ASCE7-98

Loads on single story building with roof slope 10-30 degrees

Input Parameters

$in0 := (130 \;\; B \;\; Enclosed)$

Design Wind Speed = 130 mph
Exposure B
Enclosed

Variables for Exposure

$$\begin{pmatrix} A \\ B \\ C \\ D \end{pmatrix} \equiv \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \end{pmatrix}$$

Variables for Enclosed/Part Encl.

$Enclosed \equiv 0$

$PartEnclosed \equiv 1$

Design Parameters

$V := \left| in0^{\langle 0 \rangle} \right| \cdot mph$ $\quad V = 130\,mph$

$I := 1.0$ Importance for Class II Building

$Exp := \left| in0^{\langle 1 \rangle} \right|$ $\quad Case := 1$

Case 1 = C&C and MWFRS for low rise bldgs

$IntPressure := \left| in0^{\langle 2 \rangle} \right|$

Geometry of Building: Mercedes homes

$h := 15 \cdot ft$ ht of building

$\theta := atan\left(\frac{5.5}{12}\right)$ $\quad \theta = 24.62\,deg$ roof slope

$o := 1.0 \cdot ft$ overhang width

$o_g := 1 \cdot ft$

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

$L := 50ft + 2 \cdot o$

$\Delta := 2ft$ Truss spacing

Roof cover: Tile

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

Dead load of roof

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

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

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

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

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

$\phi := 0.6$ Fraction of DeadLoad used in combination with Wind Load

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

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

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

AREAS: Roof - Hip Roof

Vertical Projected Area: wind perpendicular to ridge

$$h_{ridge} := \frac{W}{2}\cdot\tan(\theta) \qquad h_{ridge} = 10.54\,ft$$

$$VPA_{\Gamma} := \frac{h_{ridge}}{2}\cdot[L + (L - W)] \qquad VPA_{\Gamma} = 305.71\,ft^2$$

Vertical Projected Area: wind parallel to ridge

$$VPA_{ll} := \frac{W\cdot h_{ridge}}{2} \qquad VPA_{ll} = 242.46\,ft^2$$

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

Terrain Exposure Constants

$$z_g := \begin{pmatrix} 1500 \cdot ft \\ 1200 \cdot ft \\ 900 \cdot ft \\ 700 \cdot ft \end{pmatrix} \quad \alpha := \begin{pmatrix} 5.0 \\ 7.0 \\ 9.5 \\ 11.5 \end{pmatrix} \quad h_{min} := \begin{pmatrix} 60 \\ 30 \\ 15 \\ 7 \end{pmatrix} \cdot ft$$

Exposures = A,B,C,D

$$h_{min} := \begin{cases} \begin{pmatrix} 100 \\ 30 \\ 15 \\ 15 \end{pmatrix} \cdot ft & \text{if } Case = 1 \\ \begin{pmatrix} 15 \\ 15 \\ 15 \\ 15 \end{pmatrix} \cdot ft & \text{otherwise} \end{cases}$$

$$K_z(h) := \begin{cases} 2.01 \cdot \left(\dfrac{15 \cdot ft}{z_{g_{Exp}}} \right)^{\frac{2}{\alpha_{Exp}}} & \text{if } (h < 15 \cdot ft) \\ 2.01 \cdot \left(\dfrac{h_{min_{Exp}}}{z_{g_{Exp}}} \right)^{\frac{2}{\alpha_{Exp}}} & \text{if } \left(h \le h_{min_{Exp}}\right) \\ 2.01 \cdot \left(\dfrac{h}{z_{g_{Exp}}} \right)^{\frac{2}{\alpha_{Exp}}} & \text{otherwise} \end{cases}$$

$h_{min_{Exp}} = 30\,ft$

$K_z(h) = 0.7$

$K_{zt} := 1.0$ No topographic speedup

$K_d := 0.85$ Directionality factor (0.85 used when doing combination loads - with dead load)

$$q_h := .00256 \frac{slug}{2.15111 ft^3} \cdot K_z(h) \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I$$

$= 25.76\,psf$ Dynamic Wind Pressure

Internal Pressure coefficient

$$GC_{pi} := \begin{cases} \begin{pmatrix} -0.18 \\ 0.18 \end{pmatrix} & \text{if } IntPressure = Enclosed \\ \begin{pmatrix} -0.55 \\ 0.55 \end{pmatrix} & \text{if } IntPressure = PartEnclosed \\ \begin{pmatrix} -20 \\ 20 \end{pmatrix} & \text{otherwise} \end{cases}$$

— Dummy value in Case Int Pressure is invalid

$$GC_{pi} = \begin{pmatrix} -0.18 \\ 0.18 \end{pmatrix}$$

internal pressure range variable

$posneg := 0 .. 1$

Gust Factor:

Terrain Exposure Constants from Table 6-4

$$l := \begin{pmatrix} 180 \\ 320 \\ 500 \\ 650 \end{pmatrix} \cdot ft \qquad \varepsilon := \begin{pmatrix} \frac{1}{2} \\ \frac{1}{3} \\ \frac{1}{5} \\ \frac{1}{8} \end{pmatrix} \qquad c := \begin{pmatrix} 0.45 \\ 0.3 \\ 0.2 \\ 0.15 \end{pmatrix} \qquad z_{min} := \begin{pmatrix} 60 \\ 30 \\ 15 \\ 7 \end{pmatrix} \cdot ft$$

$$z_e := \begin{pmatrix} 0.6 \cdot h \\ z_{min_{Exp}} \end{pmatrix} \qquad z_e := \max(z_e) \qquad z_e = 30\,ft$$

Equivalent height of structure

$$I_z := c_{Exp} \cdot \left(\frac{33 \cdot ft}{z_e}\right)^{\frac{1}{6}} \qquad I_z = 0.3$$

Turbulence Intensity (eqn 6-3)

$$L_z := l_{Exp} \cdot \left(\frac{z_e}{33 \cdot ft}\right)^{\varepsilon_{Exp}} \qquad L_z = 309.99\,ft$$

Integral Length Scale of Turbulence (Eqn 6-5)

$$Q := \sqrt{\frac{1}{1 + 0.63 \cdot \left(\frac{W + h}{L_z}\right)^{0.63}}} \qquad Q = 0.9$$

Background Response (Eqn 6-4)

$$g_Q := 3.4 \qquad g_v := 3.4$$

$$G := 0.925 \cdot \left(\frac{1 + 1.7 \cdot g_Q \cdot I_z \cdot Q}{1 + 1.7 \cdot g_v \cdot I_z}\right) \qquad G = 0.87$$

Gust Factor (Eqn 6-2)

External Pressure Coefficients: Figure 6-5B

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

ASCE7-98: Figure 6-5B Gable/Hip Roofs 10 deg <θ< 30 deg

$$GCp_1 := \begin{pmatrix} -0.9 & -0.8 \\ 0.5 & 0.3 \end{pmatrix} \qquad Alim_1 := (10 \;\; 100)\cdot ft^2$$

$$GCp_2 := \begin{pmatrix} -2.1 & -1.4 \\ 0.5 & 0.3 \end{pmatrix} \qquad Alim_2 := (10 \;\; 100)\cdot ft^2$$

$$GCp_3 := \begin{pmatrix} -2.1 & -1.4 \\ 0.5 & 0.3 \end{pmatrix} \qquad Alim_3 := (10 \;\; 100)\cdot ft^2$$

ASCE7-98: Figure 6-5A

$$GCp_4 := \begin{pmatrix} -1.1 & -0.8 \\ 1.0 & 0.7 \end{pmatrix} \qquad Alim_4 := (10 \;\; 500)\cdot ft^2$$

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

$$GCp_5 := \begin{pmatrix} -1.4 & -0.8 \\ 1.0 & 0.7 \end{pmatrix} \qquad Alim_5 := (10 \;\; 500)\cdot ft^2$$

overhang coefficients

$$GCp_6 := \begin{pmatrix} -2.2 & -2.2 \\ 0 & 0 \end{pmatrix} \qquad Alim_6 := (10 \;\; 100)\cdot ft^2$$

Zone 2

$$GCp_7 := \begin{pmatrix} -3.7 & -2.5 \\ 0 & 0 \end{pmatrix} \qquad Alim_7 := (10 \;\; 100)\cdot ft^2$$

Zone 3

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

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

For Example:

$$GC_p\left(10\cdot ft^2, 4\right) = \begin{pmatrix} -1.1 \\ 1 \end{pmatrix} \qquad GC_p\left(200\cdot ft^2, 5\right) = \begin{pmatrix} -0.94 \\ 0.77 \end{pmatrix} \qquad GC_p\left(100\cdot ft^2, 1\right) = \begin{pmatrix} -0.8 \\ 0.3 \end{pmatrix}$$

$$GC_p\left(200\cdot ft^2, 4\right) = \begin{pmatrix} -0.87 \\ 0.77 \end{pmatrix} \qquad GC_p\left(10\cdot ft^2, 6\right) = \begin{pmatrix} -2.2 \\ 0 \end{pmatrix}$$

Window Design Pressure

The following input table was imported from an excel sheet that had a list of fens for this building. Each column represents the width, 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	4	1
1	16	7	112	45	0.19
2	3	4	12	4	1
3	4	5	20	4	1
4	4	5	20	4	1
5	4	5	20	4	1
6	6	6.7	40.2	4	1
7	4	5	20	4	1
8	4	5	20	4	1
9	6	6	36	5	1
10	6	6	36	4	1
11					
12					

When Zone = 45, Fraction represents portion of fen in Zone 5.

