STRUCTURAL CALCULATIONS
FOR
BENJAMIN ALTMAN
APN 3024059151
MERCER ISLAND, WA 98040
EAST LOT
FOR
BENJAMIN ALTMAN

PROJECT NO.: 2020-0196

SITE SPECIFIC VERTICAL AND LATERAL ANALYSIS AND DESIGN (DO NOT REUSE)

ORGINATED BY
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MC² MC SQUARED INCORPORATED

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SUITE 101
OLYMPIA, WA 98506
360-754-9339
360-352-2044

STRUCTURAL ● FOUNDATION ● CIVIL ENGINEERS

JESSE M. CHASE
STATE OF WASHINGTON
47564
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

01-06-22

DESIGN LOADS

Scope:

Client requested structural engineering to provide vertical and lateral engineering for a new home in Mercer Island, Washington.

Basis of design is drawings provided by client. No analysis and design of bracing, temporary or permanent, requested or conducted. All bracing, temporary and permanent, shall be the responsibility of the contractor.

CAUTION:
CONTRACTOR TO FIELD VERIFY ALL CONDITIONS AND ALL ELEVATION.

Standard Values for Calculations

Define concrete strength: $f'_c := 4000 \cdot psi$

Define rebar yield strength: $f_y := 60 \cdot ksi$

Define levels:

$$Level := \begin{pmatrix} \text{"R"} \\ \text{"2F"} \\ \text{"MF"} \end{pmatrix}$$

Design Data / Definitions:

Design loads are per 2015 IBC and ASCE 7-10.

Define occupancy category: $Category := \text{"II"}$

Define roof slope: $F_r := \frac{4}{12}$

$\theta_r := atan(F_r)$ $\theta_r = 18.43 \cdot deg$

Define patio cover length: $L_{w\_pc} := 30 \cdot ft + 0 \cdot in$

Define patio cover width: $B_{n\_pc} := 10 \cdot ft + 0 \cdot in$

Define covered veranda length: $L_{w\_cv} := 16 \cdot ft + 6 \cdot in$

Define covered veranda width: $B_{n\_cv} := 6 \cdot ft + 9 \cdot in$

Define building overall wide length:

$$L_w := \begin{pmatrix} 42 \\ 86 \\ 52 \end{pmatrix} \cdot ft + \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} \cdot in$$

Define building overall narrow width:

$$B_n := \begin{pmatrix} 42 \\ 42 \\ 42 \end{pmatrix} \cdot ft + \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} \cdot in$$

Define roof overhang (max): $L_{roh} := 2 \cdot ft + 0 \cdot in$

Define roof height (trusses height): $h_r := 7 \cdot ft + 3 \cdot in$

Define floor to floor heights:

$$H_f := \begin{pmatrix} 10 \\ 10 \\ 10 \end{pmatrix} \cdot ft + \begin{pmatrix} 1.75 \\ 1.75 \\ 1.75 \end{pmatrix} \cdot in$$

Define double top plate to sill plate heights:

$$H_s := \begin{pmatrix} 9 \\ 9 \\ 9 \end{pmatrix} \cdot ft + \begin{pmatrix} 1.125 \\ 1.125 \\ 1.125 \end{pmatrix} \cdot in$$

Define effective floor plan area per level:

$$A_{Tf} := \begin{pmatrix} 0 \\ 1764 \\ 2055 \end{pmatrix} \cdot ft^2$$

Define effective roof plan area per level:

$$A_{Tr} := \begin{pmatrix} 2116 \\ 1531 \\ 0 \end{pmatrix} \cdot ft^2$$

Define effective patio cover & covered veranda plan area per level:

$$A_{Tpc} := \begin{pmatrix} 0 \\ 408 \\ 0 \end{pmatrix} \cdot ft^2 \qquad A_{Tcv} := \begin{pmatrix} 0 \\ 160 \\ 0 \end{pmatrix} \cdot ft^2$$

Define exterior wall perimeter length:

$$P_o := \begin{pmatrix} 168 \\ 256 \\ 188 \end{pmatrix} \cdot ft + \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} \cdot in$$

Define projected wind area perp. to narrow face (loads parallel to wide face) per level:

$$A_{PRn} := \begin{pmatrix} 409 \\ 418 \\ 320 \end{pmatrix} \cdot ft^2$$

Define projected wind area perp. to wide face (loads parallel to narrow face) per level:

$$A_{PRw} := \begin{pmatrix} 417 \\ 604 \\ 482 \end{pmatrix} \cdot ft^2$$

Define patio cover projected wind area perp. to narrow face: $A_{PRpc} := 23 \cdot ft^2$

Define covered veranda projected wind area perp. to narrow face: $A_{PRcv} := 16 \cdot ft^2$

Roof eave height,

$H_e := \sum H_f$ $H_e = 30.44 ft$

Cumulative floor heights,

$H_c := reverse(Cumulative(reverse(H_f)))$

$$H_c = \begin{pmatrix} 30.44 \\ 20.29 \\ 10.15 \end{pmatrix} ft$$

Shrinkage Calculation:

Note: Floor joists and rim joists are manufactured members, therefore no shrinkage.

Define moisture content: $MC := 19 \cdot \%$

Define EMC moisture content: $EMC := 15 \cdot \%$

Define shrinkage coefficient: $C_{shrink} := 0.0020$

Define max. number of 2x6 top plates: $n_{top} := 6$

Define max. number of 2x6 sole plates: $n_{sole} := 4$

Total shrinkage dimension (assume 2 sill plates),

$D_{tot} := (1.5 \cdot in) \cdot (n_{top} + n_{sole} + 2)$ $\qquad D_{tot} = 18.00 \cdot in$

Total shrinkage,

$S_{tot} := C_{shrink} \cdot (MC - EMC) \cdot D_{tot}$ $\qquad S_{tot} = 0.00144 \cdot in$

Soils:

Define allowable sustained vert. bearing press. [Geotech Report]: $q_{Sv} := 1500 \cdot psf$

Define soil density [Assumed]: $\gamma_g := 120 \cdot pcf$

Define active lateral earth pressure [Geotech Report]: $K_a := 35 \cdot pcf$

Define at-rest lateral earth pressure [Geotech Report]: $K_o := 50 \cdot pcf$

Define allowable passive lateral earth pressure [Geotech Report]: $K_{p\_all} := 300 \cdot pcf$

Define allowable coefficient of friction [Geotech Report]: $\mu_{q\_all} := 0.40$

Define applied seismic force [Geotech Report]: $K_e = 6 \cdot H$

Define surcharge coefficient [Geotech Report]: $\nu_q := 0.30$

Effective friction angle,

$\phi'_e := 90 \cdot deg - 2 \cdot atan\left(\sqrt{\frac{K_a}{\gamma_g}}\right)$ $\qquad \phi'_e = 33.3 \cdot deg$

Wall friction angle,

$\delta_w := 0.5 \cdot \phi'_e$ $\qquad \delta_w = 16.63 \cdot deg$

Passive lateral earth pressure,

$K_p := Floor(1.5 \cdot K_{p\_all}, 5 \cdot pcf)$ $\qquad K_p = 450\, pcf$

Allowable vert. intermittent bearing press. [IBC-15, Sect. 1807.2.3, Exception],

$q_{Iv} := \frac{4}{3} \cdot q_{Sv}$ $\qquad q_{Iv} = 2000.00\, psf$

Vertical Loads:

Dead Loads:

Concrete density [ASCE 7-10, Table C3-2]: $\gamma_c := 150 \cdot pcf$

Partition dead load [ASCE 7-10, Sect. 12.7.2.2.]: $DL_p := 10 \cdot psf$

Exterior Walls Dead Loads

Define sheathing thickness: $t_{ew} := \frac{1}{2} \cdot in$ $\quad t_{ew} = 0.500 \cdot in$

Define sheathing weight [NDS-15, Table 9.2.4]: $DL_{ew\_sh} := 1.9 \cdot psf$

Define siding weight [ASCE 7-10, Table C3-1]: $DL_{ew\_s} := 4.0 \cdot psf$

Define studs weight (2x6's @ 16" o.c.) [ASCE 7-10, Table C3-1]: $DL_{ew\_st} := 1.7 \cdot psf$

Define insulation weight [ASCE 7-10, Table C3-1]: $DL_{ew\_i} := 1.0 \cdot psf$

Define GWB (5/8") [ASCE 7-10, Table C3-1]: $DL_{ew\_g} := 2.8 \cdot psf$

Define misc. dead load (piping, mechanical, electrical, etc.): $DL_{ew\_m} := 3.0 \cdot psf$

Exterior walls weight,

$$DL_{ew} := Ceil\left(DL_{ew\_sh} + DL_{ew\_s} + DL_{ew\_st} + DL_{ew\_i} + DL_{ew\_g} + DL_{ew\_m}, 1 \cdot psf\right) \qquad DL_{ew} = 15 \cdot psf$$

Interior (Walls) Dead Loads

Define sheathing thickness: $t_{pw} := \frac{1}{2} \cdot in$ $\quad t_{pw} = 0.500 \cdot in$

Define sheathing weight [NDS-15, Table 9.2.4]: $DL_{pw\_sh} := 1.5 \cdot psf$

Define studs weight (2x6's @ 16" o.c.) [ASCE 7-10, Table C3-1]: $DL_{pw\_st} := 1.7 \cdot psf$

Define insulation weight [ASCE 7-10, Table C3-1]: $DL_{pw\_i} := 1.0 \cdot psf$

Define GWB (5/8") [ASCE 7-10, Table C3-1]: $DL_{pw\_g} := 2.8 \cdot psf$

Define misc. dead load (piping, mechanical, electrical, etc.): $DL_{pw\_m} := 2.0 \cdot psf$

Interior walls weight,

$$DL_{iw} := Ceil\left(DL_{pw\_sh} + DL_{pw\_st} + DL_{pw\_i} + 2 \cdot DL_{pw\_g} + DL_{pw\_m}, 1 \cdot psf\right) \qquad DL_{iw} = 12 \cdot psf$$

Roof Dead Loads

Define sheathing thickness:	$t_r := \frac{1}{2} \cdot in$	$t_r = 0.500 \cdot in$
Define sheathing weight [NDS-15, Table C9.2.4]:	$DL_{r\_sh} := 1.9 \cdot psf$	
Define asphalt shingles weight [ASCE 7-10, Table C3-1]:	$DL_{r\_as} := 5.5 \cdot psf$	
Define insulation weight [ASCE 7-10, Table C3-1]:	$DL_{r\_i} := 1.0 \cdot psf$	
Define GWB (5/8") [ASCE 7-10, Table C3-1]:	$DL_{r\_g} := 2.8 \cdot psf$	
Define truss self weight:	$DL_{r\_tr} := 2.8 \cdot psf$	
Define misc. dead load (piping, mechanical, electrical, etc.):	$DL_{r\_m} := 3.0 \cdot psf$	

