STRUCTURAL CALCULATIONS

For

Meehan Cabin
POWDER MOUNTAIN
EDEN, UTAH

Prepared for

Lloyd Architects
573 E 600 S
Salt Lake City, Utah 84102

PLAN REVIEW ACCEPTANCE
FOR COMPLIANCE WITH THE APPLICABLE CONSTRUCTION CODES IDENTIFIED BELOW.

- [ ] BUILDING
- [x] 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: 04/18/18

WEST COAST CODE CONSULTANTS, INC.

PROFESSIONAL STRUCTURAL ENGINEER
2.27.18
SHAUN A. PACKER
191304
STATE OF UTAH

By

CALDER RICHARDS
CONSULTING ENGINEERS
634 S. 400 W, Suite 100
Salt Lake City, UT 84101
801-466-1699

CALDER RICHARDS CONSULTING ENGINEERS	JOB TITLE: Meehan Cabin	BY: DSM	DATE: 10/4/2017
	SUBJECT: Load Sheet	CHECKED:	SHEET:

CODE: 2015 INTERNATIONAL BUILDING CODE

RISK CATEGORY: II

ROOF LOADS

DEAD LOAD:

Roofing	= 6.0 psf
Sheathing	= 3.6 psf
Framing	= 7.0 psf
Insulation	= 0.3 psf
Mech./Misc	= 3.1 psf

	= 20.0 psf

SNOW LOAD:

Pg	= 275 psf
Is	= 1.0
Ce	= 1.0
Ct	= 1.0 - Typical
	= 1.2 - Exterior Decks
Pf	= 195 psf - Typical
Pf	= 234 psf - Exterior Decks

ROOF LIVE LOAD:

RLL= 20 psf

FLOOR LOADS

DEAD LOAD:

Wood Floor + Wood Sleepers +3" Topping

Flooring	= 1.0 psf
3" NW Conc.	= 37.5 psf
Sheathing	= 2.3 psf
Framing	= 3.5 psf
Sleeper System	= 2.8 psf
Mech./Misc	= 2.9 psf

	= 50.0 psf

Steel Floor

Flooring	= 1.0 psf
W2 Deck	= 2.1 psf
5" NW Conc.	= 48.3 psf
Framing	= 6.0 psf
Ceiling	= 2.8 psf
Mech./Misc	= 4.8 psf

	= 65.0 psf

Steel Floor + Wood Sleepers + 3" Topping

Flooring	= 1.0 psf
W2 Deck	= 2.1 psf
5" NW Conc.	= 48.3 psf
Framing	= 6.0 psf
Sleeper System	= 6.0 psf
3" NW Conc.	= 37.5 psf
Mech./Misc	= 6.1 psf

	= 107.0 psf

LIVE LOAD:

Residential	= 40 psf
Deck	= 60 psf or snow load

CALDER RICHARDS
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JOB TITLE: Meehan Cabin **BY:** DSM **DATE:** 10/4/2017
SUBJECT: Load Sheet **CHECKED:** **SHEET:**

SEISMIC

Sds = 0.551
Sd1 = 0.183
Ss = 0.826
S1 = 0.274

Ie = 1.0
Seismic Design Category= D
Site Class = B

Plywood Shear Walls
R = 6.5
Cd = 4.0
Ωo = 3.0

IMF
R = 4.5
Cd = 4.0
Ωo = 3.0

Tension Braced Frames
R = 3.25
Cd = 3.25
Ωo = 2.0

WIND

V = 150 mph
Exposure Category = C

Note: Local code only requires 115 mph. A higher wind load was used due to the location of the structure on top of the ridge of a mountain.

SOILS

Allowable Soil Bearing Pressure = 5000 psf
Frost Depth = 24 in with bedrock
Soils Report per IGES, dated September 16, 2014, Project No. 01628-008

CALDER RICHARDS
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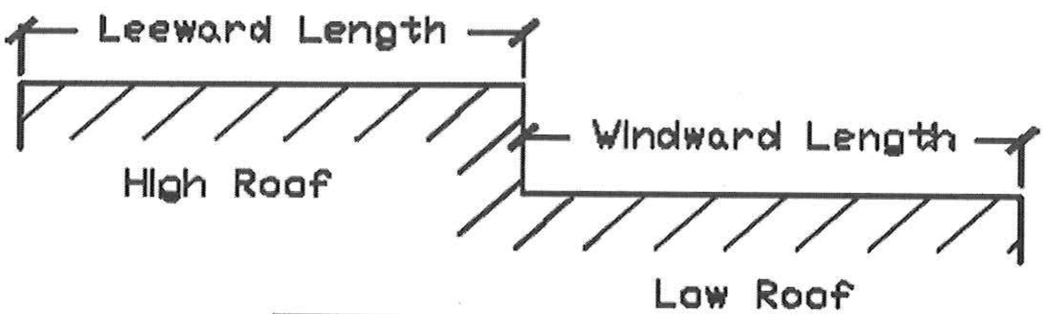
Snow Drift Analysis

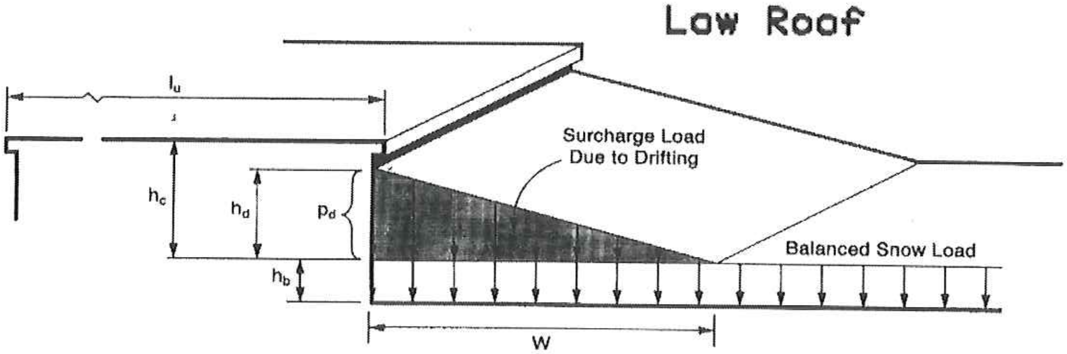
Design per ASCE 7-10 and IBC 2012

Project:	Meehan Residence
Date:	10/17/2017
Drift Location:	High Roof to Large Balcony

Ground Snow:	P_g:	275 PSF	Roof Change:	h_r:	11.00 FT
Flat Roof Snow:	P_f:	234 PSF	Max Drift Width:	W_{dmax}:	15.00 FT
Windward Length:	L_w:	4.00 FT	Leeward Length:	L_l:	31.00 FT
Snow density:	D:	30.00 PCF	Must drift be considered?		Yes
Snow base depth:	h_b:	7.80 FT	Effective leeward length:		31.00 FT
Max drift depth:	h_c:	3.20 FT	Effective windward length:		20.00 FT
Leeward drift:	h_{dl}:	4.05 FT	Drift depth:	h_d:	4.05 FT
Windward drift:	h_{dw}:	2.47 FT			

Actual drift depth:	h_d:	**3.20** FT	Max drift pressure:	p_d:	**96.0** PSF
Drift Width:	w:	**20.50** FT	Min drift pressure:	p_d @ w:	**25.77** PSF





CALDER RICHARDS
CONSULTING ENGINEERS

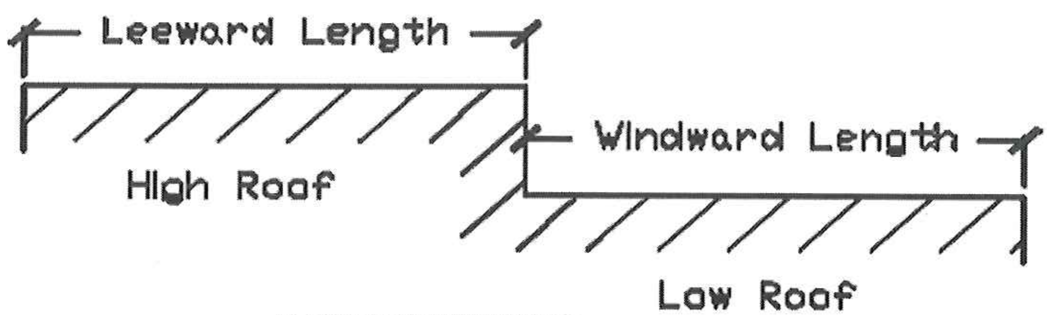
Snow Drift Analysis

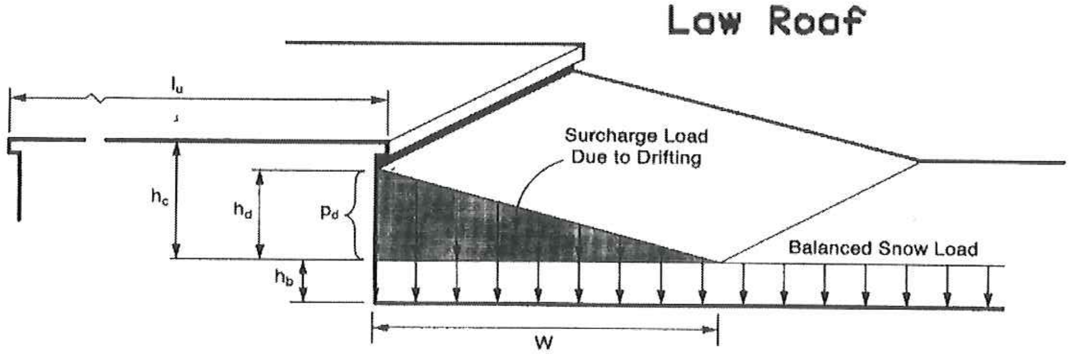
Design per ASCE 7-10 and IBC 2012

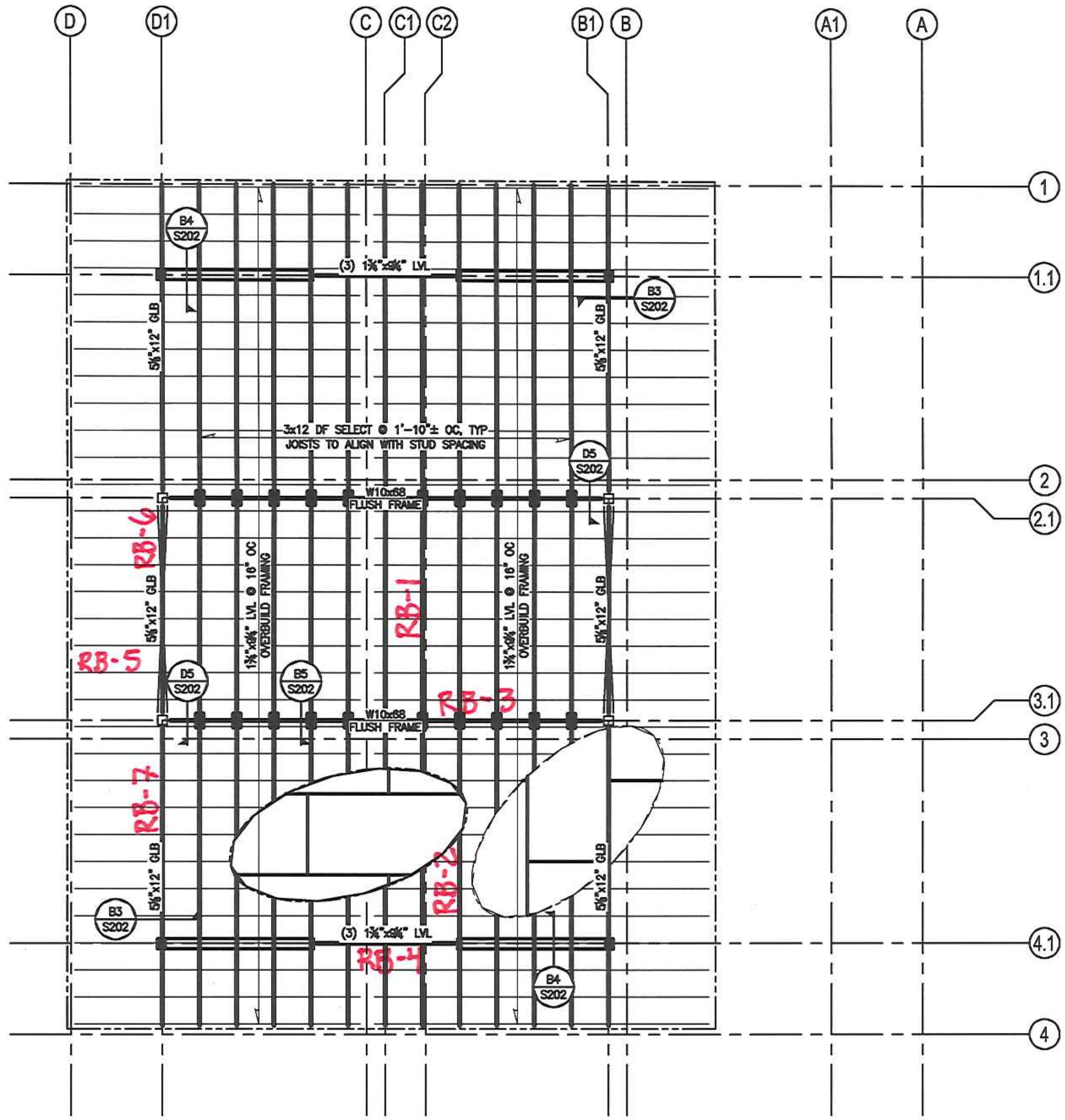
Project:	Meehan Residence
Date:	10/17/2017
Drift Location:	High Roof to Small Balcony

Ground Snow:	P_g:	275 PSF	Roof Change:	h_r:	11.00 FT
Flat Roof Snow:	P_f:	234 PSF	Max Drift Width:	W_{dmax}:	4.00 FT
Windward Length:	L_w:	4.00 FT	Leeward Length:	L_l:	0.00 FT
Snow density:	D:	30.00 PCF	Must drift be considered?		Yes
Snow base depth:	h_b:	7.80 FT	Effective leeward length:		0.00 FT
Max drift depth:	h_c:	3.20 FT	Effective windward length:		20.00 FT
Leeward drift:	h_{dl}:	0.00 FT	Drift depth:	h_d:	2.47 FT
Windward drift:	h_{dw}:	2.47 FT			

Actual drift depth:	h_d:	**2.47** FT	Max drift pressure:	p_d:	**74.2** PSF
Drift Width:	w:	**9.89** FT	Min drift pressure:	p_d @ w:	**44.15** PSF

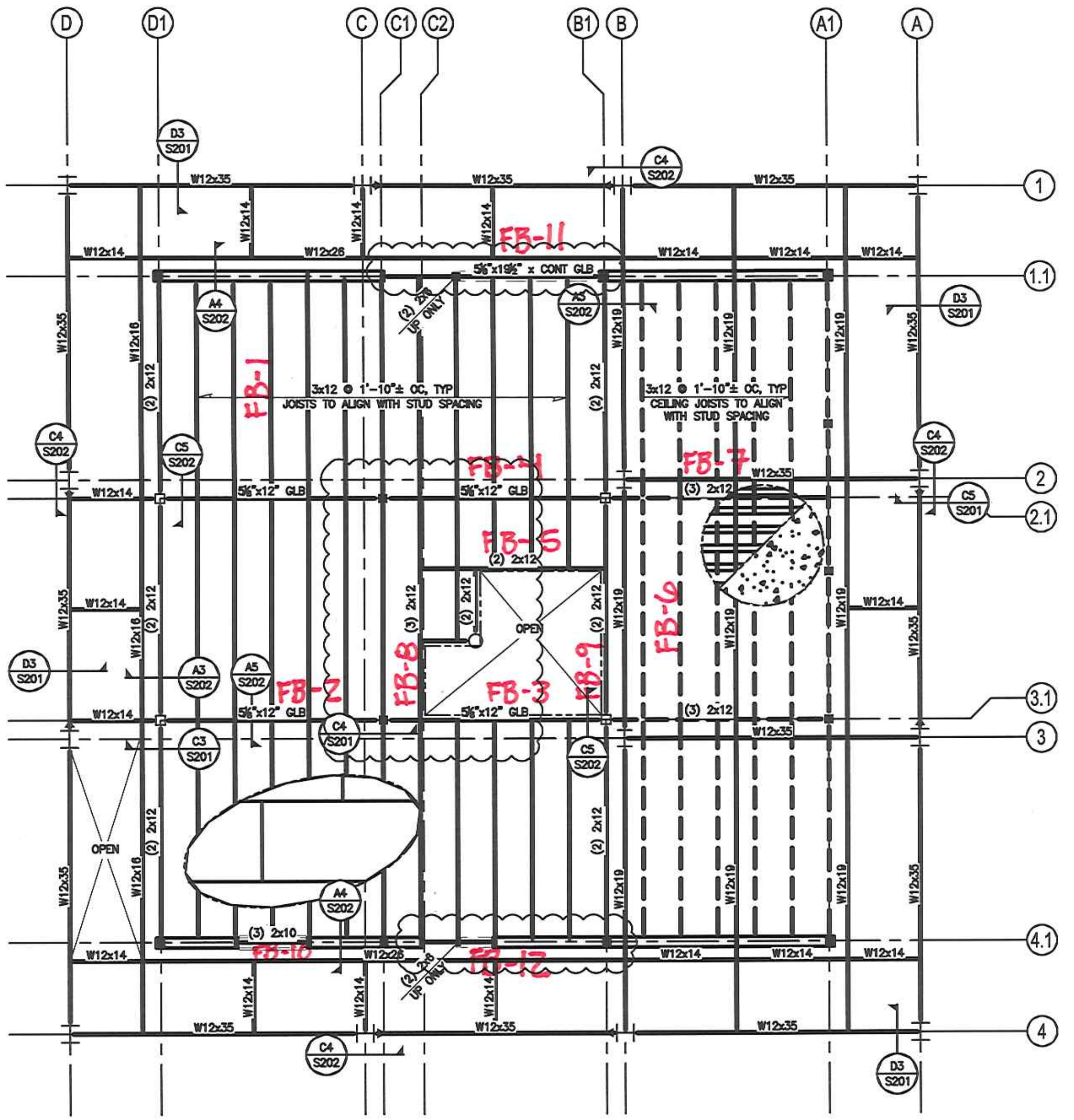






ROOF FRAMING PLAN

SCALE: 1/4" = 1'-0"



UPPER LEVEL FRAMING PLAN

SCALE: 1/4" = 1'-0"

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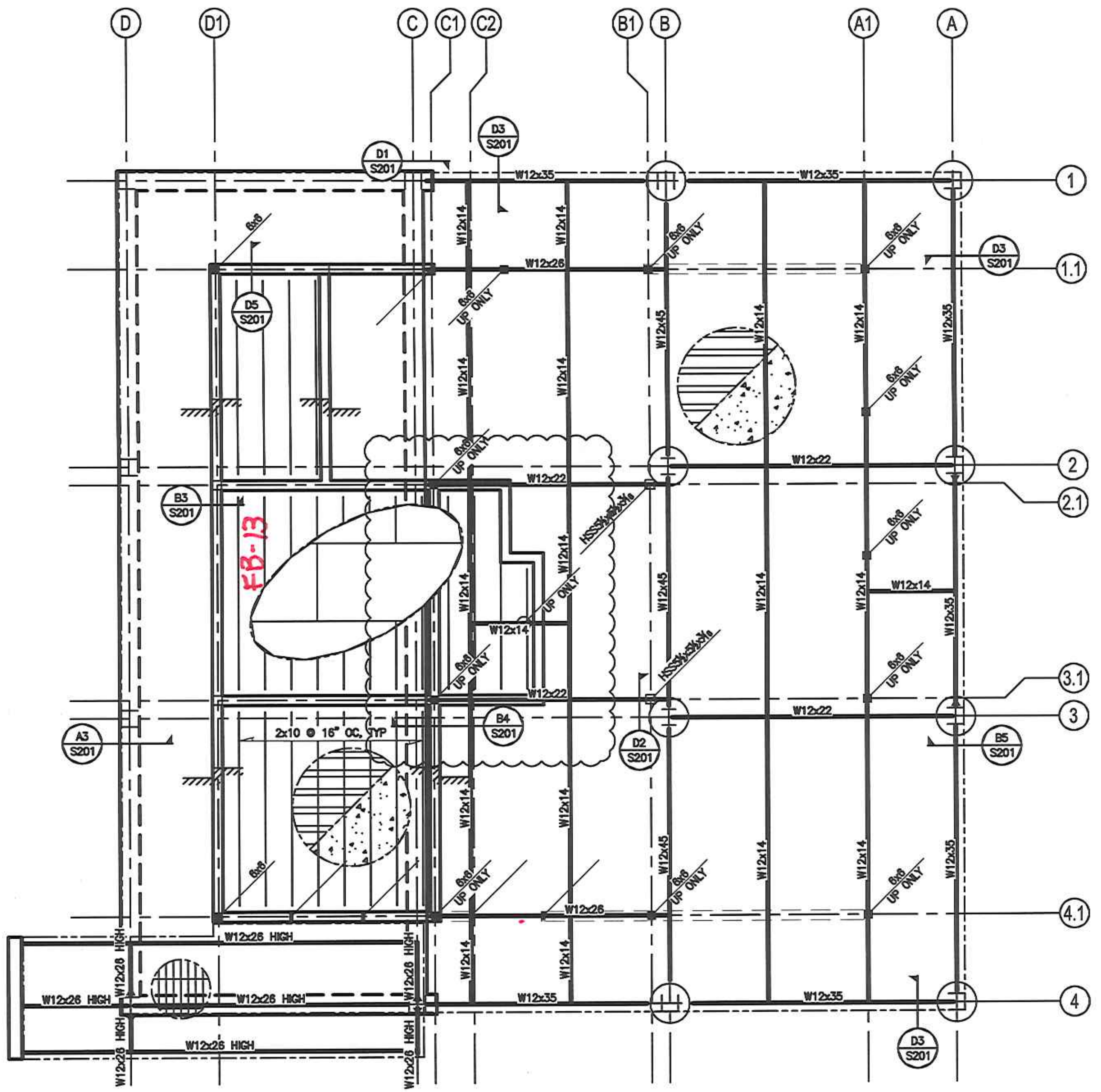
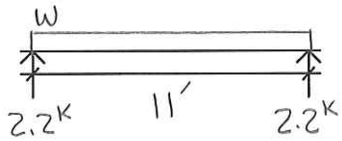
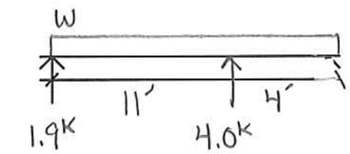
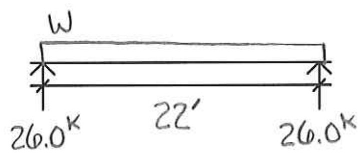
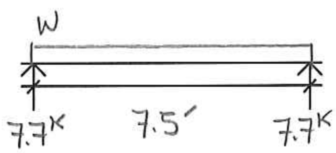


LOWER LEVEL FRAMING PLAN
SCALE: 1/4" = 1'-0"

CALDER RICHARDS CONSULTING ENGINEERS

JOB TITLE Meehan Cabin	BY DSM	DATE 10/2017
SUBJECT **BEAM DESIGN**	CHECKED	SHEET OF

BEAM MARK	LOADING (SKETCH)	V (K)	M (K-FT)	SIZE	DEFL (IN)
RB1	 W= 1.83′(20+195)	2.2	6.0	3x12 @ 1′-10″ Select Structural	Δ_L = -0.28″ Δ_T = -0.31″
RB2	 W=1.83′(20+195)	2.5	4.5 -3.1	3x12 @ 1′-10″ Select Structural	Δ_L = -0.19 Δ_T = -0.21 Δ_{L_C} = 0.11 Δ_{T_C} = 0.12
26^K = 2.4^K + 23.6^K RB3	 W=11′ (20+195)	26.0	143.1	5 1/8 x 31 1/2 GLB 6 3/4 x 27 GLB 8 3/4 x 24 GLB 10 3/4 x 22 1/2 GLB	Δ_L = -0.47″ Δ_T = -0.52″ Δ_L = -0.57″ Δ_T = -0.63″ Δ_L = -0.62″ Δ_T = -0.69″ Δ_L = -0.62″ Δ_T = -0.68″
7.7^K = 0.8^K + 6.9^K RB4	 W=9.5′(20+195)	7.7	14.4	(3) 1 3/4 x 9 1/4 LVL	Δ_L = -0.19″ Δ_T = -0.21″

CALDER RICHARDS CONSULTING ENGINEERS	JOB TITLE Meehan Cabin	BY DSM	DATE 10/2017
	SUBJECT BEAM DESIGN	CHECKED	SHEET OF

	BEAM MARK	LOADING (SKETCH)	V (K)	M (K-FT)	SIZE	DEFL (IN)
	RB5	W; 0^K 4.5′ 2.6^K 4.5′; W = 1.33′(20+195)	2.6	-2.9	1 3/4 x 9 1/4 LVL 1 3/4 x 7 1/4 LVL	Δ_L = 0.02″ Δ_T = 0.02″ Δ_{Lc} = -0.20″ Δ_{Tc} = -0.22″
$8^K = 0.7^K + 7.3^K$	RB6	W; 8.0^K 11′ 8.0^K; W = 6.75′(20+195)	8.0	22.0	5 1/8 x 12 GLB	Δ_L = -0.33″ Δ_T = -0.36″
$6.9^K = 0.6^K + 6.3^K$ $14.8 = 1.4^K + 13.4^K$	RB7	W; 6.9^K 11′ 14.8^K 4′; W = 6.75′(20+195)	9.0	16.5 -11.6	5 1/8 x 12 GLB	Δ_L = -0.22″ Δ_T = -0.25″ Δ_{Lc} = 0.12″ Δ_{Tc} = 0.14″
						Δ_L = Δ_T =

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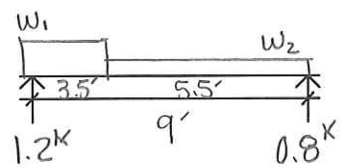
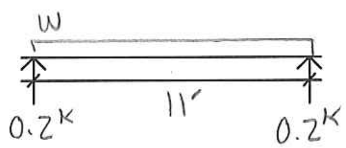
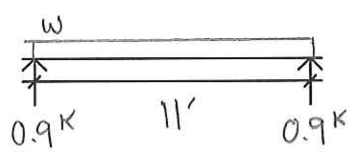
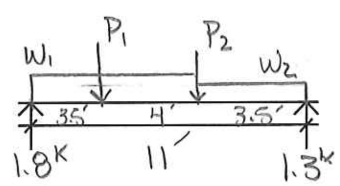
JOB TITLE Meehan Cabin	BY DSM	DATE 10/2017	
SUBJECT **BEAM DESIGN**	CHECKED	SHEET	OF

	BEAM MARK	LOADING (SKETCH)	V (K)	M (K-FT)	SIZE	DEFL (IN)
1.7=1.3+0.4	FB1	w 0.9^K 11' 0.9^K W=1.83'(50+40)	0.7	1.8	2x12 @ 1'-4" O.C.	Δ_L = -0.08" Δ_T = -0.18"
5.5^K=3.0^K+2.5^K	FB2	w 5.5^K 11' 5.5^K W=11'(50+40)	5.5	15.0	(3) 1 3/4 x 11 1/4 LVL 5 1/8 x 12 GLB	Δ_L = -0.23" Δ_T = -0.51" Δ_L = -0.11" Δ_T = -0.25"
3.8=2.1+1.7 3.0=1.6+1.4	FB3	P w 2' 9' 3.8^K 11' 3.0^K W=5.5'(50+40) P=1.3^K ≈ 0.7^{K}DL + 0.6^{K}LL	3.8	8.8	5 1/8 x 12 GLB	Δ_L = -0.07" Δ_T = -0.15"
5.1=2.8+2.3 3.9=2.1+1.8	FB4	P w 2' 9' 5.1^K 11' 3.9^K W=7.25'(50+40) P=1.8^K ≈ 1.0^{K}DL + 0.8^{K}LL	5.1	11.8	5 1/8 x 12 GLB	Δ_L = -0.09" Δ_T = -0.20"

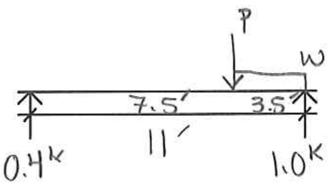
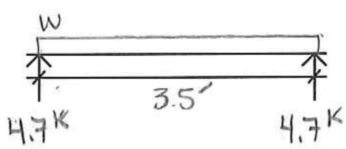
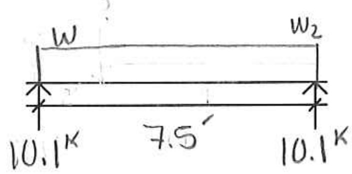
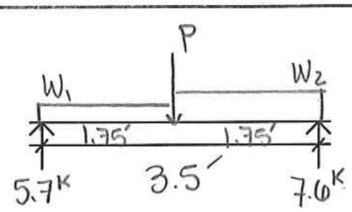
CALDER RICHARDS CONSULTING ENGINEERS	JOB TITLE Meehan Cabin	BY DSM	DATE 10/2017
	SUBJECT BEAM DESIGN	CHECKED	SHEET OF

BEAM MARK	LOADING (SKETCH)	V (K)	M (K-FT)	SIZE	DEFL (IN)
FB5	 $w_1 = 3.5'(50+40)$ $w_2 = 1.75'(50+40)$	1.2	2.1	(2) 2x10	$\Delta_L = -0.06''$ $\Delta_T = -0.13''$
FB6	 $w = 1.83'(15)$	0.2	0.4	2x8	$\Delta_L = -0.00''$ $\Delta_T = -0.16''$
FB7	 $w = 11'(15)$	0.9	2.5	(3) 2x8 (2) 2x10	$\Delta_L = -0.00''$ $\Delta_T = -0.32''$ $\Delta_L = 0.00''$ $\Delta_T = -0.23''$
FB8	 $w_1 = 1.83'(50+40)$ $w_2 = 0.92'(50+40)$ $P_1 = 1.2^K \approx 0.7^K DL + 0.5^K LL$ $P_2 = 0.3^K \approx 0.2^K DL + 0.1^K LL$	1.8	5.2	(3) 2x12 5 1/8 x 12 GLB	$\Delta_L = -0.07''$ $\Delta_T = -0.17''$ $\Delta_L = 0.03''$ $\Delta_T = 0.05''$

$1.8 = 1.0 + 0.8$

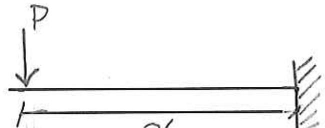
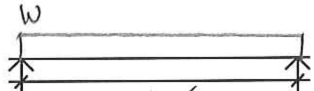
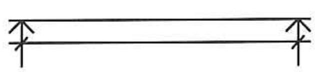
$1.3 = 0.7 + 0.6$