Garage door ratio in Zone 5 is 24.5 SF of 112 SF

Garage Effective Area should be set by considering single spanning panel that is 16ft wide, however industry practice uses entire size of door. (Difference between these two methods is about 2 psf)

$rows(Fen) = 11$

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

$$DP^{\langle j \rangle} := \begin{cases} q_h \cdot \left(GC_p\left(\overrightarrow{\left[\left(Fen^{\langle Size \rangle} \right)_j \cdot ft^2 \right]}, \overrightarrow{\left(Fen^{\langle Zone \rangle} \right)_j} \right) + GC_{pi} \right) & \text{if } \left(Fen^{\langle Zone \rangle} \right)_j \neq 45 \\ \left[\begin{array}{l} q_h \cdot \left(GC_p\left(\overrightarrow{\left[\left(Fen^{\langle Size \rangle} \right)_j \cdot ft^2 \right]}, 5 \right) + GC_{pi} \right) \cdot \left(Fen^{\langle Fraction \rangle} \right)_j \dots \\ + q_h \cdot \left(GC_p\left(\overrightarrow{\left[\left(Fen^{\langle Size \rangle} \right)_j \cdot ft^2 \right]}, 4 \right) + GC_{pi} \right) \cdot \left[1 - \left(Fen^{\langle Fraction \rangle} \right)_j \right] \end{array} \right] & \text{otherwise} \end{cases}$$

Effective Area of fenestrations should be set according to the area of the element resisting the load, as opposed to the area of the entire fenestration. For example, a sliding glass door is made of 3 doors spanning vertically, each door is 4x8. The doors do not transfer wind load horizontally, therefore the wind loads are correlated only over the single door, and thus instead of an effective area of 96 square feet, the effective area is 32 square feet.

DP =

	0	1	2	3	4	5	6	7	8	9	10
0	-31.25	-28.76	-32.62	-31.61	-31.61	-31.61	-30.23	-31.61	-31.61	-35.65	-30.45
1	28.67	25.63	30.04	29.03	29.03	29.03	27.65	29.03	29.03	27.87	27.87

psf

for Sliding Glass door : Design pressures are: $DP^{\langle 4 \rangle} = \begin{pmatrix} -31.61 \\ 29.03 \end{pmatrix} psf$

Design of Nailing Pattern for Roof Deck

Tributary area for single fastener: $Area := 10\cdot ft^2$

Zone1	Zone 2	Zone 3
$GC_p(Area,1) = \begin{pmatrix} -0.9 \\ 0.5 \end{pmatrix}$	$GC_p(Area,2) = \begin{pmatrix} -2.1 \\ 0.5 \end{pmatrix}$	$GC_p(Area,3) = \begin{pmatrix} -2.1 \\ 0.5 \end{pmatrix}$

Design load: Zone2

$p_{single} := q_h\cdot(GC_p(Area,2) + GC_{pi})$ $\qquad p_{single} = \begin{pmatrix} -58.74 \\ 17.52 \end{pmatrix} psf$

Tributary area for single sheet of plywood fastener: $Area := 32\cdot ft^2$

One 4x8ft sheet of plywood/OSB = 32 FT tributary area

Zone1	Zone 2	Zone 3
$GC_p(Area,1) = \begin{pmatrix} -0.85 \\ 0.4 \end{pmatrix}$	$GC_p(Area,2) = \begin{pmatrix} -1.75 \\ 0.4 \end{pmatrix}$	$GC_p(Area,3) = \begin{pmatrix} -1.75 \\ 0.4 \end{pmatrix}$

$p_{panel} := q_h\cdot(GC_p(Area,2) + GC_{pi})$ $\qquad p_{panel} = \begin{pmatrix} -49.63 \\ 14.92 \end{pmatrix} psf$

Resistance of single 8d Nail

Load Case : Wind + 60% of dead load

$q_r := 41\cdot\frac{lbf}{in}$ 8d common nail, NDS 1997, page 30, diameter 0.131", specific Gravity 0.55 (Southern Pine)

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

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

Southern Pine SG - 0.55 on page 29, Table 12A of NDS-S97

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

$R_{nail} := q_r\cdot l_p\cdot C_D\cdot C_m$ $\qquad R_{nail} = 131.2\,lbf$

Maximum Spacing for 8d nail:

$$A_t := \frac{R_{nail}}{\left(\left|p_{single_0} + 0.6 \cdot DL_{sheath}\right| \cdot 2 \cdot ft\right)}$$ $A_t = 13.62\,in$ maximum required spacing of fasteners

Select nailing pattern that meets max spacing criteria

practical number of nails that meets nailing spacing criteria listed above (Zone 2/3)

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

lookup nailing pattern to meet Zone2/3

$$II_s := \text{floor}\left(\text{linterp}\left(s_{possible}, II, A_t\right)\right)$$

$$s_i := s_{possible_{II_s}}$$

spacing, nails

NailSched =

4.36	12
4.8	11
5.33	10
6	9
6.86	8
8	7
9.6	6
12	5
16	4
24	3
48	2

USE the following spacing:

$s_e := 6in$ edge spacing $s_i = 12\,in$ interior spa

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

Check whole panel resistance

$$L_{panel} := \left(\left|p_{panel_0} + 0.6 \cdot DL_{sheath}\right|\right) \cdot 32ft^2$$ $L_{panel} = 1557.48\,lbf$ uplift

$R_{total} := R_{nail} \cdot N_{nails}$ $R_{total} = 4329.6\,lbf$

$Status_{RoofNail} := R_{total} > L_{panel}$ $Status_{RoofNail} = 1$ PASS = 1, FAIL = 0

ROOF STRAPS DESIGN (Uplift): Design of Typical Truss at Center of Building

Several methods of calculating the uplift on the truss have been explored here. 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.

1. The first method is considering the C&C loads that are acting on a single truss in the middle of the roof.
2. The second method is summing up the total MWFRS load pattern and dividing by the number of straps in the roof.
3. The third method is to sum moments of the MWFRS interior zone loads for a truss at the center of the building.

In addition, for comparison to prescriptive documents, the corner truss load has also be considered by two methods: C&C loads and MWFRS loads.

Edge zone

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

$$l_r := \frac{W}{2\cdot\cos(\theta)} \qquad l_r = 25.3\,ft$$ length of top chord of truss

Notes:
1. HUD RSDG 2000 and SSTD10 specifies that roof uplift for design of rood tie-downs should be determined using "MWFRS" loads

$$a_\theta := \frac{a}{\cos(\theta)}$$ length of edge zones along roof slope - assume that "a" in ASCE7 figures are widths in plan.

<u>Method 1: Center Roof Truss Design based on Components and Cladding loads from ASCE 7-98</u>

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

$$A_{eff} := \left(\begin{pmatrix} W\cdot\Delta \\ W\cdot\frac{W}{3} \end{pmatrix}\right) \quad A_{eff} = \begin{pmatrix} 92 \\ 705.33 \end{pmatrix} ft^2 \quad A_{eff} := \max(A_{eff}) \quad A_{eff} = 705.33\,ft^2$$

External Gust Factors

$$GC_p(A_{eff},1) = \begin{pmatrix} -0.8 \\ 0.3 \end{pmatrix} \quad GC_p(A_{eff},2) = \begin{pmatrix} -1.4 \\ 0.3 \end{pmatrix} \quad GC_p(A_{eff},3) = \begin{pmatrix} -1.4 \\ 0.3 \end{pmatrix}$$

$$k := 1..3 \qquad p_k := \left(GC_p(A_{eff},k)_0 + GC_{pi_0}\right)\cdot q_h$$

$$p = \begin{pmatrix} 0 \\ -25.25 \\ -40.71 \\ -40.71 \end{pmatrix} psf$$ Design Pressures for Zones 1, 2, and 3

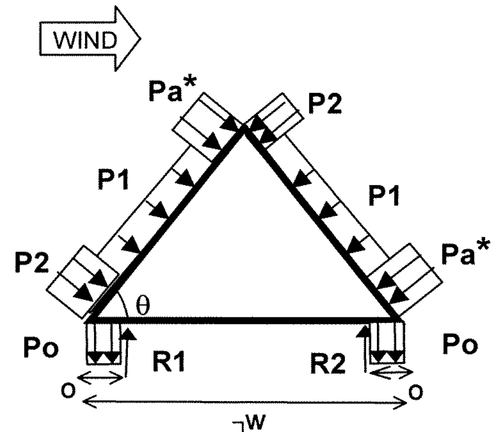
$V = 130\,mph$
$Exp = 1$

Overhang pressures $$p_o := \left(GC_p(A_{eff},6)_0\right)\cdot q_h \qquad GC_p(A_{eff},6) = \begin{pmatrix} -2.2 \\ 0 \end{pmatrix} \qquad p_o = -56.68\,psf$$

