DYNAMIC STRUCTURES, INC.
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Structural Calculations for:

MILLCREEK COMMON
COFFEE SHOP BUILDING

1300 EAST 3300 SOUTH
MILLCREEK, UTAH

PROFESSIONAL STRUCTURAL ENGINEER
NO. 190917
JAY D.
ADAMS
STATE OF UTAH

1-15-21

January 15, 2021

Service Prepared for:

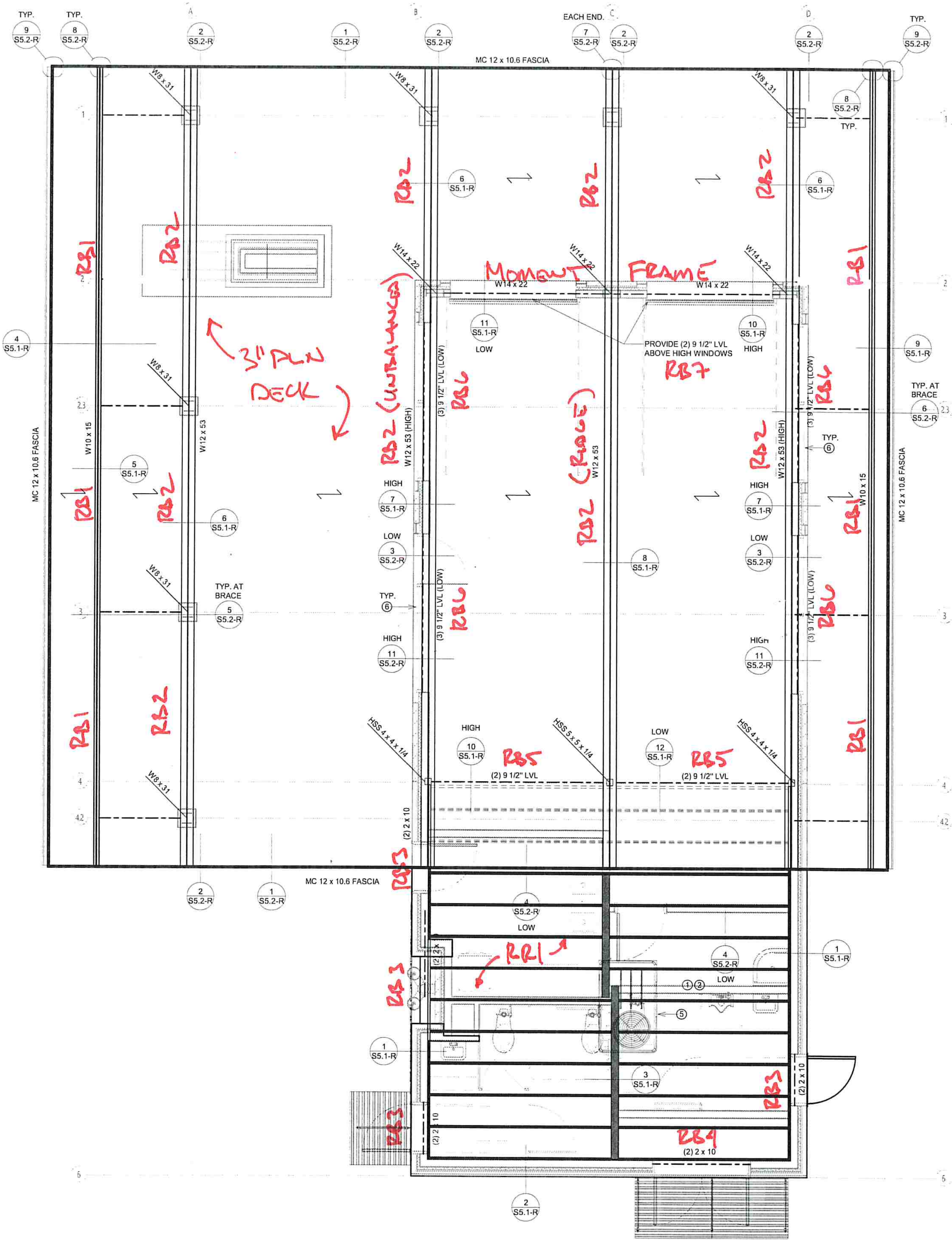
WPA ARCHITECTS

PROJECT:	MILLCREEK COMMON COFFEE SHOP BUILDING 1300 EAST 3300 SOUTH MILLCREEK, UTAH
CLIENT:	WPA ARCHITECTURE
SCOPE:	STRUCTURAL CALCULATIONS AND STRUCTURAL PLANS FOR A NEW RETAIL BUILDING
CODE CRITERIA:	2018 IBC SEISMIC RISK CATEGORY II SEISMIC DESIGN CATEGORY D WIND: 105 MPH EXP. B SNOW: Pg = 43 PSF EXP 0.7 Pf = 30 PSF SOIL: 1750 PSF (PER REPORT)

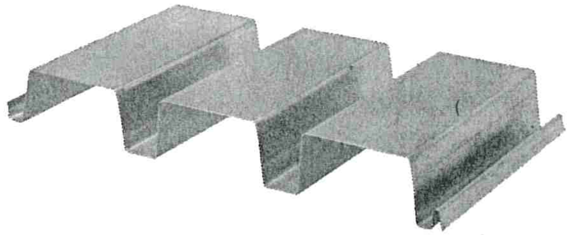
MATERIALS

STEEL:	
ROLLED SECTIONS:	ASTM A992 Fy = 50 ksi
HSS COLUMNS:	ASTM A500 GRADE B Fy = 46 ksi
PLATE, CHANNELS AND ANGLES:	ASTM A36 Fy = 36 KSI
CONCRETE:	
STRENGTH: (DESIGN)	3000 psi (min)
(CONSTRUCTION)	SEE PLAN NOTES
REINFORCING STEEL:	GRADE 60
MASONRY:	
CMU OR ATLAS:	1500 psi
REINFORCING STEEL:	GRADE 60
WOOD:	
STRUCTURAL MEMBERS	DF #2
TIMBERS:	DF #1

DESIGN LOADS:	**Roof Snow load:**	**=30.0 psf**
ROOF DEAD LOADS:	Roofing	=7.50 psf
	Roof Sheathing	=1.70 psf
	Joists	=2.30 psf
	Ceiling	=2.80 psf
	Insulation	=7.40 psf
	Misc	=3.30 psf
	Total roof dead load:	**=25.0 psf**
WALL DEAD LOADS:	Wood framed w/ light finish	=20.0 psf
	Wood framed w/ veneer	=60.0 psf



MC 12 x 10.6 FASCIA
EACH END.
TYP.
W8 x 31
W14 x 22
W12 x 53
W10 x 15
MOMENT FRAME
PROVIDE (2) 9 1/2" LVL ABOVE HIGH WINDOWS
RB7
RB1
RB2
3" PLN DECK
RB2 (UNBALANCED)
RB2 (RIDGE)
RB6
(3) 9 1/2" LVL (LOW)
W12 x 53 (HIGH)
HIGH
LOW
TYP. AT BRACE
HSS 4 x 4 x 1/4
HSS 5 x 5 x 1/4
(2) 9 1/2" LVL
RB5
(2) 2 x 10
RB3
RR1
RB4



Type PLN™-24 or N-24

3" Deep Roof Deck
Primer Painted or Galvanized

VERTICAL LOADS

Allowable Uniform Loads (psf)

25 PSF DL + 30 PSF SL = 55 PSF < 65 PSF

SPAN	GAGE		6'-0"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	13'-0"	14'-0"	15'-0"	16'-0"
SINGLE	22	Stress	160	117	102	90	80	71	64	57	52	47	43	40	34	29	26	22
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	67	57	49	42	37	32	28	22	18	15	12
	20	Stress	206	152	132	116	103	92	82	74	67	61	56	52	44	38	33	29
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	99	83	71	61	53	46	40	35	28	22	18	15
	18	Stress	295	216	189	166	147	131	118	106	96	88	80	74	63	54	47	41
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	138	116	99	85	73	64	56	49	39	31	25	21
	16	Stress	300	278	242	213	188	168	151	136	123	112	103	94	80	69	60	53
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	178	150	128	109	95	82	72	63	50	40	32	27
DOUBLE	22	Stress	195	143	125	110	97	87	78	70	64	58	53	49	41	36	31	27
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦
	20	Stress	241	177	154	136	120	107	96	87	79	72	66	60	51	44	39	34
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦
	18	Stress	300	240	209	184	163	145	130	118	107	97	89	82	70	60	52	46
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦
	16	Stress	300	298	260	229	202	181	162	146	133	121	111	102	87	75	65	57
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦
TRIPLE	22	Stress	243	179	156	137	121	108	97	88	79	72	66	61	52	45	39	34
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	48	39	32	26
	20	Stress	300	221	193	169	150	134	120	108	98	90	82	75	64	55	48	42
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	74	58	47	38	31
	18	Stress	300	300	262	230	204	182	163	147	134	122	111	102	87	75	65	58
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	98	77	62	50	41
	16	Stress	300	300	300	286	253	226	203	183	166	151	138	127	108	93	81	71
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	122	96	77	63	52

Notes:
1. Stress = Uniform load which produces maximum allowable stress in deck.
2. L/240 = Uniform load which produces L/240 deflection in deck.
3. Self-weight of the deck should be included when determining dead load.
4. The symbol ♦♦♦ indicates allowable uniform load based on deflection exceeds allowable uniform load based on stress.

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING RAFTERS
RR1

Span Length: $l := 12 \cdot ft$

Spacing: $s := 24 \cdot in$

Loads per lineal foot: $wll := 30 \cdot 1.43 \cdot psf \cdot s$ UN-BALANCED SNOW LOAD

$wdl := 25 \cdot psf \cdot s$ $w := wdl + wll$

Concentrated loads: $pll := 0 \cdot lb$ $pdl := 0 \cdot lb$

Location of point load: $a := 0 \cdot ft$ $c := l - a$ (use larger distance for a)

$p := pll + pdl$

Calculate the bending moment: $Mx := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8} \right)$ $Mx = 2444.4 \cdot ft \cdot lb$

Calculate the shear: $Vx := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2} \right)$ $Vx = 814.8 \cdot lb$

Reference:\\SERVER\Jobs\Calculation Templates\Reference Tables\I-Joist Values 2013.xmcd

Joist Series: $Series := 210$ $j := if(Series > 110, if(Series > 210, if(Series > 360, 4, 3), 2), 1)$

Joist Depth: $d := 9.5 \cdot in$ $i := if(d > 9.5 \cdot in, if(d > 11.875 \cdot in, if(d > 14 \cdot in, 4, 3), 2), 1)$

USE: 9 1/2" TJI 210 @ 24" O.C.	**Joist Capacities**	>	**Required Capacities**
Moment Capacity:	$M_{i,j} = 3000 \cdot ft \cdot lb$	>	$Mx = 2444.4 \cdot ft \cdot lb$
Maximum Reaction:	$V_{i,j} = 1330 \cdot lb$	>	$Vx = 814.8 \cdot lb$
Deflection Constant:	$EI_{i,j} = 186 \cdot 10^6 \cdot in^2 \cdot lb$		

Check deflection:

Total deflection: $$y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot EI_{i,j} \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot EI_{i,j}} + \frac{2.67 \cdot w \cdot l^2}{d \cdot 10^5} \cdot \frac{in}{lb \cdot 12}$$

Compare total deflection with allowable: $\frac{l}{240} = 0.6 \cdot in$ > $y = 0.4 \cdot in$

Gravity Beam Design

RAM SBeam v06.00.00
RETAIL BUILDING
RB1 - WORST CASE

RAM Structural System
Bentley

01/05/21 11:41:25

STEEL CODE: AISC 360-10 LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (18.00,0.00)

Beam Size (User Selected) = W10X15 Fy = 50.0 ksi
Total Beam Length (ft) = 18.00
Mp (kip-ft) = 66.67
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL	PartL
1	0.000	0.015	0.000	0.000
	18.000	0.015	0.000	0.000
2	0.000	0.150	0.260	0.000
	18.000	0.150	0.260	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 5.53 kips 1.00Vn = 69.00 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	24.9	9.0	0.0	1.00	0.90	60.00
Controlling		1.2DL+1.6LL	24.9	9.0	0.0	1.00	0.90	60.00

REACTIONS (kips):

	Left	**Right**
DL reaction	1.49	1.49
Max +LL reaction	2.34	2.34
Max +total reaction (factored)	5.53	5.53

DEFLECTIONS:

Dead load (in)	at	9.00 ft	=	-0.195	L/D =	1107
Live load (in)	at	9.00 ft	=	-0.307	L/D =	703
Net Total load (in)	at	9.00 ft	=	-0.502	L/D =	430

Gravity Beam Design

RAM Structural System
Bentley

RAM SBeam v06.00.00
RETAIL BUILDING
RB2 - WORST CASE RIDGE

01/05/21 11:41:11

STEEL CODE: AISC 360-10 LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (36.00,0.00)

Beam Size (User Selected) = W12X53 Fy = 50.0 ksi
Total Beam Length (ft) = 36.00
Cantilever on right (ft) = 5.00
Mp (kip-ft) = 324.58
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL	PartL
1	0.000	0.053	0.000	0.000
	31.000	0.053	0.000	0.000
2	0.000	0.300	0.360	0.000
	31.000	0.300	0.360	0.000
3	31.000	0.053	0.000	0.000
	36.000	0.053	0.000	0.000
4	31.000	0.300	0.360	0.000
	36.000	0.300	0.360	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 15.90 kips 1.00Vn = 125.24 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	117.5	15.3	0.0	1.00	0.90	292.12
	Max -	1.2DL+1.6LL	-12.5	31.0	31.0	1.16	0.90	190.53
Right	Max -	1.2DL+1.6LL	-12.5	31.0	5.0	1.00	0.90	292.12
Controlling		1.2DL+1.6LL	117.5	15.3	0.0	1.00	0.90	292.12

REACTIONS (kips):

	Left	**Right**
DL reaction	5.33	7.38
Max +LL reaction	5.58	7.53
Max -LL reaction	-0.15	0.00
Max +total reaction (factored)	15.32	20.90

DEFLECTIONS:

Center span:

Dead load (in)	at	15.50 ft	=	-0.558	L/D =	667
Live load (in)	at	15.50 ft	=	-0.607	L/D =	613
Net Total load (in)	at	15.50 ft	=	-1.165	L/D =	319

Right cantilever:

Dead load (in)	=	0.271	L/D =	442
Pos Live load (in)	=	-0.037	L/D =	3284
Neg Live load (in)	=	0.313	L/D =	383
Neg Total load (in)	=	0.585	L/D =	205

Gravity Beam Design

RAM Structural System
Bentley

RAM SBeam v06.00.00
RETAIL BUILDING
RB2 - WORST CASE UNBALANCED

01/05/21 11:40:40

STEEL CODE: AISC 360-10 LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (36.00,0.00)

Beam Size (User Selected) = W12X53 Fy = 50.0 ksi
Total Beam Length (ft) = 36.00
Cantilever on right (ft) = 5.00
Mp (kip-ft) = 324.58
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL	PartL
1	0.000	0.053	0.000	0.000
	31.000	0.053	0.000	0.000
2	0.000	0.340	0.580	0.000
	31.000	0.340	0.580	0.000
3	31.000	0.053	0.000	0.000
	36.000	0.053	0.000	0.000
4	31.000	0.340	0.580	0.000
	36.000	0.340	0.580	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 22.26 kips 1.00Vn = 125.24 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	165.2	15.4	0.0	1.00	0.90	292.12
	Max -	1.2DL+1.6LL	-17.5	31.0	31.0	1.17	0.90	192.08
Right	Max -	1.2DL+1.6LL	-17.5	31.0	5.0	1.00	0.90	292.12
Controlling		1.2DL+1.6LL	165.2	15.4	0.0	1.00	0.90	292.12

REACTIONS (kips):

	Left	**Right**
DL reaction	5.93	8.22
Max +LL reaction	8.99	12.12
Max -LL reaction	-0.23	0.00
Max +total reaction (factored)	21.51	29.26

DEFLECTIONS:

Center span:

Dead load (in)	at	15.50 ft	=	-0.621	L/D	=	599
Live load (in)	at	15.50 ft	=	-0.978	L/D	=	380
Net Total load (in)	at	15.50 ft	=	-1.599	L/D	=	233

Right cantilever:

Dead load (in)	=	0.302	L/D	=	397
Pos Live load (in)	=	-0.059	L/D	=	2038
Neg Live load (in)	=	0.505	L/D	=	238
Neg Total load (in)	=	0.807	L/D	=	149

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB-3

Length: $l := 6\cdot ft$ $\quad a := 0\cdot ft$ (larger)

Concentrated load: $p := 0\cdot lb$ $\quad c := l - a$

Weight per lineal foot: $w := (25 + 30\cdot 1.43)6\cdot plf + 150\cdot plf$

Material: $i := DF2$

Allowable bending stress Fb: $Fb_i = 900\cdot psi$ Allowable shear stress Fv: $Fv_i = 180\cdot psi$

Modulus of elasticity E: $E_i = 1600000\cdot psi$

Cd = load duration factor: $Cd := 1.0$

Cr = repetitive use factor: $Cr := 1.0$

Calculate bending moment: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot \frac{l^2}{8}\right)$ $\quad \underline{M = 2508.3\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot \frac{l}{2}\right)$ $\quad \underline{V = 1672.2\cdot lb}$

