SBC97

Wind Loads by SBC97

Input Parameters

Design Wind Speed = 110 mph
Enclosed

Variables for Enclosed/Part Encl.

$\text{Enclosed} \equiv 0$

$\text{PartEnclosed} \equiv 1$

Design Parameters

$V := \left|\text{in0}^{\langle 0 \rangle}\right| \cdot \text{mph}$ $\qquad V = 110\,\text{mph}$

$\text{Use} := 1.0$ Table 1606: Use factor

$\text{IntPressure} := \text{Enclosed}$

Geometry of Building: Mercedes homes

$h := 15 \cdot \text{ft}$ ht of building

$\theta := \text{atan}\left(\frac{5.5}{12}\right)$ $\qquad \theta = 24.624\,\text{deg}$ roof slope

$o := 1.0 \cdot \text{ft}$ overhang width

$W := 44\text{ft} + 2 \cdot o$ dimensions of building

$L := 50\text{ft} + 2 \cdot o$

$\Delta := 2\text{ft}$ Truss spacing

$o_g := 1.0 \cdot \text{ft}$ overhang width

Roof cover: Tile

$h_{wall} := 8 \cdot \text{ft}$ Height of Wall

Dead load of roof

$DL_{roof} := 9 \cdot \text{psf}$ Hip roof, Tile, trusses, underlayment (from SBC Appendix A)

$DL_{sheath} := (0.5 \cdot \text{in}) \cdot \left(\frac{0.4\text{psf}}{.125 \cdot \text{in}}\right)$ $\qquad DL_{sheath} = 1.6\,\text{psf}$

Dead load of 17 psf is composed of following: Truss/Sheathing (7 psf), Tile (10psf). If shingles are used, use 2 psf instead of 10 psf.

$L_{attic} := 30 \cdot \text{psf}$ SBC Table 1604.1

$L_{floor} := 40 \cdot \text{psf}$

$L_{roof} := 16 \cdot \text{psf}$

$DL_{wall} := \begin{pmatrix} 10 \\ 55 \end{pmatrix} \cdot \text{psf}$ Wood Frame wall weight
Masonry Wall Weight

$DL_{misc} := 15 \cdot \text{psf}$ Miscellaneous: Contents, carpet, cabinets, fixtures)

AREAS: Roof - Hip Roof

Vertical Projected Area: wind perpendicular to ridge

$h_{ridge} := \frac{W}{2} \cdot \tan(\theta)$ $\qquad h_{ridge} = 10.542\ ft$

$VPA_{\Gamma} := \frac{h_{ridge}}{2} \cdot [L + (L - W)]$ $\qquad VPA_{\Gamma} = 305.708\ ft^2$

Vertical Projected Area: wind parallel to ridge

$VPA_{ll} := \frac{W \cdot h_{ridge}}{2}$ $\qquad VPA_{ll} = 242.458\ ft^2$

Horizontal Projected Area:

$HPA := W \cdot L$ $\qquad HPA = 2392\ ft^2$

AREAS: Walls

Vertical Projected Area: : wind perpendicular to ridge - half of horizontal load transferred directly to foundation

$VPA_{wall\Gamma} := \frac{h_{wall}}{2} \cdot L$ $\qquad VPA_{wall\_ll} := \frac{h_{wall}}{2} \cdot W$

$VPA_{wall\Gamma} = 208\ ft^2$ $\qquad VPA_{wall\_ll} = 184\ ft^2$

Dynamic Wind Pressure

$$h_{min} := 15\cdot ft$$

$$q_h := \begin{cases} \left[\left[.00256\cdot V^2\cdot\left(\frac{h}{33\cdot ft}\right)^{\frac{2}{7}}\cdot\frac{slug}{2.15111\cdot ft^3}\right]\right] & \text{if } (h > h_{min}) \\ \left[.00256\cdot V^2\cdot\left(\frac{15\cdot ft}{33ft}\right)^{\frac{2}{7}}\right]\cdot\frac{slug}{2.15111\cdot ft^3} & \text{otherwise} \end{cases}$$

Dynamic Wind Pressure(Table 1606.2A)

$$q_h = 24.728\,psf$$

$$a := \min\left(\begin{pmatrix} 0.1\cdot W \\ 0.1\cdot L \\ 0.4\cdot h \end{pmatrix}\right) \quad a := \max\left(\begin{pmatrix} a \\ 0.04\cdot W \\ 0.04\cdot L \\ 3\cdot ft \end{pmatrix}\right) \quad a = 4.6\,ft$$

Edge zone

$$l_r := \frac{W}{2\cdot\cos(\theta)} \quad l_r = 25.301\,ft$$

length of top chord of truss

Internal Pressure coefficient

$$GC_{pi} := \begin{cases} \begin{pmatrix} 0 \\ 0 \end{pmatrix} & \text{if IntPressure = Enclosed} \\ \begin{pmatrix} -0.4 \\ 0.1 \end{pmatrix} & \text{if IntPressure = PartEnclosed} \\ \begin{pmatrix} -20 \\ 20 \end{pmatrix} & \text{otherwise} \end{cases} \qquad GC_{pi} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$$

External Pressure Coefficients: Components & Cladding

Limits of External Pressure Coefficients for each Zone in C&C loads
(first row neg coefficients, second row positive coefficents)

$$GCp_r := \begin{pmatrix} -1.2 & -1.1 \\ 0.7 & 0.5 \end{pmatrix} \qquad Alim_r := (10 \;\; 100)\cdot ft^2$$

SBC97: Figure 1606.2E Gable/Hip Roofs 10 deg <θ< 30 deg

$$GCp_{re} := \begin{pmatrix} -1.2 & -1.1 \\ 0.7 & 0.5 \end{pmatrix} \qquad Alim_{re} := (10 \;\; 100)\cdot ft^2$$

$$GCp_{si} := \begin{pmatrix} -1.4 & -1.2 \\ 0.7 & 0.5 \end{pmatrix} \qquad Alim_{si} := (10 \;\; 100)\cdot ft^2$$

$$GCp_{se} := \begin{pmatrix} -2.1 & -1.8 \\ 0.7 & 0.5 \end{pmatrix} \qquad Alim_{se} := (40 \;\; 100)\cdot ft^2$$

$$GCp_c := \begin{pmatrix} -2.7 & -1.8 \\ 0.7 & 0.5 \end{pmatrix} \qquad Alim_c := (10 \;\; 100)\cdot ft^2$$

$$GCp_w := \begin{pmatrix} -1.3 & -1.1 \\ 1.3 & 1.0 \end{pmatrix} \qquad Alim_w := (10 \;\; 500)\cdot ft^2$$

10SF neg 500SF neg
10SF pos 500SF pos

$$GCp_e := \begin{pmatrix} -1.5 & -1.1 \\ 1.3 & 1.0 \end{pmatrix} \qquad Alim_e := (10 \;\; 500)\cdot ft^2$$

$$\begin{pmatrix} r \\ re \\ si \\ se \\ c \\ w \\ e \end{pmatrix} \equiv \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \end{pmatrix}$$

$$slope_{GCp}(Zone) := \frac{\left(GCp_{Zone}\right)^{\langle 1\rangle} - \left(GCp_{Zone}\right)^{\langle 0\rangle}}{\log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right|}{ft^2}\right] - \log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right|}{ft^2}\right]}$$

$$GC_p(Area, Zone) := \begin{cases} \left(GCp_{Zone}\right)^{\langle 0\rangle} & \text{if } Area < \left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right| \\ \left(GCp_{Zone}\right)^{\langle 1\rangle} & \text{if } Area > \left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right| \\ \left(slope_{GCp}(Zone)\right)\cdot\left[\log\left(\frac{Area}{ft^2}\right) - \log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right|}{ft^2}\right]\right] + \left(GCp_{Zone}\right)^{\langle 0\rangle} & \text{otherwise} \end{cases}$$

For Example:

$$GC_p\left(10\cdot ft^2, c\right) = \begin{pmatrix} -2.7 \\ 0.7 \end{pmatrix} \qquad GC_p\left(200\cdot ft^2, e\right) = \begin{pmatrix} -1.194 \\ 1.07 \end{pmatrix} \qquad GC_p\left(100\cdot ft^2, r\right) = \begin{pmatrix} -1.1 \\ 0.5 \end{pmatrix}$$

$$GC_p\left(200\cdot ft^2, w\right) = \begin{pmatrix} -1.147 \\ 1.07 \end{pmatrix}$$

Window Design Pressure

Figure 1606.2C

The following input table was imported from an excel sheet that had a list of fens for this building. Each column represents the widht, height, area, and zone of each fen respectively.