WIND Perpendicular to Ridge: Loading pattern according to ASCE 7-95 guide by K. Metha

Set pa equal to p1, because ASCE7-98 guidebook indicates that truss loads should follow patterns where Zone2 is not applied simultaneously to all locations according to wind tunnel tests.

$$p_a := p_1$$



Sum Moments: about R2 reaction point

$$
\begin{aligned}
R_1 := \frac{1}{W-2\cdot o}\cdot\Bigg[& p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\cos(\theta)\cdot\left(W-o-\frac{o}{2}\right)\ \ldots \\
& + p_2\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\cos(\theta)\cdot\left(W-o-\frac{a-o}{2}-o\right)\ \ldots \\
& + p_2\cdot a_\theta\cdot\Delta\cdot\cos(\theta)\cdot\left(\frac{W}{2}-o-\frac{a_\theta}{2}\cdot\cos(\theta)\right)\ \ldots \\
& + p_a\cdot a_\theta\cdot\Delta\cdot\cos(\theta)\cdot\left[W-o-\left(l_r-\frac{a_\theta}{2}\right)\cdot\cos(\theta)\right]\ \ldots \\
& + \left[p_a\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\cos(\theta)\cdot\left[\frac{1}{2}\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\right]\right]\ \ldots \\
& + \left[p_1\cdot(l_r-2\cdot a_\theta)\cdot\Delta\cdot\cos(\theta)\cdot\left[\left(W-o-\frac{l_r}{2}\cdot\cos(\theta)\right)+\left(\frac{W}{2}-o-\frac{l_r}{2}\cdot\cos(\theta)\right)\right]\right]\ \ldots \\
& + -p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\cos(\theta)\cdot\left(\frac{o}{2}\right)\ \ldots \\
& + -\left[p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{o}{2\cdot\cos(\theta)}\cdot\sin(\theta)\right)\right]\ \ldots \\
& + -\left[p_2\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left[a_\theta-\frac{1}{2}\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\right]\cdot\sin(\theta)\right]\right]\ \ldots \\
& + -\left[p_1\cdot(l_r-2\cdot a_\theta)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right]\ \ldots \\
& + -\left[p_a\cdot a_\theta\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r-\frac{a_\theta}{2}\right)\sin(\theta)\right]\ \ldots \\
& + p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{o}{2\cdot\cos(\theta)}\cdot\sin(\theta)\right)\ \ldots \\
& + p_1\cdot(l_r-2\cdot a_\theta)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\ \ldots \\
& + p_2\cdot a_\theta\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r-\frac{a_\theta}{2}\right)\cdot\sin(\theta)\ \ldots \\
& + p_a\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left[a_\theta-\frac{1}{2}\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\right]\cdot\sin(\theta)\right]\ \ldots \\
& + \phi\cdot DL_{roof}\cdot\Delta\cdot W\cdot\left(\frac{W}{2}-o\right)\Bigg]
\end{aligned}
$$

Dead load factor, ASD

$$\phi = 0.6$$

$$R_1 = -1158.89\,\text{lbf}$$

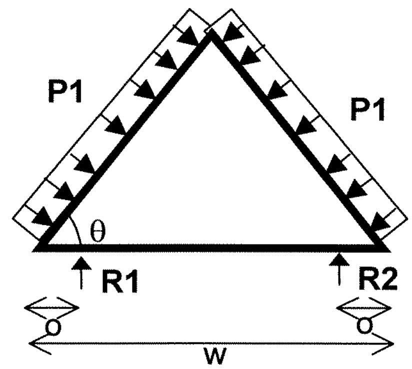
Sum Forces in Vertical

$$R_2 := \left[\left[\begin{array}{l} 2\cdot\left(p_o\cdot\frac{o}{\cos(\theta)}\cdot\cos(\theta)\cdot\Delta\right)\ \ldots \\ +\left[p_2\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta\right]\ \ldots \\ +\left(p_2\cdot a_\theta\cdot\cos(\theta)\cdot\Delta\right)\ \ldots \\ +2\cdot p_1\cdot\left(l_r - 2\cdot a_\theta\right)\cdot\cos(\theta)\cdot\Delta\ \ldots \\ +\left(p_a\cdot a_\theta\cdot\cos(\theta)\cdot\Delta\right)\ \ldots \\ +p_a\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta \end{array}\right] + \phi\cdot DL_{roof}\cdot(\Delta\cdot W)\right] - R_1$$

$$R_2 = -1046.43\ \text{lbf}$$

WIND Parallel to Ridge

$$R_3 := \frac{\Delta}{W - 2\cdot o}\cdot\left[\begin{array}{l} p_1\cdot l_r\cdot\cos(\theta)\cdot\left[\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) + \left(\frac{l_r}{2}\cdot\cos(\theta) - o\right)\right]\ \ldots \\ +\phi\cdot DL_{roof}\cdot W\cdot\left(\frac{W}{2} - o\right) \end{array}\right]$$



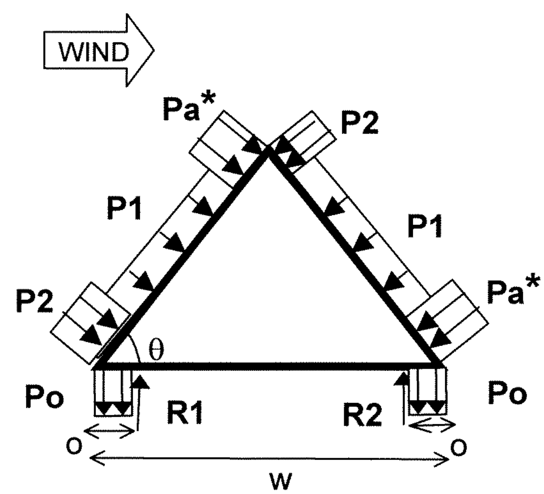
$$R_4 := 2\cdot p_1\cdot l_r\cdot\Delta\cdot\cos(\theta) - R_3 + \phi\cdot DL_{roof}\cdot\Delta\cdot W$$

$$R_3 = -913.03\ \text{lbf}$$

$$R_4 = -913.03\ \text{lbf}$$

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).

$$p_a := p_2$$



$$R_1 := \frac{1}{W - 2\cdot o}\cdot\left[\begin{array}{l} p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\cos(\theta)\cdot\left(W - o - \frac{o}{2}\right)\ ... \\ + p_2\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\cos(\theta)\cdot\left(W - o - \frac{a - o}{2} - o\right)\ ... \\ + p_2\cdot a_\theta\cdot\Delta\cdot\cos(\theta)\cdot\left(\frac{W}{2} - o - \frac{a_\theta}{2}\cdot\cos(\theta)\right)\ ... \\ + p_a\cdot a_\theta\cdot\Delta\cdot\cos(\theta)\cdot\left[W - o - \left(l_r - \frac{a_\theta}{2}\right)\cdot\cos(\theta)\right]\ ... \\ + \left[p_a\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\cos(\theta)\cdot\left[\frac{1}{2}\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\right]\right]\ ... \\ + \left[p_1\cdot(l_r - 2\cdot a_\theta)\cdot\Delta\cdot\cos(\theta)\cdot\left[\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) + \left(\frac{W}{2} - o - \frac{l_r}{2}\cdot\cos(\theta)\right)\right]\right]\ ... \\ + -p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\cos(\theta)\cdot\left(\frac{o}{2}\right)\ ... \\ + -\left[p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{o}{2\cdot\cos(\theta)}\cdot\sin(\theta)\right)\right]\ ... \\ + -\left[p_2\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left[a_\theta - \frac{1}{2}\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\right]\cdot\sin(\theta)\right]\right]\ ... \\ + -\left[p_1\cdot(l_r - 2\cdot a_\theta)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right]\ ... \\ + -\left[p_a\cdot a_\theta\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r - \frac{a_\theta}{2}\right)\sin(\theta)\right]\ ... \\ + p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{o}{2\cdot\cos(\theta)}\cdot\sin(\theta)\right)\ ... \\ + p_1\cdot(l_r - 2\cdot a_\theta)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\ ... \\ + p_2\cdot a_\theta\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r - \frac{a_\theta}{2}\right)\cdot\sin(\theta)\ ... \\ + p_a\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left[a_\theta - \frac{1}{2}\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\right]\cdot\sin(\theta)\right]\ ... \\ + \phi\cdot DL_{roof}\cdot\Delta\cdot W\cdot\left(\frac{W}{2} - o\right) \end{array}\right]$$