CALDER RICHARDS CONSULTING ENGINEERS	JOB TITLE Meehan Cabin	BY DSM	DATE 10/2017
	SUBJECT BEAM DESIGN	CHECKED	SHEET OF

	BEAM MARK	LOADING (SKETCH)	V (K)	M (K-FT)	SIZE	DEFL (IN)
0.4=0.2+0.2 1.0=0.6+0.4	FB9	 $W = 1.83'(50+40)$ $P = 0.8^K \approx 0.5^K DL + 0.3^K LL$	1.0	2.6	(2) 2x12	$\Delta_L = -0.06''$ $\Delta_T = -0.16''$
$4.7^K = 1.1^K + 3.6^K$	FB10	 $W = 5.5'(50+40) + 10'(15)$ $+ 9.5'(20+195)$	4.7	4.1	(3) 2x10	$\Delta_L = -0.02''$ $\Delta_T = -0.03''$
10.1 = 2.3+7.8	FB11	 $W = 5.5'(50+40) + 9.5(20+195)$ $+ 10'(15)$	10.1	18.9	(3) 1 3/4 x 11 7/8 LVL 5 1/8 x 12 GLB	$\Delta_L = -0.10''$ $\Delta_T = -0.13''$ $\Delta_L = -0.11''$ $\Delta_T = -0.14''$
5.7 = 1.0+4.7 7.6 = 1.3+6.3	FB12	 $W_1 = 5.5'(50+40)$ $W_2 = 5.5'(50+40) + 9.5(20+195)$ $+ 10'(15)$ $P = 7.7^K \approx 0.8^K DL + 6.9^K LL$	7.6	9.2	(3) 1 3/4 x 9 1/4 LVL 5 1/8 x 12 GLB	$\Delta_L = -0.02''$ $\Delta_T = -0.02''$ $\Delta_L = -0.01''$ $\Delta_T = -0.01''$

CALDER RICHARDS CONSULTING ENGINEERS

JOB TITLE Meehan Cabin		BY DSM		DATE 12/2017	
SUBJECT **BEAM DESIGN**		CHECKED		SHEET	OF

BEAM MARK	LOADING (SKETCH)	V (K)	M (K-FT)	SIZE	DEFL (IN)
Front Entrance Beam	 w; 13.09^K; $14.5'$; 13.09^K $w = 5.5'(35\,psf + 234\,psf)$ $+ 5.5'(59.2\,psf)$	13.09	47.4	W12x26	$\Delta_L = -0.27''$ $\Delta_T = -0.30''$
Front Entrance Beam	 P; $3'$ $P = 3.69^K = 0.38^K DL + 3.31^K LL$	3.69	11.07	W12x26 oversized to match	$\Delta_L = -0.00''$ $\Delta_T = -0.02''$
Front Entrance Beam	 w; 3.69^K; $14.5'$; 3.69^K $w = 1.5'(35\,psf + 234\,psf)$ $+ 1.5'(70.1\,psf)$	3.69	13.4	W12x26 Oversized to match.	$\Delta_L = -0.08''$ $\Delta_T = -0.09''$
					$\Delta_L =$ $\Delta_T =$

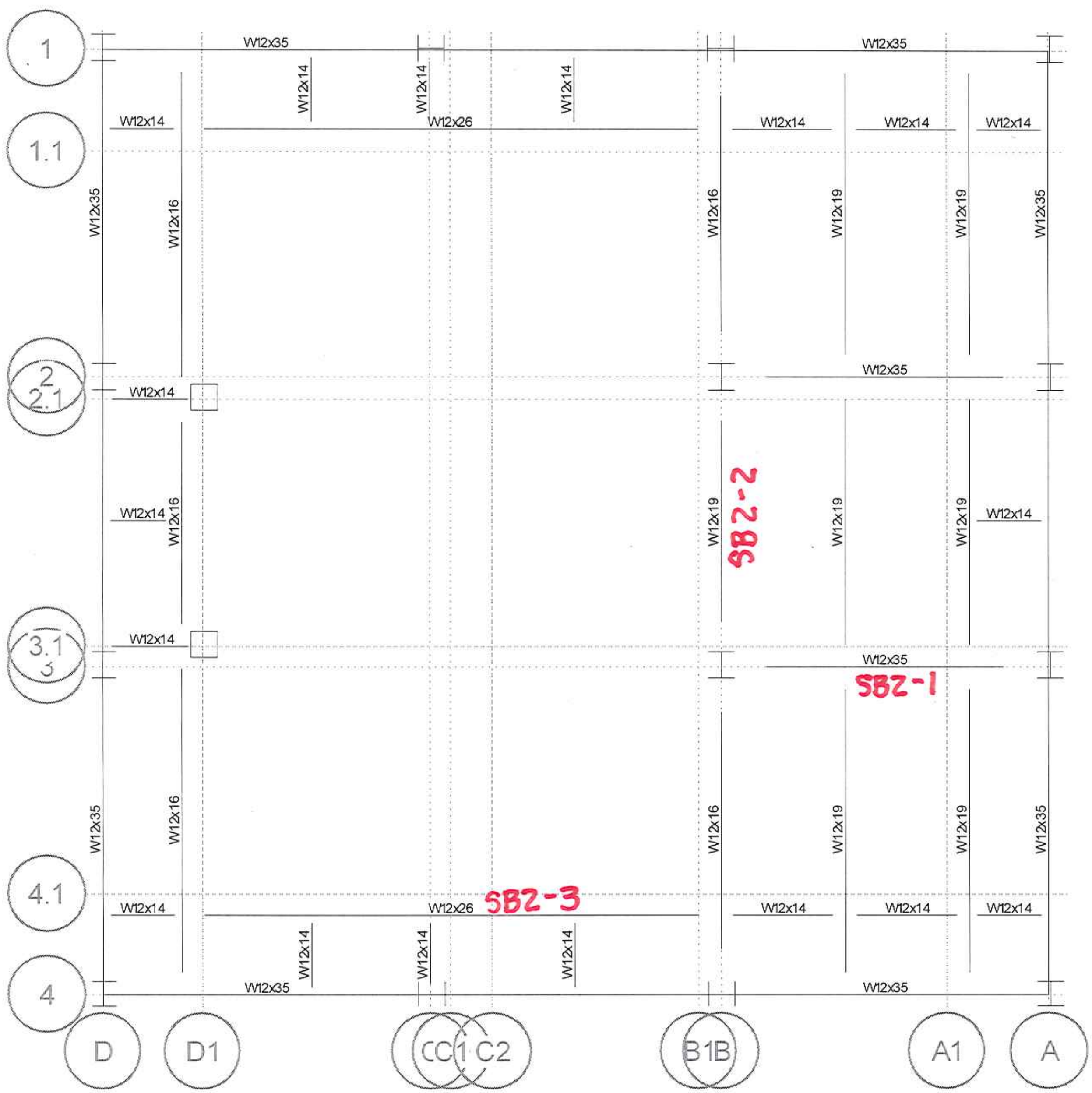
Floor Map

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/25/18 16:35:22
Steel Code: AISC 360-10 ASD

Floor Type: 2nd Floor

Beam Designs



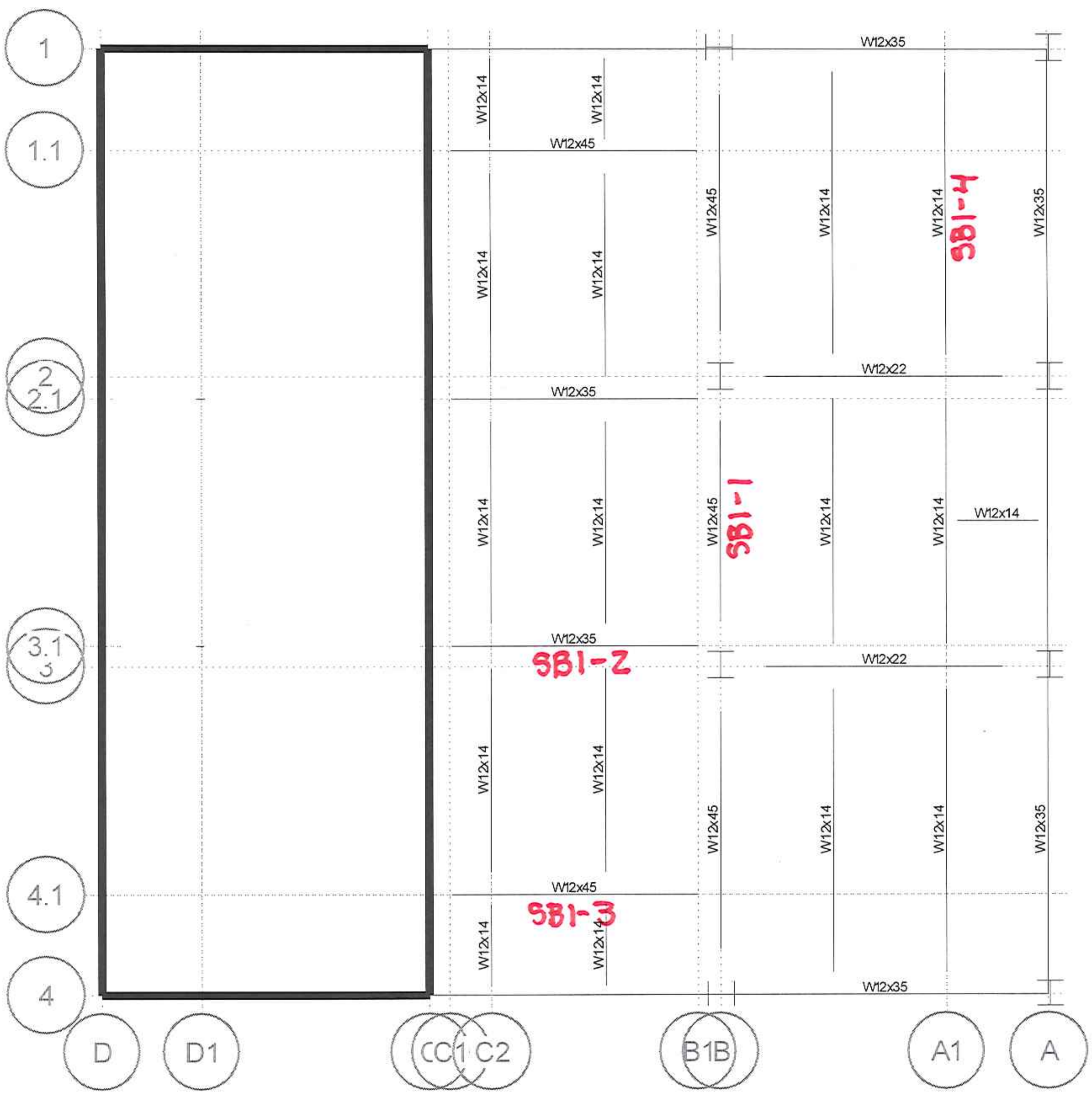
* Note: Not all steel beam calculations have been provided. Remaining calculations available upon request

Floor Map

RAM Structural System
Bentley
RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/25/18 16:35:22
Steel Code: AISC 360-10 ASD

Floor Type: 1st Floor **Beam Designs**



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Gravity Beam Design

SB2-1

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/25/18 16:35:22
Steel Code: AISC 360-10 ASD

Floor Type: 2nd Floor **Beam Number = 24**

SPAN INFORMATION (ft): I-End (27.42,14.58) J-End (42.00,14.58)

Beam Size (User Selected) = W12X35 Fy = 50.0 ksi
Total Beam Length (ft) = 14.58
Mp (kip-ft) = 213.33

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
5.500	4.90	0.00	0.0	0.00	0.00	0.0	22.88	Snow	0.00
11.000	4.05	0.00	0.0	0.00	0.00	0.0	17.44	Snow	0.00

SHEAR: Max Va (DL+LL) = 26.68 kips Vn/1.50 = 75.00 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	Max +	DL+LL	124.2	5.5	5.5	1.10	1.67	127.74
Controlling		DL+LL	124.2	5.5	5.5	1.10	1.67	127.74

REACTIONS (kips):

	Left	**Right**
DL reaction	4.05	4.90
Max +LL reaction	18.54	21.78
Max +total reaction	22.58	26.68

DEFLECTIONS:

Dead load (in)	at	7.29 ft	= -0.098	L/D =	1790
Live load (in)	at	7.22 ft	= -0.443	L/D =	395
Net Total load (in)	at	7.22 ft	= -0.541	L/D =	324

Gravity Beam Design

SB2-2

RAM Structural System
Bentley

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/25/18 16:35:22
Steel Code: AISC 360-10 ASD

Floor Type: 2nd Floor **Beam Number = 2**

SPAN INFORMATION (ft): I-End (27.42,14.58) J-End (27.42,27.42)

Beam Size (User Selected) = W12X19 Fy = 50.0 ksi
Total Beam Length (ft) = 12.83
Mp (kip-ft) = 102.92

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.000	0.000	---	NonR	0.000
	12.833	0.000	0.000			0.000
2	0.000	0.179	0.000	---	NonR	0.000
	12.833	0.179	0.000			0.000
3	0.000	0.000	0.883	---	Snow	0.000
	12.833	0.000	0.883			0.000

SHEAR: Max Va (DL+LL) = 6.81 kips Vn/1.50 = 57.34 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	Max +	DL+LL	21.9	6.4	0.0	1.00	1.67	61.63
Controlling		DL+LL	21.9	6.4	0.0	1.00	1.67	61.63

REACTIONS (kips):

	Left	**Right**
DL reaction	1.15	1.15
Max +LL reaction	5.67	5.67
Max +total reaction	6.81	6.81

DEFLECTIONS:

Dead load (in)	at	6.42 ft	=	-0.029	L/D	=	5322
Live load (in)	at	6.42 ft	=	-0.143	L/D	=	1077
Net Total load (in)	at	6.42 ft	=	-0.172	L/D	=	896

Gravity Beam Design

SB2-3

RAM Structural System
Bentley

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/25/18 16:35:22
Steel Code: AISC 360-10 ASD

Floor Type: 2nd Floor **Beam Number = 58**

SPAN INFORMATION (ft): I-End (3.58,3.58) J-End (27.42,3.58)

Beam Size (User Selected) = W12X26 Fy = 50.0 ksi
Total Beam Length (ft) = 23.83
Mp (kip-ft) = 155.00

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
5.708	0.64	0.00	0.0	0.00	0.00	0.0	2.94	Snow	0.00
11.000	0.68	0.00	0.0	0.00	0.00	0.0	3.13	Snow	0.00
17.417	0.75	0.00	0.0	0.00	0.00	0.0	3.43	Snow	0.00

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.000	0.000	---	NonR	0.000
	23.833	0.000	0.000			0.000

SHEAR: Max Va (DL+LL) = 5.90 kips Vn/1.50 = 56.12 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	Max +	DL+LL	45.9	11.0	6.4	1.09	1.67	92.81
Controlling		DL+LL	45.9	11.0	6.4	1.09	1.67	92.81

REACTIONS (kips):

	Left	**Right**
DL reaction	1.06	1.01
Max +LL reaction	4.84	4.65
Max +total reaction	5.90	5.66

DEFLECTIONS:

Dead load (in)	at	11.80 ft	=	-0.136	L/D	=	2109
Live load (in)	at	11.80 ft	=	-0.622	L/D	=	460
Net Total load (in)	at	11.80 ft	=	-0.758	L/D	=	378

Gravity Beam Design

SB1-1

RAM Structural System
Bentley

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/25/18 16:35:22
Steel Code: AISC 360-10 ASD

Floor Type: 1st Floor **Beam Number = 19**

SPAN INFORMATION (ft): I-End (27.42,14.58) J-End (27.42,27.42)

Beam Size (User Selected) = W12X45 Fy = 50.0 ksi
Total Beam Length (ft) = 12.83
Mp (kip-ft) = 267.50

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
0.917	10.58	0.00	0.0	39.46	0.00	0.0	0.00	Snow	0.00
11.917	11.13	0.00	0.0	39.88	0.00	0.0	0.00	Snow	0.00

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.353	0.202	---	NonR	0.000
	12.833	0.353	0.202			0.000

SHEAR: Max Va (DL+LL) = 54.51 kips Vn/1.50 = 81.07 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	Max +	DL+LL	57.7	6.5	0.0	1.00	1.67	160.18
Controlling		DL+LL	57.7	6.5	0.0	1.00	1.67	160.18

REACTIONS (kips):

	Left	**Right**
DL reaction	12.88	13.36
Max +LL reaction	40.79	41.15
Max +total reaction	53.67	54.51

DEFLECTIONS:

Dead load (in)	at	6.42 ft	=	-0.056	L/D =	2741
Live load (in)	at	6.42 ft	=	-0.140	L/D =	1104
Net Total load (in)	at	6.42 ft	=	-0.196	L/D =	787

RAM Structural System
Bentley

Gravity Beam Design

SB1-2

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/25/18 16:35:22
Steel Code: AISC 360-10 ASD

Floor Type: 1st Floor **Beam Number = 23**

SPAN INFORMATION (ft): I-End (14.58,15.50) J-End (27.42,15.50)

Beam Size (User Selected) = W12X35 Fy = 50.0 ksi
Total Beam Length (ft) = 12.83
Mp (kip-ft) = 213.33

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
2.750	4.07	0.00	0.0	2.27	0.00	0.0	0.00	Snow	0.00
7.792	4.51	0.00	0.0	2.45	0.00	0.0	0.00	Snow	0.00
0.917	5.10	0.00	0.0	4.20	0.00	0.0	0.00	Snow	0.00
11.917	6.40	0.00	0.0	39.60	0.00	0.0	0.00	Snow	0.00

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.917	0.350	0.220	---	NonR	0.000
	6.916	0.350	0.220			0.000
2	0.000	0.600	0.470	---	NonR	0.000
	0.916	0.600	0.470			0.000

SHEAR: Max Va (DL+LL) = 50.04 kips Vn/1.50 = 75.00 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	Max +	DL+LL	62.5	7.8	5.0	1.12	1.67	127.74
Controlling		DL+LL	62.5	7.8	5.0	1.12	1.67	127.74

REACTIONS (kips):

	Left	**Right**
DL reaction	12.15	10.58
Max +LL reaction	10.80	39.46
Max +total reaction	22.96	50.04

DEFLECTIONS:

Dead load (in)	at	6.35 ft	=	-0.099	L/D =	1559
Live load (in)	at	6.67 ft	=	-0.129	L/D =	1189
Net Total load (in)	at	6.67 ft	=	-0.228	L/D =	675

Gravity Beam Design

SB1-3

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

RAM Structural System
Bentley

02/25/18 16:35:22
Steel Code: AISC 360-10 ASD

Floor Type: 1st Floor **Beam Number = 24**

SPAN INFORMATION (ft): I-End (14.58,4.50) J-End (27.42,4.50)

Beam Size (User Selected) = W12X45 Fy = 50.0 ksi
Total Beam Length (ft) = 12.83
Mp (kip-ft) = 267.50

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
2.750	1.70	0.00	0.0	0.91	0.00	0.0	0.00	Snow	0.00
2.750	0.58	0.00	0.0	0.07	0.00	0.0	2.06	Snow	0.00
7.792	1.94	0.00	0.0	1.11	0.00	0.0	0.00	Snow	0.00
7.792	0.75	0.00	0.0	0.10	0.00	0.0	2.67	Snow	0.00
2.417	1.00	0.00	0.0	4.70	0.00	0.0	0.00	Snow	0.00
5.917	1.30	0.00	0.0	6.30	0.00	0.0	0.00	Snow	0.00
11.917	2.30	0.00	0.0	14.20	0.00	0.0	0.00	Snow	0.00

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	5.917	0.770	2.070	---	NonR	0.000
	11.916	0.770	2.070			0.000
2	11.917	0.290	0.000	---	NonR	0.000
	12.833	0.290	0.000			0.000
3	0.000	0.850	0.440	---	NonR	0.000
	2.416	0.850	0.440			0.000

SHEAR: Max Va (DL+LL) = 37.41 kips Vn/1.50 = 81.07 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	Max +	DL+LL	96.2	7.0	5.0	1.09	1.67	160.18
Controlling		DL+LL	96.2	7.0	5.0	1.09	1.67	160.18

REACTIONS (kips):

	Left	Right
DL reaction	7.80	8.70
Max +LL reaction	16.90	28.71
Max +total reaction	24.70	37.41

DEFLECTIONS:

Dead load (in)	at	6.48 ft	=	-0.077	L/D =	2002
Live load (in)	at	6.55 ft	=	-0.201	L/D =	765
Net Total load (in)	at	6.55 ft	=	-0.278	L/D =	553

Gravity Beam Design

SB1-4

RAM Structural System
Bentley

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/25/18 16:35:22
Steel Code: AISC 360-10 ASD

Floor Type: 1st Floor **Beam Number = 56**

SPAN INFORMATION (ft): I-End (37.50,27.42) J-End (37.50,42.00)

Beam Size (Optimum) = W12X14 Fy = 50.0 ksi
Total Beam Length (ft) = 14.58
Mp (kip-ft) = 72.50

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
10.083	0.93								
2.750	0.40								

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.210	0.120	---	NonR	0.000
	10.583	0.210	0.120			0.000
2	10.584	0.310	0.000	---	NonR	0.000
	14.583	0.310	0.000			0.000
3	10.584	0.000	0.777	---	Snow	0.000
	14.583	0.000	0.701			0.000
4	0.000	0.116	0.000	---	NonR	0.000
	10.583	0.116	0.000			0.000
5	0.000	0.000	0.530	---	Snow	0.000
	10.583	0.000	0.530			0.000
6	10.584	0.000	0.676	---	Snow	0.000
	11.916	0.000	0.666			0.000
7	11.917	0.000	0.666	---	Snow	0.000
	13.250	0.000	0.648			0.000
8	13.250	0.000	0.648	---	Snow	0.000
	14.583	0.000	0.626			0.000

SHEAR: Max Va (DL+LL) = 10.33 kips Vn/1.67 = 42.75 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	Max +	DL+LL	31.8	7.9	0.0	1.00	1.67	43.41
Controlling		DL+LL	31.8	7.9	0.0	1.00	1.67	43.41

REACTIONS (kips):

	Left	Right
DL reaction	2.97	3.04
Max +LL reaction	5.16	7.29
Max +total reaction	8.13	10.33

DEFLECTIONS:

Dead load (in)	at	7.36 ft	= -0.170	L/D =	1031
Live load (in)	at	7.44 ft	= -0.309	L/D =	566
Net Total load (in)	at	7.44 ft	= -0.479	L/D =	365

CALDER RICHARDS CONSULTING ENGINEERS	JOB TITLE Meehan Cabin	BY DSM	DATE 01/2018
	SUBJECT COLUMN DESIGN	CHECKED	SHEET OF

		LOAD (KIPS)	KL (ft)	COLUMN	
Grid: D1	:1.1	16.5	7'	6x6	
Grid: C1	: 1.1	0.9	7'	3x6	
Grid: C2	:2:1.1	14.9	9'	6x6	
Grid: B1	:1.1	19.6	9'	6x8	
Grid: D1	: 2.1	58.1	7'	HSS5½x5½x3/16	
Grid: C1	: 2.1	10.6	7'	6x6	
Grid: B1	: 2.1	47.5	9'	HSS5½x5½x3/16	
Grid: D1:	3.1	58.1	9'	HSS5½x5½x3/16	
Grid: C1:	3.1	9.3	7'	6x6	
Grid: B1:	3.1	46.0	9'	HSS5½x5½x3/16	
Grid: D1:	4.1	16.5	7'	6x6	
Grid: D1.2	:4.1	4.7	7'	3x6	
Grid: D1.6	:4.1	9.8	7'	6x6	
Grid: C2: 4	.1	5.7	7'	3x6	
Grid: C2.4	4.1	7.6	9'	3x6	
Grid: B1: 4	.1	16.5'	9'	6x6	
Posts w/ Omega Due to		Discontinuous	Shear	Walls + ρ=1.0	
Grid: D1:	1.1	24.91k	7'	6x10	
Grid: D1.6	:1.1	18.59k	7'	6x8	
Grid: C2.2	:1.1	27.30k	7'	6x10	
Grid: B1:	1.1	24.91k	7'	6x10	
Grid: D1: 4	.1	24.91k	7'	6x10	
Grid: D1.6:	4.1	22.46k	7'	6x10	
Grid: C2: 4	.1	10.86k	7'	6x6	
Grid: C2.3	:4.1	12.39k	7'	6x6	
Grid: B1: 4	.1	24.91k	7'	6x10	

CALDER RICHARDS CONSULTING ENGINEERS	JOB TITLE Meehan Cabin	BY DSM	DATE 01/2018
	SUBJECT	CHECKED	SHEET OF

* For ease of design, column loads were determined w/ a DL+LL+SL Load combination. This is conservative and needs to be revised for some columns

Grid C2.2:1.1

Roof = 14.8^k = 1.4^k DL + 13.4^k SL

Floor = 10.1^k = 2.3^k DL + 7.8^k LL

DL+LL = 11.5^k

DL+SL = 17.1^k

DL+ 0.75LL + 0.75SL = 19.6^k ∴ 6x8

Grid B1:1.1

Roof = 7.7^k = 0.8^k DL + 6.9^k SL

Floor = 11.0^k = 2.8^k DL + 8.2^k LL

DL+LL = 11.8^k

DL+SL = 10.5^k

DL+0.75LL + 0.75SL = 14.9^k ∴ 6x6

Grid D1.6:4.1

Roof = 7.7^k = 0.8^k DL + 6.9^k SL

Floor = 4.7^k = 1.1^k DL + 3.6^k LL

DL+LL = 5.5^k

DL+SL = 8.8^k

DL+0.75LL + 0.75SL = 9.8^k ∴ 6x6

3 x 6 STUDS I / 5.5" (strong axis)

STUD WALL BEARING CAPACITIES OF DF-South #2
ALLOWABLE STRESSES FROM 2005 NDS; NORMAL TEMP., MOISTURE & NO INCISIONS
ALLOWABLE LOADS FROM 2012 NDS, SECTION 3.7 - COLUMN DESIGN

Fce = (0.822Emin')(E)/(l/d)^2
Cp = (1+(Fce/Fc*))/(2c) - [((1+(Fce/Fc*))/(2c))^2 - (Fce/Fc*)/c]^0.5
Pa = (Cp)(Fc*)(width)(depth)

width =	2.5	inches
depth =	5.5	inches
Fcperp =	520	psi
Fc =	1350	psi
E =	1200000	psi
C_F =	1.1	
Fc* = Fc * C_F =	1485	psi
c=	0.8	for sawn lumber
Emin' =	440000	psi

L(unb) (ft)	l/d	Fce (psi)	Cp	**1 STUD** Pa (lbs)
1	2.18	75977.92	0.9960	20338
2	4.36	18994.48	0.9836	20084
3	6.55	8441.99	0.9609	19621
4	8.73	4748.62	0.9248	18882
5	10.91	3039.12	0.8709	17784
6	13.09	2110.50	0.7967	16268
7	15.27	1550.57	0.7057	14410
8	17.45	1187.15	0.6093	12442
9	19.64	938.00	0.5194	10605
10	21.82	759.78	0.4417	9020
11	24.00	627.92	0.3772	7701
12	26.18	527.62	0.3242	6620
13	28.36	449.57	0.2808	5734
14	30.55	387.64	0.2451	5005
15	32.73	337.68	0.2155	4401
16	34.91	296.79	0.1909	3897
17	37.09	262.90	0.1701	3473
18	39.27	234.50	0.1524	3112
19	41.45	210.47	0.1374	2805
20	43.64	189.94	0.1244	2540
21	45.82	172.29	0.1131	2310
22	48.00	156.98	0.1033	2110
22.9167	50.00	144.67	0.0954	1948

max. load if stud(s) bears on wood = Fcperp*Cb*AREA =

8938	lbs 1 stud
16088	lbs 2 stud
23230	lbs 3 stud

6 x 6 POST l / 5.5"

POST BEARING CAPACITIES OF Doug-Fir South #1
ALLOWABLE STRESSES FROM 2005 NDS; NORMAL TEMP., MOISTURE & NO INCISIONS
ALLOWABLE LOADS FROM 2005 NDS, SECTION 3.7 - COLUMN DESIGN

Fce = (.822)(Emin')/(l/d)^2
Cp = (1+(Fce/Fc*))/2c - [((1+(Fce/Fc*))/2c)^2 - (Fce/Fc*)/c]^0.5
Pa = (Cp)(Fc*)(width)(depth)

width =	5.5	inches
depth =	5.5	inches
Fcperp =	520	psi
Fc =	925	psi
E =	1200000	psi
C_F =	1	
Fc* = Fc * C_F =	925	psi
c=	0.8	for sawn lumber
Emin' =	440000	psi

L(unb) (ft)	l/d	Fce (psi)	Cp	Pa (lbs)
1	2.18	75977.92	0.9975	27913
2	4.36	18994.48	0.9900	27701
3	6.55	8441.99	0.9766	27326
4	8.73	4748.62	0.9562	26756
5	10.91	3039.12	0.9271	25942
6	13.09	2110.50	0.8871	24823
7	15.27	1550.57	0.8345	23351
8	17.45	1187.15	0.7695	21532
9	19.64	938.00	0.6958	19469
10	21.82	759.78	0.6196	17336
11	24.00	627.92	0.5468	15301
12	26.18	527.62	0.4812	13463
13	28.36	449.57	0.4237	11856
14	30.55	387.64	0.3743	10473
15	32.73	337.68	0.3320	9291
16	34.91	296.79	0.2960	8282
17	37.09	262.90	0.2651	7418
18	39.27	234.50	0.2386	6675
19	41.45	210.47	0.2157	6035
20	43.64	189.94	0.1958	5479
21	45.82	172.29	0.1785	4995
22	48.00	156.98	0.1633	4570
22.917	50.00	144.67	0.1510	4226

max. load if post bears on wood = Fcperp*Cb*AREA = 16800 lbs

controls for all heights in this home.

