Project #19337310

DYNAMIC STRUCTURES

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ANETH BUS GARAGE
FOUNDATION AND MEZZANINE

ANETH, UTAH

PROFESSIONAL STRUCTURAL ENGINEER
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9/16/19

DIVISION OF FACILITIES CONSTRUCTION AND MANAGEMENT
REVIEWED FOR CODE COMPLIANCE
SIGNATURE:
DATE: 05/05/2020

September 16, 2019

Service Prepared for:

CURTIS MINER ARCHITECTURE

PROJECT: ANETH BUS GARAGE
FOUNDATION AND MEZZANINE
ANETH, UTAH

CLIENT: CURTIS MINER ARCHITECTURE

SCOPE: PROVIDE STRUCTURAL ANALYSIS AND STRUCTURAL PLANS FOR A STORAGE MEZZANINE AND A FOUNDATION FOR A PRE-FABRICATED METAL BUILDING (METAL BUILDING SHELL BY OTHERS)

CODE CRITERIA: 2018 IBC
SEISMIC DESIGN CATEGORY B
SOIL: 2,000 psf (assumed)

DESIGN LOADS:

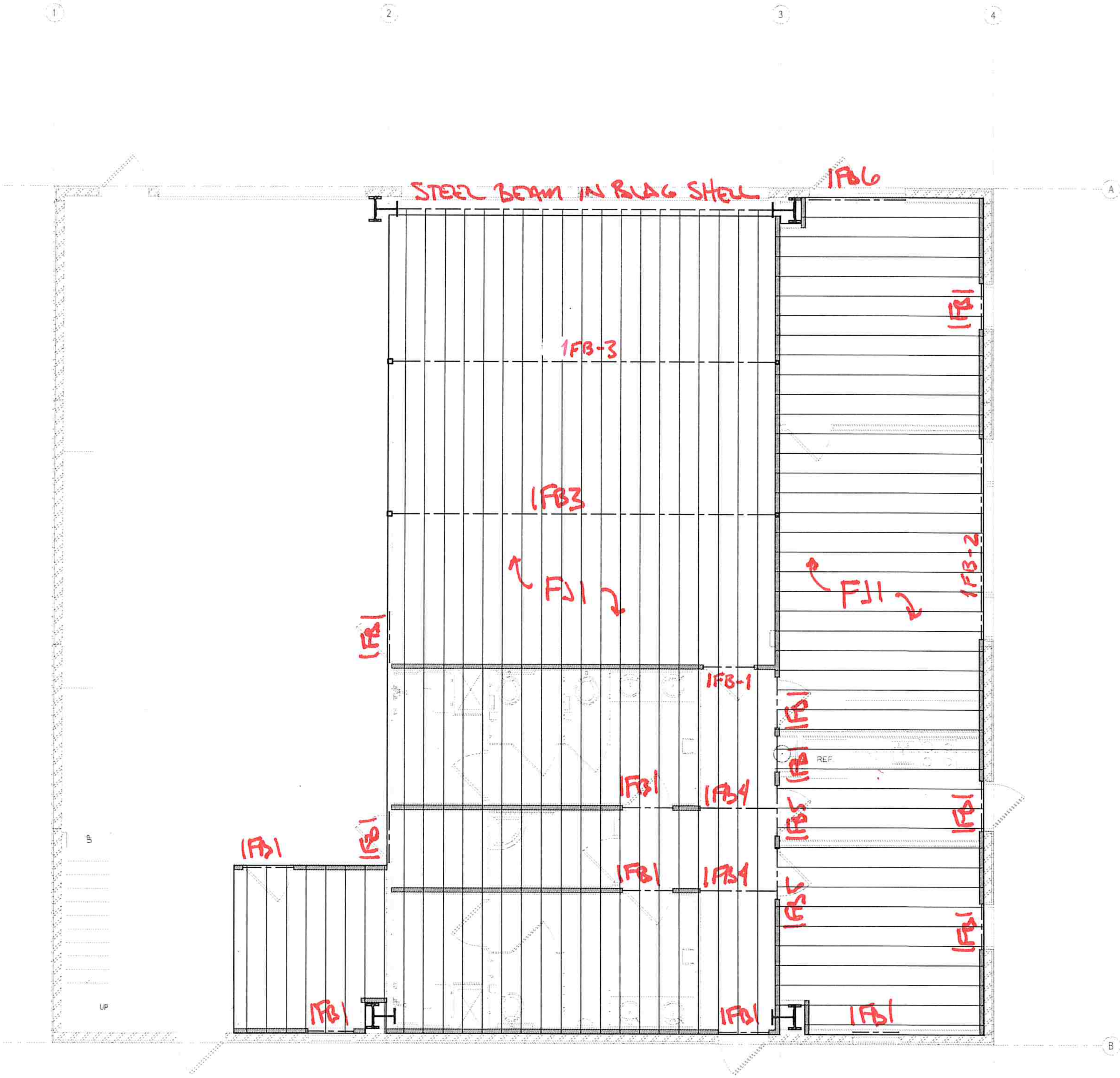
Floor Live load (storage):	**=125.0 psf**

FLOOR DEAD LOADS:

Flooring	=2.50 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:	**=15.0 psf**

INTERIOR WALL DEAD LOADS:

Wall Sheathing	=1.20 psf
Studs	=2.10 psf
Insulation	=1.50 psf
Gypsum board.	=2.80 psf
Misc	=2.40 psf
Total wall dead load:	**=10.0 psf**



MEZZANINE FRAMING

KEY PLAN

ROOF SNOW LOADS- BALANCED, UNBALANCED, SEISMIC

Elevation:	$A := 4.5$	feet/1000'
Base Ground Snow Elevation:	$Ao := 6.5$	feet/1000'
Base Ground Snow Load:	$Po := 43$	psf
Change in Ground Snow Load with Elevation Change:	$S := 63$	psf/100'

$$Ao := \begin{cases} Ao & \text{if } A > Ao \\ A & \text{otherwise} \end{cases}$$

