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Date: 2022.11.15 15:37:20 -06'00'

JOB: TS Accessory Building - TBS #610-1872111 NO. 29632

CALC. BY SC DATE 11/15/2022

Structural Calculations for:
Lewis Cox
Tuff Shed Project No. 610-1872111

Project Location:	Bunnlevel, NC
Building Code:	2018 IBC

Building Dimensions: Overall Building Width: $B := 18ft$

Overall Building Length: $L := 24ft$ (nb. ridge runs parallel to Length)

Plate Heights: Finished Floor Elevation: $h_{floor_1} := 6in$

Top of Plate Elevation: $h_{floor_2} := 7ft + 8.25in$ $h_{plate_1} := h_{floor_2}$

$h_{floor_3} := 14ft + 3.875in$ $h_{plate_2} := h_{floor_3} - h_{floor_2}$

Roof Pitches: $pitch := \frac{5}{12}$ Roof Angle: $\Theta := atan(pitch)$ $\Theta = 22.62 \cdot deg$

Peak Roof Height: $h_{peak} := 16ft + 10.63in$

Mean Building Height: $h_{mean} := \left(h_{floor_3} + h_{peak}\right) \cdot 0.5$ $h_{mean} = 15.6\,ft$

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11/15/2022

Consider Vertical Loads:

Dead Loads: Roof Dead Load: $DL_{roof} := 10psf$

Loft Dead Load: $DL_{loft} := 10psf$

Wall Dead Load: $DL_{wall} := 8psf$

Live Loads: Roof Live Load: $LL_{roof} = 30 \cdot psf$

Loft Live Load: $LL_{loft} := 30psf$

Snow Loads: Roof Snow Load: $SL_{roof} := 20psf$

Roof Load Summary	
Roof Support:	Trusses @ 24 in. c/c
Truss Span (ft):	18
Roof Dead Load (psf):	10
Roof Live Load (psf):	30
Roof Snow Load (psf):	20.00
Governing Total Load (psf):	40
See Calculations to Follow	

Roof End Reactions:

Dead Load: $R_{roof\_DL} := \frac{B}{2} \cdot \left(DL_{roof} + DL_{loft}\right) \cdot rt_{spacing}$

$R_{roof\_DL} = 360\,lb$

Live Load: $R_{roof\_LL} := 0.75 \cdot \frac{B}{2} \cdot \left[max\left(\begin{pmatrix} SL_{roof} \\ LL_{roof} \end{pmatrix}\right) + LL_{loft} \right] \cdot rt_{spacing}$

$R_{roof\_LL} = 810\,lb$

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3&4 Foot Opening Headers

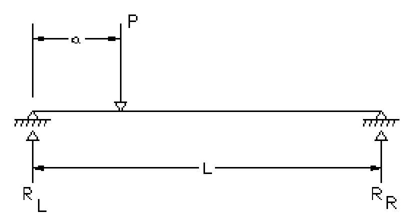
Beam Design Data					
Member:	2x4	Beam Span (ft):			4
# of Memb:	2	Unbraced Length (ft):			4
Adjustment Factors - NDS Table 2.3.1					
C_D	C_M	C_t	C_L	C_i	C_r
1.15	1.00	1.00	See Below	1.00	1.00

Effective Length Factor: $k \equiv 1.84$
(Ref NDS Table 3.3.3)

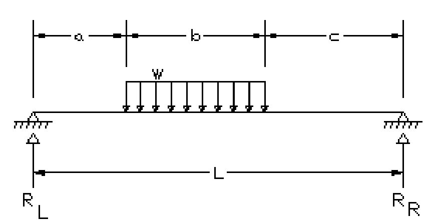
$L_e := k \cdot L_u$ $L_e = 7.36\,ft$

Selected Member Properties									
Species	Commercial Grade	Member Size	# of Mem.	b (in)	d (in)	Area (in^2)	S_x (in^3)	I_x (in^4)	E (ksi)
Spruce-Pine-Fir	No. 2	2x4	2	1.5	3.5	10.50	6.13	10.72	1400

Concentrated Load Data				
Number	DL (kips)	LL (kips)	a dist. (ft)	Description
P1	0.00	0.00	0.00	
P2	0.00	0.00	0.00	
P3	0.00	0.00	0.00	
P4	0.00	0.00	0.00	
P5	0.00	0.00	0.00	
P6	0.00	0.00	0.00	



