DYNAMIC STRUCTURES
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Structural Calculations for:

HORIZON NEIGHBORHOOD CABINS AT SUMMIT POWDER MOUNTAIN
1500+ MODEL
EDEN, UTAH

PROFESSIONAL STRUCTURAL ENGINEER
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7/28/17

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Service Prepared for:

SIGNATURE BUILDERS

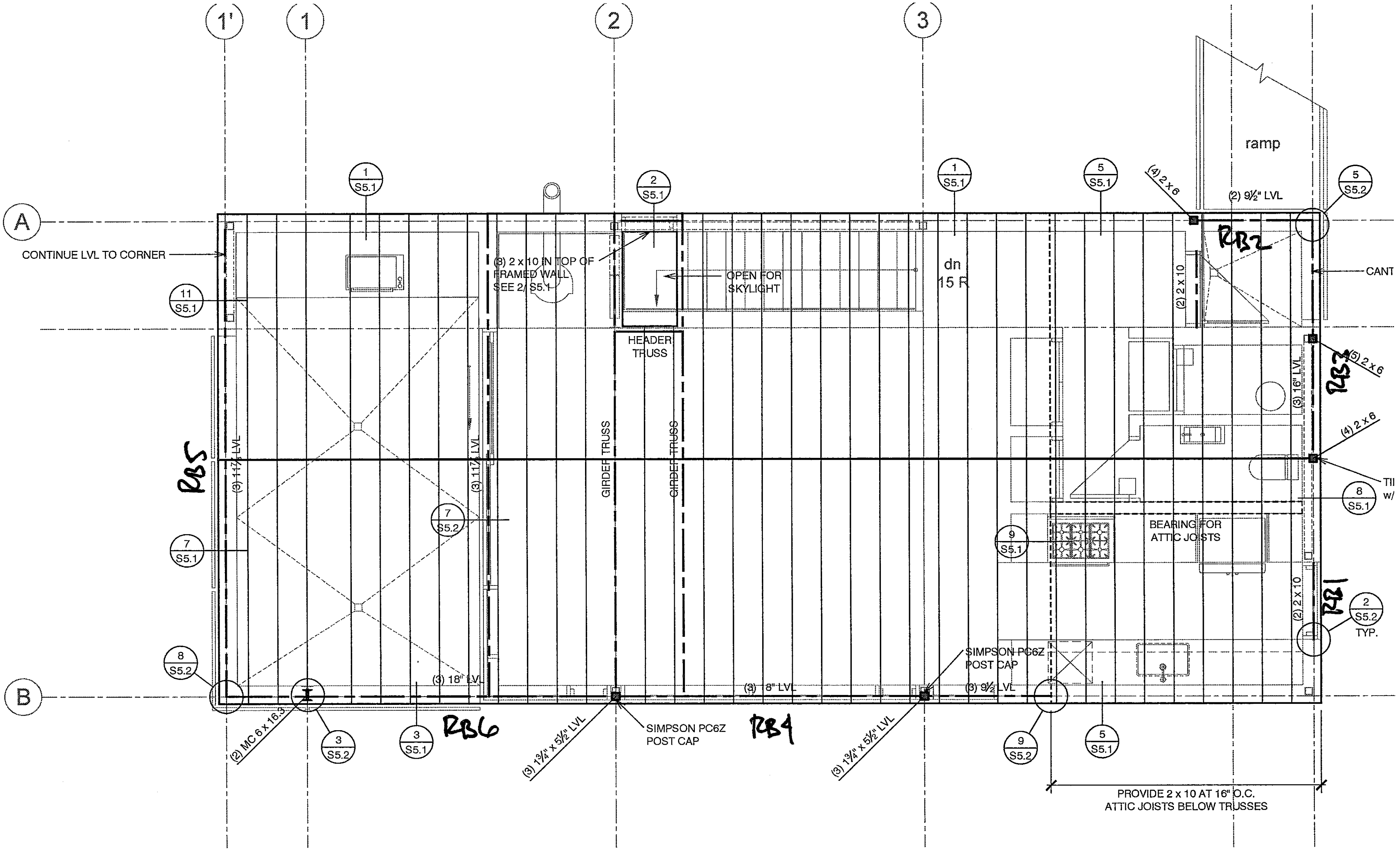
PROJECT:	HORIZON NEIGHBORHOOD CABINS AT SUMMIT POWDER MOUNTAIN 1500+ MODEL EDEN, UTAH
CLIENT:	SIGNATURE BUILDERS
SCOPE:	PROVIDE STRUCTURAL ANALYSIS, STRUCTURAL PLANS AND STRUCTURAL DETAIL SHEETS FOR A MOUNTAIN CABIN
CODE CRITERIA:	2015 IBC SEISMIC DESIGN CATEGORY D WIND: 115 MPH EXP. B (STRENGTH) SNOW: Pg = 274 PSF EXP 0.7 Pf = 192 PSF SOIL: 1500 psf

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)	2500 psi
(CONSTRUCTION)	SEE PLAN NOTES
REINFORCING STEEL:	GRADE 60
WOOD:	
STRUCTURAL MEMBERS	DF #2
TIMBERS:	DF #1

DESIGN LOADS:

Roof Snow Load:	**=192.0 psf**
Roof Dead Load:	**=15.0 psf**
Floor Live load:	**=40.0 psf**
Floor Dead Load (w/ 3" Concrete):	**=50.0 psf**
Seismic Design Category:	**=D**
Design Wind:	**=115 mph Exp. B**



ROOF FRAMING KEY PLAN

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB1

Length: $l := 4 \cdot ft$ $a := 0 \cdot ft$ (larger)

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

Weight per lineal foot: $w := (192 + 15)1 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 414 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 414 \cdot lb$

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 $C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1}$ $C_3 := if(C_3 > 1, 1, C_3)$ $C_3 = 1$

(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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 5 \cdot in^3$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 2.1 \cdot in^2$

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: $\frac{l}{240} = 0.2 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 3.766 \times 10^{-3} \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB2

Length: $l := 5.5\cdot ft$ $a := 0\cdot ft$ (larger)

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

Weight per lineal foot: $w := (192 + 15)11\cdot plf$

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: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\underline{M = 8609.9\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\underline{V = 6261.8\cdot lb}$

Use: **(2) 1 3/4 X 9 1/2 LVL**

$b := 3.5\cdot in$ $d := 9.5\cdot in$ $t := if(d \le 12\cdot in, 12\cdot in, d)$

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\underline{S = 39.7\cdot in^3}$

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

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\underline{A = 23.5\cdot in^2}$

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

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

Allowable deflection: $\frac{l}{240} = 0.275\cdot in$

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $y = 0.094\cdot in$

BEAM OVERHANGING ONE SUPPORT - CONCENTRATED LOAD AT OVERHANG

ROOF FRAMING
RB3

Simple length: $l := 5.5 \cdot ft$

Cantilever length: $a := 5.5 \cdot ft$ Cd = load duration factor: $Cd := 1.0$

Uniform load: $w := (192 + 15) \cdot 1 \cdot plf$ Cr = repetitive use factor: $Cr := 1.0$

Point load at cantilever: $P := 6300 \cdot lb$

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: $E_i = 2000000 \cdot psi$

Determine reactions:

$$R1 := \frac{w}{2 \cdot l} \cdot \left(l^2 - a^2\right) - \frac{P \cdot a}{l} \qquad R1 = -6300 \cdot lb$$

$$R2 := \frac{w}{2 \cdot l} \cdot (l + a)^2 + \frac{P}{l} \cdot (l + a) \qquad R2 = 14877 \cdot lb$$

Determine shear forces:

$$V1 := R1 \qquad V2 := w \cdot a + P \qquad V1 = -6300 \cdot lb \qquad V2 = 7438.5 \cdot lb$$

$$V3 := \frac{w}{2 \cdot l} \cdot \left(l^2 + a^2\right) + \frac{P \cdot a}{l} \qquad V3 = 7438.5 \cdot lb$$

Determine max locations:

$$x := \frac{l}{2} \cdot \left[1 - \left(\frac{a^2}{l^2}\right)\right] \qquad x = 0 \cdot ft$$

Determine moments:
(between supports)

$$M1 := \frac{w}{8 \cdot l^2} \cdot (l + a)^2 \cdot (l - a)^2 - \frac{P \cdot a \cdot x}{l} \qquad M1 = 0 \cdot ft \cdot lb$$

$$M1 := |M1|$$

(at cantilever
end)

$$M2 := \frac{w \cdot a^2}{2} + P \cdot a \qquad M2 = 37780.9 \cdot ft \cdot lb$$

$$M := if(M1 > M2, M1, M2) \qquad M = 37780.9 \cdot ft \cdot lb$$

$$V := if(V1 > V2, if(V1 > V3, V1, V3), if(V2 > V3, V2, \qquad V = 7438.5 \cdot lb$$

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)

$$C_1 := if\left(d > 10 \cdot in, if(d > 12 \cdot in, 0.9, 1.0), (\right. \quad C_2 := C_1 \qquad C_1 = 0.9$$

Cv = volume factor
(glu-laminated lumber only)

$$C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1} \qquad C_3 := if\left(C_3 > 1, 1, C_3\right) \quad C_3 = 1$$

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: $Sr := \frac{M}{(1\cdot Fb)_i\cdot Cd\cdot Cr\cdot C_i}$

Actual section modulus: $Sa := b\cdot\frac{d^2}{6}$ $Sa = 224\cdot in^3$ > $Sr = 181.3\cdot in^3$

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

Actual area: $Aa := b\cdot d$ $Aa = 84\cdot in^2$ > $Ar = 37.7\cdot in^2$

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

$y := \frac{w\cdot x}{24\cdot E_i\cdot I\cdot l}\cdot\left(l^4 - 2\cdot l^2\cdot x^2 + l\cdot x^3 - 2\cdot a^2\cdot l^2 + 2\cdot a^2\cdot x^2\right)$ $yc := 0.06415\cdot\frac{P\cdot a\cdot l^2}{E_i\cdot I}$ $y := y - yc$

(between supports) $\frac{l}{240} = 0.275\cdot in$ > $y = -0.032\cdot in$

$y1 := \frac{w\cdot a}{24\cdot E_i\cdot I}\cdot\left(3\cdot a^3 - 4\cdot a^2\cdot l - l^3\right)$ $y1c := \frac{P\cdot a^2}{3\cdot E_i\cdot I}\cdot(l + a)$ $y1 := y1 + y1c$

(at end of cantilever) $\frac{2\cdot a}{240} = 0.55\cdot in$ > $y1 = 0.329\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB4

Length: $l := 14 \cdot ft$ $a := 0 \cdot ft$ (larger)
Concentrated load: $p := 0 \cdot lb$ $c := l - a$
Weight per lineal foot: $w := (192 + 15)11 \cdot plf$
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: $Cd := 1.15$
Cr = repetitive use factor: $Cr := 1.0$

Calculate bending moment: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $\underline{M = 55786.5 \cdot ft \cdot lb}$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $\underline{V = 15939 \cdot lb}$

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 $C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1}$ $C_3 := if(C_3 > 1, 1, C_3)$ $C_3 = 0.998$
(glu-laminated lumber only)

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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $\underline{S = 236.6 \cdot in^3}$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $\underline{A = 65.9 \cdot in^2}$

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: $\frac{l}{240} = 0.7 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.386 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

ROOF FRAMING
RB5

Length: $l := 18\cdot ft$ $\quad a := 0\cdot ft$ (larger)

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

Weight per lineal foot: $w := (192 + 15)1\cdot plf$

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: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\quad \underline{M = 8383.5\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\quad \underline{V = 1863\cdot lb}$

Use: (3) 1 3/4 X 11 7/8 LVL $b := 5.25\cdot in$ $\quad d := 11.875\cdot in$ $\quad t := \text{if}(d \le 12\cdot in, 12\cdot in, d)$

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\quad \underline{S = 38.7\cdot in^3}$

Actual section modulus: $S := \frac{b}{6}\cdot d^2$ $\quad S = 123.4\cdot in^3$

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\quad \underline{A = 8.7\cdot in^2}$

Actual area: $A := b\cdot d$ $\quad A = 62.3\cdot in^2$

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

Allowable deflection: $\frac{l}{240} = 0.9\cdot in$

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $\quad y = 0.334\cdot in$

BEAM OVERHANG ONE SUPPORT -

ROOF FRAMING
RB-6

Simple length: $l := 14 \cdot ft$

Cantilever length: $a := 3.5 \cdot ft$

Uniform load: $w := (15 + 192) \cdot 13 \cdot plf$

Added load on cantilever: $w1 := 0 \cdot plf$

Cd = load duration factor: $Cd := 1.15$

Material: $i := LVL$

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

Allowable bending stress Fb: $Fb_i = 2600 \cdot psi$

Allowable shear stress Fv: $Fv_i = 285 \cdot psi$

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

Determine reactions:

$$R1 := \frac{w}{2 \cdot l} \cdot \left(l^2 - a^2\right) - \frac{w1 \cdot a^2}{2 \cdot l} \qquad R1 = 17660 \cdot lb$$

$$R2 := \frac{w}{2 \cdot l} \cdot (l + a)^2 + \frac{w1 \cdot a}{2 \cdot l} \cdot (2 \cdot l + a) \qquad R2 = 29433 \cdot lb$$

Determine shear forces:

$$V1 := R1 \qquad V1 = 17659.7 \cdot lb$$

$$V3 := \frac{w}{2 \cdot l} \cdot \left(l^2 + a^2\right) + \frac{w1 \cdot a^2}{2 \cdot l} \qquad V2 := w \cdot a + w1 \cdot a \qquad V2 = 9418.5 \cdot lb$$

$$V3 = 20014.3 \cdot lb$$

Determine max locations:

$$x := \frac{1}{2} \cdot \left[1 - \left(\frac{a^2}{l^2}\right)\right] \qquad x = 6.6 \cdot ft$$

Determine moments:
(between supports)

$$M1 := \frac{w}{8 \cdot l^2} \cdot (l + a)^2 \cdot (l - a)^2 - \frac{w1 \cdot a^2 \cdot x}{2 \cdot l} \qquad M1 = 57945.8 \cdot ft \cdot lb$$

(at cantilever end)

$$M2 := \frac{w \cdot a^2}{2} + \frac{w1 \cdot a^2}{2} \qquad M2 = 16482.4 \cdot ft \cdot lb$$

$$M := if(M1 > M2, M1, M2) \qquad M = 57945.8 \cdot ft \cdot lb$$

$$V := if(V1 > V2, if(V1 > V3, V1, V3), if(V2 > V3, V2, V3)) \qquad V = 20014.3 \cdot lb$$

Use: **(3) 1 3/4 X 18 LVL** $b := 5.125 \cdot in$ $d := 18 \cdot in$ $t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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)

$$C_1 := if\left(d > 10 \cdot in, if(d > 12 \cdot in, 0.9, 1.0), \right. \quad C_2 := C_1 \qquad C_1 = 0.9$$

Cv = volume factor
(glu-laminated lumber only)

$$C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1} \qquad C_3 := if\left(C_3 > 1, 1, C_3\right) \quad C_3 = 1$$

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

BEAM OVERHANG ONE SUPPORT - CONTINUED

Required section modulus: $Sr := \frac{M}{(1\cdot Fb)_i\cdot Cd\cdot Cr\cdot C_i}$

Actual section modulus: $Sa := b\cdot\frac{d^2}{6}$ $Sa = 276.8\cdot in^3$ > $Sr = 245.7\cdot in^3$

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

Actual area: $Aa := b\cdot d$ $Aa = 92.3\cdot in^2$ > $Ar = 84.1\cdot in^2$

Determine deflections: $I := b\cdot\frac{d^3}{12}$ $I = 2490.7\cdot in^4$

(between supports)

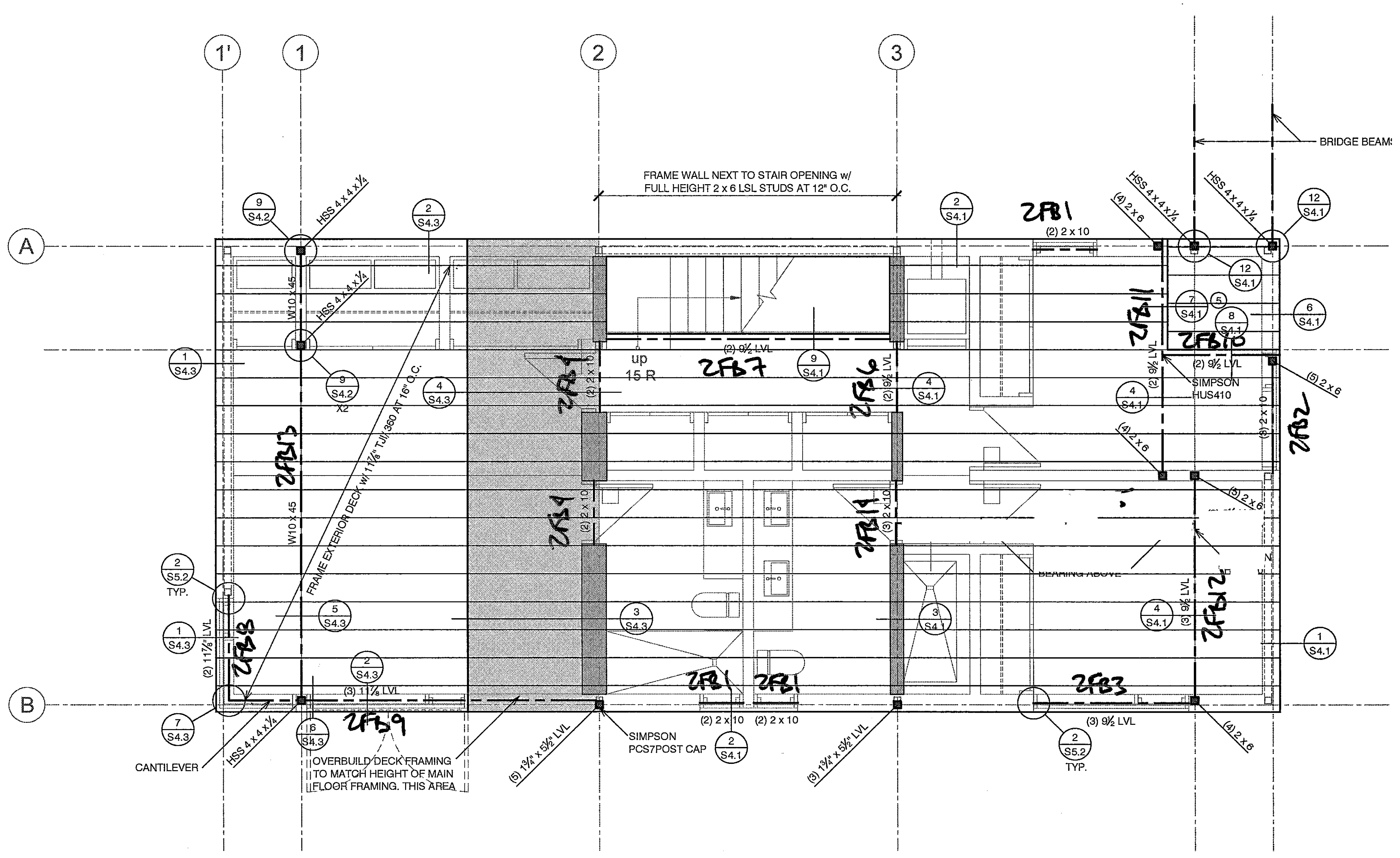
$$y := \frac{w\cdot x}{24\cdot E_i\cdot I\cdot l}\cdot\left(l^4 - 2\cdot l^2\cdot x^2 + l\cdot x^3 - 2\cdot a^2\cdot l^2 + 2\cdot a^2\cdot x^2\right)$$

$yc := 0.03208\cdot\frac{w1\cdot a^2\cdot l^2}{E_i\cdot I}$ $y := y - yc$ $\frac{l}{240} = 0.7\cdot in$ > $y = 0.396\cdot in$

(at end of cantilever)

$y1 := \frac{w\cdot a}{24\cdot E_i\cdot I}\cdot\left(3\cdot a^3 + 4\cdot a^2\cdot l - l^3\right)$ $y1c := \frac{w1\cdot a}{24\cdot E_i\cdot I}\cdot\left(3\cdot a^3 + 4\cdot a^2\cdot l - l^3\right)$

$y1 := y1 + y1c$ $\frac{a\cdot 2}{240} = 0.35\cdot in$ > $y1 = -0.263\cdot in$



SECOND FLOOR FRAMING KEY PLAN

2FB5 NOT USED

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
FLOOR JOISTS

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

Spacing: $s := 16\cdot in$

Loads per lineal foot: $wll := 40\cdot psf\cdot s$

$wdl := 50\cdot psf\cdot s$ $\quad w := wdl + wll$

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: $Mx := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\quad \underline{Mx = 2940\cdot ft\cdot lb}$

Calculate the shear: $Vx := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\quad \underline{Vx = 840\cdot lb}$

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

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

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

USE: 9 1/2" TJI 210 @ 16" O.C.

