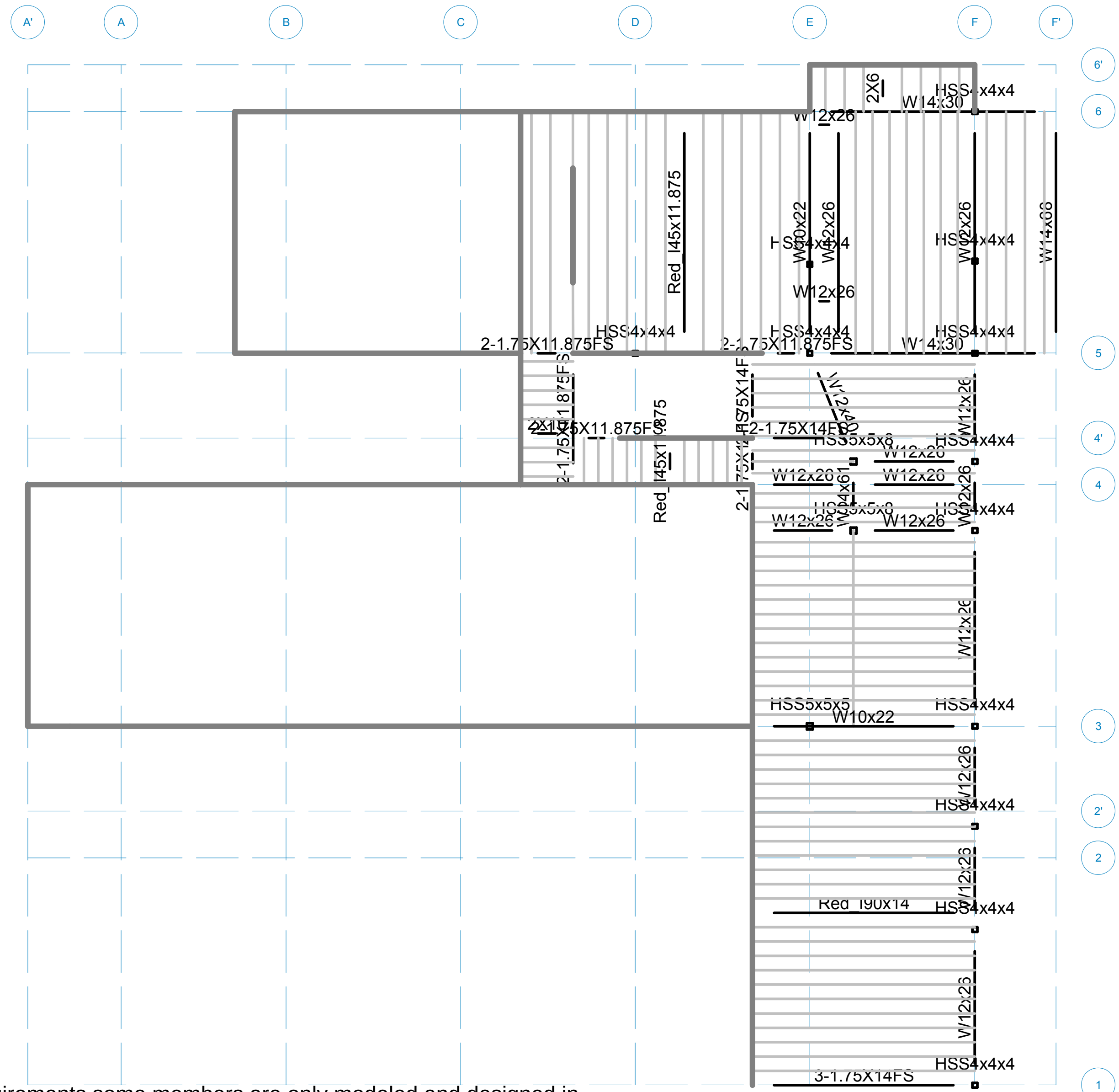


Blackwell

Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	12'-8"	SHAPES
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:10 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

Blackwell

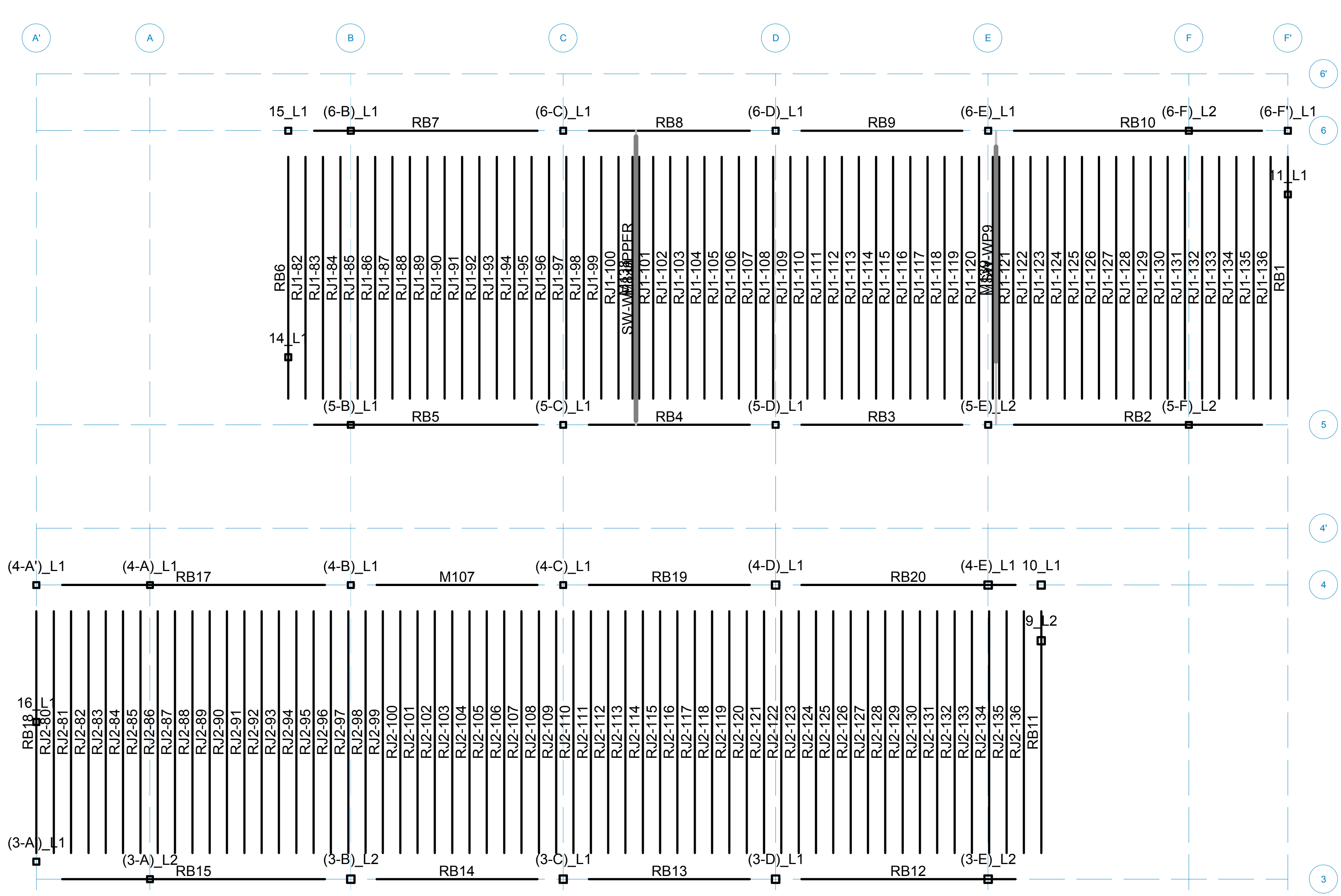


Note due to modeling requirements some members are only modeled and designed in RISA3D. Not all joists shown for clarity.

Blackwell Structural Engineers	10' Plaza Level	SHAPES
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:35 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

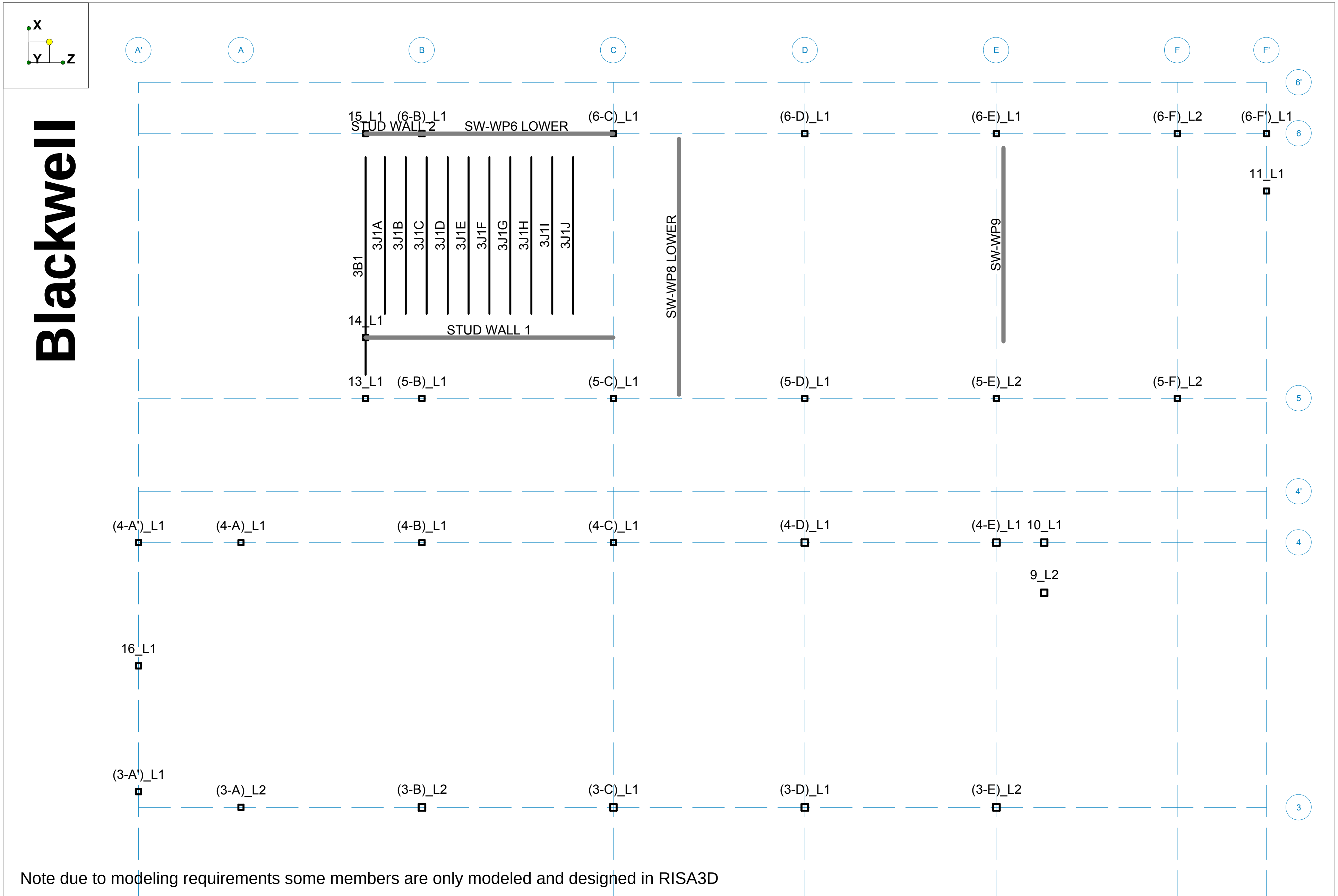
Gravity Wall and Member Designation

Blackwell



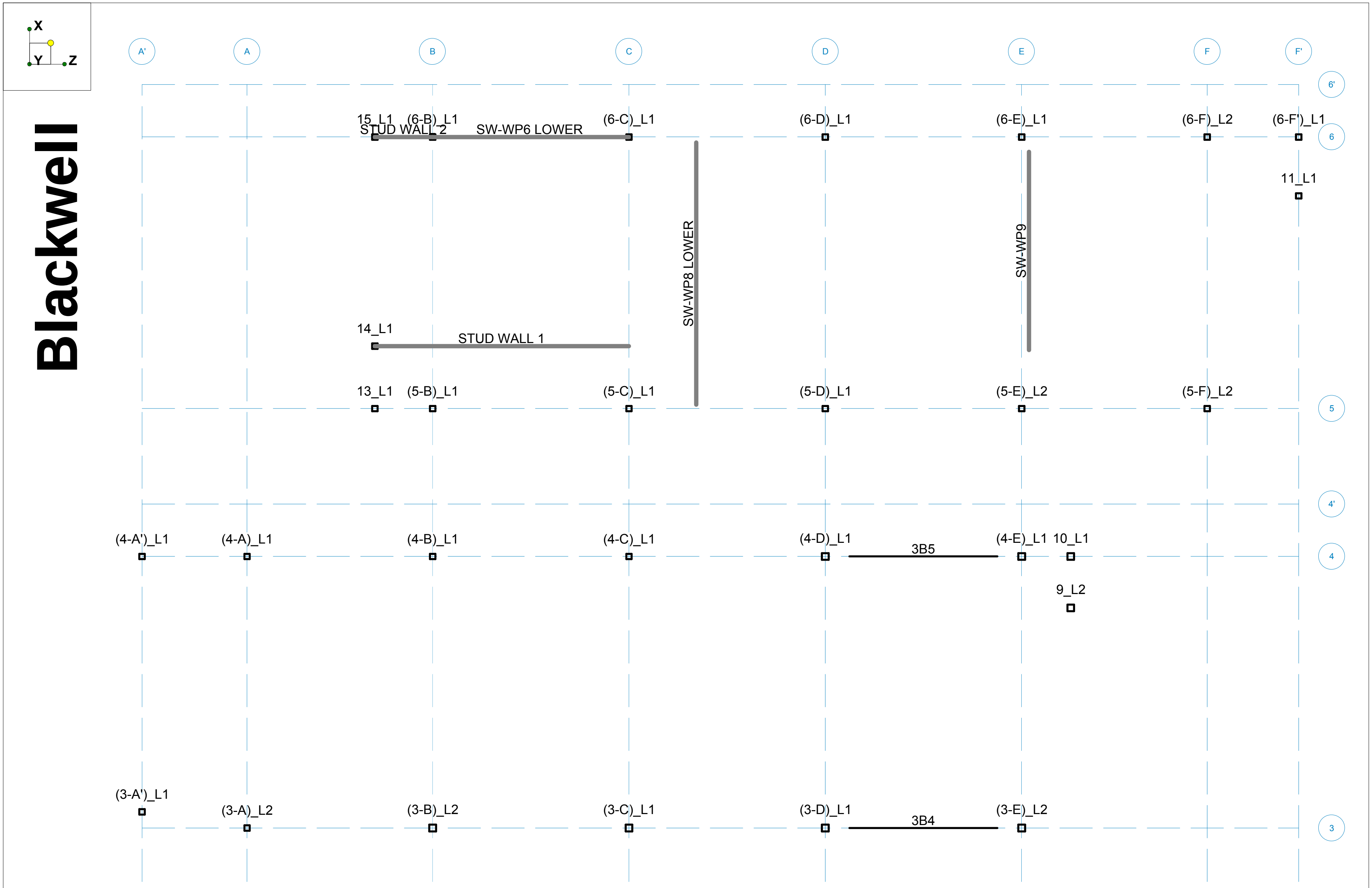
Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	25' 1/4" Roof	MEMBER DESIGNATIONS
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:12 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



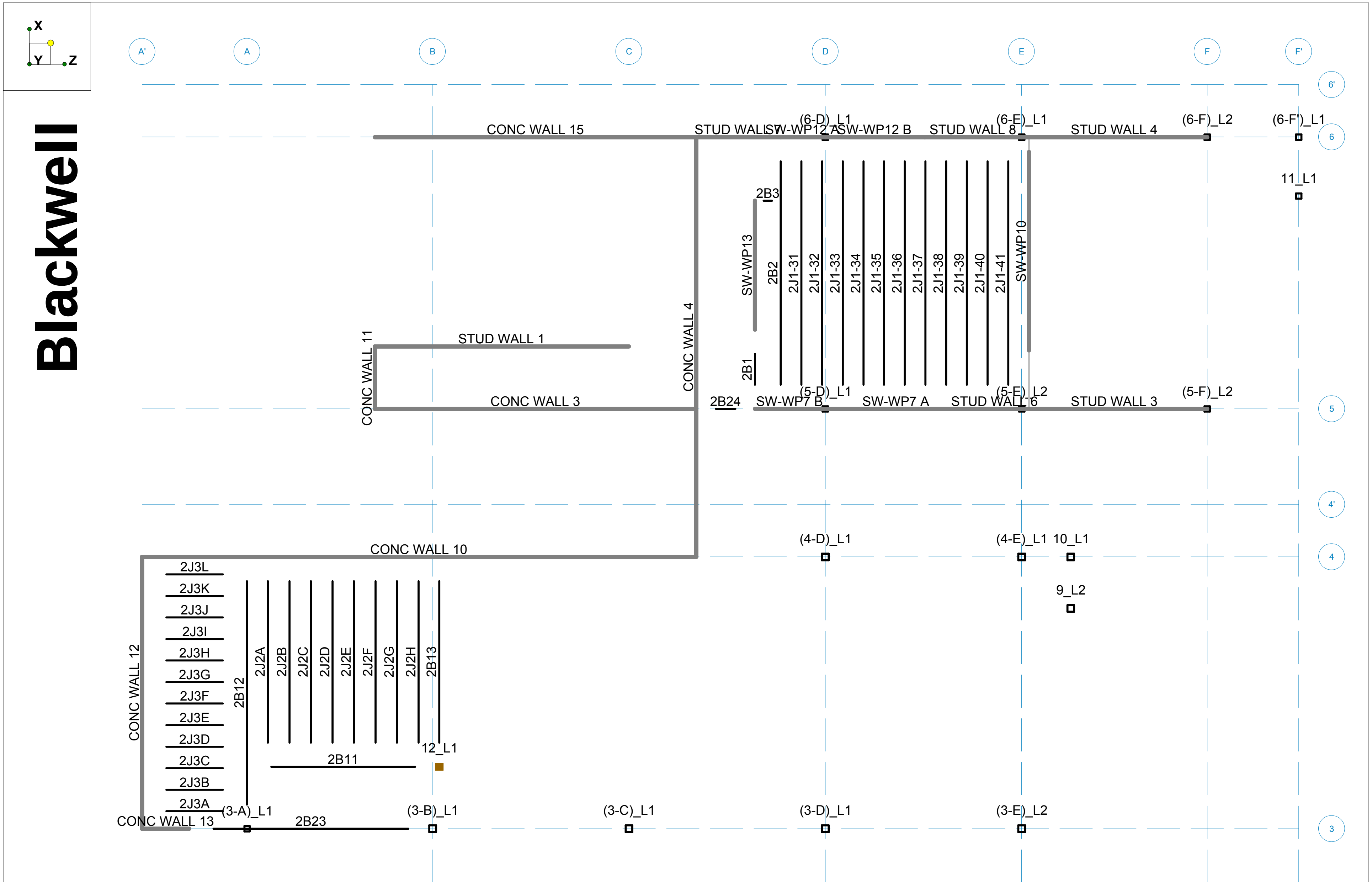
Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	23'-4" V3 Nanny	MEMBER DESIGNATIONS
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:12 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



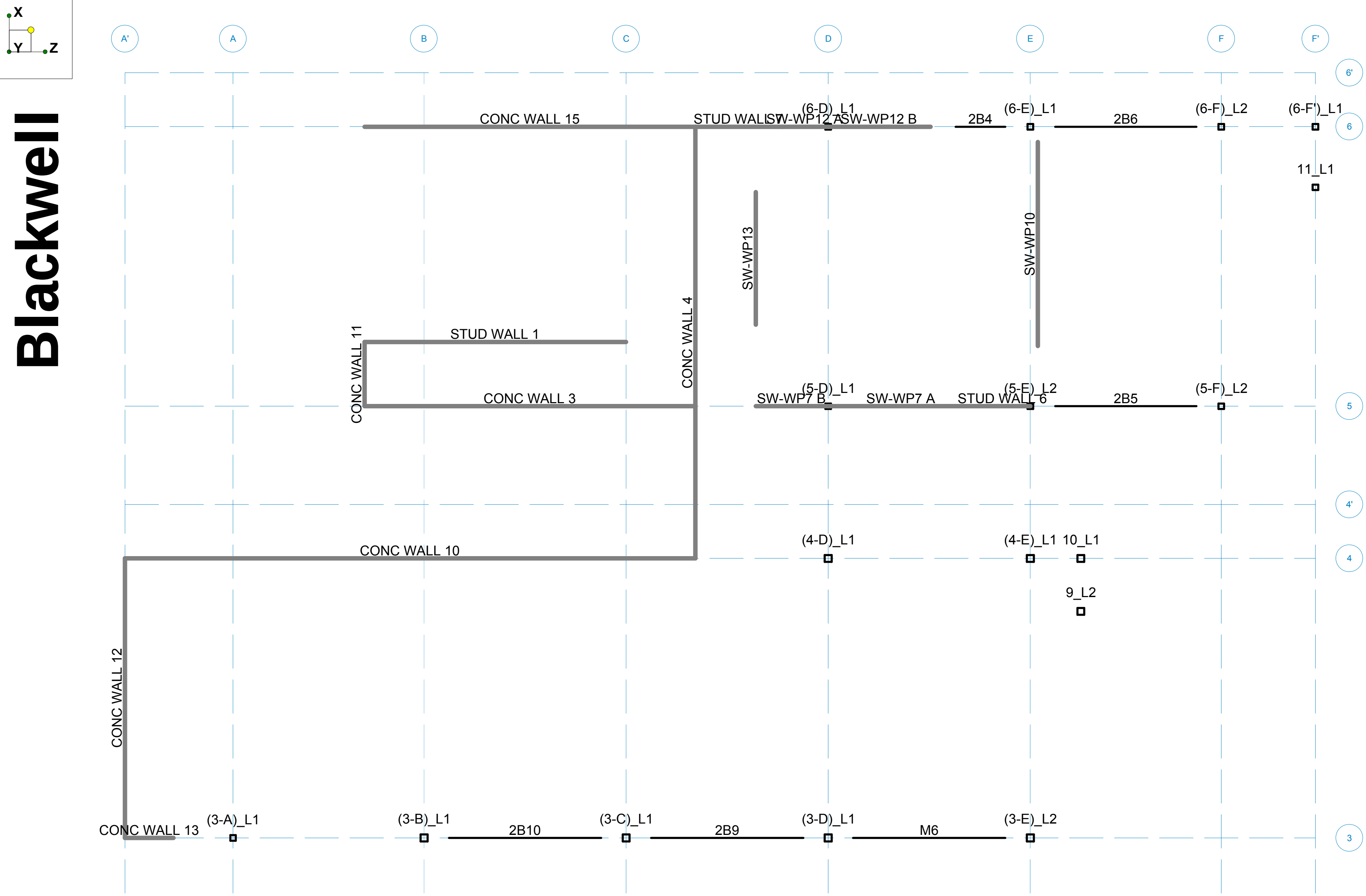
Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	22' V2 Top of Window	MEMBER DESIGNATIONS
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:12 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



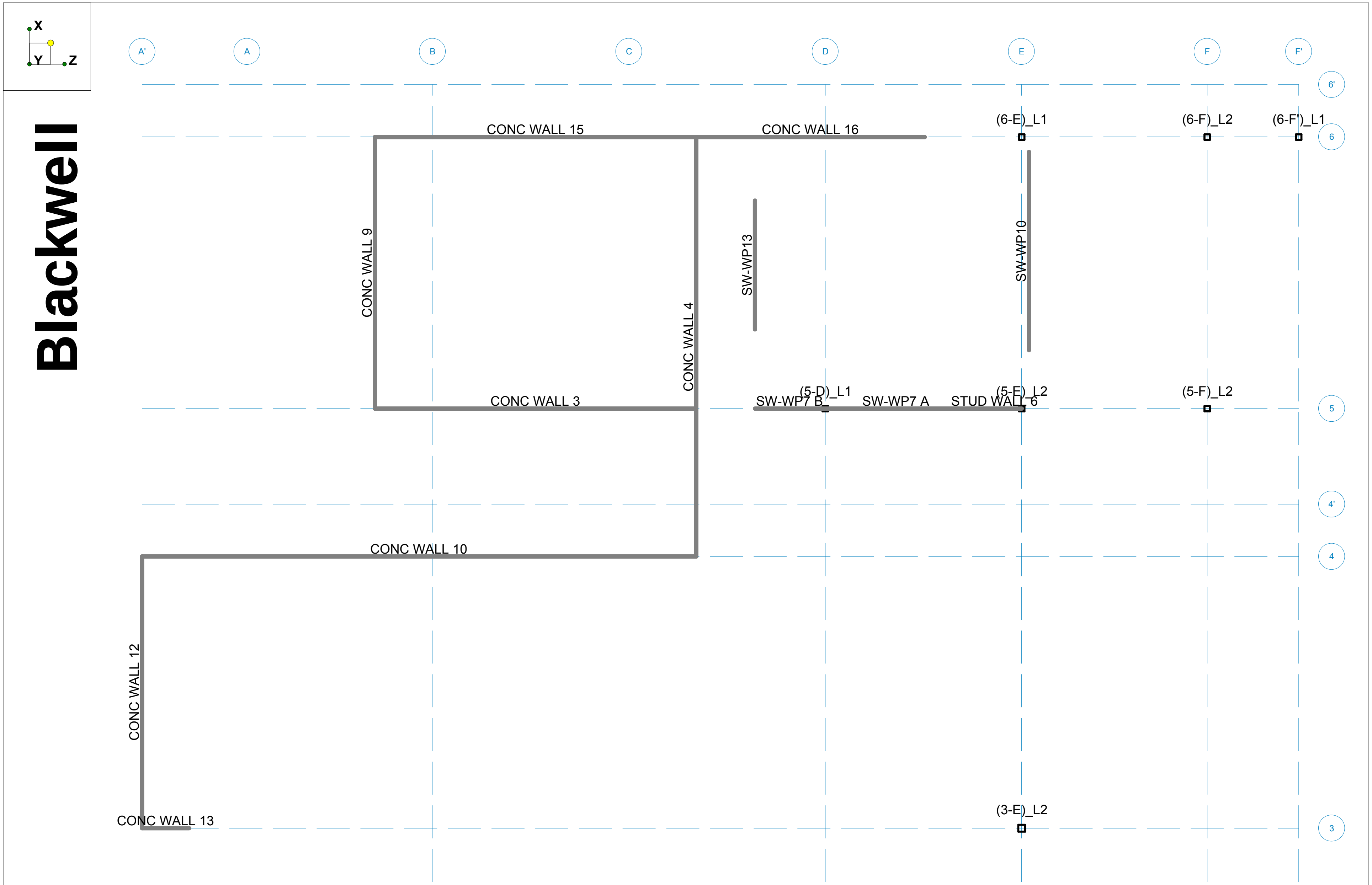
Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	19' V1 Mezz V3 Study	MEMBER DESIGNATIONS
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:12 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



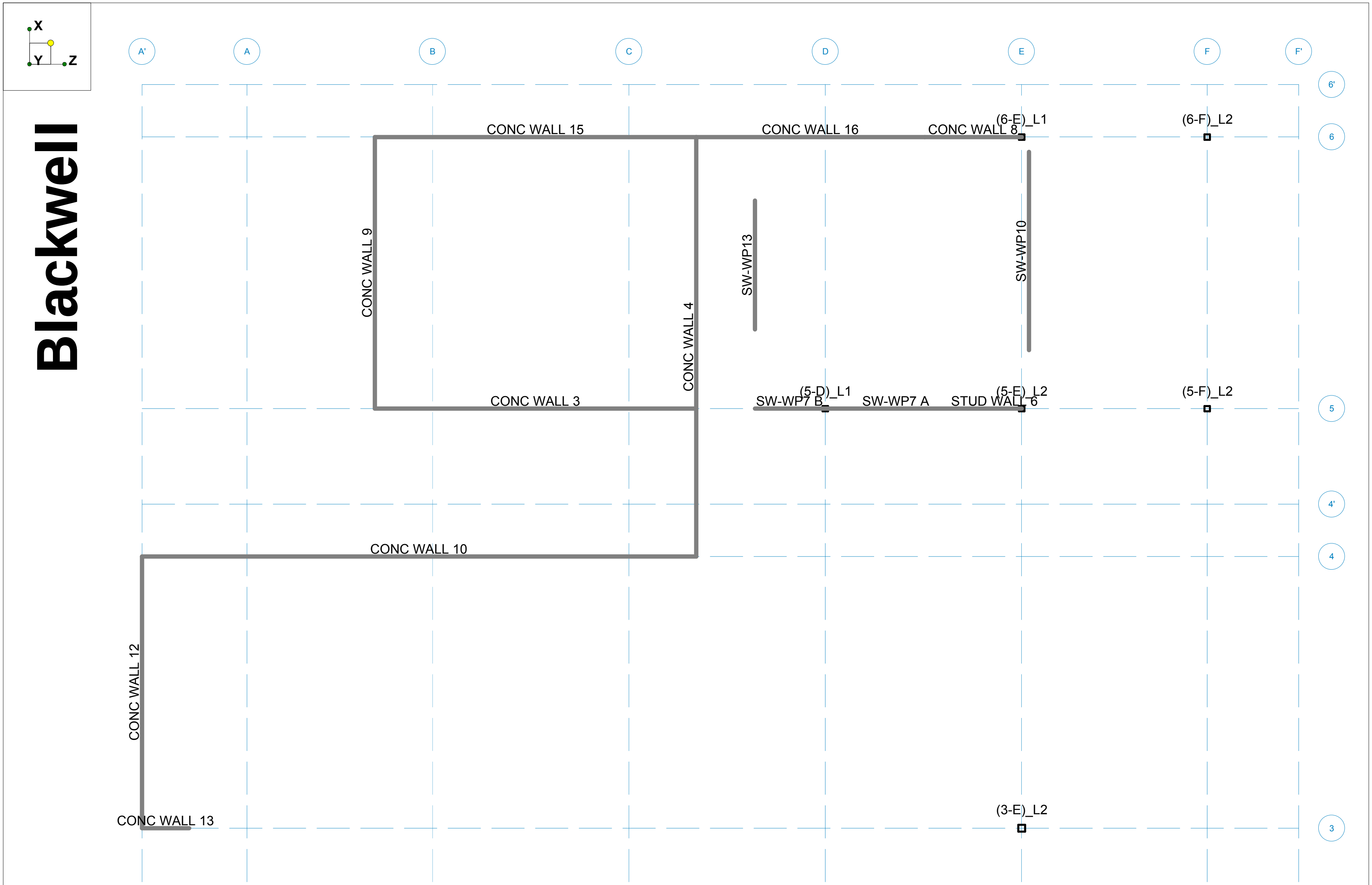
Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	17.5' V2 V3 Top of (low) windows	MEMBER DESIGNATIONS
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:13 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	15'-8"	MEMBER DESIGNATIONS
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:13 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Note due to modeling requirements some members are only modeled and designed in RISA3D

Blackwell Structural Engineers	12'-8"	MEMBER DESIGNATIONS
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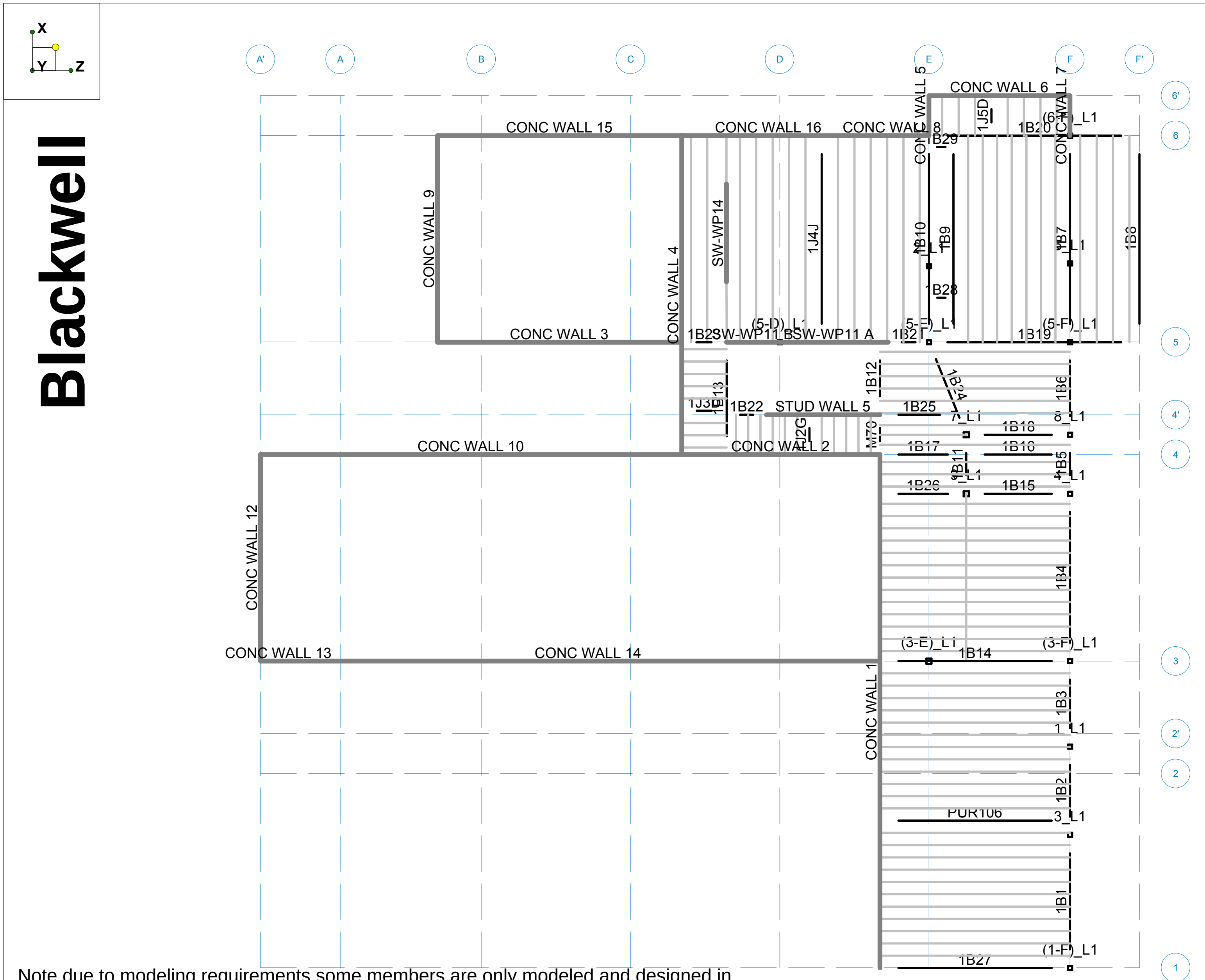
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Note due to modeling requirements some members are only modeled and designed in RISA3D. Not all joists shown for clarity.

Blackwell Structural Engineers	10' Plaza Level	MEMBER DESIGNATION
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:36 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.r3d

Column Stacks

	Stack L...	Project Grid	Z [ft]	X [ft]	Lift ...	Leng...	Bot E...	Top ...	Shape	Material	Func...	Design...	Flexural L...	Shear La...
1	(5-E)	5-E	54.5	51	1	10.216	0	10.216	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
2					2	17.708	10.216	27.924	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
3	(5-F)	5-F	66	51	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
4					2	16.007	10	26.007	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
5	(1-F)	1-F	66	0	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
6	1	-	66	18.0...	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
7	(3-F)	3-F	66	25	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
8	2	-	54.5	57.1...	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
9	3	-	66	10.8...	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
10	4	-	66	38.6...	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
11	5	-	66	57.4...	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
12	(5-D)	5-D	42.3...	51	1	29.952	0	29.952	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
13	(6-F)	6-F	66	67.8...	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
14					2	16.007	10	26.007	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
15	(3-E)	3-E	54.5	25	1	10	0	10	HSS5x5x5	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
16					2	17.924	10	27.924	HSS5x5x5	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
17	7	-	57.5...	43.4...	1	10	0	10	HSS5x5x8	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
18	8	-	66	43.4...	1	10	0	10	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
19	9	-	57.5...	38.6...	1	10	0	10	HSS5x5x8	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
20					2	17.417	10	27.417	HSS5x5x8	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
21	(6-E)	6-E	54.5	67.8...	1	17.924	10	27.924	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
22	(6-F')	6-F'	71.6...	67.8...	1	15.063	10	25.063	HSS4x4x8	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
23	(6-D)	6-D	42.3...	67.8...	1	14.285	15.667	29.952	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
24	(3-A)	3-A	6.5	25	1	9	10	19	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
25					2	16.924	19	35.924	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
26	(3-B)	3-B	18	25	1	9.05	10	19.05	HSS5x5x5	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
27					2	14.957	19.05	34.008	HSS5x5x5	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
28	(3-C)	3-C	30.1...	25	1	21.98	10	31.98	HSS5x5x5	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
29	(3-D)	3-D	42.3...	25	1	19.952	10	29.952	HSS5x5x5	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
30	(4-D)	4-D	42.3...	41.8...	1	19.952	10	29.952	HSS5x5x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
31	(4-E)	4-E	54.5	41.8...	1	17.924	10	27.924	HSS5x5x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
32	10	-	57.5...	41.8...	1	17.417	10	27.417	HSS5x5x6	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
33	11	-	71.6...	64.1...	1	15.063	10	25.063	HSS4x4x8	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
34	12	-	18.4...	28.8...	1	9	10	19	4-2X6	Spruce-Pine-Fir	Gravity	Typical	N/A	N/A
35	(5-B)	5-B	18	51	1	15.007	19	34.008	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
36	(5-C)	5-C	30.1...	51	1	12.98	19	31.98	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
37	(3-A')	-	0	26	1	18.007	19	37.008	HSS4x4x5	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
38	(4-A')	4-A'	0	41.8...	1	18.007	19	37.008	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
39	(4-A)	4-A	6.5	41.8...	1	16.924	19	35.924	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
40	(4-B)	4-B	18	41.8...	1	15.007	19	34.008	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
41	(4-C)	4-C	30.1...	41.8...	1	12.98	19	31.98	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
42	13	-	14.4...	51	1	4.333	19	23.333	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
43	14	-	14.4...	54.8...	1	15.605	19	34.605	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
44	15	-	14.4...	67.8...	1	15.605	19	34.605	HSS4x4x4	A500 Gr.B Rect	Lateral	Typical	N/A	N/A
45	(6-B)	6-B	18	67.8...	1	15.007	19	34.008	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
46	(6-C)	6-C	30.1...	67.8...	1	12.98	19	31.98	HSS4x4x4	A500 Gr.B Rect	Gravity	Typical	N/A	N/A
47	16	-	0	34	1	18.007	19	37.008	HSS4x4x5	A500 Gr.B Rect	Lateral	Typical	N/A	N/A

Beam Primary Data : 25' 1/4" Roof

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
1	RB19	N154	N156	3-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
2	RB15	N246	N151	W12x26	A992	Typical	Gravity	Strong Axis	Pinned	Pinned	
3	RB14	N151	N153	3-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
4	RB13	N153	N155	3-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
5	RB11	N159	N160	W12x22	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
6	RB18	N246	N148	W12x22	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
7	RB1	N90	N95	W12x22	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
8	RB6	N247	N248	W12x22	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
9	RB20	N156	N160	W12x26	A992	Typical	Gravity	Strong Axis	Pinned	Pinned	
10	RB9	N87	N88	3-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
11	RB4	N92	N91	3-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
12	RB8	N86	N87	3-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
13	RB17	N152	N148	W12x26	A992	Typical	Gravity	Strong Axis	Pinned	Pinned	
14	RB5	N91	N248	W12x26	A992	Typical	Gravity	Strong Axis	Pinned	Pinned	
15	RB12	N155	N159	W12x26	A992	Typical	Gravity	Strong Axis	Pinned	Fixed	
16	RB10	N88	N90	W12x26	A992	Typical	Gravity	Strong Axis	Pinned	Pinned	
17	M120	N244	N203	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
18	M121	N245	N204	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
19	M107	N154	N152	3-1.74X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
20	RB7	N86	N247	W12x26	A992	Typical	Gravity	Strong Axis	Pinned	Pinned	
21	M94	N202	N342	2-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
22	RJ2-80	N178	N179	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
23	RJ2-81	N180	N181	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
24	RJ2-82	N182	N183	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
25	RJ2-83	N184	N185	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
26	RJ2-84	N186	N193	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
27	RJ2-85	N194	N195	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
28	RJ2-86	N196	N197	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
29	RJ2-87	N198	N205	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
30	RJ2-88	N206	N207	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
31	RJ2-89	N208	N209	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
32	RJ2-90	N210	N211	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
33	RJ2-91	N212	N213	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
34	RJ2-92	N214	N215	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
35	RJ2-93	N216	N217	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
36	RJ2-94	N218	N219	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
37	RJ2-95	N220	N221	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
38	RJ2-96	N222	N223	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
39	RJ2-97	N224	N225	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
40	RJ2-98	N226	N262	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
41	RJ2-99	N263	N264	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
42	RJ2-100	N265	N266	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
43	RJ2-101	N267	N268	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
44	RJ2-102	N269	N270	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
45	RJ2-103	N271	N272	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
46	RJ2-104	N273	N274	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
47	RJ2-105	N275	N276	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
48	RJ2-106	N277	N278	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
49	RJ2-107	N279	N280	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
50	RJ2-108	N281	N282	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
51	RJ2-109	N283	N284	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
52	RJ2-110	N285	N286	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 25' 1/4" Roof (Continued)

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
53	RJ2-111	N287	N288	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
54	RJ2-112	N289	N290	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
55	RJ2-113	N291	N292	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
56	RJ2-114	N293	N294	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
57	RJ2-115	N295	N296	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
58	RJ2-116	N297	N298	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
59	RJ2-117	N299	N300	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
60	RJ2-118	N301	N302	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
61	RJ2-119	N303	N304	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
62	RJ2-120	N305	N306	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
63	RJ2-121	N307	N308	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
64	RJ2-122	N309	N310	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
65	RJ2-123	N311	N312	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
66	RJ2-124	N313	N314	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
67	RJ2-125	N315	N316	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
68	RJ2-126	N317	N318	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
69	RJ2-127	N319	N320	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
70	RJ2-128	N321	N322	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
71	RJ2-129	N323	N324	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
72	RJ2-130	N325	N326	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
73	RJ2-131	N327	N328	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
74	RJ2-132	N329	N330	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
75	RJ2-133	N331	N332	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
76	RJ2-134	N333	N334	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
77	RJ2-135	N335	N336	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
78	RJ2-136	N337	N338	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
79	M137	N339	N341	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
80	RJ1-82	N161	N162	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
81	RJ1-83	N163	N164	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
82	RJ1-84	N165	N166	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
83	RJ1-85	N167	N168	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
84	RJ1-86	N169	N170	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
85	RJ1-87	N171	N172	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
86	RJ1-88	N173	N174	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
87	RJ1-89	N175	N176	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
88	RJ1-90	N177	N187	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
89	RJ1-91	N188	N189	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
90	RJ1-92	N190	N191	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
91	RJ1-93	N192	N227	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
92	RJ1-94	N228	N229	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
93	RJ1-95	N230	N231	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
94	RJ1-96	N232	N233	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
95	RJ1-97	N234	N235	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
96	RJ1-98	N236	N237	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
97	RJ1-99	N238	N239	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
98	RJ1-100	N240	N241	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
99	RJ1-101	N249	N250	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
100	RJ1-102	N251	N252	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
101	RJ1-103	N253	N254	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
102	RJ1-104	N255	N256	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
103	RJ1-105	N257	N258	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
104	RJ1-106	N259	N260	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 25' 1/4" Roof (Continued)

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
105	RJ1-107	N261	N340	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
106	RJ1-108	N343	N344	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
107	RJ1-109	N345	N346	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
108	RJ1-110	N347	N348	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
109	RJ1-111	N349	N350	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
110	RJ1-112	N351	N352	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
111	RJ1-113	N353	N354	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
112	RJ1-114	N355	N356	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
113	RJ1-115	N357	N358	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
114	RJ1-116	N359	N360	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
115	RJ1-117	N361	N362	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
116	RJ1-118	N363	N364	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
117	RJ1-119	N365	N366	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
118	RJ1-120	N367	N368	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
119	RJ1-121	N369	N370	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
120	RJ1-122	N371	N372	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
121	RJ1-123	N373	N374	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
122	RJ1-124	N375	N376	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
123	RJ1-125	N377	N378	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
124	RJ1-126	N379	N380	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
125	RJ1-127	N381	N382	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
126	RJ1-128	N383	N384	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
127	RJ1-129	N385	N386	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
128	RJ1-130	N387	N388	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
129	RJ1-131	N389	N390	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
130	RJ1-132	N391	N392	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
131	RJ1-133	N393	N394	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
132	RJ1-134	N395	N396	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
133	RJ1-135	N397	N398	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
134	RJ1-136	N399	N400	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
135	RB3	N92	N93	3-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
136	RB2	N93	N95	W12x26	A992	Typical	Gravity	Strong Axis	Pinned	Fixed	
137	M138	N401	N402	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
138	M139	N403	N404	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
139	M140	N405	N406	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
140	M141	N407	N408	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 23'-4" V3 Nanny

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
1	3B1	N248	N247	W8x18	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
2	3J1A	N77	N78	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
3	3J1B	N79	N80	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
4	3J1C	N81	N82	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
5	3J1D	N83	N96	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
6	3J1E	N97	N98	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
7	3J1F	N99	N100	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
8	3J1G	N101	N102	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
9	3J1H	N103	N104	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
10	3J1I	N105	N106	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
11	3J1J	N107	N108	Red_I45x9.5	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 22' V2 Top of Window

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
1	3B4	N155	N157	HSS4x3x4	A500 Gr.B Rect	Typical	Gravity	Weak Axis	Pinned	Pinned	
2	3B5	N156	N158	3-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 19' V1 Mezz V3 Study

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
1	2B12	N149	GRD...	3-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
2	2B11	N32	N34	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
3	2B13	N32	N35	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
4	2J2A	N68	N69	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
5	2J2B	N70	N71	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
6	2J2C	N72	N73	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
7	2J2D	N74	N75	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
8	2J2E	N76	N77	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
9	2J2F	N78	N79	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
10	2J2G	N80	N81	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
11	2J2H	N82	N83	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
12	2B2	N110	N109	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
13	2B3	N104	N108	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
14	M22	N67	N133	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
15	2B23	N59	N151	W10x22	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
16	2B24	N106	N64	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
17	2J3A	N196	N198	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
18	2J3B	N201	N202	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
19	2J3C	N203	N204	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
20	2J3D	N205	N206	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
21	2J3E	N207	N208	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
22	2J3F	N209	N210	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
23	2J3G	N211	N212	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
24	2J3H	N213	N214	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
25	2J3I	N215	N216	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
26	2J3J	N217	N218	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
27	2J3K	N219	N220	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
28	2J3L	N221	N222	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
29	2B1	N106	N119	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
30	2J1-31	N96	N98	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
31	2J1-32	N100	N102	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
32	2J1-33	N111	N112	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
33	2J1-34	N113	N114	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
34	2J1-35	N115	N116	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
35	2J1-36	N117	N120	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
36	2J1-37	N121	N123	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
37	2J1-38	N124	N125	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
38	2J1-39	N126	N127	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
39	2J1-40	N128	N129	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
40	2J1-41	N130	N132	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
41	M43	N131	N137	Rectangular	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 17.5' V2 V3 Top of (low) windows

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
1	2B9	N153	N155	HSS4x3x4	A500 Gr.B Rect	Typical	Gravity	Weak Axis	Pinned	Pinned	
2	2B10	N151	N153	HSS4x3x4	A500 Gr.B Rect	Typical	Gravity	Weak Axis	Pinned	Pinned	
3	2B4	N32	N88	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
4	2B6	N88	N89	3-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
5	2B5	N93	N94	HSS4x3x4	A500 Gr.B Rect	Typical	Gravity	Weak Axis	Pinned	Pinned	
6	M6	N155	N157	Tube	A500 Gr.B RND	Typical	Gravity	Weak Axis	Pinned	Pinned	

Beam Primary Data : 15'-8"

Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...

No Data to Print ...

Beam Primary Data : 12'-8"

Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...

No Data to Print ...

Beam Primary Data : 10' Plaza Level

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
1	1B1	N25	N33	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
2	1B3	N26	N27	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
3	1B2	N26	N33	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
4	1B4	N27	N34	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
5	1B7	N89	N94	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
6	1B20	N88	N90	W14x30	A992	Typical	Lateral	Strong Axis	Pinned	Fixed	
7	1B19	N93	N95	W14x30	A992	Typical	Lateral	Strong Axis	Pinned	Fixed	
8	1B8	N90	N95	W14x68	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
9	1B21	N54	N93	2-1.75X11.875FS	2.0E Microllam...	Typical	Lateral	Strong Axis	Pinned	Pinned	
10	1B13	N58	N59	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
11	1B22	N56	N60	2-1.75X11.875FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
12	1B27	N53	N25	3-1.75X14FS	2.0E Microllam...	Typical	Lateral	Strong Axis	Pinned	Pinned	
13	1J4A	N362	N363	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
14	1J4B	N364	N365	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
15	1J4D	N368	N369	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
16	1J4E	N370	N371	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
17	1J4G	N372	N373	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
18	1J4H	N374	N375	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
19	1J4I	N376	N377	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
20	1J4J	N378	N379	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
21	1J4K	N380	N381	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
22	1J4L	N382	N383	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
23	1J4M	N384	N385	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
24	1J4N	N386	N387	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
25	1J4O	N388	N389	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
26	1J4P	N390	N391	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
27	1J4Y	N330	N331	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
28	1J4Z	N332	N333	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
29	1J4AA	N334	N335	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
30	1J4BB	N336	N337	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
31	1B23	N366	N43	2-1.75X11.875FS	2.0E Microllam...	Typical	Lateral	Strong Axis	Pinned	Pinned	
32	M70	N55	N36	2-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
33	1B12	N55	N132	2-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 10' Plaza Level (Continued)

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
34	1J2A	N155	N156	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
35	1J2B	N157	N158	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
36	1J2C	N159	N160	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
37	1J2D	N161	N162	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
38	1J2E	N163	N164	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
39	1J2F	N165	N166	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
40	1J2G	N167	N168	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
41	1J2H	N169	N170	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
42	1J2I	N171	N172	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
43	1J2J	N173	N174	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
44	1J2K	N175	N176	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
45	1J2L	N177	N178	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
46	1J3A	N191	N192	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
47	1J3B	N193	N194	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
48	1J3C	N195	N196	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
49	1J3D	N197	N198	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
50	1J3E	N199	N200	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
51	1J3F	N201	N202	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
52	1J3G	N203	N204	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
53	1J3H	N205	N206	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
54	1J3I	N207	N208	2X10	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
55	1J5A	N222	N223	2X6	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
56	1J5B	N224	N225	2X6	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
57	1J5C	N226	N227	2X6	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
58	1J5D	N228	N229	2X6	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
59	1J5E	N230	N231	2X6	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
60	1J5F	N232	N233	2X6	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
61	1J5G	N234	N235	2X6	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
62	1J5H	N236	N237	2X6	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
63	1B26	GRD...	N38	W12x26	A992	14"	Lateral	Strong Axis	Pinned	Fixed	
64	1B6	N439	N94	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
65	PUR86	N179	N180	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
66	PUR87	N181	N182	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
67	PUR88	N183	N184	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
68	PUR89	N185	N186	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
69	PUR90	N187	N188	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
70	PUR91	N189	N190	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
71	PUR101	N209	N210	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
72	PUR102	N211	N212	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
73	PUR103	N213	N214	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
74	PUR104	N215	N216	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
75	PUR105	N217	N218	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
76	PUR106	N219	N220	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
77	PUR107	N221	N238	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
78	PUR116	N239	N240	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
79	PUR117	N241	N242	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
80	PUR118	N243	N244	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
81	PUR119	N245	N246	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
82	PUR120	N247	N248	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
83	PUR121	N249	N250	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
84	PUR122	N251	N252	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
85	PUR123	N253	N254	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 10' Plaza Level (Continued)

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
86	PUR124	N255	N256	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
87	PUR125	N257	N258	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
88	PUR126	N259	N260	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
89	PUR153	N313	N314	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
90	PUR154	N315	N316	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
91	PUR155	N317	N318	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
92	PUR156	N319	N320	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
93	PUR157	N321	N322	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
94	PUR158	N323	N324	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
95	PUR159	N325	N326	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
96	PUR160	N328	N329	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
97	PUR161	N338	N339	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
98	PUR162	N340	N341	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
99	PUR163	N358	N359	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
100	PUR164	N360	N361	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
101	PUR165	N392	N393	Red_I90x14	Wood Product...	14"	Gravity	Strong Axis	Pinned	Pinned	
102	M139	GRD...	N296	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
103	M140	N296	N295	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
104	M141	N295	N294	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
105	M142	N294	N293	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
106	M143	N293	N292	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
107	M144	N292	N291	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
108	M145	N291	N290	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
109	M146	N290	N289	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
110	M147	N289	N288	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
111	M148	N288	N287	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
112	M149	N287	N286	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
113	M150	N286	N285	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
114	M155	N285	N284	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
115	M156	N284	GRD...	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
116	1B11	GRD...	N263	W14x61	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
117	1B10	N93	N88	W10x22	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
118	1B14	N22	N27	W10x22	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
119	1B17	N36	GRD...	W12x26	A992	Typical	Lateral	Strong Axis	Fixed	Pinned	
120	1B16	GRD...	N37	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
121	PUR138	N279	N276	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
122	PUR139	N281	N278	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
123	PUR140	N283	N282	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
124	PUR144	N276	N297	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
125	PUR145	N278	N299	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
126	PUR146	N282	N301	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
127	PUR187	N404	N405	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
128	PUR188	N406	N263	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
129	PUR215	N405	N437	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
130	1B18	N263	N439	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
131	1B5	N439	N34	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
132	1B24	N93	N263	W12x40	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
133	PUR178	N398	N427	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
134	PUR204	N400	N429	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
135	PUR205	N402	N431	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
136	PUR206	N407	N433	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
137	PUR207	N409	N435	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 10' Plaza Level (Continued)

	Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
138	PUR208	N411	N438	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
139	PUR209	N413	N441	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
140	PUR214	N427	N399	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
141	PUR216	N429	N401	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
142	PUR217	N431	N403	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
143	PUR218	N433	N408	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
144	PUR219	N435	N410	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
145	PUR220	N438	N412	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
146	PUR221	N441	N414	Red_I90x14	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
147	1B25	N55	N302	2-1.75X14FS	2.0E Microllam...	Typical	Gravity	Strong Axis	Pinned	Pinned	
148	1B15	GRD...	N34	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
149	DRAG	N367	N61	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
150	DRAG159	N59	N304	Rectangular	Spruce-Pine-Fir	Typical	Gravity	Strong Axis	Pinned	Pinned	
151	1B9	N417	N418	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
152	1B28	N298	N300	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
153	1B29	N303	N307	W12x26	A992	Typical	Lateral	Strong Axis	Pinned	Pinned	
154	1J4-154	N308	N309	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
155	1J4-155	N310	N311	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
156	1J4-156	N312	N327	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
157	1J4-157	N394	N395	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
158	1J4-158	N396	N397	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
159	1J4-159	N415	N416	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	
160	1J4-160	N419	N420	Red_I45x11.875	Wood Product...	Typical	Gravity	Strong Axis	Pinned	Pinned	

Beam Primary Data : 0' Basement Floor

Label	Start ...	End P...	Shape	Material	Design Rules	Functi...	Orientation	Start ...	End R...	Outrig...
No Data to Print ...										

Wood Wall Panel Parameters

	Label	Top Plate	Sill Plate	Studs	Min Stud Sp...	Max Stud Sp...	Green Lumb...	Header Size	Header Matl
1	Typical	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall
2	6"	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall
3	4"	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall
4	2"	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall

Additional Wood Wall Panel Parameters

	Label	Schedule	Min. Panel...	Max. Pane...	Double Sid...	Max. Nail ...	Min. Nail S...	HD Chords	HD Chord ...	Hold Down
1	Typical	AWC 2015 PLY	.469	.469	No	6-in.	2-in.	2-2X6	Same as ...	HDU_DF-SP
2	6"	S1_15/32_8d@6	.469	.469	No	6-in.	6-in.	2-2X6	Same as ...	HDU_DF-SP
3	4"	S1_15/32_8d@4	.469	.469	No	4-in.	4-in.	2-2X6	Same as ...	HDU_DF-SP
4	2"	S1_15/32_8d@2	.469	.469	No	2-in.	2-in.	2-2X6	Same as ...	HDU_DF-SP

Wall Panel Parameters

	Label	Top Floor	Bottom Floor	Start...	End ...	Material ...	Material ...	Thickn...	Function	Design R...	Distance Belo...	Parape...
1	CONC WALL 1	10' Plaza Level	0' Basemen...	N53	N36	Concrete	Conc350...	8	Gravity	Typical	10	Default...
2	CONC WALL 2	10' Plaza Level	0' Basemen...	N40	N36	Concrete	Conc350...	8	Gravity	Typical	10	NA
3	CONC WALL 3	19' V1 Mezz V3...	0' Basemen...	N248	N64	Concrete	Conc350...	8	Gravity	Typical	10	NA
4	CONC WALL 4	19' V1 Mezz V3...	0' Basemen...	N65	N464	Concrete	Conc350...	8	Gravity	Typical	10	NA
5	CONC WALL 5	10' Plaza Level	0' Basemen...	N88	N45	Concrete	Conc350...	8	Gravity	Typical	10	Default...
6	CONC WALL 6	10' Plaza Level	0' Basemen...	N45	N46	Concrete	Conc350...	8	Gravity	Typical	10	Default...
7	CONC WALL 7	10' Plaza Level	0' Basemen...	N46	N89	Concrete	Conc350...	8	Gravity	Typical	10	Default...
8	CONC WALL 8	12'-8"	0' Basemen...	N1	N2	Concrete	Conc350...	8	Gravity	Typical	10	NA
9	CONC WALL 9	15'-8"	0' Basemen...	N2	N3	Concrete	Conc350...	8	Gravity	Typical	10	NA
10	STUD WALL 1	23'-4" V3 Nanny	15'-8"	N31	N32	Wood	Spruce-P...	5.5 (st...	Gravity	6"	1.833	NA
11	STUD WALL 2	23'-4" V3 Nanny	19' V1 Mez...	N247	N84	Wood	Spruce-P...	5.5 (st...	Gravity	6"	3	NA
12	STUD WALL 3	19' V1 Mezz V3...	17.5' V2 V3...	N93	N94	Wood	Spruce-P...	5.5 (st...	Gravity	6"	1.5	NA
13	STUD WALL 4	19' V1 Mezz V3...	17.5' V2 V3...	N88	N89	Wood	Spruce-P...	5.5 (st...	Gravity	6"	1.5	NA
14	STUD WALL 5	10' Plaza Level	0' Basemen...	N55	N56	Wood	Spruce-P...	5.5 (st...	Gravity	6"	10	NA
15	CONC WALL ...	19' V1 Mezz V3...	0' Basemen...	N197	N464	Concrete	Conc350...	8	Gravity	Typical	10	NA
16	CONC WALL ...	19' V1 Mezz V3...	15'-8"	N248	N61	Concrete	Conc350...	8	Gravity	Typical	1.833	NA
17	SW-WP7 B	19' V1 Mezz V3...	10' Plaza L...	N92	N106	Wood	Spruce-P...	5.5 (st...	Lateral	6"	2.667	NA
18	SW-WP11 A	10' Plaza Level	0' Basemen...	N54	N92	Wood	Spruce-P...	5.5 (st...	Lateral	6"	10	NA
19	SW-WP11 B	10' Plaza Level	0' Basemen...	N92	N366	Wood	Spruce-P...	5.5 (st...	Lateral	6"	10	NA
20	SW-WP8 UP...	25' 1/4" Roof	23'-4" V3 N...	N244	N245	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	1.73	NA
21	SW-WP8 LO...	23'-4" V3 Nanny	19' V1 Mez...	N59	N60	Wood	Spruce-P...	5.5 (st...	Lateral	6"	3	NA
22	SW-WP13	19' V1 Mezz V3...	10' Plaza L...	N104	N119	Wood	Spruce-P...	5.5 (st...	Lateral	6"	2.667	NA
23	SW-WP14	10' Plaza Level	0' Basemen...	N61	N304	Wood	Spruce-P...	5.5 (st...	Lateral	4"	10	NA
24	SW-WP12 B	19' V1 Mezz V3...	15'-8"	N105	N87	Wood	Spruce-P...	5.5 (st...	Lateral	6"	1.833	NA
25	STUD WALL 8	19' V1 Mezz V3...	17.5' V2 V3...	N88	N105	Wood	Spruce-P...	5.5 (st...	Gravity	6"	1.5	NA
26	CONC WALL ...	19' V1 Mezz V3...	0' Basemen...	N197	N122	Concrete	Conc300...	10	Gravity	Typical	10	NA
27	CONC WALL ...	19' V1 Mezz V3...	0' Basemen...	N122	N59	Concrete	Conc300...	10	Gravity	Typical	10	NA
28	CONC WALL ...	10' Plaza Level	0' Basemen...	N22	N306	Concrete	Conc300...	10	Gravity	Typical	10	NA
29	SW-WP6 LO...	23'-4" V3 Nanny	19' V1 Mez...	N86	N84	Wood	Spruce-P...	5.5 (st...	Lateral	4"	3	NA
30	SW-WP7 A	19' V1 Mezz V3...	10' Plaza L...	N118	N92	Wood	Spruce-P...	5.5 (st...	Lateral	6"	2.667	NA
31	STUD WALL 6	19' V1 Mezz V3...	10' Plaza L...	N93	N118	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	2.667	NA
32	SW-WP12 A	19' V1 Mezz V3...	15'-8"	N87	N110	Wood	Spruce-P...	5.5 (st...	Lateral	6"	1.833	NA
33	STUD WALL 7	19' V1 Mezz V3...	15'-8"	N110	N65	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	1.833	NA
34	CONC WALL ...	19' V1 Mezz V3...	0' Basemen...	N247	N65	Concrete	Conc300...	12	Gravity	Typical	10	NA
35	CONC WALL ...	15'-8"	0' Basemen...	N4	N1	Concrete	Conc300...	12	Gravity	Typical	10	NA
36	SW-WP10	19' V1 Mezz V3...	10' Plaza L...	N131	N67	Wood	Spruce-P...	5.5 (st...	Lateral	6"	2.667	NA
37	SW-WP9	25' 1/4" Roof	19' V1 Mez...	N341	N202	Wood	Spruce-P...	5.5 (st...	Gravity	Typical	3	NA

Gravity Loading

Floors

	Label	Elevation[ft]	Area Loa...	Floor...	Deck ...	Deck...	Parent	Des...	Bot Rebar D...	Continu...	Top R...	Ina...	Sp..	Splic...	No ...	Par...
1	25' 1/4" Roof	25.063	Roof	Floor...	Roof ...	0	None	N/A	N/A		N/A		.083	Shear		0
2	23'-4" V3 N...	23.333	Interior Fl...	Floor...	Interor...	0	None	N/A	N/A		N/A		.083	Shear		0
3	22' V2 Top ...	22	Interior Fl...	Floor...	Interor...	0	None	N/A	N/A		N/A		.083	Shear		0
4	19' V1 Mezz...	19	Interior Fl...	Floor...	Interor...	0	None	N/A	N/A		N/A		.083	Shear		0
5	17.5' V2 V3 ...	17.5	Interior Fl...	Floor...	Interor...	0	None	N/A	N/A		N/A		.083	Shear		0
6	15'-8"	15.667	Interior Fl...	Floor...	Interor...	0	None	N/A	N/A		N/A		.083	Shear		0
7	12'-8"	12.667	Interior Fl...	Floor...	Interor...	0	None	N/A	N/A		N/A		.083	Shear		0
8	10' Plaza Le...	10	Plaza Roof	Floor...	Plaza ...	90	None	N/A	N/A		N/A		10	Shear		0
9	0' Basement...	0	Interior Fl...	Floor...	Interor...	0	None	N/A	N/A		N/A		.083	Shear		0

Deck General Properties

	Label	Material Type	Deck	Unbraced[ft]	Max Spa...
1	Roof Wood	Wood Deck	1 in	.5	4
2	Interor Wood	Wood Deck	1 in	.5	4
3	Plaza Roof Wood	Wood Deck	1 in	.5	4

Deck Loads

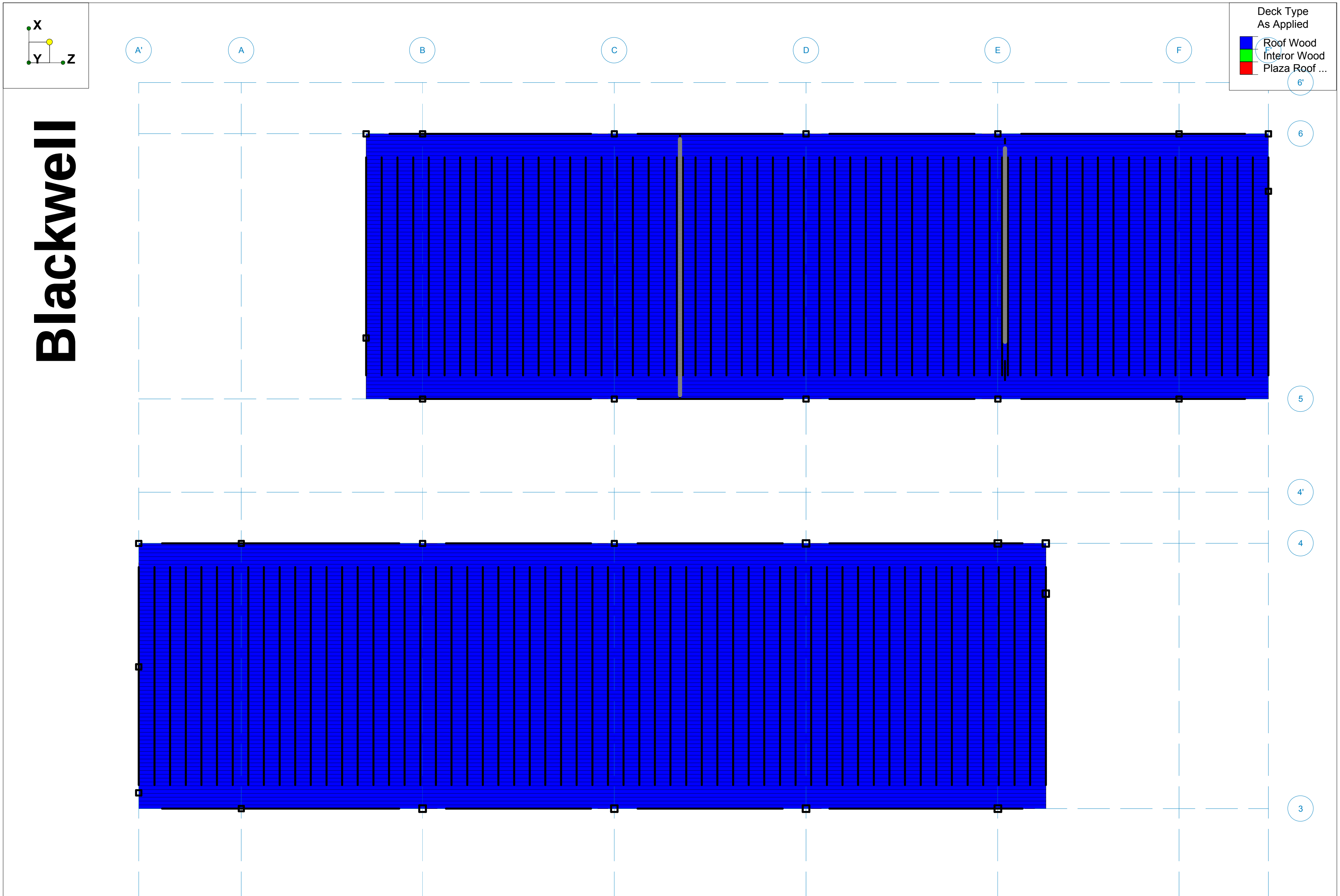
	Label	Two Way	Self Wt [psf]	Super DL [psf]	Const DL [psf]	Const LL [psf]
1	Roof Wood		3	25	0	0
2	Interor Wood		3	40	0	0
3	Plaza Roof Wood		3	50	0	0

Uniform Area Loads

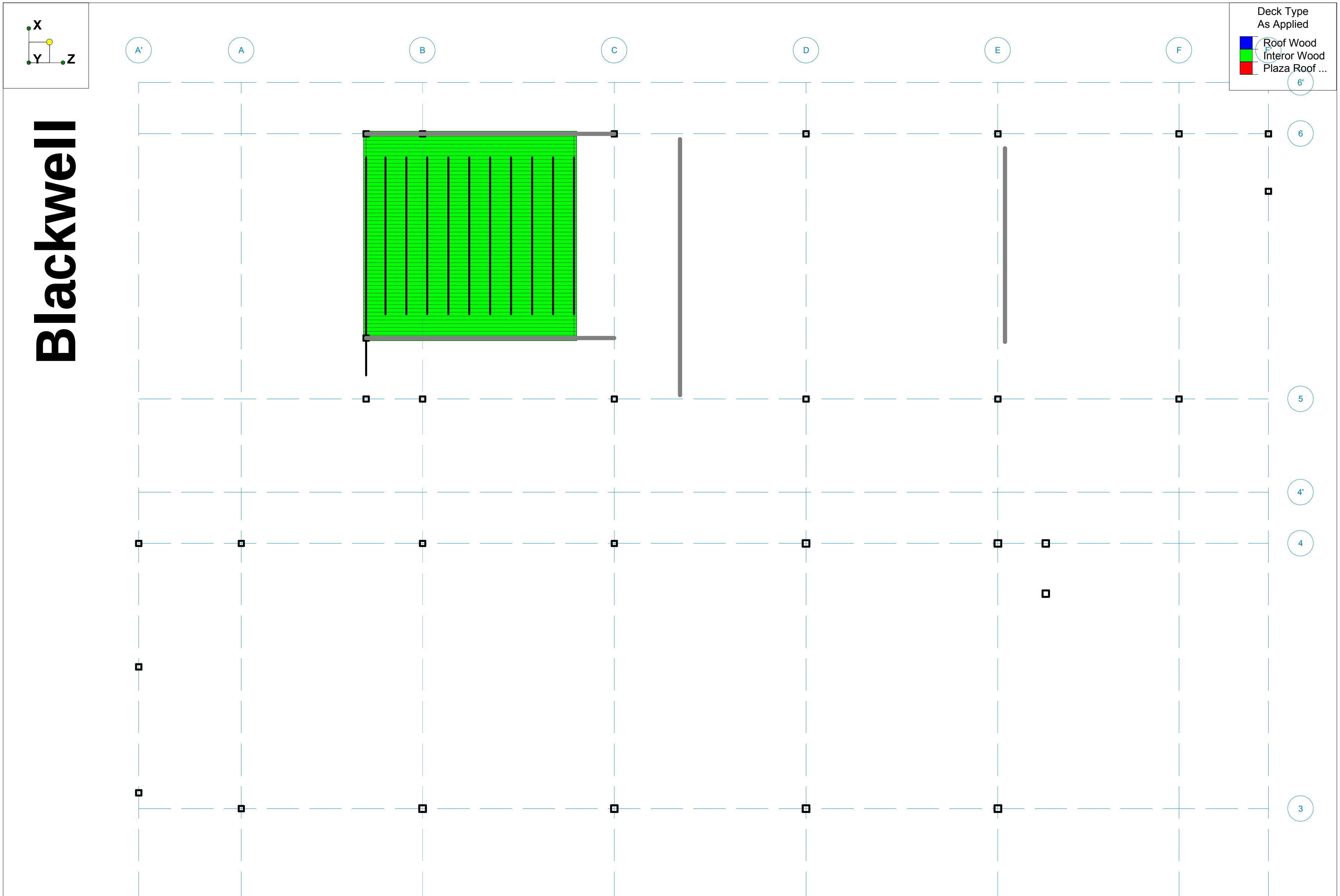
	Label	Additive	PreDL[psf]	PostDL[psf]	LL[psf]	LL Type	VL[psf]	Dyn Load[psf]
1	Interior Floor				40	LL-Non		
2	Roof				192	SL		57.6
3	Plaza Roof				274	SL		85.2

Combinations

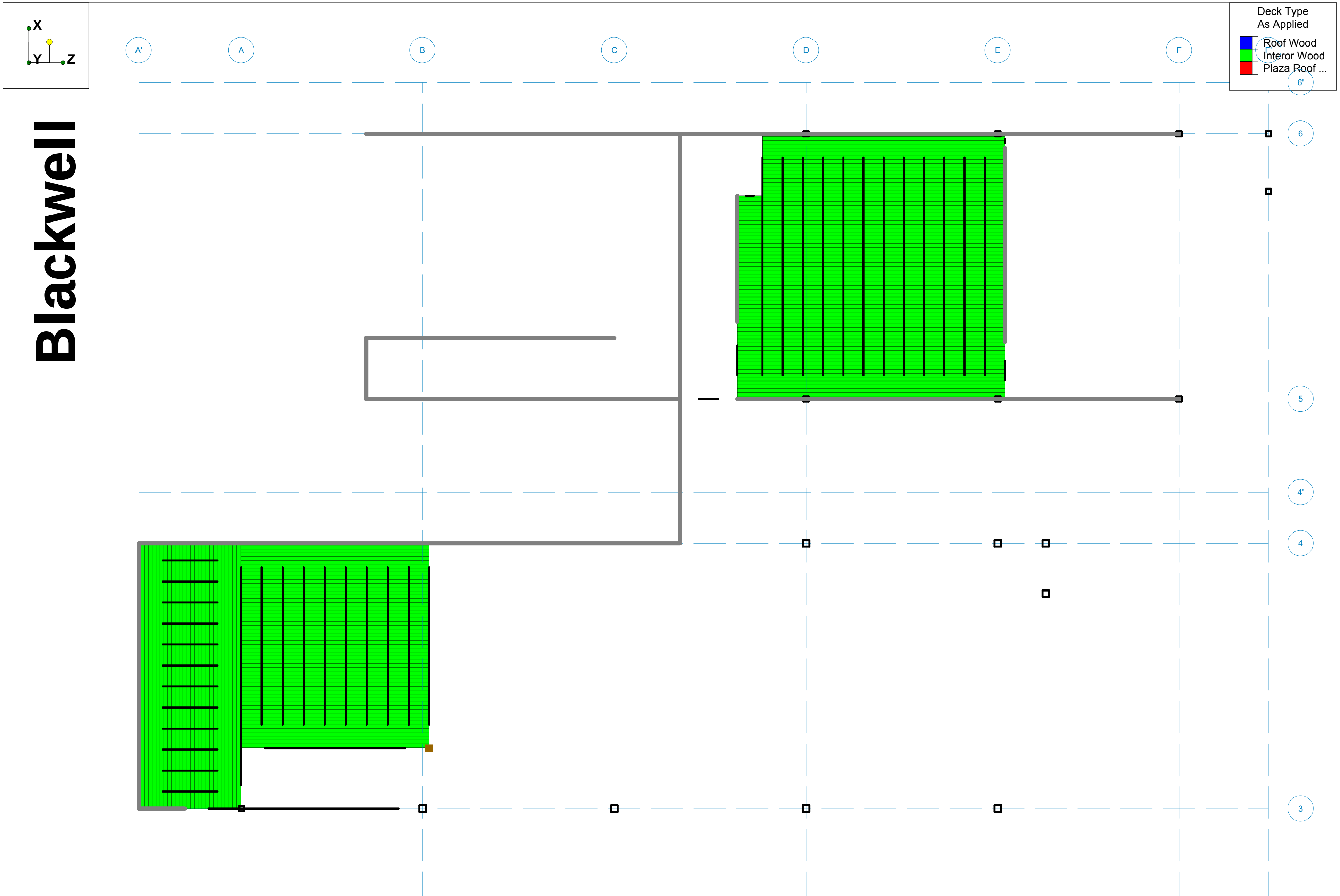
	Label	Sol..	Cat...	Fa...	Cat...	Fa...	Cat...	Fa...	Cat...	Fa...	Cat...	Fac...	Cat...	Fac...	Cat...	Fac...	Cat...	Fac...	Cat...	Fac...	Cat...	Fac...
1	ASCE Strength 1 ...	Yes	DL	1.4																		
2	ASCE Strength 2 (...	Yes	DL	1.2	LL	1.6																
3	ASCE Strength 2 (...	Yes	DL	1.2	LL	1.6	SL	.5														
4	ASCE Strength 3 (...	Yes	DL	1.2	LL	.5																
5	ASCE Strength 3 (...	Yes	DL	1.2	SL	1.6	LL	.5														
6	ASCE ASD 1 Post	Yes	DL	1																		
7	ASCE ASD 2 Post	Yes	DL	1	LL	1																
8	ASCE ASD 3 (b) ...	Yes	DL	1	SL	1																
9	ASCE ASD 4 (a) ...	Yes	DL	1	LL	.75																
10	ASCE ASD 4 (b) ...	Yes	DL	1	LL	.75	SL	.75														
11	D		DL	1																		
12	L		LL	1																		
13	S		SL	1																		



Blackwell Structural Engineers	25' 1/4" Roof	DECK ASSIGNMENT
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:15 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Blackwell Structural Engineers	23'-4" V3 Nanny	DECK ASSIGNMENT
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:15 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Blackwell Structural Engineers	19' V1 Mezz V3 Study	DECK ASSIGNMENT
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:15 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

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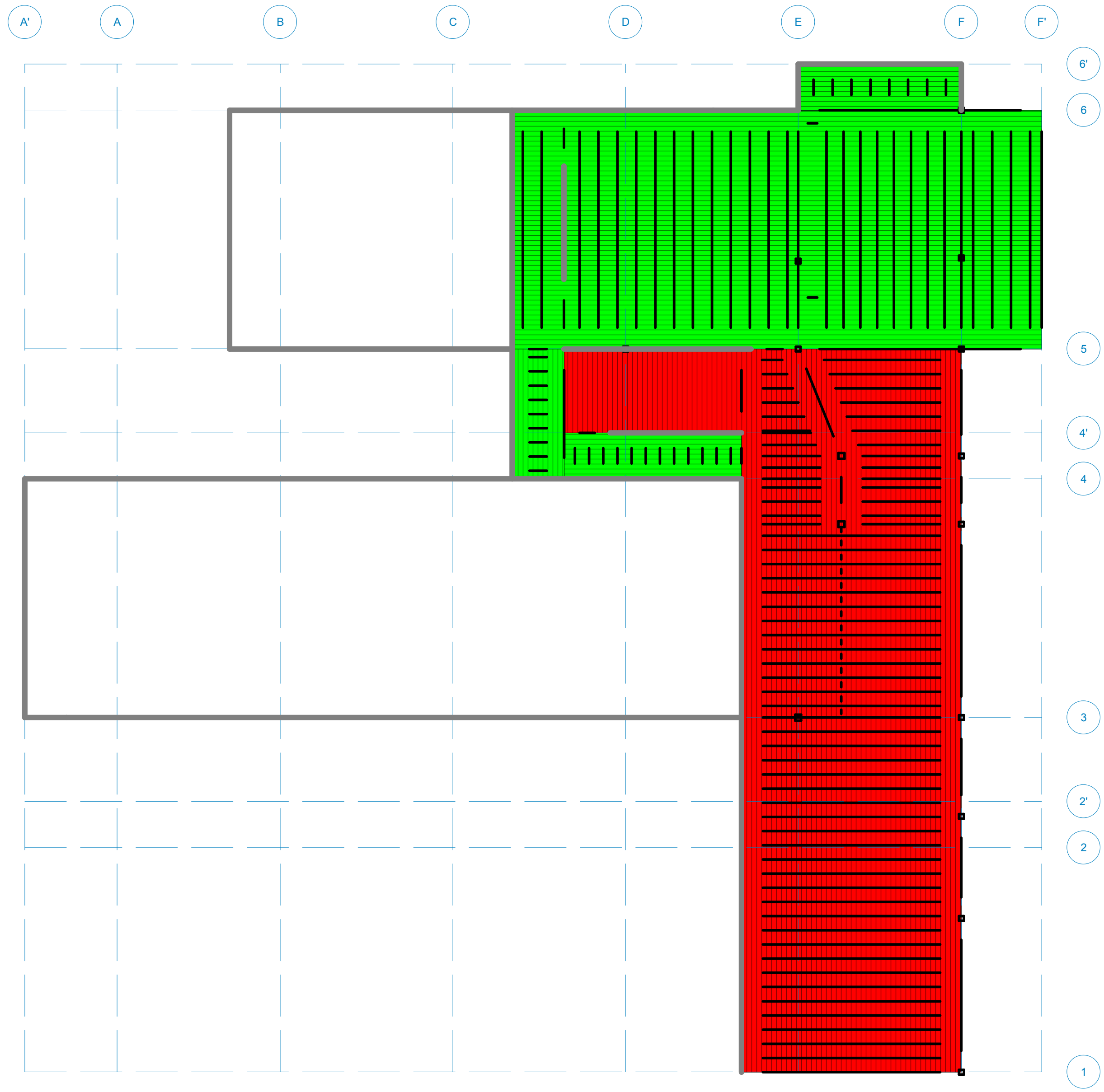
Deck Type
As Applied

Roof Wood
Interor Wood
Plaza Roof ...



Results for LC 1, ASCE Strength 1 Post

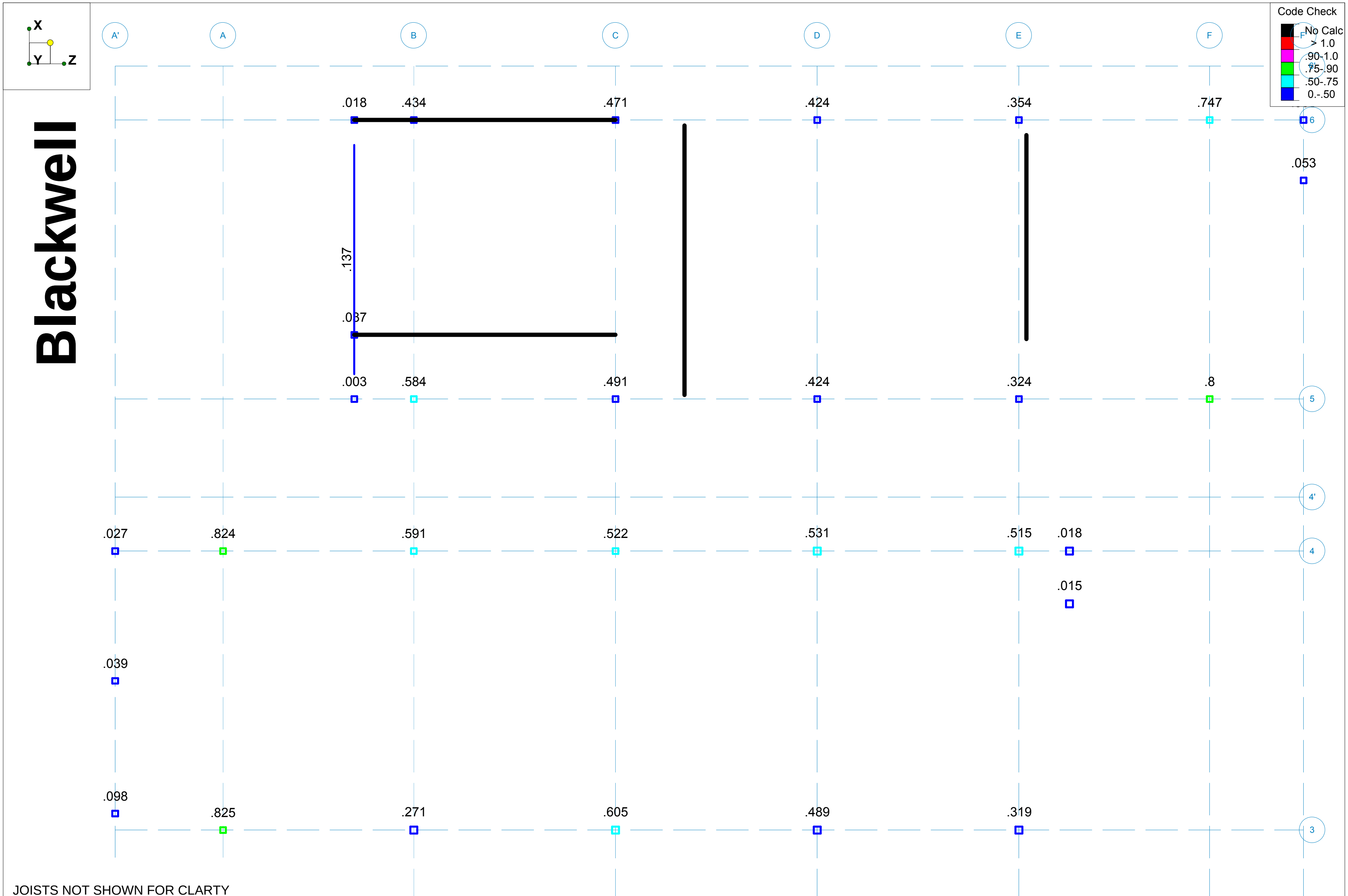
Blackwell Structural Engineers	10' Plaza Level	DECK ASSIGNMENT
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:40 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

Blackwell

Structural Engineers

Gravity Steel and Wood Member Utilization

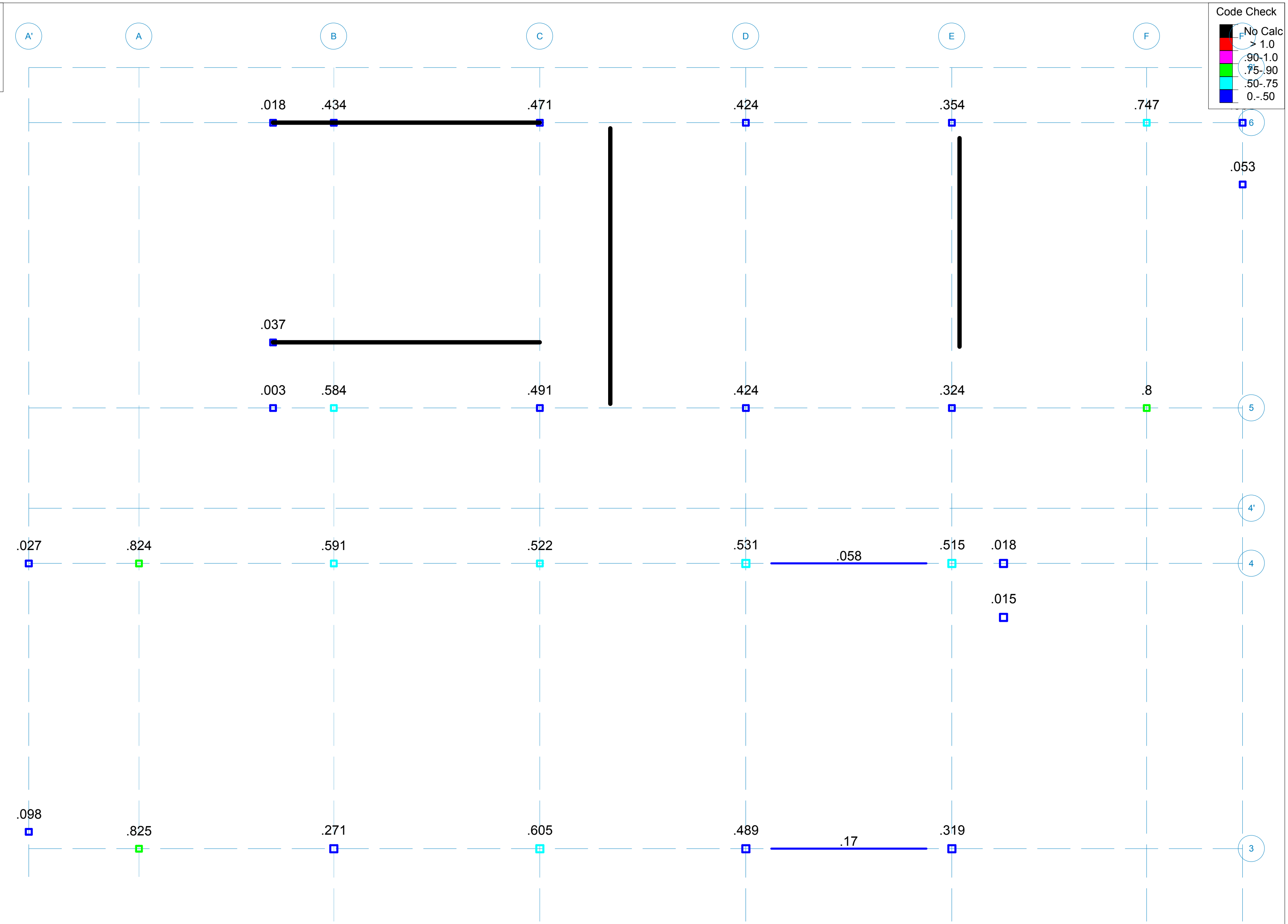
Blackwell Structural Engineers	25' 1/4" Roof	BENDING CHECK
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:18 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



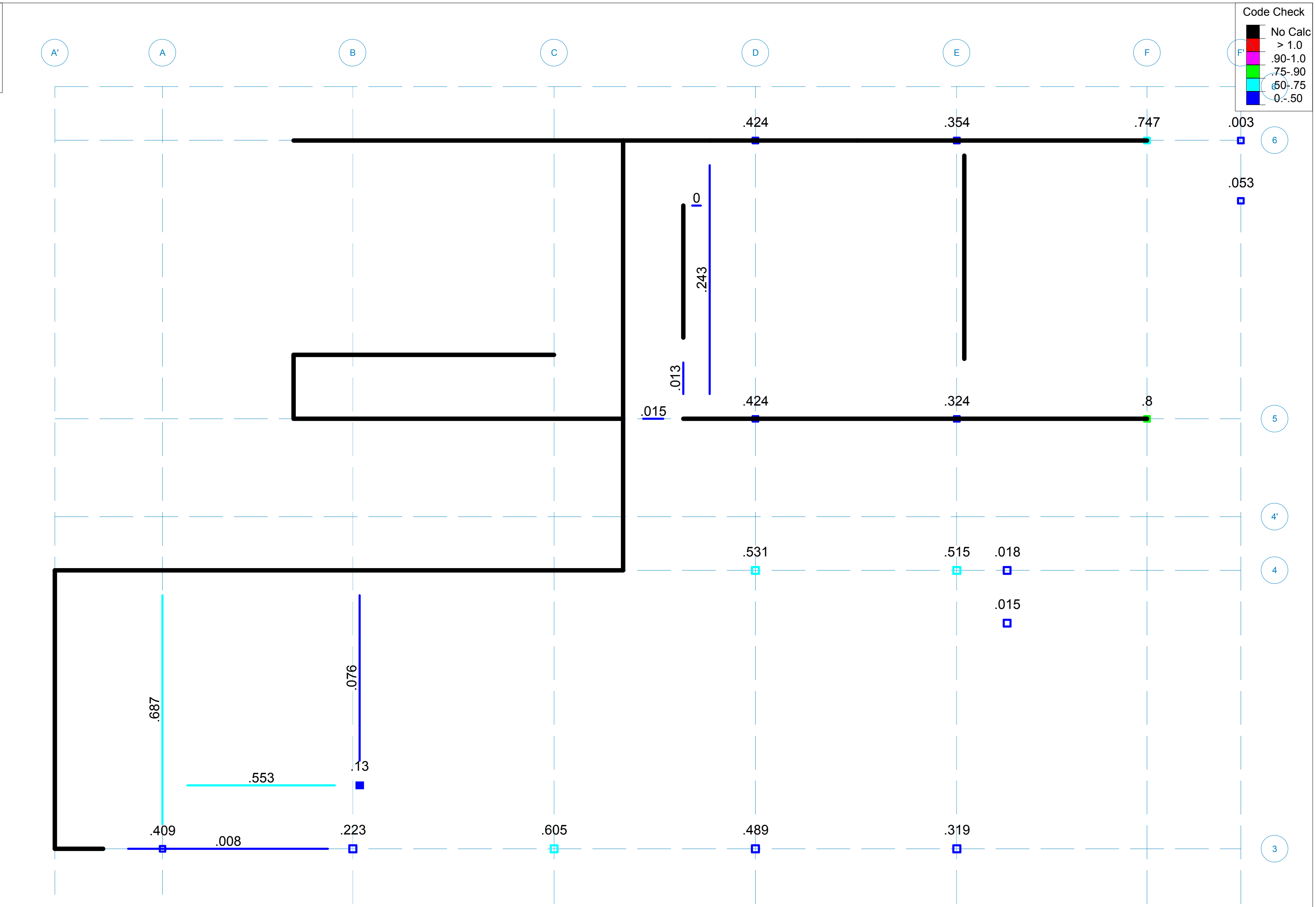
Blackwell

JOISTS NOT SHOWN FOR CLARTY

Blackwell Structural Engineers	23'-4" V3 Nanny	BENDING CHECK
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:18 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

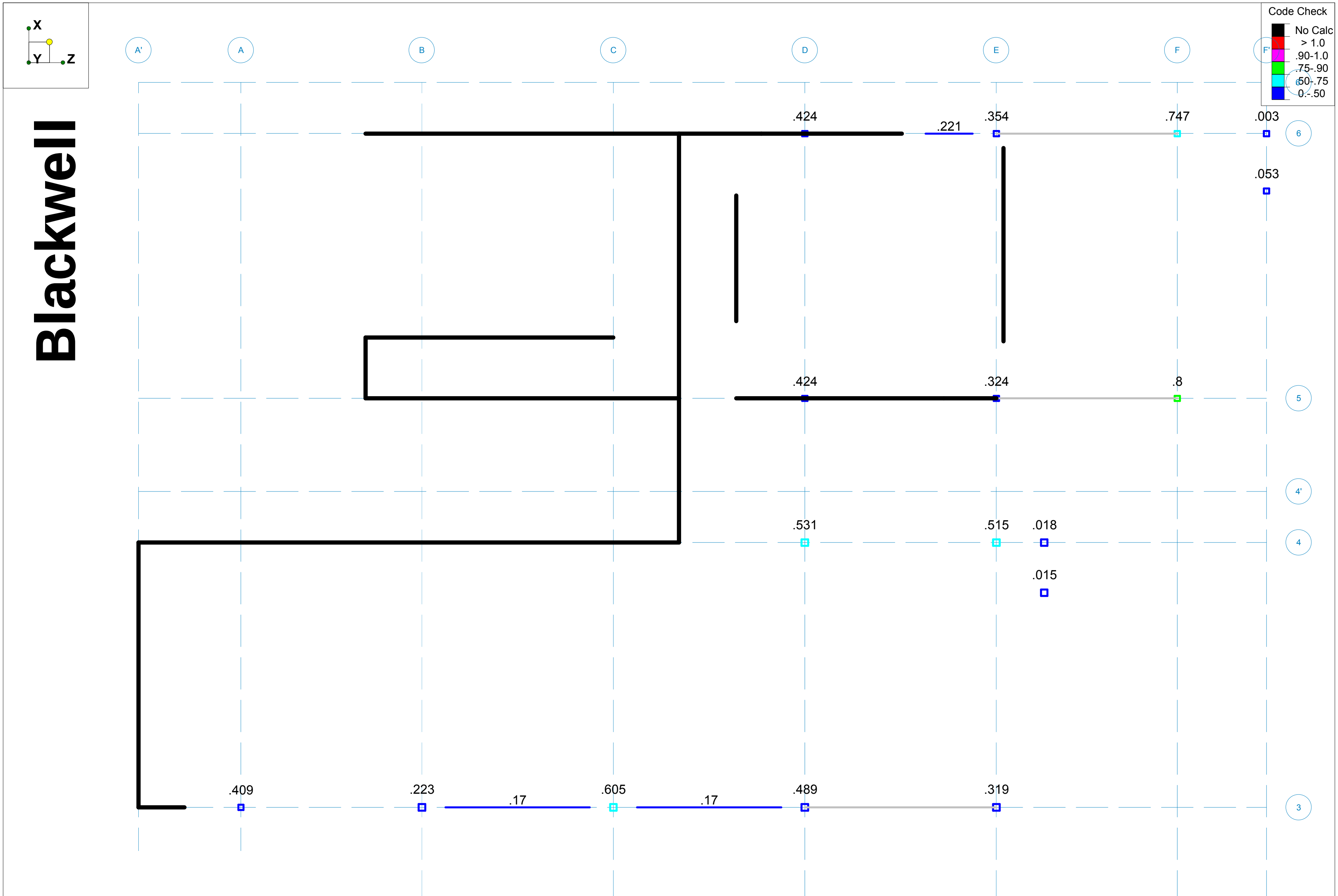


Blackwell Structural Engineers	22' V2 Top of Window	BENDING CHECK
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:19 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.r3d

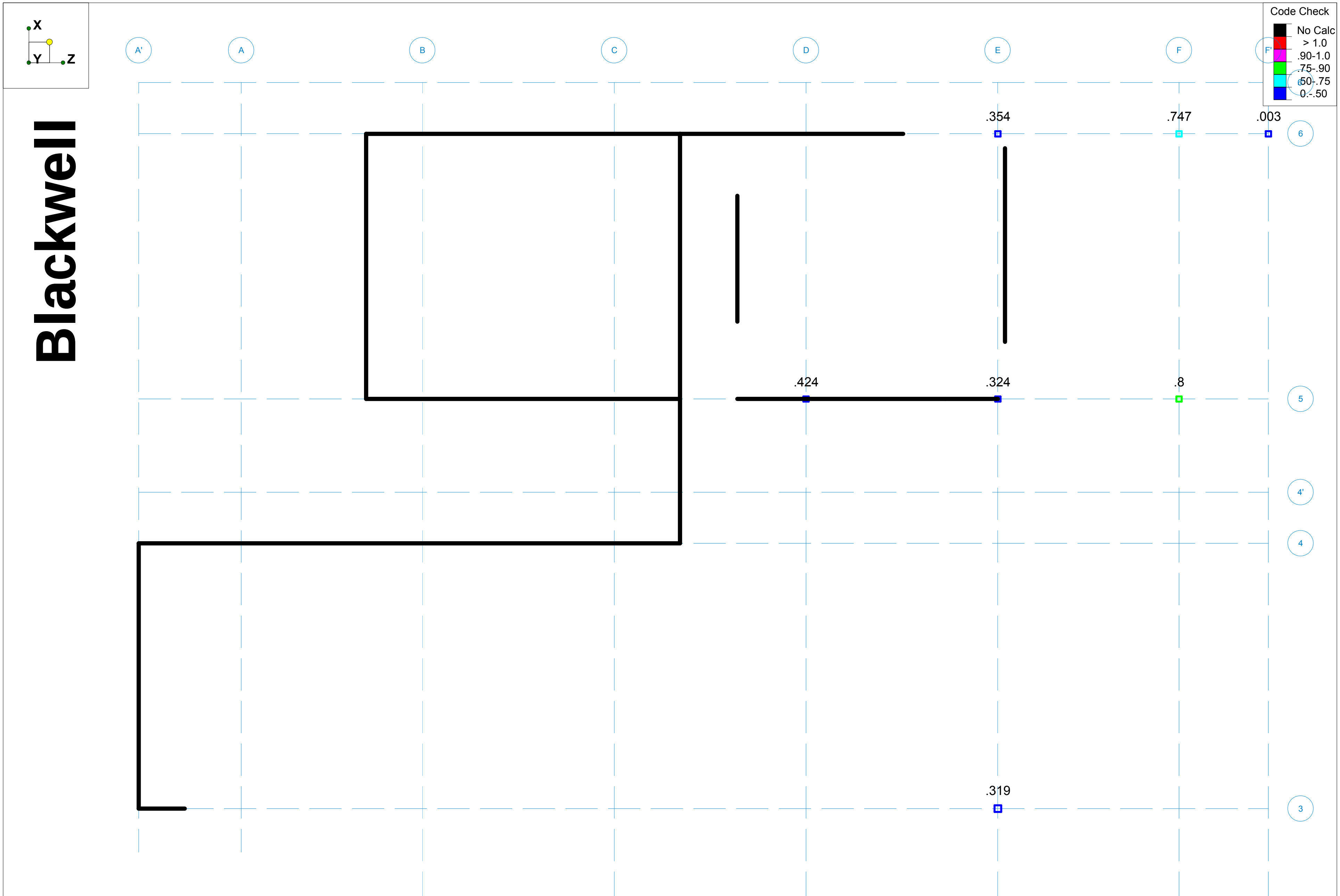


JOISTS NOT SHOWN FOR CLARTY

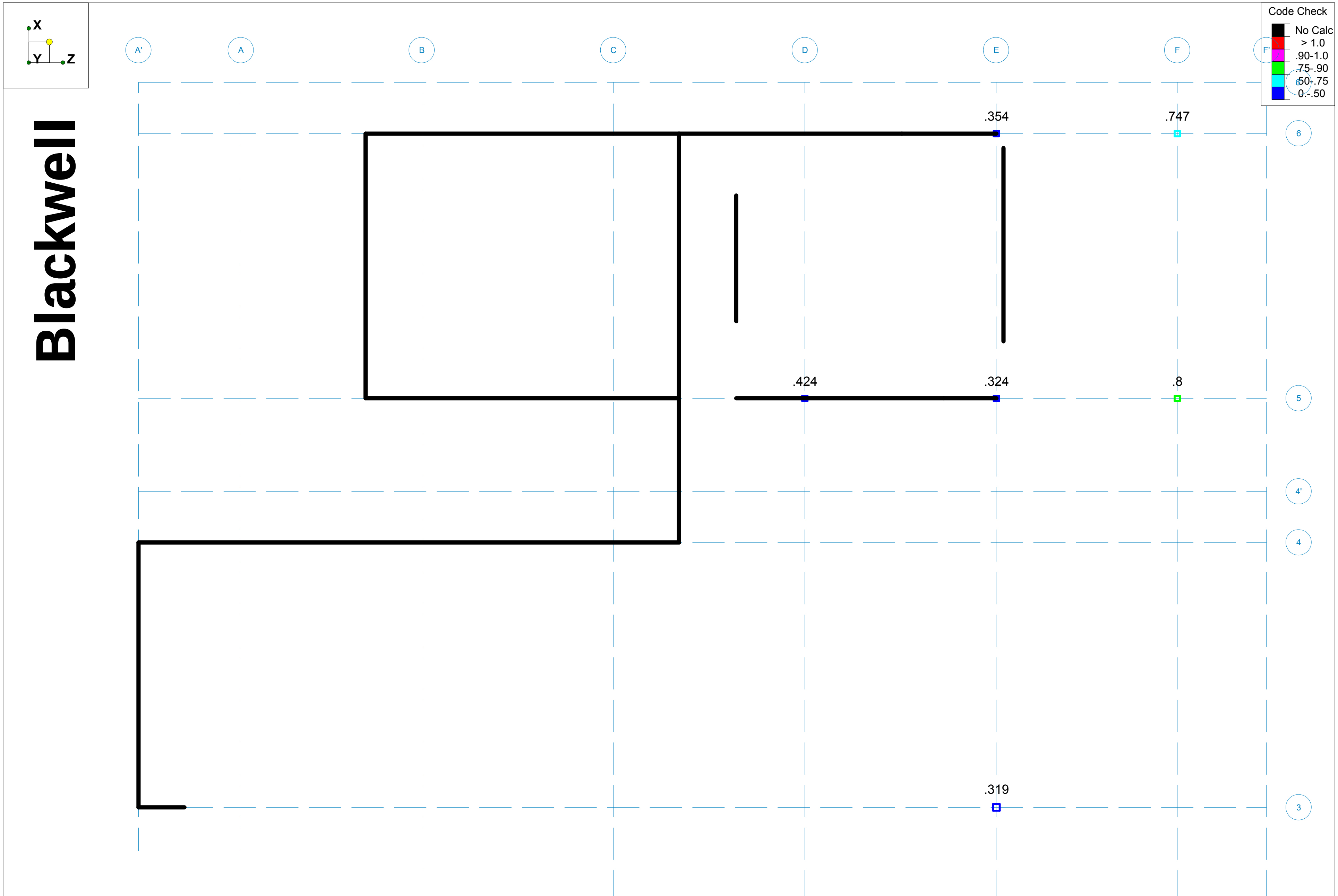
Blackwell Structural Engineers	19' V1 Mezz V3 Study	BENDING CHECK
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:19 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



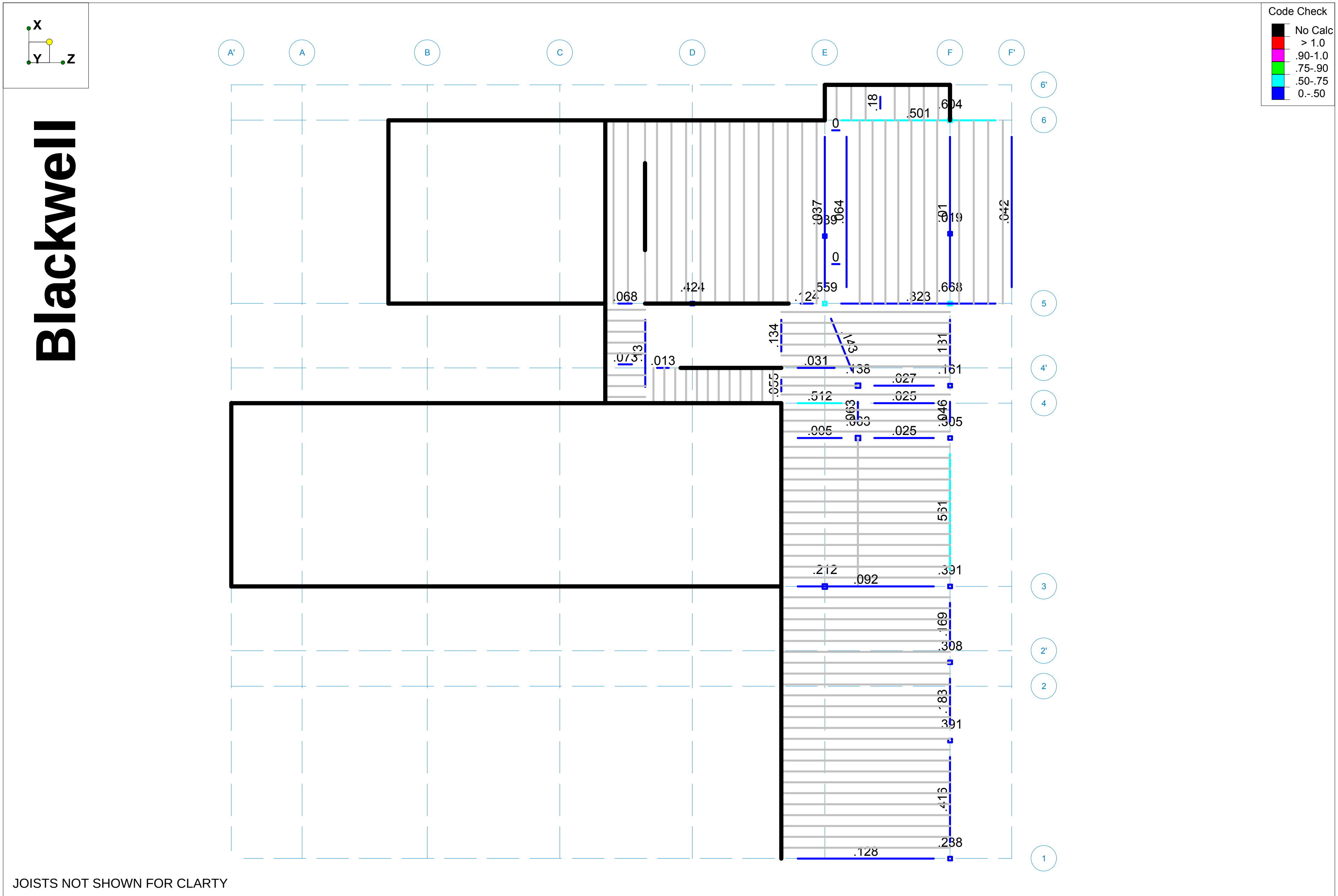
Blackwell Structural Engineers	17.5' V2 V3 Top of (low) windows	BENDING CHECK
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:20 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Blackwell Structural Engineers	15'-8"	BENDING CHECK
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:20 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Blackwell Structural Engineers	12'-8"	BENDING CHECK
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:20 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl



Blackwell Structural Engineers	10' Plaza Level	BENDING CHECK
BG	Kimmelman May Residence Volume 2, 3 and 4	Feb 15, 2018 at 4:38 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

Beam Code Summary for Hot Rolled : 25' 1/4" Roof

	Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
1	RB15	W12x26	Yes		0	A992	.528	6.463	5	.157	12.926	DL+...	.236	6.653	5
2	RB11	W12x22	Yes		0	A992	.045	6.838	5	.037	6.838	DL+...	.014	0	5
3	RB18	W12x22	Yes		0	A992	.088	.877	5	.025	0	LL	.031	.877	5
4	RB1	W12x22	Yes		0	A992	.109	3.682	5	.025	0	LL	.022	3.507	5
5	RB6	W12x22	Yes		0	A992	.085	9.644	5	.065	7.189	DL+...	.026	16.8...	5
6	RB20	W12x26	Yes		0	A992	.372	12.367	5	.183	5.621	DL+...	.234	12.2...	5
7	RB17	W12x26	Yes		0	A992	.306	11.595	5	.158	5.322	DL+...	.236	11.5...	5
8	RB5	W12x26	Yes		0	A992	.314	5.821	5	.232	5.821	DL+...	.236	12.3...	5
9	RB12	W12x26	Yes		0	A992	.344	5.461	5	.296	15.419	DL+...	.209	12.2...	5
10	RB10	W12x26	Yes		0	A992	.332	11.603	5	.159	5.439	DL+...	.243	11.6...	5
11	RB7	W12x26	Yes		0	A992	.482	12.474	5	.183	5.655	DL+...	.252	12.3...	5
12	RB2	W12x26	Yes		0	A992	.619	11.784	5	.132	5.257	DL+...	.253	11.6...	5

Beam Code Summary for Hot Rolled : 23'-4" V3 Nanny

	Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
1	3B1	W8x18	Yes		0	A992	.137	3.858	2	.082	11.047	DL+...	.044	4.033	2

Beam Code Summary for Hot Rolled : 22' V2 Top of Window

	Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
1	3B4	HSS4x3x4	Yes		0	A500 ...	.17	6.084	1	.526	6.084	DL+...	.023	12.1...	1

Beam Code Summary for Hot Rolled : 19' V1 Mezz V3 Study

	Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
1	2B23	W10x22	Yes		0	A992	.008	3.457	1	.025	0	LL	.011	3.457	1

Beam Code Summary for Hot Rolled : 17.5' V2 V3 Top of (low) windows

	Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
1	2B9	HSS4x3x4	Yes		0	A500 ...	.17	6.083	1	.526	6.083	DL+...	.023	0	1
2	2B10	HSS4x3x4	Yes		0	A500 ...	.17	6.084	1	.526	6.084	DL+...	.023	12.1...	1
3	2B5	HSS4x3x4	Yes		0	A500 ...	.249	5.87	1	.977	5.75	DL+...	.05	0	1
4	M6	HSS2.5x2....	No		0	A500 ...	.263	6.084	1	.933	6.084	DL+...	.023	12.1...	1

Beam Code Summary for Hot Rolled : 15'-8"

Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
No Data to Print ...														

Beam Code Summary for Hot Rolled : 12'-8"

Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
No Data to Print ...														

Beam Code Summary for Hot Rolled : 10' Plaza Level

	Label	Size	Explicit	Studs	Camb..	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
1	1B1	W12x26	Yes		0	A992	.416	5.08	5	.291	5.419	DL+...	.239	10.8...	5
2	1B3	W12x26	Yes		0	A992	.169	3.917	5	.087	3.482	DL+...	.142	0	5
3	1B2	W12x26	Yes		0	A992	.183	3.974	5	.096	3.599	DL+...	.192	0	5
4	1B4	W12x26	Yes		0	A992	.561	6.823	5	.49	6.823	DL+...	.256	0	5
5	1B7	W12x26	Yes		0	A992	.01	10.345	2	.025	0	LL	.011	10.3...	2
6	1B20	W14x30	Yes		0	A992	.501	11.623	5	.346	17.167	DL+...	.098	11.4...	3

Beam Code Summary for Hot Rolled : 10' Plaza Level (Continued)

	Label	Size	Explicit	Studs	Camb...	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear ...	Loc[ft]	LC
7	1B19	W14x30	Yes		0	A992	.323	11.623	3	.169	17.167	DL+...	.08	11.4...	3
8	1B8	W14x68	Yes		0	A992	.042	3.682	5	.041	7.715	DL+...	.032	0	5
9	1B26	W12x26	Yes		0	A992	.005	3.521	2	.036	0	LL	.005	0	2
10	1B6	W12x26	Yes		0	A992	.131	3.771	5	.07	3.771	DL+...	.108	7.542	5
11	1B11	W14x61	Yes		0	A992	.063	3.159	5	.029	2.958	DL+...	.105	4.813	5
12	1B10	W10x22	Yes		0	A992	.037	6.137	3	.025	0	LL	.026	6.312	3
13	1B14	W10x22	Yes		0	A992	.092	4.036	5	.041	10.172	DL+...	.044	4.036	5
14	1B17	W12x26	Yes		0	A992	.512	3.961	5	.227	3.961	DL+...	.291	7.042	5
15	1B16	W12x26	Yes		0	A992	.025	4.229	5	.025	0	LL	.019	0	5
16	1B18	W12x26	Yes		0	A992	.027	4.229	5	.025	0	LL	.022	0	5
17	1B5	W12x26	Yes		0	A992	.046	2.206	5	.025	0	LL	.055	4.813	5
18	1B24	W12x40	Yes		0	A992	.143	4.066	5	.085	4.066	DL+...	.129	8.132	5
19	1B15	W12x26	Yes		0	A992	.025	4.229	5	.025	0	LL	.019	0	5
20	1B9	W12x26	Yes		0	A992	.064	8.241	3	.075	8.241	DL+...	.025	16.8...	3
21	1B28	W12x26	Yes		0	A992	0	.458	5	.025	0	LL	.003	0	5
22	1B29	W12x26	Yes		0	A992	0	.625	5	.025	0	LL	0	0	5

Beam Code Summary for Wood : 25' 1/4" Roof

	Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
1	RB19	3-1.75X14FS	Yes	2.0E ...	.829	6.552	8	.699	6.167	DL+...	.704	0	8
2	RB14	3-1.75X14FS	Yes	2.0E ...	.832	5.91	8	.698	6.167	DL+...	.726	12.3...	8
3	RB13	3-1.75X14FS	Yes	2.0E ...	.829	6.552	8	.699	6.167	DL+...	.704	0	8
4	RB9	3-1.75X14FS	Yes	2.0E ...	.83	5.782	8	.697	6.167	DL+...	.701	12.3...	8
5	RB4	3-1.75X14FS	Yes	2.0E ...	.817	6.167	8	.685	6.167	DL+...	.75	0	8
6	RB8	3-1.75X14FS	Yes	2.0E ...	.817	6.167	8	.685	6.167	DL+...	.75	12.3...	8
7	M120	2X2	No	Spruc...	.008	.166	8	.025	0	LL	.03	0	8
8	M121	2X2	No	Spruc...	.005	.125	8	.025	0	LL	.023	0	8
9	M107	3-1.74X14FS	Yes	2.0E ...	.836	6.424	8	.702	6.167	DL+...	.73	0	8
10	M94	2-1.75X14FS	Yes	2.0E ...	.003	1.813	8	.025	0	LL	.009	0	8
11	M137	2X2	No	Spruc...	.06	.458	8	.025	0	LL	.08	0	8
12	RB3	3-1.75X14FS	Yes	2.0E ...	.83	5.782	8	.697	6.167	DL+...	.701	12.3...	8

Beam Code Summary for Wood : 23'-4" V3 Nanny

	Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
No Data to Print ...													

Beam Code Summary for Wood : 22' V2 Top of Window

	Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
1	3B5	3-1.75X11.875FS	Yes	2.0E ...	.058	6.084	6	.043	6.084	DL+...	.043	12.1...	6

Beam Code Summary for Wood : 19' V1 Mezz V3 Study

	Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
1	2B12	3-1.75X11.875FS	Yes	2.0E ...	.687	6.488	7	.798	8.066	DL+...	.412	0	7
2	2B11	2-1.75X11.875FS	Yes	2.0E ...	.553	5.959	7	.459	5.959	DL+...	.379	0	7
3	2B13	2-1.75X11.875FS	Yes	2.0E ...	.076	6.5	7	.067	6.5	DL+...	.052	0	7
4	2B2	2-1.75X11.875FS	Yes	2.0E ...	.243	8.767	7	.278	8.767	DL+...	.134	16.8...	7
5	2B3	2-1.75X11.875FS	Yes	2.0E ...	0	.791	6	.025	0	LL	.001	0	6
6	M22	2-1.75X11.875FS	Yes	2.0E ...	.006	1.813	7	.025	0	LL	.015	0	7
7	2B24	2-1.75X11.875FS	Yes	2.0E ...	.015	1.823	6	.025	0	LL	.037	3.646	6
8	2B1	2-1.75X11.875FS	Yes	2.0E ...	.013	2.456	7	.025	0	LL	.024	0	7

Beam Code Summary for Wood : 19' V1 Mezz V3 Study (Continued)

	Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
9	M43	2X2	No	2.0E ...	.046	.458	7	.025	0	LL	.058	0	7

Beam Code Summary for Wood : 17.5' V2 V3 Top of (low) windows

	Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
1	2B4	2-1.75X11.875FS	Yes	2.0E ...	.221	2.625	7	.089	3	DL+...	.386	0	7
2	2B6	3-1.75X11.875FS	Yes	2.0E ...	.109	5.75	6	.077	5.75	DL+...	.086	0	6

Beam Code Summary for Wood : 15'-8"

Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
No Data to Print ...												

Beam Code Summary for Wood : 12'-8"

Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
No Data to Print ...												

Beam Code Summary for Wood : 10' Plaza Level

	Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
1	1B21	2-1.75X11.875FS	Yes	2.0E ...	.124	1.25	7	.035	1.597	DL+...	.291	3.333	7
2	1B13	2-1.75X11.875FS	Yes	2.0E ...	.13	4.297	7	.083	4.488	DL+...	.123	0	7
3	1B22	2-1.75X11.875FS	Yes	2.0E ...	.013	1.49	7	.025	0	LL	.033	0	7
4	1B27	3-1.75X14FS	Yes	2.0E ...	.128	7.75	8	.132	7.75	DL+...	.088	0	8
5	1B23	2-1.75X11.875FS	Yes	2.0E ...	.068	1.557	7	.025	0	LL	.139	3.646	7
6	M70	2-1.75X14FS	Yes	2.0E ...	.055	1.625	8	.025	0	LL	.137	3.25	8
7	1B12	2-1.75X14FS	Yes	2.0E ...	.134	3.143	8	.058	2.959	DL+...	.256	0	8
8	1J3A	2X10	Yes	Spruc...	.058	1.834	7	.025	0	LL	.1	3.667	7
9	1J3B	2X10	Yes	Spruc...	.073	1.834	7	.025	0	LL	.126	3.667	7
10	1J3C	2X10	Yes	Spruc...	.073	1.834	7	.025	0	LL	.126	3.667	7
11	1J3D	2X10	Yes	Spruc...	.073	1.834	7	.025	0	LL	.126	3.667	7
12	1J3E	2X10	Yes	Spruc...	.073	1.834	7	.025	0	LL	.126	3.667	7
13	1J3F	2X10	Yes	Spruc...	.073	1.834	7	.025	0	LL	.126	3.667	7
14	1J3G	2X10	Yes	Spruc...	.073	1.834	7	.025	0	LL	.126	3.667	7
15	1J3H	2X10	Yes	Spruc...	.073	1.834	7	.025	0	LL	.126	3.667	7
16	1J3I	2X10	Yes	Spruc...	.058	1.834	7	.025	0	LL	.1	3.667	7
17	1J5A	2X6	Yes	Spruc...	.163	1.625	7	.056	1.625	DL+...	.223	0	7
18	1J5B	2X6	Yes	Spruc...	.18	1.625	7	.062	1.625	DL+...	.245	0	7
19	1J5C	2X6	Yes	Spruc...	.18	1.625	7	.062	1.625	DL+...	.245	0	7
20	1J5D	2X6	Yes	Spruc...	.18	1.625	7	.062	1.625	DL+...	.245	0	7
21	1J5E	2X6	Yes	Spruc...	.18	1.625	7	.062	1.625	DL+...	.245	0	7
22	1J5F	2X6	Yes	Spruc...	.18	1.625	7	.062	1.625	DL+...	.245	0	7
23	1J5G	2X6	Yes	Spruc...	.18	1.625	7	.062	1.625	DL+...	.245	0	7
24	1J5H	2X6	Yes	Spruc...	.163	1.625	7	.056	1.625	DL+...	.223	0	7
25	M139	2X2	No	Spruc...	.184	.411	6	.053	.411	DL+...	.272	0	6
26	M140	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
27	M141	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
28	M142	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
29	M143	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
30	M144	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
31	M145	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	1	6
32	M146	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
33	M147	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6

Beam Code Summary for Wood : 10' Plaza Level (Continued)

	Label	Size	Explicit	Material	Bending Check	Loc[ft]	LC	Defl Ch...	Loc[ft]	Cat	Shear Check	Loc[ft]	LC
34	M148	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
35	M149	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
36	M150	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
37	M155	2X2	No	Spruc...	.272	.5	6	.096	.5	DL+...	.33	0	6
38	M156	2X2	No	Spruc...	.184	.411	6	.053	.411	DL+...	.272	.823	6
39	1B25	2-1.75X14FS	Yes	2.0E ...	.031	3.193	8	.025	0	LL	.052	0	8
40	DRAG	2X4	No	Spruc...	.647	1.97	7	.426	1.97	DL+...	.465	3.939	7
41	DRAG159	2X6	No	Spruc...	.476	2.456	7	.216	2.456	DL+...	.374	0	7

Beam Design for Wood Products : 25' 1/4" Roof

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
1	RJ2-80	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
2	RJ2-81	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
3	RJ2-82	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
4	RJ2-83	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
5	RJ2-84	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
6	RJ2-85	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
7	RJ2-86	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
8	RJ2-87	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
9	RJ2-88	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
10	RJ2-89	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
11	RJ2-90	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
12	RJ2-91	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
13	RJ2-92	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
14	RJ2-93	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
15	RJ2-94	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
16	RJ2-95	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
17	RJ2-96	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
18	RJ2-97	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
19	RJ2-98	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
20	RJ2-99	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
21	RJ2-100	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
22	RJ2-101	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
23	RJ2-102	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
24	RJ2-103	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
25	RJ2-104	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
26	RJ2-105	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
27	RJ2-106	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
28	RJ2-107	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
29	RJ2-108	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
30	RJ2-109	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
31	RJ2-110	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
32	RJ2-111	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
33	RJ2-112	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
34	RJ2-113	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
35	RJ2-114	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
36	RJ2-115	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
37	RJ2-116	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
38	RJ2-117	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
39	RJ2-118	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
40	RJ2-119	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278

Beam Design for Wood Products : 25' 1/4" Roof (Continued)

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
41	RJ2-120	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
42	RJ2-121	Red_I90x14	Yes	-1.88	2.921	-7.911	13.144	1.88	1.88	.277	.277
43	RJ2-122	Red_I90x14	Yes	-1.88	2.921	-7.911	13.144	1.88	1.88	.277	.277
44	RJ2-123	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
45	RJ2-124	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
46	RJ2-125	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
47	RJ2-126	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
48	RJ2-127	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
49	RJ2-128	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
50	RJ2-129	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
51	RJ2-130	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
52	RJ2-131	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
53	RJ2-132	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
54	RJ2-133	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
55	RJ2-134	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
56	RJ2-135	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
57	RJ2-136	Red_I90x14	Yes	-1.881	2.921	-7.918	13.144	1.881	1.881	.278	.278
58	RJ1-82	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
59	RJ1-83	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
60	RJ1-84	Red_I90x14	Yes	-1.887	2.921	-7.94	13.144	1.887	1.887	.278	.278
61	RJ1-85	Red_I90x14	Yes	-1.887	2.921	-7.94	13.144	1.887	1.887	.278	.278
62	RJ1-86	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
63	RJ1-87	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
64	RJ1-88	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
65	RJ1-89	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
66	RJ1-90	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
67	RJ1-91	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
68	RJ1-92	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
69	RJ1-93	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
70	RJ1-94	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
71	RJ1-95	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
72	RJ1-96	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
73	RJ1-97	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
74	RJ1-98	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
75	RJ1-99	Red_I90x14	Yes	-1.888	2.921	-7.947	13.144	1.888	1.888	.279	.279
76	RJ1-100	Red_I90x14	Yes	-1.707	2.921	-7.182	13.144	1.707	1.707	.256	.256
77	RJ1-101	Red_I90x14	Yes	-1.694	2.921	-7.129	13.144	1.694	1.694	.254	.254
78	RJ1-102	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
79	RJ1-103	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
80	RJ1-104	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
81	RJ1-105	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
82	RJ1-106	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
83	RJ1-107	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
84	RJ1-108	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
85	RJ1-109	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
86	RJ1-110	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
87	RJ1-111	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
88	RJ1-112	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
89	RJ1-113	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
90	RJ1-114	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
91	RJ1-115	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
92	RJ1-116	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276

Beam Design for Wood Products : 25' 1/4" Roof (Continued)

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
93	RJ1-117	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
94	RJ1-118	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
95	RJ1-119	Red_I90x14	Yes	-1.863	2.921	-7.84	13.144	1.863	1.863	.276	.276
96	RJ1-120	Red_I90x14	Yes	-1.693	2.921	-7.124	13.144	1.693	1.693	.253	.253
97	RJ1-121	Red_I90x14	Yes	-1.695	2.921	-7.132	13.144	1.695	1.695	.254	.254
98	RJ1-122	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
99	RJ1-123	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
100	RJ1-124	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
101	RJ1-125	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
102	RJ1-126	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
103	RJ1-127	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
104	RJ1-128	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
105	RJ1-129	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
106	RJ1-130	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
107	RJ1-131	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
108	RJ1-132	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
109	RJ1-133	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
110	RJ1-134	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
111	RJ1-135	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
112	RJ1-136	Red_I90x14	Yes	-1.864	2.921	-7.846	13.144	1.864	1.864	.276	.276
113	M138	Red_I90x14	Yes	-.964	2.921	-4.059	13.144	.964	.964	.16	.16
114	M139	Red_I90x14	Yes	-.951	2.921	-4	13.144	.951	.951	.157	.157
115	M140	Red_I90x14	Yes	-.952	2.921	-4.006	13.144	.952	.952	.158	.158
116	M141	Red_I90x14	Yes	-.953	2.921	-4.008	13.144	.953	.953	.158	.158

Beam Design for Wood Products : 23'-4" V3 Nanny

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
1	3J1A	Red_I45x9.5	Yes	.701	1.59	-2.272	3.62	.701	.701	.37	.37
2	3J1B	Red_I45x9.5	Yes	.729	1.59	-2.363	3.62	.729	.729	.385	.385
3	3J1C	Red_I45x9.5	Yes	.729	1.59	-2.363	3.62	.729	.729	.385	.385
4	3J1D	Red_I45x9.5	Yes	.729	1.59	-2.363	3.62	.729	.729	.385	.385
5	3J1E	Red_I45x9.5	Yes	.729	1.59	-2.363	3.62	.729	.729	.385	.385
6	3J1F	Red_I45x9.5	Yes	.729	1.59	-2.363	3.62	.729	.729	.385	.385
7	3J1G	Red_I45x9.5	Yes	.729	1.59	-2.363	3.62	.729	.729	.385	.385
8	3J1H	Red_I45x9.5	Yes	.729	1.59	-2.363	3.62	.729	.729	.385	.385
9	3J1I	Red_I45x9.5	Yes	.729	1.59	-2.363	3.62	.729	.729	.385	.385
10	3J1J	Red_I45x9.5	Yes	.461	1.59	-1.495	3.62	.461	.461	.246	.246

Beam Design for Wood Products : 22' V2 Top of Window

Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
No Data to Print ...										

Beam Design for Wood Products : 19' V1 Mezz V3 Study

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
1	2J2A	Red_I45x11.875	Yes	.725	1.785	-2.355	4.685	.725	.725	.383	.383
2	2J2B	Red_I45x11.875	Yes	.735	1.785	-2.39	4.685	.735	.735	.389	.389
3	2J2C	Red_I45x11.875	Yes	.735	1.785	-2.39	4.685	.735	.735	.389	.389
4	2J2D	Red_I45x11.875	Yes	.735	1.785	-2.39	4.685	.735	.735	.389	.389
5	2J2E	Red_I45x11.875	Yes	.735	1.785	-2.39	4.685	.735	.735	.389	.389
6	2J2F	Red_I45x11.875	Yes	.735	1.785	-2.39	4.685	.735	.735	.389	.389

Beam Design for Wood Products : 19' V1 Mezz V3 Study (Continued)

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
7	2J2G	Red_I45x11.875	Yes	.735	1.785	-2.39	4.685	.735	.735	.389	.389
8	2J2H	Red_I45x11.875	Yes	.725	1.785	-2.355	4.685	.725	.725	.383	.383
9	2J3A	Red_I45x11.875	Yes	.334	1.785	-.543	4.685	.334	.334	.177	.177
10	2J3B	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
11	2J3C	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
12	2J3D	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
13	2J3E	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
14	2J3F	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
15	2J3G	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
16	2J3H	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
17	2J3I	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
18	2J3J	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
19	2J3K	Red_I45x11.875	Yes	.368	1.785	-.598	4.685	.368	.368	.194	.194
20	2J3L	Red_I45x11.875	Yes	.334	1.785	-.543	4.685	.334	.334	.177	.177
21	2J1-31	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
22	2J1-32	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
23	2J1-33	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
24	2J1-34	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
25	2J1-35	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
26	2J1-36	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
27	2J1-37	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
28	2J1-38	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
29	2J1-39	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
30	2J1-40	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485
31	2J1-41	Red_I45x11.875	Yes	-.917	1.785	-3.86	4.685	.917	.917	.485	.485

Beam Design for Wood Products : 17.5' V2 V3 Top of (low) windows

Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
No Data to Print ...										

Beam Design for Wood Products : 15'-8"

Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
No Data to Print ...										

Beam Design for Wood Products : 12'-8"

Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
No Data to Print ...										

Beam Design for Wood Products : 10' Plaza Level

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
1	1J4A	Red_I45x11.875	Yes	-.749	1.785	-3.154	4.685	.749	.749	.398	.398
2	1J4B	Red_I45x11.875	Yes	-1.039	1.785	-4.373	4.685	1.039	1.039	.548	.548
3	1J4D	Red_I45x11.875	Yes	-.873	1.785	-3.673	4.685	.873	.873	.462	.462
4	1J4E	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
5	1J4G	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
6	1J4H	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
7	1J4I	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
8	1J4J	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
9	1J4K	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
10	1J4L	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503

Beam Design for Wood Products : 10' Plaza Level (Continued)

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
11	1J4M	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
12	1J4N	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
13	1J4O	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
14	1J4P	Red_I45x11.875	Yes	-.749	1.785	-3.154	4.685	.749	.749	.398	.398
15	1J4Y	Red_I45x11.875	Yes	-.778	1.785	-3.274	4.685	.778	.778	.413	.413
16	1J4Z	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
17	1J4AA	Red_I45x11.875	Yes	-.952	1.785	-4.007	4.685	.952	.952	.503	.503
18	1J4BB	Red_I45x11.875	Yes	-.778	1.785	-3.274	4.685	.778	.778	.413	.413
19	1J2A	Red_I45x11.875	Yes	.122	1.785	-.099	4.685	.122	.122	.065	.065
20	1J2B	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
21	1J2C	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
22	1J2D	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
23	1J2E	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
24	1J2F	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
25	1J2G	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
26	1J2H	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
27	1J2I	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
28	1J2J	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
29	1J2K	Red_I45x11.875	Yes	.139	1.785	-.113	4.685	.139	.139	.074	.074
30	1J2L	Red_I45x11.875	Yes	.122	1.785	-.099	4.685	.122	.122	.065	.065
31	PUR86	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
32	PUR87	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
33	PUR88	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
34	PUR89	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
35	PUR90	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
36	PUR91	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
37	PUR101	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
38	PUR102	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
39	PUR103	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
40	PUR104	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
41	PUR105	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
42	PUR106	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
43	PUR107	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
44	PUR116	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
45	PUR117	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
46	PUR118	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
47	PUR119	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
48	PUR120	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
49	PUR121	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
50	PUR122	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
51	PUR123	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
52	PUR124	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
53	PUR125	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
54	PUR126	Red_I90x14	Yes	2.572	2.921	-9.967	13.144	2.572	2.572	.449	.449
55	PUR153	Red_I90x14	Yes	-1.998	2.921	-6.578	13.144	1.247	2.056	.472	.462
56	PUR154	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
57	PUR155	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
58	PUR156	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
59	PUR157	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
60	PUR158	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
61	PUR159	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
62	PUR160	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503

Beam Design for Wood Products : 10' Plaza Level (Continued)

	Label	Size	Explicit	Vmax[k]	V'[k]	Mmax[k-...	M'r[k-ft]	Max Start Re...	Max End ...	Min Start R...	Min End Rea...
63	PUR161	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
64	PUR162	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
65	PUR163	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
66	PUR164	Red_I90x14	Yes	-2.189	2.921	-7.205	13.144	1.364	2.253	.514	.503
67	PUR165	Red_I90x14	Yes	-1.998	2.921	-6.578	13.144	1.247	2.056	.472	.462
68	PUR138	Red_I90x14	Yes	-.278	2.54	-.49	11.43	.278	.278	.166	.166
69	PUR139	Red_I90x14	Yes	-.345	2.54	-.607	11.43	.345	.345	.204	.204
70	PUR140	Red_I90x14	Yes	-.278	2.54	-.49	11.43	.278	.278	.166	.166
71	PUR144	Red_I90x14	Yes	1.123	2.921	-2.374	13.144	1.123	1.123	.199	.199
72	PUR145	Red_I90x14	Yes	1.404	2.921	-2.968	13.144	1.404	1.404	.245	.245
73	PUR146	Red_I90x14	Yes	1.123	2.921	-2.374	13.144	1.123	1.123	.199	.199
74	PUR187	Red_I90x14	Yes	-.953	2.921	-1.677	13.144	.953	.953	.169	.169
75	PUR188	Red_I90x14	Yes	.929	2.921	-1.635	13.144	.929	.91	.165	.162
76	PUR215	Red_I90x14	Yes	1.144	2.921	-2.42	13.144	1.144	1.144	.203	.203
77	PUR178	Red_I90x14	Yes	.91	2.921	-1.531	13.144	.91	.887	.161	.158
78	PUR204	Red_I90x14	Yes	.608	2.921	-.96	13.144	.608	.576	.111	.106
79	PUR205	Red_I90x14	Yes	.982	2.921	-1.454	13.144	.982	.951	.171	.166
80	PUR206	Red_I90x14	Yes	.915	2.921	-1.262	13.144	.915	.884	.16	.155
81	PUR207	Red_I90x14	Yes	.848	2.921	-1.084	13.144	.848	.817	.148	.143
82	PUR208	Red_I90x14	Yes	.781	2.921	-.92	13.144	.781	.75	.136	.131
83	PUR209	Red_I90x14	Yes	.634	2.921	-.683	13.144	.634	.616	.112	.109
84	PUR214	Red_I90x14	Yes	-1.291	2.921	-2.829	13.144	1.272	1.291	.224	.227
85	PUR216	Red_I90x14	Yes	-1.522	2.921	-3.488	13.144	1.49	1.522	.26	.265
86	PUR217	Red_I90x14	Yes	-1.589	2.921	-3.802	13.144	1.557	1.589	.272	.277
87	PUR218	Red_I90x14	Yes	-1.656	2.921	-4.129	13.144	1.623	1.656	.284	.289
88	PUR219	Red_I90x14	Yes	-1.723	2.921	-4.47	13.144	1.69	1.723	.295	.3
89	PUR220	Red_I90x14	Yes	-1.789	2.921	-4.824	13.144	1.757	1.789	.307	.312
90	PUR221	Red_I90x14	Yes	-1.647	2.921	-4.606	13.144	1.615	1.647	.285	.29
91	1J4-154	Red_I45x11.875	Yes	-.851	1.785	-3.58	4.685	.851	.851	.451	.451
92	1J4-155	Red_I45x11.875	Yes	-.851	1.785	-3.58	4.685	.851	.851	.451	.451
93	1J4-156	Red_I45x11.875	Yes	-.851	1.785	-3.58	4.685	.851	.851	.451	.451
94	1J4-157	Red_I45x11.875	Yes	-.851	1.785	-3.58	4.685	.851	.851	.451	.451
95	1J4-158	Red_I45x11.875	Yes	-.851	1.785	-3.58	4.685	.851	.851	.451	.451
96	1J4-159	Red_I45x11.875	Yes	-.851	1.785	-3.58	4.685	.851	.851	.451	.451
97	1J4-160	Red_I45x11.875	Yes	-.851	1.785	-3.58	4.685	.851	.851	.451	.451

Hot Rolled Steel Column Code Checks

	Stack	...	Shape	Code Check	Elev[ft]	...	Shear Check	Ele...	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn ...	phi*Mn z-...	Cb	Eqn
1	(5-E)	1	HSS4...	.559	0	5	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
2		2	HSS4...	.324	10.216	5	.000	10....		10	99.405	139.518	16.181	16.181	1	H1-...
3	(5-F)	1	HSS4...	.668	0	5	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
4		2	HSS4...	.800	10	5	.000	10		10	47.756	139.518	16.181	16.181	1	H1-...
5	(1-F)	1	HSS4...	.238	0	5	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
6	1	1	HSS4...	.308	0	5	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
7	(3-F)	1	HSS4...	.391	0	5	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
8	2	1	HSS4...	.039	0	3	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
9	3	1	HSS4...	.391	0	5	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
10	4	1	HSS4...	.305	0	5	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
11	5	1	HSS4...	.019	0	3	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
12	(5-D)	1	HSS4...	.424	19.032	5	.000	0		10	84.454	139.518	16.181	16.181	1	H1-...
13	(6-F)	1	HSS4...	.604	0	5	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...

Hot Rolled Steel Column Code Checks (Continued)

	Stack	...	Shape	Code Check	Elev[ft]	...	Shear Check	Ele...	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn ...	phi*Mn z-...	Cb	Eqn
14		2	HSS4...	.747	10	5	.000	10		10	47.756	139.518	16.181	16.181	1	H1-...
15	(3-E)	1	HSS5...	.212	0	5	.000	0		10	166.542	217.764	31.602	31.602	1	H1-...
16		2	HSS5...	.319	10	5	.000	10		10	92.007	217.764	31.602	31.602	1	H1-...
17	7	1	HSS5...	.138	0	5	.000	0		10	243.236	326.232	45.195	45.195	1	H1-...
18	8	1	HSS4...	.161	0	5	.000	0		10	91.807	139.518	16.181	16.181	1	H1-...
19	9	1	HSS5...	.063	0	5	.000	0		10	243.236	326.232	45.195	45.195	1	H1-...
20		2	HSS5...	.015	10	5	.000	10		10	133.89	326.232	45.195	45.195	1	H1-...
21	(6-E)	1	HSS4...	.354	10	5	.000	10		10	99.405	139.518	16.181	16.181	1	H1-...
22	(6-F')	1	HSS4...	.003	10	1	.000	10		10	82.281	249.228	26.565	26.565	1	H1-...
23	(6-D)	1	HSS4...	.424	19.089	5	.000	15....		10	84.454	139.518	16.181	16.181	1	H1-...
24	(3-A)	1	HSS4...	.409	10	5	.000	10		10	99.405	139.518	16.181	16.181	1	H1-...
25		2	HSS4...	.825	19	5	.000	19		10	42.723	139.518	16.181	16.181	1	H1-...
26	(3-B)	1	HSS5...	.223	10	5	.000	10		10	148.006	217.764	31.602	31.602	1	H1-...
27		2	HSS5...	.271	19.05	5	.000	19....		10	119.037	217.764	31.602	31.602	1	H1-...
28	(3-C)	1	HSS5...	.605	10	5	.000	10		10	61.701	217.764	31.602	31.602	1	H1-...
29	(3-D)	1	HSS5...	.489	10	5	.000	10		10	74.879	217.764	31.602	31.602	1	H1-...
30	(4-D)	1	HSS5...	.531	10	5	.000	10		10	63.056	178.02	26.255	26.255	1	H1-...
31	(4-E)	1	HSS5...	.515	10	5	.000	10		10	77.133	178.02	26.255	26.255	1	H1-...
32	10	1	HSS5...	.018	23.426	5	.000	10		10	110.801	255.852	36.57	36.57	1	H1-...
33	11	1	HSS4...	.053	10	5	.000	10		10	82.281	249.228	26.565	26.565	1	H1-...
34	(5-B)	1	HSS4...	.584	19	5	.000	19		10	54.332	139.518	16.181	16.181	1	H1-...
35	(5-C)	1	HSS4...	.491	19	5	.000	19		10	68.932	139.518	16.181	16.181	1	H1-...
36	(3-A')	1	HSS4...	.098	19	5	.000	19		10	44.22	169.74	19.285	19.285	1	H1-...
37	(4-A')	1	HSS4...	.027	19	5	.000	19		10	138.935	139.518	16.181	16.181	1	H1-...
38	(4-A)	1	HSS4...	.824	19	5	.000	19		10	42.723	139.518	16.181	16.181	1	H1-...
39	(4-B)	1	HSS4...	.591	19	5	.000	19		10	54.332	139.518	16.181	16.181	1	H1-...
40	(4-C)	1	HSS4...	.522	19	5	.000	19		10	68.932	139.518	16.181	16.181	1	H1-...
41	13	1	HSS4...	.003	22.024	3	.000	19		10	138.935	139.518	16.181	16.181	1	H1-...
42	14	1	HSS4...	.037	19	1	.000	19		10	128.975	139.518	16.181	16.181	1	H1-...
43	15	1	HSS4...	.018	23.389	5	.000	19		10	81.98	139.518	16.181	16.181	1	H1-...
44	(6-B)	1	HSS4...	.434	23.377	5	.000	19		10	86.602	139.518	16.181	16.181	1	H1-...
45	(6-C)	1	HSS4...	.471	19	5	.000	19		10	68.932	139.518	16.181	16.181	1	H1-...
46	16	1	HSS4...	.039	19	5	.000	19		10	44.22	169.74	19.285	19.285	1	H1-...

Wood Column Code Checks

	Stack	Lift	Shape	Code C...	Elev[ft]	LC	Shear C...	Elev[ft]	Dir	LC	Fc' [ksi]	Ft' [ksi]	Fb1' [ksi]	Fb2' [ksi]	Fv' [ksi]	Eqn
1	12	1	4-2X6	.130	10	7	.000	10	z	10	.807	.585	1.136	1.308	.135	3.6.3

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Gravity Wall Utilization

Wall Results, Wood Wall Panel

	Wall Panel	Region	Stud Size	Stud Spacing[in]	Axial Check	Gov LC
1	STUD WALL 1	R1	2X6	16	.09	7
2	STUD WALL 2	R1	2X6	16	.039	7
3	STUD WALL 3	R1	2X6	16	.018	6
4	STUD WALL 4	R1	2X6	16	.018	6
5	STUD WALL 5	R1	2X6	16	.115	8
6	SW-WP7 B	R1	2X6	16	.146	7
7	SW-WP11 A	R1	2X6	16	.353	7
8	SW-WP11 B	R1	2X6	16	.368	7
9	SW-WP8 UPP...	R1	2X6	16	.007	8
10	SW-WP8 LO...	R1	2X6	16	.009	8
11	SW-WP13	R1	2X6	16	.019	7
12	SW-WP14	R1	2X6	16	.06	7
13	SW-WP12 B	R1	2X6	16	.125	7
14	STUD WALL 8	R1	2X6	16	.105	7
15	SW-WP6 LO...	R1	2X6	16	.059	7
16	SW-WP7 A	R1	2X6	16	.141	7
17	STUD WALL 6	R1	2X6	16	.086	7
18	SW-WP12 A	R1	2X6	16	.102	7
19	STUD WALL 7	R1	2X6	16	.017	6
20	SW-WP10	R1	2X6	16	.025	10
21	SW-WP9	R1	2X6	16	.009	8

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VOLUME 2, 3 AND BASEMENT LATERAL SYSTEM

Designed using RISA3D linked with RISAFloor

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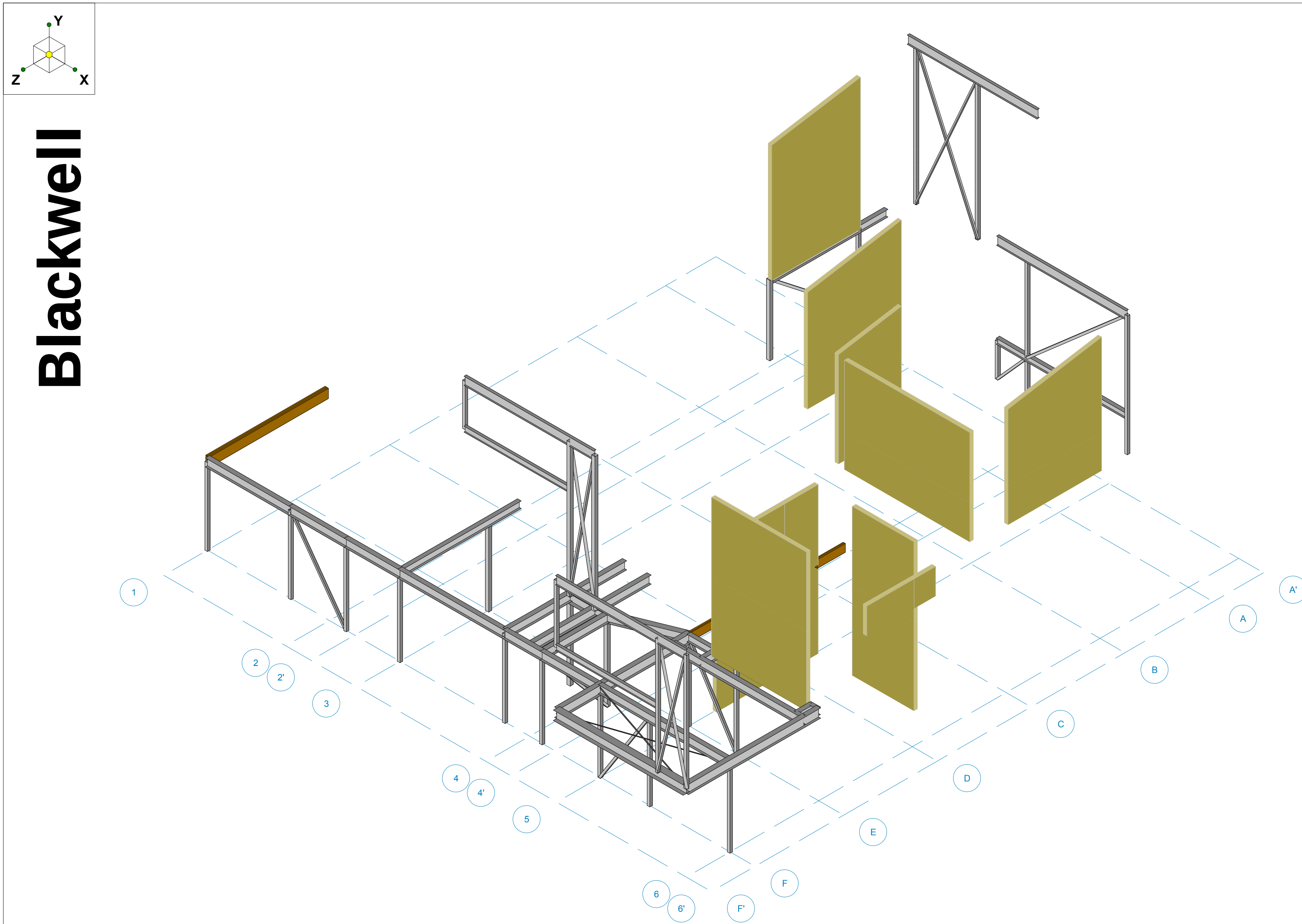
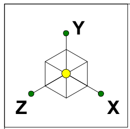


*RENDERED VIEW SHOWN FOR CONTEXT ONLY. REFER TO MEMBER PROPERTIES AND STRUCTURAL DRAWINGS FOR DETAILS.

Blackwell Structural Engineers	Kimmelman May Residence Volume 2, 3 and 4	GENERAL LATERAL RENDER
BG		Feb 15, 2018 at 5:17 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

Lateral Geometry Definition



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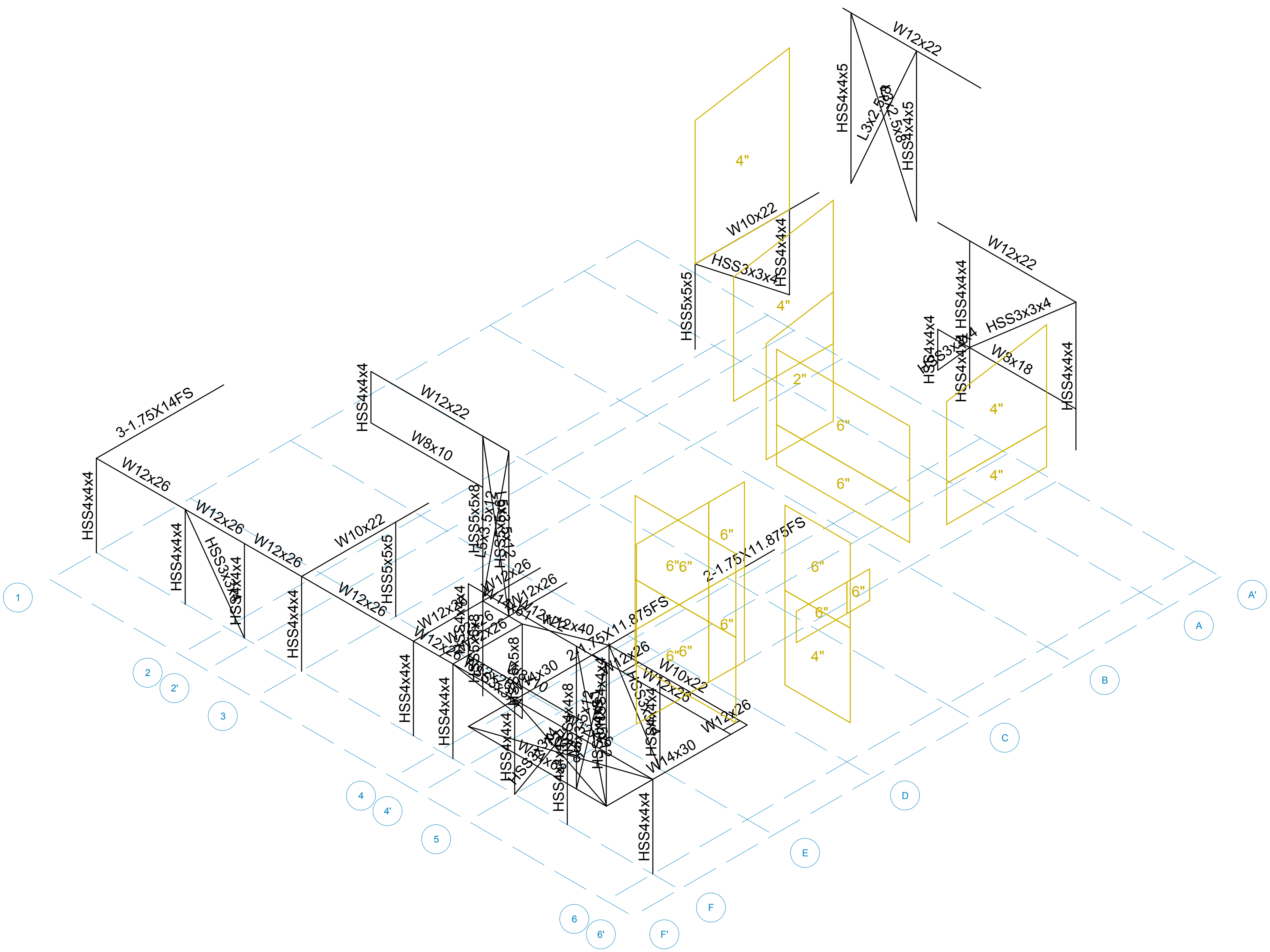
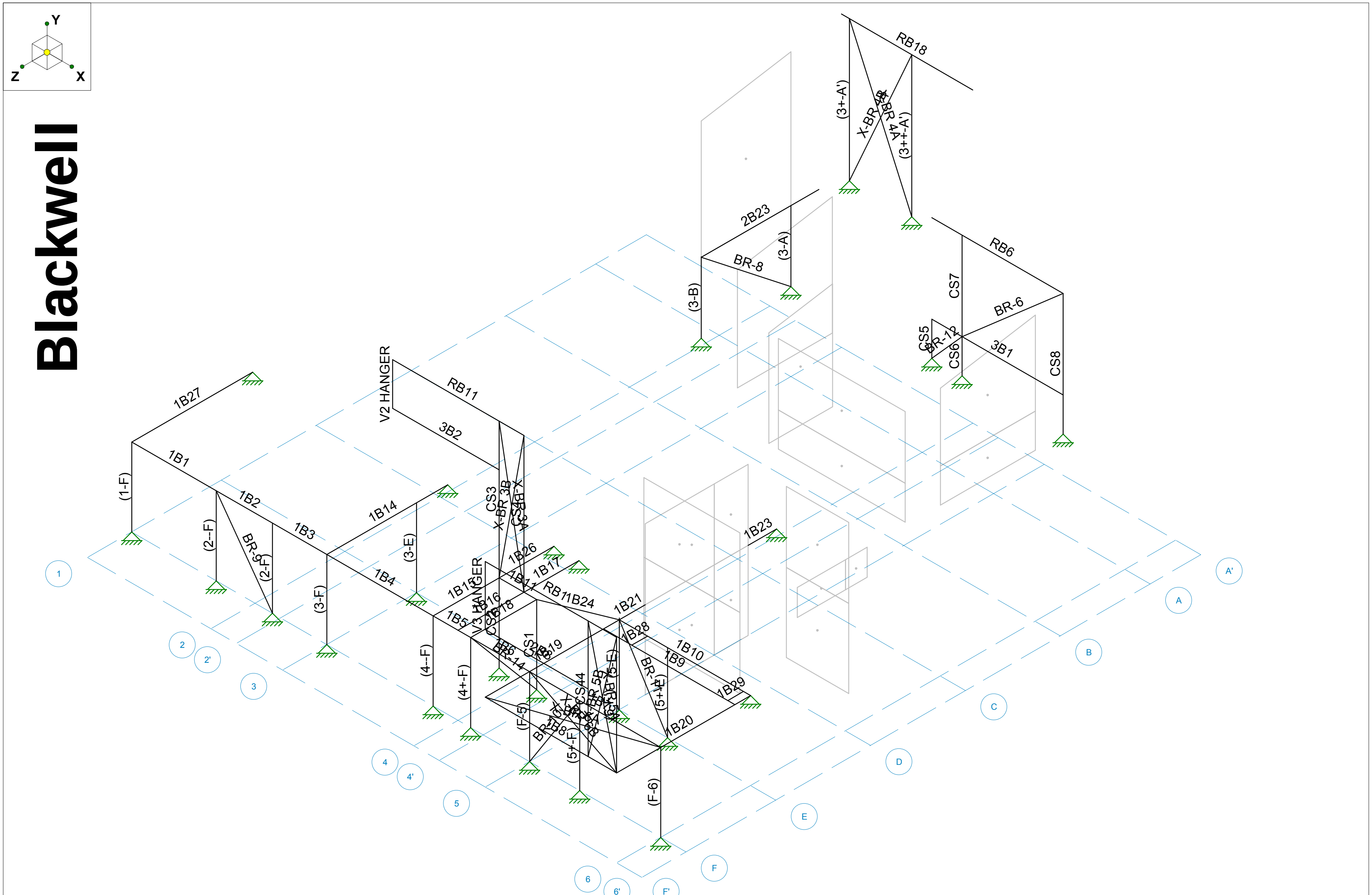


Blackwell Structural Engineers	Kimmelman May Residence Volume 2, 3 and 4	SHAPES AND NAILING PATTERN
BG		Feb 15, 2018 at 5:20 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

Lateral Wall and Member Designation



NOTE THAT COLUMN LABELS DO NOT NECESSARILY CORRELATE WITH THE GRAVITY LABELS.

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Blackwell Structural Engineers	Kimmelman May Residence Volume 2, 3 and 4	MEMBER DESIGNATIONS
BG		Feb 15, 2018 at 5:46 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [in2]	Iyy [in...	Izz [in...	J [in4]
1	T/C Brace	HSS3x3x4	VBrace	None	A500 Gr.B Rect	Typical	2.44	3.02	3.02	5.08
2	V2 V3 FRONT X-BRACE	L5x3.5x12	VBrace	Single Angle	A36 Gr.36	Typical	5.85	5.52	13.9	1.09
3	CABLE	R0.75	HBrace	None	A36 Gr.36	Typical	.442	.016	.016	.031
4	V2 REAR X-BRACE	L3x2.5x8	VBrace	Single Angle	A36 Gr.36	Typical	2.5	1.29	2.07	.213

Member Primary Data

	Label	I Joint	J Joint	K Joint	Rotate(d...	Section/Shape	Type	Design List	Material	Design Rul...
1	BR-11	N111	N128			T/C Brace	VBrace	None	A500 Gr....	Typical
2	BR-10	F8_N35	N125			T/C Brace	VBrace	None	A500 Gr....	Typical
3	3B2	N137	N1			W8x10	Beam	Wide Flange	A992	Typical
4	2B8	N136	N96			W8x10	Beam	Wide Flange	A992	Typical
5	X-BR 3A	N134	F1_N200			V2 V3 FRONT X-BRACE	VBrace	Single Angle	A36 Gr.36	Typical
6	X-BR 3B	F8_GRD...	N16			V2 V3 FRONT X-BRACE	VBrace	Single Angle	A36 Gr.36	Typical
7	X-BR 5A	N109	F1_N199			V2 V3 FRONT X-BRACE	VBrace	Single Angle	A36 Gr.36	Typical
8	X-BR 5B	N236	N4			V2 V3 FRONT X-BRACE	VBrace	Single Angle	A36 Gr.36	Typical
9	BR-9	F8_N33	N179			T/C Brace	VBrace	None	A500 Gr....	Typical
10	V2 HANGER	V2_DEF	N137			HSS4x4x4	Column	Tube	A500 Gr....	Typical
11	V3 HANGER	V3_DEF	N136			HSS4x4x4	Column	Tube	A500 Gr....	Typical
12	X-BR 6B	F8_N94	N109			CABLE	HBrace	None	A36 Gr.36	Typical
13	X-BR 6A	F8_N89	F8_N95			CABLE	HBrace	None	A36 Gr.36	Typical
14	BR-14	N110	F8_N439			T/C Brace	VBrace	None	A500 Gr....	Typical
15	RB11	V2_DEF	N16			W12x22	Beam	None	A992	Typical
16	RB1	N4	V3_DEF			W12x22	Beam	None	A992	Typical
17	1B1	F8_N25	F8_N33			W12x26	Beam	None	A992	Typical
18	1B3	F8_N26	F8_N27			W12x26	Beam	None	A992	Typical
19	1B2	F8_N26	F8_N33			W12x26	Beam	None	A992	Typical
20	1B4	F8_N27	F8_N34			W12x26	Beam	None	A992	Typical
21	1B7	F8_N89	F8_N94			W12x26	Beam	None	A992	Typical
22	1B20	F8_N88	N109			W14x30	Beam	None	A992	Typical
23	1B19	N111	F8_N95			W14x30	Beam	None	A992	Typical
24	1B8	N109	F8_N95			W14x68	Beam	None	A992	Typical
25	1B21	F8_N54	N111			2-1.75X11.875FS	Beam	None	2.0E Mic...	Typical
26	1B23	F8_N327	F8_N43			2-1.75X11.875FS	Beam	None	2.0E Mic...	Typical
27	1B26	F8_GRD...	F8_N38			W12x26	Beam	None	A992	14"
28	1B6	F8_N439	F8_N94			W12x26	Beam	None	A992	Typical
29	1B11	F8_GRD...	F8_N263			W14x61	Beam	None	A992	Typical
30	1B10	N111	F8_N88			W10x22	Beam	None	A992	Typical
31	1B17	F8_N36	N134			W12x26	Beam	None	A992	Typical
32	1B16	N134	F8_N37			W12x26	Beam	None	A992	Typical
33	1B18	F8_N263	F8_N439			W12x26	Beam	None	A992	Typical
34	1B5	F8_N439	F8_N34			W12x26	Beam	None	A992	Typical
35	1B24	N111	F8_N263			W12x40	Beam	None	A992	Typical
36	(5-E)	N118	N119			HSS4x4x4	Column	None	A500 Gr....	Typical
37	(5-F)	N125	F8_N94			HSS4x4x4	Column	None	A500 Gr....	Typical
38	(1-F)	N120	F8_N25			HSS4x4x4	Column	None	A500 Gr....	Typical
39	(2-F)	N179	F8_N26			HSS4x4x4	Column	None	A500 Gr....	Typical
40	(3-F)	N121	F8_N27			HSS4x4x4	Column	None	A500 Gr....	Typical
41	(5+-E)	N128	F8_N32			HSS4x4x4	Column	None	A500 Gr....	Typical
42	(2--F)	N122	F8_N33			HSS4x4x4	Column	None	A500 Gr....	Typical

Member Primary Data (Continued)

	Label	I Joint	J Joint	K Joint	Rotate(d...	Section/Shape	Type	Design List	Material	Design Rul...
43	(4--F)	N123	F8_N34			HSS4x4x4	Column	None	A500 Gr....	Typical
44	(5+-F)	N124	F8_N35			HSS4x4x4	Column	None	A500 Gr....	Typical
45	(6-F)	N125A	F8_N89			HSS4x4x4	Column	None	A500 Gr....	Typical
46	CS1	N110	F8_N263			HSS5x5x8	Column	None	A500 Gr....	Typical
47	(4+-F)	N126	F8_N439			HSS4x4x4	Column	None	A500 Gr....	Typical
48	CS2	N127	F8_GRD...			HSS5x5x8	Column	None	A500 Gr....	Typical
49	CS3	F8_GRD...	F1_N200			HSS5x5x8	Column	None	A500 Gr....	Typical
50	(6-F')	N109	N4			HSS4x4x8	Column	None	A500 Gr....	Typical
51	CS4	N134	N16			HSS5x5x6	Column	None	A500 Gr....	Typical
52	CS44	N236	F1_N199			HSS4x4x8	Column	None	A500 Gr....	Typical
53	1B15	F8_GRD...	F8_N34			W12x26	Beam	None	A992	Typical
54	1B14	F8_N22	F8_N27			W10x22	Beam	None	A992	Typical
55	M	N113	F8_GRD...			HSS5x5x5	Column	None	A500 Gr....	Typical
56	(3+-A')	N220A	F1_N147			HSS4x4x5	Column	None	A500 Gr....	Typical
57	(3++-A')	N192	F1_N201			HSS4x4x5	Column	None	A500 Gr....	Typical
58	RB18	F1_N246	F1_N148			W12x22	Beam	None	A992	Typical
59	RB6	F1_N247	F1_N248			W12x22	Beam	None	A992	Typical
60	3B1	F2_N248	F2_N247			W8x18	Beam	None	A992	Typical
61	2B23	F4_N59	N196			W10x22	Beam	None	A992	Typical
62	M66A	N213	N195			HSS4x4x4	Column	None	A500 Gr....	Typical
63	(3-B)	N214	N215			HSS5x5x5	Column	None	A500 Gr....	Typical
64	CS5	N214A	F2_N248			HSS4x4x4	Column	None	A500 Gr....	Typical
65	BR-12	N214A	F2_N31			T/C Brace	VBrace	None	A500 Gr....	Typical
66	BR-6	F2_N31	F1_N247			T/C Brace	VBrace	None	A500 Gr....	Typical
67	BR-8	N213	N196			T/C Brace	VBrace	None	A500 Gr....	Typical
68	X-BR 4A	F1_N147	N192			V2 REAR X-BRACE	VBrace	Single Angle	A36 Gr.36	Typical
69	X-BR 4B	N220A	F1_N201			V2 REAR X-BRACE	VBrace	Single Angle	A36 Gr.36	Typical
70	CS7	F1_N242	F2_N31			HSS4x4x4	Beam	Wide Flange	A992	Typical
71	CS6	N211	F2_N31			HSS4x4x4	Beam	Wide Flange	A992	Typical
72	1B27	F8_N53	F8_N25			3-1.75X14FS	Beam	None	2.0E Mic...	Typical
73	CS8	F4_N247	F1_N247			HSS4x4x4	Column	None	A500 Gr....	Typical
74	(3-E)	N113	F8_GRD...			HSS5x5x5	Column	None	A500 Gr....	Typical
75	(F-5)	N125	F8_N94			HSS4x4x4	Column	None	A500 Gr....	Typical
76	(F-6)	N125A	F8_N89			HSS4x4x4	Column	None	A500 Gr....	Typical
77	1B9	F8_N417	F8_N418			W12x26	Beam	None	A992	Typical
78	1B28	F8_N29...	F8_N300			W12x26	Beam	None	A992	Typical
79	1B29	F8_N303	F8_N307			W12x26	Beam	None	A992	Typical
80	(3-A)	N213	N195			HSS4x4x4	Column	None	A500 Gr....	Typical

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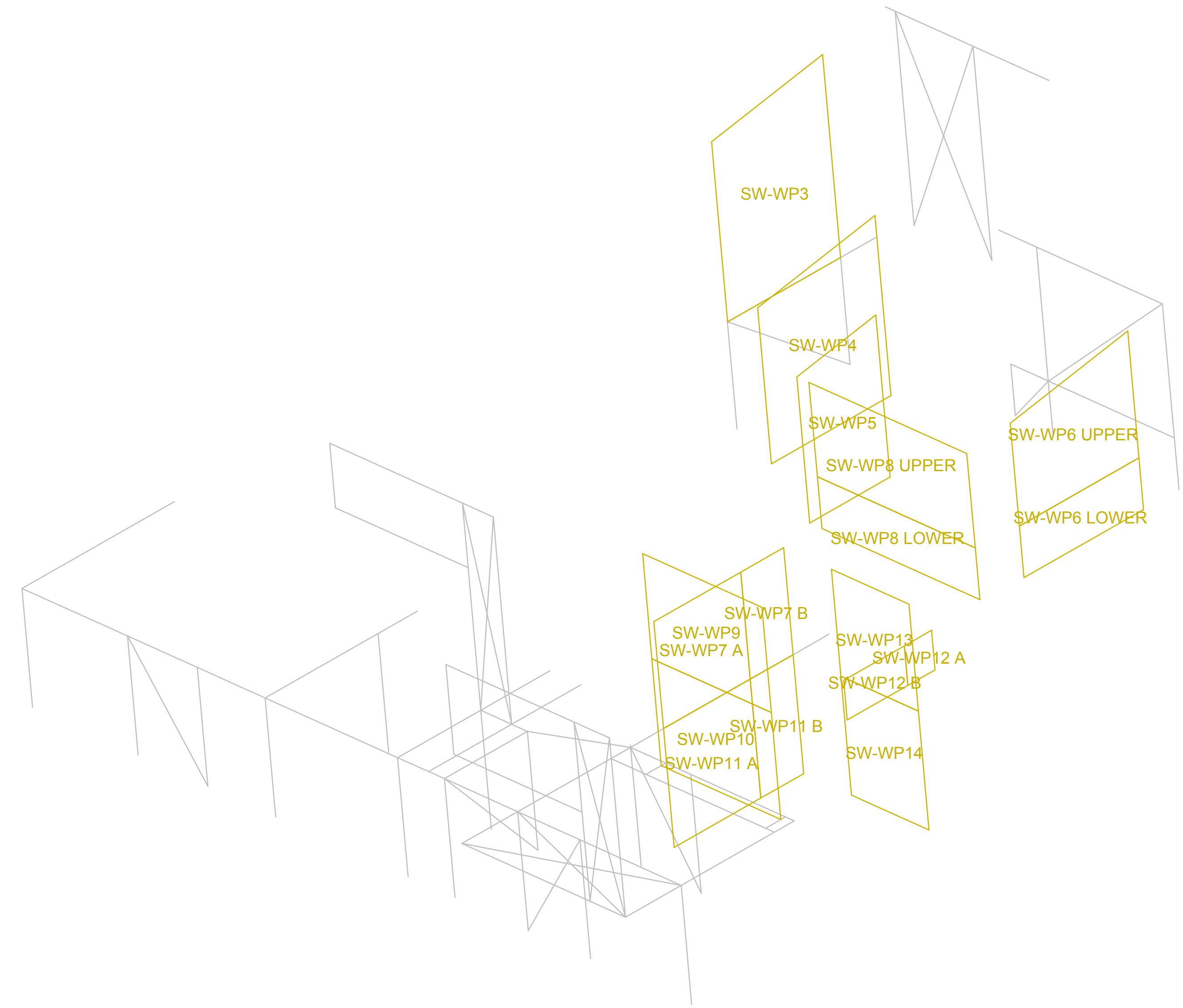

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Wall Panel Data

	Label	A Joint	B Joint	C Joint	D Joint	Material Ty..	Material Set	Thickne...	Design Rule	Panel/Spacing
1	SW-WP7 B	F4_N92	F4_N106	F8_N327	F8_N92	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
2	SW-WP11 A	F8_N54	F8_N92	N130	N199	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
3	SW-WP11 B	F8_N92	F8_N327	N131	N130	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
4	SW-WP8 LOWER	F2_N59	F2_N60	N201	N200	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
5	SW-WP13	F4_N104	F4_N119	F8_N304	F8_N61	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
6	SW-WP14	F8_N61	F8_N304	N203	N217A	Wood	Spruce-Pi...	5.5 (stud)	4"	S1_15/32_8d@4 | 16
7	SW-WP12 B	F4_N105	F4_N87	N204	N206A	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
8	SW-WP3	N216	N217	N196	N195	Wood	Spruce-Pi...	5.5 (stud)	4"	S1_15/32_8d@4 | 16
9	SW-WP4	N224	N225	N227	N226	Wood	Spruce-Pi...	5.5 (stud)	4"	S1_15/32_8d@4 | 16
10	SW-WP6 LOWER	F2_N86	F2_N84	F4_N84	F4_N86	Wood	Spruce-Pi...	5.5 (stud)	4"	S1_15/32_8d@4 | 16
11	SW-WP6 UPPER	N240A	N241A	F2_N84	F2_N86	Wood	Spruce-Pi...	5.5 (stud)	4"	S1_15/32_8d@4 | 16
12	SW-WP5	N242	N243	N245	N244	Wood	Spruce-Pi...	5.5 (stud)	2"	S1_15/32_8d@2 | 16
13	SW-WP8 UPPER	N222	N223	F2_N60	F2_N59	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
14	SW-WP7 A	F4_N118	F4_N92	F8_N92	F8_N54	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
15	SW-WP12 A	F4_N87	F4_N110	N214B	N204	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
16	SW-WP10	F4_N131	N85	N234A	N233	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16
17	SW-WP9	N219	N218	N85	F4_N131	Wood	Spruce-Pi...	5.5 (stud)	6"	S1_15/32_8d@6 | 16

Wall Panel Advanced Data

	Label	Seismic ...	Design...	SSAF	In-plane Ic...	Out-plane Ic...	K	Cm In-...	Cm Ou...	Inactive
1	SW-WP7 B	N/A	Segme...	1	N/A	N/A		N/A	N/A	
2	SW-WP11 A	N/A	Segme...	1	N/A	N/A		N/A	N/A	
3	SW-WP11 B	N/A	Segme...	1	N/A	N/A		N/A	N/A	
4	SW-WP8 LOWER	N/A	Segme...	1	N/A	N/A		N/A	N/A	
5	SW-WP13	N/A	Segme...	1	N/A	N/A		N/A	N/A	
6	SW-WP14	N/A	Segme...	1	N/A	N/A		N/A	N/A	
7	SW-WP12 B	N/A	Segme...	1	N/A	N/A		N/A	N/A	
8	SW-WP3	N/A	Segme...	1	N/A	N/A		N/A	N/A	
9	SW-WP4	N/A	Segme...	1	N/A	N/A		N/A	N/A	
10	SW-WP6 LOWER	N/A	Segme...	1	N/A	N/A		N/A	N/A	
11	SW-WP6 UPPER	N/A	Segme...	1	N/A	N/A		N/A	N/A	
12	SW-WP5	N/A	Segme...	1	N/A	N/A		N/A	N/A	
13	SW-WP8 UPPER	N/A	Segme...	1	N/A	N/A		N/A	N/A	
14	SW-WP7 A	N/A	Segme...	1	N/A	N/A		N/A	N/A	
15	SW-WP12 A	N/A	Segme...	1	N/A	N/A		N/A	N/A	
16	SW-WP10	N/A	Segme...	1	N/A	N/A		N/A	N/A	
17	SW-WP9	N/A	Segme...	1	N/A	N/A		N/A	N/A	

Wood Wall Panel Parameters

	Label	Top Plate	Sill Plate	Studs	Min Stud ...	Max Stud ...	Green Lumber?	Header Size	Header Matl
1	Typical	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall
2	6"	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall
3	4"	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall
4	2"	2-2X6	2X6	2X6	16	16		4-2X6	Same as Wall

Additional Wood Wall Panel Parameters

	Label	Schedule	Min. P...	Max. P...	Double ...	Max. N...	Min. N...	HD Chor...	HD Chord M...	Hold Down	Ecc...
1	Typical	AWC 2015 PLY	.469	.469	No	6-in.	2-in.	2-2X6	Same as Wall	HDU_DF-SP	Yes
2	6"	S1_15/32_8d@6	.469	.469	No	6-in.	6-in.	2-2X6	Same as Wall	HDU_DF-SP	Yes
3	4"	S1_15/32_8d@4	.469	.469	No	4-in.	4-in.	2-2X6	Same as Wall	HDU_DF-SP	Yes
4	2"	S1_15/32_8d@2	.469	.469	No	2-in.	2-in.	2-2X6	Same as Wall	HDU_DF-SP	Yes

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Lateral Model Loading
Note: vertical loads generated and applied via RISAFloor

Basic Load Cases

	BLC Description	Category	X Grav..	Y Grav..	Z Grav...	Joint	Point	Distributed	Area(...	Surfac...
1	Dead Load	DL		-1		13	110	39		
2	Live Load	LL				5	52	10		
3	Live Load Special (public assemb	LLS								
4	Roof Live Load	RLL								
5	Snow Load	SL				10	70	24		
6	Snow Load Nonshedding	SLN								
7	Rain Load	RL								
8	Wind Load X	WLX								
9	Partial X Wind Load 1	WLXP1								
10	Partial X Wind Load 2	WLXP2								
11	Wind Load Z	WLZ								
12	Partial Z Wind Load 1	WLZP1								
13	Partial Z Wind Load 2	WLZP2								
14	Earthquake Load X	ELX						19		
15	Earthquake Load X Plus Z Eccentr	ELX+Z								
16	Earthquake Load X Minus Z Eccent	ELX-Z								
17	Earthquake Load Z	ELZ						15		
18	Earthquake Load Z Plus X Eccentr	ELZ+X								
19	Earthquake Load Z Minus X Eccent	ELZ-X								
20	Other Load 1	OL1								
21	Other Load 2	OL2								
22	Other Load 3	OL3								
23	Other Load 4	OL4								
24	Wind Load Roof +X	WLX+R								
25	Wind Load Roof -X	WLX-R								
26	Wind Load Roof +Z	WLZ+R								
27	Wind Load Roof -Z	WLZ-R								
28	BLC 14 Transient Area Loads	None								
29	BLC 17 Transient Area Loads	None								

Seismic Forces

Consideration for the different lateral resisting systems in each direction was taken into account by use of the response modification factor, R, for wood frame shear wall and steel ordinary concentrically braced frame. See Design Loading Report in Appendix A. ASCE7-10 cl. 12.2.3.3 and cl. 12.2.3.1 were considered in determining the appropriate response modification factor. Note that 30% of the roof or ground snow load (as appropriate) was considered in the seismic force calculations as per Section 1605.3.1 and 1605.3.2 of the Utah Amended Code.