Use: **(2) 2 X 10** $\quad b := 3.0\cdot in$ $\quad d := 9.25\cdot in$ $\quad t := if(d \le 12\cdot in, 12\cdot in, d)$

CF = Size factor $\quad C_1 := if(d > 4\cdot in, if(d > 6\cdot in, if(d > 8\cdot in, 1.1, 1.2), 1.3), 1.5)$

(sawn lumber only) $\quad C_1 := if(d > 10\cdot in, if(d > 12\cdot in, 0.9, 1.0), C_2 := C_1$ $\quad C_1 = 1.1$

Cv = volume factor (glu-laminated lumber only) $\quad C_3 := 1\cdot\left(21\cdot\frac{ft}{l}\right)^{.1}\cdot\left(12\cdot\frac{in}{d}\right)^{.1}\cdot\left(5.125\cdot\frac{in}{b}\right)^{.1}$ $\quad C_3 := if(C_3 > 1, 1, C_3)$ $\quad C_3 = 1$

CF = LSL size factor: $C_4 := \left(\frac{12\cdot in}{t}\right)^{0.092}$ $\quad C_4 = 1$ $\quad$ CF = LVL size factor: $C_5 := \left(\frac{12\cdot in}{t}\right)^{0.136}$ $\quad C_5 = 1$

CF = PSL size factor: $C_6 := \left(\frac{12\cdot in}{t}\right)^{0.111}$ $\quad C_6 = 1$ $\quad C_8 := C_1$ $\quad C_9 := C_1$

Required section modulus: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\quad \underline{S = 30.4\cdot in^3}$

Actual section modulus: $S := \frac{b}{6}\cdot d^2$ $\quad S = 42.8\cdot in^3$

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\quad \underline{A = 10.4\cdot in^2}$

Actual area: $A := b\cdot d$ $\quad A = 27.7\cdot in^2$

Check deflection: $I := \frac{b}{12}\cdot d^3$ $\quad I = 197.9\cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.3\cdot in$

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $\quad y = 0.05\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB-4

Length: $l := 6 \cdot ft$ $a := 0 \cdot ft$ (larger)

Concentrated load: $p := 0 \cdot lb$ $c := l - a$

Weight per lineal foot: $w := (25 + 30 \cdot 1.43) 1 \cdot plf + 150 \cdot plf$

Material: $i := DF2$

Allowable bending stress Fb: $Fb_i = 900 \cdot psi$ Allowable shear stress Fv: $Fv_i = 180 \cdot psi$

Modulus of elasticity E: $E_i = 1600000 \cdot psi$

Cd = load duration factor: $Cd := 1.0$

Cr = repetitive use factor: $Cr := 1.0$

Calculate bending moment: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8} \right)$ $\underline{M = 980.6 \cdot ft \cdot lb}$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2} \right)$ $\underline{V = 653.7 \cdot lb}$

Use: **(2) 2 X 10** $b := 3.0 \cdot in$ $d := 9.25 \cdot in$ $t := if(d \leq 12 \cdot in, 12 \cdot in, d)$

CF = Size factor $C_1 := if(d > 4 \cdot in, if(d > 6 \cdot in, if(d > 8 \cdot in, 1.1, 1.2), 1.3), 1.5)$

(sawn lumber only) $C_1 := if(d > 10 \cdot in, if(d > 12 \cdot in, 0.9, 1.0), (C_2 := C_1$ $C_1 = 1.1$

Cv = volume factor (glu-laminated lumber only) $C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1}$ $C_3 := if(C_3 > 1, 1, C_3)$ $C_3 = 1$

CF = LSL size factor: $C_4 := \left(\frac{12 \cdot in}{t}\right)^{0.092}$ $C_4 = 1$ CF = LVL size factor: $C_5 := \left(\frac{12 \cdot in}{t}\right)^{0.136}$ $C_5 = 1$

CF = PSL size factor: $C_6 := \left(\frac{12 \cdot in}{t}\right)^{0.111}$ $C_6 = 1$ $C_8 := C_1$ $C_9 := C_1$

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $\underline{S = 11.9 \cdot in^3}$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 42.8 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i} \right)$ $\underline{A = 4 \cdot in^2}$

Actual area: $A := b \cdot d$ $A = 27.7 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 197.9 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.3 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.02 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB-5

Length: $l := 12 \cdot ft$ $a := 0 \cdot ft$ (larger)

Concentrated load: $p := 0 \cdot lb$ $c := l - a$

Weight per lineal foot: $w := (25 + 30 \cdot 1.43)1 \cdot plf + 150 \cdot plf$

Material: $i := LVL$

Allowable bending stress Fb: $Fb_i = 2600 \cdot psi$ Allowable shear stress Fv: $Fv_i = 285 \cdot psi$

Modulus of elasticity E: $E_i = 2000000 \cdot psi$

Cd = load duration factor: $Cd := 1.0$

Cr = repetitive use factor: $Cr := 1.0$

Calculate bending moment: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 3922.2 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 1307.4 \cdot lb$

Use: **(2) 1 3/4 X 9 1/2 LVL** $b := 3.5 \cdot in$ $d := 9.5 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

CF = Size factor $C_i := if(d > 4 \cdot in, if(d > 6 \cdot in, if(d > 8 \cdot in, 1.1, 1.2), 1.3), 1.5)$

(sawn lumber only) $C_1 := if(d > 10 \cdot in, if(d > 12 \cdot in, 0.9, 1.0), C_2 := C_1$ $C_1 = 1.1$

Cv = volume factor
(glu-laminated lumber only) $C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1}$ $C_3 := if(C_3 > 1, 1, C_3)$ $C_3 = 1$

CF = LSL size factor: $C_4 := \left(\frac{12 \cdot in}{t}\right)^{0.092}$ $C_4 = 1$ CF = LVL size factor: $C_5 := \left(\frac{12 \cdot in}{t}\right)^{0.136}$ $C_5 = 1$

CF = PSL size factor: $C_6 := \left(\frac{12 \cdot in}{t}\right)^{0.111}$ $C_6 = 1$ $C_8 := C_1$ $C_9 := C_1$

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 18.1 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 52.6 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 6 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 33.2 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 250.1 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.6 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.2 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB-6

Length:	$l := 10 \cdot ft$	$a := 0 \cdot ft$	(larger)
Concentrated load:	$p := 0 \cdot lb$	$c := l - a$	
Weight per lineal foot:	$w := 250 \cdot plf$		
Material:	$i := LVL$		

Allowable bending stress Fb: $Fb_i = 2600 \cdot psi$ Allowable shear stress Fv: $Fv_i = 285 \cdot psi$

Modulus of elasticity E: $E_i = 2000000 \cdot psi$

Cd = load duration factor: $Cd := 1.0$
Cr = repetitive use factor: $Cr := 1.0$

Calculate bending moment: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 3125 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 1250 \cdot lb$

Use: **(3) 1 3/4 X 9 1/2 LVL** $b := 5.25 \cdot in$ $d := 9.5 \cdot in$ $t := \text{if}(d \le 12 \cdot in, 12 \cdot in, d)$

CF = Size factor $C_1 := \text{if}(d > 4 \cdot in, \text{if}(d > 6 \cdot in, \text{if}(d > 8 \cdot in, 1.1, 1.2), 1.3), 1.5)$

(sawn lumber only) $C_1 := \text{if}(d > 10 \cdot in, \text{if}(d > 12 \cdot in, 0.9, 1.0), \text{(}C_2 := C_1$ $C_1 = 1.1$

Cv = volume factor
(glu-laminated lumber only) $C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1}$ $C_3 := \text{if}(C_3 > 1, 1, C_3)$ $C_3 = 1$

CF = LSL size factor: $C_4 := \left(\frac{12 \cdot in}{t}\right)^{0.092}$ $C_4 = 1$ CF = LVL size factor: $C_5 := \left(\frac{12 \cdot in}{t}\right)^{0.136}$ $C_5 = 1$

CF = PSL size factor: $C_6 := \left(\frac{12 \cdot in}{t}\right)^{0.111}$ $C_6 = 1$ $C_8 := C_1$ $C_9 := C_1$

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 14.4 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 79 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 5.5 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 49.9 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 375.1 \cdot in^4$

Allowable deflection: $\frac{l}{600} = 0.2 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.07 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB-7

Length: $l := 8 \cdot ft$ $a := 0 \cdot ft$ (larger)

Concentrated load: $p := 0 \cdot lb$ $c := l - a$

Weight per lineal foot: $w := 150 \cdot plf$

Material: $i := LVL$

Allowable bending stress Fb: $Fb_i = 2600 \cdot psi$ Allowable shear stress Fv: $Fv_i = 285 \cdot psi$

Modulus of elasticity E: $E_i = 2000000 \cdot psi$

Cd = load duration factor: $Cd := 1.0$

Cr = repetitive use factor: $Cr := 1.0$

Calculate bending moment: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 1200 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 600 \cdot lb$

Use: **(2) 1 3/4 X 9 1/2 LVL** $b := 3.5 \cdot in$ $d := 9.5 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

CF = Size factor $C_1 := if(d > 4 \cdot in, if(d > 6 \cdot in, if(d > 8 \cdot in, 1.1, 1.2), 1.3), 1.5)$

(sawn lumber only) $C_1 := if(d > 10 \cdot in, if(d > 12 \cdot in, 0.9, 1.0), C_2 := C_1$ $C_1 = 1.1$

Cv = volume factor
(glu-laminated lumber only) $C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1}$ $C_3 := if(C_3 > 1, 1, C_3)$ $C_3 = 1$

CF = LSL size factor: $C_4 := \left(\frac{12 \cdot in}{t}\right)^{0.092}$ $C_4 = 1$ CF = LVL size factor: $C_5 := \left(\frac{12 \cdot in}{t}\right)^{0.136}$ $C_5 = 1$

CF = PSL size factor: $C_6 := \left(\frac{12 \cdot in}{t}\right)^{0.111}$ $C_6 = 1$ $C_8 := C_1$ $C_9 := C_1$

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 5.5 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 52.6 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 2.5 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 33.2 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 250.1 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.4 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.03 \cdot in$

Gravity Beam Design

RAM Structural System
Bentley

RAM SBeam v06.00.00
MILLCREEK COMMONS
VENEER LINTEL

01/05/21 11:40:18

STEEL CODE: AISC 360-10 LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (12.00,0.00)

Beam Size (User Selected) = C8X11.5 Fy = 36.0 ksi
Total Beam Length (ft) = 12.00
Mp (kip-ft) = 28.89
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL	PartL
1	0.000	0.011	0.000	0.000
	12.000	0.011	0.000	0.000
2	0.000	0.300	0.000	0.000
	12.000	0.300	0.000	0.000

SHEAR (Ultimate): Max Vu (1.4DL) = 2.62 kips 0.90Vn = 34.21 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.4DL	7.8	6.0	0.0	1.00	0.90	26.00
Controlling		1.4DL	7.8	6.0	0.0	1.00	0.90	26.00

REACTIONS (kips):

	Left	**Right**
DL reaction	1.87	1.87
Max +total reaction (factored)	2.62	2.62

DEFLECTIONS:

Dead load (in)	at	6.00 ft	=	-0.154	L/D = 934
Live load (in)	at	6.00 ft	=	-0.000	
Net Total load (in)	at	6.00 ft	=	-0.154	L/D = 934

HSS COLUMN - GRID 4, B AND D

Unsupported Length: $L := 16 \cdot ft$

Yield Stress: $Fy := 46 \cdot ksi$

Modulus of elasticity: $E := 29000 \cdot ksi$

Effective Lenght Factor: $K := 1$

Design Method: (•) LRFD ( ) ASD

HSS 5 X 5 X 1/4

Section :=
HSS 5 x 5 x 1/4
HSS 5 x 5 x 3/16

SaveData("", "Get", 0)

AXIAL LOADS:

Dead Load: $D := 9 \cdot k$

Live Load: $Lf := 0 \cdot k$

Snow Load: $Lr := 13 \cdot k$

MOMENTS (X):

Dead Load: $Mxd := 1 \cdot in \cdot D$

Live Load: $MxLf := 0 \cdot in \cdot Lf$

Snow Load: $MxLr := 1 \cdot in \cdot Lr$

MOMENTS (Y):

Dead Load: $Myd := 0 \cdot in \cdot D$

Live Load: $MyLf := 0 \cdot in \cdot Lf$

Snow Load: $MyLr := 0 \cdot in \cdot Lr$

$$\phi := \begin{cases} 0.9 \text{ if Design} = \text{"LRFD"} \\ 0.6 \text{ if Design} = \text{"ASD"} \end{cases}$$

Design Values:

$Area = 4.3 \cdot in^2$ $r = 1.93 \cdot in$ $Z = 7.61 \cdot in^3$

$t = 0.233 \cdot in$ $b\_t = 18.5$ $S = 6.41 \cdot in^3$

Design Axial Loads:

LRFD:

$P_{L1} := 1.4 \cdot D$

$P_{L2} := 1.2\ D + 1.6\ Lf + 0.5\ Lr$

$P_{L3} := 1.2D + 1.6Lr + 0.5Lf$

ASD:

$P_{A1} := D$

$P_{A2} := D + Lf$

$P_{A3} := D + Lr$

$P_{A4} := D + 0.75Lr + 0.75Lf$

Design Moments (X):

LRFD:

$Mx_{L1} := 1.4 \cdot Mxd$

$Mx_{L2} := 1.2\ Mxd + 1.6\ MxLf + 0.5\ MxLr$

$Mx_{L3} := 1.2Mxd + 1.6MxLr + 0.5MxLf$

ASD:

$Mx_{A1} := Mxd$

$Mx_{A2} := Mxd + MxLf$

$Mx_{A3} := Mxd + MxLr$

$Mx_{A4} := Mxd + 0.75MxLr + 0.75MxLf$

Design Moments (Y):

LRFD:

$My_{L1} := 1.4 \cdot Myd$

$My_{L2} := 1.2\ Myd + 1.6\ MyLf + 0.5\ MyLr$

$My_{L3} := 1.2\ Myd + 1.6\ MyLf + 0.5\ MyLr$

ASD:

$My_{A1} := Myd$

$My_{A2} := Myd + MyLf$

$My_{A3} := Myd + MyLr$

$My_{A4} := Myd + 0.75MyLr + 0.75MyLf$

Check Slenderness of Members:

$\text{Flanges} = \text{"Compact"}$

$$\frac{K \cdot L}{r} = 99.482$$

Calculate A_{eff}:

$$b_e := 1.92 \cdot t \cdot \sqrt{\frac{E}{Fy}} \cdot \left[1 - \left(\frac{.38}{b\_t}\right) \cdot \sqrt{\frac{E}{Fy}}\right] \qquad b_e = 5.439 \cdot in$$

$$b_{useable} := b - 3 \cdot t \qquad b_x := b_{useable} - b_e$$

$$A_{eff} := Area - 4 \cdot t \cdot b_x \qquad A_{eff} = 5.361 \cdot in^2$$

$$Q := \begin{vmatrix} \left(\frac{A_{eff}}{Area}\right) \text{ if Flanges} = \text{"Slender"} \\ 1 \text{ otherwise} \end{vmatrix} \qquad Q = 1$$

$$Fe := \frac{\left(\pi^2 \cdot E\right)}{\left(K \cdot \frac{L}{r}\right)^2} \qquad Fe = 28.9 \cdot ksi$$

$$Fcr := \begin{vmatrix} Q \cdot \left(0.658^{\frac{Q \cdot Fy}{Fe}}\right) \cdot Fy \text{ if } \left[\frac{(K \cdot L)}{r}\right] \le 4.71 \cdot \sqrt{\frac{E}{Q \cdot Fy}} \\ (0.877 \cdot Fe) \text{ otherwise} \end{vmatrix} \qquad Fcr = 23.639 \cdot ksi$$

$$P_n := Fcr \cdot Area \qquad P_n = 102 \cdot kip$$

Design Axial Strength: $\phi \cdot P_n = 91 \cdot kip$

Yielding:

$M_p := Fy \cdot Z$ $\qquad M_p = 29.2 \cdot k \cdot ft$

Flange Local Buckling:

$$I_{eff} := I - 2\left[\left(b_x \cdot \frac{t^3}{12}\right) + \left[b_x \cdot t \cdot \left(\frac{d}{2} - \frac{t}{2}\right)^2\right]\right] \qquad I_{eff} = 19 \cdot in^4$$

$$S_{eff} := I_{eff} \cdot \frac{2}{d} \qquad S_{eff} = 7.6 \cdot in^3$$

$$M_{nf} := \begin{cases} \text{"NA"} & \text{if Flanges = "Compact"} \\ \left(Fy \cdot S_{eff}\right) & \text{if Flanges = "Slender"} \\ M_p - \left(M_p - Fy \cdot S\right) \cdot \left(3.57 \cdot b\_t \cdot \sqrt{\frac{Fy}{E}} - 4.0\right) & \text{otherwise} \end{cases} \qquad M_{nf} = 1 \times 10^{304} \cdot k \cdot ft$$

Web Local Buckling:

$$M_{nw} := \begin{cases} \text{"NA"} & \text{if Flanges = "Compact"} \\ \left[M_p - \left(M_p - Fy \cdot S\right)\left(0.305 \cdot b\_t \cdot \sqrt{\frac{Fy}{E}} - 0.738\right)\right] & \text{otherwise} \end{cases} \qquad M_{nw} = 1 \times 10^{304} \cdot k \cdot ft$$