Width Height $Size := 2$ $Zone := 3$ $Fraction := 4$

$$\begin{pmatrix} D309D01 \\ D311G01 \\ D608W01 \\ D508W01 \\ D508W01 \\ D508W01 \\ D510S01 \\ D408W01 \\ D408W01 \\ D308W01 \\ D308W01 \end{pmatrix}$$

Fen :=

	0	1	2	3	4
0	3	8	24	5	1
1	16	7	112	45	0.19
2	3	4	12	5	1
3	4	5	20	5	1
4	4	5	20	5	1
5	4	5	20	5	1
6	6	6.7	40.2	5	1
7	4	5	20	5	1
8	4	5	20	5	1
9	6	6	36	6	1
10	6	6	36	5	1
11					
12					

When Zone = 45, Fraction represents portion of fen in Zone e.

$rows(Fen) = 11$

$j := 0..rows(Fen) - 1$

$p_{wall} := q_h \cdot \left(GC_p\left(10 \cdot ft^2, w\right) + GC_{pi}\right)$ $\quad GC_p\left(10 \cdot ft^2, e\right) = \begin{pmatrix} -1.5 \\ 1.3 \end{pmatrix}$ $\quad p_{wall} = \begin{pmatrix} -32.147 \\ 32.147 \end{pmatrix} psf$

$$DP^{\langle j \rangle} := \begin{cases} q_h \cdot \left(GC_p\left(\overrightarrow{\left[\left(Fen^{\langle Size \rangle}\right)_j \cdot ft^2\right]}, \overrightarrow{\left(Fen^{\langle Zone \rangle}\right)_j}\right) + GC_{pi}\right) & \text{if } \left(Fen^{\langle Zone \rangle}\right)_j \neq 45 \\ \left[q_h \cdot \left(GC_p\left(\overrightarrow{\left[\left(Fen^{\langle Size \rangle}\right)_j \cdot ft^2\right]}, e\right) + GC_{pi}\right) \cdot \left(Fen^{\langle Fraction \rangle}\right)_j \ldots \\ + q_h \cdot \left(GC_p\left(\overrightarrow{\left[\left(Fen^{\langle Size \rangle}\right)_j \cdot ft^2\right]}, w\right) + GC_{pi}\right) \cdot \left[1 - \left(Fen^{\langle Fraction \rangle}\right)_j\right] \right] & \text{otherwise} \end{cases}$$

DP =

	0	1	2	3	4	5	6	7	8	9	10
0	-31.04	-29.45	-31.92	-31.27	-31.27	-31.27	-30.39	-31.27	-31.27	-33.85	-30.53
1	30.49	27.57	31.8	30.83	30.83	30.83	29.51	30.83	30.83	29.72	29.72

psf

$DP^{\langle 4 \rangle} = \begin{pmatrix} -31.27 \\ 30.832 \end{pmatrix} psf$ $\quad \left(Fen^{\langle Zone \rangle}\right)_4 = 5$ $\quad GC_p\left(20 \cdot ft^2, 6\right) = \begin{pmatrix} -1.429 \\ 1.247 \end{pmatrix}$

Design of Nailing Pattern for Roof Deck

Note there is no c zone for hip roof

Load on one nail: use 10 SF as effective area

$Area := 10 \cdot ft^2$ $\quad GC_p(Area, r) = \begin{pmatrix} -1.2 \\ 0.7 \end{pmatrix}$ $\quad GC_p(Area, si) = \begin{pmatrix} -1.4 \\ 0.7 \end{pmatrix}$ $\quad GC_p(Area, c) = \begin{pmatrix} -2.7 \\ 0.7 \end{pmatrix}$

Design Load: Zone si

$p_{single} := q_h \cdot \left(GC_p(Area, si) + GC_{pi}\right)$ $\qquad p_{single} = \begin{pmatrix} -34.619 \\ 17.31 \end{pmatrix} psf$

Tributary Area of single sheet of plwood: (4ftx8ft)

$Area := 32 \cdot ft^2$ $\quad GC_p(Area, r) = \begin{pmatrix} -1.149 \\ 0.599 \end{pmatrix}$ $\quad GC_p(Area, si) = \begin{pmatrix} -1.299 \\ 0.599 \end{pmatrix}$ $\quad GC_p(Area, c) = \begin{pmatrix} -2.245 \\ 0.599 \end{pmatrix}$

$p_{panel} := q_h \cdot \left(GC_p(Area, si) + GC_{pi}\right)$ $\qquad p_{panel} = \begin{pmatrix} -32.121 \\ 14.811 \end{pmatrix} psf$

Resistance of Single Nail

6d common nail

$q_r := 35 \cdot \frac{lbf}{in}$ 6d common nail, Southern Pine (specifig gravtiy =0.55) NDS 1997-S Table 12.2A

$l_{nail} := 2.0in$ length of nail, 6d

$t := .5 \cdot in$ Plywood thickness = 1/2" (min thickness of code)

$l_p := l_{nail} - t$ $\quad l_p = 1.5\,in$ penetration length

$C_D := 1.6$ Duration factor for short term loads - wind = 10 minutes

$C_m := 1.0$ Condition Factor = assume that wood moisture content at time of construction is same as long term value

$R_{nail_0} := q_r \cdot l_p \cdot C_D \cdot C_m$

8d common nail

$q_r := 41 \cdot \frac{lbf}{in}$

$l_{nail} := 2.5in$ length of nail, 8d, Southern Pine (SG=0.55), NDS 97-S Table 12.2A

$t := .5 \cdot in$ Plywood thickness = 1/2" (min thickness of code)

$l_p := l_{nail} - t$ $\quad l_p = 2\,in$ penetration length

$R_{nail_1} := q_r \cdot l_p \cdot C_D \cdot C_m$ $\qquad R_{nail} = \begin{pmatrix} 84 \\ 131.2 \end{pmatrix} lbf$ Resistance of single Nail, 6d and 8d respectively

Maximum Spacing for nails:

$$A_t := \frac{R_{nail}}{\left(\left|p_{single_0} + DL_{sheath}\right| \cdot 2 \cdot ft\right)} \qquad A_t = \begin{pmatrix} 15.264 \\ 23.841 \end{pmatrix} in$$

maximum allowable spacing of fasteners

Select nailing pattern that meets max spacing criteria

Check 6d nail first

number of nails that meets nailing pattern criteria for Zone si

$$\text{ceil}\left(\text{linterp}\left(s_{possible}, N_{possible}, A_{t_0}\right)\right) = 5$$

lookup nailing pattern to meet Zone2/3

$$II_s := \text{floor}\left(\text{linterp}\left(s_{possible}, II, A_{t_0}\right)\right) \qquad II_s = 7$$

$$s_{i6} := s_{possible_{II_s}} \qquad s_{i6} = 12\,in$$

check 8d nail

$$\text{ceil}\left(\text{linterp}\left(s_{possible}, N_{possible}, A_{t_1}\right)\right) = 4$$

$$II_s := \text{floor}\left(\text{linterp}\left(s_{possible}, II, A_{t_1}\right)\right) \qquad II_s = 8$$

$$s_{i8} := s_{possible_{II_s}} \qquad s_{i8} = 16\,in$$

spacing, nails

NailSched =

4.364	12
4.8	11
5.333	10
6	9
6.857	8
8	7
9.6	6
12	5
16	4
24	3
48	2

USE the following spacing:

edge spacing $s_e := 6in$

interior spacing

$$s_i := \begin{cases} s_{i8} & \text{if } s_{i6} < 6 \cdot in \\ s_{i6} & \text{otherwise} \end{cases} \qquad nailsize := \begin{cases} 8 & \text{if } s_{i6} < 6 \cdot in \\ 6 & \text{otherwise} \end{cases}$$