Load Combinations

	Description	So...	PDelta	S...	BLC	Factor	BLC	Fa...	BLC	Fa...	BLC	Fa...	BLC	Fa...	B...	Fa...	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	Dead Load		Y		DL	1																		
2	Live Load		Y		LL	1																		
3	Snow Load		Y		SL	1																		
4	Earthquake Load X		Y		ELX	1																		
5	Earthquake Load Z		Y		ELZ	1																		
6			Y																					
7			Y																					
8	LFRD	Yes	Y																					
9	ASCE Strength 5 (..	Yes	Y		DL	1.2	Sds...	.2	Rho...	1	LL	.5	LLS	1	SL	.2	SLN	.2						
10	ASCE Strength 5 (..	Yes	Y		DL	1.2	Sds...	.2	Rho...	1	LL	.5	LLS	1	SL	.2	SLN	.2						
11	ASCE Strength 5 (..	Yes	Y		DL	1.2	Sds...	.2	Rho...	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
12	ASCE Strength 5 (..	Yes	Y		DL	1.2	Sds...	.2	Rho...	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
13	ASCE Strength 7 (..	Yes	Y		DL	.9	Sds...	-.2	Rho...	1														
14	ASCE Strength 7 (..	Yes	Y		DL	.9	Sds...	-.2	Rho...	1														
15	ASCE Strength 7 (..	Yes	Y		DL	.9	Sds...	-.2	Rho...	-1														
16	ASCE Strength 7 (..	Yes	Y		DL	.9	Sds...	-.2	Rho...	-1														
17	LFRD OVERSTR...	Yes	Y																					
18	ASCE Strength 5 (..	Yes	Y		DL	1.2	Sds...	.2	Om...	1	LL	.5	LLS	1	SL	.2	SLN	.2						
19	ASCE Strength 5 (..	Yes	Y		DL	1.2	Sds...	.2	Om...	1	LL	.5	LLS	1	SL	.2	SLN	.2						
20	ASCE Strength 5 (..	Yes	Y		DL	1.2	Sds...	.2	Om...	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
21	ASCE Strength 5 (..	Yes	Y		DL	1.2	Sds...	.2	Om...	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
22	ASCE Strength 7 (..	Yes	Y		DL	.9	Sds...	-.2	Om...	1														
23	ASCE Strength 7 (..	Yes	Y		DL	.9	Sds...	-.2	Om...	1														
24	ASCE Strength 7 (..	Yes	Y		DL	.9	Sds...	-.2	Om...	-1														
25	ASCE Strength 7 (..	Yes	Y		DL	.9	Sds...	-.2	Om...	-1														
26	LFRD DEFLECTI...		Y																					
27	DEF ASCE Streng...		Y		DL	1.2	Sds...	.2	ELX	1	LL	.5	LLS	1	SL	.2	SLN	.2						
28	DEF ASCE Streng...		Y		DL	1.2	Sds...	.2	ELZ	1	LL	.5	LLS	1	SL	.2	SLN	.2						
29	DEF ASCE Streng...		Y		DL	1.2	Sds...	.2	ELX	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
30	DEF ASCE Streng...		Y		DL	1.2	Sds...	.2	ELZ	-1	LL	.5	LLS	1	SL	.2	SLN	.2						
31	DEF ASCE Streng...		Y		DL	.9	Sds...	-.2	ELX	1														
32	DEF ASCE Streng...		Y		DL	.9	Sds...	-.2	ELZ	1														
33	DEF ASCE Streng...		Y		DL	.9	Sds...	-.2	ELX	-1														
34	DEF ASCE Streng...		Y		DL	.9	Sds...	-.2	ELZ	-1														
35			Y																					
36	ASD	Yes	Y																					
37	ASCE ASD 5 (b) (a)	Yes	Y		DL	1	Sds...	.14	Rho...	.7														
38	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Rho...	.7														
39	ASCE ASD 5 (b) (c)	Yes	Y		DL	1	Sds...	.14	Rho...	-.7														
40	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Rho...	-.7														
41	ASCE ASD 6 (b) (a)	Yes	Y		DL	1	Sds...	.105	Rho...	.525	LL	.75	LLS	.75										
42	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Rho...	.525	LL	.75	LLS	.75										
43	ASCE ASD 6 (b) (c)	Yes	Y		DL	1	Sds...	.105	Rho...	-.5...	LL	.75	LLS	.75										
44	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Rho...	-.5...	LL	.75	LLS	.75										
45	ASCE ASD 6 (d) (a)	Yes	Y		DL	1	Sds...	.105	Rho...	.525	LL	.75	LLS	.75	SL	.75	SLN	.75						
46	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Rho...	.525	LL	.75	LLS	.75	SL	.75	SLN	.75						
47	ASCE ASD 6 (d) (c)	Yes	Y		DL	1	Sds...	.105	Rho...	-.5...	LL	.75	LLS	.75	SL	.75	SLN	.75						
48	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Rho...	-.5...	LL	.75	LLS	.75	SL	.75	SLN	.75						
49	ASCE ASD 8 (a)	Yes	Y		DL	.6	Sds...	-.14	Rho...	.7														
50	ASCE ASD 8 (b)	Yes	Y		DL	.6	Sds...	-.14	Rho...	.7														
51	ASCE ASD 8 (c)	Yes	Y		DL	.6	Sds...	-.14	Rho...	-.7														

Load Combinations (Continued)

	Description	So...	PDelta	S...	BLC	Factor	BLC	Fa...	BLC	Fa...	BLC	Fa...	BLC	Fa...	B...	Fa...	BLC	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
52	ASCE ASD 8 (d)	Yes	Y		DL	.6	Sds...	-.14	Rho...	-.7														
53	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Om...	.7														
54	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Om...	.7														
55	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Om...	-.7														
56	ASCE ASD 5 (b) (...	Yes	Y		DL	1	Sds...	.14	Om...	-.7														
57	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Om...	.525	LL	.75	LLS	.75										
58	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Om...	.525	LL	.75	LLS	.75										
59	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Om...	-.5...	LL	.75	LLS	.75										
60	ASCE ASD 6 (b) (...	Yes	Y		DL	1	Sds...	.105	Om...	-.5...	LL	.75	LLS	.75										
61	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Om...	.525	LL	.75	LLS	.75	SL	.75	SLN	.75						
62	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Om...	.525	LL	.75	LLS	.75	SL	.75	SLN	.75						
63	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Om...	-.5...	LL	.75	LLS	.75	SL	.75	SLN	.75						
64	ASCE ASD 6 (d) (...	Yes	Y		DL	1	Sds...	.105	Om...	-.5...	LL	.75	LLS	.75	SL	.75	SLN	.75						
65	ASCE ASD 8 (os-a)	Yes	Y		DL	.6	Sds...	-.14	Om...	.7														
66	ASCE ASD 8 (os-b)	Yes	Y		DL	.6	Sds...	-.14	Om...	.7														
67	ASCE ASD 8 (os-c)	Yes	Y		DL	.6	Sds...	-.14	Om...	-.7														
68	ASCE ASD 8 (os-d)	Yes	Y		DL	.6	Sds...	-.14	Om...	-.7														

Frame / HR Column Seismic Design Rule

	Label	Frame Ductility	Overstrength Reqd
1	OCBF	Minimal	Yes
2	SCBF	High	Yes
3	OMF	Minimal	Yes
4	IMF	Moderate	Yes
5	SMF-RBS	High	Yes
6	SMF-Kaiser	High	Yes

HR Beam Seismic Design Rule

	Label	Moment Connection	Overstrength Reqd	Z Factor	Hinge Location[in]
1	OCBF	Other/None	Yes		
2	SCBF	Other/None	Yes		
3	OMF	BUEEP			12
4	IMF	BFP			12
5	SMF-RBS	RBS		.685	14.625
6	SMF-Kaiser	KBB-B			12

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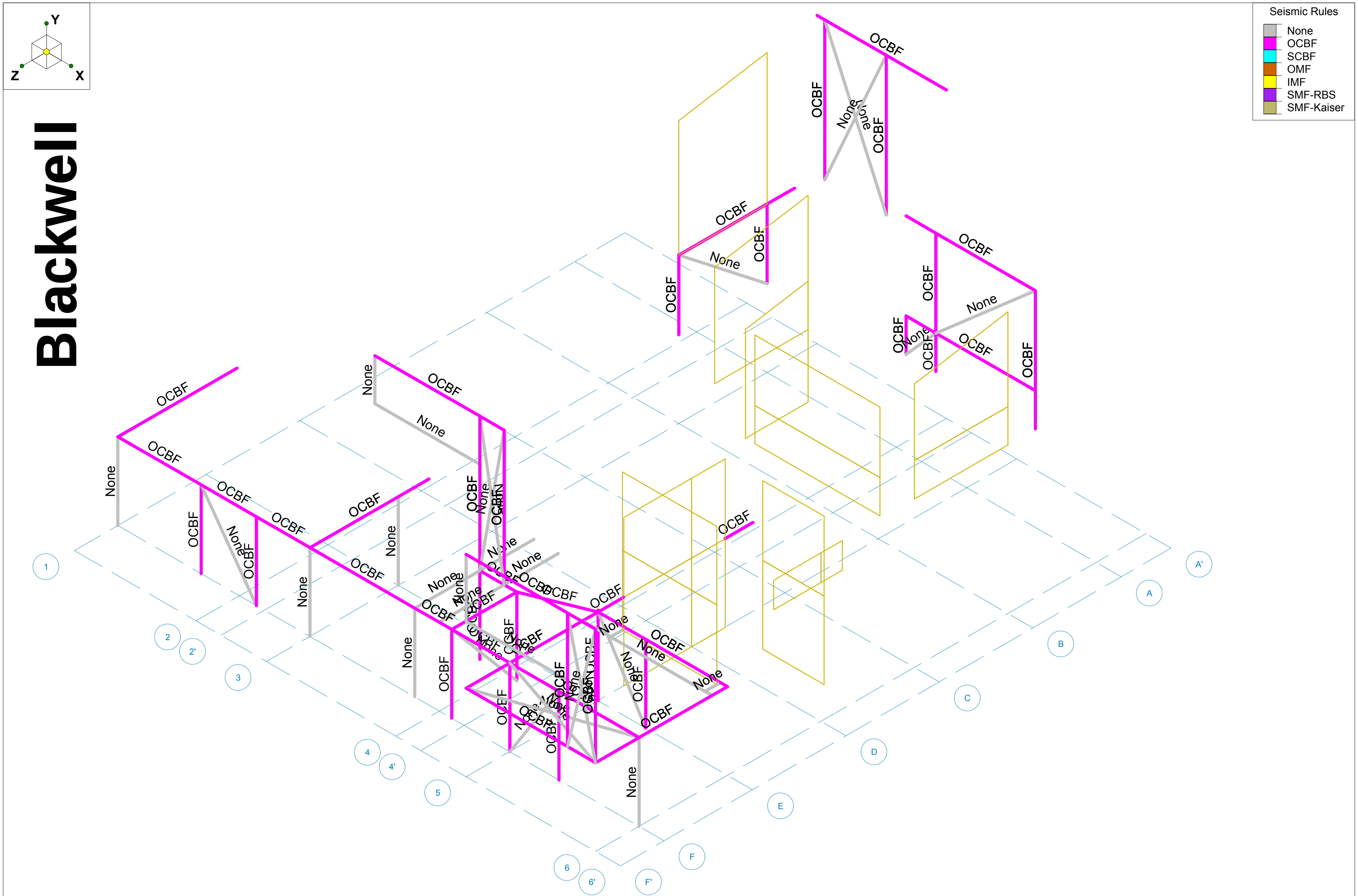

Member Seismic Design Rule Displayed

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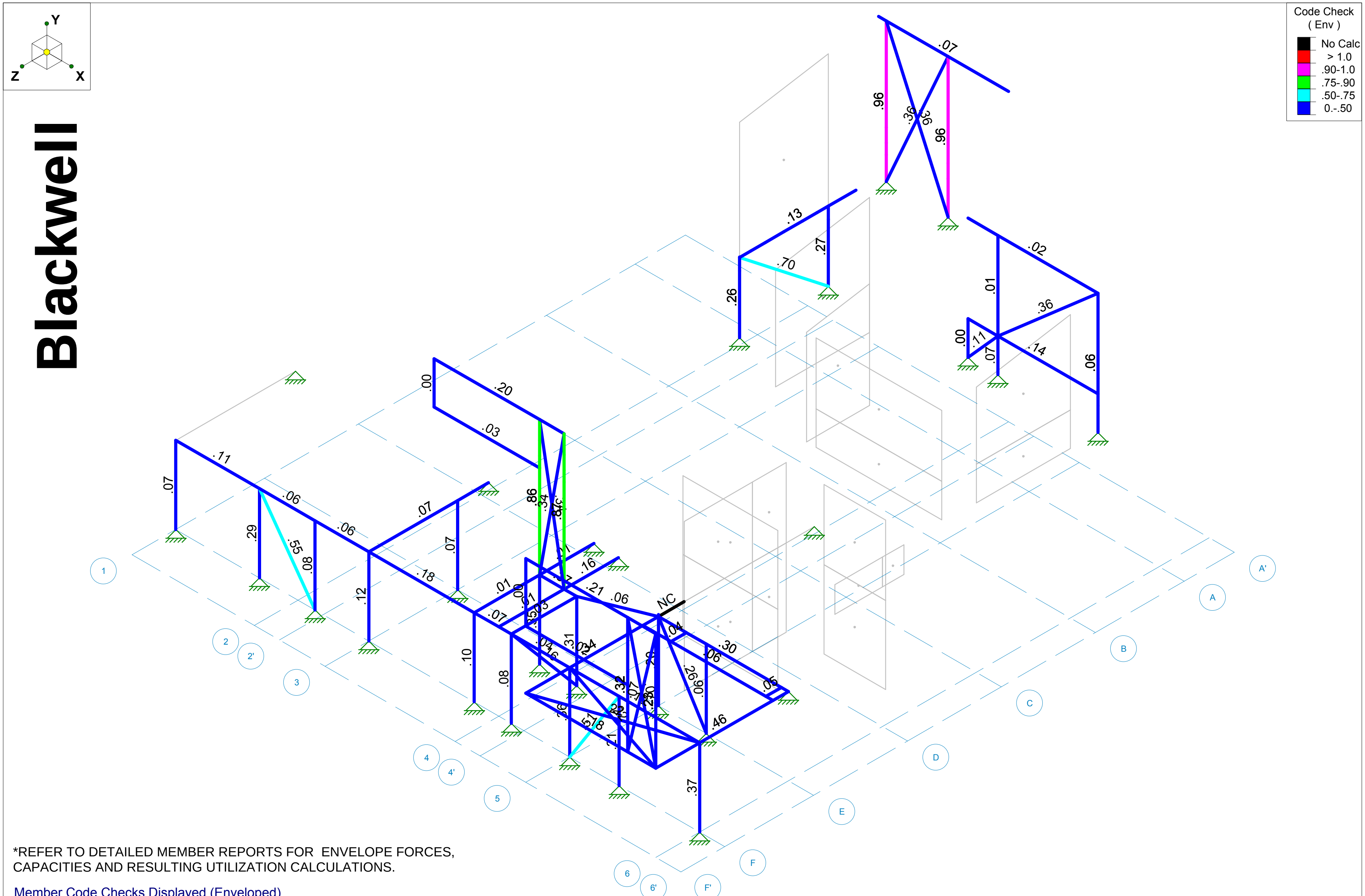
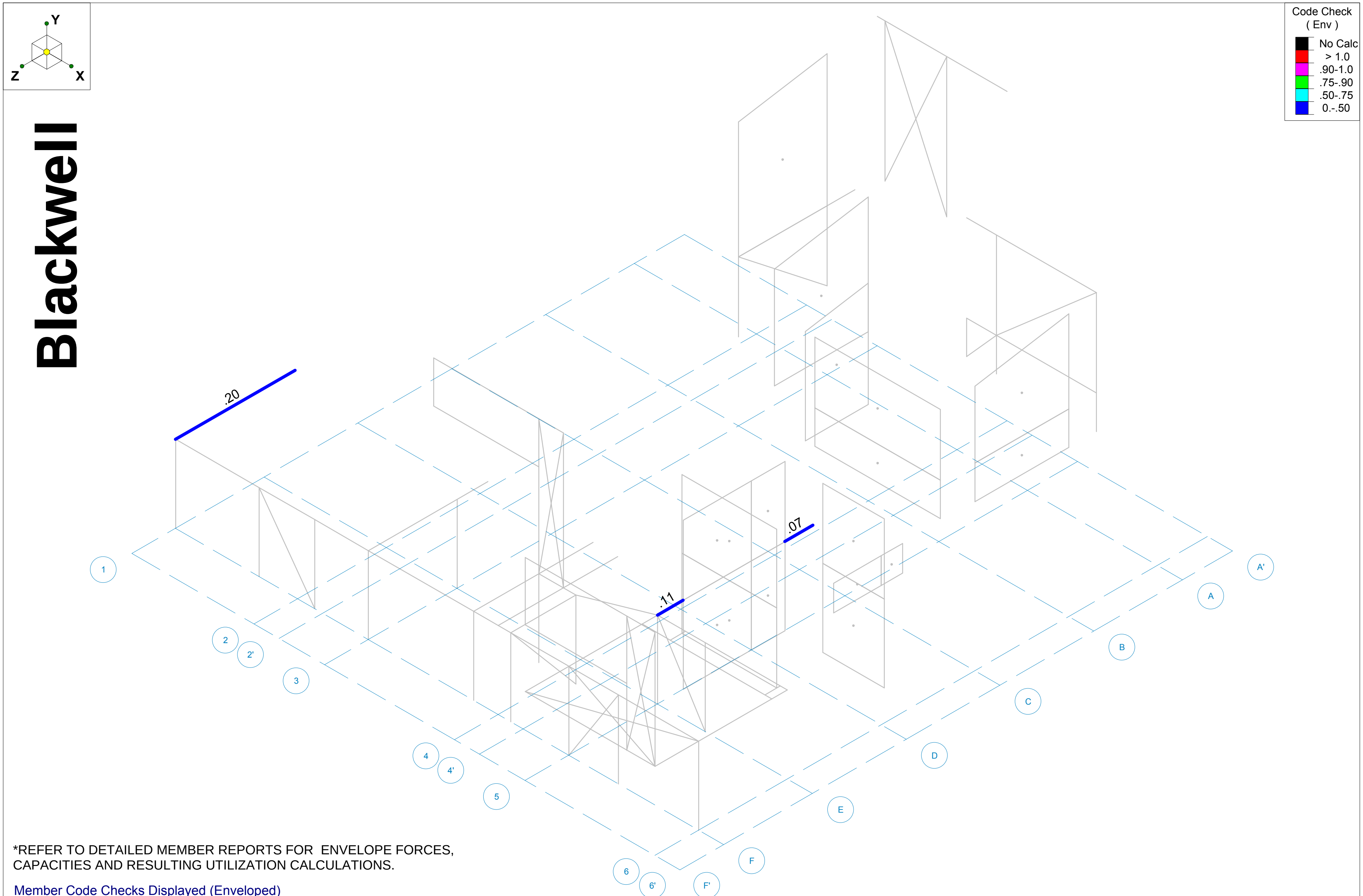
Lateral Steel and Wood Member Utilization



*REFER TO DETAILED MEMBER REPORTS FOR ENVELOPE FORCES, CAPACITIES AND RESULTING UTILIZATION CALCULATIONS.

Member Code Checks Displayed (Enveloped)

Blackwell Structural Engineers	Kimmelman May Residence Volume 2, 3 and 4	STEEL UNITY CHECK
BG		Feb 15, 2018 at 6:02 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.r3d



*REFER TO DETAILED MEMBER REPORTS FOR ENVELOPE FORCES, CAPACITIES AND RESULTING UTILIZATION CALCULATIONS.

Member Code Checks Displayed (Enveloped)

Blackwell Structural Engineers	Kimmelman May Residence Volume 2, 3 and 4	WOOD UNITY CHECK
BG		Feb 15, 2018 at 6:05 PM
170266		KMR Volume 2 3 4 Revised Permit Submission.rfl

Envelope AISC 14th(360-10): LRFD Steel Code Checks

	Member	Shape	Code Che...	Loc[ft]	LC	Shear C...	Loc[ft]	Dir	LC	phi*Pnc [..	phi*Pnt [..	phi*Mn y...	phi*Mn z-...	Cb	Eqn
1	BR-11	HSS3x3x4	.256	6.37	9	.001	11.76	y	12	34.26	101.016	8.556	8.556	1.1...	H1-1a
2	BR-10	HSS3x3x4	.507	6.436	11	.004	0	y	11	33.56	101.016	8.556	8.556	1.1...	H1-1a
3	3B2	W8x10	.027	6.823	9	.002	0	y	10	17.611	133.2	6.119	11.427	1.1...	H1-1b
4	2B8	W8x10	.024	6.593	9	.003	13.187	y	9	18.855	133.2	6.119	11.994	1.1...	H1-1b
5	X-BR 3A	L5x3.5x12	.344	17.707	11	.002	17.707	y	10	16.204	189.54	7.058	16.689	1.1...	H2-1*
6	X-BR 3B	L5x3.5x12	.343	17.707	9	.002	0	y	10	16.204	189.54	7.058	16.689	1.1...	H2-1*
7	X-BR 5A	L5x3.5x12	.069	15.498	11	.002	0	y	15	21.151	189.54	7.058	17.242	1.1...	H2-1*
8	X-BR 5B	L5x3.5x12	.069	15.498	9	.003	0	y	9	21.151	189.54	7.058	17.242	1.1...	H2-1*
9	BR-9	HSS3x3x4	.549	6.546	9	.002	0	y	12	31.209	101.016	8.556	8.556	1.1...	H1-1a
10	V2 HAN...	HSS4x4x4	.001	0	10	.000	0	z	10	123.394	139.518	16.181	16.181	1	H1-1b
11	V3 HAN...	HSS4x4x4	.001	0	9	.000	0	z	9	109.816	139.518	16.181	16.181	1	H1-1b
12	X-BR 6B	R0.75	.217	0	13	.004	17.761		12	.077	14.314	.179	.179	1.1...	H1-1...
13	X-BR 6A	R0.75	.218	0	11	.004	17.761		12	.077	14.314	.179	.179	1.1...	H1-1...
14	BR-14	HSS3x3x4	.160	0	12	.002	13.097	y	12	27.62	101.016	8.556	8.556	1.1...	H1-1...
15	RB11	W12x22	.197	13.677	18*	.030	13.677	y	19*	246.774	291.6	13.725	48.409	1.6...	H1-1b
16	RB1	W12x22	.214	3.682	18*	.027	3.507	y	18*	246.774	291.6	13.725	45.965	1.6...	H1-1b
17	1B1	W12x26	.113	5.419	18*	.064	10.838	y	21*	306.059	344.25	30.637	139.5	1	H1-1b
18	1B3	W12x26	.056	2.974	18*	.038	0	y	21*	312.914	344.25	30.637	139.5	1	H1-1b
19	1B2	W12x26	.063	3.974	18*	.051	0	y	12	312.582	344.25	30.637	139.5	1	H1-1b
20	1B4	W12x26	.184	6.823	18*	.082	0	y	19*	299.331	344.25	30.637	139.5	1	H1-1b
21	1B7	W12x26	.121	10.345	20*	.027	10.521	y	20*	95.786	344.25	30.637	113.534	1.5...	H1-1b
22	1B20	W14x30	.455	11.445	18*	.128	11.623	y	18*	339.501	398.25	33.713	151.462	1.7...	H1-1b
23	1B19	W14x30	.339	11.445	20*	.100	11.623	y	20*	339.501	398.25	33.713	155.637	1.7...	H1-1b
24	1B8	W14x68	.176	3.682	20*	.123	0	y	20*	828.613	900	138.375	431.25	1	H1-1b
25	1B26	W12x26	.011	0	10	.004	0	y	10	262.33	344.25	30.637	139.5	1	H1-1...
26	1B6	W12x26	.040	3.771	20*	.034	7.542	y	20*	312.075	344.25	30.637	139.5	1	H1-1b
27	1B11	W14x61	.371	3.159	18*	.445	4.813	y	18*	800.024	805.5	123	382.5	1	H1-1b
28	1B10	W10x22	.303	15.781	22*	.099	16.833	y	18*	118.068	292.05	22.875	97.5	1	H1-1a
29	1B17	W12x26	.161	3.961	12	.090	7.042	y	12	262.33	344.25	30.637	139.5	1	H1-1b
30	1B16	W12x26	.010	4.229	12	.007	0	y	9	240.089	344.25	30.637	139.5	1	H1-1b
31	1B18	W12x26	.026	0	25*	.008	0	y	20*	240.089	344.25	30.637	139.5	1	H1-1...
32	1B5	W12x26	.073	1.654	19*	.016	4.813	y	19*	315.459	344.25	30.637	139.5	1	H1-1b
33	1B24	W12x40	.058	4.066	18*	.037	8.132	y	18*	510.7	526.5	63	213.75	1	H1-1b
34	(5-E)	HSS4x4x4	.195	0	20*	.000	0	y	22*	91.807	139.518	16.181	16.181	1.6...	H1-1...
35	(1-F)	HSS4x4x4	.066	0	11	.000	0	z	12	91.807	139.518	16.181	16.181	1	H1-1...
36	(2-F)	HSS4x4x4	.083	0	10	.000	0	z	12	91.807	139.518	16.181	16.181	1	H1-1...
37	(3-F)	HSS4x4x4	.119	0	11	.000	0	z	10	91.807	139.518	16.181	16.181	1	H1-1...
38	(5+-E)	HSS4x4x4	.064	0	20*	.000	0	y	9	91.807	139.518	16.181	16.181	1	H1-1...
39	(2--F)	HSS4x4x4	.293	0	20*	.000	0	z	10	91.807	139.518	16.181	16.181	1	H1-1a
40	(4--F)	HSS4x4x4	.096	0	11	.000	0	y	13	91.807	139.518	16.181	16.181	1.6...	H1-1...
41	(5+-F)	HSS4x4x4	.208	0	18*	.000	0	y	9	91.807	139.518	16.181	16.181	1	H1-1a
42	CS1	HSS5x5x8	.306	0	18*	.000	0	z	18*	243.236	326.232	45.195	45.195	1.6...	H1-1a
43	(4+-F)	HSS4x4x4	.084	0	19*	.000	0	z	20*	91.807	139.518	16.181	16.181	1.6...	H1-1...
44	CS2	HSS5x5x8	.351	10	20*	.005	0	y	22*	243.236	326.232	45.195	45.195	1.6...	H1-1a
45	CS3	HSS5x5x8	.862	7.62	20*	.009	15.24	y	20*	133.89	326.232	45.195	45.195	1.0...	H1-1a
46	(6-F')	HSS4x4x8	.227	0	22*	.000	0	z	18*	82.281	249.228	26.565	26.565	1.6...	H1-1a
47	CS4	HSS5x5x6	.871	0	22*	.000	15.24	y	18*	110.801	255.852	36.57	36.57	1.6...	H1-1a
48	CS44	HSS4x4x8	.317	0	20*	.015	12.082	y	18*	82.281	249.228	26.565	26.565	1.1...	H1-1a
49	1B15	W12x26	.009	4.229	10	.009	8.458	y	9	240.089	344.25	30.637	139.5	1	H1-1b
50	1B14	W10x22	.068	0	21*	.019	4.036	y	10	216.953	292.05	22.875	97.5	1	H1-1...
51	(3+-A')	HSS4x4x5	.956	0	20*	.000	0	y	20*	44.22	169.74	19.285	19.285	1.6...	H1-1a

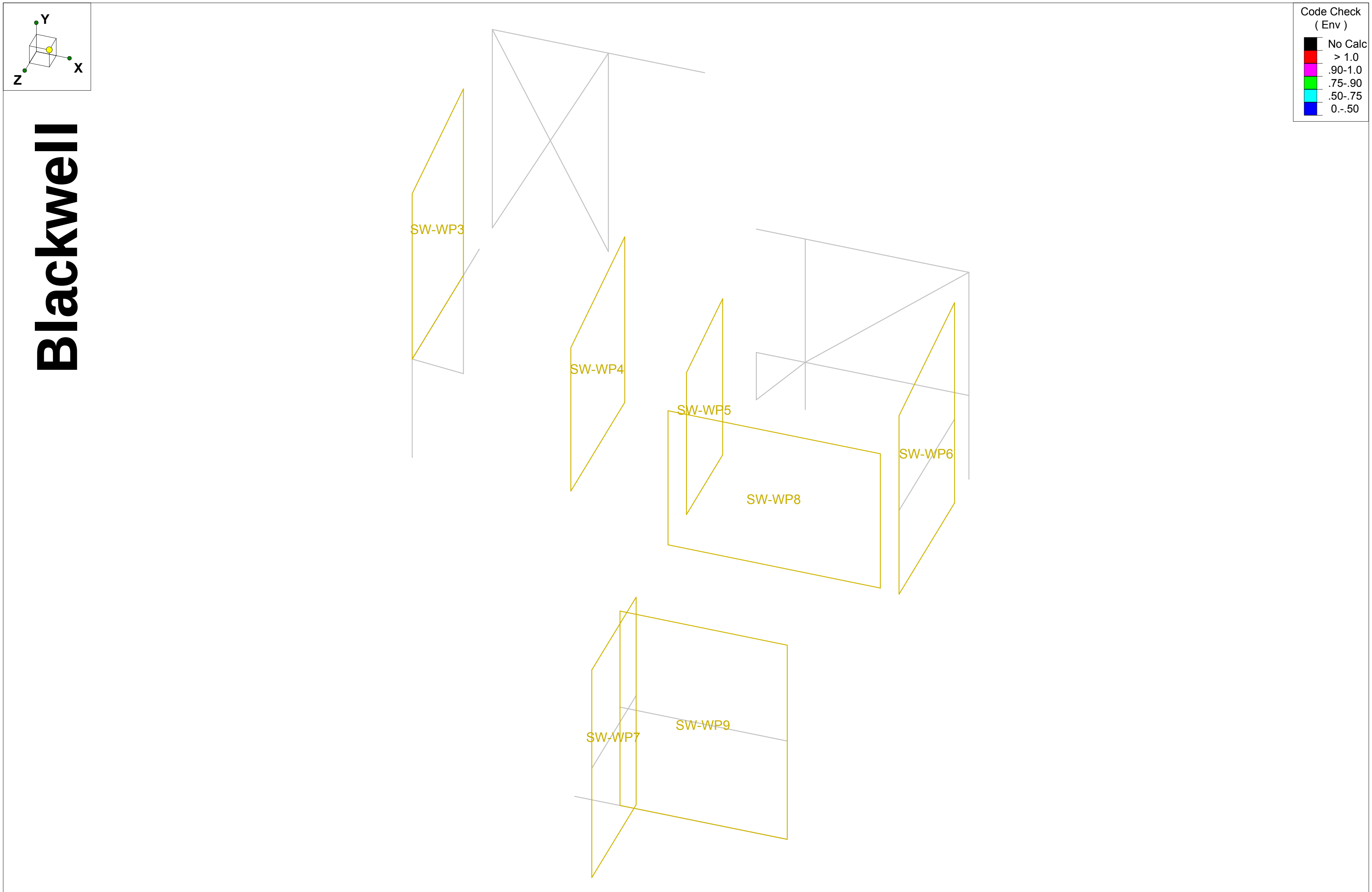
Envelope AISC 14th(360-10): LRFD Steel Code Checks (Continued)

	Member	Shape	Code Che...	Loc[ft]	LC	Shear C...	Loc[ft]	Dir	LC	phi*Pnc [...	phi*Pnt [...	phi*Mn y...	phi*Mn z-...	Cb	Eqn
52	(3++-A')	HSS4x4x5	.957	0	18*	.000	0	y	18*	44.22	169.74	19.285	19.285	1.7...	H1-1a
53	RB18	W12x22	.069	1.052	20*	.008	.877	y	10	246.774	291.6	13.725	44.32	1.5...	H1-1...
54	RB6	W12x22	.024	0	18*	.005	12.8	y	11	246.774	291.6	13.725	109.875	2.0...	H1-1...
55	3B1	W8x18	.137	4.033	20*	.043	4.033	y	20*	74.466	236.7	17.475	37.789	1.2...	H1-1b
56	2B23	W10x22	.134	13.669	19*	.073	15.083	y	19*	132.186	292.05	22.875	97.5	1.7...	H1-1b
57	(3-B)	HSS5x5x5	.260	7.447	19*	.205	9.05	z	19*	148.006	217.764	31.602	31.602	1	H1-1a
58	CS5	HSS4x4x4	.003	4.333	20*	.000	4.288	y	9	138.935	139.518	16.181	16.181	1.6...	H1-1b
59	BR-12	HSS3x3x4	.105	0	11	.001	5.813	y	11	77.543	101.016	8.556	8.556	1.1...	H1-1...
60	BR-6	HSS3x3x4	.356	8.051	11	.003	17.174	y	11	16.063	101.016	8.556	8.556	1.1...	H1-1a
61	BR-8	HSS3x3x4	.695	6.997	12	.003	0	y	12	22.218	101.016	8.556	8.556	1.1...	H1-1a
62	X-BR 4A	L3x2.5x8	.361	0	11	.002	0	y	11	2.69	81	2.276	3.947	1.1...	H2-1*
63	X-BR 4B	L3x2.5x8	.361	19.705	9	.002	19.705	y	9	2.69	81	2.276	3.947	1.1...	H2-1*
64	CS7	HSS4x4x4	.011	11.272	10	.000	9.628	y	11	85.082	151.65	17.588	17.588	1	H1-1...
65	CS6	HSS4x4x4	.068	0	18*	.000	0	y	9	139.236	151.65	17.588	17.588	1	H1-1...
66	CS8	HSS4x4x4	.061	4.389	22*	.000	0	y	18*	81.98	139.518	16.181	16.181	1.6...	H1-1...
67	(3-E)	HSS5x5x5	.067	0	12	.000	0	y	9	166.542	217.764	31.602	31.602	1	H1-1...
68	(F-5)	HSS4x4x4	.356	0	20*	.001	0	z	20*	91.807	139.518	16.181	16.181	1	H1-1a
69	(F-6)	HSS4x4x4	.367	0	9	.000	0	y	9	91.807	139.518	16.181	16.181	1	H1-1a
70	1B9	W12x26	.059	5.962	11	.030	16.833	y	9	170.358	344.25	30.637	139.5	1	H1-1b
71	1B28	W12x26	.037	.438	11	.066	0	y	11	313.048	344.25	30.637	139.5	1.5...	H1-1b
72	1B29	W12x26	.046	.438	9	.058	0	y	9	313.048	344.25	30.637	139.5	1.5...	H1-1b
73	(3-A)	HSS4x4x4	.271	0	21*	.000	7.5	z	10	99.405	139.518	16.181	16.181	1	H1-1a

Envelope Wood Code Checks

	Mem...	Shape	Code Check	Loc[ft]	LC	Shear Ch...	Loc[ft]	Dir	LC	Fc' [...	Ft' [...	Fb1'...	Fb2'...	Fv' [...	RB	CL	CP	Eqn
1	1B21	2-1.75X...	No Val															
2	1B23	2-1.75X...	No Val															
3	1B27	3-1.75X...	No Val															

Shear Wall Utilization



Blackwell Structural Engineers	KMR V2 V3 V4 Shear Walls	SHEAR WALL DESIGNATION
BG		Nov 1, 2017 at 6:51 PM
170266	LATERAL STEEL SYSTEM AT NORTH END OF RESIDENCE AND SHEAR WALLS	KMR V2 V3 V4 Shear Wall Rev 2 POST COUN...

Wood Wall Panel Axial Code Checks (AWC NDS-15: ASD)

	Wall Panel	Region	Stud Size	Stud Spaci...	Axial Check	Gov LC	Chord Size	Chord Axial Ch...	Gov LC
1	SW-WP7 B	R1	2X6	16	0	N/A	2-2X6	.032	38
2	SW-WP11 A	R1	2X6	16	0	N/A	2-2X6	.093	46
3	SW-WP11 B	R1	2X6	16	0	N/A	2-2X6	.048	44
4	SW-WP8 L...	R1	2X6	16	0	N/A	2-2X6	.139	51
5	SW-WP13	R1	2X6	16	0	N/A	2-2X6	.028	39
6	SW-WP14	R1	2X6	16	0	N/A	2-2X6	.27	37
7	SW-WP12 B	R1	2X6	16	0	N/A	2-2X6	.01	52
8	SW-WP3	R1	2X6	16	0	N/A	2-2X6	.939	40
9	SW-WP4	R1	2X6	16	0	N/A	2-2X6	.66	40
10	SW-WP6 L...	R1	2X6	16	0	N/A	2-2X6	.337	52
11	SW-WP6 ...	R1	2X6	16	0	N/A	2-2X6	.195	52
12	SW-WP5	R1	2X6	16	0	N/A	2-2X6	.819	40
13	SW-WP8 ...	R1	2X6	16	0	N/A	2-2X6	.079	51
14	SW-WP7 A	R1	2X6	16	0	N/A	2-2X6	.025	52
15	SW-WP12 A	R1	2X6	16	0	N/A	2-2X6	.011	50
16	SW-WP10	R1	2X6	16	0	N/A	2-2X6	.258	51
17	SW-WP9	R1	2X6	16	0	N/A	2-2X6	.112	51

See note below regarding tie down forces

Wood Wall Panel In Plane Code Checks (AWC NDS-15: ASD)

	Wall Panel	Shear Panel Label	Region	Shear Check	Shear Forc...	Gov LC	Hold-Down ...	Tension Ch...	Tie-Dow...	Gov LC
1	SW-WP7 B	S1_15/32_8d@6	R1	.414	.112	39	NC	NC	.496	38
2	SW-WP11 A	S1_15/32_8d@6	R1	.27	.076	37	Not Req'd	NC	NC	NC
3	SW-WP11 B	S1_15/32_8d@6	R1	.33	.08	37	HDU2-SDS...	.077	.237	50
4	SW-WP8 L...	S1_15/32_8d@6	R1	.579	.162	37	HDU2-SDS...	.618	1.9	51
5	SW-WP13	S1_15/32_8d@6	R1	.163	.046	37	NC	NC	.367	51
6	SW-WP14	S1_15/32_8d@4	R1	.62	.267	37	HDU4-SDS...	.691	3.153	49
7	SW-WP12 B	S1_15/32_8d@6	R1	.164	.046	40	HDU2-SDS...	.046	.141	52
8	SW-WP3	S1_15/32_8d@4	R1	.837	.36	38	NC	NC	5.796	52
9	SW-WP4	S1_15/32_8d@4	R1	.772	.332	38	HDU5-SDS...	.845	4.771	52
10	SW-WP6 L...	S1_15/32_8d@4	R1	.835	.359	38	HDU5-SDS...	.823	4.647	52
11	SW-WP6 ...	S1_15/32_8d@4	R1	.729	.314	38	NC	NC	3.01	52
12	SW-WP5	S1_15/32_8d@2	R1	.657	.48	38	HDU8-SDS...	.967	6.543	50
13	SW-WP8 ...	S1_15/32_8d@6	R1	.579	.162	37	NC	NC	NC	NC
14	SW-WP7 A	S1_15/32_8d@6	R1	.203	.057	40	NC	NC	.378	52
15	SW-WP12 A	S1_15/32_8d@6	R1	.162	.045	40	HDU2-SDS...	.053	.162	50
16	SW-WP10	S1_15/32_8d@6	R1	.91	.255	37	NC	NC	3.985	51
17	SW-WP9	S1_15/32_8d@6	R1	.715	.2	37	NC	NC	1.726	51

Tie Down Anchorage Note:

The tie down forces in SW-WP3, SW-WP4 and SW-WP6 as well as SW-WP2 in volume 1 will be resolved through connection to the adjacent columns using Simpson Strong-Drive TB Wood-to-Steel Screws. See data sheet on next page.

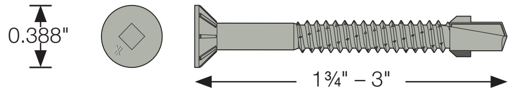
Fastener Calculation

Max Tie Down Force / Height of Adjacent Column = 5.798 kips / 14 ft = 414 lbs per foot
Fastener Resistance to Seismic Force = 345 lbs
Therefore, minimum spacing of fastener = 345/414 = 0.833 ft. Use 10" c/c.

Strong-Drive®
TB WOOD-TO-STEEL Screw

Common Applications:
- Wood to hot-rolled steel (Maximum recommended thicknesses: 5⁄16")

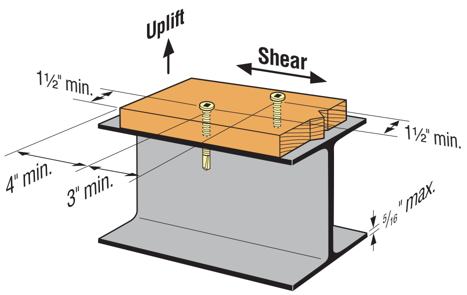
For More Product Information, see p. 100

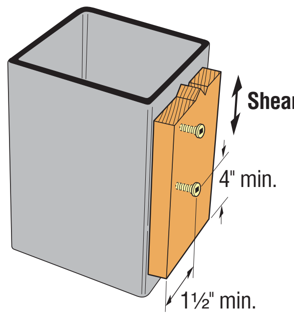


TB – Allowable Loads – DF and SP Lumber Attachment to Steel (Steel Members 16 ga. - 5⁄16" Thick)

Model No.	Length in. (mm)	Nominal Wood Thickness (in.)	Steel Thickness mil (ga.)	DF/SP Allowable Load (lb.)			
				Uplift		Shear	
				C_D=1.0	C_D=1.6	C_D=1.0	C_D=1.6
TB1460S	2⅜ (60)	2x	54 (16)	195	195	210	335
			68 (14)	225	225	210	335
			97-312 (12 – 5⁄16")	245	390	215	345
TB1475S	3 (75)		54 (16)	195	195	210	335
			68 (14)	225	225	210	335
			97-312 (12 – 5⁄16")	245	390	215	345

1. For use with structural steel members up to 5⁄16" thick or cold-formed steel members 54 mil (16 ga.) or thicker.
2. Standard product available in a black phosphate, yellow zinc or N2000 coating for additional corrosion protection *(TBG1460S or TBG1475S).*
3. For use with 2x (1½") DF/SP only.
4. For use with QD HSD60 or HSD75 Tool.
5. Use increased allowable loads (C_D=1.6) only when resisting wind or seismic forces.





Blackwell
Structural Engineers

Lateral Member Detailed Reports are Included in Appendix D

Blackwell
Structural Engineers

Lateral Shear Wall Detail Reports

CRITERIA

Code : **AWC NDS-15:ASD**

Wall Material : **Spruce-Pine-Fir**
Panel Schedule : **User Selected**

Optimize HD : **Yes**
HD Manufacturer: **SIMPSON**

MATERIALS

Wall Studs : **Spruce-Pine-Fir**
Stud Size : **2X6**

Chord Material : **Spruce-Pine-Fir**
Chord Size : **2-2X6**

Top Pl & Sill : **Spruce-Pine-Fir**
Top Pl Size : **2-2X6**
Sill Pl Size : **2X6**

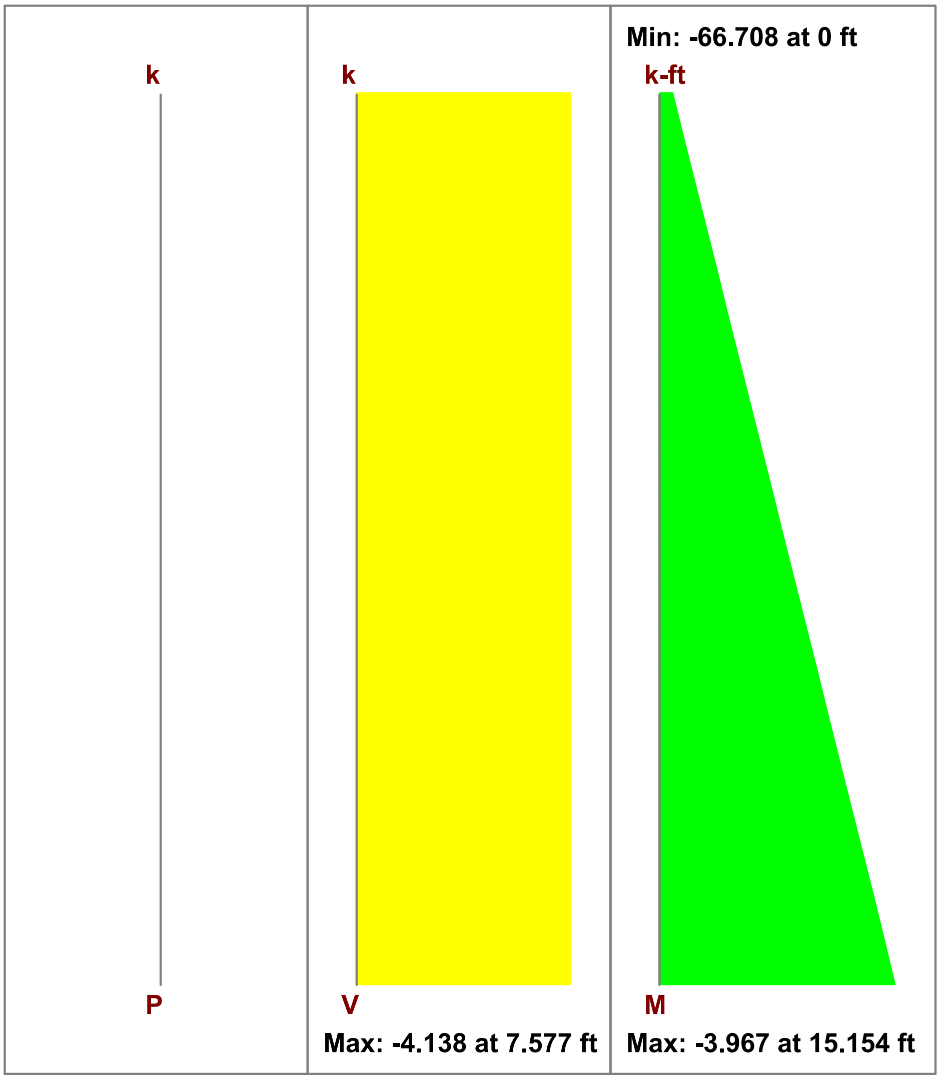
GEOMETRY

Total Height : **15.154 ft**
Total Length : **11.5 ft**

Region H/W : **1.32**
Cap. Adj. (2w/h) : **1.00**

Stud Spacing : **16 in**
K : **1.00**

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap : **.36 k/ft**
Provided Cap : **.43 k/ft**
Ratio : **.837**
Governing LC : **38 (Seismic)**

CHORDS

Max Comp Force: **5.985 k**
Comp Capacity : **6.371 k**
Comp Ratio : **.939**
Gov Comp LC : **40**
Max Tens Force : **5.796 k**
Tens Capacity : **15.444 k**
Tens Ratio : **.375**
Gov Tens LC : **52**

STUDS

No gravity-only LC solved.

DEFLECTIONS

Flexure Comp : **.452 in**
Shear Comp : **.389 in**
HD Elong : **0 in**
Tot Deflection : **.842 in**
Governing LC : **38**

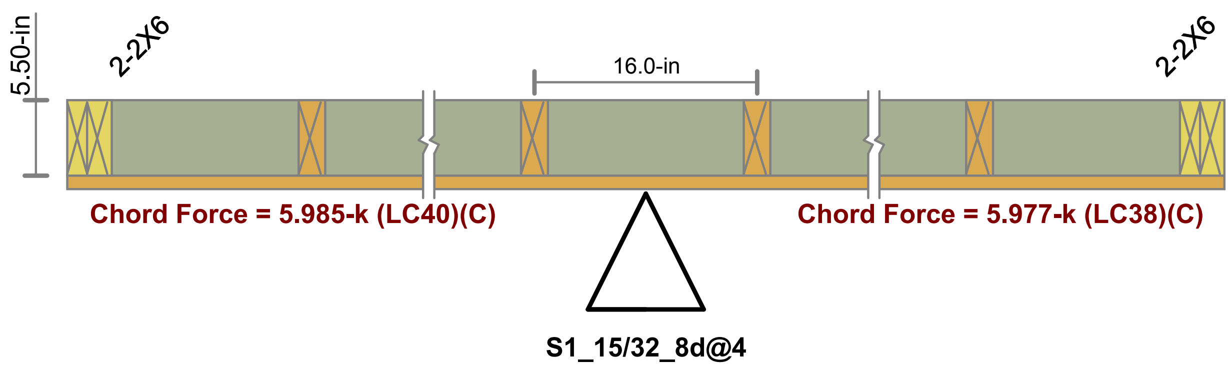
DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@4

Panel Grade	: **St-I**		Nail Size	: **8d**		Num Sides	: **One**	
Panel Thick	: **0.469**	**in**	Reqd Pen	: **1.250**	**in**	Over Gyp Brd.	: **No**	
			Reqd. Spacing	: **4**	**in**	Shear Capacity	: **0.430**	**k/ft**
						Adjusted Cap	: **0.430**	**k/ft**

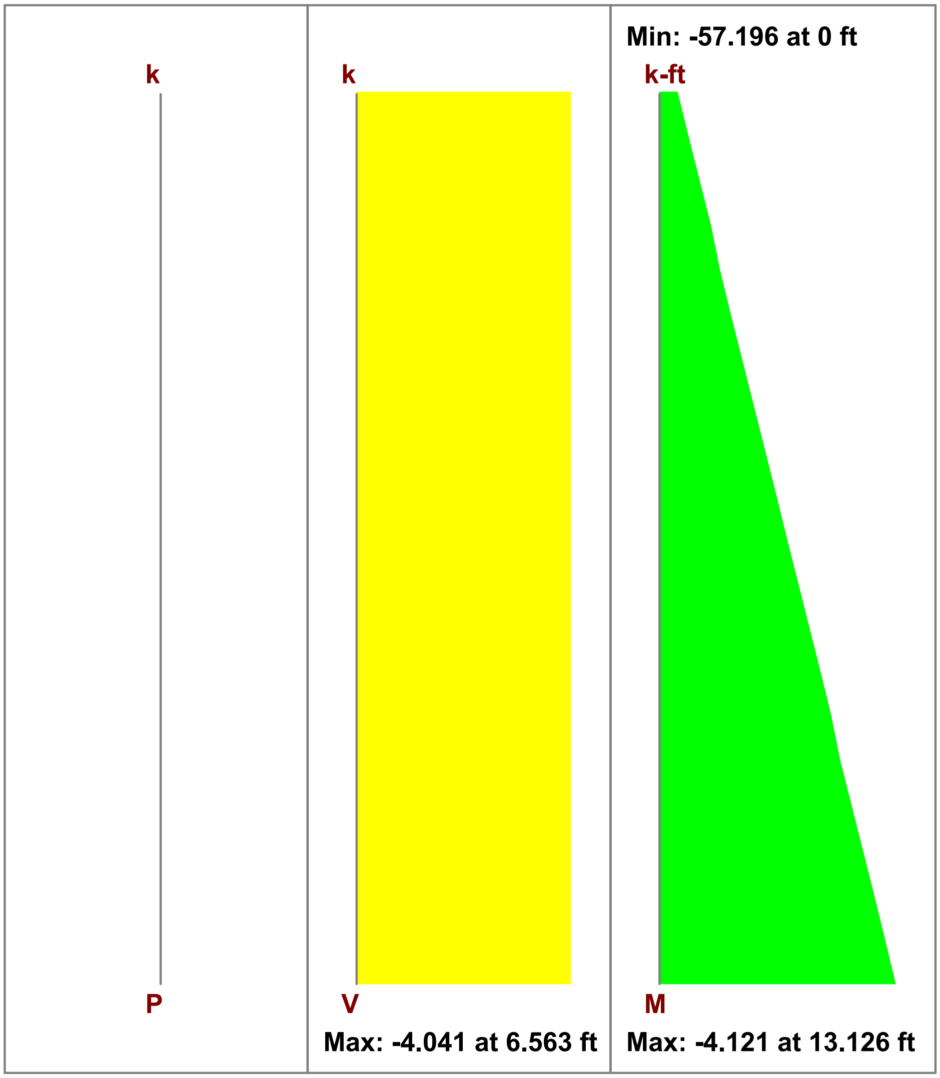
NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

CROSS SECTION DETAILING



CRITERIA		MATERIALS		GEOMETRY	
Code	: **AWC NDS-15:ASD**	Wall Studs	: **Spruce-Pine-Fir**	Total Height	: **13.126 ft**
		Stud Size	: **2X6**	Total Length	: **12.167 ft**
Wall Material	: **Spruce-Pine-Fir**	Chord Material	: **Spruce-Pine-Fir**	Region H/W	: **1.08**
Panel Schedule	: **User Selected**	Chord Size	: **2-2X6**	Cap. Adj. (2w/h)	: **1.00**
Optimize HD	: **Yes**	Top Pl & Sill	: **Spruce-Pine-Fir**	Stud Spacing	: **16 in**
HD Manufacturer	: **SIMPSON**	Top Pl Size	: **2-2X6**	K	: **1.00**
		Sill Pl Size	: **2X6**	HD Eccentricity	: **4.313 in**

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: **.332**	**k/ft**
Provided Cap	: **.43**	**k/ft**
Ratio	: **.772**	
Governing LC	: **38 (Seismic)**	

CHORDS

Max Comp Force	: **4.85 k**
Comp Capacity	: **7.348 k**
Comp Ratio	: **.66**
Gov Comp LC	: **40**
Max Tens Force	: **4.677 k**
Tens Capacity	: **13.514 k**
Tens Ratio	: **.346**
Gov Tens LC	: **52**

STUDS

No gravity-only LC solved.

HOLD-DOWNS

Required Cap	: **4.771**	**k**
Provided Cap	: **5.645**	**k**
Ratio	: **.845**	
Governing LC	: **52**	

DEFLECTIONS

Flexure Comp	: **.257**	**in**
Shear Comp	: **.311**	**in**
HD Elong	: **.102**	**in**
Tot Deflection	: **.67**	**in**
Governing LC	: **38**	

DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@4

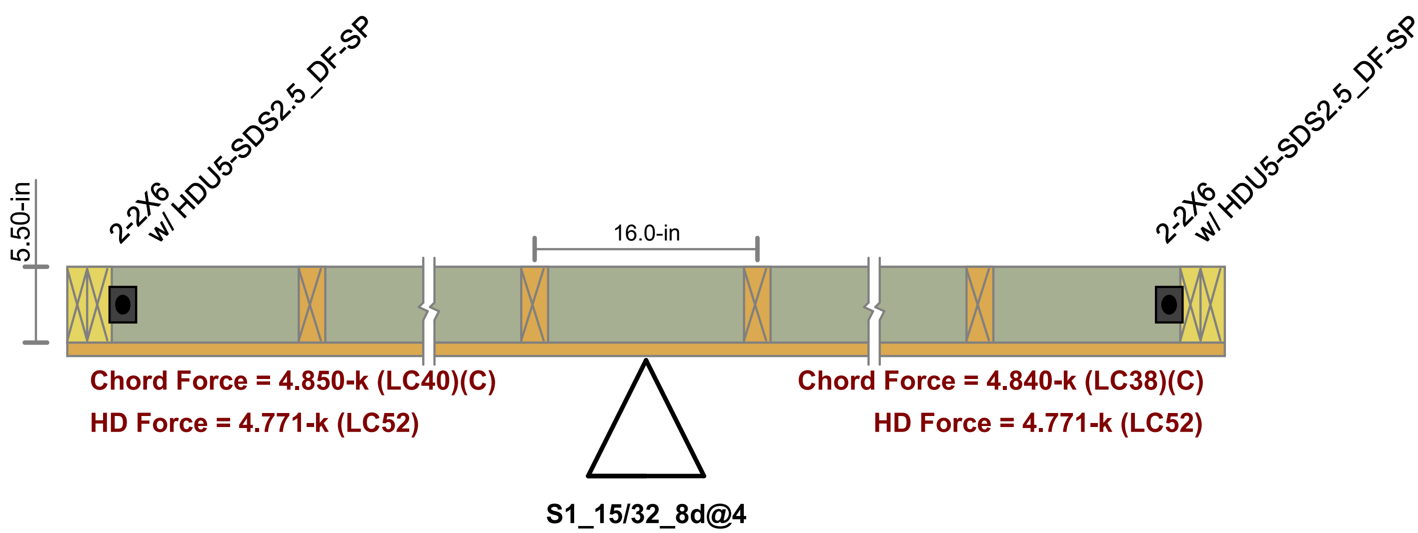
Panel Grade	: St-I		Nail Size	: 8d		Num Sides	: One	
Panel Thick	: 0.469	in	Reqd Pen	: 1.250	in	Over Gyp Brd.	: No	
			Reqd. Spacing	: 4	in	Shear Capacity	: 0.430	k/ft
						Adjusted Cap	: 0.430	k/ft

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

SELECTED HOLD-DOWN : HDU5-SDS2.5_DF-SP

Min Chord Thk	: 3.00	in	Bolt Size:	: .625	in	Base Cap(CD=1):	3.528	k
Reqd Chord Mat	: Douglas Fir					CD factor	: 1.6	
						Adjusted Cap	: 5.645	k

CROSS SECTION DETAILING



CRITERIA

Code	: **AWC NDS-15:ASD**
Wall Material	: **Spruce-Pine-Fir**
Panel Schedule	: **User Selected**
Optimize HD	: **Yes**
HD Manufacturer	: **SIMPSON**

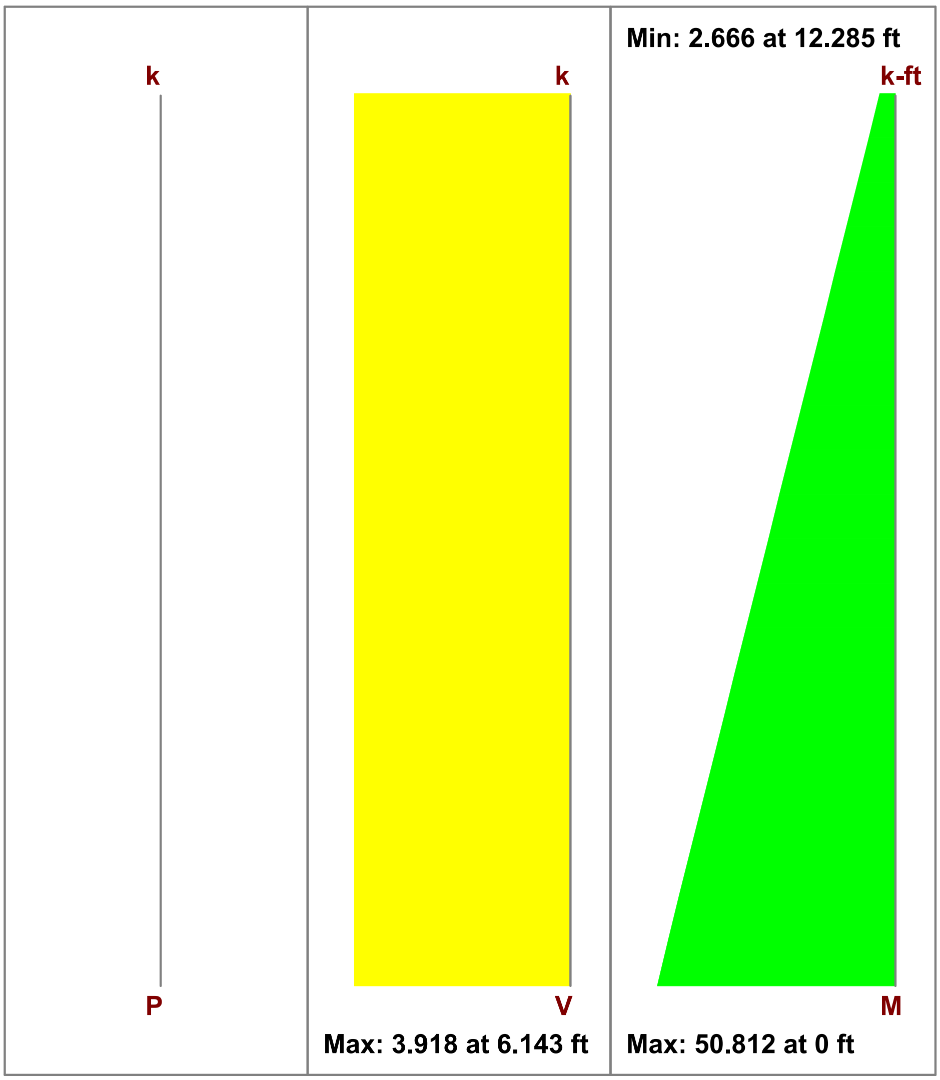
MATERIALS

Wall Studs	: **Spruce-Pine-Fir**
Stud Size	: **2X6**
Chord Material	: **Spruce-Pine-Fir**
Chord Size	: **2-2X6**
Top Pl & Sill	: **Spruce-Pine-Fir**
Top Pl Size	: **2-2X6**
Sill Pl Size	: **2X6**

GEOMETRY

Total Height	: **12.285 ft**
Total Length	: **8.167 ft**
Region H/W	: **1.50**
Cap. Adj. (2w/h)	: **1.00**
Stud Spacing	: **16 in**
K	: **1.00**
HD Eccentricity	: **4.375 in**

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: **.48 k/ft**
Provided Cap	: **.73 k/ft**
Ratio	: **.657**
Governing LC	: **38 (Seismic)**

CHORDS

Max Comp Force	: **6.46 k**
Comp Capacity	: **7.892 k**
Comp Ratio	: **.819**
Gov Comp LC	: **40**
Max Tens Force	: **6.347 k**
Tens Capacity	: **12.812 k**
Tens Ratio	: **.495**
Gov Tens LC	: **52**

STUDS

No gravity-only LC solved.

HOLD-DOWNS

Required Cap	: **6.543 k**
Provided Cap	: **6.765 k**
Ratio	: **.967**
Governing LC	: **50**

DEFLECTIONS

Flexure Comp	: **.453 in**
Shear Comp	: **.256 in**
HD Elong	: **.158 in**
Tot Deflection	: **.867 in**
Governing LC	: **38**

DESIGN DETAILS

SELECTED SHEAR PANEL : **S1_15/32_8d@2**

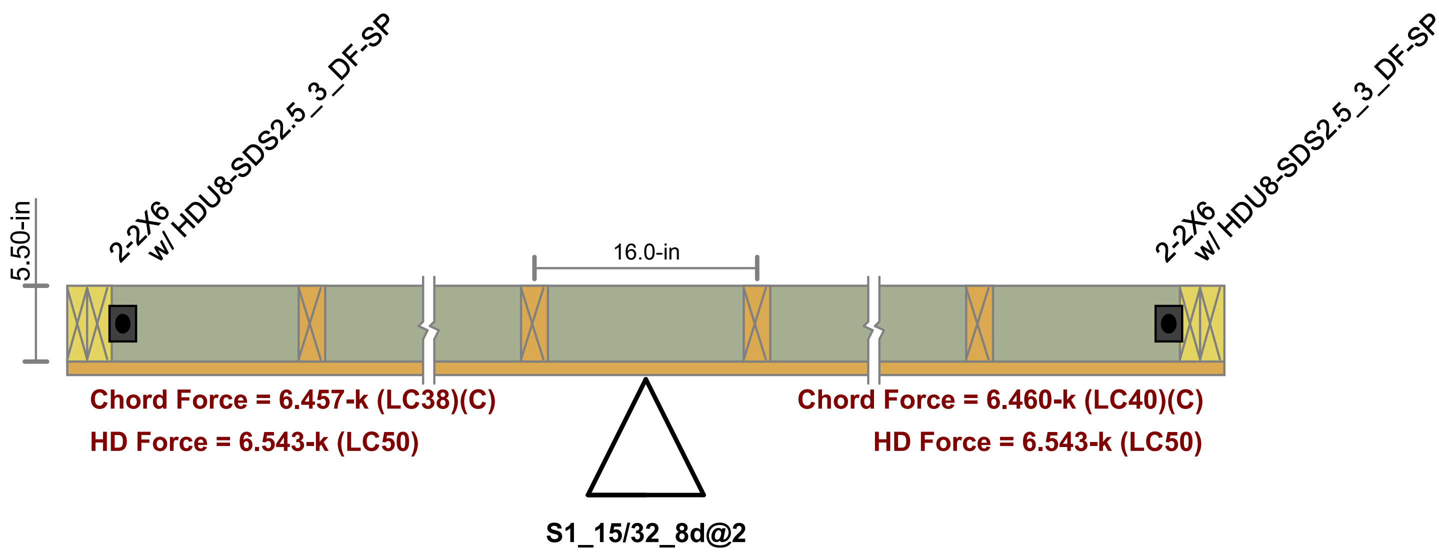
Panel Grade	:	**St-I**		Nail Size	:	**8d**		Num Sides	:	**One**	
Panel Thick	:	**0.469**	**in**	Reqd Pen	:	**1.250**	**in**	Over Gyp Brd.	:	**No**	
				Reqd. Spacing	:	**2**	**in**	Shear Capacity	:	**0.730**	**k/ft**
								Adjusted Cap	:	**0.730**	**k/ft**

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

SELECTED HOLD-DOWN : **HDU8-SDS2.5_3_DF-SP**

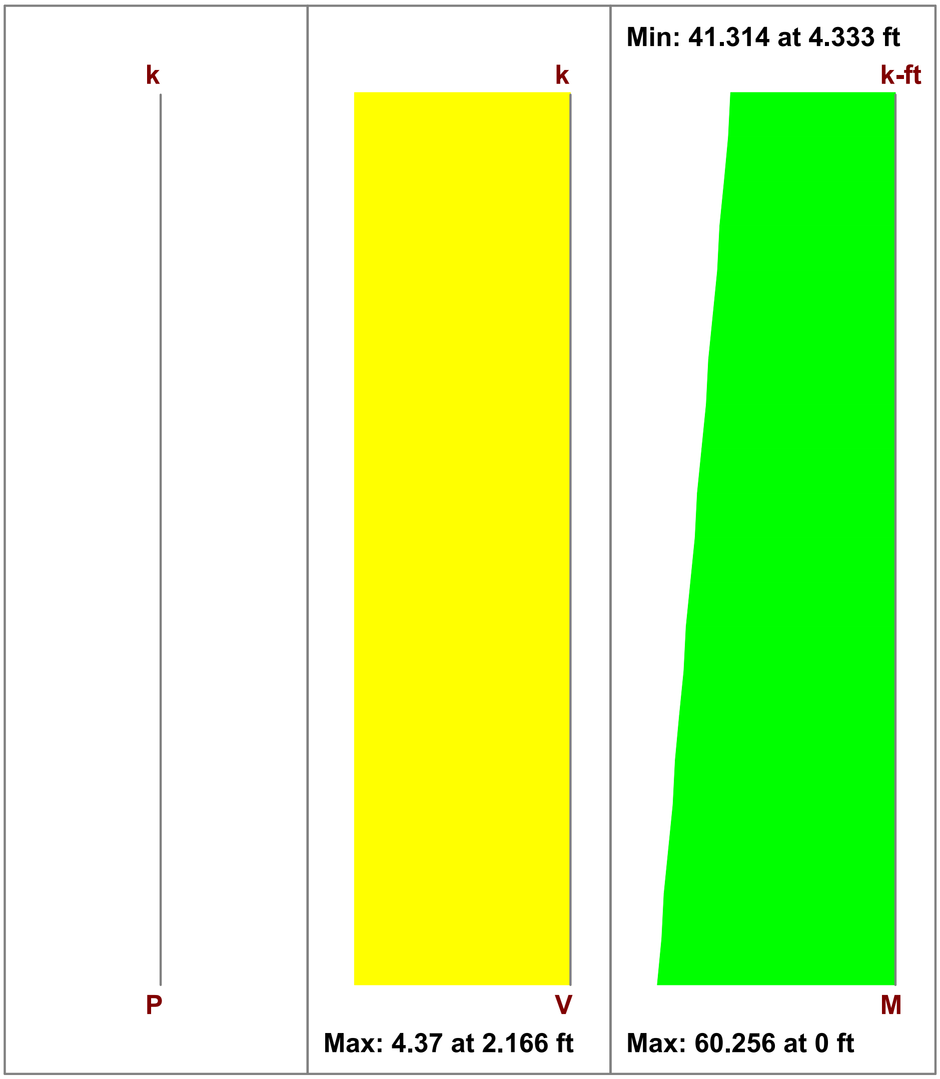
Min Chord Thk	:	**3.00**	**in**	Bolt Size:	:	**.875**	**in**	Base Cap(CD=1)	:	**4.228**	**k**
Reqd Chord Mat	:	**Douglas Fir**						CD factor	:	**1.6**	
								Adjusted Cap	:	**6.765**	**k**

CROSS SECTION DETAILING



CRITERIA		**MATERIALS**		**GEOMETRY**	
Code	: **AWC NDS-15:ASD**	Wall Studs	: **Spruce-Pine-Fir**	Total Height	: **4.333 ft**
		Stud Size	: **2X6**	Total Length	: **12.167 ft**
Wall Material	: **Spruce-Pine-Fir**	Chord Material	: **Spruce-Pine-Fir**	Region H/W	: **0.36**
Panel Schedule	: **User Selected**	Chord Size	: **2-2X6**	Cap. Adj. (2w/h)	: **1.00**
Optimize HD	: **Yes**	Top Pl & Sill	: **Spruce-Pine-Fir**	Stud Spacing	: **16 in**
HD Manufacturer	: **SIMPSON**	Top Pl Size	: **2-2X6**	K	: **1.00**
		Sill Pl Size	: **2X6**	HD Eccentricity	: **4.313 in**

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: **.359 k/ft**
Provided Cap	: **.43 k/ft**
Ratio	: **.835**
Governing LC	: **38 (Seismic)**

CHORDS

Max Comp Force	: **5.437 k**
Comp Capacity	: **26.631 k**
Comp Ratio	: **.204**
Gov Comp LC	: **40**
Max Tens Force	: **3.953 k**
Tens Capacity	: **13.514 k**
Tens Ratio	: **.293**
Gov Tens LC	: **52**

STUDS

No gravity-only LC solved.

HOLD-DOWNS

Required Cap	: **4.032 k**
Provided Cap	: **4.565 k**
Ratio	: **.883**
Governing LC	: **52**

DEFLECTIONS

Flexure Comp	: **.01 in**
Shear Comp	: **.111 in**
HD Elong	: **.022 in**
Tot Deflection	: **.143 in**
Governing LC	: **38**

DESIGN DETAILS

SELECTED SHEAR PANEL : **S1_15/32_8d@4**

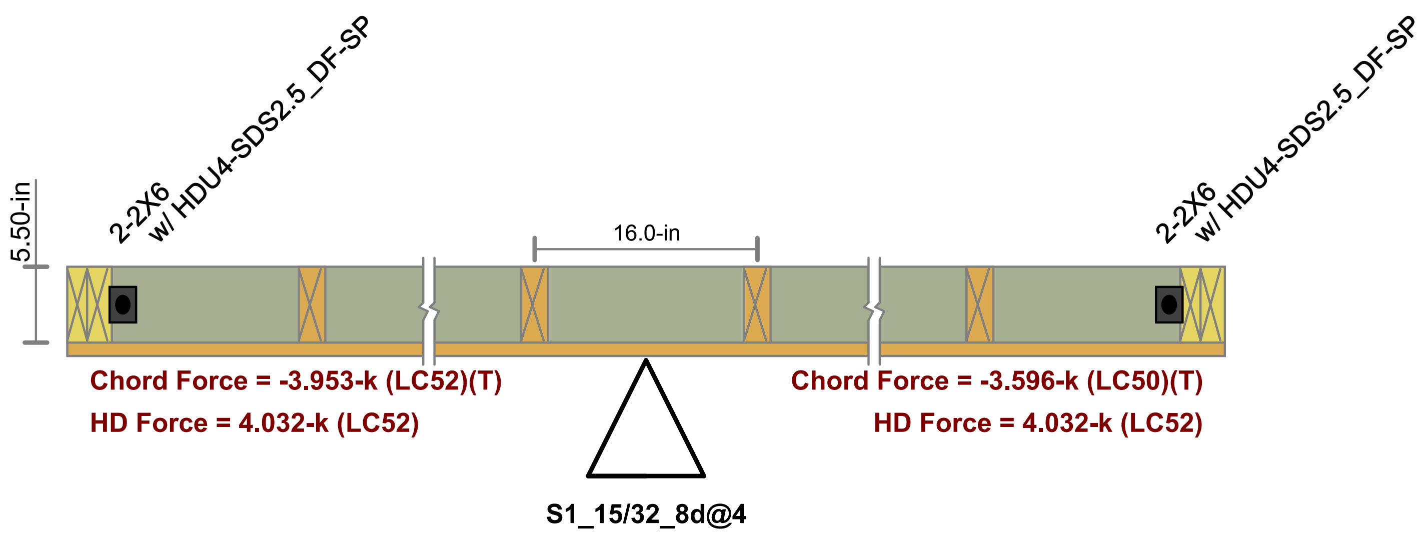
Panel Grade	: St-I		Nail Size	: 8d		Num Sides	: One	
Panel Thick	: 0.469	in	Reqd Pen	: 1.250	in	Over Gyp Brd.	: No	
			Reqd. Spacing	: 4	in	Shear Capacity	: 0.430	k/ft
						Adjusted Cap	: 0.430	k/ft

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

SELECTED HOLD-DOWN : **HDU4-SDS2.5_DF-SP**

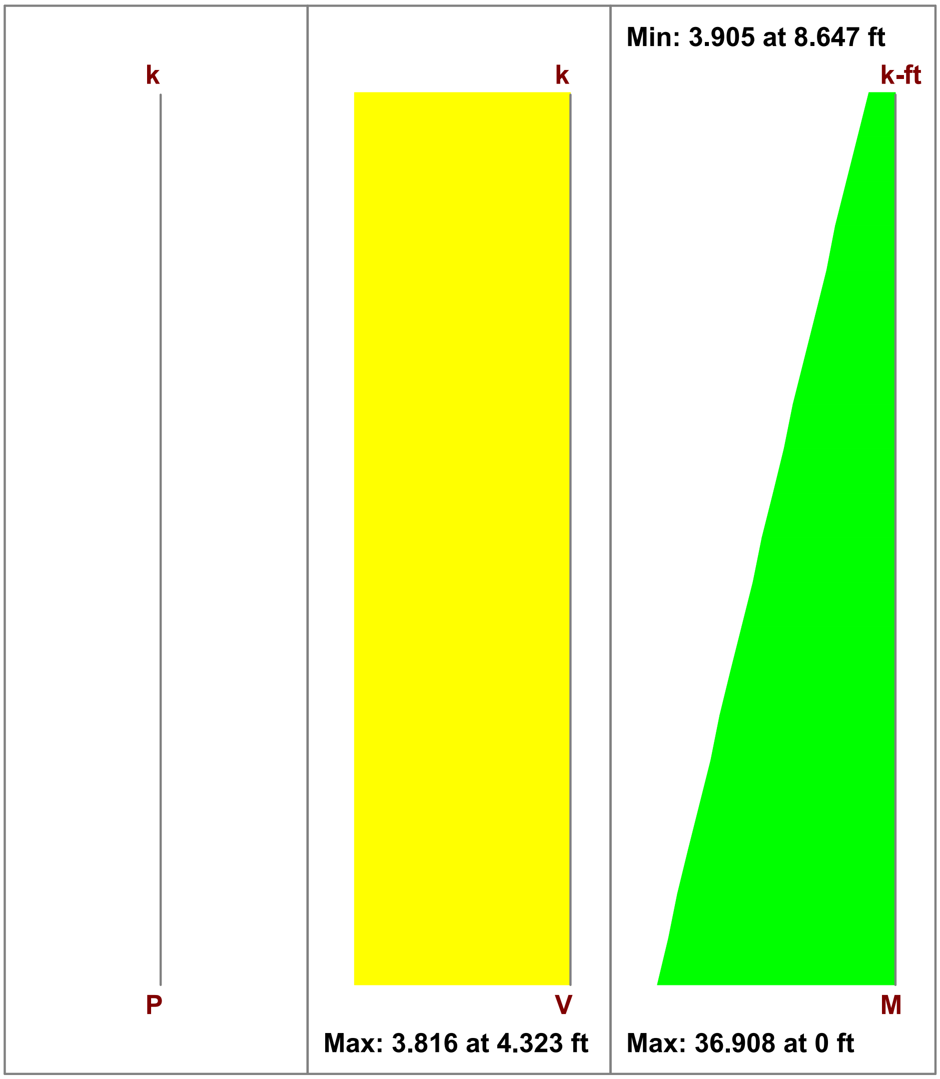
Min Chord Thk	: 3.00	in	Bolt Size:	: .625	in	Base Cap(CD=1)	: 2.853	k
Reqd Chord Mat	: Douglas Fir					CD factor	: 1.6	
						Adjusted Cap	: 4.565	k

CROSS SECTION DETAILING



CRITERIA		MATERIALS		GEOMETRY	
Code	: **AWC NDS-15:ASD**	Wall Studs	: **Spruce-Pine-Fir**	Total Height	: **8.647 ft**
		Stud Size	: **2X6**	Total Length	: **12.167 ft**
Wall Material	: **Spruce-Pine-Fir**	Chord Material	: **Spruce-Pine-Fir**	Region H/W	: **0.71**
Panel Schedule	: **User Selected**	Chord Size	: **2-2X6**	Cap. Adj. (2w/h)	: **1.00**
Optimize HD	: **Yes**	Top Pl & Sill	: **Spruce-Pine-Fir**	Stud Spacing	: **16 in**
HD Manufacturer	: **SIMPSON**	Top Pl Size	: **2-2X6**	K	: **1.00**
		Sill Pl Size	: **2X6**		

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap : **.314 k/ft**
Provided Cap : **.43 k/ft**
Ratio : **.729**
Governing LC : **38 (Seismic)**

CHORDS

Max Comp Force: **3.133 k**
Comp Capacity : **17.467 k**
Comp Ratio : **.179**
Gov Comp LC : **40**
Max Tens Force : **3.01 k**
Tens Capacity : **15.444 k**
Tens Ratio : **.195**
Gov Tens LC : **52**

STUDS

No gravity-only LC solved.

DEFLECTIONS

Flexure Comp : **.069 in**
Shear Comp : **.194 in**
HD Elong : **0 in**
Tot Deflection : **.263 in**
Governing LC : **38**

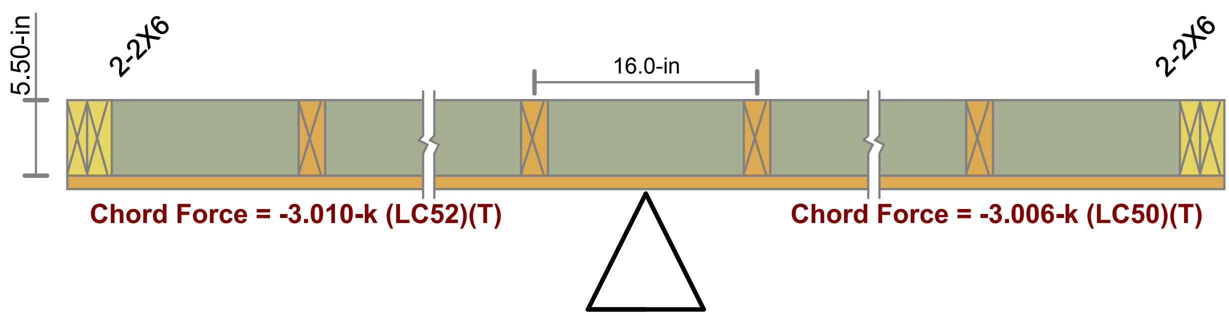
DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@4

Panel Grade	: **St-I**		Nail Size	: **8d**		Num Sides	: **One**	
Panel Thick	: **0.469**	**in**	Reqd Pen	: **1.250**	**in**	Over Gyp Brd.	: **No**	
			Reqd. Spacing	: **4**	**in**	Shear Capacity	: **0.430**	**k/ft**
						Adjusted Cap	: **0.430**	**k/ft**

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

CROSS SECTION DETAILING



S1_15/32_8d@4

CRITERIA

Code : **AWC NDS-15:ASD**

Wall Material : **Spruce-Pine-Fir**
Panel Schedule : **User Selected**

Optimize HD : **Yes**
HD Manufacturer: **SIMPSON**

MATERIALS

Wall Studs : **Spruce-Pine-Fir**
Stud Size : **2X6**

Chord Material : **Spruce-Pine-Fir**
Chord Size : **2-2X6**

Top Pl & Sill : **Spruce-Pine-Fir**
Top Pl Size : **2-2X6**
Sill Pl Size : **2X6**

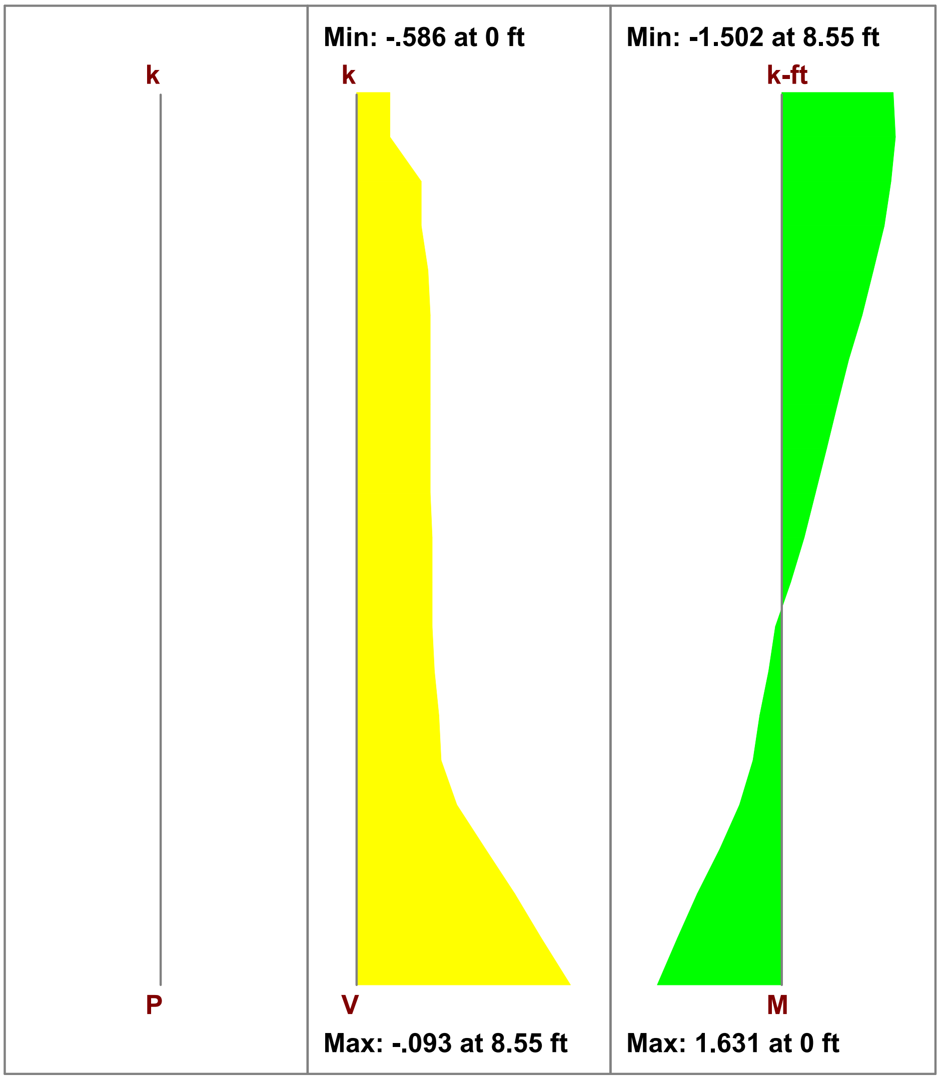
GEOMETRY

Total Height : **9 ft**
Total Length : **8.834 ft**

Region H/W : **1.02**
Cap. Adj. (2w/h) : **1.00**

Stud Spacing : **16 in**
K : **1.00**

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap : **.066 k/ft**
Provided Cap : **.28 k/ft**
Ratio : **.237**
Governing LC : **47 (Seismic)**

CHORDS

Max Comp Force: **1.182 k**
Comp Capacity : **16.415 k**
Comp Ratio : **.072**
Gov Comp LC : **47**
Max Tens Force : **0 k**
Tens Capacity : **15.444 k**
Tens Ratio : **0**
Gov Tens LC : **N/A**

STUDS

No gravity-only LC solved.

DEFLECTIONS

Flexure Comp : **.023 in**
Shear Comp : **.054 in**
HD Elong : **0 in**
Tot Deflection : **.077 in**
Governing LC : **47**

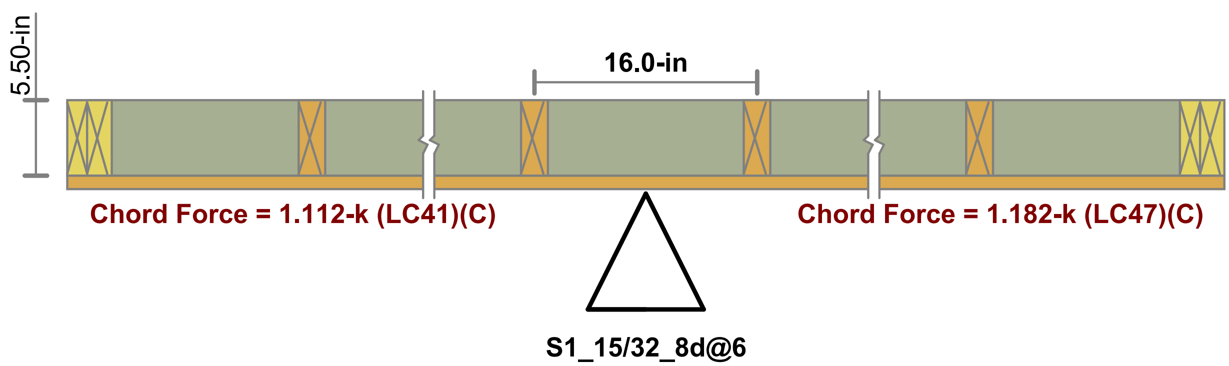
DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	:	**St-I**		Nail Size	:	**8d**		Num Sides	:	**One**	
Panel Thick	:	**0.469**	**in**	Reqd Pen	:	**1.250**	**in**	Over Gyp Brd.	:	**No**	
				Reqd. Spacing	:	**6**	**in**	Shear Capacity	:	**0.280**	**k/ft**
								Adjusted Cap	:	**0.280**	**k/ft**

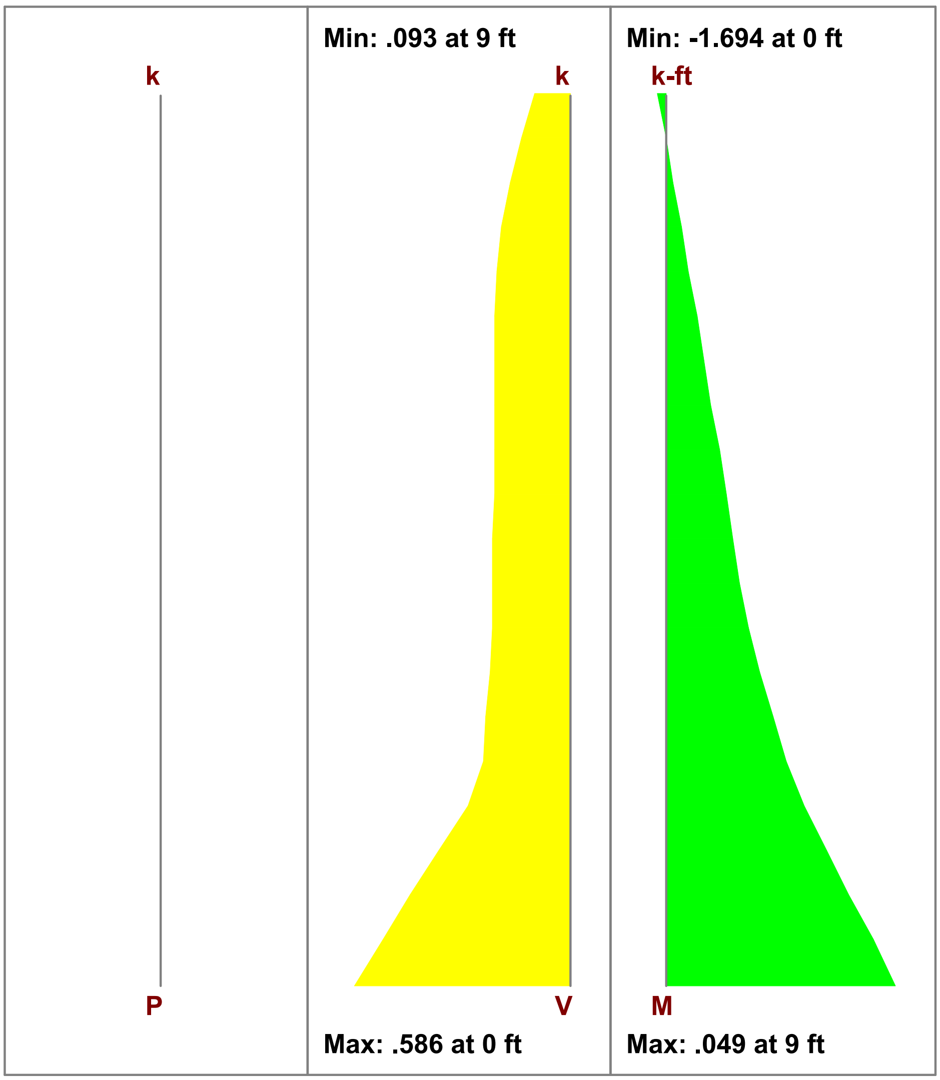
NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

CROSS SECTION DETAILING



CRITERIA		MATERIALS		GEOMETRY	
Code	: **AWC NDS-15:ASD**	Wall Studs	: **Spruce-Pine-Fir**	Total Height	: **9 ft**
		Stud Size	: **2X6**	Total Length	: **4.354 ft**
Wall Material	: **Spruce-Pine-Fir**	Chord Material	: **Spruce-Pine-Fir**	Region H/W	: **2.07**
Panel Schedule	: **User Selected**	Chord Size	: **2-2X6**	Cap. Adj. (2w/h)	: **0.97**
Optimize HD	: **Yes**	Top Pl & Sill	: **Spruce-Pine-Fir**	Stud Spacing	: **16 in**
HD Manufacturer	: **SIMPSON**	Top Pl Size	: **2-2X6**	K	: **1.00**
		Sill Pl Size	: **2X6**		

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: **.135 k/ft**
Provided Cap	: **.271 k/ft**
Ratio	: **.497**
Governing LC	: **47 (Seismic)**

CHORDS

Max Comp Force	: **1.27 k**
Comp Capacity	: **16.415 k**
Comp Ratio	: **.077**
Gov Comp LC	: **43**
Max Tens Force	: **0 k**
Tens Capacity	: **15.444 k**
Tens Ratio	: **0**
Gov Tens LC	: **N/A**

STUDS

No gravity-only LC solved.

DEFLECTIONS

Flexure Comp	: **.094 in**
Shear Comp	: **.11 in**
HD Elong	: **0 in**
Tot Deflection	: **.204 in**
Governing LC	: **47**

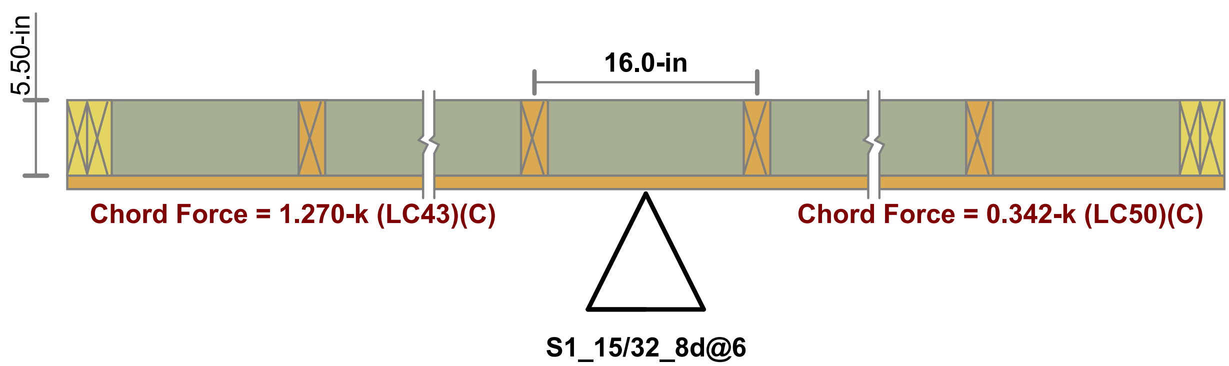
DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	:	**St-I**		Nail Size	:	**8d**		Num Sides	:	**One**	
Panel Thick	:	**0.469**	**in**	Reqd Pen	:	**1.250**	**in**	Over Gyp Brd.	:	**No**	
				Reqd. Spacing	:	**6**	**in**	Shear Capacity	:	**0.280**	**k/ft**
								Adjusted Cap	:	**0.271**	**k/ft**

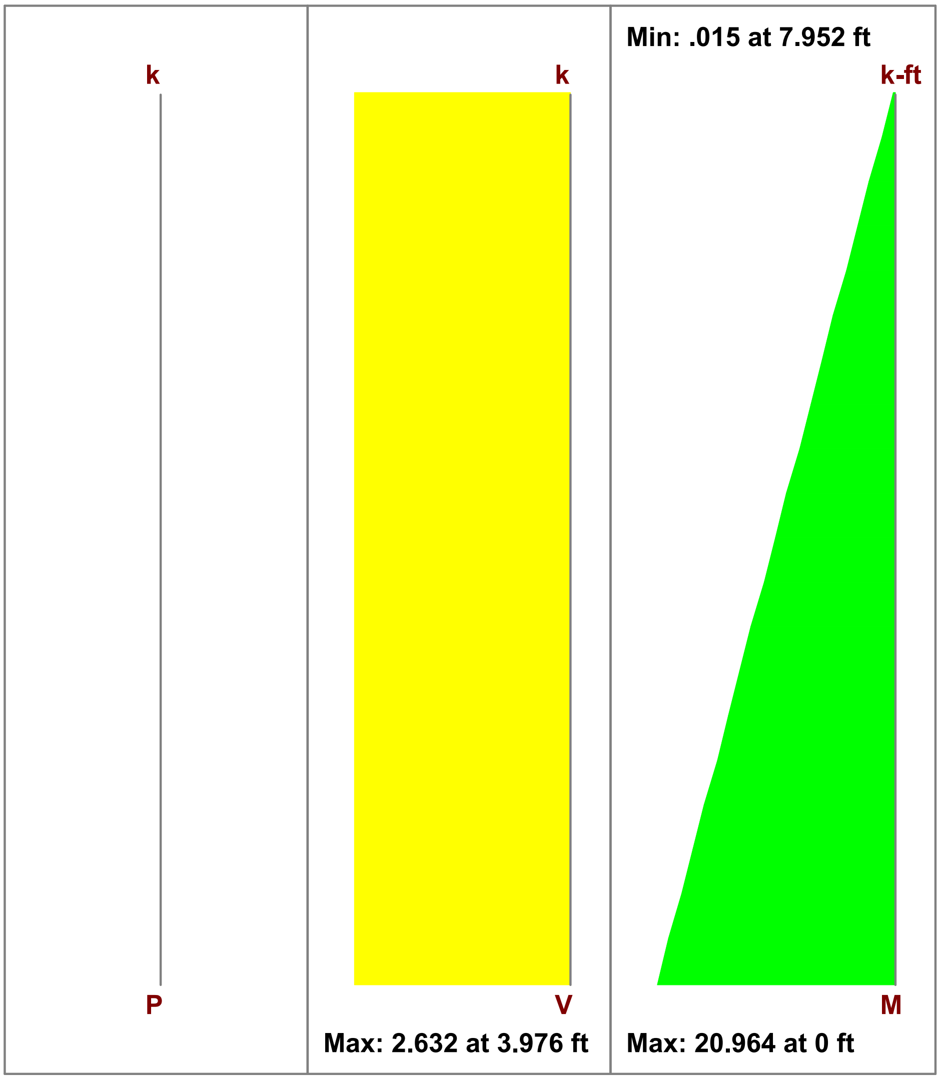
NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

CROSS SECTION DETAILING



CRITERIA		**MATERIALS**		**GEOMETRY**	
Code	: **AWC NDS-15:ASD**	Wall Studs	: **Spruce-Pine-Fir**	Total Height	: **7.952ft**
		Stud Size	: **2X6**	Total Length	: **16.25ft**
Wall Material	: **Spruce-Pine-Fir**	Chord Material	: **Spruce-Pine-Fir**	Region H/W	: **0.49**
Panel Schedule	: **User Selected**	Chord Size	: **2-2X6**	Cap. Adj. (2w/h)	: **1.00**
Optimize HD	: **Yes**	Top Pl & Sill	: **Spruce-Pine-Fir**	Stud Spacing	: **16 in**
HD Manufacturer	: **SIMPSON**	Top Pl Size	: **2-2X6**	K	: **1.00**
		Sill Pl Size	: **2X6**		

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap : **.162 k/ft**
Provided Cap : **.28 k/ft**
Ratio : **.579**
Governing LC : **37 (Seismic)**

CHORDS

Max Comp Force: **1.34 k**
Comp Capacity : **19.706 k**
Comp Ratio : **.068**
Gov Comp LC : **39**
Max Tens Force : **1.215 k**
Tens Capacity : **15.444 k**
Tens Ratio : **.079**
Gov Tens LC : **51**

STUDS

No gravity-only LC solved.

DEFLECTIONS

Flexure Comp : **.021 in**
Shear Comp : **.117 in**
HD Elong : **0 in**
Tot Deflection : **.138 in**
Governing LC : **37**

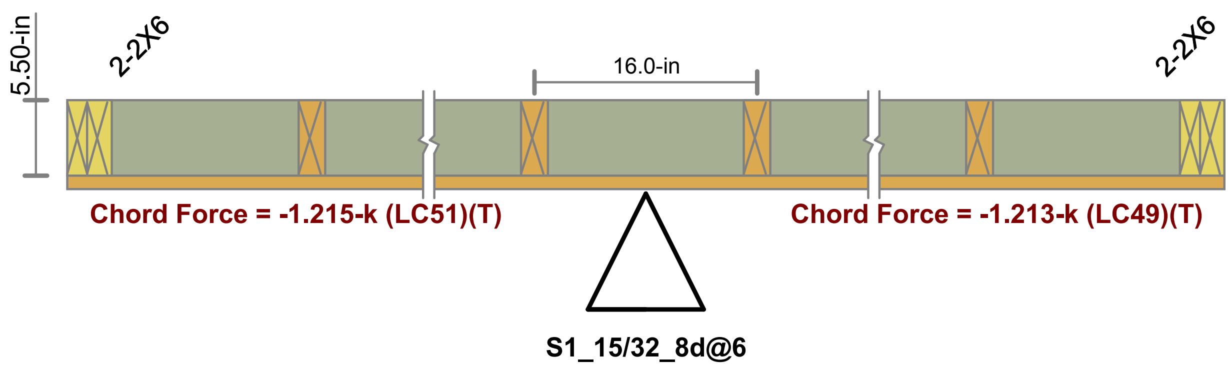
DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	: **St-I**		Nail Size	: **8d**		Num Sides	: **One**	
Panel Thick	: **0.469**	**in**	Reqd Pen	: **1.250**	**in**	Over Gyp Brd.	: **No**	
			Reqd. Spacing	: **6**	**in**	Shear Capacity	: **0.280**	**k/ft**
						Adjusted Cap	: **0.280**	**k/ft**

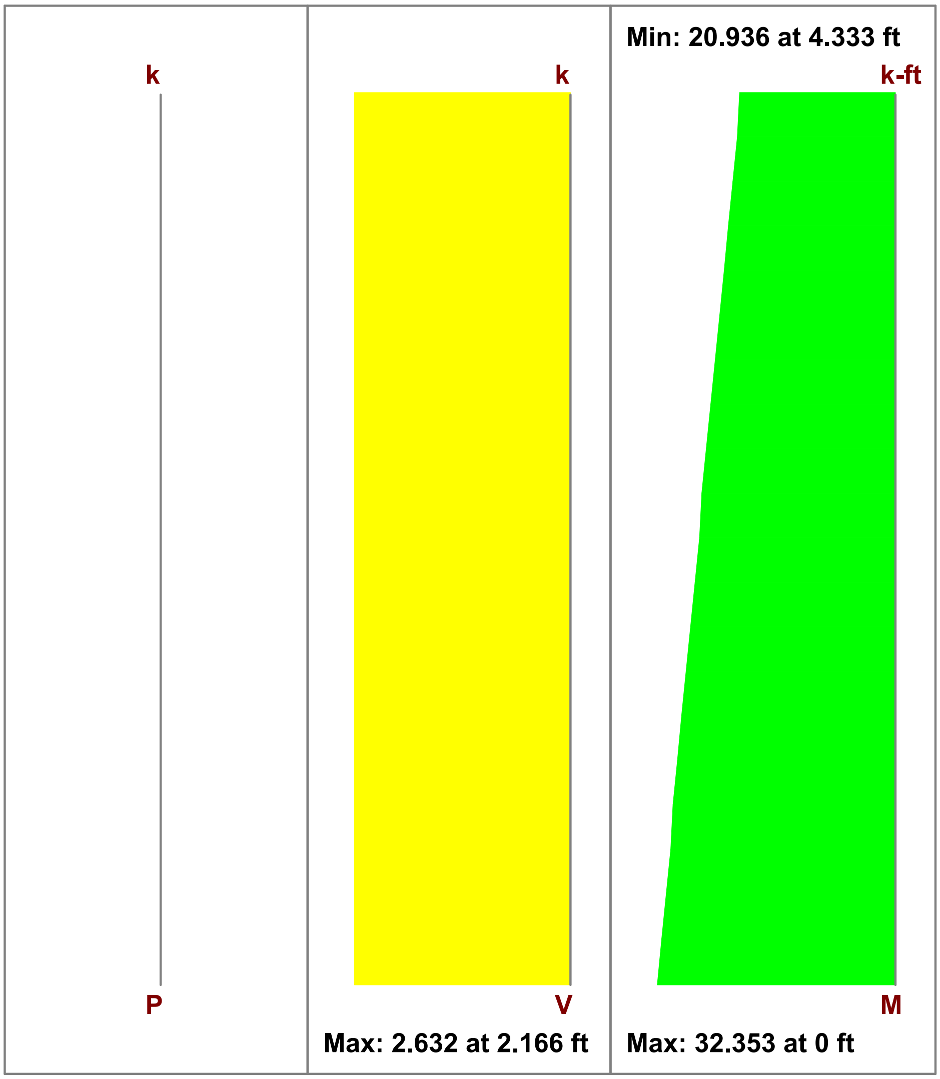
NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

CROSS SECTION DETAILING



CRITERIA		MATERIALS		GEOMETRY	
Code	: AWC NDS-15:ASD	Wall Studs	: Spruce-Pine-Fir	Total Height	: 4.333 ft
		Stud Size	: 2X6	Total Length	: 16.25 ft
Wall Material	: Spruce-Pine-Fir	Chord Material	: Spruce-Pine-Fir	Region H/W	: 0.27
Panel Schedule	: User Selected	Chord Size	: 2-2X6	Cap. Adj. (2w/h)	: 1.00
Optimize HD	: Yes	Top Pl & Sill	: Spruce-Pine-Fir	Stud Spacing	: 16 in
HD Manufacturer	: SIMPSON	Top Pl Size	: 2-2X6	K	: 1.00
		Sill Pl Size	: 2X6	HD Eccentricity	: 4.313 in

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL
Required Cap : .162 k/ft
Provided Cap : .28 k/ft
Ratio : .579
Governing LC : 37 (Seismic)

CHORDS
Max Comp Force: 2.109 k
Comp Capacity : 26.631 k
Comp Ratio : .079
Gov Comp LC : 39
Max Tens Force : 1.752 k
Tens Capacity : 13.514 k
Tens Ratio : .13
Gov Tens LC : 51

STUDS
No gravity-only LC solved.

HOLD-DOWNS
Required Cap : 1.778 k
Provided Cap : 3.075 k
Ratio : .578
Governing LC : 51

DEFLECTIONS
Flexure Comp : .003 in
Shear Comp : .064 in
HD Elong : .011 in
Tot Deflection : .078 in
Governing LC : 37

DESIGN DETAILS

SELECTED SHEAR PANEL : **S1_15/32_8d@6**

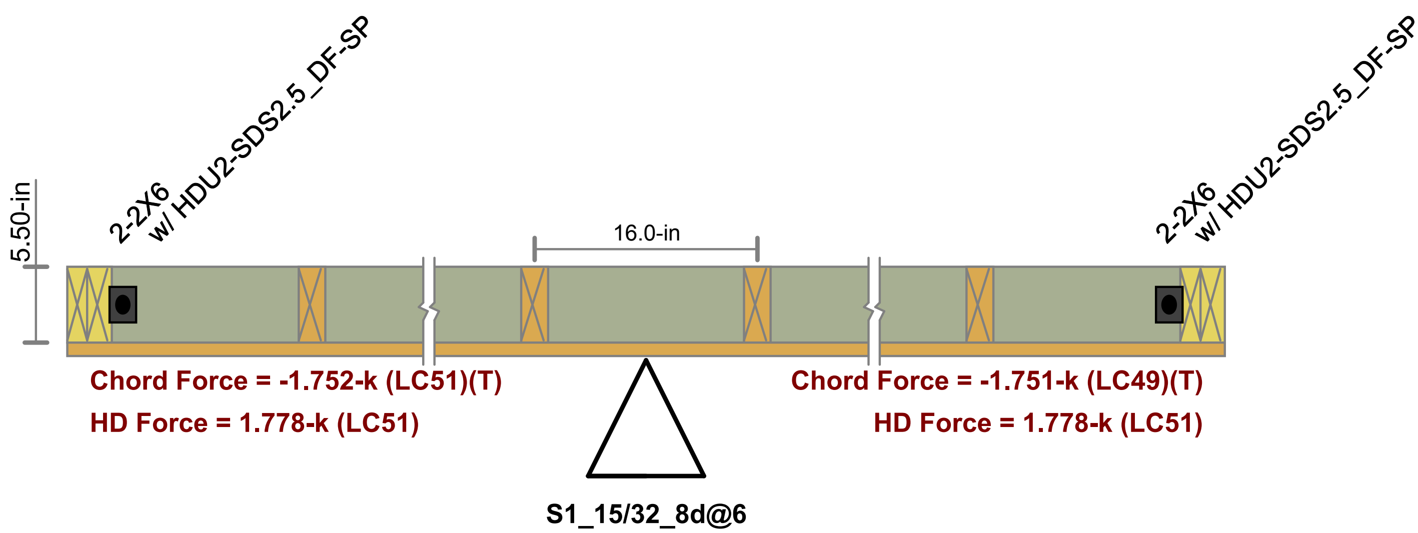
Panel Grade	:	**St-I**	Nail Size	:	**8d**		Num Sides	:	**One**	
Panel Thick	:	**0.469** **in**	Reqd Pen	:	**1.250**	**in**	Over Gyp Brd.	:	**No**	
			Reqd. Spacing	:	**6**	**in**	Shear Capacity	:	**0.280**	**k/ft**
							Adjusted Cap	:	**0.280**	**k/ft**

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

SELECTED HOLD-DOWN : **HDU2-SDS2.5_DF-SP**

Min Chord Thk	:	**3.00** **in**	Bolt Size:	:	**.625**	**in**	Base Cap(CD=1)	:	**1.922**	**k**
Reqd Chord Mat	:	**Douglas Fir**					CD factor	:	**1.6**	
							Adjusted Cap	:	**3.075**	**k**

CROSS SECTION DETAILING



CRITERIA

Code	: **AWC NDS-15:ASD**
Wall Material	: **Spruce-Pine-Fir**
Panel Schedule	: **User Selected**
Optimize HD	: **Yes**
HD Manufacturer	: **SIMPSON**

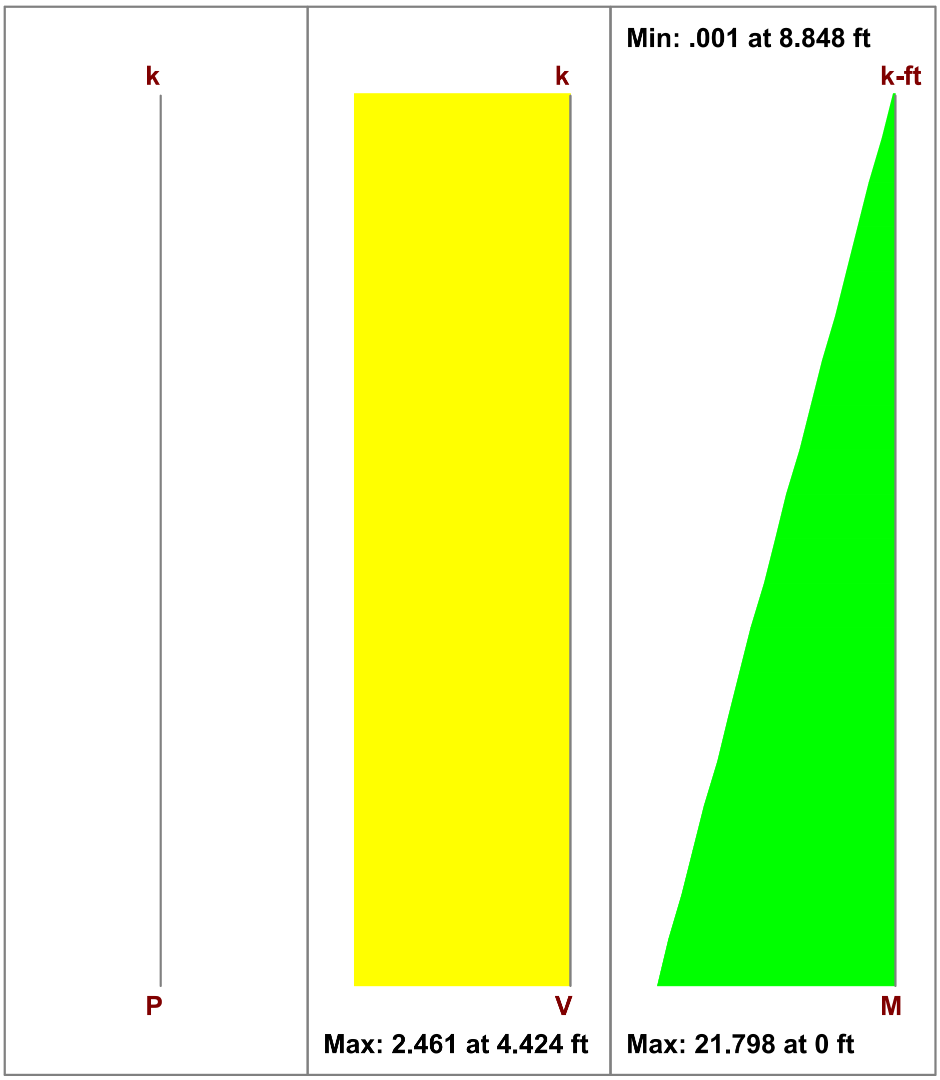
MATERIALS

Wall Studs	: **Spruce-Pine-Fir**
Stud Size	: **2X6**
Chord Material	: **Spruce-Pine-Fir**
Chord Size	: **2-2X6**
Top Pl & Sill	: **Spruce-Pine-Fir**
Top Pl Size	: **2-2X6**
Sill Pl Size	: **2X6**

GEOMETRY

Total Height	: **8.848 ft**
Total Length	: **12.292 ft**
Region H/W	: **0.72**
Cap. Adj. (2w/h)	: **1.00**
Stud Spacing	: **16 in**
K	: **1.00**

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: **.2 k/ft**
Provided Cap	: **.28 k/ft**
Ratio	: **.715**
Governing LC	: **37 (Seismic)**

CHORDS

Max Comp Force	: **1.845 k**
Comp Capacity	: **16.859 k**
Comp Ratio	: **.109**
Gov Comp LC	: **39**
Max Tens Force	: **1.726 k**
Tens Capacity	: **15.444 k**
Tens Ratio	: **.112**
Gov Tens LC	: **51**

STUDS

No gravity-only LC solved.

DEFLECTIONS

Flexure Comp	: **.047 in**
Shear Comp	: **.161 in**
HD Elong	: **0 in**
Tot Deflection	: **.208 in**
Governing LC	: **37**

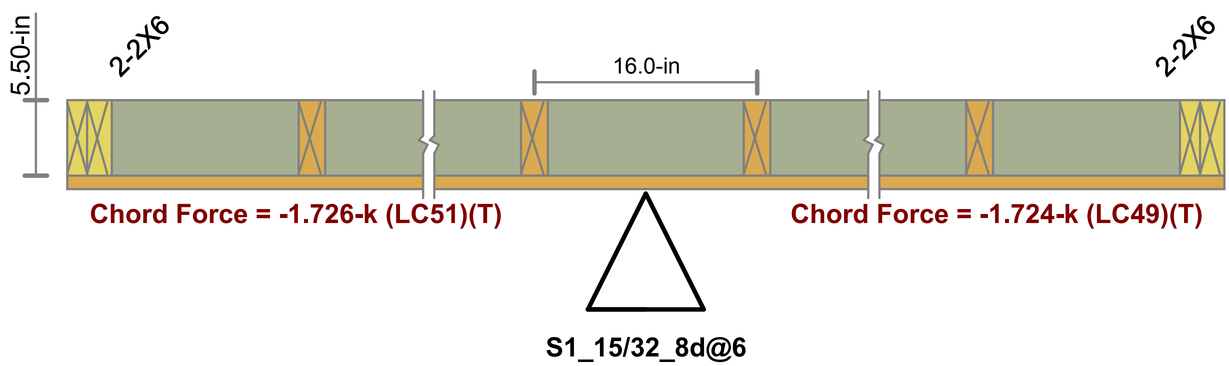
DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	: St-I		Nail Size	: 8d		Num Sides	: One	
Panel Thick	: 0.469	in	Reqd Pen	: 1.250	in	Over Gyp Brd.	: No	
			Reqd. Spacing	: 6	in	Shear Capacity	: 0.280	k/ft
						Adjusted Cap	: 0.280	k/ft

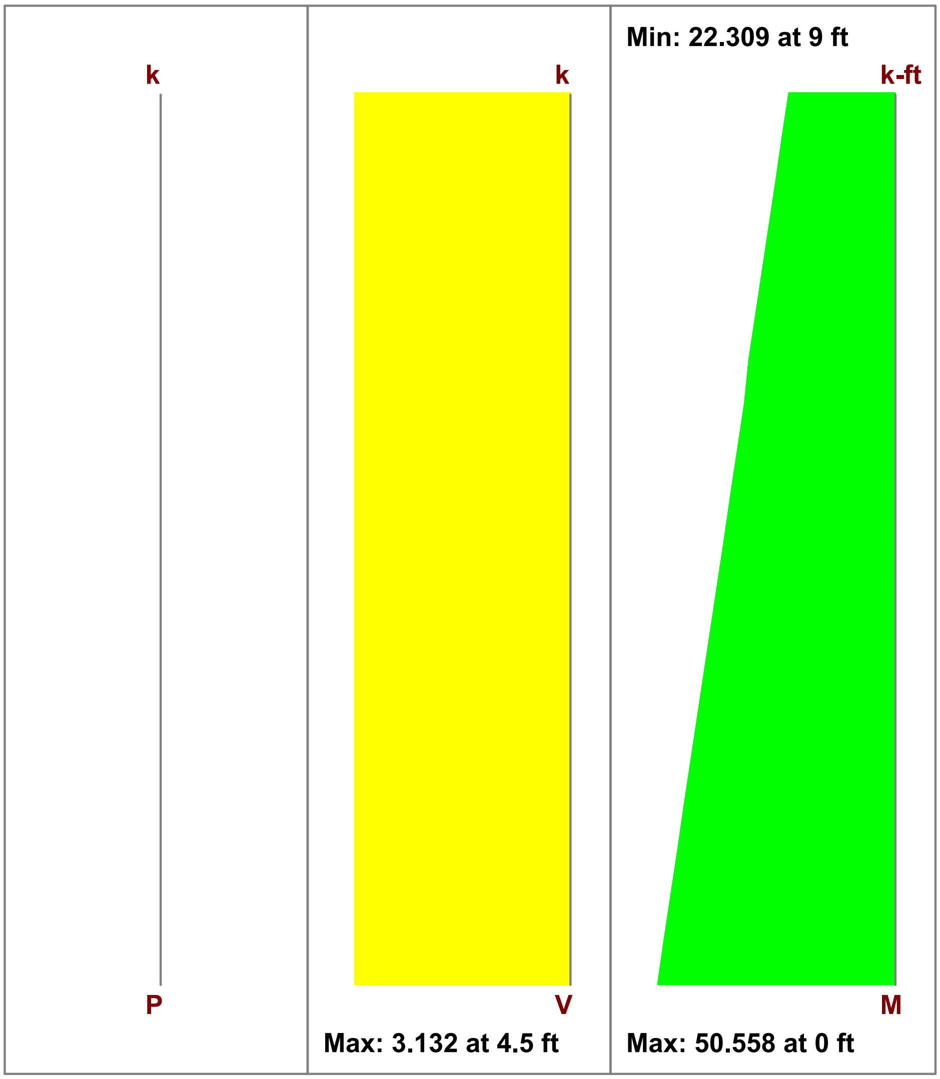
NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

CROSS SECTION DETAILING



CRITERIA		MATERIALS		GEOMETRY	
Code	: AWC NDS-15:ASD	Wall Studs	: Spruce-Pine-Fir	Total Height	: 9 ft
		Stud Size	: 2X6	Total Length	: 12.292 ft
Wall Material	: Spruce-Pine-Fir	Chord Material	: Spruce-Pine-Fir	Region H/W	: 0.73
Panel Schedule	: User Selected	Chord Size	: 2-2X6	Cap. Adj. (2w/h)	: 1.00
Optimize HD	: Yes	Top Pl & Sill	: Spruce-Pine-Fir	Stud Spacing	: 16 in
HD Manufacturer	: SIMPSON	Top Pl Size	: 2-2X6	K	: 1.00
		Sill Pl Size	: 2X6		

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: .255 k/ft
Provided Cap	: .28 k/ft
Ratio	: .91
Governing LC	: 37 (Seismic)

CHORDS

Max Comp Force:	4.358 k
Comp Capacity	: 16.415 k
Comp Ratio	: .266
Gov Comp LC	: 39
Max Tens Force	: 3.791 k
Tens Capacity	: 15.444 k
Tens Ratio	: .245
Gov Tens LC	: 51

STUDS

No gravity-only LC solved.

DEFLECTIONS

Flexure Comp	: .063 in
Shear Comp	: .208 in
HD Elong	: .062 in
Tot Deflection	: .333 in
Governing LC	: 37

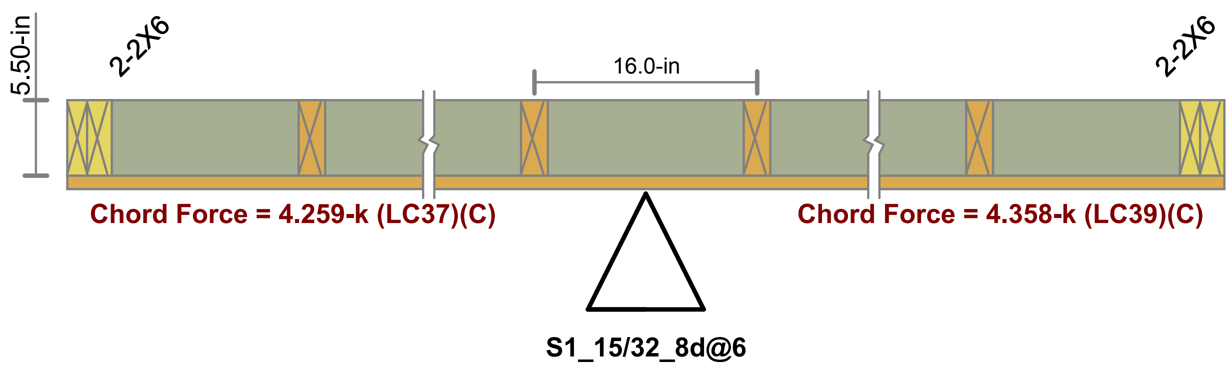
DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	:	**St-I**		Nail Size	:	**8d**		Num Sides	:	**One**	
Panel Thick	:	**0.469**	**in**	Reqd Pen	:	**1.250**	**in**	Over Gyp Brd.	:	**No**	
				Reqd. Spacing	:	**6**	**in**	Shear Capacity	:	**0.280**	**k/ft**
								Adjusted Cap	:	**0.280**	**k/ft**

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

CROSS SECTION DETAILING



CRITERIA

Code : **AWC NDS-15:ASD**

Wall Material : **Spruce-Pine-Fir**
Panel Schedule : **User Selected**

Optimize HD : **Yes**
HD Manufacturer: **SIMPSON**

MATERIALS

Wall Studs : **Spruce-Pine-Fir**
Stud Size : **2X6**

Chord Material : **Spruce-Pine-Fir**
Chord Size : **2-2X6**

Top Pl & Sill : **Spruce-Pine-Fir**
Top Pl Size : **2-2X6**
Sill Pl Size : **2X6**

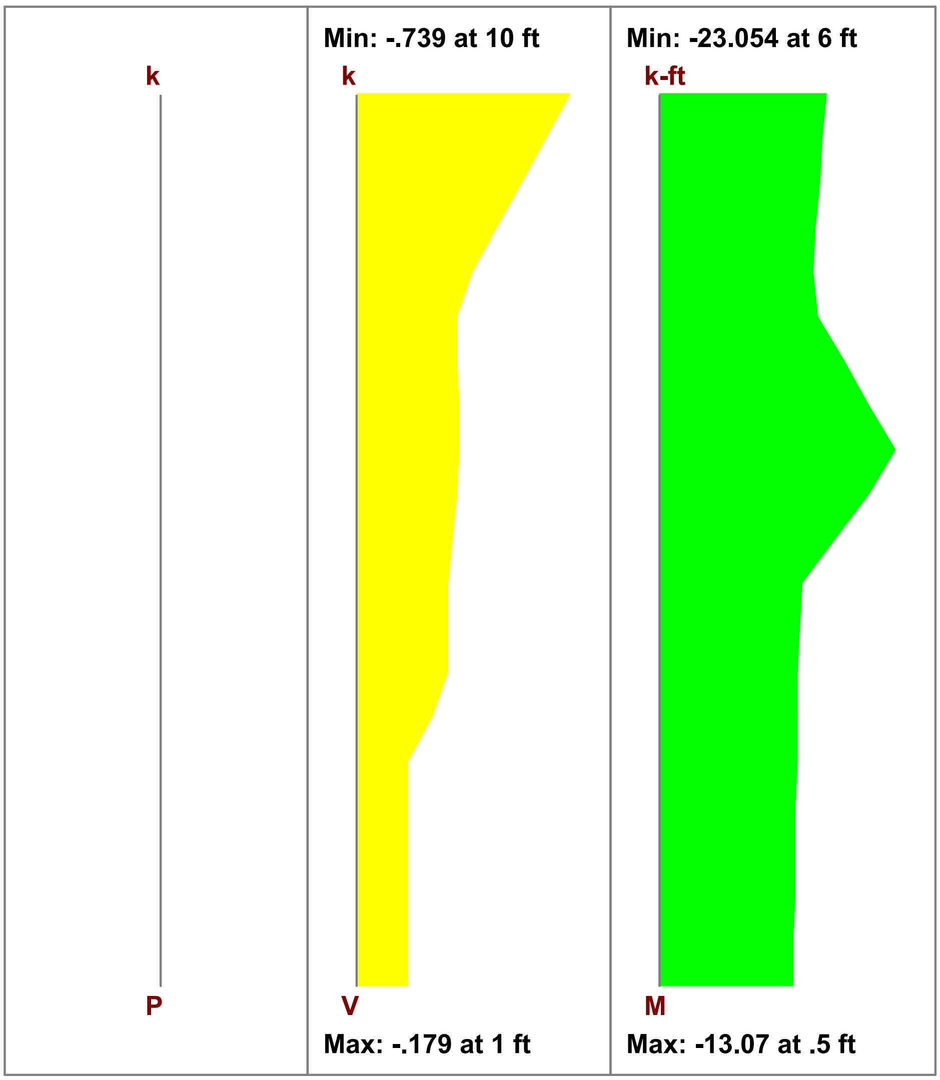
GEOMETRY

Total Height : **10 ft**
Total Length : **8.834 ft**

Region H/W : **1.13**
Cap. Adj. (2w/h) : **1.00**

Stud Spacing : **16 in**
K : **1.00**

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap : **.084 k/ft**
Provided Cap : **.28 k/ft**
Ratio : **.299**
Governing LC : **45 (Seismic)**

CHORDS

Max Comp Force: **4.697 k**
Comp Capacity : **13.789 k**
Comp Ratio : **.341**
Gov Comp LC : **45**
Max Tens Force : **0 k**
Tens Capacity : **15.444 k**
Tens Ratio : **0**
Gov Tens LC : **N/A**

STUDS

No gravity-only LC solved.

HOLD-DOWNS

Hold-Downs are not required

DEFLECTIONS

Flexure Comp : **.039 in**
Shear Comp : **.076 in**
HD Elong : **0 in**
Tot Deflection : **.115 in**
Governing LC : **45**

DESIGN DETAILS

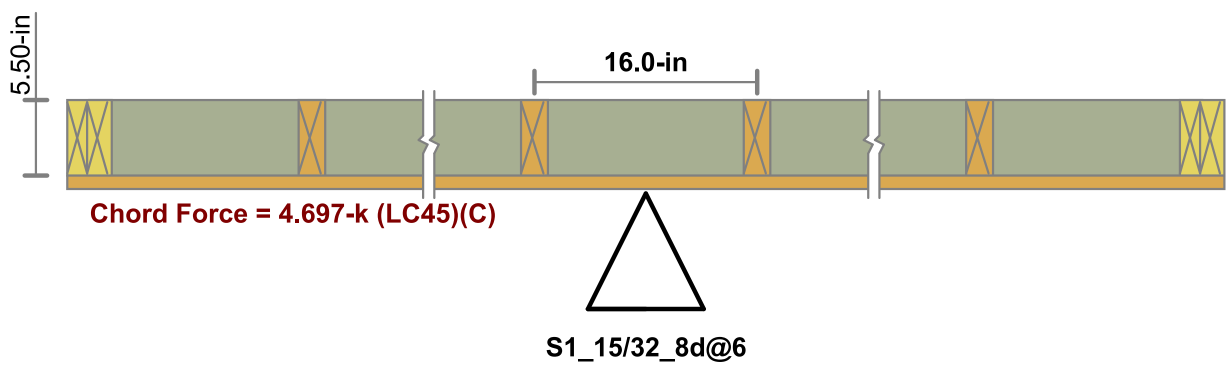
SELECTED SHEAR PANEL : **S1_15/32_8d@6**

Panel Grade	: **St-I**	Nail Size	: **8d**	Num Sides	: **One**
Panel Thick	: **0.469 in**	Reqd Pen	: **1.250 in**	Over Gyp Brd.	: **No**
		Reqd. Spacing	: **6 in**	Shear Capacity	: **0.280 k/ft**
				Adjusted Cap	: **0.280 k/ft**

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

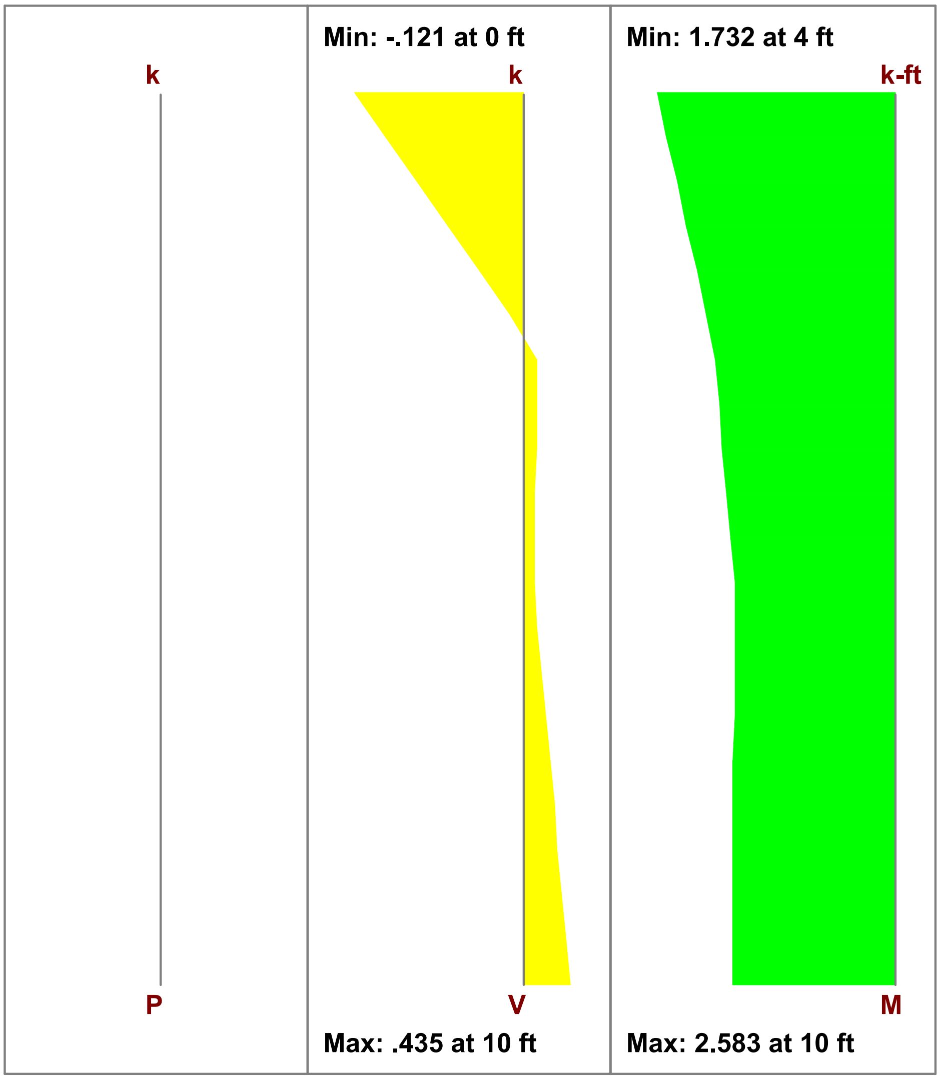
SELECTED HOLD-DOWN : **None required**

CROSS SECTION DETAILING



CRITERIA		MATERIALS		GEOMETRY	
Code	: AWC NDS-15:ASD	Wall Studs	: Spruce-Pine-Fir	Total Height	: 10 ft
		Stud Size	: 2X6	Total Length	: 4.354 ft
Wall Material	: Spruce-Pine-Fir	Chord Material	: Spruce-Pine-Fir	Region H/W	: 2.30
Panel Schedule	: User Selected	Chord Size	: 2-2X6	Cap. Adj. (2w/h)	: 0.87
Optimize HD	: Yes	Top Pl & Sill	: Spruce-Pine-Fir	Stud Spacing	: 16 in
HD Manufacturer	: SIMPSON	Top Pl Size	: 2-2X6	K	: 1.00
		Sill Pl Size	: 2X6		

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap : .1 k/ft
Provided Cap : .244 k/ft
Ratio : .41
Governing LC : 37 (Seismic)

CHORDS

Max Comp Force: 2.665 k
Comp Capacity : 13.789 k
Comp Ratio : .193
Gov Comp LC : 44
Max Tens Force : 0 k
Tens Capacity : 15.444 k
Tens Ratio : 0
Gov Tens LC : N/A

STUDS

No gravity-only LC solved.

HOLD-DOWNS

Hold-Downs are not required

DEFLECTIONS

Flexure Comp : .095 in
Shear Comp : .091 in
HD Elong : 0 in
Tot Deflection : .186 in
Governing LC : 37

DESIGN DETAILS

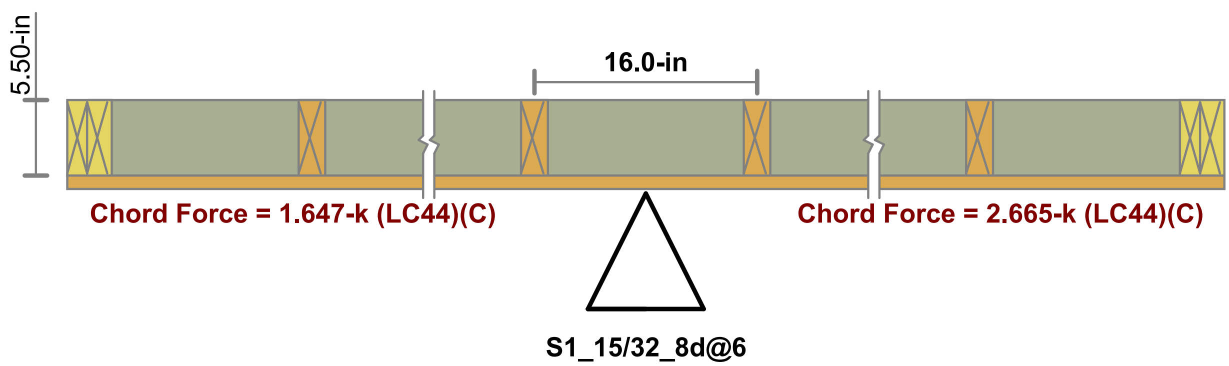
SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	: St-I		Nail Size	: 8d		Num Sides	: One	
Panel Thick	: 0.469	in	Reqd Pen	: 1.250	in	Over Gyp Brd.	: No	
			Reqd. Spacing	: 6	in	Shear Capacity	: 0.280	k/ft
						Adjusted Cap	: 0.244	k/ft

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

SELECTED HOLD-DOWN : None required

CROSS SECTION DETAILING



CRITERIA

Code	: AWC NDS-15:ASD
Wall Material	: Spruce-Pine-Fir
Panel Schedule	: User Selected
Optimize HD	: Yes
HD Manufacturer	: SIMPSON

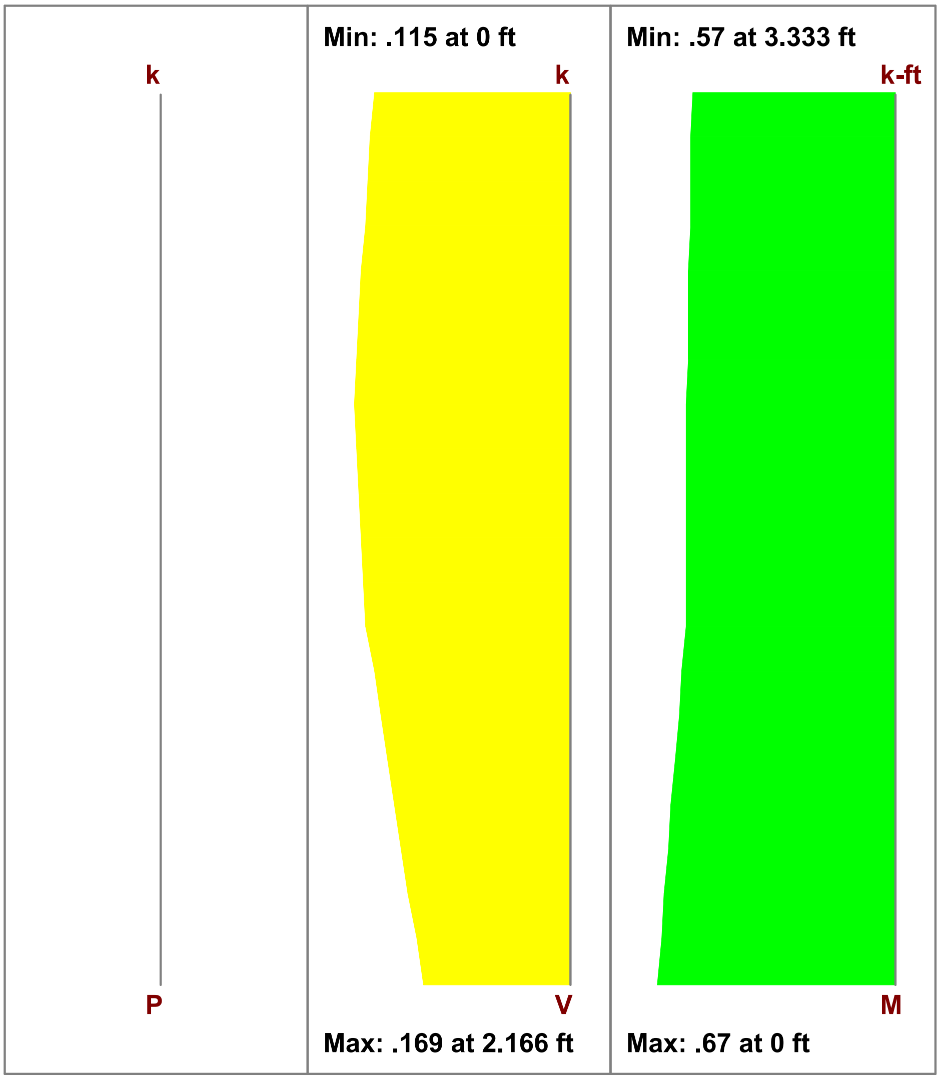
MATERIALS

Wall Studs	: Spruce-Pine-Fir
Stud Size	: 2X6
Chord Material	: Spruce-Pine-Fir
Chord Size	: 2-2X6
Top Pl & Sill	: Spruce-Pine-Fir
Top Pl Size	: 2-2X6
Sill Pl Size	: 2X6

GEOMETRY

Total Height	: 3.333 ft
Total Length	: 2.771 ft
Region H/W	: 1.20
Cap. Adj. (2w/h)	: 1.00
Stud Spacing	: 16 in
K	: 1.00

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: .061 k/ft
Provided Cap	: .28 k/ft
Ratio	: .217
Governing LC	: 38 (Seismic)

CHORDS

Max Comp Force	: 1.176 k
Comp Capacity	: 31.886 k
Comp Ratio	: .037
Gov Comp LC	: 44
Max Tens Force	: 0 k
Tens Capacity	: 15.444 k
Tens Ratio	: 0
Gov Tens LC	: N/A

STUDS

No gravity-only LC solved.

HOLD-DOWNS

Hold-Downs are not required

DEFLECTIONS

Flexure Comp	: .003 in
Shear Comp	: .018 in
HD Elong	: 0 in
Tot Deflection	: .022 in
Governing LC	: 38

DESIGN DETAILS

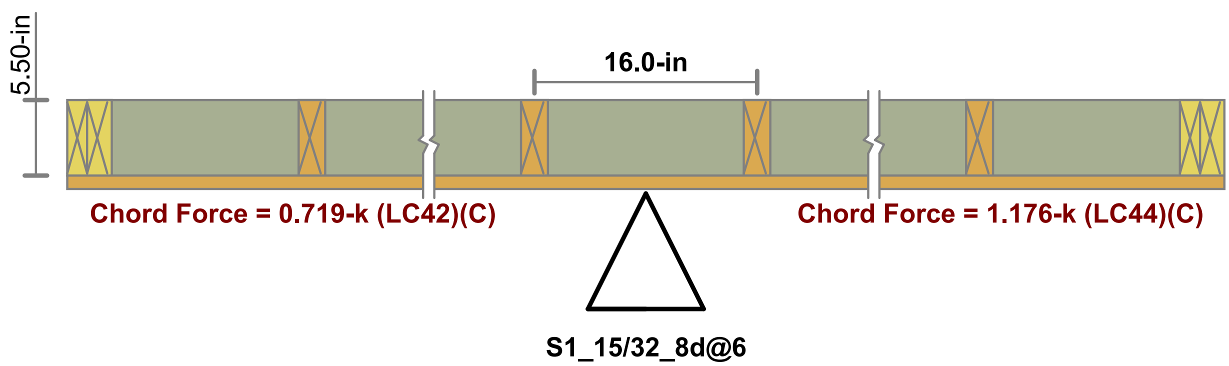
SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	: **St-I**		Nail Size	: **8d**		Num Sides	: **One**	
Panel Thick	: **0.469**	**in**	Reqd Pen	: **1.250**	**in**	Over Gyp Brd.	: **No**	
			Reqd. Spacing	: **6**	**in**	Shear Capacity	: **0.280**	**k/ft**
						Adjusted Cap	: **0.280**	**k/ft**

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

SELECTED HOLD-DOWN : None required

CROSS SECTION DETAILING



CRITERIA

Code	: **AWC NDS-15:ASD**
Wall Material	: **Spruce-Pine-Fir**
Panel Schedule	: **User Selected**
Optimize HD	: **Yes**
HD Manufacturer	: **SIMPSON**

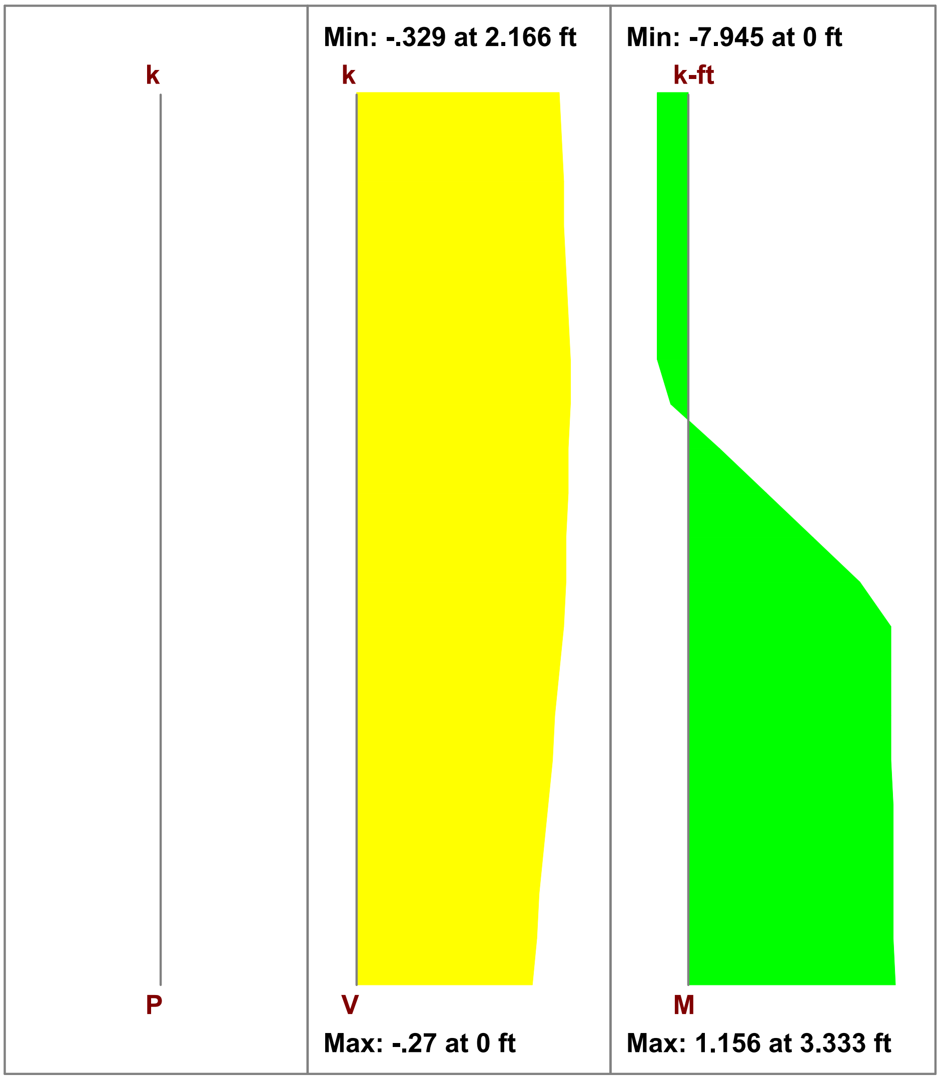
MATERIALS

Wall Studs	: **Spruce-Pine-Fir**
Stud Size	: **2X6**
Chord Material	: **Spruce-Pine-Fir**
Chord Size	: **2-2X6**
Top Pl & Sill	: **Spruce-Pine-Fir**
Top Pl Size	: **2-2X6**
Sill Pl Size	: **2X6**

GEOMETRY

Total Height	: **3.333 ft**
Total Length	: **6.167 ft**
Region H/W	: **0.54**
Cap. Adj. (2w/h)	: **1.00**
Stud Spacing	: **16 in**
K	: **1.00**

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: **.053 k/ft**
Provided Cap	: **.28 k/ft**
Ratio	: **.19**
Governing LC	: **40 (Seismic)**

CHORDS

Max Comp Force	: **2.605 k**
Comp Capacity	: **31.886 k**
Comp Ratio	: **.082**
Gov Comp LC	: **42**
Max Tens Force	: **0 k**
Tens Capacity	: **15.444 k**
Tens Ratio	: **0**
Gov Tens LC	: **N/A**

STUDS

No gravity-only LC solved.

HOLD-DOWNS

Hold-Downs are not required

DEFLECTIONS

Flexure Comp	: **.001 in**
Shear Comp	: **.016 in**
HD Elong	: **0 in**
Tot Deflection	: **.017 in**
Governing LC	: **40**

DESIGN DETAILS

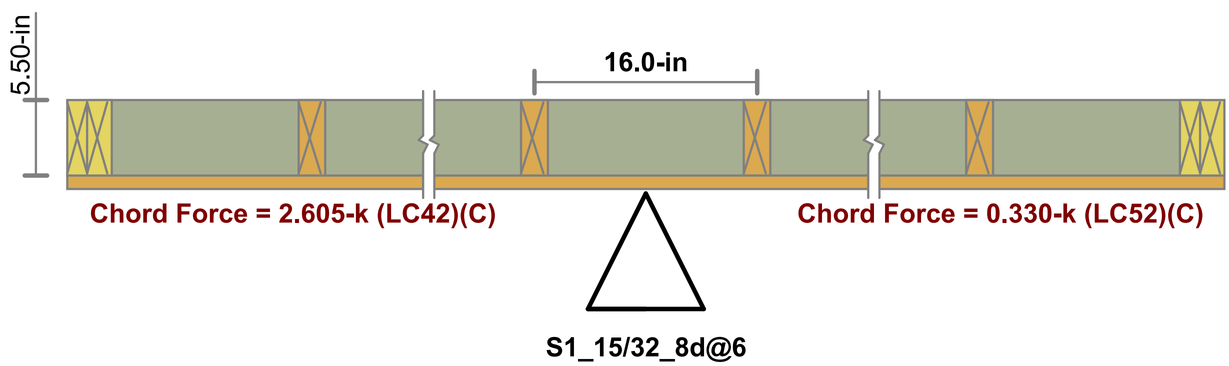
SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	:	**St-I**		Nail Size	:	**8d**		Num Sides	:	**One**	
Panel Thick	:	**0.469**	**in**	Reqd Pen	:	**1.250**	**in**	Over Gyp Brd.	:	**No**	
				Reqd. Spacing	:	**6**	**in**	Shear Capacity	:	**0.280**	**k/ft**
								Adjusted Cap	:	**0.280**	**k/ft**

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

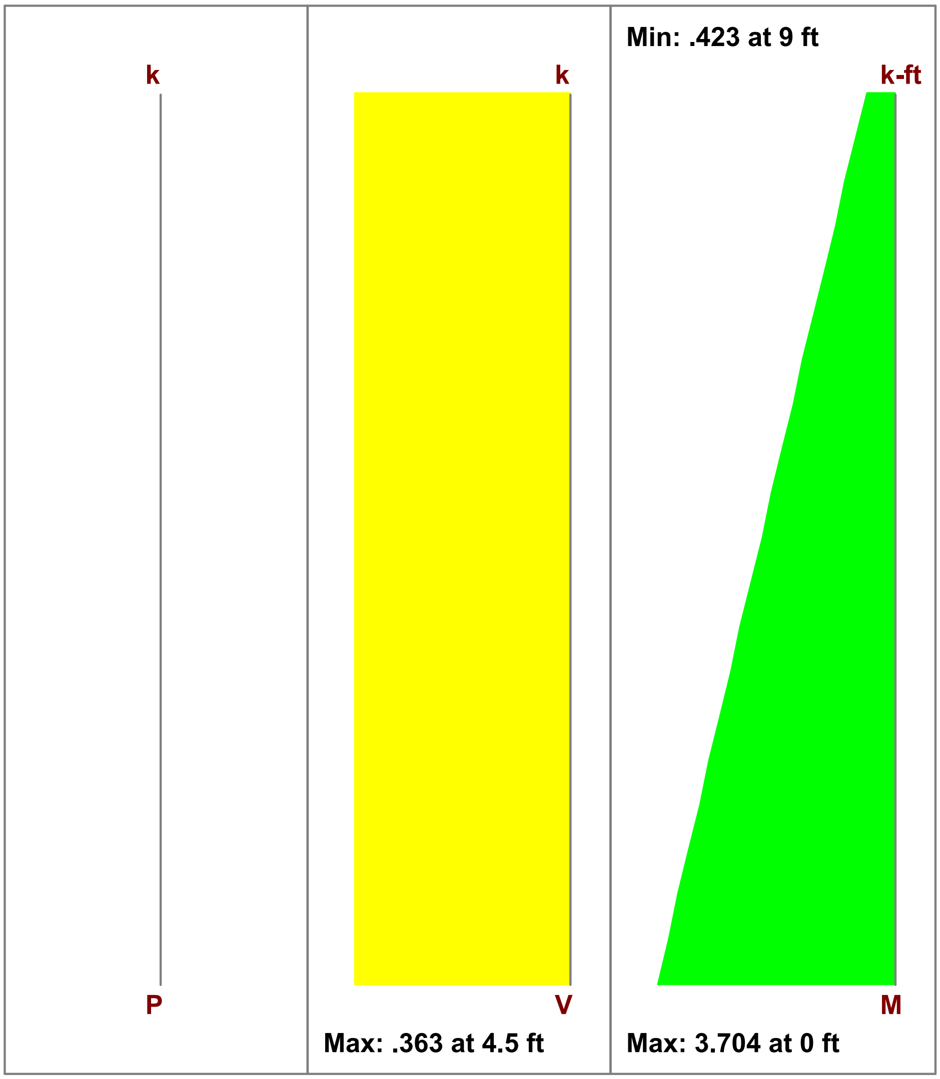
SELECTED HOLD-DOWN : None required

CROSS SECTION DETAILING



CRITERIA		MATERIALS		GEOMETRY	
Code	: **AWC NDS-15:ASD**	Wall Studs	: **Spruce-Pine-Fir**	Total Height	: **9 ft**
		Stud Size	: **2X6**	Total Length	: **7.983 ft**
Wall Material	: **Spruce-Pine-Fir**	Chord Material	: **Spruce-Pine-Fir**	Region H/W	: **1.13**
Panel Schedule	: **User Selected**	Chord Size	: **2-2X6**	Cap. Adj. (2w/h)	: **1.00**
Optimize HD	: **Yes**	Top Pl & Sill	: **Spruce-Pine-Fir**	Stud Spacing	: **16 in**
HD Manufacturer	: **SIMPSON**	Top Pl Size	: **2-2X6**	K	: **1.00**
		Sill Pl Size	: **2X6**		

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL

Required Cap	: **.046 k/ft**
Provided Cap	: **.28 k/ft**
Ratio	: **.163**
Governing LC	: **37 (Seismic)**

CHORDS

Max Comp Force	: **.572 k**
Comp Capacity	: **16.415 k**
Comp Ratio	: **.035**
Gov Comp LC	: **39**
Max Tens Force	: **.294 k**
Tens Capacity	: **15.444 k**
Tens Ratio	: **.019**
Gov Tens LC	: **51**

STUDS

No gravity-only LC solved.

DEFLECTIONS

Flexure Comp	: **.017 in**
Shear Comp	: **.037 in**
HD Elong	: **0 in**
Tot Deflection	: **.054 in**
Governing LC	: **37**

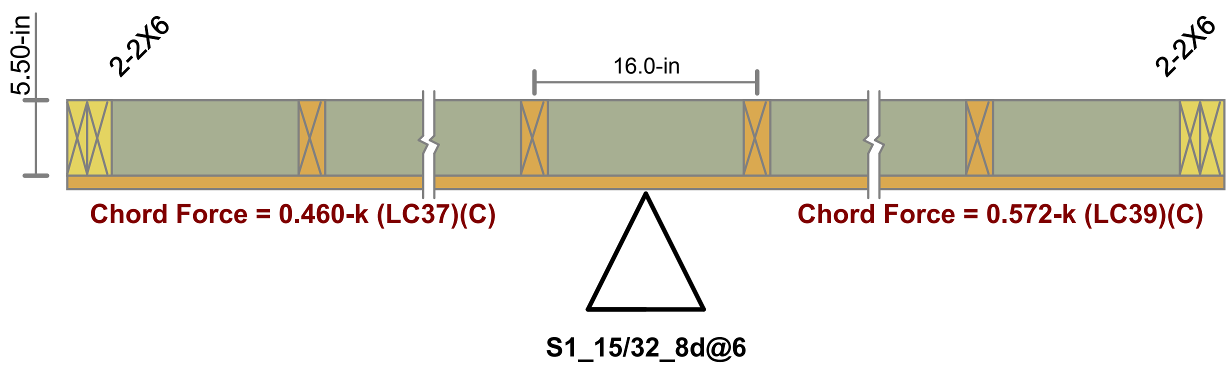
DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@6

Panel Grade	: **St-I**		Nail Size	: **8d**		Num Sides	: **One**	
Panel Thick	: **0.469**	**in**	Reqd Pen	: **1.250**	**in**	Over Gyp Brd.	: **No**	
			Reqd. Spacing	: **6**	**in**	Shear Capacity	: **0.280**	**k/ft**
						Adjusted Cap	: **0.280**	**k/ft**

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

CROSS SECTION DETAILING



CRITERIA

Code : **AWC NDS-15:ASD**

Wall Material : **Spruce-Pine-Fir**
Panel Schedule : **User Selected**

Optimize HD : **Yes**
HD Manufacturer: **SIMPSON**

MATERIALS

Wall Studs : **Spruce-Pine-Fir**
Stud Size : **2X6**

Chord Material : **Spruce-Pine-Fir**
Chord Size : **2-2X6**

Top Pl & Sill : **Spruce-Pine-Fir**
Top Pl Size : **2-2X6**
Sill Pl Size : **2X6**

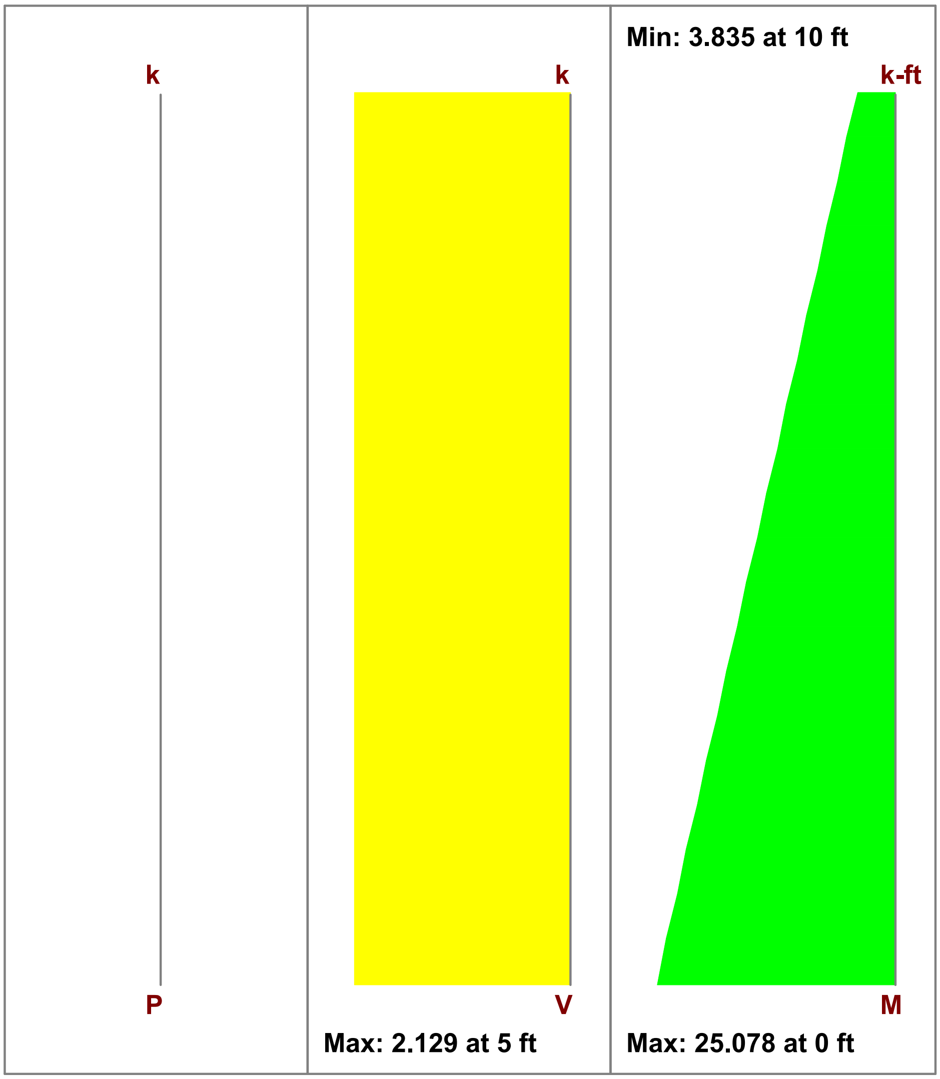
GEOMETRY

Total Height : **10 ft**
Total Length : **7.983 ft**

Region H/W : **1.25**
Cap. Adj. (2w/h) : **1.00**

Stud Spacing : **16 in**
K : **1.00**
HD Eccentricity : **4.313 in**

ENVELOPE DIAGRAMS



DESIGN SUMMARY

SHEAR PANEL
Required Cap : **.267 k/ft**
Provided Cap : **.43 k/ft**
Ratio : **.62**
Governing LC : **37 (Seismic)**

CHORDS
Max Comp Force: **3.486 k**
Comp Capacity : **12.066 k**
Comp Ratio : **.289**
Gov Comp LC : **39**
Max Tens Force : **2.801 k**
Tens Capacity : **13.514 k**
Tens Ratio : **.207**
Gov Tens LC : **51**

STUDS
No gravity-only LC solved.

HOLD-DOWNS
Required Cap : **2.889 k**
Provided Cap : **3.075 k**
Ratio : **.939**
Governing LC : **51**

DEFLECTIONS
Flexure Comp : **.139 in**
Shear Comp : **.19 in**
HD Elong : **.083 in**
Tot Deflection : **.413 in**
Governing LC : **37**

DESIGN DETAILS

SELECTED SHEAR PANEL : S1_15/32_8d@4

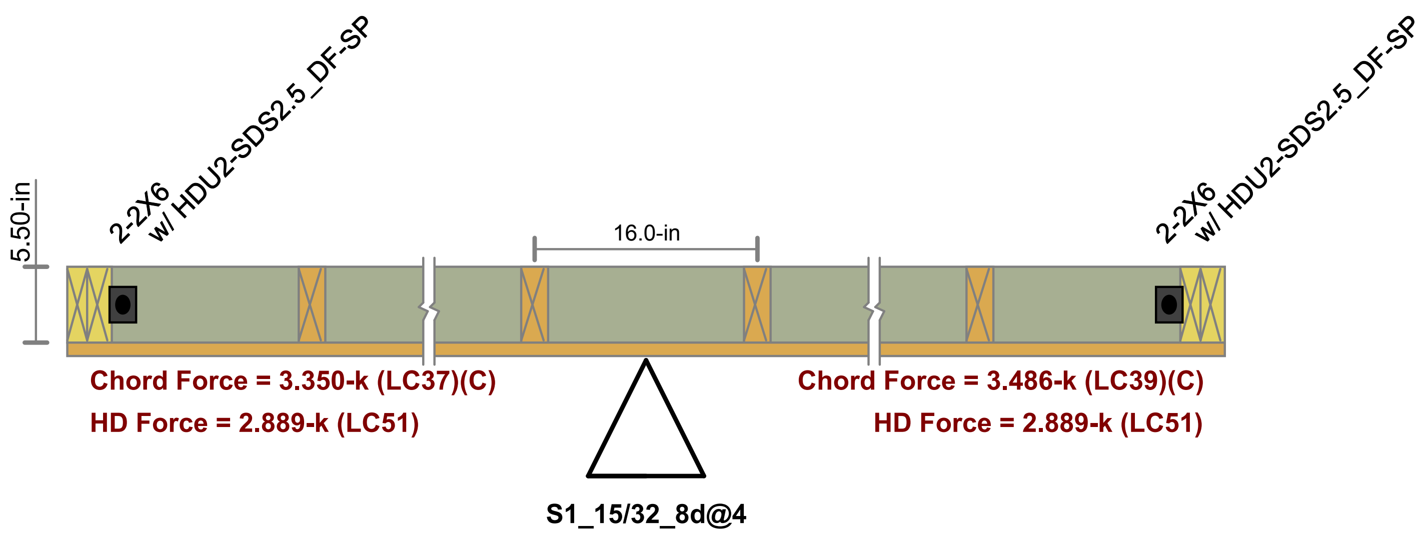
Panel Grade	: St-I		Nail Size	: 8d		Num Sides	: One	
Panel Thick	: 0.469	in	Reqd Pen	: 1.250	in	Over Gyp Brd.	: No	
			Reqd. Spacing	: 4	in	Shear Capacity	: 0.430	k/ft
						Adjusted Cap	: 0.430	k/ft

NOTE: AWC NDS-15 defines a 8d nail as being 2.5" x 0.1310" common, or 2.5" x 0.113" galvanized box

SELECTED HOLD-DOWN : HDU2-SDS2.5_DF-SP

Min Chord Thk	: 3.00	in	Bolt Size:	: .625	in	Base Cap(CD=1):	1.922	k
Reqd Chord Mat	: Douglas Fir					CD factor	: 1.6	
						Adjusted Cap	: 3.075	k

CROSS SECTION DETAILING

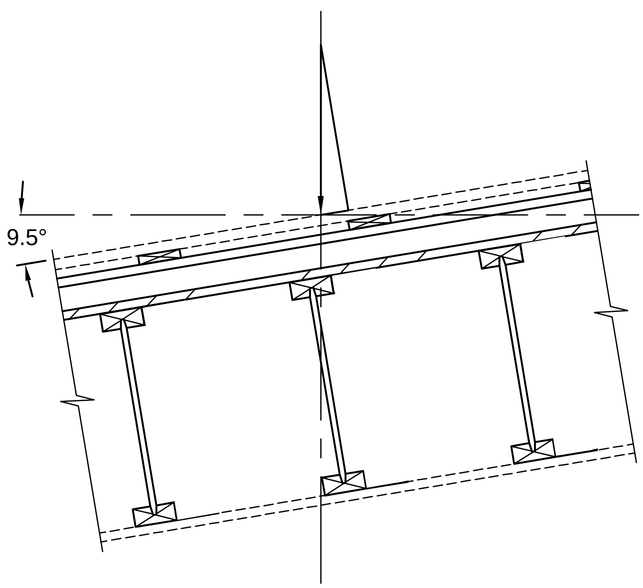


Blackwell
Structural Engineers

Oblique Angled I-Joist Framing Calculation

LOOK AT OBLIQUE ANGLED I-JOIST FRAMING

LOOK AT LATERAL COMPONENT



NORMAL TO SURFACE LOADS:

SNOW: 192psf x sin9.5 = 31.7 psf TANGENTIAL

DEAD: 25psf x sin9.5 = 4.1 psf TANGENTIAL

BUILDING WIDTH = 16'-10" THEREFORE
TRIBUTARY WIDTH FOR EACH GRID IS 8'-5"

ALONG GRIDLINES 3&4 DESIGN FOR ADDITIONAL
SPECIFIED LOAD OF SL=267plf DL=34.5plf

LOOK AT ROOF DIAPHRAGM

MAXIMUM SHEAR IN DIAPHRAGM FROM SEISMIC = 4.5kips (9.0/2)
DIAPHRAGM DEPTH IS AXIS OF CONSIDERATION = 55'-0"
UNIT SHEAR FROM SEISMIC = 0.081kips/ft

UNIT SHEAR IN DIAPHRAGM FROM SNOW = 267plf X (1.60/1.15) = 0.371kips/ft
UNIT SHEAR IN DIPAHRAGM FROM DEAD = 34.5plf X (1.60/1) = 0.055kips/ft

**1.60/1.15 ADDED TO REMOVE DURATION FACTOR FOR VERIFYING IN NDS DIAPHRAGM TABLES.

VERFIY COVERNING LOAD COMPINATION FROM ASD DESIGN:

CASE 3: D + S ----------------------------> UNIT SHEAR = 0.426kip/ft

CASE 6b: D + 0.75L + 0.75(0.7E) + 0.75(0.3S) -------> UNIT SHEAR = 0.181kip/ft (GOVERNS)

CASE 8: 0.6D + 0.7E ----------------------------> UNIT SHEAR = 0.090kip/ft

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LOOK AT BOTTOM CHORD.

- NO LOADING IS TO BE APPLIED DIRECTLY FROM BOTTOM CHORD.
- ON INSPECTION, BOTTOM CHORD IS INSUFFICIENT FOR WEAK AXIS BEANDING
- PROVIDE LOWER DIAPHRAGM FASTEN DIRECTLY TO JOISTS TO RESIST WEAK AXIS BENDING.

CEILING LOAD = 3psf
1/2 JOISTS WEIGHT = 4.9lb/ft x 1 JOIST x (1/1) SPACING = 4.9psf

$\sum$ = 7.9psf

NORMAL COMPONENT: 7.8psf (RESISTED BY JOISTS)

TANGENTIAL: 1.32psf (RESISTED BY LOWER DIAPHRAGM)

MAX SHEAR IN LOW DIAPHRAGM = SPAN/2 x FORCE = 11.11lbs/ft (ASD)

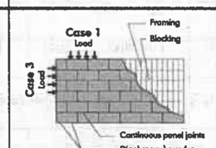
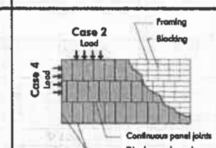
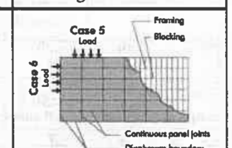
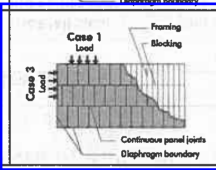
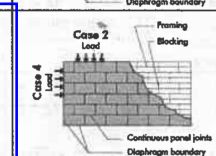
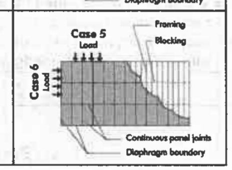
** ON INSPECTION, 5/8" GYPSUM CEILING IS SUFFICIENT TO RESIST LOAD.
** PROVIDE SOLID BLOCKING BETWEEN JOISTS @ 8" c/c FOR ADDITIONAL SUPPORT AND RIGIDITY.

Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Blocked Wood Structural Panel Diaphragms[1,2,3,4,5]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	A SEISMIC												B WIND			
					Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)												Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)			
					6			4			2-1/2			2			6	4	2-1/2	2
					Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)												Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)			
					6			6			4			3			6	6	4	3
					v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
						OSB	PLY		OSB	PLY		OSB	PLY		OSB	PLY				
Structural I	6d	1-1/4	5/16	2	370	15	12	500	8.5	7.5	750	12	10	840	20	15	520	700	1050	1175
				3	420	12	9.5	560	7.0	6.0	840	9.5	8.5	950	17	13	590	785	1175	1330
	8d	1-3/8	3/8	2	540	14	11	720	9.0	7.5	1060	13	10	1200	21	15	755	1010	1485	1680
				3	600	12	10	800	7.5	6.5	1200	10	9.0	1350	18	13	840	1120	1680	1890
	10d	1-1/2	15/32	2	640	24	17	850	15	12	1280	20	15	1460	31	21	895	1190	1790	2045
				3	720	20	15	960	12	9.5	1440	16	13	1640	26	18	1010	1345	2015	2295
Sheathing and Single-Floor	6d	1-1/4	5/16	2	340	15	10	450	9.0	7.0	670	13	9.5	760	21	13	475	630	940	1065
				3	380	12	9.0	500	7.0	6.0	760	10	8.0	860	17	12	530	700	1065	1205
			3/8	2	370	13	9.5	500	7.0	6.0	750	10	8.0	840	18	12	520	700	1050	1175
				3	420	10	8.0	560	5.5	5.0	840	8.5	7.0	950	14	10	590	785	1175	1330
	8d	1-3/8	3/8	2	480	15	11	640	9.5	7.5	960	13	9.5	1090	21	13	670	895	1345	1525
				3	540	12	9.5	720	7.5	6.0	1080	11	8.5	1220	18	12	755	1010	1510	1710
			7/16	2	510	14	10	680	8.5	7.0	1010	12	9.5	1150	20	13	715	950	1415	1610
				3	570	11	9.0	760	7.0	6.0	1140	10	8.0	1290	17	12	800	1065	1595	1805
			15/32	2	540	13	9.5	720	7.5	6.5	1060	11	8.5	1200	19	13	755	1010	1485	1680
				3	600	10	8.5	800	6.0	5.5	1200	9.0	7.5	1350	15	11	840	1120	1680	1890
	10d	1-1/2	15/32	2	580	25	15	770	15	11	1150	21	14	1310	33	18	810	1080	1610	1835
				3	650	21	14	860	12	9.5	1300	17	12	1470	28	16	910	1205	1820	2060
			19/32	2	640	21	14	850	13	9.5	1280	18	12	1460	28	17	895	1190	1790	2045
				3	720	17	12	960	10	8.0	1440	14	11	1640	24	15	1010	1345	2015	2295

1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the *NDS* (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
3. Apparent shear stiffness values, G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Diaphragm resistance depends on the direction of continuous panel joints with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

	Cases 1&3: Continuous Panel Joints Perpendicular to Framing	Cases 2&4: Continuous Panel Joints Parallel to Framing	Cases 5&6: Continuous Panel Joints Perpendicular and Parallel to Framing
Long Panel Direction Perpendicular to Supports			
Long Panel Direction Parallel to Supports[a]			

(a) Panel span rating for out-of-plane loads may be lower than the span rating with the long panel direction perpendicular to supports (See Section 3.2.2 and Section 3.2.3)

STEEL CONNECTION DESIGN

Blackwell

Seal	Title	Project #	Date
	KIMMELMAN RESIDENCE	170266.	2018/01/10.
		Designer	Scale
			NTS.
		Checked by	Sheet #
			INDEX.

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Seal	Title	Project #	Date
	KIMMELMAN RESIDENCE	170266.	2018/01/10.
		Designer	Scale
		DMV	NTS.
		Checked by	Sheet #
	GENERAL PROJECT INFORMATION.		A-01.

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PROBLEM: WE ARE REQUIRED TO DESIGN CONNECTIONS FOR BRACES. AVB + BG HAVE PROVIDEDED THE BRACE SIZES + FORCES WHICH WILL BE USED FOR THE DESIGN. USE THE LFRD DESIGN PROCEDURE USING AN OVER-STRENGTH FACTOR (Ω) = 2.0. THIS HAS ALREADY BEEN INCORPORATED IN THE DESIGN FORCES OUTLINED ON THE ELEVATIONS. FOR THE DESIGN OF BRACE CONNECTIONS, WE ARE REQUIRED TO INCORPORATE

AISC 341 - SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS.

AISC 358. - PREQUALIFIED CONNECTIONS.

AISC 360 - SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS.

*NOTE, PER AISC 341 D2.2(b)

↳ ONLY WELDED OR BOLTED CONNECTIONS ARE PERMISSIBLE.

↳ DUE TO LAYOUT AND ABILITY OF CONNECTING ELEMENTS, USE ALL WELDS FOR CONNECTIONS.

* MATERIAL INFORMATION:

→ PLATE $F_y = 36$ ksi ; $F_u = 58$ ksi

→ BEAMS. $F_y = 50$ ksi ; $F_u = 65$ ksi

→ COLUMNS $F_y = 46$ ksi ;

→ WELDS E70xx

→ ANGLES $F_y = 36$ksi ; $F_u = 58$ksi

→ USE THE WHITEMORE SECTION (EFFECTIVE WIDTH).

→ NOMINAL WELD STRENGTHS

$$\phi R_N = 1.392\, D\ell$$

WELD SIZE	ϕR_N
3/16"	4.176 kip/in.
1/4"	5.568 kip/in.
5/16"	6.96 kip/in

* DUE TO THE REQUIREMENTS OF AISC 341, WELDS CANNOT BE USED IN COMBINATION W/ BOLTS. THUS, SPECIFY ALL CONNECTIONS AS WELDED.

→ GENERAL DESIGN CHECKS ARE PROVIDED IN SECTION B OF THIS PACKAGE AND THE ASSOCIATED GUSSET PL CHECKS/DESIGN IS PROVIDED IN SECTION C. RISA CONNECTION DESIGN WAS USED FOR THE DESIGN OF BRACES.

→ ALTHOUGH THERE ARE INSTANCES WHERE RISA DOES NOT HAVE AN EXACT FRAMING SCHEME AS OUTLINED IN THE DRAWINGS (BEAM OVER AN HSS COLUMN FOR EXAMPLE) THIS IS, BY INSPECTION, ACCEPTABLE BECAUSE THE DESIGN IS FOR CONNECTION TO GENERAL FRAMING.

→ FOR THE DESIGN OF COLUMN, BEAM CONNECTIONS INCLUDING STIFFENERS, REFERENCE TYP. DETAILS. IF ADDITIONAL CONNECTION REQUIREMENTS ARE REQ'D, NOTE THOSE ON THE BRACING CONNECTION DETAILS.

→ THE FOLLOWING SHEET PROVIDES THE GENERAL CONNECTION INFORMATION.

Brace Information				Top Connection					Bottom Connection				
ID	Member	Force	Angle	Beam ID	Section	Column	Beam Reaction	Column Force	Beam ID	Section	Column	Beam Reaction	Column Force
X-1	L2x2x1/4	25	13	RB21	W12x26	HSS4x4x1/4	7.5	7.5	2B16	W10x26	HSS4x4x1/4	22	30
X-2	L2x2x1/4	34	13 / 14	RB24	W12x26	HSS4x4x3/8	30	35	2B15	W10x45	HSS4x4x3/8	16	45
X-3	L5x3-1/2x1/2	102.5	10	RB11	W12x26	HSS5x5x1/2	5	5	1B11	W14x61	HSS5x5x1/2	70	120
X-4	L3x2-1/2x1/2	41	27	RB16	W12x26	HSS4x4x5/16	1	2	-	-	HSS4x4x5/16	-	2
X-5	L5x3-1/2x1/2	20.1	12	RB1	W12x22	HSS4x4x3/8	2	5	1B8	W14x68	HSS4x4x3/8	27	35
X-7	L2x2x1/4	8.5	19	2B20	W8x18	HSS4x4x1/4	2	26	-	-	HSS4x4x1/4	2	26
BR6	HSS3x3x1/4	10	50	RB6	W12x22	HSS4x4x1/4	2	2	3B1	W8x18	HSS4x4x1/4	2	2
BR7	HSS3x3x1/4	8	54	2B22	W10x22	HSS4x4x1/4	2	3	-	-	HSS4x4x1/4	-	4
BR8	HSS3x3x1/4	25	54	2B23	W10x22	HSS5x5x5/16	12.5	45.5	-	-	HSS4x4x1/4	-	66.5
BR9	HSS3x3x1/4	28	39	1B2	W12x26	HSS4x4x1/4	16.5	37	-	-	HSS4x4x1/4	-	28.5
BR10	HSS3x3x1/4	28	35	1B7	W12x26	HSS4x4x1/4	2	2	-	-	HSS4x4x1/4	2	38
BR11	HSS3x3x1/4	11	32	1B10	W12x26	HSS4x4x1/4	2	2	-	-	HSS4x4x1/4	-	65
BR12	HSS3x3x1/4	13	49	3B1	W8x18	HSS4x4x1/4	2	2	-	-	HSS4x4x1/4	-	4
BR14	HSS3x3x1/4	6		1B18	W12x26	HSS4x4x1/4	2	16.5	-	-	HSS5x5x1/2	-	53.5

Seal	Title	Project #	Date
Blackwell	KIMMELMAN RESIDENCE TYPICAL BRACING DETAILS.	170266	2018/01/16
		Designer: DMV	Scale: NTS
		Checked by:	Sheet #: B-01

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PROBLEM: UPON REVIEW OF THE BRACING ELEVATIONS, THERE ARE A TOTAL OF 3-TYPES OF BRACES (DIAGONAL HSS; DIAGONAL ANGLE; X-BRACE ANGLE). DUE TO SIMILAR REQUIREMENTS, GENERATE TYPICAL DESIGN CHECKS FOR CONNECTIONS.

SUMMARY OF BRACES: *REFER ALSO TO SHEET A-03.

BRACE ID.	TYPE	MEMBER	FORCE [kip]	l_{WELD} *
X-1	X-BRACE	L2×2×1/4	25.0	2½"
X-2	X-BRACE	L2×2×1/4	34.0	3½"
X-3	X-BRACE	L5×3½×½	102.5	8" * 5/16" WELD.
X-4	X-BRACE	L3×2½×½	41.0	5"
X-5	X-BRACE	L5×3½×½	20.1	2½"
X-6	FLOOR BRACE	3/4" ϕ ROD.	6.0	—
X-7	X-BRACE	L2×2×1/4.	8.5	2"
BR-6	DIAGONAL	HSS 3×3×1/4	10.0	—
BR-7	DIAGONAL	HSS 3×3×1/4	8.0	—
BR-8	DIAGONAL	HSS3×3×1/4	25.0	—
BR-9	DIAGONAL	HSS 3×3×1/4	28.0	—
BR-10	DIAGONAL	HSS3×3×1/4	28.0	—
BR-11	DIAGONAL	HSS3×3×1/4	11.0	—
BR-12	DIAGONAL	HSS3×3×1/4	13.0	—
BR-14.	DIAGONAL	HSS3×3×1/4	6.0	—

* REFER TO SHEET B-05

→ FROM THE ABOVE TABLE, DESIGN CONNECTIONS FOR:

DIAGONAL BRACES: HSS3×3×1/4 → $C_f / T_f = 28.0$ kip.

X-BRACE: L2×2×1/4 → $C_f / T_f = 34.0$ kip.

*NOTE, A FEW GENERAL CHECKS WILL BE PROVIDED IN THE FOLLOWING SHEETS. REFER TO SECTION C FOR DETAILED RISA DESIGN/CHECKS.

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Seal	Title	Project #	Date
	KIMMELMAN RESIDENCE	170266.	2018/01/16.
		Designer: DMV	Scale: NTS.
	TYPICAL BRACING DETAILS.	Checked by:	Sheet #: B-02.

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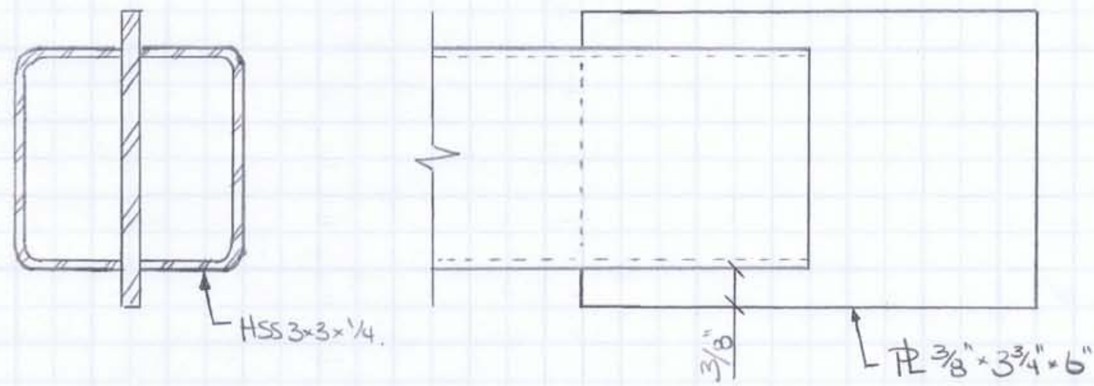
DIAGONAL BRACES.

FOR EASE OF ERECTION / FABRICATION, USE AN INTERNAL PLATE WELDED INTO SLOTS:



→ INTERNAL PLATE CHECKS.

• TENSION / COMPRESSION CAPACITY:

$$b/t = \frac{3.75\,in.}{0.375\,in} = 10.0 < 0.45\sqrt{\frac{E}{F_y}} \quad /\underline{OK}$$

$$T_N = \phi A_g F_y = (0.90)(3.75\,in \times 0.375\,in)(36\,ksi)$$

$$T_N = 45.5\ kip > 28.0\ kip \quad /\underline{OK}$$

$$T_N = \phi_u A F_u = (0.75)(3.75\,in \times 0.375\,in)(58\,ksi)$$

$$T_N = 61.2\ kip > 28.0\ kip. \quad /\underline{OK}$$

$$C_N = \phi F_{cr} A = (0.90)(3.75\,in \times 0.375\,in)(32\,ksi)$$

$$C_N = 40.7\ kip > 28.0\ kip. \quad /\underline{OK}$$

$$\frac{kL}{r} = 46.1 < 4.71\sqrt{E/F_y}$$

$$\rightarrow F_{cr} = 32.2\,ksi$$

∴ USE PL 3/8"×3 3/4"×6"

→ THIS IS TYPICAL. USE LARGER AS REQ'D FOR GEOMETRY. SEE RISA DESIGN LOCATED IN SECTION C.

→ PLATE CONNECTION TO HSS

- THERE WILL BE A TOTAL OF 4 WELDS. FOR $C_f/T_f = 28.0$ kip → 7.0 kip/WELD.
- BY USING 3" WELD LENGTH → $v_f = 2.33$ kip/in
- REFER TO SHEET A-02, USING 3/16 FILLET WELD → $v_r = 4.176$ kip/in /OK
- HSS SHEAR RUPTURE @ WELDS

$$t_{min} = \frac{3.09\,D}{F_u} = \frac{3.09(3)}{58\,ksi}$$

$$t_{min} = 0.160\,in < 0.233\,in \quad /OK$$

- PLATE RUPTURE STRENGTH @ WELDS

$$t_{min} = \frac{6.19\,D}{F_u} = \frac{6.19(3)}{58\,ksi}$$

$$t_{min} = 0.320\,in < 3/8" \quad /OK$$

- SHEAR LAG (TENSILE RUPTURE OF BRACE)

$$A_e = A_n U$$

$$A_e = (2.23\,in^2)(0.625)$$

$$A_e = 1.40\,in^2$$

$$P_n = \phi A_e F_u = (0.75)(1.40\,in^2)(58\,ksi)$$

$$P_N = 60.9\,kip > 28.0\,kip \quad /OK$$

$$A_n = 2.44\,in^2 - 2(0.233\,in)(0.375\,in + 1/16\,in)$$

$$\rightarrow A_n = 2.23\,in^2$$

$$\bar{X} = \frac{B^2 + 2BH}{4(B+H)} = \frac{(3in)^2 + 2(3in)(3in)}{4(3in + 3in)}$$

$$\bar{X} = 1.125\,in$$

$$U = 1 - \bar{X}/L = 1 - \frac{1.125\,in}{3\,in}$$

$$U = 0.625$$

Seal	Title	Project #	Date
	KIMMELMAN RESIDENCE	170266.	2018/01/19.
		Designer: DMN	Scale: NTS.
	TYPICAL BRACING DETAILS.	Checked by	Sheet #: B-04.

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→ PLATE CONNECTION TO GUSSET

- PROVIDE WELDS ON 3 SIDES FOR THE CONNECTION TO THE GUSSET PL
- TOTAL AVAILABLE WELD LENGTH = $l = 2(2\frac{1}{2}") + 3.75\,in. = 8.75\,in.$
- RECALL, $C_f / T_f = 28.0$ kip. ⟶ $v_f = 3.20$ kip/in

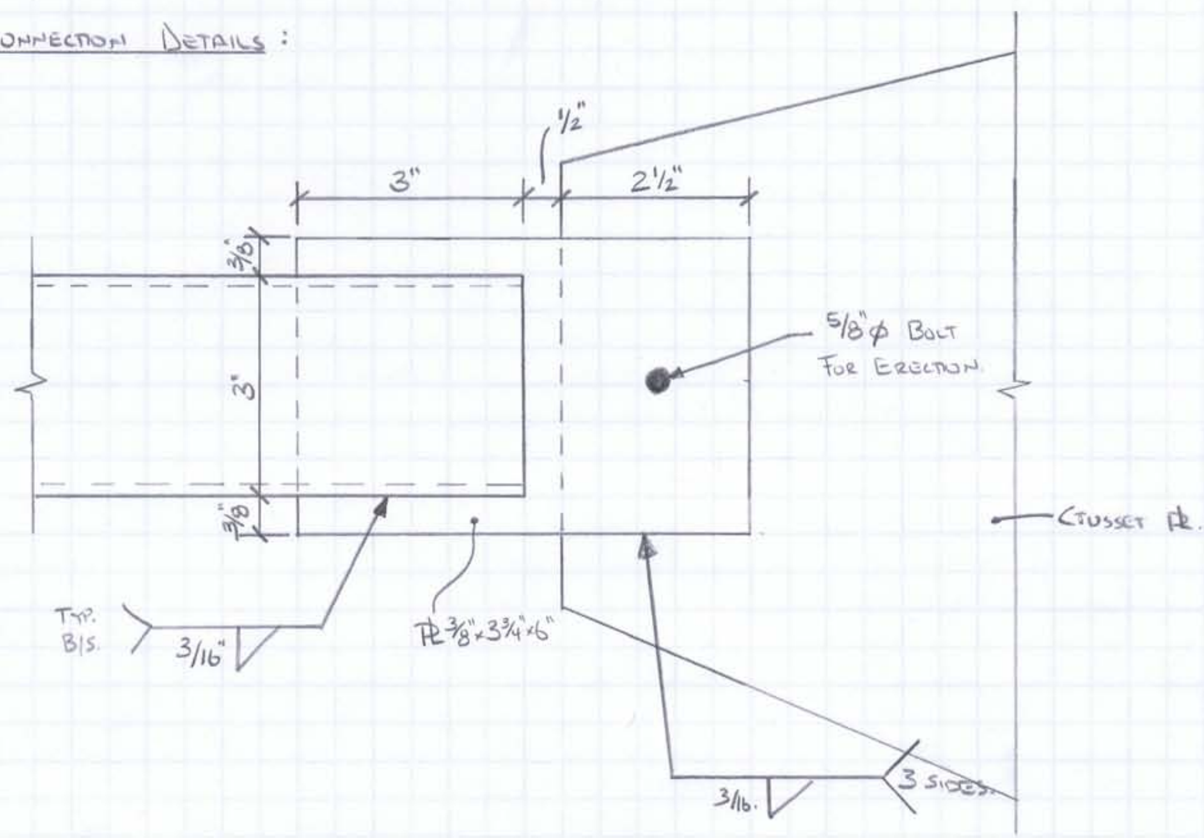
∴ USE 3/16" WELDS FOR PL TO GUSSET.

- SHEAR LAG.

FROM TABLE 3.1 IN AISC, $l = 2\frac{1}{2}" < w \Rightarrow$ CASE 1 w/ $U = 1.0$.

∴ NO SHEAR-LAG EFFECTS TO CONSIDER.

CONNECTION DETAILS :



X-BRACING

THE CROSS-BRACING (X-BRACING) WILL BE WELDED TO GUSSET PL'S. THE SUMMARY TABLE ON SHEET B-01 OUTLINES THE FORCES + MEMBERS.

→ THE L2×2×1/4 : $C_f / T_f = 34.0$ kip.
→ THE L5×3½×1/2 : $C_f / T_f = 102.5$ kip.
→ THE L5×2½×1/2 : $C_f / T_f = 41.0$ kip.

→ L2×2×1/4

- FOR 1/4" FILLET WELD → $v_r = 5.568$ kip/in.

  → $l_{WELD} \geq 6.10$ in.

  → USE (2) - 3½" LG. WELDS + PROVIDE AN END WELD (3-SIDES) TYP.

- SHEAR LAG

$$P_N = \phi A_e F_u = (0.75)(0.783 in^2)(58 ksi) \qquad A_n = A_g = 0.944 in^2$$

$$P_N = 34.0 \text{ kip} \quad /\underline{OK} \qquad U = 1 - \bar{x}/l = 1 - \frac{0.586 in}{3.5 in}$$

$$U = 0.83$$

$$\rightarrow A_e = UA_n = (0.83)(0.944 in^2)$$

$$A_e = 0.783 in^2$$

REFER TO NEXT SHEET FOR SUMMARY OF REMAINING X-BRACING CHECKS

*USE THIS INFO FOR BASIS OF GEOMETRY FOR RISA DESIGN.

