DYNAMIC STRUCTURES, INC.
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Structural Calculations for:

MILLCREEK COMMON
ICE SUPPORT BUILDING

1300 EAST 3300 SOUTH
MILLCREEK, UTAH

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

1-15-21

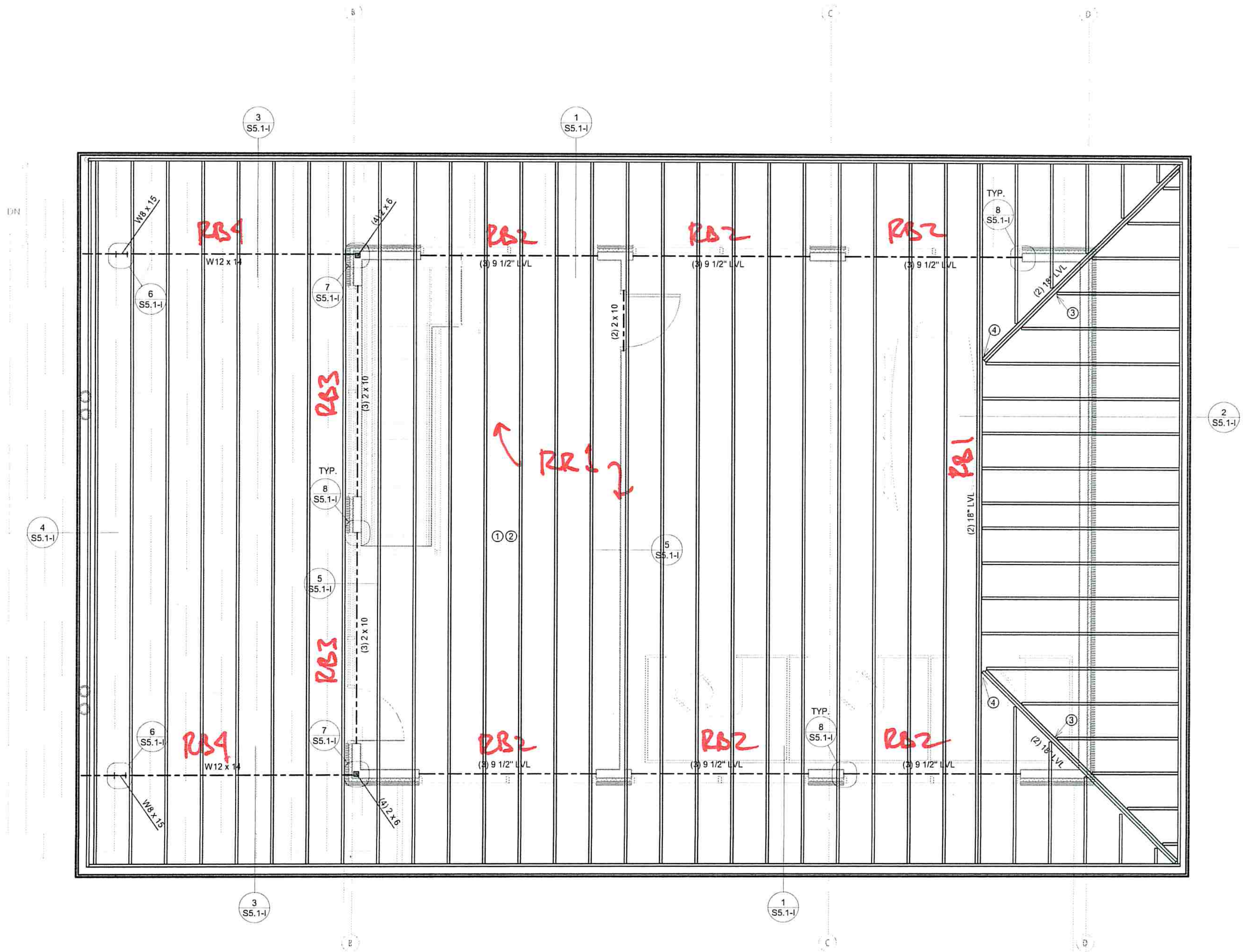
January 15, 2021

Service Prepared for:

WPA ARCHITECTS

PROJECT:	MILLCREEK COMMON ICE SUPPORT BUILDING 1300 EAST 3300 SOUTH MILLCREEK, UTAH	
CLIENT:	WPA ARCHITECTURE	
SCOPE:	STRUCTURAL CALCULATIONS AND STRUCTURAL PLANS FOR A NEW MAINTENANCE 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**
	Floor / Deck Live Load:	**=100.0 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**
FLOOR DEAD LOADS (INTERIOR):	Flooring / Topping	=17.50 psf
	Floor Sheathing	=2.50 psf
	Joists	=2.00 psf
	Ceiling	=3.00 psf
	Insulation	=3.00 psf
	Misc	=2.00 psf
	Total floor dead load:	**=30.0 psf**
FLOOR DEAD LOADS (EXTERIOR DECK):	Concrete Pavers	=45.00 psf
	Floor Sheathing	=2.50 psf
	Joists	=2.00 psf
	Ceiling	=3.00 psf
	Insulation	=5.50 psf
	Misc	=2.00 psf
	Total floor dead load:	**=60.0 psf**
WALL DEAD LOADS:	Wood framed w/ light finish	=20.0 psf
	Wood framed w/ veneer	=60.0 psf
	CMU masonry	=60.0 psf



ROOF FRAMING KEY PLAN

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
JOISTS FRAMING HIGH ROOF
RR1

Span Length: $l := 29.5 \cdot ft$

Spacing: $s := 24 \cdot in$

Loads per lineal foot: $wll := 30 \cdot psf \cdot s$

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

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

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

$p := pll + pdl$

Calculate the bending moment: $Mx := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8} \right)$ $\quad Mx = 11965.9 \cdot ft \cdot lb$

Calculate the shear: $Vx := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2} \right)$ $\quad Vx = 1622.5 \cdot lb$

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

Joist Series: $Series := 560$ $\quad j := if(Series > 110, if(Series > 210, if(Series > 360, 4, 3), 2), 1)$

Joist Depth: $d := 18 \cdot in$ $\quad i := if(d > 9.5 \cdot in, if(d > 11.875 \cdot in, if(d > 14 \cdot in, 4, 3), 2), 1)$

USE: 18" TJI 560 @ 24" O.C.

	Joist Capacities	>	**Required Capacities**
Moment Capacity:	$M := 14550 \cdot lb \cdot ft$	>	$Mx = 11965.9 \cdot ft \cdot lb$
Maximum Reaction:	$V := 1631 \cdot lb$	>	$Vx = 1622.5 \cdot lb$
Deflection Constant:	$EI_{i,j} = 1.252 \times 10^3 \cdot 10^6 \cdot in^2 \cdot lb$		

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: $\frac{l}{240} = 1.48 \cdot in$ $\quad > \quad$ $y = 1.64 \cdot in$

18" 560 AT 24" O.C. $\quad y := \frac{y \cdot 1252}{1631} = 1.26 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB-2

Length: $l := 10 \cdot ft$ $a := 0 \cdot ft$ (larger)
Concentrated load: $p := 0 \cdot lb$ $c := l - a$
Weight per lineal foot: $w := 1350 \cdot plf$
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 16875 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 6750 \cdot lb$

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

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 77.9 \cdot in^3$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 29.9 \cdot in^2$

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: $\frac{l}{240} = 0.5 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.405 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB-3

Length: $l := 12 \cdot ft$ $a := 0 \cdot ft$ (larger)
Concentrated load: $p := 0 \cdot lb$ $c := l - a$
Weight per lineal foot: $w := (25 + 30) \cdot 2 \cdot plf$
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 1980 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 660 \cdot lb$

Use: **(3) 2 X 10** $b := 4.5 \cdot in$ $d := 9.25 \cdot in$ $t := \mathrm{if}(d \le 12 \cdot in, 12 \cdot in, d)$

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

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

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

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 24 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 64.2 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 4.8 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 41.6 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 296.8 \cdot in^4$

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

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.108 \cdot in$

Gravity Beam Design

RAM SBeam v06.00.00
ICE BUILDING
RB4

RAM Structural System
Bentley

12/10/20 11:53:35

STEEL CODE: AISC 360-10 LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (16.00,0.00)

Beam Size (Optimum)	=	W12X14	Fy	= 50.0 ksi
Total Beam Length (ft)	=	16.00		
Cantilever on left (ft)	=	2.50		

Mp (kip-ft) = 72.50
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL	PartL
1	0.000	0.014	0.000	0.000
	2.500	0.014	0.000	0.000
2	0.000	1.350	0.000	0.000
	2.500	1.350	0.000	0.000
3	2.500	0.014	0.000	0.000
	16.000	0.014	0.000	0.000
4	2.500	1.350	0.000	0.000
	16.000	1.350	0.000	0.000

SHEAR (Ultimate): Max Vu (1.4DL) = 13.33 kips 0.90Vn = 64.26 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Left	Max -	1.4DL	-6.0	2.5	2.5	1.00	0.90	65.25
Center	Max +	1.4DL	40.6	9.5	0.0	1.00	0.90	65.25
	Max -	1.4DL	-6.0	2.5	13.5	1.15	0.90	17.83
Controlling		1.4DL	40.6	9.5	0.0	1.00	0.90	65.25

REACTIONS (kips):