$$nailsize = 6 \qquad s_i = 12\,in$$

Check whole panel resistance

$$N_{nails} := 2 \cdot \left(\frac{48in}{s_e} + 1\right) + 3 \cdot \left(\frac{48in}{s_i} + 1\right) \qquad N_{nails} = 33$$

$$L_{panel} := \left(\left|p_{panel_0} + DL_{sheath}\right|\right) \cdot 32ft^2 \qquad L_{panel} = 976.673\,ft^2\,psf$$

uplift

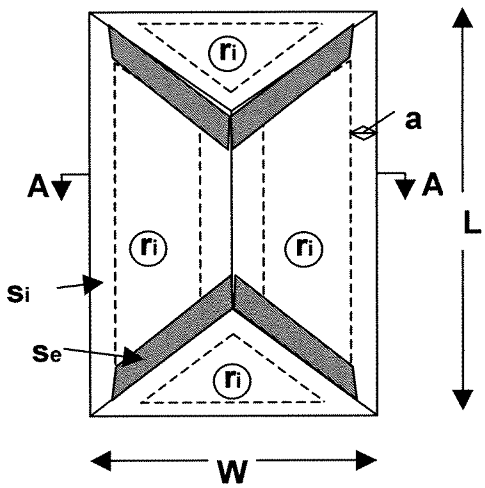
$$R_{total} := R_{nail_{\left(\frac{nailsize-6}{2}\right)}} \cdot N_n \; R_{total} = 2772\,lbf$$

$$Status_{RoofNail} := R_{total} > L_{panel} \qquad Status_{RoofNail} = 1$$

PASS = 1, FAIL = 0

ROOF STRAPS DESIGN

Roof Truss Design should be based on Components and Cladding loads



Note there is no combo load case that has a reduction in dead load in SBC (section 1609)

Effective wind area of a truss equals maximum of actual area and span times 1/3 span length

$A_{eff} := \begin{pmatrix} W \cdot \Delta \\ W \cdot \frac{W}{3} \end{pmatrix}$ $\quad A_{eff} = \begin{pmatrix} 92 \\ 705.333 \end{pmatrix} ft^2$ $\quad A_{eff} := \max(A_{eff})$

Since Aeff is greater than 100SF, Use 100SF for GCp values

External Gust Factors

$GC_p(A_{eff}, r) = \begin{pmatrix} -1.1 \\ 0.5 \end{pmatrix}$ $\qquad k := 0..4$

$GC_p(A_{eff}, si) = \begin{pmatrix} -1.2 \\ 0.5 \end{pmatrix}$

$GC_p(A_{eff}, c) = \begin{pmatrix} -1.8 \\ 0.5 \end{pmatrix}$

$p_k := \left(GC_p(A_{eff}, k)_0 + GC_{pi_0}\right) q_h$

$p = \begin{pmatrix} -27.201 \\ -27.201 \\ -29.674 \\ -44.511 \\ -44.511 \end{pmatrix} psf$

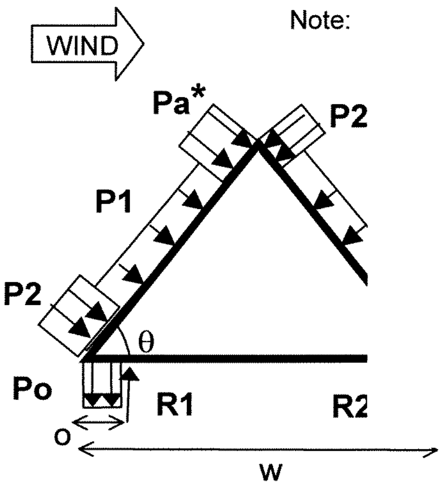
Negative pressures for r, ri, si, se and c zone

WIND Perpendicular to Ridge at section A-A

STRAP RESISTANCE used in ARA model

Sum Moments (note that in the mathcad formulas p0 is zone r pressures and p2 is zone si pressure)

Note:



$$R_0 := \frac{-\Delta}{(2\cdot o - W)}\cdot\left[\begin{array}{l} p_{si}\cdot a\cdot\cos(\theta)\cdot\left(W - o - \frac{a}{2}\cdot\cos(\theta)\right)\ ... \\ +p_r\cdot(l_r - a)\cdot\cos(\theta)\cdot\left[W - o - \frac{(l_r + a)}{2}\cdot\cos(\theta)\right]\ ... \\ +p_{si}\cdot a\cdot\cos(\theta)\cdot\left(\frac{W}{2} - o - \frac{a}{2}\cdot\cos(\theta)\right)\ ... \\ +p_r\cdot(l_r - a)\cdot\cos(\theta)\cdot\left(\frac{l_r - a}{2}\cdot\cos(\theta) - o\right)\ ... \\ + -p_{si}\cdot a\cdot\frac{a}{2}\cdot\sin(\theta)^2 - p_r\cdot(l_r - a)\cdot\frac{(l_r + a)}{2}\cdot\sin(\theta)^2\ ... \\ +p_{si}\cdot(a)\cdot\left(l_r - \frac{a}{2}\right)\cdot\sin(\theta)^2\ ... \\ +\left[p_r\cdot(l_r - a)\cdot\frac{(l_r - a)}{2}\cdot\sin(\theta)^2\right]\ ... \\ +1.0\cdot DL_{roof}\cdot W\cdot\left(\frac{W}{2} - o\right) \end{array}\right]$$

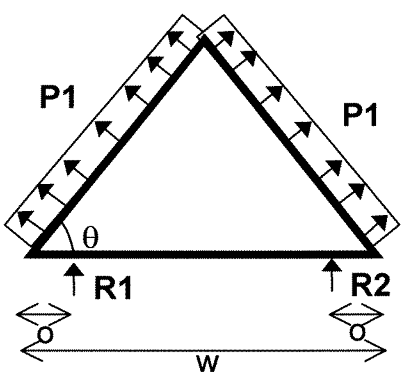
Sum Forces in Vertical

$$R_1 := \left[2\cdot\Delta\cdot\left[p_{si}\cdot a\cdot\cos(\theta) + p_{0r}\cdot(l_r - a)\cdot\cos(\theta)\right] - R_0 + DL_{roof}\cdot\Delta\cdot W\right]$$

$R_0 = -868.625$ lbf

$R_1 = -847.219$ lbf

WIND Parallel to Ridge at Section A-A



$$R_2 := \frac{-\Delta}{2\cdot o - W}\cdot\left[\begin{array}{l} p_r\cdot l_r\cdot\cos(\theta)\cdot\left(\begin{array}{l} W - o - \frac{l_r}{2}\cdot\cos(\theta)\ ... \\ +\frac{l_r}{2}\cdot\cos(\theta) - o \end{array}\right)\ ... \\ +DL_{roof}\cdot W\cdot\left(\frac{W}{2} - o\right) \end{array}\right]$$

$$R_3 := 2\cdot p_r\cdot l_r\cdot\Delta\cdot\cos(\theta) - R_2 + DL_{roof}\cdot\Delta\cdot W$$

$R_2 = -837.241$ lbf

$R_3 = -837.241$ lbf

Wind perpendicular to ridge, applied at all edge zones simultaneously (note that this is an unrealistic condition, but is one that may be checked by a designer).