Truss roof weight,

$$DL_r := Ceil\left(DL_{r\_sh} + DL_{r\_as} + DL_{r\_i} + DL_{r\_g} + DL_{r\_tr} \ldots + DL_{r\_m}, 1 \cdot psf\right) \qquad DL_r = 17 \cdot psf$$

Floor Dead Loads

Define sheathing thickness:	$t_f := \frac{3}{4} \cdot in$	$t_f = 0.750 \cdot in$
Define sheathing weight [NDS-15, Table C9.2.4]:	$DL_{f\_sh} := 2.3 \cdot psf$	
Define I-Joists weight [TrusJoist]:	$DL_{Ij} := 3.6 \cdot psf$	
Define insulation weight [ASCE 7-10, Table C3-1]:	$DL_{f\_i} := 1.5 \cdot psf$	
Define GWB (5/8") [ASCE 7-10, Table C3-1]:	$DL_{f\_g} := 2.8 \cdot psf$	
Define misc. dead load (piping, mechanical, electrical, etc.):	$DL_{f\_m} := 4.0 \cdot psf$	

Floor weight,

$$DL_f := Ceil\left(DL_{f\_sh} + DL_{Ij} + DL_{f\_i} + DL_{f\_g} + DL_{f\_m}, 1 \cdot psf\right) \qquad DL_f = 15 \cdot psf$$

Live Loads:

Roof live load [ASCE 7-10,Table 4-1]:	$LL_r := 20 \cdot psf$
Floor live load [ASCE 7-10,Table 4-1]:	$LL_f := 40 \cdot psf$
Main floor / slab on grade [ASCE 7-10,Table 4-1]:	$LL_m := 100 \cdot psf$

Snow Loads:

Define ground snow load:	$p_g := 15 \cdot psf$
Define sloped snow [City of Mercer Island]:	$SL_s := 25 \cdot psf$

Lateral Loads:

Live Loads:

Define wall live load [IBC 2015, Sect. 1607.14]: $LL_w := 5 \cdot psf$

Wind Loads:

Define wind speed [City of Mercer Island]: $V_{ult} := 110 \cdot mph$

Define exposure category [City of Mercer Island]: $Exposure_W := \text{"C"}$

Topographic factor [City of Mercer Island]: $K_{zt} := 1.0$

Nominal wind speed [IBC 2015, Sect. 1609.3.1, Eq. 16-33],

$V_{asd} := V_{ult} \cdot \sqrt{0.6}$ $\qquad V_{asd} = 85.21 \cdot mph$

Mean roof height,

$h_m := H_e + 0.5 \cdot h_r$ $\qquad h_m = 34.06 ft$

MWFRS Wind Loads:

$\theta_r = 18.43 \cdot deg$

Minimum design wind pressure [ASCE 7-10, Sect. 28.6.4]: $WL_{MWmin} := 16 \cdot psf$

Building height adjustment factor. [ASCE 7-10, Fig. 28.6-1]:

$\lambda_{MW} := linterp\left[\begin{pmatrix} 30 \\ 35 \end{pmatrix} \cdot ft, \begin{pmatrix} 1.40 \\ 1.45 \end{pmatrix}, h_m\right]$ $\qquad \lambda_{MW} = 1.44$

Horizontal Loads

Design wind pressure (Ave. of Zones A & C) [ASCE 7-10, Fig. 28.6-1]:

$p_{S30h} := 0.5 \cdot (26.6 \cdot psf + 17.7 \cdot psf)$ $\qquad p_{S30h} = 22.15 psf$

Net design horizontal wind pressure [ASCE 7-10, Sect. 28.6.3],

$p_{Sh} := \lambda_{MW} \cdot K_{zt} \cdot p_{S30h}$ $\qquad p_{Sh} = 31.91 psf$

Strength horizontal wind pressure [ASCE 7-10, Sect. 2.3.1],

$WL_{MWhLRFD} := max(p_{Sh}, WL_{MWmin})$ $\qquad WL_{MWhLRFD} = 31.9 psf$

Service horizontal wind pressure [ASCE 7-10, Sect. 2.4.1],

$WL_{MWhASD} := 0.6 \cdot WL_{MWhLRFD}$ $\qquad WL_{MWhASD} = 19.1 psf$

Vertical Loads

Design wind pressure (Ave. of Zones E & G) [ASCE 7-10, Fig. 28.6-1]:

$p_{S30v} := 0.5 \cdot (-23.1 \cdot psf + -16.0 \cdot psf)$ $\qquad p_{S30v} = -19.55 psf$

Net design vertical wind pressure [ASCE 7-10, Sect. 28.6.3],

$p_{Sv} := \lambda_{MW} \cdot K_{zt} \cdot p_{S30v}$ $\qquad p_{Sv} = -28.16 psf$

Strength vertical wind pressure [ASCE 7-10, Sect. 2.3.1],

$WL_{MWvLRFD} := min(p_{Sv}, -WL_{MWmin})$ $\qquad WL_{MWvLRFD} = -28.2 psf$

Service vertical wind pressure [ASCE 7-10, Sect. 2.4.1],

$WL_{MWvASD} := 0.6 \cdot WL_{MWvLRFD}$ $\qquad WL_{MWvASD} = -16.9 psf$

MWFRS Patio Cover Force:

Horizontal deck force,

$F_{wl\_pc} := WL_{MWhLRFD} \cdot A_{PRpc}$ $\qquad F_{wl\_pc} = 0.73 kip$ $\qquad F_{wl\_pc} = 733.93 \cdot lbf$

MWFRS Covered Veranda Force:

Horizontal deck force,

$F_{wl\_cv} := WL_{MWhLRFD} \cdot A_{PRcv}$ $\qquad F_{wl\_cv} = 0.51 kip$ $\qquad F_{wl\_cv} = 510.56 \cdot lbf$

MWFRS Forces Perpendicular to Wide Face:

Horizontal force per level perp. to wide face,

$$F_{wl\_w} := WL_{MWhLRFD} \cdot A_{PRw} \qquad F_{wl\_w} = \begin{pmatrix} 13.31 \\ 19.27 \\ 15.38 \end{pmatrix} kip \qquad F_{wl\_w} = \begin{pmatrix} 13306 \\ 19274 \\ 15381 \end{pmatrix} \cdot lbf$$

Cumulative horizontal force per level perp. to wide face,

$$\Sigma F_{wl\_w} := Cumulative(F_{wl\_w}) \qquad \Sigma F_{wl\_w} = \begin{pmatrix} 13.31 \\ 32.58 \\ 47.96 \end{pmatrix} kip$$

Base shear perp. to wide face,

$$V_{wLRFD\_w} := \sum F_{wl\_w} \qquad V_{wLRFD\_w} = 48.0\, kip$$

$$V_{wASD\_w} := 0.6 \cdot \sum F_{wl\_w} \qquad V_{wASD\_w} = 28.8\, kip$$

Distributed ASD wind loads per level,

$$w_{ASD\_w} := \frac{F_{wl\_w}}{L_w} \qquad w_{ASD\_w} = \begin{pmatrix} 316.82 \\ 224.11 \\ 295.78 \end{pmatrix} plf$$

MWFRS Forces Perpendicular to Narrow Face:

Horizontal force per level perp. to narrow face,

$$F_{wl\_n} := WL_{MWhLRFD} \cdot A_{PRn} + \begin{pmatrix} 0 \\ F_{wl\_pc} + F_{wl\_cv} \\ 0 \end{pmatrix} \qquad F_{wl\_n} = \begin{pmatrix} 13.05 \\ 14.58 \\ 10.21 \end{pmatrix} kip \qquad F_{wl\_n} = \begin{pmatrix} 13051 \\ 14583 \\ 10211 \end{pmatrix} \cdot lbf$$

Cumulative horizontal force per level perp. to narrow face,

$$\Sigma F_{wl\_n} := Cumulative(F_{wl\_n}) \qquad \Sigma F_{wl\_n} = \begin{pmatrix} 13.05 \\ 27.63 \\ 37.85 \end{pmatrix} kip$$

Base shear perp. to narrow face,

$$V_{wLRFD\_n} := \sum F_{wl\_n} \qquad V_{wLRFD\_n} = 37.8\, kip$$

$$V_{wASD\_n} := 0.6 \cdot \sum F_{wl\_n} \qquad V_{wASD\_n} = 22.7\, kip$$

Distributed ASD wind loads per level,

$$w_{ASD\_n} := \frac{F_{wl\_n}}{B_n} \qquad w_{ASD\_n} = \begin{pmatrix} 310.74 \\ 347.21 \\ 243.12 \end{pmatrix} plf$$

C&C Wind Loads:

Minimum design wind pressure [ASCE 7-10, Sect. 30.2.2]: $WL_{CCmin} := 16 \cdot psf$

Building height adjustment factor. [ASCE 7-10, Fig. 30.5-1]:

$$\lambda_{CC} := linterp\left[\begin{pmatrix} 30 \\ 35 \end{pmatrix} \cdot ft, \begin{pmatrix} 1.40 \\ 1.45 \end{pmatrix}, h_m\right] \qquad \lambda_{CC} = 1.44$$

Vertical Loads $\theta_r = 18.43 \cdot deg$

Design vertical wind pressure (Aeff = 10 sq. ft. in Zone 3) [ASCE 7-10, Fig. 30.5-1]:

$$p_{net30\_v} := \begin{pmatrix} 12.5 \\ -51.3 \end{pmatrix} \cdot psf \qquad p_{net30\_v} = \begin{pmatrix} 12.50 \\ -51.30 \end{pmatrix} psf$$

Net design vertical wind pressure [ASCE 7-10, Sect. 28.6.3],

$$p_{net\_v} := \lambda_{CC} \cdot K_{zt} \cdot p_{net30\_v} \qquad p_{net\_v} = \begin{pmatrix} 18.01 \\ -73.90 \end{pmatrix} psf$$

Strength vertical wind pressure [ASCE 7-10, Sect. 2.3.1],

$$WL_{CCvLRFD} := stack\left(max\left(p_{net\_v}, WL_{CCmin}\right), min\left(p_{net\_v}, -WL_{CCmin}\right)\right) \qquad WL_{CCvLRFD} = \begin{pmatrix} 18.0 \\ -73.9 \end{pmatrix} psf$$

Service vertical wind pressure [ASCE 7-10, Sect. 2.4.1],

$$WL_{CCvASD} := 0.6 \cdot WL_{CCvLRFD} \qquad WL_{CCvASD} = \begin{pmatrix} 10.8 \\ -44.3 \end{pmatrix} psf$$