6 x 8 POST l / 5.5" (weak axis)

POST BEARING CAPACITIES OF Doug-Fir South #1
ALLOWABLE STRESSES FROM 2005 NDS; NORMAL TEMP., MOISTURE & NO INCISIONS
ALLOWABLE LOADS FROM 2005 NDS, SECTION 3.7 - COLUMN DESIGN

Fce = (.822)(Emin')/(l/d)^2
Cp = (1+(Fce/Fc*))/2c - [((1+(Fce/Fc*))/2c)^2 - (Fce/Fc*)/c]^0.5
Pa = (Cp)(Fc*)(width)(depth)

width =	7.5	inches
depth =	5.5	inches
Fcperp =	520	psi
Fc =	925	psi
E =	1200000	psi
C_F =	1	
Fc* = Fc * C_F =	925	psi
c=	0.8	for sawn lumber
Emin' =	440000	psi

L(unb) (ft)	l/d	Fce (psi)	Cp	Pa (lbs)
1	2.18	75977.92	0.9975	38063
2	4.36	18994.48	0.9900	37774
3	6.55	8441.99	0.9766	37263
4	8.73	4748.62	0.9562	36486
5	10.91	3039.12	0.9271	35375
6	13.09	2110.50	0.8871	33849
7	15.27	1550.57	0.8345	31842
8	17.45	1187.15	0.7695	29362
9	19.64	938.00	0.6958	26548
10	21.82	759.78	0.6196	23641
11	24.00	627.92	0.5468	20866
12	26.18	527.62	0.4812	18359
13	28.36	449.57	0.4237	16167
14	30.55	387.64	0.3743	14282
15	32.73	337.68	0.3320	12670
16	34.91	296.79	0.2960	11293
17	37.09	262.90	0.2651	10115
18	39.27	234.50	0.2386	9103
19	41.45	210.47	0.2157	8229
20	43.64	189.94	0.1958	7471
21	45.82	172.29	0.1785	6811
22	48.00	156.98	0.1633	6232
22.917	50.00	144.67	0.1510	5763

max. load if post bears on wood = Fcperp*Cb*AREA = 21450 lbs

Controls for all heights in this home

6 x 10 POST l / 5.5" Weak Axis

POST BEARING CAPACITIES OF Doug-Fir South #1
ALLOWABLE STRESSES FROM 2005 NDS; NORMAL TEMP., MOISTURE & NO INCISIONS
ALLOWABLE LOADS FROM 2005 NDS, SECTION 3.7 - COLUMN DESIGN

Fce = (.822)(Emin')/(l/d)^2
Cp = (1+(Fce/Fc*))/2c - [((1+(Fce/Fc*))/2c)^2 - (Fce/Fc*)/c]^0.5
Pa = (Cp)(Fc*)(width)(depth)

width =	9.5	inches
depth =	5.5	inches
Fcperp =	520	psi
Fc =	925	psi
E =	1200000	psi
C_F =	1	
Fc* = Fc * C_F =	925	psi
c=	0.8	for sawn lumber
Emin' =	440000	psi

L(unb) (ft)	l/d	Fce (psi)	Cp	Pa (lbs)
1	2.18	75977.92	0.9975	48213
2	4.36	18994.48	0.9900	47847
3	6.55	8441.99	0.9766	47200
4	8.73	4748.62	0.9562	46215
5	10.91	3039.12	0.9271	44808
6	13.09	2110.50	0.8871	42876
7	15.27	1550.57	0.8345	40334
8	17.45	1187.15	0.7695	37192
9	19.64	938.00	0.6958	33628
10	21.82	759.78	0.6196	29945
11	24.00	627.92	0.5468	26430
12	26.18	527.62	0.4812	23255
13	28.36	449.57	0.4237	20479
14	30.55	387.64	0.3743	18090
15	32.73	337.68	0.3320	16048
16	34.91	296.79	0.2960	14305
17	37.09	262.90	0.2651	12812
18	39.27	234.50	0.2386	11530
19	41.45	210.47	0.2157	10424
20	43.64	189.94	0.1958	9464
21	45.82	172.29	0.1785	8627
22	48.00	156.98	0.1633	7894
23	50.18	143.63	0.1500	7249
24	52.36	131.91	0.1382	6678
25	54.55	121.56	0.1277	6171
26	56.73	112.39	0.1183	5719
27	58.91	104.22	0.1100	5314

max. load if post bears on wood = Fcperp*Cb*AREA = 27170 lbs

CONTROLS

Table 4-4 (continued)
Available Strength in Axial Compression, kips

$F_y = 46$ ksi

HSS5½-HSS5

Square HSS

Shape		HSS5½×5½×										HSS5×5×	
		3/8		5/16		1/4		3/16		1/8[c]		1/2	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116		0.465	
lb/ft		24.9		21.2		17.3		13.3		9.01		28.4	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, KL (ft), with respect to least radius of gyration, r_y	0	190	285	161	242	131	197	100	150	58.0	87.2	217	326
	1	189	284	161	242	131	197	99.8	150	58.0	87.1	216	325
	2	188	282	160	240	130	196	99.2	149	57.8	86.9	215	322
	3	186	279	158	237	129	194	98.1	147	57.5	86.4	211	318
	4	183	275	156	234	127	191	96.7	145	57.0	85.7	207	311
	5	179	269	153	229	125	187	94.9	143	56.4	84.8	202	303
	6	175	263	149	224	122	183	92.8	139	55.7	83.7	195	294
	7	170	255	145	218	118	178	90.3	136	54.8	82.4	188	283
	8	164	247	140	211	115	172	87.5	132	53.8	80.9	180	271
	9	158	238	135	203	111	166	84.5	127	52.7	79.2	171	257
	10	151	228	130	195	106	160	81.2	122	51.4	77.3	162	244
	11	145	217	124	186	101	153	77.8	117	50.0	75.2	152	229
	12	137	206	118	177	96.6	145	74.1	111	48.5	72.8	142	214
	13	130	195	112	168	91.6	138	70.4	106	46.7	70.3	132	199
	14	122	184	105	158	86.5	130	66.6	100	44.9	67.5	122	184
	15	115	172	98.8	148	81.3	122	62.7	94.2	42.9	64.5	112	169
	16	107	161	92.3	139	76.1	114	58.8	88.3	40.4	60.7	103	154
	17	99.2	149	85.9	129	70.9	107	54.9	82.5	37.8	56.8	93.2	140
	18	91.7	138	79.6	120	65.8	98.9	51.0	76.7	35.2	52.9	84.1	126
	19	84.5	127	73.5	110	60.8	91.4	47.3	71.0	32.7	49.1	75.5	113
	20	77.4	116	67.5	101	55.9	84.1	43.6	65.5	30.2	45.4	68.1	102
	21	70.5	106	61.6	92.7	51.2	77.0	40.0	60.2	27.8	41.8	61.8	92.9
	22	64.2	96.5	56.2	84.4	46.7	70.1	36.5	54.9	25.4	38.2	56.3	84.6
	23	58.7	88.3	51.4	77.2	42.7	64.2	33.4	50.2	23.3	35.0	51.5	77.4
	24	53.9	81.1	47.2	70.9	39.2	58.9	30.7	46.1	21.4	32.1	47.3	71.1
	25	49.7	74.7	43.5	65.4	36.1	54.3	28.3	42.5	19.7	29.6	43.6	65.5
	26	46.0	69.1	40.2	60.4	33.4	50.2	26.2	39.3	18.2	27.4	40.3	60.6
	27	42.6	64.1	37.3	56.0	31.0	46.6	24.2	36.4	16.9	25.4	37.4	56.2
	28	39.6	59.6	34.7	52.1	28.8	43.3	22.5	33.9	15.7	23.6	34.8	52.2
	29	36.9	55.5	32.3	48.6	26.9	40.4	21.0	31.6	14.6	22.0	32.4	48.7
	30	34.5	51.9	30.2	45.4	25.1	37.7	19.6	29.5	13.7	20.6	30.3	45.5

Properties

	3/8	5/16	1/4	3/16	1/8[c]	1/2
A_g, in.^{2}	6.88	5.85	4.77	3.63	2.46	7.88
$I_x = I_y$, in.^{4}	29.7	25.9	21.7	17.0	11.8	26.0
$r_x = r_y$, in.	2.08	2.11	2.13	2.16	2.19	1.82

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 46$ ksi.

Table 4-4 (continued)
Available Strength in Axial Compression, kips

$F_y = 46$ ksi

HSS5-

Square HSS

Shape		HSS5×5×									
		3/8		5/16		1/4		3/16		1/8[c]	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		22.4		19.1		15.6		12.0		8.16	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, KL (ft), with respect to least radius of gyration, r_y	0	170	256	145	218	118	178	90.3	136	56.4	84.[illegible]
	1	170	255	144	217	118	178	90.1	135	56.4	84.[illegible]
	2	168	253	143	215	117	176	89.4	134	56.1	84.[illegible]
	3	166	250	141	213	116	174	88.3	133	55.7	83.[illegible]
	4	163	245	139	209	114	171	86.8	130	55.1	82.[illegible]
	5	159	239	135	204	111	167	84.8	127	54.3	81.[illegible]
	6	154	232	132	198	108	162	82.5	124	53.4	80.[illegible]
	7	149	223	127	191	104	157	79.8	120	52.3	78.[illegible]
	8	143	214	122	183	100	151	76.9	116	51.0	76.[illegible]
	9	136	204	117	175	95.9	144	73.7	111	49.5	74.[illegible]
	10	129	194	111	167	91.3	137	70.2	106	47.8	71.[illegible]
	11	122	183	105	157	86.5	130	66.6	100	45.7	68.[illegible]
	12	114	172	98.5	148	81.4	122	62.8	94.4	43.2	64.[illegible]
	13	107	160	92.1	138	76.3	115	59.0	88.7	40.6	61.[illegible]
	14	98.9	149	85.6	129	71.1	107	55.1	82.8	38.0	57.[illegible]
	15	91.3	137	79.2	119	66.0	99.2	51.2	77.0	35.4	53.[illegible]
	16	83.8	126	72.9	110	60.9	91.5	47.4	71.2	32.8	49.[illegible]
	17	76.4	115	66.7	100	55.9	84.0	43.6	65.5	30.3	45.[illegible]
	18	69.4	104	60.7	91.3	51.0	76.7	39.9	60.0	27.8	41.[illegible]
	19	62.5	93.9	54.9	82.5	46.3	69.6	36.4	54.6	25.4	38.[illegible]
	20	56.4	84.8	49.6	74.5	41.8	62.8	32.9	49.4	23.0	34.[illegible]
	21	51.2	76.9	44.9	67.6	37.9	57.0	29.8	44.8	20.9	31.[illegible]
	22	46.6	70.0	41.0	61.5	34.5	51.9	27.2	40.8	19.0	28.[illegible]
	23	42.6	64.1	37.5	56.3	31.6	47.5	24.9	37.4	17.4	26.[illegible]
	24	39.2	58.9	34.4	51.7	29.0	43.6	22.8	34.3	16.0	24.[illegible]
	25	36.1	54.2	31.7	47.7	26.7	40.2	21.0	31.6	14.7	22.[illegible]
	26	33.4	50.2	29.3	44.1	24.7	37.2	19.5	29.2	13.6	20.[illegible]
	27	30.9	46.5	27.2	40.9	22.9	34.5	18.0	27.1	12.6	19.[illegible]
	28	28.8	43.2	25.3	38.0	21.3	32.1	16.8	25.2	11.8	17.[illegible]
	29	26.8	40.3	23.6	35.4	19.9	29.9	15.6	23.5	11.0	16.[illegible]

Properties

	3/8	5/16	1/4	3/16	1/8[c]
A_g, in.^{2}	6.18	5.26	4.30	3.28	2.23
$I_x = I_y$, in.^{4}	21.7	19.0	16.0	12.6	8.30
$r_x = r_y$, in.	1.87	1.90	1.93	1.96	1.99

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 46$ ksi.
Note: Heavy line indicates KL/r_y equal to or greater than 200.

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Gravity Column Design

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 13:01:57
Steel Code: AISC 360-10 ASD

Story level 2nd, Column Line D-4, Column # 1

Fy (ksi) = 50.00 Column Size = W10X33
Orientation (deg.) = 90.0

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		9.00	9.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.36	6.48
	Bottom	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	2.82	0.00	12.66

DEMAND CAPACITY RATIO: (DL + RF)

Pa (kip)	=	15.48	Pnx/1.67 (kip)	=	276.98	Pa/(Pnx/1.67)	=	0.056
			Pny/1.67 (kip)	=	231.85	Pa/(Pny/1.67)	=	0.067
			Pn/1.67 (kip)	=	231.85	Pa/(Pn/1.67)	=	0.067

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 1:

			Dead	Live	Roof
Axial (kip)			2.82	0.00	12.66
Moments	Top	Mx (kip-ft)	0.52	0.00	2.30
		My (kip-ft)	1.06	0.00	4.82
	Bot	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (DL + RF)

Pa (kip)	=	15.48	Pn/1.67 (kip)	=	231.85
Max (kip-ft)	=	2.82	Mnx/1.67 (kip-ft)	=	96.81
May (kip-ft)	=	5.88	Mny/1.67 (kip-ft)	=	34.93
Rm	=	1.00			
Cbx	=	1.67			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	4196.10	Pey (kip)	=	898.11
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.067
Eq H1-1b: 0.033 + 0.029 + 0.168 = 0.231

Gravity Column Design

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 13:02:15
Steel Code: AISC 360-10 ASD

Story level 1st, Column Line A-1, Column # 4

Fy (ksi)	= 50.00	Column Size	= W10X49
Orientation (deg.)	= 90.0		

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		10.67	10.67
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.50	7.50
	Bottom	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	7.82	0.00	26.22

DEMAND CAPACITY RATIO: (DL + RF)

Pa (kip) = 34.05	Pnx/1.67 (kip)	=	404.63	Pa/(Pnx/1.67)	=	0.084
	Pny/1.67 (kip)	=	358.39	Pa/(Pny/1.67)	=	0.095
	Pn/1.67 (kip)	=	358.39	Pa/(Pn/1.67)	=	0.095

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 1:

			Dead	Live	Roof
Axial (kip)			7.82	0.00	26.22
Moments	Top	Mx (kip-ft)	-0.30	0.00	-1.34
		My (kip-ft)	-0.95	0.00	-1.85
	Bot	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (DL + RF)

Pa (kip)	=	34.05	Pn/1.67 (kip)	=	358.39
Max (kip-ft)	=	-1.64	Mnx/1.67 (kip-ft)	=	150.70
May (kip-ft)	=	-2.80	Mny/1.67 (kip-ft)	=	70.61
Rm	=	1.00			
Cbx	=	1.67			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	4748.71	Pey (kip)	=	1630.62
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.095
Eq H1-1b: 0.047 + 0.011 + 0.040 = 0.098

Gravity Column Design

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 13:02:39
Steel Code: AISC 360-10 ASD

Story level 2nd, Column Line D-1, Column # 4

Fy (ksi)	= 50.00		Column Size	= W10X33
Orientation (deg.)	= 90.0			

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		9.00	9.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.36	6.48
	Bottom	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	2.82	0.00	12.66

DEMAND CAPACITY RATIO: (DL + RF)

Pa (kip)	=	15.48	Pnx/1.67 (kip)	=	276.98	Pa/(Pnx/1.67)	=	0.056
			Pny/1.67 (kip)	=	231.85	Pa/(Pny/1.67)	=	0.067
			Pn/1.67 (kip)	=	231.85	Pa/(Pn/1.67)	=	0.067

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 1:

			Dead	Live	Roof
Axial (kip)			2.82	0.00	12.66
Moments	Top	Mx (kip-ft)	-0.52	0.00	-2.30
		My (kip-ft)	1.06	0.00	4.82
	Bot	Mx (kip-ft)	0.00	0.00	-0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (DL + RF)

Pa (kip)	=	15.48	Pn/1.67 (kip)	=	231.85
Max (kip-ft)	=	-2.82	Mnx/1.67 (kip-ft)	=	96.81
May (kip-ft)	=	5.88	Mny/1.67 (kip-ft)	=	34.93
Rm	=	1.00			
Cbx	=	1.67			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	4196.10	Pey (kip)	=	898.11
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.067

Eq H1-1b: 0.033 + 0.029 + 0.168 = 0.231

Gravity Column Design

Story level 1st, Column Line A-4, Column # 5

Fy (ksi)	= 50.00	Column Size	= W10X49
Orientation (deg.)	= 90.0		

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		10.67	10.67
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.50	7.50
	Bottom	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	7.77	0.00	26.22

DEMAND CAPACITY RATIO: (DL + RF)

Pa (kip) =	33.99	Pnx/1.67 (kip)	=	404.63	Pa/(Pnx/1.67)	=	0.084
		Pny/1.67 (kip)	=	358.39	Pa/(Pny/1.67)	=	0.095
		Pn/1.67 (kip)	=	358.39	Pa/(Pn/1.67)	=	0.095

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 1:

			Dead	Live	Roof
Axial (kip)			7.77	0.00	26.22
Moments	Top	Mx (kip-ft)	0.30	0.00	1.34
		My (kip-ft)	-0.93	0.00	-1.85
	Bot	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (DL + RF)

Pa (kip)	=	33.99	Pn/1.67 (kip)	=	358.39
Max (kip-ft)	=	1.64	Mnx/1.67 (kip-ft)	=	150.70
May (kip-ft)	=	-2.79	Mny/1.67 (kip-ft)	=	70.61
Rm	=	1.00			
Cbx	=	1.67			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	4748.71	Pey (kip)	=	1630.62
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.095
Eq H1-1b: 0.047 + 0.011 + 0.039 = 0.098

Gravity Column Design

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 13:02:15
Steel Code: AISC 360-10 ASD

Story level 2nd, Column Line A-1, Column # 7

Fy (ksi)	= 50.00	Column Size	= W10X49
Orientation (deg.)	= 90.0		

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	9.00	9.00
K	1	1
Braced Against Joint Translation	Yes	Yes
Column Eccentricity (in) Top	7.50	7.50
Bottom	7.50	7.50

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	3.46	0.00	15.06

DEMAND CAPACITY RATIO: (DL + RF)

Pa (kip)	= 18.51	Pnx/1.67 (kip)	=	412.10	Pa/(Pnx/1.67)	=	0.045
		Pny/1.67 (kip)	=	378.02	Pa/(Pny/1.67)	=	0.049
		Pn/1.67 (kip)	=	378.02	Pa/(Pn/1.67)	=	0.049

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	3.46	0.00	15.06
Moments Top Mx (kip-ft)	-0.53	0.00	-2.17
My (kip-ft)	-1.63	0.00	-7.24
Bot Mx (kip-ft)	-0.36	0.00	-1.59
My (kip-ft)	-1.12	0.00	-2.20

Reverse curvature about X-Axis
Reverse curvature about Y-Axis

CALCULATED PARAMETERS: (DL + RF)

Pa (kip)	=	18.51	Pn/1.67 (kip)	=	378.02
Max (kip-ft)	=	-2.70	Mnx/1.67 (kip-ft)	=	150.70
May (kip-ft)	=	-8.87	Mny/1.67 (kip-ft)	=	70.61
Rm	=	1.00			
Cbx	=	2.22			
Cmx	=	0.31	Cmy	=	0.45
Pex (kip)	=	6674.51	Pey (kip)	=	2291.91
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.049
Eq H1-1b: 0.024 + 0.018 + 0.126 = 0.168

Gravity Column Design

RAM Structural System
Bentley
RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 13:01:35
Steel Code: AISC 360-10 ASD

Story level 2nd, Column Line A-4, Column # 10

Fy (ksi)	=	50.00	Column Size	=	W10X49
Orientation (deg.)	=	90.0			

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		9.00	9.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.50	7.50
	Bottom	7.50	7.50

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	3.46	0.00	15.06

DEMAND CAPACITY RATIO: (DL + RF)

Pa (kip)	=	18.51	Pnx/1.67 (kip)	=	412.10	Pa/(Pnx/1.67)	=	0.045
			Pny/1.67 (kip)	=	378.02	Pa/(Pny/1.67)	=	0.049
			Pn/1.67 (kip)	=	378.02	Pa/(Pn/1.67)	=	0.049

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 1:

			Dead	Live	Roof
Axial (kip)			3.46	0.00	15.06
Moments	Top	Mx (kip-ft)	0.53	0.00	2.17
		My (kip-ft)	-1.63	0.00	-7.24
	Bot	Mx (kip-ft)	0.36	0.00	1.59
		My (kip-ft)	-1.10	0.00	-2.20

Reverse curvature about X-Axis
Reverse curvature about Y-Axis

CALCULATED PARAMETERS: (DL + RF)

Pa (kip)	=	18.51	Pn/1.67 (kip)	=	378.02
Max (kip-ft)	=	2.70	Mnx/1.67 (kip-ft)	=	150.70
May (kip-ft)	=	-8.87	Mny/1.67 (kip-ft)	=	70.61
Rm	=	1.00			
Cbx	=	2.22			
Cmx	=	0.31	Cmy	=	0.45
Pex (kip)	=	6674.51	Pey (kip)	=	2291.91
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.049
Eq H1-1b: 0.024 + 0.018 + 0.126 = 0.168

Gravity Column Design

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 13:01:12
Steel Code: AISC 360-10 ASD

Story level 2nd, Column Line B-2, Column # 13

Fy (ksi)	= 50.00		Column Size	= W10X33
Orientation (deg.)	= 90.0			

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		9.00	9.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.36	6.48
	Bottom	7.36	6.48

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	6.84	0.00	32.21

DEMAND CAPACITY RATIO: (DL + RF)

Pa (kip) = 39.05	Pnx/1.67 (kip)	=	276.98	Pa/(Pnx/1.67)	=	0.141
	Pny/1.67 (kip)	=	231.85	Pa/(Pny/1.67)	=	0.168
	Pn/1.67 (kip)	=	231.85	Pa/(Pn/1.67)	=	0.168

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 3:

			Dead	Live	Roof
Axial (kip)			6.84	0.00	32.21
Moments	Top	Mx (kip-ft)	0.31	0.00	1.43
		My (kip-ft)	2.19	0.00	10.01
	Bot	Mx (kip-ft)	-2.71	-13.70	0.00
		My (kip-ft)	1.49	0.65	0.86

Single curvature about X-Axis
Reverse curvature about Y-Axis

CALCULATED PARAMETERS: (DL + 0.75LL + 0.75RF)

Pa (kip)	= 30.99	Pn/1.67 (kip)	=	231.85	
Max (kip-ft)	= -12.98	Mnx/1.67 (kip-ft)	=	96.81	
May (kip-ft)	= 9.69	Mny/1.67 (kip-ft)	=	34.93	
Rm	= 1.00				
Cbx	= 1.56				
Cmx	= 0.64	Cmy	=	0.49	
Pex (kip)	= 4196.10	Pey (kip)	=	898.11	
B1x	= 1.00	B1y	=	1.00	

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.134
Eq H1-1b: 0.067 + 0.134 + 0.277 = 0.478

Gravity Column Design

Story level 1st, Column Line B-2, Column # 13

Fy (ksi)	= 50.00	Column Size	= W10X33
Orientation (deg.)	= 90.0		

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	10.67	10.67
K	1	1
Braced Against Joint Translation	Yes	Yes
Column Eccentricity (in) Top	7.36	6.48
Bottom	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	30.52	52.94	36.60

DEMAND CAPACITY RATIO: (DL + 0.75LL + 0.75RF)

Pa (kip) = 97.68	Pnx/1.67 (kip)	= 271.59	Pa/(Pnx/1.67)	=	0.360
	Pny/1.67 (kip)	= 211.52	Pa/(Pny/1.67)	=	0.462
	Pn/1.67 (kip)	= 211.52	Pa/(Pn/1.67)	=	0.462

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 1:

		Dead	Live	Roof
Axial (kip)		30.52	52.94	36.60
Moments	Top Mx (kip-ft)	-2.28	-8.87	0.41
	My (kip-ft)	1.26	0.55	0.72
	Bot Mx (kip-ft)	0.00	0.00	0.00
	My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (DL + 0.75LL + 0.75RF)

Pa (kip)	= 97.68	Pn/1.67 (kip)	= 211.52
Max (kip-ft)	= -8.63	Mnx/1.67 (kip-ft)	= 96.81
May (kip-ft)	= 2.22	Mny/1.67 (kip-ft)	= 34.93
Rm	= 1.00		
Cbx	= 1.67		
Cmx	= 0.60	Cmy	= 0.60
Pex (kip)	= 2985.40	Pey (kip)	= 638.98
B1x	= 1.00	B1y	= 1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.462

Eq H1-1a: 0.462 + 0.079 + 0.056 = 0.597

Gravity Column Design

RAM Steel 15.07.00.17
DataBase: Meehan Cabin 2018-02-19 02/27/18 13:00:46
Building Code: IBC Steel Code: AISC 360-10 ASD

Story level 2nd, Column Line B-3, Column # 14

Fy (ksi)	= 50.00	Column Size	= W10X33
Orientation (deg.)	= 90.0		

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		9.00	9.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	7.36	6.48
	Bottom	7.36	6.48

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	6.84	0.00	32.21

DEMAND CAPACITY RATIO: (DL + RF)

Pa (kip)	=	39.05	Pnx/1.67 (kip)	=	276.98	Pa/(Pnx/1.67)	=	0.141
			Pny/1.67 (kip)	=	231.85	Pa/(Pny/1.67)	=	0.168
			Pn/1.67 (kip)	=	231.85	Pa/(Pn/1.67)	=	0.168

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 2:

			Dead	Live	Roof
Axial (kip)			6.84	0.00	32.21
Moments	Top	Mx (kip-ft)	-0.31	0.00	-1.43
		My (kip-ft)	2.19	0.00	10.01
	Bot	Mx (kip-ft)	2.47	13.58	0.00
		My (kip-ft)	1.49	0.65	0.86

Single curvature about X-Axis
Reverse curvature about Y-Axis

CALCULATED PARAMETERS: (DL + 0.75LL + 0.75RF)

Pa (kip)	=	30.99	Pn/1.67 (kip)	=	231.85
Max (kip-ft)	=	12.65	Mnx/1.67 (kip-ft)	=	96.81
May (kip-ft)	=	9.69	Mny/1.67 (kip-ft)	=	34.93
Rm	=	1.00			
Cbx	=	1.55			
Cmx	=	0.64	Cmy	=	0.49
Pex (kip)	=	4196.10	Pey (kip)	=	898.11
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.134
Eq H1-1b: 0.067 + 0.131 + 0.277 = 0.475

Gravity Column Design

Story level 1st, Column Line B-3, Column # 14

Fy (ksi)	= 50.00	Column Size	= W10X33
Orientation (deg.)	= 90.0		

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	10.67	10.67
K	1	1
Braced Against Joint Translation	Yes	Yes
Column Eccentricity (in) Top	7.36	6.48
Bottom	0.00	0.00

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	30.26	52.59	36.60

DEMAND CAPACITY RATIO: (DL + 0.75LL + 0.75RF)

Pa (kip) = 97.16	Pnx/1.67 (kip)	= 271.59	Pa/(Pnx/1.67)	=	0.358
	Pny/1.67 (kip)	= 211.52	Pa/(Pny/1.67)	=	0.459
	Pn/1.67 (kip)	= 211.52	Pa/(Pn/1.67)	=	0.459

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 1:

		Dead	Live	Roof
Axial (kip)		30.26	52.59	36.60
Moments	Top Mx (kip-ft)	2.08	8.76	-0.41
	My (kip-ft)	1.25	0.55	0.72
	Bot Mx (kip-ft)	0.00	0.00	0.00
	My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (DL + 0.75LL + 0.75RF)

Pa (kip)	= 97.16	Pn/1.67 (kip)	= 211.52
Max (kip-ft)	= 8.35	Mnx/1.67 (kip-ft)	= 96.81
May (kip-ft)	= 2.21	Mny/1.67 (kip-ft)	= 34.93
Rm	= 1.00		
Cbx	= 1.67		
Cmx	= 0.60	Cmy	= 0.60
Pex (kip)	= 2985.40	Pey (kip)	= 638.98
B1x	= 1.00	B1y	= 1.00

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.459
Eq H1-1a: 0.459 + 0.077 + 0.056 = 0.592

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USGS Design Maps Summary Report

User–Specified Input

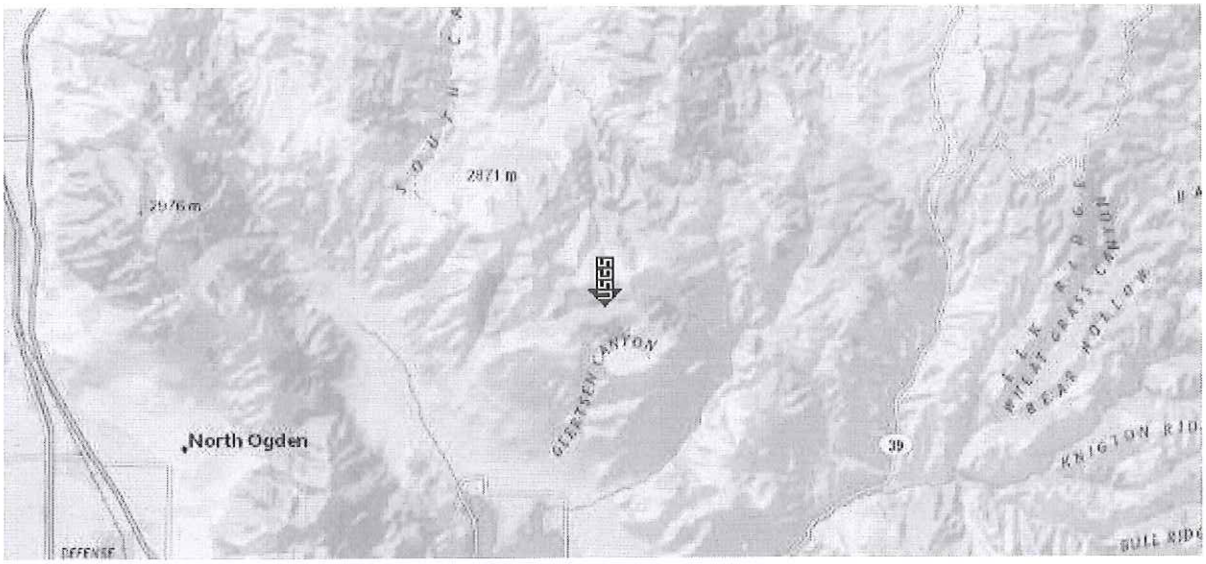
Report Title Meehan Cabin
Tue February 27, 2018 17:54:36 UTC

Building Code Reference Document 2012/2015 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 41.3696°N, 111.7579°W

Site Soil Classification Site Class B – "Rock"

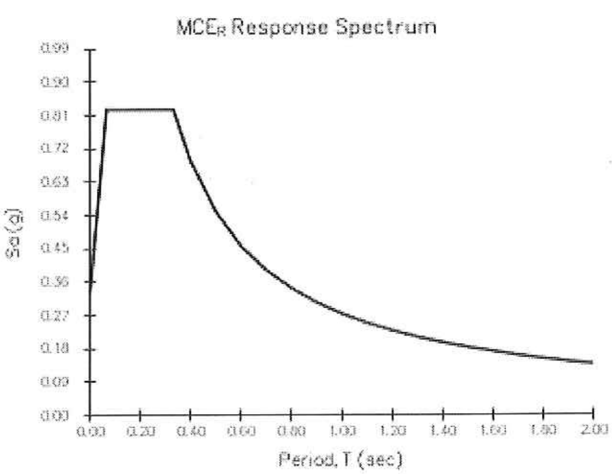
Risk Category I/II/III

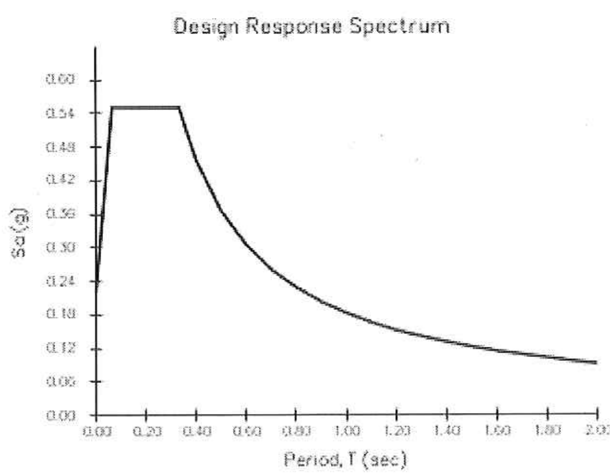


USGS–Provided Output

S_S =	0.826 g	S_{MS} =	0.826 g	S_{DS} =	0.551 g
S_1 =	0.274 g	S_{M1} =	0.274 g	S_{D1} =	0.183 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.