Welds	ΦR_N
3/16	4.176 kip/in
1/4	5.568 kip/in
5/16	6.96 kip/in
F_{EXX}=70 ksi	

Brace ID	Member	Cf/Tf [kip]	Weld				Shear Lag					
			D [in]	L_{WELD} [in]	Vr [kip]	Status	y [in]	U	A_N [in^2]	A_E [in^2]	P_N [kip]	Status
X-1	L2x2x1/4	25	1/4	2.5	27.84	OK	0.586	0.77	0.94	0.72	31.44	OK
X-2	L2x2x1/4	34	1/4	3.5	38.98	OK	0.586	0.83	0.94	0.79	34.19	OK
X-3	L5x3-1/2x1/2	102.5	5/16	8	111.36	OK	0.901	0.89	4.00	3.55	154.40	OK
X-4	L3x2-1/2x1/2	41	1/4	5	55.68	OK	0.746	0.85	2.50	2.13	92.52	OK
X-5	L5x3-1/2x1/2	20.1	1/4	2.5	27.84	OK	0.901	0.64	4.00	2.56	111.29	OK
X-7	L2x2x1/4	8.5	1/4	2	22.27	OK	0.586	0.71	0.94	0.67	29.03	OK

Seal	Title	Project #	Date
	KIMMELMAN RESIDENCE	170266.	2018/01/16.
		Designer	**Scale**
		DMV	
		Checked by	**Sheet #**
	BRACING CONNECTIONS (RISA)		C-01

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THE FOLLOWING SHEETS ARE OUTPUTS FROM RISA CONNECTION DESIGN. THE PREVIOUS SECTIONS FORMED SOME BASIC DESIGN CHECKS TO AID IN GENERAL GEOMETRY OF CONNECTIONS, MIN. WELD REQUIREMENTS.

REFER TO A SUMMARY OF MEMBERS + DESIGN FORCES ON SHEET A-03

THE ABOVE REFERENCED RISA CONNECTION OUTPUT FILES ARE FOUND IN APPENDIX E

Blackwell

Seal	Title	Project #	Date
	KIMMELMAN RESIDENCE	170266	2018/02/01
		Designer	Scale
		DMV.	NTS
		Checked by	Sheet #
	FLOOR BRACE (X-BRACE 6)		D-01

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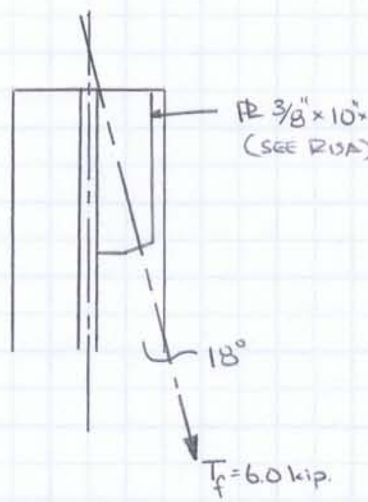
PROBLEM: THERE IS ONE ADDITIONAL CONNECTION THAT MUST BE DESIGNED. X-BRACE 6 IS LOCATED WITHIN THE FLOOR. THE BRACE IS A 3/4" ϕ ROD AND HAS A FACTORED TENSION FORCE OF 6.0 KIP. A GENERAL DESIGN REVIEW WAS COMPLETED USING RISA (ASSUMING A L2x2x1/4 ANGLE IN LIEU OF THE 3/4"ϕ ROD. ADDITIONAL DESIGN CHECKS ARE PROVIDED BELOW:

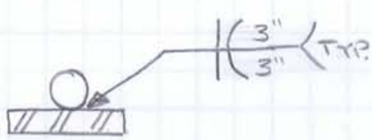


→ CHECK 3/4"ϕ ROD

$$\phi T_N = 0.90\left[\frac{\pi}{4}(0.75in)^2\right](36\,ksi)$$

$$\phi T_N = 14.30 \text{ kip} \quad /\underline{OK}$$

→ WELD OF 3/4"ϕ ROD TO GUSSET PL



$D = 5/16\ R$ [TABLE J2.2]

$$D = \left(\frac{5}{16}\right)\left(\frac{12}{16}\right) = 3.75/16$$

$$\phi R_N = 1.392(3.75)(3")(2)$$

$$\phi R_N = 31.0 \text{ kip} \quad /\underline{OK}$$

RISA

RISAConnection version 7.0.1

02/08/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

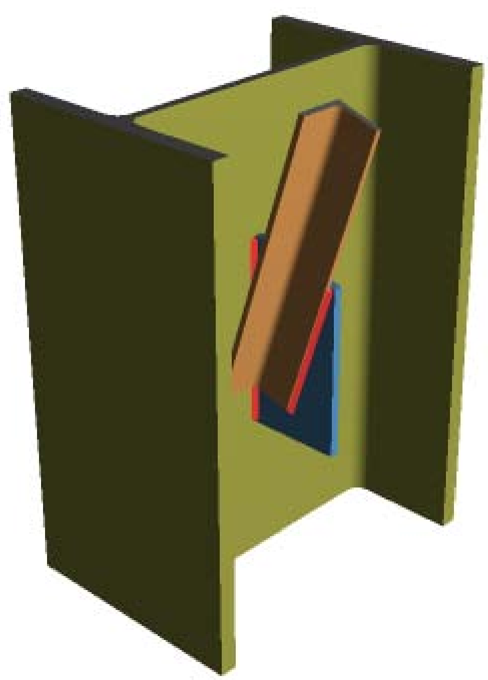
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

X-6 Detail	**PASS(UC-0.2)**

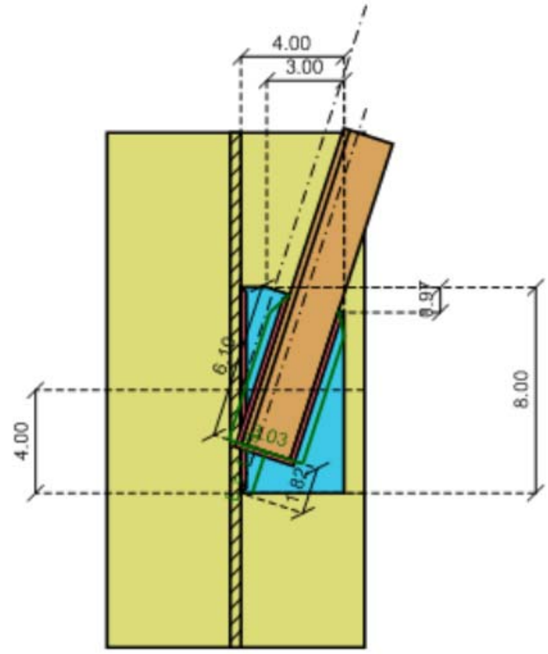
X-6 Detail: 3D View

Knee Brace Connection

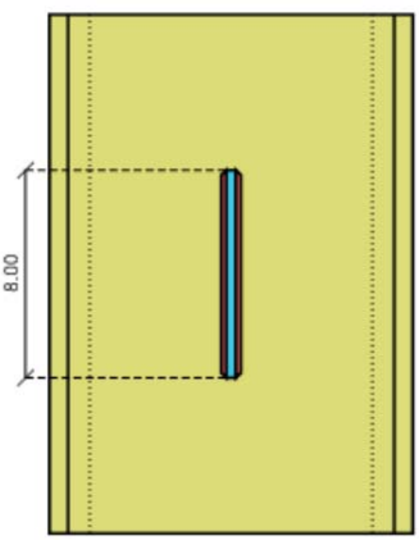


X-6 Detail: 2D Views

Left view

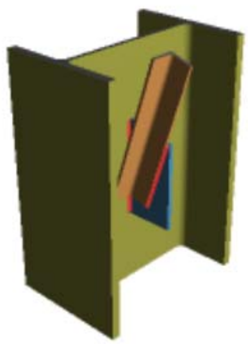


Front view



X-6 Detail: LRFD Results Report

Knee Brace Connection



Material Properties:

Column	W14x68	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x4.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	6.00 kips	*Brace Axial (compression)*
Shear Load	5.71 kips	*Calculated Shear Load*
Axial Load	1.85 kips	*Calculated Axial Load (compression)*
Moment Load	0.62 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Geometry Restrictions at Column					**PASS**
Check Workpoint Vert. Offset	**Pass**		*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	4.00 in		*Vertical brace workpoint offset*		
d_{gusset}	8.00 in		*Depth of gusset plate*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.25 in		*Max Size Allowed*		
t	0.25 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.25 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	6.10 in		*Max weld segment length*		
Plate Shear Yield		5.71 kips	64.80 kips	**0.09**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		
Plate Shear Rupture		5.71 kips	78.30 kips	**0.07**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.00 in^2		*Net area subject to shear*		
φR_n	78.30 kips		*Shear rupture strength*		

Plate Axial Yield		1.85 kips	97.20 kips	**0.02**	**PASS**
$R_n = F_y*A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	1.85 kips		*Calculated axial load*		
V_r	5.71 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.62 kips-ft		*Calculated moment*		
M_c	16.20 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.01**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	5.71 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.00 in^2		*Net area of the plate*		
Z_{net}	6.00 in^3		*Plastic modulus of net section*		
V_c	78.30 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.62 kips-ft		*Calculated moment*		
M_c	21.75 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		1.00 kips/in	6.68 kips/in	**0.15**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
V	5.71 kips		*Shear Load*		
P	1.85 kips		*Axial Load*		
M	0.62 kips-ft		*Moment*		
e_{eff}	1.30 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	1.00 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Column Web Yielding		1.85 kips	301.91 kips	**0.01**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		**φ = 1.00**	*(J10-2)*		
k	1.31 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	8.00 in		*Length of bearing*		

F_y	50.00 ksi		*Minimum yield stress of column*		
t_w	0.41 in		*Column web thickness*		
φR_n	301.91 kips		*Column web local yielding*		
Gusset Plate Compression (Whitmore)		6.00 kips	36.80 kips	**0.16**	**PASS**
$P_n = F_y * A_g$		$\varphi = 0.9$	*(J4-6)*		
K	1.20		*Effective length factor*		
L	1.69 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	18.71		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	1.14 in^2		*Gross area of plate (Whitmore section)*		
φP_n	36.80 kips		*Gusset plate compressive strength*		
Brace Weld Strength		6.00 kips	59.27 kips	**0.10**	**PASS**
$\varphi R_n = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	14.19 in		*Weld length*		
φR_n	59.27 kips		*Weld strength*		

X-6 Detail: Members Report

Knee Brace Connection

Column		***W14x68***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	10.00 in	*Flange width*
d	14.00 in	*Overall depth*
t_w	0.41 in	*Web thickness*
t_f	0.72 in	*Flange thickness*
a	20.00 in^2	*Area*
k_{des}	1.31 in	*Kdes*
k_{det}	1.56 in	*Kdet*
k_1	1.06 in	*K1*

Brace		***L2x2x4***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	2.00 in	*Flange width*
d	2.00 in	*Overall depth*
a	0.94 in^2	*Area*
t_{f1}	0.25 in	*Flange thickness*
t_{f2}	0.25 in	*Flange thickness*
k_{des}	0.50 in	*Kdes*

k_{det}	0.50 in	*Kdet*

X-6 Detail: Components Report

Knee Brace Connection

Gusset		***P0.38x4.00x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	3.00 in	*Horiz. Clip*
V_clip	0.97 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

DIAPHRAGM DESIGN

Blackwell

Title: Structural Panel Diaphragm Design Per AWC SDPWS

Master Created By: B. Guzar
Date: 2017.12.20
Master Last modified by:
Date:
Notes: Design of structural panel diphragms in accordance with American Wood Council (AWS) Special Design Provisions for Wind & Seismic (SDPWS) 2015 Edition.

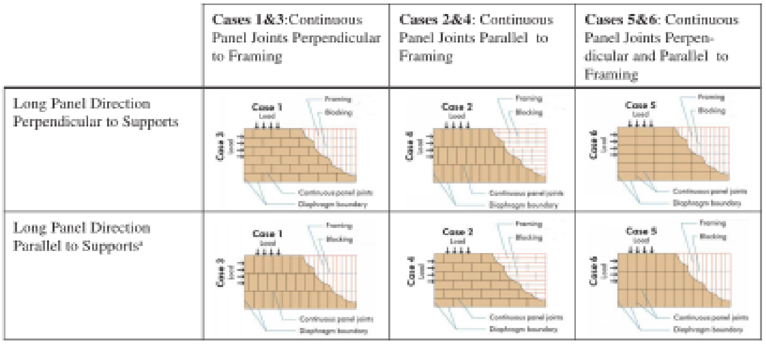
Project Name: Kimmelman May Residene
Project Number: 170266
Name: Adam van Bruinessen
Date: 2017.08.08

ROOF DIAPHRAGAM NOTES:

- BASED ON UPPER ROOF AT VOLUME 2. USE SAME NAILING FOR ALL ROOF DIAPHRAGMS.
- PROVIDE NOTE ON PLANS TO ORIENT SHEATHING SUCH THAT CASE1 LOADING IN SHORT DIRECTION
- PER 12.10.2.1 EXCEPTION 2, PORTIONS OF BUILDING SUPPORTED ENTIRELY BY WOOD SHEARWALLS NEED NOT BE DESIGNED FOR OVERSRENGTH (VOLUME 2 AND 3 IN N-S DIRECTION).
-DESIGN DIAPHRAGM EDGE NAIILING AND NAILER CONNECTION TO CHORD/DRAG MEMBERS FOR OCBF OVERSTRENGTH OF OCBF SYSTEM Ω = 2.0.
- GRID A' & E OF VOLUME 2 HAS WORST CASE SHEAR FLOW. ALL OTHER CHORDS TO BE DESIGNED FOR NEXT WORST CASE (VOLUME 1 - 383lb/ft)

APPLIED FORCES

ASD or LFRD?	LFRD		
Seismic or Wind?	Seismic		
	Strong Direction	**Weak Direction**	
Total Lateral Force	18.40	9.20	kips
Length of Diaphragm Edge	17.25	56.50	ft
Unit Shear Force in Diaphragm	533	81	lbs/ft



DIAPHRAGM SHEAR ADJUSTMENT FACTORS

Nominal Shear Adjustment Factor	0.8	*NDS SDPWS 4.2.3*
Species of Framing	Southern Pine	
Specific Gravity Adjustment Factor	1	*NDS Table 12.3.3A*

RESULTS

Filter Options:	
Blocking	Either
High Strength (multiple fastener lines)	Either
Panel Grade	Either
Panel Thickness	All
Nail Size	10d
Case	All
Total Number of Passing Diaphragms	462 of 516
Total Number of Failing Diaphragms	54 of 516
Number of Filtered Passing Results	1 of 462
Manual Diaphragm Slection	-----

Label (caseblocked_framing width_panel grade_panel thickness_nails@boundary spacing/ other spacing / nail lines)	Case	Blocked?	High Load Diaphragm?	Framing Width	Panel Grade	Panel Thickness (in)	Nail Size	Min Penetration (in)	Boundary/Cont Edge Spacing (in)	Other Edge Spacing (in)	Nail Lines	OVERALL CHECK	Vf/Vr %
C1/3B_2_S1_15/32_10d@6/6/1	1/3	Yes	No	2	Structural-I	15/32	10d	1.5	6	6	1	FULL PASS	67%

VOLUME 2 GRIDLINES A' & E

LINE LOAD SHEAR FORCE: (PER DIAPHRAGM DESIGN SPREADSHEET)

533lb/tf x 2.0 = 1.07 kip/ft

APPLICABLE ADJUSTMENT FACTORS FOR CONNECTION		
		Notes
Z'	____	*D-fir G=0.49 Z||lbs*
C_M	1.00	
C_t	1.00	
C_g	1.00	*d < 1/4"*
C_Δ	1.00	
C_{eg}	1.00	
C_{di}	1.10	*Diaphragm connections only*
C_{tn}	1.00	
K_F	3.32	
φ	0.65	
λ	1.00	

3/4" DIAPHRAGM SHEATHING TO 1 1/2" NAILER CAPACITY (CWN)(LFRD)				
Spacing (__" c/c)	6d	8d	10d	16d
Z' D-fir G=0.49 Z||lbs	71	87	102	117
12	169	207	242	278
11	184	225	264	303
10	202	248	291	333
9	225	275	323	370
8	253	310	363	417
7	289	354	415	476
6	337	413	484	555
5	404	496	2 ROWS 581	667
4	506	620	$\boxed{726}$	833
3	674	826	969	1111
2	1011	1239	1453	1666

VOLUME 2 GRIDLINES A' & E

LINE LOAD SHEAR FORCE: (PER DIAPHRAGM DESIGN SPREADSHEET)

533lb/tf x 2.0 = 1.07 kip/ft

APPLICABLE ADJUSTMENT FACTORS FOR CONNECTION		
		Notes
Z'	___	*D-fir G=0.49 Z* $_{\|\|lbs}$
C_M	1.00	
C_t	1.00	
C_g	1.00	*Linear force input. Each bolt is own group*
C_Δ	1.00	
C_{eg}	1.00	
C_{di}	1.00	
C_{tn}	1.00	
K_F	3.32	
ϕ	0.65	
λ	1.00	*Seismic Loading per table N3*

1 1/2" NAILER SHEAR CAPACITY/ft (LFRD)				
Spacing (__" c/c)	1/2" Bolts	5/8"Ø Bolts	3/4"Ø Bolts	7/8"Ø Bolts
Z' D-fir G=0.49 Z$_{\|\|lbs}$	580	720	860	1010
32	469	583	696	817
24	626	777	928	1090
12	1252	$\boxed{1554}$	1856	2180
9	1669	2072	2475	2906
8	1877	2331	2784	3269
6	2503	3108	3712	4359
Notes:	1. Verify all adjustment factors prior to using tables above. 2. For Siesmic chords/drags, ensure last bolt placed with minumum 7D			

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ALL OTHER ROOF (SFRS) BEAMS

LINE LOAD SHEAR FORCE: (PER DIAPHRAGM DESIGN SPREADSHEET)

183lb/tf x 2.0 = 0.366 kip/ft

APPLICABLE ADJUSTMENT FACTORS FOR CONNECTION		
		Notes
Z'	____	*D-fir G=0.49 Z||lbs*
C_M	1.00	
C_t	1.00	
C_g	1.00	*d < 1/4"*
C_Δ	1.00	
C_{eg}	1.00	
C_{di}	1.10	*Diaphragm connections only*
C_{tn}	1.00	
K_F	3.32	
ϕ	0.65	
λ	1.00	

3/4" DIAPHRAGM SHEATHING TO 1 1/2" NAILER CAPACITY (CWN)(LFRD)				
Spacing (__" c/c)	6d	8d	10d	16d
Z' D-fir G=0.49 Z||lbs	71	87	102	117
12	169	207	242	278
11	184	225	264	303
10	202	248	291	333
9	225	275	323	370
8	253	310	363	417
7	289	354	415	476
6	337	413	$\boxed{484}$	555
5	404	496	581	667
4	506	620	726	833
3	674	826	969	1111
2	1011	1239	1453	1666

ALL OTHER ROOF (SFRS) BEAMS

LINE LOAD SHEAR FORCE: (PER DIAPHRAGM DESIGN SPREADSHEET)

183lb/tf x 2.0 = 0.366 kip/ft

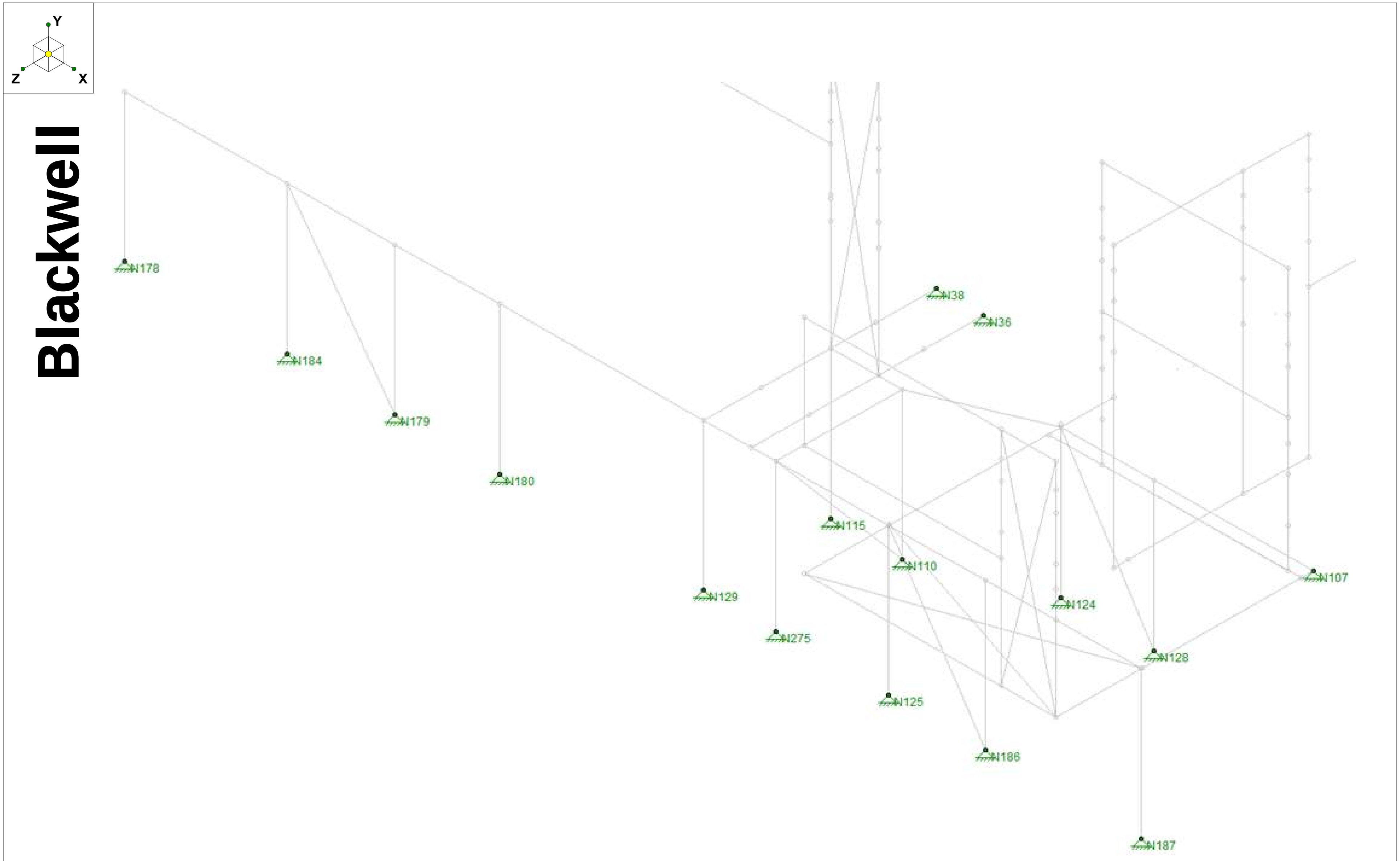
APPLICABLE ADJUSTMENT FACTORS FOR CONNECTION		
		Notes
Z'	___	*D-fir G=0.49 Z ||lbs*
C_M	1.00	
C_t	1.00	
C_g	1.00	*Linear force input. Each bolt is own group*
C_Δ	1.00	
C_{eg}	1.00	
C_{di}	1.00	
C_{tn}	1.00	
K_F	3.32	
ϕ	0.65	
λ	1.00	*Seismic Loading per table N3*

1 1/2" NAILER SHEAR CAPACITY/ft (LFRD)				
Spacing (__" c/c) Z' D-fir G=0.49 Z||lbs	1/2" Bolts 580	5/8"Ø Bolts 720	3/4"Ø Bolts 860	7/8"Ø Bolts 1010
32	469	583	696	817
24	626	$\boxed{777}$	928	1090
12	1252	1554	1856	2180
9	1669	2072	2475	2906
8	1877	2331	2784	3269
6	2503	3108	3712	4359
Notes:	1. Verify all adjustment factors prior to using tables above. 2. For Siesmic chords/drags, ensure last bolt placed with minumum 7D			

ANCHORAGE DESIGN

Blackwell
Structural Engineers

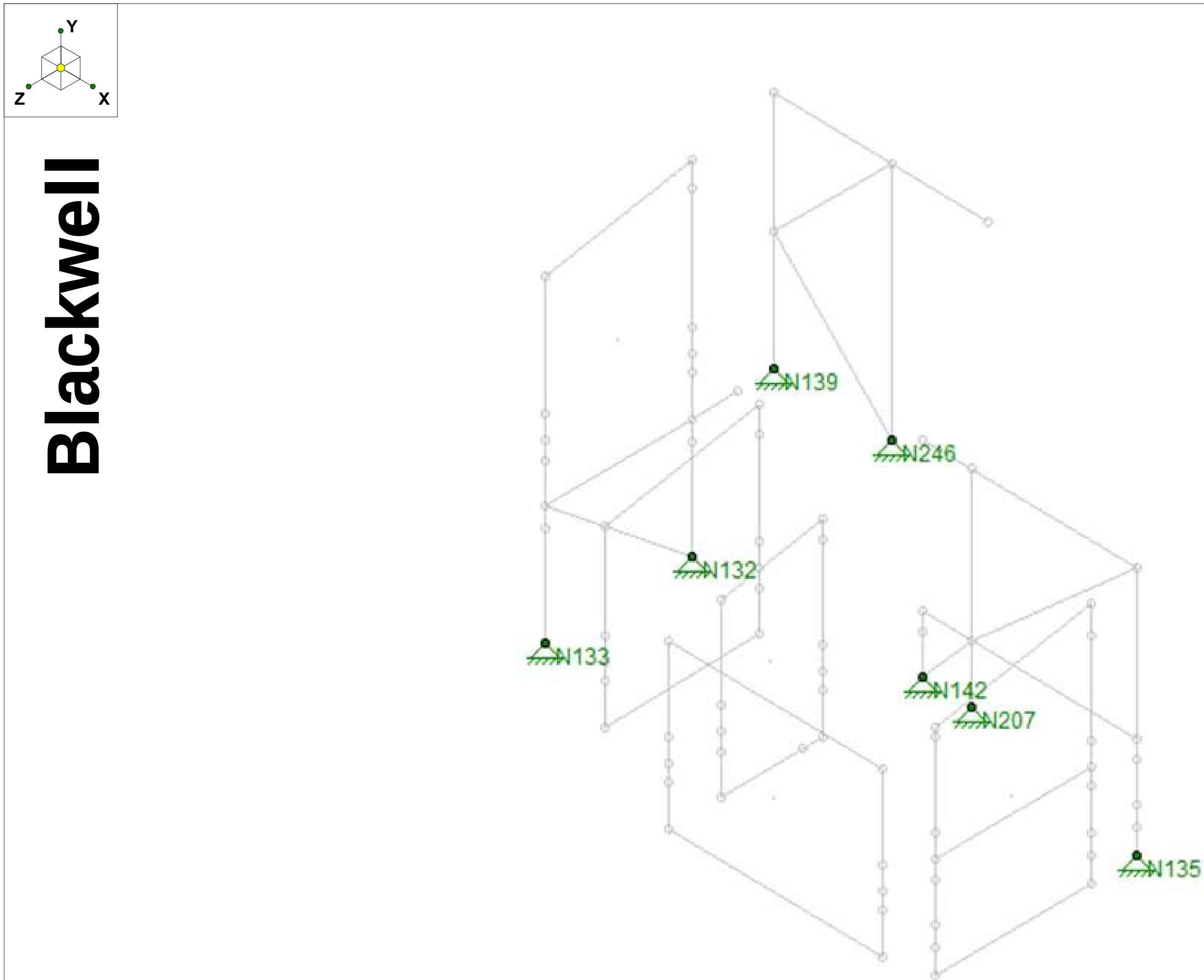
Anchorage Design Loads



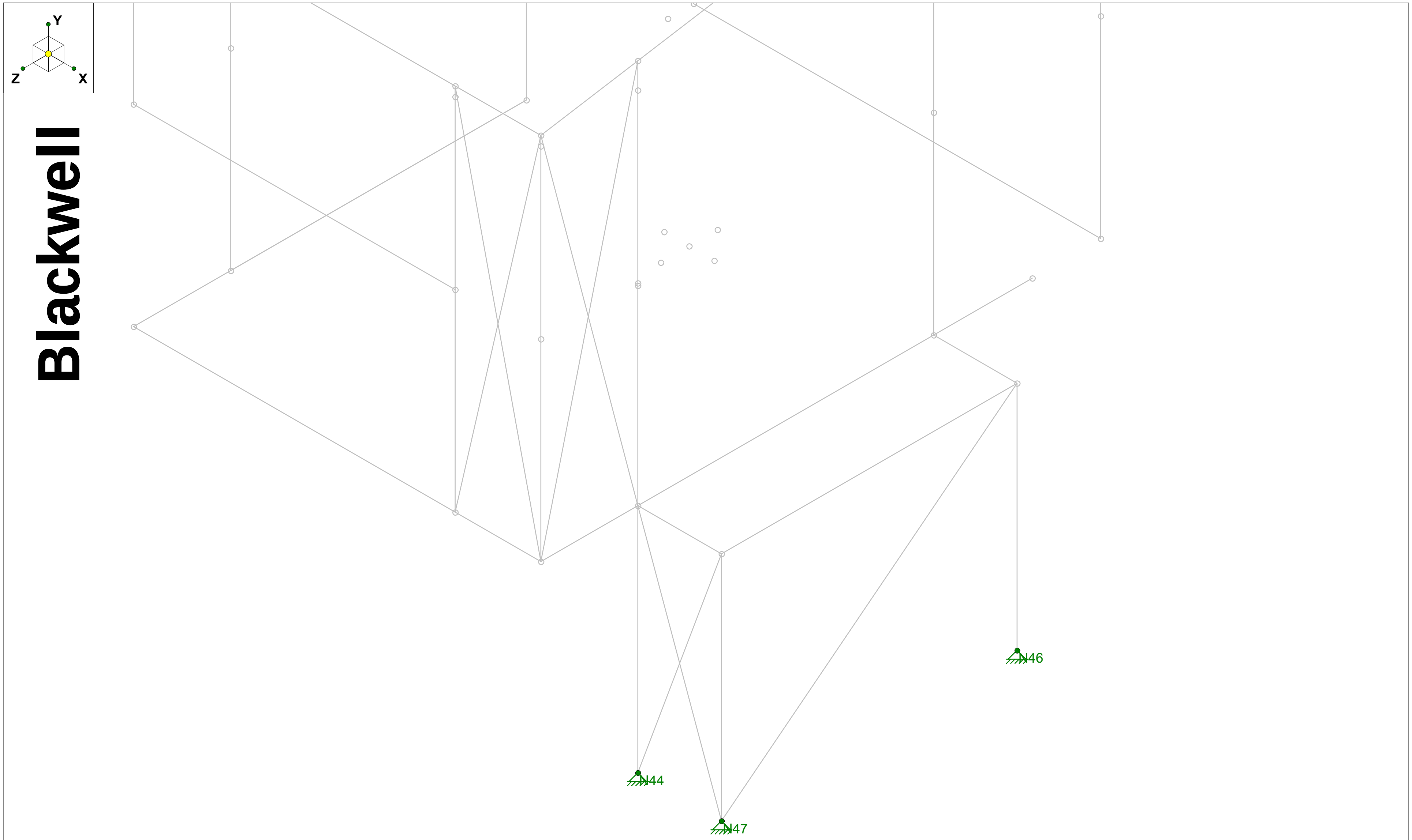
Blackwell

Blackwell Structural Engineers	KMR V2 V3 V4 Lateral	BASE PLATE NODES
BG	LATERAL SYSTEM AT SOUTH END OF RESIDENCE	
170266		

Blackwell



Blackwell Structural Engineers	KMR V2 V3 V4 Lateral	BASE PLATE NODES
BG	LATERAL STEEL SYSTEM AT NORTH END OF RESIDENCE AND SHEAR WALLS	
170266		



Blackwell

Blackwell Structural Engineers	Kimmelman May Residence Volume 1 Rev2	BASE PLATE NODES
BG		Nov 2, 2017 at 12:18 PM
170266		Volume 1 Rev 2 post county comments.rfl

Blackwell

Title: Anchorage Design Envelope Loads & Cases

Notes: Using LFRD and LFRD overstrength results

Project Name: Kimmelman May Residence
Project Number: 170266
Date: November 2, 2017

Volume 2, 3 and 4

Joint	X [k]	LC	Y [k]	LC	Z [k]	LC	Baseplate Configuration
N125	**-16.032**	**22**	**-21.237**	**22**	**-0.033**	**22**	A
N125	-10.275	13	-9.517	13	-0.014	13	A
N125	11.685	11	50.403	11	0.131	11	A
N125	**17.422**	**20**	**62.191**	**20**	**0.175**	**20**	A
N132	**0**	**31**	**-29.042**	**36**	**-20.885**	**32**	A
N132	0	8	-18.189	12	-13.567	8	A
N132	0	8	26.504	10	13.536	10	A
N132	**0**	**31**	**37.361**	**34**	**20.812**	**34**	A
N179	**-18.79**	**18**	**-21.263**	**24**	**0**	**18**	A
N179	-12.571	9	-12.668	15	0	9	A
N179	**11.126**	**15**	**25.176**	**9**	**0**	**15**	A
N179	17.334	24	33.854	18	0	24	A
N129	-0.014	11	0	8	0	9	B
N129	**-0.02**	**20**	**2.734**	**22**	**0**	**18**	B
N129	0.007	9	9.154	11	0	11	B
N129	**0.012**	**18**	**9.393**	**20**	**0**	**20**	B
N141	0	7	-4.793	14	-1.042	8	B
N180	-0.015	11	0	8	0	9	B
N180	**-0.022**	**20**	**4.092**	**22**	**0**	**18**	B
N180	0.008	9	10.624	11	0	15	B
N180	**0.015**	**18**	**10.624**	**20**	**0**	**24**	B
N187	-0.036	11	0	8	-0.019	15	B
N187	**-0.043**	**20**	**4.17**	**24**	**-0.054**	**24**	B
N187	0.028	9	34.14	9	0.148	9	B
N187	**0.054**	**18**	**37.288**	**18**	**0.184**	**18**	B
N207	**-0.001**	**33**	**0.721**	**53**	**0**	**33**	B
N207	0	9	1.147	29	0	9	B
N207	0.001	23	6.016	23	0	11	B
N207	**0.003**	**31**	**7.332**	**31**	**0**	**31**	B
N275	**-0.001**	**19**	**-1.454**	**24**	**0**	**22**	B
N275	-0.002	11	-0.475	15	0	13	B
N275	0.004	9	5.519	9	0	11	B
N275	**0.009**	**18**	**6.544**	**18**	**0.002**	**24**	B
N184	**-0.054**	**20**	**-21.768**	**22**	**0**	**24**	C
N184	-0.027	11	-13.113	13	0	15	C
N184	0	17	24.321	11	0	9	C
N184	**0**	**23**	**32.94**	**20**	**0**	**18**	C

Volume 2, 3 and 4

Joint	X [k]	LC	Y [k]	LC	Z [k]	LC	Baseplate Configuration
N186	**0**	**19**	**-23.948**	**24**	**-0.013**	**20**	C
N186	0	10	-15.364	15	-0.006	11	C
N186	0.017	13	16.762	9	0	10	C
N186	**0.041**	**20**	**25.308**	**18**	**0**	**19**	C
N110	**-0.001**	**19**	**-63.576**	**24**	**-2.234**	**22**	D
N110	-0.001	10	-39.726	15	-1.376	13	D
N110	0.021	9	54.287	9	2.152	11	D
N110	**0.047**	**18**	**77.956**	**18**	**2.986**	**20**	D
N115	**-0.262**	**24**	**-64.232**	**22**	**-0.004**	**22**	D
N115	-0.241	11	-40.054	13	-0.001	13	D
N115	0.277	9	54.738	11	0.017	11	D
N115	**0.409**	**18**	**79.038**	**20**	**0.021**	**20**	D
N142	**-6.946**	**31**	**-8.467**	**31**	**-0.007**	**33**	E
N142	-4.513	11	-5.413	11	-0.005	9	E
N142	4.513	9	4.686	13	0.003	11	E
N142	**6.942**	**33**	**7.405**	**37**	**0.008**	**31**	E
N139	**-21.002**	**31**	**-40.142**	**35**	**0**	**31**	G
N139	-13.621	11	-25.748	11	0	11	G
N139	0.548	24	29.449	9	0	24	G
N139	**0.548**	**48**	**43.858**	**33**	**0**	**48**	G
N246	**-0.548**	**48**	**-40.465**	**37**	**0**	**33**	G
N246	-0.548	24	-26.061	13	0	9	G
N246	13.626	9	27.437	11	0	19	G
N246	**21.007**	**33**	**42.593**	**31**	**0**	**43**	G
N107	-16.164	13	0	8	-1.337	11	H
N107	**-25.553**	**22**	**0.908**	**22**	**-1.999**	**20**	H
N107	20.607	11	4.195	11	1.179	13	H
N107	**29.843**	**20**	**4.421**	**20**	**1.851**	**22**	H
N133	**0**	**31**	**-29.452**	**38**	**0**	**51**	I
N133	0	8	-18.596	14	0	27	I
N133	0	8	24.579	8	0.019	8	I
N133	**0**	**31**	**35.434**	**32**	**0.042**	**32**	I
N124	-0.002	11	0	8	-0.004	9	J
N124	**-0.003**	**20**	**2.271**	**22**	**-0.006**	**18**	J
N124	0	9	15.244	11	0.008	11	J
N124	**0**	**18**	**16.26**	**20**	**0.013**	**20**	J
N128	**-5.47**	**18**	**-0.882**	**24**	**-0.007**	**18**	J
N128	-4.674	9	0	8	-0.004	9	J
N128	0.225	15	8.946	9	0.002	11	J
N128	**1.012**	**24**	**10.215**	**18**	**0.002**	**20**	J
N135	**-0.006**	**33**	**-4.401**	**37**	**-0.387**	**35**	K
N135	-0.004	9	-2.291	13	-0.252	11	K
N135	0.004	11	6.441	23	0.254	13	K
N135	**0.007**	**31**	**9.486**	**31**	**0.392**	**37**	K
N140	0	7	-4.793	12	-0.851	12	K
N178	-0.007	11	0	8	0	15	K
N178	**-0.01**	**20**	**1.935**	**23**	**0**	**24**	K
N178	0.004	9	5.354	11	0	9	K
N178	**0.007**	**18**	**5.354**	**20**	**0**	**18**	K

Volume 1 Joint	X [k]	LC	Y [k]	LC	Z [k]	LC	Baseplate Configuration
V1 N44	0.263	41	43.535	25	0.279	46	B
V1 N44	**0.465**	**81**	**33.666**	**98**	**0.447**	**86**	B
V1 N44	-0.281	43	0.836	80	-0.406	44	B
V1 N44	**-0.373**	**83**	**-1.802**	**88**	**-0.575**	**84**	B
V1 N46	**0.006**	**81**	**1.216**	**84**	**0.003**	**84**	B
V1 N46	0.004	41	1.214	44	0.003	28	B
V1 N46	-0.004	43	0.376	78	0	46	B
V1 N46	**-0.006**	**83**	**0.375**	**102**	**-0.001**	**86**	B
V1 N47	**0.013**	**81**	**1.224**	**82**	**0.001**	**86**	B
V1 N47	0.009	41	1.223	42	0	46	B
V1 N47	-0.009	43	0.379	80	-0.003	28	B
V1 N47	**-0.013**	**83**	**0.378**	**104**	**-0.003**	**84**	B

NOTE THAT BOLDED LINES ARE OVERSTRENGTH LOAD CASES

Anchorage Design

Design done using Hilti PROFIS software for all base plates shear wall hold downs and lateral force resisting beam connections. Results output from base plate D, H and I included in this report to minimize length.

BASE PLATE D

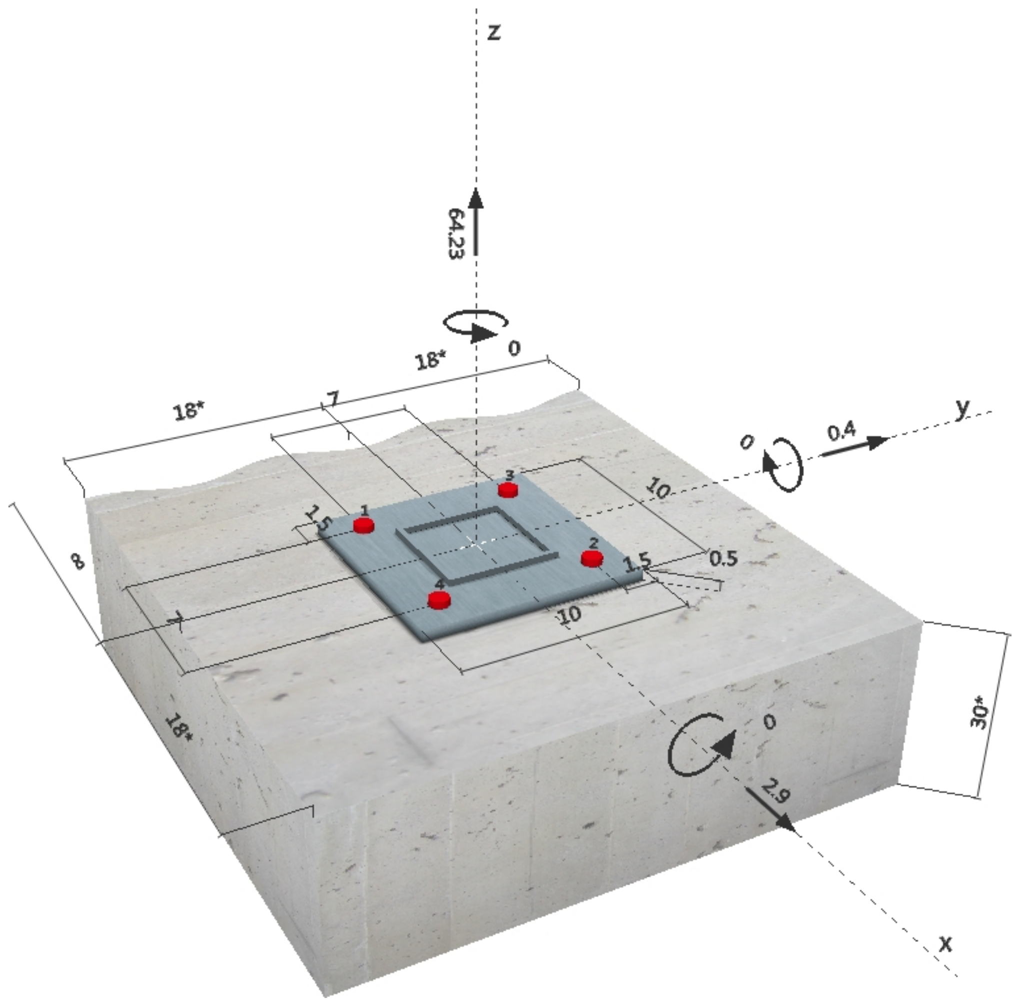
Company:	Blackwell Structural Engineers	Page:	1
Specifier:		Project:	Kimmelman May Res
Address:		Sub-Project I Pos. No.:	170266
Phone I Fax:	|	Date:	11/2/2017
E-Mail:			

Specifier's comments: Base Plate D

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 7/8**
Effective embedment depth:	h_{ef} = 24.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 10.000 in. x 10.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	Square HSS (AISC); (L x W x T) = 5.000 in. x 5.000 in. x 0.250 in.
Base material:	cracked concrete, , f_c' = 3500 psi; h = 30.000 in.
Reinforcement:	tension: condition A, shear: condition B; anchor reinforcement: tension
	edge reinforcement: > No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (b))
	Shear load: yes (17.2.3.5.3 (a))

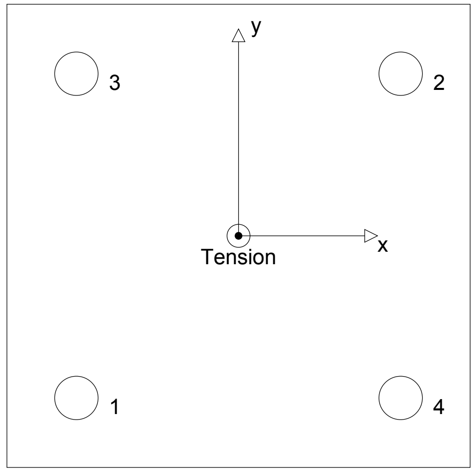
Geometry [in.] & Loading [kip, ft.kip]



Company:	Blackwell Structural Engineers	Page:	2
Specifier:		Project:	Kimmelman May Res
Address:		Sub-Project I Pos. No.:	170266
Phone I Fax:	|	Date:	11/2/2017
E-Mail:			

2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	16.058	0.732	0.725	0.100
2	16.058	0.732	0.725	0.100
3	16.058	0.732	0.725	0.100
4	16.058	0.732	0.725	0.100

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 64.230 [kip]
resulting compression force in (x/y)=(0.000/0.000): 0.000 [kip]

3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	16.058	20.097	80	OK
Pullout Strength*	16.058	17.464	92	OK
Concrete Breakout Strength**[1]	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)
[1] Tension Anchor Reinforcement has been selected!

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.46	58000

Calculations

N_{sa} [kip]
26.796

Results

N_{sa} [kip]	ϕ_{steel}	ϕ N_{sa} [kip]	N_{ua} [kip]
26.796	0.750	20.097	16.058

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-14 Eq. (17.4.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-14 Eq. (17.4.3.4)
$\phi N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.19	1.000	3500

Calculations

N_p [kip]
33.264

Results

N_{pn} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{pn} [kip]	N_{ua} [kip]
33.264	0.700	0.750	1.000	17.464	16.058

4 Shear load

	Load V_{ua} [kip]	Capacity ϕV_n [kip]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	0.732	10.450	8	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	2.927	92.066	4	OK
Concrete edge failure in direction x+**	2.927	18.393	16	OK

* anchor having the highest loading   **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.46	58000

Calculations

V_{sa} [kip]
16.078

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
16.078	0.650	10.450	0.732

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\, \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	9.667	0.000	0.000	14.500

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	3500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
1296.00	841.00	1.000	1.000	1.000	1.000	42.674

Results

V_{cpg} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [kip]	V_{ua} [kip]
131.522	0.700	1.000	1.000	92.066	2.927

4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$	ACI 318-14 Eq. (17.5.2.1b)
$\phi\ V_{cbg} \geq V_{ua}$	ACI 318-14 Table 17.3.1.1
A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)	
$A_{Vc0} = 4.5\ c_{a1}^2$	ACI 318-14 Eq. (17.5.2.1c)
$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$	ACI 318-14 Eq. (17.5.2.5)
$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$	ACI 318-14 Eq. (17.5.2.6b)
$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$	ACI 318-14 Eq. (17.5.2.8)
$V_b = 9\ \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$	ACI 318-14 Eq. (17.5.2.2b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
14.500	14.500	0.000	1.200	30.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
7.000	1.000	0.875	3500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
783.00	946.13	1.000	0.900	1.000	29.399

Results

V_{cbg} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cbg}$ [kip]	V_{ua} [kip]
26.276	0.700	1.000	1.000	18.393	2.927

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.919	0.159	1.000	90	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- The design of Anchor Reinforcement is beyond the scope of PROFIS Anchor. Refer to ACI 318-14, Section 17.4.2.9 for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Anchor calculations.

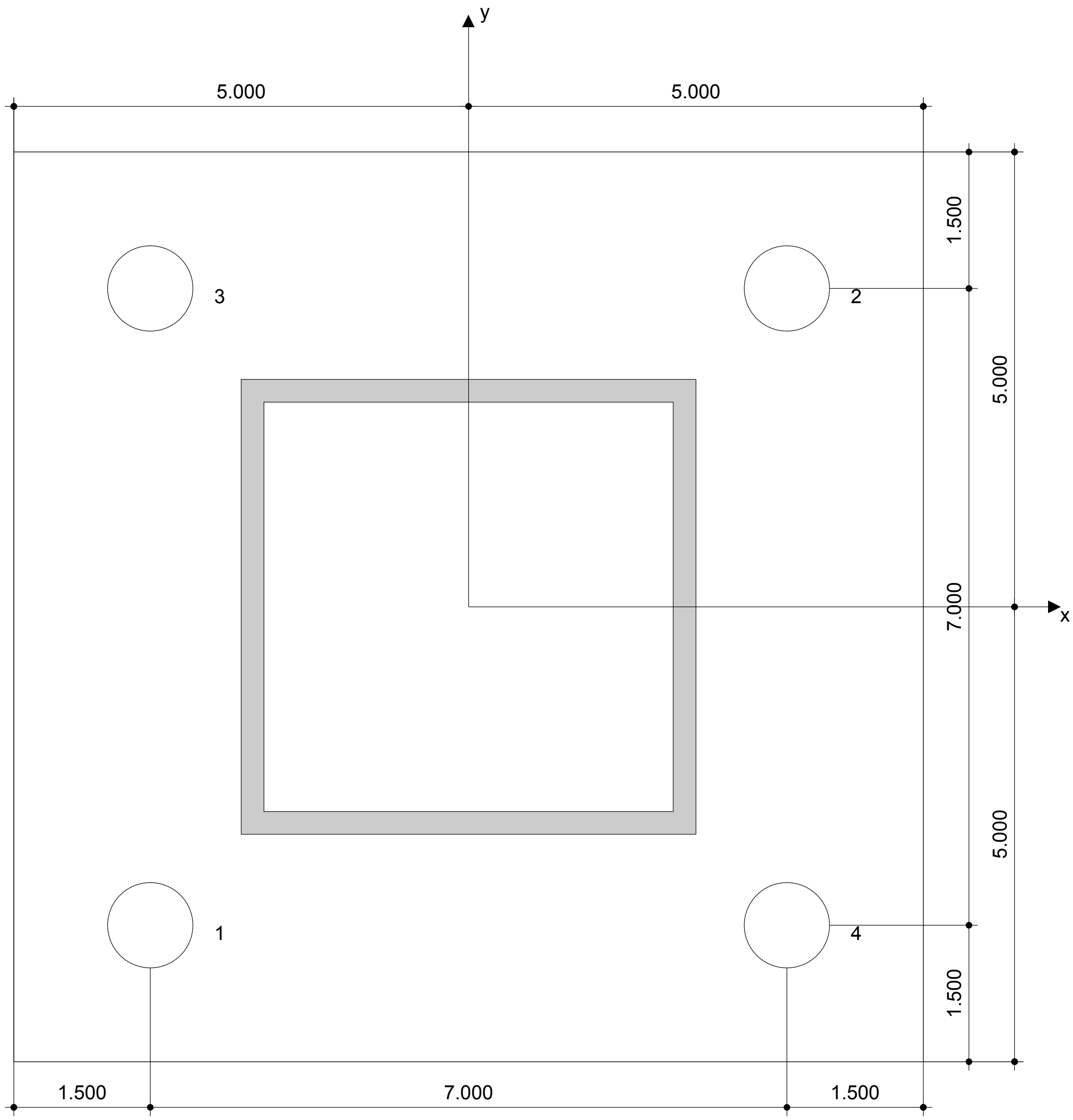
Fastening meets the design criteria!

Company:	Blackwell Structural Engineers	Page:	7
Specifier:		Project:	Kimmelman May Res
Address:		Sub-Project I Pos. No.:	170266
Phone I Fax:	|	Date:	11/2/2017
E-Mail:			

7 Installation data

Anchor plate, steel: -
Profile: Square HSS (AISC); 5.000 x 5.000 x 0.250 in.
Hole diameter in the fixture: d_f = 0.938 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Drilling method: -
Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 7/8
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 24.000 in.
Minimum thickness of the base material: 25.052 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.500	-3.500	-	21.500	14.500	21.500
2	3.500	3.500	-	14.500	21.500	14.500
3	-3.500	3.500	-	21.500	21.500	14.500
4	3.500	-3.500	-	14.500	14.500	21.500

Company:	Blackwell Structural Engineers	Page:	8
Specifier:		Project:	Kimmelman May Res
Address:		Sub-Project I Pos. No.:	170266
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E-Mail:			

8 Remarks; Your Cooperation Duties

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

BASE PLATE H

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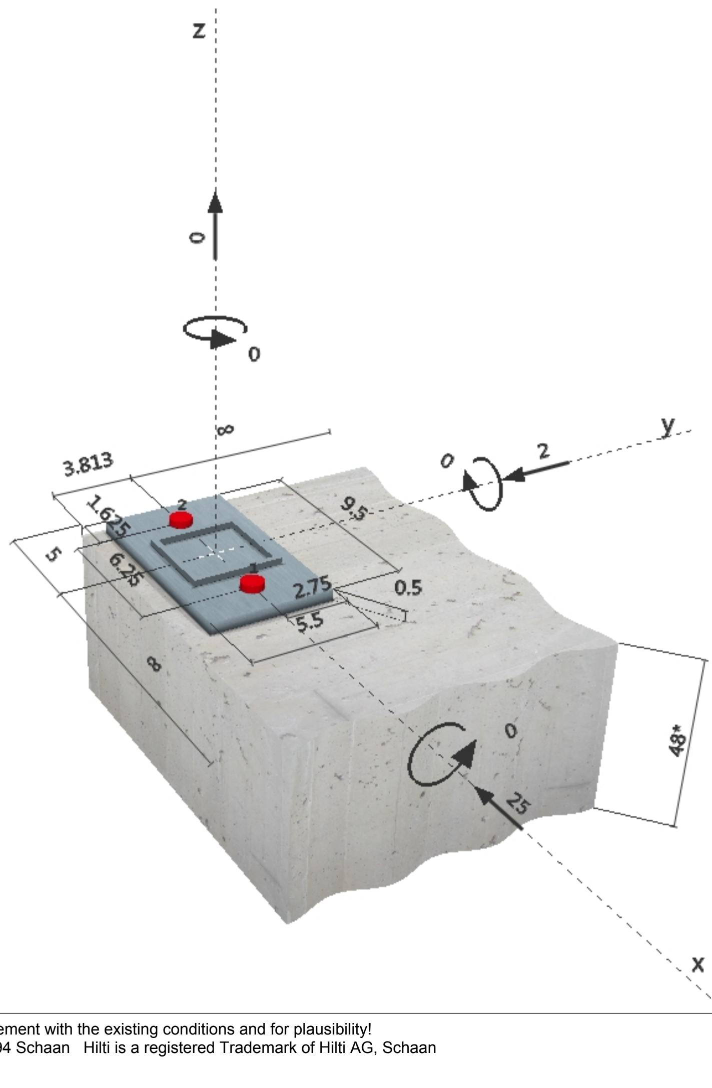
Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1**
Effective embedment depth:	h_{ef} = 12.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 9.500 in. x 5.500 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	Square HSS (AISC); (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.
Base material:	cracked concrete, , f_c' = 3500 psi; h = 48.000 in.
Reinforcement:	tension: condition A, shear: condition A; anchor reinforcement: shear
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

Geometry [in.] & Loading [kip, ft.kip]



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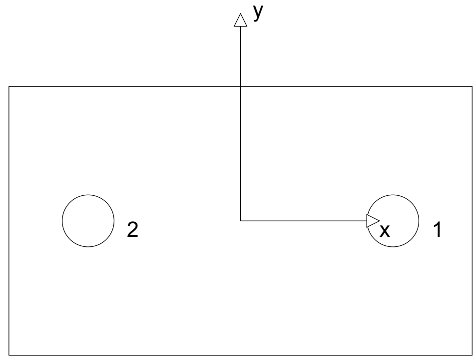
2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [kip]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	12.540	-12.500	-1.000
2	0.000	12.540	-12.500	-1.000

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0.000 [kip]
resulting compression force in (x/y)=(0.000/0.000): 0.000 [kip]

3 Tension load

	Load N_{ua} [kip]	**Capacity ϕN_n [kip]**	**Utilization $\beta_N = N_{ua}/\phi N_n$**	**Status**
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

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4 Shear load

	Load V_{ua} [kip]	Capacity ϕV_n [kip]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	12.540	13.708	92	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	25.080	26.801	94	OK
Concrete edge failure in direction **[1]	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

[1] Shear Anchor Reinforcement has been selected!

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	58000

Calculations

V_{sa} [kip]
21.089

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
21.089	0.650	13.708	12.540

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e'_N}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = 16\, \lambda_a \sqrt{f'_c}\, h_{ef}^{5/3}$ ACI 318-14 Eq. (17.4.2.2b)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	12.000	0.000	0.000	1.875

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	16	1.000	3500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
569.86	1296.00	1.000	1.000	0.731	1.000	59.537

Results

V_{cpg} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [kip]	V_{ua} [kip]
38.287	0.700	1.000	1.000	26.801	25.080

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5 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- The design of Anchor Reinforcement is beyond the scope of PROFIS Anchor. Refer to ACI 318-14, Section 17.5.2.9 for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Anchor calculations.

Fastening meets the design criteria!

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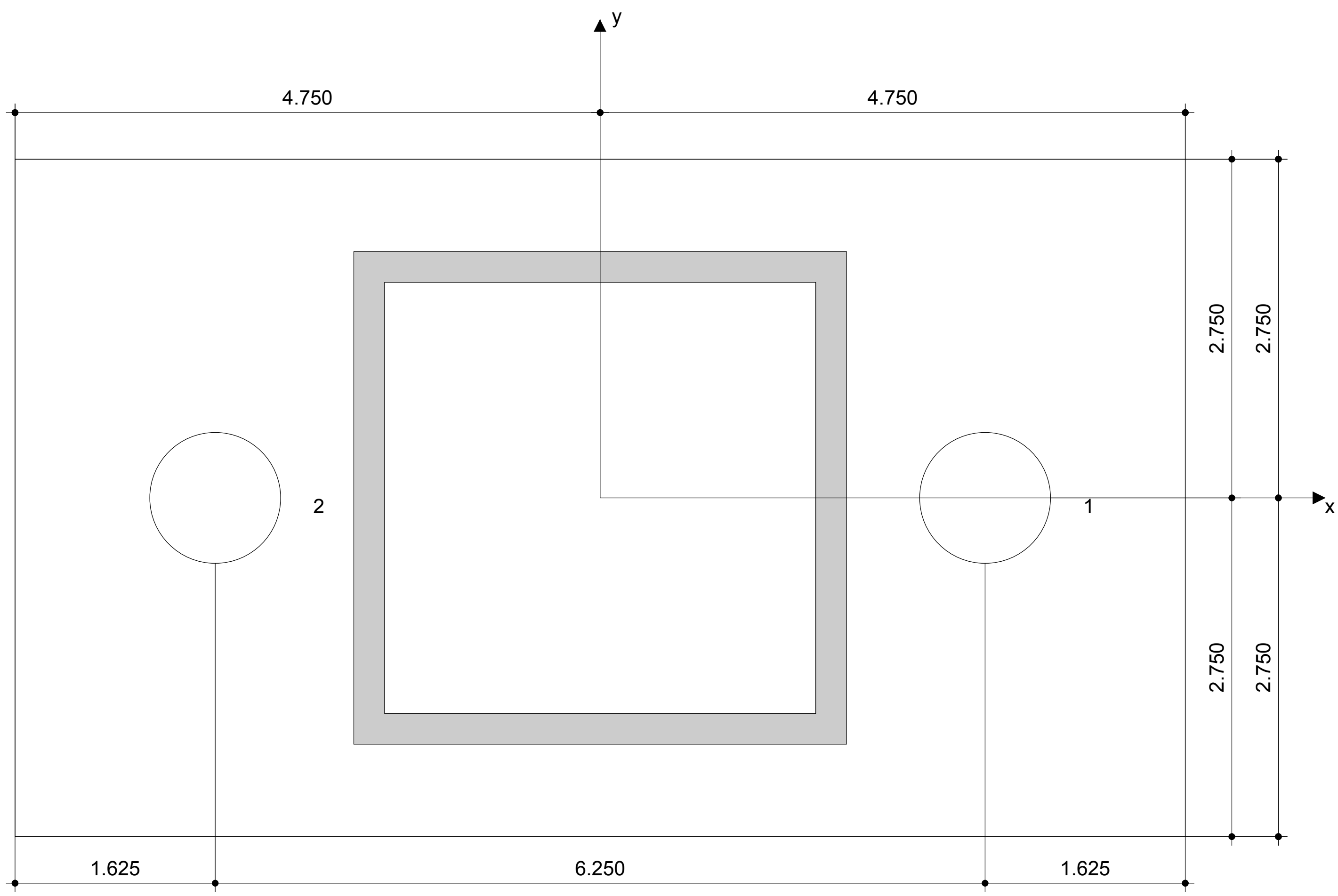
6 Installation data

Anchor plate, steel: -
Profile: Square HSS (AISC); 4.000 x 4.000 x 0.250 in.
Hole diameter in the fixture: d_f = 1.063 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Drilling method: -
Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 12.000 in.
Minimum thickness of the base material: 13.172 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	3.125	0.000	8.125	-	3.813	-
2	-3.125	0.000	1.875	-	3.813	-

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7 Remarks; Your Cooperation Duties

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

BASE PLATE I

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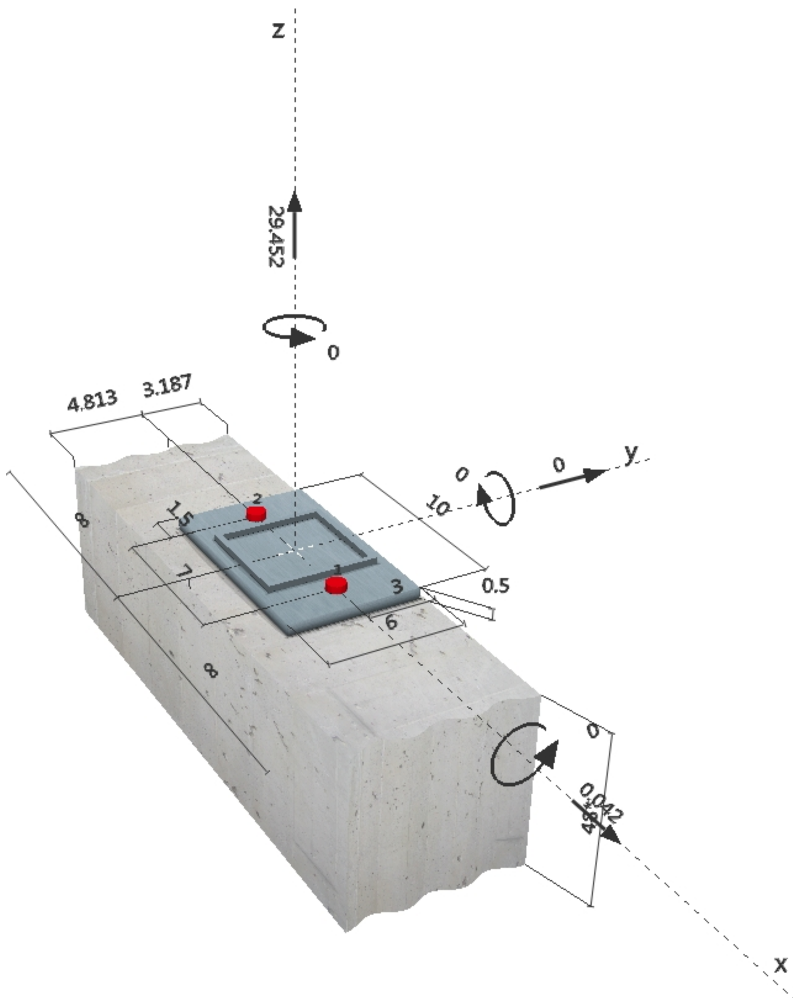
Specifier's comments:

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 7/8**
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 10.000 in. x 6.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	Square HSS (AISC); (L x W x T) = 5.000 in. x 5.000 in. x 0.250 in.
Base material:	cracked concrete, , f_c' = 3500 psi; h = 48.000 in.
Reinforcement:	tension: condition A, shear: condition A; anchor reinforcement: tension
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

Geometry [in.] & Loading [kip, ft.kip]



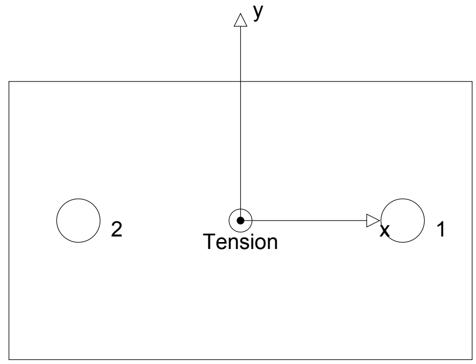
2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [kip]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	14.726	0.021	0.021	0.000
2	14.726	0.021	0.021	0.000

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 29.452 [kip]
resulting compression force in (x/y)=(0.000/0.000): 0.000 [kip]



3 Tension load

	Load N_{ua} [kip]	Capacity ϕ N_n [kip]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	14.726	20.097	74	OK
Pullout Strength*	14.726	17.464	85	OK
Concrete Breakout Strength**[1]	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)
[1] Tension Anchor Reinforcement has been selected!

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.46	58000

Calculations

N_{sa} [kip]
26.796

Results

N_{sa} [kip]	ϕ_{steel}	ϕ N_{sa} [kip]	N_{ua} [kip]
26.796	0.750	20.097	14.726

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-14 Eq. (17.4.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-14 Eq. (17.4.3.4)
$\phi N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.19	1.000	3500

Calculations

N_p [kip]
33.264

Results

N_{pn} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{pn} [kip]	N_{ua} [kip]
33.264	0.700	0.750	1.000	17.464	14.726

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4 Shear load

	Load V_{ua} [kip]	Capacity ϕV_n [kip]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	0.021	10.450	1	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	0.042	14.539	1	OK
Concrete edge failure in direction y+**	0.042	7.871	1	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.46	58000

Calculations

V_{sa} [kip]
16.078

Results

V_{sa} [kip]	ϕ_{steel}	$\phi\ V_{sa}$ [kip]	V_{ua} [kip]
16.078	0.650	10.450	0.021

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\, \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	3.187

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	3500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
200.00	324.00	1.000	1.000	0.806	1.000	20.868

Results

V_{cpg} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [kip]	V_{ua} [kip]
20.771	0.700	1.000	1.000	14.539	0.042

4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\, \psi_{ed,V}\, \psi_{c,V}\, \psi_{h,V}\, \psi_{parallel,V}\, V_b$ ACI 318-14 Eq. (17.5.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = 9\ \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
3.187	-	0.000	1.000	48.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
6.000	1.000	0.875	3500	2.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [kip]
79.17	45.71	1.000	1.000	1.000	3.029

Results

V_{cbg} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cbg}$ [kip]	V_{ua} [kip]
10.495	0.750	1.000	1.000	7.871	0.042

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.843	0.005	1.000	71	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- The design of Anchor Reinforcement is beyond the scope of PROFIS Anchor. Refer to ACI 318-14, Section 17.4.2.9 for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Anchor calculations.

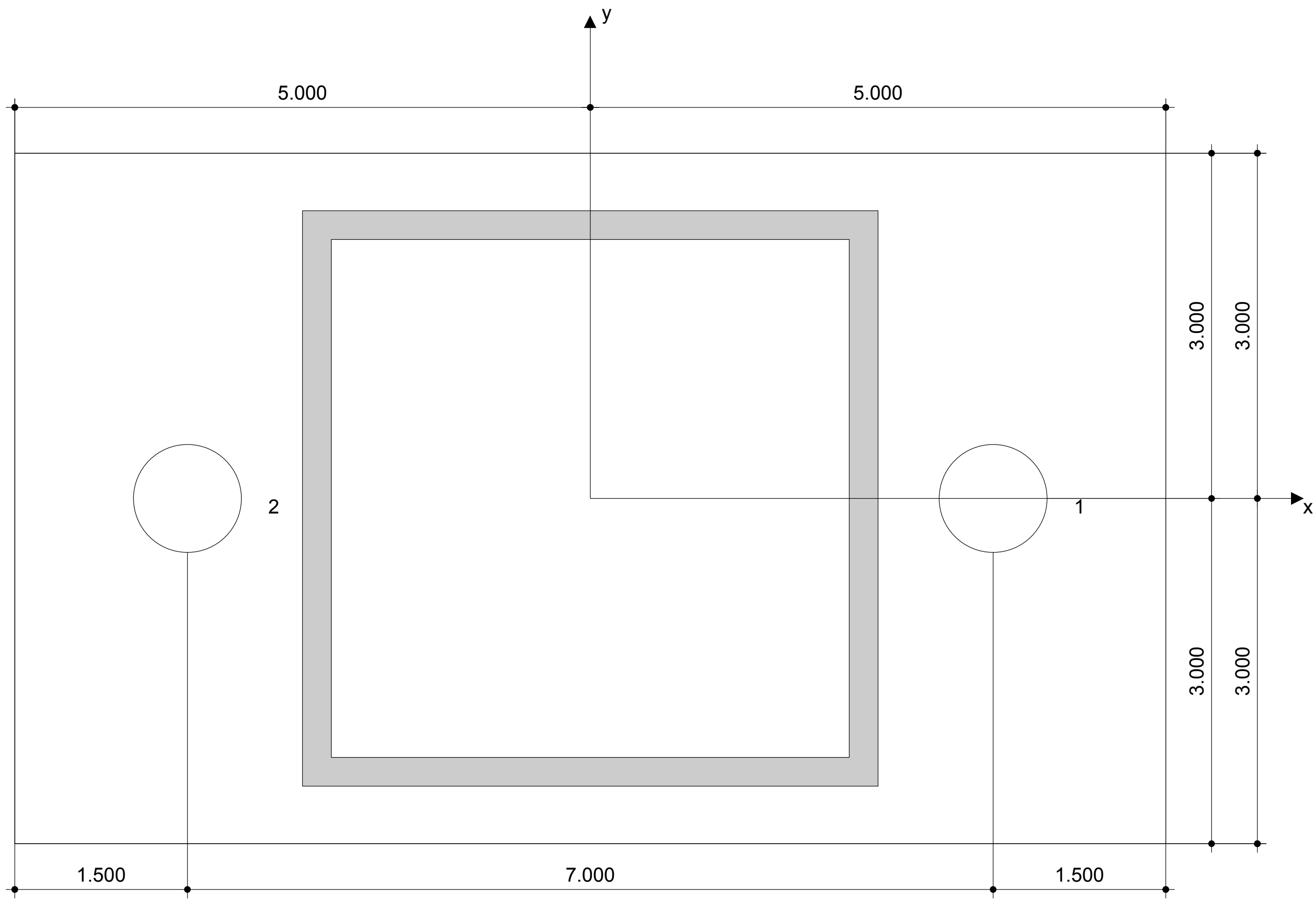
Fastening meets the design criteria!

Company:	Blackwell Structural Engineers	Page:	7
Specifier:		Project:	Kimmelman May Res
Address:		Sub-Project I Pos. No.:	170266
Phone I Fax:	|	Date:	11/2/2017
E-Mail:			

7 Installation data

Anchor plate, steel: -
Profile: Square HSS (AISC); 5.000 x 5.000 x 0.250 in.
Hole diameter in the fixture: d_f = 0.938 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Drilling method: -
Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 7/8
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 6.000 in.
Minimum thickness of the base material: 7.052 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	3.500	0.000	-	-	4.813	3.187
2	-3.500	0.000	-	-	4.813	3.187

Company:	Blackwell Structural Engineers	Page:	8
Specifier:		Project:	Kimmelman May Res
Address:		Sub-Project I Pos. No.:	170266
Phone I Fax:	|	Date:	11/2/2017
E-Mail:			

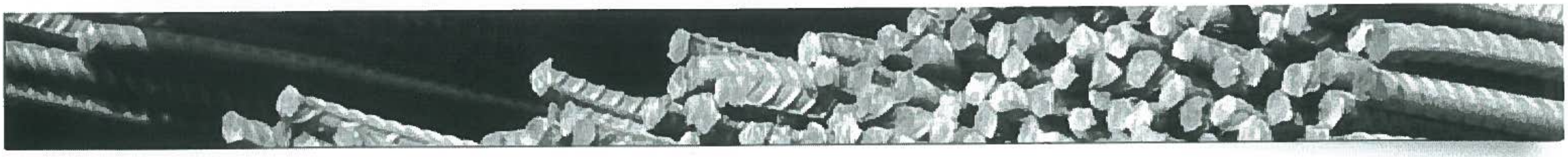
8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

Blackwell

Structural Engineers

CONCRETE FOUNDATION DESIGN

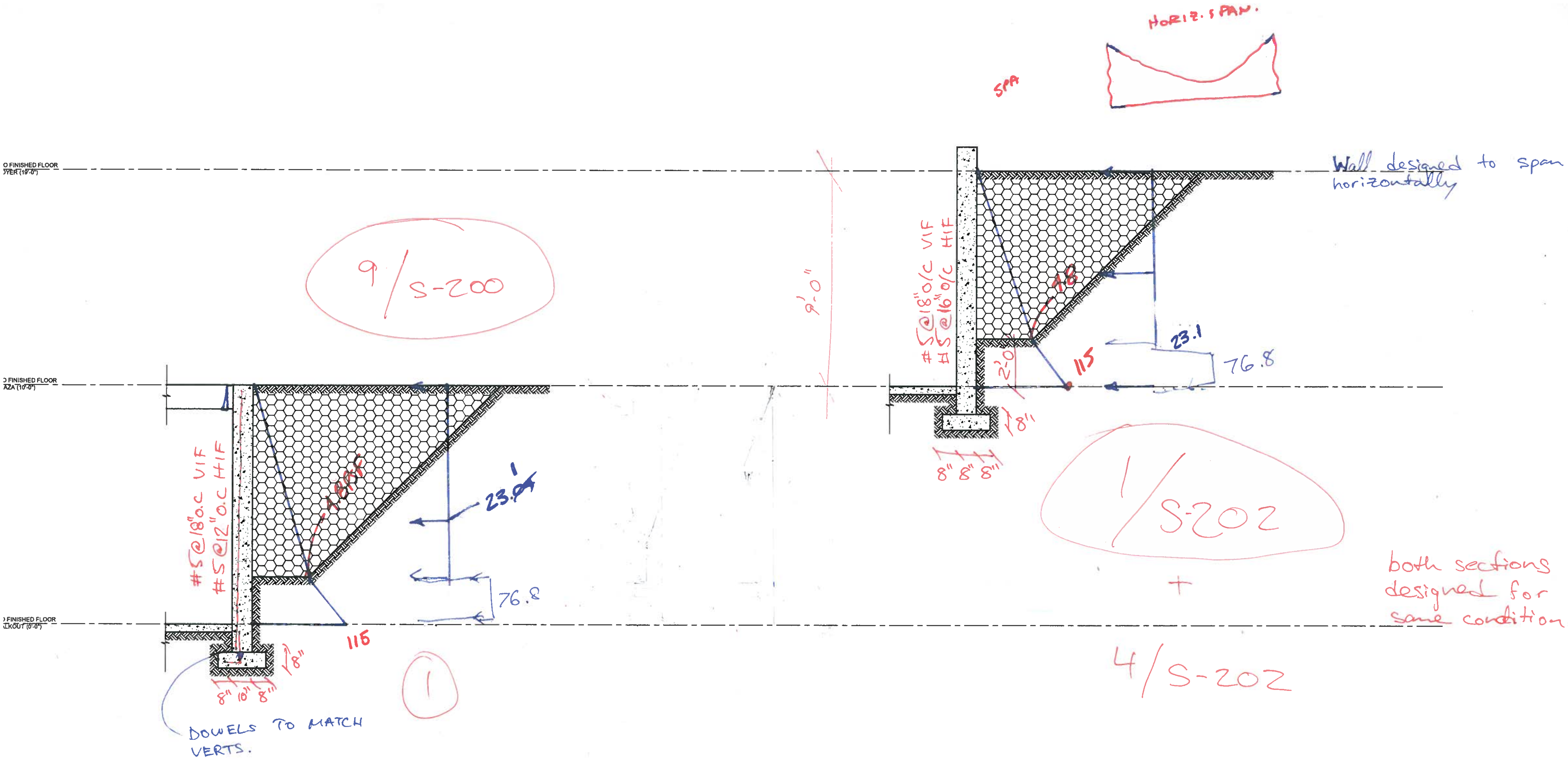


Steel Rebar Sizes & Rebar Stock

We stock a wide variety of rebar sizes from #3-#10; grades: 40, 60 and A706 (weldable); lengths: 20', 30', 40' and 60'; and finishes: black, epoxy, or A1035 (MMFX). We are committed to having what our customers need when they need it. As a subsidiary of Nucor, we have access to rebar across the US and internationally through Nucor Trading. We can find what you're looking for, whatever it is.

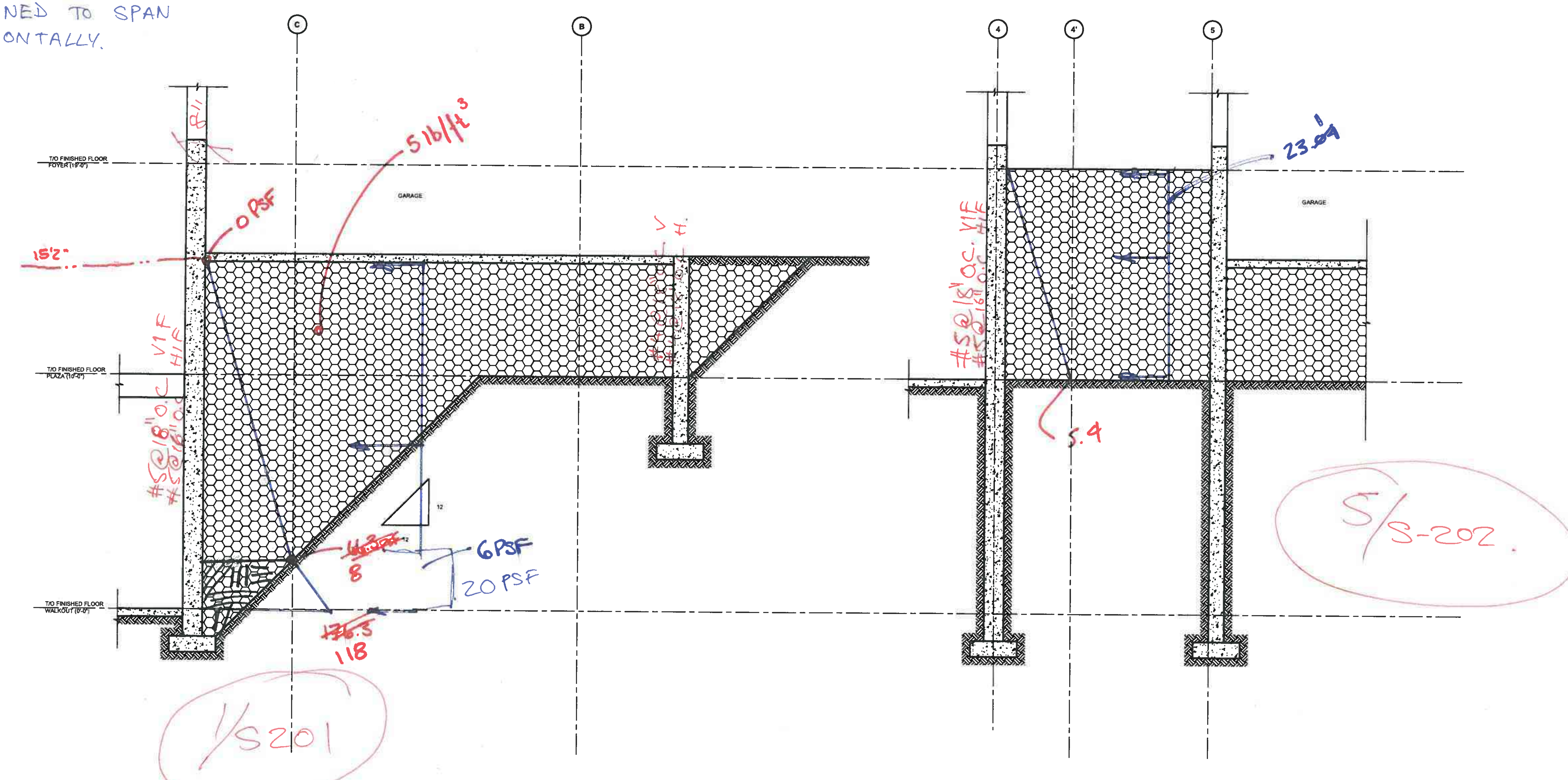
Rebar Sizes We Stock:

.1M	Imperial Bar Size	"Soft" Metric Size	Weight per unit length (lb/ft)	Mass per unit length (kg/m)	Nominal Diameter (in)	Nominal Diameter (mm)	Nominal Area(in^2)	Nominal Area (mm^2)
OM	#3	#10	0.376	0.561	0.375 9.5	9.525	0.11	71
	#4	#13	0.668	0.996	0.500 12.7m	12.7	0.2	129
5M	#5	#16	1.043	1.556	0.625 15.875	15.875	0.31	200
0M	#6	#19	1.502	2.24	0.750 19.05	19.05	0.44	284
	#7	#22	2.044	3.049	0.875 22.3	22.225	0.6	387
5M	#8	#25	2.67	3.982	1.000 25.4	25.4	0.79	509
	#9	#29	3.4	5.071	1.128	28.65	1	645
	#10	#32	4.303	6.418	1.27	32.26	1.27	819
	#11	#36	5.313	7.924	1.41	35.81	1.56	1006
	#14	#43	7.65	11.41	1.693	43	2.25	1452
	#18	#57	13.6	20.284	2.257	57.33	4	2581

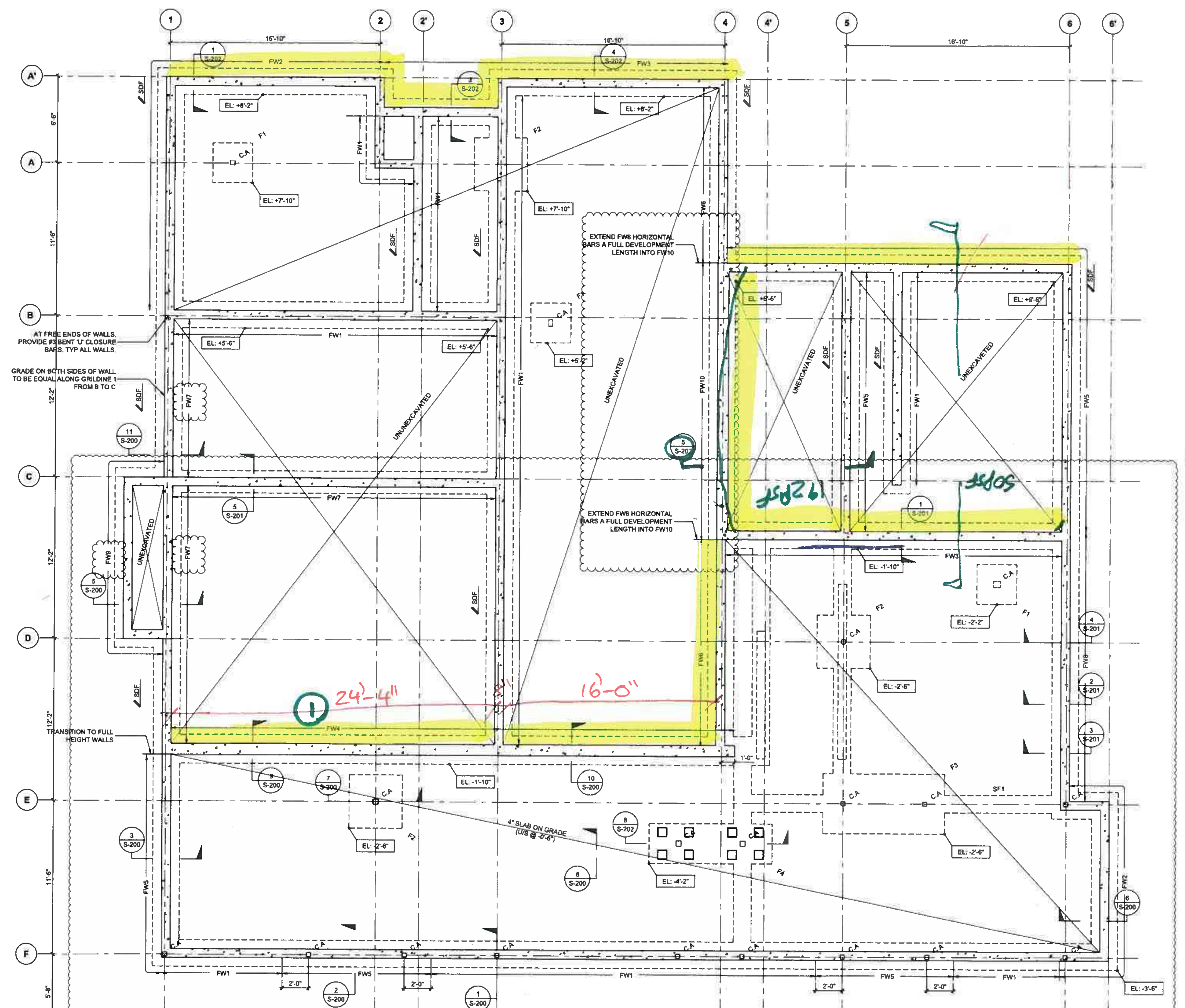


HORIZ. SPAN.
Wall designed to span horizontally
FINISHED FLOOR
FINISHED FLOOR
FINISHED FLOOR
9/S-200
9'-0"
#5@18"o/c VIF
#5@16"o/c HIF
48
23.1
76.8
115
2'-0"
8"
8" 8" 8"
1/S-202
both sections designed for same condition.
4/S-202
#5@18"o.c VIF
#5@12"o.c HIF
23.1
76.8
115
8"
8" 10" 8"
DOWELS TO MATCH VERTS.

DESIGNED TO SPAN HORIZONTALLY.



Poisson's Ratio Geofoam ~ K_a = 0.12kPa



1 S-100 FOUNDATION PLAN 1/4" = 1'-0"

NOTES:

1. A GEOTCHNICAL REPORT HAS BEE PREPARED BY IGES INC. THE CONTRACTOR IS TO READ THE REPORT AND BECOME FRAMILIAR WITH ITS CONTENTS.
2. SHALLOW FOUNDATIONS HAVE BEEN DESIGNED WITH AN ALLOWABLE BEARING CAPCITY OF 2,900psf FOR DEAD AND LIVE LOADS. ⅓ INCREASE FOR SEISMIC AND WIND.
3. NO FOOTINGS ARE TO BE CAST WITHOUT PRIOR APPROVAL FROM THE GEOTECHNICAL CONSULTANT.
4. ASSUMED SPOT ELEVATIONS OF FOOTINGS ARE GIVEN AS UNDERSIDE OF FOOTING AND ARE RELATIVE WALKOUT FINISHED FLOOR OF (0'-0"). U/S OF FOOTINGS MAY BE REQUIRED TO VARY BASED ON COMMENTS FROM IGES FOLLOWING EXCAVATION.
5. REFER TO GENERAL NOTES AND TYPICAL DETAILS FOR ADDITIONAL INFORMATION.

FOUNDATION MEMBER SCHEDULE

MEMBER MARK	MEMBER DESCRIPTION	REMARKS
FW1	8" CONCRETE FOUNDATION WALL	V.I.F N/A V.O.F N/A H.I.F N/A H.O.F N/A ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS
FW2	8" CONCRETE FOUNDATION WALL	V.I.F #5@ 12"c/c V.O.F N/A H.I.F #5@ 16"c/c H.O.F N/A ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS
FW3	8" CONCRETE FOUNDATION WALL	V.I.F #5@ 8"c/c V.O.F #5@ 8"c/c H.I.F #5@ 12"c/c H.O.F #5@ 12"c/c ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS
FW4	10" CONCRETE FOUNDATION WALL	V.I.F #5@ 16"c/c V.O.F N/A H.I.F #5@ 16"c/c H.O.F N/A ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS
FW5	8" CONCRETE FOUNDATION WALL	V.I.F #5@ 16"c/c V.O.F #5@ 16"c/c H.I.F #5@ 16"c/c H.O.F #5@ 16"c/c ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS.
FW6	8" CONCRETE FOUNDATION WALL	BELOW S.O.G. ELEVATION: V.I.F #5@ 12"c/c V.O.F #5@ 12"c/c H.I.F #5@ 12"c/c H.O.F #5@ 12"c/c ABOVE S.O.G. ELEVATION V.I.F #5@ 12"c/c V.O.F #6@ 10"c/c H.I.F #5@ 10"c/c H.O.F #6@ 8"c/c ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS.
FW7	8" CONCRETE FOUNDATION WALL	#5@ 16"c/c MIDDLE OF WALL #5@ 16"c/c MIDDLE OF WALL ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS
FW8	8" CONCRETE FOUNDATION WALL	V.I.F #5@ 12"c/c V.O.F #5@ 18"c/c H.I.F #5@ 16"c/c H.O.F #5@ 18"c/c ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS
FW9	8" CONCRETE FOUNDATION WALL	V.I.F N/A V.O.F #5@ 18"c/c H.I.F N/A H.O.F #5@ 18"c/c ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS
FW10	10" CONCRETE FOUNDATION WALL	BELOW S.O.G. ELEVATION: V.I.F #5@ 12"c/c V.O.F #5@ 12"c/c H.I.F #5@ 12"c/c H.O.F #5@ 12"c/c ABOVE S.O.G. ELEVATION V.I.F #5@ 12"c/c V.O.F #6@ 10"c/c H.I.F #5@ 10"c/c H.O.F #6@ 8"c/c ON 8"x24" CONTINUOUS STRIP FOOTING r/w 2 #5 CONTINUOUS BARS
F1	3'-0" x3'-0" x 1'-0" CONCRETE FOOTING	r/w 4 #5 BARS B.E.W.
F2	4'-0" x4'-0" x 1'-4" CONCRETE FOOTING	r/w 6 6 BARS B.E.W.
F3	4'-6" x7'-0" x 1'-4" CONCRETE FOOTING	r/w 6 #5 BARS T&B LONG DIRECTION 8 #5 BARS T&B SHORT DIRECTION
F4	3'-0" x 9'-3" x 3'-0" CONC. GRADE BEAM	r/w 7 #6 BARS T&B LONG DIRECTION 9 #5 BARS T&B SHORT DIRECTION #3 3 LEGGED STIRRUPS @ 4"c/c 8 HELICAL PIERS EACH RATED FOR 20 (-16)KIPS (LFRD)
SF1	1'-4"x8" CONTINUOUS STRIP FOOTING	r/w 2 #5 CONTINUOUS BARS

NOTES:

1. PROVIDE MINIMUM 2 #5 CONTINUOUS TOP BARS IN ALL WALLS.
2. DOWELLING FROM FOOTINGS TO WALLS TO MATCH VERTICAL REINFORCING OF WALLS ABOVE.
3. AT ABUTTING CORNERS PROVIDE CORNER BARS MATCHING REINFORCING OF MOST STRINGENT WALL AT SPACING MATCHING HORIZONTAL REINFORCING. REFER TO TYPICAL DETAIL 0312.
4. ALL SLABS SHALL BE REINFORCED WITH 4"x4" W2.9xW2.9 WELDED WIRE FABRIC PLACED WITHIN THE MIDDLE THIRD OF THE SLAB.

Blackwell

134 Peter St. #1301, Toronto, M5V 2H2 T 416.593.5300 | 31 King St. N., 2nd Fl., Waterloo, N2J 2W6 T 519.616.6895 | blackwell.ca

Seal

SULLAWAY
ENGINEERING
10815 RANCHO BERNARDO ROAD
SUITE 210
SAN DIEGO, CA 92127
(858) 312-5150
www.sullawayeng.com

MARK	DATE	DESCRIPTION
2	17.11.24	STEEL SHOP DRAWING DEVELOPMENT
1	17.09.29	REVISED PERMIT SET
	17.09.13	INTERNAL COORD SET
	17.07.31	ISSUED FOR PERMIT
	17.07.26	ISSUED FOR COORDINATION

ISSUE:

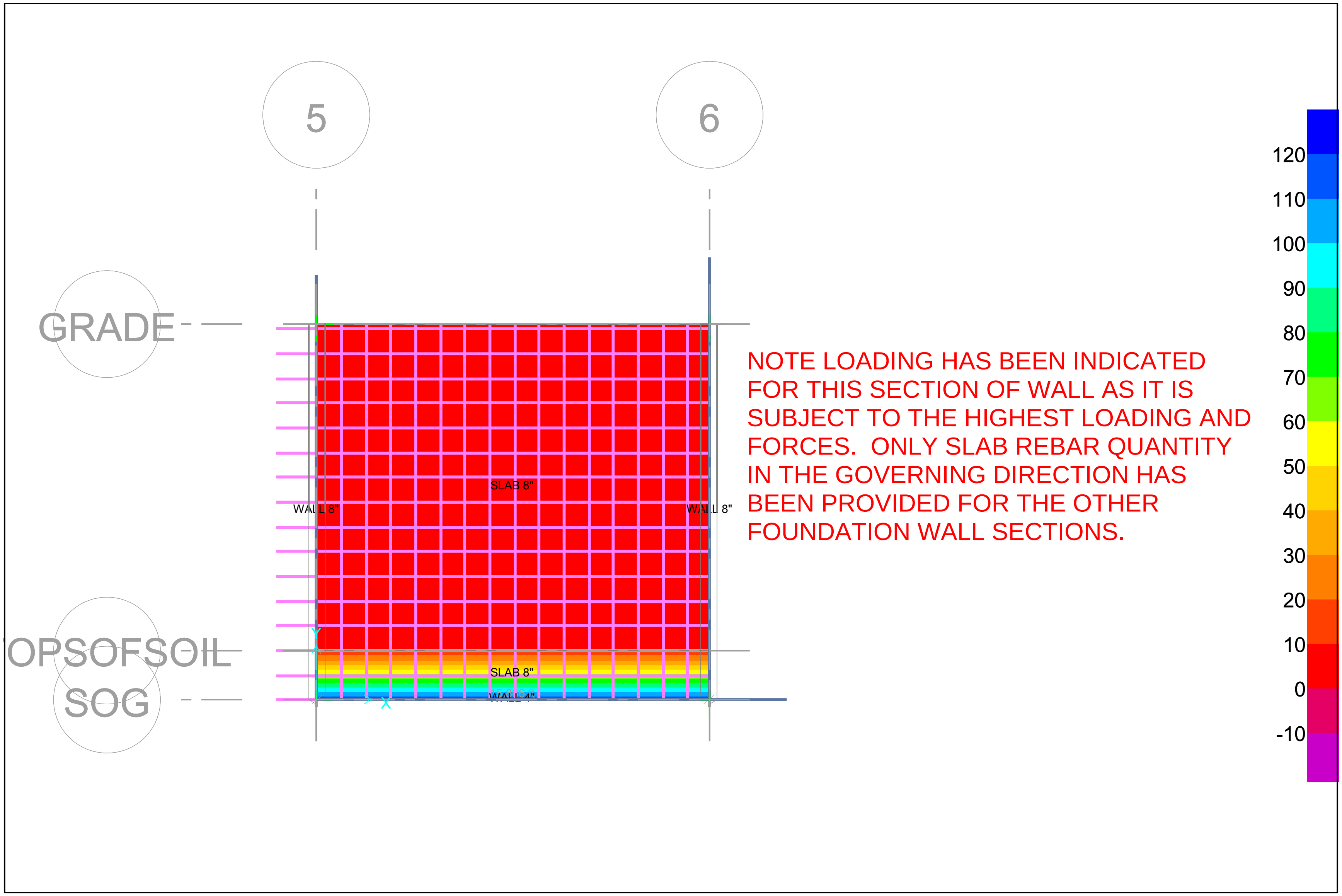
Project Name
KIMMELMAN MAY RESIDENCE

Address
SUMMIT POWDER MOUNTAIN
EDEN, UTAH

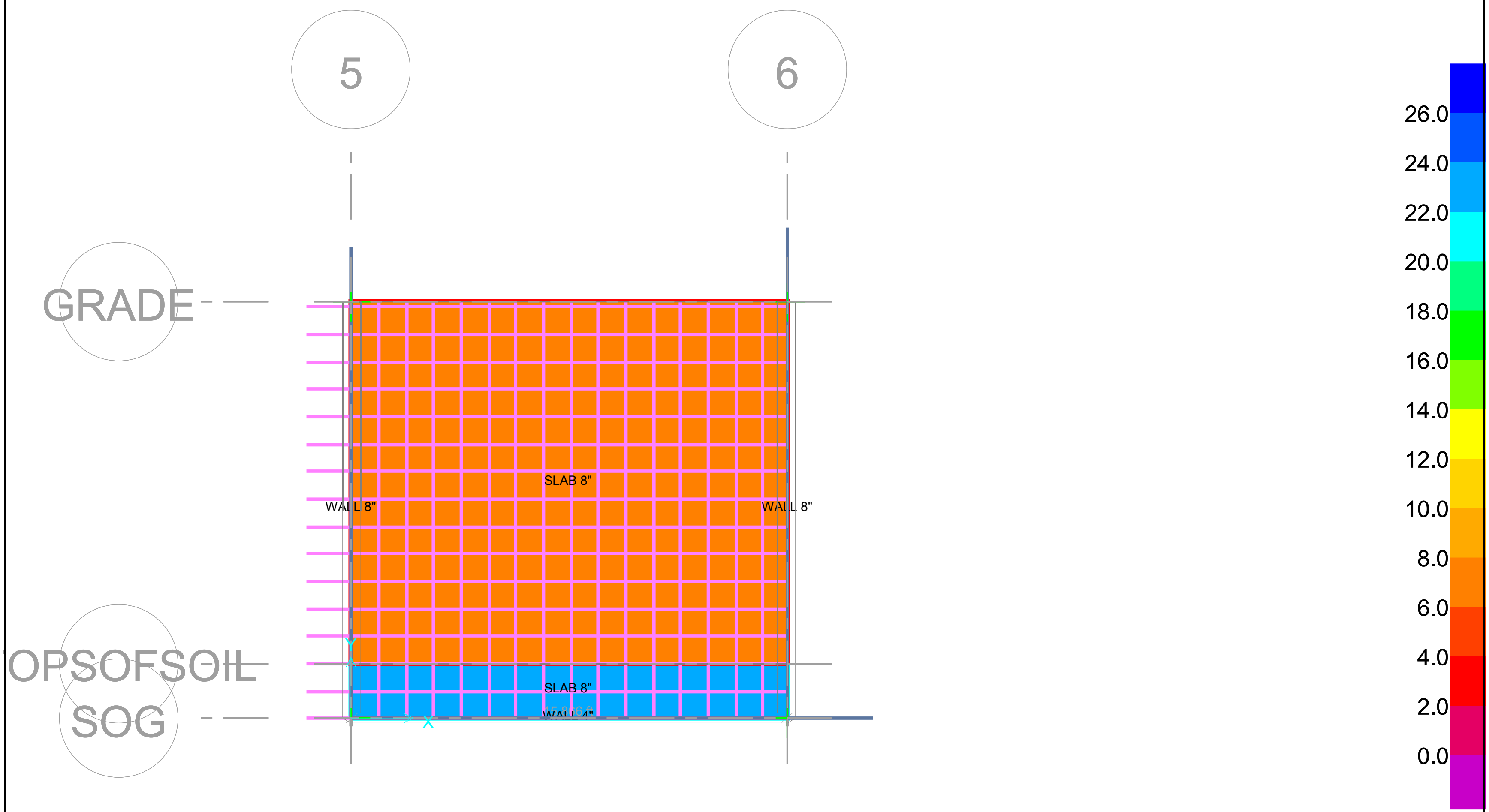
File Name: FILENAME.EXT	CAD/BIM Program: AUTOCAD
Drawn by: AVB	Checked by: DB
Scale: AS NOTED	Project #: 170266

Sheet Title
FOUNDATION PLANS

S-100



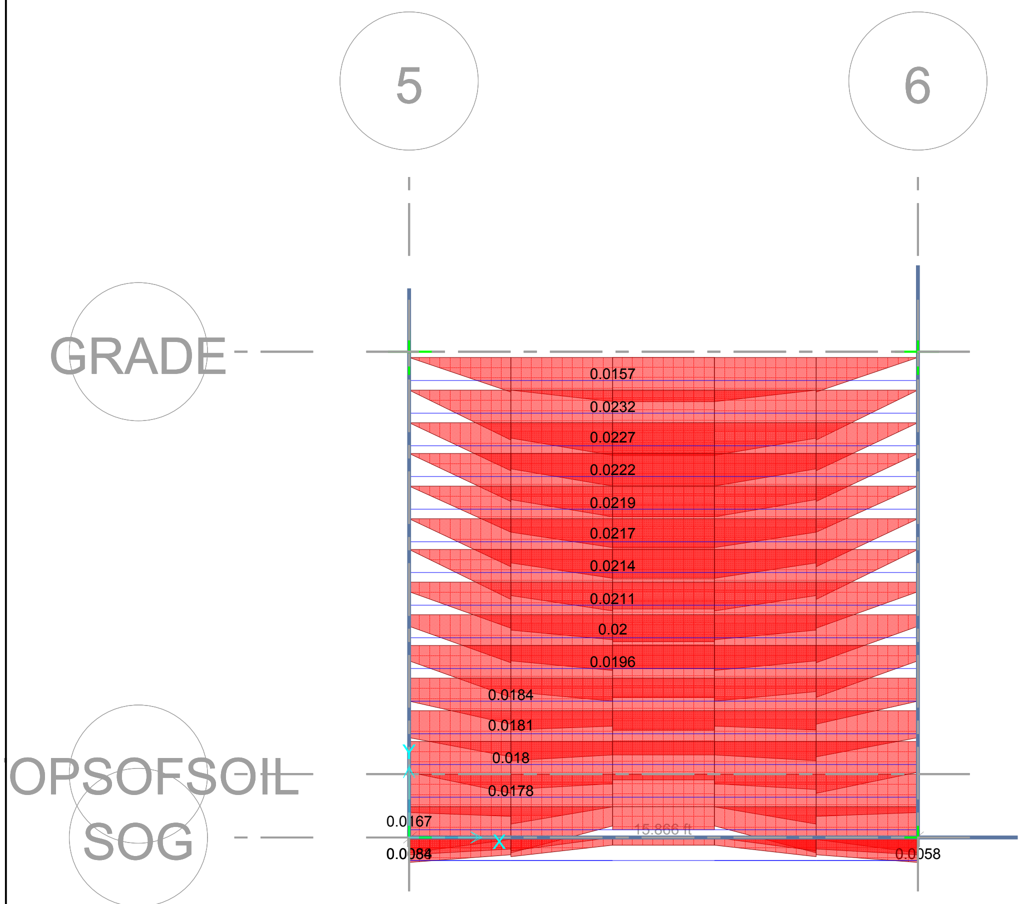
SAFE 2016 16.0.2 Slab Surface Loading in Gravity Direction (SOIL) [lb/ft2] kip - ft



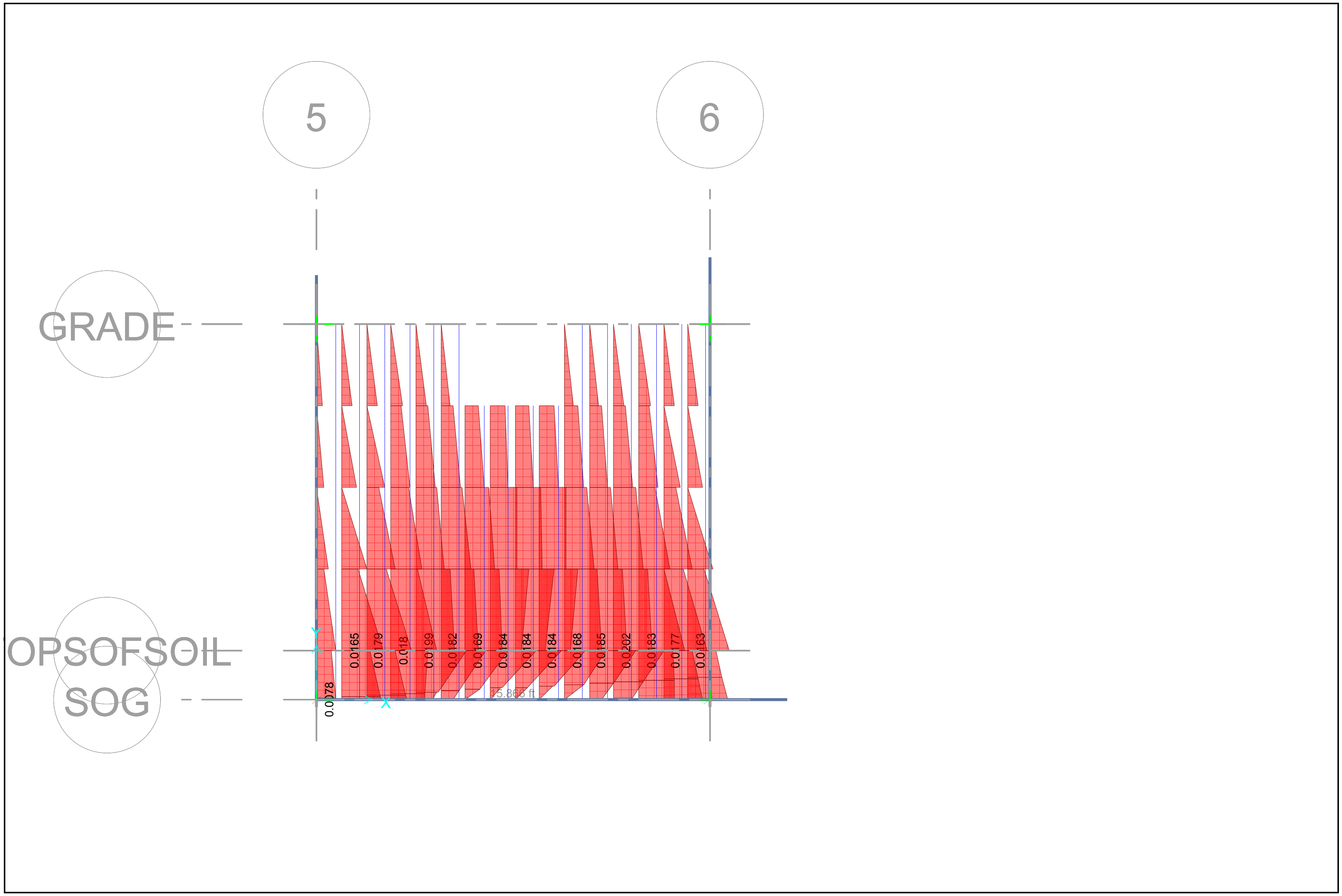
SAFE 2016 16.0.2

Slab Surface Loading in Gravity Direction (LIVE) [lb/ft2]

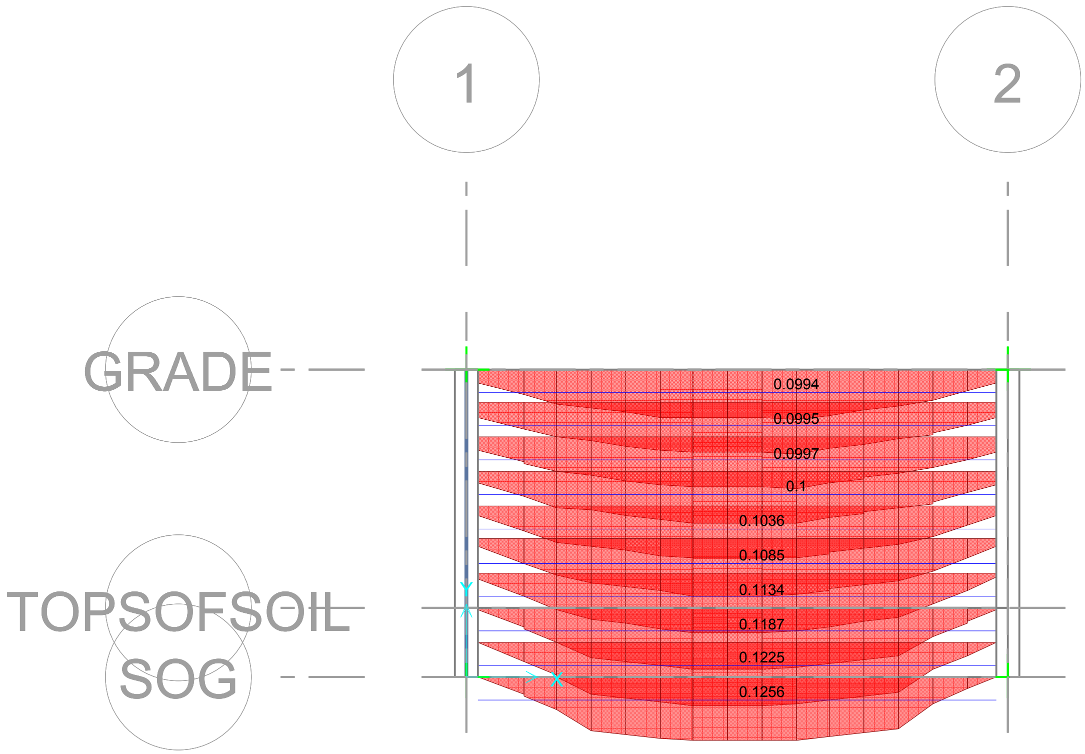
kip - ft



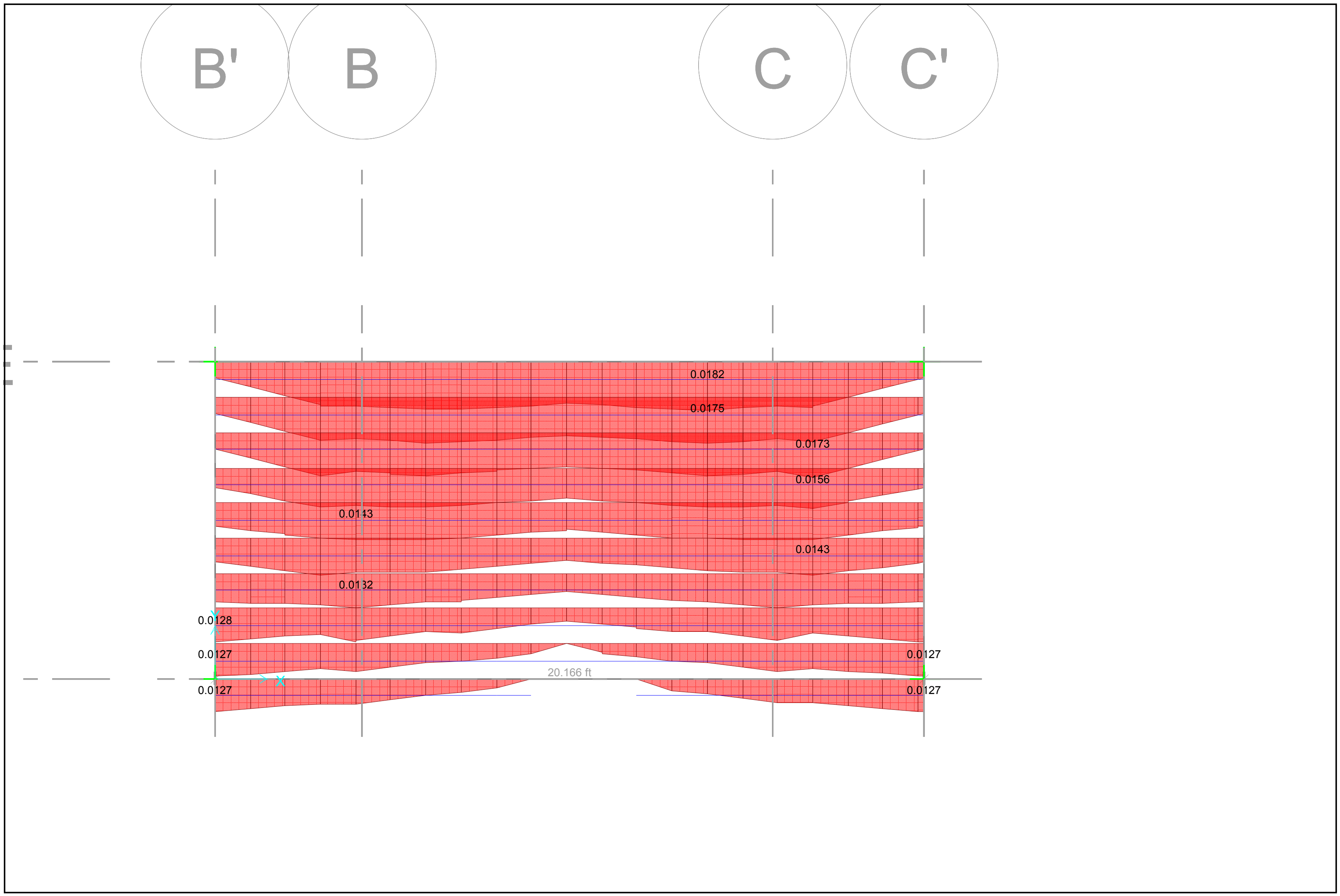
SAFE 2016 16.0.2 Slab Strip Design - Layer A - Bottom Reinforcement Area (Enveloping Flexural) [in2] kip - ft



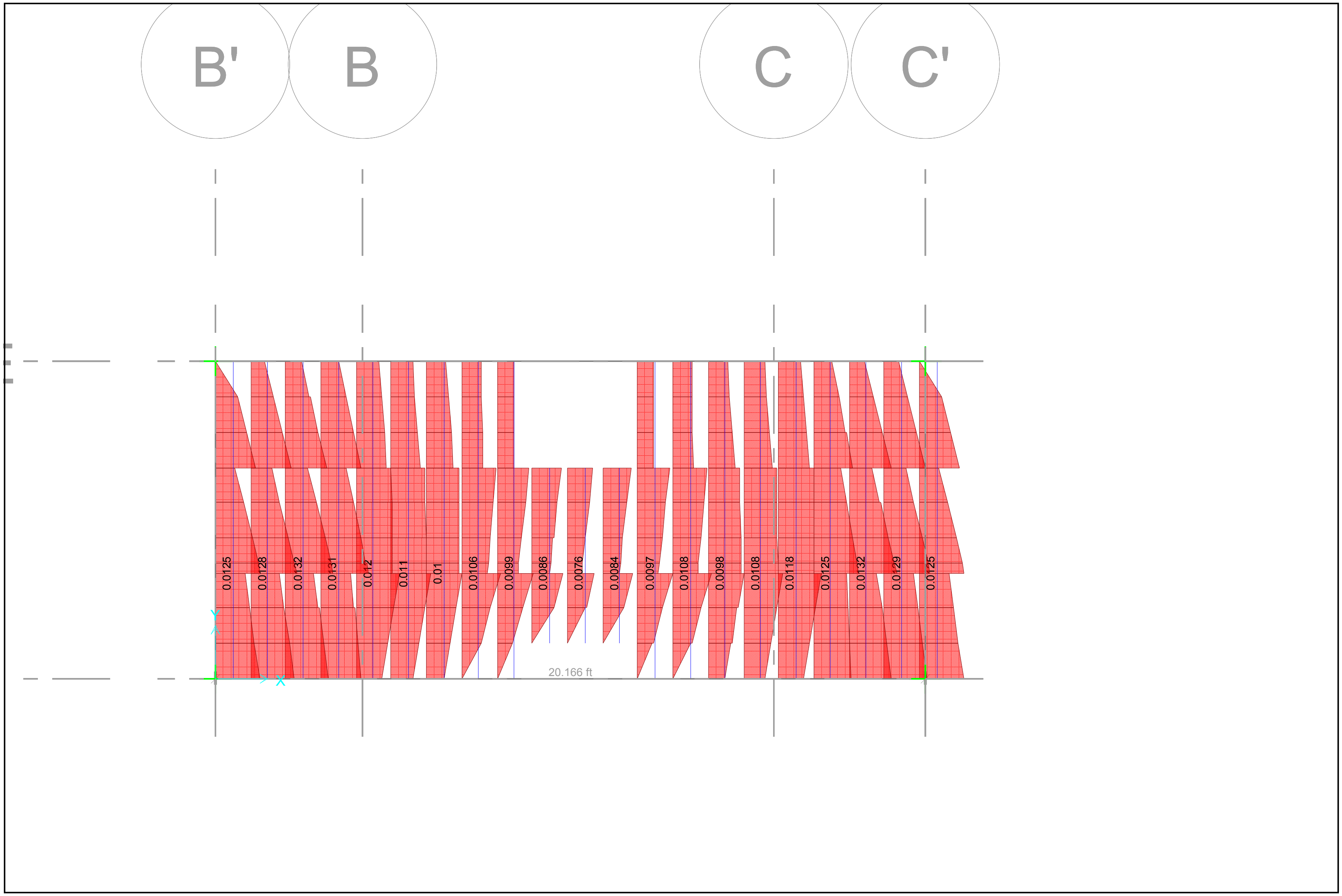
SAFE 2016 16.0.2 Slab Strip Design - Layer B - Bottom Reinforcement Area (Enveloping Flexural) [in2] kip - ft



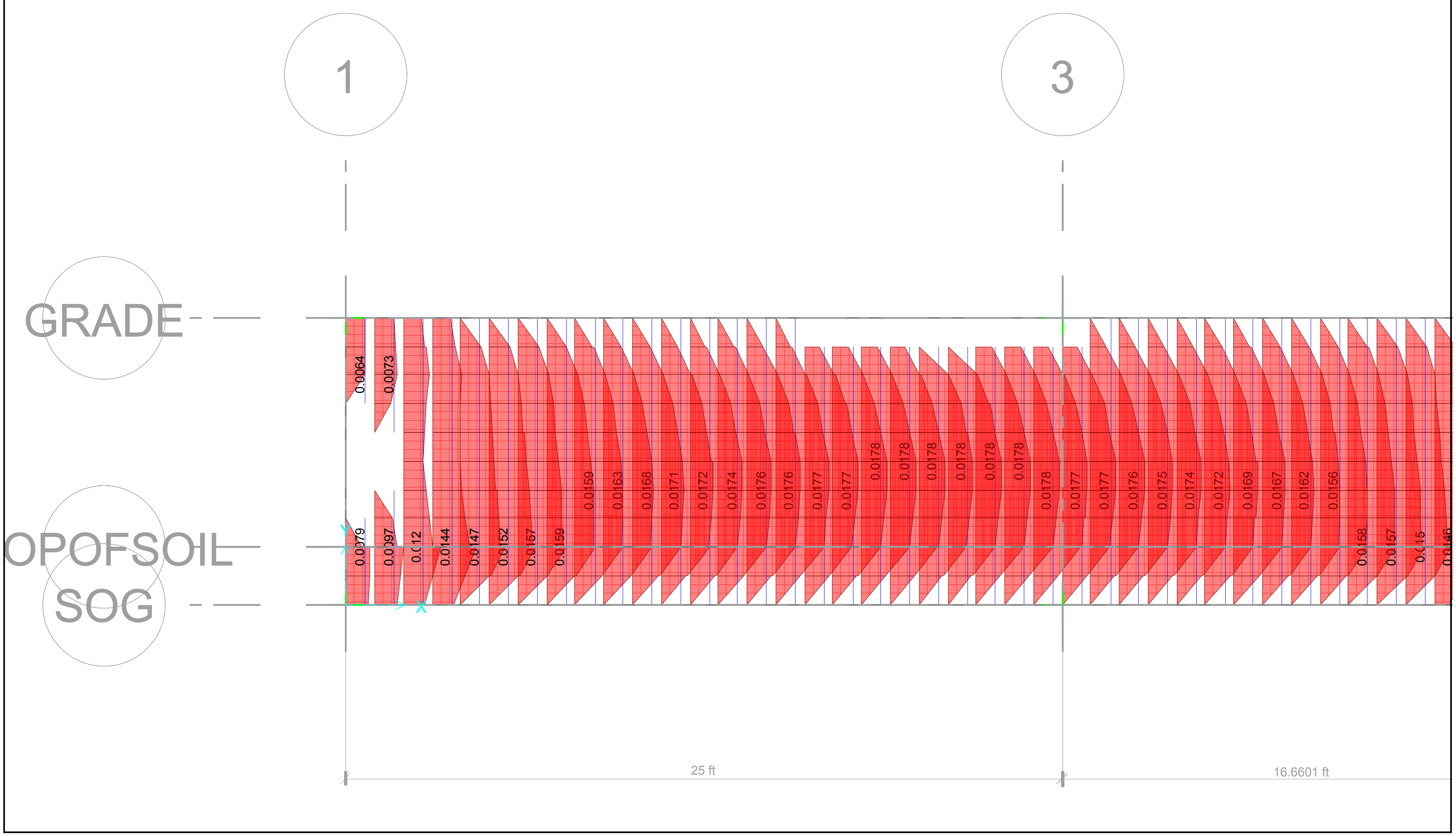
SAFE 2016 16.0.2 Slab Strip Design - Layer A - Bottom Reinforcement Intensity (Enveloping Flexural) [in2/ft] kip - ft



SAFE 2016 16.0.2 Slab Strip Design - Layer A - Bottom Reinforcement Intensity (Enveloping Flexural) [in2/ft] kip - ft



SAFE 2016 16.0.2 Slab Strip Design - Layer B - Bottom Reinforcement Intensity (Enveloping Flexural) [in2/ft] kip - ft


1
3
GRADE
TOPOFSOIL
SOG
0.0064
0.0073
0.0079
0.0097
0.012
0.0144
0.0147
0.0152
0.0157
0.0159
0.0159
0.0163
0.0168
0.0171
0.0172
0.0174
0.0176
0.0176
0.0177
0.0177
0.0178
0.0178
0.0178
0.0178
0.0178
0.0178
0.0178
0.0177
0.0177
0.0176
0.0175
0.0174
0.0172
0.0169
0.0167
0.0162
0.0156
0.0158
0.0157
0.015
0.0146
25 ft
16.6601 ft

Blackwell

Title:	Concrete Flexure
Master Created By:	J. David Bowick
Date:	8/8/2016
Master Last modified by:	
Date:	2017.07.29
Notes:	DESIGN FOR PAD FOOTING IN BASEMENT SUPPORTING VOLUME 2 BRACED FRAME SEISMIC LOADING
Project Name:	KIMMELMAN RESIDENC
Project Number:	170266
Name:	AVB
Date:	2017.07.29

Load		
Mf	112	kNm

Material		
ϕ_s	0.85	
fy	400	MPa
ϕ_c	0.65	
f'_c	30	MPa
α_1	0.805	

Geometry		
b	1000	mm
h	400	mm
Cover	75	mm
Aggregate	20	mm

Trial		
Bar Size	#6	
d_b	19.05	mm
A_b	284	mm^2
d	315.475	mm
A_{sreq}	1085	mm^2
No. Bars	4	

Reinforcing		
Min. Clear	41.9	mm
s_{min}	71.8	mm
s	75	mm
Row 1		
Bar Size	#6	
Bars	6	
A_s	1704	mm^2
d	315.475	mm
Row 2		
Bar Size	30M	
Bars		
A_s		mm^2
d	240.475	mm
Row 3		
Bar Size	35M	
Bars		
A_s		mm^2
d	165.475	mm

Calculations		
T	579.36	kN
d	315.5	mm
$\beta_1 c$	36.9	mm
Mr	172.1	kNm
Mf/Mr	0.65	

Bar Size	d_b (mm)	A_b (mm^2)
10M	11.3	100.0
15M	16	200.0
20M	19.5	300.0
25M	25.2	500.0
30M	29.9	700.0
35M	35.7	1000.0
45M	45	1500.0
55M	55	2500.0
#3	9.525	71.0
#4	12.7	129.0
#5	15.875	200.0
#6	19.05	284.0
#7	22.225	387.0
#8	25.4	509.0
#9	28.65	645.0
#10	32.26	819.0
#11	35.81	1006.0
#14	43	1452.0
#18	57.3	2581.0
#18J	59.4	2678.0

Blackwell
Structural Engineers

APPENDIX A - DESIGN LOADS

10815 Rancho Bernardo RD., SD, CA 92127
projectmanager@sullawayeng.com
Phone: 858-312-5150 Fax: 858-777-3534

Design Loads

for
The Kimmelman Residence

Summit Powder Mountain
Eden, UT

Project # 14663

date;
2017-8-14

SULLAWAY ENGINEERING

10815 Rancho Bernardo RD., SD, CA 92127
projectmanager@sullawayeng.com
Phone: 858-312-5150 Fax: 858-777-3534

PROJECT: Kimmelman Residence DATE: 6/1/2017
PROJ. NO.: 14663 ENGINEER: mfs
CLIENT: Blackwell

building code; IBC 2015 units; pounds, feet unless noted otherwise

Seismic Analysis- Building Structure

Design Force (ASCE 12)

Latitude 41.3007
Longitude -111.8127

S_1= 0.304 (from USGS) I= 1.0
S_{DS}= 0.683 Risk Category II
S_{D1}= 0.363 Seismic Design Cat. D
Ss= 0.898
F_a= 1.14
F_v= 1.80

R	Ω	Cd	
3.25	2	3.25	ASCE Table 12.2-1 B.3. "Steel ordinary concentrically braced frame"
6.5	3	4	ASCE Table 12.2-1 B.3. "Wood frame sheer wall"

$V=C_sW$ $C_s=S_{DS}/(R/I)$

Vertical Seismic Loads

$E_v=0.2S_{DS}DL$

Live Loads

Typical L_o= 40 psf Roof 20 psf
Reduction

$L=L_o(0.25+15/sqrt(K_{LL}A_T))$ R1= 0.6
K_{LL}= 1 R2= 0.6
A_T= 1044 $L_r=L_oR_1R_2$= 7.20 psf

A_T (sf)	L (psf)
1000	28.57
1500	25.49
2000	23.42
2500	22.00

SULLAWAY ENGINEERING

10815 Rancho Bernardo RD., SD, CA 92127
projectmanager@sullawayeng.com
Phone: 858-312-5150 Fax: 858-777-3534

PROJECT: Kimmelman Residence DATE: 6/1/2017
PROJ. NO.: 14663 ENGINEER: mfs
CLIENT: Blackwell

building code; IBC 2015 units; pounds, feet unless noted otherwise

Snow Load ASCE Chap. 7

Exposure Factor:	C_e =		=	1.0	
Thermal Factor:	C_t =		=	1.0	
Importance Factor:	I =		=	1.0	
Roof Slope Factor:	C_s =		=	1.0	
Ground Snow Load:	p_g =		=	274	psf
Flat Roof Snow Load:	p_f =	0.7 * Ce * Ct * I * pg =		192	psf
Sloped Roof Snow Load:	p_s =	C_s * p_f =		192	psf

Drift note- No snow drift on roof

l_u=	18	ft				
h_d=.43*(l_u)^0.33*(p_g+10)^0.25 -1.5 =	3.1	ft	leeward			
h_d=	2.2	ft	windward	w=4h_d=	8.8	ft
h_c = 15.6-h_d=	13.4	ft				
γ=0.13pg+14 <30 =	30	pcf				
h_b=	6.4	ft				
drift load = p_d = $h_d\gamma$ =	66	psf				

p_d = 0 at a distance of 'w' from wall

Unbalanced Snow Load

ASCE 7.6.1

W= 13.5 ft (W<20ft, therefore unbalanced load= Ip_g & slope == 26.4 deg.)

Use p_f = 0 psf per engineering judgement

Frost Depth

40 inches

Design Maps Summary Report

User–Specified Input

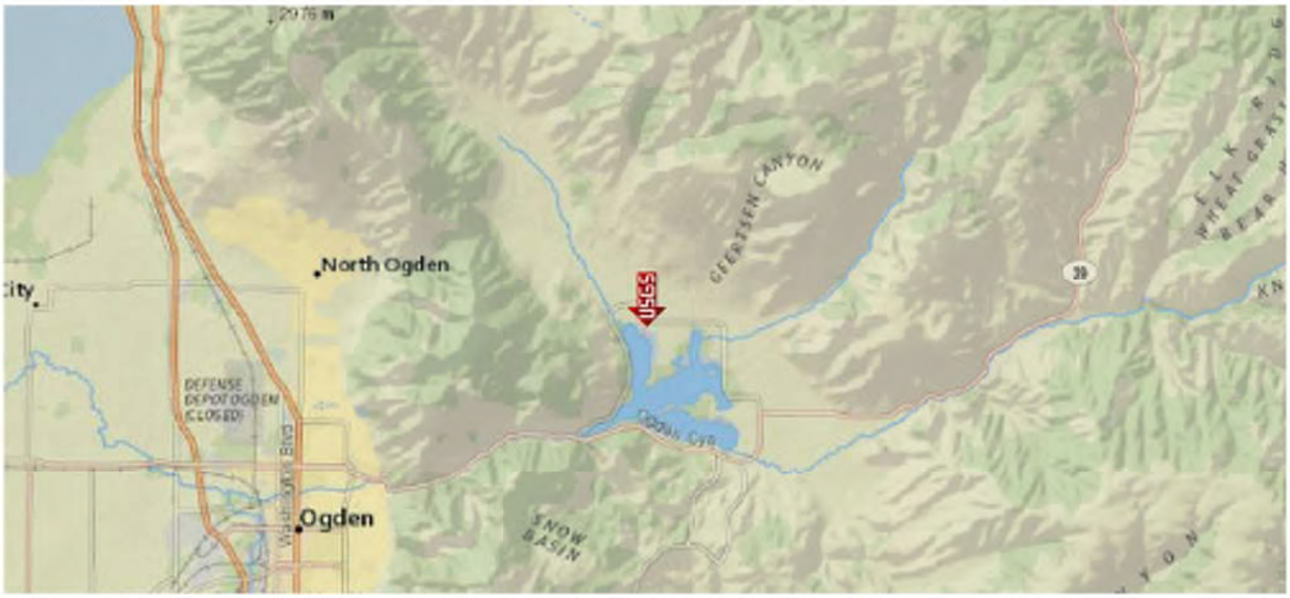
Report Title Summit Horizon, Eden, UT
Fri March 25, 2016 18:16:11 UTC

Building Code Reference Document 2012 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 41.3007°N, 111.8127°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III

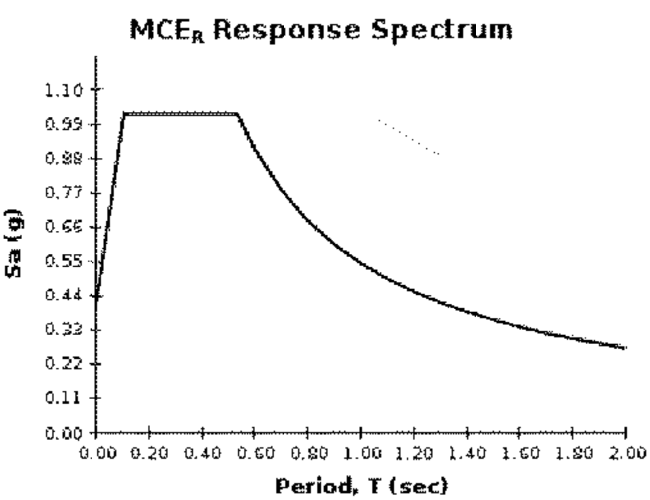


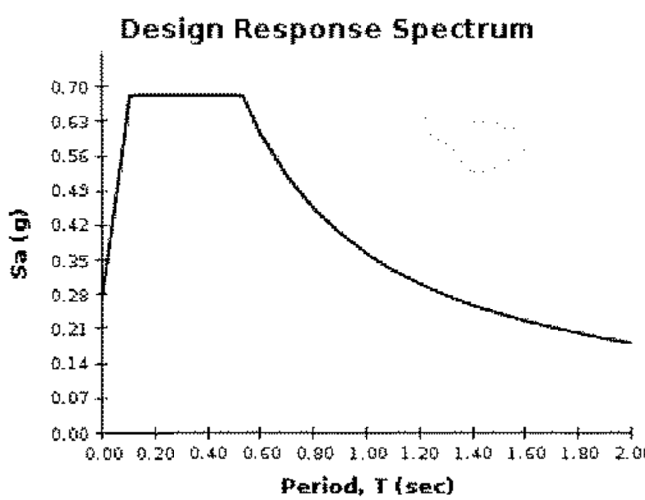
USGS–Provided Output

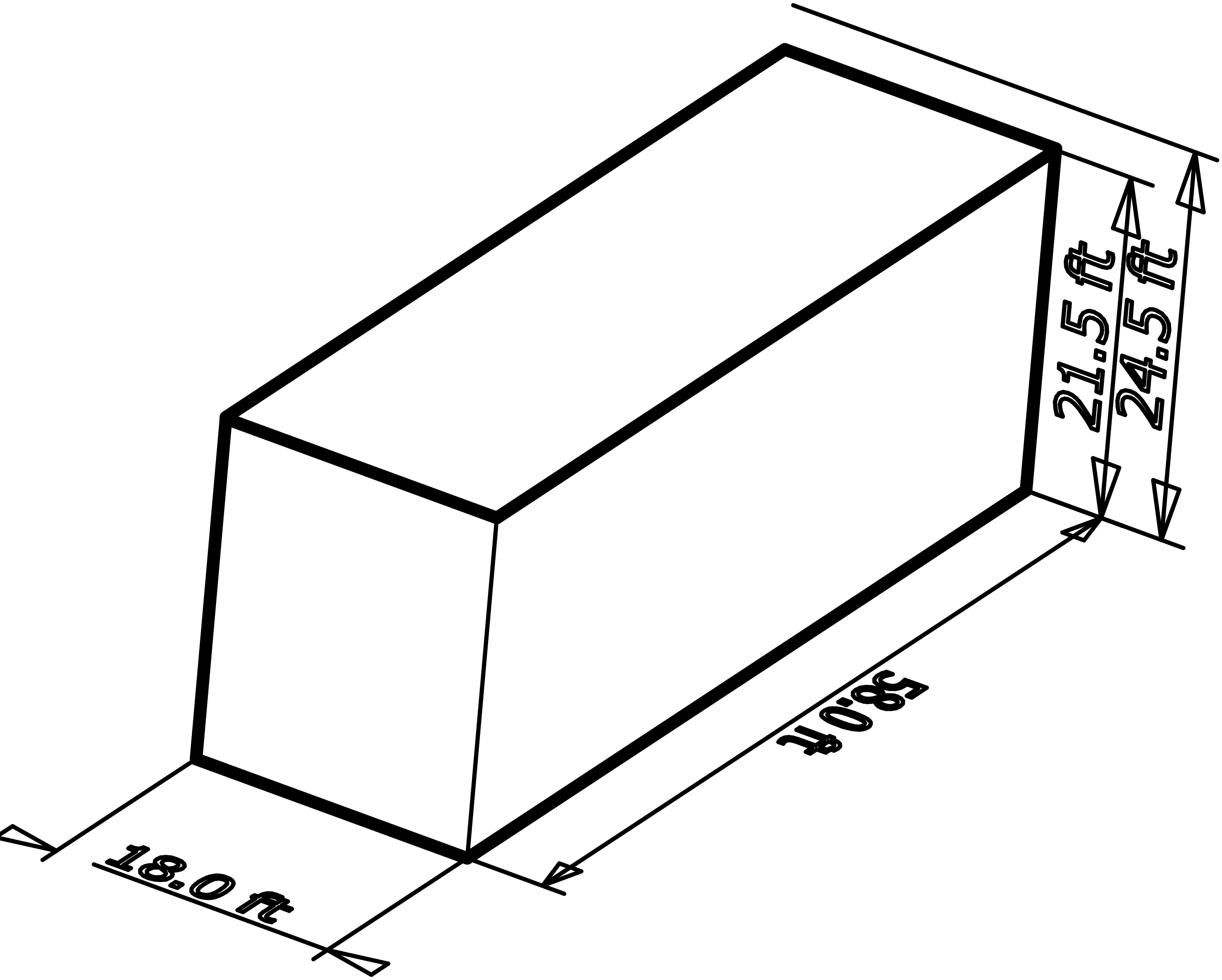
$S_S =$ 0.898 g $S_{MS} =$ 1.025 g $S_{DS} =$ 0.683 g

$S_1 =$ 0.304 g $S_{M1} =$ 0.545 g $S_{D1} =$ 0.363 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.






21.5 ft
24.5 ft
58.0 ft
18.0 ft

MecaWind Pro v2.2.7.6 per ASCE 7-10

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```
Date         : 6/1/2017                              Project No.    : JobNo
Company Name : True                                  Designed By    : Engineer
Address      : Address                               Description    : Description
City         : City                                  Customer Name  : Customer
State        : State                                 Proj Location  : Location
File Location: C:\Users\mikesullaway\AppData\Roaming\MecaWind\Default.wnd
```

Directional Procedure Simplified Diaphragm Building (Ch 27 Part 2)

```
     Basic Wind Speed(V)       =  115.00 mph
     Structural Category       =     II          Exposure Category       =      C
     Natural Frequency         =   N/A           Flexible Structure      =     No
     Importance Factor         =    1.00         Kd Directional Factor   =   0.85
     Alpha                     =    9.50         Zg                      =  900.00 ft
     At                        =    0.11         Bt                      =   1.00
     Am                        =    0.15         Bm                      =   0.65
     Cc                        =    0.20         l                       =  500.00 ft
     Epsilon                   =    0.20         Zmin                    =  15.00 ft
     Pitch of Roof             =  0 : 12         Slope of Roof(Theta)    =    .00 Deg
     h: Mean Roof Ht           =   21.50 ft      Type of Roof            = FLAT
     RHt: Ridge Ht             =   21.50 ft      Eht: Eave Height        =   21.50 ft
     OH: Roof Overhang at Eave=     .00 ft       Overhead Type           = Overhang
     Bldg Length Along Ridge   =   58.00 ft      Bldg Width Across Ridge=   18.00 ft
```

Gust Factor Calculations

```
     Gust Factor Category I Rigid Structures - Simplified Method
     Gust1:  For Rigid Structures (Nat. Freq.>1 Hz) use 0.85          =  0.85

     Gust Factor Category II Rigid Structures - Complete Analysis
     Zm:    0.6*Ht                                                    =  15.00 ft
     lzm:   Cc*(33/Zm)^0.167                                          =   0.23
     Lzm:   l*(Zm/33)^Epsilon                                         = 427.06 ft
     Q:     (1/(1+0.63*((B+Ht)/Lzm)^0.63))^0.5                        =   0.94
     Gust2: 0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))                 =   0.89

     Gust Factor Summary
     Not a Flexible Structure use the Lessor of Gust1 or Gust2        =   0.85
```

Table 26.11-1 Internal Pressure Coefficients for Buildings, GCpi

```
     GCPi   : Internal Pressure Coefficient                           = +/-0.18
```

Topographic Adjustment

```
     0.33*z                                                           =    1.00
     Kzt (0.33*z): Topographic factor at elevation 0.33*z             =    1.00
     Vtopo: Adjust V per Para 27.5.2: V * [Kzt(0.33*z)]^0.5           =   115.00 mph
```

MWFRS Diaphragm Building Wind Pressures per Ch 27 Pt 2

All pressures shown are based upon STRENGTH Design, with a Load Factor of 1

MWFRS Pressures for Wind Normal to 58 ft wall (Normal to Ridge)

```
     WALL PRESSURES PER TABLE 27.6-1
     L/B: Bldg Dim in Wind Dir / Bldg Dim Normal to Wind Dir          =    0.31
     h:   Height to top of Windward Wall                              =   21.50 ft
     ph:  Net Pressure at top of wall (windward + leeward)            =   28.99 psf
     p0:  Net Pressure at bottom of wall (windward + leeward)         =   28.47 psf

     ps:  Side wall pressure acting away from wall = .54 * ph         =  -15.65 psf
     pl:  Leeward wall pressure acting away from wall = .38 * ph      =  -11.02 psf
     pwh: Windward wall press @ top acting toward wall = ph-pl        =   17.97 psf
     pw0: Windward wall press @ bot acting toward wall = p0-pl        =   17.45 psf

     ROOF PRESSURES PER TABLE 27.6-2
     h:      Mean Roof Height                                         =   21.500 ft
     Lambda: Exposure Adjustment Factor                               =   1.000
     Slope:  Roof Slope                                               =   .00 Deg

     Any slope less than 9.46 Deg is treated as a 'Flat' roof per Table 27.6-2
```

Zone	Load Case1 psf	Load Case2 psf

1	.00	.00
2	.00	.00
3	-27.88	.00
4	-24.83	.00
5	-20.37	.00

Note: A value of '0' indicates that the zone/load case is not applicable.

ROOF OVERHANG LOADS (FIGURE 27.6-3):

LOAD CASE 1:
Povh1: Overhang pressure for zone 1 = .00 psf
Povh3: Overhang pressure for zone 3 = -20.91 psf

LOAD CASE 2:
Povh1: Overhang pressure for zone 1 = .00 psf
Povh3: Overhang pressure for zone 3 = .00 psf

Normal to Ridge - Base Reactions - Walls+Roof +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Roof (0 to h/2)	-27.88	624	.00	.00	17.38	63.0	.0	.0
Roof (h/2 to h)	-24.83	421	.00	.00	10.44	-56.1	.0	.0
Windward Wall	17.97	580	.00	10.42	.00	172.0	.0	.0
Windward Wall	17.73	580	.00	10.28	.00	66.8	.0	.0
Windward Wall	17.49	87	.00	1.52	.00	1.1	.0	.0
Leeward Wall	-11.02	1247	.00	13.74	.00	147.7	.0	.0
Side Wall	-15.65	387	-6.06	.00	.00	.0	65.1	.0
Side Wall	-15.65	387	6.06	.00	.00	.0	-65.1	.0
Total	.00	4312	.00	35.97	27.82	394.5	.0	.0

Normal to Ridge - Base Reactions - Walls Only +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	17.97	580	.00	10.42	.00	172.0	.0	.0
Windward Wall	17.73	580	.00	10.28	.00	66.8	.0	.0
Windward Wall	17.49	87	.00	1.52	.00	1.1	.0	.0
Leeward Wall	-11.02	1247	.00	13.74	.00	147.7	.0	.0
Side Wall	-15.65	387	-6.06	.00	.00	.0	65.1	.0
Side Wall	-15.65	387	6.06	.00	.00	.0	-65.1	.0
Total	.00	3268	.00	35.97	.00	387.7	.0	.0

Normal to Ridge - Base Reactions - Walls+Roof -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	17.97	580	.00	10.42	.00	172.0	.0	.0
Windward Wall	17.73	580	.00	10.28	.00	66.8	.0	.0
Windward Wall	17.49	87	.00	1.52	.00	1.1	.0	.0
Leeward Wall	-11.02	1247	.00	13.74	.00	147.7	.0	.0
Side Wall	-15.65	387	-6.06	.00	.00	.0	65.1	.0
Side Wall	-15.65	387	6.06	.00	.00	.0	-65.1	.0
Total	.00	3268	.00	35.97	.00	387.7	.0	.0

Normal to Ridge - Base Reactions - Walls Only -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	17.97	580	.00	10.42	.00	172.0	.0	.0
Windward Wall	17.73	580	.00	10.28	.00	66.8	.0	.0
Windward Wall	17.49	87	.00	1.52	.00	1.1	.0	.0
Leeward Wall	-11.02	1247	.00	13.74	.00	147.7	.0	.0
Side Wall	-15.65	387	-6.06	.00	.00	.0	65.1	.0
Side Wall	-15.65	387	6.06	.00	.00	.0	-65.1	.0
Total	.00	3268	.00	35.97	.00	387.7	.0	.0

Normal to Ridge - Base Reactions - Walls+Roof MIN

Description	Press psf	Area* ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Total	.00	0	.00	.00	.00	.0	.0	.0

Notes - Normal to Ridge

Note (1) X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical
Note (2) MIN = Minimum pressures on Walls = 16 psf and Roof = 8 psf
Note (3) Area* = Area of the surface projected onto a vertical plane normal to wind.

MWFRS Pressures for Wind Normal to 18 ft wall (Along Ridge)

WALL PRESSURES PER TABLE 27.6-1

L/B: Bldg Dim in Wind Dir / Bldg Dim Normal to Wind Dir	=	3.22
h: Height to top of Windward Wall	=	21.50 ft
ph: Net Pressure at top of wall (windward + leeward)	=	25.05 psf
p0: Net Pressure at bottom of wall (windward + leeward)	=	24.42 psf
ps: Side wall pressure acting away from wall = .64 * ph	=	-16.03 psf
pl: Leeward wall pressure acting away from wall = .27 * ph	=	-6.76 psf
pwh: Windward wall press @ top acting toward wall = ph-pl	=	18.28 psf
pw0: Windward wall press @ bot acting toward wall = p0-pl	=	17.66 psf

ROOF PRESSURES PER TABLE 27.6-2

h: Mean Roof Height	=	21.500 ft
Lambda: Exposure Adjustment Factor	=	1.000
Slope: Roof Slope	=	.00 Deg

Any slope less than 9.46 Deg is treated as a 'Flat' roof per Table 27.6-2

Zone	Load Case1 psf	Load Case2 psf
1	.00	.00
2	.00	.00
3	-27.88	.00
4	-24.83	.00
5	-20.37	.00

Note: A value of '0' indicates that the zone/load case is not applicable.

ROOF OVERHANG LOADS (FIGURE 27.6-3):

LOAD CASE 1:

Povh1: Overhang pressure for zone 1	=	.00 psf
Povh3: Overhang pressure for zone 3	=	-20.91 psf

LOAD CASE 2:

Povh1: Overhang pressure for zone 1	=	.00 psf
Povh3: Overhang pressure for zone 3	=	.00 psf

Along Ridge - Base Reactions - Walls+Roof +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Roof (0 to h/2)	-27.88	194	.00	.00	5.39	.0	-127.4	.0
Roof (h/2 to h)	-24.83	194	.00	.00	4.80	.0	-61.9	.0
Roof (h to 2h)	-20.37	387	.00	.00	7.88	.0	25.6	.0
Roof (>2h)	-20.37	270	.00	.00	5.50	.0	118.2	.0
Windward Wall	18.28	180	3.29	.00	.00	.0	-54.3	.0
Windward Wall	17.99	180	3.24	.00	.00	.0	-21.1	.0
Windward Wall	17.70	27	0.48	.00	.00	.0	-0.4	.0
Leeward Wall	-6.76	387	2.62	.00	.00	.0	-28.1	.0
Side Wall	-16.03	1247	.00	19.99	.00	214.9	.0	.0
Side Wall	-16.03	1247	.00	-19.99	.00	-214.9	.0	.0
Total	.00	4312	9.62	.00	23.58	.0	-149.3	.0

Along Ridge - Base Reactions - Walls Only +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	18.28	180	3.29	.00	.00	.0	-54.3	.0
Windward Wall	17.99	180	3.24	.00	.00	.0	-21.1	.0
Windward Wall	17.70	27	0.48	.00	.00	.0	-0.4	.0
Leeward Wall	-6.76	387	2.62	.00	.00	.0	-28.1	.0
Side Wall	-16.03	1247	.00	19.99	.00	214.9	.0	.0
Side Wall	-16.03	1247	.00	-19.99	.00	-214.9	.0	.0
Total	.00	3268	9.62	.00	.00	.0	-103.8	.0

Along Ridge - Base Reactions - Walls+Roof -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	18.28	180	3.29	.00	.00	.0	-54.3	.0
Windward Wall	17.99	180	3.24	.00	.00	.0	-21.1	.0
Windward Wall	17.70	27	0.48	.00	.00	.0	-0.4	.0
Leeward Wall	-6.76	387	2.62	.00	.00	.0	-28.1	.0
Side Wall	-16.03	1247	.00	19.99	.00	214.9	.0	.0
Side Wall	-16.03	1247	.00	-19.99	.00	-214.9	.0	.0
Total	.00	3268	9.62	.00	.00	.0	-103.8	.0

Along Ridge - Base Reactions - Walls Only -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	18.28	180	3.29	.00	.00	.0	-54.3	.0
Windward Wall	17.99	180	3.24	.00	.00	.0	-21.1	.0
Windward Wall	17.70	27	0.48	.00	.00	.0	-0.4	.0
Leeward Wall	-6.76	387	2.62	.00	.00	.0	-28.1	.0
Side Wall	-16.03	1247	.00	19.99	.00	214.9	.0	.0
Side Wall	-16.03	1247	.00	-19.99	.00	-214.9	.0	.0
Total	.00	3268	9.62	.00	.00	.0	-103.8	.0

Along Ridge - Base Reactions - Walls+Roof MIN

Description	Press psf	Area* ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Total	.00	0	.00	.00	.00	.0	.0	.0

Notes - Along Ridge

Note (1) X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical
Note (2) MIN = Minimum pressures on Walls = 16 psf and Roof = 8 psf
Note (3) Area* = Area of the surface projected onto a vertical plane normal to wind.

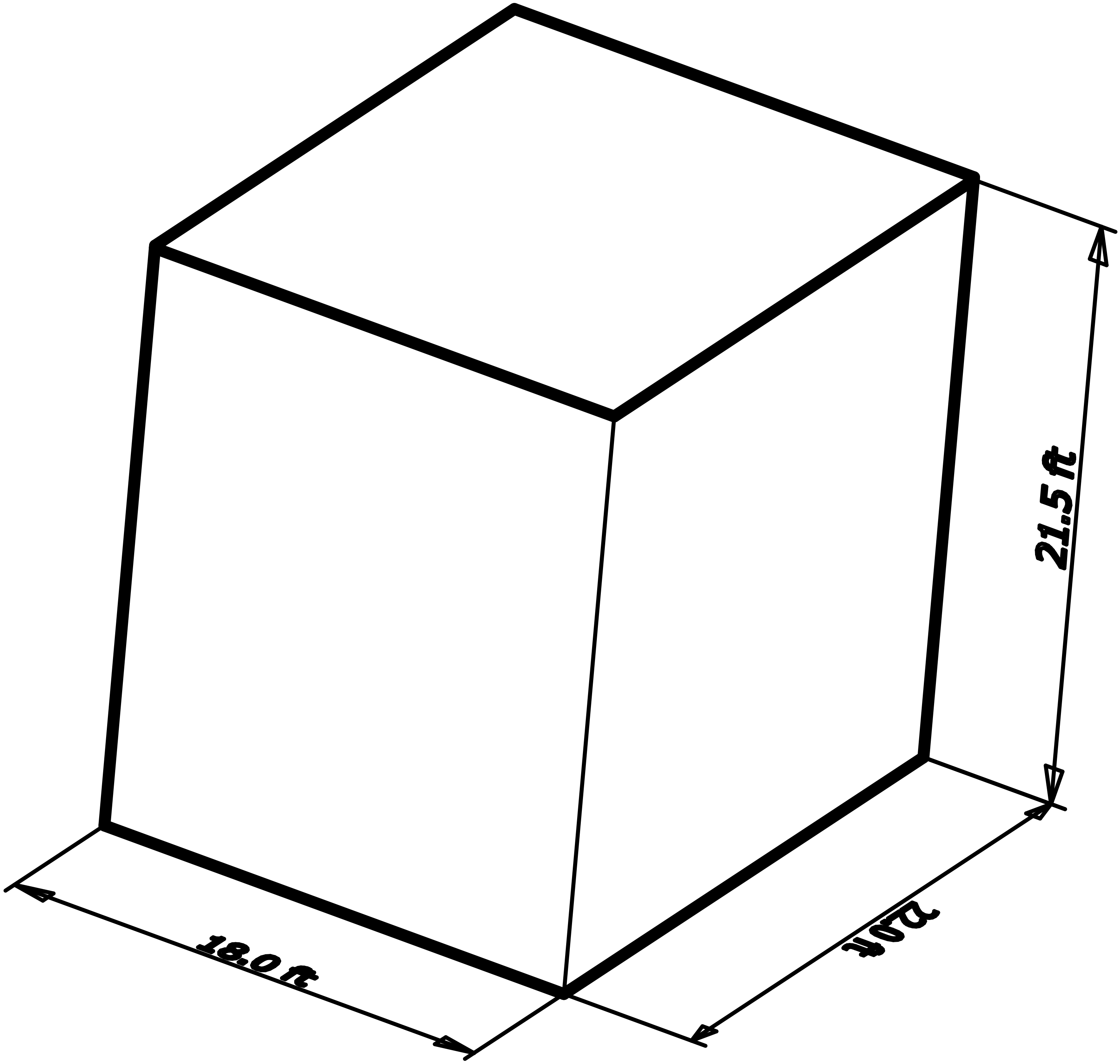
Total Base Reaction Summary

Description	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Normal to Ridge Walls+Roof +GCpi	.0	36.0	27.8	394.5	.0	.0
Normal to Ridge Walls Only +GCpi	.0	36.0	.0	387.7	.0	.0
Normal to Ridge Walls+Roof -GCpi	.0	36.0	.0	387.7	.0	.0
Normal to Ridge Walls Only -GCpi	.0	36.0	.0	387.7	.0	.0
Normal to Ridge Walls+Roof MIN	.0	.0	.0	.0	.0	.0
Along Ridge Walls+Roof +GCpi	9.6	.0	23.6	.0	-149.3	.0
Along Ridge Walls Only +GCpi	9.6	.0	.0	.0	-103.8	.0
Along Ridge Walls+Roof -GCpi	9.6	.0	.0	.0	-103.8	.0
Along Ridge Walls Only -GCpi	9.6	.0	.0	.0	-103.8	.0
Along Ridge Walls+Roof MIN	.0	.0	.0	.0	.0	.0

Notes Applying to MWFRS Reactions:

Note (1) Per Fig 27.4-1, Note 9, Use greater of Shear calculated with or without roof.
Note (2) X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical
Note (3) MIN = Minimum pressures on Walls = 16 psf and Roof = 8 psf
Note (4) MIN area is the area of the surface onto a vertical plane normal to wind.
Note (5) Total Roof Area (incl OH Top) = 1044.00 sq. ft

Note (6) LC = Load Case (Some pressures can be zero, ref ASCE 7-10 Ch 27 Pt 2)


21.5 ft
18.0 ft
2.0 ft

MecaWind Pro v2.2.7.6 per ASCE 7-10

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```
Date          : 6/1/2017                      Project No.    : JobNo
Company Name  : True                          Designed By    : Engineer
Address       : Address                       Description    : Description
City          : City                          Customer Name  : Customer
State         : State                         Proj Location  : Location
File Location: C:\Users\mikesullaway\SharePoint\Sullaway Engineering\Sullaway - Documents\Projects
\14600\14663\Load Report\meca 18x22.wnd
```

Directional Procedure Simplified Diaphragm Building (Ch 27 Part 2)

```
     Basic Wind Speed(V)        =  115.00 mph
     Structural Category        =      II         Exposure Category      =      C
     Natural Frequency          =    N/A          Flexible Structure     =     No
     Importance Factor          =     1.00        Kd Directional Factor  =    0.85
     Alpha                      =     9.50        Zg                     =  900.00 ft
     At                         =     0.11        Bt                     =    1.00
     Am                         =     0.15        Bm                     =    0.65
     Cc                         =     0.20        l                      =  500.00 ft
     Epsilon                    =     0.20        Zmin                   =   15.00 ft
     Pitch of Roof              =  0 : 12         Slope of Roof(Theta)   =     .00 Deg
     h: Mean Roof Ht            =   21.50 ft      Type of Roof           = FLAT
     RHt: Ridge Ht              =   21.50 ft      Eht: Eave Height       =   21.50 ft
     OH: Roof Overhang at Eave=       .00 ft      Overhead Type          = OH w/ soffit
     Bldg Length Along Ridge    =   22.00 ft      Bldg Width Across Ridge=   18.00 ft
```

Gust Factor Calculations

```
     Gust Factor Category I Rigid Structures - Simplified Method
     Gust1:  For Rigid Structures (Nat. Freq.>1 Hz) use 0.85        =  0.85

     Gust Factor Category II Rigid Structures - Complete Analysis
     Zm:     0.6*Ht                                                 =  15.00 ft
     lzm:    Cc*(33/Zm)^0.167                                       =   0.23
     Lzm:    l*(Zm/33)^Epsilon                                      = 427.06 ft
     Q:      (1/(1+0.63*((B+Ht)/Lzm)^0.63))^0.5                     =   0.94
     Gust2: 0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))               =   0.89

     Gust Factor Summary
     Not a Flexible Structure use the Lessor of Gust1 or Gust2      =   0.85
```

Table 26.11-1 Internal Pressure Coefficients for Buildings, GCpi

```
     GCPi    : Internal Pressure Coefficient                        = +/-0.18
```

Topographic Adjustment

```
     0.33*z                                                         =    1.00
     Kzt (0.33*z): Topographic factor at elevation 0.33*z           =    1.00
     Vtopo: Adjust V per Para 27.5.2: V * [Kzt(0.33*z)]^0.5         =   115.00 mph
```

MWFRS Diaphragm Building Wind Pressures per Ch 27 Pt 2

All pressures shown are based upon STRENGTH Design, with a Load Factor of 1

MWFRS Pressures for Wind Normal to 22 ft wall (Normal to Ridge)

```
     WALL PRESSURES PER TABLE 27.6-1
     L/B: Bldg Dim in Wind Dir / Bldg Dim Normal to Wind Dir        =    0.82
     h:   Height to top of Windward Wall                            =   21.50 ft
     ph:  Net Pressure at top of wall (windward + leeward)          =   28.99 psf
     p0:  Net Pressure at bottom of wall (windward + leeward)       =   28.47 psf

     ps:  Side wall pressure acting away from wall = .54 * ph       =  -15.65 psf
     pl:  Leeward wall pressure acting away from wall = .38 * ph    =  -11.02 psf
     pwh: Windward wall press @ top acting toward wall = ph-pl      =   17.97 psf
     pw0: Windward wall press @ bot acting toward wall = p0-pl      =   17.45 psf

     ROOF PRESSURES PER TABLE 27.6-2
     h:      Mean Roof Height                                       =   21.500 ft
     Lambda: Exposure Adjustment Factor                             =    1.000
     Slope:  Roof Slope                                             =    .00 Deg

     Any slope less than 9.46 Deg is treated as a 'Flat' roof per Table 27.6-2
```

Zone	Load Case1 psf	Load Case2 psf

----	----------	----------
1	.00	.00
2	.00	.00
3	-27.88	.00
4	-24.83	.00
5	-20.37	.00

Note: A value of '0' indicates that the zone/load case is not applicable.

ROOF OVERHANG LOADS (FIGURE 27.6-3):

LOAD CASE 1:
Povh1: Overhang pressure for zone 1 = .00 psf
Povh3: Overhang pressure for zone 3 = -20.91 psf

LOAD CASE 2:
Povh1: Overhang pressure for zone 1 = .00 psf
Povh3: Overhang pressure for zone 3 = .00 psf

Normal to Ridge - Base Reactions - Walls+Roof +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Roof (0 to h/2)	-27.88	237	.00	.00	6.59	23.9	.0	.0
Roof (h/2 to h)	-24.83	160	.00	.00	3.96	-21.3	.0	.0
Windward Wall	17.97	220	.00	3.95	.00	65.2	.0	.0
Windward Wall	17.73	220	.00	3.90	.00	25.4	.0	.0
Windward Wall	17.49	33	.00	0.58	.00	0.4	.0	.0
Leeward Wall	-11.02	473	.00	5.21	.00	56.0	.0	.0
Side Wall	-15.65	387	-6.06	.00	.00	.0	65.1	.0
Side Wall	-15.65	387	6.06	.00	.00	.0	-65.1	.0
Total	.00	2116	.00	13.64	10.55	149.7	.0	.0

Normal to Ridge - Base Reactions - Walls Only +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	17.97	220	.00	3.95	.00	65.2	.0	.0
Windward Wall	17.73	220	.00	3.90	.00	25.4	.0	.0
Windward Wall	17.49	33	.00	0.58	.00	0.4	.0	.0
Leeward Wall	-11.02	473	.00	5.21	.00	56.0	.0	.0
Side Wall	-15.65	387	-6.06	.00	.00	.0	65.1	.0
Side Wall	-15.65	387	6.06	.00	.00	.0	-65.1	.0
Total	.00	1720	.00	13.64	.00	147.0	.0	.0

Normal to Ridge - Base Reactions - Walls+Roof -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	17.97	220	.00	3.95	.00	65.2	.0	.0
Windward Wall	17.73	220	.00	3.90	.00	25.4	.0	.0
Windward Wall	17.49	33	.00	0.58	.00	0.4	.0	.0
Leeward Wall	-11.02	473	.00	5.21	.00	56.0	.0	.0
Side Wall	-15.65	387	-6.06	.00	.00	.0	65.1	.0
Side Wall	-15.65	387	6.06	.00	.00	.0	-65.1	.0
Total	.00	1720	.00	13.64	.00	147.0	.0	.0

Normal to Ridge - Base Reactions - Walls Only -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	17.97	220	.00	3.95	.00	65.2	.0	.0
Windward Wall	17.73	220	.00	3.90	.00	25.4	.0	.0
Windward Wall	17.49	33	.00	0.58	.00	0.4	.0	.0
Leeward Wall	-11.02	473	.00	5.21	.00	56.0	.0	.0
Side Wall	-15.65	387	-6.06	.00	.00	.0	65.1	.0
Side Wall	-15.65	387	6.06	.00	.00	.0	-65.1	.0

Total	.00	1720	.00	13.64	.00	147.0	.0	.0

Normal to Ridge - Base Reactions - Walls+Roof MIN

Description	Press psf	Area* ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Total	.00	0	.00	.00	.00	.0	.0	.0

Notes - Normal to Ridge
Note (1) X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical
Note (2) MIN = Minimum pressures on Walls = 16 psf and Roof = 8 psf
Note (3) Area* = Area of the surface projected onto a vertical plane normal to wind.

MWFRS Pressures for Wind Normal to 18 ft wall (Along Ridge)

WALL PRESSURES PER TABLE 27.6-1

L/B: Bldg Dim in Wind Dir / Bldg Dim Normal to Wind Dir	=	1.22
h: Height to top of Windward Wall	=	21.50 ft
ph: Net Pressure at top of wall (windward + leeward)	=	28.11 psf
p0: Net Pressure at bottom of wall (windward + leeward)	=	27.57 psf
ps: Side wall pressure acting away from wall = .56 * ph	=	-15.81 psf
pl: Leeward wall pressure acting away from wall = .36 * ph	=	-10.00 psf
pwh: Windward wall press @ top acting toward wall = ph-pl	=	18.12 psf
pw0: Windward wall press @ bot acting toward wall = p0-pl	=	17.57 psf

ROOF PRESSURES PER TABLE 27.6-2

h: Mean Roof Height	=	21.500 ft
Lambda: Exposure Adjustment Factor	=	1.000
Slope: Roof Slope	=	.00 Deg

Any slope less than 9.46 Deg is treated as a 'Flat' roof per Table 27.6-2

Zone	Load Case1 psf	Load Case2 psf
1	.00	.00
2	.00	.00
3	-27.88	.00
4	-24.83	.00
5	-20.37	.00

Note: A value of '0' indicates that the zone/load case is not applicable.

ROOF OVERHANG LOADS (FIGURE 27.6-3):

LOAD CASE 1:

Povh1: Overhang pressure for zone 1	=	.00 psf
Povh3: Overhang pressure for zone 3	=	-20.91 psf

LOAD CASE 2:

Povh1: Overhang pressure for zone 1	=	.00 psf
Povh3: Overhang pressure for zone 3	=	.00 psf

Along Ridge - Base Reactions - Walls+Roof +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Roof (0 to h/2)	-27.88	194	.00	.00	5.39	.0	-30.3	.0
Roof (h/2 to h)	-24.83	194	.00	.00	4.80	.0	24.6	.0
Roof (h to 2h)	-20.37	9	.00	.00	0.18	.0	2.0	.0
Windward Wall	18.12	180	3.26	.00	.00	.0	-53.8	.0
Windward Wall	17.86	180	3.22	.00	.00	.0	-20.9	.0
Windward Wall	17.61	27	0.48	.00	.00	.0	-0.4	.0
Leeward Wall	-10.00	387	3.87	.00	.00	.0	-41.6	.0
Side Wall	-15.81	473	.00	7.48	.00	80.4	.0	.0
Side Wall	-15.81	473	.00	-7.48	.00	-80.4	.0	.0
Total	.00	2116	10.82	.00	10.38	.0	-120.4	.0

Along Ridge - Base Reactions - Walls Only +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	18.12	180	3.26	.00	.00	.0	-53.8	.0
Windward Wall	17.86	180	3.22	.00	.00	.0	-20.9	.0
Windward Wall	17.61	27	0.48	.00	.00	.0	-0.4	.0
Leeward Wall	-10.00	387	3.87	.00	.00	.0	-41.6	.0
Side Wall	-15.81	473	.00	7.48	.00	80.4	.0	.0
Side Wall	-15.81	473	.00	-7.48	.00	-80.4	.0	.0
Total	.00	1720	10.82	.00	.00	.0	-116.7	.0

Along Ridge - Base Reactions - Walls+Roof -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	18.12	180	3.26	.00	.00	.0	-53.8	.0
Windward Wall	17.86	180	3.22	.00	.00	.0	-20.9	.0
Windward Wall	17.61	27	0.48	.00	.00	.0	-0.4	.0
Leeward Wall	-10.00	387	3.87	.00	.00	.0	-41.6	.0
Side Wall	-15.81	473	.00	7.48	.00	80.4	.0	.0
Side Wall	-15.81	473	.00	-7.48	.00	-80.4	.0	.0
Total	.00	1720	10.82	.00	.00	.0	-116.7	.0

Along Ridge - Base Reactions - Walls Only -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	18.12	180	3.26	.00	.00	.0	-53.8	.0
Windward Wall	17.86	180	3.22	.00	.00	.0	-20.9	.0
Windward Wall	17.61	27	0.48	.00	.00	.0	-0.4	.0
Leeward Wall	-10.00	387	3.87	.00	.00	.0	-41.6	.0
Side Wall	-15.81	473	.00	7.48	.00	80.4	.0	.0
Side Wall	-15.81	473	.00	-7.48	.00	-80.4	.0	.0
Total	.00	1720	10.82	.00	.00	.0	-116.7	.0

Along Ridge - Base Reactions - Walls+Roof MIN

Description	Press psf	Area* ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Total	.00	0	.00	.00	.00	.0	.0	.0

Notes - Along Ridge

Note (1) X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical
Note (2) MIN = Minimum pressures on Walls = 16 psf and Roof = 8 psf
Note (3) Area* = Area of the surface projected onto a vertical plane normal to wind.

Total Base Reaction Summary

Description	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Normal to Ridge Walls+Roof +GCpi	.0	13.6	10.6	149.7	.0	.0
Normal to Ridge Walls Only +GCpi	.0	13.6	.0	147.0	.0	.0
Normal to Ridge Walls+Roof -GCpi	.0	13.6	.0	147.0	.0	.0
Normal to Ridge Walls Only -GCpi	.0	13.6	.0	147.0	.0	.0
Normal to Ridge Walls+Roof MIN	.0	.0	.0	.0	.0	.0
Along Ridge Walls+Roof +GCpi	10.8	.0	10.4	.0	-120.4	.0
Along Ridge Walls Only +GCpi	10.8	.0	.0	.0	-116.7	.0
Along Ridge Walls+Roof -GCpi	10.8	.0	.0	.0	-116.7	.0
Along Ridge Walls Only -GCpi	10.8	.0	.0	.0	-116.7	.0
Along Ridge Walls+Roof MIN	.0	.0	.0	.0	.0	.0

Notes Applying to MWFRS Reactions:

Note (1) Per Fig 27.4-1, Note 9, Use greater of Shear calculated with or without roof.
Note (2) X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical
Note (3) MIN = Minimum pressures on Walls = 16 psf and Roof = 8 psf
Note (4) MIN area is the area of the surface onto a vertical plane normal to wind.
Note (5) Total Roof Area (incl OH Top) = 396.00 sq. ft

Note (6) LC = Load Case (Some pressures can be zero, ref ASCE 7-10 Ch 27 Pt 2)

Blackwell
Structural Engineers

APPENDIX B - TYPICAL STEEL CONNECTION DESIGN

PROBLEM: WE NEED TO DESIGN CONNECTIONS FOR VARIOUS STRUCTURAL STEEL CONNECTIONS.

IN DISCUSSIONS W/ AVB, WE REQUIRE SINGLE TAB CONNECTIONS FOR BEAMS UP TO AND INCLUDING W18 [W460]. FOLLOW THE LFRD APPROACH IN AISC. THE DESIGN IS BASED ON AISC TABLE 10-10a WHICH HAS BEEN PROVIDED WITHIN THIS PACKAGE.

STEEL WF BEAM TO HSS SHEAR TAB CONNECTIONS.

→ FROM AISC: HSS WALL SLENDERNESS IS GIVEN AS:

$$\frac{(B-3t)}{t} \leq 1.40\sqrt{\frac{E}{F_y}} \qquad * \; t = 0.225 \text{ in.}$$

$$B \leq \left[1.40\sqrt{\frac{E}{F_y}}\right](t) + 3t$$

$$B \leq \left[1.40\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}}\right](0.225 \text{ in}) + 3(0.225 \text{ in})$$

$$B \leq 8.26 \text{ in.}$$

∴ HSS SECTIONS MUST BE LESS THAN 8"

SO SLENDERNESS CAN BE NEGLECTED!

NOTE, AVB STATED THAT COLUMNS WILL NOT BE LARGER THAN HSS6"×6"×1/4" THUS:

$$\frac{6 \text{ in} - 3(0.225 \text{ in})}{0.225 \text{ in}} = 23.6 \quad \underline{\text{OK!}}$$

DUCTILITY IS ALSO OKAY PER TABLE K1.2A.

→ MAXIMUM SINGLE PL THICKNESS:

$$t_{PL} \leq \frac{F_u}{F_{yp}}(t) = \frac{(62\,ksi)}{36\,ksi}(0.225\,in)$$

$$t_{PL} \leq 0.3875\,in\ [9.8mm]$$

↳ WE ARE USING 5/16 in [8.0mm] OK!

SHEAR TAB CONNECTION DETAILS

→ SIMPLE SHEAR TAB CONNECTIONS ARE OUTLINED ON THE FOLLOWING SHEETS AND IS RE-PRODUCED FROM AISC

→ SINCE CONNECTIONS ARE GOVERNED BY THE SHEAR TAB, LIMIT THE MAX SHEAR FORCE IN THE MEMBER CONNECTION SCHEDULE.

→ BEAM COPE CHECKS ARE TAKEN FROM TABLES 9-3b AND 9-3c FROM AISC AND SUMMARIZED IN THE SPREADSHEET.

E	29000 ksi
(Fy)b	50 ksi
(Fu)b	65 ksi
(Fy)pl	36 ksi
Bolts	3/4" - A325
Weld	1/4"

Shear Tab		Cope Checks	
Plate	5/16 in	Shear Yield	Rupture
Bolts	Vr [kips]		
5	81.3	309	287
4	62.5	242	225
3	43.4	174	163
2	24.8	107	101

Section	d	tw	bf	tf	k	gauge	Vr [kip]	# of Bolts	Vr [kips]	Shear Yield	Shear Rupture	Vr (max) [kips]
W18x71	18.5	0.495	7.64	0.81	1.21	3.5	**275**	5	81.3	153.0	142.1	**81.3**
x65	18.4	0.45	7.59	0.75	1.15	3.5	**248**	5	81.3	139.1	129.2	**81.3**
x60	18.2	0.415	7.56	0.695	1.1	3.5	**227**	5	81.3	128.2	119.1	**81.3**
x55	18.1	0.39	7.53	0.63	1.03	3.5	**212**	5	81.3	120.5	111.9	**81.3**
x50	18	0.355	7.5	0.57	0.972	3.5	**192**	5	81.3	109.7	101.9	**81.3**
W18x46	18.1	0.36	6.06	0.605	1.01	3.5	**195**	5	81.3	111.2	103.3	**81.3**
x40	17.9	0.315	6.02	0.525	0.927	3.5	**169**	5	81.3	97.3	90.4	**81.3**
x35	17.7	0.3	6	0.425	0.827	3.5	**159**	5	81.3	92.7	86.1	**81.3**
W16x57	16.4	0.43	7.12	0.715	1.12	3.5	**212**	4	62.5	104.1	96.8	**62.5**
x50	16.3	0.38	7.07	0.63	1.03	3.5	**186**	4	62.5	92.0	85.5	**62.5**
x45	16.1	0.345	7.04	0.565	0.967	3.5	**167**	4	62.5	83.5	77.6	**62.5**
x40	16	0.305	7	0.505	0.907	3.5	**146**	4	62.5	73.8	68.6	**62.5**
x36	15.9	0.295	6.99	0.43	0.832	3.5	**141**	4	62.5	71.4	66.4	**62.5**
W16x31	15.9	0.275	5.53	0.44	0.842	3.5	**131**	4	62.5	66.6	61.9	**61.9**
x26	15.7	0.25	5.5	0.345	0.747	3.5	**106**	4	62.5	60.5	56.3	**56.3**
W14x99	14.16	0.485	14.565	0.78	0.4375	5.5	**265**	3	43.4	84.4	79.1	**43.4**
W14x82	14.3	0.51	10.1	0.855	1.45	5.5	**219**	3	43.4	88.7	83.1	**43.4**
x74	14.2	0.45	10.1	0.785	1.38	5.5	**192**	3	43.4	78.3	73.4	**43.4**
x68	14	0.415	10	0.72	1.31	5.5	**174**	3	43.4	72.2	67.6	**43.4**
x61	13.9	0.375	10	0.645	1.24	5.5	**156**	3	43.4	65.3	61.1	**43.4**
W14x53	13.9	0.37	8.06	0.66	1.25	5.5	**154**	3	43.4	64.4	60.3	**43.4**
x48	13.8	0.34	8.03	0.595	1.19	5.5	**141**	3	43.4	59.2	55.4	**43.4**
x43	13.7	0.305	8	0.53	1.12	5.5	**125**	3	43.4	53.1	49.7	**43.4**
W14x38	14.1	0.31	6.77	0.515	0.915	3.5	**131**	3	43.4	53.9	50.5	**43.4**
x34	14	0.285	6.75	0.455	0.855	3.5	**120**	3	43.4	49.6	46.5	**43.4**
x30	13.8	0.27	6.73	0.385	0.785	3.5	**112**	3	43.4	47.0	44.0	**43.4**
W14x26	13.9	0.255	5.03	0.42	0.82	2.75	**106**	3	43.4	44.4	41.6	**41.6**
x22	13.7	0.23	5	0.335	0.735	2.75	**94.5**	3	43.4	40.0	37.5	**37.5**
W12x96	12.7	0.55	12.2	0.9	1.5	5.5	**210**	3	43.4	95.7	89.7	**43.4**
x87	12.5	0.515	12.1	0.81	1.41	5.5	**193**	3	43.4	89.6	83.9	**43.4**
x79	12.4	0.47	12.1	0.735	1.33	5.5	**175**	3	43.4	81.8	76.6	**43.4**
x72	12.3	0.43	12	0.67	1.27	5.5	**159**	3	43.4	74.8	70.1	**43.4**
x65	12.1	0.39	12	0.605	1.2	5.5	**142**	3	43.4	67.9	63.6	**43.4**
W12x58	12.2	0.36	10	0.64	1.24	5.5	**132**	3	43.4	62.6	58.7	**43.4**
x53	12.1	0.345	10	0.575	1.18	5.5	**125**	3	43.4	60.0	56.2	**43.4**
W12x50	12.2	0.37	8.08	0.64	1.14	5.5	**135**	3	43.4	64.4	60.3	**43.4**
x45	12.1	0.335	8.05	0.575	1.08	5.5	**122**	3	43.4	58.3	54.6	**43.4**
x40	11.9	0.295	8.01	0.515	1.02	5.5	**105**	3	43.4	51.3	48.1	**43.4**
W12x35	12.5	0.3	6.56	0.52	0.82	3.5	**113**	3	43.4	52.2	48.9	**43.4**
x30	12.3	0.26	6.52	0.44	0.74	3.5	**95.9**	3	43.4	45.2	42.4	**42.4**
x26	12.2	0.23	6.49	0.38	0.68	3.5	**84.2**	3	43.4	40.0	37.5	**37.5**
W12x22	12.3	0.26	4.03	0.425	0.725	2.25	**95.9**	3	43.4	45.2	42.4	**42.4**
x19	12.2	0.235	4.01	0.35	0.65	2.25	**86**	3	43.4	40.9	38.3	**38.3**
x16	12	0.22	3.99	0.265	0.565	2.25	**79.2**	3	43.4	38.3	35.9	**35.9**
x14	11.9	0.2	3.97	0.225	0.525	2.25	**64.3**	3	43.4	34.8	32.6	**32.6**
W10x45	10.1	0.35	8.08	0.62	1.12	5.5	**106**	2	24.8	37.5	35.4	**24.8**
x39	9.92	0.315	7.99	0.53	1.03	5.5	**93.7**	2	24.8	33.7	31.8	**24.8**
x33	9.73	0.29	7.96	0.435	0.935	5.5	**84.7**	2	24.8	31.0	29.3	**24.8**
W10x30	10.5	0.3	5.81	0.51	0.81	2.75	**94.5**	2	24.8	32.1	30.3	**24.8**
x26	10.3	0.26	5.77	0.44	0.74	2.75	**80.3**	2	24.8	27.8	26.3	**24.8**
x22	10.2	0.24	5.75	0.36	0.66	2.75	**73.4**	2	24.8	25.7	24.2	**24.2**
W10x19	10.2	0.25	4.02	0.395	0.695	2.25	**76.5**	2	24.8	26.8	25.3	**24.8**
x17	10.1	0.24	4.01	0.33	0.63	2.25	**72.7**	2	24.8	25.7	24.2	**24.2**
x15	9.99	0.23	4	0.27	0.57	2.25	**68.9**	2	24.8	24.6	23.2	**23.2**
x12	9.87	0.19	3.96	0.21	0.51	2.25	**56.3**	2	24.8	20.3	19.2	**19.2**
W8x67	9	0.57	8.28	0.935	1.33	5.5	**154**	2	24.8	61.0	57.6	**24.8**
x58	8.75	0.51	8.22	0.81	1.22	5.5	**134**	2	24.8	54.6	51.5	**24.8**
x48	8.5	0.4	8.11	0.685	1.08	5.5	**102**	2	24.8	42.8	40.4	**24.8**
x40	8.25	0.36	8.07	0.56	0.954	5.5	**89.1**	2	24.8	38.5	36.4	**24.8**
x35	8.12	0.31	8.02	0.495	0.889	5.5	**75.5**	2	24.8	33.2	31.3	**24.8**
x31	8	0.285	8	0.435	0.829	5.5	**68.4**	2	24.8	30.5	28.8	**24.8**
W8x28	8.06	0.285	6.54	0.465	0.859	4	**68.9**	2	24.8	30.5	28.8	**24.8**
x24	7.93	0.245	6.5	0.4	0.794	4	**58.3**	2	24.8	26.2	24.7	**24.7**
W8x21	8.28	0.25	5.27	0.4	0.7	2.75	**62.1**	2	24.8	26.8	25.3	**24.8**
x18	8.14	0.23	5.25	0.33	0.63	2.75	**56.2**	2	24.8	24.6	23.2	**23.2**
W8x15	8.11	0.245	4.02	0.315	0.615	2.25	**59.6**	2	24.8	26.2	24.7	**24.7**
x13	7.99	0.23	4	0.255	0.555	2.25	**55.1**	2	24.8	24.6	23.2	**23.2**
x10	7.89	0.17	3.94	0.205	0.505	2.25	**40.2**	2	24.8	18.2	17.2	**17.2**

A-04

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

Plate $F_y = 36$ ksi

$3/4$-in.-diameter bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
8 ($L = 23^1/_2$)	Group A	N	STD	67.8	102	84.7	127	—	—	—	—	—	—	—	—
			SSLT	67.1	101	83.9	126	90.8	137	90.8	137	—	—	—	—
		X	STD	67.8	102	84.7	127	—	—	—	—	—	—	—	—
			SSLT	67.1	101	83.9	126	101	151	114	172	—	—	—	—
	Group B	N	STD	67.8	102	84.7	127	—	—	—	—	—	—	—	—
			SSLT	67.1	101	83.9	126	101	151	114	172	—	—	—	—
		X	STD	67.8	102	84.7	127	—	—	—	—	—	—	—	—
			SSLT	67.1	101	83.9	126	101	151	117	176	—	—	—	—
7 ($L = 20^1/_2$)	Group A	N	STD	59.7	89.5	72.1	108	—	—	—	—	—	—	—	—
			SSLT	59.0	88.5	73.7	111	78.7	118	78.7	118	—	—	—	—
		X	STD	59.7	89.5	74.6	112	—	—	—	—	—	—	—	—
			SSLT	59.0	88.5	73.7	111	88.5	133	99.2	149	—	—	—	—
	Group B	N	STD	59.7	89.5	74.6	112	—	—	—	—	—	—	—	—
			SSLT	59.0	88.5	73.7	111	88.5	133	99.2	149	—	—	—	—
		X	STD	59.7	89.5	74.6	112	—	—	—	—	—	—	—	—
			SSLT	59.0	88.5	73.7	111	88.5	133	103	155	—	—	—	—
6 ($L = 17^1/_2$)	Group A	N	STD	51.6	77.4	59.3	89.1	—	—	—	—	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	66.5	100	66.5	100	—	—	—	—
		X	STD	51.6	77.4	64.5	96.7	—	—	—	—	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	76.3	115	83.8	126	—	—	—	—
	Group B	N	STD	51.6	77.4	64.5	96.7	—	—	—	—	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	76.3	115	83.8	126	—	—	—	—
		X	STD	51.6	77.4	64.5	96.7	—	—	—	—	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	76.3	115	89.1	134	—	—	—	—
5 ($L = 14^1/_2$)	Group A	N	STD	43.5	65.2	54.1	81.3	54.1	81.3	54.1	81.3	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	54.1	81.3	54.1	81.3	54.1	81.3	54.1	81.3
		X	STD	43.5	65.2	54.3	81.5	65.2	97.8	68.1	102	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	68.1	102	68.1	102	68.1	102
	Group B	N	STD	43.5	65.2	54.3	81.5	65.2	97.8	68.1	102	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	68.1	102	68.1	102	68.1	102
		X	STD	43.5	65.2	54.3	81.5	65.2	97.8	76.1	114	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	74.9	112	84.5	126	84.5	126
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes N = Threads included

A-05

Table 10-10a (continued)
Single-Plate Connections

3/4-in.-diameter bolts

Bolt, Weld and Single-Plate Available Strengths, kips

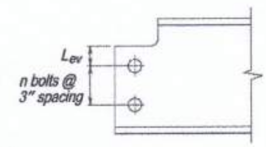
Plate F_y = 36 ksi

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
4 (L = 11 1/2)	Group A	N	STD	34.8	52.2	41.5	62.5	41.5	62.5	41.5	62.5	—	—	—	—
			SSLT	34.7	52.0	41.5	62.5	41.5	62.5	41.5	62.5	41.5	62.5	41.5	62.5
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	52.4	78.5	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	52.4	78.5	52.4	78.5	52.4	78.5
	Group B	N	STD	34.8	52.2	43.5	65.3	52.2	78.3	52.4	78.5	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	52.4	78.5	52.4	78.5	52.4	78.5
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	60.7	91.1	64.9	97.0	64.9	97.0
3 (L = 8 1/2)	Group A	N	STD	25.6	38.3	28.8	43.4	28.8	43.4	28.8	43.4	—	—	—	—
			SSLT	25.6	38.3	28.8	43.4	28.8	43.4	28.8	43.4	28.8	43.4	28.8	43.4
		X	STD	25.6	38.3	31.9	47.9	36.3	54.5	36.3	54.5	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	36.3	54.5	36.3	54.5	36.3	54.5	36.3	54.5
	Group B	N	STD	25.6	38.3	31.9	47.9	36.3	54.5	36.3	54.5	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	36.3	54.5	36.3	54.5	36.3	54.5	36.3	54.5
		X	STD	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	45.1	67.3	45.1	67.3
2 (L = 5 1/2)	Group A	N	STD	16.3	24.5	16.5	24.8	16.5	24.8	16.5	24.8	—	—	—	—
			SSLT	16.3	24.5	16.5	24.8	16.5	24.8	16.5	24.8	16.5	24.8	16.5	24.8
		X	STD	16.3	24.5	20.4	30.6	20.8	31.2	20.8	31.2	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	20.8	31.2	20.8	31.2	20.8	31.2	20.8	31.2
	Group B	N	STD	16.3	24.5	20.4	30.6	20.8	31.2	20.8	31.2	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	20.8	31.2	20.8	31.2	20.8	31.2	20.8	31.2
		X	STD	16.3	24.5	20.4	30.6	24.5	36.7	25.8	38.5	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	25.8	38.5	25.8	38.5	25.8	38.5
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes N = Threads included

A-06

Table 9-3b (continued)
Block Shear
Shear Yielding Component
per inch of thickness, kips/in.