Horizontal Loads

Design horizontal wind pressure (Aeff = 10 sq. ft. in Zone 5) [ASCE 7-10, Fig. 30.5-1]:

$$p_{net30\_h} := \begin{pmatrix} 21.8 \\ -29.1 \end{pmatrix} \cdot psf \qquad p_{net30\_h} = \begin{pmatrix} 21.80 \\ -29.10 \end{pmatrix} psf$$

Net design horizontal wind pressure [ASCE 7-10, Sect. 28.6.3],

$$p_{net\_h} := \lambda_{CC} \cdot K_{zt} \cdot p_{net30\_h} \qquad p_{net\_h} = \begin{pmatrix} 31.41 \\ -41.92 \end{pmatrix} psf$$

Strength horizontal wind pressure [ASCE 7-10, Sect. 2.3.1],

$$WL_{CChLRFD} := stack\left(max\left(p_{net\_h}, WL_{CCmin}\right), min\left(p_{net\_h}, -WL_{CCmin}\right)\right) \qquad WL_{CChLRFD} = \begin{pmatrix} 31.4 \\ -41.9 \end{pmatrix} psf$$

Service horizontal wind pressure [ASCE 7-10, Sect. 2.4.1],

$$WL_{CChASD} := 0.6 \cdot WL_{CChLRFD} \qquad WL_{CChASD} = \begin{pmatrix} 18.8 \\ -25.2 \end{pmatrix} psf$$

Seismic Loads:

ATC Hazards by Location

Search Information

Address: 6423 E Mercer Way, Mercer Island, WA 98040, USA

Coordinates: 47.54513399999999, -122.2129469

Elevation: 172 ft

Timestamp: 2020-04-23T15:08:11.408Z

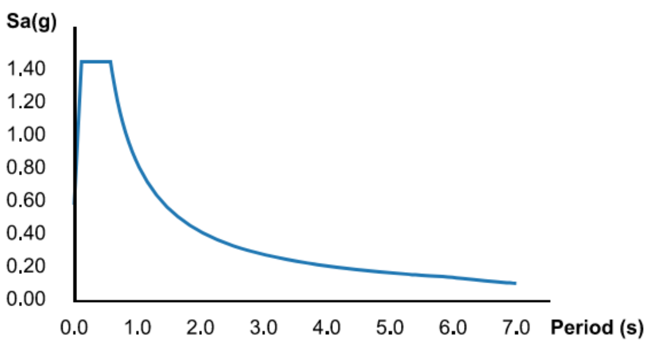
Hazard Type: Seismic

Reference Document: ASCE7-10

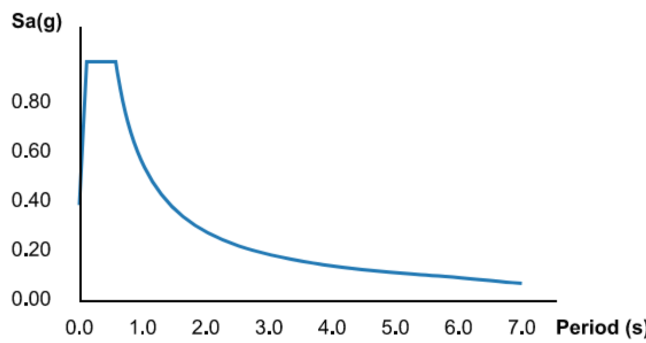
Risk Category: II

Site Class: D

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	1.449	MCE_R ground motion (period=0.2s)
S_1	0.554	MCE_R ground motion (period=1.0s)
S_{MS}	1.449	Site-modified spectral acceleration value
S_{M1}	0.832	Site-modified spectral acceleration value
S_{DS}	0.966	Numeric seismic design value at 0.2s SA
S_{D1}	0.554	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	1.5	Site amplification factor at 1.0s

Seismic Parameters:

0.2sec mapped & design spectral resp. coef. [Geotech Report]: $S_S := 1.449$ $S_{DS} := 0.996$

1.0sec mapped & design spectral resp. coef. [Geotech Report]: $S_1 := 0.554$ $S_{D1} := 0.554$

Soil site class [Geotech Report]: $Site := \text{"D"}$

Seismic resisting factor [ASCE 7-10, Table 12.2-1.A.15]: $R_E := 6.5$

Overstrength factor [ASCE 7-10, Table 12.2-1.A.15]: $\Omega_o := 2.5$ ***ASCE 7-10, Table 12.2-1, Footnote g.***

Deflection amplification factor [ASCE 7-10, Table 12.2-1.A.15]: $C_d := 4.0$

Long period transition period [ASCE 7-10, Fig. 22-12]: $T_L := 6 \cdot s$

Seismic reliability factor [ASCE 7-10, Sect. 12.3.4.2]: $\rho_o := 1.3$

Approx. period coefficient [ASCE 7-10, Table 12.8-2]: $C_{ta} := 0.02$

Approx. period exponent [ASCE 7-10, Table 12.8-2]: $x_a := 0.75$

Approx. period of structure [ASCE 7-10, Sect. 12.8.2.1, Eq. 12.8-7],

$h_n := h_m$ $\qquad h_n = 34.06\,ft$

$T_a := C_{ta} \cdot \left(h_n \cdot ft^{-1}\right)^{x_a} \cdot s$ $\qquad T_a = 0.282\,s$

Vertical distribution exponential factor [ASCE 7-10, Sect. 12.8.3],

$$k_E := \left| \begin{array}{l} 1 \;\; if \;\; T_a \le 0.5 \cdot s \\ linterp\left[\begin{pmatrix} 0.5 \cdot s \\ 2.5 \cdot s \end{pmatrix}, \begin{pmatrix} 1 \\ 2 \end{pmatrix}, T_a\right] \;\; if \;\; 0.5 \cdot s < T_a < 2.5 \cdot s \\ 2 \;\; if \;\; T_a \ge 2.5 \cdot s \end{array} \right. \qquad k_E = 1.00$$

Importance factor [ASCE 7-10, Table 1.5-2],

$$I_E := \left| \begin{array}{l} 1.00 \;\; if \;\; Category = \text{"I"} \\ 1.00 \;\; if \;\; Category = \text{"II"} \\ 1.25 \;\; if \;\; Category = \text{"III"} \\ 1.50 \;\; if \;\; Category = \text{"IV"} \end{array} \right. \qquad I_E = 1.00$$

▶ Seismic Design Category

Seismic design category [ASCE 7-10, Sect. 11.6], $SDC = \text{"D"}$

Structure plan weight,

$$P_{DL\_p} := DL_r \cdot A_{Tr} + \left(DL_f + DL_p\right) \cdot A_{Tf} \; ... \; + DL_r \cdot A_{Tpc} + DL_r \cdot A_{Tcv}$$

$$P_{DL\_p} = \begin{pmatrix} 35.97 \\ 79.78 \\ 51.38 \end{pmatrix} kip$$

Structure elevation weight,

$$P_{DL\_el_k} := DL_{ew} \cdot P_{o_k} \cdot if\left(k = 1, \frac{H_{f_k}}{2}, \frac{H_{f_{k-1}}}{2} + \frac{H_{f_k}}{2}\right)$$

$$P_{DL\_el} = \begin{pmatrix} 12.78 \\ 38.96 \\ 28.61 \end{pmatrix} kip$$

Structure total weight,

$$W_{x_k} := P_{DL\_p_k} + P_{DL\_el_k}$$

$$W_x = \begin{pmatrix} 48.76 \\ 118.74 \\ 79.99 \end{pmatrix} kip$$

Cumulative weight per floor,

$$\Sigma W_x := Cumulative\left(W_x\right)$$

$$\Sigma W_x = \begin{pmatrix} 48.76 \\ 167.50 \\ 247.49 \end{pmatrix} kip$$

Total weight,

$W_X := \sum W_x$ $\qquad W_X = 247.49\,kip$

MSFRS:

Minimum seismic response coefficient [ASCE 7-10, Sect. 12.8.1.1, Eq. 12.8-5 & 12.8-6],

$$C_{s\_min} := if\left(S_1 \geq 0.6, \frac{0.5 \cdot S_1 \cdot I_E}{R_E}, max\left(0.044 \cdot S_{DS} \cdot I_E, 0.01\right)\right) \qquad C_{s\_min} = 0.044$$

Seismic response coefficient [ASCE 7-10, Sect. 12.8.1.1, Eq. 12.8-2],

$$C_s := \frac{S_{DS} \cdot I_E}{R_E} \qquad C_s = 0.153$$

Maximum seismic response coefficient [ASCE 7-10, Sect. 12.8.1.1, Eq. 12.8-3 & Eq. 12.8-4],

$$C_{s\_maw} := if\left(T_a \leq T_L, \frac{S_{D1} \cdot s \cdot I_E}{T_a \cdot R_E}, \frac{S_{D1} \cdot T_L \cdot s \cdot I_E}{T_a \cdot T_a \cdot R_E}\right) \qquad C_{s\_maw} = 0.302$$

Design horizontal acceleration [ASCE 7-10, Sect. 12.8.1.1],

$$C_{sLRFD} := max\left(min\left(C_s, C_{s\_maw}\right), C_{s\_min}\right) \qquad C_{sLRFD} = 0.153$$

$$C_{sASD} := 0.7 \cdot C_{sLRFD} \qquad C_{sASD} = 0.107$$

Base shear [ASCE 7-10, Sect. 12.8, Eq. 12.8-1],

$$V_{eLRFD} := C_{sLRFD} \cdot W_X \qquad V_{eLRFD} = 37.9\ kip \qquad \rho_o \cdot V_{eLRFD} = 49.3\ kip$$

$$V_{eASD} := 0.7 \cdot V_{eLRFD} \qquad V_{eASD} = 26.5\ kip \qquad \rho_o \cdot V_{eASD} = 34.5\ kip$$

Vertical distribution of seismic forces [ASCE 7-10, Sect. 12.8.3, Eq. 12.8-12],

$Table_{Fx} =$

"Level"	"w (kip)"	"h (ft)"	"wh^k"	"Cvx"	"Fx (kip)"	"Vx (kip)"
"R"	48.76	30.44	1484.00	0.32	11.96	11.96
"2F"	118.74	20.29	2409.49	0.51	19.42	31.38
"MF"	79.99	10.15	811.53	0.17	6.54	37.92
"Sum"	247.49	""	4705.02	1.00	37.92	""

Diaphragm Seismic Forces:

Minimum diaphragm coeff. [ASCE 7-10, Sect. 12.10.1.1, Eq. 12.10-2],

$C_{px\_min} := 0.2 \cdot S_{DS} \cdot I_E$ $\qquad C_{px\_min} = 0.199$

Maximum diaphragm coeff. [ASCE 7-10, Sect. 12.10.1.1, Eq. 12.10-3],

$C_{px\_max} := 0.4 \cdot S_{DS} \cdot I_E$ $\qquad C_{px\_max} = 0.398$

Actual diaphragm forces [ASCE 7-10, Sect. 12.10.1.1, Eq. 12.10-1],

$Table_{Fpx} =$

"Level"	"wpx (kip)"	"Σwpx (kip)"	"Fx (kip)"	"ΣFx (kip)"	"ΣFx/Σwpx"	"Fpx (kip)"	"γ"
"R"	48.76	48.76	11.96	11.96	0.25	11.96	1.00
"2F"	118.74	167.50	19.42	31.38	0.19	23.65	1.22
"MF"	79.99	247.49	6.54	37.92	0.15	15.93	2.44
"Sum"	247.49	""	37.92	""	""	""	""

Shear force per floor,

$F_{ELp} := 0.7 \cdot F_{px}$ $\qquad F_{ELp} = \begin{pmatrix} 8373 \\ 16558 \\ 11153 \end{pmatrix} \cdot lbf \qquad \gamma_{px} = \begin{pmatrix} 1.000 \\ 1.218 \\ 2.436 \end{pmatrix}$

Uniform seismic load,

$EL_p := \dfrac{F_{ELp}}{A_{Tr} + A_{Tf} + A_{Tpc} + A_{Tcv}}$ $\qquad EL_p = \begin{pmatrix} 3.96 \\ 4.29 \\ 5.43 \end{pmatrix} psf$

Retaining Wall Loads:

Define back slope angle: $\alpha_b := 0 \cdot deg$

Define retaining wall slope measured from vert CCW: $\beta_w := 0 \cdot deg$

Define retained wall height from bottom of footing: $H_{rw} := 9.50 \cdot ft$

Seismic load + active earth pressure,,

$$K_{ae} := 6 \cdot \left(\frac{H_{rw}}{ft}\right) \cdot pcf \qquad K_{ae} = 57.00\,pcf$$

Seismic load only,

$$K_e := K_{ae} - K_a \qquad K_e = 22.00\,pcf$$

Seismic factor,

$$k_e := \frac{K_e}{\gamma_g} \qquad k_e = 0.183$$

Active earth pressure coefficient,

$$k_a := \frac{K_a}{\gamma_g} \qquad k_a = 0.292$$

Seismic-active earth pressure coefficient,

$$k_{ae} := k_e + k_a \qquad k_{ae} = 0.475 \qquad \frac{K_{ae}}{\gamma_g} = 0.475$$

Note: Use Monobe-Okave as a starting point.

Monobe-Okave/Seed-Whitman horizontal acceleration [2003 Commentary FEMA 450-2, Sect. 7.5.1],

$$k_{h0\_start} := \frac{S_{DS}}{2.5} \qquad k_{h0\_start} = 0.398$$

Design horizontal acceleration [AASHTO-12, Sect. 11.6.5.2.2],

$$k_{h\_start} := 0.5 \cdot k_{h0\_start} \qquad k_{h\_start} = 0.199$$

Seismic coefficient angle,

$$\theta_{e\_start} := atan\left(k_{h\_start}\right) \qquad \theta_{e\_start} = 11.27 \cdot deg$$

Seismic coefficient angle, Guess = $\theta_g := 15.35 \cdot deg$

$k_{ae} = 0.475$

$$k_{AE} := \frac{cos\left(\phi'_e - \theta_g - \beta_w\right)^2}{cos\left(\theta_g\right) \cdot cos\left(\beta_w\right)^2 \cdot cos\left(\delta_w + \beta_w + \theta_g\right)} \cdot \frac{1}{\left(1 + \sqrt{\frac{sin\left(\phi'_e + \delta_w\right) \cdot sin\left(\phi'_e - \theta_g - \alpha_b\right)}{cos\left(\delta_w + \beta_w + \theta_g\right) \cdot cos\left(\alpha_b - \beta_w\right)}}\right)^2}$$

$k_{AE} = 0.475$

Design horizontal acceleration,

$$k_h := tan\left(\theta_g\right) \qquad k_h = 0.275$$

LATERAL DESIGN

Shear Wall Design:

Multiple Rows of Nails Proof

Per ASCE 7-10, Sect. 1.3.1.3.1 Analysis, "Analysis shall employ rational methods based on accepted principles of engineering mechanics and shall consider all significant sources of deformation and resistance. Assumptions on stiffness, strength, damping, and other properties of components and connections incorporated in the analysis shall be based on approved test data or referenced standards."

Therefore...

Panel Capacity

In-Plane shear capacity of 7/16" OSB (min. thickness) for 24/16 span rating [APA 510, Table 8]: $Fvtv := 165 \cdot \frac{lbf}{in}$

Adjusted in-plane shear capacity for wind/seismic [NDS-15, Table 9.3.1],

$C_{d\_Fv} := 1.6$ Load duration factor [NDS-15, Table 2.3.2]

$Fvtv' := Fvtv \cdot C_{d\_Fv}$ $Fvtv' = 3168\ plf$

Nail Capacity $\boxed{nail := 1}$

Nail capacity for 10d common nails in DFL-N in panel w/G = 0.42 [NDS-15, Table 12R]: $Z_{n\_v} := 76 \cdot \frac{lbf}{nail}$

Adjusted nail capacity for wind/seismic [NDS-15, Table 11.3.1],

$C_{d\_Z} := 1.6$ Load duration factor [NDS-15, Table 2.3.2]

$C_{di\_Z} := 1.1$ Diaphragm factor [NDS-15, Sect. 12.5.3]

$Z'_n := Z_{n\_v} \cdot C_{d\_Z} \cdot C_{di\_Z}$ $Z'_n = 133.76 \cdot \frac{lbf}{nail}$

$\boxed{nl := 1..7}$

Rows of nails, $row_{nail} := \begin{pmatrix} 2 \\ 2 \\ 2 \\ 2 \\ 3 \\ 3 \\ 3 \end{pmatrix} \cdot nail$ Spacing of nails, $s_{nail} := \begin{pmatrix} 4.0 \\ 3.0 \\ 2.5 \\ 2.0 \\ 4.0 \\ 3.0 \\ 2.5 \end{pmatrix} \cdot in$

Nails per foot, Capacity,

$tot_{nail_{nl}} := \frac{row_{nail_{nl}}}{s_{nail_{nl}}}$ $tot_{nail} = \begin{pmatrix} 6 \\ 8 \\ 9.6 \\ 12 \\ 9 \\ 12 \\ 14.4 \end{pmatrix} \cdot \frac{nail}{ft}$ $Z'_{tot} := Z'_n \cdot tot_{nail}$ $Z'_{tot} = \begin{pmatrix} 803 \\ 1070 \\ 1284 \\ 1605 \\ 1204 \\ 1605 \\ 1926 \end{pmatrix} plf$

Conclusion: The maximum capacity for 3 rows of 10d common nails spaced at 2.5" is 1926 plf and is less than the the in-plane shear capacity of a 7/16" thick 24/16 span rated panel w/capacity of 3168 plf, therefore multiple rows of nails is acceptable. Note that this is in a structural panel w/G = 0.42 and using studs that are DFL-N. If studs are DFL and panels are G=0.50, then the shear wall capacity is higher yet.

Strap Development Length:

$k := 1..4$

Define strap capacity per Simpson:

$F_{strap} := stack(2490, 4585, 6490, 9215) \cdot lbf$

$$F_{strap} = \begin{pmatrix} 2.49 \\ 4.59 \\ 6.49 \\ 9.21 \end{pmatrix} kip$$

Define nail spacing:

$s_{strap} := stack(2, 3, 3.5, 3.5) \cdot in$

$$s_{strap} = \begin{pmatrix} 2.00 \\ 3.00 \\ 3.50 \\ 3.50 \end{pmatrix} \cdot in$$

Define nail sizes:

$Nail_{strap} := stack(\text{"10d"}, \text{"16d Sinker"}, \text{"16d"}, \text{"16d"})$

$$Nail_{strap} = \begin{pmatrix} \text{"10d"} \\ \text{"16d Sinker"} \\ \text{"16d"} \\ \text{"16d"} \end{pmatrix}$$

Define number of nails per member:

$n_{strap} := stack\left(\frac{26}{2}, \frac{50}{2}, \frac{56}{2}, \frac{74}{2}\right)$

$$n_{strap} = \begin{pmatrix} 13 \\ 25 \\ 28 \\ 37 \end{pmatrix}$$

Define nail shear capacity[NDS-15, Table 12P]:

$Z_{n\_strap} := stack(119, 116, 119, 127) \cdot lbf$

$$Z_{n\_strap} = \begin{pmatrix} 119 \\ 116 \\ 119 \\ 127 \end{pmatrix} \cdot lbf$$

Adjusted nail shear capacity [NDS-15, Sect. 11.3],

$Z'_{n\_strap} := Floor\left(Z_{n\_strap} \cdot 1.6, 1 \cdot lbf\right)$

$$Z'_{n\_strap} = \begin{pmatrix} 190 \\ 185 \\ 190 \\ 203 \end{pmatrix} \cdot lbf$$

Strap development length,

$L_{strap_k} := Ceil\left[s_{strap_k} \cdot \left(0.5 \cdot n_{strap_k} - 1\right), 3 \cdot in\right]$

$$L_{strap} = \begin{pmatrix} 12 \\ 36 \\ 48 \\ 63 \end{pmatrix} \cdot in \qquad L_{strap} = \begin{pmatrix} \text{"1' 0""} \\ \text{"3' 0""} \\ \text{"4' 0""} \\ \text{"5' 3""} \end{pmatrix} \cdot FIF \qquad 2 \cdot L_{strap} = \begin{pmatrix} \text{"2' 0""} \\ \text{"6' 0""} \\ \text{"8' 0""} \\ \text{"10' 6""} \end{pmatrix} \cdot FIF$$

Strap diaphragm development length,

$L_{diaph_k} := Ceil\left(\frac{F_{strap_k}}{180 \cdot plf}, 3 \cdot in\right)$

$$L_{diaph} = \begin{pmatrix} 168 \\ 306 \\ 435 \\ 615 \end{pmatrix} \cdot in \qquad L_{diaph} = \begin{pmatrix} \text{"14' 0""} \\ \text{"25' 6""} \\ \text{"36' 3""} \\ \text{"51' 3""} \end{pmatrix} \cdot FIF \qquad 2 \cdot L_{diaph} = \begin{pmatrix} \text{"28' 0""} \\ \text{"51' 0""} \\ \text{"72' 6""} \\ \text{"102' 6""} \end{pmatrix} \cdot FIF$$

Rim Joist Capacity:

Define area of single 1-1/4x11-7/8 rim joist: $A_{rj} := 14.84 \cdot in^2$

Define nominal tensile stress [Trus Joist]: $F_{t\_rj} := 1070 \cdot psi$

Adjusted tensile force capacity [NDS-15, Sect. 4],

$P'_{t\_rj} := Floor(A_{rj} \cdot F_{t\_rj} \cdot 1.6, 10 \cdot lbf)$ $\quad P'_{t\_rj} = 25400 \cdot lbf$

I-Joist Capacity:

Define area of single TJI 110 flange area:

$A_{IJ} := (1.75 \cdot in) \cdot (1.25 \cdot in)$ $\quad A_{IJ} = 2.19 \cdot in^2$

Define nominal tensile stress [#2 DF]: $F_{t\_IJ} := 575 \cdot psi$

Adjusted tensile force capacity [NDS-15, Sect. 4],

$P'_{t\_IJ} := Floor(A_{IJ} \cdot F_{t\_IJ} \cdot 1.6, 10 \cdot lbf)$ $\quad P'_{t\_IJ} = 2010 \cdot lbf$

Top Plate Capacity:

Define cross sectional area of single #2 DF 2x4 & 2x6 top plate: $A_{dtp} := \begin{pmatrix} 5.25 \\ 8.25 \end{pmatrix} \cdot in^2$

Define nominal tensile stress [NDS-15 Suppl., Table 4A] $F_{t\_dtp} := 575 \cdot psi$

Adjusted tensile force capacity [NDS-15, Sect. 4],

$P'_{t\_dtp} := Floor(1.6A_{dtp} \cdot F_{t\_dtp}, 10 \cdot lbf)$ $\quad P'_{t\_dtp} = \begin{pmatrix} 4820 \\ 7580 \end{pmatrix} \cdot lbf$

Double Top Plate Splice Nailing Capacity:

Define lap splice length: $L_{dtp} := 48.0 \cdot in$

Define nail edge distance: $d_{e\_dtp} := 1.0 \cdot in$

Define nominal 10d Common nail capacity [NDS-15, Table 12N]: $Z_{n\_dtp} := 141 \cdot lbf$

Adjusted nail capacity per [NDS-15, Sect. 11],

$Z'_{n\_dtp} := Floor(Z_{n\_dtp} \cdot 1.6 \cdot 1.1, 1 \cdot lbf)$ $\quad Z'_{n\_dtp} = 248 \cdot lbf$

Minimum number of nails for double top late transfer force,

$$n_{dtp} := Ceil\left(\frac{P'_{t\_dtp}}{2 \cdot Z'_{n\_dtp}}, 1\right) \qquad n_{dtp} = \begin{pmatrix} 10 \\ 16 \end{pmatrix}$$

Spacing of nails over 48" lap splice length,

$$s_{dtp} := Floor\left(\frac{L_{dtp} - 2 \cdot d_{e\_dtp}}{n_{dtp} - 1}, 1 \cdot in\right) \qquad s_{dtp} = \begin{pmatrix} 5.00 \\ 3.00 \end{pmatrix} \cdot in$$

Anchor Bolt Capacity:

Define anchor bolt diameter: $D_{ab} := \frac{5}{8} \cdot in$

Define nominal anchor bolt capacity [NDS-15, Table 12E]:

$$Z_{AB} := stack\left[860, linterp\left[\begin{pmatrix} 2.5 \\ 3.5 \end{pmatrix} \cdot in, \begin{pmatrix} 1070 \\ 1140 \end{pmatrix}, 3.0 \cdot in\right]\right] \cdot lbf \qquad Z_{AB} = \begin{pmatrix} 860.00 \\ 1105.00 \end{pmatrix} \cdot lbf$$

Adjusted anchor bolt capacity [NDS-15, Sect. 11.3],

$$Z'_{AB} := Floor(Z_{AB} \cdot 1.6, 10 \cdot lbf) \qquad Z'_{AB} = \begin{pmatrix} 1370 \\ 1760 \end{pmatrix} \cdot lbf$$

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Anchor Designer™
Software
Version 2.9.7376.0

Company:		Date:	8/13/2018
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Cast-In-Place Anchor Bolts
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 7.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 8.38
C_{min} (inch): 0.81
S_{min} (inch): 2.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 32.00
State: Cracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: Yes
Build-up grout pad: No

Recommended Anchor
Anchor Name: J- or L-Bolt - 5/8"Ø J- or L-Bolt, F1554 Gr. 36

Anchor Designer™
Software
Version 2.9.7376.0

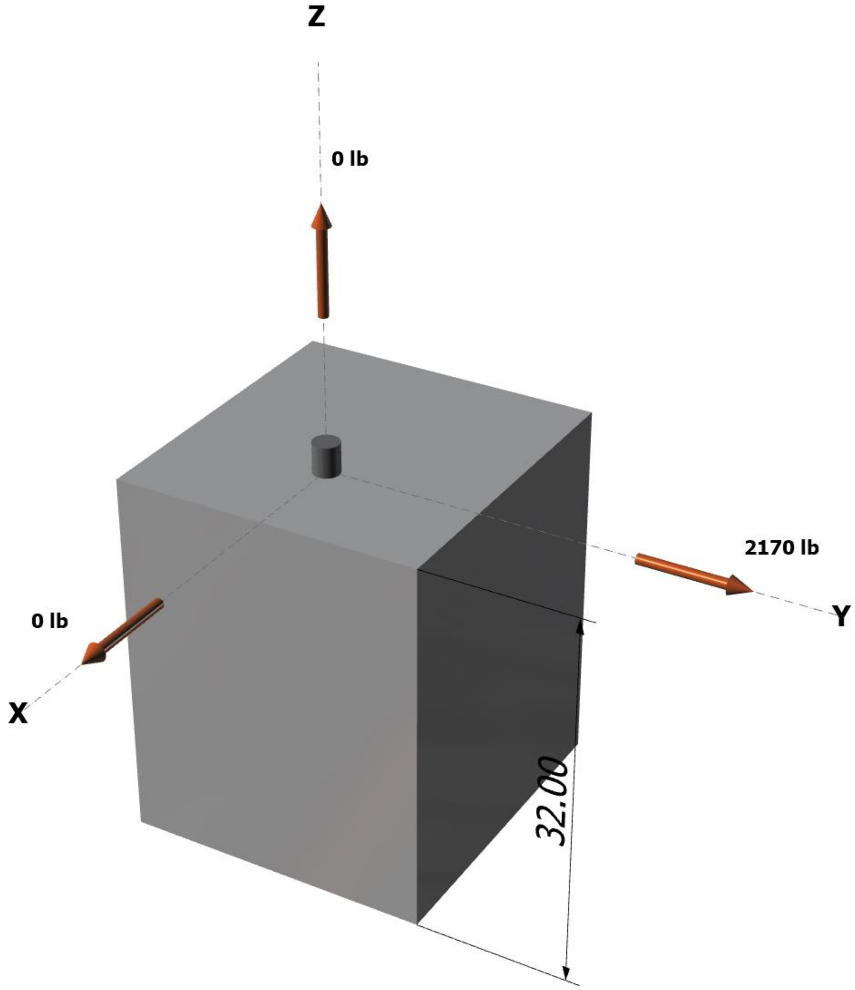
Company:		Date:	8/13/2018
Engineer:		Page:	2/5
Project:			
Address:			
Phone:			
E-mail:			

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (d) is satisfied
Ductility section for shear: 17.2.3.5.3 (c) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

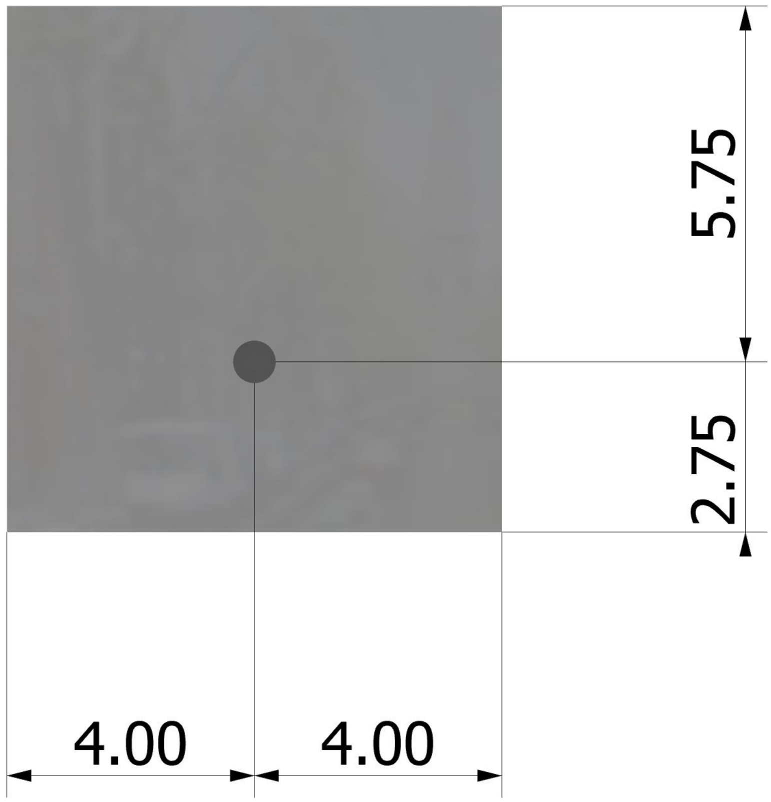
Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 2170

<Figure 1>



<Figure 2>



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Version 2.9.7376.0

Company:		Date:	8/13/2018
Engineer:		Page:	4/5
Project:			
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	0.0	2170.0	2170.0
Sum	0.0	0.0	2170.0	2170.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
7865	1.0	0.65	5112

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp} N_{cb} = \phi k_{cp}(A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	68.00	132.25	0.843	1.000	1.000	9866	0.70	5990

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	**2170**	**5112**	**0.42**	**Pass (Governs)**
Pryout	2170	5990	0.36	Pass

5/8"Ø J- or L-Bolt, F1554 Gr. 36 with hef = 7.000 inch meets the selected design criteria.