Uniform Load Data						
Number	DL (kip/ft)	LL (kip/ft)	a dist. (ft)	c dist. (ft)	trib. wt. (ft)	Description
w1	0.02	0.06	0.00	0.00	2.00	Roof Load
w2	0.02	0.06	0.00	0.00	2.00	
w3	0.00	0.00	0.00	0.00	0.00	
w4	0.00	0.00	0.00	0.00	0.00	
w5	0.00	0.00	0.00	0.00	0.00	
w6	0.00	0.00	0.00	0.00	0.00	



Calculated Adjustment Factor	C_L =	0.9971

Reactions			
	DL (kips)	LL (kips)	TL (kips)
Left End	0.09	0.24	0.33
Right End	0.09	0.24	0.33

Shears, Moments, & Deflections		
Maximum Shear Stress (psi)	39	
Allowable Shear Stress (psi)	173	**OK**
Maximum Bending Stress (psi)	532	
Allowable Bending Stress (psi)	975	**OK**
Max. Dead Load Deflection (in)	0.018	2728
Max. Live Load Deflection (in)	0.046	1042
Max. Total Load Deflection (in)	0.064	754
USE	**2 - 2x4**	

Lateral Loads:

Project Location:	Bunnlevel, NC
Building Code:	2018 IBC

Determine Wind Loads (ASCE 7-16):

Risk Category II (Table 1.5-1)

Basic Wind Speed (3 second gust): $V_{wind} := 110mph$ (Figure 26.5-1A, B or C)

Exposure Category: $Exposure := "C"$

Height and Exposure Coefficient: $\lambda = 1.29$

Basic Wind Pressures: nb. intermediate roof angles are interpolated

Main Windforce Resisting System Loads: p_{s30} (ASCE 7-16 Figure 28.5-1)					
Horizontal Loads (psf)			**Vertical Loads (psf)**		
Interior Zone:	Wall	Roof	**Interior Zone:**	Windward Roof	Leeward Roof
Transverse	17.6	0.2	Transverse	-9.9	-12.0
Longitudinal	12.7	-5.9	Longitudinal	-16.0	-10.1

Calculated Wind Pressures: 0psf minimum roof lateral load - Figure 28.5-1 footnote 7.

Calculated Wind Loads					
Horizontal Loads (psf)			**Vertical Loads (psf)**		
Interior Zone:	Wall	Roof	**Interior Zone:**	Windward Roof	Leeward Roof
Transverse	22.7	0.3	Transverse	-12.8	-15.4
Longitudinal	16.4	0.0	Longitudinal	-20.6	-13.0

Wind Loads Transverse Direction: $B = 18\,ft$ $L = 24\,ft$ $h_{peak} = 16.89\,ft$

$Trib\_area_{roof\_t} := h_{peak} - h_{floor_3}$ $Trib\_area_{roof\_t} = 2.56\,ft$

$Trib\_area_{roof\_l} := 0.5 \cdot \left(h_{peak} - h_{floor_3}\right)$ $Trib\_area_{roof\_l} = 1.28\,ft$

$Trib\_area_{wall} := h_{floor_3} - 0.5 \cdot h_{floor_2}$ $Trib\_area_{wall} = 10.48\,ft$

Transverse wind forces: Load Combination Factor: $LC_W := 0.6$

Distributed forces at roof: $P_{whtr} = 0.28 \cdot psf$ $P_{whtw} = 22.66 \cdot psf$

$$Fd_{wind\_t} := LC_W \cdot max\left(P_{whtr} \cdot Trib\_area_{roof\_t} + P_{whtw} \cdot Trib\_area_{wall}, 8psf\,Trib\_area_{roof\_t} + 16psf\,Trib\_area_{wall}\right)$$

Force at roof: $F_{wind\_t} := Fd_{wind\_t} \cdot L$ $F_{wind\_t} = 3430\,lb$ $Fd_{wind\_t} = 142.92 \cdot plf$

Wind Loads Longitudinal Direction: $B = 18\,ft$ $L = 24\,ft$ $h_{mean} = 15.6\,ft$

Longitudinal wind forces: $P_{whlw} = 16.38 \cdot psf$

Distributed forces at roof: $Fd_{wind\_l} := LC_W \cdot max\left(P_{whlw}, 16psf\right) \cdot \left(Trib\_area_{wall} + Trib\_area_{roof\_l}\right)$ $Fd_{wind\_l} = 115.6 \cdot plf$

Force at roof: $F_{wind\_l} := Fd_{wind\_l} \cdot B$ $F_{wind\_l} = 2081\,lb$

SEISMIC LOADS: (ASCE 7-16)