CALDER RICHARDS CONSULTING ENGINEERS	JOB TITLE Meehan Cabin	BY DSM	DATE 10/2017
	SUBJECT Lateral	CHECKED	SHEET OF

Lateral Loads

Seismic:

$C_s = S_{DS}/\left(\frac{R}{I}\right) = 0.551/\left(\frac{3.25}{1.0}\right) = 0.1695$ $\quad C_{s\,max} = \frac{0.183}{0.35\times\left(\frac{3.25}{1}\right)} = 0.161$ ← controls

% Snow = 0.20 + 0.025 (8.81 − 5) = 0.30

Weights:

Roof = (32'×42'×20 psf) + (32'×42'×195 psf×0.30) = 105.50 Kip

Walls = 115'×9'/2×15 psf = 7.76 Kip

← Snow Weight

2ND Floor = (42'×42' − 23.5'×34')×(65+70.2) + 23.5'×34'×50 psf = 170.42 Kip

Walls = 115'×(10.67+9)/2×15 psf = 16.97 K

1ST Floor = (43'×43' − 34'×34')×65 psf + 34'×34'×70 psf = 125.96 Kip

Walls = 136'×10.67'/2×15 psf + 118'×6.5'/2×150 psf = 68.41 Kip

$V = 300.65^K \times 0.161 \times \underbrace{0.7}_{ASD} = \underline{33.88\ Kip}$

Wind: * Local code requires 115 mph. This seemed inadequate so a higher load was used.

V = 150 mph, Exposure C, h ≈ 27', Θ = 0.60°, λ = 1.40, K_{zt} = 1.0

P_{s30_A} = 35.7 psf

P_{s30_C} = 23.7 psf

$P_{s_A} = 1.40 \times 1.0 \times 35.7\text{ psf} \times \underbrace{0.6}_{ASD} = 29.99$ psf

$P_{s_C} = 1.40 \times 1.0 \times 23.7$ psf × 0.6 = 19.91 psf

a = 0.10×24' = 2.4
0.40×27' = 10.8
0.04×24' = 0.96
= 3 ← Controls

a = 0.10×34' = 3.4' ← Controls
0.40×27' = 10.8'
0.04×34' = 1.36'
= 3'

Roof

$V_{long} = (28' \times 9'/2 \times 19.91\ psf) + (6' \times 9'/2 \times 29.99\ psf) = 3.32$ Kip

$V_{short} = (21' \times 9'/2 \times 19.91\ psf) + (3' \times 9'/2 \times 29.99\ psf) = 2.29$ Kip

2ND Floor

$V_{long} = (28' \times (10.67 + 9')/2 \times 19.91\ psf) + (6' \times (10.67 + 9')/2 \times 29.99\ psf) = 7.25$ Kip

$V_{SHort} = (21' \times (10.67 + 9)/2 \times 19.91\ psf) + (3' \times (10.67' + 9')/2 \times 29.99\ psf)$
$+ (10' \times 10.67'/2 \times 19.91\ psf) = 6.06$ Kip

1ST Floor

$V_{long} = (28' \times 10.67/2 \times 19.91\ psf) + (6' \times 10.67/2 \times 29.99\ psf) = 3.93$ Kip

$V_{short} = (31' \times 10.67/2 \times 19.91\ psf) + (3' \times 10.67/2 \times 29.99\ psf) = 3.77$ Kip

$\Sigma\ V_{LONG} = 3.32 + 7.25 = \underline{\underline{10.57\ Kip}}$

$V_{SHORT} = 2.29 + 6.06 = \underline{\underline{8.35\ Kip}}$

$\therefore$ Seismic Governs the design in both directions

Vertical Distribution of Seismic Loads

$\therefore$ See Next Sheet

Seismic Force Vertical Distribution

Project	Meehan Cabin
Designer	DSM
Date	10/5/2017

SDS =	0.551	
SD1 =	0.183	
S1 =	0.274	
I =	1.00	
R =	3.3	
Ct =	0.020	
Hn =	45.00	(ft)
x =	0.75	

T =	Ct * Hn^x =	0.35	sec
k=		1.000	
Cs =	SDS/(R/I) =	0.170	
	SD1/(T(R/I)) =	0.162	need not exceed
	0.044*I*SDS =	0.024	not less than
	0.5*S1/(R/I) =	0.042	not less than
Cs =		0.1620	
Vbase =	Cs * Wtot * 0.7 =	34.64	(kips) ASD

Level	Wx (kips)	Hx (ft)	Wx * Hx^k (kip-feet)	Cvx	Story Force Fx ASD (kips)	Story Shear Vx ASD (kips)
Roof	113.20	19.67	2226.6	0.52	18.03	0.00
2nd	192.17	10.67	2050.5	0.48	16.61	18.03
			0.0	0.00	0.00	34.64
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
			0.0	0.00	0.00	0.00
Totals	305.37		4277.1	1.00	34.64	

Diaphragm Force

Level	Fpx Formula Wx*sum(F)/sum(W) ASD (kips)	Fpx max 0.4*SDS*I*Wx ASD (kips)	Fpx min 0.2*SDS*I*Wx ASD (kips)	Fpx ASD (kips)
Roof	18.03	17.46	8.73	17.46
2nd	21.80	29.65	14.82	21.80
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00

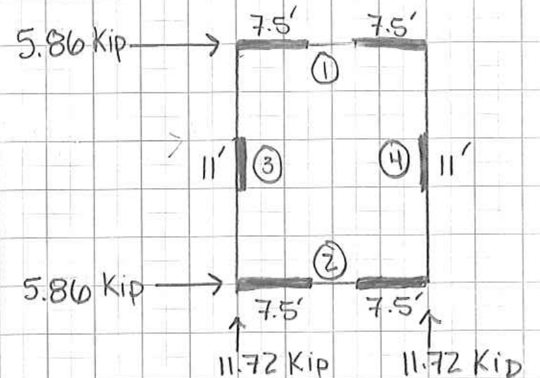
Lateral Systems

Roof:

E/W Direction

Wood Shear Walls $R = 6.5$

$V = 18.03\,\text{Kip ASD} \times \left(\frac{3.25}{6.5}\right) \times 1.3 = \underline{11.72\,\text{Kip ASD}}$ ρ



$v_{1+2} = 5.86\,\text{Kip} / (7.5' + 7.5') = 391\,\text{plf} < 490\,\text{plf}$

∴ 15/32" w/ 8d @ 3" O.C.

N/S Direction

Cable Tension Braces $R = 3.25$

$V = \underline{18.03\,\text{Kip ASD}}$

$V_{3+4} = 18.03\,\text{Kip}/2 \times 1.3 = \underline{11.72\,\text{Kip ASD}}$

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Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,3,6,7]

Wood-based Panels[4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimun Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC — Panel Edge Fastener Spacing (in.) 6: v_s (plf)	6: G_a (kips/in.) OSB	6: G_a (kips/in.) PLY	4: v_s (plf)	4: G_a (kips/in.) OSB	4: G_a (kips/in.) PLY	3: v_s (plf)	3: G_a (kips/in.) OSB	3: G_a (kips/in.) PLY	2: v_s (plf)	2: G_a (kips/in.) OSB	2: G_a (kips/in.) PLY	B WIND — Panel Edge Fastener Spacing (in.) 6: v_w (plf)	4: v_w (plf)	3: v_w (plf)	2: v_w (plf)
Wood Structural Panels - Structural I[4,5]	5/16	1-1/4	Nail (common or galvanized box) 6d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
	3/8[2]	1-3/8	8d	460	19	14	720	24	17	920	30	20	1220	43	24	645	1010	1290	1710
	7/16[2]			510	16	13	790	21	16	1010	27	19	1340	40	24	715	1105	1415	1875
	15/32			560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
	15/32	1-1/2	10d	680	22	16	1020	29	20	1330	36	22	1740	51	28	950	1430	1860	2435
Wood Structural Panels – Sheathing[4,5]	5/16	1-1/4	6d	360	13	9.5	540	18	12	700	24	14	900	37	18	505	755	980	1260
	3/8			400	11	8.5	600	15	11	780	20	13	1020	32	17	560	840	1090	1430
	3/8[2]	1-3/8	8d	440	17	12	640	25	15	820	31	17	1060	45	20	615	895	1150	1485
	7/16[2]			480	15	11	700	22	14	900	28	17	1170	42	21	670	980	1260	1640
	15/32			520	13	10	760	19	13	980	25	15	1280	39	20	730	1065	1370	1790
	15/32	1-1/2	10d	620	22	14	920	30	17	1200	37	19	1540	52	23	870	1290	1680	2155
	19/32			680	19	13	1020	26	16	1330	33	18	1740	48	22	950	1430	1860	2435
Plywood Siding	5/16	1-1/4	Nail (galvanized casing) 6d	280	13		420	16		550	17		720	21		390	590	770	1010
	3/8	1-3/8	8d	320	16		480	18		620	20		820	22		450	670	870	1150
Particleboard Sheathing - (M-S "Exterior Glue" and M-2 "Exterior Glue")	3/8		Nail (common or galvanized box) 6d	240	15		360	17		460	19		600	22		335	505	645	840
	3/8		8d	260	18		380	20		480	21		630	23		365	530	670	880
	1/2			280	18		420	20		540	22		700	24		390	590	755	980
	1/2		10d	370	21		550	23		720	24		920	25		520	770	1010	1290
	5/8			400	21		610	23		790	24		1040	26		560	855	1105	1455
Structural Fiberboard Sheathing	1/2		Nail (galvanized roofing) 11 ga. galv. roofing nail (0.120" x 1-1/2" long x 7/16" head)				340	4.0		460	5.0		520	5.5			475	645	730
	25/32		11 ga. galv. roofing nail (0.120" x 1-3/4" long x 3/8" head)				340	4.0		460	5.0		520	5.5			475	645	730

1. Nominal unit shear values shall be adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls, 4.3.7.2 for particleboard shear walls, and 4.3.7.3 for fiberboard shear walls. See Appendix A for common and box nail dimensions.
2. Shears are permitted to be increased to values shown for 15/32 inch sheathing with same nailing provided (a) studs are spaced a maximum of 16 inches on center, or (b) panels are applied with long dimension across studs.
3. 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 11.3.2A). The Specific Gravity Adjustment Factor shall not be greater than 1.
4. 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 shear walls 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 increased by 1.2.
5. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
6. Where panels are applied on both faces of a shear wall and nail spacing is less than 6" on center on either side, panel joints shall be offset to fall on different framing members. Alternatively, the width of the nailed face of framing members shall be 3" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.
7. Galvanized nails shall be hot-dipped or tumbled.

÷ Values By 2.0 for ASD

CALDER RICHARDS
CONSULTING ENGINEERS

JOB TITLE Meehan Cabin | BY DSM | DATE 01/2018
SUBJECT Lateral | CHECKED | SHEET OF

Check Diaphragms of Roof level

$v = \left(17.46 \text{ Kip ASD} \times \left(\frac{3.25}{6.5}\right)\right) = 8.88^K / 2 \text{ sides} = 4.44^K / 23' = 193 \text{ plf} < 240 \text{ plf}$

$v = 17.46 \text{ Kip ASD} / 2 \text{ sides} = 8.73^K / 42' = 208 \text{ plf} < 320 \text{ plf}$

∴ 3/4" Sheathing w/
10d @ 6" O.C.

Check Diaphragms of Upper Level

25% Increase For Irregularity

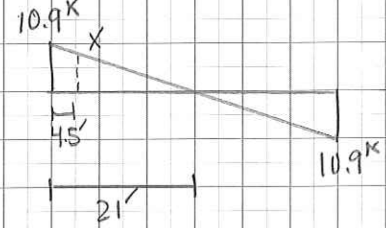
$V = 21.80^K / 2 \times 12.83' = 850 \text{ plf} \times 1.25 = 1062 \text{ plf}$

22 gauge, W2, 5" N.W. Concrete, Span = 6'-0", 36/3 ⇒ q = 2152 plf

∴ 22 Gauge W2, 36/3 Welds
w/ BP @ 36" O.C.

Check Wood Diaphram of Upper Level

$V = 21.80^K$



$\frac{10.9^K}{21'} = \frac{X}{(21' - 4.5')}$ $X = 8.56^K$

$v = 8.56^K / 35' = 245 \text{ plf} \approx 240 \text{ plf}$

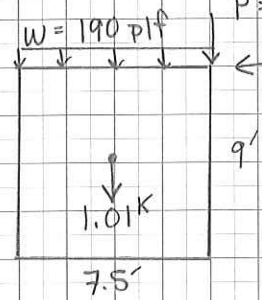
∴ 3/4" Sheathing w/
10 d @ 6" O.C.

CALDER RICHARDS
CONSULTING ENGINEERS

JOB TITLE Meehan Cabin	BY DSM	DATE 01/2018	
SUBJECT	CHECKED	SHEET	OF

Check Overturn of Wood Shear Wall

$P = 0.71^K + 0.27^K = 0.98^K$



DL Red = $0.6 - 0.14(0.551) = 0.52$

$M_{OT} = 2.93^K \times 9' = 26.37^{K'}$

$M_{RES} = (1.01^K \times 7.5'/2) + (1.43^K \times 7.5'/2) + (0.98^K \times 7.5) = 16.5^{K'} \times 0.52 = 8.58^{K'}$

$M_{NET} = 26.37^{K'} - 8.58^{K'} = 17.79^{K'}$

$T = C = M/d = 17.79^{K'} / 7.5' = 2.37^K$

∴ Provide Simpson MSTC40 w/ (32) 16d Nails. $T = 3.08^K$

∴ Where 9 1/4" beam occurs below use (2) CS14 Straps.

End Length Req = 15"
Actual End Length = 9" ⟹ 60% Reduction in tension

$T = 2{,}490 \times 0.60 \times 2$ Straps $= 2.99^K$

48

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

Unblocked Wood Structural Panel Diaphragms[1,2,3,4]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC — 6 in. Nail Spacing at diaphragm boundaries and supporting members — Case 1 Parallel — v_s (plf)	Case 1 — G_a (kips/in.) OSB	Case 1 — G_a (kips/in.) PLY	Cases 2,3,4,5,6 Perp — v_s (plf)	Cases 2,3,4,5,6 — G_a (kips/in.) OSB	Cases 2,3,4,5,6 — G_a (kips/in.) PLY	B WIND — 6 in. Nail Spacing at diaphragm boundaries and supporting members — Case 1 — v_w (plf)	Cases 2,3,4,5,6 — v_w (plf)
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350
				3	370	7.0	6.0	280	4.5	4.0	520	390
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505
				3	530	7.5	6.0	400	5.0	4.0	740	560
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600
				3	640	12	9.0	480	8.0	6.0	895	670
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310
				3	340	7.0	5.5	250	5.0	3.5	475	350
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350
				3	370	6.0	4.5	280	4.0	3.0	520	390
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450
				3	480	7.5	5.5	360	5.0	3.5	670	505
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475
				3	510	7.0	5.5	380	4.5	3.5	715	530
			15/32	2	480	7.5	5.5	360	5.0	4.0	670	505 — 253
				3	530	6.5	5.0	400	4.0	3.5	740	560
	10d	1-1/2 5/8"	15/32	2	510	15	9.0	380	10	6.0	715	530
				3	580	12	8.0	430	8.0	5.5	810	600
			19/32	2	570	13	8.5	430	8.5	5.5	800	600
				3	640	10	7.5	480	7.0	5.0	895	670

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 11.3.2A). 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 increased 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

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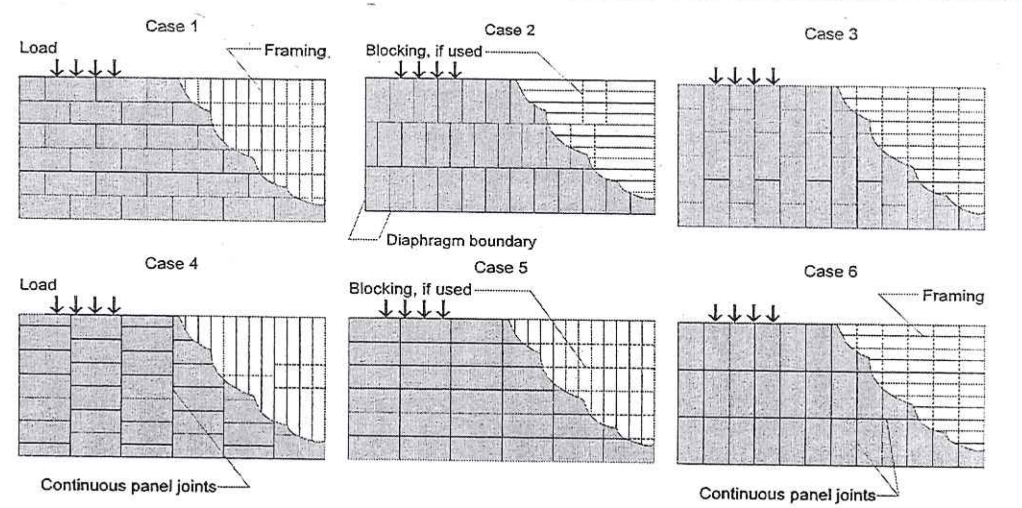
Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms

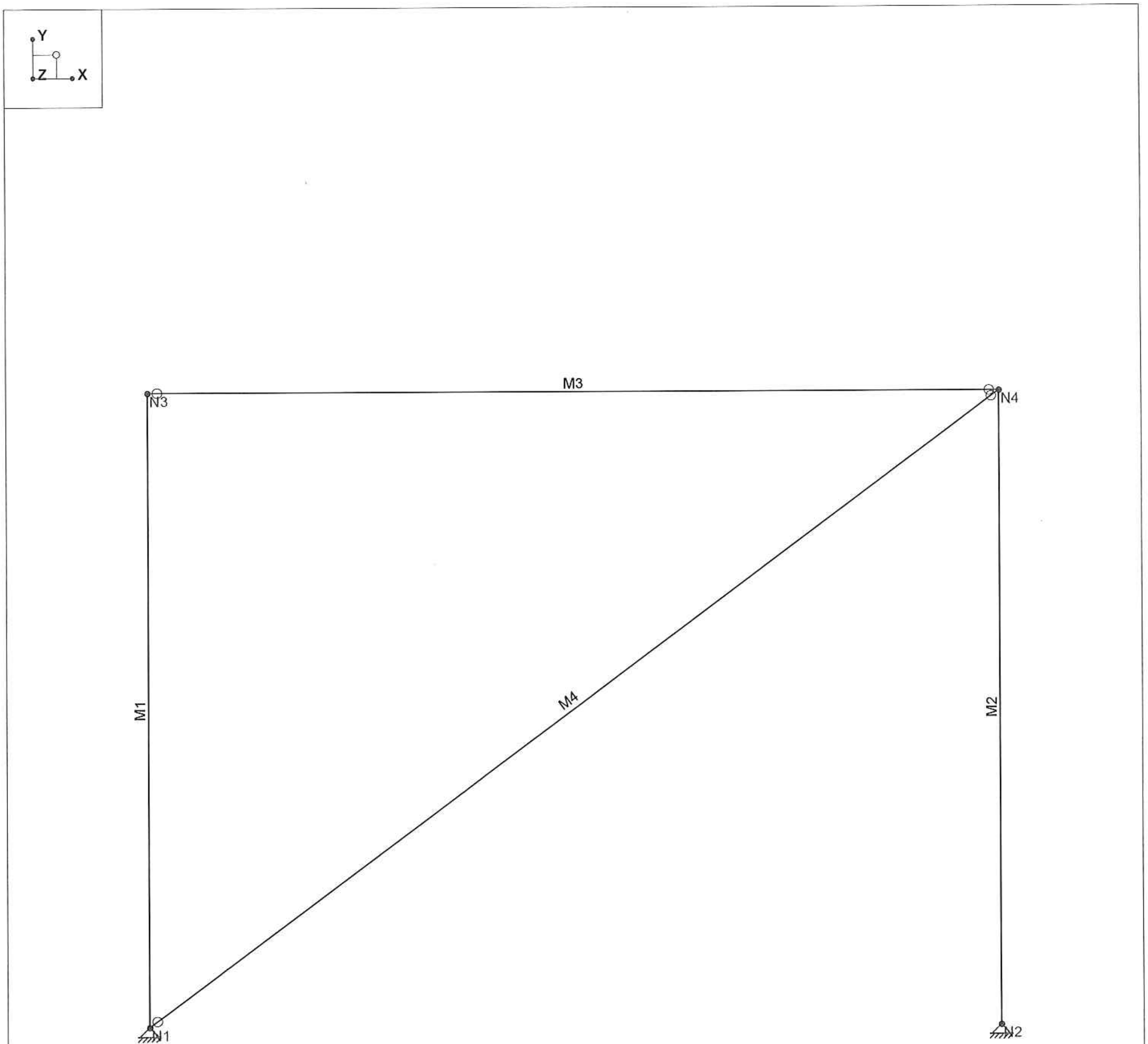
Blocked Wood Structural Panel Diaphragms[1,2,3,4]

×2 For ASD

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: 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			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): 6	4	2-1/2	2
					Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4): 6			6			4			3			Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4): 6	6	4	3
					v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
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 11.3.2A). 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 increased 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.





Envelope Only Solution

		SK - 1
		Jan 17, 2018 at 5:08 PM
		Braced Frame.r2d

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design R...	A [in2]	I (90,27...	I (0,180...
1	Columns	HSS5.5x...	Column	SquareTube	A500 Gr.B Rect	Typical	3.63	17	17
2	Brace	1" Bar	None	None	A36 Gr.36	Typical	.785	.049	.049

Wood Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [in2]	I (90,270) ...	I (0,180) [i...
1	Beam	5.125X12FS	Beam	None	24F-1.8E D...	Typical	61.5	134.611	738

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Temp [F]
1	N1	0	0	0
2	N2	11	0	0
3	N3	0	8.167	0
4	N4	11	8.167	0

Hot Rolled Steel Design Parameters

	Label	Shape	Length[ft]	Lb-out[ft]	Lb-in[ft]	Lcomp top[ft]	Lcomp bot[ft]	L-torqu...	K-out	K-in	Cb	Function
1	M1	Columns	8.167									Lateral
2	M2	Columns	8.167									Lateral
3	M4	Brace	13.7									Lateral

Member Advanced Data

	Label	I Release	J Release	I Offset[in]	J Offset[in]	T/C Only	Physical	TOM	Inactive
1	M1						Yes		
2	M2						Yes		
3	M3	PIN	PIN				Yes		
4	M4	PIN	PIN				Yes		

Member Primary Data

	Label	I Joint	J Joint	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N3		Columns	Column	SquareTube	A500 Gr.B ...	Typical
2	M2	N2	N4		Columns	Column	SquareTube	A500 Gr.B ...	Typical
3	M3	N3	N4		Beam	Beam	None	24F-1.8E D...	Typical
4	M4	N1	N4		Brace	None	None	A36 Gr.36	Typical

Joint Loads and Enforced Displacements (BLC 1 : DL)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s^2*ft)]
1	N3	L	Y	-3
2	N4	L	Y	-3

Joint Loads and Enforced Displacements (BLC 2 : SL)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s^2*ft)]
1	N3	L	Y	-29.9
2	N4	L	Y	-29.9

Member Distributed Loads (BLC 1 : DL)

	Member Label	Direction	Start Magnitude[k/ft,F]	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M3	Y	-.135	-.135	0	0

Member Distributed Loads (BLC 2 : SL)

	Member Label	Direction	Start Magnitude[k/ft,F]	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M3	Y	-1.316	-1.316	0	0

Member Distributed Loads (BLC 3 : EL)

	Member Label	Direction	Start Magnitude[k/ft,F]	End Magnitude[k/ft,F]	Start Location[ft,%]	End Location[ft,%]
1	M3	X	1.065	1.065	0	0

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distribut...
1	DL	DL			2		1
2	SL	SL			2		1
3	EL	EL					1

Load Combinations

	Description	S...	PDelta	S...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	DL	Yes	Y		DL	1																		
2	DL + SL	Yes	Y		DL	1	SL	1																
3	DL + 0.7EL	Yes	Y		DL	1	EL	1																
4	DL + 0.525EL + 0.75...	Yes	Y		DL	1	EL	.75	SL	.75														
5	0.6DL + 0.7EL	Yes	Y		DL	.6	EL	1																

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Moment [k-ft]	LC
1	N1	max	0	1	40.846	2	0	1
2		min	-11.756	3	-6.465	5	0	1
3	N2	max	.099	4	40.918	2	0	1
4		min	0	1	3.743	1	0	1
5	Totals:	max	0	2	81.764	2		
6		min	-11.72	5	4.491	5		

Envelope Joint Displacements

	Joint		X [in]	LC	Y [in]	LC	Rotation [rad]	LC
1	N1	max	0	3	0	5	-3.306e-5	1
2		min	0	1	0	2	-1.865e-3	3
3	N2	max	0	1	0	1	-3.305e-5	1
4		min	0	4	0	2	-1.789e-3	3
5	N3	max	.183	3	-.003	5	-3.306e-5	1
6		min	.003	1	-.048	2	-1.865e-3	3
7	N4	max	.175	3	-.004	1	-3.305e-5	1
8		min	.003	1	-.048	2	-1.789e-3	3

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

	Member	Shape	Code Check	Loc[..	LC	Shea...	Loc[..	LC	Pnc/o...	Pnt/om...	Mn/om...	Cb	Eqn
1	M1	HSS5.5x5..	.469	0	2	.000	0	1	87.103	99.988	16.347	1	H1-1a
2	M2	HSS5.5x5..	.470	0	2	.000	0	1	87.103	99.988	16.347	1	H1-1a
3	M4	1" Bar	.865	0	3	.000	0	1	.273	16.931	.282	1	H1-1a

< 1.0 ∴ OK

Envelope Wood Code Checks

	Member	Shape	Code Check	Loc[ft]	LC	Shear C...	Loc[ft]	LC	Fc'[ksi]	Ft'[ksi]	Fb'[ksi]	Fv'[ksi]	Eqn
1	M3	5.125X12FS	.783	5.5	2	.639	0	2	.947	1.265	2.733	.305	3.9-3

CALDER RICHARDS
CONSULTING ENGINEERS

JOB TITLE Meehan Cabin | BY DSM | DATE 01/2018
SUBJECT | CHECKED | SHEET | OF

Connection of Wood Beam to Joint

5 1/8 x 12 GLB w/ (3) 1" Thru Bolts in double shear w/ 1/4" Steel Plates

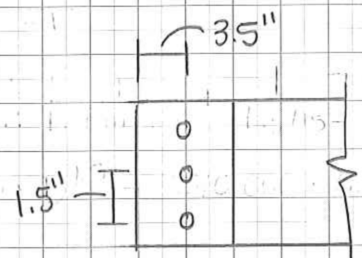
NDS: (1) 1" Bolt ⇒ $Z = 5510$ lbs

$Z' = Z \times C_D \times C_M \times C_t \times C_g \times C_\Delta \times C_{eg} \times C_{di} \times C_{tn}$

$C_D = 1.6$
$C_m = 1.0$
$C_t = 1.0$
$C_g = 1.0$
$C_\Delta =$ See Below
$C_{eg} = 1.0$
$C_{di} = 1.0$
$C_{tn} = 1.0$

$$\left.\begin{array}{l}\text{Min End Distance} = 3.5 \times D = 3.5'' \\ \text{Min Spacing of Row} = 3D = 3'' \\ \text{Min Spacing Btwn Rows} = 1.5D = 1.5''\end{array}\right\} C_\Delta = 0.5$$

$Z' = 5510 \times 1.6 \times 0.5 = 4{,}408 \text{ lbs} \times 3 = 13.2^k > 11.72^k$



Weld of Shear Plate to Top Plate

$11.72^k = 0.928 \times 3 \times \ell_{weld} = 4.21''$ ∴ 5" of 3/16" Weld

Weld of Top Plate to Gusset Plate

∴ 5" of 3/16" Weld

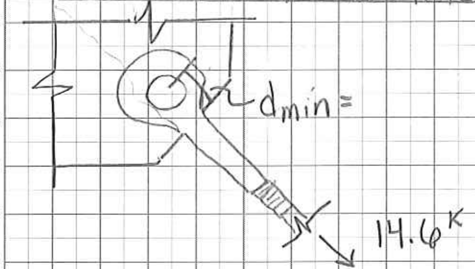
Check Clevise

Axial Load on Rod = 14.6^k $D = 1''$ ∴ #3 Clevis Required

CALDER RICHARDS
CONSULTING ENGINEERS

JOB TITLE Meehan Cabin | BY DSM | DATE 01/2018
SUBJECT | CHECKED | SHEET | OF

Check Gusset Plate thickness



Min Edge Distance = $1\frac{1}{4} \times 1\frac{3}{4} = 2.19''$ ← AISC Table J3.4

∴ Use 2.5" min.

Bearing Strength:

$R_n = 1.2\, l_c\, t\, F_u \leq 2.4 D t F_u$

$l_c = d_{min} - 1\frac{3}{4}/2 = 2.5'' - (1\frac{3}{4})/2 = 1.625''$

$F_u = 58$ Ksi

$D = 1\frac{3}{4}''$

Substitute actual load to determine t_{req}.

$14.6^K = \dfrac{1.2 \times 1.625'' \times t_{req} \times 58 \text{ Ksi}}{2.0}$ $\quad t_{req} = 0.26''$

Check Upper limit:

$2.4 \times 1.75 \times 0.26 \times 58 = 63.34^K \geq 14.6^K$ ∴ OK

∴ Provide 1/2" Gusset Plate w/ Min Edge Distance to Hole of 2.5"

Drift

RAM Structural System 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/23/18 16:32:41
Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: 1st

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosSnowLoad	RAMUSER
E1	Seismic	EQ_ASCE710_X_+E_F
E2	Seismic	EQ_ASCE710_X_-E_F
E3	Seismic	EQ_ASCE710_Y_+E_F
E4	Seismic	EQ_ASCE710_Y_-E_F

RESULTS:

Location (ft): (19.830, 21.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
2nd	D	0.0014	0.0000	0.0014	0.0000	0.0000	0.0000
	Lp	0.0024	-0.0000	0.0024	-0.0000	0.0000	0.0000
	Sp	0.0017	-0.0001	0.0017	-0.0001	0.0000	0.0000
	E1	0.3710	0.0009	0.3710	0.0009	0.0034	0.0000
	E2	0.3710	-0.0009	0.3710	-0.0009	0.0034	0.0000
	E3	-0.0000	0.4189	-0.0000	0.4189	0.0000	0.0039
	E4	0.0000	0.4207	0.0000	0.4207	0.0000	0.0039
1st	D	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Lp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Sp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Location (ft): (0.000, 42.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
2nd	D	0.0014	-0.0000	0.0014	-0.0000	0.0000	0.0000

Drift

Story	LdC	Displacement		Story Drift		Drift Ratio	
	Lp	0.0024	-0.0000	0.0024	-0.0000	0.0000	0.0000
	Sp	0.0016	-0.0001	0.0016	-0.0001	0.0000	0.0000
	E1	0.3904	0.0192	0.3904	0.0192	0.0036	0.0002
	E2	0.3516	-0.0192	0.3516	-0.0192	0.0033	0.0002
	E3	-0.0106	0.4089	-0.0106	0.4089	0.0001	0.0038
	E4	0.0282	0.4473	0.0282	0.4473	0.0003	0.0041
1st	D	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Lp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Sp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Location (ft): (42.000, 42.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
2nd	D	0.0014	0.0000	0.0014	0.0000	0.0000	0.0000
	Lp	0.0024	0.0000	0.0024	0.0000	0.0000	0.0000
	Sp	0.0016	-0.0001	0.0016	-0.0001	0.0000	0.0000
	E1	0.3904	-0.0196	0.3904	-0.0196	0.0036	0.0002
	E2	0.3516	0.0196	0.3516	0.0196	0.0033	0.0002
	E3	-0.0106	0.4301	-0.0106	0.4301	0.0001	0.0040
	E4	0.0282	0.3909	0.0282	0.3909	0.0003	0.0036
1st	D	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Lp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Sp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Location (ft): (0.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
2nd	D	0.0014	-0.0000	0.0014	-0.0000	0.0000	0.0000
	Lp	0.0024	-0.0000	0.0024	-0.0000	0.0000	0.0000
	Sp	0.0017	-0.0001	0.0017	-0.0001	0.0000	0.0000
	E1	0.3516	0.0192	0.3516	0.0192	0.0033	0.0002

Drift

Story	LdC	Displacement		Story Drift		Drift Ratio	
	E2	0.3904	-0.0192	0.3904	-0.0192	0.0036	0.0002
	E3	0.0106	0.4089	0.0106	0.4089	0.0001	0.0038
	E4	-0.0282	0.4473	-0.0282	0.4473	0.0003	0.0041
1st	D	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Lp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Sp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Location (ft): (42.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
2nd	D	0.0014	0.0000	0.0014	0.0000	0.0000	0.0000
	Lp	0.0024	0.0000	0.0024	0.0000	0.0000	0.0000
	Sp	0.0017	-0.0001	0.0017	-0.0001	0.0000	0.0000
	E1	0.3516	-0.0196	0.3516	-0.0196	0.0033	0.0002
	E2	0.3904	0.0196	0.3904	0.0196	0.0036	0.0002
	E3	0.0106	0.4301	0.0106	0.4301	0.0001	0.0040
	E4	-0.0282	0.3909	-0.0282	0.3909	0.0003	0.0036
1st	D	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Lp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Sp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

CALDER RICHARDS
CONSULTING ENGINEERS

JOB TITLE Meehan Cabin | BY DSM | DATE 02/2018
SUBJECT Drift Check | CHECKED | SHEET | OF

Drift

$\Delta_{Allow} = \frac{0.020 \times 9' \times 12}{4.0} \times 1.0 = 0.54"$ ← Story Drift of Moment Frame Deck Level

*Note: Worst Case Cd = 4.0 from Intermediate moment frame used while R = 3.25 from tension brace was used.