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Version 2.9.7376.0

Company:		Date:	8/13/2018
Engineer:		Page:	5/5
Project:			
Address:			
Phone:			
E-mail:			

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

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Software
Version 2.9.7376.0

Company:		Date:	8/13/2018
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description: Post Installed Anchor Bolts
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.625
Nominal Embedment depth (inch): 4.000
Effective Embedment depth, h_{ef} (inch): 2.970
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 6.00
c_{ac} (inch): 4.50
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Recommended Anchor
Anchor Name: Titen HD® - 5/8"Ø Titen HD (THDB model), hnom:4" (102mm)
Code Report: ICC-ES ESR-2713

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Anchor Designer™
Software
Version 2.9.7376.0

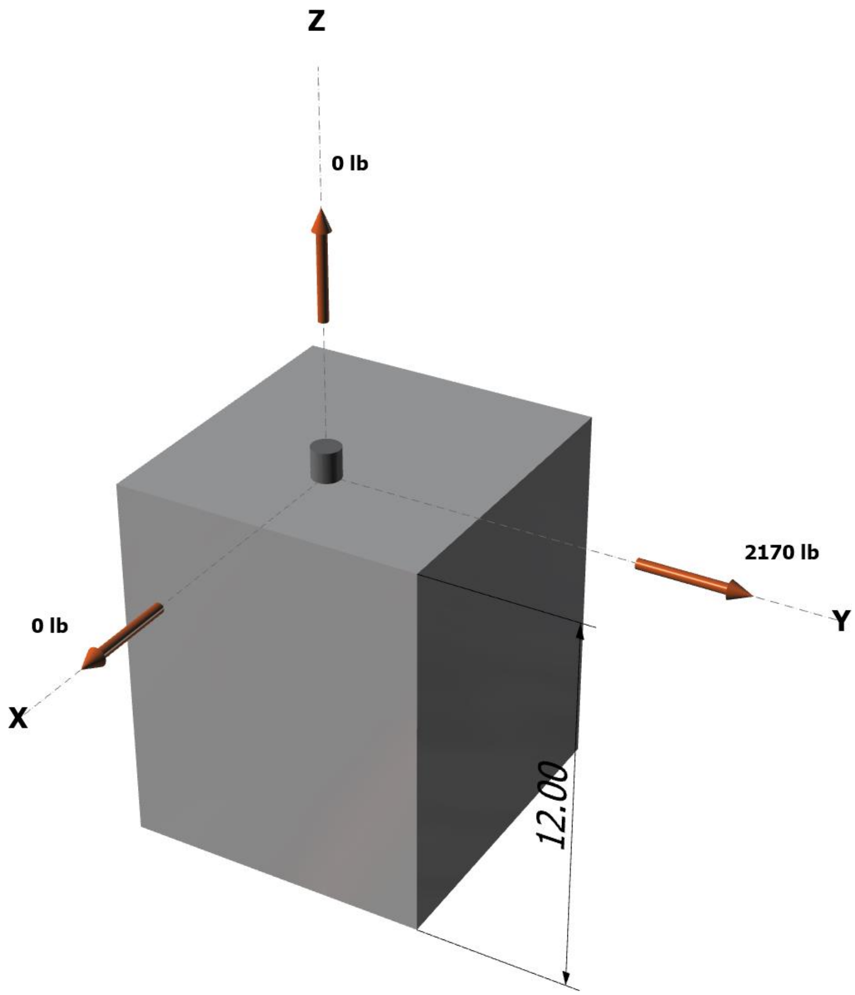
Company:		Date:	8/13/2018
Engineer:		Page:	2/5
Project:			
Address:			
Phone:			
E-mail:			

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (d) is satisfied
Ductility section for shear: 17.2.3.5.3 (c) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 2170

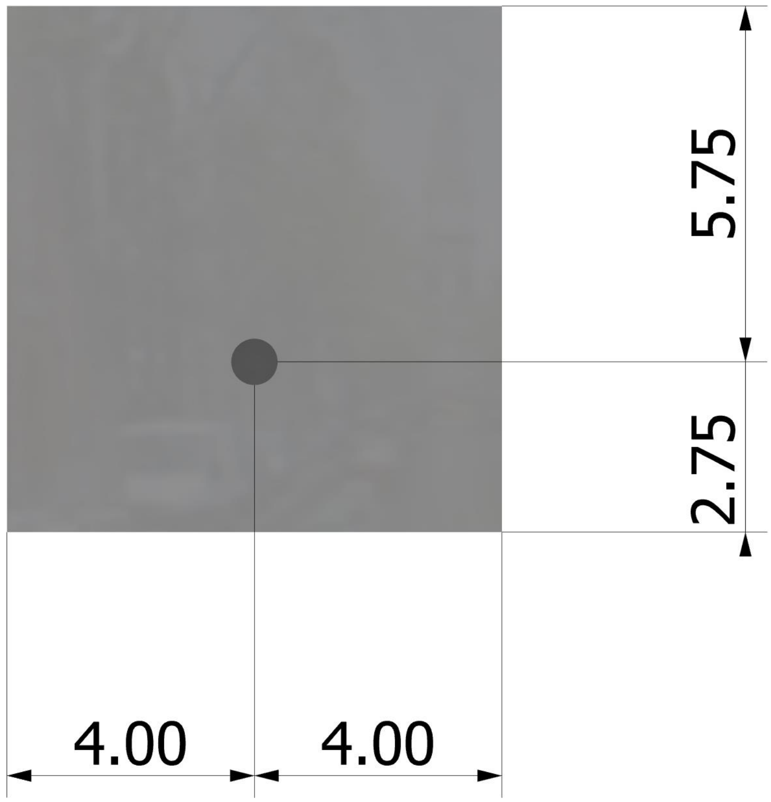
<Figure 1>



Anchor Designer™
Software
Version 2.9.7376.0

Company:		Date:	8/13/2018
Engineer:		Page:	3/5
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>



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Anchor Designer™
Software
Version 2.9.7376.0

Company:		Date:	8/13/2018
Engineer:		Page:	4/5
Project:			
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	0.0	2170.0	2170.0
Sum	0.0	0.0	2170.0	2170.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
8000	1.0	0.60	4800

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp} N_{cb} = \phi k_{cp}(A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	54.00	64.00	0.906	1.000	1.000	4055	0.70	4341

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	2170	4800	0.45	Pass
Pryout	**2170**	**4341**	**0.50**	**Pass (Governs)**

5/8"Ø Titen HD (THDB model), hnom:4" (102mm) meets the selected design criteria.

Anchor Designer™
Software
Version 2.9.7376.0

Company:		Date:	8/13/2018
Engineer:		Page:	5/5
Project:			
Address:			
Phone:			
E-mail:			

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

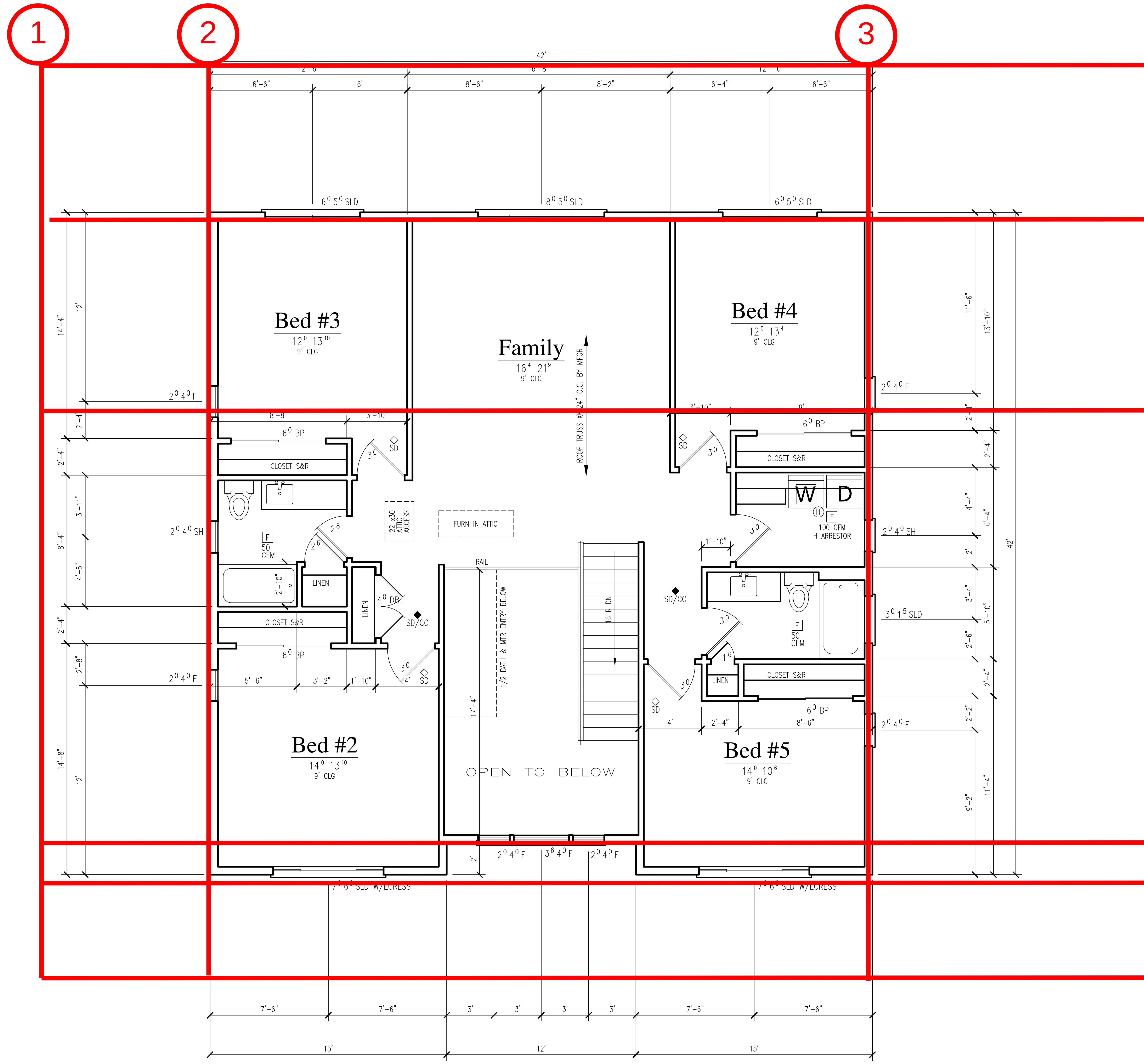
- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