$$p_a := p_{si}$$

$$R_1 := \frac{-1}{2\cdot o - W}\cdot\left[\begin{aligned}&p_{si}\cdot a\cdot\Delta\cdot\cos(\theta)\cdot\left(W - o - \frac{a}{2}\cdot\cos(\theta)\ ...\ + \frac{W}{2} - o - \frac{a}{2}\cdot\cos(\theta)\right)\ ...\\ &+ p_a\cdot a\cdot\Delta\cdot\cos(\theta)\cdot\left[\left[W - o - \left(l_r - \frac{a}{2}\right)\cdot\cos(\theta)\right]\ ...\ + \left(\frac{a}{2}\cdot\cos(\theta) - o\right)\right]\ ...\\ &+ p_r\cdot\left(l_r - 2a\right)\cdot\Delta\cdot\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\ ...\ + \frac{W}{2} - o - \frac{l_r}{2}\cdot\cos(\theta)\right)\ ...\\ &+ -\left[p_{si}\cdot a\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{a}{2}\cdot\sin(\theta)\right)\right]\ ...\\ &+ -\left[p_r\cdot\left(l_r - 2\cdot a\right)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right]\ ...\\ &+ -\left[p_a\cdot a\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r - \frac{a}{2}\right)\sin(\theta)\right]\ ...\\ &+ p_r\cdot\left(l_r - 2\cdot a\right)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\ ...\\ &+ p_{si}\cdot a\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r - \frac{a}{2}\right)\sin(\theta)\ ...\\ &+ p_a\cdot a\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{a}{2}\cdot\sin(\theta)\right)\ ...\\ &+ DL_{roof}\cdot\Delta\cdot W\cdot\left(\frac{W}{2} - o\right)\end{aligned}\right]$$

$$R_1 = -878.603\ \text{lbf}$$

$$R_2 := \left[\left[\begin{aligned}&2\cdot p_{si}\cdot a\cdot\cos(\theta)\cdot\Delta\ ...\\ &+ 2\cdot p_r\cdot\left(l_r - 2\cdot a\right)\cdot\cos(\theta)\cdot\Delta\ ...\\ &+ 2\cdot p_a\cdot a\cdot\cos(\theta)\cdot\Delta\end{aligned}\right] + DL_{roof}\cdot(\Delta\cdot W)\right] - R_1$$

$$R_2 = -878.603\ \text{lbf}$$

Check the 'section' of the jack hip rafters as worst case scenario - apply se zone loads uniformly across span.

$$Rse_0 := \frac{\Delta}{(W - 2\cdot o)}\cdot\left[p_{se}\cdot l_r\cdot\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\ \ldots \atop + \frac{l_r}{2}\cdot\cos(\theta) - o\right)\ \ldots \atop + DL_{roof}\cdot W\cdot\left(\frac{W}{2} - o\right)\right]$$

$$Rse_1 := \left[\left(2\cdot p_{se}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) - Rse_0\right] + DL_{roof}\cdot\Delta\cdot W$$

$$Rse = \begin{pmatrix} -1633.486 \\ -1633.486 \end{pmatrix} lbf$$

Therefore, house will need larger straps at corners of hip - exact configuration and size is dependent on the detailed configuration of the trusses - and how the load is transferred from one truss to the next. (.ie. are loads from jack trusses carried to a step-down hip truss, or to jack trusses,etc.)

The ARA roof-strap model simulates failure of the entire roof assembly as a whole, and not any one specific truss connection. Therefore, strap size in model should be based on strap representative of the majority of the connections, and therefore is based on section at middle of structure.

If house was a gable house, then end truss would be loaded with zone c loads and zone se loads

$$Rc_0 := \frac{\Delta}{W - 2\cdot o}\cdot\begin{bmatrix} p_{se}\cdot(l_r - 2\cdot 2\cdot a)\cdot\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\ \ldots \atop + \frac{l_r}{2}\cdot\cos(\theta) - o\right)\ \ldots \\ + p_c\cdot(2a)\cdot\cos(\theta)\cdot\begin{bmatrix} a\cdot\cos(\theta) - o\ \ldots \\ + (l_r - a)\cdot\cos(\theta) - o\ \ldots \\ + W - (l_r - a)\cos(\theta) - o\ \ldots \\ + W - a\cdot\cos(\theta) - o \end{bmatrix}\ \ldots \\ + DL_{roof}\cdot W\cdot\left(\frac{W}{2} - o\right) \end{bmatrix}$$

$$p_{se} = -44.511\ psf$$

$$p_c = -44.511\ psf$$

$$Rc_1 := \left[\left[2\cdot p_{se}\cdot(l_r - 2\cdot 2\cdot a)\cdot\Delta\cdot\cos(\theta)\right] + \left(4\cdot p_c\cdot 2\cdot a\cdot\Delta\cdot\cos(\theta)\right) - Rc_0\right] + DL_{roof}\cdot\Delta\cdot W$$

$$Rc = \begin{pmatrix} -1633.486 \\ -1633.486 \end{pmatrix} lbf$$

Check MWFRS loading conditions: There are 4 external loading conditions for the upper roof and two internal pressure conditions

Corner 1: CASE A wind perpendicular to ridge
Corner 1: CASE B wind parallel to ridge
Corner 2: CASE A wind perpendicular to 'imaginary ridge'
Corner 2: CASE B wind parallel to 'imaginary ridge'

Figure 1606.2B2:

Roof Angle = 20-30 degrees

$$GC_{pfAp} := \begin{cases} (0.4\ \ -0.75\ \ -0.75\ \ -0.70\ \ 0.7\ \ -1.0\ \ -1.0\ \ -0.95)^T & \text{if IntPressure = Enclosed} \\ (0\ \ -1.2\ \ -1.2\ \ -1.1\ \ 0.3\ \ -1.4\ \ -1.4\ \ -1.4)^T & \text{if IntPressure = PartEnclosed} \\ \left((0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)^T\right) & \text{otherwise} \end{cases}$$

$$GC_{pfAn} := \begin{cases} (0.8\ \ -0.35\ \ -0.35\ \ -0.30\ \ 1.1\ \ -0.6\ \ -0.6\ \ -0.55)^T & \text{if IntPressure = Enclosed} \\ (0.9\ \ -0.25\ \ -0.25\ \ -0.2\ \ 1.2\ \ -0.5\ \ -0.5\ \ -0.45)^T & \text{if IntPressure = PartEnclosed} \\ \left((0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)^T\right) & \text{otherwise} \end{cases}$$

$$GC_{pfBp} := \begin{cases} (0\ \ -1.0\ \ -0.65\ \ 0\ \ 0.25\ \ -0.55\ \ 0\ \ -1.4\ \ -0.8\ \ 0\ \ 0.5\ \ -0.70)^T & \text{if IntPressure = Enclosed} \\ (0\ \ -1.4\ \ -1.05\ \ 0\ \ -0.15\ \ -0.95\ \ 0\ \ -1.8\ \ -1.2\ \ 0\ \ 0.1\ \ -1.10)^T & \text{if IntPressure = PartEnclosed} \\ \left((0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)^T\right) & \text{otherwise} \end{cases}$$

$$GC_{pfBn} := \begin{cases} (0\ \ -0.60\ \ -0.25\ \ 0\ \ 0.65\ \ -0.15\ \ 0\ \ -1.00\ \ -0.4\ \ 0\ \ 0.90\ \ -0.30)^T & \text{if IntPressure = Enclosed} \\ (0\ \ -0.5\ \ -0.15\ \ 0\ \ 0.75\ \ -0.05\ \ 0\ \ -0.90\ \ -0.3\ \ 0\ \ 1.0\ \ -0.2)^T & \text{if IntPressure = PartEnclosed} \\ \left((0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)^T\right) & \text{otherwise} \end{cases}$$

$$p_{Ap} := q_h \cdot \left(\overrightarrow{(GC_{pfAp})}\right) \qquad p_{Bp} := q_h \cdot \left(\overrightarrow{(GC_{pfBp})}\right)$$

$$p_{An} := q_h \cdot \left(\overrightarrow{(GC_{pfAn})}\right) \qquad p_{Bn} := q_h \cdot \left(\overrightarrow{(GC_{pfBn})}\right)$$

$p_{An}^{T} = (19.782\ \ -8.655\ \ -8.655\ \ -7.418\ \ 27.201\ \ -14.837\ \ -14.837\ \ -13.6)\,psf$

$p_{Ap}^{T} = (9.891\ \ -18.546\ \ -18.546\ \ -17.31\ \ 17.31\ \ -24.728\ \ -24.728\ \ -23.492)\,psf$

$p_{Bn}^{T} =$ (psf)

	0	1	2	3	4	5	6	7	8
0	0	-14.837	-6.182	0	16.073	-3.709	0	-24.728	-9.891

$p_{Bp}^{T} =$ (psf)

	0	1	2	3	4	5	6	7	8
0	0	-24.728	-16.073	0	6.182	-13.6	0	-34.619	-19.782

$GC_{p\_overhang} := \begin{pmatrix} 0.2 \\ -1.5 \end{pmatrix}$

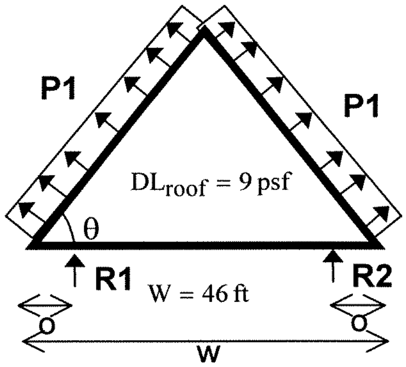
From Table 1606.2D: Coefficients from Roof Overhangs - apply to windward overhang only.