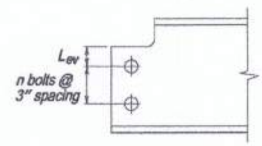


L_{ev}, in.	n	F_y, ksi				n	F_y, ksi			
		36		50			36		50	
		$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$	$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$		$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$	$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$
		ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD
1¼		175	263	244	366		78.3	117	109	163
1⅜		177	265	246	368		79.6	119	111	166
1½		178	267	248	371		81.0	121	113	169
1⅝		180	269	249	374		82.3	124	114	172
1¾		181	271	251	377		83.7	126	116	174
1⅞	6	182	273	253	380	3	85.0	128	118	177
2		184	275	255	383		86.4	130	120	180
2¼		186	279	259	388		89.1	134	124	186
2½		189	283	263	394		91.8	138	128	191
2¾		192	288	266	399		94.5	142	131	197
3		194	292	270	405		97.2	146	135	203
1¼		143	215	199	298		45.9	68.8	63.8	95.6
1⅜		144	217	201	301		47.2	70.9	65.6	98.4
1½		146	219	203	304		48.6	72.9	67.5	101
1⅝		147	221	204	307		49.9	74.9	69.4	104
1¾		148	223	206	309		51.3	76.9	71.3	107
1⅞	5	150	225	208	312	2	52.7	79.0	73.1	110
2		151	227	210	315		54.0	81.0	75.0	113
2¼		154	231	214	321		56.7	85.0	78.8	118
2½		157	235	218	326		59.4	89.1	82.5	124
2¾		159	239	221	332		62.1	93.1	86.3	129
3		162	243	225	338		64.8	97.2	90.0	135
1¼		111	166	154	231					
1⅜		112	168	156	233					
1½		113	170	158	236					
1⅝		115	172	159	239					
1¾		116	174	161	242					
1⅞	4	117	176	163	245					
2		119	178	165	248					
2¼		121	182	169	253					
2½		124	186	173	259					
2¾		127	190	176	264					
3		130	194	180	270					

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

A-07

Table 9-3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kips/in.

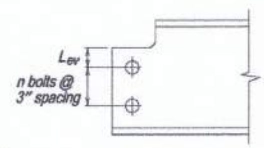


F_u, ksi		58						65					
		Bolt diameter, d, in.											
		$^3/_4$		$^7/_8$		1		$^3/_4$		$^7/_8$		1	
n	L_{ev}, in.	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	1¼	199	299	187	281	175	263	223	335	210	314	196	294
	1⅜	201	302	189	284	177	266	225	338	212	318	199	298
	1½	203	305	191	287	179	269	228	342	215	322	201	302
	1⅝	206	308	194	290	182	272	230	346	217	325	204	305
	1¾	208	312	196	294	184	276	233	349	219	329	206	309
	1⅞	210	315	198	297	186	279	235	353	222	333	208	313
	2	212	318	200	300	188	282	238	356	224	336	211	316
	2¼	216	325	204	307	192	289	243	364	229	344	216	324
	2½	221	331	209	313	197	295	247	371	234	351	221	331
	2¾	225	338	213	320	201	302	252	378	239	358	225	338
	3	229	344	217	326	206	308	257	386	244	366	230	346
5	1¼	162	243	152	228	142	214	182	272	171	256	160	239
	1⅜	164	246	154	232	145	217	184	276	173	260	162	243
	1½	166	250	157	235	147	220	186	280	176	263	165	247
	1⅝	169	253	159	238	149	223	189	283	178	267	167	250
	1¾	171	256	161	241	151	227	191	287	180	271	169	254
	1⅞	173	259	163	245	153	230	194	291	183	274	172	258
	2	175	263	165	248	156	233	196	294	185	278	174	261
	2¼	179	269	170	254	160	240	201	302	190	285	179	269
	2½	184	276	174	261	164	246	206	309	195	293	184	276
	2¾	188	282	178	268	169	253	211	316	200	300	189	283
	3	192	289	183	274	173	259	216	324	205	307	194	291
4	1¼	125	188	117	176	110	165	140	210	132	197	123	185
	1⅜	127	191	120	179	112	168	143	214	134	201	126	188
	1½	129	194	122	183	114	171	145	218	137	205	128	192
	1⅝	132	197	124	186	116	175	147	221	139	208	130	196
	1¾	134	201	126	189	119	178	150	225	141	212	133	199
	1⅞	136	204	128	192	121	181	152	229	144	216	135	203
	2	138	207	131	196	123	184	155	232	146	219	138	207
	2¼	142	214	135	202	127	191	160	239	151	227	143	214
	2½	147	220	139	209	132	197	165	247	156	234	147	221
	2¾	151	227	144	215	136	204	169	254	161	241	152	229
	3	156	233	148	222	140	210	174	261	166	249	157	236

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

A-08

Table 9-3c (continued)
Block Shear
Shear Rupture Component



per inch of thickness, kips/in.

F_u, ksi		58						65					
		Bolt diameter, d, in.											
		3/4		7/8		1		3/4		7/8		1	
n	L_{ev}, in.	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	1¼	88.1	132	82.6	124	77.2	116	98.7	148	92.6	139	86.5	130
	1⅜	90.3	135	84.8	127	79.4	119	101	152	95.1	143	89.0	133
	1½	92.4	139	87.0	131	81.6	122	104	155	97.5	146	91.4	137
	1⅝	94.6	142	89.2	134	83.7	126	106	159	99.9	150	93.8	141
	1¾	96.8	145	91.4	137	85.9	129	108	163	102	154	96.3	144
3	1⅞	99.0	148	93.5	140	88.1	132	111	166	105	157	98.7	148
	2	101	152	95.7	144	90.3	135	113	170	107	161	101	152
	2¼	105	158	100	150	94.6	142	118	177	112	168	106	159
	2½	110	165	104	157	99.0	148	123	185	117	176	111	166
	2¾	114	171	109	163	103	155	128	192	122	183	116	174
	3	119	178	113	170	108	161	133	199	127	190	121	181
	1¼	51.1	76.7	47.8	71.8	44.6	66.9	57.3	85.9	53.6	80.4	50.0	75.0
	1⅜	53.3	79.9	50.0	75.0	46.8	70.1	59.7	89.6	56.1	84.1	52.4	78.6
	1½	55.5	83.2	52.2	78.3	48.9	73.4	62.2	93.2	58.5	87.8	54.8	82.3
	1⅝	57.6	86.5	54.4	81.6	51.1	76.7	64.6	96.9	60.9	91.4	57.3	85.9
	1¾	59.8	89.7	56.6	84.8	53.3	79.9	67.0	101	63.4	95.1	59.7	89.6
2	1⅞	62.0	93.0	58.7	88.1	55.5	83.2	69.5	104	65.8	98.7	62.2	93.2
	2	64.2	96.2	60.9	91.4	57.6	86.5	71.9	108	68.3	102	64.6	96.9
	2¼	68.5	103	65.3	97.9	62.0	93.0	76.8	115	73.1	110	69.5	104
	2½	72.9	109	69.6	104	66.3	99.5	81.7	122	78.0	117	74.3	112
	2¾	77.2	116	73.9	111	70.7	106	86.5	130	82.9	124	79.2	119
	3	81.6	122	78.3	117	75.0	113	91.4	137	87.8	132	84.1	126

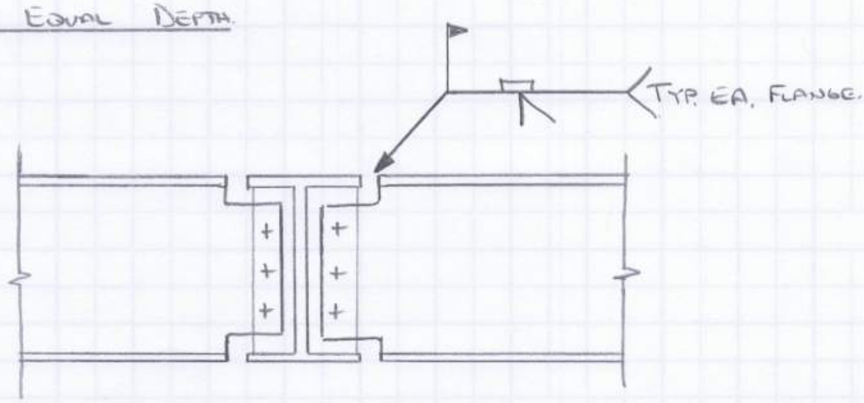
ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

PROBLEM: WE NEED TO DESIGN THROUGH-BEAM MOMENT CONNECTIONS. THERE ARE TWO SPECIFIC SITUATIONS THAT MUST BE INVESTIGATED:

1. BEAMS ARE EQUAL DEPTH
2. ONE BEAM IS SMALLER.

* REFER TO SECTION A OF THIS CALCULATION PACKAGE FOR THE SHEAR TAB DESIGN.

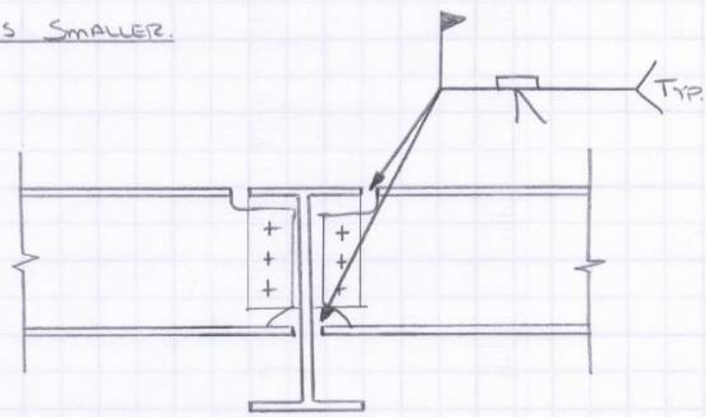
1. BEAMS ARE EQUAL DEPTH.



*NOTE, BY INSPECTION AND USING E70xx ELECTRODES w/ $F_U = 70$ ksi AND A CJP WELD THE FULL CAPACITY OF THE BEAM WILL BE AVAILABLE. THUS, USE CJP WELDS w/ BACKER BAR.

AGAIN, THE FULL SHEAR CAPACITY (SINGLE PL) IS PER THE DESIGN OUTLINED IN SECTION A.

2. ONE BEAM IS SMALLER.



SIMILAR, USING E70XX THE FULL CAPACITY OF THE BEAM WILL BE PROVIDED FOR FLEXURE. CONNECTION IS OKAY.

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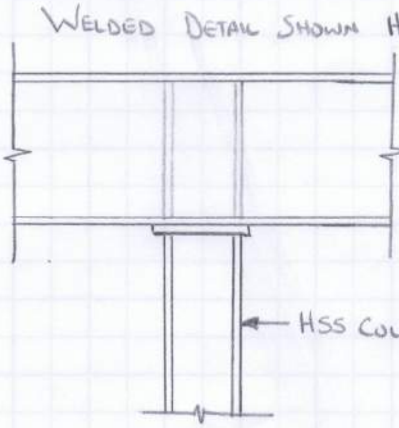
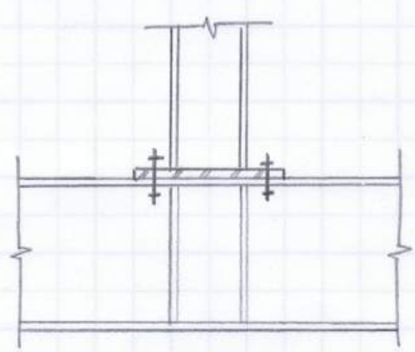
PROBLEM: WE HAVE BEEN ASKED TO DESIGN TYPICAL DETAILS FOR HSS COLUMNS WHICH ARE SUPPORTING STEEL WIDE FLANGE BEAMS AND FOR SITUATIONS WHERE HSS COLUMNS ARE SUPPORTED ON STEEL BEAMS. TO ENSURE WE COVER ALL THE BASIS, WE NEED TO CONSIDER ALL BEAMS + COLUMNS. WE WILL SETUP A SPREADSHEET TO COVER BEAMS AND COLUMNS TO SIMPLIFY THE DESIGN ASPECTS. DESIGN CONSIDERATIONS:

1. LOCAL WEB YIELDING
2. LOCAL WEB CRIPPLING
3. WEB SIDESWAY BUCKLING
4. WEB COMPRESSION BUCKLING
5. HSS CAP PLATE
6. USE OF STIFFENERS ?
7. GENERAL CONNECTION DETAILS (WELDS/BOLTS)
8. LATERAL STABILITY.

WELDED DETAIL SHOWN HERE



← HSS COLUMN



BOLTED CONNECTION SHOWN.

Seal	Title	Project #	Date
	TYPICAL STEEL CONNECTION DESIGN AS PER AISC	VARIOUS	2017/12/05
		Designer: DMV	Scale: NTS
	BEAM OVER HSS COLUMNS	Checked by:	Sheet #: C-01a

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FOR THE BASIS OF OUR DESIGN REVIEW/CHECKS WE WILL RUN THROUGH ONE BEAM/COLUMN REVIEW. AVB HAS ADVISED THAT COLUMNS WILL BE HSS3"x3" TO HSS6"x6" AND BEAMS WILL BE W8 THROUGH W16. FOR OUR REVIEW, USE:

BEAM: W14x38

COLUMN: HSS6"x6"x1/4"

THE SPREADSHEET WILL SUMMARIZE THE CAPACITIES AND CONNECTION DESIGN WILL BE BASED ON THE MINIMUM OF COLUMN, BEAM, PL CAPACITY.

*NOTE, THE DESIGN CHECKS/REVIEW FOR A STEEL BEAM OVER A HSS COLUMN AND A STEEL BEAM SUPPORTING A HSS COLUMN ARE THE SAME (ROTATED 180°). THUS CHECK FOR THE STEEL COLUMN SUPPORTING THE BEAM AND ROTATE THE DETAILS.

Seal	Title	Project #	Date
	TYPICAL STEEL CONNECTION DESIGN AS PER AISC	VARIOUS	2017/11/30
		Designer: DMN.	Scale: NTS.
	BEAM OVER HSS COLUMNS.	Checked by	Sheet #: C-02.

1. WEB LOCAL YIELDING

- AS A CONSERVATIVE MEASURE, ASSUME THE REACTION / FORCE OCCURS WITHIN 'd' OF THE SUPPORT.

$$R_N = \phi 2.50 k F_{yw} t_w + \phi l_b F_{yw} t_w$$

$$= (1.0)(2.50)(0.915\,in)(50\,ksi)(0.31\,in) + (1.0)(6.0\,in)(50\,ksi)(0.31\,in)$$

$$R_N = 128.5\ kip$$

2. WEB LOCAL CRIPPLING

- AGAIN, CONSIDER THE LOAD OCCURS WITHIN 'd' OF THE SUPPORT.

(i) FOR $l_b/d \leq 0.2$

$$R_N = \phi\left[0.40 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\right]\sqrt{\frac{E F_{yw} t_f}{t_w}}$$

(ii) FOR $l_b/d > 0.2$

$$R_N = \phi 0.40 t_w^2 \left[1 + \left(\frac{4 l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}}$$

* FOR W14×38 → $l_b/d = 0.426 > 0.2 \Rightarrow$ USE (ii)

$$R_N = (0.75)(0.40)(0.31\,in)^2\left[1 + \left(\frac{4(6.0\,in)}{14.1\,in} - 0.2\right)\left(\frac{0.31\,in}{0.515\,in}\right)^{1.5}\right]\sqrt{\frac{(29\,000\,ksi)(50\,ksi)(0.515)}{0.31}}$$

$$R_N = 76.1\ kip$$

Seal	Title	Project #	Date
	TYPICAL STEEL CONNECTION DESIGN AS PER AISC	VARIOUS	2017/11/30.
		Designer: DMV.	Scale: NTS.
	BEAM OVER HSS COLUMN.	Checked by	Sheet #: C-03.

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3. Web Sidesway Buckling

- This will generally not govern because the compression flange will be braced.
- By use of stiffeners + lateral bracing we can neglect this requirement.

* Use stiffeners + bracing.

4. Web Compression Buckling

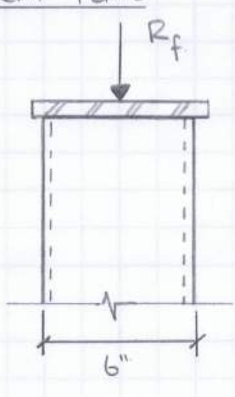
- Again, we will use transverse stiffeners.

$$R_N = \frac{\phi 24 t_w^3 \sqrt{E F_{yw}}}{h}$$

$$= \frac{(0.90)(24)(0.31in)^3 \sqrt{(29000ksi)(50ksi)}}{(14.10in - 2(0.515in))}$$

$$R_N = 59.3\ kN$$

5. HSS Cap Plate

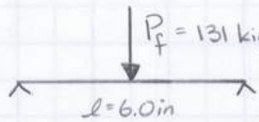


$$M_r = \phi Z_x F_y \rightarrow \text{PL Bending.}$$

* Note however, if we use a pair of transverse full height stiffeners on each flange we can neglect the plate design requirements because the loading is applied directly through the section.

→ Recall That $V_r = 131$ kip For A W14×38 (See Excel)

If We Take The HSS cap PL As A Simple Span Over The Column Flanges:



$$M_f = \frac{P_f l}{4} = \frac{(131\ kip)(6.0in)}{4}$$

$$M_f = 196.5\ kip\cdot in$$

$$\therefore Z_x \geq \frac{M_f}{\phi F_y} = \frac{196.5\ ksi}{(0.90)(36\ ksi)}$$

$$Z_x \geq 6.06\ in^3 \quad \text{Where} \quad Z_x = \tfrac{1}{4} b d^2 \rightarrow d \geq \sqrt{\frac{4Z_x}{b}}$$

$$d \geq \sqrt{\frac{4(6.0in^3)}{6.0in}}$$

$d \geq 2.0$ in. ⟹ Not Practical.

∴ As Previously Outlined, Use (2) - Full Height Stiffeners (Over The HSS Flanges) Such That The Compression Load Is Transfered Straight Through The Steel Beam. The Plate Thickness Therefore Does Not Control Or Govern The Design. Recall, We Have 2 Situations Using Bolts + Welds.

Welded Option: Use 5/16" PL That Is 3/4" Larger Than The Column.

Bolted Option: Use 1/2" PL, Size Will Vary Based On Beam Flange Width. Use Typical Gauges From AISC.

6. STIFFENERS

RECALL FROM PREVIOUS SHEETS: FOR W14x38:

BEAM SHEAR CAPACITY	= 131 kip
WEB LOCAL YIELDING	= 128 kip
WEB LOCAL CRIPPLING	= 76 kip
WEB SIDESWAY BUCKLING	= N/A
WEB COMPRESSION BUCKLING	= 59 kip. * GOVERNS.
HSS6"x6"x1/4"	= 155 kip.

*NOTE, SINCE $(V_R)_{BEAM} < (C_r)_{COLUMN}$ → BEAM SHEAR GOVERNS.

∴ STIFFENERS MUST BE SIZED FOR $P_f = 131\text{ kip} - 59\text{ kip} = 72.0\text{ kip}$.
⇒ DUE TO AXIAL COMPRESSION ONLY, $P_f = 36.0\text{ kip}$ PER PAIR OF STIFFENERS.

→ TRY USING (2) - 1/2" STIFFENERS.

A. STIFFENER YIELD @ COLUMN FLANGE

$$R_N = \phi 2 A_{st} F_y$$

$$= (0.90)(2)\left[(0.5in)(2.57in)\right](36\,ksi)$$

$R_N = 83.3$ kip /OK.

B. STIFFENER SHEAR @ COLUMN WEB

$$R_N = \phi 0.60(2) F_y A_{gv} \qquad \phi = 1.0$$

$$= (1.0)(0.60)(2)(36ksi)\left[(12.48in)(0.5in)\right]$$

$R_N = 269$ kip /OK

C. STIFFENER COMPRESSION

$$R_N = \phi 2 A_{st} F_{cr}$$

$$\frac{KL}{r} = \frac{0.75(12.48in)}{\frac{0.5in}{\sqrt{12}}} = 64.80 < 4.71\sqrt{\frac{E}{F_y}} = 133$$

$$\therefore F_{cr} = \left[0.658^{(F_y/F_E)}\right] F_y$$

$$F_E = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = 68.16\, ksi$$

$$F_{cr} = \left[0.658^{\left(\frac{36\,ksi}{68.16\,ksi}\right)}\right](36\,ksi)$$

$$F_{cr} = 28.9\, ksi$$

$$R_N = 0.9(2)\left[(0.5in)(2.57in)\right](28.9\,ksi)$$

$R_N = 67\, kip$ /OK

D. STIFFENER WELD @ COLUMN FLANGE

* ASSUME 1/4" DOUBLE FILLET WELDS.

$$\phi R_N = 1.5 \cdot 4 \cdot C_1 \cdot \alpha \cdot 1.392 \cdot D_{16}\, l$$

$$= (1.50)(4)(1.0)(0.78)(1.392)(4)(2.57\ in)$$

$$\phi R_N = 67.0\ kip \quad /OK$$

E. STIFFENER WELD @ PANEL ZONE

$$\phi R_N = 4 C_1 \alpha\, 1.392\, D_{16}\, l$$

$$= 4(1.0)(1.0)(1.392)(4)(12.27\ in)$$

$$\phi R_N = 273\ kip \quad /OK$$

7. GENERAL CONNECTION DETAILS (WELDS + BOLTS)

→ DESIGN / CHECKS OF STIFFENER WELDS IS OKAY, SEE EXCEL

→ FOR BOLTING OF BEAMS, USE PROCEDURE / GAUGES FROM AISC.

→ USE (4) - 5/8" ϕ BOLTS w/ $V_r = H_r = 12.4$ kip / BOLT w/ THREADS INC.

$$\sum (H_r)_{TOTAL} = 49.6 \text{ kip} > 5\% \, V_r \text{ FOR W18×71} = 275 \text{ kip} \cdot 0.05 = 13.75 \text{ kip.}$$

→ NOTE, PROVIDE DETAILS WHERE THE BEAM IS BOTH WIDER AND SHALLOWER THAN THE COLUMN + PL. DUE TO THE USE OF STIFFENERS, BEARING IS DIRECTLY THROUGH AND THUS PL'S ARE REALLY FOR ERECTION ONLY.

8. LATERAL STABILITY

→ FOR STEEL BEAMS CANT. OVER A COLUMN LATERAL STABILITY MUST BE CONSIDERED

→ PER AISC, IF THE BEAM FRAMING INTO THE BEAM / COLUMN JOINT IS 75% OF THE DEPTH OF THE CANT. BEAM THEN THIS IS CONSIDERED TO BE BRACED. NOTE THIS ON THE TYPICAL DETAILS.

Col.	Section	**HSS6x6x1/4**	
	Cr	155.2	kip
Cap Pl	L	6	in
	t	0.3125	in

E	**29000**	**ksi**
(Fy)b	**50**	**ksi**
(Fu)b	**65**	**ksi**
(Fy)pl	**36**	**ksi**

Section	**Vr**	**Beam Checks**				**Force Per**	**Stiffener Design and Checks**									**Stiffener Welds**				**Stiffener BRG**	
	[kip]	**Yield**	**Crip.**	**Buck.**	**Min.**	**Stiff. Set**	**b(s)**	**h(s)**	**t**	**Yield**	**Shear**	**KL/r**	**Fcr**	**Rn**	**Status**	**Size**	**Flan.**	**Web**	**Status**	**Rn**	**Status**
W18x71	**275**	223	173	187	155	0	2.95	16.75	0.750	143.1	542.7	58.0	30.2	120	OK	4	77	373	OK	149	OK
x65	**248**	200	143	140	140	7	2.92	16.65	0.750	141.9	539.5	57.7	30.2	119	OK	4	76	371	OK	148	OK
x60	**227**	182	122	111	111	22	2.97	16.58	0.750	144.2	537.0	57.4	30.3	121	OK	4	77	369	OK	150	OK
x55	**212**	167	108	92	92	32	2.95	16.48	0.750	143.5	533.8	57.1	30.3	121	OK	4	77	367	OK	149	OK
x50	**192**	150	90	69	69	43	2.94	16.38	0.750	142.8	530.6	56.7	30.4	121	OK	4	77	365	OK	149	OK
W18x46	**195**	153	92	72	72	42	2.22	16.48	0.750	107.8	533.8	57.1	30.3	91	OK	4	58	367	OK	112	OK
x40	**169**	131	71	48	48	53	2.20	16.28	0.750	106.8	527.3	56.4	30.5	90	OK	4	57	362	OK	111	OK
x35	**159**	121	65	42	42	57	2.25	16.20	0.750	109.4	524.9	56.1	30.5	93	OK	4	59	361	OK	114	OK
W16x57	**212**	189	137	138	137	9	2.69	14.65	0.500	87.0	316.4	76.1	26.5	64	OK	4	70	326	OK	91	OK
x50	**186**	163	107	95	95	30	2.72	14.68	0.500	88.2	317.0	76.3	26.5	65	OK	4	71	327	OK	92	OK
x45	**167**	145	89	71	71	42	2.71	14.48	0.500	87.7	312.7	75.2	26.7	65	OK	4	71	322	OK	91	OK
x40	**146**	126	70	49	49	48	2.69	14.38	0.500	87.1	310.5	74.7	26.8	65	OK	4	70	320	OK	91	OK
x36	**141**	119	66	44	44	48	2.75	14.40	0.500	88.9	311.0	74.8	26.8	66	OK	4	72	321	OK	93	OK
W16x31	**131**	111	57	36	36	47	2.02	14.40	0.500	65.3	311.0	74.8	26.8	49	OK	4	53	321	OK	68	OK
x26	**106**	98	48	27	27	39	2.00	14.20	0.500	64.8	306.7	73.8	27.0	49	OK	4	52	316	OK	68	OK
W14x82	**219**	245	205	274	155	0	3.99	12.18	0.500	129.2	263.0	63.3	29.2	105	OK	4	104	271	OK	135	OK
x74	**192**	213	159	188	155	0	3.99	12.08	0.500	129.2	260.8	62.7	29.3	105	OK	4	104	269	OK	135	OK
x68	**174**	192	136	148	136	9	3.94	11.88	0.500	127.6	256.5	61.7	29.5	104	OK	4	103	264	OK	133	OK
x61	**156**	171	112	109	109	23	4.00	11.90	0.500	129.6	257.0	61.8	29.4	106	OK	4	104	265	OK	135	OK
W14x53	**154**	169	108	105	105	25	3.03	11.90	0.500	98.2	257.0	61.8	29.4	80	OK	4	79	265	OK	102	OK
x48	**141**	153	92	81	81	30	3.02	11.80	0.500	97.7	254.9	61.3	29.5	80	OK	4	79	263	OK	102	OK
x43	**125**	134	74	58	58	33	3.00	11.70	0.500	97.2	252.7	60.8	29.6	80	OK	4	78	261	OK	101	OK
W14x38	**131**	128	76	59	59	36	2.57	12.48	0.500	83.3	269.5	64.8	28.9	67	OK	4	67	278	OK	87	OK
x34	**120**	116	65	46	46	37	2.63	12.50	0.500	85.1	270.0	65.0	28.8	68	OK	4	68	278	OK	89	OK
x30	**112**	107	60	39	39	36	2.62	12.30	0.500	84.7	265.7	63.9	29.0	68	OK	4	68	274	OK	88	OK
W14x26	**106**	103	52	33	33	36	1.77	12.40	0.500	57.2	267.8	64.4	28.9	46	OK	4	46	276	OK	60	OK
x22	**94.5**	90	43	24	24	35	1.75	12.20	0.500	56.7	263.5	63.4	29.1	46	OK	4	46	272	OK	59	OK
W12x96	**210**	268	253	397	155	0	4.98	10.45	0.500	161.2	225.7	54.3	30.8	138	OK	4	130	233	OK	168	OK
x87	**193**	245	225	327	155	0	4.99	10.38	0.500	161.6	224.1	53.9	30.9	139	OK	4	130	231	OK	168	OK
x79	**175**	219	188	247	155	0	4.99	10.28	0.500	161.6	221.9	53.4	31.0	139	OK	4	130	229	OK	168	OK
x72	**159**	197	158	189	155	0	4.94	10.18	0.500	160.0	219.8	52.9	31.1	138	OK	4	129	227	OK	167	OK
x65	**142**	176	132	142	132	5	5.00	10.10	0.500	162.0	218.2	52.5	31.1	140	OK	4	130	225	OK	169	OK
W12x58	**132**	164	109	111	109	12	4.06	10.33	0.500	131.6	223.0	53.7	30.9	113	OK	4	106	230	OK	137	OK
x53	**125**	154	102	98	98	14	4.06	10.23	0.500	131.6	220.9	53.1	31.0	113	OK	4	106	228	OK	137	OK
W12x50	**135**	164	116	121	116	10	3.10	10.33	0.500	100.5	223.0	53.7	30.9	86	OK	4	81	230	OK	105	OK
x45	**122**	146	95	89	89	16	3.09	10.23	0.500	100.0	220.9	53.1	31.0	86	OK	4	80	228	OK	104	OK
x40	**105**	126	74	61	61	22	3.13	10.15	0.500	101.4	219.2	52.7	31.1	88	OK	4	82	226	OK	106	OK
W12x35	**113**	121	75	61	61	26	2.53	11.00	0.500	82.0	237.6	57.2	30.3	69	OK	4	66	245	OK	85	OK
x30	**95.9**	102	57	40	40	28	2.51	10.80	0.500	81.3	233.3	56.1	30.5	69	OK	4	65	241	OK	85	OK
x26	**84.2**	89	45	28	28	28	2.50	10.70	0.500	80.8	231.1	55.6	30.6	69	OK	4	65	238	OK	84	OK
W12x22	**95.9**	102	57	40	40	28	1.39	11.05	0.500	45.0	238.7	57.4	30.3	38	OK	4	36	246	OK	47	OK
x19	**86**	90	48	29	29	28	1.44	11.08	0.500	46.7	239.2	57.5	30.2	39	OK	4	38	247	OK	49	OK
x16	**79.2**	82	45	24	24	28	1.43	10.88	0.500	46.4	234.9	56.5	30.4	39	OK	4	37	242	OK	48	OK
x14	**64.3**	73	39	18	18	23	1.42	10.78	0.500	46.1	232.7	56.0	30.5	39	OK	4	37	240	OK	48	OK
W10x45	**106**	154	113	126	106	0	3.23	8.48	0.375	78.4	137.3	58.7	30.0	65	OK	4	84	189	OK	82	OK
x39	**93.7**	135	94	92	92	1	3.18	8.30	0.375	77.3	134.4	57.5	30.3	65	OK	4	83	185	OK	81	OK
x33	**84.7**	121	83	72	72	7	3.23	8.23	0.375	78.5	133.3	57.0	30.3	66	OK	4	84	183	OK	82	OK
W10x30	**94.5**	120	82	74	74	10	2.22	9.13	0.375	53.9	147.8	63.2	29.2	44	OK	4	58	203	OK	56	OK
x26	**80.3**	102	63	49	49	16	2.20	8.93	0.375	53.4	144.6	61.8	29.4	44	OK	4	57	199	OK	56	OK
x22	**73.4**	92	55	38	38	18	2.25	8.95	0.375	54.7	145.0	62.0	29.4	45	OK	4	59	199	OK	57	OK
W10x19	**76.5**	97	59	43	43	17	1.39	8.95	0.375	33.7	145.0	62.0	29.4	27	OK	4	36	199	OK	35	OK
x17	**72.7**	91	57	38	38	17	1.44	8.98	0.375	35.1	145.4	62.2	29.4	29	OK	4	38	200	OK	37	OK
x15	**68.9**	85	57	33	33	18	1.44	8.87	0.375	34.9	143.6	61.4	29.5	29	OK	4	37	197	OK	36	OK
x12	**56.3**	69	40	19	19	19	1.42	8.75	0.375	34.4	141.7	60.6	29.7	28	OK	4	37	195	OK	36	OK
W8x67	**154**	266	327	676	154	0	3.20	7.13	0.313	64.9	96.2	59.2	29.9	54	OK	4	83	159	OK	68	OK
x58	**134**	231	269	484	134	0	3.24	7.00	0.313	65.5	94.5	58.2	30.1	55	OK	4	84	156	OK	68	OK
x48	**102**	174	164	233	102	0	3.24	6.88	0.313	65.7	92.8	57.2	30.3	55	OK	4	84	153	OK	68	OK
x40	**89.1**	151	140	170	89	0	3.22	6.63	0.313	65.3	89.4	55.1	30.7	56	OK	4	84	148	OK	68	OK
x35	**75.5**	127	104	109	76	0	3.20	6.50	0.313	64.7	87.7	54.0	30.9	56	OK	4	83	145	OK	67	OK
x31	**68.4**	115	90	84	68	0	3.25	6.50	0.313	65.8	87.8	54.0	30.9	56	OK	4	85	145	OK	69	OK
W8x28	**68.9**	116	87	84	69	0	2.65	6.81	0.313	53.6	91.9	56.6	30.4	45	OK	4	69	152	OK	56	OK
x24	**58.3**	98	65	54	54	2	2.69	6.81	0.313	54.4	91.9	56.6	30.4	46	OK	4	70	152	OK	57	OK
W8x21	**62.1**	97	67	54	54	4	2.07	7.16	0.313	42.0	96.6	59.5	29.9	35	OK	4	54	159	OK	44	OK
x18	**56.2**	87	59	42	42	7	2.06	7.02	0.313	41.8	94.7	58.3	30.1	35	OK	4	54	156	OK	44	OK
W8x15	**59.6**	92	71	51	51	4	1.45	6.99	0.313	29.3	94.3	58.1	30.1	25	OK	4	38	156	OK	31	OK
x13	**55.1**	85	68	42	42	6	1.44	6.87	0.313	29.1	92.7	57.1	30.3	25	OK	4	37	153	OK	30	OK
x10	**40.2**	62	36	17	17	12	1.47	6.89	0.313	29.8	93.0	57.3	30.3	25	OK	4	38	153	OK	31	OK

Col.	Section	**HSS5x5x1/4**	
	Cr	120	kip
Cap Pl	L	5	in
	t	0.3125	in

E	**29000**	**ksi**
(Fy)b	**50**	**ksi**
(Fu)b	**65**	**ksi**
(Fy)pl	**36**	**ksi**

Section	Vr	Beam Checks				Force Per	Stiffener Design and Checks									Stiffener Welds				Stiffener BRG	
	[kip]	Yield	Crip.	Buck.	Min.	Stiff. Set	b(s)	h(s)	t	Yield	Shear	KL/r	Fcr	Rn	Status	Size	Flan.	Web	Status	Rn	Status
W18x71	**275**	199	161	187	120	0	2.95	16.75	0.500	95.4	361.8	87.0	24.2	64	OK	4	77	373	OK	99	OK
x65	**248**	177	133	140	120	0	2.92	16.65	0.500	94.6	359.6	86.5	24.3	64	OK	4	76	371	OK	99	OK
x60	**227**	161	114	111	111	5	2.97	16.58	0.500	96.1	358.0	86.1	24.4	65	OK	4	77	369	OK	100	OK
x55	**212**	148	101	92	92	14	2.95	16.48	0.500	95.7	355.9	85.6	24.5	65	OK	4	77	367	OK	100	OK
x50	**192**	132	84	69	69	25	2.94	16.38	0.500	95.2	353.7	85.1	24.6	65	OK	4	77	365	OK	99	OK
W18x46	**195**	135	86	72	72	24	2.22	16.48	0.500	71.8	355.9	85.6	24.5	49	OK	4	58	367	OK	75	OK
x40	**169**	115	66	48	48	36	2.20	16.28	0.500	71.2	351.5	84.6	24.7	49	OK	4	57	362	OK	74	OK
x35	**159**	106	60	42	42	39	2.25	16.20	0.500	72.9	349.9	84.2	24.8	50	OK	4	59	361	OK	76	OK
W16x57	**212**	168	127	138	120	0	2.69	14.65	0.500	87.0	316.4	76.1	26.5	64	OK	4	70	326	OK	91	OK
x50	**186**	144	99	95	95	13	2.72	14.68	0.500	88.2	317.0	76.3	26.5	65	OK	4	71	327	OK	92	OK
x45	**167**	128	82	71	71	24	2.71	14.48	0.500	87.7	312.7	75.2	26.7	65	OK	4	71	322	OK	91	OK
x40	**146**	111	65	49	49	35	2.69	14.38	0.500	87.1	310.5	74.7	26.8	65	OK	4	70	320	OK	91	OK
x36	**141**	104	61	44	44	38	2.75	14.40	0.500	88.9	311.0	74.8	26.8	66	OK	4	72	321	OK	93	OK
W16x31	**131**	98	53	36	36	42	2.02	14.40	0.500	65.3	311.0	74.8	26.8	49	OK	4	53	321	OK	68	OK
x26	**106**	86	44	27	27	39	2.00	14.20	0.500	64.8	306.7	73.8	27.0	49	OK	4	52	316	OK	68	OK
W14x82	**219**	220	189	274	120	0	3.99	12.18	0.500	129.2	263.0	63.3	29.2	105	OK	4	104	271	OK	135	OK
x74	**192**	190	147	188	120	0	3.99	12.08	0.500	129.2	260.8	62.7	29.3	105	OK	4	104	269	OK	135	OK
x68	**174**	172	126	148	120	0	3.94	11.88	0.500	127.6	256.5	61.7	29.5	104	OK	4	103	264	OK	133	OK
x61	**156**	152	103	109	103	8	4.00	11.90	0.500	129.6	257.0	61.8	29.4	106	OK	4	104	265	OK	135	OK
W14x53	**154**	150	100	105	100	10	3.03	11.90	0.500	98.2	257.0	61.8	29.4	80	OK	4	79	265	OK	102	OK
x48	**141**	136	85	81	81	19	3.02	11.80	0.500	97.7	254.9	61.3	29.5	80	OK	4	79	263	OK	102	OK
x43	**125**	119	69	58	58	31	3.00	11.70	0.500	97.2	252.7	60.8	29.6	80	OK	4	78	261	OK	101	OK
W14x38	**131**	113	70	59	59	30	2.57	12.48	0.500	83.3	269.5	64.8	28.9	67	OK	4	67	278	OK	87	OK
x34	**120**	102	60	46	46	37	2.63	12.50	0.500	85.1	270.0	65.0	28.8	68	OK	4	68	278	OK	89	OK
x30	**112**	94	55	39	39	36	2.62	12.30	0.500	84.7	265.7	63.9	29.0	68	OK	4	68	274	OK	88	OK
W14x26	**106**	90	48	33	33	36	1.77	12.40	0.500	57.2	267.8	64.4	28.9	46	OK	4	46	276	OK	60	OK
x22	**94.5**	79	40	24	24	35	1.75	12.20	0.500	56.7	263.5	63.4	29.1	46	OK	4	46	272	OK	59	OK
W12x96	**210**	241	232	397	120	0	4.98	10.45	0.500	161.2	225.7	54.3	30.8	138	OK	4	130	233	OK	168	OK
x87	**193**	220	205	327	120	0	4.99	10.38	0.500	161.6	224.1	53.9	30.9	139	OK	4	130	231	OK	168	OK
x79	**175**	196	172	247	120	0	4.99	10.28	0.500	161.6	221.9	53.4	31.0	139	OK	4	130	229	OK	168	OK
x72	**159**	176	145	189	120	0	4.94	10.18	0.500	160.0	219.8	52.9	31.1	138	OK	4	129	227	OK	167	OK
x65	**142**	156	120	142	120	0	5.00	10.10	0.500	162.0	218.2	52.5	31.1	140	OK	4	130	225	OK	169	OK
W12x58	**132**	146	100	111	100	10	4.06	10.33	0.500	131.6	223.0	53.7	30.9	113	OK	4	106	230	OK	137	OK
x53	**125**	137	93	98	93	14	4.06	10.23	0.500	131.6	220.9	53.1	31.0	113	OK	4	106	228	OK	137	OK
W12x50	**135**	145	106	121	106	7	3.10	10.33	0.500	100.5	223.0	53.7	30.9	86	OK	4	81	230	OK	105	OK
x45	**122**	129	87	89	87	16	3.09	10.23	0.500	100.0	220.9	53.1	31.0	86	OK	4	80	228	OK	104	OK
x40	**105**	111	68	61	61	22	3.13	10.15	0.500	101.4	219.2	52.7	31.1	88	OK	4	82	226	OK	106	OK
W12x35	**113**	106	69	61	61	26	2.53	11.00	0.500	82.0	237.6	57.2	30.3	69	OK	4	66	245	OK	85	OK
x30	**95.9**	89	52	40	40	28	2.51	10.80	0.500	81.3	233.3	56.1	30.5	69	OK	4	65	241	OK	85	OK
x26	**84.2**	77	41	28	28	28	2.50	10.70	0.500	80.8	231.1	55.6	30.6	69	OK	4	65	238	OK	84	OK
W12x22	**95.9**	89	53	40	40	28	1.39	11.05	0.500	45.0	238.7	57.4	30.3	38	OK	4	36	246	OK	47	OK
x19	**86**	78	44	29	29	28	1.44	11.08	0.500	46.7	239.2	57.5	30.2	39	OK	4	38	247	OK	49	OK
x16	**79.2**	71	40	24	24	28	1.43	10.88	0.500	46.4	234.9	56.5	30.4	39	OK	4	37	242	OK	48	OK
x14	**64.3**	63	34	18	18	23	1.42	10.78	0.500	46.1	232.7	56.0	30.5	39	OK	4	37	240	OK	48	OK
W10x45	**106**	137	103	126	103	1	3.23	8.48	0.375	78.4	137.3	58.7	30.0	65	OK	4	84	189	OK	82	OK
x39	**93.7**	119	85	92	85	4	3.18	8.30	0.375	77.3	134.4	57.5	30.3	65	OK	4	83	185	OK	81	OK
x33	**84.7**	106	75	72	72	7	3.23	8.23	0.375	78.5	133.3	57.0	30.3	66	OK	4	84	183	OK	82	OK
W10x30	**94.5**	105	75	74	74	10	2.22	9.13	0.375	53.9	147.8	63.2	29.2	44	OK	4	58	203	OK	56	OK
x26	**80.3**	89	57	49	49	16	2.20	8.93	0.375	53.4	144.6	61.8	29.4	44	OK	4	57	199	OK	56	OK
x22	**73.4**	80	50	38	38	18	2.25	8.95	0.375	54.7	145.0	62.0	29.4	45	OK	4	59	199	OK	57	OK
W10x19	**76.5**	84	54	43	43	17	1.39	8.95	0.375	33.7	145.0	62.0	29.4	27	OK	4	36	199	OK	35	OK
x17	**72.7**	79	51	38	38	17	1.44	8.98	0.375	35.1	145.4	62.2	29.4	29	OK	4	38	200	OK	37	OK
x15	**68.9**	74	50	33	33	18	1.44	8.87	0.375	34.9	143.6	61.4	29.5	29	OK	4	37	197	OK	36	OK
x12	**56.3**	60	35	19	19	19	1.42	8.75	0.375	34.4	141.7	60.6	29.7	28	OK	4	37	195	OK	36	OK
W8x67	**154**	237	295	676	120	0	3.20	7.13	0.313	64.9	96.2	59.2	29.9	54	OK	4	83	159	OK	68	OK
x58	**134**	205	242	484	120	0	3.24	7.00	0.313	65.5	94.5	58.2	30.1	55	OK	4	84	156	OK	68	OK
x48	**102**	154	148	233	102	0	3.24	6.88	0.313	65.7	92.8	57.2	30.3	55	OK	4	84	153	OK	68	OK
x40	**89.1**	133	125	170	89	0	3.22	6.63	0.313	65.3	89.4	55.1	30.7	56	OK	4	84	148	OK	68	OK
x35	**75.5**	112	93	109	76	0	3.20	6.50	0.313	64.7	87.7	54.0	30.9	56	OK	4	83	145	OK	67	OK
x31	**68.4**	101	80	84	68	0	3.25	6.50	0.313	65.8	87.8	54.0	30.9	56	OK	4	85	145	OK	69	OK
W8x28	**68.9**	102	79	84	69	0	2.65	6.81	0.313	53.6	91.9	56.6	30.4	45	OK	4	69	152	OK	56	OK
x24	**58.3**	86	59	54	54	2	2.69	6.81	0.313	54.4	91.9	56.6	30.4	46	OK	4	70	152	OK	57	OK
W8x21	**62.1**	84	60	54	54	4	2.07	7.16	0.313	42.0	96.6	59.5	29.9	35	OK	4	54	159	OK	44	OK
x18	**56.2**	76	53	42	42	7	2.06	7.02	0.313	41.8	94.7	58.3	30.1	35	OK	4	54	156	OK	44	OK
W8x15	**59.6**	80	63	51	51	4	1.45	6.99	0.313	29.3	94.3	58.1	30.1	25	OK	4	38	156	OK	31	OK
x13	**55.1**	73	60	42	42	6	1.44	6.87	0.313	29.1	92.7	57.1	30.3	25	OK	4	37	153	OK	30	OK
x10	**40.2**	53	32	17	17	12	1.47	6.89	0.313	29.8	93.0	57.3	30.3	25	OK	4	38	153	OK	31	OK

Col.	Section	**HSS4x4x1/4**	
	Cr	85	kip
Cap Pl	L	4	in
	t	0.3125	in

E	**29000**	**ksi**
(Fy)b	**50**	**ksi**
(Fu)b	**65**	**ksi**
(Fy)pl	**36**	**ksi**

Section	Vr	Beam Checks				Force Per	Stiffener Design and Checks									Stiffener Welds				Stiffener BRG	
	[kip]	Yield	Crip.	Buck.	Min.	Stiff. Set	b(s)	h(s)	t	Yield	Shear	KL/r	Fcr	Rn	Status	Size	Flan.	Web	Status	Rn	Status
W18x71	**275**	174	149	187	85	0	2.95	16.75	0.500	95.4	361.8	87.0	24.2	64	OK	4	77	373	OK	99	OK
x65	**248**	155	124	140	85	0	2.92	16.65	0.500	94.6	359.6	86.5	24.3	64	OK	4	76	371	OK	99	OK
x60	**227**	140	106	111	85	0	2.97	16.58	0.500	96.1	358.0	86.1	24.4	65	OK	4	77	369	OK	100	OK
x55	**212**	128	93	92	85	0	2.95	16.48	0.500	95.7	355.9	85.6	24.5	65	OK	4	77	367	OK	100	OK
x50	**192**	114	77	69	69	8	2.94	16.38	0.500	95.2	353.7	85.1	24.6	65	OK	4	77	365	OK	99	OK
W18x46	**195**	117	80	72	72	7	2.22	16.48	0.500	71.8	355.9	85.6	24.5	49	OK	4	58	367	OK	75	OK
x40	**169**	100	61	48	48	18	2.20	16.28	0.500	71.2	351.5	84.6	24.7	49	OK	4	57	362	OK	74	OK
x35	**159**	91	55	42	42	22	2.25	16.20	0.500	72.9	349.9	84.2	24.8	50	OK	4	59	361	OK	76	OK
W16x57	**212**	146	117	138	85	0	2.69	14.65	0.500	87.0	316.4	76.1	26.5	64	OK	4	70	326	OK	91	OK
x50	**186**	125	92	95	85	0	2.72	14.68	0.500	88.2	317.0	76.3	26.5	65	OK	4	71	327	OK	92	OK
x45	**167**	111	76	71	71	7	2.71	14.48	0.500	87.7	312.7	75.2	26.7	65	OK	4	71	322	OK	91	OK
x40	**146**	96	59	49	49	18	2.69	14.38	0.500	87.1	310.5	74.7	26.8	65	OK	4	70	320	OK	91	OK
x36	**141**	90	55	44	44	20	2.75	14.40	0.500	88.9	311.0	74.8	26.8	66	OK	4	72	321	OK	93	OK
W16x31	**131**	84	48	36	36	24	2.02	14.40	0.500	65.3	311.0	74.8	26.8	49	OK	4	53	321	OK	68	OK
x26	**106**	73	40	27	27	29	2.00	14.20	0.500	64.8	306.7	73.8	27.0	49	OK	4	52	316	OK	68	OK
W14x82	**219**	194	173	274	85	0	3.99	12.18	0.500	129.2	263.0	63.3	29.2	105	OK	4	104	271	OK	135	OK
x74	**192**	168	135	188	85	0	3.99	12.08	0.500	129.2	260.8	62.7	29.3	105	OK	4	104	269	OK	135	OK
x68	**174**	151	116	148	85	0	3.94	11.88	0.500	127.6	256.5	61.7	29.5	104	OK	4	103	264	OK	133	OK
x61	**156**	133	95	109	85	0	4.00	11.90	0.500	129.6	257.0	61.8	29.4	106	OK	4	104	265	OK	135	OK
W14x53	**154**	132	92	105	85	0	3.03	11.90	0.500	98.2	257.0	61.8	29.4	80	OK	4	79	265	OK	102	OK
x48	**141**	119	78	81	78	3	3.02	11.80	0.500	97.7	254.9	61.3	29.5	80	OK	4	79	263	OK	102	OK
x43	**125**	104	63	58	58	13	3.00	11.70	0.500	97.2	252.7	60.8	29.6	80	OK	4	78	261	OK	101	OK
W14x38	**131**	97	64	59	59	13	2.57	12.48	0.500	83.3	269.5	64.8	28.9	67	OK	4	67	278	OK	87	OK
x34	**120**	87	54	46	46	20	2.63	12.50	0.500	85.1	270.0	65.0	28.8	68	OK	4	68	278	OK	89	OK
x30	**112**	80	49	39	39	23	2.62	12.30	0.500	84.7	265.7	63.9	29.0	68	OK	4	68	274	OK	88	OK
W14x26	**106**	77	44	33	33	26	1.77	12.40	0.500	57.2	267.8	64.4	28.9	46	OK	4	46	276	OK	60	OK
x22	**94.5**	67	36	24	24	30	1.75	12.20	0.500	56.7	263.5	63.4	29.1	46	OK	4	46	272	OK	59	OK
W12x96	**210**	213	211	397	85	0	4.98	10.45	0.500	161.2	225.7	54.3	30.8	138	OK	4	130	233	OK	168	OK
x87	**193**	194	186	327	85	0	4.99	10.38	0.500	161.6	224.1	53.9	30.9	139	OK	4	130	231	OK	168	OK
x79	**175**	172	155	247	85	0	4.99	10.28	0.500	161.6	221.9	53.4	31.0	139	OK	4	130	229	OK	168	OK
x72	**159**	154	131	189	85	0	4.94	10.18	0.500	160.0	219.8	52.9	31.1	138	OK	4	129	227	OK	167	OK
x65	**142**	137	108	142	85	0	5.00	10.10	0.500	162.0	218.2	52.5	31.1	140	OK	4	130	225	OK	169	OK
W12x58	**132**	128	92	111	85	0	4.06	10.33	0.500	131.6	223.0	53.7	30.9	113	OK	4	106	230	OK	137	OK
x53	**125**	120	84	98	84	0	4.06	10.23	0.500	131.6	220.9	53.1	31.0	113	OK	4	106	228	OK	137	OK
W12x50	**135**	127	97	121	85	0	3.10	10.33	0.500	100.5	223.0	53.7	30.9	86	OK	4	81	230	OK	105	OK
x45	**122**	112	80	89	80	3	3.09	10.23	0.500	100.0	220.9	53.1	31.0	86	OK	4	80	228	OK	104	OK
x40	**105**	97	62	61	61	12	3.13	10.15	0.500	101.4	219.2	52.7	31.1	88	OK	4	82	226	OK	106	OK
W12x35	**113**	91	63	61	61	12	2.53	11.00	0.500	82.0	237.6	57.2	30.3	69	OK	4	66	245	OK	85	OK
x30	**95.9**	76	48	40	40	22	2.51	10.80	0.500	81.3	233.3	56.1	30.5	69	OK	4	65	241	OK	85	OK
x26	**84.2**	66	37	28	28	28	2.50	10.70	0.500	80.8	231.1	55.6	30.6	69	OK	4	65	238	OK	84	OK
W12x22	**95.9**	76	48	40	40	23	1.39	11.05	0.500	45.0	238.7	57.4	30.3	38	OK	4	36	246	OK	47	OK
x19	**86**	66	39	29	29	28	1.44	11.08	0.500	46.7	239.2	57.5	30.2	39	OK	4	38	247	OK	49	OK
x16	**79.2**	60	36	24	24	28	1.43	10.88	0.500	46.4	234.9	56.5	30.4	39	OK	4	37	242	OK	48	OK
x14	**64.3**	53	30	18	18	23	1.42	10.78	0.500	46.1	232.7	56.0	30.5	39	OK	4	37	240	OK	48	OK
W10x45	**106**	119	93	126	85	0	3.23	8.48	0.375	78.4	137.3	58.7	30.0	65	OK	4	84	189	OK	82	OK
x39	**93.7**	104	77	92	77	4	3.18	8.30	0.375	77.3	134.4	57.5	30.3	65	OK	4	83	185	OK	81	OK
x33	**84.7**	92	66	72	66	9	3.23	8.23	0.375	78.5	133.3	57.0	30.3	66	OK	4	84	183	OK	82	OK
W10x30	**94.5**	90	68	74	68	9	2.22	9.13	0.375	53.9	147.8	63.2	29.2	44	OK	4	58	203	OK	56	OK
x26	**80.3**	76	51	49	49	16	2.20	8.93	0.375	53.4	144.6	61.8	29.4	44	OK	4	57	199	OK	56	OK
x22	**73.4**	68	44	38	38	18	2.25	8.95	0.375	54.7	145.0	62.0	29.4	45	OK	4	59	199	OK	57	OK
W10x19	**76.5**	72	48	43	43	17	1.39	8.95	0.375	33.7	145.0	62.0	29.4	27	OK	4	36	199	OK	35	OK
x17	**72.7**	67	45	38	38	17	1.44	8.98	0.375	35.1	145.4	62.2	29.4	29	OK	4	38	200	OK	37	OK
x15	**68.9**	62	44	33	33	18	1.44	8.87	0.375	34.9	143.6	61.4	29.5	29	OK	4	37	197	OK	36	OK
x12	**56.3**	50	30	19	19	19	1.42	8.75	0.375	34.4	141.7	60.6	29.7	28	OK	4	37	195	OK	36	OK
W8x67	**154**	209	263	676	85	0	3.20	7.13	0.313	64.9	96.2	59.2	29.9	54	OK	4	83	159	OK	68	OK
x58	**134**	180	215	484	85	0	3.24	7.00	0.313	65.5	94.5	58.2	30.1	55	OK	4	84	156	OK	68	OK
x48	**102**	134	132	233	85	0	3.24	6.88	0.313	65.7	92.8	57.2	30.3	55	OK	4	84	153	OK	68	OK
x40	**89.1**	115	111	170	85	0	3.22	6.63	0.313	65.3	89.4	55.1	30.7	56	OK	4	84	148	OK	68	OK
x35	**75.5**	96	82	109	76	0	3.20	6.50	0.313	64.7	87.7	54.0	30.9	56	OK	4	83	145	OK	67	OK
x31	**68.4**	87	71	84	68	0	3.25	6.50	0.313	65.8	87.8	54.0	30.9	56	OK	4	85	145	OK	69	OK
W8x28	**68.9**	88	70	84	69	0	2.65	6.81	0.313	53.6	91.9	56.6	30.4	45	OK	4	69	152	OK	56	OK
x24	**58.3**	73	52	54	52	3	2.69	6.81	0.313	54.4	91.9	56.6	30.4	46	OK	4	70	152	OK	57	OK
W8x21	**62.1**	72	53	54	53	5	2.07	7.16	0.313	42.0	96.6	59.5	29.9	35	OK	4	54	159	OK	44	OK
x18	**56.2**	64	46	42	42	7	2.06	7.02	0.313	41.8	94.7	58.3	30.1	35	OK	4	54	156	OK	44	OK
W8x15	**59.6**	68	54	51	51	4	1.45	6.99	0.313	29.3	94.3	58.1	30.1	25	OK	4	38	156	OK	31	OK
x13	**55.1**	62	51	42	42	6	1.44	6.87	0.313	29.1	92.7	57.1	30.3	25	OK	4	37	153	OK	30	OK
x10	**40.2**	45	27	17	17	12	1.47	6.89	0.313	29.8	93.0	57.3	30.3	25	OK	4	38	153	OK	31	OK

Col.	Section	HSS3x3x1/4
	Cr	48 kip
Cap Pl	L	3 in
	t	0.3125 in

E	29000 ksi
(Fy)b	50 ksi
(Fu)b	65 ksi
(Fy)pl	36 ksi

Section	Vr	Beam Checks				Force Per	Stiffener Design and Checks									Stiffener Welds				Stiffener BRG	
	[kip]	Yield	Crip.	Buck.	Min.	Stiff. Set	b(s)	h(s)	t	Yield	Shear	KL/r	Fcr	Rn	Status	Size	Flan.	Web	Status	Rn	Status
W18x71	**275**	149	140	187	48	0	2.95	16.75	0.375	71.6	271.4	116.0	17.7	35	OK	4	77	373	OK	75	OK
x65	**248**	132	116	140	48	0	2.92	16.65	0.375	71.0	269.7	115.4	17.9	35	OK	4	76	371	OK	74	OK
x60	**227**	119	99	111	48	0	2.97	16.58	0.375	72.1	268.5	114.8	18.0	36	OK	4	77	369	OK	75	OK
x55	**212**	109	87	92	48	0	2.95	16.48	0.375	71.7	266.9	114.1	18.1	36	OK	4	77	367	OK	75	OK
x50	**192**	96	72	69	48	0	2.94	16.38	0.375	71.4	265.3	113.4	18.3	36	OK	4	77	365	OK	74	OK
W18x46	**195**	99	75	72	48	0	2.22	16.48	0.375	53.9	266.9	114.1	18.1	27	OK	4	58	367	OK	56	OK
x40	**169**	84	57	48	48	0	2.20	16.28	0.375	53.4	263.7	112.8	18.4	27	OK	4	57	362	OK	56	OK
x35	**159**	76	50	42	42	3	2.25	16.20	0.375	54.7	262.4	112.2	18.5	28	OK	4	59	361	OK	57	OK
W16x57	**212**	125	108	138	48	0	2.69	14.65	0.375	65.2	237.3	101.5	20.9	38	OK	4	70	326	OK	68	OK
x50	**186**	106	85	95	48	0	2.72	14.68	0.375	66.2	237.7	101.7	20.9	38	OK	4	71	327	OK	69	OK
x45	**167**	93	70	71	48	0	2.71	14.48	0.375	65.8	234.5	100.3	21.2	39	OK	4	71	322	OK	69	OK
x40	**146**	80	55	49	48	0	2.69	14.38	0.375	65.3	232.9	99.6	21.4	39	OK	4	70	320	OK	68	OK
x36	**141**	75	50	44	44	2	2.75	14.40	0.375	66.7	233.3	99.8	21.3	39	OK	4	72	321	OK	69	OK
W16x31	**131**	70	44	36	36	6	2.02	14.40	0.375	49.0	233.3	99.8	21.3	29	OK	4	53	321	OK	51	OK
x26	**106**	61	36	27	27	10	2.00	14.20	0.375	48.6	230.0	98.4	21.6	29	OK	4	52	316	OK	51	OK
W14x82	**219**	169	157	274	48	0	3.99	12.18	0.375	96.9	197.2	84.4	24.8	67	OK	4	104	271	OK	101	OK
x74	**192**	145	124	188	48	0	3.99	12.08	0.375	96.9	195.6	83.7	24.9	67	OK	4	104	269	OK	101	OK
x68	**174**	130	106	148	48	0	3.94	11.88	0.375	95.7	192.4	82.3	25.2	67	OK	4	103	264	OK	100	OK
x61	**156**	114	86	109	48	0	4.00	11.90	0.375	97.2	192.8	82.4	25.2	68	OK	4	104	265	OK	101	OK
W14x53	**154**	113	84	105	48	0	3.03	11.90	0.375	73.6	192.8	82.4	25.2	51	OK	4	79	265	OK	77	OK
x48	**141**	102	71	81	48	0	3.02	11.80	0.375	73.3	191.2	81.8	25.3	52	OK	4	79	263	OK	76	OK
x43	**125**	88	57	58	48	0	3.00	11.70	0.375	72.9	189.5	81.1	25.5	52	OK	4	78	261	OK	76	OK
W14x38	**131**	82	58	59	48	0	2.57	12.48	0.375	62.5	202.1	86.4	24.3	42	OK	4	67	278	OK	65	OK
x34	**120**	73	49	46	46	1	2.63	12.50	0.375	63.8	202.5	86.6	24.3	43	OK	4	68	278	OK	66	OK
x30	**112**	67	44	39	39	4	2.62	12.30	0.375	63.5	199.3	85.2	24.6	43	OK	4	68	274	OK	66	OK
W14x26	**106**	64	40	33	33	7	1.77	12.40	0.375	42.9	200.9	85.9	24.4	29	OK	4	46	276	OK	45	OK
x22	**94.5**	56	32	24	24	12	1.75	12.20	0.375	42.5	197.6	84.5	24.7	29	OK	4	46	272	OK	44	OK
W12x96	**210**	186	190	397	48	0	4.98	10.45	0.375	120.9	169.3	72.4	27.3	92	OK	4	130	233	OK	126	OK
x87	**193**	168	166	327	48	0	4.99	10.38	0.375	121.2	168.1	71.9	27.4	92	OK	4	130	231	OK	126	OK
x79	**175**	149	139	247	48	0	4.99	10.28	0.375	121.2	166.5	71.2	27.6	93	OK	4	130	229	OK	126	OK
x72	**159**	133	117	189	48	0	4.94	10.18	0.375	120.0	164.8	70.5	27.7	92	OK	4	129	227	OK	125	OK
x65	**142**	117	96	142	48	0	5.00	10.10	0.375	121.5	163.6	70.0	27.8	94	OK	4	130	225	OK	127	OK
W12x58	**132**	110	83	111	48	0	4.06	10.33	0.375	98.7	167.3	71.5	27.5	75	OK	4	106	230	OK	103	OK
x53	**125**	103	76	98	48	0	4.06	10.23	0.375	98.7	165.6	70.8	27.6	76	OK	4	106	228	OK	103	OK
W12x50	**135**	108	87	121	48	0	3.10	10.33	0.375	75.4	167.3	71.5	27.5	58	OK	4	81	230	OK	79	OK
x45	**122**	95	72	89	48	0	3.09	10.23	0.375	75.0	165.6	70.8	27.6	58	OK	4	80	228	OK	78	OK
x40	**105**	82	56	61	48	0	3.13	10.15	0.375	76.1	164.4	70.3	27.7	59	OK	4	82	226	OK	79	OK
W12x35	**113**	76	57	61	48	0	2.53	11.00	0.375	61.5	178.2	76.2	26.5	45	OK	4	66	245	OK	64	OK
x30	**95.9**	63	43	40	40	4	2.51	10.80	0.375	61.0	175.0	74.8	26.8	45	OK	4	65	241	OK	64	OK
x26	**84.2**	54	34	28	28	10	2.50	10.70	0.375	60.6	173.3	74.1	27.0	45	OK	4	65	238	OK	63	OK
W12x22	**95.9**	63	43	40	40	4	1.39	11.05	0.375	33.8	179.0	76.6	26.4	25	OK	4	36	246	OK	35	OK
x19	**86**	54	35	29	29	9	1.44	11.08	0.375	35.1	179.4	76.7	26.4	26	OK	4	38	247	OK	37	OK
x16	**79.2**	49	31	24	24	12	1.43	10.88	0.375	34.8	176.2	75.3	26.7	26	OK	4	37	242	OK	36	OK
x14	**64.3**	43	26	18	18	15	1.42	10.78	0.375	34.6	174.6	74.7	26.8	26	OK	4	37	240	OK	36	OK
W10x45	**106**	102	84	126	48	0	3.23	8.48	0.313	65.5	114.6	70.3	27.7	50	OK	4	84	189	OK	68	OK
x39	**93.7**	88	68	92	48	0	3.18	8.30	0.313	64.5	112.2	68.9	28.0	50	OK	4	83	185	OK	67	OK
x33	**84.7**	77	58	72	48	0	3.23	8.23	0.313	65.5	111.3	68.3	28.2	51	OK	4	84	183	OK	68	OK
W10x30	**94.5**	75	60	74	48	0	2.22	9.13	0.313	45.0	123.4	75.7	26.6	33	OK	4	58	203	OK	47	OK
x26	**80.3**	63	46	49	46	1	2.20	8.93	0.313	44.6	120.7	74.1	27.0	33	OK	4	57	199	OK	46	OK
x22	**73.4**	56	39	38	38	5	2.25	8.95	0.313	45.6	121.0	74.3	26.9	34	OK	4	59	199	OK	48	OK
W10x19	**76.5**	59	42	43	42	3	1.39	8.95	0.313	28.1	121.0	74.3	26.9	21	OK	4	36	199	OK	29	OK
x17	**72.7**	55	39	38	38	5	1.44	8.98	0.313	29.3	121.4	74.5	26.9	22	OK	4	38	200	OK	30	OK
x15	**68.9**	51	37	33	33	7	1.44	8.87	0.313	29.2	119.9	73.6	27.1	22	OK	4	37	197	OK	30	OK
x12	**56.3**	41	26	19	19	15	1.42	8.75	0.313	28.8	118.2	72.6	27.3	22	OK	4	37	195	OK	30	OK
W8x67	**154**	180	231	676	48	0	3.20	7.13	0.313	64.9	96.2	59.2	29.9	54	OK	4	83	159	OK	68	OK
x58	**134**	154	188	484	48	0	3.24	7.00	0.313	65.5	94.5	58.2	30.1	55	OK	4	84	156	OK	68	OK
x48	**102**	114	117	233	48	0	3.24	6.88	0.313	65.7	92.8	57.2	30.3	55	OK	4	84	153	OK	68	OK
x40	**89.1**	97	96	170	48	0	3.22	6.63	0.313	65.3	89.4	55.1	30.7	56	OK	4	84	148	OK	68	OK
x35	**75.5**	81	72	109	48	0	3.20	6.50	0.313	64.7	87.7	54.0	30.9	56	OK	4	83	145	OK	67	OK
x31	**68.4**	72	61	84	48	0	3.25	6.50	0.313	65.8	87.8	54.0	30.9	56	OK	4	85	145	OK	69	OK
W8x28	**68.9**	73	61	84	48	0	2.65	6.81	0.313	53.6	91.9	56.6	30.4	45	OK	4	69	152	OK	56	OK
x24	**58.3**	61	45	54	45	1	2.69	6.81	0.313	54.4	91.9	56.6	30.4	46	OK	4	70	152	OK	57	OK
W8x21	**62.1**	59	46	54	46	1	2.07	7.16	0.313	42.0	96.6	59.5	29.9	35	OK	4	54	159	OK	44	OK
x18	**56.2**	53	40	42	40	4	2.06	7.02	0.313	41.8	94.7	58.3	30.1	35	OK	4	54	156	OK	44	OK
W8x15	**59.6**	56	46	51	46	1	1.45	6.99	0.313	29.3	94.3	58.1	30.1	25	OK	4	38	156	OK	31	OK
x13	**55.1**	50	43	42	42	3	1.44	6.87	0.313	29.1	92.7	57.1	30.3	25	OK	4	37	153	OK	30	OK
x10	**40.2**	36	23	17	17	12	1.47	6.89	0.313	29.8	93.0	57.3	30.3	25	OK	4	38	153	OK	31	OK

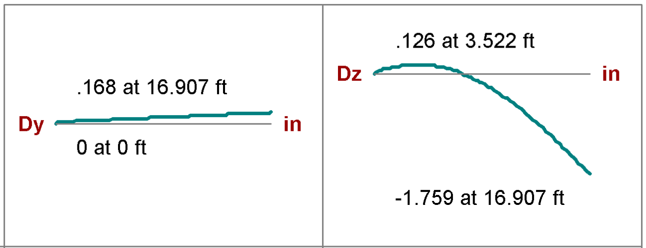
Blackwell

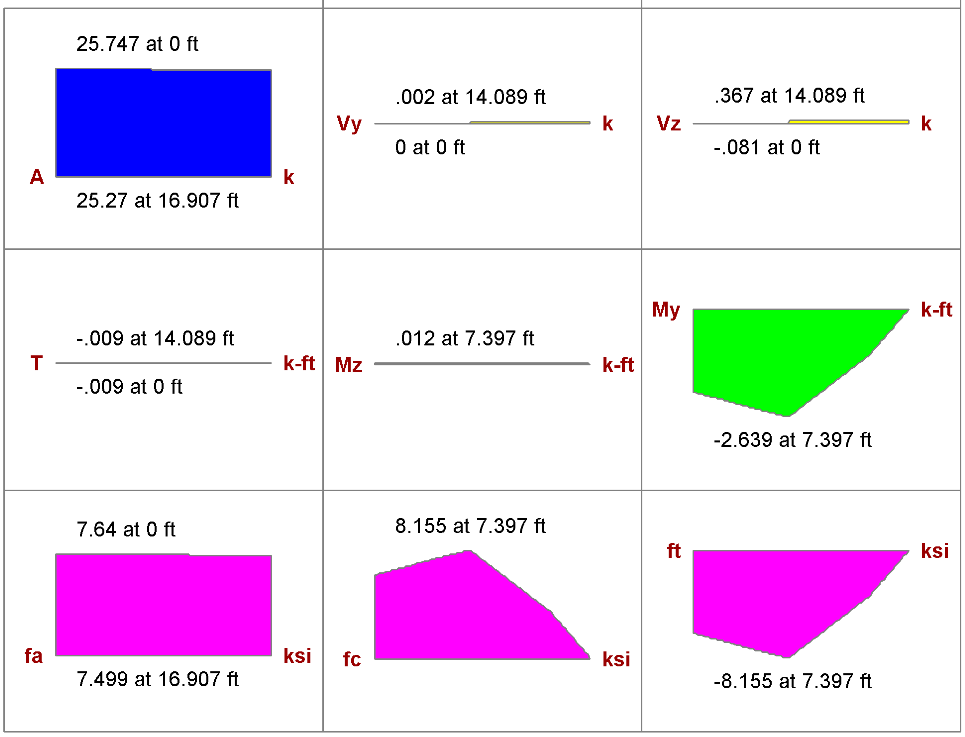
Structural Engineers

APPENDIX C - VOLUME 1 LATERAL DETAIL MEMBER REPORTS

Column: ***(2-A)_L2***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **16.907 ft**
I Joint: **N23**
J Joint: **N9**
LC 28: ASCE Strength 3 (d) (d)
Code Check: **0.744 (bending)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.744**	Max Shear Check	**0.010 (z)**
Location	**7.397 ft**	Location	**14.089 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/282**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**7.5 ft**	**16.907 ft**
phi*Pnc	**42.809 k**	KL/r	**59.158**	**133.358**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**16.907 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**16.907 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.188**			

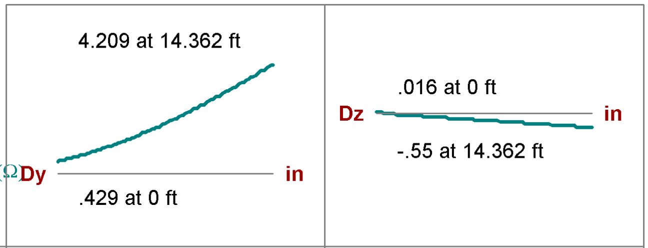
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

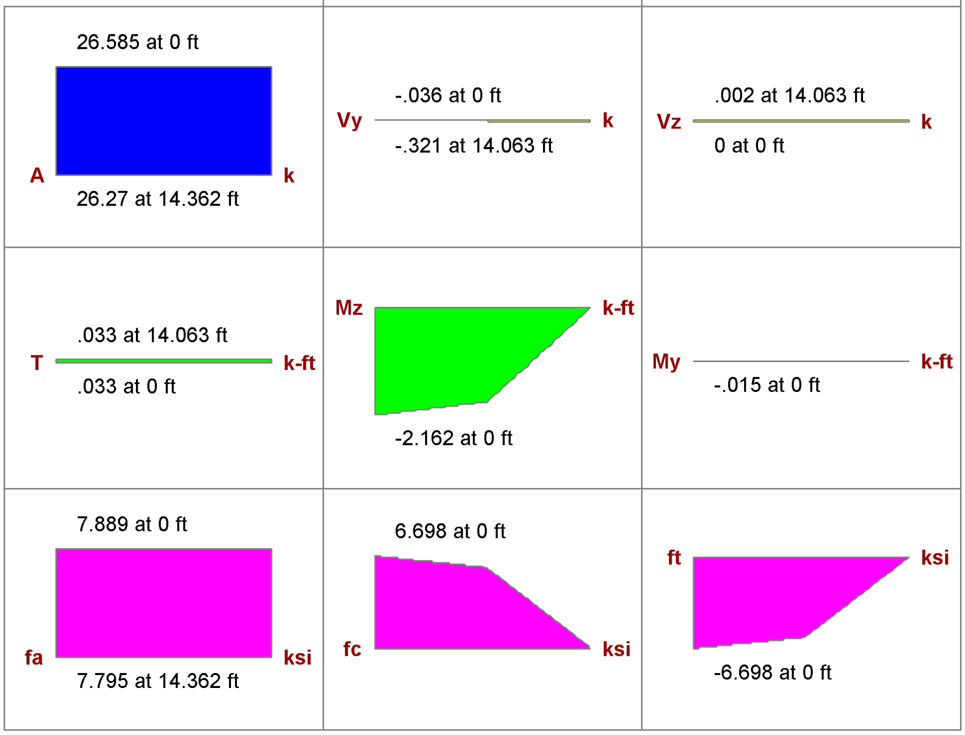
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **133.358** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***CS6_L1***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **14.362 ft**
I Joint: **N32**
J Joint: **N8**
LC 83: ASCE Strength 5 (os-c) (Ω
Code Check: **0.571 (bending)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.571**	Max Shear Check	**0.011 (y)**
Location	**0 ft**	Location	**14.063 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/418**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**14.362 ft**	**14.362 ft**
phi*Pnc	**58.847 k**	KL/r	**113.283**	**113.283**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**14.362 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**14.362 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.051**			

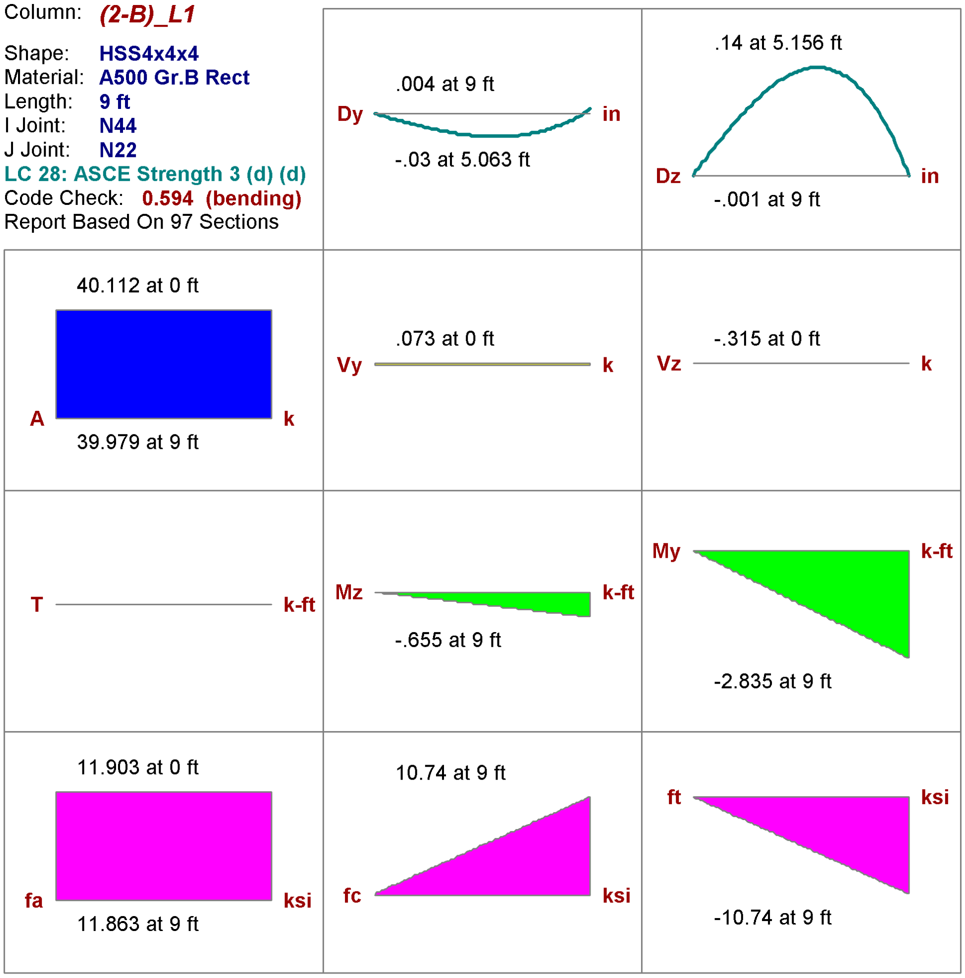
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **113.283** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(2-B)_L1***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **9 ft**
I Joint: **N44**
J Joint: **N22**
LC 28: ASCE Strength 3 (d) (d)
Code Check: **0.594 (bending)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.594**	Max Shear Check	**0.008 (z)**
Location	**9 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/767**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**9 ft**	**9 ft**
phi*Pnc	**99.405 k**	KL/r	**70.989**	**70.989**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**9 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**9 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.667**			

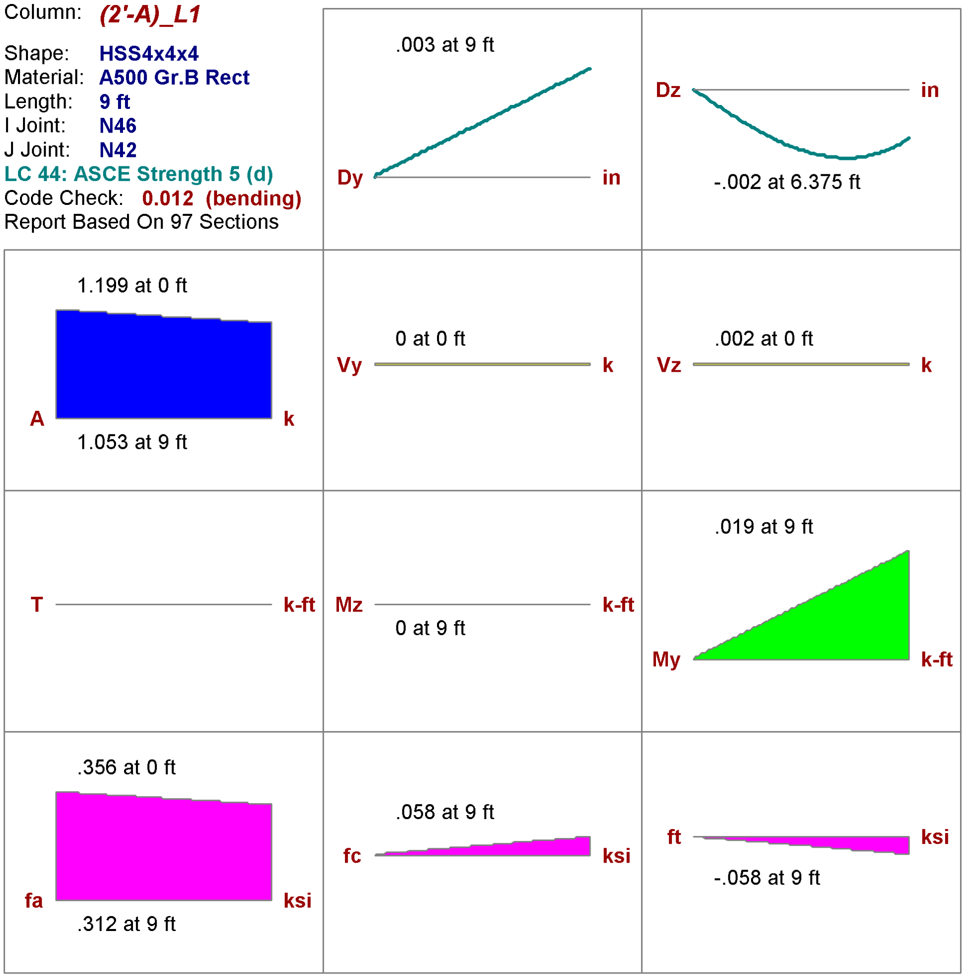
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **70.989** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(2'-A)_L1***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **9 ft**
I Joint: **N46**
J Joint: **N42**
LC 44: ASCE Strength 5 (d)
Code Check: **0.012 (bending)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.012**	Max Shear Check	**0.000 (z)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/10000**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**9 ft**	**9 ft**
phi*Pnc	**99.405 k**	KL/r	**70.989**	**70.989**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**9 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**9 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.667**			

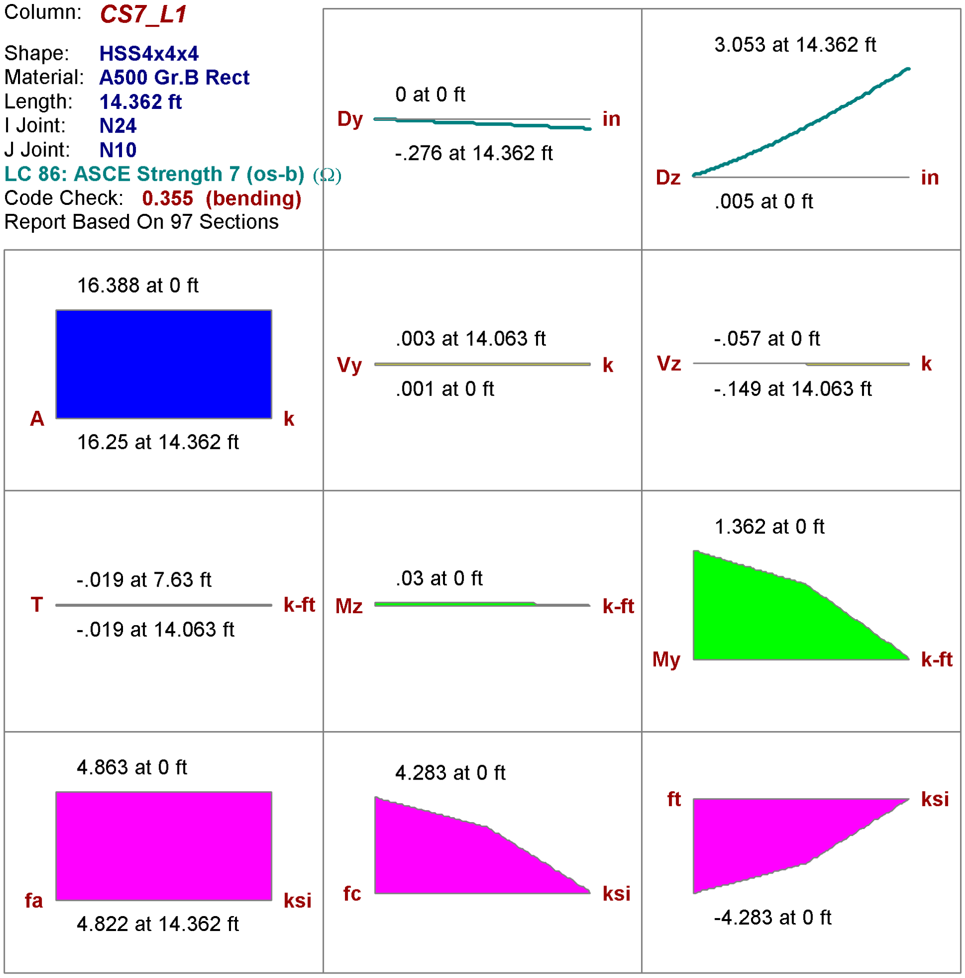
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **70.989** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***CS7_L1***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **14.362 ft**
I Joint: **N24**
J Joint: **N10**
LC 86: ASCE Strength 7 (os-b) (Ω)
Code Check: **0.355 (bending)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.355**	Max Shear Check	**0.005 (z)**
Location	**0 ft**	Location	**14.063 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/792**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**14.362 ft**	**14.362 ft**
phi*Pnc	**58.847 k**	KL/r	**113.283**	**113.283**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**14.362 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**14.362 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.148**			

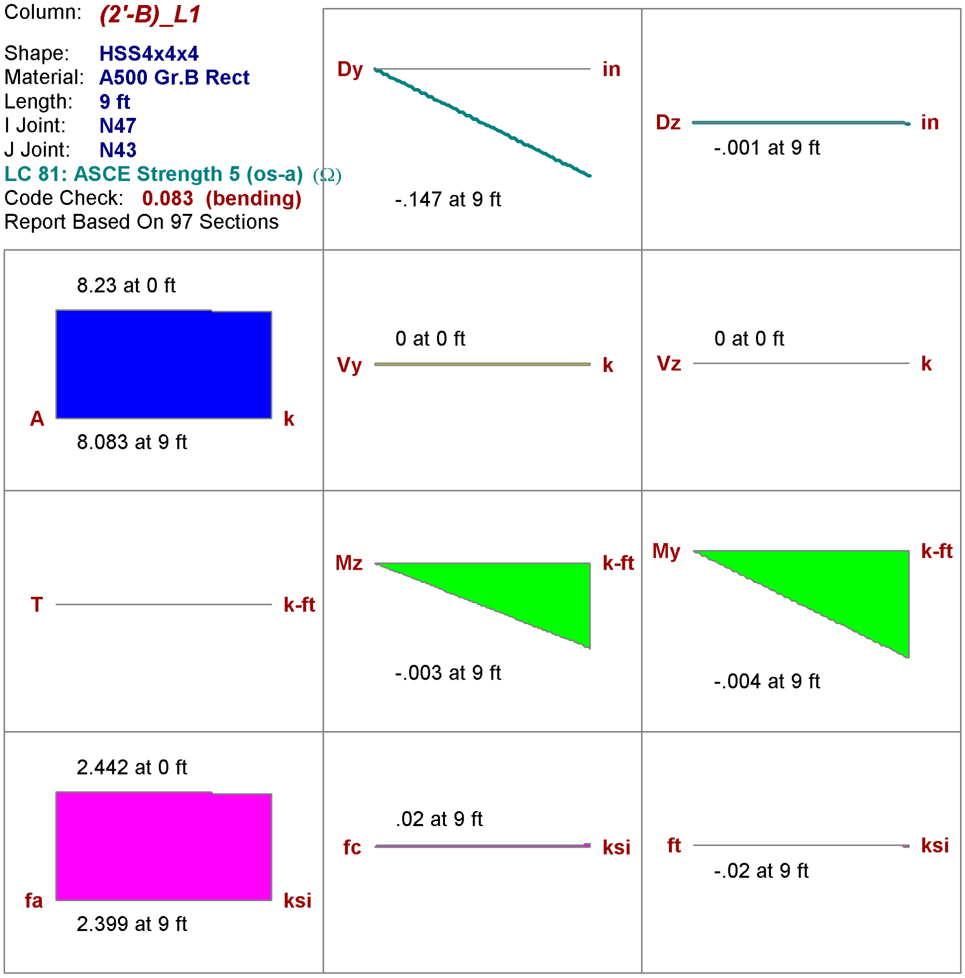
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **113.283** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(2'-B)_L1***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **9 ft**
I Joint: **N47**
J Joint: **N43**
LC 81: ASCE Strength 5 (os-a) (Ω)
Code Check: **0.083 (bending)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.083**	Max Shear Check	**0.000 (z)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/10000**
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**9 ft**	**9 ft**
phi*Pnc	**99.405 k**	KL/r	**70.989**	**70.989**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**9 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**9 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.667**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

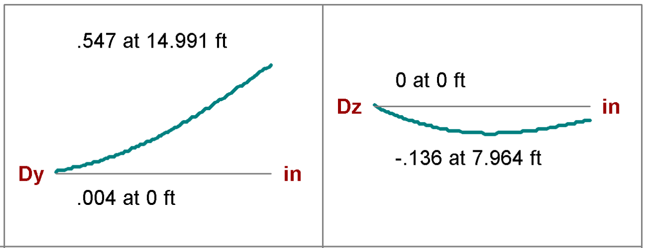
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

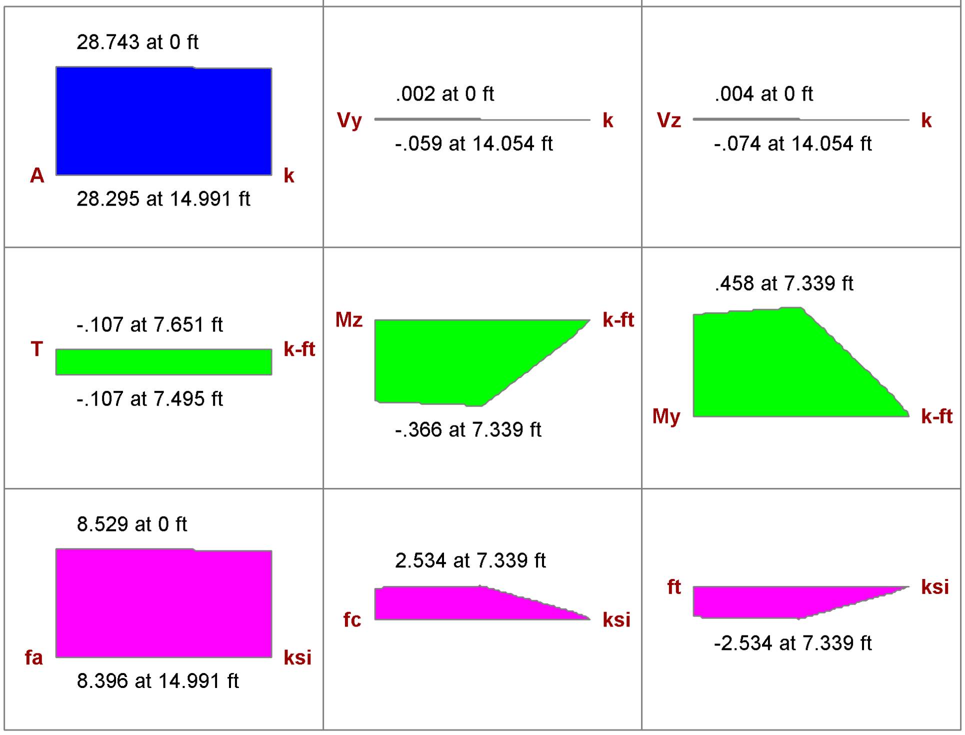
L/r = **70.989** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(2-B)_L2***

Shape:	**HSS4x4x4**
Material:	**A500 Gr.B Rect**
Length:	**14.991 ft**
I Joint:	**N22**
J Joint:	**N6**

LC 26: ASCE Strength 3 (d) (b)
Code Check: **0.571 (bending)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.571**	Max Shear Check	**0.010 (z)**
Location	**7.339 ft**	Location	**14.054 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/1759**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**7.5 ft**	**14.991 ft**
phi*Pnc	**54.475 k**	KL/r	**59.158**	**118.24**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**14.991 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**14.991 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.019**			

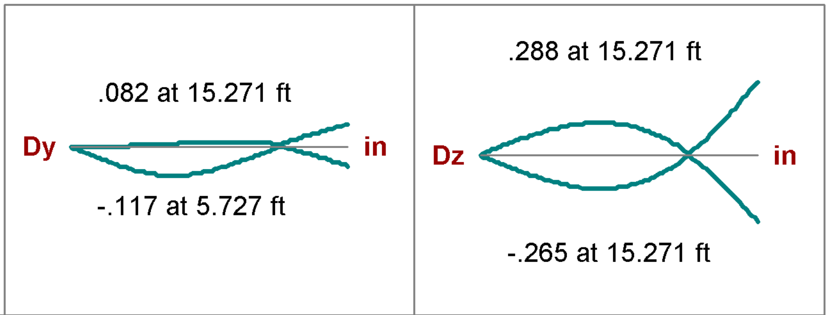
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

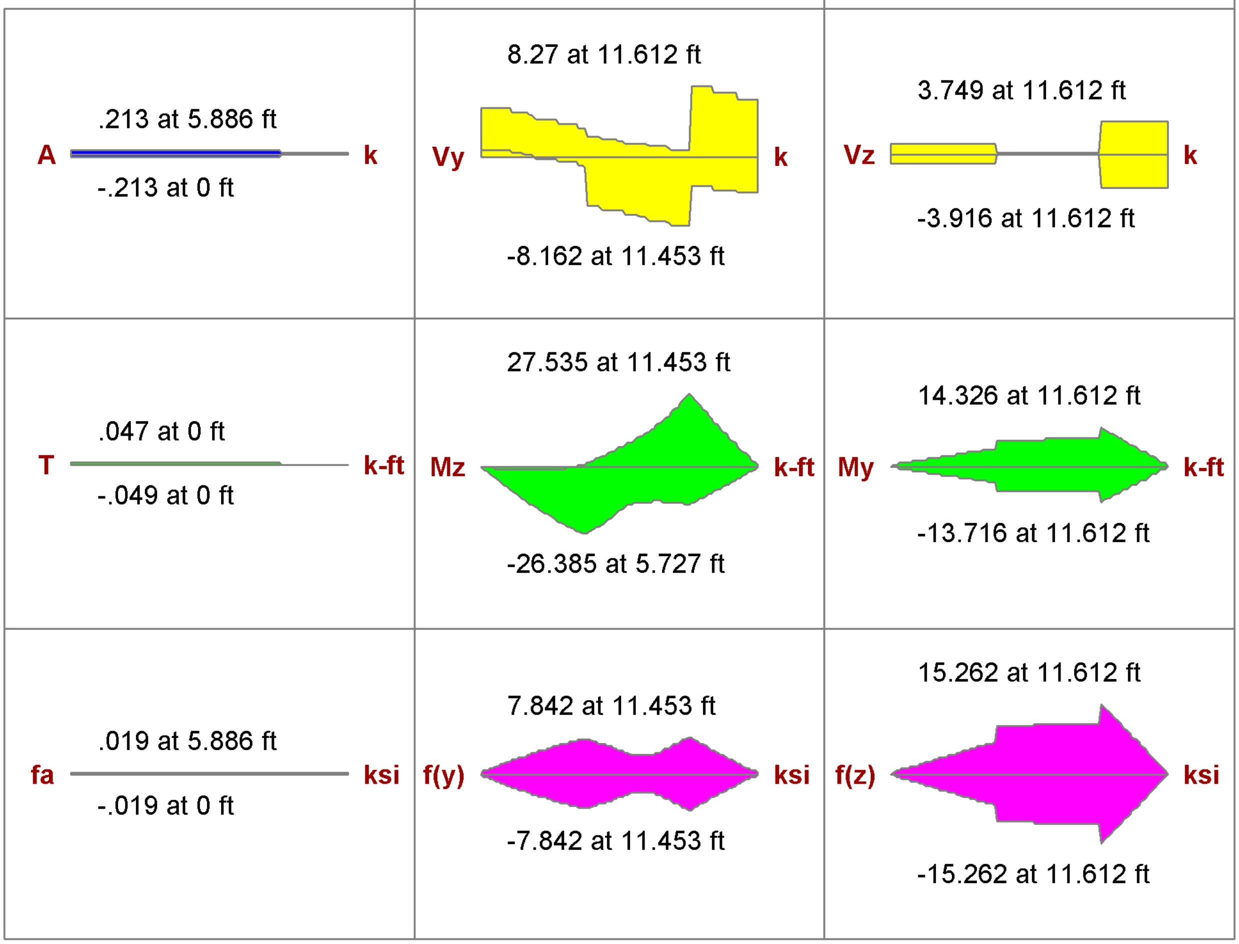
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **118.24** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Beam: ***2B14***

Shape: **W10x39**
Material: **A992**
Length: **15.271 ft**
I Joint: **N25**
J Joint: **N26**
Envelope (Ω)
Code Check: **0.376 (LC 83)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.376 (LC 83)**	Max Shear Check	**0.091 (y) (LC 2[illegible])**	Max Defl Ratio	**L/1109**
Location	**11.612 ft**	Location	**11.453 ft**	Location	**15.271 ft**
Equation	**H1-1b**			Span	**3**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**1.333 ft**	**11.5 ft**
phi*Pnc	**393.145 k**	KL/r	**8.086**	**32.371**
phi*Pnt	**517.5 k**			
phi*Mny	**64.5 k-ft**	L Comp Flange	**15.271 ft**	
phi*Mnz	**175.5 k-ft**	L-torque	**15.271 ft**	
phi*Vny	**93.744 k**	Tau_b	**1**	
phi*Vnz	**228.674 k**			
Cb	**1.927**			

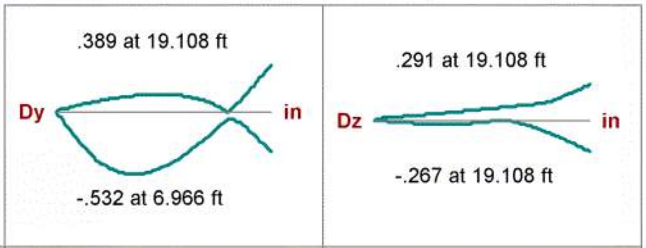
Beam: ***2B15***

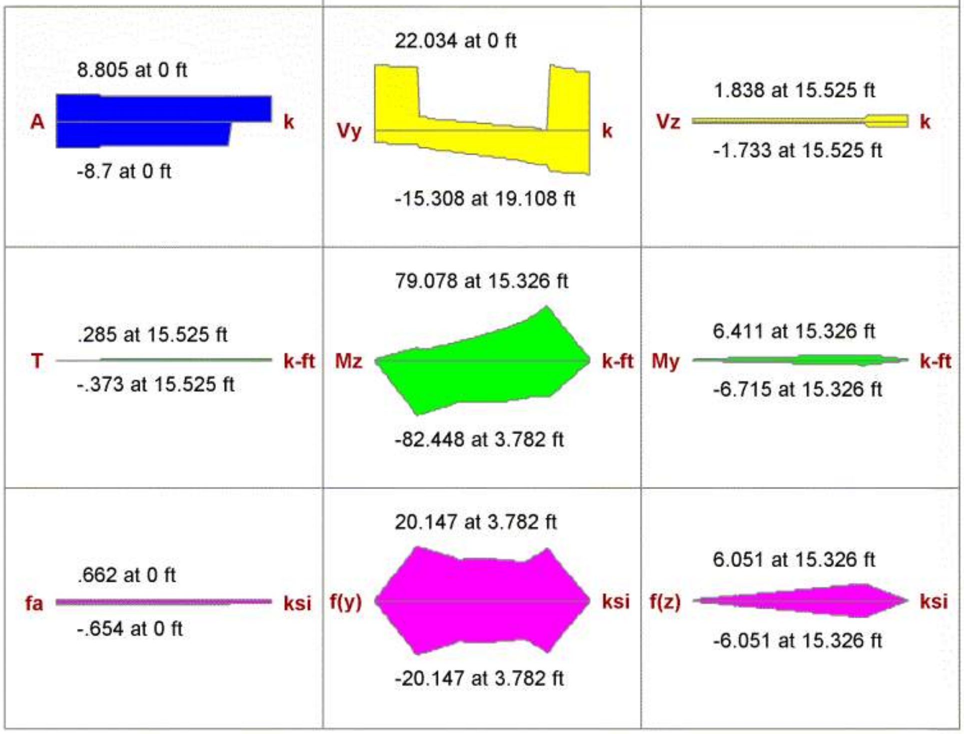
Shape:	**W10x45**
Material:	**A992**
Length:	**19.108 ft**
I Joint:	**F4_N42**
J Joint:	**N24**

Envelope

Code Check: **0.403 (LC 28)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.403 (LC 28)**	Max Shear Check	**0.210 (y) (LC 82)**	Max Defl Ratio	**L/588**
Location	**3.782 ft**	Location	**15.525 ft**	Location	**8.758 ft**
Equation	**H1-1b**			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**1.333 ft**	**19.108 ft**
phi*Pnc	**486.999 k**	KL/r	**7.983**	**53.1**
phi*Pnt	**598.5 k**			
phi*Mny	**76.125 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**205.875 k-ft**	L-torque	**19.108 ft**	
phi*Vny	**106.05 k**	Tau_b	**1**	
phi*Vnz	**268.51 k**			
Cb	**1**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

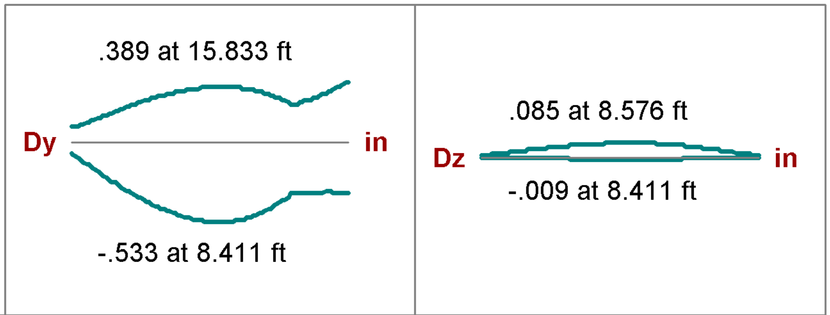
Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

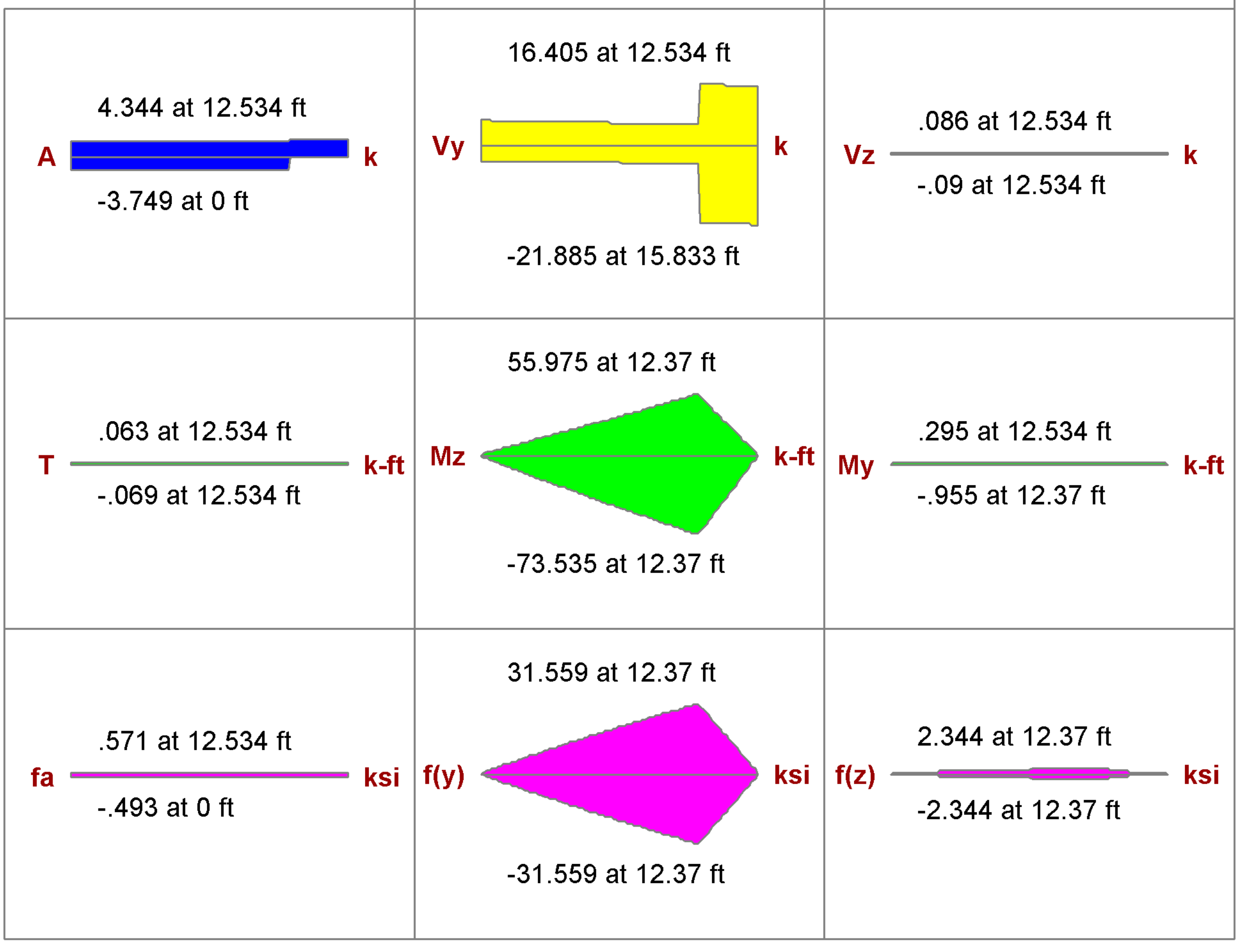
Miscellaneous Seismic Checks/Warnings:

- Seismic Framing Error: Unable to establish support information for one or both ends of beam.

Beam: ***2B16***

Shape: **W10x26**
Material: **A992**
Length: **15.833 ft**
I Joint: **N26**
J Joint: **N24**
Envelope (Ω)
Code Check: **0.640 (LC 83)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

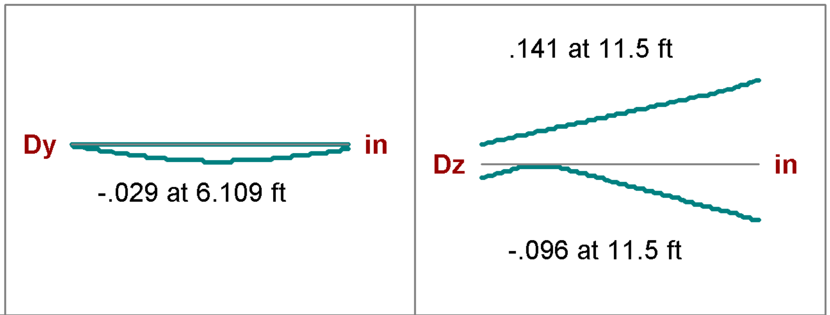
Max Bending Check	**0.640 (LC 83)**	Max Shear Check	**0.272 (y) (LC 83)**	Max Defl Ratio	**L/527**
Location	**12.37 ft**	Location	**15.833 ft**	Location	**7.092 ft**
Equation	**H1-1b**			Span	**1**

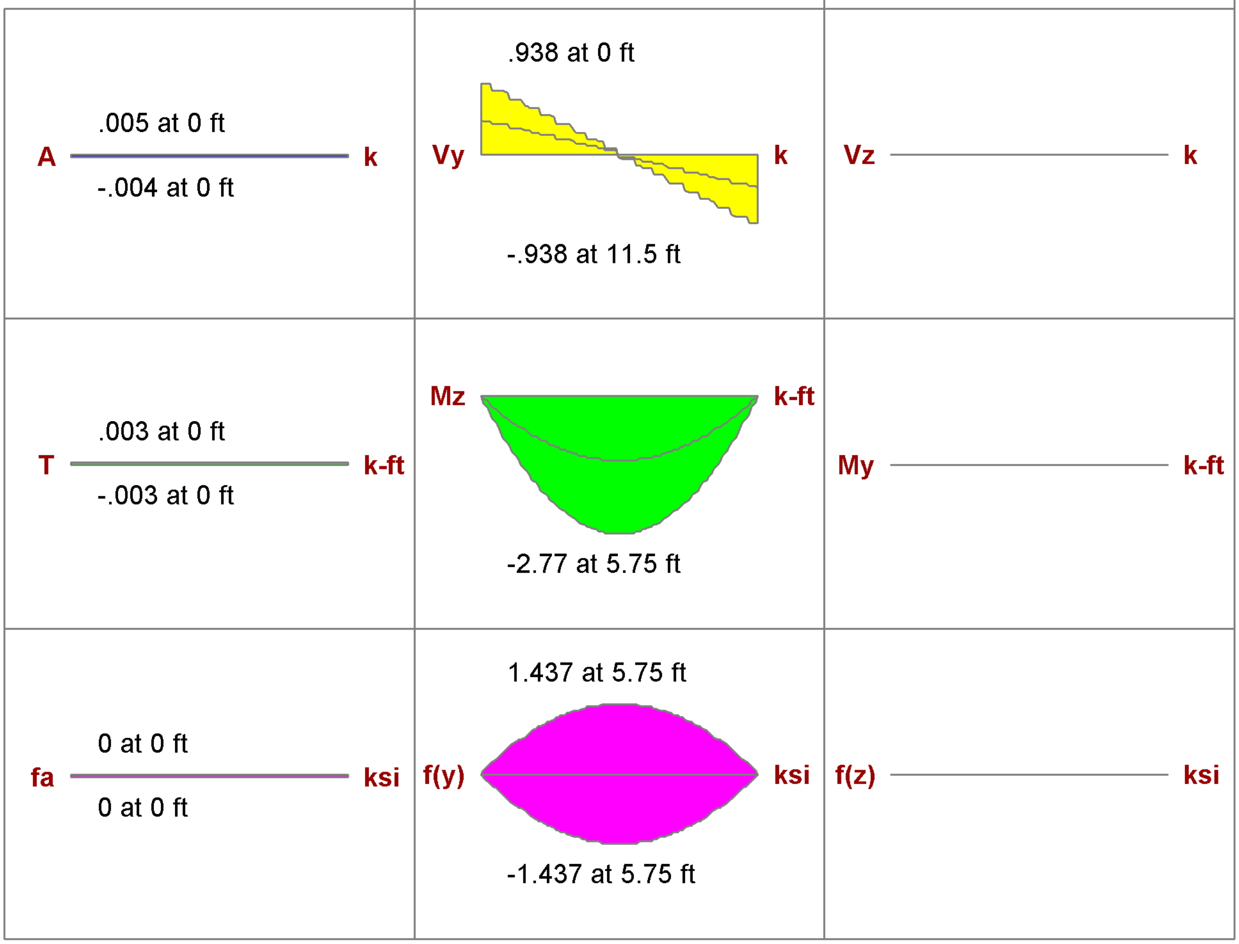
Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**50 ksi**	Lb	**2.667 ft**	**15.833 ft**
phi*Pnc	**297.865 k**	KL/r	**23.512**	**43.677**
phi*Pnt	**342.45 k**			
phi*Mny	**28.125 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**117.375 k-ft**	L-torque	**15.833 ft**	
phi*Vny	**80.34 k**	Tau_b	**1**	
phi*Vnz	**137.095 k**			
Cb	**1**			

Beam: ***2B22***

Shape: **W10x22**
Material: **A992**
Length: **11.5 ft**
I Joint: **N42**
J Joint: **N43**
Envelope (Ω)
Code Check: **0.028 (LC 84)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

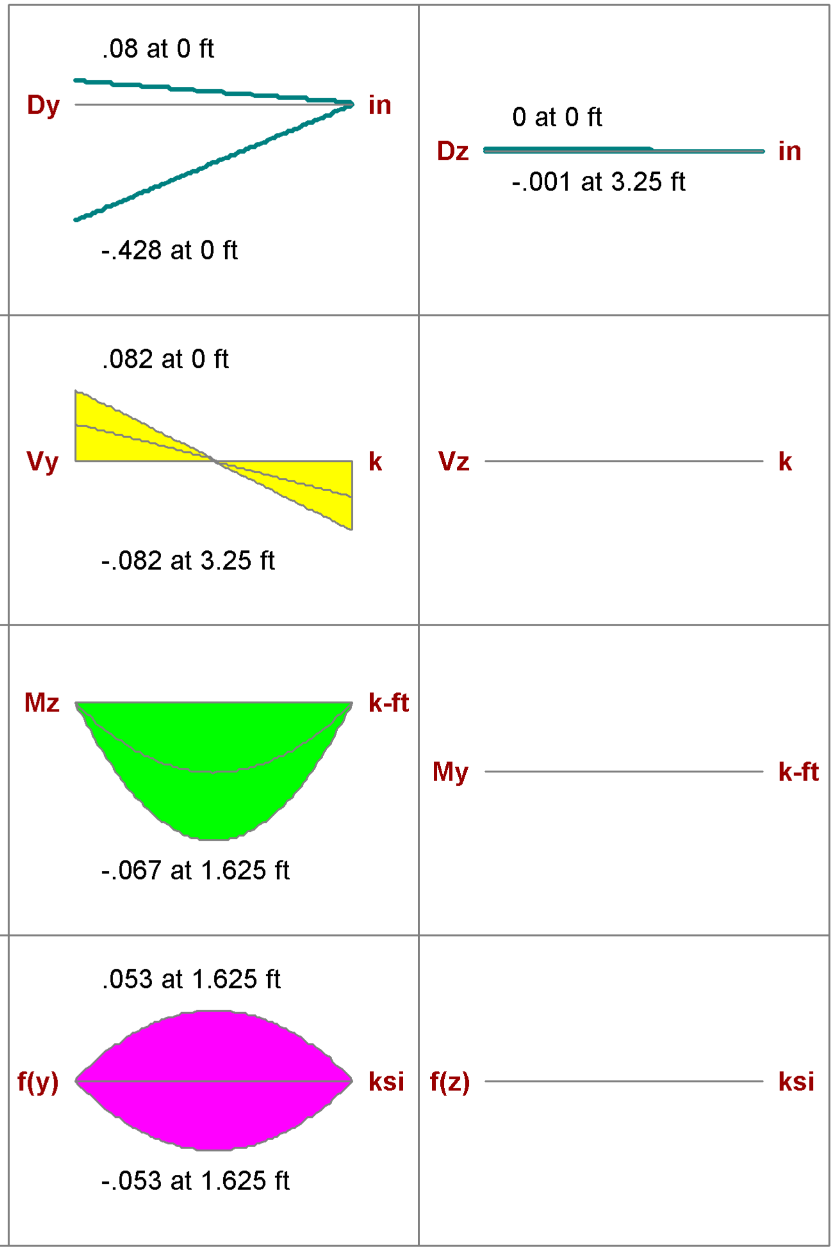
Max Bending Check	**0.028 (LC 84)**	Max Shear Check	**0.015 (y) (LC 83)**	Max Defl Ratio	**L/5740**
Location	**5.75 ft**	Location	**11.5 ft**	Location	**5.75 ft**
Equation	**H1-1b**			Span	**1**

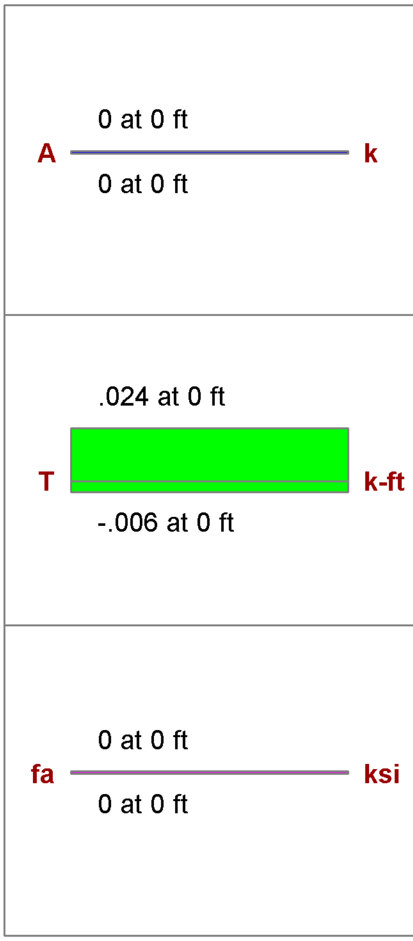
Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**50 ksi**	Lb	**11.5 ft**	**11.5 ft**
phi*Pnc	**132.186 k**	KL/r	**104.124**	**32.364**
phi*Pnt	**292.05 k**			
phi*Mny	**22.875 k-ft**	L Comp Flange	**.667 ft**	
phi*Mnz	**97.5 k-ft**	L-torque	**11.5 ft**	
phi*Vny	**73.44 k**	Tau_b	**1**	
phi*Vnz	**111.78 k**			
Cb	**1.005**			

Beam: ***2B26***

Shape: **W8x18**
Material: **A992**
Length: **3.25 ft**
I Joint: **N23**
J Joint: **N42**
Envelope
Code Check: **0.021 (LC 28)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

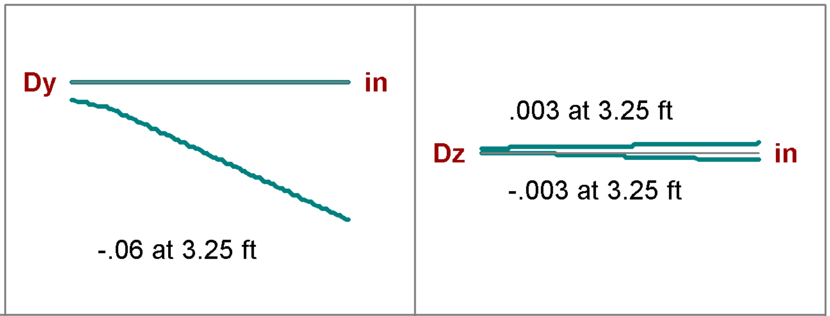
Max Bending Check	**0.001 (LC 43)**	Max Shear Check	**0.021 (z) (LC 28)**	Max Defl Ratio	**L/10000**
Location	**1.625 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**2**

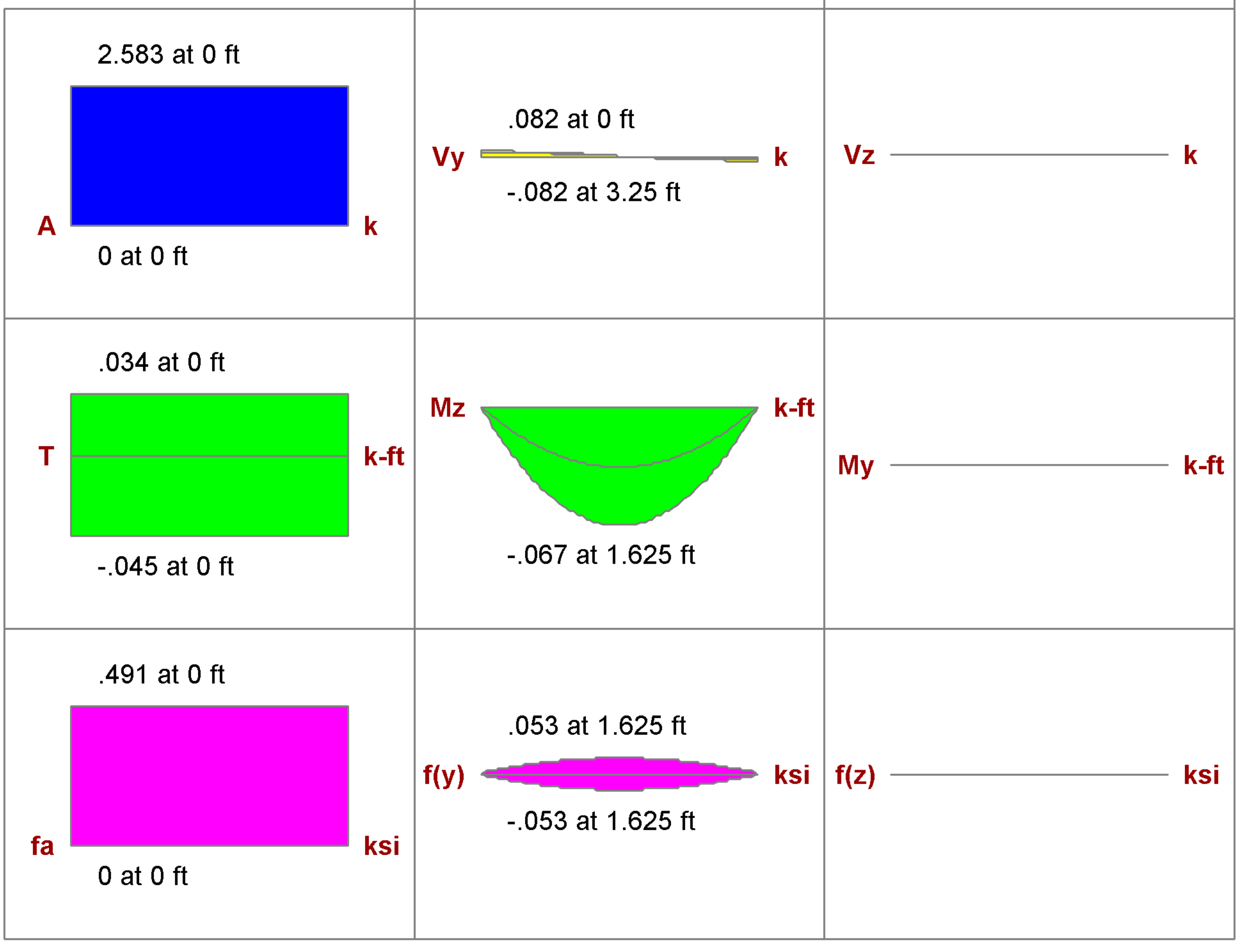
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**3.25 ft**	**3.25 ft**
phi*Pnc	**219.949 k**	KL/r	**31.683**	**11.369**
phi*Pnt	**236.7 k**			
phi*Mny	**17.475 k-ft**	L Comp Flange	**1 ft**	
phi*Mnz	**63.75 k-ft**	L-torque	**3.25 ft**	
phi*Vny	**56.166 k**	Tau_b	**1**	
phi*Vnz	**93.555 k**			
Cb	**1.016**			

Beam: **2B20**

Shape: **W8x18**
Material: **A992**
Length: **3.25 ft**
I Joint: **N43**
J Joint: **N22**
Envelope (Ω)
Code Check: **0.038 (LC 84)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.012 (LC 85)**	Max Shear Check	**0.038 (z) (LC 84)**	Max Defl Ratio	**L/10000**
Location	**0 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**3.25 ft**	**3.25 ft**
phi*Pnc	**219.949 k**	KL/r	**31.683**	**11.369**
phi*Pnt	**236.7 k**			
phi*Mny	**17.475 k-ft**	L Comp Flange	**1 ft**	
phi*Mnz	**63.75 k-ft**	L-torque	**3.25 ft**	
phi*Vny	**56.166 k**	Tau_b	**1**	
phi*Vnz	**93.555 k**			
Cb	**1.514**			

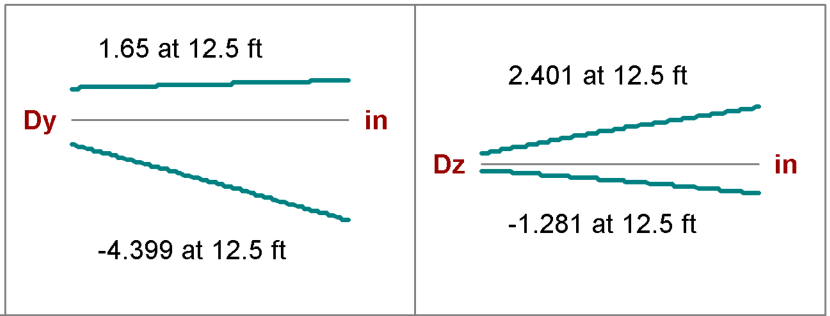
Beam: ***3B9***

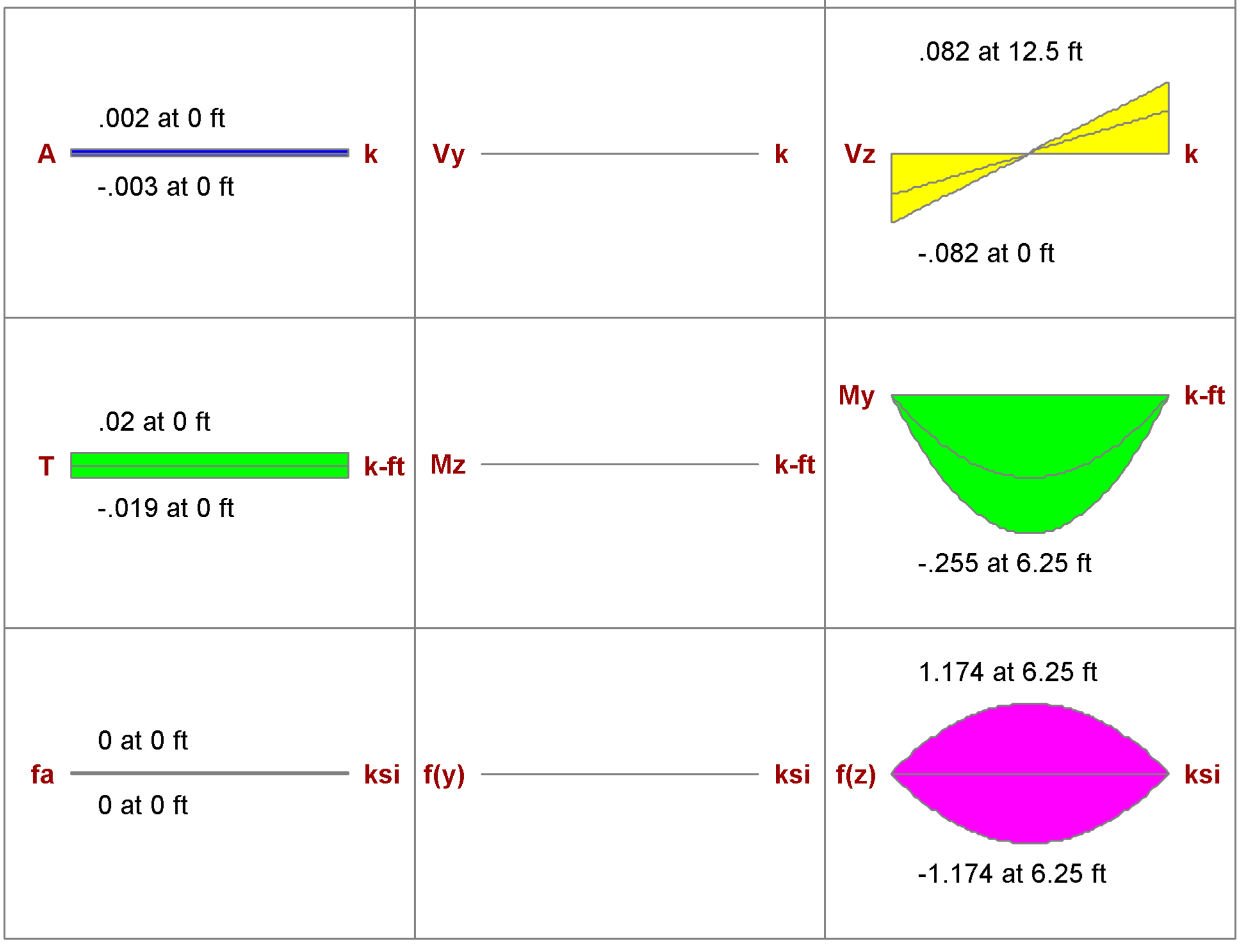
Shape:	**HSS4x3x4**
Material:	**A992**
Length:	**12.5 ft**
I Joint:	**N2**
J Joint:	**N16**

Envelope

Code Check: **0.022 (LC 41)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.022 (LC 41)**	Max Shear Check	**0.005 (z) (LC 44)**	Max Defl Ratio	**L/10000**
Location	**6.25 ft**	Location	**12.5 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**12.5 ft**	**12.5 ft**
phi*Pnc	**39.259 k**	KL/r	**129.404**	**103.181**
phi*Pnt	**130.95 k**			
phi*Mny	**11.7 k-ft**	L Comp Flange	**12.5 ft**	
phi*Mnz	**14.287 k-ft**	L-torque	**12.5 ft**	
phi*Vny	**41.533 k**	Tau_b	**1**	
phi*Vnz	**28.951 k**			
phi*Tn	**10.819 k-ft**			
Cb	**1**			

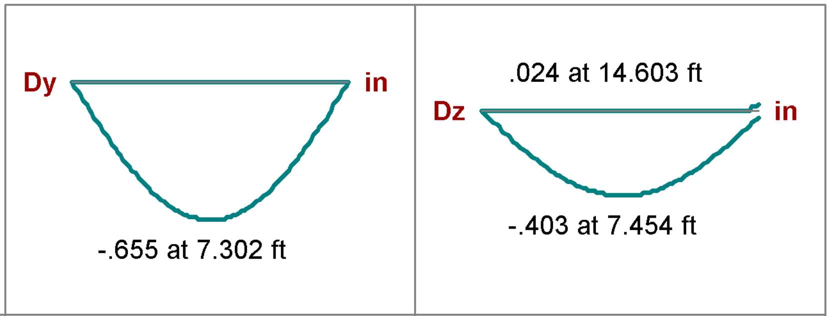
VBrace: ***BR-7***

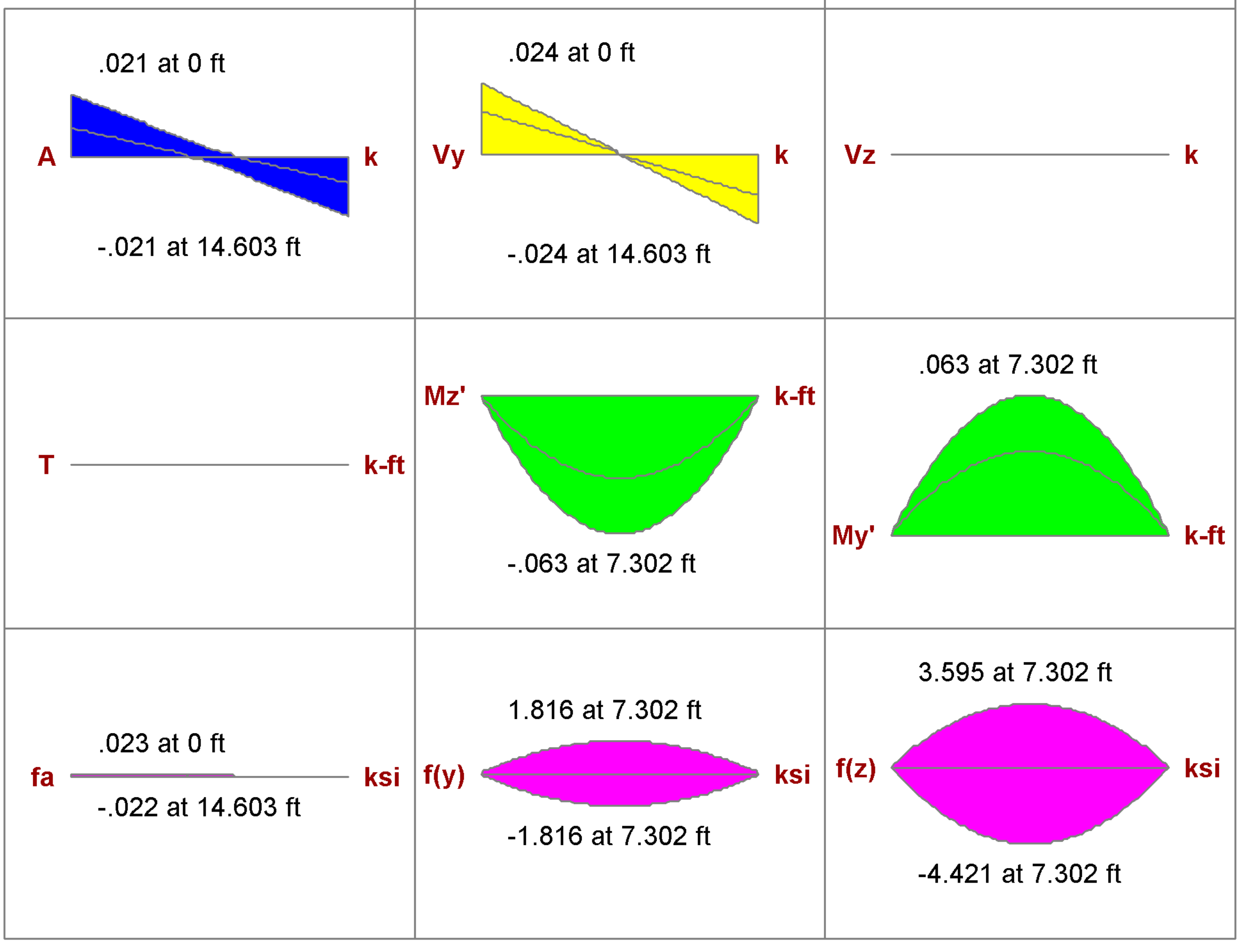
Shape:	**L2x2x4**
Material:	**A36 Gr.36**
Length:	**14.603 ft**
I Joint:	**N47**
J Joint:	**N42**

Envelope

Code Check: **0.155 (LC 42)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

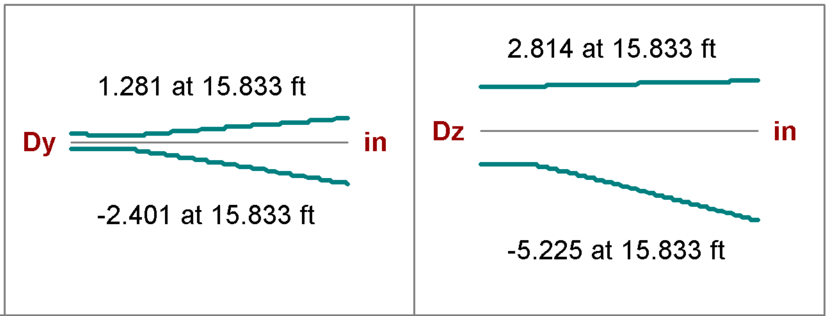
Direct Analysis Method

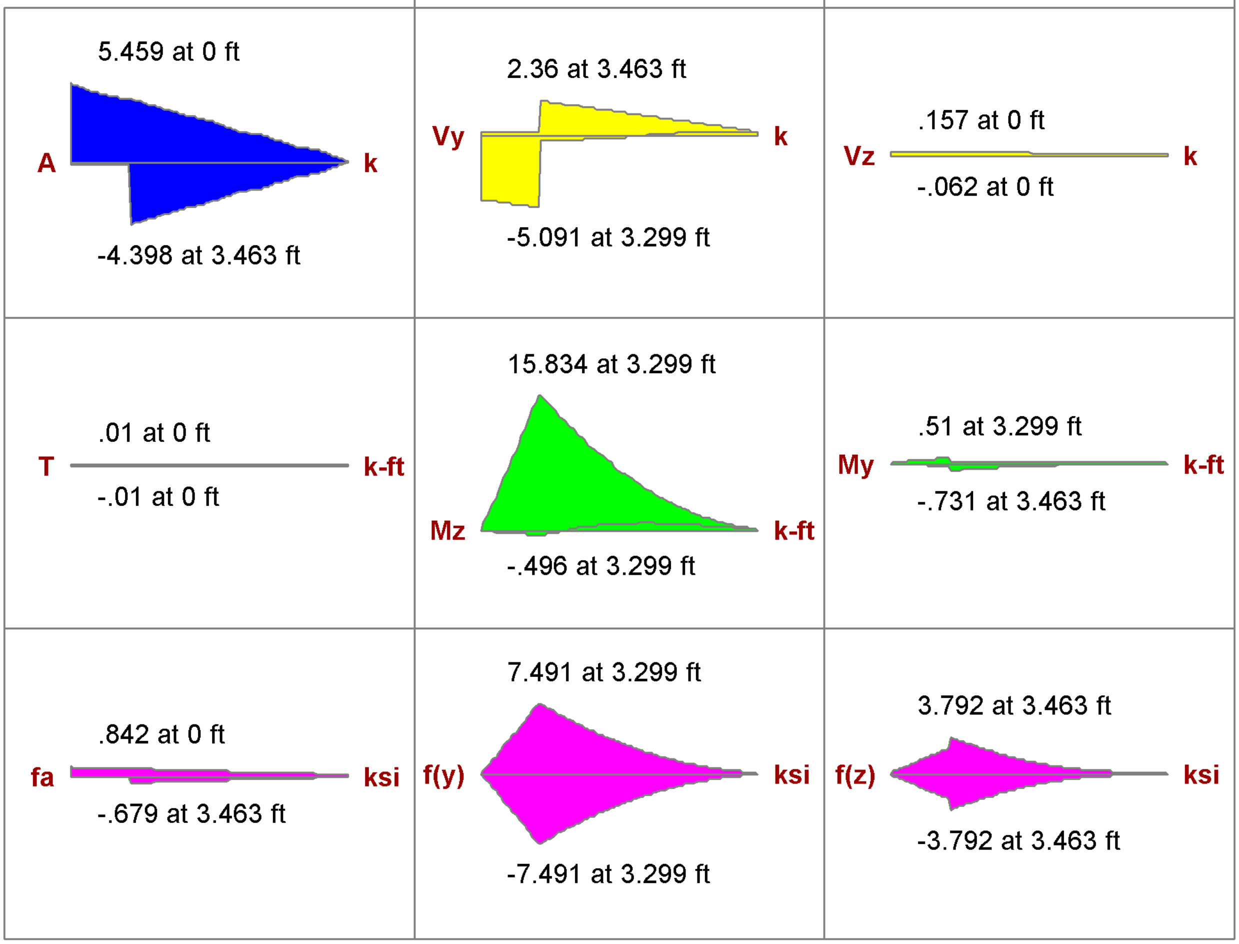
Max Bending Check	**0.155 (LC 42)**	Max Shear Check	**0.003 (y) (LC 41)**
Location	**6.845 ft**	Location	**0 ft**
Equation	**H2-1**	Max Defl Ratio	**L/0**
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**14.603 ft**	**14.603 ft**
phi*Pnc	**1.04 k**	KL/r	**452.809**	**229.449**
phi*Pnt	**30.586 k**			
phi*Mny'	**.691 k-ft**	L Comp Flange	**14.603 ft**	
phi*Mnz'	**1.028 k-ft**	L-torque	**14.603 ft**	
phi*Vny	**9.72 k**	Tau_b	**1**	
phi*Vnz	**9.72 k**			
Cb	**1.136**			

Beam: ***RB21***

Shape: **W12x22**
Material: **A992**
Length: **15.833 ft**
I Joint: **N10**
J Joint: **N1**
Envelope
Code Check: **0.318 (LC 25)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.318 (LC 25)**	Max Shear Check	**0.054 (y) (LC 25)**	Max Defl Ratio	**L/2795**
Location	**3.299 ft**	Location	**3.299 ft**	Location	**7.917 ft**
Equation	**H1-1b**			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.949**

			y-y	z-z
Fy	**50 ksi**	Lb	**2.667 ft**	**1.33 ft**
phi*Pnc	**250.738 k**	KL/r	**37.74**	**3.253**
phi*Pnt	**291.6 k**			
phi*Mny	**13.725 k-ft**	L Comp Flange	**15.833 ft**	
phi*Mnz	**52.003 k-ft**	L-torque	**15.833 ft**	
phi*Vny	**95.94 k**	Tau_b	**1**	
phi*Vnz	**92.489 k**			
Cb	**1.688**			

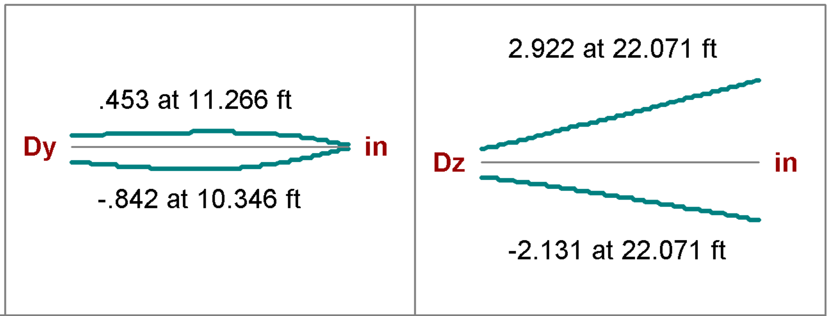
Beam: ***RB24***

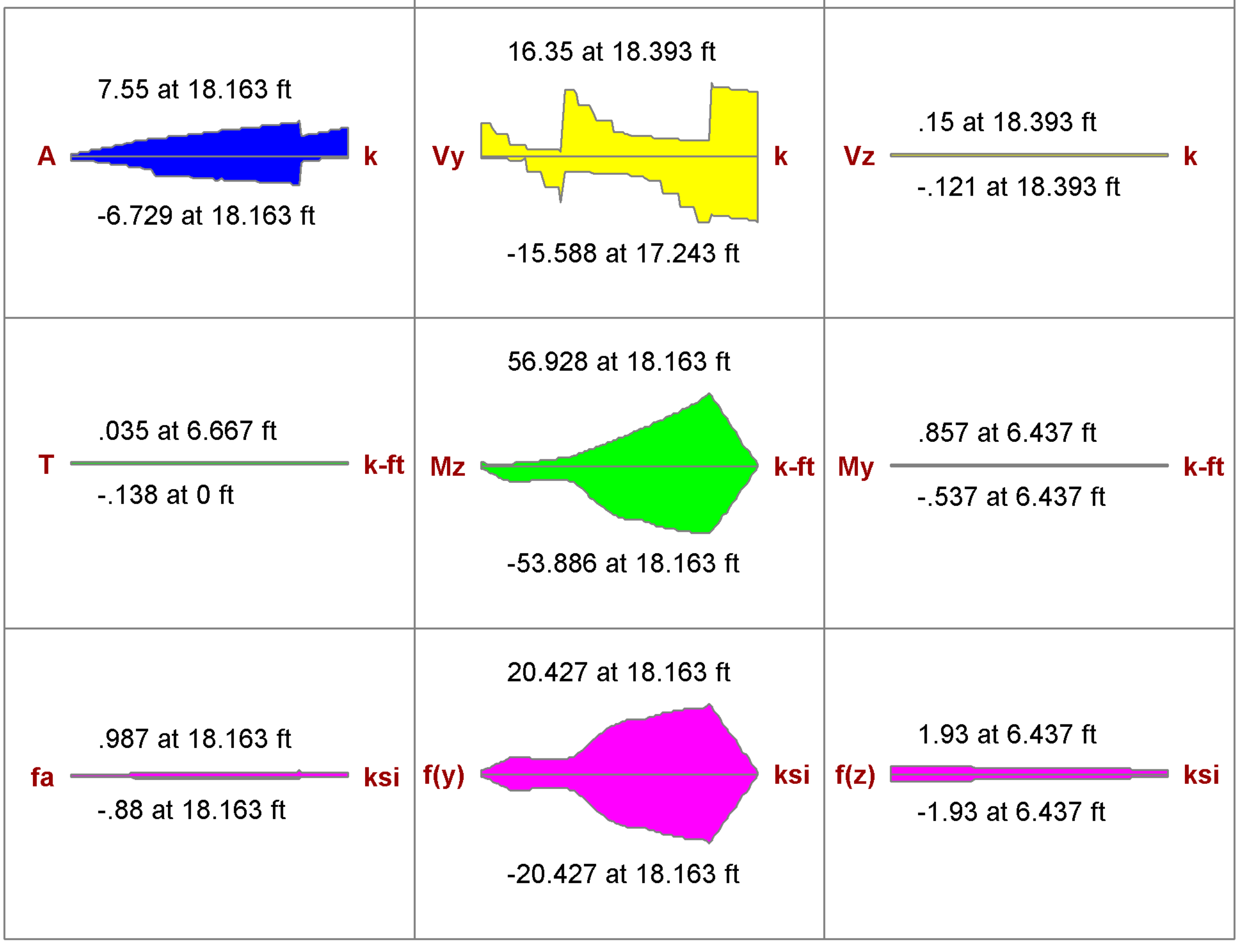
Shape:	**W12x26**
Material:	**A992**
Length:	**22.071 ft**
I Joint:	**N5**
J Joint:	**N10**

Envelope (Ω)

Code Check: **0.626 (LC 86)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.626 (LC 86)**	Max Shear Check	**0.195 (y) (LC 82)**	Max Defl Ratio	**L/615**
Location	**18.163 ft**	Location	**18.393 ft**	Location	**12.415 ft**
Equation	**H1-1b**			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.924**

			y-y	z-z
Fy	**50 ksi**	Lb	**1.33 ft**	**1.33 ft**
phi*Pnc	**315.692 k**	KL/r	**10.613**	**3.091**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**22.071 ft**	
phi*Mnz	**89.025 k-ft**	L-torque	**22.071 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1.926**			

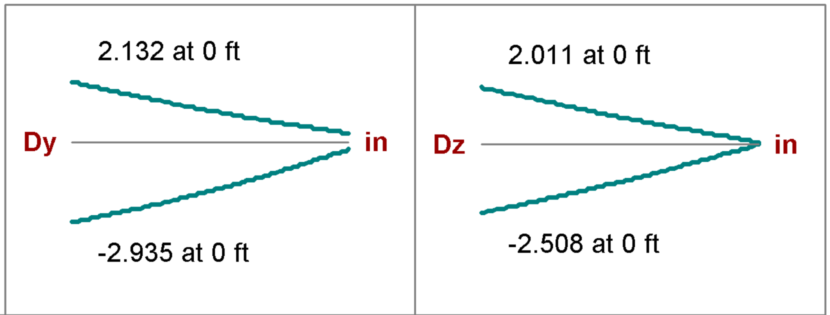
VBrace: ***X-BR 1A***

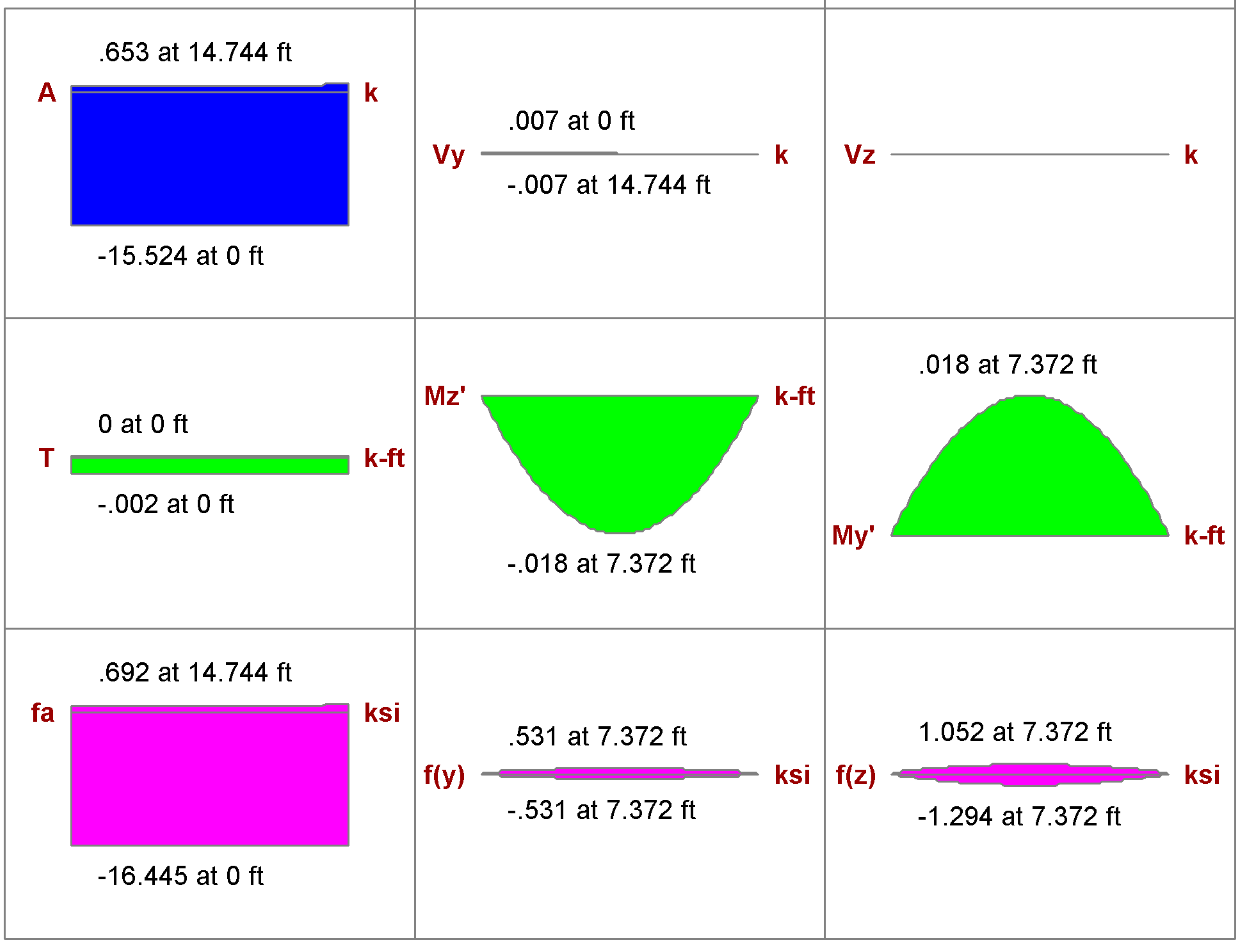
Shape:	**L2x2x4**
Material:	**A36 Gr.36**
Length:	**14.744 ft**
I Joint:	**N8**
J Joint:	**N24**

Envelope

Code Check: **0.508 (LC 43)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

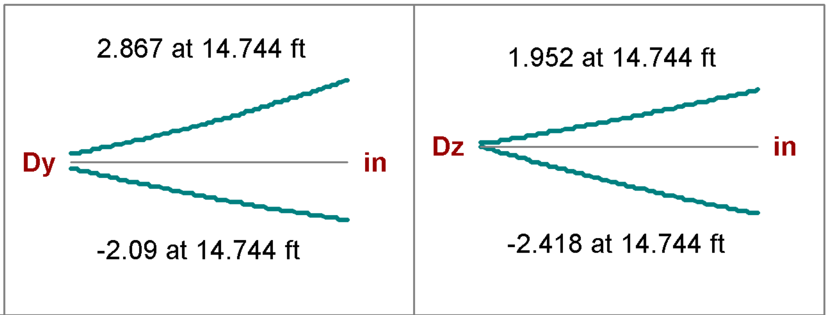
Max Bending Check	**0.508 (LC 43)**	Max Shear Check	**0.012 (y) (LC 38)**
Location	**0 ft**	Location	**14.744 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

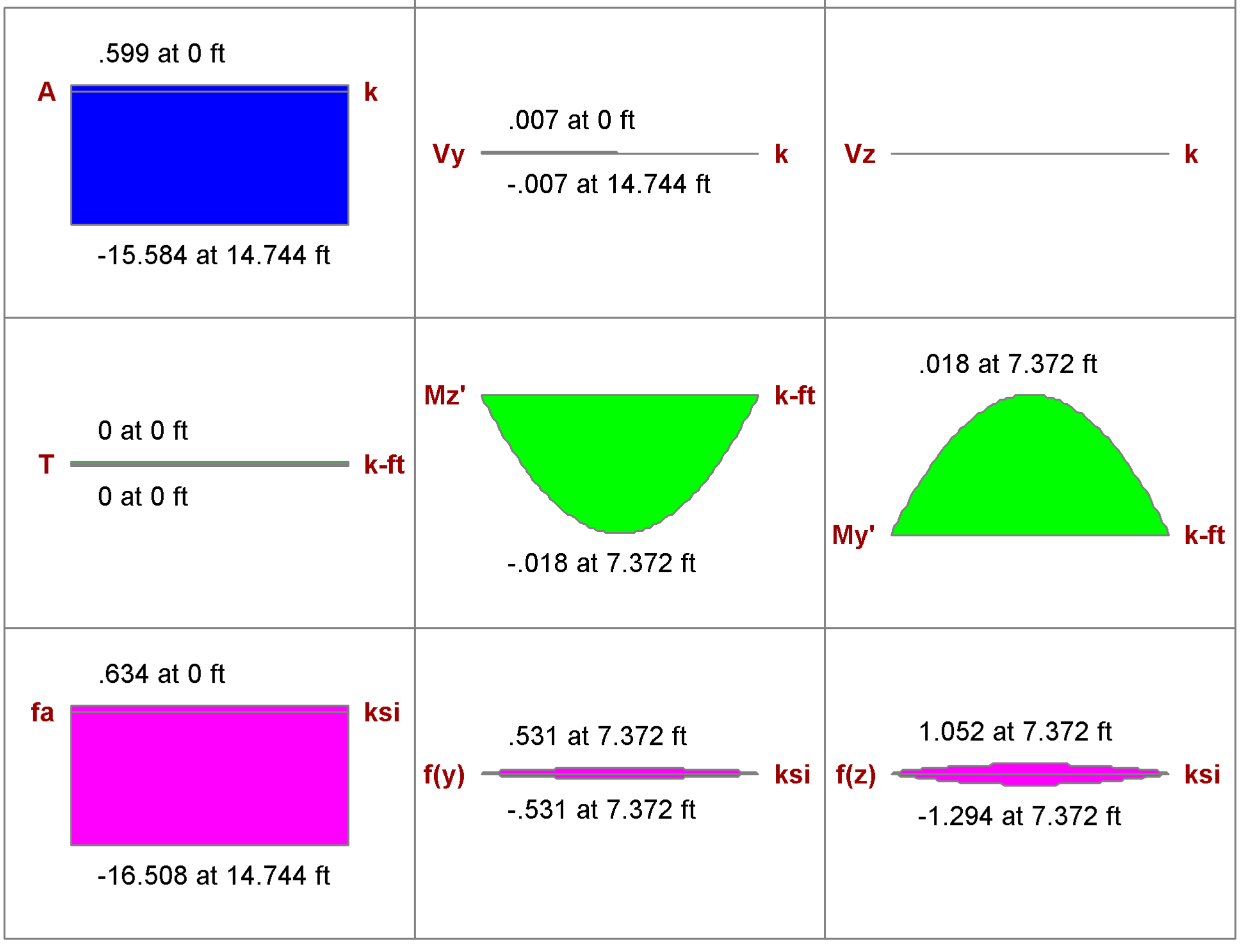
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**14.744 ft**	**14.744 ft**
phi*Pnc	**1.02 k**	KL/r	**457.168**	**231.658**
phi*Pnt	**30.586 k**			
phi*Mny'	**.691 k-ft**	L Comp Flange	**14.744 ft**	
phi*Mnz'	**1.023 k-ft**	L-torque	**14.744 ft**	
phi*Vny	**9.72 k**	Tau_b	**1**	
phi*Vnz	**9.72 k**			
Cb	**1.136**			

VBrace: ***X-BR 1B***

Shape: **L2x2x4**
Material: **A36 Gr.36**
Length: **14.744 ft**
I Joint: **N32**
J Joint: **N10**
Envelope
Code Check: **0.510 (LC 41)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

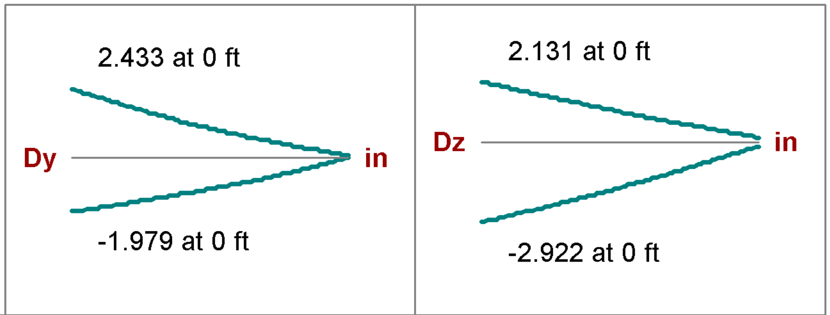
Max Bending Check	**0.510 (LC 41)**	Max Shear Check	**0.002 (y) (LC 41)**
Location	**14.744 ft**	Location	**14.744 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

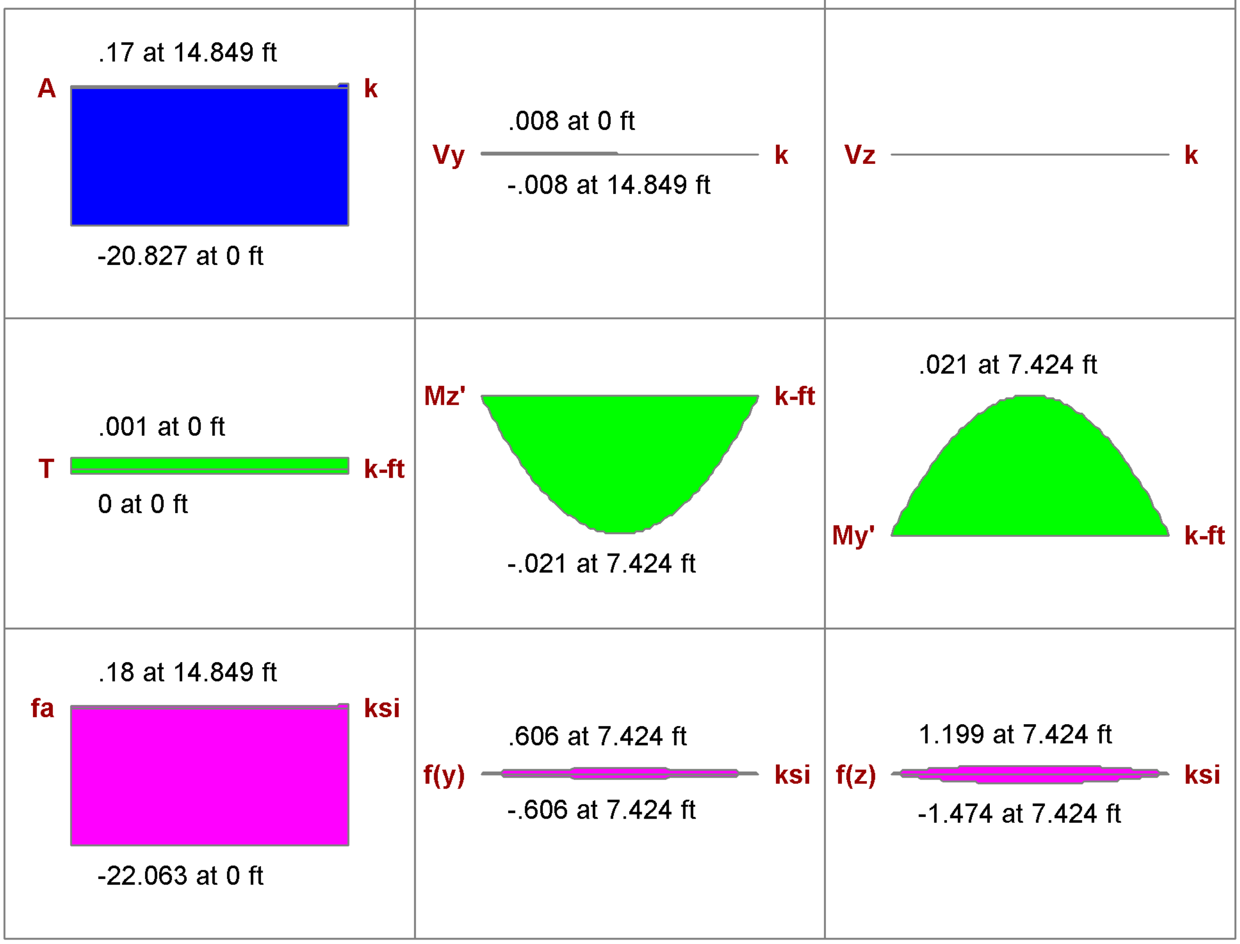
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**14.744 ft**	**14.744 ft**
phi*Pnc	**1.02 k**	KL/r	**457.168**	**231.658**
phi*Pnt	**30.586 k**			
phi*Mny'	**.691 k-ft**	L Comp Flange	**14.744 ft**	
phi*Mnz'	**1.023 k-ft**	L-torque	**14.744 ft**	
phi*Vny	**9.72 k**	Tau_b	**1**	
phi*Vnz	**9.72 k**			
Cb	**1.136**			

VBrace: ***X-BR 2A***

Shape: **L2x2x4**
Material: **A36 Gr.36**
Length: **14.849 ft**
I Joint: **N10**
J Joint: **N22**
Envelope
Code Check: **0.681 (LC 42)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

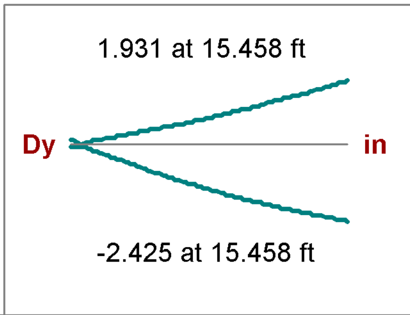
Max Bending Check	**0.681 (LC 42)**	Max Shear Check	**0.009 (y) (LC 43)**
Location	**0 ft**	Location	**14.849 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

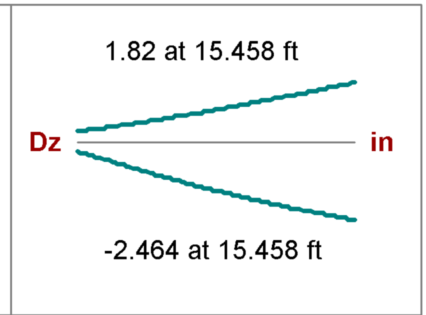
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

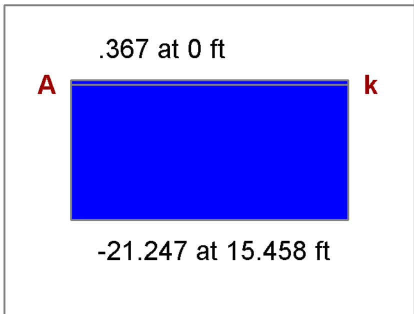
			y-y'	z-z'
Fy	**36 ksi**	Lb	**14.849 ft**	**14.849 ft**
phi*Pnc	**1.006 k**	KL/r	**460.429**	**233.31**
phi*Pnt	**30.586 k**			
phi*Mny'	**.691 k-ft**	L Comp Flange	**14.849 ft**	
phi*Mnz'	**1.02 k-ft**	L-torque	**14.849 ft**	
phi*Vny	**9.72 k**	Tau_b	**1**	
phi*Vnz	**9.72 k**			
Cb	**1.136**			

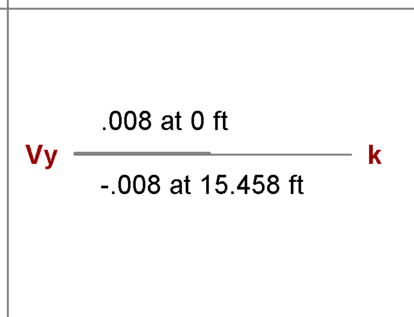
VBrace: ***X-BR 2B***

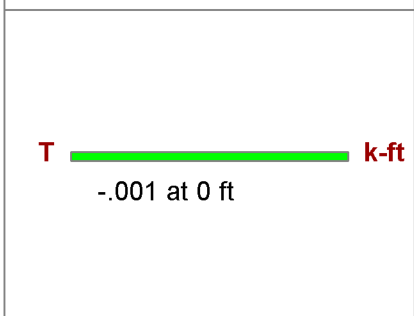
Shape: **L2x2x4**
Material: **A36 Gr.36**
Length: **15.458 ft**
I Joint: **N24**
J Joint: **N6**
Envelope
Code Check: **0.695 (LC 44)**
Report Based On 97 Sections

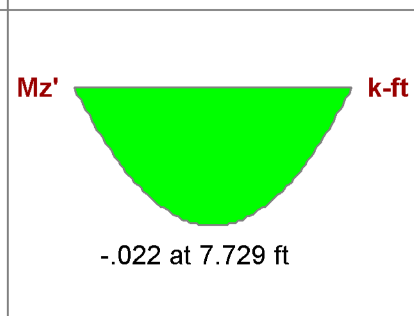


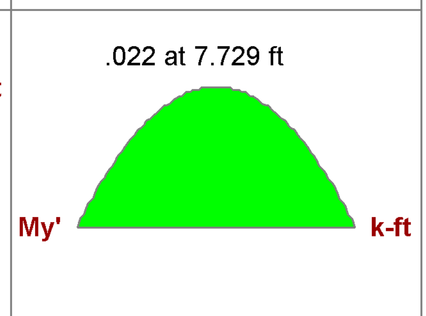


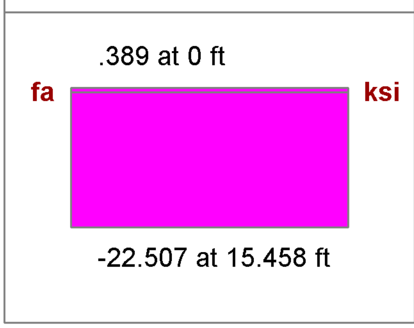


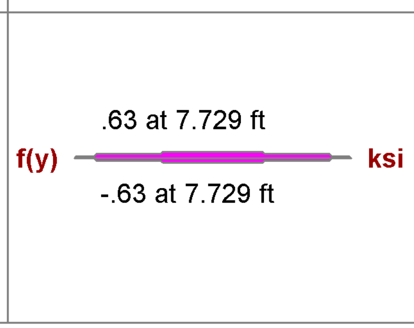


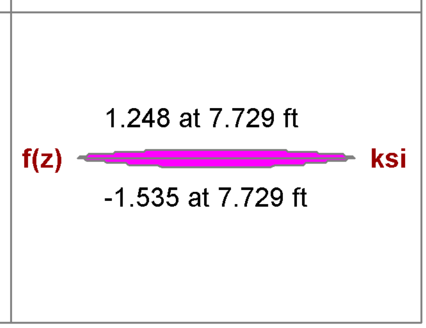












AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

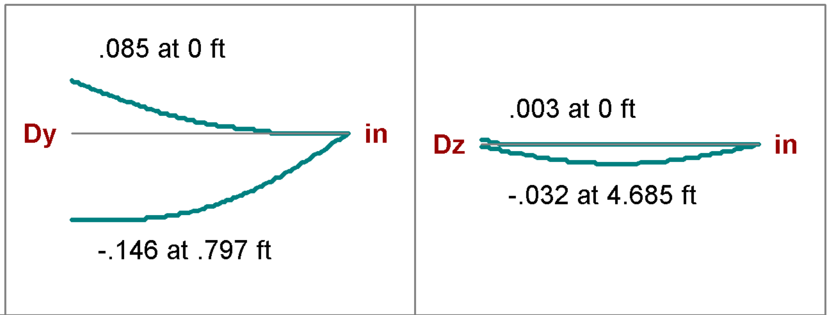
Max Bending Check	**0.695 (LC 44)**	Max Shear Check	**0.009 (y) (LC 47)**
Location	**15.458 ft**	Location	**15.458 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

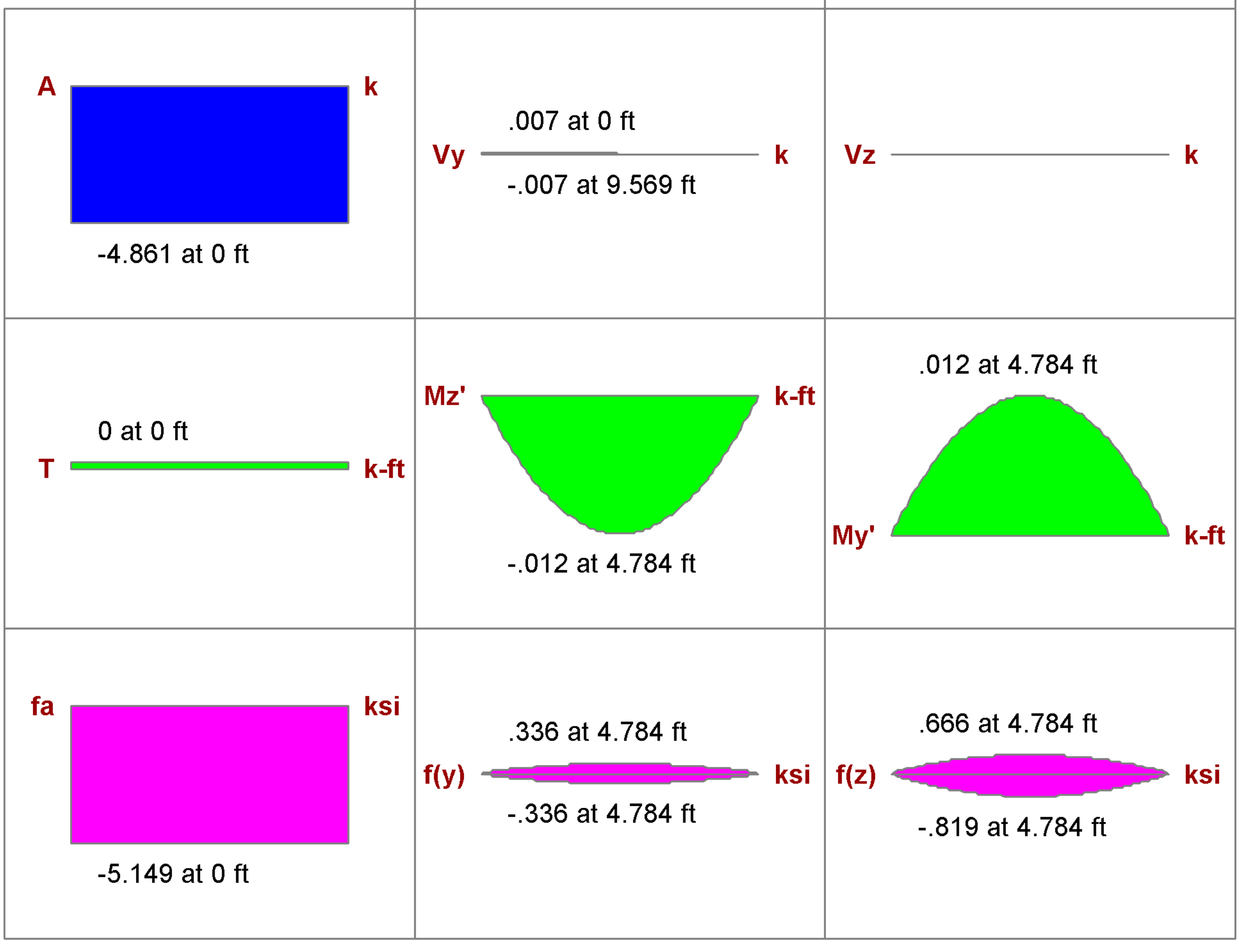
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**15.458 ft**	**15.458 ft**
phi*Pnc	**.928 k**	KL/r	**479.304**	**242.875**
phi*Pnt	**30.586 k**			
phi*Mny'	**.691 k-ft**	L Comp Flange	**15.458 ft**	
phi*Mnz'	**1 k-ft**	L-torque	**15.458 ft**	
phi*Vny	**9.72 k**	Tau_b	**1**	
phi*Vnz	**9.72 k**			
Cb	**1.136**			

VBrace: ***X-BR 6A***

Shape: **L2x2x4**
Material: **A36 Gr.36**
Length: **9.569 ft**
I Joint: **N22**
J Joint: **N47**
Envelope
Code Check: **0.159 (LC 47)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method

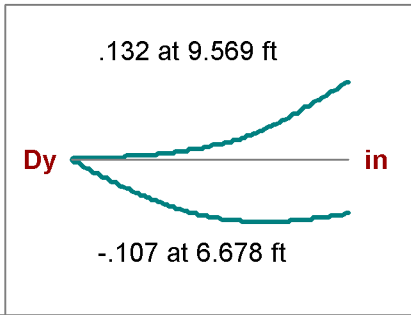
Max Bending Check	**0.159 (LC 47)**	Max Shear Check	**0.003 (y) (LC 47)**
Location	**0 ft**	Location	**0 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

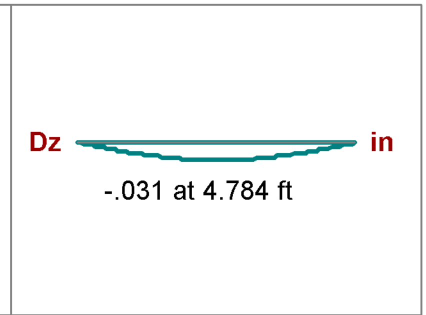
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

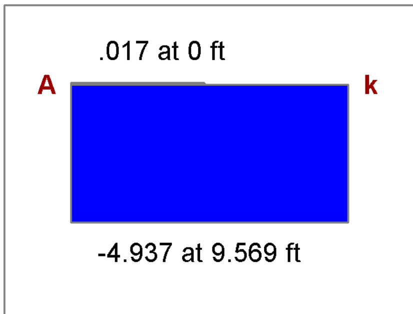
			y-y'	z-z'
Fy	**36 ksi**	Lb	**9.569 ft**	**9.569 ft**
phi*Pnc	**2.422 k**	KL/r	**296.708**	**150.349**
phi*Pnt	**30.586 k**			
phi*Mny'	**.691 k-ft**	L Comp Flange	**9.569 ft**	
phi*Mnz'	**1.217 k-ft**	L-torque	**9.569 ft**	
phi*Vny	**9.72 k**	Tau_b	**1**	
phi*Vnz	**9.72 k**			
Cb	**1.136**			

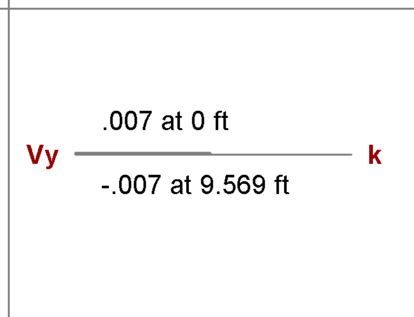
VBrace: ***X-BR 6B***

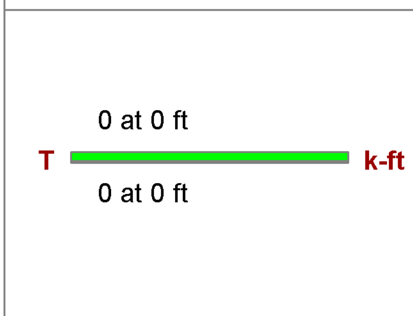
Shape: **L2x2x4**
Material: **A36 Gr.36**
Length: **9.569 ft**
I Joint: **N44**
J Joint: **N43**
Envelope
Code Check: **0.161 (LC 45)**
Report Based On 97 Sections

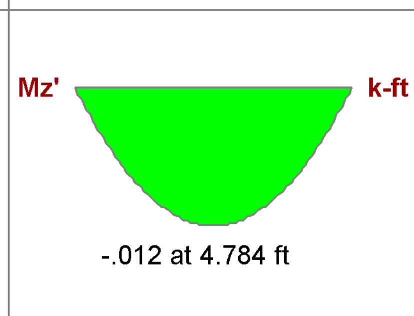


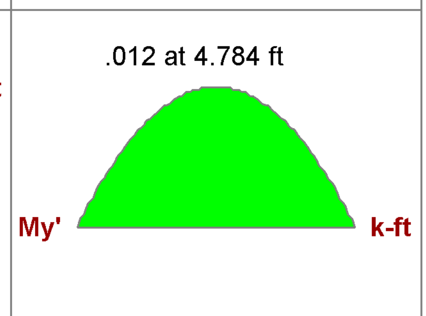


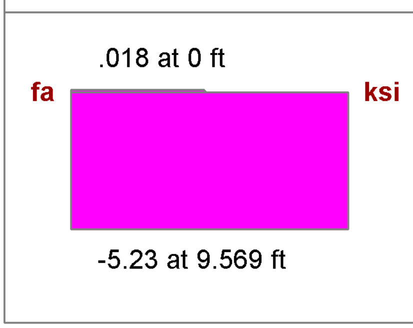


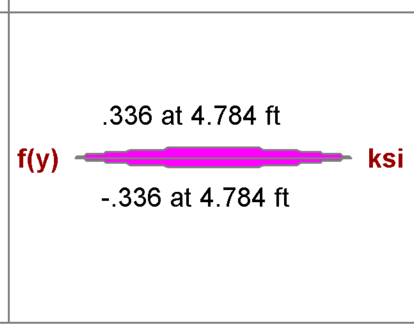


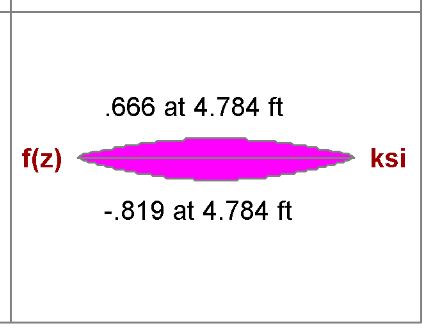












AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.161 (LC 45)**	Max Shear Check	**0.004 (y) (LC 41)**
Location	**9.569 ft**	Location	**0 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**9.569 ft**	**9.569 ft**
phi*Pnc	**2.422 k**	KL/r	**296.708**	**150.349**
phi*Pnt	**30.586 k**			
phi*Mny'	**.691 k-ft**	L Comp Flange	**9.569 ft**	
phi*Mnz'	**1.217 k-ft**	L-torque	**9.569 ft**	
phi*Vny	**9.72 k**	Tau_b	**1**	
phi*Vnz	**9.72 k**			
Cb	**1.136**			

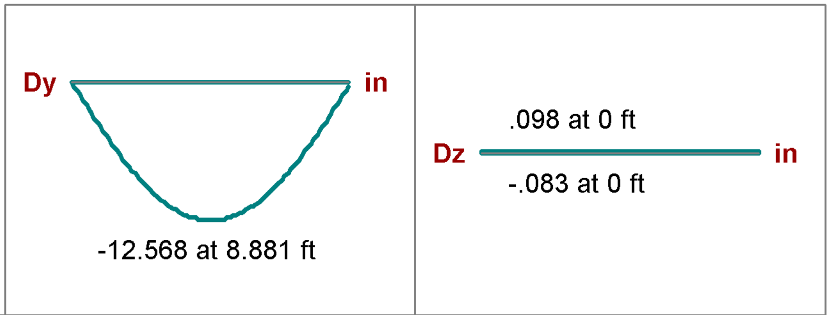
Blackwell

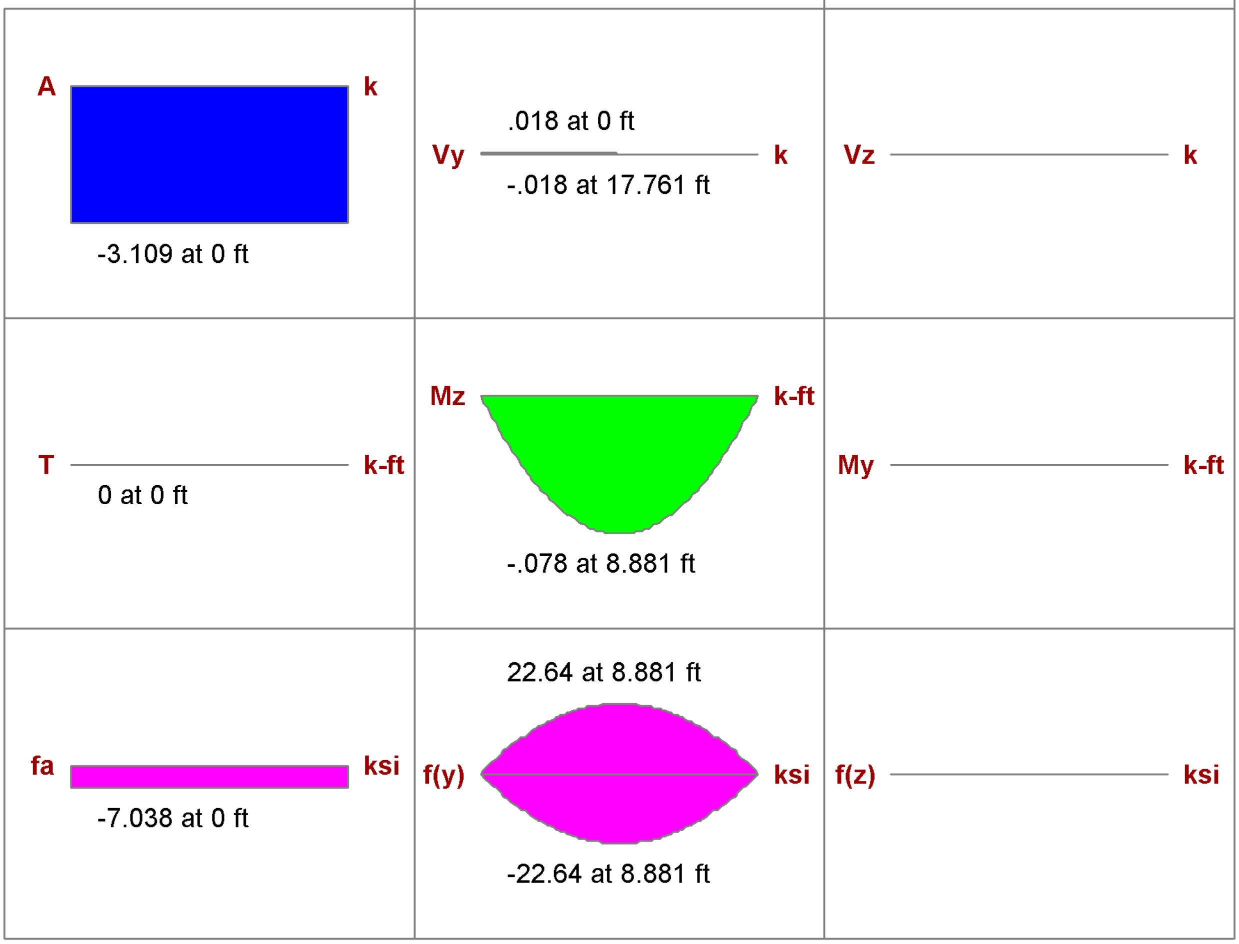
Structural Engineers

APPENDIX D - VOLUME 2, 3 AND 4
LATERAL DETAIL MEMBER REPORTS

HBrace: ***X-BR 6B***

Shape: **R0.75**
Material: **A36 Gr.36**
Length: **17.761 ft**
I Joint: **F8_N94**
J Joint: **N109**
Envelope
Code Check: **0.217 (LC 13)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method

Max Bending Check	**0.217 (LC 13)**	Max Shear Check	**0.004 (s) (LC 12)**
Location	**0 ft**	Location	**17.761 ft**
Equation	**H1-1a***	Max Defl Ratio	**L/0**

Bending	**Compact**	Compression	**Non-Slender**

			y-y	z-z
Fy	**36 ksi**	Lb	**17.761 ft**	**17.761 ft**
phi*Pnc	**.077 k**	KL/r	**1136.708**	**1136.708**
phi*Pnt	**14.314 k**			
phi*Mny	**.179 k-ft**	L Comp Flange	**17.761 ft**	
phi*Mnz	**.179 k-ft**	L-torque	**17.761 ft**	
phi*Vny	**8.588 k**	Tau_b	**1**	
phi*Vnz	**8.588 k**			
Cb	**1.136**			