	Left	**Right**
DL reaction	12.93	8.89
Max +total reaction (factored)	18.11	12.45

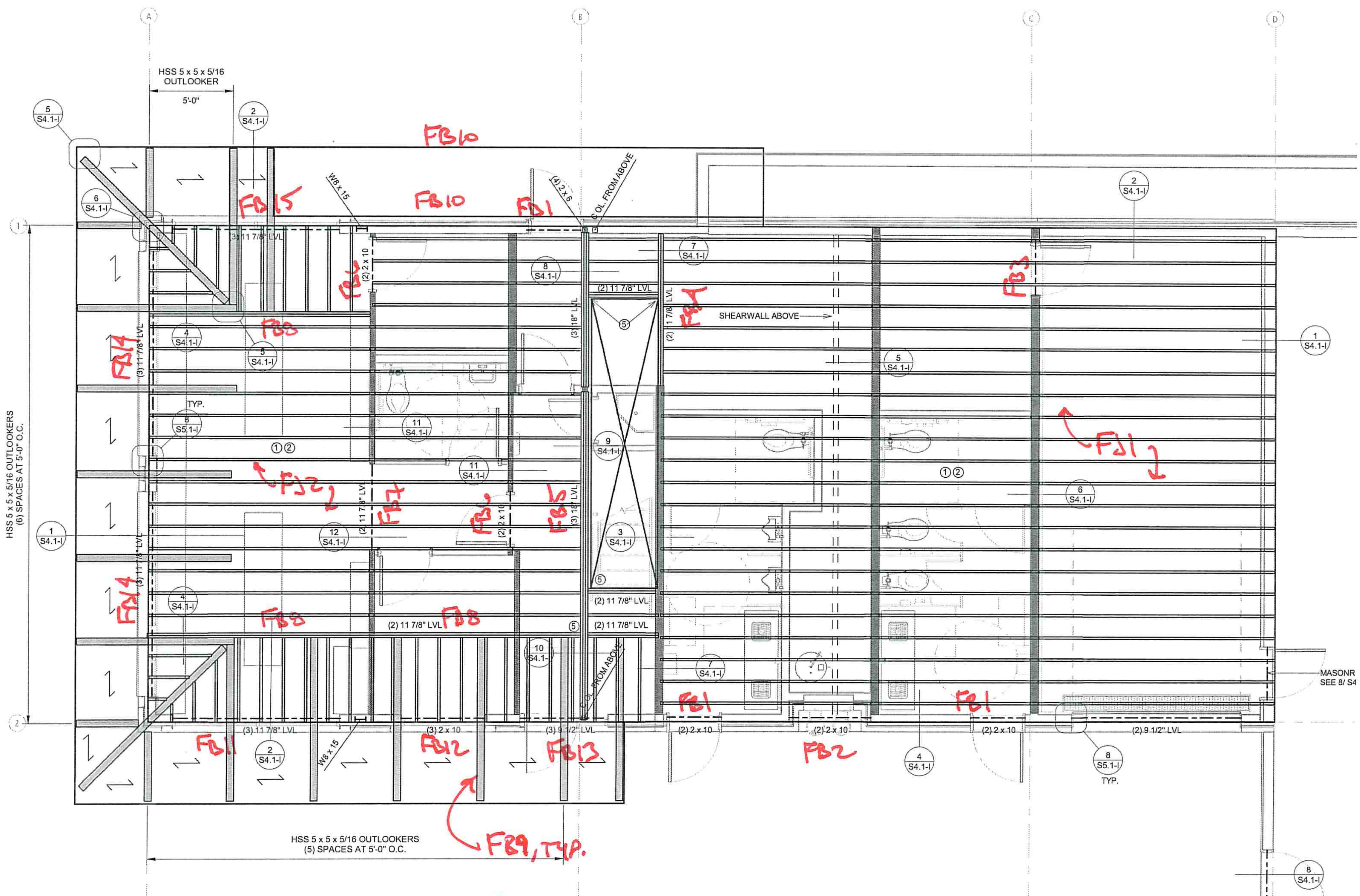
DEFLECTIONS:

Left cantilever:

Dead load (in)	=	0.198	L/D	=	302
Neg Total load (in)	=	0.198	L/D	=	302

Center span:

Dead load (in)	at	9.32 ft	=	-0.364	L/D	=	445
Live load (in)	at	9.32 ft	=	-0.000			
Net Total load (in)	at	9.32 ft	=	-0.364	L/D	=	445



FLOOR / DECK FRAMING KEY PLAN

FLOOR FRAMING
FLOOR JOISTS
FJ1

Span Length: $l := 14 \cdot ft$

Spacing: $s := 16 \cdot in$

Loads per lineal foot: $wll := 100 \cdot psf \cdot s$

$wdl := 30 \cdot psf \cdot s$ $\quad w := wdl + wll$

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

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

$p := pll + pdl$

Calculate the bending moment: $Mx := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8} \right)$ $\quad Mx = 4246.7 \cdot ft \cdot lb$

Calculate the shear: $Vx := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2} \right)$ $\quad Vx = 1213.3 \cdot lb$

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

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

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

USE: 11" TJI 360 @ 16" O.C.

	Joist Capacities	>	**Required Capacities**
Moment Capacity:	$M_{i,j} = 6180 \cdot ft \cdot lb$	>	$Mx = 4246.7 \cdot ft \cdot lb$
Maximum Reaction:	$V_{i,j} = 1705 \cdot lb$	>	$Vx = 1213.3 \cdot lb$
Deflection Constant:	$EI_{i,j} = 419 \cdot 10^6 \cdot in^2 \cdot lb$		

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: $\frac{l}{240} = 0.7 \cdot in$ $\quad > \quad y = 0.43 \cdot in$

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

Compare Live Load deflection with allowable: $\frac{l}{480} = 0.35 \cdot in$ $\quad > \quad y = 0.334 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

DECK FRAMING
DECK JOISTS
FJ2

Span Length: $l := 14 \cdot ft$

Spacing: $s := 16 \cdot in$

Loads per lineal foot: $wll := 100 \cdot psf \cdot s$

$wdl := 60 \cdot psf \cdot s$ $\quad w := wdl + wll$

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

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

$p := pll + pdl$

Calculate the bending moment: $Mx := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $\quad \underline{Mx = 5226.7 \cdot ft \cdot lb}$

Calculate the shear: $Vx := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $\quad \underline{Vx = 1493.3 \cdot lb}$

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

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

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

USE: 11" TJI 360 @ 16" O.C.

	Joist Capacities	**>**	**Required Capacities**
Moment Capacity:	$M_{i,j} = 6180 \cdot ft \cdot lb$	>	$Mx = 5226.7 \cdot ft \cdot lb$
Maximum Reaction:	$V_{i,j} = 1705 \cdot lb$	>	$Vx = 1493.3 \cdot lb$
Deflection Constant:	$EI_{i,j} = 419 \cdot 10^6 \cdot in^2 \cdot lb$		

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: $\frac{l}{240} = 0.7 \cdot in$ $\quad > \quad y = 0.53 \cdot in$

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

Compare Live Load deflection with allowable: $\frac{l}{480} = 0.35 \cdot in$ $\quad > \quad y = 0.334 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-1

Length: $l := 3 \cdot ft$ $a := 0 \cdot ft$ (larger)

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

Weight per lineal foot: $w := 1600 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 1800 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 2400 \cdot lb$

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

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 21.8 \cdot in^3$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 9.7 \cdot in^2$

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: $\frac{l}{240} = 0.15 \cdot in$ $\frac{l}{360} = 0.1 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.009 \cdot in$ $y \cdot \frac{100}{130} = 0.007 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-2

Length: $l := 5 \cdot ft$ $a := 0 \cdot ft$ (larger)

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

Weight per lineal foot: $w := 1600 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 5000 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 4000 \cdot lb$

Use: **(2) 1 3/4 X 11 7/8 LVL** $b := 3.5 \cdot in$ $d := 11.875 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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) $C_1 := if(d > 10 \cdot in, if(d > 12 \cdot in, 0.9, 1.0), ($ $C_2 := C_1$ $C_1 = 1$

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 23.1 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 82.3 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 12.7 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 41.6 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 488.4 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.25 \cdot in$ $\frac{l}{360} = 0.167 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.023 \cdot in$ $y \cdot \frac{100}{130} = 0.018 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-3

Length: $l := 3 \cdot ft$ $a := 0 \cdot ft$ (larger)

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

Weight per lineal foot: $w := 1600 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $\underline{M = 1800 \cdot ft \cdot lb}$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $\underline{V = 2400 \cdot lb}$

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

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $\underline{S = 21.8 \cdot in^3}$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $\underline{A = 9.7 \cdot in^2}$

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: $\frac{l}{240} = 0.15 \cdot in$ $\frac{l}{360} = 0.1 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.009 \cdot in$ $y \cdot \frac{100}{130} = 0.007 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-4

Length: $l := 9 \cdot ft$ $a := 5 \cdot ft$ (larger)
Concentrated load: $p := 1500 \cdot lb$ $c := l - a$
Weight per lineal foot: $w := 1200 \cdot plf$
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 15483.3 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 6233.3 \cdot lb$

Use: **(2) 1 3/4 X 11 7/8 LVL** $b := 3.5 \cdot in$ $d := 11.875 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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) $C_1 := if(d > 10 \cdot in, if(d > 12 \cdot in, 0.9, 1.0), (C_2 := C_1$ $C_1 = 1$