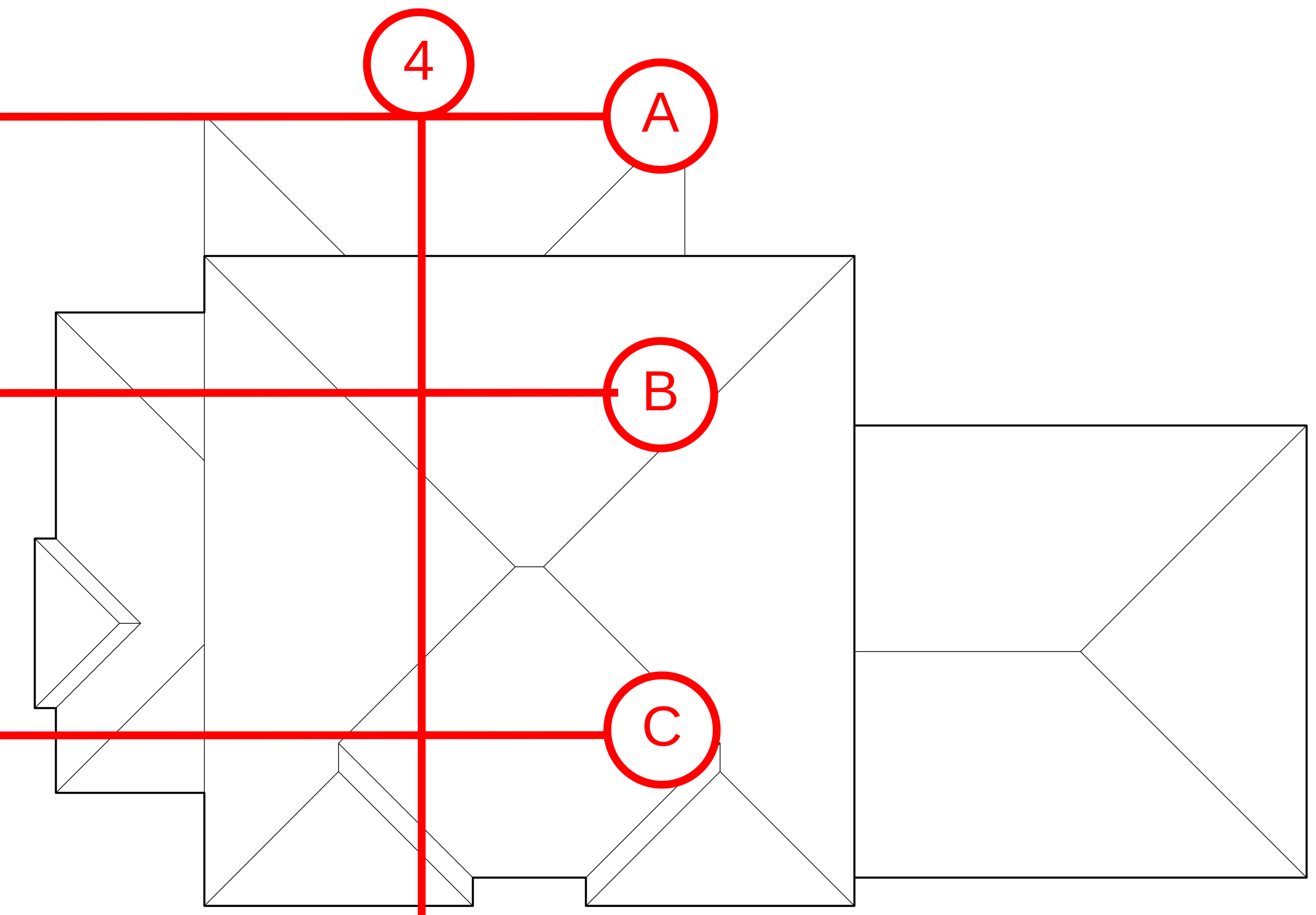
- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer’s product literature for hole cleaning and installation instructions.



Upper Floor Plan



Roof Truss Providers
PLEASE PROVIDE YOUR PLANS TO THIS OFFICE VIA EMAIL (mwcm01@gmail.com).

NOTE:
TRUSS MFGR TO VERIFY BEARING POINTS. IF NEW BEARING IS NEEDED, MFGR MUST INFORM THIS DESIGNER (509) 528-2884

Roof Plan

SCALE: 1/8" = 1'-0"

Engineering Required

ALL POSTS, SHEAR WALLS, BEAMS, FOUNDATION, FOOTINGS, & OTHER STRUCTURAL MEMBERS TO BE FULLY ENGINEERED AS NEEDED.

ALL ENGINEERING DOCUMENTATION, FLOORING, AND ROOF PACKAGES SUPERCEDED THESE DRAWINGS.

MCLEOD
HOME DESIGNS
www.mcleodhomedesigns.com
1900 Fowler Street, Suite F
Richland, WA 99352
509-528-2884

Altman's East Lot
APN 3020459151

Building Information:

Main Floor SQ FT:	2055
Second Floor SQ FT:	1527
Basement SQ FT:	1304
TOTAL SQ FT:	4886
Unfinished SQ FT:	723
Garage SQ FT:	896
Covered Area SQ FT:	0

Upper Floor Plan

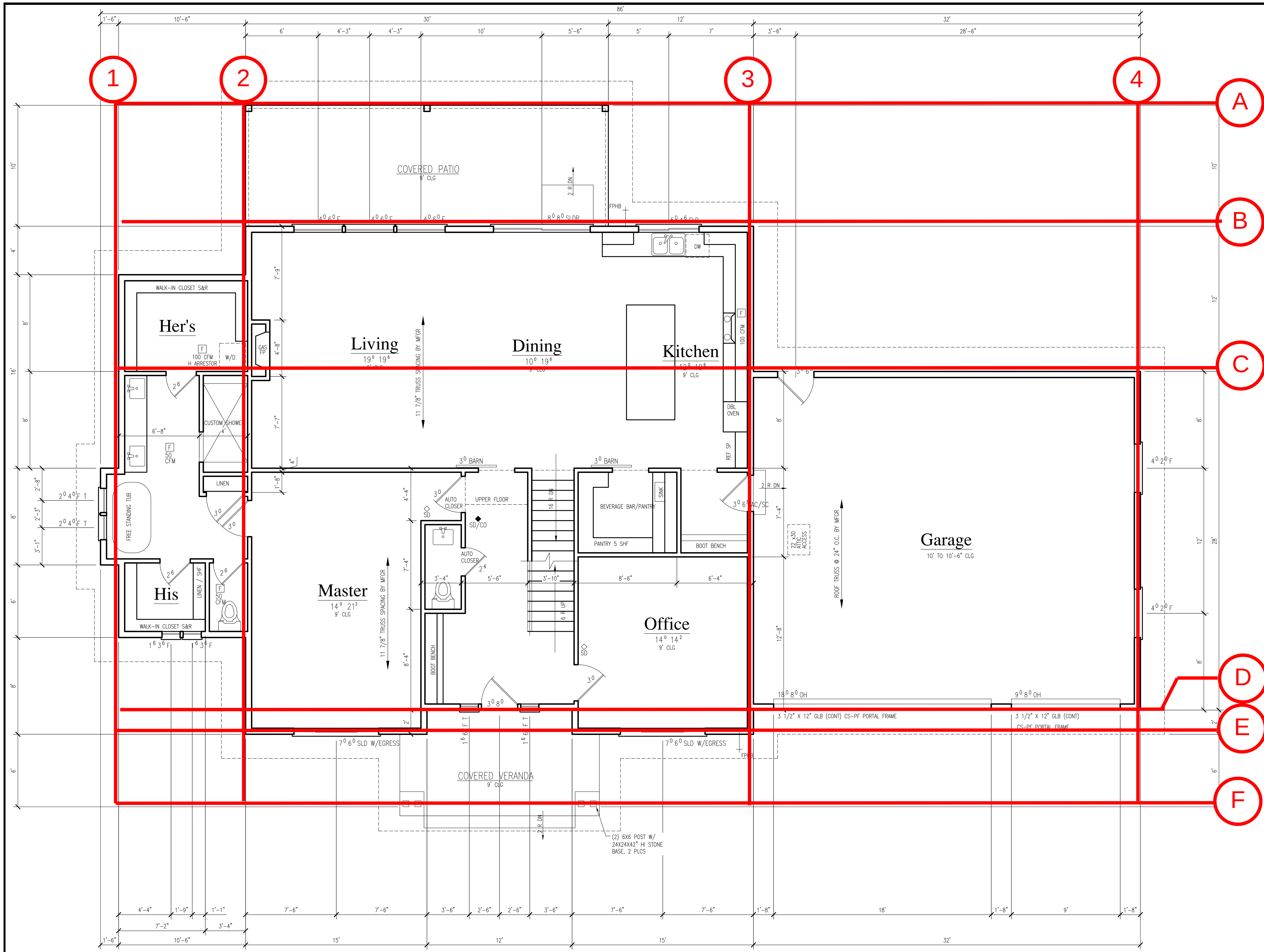
Altman's East Lot
4886 SF 2-Story

BUILDING ADDRESS: N/A

DWG	t4886x0a east lot.dwg
Date	4/8/20 5:30:PM
By:	Mark McLeod
Scale	1/4" = 1'
Approved	

3b

REV: 0 4/8/20



Main Floor Plan

ROOF TRUSS PROVIDERS

PLEASE PROVIDE YOUR PLANS TO THIS OFFICE VIA EMAIL (mwcmo1@gmail.com).

FLOOR TRUSS PROVIDERS

PLEASE PROVIDE YOUR PLANS TO THIS OFFICE VIA EMAIL (mwcmo1@gmail.com).

Braced Wall Schedule

CONTINOUS SHEATHING CONDITION (SEISMIC D , WIND 85)

ABW	PER DETAIL SH 4 (IF NEEDED)
CS-PF	PER DETAIL SH 4
CS-WSP	8d COMMON – 6" EDGE 12" FIELD
GB	1 3/8 (13 GA) GB SCREW – 7" EDGE 7" FIELD

LEGEND

SYMBOL	DESCRIPTION
H	HAMMER ARRESTOR
F	FAN VENTED TO EXTERIOR
SD SD/CO	SMOKE / CARBON MONOXIDE DETECTOR (NOTE 15)
FPHB	FROST PROOF HOSE BIB
SC/AC	SOLID CORE / AUTO CLOSER
T	SAFETY OR TEMPERED GLASS

ENERGY CREDITS

#	DESCRIPTION	CREDITS
2a	AIR LEAKAGE CONTROL AND EFFICIENT VENTILATION 2a Compliance based on on R402.4.1.2: Reduce the test air leakage to 3.0 air changes per hour maximum and All whole house ventilation requirements as determined by section M1507.3 of the international Residential Code shall be met with a high efficiency fan (maximum 0.35 including an ECM motor are allowed, provided that they are controlled to operate at low speed in ventilation only mode.	0.5
3b	HIGH EFFICIENCY HVAC EQUIPMENT 3b: Air-source heat pump with minimum HSPF of 9.0	1
5a	EFFICIENT WATER HEATING 5a. All showerhead and kitchen sink fauscets installed in the house shall be rated at 1.75 GPM or less. All other lavatory faucets shall be rated in 1.0 GPM or less.	0.5
5c	EFFICIENT WATER HEATING 5c. Electric heat pump water heater with a minimum EF of 2.0	1.5
	TOTAL	3.5

BUILDING INFORMATION

MAIN FLOOR SF:	2055
SECOND FLOOR SF:	1527
BASEMENT FLOOR SF	1304
TOTAL CONDITIONED SF:	4886
TOTAL UNCONDITIONED SF:	
UNFINISHED SF:	723
GARAGE SF:	896
COVERED AREA SF:	0

Builders Responsibility

THESE DRAWINGS ARE IN PART DIAGRAMMICAL AND DO NOT SHOW IN DETAIL HOW WORKMANSHIP, MATERIAL AND INSTALLATION OF MATERIAL ARE TO BE BROUGHT TOGETHER TO COMPLETE THE WHOLE STRUCTURE. IT IS THE RESPONSIBILITY OF THE BUILDER TO BUILD THE STRUCTURE TO COMPLY WITH ALL APPLICABLE FEDERAL, STATE, COUNTY, CITY CODES AS THEY APPLY TO EACH COMPONENT.