Design Bending Strength: $\qquad \phi \cdot M_n = 26.3 \cdot k \cdot ft$

Combined Loading

Combined Unity Equation:

$$H1\_1 := \begin{cases} \left(\frac{P_r}{\phi\, P_n}\right) + \left(\frac{8}{9}\right) \cdot \left[\left(\frac{M_{ux}}{\phi \cdot M_n}\right) + \left(\frac{M_{uy}}{\phi \cdot M_n}\right)\right] & \text{if } \left(\frac{P_r}{\phi \cdot P_n}\right) \ge 0.2 \\ \left[\frac{P_r}{2 \cdot \phi\, P_n} + \left[\left(\frac{M_{ux}}{\phi \cdot M_n}\right) + \left(\frac{M_{uy}}{\phi \cdot M_n}\right)\right]\right] & \text{if } \left(\frac{P_r}{\phi \cdot P_n}\right) < 0.2 \end{cases} \qquad H1\_1 = 0.43$$

HSS COLUMN - GRID 4,C

Unsupported Length: $L := 19 \cdot ft$

Yield Stress: $Fy := 46 \cdot ksi$

Modulus of elasticity: $E := 29000 \cdot ksi$

Effective Lenght Factor: $K := 1$

Design Method: ◉ LRFD ○ ASD

HSS 5 X 5 X 1/4

Section := HSS 5 x 5 x 1/4
HSS 5 x 5 x 3/16
SaveData("", "Get", 0)

AXIAL LOADS:

Dead Load: $D := 8 \cdot k$
Live Load: $Lf := 0 \cdot k$
Snow Load: $Lr := 8 \cdot k$

MOMENTS (X):

Dead Load: $Mxd := 1 \cdot in \cdot D$
Live Load: $MxLf := 0 \cdot in \cdot Lf$
Snow Load: $MxLr := 1 \cdot in \cdot Lr$

MOMENTS (Y):

Dead Load: $Myd := 0 \cdot in \cdot D$
Live Load: $MyLf := 0 \cdot in \cdot Lf$
Snow Load: $MyLr := 0 \cdot in \cdot Lr$

$$\phi := \begin{cases} 0.9 & \text{if Design} = \text{"LRFD"} \\ 0.6 & \text{if Design} = \text{"ASD"} \end{cases}$$

Design Values:

$Area = 4.3 \cdot in^2$ $\quad r = 1.93 \cdot in$ $\quad Z = 7.61 \cdot in^3$

$t = 0.233 \cdot in$ $\quad b\_t = 18.5$ $\quad S = 6.41 \cdot in^3$

Design Axial Loads:

LRFD:

$P_{L1} := 1.4 \cdot D$
$P_{L2} := 1.2\ D + 1.6\ Lf + 0.5\ Lr$
$P_{L3} := 1.2D + 1.6Lr + 0.5Lf$

ASD:

$P_{A1} := D$
$P_{A2} := D + Lf$
$P_{A3} := D + Lr$
$P_{A4} := D + 0.75Lr + 0.75Lf$

Design Moments (X):

LRFD:

$Mx_{L1} := 1.4 \cdot Mxd$
$Mx_{L2} := 1.2\ Mxd + 1.6\ MxLf + 0.5\ MxLr$
$Mx_{L3} := 1.2Mxd + 1.6MxLr + 0.5MxLf$

ASD:

$Mx_{A1} := Mxd$
$Mx_{A2} := Mxd + MxLf$
$Mx_{A3} := Mxd + MxLr$
$Mx_{A4} := Mxd + 0.75MxLr + 0.75MxLf$

Design Moments (Y):.

LRFD:

$My_{L1} := 1.4 \cdot Myd$
$My_{L2} := 1.2\ Myd + 1.6\ MyLf + 0.5\ MyLr$
$My_{L3} := 1.2\ Myd + 1.6\ MyLf + 0.5\ MyLr$

ASD:

$My_{A1} := Myd$
$My_{A2} := Myd + MyLf$
$My_{A3} := Myd + MyLr$
$My_{A4} := Myd + 0.75MyLr + 0.75MyLf$

Check Slenderness of Members:

$\text{Flanges} = \text{"Compact"}$

$$\frac{K \cdot L}{r} = 118.135$$

Calculate A_{eff}:

$$b_e := 1.92 \cdot t \cdot \sqrt{\frac{E}{Fy}} \cdot \left[1 - \left(\frac{.38}{b\_t}\right) \cdot \sqrt{\frac{E}{Fy}}\right] \qquad b_e = 5.439 \cdot in$$

$$b_{useable} := b - 3 \cdot t \qquad b_x := b_{useable} - b_e$$

$$A_{eff} := Area - 4 \cdot t \cdot b_x \qquad A_{eff} = 5.361 \cdot in^2$$

$$Q := \begin{cases} \left(\dfrac{A_{eff}}{Area}\right) & \text{if Flanges} = \text{"Slender"} \\ 1 & \text{otherwise} \end{cases} \qquad Q = 1$$

$$Fe := \frac{\left(\pi^2 \cdot E\right)}{\left(K \cdot \frac{L}{r}\right)^2} \qquad Fe = 20.5 \cdot ksi$$

$$Fcr := \begin{cases} Q \cdot \left(0.658^{\frac{Q \cdot Fy}{Fe}}\right) \cdot Fy & \text{if } \left[\dfrac{(K \cdot L)}{r}\right] \le 4.71 \cdot \sqrt{\dfrac{E}{Q \cdot Fy}} \\ (0.877 \cdot Fe) & \text{otherwise} \end{cases} \qquad Fcr = 17.991 \cdot ksi$$

$$P_n := Fcr \cdot Area \qquad P_n = 77 \cdot kip$$

Design Axial Strength: $\phi \cdot P_n = 70 \cdot kip$

Yielding:

$M_p := Fy \cdot Z$ $\qquad M_p = 29.2 \cdot k \cdot ft$

Flange Local Buckling:

$$I_{eff} := I - 2\left[\left(b_x \cdot \frac{t^3}{12}\right) + \left[b_x \cdot t \cdot \left(\frac{d}{2} - \frac{t}{2}\right)^2\right]\right] \qquad I_{eff} = 19 \cdot in^4$$

$$S_{eff} := I_{eff} \cdot \frac{2}{d} \qquad S_{eff} = 7.6 \cdot in^3$$

$$M_{nf} := \begin{array}{|l} \text{"NA" if Flanges = "Compact"} \\ \left(Fy \cdot S_{eff}\right) \text{ if Flanges = "Slender"} \\ M_p - \left(M_p - Fy \cdot S\right) \cdot \left(3.57 \cdot b\_t \cdot \sqrt{\frac{Fy}{E}} - 4.0\right) \text{ otherwise} \end{array} \qquad M_{nf} = 1 \times 10^{304} \cdot k \cdot ft$$

Web Local Buckling:

$$M_{nw} := \begin{array}{|l} \text{"NA" if Flanges = "Compact"} \\ \left[M_p - \left(M_p - Fy \cdot S\right)\left(0.305 \cdot b\_t \cdot \sqrt{\frac{Fy}{E}} - 0.738\right)\right] \text{ otherwise} \end{array} \qquad M_{nw} = 1 \times 10^{304} \cdot k \cdot ft$$

Design Bending Strength: $\qquad \phi \cdot M_n = 26.3 \cdot k \cdot ft$

Combined Loading

Combined Unity Equation:

$$H1\_1 := \begin{array}{|l} \left(\frac{P_r}{\phi\, P_n}\right) + \left(\frac{8}{9}\right) \cdot \left[\left(\frac{M_{ux}}{\phi \cdot M_n}\right) + \left(\frac{M_{uy}}{\phi \cdot M_n}\right)\right] \text{ if } \left(\frac{P_r}{\phi \cdot P_n}\right) \ge 0.2 \\ \left[\frac{P_r}{2 \cdot \phi\, P_n} + \left[\left(\frac{M_{ux}}{\phi \cdot M_n}\right) + \left(\frac{M_{uy}}{\phi \cdot M_n}\right)\right]\right] \text{ if } \left(\frac{P_r}{\phi \cdot P_n}\right) < 0.2 \end{array} \qquad H1\_1 = 0.38$$

WF COLUMN - NO BRACE

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	19	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	Y Rot [k-ft/rad]
1	N1	Reaction	Reaction	Reaction	Fixed
2	N2	Reaction		Reaction	Fixed

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Column	W8X31	Column	Wide Flange	A992	Typical	9.13	37.1	110	0.536

Nodal Loads and Enforced Displacements

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]	Inactive [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N2	L	Y	-5	Active
2	N2	L	MX	1.25	Active

Nodal Loads and Enforced Displacements

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]	Inactive [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N2	L	Y	-8	Active
2	N2	L	MX	2	Active

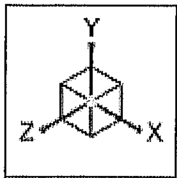
Node Reactions

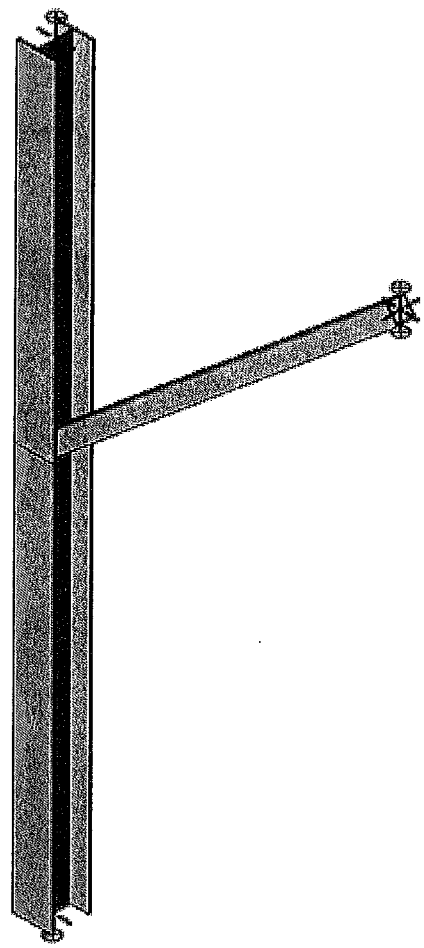
	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N1	0	18.8	0.2474	0	NC	0
2	1	N2	0	0	-0.2474	0	NC	0
3	1	Totals:	0	18.8	0			
4	1	COG (ft):	X: 0	Y: 19	Z: 0			

Member AISC 15th (360-16): LRFD Steel Code Checks

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	phi*Pnc[k]	phi*Pnt[k]	phi*Mnyy[k-ft]	phi*Mnzz[k-ft]	Cb	Eqn
1	1	M1	W8X31	0.1473	19	0.0013	19	z	161.2332	410.85	52.7913	85.8546	1	H1-1b

≤1.0





Dynamic Structures	WF Columns with Braces	SK-2
Jay Adams		Jan 07, 2021
		Column and Brace.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	7	0	
3	N3	0	13	0	
4	N4	6	12	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	Y Rot [k-ft/rad]
1	N1	Reaction	Reaction	Reaction	Fixed
2	N3	Reaction		Reaction	Fixed
3	N4	Reaction		Reaction	Fixed

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Column	W8X31	Column	Wide Flange	A992	Typical	9.13	37.1	110	0.536
2	Brace	HSS4X4X4	Column	SquareTube	A500 Gr.B Rect	Typical	3.37	7.8	7.8	12.8

Nodal Loads and Enforced Displacements

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]	Inactive [(k, k-ft), (in, rad), (k*s²/ft, k*s²*ft)]
1	N4	L	Y	-6	Active
2	N3	L	Y	-11	Active

Node Reactions

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N1	3.4133	17	0	0	NC	0
2	1	N3	4.0783	0	0	0	NC	0
3	1	N4	-7.4916	0	0	0	NC	0
4	1	Totals:	0	17	0			
5	1	COG (ft):	X: 2.1176	Y: 12.6471	Z: 0			

HORIZ. DEFLECTION

Node Deflections

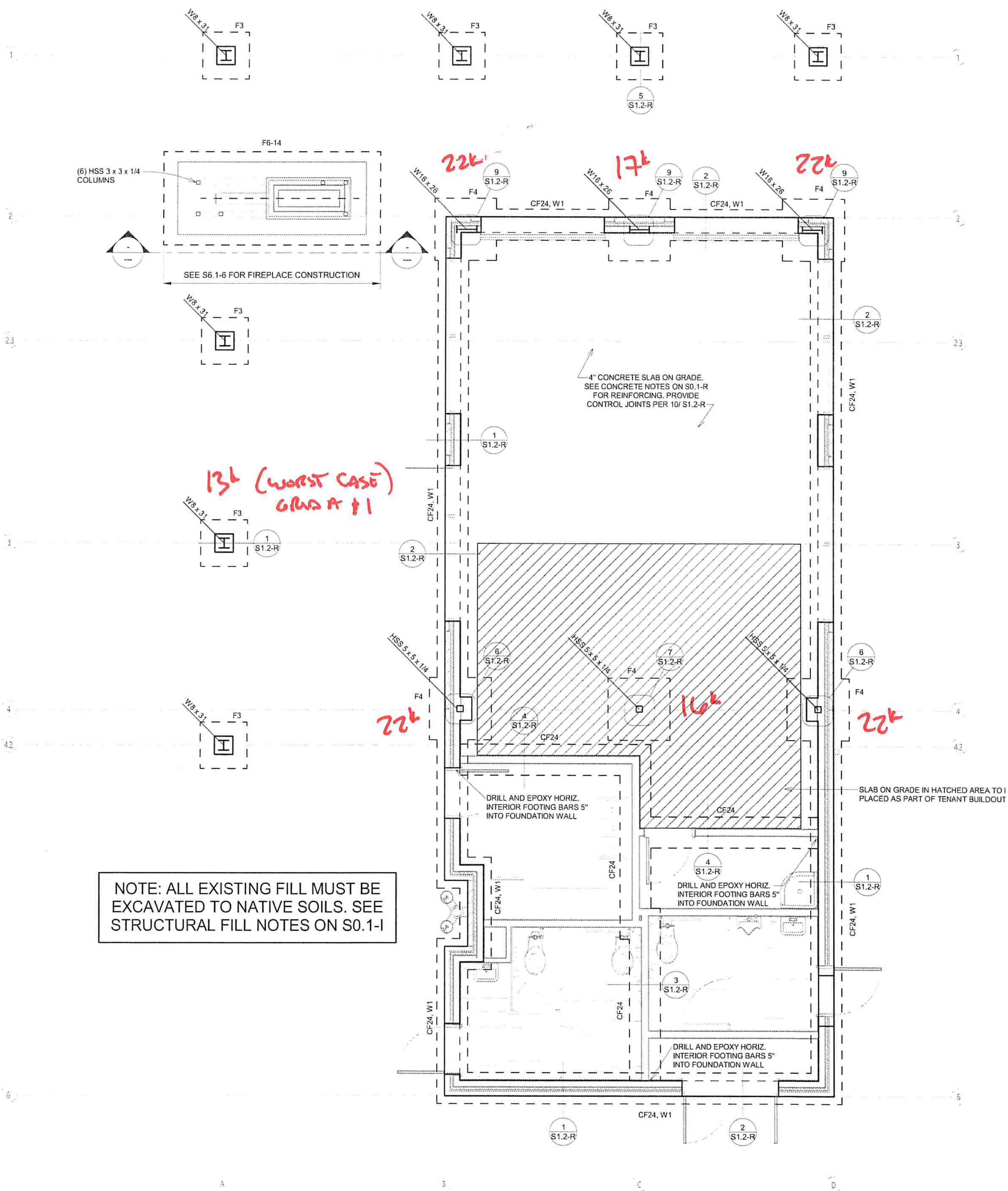
	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
1	1	N1	0	0	0	0	0	1.3524e-2
2	1	N2	-0.7214	-0.0067	0	0	0	-1.4236e-3
3	1	N3	0	-0.0105	0	0	0	-1.4236e-2
4	1	N4	0	-0.8904	0	0	0	-1.2171e-2

Member AISC 15th (360-16): ASD Steel Code Checks

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
1	1	M1	W8X31	0.7621	7	0.0292	7	z	240.7605	273.3533	35.124	75.7667	1	H1-1b
2	1	M2	W8X31	0.7489	0	0.034	6	z	249.0084	273.3533	35.124	75.7667	1	H1-1b
3	1	M3	HSS4X4X4	0.1334	7.8102	0	7.8102	y	71.9117	92.8263	10.7655	10.7655	1	H1-1b*

≤ 1.0

WORST CASE CONTINUOUS FOOTING LOAD = 1200 PLF



CONTINUOUS FOOTING DESIGN

Soil bearing pressure (assumed): $p := 1750 \cdot psf$

Continuous wall load CF24: $w1 := 1200 \cdot plf$ WORST CASE CONTINUOUS FOOTING

Continous footing CF24:

$w := \frac{w1}{p}$ $w = 8.2 \cdot in$ use 24" x 12" x cont. w/(3) #4 cont

Allowable point load on continuous footing:

$P := 24 \cdot in \cdot 1.5 \cdot 24 \cdot in \cdot p$ $P = 10 \cdot k$

Point loads greater than 10 kips require spot footings

SPOT FOOTING DESIGN

Soil bearing pressure:	$p := 1750 \cdot psf$
Maximum point load for spread footing F2:	$P2 := 7000 \cdot lb$
Maximum point load for spread footing F3:	$P3 := 15750 \cdot lb$
Maximum point load for spread footing F4:	$P4 := 28000 \cdot lb$