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

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 71.5 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 82.3 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 26.6 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 41.6 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 488.4 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.45 \cdot in$ $\frac{l}{360} = 0.3 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.221 \cdot in$ $y \cdot \frac{100}{130} = 0.17 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-5

Length: $l := 20 \cdot ft$ $a := 0 \cdot ft$ (larger)

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

Weight per lineal foot: $w := (60 + 100)2.5 \cdot plf + 150 \cdot plf + (25 + 30) \cdot 2 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 33000 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 6600 \cdot lb$

Use: (3) 1 3/4 X 18 LVL $b := 5.25 \cdot in$ $d := 18 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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

Cv = volume factor (glu-laminated lumber only) $C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1}$ $C_3 := if(C_3 > 1, 1, C_3)$ $C_3 = 0.963$

CF = LSL size factor: $C_4 := \left(\frac{12 \cdot in}{t}\right)^{0.092}$ $C_4 = 0.963$ CF = LVL size factor: $C_5 := \left(\frac{12 \cdot in}{t}\right)^{0.136}$ $C_5 = 0.946$

CF = PSL size factor: $C_6 := \left(\frac{12 \cdot in}{t}\right)^{0.111}$ $C_6 = 0.956$ $C_8 := C_1$ $C_9 := C_1$

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 160.9 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 283.5 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 29.5 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 94.5 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 2551.5 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 1 \cdot in$ $\frac{l}{360} = 0.667 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.466 \cdot in$ $y \cdot \frac{40}{50} = 0.372 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-6

Length: $l := 3 \cdot ft$ $a := 0 \cdot ft$ (larger)
Concentrated load: $p := 0 \cdot lb$ $c := l - a$
Weight per lineal foot: $w := 1100 \cdot plf$
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 1237.5 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 1650 \cdot lb$

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

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 15 \cdot in^3$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 6.7 \cdot in^2$

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: $\frac{l}{240} = 0.15 \cdot in$ $\frac{l}{360} = 0.1 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.006 \cdot in$ $y \cdot \frac{100}{130} = 0.005 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-7

Length: $l := 6 \cdot ft$ $a := 0 \cdot ft$ (larger)
Concentrated load: $p := 0 \cdot lb$ $c := l - a$
Weight per lineal foot: $w := 1700 \cdot plf$
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 7650 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 5100 \cdot lb$

Use: **(2) 1 3/4 X 11 7/8 LVL** $b := 3.5 \cdot in$ $d := 11.875 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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) $C_1 := if(d > 10 \cdot in, if(d > 12 \cdot in, 0.9, 1.0), C_2 := C_1$ $C_1 = 1$

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 35.3 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 82.3 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 18 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 41.6 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 488.4 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.3 \cdot in$ $\frac{l}{360} = 0.2 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.051 \cdot in$ $y \cdot \frac{100}{130} = 0.039 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-8

Length: $l := 13 \cdot ft$ $a := 0 \cdot ft$ (larger)

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

Weight per lineal foot: $w := 500 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 10562.5 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 3250 \cdot lb$

Use: **(2) 1 3/4 X 11 7/8 LVL** $b := 3.5 \cdot in$ $d := 11.875 \cdot in$ $t := \text{if}(d \le 12 \cdot in, 12 \cdot in, d)$

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) $C_1 := \text{if}(d > 10 \cdot in, \text{if}(d > 12 \cdot in, 0.9, 1.0), (C_2 := C_1$ $C_1 = 1$

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

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 48.8 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 82.3 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 14.5 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 41.6 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 488.4 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.65 \cdot in$ $\frac{l}{360} = 0.433 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.329 \cdot in$ $y \cdot \frac{100}{130} = 0.253 \cdot in$

Gravity Beam Design

RAM SBeam v06.00.00
ICE BUILDING
FB-9

RAM Structural System
Bentley

12/10/20 13:17:36

STEEL CODE: AISC 360-10 ASD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (10.00,0.00)

Maximum Depth Limitation specified = 5.00 in
Beam Size (Optimum) = HSS5X5X5/16 Fy = 46.0 ksi
Total Beam Length (ft) = 10.00
Cantilever on left (ft) = 5.00
Mp (kip-ft) = 35.11
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL	PartL
1	0.000	0.018	0.000	0.000
	5.000	0.018	0.000	0.000
2	5.000	0.018	0.000	0.000
	10.000	0.018	0.000	0.000
3	0.000	0.800	0.000	0.000
	5.000	0.800	0.000	0.000

SHEAR: Max Va (DL+LL) = 4.09 kips Vn/1.67 = 39.70 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Left	Max -	DL	-10.2	5.0	5.0	1.00	1.67	21.03
Center	Max -	DL	-10.2	5.0	5.0	1.68	1.67	21.03
Controlling		DL	-10.2	5.0	5.0	1.00	1.67	21.03

REACTIONS (kips):

	Left	**Right**
DL reaction	6.18	-2.00
Max +total reaction	6.18	-2.00
Max -total reaction	6.18	-2.00

DEFLECTIONS:

Left cantilever:

Dead load (in)	= -0.466	L/D	=	257
Pos Total load (in)	= -0.466	L/D	=	257

Center span:

Dead load (in)	at	7.10 ft	=	0.051	L/D =	1177
Live load (in)	at	7.10 ft	=	-0.000		
Net Total load (in)	at	7.10 ft	=	0.051	L/D =	1177

Gravity Beam Design

RAM SBeam v06.00.00
Millcreek Ice Building
Ramp Stringer - FB-10

RAM Structural System
Bentley

12/10/20 13:17:56

STEEL CODE: AISC 360-10 ASD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (26.00,0.00)

Maximum Depth Limitation specified = 12.00 in
Beam Size (User Selected) = HSS12X4X5/16 Fy = 46.0 ksi
Total Beam Length (ft) = 26.00
Mp (kip-ft) = 119.98
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL	PartL
1	0.000	0.030	0.000	0.000
	26.000	0.030	0.000	0.000
2	0.000	0.100	0.200	0.000
	26.000	0.100	0.200	0.000

SHEAR: Max Va (DL+LL) = 4.29 kips Vn/1.67 = 107.03 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Center	Max +	DL+LL	27.9	13.0	0.0	1.00	1.67	71.85
Controlling		DL+LL	27.9	13.0	0.0	1.00	1.67	71.85

REACTIONS (kips):

	Left	**Right**
DL reaction	1.69	1.69
Max +LL reaction	2.60	2.60
Max +total reaction	4.29	4.29

DEFLECTIONS:

Dead load (in)	at	13.00 ft	=	-0.320	L/D	=	976
Live load (in)	at	13.00 ft	=	-0.492	L/D	=	634
Net Total load (in)	at	13.00 ft	=	-0.812	L/D	=	384

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-11

Length: $l := 10 \cdot ft$ $a := 0 \cdot ft$ (larger)
Concentrated load: $p := 0 \cdot lb$ $c := l - a$
Weight per lineal foot: $w := 1600 \cdot plf$
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 20000 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 8000 \cdot lb$

Use: (3) 1 3/4 X 11 7/8 LVL $b := 5.25 \cdot in$ $d := 16 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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

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: $C_4 := \left(\frac{12 \cdot in}{t}\right)^{0.092}$ $C_4 = 0.974$ CF = LVL size factor: $C_5 := \left(\frac{12 \cdot in}{t}\right)^{0.136}$ $C_5 = 0.962$

CF = PSL size factor: $C_6 := \left(\frac{12 \cdot in}{t}\right)^{0.111}$ $C_6 = 0.969$ $C_8 := C_1$ $C_9 := C_1$

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 96 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 224 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 30.9 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 84 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 1792 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.5 \cdot in$ $\frac{l}{360} = 0.333 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.1 \cdot in$ $y \cdot \frac{100}{160} = 0.063 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-12

Length: $l := 3 \cdot ft$ $a := 0 \cdot ft$ (larger)

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

Weight per lineal foot: $w := 1300 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 1462.5 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 1950 \cdot lb$

Use: **(3) 2 X 10** $b := 4.5 \cdot in$ $d := 9.25 \cdot in$ $t := \mathrm{if}(d \le 12 \cdot in, 12 \cdot in, d)$

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

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

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

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 17.7 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 64.2 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 7.9 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 41.6 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 296.8 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.15 \cdot in$ $\frac{l}{360} = 0.1 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.005 \cdot in$ $y \cdot \frac{100}{130} = 0.004 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-13

Length:	$l := 5 \cdot ft$	$a := 3.5 \cdot ft$	(larger)
Concentrated load:	$p := 9000 \cdot lb$	$c := l - a$	
Weight per lineal foot:	$w := 1200 \cdot plf$		
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 13200 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 9300 \cdot lb$

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

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 60.9 \cdot in^3$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 43.9 \cdot in^2$

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: $\frac{l}{240} = 0.25 \cdot in$ $\frac{l}{360} = 0.167 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.061 \cdot in$ $y \cdot \frac{100}{160} = 0.038 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-14

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

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

Weight per lineal foot: $w := 2200 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $\underline{M = 39600 \cdot ft \cdot lb}$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $\underline{V = 13200 \cdot lb}$

Use: **(3) 1 3/4 X 16 LVL** $b := 5.25 \cdot in$ $d := 16 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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

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: $C_4 := \left(\frac{12 \cdot in}{t}\right)^{0.092}$ $C_4 = 0.974$ CF = LVL size factor: $C_5 := \left(\frac{12 \cdot in}{t}\right)^{0.136}$ $C_5 = 0.962$