$p_{overhang} := q_h \cdot GC_{p\_overhang}$ $p_{overhang} = \begin{pmatrix} 4.946 \\ -37.092 \end{pmatrix} psf$

z = width of zone 2 on roof parallel to wind direction, varies for some cases

$z := \dfrac{W}{2 \cdot \cos(\theta)}$ $z = 25.301\ ft$

Calculate uplift on corner truss by end zone pressure from MWFRS loads



Apply edge zone loads on trib area between end truss and next truss. Apply overhang load on full width of overhang for end truss

$p_{overhang_1} = -37.092\,psf$

$2\cdot a = 9.2\,ft$

$\Delta = 2\,ft$ $\qquad o_g := 1\cdot ft$

$$R_{MWFc_0} := \frac{1}{W-2\cdot o}\cdot\left[\left[\left(pAn_{A2E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) \ldots \atop + \left(pAn_{A3E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta) - o\right)\right] \ldots \atop + DL_{roof}\cdot W\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot\left(\frac{W}{2} - o\right)\right]$$

$pAn_{A2E} = -14.837\,psf$

$pAn_{A3E} = -14.837\,psf$

$R_{MWFc_0} = -780.367\,lbf$

$$R_{MWFc_1} := \left[\left(pAn_{A2E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\right] \ldots + \left[\left(pAn_{A3E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\right] - R_{MWFc_0} + DL_{roof}\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot W$$

$R_{MWFc_1} = -780.367\,lbf$

$$R_{MWFc_2} := \frac{1}{W-2\cdot o}\cdot\left[\left[\left(pAp_{A2E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) \ldots \atop + \left(pAp_{A3E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta) - o\right)\right] \ldots \atop + DL_{roof}\cdot W\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot\left(\frac{W}{2} - o\right)\right]$$

$pAp_{A2E} = -24.728\,psf$

$pAp_{A3E} = -24.728\,psf$

$R_{MWFc_2} = -1007.865\,lbf$

$$R_{MWFc_3} := \left[\left(pAp_{A2E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\right] \ldots + \left[\left(pAp_{A3E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\right] - R_{MWFc_2} + DL_{roof}\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot W$$

$R_{MWFc_3} = -1007.865\,lbf$

$$R_{MWFc_4} := \frac{1}{W - 2\cdot o}\cdot\left[\left[\left(pBp_{B2E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) \ldots \atop + \left(pBp_{B3E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta) - o\right)\right] \ldots \atop + DL_{roof}\cdot W\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot\left(\frac{W}{2} - o\right)\right]$$

$pBp_{B2E} = -34.619\,psf$

$pBp_{B3E} = -19.782\,psf$

$R_{MWFc_4} = -1153.929\,lbf$

$$R_{MWFc_5} := \left[\left(pBp_{B2E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta) \ldots \atop + \left(pBp_{B3E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\right] - R_{MWFc_4} + DL_{roof}\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot W$$

$R_{MWFc_5} = -975.55\,lbf$

$$R_{MWFc_6} := \frac{1}{W - 2\cdot o}\cdot\left[\left[\left(pBn_{B2E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) \ldots \atop + \left(pBn_{B3E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta) - o\right)\right] \ldots \atop + DL_{roof}\cdot W\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot\left(\frac{W}{2} - o\right)\right]$$

$pBn_{B2E} = -24.728\,psf$

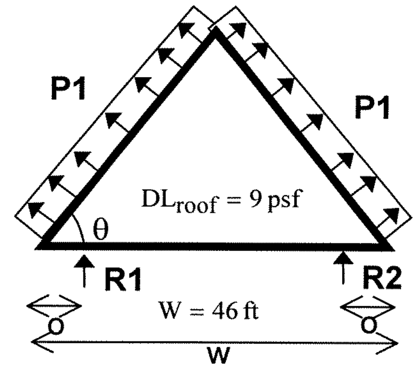
$pBn_{B3E} = -9.891\,psf$

$R_{MWFc_6} = -926.431\,lbf$

$$R_{MWFc_7} := \left[\left(pBn_{B2E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta) \ldots \atop + \left(pBn_{B3E}\cdot l_r\cdot\frac{\Delta}{2} + p_{overhang_1}\cdot l_r\cdot o_g\right)\cdot\cos(\theta)\right] - R_{MWFc_6} + DL_{roof}\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot W$$

$R_{MWFc_7} = -748.051\,lbf$

Calculate uplift on interior truss by interior zone pressure from MWFRS loads



$$R_{MWF_0} := \frac{1}{W-2\cdot o}\cdot\left[\left[p{An}_{A2}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(W-o-\frac{l_r}{2}\cdot\cos(\theta)\right)\ldots + p{An}_{A3}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta)-o\right)\right]\ldots + DL_{roof}\cdot W\cdot\Delta\cdot\left(\frac{W}{2}-o\right)\right]$$

$R_{MWF_0} = 15.878\,lbf$

$$R_{MWF_1} := \left(p{An}_{A2}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) + \left(p{An}_{A3}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) - R_{MWF_0} + DL_{roof}\cdot\Delta\cdot W$$

$p{An}_{A2} = -8.655\,psf$

$R_{MWF_1} = 15.878\,lbf$

$p{An}_{A3} = -8.655\,psf$

$$R_{MWF_2} := \frac{1}{W-2\cdot o}\cdot\left[\left[p{Ap}_{A2}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(W-o-\frac{l_r}{2}\cdot\cos(\theta)\right)\ldots + p{Ap}_{A3}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta)-o\right)\right]\ldots + DL_{roof}\cdot W\cdot\Delta\cdot\left(\frac{W}{2}-o\right)\right]$$

$p{Ap}_{A2} = -18.546\,psf$

$p{Ap}_{A3} = -18.546\,psf$

$R_{MWF_2} = -439.119\,lbf$

$$R_{MWF_3} := \left(p{Ap}_{A2}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) + \left(p{Ap}_{A3}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) - R_{MWF_2} + DL_{roof}\cdot\Delta\cdot W$$

$R_{MWF_3} = -439.119\,lbf$

$$R_{MWF_4} := \frac{1}{W-2\cdot o}\cdot\left[\left[p{Bp}_{B2}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(W-o-\frac{l_r}{2}\cdot\cos(\theta)\right)\ldots + p{Bp}_{B3}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta)-o\right)\right]\ldots + DL_{roof}\cdot W\cdot\Delta\cdot\left(\frac{W}{2}-o\right)\right]$$

$p{Bp}_{B2} = -24.728\,psf$

$p{Bp}_{B3} = -16.073\,psf$

$R_{MWF_4} = -628.486\,lbf$

$$R_{MWF_5} := \left(p{Bp}_{B2}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) + \left(p{Bp}_{B3}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) - R_{MWF_4} + DL_{roof}\cdot\Delta\cdot W$$

$R_{MWF_5} = -420.376\,lbf$

$$R_{MWF_6} := \frac{1}{W - 2\cdot o}\cdot\left[\left[p_{Bn_{B2}}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right)\ ... \atop + p_{Bn_{B3}}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta) - o\right)\right]\ ... \atop + DL_{roof}\cdot W\cdot\Delta\cdot\left(\frac{W}{2} - o\right)\right]$$

$p_{Bn_{B2}} = -14.837\ psf$

$p_{Bn_{B3}} = -6.182\ psf$

$R_{MWF_6} = -173.489\ lbf$

$$R_{MWF_7} := \left(p_{Bn_{B2}}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) + \left(p_{Bn_{B3}}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) - R_{MWF_6} + DL_{roof}\cdot\Delta\cdot W$$