Center of Mass = 19.83, 21.00, 19.67 $\Delta x_{max} = 0.3710$ $\Delta y_{max} = 0.4207$
Corner 1 = 0.00, 42.00, 19.67 $\Delta x_{max} = 0.3904$ $\Delta y_{max} = 0.4473$
Corner 2 = 42.00, 42.00, 19.67 $\Delta x_{max} = 0.3904$ $\Delta y_{max} = 0.4301$
Corner 3 = 0.00, 0.00, 19.67 $\Delta x_{max} = 0.3904$ $\Delta y_{max} = 0.4473$
Corner 4 = 42.00, 0.00, 19.67 $\Delta x_{max} = 0.3904$ $\Delta y_{max} = 0.4301$

∴ Drift OKay

Check Torsion

$\Delta_{y\,max} = 0.4473$

$\Delta_{y\,AVERAGE} = \frac{0.4473 + 0.4301}{2} = 0.4387 \times 1.2 = 0.5264" > 0.4473"$

∴ No Torsion in Y Direction.

∴ By Inspection no torsion in X Direction

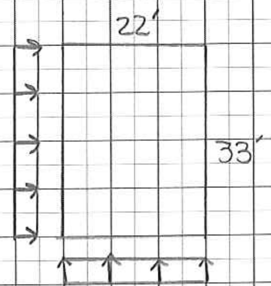
Additional Irregularities

Horizontal - Out-of-plane Offset Irregularity

- Increase diaphragm collector forces 25%
- Check elements supporting discontinuous w/ Ω_0
- 3D Model

Chord Forces @ Roof

$w = 11.72^k / 33' = 355$ plf



$w = 11.72^k / 22' = 533$ plf

$M = wl^2/8 = 355 \times 33^2/8 = 48.32^{k \cdot '}$

$T/C = 48.32^{k \cdot '} / 22' = 2.20^k$ ⇐ Controls

$M = wl^2/8 = 533 \times 22^2/8 = 32.25^{k \cdot '}$

$T/C = 32.25^{k \cdot '} / 33' = 0.98^k$

∴ Splice single top plate w/ CS14 × 3'-0"
w/ (30) 8d Nails on each end min

Frame Story Shears

RAM Structural System RAM Frame 15.07.00.17
Bentley DataBase: Meehan Cabin 2018-02-19 02/27/18 11:58:39

CRITICIAL:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0100
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

Frame #1

Load Case: D DeadLoad RAMUSER

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	0.00	0.00
1st	0.00	0.00	-0.03	-0.03

Load Case: Lp PosLiveLoad RAMUSER

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	0.00	0.00
1st	0.00	0.00	0.03	0.03

Load Case: Sp PosSnowLoad RAMUSER

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	0.00	0.00
1st	0.00	0.00	0.45	0.45

Load Case: E1 Seismic EQ_ASCE710_X_+E_F

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	1.49	1.49
1st	0.01	0.01	4.18	2.69

Load Case: E2 Seismic EQ_ASCE710_X_-E_F

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	-1.49	-1.49

Frame Story Shears

1st	0.01	0.01	-4.23	-2.74

Load Case: E3 **Seismic EQ_ASCE710_Y_+E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	31.80	31.80
1st	0.00	0.00	9.48	-22.32

Load Case: E4 **Seismic EQ_ASCE710_Y_-E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	34.78	34.78
1st	0.00	0.00	17.90	-16.89

Frame #2

Load Case: D **DeadLoad RAMUSER**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	0.00	0.00
1st	-0.01	-0.01	0.00	0.00

Load Case: Lp **PosLiveLoad RAMUSER**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	0.00	0.00
1st	0.01	0.01	0.00	0.00

Load Case: Sp **PosSnowLoad RAMUSER**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.01	0.01	0.00	0.00
1st	0.16	0.15	0.00	0.00

Load Case: E1 **Seismic EQ_ASCE710_X_+E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	30.48	30.48	0.00	0.00
1st	40.20	9.73	0.00	0.00

Load Case: E2 **Seismic EQ_ASCE710_X_-E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips

	Shear-X	Change-X	Shear-Y	Change-Y
2nd	33.85	33.85	0.00	0.00
1st	46.47	12.62	0.00	0.00

Load Case: E3 **Seismic** **EQ_ASCE710_Y_+E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.93	0.93	0.00	0.00
1st	25.08	24.15	0.00	0.00

Load Case: E4 **Seismic** **EQ_ASCE710_Y_-E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	-2.44	-2.44	0.00	0.00
1st	18.82	21.26	0.01	0.01

Frame #3

Load Case: D **DeadLoad** **RAMUSER**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	0.00	0.00
1st	0.00	0.00	0.00	0.00

Load Case: Lp **PosLiveLoad** **RAMUSER**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	0.00	0.00
1st	0.00	0.00	0.00	0.00

Load Case: Sp **PosSnowLoad** **RAMUSER**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	0.00	0.00
1st	0.00	0.00	0.00	0.00

Load Case: E1 **Seismic** **EQ_ASCE710_X_+E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	0.00	0.00	-1.49	-1.49
1st	0.00	0.00	0.40	1.89

Load Case: E2 **Seismic** **EQ_ASCE710_X_-E_F**

Level	Shear-X	Change-X	Shear-Y	Change-Y

	kips	kips	kips	kips
2nd	0.00	0.00	1.49	1.49
1st	0.00	0.00	-0.40	-1.89

Load Case: E3 Seismic EQ_ASCE710_Y_+E_F

Level	Shear-X	Change-X	Shear-Y	Change-Y
	kips	kips	kips	kips
2nd	0.00	0.00	32.69	32.69
1st	0.00	0.00	-9.27	-41.96

Load Case: E4 Seismic EQ_ASCE710_Y_-E_F

Level	Shear-X	Change-X	Shear-Y	Change-Y
	kips	kips	kips	kips
2nd	0.00	0.00	29.72	29.72
1st	0.00	0.00	-8.47	-38.19

Frame #4

Load Case: D DeadLoad RAMUSER

Level	Shear-X	Change-X	Shear-Y	Change-Y
	kips	kips	kips	kips
2nd	0.00	0.00	0.00	0.00
1st	0.01	0.01	0.00	0.00

Load Case: Lp PosLiveLoad RAMUSER

Level	Shear-X	Change-X	Shear-Y	Change-Y
	kips	kips	kips	kips
2nd	0.00	0.00	0.00	0.00
1st	-0.01	-0.01	0.00	0.00

Load Case: Sp PosSnowLoad RAMUSER

Level	Shear-X	Change-X	Shear-Y	Change-Y
	kips	kips	kips	kips
2nd	0.00	0.00	0.00	0.00
1st	-0.16	-0.16	0.00	0.00

Load Case: E1 Seismic EQ_ASCE710_X_+E_F

Level	Shear-X	Change-X	Shear-Y	Change-Y
	kips	kips	kips	kips
2nd	33.85	33.85	0.00	0.00
1st	46.43	12.59	0.00	0.00

Frame Story Shears

Load Case: E2 **Seismic EQ_ASCE710_X_-E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	30.48	30.48	0.00	0.00
1st	40.17	9.69	0.00	0.00

Load Case: E3 **Seismic EQ_ASCE710_Y_+E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	-0.93	-0.93	0.00	0.00
1st	-25.08	-24.15	0.00	0.00

Load Case: E4 **Seismic EQ_ASCE710_Y_-E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
2nd	2.44	2.44	0.00	0.00
1st	-18.82	-21.26	0.01	0.01

Frame #5

Load Case: D **DeadLoad RAMUSER**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
1st	0.00	0.00	0.03	0.03

Load Case: Lp **PosLiveLoad RAMUSER**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
1st	0.00	0.00	-0.03	-0.03

Load Case: Sp **PosSnowLoad RAMUSER**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
1st	0.00	0.00	-0.45	-0.45

Load Case: E1 **Seismic EQ_ASCE710_X_+E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
1st	0.01	0.01	-4.59	-4.59

Load Case: E2 **Seismic EQ_ASCE710_X_-E_F**

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips

Frame Story Shears

1st	0.01	0.01	4.63	4.63

Load Case: E3 Seismic EQ_ASCE710_Y_+E_F

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
1st	0.00	0.00	86.42	86.42

Load Case: E4 Seismic EQ_ASCE710_Y_-E_F

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
1st	0.00	0.00	77.21	77.21

Member Code Check

RAM Frame 15.07.00.17
DataBase: Meehan Cabin 2018-02-19 — 02/27/18 11:56:28
Building Code: IBC — Steel Code: AISC360-10 ASD

RAM Structural System
Bentley

COLUMN INFORMATION:

Story Level = 2nd — Frame Number = 1 — Column Number = 2
Fy (ksi) = 50.00
Column Size = W10X49

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	9.00	9.00
Lu for Bending (ft)	9.00	9.00
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D - 0.910 E4

Shear	Top	Vmajor (kip)	-16.04
		Vminor (kip)	-0.00
Shear	Bot.	Vmajor (kip)	-16.04
		Vminor (kip)	-0.00

SHEAR CHECK:

Vax (kip)	= -16.04	Vnx/1.50 (kip)	=	68.00	Vax/(Vnx/1.50)	=	0.236
Vay (kip)	= -0.00	Vny/1.67 (kip)	=	201.20	Vay/(Vny/1.67)	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Sp - 0.683 E4

AXIAL CHECK:

Pa (kip)	= 16.27	Pnx/1.67 (kip)	=	412.10	Pa/(Pnx/1.67)	=	0.039
		Pny/1.67 (kip)	=	378.02	Pa/(Pny/1.67)	=	0.043
		Pn/1.67 (kip)	=	378.02	Pa/(Pn/1.67)	=	0.043

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D - 0.910 E4

Axial		Load (kip)	12.23
Moment	Top	Mmajor (kip-ft)	57.87
		Mminor (kip-ft)	0.00
Moment	Bot.	Mmajor (kip-ft)	-78.16
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip)	=	12.23	Pnx/1.67 (kip)	=	412.10
			Pny/1.67 (kip)	=	378.02
Max (kip-ft)	=	-78.16	Mnx/1.67 (kip-ft)	=	150.70
May (kip-ft)	=	0.00	Mny/1.67 (kip-ft)	=	70.61
KL/Rx	=	24.85	KL/Ry	=	42.41
			Mcx (kip-ft)	=	150.69
Cbx	=	2.22			

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.030
Eq H1-2: 0.048 + 0.055 = 0.103
Eq H1-1b Per H1.3: 0.015 + 0.519 + 0.000 = 0.533

68

Member Code Check

RAM Frame 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:56:28
Steel Code: AISC360-10 ASD

COLUMN INFORMATION:

Story Level	= 2nd	Frame Number	= 1	Column Number	= 3
Fy (ksi)	= 50.00				
Column Size	= W10X49				

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	9.00	9.00
Lu for Bending (ft)	9.00	9.00
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D + 0.910 E4

Shear	Top	Vmajor (kip)	16.04
		Vminor (kip)	-0.00
Shear	Bot.	Vmajor (kip)	16.04
		Vminor (kip)	-0.00

SHEAR CHECK:

Vax (kip) =	16.04	Vnx/1.50 (kip) =	68.00	Vax/(Vnx/1.50) =	0.236
Vay (kip) =	-0.00	Vny/1.67 (kip) =	201.20	Vay/(Vny/1.67) =	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 0.683 E4

AXIAL CHECK:

Pa (kip) =	16.28	Pnx/1.67 (kip) =	412.10	Pa/(Pnx/1.67) =	0.039
		Pny/1.67 (kip) =	378.02	Pa/(Pny/1.67) =	0.043
		Pn/1.67 (kip) =	378.02	Pa/(Pn/1.67) =	0.043

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D + 0.910 E4

Axial		Load (kip)	12.23
Moment	Top	Mmajor (kip-ft)	-57.87
		Mminor (kip-ft)	0.00
Moment	Bot.	Mmajor (kip-ft)	78.15
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip) =	12.23	Pnx/1.67 (kip) =	412.10
		Pny/1.67 (kip) =	378.02
Max (kip-ft) =	78.15	Mnx/1.67 (kip-ft) =	150.70
May (kip-ft) =	0.00	Mny/1.67 (kip-ft) =	70.61
KL/Rx =	24.85	KL/Ry =	42.41
		Mcx (kip-ft) =	150.69
Cbx =	2.22		

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.030
Eq H1-2: 0.048 + 0.055 = 0.103
Eq H1-1b Per H1.3: 0.015 + 0.519 + 0.000 = 0.533

Member Code Check

BEAM INFORMATION:

Story Level = 2nd   Frame Number = 1   Beam Number = 12
Fy (ksi) = 50.00
Beam Size = W12X35
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	12.83	0.92
Lu for Bending (ft)	12.83	0.92
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 0.683 E4

Segment distance (ft) i - end: 11.92
j - end: 12.83

SHEAR CHECK:

Vax (kip) =	-12.34	Vnx/1.50 (kip) =	75.00	Vax/(Vnx/1.50) =	0.165
Vay (kip) =	0.00	Vny/1.67 (kip) =	122.56	Vay/(Vny/1.67) =	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.000 D

Segment distance (ft) i - end: 0.92
j - end: 6.50

AXIAL CHECK:

Pa (kip) =	0.00	Pnx/1.67 (kip) =	308.38	Pa/(Pnx/1.67) =	0.000
		Pny/1.67 (kip) =	308.38	Pa/(Pny/1.67) =	0.000
		Pn/1.67 (kip) =	308.38	Pa/(Pn/1.67) =	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.077 D - 0.910 E4

Segment distance (ft) i - end: 0.00
j - end: 0.92

CALCULATED PARAMETERS:

Pa (kip) =	0.00	Pn/1.67 (kip) =	308.38
Max (kip-ft) =	-61.51	Mnx/1.67 (kip-ft) =	127.74
May (kip-ft) =	0.00	Mny/1.67 (kip-ft) =	28.69
Cbx =	1.04		

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.000
Eq H1-1b: 0.000 + 0.481 + 0.000 = 0.481

Member Code Check

COLUMN INFORMATION:

Story Level = 2nd   Frame Number = 2   Column Number = 11
Fy (ksi) = 50.00
Column Size = W10X49

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	9.00	9.00
Lu for Bending (ft)	9.00	9.00
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D + 0.910 E2

Shear	Top	Vmajor (kip)	13.41
		Vminor (kip)	0.00
Shear	Bot.	Vmajor (kip)	13.41
		Vminor (kip)	0.00

SHEAR CHECK:

Vax (kip) = 13.41	Vnx/1.50 (kip) = 68.00	Vax/(Vnx/1.50) = 0.197
Vay (kip) = 0.00	Vny/1.67 (kip) = 201.20	Vay/(Vny/1.67) = 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Sp + 0.683 E2

AXIAL CHECK:

Pa (kip) = 30.70	Pnx/1.67 (kip) = 412.10	Pa/(Pnx/1.67) = 0.074
	Pny/1.67 (kip) = 378.02	Pa/(Pny/1.67) = 0.081
	Pn/1.67 (kip) = 378.02	Pa/(Pn/1.67) = 0.081

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D + 0.910 E2

Axial		Load (kip)	14.87
Moment	Top	Mmajor (kip-ft)	-50.66
		Mminor (kip-ft)	-0.00
Moment	Bot.	Mmajor (kip-ft)	56.09
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip) = 14.87	Pnx/1.67 (kip) = 412.10
	Pny/1.67 (kip) = 378.02
Max (kip-ft) = 56.09	Mnx/1.67 (kip-ft) = 150.70
May (kip-ft) = 0.00	Mny/1.67 (kip-ft) = 70.61
KL/Rx = 24.85	KL/Ry = 42.41
	Mcx (kip-ft) = 150.69
Cbx = 2.25	

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.036
Eq H1-2: 0.058 + 0.027 = 0.086
Eq H1-1b Per H1.3: 0.018 + 0.372 + 0.000 = 0.390

Member Code Check

RAM Structural System
Bentley

RAM Frame 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:56:28
Steel Code: AISC360-10 ASD

COLUMN INFORMATION:

Story Level = 2nd   Frame Number = 2   Column Number = 12
Fy (ksi) = 50.00
Column Size = W10X49

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	9.00	9.00
Lu for Bending (ft)	9.00	9.00
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D - 0.910 E2

Shear	Top	Vmajor (kip)	-17.69
		Vminor (kip)	0.00
Shear	Bot.	Vmajor (kip)	-17.69
		Vminor (kip)	0.00

SHEAR CHECK:

Vax (kip)	= -17.69	Vnx/1.50 (kip)	=	68.00	Vax/(Vnx/1.50)	=	0.260
Vay (kip)	= 0.00	Vny/1.67 (kip)	=	201.20	Vay/(Vny/1.67)	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp - 0.683 E2

AXIAL CHECK:

Pa (kip)	= 15.99	Pnx/1.67 (kip)	=	412.10	Pa/(Pnx/1.67)	=	0.039
		Pny/1.67 (kip)	=	378.02	Pa/(Pny/1.67)	=	0.042
		Pn/1.67 (kip)	=	378.02	Pa/(Pn/1.67)	=	0.042

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D - 0.910 E2

Axial		Load (kip)	11.46
Moment	Top	Mmajor (kip-ft)	53.64
		Mminor (kip-ft)	-0.00
Moment	Bot.	Mmajor (kip-ft)	-87.17
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip)	=	11.46	Pnx/1.67 (kip)	=	412.10
			Pny/1.67 (kip)	=	378.02
Max (kip-ft)	=	-87.17	Mnx/1.67 (kip-ft)	=	150.70
May (kip-ft)	=	0.00	Mny/1.67 (kip-ft)	=	70.61
KL/Rx	=	24.85	KL/Ry	=	42.41
			Mcx (kip-ft)	=	150.69
Cbx	=	2.20			

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.028
Eq H1-2: 0.045 + 0.069 = 0.114
Eq H1-1b Per H1.3: 0.014 + 0.578 + 0.000 = 0.592

Member Code Check

RAM Structural System
Bentley
RAM Frame 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:56:28
Steel Code: AISC360-10 ASD

BEAM INFORMATION:

Story Level = 2nd Frame Number = 2 Beam Number = 9
Fy (ksi) = 50.00
Beam Size = W12X35
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	12.83	6.42
Lu for Bending (ft)	12.83	6.42
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.077 D - 0.910 E2

Segment distance (ft) i - end	0.00
j - end	6.42

SHEAR CHECK:

Vax (kip) =	9.68	Vnx/1.50 (kip)	=	75.00	Vax/(Vnx/1.50)	=	0.129
Vay (kip) =	-0.00	Vny/1.67 (kip)	=	122.56	Vay/(Vny/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.000 D

Segment distance (ft) i - end	0.00
j - end	6.42

AXIAL CHECK:

Pa (kip) =	0.00	Pnx/1.67 (kip)	=	308.38	Pa/(Pnx/1.67)	=	0.000
		Pny/1.67 (kip)	=	308.38	Pa/(Pny/1.67)	=	0.000
		Pn/1.67 (kip)	=	308.38	Pa/(Pn/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.077 D - 0.910 E2

Segment distance (ft) i - end	0.00
j - end	6.42

CALCULATED PARAMETERS:

Pa (kip)	=	0.00	Pn/1.67 (kip)	=	308.38
Max (kip-ft)	=	-58.81	Mnx/1.67 (kip-ft)	=	127.74
May (kip-ft)	=	-0.00	Mny/1.67 (kip-ft)	=	28.69
Cbx	=	1.65			

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.000
Eq H1-1b: 0.000 + 0.460 + 0.000 = 0.460

Member Code Check

RAM Structural System
Bentley

RAM Frame 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:56:28
Steel Code: AISC360-10 ASD

COLUMN INFORMATION:

Story Level	= 1st	Frame Number	=	2	Column Number	=	6
Fy (ksi)	= 50.00						
Column Size	= W10X49						

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	10.67	10.67
Lu for Bending (ft)	10.67	10.67
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D - 0.910 E2

Shear	Top	Vmajor (kip)	4.38
		Vminor (kip)	-0.00
Shear	Bot.	Vmajor (kip)	4.38
		Vminor (kip)	-0.00

SHEAR CHECK:

Vax (kip)	=	4.38	Vnx/1.50 (kip)	= 68.00	Vax/(Vnx/1.50)	=	0.064
Vay (kip)	=	-0.00	Vny/1.67 (kip)	= 201.20	Vay/(Vny/1.67)	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 0.683 E2

AXIAL CHECK:

Pa (kip)	=	71.91	Pnx/1.67 (kip)	= 404.63	Pa/(Pnx/1.67)	=	0.178
			Pny/1.67 (kip)	= 358.39	Pa/(Pny/1.67)	=	0.201
			Pn/1.67 (kip)	= 358.39	Pa/(Pn/1.67)	=	0.201

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 0.683 E2

Axial		Load (kip)	71.91
Moment	Top	Mmajor (kip-ft)	21.99
		Mminor (kip-ft)	0.00
Moment	Bot.	Mmajor (kip-ft)	-9.87
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip)	=	71.91	Pnx/1.67 (kip)	=	404.63
			Pny/1.67 (kip)	=	358.39
Max (kip-ft)	=	21.99	Mnx/1.67 (kip-ft)	=	150.70
May (kip-ft)	=	0.00	Mny/1.67 (kip-ft)	=	70.61
KL/Rx	=	29.46	KL/Ry	=	50.28
			Mcx (kip-ft)	=	146.60
Cbx	=	2.16			

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.201
Eq H1-1b Per H1.3: 0.089 + 0.146 + 0.000 = 0.235
Eq H1-2: 0.281 + 0.005 = 0.286

Member Code Check

BEAM INFORMATION:

Story Level = 1st Frame Number = 2 Beam Number = 7
Fy (ksi) = 50.00
Beam Size = W12X35
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	12.83	5.04
Lu for Bending (ft)	12.83	5.04
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp - 0.683 E2

Segment distance (ft) i - end	0.00
j - end	2.75

SHEAR CHECK:

Vax (kip)	= 6.32	Vnx/1.50 (kip)	=	75.00	Vax/(Vnx/1.50)	=	0.084
Vay (kip)	= 0.00	Vny/1.67 (kip)	=	122.56	Vay/(Vny/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.000 D

Segment distance (ft) i - end	2.75
j - end	7.79

AXIAL CHECK:

Pa (kip)	= 0.00	Pnx/1.67 (kip)	=	308.38	Pa/(Pnx/1.67)	=	0.000
		Pny/1.67 (kip)	=	308.38	Pa/(Pny/1.67)	=	0.000
		Pn/1.67 (kip)	=	308.38	Pa/(Pn/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.077 D + 0.910 E2

Segment distance (ft) i - end	7.79
j - end	12.83

CALCULATED PARAMETERS:

Pa (kip)	=	0.00	Pn/1.67 (kip)	=	308.38
Max (kip-ft)	=	-28.90	Mnx/1.67 (kip-ft)	=	127.74
May (kip-ft)	=	-0.00	Mny/1.67 (kip-ft)	=	28.69
Cbx	=	1.33			

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.000
Eq H1-1b: 0.000 + 0.226 + 0.000 = 0.226

Member Code Check

RAM Frame 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:56:28
Steel Code: AISC360-10 ASD

COLUMN INFORMATION:

Story Level = 2nd   Frame Number = 3   Column Number = 8
Fy (ksi) = 50.00
Column Size = W10X49

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	9.00	9.00
Lu for Bending (ft)	9.00	9.00
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D + 0.910 E3

Shear	Top	Vmajor (kip)	15.15
		Vminor (kip)	0.00
Shear	Bot.	Vmajor (kip)	15.15
		Vminor (kip)	0.00

SHEAR CHECK:

Vax (kip)	=	15.15	Vnx/1.50 (kip)	=	68.00	Vax/(Vnx/1.50)	=	0.223
Vay (kip)	=	0.00	Vny/1.67 (kip)	=	201.20	Vay/(Vny/1.67)	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 0.683 E3

AXIAL CHECK:

Pa (kip)	=	35.28	Pnx/1.67 (kip)	=	412.10	Pa/(Pnx/1.67)	=	0.086
			Pny/1.67 (kip)	=	378.02	Pa/(Pny/1.67)	=	0.093
			Pn/1.67 (kip)	=	378.02	Pa/(Pn/1.67)	=	0.093

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D + 0.910 E3

Axial		Load (kip)	16.57
Moment	Top	Mmajor (kip-ft)	-54.81
		Mminor (kip-ft)	-0.00
Moment	Bot.	Mmajor (kip-ft)	65.74
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip)	=	16.57	Pnx/1.67 (kip)	=	412.10
			Pny/1.67 (kip)	=	378.02
Max (kip-ft)	=	65.74	Mnx/1.67 (kip-ft)	=	150.70
May (kip-ft)	=	0.00	Mny/1.67 (kip-ft)	=	70.61
KL/Rx	=	24.85	KL/Ry	=	42.41
			Mcx (kip-ft)	=	150.69
Cbx	=	2.24			

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.040
Eq H1-2: 0.065 + 0.038 = 0.103
Eq H1-1b Per H1.3: 0.020 + 0.436 + 0.000 = 0.456

Member Code Check

COLUMN INFORMATION:

Story Level	= 2nd	Frame Number	= 3	Column Number	= 9
Fy (ksi)	= 50.00				
Column Size	= W10X49				

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	9.00	9.00
Lu for Bending (ft)	9.00	9.00
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D - 0.910 E3

Shear	Top	Vmajor (kip)	-15.15
		Vminor (kip)	0.00
Shear	Bot.	Vmajor (kip)	-15.15
		Vminor (kip)	0.00

SHEAR CHECK:

Vax (kip)	= -15.15	Vnx/1.50 (kip)	= 68.00	Vax/(Vnx/1.50)	= 0.223
Vay (kip)	= 0.00	Vny/1.67 (kip)	= 201.20	Vay/(Vny/1.67)	= 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Sp - 0.683 E3

AXIAL CHECK:

Pa (kip)	= 35.28	Pnx/1.67 (kip)	= 412.10	Pa/(Pnx/1.67)	= 0.086
		Pny/1.67 (kip)	= 378.02	Pa/(Pny/1.67)	= 0.093
		Pn/1.67 (kip)	= 378.02	Pa/(Pn/1.67)	= 0.093

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D - 0.910 E3

Axial		Load (kip)	16.57
Moment	Top	Mmajor (kip-ft)	54.81
		Mminor (kip-ft)	-0.00
Moment	Bot.	Mmajor (kip-ft)	-65.73
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip)	= 16.57	Pnx/1.67 (kip)	= 412.10
		Pny/1.67 (kip)	= 378.02
Max (kip-ft)	= -65.73	Mnx/1.67 (kip-ft)	= 150.70
May (kip-ft)	= 0.00	Mny/1.67 (kip-ft)	= 70.61
KL/Rx	= 24.85	KL/Ry	= 42.41
		Mcx (kip-ft)	= 150.69
Cbx	= 2.24		

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.040

Eq H1-2: 0.065 + 0.038 = 0.103

Eq H1-1b Per H1.3: 0.020 + 0.436 + 0.000 = 0.456

Member Code Check

BEAM INFORMATION:

Story Level = 2nd Frame Number = 3 Beam Number = 6
Fy (ksi) = 50.00
Beam Size = W12X35
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	12.83	6.33
Lu for Bending (ft)	12.83	6.33
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.077 D + 0.910 E3

Segment distance (ft) i - end 6.50
j - end 12.83

SHEAR CHECK:

Vax (kip)	= -10.32	Vnx/1.50 (kip)	=	75.00	Vax/(Vnx/1.50)	=	0.138
Vay (kip)	= -0.00	Vny/1.67 (kip)	=	122.56	Vay/(Vny/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.000 D

Segment distance (ft) i - end 0.00
j - end 6.50

AXIAL CHECK:

Pa (kip)	= 0.00	Pnx/1.67 (kip)	=	308.38	Pa/(Pnx/1.67)	=	0.000
		Pny/1.67 (kip)	=	308.38	Pa/(Pny/1.67)	=	0.000
		Pn/1.67 (kip)	=	308.38	Pa/(Pn/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.077 D + 0.910 E3

Segment distance (ft) i - end 6.50
j - end 12.83

CALCULATED PARAMETERS:

Pa (kip)	=	0.00	Pn/1.67 (kip)	=	308.38
Max (kip-ft)	=	-58.37	Mnx/1.67 (kip-ft)	=	127.74
May (kip-ft)	=	-0.00	Mny/1.67 (kip-ft)	=	28.69
Cbx	=	1.69			

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.000
Eq H1-1b: 0.000 + 0.457 + 0.000 = 0.457

Member Code Check

COLUMN INFORMATION:

Story Level = 1st   Frame Number = 3   Column Number = 11
Fy (ksi) = 50.00
Column Size = W10X49

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	10.67	10.67
Lu for Bending (ft)	10.67	10.67
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D - 0.910 E3

Shear	Top	Vmajor (kip)	4.31
		Vminor (kip)	-0.00
Shear	Bot.	Vmajor (kip)	4.31
		Vminor (kip)	-0.00

SHEAR CHECK:

Vax (kip)	= 4.31	Vnx/1.50 (kip)	= 68.00	Vax/(Vnx/1.50)	= 0.063
Vay (kip)	= -0.00	Vny/1.67 (kip)	= 201.20	Vay/(Vny/1.67)	= 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 0.683 E3

AXIAL CHECK:

Pa (kip)	= 60.32	Pnx/1.67 (kip)	= 404.63	Pa/(Pnx/1.67)	= 0.149
		Pny/1.67 (kip)	= 358.39	Pa/(Pny/1.67)	= 0.168
		Pn/1.67 (kip)	= 358.39	Pa/(Pn/1.67)	= 0.168

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D + 0.910 E3

Axial		Load (kip)	31.00
Moment	Top	Mmajor (kip-ft)	28.92
		Mminor (kip-ft)	-0.00
Moment	Bot.	Mmajor (kip-ft)	-13.00
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip)	= 31.00	Pnx/1.67 (kip)	= 404.63
		Pny/1.67 (kip)	= 358.39
Max (kip-ft)	= 28.92	Mnx/1.67 (kip-ft)	= 150.70
May (kip-ft)	= -0.00	Mny/1.67 (kip-ft)	= 70.61
KL/Rx	= 29.46	KL/Ry	= 50.28
		Mcx (kip-ft)	= 146.60
Cbx	= 2.16		

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.077
Eq H1-2: 0.126 + 0.008 = 0.134
Eq H1-1b Per H1.3: 0.038 + 0.192 + 0.000 = 0.230

Member Code Check

COLUMN INFORMATION:

Story Level = 1st   Frame Number = 3   Column Number = 12
Fy (ksi) = 50.00
Column Size = W10X49

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	10.67	10.67
Lu for Bending (ft)	10.67	10.67
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D + 0.910 E3

Shear	Top	Vmajor (kip)	-4.31
		Vminor (kip)	-0.00
Shear	Bot.	Vmajor (kip)	-4.31
		Vminor (kip)	-0.00

SHEAR CHECK:

Vax (kip) = -4.31	Vnx/1.50 (kip) = 68.00	Vax/(Vnx/1.50) = 0.063
Vay (kip) = -0.00	Vny/1.67 (kip) = 201.20	Vay/(Vny/1.67) = 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp - 0.683 E3

AXIAL CHECK:

Pa (kip) = 60.25	Pnx/1.67 (kip) = 404.63	Pa/(Pnx/1.67) = 0.149
	Pny/1.67 (kip) = 358.39	Pa/(Pny/1.67) = 0.168
	Pn/1.67 (kip) = 358.39	Pa/(Pn/1.67) = 0.168

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D - 0.910 E3

Axial		Load (kip)	30.93
Moment	Top	Mmajor (kip-ft)	-28.91
		Mminor (kip-ft)	-0.00
Moment	Bot.	Mmajor (kip-ft)	13.00
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip) = 30.93	Pnx/1.67 (kip) = 404.63
	Pny/1.67 (kip) = 358.39
Max (kip-ft) = -28.91	Mnx/1.67 (kip-ft) = 150.70
May (kip-ft) = -0.00	Mny/1.67 (kip-ft) = 70.61
KL/Rx = 29.46	KL/Ry = 50.28
	Mcx (kip-ft) = 146.60
Cbx = 2.16	

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.076
Eq H1-2: 0.126 + 0.008 = 0.134
Eq H1-1b Per H1.3: 0.038 + 0.192 + 0.000 = 0.230

Member Code Check

RAM Structural System
Bentley
RAM Frame 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:56:28
Steel Code: AISC360-10 ASD

BEAM INFORMATION:

Story Level = 1st   Frame Number = 3   Beam Number = 16
Fy (ksi) = 50.00
Beam Size = W12X35
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	12.83	6.33
Lu for Bending (ft)	12.83	6.33
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.058 D + 0.750 Sp - 0.683 E3

Segment distance (ft) i - end	0.00
j - end	6.50

SHEAR CHECK:

Vax (kip)	= 8.57	Vnx/1.50 (kip)	=	75.00	Vax/(Vnx/1.50)	=	0.114
Vay (kip)	= -0.00	Vny/1.67 (kip)	=	122.56	Vay/(Vny/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.000 D

Segment distance (ft) i - end	0.00
j - end	6.50

AXIAL CHECK:

Pa (kip)	= 0.00	Pnx/1.67 (kip)	=	308.38	Pa/(Pnx/1.67)	=	0.000
		Pny/1.67 (kip)	=	308.38	Pa/(Pny/1.67)	=	0.000
		Pn/1.67 (kip)	=	308.38	Pa/(Pn/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.077 D + 0.910 E3

Segment distance (ft) i - end	6.50
j - end	12.83

CALCULATED PARAMETERS:

Pa (kip)	=	0.00	Pn/1.67 (kip)	=	308.38
Max (kip-ft)	=	-39.50	Mnx/1.67 (kip-ft)	=	127.74
May (kip-ft)	=	-0.00	Mny/1.67 (kip-ft)	=	28.69
Cbx	=	1.73			

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.000
Eq H1-1b: 0.000 + 0.309 + 0.000 = 0.309

Member Code Check

COLUMN INFORMATION:

Story Level = 2nd   Frame Number = 4   Column Number = 5
Fy (ksi) = 50.00
Column Size = W10X49

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	9.00	9.00
Lu for Bending (ft)	9.00	9.00
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D - 0.910 E1

Shear	Top	Vmajor (kip)	-17.69
		Vminor (kip)	-0.00
Shear	Bot.	Vmajor (kip)	-17.69
		Vminor (kip)	-0.00

SHEAR CHECK:

Vax (kip) =	-17.69	Vnx/1.50 (kip) =	68.00	Vax/(Vnx/1.50) =	0.260
Vay (kip) =	-0.00	Vny/1.67 (kip) =	201.20	Vay/(Vny/1.67) =	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp - 0.683 E1

AXIAL CHECK:

Pa (kip) =	15.99	Pnx/1.67 (kip) =	412.10	Pa/(Pnx/1.67) =	0.039
		Pny/1.67 (kip) =	378.02	Pa/(Pny/1.67) =	0.042
		Pn/1.67 (kip) =	378.02	Pa/(Pn/1.67) =	0.042

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D - 0.910 E1

Axial		Load (kip)	11.46
Moment	Top	Mmajor (kip-ft)	53.64
		Mminor (kip-ft)	0.00
Moment	Bot.	Mmajor (kip-ft)	-87.17
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip) =	11.46	Pnx/1.67 (kip) =	412.10
		Pny/1.67 (kip) =	378.02
Max (kip-ft) =	-87.17	Mnx/1.67 (kip-ft) =	150.70
May (kip-ft) =	0.00	Mny/1.67 (kip-ft) =	70.61
KL/Rx =	24.85	KL/Ry =	42.41
		Mcx (kip-ft) =	150.69
Cbx =	2.20		

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.028
Eq H1-2: 0.045 + 0.069 = 0.114
Eq H1-1b Per H1.3: 0.014 + 0.578 + 0.000 = 0.592

Member Code Check

COLUMN INFORMATION:

Story Level = 2nd	Frame Number = 4	Column Number = 6	
Fy (ksi) = 50.00			
Column Size = W10X49			

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	9.00	9.00
Lu for Bending (ft)	9.00	9.00
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D + 0.910 E1

Shear	Top	Vmajor (kip)	13.42
		Vminor (kip)	-0.00
Shear	Bot.	Vmajor (kip)	13.42
		Vminor (kip)	-0.00

SHEAR CHECK:

Vax (kip) =	13.42	Vnx/1.50 (kip) =	68.00	Vax/(Vnx/1.50) =	0.197
Vay (kip) =	-0.00	Vny/1.67 (kip) =	201.20	Vay/(Vny/1.67) =	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Sp + 0.683 E1

AXIAL CHECK:

Pa (kip) =	30.70	Pnx/1.67 (kip) =	412.10	Pa/(Pnx/1.67) =	0.074
		Pny/1.67 (kip) =	378.02	Pa/(Pny/1.67) =	0.081
		Pn/1.67 (kip) =	378.02	Pa/(Pn/1.67) =	0.081

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.077 D + 0.910 E1

Axial		Load (kip)	14.87
Moment	Top	Mmajor (kip-ft)	-50.67
		Mminor (kip-ft)	0.00
Moment	Bot.	Mmajor (kip-ft)	56.09
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip) =	14.87	Pnx/1.67 (kip) =	412.10
		Pny/1.67 (kip) =	378.02
Max (kip-ft) =	56.09	Mnx/1.67 (kip-ft) =	150.70
May (kip-ft) =	0.00	Mny/1.67 (kip-ft) =	70.61
KL/Rx =	24.85	KL/Ry =	42.41
		Mcx (kip-ft) =	150.69
Cbx =	2.25		

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.036

Eq H1-2: 0.058 + 0.027 = 0.086

Eq H1-1b Per H1.3: 0.018 + 0.372 + 0.000 = 0.390

Member Code Check

BEAM INFORMATION:

Story Level = 2nd   Frame Number = 4   Beam Number = 15
Fy (ksi) = 50.00
Beam Size = W12X35
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	12.83	6.42
Lu for Bending (ft)	12.83	6.42
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.077 D - 0.910 E1

Segment distance (ft) i - end	0.00
j - end	6.42

SHEAR CHECK:

Vax (kip)	=	9.68	Vnx/1.50 (kip)	=	75.00	Vax/(Vnx/1.50)	=	0.129
Vay (kip)	=	0.00	Vny/1.67 (kip)	=	122.56	Vay/(Vny/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.000 D

Segment distance (ft) i - end	0.00
j - end	6.42

AXIAL CHECK:

Pa (kip)	=	0.00	Pnx/1.67 (kip)	=	308.38	Pa/(Pnx/1.67)	=	0.000
			Pny/1.67 (kip)	=	308.38	Pa/(Pny/1.67)	=	0.000
			Pn/1.67 (kip)	=	308.38	Pa/(Pn/1.67)	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.077 D - 0.910 E1

Segment distance (ft) i - end	0.00
j - end	6.42

CALCULATED PARAMETERS:

Pa (kip)	=	0.00	Pn/1.67 (kip)	=	308.38
Max (kip-ft)	=	-58.80	Mnx/1.67 (kip-ft)	=	127.74
May (kip-ft)	=	0.00	Mny/1.67 (kip-ft)	=	28.69
Cbx	=	1.65			

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.000
Eq H1-1b: 0.000 + 0.460 + 0.000 = 0.460

Member Code Check

RAM Structural System
Bentley

RAM Frame 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:56:28
Steel Code: AISC360-10 ASD

COLUMN INFORMATION:

Story Level = 1st		Frame Number = 4	Column Number = 3
Fy (ksi) = 50.00			
Column Size = W10X49			

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	10.67	10.67
Lu for Bending (ft)	10.67	10.67
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.077 D - 0.910 E1

Shear	Top	Vmajor (kip)	4.38
		Vminor (kip)	0.00
Shear	Bot.	Vmajor (kip)	4.38
		Vminor (kip)	0.00

SHEAR CHECK:

Vax (kip) =	4.38	Vnx/1.50 (kip) =	68.00	Vax/(Vnx/1.50) =	0.064
Vay (kip) =	0.00	Vny/1.67 (kip) =	201.20	Vay/(Vny/1.67) =	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 0.683 E1

AXIAL CHECK:

Pa (kip) =	71.33	Pnx/1.67 (kip) =	404.63	Pa/(Pnx/1.67) =	0.176
		Pny/1.67 (kip) =	358.39	Pa/(Pny/1.67) =	0.199
		Pn/1.67 (kip) =	358.39	Pa/(Pn/1.67) =	0.199

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 0.683 E1

Axial		Load (kip)	71.33
Moment	Top	Mmajor (kip-ft)	21.97
		Mminor (kip-ft)	-0.00
Moment	Bot.	Mmajor (kip-ft)	-9.86
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip) =	71.33	Pn/1.67 (kip) =	358.39
Max (kip-ft) =	21.97	Mnx/1.67 (kip-ft) =	150.70
May (kip-ft) =	-0.00	Mny/1.67 (kip-ft) =	70.61
KL/Rx =	29.46	KL/Ry =	50.28
Cbx =	2.16		

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.199

Eq H1-1b: 0.100 + 0.146 + 0.000 = 0.245

Member Code Check

RAM Structural System
Bentley

RAM Frame 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:56:28
Steel Code: AISC360-10 ASD

BEAM INFORMATION:

Story Level = 1st   Frame Number = 4   Beam Number = 2
Fy (ksi) = 50.00
Beam Size = W12X35
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	12.83	5.04
Lu for Bending (ft)	12.83	5.04
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.058 D + 0.750 Lp + 0.750 Sp - 0.683 E1

Segment distance (ft) i - end	0.00
j - end	2.75

SHEAR CHECK:

Vax (kip)	= 6.32	Vnx/1.50 (kip)	= 75.00	Vax/(Vnx/1.50)	= 0.084
Vay (kip)	= -0.00	Vny/1.67 (kip)	= 122.56	Vay/(Vny/1.67)	= 0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.000 D

Segment distance (ft) i - end	2.75
j - end	7.79

AXIAL CHECK:

Pa (kip)	= 0.00	Pnx/1.67 (kip)	= 308.38	Pa/(Pnx/1.67)	= 0.000
		Pny/1.67 (kip)	= 308.38	Pa/(Pny/1.67)	= 0.000
		Pn/1.67 (kip)	= 308.38	Pa/(Pn/1.67)	= 0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.077 D + 0.910 E1

Segment distance (ft) i - end	7.79
j - end	12.83

CALCULATED PARAMETERS:

Pa (kip)	= 0.00	Pn/1.67 (kip)	= 308.38
Max (kip-ft)	= -28.92	Mnx/1.67 (kip-ft)	= 127.74
May (kip-ft)	= -0.00	Mny/1.67 (kip-ft)	= 28.69
Cbx	= 1.33		

INTERACTION EQUATION:

Pa/(Pn/1.67)),=,0.000
Eq H1-1b: 0.000 + 0.226 + 0.000 = 0.226

Joint Code Check

Story Number: 1 **Joint Number:** 16

Final Design

No Web Plate Required
No Top Flange Stiffener Required
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00
Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	0.00	---	50.00
Beam SideB :	W12X35	180.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
B	-28.93	0.00	-3.85	1.077 D + 0.910 E1

Shear Force In Column Above Joint(kip) = 13.42
Controlling Shear Force (kip) = 33.31
Column Web Capacity w/o Web Plate (kip) = 74.60

Compression		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	28.0	41	66.0	NO
	Bot	---	---	---	29.0	5	66.0	NO
Web Crippling	Top	---	---	---	28.0	41	76.7	NO
	Bot	---	---	---	29.0	5	76.7	NO

Tension		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	29.0	5	66.0	NO
	Bot	---	---	---	28.0	41	66.0	NO
Flange Bend.	Top	---	---	---	29.0	5	58.7	NO
	Bot	---	---	---	28.0	41	58.7	NO

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

RAM Structural System 15.07.00.17
DataBase: Meehan Cabin 2018-02-19 02/27/18 11:56:28
Building Code: IBC Steel Code: AISC360-10 ASD

Frame #1:
Story Number: 2 **Joint Number:** 1

Final Design

No Web Plate Required

Top	SideA Stiffener LxWxT (in) :	4.500	x	3.125	x	0.375
Bot	SideA Stiffener LxWxT (in) :	4.500	x	3.125	x	0.375

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00
Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	90.00	---	50.00
Beam SideA :	W12X35	90.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
A	-61.52	-0.00	11.26	1.077 D - 0.910 E4

Controlling Shear Force (kip) = 45.58
Column Web Capacity w/o Web Plate (kip) = 74.60

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideA	Top	1.00	**OK**
SideA	Bot	0.05	**OK**

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	60.2	40	35.9	---	---	---	YES
	Bot	61.6	12	66.0	---	---	---	NO
Web Crippling	Top	60.2	40	38.4	---	---	---	YES
	Bot	61.6	12	76.7	---	---	---	NO

Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	61.6	12	35.9	---	---	---	YES
	Bot	60.2	40	66.0	---	---	---	NO
Flange Bend.	Top	61.6	12	29.3	---	---	---	YES
	Bot	60.2	40	58.7	---	---	---	YES

Joint Code Check

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Story Number: 2 **Joint Number:** 2

Final Design

No Web Plate Required

Top	SideB Stiffener LxWxT (in) :	4.500	x	3.125	x	0.375
Bot	SideB Stiffener LxWxT (in) :	4.500	x	3.125	x	0.375

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00
Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	90.00	---	50.00
Beam SideB :	W12X35	270.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
B	-61.52	0.00	-11.26	1.077 D + 0.910 E4

Controlling Shear Force (kip) = 45.58
Column Web Capacity w/o Web Plate (kip) = 74.60

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideB	Top	1.00	**OK**
SideB	Bot	0.05	**OK**

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	60.2	44	35.9	YES
	Bot	---	---	---	61.6	8	66.0	NO
Web Crippling	Top	---	---	---	60.2	44	38.4	YES
	Bot	---	---	---	61.6	8	76.7	NO

Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	61.6	8	35.9	YES
	Bot	---	---	---	60.2	44	66.0	NO
Flange Bend.	Top	---	---	---	61.6	8	29.3	YES
	Bot	---	---	---	60.2	44	58.7	YES

Joint Code Check

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Frame #2:
Story Number: 2 **Joint Number:** 5

Final Design

No Web Plate Required
Top SideB Stiffener LxWxT (in) : 4.500 x 3.125 x 0.375
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00
Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	0.00	---	50.00
Beam SideB :	W12X35	180.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
B	-53.64	-0.00	-9.62	1.077 D + 0.910 E2

Controlling Shear Force (kip) = 40.32
Column Web Capacity w/o Web Plate (kip) = 74.60

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideB	Top	0.75	**OK**

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	52.9	42	35.9	YES
	Bot	---	---	---	53.7	6	66.0	NO
Web Crippling	Top	---	---	---	52.9	42	38.4	YES
	Bot	---	---	---	53.7	6	76.7	NO

Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	53.7	6	35.9	YES
	Bot	---	---	---	52.9	42	66.0	NO
Flange Bend.	Top	---	---	---	53.7	6	29.3	YES
	Bot	---	---	---	52.9	42	58.7	NO

Joint Code Check

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Story Number: 2 **Joint Number:** 3

Final Design

No Web Plate Required

Top SideA Stiffener LxWxT (in) : 4.500 x 3.125 x 0.375

No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) 36.00

Stiffener Nominal Yield (ksi) 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	0.00	---	50.00
Beam SideA :	W12X35	0.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
A	-58.82	0.00	9.68	1.077 D - 0.910 E2

Controlling Shear Force (kip) = 41.23

Column Web Capacity w/o Web Plate (kip) = 74.60

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideA	Top	0.91	**OK**

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	57.5	38	35.9	---	---	---	YES
	Bot	58.9	10	66.0	---	---	---	NO
Web Crippling	Top	57.5	38	38.4	---	---	---	YES
	Bot	58.9	10	76.7	---	---	---	NO

Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	58.9	10	35.9	---	---	---	YES
	Bot	57.5	38	66.0	---	---	---	NO
Flange Bend.	Top	58.9	10	29.3	---	---	---	YES
	Bot	57.5	38	58.7	---	---	---	NO

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Story Number: 1 **Joint Number:** 15

Final Design

No Web Plate Required
No Top Flange Stiffener Required
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ______ 36.00
Stiffener Nominal Yield (ksi) ______ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	0.00	---	50.00
Beam SideB :	W12X35	180.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
B	-28.91	0.00	-3.85	1.077 D + 0.910 E2

Shear Force In Column Above Joint(kip) = 13.41
Controlling Shear Force (kip) = 33.29
Column Web Capacity w/o Web Plate (kip) = 74.60

Compression

		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	28.0	42	66.0	NO
	Bot	---	---	---	29.0	6	66.0	NO
Web Crippling	Top	---	---	---	28.0	42	76.7	NO
	Bot	---	---	---	29.0	6	76.7	NO

Tension

		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	29.0	6	66.0	NO
	Bot	---	---	---	28.0	42	66.0	NO
Flange Bend.	Top	---	---	---	29.0	6	58.7	NO
	Bot	---	---	---	28.0	42	58.7	NO

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Frame #3:
Story Number: 2 **Joint Number:** 8

Final Design

No Web Plate Required
Top SideB Stiffener LxWxT (in) : 4.500 x 3.125 x 0.375
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ———— 36.00
Stiffener Nominal Yield (ksi) ———— 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	90.00	---	50.00
Beam SideB :	W12X35	270.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
B	-58.39	-0.00	-10.32	1.077 D + 0.910 E3

Controlling Shear Force (kip) = 43.34
Column Web Capacity w/o Web Plate (kip) = 74.60

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideB	Top	0.90	**OK**

Compression

		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	57.0	43	35.9	YES
	Bot	---	---	---	58.5	7	66.0	NO
Web Crippling	Top	---	---	---	57.0	43	38.4	YES
	Bot	---	---	---	58.5	7	76.7	NO

Tension

		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	58.5	7	35.9	YES
	Bot	---	---	---	57.0	43	66.0	NO
Flange Bend.	Top	---	---	---	58.5	7	29.3	YES
	Bot	---	---	---	57.0	43	58.7	NO

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Joint Code Check

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Story Number: 2 **Joint Number:** 7

Final Design

No Web Plate Required
Top SideA Stiffener LxWxT (in) : 4.500 x 3.125 x 0.375
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00
Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	90.00	---	50.00
Beam SideA :	W12X35	90.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
A	-58.39	0.00	10.32	1.077 D - 0.910 E3

Controlling Shear Force (kip) = 43.34
Column Web Capacity w/o Web Plate (kip) = 74.60

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideA	Top	0.90	**OK**

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	57.0	39	35.9	---	---	---	YES
	Bot	58.5	11	66.0	---	---	---	NO
Web Crippling	Top	57.0	39	38.4	---	---	---	YES
	Bot	58.5	11	76.7	---	---	---	NO

Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	58.5	11	35.9	---	---	---	YES
	Bot	57.0	39	66.0	---	---	---	NO
Flange Bend.	Top	58.5	11	29.3	---	---	---	YES
	Bot	57.0	39	58.7	---	---	---	NO

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

RAM Structural System 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:56:28
Steel Code: AISC360-10 ASD

Story Number: 1 **Joint Number:** 18

Final Design

No Web Plate Required
No Top Flange Stiffener Required
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00
Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	90.00	---	50.00
Beam SideB :	W12X35	270.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
B	-39.51	0.00	-7.29	1.077 D + 0.910 E3

Shear Force In Column Above Joint(kip) = 15.15
Controlling Shear Force (kip) = 43.71
Column Web Capacity w/o Web Plate (kip) = 74.60

Compression

		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	37.5	43	66.0	NO
	Bot	---	---	---	39.6	7	66.0	NO
Web Crippling	Top	---	---	---	37.5	43	76.7	NO
	Bot	---	---	---	39.6	7	76.7	NO

Tension

		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	39.6	7	66.0	NO
	Bot	---	---	---	37.5	43	66.0	NO
Flange Bend.	Top	---	---	---	39.6	7	58.7	NO
	Bot	---	---	---	37.5	43	58.7	NO

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Story Number: 1 **Joint Number:** 17

Final Design

No Web Plate Required
No Top Flange Stiffener Required
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00
Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	90.00	---	50.00
Beam SideA :	W12X35	90.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
A	-39.51	-0.00	7.29	1.077 D - 0.910 E3

Shear Force In Column Above Joint(kip) = -15.15
Controlling Shear Force (kip) = 43.71
Column Web Capacity w/o Web Plate (kip) = 74.60

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	37.5	39	66.0	---	---	---	NO
	Bot	39.6	11	66.0	---	---	---	NO
Web Crippling	Top	37.5	39	76.7	---	---	---	NO
	Bot	39.6	11	76.7	---	---	---	NO

Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	39.6	11	66.0	---	---	---	NO
	Bot	37.5	39	66.0	---	---	---	NO
Flange Bend.	Top	39.6	11	58.7	---	---	---	NO
	Bot	37.5	39	58.7	---	---	---	NO

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Frame #4:
Story Number: 2 **Joint Number:** 4

Final Design

No Web Plate Required
Top SideA Stiffener LxWxT (in) : 4.500 x 3.125 x 0.375
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00
Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	0.00	---	50.00
Beam SideA :	W12X35	0.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
A	-58.82	0.00	9.68	1.077 D - 0.910 E1

Controlling Shear Force (kip) = 41.22
Column Web Capacity w/o Web Plate (kip) = 74.60

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideA	Top	0.91	**OK**

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	57.5	37	35.9	---	---	---	YES
	Bot	58.9	9	66.0	---	---	---	NO
Web Crippling	Top	57.5	37	38.4	---	---	---	YES
	Bot	58.9	9	76.7	---	---	---	NO

Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	58.9	9	35.9	---	---	---	YES
	Bot	57.5	37	66.0	---	---	---	NO
Flange Bend.	Top	58.9	9	29.3	---	---	---	YES
	Bot	57.5	37	58.7	---	---	---	NO

Joint Code Check

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Story Number: 2 **Joint Number:** 6

Final Design

No Web Plate Required

Top SideB Stiffener LxWxT (in) : 4.500 x 3.125 x 0.375

No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00

Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	0.00	---	50.00
Beam SideB :	W12X35	180.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.

Use actual beam moments to determine panel zone shear at the joint.

Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
B	-53.65	-0.00	-9.62	1.077 D + 0.910 E1

Controlling Shear Force (kip) = 40.32

Column Web Capacity w/o Web Plate (kip) = 74.60

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideB	Top	0.75	**OK**

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	52.9	41	35.9	YES
	Bot	---	---	---	53.7	5	66.0	NO
Web Crippling	Top	---	---	---	52.9	41	38.4	YES
	Bot	---	---	---	53.7	5	76.7	NO

Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	53.7	5	35.9	YES
	Bot	---	---	---	52.9	41	66.0	NO
Flange Bend.	Top	---	---	---	53.7	5	29.3	YES
	Bot	---	---	---	52.9	41	58.7	NO

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Joint Code Check

Story Number: 1 **Joint Number:** 16

Final Design

No Web Plate Required
No Top Flange Stiffener Required
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ____________ 36.00
Stiffener Nominal Yield (ksi) ____________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W10X49	0.00	---	50.00
Beam SideB :	W12X35	180.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
B	-28.93	0.00	-3.85	1.077 D + 0.910 E1

Shear Force In Column Above Joint(kip) = 13.42
Controlling Shear Force (kip) = 33.31
Column Web Capacity w/o Web Plate (kip) = 74.60

Compression		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	28.0	41	66.0	NO
	Bot	---	---	---	29.0	5	66.0	NO
Web Crippling	Top	---	---	---	28.0	41	76.7	NO
	Bot	---	---	---	29.0	5	76.7	NO

Tension		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	---	---	---	29.0	5	66.0	NO
	Bot	---	---	---	28.0	41	66.0	NO
Flange Bend.	Top	---	---	---	29.0	5	58.7	NO
	Bot	---	---	---	28.0	41	58.7	NO

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

Frame #5:

Seismic Provisions Member Code Check

RAM Structural System 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:58:11
Steel Code: AISC341-10 - ASD

Column Parameters

Story: 2nd   Frame No: 1   Member No: 2
Fy (ksi): 50.00   Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 26.18 --- Combination: 1.058 D + 0.750 Sp - 1.575 E4
Max Pa/(Pn/Ω) = 0.07 OK

Tension: Max Pa (kip) = 22.27 --- Combination: 0.523 D + 2.100 E4
Max Pa/(Pn/Ω) = 0.05 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 36.74 --- Combination: 1.077 D - 2.100 E4
No shear in minor axis
Moment in major axis (kip-ft)= 179.69 --- Combination: 1.077 D - 2.100 E4
Moment in minor axis (kip-ft)= 0.00 --- Combination: 1.077 D - 2.100 E4
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 26.18   Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	75.43	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Seismic Provisions Member Code Check

Column Parameters

Story: 2nd Frame No: 1 Member No: 3
Fy (ksi): 50.00 Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 26.18 --- Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 1.575 E4
Max Pa/(Pn/Ω) = 0.07 OK

Tension: Max Pa (kip) = 22.27 --- Combination: 0.523 D - 2.100 E4
Max Pa/(Pn/Ω) = 0.05 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 36.74 --- Combination: 1.077 D + 2.100 E4
No shear in minor axis
Moment in major axis (kip-ft)= 179.69 --- Combination: 1.077 D + 2.100 E4
Moment in minor axis (kip-ft)= 0.00 --- Combination: 1.077 D - 2.100 E4
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 26.18 Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	75.43	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Beam Parameters

Story: 2nd   Frame No: 1   Member No: 12
Fy (ksi): 50.00   Size: W12X35
Frame Type: Intermediate Moment Resisting Frame
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

Reduced Beam Section (RBS) Bending Check --- OK

	End I	**End J**
Distance column Flange to middle RBS (in)	7.63	7.63
Max Moment at RBS (kip-ft)	-54.45	-54.45
Load Combination Number	12	8
Reduced plastic modulus - Zrbs (in ^{3})	41.86	41.86
1.67 Ma / Zrbs Fy	0.52	0.52
	OK	OK

E2.4a Stability Bracing of Beams (D1.2a Moderately Ductile) --- OK

Max Lu (in) = 67.00   Lu Limit = 152.07   OK
Lateral Bracing Requirements along Beam
Required strength of lateral brace along beam = 3.13 kip
Required stiffness of bracing (A-6-8) = 3134.11 kip / Lb
Lateral Bracing Requirements at Plastic Hinge
Required strength of lateral brace at hinge = 7.69 kip
Required stiffness of bracing (A-6-8) = 2562.11 kip / Lb
Cd = 1.0 assumed for eqns (A-6-7/8)
Lb = distance between braces (in)
The brace closest to the point of inflection should be designed for two times these values (Cd=2)

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf =	6.31	Limit =	9.15	OK
Web h/tw =	36.20	Limit =	90.55	OK

E2.6b Beam-to-Column Connection Requirements

Beam-to-column connection must be capable of sustaining a 0.02 radian interstory drift ratio.
All beam-to-column joints to demonstrate conformance with E2.6b(1) as indicated in E2.6b(2)

Seismic Provisions Member Code Check

Column Parameters

Story: 2nd Frame No: 2 Member No: 11
Fy (ksi): 50.00 Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 39.77 --- Combination: 1.058 D + 0.750 Sp + 1.575 E2
Max Pa/(Pn/Ω) = 0.11 OK

Tension: Max Pa (kip) = 18.61 --- Combination: 0.523 D - 2.100 E2
Max Pa/(Pn/Ω) = 0.04 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 30.75 --- Combination: 1.077 D + 2.100 E2
No shear in minor axis
Moment in major axis (kip-ft)= 128.66 --- Combination: 1.077 D + 2.100 E2
Moment in minor axis (kip-ft)= 0.00 --- Combination: 1.077 D - 2.100 E4
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 39.77 Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	67.58	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

	Major	**Minor**
ΣMpc (kip-ft)	335.56	157.22

Column Splice Shear Force Required ΣMpc / 1.5H
Where ΣMpc is the sum of column plastic strengths above and below splice.
Required flexural strength for bolted column splices (kip-ft) = 184.56
See code for more information on the Required Shear Strength

Seismic Provisions Member Code Check

Column Parameters

Story: 2nd Frame No: 2 Member No: 12
Fy (ksi): 50.00 Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 25.06 --- Combination: 1.058 D + 0.750 Lp + 0.750 Sp - 1.575 E2
Max Pa/(Pn/Ω) = 0.07 OK
Tension: Max Pa (kip) = 20.27 --- Combination: 0.523 D + 2.100 E2
Max Pa/(Pn/Ω) = 0.05 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 40.63 --- Combination: 1.077 D - 2.100 E2
No shear in minor axis
Moment in major axis (kip-ft)= 200.92 --- Combination: 1.077 D - 2.100 E2
Moment in minor axis (kip-ft)= 0.00 --- Combination: 1.077 D - 2.100 E4
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 25.06 Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	76.08	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Seismic Provisions Member Code Check