$p_2 = -40.71\ \text{psf}$

$p_1 = -25.25\ \text{psf}$

$p_o = -56.68\ \text{psf}$

$p_a = -40.71\ \text{psf}$

$a_\theta = 5.06\ \text{ft}$

$W = 46\ \text{ft}$

$l_r = 25.3\ \text{ft}$

$\Delta = 2\ \text{ft}$

$R_1 = -1229.41\ \text{lbf}$

$$R_2 := \left[\left[\begin{array}{l} 2\cdot\left(p_o\cdot\frac{o}{\cos(\theta)}\cdot\cos(\theta)\cdot\Delta\right)\ ... \\ + \left[p_2\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta\right]\ ... \\ + \left(p_2\cdot a_\theta\cdot\cos(\theta)\cdot\Delta\right)\ ... \\ + 2\cdot p_1\cdot(l_r - 2\cdot a_\theta)\cdot\cos(\theta)\cdot\Delta\ ... \\ + \left(p_a\cdot a_\theta\cdot\cos(\theta)\cdot\Delta\right)\ ... \\ + p_a\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta \end{array}\right] + \phi\cdot DL_{roof}\cdot(\Delta\cdot W)\right] - R_1$$

$R_2 = -1229.41\ \text{lbf}$

Compared to theoretically correct loading pattern:

$$\frac{\text{Pattern\_Load}}{\text{Full\_Zone\_Load}} \qquad \frac{R_1}{R_1} = 0.94 \qquad \frac{R_2}{R_2} = 0.85$$

$$R_1 := R_1 \qquad R_2 := R_2$$ Use full pattern loading

My theorectically correct loading pattern produces maximum uplifts that are only ~6-7% lower than the full pattern loading. Therefore, since ASCE7 doesn't clearly indicate the pattern loading that is considered appropriate, and the difference is relatively minor, then we will default to full pattern loading.

Method 2: 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 6-4: Walls and Gable Roof

CASE A Table from Figure 6-4

$$roofAng := \begin{pmatrix} 0 \\ 5 \\ 20 \\ 30 \\ 45 \\ 90 \end{pmatrix} \quad casea := \begin{pmatrix} 0.40 & -0.69 & -0.37 & -0.29 & 0.61 & -1.07 & -0.53 & -0.43 \\ 0.40 & -0.69 & -0.37 & -0.29 & 0.61 & -1.07 & -0.53 & -0.43 \\ 0.53 & -0.69 & -0.48 & -0.43 & 0.80 & -1.07 & -0.69 & -0.64 \\ 0.56 & 0.21 & -0.43 & -0.37 & 0.69 & 0.27 & -0.53 & -0.48 \\ 0.56 & 0.21 & -0.43 & -0.37 & 0.69 & 0.27 & -0.53 & -0.48 \\ 0.56 & 0.56 & -0.37 & -0.37 & 0.69 & 0.69 & -0.48 & -0.48 \end{pmatrix}$$

$zoneA := 0..7$ range of values in CASE A table

$zoneB := 0..11$ range of values in CASE B table

$$GC_{pfA1_{zoneA}} := \text{linterp}\left(roofAng, casea^{\langle zoneA \rangle}, \frac{\theta}{deg}\right)$$

Interpolated for roof slope $\theta = 24.62\,deg$

$$GC_{pfA2_{zoneA}} := \text{linterp}\left(roofAng, casea^{\langle zoneA \rangle}, 0\right)$$

Roof Slope equal 0, See Note 2 in Figure 6-4

Corner 1 $\quad GC_{pfA1}^{T} = \begin{pmatrix} 0.54 & -0.27 & -0.46 & -0.4 & 0.75 & -0.45 & -0.62 & -0.57 \end{pmatrix}$

Corner 2 $\quad GC_{pfA2}^{T} = \begin{pmatrix} 0.4 & -0.69 & -0.37 & -0.29 & 0.61 & -1.07 & -0.53 & -0.43 \end{pmatrix}$

CASE B from Figure 6-4

Corner 1 $\quad GC_{pfB1} := \begin{pmatrix} -0.45 & -0.69 & -0.37 & -0.45 & 0.4 & -0.29 & -0.48 & -1.07 & -0.53 & -0.48 & 0.61 & -0.43 \end{pmatrix}^{T}$

Corner 2 $\quad GC_{pfB2} := GC_{pfB1}$

Pressures

$$p_{A1_{posneg,zoneA}} := q_h \cdot \left(GC_{pfA1_{zoneA}} + GC_{pi_{posneg}}\right) \qquad p_{B1_{posneg,zoneB}} := q_h \cdot \left(GC_{pfB1_{zoneB}} + GC_{pi_{posneg}}\right)$$

$$p_{A2_{posneg,zoneA}} := q_h \cdot \left(GC_{pfA2_{zoneA}} + GC_{pi_{posneg}}\right) \qquad p_{B2} := p_{B1}$$

$$p_{A1} = \begin{pmatrix} 9.37 & -11.69 & -16.41 & -15 & 14.66 & -16.24 & -20.51 & -19.22 \\ 18.65 & -2.42 & -7.13 & -5.73 & 23.94 & -6.97 & -11.23 & -9.95 \end{pmatrix} psf$$

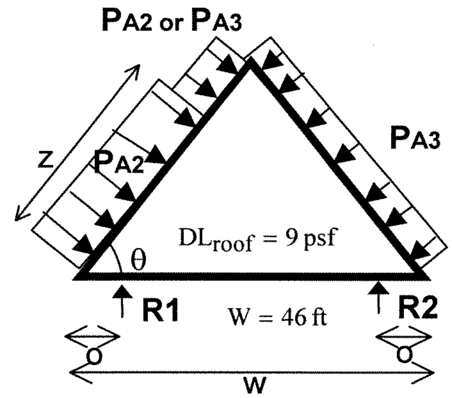
Note: No Overhang Loads as part of MWFRS

$p_{B1} =$ (psf)

-16.23	-22.41	-14.17	-16.23	5.67	-12.11	-17	-32.2	-18.29	-17	11.08	-15.72
-6.96	-13.14	-4.9	-6.96	14.94	-2.83	-7.73	-22.93	-9.02	-7.73	20.35	-6.44

$$p_{A2} = \begin{pmatrix} 5.67 & -22.41 & -14.17 & -12.11 & 11.08 & -32.2 & -18.29 & -15.72 \\ 14.94 & -13.14 & -4.9 & -2.83 & 20.35 & -22.93 & -9.02 & -6.44 \end{pmatrix} psf$$

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



z = width of zone 2 on roof parallel to wind direction

$$z := \begin{pmatrix} 0.5 \cdot W \\ 2.5 \cdot h \end{pmatrix} \qquad z = \begin{pmatrix} 23 \\ 37.5 \end{pmatrix} \text{ft}$$

$$z := \min(z) \qquad z = 23 \text{ ft}$$

Note: Figure 6-4 indicates that zone 2 pressure extends for distance of z only, if zone 2 pressure is negative

$$p_{A1}{}^{\langle A2 \rangle} = \begin{pmatrix} -11.69 \\ -2.42 \end{pmatrix} \text{psf}$$

$$p_{A1}{}^{\langle A3 \rangle} = \begin{pmatrix} -16.41 \\ -7.13 \end{pmatrix} \text{psf}$$

<u>CASE A Corner 1</u> Sum moments about R2 reaction of load distribution

$$R_{1_{posneg}} := \frac{1}{W - 2 \cdot o} \cdot \Bigg[\Bigg[\left[\left(p_{A1}{}^{\langle A2 \rangle}\right)_{posneg} \cdot z \cdot \Delta \cos(\theta) \cdot \left(W - o - \frac{z}{2} \cdot \cos(\theta) \right) \right] \ldots$$
$$+ \left[\begin{array}{ll} \left(p_{A1}{}^{\langle A3 \rangle}\right)_{posneg} & \text{if } \left(p_{A1}{}^{\langle A2 \rangle}\right)_{posneg} < 0 \\ \left(p_{A1}{}^{\langle A2 \rangle}\right)_{posneg} & \text{otherwise} \end{array} \right] \cdot (l_r - z) \cdot \Delta \cos(\theta) \cdot \left[\left(\frac{W}{2} - o \right) \ldots + \left[\left(\frac{l_r - z}{2} \right) \cdot \cos(\theta) \right] \right] \ldots$$
$$+ \left(p_{A1}{}^{\langle A3 \rangle}\right)_{posneg} \cdot l_r \cdot \Delta \cos(\theta) \cdot \left(\frac{l_r}{2} \cdot \cos(\theta) - o \right) \Bigg]$$
$$+ \Bigg[-\left(p_{A1}{}^{\langle A2 \rangle}\right)_{posneg} \cdot z \cdot \Delta \cdot \sin(\theta) \cdot \left(\frac{z}{2} \cdot \sin(\theta) \right) \ldots$$
$$+ \left[\begin{array}{ll} -\left(p_{A1}{}^{\langle A3 \rangle}\right)_{posneg} & \text{if } \left(p_{A1}{}^{\langle A2 \rangle}\right)_{posneg} < 0 \\ -\left(p_{A1}{}^{\langle A2 \rangle}\right)_{posneg} & \text{otherwise} \end{array} \right] \cdot (l_r - z) \cdot \Delta \cdot \sin(\theta) \cdot \left(z + \frac{l_r - z}{2} \right) \cdot \sin(\theta) \ldots$$
$$+ \left(p_{A1}{}^{\langle A3 \rangle}\right)_{posneg} \cdot l_r \cdot \Delta \cdot \sin(\theta) \cdot \left(\frac{l_r}{2} \cdot \sin(\theta) \right) \Bigg] \ldots$$
$$+ \phi \cdot DL_{roof} \cdot W \cdot \Delta \cdot \left(\frac{W}{2} - o \right) \Bigg]$$