	Joist Capacities	>	Required Capacities
Moment Capacity:	$M_{i,j} = 3000\cdot ft\cdot lb$	>	$Mx = 2940\cdot ft\cdot lb$
Maximum Reaction:	$V_{i,j} = 1330\cdot lb$	>	$Vx = 840\cdot lb$
Deflection Constant:	$EI_{i,j} = 186\cdot 10^6\cdot in^2\cdot lb$		

Check deflection:

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

Compare total deflection with allowable: $\frac{l}{240} = 0.7\cdot in$ $\quad > \quad y = 0.624\cdot in$

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: $\frac{l}{480} = 0.35\cdot in$ $\quad > \quad y = 0.277\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

UPPER FLOOR FRAMING
EXTERIOR DECK JOISTS

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

Spacing: $s := 16 \cdot in$

Loads per lineal foot: $wll := 192 \cdot 0.5 \cdot psf \cdot s$

$wdl := 50 \cdot psf \cdot s$ $\quad w := wdl + wll$

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: $Mx := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8} \right)$ $\quad \underline{Mx = 4769.3 \cdot ft \cdot lb}$

Calculate the shear: $Vx := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2} \right)$ $\quad \underline{Vx = 1362.7 \cdot lb}$

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 7/8" TJI 360 @ 16" O.C.

	Joist Capacities	>	Required Capacities
Moment Capacity:	$M_{i,j} = 6180 \cdot ft \cdot lb$	>	$Mx = 4769 \cdot ft \cdot lb$
Maximum Reaction:	$V_{i,j} = 1705 \cdot lb$	>	$Vx = 1363 \cdot lb$
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: $\frac{l}{240} = 0.7 \cdot in$ $\quad > \quad$ $y = 0.487 \cdot in$

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: $\frac{l}{480} = 0.35 \cdot in$ $\quad > \quad$ $y = 0.32 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
JOISTS AT ENTRY

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: $w := (50 + 40)1.33\cdot plf$

Material: $i := DF2$

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

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

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

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

Calculate bending moment: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\quad \underline{M = 374.1\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\quad \underline{V = 299.2\cdot lb}$

Use: 2 X 8 AT 16" O.C. $\quad b := 1.5\cdot in$ $\quad d := 7.25\cdot in$ $\quad t := \text{if}(d \le 12\cdot in, 12\cdot in, d)$

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

(sawn lumber only) $\quad C_1 := \text{if}\left(d > 10\cdot in, \text{if}(d > 12\cdot in, 0.9, 1.0), \right. C_2 := C_1$ $\quad C_1 = 1.2$

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

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

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

Required section modulus: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\quad \underline{S = 4.2\cdot in^3}$

Actual section modulus: $S := \frac{b}{6}\cdot d^2$ $\quad S = 13.1\cdot in^3$

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\quad \underline{A = 1.9\cdot in^2}$

Actual area: $A := b\cdot d$ $\quad A = 10.9\cdot in^2$

Check deflection: $I := \frac{b}{12}\cdot d^3$ $\quad I = 47.6\cdot in^4$

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

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $\quad y = 0.022\cdot in$ $\quad y\cdot\frac{40}{90} = 0.01\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB1

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

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

Weight per lineal foot: $w := (50 + 40) 1 \cdot plf + (192 + 15) \cdot 11 \cdot plf$

Material: $i := DF2$

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

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

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

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

Calculate bending moment: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8} \right)$ $\quad \underline{M = 2662.9 \cdot ft \cdot lb}$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2} \right)$ $\quad \underline{V = 3550.5 \cdot lb}$

Use: **(2) 2 X 10**

$b := 3.0 \cdot in$ $\quad d := 9.25 \cdot in$ $\quad t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $\quad \underline{S = 32.3 \cdot in^3}$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i} \right)$ $\quad \underline{A = 14.4 \cdot in^2}$

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

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

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

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $\quad y = 0.014 \cdot in$ $\quad y \cdot \frac{40}{90} = 0.006 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB2

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

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

Weight per lineal foot: $w := (50+40)3\cdot plf + (192+15)\cdot 1\cdot plf + 150\cdot plf$

Material: $i := DF2$

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

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

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

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

Calculate bending moment: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\quad \underline{M = 2821.5\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\quad \underline{V = 1881\cdot lb}$

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

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\quad \underline{S = 34.2\cdot in^3}$

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

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\quad \underline{A = 11.6\cdot in^2}$

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

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

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

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $\quad y = 0.039\cdot in$ $\quad y\cdot\frac{40}{90} = 0.017\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB3

Length: $l := 7\cdot ft$ $a := 0\cdot ft$ (larger)

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

Weight per lineal foot: $w := (50 + 40)1\cdot plf + (192 + 15)\cdot 11\cdot plf + 150\cdot plf$

Material: $i := LVL$

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

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

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

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

Calculate bending moment: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\underline{M = 15416.6\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\underline{V = 8809.5\cdot lb}$

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) $C_1 := if\left(d > 10\cdot in, if(d > 12\cdot in, 0.9, 1.0), (C_2 := C_1\right.$ $C_1 = 1.1$

Cv = volume factor $C_3 := 1\cdot\left(21\cdot\frac{ft}{l}\right)^{.1}\cdot\left(12\cdot\frac{in}{d}\right)^{.1}\cdot\left(5.125\cdot\frac{in}{b}\right)^{.1}$ $C_3 := if(C_3 > 1, 1, C_3)$ $C_3 = 1$
(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: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\underline{S = 71.2\cdot in^3}$

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

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\underline{A = 35.9\cdot in^2}$

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: $\frac{l}{240} = 0.35\cdot in$ $\frac{l}{360} = 0.233\cdot in$

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $y = 0.181\cdot in$ $y\cdot\frac{40}{90} = 0.081\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB4

Length: $l := 4 \cdot ft$ $\quad a := 0 \cdot ft$ (larger)

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

Weight per lineal foot: $w := (40 + 40)14 \cdot plf$

Material: $i := DF2$

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

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

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

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

Calculate bending moment: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $\quad \underline{M = 2240 \cdot ft \cdot lb}$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $\quad \underline{V = 2240 \cdot lb}$

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

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $\quad \underline{S = 27.2 \cdot in^3}$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $\quad \underline{A = 11.5 \cdot in^2}$

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

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

Allowable deflection: $\frac{l}{240} = 0.2 \cdot in$ $\quad \frac{l}{360} = 0.133 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $\quad y = 0.02 \cdot in$ $\quad y \cdot \frac{40}{90} = 0.009 \cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB6

Length: $l := 4\cdot ft$ $\quad a := 0\cdot ft$ (larger)

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

Weight per lineal foot: $w := (40 + 40)14\cdot plf$

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: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\quad \underline{M = 2240\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\quad \underline{V = 2240\cdot lb}$

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

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\quad \underline{S = 10.3\cdot in^3}$

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

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\quad \underline{A = 7.1\cdot in^2}$

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

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

Allowable deflection: $\frac{l}{240} = 0.2\cdot in$ $\quad \frac{l}{360} = 0.133\cdot in$

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $\quad y = 0.013\cdot in$ $\quad y\cdot\frac{40}{90} = 0.006\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB7

Length: $l := 14\cdot ft$ $a := 0\cdot ft$ (larger)

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

Weight per lineal foot: $w := (50 + 40)2\cdot plf$

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: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\underline{M = 4410\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\underline{V = 1260\cdot lb}$

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

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

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

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

(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: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\underline{S = 20.4\cdot in^3}$

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

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\underline{A = 5.9\cdot in^2}$

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

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

Allowable deflection: $\frac{l}{240} = 0.7\cdot in$ $\frac{l}{360} = 0.467\cdot in$

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $y = 0.311\cdot in$ $y\cdot\frac{40}{90} = 0.138\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB8

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

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

Weight per lineal foot: $w := (192 \cdot .5 + 50)2 \cdot plf + (192 + 15) \cdot 1 \cdot plf + 150 \cdot plf$

Material: $i := LVL$

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

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

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

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

Calculate bending moment: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $M = 2920.5 \cdot ft \cdot lb$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $V = 1947 \cdot lb$

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 $C1 := if(d > 4 \cdot in, if(d > 6 \cdot in, if(d > 8 \cdot in, 1.1, 1.2), 1.3), 1.5)$

(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: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $S = 13.5 \cdot in^3$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $A = 6.9 \cdot in^2$

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

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

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

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.019 \cdot in$ $y \cdot \frac{40}{90} = 8.611 \times 10^{-3} \cdot in$

BEAM OVERHANGING ONE SUPPORT - CONCENTRATED LOAD AT OVERHANG

SECOND FLOOR FRAMING
2FB9

Simple length: $l := 14 \cdot ft$

Cantilever length: $a := 4 \cdot ft$

Uniform load: $w := 200 \cdot plf$

Point load at cantilever: $P := 2000 \cdot lb$

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

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

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: $E_i = 2000000 \cdot psi$

Determine reactions:

$$R1 := \frac{w}{2 \cdot l} \cdot \left(l^2 - a^2\right) - \frac{P \cdot a}{l} \qquad R1 = 714.3 \cdot lb$$

$$R2 := \frac{w}{2 \cdot l} \cdot (l + a)^2 + \frac{P}{l} \cdot (l + a) \qquad R2 = 4885.7 \cdot lb$$

Determine shear forces:

$$V1 := R1 \qquad V2 := w \cdot a + P \qquad V1 = 714.3 \cdot lb \qquad V2 = 2800 \cdot lb$$

$$V3 := \frac{w}{2 \cdot l} \cdot \left(l^2 + a^2\right) + \frac{P \cdot a}{l} \qquad V3 = 2085.7 \cdot lb$$

Determine max locations:

$$x := \frac{l}{2} \cdot \left[1 - \left(\frac{a^2}{l^2}\right)\right] \qquad x = 6.429 \cdot ft$$

Determine moments:
(between supports)

$$M1 := \frac{w}{8 \cdot l^2} \cdot (l + a)^2 \cdot (l - a)^2 - \frac{P \cdot a \cdot x}{l} \qquad M1 = 459.2 \cdot ft \cdot lb$$

$$M1 := |M1|$$

(at cantilever
end)

$$M2 := \frac{w \cdot a^2}{2} + P \cdot a \qquad M2 = 9600 \cdot ft \cdot lb$$

$$M := if(M1 > M2, M1, M2) \qquad M = 9600 \cdot ft \cdot lb$$

$$V := if(V1 > V2, if(V1 > V3, V1, V3), if(V2 > V3, V2, \qquad V = 2800 \cdot lb$$

Use: (3) 1 3/4 X 11 7/8 LVL

$$b := 5.25 \cdot in \qquad d := 11.875 \cdot in \qquad t := if(d \le 12 \cdot in, 12 \cdot in, d)$$

CF = Size factor

(sawn lumber only)

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

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

Cv = volume factor
(glu-laminated lumber only)

$$C_3 := 1 \cdot \left(21 \cdot \frac{ft}{l}\right)^{.1} \cdot \left(12 \cdot \frac{in}{d}\right)^{.1} \cdot \left(5.125 \cdot \frac{in}{b}\right)^{.1} \qquad C_3 := if\left(C_3 > 1, 1, C_3\right) \qquad C_3 = 1$$

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: $Sr := \frac{M}{(1\cdot Fb)_i\cdot Cd\cdot Cr\cdot C_i}$

Actual section modulus: $Sa := b\cdot\frac{d^2}{6}$ $Sa = 123.4\cdot in^3$ > $Sr = 44.3\cdot in^3$

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

Actual area: $Aa := b\cdot d$ $Aa = 62.3\cdot in^2$ > $Ar = 13.7\cdot in^2$

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

$y := \frac{w\cdot x}{24\cdot E_i\cdot I\cdot l}\cdot\left(l^4 - 2\cdot l^2\cdot x^2 + l\cdot x^3 - 2\cdot a^2\cdot l^2 + 2\cdot a^2\cdot x^2\right)$ $yc := 0.06415\cdot\frac{P\cdot a\cdot l^2}{E_i\cdot I}$ $y := y - yc$

(between supports) $\frac{l}{240} = 0.7\cdot in$ > $y = -0.024\cdot in$

$y1 := \frac{w\cdot a}{24\cdot E_i\cdot I}\cdot\left(3\cdot a^3 - 4\cdot a^2\cdot l - l^3\right)$ $y1c := \frac{P\cdot a^2}{3\cdot E_i\cdot I}\cdot(1 + a)$ $y1 := y1 + y1c$

(at end of cantilever) $\frac{2\cdot a}{240} = 0.4\cdot in$ > $y1 = 0.091\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB10

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

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

Weight per lineal foot: $w := (50 + 40)2\cdot plf$

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: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\underline{M = 810\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\underline{V = 540\cdot lb}$

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

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\underline{S = 3.7\cdot in^3}$

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

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\underline{A = 2.1\cdot in^2}$

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

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

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

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $y = 0.01\cdot in$ $y\cdot\frac{40}{90} = 0.005\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB11

Length: $l := 8.5\cdot ft$ $\quad a := 5.5\cdot ft$ (larger)

Concentrated load: $p := 600\cdot lb$ $\quad c := l - a$

Weight per lineal foot: $w := (50 + 40)9\cdot plf$

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: $M := \frac{p\cdot a\cdot c}{l} + \left(w\cdot\frac{l^2}{8}\right)$ $\quad \underline{M = 8480\cdot ft\cdot lb}$

Calculate the shear: $V := \frac{p\cdot a}{l} + \left(w\cdot\frac{l}{2}\right)$ $\quad \underline{V = 3830.7\cdot lb}$

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

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i\cdot Cd\cdot Cr\cdot C_i}$ $\quad \underline{S = 39.1\cdot in^3}$

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

Required area: $A := 1.5\cdot\left(\frac{V - w\cdot d}{Fv_i}\right)$ $\quad \underline{A = 16.8\cdot in^2}$

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

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

Allowable deflection: $\frac{l}{240} = 0.425\cdot in$ $\quad \frac{l}{360} = 0.283\cdot in$

Actual deflection: $y := \frac{p\cdot a^2\cdot c^2}{3\cdot E_i\cdot I\cdot l} + \frac{5\cdot w\cdot l^4}{384\cdot E_i\cdot I}$ $\quad y = 0.212\cdot in$ $\quad y\cdot\frac{40}{80} = 0.106\cdot in$

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB12

Length: $l := 8.5 \cdot ft$ $\quad a := 5.5 \cdot ft$ (larger)

Concentrated load: $p := 3800 \cdot lb$ $\quad c := l - a$

Weight per lineal foot: $w := (50 + 40)9 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $\quad \underline{M = 14691.8 \cdot ft \cdot lb}$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $\quad \underline{V = 5901.3 \cdot lb}$

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

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $\quad \underline{S = 67.8 \cdot in^3}$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $\quad \underline{A = 27.7 \cdot in^2}$

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

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

Allowable deflection: $\frac{l}{240} = 0.425 \cdot in$ $\quad \frac{l}{360} = 0.283 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $\quad y = 0.22 \cdot in$ $\quad y \cdot \frac{40}{80} = 0.11 \cdot in$

STEEL BEAM DESIGN

GEOMETRY

Beam Designation	W10X45		
Steel Yield Strength Fy ...	50.0	ksi	OK
Modulus of Elasticity Es ..	29000	ksi	
Member Length L	17.00	ft	
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	
Unbraced Length Lb top ..	10.00	ft	
Unbraced Length Lb bot ..	17.00	ft	

PROPERTIES

Area ..	13.3	in^2	Sx ...	49.1	in^3
Depth	10.1	in	Zx ...	54.9	in^3
bf	8.0	in	rx	4.32	in
tw	0.35	in	Iy	53.4	in^4
tf	0.62	in	Sy ...	13.3	in^3
k des .	1.12	in	Zy ...	20.3	in^3
Ix	248.0	in^4	ry	2.01	in
Cw	1200.0	in^6	J	1.51	in^4

UNFACTORED LOADS (Selfweight calculated internally)

	Uniform (k/ft)		Concentrated (kip)						Moments (k-ft)	
SPAN 1	w1	w2	P1	P2	P3	P4	P5	P6	ML	MR
Const. Dead Load .	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Const. Live Load ...	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Dead Load	0.18	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load	0.90	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Roof Live Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Snow Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Wind Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Seismic Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Start Distance (ft) ..	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
End Distance (ft) ...	17.00	0.00								

Project:
Engineer:
Descrip: 2FB13

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7/28/2017

ASDIP Steel 4.1.2

STEEL BEAM DESIGN

www.asdipsoft.com

FLEXURE DESIGN (STEEL)

L. T. Buckling Cb-factor	1.21	
Max. Bending Moment M ..	61.7	k-ft

Limit States	Nominal Mn		
Yielding	228.8	k-ft	▯
Lateral Torsional Buckling	228.8	k-ft	
Flange Local Buckling	N.A.	k-ft	
Web Local Buckling	N.A.	k-ft	

Nominal Strength Mn	205.9	k-ft
Resistance Factor ϕ	0.90	
Design Strength ϕMn	205.9	k-ft
M / ϕMn Design Ratio	0.30	OK

FLEXURE DESIGN (COMPOSITE)

Overall Slab Thickness .... N.A

Interior Beam. Spacing = 5.0 ft

Effective Slab Width	N.A
Concrete Strength f'c	N.A
Concrete Density	N.A
Metal Deck Type	None None
Deck Ribs Height hr	N.A
Deck Ribs Avg. Width wr ..	N.A

No Metal Deck specified for this Beam

Max. Bending Moment M N.A

Limit States	Nominal Mn
Plastic Yielding	N.A
Elastic Yielding	N.A.

Nominal Strength Mn	N.A
Resistance Factor ϕ	0.90
Design Strength ϕMn	N.A
M / ϕMn Design Ratio	N.A

DEFLECTIONS

Stiffness factor	1.0	
Required Camber	0.00	in
Long-term Deflection	N.A.	

Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	9999	360	0.04	OK
CD+CL .	0.01	9999	240	0.02	OK
L	0.24	868	360	0.41	OK
D+L	0.29	694	240	0.35	OK

DESIGN FOR SHEAR

Shear Coefficient Cv	1.00	
Maximum Shear Force V ...	14.5	kip

Limit States	Nominal Vn		
Shear Yielding	106.1	kip	▯
Shear Buckling	106.1	kip	

Nominal Strength Vn	106.1	kip
Resistance Factor ϕ	1.00	
Design Strength ϕVn	106.1	kip
V / ϕVn Design Ratio	0.14	OK

LOCAL BUCKLING

Flanges in Flexure	Compact
Flanges in Compression	Non-compact
Web in Flexure	Compact
Web in Compression	Non-compact

SHEAR CONNECTORS

Shear Stud Diameter	N.A
Shear Stud Length	N.A
Tensile Strength Fu	N.A
Nominal Strength Qn	N.A
Horizontal Shear Force	N.A
# of Studs for Full Composite	N.A
# of Studs for Partial Composite ..	N.A
Partial Composite Action %	N.A
Minimum Spacing Allowed	N.A
# of Studs at Any Section	N.A
Max. Spacing Required	N.A

DESIGN CODES

Steel Design	AISC 360-10 (14th Ed.)
Load Combinations ...	ASCE 7-10

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

SECOND FLOOR FRAMING
2FB14

Length: $l := 4 \cdot ft$ $\quad a := 3 \cdot ft$ (larger)

Concentrated load: $p := 4000 \cdot lb$ $\quad c := l - a$

Weight per lineal foot: $w := (40 + 40)14 \cdot plf$

Material: $i := DF2$

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

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

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

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

Calculate bending moment: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $\quad \underline{M = 5240 \cdot ft \cdot lb}$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $\quad \underline{V = 5240 \cdot lb}$

Use: (2) 2 X 10 $\quad b := 4.5 \cdot in$ $\quad d := 9.25 \cdot in$ $\quad t := if(d \le 12 \cdot in, 12 \cdot in, d)$

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $\quad \underline{S = 63.5 \cdot in^3}$

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

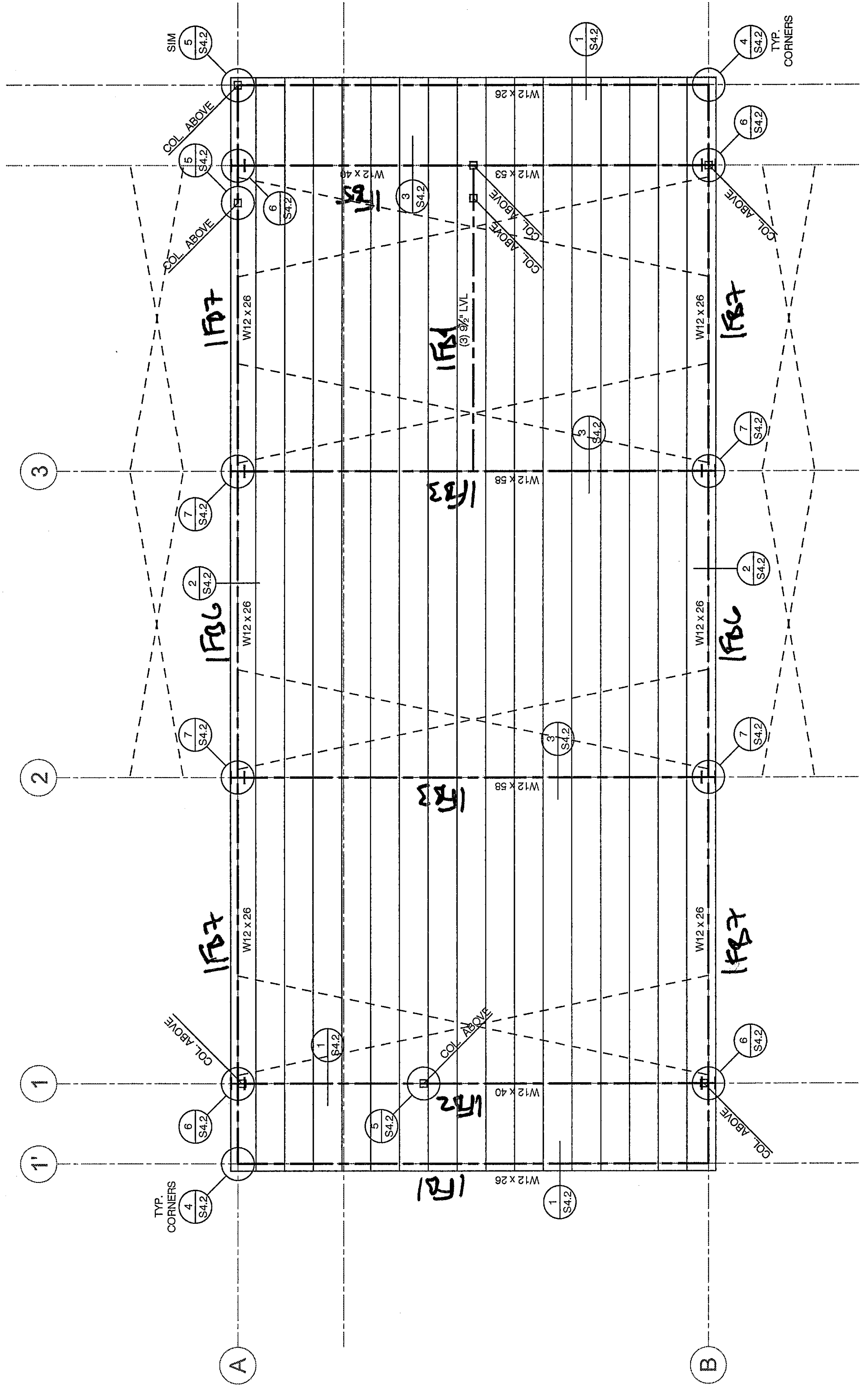
Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $\quad \underline{A = 36.5 \cdot in^2}$

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

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

Allowable deflection: $\frac{l}{240} = 0.2 \cdot in$ $\quad \frac{l}{360} = 0.133 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $\quad y = 0.025 \cdot in$ $\quad y \cdot \frac{40}{90} = 0.011 \cdot in$



MAIN FLOOR FRAMING KEY PLAN

Project:
Engineer:
Descrip: 1FB1

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ASDIP Steel 4.1.2

STEEL BEAM DESIGN

www.asdipsoft.com

GEOMETRY

Beam Designation	W12X26		
Steel Yield Strength Fy ...	50.0	ksi	OK
Modulus of Elasticity Es ..	29000	ksi	
Member Length L	22.00	ft	
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	
Unbraced Length Lb top ..	12.00	ft	
Unbraced Length Lb bot ..	22.00	ft	

PROPERTIES

Area ..	7.7	in^2	Sx ...	33.4	in^3
Depth	12.2	in	Zx ...	37.2	in^3
bf	6.5	in	rx	5.17	in
tw	0.23	in	Iy	17.3	in^4
tf	0.38	in	Sy ...	5.3	in^3
k des .	0.68	in	Zy ...	8.2	in^3
Ix	204.0	in^4	ry	1.51	in
Cw	607.0	in^6	J	0.30	in^4

UNFACTORED LOADS (Selfweight calculated internally)

	Uniform (k/ft)		Concentrated (kip)						Moments (k-ft)	
SPAN 1	w1	w2	P1	P2	P3	P4	P5	P6	ML	MR
Const. Dead Load .	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Const. Live Load ...	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Dead Load	0.35	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load	0.35	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Roof Live Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Snow Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Wind Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Seismic Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Start Distance (ft) ..	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
End Distance (ft) ...	22.00	0.00								

FLEXURE DESIGN (STEEL)

L. T. Buckling Cb-factor	1.25	
Max. Bending Moment M ..	43.8	k-ft
Limit States	Nominal Mn	
Yielding	155.0	k-ft
Lateral Torsional Buckling	143.5	k-ft ▯
Flange Local Buckling	N.A.	k-ft
Web Local Buckling	N.A.	k-ft
Nominal Strength Mn	85.9	k-ft
Safety Factor Ω	1.67	
Allowable Strength Mn/Ω ...	85.9	k-ft
M / Mn/Ω Design Ratio	0.51	OK

FLEXURE DESIGN (COMPOSITE)

Overall Slab Thickness	N.A
Interior Beam. Spacing = 5.0 ft	
Effective Slab Width	N.A
Concrete Strength f'c	N.A
Concrete Density	N.A
Metal Deck Type	None None
Deck Ribs Height hr	N.A
Deck Ribs Avg. Width wr ..	N.A
No Metal Deck specified for this Beam	
Max. Bending Moment M	N.A
Limit States	Nominal Mn
Plastic Yielding	N.A
Elastic Yielding	N.A.
Nominal Strength Mn	N.A
Safety Factor Ω	1.67
Allowable Strength Mn/Ω	N.A
M / Mn/Ω Design Ratio	N.A

DEFLECTIONS

Stiffness factor	1.0	
Required Camber	0.00	in
Long-term Deflection	N.A.	

Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	9999	360	0.04	OK
CD+CL .	0.02	9999	240	0.02	OK
L	0.31	847	360	0.43	OK
D+L	0.65	408	240	0.59	OK

DESIGN FOR SHEAR

Shear Coefficient Cv	1.00	
Maximum Shear Force V ...	8.0	kip
Limit States	Nominal Vn	
Shear Yielding	84.2	kip ▯
Shear Buckling	84.2	kip
Nominal Strength Vn	84.2	kip
Safety Factor Ω	1.50	
Allowable Strength Vn/Ω ..	56.1	kip
V / Vn/Ω Design Ratio	0.14	OK

LOCAL BUCKLING

Flanges in Flexure	Compact
Flanges in Compression	Non-compact
Web in Flexure	Compact
Web in Compression	Non-compact

SHEAR CONNECTORS

Shear Stud Diameter	N.A
Shear Stud Length	N.A
Tensile Strength Fu	N.A
Nominal Strength Qn	N.A
Horizontal Shear Force	N.A
# of Studs for Full Composite	N.A
# of Studs for Partial Composite ..	N.A
Partial Composite Action %	N.A
Minimum Spacing Allowed	N.A
# of Studs at Any Section	N.A
Max. Spacing Required	N.A

DESIGN CODES

Steel Design	AISC 360-10 (14th Ed.)
Load Combinations ...	ASCE 7-10

Project:
Engineer:
Descrip: 1FB2

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ASDIP Steel 4.1.2

STEEL BEAM DESIGN

www.asdipsoft.com

GEOMETRY

Beam Designation	W12X40		
Steel Yield Strength Fy ...	50.0	ksi	OK
Modulus of Elasticity Es ..	29000	ksi	
Member Length L	22.00	ft	
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	
Unbraced Length Lb top ..	10.00	ft	
Unbraced Length Lb bot ..	22.00	ft	

PROPERTIES

Area ..	11.7	in^2	Sx ...	51.5	in^3
Depth	11.9	in	Zx ...	57.0	in^3
bf	8.0	in	rx	5.13	in
tw	0.30	in	Iy	44.1	in^4
tf	0.52	in	Sy ...	11.0	in^3
k des .	1.02	in	Zy ...	16.8	in^3
Ix	307.0	in^4	ry	1.94	in
Cw	1440.0	in^6	J	0.91	in^4

UNFACTORED LOADS (Selfweight calculated internally)

	Uniform (k/ft)		Concentrated (kip)						Moments (k-ft)	
SPAN 1	w1	w2	P1	P2	P3	P4	P5	P6	ML	MR
Const. Dead Load .	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Const. Live Load ...	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Dead Load	0.45	0.00	2.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load	0.36	0.00	11.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Roof Live Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Snow Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Wind Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Seismic Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Start Distance (ft) ..	0.00	0.00	7.50	0.00	0.00	0.00	0.00	0.00		
End Distance (ft) ...	22.00	0.00								

Project:
Engineer:
Descrip: 1FB2

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ASDIP Steel 4.1.2 **STEEL BEAM DESIGN** *www.asdipsoft.com*

FLEXURE DESIGN (STEEL)

L. T. Buckling Cb-factor	1.32	
Max. Bending Moment M ..	169.5	k-ft
Limit States	Nominal Mn	
Yielding	237.5	k-ft ☐
Lateral Torsional Buckling	237.5	k-ft
Flange Local Buckling	N.A.	k-ft
Web Local Buckling	N.A.	k-ft
Nominal Strength Mn	213.8	k-ft
Resistance Factor ϕ	0.90	
Design Strength ϕMn	213.8	k-ft
M / ϕMn Design Ratio	0.79	OK

FLEXURE DESIGN (COMPOSITE)

Overall Slab Thickness	N.A
Interior Beam. Spacing = 5.0 ft	
Effective Slab Width	N.A
Concrete Strength f'c	N.A
Concrete Density	N.A
Metal Deck Type	None None
Deck Ribs Height hr	N.A
Deck Ribs Avg. Width wr ..	N.A
No Metal Deck specified for this Beam	
Max. Bending Moment M	N.A
Limit States	Nominal Mn
Plastic Yielding	N.A
Elastic Yielding	N.A.
Nominal Strength Mn	N.A
Resistance Factor ϕ	0.90
Design Strength ϕMn	N.A
M / ϕMn Design Ratio	N.A

DEFLECTIONS

Stiffness factor	1.0	
Required Camber	0.00	in
Long-term Deflection	N.A.	

Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	9999	360	0.04	OK
CD+CL .	0.02	9999	240	0.02	OK
L	0.65	407	360	0.88	OK
D+L	1.02	258	240	0.93	OK

DESIGN FOR SHEAR

Shear Coefficient Cv	1.00	
Maximum Shear Force V ...	27.0	kip
Limit States	Nominal Vn	
Shear Yielding	105.3	kip ☐
Shear Buckling	105.3	kip
Nominal Strength Vn	105.3	kip
Resistance Factor ϕ	1.00	
Design Strength ϕVn	105.3	kip
V / ϕVn Design Ratio	0.26	OK

LOCAL BUCKLING

Flanges in Flexure	Compact
Flanges in Compression	Non-compact
Web in Flexure	Compact
Web in Compression	Non-compact

SHEAR CONNECTORS

Shear Stud Diameter	N.A
Shear Stud Length	N.A
Tensile Strength Fu	N.A
Nominal Strength Qn	N.A
Horizontal Shear Force	N.A
# of Studs for Full Composite	N.A
# of Studs for Partial Composite ..	N.A
Partial Composite Action %	N.A
Minimum Spacing Allowed	N.A
# of Studs at Any Section	N.A
Max. Spacing Required	N.A

DESIGN CODES

Steel Design	AISC 360-10 (14th Ed.)
Load Combinations ...	ASCE 7-10

Project:
Engineer:
Descrip: 1FB3

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ASDIP Steel 4.1.2

STEEL BEAM DESIGN

www.asdipsoft.com

GEOMETRY

Beam Designation	W12X58		
Steel Yield Strength Fy ...	50.0	ksi	OK
Modulus of Elasticity Es ..	29000	ksi	
Member Length L	22.00	ft	
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	
Unbraced Length Lb top ..	0.00	ft	
Unbraced Length Lb bot ..	22.00	ft	

PROPERTIES

Area ..	17.0	in^2	Sx ...	78.0	in^3
Depth	12.2	in	Zx ...	86.4	in^3
bf	10.0	in	rx	5.28	in
tw	0.36	in	Iy	107.0	in^4
tf	0.64	in	Sy ...	21.4	in^3
k des .	1.24	in	Zy ...	32.5	in^3
Ix	475.0	in^4	ry	2.51	in
Cw	3570.0	in^6	J	2.10	in^4

UNFACTORED LOADS (Selfweight calculated internally)

	Uniform (k/ft)		Concentrated (kip)						Moments (k-ft)	
SPAN 1	w1	w2	P1	P2	P3	P4	P5	P6	ML	MR
Const. Dead Load .	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Const. Live Load ...	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Dead Load	1.20	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load	1.50	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Roof Live Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Snow Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Wind Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Seismic Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Start Distance (ft) ..	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
End Distance (ft) ...	22.00	0.00								

FLEXURE DESIGN (STEEL)

L. T. Buckling Cb-factor	1.25	
Max. Bending Moment M ..	236.0	k-ft

Limit States	Nominal Mn		
Yielding	360.0	k-ft	☐
Lateral Torsional Buckling	360.0	k-ft	
Flange Local Buckling	N.A.	k-ft	
Web Local Buckling	N.A.	k-ft	

Nominal Strength Mn	324.0	k-ft	
Resistance Factor ϕ	0.90		
Design Strength ϕMn	324.0	k-ft	
M / ϕMn Design Ratio	0.73		OK

FLEXURE DESIGN (COMPOSITE)

Overall Slab Thickness	N.A

Interior Beam. Spacing = 5.0 ft

Effective Slab Width	N.A
Concrete Strength f'c	N.A
Concrete Density	N.A
Metal Deck Type	None None
Deck Ribs Height hr	N.A
Deck Ribs Avg. Width wr ..	N.A

No Metal Deck specified for this Beam

Max. Bending Moment M	N.A

Limit States	Nominal Mn
Plastic Yielding	N.A
Elastic Yielding	N.A.

Nominal Strength Mn	N.A
Resistance Factor ϕ	0.90
Design Strength ϕMn	N.A
M / ϕMn Design Ratio	N.A

DEFLECTIONS

Stiffness factor	1.0	
Required Camber	0.00	in
Long-term Deflection	N.A.	

Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	9999	360	0.04	OK
CD+CL .	0.02	9999	240	0.02	OK
L	0.57	460	360	0.78	OK
D+L	1.06	250	240	0.96	OK

DESIGN FOR SHEAR

Shear Coefficient Cv	1.00	
Maximum Shear Force V ...	43.0	kip

Limit States	Nominal Vn		
Shear Yielding	131.8	kip	☐
Shear Buckling	131.8	kip	

Nominal Strength Vn	131.8	kip	
Resistance Factor ϕ	1.00		
Design Strength ϕVn	131.8	kip	
V / ϕVn Design Ratio	0.33		OK

LOCAL BUCKLING

Flanges in Flexure	Compact
Flanges in Compression	Non-compact
Web in Flexure	Compact
Web in Compression	Non-compact

SHEAR CONNECTORS

Shear Stud Diameter	N.A
Shear Stud Length	N.A
Tensile Strength Fu	N.A
Nominal Strength Qn	N.A
Horizontal Shear Force	N.A
# of Studs for Full Composite	N.A
# of Studs for Partial Composite ..	N.A
Partial Composite Action %	N.A
Minimum Spacing Allowed	N.A
# of Studs at Any Section	N.A
Max. Spacing Required	N.A

DESIGN CODES

Steel Design	AISC 360-10 (14th Ed.)
Load Combinations ...	ASCE 7-10

SIMPLE BEAM - CONCENTRATED, AND UNIFORM LOAD

LOWER FLOOR FRAMING
1FB4

Length: $l := 14 \cdot ft$ $a := 11.5 \cdot ft$ (larger)

Concentrated load: $p := 3800 \cdot lb$ $c := l - a$

Weight per lineal foot: $w := 90 \cdot 1.33 \cdot plf$

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: $M := \frac{p \cdot a \cdot c}{l} + \left(w \cdot \frac{l^2}{8}\right)$ $\underline{M = 10736.2 \cdot ft \cdot lb}$

Calculate the shear: $V := \frac{p \cdot a}{l} + \left(w \cdot \frac{l}{2}\right)$ $\underline{V = 3959.3 \cdot lb}$

Use: (3) 1 3/4 X 9 1/2 LVL

$b := 5.25 \cdot in$ $d := 9.5 \cdot in$ $t := \text{if}(d \le 12 \cdot in, 12 \cdot in, d)$

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

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

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

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

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

Required section modulus: $S := \frac{M}{Fb_i \cdot Cd \cdot Cr \cdot C_i}$ $\underline{S = 49.6 \cdot in^3}$

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

Required area: $A := 1.5 \cdot \left(\frac{V - w \cdot d}{Fv_i}\right)$ $\underline{A = 20.3 \cdot in^2}$

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: $\frac{l}{240} = 0.7 \cdot in$ $\frac{l}{360} = 0.467 \cdot in$

Actual deflection: $y := \frac{p \cdot a^2 \cdot c^2}{3 \cdot E_i \cdot I \cdot l} + \frac{5 \cdot w \cdot l^4}{384 \cdot E_i \cdot I}$ $y = 0.31 \cdot in$ $y \cdot \frac{40}{90} = 0.138 \cdot in$

Project:
Engineer:
Descrip: 1FB5

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ASDIP Steel 4.1.2

STEEL BEAM DESIGN

www.asdipsoft.com

GEOMETRY

Beam Designation	W12X40		
Steel Yield Strength Fy ...	50.0	ksi	OK
Modulus of Elasticity Es ..	29000	ksi	
Member Length L	22.00	ft	
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	
Unbraced Length Lb top ..	10.00	ft	
Unbraced Length Lb bot ..	22.00	ft	

PROPERTIES

Area ..	11.7	in^2	Sx ...	51.5	in^3
Depth	11.9	in	Zx ...	57.0	in^3
bf	8.0	in	rx	5.13	in
tw	0.30	in	Iy	44.1	in^4
tf	0.52	in	Sy ...	11.0	in^3
k des .	1.02	in	Zy ...	16.8	in^3
Ix	307.0	in^4	ry	1.94	in
Cw	1440.0	in^6	J	0.91	in^4

UNFACTORED LOADS (Selfweight calculated internally)

	Uniform (k/ft)		Concentrated (kip)						Moments (k-ft)	
SPAN 1	w1	w2	P1	P2	P3	P4	P5	P6	ML	MR
Const. Dead Load .	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Const. Live Load ...	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Dead Load	0.50	0.00	8.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load	0.36	0.00	6.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Roof Live Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Snow Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Wind Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Seismic Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Start Distance (ft) ..	0.00	0.00	7.50	0.00	0.00	0.00	0.00	0.00		
End Distance (ft) ...	22.00	0.00								

Project:
Engineer:
Descrip: 1FB5

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ASDIP Steel 4.1.2

STEEL BEAM DESIGN

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FLEXURE DESIGN (STEEL)

L. T. Buckling Cb-factor	1.32	
Max. Bending Moment M ..	166.6	k-ft

Limit States	Nominal Mn	
Yielding	237.5	k-ft ☐
Lateral Torsional Buckling	237.5	k-ft
Flange Local Buckling	N.A.	k-ft
Web Local Buckling	N.A.	k-ft

Nominal Strength Mn	213.8	k-ft	
Resistance Factor ϕ	0.90		
Design Strength ϕMn	213.8	k-ft	
M / ϕMn Design Ratio	0.78		OK

FLEXURE DESIGN (COMPOSITE)

Overall Slab Thickness	N.A

Interior Beam. Spacing = 5.0 ft

Effective Slab Width	N.A
Concrete Strength f'c	N.A
Concrete Density	N.A
Metal Deck Type	None None
Deck Ribs Height hr	N.A
Deck Ribs Avg. Width wr ..	N.A

No Metal Deck specified for this Beam

Max. Bending Moment M	N.A

Limit States	Nominal Mn
Plastic Yielding	N.A
Elastic Yielding	N.A.

Nominal Strength Mn	N.A
Resistance Factor ϕ	0.90
Design Strength ϕMn	N.A
M / ϕMn Design Ratio	N.A

DEFLECTIONS

Stiffness factor	1.0	
Required Camber	0.00	in
Long-term Deflection	N.A.	

Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	9999	360	0.04	OK
CD+CL .	0.02	9999	240	0.02	OK
L	0.44	595	360	0.61	OK
D+L	1.08	244	240	0.98	OK

DESIGN FOR SHEAR

Shear Coefficient Cv	1.00	
Maximum Shear Force V ...	26.8	kip

Limit States	Nominal Vn	
Shear Yielding	105.3	kip ☐
Shear Buckling	105.3	kip

Nominal Strength Vn	105.3	kip	
Resistance Factor ϕ	1.00		
Design Strength ϕVn	105.3	kip	
V / ϕVn Design Ratio	0.25		OK

LOCAL BUCKLING

Flanges in Flexure	Compact
Flanges in Compression	Non-compact
Web in Flexure	Compact
Web in Compression	Non-compact

SHEAR CONNECTORS

Shear Stud Diameter	N.A
Shear Stud Length	N.A
Tensile Strength Fu	N.A
Nominal Strength Qn	N.A
Horizontal Shear Force	N.A
# of Studs for Full Composite	N.A
# of Studs for Partial Composite ..	N.A
Partial Composite Action %	N.A
Minimum Spacing Allowed	N.A
# of Studs at Any Section	N.A
Max. Spacing Required	N.A

DESIGN CODES

Steel Design	AISC 360-10 (14th Ed.)
Load Combinations ...	ASCE 7-10

Project:
Engineer:
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ASDIP Steel 4.1.2

STEEL BEAM DESIGN

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GEOMETRY

Beam Designation	W12X26		
Steel Yield Strength Fy ...	50.0	ksi	OK
Modulus of Elasticity Es ..	29000	ksi	
Member Length L	14.00	ft	
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	
Unbraced Length Lb top ..	10.00	ft	
Unbraced Length Lb bot ..	14.00	ft	

PROPERTIES

Area ..	7.7	in^2	Sx ...	33.4	in^3
Depth	12.2	in	Zx ...	37.2	in^3
bf	6.5	in	rx	5.17	in
tw	0.23	in	Iy	17.3	in^4
tf	0.38	in	Sy ...	5.3	in^3
k des .	0.68	in	Zy ...	8.2	in^3
Ix	204.0	in^4	ry	1.51	in
Cw	607.0	in^6	J	0.30	in^4

UNFACTORED LOADS (Selfweight calculated internally)

	Uniform (k/ft)		Concentrated (kip)						Moments (k-ft)	
SPAN 1	w1	w2	P1	P2	P3	P4	P5	P6	ML	MR
Const. Dead Load .	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Const. Live Load ...	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Dead Load	0.70	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load	2.60	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Roof Live Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Snow Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Wind Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Seismic Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Start Distance (ft) ..	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
End Distance (ft) ...	14.00	0.00								

Project:
Engineer:
Descrip: 1FB6

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ASDIP Steel 4.1.2 **STEEL BEAM DESIGN** *www.asdipsoft.com*

FLEXURE DESIGN (STEEL)

L. T. Buckling Cb-factor	1.14	
Max. Bending Moment M ..	122.6	k-ft

Limit States	Nominal Mn	
Yielding	155.0	k-ft
Lateral Torsional Buckling	144.7	k-ft ▯
Flange Local Buckling	N.A.	k-ft
Web Local Buckling	N.A.	k-ft

Nominal Strength Mn	130.2	k-ft	
Resistance Factor ϕ	0.90		
Design Strength ϕMn	130.2	k-ft	
M / ϕMn Design Ratio	0.94		OK

FLEXURE DESIGN (COMPOSITE)

Overall Slab Thickness	N.A

Interior Beam. Spacing = 5.0 ft

Effective Slab Width	N.A
Concrete Strength f'c	N.A
Concrete Density	N.A
Metal Deck Type	None None
Deck Ribs Height hr	N.A
Deck Ribs Avg. Width wr ..	N.A

No Metal Deck specified for this Beam

Max. Bending Moment M	N.A

Limit States	Nominal Mn
Plastic Yielding	N.A
Elastic Yielding	N.A.

Nominal Strength Mn	N.A
Resistance Factor ϕ	0.90
Design Strength ϕMn	N.A
M / ϕMn Design Ratio	N.A

DEFLECTIONS

Stiffness factor	1.0	
Required Camber	0.00	in
Long-term Deflection	N.A.	

Loading	δ (in)	L/δ	L/δ Min	Ratio	
CL	0.00	9999	360	0.04	OK
CD+CL .	0.00	9999	240	0.02	OK
L	0.38	442	360	0.81	OK
D+L	0.49	346	240	0.69	OK

DESIGN FOR SHEAR

Shear Coefficient Cv	1.00	
Maximum Shear Force V ...	35.2	kip

Limit States	Nominal Vn	
Shear Yielding	84.2	kip ▯
Shear Buckling	84.2	kip

Nominal Strength Vn	84.2	kip	
Resistance Factor ϕ	1.00		
Design Strength ϕVn	84.2	kip	
V / ϕVn Design Ratio	0.42		OK

LOCAL BUCKLING

Flanges in Flexure	Compact
Flanges in Compression	Non-compact
Web in Flexure	Compact
Web in Compression	Non-compact

SHEAR CONNECTORS

Shear Stud Diameter	N.A
Shear Stud Length	N.A
Tensile Strength Fu	N.A
Nominal Strength Qn	N.A
Horizontal Shear Force	N.A
# of Studs for Full Composite	N.A
# of Studs for Partial Composite ..	N.A
Partial Composite Action %	N.A
Minimum Spacing Allowed	N.A
# of Studs at Any Section	N.A
Max. Spacing Required	N.A

DESIGN CODES

Steel Design	AISC 360-10 (14th Ed.)
Load Combinations ...	ASCE 7-10

Gravity Beam Design

RAM SBeam v5.01
1FB7

07/28/17 08:55:45

STEEL CODE: AISC 360-05 LRFD

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

```
Maximum Depth Limitation specified = 13.00 in
Beam Size (User Selected)        =  W12X26                        Fy  = 50.0 ksi
Total Beam Length (ft)           =  18.00
Cantilever on right (ft)         =  4.00
Mp (kip-ft)      =    155.00
Top flange braced by decking.
```

POINT LOADS (kips):

			Flange Bracing	
Dist (ft)	DL	LL	Top	Bottom
18.000	0.60	0.40	No	No
18.000	1.20	11.50	No	No
13.000	1.20	11.50	No	No

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL
1	0.000	0.026	0.000
	14.000	0.026	0.000
2	0.000	0.700	2.600
	14.000	0.700	2.600
3	14.000	0.026	0.000
	18.000	0.026	0.000
4	14.000	0.700	2.600
	18.000	0.700	2.600

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 62.57 kips 1.00Vn = 84.18 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	125.4	7.1	0.0	1.00	0.90	139.50
	Max -	1.2DL+1.6LL	-125.0	14.0	14.0	2.09	0.90	139.50
Right	Max -	1.2DL+1.6LL	-125.0	14.0	4.0	1.00	0.90	139.50
Controlling		1.2DL+1.6LL	125.4	7.1	0.0	1.00	0.90	139.50

REACTIONS (kips):

	Left	**Right**
DL reaction	4.24	11.83
Max +LL reaction	19.02	56.06
Max -LL reaction	-4.89	0.00
Max +total reaction (factored)	35.52	103.90
Max -total reaction	5.93	16.56

DEFLECTIONS:

Center span:

```
Dead load (in)          at      7.00 ft  =      -0.064       L/D  =   2633
Live load (in)          at      7.00 ft  =      -0.421       L/D  =    399
```

Center span:

Net Total load (in) at 7.00 ft = -0.485 L/D = 347

Right cantilever:

Dead load (in) = 0.014 L/D = 6898
Pos Live load (in) = -0.471 L/D = 204
Neg Live load (in) = 0.403 L/D = 238
Pos Total load (in) = -0.457 L/D = 210
Neg Total load (in) = 0.417 L/D = 230

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Engineer:		7/28/2017
Descrip:	Bottom Col	

ASDIP Steel 4.1.2

STEEL COLUMN DESIGN

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GEOMETRY

Column Designation	W8X24		
Steel Yield Strength Fy	50.0	ksi	
Modulus of Elasticity Es	29000	ksi	
Member Length L	10.00	ft	
Effective Length Kx-factor	1.00		
Effective Length Ky-factor	1.00		
Unbraced Length Lb	10.00	ft	OK

ASD SERVICE LOADS (2nd-Order Analysis)

Axial Force P	70.0		kip
	Bottom	Top	
Mx due to Gravity	0.0	0.0	k-ft
Mx due to Lateral	20.0	30.0	k-ft
My due to Gravity	10.0	12.0	k-ft
My due to Lateral	7.0	10.0	k-ft

COMPRESSION

Slenderness Ratio Kx L / rx	35.1		
Slenderness Ratio Ky L / ry	74.5		
Max. Slenderness Ratio	74.5	OK	

Limit States	Nominal Pn		
Flexural Buckling	235.8	kip	☐
Torsional Buckling	280.1	kip	
Flexural-Torsional Buckling	N.A.	kip	

Nominal Strength Pn	235.8	kip
Safety Factor Ω	1.67	
Allowable Strength Pn/Ω	141.2	kip
P / Pn/Ω Design Ratio	0.50	OK

BENDING ABOUT Y-Y

Limit States	Nominal Mn		
Yielding	35.7	k-ft	☐
Lateral-Torsional Buckling	N.A.	k-ft	
Flange Local Buckling	N.A.	k-ft	
Web Local Buckling	N.A.	k-ft	

Nominal Strength Mn	35.7	k-ft
Safety Factor Ω	1.67	
Allowable Strength Mn/Ω	21.4	k-ft
M / Mn/Ω Design Ratio	0.56	OK

PROPERTIES

Area	7.1	in^2	Sx	20.9	in^3
Depth	7.9	in	Zx	23.1	in^3
bf	6.5	in	rx	3.42	in
tw	0.25	in	Iy	18.3	in^4
tf	0.40	in	Sy	5.6	in^3
k des	0.79	in	Zy	8.6	in^3
Ix	82.7	in^4	ry	1.61	in
Cw	259.0	in^6	J	0.35	in^4

LOCAL BUCKLING

Flanges in Flexure	Compact
Flanges in Compression	Non-slender
Web in Flexure	Compact
Web in Compression	Non-slender

BENDING ABOUT X-X

Moment at 1/4 point of Lb	N.A.	k-ft
Moment at 1/2 point of Lb	N.A.	k-ft
Moment at 3/4 point of Lb	N.A.	k-ft
L. T. Buckling Cb-factor	1.75	

Limit States	Nominal Mn		
Yielding	96.3	k-ft	☐
Lateral-Torsional Buckling	96.3	k-ft	
Flange Local Buckling	N.A.	k-ft	
Web Local Buckling	N.A.	k-ft	

Nominal Strength Mn	96.3	k-ft
Safety Factor Ω	1.67	
Allowable Strength Mn/Ω	57.6	k-ft
M / Mn/Ω Design Ratio	0.00	OK

COMBINED FORCES

AISC Equation {H1-1a}	0.99	OK
AISC Equation {H1-1b}	N.A.	

DESIGN CODES

Steel Design	AISC 360-10 (14th Ed.)
Load Combinations	User-defined

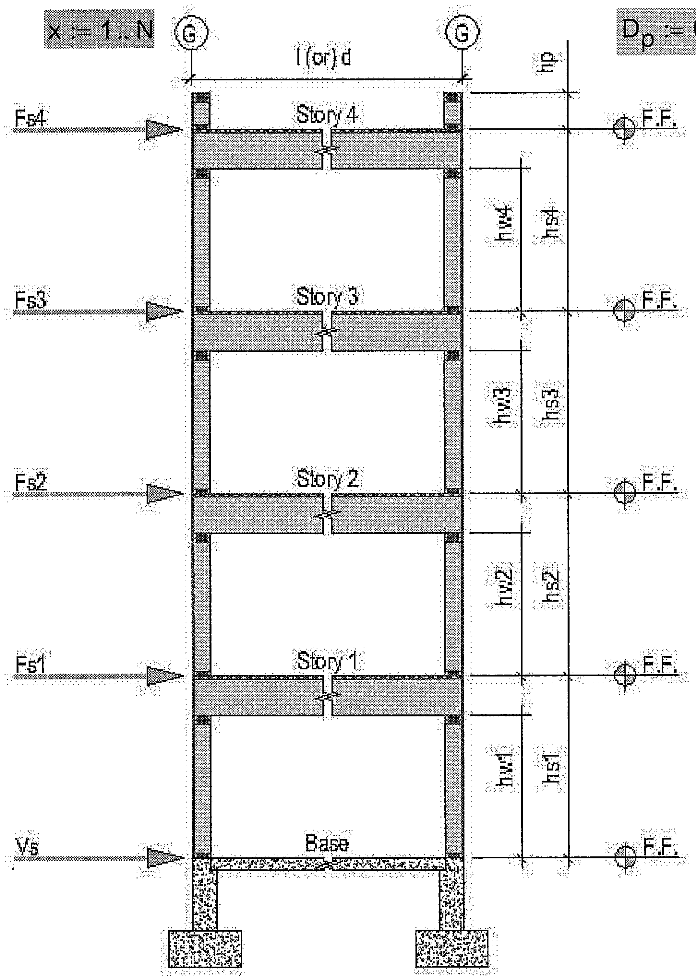
Seismic Base Shear

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

$l_2 := 50\cdot ft$

$d_2 := 22\cdot ft$

$h_{s_2} := 9\cdot ft$

30% snow $D_2 := (15 + 58)\cdot psf$

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

Story 1

$l_1 := 50\cdot ft$

$d_1 := 22\cdot ft$

$h_{s_1} := 10\cdot ft$

$D_1 := 50\cdot psf$

Wood Shear Walls: $R := 6.5$ (Table 12.2-1) $D_{w_1} := 15\cdot psf$

Determine the MCE SRA Parameters per Section 11.4:

Site Class: $SC := \text{"D"}$ (Table 20.3-1)

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

$S_s := 0.898$ $S_1 := 0.304$ (Figures 22-1 thr. 22-14)

$S_{ds} := 0.683$ $S_{d1} := 0.363$ (Equations 11.4-3 & 11.4-4)

Determine the Seismic Design Category per Section 11.6:

Occupancy Category: $OC := 2$ (IBC Table 1604.5)

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

$SDC = \text{"D"}$ (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: $W := \sum w$

$W = 165.5 \cdot kip$

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)

$T = 0.273$

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)

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

$C_s = 0.105$

Calculate the Seismic Base Shear per Section 12.8.1:

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

$V_s = 17.4 \cdot \text{kip}$ STRENGTH

$\dfrac{V_s}{1.4} = 12.4 \cdot \text{kip}$ ALLOWABLE

Vertically Distribute the Seismic Base Shear per Section 12.8.3:

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

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

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

	STRENGTH	ALLOWABLE
(Story 4)	$F_{s_4} = ▪ \cdot kip$	$0.71 F_{s_4} = ▪ \cdot kip$
(Story 3)	$F_{s_3} = ▪ \cdot kip$	$0.71 F_{s_3} = ▪ \cdot kip$
(Story 2)	$F_{s_2} = 12.1 \cdot kip$	$0.71 F_{s_2} = 8.6 \cdot kip$
(Story 1)	$F_{s_1} = 5.3 \cdot kip$	$0.71 F_{s_1} = 3.8 \cdot kip$

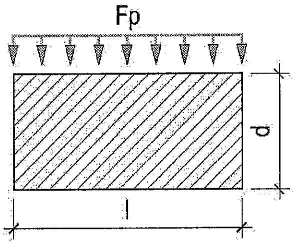
Calculate the Diaphragm Design Force (perpendicular to L) per Section 12.10.1.1:

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

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

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

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



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

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

$F_{psl_2} = 237.8 \cdot plf$ (Story 2)

$F_{psl_1} = 189.2 \cdot plf$ (Story 1)

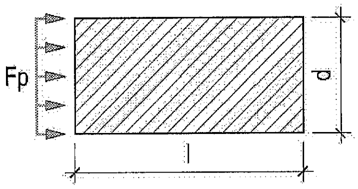
Calculate the Diaphragm Design Force (perpendicular to D) per Section 12.10.1.1:

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

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

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

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



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

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

$F_{psd_2} = 517.0 \cdot plf$ (Story 2)

$F_{psd_1} = 380.4 \cdot plf$ (Story 1)

SHEAR WALLS - LINE 1'

STORY 2

Piers in Shear Line: $n2 := 1$ (n = 8 max)

Story Shear: $Fa_2 := 8.6 \cdot k$ (Allowable)

Shear Attributed To Line: $Va_2 := 3.8 \cdot k$ (Allowable)

Story DL: $DL_2 := 15 \cdot psf$

Wall DL: $DLw_2 := 15 \cdot psf$

Redundancy $\rho_2 := 1$

PIERS	Length	Height	Tributary
1:	$l2_1 := 5.5 \cdot ft$	$h2_1 := 9 \cdot ft$	$t2_1 := 1 \cdot ft$
2:	$l2_2 := 0 \cdot ft$	$h2_2 := 0 \cdot ft$	$t2_2 := 0 \cdot ft$
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: $Mo2_{i2} := v_2 \cdot h2_{i2} \cdot l2_{i2}$ $Mo2 = (34.2) \cdot k \cdot ft$

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 in Shear Line:	$n1 := 1$	(n = 8 max)
Story Shear:	$Fa_1 := 3.8\cdot k$	(Allowable)
Shear Attributed To Line:	$Va_1 := 1.9\cdot k$	(Allowable)
Story DL:	$DL_1 := 50\cdot psf$	
Wall DL:	$DLw_1 := 15\cdot psf$	
Sill Plate Length:	$Ls_1 := 16.5\cdot ft$	
Redundancy	$\rho_1 := 1$	

PIERS	Length	Height	Tributary
1:	$l1_1 := 16.5\cdot ft$	$h1_1 := 10\cdot ft$	$t1_1 := 2\cdot ft$
2:	$l1_2 := 0\cdot ft$	$h1_2 := 0\cdot ft$	$t1_2 := 0\cdot ft$
3:	$l1_3 := 0\cdot ft$	$h1_3 := 0\cdot ft$	$t1_3 := 0\cdot ft$
4:	$l1_4 := 0\cdot ft$	$h1_4 := 0\cdot ft$	$t1_4 := 0\cdot ft$
5:	$l1_5 := 0\cdot ft$	$h1_5 := 0\cdot ft$	$t1_5 := 0\cdot ft$
6:	$l1_6 := 0\cdot ft$	$h1_6 := 0\cdot ft$	$t1_6 := 0\cdot ft$
7:	$l1_7 := 0\cdot ft$	$h1_7 := 0\cdot ft$	$t1_7 := 0\cdot ft$
8:	$l1_8 := 0\cdot ft$	$h1_8 := 0\cdot ft$	$t1_8 := 0\cdot ft$

SHEAR CALCULATIONS

Unit Shear (for walls):

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

OVERTURNING CALCULATIONS

$i1 := 1\,..\,n1$

Overturning Moment:

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

Resisting Moment:

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

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

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

Unit Shear

$$\frac{v2}{r2} = 691\cdot\mathrm{plf}$$

SHEAR WALLS

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

SHEATH BOTH SIDES OF WALL

	Uplift	HOLD DOWN
Pier 1:	$T2_1 = 5971\cdot\mathrm{lb}$	MST72
Pier 2:	$T2_2 = ■\cdot\mathrm{lb}$	
Pier 3:	$T2_3 = ■\cdot\mathrm{lb}$	
Pier 4:	$T2_4 = ■\cdot\mathrm{lb}$	
Pier 5:	$T2_5 = ■\cdot\mathrm{lb}$	
Pier 6:	$T2_6 = ■\cdot\mathrm{lb}$	
Pier 7:	$T2_7 = ■\cdot\mathrm{lb}$	
Pier 8:	$T2_8 = ■\cdot\mathrm{lb}$	

SUMMARY, STORY 1

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

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

Unit Shear

$$\frac{v1}{r1} = 345\cdot\mathrm{plf}$$

SHEAR WALLS

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

	Uplift	HOLD DOWN
Pier 1:	$T1_1 = 2217\cdot\mathrm{lb}$	MST48
Pier 2:	$T1_2 = ■\cdot\mathrm{lb}$	
Pier 3:	$T1_3 = ■\cdot\mathrm{lb}$	
Pier 4:	$T1_4 = ■\cdot\mathrm{lb}$	
Pier 5:	$T1_5 = ■\cdot\mathrm{lb}$	
Pier 6:	$T1_6 = ■\cdot\mathrm{lb}$	
Pier 7:	$T1_7 = ■\cdot\mathrm{lb}$	
Pier 8:	$T1_8 = ■\cdot\mathrm{lb}$	

SHEAR WALLS - LINE 4'

STORY 2

\# Piers in Shear Line: $n2 := 1$ (n = 8 max)

Story Shear: $Fa_2 := 8.6 \cdot k$ (Allowable)

Shear Attributed To Line: $Va_2 := 4.3 \cdot k$ (Allowable)

Story DL: $DL_2 := 15 \cdot psf$

Wall DL: $DLw_2 := 15 \cdot psf$

Redundancy $\rho_2 := 1$

PIERS	Length	Height	Tributary
1:	$l2_1 := 11 \cdot ft$	$h2_1 := 9 \cdot ft$	$t2_1 := 1 \cdot ft$
2:	$l2_2 := 0 \cdot ft$	$h2_2 := 0 \cdot ft$	$t2_2 := 0 \cdot ft$
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:

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

$$Mo2 = (38.7) \cdot k \cdot ft$$

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 in Shear Line: $n1 := 1$ (n = 8 max)

Story Shear: $Fa_1 := 3.8\cdot k$ (Allowable)

Shear Attributed To Line: $Va_1 := 1.9\cdot k$ (Allowable)

Story DL: $DL_1 := 50\cdot psf$

Wall DL: $DLw_1 := 15\cdot psf$

Sill Plate Length: $Ls_1 := 22\cdot ft$

Redundancy $\rho_1 := 1$

PIERS	Length	Height	Tributary
1:	$l1_1 := 22\cdot ft$	$h1_1 := 10\cdot ft$	$t1_1 := 2\cdot ft$
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):

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

OVERTURNING CALCULATIONS $i1 := 1..n1$

Overturning Moment:

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

Resisting Moment:

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

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

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

Unit Shear

$\dfrac{v2}{r2} = 391\cdot\mathrm{plf}$

	Uplift	HOLD DOWN
Pier 1:	$T2_1 = 3023\cdot\mathrm{lb}$	MST48
Pier 2:	$T2_2 = ■\cdot\mathrm{lb}$	
Pier 3:	$T2_3 = ■\cdot\mathrm{lb}$	
Pier 4:	$T2_4 = ■\cdot\mathrm{lb}$	
Pier 5:	$T2_5 = ■\cdot\mathrm{lb}$	
Pier 6:	$T2_6 = ■\cdot\mathrm{lb}$	
Pier 7:	$T2_7 = ■\cdot\mathrm{lb}$	
Pier 8:	$T2_8 = ■\cdot\mathrm{lb}$	

SHEAR WALLS

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

SUBTRACT OPENINGS

$\mathrm{ratio}_{i1} := \dfrac{l1_{i1}}{h1_{i1}}$ $\qquad r1 := \mathrm{if}(2\cdot\min(\mathrm{ratio}) > 1.0, 1.0, 2\cdot\min(\mathrm{ratio}))$ $\qquad r1 = 1$ $\qquad t1 := \dfrac{(22 - 1\cdot 6)}{22}$ $\qquad t1 = 0.73$

Unit Shear

$\dfrac{v1}{r1\cdot t1} = 387\cdot\mathrm{plf}$

	Uplift	HOLD DOWN
Pier 1:	$T1_1 = 1168\cdot\mathrm{lb}$	MST48
Pier 2:	$T1_2 = ■\cdot\mathrm{lb}$	
Pier 3:	$T1_3 = ■\cdot\mathrm{lb}$	
Pier 4:	$T1_4 = ■\cdot\mathrm{lb}$	
Pier 5:	$T1_5 = ■\cdot\mathrm{lb}$	
Pier 6:	$T1_6 = ■\cdot\mathrm{lb}$	
Pier 7:	$T1_7 = ■\cdot\mathrm{lb}$	
Pier 8:	$T1_8 = ■\cdot\mathrm{lb}$	

SHEAR WALLS

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

SHEAR WALLS - LINE A

STORY 2

Piers in Shear Line: $n2 := 2$ (n = 8 max)

Story Shear: $Fa_2 := 8.6 \cdot k$ (Allowable)

Shear Attributed To Line: $Va_2 := 4.3 \cdot k$ (Allowable)

Story DL: $DL_2 := 15 \cdot psf$

Wall DL: $DLw_2 := 15 \cdot psf$

Redundancy $\rho_2 := 1$

PIERS	Length	Height	Tributary
1:	$l2_1 := 18 \cdot ft$	$h2_1 := 9 \cdot ft$	$t2_1 := 11 \cdot ft$
2:	$l2_2 := 24 \cdot ft$	$h2_2 := 9 \cdot ft$	$t2_2 := 11 \cdot ft$
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): $v_2 := \dfrac{\rho_2 \cdot Va_2}{\sum l2}$

OVERTURNING CALCULATIONS $i2 := 1 .. n2$

Overturning Moment: $Mo2_{i2} := v_2 \cdot h2_{i2} \cdot l2_{i2}$ $Mo2 = \begin{pmatrix} 16.586 \\ 22.114 \end{pmatrix} \cdot k \cdot ft$

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: $T2_{i2} := \dfrac{M2_{i2}}{l2_{i2}}$

STORY 1

# Piers in Shear Line:	$n1 := 1$	(n = 8 max)
Story Shear:	$Fa_1 := 3.8 \cdot k$	(Allowable)
Shear Attributed To Line:	$Va_1 := 1.9 \cdot k$	(Allowable)
Story DL:	$DL_1 := 50 \cdot psf$	
Wall DL:	$DLw_1 := 15 \cdot psf$	
Sill Plate Length:	$Ls_1 := 38 \cdot ft$	
Redundancy	$\rho_1 := 1$	

PIERS	Length	Height	Tributary
1:	$l1_1 := 38 \cdot ft$	$h1_1 := 10 \cdot ft$	$t1_1 := 1 \cdot ft$
2:	$l1_2 := 0 \cdot ft$	$h1_2 := 0 \cdot ft$	$t1_2 := 0 \cdot ft$
3:	$l1_3 := 0 \cdot ft$	$h1_3 := 0 \cdot ft$	$t1_3 := 0 \cdot ft$
4:	$l1_4 := 0 \cdot ft$	$h1_4 := 0 \cdot ft$	$t1_4 := 0 \cdot ft$
5:	$l1_5 := 0 \cdot ft$	$h1_5 := 0 \cdot ft$	$t1_5 := 0 \cdot ft$
6:	$l1_6 := 0 \cdot ft$	$h1_6 := 0 \cdot ft$	$t1_6 := 0 \cdot ft$
7:	$l1_7 := 0 \cdot ft$	$h1_7 := 0 \cdot ft$	$t1_7 := 0 \cdot ft$
8:	$l1_8 := 0 \cdot ft$	$h1_8 := 0 \cdot ft$	$t1_8 := 0 \cdot ft$

SHEAR CALCULATIONS

Unit Shear (for walls): $v_1 := \dfrac{\left(\rho_2 \cdot Va_2 + \rho_1 \cdot Va_1\right)}{\sum l1}$

OVERTURNING CALCULATIONS $i1 := 1 .. n1$

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

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

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

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

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

SUMMARY, STORY 2

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

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

Unit Shear

$\frac{v_2}{r2} = 102\cdot\text{plf}$

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:	$T2_1 = -699\cdot\text{lb}$	NONE REQUIRED
Pier 2:	$T2_2 = -1239\cdot\text{lb}$	NONE REQUIRED
Pier 3:	$T2_3 = ▮\cdot\text{lb}$	
Pier 4:	$T2_4 = ▮\cdot\text{lb}$	
Pier 5:	$T2_5 = ▮\cdot\text{lb}$	
Pier 6:	$T2_6 = ▮\cdot\text{lb}$	
Pier 7:	$T2_7 = ▮\cdot\text{lb}$	
Pier 8:	$T2_8 = ▮\cdot\text{lb}$	

SUMMARY, STORY 1

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

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

Unit Shear

$\frac{v_1}{r1} = 163\cdot\text{plf}$

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:	$T1_1 = -648\cdot\text{lb}$	NONE REQUIRED
Pier 2:	$T1_2 = ▮\cdot\text{lb}$	
Pier 3:	$T1_3 = ▮\cdot\text{lb}$	
Pier 4:	$T1_4 = ▮\cdot\text{lb}$	
Pier 5:	$T1_5 = ▮\cdot\text{lb}$	
Pier 6:	$T1_6 = ▮\cdot\text{lb}$	
Pier 7:	$T1_7 = ▮\cdot\text{lb}$	
Pier 8:	$T1_8 = ▮\cdot\text{lb}$	

SHEAR WALLS - LINE B

STORY 2

Piers in Shear Line: $n2 := 1$ (n = 8 max)

Story Shear: $Fa_2 := 8.6 \cdot k$ (Allowable)

Shear Attributed To Line: $Va_2 := 4.3 \cdot k$ (Allowable)

Story DL: $DL_2 := 15 \cdot psf$

Wall DL: $DLw_2 := 15 \cdot psf$

Redundancy $\rho_2 := 1$

PIERS	Length	Height	Tributary
1:	$l2_1 := 14 \cdot ft$	$h2_1 := 9 \cdot ft$	$t2_1 := 11 \cdot ft$
2:	$l2_2 := 0 \cdot ft$	$h2_2 := 0 \cdot ft$	$t2_2 := 0 \cdot ft$
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): $v_2 := \dfrac{\rho_2 \cdot Va_2}{\sum l2}$

OVERTURNING CALCULATIONS $i2 := 1 .. n2$

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

$Mo2 = (38.7) \cdot k \cdot ft$

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: $T2_{i2} := \dfrac{M2_{i2}}{l2_{i2}}$

STORY 1

Piers in Shear Line: $n1 := 2$ (n = 8 max)

Story Shear: $Fa_1 := 3.8 \cdot k$ (Allowable)

Shear Attributed To Line: $Va_1 := 1.9 \cdot k$ (Allowable)

Story DL: $DL_1 := 50 \cdot psf$

Wall DL: $DLw_1 := 15 \cdot psf$

Sill Plate Length: $Ls_1 := 22 \cdot ft$

Redundancy $\rho_1 := 1$

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

SHEAR CALCULATIONS

Unit Shear (for walls):

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

OVERTURNING CALCULATIONS $i1 := 1 .. n1$

Overturning Moment:

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

Resisting Moment:

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

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

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

Unit Shear

$\frac{v2}{r2} = 307 \cdot plf$

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:	$T2_1 = 1504 \cdot lb$	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

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

Unit Shear

$\frac{v1}{r1} = 282 \cdot plf$

SHEAR WALLS

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

	Uplift	HOLD DOWN
Pier 1:	$T1_1 = 2158 \cdot lb$	MST48
Pier 2:	$T1_2 = 2158 \cdot lb$	MST48
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:	$T1_8 = \blacksquare \cdot lb$	

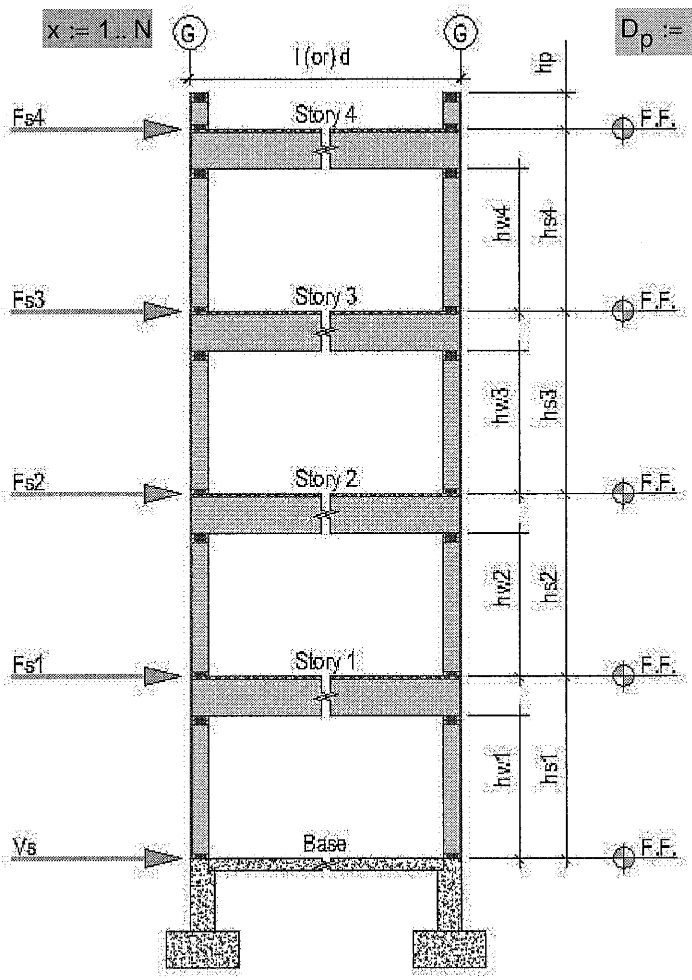
Seismic Base Shear (full structure)

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

$N := 3$

$x := 1..N$



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

$l_3 := 63 \cdot ft$

$d_3 := 26 \cdot ft$

$h_{s_3} := 9 \cdot ft$

30% snow $D_3 := (15 + 58) \cdot psf$

$D_{w_3} := 15 \cdot psf$

Story 2

$l_2 := 50 \cdot ft$

$d_2 := 22 \cdot ft$

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

$D_2 := 50 \cdot psf$

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

Story 1

$l_1 := 50 \cdot ft$

$d_1 := 22 \cdot ft$

$h_{s_1} := 8 \cdot ft$

$D_1 := 50 \cdot psf$

Wood Shear Walls: $R := 6.5$ (Table 12.2-1) $D_{w_1} := 0 \cdot psf$

Determine the MCE SRA Parameters per Section 11.4:

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

@ Short Periods	**@ 1-second Period**	
$S_s := 0.898$	$S_1 := 0.304$	(Figures 22-1 thr. 22-14)
$S_{ds} := 0.683$	$S_{d1} := 0.363$	(Equations 11.4-3 & 11.4-4)

Determine the Seismic Design Category per Section 11.6:

Occupancy Category: $OC := 2$ (IBC Table 1604.5)

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

$SDC = "D"$ (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: $W := \sum w$

$W = 272.9 \cdot kip$

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)

$T = 0.355$

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:

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

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

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

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

$C_s = 0.105$

Calculate the Seismic Base Shear per Section 12.8.1:

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

$V_s = 28.7 \cdot \text{kip}$ STRENGTH

$\frac{V_s}{1.4} = 20.5 \cdot \text{kip}$ ALLOWABLE

Vertically Distribute the Seismic Base Shear per Section 12.8.3:

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

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

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

	STRENGTH	ALLOWABLE
(Story 4)	$F_{s_4} = ▮ \cdot kip$	$0.71 F_{s_4} = ▮ \cdot kip$
(Story 3)	$F_{s_3} = 18.7 \cdot kip$	$0.71 F_{s_3} = 13.3 \cdot kip$
(Story 2)	$F_{s_2} = 7.2 \cdot kip$	$0.71 F_{s_2} = 5.1 \cdot kip$
(Story 1)	$F_{s_1} = 2.8 \cdot kip$	$0.71 F_{s_1} = 2.0 \cdot kip$

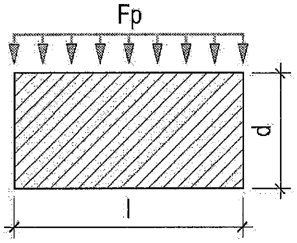
Calculate the Diaphragm Design Force (perpendicular to L) per Section 12.10.1.1:

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

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

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

$$F_{psl_x} := \begin{array}{|l} F_{pslmin_x} \ \text{ if } \ F_{psl_x} < F_{pslmin_x} \\ F_{pslmax_x} \ \text{ if } \ F_{psl_x} > F_{pslmax_x} \\ F_{psl_x} \ \text{ otherwise} \end{array}$$



$F_{psl_4} = ■\cdot plf$ (Story 4)

$F_{psl_3} = 289.4\cdot plf$ (Story 3)

$F_{psl_2} = 189.2\cdot plf$ (Story 2)

$F_{psl_1} = 170.8\cdot plf$ (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)

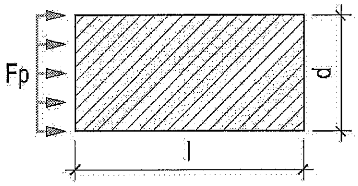
$$F_{psdmin_x} := 0.2\cdot S_{ds}\cdot I_e\cdot\left(\frac{wd_x}{d_x} + 2\cdot ww_x\right)$$

(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)

$$F_{psd_x} := \begin{array}{|l} F_{psdmin_x} \ \text{ if } \ F_{psd_x} < F_{psdmin_x} \\ F_{psdmax_x} \ \text{ if } \ F_{psd_x} > F_{psdmax_x} \\ F_{psd_x} \ \text{ otherwise} \end{array}$$



$F_{psd_4} = ■\cdot plf$ (Story 4)

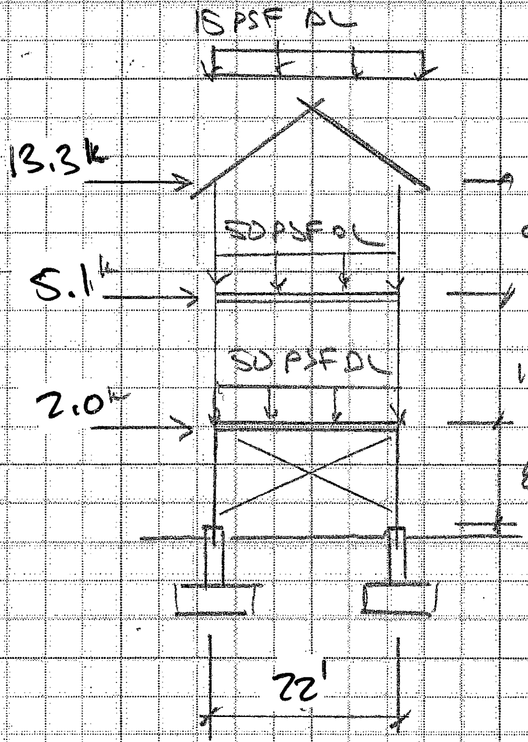
$F_{psd_3} = 673.9\cdot plf$ (Story 3)

$F_{psd_2} = 380.4\cdot plf$ (Story 2)

$F_{psd_1} = 362.0\cdot plf$ (Story 1)

GLOBAL OVERTURNING

SHORT DIRECTION WILL CONTROL



OVERTURNING (ASD)

$$OTM = 0.7\left[(13.3^{k})(27') + 5.1^{k}(18') + 2.0^{k}(8')\right] = 327^{k\text{-}FT}$$

RIGHTING FORCE (ASD)

$$RM = 0.6(15\,PSF + 50\,PSF + 50\,PSF)(22')(63')\left(\frac{22'}{2}\right)/1000 = 1052^{k\text{-}FT}$$

BUILDING STABLE

ASD FORCE ON FOOTING $= 327^{k\text{-}FT}/22' = +/- 14.9^{k}$

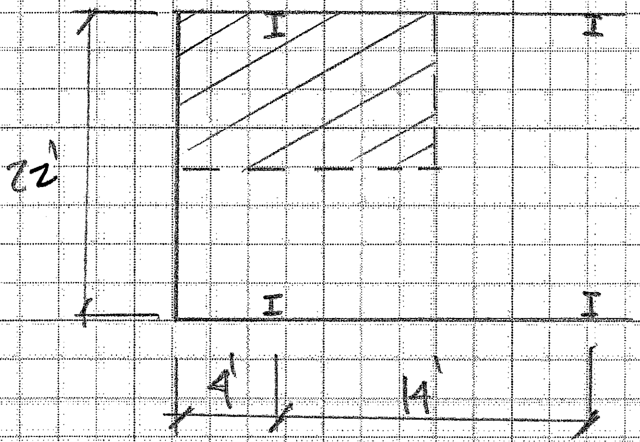
JOB ____________

SHEET NO ________ OF ________

CALCULATED BY ________ DATE ________

FOUNDATIONS

WORST CASE TRIBUTARY TO COLUMN



AREA $= 11'(4'+7')$
$= 121 \text{ ft}^2$

$$P = \frac{\left[15\text{psf} + 192\text{psf} + 2(50\text{psf} + 40\text{psf})\right]}{1000} 121 \text{ ft}^2 = 47^k$$

ADD 15^k FROM OVERTURNING

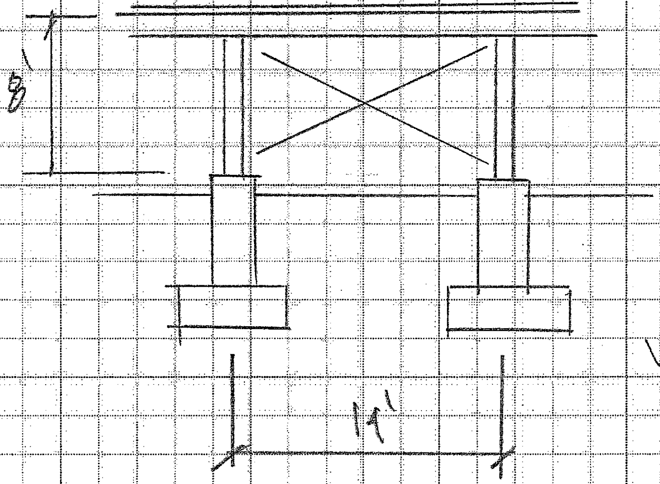
$$47^k + 15^k = 62^k$$

ALLOWABLE SOIL PRESSURE = 2600 PSF

$(62^k / 2.6\text{ksf})^{1/2} = 4.9'$ USE $6' \times 6' \times 14''$

CROSS BRACING BETWEEN LOWER LEVEL AND FOUNDATION

(4) BRACES IN SHORT DIRECTION
(4) BRACES IN LONG DIRECTION.



UPPER LEVEL BASE SHEAR FOR $R=6.5$
$=20.4^k$

R FOR ORDINARY STEEL CONCENTRICALLY BRACED FRAMES $=3.25$

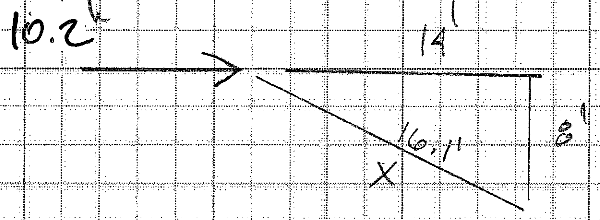
$V=20.4^k\left(\frac{6.5}{3.25}\right)=40.8^k$

$40.8^k/4$ BAYS $=10.2^k$ PER BAY

JOB ______________________

SHEET NO ____________ OF ____________

CALCULATED BY ____________ DATE ____________



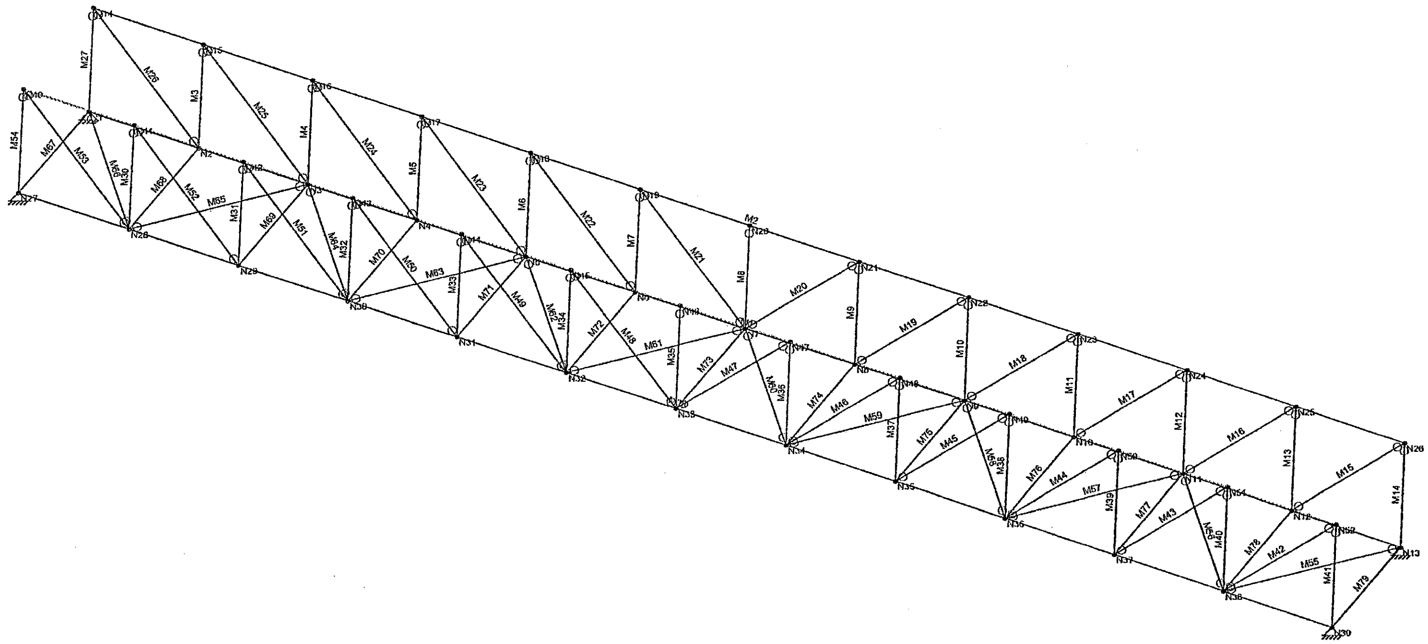
FORCE IN ROD

$$\frac{X}{16.1'} = \frac{10.2^k}{14'} \qquad X = 12^k$$

$$A_{REQ'D} = \frac{12^k}{0.9\,(36\,ksi)} = 0.37\,in^2$$

USE 1 1/4" DIA GR 36 ROD, $A = 1.23\,IN^2$

TO MATCH WORST CASE
ON OTHER MODELS



ACCESS RAMP / BRIDGE
48' MAX. SPAN

(Global) Model Settings

Display Sections for Member Calcs	5
Max Internal Sections for Member Calcs	97
Include Shear Deformation?	Yes
Increase Nailing Capacity for Wind?	Yes
Include Warping?	Yes
Trans Load Btwn Intersecting Wood Wall?	Yes
Area Load Mesh (in^2)	144
Merge Tolerance (in)	.12
P-Delta Analysis Tolerance	0.50%
Include P-Delta for Walls?	Yes
Automatically Iterate Stiffness for Walls?	Yes
Max Iterations for Wall Stiffness	3
Gravity Acceleration (ft/sec^2)	32.2
Wall Mesh Size (in)	24
Eigensolution Convergence Tol. (1.E-)	4
Vertical Axis	Y
Global Member Orientation Plane	XZ
Static Solver	Sparse Accelerated
Dynamic Solver	Accelerated Solver

Hot Rolled Steel Code	AISC 14th(360-10): LRFD
Adjust Stiffness?	Yes(Iterative)
RISAConnection Code	AISC 14th(360-10): LRFD
Cold Formed Steel Code	None
Wood Code	None
Wood Temperature	< 100F
Concrete Code	None
Masonry Code	None
Aluminum Code	None - Building

Number of Shear Regions	4
Region Spacing Increment (in)	4
Biaxial Column Method	Exact Integration
Parme Beta Factor (PCA)	.65
Concrete Stress Block	Rectangular
Use Cracked Sections?	Yes
Use Cracked Sections Slab?	Yes
Bad Framing Warnings?	No
Unused Force Warnings?	Yes
Min 1 Bar Diam. Spacing?	No
Concrete Rebar Set	REBAR_SET_ASTMA615
Min % Steel for Column	1
Max % Steel for Column	8

(Global) Model Settings, Continued

Seismic Code	None
Seismic Base Elevation (ft)	Not Entered
Add Base Weight?	Yes
Ct X	.02
Ct Z	.02
T X (sec)	Not Entered
T Z (sec)	Not Entered
R X	3
R Z	3

Hot Rolled Steel Properties

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

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design ...	A [in2]	Iyy [in4]	Izz [in4]	J [in4]
1	ContCho...	L4x4x6	Beam	Wide Flange	A992	Typical	2.86	4.32	4.32	.141
2	VertGuard	LL3.5x3.5x6x3	Beam	Wide Flange	A992	Typical	5	12.8	5.72	.246
3	CrossBr...	L3.5x3.5x6	Beam	Wide Flange	A992	Typical	2.5	2.86	2.86	.123
4	BottomC...	C6x10.5	Beam	Channel	A992	Typical	3.07	.86	15.1	.128
5	DiagChord	L3.5x3.5x4	None	None	A992	Typical	1.7	2	2	.039
6	End Me...	HSS3.5x3.5x6	None	None	A992	Typical	4.09	6.49	6.49	11.2

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
1	N1	0	0	0	0	
2	N2	4	0	0	0	
3	N3	8	0	0	0	
4	N4	12	0	0	0	
5	N5	16	0	0	0	
6	N6	20	0	0	0	
7	N7	24	0	0	0	
8	N8	28	0	0	0	
9	N9	32	0	0	0	
10	N10	36	0	0	0	
11	N11	40	0	0	0	
12	N12	44	0	0	0	
13	N13	48	0	0	0	
14	N14	0	4.5	0	0	
15	N15	4	4.5	0	0	
16	N16	8	4.5	0	0	
17	N17	12	4.5	0	0	
18	N18	16	4.5	0	0	
19	N19	20	4.5	0	0	
20	N20	24	4.5	0	0	
21	N21	28	4.5	0	0	
22	N22	32	4.5	0	0	

Joint Coordinates and Temperatures (Continued)

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
23	N23	36	4.5	0	0	
24	N24	40	4.5	0	0	
25	N25	44	4.5	0	0	
26	N26	48	4.5	0	0	
27	N27	0	0	5	0	
28	N28	4	0	5	0	
29	N29	8	0	5	0	
30	N30	12	0	5	0	
31	N31	16	0	5	0	
32	N32	20	0	5	0	
33	N33	24	0	5	0	
34	N34	28	0	5	0	
35	N35	32	0	5	0	
36	N36	36	0	5	0	
37	N37	40	0	5	0	
38	N38	44	0	5	0	
39	N39	48	0	5	0	
40	N40	0	4.5	5	0	
41	N41	4	4.5	5	0	
42	N42	8	4.5	5	0	
43	N43	12	4.5	5	0	
44	N44	16	4.5	5	0	
45	N45	20	4.5	5	0	
46	N46	24	4.5	5	0	
47	N47	28	4.5	5	0	
48	N48	32	4.5	5	0	
49	N49	36	4.5	5	0	
50	N50	40	4.5	5	0	
51	N51	44	4.5	5	0	
52	N52	48	4.5	5	0	

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]
1	N1	Reaction	Reaction	Fixed			
2	N13	Reaction	Reaction	Fixed			
3	N27	Reaction	Reaction	Fixed			
4	N39	Reaction	Reaction	Fixed			

Member Primary Data

	Label	I Joint	J Joint	K Joint	Rotate(...	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N13			ContChord	Beam	Wide Flange	A992	Typical
2	M2	N14	N26			ContChord	Beam	Wide Flange	A992	Typical
3	M3	N2	N15		90	VertGuard	Beam	Wide Flange	A992	Typical
4	M4	N3	N16		90	VertGuard	Beam	Wide Flange	A992	Typical
5	M5	N4	N17		90	VertGuard	Beam	Wide Flange	A992	Typical
6	M6	N5	N18		90	VertGuard	Beam	Wide Flange	A992	Typical
7	M7	N6	N19		90	VertGuard	Beam	Wide Flange	A992	Typical
8	M8	N7	N20		90	VertGuard	Beam	Wide Flange	A992	Typical
9	M9	N8	N21		90	VertGuard	Beam	Wide Flange	A992	Typical
10	M10	N9	N22		90	VertGuard	Beam	Wide Flange	A992	Typical
11	M11	N10	N23		90	VertGuard	Beam	Wide Flange	A992	Typical
12	M12	N11	N24		90	VertGuard	Beam	Wide Flange	A992	Typical
13	M13	N12	N25		90	VertGuard	Beam	Wide Flange	A992	Typical
14	M14	N13	N26			End Members	None	None	A992	Typical

Member Primary Data (Continued)