C
R
n
n

Occupancy Category: $Occupancy := "II"$

Redundancy Factor: $\rho := 1.0$

Max Considered Earthquake Coefficients: $S_s := 0.137$ $S_1 := 0.072$

Load Combination modifer (service load) $LC_E := 0.7$

Seismic Site Class: $Site\_Class := "D"$ (As Allowed Per 11.4.3. Subject to Requirements of 11.4.4 and 11.4.8)

Seismic Importance Factor: $I_E := 1.0$

Site Coefficients: {tables 11.4(1-2)} $F_a = 1.6$ (Per 11.4.4) $F_v = 2.4$

Adjusted Max considered Earthquake: $S_{ms} = 0.22$ $S_{m1} = 0.17$

Design Spectral Response Acceleration: $S_{ds} = 0.15$ $S_{d1} = 0.12$

Approximate Fundamental Period: $T_a := .020 \cdot \left(\frac{h_{peak}}{ft}\right)^{.75}$ $T_a = 0.17$ $T_L := 8$

$T_s := (S_{d1} \div S_{ds})$ $T_s = 0.79$

Seismic Design Category: $SDC = "A"$

Structural System: $Basic\_Structural\_System := "Bearing Wall System"$

$Seismic\_Resisting\_System := "Light Framed Walls w/ Wood Shear Panels"$

Determine C_s: $C_u := 1.4$ ASCE 7 Table 12.8-1

$T := C_u \cdot T_a$ $T = 0.23$ 12.8.2

Maximum S_{DS} value for determination of C_s: $S_{ds\_max} := \mathrm{if}(0.7S_{ds} > 1, 0.7 \cdot S_{ds}, \mathrm{if}(T > 0.5, S_{ds}, 1.0))$

(12.8.1.3)

Design Spectral Response Acceleration: $S_{ds\_max} = 1$

Response Modification Factor: $R := 6.5$ {ASCE 7 table 12.2-1}

Seismic Response Coefficient:

{12.8.1.1 adjusted to meet the requirements of 11.4.8}

$$C_{s\_min} := \mathrm{if}\left(S_1 < 0.6, \max\left(0.01, \frac{S_{ds\_max}}{R \div I_E}\right), \max\left(0.01, \frac{S_{ds\_max}}{R \div I_E}, \frac{0.5 \cdot S_1}{R \div I_E}\right)\right)$$

$C_{s\_min} = 0.15$

$$C_{s\_max} := 1.5 \cdot \mathrm{if}\left[1.5 \cdot T_s > T \le T_L, \left[\frac{S_{d1}}{T \cdot (R \div I_E)}\right], \frac{S_{d1} \cdot T_L}{T^2 \cdot (R \div I_E)}\right]$$

$C_{s\_max} = 0.11$

$C_s := \mathrm{if}(T < 1.5T_s, C_{s\_min}, C_{s\_max})$ $C_s = 0.15$

Determine Structural Dead Load: Ws

Area of Roof & Floor: $Area := L \cdot B$ $\boxed{Area = 432\,ft^2}$

Total Roof Dead Load: $W_{roof} := Area \cdot DL_{roof} + DL_{loft} \cdot 128ft^2$ $\boxed{W_{roof} = 5.6 \cdot kip}$

$$W_{roof\_shear} := if\left[\frac{SL_{roof}}{psf} > 30, Area \cdot \left(DL_{roof} + \frac{SL_{roof}}{5}\right), W_{roof}\right]$$

Exterior Wall Dead Loads — Wall load tributary to roof:

Length of wall: $L_{wall} := L + B + L + B$ $\boxed{L_{wall} = 84\,ft}$

Wall Dead Load: $W_{wall\_shear} := DL_{wall} \cdot L_{wall} \cdot \frac{h_{mean}}{2}$ $\boxed{W_{wall\_shear} = 5.24 \cdot kip}$

$W_{wall\_ot} := DL_{wall} \cdot L_{wall} \cdot h_{mean}$ $\boxed{W_{wall\_ot} = 10.49 \cdot kip}$

Total Structural Dead Load: $W_{s\_shear} := W_{roof\_shear} + W_{wall\_shear} + 0.25 \cdot LL_{loft} \cdot 128ft^2$ $\boxed{W_{s\_shear} = 12 \cdot kip}$

$W_{s\_ot} := W_{roof} + W_{wall\_ot}$ $\boxed{W_{s\_ot} = 16.09 \cdot kip}$

Determine Total Base Shear Forces: $F_{EQ} := LC_E \cdot \rho \cdot C_s \cdot W_{s\_shear}$

$Fd_{EQ\_t} := F_{EQ} \div L$ $Fd_{EQ\_l} := F_{EQ} \div B$

Compare Seismic Forces at Roof:

Wind and Seismic Forces:

$F_{EQ} = 1.3 \cdot kip$ $F_{wind\_t} = 3.4 \cdot kip$ $F_{wind\_l} = 2.1 \cdot kip$

$Fd_{EQ\_t} = 52.96 \cdot plf$ $Fd_{wind\_t} = 142.92 \cdot plf$ $Fd_{wind\_l} = 115.6 \cdot plf$

$Fd_{EQ\_l} = 70.62 \cdot plf$

Use Maximum Forces:

$F_{roof\_t} := max(F_{EQ}, F_{wind\_t})$ $\boxed{F_{roof\_t} = 3430.15\,lb}$

$F_{roof\_l} := max(F_{EQ}, F_{wind\_l})$ $\boxed{F_{roof\_l} = 2080.88\,lb}$

$Fd_{roof\_t} := max(Fd_{EQ\_t}, Fd_{wind\_t})$ $\boxed{Fd_{roof\_t} = 142.92 \cdot plf}$

$Fd_{roof\_l} := max(Fd_{EQ\_l}, Fd_{wind\_l})$ $\boxed{Fd_{roof\_l} = 115.6 \cdot plf}$

Wind Governs Transverse Load. Wind Governs Longitudinal Load.

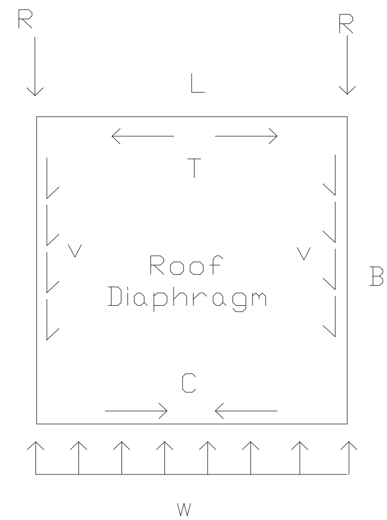
Roof Diaphragm Design:

Controlling Lateral Load: **Transverse Load Governs Diaphragm Design.**

Strut Reaction, R: $R_{roof} := Fd_{roof\_t} \cdot L \cdot \frac{1}{2}$ $\boxed{R_{roof} = 1715\ lb}$

Uniform Shear in Strut, v: $v_d := \frac{R_{roof}}{B}$ $\boxed{v_d = 95\ \frac{lb}{ft}}$

Chord Forces (T = C): $T_{chord} := \frac{Fd_{roof\_t} \cdot L^2}{8 \cdot B}$ $\boxed{T_{chord} = 572\ lb}$



Diaphragm and Fasteners Design Strength:

Roof deck is 7/16" wood structural panel with 8d common nails @6" o.c. at edges. Use 8d common nails @12" o.c. for interior nailing.

$v_{allow} := 214 \cdot \frac{lb}{ft}$

Since v_{allow} = 214.0 lb/ft >= v_d = 95 lb/ft **OK**

Consider Diaphragm Chord/Drag: chord := "Double 2x4 Hem Fir Top Plate"

Area of Chord: $A_{chord} := 5.25 \cdot in^2$

Chord Modulus of Elasticity: $E_{chord} := 1200000 \cdot psi$

Allowable Tension in Chord: $F_t := 400 \cdot psi$

Load duration factor: $C_D := 1.6$

$P_a := F_t \cdot A_{chord} \cdot C_D$ $P_a = 3360\ lb$

Fastener type: fastener := "16d nails"

Allowable shear per fastener: $v_{fastener} := 122 \cdot lb$

Number of Nails Required: $N_{fasteners} := \frac{T_{chord}}{v_{fastener} \cdot C_D}$

Use Double 2x4 Hem Fir Top Plate with 8 - 16d nails each side of splice minimum.

Consider Drag Strut: $drag_{spacing} := \frac{B}{R_{roof} \div v_{fastener}}$ $\boxed{drag_{spacing} = 15.36 \cdot in}$

Check Diaphragm Deflection:

Panel Shear Modulus: (7/16" sheathing) $Gt_{panel} := 83500\ \frac{lb}{in}$

Nail Deformation: $e_n = 0 \cdot in$

Allowable Deflection: $\Delta_{allow} := 0.025 h_{plate}$

Deflection Amplification: $\boxed{C_d := 4}$ {for WSP shear walls} $\Delta_{allow} = \begin{pmatrix} 2.31 \\ 1.99 \end{pmatrix} \cdot in$