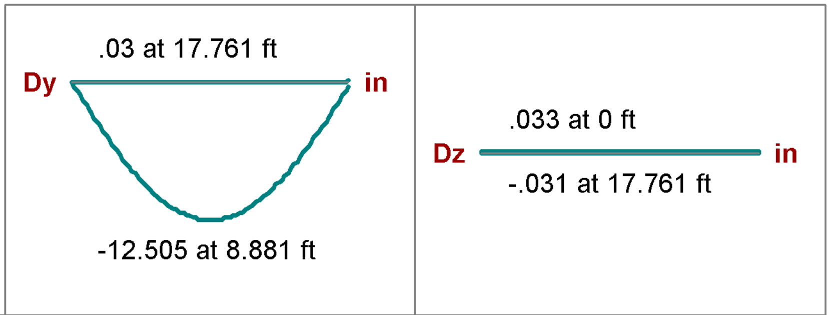
HBrace: ***X-BR 6A***

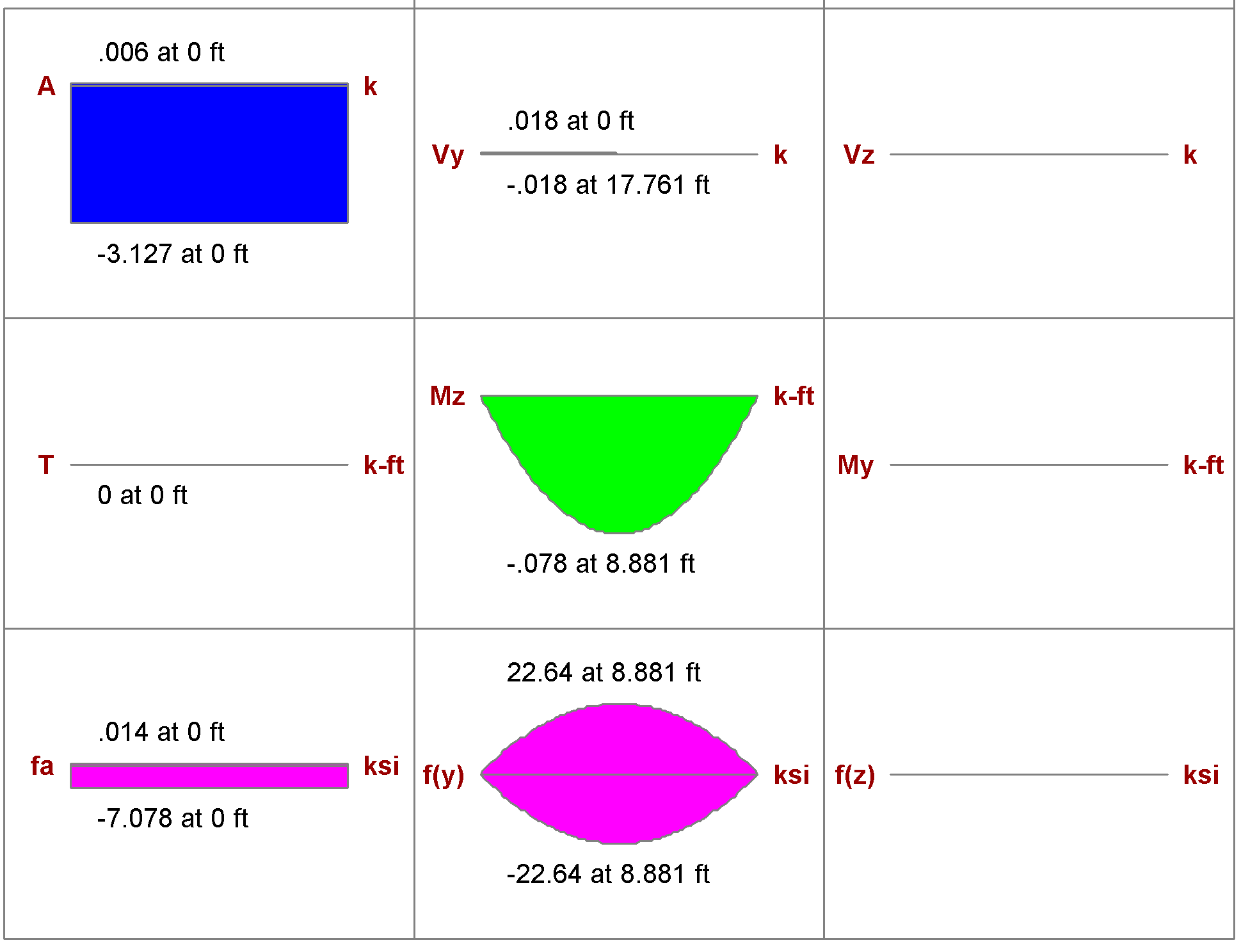
Shape:	**R0.75**
Material:	**A36 Gr.36**
Length:	**17.761 ft**
I Joint:	**F8_N89**
J Joint:	**F8_N95**

Envelope

Code Check: **0.218 (LC 11)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

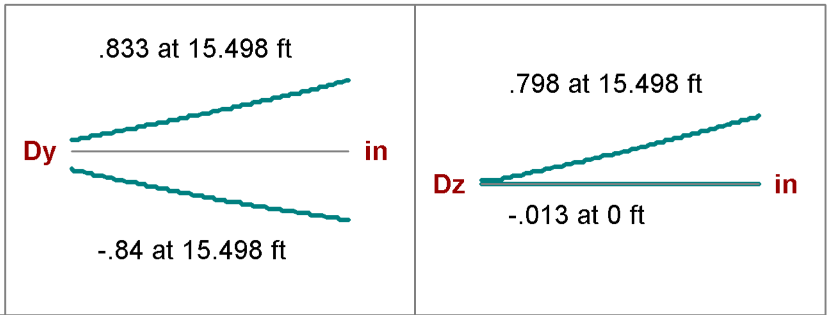
Direct Analysis Method

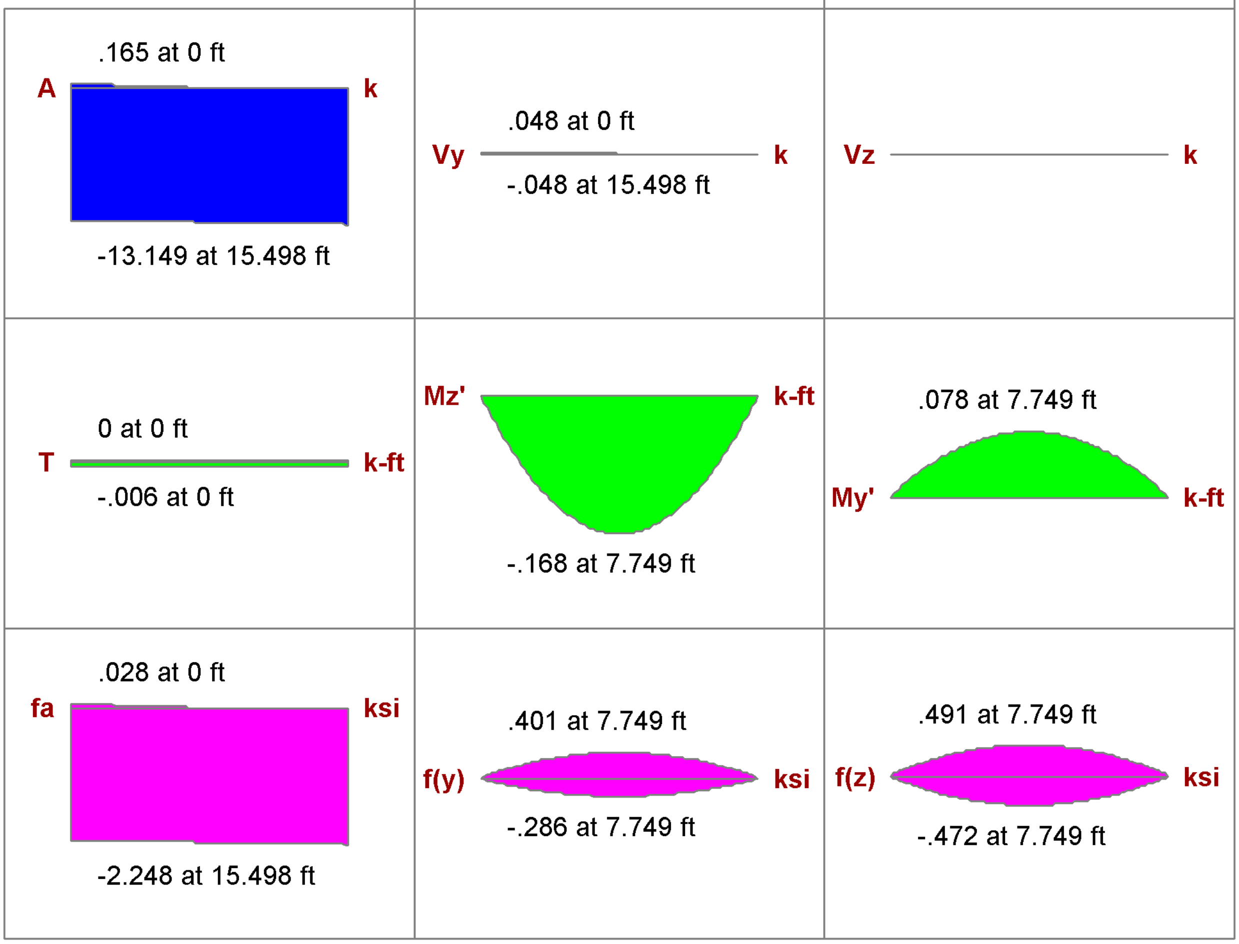
Max Bending Check	**0.218 (LC 11)**	Max Shear Check	**0.004 (s) (LC 12)**
Location	**0 ft**	Location	**17.761 ft**
Equation	**H1-1a***	Max Defl Ratio	**L/0**
Bending	**Compact**	Compression	**Non-Slender**

			y-y	z-z
Fy	**36 ksi**	Lb	**17.761 ft**	**17.761 ft**
phi*Pnc	**.077 k**	KL/r	**1136.708**	**1136.708**
phi*Pnt	**14.314 k**			
phi*Mny	**.179 k-ft**	L Comp Flange	**17.761 ft**	
phi*Mnz	**.179 k-ft**	L-torque	**17.761 ft**	
phi*Vny	**8.588 k**	Tau_b	**1**	
phi*Vnz	**8.588 k**			
Cb	**1.136**			

VBrace: ***X-BR 5B***

Shape: **L5x3.5x12**
Material: **A36 Gr.36**
Length: **15.498 ft**
I Joint: **N236**
J Joint: **N4**
Envelope
Code Check: **0.069 (LC 9)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method

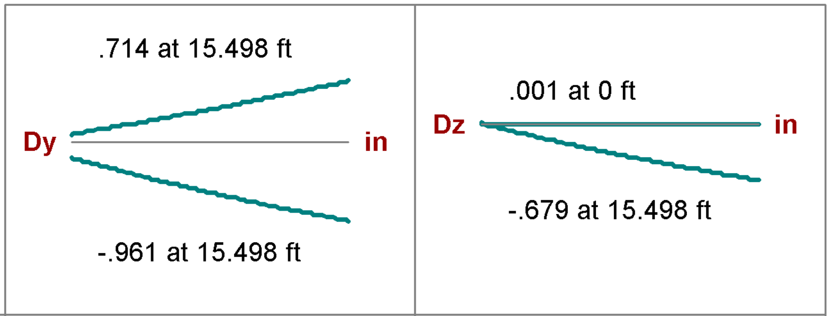
Max Bending Check	**0.069 (LC 9)**	Max Shear Check	**0.003 (y) (LC 9)**
Location	**15.498 ft**	Location	**15.498 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

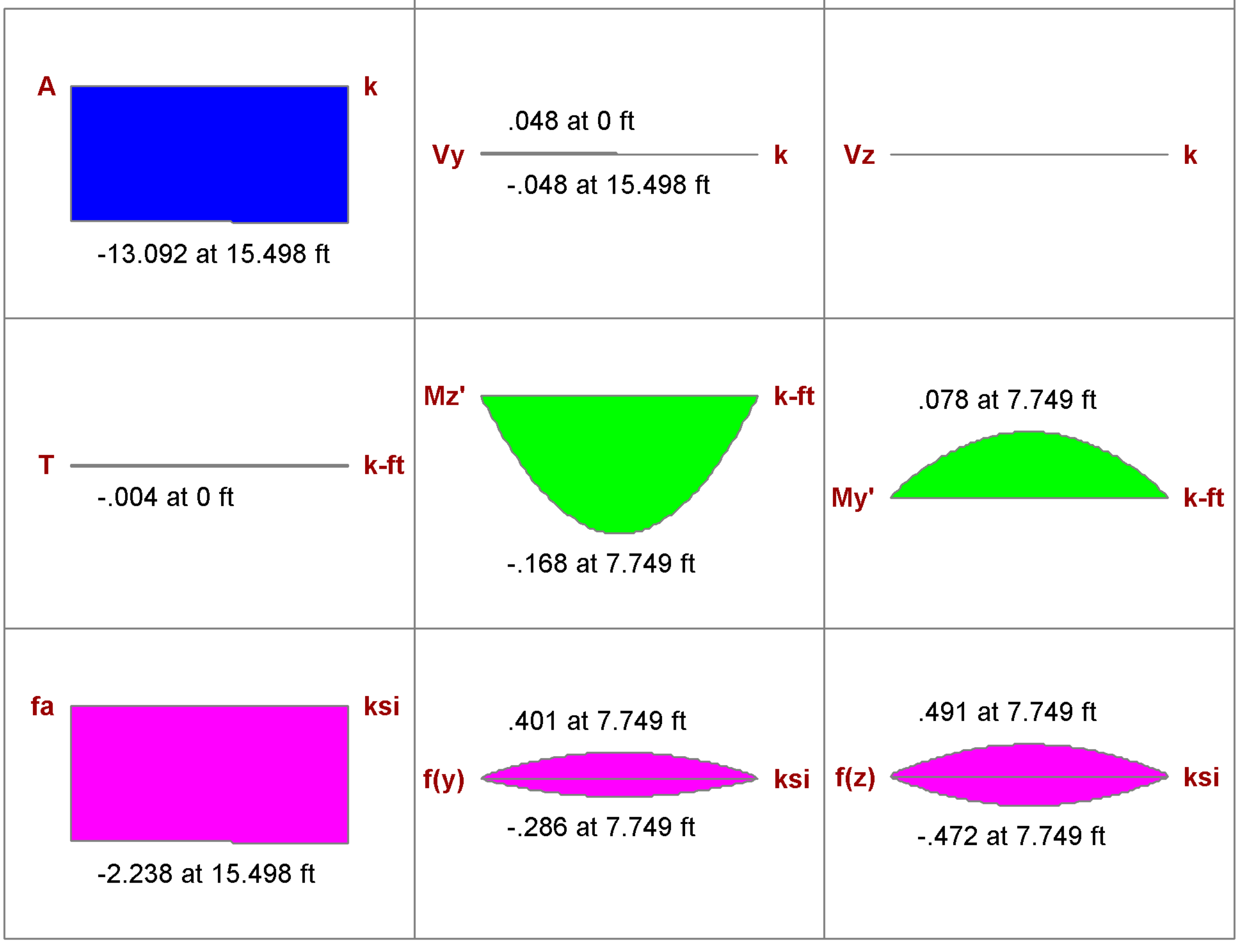
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**15.498 ft**	**15.498 ft**
phi*Pnc	**21.151 k**	KL/r	**249.967**	**111.82**
phi*Pnt	**189.54 k**			
phi*Mny'	**7.058 k-ft**	L Comp Flange	**15.498 ft**	
phi*Mnz'	**17.242 k-ft**	L-torque	**15.498 ft**	
phi*Vny	**72.9 k**	Tau_b	**1**	
phi*Vnz	**51.03 k**			
Cb	**1.136**			

VBrace: ***X-BR 5A***

Shape: **L5x3.5x12**
Material: **A36 Gr.36**
Length: **15.498 ft**
I Joint: **N109**
J Joint: **F1_N199**
Envelope
Code Check: **0.069 (LC 11)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.069 (LC 11)**	Max Shear Check	**0.002 (y) (LC 15)**
Location	**15.498 ft**	Location	**0 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**15.498 ft**	**15.498 ft**
phi*Pnc	**21.151 k**	KL/r	**249.967**	**111.82**
phi*Pnt	**189.54 k**			
phi*Mny'	**7.058 k-ft**	L Comp Flange	**15.498 ft**	
phi*Mnz'	**17.242 k-ft**	L-torque	**15.498 ft**	
phi*Vny	**72.9 k**	Tau_b	**1**	
phi*Vnz	**51.03 k**			
Cb	**1.136**			

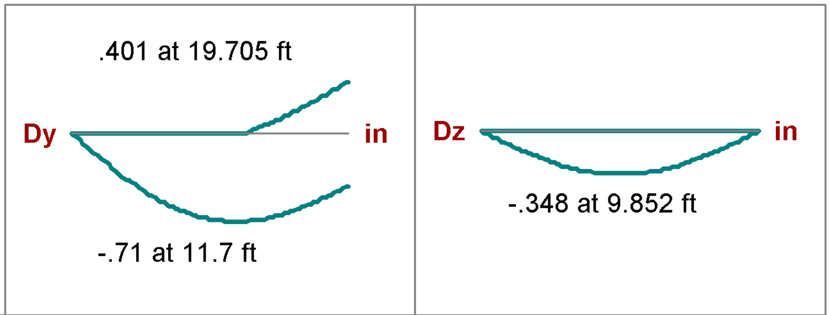
VBrace: ***X-BR 4B***

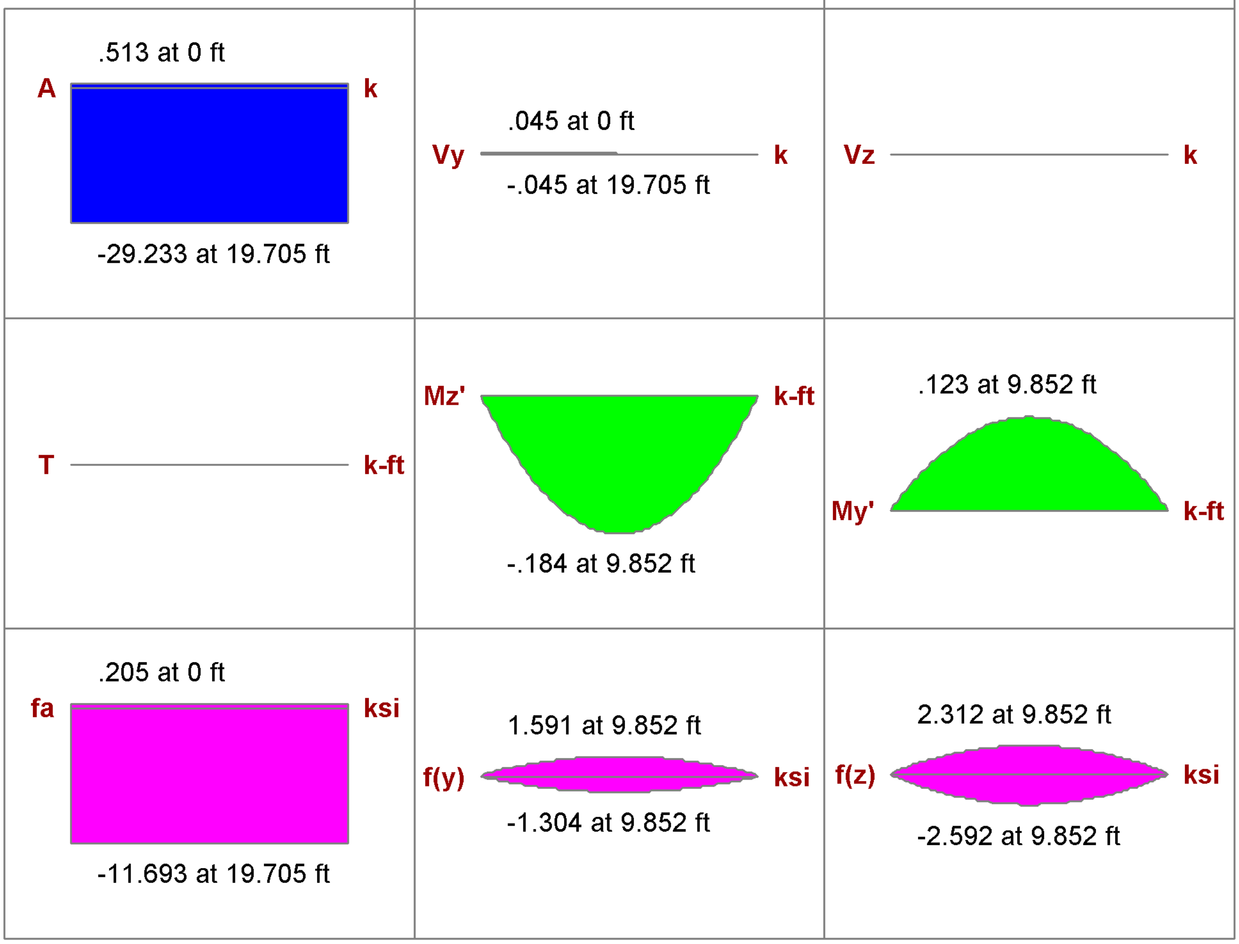
Shape:	**L3x2.5x8**
Material:	**A36 Gr.36**
Length:	**19.705 ft**
I Joint:	**N220A**
J Joint:	**F1_N201**

Envelope

Code Check: **0.361 (LC 9)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.361 (LC 9)**	Max Shear Check	**0.002 (y) (LC 9)**
Location	**19.705 ft**	Location	**19.705 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**19.705 ft**	**19.705 ft**
phi*Pnc	**2.69 k**	KL/r	**458.246**	**227.767**
phi*Pnt	**81 k**			
phi*Mny'	**2.276 k-ft**	L Comp Flange	**19.705 ft**	
phi*Mnz'	**3.947 k-ft**	L-torque	**19.705 ft**	
phi*Vny	**29.16 k**	Tau_b	**1**	
phi*Vnz	**24.3 k**			
Cb	**1.136**			

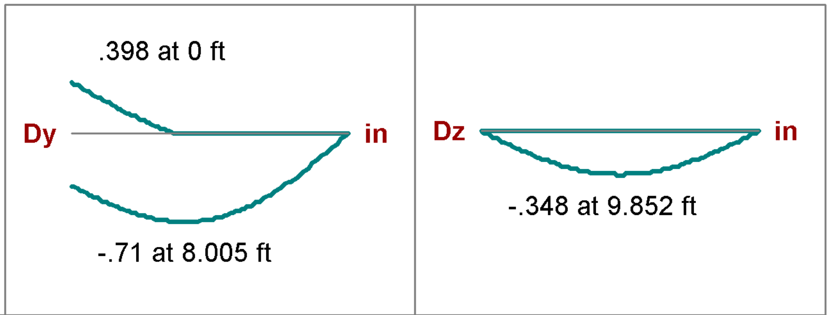
VBrace: ***X-BR 4A***

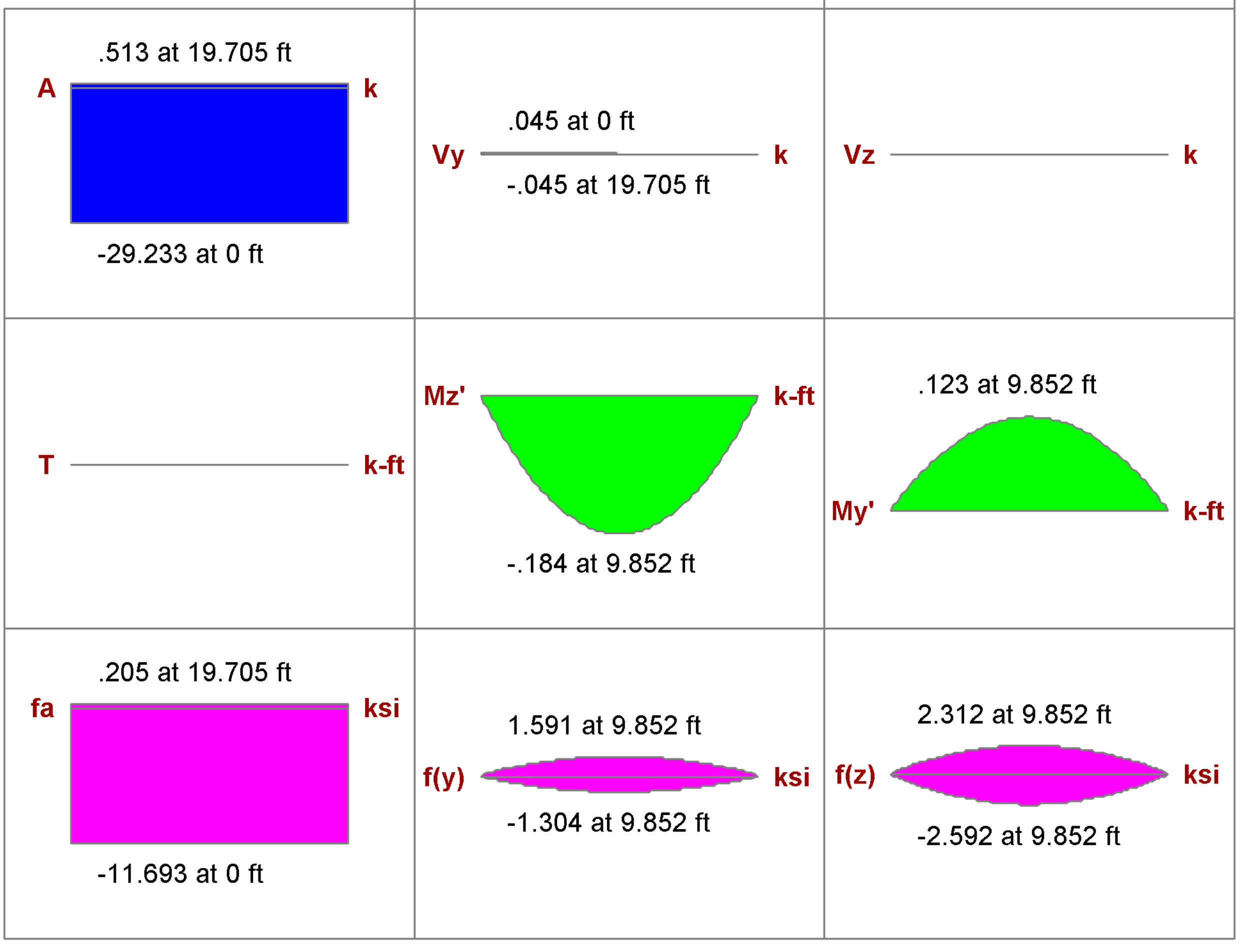
Shape:	**L3x2.5x8**
Material:	**A36 Gr.36**
Length:	**19.705 ft**
I Joint:	**F1_N147**
J Joint:	**N192**

Envelope

Code Check: **0.361 (LC 11)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.361 (LC 11)**	Max Shear Check	**0.002 (y) (LC 11)**
Location	**0 ft**	Location	**0 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**19.705 ft**	**19.705 ft**
phi*Pnc	**2.69 k**	KL/r	**458.246**	**227.767**
phi*Pnt	**81 k**			
phi*Mny'	**2.276 k-ft**	L Comp Flange	**19.705 ft**	
phi*Mnz'	**3.947 k-ft**	L-torque	**19.705 ft**	
phi*Vny	**29.16 k**	Tau_b	**1**	
phi*Vnz	**24.3 k**			
Cb	**1.136**			

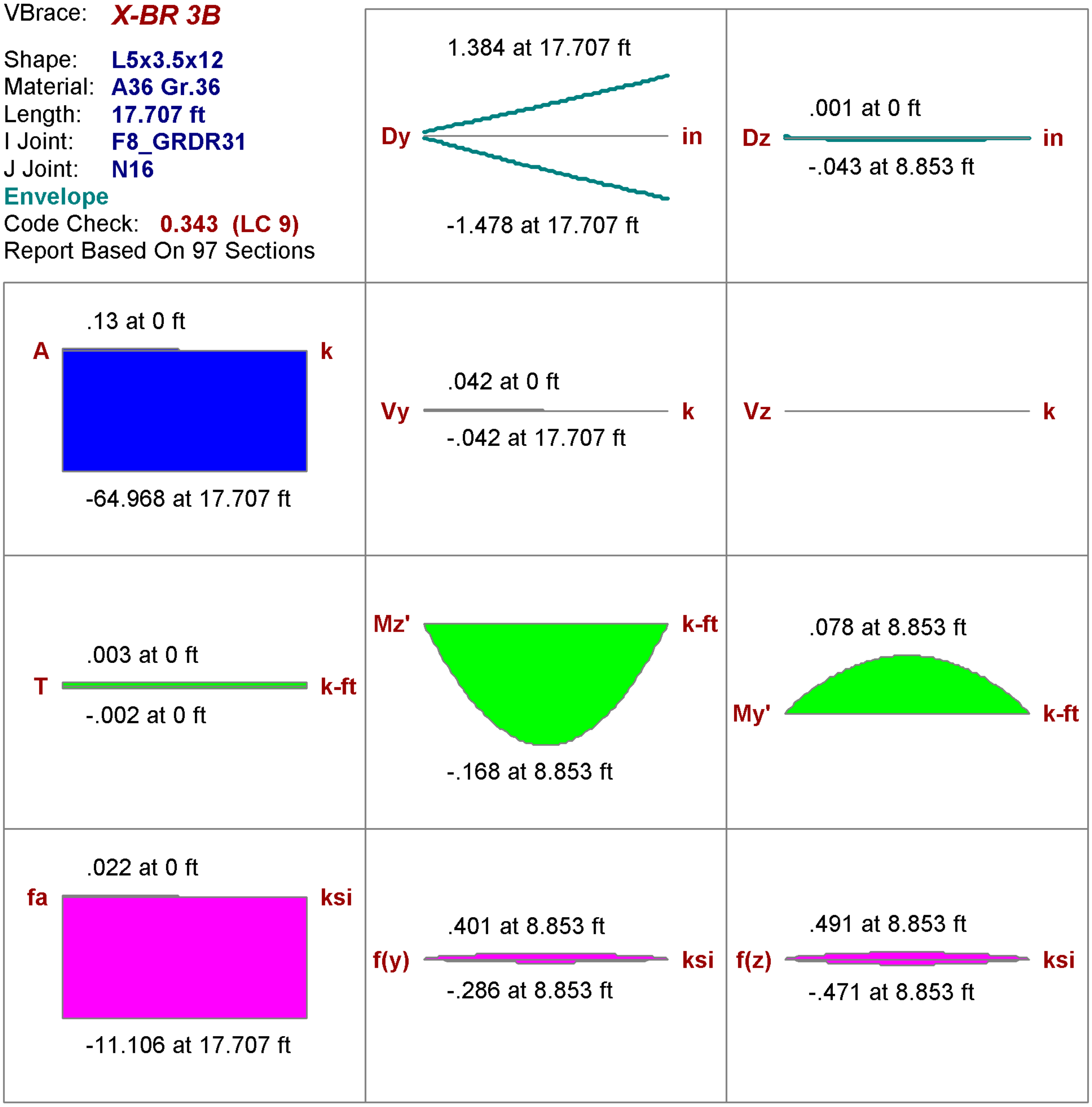
VBrace: ***X-BR 3B***

Shape:	**L5x3.5x12**
Material:	**A36 Gr.36**
Length:	**17.707 ft**
I Joint:	**F8_GRDR31**
J Joint:	**N16**

Envelope

Code Check: **0.343 (LC 9)**

Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

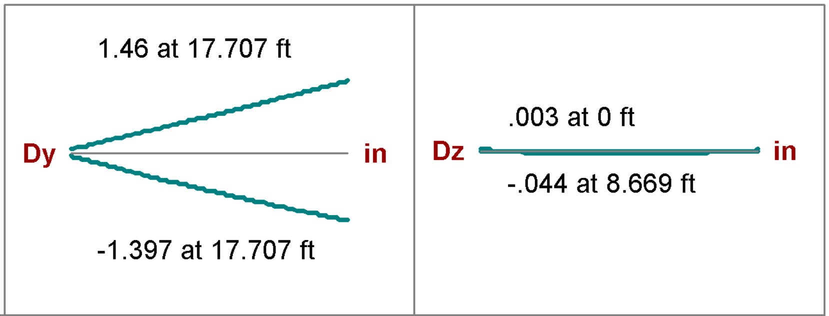
Max Bending Check	**0.343 (LC 9)**	Max Shear Check	**0.002 (y) (LC 10)**
Location	**17.707 ft**	Location	**0 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

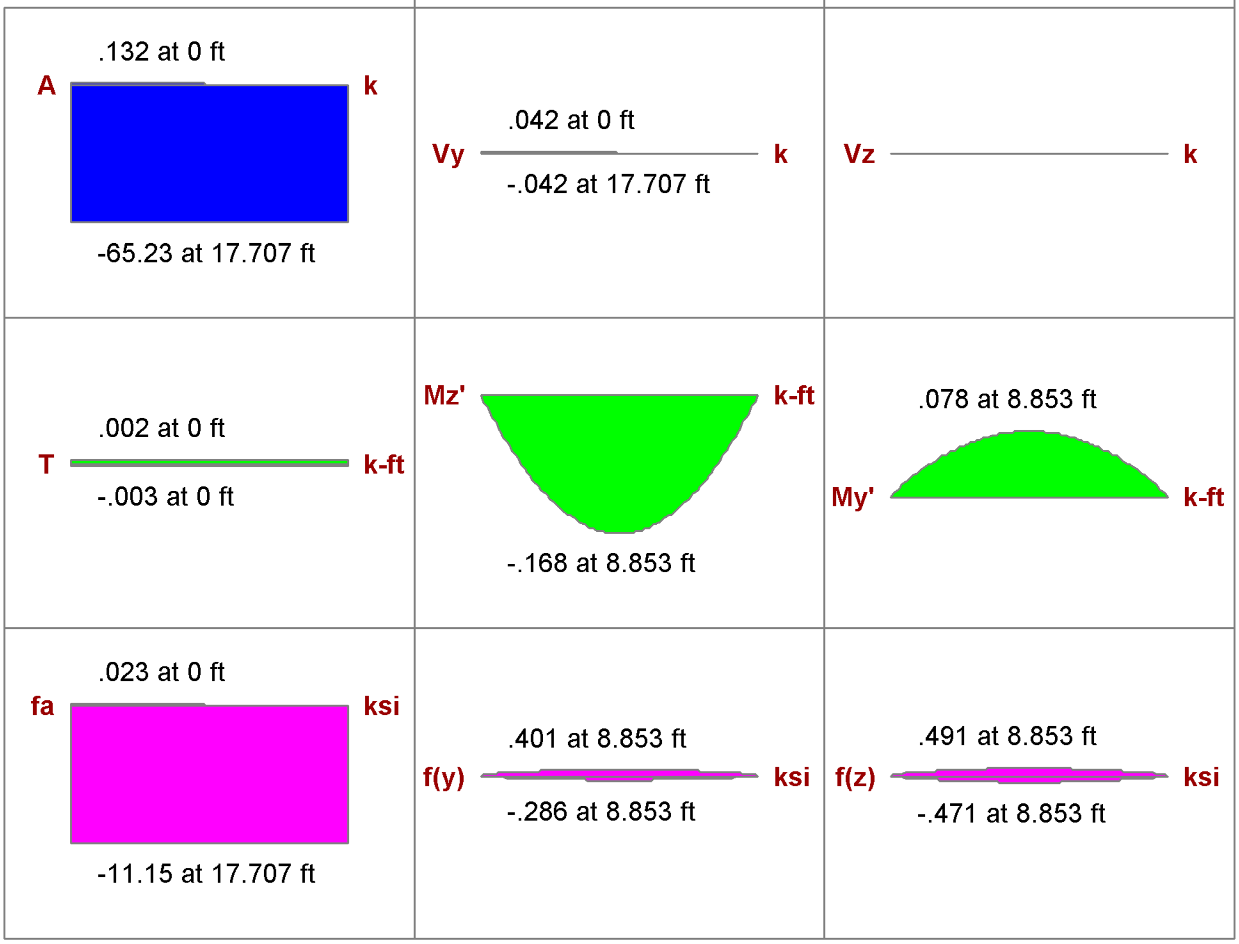
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**17.707 ft**	**17.707 ft**
phi*Pnc	**16.204 k**	KL/r	**285.589**	**127.755**
phi*Pnt	**189.54 k**			
phi*Mny'	**7.058 k-ft**	L Comp Flange	**17.707 ft**	
phi*Mnz'	**16.689 k-ft**	L-torque	**17.707 ft**	
phi*Vny	**72.9 k**	Tau_b	**1**	
phi*Vnz	**51.03 k**			
Cb	**1.136**			

VBrace: ***X-BR 3A***

Shape: **L5x3.5x12**
Material: **A36 Gr.36**
Length: **17.707 ft**
I Joint: **N134**
J Joint: **F1_N200**
Envelope
Code Check: **0.344 (LC 11)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.344 (LC 11)**	Max Shear Check	**0.002 (y) (LC 10)**
Location	**17.707 ft**	Location	**17.707 ft**
Equation	**H2-1***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**17.707 ft**	**17.707 ft**
phi*Pnc	**16.204 k**	KL/r	**285.589**	**127.755**
phi*Pnt	**189.54 k**			
phi*Mny'	**7.058 k-ft**	L Comp Flange	**17.707 ft**	
phi*Mnz'	**16.689 k-ft**	L-torque	**17.707 ft**	
phi*Vny	**72.9 k**	Tau_b	**1**	
phi*Vnz	**51.03 k**			
Cb	**1.136**			

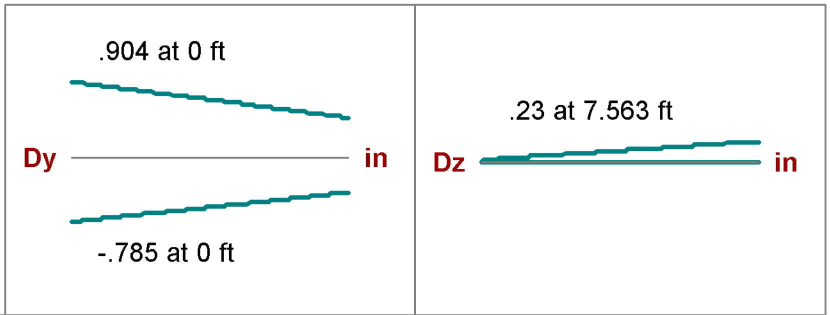
Column: ***V3 HANGER***

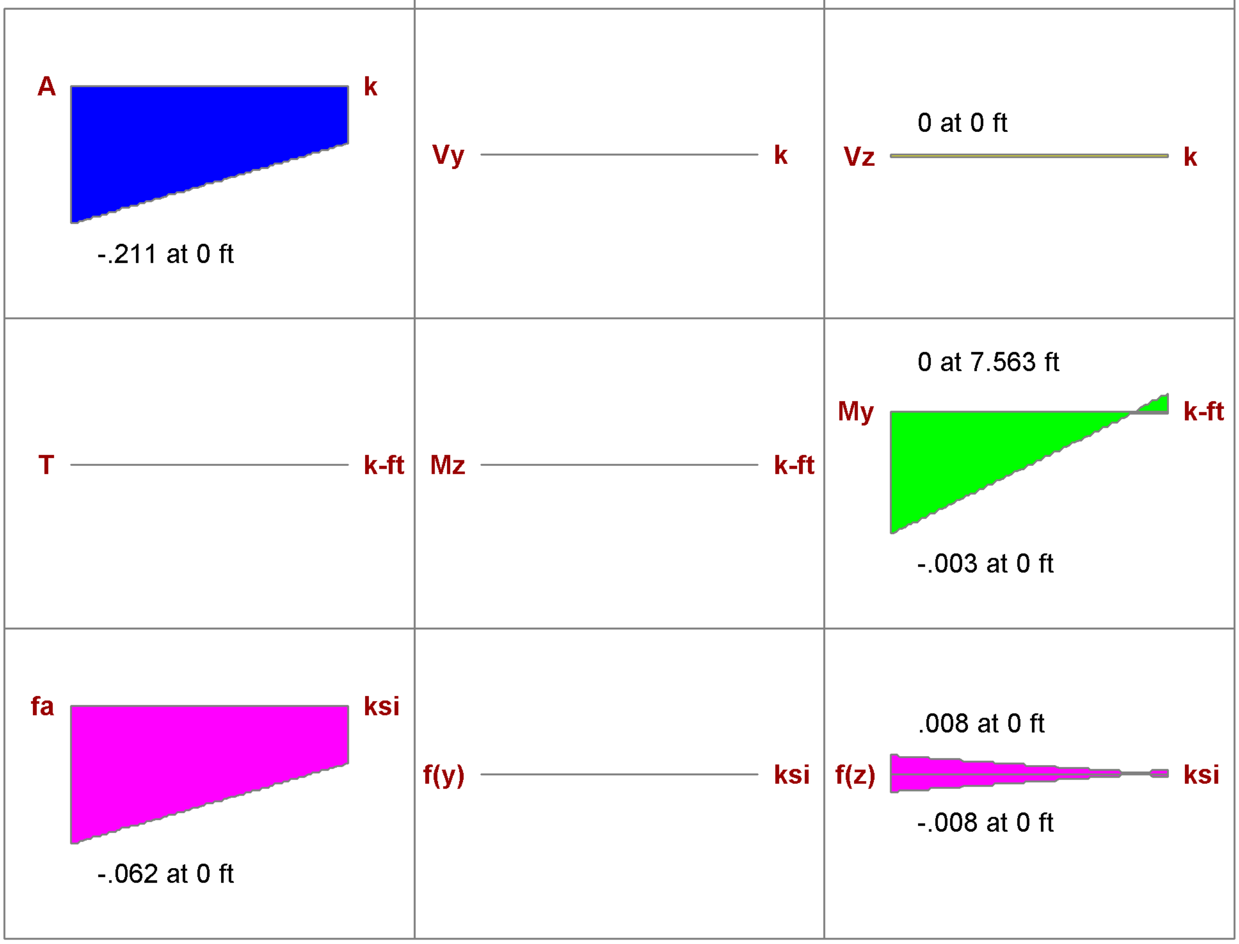
Shape:	**HSS4x4x4**
Material:	**A500 Gr.B Rect**
Length:	**7.563 ft**
I Joint:	**V3_DEF**
J Joint:	**N136**

Envelope

Code Check: **0.001 (LC 9)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

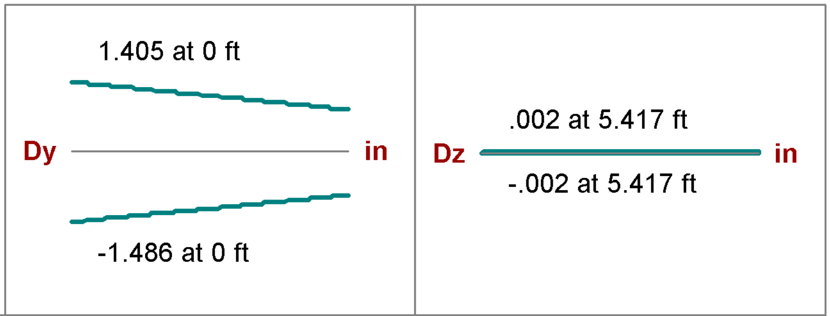
Max Bending Check	**0.001 (LC 9)**	Max Shear Check	**0.000 (z) (LC 9)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b**	Max Defl Ratio	**L/0**

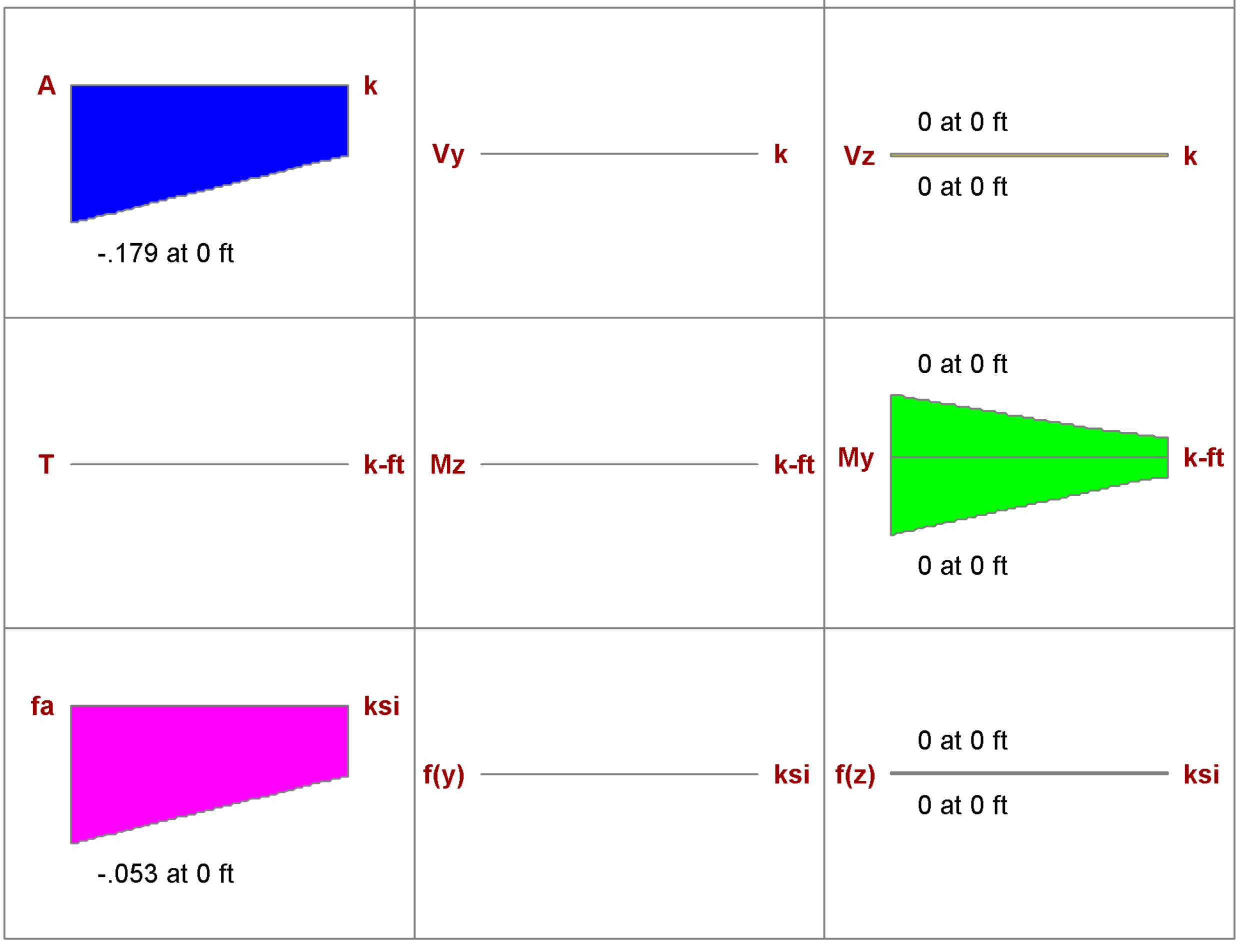
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**7.563 ft**	**7.563 ft**
phi*Pnc	**109.816 k**	KL/r	**59.654**	**59.654**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**7.563 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**7.563 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

Column: ***V2 HANGER***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **5.417 ft**
I Joint: **V2_DEF**
J Joint: **N137**
Envelope
Code Check: **0.001 (LC 10)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.001 (LC 10)**	Max Shear Check	**0.000 (z) (LC 10)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

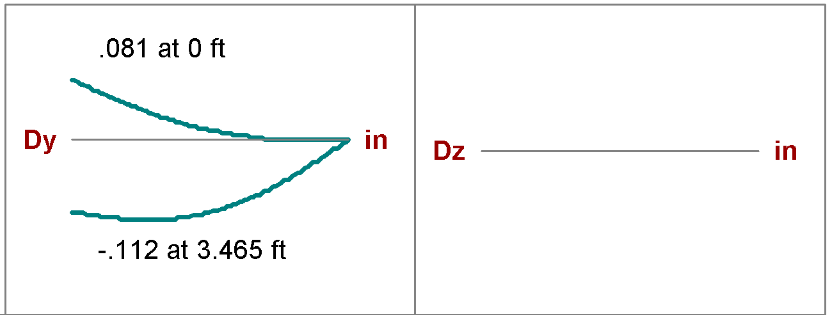
			y-y	z-z
Fy	**46 ksi**	Lb	**5.417 ft**	**5.417 ft**
phi*Pnc	**123.394 k**	KL/r	**42.729**	**42.729**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**5.417 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**5.417 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

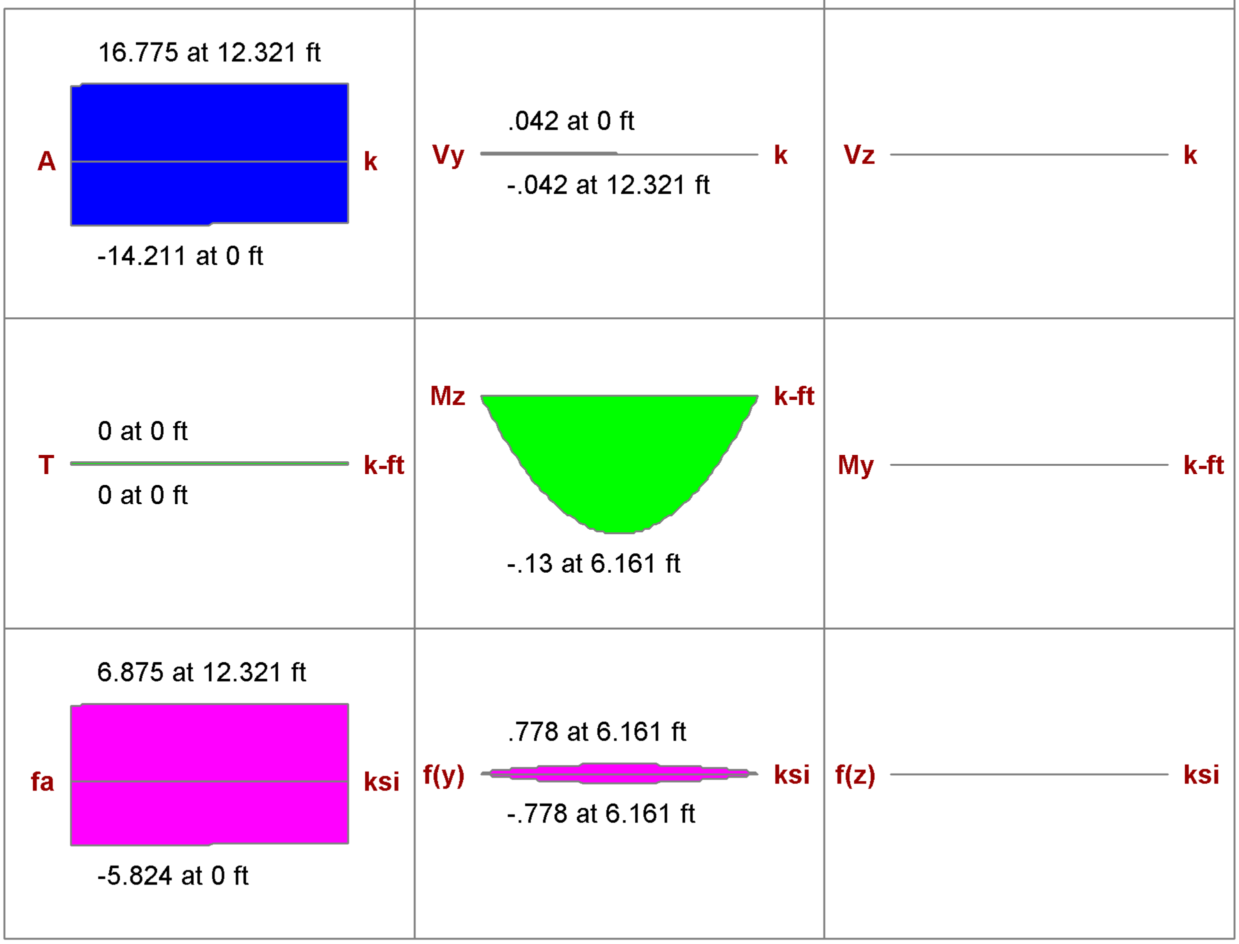
VBrace:	***BR-9***
Shape:	**HSS3x3x4**
Material:	**A500 Gr.B Rect**
Length:	**12.321 ft**
I Joint:	**F8_N33**
J Joint:	**N179**

Envelope

Code Check: **0.549 (LC 9)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.549 (LC 9)**	Max Shear Check	**0.002 (y) (LC 12)**
Location	**6.546 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**12.321 ft**	**12.321 ft**
phi*Pnc	**31.209 k**	KL/r	**132.9**	**132.9**
phi*Pnt	**101.016 k**			
phi*Mny	**8.556 k-ft**	L Comp Flange	**12.321 ft**	
phi*Mnz	**8.556 k-ft**	L-torque	**12.321 ft**	
phi*Vny	**26.635 k**	Tau_b	**1**	
phi*Vnz	**26.635 k**			
phi*Tn	**7.284 k-ft**			
Cb	**1.136**			

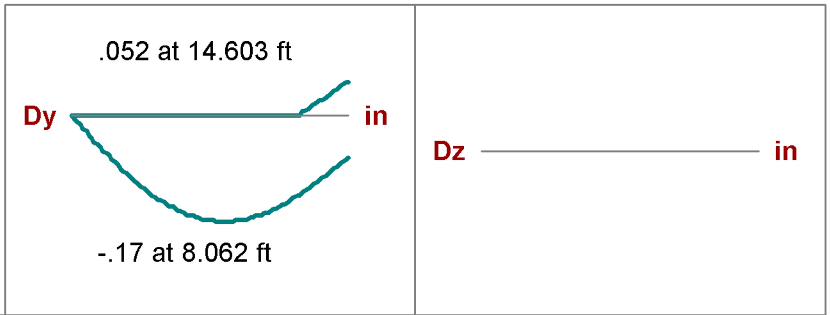
VBrace: ***BR-8***

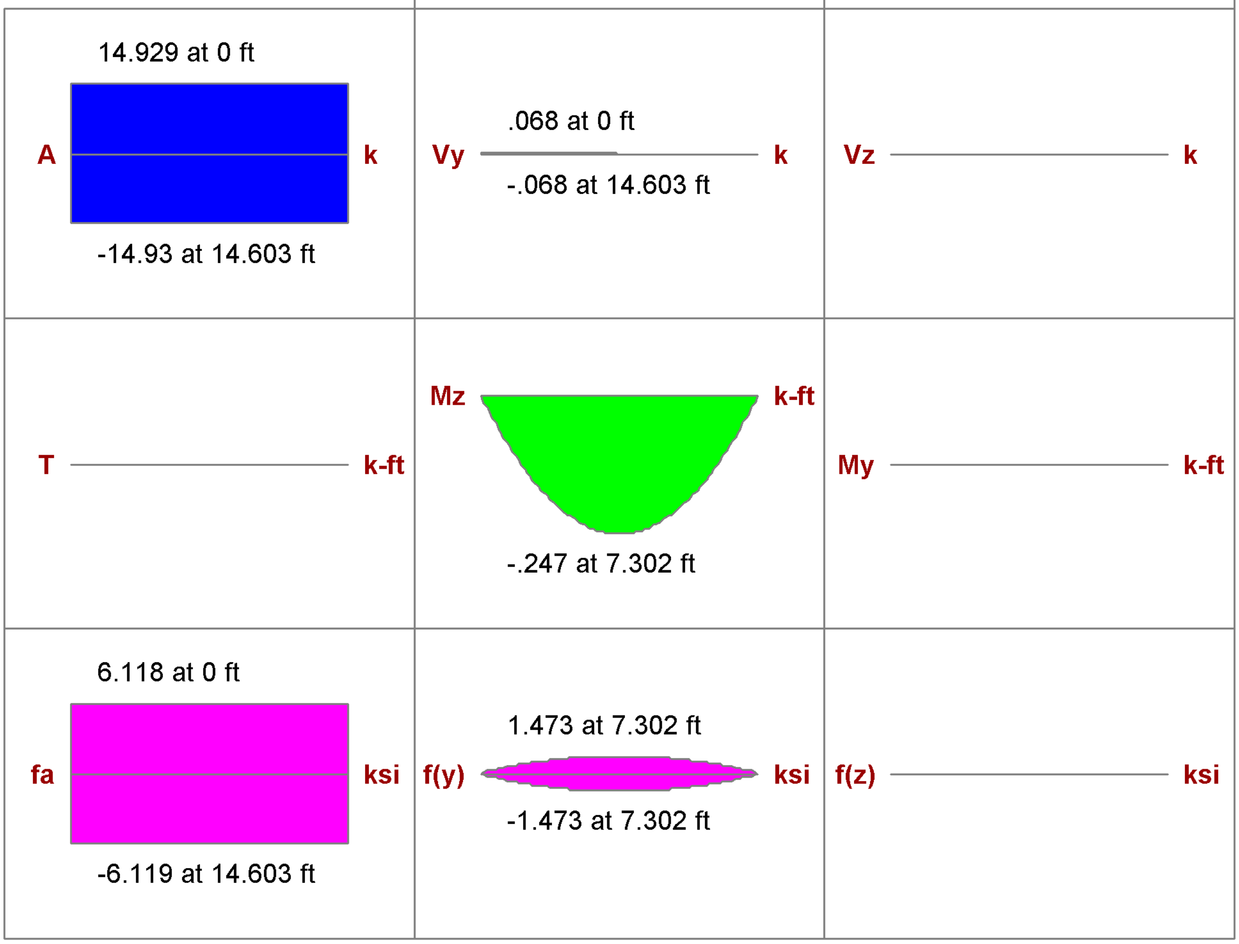
Shape:	**HSS3x3x4**
Material:	**A500 Gr.B Rect**
Length:	**14.603 ft**
I Joint:	**N213**
J Joint:	**N196**

Envelope

Code Check: **0.695 (LC 12)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

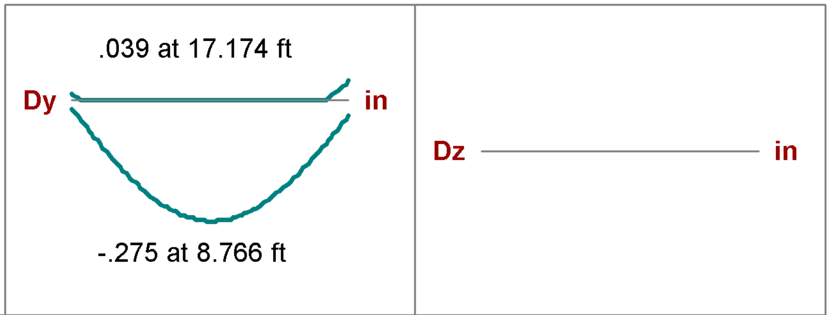
Max Bending Check	**0.695 (LC 12)**	Max Shear Check	**0.003 (y) (LC 12)**
Location	**6.997 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

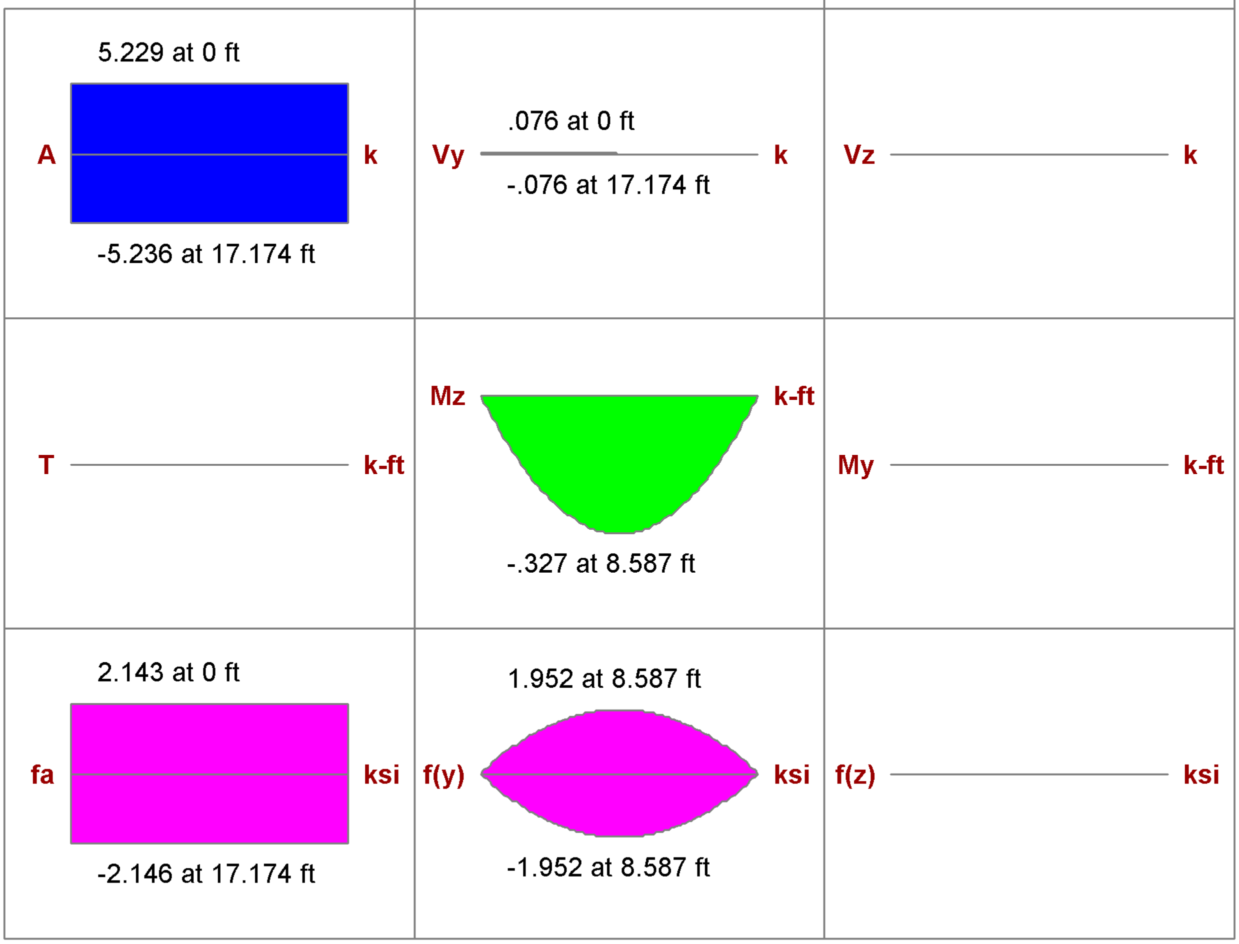
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**14.603 ft**	**14.603 ft**
phi*Pnc	**22.218 k**	KL/r	**157.513**	**157.513**
phi*Pnt	**101.016 k**			
phi*Mny	**8.556 k-ft**	L Comp Flange	**14.603 ft**	
phi*Mnz	**8.556 k-ft**	L-torque	**14.603 ft**	
phi*Vny	**26.635 k**	Tau_b	**1**	
phi*Vnz	**26.635 k**			
phi*Tn	**7.284 k-ft**			
Cb	**1.136**			

VBrace: ***BR-6***

Shape: **HSS3x3x4**
Material: **A500 Gr.B Rect**
Length: **17.174 ft**
I Joint: **F2_N31**
J Joint: **F1_N247**
Envelope
Code Check: **0.356 (LC 11)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method

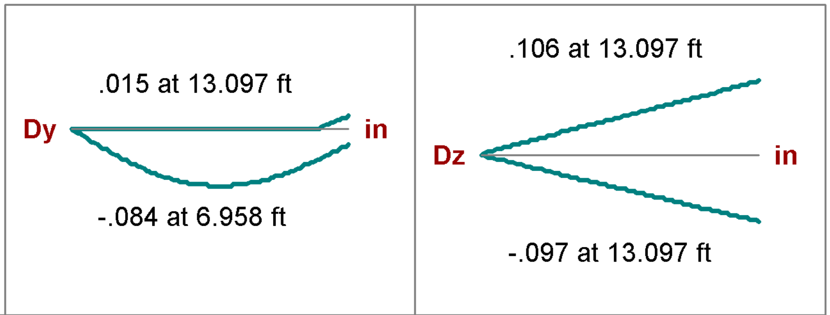
Max Bending Check	**0.356 (LC 11)**	Max Shear Check	**0.003 (y) (LC 11)**
Location	**8.051 ft**	Location	**17.174 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

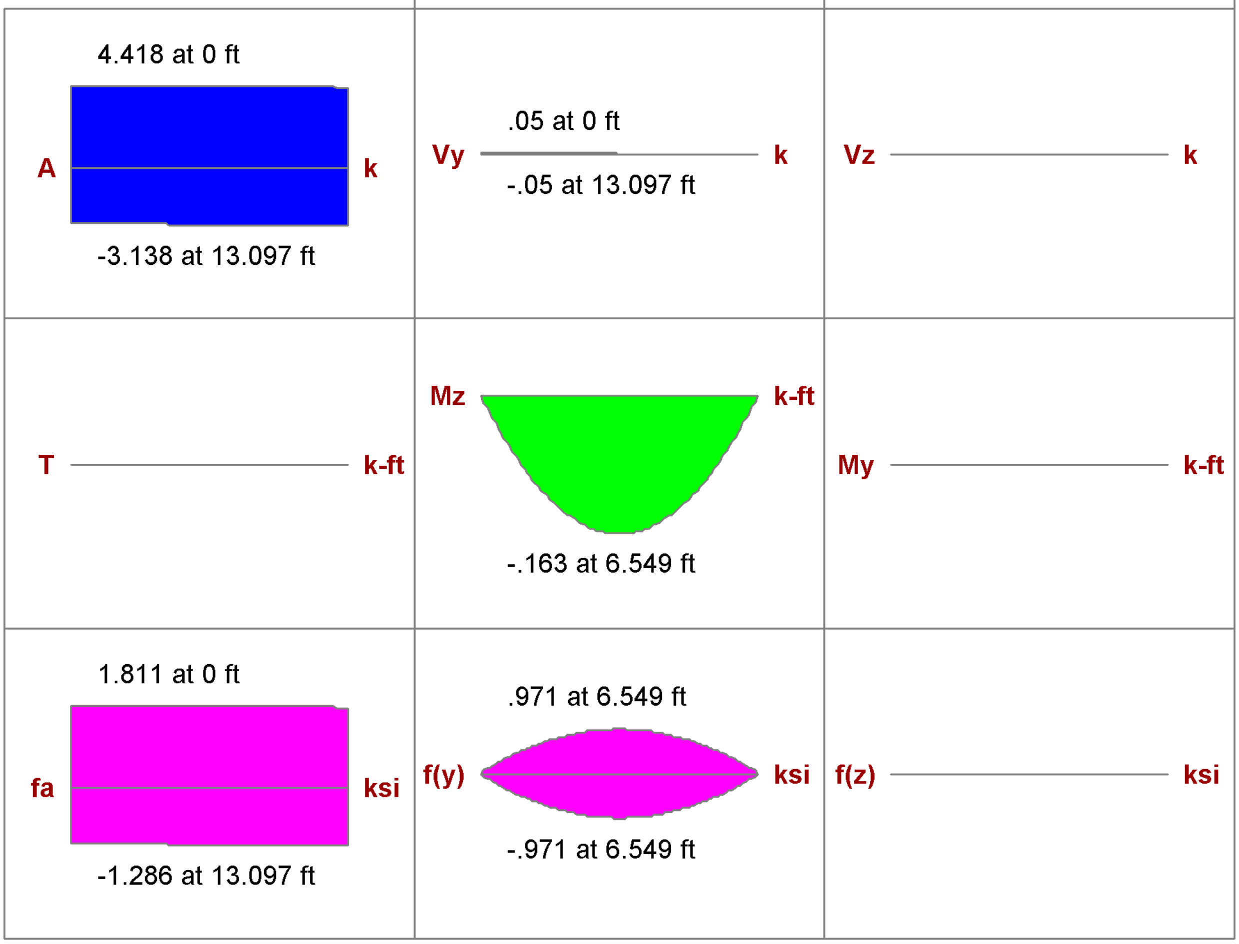
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**17.174 ft**	**17.174 ft**
phi*Pnc	**16.063 k**	KL/r	**185.248**	**185.248**
phi*Pnt	**101.016 k**			
phi*Mny	**8.556 k-ft**	L Comp Flange	**17.174 ft**	
phi*Mnz	**8.556 k-ft**	L-torque	**17.174 ft**	
phi*Vny	**26.635 k**	Tau_b	**1**	
phi*Vnz	**26.635 k**			
phi*Tn	**7.284 k-ft**			
Cb	**1.136**			

VBrace: ***BR-14***

Shape: **HSS3x3x4**
Material: **A500 Gr.B Rect**
Length: **13.097 ft**
I Joint: **N110**
J Joint: **F8_N439**
Envelope
Code Check: **0.160 (LC 12)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.160 (LC 12)**	Max Shear Check	**0.002 (y) (LC 12)**
Location	**0 ft**	Location	**13.097 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**13.097 ft**	**13.097 ft**
phi*Pnc	**27.62 k**	KL/r	**141.271**	**141.271**
phi*Pnt	**101.016 k**			
phi*Mny	**8.556 k-ft**	L Comp Flange	**13.097 ft**	
phi*Mnz	**8.556 k-ft**	L-torque	**13.097 ft**	
phi*Vny	**26.635 k**	Tau_b	**1**	
phi*Vnz	**26.635 k**			
phi*Tn	**7.284 k-ft**			
Cb	**1.136**			

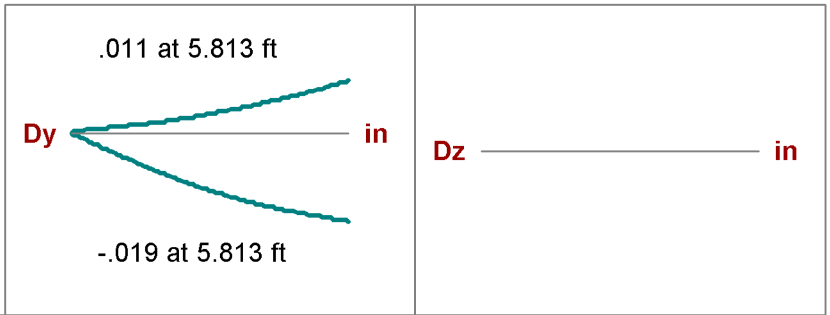
VBrace: ***BR-12***

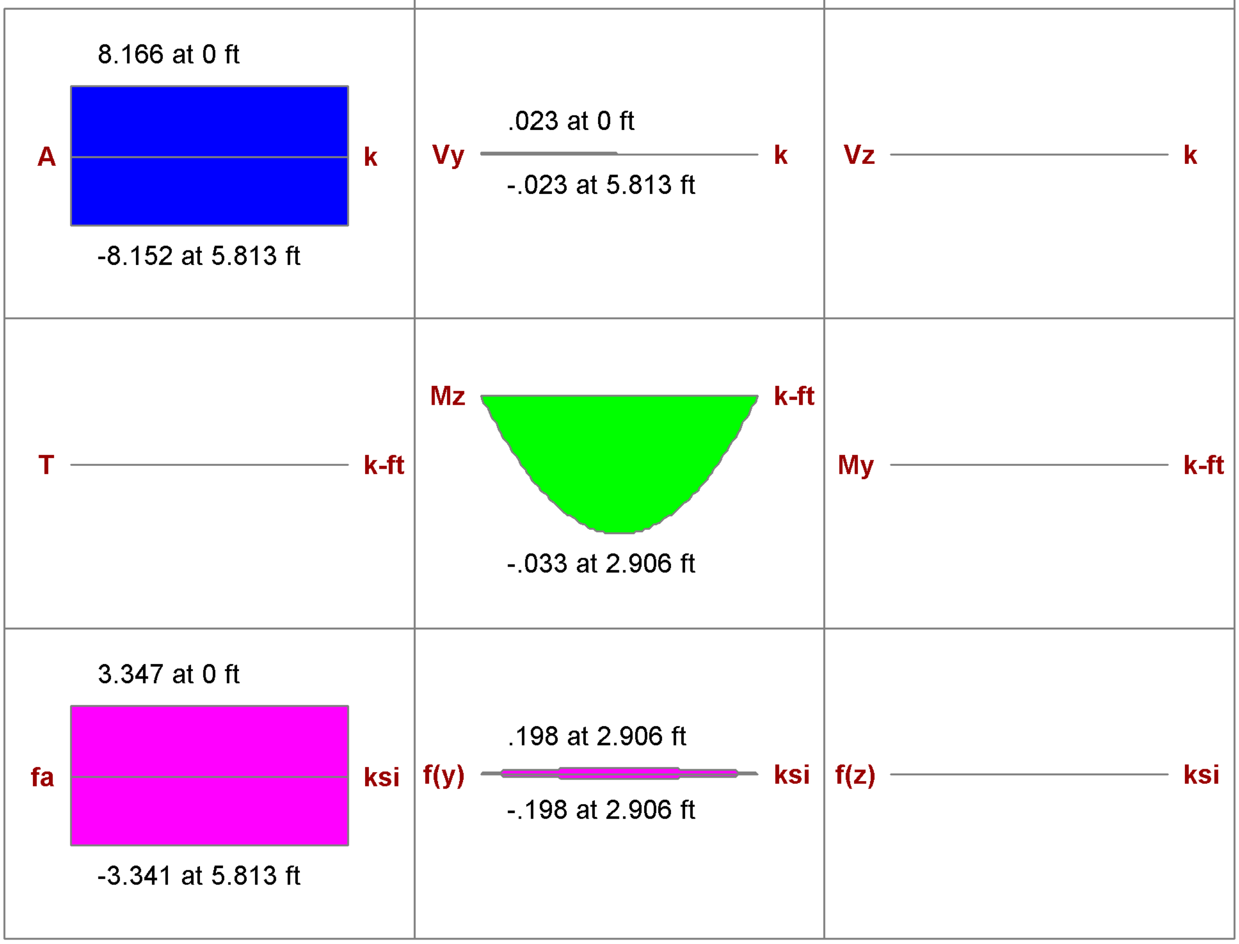
Shape:	**HSS3x3x4**
Material:	**A500 Gr.B Rect**
Length:	**5.813 ft**
I Joint:	**N214A**
J Joint:	**F2_N31**

Envelope

Code Check: **0.105 (LC 11)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.105 (LC 11)**	Max Shear Check	**0.001 (y) (LC 11)**
Location	**0 ft**	Location	**5.813 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**5.813 ft**	**5.813 ft**
phi*Pnc	**77.543 k**	KL/r	**62.7**	**62.7**
phi*Pnt	**101.016 k**			
phi*Mny	**8.556 k-ft**	L Comp Flange	**5.813 ft**	
phi*Mnz	**8.556 k-ft**	L-torque	**5.813 ft**	
phi*Vny	**26.635 k**	Tau_b	**1**	
phi*Vnz	**26.635 k**			
phi*Tn	**7.284 k-ft**			
Cb	**1.136**			

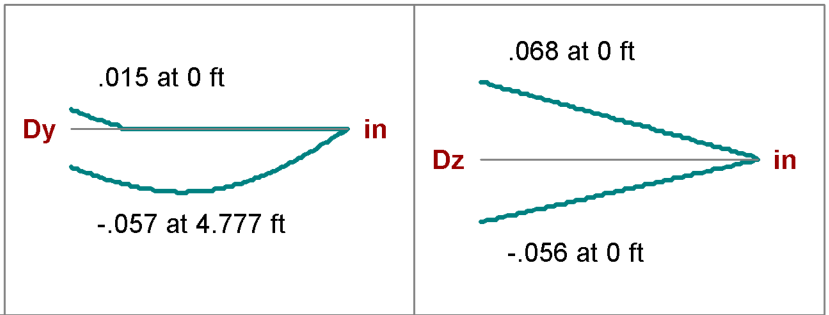
VBrace: ***BR-11***

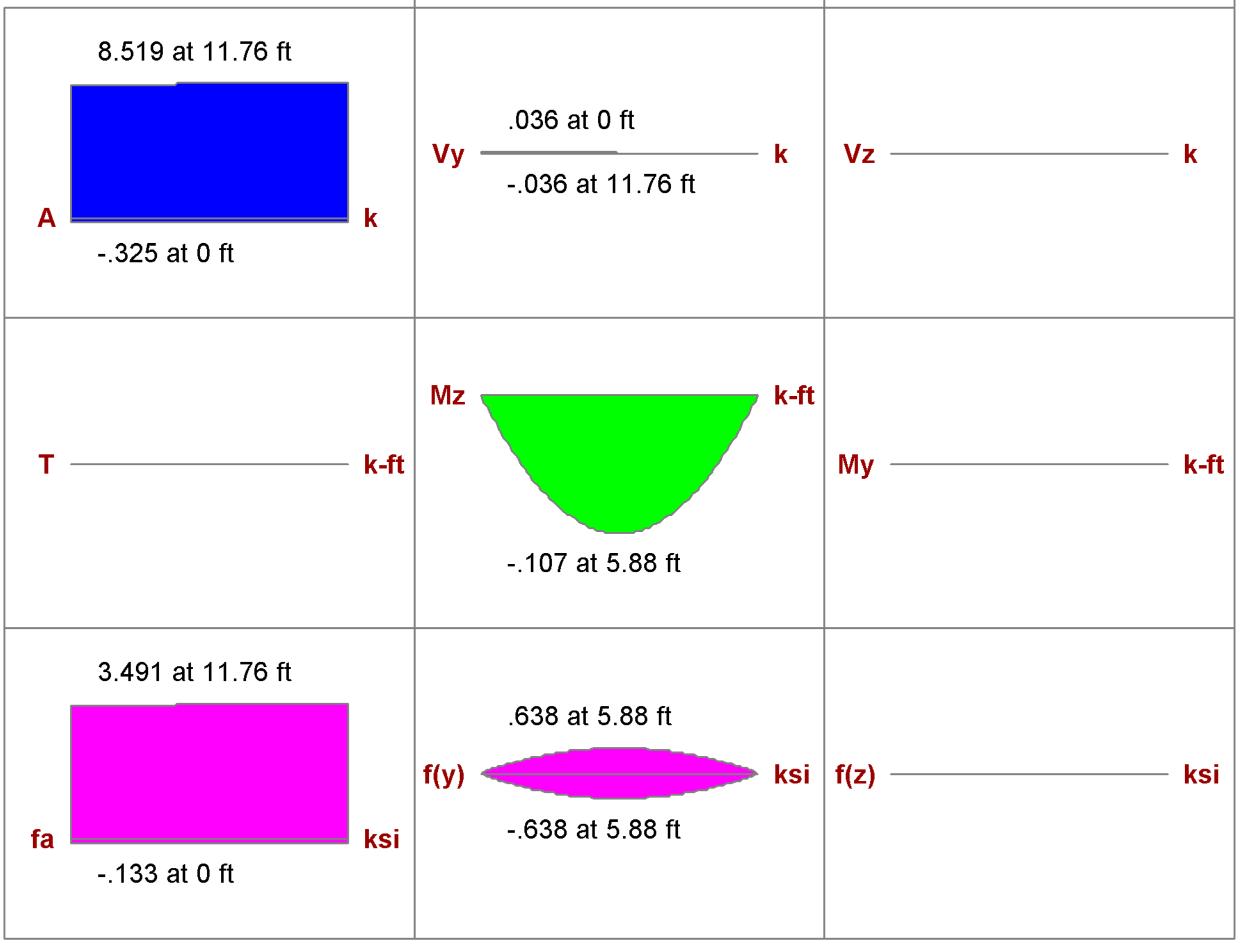
Shape:	**HSS3x3x4**
Material:	**A500 Gr.B Rect**
Length:	**11.76 ft**
I Joint:	**N111**
J Joint:	**N128**

Envelope

Code Check: **0.258 (LC 9)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

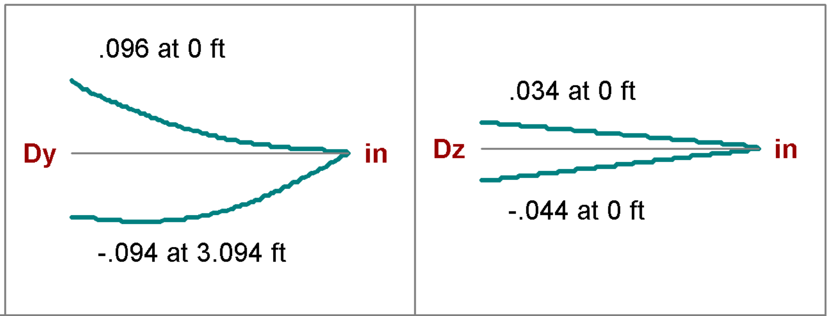
Max Bending Check	**0.258 (LC 9)**	Max Shear Check	**0.001 (y) (LC 12)**
Location	**6.37 ft**	Location	**11.76 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

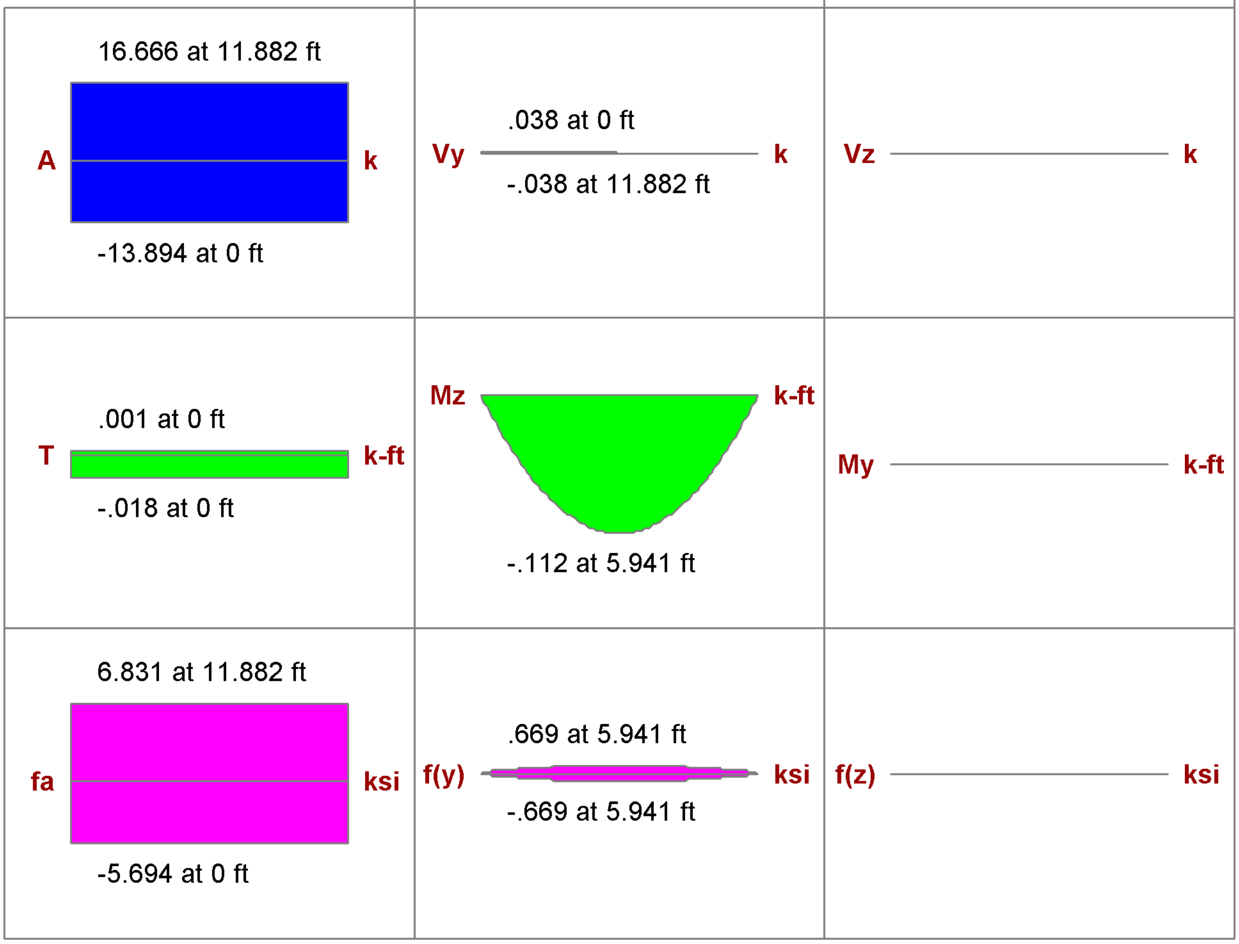
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**11.76 ft**	**11.76 ft**
phi*Pnc	**34.26 k**	KL/r	**126.844**	**126.844**
phi*Pnt	**101.016 k**			
phi*Mny	**8.556 k-ft**	L Comp Flange	**11.76 ft**	
phi*Mnz	**8.556 k-ft**	L-torque	**11.76 ft**	
phi*Vny	**26.635 k**	Tau_b	**1**	
phi*Vnz	**26.635 k**			
phi*Tn	**7.284 k-ft**			
Cb	**1.136**			

VBrace: ***BR-10***

Shape: **HSS3x3x4**
Material: **A500 Gr.B Rect**
Length: **11.882 ft**
I Joint: **F8_N35**
J Joint: **N125**
Envelope
Code Check: **0.507 (LC 11)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method

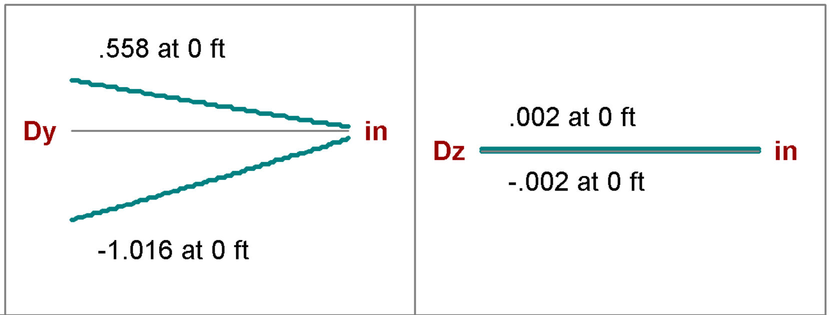
Max Bending Check	**0.507 (LC 11)**	Max Shear Check	**0.004 (y) (LC 11)**
Location	**6.436 ft**	Location	**11.882 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

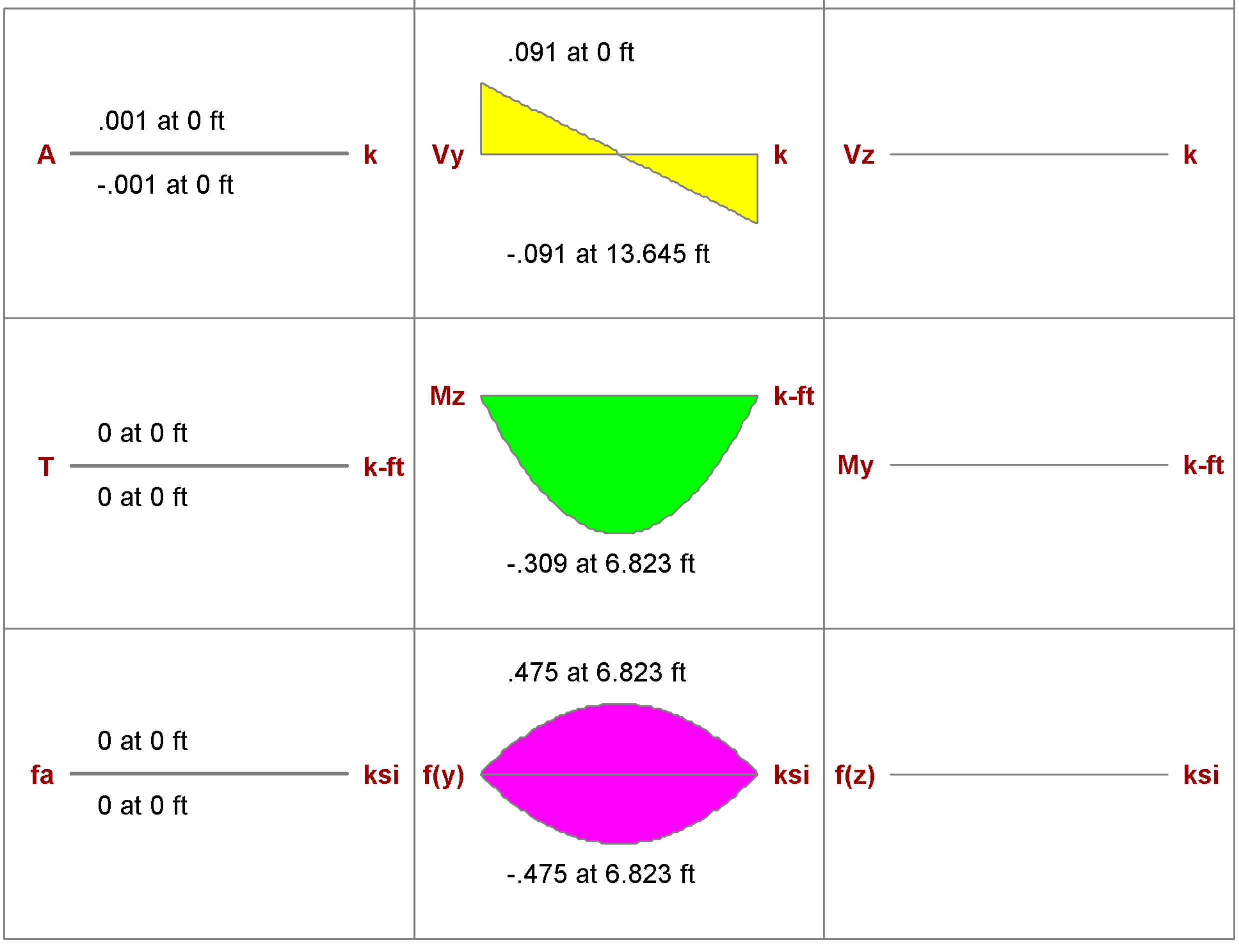
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**11.882 ft**	**11.882 ft**
phi*Pnc	**33.56 k**	KL/r	**128.161**	**128.161**
phi*Pnt	**101.016 k**			
phi*Mny	**8.556 k-ft**	L Comp Flange	**11.882 ft**	
phi*Mnz	**8.556 k-ft**	L-torque	**11.882 ft**	
phi*Vny	**26.635 k**	Tau_b	**1**	
phi*Vnz	**26.635 k**			
phi*Tn	**7.284 k-ft**			
Cb	**1.136**			

Beam: ***3B2***

Shape: **W8x10**
Material: **A992**
Length: **13.645 ft**
I Joint: **N137**
J Joint: **N1**
Envelope
Code Check: **0.027 (LC 9)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

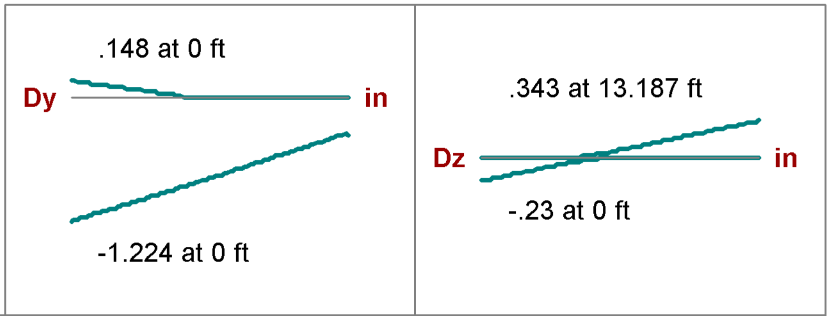
Max Bending Check	**0.027 (LC 9)**	Max Shear Check	**0.002 (y) (LC 10)**	Max Defl Ratio	**L/10000**
Location	**6.823 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**1**

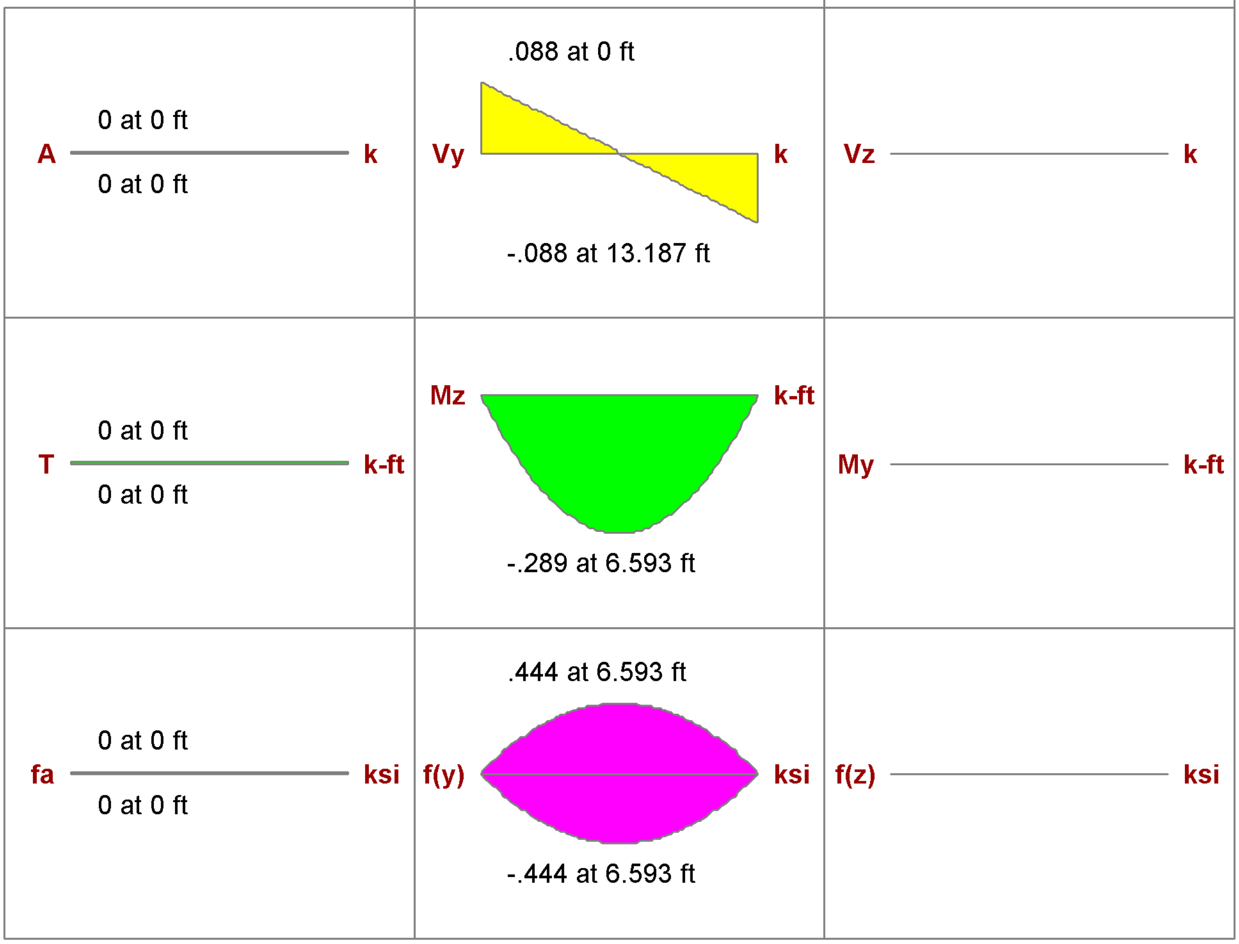
Bending Flange	**Non-Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**50 ksi**	Lb	**13.645 ft**	**13.645 ft**
phi*Pnc	**17.611 k**	KL/r	**194.862**	**50.76**
phi*Pnt	**133.2 k**			
phi*Mny	**6.119 k-ft**	L Comp Flange	**13.645 ft**	
phi*Mnz	**11.427 k-ft**	L-torque	**13.645 ft**	
phi*Vny	**40.239 k**	Tau_b	**1**	
phi*Vnz	**43.616 k**			
Cb	**1.136**			

Beam: ***2B8***

Shape: **W8x10**
Material: **A992**
Length: **13.187 ft**
I Joint: **N136**
J Joint: **N96**
Envelope
Code Check: **0.024 (LC 9)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

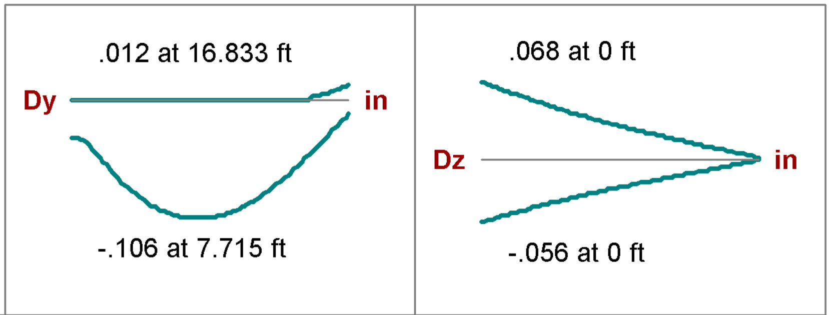
Max Bending Check	**0.024 (LC 9)**	Max Shear Check	**0.003 (y) (LC 9)**	Max Defl Ratio	**L/10000**
Location	**6.593 ft**	Location	**13.187 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**1**

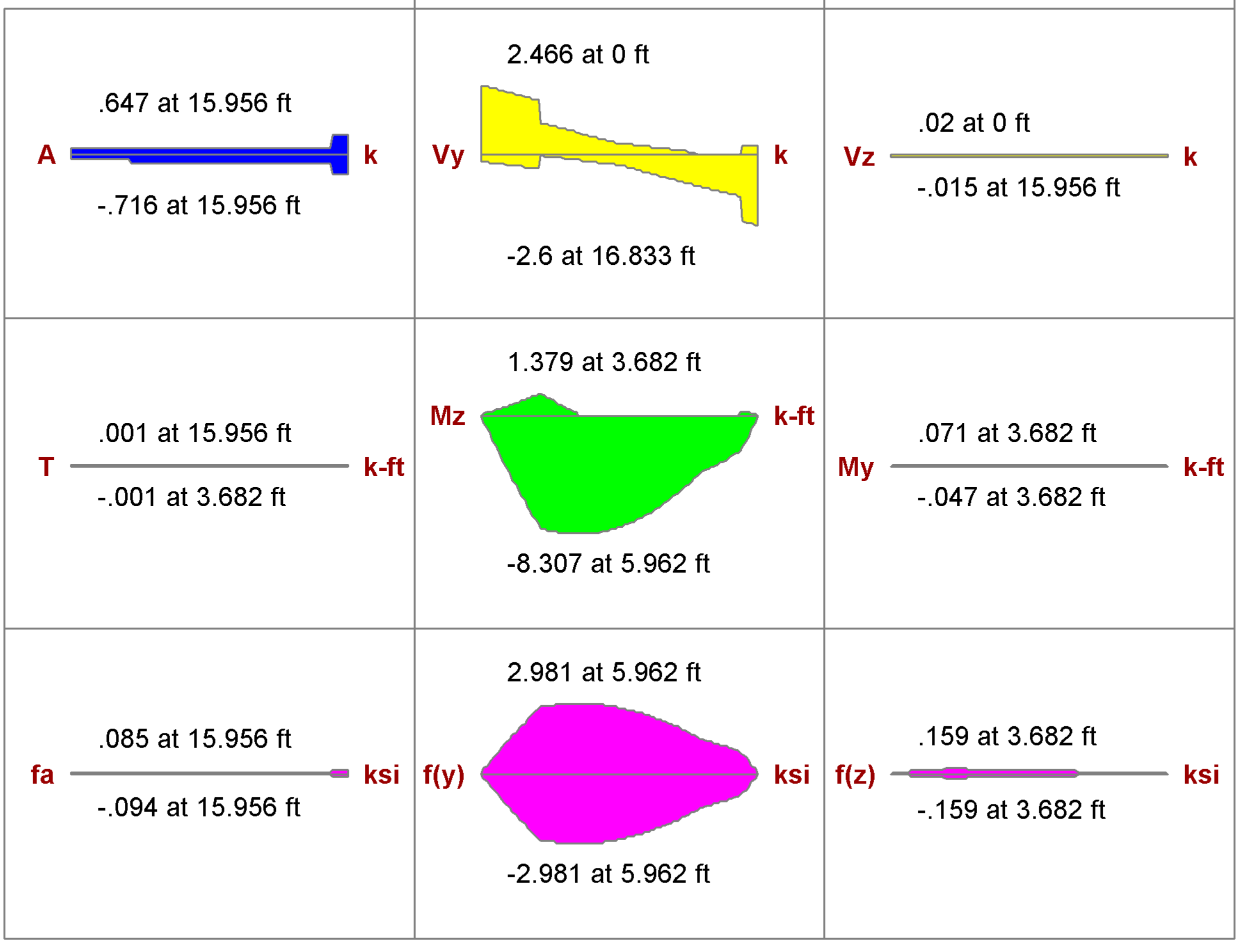
Bending Flange	**Non-Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**50 ksi**	Lb	**13.187 ft**	**13.187 ft**
phi*Pnc	**18.855 k**	KL/r	**188.322**	**49.057**
phi*Pnt	**133.2 k**			
phi*Mny	**6.119 k-ft**	L Comp Flange	**13.187 ft**	
phi*Mnz	**11.994 k-ft**	L-torque	**13.187 ft**	
phi*Vny	**40.239 k**	Tau_b	**1**	
phi*Vnz	**43.616 k**			
Cb	**1.136**			

Beam: *1B9*

Shape: **W12x26**
Material: **A992**
Length: **16.833 ft**
I Joint: **F8_N417**
J Joint: **F8_N418**
Envelope
Code Check: **0.062 (LC 11)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

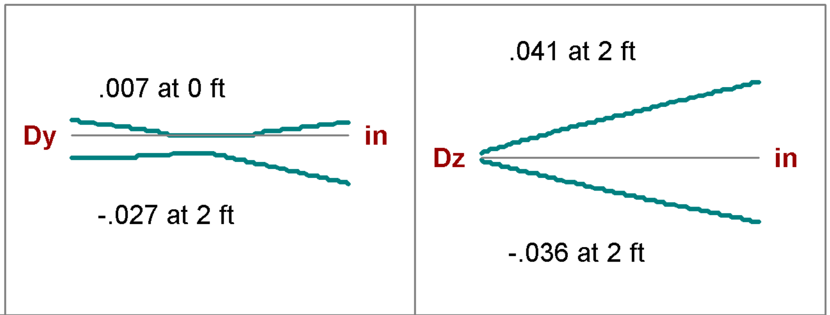
Max Bending Check	**0.062 (LC 11)**	Max Shear Check	**0.032 (y) (LC 9)**	Max Defl Ratio	**L/2255**
Location	**5.786 ft**	Location	**16.833 ft**	Location	**7.89 ft**
Equation	**H1-1b**			Span	**1**

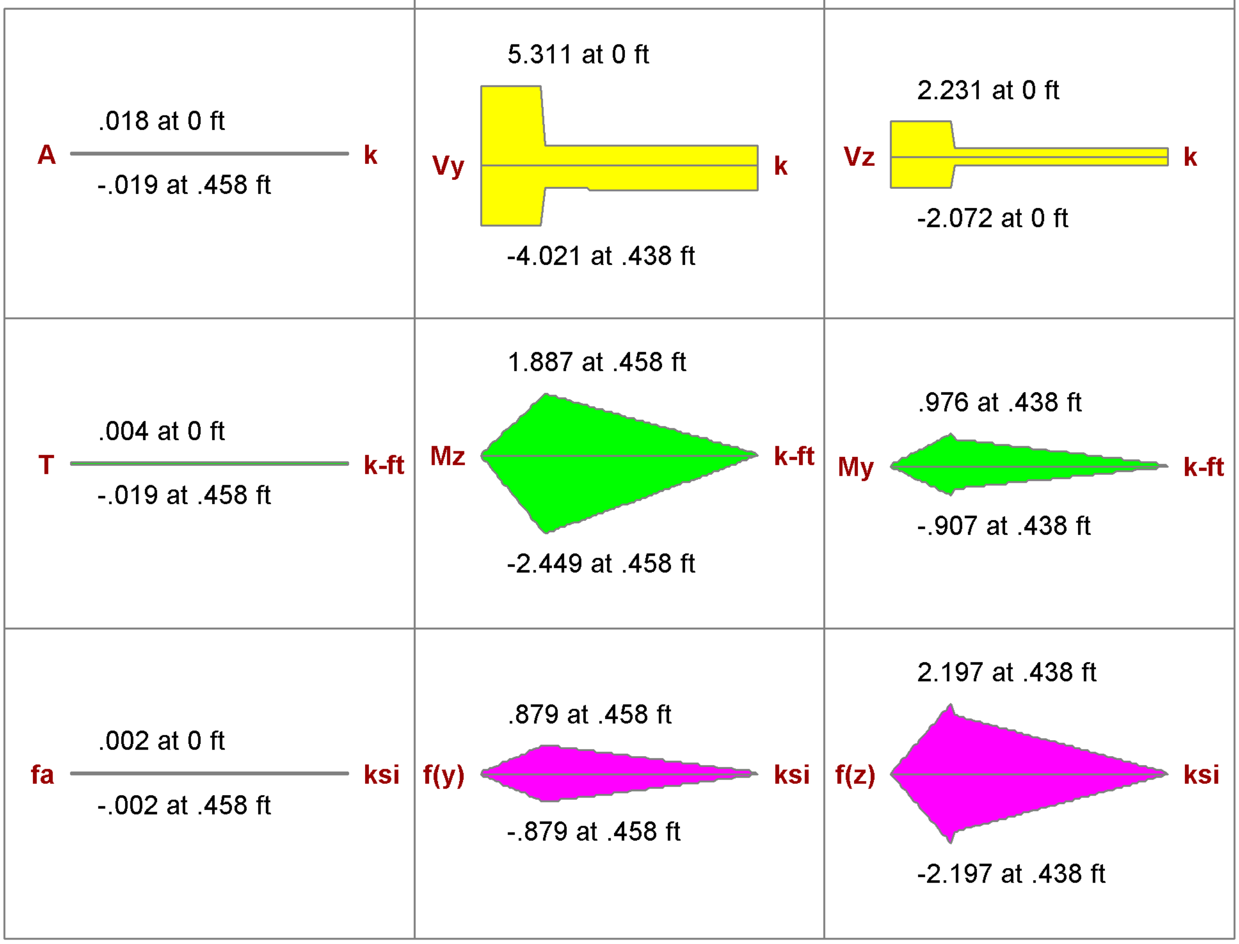
Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**50 ksi**	Lb	**12.292 ft**	**16.833 ft**
phi*Pnc	**170.358 k**	KL/r	**98.087**	**39.116**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**16.833 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

Beam: ***1B29***

Shape: **W12x26**
Material: **A992**
Length: **2 ft**
I Joint: **F8_N303**
J Joint: **F8_N307**
Envelope
Code Check: **0.064 (LC 9)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

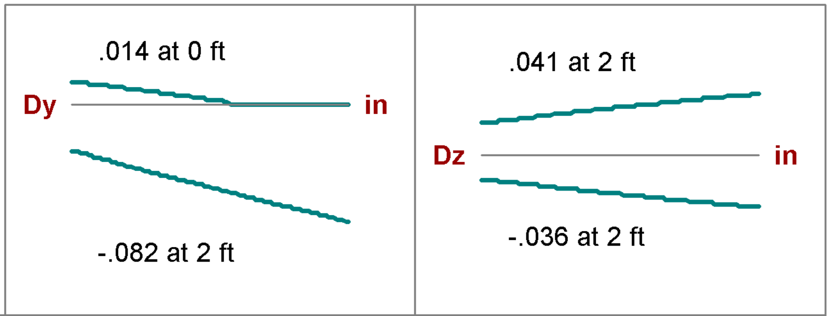
Max Bending Check	**0.049 (LC 9)**	Max Shear Check	**0.064 (y) (LC 9)**	Max Defl Ratio	**L/10000**
Location	**.438 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**1**

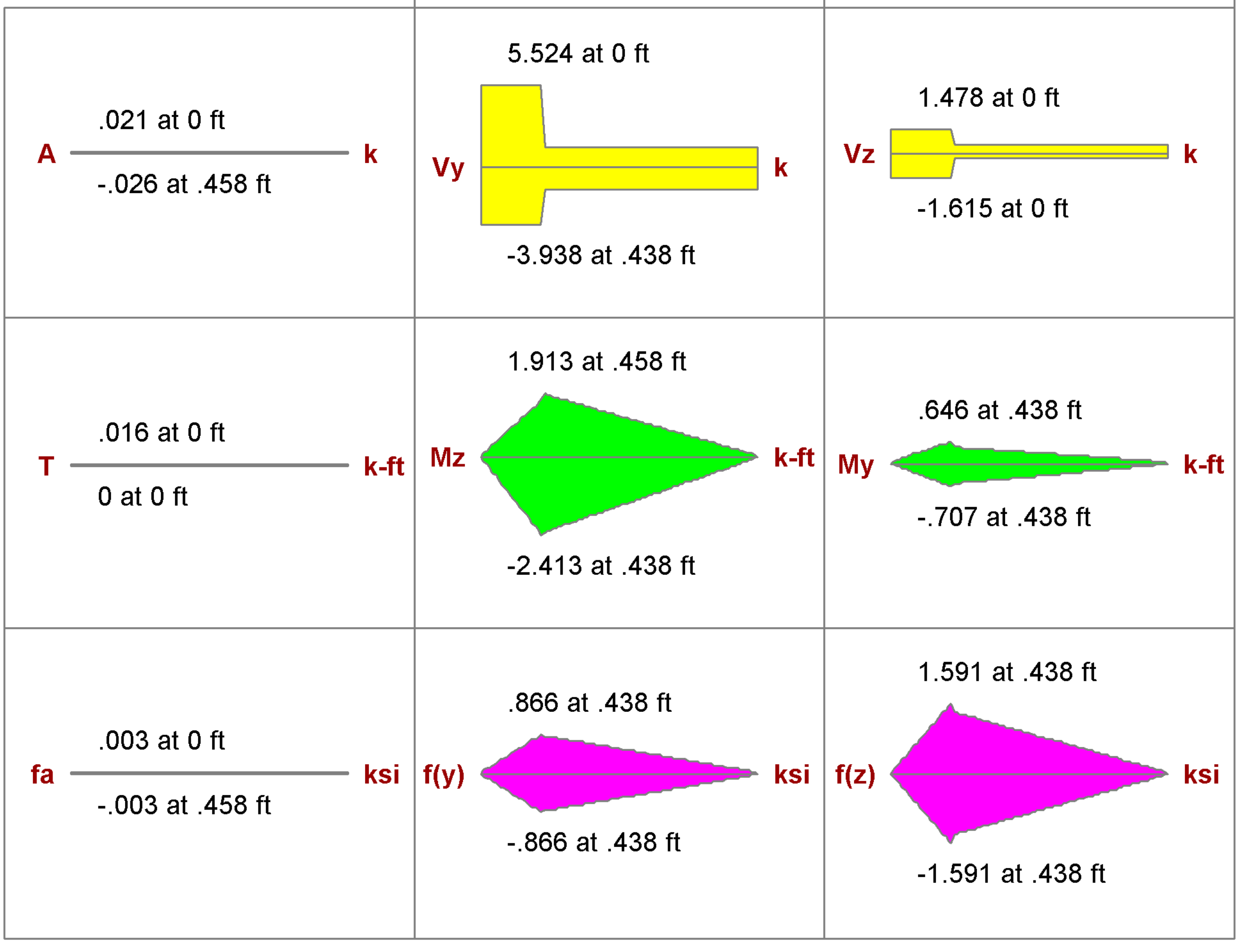
Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.925**

			y-y	z-z
Fy	**50 ksi**	Lb	**2 ft**	**2 ft**
phi*Pnc	**313.048 k**	KL/r	**15.959**	**4.648**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**2 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1.579**			

Beam: ***1B28***

Shape: **W12x26**
Material: **A992**
Length: **2 ft**
I Joint: **F8_N298229A**
J Joint: **F8_N300**
Envelope
Code Check: **0.074 (LC 11)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

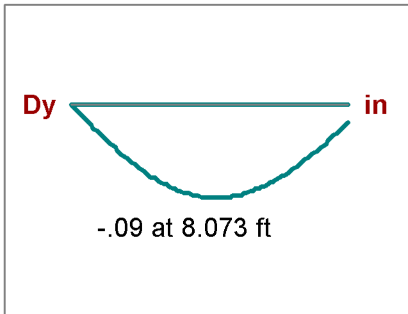
Max Bending Check	**0.040 (LC 11)**	Max Shear Check	**0.074 (y) (LC 11)**	Max Defl Ratio	**L/10000**
Location	**.438 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**1**

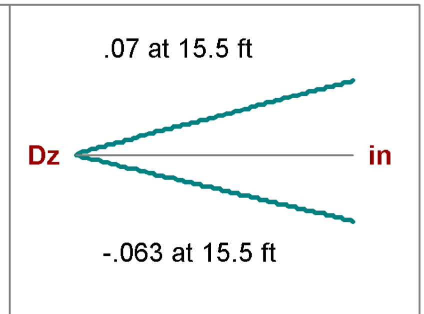
Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.925**

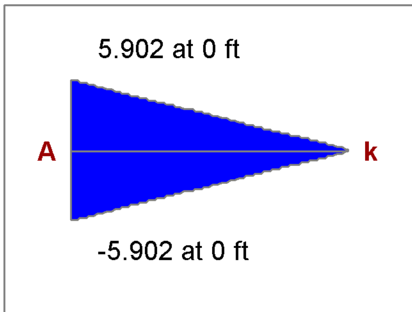
			y-y	z-z
Fy	**50 ksi**	Lb	**2 ft**	**2 ft**
phi*Pnc	**313.048 k**	KL/r	**15.959**	**4.648**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**2 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1.521**			

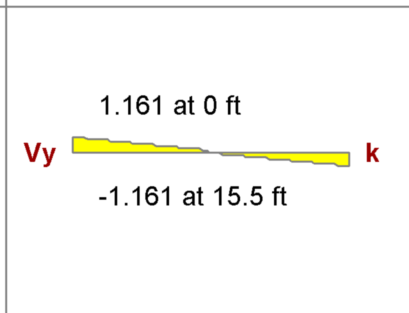
Beam: ***1B27***

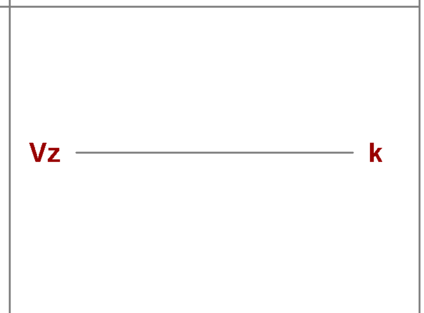
Shape: **3-1.75X14FS**
Material: **2.0E Microllam LVL**
Length: **15.5 ft**
I Joint: **F8_N53**
J Joint: **F8_N25**
Envelope (Ω)
Code Check: **No Calc**
Report Based On 97 Sections

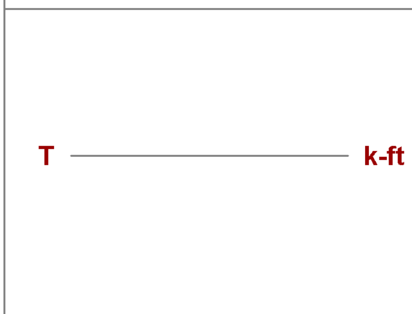


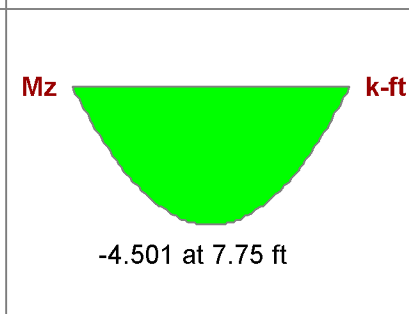


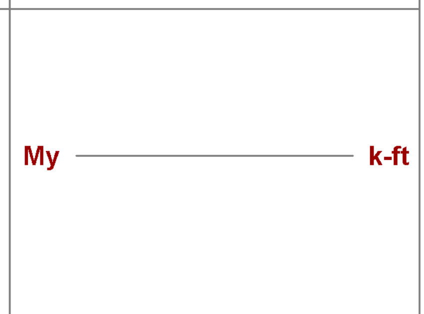


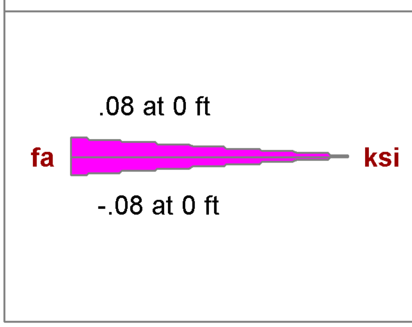


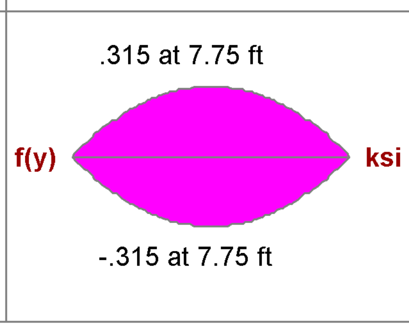


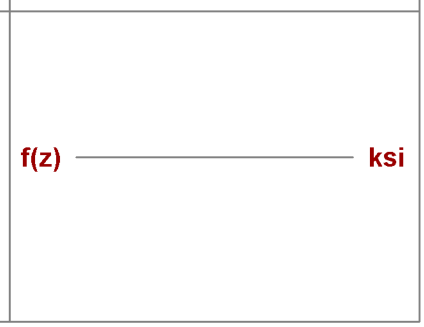












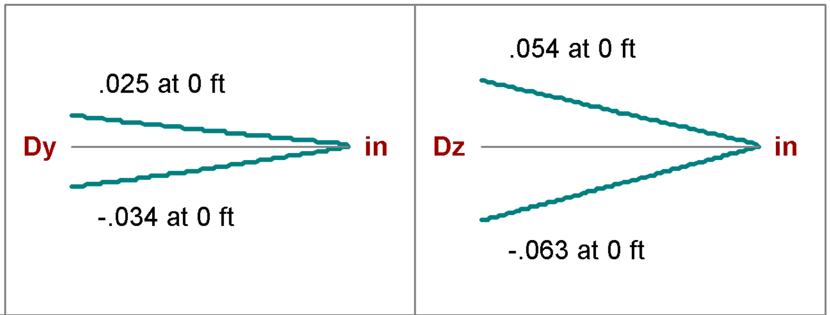
AWC NDS-15: ASD Code Check

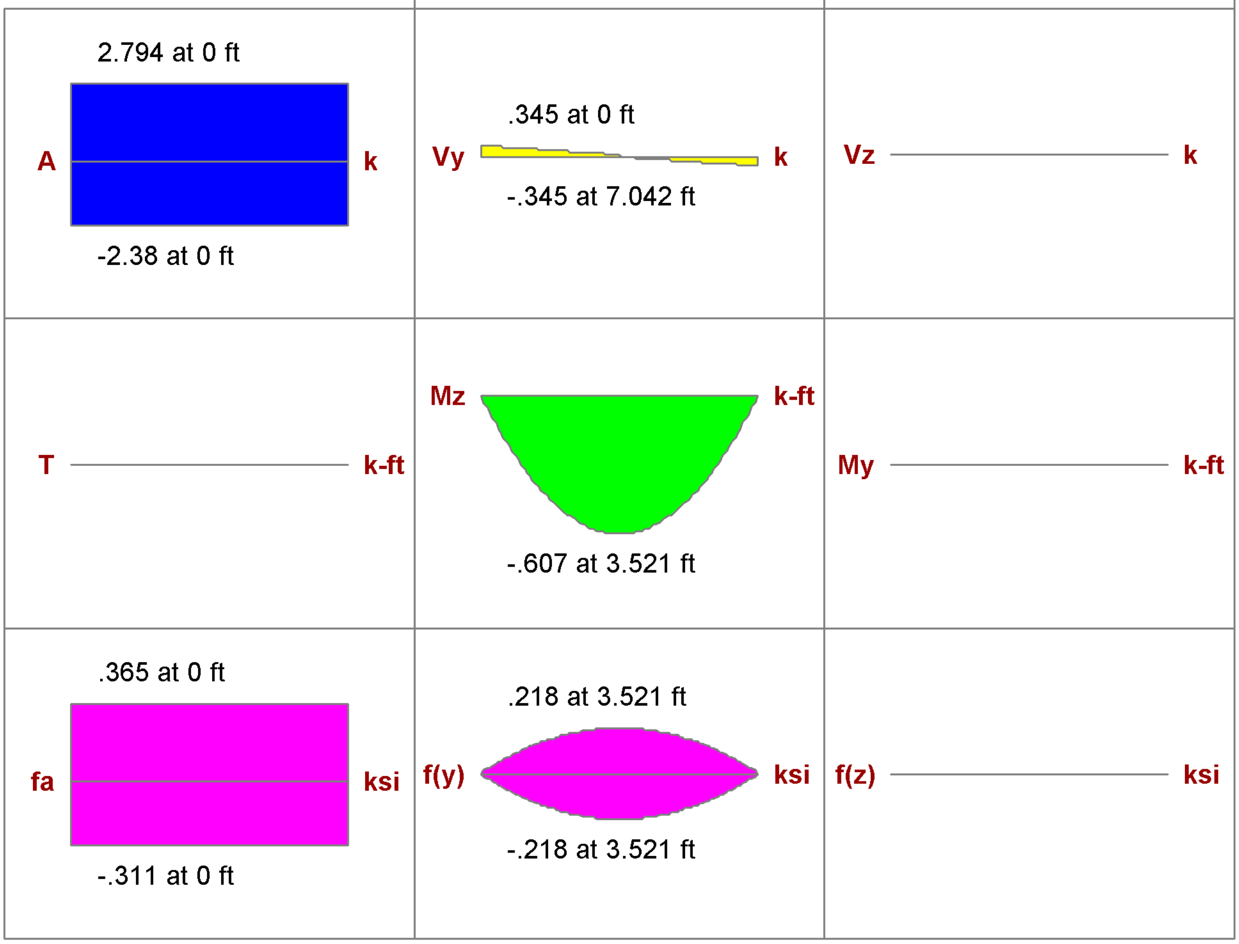
- Size from RISAFloor governed optimization -
- This load combination was not selected for wood design -

Max Defl Ratio **L/2294**

Beam: *1B26*

Shape: **W12x26**
Material: **A992**
Length: **7.042 ft**
I Joint: **F8_GRDR31**
J Joint: **F8_N38**
Envelope
Code Check: **0.011 (LC 10)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

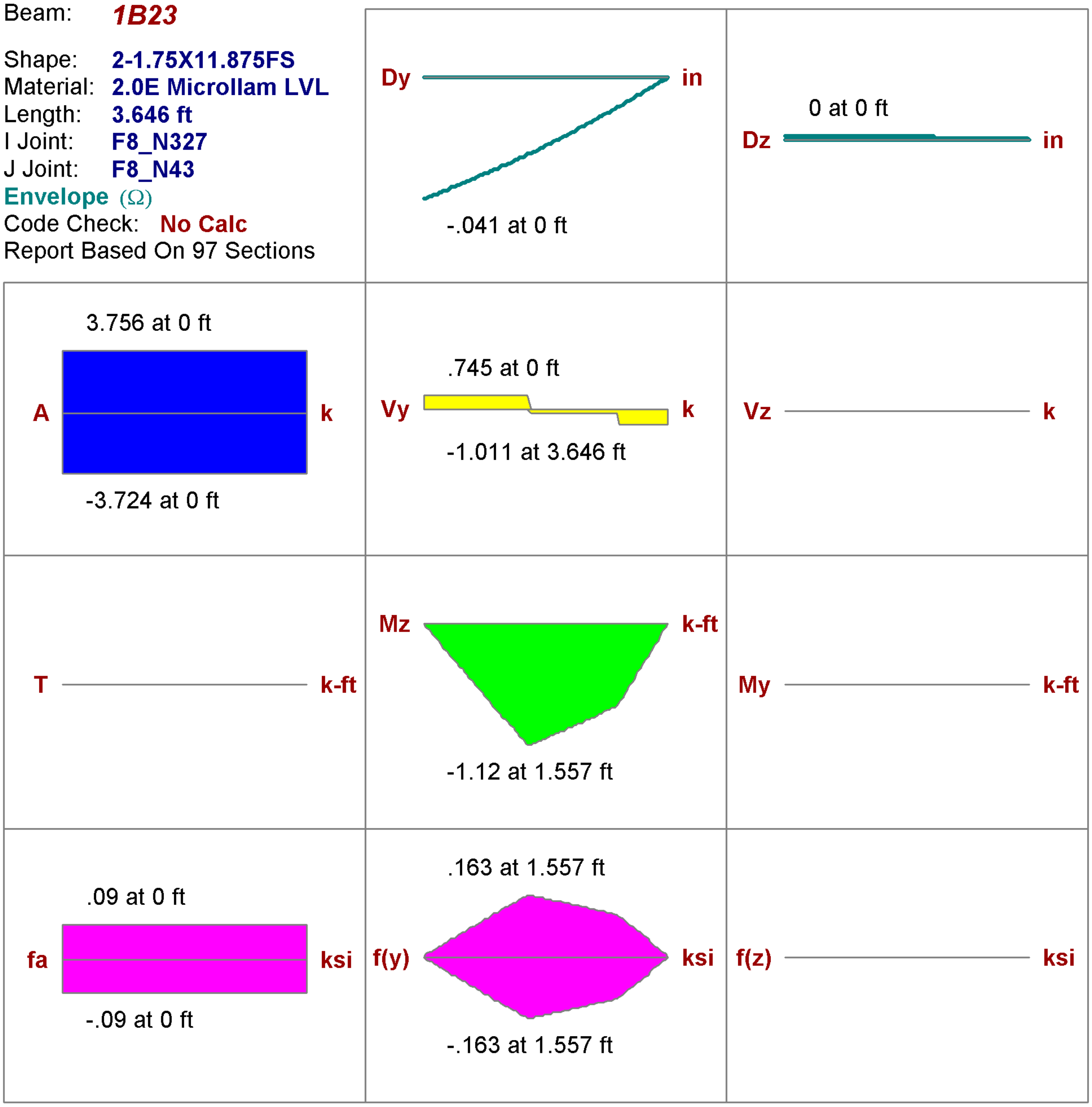
Max Bending Check	**0.011 (LC 10)**	Max Shear Check	**0.004 (y) (LC 10)**	Max Defl Ratio	**L/10000**
Location	**0 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.949**

			y-y	z-z
Fy	**50 ksi**	Lb	**7.042 ft**	**7.042 ft**
phi*Pnc	**262.33 k**	KL/r	**56.193**	**16.364**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**7.042 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

Beam: ***1B23***

Shape: **2-1.75X11.875FS**
Material: **2.0E Microllam LVL**
Length: **3.646 ft**
I Joint: **F8_N327**
J Joint: **F8_N43**
Envelope (Ω)
Code Check: **No Calc**
Report Based On 97 Sections



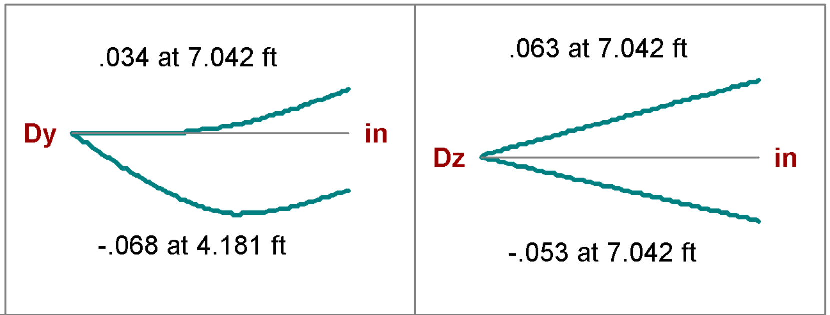
AWC NDS-15: ASD Code Check

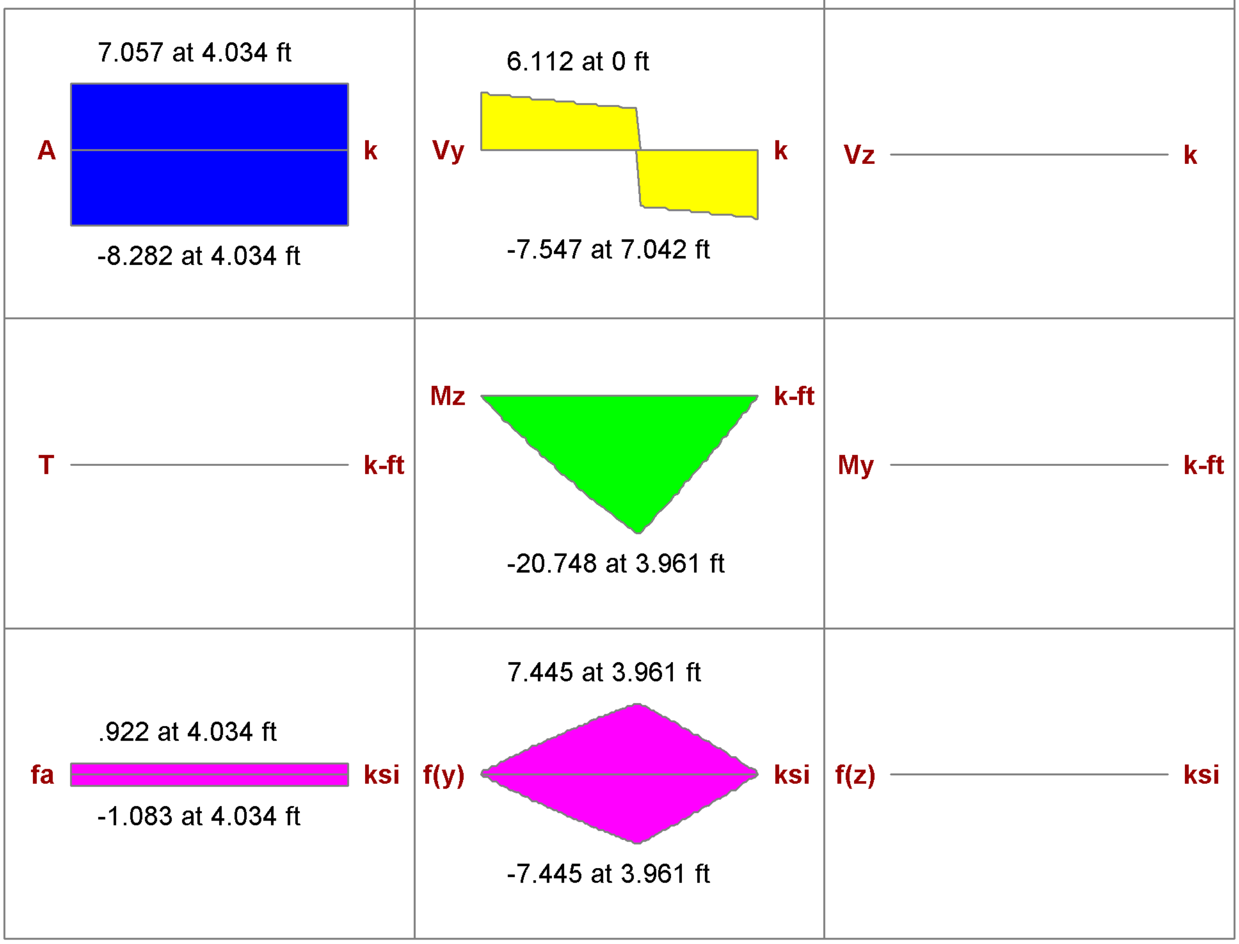
- Size from RISAFloor governed optimization -
- This load combination was not selected for wood design -

Max Defl Ratio **L/10000**

Beam: *1B17*

Shape: **W12x26**
Material: **A992**
Length: **7.042 ft**
I Joint: **F8_N36**
J Joint: **N134**
Envelope
Code Check: **0.161 (LC 12)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

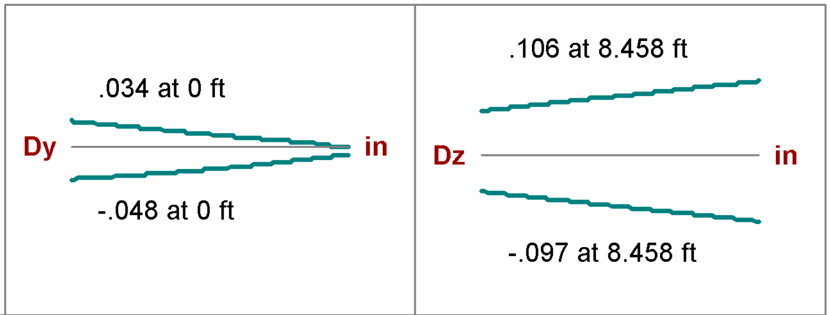
Max Bending Check	**0.161 (LC 12)**	Max Shear Check	**0.090 (y) (LC 12)**	Max Defl Ratio	**L/2073**
Location	**3.961 ft**	Location	**7.042 ft**	Location	**3.961 ft**
Equation	**H1-1b**			Span	**1**

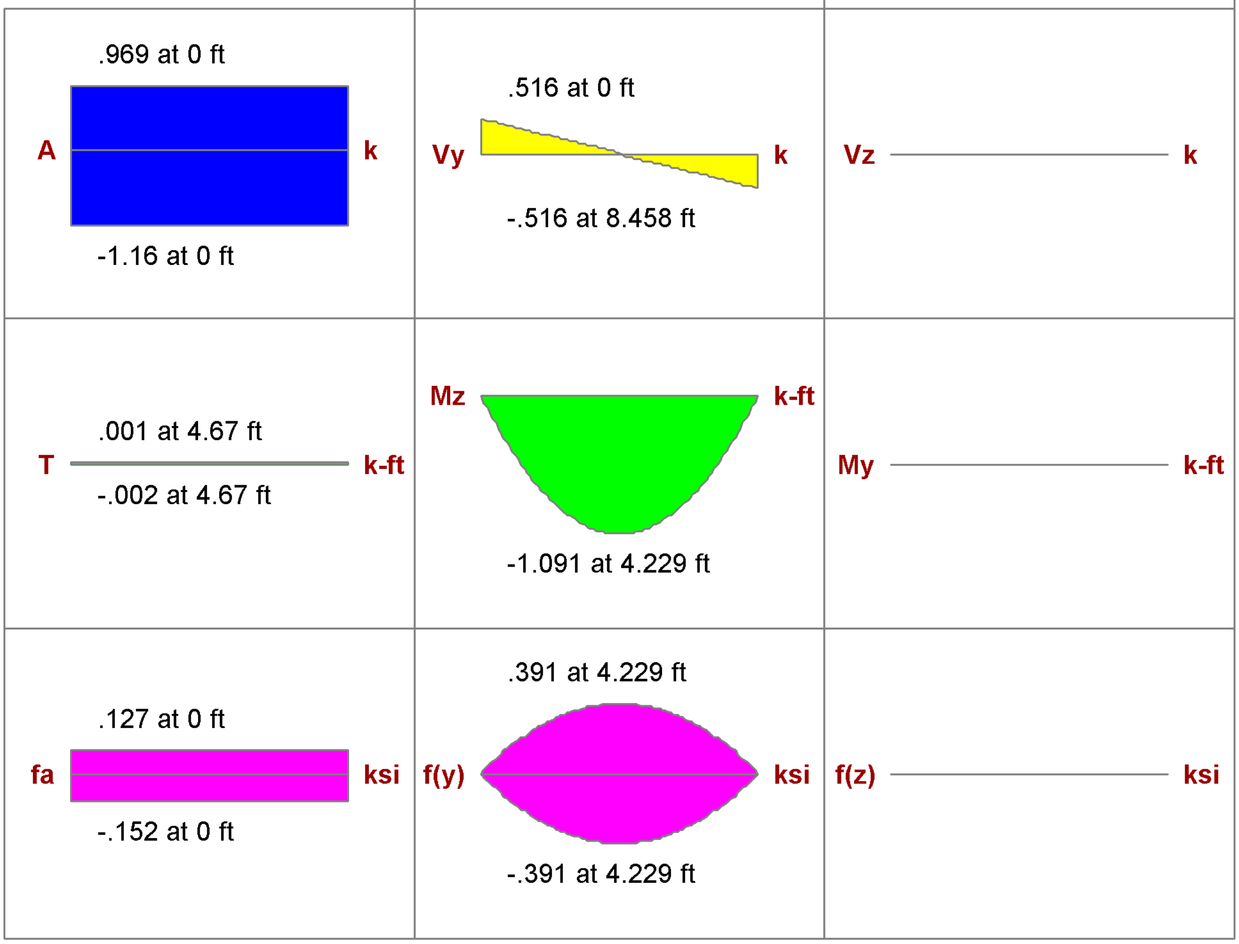
Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.949**

			y-y	z-z
Fy	**50 ksi**	Lb	**7.042 ft**	**7.042 ft**
phi*Pnc	**262.33 k**	KL/r	**56.193**	**16.364**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**7.042 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

Beam: ***1B16***

Shape: **W12x26**
Material: **A992**
Length: **8.458 ft**
I Joint: **N134**
J Joint: **F8_N37**
Envelope
Code Check: **0.010 (LC 12)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.010 (LC 12)**	Max Shear Check	**0.007 (y) (LC 9)**	Max Defl Ratio	**L/10000**
Location	**4.229 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.96**

			y-y	z-z
Fy	**50 ksi**	Lb	**8.458 ft**	**8.458 ft**
phi*Pnc	**240.089 k**	KL/r	**67.493**	**19.655**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**8.458 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

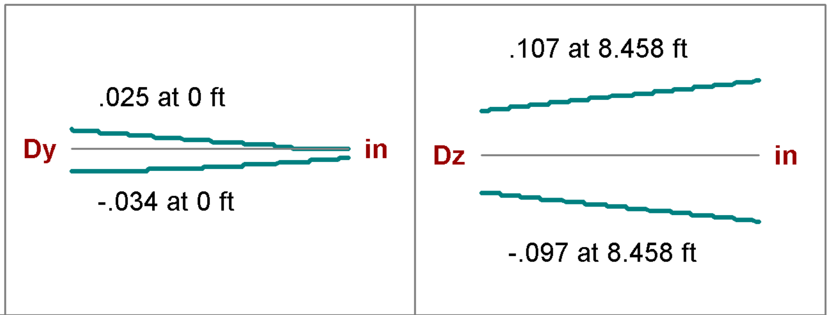
Beam: ***1B15***

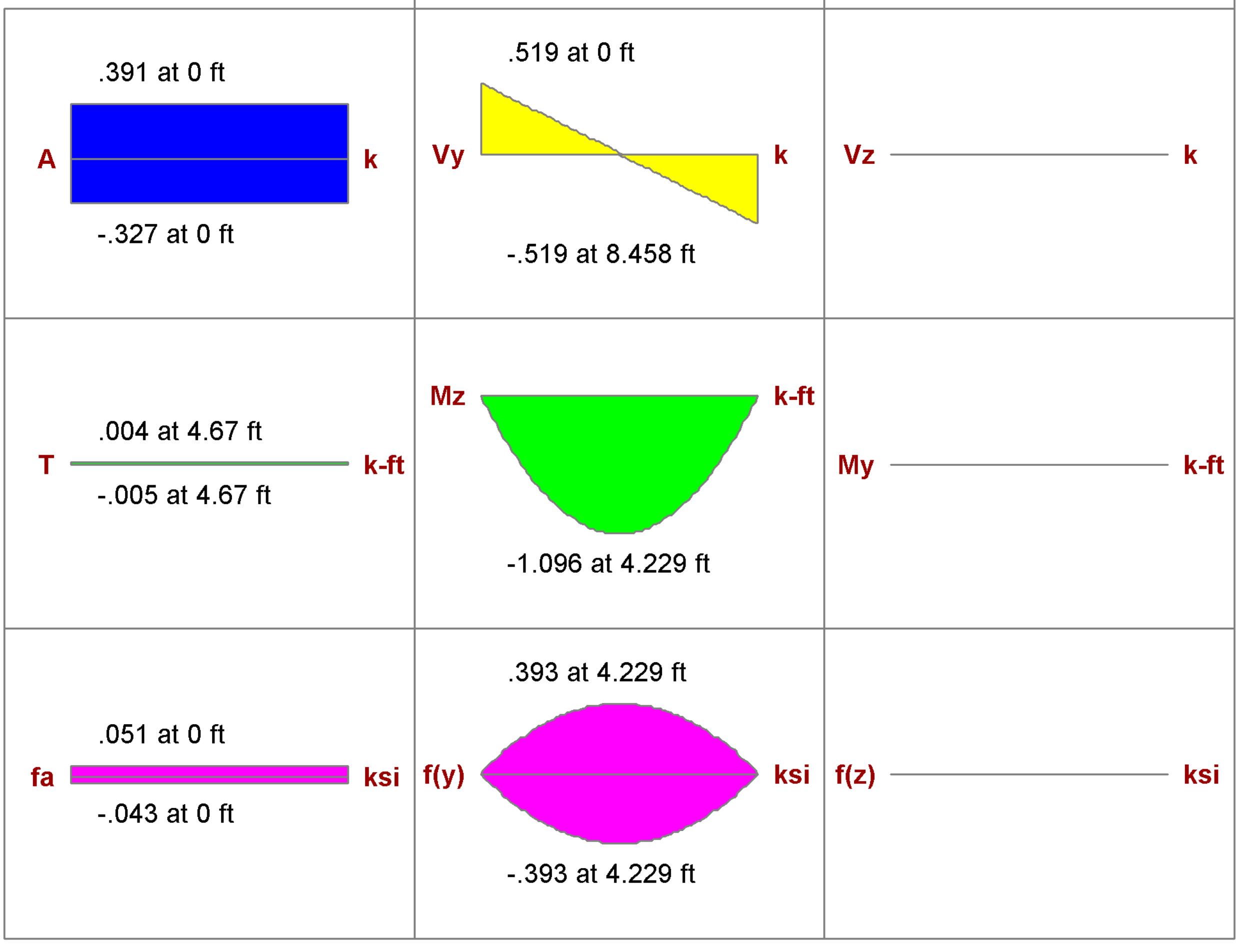
Shape:	**W12x26**
Material:	**A992**
Length:	**8.458 ft**
I Joint:	**F8_GRDR31**
J Joint:	**F8_N34**

Envelope

Code Check: **0.009 (LC 9)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

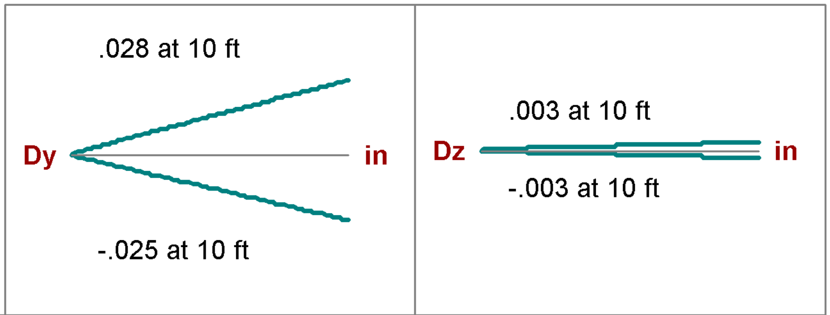
Max Bending Check	**0.009 (LC 9)**	Max Shear Check	**0.009 (y) (LC 9)**	Max Defl Ratio	**L/10000**
Location	**4.229 ft**	Location	**8.458 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**1**

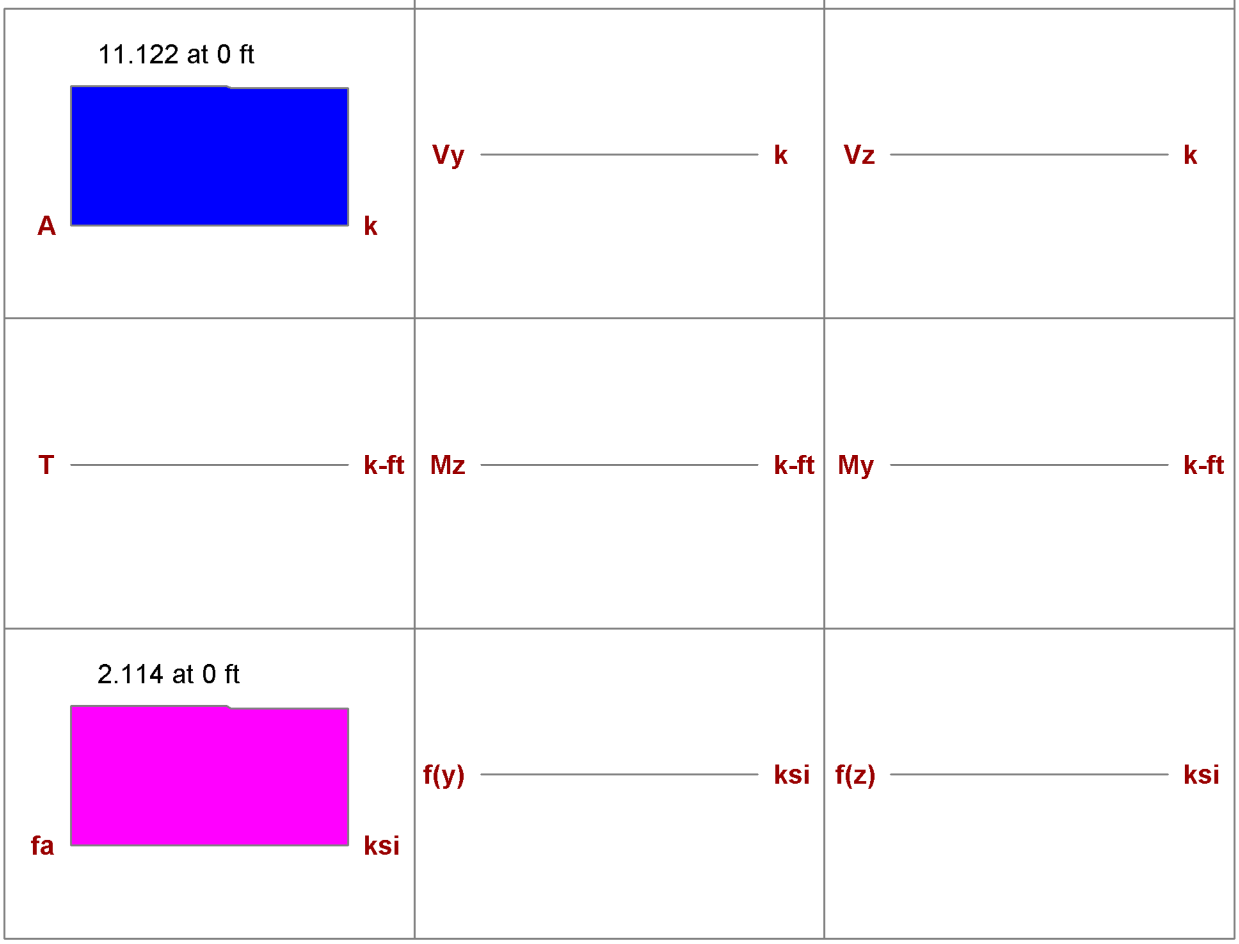
Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.96**

			y-y	z-z
Fy	**50 ksi**	Lb	**8.458 ft**	**8.458 ft**
phi*Pnc	**240.089 k**	KL/r	**67.493**	**19.655**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**8.458 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

Column: ***(3-E)***

Shape: **HSS5x5x5**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N113**
J Joint: **F8_GRDR131**
Envelope
Code Check: **0.067 (LC 12)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

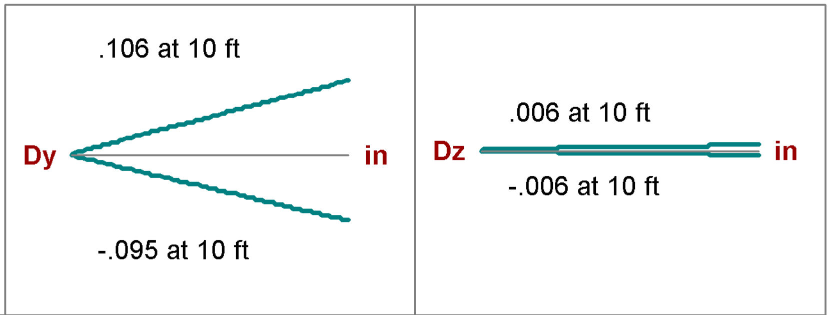
Max Bending Check	**0.067 (LC 12)**	Max Shear Check	**0.000 (y) (LC 9)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

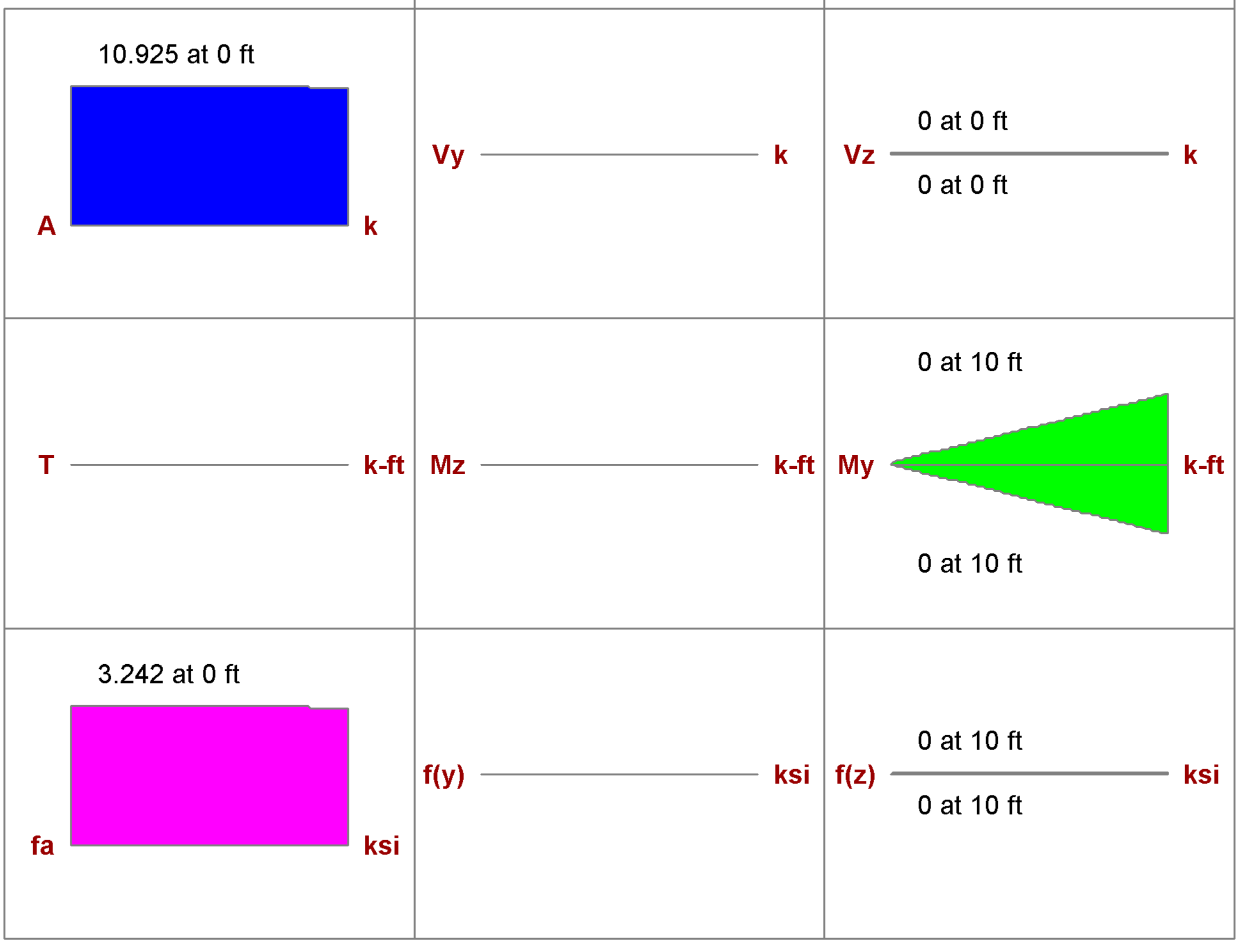
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**166.542 k**	KL/r	**63.139**	**63.139**
phi*Pnt	**217.764 k**			
phi*Mny	**31.602 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**31.602 k-ft**	L-torque	**10 ft**	
phi*Vny	**59.664 k**	Tau_b	**1**	
phi*Vnz	**59.664 k**			
phi*Tn	**26.518 k-ft**			
Cb	**1**			

Column: ***(3-F)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N121**
J Joint: **F8_N27**
Envelope
Code Check: **0.119 (LC 11)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

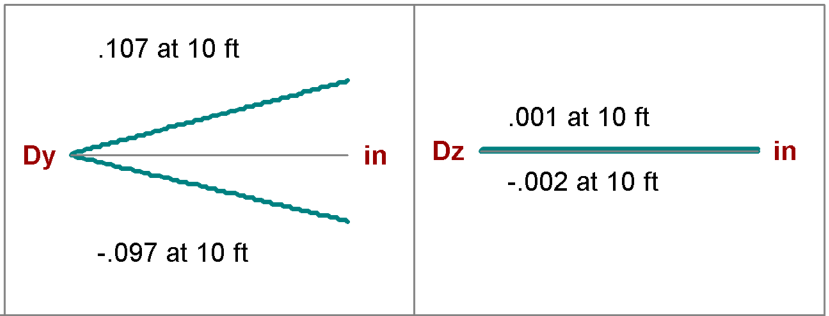
Max Bending Check	**0.119 (LC 11)**	Max Shear Check	**0.000 (z) (LC 10)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

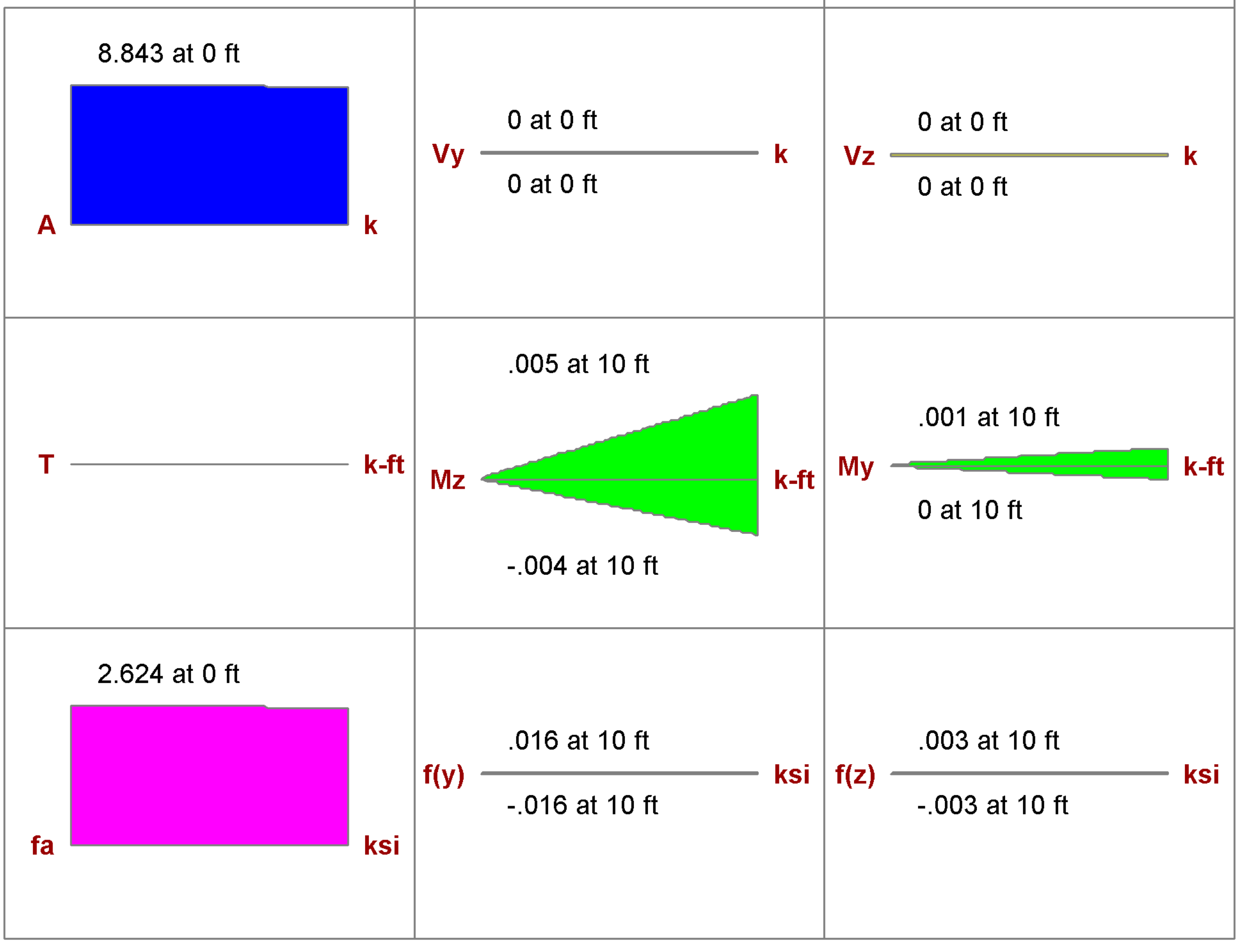
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

Column: ***(4--F)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N123**
J Joint: **F8_N34**
Envelope
Code Check: **0.096 (LC 11)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

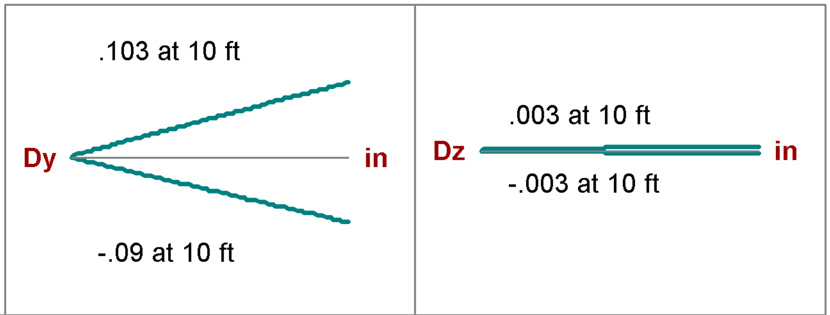
Max Bending Check	**0.096 (LC 11)**	Max Shear Check	**0.000 (y) (LC 13)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

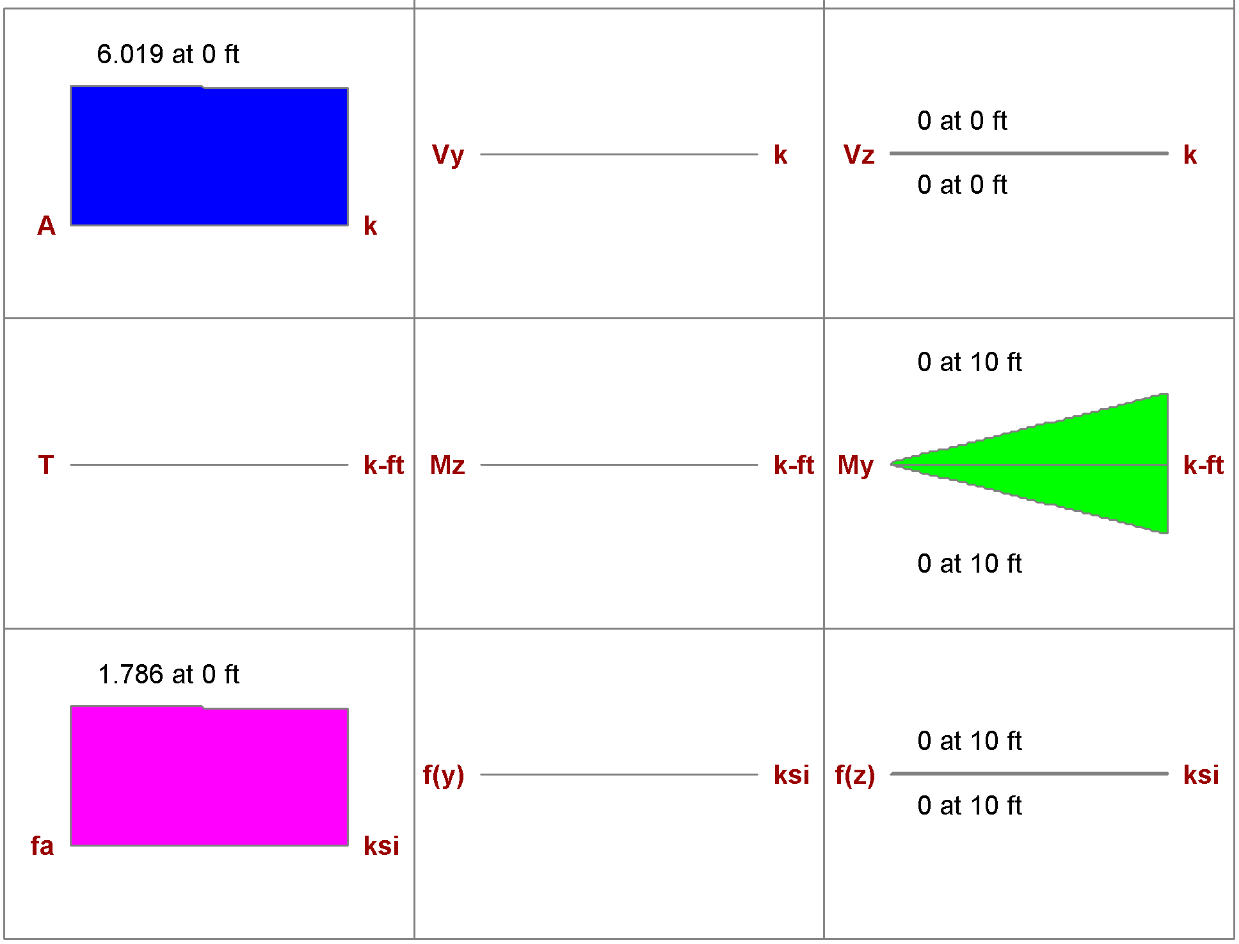
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.667**			

Column: ***(1-F)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N120**
J Joint: **F8_N25**
Envelope
Code Check: **0.066 (LC 11)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

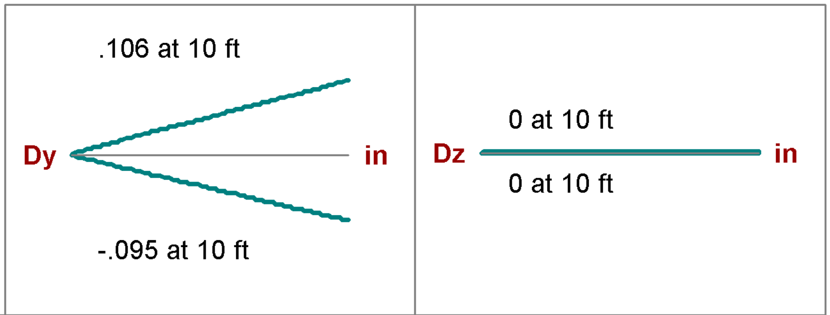
Max Bending Check	**0.066 (LC 11)**	Max Shear Check	**0.000 (z) (LC 12)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

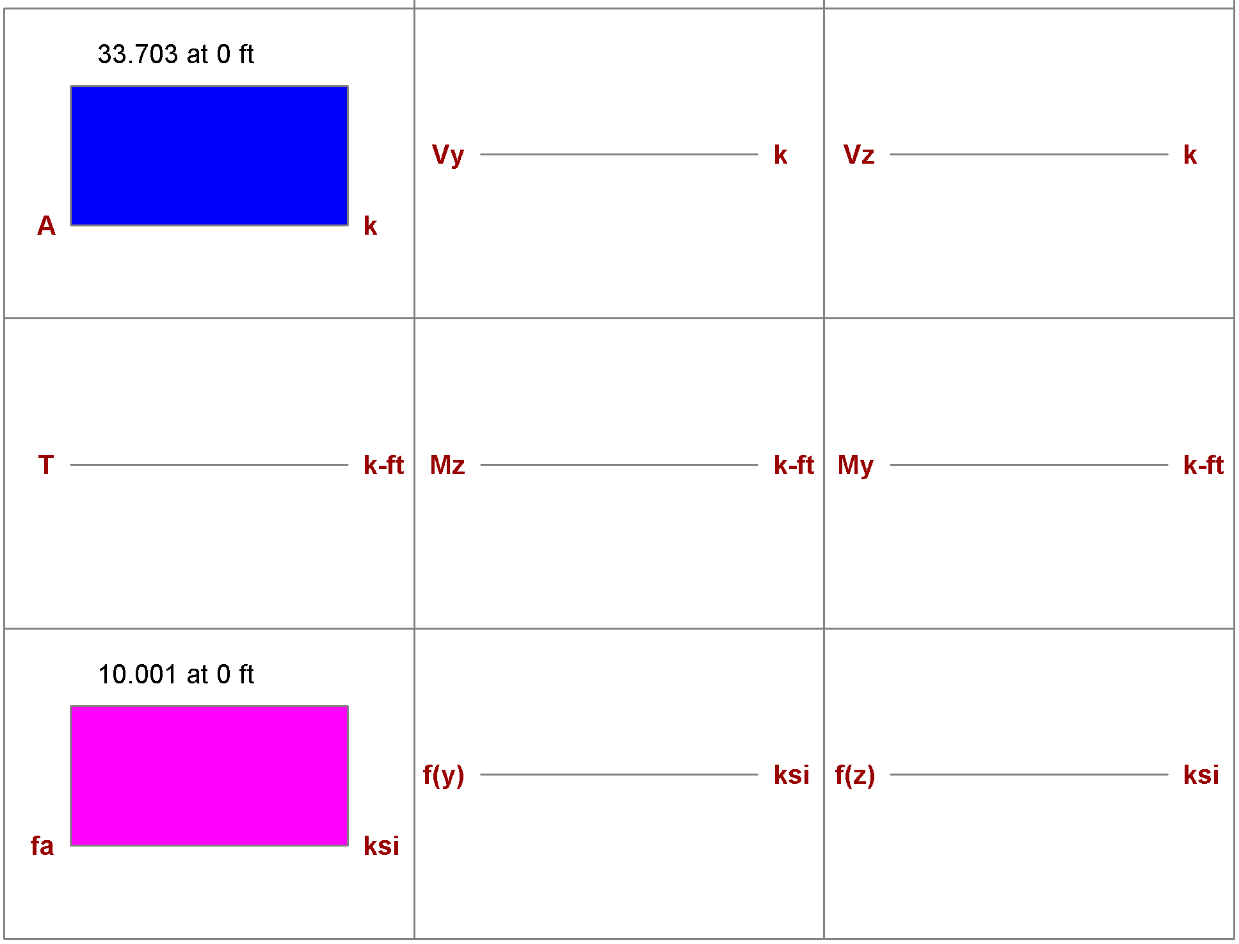
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

Column: ***(F-6)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N125A**
J Joint: **F8_N89**
Envelope
Code Check: **0.367 (LC 9)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method

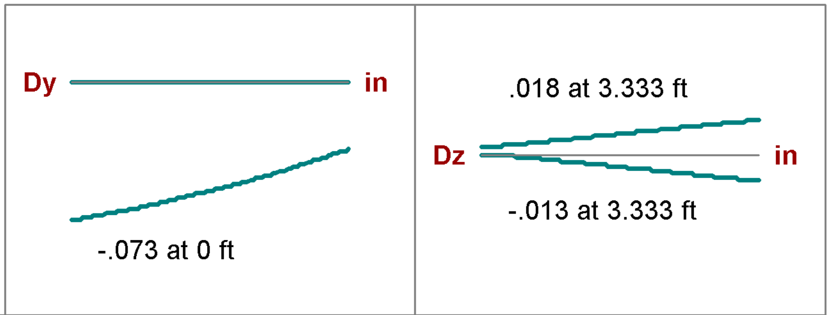
Max Bending Check	**0.367 (LC 9)**	Max Shear Check	**0.000 (y) (LC 9)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

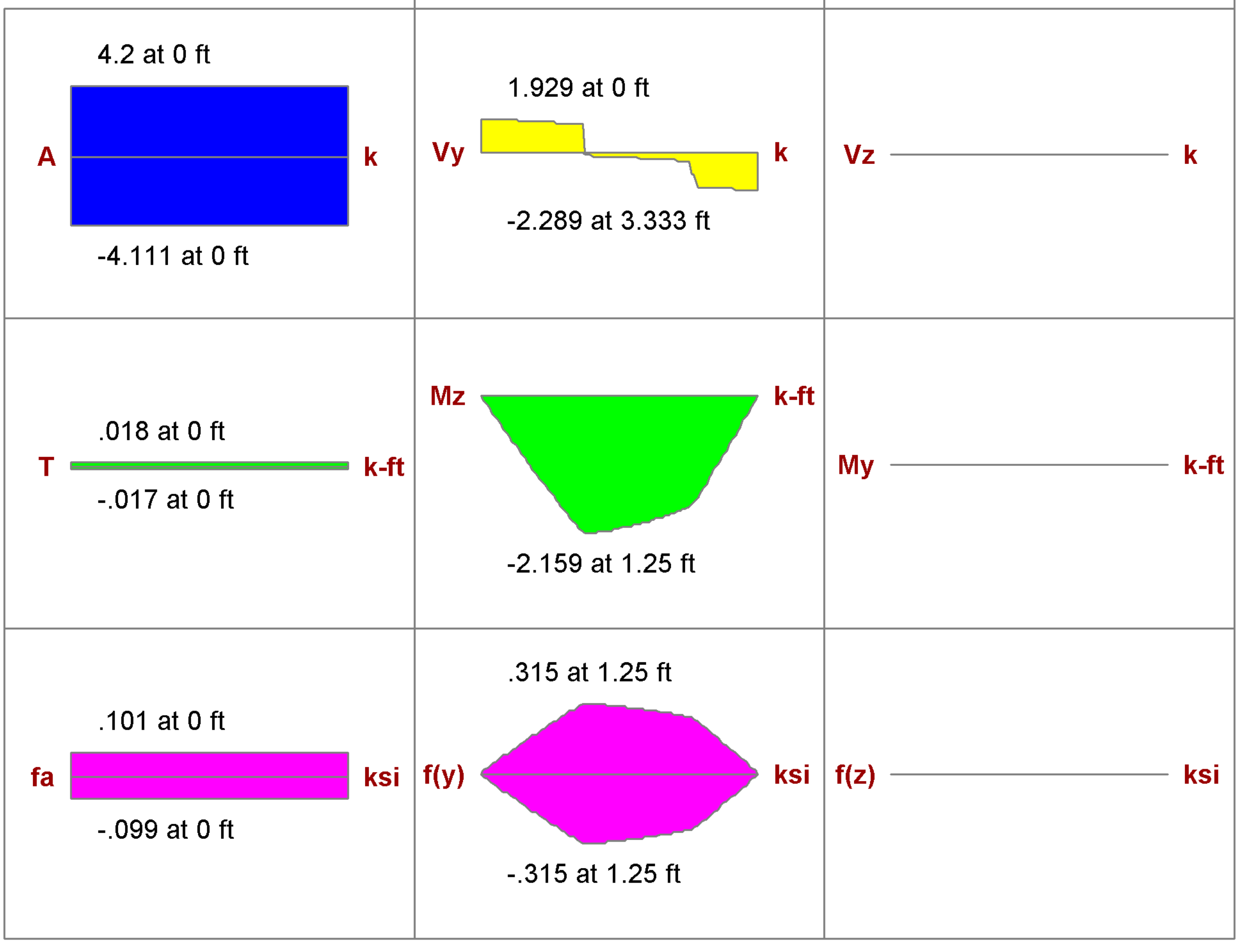
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

Beam: *1B21*

Shape: **2-1.75X11.875FS**
Material: **2.0E Microllam LVL**
Length: **3.333 ft**
I Joint: **F8_N54**
J Joint: **N111**
Envelope (Ω)
Code Check: **No Calc**
Report Based On 97 Sections





AWC NDS-15: ASD Code Check

- Size from RISAFloor governed optimization -
- This load combination was not selected for wood design -

Max Defl Ratio **L/9233**

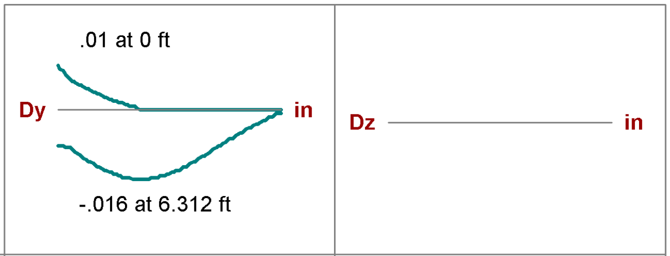
Beam: ***RB6***

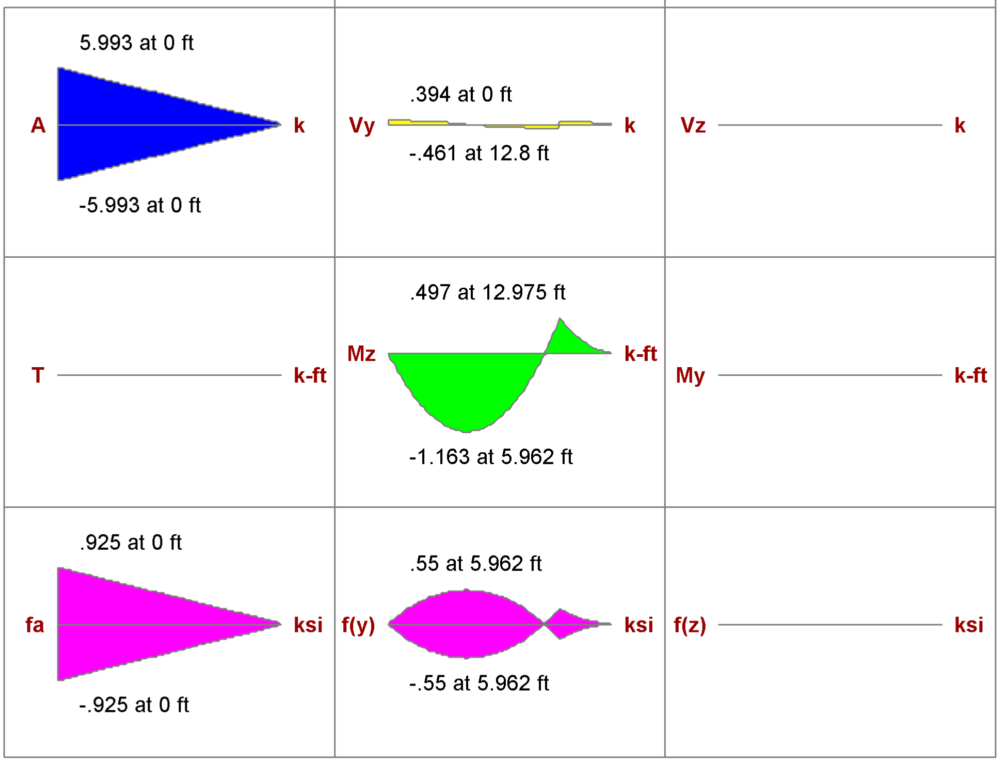
Shape:	**W12x22**
Material:	**A992**
Length:	**16.833 ft**
I Joint:	**F1_N247**
J Joint:	**F1_N248**

Envelope (Ω)

Code Check: **0.024 (LC 18)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.024 (LC 18)**	Max Shear Check	**0.005 (y) (LC 17)**	Max Defl Ratio	**L/10000**
Location	**0 ft**	Location	**12.8 ft**	Location	**0 ft**
Equation	**H1-1b***			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.952**

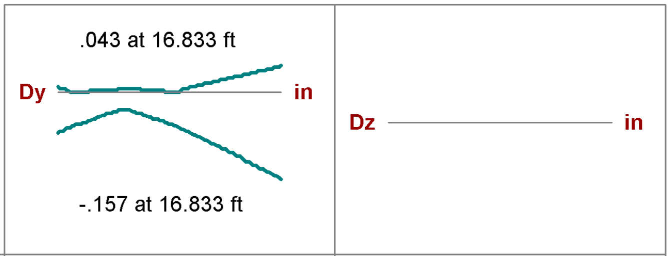
			y-y	z-z
Fy	**50 ksi**	Lb	**2.667 ft**	**16.833 ft**
phi*Pnc	**246.774 k**	KL/r	**37.74**	**41.169**
phi*Pnt	**291.6 k**			
phi*Mny	**13.725 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**109.875 k-ft**	L-torque	**16.833 ft**	
phi*Vny	**95.94 k**	Tau_b	**1**	
phi*Vnz	**92.489 k**			
Cb	**1.006**			

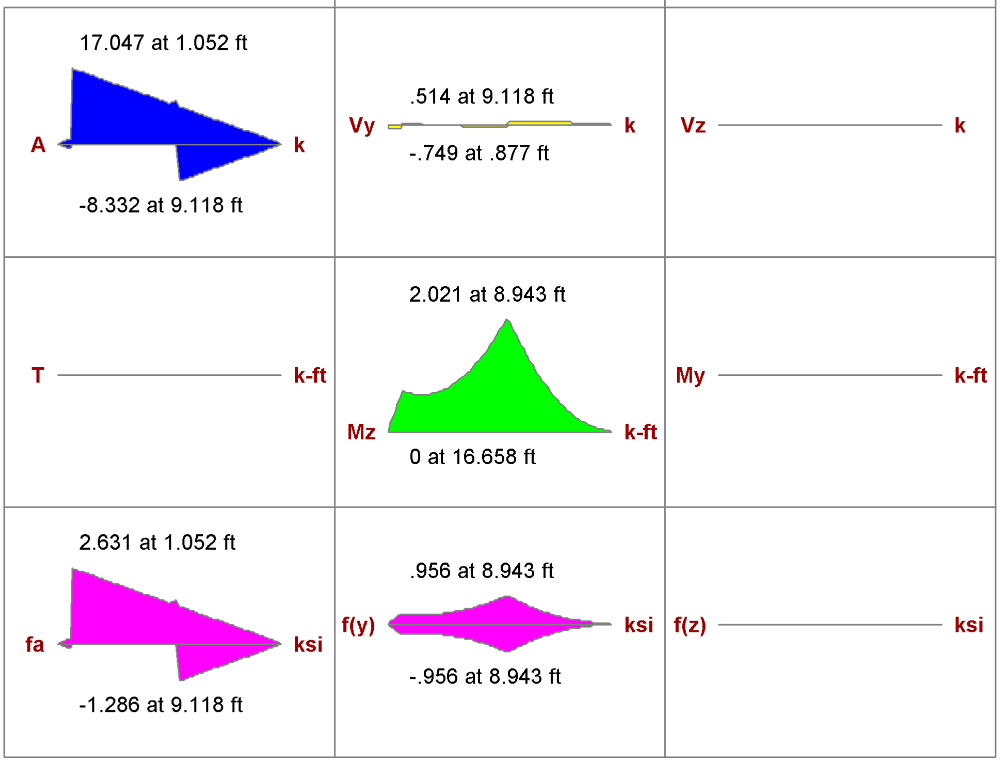
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***RB18***

Shape: **W12x22**
Material: **A992**
Length: **16.833 ft**
I Joint: **F1_N246**
J Joint: **F1_N148**
Envelope (Ω)
Code Check: **0.069 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.069 (LC 20)**	Max Shear Check	**0.008 (y) (LC 10)**	Max Defl Ratio	**L/916**
Location	**1.052 ft**	Location	**.877 ft**	Location	**0 ft**
Equation	**H1-1b***			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.952**

			y-y	z-z
Fy	**50 ksi**	Lb	**2.667 ft**	**16.833 ft**
phi*Pnc	**246.774 k**	KL/r	**37.74**	**41.169**
phi*Pnt	**291.6 k**			
phi*Mny	**13.725 k-ft**	L Comp Flange	**16.833 ft**	
phi*Mnz	**44.32 k-ft**	L-torque	**16.833 ft**	
phi*Vny	**95.94 k**	Tau_b	**1**	
phi*Vnz	**92.489 k**			
Cb	**1.555**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

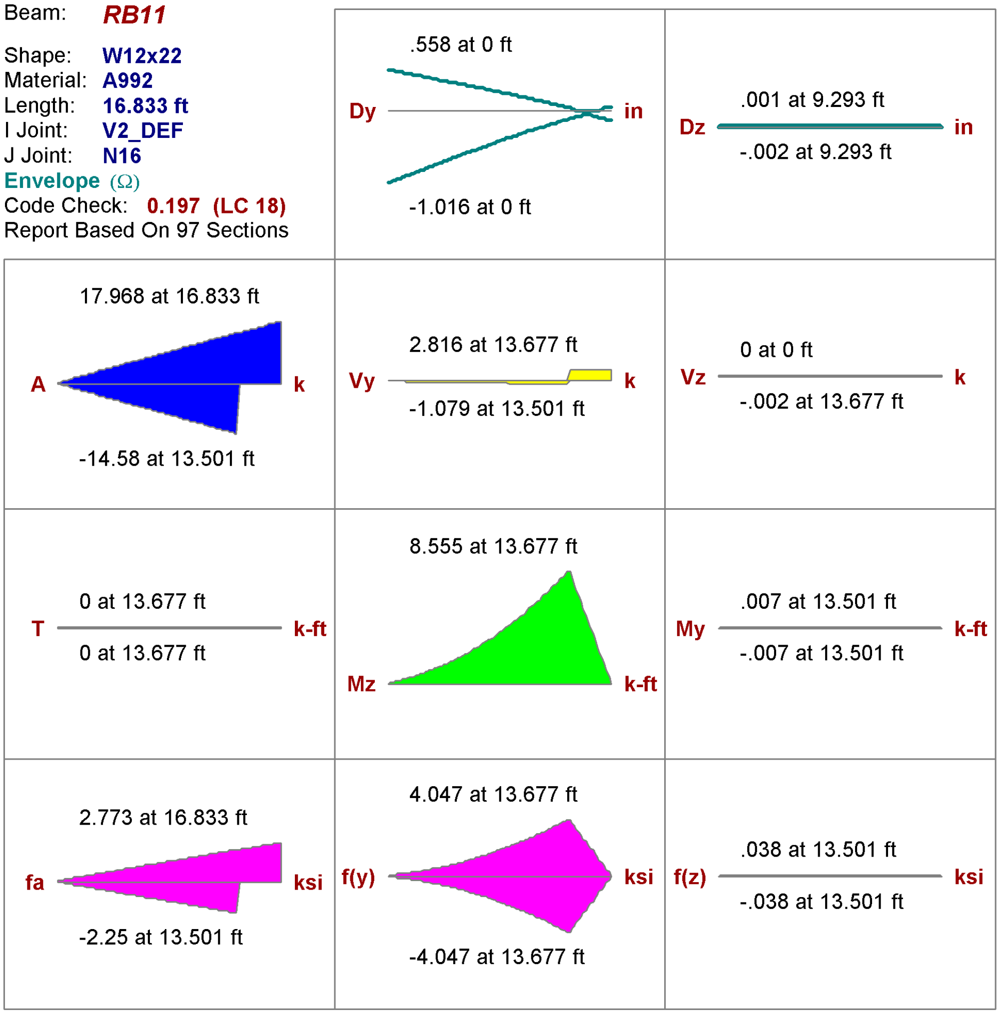
Beam: ***RB11***

Shape:	**W12x22**
Material:	**A992**
Length:	**16.833 ft**
I Joint:	**V2_DEF**
J Joint:	**N16**