	Label	I Joint	J Joint	K Joint	Rotate(...	Section/Shape	Type	Design List	Material	Design Rules
15	M15	N26	N12			CrossBrace	Beam	Wide Flange	A992	Typical
16	M16	N25	N11			CrossBrace	Beam	Wide Flange	A992	Typical
17	M17	N24	N10			CrossBrace	Beam	Wide Flange	A992	Typical
18	M18	N23	N9			CrossBrace	Beam	Wide Flange	A992	Typical
19	M19	N22	N8			CrossBrace	Beam	Wide Flange	A992	Typical
20	M20	N21	N7			CrossBrace	Beam	Wide Flange	A992	Typical
21	M21	N7	N19			CrossBrace	Beam	Wide Flange	A992	Typical
22	M22	N18	N6			CrossBrace	Beam	Wide Flange	A992	Typical
23	M23	N5	N17			CrossBrace	Beam	Wide Flange	A992	Typical
24	M24	N4	N16			CrossBrace	Beam	Wide Flange	A992	Typical
25	M25	N3	N15			CrossBrace	Beam	Wide Flange	A992	Typical
26	M26	N2	N14			CrossBrace	Beam	Wide Flange	A992	Typical
27	M27	N1	N14			End Members	None	None	A992	Typical
28	M28	N27	N39			ContChord	Beam	Wide Flange	A992	Typical
29	M29	N40	N52			ContChord	Beam	Wide Flange	A992	Typical
30	M30	N28	N41		270	VertGuard	Beam	Wide Flange	A992	Typical
31	M31	N29	N42		270	VertGuard	Beam	Wide Flange	A992	Typical
32	M32	N30	N43		270	VertGuard	Beam	Wide Flange	A992	Typical
33	M33	N31	N44		270	VertGuard	Beam	Wide Flange	A992	Typical
34	M34	N32	N45		270	VertGuard	Beam	Wide Flange	A992	Typical
35	M35	N33	N46		270	VertGuard	Beam	Wide Flange	A992	Typical
36	M36	N34	N47		270	VertGuard	Beam	Wide Flange	A992	Typical
37	M37	N35	N48		270	VertGuard	Beam	Wide Flange	A992	Typical
38	M38	N36	N49		270	VertGuard	Beam	Wide Flange	A992	Typical
39	M39	N37	N50		270	VertGuard	Beam	Wide Flange	A992	Typical
40	M40	N38	N51		270	VertGuard	Beam	Wide Flange	A992	Typical
41	M41	N39	N52			End Members	None	None	A992	Typical
42	M42	N52	N38			CrossBrace	Beam	Wide Flange	A992	Typical
43	M43	N51	N37			CrossBrace	Beam	Wide Flange	A992	Typical
44	M44	N50	N36			CrossBrace	Beam	Wide Flange	A992	Typical
45	M45	N49	N35			CrossBrace	Beam	Wide Flange	A992	Typical
46	M46	N48	N34			CrossBrace	Beam	Wide Flange	A992	Typical
47	M47	N47	N33			CrossBrace	Beam	Wide Flange	A992	Typical
48	M48	N33	N45			CrossBrace	Beam	Wide Flange	A992	Typical
49	M49	N44	N32			CrossBrace	Beam	Wide Flange	A992	Typical
50	M50	N31	N43			CrossBrace	Beam	Wide Flange	A992	Typical
51	M51	N30	N42			CrossBrace	Beam	Wide Flange	A992	Typical
52	M52	N29	N41			CrossBrace	Beam	Wide Flange	A992	Typical
53	M53	N28	N40			CrossBrace	Beam	Wide Flange	A992	Typical
54	M54	N27	N40			End Members	None	None	A992	Typical
55	M55	N13	N38			DiagChord	None	None	A992	Typical
56	M56	N38	N11			DiagChord	None	None	A992	Typical
57	M57	N11	N36			DiagChord	None	None	A992	Typical
58	M58	N36	N9			DiagChord	None	None	A992	Typical
59	M59	N9	N34			DiagChord	None	None	A992	Typical
60	M60	N34	N7			DiagChord	None	None	A992	Typical
61	M61	N7	N32			DiagChord	None	None	A992	Typical
62	M62	N32	N5			DiagChord	None	None	A992	Typical
63	M63	N5	N30			DiagChord	None	None	A992	Typical
64	M64	N30	N3			DiagChord	None	None	A992	Typical
65	M65	N3	N28			DiagChord	None	None	A992	Typical
66	M66	N28	N1			DiagChord	None	None	A992	Typical
67	M67	N1	N27			End Members	None	None	A992	Typical
68	M68	N2	N28			BottomChord	Beam	Channel	A992	Typical
69	M69	N3	N29			BottomChord	Beam	Channel	A992	Typical
70	M70	N4	N30			BottomChord	Beam	Channel	A992	Typical
71	M71	N5	N31			BottomChord	Beam	Channel	A992	Typical

Member Primary Data (Continued)

	Label	I Joint	J Joint	K Joint	Rotate(...	Section/Shape	Type	Design List	Material	Design Rules
72	M72	N6	N32			BottomChord	Beam	Channel	A992	Typical
73	M73	N7	N33			BottomChord	Beam	Channel	A992	Typical
74	M74	N8	N34			BottomChord	Beam	Channel	A992	Typical
75	M75	N9	N35			BottomChord	Beam	Channel	A992	Typical
76	M76	N10	N36			BottomChord	Beam	Channel	A992	Typical
77	M77	N11	N37			BottomChord	Beam	Channel	A992	Typical
78	M78	N12	N38			BottomChord	Beam	Channel	A992	Typical
79	M79	N13	N39			End Members	None	None	A992	Typical

Member Advanced Data

	Label	I Release	J Release	I Offset[in]	J Offset[in]	T/C Only	Physical	Analysis Offset[in]	Inactive	Seismic Desig...
1	M1						Yes			None
2	M2						Yes			None
3	M3		BenPIN				Yes			None
4	M4		BenPIN				Yes			None
5	M5		BenPIN				Yes			None
6	M6		BenPIN				Yes			None
7	M7		BenPIN				Yes			None
8	M8		BenPIN				Yes			None
9	M9		BenPIN				Yes			None
10	M10		BenPIN				Yes			None
11	M11		BenPIN				Yes			None
12	M12		BenPIN				Yes			None
13	M13		BenPIN				Yes			None
14	M14		BenPIN				Yes			None
15	M15	BenPIN	BenPIN				Yes			None
16	M16	BenPIN	BenPIN				Yes			None
17	M17	BenPIN	BenPIN				Yes			None
18	M18	BenPIN	BenPIN				Yes			None
19	M19	BenPIN	BenPIN				Yes			None
20	M20	BenPIN	BenPIN				Yes			None
21	M21	BenPIN	BenPIN				Yes			None
22	M22	BenPIN	BenPIN				Yes			None
23	M23	BenPIN	BenPIN				Yes			None
24	M24	BenPIN	BenPIN				Yes			None
25	M25	BenPIN	BenPIN				Yes			None
26	M26	BenPIN	BenPIN				Yes			None
27	M27		BenPIN				Yes			None
28	M28						Yes			None
29	M29						Yes			None
30	M30		BenPIN				Yes			None
31	M31		BenPIN				Yes			None
32	M32		BenPIN				Yes			None
33	M33		BenPIN				Yes			None
34	M34		BenPIN				Yes			None
35	M35		BenPIN				Yes			None
36	M36		BenPIN				Yes			None
37	M37		BenPIN				Yes			None
38	M38		BenPIN				Yes			None
39	M39		BenPIN				Yes			None
40	M40		BenPIN				Yes			None
41	M41		BenPIN				Yes			None
42	M42	BenPIN	BenPIN				Yes			None
43	M43	BenPIN	BenPIN				Yes			None
44	M44	BenPIN	BenPIN				Yes			None

Member Advanced Data (Continued)

	Label	I Release	J Release	I Offset[in]	J Offset[in]	T/C Only	Physical	Analysis Offset[in]	Inactive	Seismic Desig...
45	M45	BenPIN	BenPIN				Yes			None
46	M46	BenPIN	BenPIN				Yes			None
47	M47	BenPIN	BenPIN				Yes			None
48	M48	BenPIN	BenPIN				Yes			None
49	M49	BenPIN	BenPIN				Yes			None
50	M50	BenPIN	BenPIN				Yes			None
51	M51	BenPIN	BenPIN				Yes			None
52	M52	BenPIN	BenPIN				Yes			None
53	M53	BenPIN	BenPIN				Yes			None
54	M54		BenPIN				Yes			None
55	M55	BenPIN	BenPIN				Yes			None
56	M56	BenPIN	BenPIN				Yes			None
57	M57	BenPIN	BenPIN				Yes			None
58	M58	BenPIN	BenPIN				Yes			None
59	M59	BenPIN	BenPIN				Yes			None
60	M60	BenPIN	BenPIN				Yes			None
61	M61	BenPIN	BenPIN				Yes			None
62	M62	BenPIN	BenPIN				Yes			None
63	M63	BenPIN	BenPIN				Yes			None
64	M64	BenPIN	BenPIN				Yes			None
65	M65	BenPIN	BenPIN				Yes			None
66	M66	BenPIN	BenPIN				Yes			None
67	M67						Yes			None
68	M68						Yes			None
69	M69						Yes			None
70	M70						Yes			None
71	M71						Yes			None
72	M72						Yes			None
73	M73						Yes			None
74	M74						Yes			None
75	M75						Yes			None
76	M76						Yes			None
77	M77						Yes			None
78	M78						Yes			None
79	M79						Yes			None

Hot Rolled Steel Design Parameters

	Label	Shape	Length[ft]	Lbyy[ft]	Lbzz[ft]	Lcomp top[ft]	Lcomp bot[ft]	L-torqu...	Kyy	Kzz	Cb	Function
1	M1	ContChord	48	4	4	4	4					Lateral
2	M2	ContChord	48	4	4	4	4					Lateral
3	M3	VertGuard	4.5									Lateral
4	M4	VertGuard	4.5									Lateral
5	M5	VertGuard	4.5									Lateral
6	M6	VertGuard	4.5									Lateral
7	M7	VertGuard	4.5									Lateral
8	M8	VertGuard	4.5									Lateral
9	M9	VertGuard	4.5									Lateral
10	M10	VertGuard	4.5									Lateral
11	M11	VertGuard	4.5									Lateral
12	M12	VertGuard	4.5									Lateral
13	M13	VertGuard	4.5									Lateral
14	M14	End Membe...	4.5									Lateral
15	M15	CrossBrace	6.021									Lateral
16	M16	CrossBrace	6.021									Lateral
17	M17	CrossBrace	6.021									Lateral

Hot Rolled Steel Design Parameters (Continued)

	Label	Shape	Length[ft]	Lbyy[ft]	Lbzz[ft]	Lcomp top[ft]	Lcomp bot[ft]	L-torqu...	Kyy	Kzz	Cb	Function
18	M18	CrossBrace	6.021									Lateral
19	M19	CrossBrace	6.021									Lateral
20	M20	CrossBrace	6.021									Lateral
21	M21	CrossBrace	6.021									Lateral
22	M22	CrossBrace	6.021									Lateral
23	M23	CrossBrace	6.021									Lateral
24	M24	CrossBrace	6.021									Lateral
25	M25	CrossBrace	6.021									Lateral
26	M26	CrossBrace	6.021									Lateral
27	M27	End Membe...	4.5									Lateral
28	M28	ContChord	48	4	4	4	4					Lateral
29	M29	ContChord	48	4	4	4	4					Lateral
30	M30	VertGuard	4.5									Lateral
31	M31	VertGuard	4.5									Lateral
32	M32	VertGuard	4.5									Lateral
33	M33	VertGuard	4.5									Lateral
34	M34	VertGuard	4.5									Lateral
35	M35	VertGuard	4.5									Lateral
36	M36	VertGuard	4.5									Lateral
37	M37	VertGuard	4.5									Lateral
38	M38	VertGuard	4.5									Lateral
39	M39	VertGuard	4.5									Lateral
40	M40	VertGuard	4.5									Lateral
41	M41	End Membe...	4.5									Lateral
42	M42	CrossBrace	6.021									Lateral
43	M43	CrossBrace	6.021									Lateral
44	M44	CrossBrace	6.021									Lateral
45	M45	CrossBrace	6.021									Lateral
46	M46	CrossBrace	6.021									Lateral
47	M47	CrossBrace	6.021									Lateral
48	M48	CrossBrace	6.021									Lateral
49	M49	CrossBrace	6.021									Lateral
50	M50	CrossBrace	6.021									Lateral
51	M51	CrossBrace	6.021									Lateral
52	M52	CrossBrace	6.021									Lateral
53	M53	CrossBrace	6.021									Lateral
54	M54	End Membe...	4.5									Lateral
55	M55	DiagChord	6.403									Lateral
56	M56	DiagChord	6.403									Lateral
57	M57	DiagChord	6.403									Lateral
58	M58	DiagChord	6.403									Lateral
59	M59	DiagChord	6.403									Lateral
60	M60	DiagChord	6.403									Lateral
61	M61	DiagChord	6.403									Lateral
62	M62	DiagChord	6.403									Lateral
63	M63	DiagChord	6.403									Lateral
64	M64	DiagChord	6.403									Lateral
65	M65	DiagChord	6.403									Lateral
66	M66	DiagChord	6.403									Lateral
67	M67	End Membe...	5									Lateral
68	M68	BottomChord	5									Lateral
69	M69	BottomChord	5									Lateral
70	M70	BottomChord	5									Lateral
71	M71	BottomChord	5									Lateral
72	M72	BottomChord	5									Lateral
73	M73	BottomChord	5									Lateral
74	M74	BottomChord	5									Lateral

Envelope Joint Displacements (Continued)

	Joint		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotati...	LC	Y Rotation...	LC	Z Rotation [rad]	LC
17	N9	max	.03	2	-.145	1	.015	2	1.417e-03	2	7.282e-05	2	2.405e-03	2
18		min	.005	1	-.833	2	.003	1	1.466e-04	1	1.265e-05	1	4.175e-04	1
19	N10	max	.037	2	-.117	1	.01	2	1.637e-03	2	3.722e-04	2	3.348e-03	2
20		min	.006	1	-.674	2	.002	1	1.68e-04	1	6.448e-05	1	5.811e-04	1
21	N11	max	.036	2	-.082	1	.009	2	1.413e-03	2	1.151e-04	2	4.005e-03	2
22		min	.006	1	-.474	2	.002	1	1.503e-04	1	2.e-05	1	6.954e-04	1
23	N12	max	.024	2	-.042	1	.018	2	1.568e-03	2	5.466e-04	2	4.349e-03	2
24		min	.004	1	-.242	2	.003	1	1.663e-04	1	9.495e-05	1	7.541e-04	1
25	N13	max	0	2	0	1	0	1	1.36e-03	2	2.729e-04	2	4.319e-03	2
26		min	0	1	0	2	0	1	1.807e-04	1	4.248e-05	1	7.628e-04	1
27	N14	max	.187	2	-.002	1	.08	2	1.716e-03	2	-1.679e-05	1	-9.082e-04	1
28		min	.033	1	-.011	2	.01	1	2.159e-04	1	-2.955e-04	2	-5.026e-03	2
29	N15	max	.175	2	-.044	1	.092	2	1.681e-03	2	-7.472e-06	1	-8.482e-04	1
30		min	.03	1	-.249	2	.011	1	2.061e-04	1	-1.407e-04	2	-4.933e-03	2
31	N16	max	.152	2	-.083	1	.093	2	1.723e-03	2	1.358e-05	1	-7.888e-04	1
32		min	.026	1	-.48	2	.011	1	2.069e-04	1	-2.269e-06	2	-4.529e-03	2
33	N17	max	.12	2	-.118	1	.095	2	1.68e-03	2	1.671e-06	1	-6.496e-04	1
34		min	.021	1	-.678	2	.011	1	1.952e-04	1	-9.535e-06	2	-3.745e-03	2
35	N18	max	.083	2	-.145	1	.094	2	1.687e-03	2	-8.339e-07	1	-4.675e-04	1
36		min	.014	1	-.835	2	.01	1	1.919e-04	1	-4.26e-06	2	-2.69e-03	2
37	N19	max	.042	2	-.163	1	.094	2	1.645e-03	2	7.308e-05	2	-2.702e-04	1
38		min	.007	1	-.935	2	.01	1	1.843e-04	1	9.794e-06	1	-1.555e-03	2
39	N20	max	0	1	-.17	1	.089	2	1.645e-03	2	0	2	0	1
40		min	0	2	-.978	2	.01	1	1.843e-04	1	0	1	0	2
41	N21	max	-.007	1	-.163	1	.094	2	1.645e-03	2	-9.794e-06	1	1.555e-03	2
42		min	-.042	2	-.935	2	.01	1	1.843e-04	1	-7.308e-05	2	2.702e-04	1
43	N22	max	-.014	1	-.145	1	.094	2	1.687e-03	2	4.26e-06	2	2.69e-03	2
44		min	-.083	2	-.835	2	.01	1	1.919e-04	1	8.339e-07	1	4.675e-04	1
45	N23	max	-.021	1	-.118	1	.095	2	1.68e-03	2	9.535e-06	2	3.745e-03	2
46		min	-.12	2	-.678	2	.011	1	1.952e-04	1	-1.671e-06	1	6.496e-04	1
47	N24	max	-.026	1	-.083	1	.093	2	1.723e-03	2	2.269e-06	2	4.529e-03	2
48		min	-.152	2	-.48	2	.011	1	2.069e-04	1	-1.358e-05	1	7.888e-04	1
49	N25	max	-.03	1	-.044	1	.092	2	1.681e-03	2	1.407e-04	2	4.933e-03	2
50		min	-.175	2	-.249	2	.011	1	2.061e-04	1	7.472e-06	1	8.482e-04	1
51	N26	max	-.033	1	-.002	1	.08	2	1.716e-03	2	2.955e-04	2	5.026e-03	2
52		min	-.187	2	-.011	2	.01	1	2.159e-04	1	1.679e-05	1	9.082e-04	1
53	N27	max	0	1	0	1	0	1	-1.814e-...	1	-4.899e-05	1	-7.681e-04	1
54		min	0	2	0	2	0	1	-1.413e-...	2	-2.909e-04	2	-4.38e-03	2
55	N28	max	-.004	1	-.043	1	.018	2	-1.438e-...	1	-9.383e-05	1	-7.562e-04	1
56		min	-.024	2	-.246	2	.003	1	-1.291e-...	2	-5.438e-04	2	-4.374e-03	2
57	N29	max	-.006	1	-.083	1	.01	2	-1.513e-...	1	-1.986e-05	1	-6.97e-04	1
58		min	-.036	2	-.477	2	.002	1	-1.45e-03	2	-1.135e-04	2	-4.027e-03	2
59	N30	max	-.006	1	-.118	1	.01	2	-1.322e-...	1	-6.404e-05	1	-5.819e-04	1
60		min	-.037	2	-.681	2	.002	1	-1.242e-...	2	-3.702e-04	2	-3.362e-03	2
61	N31	max	-.005	1	-.145	1	.015	2	-1.529e-...	1	-1.241e-05	1	-4.18e-04	1
62		min	-.029	2	-.837	2	.003	1	-1.453e-...	2	-7.168e-05	2	-2.414e-03	2
63	N32	max	-.003	1	-.163	1	.005	2	-1.299e-...	1	-3.368e-05	1	-2.293e-04	1
64		min	-.017	2	-.942	2	0	1	-1.217e-...	2	-1.939e-04	2	-1.323e-03	2
65	N33	max	0	1	-.17	1	.018	2	-1.565e-...	1	0	2	0	1
66		min	0	2	-.982	2	.003	1	-1.467e-...	2	0	1	0	2
67	N34	max	.017	2	-.163	1	.005	2	-1.299e-...	1	1.939e-04	2	1.323e-03	2
68		min	.003	1	-.942	2	0	1	-1.217e-...	2	3.368e-05	1	2.293e-04	1
69	N35	max	.029	2	-.145	1	.015	2	-1.529e-...	1	7.168e-05	2	2.414e-03	2
70		min	.005	1	-.837	2	.003	1	-1.453e-...	2	1.241e-05	1	4.18e-04	1
71	N36	max	.037	2	-.118	1	.01	2	-1.322e-...	1	3.702e-04	2	3.362e-03	2
72		min	.006	1	-.681	2	.002	1	-1.242e-...	2	6.404e-05	1	5.819e-04	1
73	N37	max	.036	2	-.083	1	.01	2	-1.513e-...	1	1.135e-04	2	4.027e-03	2

1" MAX DEFLECTION

Envelope Joint Displacements (Continued)