Deflection: $\Delta_d := \frac{5 \cdot v_d \cdot L^3}{8 \cdot E_{chord} \cdot A_{chord} \cdot B} + \frac{v_d \cdot L}{4 \cdot Gt_{panel}} + \frac{0.188}{in} \cdot L \cdot e_n$ $\Delta_d = 0.09 \cdot in$ OK

Amplified Deflection: $\Delta_{d\_ampl} := \frac{C_d \cdot \Delta_d}{I_E}$ $\Delta_{d\_ampl} = 0.38 \cdot in$ OK

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Consider 2x Wood Studs - $\text{Member} := \text{"2x4"}$ Number of Members: $n := 1$

Selected Member Properties									
Species	Commercial Grade	Member Size	# of Mem.	b (in)	d (in)	Area (in^2)	S_x (in^3)	I_x (in^4)	E (ksi)
Hem-Fir	Stud	2x4	1	1.5	3.5	5.25	3.06	5.36	1200

Plate Height: $ht := h_{floor_2} - 4.5in$ Unbraced Length Factor: $k := 1.0$ Unbraced Length: $kl := k \cdot ht$ $kl = 7.31\,ft$

Compression Parallel to Grain Non-Adjusted Design Value: $F_c = 800 \cdot psi$

Grading Adjustment Factor: $K_{cE} := 0.3$ (See NDS Section 3.7.1)

Coefficient Used for Column Stability Factor: $c := 0.8$ (See NDS Section 3.7.1)

Maximum Bending Unbraced Length: $L_u := 12in$ Effective Length Factor: (Ref NDS Table 3.3.3) $k := 1.84$ $L_e := k \cdot L_u$ $L_e = 1.84\,ft$

Determine the allowable axial compressive stress per NDS Section 3.6

Adjustment Factors - NDS Table 2.3.1					
C_M	C_t	C_F	C_i	C_r	C_T
1.00	1.00	1.00	1.00	1.15	1.00

Modified Modulus of Elasticity: $E' := E \cdot C_M \cdot C_t \cdot C_i \cdot C_T$ $E' = 1200000 \cdot psi$

Stress used for column stability factor:

$$F_{cE} := \frac{K_{cE} \cdot E'}{\left(\frac{kl}{d_{dressed}(id)}\right)^2} \qquad F_{cE} = 572.72 \cdot psi$$

Column stability factor:

$$C_P := \frac{1 + \left(\frac{F_{cE}}{F_c}\right)}{2c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_c}\right)}{2c}\right]^2 - \frac{\frac{F_{cE}}{F_c}}{c}} \qquad C_P = 0.57$$

Allowable axial compression design stress w/ no lateral load: $C_D := 1.15$ $F'_{c\_No\_LL} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_P$ $F'_{c\_No\_LL} = 522 \cdot psi$

Maximum Allowable Axial Load w/ no lateral load: $P_{allow\_No\_LL} := A(id) \cdot F'_{c\_No\_LL}$ $P_{allow\_No\_LL} = 2740\,lb$

Allowable axial compression design stress with lateral load: $C_D := 1.6$ $F'_{c\_w\_LL} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_P$ $F'_{c\_w\_LL} = 726 \cdot psi$

Determine the allowable bending stress per NDS Section 3.3

Determine the slenderness ratio: $R_B := \text{if}\left(L_u > 0\cdot ft, \sqrt{\frac{L_e\cdot d_{dressed}(id)}{b_{dressed}(id)^2}}, 1\right)$ $R_B = 5.86$

Coefficient used for beam stability factor: $K_{bE} := 0.439$

Stress value used for beam stability factor: $F_{bE} := \frac{K_{bE}\cdot E'}{R_B^{\;2}}$ $F_{bE} = 15.34\cdot ksi$

Tabulated bending design value from Table 4A: $F_{b\_material} = 675\cdot psi$

Tabulated bending design value multiplied by applicable adjustment factors: $F_{bstar} := F_{b\_material}\cdot C_M\cdot C_t\cdot C_i\cdot C_D\cdot C_r$

Beam stability factor: $C_L := \text{if}\left[L_u > 0\cdot ft, \frac{1+\left(\frac{F_{bE}}{F_{bstar}}\right)}{1.9} - \sqrt{\left[\frac{1+\left(\frac{F_{bE}}{F_{bstar}}\right)}{1.9}\right]^2 - \frac{\frac{F_{bE}}{F_{bstar}}}{0.95}}, 1.0\right]$ $C_L = 1$