Spread footing F2:

$w := \sqrt{\frac{P2}{p}}$ $\quad w = 2 \cdot ft$ **USE: 2'-0" x 2'-0" x 10" w/ (3) #4 each way**

Spread footing F3:

$w := \sqrt{\frac{P3}{p}}$ $\quad w = 3 \cdot ft$ **USE: 3'-0" x 3'-0" x 10" w/ (4) #4 each way**

Spread footing F4:

$w := \sqrt{\frac{P4}{p}}$ $\quad w = 4 \cdot ft$ **USE: 4'-0" x 4'-0" x 10" w/ (5) #4 each way**

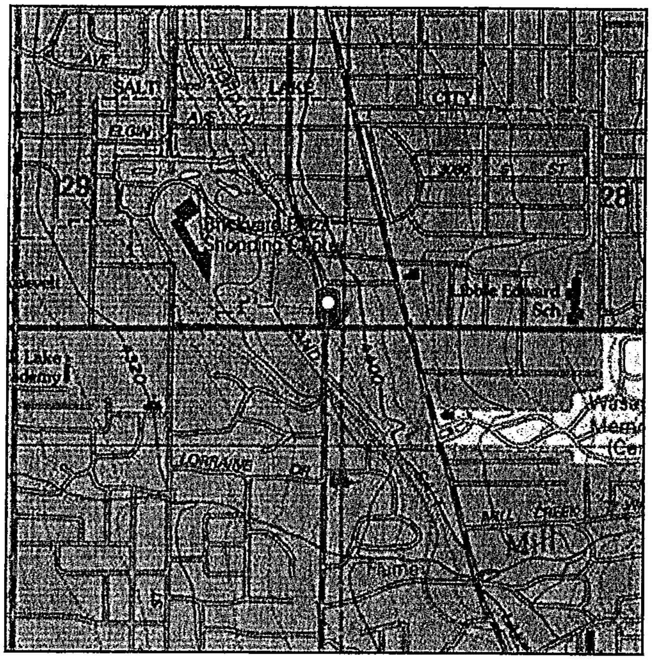
ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

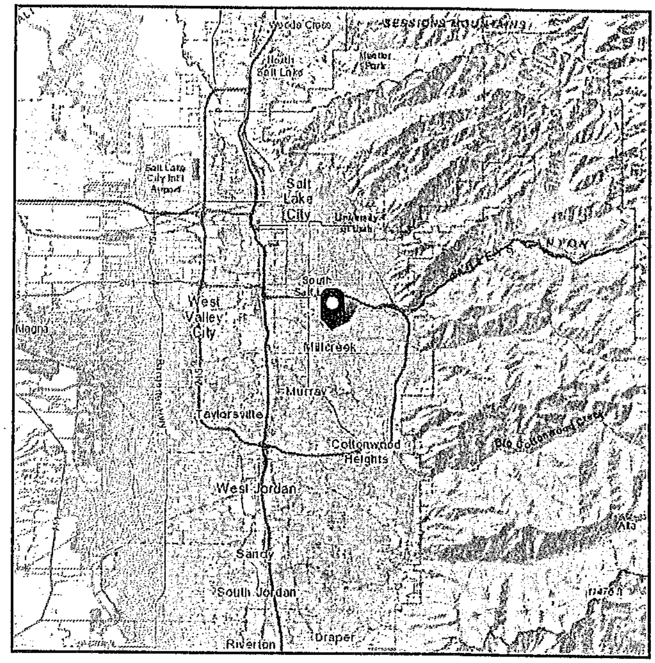
ASCE 7 Hazards Report

Address:
1300 E 3300 S
Salt Lake City, Utah
84106

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Default (see Section 11.4.3)

Elevation: 4395.72 ft (NAVD 88)
Latitude: 40.699834
Longitude: -111.853964





Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.411	S_{D1} :	N/A
S_1 :	0.521	T_L :	8
F_a :	1.2	PGA :	0.64
F_v :	N/A	PGA_M :	0.768
S_{MS} :	1.693	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.129	C_v :	1.382

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Thu Dec 10 2020

Date Source: USGS Seismic Design Maps

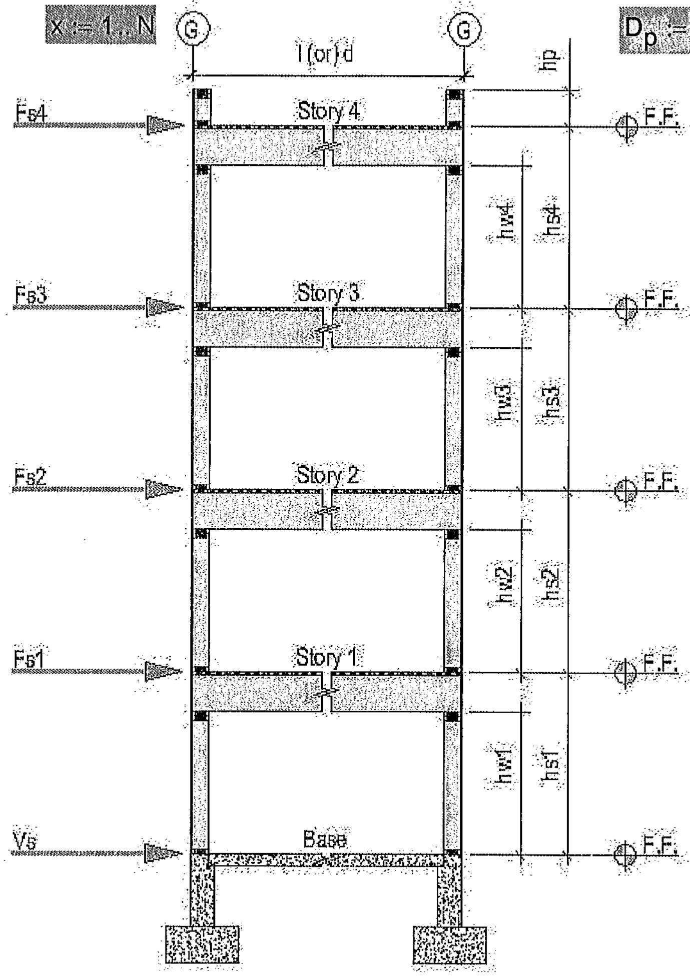
Seismic Base Shear - High Roof Area

This worksheet calculates the Seismic Loads applied to the building's main lateral load resisting system per ASCE 7-10, Chapters 11 and 12.

<u>Number of Stories</u>

$N := 1$

$x := 1..N$



<u>Parapet</u>

$h_p := 0 \cdot ft$

$D_p := 0 \cdot psf$

<u>Story 4</u>

$l_4 := 0 \cdot ft$

$d_4 := 0 \cdot ft$

$h_{s_4} := 0 \cdot ft$

$D_4 := 0 \cdot psf$

$D_{w_4} := 0 \cdot psf$

<u>Story 3</u>

$l_3 := 0 \cdot ft$

$d_3 := 0 \cdot ft$

$h_{s_3} := 0 \cdot ft$

$D_3 := 0 \cdot psf$

$D_{w_3} := 0 \cdot psf$

<u>Story 2</u>

$l_2 := 0 \cdot ft$

$d_2 := 0 \cdot ft$

$h_{s_2} := 0 \cdot ft$

$D_2 := 0 \cdot psf$

$D_{w_2} := 0 \cdot psf$

<u>Story 1</u>

$l_1 := 54 \cdot ft$

$d_1 := 50 \cdot ft$

$h_{s_1} := 16 \cdot ft$ avg

$D_1 := 25 \cdot psf$

$D_{w_1} := 0 \cdot psf$

Wood Shear Walls $R := 6.5$ (Table 12.2-1)

Determine the MCE SRA Parameters per Section 11.4:

Site Class: $SC := "D"$ (Table 20.3-1)

@ Short Periods **@ 1-second Period**

$S_s := 1.41$ $S_1 := 0.52$ (Figures 22-1 thr. 22-14)

$S_{ds} := 1.12$ $S_{d1} := 0.63$

Determine the Seismic Risk Category per Section 11.6:

Risk Category: $RC := 2$ (IBC Table 1604.5)

$I_e := 1.0$ (Table 11.5-1)

$RC = "D"$ (Table 11.6-1)

Calculate the Effective Seismic Weight per Section 12.7.2:

Diaphragms: $wd_x := D_x \cdot l_x \cdot d_x$

$h_{s_{N+1}} := 2 \cdot h_{s_{N+1}}$

Walls: $ww_x := \left(D_{w_x} \cdot \frac{h_{s_x}}{2} + D_{w_{x+1}} \cdot \frac{h_{s_{x+1}}}{2} \right)$

Story Weight: $w_x := wd_x + ww_x \cdot (2 \cdot l_x + 2 \cdot d_x)$

$h_{s_{N+1}} := 0.5 \cdot h_{s_{N+1}}$

Total Weight: $W := \sum w$

$W = 67.5 \cdot kip$

Calculate the Approximate Fundamental Period per Section 12.7.2:

$$h_n := \sum_{i=1}^{N+1} \frac{h_{s_i}}{ft}$$ (Section 12.8.2.1)

$C_t := 0.02$ (Table 12.8-2)

$X := 0.75$ (Table 12.8-2)

$T_a := C_t \cdot h_n^{X}$ (Equation 12.8-7)

Calculate the Fundamental Period per Section 12.7.2:

$C_u := 1.5$ (Table 12.8-1)

$T := C_u \cdot T_a$ (Section 12.8.2)

$T = 0.24$

Determine the Long-period Transition Period per Section 11.4.5:

$T_L := 8.0$ (Figure 22-15)

Calculate the Seismic Response Coefficient per Section 12.8.1.1:

$$C_s := \frac{S_{ds} \cdot I_e}{R}$$ (Equation 12.8-2)

$$C_{smin} := \begin{cases} \dfrac{0.5 \cdot S_1 \cdot I_e}{R} & \text{if } S_1 > 0.60 \\ 0.01 & \text{otherwise} \end{cases}$$ (Equations 12.8-5 & 12.8-6)

$$C_{smax} := \begin{cases} \dfrac{S_{d1} \cdot I_e}{R \cdot T} & \text{if } T \le T_L \\ \dfrac{S_{d1} \cdot I_e \cdot T_L}{R \cdot T^2} & \text{if } T > T_L \end{cases}$$ (Equations 12.8-3 & 12.8-4)

$$C_s := \begin{cases} C_{smin} & \text{if } C_s < C_{smin} \\ C_{smax} & \text{if } C_s > C_{smax} \\ C_s & \text{otherwise} \end{cases}$$

$C_s = 0.172$

Calculate the Seismic Base Shear per Section 12.8.1:

$V_s := C_s \cdot W$ (Equation 12.8-1)

$V_s = 11.6 \cdot kip$ STRENGTH

$\dfrac{V_s}{1.4} = 8.3 \cdot kip$ ALLOWABLE

Vertically Distribute the Seismic Base Shear per Section 12.8.3:

$$k1 := \begin{pmatrix} 1 \\ 2 \end{pmatrix} \qquad t1 := \begin{pmatrix} 0.5 \\ 2.5 \end{pmatrix} \qquad k := \left| \begin{array}{l} 1.0 \text{ if } T < 0.5 \\ \text{linterp}(t1, k1, T) \text{ if } 0.5 \le T \le 2.5 \\ 2.0 \text{ if } T > 2.5 \end{array} \right.$$

$$C_{v_x} := \frac{w_x \cdot \left(\sum_{i=1}^{x} \frac{h_{s_i}}{ft} \right)^k}{\sum_{i=1}^{N} \left[w_i \cdot \left(\sum_{j=1}^{i} \frac{h_{s_j}}{ft} \right)^k \right]}$$ (Equation 12.8-12)

$$F_{s_x} := C_{v_x} \cdot V_s$$ (Equation 12.8-11)

	STRENGTH	ALLOWABLE	
(Story 4)	$F_{s_4} = ■ \cdot kip$	$0.71 F_{s_4} = ■ \cdot kip$	
(Story 3)	$F_{s_3} = ■ \cdot kip$	$0.71 F_{s_3} = ■ \cdot kip$	
(Story 2)	$F_{s_2} = ■ \cdot kip$	$0.71 F_{s_2} = ■ \cdot kip$	
(Story 1)	$F_{s_1} = 11.6 \cdot kip$	$0.71 F_{s_1} = 8.3 \cdot kip$	**USE 9 KIPS**

Calculate the Diaphragm Design Force (perpendicular to L) per Section 12.10.1.1:

$$F_{psl_x} := \frac{\sum_{i=x}^{N} F_{s_i}}{\sum_{i=x}^{N} w_i} \cdot \left(\frac{wd_x}{l_x} + 2 \cdot ww_x \right)$$ (Equation 12.10-1)

$$F_{pslmin_x} := 0.2 \cdot S_{ds} \cdot I_e \cdot \left(\frac{wd_x}{l_x} + 2 \cdot ww_x \right)$$ (Section 12.10.1.1)

$$F_{pslmax_x} := 0.4 \cdot S_{ds} \cdot I_e \cdot \left(\frac{wd_x}{l_x} + 2 \cdot ww_x \right)$$ (Section 12.10.1.1)

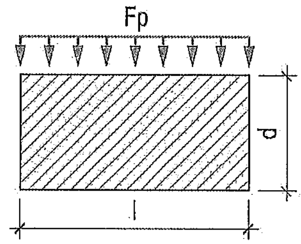
$$F_{psl_x} := \begin{cases} F_{pslmin_x} & \text{if } F_{psl_x} < F_{pslmin_x} \\ F_{pslmax_x} & \text{if } F_{psl_x} > F_{pslmax_x} \\ F_{psl_x} & \text{otherwise} \end{cases}$$

$F_{psl_4} = \blacksquare \cdot plf$ (Story 4)

$F_{psl_3} = \blacksquare \cdot plf$ (Story 3)

$F_{psl_2} = \blacksquare \cdot plf$ (Story 2)

$F_{psl_1} = 280 \cdot plf$ (Story 1)



Calculate the Diaphragm Design Force (perpendicular to D) per Section 12.10.1.1:

$$F_{psd_x} := \frac{\sum_{i=x}^{N} F_{s_i}}{\sum_{i=x}^{N} w_i} \cdot \left(\frac{wd_x}{d_x} + 2 \cdot ww_x \right)$$ (Equation 12.10-1)

$$F_{psdmin_x} := 0.2 \cdot S_{ds} \cdot I_e \cdot \left(\frac{wd_x}{d_x} + 2 \cdot ww_x \right)$$ (Section 12.10.1.1)

$$F_{psdmax_x} := 0.4 \cdot S_{ds} \cdot I_e \cdot \left(\frac{wd_x}{d_x} + 2 \cdot ww_x \right)$$ (Section 12.10.1.1)

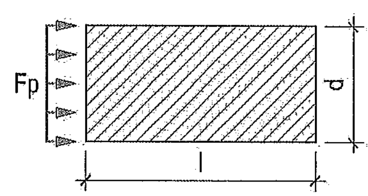
$$F_{psd_x} := \begin{cases} F_{psdmin_x} & \text{if } F_{psd_x} < F_{psdmin_x} \\ F_{psdmax_x} & \text{if } F_{psd_x} > F_{psdmax_x} \\ F_{psd_x} & \text{otherwise} \end{cases}$$

$F_{psd_4} = \blacksquare \cdot plf$ (Story 4)

$F_{psd_3} = \blacksquare \cdot plf$ (Story 3)

$F_{psd_2} = \blacksquare \cdot plf$ (Story 2)

$F_{psd_1} = 302 \cdot plf$ (Story 1)



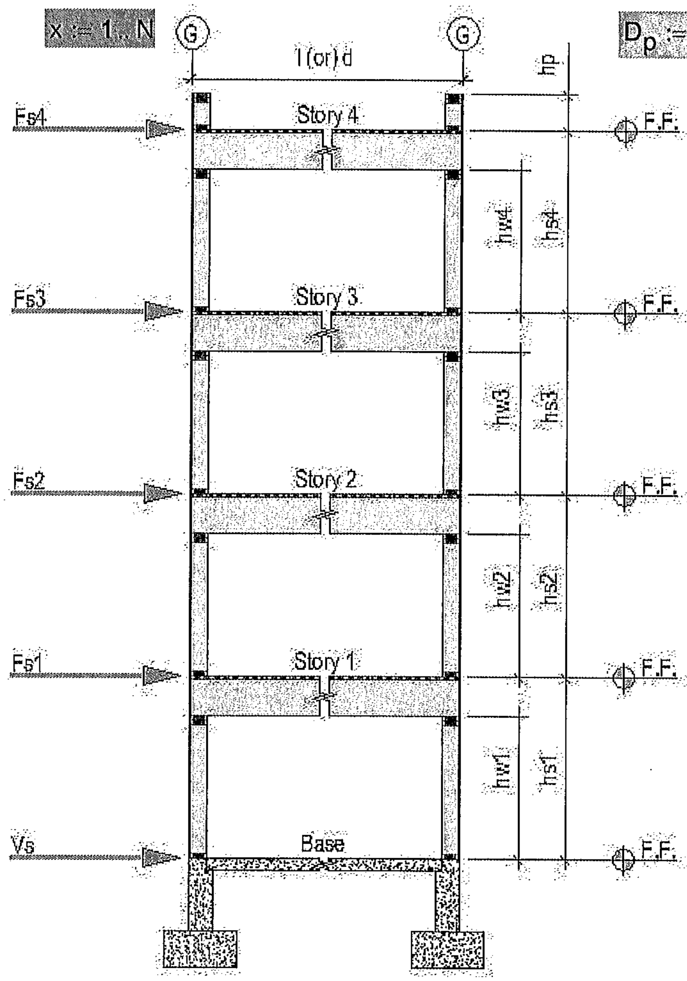
Seismic Base Shear - Enclosed Portion

This worksheet calculates the Seismic Loads applied to the building's main lateral load resisting system per ASCE 7-10, Chapters 11 and 12.