$R_{MWF_7} = 34.621\ lbf$

WALL DESIGN for Wood Frame Walls

Nominal Wall Design Parameters

Exterior Surface: 7/16" OSB $t_{OSB} := \frac{7}{16} \cdot in$

Interior Surface: 1/2" Gypsum

Nail Size: 8d common

$\Delta_{stud} := \begin{pmatrix} 12 \\ 16 \end{pmatrix} in$ Spacing of studs in wall $sp := 0..1$ spacing of studs option variable

Wall Sheathing Attachment - Suction Loads for Zone e C&C loads

Loads:

$Area := 32 \cdot ft^2$ $Area = 32\,ft^2$ $A_{eff} := 10 \cdot ft^2$ for cladding fasteners

$p_{wall} := q_h \cdot (GC_p(A_{eff}, e) + GC_{pi})$ $p_{wall_0} = -37.092\,psf$

$L_{total} := (-p_{wall})_0 \cdot Area$ $L_{total} = 1186.948\,lbf$ suction

Resistance:

$q_r := 41 \cdot \frac{lbf}{in}$ 8d common nail

$l_{nail} := 2.5in$ length of nail, 8d

$l_p := l_{nail} - t_{OSB}$ $l_p = 2.063\,in$ penetration length

$C_D := 1.6$ Duration factor for short term loads - wind = 10 minutes

$C_m := 1.0$ Condition Factor = assume that wood moisture content at time of construction is same as long term value

$R_{nail} := q_r \cdot l_p \cdot C_D \cdot C_m$ $R_{nail} = 135.3\,lbf$ per nail

$$Nnails_{wall} := 2 \cdot \left[\frac{(8 \cdot ft)}{12 \cdot in} + 1\right] + \left(\frac{4 \cdot ft}{\Delta_{stud}} - 1\right) \cdot \left(\frac{8 \cdot ft}{6 \cdot in} + 1\right) + \left[\frac{4 \cdot ft}{6 \cdot in} - \left(\frac{4 \cdot ft}{\Delta_{stud}} - 1\right)\right] \cdot 2$$

Int Nails at 12" Edge nails at 6" Top/Bottom Plate at 6"

$Nnails_{wall} = \begin{pmatrix} 79 \\ 64 \end{pmatrix}$

$R_{total} := Nnails_{wall} \cdot R_{nail}$ $R_{total} = \begin{pmatrix} 1.069 \times 10^4 \\ 8659.2 \end{pmatrix} lbf$ $R_{total} := \min(R_{total})$

$Status_{WallSuction} := \begin{cases} PASS & \text{if } (R_{total} > L_{total}) \\ FAIL & \text{otherwise} \end{cases}$ $Status_{WallSuction} = 1$

Wall Bending & Axial Loads

Wind Load:

$A_{eff} := 32 \cdot ft^2$ $\quad A_{eff} = 32\,ft^2$ $\quad$ Zone e

$p_{wall} := q_h \cdot \left(GC_p\left(A_{eff}, e\right) + GC_{pi}\right)$ $\quad GC_p(Area, e) + GC_{pi} = \begin{pmatrix} -1.381 \\ 1.211 \end{pmatrix}$ $\quad p_{wall} = \begin{pmatrix} -34.151 \\ 29.941 \end{pmatrix} psf$

$\omega := p_{wall_0} \cdot \Delta_{stud}$ $\quad \omega = \begin{pmatrix} -34.151 \\ -45.535 \end{pmatrix} \frac{1}{ft}\,lbf$ $\quad M := \frac{\omega \cdot h_{wall}^2}{8}$ $\quad M = \begin{pmatrix} -273.21 \\ -364.279 \end{pmatrix} ft\,lbf$

Axial Load:

$DL_{roof} = 9\,psf$ $\quad L = 52\,ft$ $\quad W = 46\,ft$

$Load_{stud} := \frac{\left(DL_{roof} \cdot W \cdot L\right)}{2 \cdot L} \cdot \Delta_{stud}$ $\quad Load_{stud} = \begin{pmatrix} 207 \\ 276 \end{pmatrix} lbf$

assume all load carried by long walls

Resistance of Wall (Wood)

$size := 0..2$

$Stud_w := \begin{pmatrix} 1.5 \cdot in \\ 1.5 \cdot in \\ 1.5 \cdot in \end{pmatrix}$ $\quad Stud_d := \begin{pmatrix} 3.5 \cdot in \\ 5.5 \cdot in \\ 7.25 \cdot in \end{pmatrix}$

2x4 wall, Dressed dim, Table 1A from NDS97-S
2x6 wall
2x8 wall

$Stud_{area} := \overrightarrow{\left(Stud_w \cdot Stud_d\right)}$ $\quad Stud_{area} = \begin{pmatrix} 5.25 \\ 8.25 \\ 10.875 \end{pmatrix} in^2$

Section modulus: NDS-S97 $\qquad$ Moment of Inertia

$S_{xx} := \begin{pmatrix} 3.063 \\ 7.563 \\ 13.14 \end{pmatrix} \cdot in^3$ $\quad S_{yy} := \begin{pmatrix} 1.313 \\ 2.063 \\ 2.719 \end{pmatrix} \cdot in^3$ $\quad I_{xx} := \begin{pmatrix} 5.359 \\ 20.80 \\ 47.63 \end{pmatrix} \cdot in^4$ $\quad I_{yy} := \begin{pmatrix} 0.984 \\ 1.547 \\ 2.039 \end{pmatrix} \cdot in^4$

Design Values from Table 4A, NDS-S 1997

$F_b := 875 \cdot psi$ Bending stress, allowable

$F_t := 450 \cdot psi$ Tension Parallel to grain, allowable

$F_v := 70 \cdot psi$ Shear parallel to grain, allowablw

$Fcp := 425 \cdot psi$ Compression Perpendicular to grain

$F_c := 1150 \cdot psi$ Compression Parallel to grain

$E := 1400000 \cdot psi$ Modulus of Elasticity

Species and Grade:
SPF No.2

Lumber Property Adjustments

$C_{Dwind} := 1.6$

$C_{Dgravity} := 1.25$

$C_r := 1.15$ Repetative Loading Factor, NDS

$C_L := 1.0$ Continuous Lateral Bracing (from sheathing)

$C_F := \begin{pmatrix} 1.05 \\ 1.1 \\ 1.1 \end{pmatrix}$ for compression / for tension / for bending

Calculate Adjusted Bending Capacity

$F_{b\_a} := F_b \cdot C_{Dwind} \cdot C_L \cdot C_{F_2} \cdot C_r$ $\qquad F_{b\_a} = 1771\,psi$

Calculate adjusted compressive Capacity

$F_{c\_star} := F_c \cdot C_{Dwind} \cdot C_{F_0}$ $\qquad F_{c\_star} = 1932\,psi$

Euler Buckling Load

$K_{cE} := 0.3$ visually graded lumber

$c := 0.8$ sawn lumber

$K_l := 1.0$ Effective length factor (Assume pin-pin column)

$$F_{cE} := \frac{K_{cE} \cdot E}{\left[\left(\frac{K_l \cdot h_{wall}}{Stud_d}\right)^2\right]} \qquad F_{cE} = \begin{pmatrix} 558.268 \\ 1378.581 \\ 2395.426 \end{pmatrix} psi$$

Euler buckling pressure

$$C_p := \frac{1 + \frac{F_{cE}}{F_{c\_star}}}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{cE}}{F_{c\_star}}}{2 \cdot c}\right)^2 - \frac{\frac{F_{cE}}{F_{c\_star}}}{c}} \qquad C_p = \begin{pmatrix} 0.269 \\ 0.566 \\ 0.76 \end{pmatrix}$$

Column stability factor

$$F_{c\_a} := \overrightarrow{\left(F_c \cdot C_{Dwind} \cdot C_{F_0} \cdot C_p\right)} \qquad F_{c\_a} = \begin{pmatrix} 519.973 \\ 1093.431 \\ 1467.657 \end{pmatrix} psi$$

Combined Bending and Axial Compression Capactiy for Wind and Gravity (Dead Load) using combined stress interaction equation NDS 3.9.2 (also see p3.27 of Wood Engineering and Construction Handbook)