CF = PSL size factor: $C_6 := \left(\frac{12 \cdot in}{t}\right)^{0.111}$ $C_6 = 0.969$ $C_8 := C_1$ $C_9 := C_1$

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 190.1 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 224 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 54 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 84 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 1792 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.6 \cdot in$ $\frac{l}{360} = 0.4 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.286 \cdot in$ $y \cdot \frac{100}{160} = 0.179 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FB-15

Length: $l := 10 \cdot ft$ $a := 0 \cdot ft$ (larger)
Concentrated load: $p := 0 \cdot lb$ $c := l - a$
Weight per lineal foot: $w := 1600 \cdot plf$
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 20000 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 8000 \cdot lb$

Use: **(3) 1 3/4 X 11 7/8 LVL** $b := 5.25 \cdot in$ $d := 16 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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

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: $C_4 := \left(\frac{12 \cdot in}{t}\right)^{0.092}$ $C_4 = 0.974$ CF = LVL size factor: $C_5 := \left(\frac{12 \cdot in}{t}\right)^{0.136}$ $C_5 = 0.962$

CF = PSL size factor: $C_6 := \left(\frac{12 \cdot in}{t}\right)^{0.111}$ $C_6 = 0.969$ $C_8 := C_1$ $C_9 := C_1$

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 96 \cdot in^3$

Actual section modulus: $S := \frac{b}{6} \cdot d^2$ $S = 224 \cdot in^3$

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 30.9 \cdot in^2$

Actual area: $A := b \cdot d$ $A = 84 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 1792 \cdot in^4$

Allowable deflection: $\frac{l}{240} = 0.5 \cdot in$ $\frac{l}{360} = 0.333 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.1 \cdot in$ $y \cdot \frac{100}{160} = 0.063 \cdot in$

W8X15 WF Column

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	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	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	W8X15	Column	Wide Flange	A992	Typical	4.44	3.41	48	0.137

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	-6	Active
2	N2	L	MX	0.5	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	-7	Active
2	N2	L	MX	0.6	Active

Load Combinations

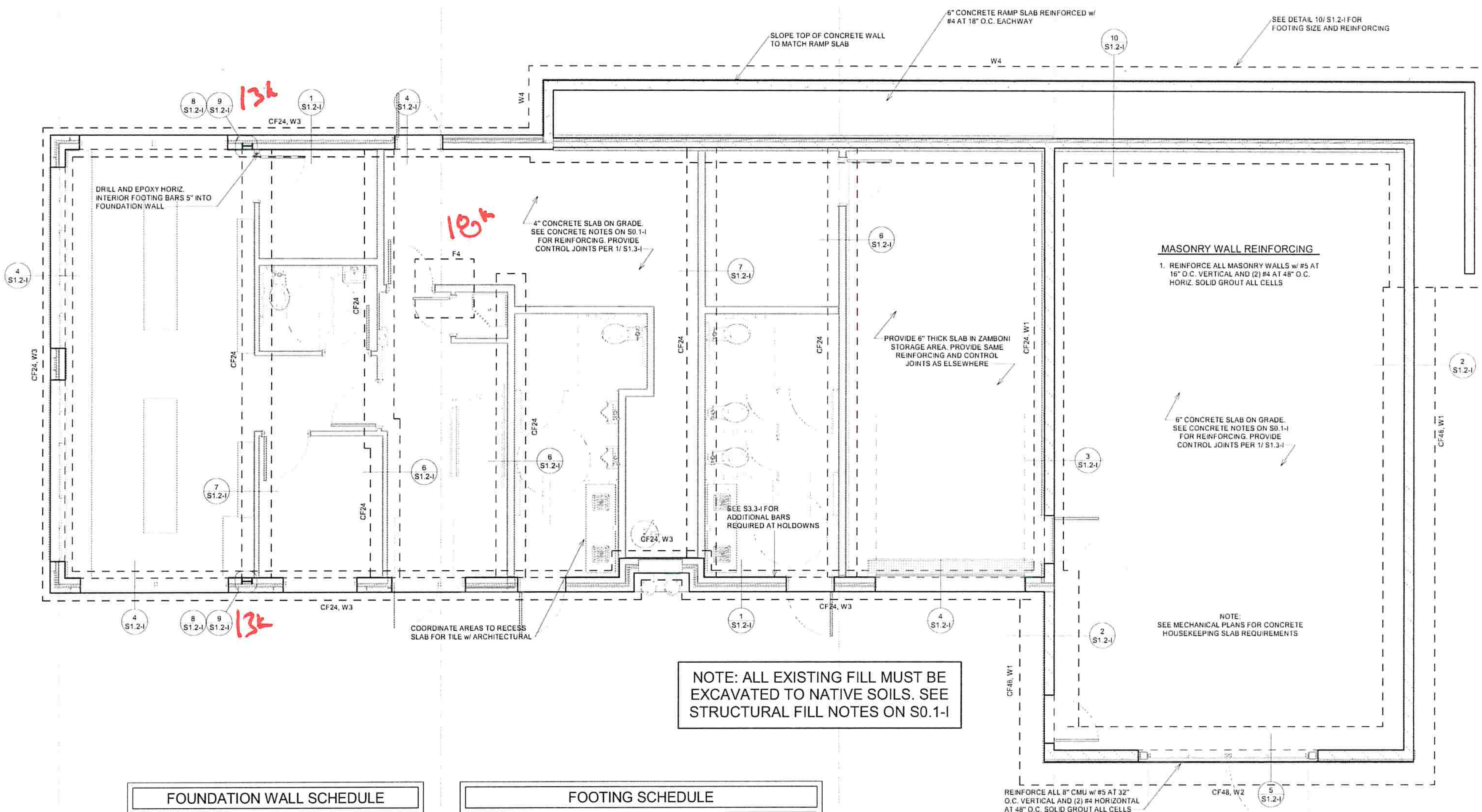
	Description	Solve	PDelta	BLC	Factor	BLC	Factor
1	Total Load	Yes	Y	1	1.2	2	1.6

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	W8X15	0.6338	12	0.0019	12	z	37.1509	199.8	10.0125	24.376	1	H1-1a

≤ 1.0

WORST CASE UNIFORM LOAD = 1800 PLF



NOTE: ALL EXISTING FILL MUST BE EXCAVATED TO NATIVE SOILS. SEE STRUCTURAL FILL NOTES ON S0.1-I

FOUNDATION WALL SCHEDULE

MARK	WALL WIDTH	WALL HEIGHT	VERT. REINFORCING SIZE	VERT. REINFORCING SPACING	HORIZ. REINFORCING SIZE	HORIZ. REINFORCING SPACING
W1	8"	4'-0" MAX	#4	18" O.C.	#4	12" O.C.
W2	10"	4'-0" MAX	#4	18" O.C.	#4	12" O.C.
★ W3	12"	4'-0" MAX	#4	18" O.C.	#4	12" O.C.
W4	10"	16'-0" MAX	#5	12" O.C.	#4	12" O.C.

1. PLACE (2) #4 HORIZ. BARS AT TOP OF WALL CONTINUOUS, TYP.
2. RECESS TOP OF WALL AT OPENINGS and POUR SLAB THROUGH, SEE DETAILS

★ THIS WALL REQUIRES (2) MATS OF REINFORCING (1) MAT 2" OFF EA. FACE AS SPECIFIED ABOVE

FOOTING SCHEDULE

MARK	WIDTH	LENGTH	DEPTH	CROSS-WISE No.	CROSS-WISE SIZE	CROSS-WISE LENGTH	CROSS-WISE SPACING	LENGTH-WISE No.	LENGTH-WISE SIZE	LENGTH-WISE LENGTH	LENGTH-WISE SPACING
CF24	24"	CONT.	12"	-	-	-	-	(3)	#4	CONT.	EQUAL
CF48	48"	CONT.	12"	-	#4	3'-6"	12" O.C.	(6)	#4	CONT.	EQUAL
F2	2'-0"	2'-0"	12"	(3)	#4	1'-6"	EQUAL	(3)	#4	1'-6"	EQUAL
F3	3'-0"	3'-0"	12"	(4)	#4	2'-6"	EQUAL	(4)	#4	2'-6"	EQUAL
F4	4'-0"	4'-0"	12"	(5)	#4	3'-6"	EQUAL	(5)	#4	3'-6"	EQUAL

FOUNDATION KEY PLAN

CONTINUOUS FOOTING DESIGN

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

Continuous wall load CF24: $w1 := 1800 \cdot plf$ WORST CASE CONTINUOUS FOOTING

Continous footing CF24:

$w := \frac{w1}{p}$ $w = 12.3 \cdot in$ 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:

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

Spread footing F3:

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

Spread footing F4:

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

ASCE
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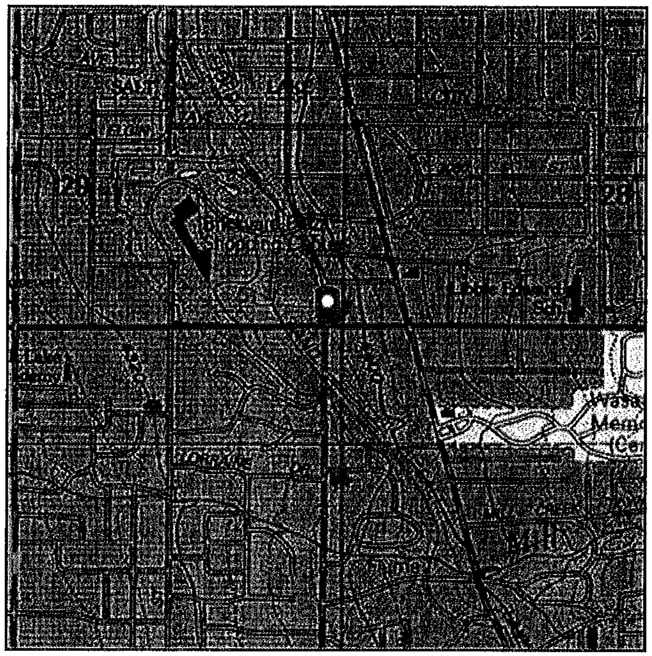
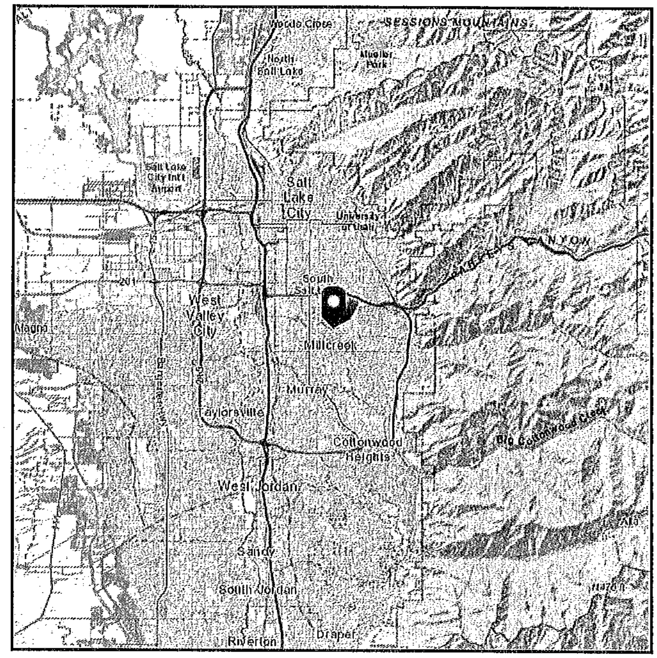
ASCE 7 Hazards Report

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





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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

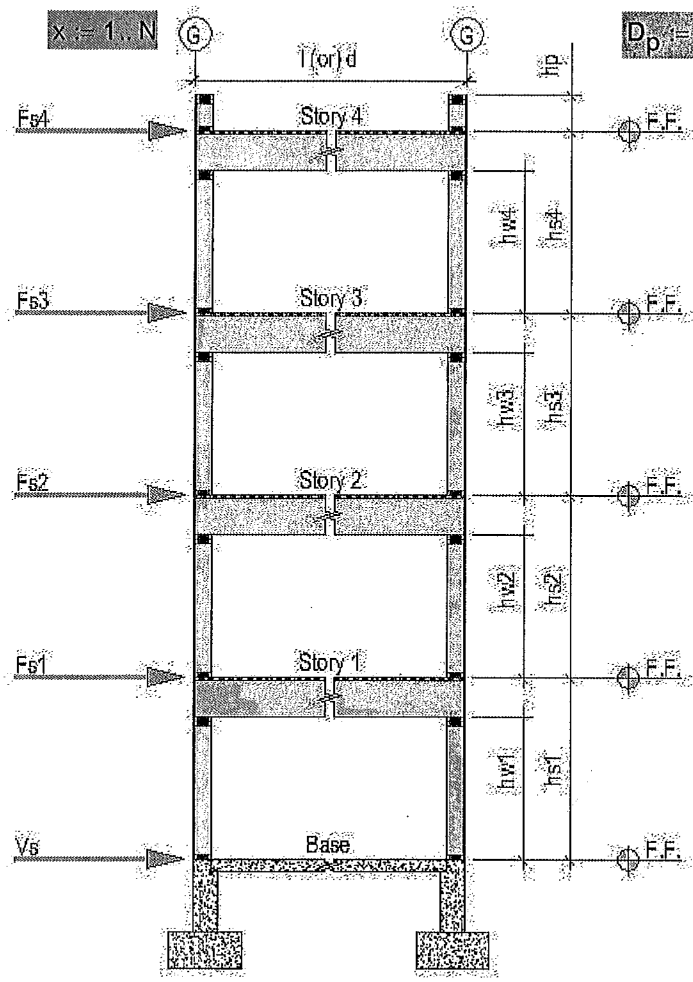
Seismic Base Shear

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

$N := 2$

$x := 1 .. N$



Parapet

$h_p := 0 \cdot ft$

$D_p := 0 \cdot psf$

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

$l_2 := 62 \cdot ft$

$d_2 := 40 \cdot ft$

$h_{s_2} := 10 \cdot ft$

$D_2 := 15 \cdot psf$ smc mass

$D_{w_2} := 15 \cdot psf$ avg

Story 1

$l_1 := 68 \cdot ft$

$d_1 := 30 \cdot ft$

$h_{s_1} := 14 \cdot ft$

$D_1 := 45 \cdot psf$ avg

$D_{w_1} := 60 \cdot psf$

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

(Amplify base shear at masonry shear wall lines by a ration of 5.0 / 6.5 per ASCE 7-16 12.2.3.3 Exception)

Determine the MCE SRA Parameters per Section 11.4:

Site Class:	$SC := \text{"D"}$	(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)	
$RC = \text{"D"}$	(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: $W := \sum w$

$W = 241.3 \cdot kip$

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)

$T = 0.325$

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)

$V_s = 41.6 \cdot kip$ STRENGTH

$\dfrac{V_s}{1.4} = 29.7 \cdot kip$ ALLOWABLE

Vertically Distribute the Seismic Base Shear per Section 12.8.3:

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

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

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

	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)	$F_{s_2} = 13.4 \cdot kip$	$0.71 F_{s_2} = 9.5 \cdot kip$	**USE 10 KIPS**
(Story 1)	$F_{s_1} = 28.2 \cdot kip$	$0.71 F_{s_1} = 20.0 \cdot kip$	**USE 20 KIPS**

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

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

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

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

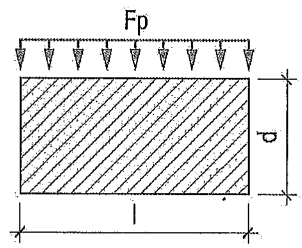
$$F_{psl_x} := \begin{cases} F_{pslmin_x} & \text{if } F_{psl_x} < F_{pslmin_x} \\ F_{pslmax_x} & \text{if } F_{psl_x} > F_{pslmax_x} \\ F_{psl_x} & \text{otherwise} \end{cases}$$

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

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

$F_{psl_2} = 192 \cdot plf$ (Story 2)

$F_{psl_1} = 524 \cdot plf$ (Story 1)



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

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

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

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

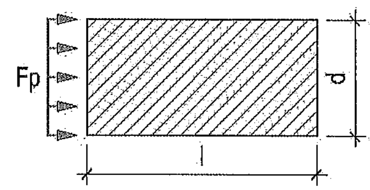
$$F_{psd_x} := \begin{cases} F_{psdmin_x} & \text{if } F_{psd_x} < F_{psdmin_x} \\ F_{psdmax_x} & \text{if } F_{psd_x} > F_{psdmax_x} \\ F_{psd_x} & \text{otherwise} \end{cases}$$

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

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

$F_{psd_2} = 276 \cdot plf$ (Story 2)

$F_{psd_1} = 907 \cdot plf$ (Story 1)



SHEAR WALLS - LINE 1

STORY 2

			PIERS	Length	Height	Tributary
# Piers in Shear Line:	$n2 := 2$	(n = 8 max)	1:	$l2_1 := 4 \cdot ft$	$h2_1 := 7 \cdot ft$	$t2_1 := 7 \cdot ft$
Story Shear:	$Fa_2 := 10 \cdot k$	(Allowable)	2:	$l2_2 := 4 \cdot ft$	$h2_2 := 7 \cdot ft$	$t2_2 := 7 \cdot ft$
Shear Attributed To Line:	$Va_2 := 5 \cdot k$	(Allowable)	3:	$l2_3 := 0 \cdot ft$	$h2_3 := 0 \cdot ft$	$t2_3 := 0 \cdot ft$
Story DL:	$DL_2 := 15 \cdot psf$		4:	$l2_4 := 0 \cdot ft$	$h2_4 := 0 \cdot ft$	$t2_4 := 0 \cdot ft$
Wall DL:	$DLw_2 := 20 \cdot psf$		5:	$l2_5 := 0 \cdot ft$	$h2_5 := 0 \cdot ft$	$t2_5 := 0 \cdot ft$
Redundancy	$\rho_2 := 1$		6:	$l2_6 := 0 \cdot ft$	$h2_6 := 0 \cdot ft$	$t2_6 := 0 \cdot ft$
			7:	$l2_7 := 0 \cdot ft$	$h2_7 := 0 \cdot ft$	$t2_7 := 0 \cdot ft$
			8:	$l2_8 := 0 \cdot ft$	$h2_8 := 0 \cdot ft$	$t2_8 := 0 \cdot ft$

SHEAR CALCULATIONS

Unit Shear (for walls): $$v_2 := \frac{\rho_2 \cdot Va_2}{\sum l2}$$

OVERTURNING CALCULATIONS $i2 := 1 .. n2$

Overturning Moment: $$Mo2_{i2} := v_2 \cdot h2_{i2} \cdot l2_{i2} \qquad Mo2 = \begin{pmatrix} 17.5 \\ 17.5 \end{pmatrix} \cdot k \cdot ft$$