	Joint		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotati...	LC	Y Rotation...	LC	Z Rotation [rad]	LC
74		min	.006	1	-.477	2	.002	1	-1.45e-03	2	1.986e-05	1	6.97e-04	1
75	N38	max	.024	2	-.043	1	.018	2	-1.438e-...	1	5.438e-04	2	4.374e-03	2
76		min	.004	1	-.246	2	.003	1	-1.291e-...	2	9.383e-05	1	7.562e-04	1
77	N39	max	0	2	0	1	0	1	-1.814e-...	1	2.909e-04	2	4.38e-03	2
78		min	0	1	0	2	0	1	-1.413e-...	2	4.899e-05	1	7.681e-04	1
79	N40	max	.189	2	-.002	1	-.009	1	-1.111e-...	1	-4.103e-05	1	-9.243e-04	1
80		min	.033	1	-.012	2	-.071	2	-1.115e-...	2	-2.488e-04	2	-5.116e-03	2
81	N41	max	.176	2	-.044	1	-.006	1	-1.179e-...	1	-4.086e-05	1	-8.472e-04	1
82		min	.03	1	-.254	2	-.061	2	-1.155e-...	2	-1.048e-04	2	-4.948e-03	2
83	N42	max	.153	2	-.084	1	-.006	1	-1.102e-...	1	3.218e-05	2	-7.914e-04	1
84		min	.026	1	-.483	2	-.061	2	-1.121e-...	2	1.336e-06	1	-4.555e-03	2
85	N43	max	.121	2	-.119	1	-.006	1	-1.156e-...	1	1.983e-07	1	-6.501e-04	1
86		min	.021	1	-.685	2	-.061	2	-1.173e-...	2	-2.336e-07	2	-3.757e-03	2
87	N44	max	.083	2	-.146	1	-.006	1	-1.152e-...	1	1.607e-06	1	-4.684e-04	1
88		min	.014	1	-.84	2	-.061	2	-1.173e-...	2	-1.316e-06	2	-2.702e-03	2
89	N45	max	.042	2	-.163	1	-.006	1	-1.211e-...	1	6.617e-05	2	-2.697e-04	1
90		min	.007	1	-.943	2	-.062	2	-1.218e-...	2	1.101e-05	1	-1.553e-03	2
91	N46	max	0	1	-.17	1	-.006	1	-1.211e-...	1	0	2	0	1
92		min	0	2	-.983	2	-.065	2	-1.218e-...	2	0	1	0	2
93	N47	max	-.007	1	-.163	1	-.006	1	-1.211e-...	1	-1.101e-05	1	1.553e-03	2
94		min	-.042	2	-.943	2	-.062	2	-1.218e-...	2	-6.617e-05	2	2.697e-04	1
95	N48	max	-.014	1	-.146	1	-.006	1	-1.152e-...	1	1.316e-06	2	2.702e-03	2
96		min	-.083	2	-.84	2	-.061	2	-1.173e-...	2	-1.607e-06	1	4.684e-04	1
97	N49	max	-.021	1	-.119	1	-.006	1	-1.156e-...	1	2.336e-07	2	3.757e-03	2
98		min	-.121	2	-.685	2	-.061	2	-1.173e-...	2	-1.983e-07	1	6.501e-04	1
99	N50	max	-.026	1	-.084	1	-.006	1	-1.102e-...	1	-1.336e-06	1	4.555e-03	2
100		min	-.153	2	-.483	2	-.061	2	-1.121e-...	2	-3.218e-05	2	7.914e-04	1
101	N51	max	-.03	1	-.044	1	-.006	1	-1.179e-...	1	1.048e-04	2	4.948e-03	2
102		min	-.176	2	-.254	2	-.061	2	-1.155e-...	2	4.086e-05	1	8.472e-04	1
103	N52	max	-.033	1	-.002	1	-.009	1	-1.111e-...	1	2.488e-04	2	5.116e-03	2
104		min	-.189	2	-.012	2	-.071	2	-1.115e-...	2	4.103e-05	1	9.243e-04	1

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

	LC	Member	Shape	UC Max	Loc[ft]	Shear...	Loc[ft]...		phi*P...	phi*P...	phi*M...	phi*M...	Cb	Eqn
1	1	M1	L4x4x6	.080	4	.002	48	y	97.503	128.7	6.109	12.844	1	H2-1
2	1	M2	L4x4x6	.111	22.5	.001	28	y	97.503	128.7	6.109	12.844	1	H2-1
3	1	M3	LL3.5x3...	.017	0	.001	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
4	1	M4	LL3.5x3...	.013	0	.000	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
5	1	M5	LL3.5x3...	.010	0	.000	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
6	1	M6	LL3.5x3...	.007	0	.000	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
7	1	M7	LL3.5x3...	.004	0	.000	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
8	1	M8	LL3.5x3...	.002	0	.000	0	y	186.7...	225	20.827	8.58	1	H1-1b
9	1	M9	LL3.5x3...	.004	0	.000	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
10	1	M10	LL3.5x3...	.007	0	.000	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
11	1	M11	LL3.5x3...	.010	0	.000	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
12	1	M12	LL3.5x3...	.013	0	.000	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
13	1	M13	LL3.5x3...	.017	0	.001	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
14	1	M14	HSS3.5...	.022	0	.001	0	y	160.91	184.05	17.588	17.588	1.6...	H1-...
15	1	M15	L3.5x3....	.048	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
16	1	M16	L3.5x3....	.042	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1...	H2-1
17	1	M17	L3.5x3....	.033	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
18	1	M18	L3.5x3....	.027	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
19	1	M19	L3.5x3....	.019	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
20	1	M20	L3.5x3....	.012	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1...	H2-1
21	1	M21	L3.5x3....	.012	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1

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

	LC	Member	Shape	UC Max	Loc[ft]	Shear..	Loc[ft]	...	phi*P...	phi*P...	phi*M...	phi*M...	Cb	Eqn
22	1	M22	L3.5x3....	.019	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
23	1	M23	L3.5x3....	.027	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
24	1	M24	L3.5x3....	.033	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
25	1	M25	L3.5x3....	.042	3.073	.001	0	y	49.639	112.5	4.639	9.175	1.1...	H2-1
26	1	M26	L3.5x3....	.048	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
27	1	M27	HSS3.5...	.022	0	.001	0	y	160.91	184.05	17.588	17.588	1.6...	H1-...
28	1	M28	L4x4x6	.080	44	.002	48	y	97.503	128.7	6.109	12.844	1	H2-1
29	1	M29	L4x4x6	.111	25.5	.001	28	y	97.503	128.7	6.109	12.844	1	H2-1
30	1	M30	LL3.5x3...	.017	0	.001	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
31	1	M31	LL3.5x3...	.014	0	.000	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
32	1	M32	LL3.5x3...	.010	0	.000	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
33	1	M33	LL3.5x3...	.007	0	.000	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
34	1	M34	LL3.5x3...	.004	0	.000	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
35	1	M35	LL3.5x3...	.001	0	.000	0	y	186.7...	225	20.827	13.728	1	H1-1b
36	1	M36	LL3.5x3...	.004	0	.000	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
37	1	M37	LL3.5x3...	.007	0	.000	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
38	1	M38	LL3.5x3...	.010	0	.000	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
39	1	M39	LL3.5x3...	.014	0	.000	0	z	186.7...	225	20.827	8.58	1.6...	H1-1b
40	1	M40	LL3.5x3...	.017	0	.001	0	z	186.7...	225	20.827	13.728	1.6...	H1-1b
41	1	M41	HSS3.5...	.022	0	.001	0	y	160.91	184.05	17.588	17.588	1.6...	H1-...
42	1	M42	L3.5x3....	.050	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
43	1	M43	L3.5x3....	.041	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
44	1	M44	L3.5x3....	.035	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1...	H2-1
45	1	M45	L3.5x3....	.026	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1...	H2-1
46	1	M46	L3.5x3....	.020	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
47	1	M47	L3.5x3....	.011	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1...	H2-1
48	1	M48	L3.5x3....	.011	3.073	.001	0	y	49.639	112.5	4.639	9.175	1.1...	H2-1
49	1	M49	L3.5x3....	.020	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
50	1	M50	L3.5x3....	.026	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
51	1	M51	L3.5x3....	.035	3.073	.001	0	y	49.639	112.5	4.639	9.175	1.1...	H2-1
52	1	M52	L3.5x3....	.041	3.073	.001	0	y	49.639	112.5	4.639	9.175	1.1...	H2-1
53	1	M53	L3.5x3....	.050	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1...	H2-1
54	1	M54	HSS3.5...	.022	0	.001	0	y	160.91	184.05	17.588	17.588	1.6...	H1-...
55	1	M55	L3.5x3....	.093	3.202	.007	0	y	30.289	76.5	3.313	5.593	1.1...	H2-1
56	1	M56	L3.5x3....	.091	3.202	.007	0	y	30.289	76.5	3.313	5.593	1.1...	H2-1
57	1	M57	L3.5x3....	.091	3.202	.007	0	y	30.289	76.5	3.313	5.593	1.1...	H2-1
58	1	M58	L3.5x3....	.091	3.202	.007	0	y	30.289	76.5	3.313	5.593	1.1...	H2-1
59	1	M59	L3.5x3....	.091	3.202	.007	0	y	30.289	76.5	3.313	5.593	1.1...	H2-1
60	1	M60	L3.5x3....	.091	3.202	.007	6.403	y	30.289	76.5	3.313	5.593	1.1...	H2-1
61	1	M61	L3.5x3....	.091	3.202	.007	6.403	y	30.289	76.5	3.313	5.593	1.1...	H2-1
62	1	M62	L3.5x3....	.091	3.202	.007	0	y	30.289	76.5	3.313	5.593	1.1...	H2-1
63	1	M63	L3.5x3....	.091	3.202	.007	6.403	y	30.289	76.5	3.313	5.593	1.1...	H2-1
64	1	M64	L3.5x3....	.091	3.202	.007	6.403	y	30.289	76.5	3.313	5.593	1.1...	H2-1
65	1	M65	L3.5x3....	.091	3.202	.007	6.403	y	30.289	76.5	3.313	5.593	1.1...	H2-1
66	1	M66	L3.5x3....	.093	3.202	.007	0	y	30.289	76.5	3.313	5.593	1.1...	H2-1
67	1	M67	HSS3.5...	.009	3.177	.003	5	y	155.9...	184.05	17.588	17.588	1.1...	H1-1b
68	1	M68	C6x10.5	.012	1.771	.005	0	y	53.985	138.15	3.373	21.675	1.1...	H1-1b
69	1	M69	C6x10.5	.010	2.865	.004	5	y	53.985	138.15	3.373	21.756	1.1...	H1-1b
70	1	M70	C6x10.5	.011	1.875	.004	0	y	53.985	138.15	3.373	21.835	1.1...	H1-1b
71	1	M71	C6x10.5	.010	2.917	.004	5	y	53.985	138.15	3.373	21.875	1.1...	H1-1b
72	1	M72	C6x10.5	.011	1.979	.004	0	y	53.985	138.15	3.373	21.874	1.1...	H1-1b
73	1	M73	C6x10.5	.011	2.865	.004	5	y	53.985	138.15	3.373	21.98	1.1...	H1-1b
74	1	M74	C6x10.5	.011	1.979	.004	0	y	53.985	138.15	3.373	21.874	1.1...	H1-1b
75	1	M75	C6x10.5	.010	2.917	.004	5	y	53.985	138.15	3.373	21.875	1.1...	H1-1b
76	1	M76	C6x10.5	.011	1.875	.004	0	y	53.985	138.15	3.373	21.835	1.1...	H1-1b
77	1	M77	C6x10.5	.010	2.865	.004	5	y	53.985	138.15	3.373	21.756	1.1...	H1-1b
78	1	M78	C6x10.5	.012	1.771	.005	0	y	53.985	138.15	3.373	21.675	1.1...	H1-1b

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

	LC	Member	Shape	UC Max	Loc[ft]	Shear..	Loc[ft]	...	phi*P...	phi*P...	phi*M...	phi*M...	Cb	Eqn
79	1	M79	HSS3.5...	.009	3.177	.003	5	y	155.9..	184.05	17.588	17.588	1.1..	H1-1b
80	2	M1	L4x4x6	.452	44	.009	48	y	97.503	128.7	6.109	12.844	1	H2-1
81	2	M2	L4x4x6	.643	24	.002	28	y	97.503	128.7	6.109	12.844	1	H2-1
82	2	M3	LL3.5x3...	.102	0	.004	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
83	2	M4	LL3.5x3...	.074	0	.003	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
84	2	M5	LL3.5x3...	.059	0	.003	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
85	2	M6	LL3.5x3...	.037	0	.002	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
86	2	M7	LL3.5x3...	.023	0	.002	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
87	2	M8	LL3.5x3...	.006	0	.000	0	y	186.7..	225	20.827	8.58	1	H1-1b
88	2	M9	LL3.5x3...	.023	0	.002	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
89	2	M10	LL3.5x3...	.037	0	.002	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
90	2	M11	LL3.5x3...	.059	0	.003	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
91	2	M12	LL3.5x3...	.074	0	.003	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
92	2	M13	LL3.5x3...	.102	0	.004	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
93	2	M14	HSS3.5...	.120	0	.004	0	y	160.91	184.05	17.588	17.588	1.6..	H1-...
94	2	M15	L3.5x3....	.235	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
95	2	M16	L3.5x3....	.207	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
96	2	M17	L3.5x3....	.151	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1..	H2-1
97	2	M18	L3.5x3....	.120	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
98	2	M19	L3.5x3....	.066	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
99	2	M20	L3.5x3....	.033	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
100	2	M21	L3.5x3....	.033	3.073	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
101	2	M22	L3.5x3....	.066	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1..	H2-1
102	2	M23	L3.5x3....	.120	3.073	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
103	2	M24	L3.5x3....	.151	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1..	H2-1
104	2	M25	L3.5x3....	.207	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1..	H2-1
105	2	M26	L3.5x3....	.235	3.073	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
106	2	M27	HSS3.5...	.120	0	.004	0	y	160.91	184.05	17.588	17.588	1.6..	H1-...
107	2	M28	L4x4x6	.458	44	.010	48	y	97.503	128.7	6.109	12.844	1	H2-1
108	2	M29	L4x4x6	.636	24.5	.001	8	y	97.503	128.7	6.109	12.844	1	H2-1
109	2	M30	LL3.5x3...	.093	0	.004	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
110	2	M31	LL3.5x3...	.082	0	.003	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
111	2	M32	LL3.5x3...	.056	0	.003	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
112	2	M33	LL3.5x3...	.041	0	.002	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
113	2	M34	LL3.5x3...	.021	0	.002	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
114	2	M35	LL3.5x3...	.003	0	.000	0	y	186.7..	225	20.827	13.728	1	H1-1b
115	2	M36	LL3.5x3...	.021	0	.002	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
116	2	M37	LL3.5x3...	.041	0	.002	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
117	2	M38	LL3.5x3...	.056	0	.003	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
118	2	M39	LL3.5x3...	.082	0	.003	0	z	186.7..	225	20.827	8.58	1.6..	H1-1b
119	2	M40	LL3.5x3...	.093	0	.004	0	z	186.7..	225	20.827	13.728	1.6..	H1-1b
120	2	M41	HSS3.5...	.126	0	.004	0	y	160.91	184.05	17.588	17.588	1.6..	H1-...
121	2	M42	L3.5x3....	.247	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
122	2	M43	L3.5x3....	.194	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
123	2	M44	L3.5x3....	.163	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
124	2	M45	L3.5x3....	.107	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
125	2	M46	L3.5x3....	.078	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1..	H2-1
126	2	M47	L3.5x3....	.020	2.948	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
127	2	M48	L3.5x3....	.020	3.073	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
128	2	M49	L3.5x3....	.078	2.948	.001	6.021	y	49.639	112.5	4.639	9.175	1.1..	H2-1
129	2	M50	L3.5x3....	.107	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1..	H2-1
130	2	M51	L3.5x3....	.163	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1..	H2-1
131	2	M52	L3.5x3....	.194	3.073	.001	0	y	49.639	112.5	4.639	9.175	1.1..	H2-1
132	2	M53	L3.5x3....	.247	3.073	.001	6.021	y	49.639	112.5	4.639	9.175	1.1..	H2-1
133	2	M54	HSS3.5...	.126	0	.004	0	y	160.91	184.05	17.588	17.588	1.6..	H1-...
134	2	M55	L3.5x3....	.930	3.202	.073	0	y	30.289	76.5	3.313	5.593	1.1..	H2-1
135	2	M56	L3.5x3....	.919	3.202	.073	0	y	30.289	76.5	3.313	5.593	1.1..	H2-1

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

	LC	Member	Shape	UC Max	Loc[ft]	Shear..	Loc[ft]	...	phi*P...	phi*P...	phi*M...	phi*M...	Cb	Eqn
136	2	M57	L3.5x3....	.917	3.202	.072	6.403	y	30.289	76.5	3.313	5.593	1.1..	H2-1
137	2	M58	L3.5x3....	.915	3.202	.072	0	y	30.289	76.5	3.313	5.593	1.1..	H2-1
138	2	M59	L3.5x3....	.916	3.202	.072	6.403	y	30.289	76.5	3.313	5.593	1.1..	H2-1
139	2	M60	L3.5x3....	.918	3.202	.072	0	y	30.289	76.5	3.313	5.593	1.1..	H2-1
140	2	M61	L3.5x3....	.918	3.202	.072	0	y	30.289	76.5	3.313	5.593	1.1..	H2-1
141	2	M62	L3.5x3....	.916	3.202	.072	0	y	30.289	76.5	3.313	5.593	1.1..	H2-1
142	2	M63	L3.5x3....	.915	3.202	.072	6.403	y	30.289	76.5	3.313	5.593	1.1..	H2-1
143	2	M64	L3.5x3....	.917	3.202	.072	0	y	30.289	76.5	3.313	5.593	1.1..	H2-1
144	2	M65	L3.5x3....	.919	3.202	.073	6.403	y	30.289	76.5	3.313	5.593	1.1..	H2-1
145	2	M66	L3.5x3....	.930	3.202	.073	6.403	y	30.289	76.5	3.313	5.593	1.1..	H2-1
146	2	M67	HSS3.5...	.068	3.177	.026	5	y	155.9..	184.05	17.588	17.588	1.1..	H1-1b
147	2	M68	C6x10.5	.103	1.875	.045	0	y	53.985	138.15	3.373	21.894	1.1..	H1-1b
148	2	M69	C6x10.5	.099	2.917	.045	5	y	53.985	138.15	3.373	21.913	1.1..	H1-1b
149	2	M70	C6x10.5	.102	1.927	.044	0	y	53.985	138.15	3.373	21.955	1.1..	H1-1b
150	2	M71	C6x10.5	.099	2.917	.044	5	y	53.985	138.15	3.373	21.98	1.1..	H1-1b
151	2	M72	C6x10.5	.102	2.031	.044	0	y	53.985	138.15	3.373	21.983	1.1..	H1-1b
152	2	M73	C6x10.5	.100	2.917	.044	5	y	53.985	138.15	3.373	22.07	1.1..	H1-1b
153	2	M74	C6x10.5	.102	2.031	.044	0	y	53.985	138.15	3.373	21.983	1.1..	H1-1b
154	2	M75	C6x10.5	.099	2.917	.044	5	y	53.985	138.15	3.373	21.98	1.1..	H1-1b
155	2	M76	C6x10.5	.102	1.927	.044	0	y	53.985	138.15	3.373	21.955	1.1..	H1-1b
156	2	M77	C6x10.5	.099	2.917	.045	5	y	53.985	138.15	3.373	21.913	1.1..	H1-1b
157	2	M78	C6x10.5	.103	1.875	.045	0	y	53.985	138.15	3.373	21.894	1.1..	H1-1b
158	2	M79	HSS3.5...	.068	3.177	.026	5	y	155.9..	184.05	17.588	17.588	1.1..	H1-1b

↖ ALL ≤ 1.0