Sum forces in vertical direction

$$R_1 = \begin{pmatrix} -361.44 \\ 65.21 \end{pmatrix} \text{lbf}$$

$$R_{2_{posneg}} := \left(p_{A1}{}^{\langle A2 \rangle}\right)_{posneg} \cdot z \cdot \Delta \cdot \cos(\theta) \ldots$$
$$+ \left[\begin{array}{ll} \left(p_{A1}{}^{\langle A3 \rangle}\right)_{posneg} & \text{if } \left(p_{A1}{}^{\langle A2 \rangle}\right)_{posneg} < 0 \\ \left(p_{A1}{}^{\langle A2 \rangle}\right)_{posneg} & \text{otherwise} \end{array} \right] \cdot (l_r - z) \cdot \Delta \cdot \cos(\theta) \ldots$$
$$+ \left(p_{A1}{}^{\langle A3 \rangle}\right)_{posneg} \cdot l_r \cdot \Delta \cdot \cos(\theta) \ldots$$
$$+ -R_{1_{posneg}} + \phi \cdot DL_{roof} \cdot \Delta \cdot W$$

$$R_2 = \begin{pmatrix} -454.19 \\ -27.54 \end{pmatrix} \text{lbf}$$

$$R_T := \text{stack}(R_1, R_2) \qquad R_{MWF_0} := \min((R_T)) \qquad R_{MWF_0} = -454.19 \text{ lbf}$$

CASE B Corner 1

$$R_1 := \frac{1}{W - 2\cdot o}\cdot\left[\left[p_{B1}^{\langle B2\rangle}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) \ldots + p_{B1}^{\langle B3\rangle}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta) - o\right)\right] \ldots + \left[-p_{B1}^{\langle B2\rangle}\cdot l_r\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right) \ldots + p_{B1}^{\langle B3\rangle}\cdot l_r\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right] \ldots + \phi\cdot DL_{roof}\cdot W\cdot\Delta\cdot\left(\frac{W}{2} - o\right)\right]$$

$$p_{B1}^{\langle B2\rangle} = \begin{pmatrix}-22.41\\-13.14\end{pmatrix} psf$$

$$p_{B1}^{\langle B3\rangle} = \begin{pmatrix}-14.17\\-4.9\end{pmatrix} psf$$

$$R_1 = \begin{pmatrix}-671.35\\-244.7\end{pmatrix} lbf$$

$$R_2 := \left(p_{B1}^{\langle B2\rangle}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) + \left(p_{B1}^{\langle B3\rangle}\cdot l_r\cdot\Delta\cdot\cos(\theta)\right) - R_1 + DL_{roof}\cdot\Delta\cdot W$$

$$R_2 = \begin{pmatrix}-183.55\\243.1\end{pmatrix} lbf$$

$R_T := \text{stack}(R_1, R_2)$ $\quad R_{MWF_1} := \min((R_T))$ $\quad R_{MWF_1} = -671.35\, lbf$

CASE A Corner 2

Assume truss is on windward side of imaginary line drawn for distance z from windward edge. All wind zones are Zone 2 or 2E.

$$R_1 := \frac{1}{W - 2\cdot o}\cdot\left[\left[\left[p_{A2}^{\langle A2E\rangle}\cdot 2\cdot a_\theta\cdot\Delta\cos(\theta)\cdot\left(W - o - a_\theta\cdot\cos(\theta)\right)\right] \ldots + \left[p_{A2}^{\langle A2\rangle}\cdot\left(l_r - 2\cdot a_\theta\right)\cdot\Delta\cos(\theta)\cdot\left[W - o - 2\cdot a_\theta\cdot\cos(\theta) - \frac{\left(l_r - 2\cdot a_\theta\right)}{2}\cdot\cos(\theta)\right]\right] \ldots + p_{A2}^{\langle A2\rangle}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta) - o\right)\right] \ldots + \left[\left[-p_{A2}^{\langle A2E\rangle}\cdot\left(2\cdot a_\theta\right)\cdot\Delta\cdot\sin(\theta)\cdot\left(a_\theta\cdot\sin(\theta)\right)\right] \ldots + \left[-p_{A2}^{\langle A2\rangle}\cdot\left(l_r - 2\cdot a_\theta\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left(2\cdot a_\theta + \frac{l_r - 2\cdot a_\theta}{2}\right)\cdot\sin(\theta)\right]\right] \ldots + p_{A2}^{\langle A2\rangle}\cdot\left(l_r\cdot\Delta\right)\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right] \ldots + \phi\cdot DL_{roof}\cdot W\cdot\Delta\cdot\left(\frac{W}{2} - o\right)\right]$$

$$R_1 = \begin{pmatrix}-944.11\\-517.47\end{pmatrix} lbf$$

$$p_{A2}^{\langle A2E\rangle} = \begin{pmatrix}-32.2\\-22.93\end{pmatrix} psf$$

$$p_{A2}^{\langle A2\rangle} = \begin{pmatrix}-22.41\\-13.14\end{pmatrix} psf$$

$$R_2 := p_{A2}^{\langle A2E\rangle}\cdot 2\cdot a_\theta\cdot\Delta\cdot\cos(\theta) + p_{A2}^{\langle A2\rangle}\cdot\left(2\cdot l_r - 2\cdot a_\theta\right)\cdot\Delta\cdot\cos(\theta) \ldots + -R_1 + \phi\cdot DL_{roof}\cdot\Delta\cdot W$$

$$R_2 = \begin{pmatrix}-801.36\\-374.71\end{pmatrix} lbf$$

$R_T := \text{stack}(R_1, R_2)$ $\quad R_{MWF_2} := \min((R_T))$ $\quad R_{MWF_2} = -944.11\, lbf$

CASE B Corner 2

Assume truss is on windward side of imaginary ridge line. All wind zones are Zone 2 or 2E.

$$R_1 := \frac{1}{W-2\cdot o}\cdot\left[\left[\left[p_{B2}^{\langle B2E\rangle}\cdot 2\cdot a_\theta\cdot\Delta\cos(\theta)\cdot\left(W-o-a_\theta\cdot\cos(\theta)\right)\right]\ldots + \left[p_{B2}^{\langle B2\rangle}\cdot\left(l_r-2\cdot a_\theta\right)\cdot\Delta\cos(\theta)\cdot\left[W-o-2\cdot a_\theta\cdot\cos(\theta)-\frac{\left(l_r-2\cdot a_\theta\right)}{2}\cdot\cos(\theta)\right]\right]\ldots + p_{B2}^{\langle B2\rangle}\cdot l_r\cdot\Delta\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta)-o\right)\right]\ldots + \left[\left[-p_{B2}^{\langle B2E\rangle}\cdot 2\cdot a_\theta\cdot\Delta\cdot\sin(\theta)\cdot\left(a_\theta\cdot\sin(\theta)\right)\right]\ldots + \left[-p_{B2}^{\langle B2\rangle}\cdot\left(l_r-2\cdot a_\theta\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left(2\cdot a_\theta+\frac{l_r-2\cdot a_\theta}{2}\right)\cdot\sin(\theta)\right]\right]\ldots + p_{B2}^{\langle B2\rangle}\cdot l_r\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right]\ldots + \phi\cdot DL_{roof}\cdot W\cdot\Delta\cdot\left(\frac{W}{2}-o\right)\right]$$

$$p_{B2}^{\langle B2E\rangle} = \begin{pmatrix}-32.2\\-22.93\end{pmatrix}\text{psf}$$

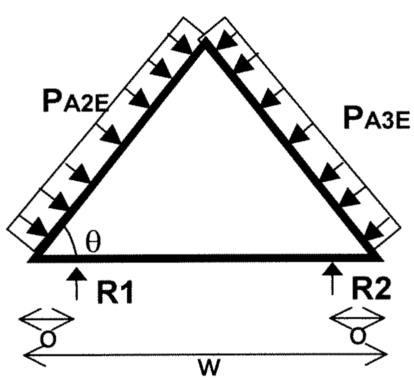
$$R_1 = \begin{pmatrix}-944.11\\-517.47\end{pmatrix}\text{lbf}$$

$$p_{B2}^{\langle B2\rangle} = \begin{pmatrix}-22.41\\-13.14\end{pmatrix}\text{psf}$$

$$R_2 := p_{B2}^{\langle B2E\rangle}\cdot 2\cdot a_\theta\cdot\Delta\cdot\cos(\theta) + p_{B2}^{\langle B2\rangle}\cdot\left(2\cdot l_r-2\cdot a_\theta\right)\cdot\Delta\cdot\cos(\theta)\ldots + -R_1 + \phi\cdot DL_{roof}\cdot\Delta\cdot W$$

$$R_2 = \begin{pmatrix}-801.36\\-374.71\end{pmatrix}\text{lbf} \qquad R_T := \text{stack}(R_1, R_2) \qquad R_{MWF_3} := \min\left(\left(R_T\right)\right) \qquad R_{MWF_3} = -944.11\,\text{lbf}$$

Corner Straps - 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.