Ground Snow Load: $Pg := \left[Po^2 + S^2\cdot(A - Ao)^2\right]^{.5}$ $\quad Pg = 43$ psf

Temperature Coeffecient:	$Ctl := 1.0$	$Ctf := 1.2$
Pitch Coeffecient:	$Cf := 1.0$	
Other Coeffecient:	$Cv := 1.0$	
Snow Importance Factor:	$Is := 1.0$	

Balanced Roof Snow Load (for livable structures):	$Pf := Pg\cdot 0.7\cdot Ctl\cdot Cf\cdot Cv\cdot Is$	$Pf = 30.1$	psf
Balanced Roof Snow Load without Importance factor:	$Pfi := Pg\cdot 0.7\cdot Ctl\cdot Cf\cdot Cv$	$Pfi = 30.1$	psf
Unbalanced Roof Snow Load:	$Pf := Pg\cdot 0.7\cdot Ctl\cdot Cf\cdot Cv\cdot Is\cdot 1.5$	$Pf = 45.1$	psf
Balanced Roof Snow Load (for frozen structures):	$Pf := Pg\cdot 0.7\cdot Ctf\cdot Cf\cdot Cv\cdot Is$	$Pf = 36.1$	psf

Seismic Snow Load:

$$Ws := \begin{cases} [0.2 + 0.025(A - 5)]\cdot Pfi & \text{if } Pfi > 35 \\ 0.0 & \text{otherwise} \end{cases}$$

$Ws = 0$ psf

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FLOOR FRAMING
FJ-1

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

Spacing: $s := 16 \cdot in$

Loads per lineal foot: $wll := 125 \cdot psf \cdot s$

$wdl := 15 \cdot psf \cdot s$ $\qquad w := wdl + wll$

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

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

$p := pll + pdl$

Calculate the bending moment: $$Mx := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8} \right) \qquad Mx = 4573.3 \cdot ft \cdot lb$$

Calculate the shear: $$Vx := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2} \right) \qquad Vx = 1306.7 \cdot lb$$

Joist Series: $Series := 360$

Joist Depth: $d := 11.875 \cdot in$

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

$$i := if(d > 9.5 \cdot in, if(d > 11.875 \cdot in, if(d > 14 \cdot in, if(d > 16in, if(d > 18in, 0, 5), 4), 3), 2), 1)$$

USE: 11 7/8" TJI 360 @ 16" O.C.

	Joist Capacities	>	Required Capacities
Moment Capacity:	$M_{i,j} = 6180 \cdot ft \cdot lb$	>	$Mx = 4573.3 \cdot ft \cdot lb$
Maximum Reaction:	$V_{i,j} = 1705 \cdot lb$	>	$Vx = 1306.7 \cdot lb$
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: $\frac{l}{240} = 0.7 \cdot in$ $\quad > \quad$ $y = 0.467 \cdot in$

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: $\frac{l}{360} = 0.467 \cdot in$ $\quad > \quad$ $y = 0.417 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FIRST FLOOR FRAMING
1FB-1

Length:	$l := 3 \cdot ft$	$a := 0 \cdot ft$	(larger) (WORST CASE)
Concentrated load:	$p := 0 \cdot lb$	$c := l - a$	
Weight per lineal foot:	$w := (15 + 125) \cdot 10 \cdot plf$		
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 1575 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 2100 \cdot lb$

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) $C_{\cdot} := if(d > 10 \cdot in, if(d > 12 \cdot in, 0.9, 1.0), C$ $C_2 := C_1$ $C_1 = 1.1$

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: $Sr := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$

Actual section modulus: $Sa := \frac{b}{6} \cdot d^2$ $Sa = 42.8 \cdot in^3$ $\geq$ $Sr = 19.1 \cdot in^3$

Required area: $Ar := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$

Actual area: $Aa := b \cdot d$ $Aa = 27.7 \cdot in^2$ $\geq$ $Ar = 8.5 \cdot in^2$

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

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $\frac{l}{240} = 0.15 \cdot in$ $\geq$ $y = 0.01 \cdot in$

Allowable deflection: $\frac{l}{360} = 0.1 \cdot in$ $\geq$ $y \cdot \frac{45}{65} = 0.01 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FIRST FLOOR FRAMING
1FB-2

Length:	$l := 14 \cdot ft$	$a := 0 \cdot ft$	(larger)
Concentrated load:	$p := 0 \cdot lb$	$c := l - a$	
Weight per lineal foot:	$w := (15 + 125) \cdot 7 \cdot plf$		
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 24010 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 6860 \cdot lb$

Use: **(3) 11 7/8" LVL** $b := 5.25 \cdot in$ $d := 11.875 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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

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: $Sr := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$

Actual section modulus: $Sa := \frac{b}{6} \cdot d^2$ $Sa = 123.4 \cdot in^3$ > $Sr = 110.8 \cdot in^3$

Required area: $Ar := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$

Actual area: $Aa := b \cdot d$ $Aa = 62.3 \cdot in^2$ > $Ar = 31 \cdot in^2$

Check deflection: $I := \frac{b}{12} \cdot d^3$ $I = 732.6 \cdot in^4$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $\frac{l}{240} = 0.7 \cdot in$ > $y = 0.58 \cdot in$

Allowable deflection: $\frac{l}{360} = 0.47 \cdot in$ > $y \cdot \frac{45}{65} = 0.4 \cdot in$