Allowable bending stress: $F_{b\_allow} := F_{b\_material}\cdot C_L\cdot C_M\cdot C_i\cdot C_D\cdot C_t\cdot C_r$ $F_{b\_allow} = 1237\cdot psi$

Lateral pressure: $P_{whtw} = 22.66\cdot psf$ Tributary width of member: $t_{width} := 16in$

Maximum moment due to lateral pressure: $M_{max} := 0.75\cdot(0.6\cdot P_{whtw})\cdot t_{width}\cdot ht^2\cdot 0.125$ $M_{max} = 91\,ft\cdot lb$ (ASCE LC ASD 6a)

Maximum bending stress: $f_b := \frac{M_{max}}{S_x(id)}$ $f_b = 356\cdot psi$

Find the maximum compressive stress to satisfy the bending and axial compression interaction equation. (NDS equation 3.9-3)

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_b}{F_{b\_allow}\cdot\left(1-\frac{f_c}{F_{cE}}\right)} \le 1.0$$ (NDS equation 3.9-3)

Solving for the compressive stress results in: $f_c = 356\cdot psi$

Maximum allowable axial load w/ lateral load: $P_{allow\_w\_LL} := A(id)\cdot f_c$ $P_{allow\_w\_LL} = 1867\,lb$

Controlling allowable axial load: $P_{allow} = 1867\,lb$ Axial load w/ lateral load governs

Required axial load capacity: $P_{required} := \frac{R_{roof\_DL} + 0.75R_{roof\_LL}}{rt_{spacing} \div t_{width}}$

$P_{required} = 645\,lb$

Stud deflection @ midheight: $\Delta_{lateral} := \frac{5P_{whtw}\cdot t_{width}\cdot ht^4}{384E\cdot I_x(id)}$ $\Delta_{lateral} = 0.3\cdot in$ $\frac{ht}{\Delta_{lateral}} = 290$

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SHEET NO. 9 OF 12
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Shear Wall Design:

Consider Wall A:

{Note: 2-7'-0" panels each resist 25% of lateral force = 50% total}

Percentage of lateral force taken by wall A: $Percent := 25\%$

Total Wall Length: $l_{wall} := 7ft$

Length of Shear Wall: $l_{shearwall} := 7ft$

Seismic Force: $F_{EQ} = 1271.1\ lb$

Tranverse Wind Force: $F_{wind\_t} = 3430.15\ lb$

Fastener type (penny weight): $nail\_size := 8$

Sheathing grade: $sheathing :=$ "Smart Panel"

Fastener spacing at edge: $edge\_spacing := 6 \cdot in$

Sheathing thickness: $thickness := \frac{3}{8} \cdot in$

Code provisions:

{18% reduction if hem-fir studs are used}

{Reduction for panels applied over gypsum}

{40% allowable increase for wind design permitted}

{Aspect ratio reduction for seismic}

Adjustment for perforated shearwall: $max\_height\_opening := 0 \cdot ft$

Minimum panel: $l_{panel} := 7ft$

Perfotated multiplier: $C_o = 1$ SDPWS Table 4.3.3.5

Shear Stress:

$$v_{seismic} := \frac{F_{EQ} \cdot Percent}{l_{shearwall}} \qquad v_{seismic} = 45 \cdot plf \qquad v_{wind} := \frac{F_{wind\_t} \cdot Percent}{l_{shearwall}} \qquad v_{wind} = 123 \cdot plf$$

For 0.375 in. Smart Panel applied directly to framing w/ 8d nails @ 6 in. edge spacing.

Seismic Design: Since $v_{seismic}$ = 45 lb/ft <= v_{allow} = 148 lb/ft **OK**

Wind Design: Since v_{wind} = 123 lb/ft <= v_{allow} = 148 lb/ft **OK**

Check Uplift:

Roof Tributary Area: $A_t := 2.5ft$

Overturning Moment:

$$M_{ot\_EQ} := F_{EQ} \cdot Percent \cdot h_{floor_2} = 2.44 \cdot kip \cdot ft \qquad M_{ot\_wind} := F_{wind\_t} \cdot Percent \cdot h_{floor_2} = 6.59 \cdot kip \cdot ft$$

Resisting Moment:

$$M_{res\_EQ} := \left(0.6 - 0.14 \cdot S_{ds}\right) \cdot \left(DL_{roof} \cdot \frac{l_{wall}^{\,2}}{2} \cdot A_t + DL_{wall} \cdot l_{shearwall} \cdot h_{floor_2} \cdot \frac{l_{wall}}{2}\right) \qquad M_{res\_EQ} = 1.23 \cdot kip \cdot ft$$

$$M_{res\_wind} := 0.6 \cdot \left(DL_{roof} \cdot \frac{l_{wall}^{\,2}}{2} \cdot A_t + DL_{wall} \cdot l_{shearwall} \cdot h_{floor_2} \cdot \frac{l_{wall}}{2}\right) \qquad M_{res\_wind} = 1.27 \cdot kip \cdot ft$$