$l_r = 25.3\,\text{ft}$

$DL_{roof} = 9\,\text{psf}$

$2\cdot a = 9.2\,\text{ft}$

$\Delta = 2\,\text{ft}$

$o_g = 1\,\text{ft}$

CASE A: Corner 1

$$R_1 := \frac{1}{W - 2\cdot o}\cdot\left[\left[\left[p_{A1}^{\langle A2E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) \ldots \\ + \left[p_{A1}^{\langle A3E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta) - o\right) \ldots \\ + \left[\left[-p_{A1}^{\langle A2E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right] \ldots \\ + \left[p_{A1}^{\langle A3E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right] \ldots \\ + \phi\cdot DL_{roof}\cdot W\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot\left(\frac{W}{2} - o\right)\right]$$

$$p_{A1}^{\langle A2E\rangle} = \begin{pmatrix} -16.24 \\ -6.97 \end{pmatrix} psf \qquad p_{A1}^{\langle A3E\rangle} = \begin{pmatrix} -20.51 \\ -11.23 \end{pmatrix} psf$$

$$R_1 = \begin{pmatrix} -556.36 \\ -129.71 \end{pmatrix} lbf$$

$$R_2 := \left[\left[p_{A1}^{\langle A2E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\cos(\theta)\right] \ldots \\ + \left[\left[p_{A1}^{\langle A3E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\cos(\theta)\right] - R_1 + \phi\cdot DL_{roof}\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot W$$

$$R_2 = \begin{pmatrix} -637.39 \\ -210.74 \end{pmatrix} lbf$$

$$R_T := \mathrm{stack}(R_1, R_2) \qquad R_{MWFc_0} := \min((R_T)) \qquad R_{MWFc_0} = -637.39\,lbf$$

CASE B: Corner 1

$$R_1 := \frac{1}{W - 2\cdot o}\cdot\left[\left[\left[p_{B1}^{\langle B2E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\cos(\theta)\cdot\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) \ldots \\ + \left[p_{B1}^{\langle B3E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\cos(\theta)\cdot\left(\frac{l_r}{2}\cdot\cos(\theta) - o\right) \ldots \\ + \left[\left[-p_{B1}^{\langle B2E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right] \ldots \\ + \left[p_{B1}^{\langle B3E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right] \ldots \\ + \phi\cdot DL_{roof}\cdot W\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot\left(\frac{W}{2} - o\right)\right]$$

$$p_{B1}^{\langle B2E\rangle} = \begin{pmatrix} -32.2 \\ -22.93 \end{pmatrix} psf \qquad p_{B1}^{\langle B3E\rangle} = \begin{pmatrix} -18.29 \\ -9.02 \end{pmatrix} psf$$

$$R_1 = \begin{pmatrix} -1045.16 \\ -618.51 \end{pmatrix} lbf$$

$$R_2 := \left[\left[p_{B1}^{\langle B2E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\cos(\theta) \ldots \\ + \left[p_{B1}^{\langle B3E\rangle}\cdot l_r\cdot\left(\frac{\Delta}{2} + o_g\right)\right]\cdot\cos(\theta)\right] - R_1 + \phi\cdot DL_{roof}\cdot\left(\frac{\Delta}{2} + o_g\right)\cdot W$$

$$R_2 = \begin{pmatrix} -780.91 \\ -354.26 \end{pmatrix} lbf$$

$$R_T := \mathrm{stack}(R_1, R_2) \qquad R_{MWFc_1} := \min((R_T)) \qquad R_{MWFc_1} = -1045.16\,lbf$$

CASE A and B for Corner 2

will not govern end truss uplift by inspection.

Summary of Strap Design

Strap Design of interior zone truss:

Components and Cladding: Interior Truss

$$R = \begin{pmatrix} 0 \\ -1229.41 \\ -1229.41 \\ -913.03 \\ -913.03 \end{pmatrix} \text{lbf}$$

$$\min(R) = -1229.41 \text{ lbf}$$

MWFRS loads: interior zone on single truss (4 values are max uplift from corner 1 Case A & B, corner 2, Case A&B)

$$R_{MWF} = \begin{pmatrix} -454.19 \\ -671.35 \\ -944.11 \\ -944.11 \end{pmatrix} \text{lbf}$$

$$\min(R_{MWF}) = -944.11 \text{ lbf}$$

Corner Truss Design

$$R_{MWFc} = \begin{pmatrix} -637.39 \\ -1045.16 \end{pmatrix} \text{lbf}$$

$$\min(R_{MWFc}) = -1045.16 \text{ lbf}$$

Behavior of whole roof is governed by sum of all strap resistances - i.e. overall moment of loads, and therefore the modeled value is representative of the bulk of the straps used in house. Therefore base "design" for HURLOSS on interior truss loads from C&C loads (worst case loads).

$$R_{design} := \min(R)$$

$$R_{design} = -1229.41 \text{ lbf}$$

Shear on Roof-Wall Connectors

Lateral shear loads on connectors are assumed to be adequate.

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, 2 options considered

1. Wall Sheathing Attachment - Suction Loads for Zone 5 C&C loads

Loads:

$Area := 32 \cdot ft^2$ Cladding loads $A_{eff} := 10 \cdot ft^2$ Effective Area for one fastener

$p_{wall} := q_h \cdot \left(GC_p\left(A_{eff}, 5\right) + GC_{pi}\right)$ $p_{wall_0} = -40.71\,psf$

$L_{total} := \left(-p_{wall}\right)_0 \cdot Area$ $L_{total} = 1302.62\,lbf$ suction

Resistance of nails in panel:

$q_r := 41 \cdot \frac{lbf}{in}$ 8d common nail in Southern Pine (SG = 0.55)

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

$l_p := l_{nail} - t_{OSB}$ $l_p = 2.06\,in$ 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

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

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

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

$R_{total} := Nnails_{wall} \cdot R_{nail}$ $R_{total} = \begin{pmatrix} 10688.7 \\ 8659.2 \end{pmatrix} lbf$

$$Status_{WallSuction} := \begin{vmatrix} PASS & \text{if } \left(\min\left(R_{total}\right) > L_{total}\right) \\ FAIL & \text{otherwise} \end{vmatrix}$$

$Status_{WallSuction} = 1$

Resistance of Wall (Wood): Consider three stud sizes - 2x4, 2x6 and 2x8's

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

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

$isize := 0..2$

$$Stud_{area} := \overrightarrow{(Stud_w \cdot Stud_d)} \qquad Stud_{area} = \begin{pmatrix} 5.25 \\ 8.25 \\ 10.88 \end{pmatrix} in^2$$

Section modulus: NDS-S97

$$S_{xx} := \begin{pmatrix} 3.063 \\ 7.563 \\ 13.14 \end{pmatrix} \cdot in^3 \qquad S_{yy} := \begin{pmatrix} 1.313 \\ 2.063 \\ 2.719 \end{pmatrix} \cdot in^3$$

Moment of Inertia

$$I_{xx} := \begin{pmatrix} 5.359 \\ 20.80 \\ 47.63 \end{pmatrix} \cdot in^4 \qquad I_{yy} := \begin{pmatrix} 0.984 \\ 1.547 \\ 2.039 \end{pmatrix} \cdot in^4$$

Design Values from Table 4B, 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, allowable

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

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

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

Species and Grade:
Spruce Pine Fir No 2.