Resisting Moment: $$Mr2_{i2} := 0.6 \cdot \left[\left[(DL_2 \cdot t2_{i2}) \cdot l2_{i2} \cdot \left(\frac{l2_{i2}}{2} \right) \right] + \left[(DLw_2 \cdot h2_{i2}) \cdot l2_{i2} \cdot \left(\frac{l2_{i2}}{2} \right) \right] \right]$$

Nominal Overturning: $$M2_{i2} := Mo2_{i2} - Mr2_{i2}$$

Tension at Pier Ends: $$T2_{i2} := \frac{M2_{i2}}{l2_{i2}}$$

STORY 1

			PIERS	Length	Height	Tributary
# Piers in Shear Line:	$n1 := 2$	(n = 8 max)	1:	$l1_1 := 11 \cdot ft$	$h1_1 := 14 \cdot ft$	$t1_1 := 2 \cdot ft$
Story Shear:	$Fa_1 := 20 \cdot k$	(Allowable)	2:	$l1_2 := 7 \cdot ft$	$h1_2 := 14 \cdot ft$	$t1_2 := 2 \cdot ft$
Shear Attributed To Line:	$Va_1 := 10 \cdot k$	(Allowable)	3:	$l1_3 := 0 \cdot ft$	$h1_3 := 0 \cdot ft$	$t1_3 := 0 \cdot ft$
Story DL:	$DL_1 := 30 \cdot psf$		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:	$Ls_1 := 17 \cdot ft$		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:	$l1_8 := 0 \cdot ft$	$h1_8 := 0 \cdot ft$	$t1_8 := 0 \cdot ft$

NOTE: TAKE 50% OF FORCE CONSERVATIVELY INTO MASONRY SHEAR WALLS

SHEAR CALCULATIONS

Unit Shear (for walls):

$$v_1 := \frac{(\rho_2 \cdot Va_2 + \rho_1 \cdot Va_1) \cdot 0.5}{\sum l1}$$

OVERTURNING CALCULATIONS $i1 := 1 .. n1$

Overturning Moment:

$$Mo1_{i1} := \left[\frac{(\rho_2 \cdot Va_2 + \rho_1 \cdot Va_1) \cdot 0.5 \cdot h1_{i1}}{\sum l1}\right] \cdot l1_{i1}$$

Resisting Moment:

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

Nominal Overturning:

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

Tension at Pier Ends:

$$T1_{i1} := \frac{M1_{i1}}{l1_{i1}}$$

ANCHOR BOLTS

Unit Shear (for bolts):

$$vb_1 := \frac{\left[\sum_{i=1}^{2} (\rho_i \cdot Va_i)\right]}{Ls_1}$$

1/2" bolt in 1 1/2" sill:

$$s_{0.5} := \frac{(650 \cdot lb) \cdot 1.6}{vb_1}$$

5/8" bolt in 1 1/2" sill:

$$s_{0.625} := \frac{(930 \cdot lb) \cdot 1.6}{vb_1}$$

SUMMARY, STORY 2

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

$ratio_{i2} := \frac{l2_{i2}}{h2_{i2}}$ $r2 := if(2 \cdot min(ratio) > 1.0, 1.0, 2 \cdot min(ratio))$ $r2 = 1$

Unit Shear

$\frac{v2}{r2} = 625 \cdot plf$

	Uplift	HOLD DOWN
Pier 1:	$T2_1 = 4081 \cdot lb$	
Pier 2:	$T2_2 = 4081 \cdot lb$	
Pier 3:	$T2_3 = \blacksquare \cdot lb$	
Pier 4:	$T2_4 = \blacksquare \cdot lb$	
Pier 5:	$T2_5 = \blacksquare \cdot lb$	
Pier 6:	$T2_6 = \blacksquare \cdot lb$	
Pier 7:	$T2_7 = \blacksquare \cdot lb$	
Pier 8:	$T2_8 = \blacksquare \cdot lb$	

SHEAR WALLS

Sheathing: 7/16", APA, Exp. 1
Blocking: All Panel Edges
Edge Nailing: 8d @ 2" o.c.
Field Nailing: 8d @ 12" o.c.

SUMMARY, STORY 1

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

$ratio_{i1} := \frac{l1_{i1}}{h1_{i1}}$ $r1 := if(2 \cdot min(ratio) > 1.0, 1.0, 2 \cdot min(ratio))$ $r1 = 1$

Unit Shear

$\frac{v1}{r1} = 417 \cdot plf$

	Uplift	HOLD DOWN
Pier 1:	$T1_1 = 4711 \cdot lb$	HDU8
Pier 2:	$T1_2 = 5119 \cdot lb$	HDU8
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:	$T1_8 = \blacksquare \cdot lb$	

SHEAR WALLS

Sheathing: 7/16", APA, Exp. 1
Blocking: All Panel Edges
Edge Nailing: 8d @ 2" o.c.
Field Nailing: 8d @ 12" o.c.

ANCHOR BOLTS	1/2" A.Bolts	5/8" A.Bolts
	$s_{0.5} = 14 \cdot in$	$s_{0.625} = 20 \cdot in$

USE:
5/8" dia. x 10" J-bolts
Spacing = 16" o.c.

SHEAR WALLS - LINE 2

STORY 2

# Piers in Shear Line:	$n2 := 2$	(n = 8 max)
Story Shear:	$Fa_2 := 10 \cdot k$	(Allowable)
Shear Attributed To Line:	$Va_2 := 5 \cdot k$	(Allowable)
Story DL:	$DL_2 := 15 \cdot psf$	
Wall DL:	$DLw_2 := 20 \cdot psf$	
Redundancy	$\rho_2 := 1$	

PIERS	Length	Height	Tributary
1:	$l2_1 := 4 \cdot ft$	$h2_1 := 7 \cdot ft$	$t2_1 := 7 \cdot ft$
2:	$l2_2 := 4 \cdot ft$	$h2_2 := 7 \cdot ft$	$t2_2 := 7 \cdot ft$
3:	$l2_3 := 0 \cdot ft$	$h2_3 := 0 \cdot ft$	$t2_3 := 0 \cdot ft$
4:	$l2_4 := 0 \cdot ft$	$h2_4 := 0 \cdot ft$	$t2_4 := 0 \cdot ft$
5:	$l2_5 := 0 \cdot ft$	$h2_5 := 0 \cdot ft$	$t2_5 := 0 \cdot ft$
6:	$l2_6 := 0 \cdot ft$	$h2_6 := 0 \cdot ft$	$t2_6 := 0 \cdot ft$
7:	$l2_7 := 0 \cdot ft$	$h2_7 := 0 \cdot ft$	$t2_7 := 0 \cdot ft$
8:	$l2_8 := 0 \cdot ft$	$h2_8 := 0 \cdot ft$	$t2_8 := 0 \cdot ft$

SHEAR CALCULATIONS

Unit Shear (for walls): $$v_2 := \frac{\rho_2 \cdot Va_2}{\sum l2}$$

OVERTURNING CALCULATIONS $i2 := 1 .. n2$

Overturning Moment: $Mo2_{i2} := v_2 \cdot h2_{i2} \cdot l2_{i2}$ $\quad Mo2 = \begin{pmatrix} 17.5 \\ 17.5 \end{pmatrix} \cdot k \cdot ft$

Resisting Moment: $$Mr2_{i2} := 0.6 \cdot \left[\left[\left(DL_2 \cdot t2_{i2} \right) \cdot l2_{i2} \cdot \left(\frac{l2_{i2}}{2} \right) \right] + \left[\left(DLw_2 \cdot h2_{i2} \right) \cdot l2_{i2} \cdot \left(\frac{l2_{i2}}{2} \right) \right] \right]$$

Nominal Overturning: $M2_{i2} := Mo2_{i2} - Mr2_{i2}$

Tension at Pier Ends: $$T2_{i2} := \frac{M2_{i2}}{l2_{i2}}$$

STORY 1

			PIERS	Length	Height	Tributary
# Piers in Shear Line:	$n1 := 4$	(n = 8 max)	1:	$l1_1 := 3.5 \cdot ft$	$h1_1 := 9 \cdot ft$	$t1_1 := 7 \cdot ft$
Story Shear:	$Fa_1 := 20 \cdot k$	(Allowable)	2:	$l1_2 := 4 \cdot ft$	$h1_2 := 9 \cdot ft$	$t1_2 := 7 \cdot ft$
Shear Attributed To Line:	$Va_1 := 10 \cdot k$	(Allowable)	3:	$l1_3 := 4 \cdot ft$	$h1_3 := 9 \cdot ft$	$t1_3 := 7 \cdot ft$
Story DL:	$DL_1 := 30 \cdot psf$		4:	$l1_4 := 6 \cdot ft$	$h1_4 := 9 \cdot ft$	$t1_4 := 7 \cdot ft$
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:	$Ls_1 := 17.5 \cdot ft$		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): $$v_1 := \frac{(\rho_2 \cdot Va_2 + \rho_1 \cdot Va_1)}{\sum l1}$$

OVERTURNING CALCULATIONS $i1 := 1 .. n1$

Overturning Moment: $$Mo1_{i1} := \left[\frac{(\rho_2 \cdot Va_2 + \rho_1 \cdot Va_1) \cdot h1_{i1}}{\sum l1}\right] \cdot l1_{i1}$$