Chord Forces:

$$tension := max\left(v_{seismic} \cdot h_{plate}, v_{wind} \cdot h_{plate}\right) \qquad tension = 942\ lb$$

OK

Uplift Anchorage at shear wall ends:

$$uplift := max\left(\frac{M_{ot\_EQ} - M_{res\_EQ}}{l_{wall}}, \frac{M_{ot\_wind} - M_{res\_wind}}{l_{wall}}\right) \qquad uplift = 760\ lb$$

Provide MR88 Earth Anchors

Shear Wall Design: Consider Wall C:

Percentage of lateral force taken by wall C: $\text{Percent} := 50\%$

Total Wall Length: $l_{wall} := 18ft$ Length of Shear Wall: $l_{shearwall} := 18ft$

Seismic Force: $F_{EQ} = 1271.1\ lb$ Tranverse Wind Force: $F_{wind\_t} = 3430.15\ lb$

Fastener type (penny weight): $nail\_size := 8$ Sheathing grade: $sheathing :=$ "Smart Panel"

Fastener spacing at edge: $edge\_spacing := 4 \cdot in$ Sheathing thickness: $thickness := \frac{3}{8} \cdot in$

Code provisions:

{18% reduction if hem-fir studs are used} {Reduction for panels applied over gypsum}

{40% allowable increase for wind design permitted} {Aspect ratio reduction for seismic}

Adjustment for perforated shearwall: $max\_height\_opening := 0 \cdot ft$ Minimum panel: $l_{panel} := 18ft$

Perforated multiplier: $C_o = 1$ SDPWS Table 4.3.3.5

Shear Stress: $v_{seismic} := \frac{F_{EQ} \cdot Percent}{l_{shearwall}}$ $v_{seismic} = 35 \cdot plf$ $v_{wind} := \frac{F_{wind\_t} \cdot Percent}{l_{shearwall}}$ $v_{wind} = 95 \cdot plf$

For 0.375 in. Smart Panel applied directly to framing w/ 8d nails @ 4 in. edge spacing.

Seismic Design: Since $v_{seismic} =$ 35 lb/ft <= $v_{allow} =$ 221 lb/ft **OK**

Wind Design: Since $v_{wind} =$ 95 lb/ft <= $v_{allow} =$ 221 lb/ft **OK**

Check Uplift: Roof Tributary Area: $A_t := 2.5ft$

Overturning Moment: $M_{ot\_EQ} := F_{EQ} \cdot Percent \cdot h_{floor_2} = 4.89 \cdot kip \cdot ft$ $M_{ot\_wind} := F_{wind\_t} \cdot Percent \cdot h_{floor_2} = 13.18 \cdot kip \cdot ft$

Resisting Moment:

$$M_{res\_EQ} := (0.6 - 0.14 \cdot S_{ds}) \cdot \left(DL_{roof} \cdot \frac{l_{wall}^2}{2} \cdot A_t + DL_{wall} \cdot l_{shearwall} \cdot h_{floor_2} \cdot \frac{l_{wall}}{2} \right) \quad M_{res\_EQ} = 8.12 \cdot kip \cdot ft$$

$$M_{res\_wind} := 0.6 \cdot \left(DL_{roof} \cdot \frac{l_{wall}^2}{2} \cdot A_t + DL_{wall} \cdot l_{shearwall} \cdot h_{floor_2} \cdot \frac{l_{wall}}{2} \right) \quad M_{res\_wind} = 8.41 \cdot kip \cdot ft$$

Chord Forces: $tension := max(v_{seismic} \cdot h_{plate}, v_{wind} \cdot h_{plate})$ $tension = 732\ lb$ OK

Uplift Anchorage at shear wall ends: $uplift := max\left(\frac{M_{ot\_EQ} - M_{res\_EQ}}{l_{wall}}, \frac{M_{ot\_wind} - M_{res\_wind}}{l_{wall}} \right)$ $uplift = 265\ lb$

Provide MR88 Earth Anchors

Shear Wall Design: Consider Wall B&D:

Percentage of lateral force taken by wall: $\text{Percent} := 50\%$

Total Wall Length: $l_{wall} := 24ft$ Length of Shear Wall: $l_{shearwall} := 24ft$