Gravity Beam Design

RAM SBeam v5.01
ANITH BUS BUILDING
1FB-3

07/15/19 11:28:25

STEEL CODE: AISC 360-05 LRFD — SAME RESULTS FROM 360-16 FOR BRACED TOP FLANGE

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

Maximum Depth Limitation specified = 11.00 in
Beam Size (Optimum) = W10X77 Fy = 50.0 ksi
Total Beam Length (ft) = 26.00
Mp (kip-ft) = 406.67
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL
1	0.000	0.077	0.000
	26.000	0.077	0.000
2	0.000	0.154	1.280
	26.000	0.154	1.280

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 30.23 kips 1.00Vn = 168.54 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	196.5	13.0	0.0	1.00	0.90	366.00
Controlling		1.2DL+1.6LL	196.5	13.0	0.0	1.00	0.90	366.00

REACTIONS (kips):

	Left	Right
DL reaction	3.00	3.00
Max +LL reaction	16.64	16.64
Max +total reaction (factored)	30.23	30.23

DEFLECTIONS:

Dead load (in)	at	13.00 ft	= -0.180	L/D =	1734
Live load (in)	at	13.00 ft	= -0.997	L/D =	313
Net Total load (in)	at	13.00 ft	= -1.177	L/D =	265

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FIRST FLOOR FRAMING
1FB-4

Length: $l := 5 \cdot ft$ $\quad a := 0 \cdot ft$ (larger)
Concentrated load: $p := 0 \cdot lb$ $\quad c := l - a$
Weight per lineal foot: $w := (15 + 125) \cdot 8 \cdot plf$
Material: $i := LVL$

Allowable bending stress Fb: $Fb_i = 2600 \cdot psi$ $\quad$ 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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $\quad M = 3500 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $\quad V = 2800 \cdot lb$

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

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

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

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

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: $Sr := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$

Actual section modulus: $Sa := \frac{b}{6} \cdot d^2$ $\quad Sa = 82.3 \cdot in^3 \ \ge \ Sr = 16.2 \cdot in^3$

Required area: $Ar := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$

Actual area: $Aa := b \cdot d$ $\quad Aa = 41.6 \cdot in^2 \ \ge \ Ar = 8.9 \cdot in^2$

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

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $\quad \frac{l}{240} = 0.25 \cdot in \ \ge \ y = 0.02 \cdot in$

Allowable deflection:

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FIRST FLOOR FRAMING
1FB-5

Length: $l := 3 \cdot ft$ $a := 1.5 \cdot ft$ (larger)

Concentrated load: $p := 2800 \cdot lb$ $c := l - a$

Weight per lineal foot: $w := (15 + 125) \cdot 1 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 2257.5 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 1610 \cdot lb$

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: $Sr := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$

Actual section modulus: $Sa := \frac{b}{6} \cdot d^2$ $Sa = 52.6 \cdot in^3$ $\ge$ $Sr = 10.4 \cdot in^3$

Required area: $Ar := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$

Actual area: $Aa := b \cdot d$ $Aa = 33.2 \cdot in^2$ $\ge$ $Ar = 7.9 \cdot in^2$

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

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $\frac{l}{240} = 0.15 \cdot in$ $\ge$ $y = 0.01 \cdot in$

Allowable deflection:

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

FIRST FLOOR FRAMING
1FB-6

Length:	$l := 7 \cdot ft$	$a := 0 \cdot ft$	(larger)
Concentrated load:	$p := 0 \cdot lb$	$c := l - a$	
Weight per lineal foot:	$w := (15 + 125) \cdot 1 \cdot plf$		
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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 857.5 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 490 \cdot lb$

Use: (3) 2 X 10

$b := 4.5 \cdot in$ $d := 9.25 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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: $Sr := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$

Actual section modulus: $Sa := \frac{b}{6} \cdot d^2$ $Sa = 64.2 \cdot in^3$ $\ge$ $Sr = 10.4 \cdot in^3$

Required area: $Ar := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$

Actual area: $Aa := b \cdot d$ $Aa = 41.6 \cdot in^2$ $\ge$ $Ar = 3.2 \cdot in^2$

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

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $\frac{l}{240} = 0.35 \cdot in$ $\ge$ $y = 0.02 \cdot in$

Allowable deflection:

Seismic Base Shear (MEZZANINE)