Number of Stories

$N := 1$

$x := 1..N$



Parapet

$h_p := 3 \cdot ft$

$D_p := 20 \cdot psf$

Story 4

$l_4 := 0 \cdot ft$

$d_4 := 0 \cdot ft$

$h_{s_4} := 0 \cdot ft$

$D_4 := 0 \cdot psf$

$D_{W_4} := 0 \cdot psf$

Story 3

$l_3 := 0 \cdot ft$

$d_3 := 0 \cdot ft$

$h_{s_3} := 0 \cdot ft$

$D_3 := 0 \cdot psf$

$D_{W_3} := 0 \cdot psf$

Story 2

$l_2 := 0 \cdot ft$

$d_2 := 0 \cdot ft$

$h_{s_2} := 0 \cdot ft$

$D_2 := 0 \cdot psf$

$D_{W_2} := 0 \cdot psf$

Story 1

$l_1 := 56 \cdot ft$

$d_1 := 24 \cdot ft$

$h_{s_1} := 11 \cdot ft$

dead load reduced to acocunt for double areas used in roof area base shear $D_1 := 11 \cdot psf$

Wood Shear Walls $R := 6.5$ (Table 12.2-1) $D_{W_1} := 40 \cdot psf$ avg

Determine the MCE SRA Parameters per Section 11.4:

Site Class:	SC := "D"	(Table 20.3-1)
@ Short Periods	**@ 1-second Period**	
$S_s := 1.41$	$S_1 := 0.52$	(Figures 22-1 thr. 22-14)
$S_{ds} := 1.12$	$S_{d1} := 0.63$	

Determine the Seismic Risk Category per Section 11.6:

Risk Category:	RC := 2	(IBC Table 1604.5)
$I_e := 1.0$	(Table 11.5-1)	
RC = "D"	(Table 11.6-1)	

Calculate the Effective Seismic Weight per Section 12.7.2:

Diaphragms: $wd_x := D_x \cdot l_x \cdot d_x$

$h_{s_{N+1}} := 2 \cdot h_{s_{N+1}}$

Walls: $ww_x := \left(D_{w_x} \cdot \frac{h_{s_x}}{2} + D_{w_{x+1}} \cdot \frac{h_{s_{x+1}}}{2} \right)$

Story Weight: $w_x := wd_x + ww_x \cdot (2 \cdot l_x + 2 \cdot d_x)$

$h_{s_{N+1}} := 0.5 \cdot h_{s_{N+1}}$

Total Weight: $W := \sum w$

$W = 59.6 \cdot kip$

Calculate the Approximate Fundamental Period per Section 12.7.2:

$h_n := \sum_{i=1}^{N+1} \frac{h_{s_i}}{ft}$ (Section 12.8.2.1)

$C_t := 0.02$ (Table 12.8-2)

$X := 0.75$ (Table 12.8-2)

$T_a := C_t \cdot h_n^{\,X}$ (Equation 12.8-7)

Calculate the Fundamental Period per Section 12.7.2:

$C_u := 1.5$ (Table 12.8-1)

$T := C_u \cdot T_a$ (Section 12.8.2)

$T = 0.217$

Determine the Long-period Transition Period per Section 11.4.5:

$T_L := 8.0$ (Figure 22-15)

Calculate the Seismic Response Coefficient per Section 12.8.1.1:

$$C_s := \frac{S_{ds} \cdot I_e}{R}$$ (Equation 12.8-2)

$$C_{smin} := \begin{cases} \dfrac{0.5 \cdot S_1 \cdot I_e}{R} & \text{if } S_1 > 0.60 \\ 0.01 & \text{otherwise} \end{cases}$$ (Equations 12.8-5 & 12.8-6)

$$C_{smax} := \begin{cases} \dfrac{S_{d1} \cdot I_e}{R \cdot T} & \text{if } T \le T_L \\ \dfrac{S_{d1} \cdot I_e \cdot T_L}{R \cdot T^2} & \text{if } T > T_L \end{cases}$$ (Equations 12.8-3 & 12.8-4)

$$C_s := \begin{cases} C_{smin} & \text{if } C_s < C_{smin} \\ C_{smax} & \text{if } C_s > C_{smax} \\ C_s & \text{otherwise} \end{cases}$$

$C_s = 0.172$

Calculate the Seismic Base Shear per Section 12.8.1:

$V_s := C_s \cdot W$ (Equation 12.8-1)

$V_s = 10.3 \cdot kip$ STRENGTH

$\frac{V_s}{1.4} = 7.3 \cdot kip$ ALLOWABLE

Vertically Distribute the Seismic Base Shear per Section 12.8.3:

$k1 := \begin{pmatrix} 1 \\ 2 \end{pmatrix}$ $\quad t1 := \begin{pmatrix} 0.5 \\ 2.5 \end{pmatrix}$ $\quad k := \begin{cases} 1.0 & \text{if } T < 0.5 \\ \text{linterp}(t1, k1, T) & \text{if } 0.5 \le T \le 2.5 \\ 2.0 & \text{if } T > 2.5 \end{cases}$

$$C_{v_x} := \frac{w_x \cdot \left(\sum_{i=1}^{x} \frac{h_{s_i}}{ft}\right)^k}{\sum_{i=1}^{N}\left[w_i \cdot \left(\sum_{j=1}^{i} \frac{h_{s_j}}{ft}\right)^k\right]}$$ (Equation 12.8-12)

$$F_{s_x} := C_{v_x} \cdot V_s$$ (Equation 12.8-11)

	STRENGTH	ALLOWABLE	
(Story 4)	$F_{s_4} = \blacksquare \cdot kip$	$0.71 F_{s_4} = \blacksquare \cdot kip$	
(Story 3)	$F_{s_3} = \blacksquare \cdot kip$	$0.71 F_{s_3} = \blacksquare \cdot kip$	
(Story 2)	$F_{s_2} = \blacksquare \cdot kip$	$0.71 F_{s_2} = \blacksquare \cdot kip$	
(Story 1)	$F_{s_1} = 10.3 \cdot kip$	$0.71 F_{s_1} = 7.3 \cdot kip$	**USE 8 KIPS**

Calculate the Diaphragm Design Force (perpendicular to L) per Section 12.10.1.1:

$$F_{psl_x} := \frac{\sum_{i=x}^{N} F_{s_i}}{\sum_{i=x}^{N} w_i} \cdot \left(\frac{wd_x}{l_x} + 2 \cdot ww_x\right)$$ (Equation 12.10-1)

$$F_{pslmin_x} := 0.2 \cdot S_{ds} \cdot I_e \cdot \left(\frac{wd_x}{l_x} + 2 \cdot ww_x\right)$$ (Section 12.10.1.1)

$$F_{pslmax_x} := 0.4 \cdot S_{ds} \cdot I_e \cdot \left(\frac{wd_x}{l_x} + 2 \cdot ww_x\right)$$ (Section 12.10.1.1)

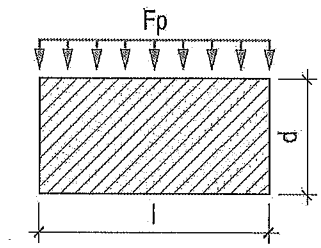
$$F_{psl_x} := \begin{cases} F_{pslmin_x} & \text{if } F_{psl_x} < F_{pslmin_x} \\ F_{pslmax_x} & \text{if } F_{psl_x} > F_{pslmax_x} \\ F_{psl_x} & \text{otherwise} \end{cases}$$

$F_{psl_4} = \blacksquare \cdot plf$ (Story 4)

$F_{psl_3} = \blacksquare \cdot plf$ (Story 3)

$F_{psl_2} = \blacksquare \cdot plf$ (Story 2)

$F_{psl_1} = \mathbf{185} \cdot plf$ (Story 1)



Calculate the Diaphragm Design Force (perpendicular to D) per Section 12.10.1.1:

$$F_{psd_x} := \frac{\sum_{i=x}^{N} F_{s_i}}{\sum_{i=x}^{N} w_i} \cdot \left(\frac{wd_x}{d_x} + 2 \cdot ww_x \right)$$ (Equation 12.10-1)

$$F_{psdmin_x} := 0.2 \cdot S_{ds} \cdot I_e \cdot \left(\frac{wd_x}{d_x} + 2 \cdot ww_x \right)$$ (Section 12.10.1.1)

$$F_{psdmax_x} := 0.4 \cdot S_{ds} \cdot I_e \cdot \left(\frac{wd_x}{d_x} + 2 \cdot ww_x \right)$$ (Section 12.10.1.1)

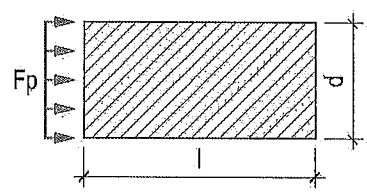
$$F_{psd_x} := \begin{cases} F_{psdmin_x} & \text{if } F_{psd_x} < F_{psdmin_x} \\ F_{psdmax_x} & \text{if } F_{psd_x} > F_{psdmax_x} \\ F_{psd_x} & \text{otherwise} \end{cases}$$

$F_{psd_4} = \blacksquare \cdot plf$ (Story 4)

$F_{psd_3} = \blacksquare \cdot plf$ (Story 3)

$F_{psd_2} = \blacksquare \cdot plf$ (Story 2)

$F_{psd_1} = \mathbf{263} \cdot plf$ (Story 1)



SHEAR WALLS - GRID B

STORY 1

# Piers in Shear Line:	$n1 := 3$	(n = 8 max)
Story Shear:	$Fa_1 := 17 \cdot k$	(Allowable)
Shear Attributed To Line:	$Va_1 := 10k$	(Allowable)
Story DL:	$DL_1 := 25 \cdot psf$	
Wall DL:	$DLw_1 := 20 \cdot psf$	
Sill Plate Length:	$Ls_1 := 17 \cdot ft$	
Redundancy	$\rho_1 := 1$	

PIERS	Length	Height	Tributary
1:	$l1_1 := 9.5 \cdot ft$	$h1_1 := 10 \cdot ft$	$t1_1 := 13 \cdot ft$
2:	$l1_2 := 3.5 \cdot ft$	$h1_2 := 7 \cdot ft$	$t1_2 := 13 \cdot ft$
3:	$l1_3 := 4 \cdot ft$	$h1_3 := 7 \cdot ft$	$t1_3 := 13 \cdot ft$
4:	$l1_4 := 0 \cdot ft$	$h1_4 := 0 \cdot ft$	$t1_4 := 0 \cdot ft$
5:	$l1_5 := 0 \cdot ft$	$h1_5 := 0 \cdot ft$	$t1_5 := 0 \cdot ft$
6:	$l1_6 := 0 \cdot ft$	$h1_6 := 0 \cdot ft$	$t1_6 := 0 \cdot ft$
7:	$l1_7 := 0 \cdot ft$	$h1_7 := 0 \cdot ft$	$t1_7 := 0 \cdot ft$
8:	$l1_8 := 0 \cdot ft$	$h1_8 := 0 \cdot ft$	$t1_8 := 0 \cdot ft$

TOP OF PIERS STRAPPED
TO LIMIT HEIGHT

SHEAR CALCULATIONS

Unit Shear (for walls): $v_1 := \dfrac{\rho_1 \cdot Va_1}{\sum l1}$

OVERTURNING CALCULATIONS $i1 := 1 .. n1$

Overturning Moment: $Mo1_{i1} := v_1 \cdot h1_{i1} \cdot l1_{i1}$

Resisting Moment: $Mr1_{i1} := 0.6 \cdot \left[\left[\left(DL_1 \cdot t1_{i1}\right) \cdot l1_{i1} \cdot \left(\dfrac{l1_{i1}}{2}\right)\right] + \left[\left(DLw_1 \cdot h1_{i1}\right) \cdot l1_{i1} \cdot \left(\dfrac{l1_{i1}}{2}\right)\right]\right]$

Nominal Overturning: $M1_{i1} := Mo1_{i1} - Mr1_{i1}$

Tension at Pier Ends: $T1_{i1} := \dfrac{M1_{i1}}{l1_{i1}}$

ANCHOR BOLTS

Unit Shear (for bolts): $vb_1 := \dfrac{\rho_1 \cdot Va_1}{Ls_1}$

1/2" bolt in 1 1/2" sill: $s_{0.5} := \dfrac{(650 \cdot lb) \cdot 1.6}{vb_1}$

5/8" bolt in 1 1/2" sill: $s_{0.625} := \dfrac{(930 \cdot lb) \cdot 1.6}{vb_1}$

SHEAR WALLS - GRID D

STORY 1

			PIERS	Length	Height	Tributary
# Piers in Shear Line:	$n1 := 1$	(n = 8 max)	1:	$l1_1 := 22.5 \cdot ft$	$h1_1 := 10 \cdot ft$	$t1_1 := 8 \cdot ft$
Story Shear:	$Fa_1 := 17 \cdot k$	(Allowable)	2:	$l1_2 := 0 \cdot ft$	$h1_2 := 0 \cdot ft$	$t1_2 := 0 \cdot ft$
Shear Attributed To Line:	$Va_1 := 7k$	(Allowable)	3:	$l1_3 := 0 \cdot ft$	$h1_3 := 0 \cdot ft$	$t1_3 := 0 \cdot ft$
Story DL:	$DL_1 := 25 \cdot psf$		4:	$l1_4 := 0 \cdot ft$	$h1_4 := 0 \cdot ft$	$t1_4 := 0 \cdot ft$
Wall DL:	$DLw_1 := 20 \cdot psf$		5:	$l1_5 := 0 \cdot ft$	$h1_5 := 0 \cdot ft$	$t1_5 := 0 \cdot ft$
Sill Plate Length:	$Ls_1 := 22.5 \cdot ft$		6:	$l1_6 := 0 \cdot ft$	$h1_6 := 0 \cdot ft$	$t1_6 := 0 \cdot ft$
Redundancy	$\rho_1 := 1$		7:	$l1_7 := 0 \cdot ft$	$h1_7 := 0 \cdot ft$	$t1_7 := 0 \cdot ft$
			8:	$l1_8 := 0 \cdot ft$	$h1_8 := 0 \cdot ft$	$t1_8 := 0 \cdot ft$

SHEAR CALCULATIONS

Unit Shear (for walls): $v_1 := \dfrac{\rho_1 \cdot Va_1}{\sum l1}$

OVERTURNING CALCULATIONS $i1 := 1 .. n1$

Overturning Moment: $Mo1_{i1} := v_1 \cdot h1_{i1} \cdot l1_{i1}$

Resisting Moment: $Mr1_{i1} := 0.6 \cdot \left[\left[\left(DL_1 \cdot t1_{i1} \right) \cdot l1_{i1} \cdot \left(\frac{l1_{i1}}{2} \right) \right] + \left[\left(DLw_1 \cdot h1_{i1} \right) \cdot l1_{i1} \cdot \left(\frac{l1_{i1}}{2} \right) \right] \right]$