Envelope (Ω)

Code Check: **0.197 (LC 18)**

Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.197 (LC 18)**	Max Shear Check	**0.030 (y) (LC 19)**	Max Defl Ratio	**L/4417**
Location	**13.677 ft**	Location	**13.677 ft**	Location	**8.416 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.952**

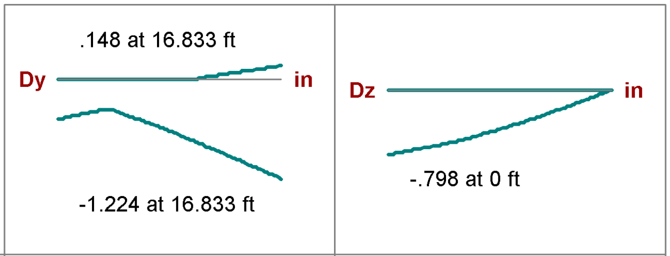
			y-y	z-z
Fy	**50 ksi**	Lb	**2.667 ft**	**16.833 ft**
phi*Pnc	**246.774 k**	KL/r	**37.74**	**41.169**
phi*Pnt	**291.6 k**			
phi*Mny	**13.725 k-ft**	L Comp Flange	**16.833 ft**	
phi*Mnz	**48.409 k-ft**	L-torque	**16.833 ft**	
phi*Vny	**95.94 k**	Tau_b	**1**	
phi*Vnz	**92.489 k**			
Cb	**1.687**			

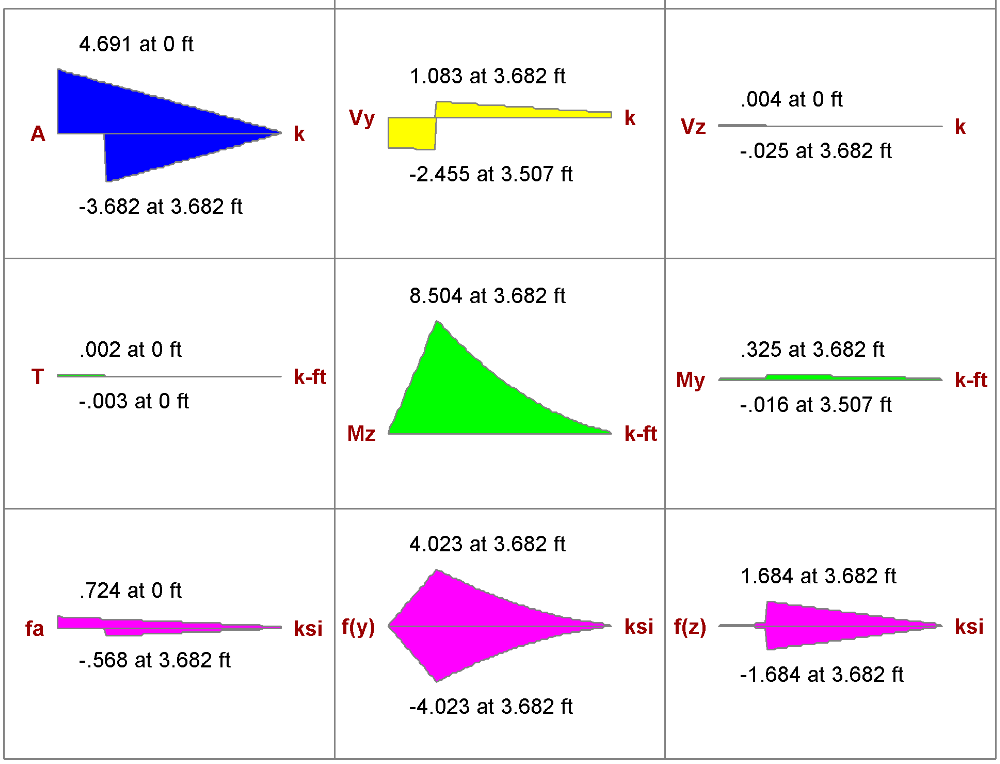
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***RB1***

Shape: **W12x22**
Material: **A992**
Length: **16.833 ft**
I Joint: **N4**
J Joint: **V3_DEF**
Envelope (Ω)
Code Check: **0.215 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.215 (LC 18)**	Max Shear Check	**0.027 (y) (LC 18)**	Max Defl Ratio	**L/4088**
Location	**3.682 ft**	Location	**3.507 ft**	Location	**8.416 ft**
Equation	**H1-1b**			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.952**

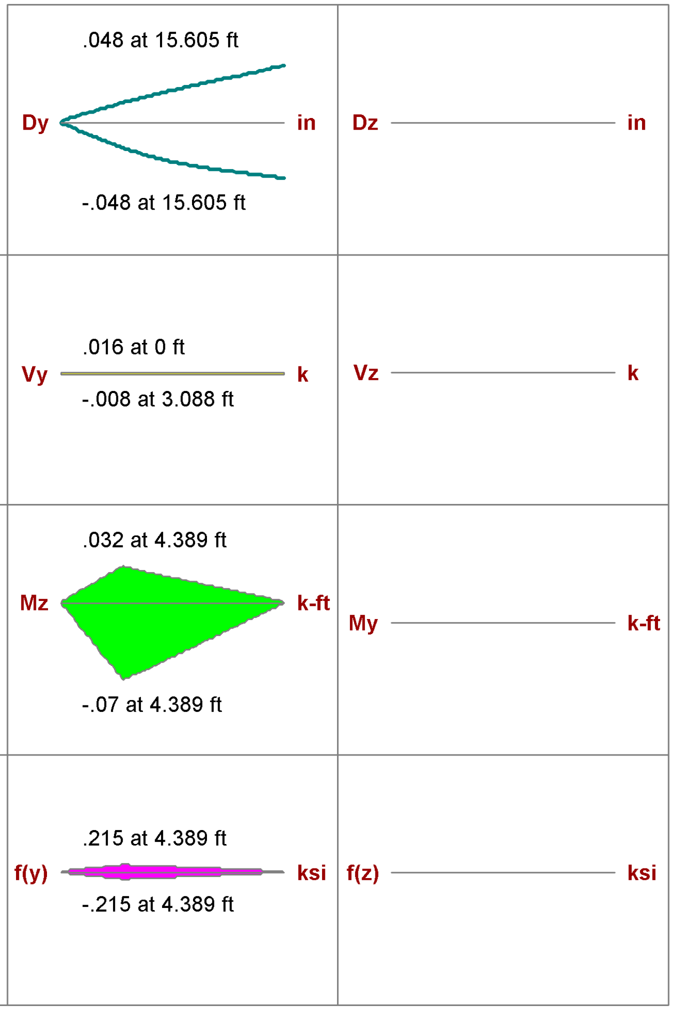
			y-y	z-z
Fy	**50 ksi**	Lb	**2.667 ft**	**16.833 ft**
phi*Pnc	**246.774 k**	KL/r	**37.74**	**41.169**
phi*Pnt	**291.6 k**			
phi*Mny	**13.725 k-ft**	L Comp Flange	**16.833 ft**	
phi*Mnz	**45.965 k-ft**	L-torque	**16.833 ft**	
phi*Vny	**95.94 k**	Tau_b	**1**	
phi*Vnz	**92.489 k**			
Cb	**1.611**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Column: **CS8**

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **15.605 ft**
I Joint: **F4_N247**
J Joint: **F1_N247**
Envelope (Ω)
Code Check: **0.061 (LC 22)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.061 (LC 22)**	Max Shear Check	**0.000 (y) (LC 18)**
Location	**4.389 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**11.272 ft**	**11.272 ft**
phi*Pnc	**81.98 k**	KL/r	**88.907**	**88.907**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**11.272 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**11.272 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.634**			

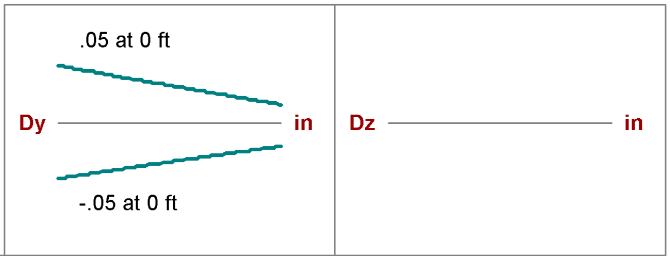
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

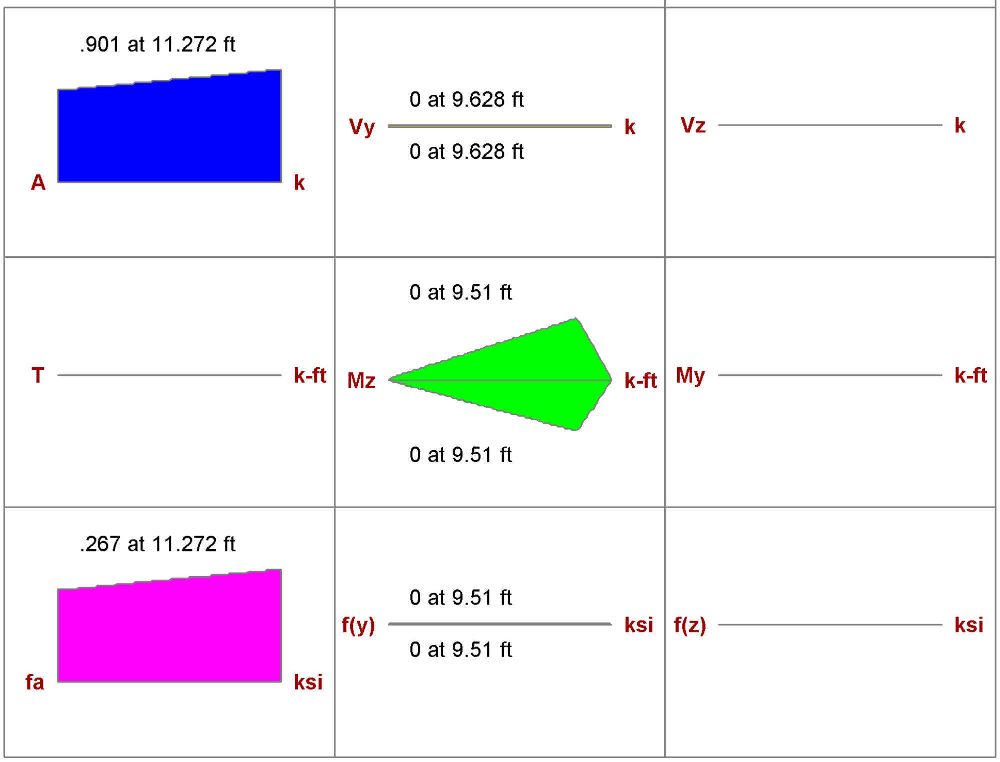
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **88.907** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Beam: ***CS7***

Shape: **HSS4x4x4**
Material: **A992**
Length: **11.272 ft**
I Joint: **F1_N242**
J Joint: **F2_N31**
Envelope
Code Check: **0.011 (LC 10)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method

Max Bending Check	**0.011 (LC 10)**	Max Shear Check	**0.000 (y) (LC 1)**	Max Defl Ratio	**L/10000**
Location	**11.272 ft**	Location	**9.628 ft**	Location	**0 ft**
Equation	**H1-1b***			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

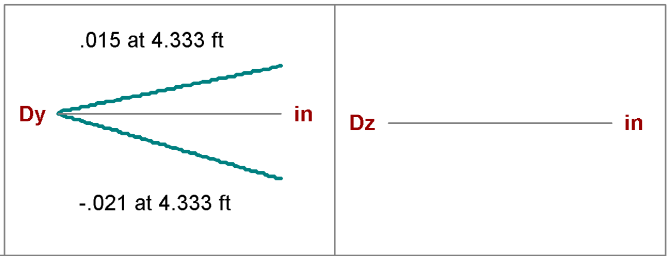
			y-y	z-z
Fy	**50 ksi**	Lb	**11.272 ft**	**11.272 ft**
phi*Pnc	**85.082 k**	KL/r	**88.907**	**88.907**
phi*Pnt	**151.65 k**			
phi*Mny	**17.588 k-ft**	L Comp Flange	**11.272 ft**	
phi*Mnz	**17.588 k-ft**	L-torque	**11.272 ft**	
phi*Vny	**41.533 k**	Tau_b	**1**	
phi*Vnz	**41.533 k**			
phi*Tn	**14.769 k-ft**			
Cb	**1**			

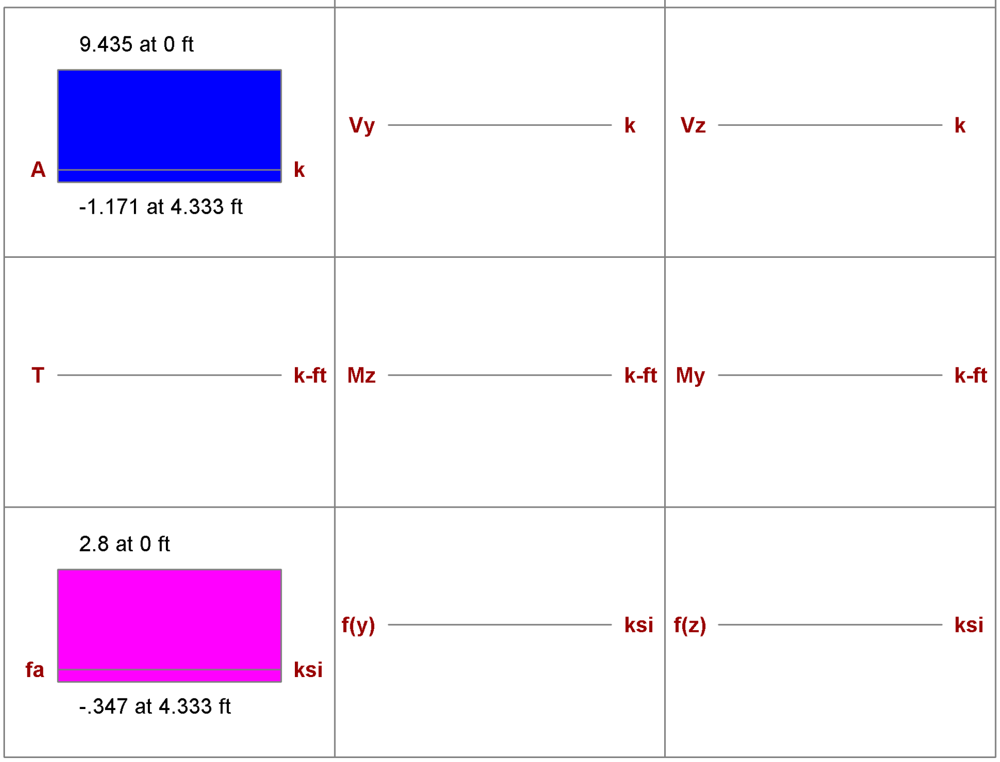
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***CS6***

Shape: **HSS4x4x4**
Material: **A992**
Length: **4.333 ft**
I Joint: **N211**
J Joint: **F2_N31**
Envelope (Ω)
Code Check: **0.068 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.068 (LC 18)**	Max Shear Check	**0.000 (y) (LC 9)**	Max Defl Ratio	**L/10000**
Location	**0 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**4.333 ft**	**4.333 ft**
phi*Pnc	**139.236 k**	KL/r	**34.177**	**34.177**
phi*Pnt	**151.65 k**			
phi*Mny	**17.588 k-ft**	L Comp Flange	**4.333 ft**	
phi*Mnz	**17.588 k-ft**	L-torque	**4.333 ft**	
phi*Vny	**41.533 k**	Tau_b	**1**	
phi*Vnz	**41.533 k**			
phi*Tn	**14.769 k-ft**			
Cb	**1**			

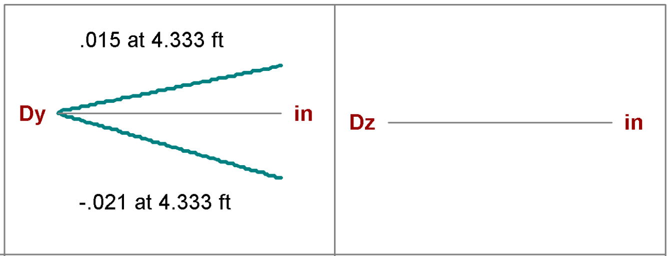
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

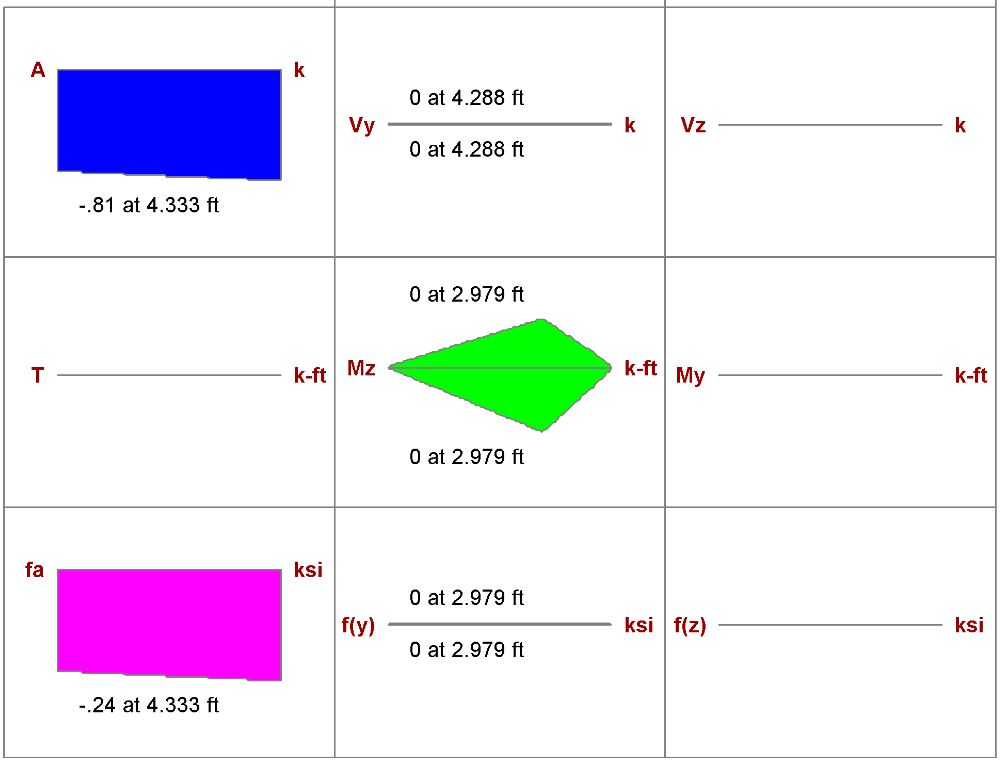
Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Miscellaneous Seismic Checks/Warnings:

Column: ***CS5***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **4.333 ft**
I Joint: **N214A**
J Joint: **F2_N248**
Envelope (Ω)
Code Check: **0.003 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.003 (LC 20)**	Max Shear Check	**0.000 (y) (LC 9)**
Location	**4.333 ft**	Location	**4.288 ft**
Equation	**H1-1b**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**1 ft**	**1 ft**
phi*Pnc	**138.935 k**	KL/r	**7.888**	**7.888**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**4.333 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**4.333 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

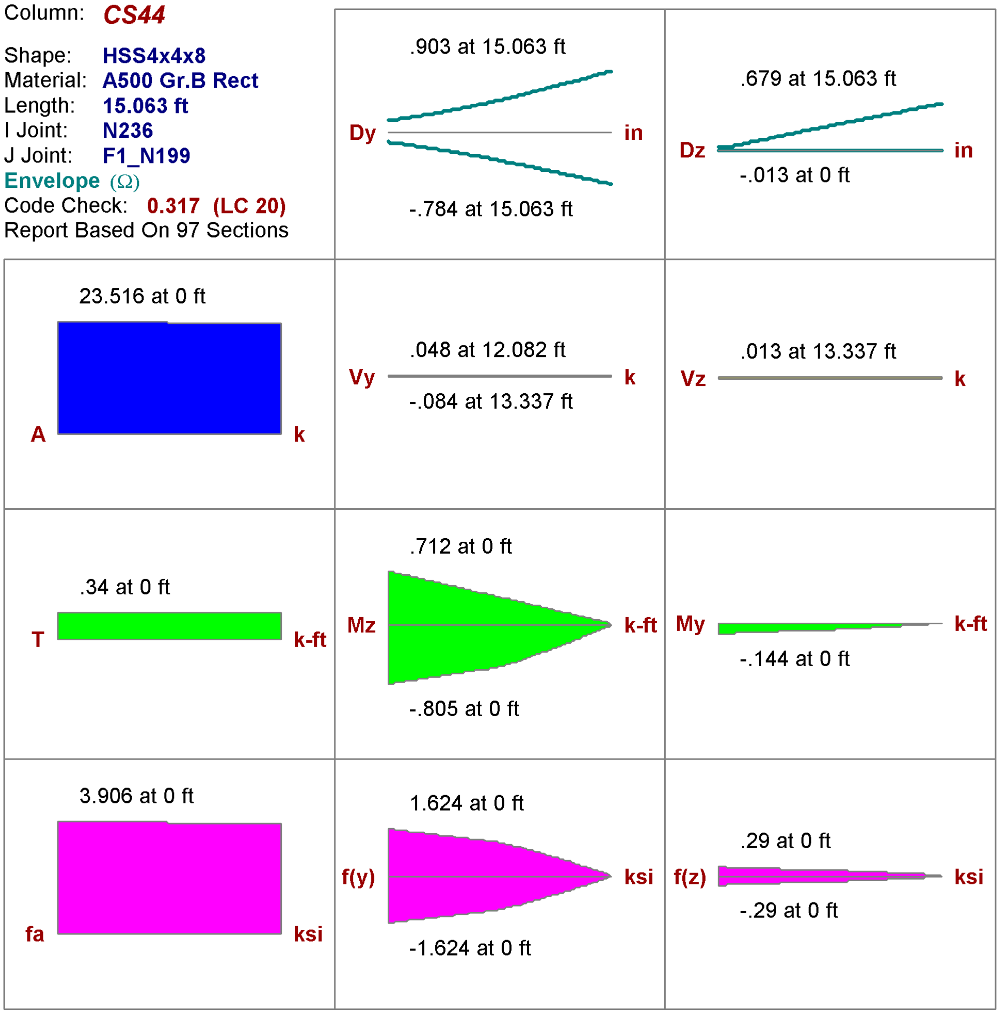
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **7.888** **<= 60 per 341-10 E3.4c(2)(2)**

Column: ***CS44***

Shape: **HSS4x4x8**
Material: **A500 Gr.B Rect**
Length: **15.063 ft**
I Joint: **N236**
J Joint: **F1_N199**
Envelope (Ω)
Code Check: **0.317 (LC 20)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.317 (LC 20)**	Max Shear Check	**0.015 (y) (LC 18)**
Location	**0 ft**	Location	**12.082 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**15.063 ft**	**15.063 ft**
phi*Pnc	**82.281 k**	KL/r	**128.563**	**128.563**
phi*Pnt	**249.228 k**			
phi*Mny	**26.565 k-ft**	L Comp Flange	**15.063 ft**	
phi*Mnz	**26.565 k-ft**	L-torque	**15.063 ft**	
phi*Vny	**60.179 k**	Tau_b	**1**	
phi*Vnz	**60.179 k**			
phi*Tn	**23.253 k-ft**			
Cb	**1.163**			

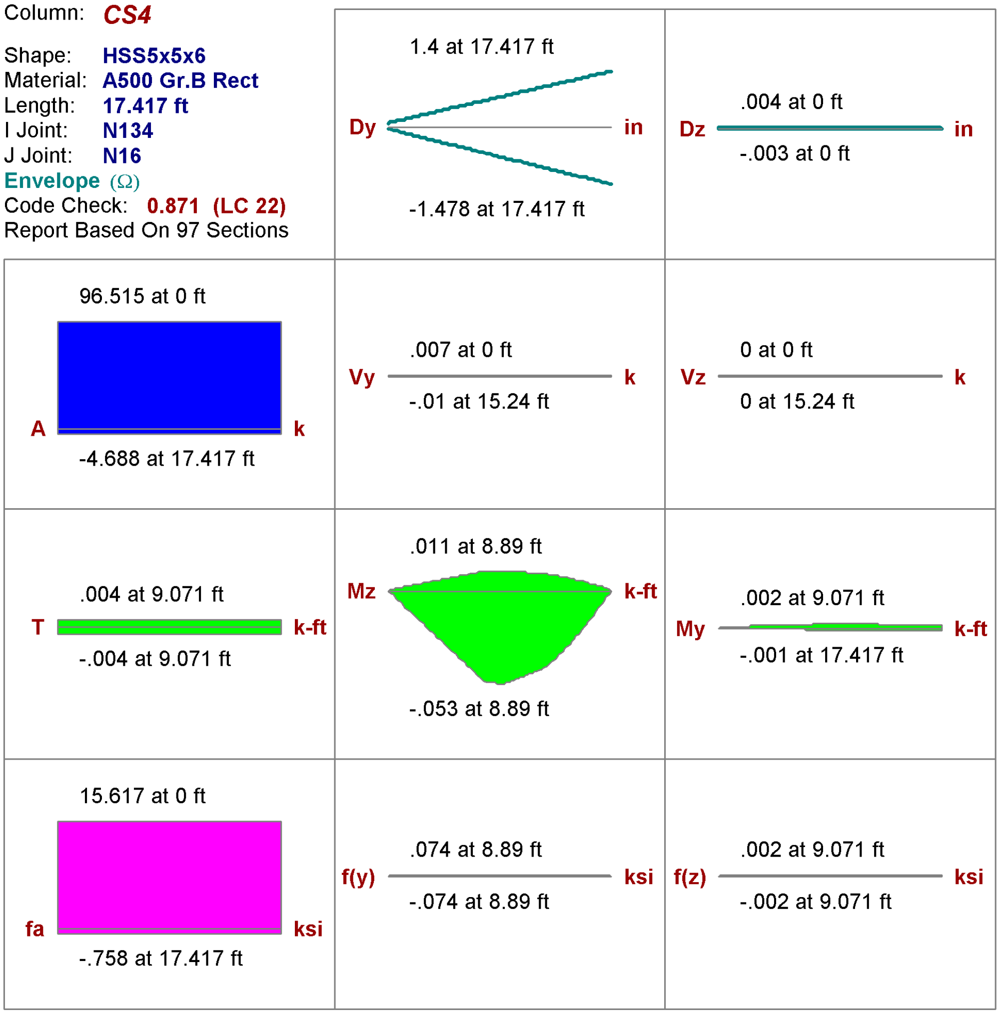
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Column**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **128.563** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: **CS4**

Shape: **HSS5x5x6**
Material: **A500 Gr.B Rect**
Length: **17.417 ft**
I Joint: **N134**
J Joint: **N16**
Envelope (Ω)
Code Check: **0.871 (LC 22)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.871 (LC 22)**	Max Shear Check	**0.000 (y) (LC 18)**
Location	**0 ft**	Location	**15.24 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**17.417 ft**	**17.417 ft**
phi*Pnc	**110.801 k**	KL/r	**111.538**	**111.538**
phi*Pnt	**255.852 k**			
phi*Mny	**36.57 k-ft**	L Comp Flange	**17.417 ft**	
phi*Mnz	**36.57 k-ft**	L-torque	**17.417 ft**	
phi*Vny	**68.538 k**	Tau_b	**1**	
phi*Vnz	**68.538 k**			
phi*Tn	**30.915 k-ft**			
Cb	**1.694**			

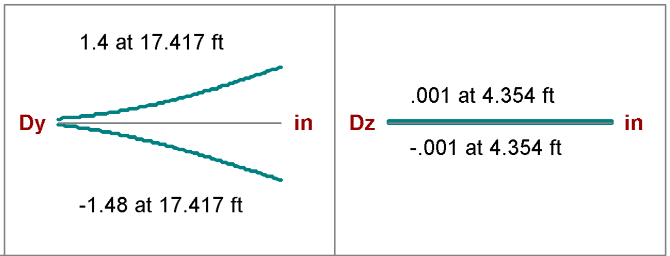
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

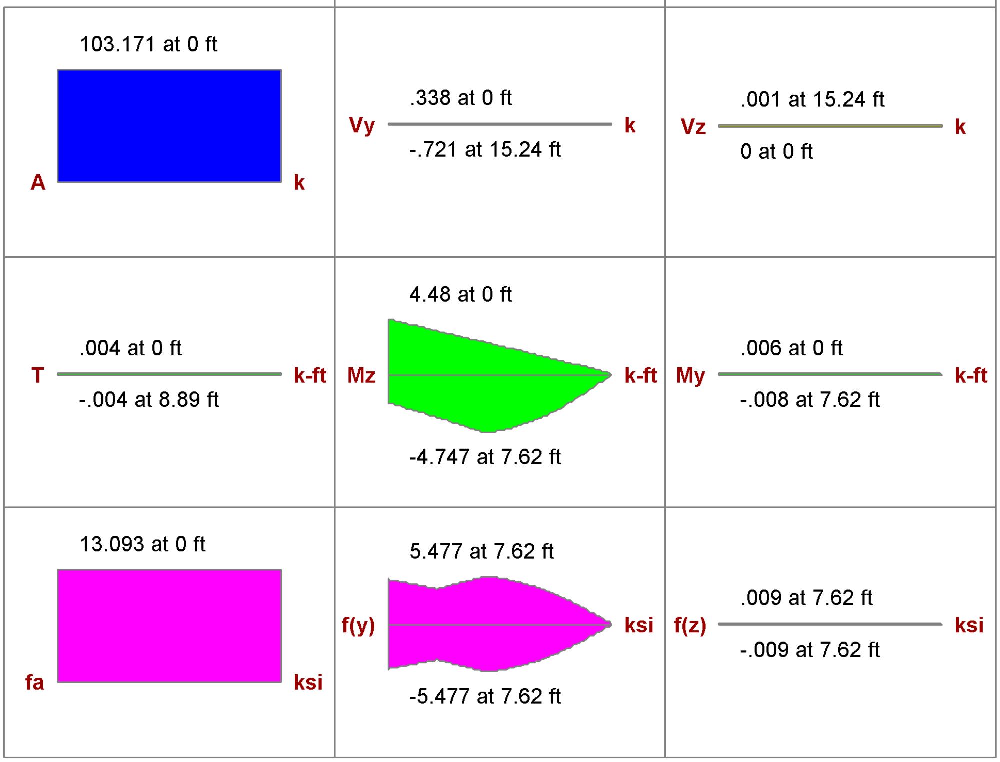
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **111.538** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***CS3***

Shape: **HSS5x5x8**
Material: **A500 Gr.B Rect**
Length: **17.417 ft**
I Joint: **F8_GRDR31**
J Joint: **F1_N200**
Envelope (Ω)
Code Check: **0.862 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.862 (LC 20)**	Max Shear Check	**0.009 (y) (LC 20)**
Location	**7.62 ft**	Location	**15.24 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**17.417 ft**	**17.417 ft**
phi*Pnc	**133.89 k**	KL/r	**115.063**	**115.063**
phi*Pnt	**326.232 k**			
phi*Mny	**45.195 k-ft**	L Comp Flange	**17.417 ft**	
phi*Mnz	**45.195 k-ft**	L-torque	**17.417 ft**	
phi*Vny	**83.28 k**	Tau_b	**1**	
phi*Vnz	**83.28 k**			
phi*Tn	**38.788 k-ft**			
Cb	**1.054**			

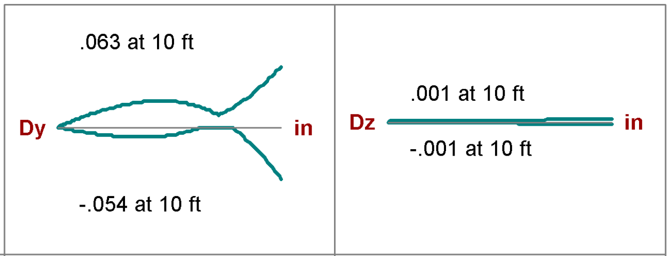
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

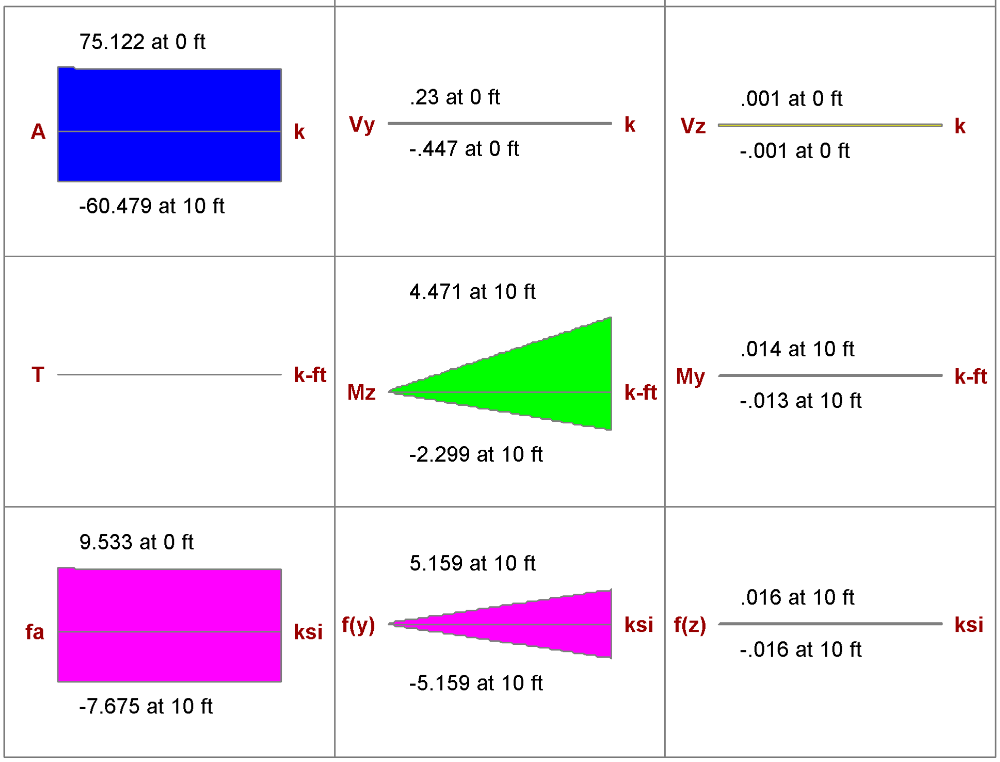
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **115.063** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: **CS2**

Shape: **HSS5x5x8**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N127**
J Joint: **F8_GRDR31**
Envelope (Ω)
Code Check: **0.351 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.351 (LC 20)**	Max Shear Check	**0.005 (y) (LC 22)**
Location	**10 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**243.236 k**	KL/r	**66.063**	**66.063**
phi*Pnt	**326.232 k**			
phi*Mny	**45.195 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**45.195 k-ft**	L-torque	**10 ft**	
phi*Vny	**83.28 k**	Tau_b	**1**	
phi*Vnz	**83.28 k**			
phi*Tn	**38.788 k-ft**			
Cb	**1.667**			

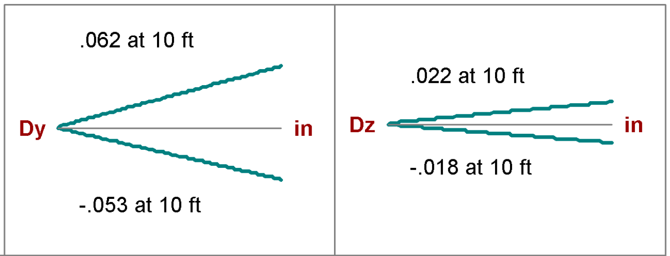
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

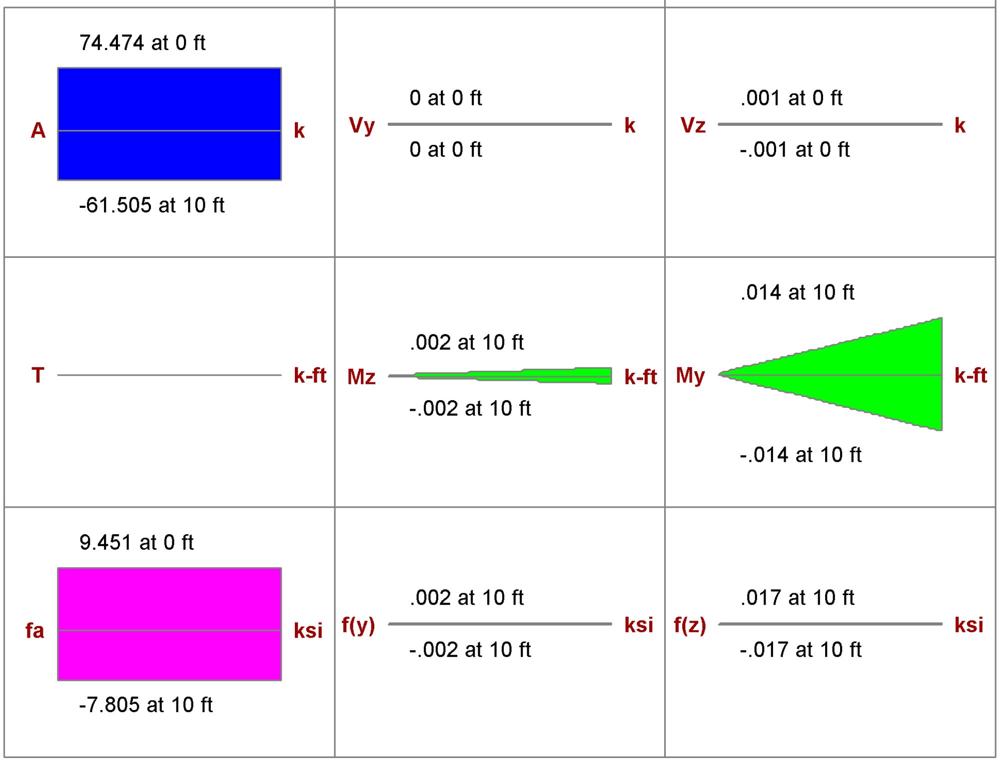
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **66.063** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***CS1***

Shape: **HSS5x5x8**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N110**
J Joint: **F8_N263**
Envelope (Ω)
Code Check: **0.306 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.306 (LC 18)**	Max Shear Check	**0.000 (z) (LC 18)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**243.236 k**	KL/r	**66.063**	**66.063**
phi*Pnt	**326.232 k**			
phi*Mny	**45.195 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**45.195 k-ft**	L-torque	**10 ft**	
phi*Vny	**83.28 k**	Tau_b	**1**	
phi*Vnz	**83.28 k**			
phi*Tn	**38.788 k-ft**			
Cb	**1.667**			

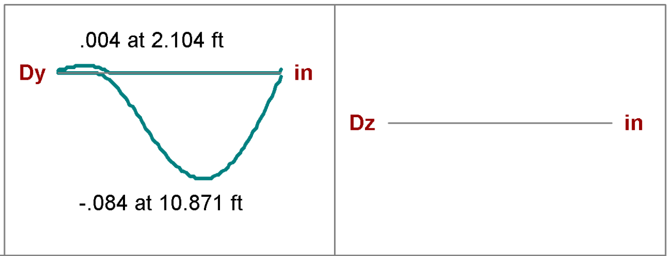
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

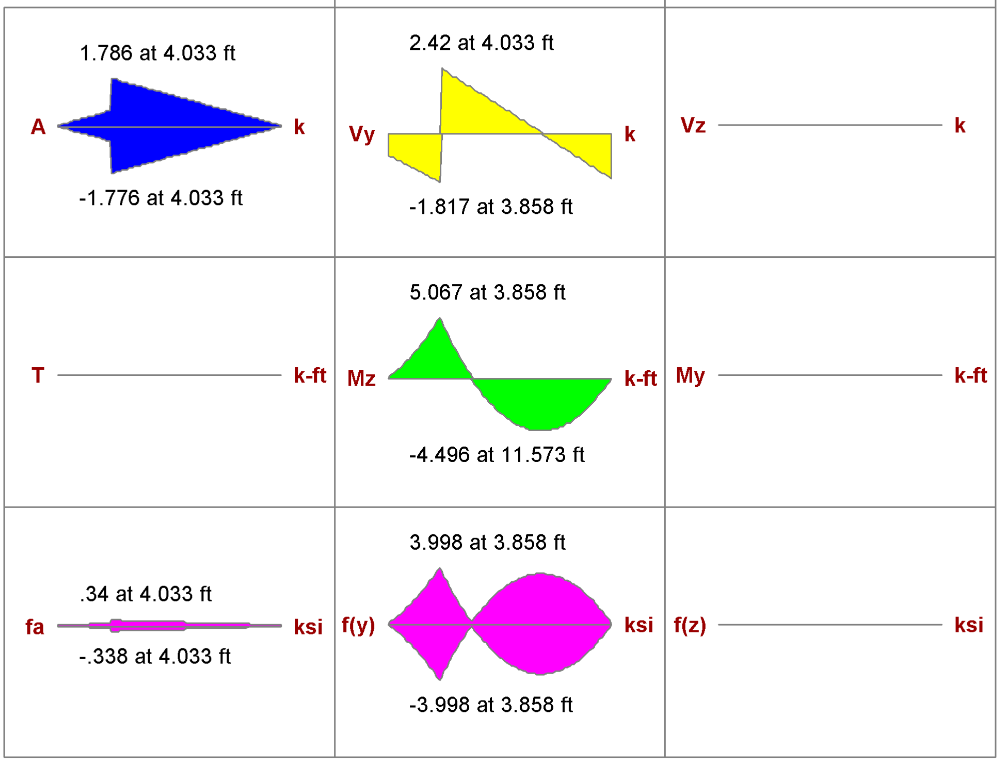
Member Type	**Column**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **66.063** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Beam: ***3B1***

Shape: **W8x18**
Material: **A992**
Length: **16.833 ft**
I Joint: **F2_N248**
J Joint: **F2_N247**
Envelope (Ω)
Code Check: **0.137 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.137 (LC 20)**	Max Shear Check	**0.043 (y) (LC 20)**	Max Defl Ratio	**L/2450**
Location	**4.033 ft**	Location	**4.033 ft**	Location	**10.871 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

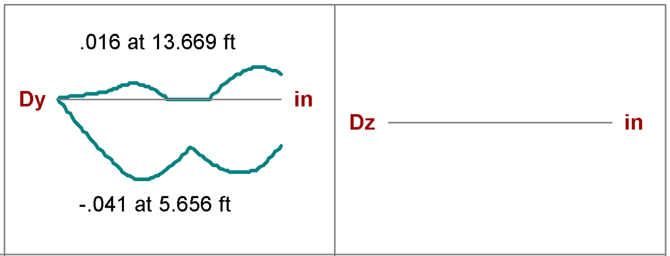
			y-y	z-z
Fy	**50 ksi**	Lb	**12.958 ft**	**16.833 ft**
phi*Pnc	**74.466 k**	KL/r	**126.323**	**58.883**
phi*Pnt	**236.7 k**			
phi*Mny	**17.475 k-ft**	L Comp Flange	**16.833 ft**	
phi*Mnz	**37.789 k-ft**	L-torque	**16.833 ft**	
phi*Vny	**56.166 k**	Tau_b	**1**	
phi*Vnz	**93.555 k**			
Cb	**1.272**			

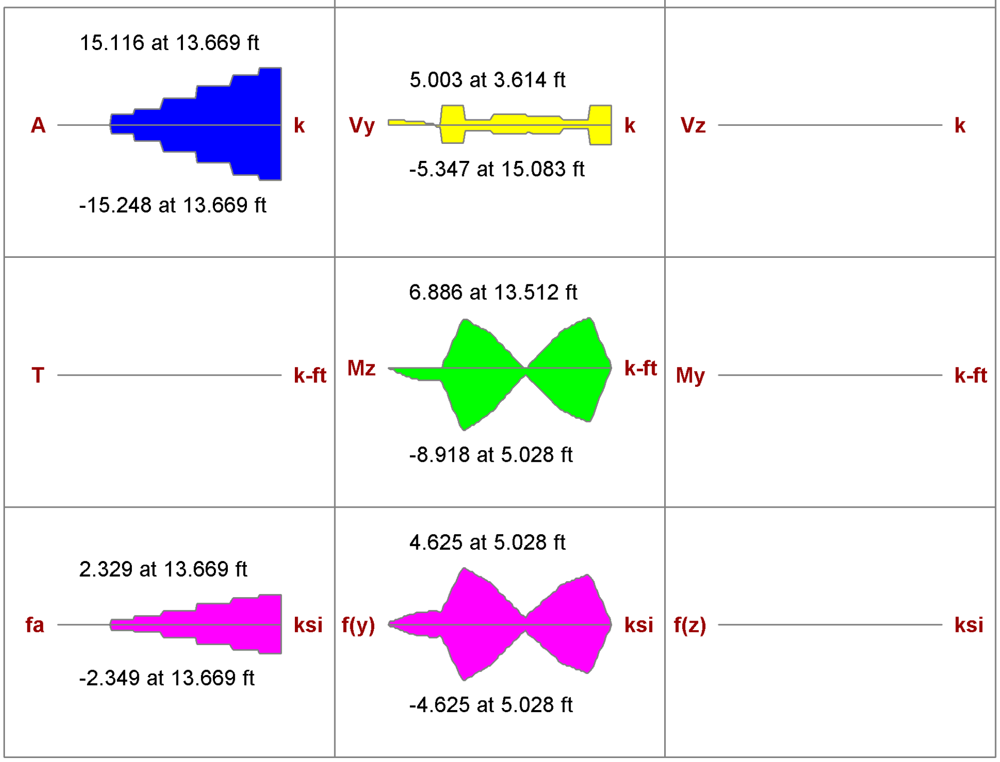
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***2B23***

Shape: **W10x22**
Material: **A992**
Length: **15.083 ft**
I Joint: **F4_N59**
J Joint: **N196**
Envelope (Ω)
Code Check: **0.134 (LC 19)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.134 (LC 19)**	Max Shear Check	**0.073 (y) (LC 19)**	Max Defl Ratio	**L/10000**
Location	**13.669 ft**	Location	**15.083 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**50 ksi**	Lb	**11.5 ft**	**15.083 ft**
phi*Pnc	**132.186 k**	KL/r	**104.124**	**42.447**
phi*Pnt	**292.05 k**			
phi*Mny	**22.875 k-ft**	L Comp Flange	**11.5 ft**	
phi*Mnz	**97.5 k-ft**	L-torque	**15.083 ft**	
phi*Vny	**73.44 k**	Tau_b	**1**	
phi*Vnz	**111.78 k**			
Cb	**1.739**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

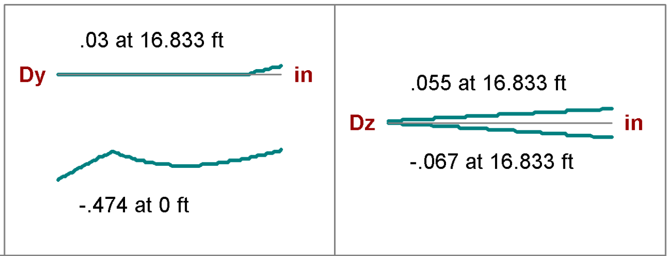
Beam: **1B8**

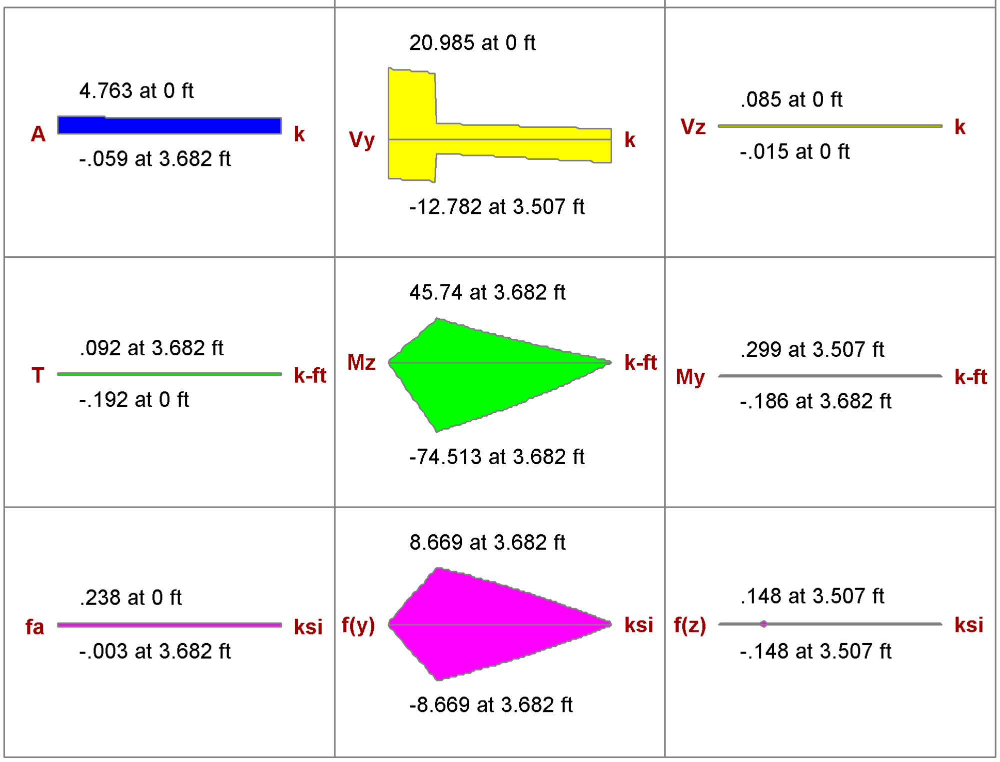
Shape:	**W14x68**
Material:	**A992**
Length:	**16.833 ft**
I Joint:	**N109**
J Joint:	**F8_N95**

Envelope (Ω)

Code Check: **0.176 (LC 20)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.176 (LC 20)**	Max Shear Check	**0.123 (y) (LC 20)**	Max Defl Ratio	**L/2191**
Location	**3.682 ft**	Location	**0 ft**	Location	**9.293 ft**
Equation	**H1-1b**			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

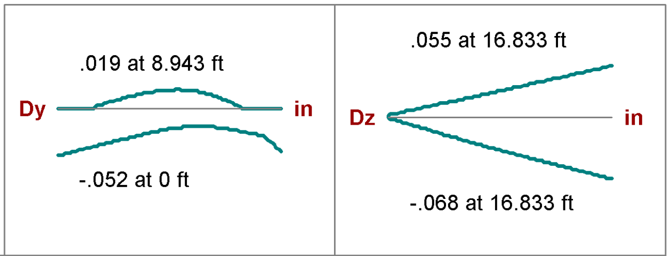
			y-y	z-z
Fy	**50 ksi**	Lb	**2.667 ft**	**16.833 ft**
phi*Pnc	**828.613 k**	KL/r	**13.011**	**33.619**
phi*Pnt	**900 k**			
phi*Mny	**138.375 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**431.25 k-ft**	L-torque	**16.833 ft**	
phi*Vny	**174.3 k**	Tau_b	**1**	
phi*Vnz	**388.8 k**			
Cb	**1**			

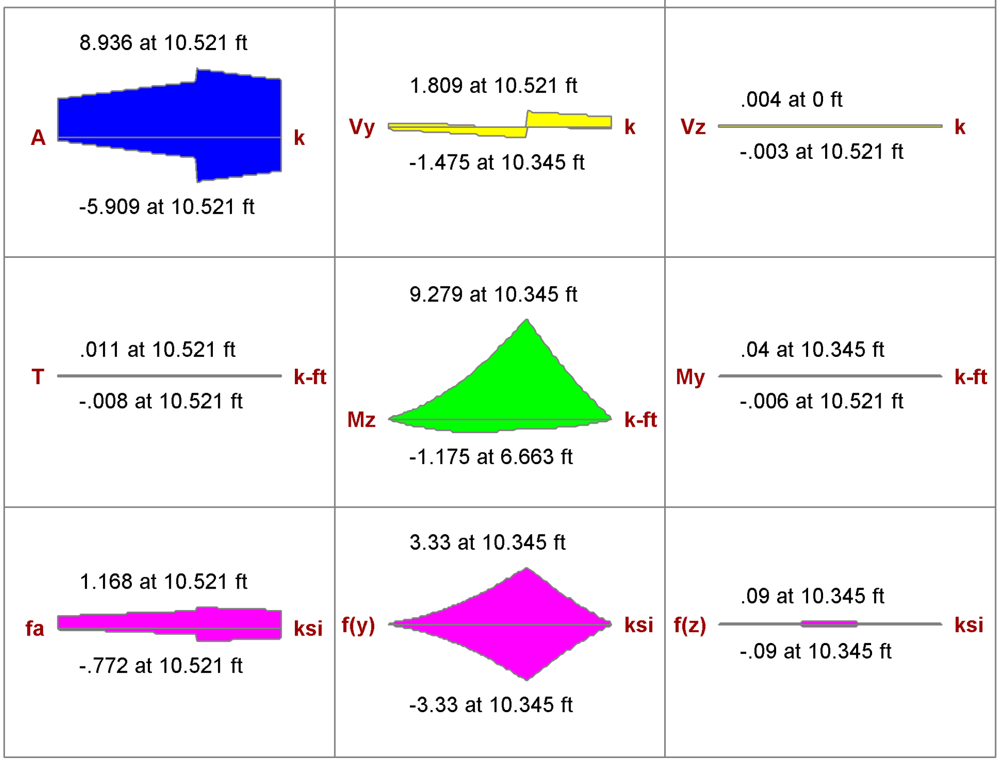
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: *1B7*

Shape: **W12x26**
Material: **A992**
Length: **16.833 ft**
I Joint: **F8_N89**
J Joint: **F8_N94**
Envelope (Ω)
Code Check: **0.121 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.121 (LC 20)**	Max Shear Check	**0.027 (y) (LC 20)**	Max Defl Ratio	**L/8598**
Location	**10.345 ft**	Location	**10.521 ft**	Location	**6.312 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

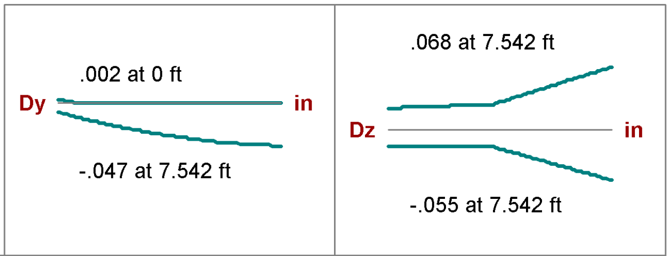
			y-y	z-z
Fy	**50 ksi**	Lb	**16.833 ft**	**16.833 ft**
phi*Pnc	**95.786 k**	KL/r	**134.323**	**39.116**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**16.833 ft**	
phi*Mnz	**113.528 k-ft**	L-torque	**16.833 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1.605**			

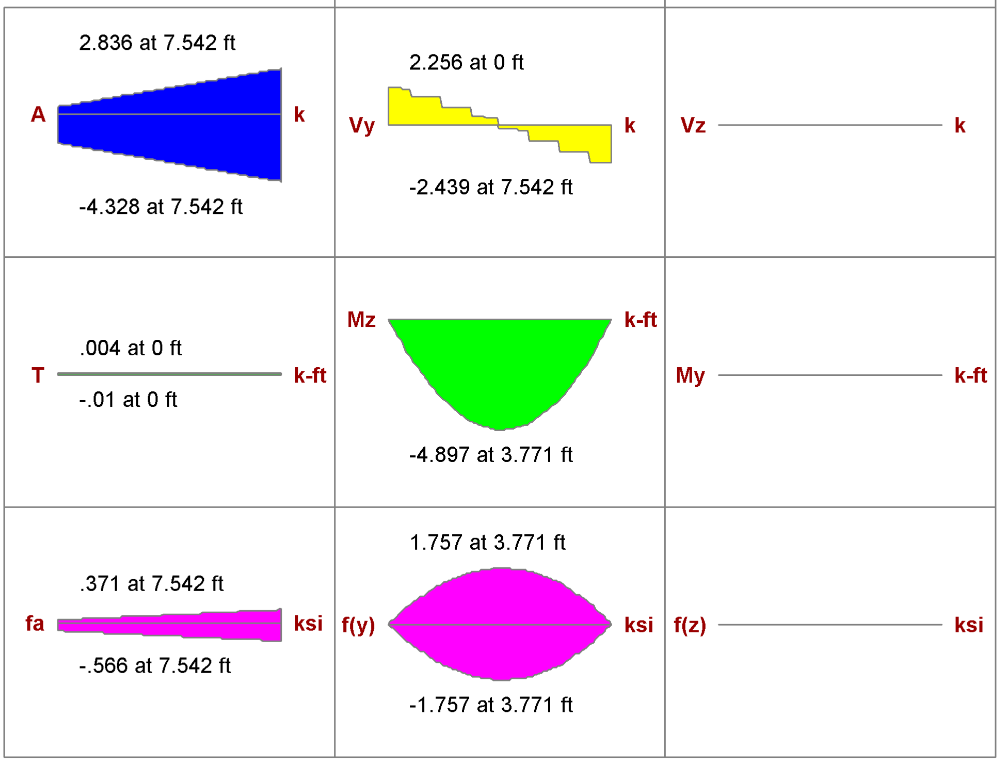
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***1B6***

Shape: **W12x26**
Material: **A992**
Length: **7.542 ft**
I Joint: **F8_N439**
J Joint: **F8_N94**
Envelope (Ω)
Code Check: **0.040 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.040 (LC 20)**	Max Shear Check	**0.034 (y) (LC 20)**	Max Defl Ratio	**L/8655**
Location	**3.771 ft**	Location	**7.542 ft**	Location	**3.771 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.926**

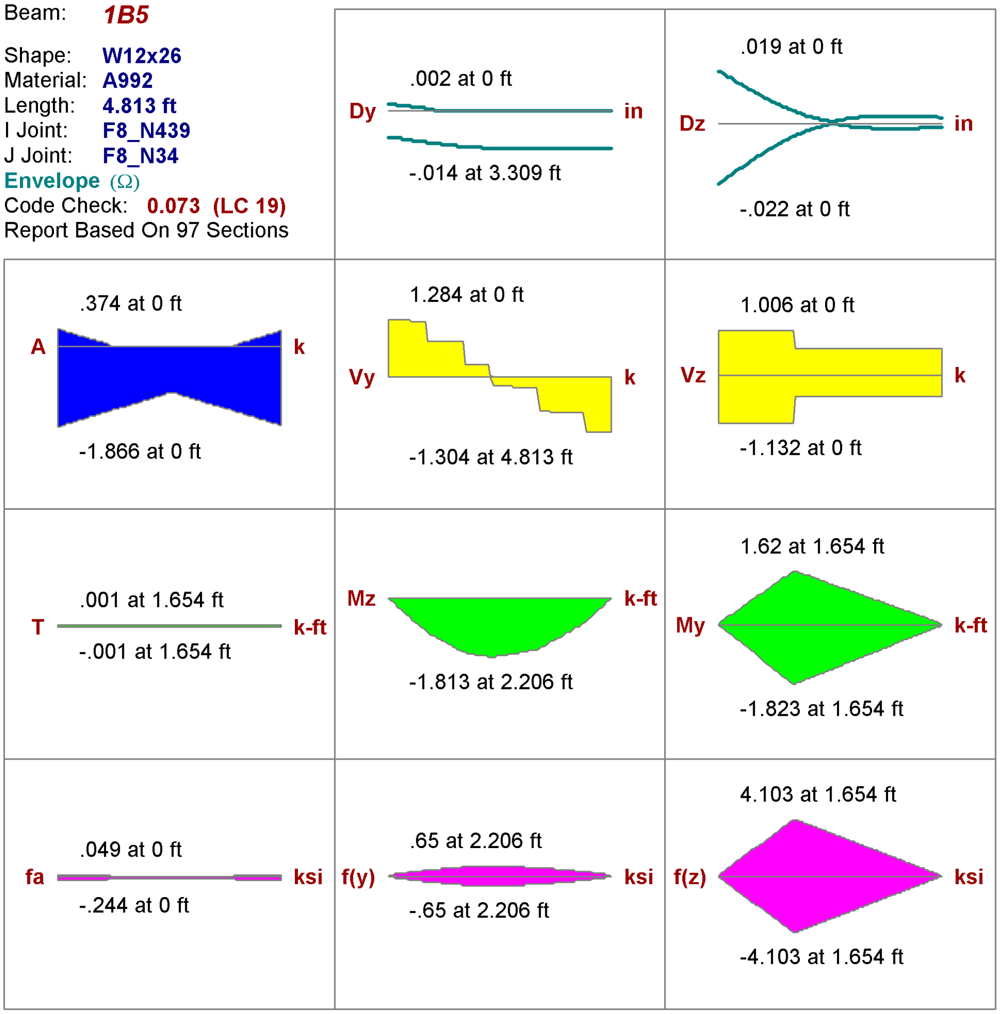
			y-y	z-z
Fy	**50 ksi**	Lb	**1 ft**	**7.542 ft**
phi*Pnc	**312.075 k**	KL/r	**7.98**	**17.526**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**7.542 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***1B5***

Shape: **W12x26**
Material: **A992**
Length: **4.813 ft**
I Joint: **F8_N439**
J Joint: **F8_N34**
Envelope (Ω)
Code Check: **0.073 (LC 19)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.073 (LC 19)**	Max Shear Check	**0.016 (y) (LC 19)**	Max Defl Ratio	**L/10000**
Location	**1.654 ft**	Location	**4.813 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.924**

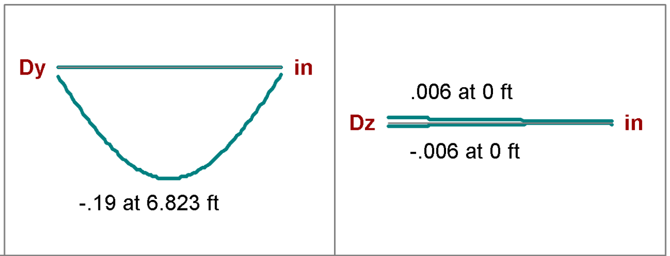
			y-y	z-z
Fy	**50 ksi**	Lb	**.594 ft**	**4.813 ft**
phi*Pnc	**315.459 k**	KL/r	**4.74**	**11.184**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**4.813 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

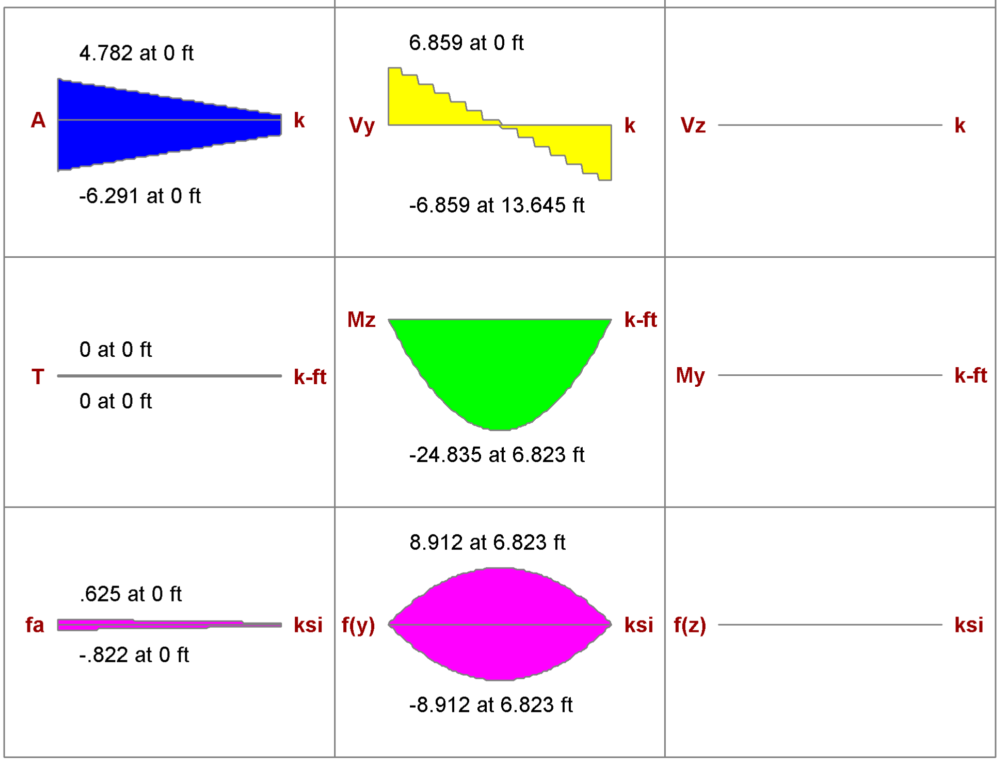
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: *1B4*

Shape: **W12x26**
Material: **A992**
Length: **13.645 ft**
I Joint: **F8_N27**
J Joint: **F8_N34**
Envelope (Ω)
Code Check: **0.184 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.184 (LC 18)**	Max Shear Check	**0.082 (y) (LC 19)**	Max Defl Ratio	**L/935**
Location	**6.823 ft**	Location	**0 ft**	Location	**6.823 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.931**

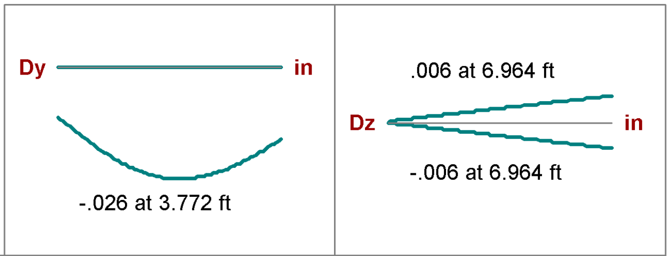
			y-y	z-z
Fy	**50 ksi**	Lb	**1 ft**	**13.645 ft**
phi*Pnc	**299.331 k**	KL/r	**7.98**	**31.708**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**13.645 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

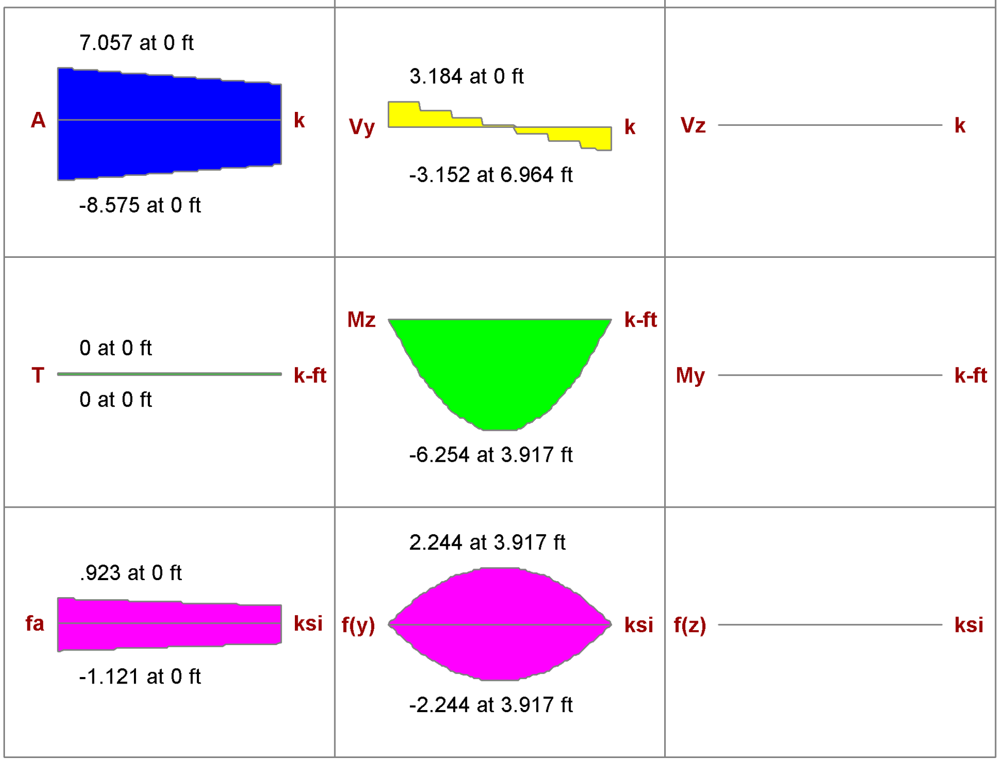
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***1B3***

Shape: **W12x26**
Material: **A992**
Length: **6.964 ft**
I Joint: **F8_N26**
J Joint: **F8_N27**
Envelope (Ω)
Code Check: **0.056 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.056 (LC 18)**	Max Shear Check	**0.038 (y) (LC 20)**	Max Defl Ratio	**L/7218**
Location	**2.974 ft**	Location	**0 ft**	Location	**3.482 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.925**

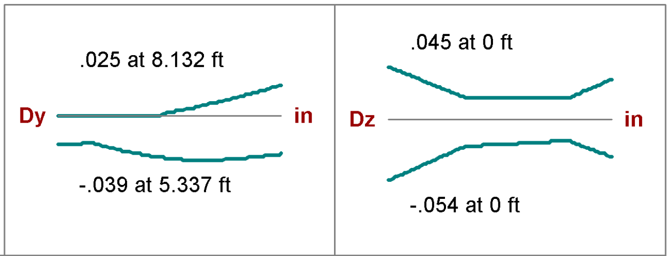
			y-y	z-z
Fy	**50 ksi**	Lb	**1 ft**	**6.964 ft**
phi*Pnc	**312.914 k**	KL/r	**7.98**	**16.183**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**6.964 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

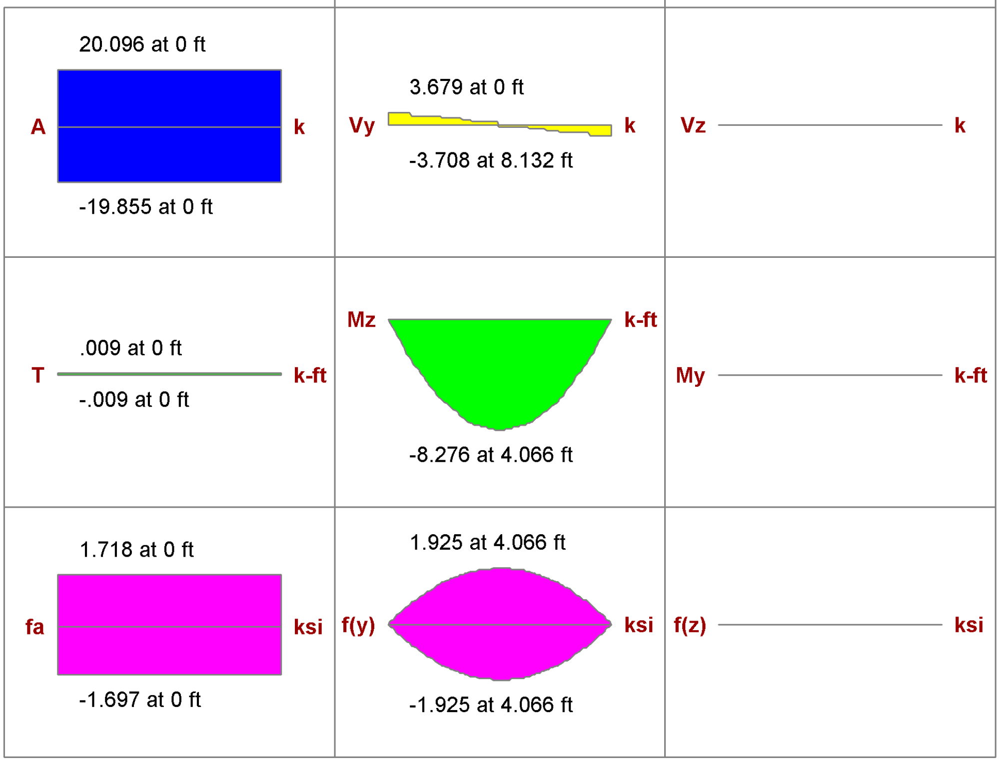
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: **1B24**

Shape: **W12x40**
Material: **A992**
Length: **8.132 ft**
I Joint: **N111**
J Joint: **F8_N263**
Envelope (Ω)
Code Check: **0.058 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.058 (LC 18)**	Max Shear Check	**0.037 (y) (LC 20)**	Max Defl Ratio	**L/7138**
Location	**4.066 ft**	Location	**8.132 ft**	Location	**4.066 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.996**

			y-y	z-z
Fy	**50 ksi**	Lb	**1.078 ft**	**8.132 ft**
phi*Pnc	**510.7 k**	KL/r	**6.665**	**19.051**
phi*Pnt	**526.5 k**			
phi*Mny	**63 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**213.75 k-ft**	L-torque	**8.132 ft**	
phi*Vny	**105.315 k**	Tau_b	**1**	
phi*Vnz	**222.758 k**			
Cb	**1**			

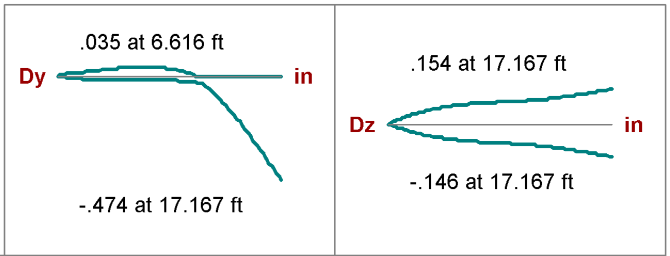
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

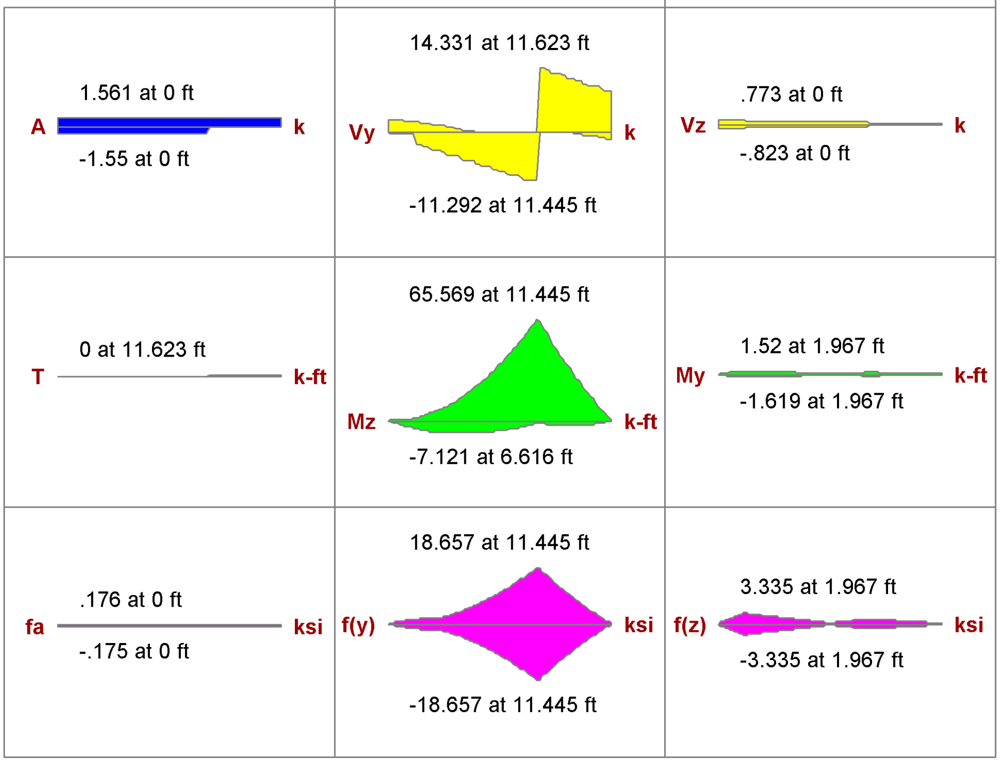
Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: **1B20**

Shape:	W14x30
Material:	A992
Length:	17.167 ft
I Joint:	F8_N88
J Joint:	N109

Envelope (Ω)
Code Check: **0.455 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	0.455 (LC 18)	Max Shear Check	0.128 (y) (LC 18)	Max Defl Ratio	L/2122
Location	11.445 ft	Location	11.623 ft	Location	7.511 ft
Equation	H1-1b			Span	1

Bending Flange	Compact	Compression Flange	Non-Slender	Qs=1
Bending Web	Compact	Compression Web	Slender	Qa=.931

			y-y	z-z
Fy	50 ksi	Lb	1.085 ft	17.167 ft
phi*Pnc	339.501 k	KL/r	8.745	35.925
phi*Pnt	398.25 k			
phi*Mny	33.713 k-ft	L Comp Flange	17.167 ft	
phi*Mnz	151.779 k-ft	L-torque	17.167 ft	
phi*Vny	111.78 k	Tau_b	1	
phi*Vnz	139.917 k			
Cb	1.789			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	Beam	Design to Ω_0 Loads	Yes
Seismic Design Rule	OCBF	Frame Ductility Req'd	Minimal
Moment Conn Type	Other/None		
Flange	Compact	Web	Compact

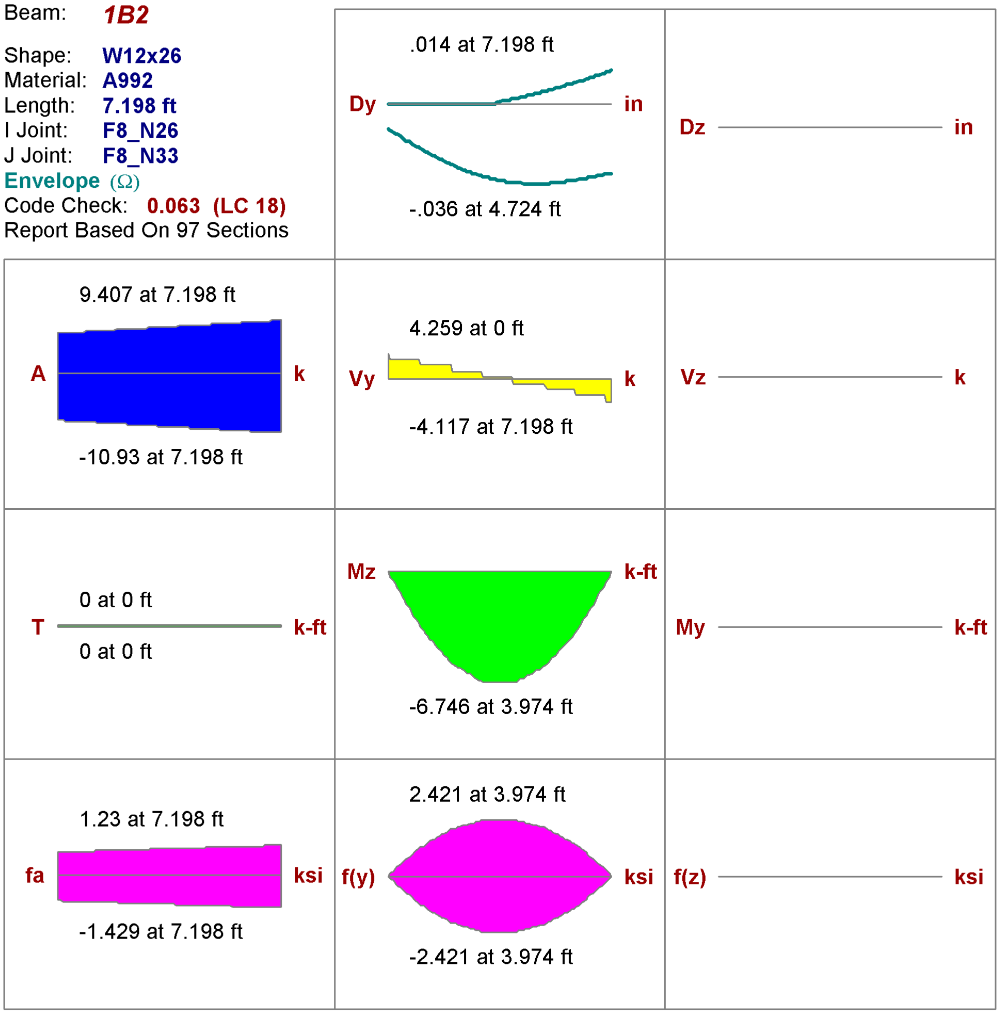
Beam: *1B2*

Shape:	**W12x26**
Material:	**A992**
Length:	**7.198 ft**
I Joint:	**F8_N26**
J Joint:	**F8_N33**

Envelope (Ω)

Code Check: **0.063 (LC 18)**

Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.063 (LC 18)**	Max Shear Check	**0.051 (y) (LC 12)**	Max Defl Ratio	**L/6490**
Location	**3.974 ft**	Location	**0 ft**	Location	**3.599 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.925**

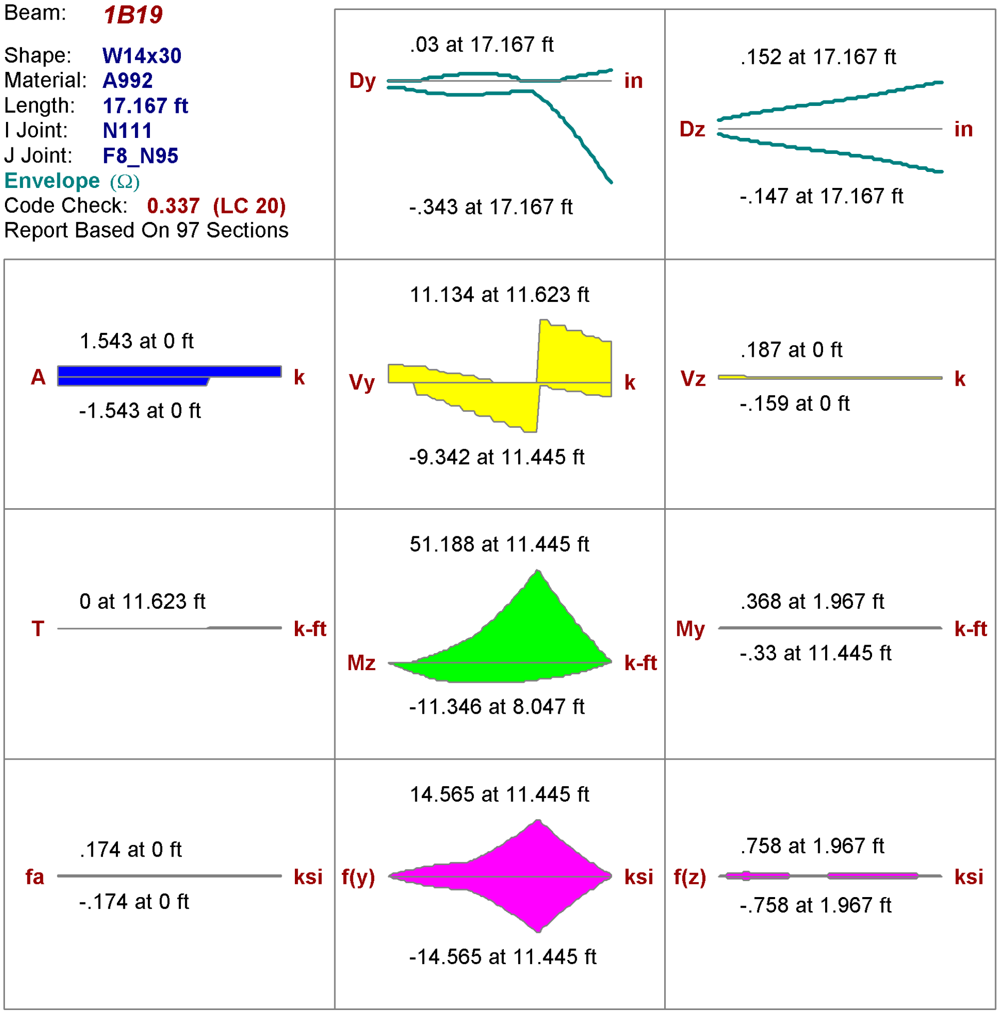
			y-y	z-z
Fy	**50 ksi**	Lb	**1 ft**	**7.198 ft**
phi*Pnc	**312.582 k**	KL/r	**7.98**	**16.727**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**7.198 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***1B19***

Shape: **W14x30**
Material: **A992**
Length: **17.167 ft**
I Joint: **N111**
J Joint: **F8_N95**
Envelope (Ω)
Code Check: **0.337 (LC 20)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.337 (LC 20)**	Max Shear Check	**0.100 (y) (LC 20)**	Max Defl Ratio	**L/455**
Location	**11.445 ft**	Location	**11.623 ft**	Location	**17.167 ft**
Equation	**H1-1b**			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.931**

			y-y	z-z
Fy	**50 ksi**	Lb	**1.188 ft**	**17.167 ft**
phi*Pnc	**339.501 k**	KL/r	**9.575**	**35.925**
phi*Pnt	**398.25 k**			
phi*Mny	**33.713 k-ft**	L Comp Flange	**17.167 ft**	
phi*Mnz	**155.991 k-ft**	L-torque	**17.167 ft**	
phi*Vny	**111.78 k**	Tau_b	**1**	
phi*Vnz	**139.917 k**			
Cb	**1.868**			

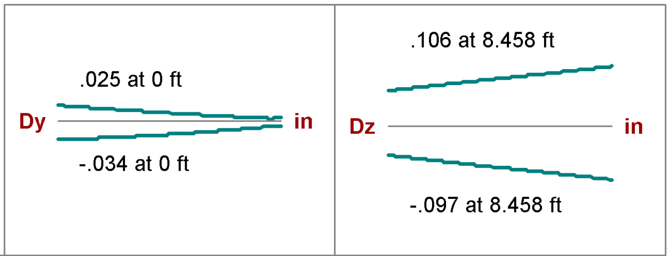
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

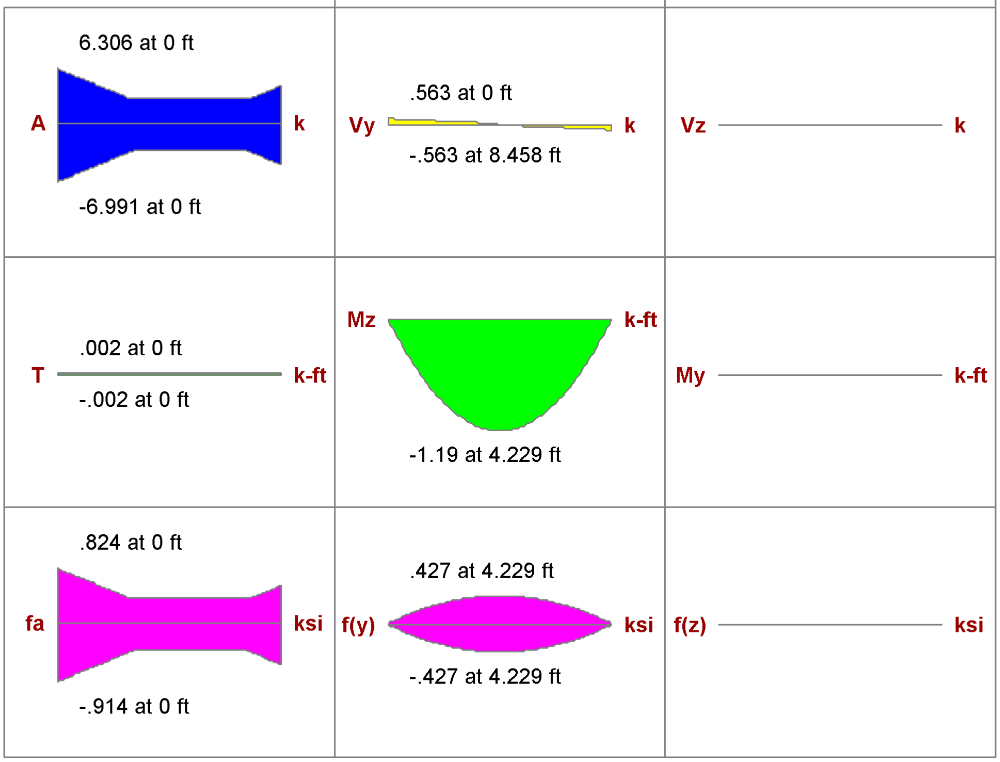
Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Miscellaneous Seismic Checks/Warnings:

Beam: ***1B18***

Shape: **W12x26**
Material: **A992**
Length: **8.458 ft**
I Joint: **F8_N263**
J Joint: **F8_N439**
Envelope (Ω)
Code Check: **0.026 (LC 25)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.026 (LC 25)**	Max Shear Check	**0.007 (y) (LC 20)**	Max Defl Ratio	**L/10000**
Location	**0 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.96**

			y-y	z-z
Fy	**50 ksi**	Lb	**8.458 ft**	**8.458 ft**
phi*Pnc	**240.089 k**	KL/r	**67.493**	**19.655**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**8.458 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

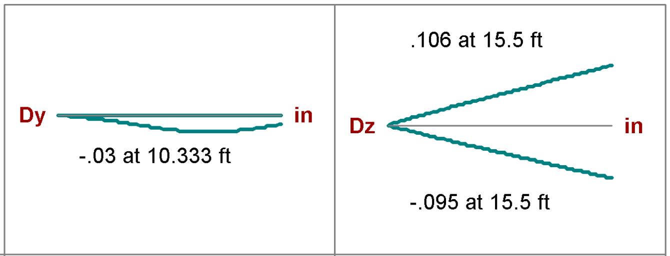
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

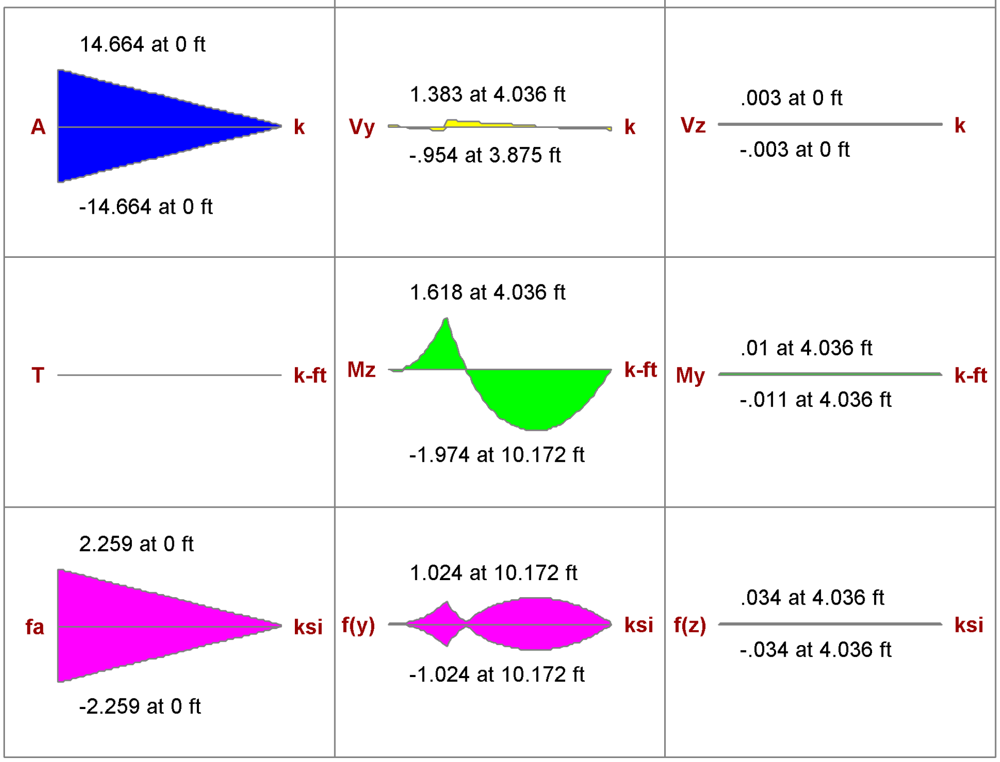
Member Type	**Beam**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***1B14***

Shape:	**W10x22**
Material:	**A992**
Length:	**15.5 ft**
I Joint:	**F8_N22**
J Joint:	**F8_N27**

Envelope (Ω)
Code Check: **0.068 (LC 21)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.068 (LC 21)**	Max Shear Check	**0.019 (y) (LC 10)**	Max Defl Ratio	**L/8589**
Location	**0 ft**	Location	**4.036 ft**	Location	**10.01 ft**
Equation	**H1-1b***			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**50 ksi**	Lb	**7.042 ft**	**15.5 ft**
phi*Pnc	**216.953 k**	KL/r	**63.76**	**43.621**
phi*Pnt	**292.05 k**			
phi*Mny	**22.875 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**97.5 k-ft**	L-torque	**15.5 ft**	
phi*Vny	**73.44 k**	Tau_b	**1**	
phi*Vnz	**111.78 k**			
Cb	**1**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

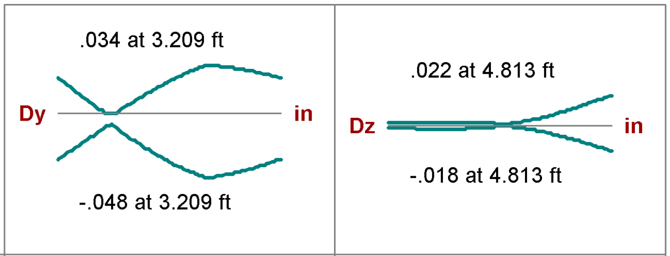
Beam: ***1B11***

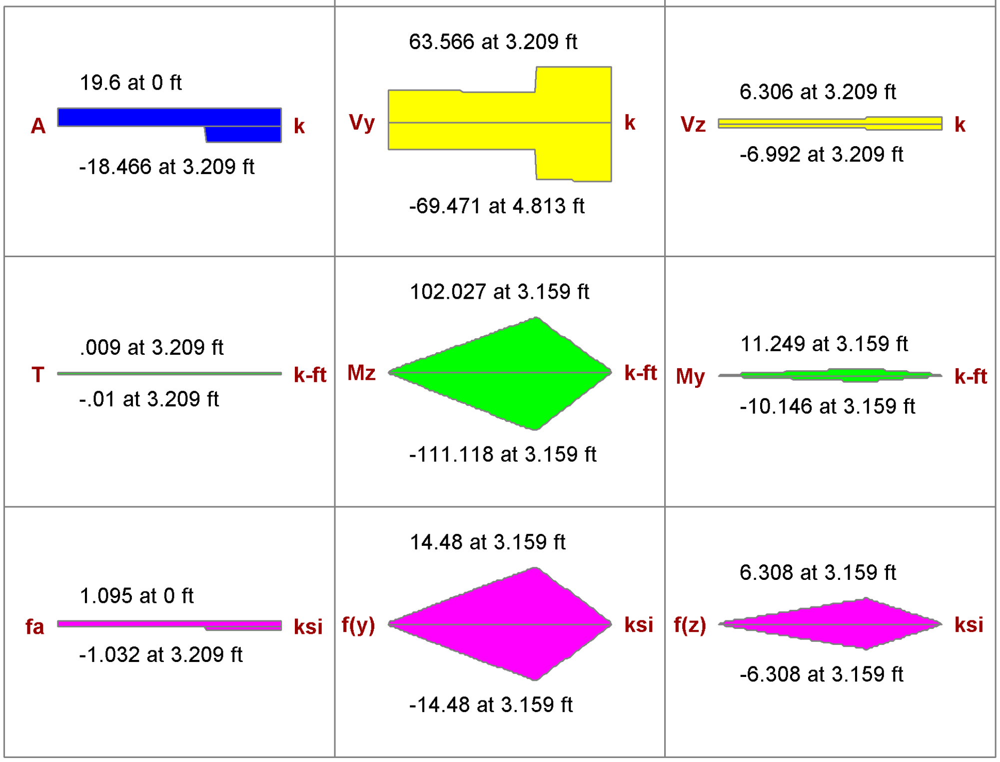
Shape:	**W14x61**
Material:	**A992**
Length:	**4.813 ft**
I Joint:	**F8_GRDR31**
J Joint:	**F8_N263**

Envelope (Ω)

Code Check: **0.445 (LC 18)**

Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.371 (LC 18)**	Max Shear Check	**0.445 (y) (LC 18)**	Max Defl Ratio	**L/6964**
Location	**3.159 ft**	Location	**4.813 ft**	Location	**1.805 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

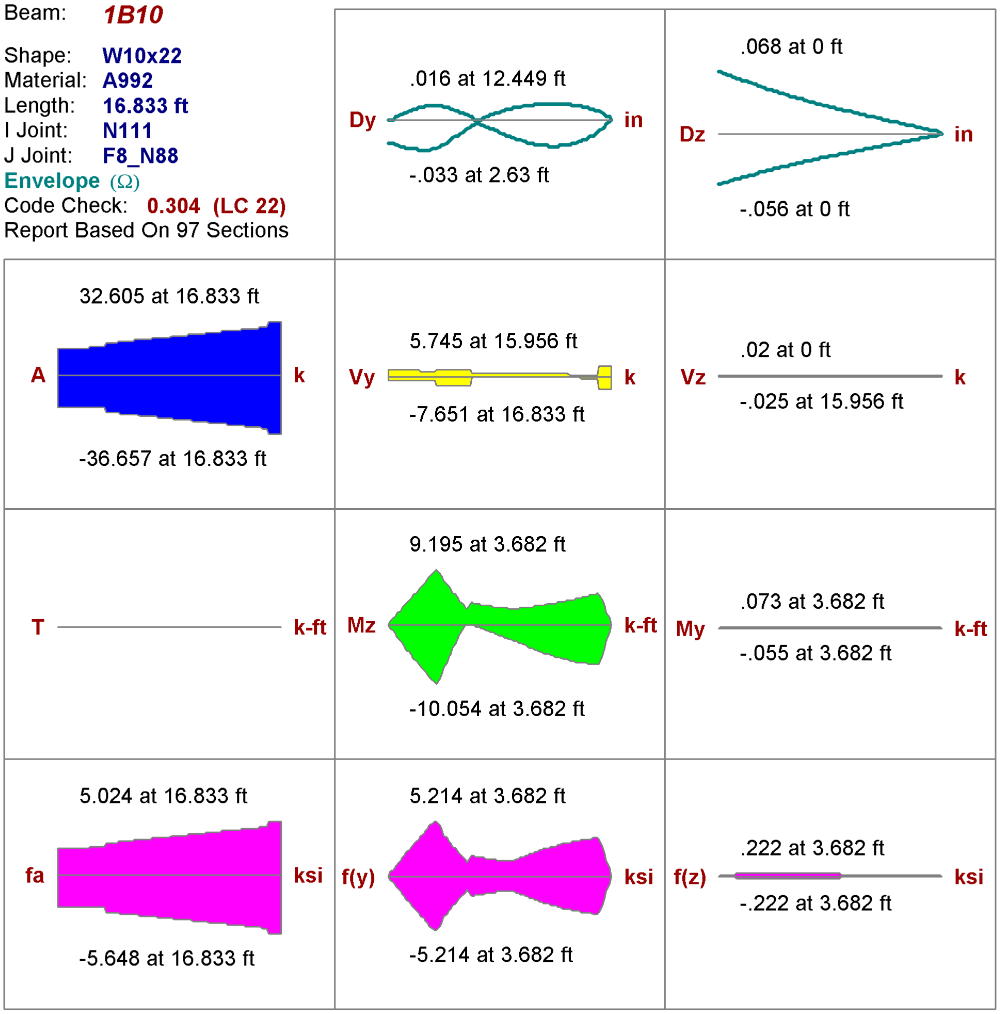
			y-y	z-z
Fy	**50 ksi**	Lb	**.594 ft**	**4.813 ft**
phi*Pnc	**800.024 k**	KL/r	**2.915**	**9.659**
phi*Pnt	**805.5 k**			
phi*Mny	**123 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**382.5 k-ft**	L-torque	**4.813 ft**	
phi*Vny	**156.375 k**	Tau_b	**1**	
phi*Vnz	**348.3 k**			
Cb	**1**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: ***1B10***

Shape: **W10x22**
Material: **A992**
Length: **16.833 ft**
I Joint: **N111**
J Joint: **F8_N88**
Envelope (Ω)
Code Check: **0.304 (LC 22)**
Report Based On 97 Sections



AISC 14th(360-10): LRFD Code Check

Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.304 (LC 22)**	Max Shear Check	**0.104 (y) (LC 18)**	Max Defl Ratio	**L/4359**
Location	**15.781 ft**	Location	**16.833 ft**	Location	**3.507 ft**
Equation	**H1-1a**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

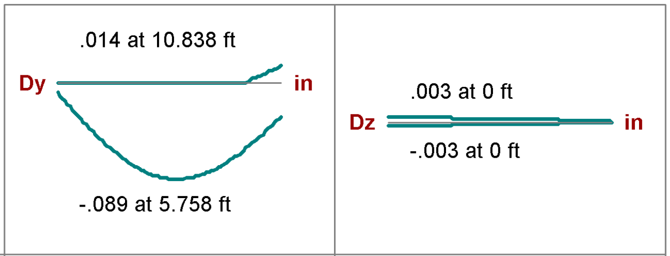
			y-y	z-z
Fy	**50 ksi**	Lb	**12.292 ft**	**16.833 ft**
phi*Pnc	**118.068 k**	KL/r	**111.295**	**47.372**
phi*Pnt	**292.05 k**			
phi*Mny	**22.875 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**97.5 k-ft**	L-torque	**16.833 ft**	
phi*Vny	**73.44 k**	Tau_b	**1**	
phi*Vnz	**111.78 k**			
Cb	**1**			

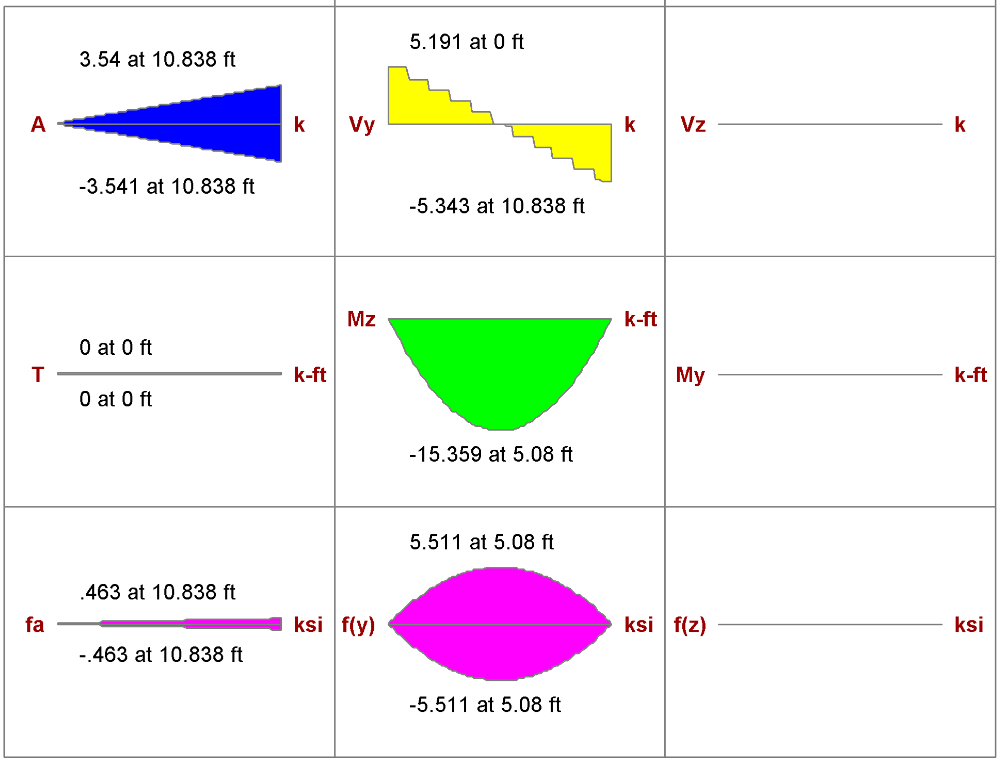
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Beam: **1B1**

Shape: **W12x26**
Material: **A992**
Length: **10.838 ft**
I Joint: **F8_N25**
J Joint: **F8_N33**
Envelope (Ω)
Code Check: **0.113 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.113 (LC 18)**	Max Shear Check	**0.064 (y) (LC 20)**	Max Defl Ratio	**L/1894**
Location	**5.419 ft**	Location	**10.838 ft**	Location	**5.419 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.928**

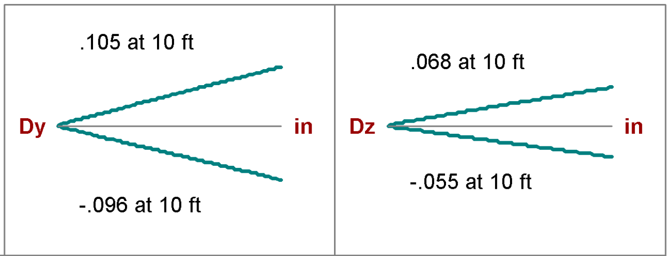
			y-y	z-z
Fy	**50 ksi**	Lb	**1 ft**	**10.838 ft**
phi*Pnc	**306.059 k**	KL/r	**7.98**	**25.185**
phi*Pnt	**344.25 k**			
phi*Mny	**30.637 k-ft**	L Comp Flange	**.5 ft**	
phi*Mnz	**139.5 k-ft**	L-torque	**10.838 ft**	
phi*Vny	**84.18 k**	Tau_b	**1**	
phi*Vnz	**133.175 k**			
Cb	**1**			

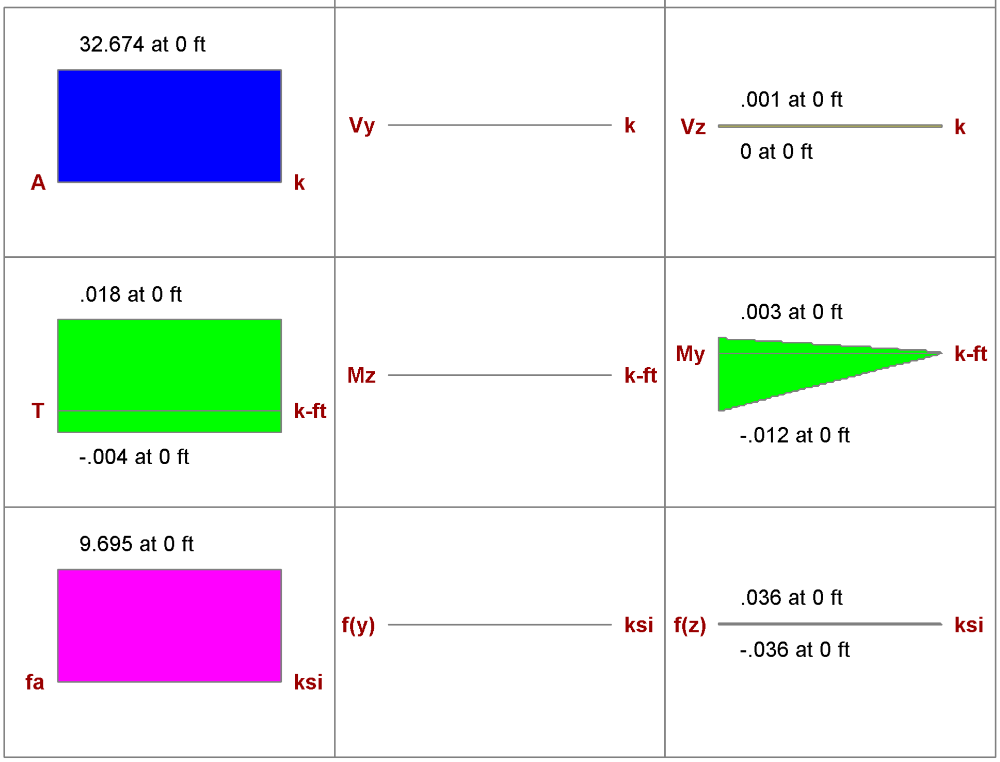
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Beam**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

Column: ***(F-5)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N125**
J Joint: **F8_N94**
Envelope (Ω)
Code Check: **0.357 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

Max Bending Check	**0.357 (LC 20)**	Max Shear Check	**0.001 (z) (LC 20)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

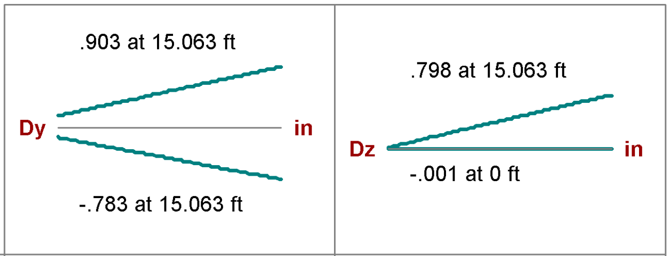
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

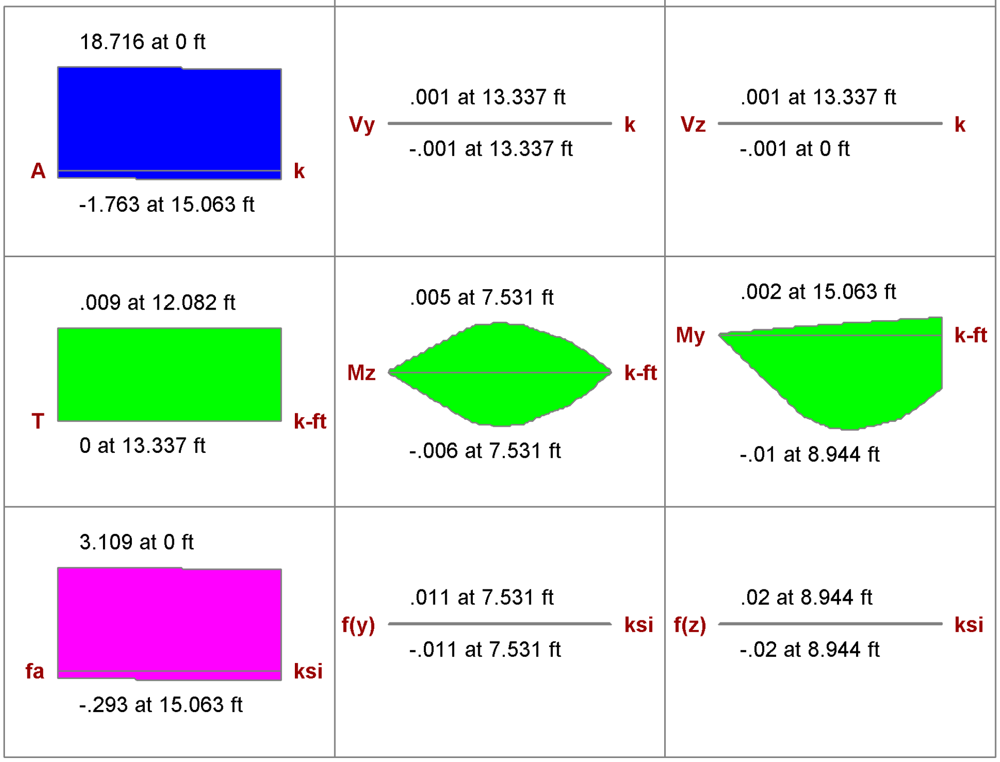
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **78.877** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(6-F')***

Shape: **HSS4x4x8**
Material: **A500 Gr.B Rect**
Length: **15.063 ft**
I Joint: **N109**
J Joint: **N4**
Envelope (Ω)
Code Check: **0.227 (LC 22)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.227 (LC 22)**	Max Shear Check	**0.000 (z) (LC 18)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**15.063 ft**	**15.063 ft**
phi*Pnc	**82.281 k**	KL/r	**128.563**	**128.563**
phi*Pnt	**249.228 k**			
phi*Mny	**26.565 k-ft**	L Comp Flange	**15.063 ft**	
phi*Mnz	**26.565 k-ft**	L-torque	**15.063 ft**	
phi*Vny	**60.179 k**	Tau_b	**1**	
phi*Vnz	**60.179 k**			
phi*Tn	**23.253 k-ft**			
Cb	**1.667**			

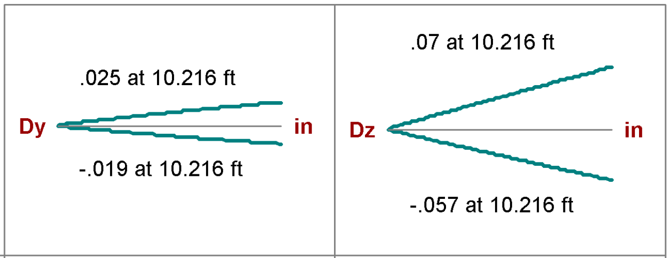
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

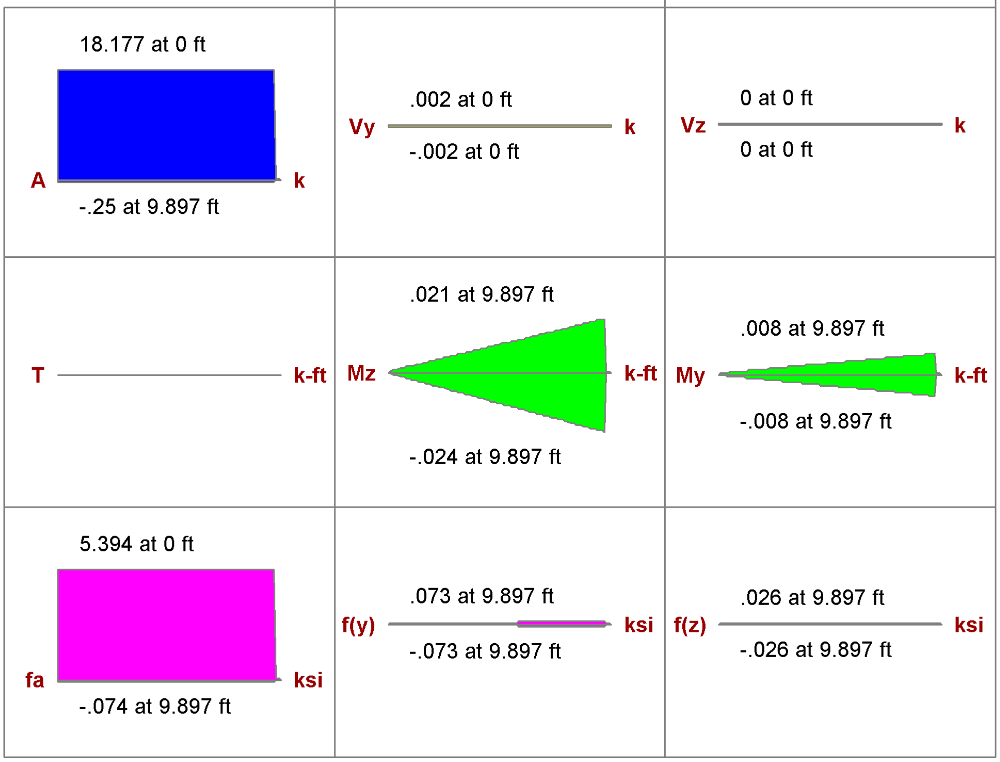
Member Type	**Column**	Design to Ω_0 Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **128.563 > 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(5-E)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10.216 ft**
I Joint: **N118**
J Joint: **N119**
Envelope (Ω)
Code Check: **0.198 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.198 (LC 20)**	Max Shear Check	**0.000 (y) (LC 22)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.679**			

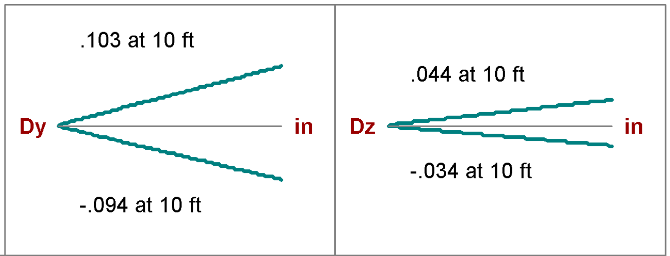
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

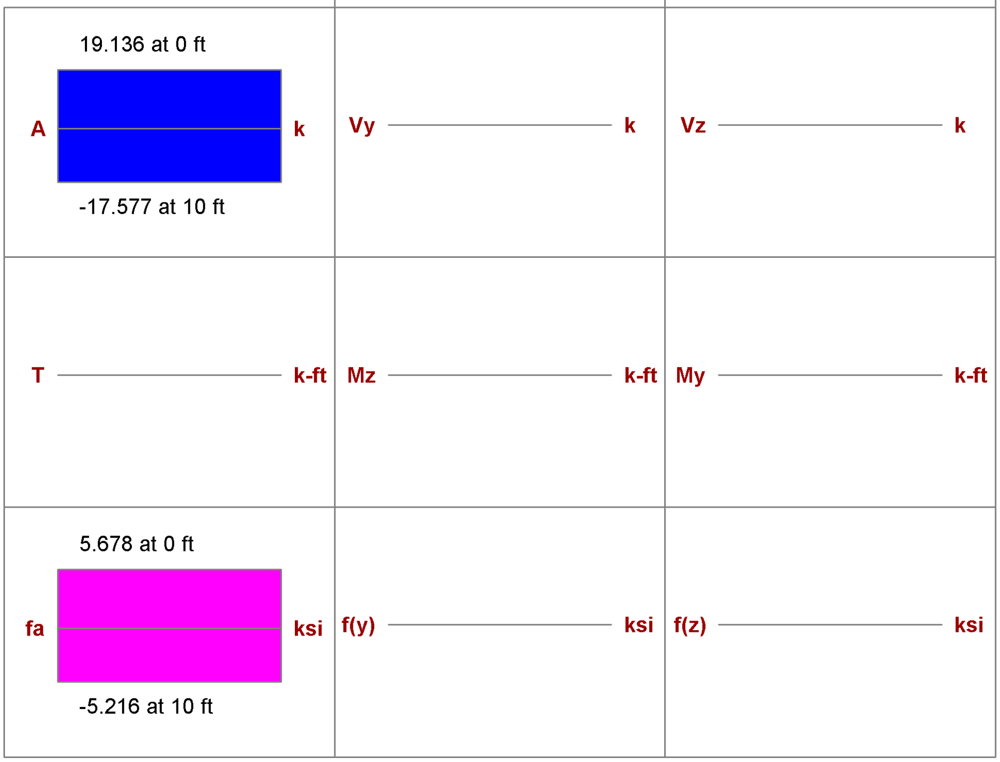
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **78.877** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(5+-F)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N124**
J Joint: **F8_N35**
Envelope (Ω)
Code Check: **0.208 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.208 (LC 18)**	Max Shear Check	**0.000 (y) (LC 9)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

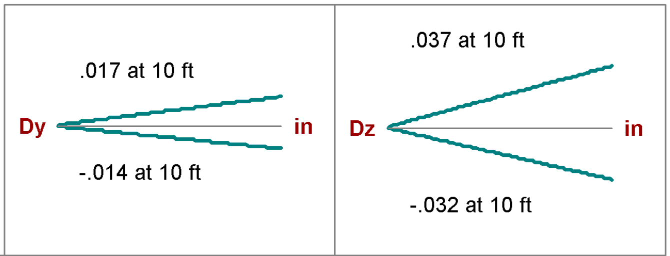
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

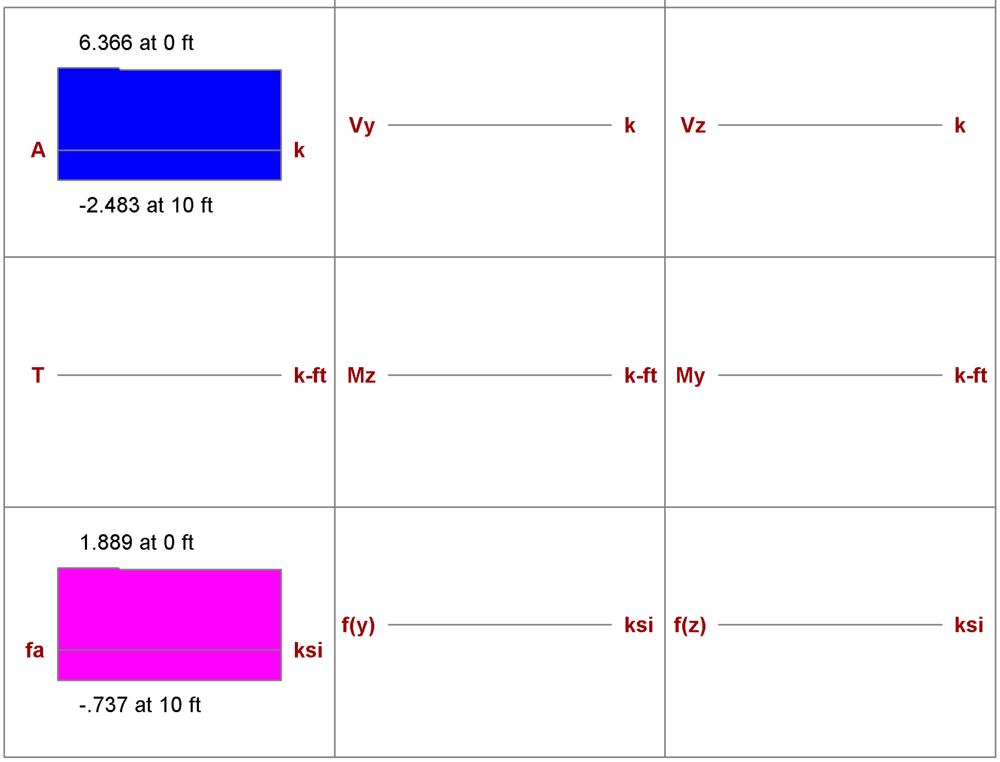
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **78.877** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(5+-E)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N128**
J Joint: **F8_N32**
Envelope (Ω)
Code Check: **0.069 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.069 (LC 20)**	Max Shear Check	**0.000 (y) (LC 9)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

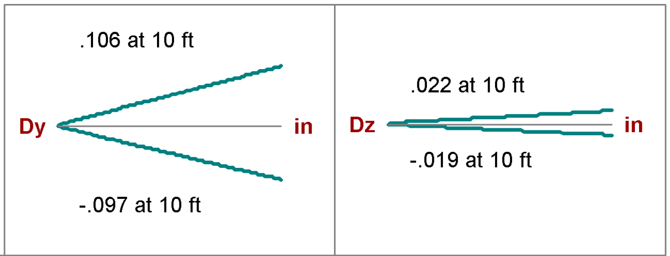
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

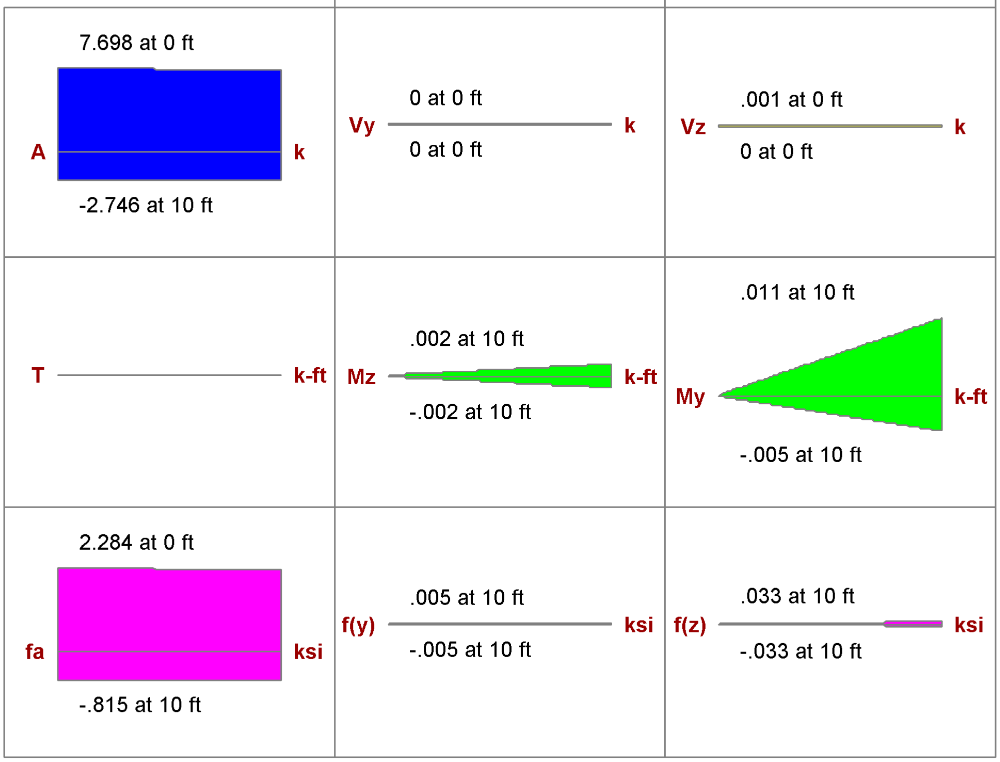
L/r = **78.877** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(4+-F)***

Shape:	**HSS4x4x4**
Material:	**A500 Gr.B Rect**
Length:	**10 ft**
I Joint:	**N126**
J Joint:	**F8_N439**

Envelope (Ω)
Code Check: **0.084 (LC 19)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.084 (LC 19)**	Max Shear Check	**0.000 (z) (LC 20)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1.667**			

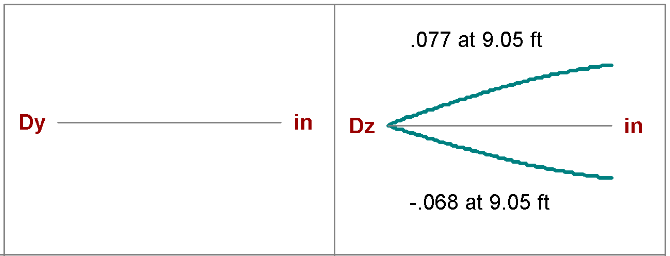
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

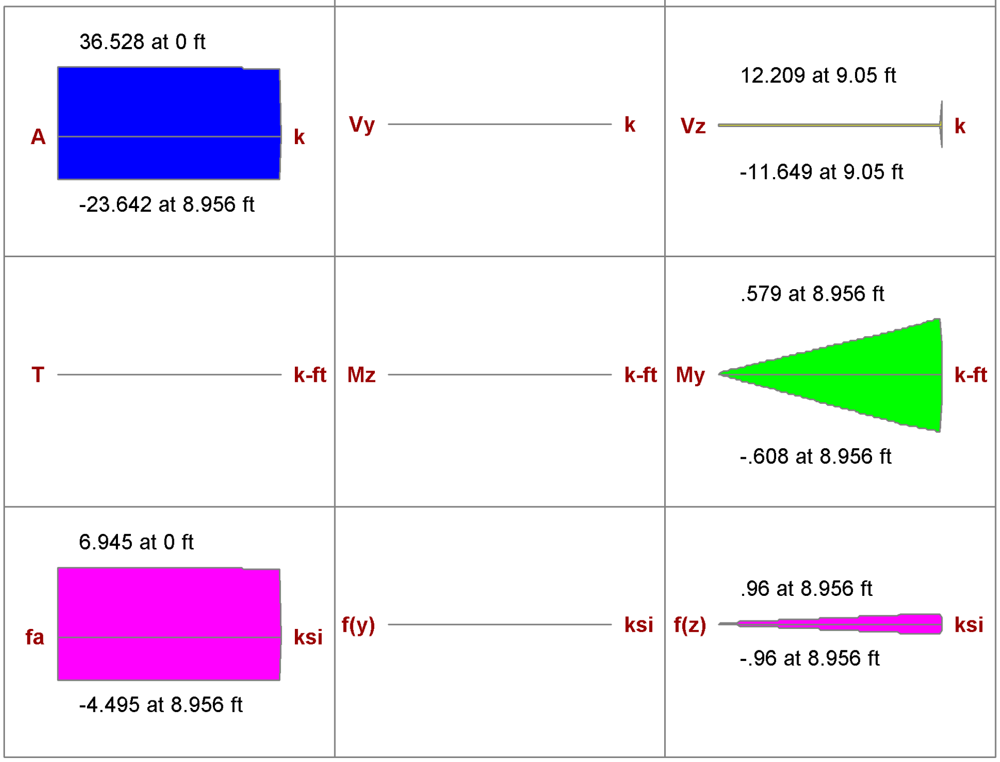
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **78.877** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(3-B)***

Shape: **HSS5x5x5**
Material: **A500 Gr.B Rect**
Length: **9.05 ft**
I Joint: **N214**
J Joint: **N215**
Envelope (Ω)
Code Check: **0.260 (LC 19)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.260 (LC 19)**	Max Shear Check	**0.205 (z) (LC 19)**
Location	**7.447 ft**	Location	**9.05 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**1 ft**	**12 ft**
phi*Pnc	**148.006 k**	KL/r	**6.314**	**75.767**
phi*Pnt	**217.764 k**			
phi*Mny	**31.602 k-ft**	L Comp Flange	**12 ft**	
phi*Mnz	**31.602 k-ft**	L-torque	**12 ft**	
phi*Vny	**59.664 k**	Tau_b	**1**	
phi*Vnz	**59.664 k**			
phi*Tn	**26.518 k-ft**			
Cb	**1**			

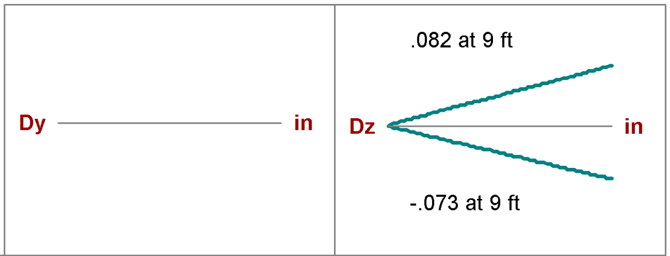
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

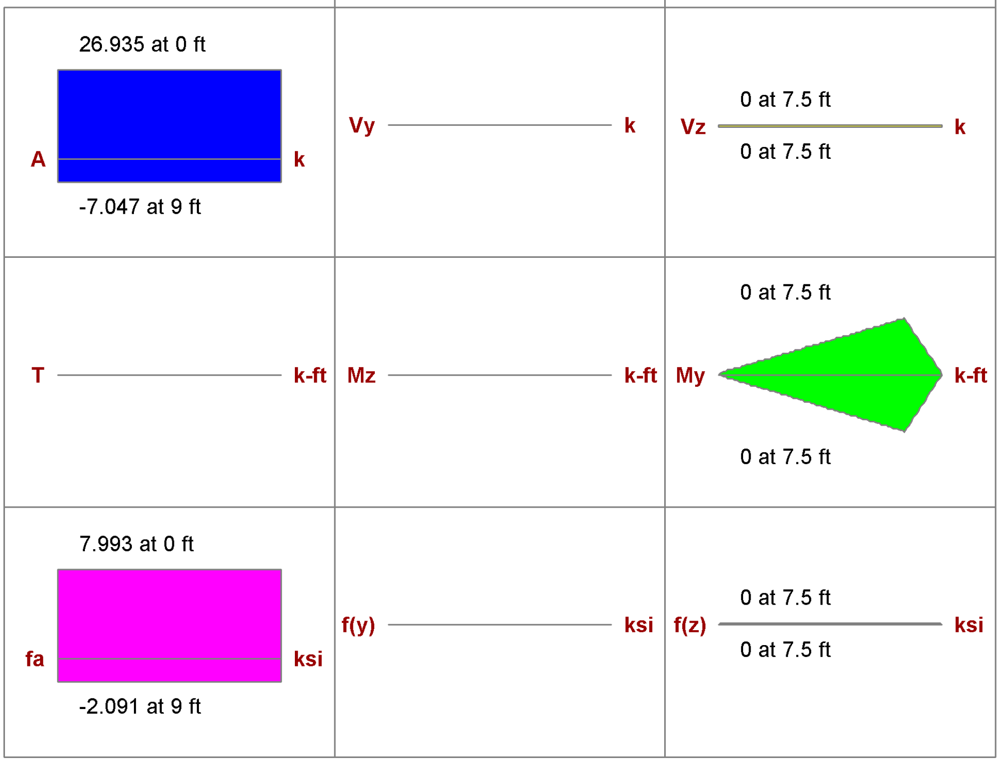
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **75.767** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(3-A)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **9 ft**
I Joint: **N213**
J Joint: **N195**
Envelope (Ω)
Code Check: **0.271 (LC 21)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method

Max Bending Check	**0.271 (LC 21)**	Max Shear Check	**0.000 (z) (LC 10)**
Location	**0 ft**	Location	**7.5 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**9 ft**	**9 ft**
phi*Pnc	**99.405 k**	KL/r	**70.989**	**70.989**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**9 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**9 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

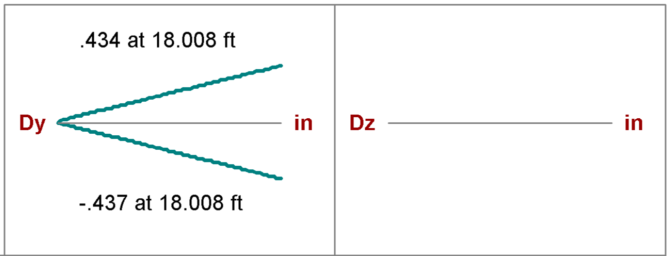
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

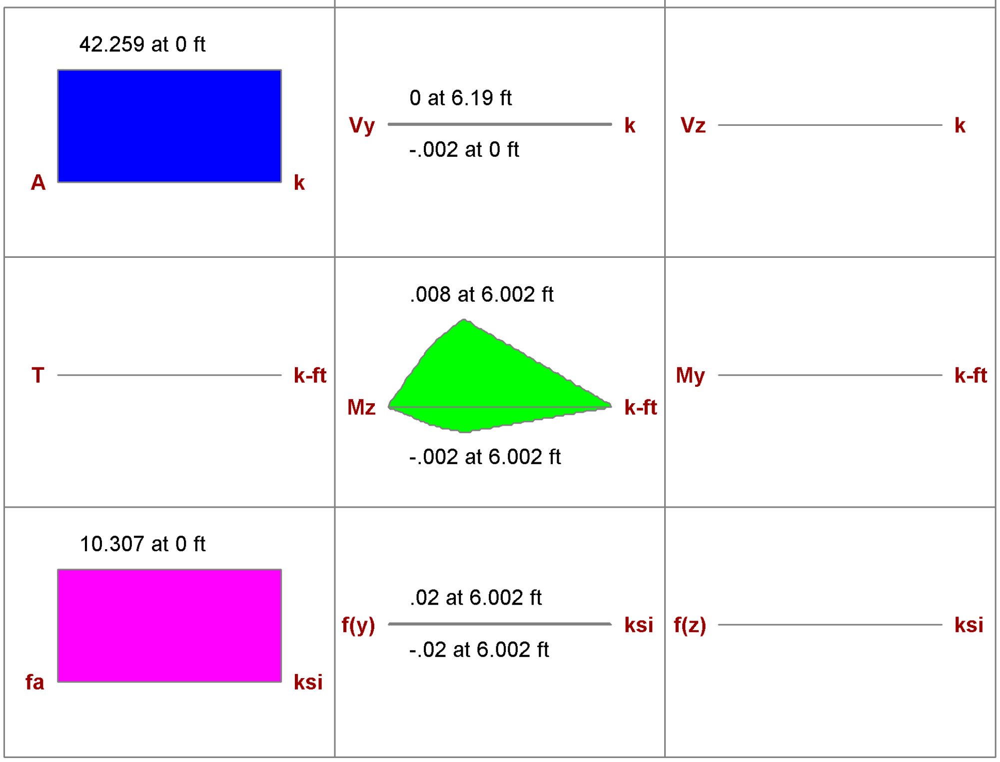
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **70.989** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(3+-A')***

Shape: **HSS4x4x5**
Material: **A500 Gr.B Rect**
Length: **18.008 ft**
I Joint: **N220A**
J Joint: **F1_N147**
Envelope (Ω)
Code Check: **0.956 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.956 (LC 20)**	Max Shear Check	**0.000 (y) (LC 20)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**18.008 ft**	**18.008 ft**
phi*Pnc	**44.22 k**	KL/r	**144.728**	**144.728**
phi*Pnt	**169.74 k**			
phi*Mny	**19.285 k-ft**	L Comp Flange	**18.008 ft**	
phi*Mnz	**19.285 k-ft**	L-torque	**18.008 ft**	
phi*Vny	**45.207 k**	Tau_b	**1**	
phi*Vnz	**45.207 k**			
phi*Tn	**16.376 k-ft**			
Cb	**1.667**			

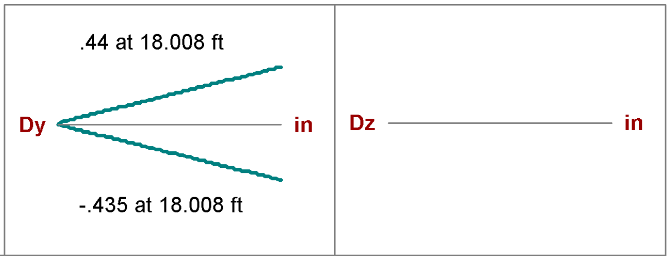
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

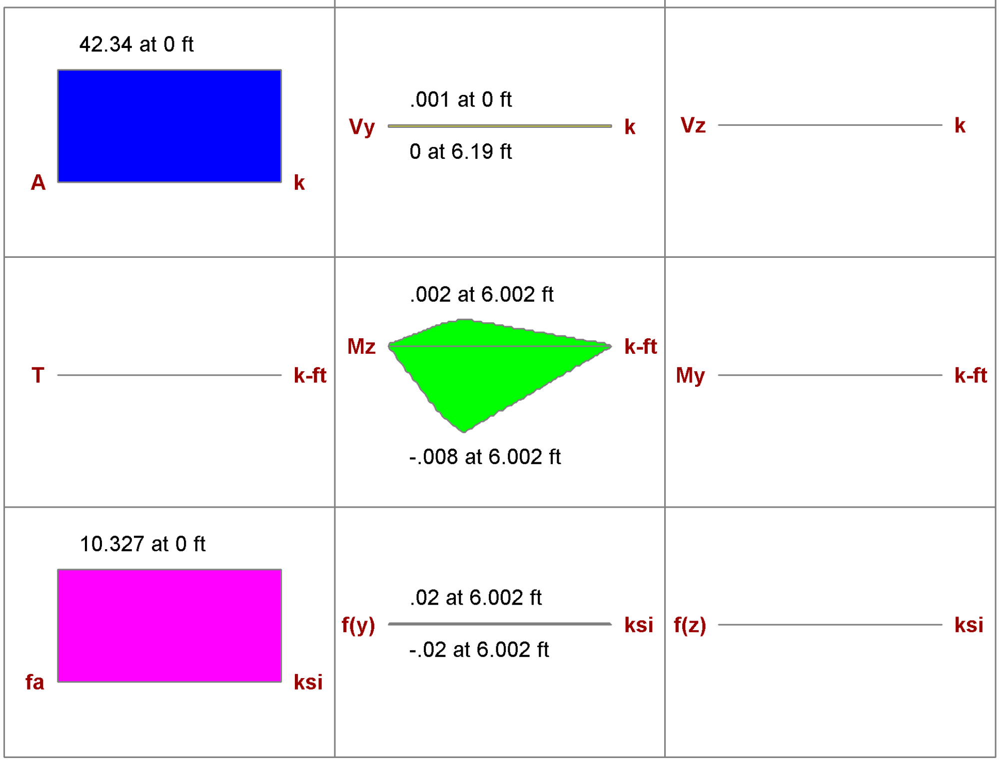
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **144.728** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(3++-A')***

Shape: **HSS4x4x5**
Material: **A500 Gr.B Rect**
Length: **18.008 ft**
I Joint: **N192**
J Joint: **F1_N201**
Envelope (Ω)
Code Check: **0.957 (LC 18)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.957 (LC 18)**	Max Shear Check	**0.000 (y) (LC 18)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**18.008 ft**	**18.008 ft**
phi*Pnc	**44.22 k**	KL/r	**144.728**	**144.728**
phi*Pnt	**169.74 k**			
phi*Mny	**19.285 k-ft**	L Comp Flange	**18.008 ft**	
phi*Mnz	**19.285 k-ft**	L-torque	**18.008 ft**	
phi*Vny	**45.207 k**	Tau_b	**1**	
phi*Vnz	**45.207 k**			
phi*Tn	**16.376 k-ft**			
Cb	**1.716**			

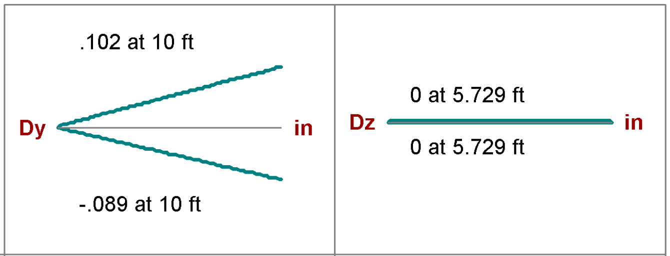
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

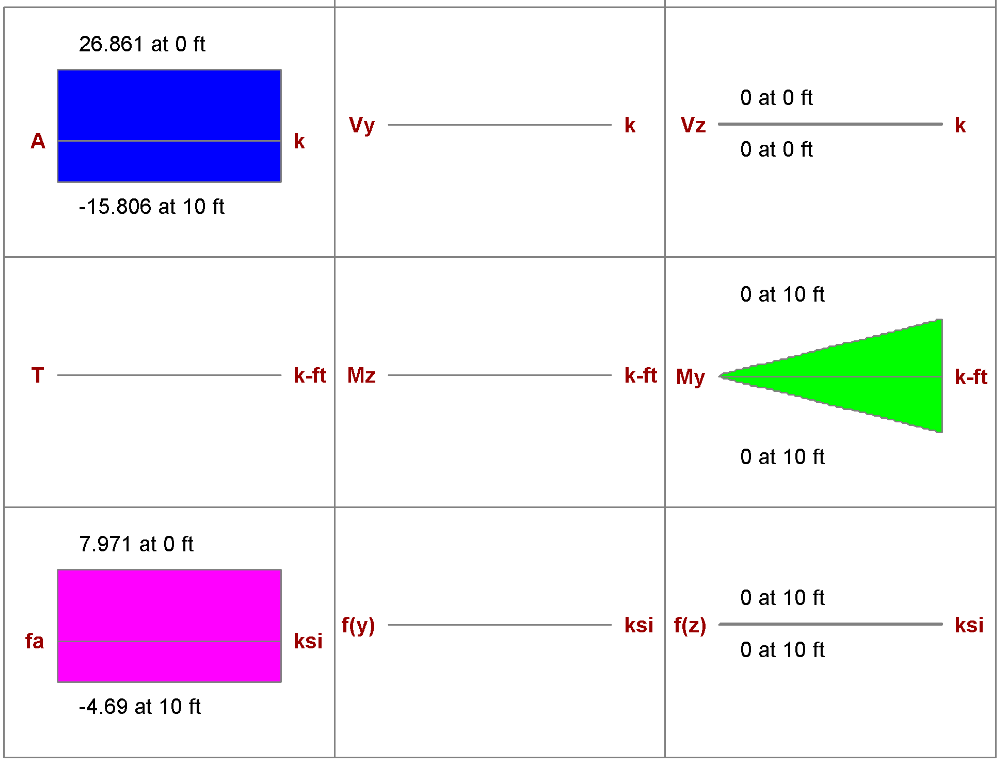
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **144.728 > 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(2--F)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N122**
J Joint: **F8_N33**
Envelope (Ω)
Code Check: **0.293 (LC 20)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check

Direct Analysis Method

- Size from RISAFloor governed optimization -

Max Bending Check	**0.293 (LC 20)**	Max Shear Check	**0.000 (z) (LC 10)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

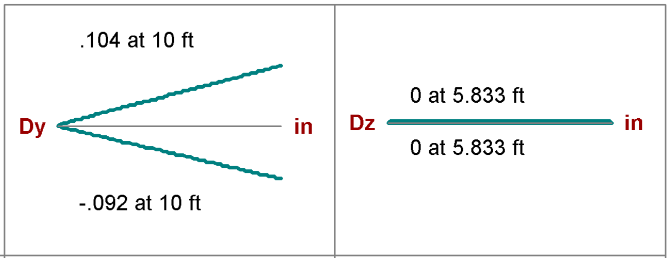
Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

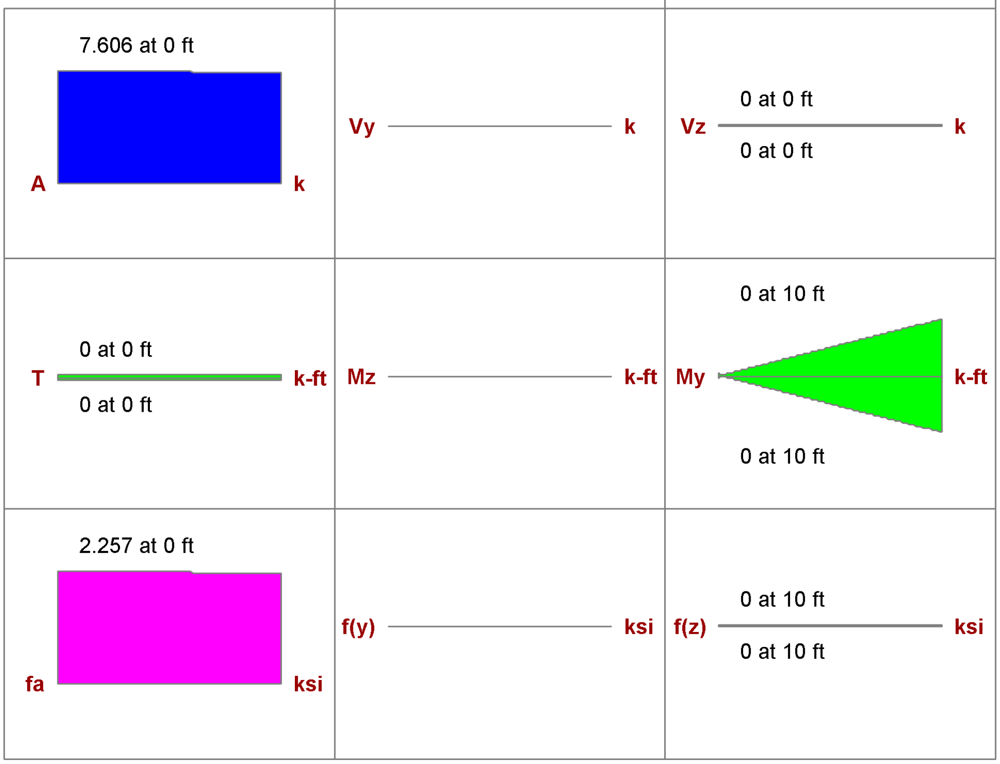
Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **78.877** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Column: ***(2-F)***

Shape: **HSS4x4x4**
Material: **A500 Gr.B Rect**
Length: **10 ft**
I Joint: **N179**
J Joint: **F8_N26**
Envelope
Code Check: **0.083 (LC 10)**
Report Based On 97 Sections





AISC 14th(360-10): LRFD Code Check
Direct Analysis Method
- Size from RISAFloor governed optimization -

Max Bending Check	**0.083 (LC 10)**	Max Shear Check	**0.000 (z) (LC 12)**
Location	**0 ft**	Location	**0 ft**
Equation	**H1-1b***	Max Defl Ratio	**L/0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**46 ksi**	Lb	**10 ft**	**10 ft**
phi*Pnc	**91.807 k**	KL/r	**78.877**	**78.877**
phi*Pnt	**139.518 k**			
phi*Mny	**16.181 k-ft**	L Comp Flange	**10 ft**	
phi*Mnz	**16.181 k-ft**	L-torque	**10 ft**	
phi*Vny	**38.211 k**	Tau_b	**1**	
phi*Vnz	**38.211 k**			
phi*Tn	**13.587 k-ft**			
Cb	**1**			

Enveloped Seismic Detailing Results (AISC 341/358 - 2010)

Member Type	**Column**	Design to Ω_o Loads	**Yes**
Seismic Design Rule	**OCBF**	Frame Ductility Req'd	**Minimal**
Moment Conn Type	**Other/None**		
Flange	**Compact**	Web	**Compact**

L/r = **78.877** **> 60 per 341-10 E3.4c(2)(2) (For Reference Only)**

Blackwell

Structural Engineers

APPENDIX E - RISA CONNECTION OUTPUT FILES

RISAConnection version 7.0.1

02/07/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

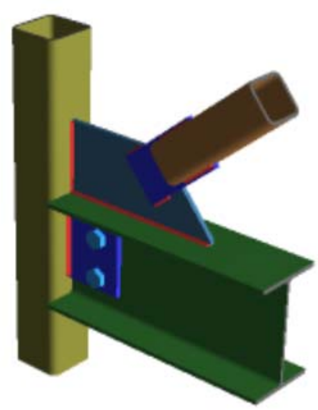
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

BR-6 Bottom Detail	**PASS(UC-0.7)**
BR-6 Top Detail	**PASS(UC-0.6)**

BR-6 Bottom Detail: 3D View

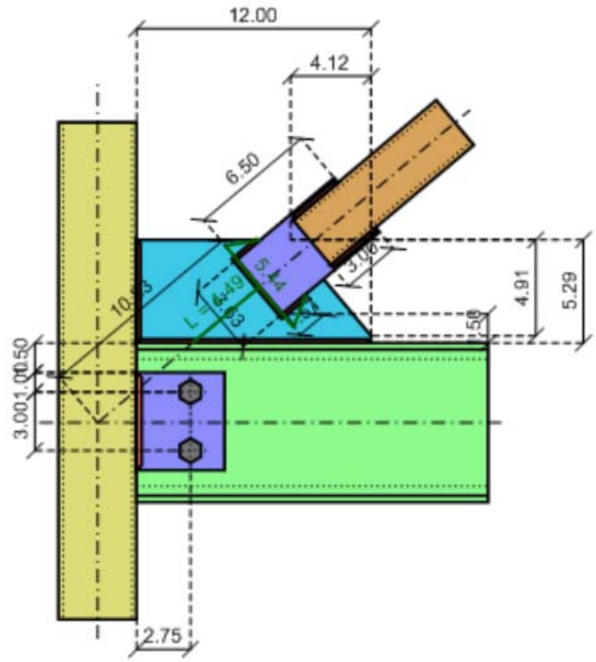
Vertical Brace Diagonal Connection



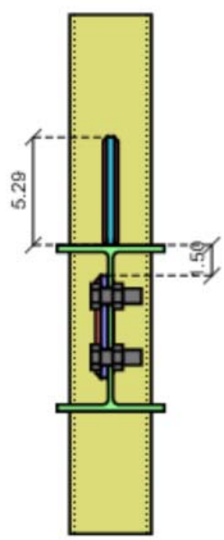
BR-6 Bottom Detail: 2D Views

Vertical Brace Diagonal Connection

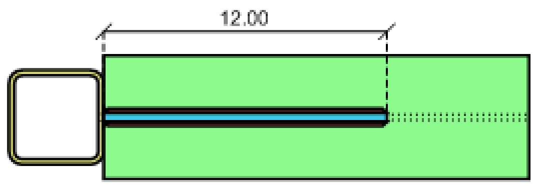
Side view



Front view

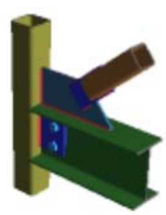


Top view



BR-6 Bottom Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.63x6.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x5.29x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	5.00 kips-ft	*User Input Column Moment*
Top Brace Axial	10.00 kips	*User Input Top Brace Axial Force (compression)*

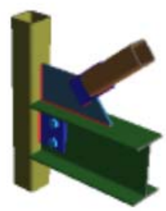
Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	Plate Flexural Buckling	**0.64**	**PASS**
Top Gusset/Beam connection	Beam Weld Strength	**0.07**	**PASS**
Top Gusset/Column connection	HSS Transverse Plastification	**0.11**	**PASS**
Top Gusset/Brace connection	Knife Plate Interaction	**0.68**	**PASS**

BR-6 Bottom Detail: Beam/Column Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.63x6.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x5.29x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	5.90 kips	*Calculated Shear Load*
Total Axial Load	6.92 kips	*Calculated Axial Load (compression)*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	5.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
$t_{p\text{-}max}$	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.23 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		

D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	5.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	5.00 in		*Max weld segment length*		
Beam Shear Yield		5.90 kips	56.17 kips	**0.10**	**PASS**
$R_n = 0.6 * F_y * A_{gv} * C_v$		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	1.87 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	56.17 kips		*Shear yield strength*		
Plate Shear Yield		5.90 kips	40.50 kips	**0.15**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	1.88 in^2		*Gross area subject to shear*		
φR_n	40.50 kips		*Shear yield strength*		
Beam Shear Rupture		5.90 kips	42.99 kips	**0.14**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	1.47 in^2		*Net area subject to shear*		
φR_n	42.99 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		5.90 kips	31.81 kips	**0.19**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	1.22 in^2		*Net area subject to shear*		
φR_n	31.81 kips		*Shear rupture strength*		
Beam Axial Yield		6.92 kips	236.70 kips	**0.03**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	5.26 in^2		*Gross area subject to tension*		
φR_n	236.70 kips		*Tensile yield strength*		
Plate Axial Yield		6.92 kips	60.75 kips	**0.11**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.88 in^2		*Gross area subject to tension*		
φR_n	60.75 kips		*Tensile yield strength*		
Beam Block Shear		5.90 kips	95.23 kips	**0.06**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		**φ = 0.75**	*(J4-5)*		
A_{gv}	3.08 in^2		*Gross area subject to shear*		
A_{nv}	2.78 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.53 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	95.23 kips		*Block shear strength*		

Plate Block Shear at Beam		5.90 kips	45.71 kips	0.13	PASS
$R_n = [\min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]$		$\varphi = 0.75$	*(J4-5)*		
A_{gv}	1.50 in^2		*Gross area subject to shear*		
A_{nv}	1.01 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.49 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	45.71 kips		*Block shear strength*		

Compression Buckling of the Plate		6.92 kips	58.73 kips	0.12	PASS
$R_n = F_{cr} * A_g$		$\varphi = 0.9$	*(E3-1)*		
K	1.00		*Effective length factor*		
L	2.75 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	25.37		*Plate slenderness check from J4-6*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.88 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	444.52 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.80 ksi		*Critical stress (E3-2)*		
φR_n	58.73 kips		*Compressive strength*		

Plate Flexural Yield				0.13	PASS
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	6.92 kips		*Calculated axial load*		
V_r	5.90 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.88 in^2		*Gross area of the plate*		
Z_{pl}	2.34 in^3		*Plastic modulus of the shear plate*		
P_c	60.75 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	40.50 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.07 in		*Vertical eccentricity*		
M_r	1.39 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	6.33 kips-ft		*Available moment $M_c = \varphi*(F_y* Z)$, φ=0.90*		
UC	0.13		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				0.11	PASS
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	6.92 kips		*Calculated axial load*		
V_r	5.90 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	1.22 in^2		*Net area of the plate*		
Z_{net}	1.36 in^3		*Plastic modulus of net section*		
V_c	31.81 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.07 in		*Vertical eccentricity*		
M_r	1.39 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	4.93 kips-ft		*Available moment $M_c = \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.11		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Plate Flexural Buckling				0.64	PASS
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		$\varphi = 0.90$	*(AISC 14th Edition)*		
P	6.92 kips		*Calculated axial load*		
V	5.90 kips		*Calculated shear load*		
L	2.75 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	25.37		*Slenderness ratio*		
F_e	444.52 ksi		*Elastic critical buckling stress, per eqn E3-4,* $F_e = (\pi^2 *E)/(KL/r)^2$		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	34.80 ksi		*Compression stress, per eqn E3-2, Fcr = (0.658 ^ (Fy / Fe)) * Fy*		
A_g	1.88 in^2		*Gross area of the plate*		
λ	0.28		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	0.96 in^3		*Section modulus of net section*		
a	2.75 in		*Design eccentricity*		
P_n	65.25 kips		*Compressive capacity, per eqn E4-1, Pn = Fcr_Comp * Ag*		
V_n	12.50 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.64		*Unity check per interaction equation, P/(Pn*Φ) + V/(Vn*Φ) <= 1*		

Bolt Bearing on Beam		9.09 kips	35.78 kips	0.25	PASS
$R_n = 1*R_{n\_boltA} + 1*R_{n\_boltB}$		$\varphi = 0.75$	*(J3-6b)*		
V	5.90 kips		*Applied shear force*		
P	6.92 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	9.09 kips		*Resultant shear force*		
Θ	40.46 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	2.64 in		*Vertical edge spacing*		
e_h	2.75 in		*Horizontal edge spacing*		
L_{c_boltA}	3.21 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min(* $(e_v/\sin(\Theta), (e_h/\cos(\Theta)) - d_c$		
L_{c_boltB}	3.21 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min(* $(s_v - 0.5 * d_v/\sin(\Theta)), (e_h/\cos(\Theta)) - d_c$		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0**d_b ** t * Fu),* R_{n-bolt}*]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0**d_b ** t * Fu),* R_{n-bolt}*]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength* $R_{n-bolt}=F_{nv}*A_{bolt}$		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	35.78 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		9.09 kips	35.78 kips	0.25	PASS
$R_n = 1*R_{n\_boltA} + 1*R_{n\_boltB}$		$\varphi = 0.75$	*(J3-6b)*		
V	5.90 kips		*Applied shear force*		
P	6.92 kips		*Applied axial force*		

$R=(V^2 + P^2)^{0.5}$	9.09 kips		*Resultant shear force*		
Θ	40.46 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.00 in		*Vertical edge spacing*		
e_h	1.75 in		*Horizontal edge spacing*		
L_{c_boltA}	1.13 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	1.89 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	35.78 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		9.09 kips	34.83 kips	**0.26**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		φ **= 0.75**	*(J3-1)*		
V	5.90 kips		*Applied shear force*		
P	6.92 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	9.09 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	2		*Number of bolts*		
C	0.97		*Eccentricity coefficient*		
φR_n	34.83 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***0.97***
Elastic method		*(AISC 14th p.7-6)*	
C	0.97	*Coefficient (1.9469 / 2)*	
N_{rows}	1	*Number of rows of bolts*	
N_{cols}	2	*Number of bolts per row*	
D_x	0.00 in	*Horizontal bolt spacing*	
D_y	3.00 in	*Vertical bolt spacing*	
E_x	0.00 in	*Horizontal eccentricity*	
E_y	0.07 in	*Vertical eccentricity*	
Ang	40.46	*Angle of force in degrees, relative X axis*	

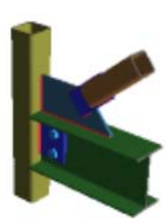
Weld at Column		9.09 kips	48.99 kips	**0.19**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
1.392 = φ * 0.6 * F_{E70} * $2^{0.5}$/2 * 1/16, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	5.00 in		*Weld length*		
φRn	48.99 kips		*Weld strength*		

HSS Transverse Plastification		6.92 kips	17.38 kips	**0.40**	**PASS**
$R_n = F_y t^2 / (1 - t_p/B) * (2 l_b/B + 4 * Q_f * (1 - t_p/B)^{0.5})$		φ = 1.00	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	5.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	17.38 kips		*Transverse plastification*		

HSS Flexural Plastification		0.00 kips-ft	4.44 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	5.00 in		*Depth of plate*		
η	1.25		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	4.44 kips-ft		*Flexural plastification*		

BR-6 Bottom Detail: Top Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.63x6.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x5.29x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	5.75 kips	*Calculated Shear Load*
Axial Load	3.90 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

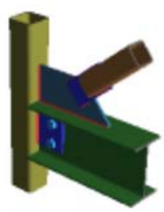
Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.33 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		

L_{min}	12.00 in		*Min weld segment length*		
Plate Shear Yield		5.75 kips	97.20 kips	**0.06**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	4.50 in^2		*Gross area subject to shear*		
φR_n	97.20 kips		*Shear yield strength*		
Plate Shear Rupture		5.75 kips	117.45 kips	**0.05**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	4.50 in^2		*Net area subject to shear*		
φR_n	117.45 kips		*Shear rupture strength*		
Plate Axial Yield		3.90 kips	145.80 kips	**0.03**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.50 in^2		*Gross area subject to tension*		
φR_n	145.80 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	3.90 kips		*Calculated axial load*		
V_r	5.75 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.50 in^2		*Gross area of the plate*		
Z_{pl}	13.50 in^3		*Plastic modulus of the shear plate*		
P_c	145.80 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	97.20 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	36.45 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	5.75 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	4.50 in^2		*Net area of the plate*		
Z_{net}	13.50 in^3		*Plastic modulus of net section*		
V_c	117.45 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	48.94 kips-ft		*Available moment M_c= φ*(F_u* Znet), φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Beam Weld Strength		5.75 kips	80.18 kips	**0.07**	**PASS**
φRn = 2 * C_1 * α * β * 1.392 * D_{16} * L Double Fillet					
1.392 = φ * 0.6 * F_{E70} * $2^{0.5}$/2 * 1/16, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		

β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	12.00 in		*Weld length*		
φRn	80.18 kips		*Weld strength*		
Beam Web Yielding		3.90 kips	174.22 kips	**0.02**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		**φ = 1.00**	*(J10-2)*		
k	0.63 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	12.00 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.23 in		*Beam web thickness*		
φRn	174.22 kips		*Beam web local yielding*		

BR-6 Bottom Detail: Top Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.63x6.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x5.29x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.53 kips	*Calculated Shear Load*
Axial Load	1.92 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

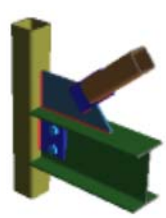
Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
F_{y-max}	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
t_{p-max}	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**

Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	5.29 in		*Min weld segment length*		
Plate Shear Yield		2.53 kips	42.81 kips	**0.06**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	1.98 in^2		*Gross area subject to shear*		
φR_n	42.81 kips		*Shear yield strength*		
Plate Shear Rupture		2.53 kips	51.73 kips	**0.05**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	1.98 in^2		*Net area subject to shear*		
φR_n	51.73 kips		*Shear rupture strength*		
Plate Axial Yield		1.92 kips	64.22 kips	**0.03**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.98 in^2		*Gross area subject to tension*		
φR_n	64.22 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	1.92 kips		*Calculated axial load*		
V_r	2.53 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.98 in^2		*Gross area of the plate*		
Z_{pl}	2.62 in^3		*Plastic modulus of the shear plate*		
P_c	64.22 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	42.81 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	7.07 kips-ft		*Available moment $M_c = \varphi*(F_y * Z)$, φ=0.90*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	2.53 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	1.98 in^2		*Net area of the plate*		
Z_{net}	2.62 in^3		*Plastic modulus of net section*		
V_c	51.73 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	9.49 kips-ft		*Available moment $M_c = \varphi*(F_u * Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Column Weld Strength		2.53 kips	35.32 kips	**0.07**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					

Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	5.29 in		*Weld length*		
φRn	35.32 kips		*Weld strength*		
HSS Transverse Plastification		1.92 kips	17.78 kips	**0.11**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	5.29 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	17.78 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	4.65 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	5.29 in		*Depth of plate*		
η	1.32		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	4.65 kips-ft		*Flexural plastification*		

BR-6 Bottom Detail: Top Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.63x6.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x5.29x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	10.00 kips	*Brace Axial (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
Gusset Weld Limitations				**PASS**

Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	1.57 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.63 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		10.00 kips	66.06 kips	**0.15**	**PASS**
$P_n = F_y*A_g$		**φ = 0.9**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	4.49 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	20.76		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.04 in^2		*Gross area of plate (Whitmore section)*		
φP_n	66.06 kips		*Gusset plate compressive strength*		

Knife Plate Buckling		10.00 kips	36.25 kips	**0.28**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	2.10		*Effective length factor*		
L	3.14 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	60.83		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.36 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	77.34 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	29.63 ksi		*Critical stress (E3-2)*		
φR_n	36.25 kips		*Compressive strength*		

Knife Plate Flexure		0.16 kips-ft	0.34 kips-ft	**0.45**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	10.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.38 in		*Thickness of knife plate*		
e	0.38 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) /2$*		
W_p	3.63 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.13 in^3		*Plastic section modulus, $Z = W_p* (t_{plate})^2/4$*		
M_u	0.16 kips-ft		*Required moment demand on knife plate,*		

		$M_u = R_u * e / 2$		
φM_n	0.34 kips-ft	*Available flexural strength*		

Knife Plate Interaction			**0.68**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$		*(H1-1a)*		
P_u	10.00 kips	*User input brace axial load*		
φP_c	36.25 kips	*Available buckling strength of knife plate*		
M_u	0.16 kips-ft	*Required moment demand on knife plate*		
φM_p	0.34 kips-ft	*Available flexural strength of knife plate*		

Gusset Weld Strength		10.00 kips	28.24 kips	**0.35**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	6.76 in		*Weld length*		
φRn	28.24 kips		*Weld strength*		

Brace Weld Strength		10.00 kips	50.11 kips	**0.20**	**PASS**
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		

BR-6 Bottom Detail: Members Report

Vertical Brace Diagonal Connection

	Beam		***W8x18***
Material			
	Name	A992	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	65.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b_f	5.25 in	*Flange width*
	d	8.14 in	*Overall depth*
	t_w	0.23 in	*Web thickness*
	t_f	0.33 in	*Flange thickness*
	a	5.26 in^2	*Area*
	k_{des}	0.63 in	*Kdes*
	k_{det}	0.81 in	*Kdet*
	k_1	0.56 in	*K1*
Web Hole Type			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	2	*Number of holes per row*

R_S	3.00 in	*Row Spacing*
C_S	3.00 in	*Column Spacing*

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Top Brace		***HSS3x3x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-6 Bottom Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.50x5.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.50 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	2	*Number of holes per row*
R_S	3.00 in	*Row Spacing*
C_S	3.00 in	*Column Spacing*

Knife Plate		***P0.38x3.63x6.50***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.63 in	*Width*
t	0.38 in	*Thickness*

Top Gusset		***P0.38x5.29x12.00***

Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	5.29 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	4.12 in	*Horiz. Clip*
V_clip	4.91 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

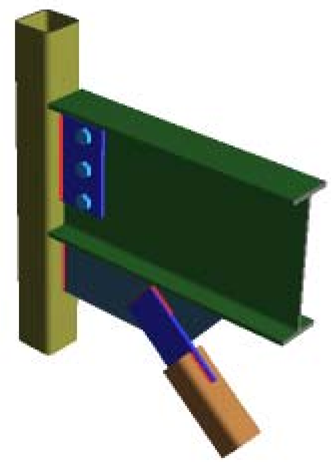
Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

BR-6 Top Detail: 3D View

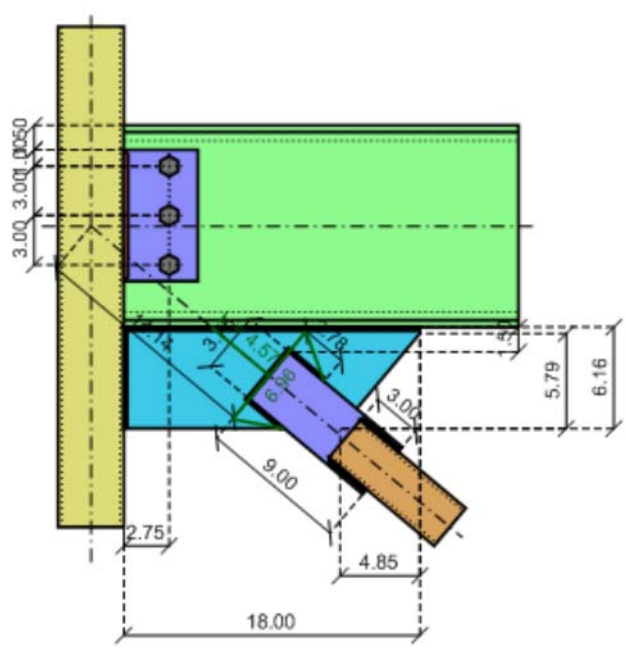
Vertical Brace Diagonal Connection



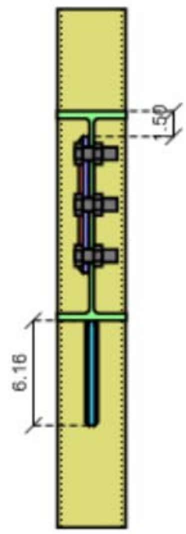
BR-6 Top Detail: 2D Views

Vertical Brace Diagonal Connection

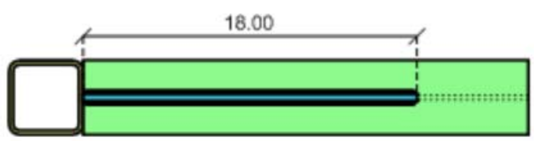
Side view

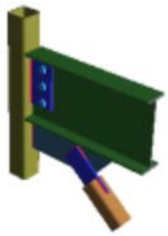


Front view



Bottom view





Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.38x3.75x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.16x18.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	10.00 kips	*User Input Bottom Brace Axial Force (compression)*

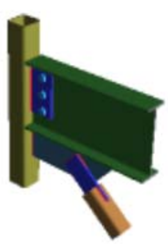
Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	HSS Transverse Plastification	**0.30**	**PASS**
Bottom Gusset/Beam connection	Beam Weld Strength	**0.05**	**PASS**
Bottom Gusset/Column connection	HSS Transverse Plastification	**0.07**	**PASS**
Bottom Gusset/Brace connection	Knife Plate Interaction	**0.64**	**PASS**

BR-6 Top Detail: Beam/Column Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.38x3.75x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.16x18.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-2.28 kips	*Calculated Shear Load*
Total Axial Load	6.39 kips	*Calculated Axial Load (compression)*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		

$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
$t_{p\text{-}max}$	0.38 in		*Maximum allowed plate thickness*		
Geometry Restrictions at Beam					**PASS**
Check Min Bolt Spacing	**Pass**		*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in		*Min bolt spacing*		
d_{bolt}	0.75 in		*Bolt diameter*		
Check Max Bolt Spacing	**Pass**		*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in		*Max bolt spacing*		
t	0.26 in		*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**		*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**		*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.25 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	8.00 in		*Max weld segment length*		
Beam Shear Yield		2.28 kips	95.94 kips	**0.02**	**PASS**
$R_n = 0.6 * F_y * A_{gv} * C_v$		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.20 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	95.94 kips		*Shear yield strength*		
Plate Shear Yield		2.28 kips	64.80 kips	**0.04**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		
Beam Shear Rupture		2.28 kips	73.58 kips	**0.03**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.52 in^2		*Net area subject to shear*		
φR_n	73.58 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		2.28 kips	52.61 kips	**0.04**	**PASS**

R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.02 in^2		*Net area subject to shear*		
φR_n	52.61 kips		*Shear rupture strength*		
Beam Axial Yield		6.39 kips	291.60 kips	**0.02**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	6.48 in^2		*Gross area subject to tension*		
φR_n	291.60 kips		*Tensile yield strength*		
Plate Axial Yield		6.39 kips	97.20 kips	**0.07**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		
Beam Block Shear		2.28 kips	131.63 kips	**0.02**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	4.55 in^2		*Gross area subject to shear*		
A_{nv}	3.98 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.60 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	131.63 kips		*Block shear strength*		
Plate Block Shear at Beam		2.28 kips	63.94 kips	**0.04**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.63 in^2		*Gross area subject to shear*		
A_{nv}	1.80 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.49 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	63.94 kips		*Block shear strength*		
Compression Buckling of the Plate		6.39 kips	93.96 kips	**0.07**	**PASS**
R_n = F_{cr} * A_g		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	2.75 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	25.37		*Plate slenderness check from J4-6*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	444.52 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.80 ksi		*Critical stress (E3-2)*		
φR_n	93.96 kips		*Compressive strength*		
Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	6.39 kips		*Calculated axial load*		
V_r	-2.28 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		

P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.65 in		*Vertical eccentricity*		
M_r	-0.18 kips-ft		*Moment due to eccentricity =* V_r*ex + P_r*ey		
M_c	16.20 kips-ft		*Available moment* $M_c=\varphi*(F_y* Z)$, $\varphi=0.90$		
UC	0.01		*Unity check per interaction equation,* $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	6.39 kips		*Calculated axial load*		
V_r	-2.28 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.02 in^2		*Net area of the plate*		
Z_{net}	3.96 in^3		*Plastic modulus of net section*		
V_c	52.61 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.65 in		*Vertical eccentricity*		
M_r	-0.18 kips-ft		*Moment due to eccentricity =* V_r*ex + P_r*ey		
M_c	14.35 kips-ft		*Available moment* $M_c= \varphi*(F_u* Znet)$, $\varphi=0.75$		
UC	0.00		*Unity check per interaction equation,* $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		

Plate Flexural Buckling				**0.15**	**PASS**
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**φ = 0.90**	*(AISC 14th Edition)*		
P	6.39 kips		*Calculated axial load*		
V	2.28 kips		*Calculated shear load*		
L	2.75 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	25.37		*Slenderness ratio*		
F_e	444.52 ksi		*Elastic critical buckling stress, per eqn E3-4,* $F_e = (\pi^2*E)/(KL/r)^2$		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	34.80 ksi		*Compression stress, per eqn E3-2, Fcr = (0.658 ^ (Fy / Fe)) * Fy*		
A_g	3.00 in^2		*Gross area of the plate*		
λ	0.35		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	2.51 in^3		*Section modulus of net section*		
a	2.75 in		*Design eccentricity*		
P_n	104.40 kips		*Compressive capacity, per eqn E4-1, Pn = Fcr_Comp * Ag*		
V_n	32.83 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.15		*Unity check per interaction equation,* $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$		

Bolt Bearing on Beam		6.79 kips	53.68 kips	**0.13**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-2.28 kips		*Applied shear force*		
P	6.39 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	6.79 kips		*Resultant shear force*		
Θ	19.65 degrees		*Angle between the resultant shear force and horizontal*		

d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	3.80 in		*Vertical edge spacing*		
e_h	2.75 in		*Horizontal edge spacing*		
L_{c_boltA}	2.51 in		*Minimum clear distance for the corner edge bolt:* $Lc\_boltA = min((e_v /sin(\Theta), (e_h /cos(\Theta)) - d_c$		
L_{c_boltB}	2.51 in		*Minimum clear distance for the side edge bolts:* $Lc\_boltB = min((s_v - 0.5 * d_v /sin(\Theta)), (e_h /cos(\Theta)) - d_c$		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt:* $Rn\_boltA = min[(1.5* Lc\_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt:* $Rn\_boltB = min[(1.5* Lc\_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$		
R_{n-bolt}	23.86 kips		*Bolt shear strength* $R_{n-bolt}=F_{nv}*A_{bolt}$		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		6.79 kips	53.68 kips	**0.13**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		φ **= 0.75**	*(J3-6b)*		
V	-2.28 kips		*Applied shear force*		
P	6.39 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	6.79 kips		*Resultant shear force*		
Θ	19.65 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.00 in		*Vertical edge spacing*		
e_h	1.75 in		*Horizontal edge spacing*		
L_{c_boltA}	1.45 in		*Minimum clear distance for the corner edge bolt:* $Lc\_boltA = min((e_v /sin(\Theta), (e_h /cos(\Theta)) - d_c$		
L_{c_boltB}	1.45 in		*Minimum clear distance for the side edge bolts:* $Lc\_boltB = min((s_v - 0.5 * d_v /sin(\Theta)), (e_h /cos(\Theta)) - d_c$		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt:* $Rn\_boltA = min[(1.5* Lc\_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt:* $Rn\_boltB = min[(1.5* Lc\_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$		
R_{n-bolt}	23.86 kips		*Bolt shear strength* $R_{n-bolt}=F_{nv}*A_{bolt}$		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		6.79 kips	41.54 kips	**0.16**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		φ **= 0.75**	*(J3-1)*		
V	-2.28 kips		*Applied shear force*		
P	6.39 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	6.79 kips		*Resultant force in bolts*		

F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.77		*Eccentricity coefficient*		
φR_n	41.54 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***0.77***		
Elastic method			*(AISC 14th p.7-6)*		
C	0.77		*Coefficient (2.3214 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.65 in		*Vertical eccentricity*		
Ang	19.65		*Angle of force in degrees, relative X axis*		

Weld at Column		6.79 kips	78.38 kips	**0.09**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ **(AISC 14th Eqn 8-2a)**					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φRn	78.38 kips		*Weld strength*		

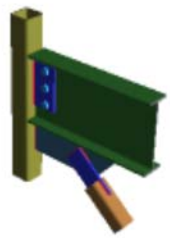
HSS Transverse Plastification		6.39 kips	21.52 kips	**0.30**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		φ **= 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		

HSS Flexural Plastification		0.00 kips-ft	8.45 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ **= 1.0**	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	8.45 kips-ft		*Flexural plastification*		

BR-6 Top Detail: Bot Gusset/Beam

LRFD
Vertical Brace Diagonal Connection

Report



Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.38x3.75x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.16x18.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	6.27 kips	*Calculated Shear Load*
Axial Load	4.28 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	18.00 in		*Min weld segment length*		
Plate Shear Yield		6.27 kips	145.80 kips	**0.04**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	6.75 in^2		*Gross area subject to shear*		
φR_n	145.80 kips		*Shear yield strength*		
Plate Shear Rupture		6.27 kips	176.17 kips	**0.04**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	6.75 in^2		*Net area subject to shear*		
φR_n	176.17 kips		*Shear rupture strength*		
Plate Axial Yield		4.28 kips	218.70 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.75 in^2		*Gross area subject to tension*		
φR_n	218.70 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14^{th} Eq.10-5)*		
P_r	4.28 kips		*Calculated axial load*		
V_r	6.27 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.75 in^2		*Gross area of the plate*		
Z_{pl}	30.38 in^3		*Plastic modulus of the shear plate*		
P_c	218.70 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	145.80 kips		*Available shear strength (see check 'Shear Yield')*		

M_r	0.00 kips-ft		*Calculated moment*		
M_c	82.01 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	6.27 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	6.75 in^2		*Net area of the plate*		
Z_{net}	30.38 in^3		*Plastic modulus of net section*		
V_c	176.17 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	110.11 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Beam Weld Strength		6.27 kips	120.27 kips	**0.05**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	18.00 in		*Weld length*		
φRn	120.27 kips		*Weld strength*		

Beam Web Yielding		4.28 kips	281.13 kips	**0.02**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		**φ = 1.00**	*(J10-2)*		
k	0.72 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	18.00 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.26 in		*Beam web thickness*		
φR_n	281.13 kips		*Beam web local yielding*		

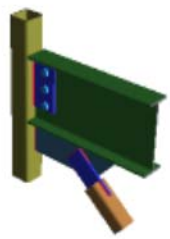
BR-6 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.38x3.75x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.16x18.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:		
Shear Load	2.14 kips	*Calculated Shear Load*
Axial Load	1.39 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*



Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
$t_{p\text{-}max}$	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	6.16 in		*Min weld segment length*		
Plate Shear Yield		2.14 kips	49.90 kips	**0.04**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.31 in^2		*Gross area subject to shear*		
φR_n	49.90 kips		*Shear yield strength*		
Plate Shear Rupture		2.14 kips	60.29 kips	**0.04**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.31 in^2		*Net area subject to shear*		
φR_n	60.29 kips		*Shear rupture strength*		
Plate Axial Yield		1.39 kips	74.85 kips	**0.02**	**PASS**

$R_n = F_y*A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.31 in^2		*Gross area subject to tension*		
φR_n	74.85 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14^{th} Eq.10-5)*		
P_r	1.39 kips		*Calculated axial load*		
V_r	2.14 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.31 in^2		*Gross area of the plate*		
Z_{pl}	3.56 in^3		*Plastic modulus of the shear plate*		
P_c	74.85 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	49.90 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	9.61 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	2.14 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.31 in^2		*Net area of the plate*		
Z_{net}	3.56 in^3		*Plastic modulus of net section*		
V_c	60.29 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	12.90 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

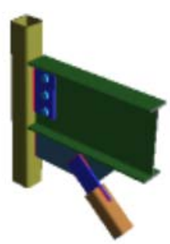
Column Weld Strength		2.14 kips	41.16 kips	**0.05**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	6.16 in		*Weld length*		
φR_n	41.16 kips		*Weld strength*		

HSS Transverse Plastification		1.39 kips	18.98 kips	**0.07**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	6.16 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		

φR_n	18.98 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	5.69 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	6.16 in		*Depth of plate*		
η	1.54		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	5.69 kips-ft		*Flexural plastification*		

BR-6 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.38x3.75x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.16x18.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	10.00 kips	*Brace Axial (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Gusset Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.78 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		
Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		

Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		10.00 kips	84.58 kips	**0.12**	**PASS**
$P_n = F_y * A_g$		**φ = 0.9**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	4.57 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	21.10		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.61 in^2		*Gross area of plate (Whitmore section)*		
φP_n	84.58 kips		*Gusset plate compressive strength*		

Knife Plate Buckling		10.00 kips	40.41 kips	**0.25**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	5.17 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	47.75		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.41 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	125.51 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	31.93 ksi		*Critical stress (E3-2)*		
φR_n	40.41 kips		*Compressive strength*		

Knife Plate Flexure		0.16 kips-ft	0.36 kips-ft	**0.44**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	10.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.38 in		*Thickness of knife plate*		
e	0.38 in		*Eccentricity, $e = (t_{plate} + t_{gusset})/2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.13 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.16 kips-ft		*Required moment demand on knife plate, $M_u = R_u * e / 2$*		
φM_n	0.36 kips-ft		*Available flexural strength*		

Knife Plate Interaction				**0.64**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	10.00 kips		*User input brace axial load*		
φP_c	40.41 kips		*Available buckling strength of knife plate*		
M_u	0.16 kips-ft		*Required moment demand on knife plate*		
φM_p	0.36 kips-ft		*Available flexural strength of knife plate*		

Gusset Weld Strength		10.00 kips	38.89 kips	**0.26**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	9.31 in		*Weld length*		
φRn	38.89 kips		*Weld strength*		

Brace Weld Strength	10.00 kips	50.11 kips	**0.20**	**PASS**

$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$

Single Fillet

$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)

C_1	1.00	*Electrode strength coefficient (AISC 14th table 8-3)*
α	1.00	*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*
D_{16}	3.00	*Weld fillet size in sixteenths of an inch*
L	3.00 in	*Weld length*
φRn	50.11 kips	*Weld strength*

BR-6 Top Detail: Members Report

Vertical Brace Diagonal Connection

	Beam		***W12x22***
Material			
	Name	A992	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	65.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b_f	4.03 in	*Flange width*
	d	12.30 in	*Overall depth*
	t_w	0.26 in	*Web thickness*
	t_f	0.42 in	*Flange thickness*
	a	6.48 in^2	*Area*
	k_{des}	0.72 in	*Kdes*
	k_{det}	0.94 in	*Kdet*
	k_1	0.63 in	*K1*
Web Hole Type			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	3	*Number of holes per row*
	R_s	3.00 in	*Row Spacing*
	C_s	3.00 in	*Column Spacing*

	Column		***HSS4x4x4***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	4.00 in	*Depth*
	b	4.00 in	*Width*
	a	3.37 in^2	*Area*
	t_{des}	0.23 in	*Wall Thickness*