Beam Parameters

Story: 2nd   Frame No: 2   Member No: 9
Fy (ksi): 50.00   Size: W12X35
Frame Type: Intermediate Moment Resisting Frame
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

Reduced Beam Section (RBS) Bending Check --- OK

	End I	**End J**
Distance column Flange to middle RBS (in)	7.63	7.63
Max Moment at RBS (kip-ft)	-52.67	-47.53
Load Combination Number	10	6
Reduced plastic modulus - Zrbs (in^3)	41.86	41.86
1.67 Ma / Zrbs Fy	0.50	0.46
	OK	OK

E2.4a Stability Bracing of Beams (D1.2a Moderately Ductile) --- OK

Max Lu (in) = 77.00   Lu Limit = 152.07   OK
Lateral Bracing Requirements along Beam
Required strength of lateral brace along beam = 3.13 kip
Required stiffness of bracing (A-6-8) = 3134.11 kip / Lb
Lateral Bracing Requirements at Plastic Hinge
Required strength of lateral brace at hinge = 7.69 kip
Required stiffness of bracing (A-6-8) = 2562.11 kip / Lb
Cd = 1.0 assumed for eqns (A-6-7/8)
Lb = distance between braces (in)
The brace closest to the point of inflection should be designed for two times these values (Cd=2)

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf =	6.31	Limit =	9.15	OK
Web h/tw =	36.20	Limit =	90.55	OK

E2.6b Beam-to-Column Connection Requirements

Beam-to-column connection must be capable of sustaining a 0.02 radian interstory drift ratio.
All beam-to-column joints to demonstrate conformance with E2.6b(1) as indicated in E2.6b(2)

Seismic Provisions Member Code Check

Column Parameters

Story: 1st　　Frame No: 2　　Member No: 6
Fy (ksi): 50.00　　Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression:	Max Pa (kip) = 84.30 --- Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 1.575 E2
	Max Pa/(Pn/Ω) = 0.24 OK
Tension:	Max Pa (kip) = 19.83 --- Combination: 0.523 D - 2.100 E2
	Max Pa/(Pn/Ω) = 0.05 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 10.08 --- Combination: 1.077 D - 2.100 E2
No shear in minor axis
Moment in major axis (kip-ft)= 70.43 --- Combination: 1.077 D - 2.100 E2
No moment in minor axis
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 84.30　Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	57.57	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Seismic Provisions Member Code Check

Beam Parameters

Story: 1st   Frame No: 2   Member No: 7
Fy (ksi): 50.00   Size: W12X35
Frame Type: Intermediate Moment Resisting Frame
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

Reduced Beam Section (RBS) Bending Check --- OK

	End I	**End J**
Distance column Flange to middle RBS (in)	7.63	7.63
Max Moment at RBS (kip-ft)	-17.36	-26.46
Load Combination Number	26	6
Reduced plastic modulus - Zrbs (in ^{3})	41.86	41.86
1.67 Ma / Zrbs Fy	0.17	0.25
	OK	OK

E2.4a Stability Bracing of Beams (D1.2a Moderately Ductile) --- OK

Max Lu (in) = 60.50   Lu Limit = 152.07   OK
Lateral Bracing Requirements along Beam
Required strength of lateral brace along beam = 3.13 kip
Required stiffness of bracing (A-6-8) = 3134.11 kip / Lb
Lateral Bracing Requirements at Plastic Hinge
Required strength of lateral brace at hinge = 7.69 kip
Required stiffness of bracing (A-6-8) = 2562.11 kip / Lb
Cd = 1.0 assumed for eqns (A-6-7/8)
Lb = distance between braces (in)
The brace closest to the point of inflection should be designed for two times these values (Cd=2)

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf =	6.31	Limit =	9.15	OK
Web h/tw =	36.20	Limit =	90.55	OK

E2.6b Beam-to-Column Connection Requirements

Beam-to-column connection must be capable of sustaining a 0.02 radian interstory drift ratio.
All beam-to-column joints to demonstrate conformance with E2.6b(1) as indicated in E2.6b(2)

Seismic Provisions Member Code Check

Column Parameters

Story: 2nd Frame No: 3 Member No: 8
Fy (ksi): 50.00 Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 44.66 --- Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 1.575 E3
Max Pa/(Pn/Ω) = 0.12 OK
Tension: Max Pa (kip) = 18.69 --- Combination: 0.523 D - 2.100 E3
Max Pa/(Pn/Ω) = 0.04 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 34.60 --- Combination: 1.077 D + 2.100 E3
No shear in minor axis
Moment in major axis (kip-ft)= 150.38 --- Combination: 1.077 D + 2.100 E3
Moment in minor axis (kip-ft)= 0.00 --- Combination: 1.077 D - 2.100 E4
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 44.66 Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	64.76	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

	Major	Minor
ΣMpc (kip-ft)	335.56	157.22

Column Splice Shear Force Required ΣMpc / 1.5H
Where ΣMpc is the sum of column plastic strengths above and below splice.
Required flexural strength for bolted column splices (kip-ft) = 184.56
See code for more information on the Required Shear Strength

Seismic Provisions Member Code Check

Column Parameters

Story: 2nd	Frame No: 3	Member No: 9
Fy (ksi): 50.00	Size: W10X49	

Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression:	Max Pa (kip) = 44.67 --- Combination: 1.058 D + 0.750 Sp - 1.575 E3
	Max Pa/(Pn/Ω) = 0.12 OK
Tension:	Max Pa (kip) = 18.69 --- Combination: 0.523 D + 2.100 E3
	Max Pa/(Pn/Ω) = 0.04 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 34.60 --- Combination: 1.077 D - 2.100 E3
No shear in minor axis
Moment in major axis (kip-ft)= 150.37 --- Combination: 1.077 D - 2.100 E3
Moment in minor axis (kip-ft)= 0.00 --- Combination: 1.077 D - 2.100 E4
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 44.67 Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	64.75	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

	Major	**Minor**
ΣMpc (kip-ft)	335.56	157.22

Column Splice Shear Force Required ΣMpc / 1.5H
Where ΣMpc is the sum of column plastic strengths above and below splice.
Required flexural strength for bolted column splices (kip-ft) = 184.56
See code for more information on the Required Shear Strength

Seismic Provisions Member Code Check

Beam Parameters

Story: 2nd   Frame No: 3   Member No: 6
Fy (ksi): 50.00   Size: W12X35
Frame Type: Intermediate Moment Resisting Frame
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

Reduced Beam Section (RBS) Bending Check --- OK

	End I	**End J**
Distance column Flange to middle RBS (in)	7.63	7.63
Max Moment at RBS (kip-ft)	-51.85	-51.86
Load Combination Number	11	7
Reduced plastic modulus - Zrbs (in^3)	41.86	41.86
1.67 Ma / Zrbs Fy	0.50	0.50
	OK	OK

E2.4a Stability Bracing of Beams (D1.2a Moderately Ductile) --- OK

Max Lu (in) = 78.00   Lu Limit = 152.07   OK
Lateral Bracing Requirements along Beam
  Required strength of lateral brace along beam = 3.13 kip
  Required stiffness of bracing (A-6-8) = 3134.11 kip / Lb
Lateral Bracing Requirements at Plastic Hinge
  Required strength of lateral brace at hinge = 7.69 kip
  Required stiffness of bracing (A-6-8) = 2562.11 kip / Lb
    Cd = 1.0 assumed for eqns (A-6-7/8)
    Lb = distance between braces (in)
    The brace closest to the point of inflection should be designed for two times these values (Cd=2)

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf =	6.31	Limit =	9.15	OK
Web h/tw =	36.20	Limit =	90.55	OK

E2.6b Beam-to-Column Connection Requirements

Beam-to-column connection must be capable of sustaining a 0.02 radian interstory drift ratio.
All beam-to-column joints to demonstrate conformance with E2.6b(1) as indicated in E2.6b(2)

Seismic Provisions Member Code Check

Column Parameters

Story: 1st Frame No: 3 Member No: 11
Fy (ksi): 50.00 Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 75.94 --- Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 1.575 E3
Max Pa/(Pn/Ω) = 0.21 OK
Tension: Max Pa (kip) = 29.42 --- Combination: 0.523 D - 2.100 E3
Max Pa/(Pn/Ω) = 0.07 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 9.82 --- Combination: 1.077 D - 2.100 E3
No shear in minor axis
Moment in major axis (kip-ft)= 68.71 --- Combination: 1.077 D - 2.100 E3
No moment in minor axis
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 75.94 Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	58.10	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Seismic Provisions Member Code Check

Column Parameters

Story: 1st Frame No: 3 Member No: 12
Fy (ksi): 50.00 Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 75.87 --- Combination: 1.058 D + 0.750 Lp + 0.750 Sp - 1.575 E3
Max Pa/(Pn/Ω) = 0.21 OK
Tension: Max Pa (kip) = 29.46 --- Combination: 0.523 D + 2.100 E3
Max Pa/(Pn/Ω) = 0.07 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 9.82 --- Combination: 1.077 D + 2.100 E3
No shear in minor axis
Moment in major axis (kip-ft)= 68.72 --- Combination: 1.077 D + 2.100 E3
No moment in minor axis
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 75.87 Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	58.10	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Seismic Provisions Member Code Check

Beam Parameters

Story: 1st Frame No: 3 Member No: 16
Fy (ksi): 50.00 Size: W12X35
Frame Type: Intermediate Moment Resisting Frame
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

Reduced Beam Section (RBS) Bending Check --- OK

	End I	End J
Distance column Flange to middle RBS (in)	7.63	7.63
Max Moment at RBS (kip-ft)	-34.92	-34.92
Load Combination Number	11	7
Reduced plastic modulus - Zrbs (in^3)	41.86	41.86
1.67 Ma / Zrbs Fy	0.33	0.33
	OK	OK

E2.4a Stability Bracing of Beams (D1.2a Moderately Ductile) --- OK

Max Lu (in) = 78.00 Lu Limit = 152.07 OK
Lateral Bracing Requirements along Beam
Required strength of lateral brace along beam = 3.13 kip
Required stiffness of bracing (A-6-8) = 3134.11 kip / Lb
Lateral Bracing Requirements at Plastic Hinge
Required strength of lateral brace at hinge = 7.69 kip
Required stiffness of bracing (A-6-8) = 2562.11 kip / Lb
Cd = 1.0 assumed for eqns (A-6-7/8)
Lb = distance between braces (in)
The brace closest to the point of inflection should be designed for two times these values (Cd=2)

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf =	6.31	Limit =	9.15	OK
Web h/tw =	36.20	Limit =	90.55	OK

E2.6b Beam-to-Column Connection Requirements

Beam-to-column connection must be capable of sustaining a 0.02 radian interstory drift ratio.
All beam-to-column joints to demonstrate conformance with E2.6b(1) as indicated in E2.6b(2)

Seismic Provisions Member Code Check

Column Parameters

Story: 2nd　Frame No: 4　Member No: 5
Fy (ksi): 50.00　Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 25.06 --- Combination: 1.058 D + 0.750 Lp + 0.750 Sp - 1.575 E1
Max Pa/(Pn/Ω) = 0.07 OK

Tension: Max Pa (kip) = 20.27 --- Combination: 0.523 D + 2.100 E1
Max Pa/(Pn/Ω) = 0.05 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 40.63 --- Combination: 1.077 D - 2.100 E1
No shear in minor axis
Moment in major axis (kip-ft)= 200.92 --- Combination: 1.077 D - 2.100 E1
Moment in minor axis (kip-ft)= 0.00 --- Combination: 1.077 D - 2.100 E4
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 25.06　Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	76.08	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Seismic Provisions Member Code Check

Column Parameters

Story: 2nd   Frame No: 4   Member No: 6
Fy (ksi): 50.00   Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 39.77 --- Combination: 1.058 D + 0.750 Sp + 1.575 E1
Max Pa/(Pn/Ω) = 0.11 OK

Tension: Max Pa (kip) = 18.61 --- Combination: 0.523 D - 2.100 E1
Max Pa/(Pn/Ω) = 0.04 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 30.76 --- Combination: 1.077 D + 2.100 E1
No shear in minor axis
Moment in major axis (kip-ft)= 128.67 --- Combination: 1.077 D + 2.100 E1
Moment in minor axis (kip-ft)= 0.00 --- Combination: 1.077 D - 2.100 E4
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 39.77   Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	67.58	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

	Major	**Minor**
ΣMpc (kip-ft)	335.56	157.22

Column Splice Shear Force Required ΣMpc / 1.5H
Where ΣMpc is the sum of column plastic strengths above and below splice.
Required flexural strength for bolted column splices (kip-ft) = 184.56
See code for more information on the Required Shear Strength

Seismic Provisions Member Code Check

Beam Parameters

Story: 2nd Frame No: 4 Member No: 15
Fy (ksi): 50.00 Size: W12X35
Frame Type: Intermediate Moment Resisting Frame
Left Connection - Reduced Beam Section
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

Reduced Beam Section (RBS) Bending Check --- OK

	End I	**End J**
Distance column Flange to middle RBS (in)	7.63	7.63
Max Moment at RBS (kip-ft)	-52.67	-47.54
Load Combination Number	9	5
Reduced plastic modulus - Zrbs (in ^{3})	41.86	41.86
1.67 Ma / Zrbs Fy	0.50	0.46
	OK	OK

E2.4a Stability Bracing of Beams (D1.2a Moderately Ductile) --- OK

Max Lu (in) = 77.00 Lu Limit = 152.07 OK
Lateral Bracing Requirements along Beam
Required strength of lateral brace along beam = 3.13 kip
Required stiffness of bracing (A-6-8) = 3134.11 kip / Lb
Lateral Bracing Requirements at Plastic Hinge
Required strength of lateral brace at hinge = 7.69 kip
Required stiffness of bracing (A-6-8) = 2562.11 kip / Lb
Cd = 1.0 assumed for eqns (A-6-7/8)
Lb = distance between braces (in)
The brace closest to the point of inflection should be designed for two times these values (Cd=2)

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf =	6.31	Limit =	9.15	OK
Web h/tw =	36.20	Limit =	90.55	OK

E2.6b Beam-to-Column Connection Requirements

Beam-to-column connection must be capable of sustaining a 0.02 radian interstory drift ratio.
All beam-to-column joints to demonstrate conformance with E2.6b(1) as indicated in E2.6b(2)

Seismic Provisions Member Code Check

Column Parameters

Story: 1st   Frame No: 4   Member No: 3
Fy (ksi): 50.00   Size: W10X49
Frame Type: Intermediate Moment Resisting Frame

D1.4a Required Strength --- OK

Compression: Max Pa (kip) = 83.72 --- Combination: 1.058 D + 0.750 Lp + 0.750 Sp + 1.575 E1
Max Pa/(Pn/Ω) = 0.23 OK
Tension: Max Pa (kip) = 20.10 --- Combination: 0.523 D - 2.100 E1
Max Pa/(Pn/Ω) = 0.05 OK

D2.5b Column Splices - Required Strength

Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 10.08 --- Combination: 1.077 D - 2.100 E1
No shear in minor axis
Moment in major axis (kip-ft)= 70.44 --- Combination: 1.077 D - 2.100 E1
No moment in minor axis
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

For calculation of web limit: Pa (kip) = 83.72 Py/Ω(kip) = 431.14

Flange b/tf =	8.93	Limit =	9.15	OK
Web h/tw =	23.18	Limit =	57.61	OK

E2.6g Column Splices (D2.5c Required Shear Strength)

Column is at the lowest story. No column splice required

Seismic Provisions Member Code Check

Beam Parameters

Story: 1st	Frame No: 4	Member No: 2
Fy (ksi): 50.00	Size: W12X35	

Frame Type: Intermediate Moment Resisting Frame
Right Connnection - Reduced Beam Section
a(in) = 3.50 b(in) = 8.25 c(in) = 0.75
Use Reduced Section Properties in Analysis

Reduced Beam Section (RBS) Bending Check --- OK

	End I	**End J**
Distance column Flange to middle RBS (in)	7.63	7.63
Max Moment at RBS (kip-ft)	-17.33	-26.48
Load Combination Number	25	5
Reduced plastic modulus - Zrbs (in^3)	41.86	41.86
1.67 Ma / Zrbs Fy	0.17	0.25
	OK	OK

E2.4a Stability Bracing of Beams (D1.2a Moderately Ductile) --- OK

Max Lu (in) = 60.50 Lu Limit = 152.07 OK

Lateral Bracing Requirements along Beam
Required strength of lateral brace along beam = 3.13 kip
Required stiffness of bracing (A-6-8) = 3134.11 kip / Lb

Lateral Bracing Requirements at Plastic Hinge
Required strength of lateral brace at hinge = 7.69 kip
Required stiffness of bracing (A-6-8) = 2562.11 kip / Lb

Cd = 1.0 assumed for eqns (A-6-7/8)
Lb = distance between braces (in)
The brace closest to the point of inflection should be designed for two times these values (Cd=2)

E2.5a Basic Requirements (D1.1 Moderately Ductile) --- OK

Flange b/tf =	6.31	Limit =	9.15	OK
Web h/tw =	36.20	Limit =	90.55	OK

E2.6b Beam-to-Column Connection Requirements

Beam-to-column connection must be capable of sustaining a 0.02 radian interstory drift ratio.
All beam-to-column joints to demonstrate conformance with E2.6b(1) as indicated in E2.6b(2)

Seismic Provisions Joint Code Check

RAM Structural System 15.07.00.17
DataBase: Meehan Cabin 2018-02-19
Building Code: IBC

02/27/18 11:58:39
Steel Code: AISC341-10 - ASD

Frame #1:

Joint Parameters

Story: 2nd Frame No: 1 Joint No: 1
Fy (ksi): 50.00 Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
12	Flange	211.02	10.729	23.39	24.46	8

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
12	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Joint Parameters

Story: 2nd   Frame No: 1   Joint No: 2
Fy (ksi): 50.00   Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
12	Flange	211.02	10.729	23.39	24.46	4

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
12	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Frame #2:

Joint Parameters

Story: 2nd Frame No: 2 Joint No: 5
Fy (ksi): 50.00 Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
9	Flange	211.02	10.729	22.19	21.71	2

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
9	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Joint Parameters

Story: 2nd   Frame No: 2   Joint No: 3
Fy (ksi): 50.00   Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
9	Flange	211.02	10.729	22.40	21.78	6

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
9	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Joint Parameters

Story: 1st   Frame No: 2   Joint No: 15
Fy (ksi): 50.00   Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
7	Flange	211.02	11.146	21.85	8.28	2

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
7	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Frame #3:

Joint Parameters

Story: 2nd Frame No: 3 Joint No: 8
Fy (ksi): 50.00 Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
6	Flange	211.02	10.729	23.23	22.84	3

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
6	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Joint Parameters

Story: 2nd | Frame No: 3 | Joint No: 7

Fy (ksi): 50.00 | Column Size: W10X49

Joint Frame Type: Intermediate Moment Resisting Frame

Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
6	Flange	211.02	10.729	23.23	22.84	7

V Max = Max shear from applicable load combinations (with Emh)

Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column

Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
6	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Joint Parameters

Story: 1st Frame No: 3 Joint No: 18
Fy (ksi): 50.00 Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
16	Flange	211.02	10.729	24.06	15.59	3

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
16	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Joint Parameters

Story: 1st   Frame No: 3   Joint No: 17
Fy (ksi): 50.00   Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
16	Flange	211.02	10.729	24.06	15.59	7

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
16	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Frame #4:

Joint Parameters

Story: 2nd Frame No: 4 Joint No: 4
Fy (ksi): 50.00 Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
15	Flange	211.02	10.729	22.40	21.77	5

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
15	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Joint Parameters

Story: 2nd　Frame No: 4　Joint No: 6
Fy (ksi): 50.00　Column Size: W10X49
Joint Frame Type: Intermediate Moment Resisting Frame
Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
15	Flange	211.02	10.729	22.19	21.71	1

V Max = Max shear from applicable load combinations (with Emh)
Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column
Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
15	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Joint Parameters

Story: 1st   Frame No: 4   Joint No: 16

Fy (ksi): 50.00   Column Size: W10X49

Joint Frame Type: Intermediate Moment Resisting Frame

Strain Hardening Factor Cpr = 1.10

E2.6d Connection Required Shear Strength

Bm No.	Col Side	Mpe (kip-ft)	L (ft)	V Max (kip)	V LCo (kip)	Comb.
2	Flange	211.02	11.146	21.86	8.28	1

V Max = Max shear from applicable load combinations (with Emh)

Vu LCo = Max shear due to the special seismic load combination no. shown in Combo column

Mpr = 1.1 x Ry x Fy x Zrbs, where Zrbs = Z if no RBS exists

E2.6f Continuity Plates --- OK With Web Plate / Stiffeners

Bm.	Size	Fyb (ksi)	Bbf (in)	Tbf (in)	Tf Req. (in)	Tf Prov (in)	Stiffen
2	W12X35	50.00	6.56	0.520	1.09	0.56	Yes

Seismic Provisions Joint Code Check

Frame #5:

CALDER RICHARDS CONSULTING ENGINEERS

JOB TITLE Meehan Cabin BY DSM DATE 02/2018

SUBJECT Column Base Loads CHECKED SHEET OF

Columns on Circular Pier

Required Axial

Compression = 93.57^K LRFD

Tension = -31.32^K LRFD

Required Shear

D2.5b: $V = 12.39^K$ LRFD

D2.5c: $V = M_{pc}/H = 50\text{ ksi} \times 60.4\text{ in}^3 / 10.67' \times 12 = 23.58^K$ LRFD

Required Flexure

a) $1.1 \times R_y \times F_y \times Z = 1.1 \times 1.1 \times 50\text{ ksi} \times 60.4\text{ in}^3 = 304.52^{K'}$ LRFD

b) Overstrength = $40.71^{K'}$ LRFD

Max Compression

$P = 93.57^K$

$V = \cancel{12.11^K} \rightarrow 23.58^K$

$M = 39.80^{K'}$

Max Uplift

$P = -31.32^K$

$V = \cancel{11.91^K} \rightarrow 23.58^K$

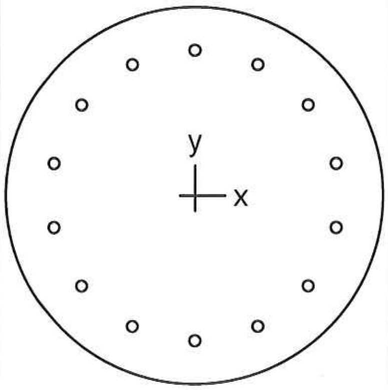
$M = 39.12^{K'}$

Max Shear/Moment

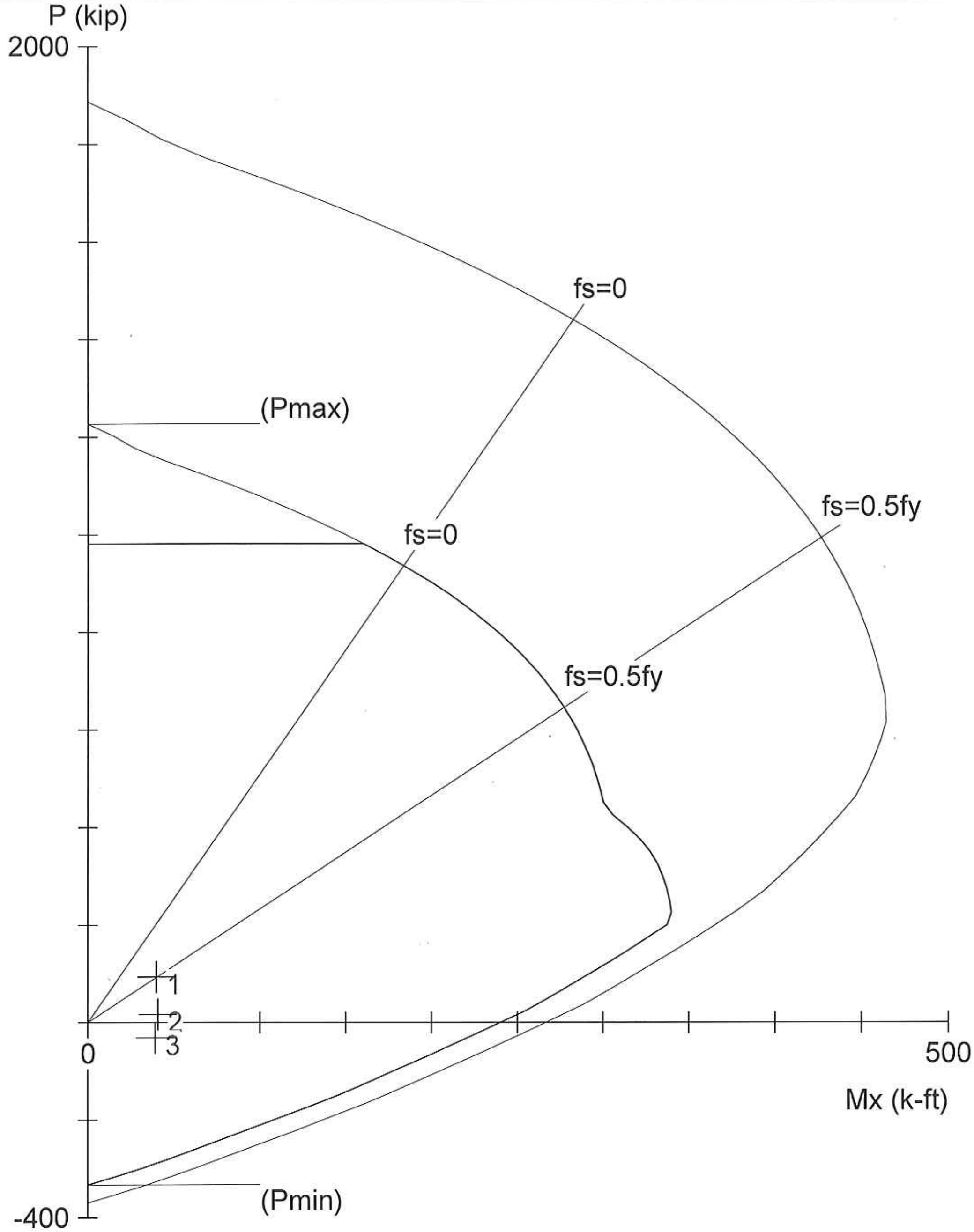
$P = 16.70^K$

$V = \cancel{12.39^K} \rightarrow 23.58^K$

$M = 40.71^{K'}$



24 in diam.



Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 02/27/18

Time: 15:09:33

STRUCTUREPOINT - spColumn v6.00 (TM). Licensed to: Calder Richards Consulting Engineers. License ID: 68548-1061382-4-32C47-28C5

File: x:\project files\2017\17xxx meehan cabin\calculations\piers\round column.col

Project:

Column:

Engineer:

f'c = 4 ksi	fy = 60 ksi	Ag = 452.389 in^2	14 #6 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 6.16 in^2	rho = 1.36%
fc = 3.4 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 16286 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 16286 in^4
Beta1 = 0.85		Min clear spacing = 3.37 in	Clear cover = 2.38 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

RAM BasePlate V1.5

Meehan Cabin
Pier Connections

Detailed Design Results
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CRITERIA:
Analysis
    Maintain Strain Compatibility
    Use min. effective plate area for axial only compression load on plate.
Design
    Use ASD 9th to check plate bending
    Max concrete bearing per AISC J9.
    Anchor Shear Check Per AISC Specifications.
    Anchor Tension Check Per AISC Specifications.

INPUT DATA:
Column
  Column Size............................. W10X49

Dim:	BfTop	TfTop	BfBot	TfBot	TW	Depth
(in)	10.00	0.560	10.00	0.560	0.340	9.98

Base Plate
  Plate Fy (ksi) 36.000
  N (Parallel to Web) (in)................ 18.000
  B (Perpendicular to Web) (in)........... 18.000
  Plate Thickness (in).................... 1.500
Anchor
  Anchor Size............................. 1 1/8"
  Anchor Area (in^2)...................... 0.994
  Anchor Material......................... A325-105
  Anchor Modulus (ksi) 29000.00
  Anchor Strength Fu (ksi) 105.00
  Thread Included in Shear Plane
Footing
  Footing Strength f'c (ksi) 3.00
  Concrete Modulus (ksi) 3122.02
  Dimension (Parallel to web) (ft)........ 2.00
  Dimension (Perpendicular to web) (ft)... 2.00
Design Load
  Building Code: - None -
  Load combination: Single Load Case
  Axial (kip)......................... -31.32
  Vx (kip)............................ 23.58
  Mx (kip-ft)......................... 40.71
  Allowable Stress Increase Factor 1.00

RESULTS:
Analysis
  YBar (in) 4.23
  Resultant Angle (°)............................... 0.00
Plate Bending
  Max bending moment from anchor/s #1 in tension
  Allowable Stress Increase Factor 1.00
  m [N-0.95d]/2.0 (in).............................. 4.260
  n [B-0.80b]/2.0 (in).............................. 5.000
  Controlling effective width to resist moment (in) ... 4.750
  Controlling plate bending moment (kip-ft) 2.85
  fb (ksi) 19.21
  Fb (ksi) 27.00
  fb/Fb 0.71
  Thickness Required (in)........................... 1.265

RAM BasePlate V1.5

Meehan Cabin
Pier Connections

Detailed Design Results
2/27/18 14:46

Thickness controlled by cantilever action.