Seismic Force: $F_{EQ} = 1271.1\ lb$ Longitudinal Wind Force: $F_{wind\_l} = 2080.88\ lb$

Fastener type (penny weight): $nail\_size := 8$ Sheathing grade: $sheathing := \text{"Smart Panel"}$

Fastener spacing at edge: $edge\_spacing := 6 \cdot in$ Sheathing thickness: $thickness := \frac{3}{8} \cdot in$

Code provisions:

{18% reduction if hem-fir studs are used} {Reduction for panels applied over gypsum}

{40% allowable increase for wind design permitted} {Aspect ratio reduction for seismic}

Adjustment for perforated shearwall: $max\_height\_opening := 0 \cdot ft$ Minimum panel: $l_{panel} := 24ft$

Perfortated multiplier: $C_o = 1$ SDPWS Table 4.3.3.5

Shear Stress: $v_{seismic} := \frac{F_{EQ} \cdot \text{Percent}}{l_{shearwall}}$ $v_{seismic} = 26 \cdot plf$ $v_{wind} := \frac{F_{wind\_l} \cdot \text{Percent}}{l_{shearwall}}$ $v_{wind} = 43 \cdot plf$

For 0.375 in. Smart Panel applied directly to framing w/ 8d nails @ 6 in. edge spacing.

Seismic Design: Since $v_{seismic}$ = 26 lb/ft <= v_{allow} = 148 lb/ft **OK**

Wind Design: Since v_{wind} = 43 lb/ft <= v_{allow} = 148 lb/ft **OK**

Check Uplift:

Roof Tributary Area: $A_t := 0.5B$

Overturning Moment: $M_{ot\_EQ} := F_{EQ} \cdot \text{Percent} \cdot h_{floor_2} = 4.89 \cdot kip \cdot ft$ $M_{ot\_wind} := F_{wind\_l} \cdot \text{Percent} \cdot h_{floor_2} = 8 \cdot kip \cdot ft$

Resisting Moment:

$$M_{res\_EQ} := (0.6 - 0.14 \cdot S_{ds}) \cdot \left(DL_{roof} \cdot \frac{l_{wall}^2}{2} \cdot A_t + DL_{wall} \cdot l_{shearwall} \cdot h_{floor_2} \cdot \frac{l_{wall}}{2} \right)$$ $M_{res\_EQ} = 25.29 \cdot kip \cdot ft$

$$M_{res\_wind} := 0.6 \cdot \left(DL_{roof} \cdot \frac{l_{wall}^2}{2} \cdot A_t + DL_{wall} \cdot l_{shearwall} \cdot h_{floor_2} \cdot \frac{l_{wall}}{2} \right)$$ $M_{res\_wind} = 26.18 \cdot kip \cdot ft$

Chord Forces: $tension := \max(v_{seismic} \cdot h_{plate}, v_{wind} \cdot h_{plate})$ $tension = 333\ lb$ OK

Uplift Anchorage at shear wall ends: $$uplift := \max\left(\frac{M_{ot\_EQ} - M_{res\_EQ}}{l_{wall}}, \frac{M_{ot\_wind} - M_{res\_wind}}{l_{wall}} \right)$$ $uplift = -758\ lb$

No Anchors Required

Overall Overturning:

Driving Forces:

Overturning Moments:

$$M_{ot1} := 0.6L\cdot\left(P_{whtw}\cdot Trib\_area_{wall}\cdot h_{floor_2} + P_{whtr}\cdot Trib\_area_{roof\_t}\cdot h_{floor_3}\right) \qquad M_{ot1} = 26.44\cdot kip\cdot ft$$

$$M_{ot2} := F_{EQ}\cdot h_{floor_2} \qquad M_{ot2} = 9.77\cdot kip\cdot ft$$

Uplifting Moment: $M_{uplift} := 0.6\cdot\left(-P_{wvtw}\cdot L\cdot 3\cdot\frac{B^2}{8} - P_{wvtl}\cdot L\cdot\frac{B^2}{8}\right)$ $\qquad M_{uplift} = 31.39\cdot kip\cdot ft$

Combined Driving Moment: $M_{driving} := \max\left(M_{ot1} + M_{uplift}, M_{ot2}\right)$ $\qquad M_{driving} = 57.83\cdot kip\cdot ft$

Resisting Moment: $M_{res} := \left(W_{s\_ot}\cdot\frac{B}{2}\right)\cdot 0.6$ $\qquad M_{res} = 86.87\cdot kip\cdot ft$

Since	M_{res}	86.9	>=	$M_{driving}$	57.8	**OK**