	Bottom Brace		***HSS3x3x4***
Material			
	Name	A500 Gr.C Rect	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	62.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			

d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-6 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.50x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.50 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Knife Plate		***P0.38x3.75x9.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*
t	0.38 in	*Thickness*

Bottom Gusset		***P0.38x6.16x18.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	6.16 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	4.85 in	*Horiz. Clip*
V_clip	5.79 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*

T	90.00 ksi	*Tensile strength*

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

RISAConnection version 7.0.1

02/07/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

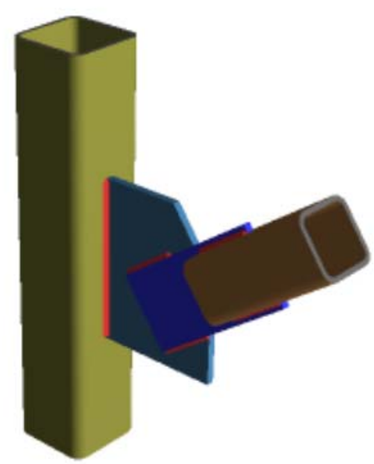
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

BR-7 Bottom Detail	**PASS(UC-0.7)**
BR-7 Top Detail	**PASS(UC-0.5)**

BR-7 Bottom Detail: 3D View

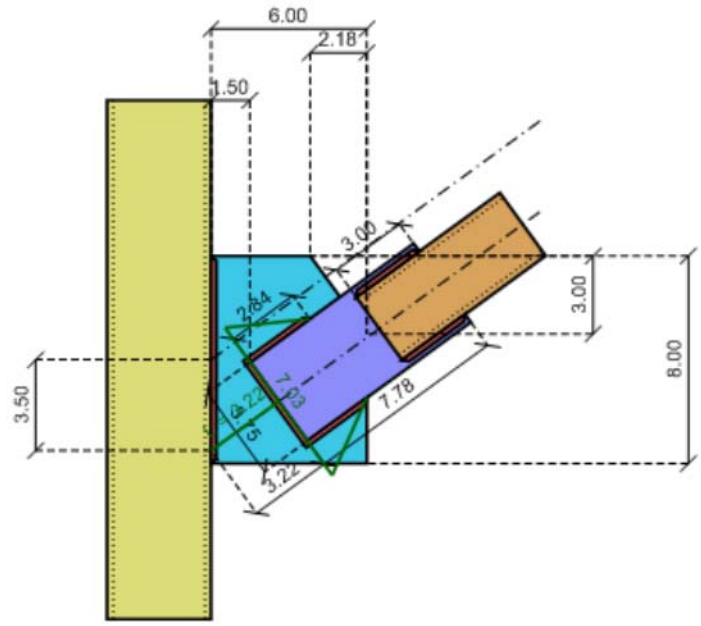
Knee Brace Connection



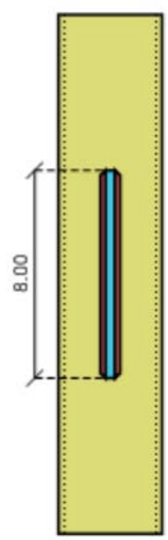
BR-7 Bottom Detail: 2D Views

Knee Brace Connection

Left view

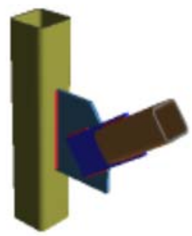


Front view



BR-7 Bottom Detail: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:

Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x6.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.78	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	8.00 kips	*Brace Axial (compression)*
Shear Load	4.70 kips	*Calculated Shear Load*
Axial Load	6.47 kips	*Calculated Axial Load (compression)*
Moment Load	1.89 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	3.50 in	*Vertical brace workpoint offset*		
d_{gusset}	8.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		
t_{min}	0.23 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	8.00 in	*Min weld segment length*		
Gusset Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			

D	0.19 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.84 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Plate Shear Yield		4.70 kips	64.80 kips	**0.07**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		

Plate Shear Rupture		4.70 kips	78.30 kips	**0.06**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.00 in^2		*Net area subject to shear*		
φR_n	78.30 kips		*Shear rupture strength*		

Plate Axial Yield		6.47 kips	97.20 kips	**0.07**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.04**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	6.47 kips		*Calculated axial load*		
V_r	4.70 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	1.89 kips-ft		*Calculated moment*		
M_c	16.20 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		
UC	0.04		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture			**0.01**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		*(Eq.10-5)*		
P_r	0.00 kips	*Calculated axial load*		
V_r	4.70 kips	*Calculated shear load*		
F_u	58.00 ksi	*Minimum tensile stress of material*		
A_n	3.00 in^2	*Net area of the plate*		
Z_{net}	6.00 in^3	*Plastic modulus of net section*		
V_c	78.30 kips	*Available shear strength (see check 'Shear Rupture')*		
M_r	1.89 kips-ft	*Calculated moment*		
M_c	21.75 kips-ft	*Available moment $M_c= \varphi*(F_u* Znet), \varphi=0.75$*		
UC	0.01	*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		2.20 kips/in	6.68 kips/in	**0.33**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
V	4.70 kips		*Shear Load*		
P	6.47 kips		*Axial Load*		
M	1.89 kips-ft		*Moment*		
e_{eff}	4.82 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	2.20 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		8.00 kips	61.23 kips	**0.13**	**PASS**
$P_n = F_{cr}*A_g$		**$\varphi = 0.9$**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	3.22 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	35.65		*Plate slenderness*		
F_{cr}	33.67 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.02 in^2		*Gross area of plate (Whitmore section)*		
φP_n	61.23 kips		*Gusset plate compressive strength*		

Gusset Weld Strength		8.00 kips	39.36 kips	**0.20**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	9.43 in		*Weld length*		
φRn	39.36 kips		*Weld strength*		

Brace Weld Strength		8.00 kips	50.11 kips	**0.16**	**PASS**
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table*		

			8-3)		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φ**Rn**	50.11 kips		*Weld strength*		

Knife Plate Flexure		0.13 kips-ft	0.36 kips-ft	**0.35**	**PASS**
$M_n = F_y * Z$		φ **= 0.9**	*(F2-1)*		
R_u	8.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.38 in		*Thickness of knife plate*		
e	0.38 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) /2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.13 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.13 kips-ft		*Required moment demand on knife plate, $M_u = R_u * e / 2$*		
φM_n	0.36 kips-ft		*Available flexural strength*		

Knife Plate Interaction				**0.67**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	8.00 kips		*User input brace axial load*		
φP_c	22.19 kips		*Available buckling strength of knife plate*		
M_u	0.13 kips-ft		*Required moment demand on knife plate*		
φM_p	0.36 kips-ft		*Available flexural strength of knife plate*		

HSS Transverse Plastification		6.47 kips	21.52 kips	**0.30**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		φ **= 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		

HSS Flexural Plastification		1.89 kips-ft	8.21 kips-ft	**0.23**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ **= 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	1.89 kips-ft		*Required flexural plastification*		
φM_n	8.21 kips-ft		*Flexural plastification*		

Knife Plate Buckling		8.00 kips	22.19 kips	**0.36**	**PASS**
$R_n = F_{cr} * A_g$		φ **= 0.9**	*(E3-1)*		
K	2.10		*Effective length factor*		
L	6.03 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	116.91		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.41 in^2		*Gross area subject to compression*		

E	29000.00 ksi	*Modulus of elasticity*
F_e	20.94 ksi	*Elastic buckling stress (E3-4)*
F_{cr}	17.53 ksi	*Critical stress (E3-2)*
φR_n	22.19 kips	*Compressive strength*

BR-7 Bottom Detail: Members Report

Knee Brace Connection

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Brace		***HSS3x3x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-7 Bottom Detail: Components Report

Knee Brace Connection

Gusset		***P0.38x6.00x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	6.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	2.18 in	*Horiz. Clip*
V_clip	3.00 in	*Vert. Clip*

Knife Plate		***P0.38x3.75x7.78***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*
t	0.38 in	*Thickness*

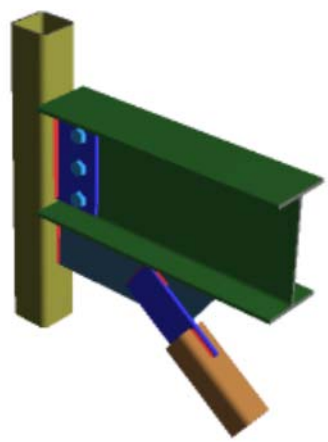
	Column Weld	***E70***
Weld Properties		
	Type	Double Fillet
	Fillet Size	0.19 in

	Gusset Plate Weld	***E70***
Weld Properties		
	Type	Single Fillet
	Fillet Size	0.19 in

	Knife Plate Brace Weld	***E70***
Weld Properties		
	Type	Single Fillet
	Fillet Size	0.19 in

BR-7 Top Detail: 3D View

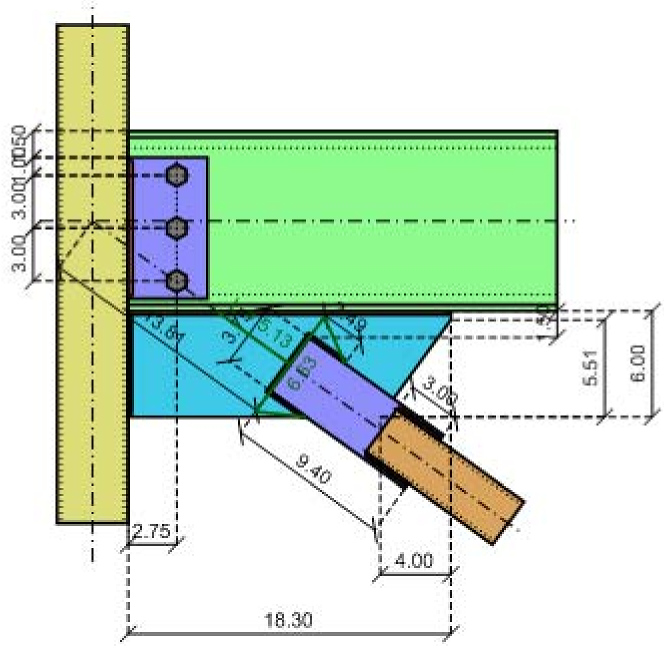
Vertical Brace Diagonal Connection



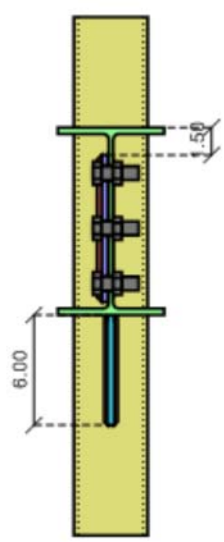
BR-7 Top Detail: 2D Views

Vertical Brace Diagonal Connection

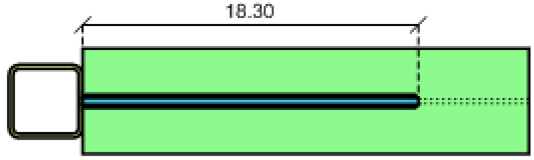
Side view



Front view

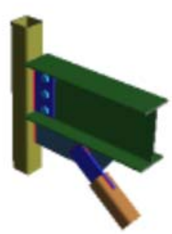


Bottom view



BR-7 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x9.40	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x18.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	3.00 kips	*User Input Column Force*
Column Moment	2.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	8.00 kips	*User Input Bottom Brace Axial Force (compression)*

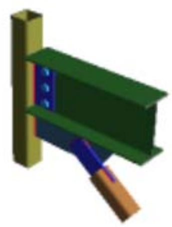
Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	HSS Transverse Plastification	**0.29**	**PASS**
Bottom Gusset/Beam connection	Beam Weld Strength	**0.04**	**PASS**
Bottom Gusset/Column connection	HSS Transverse Plastification	**0.06**	**PASS**
Bottom Gusset/Brace connection	Knife Plate Interaction	**0.52**	**PASS**

BR-7 Top Detail: Beam/Column Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x9.40	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x18.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-0.96 kips	*Calculated Shear Load*
Total Axial Load	6.16 kips	*Calculated Axial Load (compression)*
Column Force	3.00 kips	*User Input Column Force*
Column Moment	2.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.24 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		

t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	8.00 in		*Max weld segment length*		

Beam Shear Yield		0.96 kips	73.44 kips	**0.01**	**PASS**
$R_n = 0.6 * F_y * A_{gv} * C_v$		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.45 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	73.44 kips		*Shear yield strength*		

Plate Shear Yield		0.96 kips	64.80 kips	**0.01**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		

Beam Shear Rupture		0.96 kips	53.18 kips	**0.02**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	1.82 in^2		*Net area subject to shear*		
φR_n	53.18 kips		*Shear rupture strength*		

Plate Shear Rupture at Beam		0.96 kips	52.61 kips	**0.02**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.02 in^2		*Net area subject to shear*		
φR_n	52.61 kips		*Shear rupture strength*		

Beam Axial Yield		6.16 kips	292.05 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	6.49 in^2		*Gross area subject to tension*		
φR_n	292.05 kips		*Tensile yield strength*		

Plate Axial Yield		6.16 kips	97.20 kips	**0.06**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		

Beam Block Shear		0.96 kips	117.51 kips	**0.01**	**PASS**
R_n = [min($0.6 * F_u * A_{nv}$, $0.6 * F_y * A_{gv}$) + $U_{bs} * F_u * A_{nt}$]		**φ = 0.75**	*(J4-5)*		
A_{gv}	4.02 in^2		*Gross area subject to shear*		
A_{nv}	3.50 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.55 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	117.51 kips		*Block shear strength*		

Plate Block Shear at Beam		0.96 kips	63.94 kips	**0.02**	**PASS**

R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		φ = 0.75	*(J4-5)*		
A_{gv}	2.63 in^2		*Gross area subject to shear*		
A_{nv}	1.80 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.49 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	63.94 kips		*Block shear strength*		
Compression Buckling of the Plate		6.16 kips	93.96 kips	**0.07**	**PASS**
R_n = F_{cr} * A_g		φ = 0.9	*(E3-1)*		
K	1.00		*Effective length factor*		
L	2.75 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	25.37		*Plate slenderness check from J4-6*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	444.52 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.80 ksi		*Critical stress (E3-2)*		
φR_n	93.96 kips		*Compressive strength*		
Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	6.16 kips		*Calculated axial load*		
V_r	-0.96 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	-0.40 in		*Vertical eccentricity*		
M_r	-0.43 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	16.20 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	6.16 kips		*Calculated axial load*		
V_r	-0.96 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.02 in^2		*Net area of the plate*		
Z_{net}	3.96 in^3		*Plastic modulus of net section*		
V_c	52.61 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	-0.40 in		*Vertical eccentricity*		
M_r	-0.43 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	14.35 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Plate Flexural Buckling				**0.10**	**PASS**

$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**φ = 0.90**	*(AISC 14th Edition)*		
P	6.16 kips		*Calculated axial load*		
V	0.96 kips		*Calculated shear load*		
L	2.75 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	25.37		*Slenderness ratio*		
F_e	444.52 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2*E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	34.80 ksi		*Compression stress, per eqn E3-2, Fcr = (0.658 ^ (Fy / Fe)) * Fy*		
A_g	3.00 in^2		*Gross area of the plate*		
λ	0.35		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	2.51 in^3		*Section modulus of net section*		
a	2.75 in		*Design eccentricity*		
P_n	104.40 kips		*Compressive capacity, per eqn E4-1, Pn = Fcr_Comp * Ag*		
V_n	32.83 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.10		*Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$*		

Bolt Bearing on Beam		6.24 kips	53.68 kips	**0.12**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-0.96 kips		*Applied shear force*		
P	6.16 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	6.24 kips		*Resultant shear force*		
Θ	8.86 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.70 in		*Vertical edge spacing*		
e_h	2.75 in		*Horizontal edge spacing*		
L_{c_boltA}	2.38 in		*Minimum clear distance for the corner edge bolt: $Lc\_boltA = min((e_v /sin(\Theta), (e_h /cos(\Theta)) - d_c$*		
L_{c_boltB}	2.38 in		*Minimum clear distance for the side edge bolts: $Lc\_boltB = min((s_v - 0.5 * d_v /sin(\Theta)), (e_h /cos(\Theta)) - d_c$*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: $Rn\_boltA = min[(1.5* Lc\_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: $Rn\_boltB = min[(1.5* Lc\_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		6.24 kips	53.68 kips	**0.12**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-0.96 kips		*Applied shear force*		
P	6.16 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	6.24 kips		*Resultant shear force*		

Θ	8.86 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.00 in		*Vertical edge spacing*		
e_h	1.75 in		*Horizontal edge spacing*		
L_{c_boltA}	1.36 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	1.36 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength R_{n-bolt}=F_{nv}*A_{bolt}*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		6.24 kips	44.89 kips	**0.14**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		**φ = 0.75**	*(J3-1)*		
V	-0.96 kips		*Applied shear force*		
P	6.16 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	6.24 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.84		*Eccentricity coefficient*		
φR_n	44.89 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***0.84***		
Elastic method			*(AISC 14th p.7-6)*		
C	0.84		*Coefficient (2.5091 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.40 in		*Vertical eccentricity*		
Ang	8.86		*Angle of force in degrees, relative X axis*		

Weld at Column		6.24 kips	78.38 kips	**0.08**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φR_n	78.38 kips		*Weld strength*		

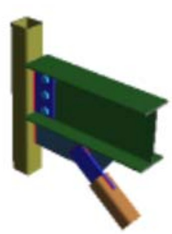
HSS Transverse Plastification		6.16 kips	21.52 kips	**0.29**	**PASS**

$R_n = F_y{*}t^2 / (1-t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1-t_p/B)^{0.5})$		$\varphi = 1.00$	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	8.45 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		$\varphi = 1.0$	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	8.45 kips-ft		*Flexural plastification*		

BR-7 Top Detail: Bot Gusset/Beam Report

LRFD

Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x9.40	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x18.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	5.31 kips	*Calculated Shear Load*
Axial Load	2.96 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.36 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	18.30 in		*Min weld segment length*		

Plate Shear Yield		5.31 kips	148.21 kips	**0.04**	PASS
$R_n = 0.6 * F_y * A_{gv}$		**$\varphi = 1.00$**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	6.86 in^2		*Gross area subject to shear*		
φR_n	148.21 kips		*Shear yield strength*		

Plate Shear Rupture		5.31 kips	179.09 kips	**0.03**	PASS
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	6.86 in^2		*Net area subject to shear*		
φR_n	179.09 kips		*Shear rupture strength*		

Plate Axial Yield		2.96 kips	222.31 kips	**0.01**	PASS
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.86 in^2		*Gross area subject to tension*		
φR_n	222.31 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.00**	PASS
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	2.96 kips		*Calculated axial load*		
V_r	5.31 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.86 in^2		*Gross area of the plate*		
Z_{pl}	31.39 in^3		*Plastic modulus of the shear plate*		
P_c	222.31 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	148.21 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	84.74 kips-ft		*Available moment $M_c=\varphi*(F_y*Z)$, $\varphi=0.90$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

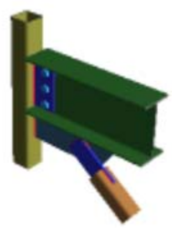
Plate Flexural Rupture				**0.00**	PASS
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	5.31 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	6.86 in^2		*Net area of the plate*		
Z_{net}	31.39 in^3		*Plastic modulus of net section*		
V_c	179.09 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	113.78 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, $\varphi=0.75$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Beam Weld Strength		5.31 kips	122.26 kips	**0.04**	PASS
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		

L	18.30 in		*Weld length*		
φRn	122.26 kips		*Weld strength*		
Beam Web Yielding		2.96 kips	259.17 kips	**0.01**	PASS
$R_n = (5 * k + N) * F_y * t_w$		**φ = 1.00**	*(J10-2)*		
k	0.66 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	18.30 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.24 in		*Beam web thickness*		
φRn	259.17 kips		*Beam web local yielding*		

BR-7 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x9.40	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x18.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	1.74 kips	*Calculated Shear Load*
Axial Load	1.16 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					PASS
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
F_{y-max}	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
t_{p-max}	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					PASS
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		

D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	6.00 in		*Min weld segment length*		

Plate Shear Yield		1.74 kips	48.60 kips	**0.04**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.25 in^2		*Gross area subject to shear*		
φR_n	48.60 kips		*Shear yield strength*		

Plate Shear Rupture		1.74 kips	58.72 kips	**0.03**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.25 in^2		*Net area subject to shear*		
φR_n	58.72 kips		*Shear rupture strength*		

Plate Axial Yield		1.16 kips	72.90 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.25 in^2		*Gross area subject to tension*		
φR_n	72.90 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	1.16 kips		*Calculated axial load*		
V_r	1.74 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.25 in^2		*Gross area of the plate*		
Z_{pl}	3.38 in^3		*Plastic modulus of the shear plate*		
P_c	72.90 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	48.60 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	9.11 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

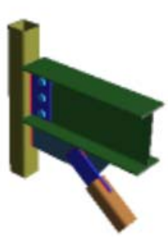
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	1.74 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.25 in^2		*Net area of the plate*		
Z_{net}	3.38 in^3		*Plastic modulus of net section*		
V_c	58.72 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	12.23 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		1.74 kips	40.09 kips	**0.04**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					

C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	6.00 in		*Weld length*		
φRn	40.09 kips		*Weld strength*		
HSS Transverse Plastification		1.16 kips	18.76 kips	**0.06**	**PASS**
$R_n = F_y{*}t^2\,/(1-t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1-t_p/B)^{0.5})$		φ = 1.00	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	6.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	18.76 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	5.49 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	6.00 in		*Depth of plate*		
η	1.50		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	5.49 kips-ft		*Flexural plastification*		

BR-7 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x9.40	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x18.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	8.00 kips	*Brace Axial (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Gusset Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				

D	0.19 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.49 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		8.00 kips	80.56 kips	**0.10**	**PASS**
$P_n = Fy*A_g$		**φ = 0.9**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	5.13 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	23.71		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.49 in^2		*Gross area of plate (Whitmore section)*		
φP_n	80.56 kips		*Gusset plate compressive strength*		

Knife Plate Buckling		8.00 kips	39.40 kips	**0.20**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	5.69 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	52.54		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.41 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	103.67 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	31.13 ksi		*Critical stress (E3-2)*		
φR_n	39.40 kips		*Compressive strength*		

Knife Plate Flexure		0.13 kips-ft	0.36 kips-ft	**0.35**	**PASS**
$M_n = Fy * Z$		**φ = 0.9**	*(F2-1)*		
R_u	8.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.38 in		*Thickness of knife plate*		
e	0.38 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) /2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.13 in^3		*Plastic section modulus, $Z = W_p* (t_{plate})^2/4$*		
M_u	0.13 kips-ft		*Required moment demand on knife plate, $M_u=R_u * e / 2$*		
φM_n	0.36 kips-ft		*Available flexural strength*		

Knife Plate Interaction			0.52	PASS
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$		*(H1-1a)*		
P_u	8.00 kips	*User input brace axial load*		
φP_c	39.40 kips	*Available buckling strength of knife plate*		
M_u	0.13 kips-ft	*Required moment demand on knife plate*		
φM_p	0.36 kips-ft	*Available flexural strength of knife plate*		

Gusset Weld Strength		8.00 kips	36.49 kips	0.22	PASS
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	8.74 in		*Weld length*		
φRn	36.49 kips		*Weld strength*		

Brace Weld Strength		8.00 kips	50.11 kips	0.16	PASS
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		

BR-7 Top Detail: Members Report

Vertical Brace Diagonal Connection

	Beam		***W10x22***
Material			
	Name	A992	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	65.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b_f	5.75 in	*Flange width*
	d	10.20 in	*Overall depth*
	t_w	0.24 in	*Web thickness*
	t_f	0.36 in	*Flange thickness*
	a	6.49 in^2	*Area*
	k_{des}	0.66 in	*Kdes*
	k_{det}	0.94 in	*Kdet*
	k_1	0.63 in	*K1*
Web Hole Type			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	3	*Number of holes per row*
	R_s	3.00 in	*Row Spacing*
	C_s	3.00 in	*Column Spacing*

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Bottom Brace		***HSS3x3x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-7 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.50x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.50 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Knife Plate		***P0.38x3.75x9.40***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*
t	0.38 in	*Thickness*

Bottom Gusset		***P0.38x6.00x18.30***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*

F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	6.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	4.00 in	*Horiz. Clip*
V_clip	5.51 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

RISAConnection version 7.0.1

02/07/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

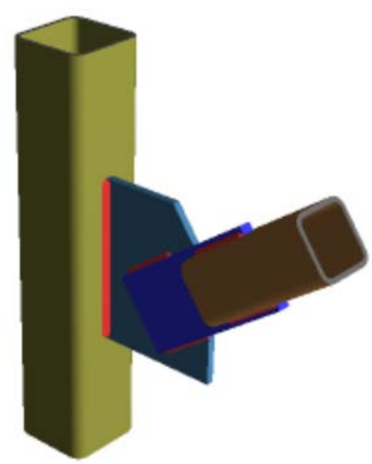
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

BR-8 Bottom Detail	**PASS(UC-0.9)**
BR-8 Top Detail	**PASS(UC-0.9)**

BR-8 Bottom Detail: 3D View

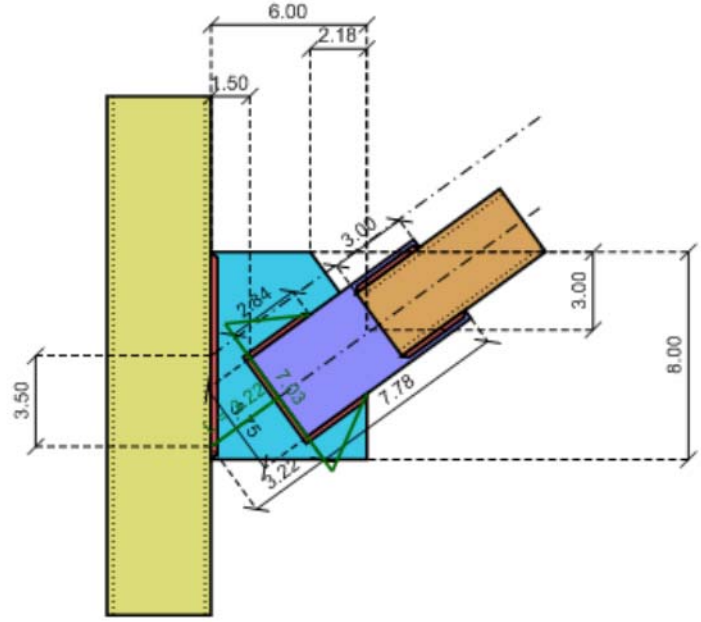
Knee Brace Connection



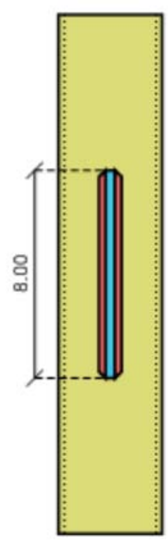
BR-8 Bottom Detail: 2D Views

Knee Brace Connection

Left view

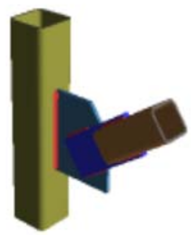


Front view



BR-8 Bottom Detail: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:

Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x6.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x7.78	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	25.00 kips	*Brace Axial (compression)*
Shear Load	14.69 kips	*Calculated Shear Load*
Axial Load	20.23 kips	*Calculated Axial Load (compression)*
Moment Load	5.90 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	3.50 in	*Vertical brace workpoint offset*		
d_{gusset}	8.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		
t_{min}	0.23 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in	*Weld size*		
L_{min}	8.00 in	*Min weld segment length*		
Gusset Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			

D	0.19 in		*Weld size*		
D_{max}	0.56 in		*Max Size Allowed*		
t	0.63 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.84 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Plate Shear Yield		14.69 kips	64.80 kips	**0.23**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		

Plate Shear Rupture		14.69 kips	78.30 kips	**0.19**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.00 in^2		*Net area subject to shear*		
φR_n	78.30 kips		*Shear rupture strength*		

Plate Axial Yield		20.23 kips	97.20 kips	**0.21**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.38**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14^{th} Eq.10-5)*		
P_r	20.23 kips		*Calculated axial load*		
V_r	14.69 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	5.90 kips-ft		*Calculated moment*		
M_c	16.20 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		
UC	0.38		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture			**0.11**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		*(Eq.10-5)*		
P_r	0.00 kips	*Calculated axial load*		
V_r	14.69 kips	*Calculated shear load*		
F_u	58.00 ksi	*Minimum tensile stress of material*		
A_n	3.00 in^2	*Net area of the plate*		
Z_{net}	6.00 in^3	*Plastic modulus of net section*		
V_c	78.30 kips	*Available shear strength (see check 'Shear Rupture')*		
M_r	5.90 kips-ft	*Calculated moment*		
M_c	21.75 kips-ft	*Available moment $M_c = \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.11	*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		6.89 kips/in	7.84 kips/in	**0.88**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
V	14.69 kips		*Shear Load*		
P	20.23 kips		*Axial Load*		
M	5.90 kips-ft		*Moment*		
e_{eff}	4.82 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
r_u	6.89 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	7.84 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		25.00 kips	61.23 kips	**0.41**	**PASS**
$P_n = F_{cr}*A_g$		**φ = 0.9**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	3.22 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	35.65		*Plate slenderness*		
F_{cr}	33.67 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.02 in^2		*Gross area of plate (Whitmore section)*		
φP_n	61.23 kips		*Gusset plate compressive strength*		

Gusset Weld Strength		25.00 kips	39.36 kips	**0.64**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	9.43 in		*Weld length*		
φRn	39.36 kips		*Weld strength*		

Brace Weld Strength		25.00 kips	50.11 kips	**0.50**	**PASS**
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table*		

			8-3)		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		

Knife Plate Flexure		0.52 kips-ft	0.99 kips-ft	**0.53**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	25.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.63 in		*Thickness of knife plate*		
e	0.50 in		*Eccentricity, $e = (t_{plate} + t_{gusset})/2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.37 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.52 kips-ft		*Required moment demand on knife plate, $M_u=R_u * e / 2$*		
φM_n	0.99 kips-ft		*Available flexural strength*		

Knife Plate Interaction				**0.89**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	25.00 kips		*User input brace axial load*		
φP_c	58.61 kips		*Available buckling strength of knife plate*		
M_u	0.52 kips-ft		*Required moment demand on knife plate*		
φM_p	0.99 kips-ft		*Available flexural strength of knife plate*		

HSS Transverse Plastification		20.23 kips	21.52 kips	**0.94**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		

HSS Flexural Plastification		5.90 kips-ft	8.45 kips-ft	**0.70**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	5.90 kips-ft		*Required flexural plastification*		
φM_n	8.45 kips-ft		*Flexural plastification*		

Knife Plate Buckling		25.00 kips	58.61 kips	**0.43**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	2.10		*Effective length factor*		
L	6.03 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	70.15		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.34 in^2		*Gross area subject to compression*		

E	29000.00 ksi	*Modulus of elasticity*
F_e	58.17 ksi	*Elastic buckling stress (E3-4)*
F_{cr}	27.78 ksi	*Critical stress (E3-2)*
φR_n	58.61 kips	*Compressive strength*

BR-8 Bottom Detail: Members Report

Knee Brace Connection

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Brace		***HSS3x3x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-8 Bottom Detail: Components Report

Knee Brace Connection

Gusset		***P0.38x6.00x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	6.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	2.18 in	*Horiz. Clip*
V_clip	3.00 in	*Vert. Clip*

Knife Plate		***P0.63x3.75x7.78***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*
t	0.63 in	*Thickness*

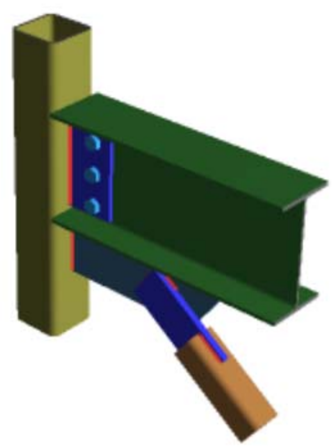
Column Weld	***E70***
Weld Properties	
Type	Double Fillet
Fillet Size	0.25 in

Gusset Plate Weld	***E70***
Weld Properties	
Type	Single Fillet
Fillet Size	0.19 in

Knife Plate Brace Weld	***E70***
Weld Properties	
Type	Single Fillet
Fillet Size	0.19 in

BR-8 Top Detail: 3D View

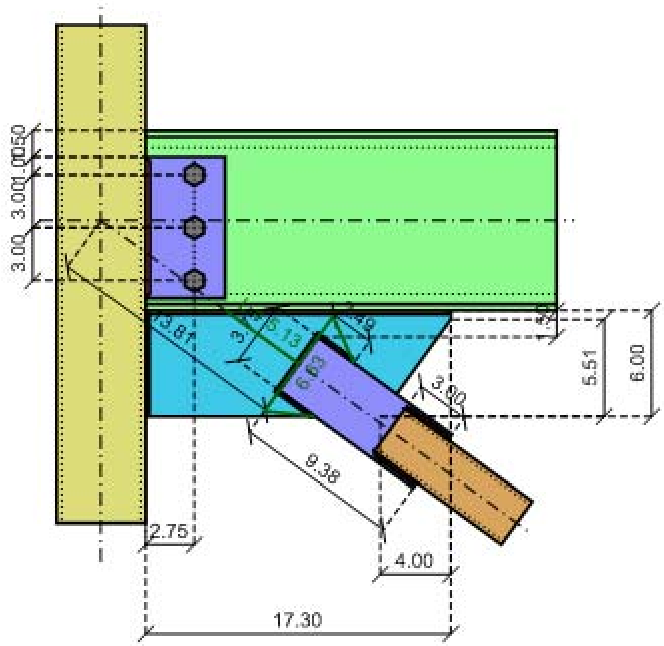
Vertical Brace Diagonal Connection



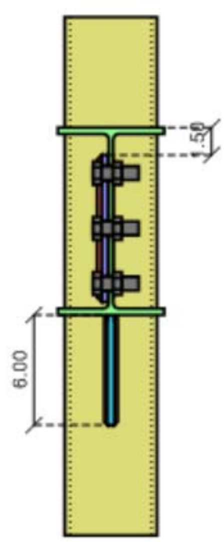
BR-8 Top Detail: 2D Views

Vertical Brace Diagonal Connection

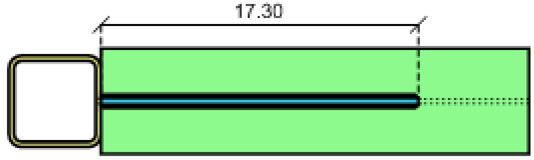
Side view



Front view

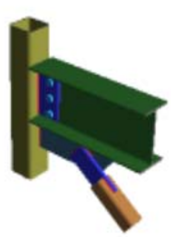


Bottom view



BR-8 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x9.38	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x17.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	12.50 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	46.00 kips	*User Input Column Force*
Column Moment	2.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	28.00 kips	*User Input Bottom Brace Axial Force (compression)*

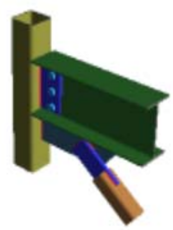
Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	HSS Transverse Plastification	**0.34**	**PASS**
Bottom Gusset/Beam connection	Beam Weld Strength	**0.15**	**PASS**
Bottom Gusset/Column connection	HSS Transverse Plastification	**0.19**	**PASS**
Bottom Gusset/Brace connection	Knife Plate Interaction	**0.91**	**PASS**

BR-8 Top Detail: Beam/Column Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x9.38	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x17.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	2.14 kips	*Calculated Shear Load*
Total Axial Load	10.08 kips	*Calculated Axial Load (compression)*
Column Force	46.00 kips	*User Input Column Force*
Column Moment	2.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.29 in	*Column wall thickness*		
B	5.00 in	*Column face width*		
(B - 3 * t) / t	14.18	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.18	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.47 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.24 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		

t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.29 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	8.00 in		*Max weld segment length*		
Beam Shear Yield		2.14 kips	73.44 kips	**0.03**	**PASS**
R_n = 0.6 *Fy*A_{gv}*C_v		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.45 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φRn	73.44 kips		*Shear yield strength*		
Plate Shear Yield		2.14 kips	64.80 kips	**0.03**	**PASS**
R_n = 0.6 *Fy*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φRn	64.80 kips		*Shear yield strength*		
Beam Shear Rupture		2.14 kips	53.18 kips	**0.04**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	1.82 in^2		*Net area subject to shear*		
φRn	53.18 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		2.14 kips	52.61 kips	**0.04**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.02 in^2		*Net area subject to shear*		
φRn	52.61 kips		*Shear rupture strength*		
Beam Axial Yield		10.08 kips	292.05 kips	**0.03**	**PASS**
R_n = Fy*A_g		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	6.49 in^2		*Gross area subject to tension*		
φRn	292.05 kips		*Tensile yield strength*		
Plate Axial Yield		10.08 kips	97.20 kips	**0.10**	**PASS**
R_n = Fy*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φRn	97.20 kips		*Tensile yield strength*		
Beam Block Shear		2.14 kips	117.51 kips	**0.02**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*Fu*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	4.02 in^2		*Gross area subject to shear*		
A_{nv}	3.50 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.55 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φRn	117.51 kips		*Block shear strength*		
Plate Block Shear at Beam		2.14 kips	63.94 kips	**0.03**	**PASS**

R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.63 in^2		*Gross area subject to shear*		
A_{nv}	1.80 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.49 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	63.94 kips		*Block shear strength*		
Compression Buckling of the Plate		10.08 kips	93.96 kips	**0.11**	**PASS**
R_n = F_{cr} * A_g		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	2.75 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	25.37		*Plate slenderness check from J4-6*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	444.52 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.80 ksi		*Critical stress (E3-2)*		
φR_n	93.96 kips		*Compressive strength*		
Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	10.08 kips		*Calculated axial load*		
V_r	2.14 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	-0.40 in		*Vertical eccentricity*		
M_r	0.15 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	16.20 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	10.08 kips		*Calculated axial load*		
V_r	2.14 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.02 in^2		*Net area of the plate*		
Z_{net}	3.96 in^3		*Plastic modulus of net section*		
V_c	52.61 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	-0.40 in		*Vertical eccentricity*		
M_r	0.15 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	14.35 kips-ft		*Available moment M_c= φ*(F_u* Znet), φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Plate Flexural Buckling				**0.18**	**PASS**

$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**$\varphi = 0.90$**	*(AISC 14^{th} Edition)*		
P	10.08 kips		*Calculated axial load*		
V	2.14 kips		*Calculated shear load*		
L	2.75 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	25.37		*Slenderness ratio*		
F_e	444.52 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2*E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	34.80 ksi		*Compression stress, per eqn E3-2, Fcr = (0.658 ^ (Fy / Fe)) * Fy*		
A_g	3.00 in^2		*Gross area of the plate*		
λ	0.35		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	2.51 in^3		*Section modulus of net section*		
a	2.75 in		*Design eccentricity*		
P_n	104.40 kips		*Compressive capacity, per eqn E4-1, Pn = Fcr_Comp * Ag*		
V_n	32.83 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.18		*Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$*		

Bolt Bearing on Beam		10.30 kips	53.68 kips	**0.19**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**$\varphi = 0.75$**	*(J3-6b)*		
V	2.14 kips		*Applied shear force*		
P	10.08 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	10.30 kips		*Resultant shear force*		
Θ	11.97 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.70 in		*Vertical edge spacing*		
e_h	2.75 in		*Horizontal edge spacing*		
L_{c_boltA}	2.40 in		*Minimum clear distance for the corner edge bolt: $Lc\_boltA = min((e_v /sin(\Theta), (e_h /cos(\Theta)) - d_c$*		
L_{c_boltB}	2.40 in		*Minimum clear distance for the side edge bolts: $Lc\_boltB = min((s_v - 0.5 * d_v /sin(\Theta)), (e_h /cos(\Theta))) - d_c$*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: $Rn\_boltA = min[(1.5* Lc\_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: $Rn\_boltB = min[(1.5* Lc\_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		10.30 kips	53.68 kips	**0.19**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**$\varphi = 0.75$**	*(J3-6b)*		
V	2.14 kips		*Applied shear force*		
P	10.08 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	10.30 kips		*Resultant shear force*		

Θ	11.97 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.00 in		*Vertical edge spacing*		
e_h	1.75 in		*Horizontal edge spacing*		
L_{c_boltA}	1.38 in		*Minimum clear distance for the corner edge bolt:* $Lc\_boltA = \min((e_v / \sin(\Theta)), (e_h / \cos(\Theta)) - d_c$		
L_{c_boltB}	1.38 in		*Minimum clear distance for the side edge bolts:* $Lc\_boltB = \min((s_v - 0.5 * d_v / \sin(\Theta)), (e_h / \cos(\Theta))) - d_c$		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt:* $Rn\_boltA = \min[(1.5 * Lc\_boltA * t * Fu), (3.0 * d_b * t * Fu), R_{n\text{-}bolt}]$		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt:* $Rn\_boltB = \min[(1.5 * Lc\_boltB * t * Fu), (3.0 * d_b * t * Fu), R_{n\text{-}bolt}]$		
$R_{n\text{-}bolt}$	23.86 kips		*Bolt shear strength* $R_{n\text{-}bolt} = F_{nv} * A_{bolt}$		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		10.30 kips	45.03 kips	**0.23**	**PASS**
$R_n = F_{nv} * A_b * N_{bolt} * C$		φ **= 0.75**	*(J3-1)*		
V	2.14 kips		*Applied shear force*		
P	10.08 kips		*Applied axial force*		
$R = (V^2 + P^2)^{0.5}$	10.30 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.84		*Eccentricity coefficient*		
φR_n	45.03 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity		***0.84***			
Elastic method			*(AISC 14th p.7-6)*		
C	0.84		*Coefficient (2.5166 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.40 in		*Vertical eccentricity*		
Ang	11.97		*Angle of force in degrees, relative X axis*		

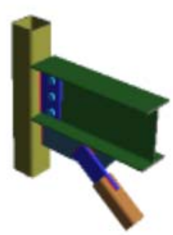
Weld at Column		10.30 kips	78.38 kips	**0.13**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ**=0.75 (AISC 14th Eqn 8-2a)**					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φRn	78.38 kips		*Weld strength*		

HSS Transverse Plastification		10.08 kips	29.68 kips	**0.34**	**PASS**

$R_n = F_y{*}t^2\ /(1-t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1-t_p/B)^{0.5})$		φ = 1.00	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.29 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	5.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	29.68 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	11.57 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	5.00 in		*Column width*		
β	0.17		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.29 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	1.60		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	11.57 kips-ft		*Flexural plastification*		

BR-8 Top Detail: Bot Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x9.38	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x17.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	17.57 kips	*Calculated Shear Load*
Axial Load	10.36 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.36 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	17.30 in		*Min weld segment length*		

Plate Shear Yield		17.57 kips	140.11 kips	**0.13**	PASS
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	6.49 in^2		*Gross area subject to shear*		
φR_n	140.11 kips		*Shear yield strength*		

Plate Shear Rupture		17.57 kips	169.30 kips	**0.10**	PASS
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	6.49 in^2		*Net area subject to shear*		
φR_n	169.30 kips		*Shear rupture strength*		

Plate Axial Yield		10.36 kips	210.16 kips	**0.05**	PASS
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.49 in^2		*Gross area subject to tension*		
φR_n	210.16 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.02**	PASS
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	10.36 kips		*Calculated axial load*		
V_r	17.57 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.49 in^2		*Gross area of the plate*		
Z_{pl}	28.05 in^3		*Plastic modulus of the shear plate*		
P_c	210.16 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	140.11 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	75.73 kips-ft		*Available moment $M_c=\varphi*(F_y*Z)$, $\varphi=0.90$*		
UC	0.02		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.01**	PASS
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	17.57 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	6.49 in^2		*Net area of the plate*		
Z_{net}	28.05 in^3		*Plastic modulus of net section*		
V_c	169.30 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	101.68 kips-ft		*Available moment $M_c= \varphi*(F_u*Znet)$, $\varphi=0.75$*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

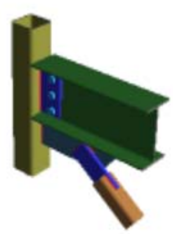
Beam Weld Strength		17.57 kips	115.57 kips	**0.15**	PASS
φRn = 2 * C_1 * α * β * 1.392 * D_{16} * L					
Double Fillet					
1.392 = φ * 0.6 * F_{E70} * $2^{0.5}$/2 * 1/16, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		

L	17.30 in		*Weld length*		
φRn	115.57 kips		*Weld strength*		

Beam Web Yielding		10.36 kips	247.17 kips	**0.04**	**PASS**
R_n = (5 * k + N) * F_y * t_w		**φ = 1.00**	*(J10-2)*		
k	0.66 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	17.30 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.24 in		*Beam web thickness*		
φRn	247.17 kips		*Beam web local yielding*		

BR-8 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x9.38	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x17.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	6.10 kips	*Calculated Shear Load*
Axial Load	5.08 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.29 in		*Column wall thickness*		
B	5.00 in		*Column face width*		
(B - 3 * t) / t	14.18		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.18		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
F_{y-max}	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
t_{p-max}	0.47 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		

D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.29 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	6.00 in		*Min weld segment length*		

Plate Shear Yield		6.10 kips	48.60 kips	**0.13**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.25 in^2		*Gross area subject to shear*		
φR_n	48.60 kips		*Shear yield strength*		

Plate Shear Rupture		6.10 kips	58.72 kips	**0.10**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.25 in^2		*Net area subject to shear*		
φR_n	58.72 kips		*Shear rupture strength*		

Plate Axial Yield		5.08 kips	72.90 kips	**0.07**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.25 in^2		*Gross area subject to tension*		
φR_n	72.90 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.02**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	5.08 kips		*Calculated axial load*		
V_r	6.10 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.25 in^2		*Gross area of the plate*		
Z_{pl}	3.38 in^3		*Plastic modulus of the shear plate*		
P_c	72.90 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	48.60 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	9.11 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.02		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

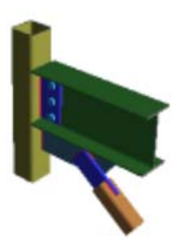
Plate Flexural Rupture				**0.01**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	6.10 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.25 in^2		*Net area of the plate*		
Z_{net}	3.38 in^3		*Plastic modulus of net section*		
V_c	58.72 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	12.23 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		6.10 kips	40.09 kips	**0.15**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					

C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	6.00 in		*Weld length*		
φRn	40.09 kips		*Weld strength*		
HSS Transverse Plastification		5.08 kips	26.31 kips	**0.19**	**PASS**
$R_n = F_y{*}t^2 / (1-t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1-t_p/B)^{0.5})$		φ = 1.00	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.29 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	6.00 in		*Plate length*		
B	5.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	26.31 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	7.79 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	5.00 in		*Column width*		
β	0.15		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.29 in		*Column wall thickness*		
H_b	6.00 in		*Depth of plate*		
η	1.20		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	7.79 kips-ft		*Flexural plastification*		

BR-8 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W10x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x9.38	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x17.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	28.00 kips	*Brace Axial (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Gusset Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				

D	0.19 in		*Weld size*		
D_{max}	0.56 in		*Max Size Allowed*		
t	0.63 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.49 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		28.00 kips	80.56 kips	**0.35**	**PASS**
$P_n = F_y*A_g$		**φ = 0.9**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	5.13 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	23.71		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.49 in^2		*Gross area of plate (Whitmore section)*		
$φP_n$	80.56 kips		*Gusset plate compressive strength*		

Knife Plate Buckling		28.00 kips	72.10 kips	**0.39**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	5.66 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	31.39		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.34 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	290.53 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.18 ksi		*Critical stress (E3-2)*		
$φR_n$	72.10 kips		*Compressive strength*		

Knife Plate Flexure		0.58 kips-ft	0.99 kips-ft	**0.59**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	28.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.63 in		*Thickness of knife plate*		
e	0.50 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) /2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.37 in^3		*Plastic section modulus, $Z = W_p* (t_{plate})^2/4$*		
M_u	0.58 kips-ft		*Required moment demand on knife plate, $M_u=R_u * e / 2$*		
$φM_n$	0.99 kips-ft		*Available flexural strength*		

Knife Plate Interaction				0.91	PASS
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	28.00 kips		*User input brace axial load*		
φP_c	72.10 kips		*Available buckling strength of knife plate*		
M_u	0.58 kips-ft		*Required moment demand on knife plate*		
φM_p	0.99 kips-ft		*Available flexural strength of knife plate*		

Gusset Weld Strength		28.00 kips	36.49 kips	0.77	PASS
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	8.74 in		*Weld length*		
φRn	36.49 kips		*Weld strength*		

Brace Weld Strength		28.00 kips	50.11 kips	0.56	PASS
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		

BR-8 Top Detail: Members Report

Vertical Brace Diagonal Connection

	Beam		***W10x22***
Material			
	Name	A992	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	65.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b_f	5.75 in	*Flange width*
	d	10.20 in	*Overall depth*
	t_w	0.24 in	*Web thickness*
	t_f	0.36 in	*Flange thickness*
	a	6.49 in^2	*Area*
	k_{des}	0.66 in	*Kdes*
	k_{det}	0.94 in	*Kdet*
	k_1	0.63 in	*K1*
Web Hole Type			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	3	*Number of holes per row*
	R_s	3.00 in	*Row Spacing*
	C_s	3.00 in	*Column Spacing*

	Column		***HSS5x5x5***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	5.00 in	*Depth*
	b	5.00 in	*Width*
	a	5.26 in^2	*Area*
	t_{des}	0.29 in	*Wall Thickness*

	Bottom Brace		***HSS3x3x4***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.00 in	*Depth*
	b	3.00 in	*Width*
	a	2.44 in^2	*Area*
	t_{des}	0.23 in	*Wall Thickness*

BR-8 Top Detail: Components Report

Vertical Brace Diagonal Connection

	Plate		***P0.38x4.50x8.00***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	4.50 in	*Width*
	t	0.38 in	*Thickness*
Hole			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	3	*Number of holes per row*
	R_s	3.00 in	*Row Spacing*
	C_s	3.00 in	*Column Spacing*

	Knife Plate		***P0.63x3.75x9.38***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.75 in	*Width*
	t	0.63 in	*Thickness*

	Bottom Gusset		***P0.38x6.00x17.30***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*

F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	6.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	4.00 in	*Horiz. Clip*
V_clip	5.51 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

RISAConnection version 7.0.1

02/07/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

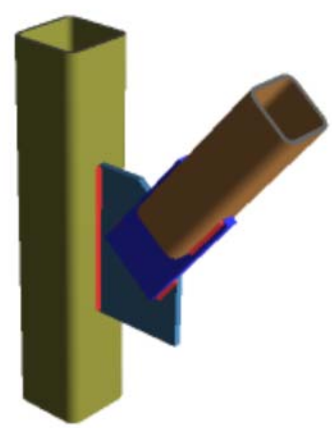
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

BR-9 Bottom Detail	**PASS(UC-1.0)**
BR-9 Top Detail	**PASS(UC-0.9)**

BR-9 Bottom Detail: 3D View

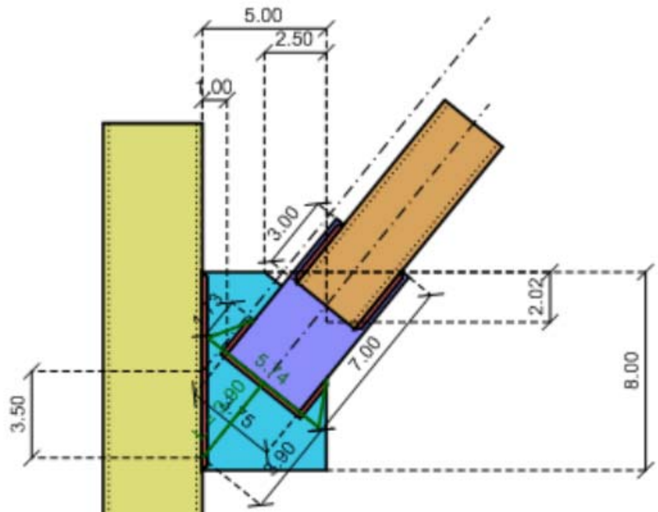
Knee Brace Connection



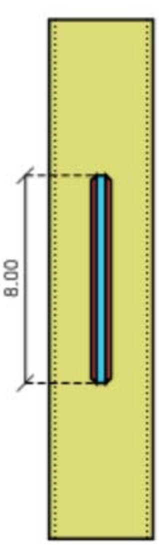
BR-9 Bottom Detail: 2D Views

Knee Brace Connection

Left view

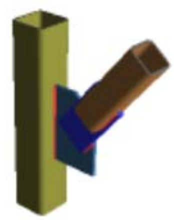


Front view



BR-9 Bottom Detail: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:

Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x5.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x7.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	28.00 kips	*Brace Axial (compression)*
Shear Load	21.76 kips	*Calculated Shear Load*
Axial Load	17.62 kips	*Calculated Axial Load (compression)*
Moment Load	5.14 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
$t_{p\text{-}max}$	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	3.50 in	*Vertical brace workpoint offset*		
d_{gusset}	8.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		
t_{min}	0.23 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	8.00 in	*Min weld segment length*		
Gusset Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			

D	0.19 in		*Weld size*		
D_{max}	0.56 in		*Max Size Allowed*		
t	0.63 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	1.73 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Plate Shear Yield		21.76 kips	64.80 kips	**0.34**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		

Plate Shear Rupture		21.76 kips	78.30 kips	**0.28**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.00 in^2		*Net area subject to shear*		
φR_n	78.30 kips		*Shear rupture strength*		

Plate Axial Yield		17.62 kips	97.20 kips	**0.18**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.36**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	17.62 kips		*Calculated axial load*		
V_r	21.76 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	5.14 kips-ft		*Calculated moment*		
M_c	16.20 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		
UC	0.36		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture			**0.13**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		*(Eq.10-5)*		
P_r	0.00 kips	*Calculated axial load*		
V_r	21.76 kips	*Calculated shear load*		
F_u	58.00 ksi	*Minimum tensile stress of material*		
A_n	3.00 in^2	*Net area of the plate*		
Z_{net}	6.00 in^3	*Plastic modulus of net section*		
V_c	78.30 kips	*Available shear strength (see check 'Shear Rupture')*		
M_r	5.14 kips-ft	*Calculated moment*		
M_c	21.75 kips-ft	*Available moment $M_c = \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.13	*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		6.39 kips/in	6.68 kips/in	**0.96**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
V	21.76 kips		*Shear Load*		
P	17.62 kips		*Axial Load*		
M	5.14 kips-ft		*Moment*		
e_{eff}	2.83 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	6.39 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		28.00 kips	63.21 kips	**0.44**	**PASS**
$P_n = F_{cr}*A_g$		**φ = 0.9**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	3.90 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	43.28		*Plate slenderness*		
F_{cr}	32.62 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.15 in^2		*Gross area of plate (Whitmore section)*		
φP_n	63.21 kips		*Gusset plate compressive strength*		

Gusset Weld Strength		28.00 kips	30.07 kips	**0.93**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	7.20 in		*Weld length*		
φRn	30.07 kips		*Weld strength*		

Brace Weld Strength		28.00 kips	50.11 kips	**0.56**	**PASS**
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table*		

			8-3)		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		
Knife Plate Flexure		0.58 kips-ft	0.99 kips-ft	**0.59**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	28.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.63 in		*Thickness of knife plate*		
e	0.50 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) /2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.37 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.58 kips-ft		*Required moment demand on knife plate, $M_u=R_u * e / 2$*		
φM_n	0.99 kips-ft		*Available flexural strength*		
Knife Plate Interaction				**0.99**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	28.00 kips		*User input brace axial load*		
φP_c	60.60 kips		*Available buckling strength of knife plate*		
M_u	0.58 kips-ft		*Required moment demand on knife plate*		
φM_p	0.99 kips-ft		*Available flexural strength of knife plate*		
HSS Transverse Plastification		17.62 kips	21.52 kips	**0.82**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		
HSS Flexural Plastification		5.14 kips-ft	8.21 kips-ft	**0.63**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	5.14 kips-ft		*Required flexural plastification*		
φM_n	8.21 kips-ft		*Flexural plastification*		
Knife Plate Buckling		28.00 kips	60.60 kips	**0.46**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	2.10		*Effective length factor*		
L	5.62 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	65.47		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.34 in^2		*Gross area subject to compression*		

E	29000.00 ksi	*Modulus of elasticity*
F_e	66.77 ksi	*Elastic buckling stress (E3-4)*
F_{cr}	28.73 ksi	*Critical stress (E3-2)*
φR_n	60.60 kips	*Compressive strength*

BR-9 Bottom Detail: Members Report

Knee Brace Connection

	Column		***HSS4x4x4***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	4.00 in	*Depth*
	b	4.00 in	*Width*
	a	3.37 in^2	*Area*
	t_{des}	0.23 in	*Wall Thickness*

	Brace		***HSS3x3x4***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.00 in	*Depth*
	b	3.00 in	*Width*
	a	2.44 in^2	*Area*
	t_{des}	0.23 in	*Wall Thickness*

BR-9 Bottom Detail: Components Report

Knee Brace Connection

	Gusset		***P0.38x5.00x8.00***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	5.00 in	*Width*
	t	0.38 in	*Thickness*
Clip			
	H_clip	2.50 in	*Horiz. Clip*
	V_clip	2.02 in	*Vert. Clip*

	Knife Plate		***P0.63x3.75x7.00***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.75 in	*Width*
	t	0.63 in	*Thickness*

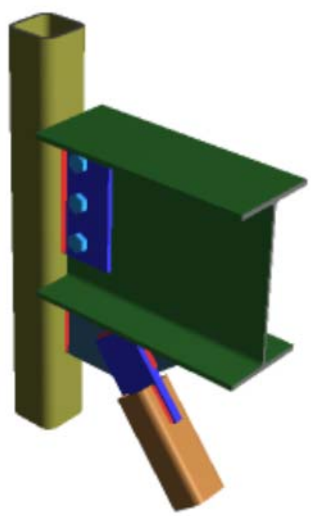
Column Weld	***E70***	
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Gusset Plate Weld	***E70***	
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld	***E70***	
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

BR-9 Top Detail: 3D View

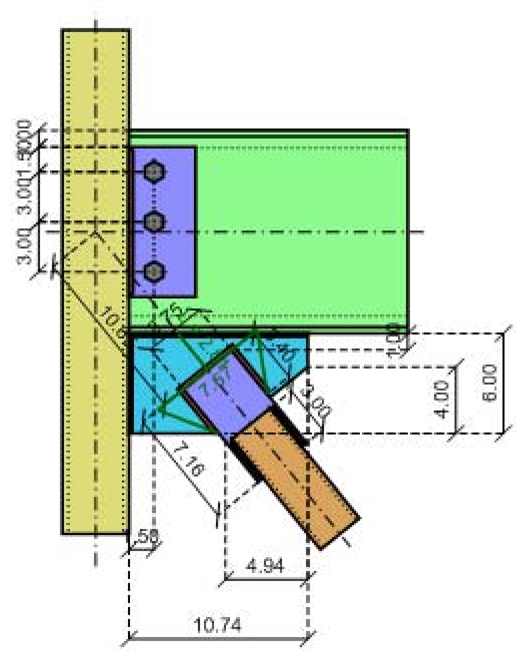
Vertical Brace Diagonal Connection



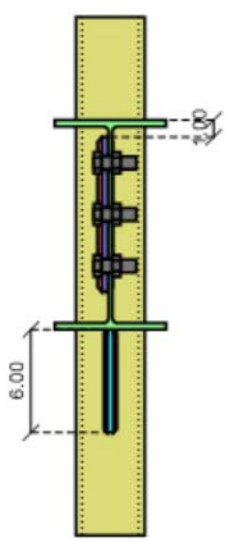
BR-9 Top Detail: 2D Views

Vertical Brace Diagonal Connection

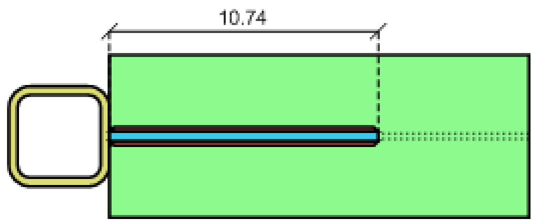
Side view



Front view

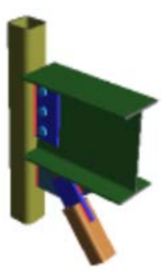


Bottom view



BR-9 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.63x3.75x7.16	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x10.74	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	16.50 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	37.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	28.00 kips	*User Input Bottom Brace Axial Force (compression)*

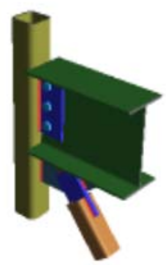
Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	Bolt Shear at Beam	**0.24**	**PASS**
Bottom Gusset/Beam connection	Beam Weld Strength	**0.18**	**PASS**
Bottom Gusset/Column connection	Column Weld Strength	**0.18**	**PASS**
Bottom Gusset/Brace connection	Knife Plate Interaction	**0.89**	**PASS**

BR-9 Top Detail: Beam/Column Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.63x3.75x7.16	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x10.74	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	1.91 kips	*Calculated Shear Load*
Total Axial Load	9.78 kips	*Calculated Axial Load (compression)*
Column Force	37.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.35 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	8.46	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	11.46	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.56 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.23 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		

t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.35 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	9.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	9.00 in		*Max weld segment length*		
Beam Shear Yield		1.91 kips	84.18 kips	**0.02**	**PASS**
$R_n = 0.6 * F_y * A_{gv} * C_v$		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	84.18 kips		*Shear yield strength*		
Plate Shear Yield		1.91 kips	72.90 kips	**0.03**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.38 in^2		*Gross area subject to shear*		
φR_n	72.90 kips		*Shear yield strength*		
Beam Shear Rupture		1.91 kips	64.42 kips	**0.03**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.20 in^2		*Net area subject to shear*		
φR_n	64.42 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		1.91 kips	62.40 kips	**0.03**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.39 in^2		*Net area subject to shear*		
φR_n	62.40 kips		*Shear rupture strength*		
Beam Axial Yield		9.78 kips	344.25 kips	**0.03**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	7.65 in^2		*Gross area subject to tension*		
φR_n	344.25 kips		*Tensile yield strength*		
Plate Axial Yield		9.78 kips	109.35 kips	**0.09**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to tension*		
φR_n	109.35 kips		*Tensile yield strength*		
Beam Block Shear		1.91 kips	81.55 kips	**0.02**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		**φ = 0.75**	*(J4-5)*		
A_{gv}	3.10 in^2		*Gross area subject to shear*		
A_{nv}	2.59 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.24 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	81.55 kips		*Block shear strength*		
Plate Block Shear at Beam		1.91 kips	79.21 kips	**0.02**	**PASS**

R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		φ = 0.75	*(J4-5)*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
A_{nv}	1.99 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.77 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	79.21 kips		*Block shear strength*		
Compression Buckling of the Plate		9.78 kips	109.35 kips	**0.09**	**PASS**
R_n = F_y * A_g		φ = 0.9	*(J4-6)*		
K	1.00		*Effective length factor*		
L	1.50 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	13.84		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	3.38 in^2		*Gross area subject to compression*		
φR_n	109.35 kips		*Compressive strength*		
Plate Flexural Yield				**0.02**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	9.78 kips		*Calculated axial load*		
V_r	1.91 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area of the plate*		
Z_{pl}	7.59 in^3		*Plastic modulus of the shear plate*		
P_c	109.35 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	72.90 kips		*Available shear strength (see check 'Shear Yield')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.60 in		*Vertical eccentricity*		
M_r	0.73 kips-ft		*Moment due to eccentricity =* V_r**ex +* P_r**ey*		
M_c	20.50 kips-ft		*Available moment* $M_c=\varphi*(F_y* Z)$, $\varphi=0.90$		
UC	0.02		*Unity check per interaction equation,* $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	9.78 kips		*Calculated axial load*		
V_r	1.91 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.39 in^2		*Net area of the plate*		
Z_{net}	5.55 in^3		*Plastic modulus of net section*		
V_c	62.40 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.60 in		*Vertical eccentricity*		
M_r	0.73 kips-ft		*Moment due to eccentricity =* V_r**ex +* P_r**ey*		
M_c	20.13 kips-ft		*Available moment* $M_c= \varphi*(F_u* Znet)$, $\varphi=0.75$		
UC	0.00		*Unity check per interaction equation,* $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		
Plate Flexural Buckling				**0.11**	**PASS**
P / (P_n*φ) + V / (V_n*φ) <= 1.0		φ = 0.90	*(AISC 14th Edition)*		
P	9.78 kips		*Calculated axial load*		
V	1.91 kips		*Calculated shear load*		

L	1.50 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	13.84		*Slenderness ratio*		
F_e	1494.08 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	35.64 ksi		*Compression stress = F_y when KL/r <= 25, per J4.4*		
A_g	3.38 in^2		*Gross area of the plate*		
λ	0.33		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	3.74 in^3		*Section modulus of net section*		
a	1.50 in		*Design eccentricity*		
P_n	121.50 kips		*Compressive capacity, per eqn J4-1, Pn = Fy * Ag*		
V_n	89.67 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.11		*Unity check per interaction equation, P/(Pn*Φ) + V/(Vn*Φ) <= 1*		

Bolt Bearing on Beam		9.97 kips	53.68 kips	**0.19**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	1.91 kips		*Applied shear force*		
P	9.78 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	9.97 kips		*Resultant shear force*		
Θ	11.07 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	3.70 in		*Vertical edge spacing*		
e_h	1.50 in		*Horizontal edge spacing*		
L_{c_boltA}	1.12 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	1.12 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), $R_{n\text{-}bolt}$]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), $R_{n\text{-}bolt}$]*		
$R_{n\text{-}bolt}$	23.86 kips		*Bolt shear strength $R_{n\text{-}bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		9.97 kips	53.68 kips	**0.19**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	1.91 kips		*Applied shear force*		
P	9.78 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	9.97 kips		*Resultant shear force*		
Θ	11.07 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		

d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.50 in		*Vertical edge spacing*		
e_h	2.50 in		*Horizontal edge spacing*		
L_{c_boltA}	2.14 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.14 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength R_{n-bolt}=F_{nv}*A_{bolt}*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		9.97 kips	41.61 kips	**0.24**	**PASS**
$R_n = F_{nv} * A_b * N_{bolt} * C$		**φ = 0.75**	*(J3-1)*		
V	1.91 kips		*Applied shear force*		
P	9.78 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	9.97 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.78		*Eccentricity coefficient*		
φR_n	41.61 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***0.78***		
Elastic method			*(AISC 14^{th} p.7-6)*		
C	0.78		*Coefficient (2.3253 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.60 in		*Vertical eccentricity*		
Ang	11.07		*Angle of force in degrees, relative X axis*		

Weld at Column		9.97 kips	88.18 kips	**0.11**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	9.00 in		*Weld length*		
φRn	88.18 kips		*Weld strength*		

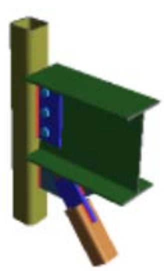
HSS Transverse Plastification		9.78 kips	51.36 kips	**0.19**	**PASS**
$R_n = F_y * t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		

F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	9.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	51.36 kips		*Transverse plastification*		

HSS Flexural Plastification		0.00 kips-ft	22.54 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	9.00 in		*Depth of plate*		
η	2.25		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	22.54 kips-ft		*Flexural plastification*		

BR-9 Top Detail: Bot Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.63x3.75x7.16	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x10.74	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	12.84 kips	*Calculated Shear Load*
Axial Load	14.59 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

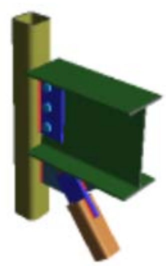
Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	10.74 in		*Min weld segment length*		
Plate Shear Yield		12.84 kips	86.98 kips	**0.15**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		

A_{gv}	4.03 in^2		*Gross area subject to shear*		
φR_n	86.98 kips		*Shear yield strength*		
Plate Shear Rupture		12.84 kips	105.10 kips	**0.12**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	4.03 in^2		*Net area subject to shear*		
φR_n	105.10 kips		*Shear rupture strength*		
Plate Axial Yield		14.59 kips	130.47 kips	**0.11**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.03 in^2		*Gross area subject to tension*		
φR_n	130.47 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.03**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14^{th} Eq.10-5)*		
P_r	14.59 kips		*Calculated axial load*		
V_r	12.84 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.03 in^2		*Gross area of the plate*		
Z_{pl}	10.81 in^3		*Plastic modulus of the shear plate*		
P_c	130.47 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	86.98 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	29.19 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, $\varphi=0.90$*		
UC	0.03		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.01**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	12.84 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	4.03 in^2		*Net area of the plate*		
Z_{net}	10.81 in^3		*Plastic modulus of net section*		
V_c	105.10 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	39.19 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, $\varphi=0.75$*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Beam Weld Strength		12.84 kips	71.75 kips	**0.18**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	10.74 in		*Weld length*		
φR_n	71.75 kips		*Weld strength*		

		Required	Available	Unity Check	Result
Beam Web Yielding		14.59 kips	162.59 kips	**0.09**	**PASS**
R_n = (5 * k + N) * F_y * t_w		**φ = 1.00**	*(J10-2)*		
k	0.68 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	10.74 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.23 in		*Beam web thickness*		
φR_n	162.59 kips		*Beam web local yielding*		

BR-9 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.63x3.75x7.16	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x10.74	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	7.17 kips	*Calculated Shear Load*
Axial Load	4.78 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	8.46		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	11.46		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
F_{y-max}	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
t_{p-max}	0.56 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.35 in		*Controlling member thickness*		

Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	6.00 in		*Min weld segment length*		

Plate Shear Yield		7.17 kips	48.60 kips	**0.15**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.25 in^2		*Gross area subject to shear*		
φR_n	48.60 kips		*Shear yield strength*		

Plate Shear Rupture		7.17 kips	58.72 kips	**0.12**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.25 in^2		*Net area subject to shear*		
φR_n	58.72 kips		*Shear rupture strength*		

Plate Axial Yield		4.78 kips	72.90 kips	**0.07**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.25 in^2		*Gross area subject to tension*		
φR_n	72.90 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.03**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	4.78 kips		*Calculated axial load*		
V_r	7.17 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.25 in^2		*Gross area of the plate*		
Z_{pl}	3.38 in^3		*Plastic modulus of the shear plate*		
P_c	72.90 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	48.60 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	9.11 kips-ft		*Available moment $M_c = \varphi*(F_y* Z)$, φ=0.90*		
UC	0.03		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

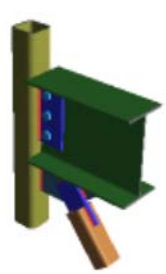
Plate Flexural Rupture				**0.01**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	7.17 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.25 in^2		*Net area of the plate*		
Z_{net}	3.38 in^3		*Plastic modulus of net section*		
V_c	58.72 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	12.23 kips-ft		*Available moment $M_c = \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		7.17 kips	40.09 kips	**0.18**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement*		

			of AISC 14th Eqn 9-2)		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	6.00 in		*Weld length*		
φRn	40.09 kips		*Weld strength*		
HSS Transverse Plastification		4.78 kips	42.09 kips	**0.11**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	6.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	42.09 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	12.32 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	6.00 in		*Depth of plate*		
η	1.50		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	12.32 kips-ft		*Flexural plastification*		

BR-9 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.C Rect	F_y = 50.00 ksi	F_u = 62.00 ksi
Knife Plate	P0.63x3.75x7.16	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.00x10.74	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	28.00 kips	*Brace Axial (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Gusset Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.25 in		*Weld size*		
D_{max}	0.56 in		*Max Size Allowed*		
t	0.63 in		*Min shelf dimension*		

Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	3.40 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		28.00 kips	93.22 kips	**0.30**	**PASS**
$P_n = F_y * A_g$		**φ = 0.9**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	2.81 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	12.96		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.88 in^2		*Gross area of plate (Whitmore section)*		
φP_n	93.22 kips		*Gusset plate compressive strength*		

Knife Plate Buckling		28.00 kips	75.94 kips	**0.37**	**PASS**
$R_n = F_y * A_g$		**φ = 0.9**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	3.06 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	16.97		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	2.34 in^2		*Gross area subject to compression*		
φR_n	75.94 kips		*Compressive strength*		

Knife Plate Flexure		0.58 kips-ft	0.99 kips-ft	**0.59**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	28.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.63 in		*Thickness of knife plate*		
e	0.50 in		*Eccentricity, $e = (t_{plate} + t_{gusset})/2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.37 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.58 kips-ft		*Required moment demand on knife plate, $M_u = R_u * e / 2$*		
φM_n	0.99 kips-ft		*Available flexural strength*		

Knife Plate Interaction				**0.89**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	28.00 kips		*User input brace axial load*		
φP_c	75.94 kips		*Available buckling strength of knife plate*		
M_u	0.58 kips-ft		*Required moment demand on knife plate*		

φM_p	0.99 kips-ft	*Available flexural strength of knife plate*		

Gusset Weld Strength	28.00 kips	58.71 kips	**0.48**	PASS

$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$
Single Fillet
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)

C_1	1.00	*Electrode strength coefficient (AISC 14th table 8-3)*
α	1.00	*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*
D_{16}	4.00	*Weld fillet size in sixteenths of an inch*
L	10.54 in	*Weld length*
φRn	58.71 kips	*Weld strength*

Brace Weld Strength	28.00 kips	50.11 kips	**0.56**	PASS

$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$
Single Fillet
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)

C_1	1.00	*Electrode strength coefficient (AISC 14th table 8-3)*
α	1.00	*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*
D_{16}	3.00	*Weld fillet size in sixteenths of an inch*
L	3.00 in	*Weld length*
φRn	50.11 kips	*Weld strength*

BR-9 Top Detail: Members Report

Vertical Brace Diagonal Connection

	Beam		***W12x26***
Material			
	Name	A992	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	65.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b_f	6.49 in	*Flange width*
	d	12.20 in	*Overall depth*
	t_w	0.23 in	*Web thickness*
	t_f	0.38 in	*Flange thickness*
	a	7.65 in^2	*Area*
	k_{des}	0.68 in	*Kdes*
	k_{det}	1.06 in	*Kdet*
	k_1	0.75 in	*K1*
Web Hole Type			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	3	*Number of holes per row*
	R_s	3.00 in	*Row Spacing*
	C_s	3.00 in	*Column Spacing*

	Column		***HSS4x4x6***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*

E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	4.78 in^2	*Area*
t_{des}	0.35 in	*Wall Thickness*

Bottom Brace		***HSS3x3x4***
Material		
Name	A500 Gr.C Rect	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	62.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-9 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.00x9.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Knife Plate		***P0.63x3.75x7.16***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*
t	0.63 in	*Thickness*

Bottom Gusset		***P0.38x6.00x10.74***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	6.00 in	*Width*
t	0.38 in	*Thickness*

Clip

H_clip	4.94 in	*Horiz. Clip*
V_clip	4.00 in	*Vert. Clip*

Column Weld ***E70***

Weld Properties

Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts ***3/4" A325***

Bolt Properties

Type	A325	
d	0.75 in	*Diameter*

Strength

S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Gusset Plate Weld ***E70***

Weld Properties

Type	Single Fillet	
Fillet Size	0.25 in	

Knife Plate Brace Weld ***E70***

Weld Properties

Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld ***E70***

Weld Properties

Type	Double Fillet	
Fillet Size	0.19 in	

Column Weld ***E70***

Weld Properties

Type	Double Fillet	
Fillet Size	0.19 in	

RISAConnection version 7.0.1

02/07/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

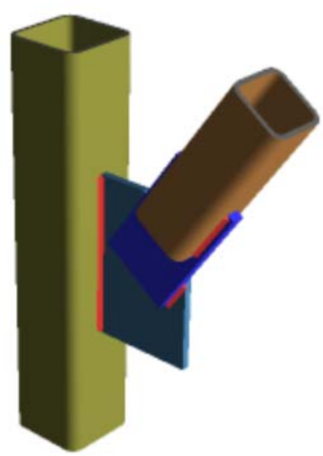
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

BR-10 Bottom Detail	**PASS(UC-1.0)**
BR-10 Top Detail	**PASS(UC-0.9)**

BR-10 Bottom Detail: 3D View

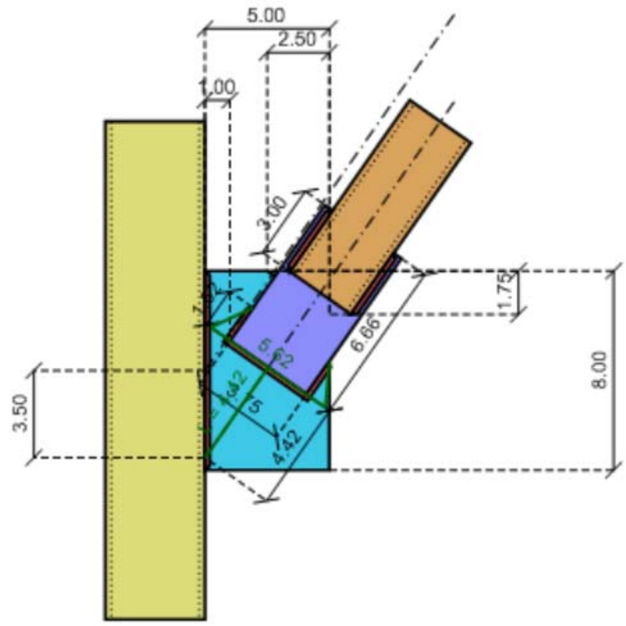
Knee Brace Connection



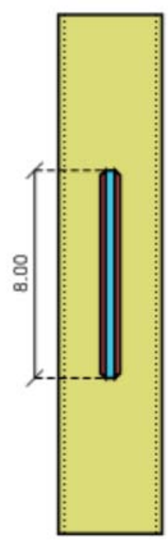
BR-10 Bottom Detail: 2D Views

Knee Brace Connection

Left view

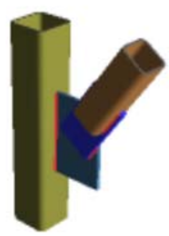


Front view



BR-10 Bottom Detail: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:

Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x5.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x6.66	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	28.00 kips	*Brace Axial (compression)*
Shear Load	22.94 kips	*Calculated Shear Load*
Axial Load	16.06 kips	*Calculated Axial Load (compression)*
Moment Load	4.68 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	3.50 in	*Vertical brace workpoint offset*		
d_{gusset}	8.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		
t_{min}	0.23 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	8.00 in	*Min weld segment length*		
Gusset Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		

Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.56 in		*Max Size Allowed*		
t	0.63 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	1.62 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Plate Shear Yield		22.94 kips	64.80 kips	**0.35**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		

Plate Shear Rupture		22.94 kips	78.30 kips	**0.29**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.00 in^2		*Net area subject to shear*		
φR_n	78.30 kips		*Shear rupture strength*		

Plate Axial Yield		16.06 kips	97.20 kips	**0.17**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.33**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	16.06 kips		*Calculated axial load*		
V_r	22.94 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	4.68 kips-ft		*Calculated moment*		
M_c	16.20 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.33		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.13**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	22.94 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.00 in^2		*Net area of the plate*		
Z_{net}	6.00 in^3		*Plastic modulus of net section*		
V_c	78.30 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	4.68 kips-ft		*Calculated moment*		
M_c	21.75 kips-ft		*Available moment $M_c = \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.13		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		6.00 kips/in	6.68 kips/in	**0.90**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
V	22.94 kips		*Shear Load*		
P	16.06 kips		*Axial Load*		
M	4.68 kips-ft		*Moment*		
e_{eff}	2.45 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	6.00 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		28.00 kips	60.16 kips	**0.47**	**PASS**
$P_n = F_{cr}*A_g$		φ = 0.9	*(E3-1)*		
K	1.20		*Effective length factor*		
L	4.42 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	49.01		*Plate slenderness*		
F_{cr}	31.72 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.11 in^2		*Gross area of plate (Whitmore section)*		
φP_n	60.16 kips		*Gusset plate compressive strength*		

Gusset Weld Strength		28.00 kips	29.18 kips	**0.96**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	6.99 in		*Weld length*		
φRn	29.18 kips		*Weld strength*		

Brace Weld Strength		28.00 kips	50.11 kips	**0.56**	**PASS**
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table*		

			8-3)		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φR_n	50.11 kips		*Weld strength*		
Knife Plate Flexure		0.58 kips-ft	0.99 kips-ft	**0.59**	**PASS**
$M_n = F_y * Z$		**$\varphi = 0.9$**	*(F2-1)*		
R_u	28.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.63 in		*Thickness of knife plate*		
e	0.50 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) / 2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.37 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2 / 4$*		
M_u	0.58 kips-ft		*Required moment demand on knife plate, $M_u = R_u * e / 2$*		
φM_n	0.99 kips-ft		*Available flexural strength*		
Knife Plate Interaction				**0.97**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	28.00 kips		*User input brace axial load*		
φP_c	63.00 kips		*Available buckling strength of knife plate*		
M_u	0.58 kips-ft		*Required moment demand on knife plate*		
φM_p	0.99 kips-ft		*Available flexural strength of knife plate*		
HSS Transverse Plastification		16.06 kips	21.52 kips	**0.75**	**PASS**
$R_n = F_y * t^2 / (1 - t_p/B) * (2 l_b/B + 4 * Q_f * (1 - t_p/B)^{0.5})$		**$\varphi = 1.00$**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		
HSS Flexural Plastification		4.68 kips-ft	8.21 kips-ft	**0.57**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**$\varphi = 1.0$**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	4.68 kips-ft		*Required flexural plastification*		
φM_n	8.21 kips-ft		*Flexural plastification*		
Knife Plate Buckling		28.00 kips	63.00 kips	**0.44**	**PASS**
$R_n = F_{cr} * A_g$		**$\varphi = 0.9$**	*(E3-1)*		
K	2.10		*Effective length factor*		
L	5.12 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	59.57		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.34 in^2		*Gross area subject to compression*		

E	29000.00 ksi	*Modulus of elasticity*
F_e	80.66 ksi	*Elastic buckling stress (E3-4)*
F_{cr}	29.87 ksi	*Critical stress (E3-2)*
φR_n	63.00 kips	*Compressive strength*

BR-10 Bottom Detail: Members Report

Knee Brace Connection

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Brace		***HSS3x3x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-10 Bottom Detail: Components Report

Knee Brace Connection

Gusset		***P0.38x5.00x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	5.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	2.50 in	*Horiz. Clip*
V_clip	1.75 in	*Vert. Clip*

Knife Plate		***P0.63x3.75x6.66***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*

t	0.63 in	*Thickness*

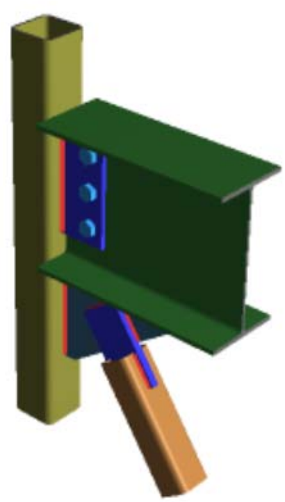
Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

BR-10 Top Detail: 3D View

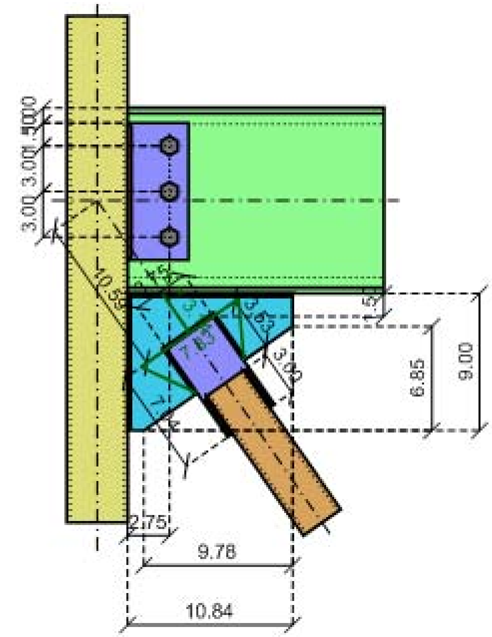
Vertical Brace Diagonal Connection



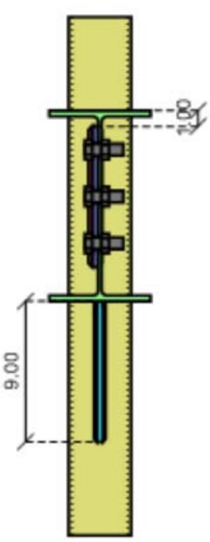
BR-10 Top Detail: 2D Views

Vertical Brace Diagonal Connection

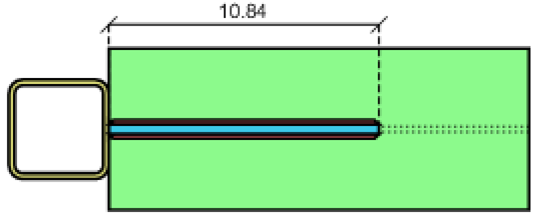
Side view



Front view

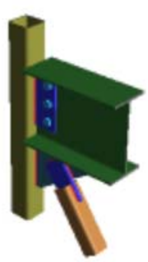


Bottom view



BR-10 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x7.04	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x9.00x10.84	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	28.00 kips	*User Input Bottom Brace Axial Force (compression)*

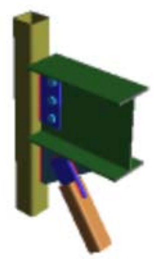
Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	HSS Transverse Plastification	**0.41**	**PASS**
Bottom Gusset/Beam connection	Beam Weld Strength	**0.16**	**PASS**
Bottom Gusset/Column connection	HSS Transverse Plastification	**0.19**	**PASS**
Bottom Gusset/Brace connection	Knife Plate Interaction	**0.89**	**PASS**

BR-10 Top Detail: Beam/Column Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x7.04	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x9.00x10.84	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-11.20 kips	*Calculated Shear Load*
Total Axial Load	9.33 kips	*Calculated Axial Load (compression)*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.23 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		

t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	9.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	9.00 in		*Max weld segment length*		
Beam Shear Yield		11.20 kips	84.18 kips	**0.13**	**PASS**
$R_n = 0.6 * F_y * A_{gv} * C_v$		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	84.18 kips		*Shear yield strength*		
Plate Shear Yield		11.20 kips	72.90 kips	**0.15**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.38 in^2		*Gross area subject to shear*		
φR_n	72.90 kips		*Shear yield strength*		
Beam Shear Rupture		11.20 kips	64.42 kips	**0.17**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.20 in^2		*Net area subject to shear*		
φR_n	64.42 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		11.20 kips	62.40 kips	**0.18**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.39 in^2		*Net area subject to shear*		
φR_n	62.40 kips		*Shear rupture strength*		
Beam Axial Yield		9.33 kips	344.25 kips	**0.03**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	7.65 in^2		*Gross area subject to tension*		
φR_n	344.25 kips		*Tensile yield strength*		
Plate Axial Yield		9.33 kips	109.35 kips	**0.09**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to tension*		
φR_n	109.35 kips		*Tensile yield strength*		
Beam Block Shear		11.20 kips	116.94 kips	**0.10**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		**φ = 0.75**	*(J4-5)*		
A_{gv}	4.04 in^2		*Gross area subject to shear*		
A_{nv}	3.54 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.53 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	116.94 kips		*Block shear strength*		
Plate Block Shear at Beam		11.20 kips	58.82 kips	**0.19**	**PASS**

R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
A_{nv}	1.99 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.30 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	58.82 kips		*Block shear strength*		

Compression Buckling of the Plate		9.33 kips	105.71 kips	**0.09**	**PASS**
R_n = F_{cr} * A_g		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	2.75 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	25.37		*Plate slenderness check from J4-6*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	444.52 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.80 ksi		*Critical stress (E3-2)*		
φR_n	105.71 kips		*Compressive strength*		

Plate Flexural Yield				**0.06**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	9.33 kips		*Calculated axial load*		
V_r	-11.20 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area of the plate*		
Z_{pl}	7.59 in^3		*Plastic modulus of the shear plate*		
P_c	109.35 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	72.90 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.60 in		*Vertical eccentricity*		
M_r	-2.10 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.50 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		
UC	0.06		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.04**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	9.33 kips		*Calculated axial load*		
V_r	-11.20 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.39 in^2		*Net area of the plate*		
Z_{net}	5.55 in^3		*Plastic modulus of net section*		
V_c	62.40 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.60 in		*Vertical eccentricity*		
M_r	-2.10 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.13 kips-ft		*Available moment M_c= φ*(F_u* Znet), φ=0.75*		
UC	0.04		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Plate Flexural Buckling				**0.34**	**PASS**

P / (Pn*φ) + V / (Vn*φ) <= 1.0		**φ = 0.90**	*(AISC 14^{th} Edition)*		
P	9.33 kips		*Calculated axial load*		
V	11.20 kips		*Calculated shear load*		
L	2.75 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	25.37		*Slenderness ratio*		
F_e	444.52 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	34.80 ksi		*Compression stress, per eqn E3-2, Fcr = (0.658 ^ (Fy / Fe)) * Fy*		
A_g	3.38 in^2		*Gross area of the plate*		
λ	0.33		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	3.74 in^3		*Section modulus of net section*		
a	2.75 in		*Design eccentricity*		
P_n	117.45 kips		*Compressive capacity, per eqn E4-1, Pn = Fcr_Comp * Ag*		
V_n	48.91 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.34		*Unity check per interaction equation, P/(Pn*Φ) + V/(Vn*Φ) <= 1*		

Bolt Bearing on Beam		14.57 kips	53.68 kips	**0.27**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-11.20 kips		*Applied shear force*		
P	9.33 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	14.57 kips		*Resultant shear force*		
Θ	50.21 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	3.70 in		*Vertical edge spacing*		
e_h	2.75 in		*Horizontal edge spacing*		
L_{c_boltA}	3.89 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.97 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
$φR_n$	53.68 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		14.57 kips	53.68 kips	**0.27**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-11.20 kips		*Applied shear force*		
P	9.33 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	14.57 kips		*Resultant shear force*		

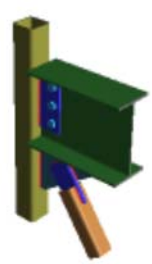
Θ	50.21 degrees		Angle between the resultant shear force and horizontal		
d_b	0.75 in		Bolt diameter		
d_v	0.81 in		Slotted hole vertical dimension		
d_h	0.81 in		Slotted hole horizontal dimension		
d_c	0.41 in		Distance from center of bolt to the edge of the hole		
F_u	58.00 ksi		Minimum tensile stress of material		
s_v	3.00 in		Vertical bolt spacing		
s_h	0.00 in		Horizontal bolt spacing		
e_v	1.50 in		Vertical edge spacing		
e_h	1.25 in		Horizontal edge spacing		
L_{c_boltA}	1.55 in		Minimum clear distance for the corner edge bolt: $Lc\_boltA = \min((e_v /\sin(\Theta)), (e_h /\cos(\Theta)) - d_c$		
L_{c_boltB}	1.55 in		Minimum clear distance for the side edge bolts: $Lc\_boltB = \min((s_v - 0.5 * d_v /\sin(\Theta)), (e_h /\cos(\Theta))) - d_c$		
R_{n_boltA}	23.86 kips		Available bearing strength for the corner edge bolt: $Rn\_boltA = \min[(1.5* Lc\_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n\text{-}bolt}]$		
R_{n_boltB}	23.86 kips		Available bearing strength for each side edge bolt: $Rn\_boltB = \min[(1.5* Lc\_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n\text{-}bolt}]$		
$R_{n\text{-}bolt}$	23.86 kips		Bolt shear strength $R_{n\text{-}bolt}=F_{nv}*A_{bolt}$		
F_{nv}	54.00 ksi		Nominal shear stress of bolt		
φR_n	53.68 kips		Total bolt bearing strength		
Bolt Shear at Beam		14.57 kips	47.40 kips	**0.31**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		φ **= 0.75**	(J3-1)		
V	-11.20 kips		Applied shear force		
P	9.33 kips		Applied axial force		
$R=(V^2 + P^2)^{0.5}$	14.57 kips		Resultant force in bolts		
F_{nv}	54.00 ksi		Shear stress N type		
A_b	0.44 in^2		Area of bolt		
N_{bolt}	3		Number of bolts		
C	0.88		Eccentricity coefficient		
φR_n	47.40 kips		Bolt shear rupture strength		
Bolt Group Eccentricity			***0.88***		
Elastic method			(AISC 14th p.7-6)		
C	0.88		Coefficient (2.6490 / 3)		
N_{rows}	1		Number of rows of bolts		
N_{cols}	3		Number of bolts per row		
D_x	0.00 in		Horizontal bolt spacing		
D_y	3.00 in		Vertical bolt spacing		
E_x	0.00 in		Horizontal eccentricity		
E_y	0.60 in		Vertical eccentricity		
Ang	50.21		Angle of force in degrees, relative X axis		
Weld at Column		14.57 kips	88.18 kips	**0.17**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		Electrode strength coefficient (AISC 14th table 8-3)		
α	0.88		Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)		
D_{16}	4.00		Weld fillet size in sixteenths of an inch		
L	9.00 in		Weld length		
φR_n	88.18 kips		Weld strength		
HSS Transverse Plastification		9.33 kips	22.89 kips	**0.41**	**PASS**

$R_n = F_y{*}t^2 / (1-t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*	
F_y	46.00 ksi		*Column yield strength*	
t	0.23 in		*Column wall thickness*	
t_p	0.38 in		*Plate thickness*	
l_b	9.00 in		*Plate length*	
B	4.00 in		*Column width*	
Q_f	1.00		*User input column stress interaction parameter*	
φR_n	22.89 kips		*Transverse plastification*	

HSS Flexural Plastification		0.00 kips-ft	10.05 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	9.00 in		*Depth of plate*		
η	2.25		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	10.05 kips-ft		*Flexural plastification*		

BR-10 Top Detail: Bot Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x7.04	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x9.00x10.84	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	11.73 kips	*Calculated Shear Load*
Axial Load	13.20 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	10.84 in		*Min weld segment length*		

Plate Shear Yield		11.73 kips	87.84 kips	**0.13**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**$\varphi = 1.00$**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	4.07 in^2		*Gross area subject to shear*		
φR_n	87.84 kips		*Shear yield strength*		

Plate Shear Rupture		11.73 kips	106.14 kips	**0.11**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	4.07 in^2		*Net area subject to shear*		
φR_n	106.14 kips		*Shear rupture strength*		

Plate Axial Yield		13.20 kips	131.76 kips	**0.10**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.07 in^2		*Gross area subject to tension*		
φR_n	131.76 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.03**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	13.20 kips		*Calculated axial load*		
V_r	11.73 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.07 in^2		*Gross area of the plate*		
Z_{pl}	11.03 in^3		*Plastic modulus of the shear plate*		
P_c	131.76 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	87.84 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	29.77 kips-ft		*Available moment $M_c = \varphi * (F_y * Z)$, $\varphi = 0.90$*		
UC	0.03		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

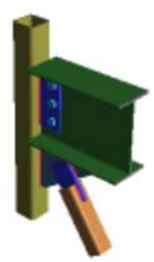
Plate Flexural Rupture				**0.01**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	11.73 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	4.07 in^2		*Net area of the plate*		
Z_{net}	11.03 in^3		*Plastic modulus of net section*		
V_c	106.14 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	39.97 kips-ft		*Available moment $M_c = \varphi * (F_u * Znet)$, $\varphi = 0.75$*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Beam Weld Strength		11.73 kips	72.46 kips	**0.16**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		

L	10.84 in		*Weld length*		
φRn	72.46 kips		*Weld strength*		
Beam Web Yielding		13.20 kips	163.81 kips	**0.08**	**PASS**
R_n = (5 * k + N) * Fy * t_w		**φ = 1.00**	*(J10-2)*		
k	0.68 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	10.84 in		*Length of bearing*		
Fy	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.23 in		*Beam web thickness*		
φRn	163.81 kips		*Beam web local yielding*		

BR-10 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x7.04	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x9.00x10.84	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	9.74 kips	*Calculated Shear Load*
Axial Load	4.33 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
F_{y-max}	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
t_{p-max}	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		

D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	9.00 in		*Min weld segment length*		

Plate Shear Yield		9.74 kips	72.90 kips	**0.13**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.38 in^2		*Gross area subject to shear*		
φR_n	72.90 kips		*Shear yield strength*		

Plate Shear Rupture		9.74 kips	88.09 kips	**0.11**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.38 in^2		*Net area subject to shear*		
φR_n	88.09 kips		*Shear rupture strength*		

Plate Axial Yield		4.33 kips	109.35 kips	**0.04**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to tension*		
φR_n	109.35 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.02**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	4.33 kips		*Calculated axial load*		
V_r	9.74 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area of the plate*		
Z_{pl}	7.59 in^3		*Plastic modulus of the shear plate*		
P_c	109.35 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	72.90 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	20.50 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.02		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.01**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	9.74 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.38 in^2		*Net area of the plate*		
Z_{net}	7.59 in^3		*Plastic modulus of net section*		
V_c	88.09 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	27.53 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		9.74 kips	60.13 kips	**0.16**	**PASS**

$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$

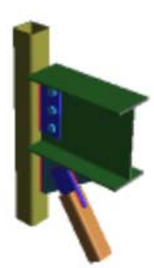
Double Fillet

$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)

C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	9.00 in		*Weld length*		
φ**Rn**	60.13 kips		*Weld strength*		
HSS Transverse Plastification		4.33 kips	22.89 kips	**0.19**	**PASS**
$R_n = F_y{*}t^2 / (1-t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1-t_p/B)^{0.5})$		φ **= 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	9.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	22.89 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	9.76 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ **= 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	9.00 in		*Depth of plate*		
η	2.25		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	9.76 kips-ft		*Flexural plastification*		

BR-10 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x7.04	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x9.00x10.84	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	28.00 kips	*Brace Axial (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Gusset Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				

D	0.19 in		*Weld size*		
D_{max}	0.56 in		*Max Size Allowed*		
t	0.63 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.53 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		28.00 kips	95.15 kips	**0.29**	**PASS**
$P_n = F_y*A_g$		**φ = 0.9**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	3.14 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	14.52		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.94 in^2		*Gross area of plate (Whitmore section)*		
φP_n	95.15 kips		*Gusset plate compressive strength*		

Knife Plate Buckling		28.00 kips	75.94 kips	**0.37**	**PASS**
$R_n = F_y * A_g$		**φ = 0.9**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	2.89 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	16.00		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	2.34 in^2		*Gross area subject to compression*		
φR_n	75.94 kips		*Compressive strength*		

Knife Plate Flexure		0.58 kips-ft	0.99 kips-ft	**0.59**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	28.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.63 in		*Thickness of knife plate*		
e	0.50 in		*Eccentricity, e = (t_{plate} + t_{gusset}) /2*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.37 in^3		*Plastic section modulus, Z = W_p* $(t_{plate})^2$/4*		
M_u	0.58 kips-ft		*Required moment demand on knife plate, M_u=R_u * e / 2*		
φM_n	0.99 kips-ft		*Available flexural strength*		

Knife Plate Interaction				**0.89**	**PASS**
$P_u/\varphi P_c$ + 8/9*($M_u/\varphi M_p$) <= 1.0			*(H1-1a)*		

P_u	28.00 kips		User input brace axial load	
φP_c	75.94 kips		Available buckling strength of knife plate	
M_u	0.58 kips-ft		Required moment demand on knife plate	
φM_p	0.99 kips-ft		Available flexural strength of knife plate	

Gusset Weld Strength		28.00 kips	45.18 kips	**0.62**	PASS
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	10.82 in		*Weld length*		
φRn	45.18 kips		*Weld strength*		

Brace Weld Strength		28.00 kips	50.11 kips	**0.56**	PASS
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		

BR-10 Top Detail: Members Report

Vertical Brace Diagonal Connection

Beam		***W12x26***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	6.49 in	*Flange width*
d	12.20 in	*Overall depth*
t_w	0.23 in	*Web thickness*
t_f	0.38 in	*Flange thickness*
a	7.65 in^2	*Area*
k_{des}	0.68 in	*Kdes*
k_{det}	1.06 in	*Kdet*
k_1	0.75 in	*K1*
Web Hole Type		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Column		***HSS4x4x4***
Material		

Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Bottom Brace		***HSS3x3x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-10 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.00x9.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Knife Plate		***P0.63x3.75x7.04***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*
t	0.63 in	*Thickness*

Bottom Gusset		***P0.38x9.00x10.84***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*

Member Properties			
	d	9.00 in	*Width*
	t	0.38 in	*Thickness*
Clip			
	H_clip	9.78 in	*Horiz. Clip*
	V_clip	6.85 in	*Vert. Clip*

	Column Weld		***E70***
Weld Properties			
	Type	Double Fillet	
	Fillet Size	0.25 in	

	Beam Bolts		***3/4" A325***
Bolt Properties			
	Type	A325	
	d	0.75 in	*Diameter*
Strength			
	S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
	T	90.00 ksi	*Tensile strength*

	Gusset Plate Weld		***E70***
Weld Properties			
	Type	Single Fillet	
	Fillet Size	0.19 in	

	Knife Plate Brace Weld		***E70***
Weld Properties			
	Type	Single Fillet	
	Fillet Size	0.19 in	

	Beam Weld		***E70***
Weld Properties			
	Type	Double Fillet	
	Fillet Size	0.19 in	

	Column Weld		***E70***
Weld Properties			
	Type	Double Fillet	
	Fillet Size	0.19 in	

RISAConnection version 7.0.1

02/08/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

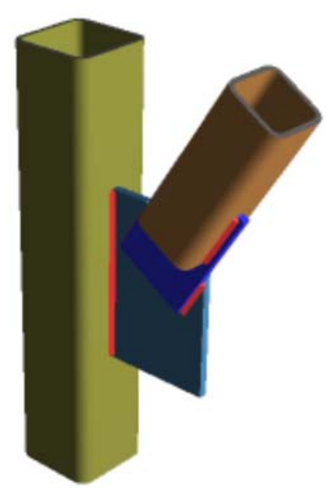
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

BR-11 Bottom Detail	**PASS(UC-0.8)**
BR-11 Top Detail	**PASS(UC-0.7)**

BR-11 Bottom Detail: 3D View

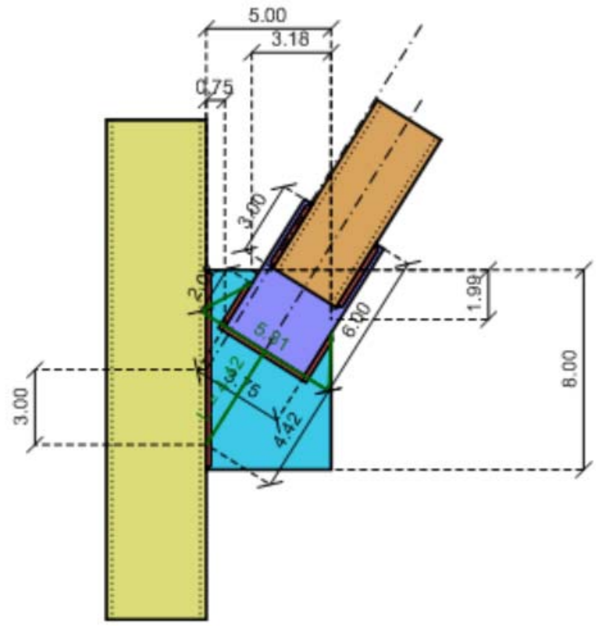
Knee Brace Connection



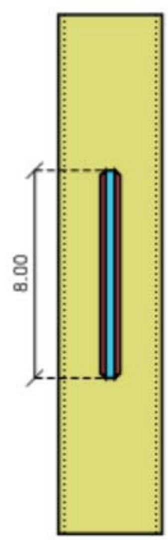
BR-11 Bottom Detail: 2D Views

Knee Brace Connection

Left view

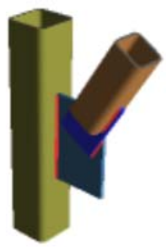


Front view



BR-11 Bottom Detail: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:

Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x5.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x6.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	11.00 kips	*Brace Axial (compression)*
Shear Load	9.33 kips	*Calculated Shear Load*
Axial Load	5.83 kips	*Calculated Axial Load (compression)*
Moment Load	1.46 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
$t_{p\text{-}max}$	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	3.00 in	*Vertical brace workpoint offset*		
d_{gusset}	8.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		
t_{min}	0.23 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	8.00 in	*Min weld segment length*		
Gusset Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		

Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.02 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Plate Shear Yield		9.33 kips	64.80 kips	**0.14**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		

Plate Shear Rupture		9.33 kips	78.30 kips	**0.12**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.00 in^2		*Net area subject to shear*		
φR_n	78.30 kips		*Shear rupture strength*		

Plate Axial Yield		5.83 kips	97.20 kips	**0.06**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.04**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	5.83 kips		*Calculated axial load*		
V_r	9.33 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	1.46 kips-ft		*Calculated moment*		
M_c	16.20 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.04		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture			**0.02**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		*(Eq.10-5)*		
P_r	0.00 kips	*Calculated axial load*		
V_r	9.33 kips	*Calculated shear load*		
F_u	58.00 ksi	*Minimum tensile stress of material*		
A_n	3.00 in^2	*Net area of the plate*		
Z_{net}	6.00 in^3	*Plastic modulus of net section*		
V_c	78.30 kips	*Available shear strength (see check 'Shear Rupture')*		
M_r	1.46 kips-ft	*Calculated moment*		
M_c	21.75 kips-ft	*Available moment $M_c = \varphi*(F_u * Znet)$, $\varphi=0.75$*		
UC	0.02	*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		2.01 kips/in	6.68 kips/in	**0.30**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ (AISC 14th Eqn 8-2a)					
V	9.33 kips		*Shear Load*		
P	5.83 kips		*Axial Load*		
M	1.46 kips-ft		*Moment*		
e_{eff}	1.87 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	2.01 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		11.00 kips	62.24 kips	**0.18**	**PASS**
$P_n = F_{cr}*A_g$		**$\varphi = 0.9$**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	4.42 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	48.95		*Plate slenderness*		
F_{cr}	31.73 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.18 in^2		*Gross area of plate (Whitmore section)*		
φP_n	62.24 kips		*Gusset plate compressive strength*		

Gusset Weld Strength		11.00 kips	32.52 kips	**0.34**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	7.79 in		*Weld length*		
φRn	32.52 kips		*Weld strength*		

Brace Weld Strength		11.00 kips	50.11 kips	**0.22**	**PASS**
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table*		

			8-3)		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		
Knife Plate Flexure		0.17 kips-ft	0.36 kips-ft	**0.48**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	11.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.38 in		*Thickness of knife plate*		
e	0.38 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) /2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.13 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.17 kips-ft		*Required moment demand on knife plate, $M_u = R_u * e / 2$*		
φM_n	0.36 kips-ft		*Available flexural strength*		
Knife Plate Interaction				**0.76**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	11.00 kips		*User input brace axial load*		
φP_c	33.46 kips		*Available buckling strength of knife plate*		
M_u	0.17 kips-ft		*Required moment demand on knife plate*		
φM_p	0.36 kips-ft		*Available flexural strength of knife plate*		
HSS Transverse Plastification		5.83 kips	21.52 kips	**0.27**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		
HSS Flexural Plastification		1.46 kips-ft	8.21 kips-ft	**0.18**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	1.46 kips-ft		*Required flexural plastification*		
φM_n	8.21 kips-ft		*Flexural plastification*		
Knife Plate Buckling		11.00 kips	33.46 kips	**0.33**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	2.10		*Effective length factor*		
L	3.95 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	76.60		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.41 in^2		*Gross area subject to compression*		

E	29000.00 ksi	*Modulus of elasticity*
F_e	48.78 ksi	*Elastic buckling stress (E3-4)*
F_{cr}	26.43 ksi	*Critical stress (E3-2)*
φR_n	33.46 kips	*Compressive strength*

BR-11 Bottom Detail: Members Report

Knee Brace Connection

	Column		***HSS4x4x4***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	4.00 in	*Depth*
	b	4.00 in	*Width*
	a	3.37 in^2	*Area*
	t_{des}	0.23 in	*Wall Thickness*

	Brace		***HSS3x3x4***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.00 in	*Depth*
	b	3.00 in	*Width*
	a	2.44 in^2	*Area*
	t_{des}	0.23 in	*Wall Thickness*

BR-11 Bottom Detail: Components Report

Knee Brace Connection

	Gusset		***P0.38x5.00x8.00***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	5.00 in	*Width*
	t	0.38 in	*Thickness*
Clip			
	H_clip	3.18 in	*Horiz. Clip*
	V_clip	1.99 in	*Vert. Clip*

	Knife Plate		***P0.38x3.75x6.00***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.75 in	*Width*

t	0.38 in	*Thickness*

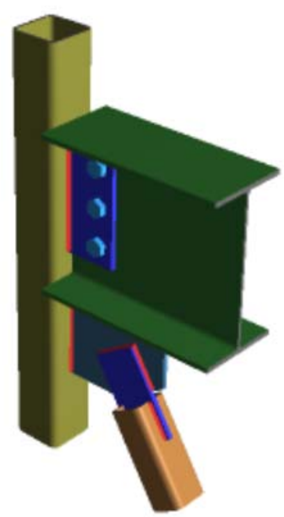
Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

BR-11 Top Detail: 3D View

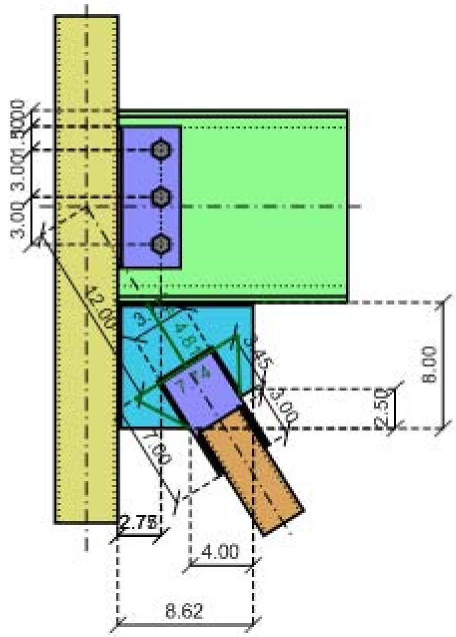
Vertical Brace Diagonal Connection



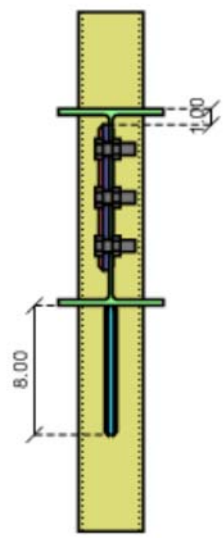
BR-11 Top Detail: 2D Views

Vertical Brace Diagonal Connection

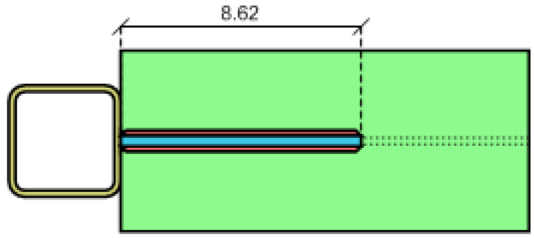
Side view



Front view

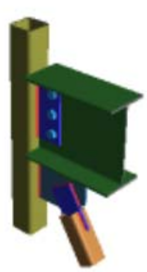


Bottom view



BR-11 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x8.00x8.62	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	11.00 kips	*User Input Bottom Brace Axial Force (compression)*

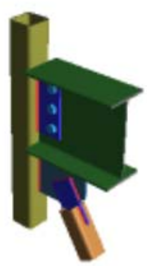
Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	HSS Transverse Plastification	**0.30**	**PASS**
Bottom Gusset/Beam connection	Beam Weld Strength	**0.07**	**PASS**
Bottom Gusset/Column connection	HSS Transverse Plastification	**0.09**	**PASS**
Bottom Gusset/Brace connection	Knife Plate Interaction	**0.68**	**PASS**

BR-11 Top Detail: Beam/Column Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x8.00x8.62	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-3.63 kips	*Calculated Shear Load*
Total Axial Load	6.85 kips	*Calculated Axial Load (compression)*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
$t_{p\text{-}max}$	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.23 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		

t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	9.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	9.00 in		*Max weld segment length*		
Beam Shear Yield		3.63 kips	84.18 kips	**0.04**	**PASS**
R_n = 0.6 *F_y*A_{gv}*C_v		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	84.18 kips		*Shear yield strength*		
Plate Shear Yield		3.63 kips	72.90 kips	**0.05**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.38 in^2		*Gross area subject to shear*		
φR_n	72.90 kips		*Shear yield strength*		
Beam Shear Rupture		3.63 kips	64.42 kips	**0.06**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.20 in^2		*Net area subject to shear*		
φR_n	64.42 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		3.63 kips	62.40 kips	**0.06**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.39 in^2		*Net area subject to shear*		
φR_n	62.40 kips		*Shear rupture strength*		
Beam Axial Yield		6.85 kips	344.25 kips	**0.02**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	7.65 in^2		*Gross area subject to tension*		
φR_n	344.25 kips		*Tensile yield strength*		
Plate Axial Yield		6.85 kips	109.35 kips	**0.06**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to tension*		
φR_n	109.35 kips		*Tensile yield strength*		
Beam Block Shear		3.63 kips	116.94 kips	**0.03**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	4.04 in^2		*Gross area subject to shear*		
A_{nv}	3.54 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.53 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	116.94 kips		*Block shear strength*		
Plate Block Shear at Beam		3.63 kips	58.82 kips	**0.06**	**PASS**

R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
A_{nv}	1.99 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.30 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	58.82 kips		*Block shear strength*		
Compression Buckling of the Plate		6.85 kips	105.71 kips	**0.06**	**PASS**
R_n = F_{cr} * A_g		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	2.75 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	25.37		*Plate slenderness check from J4-6*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	444.52 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.80 ksi		*Critical stress (E3-2)*		
φR_n	105.71 kips		*Compressive strength*		
Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14^{th} Eq.10-5)*		
P_r	6.85 kips		*Calculated axial load*		
V_r	-3.63 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area of the plate*		
Z_{pl}	7.59 in^3		*Plastic modulus of the shear plate*		
P_c	109.35 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	72.90 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.60 in		*Vertical eccentricity*		
M_r	-0.49 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.50 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	6.85 kips		*Calculated axial load*		
V_r	-3.63 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.39 in^2		*Net area of the plate*		
Z_{net}	5.55 in^3		*Plastic modulus of net section*		
V_c	62.40 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.60 in		*Vertical eccentricity*		
M_r	-0.49 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.13 kips-ft		*Available moment M_c= φ*(F_u* Znet), φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Plate Flexural Buckling				**0.15**	**PASS**

$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**$\varphi = 0.90$**	*(AISC 14th Edition)*		
P	6.85 kips		*Calculated axial load*		
V	3.63 kips		*Calculated shear load*		
L	2.75 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	25.37		*Slenderness ratio*		
F_e	444.52 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2*E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	34.80 ksi		*Compression stress, per eqn E3-2, Fcr = (0.658 ^ (Fy / Fe)) * Fy*		
A_g	3.38 in^2		*Gross area of the plate*		
λ	0.33		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	3.74 in^3		*Section modulus of net section*		
a	2.75 in		*Design eccentricity*		
P_n	117.45 kips		*Compressive capacity, per eqn E4-1, Pn = Fcr_Comp * Ag*		
V_n	48.91 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.15		*Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$*		

Bolt Bearing on Beam		7.75 kips	53.68 kips	**0.14**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**$\varphi = 0.75$**	*(J3-6b)*		
V	-3.63 kips		*Applied shear force*		
P	6.85 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	7.75 kips		*Resultant shear force*		
Θ	27.96 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	3.70 in		*Vertical edge spacing*		
e_h	2.75 in		*Horizontal edge spacing*		
L_{c_boltA}	2.71 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.71 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		7.75 kips	53.68 kips	**0.14**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**$\varphi = 0.75$**	*(J3-6b)*		
V	-3.63 kips		*Applied shear force*		
P	6.85 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	7.75 kips		*Resultant shear force*		

Θ	27.96 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.50 in		*Vertical edge spacing*		
e_h	1.25 in		*Horizontal edge spacing*		
L_{c_boltA}	1.01 in		*Minimum clear distance for the corner edge bolt:* $Lc\_boltA = \min((e_v / \sin(\Theta)), (e_h / \cos(\Theta))) - d_c$		
L_{c_boltB}	1.01 in		*Minimum clear distance for the side edge bolts:* $Lc\_boltB = \min((s_v - 0.5 * d_v / \sin(\Theta)), (e_h / \cos(\Theta))) - d_c$		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt:* $Rn\_boltA = \min[(1.5 * Lc\_boltA * t * Fu), (3.0 * d_b * t * Fu), R_{n\text{-}bolt}]$		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt:* $Rn\_boltB = \min[(1.5 * Lc\_boltB * t * Fu), (3.0 * d_b * t * Fu), R_{n\text{-}bolt}]$		
$R_{n\text{-}bolt}$	23.86 kips		*Bolt shear strength* $R_{n\text{-}bolt} = F_{nv} * A_{bolt}$		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		7.75 kips	43.28 kips	**0.18**	**PASS**
$\mathbf{R_n = F_{nv} * A_b * N_{bolt} * C}$		φ **= 0.75**	*(J3-1)*		
V	-3.63 kips		*Applied shear force*		
P	6.85 kips		*Applied axial force*		
$\mathbf{R = (V^2 + P^2)^{0.5}}$	7.75 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.81		*Eccentricity coefficient*		
φR_n	43.28 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***0.81***		
Elastic method			*(AISC 14^{th} p.7-6)*		
C	0.81		*Coefficient (2.4188 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.60 in		*Vertical eccentricity*		
Ang	27.96		*Angle of force in degrees, relative X axis*		

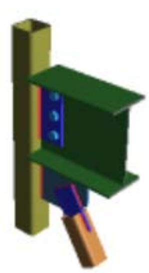
Weld at Column		7.75 kips	88.18 kips	**0.09**	**PASS**
$\mathbf{\varphi R_n = 2 * C_1 * \alpha * 1.392 * D_{16} * L}$					
Double Fillet					
$\mathbf{1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75}$ **(AISC 14^{th} Eqn 8-2a)**					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	9.00 in		*Weld length*		
φR_n	88.18 kips		*Weld strength*		

HSS Transverse Plastification		6.85 kips	22.89 kips	**0.30**	**PASS**

$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		φ = 1.00	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	9.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	22.89 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	10.05 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	9.00 in		*Depth of plate*		
η	2.25		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	10.05 kips-ft		*Flexural plastification*		

BR-11 Top Detail: Bot Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x8.00x8.62	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	3.98 kips	*Calculated Shear Load*
Axial Load	5.63 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	8.62 in		*Min weld segment length*		

Plate Shear Yield		3.98 kips	69.84 kips	**0.06**	PASS
$R_n = 0.6 * F_y * A_{gv}$		**$\varphi = 1.00$**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.23 in^2		*Gross area subject to shear*		
φR_n	69.84 kips		*Shear yield strength*		

Plate Shear Rupture		3.98 kips	84.39 kips	**0.05**	PASS
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.23 in^2		*Net area subject to shear*		
φR_n	84.39 kips		*Shear rupture strength*		

Plate Axial Yield		5.63 kips	104.76 kips	**0.05**	PASS
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.23 in^2		*Gross area subject to tension*		
φR_n	104.76 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	PASS
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	5.63 kips		*Calculated axial load*		
V_r	3.98 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.23 in^2		*Gross area of the plate*		
Z_{pl}	6.97 in^3		*Plastic modulus of the shear plate*		
P_c	104.76 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	69.84 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	18.82 kips-ft		*Available moment $M_c=\varphi*(F_y * Z)$, $\varphi=0.90$*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

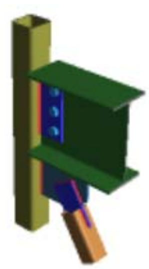
Plate Flexural Rupture				**0.00**	PASS
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	3.98 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.23 in^2		*Net area of the plate*		
Z_{net}	6.97 in^3		*Plastic modulus of net section*		
V_c	84.39 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	25.27 kips-ft		*Available moment $M_c= \varphi*(F_u * Znet)$, $\varphi=0.75$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Beam Weld Strength		3.98 kips	57.61 kips	**0.07**	PASS
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		

L	8.62 in		*Weld length*		
φRn	57.61 kips		*Weld strength*		
Beam Web Yielding		5.63 kips	138.26 kips	**0.04**	PASS
$R_n = (5 * k + N) * F_y * t_w$		**φ = 1.00**	*(J10-2)*		
k	0.68 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	8.62 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.23 in		*Beam web thickness*		
φRn	138.26 kips		*Beam web local yielding*		

BR-11 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x8.00x8.62	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	3.69 kips	*Calculated Shear Load*
Axial Load	1.85 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

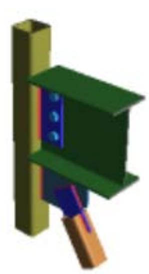
Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					PASS
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
$t_{p\text{-}max}$	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					PASS
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		

D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Plate Shear Yield		3.69 kips	64.80 kips	**0.06**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		$\varphi = 1.00$	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		
Plate Shear Rupture		3.69 kips	78.30 kips	**0.05**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		$\varphi = 0.75$	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.00 in^2		*Net area subject to shear*		
φR_n	78.30 kips		*Shear rupture strength*		
Plate Axial Yield		1.85 kips	97.20 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		$\varphi = 0.90$	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	1.85 kips		*Calculated axial load*		
V_r	3.69 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	16.20 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	3.69 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.00 in^2		*Net area of the plate*		
Z_{net}	6.00 in^3		*Plastic modulus of net section*		
V_c	78.30 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	21.75 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Column Weld Strength		3.69 kips	53.45 kips	**0.07**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					

C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φRn	53.45 kips		*Weld strength*		
HSS Transverse Plastification		1.85 kips	21.52 kips	**0.09**	**PASS**
$R_n = F_y t^2 / (1-t_p/B)(2l_b/B + 4 Q_f (1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φRn	21.52 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	8.21 kips-ft	**0.00**	**PASS**
$M_n = F_y t^2 H_b (1/(2\eta) + 2/(1-\beta)^{0.5} + \eta/(1-\beta)) Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	8.21 kips-ft		*Flexural plastification*		

BR-11 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x8.00x8.62	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	11.00 kips	*Brace Axial (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Gusset Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				

D	0.19 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.45 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		11.00 kips	94.03 kips	**0.12**	**PASS**
$P_n = F_y * A_g$		**φ = 0.9**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	4.81 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	22.20		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.90 in^2		*Gross area of plate (Whitmore section)*		
φP_n	94.03 kips		*Gusset plate compressive strength*		

Knife Plate Buckling		11.00 kips	43.90 kips	**0.25**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	2.88 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	26.61		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.41 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	404.30 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.68 ksi		*Critical stress (E3-2)*		
φR_n	43.90 kips		*Compressive strength*		

Knife Plate Flexure		0.17 kips-ft	0.36 kips-ft	**0.48**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	11.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.38 in		*Thickness of knife plate*		
e	0.38 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) / 2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.13 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.17 kips-ft		*Required moment demand on knife plate, $M_u = R_u * e / 2$*		
φM_n	0.36 kips-ft		*Available flexural strength*		

Knife Plate Interaction			0.68	PASS
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$		*(H1-1a)*		
P_u	11.00 kips	*User input brace axial load*		
φP_c	43.90 kips	*Available buckling strength of knife plate*		
M_u	0.17 kips-ft	*Required moment demand on knife plate*		
φM_p	0.36 kips-ft	*Available flexural strength of knife plate*		

Gusset Weld Strength		11.00 kips	44.51 kips	0.25	PASS
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	10.66 in		*Weld length*		
φRn	44.51 kips		*Weld strength*		

Brace Weld Strength		11.00 kips	50.11 kips	0.22	PASS
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		

BR-11 Top Detail: Members Report

Vertical Brace Diagonal Connection

	Beam		***W12x26***
Material			
	Name	A992	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	65.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b_f	6.49 in	*Flange width*
	d	12.20 in	*Overall depth*
	t_w	0.23 in	*Web thickness*
	t_f	0.38 in	*Flange thickness*
	a	7.65 in^2	*Area*
	k_{des}	0.68 in	*Kdes*
	k_{det}	1.06 in	*Kdet*
	k_1	0.75 in	*K1*
Web Hole Type			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	3	*Number of holes per row*
	R_s	3.00 in	*Row Spacing*
	C_s	3.00 in	*Column Spacing*

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Bottom Brace		***HSS3x3x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-11 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.00x9.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Knife Plate		***P0.38x3.75x7.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*
t	0.38 in	*Thickness*

Bottom Gusset		***P0.38x8.00x8.62***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*

F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	8.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	4.00 in	*Horiz. Clip*
V_clip	2.50 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

RISAConnection version 7.0.1

02/07/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

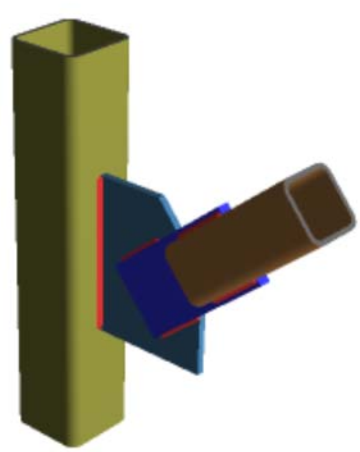
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

BR-12 Bottom Detail	**PASS(UC-0.5)**
BR-12 Top Detail	**PASS(UC-0.8)**

BR-12 Bottom Detail: 3D View

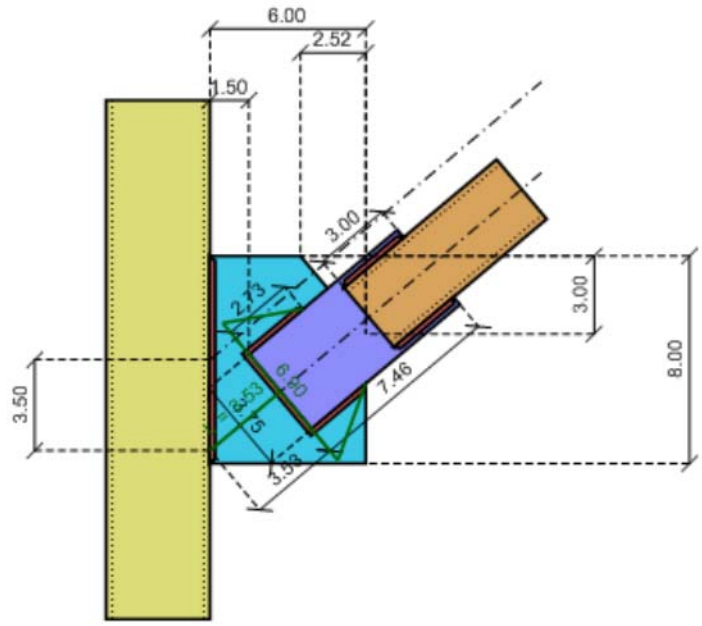
Knee Brace Connection



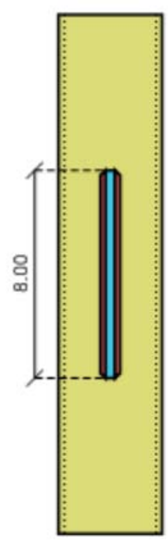
BR-12 Bottom Detail: 2D Views

Knee Brace Connection

Left view

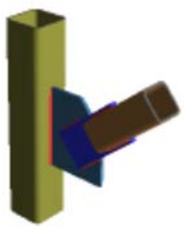


Front view



BR-12 Bottom Detail: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:

Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x6.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.63x3.75x7.46	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	13.00 kips	*Brace Axial (compression)*
Shear Load	8.36 kips	*Calculated Shear Load*
Axial Load	9.96 kips	*Calculated Axial Load (compression)*
Moment Load	2.90 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
$t_{p\text{-}max}$	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	3.50 in	*Vertical brace workpoint offset*		
d_{gusset}	8.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		
t_{min}	0.23 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	8.00 in	*Min weld segment length*		
Gusset Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		

Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.56 in		*Max Size Allowed*		
t	0.63 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.73 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Plate Shear Yield		8.36 kips	64.80 kips	**0.13**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		

Plate Shear Rupture		8.36 kips	78.30 kips	**0.11**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.00 in^2		*Net area subject to shear*		
φR_n	78.30 kips		*Shear rupture strength*		

Plate Axial Yield		9.96 kips	97.20 kips	**0.10**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.10**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	9.96 kips		*Calculated axial load*		
V_r	8.36 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	2.90 kips-ft		*Calculated moment*		
M_c	16.20 kips-ft		*Available moment $M_c = \varphi*(F_y * Z)$, φ=0.90*		
UC	0.10		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				0.03	PASS
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	8.36 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.00 in^2		*Net area of the plate*		
Z_{net}	6.00 in^3		*Plastic modulus of net section*		
V_c	78.30 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	2.90 kips-ft		*Calculated moment*		
M_c	21.75 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.03		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		3.43 kips/in	6.68 kips/in	0.51	PASS
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
V	8.36 kips		*Shear Load*		
P	9.96 kips		*Axial Load*		
M	2.90 kips-ft		*Moment*		
e_{eff}	4.17 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	3.43 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		13.00 kips	77.33 kips	0.17	PASS
$P_n = F_{cr}*A_g$		φ = 0.9	*(E3-1)*		
K	1.20		*Effective length factor*		
L	3.53 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	39.15		*Plate slenderness*		
F_{cr}	33.21 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.59 in^2		*Gross area of plate (Whitmore section)*		
φP_n	77.33 kips		*Gusset plate compressive strength*		

Gusset Weld Strength		13.00 kips	38.44 kips	0.34	PASS
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	9.21 in		*Weld length*		
φRn	38.44 kips		*Weld strength*		

Brace Weld Strength		13.00 kips	50.11 kips	0.26	PASS
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table*		

			8-3)		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		

Knife Plate Flexure		0.27 kips-ft	0.99 kips-ft	**0.27**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	13.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.63 in		*Thickness of knife plate*		
e	0.50 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) /2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.37 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.27 kips-ft		*Required moment demand on knife plate, $M_u=R_u * e / 2$*		
φM_n	0.99 kips-ft		*Available flexural strength*		

Knife Plate Interaction				**0.46**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	13.00 kips		*User input brace axial load*		
φP_c	60.24 kips		*Available buckling strength of knife plate*		
M_u	0.27 kips-ft		*Required moment demand on knife plate*		
φM_p	0.99 kips-ft		*Available flexural strength of knife plate*		

HSS Transverse Plastification		9.96 kips	21.52 kips	**0.46**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		

HSS Flexural Plastification		2.90 kips-ft	8.21 kips-ft	**0.35**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	2.90 kips-ft		*Required flexural plastification*		
φM_n	8.21 kips-ft		*Flexural plastification*		

Knife Plate Buckling		13.00 kips	60.24 kips	**0.22**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	2.10		*Effective length factor*		
L	5.70 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	66.33		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.34 in^2		*Gross area subject to compression*		

E	29000.00 ksi	*Modulus of elasticity*
F_e	65.06 ksi	*Elastic buckling stress (E3-4)*
F_{cr}	28.56 ksi	*Critical stress (E3-2)*
φR_n	60.24 kips	*Compressive strength*

BR-12 Bottom Detail: Members Report

Knee Brace Connection

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Brace		***HSS3x3x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-12 Bottom Detail: Components Report

Knee Brace Connection

Gusset		***P0.38x6.00x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	6.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	2.52 in	*Horiz. Clip*
V_clip	3.00 in	*Vert. Clip*

Knife Plate		***P0.63x3.75x7.46***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*

t	0.63 in	*Thickness*

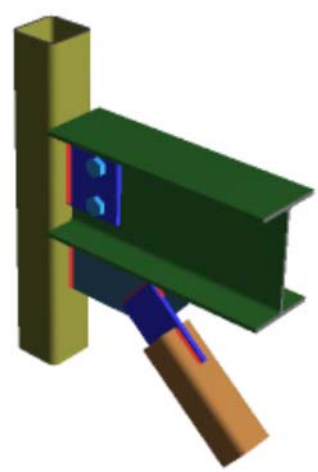
Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

BR-12 Top Detail: 3D View

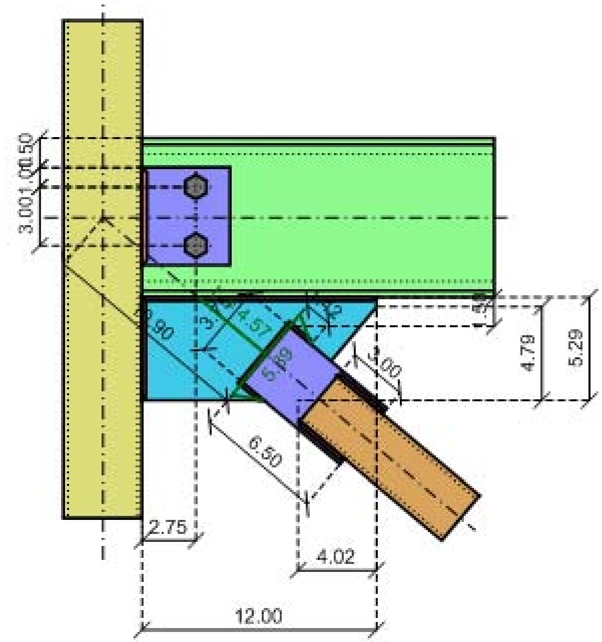
Vertical Brace Diagonal Connection



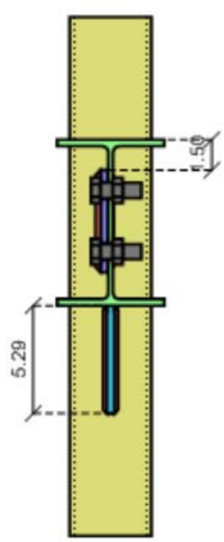
BR-12 Top Detail: 2D Views

Vertical Brace Diagonal Connection

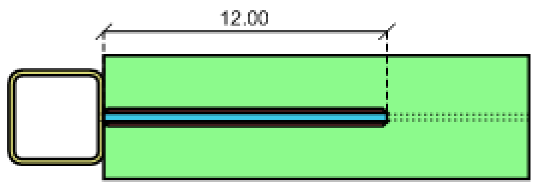
Side view



Front view

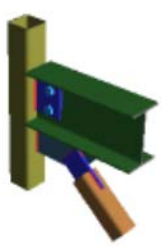


Bottom view



BR-12 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	Fy = 50.00 ksi	Fu = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	Fy = 46.00 ksi	Fu = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	Fy = 36.00 ksi	Fu = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	Fy = 46.00 ksi	Fu = 58.00 ksi
Knife Plate	P0.38x3.75x6.50	A36	Fy = 36.00 ksi	Fu = 58.00 ksi
Bottom Gusset	P0.38x5.29x12.00	A36	Fy = 36.00 ksi	Fu = 58.00 ksi

Input Data:

Shear Load	2.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	5.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	13.00 kips	*User Input Bottom Brace Axial Force (compression)*

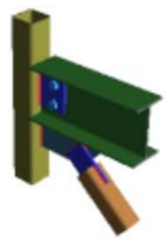
Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	HSS Transverse Plastification	**0.43**	**PASS**
Bottom Gusset/Beam connection	Beam Weld Strength	**0.09**	**PASS**
Bottom Gusset/Column connection	HSS Transverse Plastification	**0.14**	**PASS**
Bottom Gusset/Brace connection	Knife Plate Interaction	**0.81**	**PASS**

BR-12 Top Detail: Beam/Column Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x6.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x5.29x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-3.07 kips	*Calculated Shear Load*
Total Axial Load	7.49 kips	*Calculated Axial Load (compression)*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	5.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.23 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		

t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	5.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	5.00 in		*Max weld segment length*		
Beam Shear Yield		3.07 kips	56.17 kips	**0.05**	**PASS**
$R_n = 0.6 * F_y * A_{gv} * C_v$		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	1.87 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	56.17 kips		*Shear yield strength*		
Plate Shear Yield		3.07 kips	40.50 kips	**0.08**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	1.88 in^2		*Gross area subject to shear*		
φR_n	40.50 kips		*Shear yield strength*		
Beam Shear Rupture		3.07 kips	42.99 kips	**0.07**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	1.47 in^2		*Net area subject to shear*		
φR_n	42.99 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		3.07 kips	31.81 kips	**0.10**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	1.22 in^2		*Net area subject to shear*		
φR_n	31.81 kips		*Shear rupture strength*		
Beam Axial Yield		7.49 kips	236.70 kips	**0.03**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	5.26 in^2		*Gross area subject to tension*		
φR_n	236.70 kips		*Tensile yield strength*		
Plate Axial Yield		7.49 kips	60.75 kips	**0.12**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.88 in^2		*Gross area subject to tension*		
φR_n	60.75 kips		*Tensile yield strength*		
Beam Block Shear		3.07 kips	95.23 kips	**0.03**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		**φ = 0.75**	*(J4-5)*		
A_{gv}	3.08 in^2		*Gross area subject to shear*		
A_{nv}	2.78 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.53 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	95.23 kips		*Block shear strength*		
Plate Block Shear at Beam		3.07 kips	45.71 kips	**0.07**	**PASS**

R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	1.50 in^2		*Gross area subject to shear*		
A_{nv}	1.01 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.49 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	45.71 kips		*Block shear strength*		
Compression Buckling of the Plate		7.49 kips	58.73 kips	**0.13**	**PASS**
R_n = F_{cr} * A_g		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	2.75 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	25.37		*Plate slenderness check from J4-6*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.88 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	444.52 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.80 ksi		*Critical stress (E3-2)*		
φR_n	58.73 kips		*Compressive strength*		
Plate Flexural Yield				**0.06**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	7.49 kips		*Calculated axial load*		
V_r	-3.07 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.88 in^2		*Gross area of the plate*		
Z_{pl}	2.34 in^3		*Plastic modulus of the shear plate*		
P_c	60.75 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	40.50 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.07 in		*Vertical eccentricity*		
M_r	-0.66 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	6.33 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		
UC	0.06		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.03**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	7.49 kips		*Calculated axial load*		
V_r	-3.07 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	1.22 in^2		*Net area of the plate*		
Z_{net}	1.36 in^3		*Plastic modulus of net section*		
V_c	31.81 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.07 in		*Vertical eccentricity*		
M_r	-0.66 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	4.93 kips-ft		*Available moment M_c= φ*(F_u* Znet), φ=0.75*		
UC	0.03		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Plate Flexural Buckling				**0.40**	**PASS**

$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**$\varphi = 0.90$**	*(AISC 14th Edition)*		
P	7.49 kips		*Calculated axial load*		
V	3.07 kips		*Calculated shear load*		
L	2.75 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	25.37		*Slenderness ratio*		
F_e	444.52 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 *E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	34.80 ksi		*Compression stress, per eqn E3-2, Fcr = (0.658 ^ (Fy / Fe)) * Fy*		
A_g	1.88 in^2		*Gross area of the plate*		
λ	0.28		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	0.96 in^3		*Section modulus of net section*		
a	2.75 in		*Design eccentricity*		
P_n	65.25 kips		*Compressive capacity, per eqn E4-1, Pn = Fcr_Comp * Ag*		
V_n	12.50 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.40		*Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$*		

Bolt Bearing on Beam		8.09 kips	35.78 kips	**0.23**	**PASS**
$R_n = 1*R_{n\_boltA} + 1*R_{n\_boltB}$		**$\varphi = 0.75$**	*(J3-6b)*		
V	-3.07 kips		*Applied shear force*		
P	7.49 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	8.09 kips		*Resultant shear force*		
Θ	22.27 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	2.64 in		*Vertical edge spacing*		
e_h	2.75 in		*Horizontal edge spacing*		
L_{c_boltA}	2.57 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.57 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	35.78 kips		*Total bolt bearing strength*		

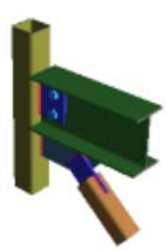
Bolt Bearing on Plate at Beam		8.09 kips	35.78 kips	**0.23**	**PASS**
$R_n = 1*R_{n\_boltA} + 1*R_{n\_boltB}$		**$\varphi = 0.75$**	*(J3-6b)*		
V	-3.07 kips		*Applied shear force*		
P	7.49 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	8.09 kips		*Resultant shear force*		

Θ	22.27 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.00 in		*Vertical edge spacing*		
e_h	1.75 in		*Horizontal edge spacing*		
L_{c_boltA}	1.48 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	1.48 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength R_{n-bolt}=F_{nv}*A_{bolt}*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	35.78 kips		*Total bolt bearing strength*		
Bolt Shear at Beam		8.09 kips	34.41 kips	**0.24**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		**φ = 0.75**	*(J3-1)*		
V	-3.07 kips		*Applied shear force*		
P	7.49 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	8.09 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	2		*Number of bolts*		
C	0.96		*Eccentricity coefficient*		
φR_n	34.41 kips		*Bolt shear rupture strength*		
Bolt Group Eccentricity			***0.96***		
Elastic method			*(AISC 14th p.7-6)*		
C	0.96		*Coefficient (1.9229 / 2)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	2		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.07 in		*Vertical eccentricity*		
Ang	22.27		*Angle of force in degrees, relative X axis*		
Weld at Column		8.09 kips	48.99 kips	**0.17**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	5.00 in		*Weld length*		
φR_n	48.99 kips		*Weld strength*		
HSS Transverse Plastification		7.49 kips	17.38 kips	**0.43**	**PASS**

$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		φ = 1.00	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	5.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	17.38 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	4.44 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	5.00 in		*Depth of plate*		
η	1.25		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	4.44 kips-ft		*Flexural plastification*		

BR-12 Top Detail: Bot Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x6.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x5.29x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	7.47 kips	*Calculated Shear Load*
Axial Load	5.07 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.33 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	12.00 in		*Min weld segment length*		

Plate Shear Yield		7.47 kips	97.20 kips	**0.08**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	4.50 in^2		*Gross area subject to shear*		
φR_n	97.20 kips		*Shear yield strength*		

Plate Shear Rupture		7.47 kips	117.45 kips	**0.06**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	4.50 in^2		*Net area subject to shear*		
φR_n	117.45 kips		*Shear rupture strength*		

Plate Axial Yield		5.07 kips	145.80 kips	**0.03**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.50 in^2		*Gross area subject to tension*		
φR_n	145.80 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	5.07 kips		*Calculated axial load*		
V_r	7.47 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.50 in^2		*Gross area of the plate*		
Z_{pl}	13.50 in^3		*Plastic modulus of the shear plate*		
P_c	145.80 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	97.20 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	36.45 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

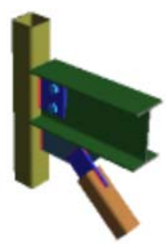
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	7.47 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	4.50 in^2		*Net area of the plate*		
Z_{net}	13.50 in^3		*Plastic modulus of net section*		
V_c	117.45 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	48.94 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Beam Weld Strength		7.47 kips	80.18 kips	**0.09**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		

L	12.00 in		*Weld length*		
φRn	80.18 kips		*Weld strength*		
Beam Web Yielding		5.07 kips	174.22 kips	**0.03**	PASS
R_n = (5 * k + N) * Fy * t_w		**φ = 1.00**	*(J10-2)*		
k	0.63 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	12.00 in		*Length of bearing*		
Fy	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.23 in		*Beam web thickness*		
φRn	174.22 kips		*Beam web local yielding*		

BR-12 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x6.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x5.29x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	3.29 kips	*Calculated Shear Load*
Axial Load	2.49 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					PASS
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
F_{y-max}	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
t_{p-max}	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					PASS
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		

D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	5.29 in		*Min weld segment length*		

Plate Shear Yield		3.29 kips	42.81 kips	**0.08**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	1.98 in^2		*Gross area subject to shear*		
φRn	42.81 kips		*Shear yield strength*		

Plate Shear Rupture		3.29 kips	51.73 kips	**0.06**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	1.98 in^2		*Net area subject to shear*		
φRn	51.73 kips		*Shear rupture strength*		

Plate Axial Yield		2.49 kips	64.22 kips	**0.04**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.98 in^2		*Gross area subject to tension*		
φRn	64.22 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14^{th} Eq.10-5)*		
P_r	2.49 kips		*Calculated axial load*		
V_r	3.29 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.98 in^2		*Gross area of the plate*		
Z_{pl}	2.62 in^3		*Plastic modulus of the shear plate*		
P_c	64.22 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	42.81 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	7.07 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

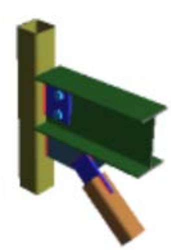
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	3.29 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	1.98 in^2		*Net area of the plate*		
Z_{net}	2.62 in^3		*Plastic modulus of net section*		
V_c	51.73 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	9.49 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		3.29 kips	35.32 kips	**0.09**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					

C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	5.29 in		*Weld length*		
φRn	35.32 kips		*Weld strength*		
HSS Transverse Plastification		2.49 kips	17.78 kips	**0.14**	**PASS**
$R_n = F_y{*}t^2 /(1-t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1-t_p/B)^{0.5})$		φ = 1.00	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	5.29 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	17.78 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	4.65 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	5.29 in		*Depth of plate*		
η	1.32		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	4.65 kips-ft		*Flexural plastification*		

BR-12 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W8x18	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x5.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x6.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x5.29x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	13.00 kips	*Brace Axial (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Gusset Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				

D	0.19 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	1.42 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		13.00 kips	65.49 kips	**0.20**	**PASS**
$P_n = F_y * A_g$		**$\varphi = 0.9$**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	4.57 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	21.10		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.02 in^2		*Gross area of plate (Whitmore section)*		
φP_n	65.49 kips		*Gusset plate compressive strength*		

Knife Plate Buckling		13.00 kips	43.52 kips	**0.30**	**PASS**
$R_n = F_{cr} * A_g$		**$\varphi = 0.9$**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	3.19 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	29.50		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.41 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	328.80 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.39 ksi		*Critical stress (E3-2)*		
φR_n	43.52 kips		*Compressive strength*		

Knife Plate Flexure		0.20 kips-ft	0.36 kips-ft	**0.57**	**PASS**
$M_n = F_y * Z$		**$\varphi = 0.9$**	*(F2-1)*		
R_u	13.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.38 in		*Thickness of knife plate*		
e	0.38 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) /2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.13 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.20 kips-ft		*Required moment demand on knife plate, $M_u = R_u * e / 2$*		
φM_n	0.36 kips-ft		*Available flexural strength*		

Knife Plate Interaction			**0.81**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$		*(H1-1a)*		
P_u	13.00 kips	*User input brace axial load*		
φP_c	43.52 kips	*Available buckling strength of knife plate*		
M_u	0.20 kips-ft	*Required moment demand on knife plate*		
φM_p	0.36 kips-ft	*Available flexural strength of knife plate*		

Gusset Weld Strength		13.00 kips	27.52 kips	**0.47**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	6.59 in		*Weld length*		
φRn	27.52 kips		*Weld strength*		

Brace Weld Strength		13.00 kips	50.11 kips	**0.26**	**PASS**
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		

BR-12 Top Detail: Members Report

Vertical Brace Diagonal Connection

	Beam		***W8x18***
Material			
	Name	A992	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	65.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b_f	5.25 in	*Flange width*
	d	8.14 in	*Overall depth*
	t_w	0.23 in	*Web thickness*
	t_f	0.33 in	*Flange thickness*
	a	5.26 in^2	*Area*
	k_{des}	0.63 in	*Kdes*
	k_{det}	0.81 in	*Kdet*
	k_1	0.56 in	*K1*
Web Hole Type			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	2	*Number of holes per row*
	R_s	3.00 in	*Row Spacing*
	C_s	3.00 in	*Column Spacing*

	Column		HSS4x4x4
Material			
	Name	A500 Gr.B Rect	Material name
	F_y	46.00 ksi	Minimum yield stress of material
	F_u	58.00 ksi	Minimum tensile stress of material
	E	29000.00 ksi	Modulus of elasticity
Member Properties			
	d	4.00 in	Depth
	b	4.00 in	Width
	a	3.37 in^2	Area
	t_{des}	0.23 in	Wall Thickness

	Bottom Brace		HSS3x3x4
Material			
	Name	A500 Gr.B Rect	Material name
	F_y	46.00 ksi	Minimum yield stress of material
	F_u	58.00 ksi	Minimum tensile stress of material
	E	29000.00 ksi	Modulus of elasticity
Member Properties			
	d	3.00 in	Depth
	b	3.00 in	Width
	a	2.44 in^2	Area
	t_{des}	0.23 in	Wall Thickness

BR-12 Top Detail: Components Report

Vertical Brace Diagonal Connection

	Plate		P0.38x4.50x5.00
Material			
	Name	A36	Material name
	F_y	36.00 ksi	Minimum yield stress of material
	F_u	58.00 ksi	Minimum tensile stress of material
	E	29000.00 ksi	Modulus of elasticity
Member Properties			
	d	4.50 in	Width
	t	0.38 in	Thickness
Hole			
	Hole type	Standard	
	D_x	0.81 in	Hole width
	D_y	0.81 in	Hole height
	R	1	Number of rows of holes
	C	2	Number of holes per row
	R_s	3.00 in	Row Spacing
	C_s	3.00 in	Column Spacing

	Knife Plate		P0.38x3.75x6.50
Material			
	Name	A36	Material name
	F_y	36.00 ksi	Minimum yield stress of material
	F_u	58.00 ksi	Minimum tensile stress of material
	E	29000.00 ksi	Modulus of elasticity
Member Properties			
	d	3.75 in	Width
	t	0.38 in	Thickness

	Bottom Gusset		P0.38x5.29x12.00
Material			
	Name	A36	Material name
	F_y	36.00 ksi	Minimum yield stress of material

F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	5.29 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	4.02 in	*Horiz. Clip*
V_clip	4.79 in	*Vert. Clip*

Column Weld	***E70***
Weld Properties	
Type	Double Fillet
Fillet Size	0.25 in

Beam Bolts	***3/4" A325***	
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Gusset Plate Weld	***E70***
Weld Properties	
Type	Single Fillet
Fillet Size	0.19 in

Knife Plate Brace Weld	***E70***
Weld Properties	
Type	Single Fillet
Fillet Size	0.19 in

Beam Weld	***E70***
Weld Properties	
Type	Double Fillet
Fillet Size	0.19 in

Column Weld	***E70***
Weld Properties	
Type	Double Fillet
Fillet Size	0.19 in

RISAConnection version 7.0.1

02/08/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

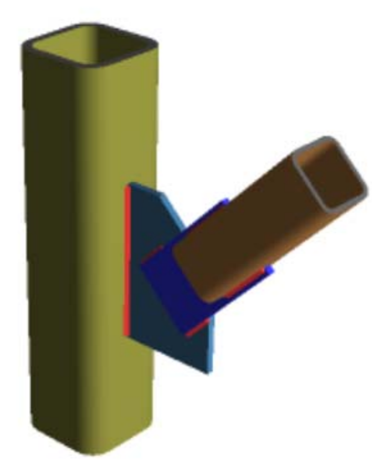
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

BR-14 Bottom Detail	**PASS(UC-0.5)**
BR-14 Top Detail	**PASS(UC-0.3)**

BR-14 Bottom Detail: 3D View

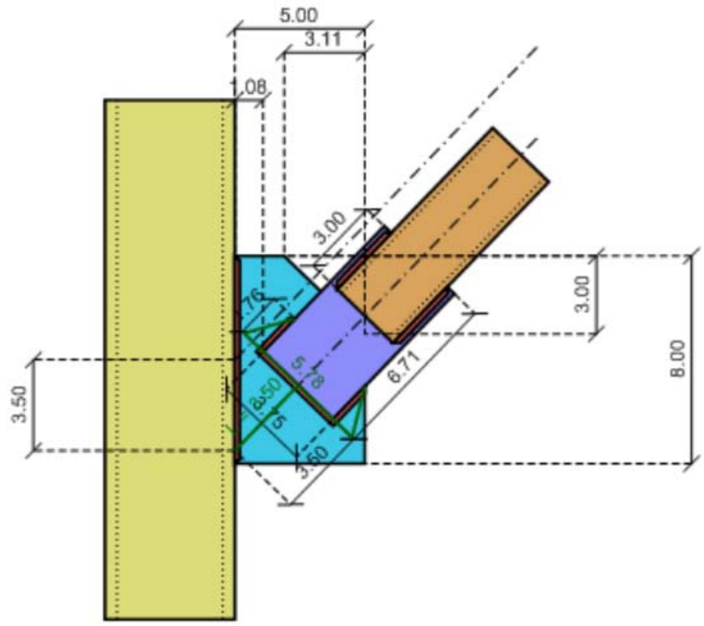
Knee Brace Connection



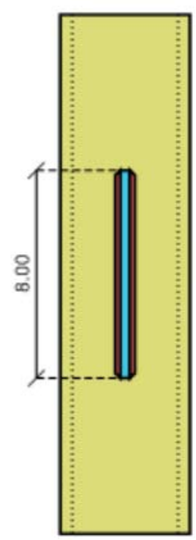
BR-14 Bottom Detail: 2D Views

Knee Brace Connection

Left view

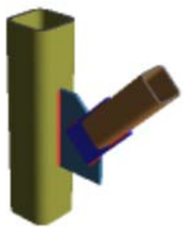


Front view



BR-14 Bottom Detail: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:

Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x5.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x6.71	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	6.00 kips	*Brace Axial (compression)*
Shear Load	4.32 kips	*Calculated Shear Load*
Axial Load	4.17 kips	*Calculated Axial Load (compression)*
Moment Load	1.22 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.47 in	*Column wall thickness*		
B	5.00 in	*Column face width*		
(B - 3 * t) / t	7.75	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	10.75	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
$t_{p\text{-}max}$	0.75 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	3.50 in	*Vertical brace workpoint offset*		
d_{gusset}	8.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.38 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	8.00 in	*Min weld segment length*		
Gusset Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		

Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	1.76 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Plate Shear Yield		4.32 kips	64.80 kips	**0.07**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
Fy	36.00 ksi		*Minimum yield stress of material*		
Agv	3.00 in^2		*Gross area subject to shear*		
φRn	64.80 kips		*Shear yield strength*		

Plate Shear Rupture		4.32 kips	78.30 kips	**0.06**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
Fu	58.00 ksi		*Minimum tensile stress of material*		
Anv	3.00 in^2		*Net area subject to shear*		
φRn	78.30 kips		*Shear rupture strength*		

Plate Axial Yield		4.17 kips	97.20 kips	**0.04**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
Fy	36.00 ksi		*Minimum yield stress of material*		
Ag	3.00 in^2		*Gross area subject to tension*		
φRn	97.20 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.02**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	4.17 kips		*Calculated axial load*		
V_r	4.32 kips		*Calculated shear load*		
Fy	36.00 ksi		*Minimum yield stress of material*		
Ag	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	1.22 kips-ft		*Calculated moment*		
M_c	16.20 kips-ft		*Available moment M_c=φ*(Fy* Z), φ=0.90*		
UC	0.02		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.01**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	4.32 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.00 in^2		*Net area of the plate*		
Z_{net}	6.00 in^3		*Plastic modulus of net section*		
V_c	78.30 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	1.22 kips-ft		*Calculated moment*		
M_c	21.75 kips-ft		*Available moment $M_c = \varphi*(F_u * Znet)$, φ=0.75*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		1.47 kips/in	6.68 kips/in	**0.22**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
V	4.32 kips		*Shear Load*		
P	4.17 kips		*Axial Load*		
M	1.22 kips-ft		*Moment*		
e_{eff}	3.38 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	1.47 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		6.00 kips	64.85 kips	**0.09**	**PASS**
$P_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	3.50 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	38.80		*Plate slenderness*		
F_{cr}	33.26 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.17 in^2		*Gross area of plate (Whitmore section)*		
φP_n	64.85 kips		*Gusset plate compressive strength*		

Gusset Weld Strength		6.00 kips	30.33 kips	**0.20**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	7.26 in		*Weld length*		
φRn	30.33 kips		*Weld strength*		

Brace Weld Strength		6.00 kips	50.11 kips	**0.12**	**PASS**
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table*		

			8-3)		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φR_n	50.11 kips		*Weld strength*		
Knife Plate Flexure		0.09 kips-ft	0.36 kips-ft	**0.26**	**PASS**
$M_n = F_y * Z$		**φ = 0.9**	*(F2-1)*		
R_u	6.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.38 in		*Thickness of knife plate*		
e	0.38 in		*Eccentricity, $e = (t_{plate} + t_{gusset}) / 2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.13 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2 / 4$*		
M_u	0.09 kips-ft		*Required moment demand on knife plate, $M_u = R_u * e / 2$*		
φM_n	0.36 kips-ft		*Available flexural strength*		
Knife Plate Interaction				**0.48**	**PASS**
$P_u/\varphi P_c + 8/9*(M_u/\varphi M_p) <= 1.0$			*(H1-1a)*		
P_u	6.00 kips		*User input brace axial load*		
φP_c	24.51 kips		*Available buckling strength of knife plate*		
M_u	0.09 kips-ft		*Required moment demand on knife plate*		
φM_p	0.36 kips-ft		*Available flexural strength of knife plate*		
HSS Transverse Plastification		4.17 kips	75.78 kips	**0.06**	**PASS**
$R_n = F_y * t^2 / (1 - t_p/B) * (2 l_b/B + 4 * Q_f * (1 - t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	5.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	75.78 kips		*Transverse plastification*		
HSS Flexural Plastification		1.22 kips-ft	28.94 kips-ft	**0.04**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	5.00 in		*Column width*		
β	0.15		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	1.60		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	1.22 kips-ft		*Required flexural plastification*		
φM_n	28.94 kips-ft		*Flexural plastification*		
Knife Plate Buckling		6.00 kips	24.51 kips	**0.24**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	2.10		*Effective length factor*		
L	5.59 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	108.53		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.41 in^2		*Gross area subject to compression*		

E	29000.00 ksi	*Modulus of elasticity*
F_e	24.30 ksi	*Elastic buckling stress (E3-4)*
F_{cr}	19.36 ksi	*Critical stress (E3-2)*
φR_n	24.51 kips	*Compressive strength*

BR-14 Bottom Detail: Members Report

Knee Brace Connection

	Column		***HSS5x5x8***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	5.00 in	*Depth*
	b	5.00 in	*Width*
	a	7.88 in^2	*Area*
	t_{des}	0.47 in	*Wall Thickness*

	Brace		***HSS3x3x4***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.00 in	*Depth*
	b	3.00 in	*Width*
	a	2.44 in^2	*Area*
	t_{des}	0.23 in	*Wall Thickness*

BR-14 Bottom Detail: Components Report

Knee Brace Connection

	Gusset		***P0.38x5.00x8.00***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	5.00 in	*Width*
	t	0.38 in	*Thickness*
Clip			
	H_clip	3.11 in	*Horiz. Clip*
	V_clip	3.00 in	*Vert. Clip*

	Knife Plate		***P0.38x3.75x6.71***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.75 in	*Width*

t	0.38 in	*Thickness*

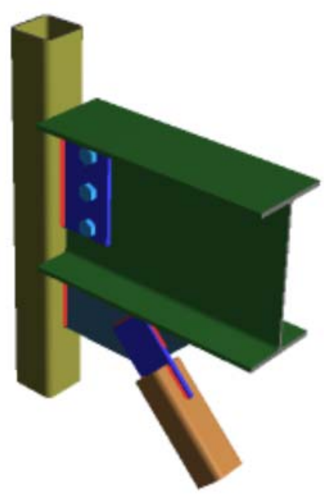
Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

BR-14 Top Detail: 3D View

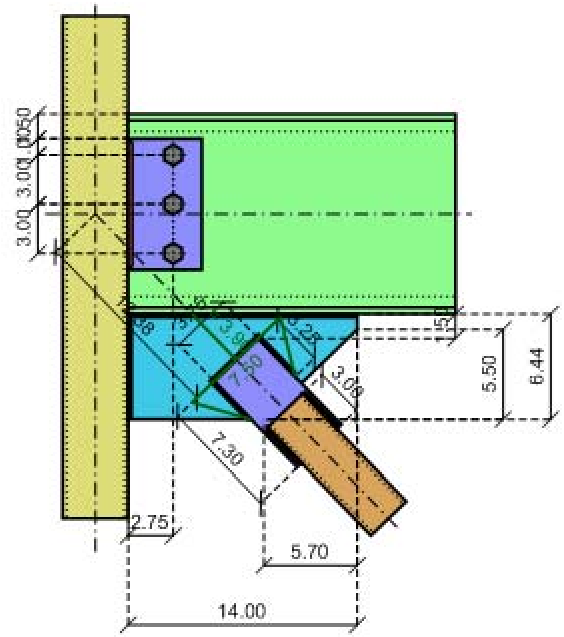
Vertical Brace Diagonal Connection



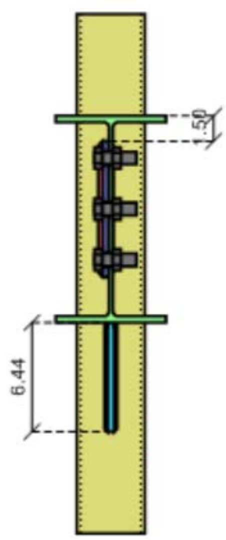
BR-14 Top Detail: 2D Views

Vertical Brace Diagonal Connection

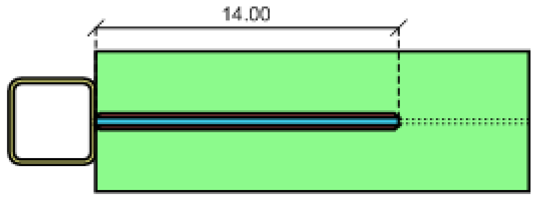
Side view



Front view

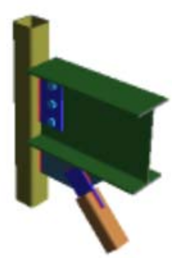


Bottom view



BR-14 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.44x14.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	17.00 kips	*User Input Column Force*
Column Moment	5.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	6.00 kips	*User Input Bottom Brace Axial Force (compression)*

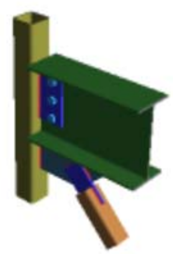
Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	HSS Transverse Plastification	**0.28**	**PASS**
Bottom Gusset/Beam connection	Beam Weld Strength	**0.03**	**PASS**
Bottom Gusset/Column connection	HSS Transverse Plastification	**0.05**	**PASS**
Bottom Gusset/Brace connection	Knife Plate Interaction	**0.33**	**PASS**

BR-14 Top Detail: Beam/Column Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.44x14.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-0.82 kips	*Calculated Shear Load*
Total Axial Load	5.93 kips	*Calculated Axial Load (compression)*
Column Force	17.00 kips	*User Input Column Force*
Column Moment	5.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.23 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		

t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	8.00 in		*Max weld segment length*		
Beam Shear Yield		0.82 kips	84.18 kips	**0.01**	**PASS**
R_n = 0.6 *F_y*A_{gv}*C_v		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	84.18 kips		*Shear yield strength*		
Plate Shear Yield		0.82 kips	64.80 kips	**0.01**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		
Beam Shear Rupture		0.82 kips	64.42 kips	**0.01**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.20 in^2		*Net area subject to shear*		
φR_n	64.42 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		0.82 kips	52.61 kips	**0.02**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.02 in^2		*Net area subject to shear*		
φR_n	52.61 kips		*Shear rupture strength*		
Beam Axial Yield		5.93 kips	344.25 kips	**0.02**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	7.65 in^2		*Gross area subject to tension*		
φR_n	344.25 kips		*Tensile yield strength*		
Plate Axial Yield		5.93 kips	97.20 kips	**0.06**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		
Beam Block Shear		0.82 kips	116.94 kips	**0.01**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	4.04 in^2		*Gross area subject to shear*		
A_{nv}	3.54 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.53 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	116.94 kips		*Block shear strength*		
Plate Block Shear at Beam		0.82 kips	63.94 kips	**0.01**	**PASS**

R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.63 in^2		*Gross area subject to shear*		
A_{nv}	1.80 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.49 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	63.94 kips		*Block shear strength*		
Compression Buckling of the Plate		5.93 kips	93.96 kips	**0.06**	**PASS**
R_n = F_{cr} * A_g		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	2.75 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	25.37		*Plate slenderness check from J4-6*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	444.52 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.80 ksi		*Critical stress (E3-2)*		
φR_n	93.96 kips		*Compressive strength*		
Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	5.93 kips		*Calculated axial load*		
V_r	-0.82 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.60 in		*Vertical eccentricity*		
M_r	0.11 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	16.20 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	5.93 kips		*Calculated axial load*		
V_r	-0.82 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.02 in^2		*Net area of the plate*		
Z_{net}	3.96 in^3		*Plastic modulus of net section*		
V_c	52.61 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.75 in		*Horizontal eccentricity*		
ey	0.60 in		*Vertical eccentricity*		
M_r	0.11 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	14.35 kips-ft		*Available moment M_c= φ*(F_u* Znet), φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Plate Flexural Buckling				**0.09**	**PASS**

$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**$\varphi = 0.90$**	*(AISC 14th Edition)*		
P	5.93 kips		*Calculated axial load*		
V	0.82 kips		*Calculated shear load*		
L	2.75 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	25.37		*Slenderness ratio*		
F_e	444.52 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	34.80 ksi		*Compression stress, per eqn E3-2, Fcr = (0.658 ^ (Fy / Fe)) * Fy*		
A_g	3.00 in^2		*Gross area of the plate*		
λ	0.35		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	2.51 in^3		*Section modulus of net section*		
a	2.75 in		*Design eccentricity*		
P_n	104.40 kips		*Compressive capacity, per eqn E4-1, Pn = Fcr_Comp * Ag*		
V_n	32.83 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.09		*Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$*		

Bolt Bearing on Beam		5.98 kips	53.68 kips	**0.11**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**$\varphi = 0.75$**	*(J3-6b)*		
V	-0.82 kips		*Applied shear force*		
P	5.93 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	5.98 kips		*Resultant shear force*		
Θ	7.92 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	3.70 in		*Vertical edge spacing*		
e_h	2.75 in		*Horizontal edge spacing*		
L_{c_boltA}	2.37 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ)), (e_h /cos(Θ))) - d_c*		
L_{c_boltB}	2.37 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ))) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		5.98 kips	53.68 kips	**0.11**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**$\varphi = 0.75$**	*(J3-6b)*		
V	-0.82 kips		*Applied shear force*		
P	5.93 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	5.98 kips		*Resultant shear force*		

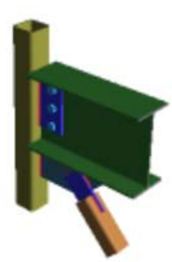
Θ	7.92 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.00 in		*Vertical edge spacing*		
e_h	1.75 in		*Horizontal edge spacing*		
L_{c_boltA}	1.36 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	1.36 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		
Bolt Shear at Beam		5.98 kips	41.45 kips	**0.14**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		**φ = 0.75**	*(J3-1)*		
V	-0.82 kips		*Applied shear force*		
P	5.93 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	5.98 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.77		*Eccentricity coefficient*		
φR_n	41.45 kips		*Bolt shear rupture strength*		
Bolt Group Eccentricity			***0.77***		
Elastic method			*(AISC 14^{th} p.7-6)*		
C	0.77		*Coefficient (2.3167 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.60 in		*Vertical eccentricity*		
Ang	7.92		*Angle of force in degrees, relative X axis*		
Weld at Column		5.98 kips	78.38 kips	**0.08**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φR_n	78.38 kips		*Weld strength*		
HSS Transverse Plastification		5.93 kips	21.52 kips	**0.28**	**PASS**

$R_n = F_y{*}t^2 / (1 - t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1 - t_p/B)^{0.5})$		$\varphi = 1.00$	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	8.45 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		$\varphi = 1.0$	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	8.45 kips-ft		*Flexural plastification*		

BR-14 Top Detail: Bot Gusset/Beam Report

LRFD

Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.44x14.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	3.24 kips	*Calculated Shear Load*
Axial Load	2.82 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	14.00 in		*Min weld segment length*		

Plate Shear Yield		3.24 kips	113.40 kips	**0.03**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**$\varphi = 1.00$**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	5.25 in^2		*Gross area subject to shear*		
φR_n	113.40 kips		*Shear yield strength*		

Plate Shear Rupture		3.24 kips	137.02 kips	**0.02**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	5.25 in^2		*Net area subject to shear*		
φR_n	137.02 kips		*Shear rupture strength*		

Plate Axial Yield		2.82 kips	170.10 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	5.25 in^2		*Gross area subject to tension*		
φR_n	170.10 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	2.82 kips		*Calculated axial load*		
V_r	3.24 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	5.25 in^2		*Gross area of the plate*		
Z_{pl}	18.38 in^3		*Plastic modulus of the shear plate*		
P_c	170.10 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	113.40 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	49.61 kips-ft		*Available moment $M_c = \varphi*(F_y * Z)$, $\varphi = 0.90$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	3.24 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	5.25 in^2		*Net area of the plate*		
Z_{net}	18.38 in^3		*Plastic modulus of net section*		
V_c	137.02 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	66.61 kips-ft		*Available moment $M_c = \varphi*(F_u * Z_{net})$, $\varphi = 0.75$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

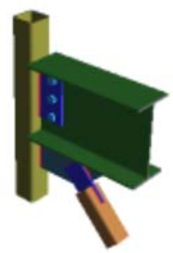
Beam Weld Strength		3.24 kips	93.54 kips	**0.03**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		

L	14.00 in		*Weld length*		
φRn	93.54 kips		*Weld strength*		

Beam Web Yielding		2.82 kips	200.10 kips	**0.01**	PASS
R_n = (5 * k + N) * F_y * t_w		**φ = 1.00**	*(J10-2)*		
k	0.68 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	14.00 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.23 in		*Beam web thickness*		
φRn	200.10 kips		*Beam web local yielding*		

BR-14 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.44x14.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	1.49 kips	*Calculated Shear Load*
Axial Load	0.93 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					PASS
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
F_{y-max}	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
t_{p-max}	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					PASS
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		

D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	6.44 in		*Min weld segment length*		

Plate Shear Yield		1.49 kips	52.16 kips	**0.03**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**$\varphi = 1.00$**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.41 in^2		*Gross area subject to shear*		
φR_n	52.16 kips		*Shear yield strength*		

Plate Shear Rupture		1.49 kips	63.03 kips	**0.02**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.41 in^2		*Net area subject to shear*		
φR_n	63.03 kips		*Shear rupture strength*		

Plate Axial Yield		0.93 kips	78.24 kips	**0.01**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.41 in^2		*Gross area subject to tension*		
φR_n	78.24 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	0.93 kips		*Calculated axial load*		
V_r	1.49 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.41 in^2		*Gross area of the plate*		
Z_{pl}	3.89 in^3		*Plastic modulus of the shear plate*		
P_c	78.24 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	52.16 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	10.50 kips-ft		*Available moment $M_c = \varphi*(F_y * Z)$, φ=0.90*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

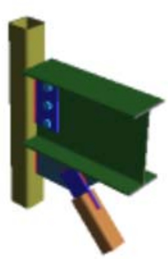
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	1.49 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.41 in^2		*Net area of the plate*		
Z_{net}	3.89 in^3		*Plastic modulus of net section*		
V_c	63.03 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	14.09 kips-ft		*Available moment $M_c = \varphi*(F_u * Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		1.49 kips	43.03 kips	**0.03**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					

C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	6.44 in		*Weld length*		
φRn	43.03 kips		*Weld strength*		
HSS Transverse Plastification		0.93 kips	19.37 kips	**0.05**	**PASS**
$R_n = F_y * t^2 / (1 - t_p/B) * (2 l_b/B + 4 * Q_f * (1 - t_p/B)^{0.5})$		φ **= 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	6.44 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	19.37 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	6.04 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ **= 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	6.44 in		*Depth of plate*		
η	1.61		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	6.04 kips-ft		*Flexural plastification*		

BR-14 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	HSS3x3x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Knife Plate	P0.38x3.75x7.30	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x6.44x14.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	6.00 kips	*Brace Axial (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Gusset Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				

D	0.19 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.25 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.75 in		*Max weld segment length*		

Brace Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	3.00 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		6.00 kips	91.09 kips	**0.07**	**PASS**
$P_n = Fy*A_g$		**φ = 0.9**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	3.90 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	17.99		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.81 in^2		*Gross area of plate (Whitmore section)*		
$φP_n$	91.09 kips		*Gusset plate compressive strength*		

Knife Plate Buckling		6.00 kips	43.42 kips	**0.14**	**PASS**
$R_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	1.00		*Effective length factor*		
L	3.27 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	30.24		*Plate slenderness, check from (J4-6)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	1.41 in^2		*Gross area subject to compression*		
E	29000.00 ksi		*Modulus of elasticity*		
F_e	312.92 ksi		*Elastic buckling stress (E3-4)*		
F_{cr}	34.31 ksi		*Critical stress (E3-2)*		
$φR_n$	43.42 kips		*Compressive strength*		

Knife Plate Flexure		0.09 kips-ft	0.36 kips-ft	**0.26**	**PASS**
$M_n = Fy * Z$		**φ = 0.9**	*(F2-1)*		
R_u	6.00 kips		*User Input Brace Axial Load*		
t_{gusset}	0.38 in		*Thickness of gusset plate*		
t_{plate}	0.38 in		*Thickness of knife plate*		
e	0.38 in		*Eccentricity, $e = (t_{plate} + t_{gusset})/2$*		
W_p	3.75 in		*Width of knife plate*		
F_y	36.00 ksi		*Minimum yield stress of material*		
Z	0.13 in^3		*Plastic section modulus, $Z = W_p * (t_{plate})^2/4$*		
M_u	0.09 kips-ft		*Required moment demand on knife plate, $M_u=R_u * e / 2$*		
$φM_n$	0.36 kips-ft		*Available flexural strength*		

Knife Plate Interaction				**0.33**	**PASS**
$0.5*P_u/\varphi P_c + M_u/\varphi M_p <= 1.0$			*(H1-1b)*		
P_u	6.00 kips		*User input brace axial load*		
φP_c	43.42 kips		*Available buckling strength of knife plate*		
M_u	0.09 kips-ft		*Required moment demand on knife plate*		
φM_p	0.36 kips-ft		*Available flexural strength of knife plate*		

Gusset Weld Strength		6.00 kips	42.77 kips	**0.14**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	10.24 in		*Weld length*		
φRn	42.77 kips		*Weld strength*		

Brace Weld Strength		6.00 kips	50.11 kips	**0.12**	**PASS**
$\varphi Rn = 4 * C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.00 in		*Weld length*		
φRn	50.11 kips		*Weld strength*		

BR-14 Top Detail: Members Report

Vertical Brace Diagonal Connection

	Beam		***W12x26***
Material			
	Name	A992	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	65.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b_f	6.49 in	*Flange width*
	d	12.20 in	*Overall depth*
	t_w	0.23 in	*Web thickness*
	t_f	0.38 in	*Flange thickness*
	a	7.65 in^2	*Area*
	k_{des}	0.68 in	*Kdes*
	k_{det}	1.06 in	*Kdet*
	k_1	0.75 in	*K1*
Web Hole Type			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	3	*Number of holes per row*
	R_s	3.00 in	*Row Spacing*
	C_s	3.00 in	*Column Spacing*

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Bottom Brace		***HSS3x3x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.00 in	*Depth*
b	3.00 in	*Width*
a	2.44 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

BR-14 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.50x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.50 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Knife Plate		***P0.38x3.75x7.30***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	3.75 in	*Width*
t	0.38 in	*Thickness*

Bottom Gusset		***P0.38x6.44x14.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*

F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	6.44 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	5.70 in	*Horiz. Clip*
V_clip	5.50 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Gusset Plate Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Knife Plate Brace Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

RISAConnection version 7.0.1

02/07/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

X-1 Bottom Detail	**FAIL(UC-1.0)**
X-1 Top Detail	**FAIL(UC-0.8)**

X-1 Bottom Detail: 3D View

Vertical Brace Diagonal Connection

Per AISC K1-3, the minimum plate thickness must be less than 0.38 in. We need a 0.5 in gusset plate for buckling of the whitemore section. This limit (AISC K1-3) is to negate the requirement of plastification of the HSS wall. By inspection, this failure mechanism is not applicable to this situation.

Furthermore, by review of punching shear and stresses on the HSS wall (Tf / Cf = 25.0 kip), base connection governs for brace X1:

Area of Gusset PL at face of HSS
A = (0.5 in)*(23.91 in) = 11.9 in^2

Stress acting on face of HSS
F_F = (25.0 kip) / (11.9 in^2) = 2.10 ksi

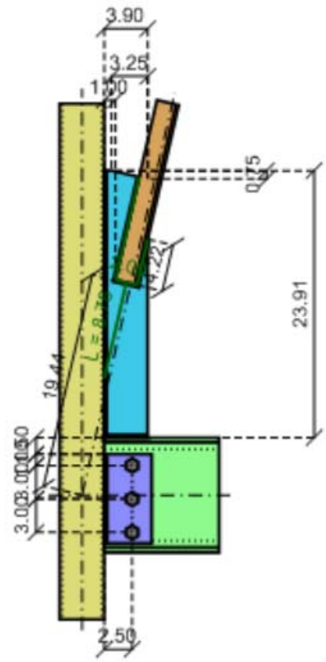
Shear strength of HSS
F_R = (0.90)*(0.6)*(36 ksi) = 19.44 ksi > F_F OK [U ~ 10%]

The overall stress on the HSS wall is approx. 10% of the capacity and is deemed more than acceptable. It should also be noted that the full brace force is used in the above which is a conservative check.