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

$h_p := 0 \cdot ft$

$D_p := 0 \cdot psf$

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

$l_2 := 0 \cdot ft$

$d_2 := 0 \cdot ft$

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

$D_2 := (0 + 0) \cdot psf$

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

Story 1

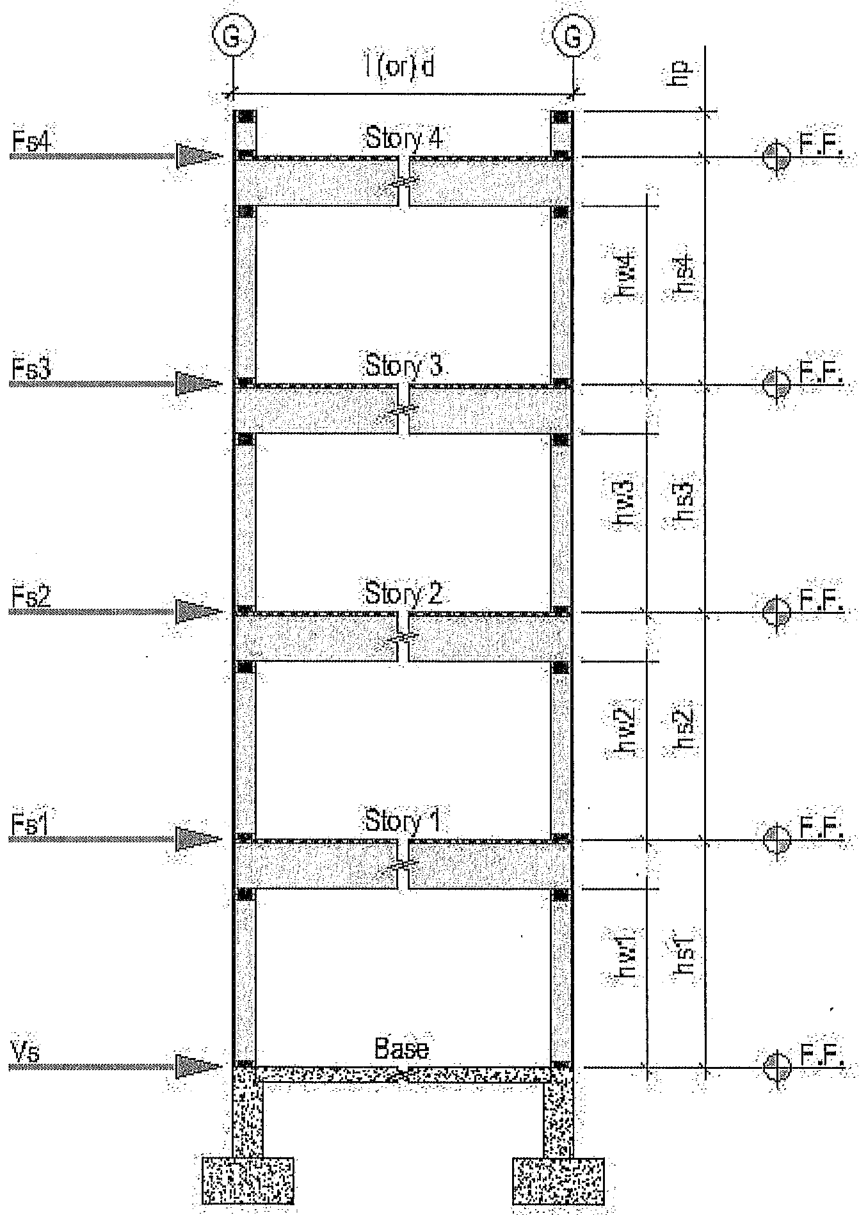
$l_1 := 58 \cdot ft$

$d_1 := 43 \cdot ft$

$h_{s_1} := 12 \cdot ft$

$D_1 := 25 \cdot psf$

$D_{w_1} := 10 \cdot psf$



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**

$S_s := 0.165$ $S_1 := 0.052$ (Figures 22-1 thr. 22-14)

$S_{ds} := 0.176$ $S_{d1} := 0.084$

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)

$RC = "C"$ (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: $W := \sum w$

$W = 74.5 \cdot kip$

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)

$T = 0.193$

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:

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

$$C_{smin} := \begin{cases} \dfrac{0.5 \cdot S_1 \cdot I_e}{R} & \text{if } S_1 > 0.60 \\ \max(0.044 \cdot S_{ds} \cdot I_e, 0.01) & \text{otherwise} \end{cases} \qquad \text{(Equations 12.8-5 \& 12.8-6)}$$

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

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

$C_s = 0.027$

Calculate the Seismic Base Shear per Section 12.8.1:

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

$V_s = 2.0 \cdot kip$ STRENGTH

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

Vertically Distribute the Seismic Base Shear per Section 12.8.3:

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

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

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

	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)	$F_{s_2} = \blacksquare \cdot kip$	$0.71 F_{s_2} = \blacksquare \cdot kip$
(Story 1)	$F_{s_1} = 2.0 \cdot kip$	$0.71 F_{s_1} = 1.4 \cdot kip$

USE 1.5 KIPS

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

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

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

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

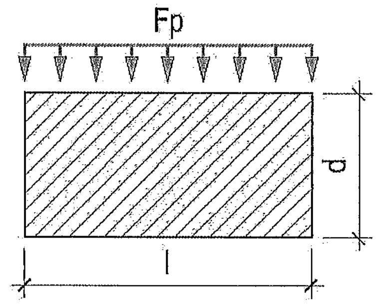
$$F_{psl_x} := \begin{cases} F_{pslmin_x} & \text{if } F_{psl_x} < F_{pslmin_x} \\ F_{pslmax_x} & \text{if } F_{psl_x} > F_{pslmax_x} \\ F_{psl_x} & \text{otherwise} \end{cases}$$

$F_{psl_4} =$ [redacted] $\cdot plf$ (Story 4)

$F_{psl_3} =$ [redacted] $\cdot plf$ (Story 3)

$F_{psl_2} =$ [redacted] $\cdot plf$ (Story 2)

$F_{psl_1} = 42.1 \cdot plf$ (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)

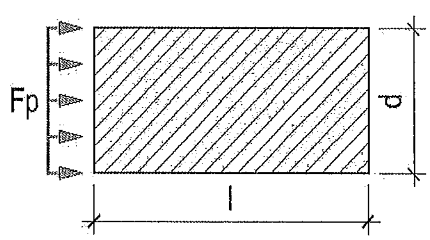
$$F_{psd_x} := \begin{cases} F_{psdmin_x} & \text{if } F_{psd_x} < F_{psdmin_x} \\ F_{psdmax_x} & \text{if } F_{psd_x} > F_{psdmax_x} \\ F_{psd_x} & \text{otherwise} \end{cases}$$