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

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

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

ANCHOR BOLTS

Unit Shear (for bolts): $$vb_1 := \frac{\left[\sum_{i=1}^{2} (\rho_i \cdot Va_i)\right]}{Ls_1}$$

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

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

SUMMARY, STORY 2

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

$\text{ratio}_{i2} := \dfrac{l2_{i2}}{h2_{i2}}$ $\qquad r2 := \text{if}(2\cdot\min(\text{ratio}) > 1.0, 1.0, 2\cdot\min(\text{ratio}))$ $\qquad r2 = 1$

Unit Shear		Uplift	HOLD DOWN
$\dfrac{v2}{r2} = 625\cdot\text{plf}$	Pier 1:	$T2_1 = 4081\cdot\text{lb}$	
	Pier 2:	$T2_2 = 4081\cdot\text{lb}$	
SHEAR WALLS	Pier 3:	$T2_3 = \blacksquare\cdot\text{lb}$	
	Pier 4:	$T2_4 = \blacksquare\cdot\text{lb}$	
	Pier 5:	$T2_5 = \blacksquare\cdot\text{lb}$	
	Pier 6:	$T2_6 = \blacksquare\cdot\text{lb}$	
	Pier 7:	$T2_7 = \blacksquare\cdot\text{lb}$	
	Pier 8:	$T2_8 = \blacksquare\cdot\text{lb}$	

Sheathing: 7/16", APA, Exp. 1
Blocking: All Panel Edges
Edge Nailing: 8d @ 2" o.c.
Field Nailing: 8d @ 12" o.c.

SUMMARY, STORY 1

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

$\text{ratio}_{i1} := \dfrac{l1_{i1}}{h1_{i1}}$ $\qquad r1 := \text{if}(2\cdot\min(\text{ratio}) > 1.0, 1.0, 2\cdot\min(\text{ratio}))$ $\qquad r1 = 0.78$

Unit Shear		Uplift	HOLD DOWN
$\dfrac{v1}{r1} = 1102\cdot\text{plf}$	Pier 1:	$T1_1 = 7305\cdot\text{lb}$	HDU8
	Pier 2:	$T1_2 = 7246\cdot\text{lb}$	HDU8
SHEAR WALLS	Pier 3:	$T1_3 = 7246\cdot\text{lb}$	HDU8
	Pier 4:	$T1_4 = 7012\cdot\text{lb}$	HDU8
	Pier 5:	$T1_5 = \blacksquare\cdot\text{lb}$	
	Pier 6:	$T1_6 = \blacksquare\cdot\text{lb}$	
	Pier 7:	$T1_7 = \blacksquare\cdot\text{lb}$	
	Pier 8:	$T1_8 = \blacksquare\cdot\text{lb}$	

Sheathing: 7/16", APA, Exp. 1
Blocking: All Panel Edges
Edge Nailing: 8d @ 3" o.c.
Field Nailing: 8d @ 12" o.c.

SHEATH BOTH SIDES OF WALL

ANCHOR BOLTS	1/2" A.Bolts	5/8" A.Bolts
	$s_{0.5} = 15\cdot\text{in}$	$s_{0.625} = 21\cdot\text{in}$

USE:
5/8" dia. x 10" J-bolts
Spacing = 16" o.c.

SHEAR WALLS - LINE A.5

STORY 1

			PIERS	Length	Height	Tributary
# Piers in Shear Line:	$n1 := 2$	(n = 8 max)	1:	$l1_1 := 10 \cdot ft$	$h1_1 := 14 \cdot ft$	$t1_1 := 11 \cdot ft$
Story Shear:	$Fa_1 := 20 \cdot k$	(Allowable)	2:	$l1_2 := 10 \cdot ft$	$h1_2 := 14 \cdot ft$	$t1_2 := 11 \cdot ft$
Shear Attributed To Line:	$Va_1 := 8.6k$	(Allowable)	3:	$l1_3 := 0 \cdot ft$	$h1_3 := 0 \cdot ft$	$t1_3 := 0 \cdot ft$
Story DL:	$DL_1 := 60 \cdot psf$		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:	$Ls_1 := 20 \cdot ft$		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: $Mr1_{i1} := 0.6 \cdot \left[\left[\left(DL_1 \cdot t1_{i1} \right) \cdot l1_{i1} \cdot \left(\dfrac{l1_{i1}}{2} \right) \right] + \left[\left(DLw_1 \cdot h1_{i1} \right) \cdot l1_{i1} \cdot \left(\dfrac{l1_{i1}}{2} \right) \right] \right]$

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

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

$r := \text{if}(2 \cdot \min(ratio) > 1.0, 1.0, 2 \cdot \min(ratio))$ $\quad r = 1$

Reduced Unit Shear:

$\frac{v1}{r} = 430 \cdot plf$

SHEAR WALLS

Sheathing:	7/16", APA, Exp. 1
Blocking:	All Panel Edges
Edge Nailing:	8d @ 3" o.c.
Field Nailing:	8d @ 12" o.c.

	Uplift	HOLD DOWN
Pier 1:	$T1_1 = 3200 \cdot lb$	HDU5
Pier 2:	$T1_2 = 3200 \cdot lb$	HDU5
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
	$s_{0.5} = 29 \cdot in$	$s_{0.625} = 42 \cdot in$

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

SHEAR WALLS - GRID B.5

STORY 2

			PIERS	Length	Height	Tributary
# Piers in Shear Line:	$n2 := 1$	(n = 8 max)	1:	$l2_1 := 24 \cdot ft$	$h2_1 := 10 \cdot ft$	$t2_1 := 1 \cdot ft$
Story Shear:	$Fa_2 := 10 \cdot k$	(Allowable)	2:	$l2_2 := 0 \cdot ft$	$h2_2 := 0 \cdot ft$	$t2_2 := 0 \cdot ft$
Shear Attributed To Line:	$Va_2 := 7 \cdot k$	(Allowable)	3:	$l2_3 := 0 \cdot ft$	$h2_3 := 0 \cdot ft$	$t2_3 := 0 \cdot ft$
Story DL:	$DL_2 := 15 \cdot psf$		4:	$l2_4 := 0 \cdot ft$	$h2_4 := 0 \cdot ft$	$t2_4 := 0 \cdot ft$
Wall DL:	$DLw_2 := 15 \cdot psf$		5:	$l2_5 := 0 \cdot ft$	$h2_5 := 0 \cdot ft$	$t2_5 := 0 \cdot ft$
Redundancy	$\rho_2 := 1$		6:	$l2_6 := 0 \cdot ft$	$h2_6 := 0 \cdot ft$	$t2_6 := 0 \cdot ft$
			7:	$l2_7 := 0 \cdot ft$	$h2_7 := 0 \cdot ft$	$t2_7 := 0 \cdot ft$
			8:	$l2_8 := 0 \cdot ft$	$h2_8 := 0 \cdot ft$	$t2_8 := 0 \cdot ft$

SHEAR CALCULATIONS

Unit Shear (for walls): $$v_2 := \frac{\rho_2 \cdot Va_2}{\sum l2}$$

OVERTURNING CALCULATIONS $i2 := 1 .. n2$

Overturning Moment: $$Mo2_{i2} := v_2 \cdot h2_{i2} \cdot l2_{i2}$$

Resisting Moment: $$Mr2_{i2} := 0.6 \cdot \left[\left[\left(DL_2 \cdot t2_{i2} \right) \cdot l2_{i2} \cdot \left(\frac{l2_{i2}}{2} \right) \right] + \left[\left(DLw_2 \cdot h2_{i2} \right) \cdot l2_{i2} \cdot \left(\frac{l2_{i2}}{2} \right) \right] \right]$$

Nominal Overturning: $$M2_{i2} := Mo2_{i2} - Mr2_{i2}$$

Tension at Pier Ends: $$T2_{i2} := \frac{M2_{i2}}{l2_{i2}}$$

STORY 1

			PIERS	Length	Height	Tributary
# Piers in Shear Line:	$n1 := 1$	(n = 8 max)	1:	$l1_1 := 28\cdot ft$	$h1_1 := 14\cdot ft$	$t1_1 := 11\cdot ft$
Story Shear:	$Fa_1 := 20\cdot k$	(Allowable)	2:	$l1_2 := 0\cdot ft$	$h1_2 := 0\cdot ft$	$t1_2 := 0\cdot ft$
Shear Attributed To Line:	$Va_1 := 8\cdot k$	(Allowable)	3:	$l1_3 := 0\cdot ft$	$h1_3 := 0\cdot ft$	$t1_3 := 0\cdot ft$
Story DL:	$DL_1 := 30\cdot psf$		4:	$l1_4 := 0\cdot ft$	$h1_4 := 0\cdot ft$	$t1_4 := 0\cdot ft$
Wall DL:	$DLw_1 := 15\cdot psf$		5:	$l1_5 := 0\cdot ft$	$h1_5 := 0\cdot ft$	$t1_5 := 0\cdot ft$
Sill Plate Length:	$Ls_1 := 28\cdot ft$		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): $$v_1 := \frac{\left(\rho_2\cdot Va_2 + \rho_1\cdot Va_1\right)}{\sum l1}$$

OVERTURNING CALCULATIONS $i1 := 1..n1$

Overturning Moment: $$Mo1_{i1} := \left[\frac{\rho_2\cdot Va_2\cdot\left(h2_{i1} + h1_{i1}\right) + \rho_1\cdot Va_1\cdot h1_{i1}}{\sum l1}\right]\cdot l1_{i1}$$