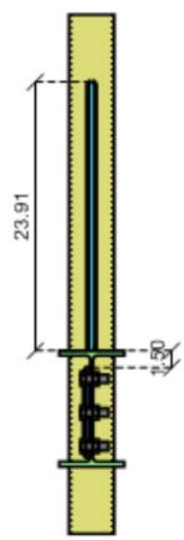
X-1 Bottom Detail: 2D Views

Vertical Brace Diagonal Connection

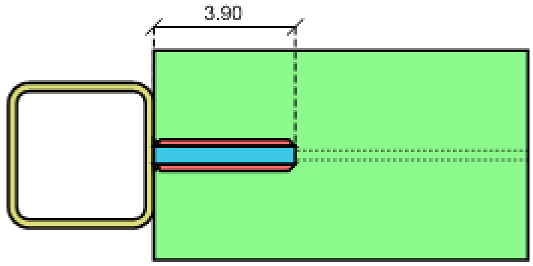
Side view



Front view



Top view



X-1 Bottom Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.25x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.50x23.91x3.90	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	22.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	30.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Top Brace Axial	25.00 kips	*User Input Top Brace Axial Force (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	Plate Flexural Buckling	**0.98**	**PASS**
Top Gusset/Beam connection	Plate Axial Yield	**0.12**	**PASS**
Top Gusset/Column connection	HSS Punching Shear		**FAIL**
Top Gusset/Brace connection	Gusset Plate Compression (Whitmore)	**0.81**	**PASS**

X-1 Bottom Detail: Beam/Column Report

Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.25x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.50x23.91x3.90	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	29.33 kips	*Calculated Shear Load*
Total Axial Load	7.85 kips	*Calculated Axial Load (compression)*
Column Force	30.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.26 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		
t	0.38 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		

t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	8.00 in		*Max weld segment length*		
Beam Shear Yield		29.33 kips	80.34 kips	**0.37**	**PASS**
$R_n = 0.6 * F_y * A_{gv} * C_v$		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.68 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φRn	80.34 kips		*Shear yield strength*		
Plate Shear Yield		29.33 kips	64.80 kips	**0.45**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φRn	64.80 kips		*Shear yield strength*		
Beam Shear Rupture		29.33 kips	58.37 kips	**0.50**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.00 in^2		*Net area subject to shear*		
φRn	58.37 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		29.33 kips	52.61 kips	**0.56**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.02 in^2		*Net area subject to shear*		
φRn	52.61 kips		*Shear rupture strength*		
Beam Axial Yield		7.85 kips	342.45 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	7.61 in^2		*Gross area subject to tension*		
φRn	342.45 kips		*Tensile yield strength*		
Plate Axial Yield		7.85 kips	97.20 kips	**0.08**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φRn	97.20 kips		*Tensile yield strength*		
Beam Block Shear		29.33 kips	125.37 kips	**0.23**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	4.41 in^2		*Gross area subject to shear*		
A_{nv}	3.84 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.54 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φRn	125.37 kips		*Block shear strength*		
Plate Block Shear at Beam		29.33 kips	63.94 kips	**0.46**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.63 in^2		*Gross area subject to shear*		

A_{nv}	1.80 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.49 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	63.94 kips		*Block shear strength*		

Compression Buckling of the Plate		7.85 kips	97.20 kips	**0.08**	**PASS**
$R_n = F_y * A_g$		**φ = 0.9**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	2.50 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	23.07		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	3.00 in^2		*Gross area subject to compression*		
φR_n	97.20 kips		*Compressive strength*		

Plate Flexural Yield				**0.40**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	7.85 kips		*Calculated axial load*		
V_r	29.33 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.50 in		*Horizontal eccentricity*		
ey	-0.35 in		*Vertical eccentricity*		
M_r	5.88 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	16.20 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.40		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.48**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	7.85 kips		*Calculated axial load*		
V_r	29.33 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.02 in^2		*Net area of the plate*		
Z_{net}	3.96 in^3		*Plastic modulus of net section*		
V_c	52.61 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.50 in		*Horizontal eccentricity*		
ey	-0.35 in		*Vertical eccentricity*		
M_r	5.88 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	14.35 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.48		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Plate Flexural Buckling				**0.98**	**PASS**
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**φ = 0.90**	*(AISC 14th Edition)*		
P	7.85 kips		*Calculated axial load*		
V	29.33 kips		*Calculated shear load*		
L	2.50 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	23.07		*Slenderness ratio*		

F_e	537.87 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	35.01 ksi		*Compression stress = F_y when KL/r <= 25, per J4.4*		
A_g	3.00 in^2		*Gross area of the plate*		
λ	0.33		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	2.51 in^3		*Section modulus of net section*		
a	2.50 in		*Design eccentricity*		
P_n	108.00 kips		*Compressive capacity, per eqn J4-1, Pn = Fy * Ag*		
V_n	36.11 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.98		*Unity check per interaction equation, P/(Pn*Φ) + V/(Vn*Φ) <= 1*		

Bolt Bearing on Beam		30.37 kips	53.68 kips	**0.57**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	29.33 kips		*Applied shear force*		
P	7.85 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	30.37 kips		*Resultant shear force*		
Θ	75.02 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.80 in		*Vertical edge spacing*		
e_h	2.50 in		*Horizontal edge spacing*		
L_{c_boltA}	1.46 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.28 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		30.37 kips	51.17 kips	**0.59**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	29.33 kips		*Applied shear force*		
P	7.85 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	30.37 kips		*Resultant shear force*		
Θ	75.02 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		

F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.00 in		*Vertical edge spacing*		
e_h	1.75 in		*Horizontal edge spacing*		
L_{c_boltA}	0.63 in		*Minimum clear distance for the corner edge bolt: $Lc\_boltA = min((e_v /sin(\Theta), (e_h /cos(\Theta)) - d_c$*		
L_{c_boltB}	2.28 in		*Minimum clear distance for the side edge bolts: $Lc\_boltB = min((s_v - 0.5 * d_v /sin(\Theta)), (e_h /cos(\Theta))) - d_c$*		
R_{n_boltA}	20.52 kips		*Available bearing strength for the corner edge bolt: $Rn\_boltA = min[(1.5* Lc\_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: $Rn\_boltB = min[(1.5* Lc\_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	51.17 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		30.37 kips	53.01 kips	**0.57**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		**φ = 0.75**	*(J3-1)*		
V	29.33 kips		*Applied shear force*		
P	7.85 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	30.37 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.99		*Eccentricity coefficient*		
φR_n	53.01 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***0.99***		
Elastic method			*(AISC 14^{th} p.7-6)*		
C	0.99		*Coefficient (2.9626 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.35 in		*Vertical eccentricity*		
Ang	75.02		*Angle of force in degrees, relative X axis*		

Weld at Column		30.37 kips	78.38 kips	**0.39**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φRn	78.38 kips		*Weld strength*		

HSS Transverse Plastification		7.85 kips	21.52 kips	**0.36**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		

B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	21.52 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	8.45 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	8.45 kips-ft		*Flexural plastification*		

X-1 Bottom Detail: Top Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.25x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.50x23.91x3.90	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.78 kips	*Calculated Shear Load*
Axial Load	7.33 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		**Required**	**Available**	**Unity Check**	**Result**
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.44 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	3.90 in		*Min weld segment length*		
Plate Shear Yield		2.78 kips	42.10 kips	**0.07**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	1.95 in^2		*Gross area subject to shear*		
φR_n	42.10 kips		*Shear yield strength*		
Plate Shear Rupture		2.78 kips	50.87 kips	**0.05**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		

A_{nv}	$1.95\ in^2$		*Net area subject to shear*		
φR_n	50.87 kips		*Shear rupture strength*		
Plate Axial Yield		7.33 kips	63.15 kips	**0.12**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	$1.95\ in^2$		*Gross area subject to tension*		
φR_n	63.15 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.02**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	7.33 kips		*Calculated axial load*		
V_r	2.78 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	$1.95\ in^2$		*Gross area of the plate*		
Z_{pl}	$1.90\ in^3$		*Plastic modulus of the shear plate*		
P_c	63.15 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	42.10 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	5.13 kips-ft		*Available moment $M_c=\varphi*(F_y*Z)$, φ=0.90*		
UC	0.02		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	2.78 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	$1.95\ in^2$		*Net area of the plate*		
Z_{net}	$1.90\ in^3$		*Plastic modulus of net section*		
V_c	50.87 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	6.89 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Beam Weld Strength		2.78 kips	26.05 kips	**0.11**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	3.90 in		*Weld length*		
φR_n	26.05 kips		*Weld strength*		
Beam Web Yielding		7.33 kips	98.78 kips	**0.07**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		**φ = 1.00**	*(J10-2)*		
k	0.74 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	3.90 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		

t_w	0.26 in	*Beam web thickness*
φR_n	98.78 kips	*Beam web local yielding*

X-1 Bottom Detail: Top Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.25x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.50x23.91x3.90	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	17.03 kips	*Calculated Shear Load*
Axial Load	2.85 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**FAIL**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Fail**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.50 in		*Plate thickness*		
$t_{p\text{-}max}$	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	23.91 in		*Min weld segment length*		
Plate Shear Yield		17.03 kips	258.23 kips	**0.07**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	11.96 in^2		*Gross area subject to shear*		

φR_n	258.23 kips		*Shear yield strength*		
Plate Shear Rupture		17.03 kips	312.03 kips	**0.05**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		$\varphi = 0.75$	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	11.96 in^2		*Net area subject to shear*		
φR_n	312.03 kips		*Shear rupture strength*		
Plate Axial Yield		2.85 kips	387.35 kips	**0.01**	**PASS**
$R_n = F_y * A_g$		$\varphi = 0.90$	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	11.96 in^2		*Gross area subject to tension*		
φR_n	387.35 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	2.85 kips		*Calculated axial load*		
V_r	17.03 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	11.96 in^2		*Gross area of the plate*		
Z_{pl}	71.46 in^3		*Plastic modulus of the shear plate*		
P_c	387.35 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	258.23 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	192.95 kips-ft		*Available moment* $M_c=\varphi*(F_y*Z)$, $\varphi=0.90$		
UC	0.00		*Unity check per interaction equation,* $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	17.03 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	11.96 in^2		*Net area of the plate*		
Z_{net}	71.46 in^3		*Plastic modulus of net section*		
V_c	312.03 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	259.05 kips-ft		*Available moment* $M_c= \varphi*(F_u*Znet)$, $\varphi=0.75$		
UC	0.00		*Unity check per interaction equation,* $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		
Column Weld Strength		17.03 kips	159.76 kips	**0.11**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$ **Double Fillet**					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ **(AISC 14th Eqn 8-2a)**					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	23.91 in		*Weld length*		
φR_n	159.76 kips		*Weld strength*		
HSS Transverse Plastification		2.85 kips	44.80 kips	**0.06**	**PASS**

$R_n = F_y{*}t^2 / (1{-}t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1{-}t_p/B)^{0.5})$		$\varphi = 1.00$	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.50 in		*Plate thickness*		
l_b	23.91 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	44.80 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	49.75 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		$\varphi = 1.0$	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	23.91 in		*Depth of plate*		
η	5.98		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	49.75 kips-ft		*Flexural plastification*		

X-1 Bottom Detail: Top Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.25x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.50x23.91x3.90	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	25.00 kips	*Brace Axial (compression)*
Brace Moment	1.75 kips-ft	*Brace Moment (not used for design)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.25 in		*Max Size Allowed*		
t	0.25 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.25 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		

D	0.19 in		*Weld size*		
L_{max}	9.47 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		25.00 kips	30.86 kips	**0.81**	**PASS**
$P_n = F_{cr} * A_g$		**$\varphi = 0.9$**	*(E3-1)*		
K	0.50		*Effective length factor*		
L	8.78 in		*Unbraced length*		
r	0.14 in		*Radius of gyration*		
KL/r	30.40		*Plate slenderness*		
F_{cr}	34.29 ksi		*Flexural buckling stress (E3-2)*		
A_g	1.00 in^2		*Gross area of plate (Whitmore section)*		
φP_n	30.86 kips		*Gusset plate compressive strength*		

Brace Weld Strength		25.00 kips	65.54 kips	**0.38**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	15.70 in		*Weld length*		
φRn	65.54 kips		*Weld strength*		

X-1 Bottom Detail: Members Report

Vertical Brace Diagonal Connection

	Beam		***W10x26***
Material			
	Name	A992	*Material name*
	F_y	50.00 ksi	*Minimum yield stress of material*
	F_u	65.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b_f	5.77 in	*Flange width*
	d	10.30 in	*Overall depth*
	t_w	0.26 in	*Web thickness*
	t_f	0.44 in	*Flange thickness*
	a	7.61 in^2	*Area*
	k_{des}	0.74 in	*Kdes*
	k_{det}	1.06 in	*Kdet*
	k_1	0.69 in	*K1*
Web Hole Type			
	Hole type	Standard	
	D_x	0.81 in	*Hole width*
	D_y	0.81 in	*Hole height*
	R	1	*Number of rows of holes*
	C	3	*Number of holes per row*
	R_s	3.00 in	*Row Spacing*
	C_s	3.00 in	*Column Spacing*

	Column		***HSS4x4x4***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			

d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*
t_{des}	0.23 in	*Wall Thickness*

Top Brace		***L2x2x4***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	2.00 in	*Flange width*
d	2.00 in	*Overall depth*
a	0.94 in^2	*Area*
t_{f1}	0.25 in	*Flange thickness*
t_{f2}	0.25 in	*Flange thickness*
k_{des}	0.50 in	*Kdes*
k_{det}	0.50 in	*Kdet*

X-1 Bottom Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.25x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.25 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Top Gusset		***P0.50x23.91x3.90***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	23.91 in	*Width*
t	0.50 in	*Thickness*
Clip		
H_clip	3.25 in	*Horiz. Clip*
V_clip	0.75 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

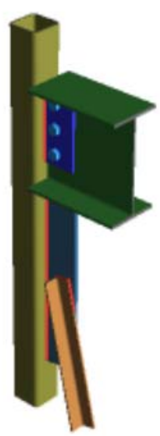
Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

X-1 Top Detail: 3D View

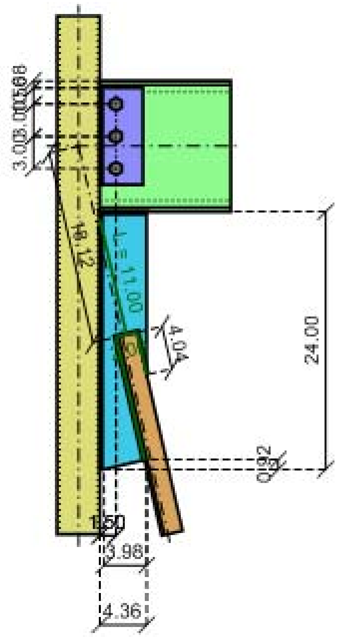
Vertical Brace Diagonal Connection



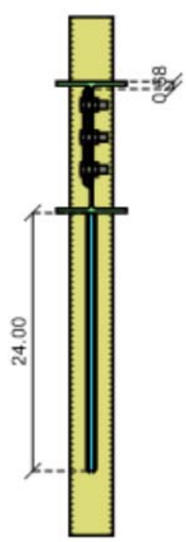
X-1 Top Detail: 2D Views

Vertical Brace Diagonal Connection

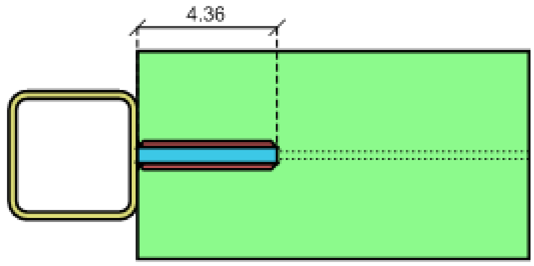
Side view



Front view

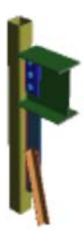


Bottom view



X-1 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.50x24.00x4.36	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	7.50 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	7.50 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	25.00 kips	*User Input Bottom Brace Axial Force (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	HSS Transverse Plastification	**0.34**	**PASS**
Bottom Gusset/Beam connection	Plate Axial Yield	**0.12**	**PASS**
Bottom Gusset/Column connection	HSS Punching Shear		**FAIL**
Bottom Gusset/Brace connection	Gusset Plate Compression (Whitmore)	**0.83**	**PASS**

X-1 Top Detail: Beam/Column Report

Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.50x24.00x4.36	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-0.71 kips	*Calculated Shear Load*
Total Axial Load	7.69 kips	*Calculated Axial Load (compression)*
Column Force	7.50 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.23 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		
t	0.38 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		

t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	9.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	9.00 in		*Max weld segment length*		
Beam Shear Yield		0.71 kips	84.18 kips	**0.01**	**PASS**
R_n = 0.6 *F_y*A_{gv}*C_v		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	84.18 kips		*Shear yield strength*		
Plate Shear Yield		0.71 kips	72.90 kips	**0.01**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.38 in^2		*Gross area subject to shear*		
φR_n	72.90 kips		*Shear yield strength*		
Beam Shear Rupture		0.71 kips	64.42 kips	**0.01**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.20 in^2		*Net area subject to shear*		
φR_n	64.42 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		0.71 kips	62.40 kips	**0.01**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.39 in^2		*Net area subject to shear*		
φR_n	62.40 kips		*Shear rupture strength*		
Beam Axial Yield		7.69 kips	344.25 kips	**0.02**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	7.65 in^2		*Gross area subject to tension*		
φR_n	344.25 kips		*Tensile yield strength*		
Plate Axial Yield		7.69 kips	109.35 kips	**0.07**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to tension*		
φR_n	109.35 kips		*Tensile yield strength*		
Beam Block Shear		0.71 kips	79.89 kips	**0.01**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	3.02 in^2		*Gross area subject to shear*		
A_{nv}	2.52 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.24 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	79.89 kips		*Block shear strength*		
Plate Block Shear at Beam		0.71 kips	79.21 kips	**0.01**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		

A_{nv}	$1.99\ in^2$		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	$0.77\ in^2$		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	79.21 kips		*Block shear strength*		

Compression Buckling of the Plate		7.69 kips	109.35 kips	**0.07**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.9$**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	1.50 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	13.84		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	$3.38\ in^2$		*Gross area subject to compression*		
φR_n	109.35 kips		*Compressive strength*		

Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	7.69 kips		*Calculated axial load*		
V_r	-0.71 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	$3.38\ in^2$		*Gross area of the plate*		
Z_{pl}	$7.59\ in^3$		*Plastic modulus of the shear plate*		
P_c	109.35 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	72.90 kips		*Available shear strength (see check 'Shear Yield')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.92 in		*Vertical eccentricity*		
M_r	0.50 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.50 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	7.69 kips		*Calculated axial load*		
V_r	-0.71 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	$2.39\ in^2$		*Net area of the plate*		
Z_{net}	$5.55\ in^3$		*Plastic modulus of net section*		
V_c	62.40 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.92 in		*Vertical eccentricity*		
M_r	0.50 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.13 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Plate Flexural Buckling				**0.08**	**PASS**
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**$\varphi = 0.90$**	*(AISC 14th Edition)*		
P	7.69 kips		*Calculated axial load*		
V	0.71 kips		*Calculated shear load*		
L	1.50 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	13.84		*Slenderness ratio*		

F_e	1494.08 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	35.64 ksi		*Compression stress = F_y when KL/r <= 25, per J4.4*		
A_g	3.38 in^2		*Gross area of the plate*		
λ	0.33		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	3.74 in^3		*Section modulus of net section*		
a	1.50 in		*Design eccentricity*		
P_n	121.50 kips		*Compressive capacity, per eqn J4-1, Pn = Fy * Ag*		
V_n	89.67 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.08		*Unity check per interaction equation, P/(Pn*Φ) + V/(Vn*Φ) <= 1*		

Bolt Bearing on Beam		7.72 kips	53.68 kips	**0.14**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-0.71 kips		*Applied shear force*		
P	7.69 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	7.72 kips		*Resultant shear force*		
Θ	5.27 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	4.02 in		*Vertical edge spacing*		
e_h	1.50 in		*Horizontal edge spacing*		
L_{c_boltA}	1.10 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	1.10 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Bearing on Plate at Beam		7.72 kips	53.68 kips	**0.14**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-0.71 kips		*Applied shear force*		
P	7.69 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	7.72 kips		*Resultant shear force*		
Θ	5.27 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		

F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.50 in		*Vertical edge spacing*		
e_h	2.50 in		*Horizontal edge spacing*		
L_{c_boltA}	2.10 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.10 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ))) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		7.72 kips	36.85 kips	**0.21**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		φ **= 0.75**	*(J3-1)*		
V	-0.71 kips		*Applied shear force*		
P	7.69 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	7.72 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.69		*Eccentricity coefficient*		
φR_n	36.85 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***0.69***		
Elastic method			*(AISC 14th p.7-6)*		
C	0.69		*Coefficient (2.0594 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.92 in		*Vertical eccentricity*		
Ang	5.27		*Angle of force in degrees, relative X axis*		

Weld at Column		7.72 kips	88.18 kips	**0.09**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ**=0.75 (AISC 14th Eqn 8-2a)**					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	9.00 in		*Weld length*		
φ**Rn**	88.18 kips		*Weld strength*		

HSS Transverse Plastification		7.69 kips	22.89 kips	**0.34**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		φ **= 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	9.00 in		*Plate length*		

B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	22.89 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	10.05 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	9.00 in		*Depth of plate*		
η	2.25		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	10.05 kips-ft		*Flexural plastification*		

X-1 Top Detail: Bot Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.50x24.00x4.36	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.93 kips	*Calculated Shear Load*
Axial Load	8.21 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

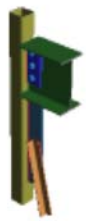
Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	4.36 in		*Min weld segment length*		
Plate Shear Yield		2.93 kips	47.06 kips	**0.06**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.18 in^2		*Gross area subject to shear*		
φR_n	47.06 kips		*Shear yield strength*		
Plate Shear Rupture		2.93 kips	56.86 kips	**0.05**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.18 in^2		*Net area subject to shear*		

φR_n	56.86 kips		*Shear rupture strength*		
Plate Axial Yield		8.21 kips	70.59 kips	**0.12**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.18 in^2		*Gross area subject to tension*		
φR_n	70.59 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.02**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	8.21 kips		*Calculated axial load*		
V_r	2.93 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.18 in^2		*Gross area of the plate*		
Z_{pl}	2.37 in^3		*Plastic modulus of the shear plate*		
P_c	70.59 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	47.06 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	6.41 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.02		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	2.93 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.18 in^2		*Net area of the plate*		
Z_{net}	2.37 in^3		*Plastic modulus of net section*		
V_c	56.86 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	8.60 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Beam Weld Strength		2.93 kips	29.11 kips	**0.10**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	4.36 in		*Weld length*		
φR_n	29.11 kips		*Weld strength*		
Beam Web Yielding		8.21 kips	89.21 kips	**0.09**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		**$\varphi = 1.00$**	*(J10-2)*		
k	0.68 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	4.36 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.23 in		*Beam web thickness*		

φR_n	89.21 kips	*Beam web local yielding*

X-1 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.50x24.00x4.36	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	16.15 kips	*Calculated Shear Load*
Axial Load	2.69 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**FAIL**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Fail**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.50 in		*Plate thickness*		
$t_{p\text{-}max}$	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	24.00 in		*Min weld segment length*		
Plate Shear Yield		16.15 kips	259.20 kips	**0.06**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	12.00 in^2		*Gross area subject to shear*		
φR_n	259.20 kips		*Shear yield strength*		

Plate Shear Rupture		16.15 kips	313.20 kips	**0.05**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		φ **= 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	12.00 in^2		*Net area subject to shear*		
φR_n	313.20 kips		*Shear rupture strength*		

Plate Axial Yield		2.69 kips	388.80 kips	**0.01**	**PASS**
$R_n = F_y * A_g$		φ **= 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	12.00 in^2		*Gross area subject to tension*		
φR_n	388.80 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	2.69 kips		*Calculated axial load*		
V_r	16.15 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	12.00 in^2		*Gross area of the plate*		
Z_{pl}	72.00 in^3		*Plastic modulus of the shear plate*		
P_c	388.80 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	259.20 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	194.40 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	16.15 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	12.00 in^2		*Net area of the plate*		
Z_{net}	72.00 in^3		*Plastic modulus of net section*		
V_c	313.20 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	261.00 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		16.15 kips	160.36 kips	**0.10**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	24.00 in		*Weld length*		
φR_n	160.36 kips		*Weld strength*		

HSS Transverse Plastification		2.69 kips	44.93 kips	**0.06**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		φ **= 1.00**	*(K1-12)*		

F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.50 in		*Plate thickness*		
l_b	24.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	44.93 kips		*Transverse plastification*		

HSS Flexural Plastification		0.00 kips-ft	50.08 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	24.00 in		*Depth of plate*		
η	6.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	50.08 kips-ft		*Flexural plastification*		

X-1 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.50x24.00x4.36	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	25.00 kips	*Brace Axial (compression)*
Brace Moment	1.75 kips-ft	*Brace Moment (not used for design)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.25 in		*Max Size Allowed*		
t	0.25 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.25 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	11.74 in		*Max weld segment length*		
Gusset Plate Compression (Whitmore)		25.00 kips	30.02 kips	**0.83**	**PASS**

$P_n = F_{cr} * A_g$		$\varphi = 0.9$	*(E3-1)*		
K	0.50		*Effective length factor*		
L	11.00 in		*Unbraced length*		
r	0.14 in		*Radius of gyration*		
KL/r	38.10		*Plate slenderness*		
F_{cr}	33.35 ksi		*Flexural buckling stress (E3-2)*		
A_g	1.00 in^2		*Gross area of plate (Whitmore section)*		
φP_n	30.02 kips		*Gusset plate compressive strength*		

Brace Weld Strength		25.00 kips	74.25 kips	**0.34**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	17.78 in		*Weld length*		
φRn	74.25 kips		*Weld strength*		

X-1 Top Detail: Members Report

Vertical Brace Diagonal Connection

Beam		***W12x26***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	6.49 in	*Flange width*
d	12.20 in	*Overall depth*
t_w	0.23 in	*Web thickness*
t_f	0.38 in	*Flange thickness*
a	7.65 in^2	*Area*
k_{des}	0.68 in	*Kdes*
k_{det}	1.06 in	*Kdet*
k_1	0.75 in	*K1*
Web Hole Type		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*

t_{des}	0.23 in	*Wall Thickness*

Bottom Brace		***L2x2x4***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	2.00 in	*Flange width*
d	2.00 in	*Overall depth*
a	0.94 in^2	*Area*
t_{f1}	0.25 in	*Flange thickness*
t_{f2}	0.25 in	*Flange thickness*
k_{des}	0.50 in	*Kdes*
k_{det}	0.50 in	*Kdet*

X-1 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.00x9.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Bottom Gusset		***P0.50x24.00x4.36***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	24.00 in	*Width*
t	0.50 in	*Thickness*
Clip		
H_clip	3.98 in	*Horiz. Clip*
V_clip	0.92 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	

d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

RISAConnection version 7.0.1

02/08/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

X-2 Bottom-L Detail	FAIL(UC-0.8)
X-2 Bottom-R Detail	FAIL(UC-0.8)
X-2 Top-L Connection	FAIL(UC-0.8)
X-2 Top-R Connection	FAIL(UC-0.6)

X-2 Bottom-L Detail: 3D View

Vertical Brace Diagonal Connection

Per AISC K1-3, the minimum plate thickness must be less than 0.38 in. We need a 0.5 in gusset plate for buckling of the whitemore section. This limit (AISC K1-3) is to negate the requirement of plastification of the HSS wall. By inspection, this failure mechanism is not applicable to this situation.

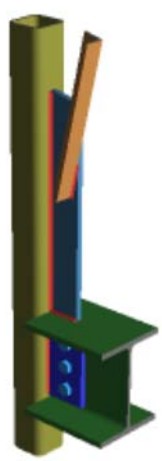
Furthermore, by review of punching shear and stresses on the HSS wall (Tf / Cf = 34.0 kip), base connection governs for brace X2:

Area of Gusset PL at face of HSS
A = (0.625 in)*(10 in) = 6.25 in^2

Stress acting on face of HSS
F_F = (34.0 kip) / (6.25 in^2) = 5.44 ksi

Shear strength of HSS
F_R = (0.90)*(0.6)*(36 ksi) = 19.44 ksi > F_F OK [U ~ 28%]

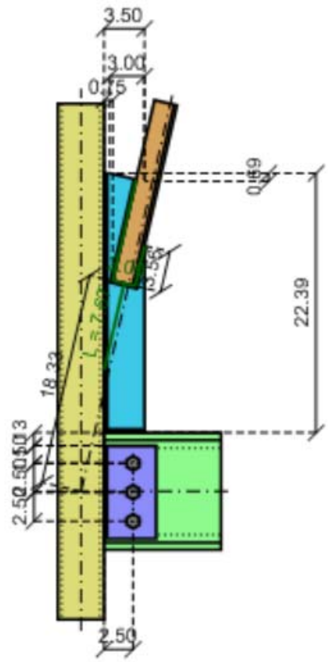
The overall stress on the HSS wall is approx. 28% of the capacity and is deemed more than acceptable. It should also be noted that the full brace force is used in the above which is a conservative check.



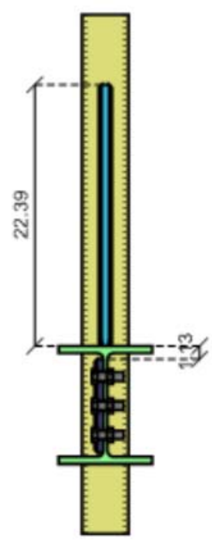
X-2 Bottom-L Detail: 2D Views

Vertical Brace Diagonal Connection

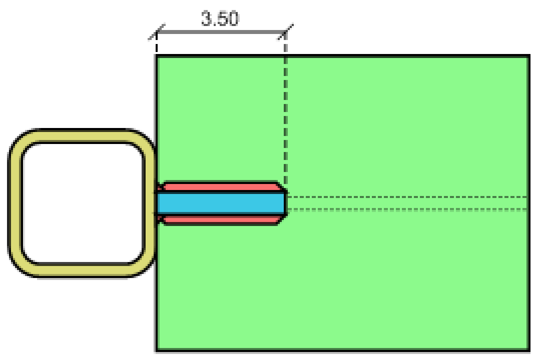
Side view



Front view



Top view



X-2 Bottom-L Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	Fy = 50.00 ksi	Fu = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	Fy = 46.00 ksi	Fu = 58.00 ksi
Plate	P0.50x4.50x8.00	A36	Fy = 36.00 ksi	Fu = 58.00 ksi
Top Brace	L2x2x4	A36	Fy = 36.00 ksi	Fu = 58.00 ksi
Top Gusset	P0.63x22.39x3.50	A36	Fy = 36.00 ksi	Fu = 58.00 ksi

Input Data:

Shear Load	16.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	30.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Top Brace Axial	34.00 kips	*User Input Top Brace Axial Force (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	Plate Flexural Buckling	**0.58**	**PASS**
Top Gusset/Beam connection	Plate Axial Yield	**0.15**	**PASS**
Top Gusset/Column connection	HSS Punching Shear		**FAIL**
Top Gusset/Brace connection	Gusset Plate Compression (Whitmore)	**0.84**	**PASS**

X-2 Bottom-L Detail: Beam/Column Report

Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.50x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.63x22.39x3.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	26.30 kips	*Calculated Shear Load*
Total Axial Load	9.08 kips	*Calculated Axial Load (compression)*
Column Force	30.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.35 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	8.46	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	11.46	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.50 in	*Plate thickness*		
$t_{p\text{-}max}$	0.56 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	2.50 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	2.50 in	*Max bolt spacing*		
t	0.35 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.44 in	*Max Size Allowed*		
t	0.50 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		

D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.35 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	8.00 in		*Max weld segment length*		
Beam Shear Yield		26.30 kips	106.05 kips	**0.25**	**PASS**
$R_n = 0.6 * F_y * A_{gv} * C_v$		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.53 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	106.05 kips		*Shear yield strength*		
Plate Shear Yield		26.30 kips	86.40 kips	**0.30**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	4.00 in^2		*Gross area subject to shear*		
φR_n	86.40 kips		*Shear yield strength*		
Beam Shear Rupture		26.30 kips	76.53 kips	**0.34**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.62 in^2		*Net area subject to shear*		
φR_n	76.53 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		26.30 kips	70.14 kips	**0.37**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.69 in^2		*Net area subject to shear*		
φR_n	70.14 kips		*Shear rupture strength*		
Beam Axial Yield		9.08 kips	598.50 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	13.30 in^2		*Gross area subject to tension*		
φR_n	598.50 kips		*Tensile yield strength*		
Plate Axial Yield		9.08 kips	129.60 kips	**0.07**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.00 in^2		*Gross area subject to tension*		
φR_n	129.60 kips		*Tensile yield strength*		
Beam Block Shear		26.30 kips	164.99 kips	**0.16**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		**φ = 0.75**	*(J4-5)*		
A_{gv}	5.77 in^2		*Gross area subject to shear*		
A_{nv}	5.00 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.72 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	164.99 kips		*Block shear strength*		
Plate Block Shear at Beam		26.30 kips	86.63 kips	**0.30**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		**φ = 0.75**	*(J4-5)*		
A_{gv}	3.25 in^2		*Gross area subject to shear*		

A_{nv}	2.16 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.78 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	86.63 kips		*Block shear strength*		
Compression Buckling of the Plate		9.08 kips	129.60 kips	**0.07**	**PASS**
$R_n = F_y * A_g$		φ = 0.9	*(J4-6)*		
K	1.00		*Effective length factor*		
L	2.50 in		*Unbraced length*		
r	0.14 in		*Radius of gyration*		
KL/r	17.30		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	4.00 in^2		*Gross area subject to compression*		
φR_n	129.60 kips		*Compressive strength*		
Plate Flexural Yield				**0.20**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	9.08 kips		*Calculated axial load*		
V_r	26.30 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.00 in^2		*Gross area of the plate*		
Z_{pl}	8.00 in^3		*Plastic modulus of the shear plate*		
P_c	129.60 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	86.40 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.50 in		*Horizontal eccentricity*		
ey	-0.08 in		*Vertical eccentricity*		
M_r	5.42 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	21.60 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.20		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.21**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	9.08 kips		*Calculated axial load*		
V_r	26.30 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.69 in^2		*Net area of the plate*		
Z_{net}	5.72 in^3		*Plastic modulus of net section*		
V_c	70.14 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.50 in		*Horizontal eccentricity*		
ey	-0.08 in		*Vertical eccentricity*		
M_r	5.42 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.72 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.21		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Plate Flexural Buckling				**0.58**	**PASS**
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		φ = 0.90	*(AISC 14th Edition)*		
P	9.08 kips		*Calculated axial load*		
V	26.30 kips		*Calculated shear load*		
L	2.50 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.14 in		*Radius of gyration of the plate*		
KL/r	17.30		*Slenderness ratio*		

F_e	956.21 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	35.44 ksi		*Compression stress = F_y when KL/r <= 25, per J4.4*		
A_g	4.00 in^2		*Gross area of the plate*		
λ	0.26		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	3.95 in^3		*Section modulus of net section*		
a	2.50 in		*Design eccentricity*		
P_n	144.00 kips		*Compressive capacity, per eqn J4-1, Pn = Fy * Ag*		
V_n	56.81 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.58		*Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$*		
Bolt Bearing on Beam		27.82 kips	53.68 kips	**0.52**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		φ **= 0.75**	*(J3-6b)*		
V	26.30 kips		*Applied shear force*		
P	9.08 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	27.82 kips		*Resultant shear force*		
Θ	70.95 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	2.50 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	2.47 in		*Vertical edge spacing*		
e_h	2.50 in		*Horizontal edge spacing*		
L_{c_boltA}	2.21 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	1.81 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		
Bolt Bearing on Plate at Beam		27.82 kips	53.68 kips	**0.52**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		φ **= 0.75**	*(J3-6b)*		
V	26.30 kips		*Applied shear force*		
P	9.08 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	27.82 kips		*Resultant shear force*		
Θ	70.95 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		

F_u	58.00 ksi	*Minimum tensile stress of material*
s_v	2.50 in	*Vertical bolt spacing*
s_h	0.00 in	*Horizontal bolt spacing*
e_v	1.50 in	*Vertical edge spacing*
e_h	2.00 in	*Horizontal edge spacing*
L_{c_boltA}	1.18 in	*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*
L_{c_boltB}	1.81 in	*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*
R_{n_boltA}	23.86 kips	*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*
R_{n_boltB}	23.86 kips	*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*
R_{n-bolt}	23.86 kips	*Bolt shear strength R_{n-bolt}=F_{nv}*A_{bolt}*
F_{nv}	54.00 ksi	*Nominal shear stress of bolt*
φR_n	53.68 kips	*Total bolt bearing strength*

Bolt Shear at Beam		27.82 kips	53.42 kips	**0.52**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		**φ = 0.75**	*(J3-1)*		
V	26.30 kips		*Applied shear force*		
P	9.08 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	27.82 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	1.00		*Eccentricity coefficient*		
φR_n	53.42 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***1.00***
Elastic method		*(AISC 14th p.7-6)*	
C	1.00	*Coefficient (2.9854 / 3)*	
N_{rows}	1	*Number of rows of bolts*	
N_{cols}	3	*Number of bolts per row*	
D_x	0.00 in	*Horizontal bolt spacing*	
D_y	2.50 in	*Vertical bolt spacing*	
E_x	0.00 in	*Horizontal eccentricity*	
E_y	0.08 in	*Vertical eccentricity*	
Ang	70.95	*Angle of force in degrees, relative X axis*	

Weld at Column		27.82 kips	89.09 kips	**0.31**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φRn	89.09 kips		*Weld strength*		

HSS Transverse Plastification		9.08 kips	49.57 kips	**0.18**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.50 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		

B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	49.57 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	19.52 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	1.00 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.25		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	19.52 kips-ft		*Flexural plastification*		

X-2 Bottom-L Detail: Top Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.50x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.63x22.39x3.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	3.57 kips	*Calculated Shear Load*
Axial Load	10.30 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.25 in		*Min size allowed per Table J2.4*		
t_{min}	0.62 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	3.50 in		*Min weld segment length*		
Plate Shear Yield		3.57 kips	47.25 kips	**0.08**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.19 in^2		*Gross area subject to shear*		
φR_n	47.25 kips		*Shear yield strength*		
Plate Shear Rupture		3.57 kips	57.09 kips	**0.06**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		

A_{nv}	2.19 in^2		*Net area subject to shear*		
φR_n	57.09 kips		*Shear rupture strength*		
Plate Axial Yield		10.30 kips	70.88 kips	**0.15**	**PASS**
$R_n = F_y*A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.19 in^2		*Gross area subject to tension*		
φ**R_n**	70.88 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.03**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	10.30 kips		*Calculated axial load*		
V_r	3.57 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.19 in^2		*Gross area of the plate*		
Z_{pl}	1.91 in^3		*Plastic modulus of the shear plate*		
P_c	70.88 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	47.25 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	5.17 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.03		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	3.57 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.19 in^2		*Net area of the plate*		
Z_{net}	1.91 in^3		*Plastic modulus of net section*		
V_c	57.09 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	6.94 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Beam Weld Strength		3.57 kips	31.18 kips	**0.11**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	3.50 in		*Weld length*		
φ**Rn**	31.18 kips		*Weld strength*		
Beam Web Yielding		10.30 kips	159.25 kips	**0.06**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		**φ = 1.00**	*(J10-2)*		
k	1.12 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	3.50 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		

t_w	0.35 in	*Beam web thickness*
φR_n	159.25 kips	*Beam web local yielding*

X-2 Bottom-L Detail: Top Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.50x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.63x22.39x3.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	22.83 kips	*Calculated Shear Load*
Axial Load	4.08 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**FAIL**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	8.46		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	11.46		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Fail**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.63 in		*Plate thickness*		
$t_{p\text{-}max}$	0.56 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.35 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	22.39 in		*Min weld segment length*		
Plate Shear Yield		22.83 kips	302.21 kips	**0.08**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	13.99 in^2		*Gross area subject to shear*		

φR_n	302.21 kips		*Shear yield strength*		

Plate Shear Rupture		22.83 kips	365.17 kips	**0.06**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	13.99 in^2		*Net area subject to shear*		
φR_n	365.17 kips		*Shear rupture strength*		

Plate Axial Yield		4.08 kips	453.32 kips	**0.01**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	13.99 in^2		*Gross area subject to tension*		
φR_n	453.32 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	4.08 kips		*Calculated axial load*		
V_r	22.83 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	13.99 in^2		*Gross area of the plate*		
Z_{pl}	78.30 in^3		*Plastic modulus of the shear plate*		
P_c	453.32 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	302.21 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	211.42 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	22.83 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	13.99 in^2		*Net area of the plate*		
Z_{net}	78.30 in^3		*Plastic modulus of net section*		
V_c	365.17 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	283.85 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		22.83 kips	199.43 kips	**0.11**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	22.39 in		*Weld length*		
φR_n	199.43 kips		*Weld strength*		

HSS Transverse Plastification		4.08 kips	98.72 kips	**0.04**	**PASS**

$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.63 in		*Plate thickness*		
l_b	22.39 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	98.72 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	106.98 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	1.13 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.28		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	22.39 in		*Depth of plate*		
η	5.60		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	106.98 kips-ft		*Flexural plastification*		

X-2 Bottom-L Detail: Top Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.50x4.50x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.63x22.39x3.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	34.00 kips	*Brace Axial (compression)*
Brace Moment	2.56 kips-ft	*Brace Moment (not used for design)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.25 in		*Max Size Allowed*		
t	0.25 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.25 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		

D	0.19 in		*Weld size*		
L_{max}	8.97 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		34.00 kips	40.50 kips	**0.84**	**PASS**
$P_n = F_y * A_g$		**$\varphi = 0.9$**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	7.67 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	21.24		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	1.25 in^2		*Gross area of plate (Whitmore section)*		
φP_n	40.50 kips		*Gusset plate compressive strength*		

Brace Weld Strength		34.00 kips	60.69 kips	**0.56**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	14.53 in		*Weld length*		
φRn	60.69 kips		*Weld strength*		

X-2 Bottom-L Detail: Members Report

Vertical Brace Diagonal Connection

Beam		***W10x45***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	8.02 in	*Flange width*
d	10.10 in	*Overall depth*
t_w	0.35 in	*Web thickness*
t_f	0.62 in	*Flange thickness*
a	13.30 in^2	*Area*
k_{des}	1.12 in	*Kdes*
k_{det}	1.31 in	*Kdet*
k_1	0.81 in	*K1*
Web Hole Type		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	2.50 in	*Column Spacing*

Column		***HSS4x4x6***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		

d	4.00 in	*Depth*
b	4.00 in	*Width*
a	4.78 in^2	*Area*
t_{des}	0.35 in	*Wall Thickness*

Top Brace		***L2x2x4***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	2.00 in	*Flange width*
d	2.00 in	*Overall depth*
a	0.94 in^2	*Area*
t_{f1}	0.25 in	*Flange thickness*
t_{f2}	0.25 in	*Flange thickness*
k_{des}	0.50 in	*Kdes*
k_{det}	0.50 in	*Kdet*

X-2 Bottom-L Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.50x4.50x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.50 in	*Width*
t	0.50 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	2.50 in	*Column Spacing*

Top Gusset		***P0.63x22.39x3.50***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	22.39 in	*Width*
t	0.63 in	*Thickness*
Clip		
H_clip	3.00 in	*Horiz. Clip*
V_clip	0.69 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

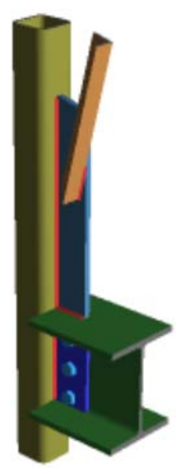
Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

X-2 Bottom-R Detail: 3D View

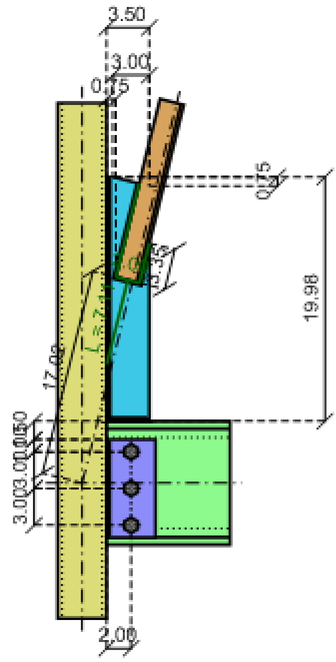
Vertical Brace Diagonal Connection



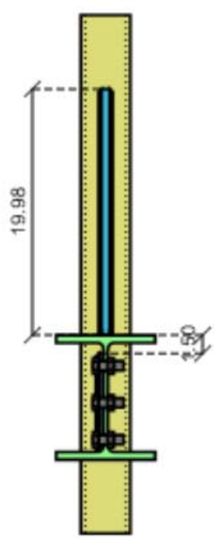
X-2 Bottom-R Detail: 2D Views

Vertical Brace Diagonal Connection

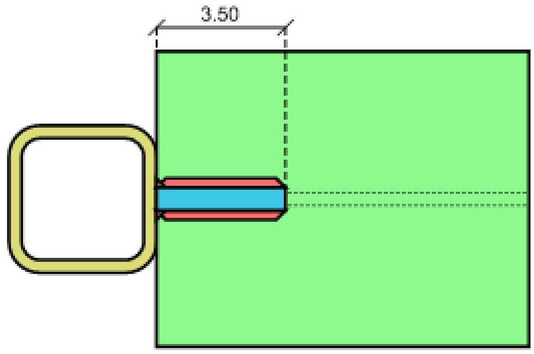
Side view



Front view



Top view



X-2 Bottom-R Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.63x19.98x3.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	16.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	45.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Top Brace Axial	34.00 kips	*User Input Top Brace Axial Force (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	Plate Flexural Buckling	**0.76**	**PASS**
Top Gusset/Beam connection	Plate Axial Yield	**0.16**	**PASS**
Top Gusset/Column connection	HSS Punching Shear		**FAIL**
Top Gusset/Brace connection	Gusset Plate Compression (Whitmore)	**0.84**	**PASS**

X-2 Bottom-R Detail: Beam/Column Report

Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.63x19.98x3.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	27.08 kips	*Calculated Shear Load*
Total Axial Load	9.39 kips	*Calculated Axial Load (compression)*
Column Force	45.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.35 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	8.46	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	11.46	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
$t_{p\text{-}max}$	0.56 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.35 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		
t	0.38 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		

D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.35 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	8.00 in		*Max weld segment length*		
Beam Shear Yield		27.08 kips	106.05 kips	**0.26**	**PASS**
$R_n = 0.6 * F_y * A_{gv} * C_v$		**$\varphi = 1.00$**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.53 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	106.05 kips		*Shear yield strength*		
Plate Shear Yield		27.08 kips	64.80 kips	**0.42**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**$\varphi = 1.00$**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.00 in^2		*Gross area subject to shear*		
φR_n	64.80 kips		*Shear yield strength*		
Beam Shear Rupture		27.08 kips	76.53 kips	**0.35**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.62 in^2		*Net area subject to shear*		
φR_n	76.53 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		27.08 kips	52.61 kips	**0.51**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.02 in^2		*Net area subject to shear*		
φR_n	52.61 kips		*Shear rupture strength*		
Beam Axial Yield		9.39 kips	598.50 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	13.30 in^2		*Gross area subject to tension*		
φR_n	598.50 kips		*Tensile yield strength*		
Plate Axial Yield		9.39 kips	97.20 kips	**0.10**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area subject to tension*		
φR_n	97.20 kips		*Tensile yield strength*		
Beam Block Shear		27.08 kips	149.40 kips	**0.18**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		**$\varphi = 0.75$**	*(J4-5)*		
A_{gv}	5.46 in^2		*Gross area subject to shear*		
A_{nv}	4.69 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.55 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	149.40 kips		*Block shear strength*		
Plate Block Shear at Beam		27.08 kips	68.01 kips	**0.40**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		**$\varphi = 0.75$**	*(J4-5)*		
A_{gv}	2.63 in^2		*Gross area subject to shear*		

A_{nv}	1.80 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.59 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	68.01 kips		*Block shear strength*		
Compression Buckling of the Plate		9.39 kips	97.20 kips	**0.10**	**PASS**
$R_n = F_y * A_g$		φ **= 0.9**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	2.00 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	18.45		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	3.00 in^2		*Gross area subject to compression*		
φR_n	97.20 kips		*Compressive strength*		
Plate Flexural Yield				**0.30**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	9.39 kips		*Calculated axial load*		
V_r	27.08 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.00 in^2		*Gross area of the plate*		
Z_{pl}	6.00 in^3		*Plastic modulus of the shear plate*		
P_c	97.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	64.80 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.00 in		*Horizontal eccentricity*		
ey	-0.45 in		*Vertical eccentricity*		
M_r	4.16 kips-ft		*Moment due to eccentricity =* $V_r*ex + P_r*ey$		
M_c	16.20 kips-ft		*Available moment* $M_c=\varphi*(F_y* Z)$, $\varphi=0.90$		
UC	0.30		*Unity check per interaction equation,* $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$		
Plate Flexural Rupture				**0.35**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	9.39 kips		*Calculated axial load*		
V_r	27.08 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.02 in^2		*Net area of the plate*		
Z_{net}	3.96 in^3		*Plastic modulus of net section*		
V_c	52.61 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.00 in		*Horizontal eccentricity*		
ey	-0.45 in		*Vertical eccentricity*		
M_r	4.16 kips-ft		*Moment due to eccentricity =* $V_r*ex + P_r*ey$		
M_c	14.35 kips-ft		*Available moment* $M_c= \varphi*(F_u* Znet)$, $\varphi=0.75$		
UC	0.35		*Unity check per interaction equation,* $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		
Plate Flexural Buckling				**0.76**	**PASS**
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		φ **= 0.90**	*(AISC 14th Edition)*		
P	9.39 kips		*Calculated axial load*		
V	27.08 kips		*Calculated shear load*		
L	2.00 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	18.45		*Slenderness ratio*		

F_e	840.42 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	35.36 ksi		*Compression stress = F_y when KL/r <= 25, per J4.4*		
A_g	3.00 in^2		*Gross area of the plate*		
λ	0.32		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	2.51 in^3		*Section modulus of net section*		
a	2.00 in		*Design eccentricity*		
P_n	108.00 kips		*Compressive capacity, per eqn J4-1, Pn = Fy * Ag*		
V_n	45.14 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.76		*Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$*		
Bolt Bearing on Beam		28.66 kips	53.68 kips	**0.53**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		φ **= 0.75**	*(J3-6b)*		
V	27.08 kips		*Applied shear force*		
P	9.39 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	28.66 kips		*Resultant shear force*		
Θ	70.88 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.60 in		*Vertical edge spacing*		
e_h	2.00 in		*Horizontal edge spacing*		
L_{c_boltA}	1.29 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.34 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		
Bolt Bearing on Plate at Beam		28.66 kips	51.74 kips	**0.55**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		φ **= 0.75**	*(J3-6b)*		
V	27.08 kips		*Applied shear force*		
P	9.39 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	28.66 kips		*Resultant shear force*		
Θ	70.88 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		

F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.00 in		*Vertical edge spacing*		
e_h	2.00 in		*Horizontal edge spacing*		
L_{c_boltA}	0.65 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.34 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	21.28 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), $R_{n\text{-}bolt}$]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), $R_{n\text{-}bolt}$]*		
$R_{n\text{-}bolt}$	23.86 kips		*Bolt shear strength $R_{n\text{-}bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	51.74 kips		*Total bolt bearing strength*		
Bolt Shear at Beam		28.66 kips	52.29 kips	**0.55**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		φ **= 0.75**	*(J3-1)*		
V	27.08 kips		*Applied shear force*		
P	9.39 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	28.66 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.97		*Eccentricity coefficient*		
φR_n	52.29 kips		*Bolt shear rupture strength*		
Bolt Group Eccentricity			***0.97***		
Elastic method			*(AISC 14th p.7-6)*		
C	0.97		*Coefficient (2.9225 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.45 in		*Vertical eccentricity*		
Ang	70.88		*Angle of force in degrees, relative X axis*		
Weld at Column		28.66 kips	78.38 kips	**0.37**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φRn	78.38 kips		*Weld strength*		
HSS Transverse Plastification		9.39 kips	48.27 kips	**0.19**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		φ **= 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	8.00 in		*Plate length*		

B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	48.27 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	18.95 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	8.00 in		*Depth of plate*		
η	2.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	18.95 kips-ft		*Flexural plastification*		

X-2 Bottom-R Detail: Top Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.63x19.98x3.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	3.84 kips	*Calculated Shear Load*
Axial Load	11.08 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.25 in		*Min size allowed per Table J2.4*		
t_{min}	0.62 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	3.50 in		*Min weld segment length*		
Plate Shear Yield		3.84 kips	47.25 kips	**0.08**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.19 in^2		*Gross area subject to shear*		
φR_n	47.25 kips		*Shear yield strength*		
Plate Shear Rupture		3.84 kips	57.09 kips	**0.07**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		

A_{nv}	2.19 in^2		*Net area subject to shear*		
φR_n	57.09 kips		*Shear rupture strength*		
Plate Axial Yield		11.08 kips	70.88 kips	**0.16**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.19 in^2		*Gross area subject to tension*		
φR_n	70.88 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.03**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	11.08 kips		*Calculated axial load*		
V_r	3.84 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.19 in^2		*Gross area of the plate*		
Z_{pl}	1.91 in^3		*Plastic modulus of the shear plate*		
P_c	70.88 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	47.25 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	5.17 kips-ft		*Available moment $M_c=\varphi*(F_y*Z)$, φ=0.90*		
UC	0.03		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	3.84 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.19 in^2		*Net area of the plate*		
Z_{net}	1.91 in^3		*Plastic modulus of net section*		
V_c	57.09 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	6.94 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Beam Weld Strength		3.84 kips	31.18 kips	**0.12**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	3.50 in		*Weld length*		
φR_n	31.18 kips		*Weld strength*		
Beam Web Yielding		11.08 kips	159.25 kips	**0.07**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		**φ = 1.00**	*(J10-2)*		
k	1.12 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	3.50 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		

t_w	0.35 in	*Beam web thickness*
φR_n	159.25 kips	*Beam web local yielding*

X-2 Bottom-R Detail: Top Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.63x19.98x3.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	21.91 kips	*Calculated Shear Load*
Axial Load	4.39 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**FAIL**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	8.46		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	11.46		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Fail**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.63 in		*Plate thickness*		
$t_{p\text{-}max}$	0.56 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.35 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	19.98 in		*Min weld segment length*		
Plate Shear Yield		21.91 kips	269.74 kips	**0.08**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	12.49 in^2		*Gross area subject to shear*		

φR_n	269.74 kips		*Shear yield strength*		

Plate Shear Rupture		21.91 kips	325.94 kips	**0.07**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		φ **= 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	12.49 in^2		*Net area subject to shear*		
φR_n	325.94 kips		*Shear rupture strength*		

Plate Axial Yield		4.39 kips	404.61 kips	**0.01**	**PASS**
$R_n = F_y * A_g$		φ **= 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	12.49 in^2		*Gross area subject to tension*		
φR_n	404.61 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	4.39 kips		*Calculated axial load*		
V_r	21.91 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	12.49 in^2		*Gross area of the plate*		
Z_{pl}	62.38 in^3		*Plastic modulus of the shear plate*		
P_c	404.61 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	269.74 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	168.43 kips-ft		*Available moment* $M_c = \varphi*(F_y * Z)$, $\varphi = 0.90$		
UC	0.01		*Unity check per interaction equation,* $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	21.91 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	12.49 in^2		*Net area of the plate*		
Z_{net}	62.38 in^3		*Plastic modulus of net section*		
V_c	325.94 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	226.13 kips-ft		*Available moment* $M_c = \varphi*(F_u * Znet)$, $\varphi = 0.75$		
UC	0.00		*Unity check per interaction equation,* $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		

Column Weld Strength		21.91 kips	178.01 kips	**0.12**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	19.98 in		*Weld length*		
φR_n	178.01 kips		*Weld strength*		

HSS Transverse Plastification		4.39 kips	90.74 kips	**0.05**	**PASS**

$R_n = F_y{*}t^2 / (1-t_p/B){*}(2l_b/B + 4{*}Q_f{*}(1-t_p/B)^{0.5})$		$\varphi = 1.00$	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.63 in		*Plate thickness*		
l_b	19.98 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	90.74 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	87.78 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		$\varphi = 1.0$	*(K3-6)*		
B_b	1.13 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.28		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	19.98 in		*Depth of plate*		
η	5.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	87.78 kips-ft		*Flexural plastification*		

X-2 Bottom-R Detail: Top Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W10x45	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.63x19.98x3.50	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	34.00 kips	*Brace Axial (compression)*
Brace Moment	2.56 kips-ft	*Brace Moment (not used for design)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.19 in		*Weld size*		
D_{max}	0.25 in		*Max Size Allowed*		
t	0.25 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.25 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		

D	0.19 in		*Weld size*		
L_{max}	7.88 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		34.00 kips	40.50 kips	**0.84**	**PASS**
$P_n = F_y*A_g$		**φ = 0.9**	*(J4-6)*		
K	0.50		*Effective length factor*		
L	7.11 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	19.71		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	1.25 in^2		*Gross area of plate (Whitmore section)*		
φP_n	40.50 kips		*Gusset plate compressive strength*		

Brace Weld Strength		34.00 kips	55.22 kips	**0.62**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	13.22 in		*Weld length*		
φRn	55.22 kips		*Weld strength*		

X-2 Bottom-R Detail: Members Report

Vertical Brace Diagonal Connection

Beam		***W10x45***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	8.02 in	*Flange width*
d	10.10 in	*Overall depth*
t_w	0.35 in	*Web thickness*
t_f	0.62 in	*Flange thickness*
a	13.30 in^2	*Area*
k_{des}	1.12 in	*Kdes*
k_{det}	1.31 in	*Kdet*
k_1	0.81 in	*K1*
Web Hole Type		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Column		***HSS4x4x6***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		

d	4.00 in	*Depth*
b	4.00 in	*Width*
a	4.78 in^2	*Area*
t_{des}	0.35 in	*Wall Thickness*

Top Brace		***L2x2x4***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	2.00 in	*Flange width*
d	2.00 in	*Overall depth*
a	0.94 in^2	*Area*
t_{f1}	0.25 in	*Flange thickness*
t_{f2}	0.25 in	*Flange thickness*
k_{des}	0.50 in	*Kdes*
k_{det}	0.50 in	*Kdet*

X-2 Bottom-R Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.00x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Top Gusset		***P0.63x19.98x3.50***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	19.98 in	*Width*
t	0.63 in	*Thickness*
Clip		
H_clip	3.00 in	*Horiz. Clip*
V_clip	0.75 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

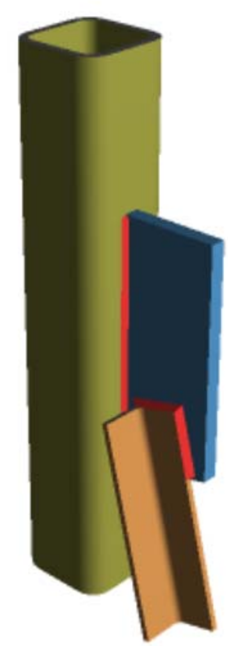
Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.19 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

X-2 Top-L Connection: 3D View

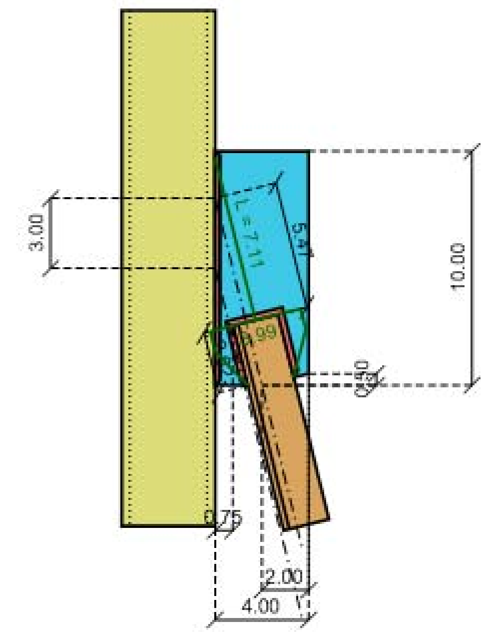
Knee Brace Connection



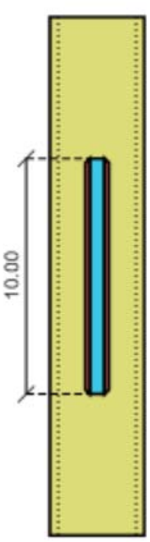
X-2 Top-L Connection: 2D Views

Knee Brace Connection

Left view

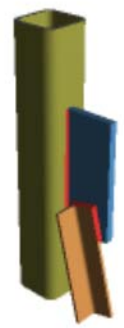


Front view



X-2 Top-L Connection: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:				
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Gusset	P0.63x4.00x10.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:		
Brace Axial	34.00 kips	*Brace Axial (compression)*
Shear Load	-32.99 kips	*Calculated Shear Load*
Axial Load	8.23 kips	*Calculated Axial Load (compression)*
Moment Load	2.06 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**FAIL**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.35 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	8.46	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	11.46	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Fail**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.63 in	*Plate thickness*		
$t_{p\text{-}max}$	0.56 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_V<= 0.5*d_{gusset}*		
WP_V	3.00 in	*Vertical brace workpoint offset*		
d_{gusset}	10.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.35 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	10.00 in	*Min weld segment length*		
Brace Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.25 in	*Max Size Allowed*		
t	0.25 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		

D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.25 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	2.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	2.77 in		*Max weld segment length*		
Plate Shear Yield		32.99 kips	135.00 kips	**0.24**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	6.25 in^2		*Gross area subject to shear*		
φR_n	135.00 kips		*Shear yield strength*		
Plate Shear Rupture		32.99 kips	163.13 kips	**0.20**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	6.25 in^2		*Net area subject to shear*		
φR_n	163.13 kips		*Shear rupture strength*		
Plate Axial Yield		8.23 kips	202.50 kips	**0.04**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.25 in^2		*Gross area subject to tension*		
φR_n	202.50 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.07**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	8.23 kips		*Calculated axial load*		
V_r	32.99 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.25 in^2		*Gross area of the plate*		
Z_{pl}	15.63 in^3		*Plastic modulus of the shear plate*		
P_c	202.50 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	135.00 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	2.06 kips-ft		*Calculated moment*		
M_c	42.19 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.07		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.04**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	32.99 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	6.25 in^2		*Net area of the plate*		
Z_{net}	15.63 in^3		*Plastic modulus of net section*		
V_c	163.13 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	2.06 kips-ft		*Calculated moment*		
M_c	56.64 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.04		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Column Weld Strength		3.62 kips/in	6.68 kips/in	**0.54**	**PASS**
φ$R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					

Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
V	32.99 kips		*Shear Load*		
P	8.23 kips		*Axial Load*		
M	2.06 kips-ft		*Moment*		
e_{eff}	0.75 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	3.62 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		34.00 kips	71.89 kips	**0.47**	**PASS**
$P_n = F_{cr}*A_g$		**φ = 0.9**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	7.11 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	47.30		*Plate slenderness*		
F_{cr}	32.00 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.50 in^2		*Gross area of plate (Whitmore section)*		
φP_n	71.89 kips		*Gusset plate compressive strength*		

Brace Weld Strength		34.00 kips	41.19 kips	**0.83**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	7.40 in		*Weld length*		
φRn	41.19 kips		*Weld strength*		

HSS Transverse Plastification		8.23 kips	57.60 kips	**0.14**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.63 in		*Plate thickness*		
l_b	10.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φRn	57.60 kips		*Transverse plastification*		

HSS Flexural Plastification		2.06 kips-ft	27.28 kips-ft	**0.08**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	1.00 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.25		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	10.00 in		*Depth of plate*		
η	2.50		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		

M_{req}	2.06 kips-ft	*Required flexural plastification*
φM_n	27.28 kips-ft	*Flexural plastification*

X-2 Top-L Connection: Members Report

Knee Brace Connection

Column		***HSS4x4x6***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	4.78 in^2	*Area*
t_{des}	0.35 in	*Wall Thickness*

Brace		***L2x2x4***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	2.00 in	*Flange width*
d	2.00 in	*Overall depth*
a	0.94 in^2	*Area*
t_{f1}	0.25 in	*Flange thickness*
t_{f2}	0.25 in	*Flange thickness*
k_{des}	0.50 in	*Kdes*
k_{det}	0.50 in	*Kdet*

X-2 Top-L Connection: Components Report

Knee Brace Connection

Gusset		***P0.63x4.00x10.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.63 in	*Thickness*
Clip		
H_clip	2.00 in	*Horiz. Clip*
V_clip	0.50 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.19 in	

Brace Gusset Weld		***E70***
Weld Properties		

Type Single Fillet
Fillet Size 0.25 in

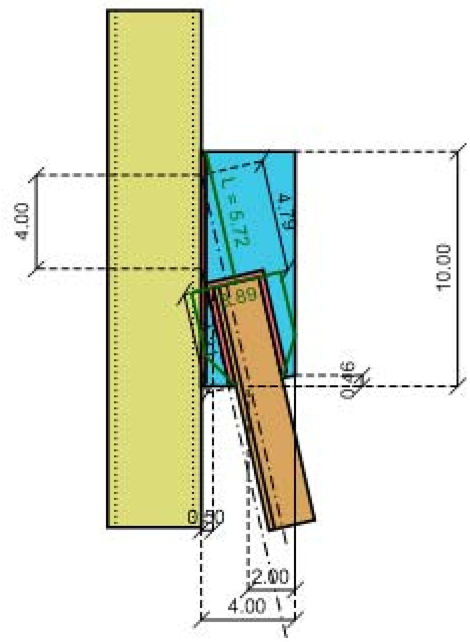
X-2 Top-R Connection: 3D View

Knee Brace Connection

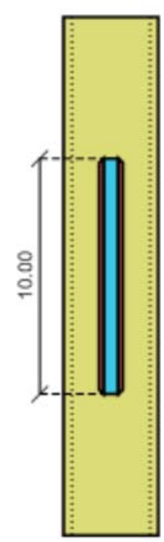
X-2 Top-R Connection: 2D Views

Knee Brace Connection

Left view

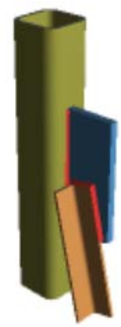


Front view



X-2 Top-R Connection: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:				
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Gusset	P0.63x4.00x10.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:		
Brace Axial	34.00 kips	*Brace Axial (compression)*
Shear Load	-33.13 kips	*Calculated Shear Load*
Axial Load	7.65 kips	*Calculated Axial Load (compression)*
Moment Load	2.55 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**FAIL**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.35 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	8.46	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	11.46	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Fail**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.63 in	*Plate thickness*		
$t_{p\text{-}max}$	0.56 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	4.00 in	*Vertical brace workpoint offset*		
d_{gusset}	10.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.35 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	10.00 in	*Min weld segment length*		
Brace Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.25 in	*Max Size Allowed*		
t	0.25 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		

D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.25 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	2.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	4.43 in		*Max weld segment length*		
Plate Shear Yield		33.13 kips	135.00 kips	**0.25**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	6.25 in^2		*Gross area subject to shear*		
φR_n	135.00 kips		*Shear yield strength*		
Plate Shear Rupture		33.13 kips	163.13 kips	**0.20**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	6.25 in^2		*Net area subject to shear*		
φR_n	163.13 kips		*Shear rupture strength*		
Plate Axial Yield		7.65 kips	202.50 kips	**0.04**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.25 in^2		*Gross area subject to tension*		
φR_n	202.50 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.07**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	7.65 kips		*Calculated axial load*		
V_r	33.13 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.25 in^2		*Gross area of the plate*		
Z_{pl}	15.63 in^3		*Plastic modulus of the shear plate*		
P_c	202.50 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	135.00 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	2.55 kips-ft		*Calculated moment*		
M_c	42.19 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.07		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.04**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	33.13 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	6.25 in^2		*Net area of the plate*		
Z_{net}	15.63 in^3		*Plastic modulus of net section*		
V_c	163.13 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	2.55 kips-ft		*Calculated moment*		
M_c	56.64 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.04		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Column Weld Strength		3.79 kips/in	6.68 kips/in	**0.57**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					

Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
V	33.13 kips		*Shear Load*		
P	7.65 kips		*Axial Load*		
M	2.55 kips-ft		*Moment*		
e_{eff}	0.92 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	3.79 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		34.00 kips	73.08 kips	**0.47**	**PASS**
$P_n = F_{cr}*A_g$		**$\varphi = 0.9$**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	5.72 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	38.03		*Plate slenderness*		
F_{cr}	33.36 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.43 in^2		*Gross area of plate (Whitmore section)*		
φP_n	73.08 kips		*Gusset plate compressive strength*		

Brace Weld Strength		34.00 kips	59.85 kips	**0.57**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	10.75 in		*Weld length*		
φRn	59.85 kips		*Weld strength*		

HSS Transverse Plastification		7.65 kips	57.60 kips	**0.13**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**$\varphi = 1.00$**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.63 in		*Plate thickness*		
l_b	10.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φRn	57.60 kips		*Transverse plastification*		

HSS Flexural Plastification		2.55 kips-ft	27.28 kips-ft	**0.09**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**$\varphi = 1.0$**	*(K3-6)*		
B_b	1.00 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.25		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	10.00 in		*Depth of plate*		
η	2.50		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		

M_{req}	2.55 kips-ft	*Required flexural plastification*
φM_n	27.28 kips-ft	*Flexural plastification*

X-2 Top-R Connection: Members Report

Knee Brace Connection

	Column		***HSS4x4x6***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	4.00 in	*Depth*
	b	4.00 in	*Width*
	a	4.78 in^2	*Area*
	t_{des}	0.35 in	*Wall Thickness*

	Brace		***L2x2x4***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b	2.00 in	*Flange width*
	d	2.00 in	*Overall depth*
	a	0.94 in^2	*Area*
	t_{f1}	0.25 in	*Flange thickness*
	t_{f2}	0.25 in	*Flange thickness*
	k_{des}	0.50 in	*Kdes*
	k_{det}	0.50 in	*Kdet*

X-2 Top-R Connection: Components Report

Knee Brace Connection

	Gusset		***P0.63x4.00x10.00***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	4.00 in	*Width*
	t	0.63 in	*Thickness*
Clip			
	H_clip	2.00 in	*Horiz. Clip*
	V_clip	0.46 in	*Vert. Clip*

	Column Weld		***E70***
Weld Properties			
	Type	Double Fillet	
	Fillet Size	0.19 in	

	Brace Gusset Weld		***E70***
Weld Properties			

Type	Single Fillet
Fillet Size	0.25 in

RISAConnection version 7.0.1

02/08/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

X-3 Bottom Detail	**PASS(UC-1.0)**
X-3 Top Detail	**PASS(UC-1.0)**

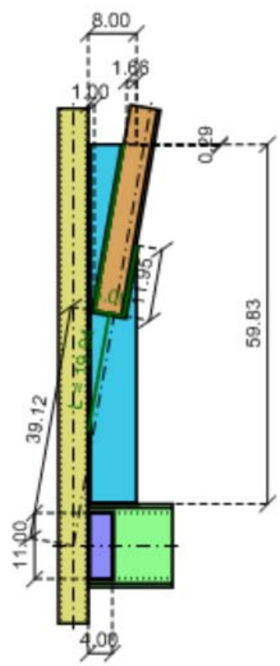
X-3 Bottom Detail: 3D View

Vertical Brace Diagonal Connection

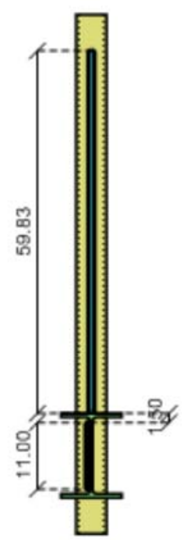
X-3 Bottom Detail: 2D Views

Vertical Brace Diagonal Connection

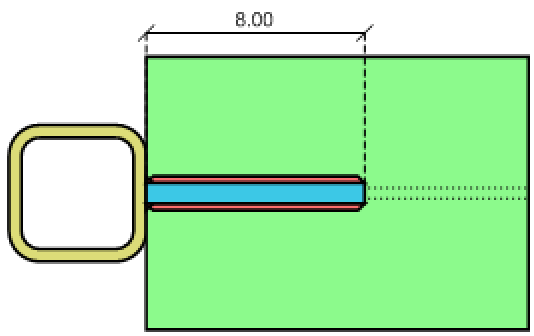
Side view



Front view



Top view



X-3 Bottom Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x61	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.63x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.74x59.83x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	70.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	120.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Top Brace Axial	102.50 kips	*User Input Top Brace Axial Force (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	Weld at Beam	**0.85**	**PASS**
Top Gusset/Beam connection	Beam Weld Strength	**0.15**	**PASS**
Top Gusset/Column connection	Column Weld Strength	**0.15**	**PASS**
Top Gusset/Brace connection	Gusset Plate Compression (Whitmore)	**0.95**	**PASS**

LRFD

X-3 Bottom Detail: Beam/Column Report

Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x61	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.63x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.74x59.83x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	89.03 kips	*Calculated Shear Load*
Total Axial Load	11.85 kips	*Calculated Axial Load (compression)*
Column Force	120.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.47 in	*Column wall thickness*		
B	5.00 in	*Column face width*		
(B - 3 * t) / t	7.75	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	10.75	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.63 in	*Plate thickness*		
$t_{p\text{-}max}$	0.75 in	*Maximum allowed plate thickness*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.56 in	*Max Size Allowed*		
t	0.63 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.47 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in	*Weld size*		
L_{min}	11.00 in	*Min weld segment length*		
Check Weld Max Length	**Pass**	*Condition: L_{max} <= 100*D*		
D	0.25 in	*Weld size*		
L_{max}	11.00 in	*Max weld segment length*		
Beam Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		

Check Weld Max Size	**Pass**				
D	0.25 in		*Weld size*		
D_{max}	0.56 in		*Max Size Allowed*		
t	0.63 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	4.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	11.00 in		*Max weld segment length*		

Beam Shear Yield		89.03 kips	156.38 kips	**0.57**	**PASS**
R_n = 0.6 *F_y*A_{gv}*C_v		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	5.21 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	156.38 kips		*Shear yield strength*		

Plate Shear Yield		89.03 kips	148.50 kips	**0.60**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	6.88 in^2		*Gross area subject to shear*		
φR_n	148.50 kips		*Shear yield strength*		

Beam Shear Rupture		89.03 kips	152.47 kips	**0.58**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	5.21 in^2		*Net area subject to shear*		
φR_n	152.47 kips		*Shear rupture strength*		

Plate Shear Rupture at Beam		89.03 kips	179.44 kips	**0.50**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	6.88 in^2		*Net area subject to shear*		
φR_n	179.44 kips		*Shear rupture strength*		

Beam Axial Yield		11.85 kips	805.50 kips	**0.01**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	17.90 in^2		*Gross area subject to tension*		
φR_n	805.50 kips		*Tensile yield strength*		

Plate Axial Yield		11.85 kips	222.75 kips	**0.05**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	6.88 in^2		*Gross area subject to tension*		
φR_n	222.75 kips		*Tensile yield strength*		

Compression Buckling of the Plate		11.85 kips	222.75 kips	**0.05**	**PASS**
R_n = F_y * A_g		**φ = 0.9**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	0.00 in		*Unbraced length*		
r	0.18 in		*Radius of gyration*		
KL/r	0.00		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	6.88 in^2		*Gross area subject to compression*		
φR_n	222.75 kips		*Compressive strength*		

Plate Flexural Yield		0.62	PASS
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$		*(AISC 14th Eq.10-5)*	
P_r	11.85 kips	*Calculated axial load*	
V_r	89.03 kips	*Calculated shear load*	
F_y	36.00 ksi	*Minimum yield stress of material*	
A_g	6.88 in^2	*Gross area of the plate*	
Z_{pl}	18.91 in^3	*Plastic modulus of the shear plate*	
P_c	222.75 kips	*Available tensile strength (see check 'Axial Yield')*	
V_c	148.50 kips	*Available shear strength (see check 'Shear Yield')*	
ex	3.16 in	*Horizontal eccentricity*	
ey	-0.05 in	*Vertical eccentricity*	
M_r	23.38 kips-ft	*Moment due to eccentricity = V_r*ex + P_r*ey*	
M_c	51.05 kips-ft	*Available moment $M_c=\varphi*(F_y* Z)$, $\varphi=0.90$*	
UC	0.62	*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*	

Plate Flexural Rupture		0.36	PASS
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		*(Eq.10-5)*	
P_r	11.85 kips	*Calculated axial load*	
V_r	89.03 kips	*Calculated shear load*	
F_u	58.00 ksi	*Minimum tensile stress of material*	
A_n	6.88 in^2	*Net area of the plate*	
Z_{net}	18.91 in^3	*Plastic modulus of net section*	
V_c	179.44 kips	*Available shear strength (see check 'Shear Rupture')*	
ex	3.16 in	*Horizontal eccentricity*	
ey	-0.05 in	*Vertical eccentricity*	
M_r	23.38 kips-ft	*Moment due to eccentricity = V_r*ex + P_r*ey*	
M_c	68.54 kips-ft	*Available moment $M_c= \varphi*(F_u* Znet)$, $\varphi=0.75$*	
UC	0.36	*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*	

Plate Flexural Buckling			0.74	PASS
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		$\varphi = 0.90$	*(AISC 14th Edition)*	
P	11.85 kips		*Calculated axial load*	
V	89.03 kips		*Calculated shear load*	
L	0.00 in		*Length of connecting element (distance between the applied load and resisting element)*	
r	0.18 in		*Radius of gyration of the plate*	
KL/r	0.00		*Slenderness ratio*	
F_e	N/A		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2*E)/(KL/r)^2$*	
F_y	36.00 ksi		*Minimum yield stress of material*	
F_{cr_Comp}	36.00 ksi		*Compression stress = F_y when KL/r <= 25, per J4.4*	
A_g	6.88 in^2		*Gross area of the plate*	
λ	0.21		*Buckling factor (pg 9.9) (eqn 9-18)*	
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*	
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*	
S_{net}	12.60 in^3		*Section modulus of net section*	
a	3.16 in		*Design eccentricity*	
P_n	247.50 kips		*Compressive capacity, per eqn J4-1, Pn = Fy * Ag*	
V_n	143.69 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*	
UC	0.74		*Unity check per interaction equation, P/(Pn*Φ) +*	

		*V/(Vn*Φ) <= 1*		

Weld at Column	89.82 kips	122.50 kips	**0.73**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$				
Double Fillet				
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)				
C_1	1.00	*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
D_{16}	4.00	*Weld fillet size in sixteenths of an inch*		
L	11.00 in	*Weld length*		
φRn	122.50 kips	*Weld strength*		

Weld at Beam	4.73 kips/in	5.57 kips/in	**0.85**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16}$				
Single Fillet				
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)				
C_1	1.00	*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
D_{16}	4.00	*Weld fillet size in sixteenths of an inch*		
r_u	4.73 kips/in	*Required weld stress per AISC 14^{th} Eqn 8-11*		
φRn	5.57 kips/in	*Weld strength*		

HSS Transverse Plastification	11.85 kips	92.55 kips	**0.13**	**PASS**
$R_n = F_y * t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$	**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi	*Column yield strength*		
t	0.47 in	*Column wall thickness*		
t_p	0.63 in	*Plate thickness*		
l_b	11.00 in	*Plate length*		
B	5.00 in	*Column width*		
Q_f	1.00	*User input column stress interaction parameter*		
φR_n	92.55 kips	*Transverse plastification*		

HSS Flexural Plastification	0.00 kips-ft	48.67 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$	**φ = 1.0**	*(K3-6)*		
B_b	1.13 in	*Plate bearing width*		
B	5.00 in	*Column width*		
β	0.23	*Width ratio (B_b / B)*		
F_y	46.00 ksi	*Column yield strength*		
t	0.47 in	*Column wall thickness*		
H_b	11.00 in	*Depth of plate*		
η	2.20	*Load length parameter (H_b / B)*		
Q_f	1.00	*User input column stress interaction parameter*		
ex	0.00 in	*Horizontal eccentricity*		
ey	0.00 in	*Vertical eccentricity*		
M_{req}	0.00 kips-ft	*Required flexural plastification = V*ex + P*ey*		
φM_n	48.67 kips-ft	*Flexural plastification*		

X-3 Bottom Detail: Top Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x61	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.63x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.74x59.83x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	10.95 kips	*Calculated Shear Load*
Axial Load	19.03 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.25 in		*Min size allowed per Table J2.4*		
t_{min}	0.65 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Plate Shear Yield		10.95 kips	128.74 kips	**0.09**	**PASS**
$R_n = 0.6 \cdot F_y \cdot A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	5.96 in^2		*Gross area subject to shear*		
φR_n	128.74 kips		*Shear yield strength*		
Plate Shear Rupture		10.95 kips	155.56 kips	**0.07**	**PASS**
$R_n = 0.6 \cdot F_u \cdot A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	5.96 in^2		*Net area subject to shear*		
φR_n	155.56 kips		*Shear rupture strength*		
Plate Axial Yield		19.03 kips	193.10 kips	**0.10**	**PASS**
$R_n = F_y \cdot A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	5.96 in^2		*Gross area subject to tension*		
φR_n	193.10 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.02**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	19.03 kips		*Calculated axial load*		
V_r	10.95 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	5.96 in^2		*Gross area of the plate*		
Z_{pl}	11.92 in^3		*Plastic modulus of the shear plate*		
P_c	193.10 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	128.74 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	32.18 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.02		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	10.95 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	5.96 in^2		*Net area of the plate*		
Z_{net}	11.92 in^3		*Plastic modulus of net section*		
V_c	155.56 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	43.21 kips-ft		*Available moment $M_c = \varphi*(F_u * Znet)$, $\varphi=0.75$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Beam Weld Strength		10.95 kips	71.27 kips	**0.15**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φRn	71.27 kips		*Weld strength*		

Beam Web Yielding		19.03 kips	266.25 kips	**0.07**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		$\varphi = 1.00$	*(J10-2)*		
k	1.24 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	8.00 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.38 in		*Beam web thickness*		
φRn	266.25 kips		*Beam web local yielding*		

X-3 Bottom Detail: Top Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x61	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.63x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.74x59.83x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	81.91 kips	*Calculated Shear Load*
Axial Load	6.85 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		

E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
B	5.00 in		*Column face width*		
(B - 3 * t) / t	7.75		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	10.75		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.74 in		*Plate thickness*		
$t_{p\text{-}max}$	0.75 in		*Maximum allowed plate thickness*		

Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.47 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	59.83 in		*Min weld segment length*		

Plate Shear Yield		81.91 kips	962.73 kips	**0.09**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	44.57 in^2		*Gross area subject to shear*		
φR_n	962.73 kips		*Shear yield strength*		

Plate Shear Rupture		81.91 kips	1163.30 kips	**0.07**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	44.57 in^2		*Net area subject to shear*		
φR_n	1163.30 kips		*Shear rupture strength*		

Plate Axial Yield		6.85 kips	1444.10 kips	**0.00**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	44.57 in^2		*Gross area subject to tension*		
φR_n	1444.10 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	6.85 kips		*Calculated axial load*		
V_r	81.91 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	44.57 in^2		*Gross area of the plate*		
Z_{pl}	666.63 in^3		*Plastic modulus of the shear plate*		
P_c	1444.10 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	962.73 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	1799.91 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		

UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$\mathbf{(V_r/V_c)^2 + (M_r/M_c)^2 <= 1}$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	81.91 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	44.57 in^2		*Net area of the plate*		
Z_{net}	666.63 in^3		*Plastic modulus of net section*		
V_c	1163.30 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	2416.54 kips-ft		*Available moment $M_c = \varphi*(F_u * Znet)$, $\varphi=0.75$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		81.91 kips	532.98 kips	**0.15**	**PASS**
$\mathbf{\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L}$					
Double Fillet					
$\mathbf{1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75}$ **(AISC 14th Eqn 8-2a)**					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	59.83 in		*Weld length*		
φRn	532.98 kips		*Weld strength*		

HSS Transverse Plastification		6.85 kips	322.83 kips	**0.02**	**PASS**
$\mathbf{R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})}$		$\mathbf{\varphi = 1.00}$	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
t_p	0.74 in		*Plate thickness*		
l_b	59.83 in		*Plate length*		
B	5.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	322.83 kips		*Transverse plastification*		

HSS Flexural Plastification		0.00 kips-ft	906.58 kips-ft	**0.00**	**PASS**
$\mathbf{M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f}$		$\mathbf{\varphi = 1.0}$	*(K3-6)*		
B_b	1.25 in		*Plate bearing width*		
B	5.00 in		*Column width*		
β	0.25		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
H_b	59.83 in		*Depth of plate*		
η	11.97		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	906.58 kips-ft		*Flexural plastification*		

X-3 Bottom Detail: Top Gusset/Brace Report

Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x61	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.63x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.74x59.83x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	102.50 kips	*Brace Axial (compression)*
Brace Moment	10.92 kips-ft	*Brace Moment (not used for design)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.38 in		*Weld size*		
D_{max}	0.44 in		*Max Size Allowed*		
t	0.50 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.38 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.50 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.38 in		*Weld size*		
L_{min}	5.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.38 in		*Weld size*		
L_{max}	28.10 in		*Max weld segment length*		
Gusset Plate Compression (Whitmore)		102.50 kips	107.78 kips	**0.95**	**PASS**
$P_n = F_{cr}*A_g$		**φ = 0.9**	*(E3-1)*		
K	0.50		*Effective length factor*		
L	19.94 in		*Unbraced length*		
r	0.22 in		*Radius of gyration*		
KL/r	46.35		*Plate slenderness*		
F_{cr}	32.15 ksi		*Flexural buckling stress (E3-2)*		
A_g	3.73 in^2		*Gross area of plate (Whitmore section)*		
φP_n	107.78 kips		*Gusset plate compressive strength*		
Brace Weld Strength		102.50 kips	376.28 kips	**0.27**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
D_{16}	6.00		*Weld fillet size in sixteenths of an inch*		
L	45.05 in		*Weld length*		
φRn	376.28 kips		*Weld strength*		

X-3 Bottom Detail: Members Report

Vertical Brace Diagonal Connection

Beam		***W14x61***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	10.00 in	*Flange width*
d	13.90 in	*Overall depth*
t_w	0.38 in	*Web thickness*
t_f	0.65 in	*Flange thickness*
a	17.90 in^2	*Area*
k_{des}	1.24 in	*Kdes*
k_{det}	1.50 in	*Kdet*
k_1	1.00 in	*K1*

Column		***HSS5x5x8***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	5.00 in	*Depth*
b	5.00 in	*Width*
a	7.88 in^2	*Area*
t_{des}	0.47 in	*Wall Thickness*

Top Brace		***L5x3.5x8***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	3.50 in	*Flange width*
d	5.00 in	*Overall depth*
a	4.00 in^2	*Area*
t_{f1}	0.50 in	*Flange thickness*
t_{f2}	0.50 in	*Flange thickness*
k_{des}	0.94 in	*Kdes*
k_{det}	0.94 in	*Kdet*

X-3 Bottom Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.63x4.00x11.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.63 in	*Thickness*

Top Gusset		***P0.74x59.83x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	59.83 in	*Width*
t	0.74 in	*Thickness*
Clip		
H_clip	1.66 in	*Horiz. Clip*
V_clip	0.29 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.25 in	

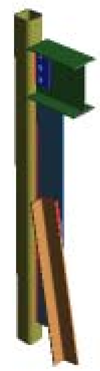
Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.38 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

X-3 Top Detail: 3D View

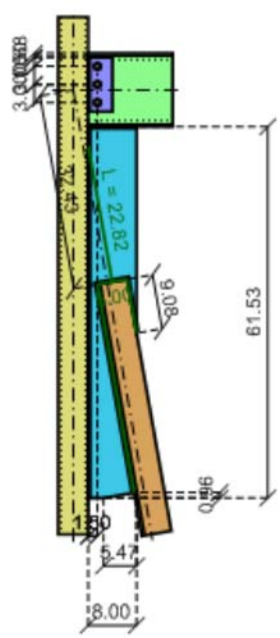
Vertical Brace Diagonal Connection



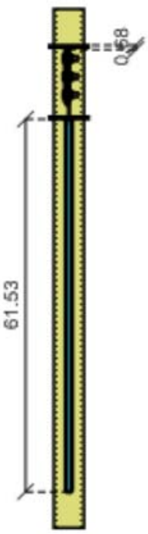
X-3 Top Detail: 2D Views

Vertical Brace Diagonal Connection

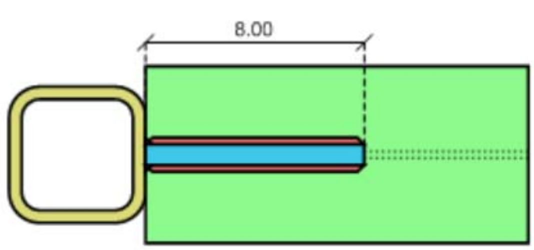
Side view



Front view



Bottom view



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.74x61.53x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	5.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	5.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	102.50 kips	*User Input Bottom Brace Axial Force (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	Bolt Shear at Beam	**0.39**	**PASS**
Bottom Gusset/Beam connection	Beam Weld Strength	**0.15**	**PASS**
Bottom Gusset/Column connection	Column Weld Strength	**0.15**	**PASS**
Bottom Gusset/Brace connection	Gusset Plate Compression (Whitmore)	**0.98**	**PASS**

X-3 Top Detail: Beam/Column Report

LRFD

Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.74x61.53x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-11.70 kips	*Calculated Shear Load*
Total Axial Load	11.85 kips	*Calculated Axial Load (compression)*
Column Force	5.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.47 in	*Column wall thickness*		
B	5.00 in	*Column face width*		
(B - 3 * t) / t	7.75	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	10.75	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		

F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
$t_{p\text{-}max}$	0.75 in		*Maximum allowed plate thickness*		
Geometry Restrictions at Beam					**PASS**
Check Min Bolt Spacing	**Pass**		*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in		*Min bolt spacing*		
d_{bolt}	0.75 in		*Bolt diameter*		
Check Max Bolt Spacing	**Pass**		*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in		*Max bolt spacing*		
t	0.23 in		*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**		*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**		*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.25 in		*Weld size*		
D_{max}	0.31 in		*Max Size Allowed*		
t	0.38 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	9.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	9.00 in		*Max weld segment length*		
Beam Shear Yield		11.70 kips	84.18 kips	**0.14**	**PASS**
R_n = 0.6 *F_y*A_{gv}*C_v		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	84.18 kips		*Shear yield strength*		
Plate Shear Yield		11.70 kips	72.90 kips	**0.16**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.38 in^2		*Gross area subject to shear*		
φR_n	72.90 kips		*Shear yield strength*		
Beam Shear Rupture		11.70 kips	64.42 kips	**0.18**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.20 in^2		*Net area subject to shear*		
φR_n	64.42 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		11.70 kips	62.40 kips	**0.19**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.39 in^2		*Net area subject to shear*		
φR_n	62.40 kips		*Shear rupture strength*		

Beam Axial Yield		11.85 kips	344.25 kips	**0.03**	**PASS**
$R_n = F_y*A_g$		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	7.65 in^2		*Gross area subject to tension*		
$φR_n$	344.25 kips		*Tensile yield strength*		

Plate Axial Yield		11.85 kips	109.35 kips	**0.11**	**PASS**
$R_n = F_y*A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to tension*		
$φR_n$	109.35 kips		*Tensile yield strength*		

Beam Block Shear		11.70 kips	79.89 kips	**0.15**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	3.02 in^2		*Gross area subject to shear*		
A_{nv}	2.52 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.24 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
$φR_n$	79.89 kips		*Block shear strength*		

Plate Block Shear at Beam		11.70 kips	79.21 kips	**0.15**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
A_{nv}	1.99 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.77 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
$φR_n$	79.21 kips		*Block shear strength*		

Compression Buckling of the Plate		11.85 kips	109.35 kips	**0.11**	**PASS**
$R_n = F_y * A_g$		**φ = 0.9**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	1.50 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	13.84		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	3.38 in^2		*Gross area subject to compression*		
$φR_n$	109.35 kips		*Compressive strength*		

Plate Flexural Yield				**0.04**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	11.85 kips		*Calculated axial load*		
V_r	-11.70 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area of the plate*		
Z_{pl}	7.59 in^3		*Plastic modulus of the shear plate*		
P_c	109.35 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	72.90 kips		*Available shear strength (see check 'Shear Yield')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.92 in		*Vertical eccentricity*		
M_r	-0.55 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.50 kips-ft		*Available moment $M_c=φ*(F_y*Z)$, $φ$=0.90*		

UC	0.04		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*	

Plate Flexural Rupture				**0.04**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	11.85 kips		*Calculated axial load*		
V_r	-11.70 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.39 in^2		*Net area of the plate*		
Z_{net}	5.55 in^3		*Plastic modulus of net section*		
V_c	62.40 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.92 in		*Vertical eccentricity*		
M_r	-0.55 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.13 kips-ft		*Available moment $M_c = \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.04		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Plate Flexural Buckling				**0.25**	**PASS**
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**φ = 0.90**	*(AISC 14th Edition)*		
P	11.85 kips		*Calculated axial load*		
V	11.70 kips		*Calculated shear load*		
L	1.50 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	13.84		*Slenderness ratio*		
F_e	1494.08 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2*E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	35.64 ksi		*Compression stress = F_y when KL/r <= 25, per J4.4*		
A_g	3.38 in^2		*Gross area of the plate*		
λ	0.33		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	3.74 in^3		*Section modulus of net section*		
a	1.50 in		*Design eccentricity*		
P_n	121.50 kips		*Compressive capacity, per eqn J4-1, Pn = Fy * Ag*		
V_n	89.67 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.25		*Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$*		

Bolt Bearing on Beam		16.65 kips	53.68 kips	**0.31**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-11.70 kips		*Applied shear force*		
P	11.85 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	16.65 kips		*Resultant shear force*		
Θ	44.65 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		

e_v	4.02 in		*Vertical edge spacing*		
e_h	1.50 in		*Horizontal edge spacing*		
L_{c_boltA}	1.70 in		*Minimum clear distance for the corner edge bolt: $Lc\_boltA = min((e_v /sin(\Theta), (e_h /cos(\Theta)) - d_c$*		
L_{c_boltB}	1.70 in		*Minimum clear distance for the side edge bolts: $Lc\_boltB = min((s_v - 0.5 * d_v /sin(\Theta)), (e_h /cos(\Theta)) - d_c$*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: $Rn\_boltA = min[(1.5* Lc\_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: $Rn\_boltB = min[(1.5* Lc\_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		
Bolt Bearing on Plate at Beam		16.65 kips	53.68 kips	**0.31**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**$\varphi = 0.75$**	*(J3-6b)*		
V	-11.70 kips		*Applied shear force*		
P	11.85 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	16.65 kips		*Resultant shear force*		
Θ	44.65 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.50 in		*Vertical edge spacing*		
e_h	2.50 in		*Horizontal edge spacing*		
L_{c_boltA}	1.73 in		*Minimum clear distance for the corner edge bolt: $Lc\_boltA = min((e_v /sin(\Theta), (e_h /cos(\Theta)) - d_c$*		
L_{c_boltB}	3.11 in		*Minimum clear distance for the side edge bolts: $Lc\_boltB = min((s_v - 0.5 * d_v /sin(\Theta)), (e_h /cos(\Theta)) - d_c$*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: $Rn\_boltA = min[(1.5* Lc\_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: $Rn\_boltB = min[(1.5* Lc\_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		
Bolt Shear at Beam		16.65 kips	42.80 kips	**0.39**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		**$\varphi = 0.75$**	*(J3-1)*		
V	-11.70 kips		*Applied shear force*		
P	11.85 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	16.65 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.80		*Eccentricity coefficient*		
φR_n	42.80 kips		*Bolt shear rupture strength*		
Bolt Group Eccentricity			***0.80***		
Elastic method			*(AISC 14^{th} p.7-6)*		

C	0.80		*Coefficient (2.3923 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.92 in		*Vertical eccentricity*		
Ang	44.65		*Angle of force in degrees, relative X axis*		
Weld at Column		16.65 kips	88.18 kips	**0.19**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16, \varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	9.00 in		*Weld length*		
φRn	88.18 kips		*Weld strength*		
HSS Transverse Plastification		11.85 kips	80.08 kips	**0.15**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	9.00 in		*Plate length*		
B	5.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	80.08 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	34.77 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	5.00 in		*Column width*		
β	0.17		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
H_b	9.00 in		*Depth of plate*		
η	1.80		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	34.77 kips-ft		*Flexural plastification*		

X-3 Top Detail: Bot Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.74x61.53x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	10.95 kips	*Calculated Shear Load*
Axial Load	16.70 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.00 in		*Min weld segment length*		
Plate Shear Yield		10.95 kips	128.74 kips	**0.09**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	5.96 in^2		*Gross area subject to shear*		
φR_n	128.74 kips		*Shear yield strength*		
Plate Shear Rupture		10.95 kips	155.56 kips	**0.07**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	5.96 in^2		*Net area subject to shear*		
φR_n	155.56 kips		*Shear rupture strength*		
Plate Axial Yield		16.70 kips	193.10 kips	**0.09**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	5.96 in^2		*Gross area subject to tension*		
φR_n	193.10 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	16.70 kips		*Calculated axial load*		
V_r	10.95 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	5.96 in^2		*Gross area of the plate*		
Z_{pl}	11.92 in^3		*Plastic modulus of the shear plate*		
P_c	193.10 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	128.74 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	32.18 kips-ft		*Available moment $M_c=\varphi*(F_y*Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	10.95 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	5.96 in^2		*Net area of the plate*		
Z_{net}	11.92 in^3		*Plastic modulus of net section*		
V_c	155.56 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	43.21 kips-ft		*Available moment $M_c = \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Beam Weld Strength		10.95 kips	71.27 kips	**0.15**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.00 in		*Weld length*		
φRn	71.27 kips		*Weld strength*		
Beam Web Yielding		16.70 kips	131.10 kips	**0.13**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		φ = 1.00	*(J10-2)*		
k	0.68 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	8.00 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.23 in		*Beam web thickness*		
φRn	131.10 kips		*Beam web local yielding*		

X-3 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.74x61.53x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	84.24 kips	*Calculated Shear Load*
Axial Load	6.85 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		

E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
B	5.00 in		*Column face width*		
(B - 3 * t) / t	7.75		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	10.75		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
F_{y-max}	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.74 in		*Plate thickness*		
t_{p-max}	0.75 in		*Maximum allowed plate thickness*		

Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.47 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	61.53 in		*Min weld segment length*		

Plate Shear Yield		84.24 kips	990.09 kips	**0.09**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	45.84 in^2		*Gross area subject to shear*		
φR_n	990.09 kips		*Shear yield strength*		

Plate Shear Rupture		84.24 kips	1196.36 kips	**0.07**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	45.84 in^2		*Net area subject to shear*		
φR_n	1196.36 kips		*Shear rupture strength*		

Plate Axial Yield		6.85 kips	1485.13 kips	**0.00**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	45.84 in^2		*Gross area subject to tension*		
φR_n	1485.13 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14^{th} Eq.10-5)*		
P_r	6.85 kips		*Calculated axial load*		
V_r	84.24 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	45.84 in^2		*Gross area of the plate*		
Z_{pl}	705.06 in^3		*Plastic modulus of the shear plate*		
P_c	1485.13 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	990.09 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	1903.65 kips-ft		*Available moment M_c=φ*(F_y* Z), φ=0.90*		

UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	84.24 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	45.84 in^2		*Net area of the plate*		
Z_{net}	705.06 in^3		*Plastic modulus of net section*		
V_c	1196.36 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	2555.82 kips-ft		*Available moment $M_c = \varphi*(F_u* Znet)$, $\varphi=0.75$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Column Weld Strength		84.24 kips	548.13 kips	**0.15**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	61.53 in		*Weld length*		
φRn	548.13 kips		*Weld strength*		
HSS Transverse Plastification		6.85 kips	330.77 kips	**0.02**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
t_p	0.74 in		*Plate thickness*		
l_b	61.53 in		*Plate length*		
B	5.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	330.77 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	955.37 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	1.25 in		*Plate bearing width*		
B	5.00 in		*Column width*		
β	0.25		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.47 in		*Column wall thickness*		
H_b	61.53 in		*Depth of plate*		
η	12.31		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	955.37 kips-ft		*Flexural plastification*		

X-3 Top Detail: Bot Gusset/Brace Report

Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS5x5x8	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.74x61.53x8.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	102.50 kips	*Brace Axial (compression)*
Brace Moment	10.92 kips-ft	*Brace Moment (not used for design)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.38 in		*Weld size*		
D_{max}	0.44 in		*Max Size Allowed*		
t	0.50 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.38 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.50 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.38 in		*Weld size*		
L_{min}	5.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.38 in		*Weld size*		
L_{max}	35.04 in		*Max weld segment length*		
Gusset Plate Compression (Whitmore)		102.50 kips	104.07 kips	**0.98**	**PASS**
$P_n = F_{cr} * A_g$		**φ = 0.9**	*(E3-1)*		
K	0.50		*Effective length factor*		
L	22.82 in		*Unbraced length*		
r	0.22 in		*Radius of gyration*		
KL/r	53.05		*Plate slenderness*		
F_{cr}	31.04 ksi		*Flexural buckling stress (E3-2)*		
A_g	3.73 in^2		*Gross area of plate (Whitmore section)*		
φP_n	104.07 kips		*Gusset plate compressive strength*		
Brace Weld Strength		102.50 kips	410.26 kips	**0.25**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
D_{16}	6.00		*Weld fillet size in sixteenths of an inch*		
L	49.12 in		*Weld length*		
φRn	410.26 kips		*Weld strength*		

X-3 Top Detail: Members Report

Vertical Brace Diagonal Connection

Beam		***W12x26***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	6.49 in	*Flange width*
d	12.20 in	*Overall depth*
t_w	0.23 in	*Web thickness*
t_f	0.38 in	*Flange thickness*
a	7.65 in^2	*Area*
k_{des}	0.68 in	*Kdes*
k_{det}	1.06 in	*Kdet*
k_1	0.75 in	*K1*
Web Hole Type		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Column		***HSS5x5x8***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	5.00 in	*Depth*
b	5.00 in	*Width*
a	7.88 in^2	*Area*
t_{des}	0.47 in	*Wall Thickness*

Bottom Brace		***L5x3.5x8***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	3.50 in	*Flange width*
d	5.00 in	*Overall depth*
a	4.00 in^2	*Area*
t_{f1}	0.50 in	*Flange thickness*
t_{f2}	0.50 in	*Flange thickness*
k_{des}	0.94 in	*Kdes*
k_{det}	0.94 in	*Kdet*

X-3 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.00x9.00***

Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Bottom Gusset		***P0.74x61.53x8.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	61.53 in	*Width*
t	0.74 in	*Thickness*
Clip		
H_clip	5.47 in	*Horiz. Clip*
V_clip	0.96 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.38 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

RISAConnection version 7.0.1

02/07/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

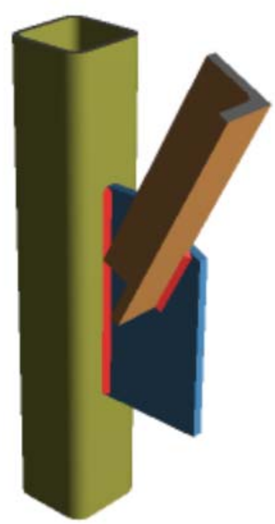
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

X-4 Bottom Detail	**PASS(UC-0.8)**
X-4 Top Detail	**PASS(UC-0.5)**

X-4 Bottom Detail: 3D View

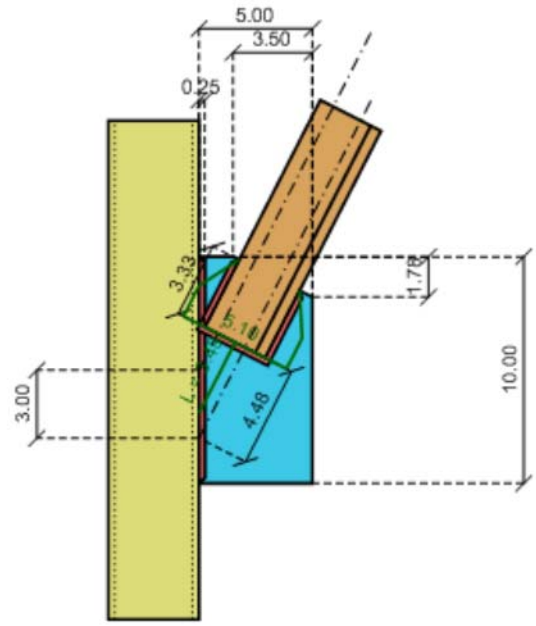
Knee Brace Connection



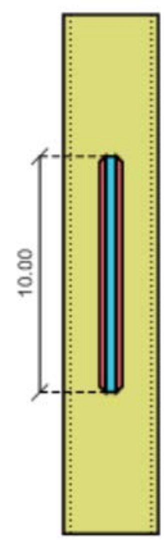
X-4 Bottom Detail: 2D Views

Knee Brace Connection

Left view

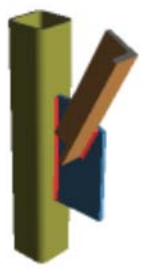


Front view



X-4 Bottom Detail: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:

Column	HSS4x4x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	L3x2.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Gusset	P0.47x5.00x10.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	41.00 kips	*Brace Axial (compression)*
Shear Load	36.53 kips	*Calculated Shear Load*
Axial Load	18.61 kips	*Calculated Axial Load (compression)*
Moment Load	4.65 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.29 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	10.75	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	13.75	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.47 in	*Plate thickness*		
t_{p-max}	0.47 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_V<= 0.5*d_{gusset}*		
WP_V	3.00 in	*Vertical brace workpoint offset*		
d_{gusset}	10.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.29 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in	*Weld size*		
L_{min}	10.00 in	*Min weld segment length*		
Brace Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		

D_{max}	0.44 in		*Max Size Allowed*		
t	0.50 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.47 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	3.33 in		*Max weld segment length*		

Plate Shear Yield		36.53 kips	100.44 kips	**0.36**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	4.65 in^2		*Gross area subject to shear*		
φRn	100.44 kips		*Shear yield strength*		

Plate Shear Rupture		36.53 kips	121.36 kips	**0.30**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	4.65 in^2		*Net area subject to shear*		
φRn	121.36 kips		*Shear rupture strength*		

Plate Axial Yield		18.61 kips	150.66 kips	**0.12**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.65 in^2		*Gross area subject to tension*		
φRn	150.66 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.21**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	18.61 kips		*Calculated axial load*		
V_r	36.53 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.65 in^2		*Gross area of the plate*		
Z_{pl}	11.63 in^3		*Plastic modulus of the shear plate*		
P_c	150.66 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	100.44 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	4.65 kips-ft		*Calculated moment*		
M_c	31.39 kips-ft		*Available moment $M_c = \varphi*(F_y * Z)$, φ=0.90*		
UC	0.21		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.10**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	36.53 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	4.65 in^2		*Net area of the plate*		
Z_{net}	11.63 in^3		*Plastic modulus of net section*		
V_c	121.36 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	4.65 kips-ft		*Calculated moment*		
M_c	42.14 kips-ft		*Available moment $M_c = \varphi*(F_u * Znet)$, φ=0.75*		
UC	0.10		*Unity check per interaction equation, $(V_r/V_c)^2$ +*		

			$(M_r/M_c)^2$ <= 1		

Column Weld Strength		4.96 kips/in	8.91 kips/in	**0.56**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					
V	36.53 kips		*Shear Load*		
P	18.61 kips		*Axial Load*		
M	4.65 kips-ft		*Moment*		
e_{eff}	1.53 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
r_u	4.96 kips/in		*Required weld stress per AISC 14^{th} Eqn 8-11*		
φRn	8.91 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		41.00 kips	73.02 kips	**0.56**	**PASS**
$P_n = F_{cr}*A_g$		**φ = 0.9**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	3.49 in		*Unbraced length*		
r	0.13 in		*Radius of gyration*		
KL/r	31.24		*Plate slenderness*		
F_{cr}	34.20 ksi		*Flexural buckling stress (E3-2)*		
A_g	2.37 in^2		*Gross area of plate (Whitmore section)*		
φP_n	73.02 kips		*Gusset plate compressive strength*		

Brace Weld Strength		41.00 kips	53.82 kips	**0.76**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14^{th} Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14^{th} table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14^{th} Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	9.67 in		*Weld length*		
φRn	53.82 kips		*Weld strength*		

HSS Transverse Plastification		18.61 kips	38.61 kips	**0.48**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.29 in		*Column wall thickness*		
t_p	0.47 in		*Plate thickness*		
l_b	10.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	38.61 kips		*Transverse plastification*		

HSS Flexural Plastification		4.65 kips-ft	18.80 kips-ft	**0.25**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.97 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.24		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		

t	0.29 in	*Column wall thickness*
H_b	10.00 in	*Depth of plate*
η	2.50	*Load length parameter (H_b / B)*
Q_f	1.00	*User input column stress interaction parameter*
M_{req}	4.65 kips-ft	*Required flexural plastification*
φM_n	18.80 kips-ft	*Flexural plastification*

X-4 Bottom Detail: Members Report

Knee Brace Connection

Column		***HSS4x4x5***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	4.10 in^2	*Area*
t_{des}	0.29 in	*Wall Thickness*

Brace		***L3x2.5x8***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	2.50 in	*Flange width*
d	3.00 in	*Overall depth*
a	2.50 in^2	*Area*
t_{f1}	0.50 in	*Flange thickness*
t_{f2}	0.50 in	*Flange thickness*
k_{des}	0.88 in	*Kdes*
k_{det}	0.88 in	*Kdet*

X-4 Bottom Detail: Components Report

Knee Brace Connection

Gusset		***P0.47x5.00x10.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	5.00 in	*Width*
t	0.47 in	*Thickness*
Clip		
H_clip	3.50 in	*Horiz. Clip*
V_clip	1.78 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

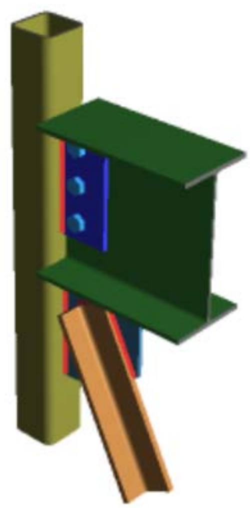
Brace Gusset Weld	***E70***	

Weld Properties

Type	Single Fillet
Fillet Size	0.25 in

X-4 Top Detail: 3D View

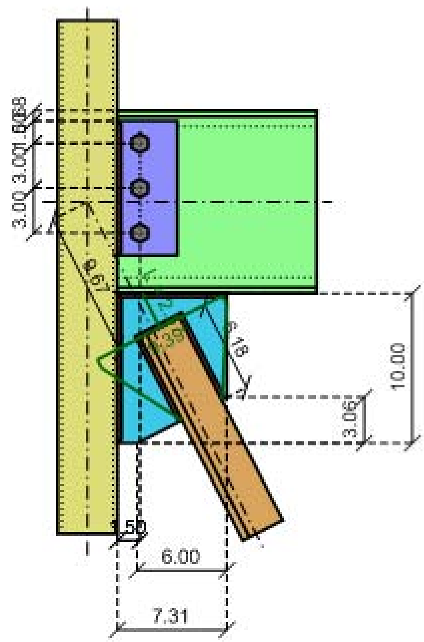
Vertical Brace Diagonal Connection



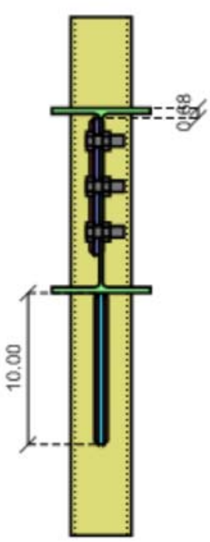
X-4 Top Detail: 2D Views

Vertical Brace Diagonal Connection

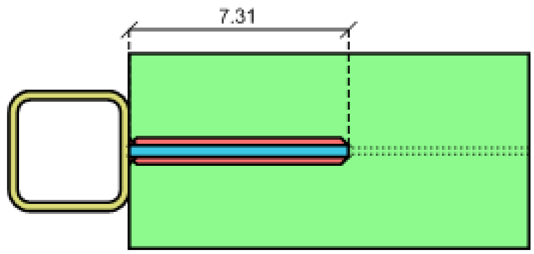
Side view



Front view

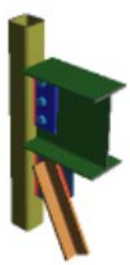


Bottom view



X-4 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L3x2.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x10.00x7.31	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	1.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	41.00 kips	*User Input Bottom Brace Axial Force (compression)*

Note: Unless specified, all code references are from AISC 360-10

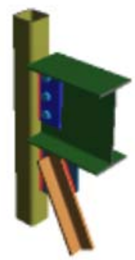
Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	Bolt Shear at Beam	**0.47**	**PASS**
Bottom Gusset/Beam connection	Plate Axial Yield	**0.23**	**PASS**
Bottom Gusset/Column connection	Column Weld Strength	**0.21**	**PASS**
Bottom Gusset/Brace connection	Gusset Plate Compression (Whitmore)	**0.54**	**PASS**

LRFD

X-4 Top Detail: Beam/Column Report

Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L3x2.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x10.00x7.31	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-19.08 kips	*Calculated Shear Load*
Total Axial Load	11.58 kips	*Calculated Axial Load (compression)*
Column Force	2.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.29 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	10.75	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	13.75	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
$t_{p\text{-}max}$	0.47 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.23 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		
t	0.38 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		

t_{min}	0.29 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	9.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	9.00 in		*Max weld segment length*		

Beam Shear Yield		19.08 kips	84.18 kips	**0.23**	**PASS**
R_n = 0.6 *F_y*A_{gv}*C_v		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	84.18 kips		*Shear yield strength*		

Plate Shear Yield		19.08 kips	72.90 kips	**0.26**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.38 in^2		*Gross area subject to shear*		
φR_n	72.90 kips		*Shear yield strength*		

Beam Shear Rupture		19.08 kips	64.42 kips	**0.30**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.20 in^2		*Net area subject to shear*		
φR_n	64.42 kips		*Shear rupture strength*		

Plate Shear Rupture at Beam		19.08 kips	62.40 kips	**0.31**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.39 in^2		*Net area subject to shear*		
φR_n	62.40 kips		*Shear rupture strength*		

Beam Axial Yield		11.58 kips	344.25 kips	**0.03**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	7.65 in^2		*Gross area subject to tension*		
φR_n	344.25 kips		*Tensile yield strength*		

Plate Axial Yield		11.58 kips	109.35 kips	**0.11**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to tension*		
φR_n	109.35 kips		*Tensile yield strength*		

Beam Block Shear		19.08 kips	79.89 kips	**0.24**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	3.02 in^2		*Gross area subject to shear*		
A_{nv}	2.52 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.24 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	79.89 kips		*Block shear strength*		

Plate Block Shear at Beam		19.08 kips	79.21 kips	**0.24**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		

A_{nv}	1.99 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.77 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	79.21 kips		*Block shear strength*		

Compression Buckling of the Plate		11.58 kips	109.35 kips	**0.11**	**PASS**
$R_n = F_y * A_g$		**φ = 0.9**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	1.50 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	13.84		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	3.38 in^2		*Gross area subject to compression*		
φR_n	109.35 kips		*Compressive strength*		

Plate Flexural Yield				**0.10**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	11.58 kips		*Calculated axial load*		
V_r	-19.08 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area of the plate*		
Z_{pl}	7.59 in^3		*Plastic modulus of the shear plate*		
P_c	109.35 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	72.90 kips		*Available shear strength (see check 'Shear Yield')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.92 in		*Vertical eccentricity*		
M_r	-1.50 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.50 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.10		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.10**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	11.58 kips		*Calculated axial load*		
V_r	-19.08 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.39 in^2		*Net area of the plate*		
Z_{net}	5.55 in^3		*Plastic modulus of net section*		
V_c	62.40 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.92 in		*Vertical eccentricity*		
M_r	-1.50 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.13 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.10		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Plate Flexural Buckling				**0.34**	**PASS**
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**φ = 0.90**	*(AISC 14th Edition)*		
P	11.58 kips		*Calculated axial load*		
V	19.08 kips		*Calculated shear load*		
L	1.50 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	13.84		*Slenderness ratio*		

F_e	1494.08 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	35.64 ksi		*Compression stress = F_y when KL/r <= 25, per J4.4*		
A_g	3.38 in^2		*Gross area of the plate*		
λ	0.33		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	3.74 in^3		*Section modulus of net section*		
a	1.50 in		*Design eccentricity*		
P_n	121.50 kips		*Compressive capacity, per eqn J4-1, Pn = Fy * Ag*		
V_n	89.67 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.34		*Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$*		
Bolt Bearing on Beam		22.32 kips	53.68 kips	**0.42**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-19.08 kips		*Applied shear force*		
P	11.58 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	22.32 kips		*Resultant shear force*		
Θ	58.74 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	4.02 in		*Vertical edge spacing*		
e_h	1.50 in		*Horizontal edge spacing*		
L_{c_boltA}	2.48 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.48 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), $R_{n\text{-}bolt}$]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), $R_{n\text{-}bolt}$]*		
$R_{n\text{-}bolt}$	23.86 kips		*Bolt shear strength $R_{n\text{-}bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		
Bolt Bearing on Plate at Beam		22.32 kips	53.68 kips	**0.42**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		**φ = 0.75**	*(J3-6b)*		
V	-19.08 kips		*Applied shear force*		
P	11.58 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	22.32 kips		*Resultant shear force*		
Θ	58.74 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		

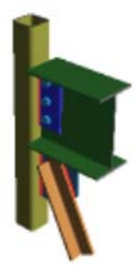
F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.50 in		*Vertical edge spacing*		
e_h	2.50 in		*Horizontal edge spacing*		
L_{c_boltA}	1.35 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.63 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength R_{n-bolt}=F_{nv}*A_{bolt}*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		
Bolt Shear at Beam		22.32 kips	46.99 kips	**0.47**	**PASS**
$R_n = F_{nv} * A_b * N_{bolt} * C$		**φ = 0.75**	*(J3-1)*		
V	-19.08 kips		*Applied shear force*		
P	11.58 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	22.32 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.88		*Eccentricity coefficient*		
φR_n	46.99 kips		*Bolt shear rupture strength*		
Bolt Group Eccentricity			***0.88***		
Elastic method			*(AISC 14th p.7-6)*		
C	0.88		*Coefficient (2.6263 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.92 in		*Vertical eccentricity*		
Ang	58.74		*Angle of force in degrees, relative X axis*		
Weld at Column		22.32 kips	88.18 kips	**0.25**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
1.392 = φ * 0.6 * F_{E70} * $2^{0.5}$/2 * 1/16, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	9.00 in		*Weld length*		
φR_n	88.18 kips		*Weld strength*		
HSS Transverse Plastification		11.58 kips	35.71 kips	**0.32**	**PASS**
$R_n = F_y * t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.29 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	9.00 in		*Plate length*		

B	4.00 in		*Column width*		
Qf	1.00		*User input column stress interaction parameter*		
φRn	35.71 kips		*Transverse plastification*		

HSS Flexural Plastification		0.00 kips-ft	15.67 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
Bb	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (Bb / B)*		
Fy	46.00 ksi		*Column yield strength*		
t	0.29 in		*Column wall thickness*		
Hb	9.00 in		*Depth of plate*		
η	2.25		*Load length parameter (Hb / B)*		
Qf	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
Mreq	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φMn	15.67 kips-ft		*Flexural plastification*		

X-4 Top Detail: Bot Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	Fy = 50.00 ksi	Fu = 65.00 ksi
Column	HSS4x4x5	A500 Gr.B Rect	Fy = 46.00 ksi	Fu = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	Fy = 36.00 ksi	Fu = 58.00 ksi
Bottom Brace	L3x2.5x8	A36	Fy = 36.00 ksi	Fu = 58.00 ksi
Bottom Gusset	P0.38x10.00x7.31	A36	Fy = 36.00 ksi	Fu = 58.00 ksi

Input Data:

Shear Load	12.03 kips	*Calculated Shear Load*
Axial Load	20.08 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

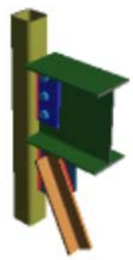
Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
Dmin	0.19 in		*Min size allowed per Table J2.4*		
tmin	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: Lmin >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
Lmin	7.31 in		*Min weld segment length*		
Plate Shear Yield		12.03 kips	59.22 kips	**0.20**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
Fy	36.00 ksi		*Minimum yield stress of material*		
Agv	2.74 in^2		*Gross area subject to shear*		
φRn	59.22 kips		*Shear yield strength*		
Plate Shear Rupture		12.03 kips	71.56 kips	**0.17**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
Fu	58.00 ksi		*Minimum tensile stress of material*		
Anv	2.74 in^2		*Net area subject to shear*		

φR_n	71.56 kips		*Shear rupture strength*		
Plate Axial Yield		20.08 kips	88.83 kips	**0.23**	**PASS**
$R_n = F_y*A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.74 in^2		*Gross area subject to tension*		
φR_n	88.83 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.09**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	20.08 kips		*Calculated axial load*		
V_r	12.03 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.74 in^2		*Gross area of the plate*		
Z_{pl}	5.01 in^3		*Plastic modulus of the shear plate*		
P_c	88.83 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	59.22 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	13.53 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, $\varphi=0.90$*		
UC	0.09		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.03**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	12.03 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.74 in^2		*Net area of the plate*		
Z_{net}	5.01 in^3		*Plastic modulus of net section*		
V_c	71.56 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	18.17 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, $\varphi=0.75$*		
UC	0.03		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Beam Weld Strength		12.03 kips	57.31 kips	**0.21**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	7.31 in		*Weld length*		
φRn	57.31 kips		*Weld strength*		
Beam Web Yielding		20.08 kips	123.18 kips	**0.16**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		**$\varphi = 1.00$**	*(J10-2)*		
k	0.68 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	7.31 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.23 in		*Beam web thickness*		

φ**Rn**	123.18 kips	*Beam web local yielding*

X-4 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L3x2.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x10.00x7.31	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	16.46 kips	*Calculated Shear Load*
Axial Load	6.58 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.29 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	10.75		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	13.75		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
$t_{p\text{-}max}$	0.47 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.29 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	10.00 in		*Min weld segment length*		
Plate Shear Yield		16.46 kips	81.00 kips	**0.20**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.75 in^2		*Gross area subject to shear*		
φ**Rn**	81.00 kips		*Shear yield strength*		

Plate Shear Rupture		16.46 kips	97.88 kips	**0.17**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.75 in^2		*Net area subject to shear*		
φR_n	97.88 kips		*Shear rupture strength*		

Plate Axial Yield		6.58 kips	121.50 kips	**0.05**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.75 in^2		*Gross area subject to tension*		
φR_n	121.50 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.04**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	6.58 kips		*Calculated axial load*		
V_r	16.46 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.75 in^2		*Gross area of the plate*		
Z_{pl}	9.38 in^3		*Plastic modulus of the shear plate*		
P_c	121.50 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	81.00 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	25.31 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.04		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.03**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	16.46 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.75 in^2		*Net area of the plate*		
Z_{net}	9.37 in^3		*Plastic modulus of net section*		
V_c	97.88 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	33.98 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.03		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		16.46 kips	78.38 kips	**0.21**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	10.00 in		*Weld length*		
φRn	78.38 kips		*Weld strength*		

HSS Transverse Plastification		6.58 kips	37.86 kips	**0.17**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Qf*(1-t_p/B)^{0.5})$		**$\varphi = 1.00$**	*(K1-12)*		

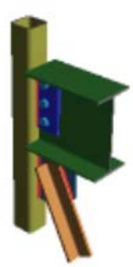
F_y	46.00 ksi	*Column yield strength*
t	0.29 in	*Column wall thickness*
t_p	0.38 in	*Plate thickness*
l_b	10.00 in	*Plate length*
B	4.00 in	*Column width*
Q_f	1.00	*User input column stress interaction parameter*
φR_n	37.86 kips	*Transverse plastification*

	Required	Available	Unity Check	Result
HSS Flexural Plastification	0.00 kips-ft	18.38 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$	φ = 1.0	*(K3-6)*		

B_b	0.88 in	*Plate bearing width*
B	4.00 in	*Column width*
β	0.22	*Width ratio (B_b / B)*
F_y	46.00 ksi	*Column yield strength*
t	0.29 in	*Column wall thickness*
H_b	10.00 in	*Depth of plate*
η	2.50	*Load length parameter (H_b / B)*
Q_f	1.00	*User input column stress interaction parameter*
M_{req}	0.00 kips-ft	*Required flexural plastification*
φM_n	18.38 kips-ft	*Flexural plastification*

X-4 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection



Material Properties:

Beam	W12x26	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x5	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L3x2.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x10.00x7.31	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	41.00 kips	*Brace Axial (compression)*
Brace Moment	3.20 kips-ft	*Brace Moment (not used for design)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.25 in		*Weld size*		
D_{max}	0.44 in		*Max Size Allowed*		
t	0.50 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	3.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	6.18 in		*Max weld segment length*		
Gusset Plate Compression (Whitmore)		41.00 kips	75.26 kips	**0.54**	**PASS**

$P_n = F_y * A_g$		$\varphi = 0.9$	*(J4-6)*		
K	0.50		*Effective length factor*		
L	2.57 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	11.88		*Plate slenderness*		
F_y	36.00 ksi		*Gusset plate yield stress*		
A_g	2.32 in^2		*Gross area of plate (Whitmore section)*		
φP_n	75.26 kips		*Gusset plate compressive strength*		

Brace Weld Strength		41.00 kips	85.49 kips	**0.48**	PASS
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	15.35 in		*Weld length*		
φRn	85.49 kips		*Weld strength*		

X-4 Top Detail: Members Report

Vertical Brace Diagonal Connection

Beam		***W12x26***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	6.49 in	*Flange width*
d	12.20 in	*Overall depth*
t_w	0.23 in	*Web thickness*
t_f	0.38 in	*Flange thickness*
a	7.65 in^2	*Area*
k_{des}	0.68 in	*Kdes*
k_{det}	1.06 in	*Kdet*
k_1	0.75 in	*K1*
Web Hole Type		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Column		***HSS4x4x5***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	4.10 in^2	*Area*

t_{des}	0.29 in	*Wall Thickness*

Bottom Brace		***L3x2.5x8***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	2.50 in	*Flange width*
d	3.00 in	*Overall depth*
a	2.50 in^2	*Area*
t_{f1}	0.50 in	*Flange thickness*
t_{f2}	0.50 in	*Flange thickness*
k_{des}	0.88 in	*Kdes*
k_{det}	0.88 in	*Kdet*

X-4 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.00x9.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Bottom Gusset		***P0.38x10.00x7.31***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	10.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	6.00 in	*Horiz. Clip*
V_clip	3.06 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	

d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.25 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

RISAConnection version 7.0.1

02/08/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

X-5 Bottom Detail	**PASS(UC-0.5)**
X-5 Top Detail	**PASS(UC-0.5)**

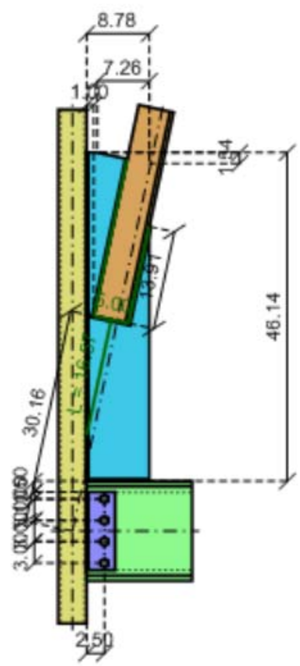
X-5 Bottom Detail: 3D View

Vertical Brace Diagonal Connection

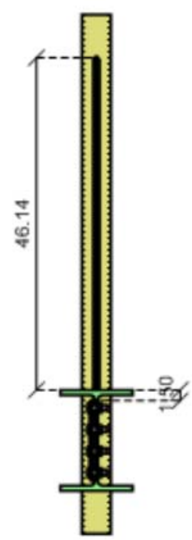
X-5 Bottom Detail: 2D Views

Vertical Brace Diagonal Connection

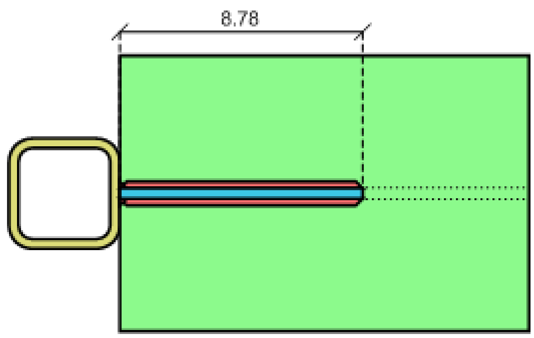
Side view



Front view



Top view



X-5 Bottom Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x68	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.50x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x46.14x8.78	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	27.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	35.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Top Brace Axial	20.10 kips	*User Input Top Brace Axial Force (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	Bolt Bearing on Beam	**0.45**	**PASS**
Top Gusset/Beam connection	Plate Axial Yield	**0.04**	**PASS**
Top Gusset/Column connection	Column Weld Strength	**0.04**	**PASS**
Top Gusset/Brace connection	Gusset Plate Compression (Whitmore)	**0.45**	**PASS**

LRFD

X-5 Bottom Detail: Beam/Column Report

Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x68	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.50x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x46.14x8.78	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	31.58 kips	*Calculated Shear Load*
Total Axial Load	6.31 kips	*Calculated Axial Load (compression)*
Column Force	35.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.35 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	8.46	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	11.46	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.50 in	*Plate thickness*		
$t_{p\text{-}max}$	0.56 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.41 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.44 in	*Max Size Allowed*		
t	0.50 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		

t_{min}	0.35 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	11.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	11.00 in		*Max weld segment length*		

Beam Shear Yield		31.58 kips	174.30 kips	**0.18**	PASS
R_n = 0.6 *F_y*A_{gv}*C_v		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	5.81 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	174.30 kips		*Shear yield strength*		

Plate Shear Yield		31.58 kips	118.80 kips	**0.27**	PASS
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	5.50 in^2		*Gross area subject to shear*		
φR_n	118.80 kips		*Shear yield strength*		

Beam Shear Rupture		31.58 kips	127.46 kips	**0.25**	PASS
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	4.36 in^2		*Net area subject to shear*		
φR_n	127.46 kips		*Shear rupture strength*		

Plate Shear Rupture at Beam		31.58 kips	97.87 kips	**0.32**	PASS
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	3.75 in^2		*Net area subject to shear*		
φR_n	97.87 kips		*Shear rupture strength*		

Beam Axial Yield		6.31 kips	900.00 kips	**0.01**	PASS
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	20.00 in^2		*Gross area subject to tension*		
φR_n	900.00 kips		*Tensile yield strength*		

Plate Axial Yield		6.31 kips	178.20 kips	**0.04**	PASS
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	5.50 in^2		*Gross area subject to tension*		
φR_n	178.20 kips		*Tensile yield strength*		

Beam Block Shear		31.58 kips	230.11 kips	**0.14**	PASS
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	8.37 in^2		*Gross area subject to shear*		
A_{nv}	7.10 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.86 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	230.11 kips		*Block shear strength*		

Plate Block Shear at Beam		31.58 kips	104.11 kips	**0.30**	PASS
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	5.00 in^2		*Gross area subject to shear*		

A_{nv}	3.47 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.53 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	104.11 kips		*Block shear strength*		
Compression Buckling of the Plate		6.31 kips	178.20 kips	**0.04**	**PASS**
$R_n = F_y * A_g$		φ **= 0.9**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	2.50 in		*Unbraced length*		
r	0.14 in		*Radius of gyration*		
KL/r	17.30		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	5.50 in^2		*Gross area subject to compression*		
φR_n	178.20 kips		*Compressive strength*		
Plate Flexural Yield				**0.11**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	6.31 kips		*Calculated axial load*		
V_r	31.58 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	5.50 in^2		*Gross area of the plate*		
Z_{pl}	15.13 in^3		*Plastic modulus of the shear plate*		
P_c	178.20 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	118.80 kips		*Available shear strength (see check 'Shear Yield')*		
ex	2.50 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_r	6.58 kips-ft		*Moment due to eccentricity =* V_r*ex + P_r*ey		
M_c	40.84 kips-ft		*Available moment* $M_c=\varphi*(F_y* Z)$, φ=0.90		
UC	0.11		*Unity check per interaction equation,* $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$		
Plate Flexural Rupture				**0.14**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	6.31 kips		*Calculated axial load*		
V_r	31.58 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.75 in^2		*Net area of the plate*		
Z_{net}	9.88 in^3		*Plastic modulus of net section*		
V_c	97.87 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	2.50 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_r	6.58 kips-ft		*Moment due to eccentricity =* V_r*ex + P_r*ey		
M_c	35.80 kips-ft		*Available moment* $M_c= \varphi*(F_u* Znet)$, φ=0.75		
UC	0.14		*Unity check per interaction equation,* $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		
Plate Flexural Buckling				**0.41**	**PASS**
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		φ **= 0.90**	*(AISC 14th Edition)*		
P	6.31 kips		*Calculated axial load*		
V	31.58 kips		*Calculated shear load*		
L	2.50 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.14 in		*Radius of gyration of the plate*		
KL/r	17.30		*Slenderness ratio*		

F_e	956.21 ksi		Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$		
F_y	36.00 ksi		Minimum yield stress of material		
F_{cr_Comp}	35.44 ksi		Compression stress = F_y when KL/r <= 25, per J4.4		
A_g	5.50 in^2		Gross area of the plate		
λ	0.26		Buckling factor (pg 9.9) (eqn 9-18)		
Q	1.00		Buckling factor (eqn 9-15 through 9-17)		
F_{cr_Flex}	36.00 ksi		Critical stress, per eqn 9-14, Fcr = Fy * Q		
S_{net}	6.48 in^3		Section modulus of net section		
a	2.50 in		Design eccentricity		
P_n	198.00 kips		Compressive capacity, per eqn J4-1, Pn = Fy * Ag		
V_n	93.36 kips		Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a		
UC	0.41		Unity check per interaction equation, $P/(Pn*\Phi) + V/(Vn*\Phi) <= 1$		
Bolt Bearing on Beam		32.20 kips	71.57 kips	**0.45**	**PASS**
$R_n = 1*R_{n\_boltA} + 3*R_{n\_boltB}$		**φ = 0.75**	(J3-6b)		
V	31.58 kips		Applied shear force		
P	6.31 kips		Applied axial force		
$R=(V^2 + P^2)^{0.5}$	32.20 kips		Resultant shear force		
Θ	78.70 degrees		Angle between the resultant shear force and horizontal		
d_b	0.75 in		Bolt diameter		
d_v	0.81 in		Slotted hole vertical dimension		
d_h	0.81 in		Slotted hole horizontal dimension		
d_c	0.41 in		Distance from center of bolt to the edge of the hole		
F_u	65.00 ksi		Minimum tensile stress of material		
s_v	3.00 in		Vertical bolt spacing		
s_h	0.00 in		Horizontal bolt spacing		
e_v	2.50 in		Vertical edge spacing		
e_h	2.50 in		Horizontal edge spacing		
L_{c_boltA}	2.14 in		Minimum clear distance for the corner edge bolt: $Lc\_boltA = min((e_v /sin(\Theta), (e_h /cos(\Theta)) - d_c$		
L_{c_boltB}	2.24 in		Minimum clear distance for the side edge bolts: $Lc\_boltB = min((s_v - 0.5 * d_v /sin(\Theta)), (e_h /cos(\Theta))) - d_c$		
R_{n_boltA}	23.86 kips		Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]		
R_{n_boltB}	23.86 kips		Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]		
R_{n-bolt}	23.86 kips		Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$		
F_{nv}	54.00 ksi		Nominal shear stress of bolt		
φR_n	71.57 kips		Total bolt bearing strength		
Bolt Bearing on Plate at Beam		32.20 kips	71.57 kips	**0.45**	**PASS**
$R_n = 1*R_{n\_boltA} + 3*R_{n\_boltB}$		**φ = 0.75**	(J3-6b)		
V	31.58 kips		Applied shear force		
P	6.31 kips		Applied axial force		
$R=(V^2 + P^2)^{0.5}$	32.20 kips		Resultant shear force		
Θ	78.70 degrees		Angle between the resultant shear force and horizontal		
d_b	0.75 in		Bolt diameter		
d_v	0.81 in		Slotted hole vertical dimension		
d_h	0.81 in		Slotted hole horizontal dimension		
d_c	0.41 in		Distance from center of bolt to the edge of the hole		

F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.00 in		*Vertical edge spacing*		
e_h	1.50 in		*Horizontal edge spacing*		
L_{c_boltA}	0.61 in		*Minimum clear distance for the corner edge bolt: $Lc\_boltA = min((e_v /sin(\Theta), (e_h /cos(\Theta)) - d_c$*		
L_{c_boltB}	2.24 in		*Minimum clear distance for the side edge bolts: $Lc\_boltB = min((s_v - 0.5 * d_v /sin(\Theta)), (e_h /cos(\Theta))) - d_c$*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: $Rn\_boltA = min[(1.5* Lc\_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: $Rn\_boltB = min[(1.5* Lc\_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]$*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	71.57 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		32.20 kips	71.57 kips	**0.45**	**PASS**
$R_n = F_{nv}*A_b*N_{bolt}*C$		$\varphi = 0.75$	*(J3-1)*		
V	31.58 kips		*Applied shear force*		
P	6.31 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	32.20 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	$0.44\ in^2$		*Area of bolt*		
N_{bolt}	4		*Number of bolts*		
C	1.00		*Eccentricity coefficient*		
φR_n	71.57 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***1.00***		
Elastic method			*(AISC 14th p.7-6)*		
C	1.00		*Coefficient (4.0000 / 4)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	4		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.00 in		*Vertical eccentricity*		
Ang	78.70		*Angle of force in degrees, relative X axis*		

Weld at Column		32.20 kips	122.50 kips	**0.26**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	11.00 in		*Weld length*		
φR_n	122.50 kips		*Weld strength*		

HSS Transverse Plastification		6.31 kips	59.18 kips	**0.11**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		$\varphi = 1.00$	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.50 in		*Plate thickness*		
l_b	11.00 in		*Plate length*		

B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	59.18 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	31.63 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	1.00 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.25		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	11.00 in		*Depth of plate*		
η	2.75		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	31.63 kips-ft		*Flexural plastification*		

X-5 Bottom Detail: Top Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x68	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.50x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x46.14x8.78	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.87 kips	*Calculated Shear Load*
Axial Load	4.58 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	8.78 in		*Min weld segment length*		
Plate Shear Yield		2.87 kips	71.14 kips	**0.04**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.29 in^2		*Gross area subject to shear*		
φR_n	71.14 kips		*Shear yield strength*		
Plate Shear Rupture		2.87 kips	85.97 kips	**0.03**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		

A_{nv}	3.29 in^2		*Net area subject to shear*		
φR_n	85.97 kips		*Shear rupture strength*		

Plate Axial Yield		4.58 kips	106.72 kips	**0.04**	**PASS**
$R_n = F_y * A_g$		φ **= 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.29 in^2		*Gross area subject to tension*		
φR_n	106.72 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	4.58 kips		*Calculated axial load*		
V_r	2.87 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.29 in^2		*Gross area of the plate*		
Z_{pl}	7.23 in^3		*Plastic modulus of the shear plate*		
P_c	106.72 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	71.14 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	19.53 kips-ft		*Available moment* $M_c=\varphi*(F_y*Z)$, $\varphi=0.90$		
UC	0.00		*Unity check per interaction equation,* $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	2.87 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	3.29 in^2		*Net area of the plate*		
Z_{net}	7.23 in^3		*Plastic modulus of net section*		
V_c	85.97 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	26.22 kips-ft		*Available moment* $M_c=\varphi*(F_u*Z_{net})$, $\varphi=0.75$		
UC	0.00		*Unity check per interaction equation,* $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		

Beam Weld Strength		2.87 kips	68.85 kips	**0.04**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$ **Double Fillet**					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ **(AISC 14th Eqn 8-2a)**					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	8.78 in		*Weld length*		
φR_n	68.85 kips		*Weld strength*		

Beam Web Yielding		4.58 kips	318.16 kips	**0.01**	**PASS**
$R_n = (5 * k + N) * F_y * t_w$		φ **= 1.00**	*(J10-2)*		
k	1.31 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	8.78 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		

t_w	0.41 in	*Beam web thickness*
φR_n	318.16 kips	*Beam web local yielding*

X-5 Bottom Detail: Top Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x68	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.50x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x46.14x8.78	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	15.08 kips	*Calculated Shear Load*
Axial Load	1.31 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	8.46		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	11.46		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
$t_{p\text{-}max}$	0.56 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.35 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	46.14 in		*Min weld segment length*		
Plate Shear Yield		15.08 kips	373.74 kips	**0.04**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	17.30 in^2		*Gross area subject to shear*		

φR_n	373.74 kips		*Shear yield strength*		

Plate Shear Rupture		15.08 kips	451.60 kips	**0.03**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		$\varphi = 0.75$	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	17.30 in^2		*Net area subject to shear*		
φR_n	451.60 kips		*Shear rupture strength*		

Plate Axial Yield		1.31 kips	560.60 kips	**0.00**	**PASS**
$R_n = F_y * A_g$		$\varphi = 0.90$	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	17.30 in^2		*Gross area subject to tension*		
φR_n	560.60 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	1.31 kips		*Calculated axial load*		
V_r	15.08 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	17.30 in^2		*Gross area of the plate*		
Z_{pl}	199.59 in^3		*Plastic modulus of the shear plate*		
P_c	560.60 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	373.74 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	538.88 kips-ft		*Available moment* $M_c = \varphi * (F_y * Z)$, $\varphi = 0.90$		
UC	0.00		*Unity check per interaction equation,* $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	15.08 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	17.30 in^2		*Net area of the plate*		
Z_{net}	199.59 in^3		*Plastic modulus of net section*		
V_c	451.60 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	723.50 kips-ft		*Available moment* $M_c = \varphi * (F_u * Znet)$, $\varphi = 0.75$		
UC	0.00		*Unity check per interaction equation,* $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$		

Column Weld Strength		15.08 kips	361.67 kips	**0.04**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi = 0.75$ **(AISC 14th Eqn 8-2a)**					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	46.14 in		*Weld length*		
φR_n	361.67 kips		*Weld strength*		

HSS Transverse Plastification		1.31 kips	166.17 kips	**0.01**	**PASS**

$R_n = F_y \cdot t^2 / (1-t_p/B) \cdot (2l_b/B + 4 \cdot Q_f \cdot (1-t_p/B)^{0.5})$		$\varphi = 1.00$	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	46.14 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	166.17 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	367.76 kips-ft	**0.00**	**PASS**
$M_n = F_y \cdot t^2 \cdot H_b \cdot (1 / (2 \cdot \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) \cdot Q_f$		$\varphi = 1.0$	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.35 in		*Column wall thickness*		
H_b	46.14 in		*Depth of plate*		
η	11.54		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	367.76 kips-ft		*Flexural plastification*		

X-5 Bottom Detail: Top Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W14x68	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x6	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.50x4.00x11.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Top Gusset	P0.38x46.14x8.78	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	20.10 kips	*Brace Axial (compression)*
Brace Moment	1.83 kips-ft	*Brace Moment (not used for design)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.25 in		*Weld size*		
D_{max}	0.44 in		*Max Size Allowed*		
t	0.50 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	5.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		

D	0.25 in		*Weld size*		
L_{max}	22.55 in		*Max weld segment length*		

Gusset Plate Compression (Whitmore)		20.10 kips	44.63 kips	**0.45**	**PASS**
$P_n = F_{cr} * A_g$		$\varphi = 0.9$	*(E3-1)*		
K	0.50		*Effective length factor*		
L	16.57 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	76.54		*Plate slenderness*		
F_{cr}	26.45 ksi		*Flexural buckling stress (E3-2)*		
A_g	1.88 in^2		*Gross area of plate (Whitmore section)*		
φP_n	44.63 kips		*Gusset plate compressive strength*		

Brace Weld Strength		20.10 kips	230.86 kips	**0.09**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
1.392 = $\varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi=0.75$ **(AISC 14th Eqn 8-2a)**					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	41.46 in		*Weld length*		
φRn	230.86 kips		*Weld strength*		

X-5 Bottom Detail: Members Report

Vertical Brace Diagonal Connection

Beam		***W14x68***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	10.00 in	*Flange width*
d	14.00 in	*Overall depth*
t_w	0.41 in	*Web thickness*
t_f	0.72 in	*Flange thickness*
a	20.00 in^2	*Area*
k_{des}	1.31 in	*Kdes*
k_{det}	1.56 in	*Kdet*
k_1	1.06 in	*K1*
Web Hole Type		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	4	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Column		***HSS4x4x6***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		

d	4.00 in	*Depth*
b	4.00 in	*Width*
a	4.78 in^2	*Area*
t_{des}	0.35 in	*Wall Thickness*

Top Brace		***L5x3.5x8***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	3.50 in	*Flange width*
d	5.00 in	*Overall depth*
a	4.00 in^2	*Area*
t_{f1}	0.50 in	*Flange thickness*
t_{f2}	0.50 in	*Flange thickness*
k_{des}	0.94 in	*Kdes*
k_{det}	0.94 in	*Kdet*

X-5 Bottom Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.50x4.00x11.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.50 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	4	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Top Gusset		***P0.38x46.14x8.78***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	46.14 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	7.26 in	*Horiz. Clip*
V_clip	1.54 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	
d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.25 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

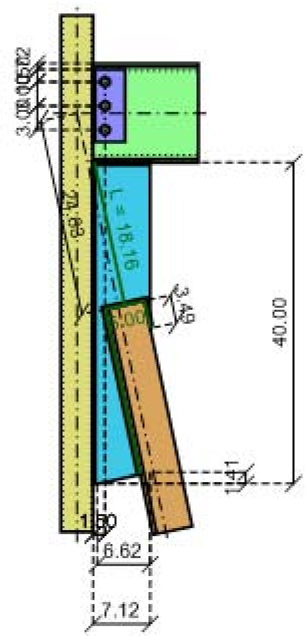
X-5 Top Detail: 3D View

Vertical Brace Diagonal Connection

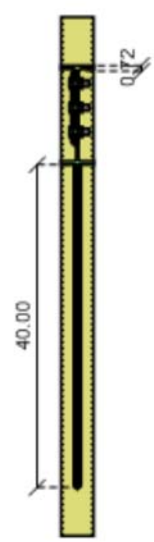
X-5 Top Detail: 2D Views

Vertical Brace Diagonal Connection

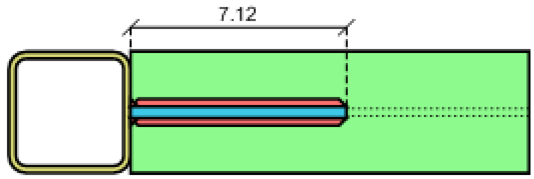
Side view



Front view



Bottom view



X-5 Top Detail: Summary Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x40.00x7.12	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.00 kips	*User Input Shear Load*
Beam Axial Load	5.00 kips	*User Input Beam Axial Force*
Column Force	5.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*
Bottom Brace Axial	20.10 kips	*User Input Bottom Brace Axial Force (compression)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Connection	Controlling Limit State	Max Unity Check	Result
Beam/Column connection	HSS Transverse Plastification	**0.28**	**PASS**
Bottom Gusset/Beam connection	Plate Axial Yield	**0.05**	**PASS**
Bottom Gusset/Column connection	Column Weld Strength	**0.05**	**PASS**
Bottom Gusset/Brace connection	Gusset Plate Compression (Whitmore)	**0.48**	**PASS**

X-5 Top Detail: Beam/Column Report

Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x40.00x7.12	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Total Shear Load	-2.62 kips	*Calculated Shear Load*
Total Axial Load	6.50 kips	*Calculated Axial Load (compression)*
Column Force	5.00 kips	*User Input Column Force*
Column Moment	0.00 kips-ft	*User Input Column Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt} (J3.3)*		
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t) (J3.5a)*		
S_{max}	3.00 in	*Max bolt spacing*		
t	0.26 in	*Thickness of governing element (Beam)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow} (J3.4)*		
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t) (J3.5)*		
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.25 in	*Weld size*		
D_{max}	0.31 in	*Max Size Allowed*		
t	0.38 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		

t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	9.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.25 in		*Weld size*		
L_{max}	9.00 in		*Max weld segment length*		
Beam Shear Yield		2.62 kips	95.94 kips	**0.03**	**PASS**
R_n = 0.6 *F_y*A_{gv}*C_v		**φ = 1.00**	*(G2-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.20 in^2		*Gross area subject to shear*		
C_v	1.00		*Web shear coefficient (G2-2)*		
φR_n	95.94 kips		*Shear yield strength*		
Plate Shear Yield		2.62 kips	72.90 kips	**0.04**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	3.38 in^2		*Gross area subject to shear*		
φR_n	72.90 kips		*Shear yield strength*		
Beam Shear Rupture		2.62 kips	73.58 kips	**0.04**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.52 in^2		*Net area subject to shear*		
φR_n	73.58 kips		*Shear rupture strength*		
Plate Shear Rupture at Beam		2.62 kips	62.40 kips	**0.04**	**PASS**
R_n = 0.6 *F_u*A_{nv}		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.39 in^2		*Net area subject to shear*		
φR_n	62.40 kips		*Shear rupture strength*		
Beam Axial Yield		6.50 kips	291.60 kips	**0.02**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	50.00 ksi		*Minimum yield stress of material*		
A_g	6.48 in^2		*Gross area subject to tension*		
φR_n	291.60 kips		*Tensile yield strength*		
Plate Axial Yield		6.50 kips	109.35 kips	**0.06**	**PASS**
R_n = F_y*A_g		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area subject to tension*		
φR_n	109.35 kips		*Tensile yield strength*		
Beam Block Shear		2.62 kips	90.27 kips	**0.03**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	3.41 in^2		*Gross area subject to shear*		
A_{nv}	2.84 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.28 in^2		*Net area subject to tension*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
F_y	50.00 ksi		*Minimum yield stress of material*		
φR_n	90.27 kips		*Block shear strength*		
Plate Block Shear at Beam		2.62 kips	79.21 kips	**0.03**	**PASS**
R_n = [min(0.6*F_u*A_{nv}, 0.6*F_y*A_{gv}) + U_{bs}*F_u*A_{nt}]		**φ = 0.75**	*(J4-5)*		
A_{gv}	2.81 in^2		*Gross area subject to shear*		

A_{nv}	1.99 in^2		*Net area subject to shear*		
U_{bs}	1.00		*Uniform tension stress factor*		
A_{nt}	0.77 in^2		*Net area subject to tension*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
F_y	36.00 ksi		*Minimum yield stress of material*		
φR_n	79.21 kips		*Block shear strength*		
Compression Buckling of the Plate		6.50 kips	109.35 kips	**0.06**	**PASS**
$R_n = F_y * A_g$		**φ = 0.9**	*(J4-6)*		
K	1.00		*Effective length factor*		
L	1.50 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	13.84		*Plate slenderness*		
F_y	36.00 ksi		*Capacity = Minimum Yield stress for KL/r <= 25*		
A_g	3.38 in^2		*Gross area subject to compression*		
φR_n	109.35 kips		*Compressive strength*		
Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	6.50 kips		*Calculated axial load*		
V_r	-2.62 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	3.38 in^2		*Gross area of the plate*		
Z_{pl}	7.59 in^3		*Plastic modulus of the shear plate*		
P_c	109.35 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	72.90 kips		*Available shear strength (see check 'Shear Yield')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.93 in		*Vertical eccentricity*		
M_r	0.17 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.50 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	6.50 kips		*Calculated axial load*		
V_r	-2.62 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.39 in^2		*Net area of the plate*		
Z_{net}	5.55 in^3		*Plastic modulus of net section*		
V_c	62.40 kips		*Available shear strength (see check 'Shear Rupture')*		
ex	1.50 in		*Horizontal eccentricity*		
ey	0.93 in		*Vertical eccentricity*		
M_r	0.17 kips-ft		*Moment due to eccentricity = V_r*ex + P_r*ey*		
M_c	20.13 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Plate Flexural Buckling				**0.09**	**PASS**
$P / (P_n*\varphi) + V / (V_n*\varphi) <= 1.0$		**φ = 0.90**	*(AISC 14th Edition)*		
P	6.50 kips		*Calculated axial load*		
V	2.62 kips		*Calculated shear load*		
L	1.50 in		*Length of connecting element (distance between the applied load and resisting element)*		
r	0.11 in		*Radius of gyration of the plate*		
KL/r	13.84		*Slenderness ratio*		

F_e	1494.08 ksi		*Elastic critical buckling stress, per eqn E3-4, $F_e = (\pi^2 * E)/(KL/r)^2$*		
F_y	36.00 ksi		*Minimum yield stress of material*		
F_{cr_Comp}	35.64 ksi		*Compression stress = F_y when KL/r <= 25, per J4.4*		
A_g	3.38 in^2		*Gross area of the plate*		
λ	0.33		*Buckling factor (pg 9.9) (eqn 9-18)*		
Q	1.00		*Buckling factor (eqn 9-15 through 9-17)*		
F_{cr_Flex}	36.00 ksi		*Critical stress, per eqn 9-14, Fcr = Fy * Q*		
S_{net}	3.74 in^3		*Section modulus of net section*		
a	1.50 in		*Design eccentricity*		
P_n	121.50 kips		*Compressive capacity, per eqn J4-1, Pn = Fy * Ag*		
V_n	89.67 kips		*Plate flexural buckling, per eqn 9-6, Vn = (Fcr_Flex * Snet) /a*		
UC	0.09		*Unity check per interaction equation, P/(Pn*Φ) + V/(Vn*Φ) <= 1*		
Bolt Bearing on Beam		7.01 kips	53.68 kips	**0.13**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		φ **= 0.75**	*(J3-6b)*		
V	-2.62 kips		*Applied shear force*		
P	6.50 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	7.01 kips		*Resultant shear force*		
Θ	21.97 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		
F_u	65.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	4.08 in		*Vertical edge spacing*		
e_h	1.50 in		*Horizontal edge spacing*		
L_{c_boltA}	1.21 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	1.21 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength $R_{n-bolt}=F_{nv}*A_{bolt}$*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		
Bolt Bearing on Plate at Beam		7.01 kips	53.68 kips	**0.13**	**PASS**
$R_n = 1*R_{n\_boltA} + 2*R_{n\_boltB}$		φ **= 0.75**	*(J3-6b)*		
V	-2.62 kips		*Applied shear force*		
P	6.50 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	7.01 kips		*Resultant shear force*		
Θ	21.97 degrees		*Angle between the resultant shear force and horizontal*		
d_b	0.75 in		*Bolt diameter*		
d_v	0.81 in		*Slotted hole vertical dimension*		
d_h	0.81 in		*Slotted hole horizontal dimension*		
d_c	0.41 in		*Distance from center of bolt to the edge of the hole*		

F_u	58.00 ksi		*Minimum tensile stress of material*		
s_v	3.00 in		*Vertical bolt spacing*		
s_h	0.00 in		*Horizontal bolt spacing*		
e_v	1.50 in		*Vertical edge spacing*		
e_h	2.50 in		*Horizontal edge spacing*		
L_{c_boltA}	2.29 in		*Minimum clear distance for the corner edge bolt: Lc_boltA = min((e_v /sin(Θ), (e_h /cos(Θ)) - d_c*		
L_{c_boltB}	2.29 in		*Minimum clear distance for the side edge bolts: Lc_boltB = min((s_v - 0.5 * d_v /sin(Θ)), (e_h /cos(Θ)) - d_c*		
R_{n_boltA}	23.86 kips		*Available bearing strength for the corner edge bolt: Rn_boltA = min[(1.5* Lc_boltA * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n_boltB}	23.86 kips		*Available bearing strength for each side edge bolt: Rn_boltB = min[(1.5* Lc_boltB * t * Fu), (3.0*d_b * t * Fu), R_{n-bolt}]*		
R_{n-bolt}	23.86 kips		*Bolt shear strength R_{n-bolt}=F_{nv}*A_{bolt}*		
F_{nv}	54.00 ksi		*Nominal shear stress of bolt*		
φR_n	53.68 kips		*Total bolt bearing strength*		

Bolt Shear at Beam		7.01 kips	38.15 kips	**0.18**	**PASS**
R_n = F_{nv}*A_b*N_{bolt}*C		**φ = 0.75**	*(J3-1)*		
V	-2.62 kips		*Applied shear force*		
P	6.50 kips		*Applied axial force*		
$R=(V^2 + P^2)^{0.5}$	7.01 kips		*Resultant force in bolts*		
F_{nv}	54.00 ksi		*Shear stress N type*		
A_b	0.44 in^2		*Area of bolt*		
N_{bolt}	3		*Number of bolts*		
C	0.71		*Eccentricity coefficient*		
φR_n	38.15 kips		*Bolt shear rupture strength*		

Bolt Group Eccentricity			***0.71***		
Elastic method			*(AISC 14th p.7-6)*		
C	0.71		*Coefficient (2.1323 / 3)*		
N_{rows}	1		*Number of rows of bolts*		
N_{cols}	3		*Number of bolts per row*		
D_x	0.00 in		*Horizontal bolt spacing*		
D_y	3.00 in		*Vertical bolt spacing*		
E_x	0.00 in		*Horizontal eccentricity*		
E_y	0.93 in		*Vertical eccentricity*		
Ang	21.97		*Angle of force in degrees, relative X axis*		

Weld at Column		7.01 kips	88.18 kips	**0.08**	**PASS**
φRn = 2 * C_1 * α * 1.392 * D_{16} * L					
Double Fillet					
1.392 = φ * 0.6 * F_{E70} * $2^{0.5}$/2 * 1/16, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	9.00 in		*Weld length*		
φRn	88.18 kips		*Weld strength*		

HSS Transverse Plastification		6.50 kips	22.89 kips	**0.28**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	9.00 in		*Plate length*		

B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	22.89 kips		*Transverse plastification*		

HSS Flexural Plastification		0.00 kips-ft	10.05 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		φ = 1.0	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	9.00 in		*Depth of plate*		
η	2.25		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
ex	0.00 in		*Horizontal eccentricity*		
ey	0.00 in		*Vertical eccentricity*		
M_{req}	0.00 kips-ft		*Required flexural plastification = V*ex + P*ey*		
φM_n	10.05 kips-ft		*Flexural plastification*		

X-5 Top Detail: Bot Gusset/Beam Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x40.00x7.12	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	2.68 kips	*Calculated Shear Load*
Axial Load	4.62 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Beam Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	7.12 in		*Min weld segment length*		
Plate Shear Yield		2.68 kips	57.65 kips	**0.05**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		φ = 1.00	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	2.67 in^2		*Gross area subject to shear*		
φR_n	57.65 kips		*Shear yield strength*		
Plate Shear Rupture		2.68 kips	69.65 kips	**0.04**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		φ = 0.75	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	2.67 in^2		*Net area subject to shear*		

φR_n	69.65 kips		*Shear rupture strength*		

Plate Axial Yield		4.62 kips	86.47 kips	**0.05**	PASS
$R_n = F_y*A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.67 in^2		*Gross area subject to tension*		
φR_n	86.47 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	PASS
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	4.62 kips		*Calculated axial load*		
V_r	2.68 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	2.67 in^2		*Gross area of the plate*		
Z_{pl}	4.75 in^3		*Plastic modulus of the shear plate*		
P_c	86.47 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	57.65 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	12.82 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	PASS
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	2.68 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	2.67 in^2		*Net area of the plate*		
Z_{net}	4.75 in^3		*Plastic modulus of net section*		
V_c	69.65 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	17.21 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, φ=0.75*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Beam Weld Strength		2.68 kips	55.78 kips	**0.05**	PASS
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	7.12 in		*Weld length*		
φRn	55.78 kips		*Weld strength*		

Beam Web Yielding		4.62 kips	139.64 kips	**0.03**	PASS
$R_n = (5 * k + N) * F_y * t_w$		**φ = 1.00**	*(J10-2)*		
k	0.72 in		*Distance from outer face of the flange to the web toe of the fillet*		
N	7.12 in		*Length of bearing*		
F_y	50.00 ksi		*Minimum yield stress of beam*		
t_w	0.26 in		*Beam web thickness*		

φR_n	139.64 kips	*Beam web local yielding*

X-5 Top Detail: Bot Gusset/Col Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x40.00x7.12	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Shear Load	15.04 kips	*Calculated Shear Load*
Axial Load	1.50 kips	*Calculated Axial Load (compression)*
Moment Load	0.00 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
HSS Punching Shear					**PASS**
Check Column Slenderness	**Pass**		*(K1.3)*		
E	29000.00 ksi		*Modulus of elasticity*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
B	4.00 in		*Column face width*		
(B - 3 * t) / t	14.17		*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15		*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**		*(K1.3)*		
B / t	17.17		*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00		*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**		*(K1.3)*		
F_y	46.00 ksi		*Column yield strength*		
F_{y-max}	52.00 ksi		*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**		*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi		*Column yield strength*		
F_u	58.00 ksi		*Column tensile strength*		
Check Punching Shear	**Pass**		*(Eqn K1-3)*		
F_{yp}	36.00 ksi		*Plate yield strength*		
t_p	0.38 in		*Plate thickness*		
t_{p-max}	0.38 in		*Maximum allowed plate thickness*		
Column Weld Limitations					**PASS**
Weld Min Size, Length			*(J2.2b)*		
Check Weld Min Size	**Pass**				
D	0.25 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.23 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.25 in		*Weld size*		
L_{min}	40.00 in		*Min weld segment length*		
Plate Shear Yield		15.04 kips	324.00 kips	**0.05**	**PASS**
R_n = 0.6 *F_y*A_{gv}		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	15.00 in^2		*Gross area subject to shear*		
φR_n	324.00 kips		*Shear yield strength*		

Plate Shear Rupture		15.04 kips	391.50 kips	**0.04**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	15.00 in^2		*Net area subject to shear*		
φR_n	391.50 kips		*Shear rupture strength*		

Plate Axial Yield		1.50 kips	486.00 kips	**0.00**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	15.00 in^2		*Gross area subject to tension*		
φR_n	486.00 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.00**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	1.50 kips		*Calculated axial load*		
V_r	15.04 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	15.00 in^2		*Gross area of the plate*		
Z_{pl}	150.00 in^3		*Plastic modulus of the shear plate*		
P_c	486.00 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	324.00 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	405.00 kips-ft		*Available moment $M_c = \varphi * (F_y * Z)$, $\varphi = 0.90$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	15.04 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	15.00 in^2		*Net area of the plate*		
Z_{net}	150.00 in^3		*Plastic modulus of net section*		
V_c	391.50 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.00 kips-ft		*Calculated moment*		
M_c	543.75 kips-ft		*Available moment $M_c = \varphi * (F_u * Znet)$, $\varphi = 0.75$*		
UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		

Column Weld Strength		15.04 kips	313.54 kips	**0.05**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16} * L$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, $\varphi = 0.75$ (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	0.88		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	4.00		*Weld fillet size in sixteenths of an inch*		
L	40.00 in		*Weld length*		
φRn	313.54 kips		*Weld strength*		

HSS Transverse Plastification		1.50 kips	65.61 kips	**0.02**	**PASS**
$R_n = F_y * t^2 / (1 - t_p/B) * (2 l_b/B + 4 * Qf * (1 - t_p/B)^{0.5})$		**$\varphi = 1.00$**	*(K1-12)*		

F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	40.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	65.61 kips		*Transverse plastification*		
HSS Flexural Plastification		0.00 kips-ft	125.80 kips-ft	**0.00**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		$\varphi = 1.0$	*(K3-6)*		
B_b	0.88 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.22		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	40.00 in		*Depth of plate*		
η	10.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		
M_{req}	0.00 kips-ft		*Required flexural plastification*		
φM_n	125.80 kips-ft		*Flexural plastification*		

X-5 Top Detail: Bot Gusset/Brace Report

LRFD
Vertical Brace Diagonal Connection

Material Properties:

Beam	W12x22	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Plate	P0.38x4.00x9.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Brace	L5x3.5x8	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Bottom Gusset	P0.38x40.00x7.12	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	20.10 kips	*Brace Axial (compression)*
Brace Moment	1.83 kips-ft	*Brace Moment (not used for design)*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State		Required	Available	Unity Check	Result
Brace Weld Limitations					**PASS**
Weld Max/Min Size, Length			*(J2.2b)*		
Check Weld Max Size	**Pass**				
D	0.38 in		*Weld size*		
D_{max}	0.44 in		*Max Size Allowed*		
t	0.50 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.38 in		*Weld size*		
D_{min}	0.19 in		*Min size allowed per Table J2.4*		
t_{min}	0.38 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.38 in		*Weld size*		
L_{min}	3.49 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.38 in		*Weld size*		
L_{max}	21.04 in		*Max weld segment length*		
Gusset Plate Compression (Whitmore)		20.10 kips	41.95 kips	**0.48**	**PASS**

$P_n = F_{cr}*A_g$		$\varphi = 0.9$	*(E3-1)*		
K	0.50		*Effective length factor*		
L	18.16 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	83.87		*Plate slenderness*		
F_{cr}	24.86 ksi		*Flexural buckling stress (E3-2)*		
A_g	1.88 in^2		*Gross area of plate (Whitmore section)*		
φP_n	41.95 kips		*Gusset plate compressive strength*		

Brace Weld Strength		20.10 kips	246.61 kips	**0.08**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	6.00		*Weld fillet size in sixteenths of an inch*		
L	29.53 in		*Weld length*		
φRn	246.61 kips		*Weld strength*		

X-5 Top Detail: Members Report

Vertical Brace Diagonal Connection

Beam		***W12x22***
Material		
Name	A992	*Material name*
F_y	50.00 ksi	*Minimum yield stress of material*
F_u	65.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b_f	4.03 in	*Flange width*
d	12.30 in	*Overall depth*
t_w	0.26 in	*Web thickness*
t_f	0.42 in	*Flange thickness*
a	6.48 in^2	*Area*
k_{des}	0.72 in	*Kdes*
k_{det}	0.94 in	*Kdet*
k_1	0.63 in	*K1*
Web Hole Type		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Column		***HSS4x4x4***
Material		
Name	A500 Gr.B Rect	*Material name*
F_y	46.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Depth*
b	4.00 in	*Width*
a	3.37 in^2	*Area*

t_{des}	0.23 in	*Wall Thickness*

Bottom Brace		***L5x3.5x8***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
b	3.50 in	*Flange width*
d	5.00 in	*Overall depth*
a	4.00 in^2	*Area*
t_{f1}	0.50 in	*Flange thickness*
t_{f2}	0.50 in	*Flange thickness*
k_{des}	0.94 in	*Kdes*
k_{det}	0.94 in	*Kdet*

X-5 Top Detail: Components Report

Vertical Brace Diagonal Connection

Plate		***P0.38x4.00x9.00***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	4.00 in	*Width*
t	0.38 in	*Thickness*
Hole		
Hole type	Standard	
D_x	0.81 in	*Hole width*
D_y	0.81 in	*Hole height*
R	1	*Number of rows of holes*
C	3	*Number of holes per row*
R_s	3.00 in	*Row Spacing*
C_s	3.00 in	*Column Spacing*

Bottom Gusset		***P0.38x40.00x7.12***
Material		
Name	A36	*Material name*
F_y	36.00 ksi	*Minimum yield stress of material*
F_u	58.00 ksi	*Minimum tensile stress of material*
E	29000.00 ksi	*Modulus of elasticity*
Member Properties		
d	40.00 in	*Width*
t	0.38 in	*Thickness*
Clip		
H_clip	6.62 in	*Horiz. Clip*
V_clip	1.41 in	*Vert. Clip*

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Beam Bolts		***3/4" A325***
Bolt Properties		
Type	A325	

d	0.75 in	*Diameter*
Strength		
S_n	54.00 ksi	*Shear strength (N-threads included in shear plane)*
T	90.00 ksi	*Tensile strength*

Brace Gusset Weld		***E70***
Weld Properties		
Type	Single Fillet	
Fillet Size	0.38 in	

Beam Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

Column Weld		***E70***
Weld Properties		
Type	Double Fillet	
Fillet Size	0.25 in	

RISAConnection version 7.0.1

02/07/2018

Global Parameters - Description:

Project Title	Bracing Connection Details
Company	Blackwell
Designer	D.M.V
Job Number	170266
Notes	

Global Parameters - Solution:

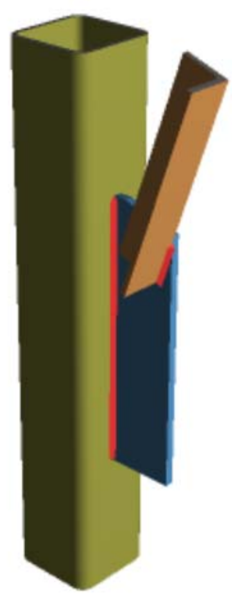
Design Method	AISC 14th (360-10): LRFD
Bolt Group Analysis Method	Elastic
Weld Analysis Method	Elastic
Consider Bolt Hole Deformation?	No
Check Weld Filler Material Matching?	Yes
Check Rotational Ductility?	Yes
Full Shear Eccentricity Considered?	No
Plastic Panel-Zone Shear Deformation Considered?	No

Project Explorer Summary:

X-7 Bottom Connection	**PASS(UC-0.4)**
X-7 Top Connection	**PASS(UC-0.4)**

X-7 Bottom Connection: 3D View

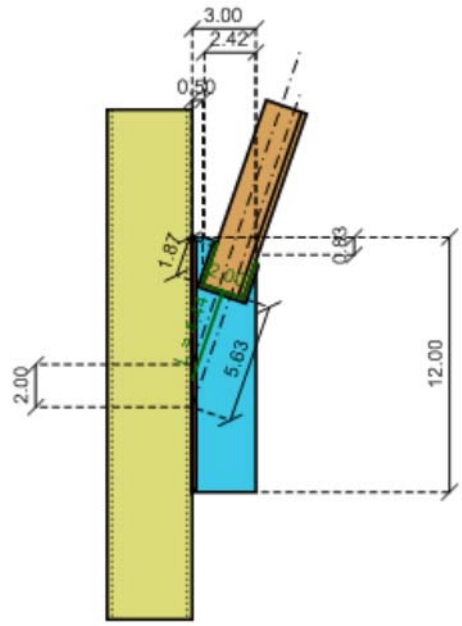
Knee Brace Connection



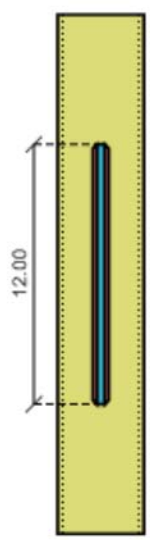
X-7 Bottom Connection: 2D Views

Knee Brace Connection

Left view

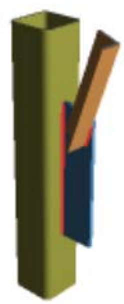


Front view



X-7 Bottom Connection: LRFD Results Report

LRFD
Knee Brace Connection



Material Properties:

Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x3.00x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Brace Axial	8.50 kips	*Brace Axial (compression)*
Shear Load	8.04 kips	*Calculated Shear Load*
Axial Load	2.77 kips	*Calculated Axial Load (compression)*
Moment Load	0.46 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
F_{y-max}	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
t_{p-max}	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	2.00 in	*Vertical brace workpoint offset*		
d_{gusset}	12.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		
t_{min}	0.23 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	12.00 in	*Min weld segment length*		
Brace Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			

D	0.19 in		*Weld size*		
D_{max}	0.25 in		*Max Size Allowed*		
t	0.25 in		*Min shelf dimension*		
Check Weld Min Size	**Pass**				
D	0.19 in		*Weld size*		
D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.25 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	1.87 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	2.13 in		*Max weld segment length*		

Plate Shear Yield		8.04 kips	97.20 kips	**0.08**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**φ = 1.00**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	4.50 in^2		*Gross area subject to shear*		
φRn	97.20 kips		*Shear yield strength*		

Plate Shear Rupture		8.04 kips	117.45 kips	**0.07**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**φ = 0.75**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	4.50 in^2		*Net area subject to shear*		
φRn	117.45 kips		*Shear rupture strength*		

Plate Axial Yield		2.77 kips	145.80 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		**φ = 0.90**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.50 in^2		*Gross area subject to tension*		
φRn	145.80 kips		*Tensile yield strength*		

Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	2.77 kips		*Calculated axial load*		
V_r	8.04 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.50 in^2		*Gross area of the plate*		
Z_{pl}	13.50 in^3		*Plastic modulus of the shear plate*		
P_c	145.80 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	97.20 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	0.46 kips-ft		*Calculated moment*		
M_c	36.45 kips-ft		*Available moment $M_c = \varphi*(F_y * Z)$, φ=0.90*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		

Plate Flexural Rupture				**0.00**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	8.04 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	4.50 in^2		*Net area of the plate*		
Z_{net}	13.50 in^3		*Plastic modulus of net section*		
V_c	117.45 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	0.46 kips-ft		*Calculated moment*		
M_c	48.94 kips-ft		*Available moment $M_c = \varphi*(F_u * Znet)$, φ=0.75*		

UC	0.00		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Column Weld Strength		0.71 kips/in	6.68 kips/in	**0.11**	**PASS**
$\varphi Rn = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					
Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
V	8.04 kips		*Shear Load*		
P	2.77 kips		*Axial Load*		
M	0.46 kips-ft		*Moment*		
e_{eff}	0.69 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	0.71 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		
Gusset Plate Compression (Whitmore)		8.50 kips	21.39 kips	**0.40**	**PASS**
$P_n = F_{cr}*A_g$		**φ = 0.9**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	4.44 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	49.22		*Plate slenderness*		
F_{cr}	31.69 ksi		*Flexural buckling stress (E3-2)*		
A_g	0.75 in^2		*Gross area of plate (Whitmore section)*		
φP_n	21.39 kips		*Gusset plate compressive strength*		
Brace Weld Strength		8.50 kips	25.04 kips	**0.34**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	6.00 in		*Weld length*		
φRn	25.04 kips		*Weld strength*		
HSS Transverse Plastification		2.77 kips	27.03 kips	**0.10**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	12.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φR_n	27.03 kips		*Transverse plastification*		
HSS Flexural Plastification		0.46 kips-ft	15.18 kips-ft	**0.03**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		

F_y	46.00 ksi	*Column yield strength*
t	0.23 in	*Column wall thickness*
H_b	12.00 in	*Depth of plate*
η	3.00	*Load length parameter (H_b / B)*
Q_f	1.00	*User input column stress interaction parameter*
M_{req}	0.46 kips-ft	*Required flexural plastification*
φM_n	15.18 kips-ft	*Flexural plastification*

X-7 Bottom Connection: Members Report

Knee Brace Connection

	Column		***HSS4x4x4***
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	4.00 in	*Depth*
	b	4.00 in	*Width*
	a	3.37 in^2	*Area*
	t_{des}	0.23 in	*Wall Thickness*

	Brace		***L2x2x4***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b	2.00 in	*Flange width*
	d	2.00 in	*Overall depth*
	a	0.94 in^2	*Area*
	t_{f1}	0.25 in	*Flange thickness*
	t_{f2}	0.25 in	*Flange thickness*
	k_{des}	0.50 in	*Kdes*
	k_{det}	0.50 in	*Kdet*

X-7 Bottom Connection: Components Report

Knee Brace Connection

	Gusset		***P0.38x3.00x12.00***
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.00 in	*Width*
	t	0.38 in	*Thickness*
Clip			
	H_clip	2.42 in	*Horiz. Clip*
	V_clip	0.83 in	*Vert. Clip*

	Column Weld		***E70***
Weld Properties			

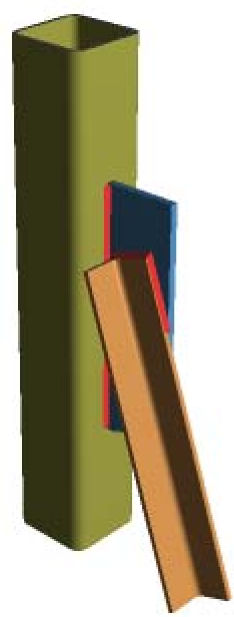
Type Double Fillet
Fillet Size 0.19 in

Brace Gusset Weld	***E70***

Weld Properties
Type Single Fillet
Fillet Size 0.19 in

X-7 Top Connection: 3D View

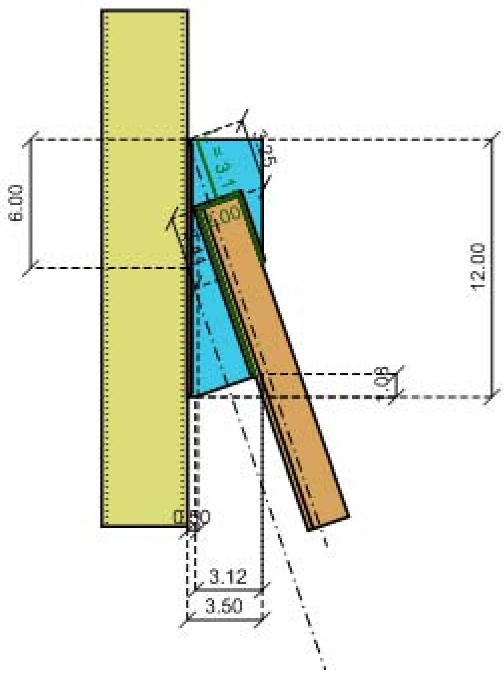
Knee Brace Connection



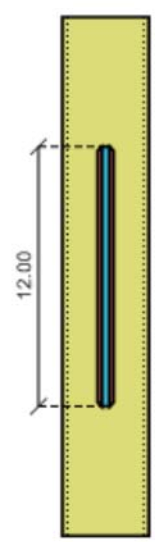
X-7 Top Connection: 2D Views

Knee Brace Connection

Left view



Front view



X-7 Top Connection: LRFD Results Report

LRFD
Knee Brace Connection

Material Properties:				
Column	HSS4x4x4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Brace	L2x2x4	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Gusset	P0.38x3.50x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:		
Brace Axial	8.50 kips	*Brace Axial (compression)*
Shear Load	-8.04 kips	*Calculated Shear Load*
Axial Load	2.77 kips	*Calculated Axial Load (compression)*
Moment Load	1.38 kips-ft	*Calculated Moment*

Note: Unless specified, all code references are from AISC 360-10

Governing LC: N/A

Limit State	Required	Available	Unity Check	Result
HSS Punching Shear				**PASS**
Check Column Slenderness	**Pass**	*(K1.3)*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	46.00 ksi	*Column yield strength*		
t	0.23 in	*Column wall thickness*		
B	4.00 in	*Column face width*		
(B - 3 * t) / t	14.17	*Column slenderness ratio for shear*		
$((B - 3 * t) / t)_{max}$	35.15	*Slender wall limit for shear (Table K1.2A)*		
Check Column Slenderness	**Pass**	*(K1.3)*		
B / t	17.17	*Column slenderness ratio for axial*		
$(B / t)_{max}$	40.00	*Slender wall limit for axial (Table K1.2A)*		
Check Column Material	**Pass**	*(K1.3)*		
F_y	46.00 ksi	*Column yield strength*		
$F_{y\text{-}max}$	52.00 ksi	*Column yield strength limit (Table K1.2A)*		
Check Column Ductility	**Pass**	*(Table K1.2A) Condition: F_y / F_u<= 0.8 or ASTM A500 Grade C*		
F_y	46.00 ksi	*Column yield strength*		
F_u	58.00 ksi	*Column tensile strength*		
Check Punching Shear	**Pass**	*(Eqn K1-3)*		
F_{yp}	36.00 ksi	*Plate yield strength*		
t_p	0.38 in	*Plate thickness*		
$t_{p\text{-}max}$	0.38 in	*Maximum allowed plate thickness*		
Geometry Restrictions at Column				**PASS**
Check Workpoint Vert. Offset	**Pass**	*Condition: 0 <=WP_v<= 0.5*d_{gusset}*		
WP_v	6.00 in	*Vertical brace workpoint offset*		
d_{gusset}	12.00 in	*Depth of gusset plate*		
Column Weld Limitations				**PASS**
Weld Min Size, Length		*(J2.2b)*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		
D_{min}	0.13 in	*Min size allowed per Table J2.4*		
t_{min}	0.23 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in	*Weld size*		
L_{min}	12.00 in	*Min weld segment length*		
Brace Weld Limitations				**PASS**
Weld Max/Min Size, Length		*(J2.2b)*		
Check Weld Max Size	**Pass**			
D	0.19 in	*Weld size*		
D_{max}	0.25 in	*Max Size Allowed*		
t	0.25 in	*Min shelf dimension*		
Check Weld Min Size	**Pass**			
D	0.19 in	*Weld size*		

D_{min}	0.13 in		*Min size allowed per Table J2.4*		
t_{min}	0.25 in		*Controlling member thickness*		
Check Weld Min Length	**Pass**		*Condition: L_{min} >= 4*D per J2.2b*		
D	0.19 in		*Weld size*		
L_{min}	2.00 in		*Min weld segment length*		
Check Weld Max Length	**Pass**		*Condition: L_{max} <= 100*D*		
D	0.19 in		*Weld size*		
L_{max}	8.21 in		*Max weld segment length*		
Plate Shear Yield		8.04 kips	97.20 kips	**0.08**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		**$\varphi = 1.00$**	*(J4-3)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_{gv}	4.50 in^2		*Gross area subject to shear*		
φR_n	97.20 kips		*Shear yield strength*		
Plate Shear Rupture		8.04 kips	117.45 kips	**0.07**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		**$\varphi = 0.75$**	*(J4-4)*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_{nv}	4.50 in^2		*Net area subject to shear*		
φR_n	117.45 kips		*Shear rupture strength*		
Plate Axial Yield		2.77 kips	145.80 kips	**0.02**	**PASS**
$R_n = F_y * A_g$		**$\varphi = 0.90$**	*(J4-1)*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.50 in^2		*Gross area subject to tension*		
φR_n	145.80 kips		*Tensile yield strength*		
Plate Flexural Yield				**0.01**	**PASS**
$(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$			*(AISC 14th Eq.10-5)*		
P_r	2.77 kips		*Calculated axial load*		
V_r	8.04 kips		*Calculated shear load*		
F_y	36.00 ksi		*Minimum yield stress of material*		
A_g	4.50 in^2		*Gross area of the plate*		
Z_{pl}	13.50 in^3		*Plastic modulus of the shear plate*		
P_c	145.80 kips		*Available tensile strength (see check 'Axial Yield')*		
V_c	97.20 kips		*Available shear strength (see check 'Shear Yield')*		
M_r	1.38 kips-ft		*Calculated moment*		
M_c	36.45 kips-ft		*Available moment $M_c=\varphi*(F_y* Z)$, $\varphi=0.90$*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (P_r/P_c + M_r/M_c)^2 <= 1$*		
Plate Flexural Rupture				**0.01**	**PASS**
$(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$			*(Eq.10-5)*		
P_r	0.00 kips		*Calculated axial load*		
V_r	8.04 kips		*Calculated shear load*		
F_u	58.00 ksi		*Minimum tensile stress of material*		
A_n	4.50 in^2		*Net area of the plate*		
Z_{net}	13.50 in^3		*Plastic modulus of net section*		
V_c	117.45 kips		*Available shear strength (see check 'Shear Rupture')*		
M_r	1.38 kips-ft		*Calculated moment*		
M_c	48.94 kips-ft		*Available moment $M_c= \varphi*(F_u* Znet)$, $\varphi=0.75$*		
UC	0.01		*Unity check per interaction equation, $(V_r/V_c)^2 + (M_r/M_c)^2 <= 1$*		
Column Weld Strength		0.96 kips/in	6.68 kips/in	**0.14**	**PASS**
$\varphi R_n = 2 * C_1 * \alpha * \beta * 1.392 * D_{16}$					

Double Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
V	8.04 kips		*Shear Load*		
P	2.77 kips		*Axial Load*		
M	1.38 kips-ft		*Moment*		
e_{eff}	2.07 in		*Effective eccentricity*		
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
β	0.80		*Force redistribution adjustment factor*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
r_u	0.96 kips/in		*Required weld stress per AISC 14th Eqn 8-11*		
φRn	6.68 kips/in		*Weld strength*		

Gusset Plate Compression (Whitmore)		8.50 kips	22.82 kips	**0.37**	**PASS**
$P_n = F_{cr}*A_g$		**φ = 0.9**	*(E3-1)*		
K	1.20		*Effective length factor*		
L	3.11 in		*Unbraced length*		
r	0.11 in		*Radius of gyration*		
KL/r	34.51		*Plate slenderness*		
F_{cr}	33.81 ksi		*Flexural buckling stress (E3-2)*		
A_g	0.75 in^2		*Gross area of plate (Whitmore section)*		
φP_n	22.82 kips		*Gusset plate compressive strength*		

Brace Weld Strength		8.50 kips	56.88 kips	**0.15**	**PASS**
$\varphi Rn = C_1 * \alpha * 1.392 * D_{16} * L$					
Single Fillet					
$1.392 = \varphi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, φ=0.75 (AISC 14th Eqn 8-2a)					
C_1	1.00		*Electrode strength coefficient (AISC 14th table 8-3)*		
α	1.00		*Base material proration factor (re-arrangement of AISC 14th Eqn 9-2)*		
D_{16}	3.00		*Weld fillet size in sixteenths of an inch*		
L	13.62 in		*Weld length*		
φRn	56.88 kips		*Weld strength*		

HSS Transverse Plastification		2.77 kips	27.03 kips	**0.10**	**PASS**
$R_n = F_y*t^2 /(1-t_p/B)*(2l_b/B + 4*Q_f*(1-t_p/B)^{0.5})$		**φ = 1.00**	*(K1-12)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
t_p	0.38 in		*Plate thickness*		
l_b	12.00 in		*Plate length*		
B	4.00 in		*Column width*		
Q_f	1.00		*User input column stress interaction parameter*		
φRn	27.03 kips		*Transverse plastification*		

HSS Flexural Plastification		1.38 kips-ft	15.18 kips-ft	**0.09**	**PASS**
$M_n = F_y * t^2 * H_b * (1 / (2 * \eta) + 2 / (1 - \beta)^{0.5} + \eta / (1 - \beta)) * Q_f$		**φ = 1.0**	*(K3-6)*		
B_b	0.75 in		*Plate bearing width*		
B	4.00 in		*Column width*		
β	0.19		*Width ratio (B_b / B)*		
F_y	46.00 ksi		*Column yield strength*		
t	0.23 in		*Column wall thickness*		
H_b	12.00 in		*Depth of plate*		
η	3.00		*Load length parameter (H_b / B)*		
Q_f	1.00		*User input column stress interaction parameter*		

M_{req}	1.38 kips-ft	*Required flexural plastification*
φM_n	15.18 kips-ft	*Flexural plastification*

X-7 Top Connection: Members Report

Knee Brace Connection

	Column		*HSS4x4x4*
Material			
	Name	A500 Gr.B Rect	*Material name*
	F_y	46.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	4.00 in	*Depth*
	b	4.00 in	*Width*
	a	3.37 in^2	*Area*
	t_{des}	0.23 in	*Wall Thickness*

	Brace		*L2x2x4*
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	b	2.00 in	*Flange width*
	d	2.00 in	*Overall depth*
	a	0.94 in^2	*Area*
	t_{f1}	0.25 in	*Flange thickness*
	t_{f2}	0.25 in	*Flange thickness*
	k_{des}	0.50 in	*Kdes*
	k_{det}	0.50 in	*Kdet*

X-7 Top Connection: Components Report

Knee Brace Connection

	Gusset		*P0.38x3.50x12.00*
Material			
	Name	A36	*Material name*
	F_y	36.00 ksi	*Minimum yield stress of material*
	F_u	58.00 ksi	*Minimum tensile stress of material*
	E	29000.00 ksi	*Modulus of elasticity*
Member Properties			
	d	3.50 in	*Width*
	t	0.38 in	*Thickness*
Clip			
	H_clip	3.12 in	*Horiz. Clip*
	V_clip	1.08 in	*Vert. Clip*

	Column Weld		*E70*
Weld Properties			
	Type	Double Fillet	
	Fillet Size	0.19 in	

	Brace Gusset Weld		*E70*
Weld Properties			
	Type	Single Fillet	

Fillet Size 0.19 in