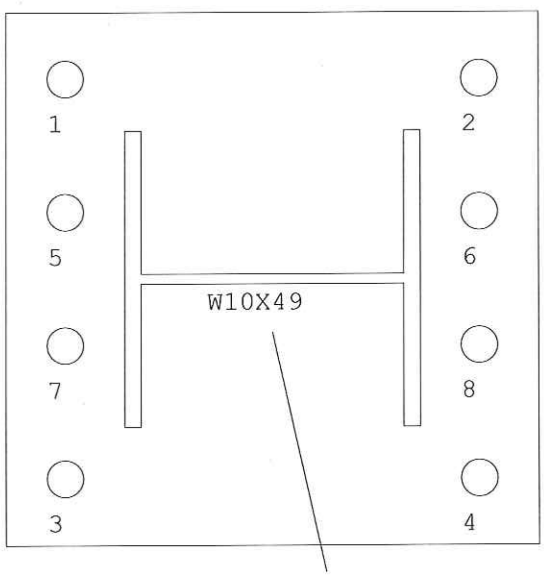
Anchors

Anchor	X(in)	Y(in)	V(kip)	T(kip)	Interaction
1	-7.00	6.75	2.95	12.44	0.29
2	7.00	6.75	2.95	0.00	0.14
3	-7.00	-6.75	2.95	12.44	0.29
4	7.00	-6.75	2.95	0.00	0.14
5	-7.00	2.25	2.95	12.44	0.29
6	7.00	2.25	2.95	0.00	0.14
7	-7.00	-2.25	2.95	12.44	0.29
8	7.00	-2.25	2.95	0.00	0.14

Bearing

Eff Area of Support A2 (in^2)........................ 576.00
Plate Area A1 (in^2)................................. 324.00
Sqrt(A2/A1).. 1.33
Allowable Bearing Pressure (ksi) 1.40
Actual Bearing Stress (ksi) 0.48

DIAGRAM:



#	X(in)	Y(in)
1	-7.000	6.750
2	7.000	6.750
3	-7.000	-6.750
4	7.000	-6.750
5	-7.000	2.250
6	7.000	2.250
7	-7.000	-2.250
8	7.000	-2.250

PL 18.00 X 18.00 X 1.50 (in)
8 - 1 1/8" A325 Anchor Bolts

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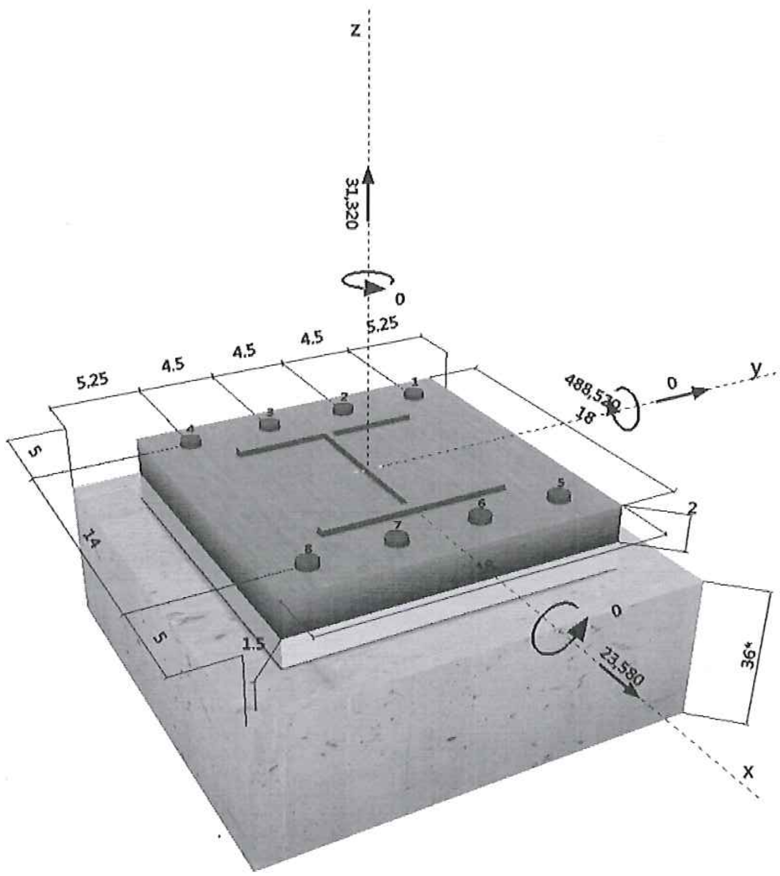
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	2/27/2018
E-Mail:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1 1/8**
Additional plate or washer (17.4.2.8):	d_{plate} = 2.000 in., t_{plate} = 0.375 in.
Effective embedment depth:	h_{ef} = 24.000 in., $h_{ef,17.4.2.8}$ = 0.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	without clamping (anchor); restraint level (anchor plate): 2.00; e_b = 1.500 in.; t = 2.000 in.
	Hilti Grout: CB-G EG, epoxy, $f_{c,Grout}$ = 14939 psi
Anchor plate:	l_x x l_y x t = 18.000 in. x 18.000 in. x 2.000 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC); (L x W x T x FT) = 9.980 in. x 10.000 in. x 0.340 in. x 0.560 in.
Base material:	cracked concrete, 2500, f_c' = 2500 psi; h = 36.000 in.
Reinforcement:	tension: condition A, shear: condition A; anchor reinforcement: tension, shear
	edge reinforcement: > No. 4 bar with stirrups
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 [lb, in.lb]



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E-Mail:			

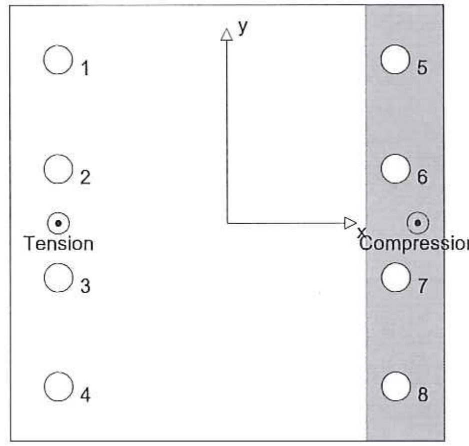
2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	12343	2947	2947	0
2	12343	2947	2947	0
3	12343	2947	2947	0
4	12343	2947	2947	0
5	0	2947	2947	0
6	0	2947	2947	0
7	0	2947	2947	0
8	0	2947	2947	0



max. concrete compressive strain: 0.14 [‰]
max. concrete compressive stress: 618 [psi]
resulting tension force in (x/y)=(-7.000/0.000): 49371 [lb]
resulting compression force in (x/y)=(7.918/0.000): 18051 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	12343	71531	18	OK
Pullout Strength*	12343	19435	64	OK
Concrete Breakout Strength**[1]	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction x-**	49371	44387	112	not recommended

OKAY. SEE CALCS.

* 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]
0.76	125001

Calculations

N_{sa} [lb]
95375

Results

N_{sa} [lb]	ϕ_{steel}	ϕ N_{sa} [lb]	N_{ua} [lb]
95375	0.750	71531	12343

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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	1.85	1.000	2500

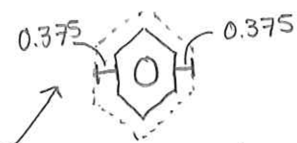
Calculations

N_p [lb]
37020

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{pn} [lb]	N_{ua} [lb]
37020	0.700	0.750	1.000	19435	12343

3.3 Concrete Side-Face Blowout, direction x-



Accounts for thickness of the plate washer

$N_{sb} = 160\, c_{a1} \sqrt{A_{brg}}\, \lambda_a \sqrt{f'_c}$ ACI 318-14 Eq. (17.4.4.1)

$N_{sbg} = \alpha_{group} N_{sb}$ ACI 318-14 Eq. (17.4.4.2)

$\phi N_{sbg} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

$\alpha_{group} = \left(1 + \frac{s}{6\, c_{a1}}\right)$ see ACI 318-14, Section 17.4.4.2, Eq. (17.4.4.2)

Variables

c_{a1} [in.]	c_{a2} [in.]	A_{brg} [in.^{2}]	λ_a	f'_c [psi]	s [in.]
5.000	5.250	$\cancel{1.85}$ 4.696	1.000	2500	13.500

Calculations

α_{group}	N_{sb} [lb]
1.450	54421

Results

N_{sbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{sbg} [lb]	$N_{ua,edge}$ [lb]
78910	0.750	0.750	1.000	44387	49371

$N_{sb} = 160 \times 5 \times \sqrt{4.696} \times 1.0 \times \sqrt{2500} = 86{,}681$ lbs

$N_{sbg} = 1.45 \times 86681 = 125{,}687$ lb

$\phi N_{sbg} = 0.75 \times 125{,}687 = 94{,}266$ lbs $> 49{,}371$ lbs.

Unity $= 49{,}371 / 94{,}266 = 0.52 \Rightarrow 52\%$

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	2947	29757	10	OK
Steel failure (with lever arm)*	2947	4727	63	OK
Pryout Strength**	23580	45506	52	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]
0.76	125001

Calculations

V_{sa} [lb]
57225

Results

V_{sa} [lb]	ϕ_{steel}	ϕ_{eb}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
57225	0.650	0.800	29757	2947

4.2 Steel failure (with lever arm)

V_s^M	$= \frac{\alpha_M \cdot M_s}{L_b}$	bending equation for stand-off
M_s	$= M_s^0 \left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$	resultant flexural resistance of anchor
M_s^0	$= (1.2)\ (S)\ (f_{u,min})$	characteristic flexural resistance of anchor
$\left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$		reduction for tensile force acting simultaneously with a shear force on the anchor
S	$= \frac{\pi (d)^3}{32}$	elastic section modulus of anchor bolt at concrete surface
L_b	$= z + (n)(d_0)$	internal lever arm adjusted for spalling of the surface concrete
$\phi\ V_s^M$	$\geq V_{ua}$	ACI 318-14 Table 17.3.1.1

Variables

α_M	$f_{u,min}$ [psi]	N_{ua} [lb]	$\phi\ N_{sa}$ [lb]	z [in.]	n	d_0 [in.]
2.00	125001	12343	71531	2.500	0.500	1.125

Calculations

M_s^0 [in.lb]	$\left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$	M_s [in.lb]	L_b [in.]
13456.937	0.827	11134.929	3.063

Results

V_s^M [lb]	ϕ_{steel}	$\phi\ V_s^M$ [lb]	V_{ua} [lb]
7272	0.650	4727	2947

4.3 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	4.667	0.000	0.000	5.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	2500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
576.00	196.00	1.000	1.000	0.914	1.000	12097

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
65008	0.700	1.000	1.000	45506	23580

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
1.112	0.624	1.000	145	not recommended

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

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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.
- ACI 318 does not specifically address anchor bending when a stand-off condition exists. PROFIS Anchor calculates a shear load corresponding to anchor bending when stand-off exists and includes the results as a shear Design Strength!
- 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.
- 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 does not meet the design criteria!

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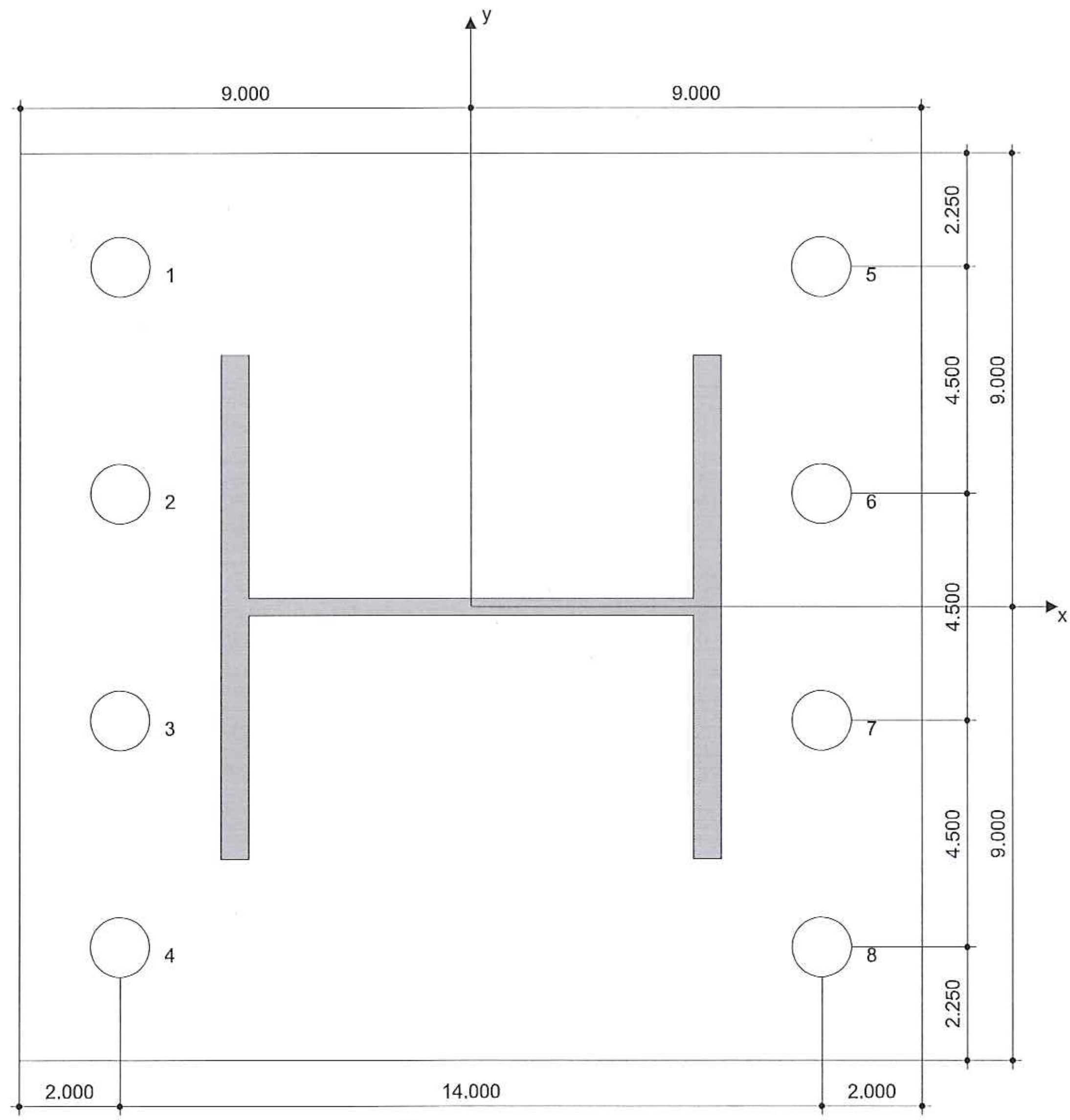
7 Installation data

Anchor plate, steel: -
Profile: W shape (AISC); 9.980 x 10.000 x 0.340 x 0.560 in.
Hole diameter in the fixture: d_f = 1.188 in.
Plate thickness (input): 2.000 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. 105 1 1/8
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 24.000 in.
Minimum thickness of the base material: 25.250 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-7.000	6.750	5.000	19.000	18.750	5.250
2	-7.000	2.250	5.000	19.000	14.250	9.750
3	-7.000	-2.250	5.000	19.000	9.750	14.250
4	-7.000	-6.750	5.000	19.000	5.250	18.750

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
5	7.000	6.750	19.000	5.000	18.750	5.250
6	7.000	2.250	19.000	5.000	14.250	9.750
7	7.000	-2.250	19.000	5.000	9.750	14.250
8	7.000	-6.750	19.000	5.000	5.250	18.750

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.

CALDER RICHARDS CONSULTING ENGINEERS	JOB TITLE Meehan Cabin	BY DSM	DATE 02/2018
	SUBJECT Column Base Loads	CHECKED	SHEET OF

Columns On Wall

Required Axial

Compression = 35.81^k LRFD

Tension = -25.49^k LRFD

Required Shear

D2.5b: V = 47.66^k LRFD

D2.5c: $V = M_{pc}/H = 50\,ksi \times 60.4\,in^3 / 9' \times 12 = 27.96^k$ LRFD

Required Flexure

a) $1.1 \times R_y \times F_y \times Z = 1.1 \times 1.1 \times 50\,ksi \times 60.4\,in^3 = 304.5^{k.ft}$ LRFD

b) Overstrength = $233.95^{k.'}$

Max Compression

$P = 35.81^k$

$V = 43.30^k$

$M = 210.21^{k.'}$

Max Uplift

$P = -25.49^k$

$V = 42.26^k$

$M = 207.79^{k.'}$

Max Shear/Moment

$P = 33.83^k$

$V = 47.66^k$

$M = 234.58^{k.'}$

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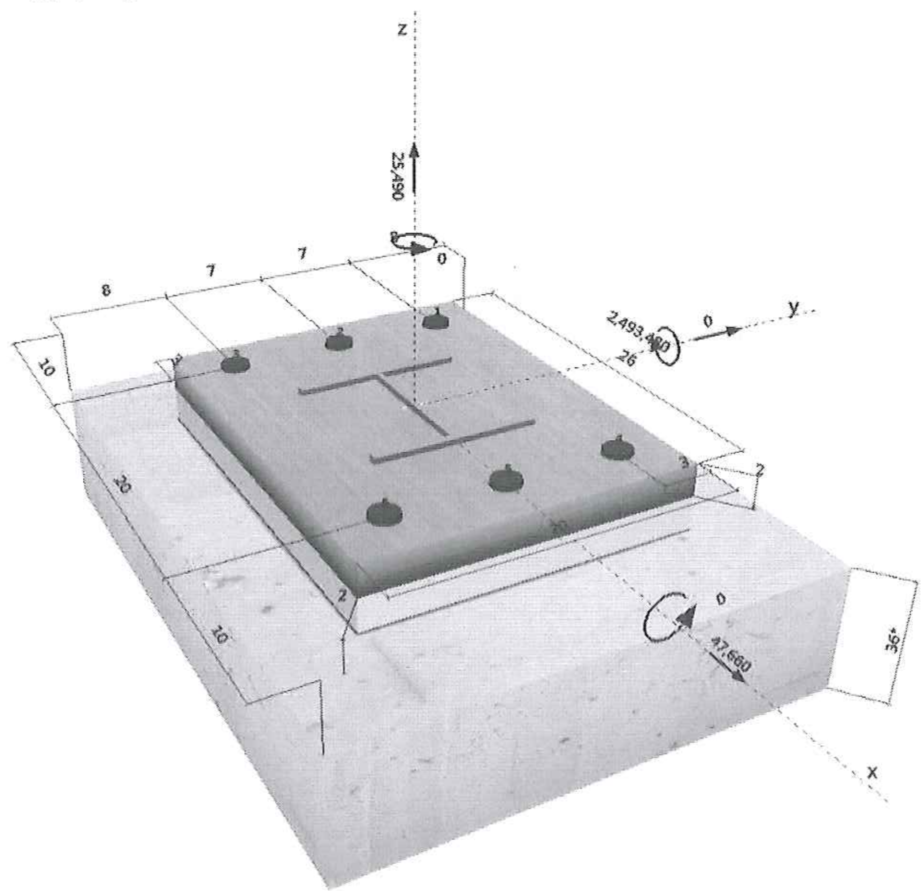
Specifier's comments:

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1 3/4**
Additional plate or washer (17.4.2.8):	d_{plate} = 3.000 in., t_{plate} = 0.500 in.
Effective embedment depth:	h_{ef} = 24.000 in., $h_{ef,17.4.2.8}$ = 0.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	without clamping (anchor); restraint level (anchor plate): 2.00; e_b = 2.000 in.; t = 2.000 in.
	Hilti Grout: CB-G EG, epoxy, $f_{c,Grout}$ = 14939 psi
Anchor plate:	l_x x l_y x t = 26.000 in. x 20.000 in. x 2.000 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC); (L x W x T x FT) = 9.980 in. x 10.000 in. x 0.340 in. x 0.560 in.
Base material:	cracked concrete, 4000, f_c' = 4000 psi; h = 36.000 in.
Reinforcement:	tension: condition A, shear: condition A; anchor reinforcement: tension, shear
	edge reinforcement: > No. 4 bar with stirrups
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 [lb, in.lb]



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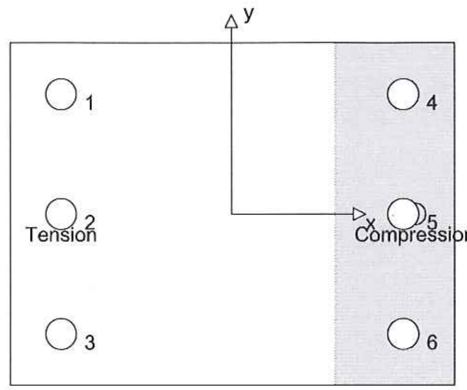
2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	44610	7943	7943	0
2	44610	7943	7943	0
3	44610	7943	7943	0
4	0	7943	7943	0
5	0	7943	7943	0
6	0	7943	7943	0

max. concrete compressive strain: 0.35 [‰]
max. concrete compressive stress: 1545 [psi]
resulting tension force in (x/y)=(-10.000/0.000): 133830 [lb]
resulting compression force in (x/y)=(10.662/0.000): 108340 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	44610	178125	26	OK
Pullout Strength*	44610	69619	65	OK
Concrete Breakout Strength**[1]	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction y-**	44610	92698	49	OK

* 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]
1.90	125001

Calculations

N_{sa} [lb]
237500

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
237500	0.750	178125	44610

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	4.14	1.000	4000

Calculations

N_p [lb]
132608

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{pn} [lb]	N_{ua} [lb]
132608	0.700	0.750	1.000	69619	44610

3.3 Concrete Side-Face Blowout, direction y-

$N_{sb} = 160\, c_{a1} \sqrt{A_{brg}}\, \lambda_a \sqrt{f'_c}$ ACI 318-14 Eq. (17.4.4.1)
$N_{sbg} = \alpha_{group} N_{sb}$ ACI 318-14 Eq. (17.4.4.2)
$\phi N_{sbg} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1
$\alpha_{group} = \left(1 + \frac{s}{6\, c_{a1}}\right)$ see ACI 318-14, Section 17.4.4.2, Eq. (17.4.4.2)

Variables

c_{a1} [in.]	c_{a2} [in.]	A_{brg} [in.^{2}]	λ_a	f'_c [psi]	s [in.]
8.000	10.000	4.14	1.000	4000	-

Calculations

α_{group}	N_{sb} [lb]
1.000	164797

Results

N_{sbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{sbg} [lb]	$N_{ua,edge}$ [lb]
164797	0.750	0.750	1.000	92698	44610

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	7943	74100	11	OK
Steel failure (with lever arm)*	7943	13354	60	OK
Pryout Strength**	47660	103153	47	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]
1.90	125001

Calculations

V_{sa} [lb]
142500

Results

V_{sa} [lb]	ϕ_{steel}	ϕ_{eb}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
142500	0.650	0.800	74100	7943

4.2 Steel failure (with lever arm)

$V_s^M = \frac{\alpha_M \cdot M_s}{L_b}$ bending equation for stand-off
$M_s = M_s^0 \left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$ resultant flexural resistance of anchor
$M_s^0 = (1.2)\,(S)\,(f_{u,min})$ characteristic flexural resistance of anchor
$\left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$ reduction for tensile force acting simultaneously with a shear force on the anchor
$S = \frac{\pi(d)^3}{32}$ elastic section modulus of anchor bolt at concrete surface
$L_b = z + (n)(d_0)$ internal lever arm adjusted for spalling of the surface concrete
$\phi V_s^M \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

α_M	$f_{u,min}$ [psi]	N_{ua} [lb]	ϕN_{sa} [lb]	z [in.]	n	d_0 [in.]
2.00	125001	44610	178125	3.000	0.500	1.750

Calculations

M_s^0 [in.lb]	$\left(1 - \frac{N_{ua}}{\phi N_{sa}}\right)$	M_s [in.lb]	L_b [in.]
53106.195	0.750	39806.126	3.875

Results

V_s^M [lb]	ϕ_{steel}	ϕV_s^M [lb]	V_{ua} [lb]
20545	0.650	13354	7943

4.3 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	6.667	0.000	0.000	8.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	4000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1200.00	400.00	1.000	1.000	0.940	1.000	26128

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
147361	0.700	1.000	1.000	103153	47660

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.641	0.595	5/3	90	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

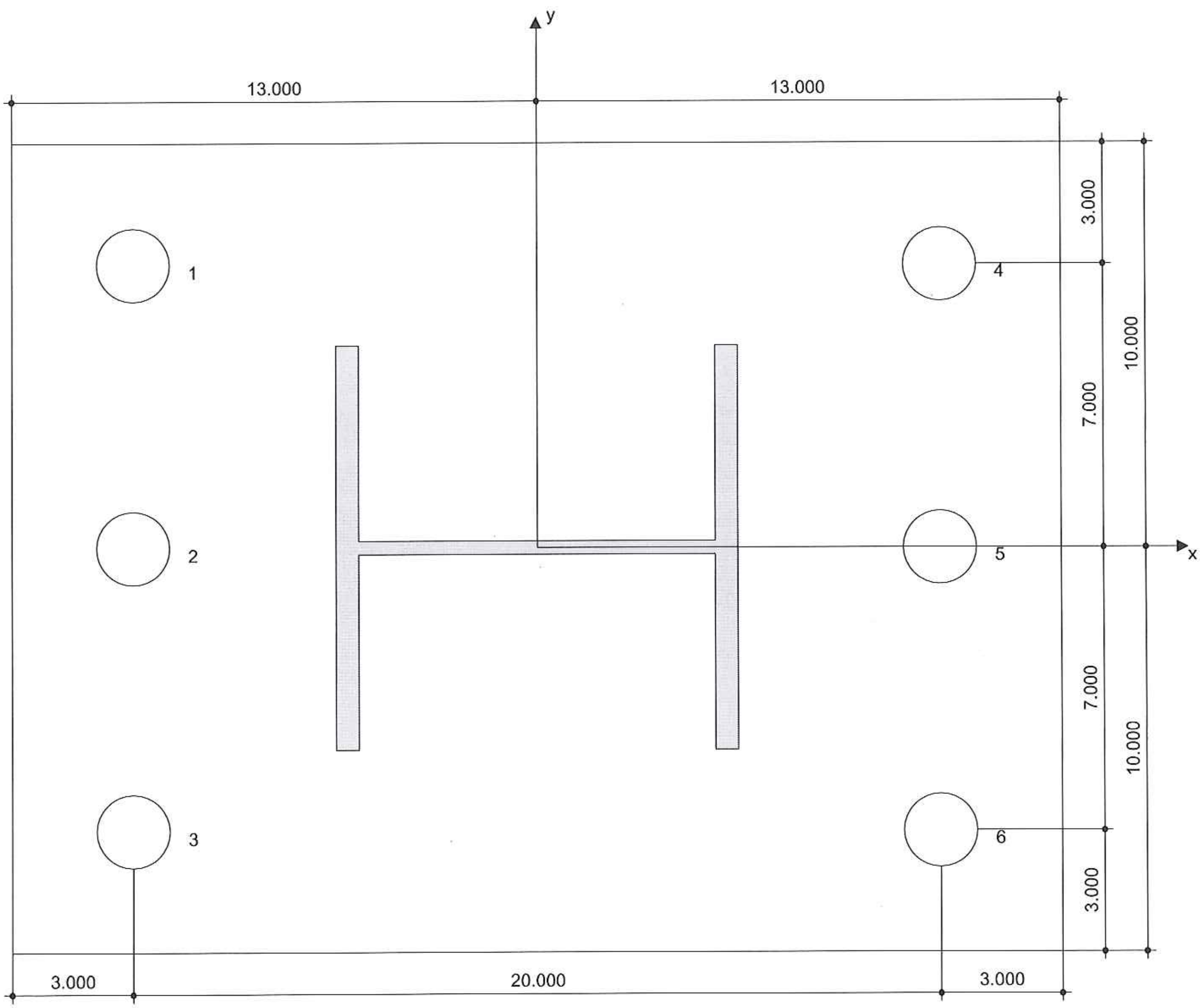
- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, 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.
- ACI 318 does not specifically address anchor bending when a stand-off condition exists. PROFIS Anchor calculates a shear load corresponding to anchor bending when stand-off exists and includes the results as a shear Design Strength!
- 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.
- 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!

7 Installation data

Anchor plate, steel: -
Profile: W shape (AISC); 9.980 x 10.000 x 0.340 x 0.560 in.
Hole diameter in the fixture: d_f = 1.813 in.
Plate thickness (input): 2.000 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. 105 1 3/4
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 24.000 in.
Minimum thickness of the base material: 25.656 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-10.000	7.000	10.000	30.000	22.000	8.000
2	-10.000	0.000	10.000	30.000	15.000	15.000
3	-10.000	-7.000	10.000	30.000	8.000	22.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
4	10.000	7.000	30.000	10.000	22.000	8.000
5	10.000	0.000	30.000	10.000	15.000	15.000
6	10.000	-7.000	30.000	10.000	8.000	22.000

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Phone I Fax:	|	Date:	2/27/2018
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8 Remarks; Your Cooperation Duties

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RAM BasePlate V1.5

Pier Connections

Detailed Design Results
2/27/18 17:38

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CRITERIA:
Analysis
    Maintain Strain Compatibility
    Use min. effective plate area for axial only compression load on plate.
Design
    Use ASD 9th to check plate bending
    Max concrete bearing per AISC J9.
    Anchor Shear Check Per AISC Specifications.
    Anchor Tension Check Per AISC Specifications.

INPUT DATA:
Column
   Column Size..............................  W10X49
   Dim:   BfTop   TfTop   BfBot   TfBot      TW   Depth
   (in)   10.00   0.560   10.00   0.560   0.340    9.98
Base Plate
   Plate Fy (ksi)   ......................    36.000
   N (Parallel to Web) (in)................    26.000
   B (Perpendicular to Web) (in)...........    20.000
   Plate Thickness (in)....................     3.000
Anchor
   Anchor Size.............................    1 3/4"
   Anchor Area (in^2)......................     2.405
   Anchor Material.........................  A325-105
   Anchor Modulus (ksi)   .................  29000.00
   Anchor Strength Fu (ksi)   .............    105.00
   Thread Included in Shear Plane
Footing
   Footing Strength f'c (ksi)   ...........      4.00
   Concrete Modulus (ksi)   ..............   3605.00
   Dimension (Parallel to web) (ft)........      2.50
   Dimension (Perpendicular to web) (ft)...      2.00
Design Load
   Building Code: - None -
   Load combination: Single Load Case
   Axial (kip).........................      -22.27
   Vx (kip)............................       40.63
   Mx (kip-ft).........................      200.92
   Allowable Stress Increase Factor .....      1.00

RESULTS:
Analysis
   YBar (in) ...................................        8.39
   Resultant Angle (°)..........................        0.00
Plate Bending
   Max bending moment from anchor/s #1, 3, 5 in tension
   Allowable Stress Increase Factor ..................     1.00
   m [N-0.95d]/2.0 (in)...............................    8.260
   n [B-0.80b]/2.0 (in)...............................    6.000
   Controlling effective width to resist moment (in) ...  20.000
   Controlling plate bending moment (kip-ft) ..........    57.23
   fb (ksi)   .........................................    22.89
   Fb (ksi)   .........................................    27.00
   fb/Fb ..............................................     0.85
   Thickness Required (in)............................    2.762
```

Thickness controlled by cantilever action.

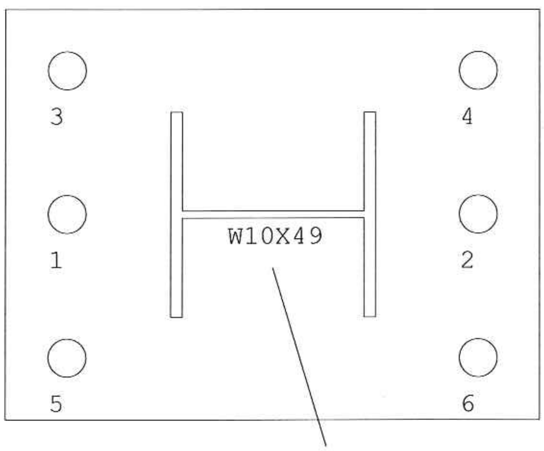
Anchors

Anchor	X(in)	Y(in)	V(kip)	T(kip)	Interaction
1	-10.00	0.00	6.77	43.53	0.42
2	10.00	0.00	6.77	0.00	0.13
3	-10.00	7.00	6.77	43.53	0.42
4	10.00	7.00	6.77	0.00	0.13
5	-10.00	-7.00	6.77	43.53	0.42
6	10.00	-7.00	6.77	0.00	0.13

Bearing

Eff Area of Support A2 (in^2)........................ 692.31
Plate Area A1 (in^2).................................. 520.00
Sqrt(A2/A1)... 1.15
Allowable Bearing Pressure (ksi) 1.62
Actual Bearing Stress (ksi) 1.29

DIAGRAM:



#	X(in)	Y(in)
1	-10.000	0.000
2	10.000	0.000
3	-10.000	7.000
4	10.000	7.000
5	-10.000	-7.000
6	10.000	-7.000

PL 26.00 X 20.00 X 3.00 (in)
6 - 1 3/4" A325 Anchor Bolts