Nominal Overturning: $M1_{i1} := Mo1_{i1} - Mr1_{i1}$

Tension at Pier Ends: $T1_{i1} := \dfrac{M1_{i1}}{l1_{i1}}$

ANCHOR BOLTS

Unit Shear (for bolts): $vb_1 := \dfrac{\rho_1 \cdot Va_1}{Ls_1}$

1/2" bolt in 1 1/2" sill: $s_{0.5} := \dfrac{(650 \cdot lb) \cdot 1.6}{vb_1}$

5/8" bolt in 1 1/2" sill: $s_{0.625} := \dfrac{(930 \cdot lb) \cdot 1.6}{vb_1}$

SUMMARY, STORY 1

Reduction in shear walls due to height to width ratio less than 2:1

$$ratio_{i1} := \frac{l1_{i1}}{h1_{i1}}$$

$$r := if(2 \cdot min(ratio) > 1.0, 1.0, 2 \cdot min(ratio)) \qquad r = 1$$

Reduced Unit Shear:

$$\frac{v1}{r} = 311 \cdot plf$$

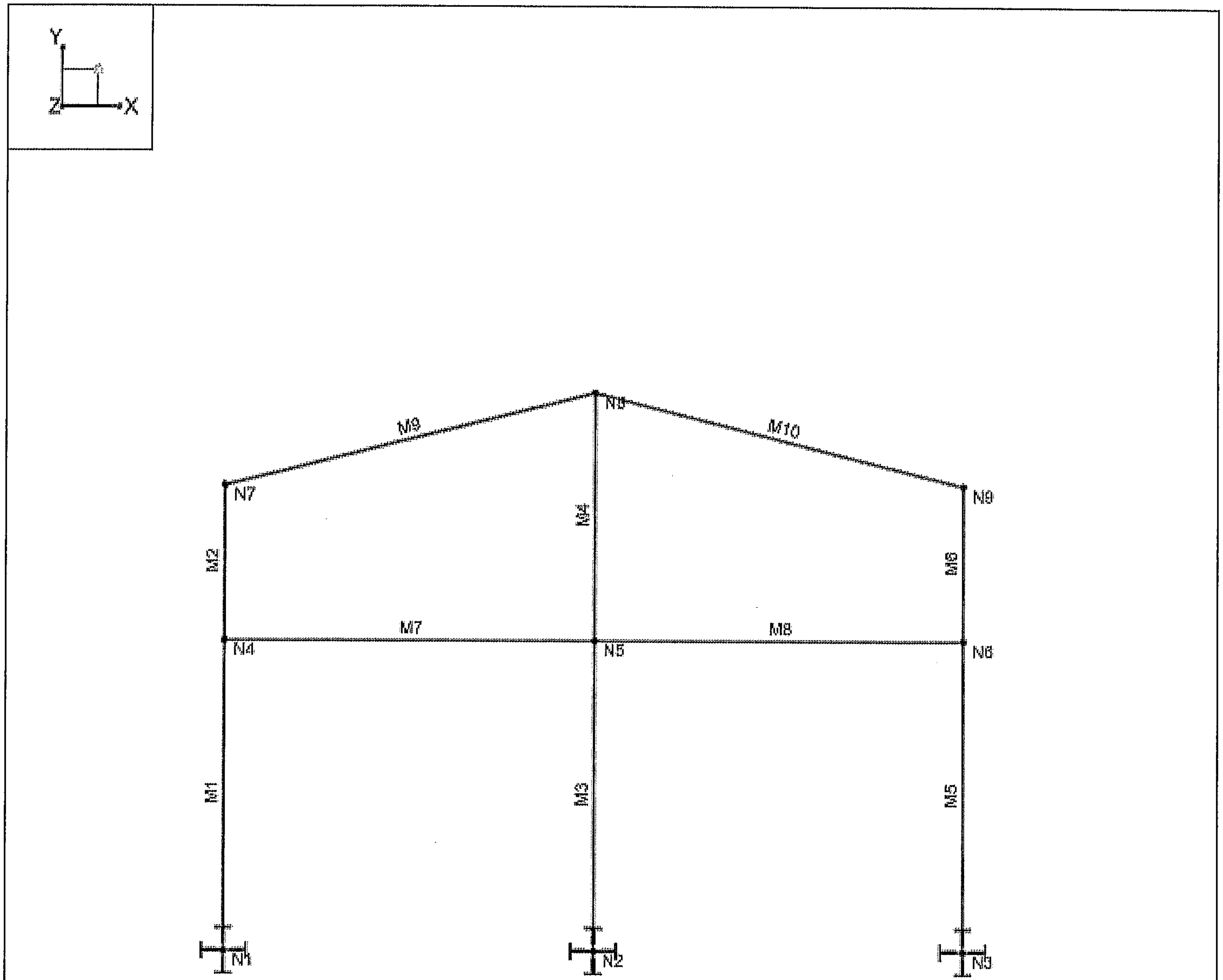
SHEAR WALLS

Sheathing:	7/16", APA, Exp. 1
Blocking:	All Panel Edges
Edge Nailing:	8d @ 4" o.c.
Field Nailing:	8d @ 12" o.c.

	Uplift	HOLD DOWN
Pier 1:	$T1_1 = 411 \cdot lb$	STHD14
Pier 2:	$T1_2 = \blacksquare \cdot lb$	
Pier 3:	$T1_3 = \blacksquare \cdot lb$	
Pier 4:	$T1_4 = \blacksquare \cdot lb$	
Pier 5:	$T1_5 = \blacksquare \cdot lb$	
Pier 6:	$T1_6 = \blacksquare \cdot lb$	
Pier 7:	$T1_7 = \blacksquare \cdot lb$	
Pier 8:	$T1_8 = \blacksquare \cdot lb$	

ANCHOR BOLTS	1/2" A.Bolts	5/8" A.Bolts
	$s_{0.5} = 40 \cdot in$	$s_{0.625} = 57 \cdot in$

USE:
5/8" dia. x 10" J-bolts
Spacing = 32" o.c.



	Moment Frame Grid 2	SK-1
		Jan 04, 2021
		MF2.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	12	0	0	
3	N3	24	0	0	
4	N4	0	10	0	
5	N5	12	10	0	
6	N6	24	10	0	
7	N7	0	15	0	
8	N8	12	18	0	
9	N9	24	15	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Y Rot [k-ft/rad]
1	N1	Reaction	Reaction	Fixed	Fixed	Fixed
2	N2	Reaction	Reaction	Fixed	Fixed	Fixed
3	N3	Reaction	Reaction	Fixed	Fixed	Fixed

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e⁵°F⁻¹]	Density [k/ft³]	Yield [ksi]	Ry	Fu [ksi]	Rt
1	A36 Gr.36	29000	11154	0.3	0.65	0.49	36	1.5	58	1.2
2	A572 Gr.50	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
3	A992	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
4	A500 Gr.42	29000	11154	0.3	0.65	0.49	42	1.4	58	1.3
5	A500 Gr.46	29000	11154	0.3	0.65	0.49	46	1.4	58	1.3

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Column	W16X26	Column	Wide Flange	A992	Typical	7.68	9.59	301	0.262
2	Beam	W16X26	Beam	Wide Flange	A992	Typical	7.68	9.59	301	0.262

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N4	Column	Column	Wide Flange	A992	Typical
2	M2	N4	N7	Column	Column	Wide Flange	A992	Typical
3	M3	N2	N5	Column	Column	Wide Flange	A992	Typical
4	M4	N5	N8	Column	Column	Wide Flange	A992	Typical
5	M5	N3	N6	Column	Column	Wide Flange	A992	Typical
6	M6	N6	N9	Column	Column	Wide Flange	A992	Typical
7	M7	N4	N5	Beam	Beam	Wide Flange	A992	Typical
8	M8	N5	N6	Beam	Beam	Wide Flange	A992	Typical
9	M9	N7	N8	Beam	Beam	Wide Flange	A992	Typical
10	M10	N8	N9	Beam	Beam	Wide Flange	A992	Typical

Basic Load Cases

	BLC Description	Category	Y Gravity	Nodal
1	Dead Load	None	-1	3
2	Live Load	None		
3	Snow Load	None		3
4	Seismic (Eh)	None		1

Load Combinations

	Description	Solve	PDelta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	1.4DL (IBC 16-1)	Yes	C	1	1.4								
2	1.2DL + 1.6LL + 0.5SL (IBC 16-2)	Yes	C	1	1.2	2	1.6	3	0.5				
3	1.2DL + 1.6SL + 0.5LL (IBC 16-3)	Yes	C	1	1.2	3	1.6	2	0.5				
4	1.2DL + 1.0Eh + 0.2SdsDL + 0.5LL + 0.2SL (IBC 16-5)	Yes	C	1	1.2	4	1	1	0.25	2	0.5	3	0.2

Load Combinations (Continued)

	Description	Solve	PDelta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
5	1.2DL -1.0Eh - 0.2SdsDL + 0.5LL + 0.2SL (IBC 16-5)	Yes	C	1	1.2	4	-1	1	-0.25	2	0.5	3	0.2
6	0.9DL + 1.0Eh + 0.2SdsDL (IBC 16-7)	Yes	C	1	0.9	4	1	1	0.25				
7	0.9DL - 1.0Eh - 0.2SdsDL (IBC 16-7)	Yes	C	1	0.9	4	-1	1	-0.25				
8	DL + LL (Service) IBC 16-9		C	1	1	2	1						
9	DL + SL (Service) IBC 16-10		C	1	1	3	1						
10	DL + .75LL + .75SL (Service) IBC 16-11		C	1	1	2	0.75	3	0.75				
11	DL + 0.7Eh + 0.7(0.2Sds)DL (Service) IBC 16-12		C	1	1	4	0.7	1	0.18				
12	DL - 0.7Eh - 0.7(0.2Sds)DL (Service) IBC 16-12		C	1	1	4	-0.7	1	-0.18				
13	DL + 0.75(0.7Eh) + 0.75(0.2Sds)DL + 0.75LL + 0.75SL		C	1	1	4	0.53	1	0.19	2	0.75	3	0.75
14	DL - 0.75(0.7Eh) - 0.75(0.2Sds)DL + 0.75LL + 0.75SL		C	1	1	4	-0.53	1	-0.19	2	0.75	3	0.75
15	0.6DL + 0.7Eh +0.7(0.2SdsDL) (Service) IBC 16-15		C	1	0.6	4	0.7	1	0.18				
16	0.6DL - 0.7Eh - 0.7(0.2SdsDL) (Service) IBC 16-15		C	1	0.6	4	-0.7	1	-0.18				

≤ 1.0

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

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
1	M1	W16X26	0.351	10	5	0.0447	10	y	4	147.8391	345.6	20.55	165.75	1.6667	H1-1b
2	M2	W16X26	0.1837	5	5	0.0591	5	y	5	249.3879	345.6	20.55	165.75	1.8866	H1-1b
3	M3	W16X26	0.4373	10	5	0.0624	10	y	4	147.8391	345.6	20.55	165.75	1.6667	H1-1b
4	M4	W16X26	0.2008	8	5	0.0503	8	y	4	193.127	345.6	20.55	165.75	2.1746	H1-1b
5	M5	W16X26	0.3727	10	4	0.0462	10	y	4	147.8391	345.6	20.55	165.75	1.6667	H1-1b
6	M6	W16X26	0.1699	5	4	0.0449	5	y	4	249.3879	345.6	20.55	165.75	1.6997	H1-1b
7	M7	W16X26	0.3165	0	4	0.0753	12	y	4	104.4801	345.6	20.55	165.75	2.2125	H1-1b
8	M8	W16X26	0.2996	12	4	0.0717	12	y	4	104.4801	345.6	20.55	165.75	2.2508	H1-1b
9	M9	W16X26	0.2046	0	4	0.034	12.3693	y	4	98.3342	345.6	20.55	165.75	2.1546	H1-1b
10	M10	W16X26	0.1683	12.3693	4	0.0294	12.3693	y	4	98.3342	345.6	20.55	165.75	2.2292	H1-1b

Envelope Story Drift - X-Direction, Strength

ALLOW DRIFT = 0.02(15')(12)/3.5 = 1.03"

	Story (Elevation)		Story Drift[in]	Loc (Z,X)	LC	Drift Ratio (%)	Loc (Z,X)	LC	2nd/1st Ratio	Loc (Z,X)	LC
1	N7 (15 ft)	max	0.897	0, 0	4	0.4988	0, 0	5	1.025	0, 0	3
2		min	-0.8978	0, 0	5	0.001	0, 0	1	1.0073	0, 0	7

< 1.03"

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	4.6245	7	31.1453	3	NC	NC	NC	NC	NC	NC	0	7
2		min	-4.7043	6	1.348	6	NC	NC	NC	NC	NC	NC	0	1
3	N2	max	6.5221	7	22.7452	3	NC	NC	NC	NC	NC	NC	0	7
4		min	-6.5489	4	6.4986	6	NC	NC	NC	NC	NC	NC	0	1
5	N3	max	4.8702	5	23.9433	3	NC	NC	NC	NC	NC	NC	0	7
6		min	-4.7538	4	-7.0184	7	NC	NC	NC	NC	NC	NC	0	1
7	Totals:	max	16	7	77.8337	3	0	7						
8		min	-16	4	15.2933	7	0	1						

PINNED BASE

Load Combinations

	Description	Solve	PDelta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	1.4DL (IBC 16-1)		C	1	1.4								
2	1.2DL + 1.6LL + 0.5SL (IBC 16-2)		C	1	1.2	2	1.6	3	0.5				
3	1.2DL + 1.6SL + 0.5LL (IBC 16-3)		C	1	1.2	3	1.6	2	0.5				
4	1.2DL + 1.0Eh + 0.2SdsDL + 0.5LL + 0.2SL (IBC 16-5)		C	1	1.2	4	1	1	0.25	2	0.5	3	0.2
5	1.2DL -1.0Eh - 0.2SdsDL + 0.5LL + 0.2SL (IBC 16-5)		C	1	1.2	4	-1	1	-0.25	2	0.5	3	0.2
6	0.9DL + 1.0Eh + 0.2SdsDL (IBC 16-7)		C	1	0.9	4	1	1	0.25				
7	0.9DL - 1.0Eh - 0.2SdsDL (IBC 16-7)		C	1	0.9	4	-1	1	-0.25				
8	DL + LL (Service) IBC 16-9	Yes	C	1	1	2	1						
9	DL + SL (Service) IBC 16-10	Yes	C	1	1	3	1						
10	DL + .75LL + .75SL (Service) IBC 16-11	Yes	C	1	1	2	0.75	3	0.75				
11	DL + 0.7Eh + 0.7(0.2Sds)DL (Service) IBC 16-12	Yes	C	1	1	4	0.7	1	0.18				
12	DL - 0.7Eh - 0.7(0.2Sds)DL (Service) IBC 16-12	Yes	C	1	1	4	-0.7	1	-0.18				
13	DL + 0.75(0.7Eh) + 0.75(0.2Sds)DL + 0.75LL + 0.75SL	Yes	C	1	1	4	0.53	1	0.19	2	0.75	3	0.75
14	DL - 0.75(0.7Eh) - 0.75(0.2Sds)DL + 0.75LL + 0.75SL	Yes	C	1	1	4	-0.53	1	-0.19	2	0.75	3	0.75
15	0.6DL + 0.7Eh +0.7(0.2SdsDL) (Service) IBC 16-15	Yes	C	1	0.6	4	0.7	1	0.18				
16	0.6DL - 0.7Eh - 0.7(0.2SdsDL) (Service) IBC 16-15	Yes	C	1	0.6	4	-0.7	1	-0.18				

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	3.2472	16	21.6573	9	NC	NC	NC	NC	NC	NC	0	16
2		min	-3.2844	15	0.7433	15	NC	NC	NC	NC	NC	NC	0	8
3	N2	max	4.5647	16	16.2145	9	NC	NC	NC	NC	NC	NC	0	16
4		min	-4.576	11	4.3483	15	NC	NC	NC	NC	NC	NC	0	8
5	N3	max	3.3949	12	21.5682	13	NC	NC	NC	NC	NC	NC	0	16
6		min	-3.3438	11	-5.1342	16	NC	NC	NC	NC	NC	NC	0	8
7	Totals:	max	11.2	16	54.5281	9	0	16						
8		min	-11.2	11	9.8818	16	0	8						

SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.6.6682.28

Company:	Dynamic Structures	Date:	1/4/2021
Engineer:		Page:	1/5
Project:	Holdown BB Anchors		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Retrofit Anchors

Location:
Fastening description:

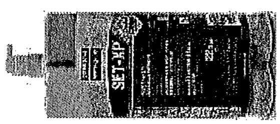
2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 8.750
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.88
c_{ac} (inch): 22.36
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Periodic
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Recommended Anchor
Anchor Name: SET-XP® - SET-XP w/ 5/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

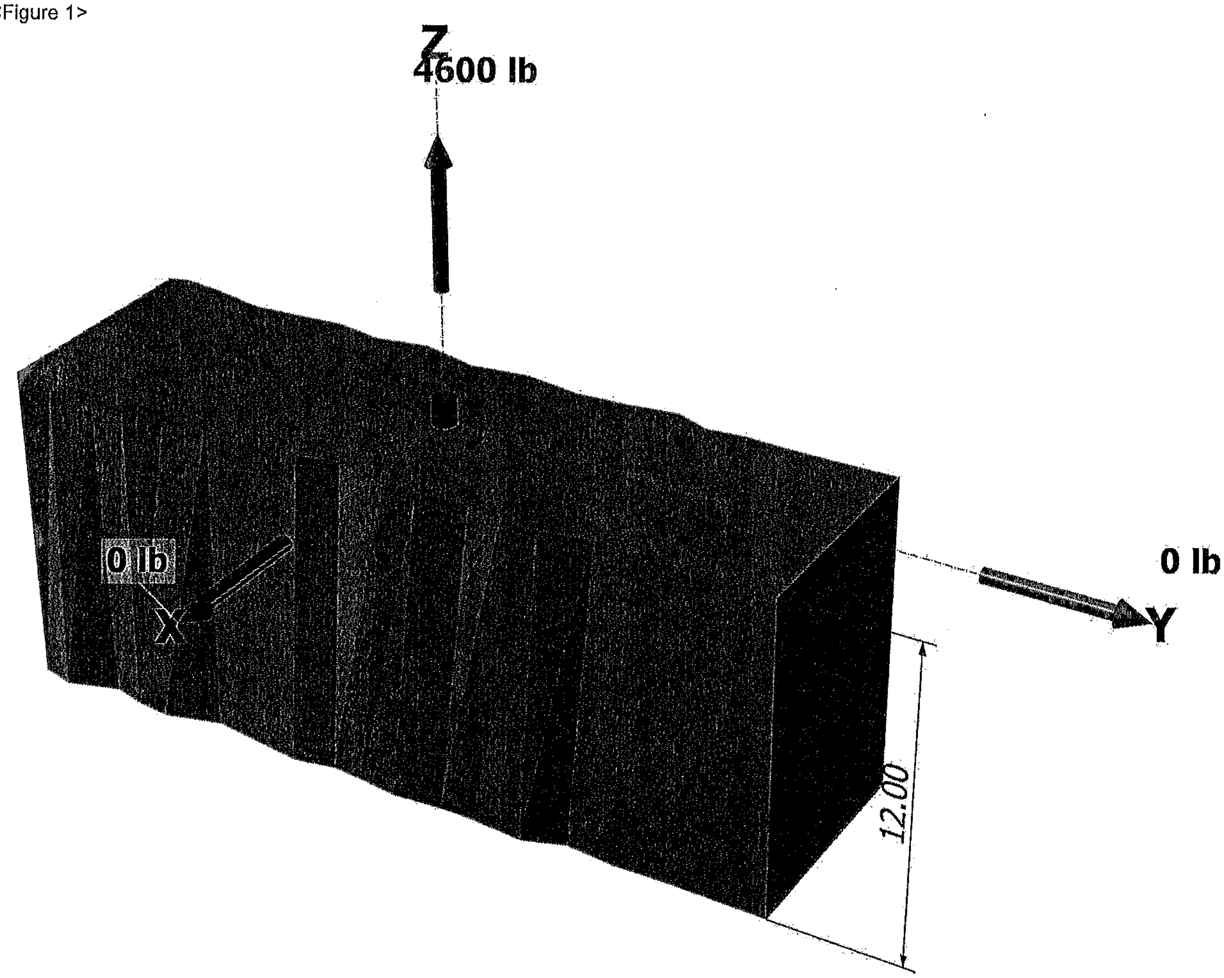


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: not satisfied
Ductility section for shear: not satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