Resisting Moment: $$Mr1_{i1} := 0.6\cdot\left[\left[\left[\left(DL_1 + DL_2\right)\cdot t1_{i1}\right]\cdot l1_{i1}\cdot\left(\frac{l1_{i1}}{2}\right)\right] + \left[\left(DLw_2\cdot h2_{i1} + DLw_1\cdot h1_{i1}\right)\cdot l1_{i1}\cdot\left(\frac{l1_{i1}}{2}\right)\right]\right]$$

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

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

ANCHOR BOLTS

Unit Shear (for bolts): $$vb_1 := \frac{\left[\sum_{i=1}^{2}\left(\rho_i\cdot Va_i\right)\right]}{Ls_1}$$

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

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

SUMMARY, STORY 2

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

$$ratio_{i2} := \frac{l2_{i2}}{h2_{i2}} \qquad r2 := \text{if}(2\cdot\min(ratio) > 1.0, 1.0, 2\cdot\min(ratio)) \qquad r2 = 1$$

Unit Shear

$$\frac{v2}{r2} = 292\cdot plf$$

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:	$T2_1 = 1729\cdot lb$	MST48
Pier 2:	$T2_2 = \blacksquare\cdot lb$	
Pier 3:	$T2_3 = \blacksquare\cdot lb$	
Pier 4:	$T2_4 = \blacksquare\cdot lb$	
Pier 5:	$T2_5 = \blacksquare\cdot lb$	
Pier 6:	$T2_6 = \blacksquare\cdot lb$	
Pier 7:	$T2_7 = \blacksquare\cdot lb$	
Pier 8:	$T2_8 = \blacksquare\cdot lb$	

SUMMARY, STORY 1

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

$$ratio_{i1} := \frac{l1_{i1}}{h1_{i1}} \qquad r1 := \text{if}(2\cdot\min(ratio) > 1.0, 1.0, 2\cdot\min(ratio)) \qquad r1 = 1$$

Unit Shear

$$\frac{v1}{r1} = 536\cdot plf$$

SHEAR WALLS

Sheathing:	7/16", APA, Exp. 1
Blocking:	All Panel Edges
Edge Nailing:	8d @ 2" o.c.
Field Nailing:	8d @ 12" o.c.

	Uplift	HOLD DOWN
Pier 1:	$T1_1 = 2818\cdot lb$	HDU5
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
	$s_{0.5} = 23\cdot in$	$s_{0.625} = 33\cdot in$

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

SHEAR WALLS - GRID D

STORY 2

			PIERS	Length	Height	Tributary
# Piers in Shear Line:	$n2 := 1$	(n = 8 max)	1:	$l2_1 := 30 \cdot ft$	$h2_1 := 10 \cdot ft$	$t2_1 := 8 \cdot ft$
Story Shear:	$Fa_2 := 10 \cdot k$	(Allowable)	2:	$l2_2 := 0 \cdot ft$	$h2_2 := 0 \cdot ft$	$t2_2 := 0 \cdot ft$
Shear Attributed To Line:	$Va_2 := 3 \cdot k$	(Allowable)	3:	$l2_3 := 0 \cdot ft$	$h2_3 := 0 \cdot ft$	$t2_3 := 0 \cdot ft$
Story DL:	$DL_2 := 15 \cdot psf$		4:	$l2_4 := 0 \cdot ft$	$h2_4 := 0 \cdot ft$	$t2_4 := 0 \cdot ft$
Wall DL:	$DLw_2 := 15 \cdot psf$		5:	$l2_5 := 0 \cdot ft$	$h2_5 := 0 \cdot ft$	$t2_5 := 0 \cdot ft$
Redundancy	$\rho_2 := 1$		6:	$l2_6 := 0 \cdot ft$	$h2_6 := 0 \cdot ft$	$t2_6 := 0 \cdot ft$
			7:	$l2_7 := 0 \cdot ft$	$h2_7 := 0 \cdot ft$	$t2_7 := 0 \cdot ft$
			8:	$l2_8 := 0 \cdot ft$	$h2_8 := 0 \cdot ft$	$t2_8 := 0 \cdot ft$

SHEAR CALCULATIONS

Unit Shear (for walls): $$v_2 := \frac{\rho_2 \cdot Va_2}{\sum l2}$$

OVERTURNING CALCULATIONS $i2 := 1 .. n2$

Overturning Moment: $$Mo2_{i2} := v_2 \cdot h2_{i2} \cdot l2_{i2}$$

Resisting Moment: $$Mr2_{i2} := 0.6 \cdot \left[\left[\left(DL_2 \cdot t2_{i2}\right) \cdot l2_{i2} \cdot \left(\frac{l2_{i2}}{2}\right)\right] + \left[\left(DLw_2 \cdot h2_{i2}\right) \cdot l2_{i2} \cdot \left(\frac{l2_{i2}}{2}\right)\right]\right]$$

Nominal Overturning: $$M2_{i2} := Mo2_{i2} - Mr2_{i2}$$

Tension at Pier Ends: $$T2_{i2} := \frac{M2_{i2}}{l2_{i2}}$$

MASONRY SHEAR WALL / FOUNDATION AT LOWER LEVEL
HAS SUFFICIENT CAPACITY BY INSPECTION

SUMMARY, STORY 2

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

$ratio_{i2} := \frac{l2_{i2}}{h2_{i2}}$ $r2 := \text{if}(2 \cdot \min(ratio) > 1.0, 1.0, 2 \cdot \min(ratio))$ $r2 = 1$

Unit Shear

$\frac{v2}{r2} = 100 \cdot plf$

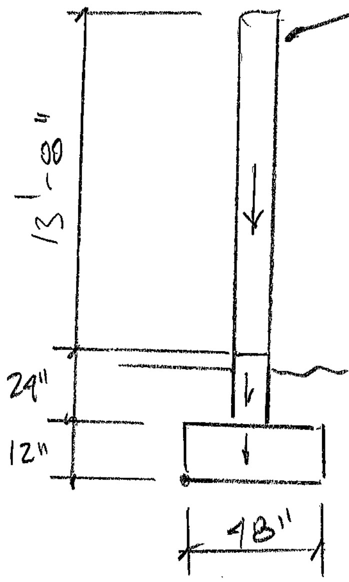
SHEAR WALLS

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:	$T2_1 = -1430 \cdot lb$	NONE REQUIRED
Pier 2:	$T2_2 = \blacksquare \cdot lb$	
Pier 3:	$T2_3 = \blacksquare \cdot lb$	
Pier 4:	$T2_4 = \blacksquare \cdot lb$	
Pier 5:	$T2_5 = \blacksquare \cdot lb$	
Pier 6:	$T2_6 = \blacksquare \cdot lb$	
Pier 7:	$T2_7 = \blacksquare \cdot lb$	
Pier 8:	$T2_8 = \blacksquare \cdot lb$	

MASONRY SHEAR WALL / FOUNDATION AT LOWER LEVEL
HAS SUFFICIENT CAPACITY BY INSPECTION

MECH. YARD WALL



SOLID GROUTED 8" CMU = 85 PSF = W_p

$$F_p = \frac{0.4\, a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)} \cancel{\left(1 + 2\frac{z}{h}\right)} W_p$$

$a_p = 1.0$ $\quad$ $I_p = 1.0$

$R_p = 2.5$

$S_{DS} = 1.12$

$$F_p = \frac{0.4\,(1.0)\,(1.12)}{\left(\frac{2.5}{1.0}\right)} (85\text{ PSF}) = 15.2\text{ PSF}$$

$$F_{p\,MIN.} = 0.3\,(1.12)(1.0)(85\text{ PSF}) = \underline{28.6\text{ PSF}}$$

M_u AT MASONRY BASE = 28.6 PSF $(13.67')\left(\frac{13.67'}{2}\right) = 2672^{\text{LB-FT}}/\text{FT}$

8" CMU w/ #5 AT 16" o.c. VERTICAL G.F. $3600^{\text{LB-FT}}/\text{FT}$.
<u>O.K.</u>

DYNAMIC STRUCTURES
1887 NORTH 1120 WEST PROVO, UTAH 84604
PH: (801) 356-1140 FAX: (801) 356-0001

PROJECT:

SUBJECT:

DATE:

FOOTING

ALLOWABLE OVERTURNING AT FOOTING.

$$M_a = 0.7\,(28.6\,PSF)\,(13.67')\left(3' + \frac{13.67'}{2}\right) = 2692\ \text{lb-FT/FT}$$

MASONRY WALL WEIGHT = 85 PSF (13.67') = 1162 LBS/FT

CONCRETE WALL WEIGHT = 100 PSF (2') = 200 LBS/FT

CONCRETE FOOTING WEIGHT = 1'(4')(150 PCF) = 600 LBS/FT

SOIL WEIGHT = 1.5'(4' − .67')(110 PCF) = 549 LBS/FT

RIGHTING MOMENT = $0.6\,(1162 + 200 + 600 + 549)\left(\frac{4'}{2}\right)$

= 3013 lb-FT/FT

o.k.

SOIL PRESSURE

$$\frac{P}{A} + \frac{M}{S} = \frac{1162+200}{4'(1')} + \frac{2692}{\frac{1}{6}(1')(4')^2} =$$

$$= 341\ PSF + 1010\ PSF = 1351\ PSF \quad \text{o.k.}$$

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