2. Wall Bending & Axial Loads

$sp := 0..1$ spacing of studs option variable

Wind Load:

$$A_{eff_{sp}} := \left(\begin{pmatrix} h_{wall} \cdot \Delta stud_{sp} \\ \dfrac{h_{wall}^2}{3} \end{pmatrix}\right)$$

For Stud Spacing: $\Delta stud_0 = 12\,in$ $\quad A_{eff_0} = \begin{pmatrix} 8 \\ 21.33 \end{pmatrix} ft^2$ $\quad A_{eff_0} := \max\left(A_{eff_0}\right)$ $\quad A_{eff_0} = 21.33\,ft^2$

For Stud Spacing: $\Delta stud_1 = 16\,in$ $\quad A_{eff_1} = \begin{pmatrix} 10.67 \\ 21.33 \end{pmatrix} ft^2$ $\quad A_{eff_1} := \max\left(A_{eff_1}\right)$ $\quad A_{eff_1} = 21.33\,ft^2$

The one third span run tends to govern for all stud spacings, therefore limit effective area to just one area.

$$A_{eff} := \max\left(A_{eff}\right) \qquad A_{eff} = 21.33\,ft^2$$

Zone 5

$$p_{wall} := q_h \cdot \left(GC_p\left(A_{eff}, 5\right) + GC_{pi}\right) \qquad GC_p(Area, 5) + GC_{pi} = \begin{pmatrix} -1.4 \\ 1.09 \end{pmatrix} \qquad p_{wall} = \begin{pmatrix} -37.71 \\ 28.9 \end{pmatrix} psf$$

$$\omega_{sp} := p_{wall_0} \cdot \Delta stud_{sp} \qquad \omega = \begin{pmatrix} -37.71 \\ -50.28 \end{pmatrix} \frac{1}{ft}\,lbf \qquad M_{sp} := \frac{\omega_{sp} \cdot h_{wall}^2}{8} \qquad M = \begin{pmatrix} -301.7 \\ -402.27 \end{pmatrix} ft\,lbf$$

Axial Load:

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

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

assume all load carried by long walls

Lumber Property Adjustments

$C_{Dwind} := 1.6$ $\quad C_L := 1.0$ Continuous Lateral Bracing (from sheathing)

$C_{Dgravity} := 1.25$

$$C_r := \begin{pmatrix} 1.5 \\ 1.4 \\ 1.3 \end{pmatrix}$$

Repetitive Loading Factor, from Table 2313.3 FBC pg 23.23 assuming 3/8 sheathing with gypsum board, and 8d nails at 6"/12" spacing

$$C_F := \begin{pmatrix} 1.15 & 1.1 & 1.05 \\ 1.5 & 1.3 & 1.2 \\ 1.5 & 1.3 & 1.2 \end{pmatrix}$$

for compression — Size adjustments for anything but Southern Pine

for tension

for bending — Size Factor, No.1 and Better Grade (Table 4A of NDS 97 Supplement, page 25)

2 x4, x6, x8

Calculate Adjusted Bending Capacity

$$Fb\_a_{isize} := F_b \cdot C_{Dwind} \cdot C_L \cdot \left(C_F^{\langle isize \rangle}\right)_2 \cdot Cr_{isize} \qquad Fb\_a = \begin{pmatrix} 3150 \\ 2548 \\ 2184 \end{pmatrix} psi \qquad \left(C_F^{\langle 0 \rangle}\right)_2 = 1.5$$

Calculate adjusted compressive Capacity

$$Fc\_star_{isize} := F_c \cdot C_{Dwind} \cdot \left(C_F^{\langle isize \rangle}\right)_0 \qquad Fc\_star = \begin{pmatrix} 2116 \\ 2024 \\ 1932 \end{pmatrix} psi$$

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)

$$F_{cE} := \frac{K_{cE} \cdot E}{\left[\left(\frac{K_l \cdot h_{wall}}{Stud_d}\right)^2\right]} \qquad F_{cE} = \begin{pmatrix} 558.27 \\ 1378.58 \\ 2395.43 \end{pmatrix} psi$$

Euler buckling pressure

$$Cp\_col := \overrightarrow{\left[\frac{1 + \frac{F_{cE}}{Fc\_star}}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{cE}}{Fc\_star}}{2 \cdot c}\right)^2 - \frac{\frac{F_{cE}}{Fc\_star}}{c}}\right]} \qquad Cp\_col = \begin{pmatrix} 0.25 \\ 0.55 \\ 0.76 \end{pmatrix}$$

Column stability factor

$$Fc\_a_{isize} := F_c \cdot C_{Dwind} \cdot \left(C_F^{\langle isize \rangle}\right)_0 \cdot Cp\_col_{isize} \qquad Fc\_a = \begin{pmatrix} 523.8 \\ 1109.42 \\ 1467.66 \end{pmatrix} psi$$

Combined Bending and Axial Compression Capacity 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)

For Stud Spacing of: $sp := 1$ $\Delta stud_{sp} = 16\,in$

Bending stress for: $f_b := \frac{(-M_{sp})}{S_{xx}}$ $f_b = \begin{pmatrix} 1575.99 \\ 638.27 \\ 367.37 \end{pmatrix} psi$

compressive stress $f_c := \frac{Load_{stud_{sp}}}{Stud_{area}}$ $f_c = \begin{pmatrix} 52.57 \\ 33.45 \\ 25.38 \end{pmatrix} psi$

Allowable values: $F_{c\_a} = \begin{pmatrix} 523.8 \\ 1109.42 \\ 1467.66 \end{pmatrix} psi$ $F_{b\_a} = \begin{pmatrix} 3150 \\ 2548 \\ 2184 \end{pmatrix} psi$

Interaction Equation:

$$axial_{isize} := \left(\frac{f_{c_{isize}}}{F_{c\_a_{isize}}}\right)^2 \qquad bend_{isize} := \frac{f_{b_{isize}}}{F_{b\_a_{isize}} \cdot \left(1 - \frac{f_{c_{isize}}}{F_{cE_{isize}}}\right)} \qquad \left(1 - \frac{f_{c_{isize}}}{F_{cE_{isize}}}\right) = \begin{pmatrix} 0.91 \\ 0.98 \\ 0.99 \end{pmatrix}$$

$$axial = \begin{pmatrix} 0.01 \\ 0.001 \\ 0 \end{pmatrix} \qquad bend = \begin{pmatrix} 0.55 \\ 0.26 \\ 0.17 \end{pmatrix}$$

$$CSIequation_{isize} := axial_{isize} + bend_{isize} \qquad CSIequation = \begin{pmatrix} 0.56 \\ 0.26 \\ 0.17 \end{pmatrix}$$

$Status_{Wood\_Bending2x4} :=$ | 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 $\qquad Status_{Wood\_Bending2x8} = 1$

$Spacing2x4 := if\left(Status_{Wood\_Bending2x4} = PASS, \Delta stud_{sp}, 0\right)$

$sp := 0$ Repeat Bending Calculations for spacing of $\Delta_{stud_{sp}} = 12\,in$

Bending stress for: $f_b := \frac{(-M_{sp})}{S_{xx}}$ $f_b = \begin{pmatrix} 1181.99 \\ 478.7 \\ 275.53 \end{pmatrix} psi$

compressive stress $f_c := \frac{Load_{stud_{sp}}}{Stud_{area}}$ $f_c = \begin{pmatrix} 39.43 \\ 25.09 \\ 19.03 \end{pmatrix} psi$

Interaction Equation:

$$axial_{isize} := \left(\frac{f_{c_{isize}}}{F_{c\_a_{isize}}}\right)^2 \qquad bend_{isize} := \frac{f_{b_{isize}}}{F_{b\_a_{isize}} \cdot \left(1 - \frac{f_{c_{isize}}}{F_{cE_{isize}}}\right)} \qquad \left(1 - \frac{f_{c_{isize}}}{F_{cE_{isize}}}\right) = \begin{array}{|c|} \hline 0.93 \\ \hline 0.98 \\ \hline 0.99 \\ \hline \end{array}$$

$$axial = \begin{pmatrix} 0.006 \\ 0.001 \\ 0 \end{pmatrix} \qquad bend = \begin{pmatrix} 0.4 \\ 0.19 \\ 0.13 \end{pmatrix}$$

$$CSIequation_{isize} := axial_{isize} + bend_{isize} \qquad CSIequation = \begin{pmatrix} 0.41 \\ 0.19 \\ 0.13 \end{pmatrix}$$

$$Status_{Wood\_Bending2x4} := \begin{cases} PASS & \text{if } (CSIequation_0) \le 1.0 \\ FAIL & \text{otherwise} \end{cases} \qquad Status_{Wood\_Bending2x4} = 1$$

$$Status_{Wood\_Bending2x6} := \begin{cases} PASS & \text{if } (CSIequation_1) \le 1.0 \\ FAIL & \text{otherwise} \end{cases} \qquad Status_{Wood\_Bending2x6} = 1$$

$$Status_{Wood\_Bending2x8} := \begin{cases} PASS & \text{if } (CSIequation_2) \le 1.0 \\ FAIL & \text{otherwise} \end{cases} \qquad Status_{Wood\_Bending2x8} = 1$$

Check if spacing of 2x4's needs to be decreased

$$Spacing2x4 := if\left(Spacing2x4 = 0, if\left(Status_{Wood\_Bending2x4} = PASS, \Delta_{stud_{sp}}, 0\right), Spacing2x4\right)$$

$Spacing2x4 = 16\,in$

3. Calculate adjusted axial load only case

$$F_{c\_star_{isize}} := F_c \cdot C_{Dgravity} \cdot \left(C_F^{\langle isize \rangle}\right)_0 \qquad F_{c\_star} = \begin{pmatrix} 1653.13 \\ 1581.25 \\ 1509.38 \end{pmatrix} psi$$

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)

$$F_{cE} := \frac{K_{cE} \cdot E}{\left[\left(\frac{K_l \cdot h_{wall}}{Stud_d}\right)^2\right]} \qquad F_{cE} = \begin{pmatrix} 558.27 \\ 1378.58 \\ 2395.43 \end{pmatrix} psi$$