$sp := 0$ stud spacing of $\Delta_{stud_{sp}} = 12\,in$

Bending stress $f_b := \frac{-M_{sp}}{S_{xx}}$ $f_b = \begin{pmatrix} 1070.361 \\ 433.494 \\ 249.506 \end{pmatrix} psi$

compressive stress $f_c := \frac{Load_{stud_{sp}}}{Stud_{area}}$ $f_c = \begin{pmatrix} 39.429 \\ 25.091 \\ 19.034 \end{pmatrix} psi$

$$CSIequation_{size} := \left[\left(\frac{f_{c_{size}}}{F_{c\_a_{size}}} \right)^2 + \frac{f_{b_{size}}}{F_{b\_a} \cdot \left(1 - \frac{f_{c_{size}}}{F_{cE_{size}}} \right)} \right] \qquad CSIequation = \begin{pmatrix} 0.656 \\ 0.25 \\ 0.142 \end{pmatrix}$$

$Status_{Wood\_Bending2x4} :=$ | PASS if $(CSIequation_0) \le 1.0$ / FAIL otherwise $\qquad Status_{Wood\_Bending2x4} = 1$

$Status_{Wood\_Bending2x6} :=$ | PASS if $(CSIequation_1) \le 1.0$ / FAIL otherwise $\qquad Status_{Wood\_Bending2x6} = 1$

$Status_{Wood\_Bending2x8} :=$ | PASS if $(CSIequation_2) \le 1.0$ / FAIL otherwise $\qquad Status_{Wood\_Bending2x8} = 1$

Calculate adjusted axial load only case

$F_{c\_star} := F_c \cdot C_{Dgravity} \cdot C_{F_0}$ $\qquad$ $F_{c\_star} = 1509.375\,psi$

Euler Buckling Load

$K_{cE} := 0.3$ visually graded lumber

$c := 0.8$ sawn lumber

$K_l := 1.0$ Effective length factor (Assume pin-pin column)

$$F_{cE} := \frac{K_{cE} \cdot E}{\left[\left(\frac{K_l \cdot h_{wall}}{Stud_d}\right)^2\right]} \qquad F_{cE} = \begin{pmatrix} 558.268 \\ 1378.581 \\ 2395.426 \end{pmatrix} psi$$

Euler buckling pressure

$$C_p := \frac{1 + \frac{F_{cE}}{F_{c\_star}}}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{cE}}{F_{c\_star}}}{2 \cdot c}\right)^2 - \frac{\frac{F_{cE}}{F_{c\_star}}}{c}} \qquad C_p = \begin{pmatrix} 0.336 \\ 0.659 \\ 0.823 \end{pmatrix}$$

Column stability factor

$$F_{c\_a} := \overrightarrow{\left(F_c \cdot C_{Dgravity} \cdot C_{F_0} \cdot C_p\right)} \qquad F_{c\_a} = \begin{pmatrix} 506.984 \\ 994.46 \\ 1241.94 \end{pmatrix} psi$$

$$CSIequation := \overrightarrow{\frac{f_c}{F_{c\_a}}} \qquad CSIequation = \begin{pmatrix} 0.078 \\ 0.025 \\ 0.015 \end{pmatrix}$$

$$Status_{Wood\_Axial} := \begin{vmatrix} PASS \text{ if } \max(CSIequation) \le 1.0 \\ FAIL \text{ otherwise} \end{vmatrix} \qquad Status_{Wood\_Axial} = 1$$

Lateral Shear Design of Walls

1. Shear Loads from Wind (MWFRS)

Note that roof pressures cancel in Case A

$$Shear\_alt_{A_0} := VPA_{wall\Gamma} \cdot \left(pAp_{A1} - pAp_{A4}\right) \cdots + VPA_{\Gamma} \cdot \left(pAp_{A2} - pAp_{A3}\right)$$

$$Shear\_alt_{A_1} := VPA_{wall\Gamma} \cdot \left(pAn_{A1} - pAn_{A4}\right) \cdots + VPA_{\Gamma} \cdot \left(pAn_{A2} - pAn_{A3}\right)$$

$$Shear\_alt_{A_2} := VPA_{wall\_II} \cdot \left(pAp_{A1} - pAp_{A4}\right) \cdots + VPA_{II} \cdot \left(pAp_{A2} - pAp_{A3}\right)$$

$$Shear\_alt_{A_3} := VPA_{wall\_II} \cdot \left(pAn_{A1} - pAn_{A4}\right) \cdots + VPA_{II} \cdot \left(pAn_{A2} - pAn_{A3}\right)$$

$VPA_{wall\Gamma} = 208\ ft^2$

$VPA_{\Gamma} = 305.708\ ft^2$

$pAp_{A4} = -17.31\ psf$

$VPA_{wall\_II} = 184\ ft^2$

$VPA_{II} = 242.458\ ft^2$

$$pAp = \begin{pmatrix} 9.891 \\ -18.546 \\ -18.546 \\ -17.31 \\ 17.31 \\ -24.728 \\ -24.728 \\ -23.492 \end{pmatrix} psf \qquad pAn = \begin{pmatrix} 19.782 \\ -8.655 \\ -8.655 \\ -7.418 \\ 27.201 \\ -14.837 \\ -14.837 \\ -13.6 \end{pmatrix} psf$$

$$Shear\_alt_A = \begin{pmatrix} 5658 \\ 5658 \\ 5005 \\ 5005 \end{pmatrix} lbf$$

$$Shear\_alt_{B_0} := VPA_{wall\_II} \cdot \left(pBp_{B5} - pBp_{B6}\right) \cdots + VPA_{II} \cdot \left[\left(\frac{pBp_{B2E} + pBp_{B3E}}{2}\right) - \left(\frac{pBp_{B3} + pBp_{B2}}{2}\right)\right]$$

$$Shear\_alt_{B_1} := VPA_{wall\_II} \cdot \left(pBn_{B5} - pBn_{B6}\right) \cdots + VPA_{II} \cdot \left[\left(\frac{pBn_{B2E} + pBn_{B3E}}{2}\right) - \left(\frac{pBn_{B3} + pBn_{B2}}{2}\right)\right]$$

$$Shear\_alt_{B_2} := VPA_{wall\Gamma} \cdot \left(pBp_{B5} - pBp_{B6}\right) \cdots + VPA_{\Gamma} \cdot \left[\left(\frac{pBp_{B2E} + pBp_{B3E}}{2}\right) - \left(\frac{pBp_{B3} + pBp_{B2}}{2}\right)\right]$$

$$Shear\_alt_{B_3} := VPA_{wall\Gamma} \cdot \left(pBn_{B5} - pBn_{B6}\right) \cdots + VPA_{\Gamma} \cdot \left[\left(\frac{pBn_{B2E} + pBn_{B3E}}{2}\right) - \left(\frac{pBn_{B3} + pBn_{B2}}{2}\right)\right]$$

$VPA_{wall\Gamma} = 208\ ft^2$

$VPA_{\Gamma} = 305.708\ ft^2$

$VPA_{wall\_II} = 184\ ft^2$

$VPA_{II} = 242.458\ ft^2$

pBp = (psf)

0
-24.7
-16.1
0
6.2
-13.6
0
-34.6
-19.8
0
12.4
-17.3

pBn =

0
-14.84
-6.18
0
16.07
-3.71
0
-24.73
-9.89
0
22.26
-7.42

$$Shear\_alt_B = \begin{pmatrix} 1991 \\ 1991 \\ 2036 \\ 2036 \end{pmatrix} lbf$$

Note: internal pressures cancel and therefore are ignored in calculating total shear

2. Shear Load per wall: (Roof loads plus half of wall loads)

$$Shear_\Gamma := \begin{pmatrix} Shear\_alt_{A_0} \\ Shear\_alt_{B_2} \end{pmatrix}$$

$$Shear_\Gamma = \begin{pmatrix} 5657.8 \\ 2035.9 \end{pmatrix} lbf \qquad Shear_\Gamma := \max(Shear_\Gamma)$$ per wall

$$Shear_{ll} := \begin{pmatrix} Shear\_alt_{A_2} \\ Shear\_alt_{B_0} \end{pmatrix} \cdot \frac{1}{2}$$

$$Shear_{ll} = \begin{pmatrix} 2502.5 \\ 995.6 \end{pmatrix} lbf \qquad Shear_{ll} := \max(Shear_{ll})$$ per wall