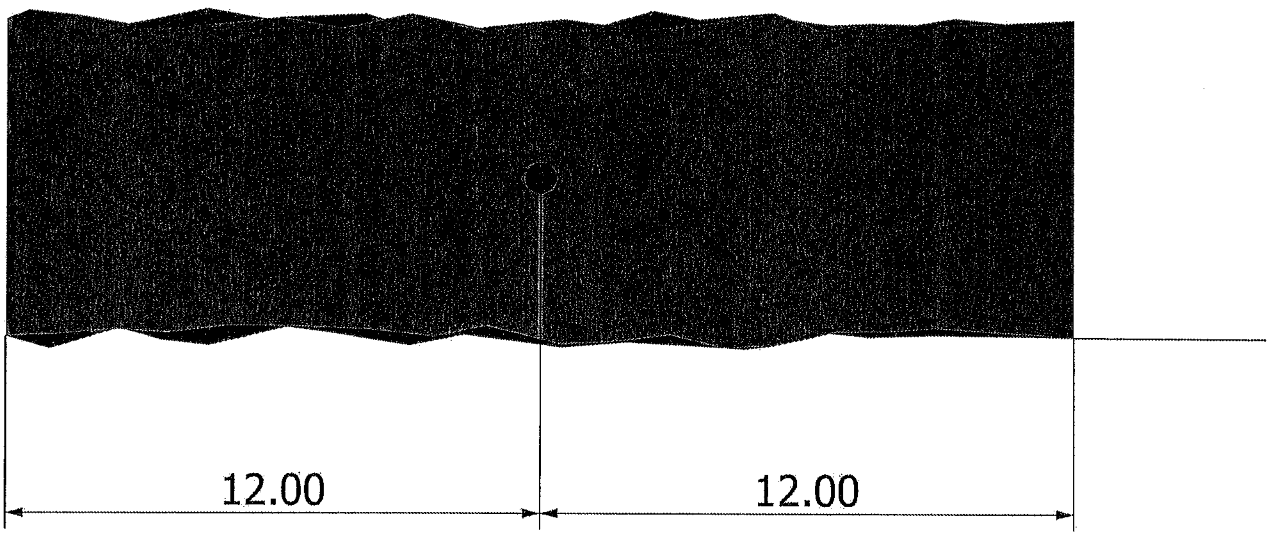
Strength level loads:

N_{ua} [lb]: 4600
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4600.0	0.0	0.0	0.0
Sum	4600.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 4600
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	8.750	22000

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
630.00	689.06	12.00	0.974	1.00	1.000	22000	0.65	9554

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} \alpha_{N,seis}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N,seis}$	$\tau_{k,cr}$ (psi)
435	1.72	1.00	1.00	748

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	748	0.63	8.750	12855

$0.75\phi N_a = 0.75\phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\,\Psi_{cp,Na} N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
258.98	258.98	8.05	12.00	1.000	1.000	12855	0.55	5302

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	4600	9833	0.47	Pass
Concrete breakout	4600	9554	0.48	Pass
Adhesive	**4600**	**5302**	**0.87**	**Pass (Governs)**

SET-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 8.750 inch meets the selected design criteria.

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Brittle failure governs for tension. Governing anchor failure mode is brittle failure. Attachment shall be designed to satisfy the requirements of ACI 318-14 Section 17.2.3.4.3 for structures assigned to Seismic Design Category C, D, E, or F when the component of the strength level earthquake force applied to anchors exceeds 20 percent of the total factored anchor force associated with the same load combination. In case when ACI 318-14 Sections 17.2.3.4.3 (a)(iii) to (vi), (b), (c) or (d) is satisfied for tension loading, select appropriate checkbox from Inputs tab to disable this message. Alternatively, Ω0 factor can be entered to satisfy ACI 318-14 Section 17.2.3.4.3(d) to increase the earthquake portion of the loads as required.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

SHEAR WALLS - GRID 4.4

STORY 1

# Piers in Shear Line:	$n1 := 2$	(n = 8 max)
Story Shear:	$Fa_1 := 17 \cdot k$	(Allowable)
Shear Attributed To Line:	$Va_1 := 7k$	(Allowable)
Story DL:	$DL_1 := 25 \cdot psf$	
Wall DL:	$DLw_1 := 15 \cdot psf$	
Sill Plate Length:	$Ls_1 := 19 \cdot ft$	
Redundancy	$\rho_1 := 1$	

PIERS	Length	Height	Tributary
1:	$l1_1 := 11 \cdot ft$	$h1_1 := 10 \cdot ft$	$t1_1 := 1 \cdot ft$
2:	$l1_2 := 8 \cdot ft$	$h1_2 := 10 \cdot ft$	$t1_2 := 1 \cdot ft$
3:	$l1_3 := 0 \cdot ft$	$h1_3 := 0 \cdot ft$	$t1_3 := 0 \cdot ft$
4:	$l1_4 := 0 \cdot ft$	$h1_4 := 0 \cdot ft$	$t1_4 := 0 \cdot ft$
5:	$l1_5 := 0 \cdot ft$	$h1_5 := 0 \cdot ft$	$t1_5 := 0 \cdot ft$
6:	$l1_6 := 0 \cdot ft$	$h1_6 := 0 \cdot ft$	$t1_6 := 0 \cdot ft$
7:	$l1_7 := 0 \cdot ft$	$h1_7 := 0 \cdot ft$	$t1_7 := 0 \cdot ft$
8:	$l1_8 := 0 \cdot ft$	$h1_8 := 0 \cdot ft$	$t1_8 := 0 \cdot ft$

SHEAR CALCULATIONS

Unit Shear (for walls): $v_1 := \dfrac{\rho_1 \cdot Va_1}{\sum l1}$

OVERTURNING CALCULATIONS $i1 := 1 .. n1$

Overturning Moment: $Mo1_{i1} := v_1 \cdot h1_{i1} \cdot l1_{i1}$

Resisting Moment: $Mr1_{i1} := 0.6 \cdot \left[\left[\left(DL_1 \cdot t1_{i1}\right) \cdot l1_{i1} \cdot \left(\dfrac{l1_{i1}}{2}\right)\right] + \left[\left(DLw_1 \cdot h1_{i1}\right) \cdot l1_{i1} \cdot \left(\dfrac{l1_{i1}}{2}\right)\right]\right]$

Nominal Overturning: $M1_{i1} := Mo1_{i1} - Mr1_{i1}$

Tension at Pier Ends: $T1_{i1} := \dfrac{M1_{i1}}{l1_{i1}}$

ANCHOR BOLTS

Unit Shear (for bolts): $vb_1 := \dfrac{\rho_1 \cdot Va_1}{Ls_1}$

1/2" bolt in 1 1/2" sill: $s_{0.5} := \dfrac{(650 \cdot lb) \cdot 1.6}{vb_1}$

5/8" bolt in 1 1/2" sill: $s_{0.625} := \dfrac{(930 \cdot lb) \cdot 1.6}{vb_1}$

SHEAR WALLS - GRID 6

STORY 1

# Piers in Shear Line:	$n1 := 1$	(n = 8 max)
Story Shear:	$Fa_1 := 17 \cdot k$	(Allowable)
Shear Attributed To Line:	$Va_1 := 1.4k$	(Allowable)
Story DL:	$DL_1 := 25 \cdot psf$	
Wall DL:	$DLw_1 := 20 \cdot psf$	
Sill Plate Length:	$Ls_1 := 14 \cdot ft$	
Redundancy	$\rho_1 := 1$	

PIERS	Length	Height	Tributary
1:	$l1_1 := 14 \cdot ft$	$h1_1 := 10 \cdot ft$	$t1_1 := 1 \cdot ft$
2:	$l1_2 := 0 \cdot ft$	$h1_2 := 0 \cdot ft$	$t1_2 := 0 \cdot ft$
3:	$l1_3 := 0 \cdot ft$	$h1_3 := 0 \cdot ft$	$t1_3 := 0 \cdot ft$
4:	$l1_4 := 0 \cdot ft$	$h1_4 := 0 \cdot ft$	$t1_4 := 0 \cdot ft$
5:	$l1_5 := 0 \cdot ft$	$h1_5 := 0 \cdot ft$	$t1_5 := 0 \cdot ft$
6:	$l1_6 := 0 \cdot ft$	$h1_6 := 0 \cdot ft$	$t1_6 := 0 \cdot ft$
7:	$l1_7 := 0 \cdot ft$	$h1_7 := 0 \cdot ft$	$t1_7 := 0 \cdot ft$
8:	$l1_8 := 0 \cdot ft$	$h1_8 := 0 \cdot ft$	$t1_8 := 0 \cdot ft$

SHEAR CALCULATIONS

Unit Shear (for walls): $v_1 := \dfrac{\rho_1 \cdot Va_1}{\sum l1}$

OVERTURNING CALCULATIONS $i1 := 1 .. n1$

Overturning Moment: $Mo1_{i1} := v_1 \cdot h1_{i1} \cdot l1_{i1}$

Resisting Moment: $Mr1_{i1} := 0.6 \cdot \left[\left[\left(DL_1 \cdot t1_{i1} \right) \cdot l1_{i1} \cdot \left(\dfrac{l1_{i1}}{2} \right) \right] + \left[\left(DLw_1 \cdot h1_{i1} \right) \cdot l1_{i1} \cdot \left(\dfrac{l1_{i1}}{2} \right) \right] \right]$

Nominal Overturning: $M1_{i1} := Mo1_{i1} - Mr1_{i1}$

Tension at Pier Ends: $T1_{i1} := \dfrac{M1_{i1}}{l1_{i1}}$

ANCHOR BOLTS

Unit Shear (for bolts): $vb_1 := \dfrac{\rho_1 \cdot Va_1}{Ls_1}$

1/2" bolt in 1 1/2" sill: $s_{0.5} := \dfrac{(650 \cdot lb) \cdot 1.6}{vb_1}$

5/8" bolt in 1 1/2" sill: $s_{0.625} := \dfrac{(930 \cdot lb) \cdot 1.6}{vb_1}$

SUMMARY, STORY 1

Reduction in shear walls due to height to width ratio less than 2:1

$$ratio_{i1} := \frac{l1_{i1}}{h1_{i1}}$$

$$r := if(2 \cdot min(ratio) > 1.0, 1.0, 2 \cdot min(ratio)) \qquad r = 1$$

Reduced Unit Shear:

$$\frac{v1}{r} = 100 \cdot plf$$

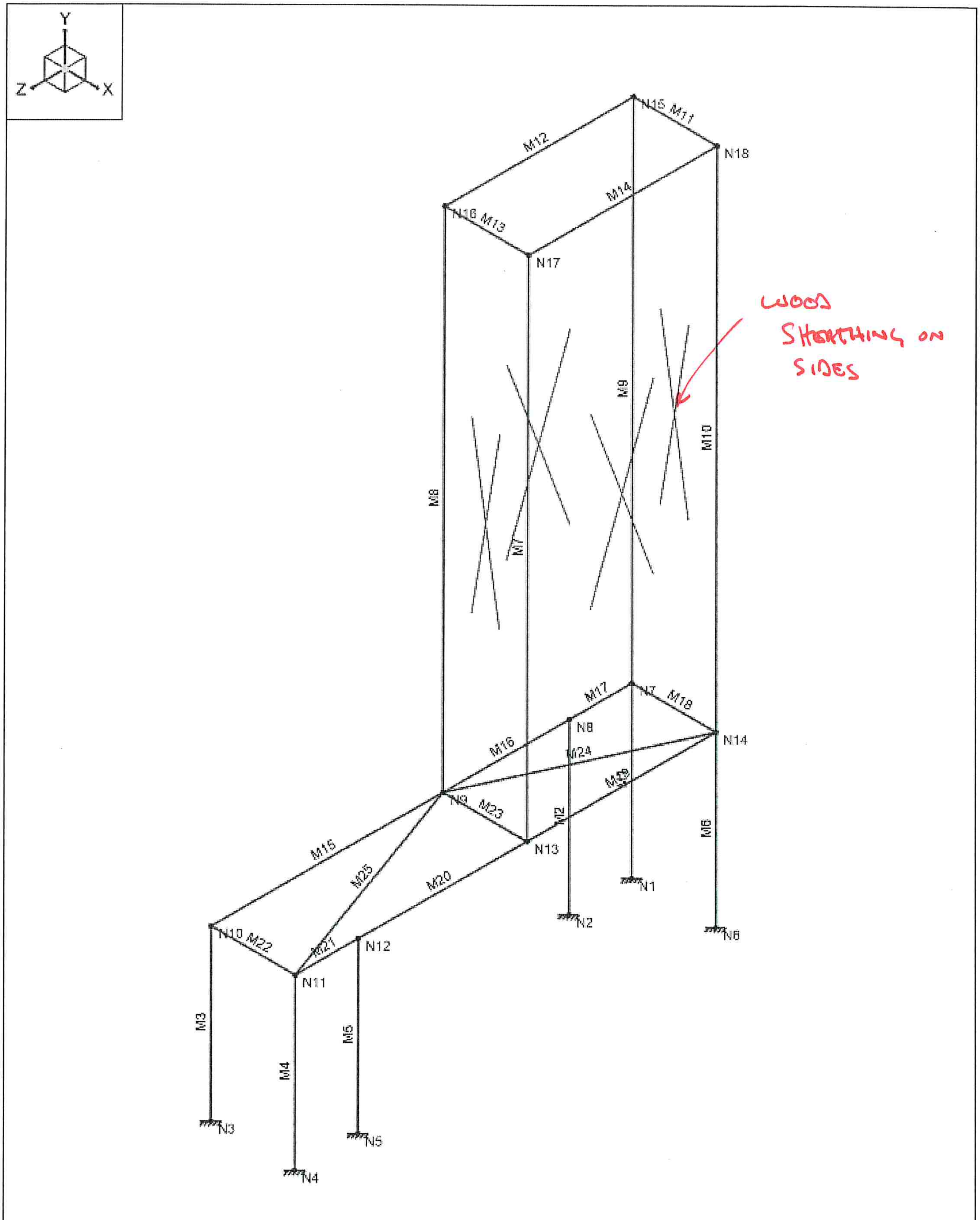
SHEAR WALLS

Sheathing:	7/16", APA, Exp. 1
Blocking:	All Panel Edges
Edge Nailing:	8d @ 6" o.c.
Field Nailing:	8d @ 12" o.c.

	Uplift	HOLD DOWN
Pier 1:	$T1_1 = 55 \cdot lb$	STHD14
Pier 2:	$T1_2 = ■ \cdot lb$	
Pier 3:	$T1_3 = ■ \cdot lb$	
Pier 4:	$T1_4 = ■ \cdot lb$	
Pier 5:	$T1_5 = ■ \cdot lb$	
Pier 6:	$T1_6 = ■ \cdot lb$	
Pier 7:	$T1_7 = ■ \cdot lb$	
Pier 8:	$T1_8 = ■ \cdot lb$	

ANCHOR BOLTS	1/2" A.Bolts	5/8" A.Bolts
	$s_{0.5} = 125 \cdot in$	$s_{0.625} = 179 \cdot in$

USE:
5/8" dia. x 10" J-bolts
Spacing = 32" o.c.