Euler buckling pressure

$$C_{p\_col} := \overrightarrow{\left[\frac{1 + \frac{F_{cE}}{F_{c\_star}}}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{cE}}{F_{c\_star}}}{2 \cdot c}\right)^2 - \frac{\frac{F_{cE}}{F_{c\_star}}}{c}}\right]} \qquad C_{p\_col} = \begin{pmatrix} 0.31 \\ 0.64 \\ 0.82 \end{pmatrix}$$

Column stability factor

$$F_{c\_a_{isize}} := \left[F_c \cdot C_{Dgravity} \cdot \left(C_F^{\langle isize \rangle}\right)_0 \cdot C_{p\_col_{isize}}\right] \qquad F_{c\_a} = \begin{pmatrix} 512.27 \\ 1014.87 \\ 1241.94 \end{pmatrix} psi$$

$$CSIequation := \overrightarrow{\frac{f_c}{F_{c\_a}}} \qquad CSIequation = \begin{pmatrix} 0.08 \\ 0.02 \\ 0.02 \end{pmatrix}$$

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

6. Bearing Capacity of Top Plate

Not a capacity limit state. OK by inspection

Lateral Shear Design of Wood Walls

1. Wind Loads

Normal to ridge for roof slope higher than 10 degrees: $\frac{L}{W} = 1.13$ $\frac{h}{L} = 0.29$

Look up values from Figure 6-3 (ASCE 7-98)

$C_{p\_wall\_windward} := 0.8$ $C_{p\_wall\_leeward} := -0.5$

Wind Normal to ridge $C_{p\_roof\_windward} := \begin{pmatrix} 0.3 \\ -0.2 \end{pmatrix}$ $C_{p\_roof\_leeward} := -0.6$

Wind Parallel to ridge $C_{p\_roof\_windward\_ll} := \begin{pmatrix} 0.3 \\ -0.9 \end{pmatrix}$ assume windward hip acts similar to wind normal to ridge case

$G = 0.87$ $C_{p\_roof\_leeward\_ll} := -0.3$ from normal to ridge section of Fig 6-3

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

Wind Perpendicular to Ridge:

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

MWFRS Roof Pressure $MWFRS_{roof\Gamma} := q_h \cdot \left[\left(G \cdot C_{p\_roof\_windward_0}\right) - \left(G \cdot C_{p\_roof\_leeward}\right)\right]$

MWFRSWall Wall Pressure $MWFRS_{wall\Gamma} := q_h \cdot \left(\left(G \cdot C_{p\_wall\_windward} - G \cdot C_{p\_wall\_leeward}\right)\right)$

$MWFRS_{roof\Gamma} = 20.12\,psf$ $MWFRS_{wall\Gamma} = 29.07\,psf$ $q_h = 25.76\,psf$

Total Shear from Roof $VPA_{\Gamma} \cdot MWFRS_{roof\Gamma} = 6151.46\,lbf$

Total Shear from Wall $VPA_{wall\Gamma} \cdot MWFRS_{wall\Gamma} = 6045.54\,lbf$

Total shear $Shear_{\Gamma} := VPA_{wall\Gamma} \cdot MWFRS_{wall\Gamma} + VPA_{\Gamma} \cdot MWFRS_{roof\Gamma}$ $Shear_{\Gamma} = 12197\,lbf$

Wind Parallel to Ridge:

MWFRS Roof Pressure $q_h \cdot \left[\left(G \cdot C_{p\_roof\_windward\_ll_0}\right) - \left(G \cdot C_{p\_roof\_leeward\_ll}\right)\right] = 13.41\,psf$

MWFRSWall Wall Pressure $q_h \cdot \left(\left(G \cdot C_{p\_wall\_windward} - G \cdot C_{p\_wall\_leeward}\right)\right) = 29.07\,psf$

Total Shear from Roof $VPA_{ll} \cdot q_h \cdot \left[\left(G \cdot C_{p\_roof\_windward\_ll_0}\right) - \left(G \cdot C_{p\_roof\_leeward\_ll}\right)\right] = 3252.49\,lbf$

Total Shear from Wall $VPA_{wall\_ll} \cdot q_h \cdot \left(G \cdot C_{p\_wall\_windward} - G \cdot C_{p\_wall\_leeward}\right) = 5347.97\,lbf$

Total shear

$$Shear_{ll} := q_h \cdot \begin{bmatrix} VPA_{ll} \cdot \left[\left(G \cdot C_{p\_roof\_windward\_ll_0}\right) - \left(G \cdot C_{p\_roof\_leeward\_ll}\right)\right] \cdots \\ + VPA_{wall\_ll} \cdot \left[\left(G \cdot C_{p\_wall\_windward}\right) - \left(G \cdot C_{p\_wall\_leeward}\right)\right] \end{bmatrix}$$

$Shear_{ll} = 8600.5\,lbf$

3. 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

$Shear_{allowable} := 310 \cdot \frac{lbf}{ft}$

Table 4.1A of Structural Use Panel Shear Wall and Diaphragm Supplement to NDS 1997
3/8" sheathing with 8d nails 6" at edges

$L_{shearMin\_\Gamma} := \frac{Shear_{\Gamma}}{Shear_{allowable}}$ $L_{shearMin\_\Gamma} = 39.35\,ft$

$L_{shearMin\_ll} := \frac{Shear_{ll}}{Shear_{allowable}}$ $L_{shearMin\_ll} = 27.74\,ft$

Actual length available for shear walls:

$L_{shearwall\_Actual\_\Gamma} := (30\ \ 24\ \ 18\ \ 20\ \ 8)^T \cdot ft$ $\sum L_{shearwall\_Actual\_\Gamma} = 100\,ft$

$L_{shearwall\_Actual\_ll} := (4\ \ 4\ \ 10\ \ 4\ \ 24\ \ 10\ \ 4\ \ 4\ \ 4\ \ 4)^T \cdot ft$ $\sum L_{shearwall\_Actual\_ll} = 72\,ft$

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

$Status_{Wood\_Shear} = 1$

3. Shear Wall "Chord" Force and hold down requirements

Distribute shear by ratio of wall length to total shear wall length

$$Shear_{\Gamma wall} := Shear_{\Gamma} \cdot \left(\frac{L_{shearwall\_Actual\_\Gamma}}{\sum L_{shearwall\_Actual\_\Gamma}} \right) \qquad Shear_{\Gamma wall} = \begin{pmatrix} 3659.1 \\ 2927.28 \\ 2195.46 \\ 2439.4 \\ 975.76 \end{pmatrix} lbf$$

$k := 0 .. length(L_{shearwall\_Actual\_\Gamma})$

$$T := \overrightarrow{\frac{Shear_{\Gamma wall} \cdot h_{wall}}{L_{shearwall\_Actual\_\Gamma}}} \qquad T = \begin{pmatrix} 975.76 \\ 975.76 \\ 975.76 \\ 975.76 \\ 975.76 \end{pmatrix} lbf$$

5. Shear of Anchor Bolts

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

$Z := 890 \cdot lbf$ 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

$Z\_a := Z \cdot C_{Dwind} \cdot C_m \cdot C_t \cdot C_g \cdot C_{others}$ $Z\_a = 1424\,lbf$ Shear capacity per bolt

$Shear_{\Gamma} = 12196.99\,lbf$ shear to resist total ...

$N_{bolts} := \frac{Shear_{\Gamma}}{Z\_a}$ $N_{bolts} = 8.57$

$\Delta_{bolt} := floor\left(\frac{2 \cdot W}{N_{bolts}}\right)$ $\Delta_{bolt} = 10\,ft$ Use one bolt every $\Delta_{bolt} = 10\,ft$

WALL DESIGN for Masonry Walls (ACI 530-99)

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 worr 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

$A_{yy} := d_{shell} \cdot h_{CMU} \cdot 2$ $\qquad A_{yy} = 19.06 \, in^2$

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

$$S_{yy} := \frac{h_{CMU} \cdot \left[d_{CMU}^{\;3} - \left(d_{CMU} - 2 \cdot d_{shell} \right)^3 \right]}{6 \cdot d_{CMU}} \qquad S_{yy} = 51.45 \, in^3$$

Limiting moment in wall

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

Wind Load:

$A_{eff} := 8 \cdot 6 \cdot ft^2$ $\qquad$ Zone 5

$p_{wall} := q_h \cdot \left(GC_p(A_{eff}, 5) + GC_{pi} \right)$ $\qquad GC_p(A_{eff}, 5) + GC_{pi} = \begin{pmatrix} -1.34 \\ 1.06 \end{pmatrix}$ $\qquad p_{wall} = \begin{pmatrix} -34.51 \\ 27.3 \end{pmatrix} \text{psf}$

$\omega := p_{wall_0} \cdot h_{CMU}$ $\qquad \omega = -21.93 \frac{1}{ft}\ \text{lbf}$

Maximum spacing of reinforcement

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

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