Allowable shear resistance from NDS Supplement for structural use panel shear wall and diaphragm

Wall properties: (see above)

Exterior Surface: 7/16" OSB $t_{OSB} = 0.438\,in$

Interior Surface: 1/2" Gypsum

Blocked construction

Nail Size: 8d common Nail spacing: 6"/12"

$\Delta_{stud} = \begin{pmatrix} 12 \\ 16 \end{pmatrix} in$ Spacing of studs in wall

$Shear_{allowable} := 225 \cdot \frac{lbf}{ft}$ Table 4.1A of Structural Use Panel Shear Wall and Diaphragm Supplement to NDS 1997

$$L_{shearMin\_\Gamma} := \begin{pmatrix} \dfrac{Shear_\Gamma}{Shear_{allowable}} \\ 0.4 \cdot W \end{pmatrix} \qquad L_{shearMin\_\Gamma} = \begin{pmatrix} 25.146 \\ 18.4 \end{pmatrix} ft$$

$$L_{shearMin\_ll} := \begin{pmatrix} \dfrac{Shear_{ll}}{Shear_{allowable}} \\ 0.4 \cdot L \end{pmatrix} \qquad L_{shearMin\_ll} = \begin{pmatrix} 11.122 \\ 20.8 \end{pmatrix} ft$$

Code required minimum length of 0.4 of total building length (SBC Section 2105.1.2)

$$L_{shearMin\_\Gamma} := \max(L_{shearMin\_\Gamma}) \qquad L_{shearMin\_ll} := \max(L_{shearMin\_ll})$$

Actual length available for shear walls:

$L_{shearwall\_Actual\_\Gamma} := (30 \cdot ft + 24 \cdot ft + 18 \cdot ft + 20 \cdot ft + 8 \cdot ft)$

$L_{shearwall\_Actual\_\Gamma} = 100\,ft$ per side

$L_{shearwall\_Actual\_ll} := \left[\begin{array}{l} 4 \cdot ft + 4 \cdot ft + 10 \cdot ft + 4 \cdot ft \ldots \\ + (24 + 10 + 4 + 4 + 4 + 4) \cdot ft \end{array} \right]$

$L_{shearwall\_Actual\_ll} = 72\,ft$

$Status_{Wood\_Shear} := \left| \begin{array}{l} PASS \quad if \; (L_{shearwall\_Actual\_\Gamma} > L_{shearMin\_\Gamma}) \cdot (L_{shearwall\_Actual\_ll} > L_{shearMin\_ll}) \\ FAIL \quad otherwise \end{array} \right.$

$Status_{Wood\_Shear} = 1$

3. Shear Wall "Chord" Force

$T := \dfrac{Shear_{\Gamma} \cdot h_{wall}}{L_{shearwall\_Actual\_\Gamma}}$ $\quad T = 452.623\,lbf$ Holddown anchors must meet this resistance

4. Shear of Anchor Bolts

Anchor bolts 5/8" diameter embedded in concrete 6" trough 2x4 bottom plate.

$Z := 850 \cdot lbf$ For Specific Gravity wood of 0.5, Table 8.2E of NDS supplement for connections

$C_t := 1.0$ temperature service factor

$C_{others} := 1.0$ bunch of other factors for end grain, toenail, etc. which are all 1.0

$C_g := 1.0$ Group Action Factor: fasteners are several feet apart and therefore behave as single fasteners

$Z\_a := Z \cdot C_{Dwind} \cdot C_m \cdot C_t \cdot C_g \cdot C_{others}$ $\quad Z\_a = 1360\,lbf$ Shear capcity per bolt

$Shear_{\Gamma} = 5657.786\,lbf$ shear to resist...

$N_{bolts} := \dfrac{Shear_{\Gamma}}{Z\_a}$ $\quad N_{bolts} = 4.16$

$\Delta_{bolt} := \dfrac{W}{N_{bolts}}$ $\quad \Delta_{bolt} = 11.057\,ft$ Use one bolt every $floor(\Delta_{bolt}) = 11\,ft$

WALL DESIGN for Masonry Walls

1. Choosing Spacing of Vertical Reinforcement in Reinforced Wall

Select Vertical Wall Reinforcement based horizontal flexure between grouted cells - horizontal span

To determining the spacing of the vertical reinforcement, we have used the method cited in "Masonry Structures Behavior and Design" by Drysdale, R. G., Hamid, A. A., and Baker, L. R. In this book it is stated that when the spacing of reinforcement is greater than beff the wall is considered as reinforced strips beff wide with unreinforced strips in between. Therefore, "The reinforced strips are designed to carry the full load and the unreinforced masonry must be capable of spanning a horizontal distance between reinforcement". In addition, ACI 530 specifies a maximum reinforcement only for seismic zones. Therefore, if you are not in a seismic zone you don't have to worry about maximum spacing as long as the unreinforced masonry can carry the load between the grouted cells. Also, a minimum horizontal reinforcement is required by the SFBC (Section 2704.1), which can be used to calculate the spacing of the vertical reinforcement. By not using this vertical reinforcement a conservative estimate of reinforcement spacing is achieved.

Masonry Wall Design Parameters

8" Concrete Block, hollow unit face shell bedding

$b_{CMU} := 15.625 \cdot in \quad d_{CMU} := 7.625 \cdot in$

$h_{CMU} := 7.625 \cdot in$

width of mortar bed on face shell $\quad d_{shell} := 1.25 \cdot in$

Steel Properties

#5 rebar: ASTM A 615

$A_{steel} := 0.31 \cdot in^2$ per bar

$f_y := 60000 \cdot psi$

$f_s := 24000 \cdot psi$

$E_{steel} := 29.5 \cdot 10^6 \cdot psi$

Masonry Properties

$f_b := 30 \cdot psi$ Allowable Flexure Tension of Hollow Unit Concrete Masonry, Ungrouted from Table 2.2.3.2 of ACI 530-99

$f_m := 1500 \cdot psi$ allowable compression stress

$E_m := 900 \cdot f_m$ for fm of 1500 psi masonry

$E_m = 1.35 \times 10^6\, psi$

Calculate section properties of concrete block bending in vertical direction: Uncracked section

$A_{yy} := d_{shell} \cdot h_{CMU} \cdot 2 \qquad A_{yy} = 19.063\, in^2$

$$I_{yy} := \frac{h_{CMU}}{12} \cdot \left[d_{CMU}^{\,3} - \left(d_{CMU} - 2 \cdot d_{shell}\right)^3 \right] \qquad I_{yy} = 196.16\, in^4$$

$$S_{yy} := \frac{h_{CMU} \cdot \left[d_{CMU}^{\,3} - \left(d_{CMU} - 2 \cdot d_{shell}\right)^3 \right]}{6 \cdot d_{CMU}} \qquad S_{yy} = 51.452\, in^3$$

Limiting moment in wall

$M_{max} := f_b \cdot S_{yy}$ $M_{max} = 128.63\ ft\ lbf$

Wind Load:

$A_{eff} := 8 \cdot 6 \cdot ft^2$ $A_{eff} = 48\ ft^2$

Zone e

$p_{wall} := q_h \cdot (GC_p(A_{eff}, e) + GC_{pi})$ $GC_p(A_{eff}, e) + GC_{pi} = \begin{pmatrix} -1.34 \\ 1.18 \end{pmatrix}$ $p_{wall} = \begin{pmatrix} -33.126 \\ 29.172 \end{pmatrix} psf$

$\omega := p_{wall_0} \cdot h_{CMU}$ $\omega = -21.049 \frac{1}{ft} lbf$

Maximum spacing of reinforcement

$\Delta_{steel} := \sqrt{\frac{12 \cdot M_{max}}{-\omega}}$ Assuming fixed-fixed end conditions

$\Delta_{steel} := floor\left(\frac{\Delta_{steel}}{8 \cdot in}\right) \cdot 8 \cdot in$ round down to nearest 8" multiple (dist between cells) $\Delta_{steel} = 96\ in$ $\Delta_{steel} = 8\ ft$