Dynamic Structures	Coffee Shop Fireplace Frame	SK-2
Jay Adams		Jan 05, 2021
		Fireplace Frame.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	0	1.5	
3	N3	0	0	10	
4	N4	2	0	10	
5	N5	2	0	8.5	
6	N6	2	0	0	
7	N7	0	4	0	
8	N8	0	4	1.5	
9	N9	0	4	4.5	
10	N10	0	4	10	
11	N11	2	4	10	
12	N12	2	4	8.5	
13	N13	2	4	4.5	
14	N14	2	4	0	
15	N15	0	16	0	
16	N16	0	16	4.5	
17	N17	2	16	4.5	
18	N18	2	16	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Y Rot [k-ft/rad]	Z Rot [k-ft/rad]
1	N1	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
2	N2	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
3	N3	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
4	N4	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
5	N5	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
6	N6	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e⁵°F⁻¹]	Density [k/ft³]	Yield [ksi]	Ry	Fu [ksi]	Rt
1	A992	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
2	A36 Gr.36	29000	11154	0.3	0.65	0.49	36	1.5	58	1.2
3	A572 Gr.50	29000	11154	0.3	0.65	0.49	50	1.1	65	1.1
4	A500 Gr.B RND	29000	11154	0.3	0.65	0.527	42	1.4	58	1.3
5	A500 Gr.B Rect	29000	11154	0.3	0.65	0.527	46	1.4	58	1.3
6	A53 Gr.B	29000	11154	0.3	0.65	0.49	35	1.6	60	1.2
7	A1085	29000	11154	0.3	0.65	0.49	50	1.25	65	1.15
8	A913 Gr.65	29000	11154	0.3	0.65	0.49	65	1.1	80	1.1

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Frame	HSS3X3X4	Column	SquareTube	A53 Gr.B	Typical	2.44	3.02	3.02	5.08
2	Beams	HSS6X3X4	Beam	Wide Flange	A53 Gr.B	Typical	3.84	5.7	17	14.2

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N7	Frame	Column	SquareTube	A53 Gr.B	Typical
2	M2	N2	N8	Frame	Column	SquareTube	A53 Gr.B	Typical
3	M3	N3	N10	Frame	Column	SquareTube	A53 Gr.B	Typical
4	M4	N4	N11	Frame	Column	SquareTube	A53 Gr.B	Typical
5	M5	N5	N12	Frame	Column	SquareTube	A53 Gr.B	Typical
6	M6	N6	N14	Frame	Column	SquareTube	A53 Gr.B	Typical
7	M7	N13	N17	Frame	Column	SquareTube	A53 Gr.B	Typical
8	M8	N9	N16	Frame	Column	SquareTube	A53 Gr.B	Typical
9	M9	N7	N15	Frame	Column	SquareTube	A53 Gr.B	Typical
10	M10	N14	N18	Frame	Column	SquareTube	A53 Gr.B	Typical

Member Primary Data (Continued)

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
11	M11	N18	N15	Frame	Column	SquareTube	A53 Gr.B	Typical
12	M12	N15	N16	Frame	Column	SquareTube	A53 Gr.B	Typical
13	M13	N16	N17	Frame	Column	SquareTube	A53 Gr.B	Typical
14	M14	N17	N18	Frame	Column	SquareTube	A53 Gr.B	Typical
15	M15	N10	N9	Beams	Beam	Wide Flange	A53 Gr.B	Typical
16	M16	N9	N8	Beams	Beam	Wide Flange	A53 Gr.B	Typical
17	M17	N8	N7	Beams	Beam	Wide Flange	A53 Gr.B	Typical
18	M18	N7	N14	Beams	Beam	Wide Flange	A53 Gr.B	Typical
19	M19	N14	N13	Beams	Beam	Wide Flange	A53 Gr.B	Typical
20	M20	N13	N12	Beams	Beam	Wide Flange	A53 Gr.B	Typical
21	M21	N12	N11	Beams	Beam	Wide Flange	A53 Gr.B	Typical
22	M22	N11	N10	Beams	Beam	Wide Flange	A53 Gr.B	Typical
23	M23	N9	N13	Beams	Beam	Wide Flange	A53 Gr.B	Typical
24	M24	N14	N9	Beams	Beam	Wide Flange	A53 Gr.B	Typical
25	M25	N9	N11	Beams	Beam	Wide Flange	A53 Gr.B	Typical

Basic Load Cases

	BLC Description	Category	Y Gravity	Distributed	Area(Member)	Surface(Plate/Wall)
1	Dead Load	DL	-1	5	1	
2	Snow Load	SL				
3	Wind X	WLX				1
4	Wind Z	WLZ				1
5	Seismic X	ELX				1
6	Seismic Z	ELZ				1
7	BLC 1 Transient Area Loads	None		22		

Load Combinations

	Description	Solve	PDelta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	IBC 16-1	Yes	Y	DL	1.4										
2	IBC 16-2 (a)	Yes	Y	DL	1.2	LL	1.6	LLS	1.6						
3	IBC 16-2 (b)	Yes	Y	DL	1.2	LL	1.6	LLS	1.6	SL	0.5	SLN	0.5		
4	IBC 16-3 (c)	Yes	Y	DL	1.2	SL	1.6	SLN	1.6	LL	0.5	LLS	1		
5	IBC 16-3 (b) (a)	Yes	Y	DL	1.2	WLX	0.5								
6	IBC 16-3 (b) (b)	Yes	Y	DL	1.2	WLZ	0.5								
7	IBC 16-3 (b) (c)	Yes	Y	DL	1.2	WLX	-0.5								
8	IBC 16-3 (b) (d)	Yes	Y	DL	1.2	WLZ	-0.5								
9	IBC 16-3 (d) (a)	Yes	Y	DL	1.2	SL	1.6	SLN	1.6	WLX	0.5				
10	IBC 16-3 (d) (b)	Yes	Y	DL	1.2	SL	1.6	SLN	1.6	WLZ	0.5				
11	IBC 16-3 (d) (c)	Yes	Y	DL	1.2	SL	1.6	SLN	1.6	WLX	-0.5				
12	IBC 16-3 (d) (d)	Yes	Y	DL	1.2	SL	1.6	SLN	1.6	WLZ	-0.5				
13	IBC 16-4 (a) (a)	Yes	Y	DL	1.2	WLX	1	LL	0.5	LLS	1				
14	IBC 16-4 (a) (b)	Yes	Y	DL	1.2	WLZ	1	LL	0.5	LLS	1				
15	IBC 16-4 (a) (c)	Yes	Y	DL	1.2	WLX	-1	LL	0.5	LLS	1				
16	IBC 16-4 (a) (d)	Yes	Y	DL	1.2	WLZ	-1	LL	0.5	LLS	1				
17	IBC 16-4 (b) (a)	Yes	Y	DL	1.2	WLX	1	LL	0.5	LLS	1	SL	0.5	SLN	0.5
18	IBC 16-4 (b) (b)	Yes	Y	DL	1.2	WLZ	1	LL	0.5	LLS	1	SL	0.5	SLN	0.5
19	IBC 16-4 (b) (c)	Yes	Y	DL	1.2	WLX	-1	LL	0.5	LLS	1	SL	0.5	SLN	0.5
20	IBC 16-4 (b) (d)	Yes	Y	DL	1.2	WLZ	-1	LL	0.5	LLS	1	SL	0.5	SLN	0.5
21	IBC 16-6 (a)	Yes	Y	DL	0.9	WLX	1								
22	IBC 16-6 (b)	Yes	Y	DL	0.9	WLZ	1								
23	IBC 16-6 (c)	Yes	Y	DL	0.9	WLX	-1								
24	IBC 16-6 (d)	Yes	Y	DL	0.9	WLZ	-1								
25	IBC 16-5 (a)	Yes	Y	DL	1.2	ELX	1	LL	0.5	LLS	1	SL	0.2	SLN	0.7
26	IBC 16-5 (b)	Yes	Y	DL	1.2	ELZ	1	LL	0.5	LLS	1	SL	0.2	SLN	0.7
27	IBC 16-5 (c)	Yes	Y	DL	1.2	ELX	-1	LL	0.5	LLS	1	SL	0.2	SLN	0.7
28	IBC 16-5 (d)	Yes	Y	DL	1.2	ELZ	-1	LL	0.5	LLS	1	SL	0.2	SLN	0.7
29	IBC 16-7 (a)	Yes	Y	DL	0.9	ELX	1								

Load Combinations (Continued)

	Description	Solve	PDelta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
30	IBC 16-7 (b)	Yes	Y	DL	0.9	ELZ	1								
31	IBC 16-7 (c)	Yes	Y	DL	0.9	ELX	-1								
32	IBC 16-7 (d)	Yes	Y	DL	0.9	ELZ	-1								

Envelope AISC 15th (360-16): LRFD Steel Code Checks

ALL ≤ 1.0

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
1	M1	HSS3X3X4	0.4478	0	25	0.067	4	y	25	69.8751	76.86	6.51	6.51	2.232	H1-1b
2	M2	HSS3X3X4	0.3427	0	27	0.0501	4	y	25	69.8751	76.86	6.51	6.51	2.201	H1-1b
3	M3	HSS3X3X4	0.2583	4	27	0.0347	4	z	27	69.8751	76.86	6.51	6.51	2.2556	H1-1b
4	M4	HSS3X3X4	0.1893	4	25	0.033	4	y	25	69.8751	76.86	6.51	6.51	2.2664	H1-1b
5	M5	HSS3X3X4	0.2297	0	25	0.0351	4	z	25	69.8751	76.86	6.51	6.51	2.1844	H1-1b
6	M6	HSS3X3X4	0.6091	4	25	0.0667	4	y	25	69.8751	76.86	6.51	6.51	2.2379	H1-1a
7	M7	HSS3X3X4	0.0366	0	26	0.0102	12	z	25	32.6062	76.86	6.51	6.51	1.2438	H1-1b
8	M8	HSS3X3X4	0.0279	0	26	0.0096	12	z	25	32.6062	76.86	6.51	6.51	1.0132	H1-1b
9	M9	HSS3X3X4	0.0873	0	31	0.0102	12	z	25	32.6062	76.86	6.51	6.51	1.1547	H1-1b*
10	M10	HSS3X3X4	0.0973	0	25	0.0095	12	z	25	32.6062	76.86	6.51	6.51	1.1508	H1-1b*
11	M11	HSS3X3X4	0.3114	0	25	0.1541	0	y	25	75.0509	76.86	6.51	6.51	2.2659	H1-1b
12	M12	HSS3X3X4	0.1251	4.5	26	0.025	4.5	y	25	68.1289	76.86	6.51	6.51	2.2906	H1-1b
13	M13	HSS3X3X4	0.1716	0	27	0.0937	0	y	27	75.0509	76.86	6.51	6.51	2.2725	H1-1b
14	M14	HSS3X3X4	0.1372	0	26	0.0168	0	y	27	68.1289	76.86	6.51	6.51	2.1005	H1-1b
15	M15	HSS6X3X4	0.1906	5.5	27	0.0526	0	y	25	104.0907	120.96	11.5762	18.8738	2.0563	H1-1b
16	M16	HSS6X3X4	0.361	3	27	0.1345	3	y	27	115.6737	120.96	11.5762	18.8738	2.0835	H1-1b
17	M17	HSS6X3X4	0.3463	0	27	0.1961	0	y	27	119.6162	120.96	11.5762	18.8738	2.1057	H1-1b
18	M18	HSS6X3X4	0.2477	0	27	0.1892	2	y	25	118.5813	120.96	11.5762	18.8738	2.2684	H1-1b
19	M19	HSS6X3X4	0.2169	3.6563	25	0.1103	0	y	25	109.3895	120.96	11.5762	18.8738	1.4272	H1-1b
20	M20	HSS6X3X4	0.2817	0	25	0.0926	4	y	25	111.7223	120.96	11.5762	18.8738	2.2182	H1-1b
21	M21	HSS6X3X4	0.2264	0	25	0.1532	0	y	25	119.6162	120.96	11.5762	18.8738	2.1782	H1-1b
22	M22	HSS6X3X4	0.1127	0	27	0.0774	2	y	27	118.5813	120.96	11.5762	18.8738	2.1452	H1-1b
23	M23	HSS6X3X4	0.0826	0	26	0.1025	0	y	1	118.5813	120.96	11.5762	18.8738	2.8631	H1-1b
24	M24	HSS6X3X4	0.1012	4.9244	25	0.0214	4.9244	z	27	107.2383	120.96	11.5762	18.8738	1.2551	H1-1b
25	M25	HSS6X3X4	0.1874	0	26	0.0585	5.8523	y	25	102.0438	120.96	11.5762	18.8738	2.0267	H1-1b

Envelope Story Drift - X-Direction, Strength

REASONABLE MAX DRIFT

	Story (Elevation)		Story Drift[in]	Loc (Z,X)	LC	Drift Ratio (%)	Loc (Z,X)	LC	2nd/1st Ratio	Loc (Z,X)	LC
1	N15 (16 ft)	max	1.3997	0, 0	25	0.972	0, 0	25	1.0044	0, 0	19
2		min	-1.0919	0, 0	31	0.0034	0, 0	7	0.9895	0, 0	7
3	N16 (16 ft)	max	1.5393	4.5, 0	25	1.0689	4.5, 0	25	1.0588	4.5, 0	11
4		min	-1.2285	4.5, 0	31	0.0009	4.5, 0	7	0.997	4.5, 0	15
5	N7 (4 ft)	max	0.6982	0, 0	25	1.4545	0, 0	25	1.0137	0, 0	1
6		min	-0.6497	0, 0	31	0.0096	0, 0	24	1.0075	0, 0	30
7	N10 (4 ft)	max	0.1736	10, 0	29	0.4133	10, 0	27	1.0175	10, 0	9
8		min	-0.1984	10, 0	27	0.005	10, 0	5	1.0008	10, 0	24

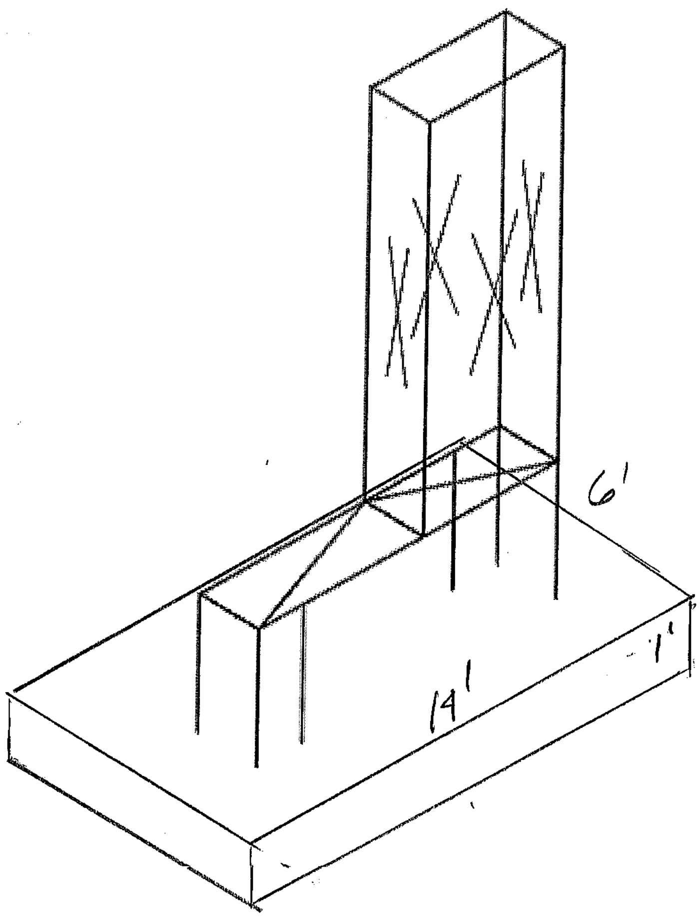
Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	0.9686	27	8.0061	31	0.2989	28	0.5725	28	0.1064	25	2.1657	25
2		min	-1.0133	25	-10.7172	25	-0.1736	30	-0.4297	30	-0.0909	31	-2.1249	27
3	N2	max	0.6107	31	11.4643	27	0.4033	28	0.7157	28	0.1085	25	1.5931	25
4		min	-0.6111	29	4.0926	29	-0.0765	30	-0.3149	30	-0.0932	31	-1.5669	27
5	N3	max	0.3227	27	2.6854	27	0.1056	32	0.3213	32	0.1087	25	0.608	29
6		min	-0.2956	29	-0.5628	29	-0.5462	26	-0.9266	26	-0.0945	31	-0.6793	27
7	N4	max	0.2628	31	-1.0617	29	0.1808	32	0.4119	32	0.1017	25	0.6135	25
8		min	-0.3035	25	-2.9885	27	-0.435	26	-0.7875	26	-0.0877	31	-0.5893	31
9	N5	max	0.2391	31	6.6451	25	0.1699	32	0.4022	32	0.1019	25	0.6286	29
10		min	-0.2364	29	2.5656	31	-0.4768	26	-0.8529	26	-0.089	31	-0.6483	27
11	N6	max	1.0099	31	16.7159	25	0.496	28	0.8291	28	0.0936	25	2.218	25
12		min	-0.9533	25	-6.9924	31	0.0518	30	-0.1513	30	-0.078	31	-2.12	31

Envelope Node Reactions (Continued)

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
13	Totals:	max	3.402	27	19.8993	1	1.512	28						
14		min	-3.402	29	12.7924	30	-1.512	30						

FIREPLACE FOOTING



MINIMUM FOOTING PER GEOMETRY = 6'-0" × 14'-0"

CHECK STABILITY IN 6'-0" DIRECTION

OVERTURNING MOMENT IN SHORT DIRECTION = $6^{k\cdot ft}$

FOOTING WEIGHT = 14'(6')(1')(.150kcf) = 12.6^k

RIGHTING MOMENT = $0.6(12.6^k)(3') = 23^{k\cdot ft} > 6^{k\cdot ft}$ O.K.

DYNAMIC STRUCTURES
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PH: (801) 356-1140 FAX: (801) 356-0001

PROJECT: ____________

SUBJECT: ____________

DATE: ____________