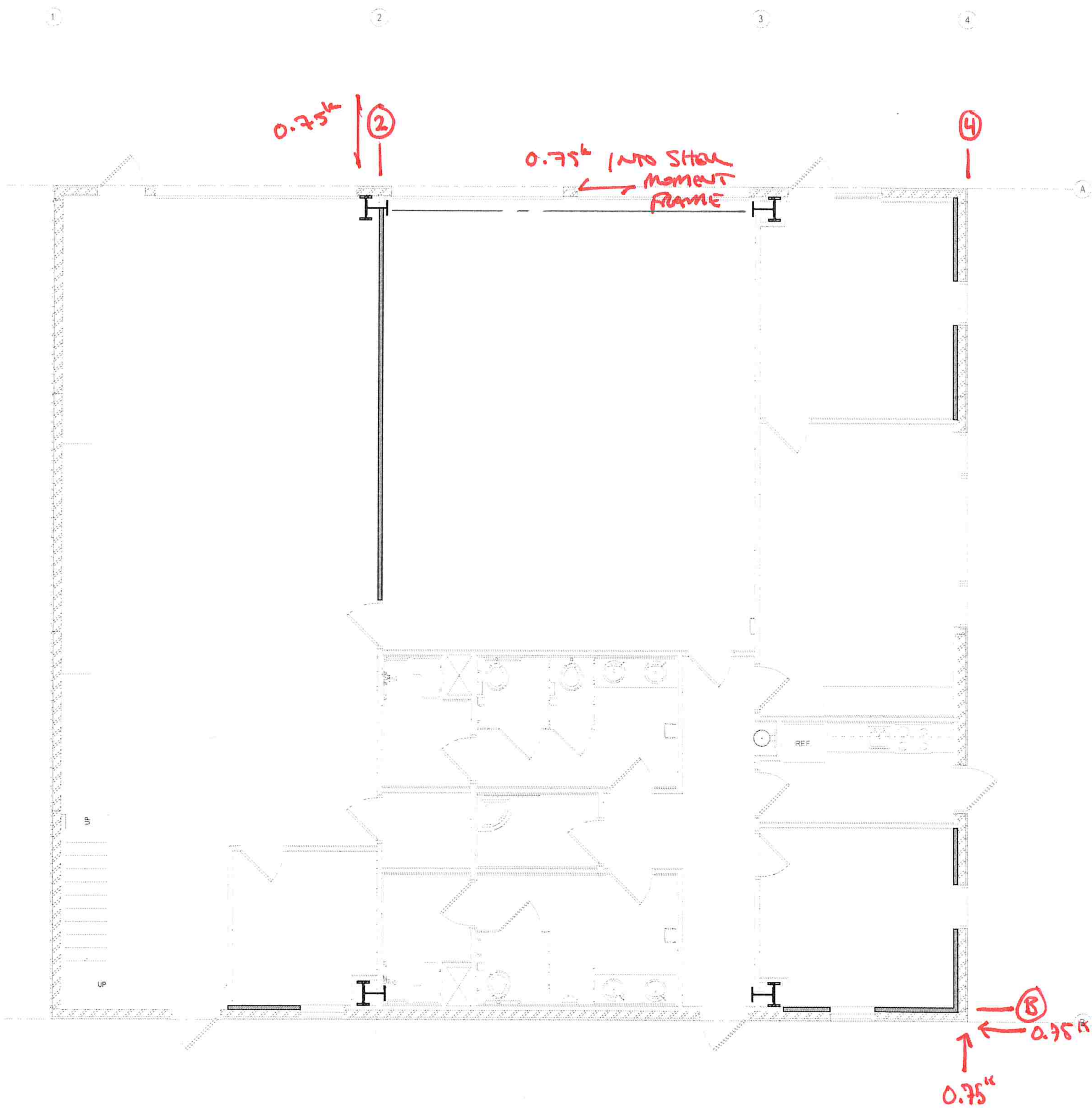
$F_{psd_4} =$ [redacted] $\cdot plf$ (Story 4)

$F_{psd_3} =$ [redacted] $\cdot plf$ (Story 3)

$F_{psd_2} =$ [redacted] $\cdot plf$ (Story 2)

$F_{psd_1} = 55.3 \cdot plf$ (Story 1)




1
2
3
4
A
B
0.75k
2
4
0.75k INTO SHEAR MOMENT FRAME
REF
UP
UP
0.75k
0.75k

SHEAR WALLS - LINE 2

STORY 1

# Piers in Shear Line:	$n1 := 1$	(n = 8 max)
Story Shear:	$Fa_1 := 1.5 \cdot k$	(Allowable)
Shear Attributed To Line:	$Va_1 := 0.75k$	(Allowable)
Story DL:	$DL_1 := 15 \cdot psf$	
Wall DL:	$DLw_1 := 10 \cdot psf$	
Sill Plate Length:	$Ls_1 := 26.5 \cdot ft$	
Redundancy	$\rho_1 := 1$	

PIERS	Length	Height	Tributary
1:	$l1_1 := 26.5 \cdot ft$	$h1_1 := 12 \cdot ft$	$t1_1 := 1 \cdot ft$
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:	$l1_8 := 0 \cdot ft$	$h1_8 := 0 \cdot ft$	$t1_8 := 0 \cdot ft$

SUBTRACT OPENINGS BELOW

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

$$\text{ratio}_{i1} := \frac{l1_{i1}}{h1_{i1}}$$

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

SUBTRACT OPENINGS

$$t := \frac{(39 - 2 \cdot 0)}{39} \qquad t = 1$$

Reduced Unit Shear:

$$\frac{v_1}{r \cdot t} = 28 \cdot \text{plf}$$

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:	$T1_1 = -734 \cdot \text{lb}$	NONE REQUIRED
Pier 2:	$T1_2 = \blacksquare \cdot \text{lb}$	
Pier 3:	$T1_3 = \blacksquare \cdot \text{lb}$	
Pier 4:	$T1_4 = \blacksquare \cdot \text{lb}$	
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:	$T1_8 = \blacksquare \cdot \text{lb}$	

ANCHOR BOLTS	1/2" A.Bolts	5/8" A.Bolts
	$s_{0.5} = 441 \cdot \text{in}$	$s_{0.625} = 631 \cdot \text{in}$

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

SHEAR WALLS - LINE 4

STORY 1

# Piers in Shear Line:	$n1 := 2$	(n = 8 max)
Story Shear:	$Fa_1 := 1.5 \cdot k$	(Allowable)
Shear Attributed To Line:	$Va_1 := 0.75k$	(Allowable)
Story DL:	$DL_1 := 15 \cdot psf$	
Wall DL:	$DLw_1 := 10 \cdot psf$	
Sill Plate Length:	$Ls_1 := 27 \cdot ft$	
Redundancy	$\rho_1 := 1$	

PIERS	Length	Height	Tributary
1:	$l1_1 := 15 \cdot ft$	$h1_1 := 12 \cdot ft$	$t1_1 := 7 \cdot ft$
2:	$l1_2 := 12 \cdot ft$	$h1_2 := 12 \cdot ft$	$t1_2 := 7 \cdot ft$
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:	$l1_8 := 0 \cdot ft$	$h1_8 := 0 \cdot ft$	$t1_8 := 0 \cdot ft$

SUBTRACT OPENINGS BELOW

SHEAR CALCULATIONS

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

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

ANCHOR BOLTS

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

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 1

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

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

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

SUBTRACT OPENINGS

$$t := \frac{(27 - 2 \cdot 3)}{27} \qquad t = 0.78$$

Reduced Unit Shear:

$$\frac{v_1}{r \cdot t} = 36 \cdot plf$$

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:	$T1_1 = -679 \cdot lb$	NONE REQUIRED
Pier 2:	$T1_2 = -477 \cdot lb$	NONE REQUIRED
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
	$s_{0.5} = 449 \cdot in$	$s_{0.625} = 643 \cdot in$

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

SHEAR WALLS - LINE B

STORY 1

			PIERS	Length	Height	Tributary
# Piers in Shear Line:	$n1 := 2$	(n = 8 max)	1:	$l1_1 := 4.5 \cdot ft$	$h1_1 := 12 \cdot ft$	$t1_1 := 5.5 \cdot ft$
Story Shear:	$Fa_1 := 1.5 \cdot k$	(Allowable)	2:	$l1_2 := 11.5 \cdot ft$	$h1_2 := 12 \cdot ft$	$t1_2 := 1 \cdot ft$
Shear Attributed To Line:	$Va_1 := 0.75k$	(Allowable)	3:	$l1_3 := 0 \cdot ft$	$h1_3 := 0 \cdot ft$	$t1_3 := 0 \cdot ft$
Story DL:	$DL_1 := 15 \cdot psf$		4:	$l1_4 := 0 \cdot ft$	$h1_4 := 0 \cdot ft$	$t1_4 := 0 \cdot ft$
Wal DL:	$DLw_1 := 10 \cdot psf$		5:	$l1_5 := 0 \cdot ft$	$h1_5 := 0 \cdot ft$	$t1_5 := 0 \cdot ft$
Sill Plate Length:	$Ls_1 := 16 \cdot ft$		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:	$l1_8 := 0 \cdot ft$	$h1_8 := 0 \cdot ft$	$t1_8 := 0 \cdot ft$

SUBTRACT OPENINGS BELOW

SHEAR CALCULATIONS

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

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

ANCHOR BOLTS

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

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 1

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

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

$r := if(2 \cdot min(ratio) > 1.0, 1.0, 2 \cdot min(ratio))$ $\quad r = 0.75$

SUBTRACT OPENINGS

$$t := \frac{(16 - 1 \cdot 3)}{16} \qquad t = 0.81$$

Reduced Unit Shear:

$$\frac{v_1}{r \cdot t} = 77 \cdot plf$$

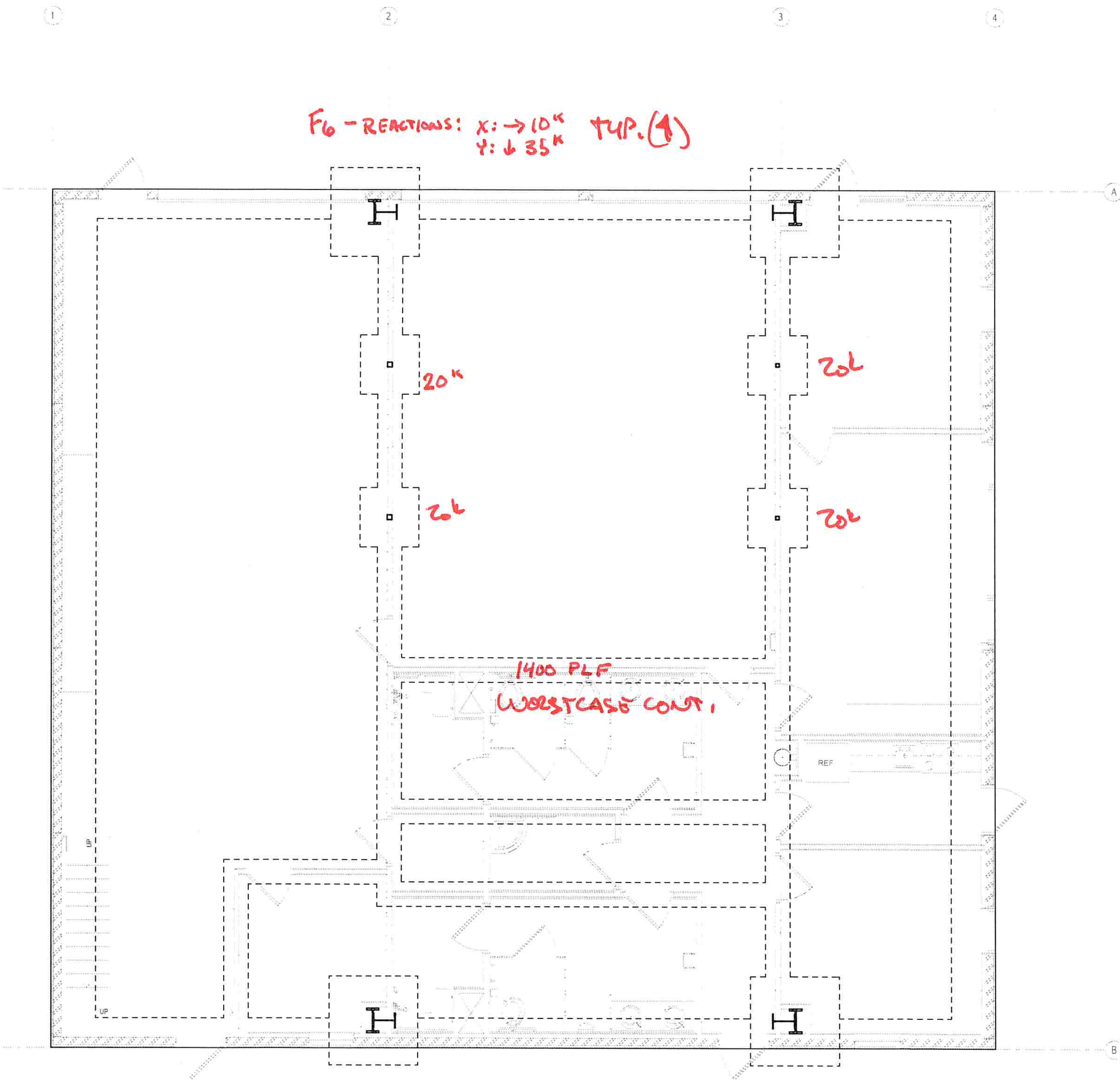
SHEAR WALLS

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:	$T1_1 = 289 \cdot lb$	STHD14 OR HDU2
Pier 2:	$T1_2 = 97 \cdot lb$	STHD14 or HDU2
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
	$s_{0.5} = 266 \cdot in$	$s_{0.625} = 381 \cdot in$

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



FOUNDATION KEY PLAN

CONTINUOUS FOOTING DESIGN **1500 psf BEARING PRESSURE**

Soil bearing pressure (assumed): $p := 1500 \cdot psf$

Maximum continuous loading for CF18: $w1 := 2250 \cdot plf$

Maximum continuous loading for CF20: $w2 := 2500 \cdot plf$

Maximum continuous loading for CF24: $w3 := 3000 \cdot plf$

Maximum continuous loading for CF30: $w4 := 3750 \cdot plf$

Maximum continuous loading for CF36: $w5 := 4500 \cdot plf$

Continous footing CF18:

$w := \frac{w1}{p}$ $w = 18 \cdot in$

USE: 18" x 10" x cont. w/(2) #4 cont.

Allowable point load on continuous footing CF18:

$P := w \cdot 1.5 \cdot w \cdot p$ $P = 5 \cdot k$

Point loads greater than 5 kips require spot footings

Continous footing CF20:

$w := \frac{w2}{p}$ $w = 20 \cdot in$

USE: 20" x 10" x cont. w/(2) #4 cont.

Allowable point load on continuous footing CF20:

$P := w \cdot 1.5 \cdot w \cdot p$ $P = 6 \cdot k$

Point loads greater than 6 kips require spot footings

Continous footing CF24:

$w := \frac{w3}{p}$ $w = 24 \cdot in$

USE: 24" x 10" x cont. w/(3) #4 cont.

Allowable point load on continuous footing CF24:

$P := w \cdot 1.5 \cdot w \cdot p$ $P = 9 \cdot k$

Point loads greater than 9 kips require spot footings

Continous footing CF30:

$w := \frac{w4}{p}$ $w = 30 \cdot in$

USE: 30" x 10" x cont. w/(3) #4 cont.

Allowable point load on continuous footing CF30:

$P := w \cdot 1.5 \cdot w \cdot p$ $P = 14 \cdot k$

Point loads greater than 14 kips require spot footings

Continous footing CF36:

$w := \frac{w5}{p}$ $w = 36 \cdot in$

USE: 36" x 10" x cont. w/(4) #4 cont.

Allowable point load on continuous footing CF36:

$P := w \cdot 1.5 \cdot w \cdot p$ $P = 20 \cdot k$

Point loads greater than 20 kips require spot footings

SPOT FOOTING DESIGN **1500 psf BEARING PRESSURE**

Soil bearing pressure (assumed):	$p := 1500 \cdot psf$
Maximum point load for spread footing F2:	$P2 := 6000 \cdot lb$
Maximum point load for spread footing F3:	$P3 := 13500 \cdot lb$
Maximum point load for spread footing F4:	$P4 := 24000 \cdot lb$
Maximum point load for spread footing F5:	$P5 := 37500 \cdot lb$
Maximum point load for spread footing F6:	$P6 := 54000 \cdot lb$

Spread footing F2:

$w := \sqrt{\frac{P2}{p}}$ $w = 2 \cdot ft$ **USE: 2'-0" x 2'-0" x 12" w/ (3) #4 each way**

Spread footing F3:

$w := \sqrt{\frac{P3}{p}}$ $w = 3 \cdot ft$ **USE: 3'-0" x 3'-0" x 12" w/ (4) #4 each way**

Spread footing F4:

$w := \sqrt{\frac{P4}{p}}$ $w = 4 \cdot ft$ **USE: 4'-0" x 4'-0" x 12" w/ (5) #4 each way**

Spread footing F5:

$w := \sqrt{\frac{P5}{p}}$ $w = 5 \cdot ft$ **USE: 5'-0" x 5'-0" x 12" w/ (6) #5 each way**

Spread footing F6:

$w := \sqrt{\frac{P6}{p}}$ $w = 6 \cdot ft$ **USE: 6'-0" x 6'-0" x 12" w/ (7) #5 each way**

ESTIMATED LOADS, WILL BE VERIFIED W/ ENGINEERING RESULTS FROM METAL BUILDING SUPPLIER

MAIN FRAME FOOTINGS
WORST CASE "Y" REACTION

Design of Square Conccrete Footing:

Service Level Point Load: $P := 35 \cdot k$

Service Level Moment: $M := 10 \cdot k \cdot 1ft$

Bearing Pressure: $ps := 1500 \cdot psf$

Wind/Seismic Bearing Pressure: $psa := 1.33 \cdot ps$ $\quad psa = 1995 \cdot psf$

Overturning Moment: $OTM := 1.0 \cdot M$

$OTM = 10 \cdot k \cdot ft$

Soil Properties:

Density: $\gamma s := 110 \cdot pcf$

Soil over Footing: $ds := 6 \cdot in$

Assumed Footing Properties:

Density: $\gamma c := 150 \cdot pcf$

Assumed footing width (perp to mom.): $b := 6 \cdot ft$ **USE 6' X 6' X 12"**

Assumed footing length (parrallel to mom.): $l := 6 \cdot ft$

Assumed footing thickness: $dc := 12 \cdot in$

Righting Moment:

Soil:

$$Ms := \gamma s \cdot ds \cdot b \cdot l \cdot \frac{b}{2}$$

$$Ms = 5.94 \cdot k \cdot ft$$

Footing:

$$Mf := \gamma c \cdot dc \cdot b \cdot l \cdot \frac{b}{2}$$

$$Mf = 16.2 \cdot k \cdot ft$$

$$RM := 0.6(Ms + Mf)$$

Verify: $RM = 13.3 \cdot k \cdot ft \quad > \quad OTM = 10 \cdot k \cdot ft$

Soil Bearing Pressure:

Toe: $$pt := \frac{P}{b \cdot l} + \frac{M}{\frac{1}{6} \cdot b \cdot l^2}$$ $psa = 1995 \cdot psf \quad > \quad pt = 1250 \cdot psf$

Heel: $$ph := \frac{P}{b \cdot l} - \frac{M}{\frac{1}{6} \cdot b \cdot l^2}$$ $psa = 1995 \cdot psf \quad > \quad ph = 694 \cdot psf$

Check Footing Depth for Punching Shear:

Base plate width: $c := 12 \cdot in$ Compressive Strength of Concrete: $fc' := 3000 \cdot psi$

Area of column: $Ag := c^2$ Yield strength of Steel: $fy := 60ksi$

$d := (dc - 3.5 \cdot in)$ $\quad d = 8.5 \cdot in$

Factored Point Load for Design: $\quad Pu := 1.5P \quad Pu = 52.5 \cdot k$

Ultimate Soil Pressure: $\quad pnet := \dfrac{Pu}{b^2} \quad pnet = 1458 \cdot psf$

Depth Determined by shear:
Two way action:

$area := b^2 - (c + d)^2 \quad area = 33.082\, ft^2$

$Vup := pnet \cdot area \quad Vup = 48.2 \cdot k$

ratio of long to short side: $\quad \beta c := 1$

perimeter of critical section: $\quad bo := (c + d) \cdot 4 \quad bo = 6.833\, ft$

for $\beta c < 2$ and $bo/d < 20$ for a four sided critical section,

$Vc := 4 \cdot \sqrt{fc' \cdot psi} \cdot bo \cdot d \quad \phi Vc := 0.85 \cdot Vc$

$\phi Vc = 129.8 \cdot k \quad > \quad Vup = 48.2 \cdot k$

Depth Determined by shear:
One way action:

$area := b \cdot \left(\dfrac{b}{2} - \dfrac{c}{2} - d \right) \quad area = 10.75\, ft^2$

$Vuw := pnet \cdot area \quad Vuw = 15.7 \cdot k$

$Vc := 2\sqrt{fc' \cdot psi} \cdot b \cdot d \quad \phi Vc := 0.85 \cdot Vc$

$\phi Vc = 57 \cdot k \quad > \quad Vuw = 15.7 \cdot k$ Assumed Depth: OK

Design for bending moment strength:

$Mu := \dfrac{1}{2} \cdot pnet \cdot b \cdot \left(\dfrac{b}{2} - \dfrac{c}{2} \right)^2 \quad Mu = 27.3 \cdot k \cdot ft$

required Rn: $\quad Rn := \dfrac{Mu}{0.9 \cdot b \cdot d^2} \quad Rn = 70.085 \cdot psi$

$m := \dfrac{fy}{0.85 \cdot fc'}$

$\rho := \dfrac{1}{m} \cdot \left(1 - \sqrt{1 - \dfrac{2 \cdot m \cdot Rn}{fy}} \right) \quad \rho = 0.0012$

required As: $\quad As := \rho \cdot b \cdot d \quad As = 0.725 \cdot in^2$

minimum As: $\quad Asmin := 0.002 \cdot b \cdot d \quad Asmin = 1.224 \cdot in^2$

$Asreq := if(As < Asmin, Asmin, As)$

$Asreq = 1.22 \cdot in^2$

Use: **(7) #5 bars each way**

$As := 7 \cdot .31 \cdot in^2 \quad As = 2.17 \cdot in^2$

$$a := \frac{As \cdot fy}{0.85 \cdot fc' \cdot b} \qquad a = 0.709 \cdot in$$

$$\phi Mn := 0.90 \cdot (As \cdot fy) \left(d - \frac{a}{2} \right)$$

$$\phi Mn = 79.5 \cdot k \cdot ft \qquad > \qquad Mu = 27.3 \cdot k \cdot ft$$