General Notes:

1. PROVIDE 30" RANGE AND HOOD W/ 100 CFM FAN VENTED TO EXTERIOR.
2. PROVIDE WATER RESISTANT GYPSUM BOARD IN TUB OR SHOWER RECESS.
3. PROVIDE 50 GALLON (MIN) WATER HEATER W/ ASME RATED TEMPERATURE AND PRESSURE RELIEF VALVE W/ 3/4" COPPER DRIP
4. BUILDER TO VERIFY ALL ASPECTS AND DIMENSIONS OF THESE DRAWINGS. ANY PROBLEMS WITH THESE DRAWINGS ARE TO BE BROUGHT TO THE IMMEDIATE ATTENTION OF THIS DESIGNER, MARK McLEOD (509) 528-2884.
5. DO NOT SCALE THESE DRAWINGS.
6. EXTERIOR WALLS OF HOUSE ARE TO BE 2 X 6, UNLESS OTHERWISE SPECIFIED.
7. INTERIOR WALL OF HOUSE ARE TO BE 2 X 4, UNLESS OTHERWISE SPECIFIED.
8. EXTERIOR WALLS OF GARAGE ARE TO BE 2 X 6, UNLESS OTHERWISE SPECIFIED.
9. HOUSE INSULATION AS NOTED BELOW:
   EXTERIOR WALLS = R-21 BATT INSULATION
   EXTERIOR CEILING = R-49 BLOWN INSULATION
   EXTERIOR FLOORS = R-30 BATT INSULATION
10. ALL FINISH GRADE WORK SHALL BE NO CLOSER THAN 6" TO FINISH SIDING.
11. ALL HEADER MATERIAL FOR BEARING WALLS TO BE 3 ½" x 9" G.L. HEADER STOCK UNLESS OTHERWISE NOTED.
12. DIMENSIONING FORMAT AS FOLLOWS:
   OVER ALL DIMENSIONS SHALL BE FROM EXTERIOR TO EXTERIOR OF BUILDING.
   BREAKS OR JOGS IN BUILDING SHALL BE DIMENSIONED FROM EXTERIOR OF BUILDING.
   INTERIOR WALL DIMENSIONS:
   VERTICALLY SHALL BE TAKEN FROM THE TOP SIDE OF THE WALL.
   HORIZONTAL WALLS SHALL BE TAKEN FROM THE LEFT SIDE OF WALL.
   OPENINGS SHALL BE DIMENSIONED FROM CENTER (EXCEPT GARAGE OPENINGS)
13. ANGULAR WALLS ARE ON A 45 DEGREE ANGLE, UNLESS OTHERWISE NOTED.
14. PROVIDE GAS FIREPLACE PER IRC 302.13 (per plan)
15. NOTE ALL SMOKE DETECTORS ARE ELECTRICALLY HARDWIRED.
16. ALL WINDOWS ARE TO BE .3 U FACTOR MAX.

Engineering Required

ALL POSTS, SHEAR WALLS, BEAMS, FOUNDATION, FOOTINGS, & OTHER STRUCTURAL MEMBERS TO BE FULLY ENGINEERED AS NEEDED.

ALL ENGINEERING DOCUMENTATION, FLOORING, AND ROOF PACKAGES SUPERCEDED THESE DRAWINGS.

MCLEOD HOME DESIGNS
www.mcleodhomedesigns.com
1900 Fowler Street, Suite F
Richland, WA 99352
509-528-2884

Altman's East Lot APN 3020459151

Building Information:

Main Floor SQ FT:	2055
Second Floor SQ FT:	1527
Basement SQ FT:	1304
TOTAL SQ FT:	4886
Unfinished SQ FT:	723
Garage SQ FT:	896
Covered Area SQ FT:	0

Main Floor Plans

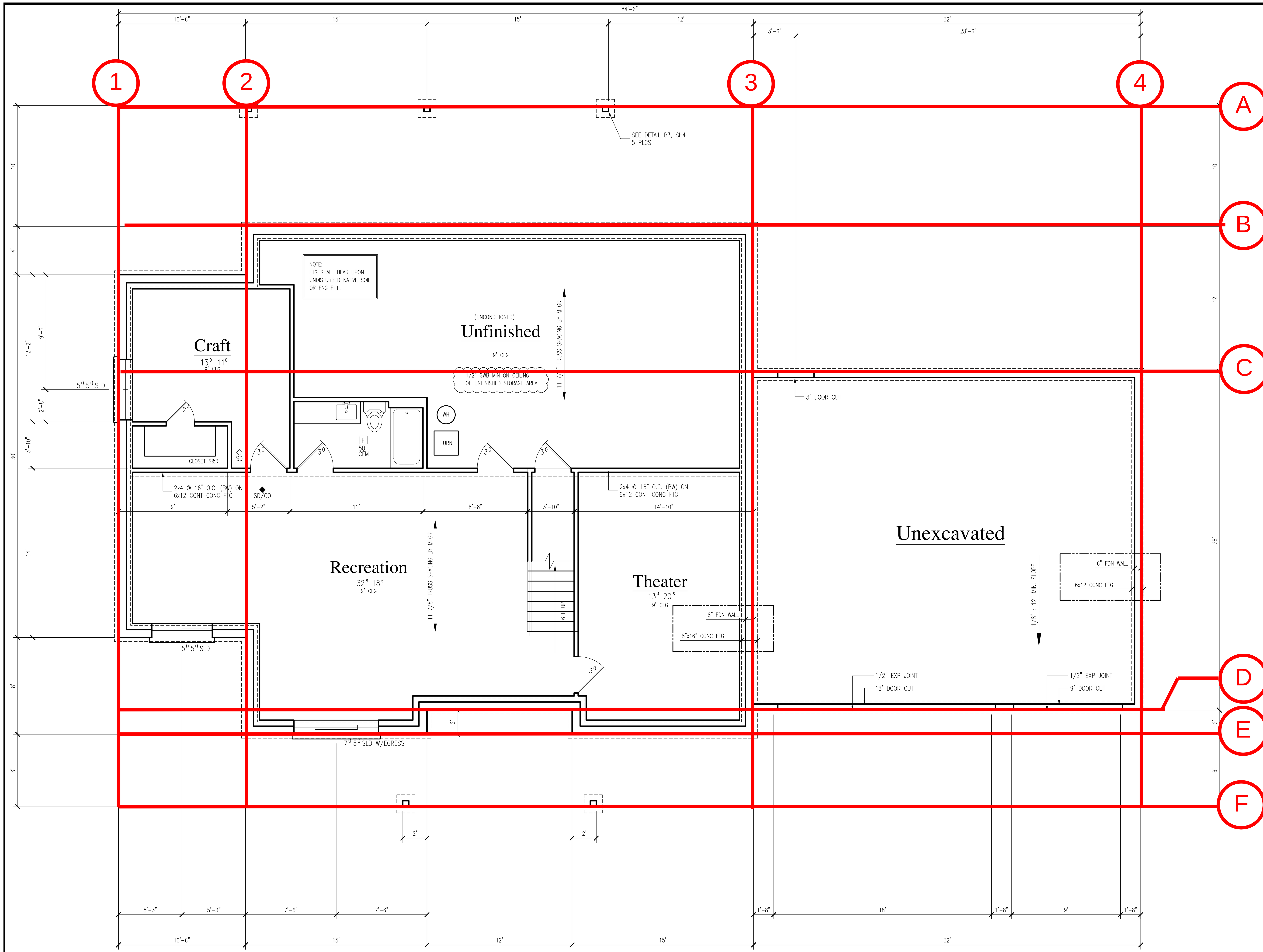
Altman's East Lot
4886 SF 2-Story

BUILDING ADDRESS: N/A

DWG	t4886x0a east lot.dwg
Date	4/8/20 5:30:PM
By:	Mark McLeod
Scale	1/4" = 1'
Approved	

3a

REV: 0 4/8/20



Basement Floor Plan
Footing & Foundation Plan

Engineering Required

ALL POSTS, SHEAR WALLS, BEAMS, FOUNDATION, FOOTINGS, & OTHER STRUCTURAL MEMBERS TO BE FULLY ENGINEERED AS NEEDED.

ALL ENGINEERING DOCUMENTATION, FLOORING, AND ROOF PACKAGES SUPERCEDED THESE DRAWINGS.

MCLEOD
HOME DESIGNS

www.mcleodhomedesigns.com
1900 Fowler Street, Suite F
Richland, WA 99352
509-528-2884

Altman's East Lot
APN 302045915 1

Building Information:

Main Floor SQ FT:	2055
Second Floor SQ FT:	1527
Basement SQ FT:	1304
TOTAL SQ FT:	4886
Unfinished SQ FT:	723
Garage SQ FT:	896
Covered Area SQ FT:	0

Ftg / Fdn / Roof Plan

Altman's East Lot
4886 SF 2-Story

BUILDING ADDRESS: N/A

DWG	t4886x0a east lot.dwg
Date	4/8/20 5:30:PM
By:	Mark McLeod
Scale	1/4" = 1'
Approved	

2

REV: 0 | 4/8/20

SERVICE DIAPHRAGM / SHEAR WALL / OVERTURNING LOADS

DIAPHRAGM LOADS

S = Simply Supported; C = Cantilever

Level	Bay	Load			L (ft)	W (ft)	L/W	w_{dist} (plf)	F_d (lbf)	Supp.	V_d (lbf)	v_d (plf)	M_d (ft-lbf)	T_d=C_d (lbf)	v_{Dcap} (plf)	INT_D	Case	Remarks
Perpendicular to Narrow Face																		
Roof	B-E	**W**			42.000	42.000	1.000	311.9	13,100	S	6,550	156	68,775	1,638	180	0.87	**3**	
					42.000													
2nd Floor	A-B	**W**			10.000	30.000	0.333	248.5	14,600	C	2,485	83	12,426	415	180	0.46	**3**	
2nd Floor	C-D/E	**W**			28.000	32.000	0.875	248.5	14,600	S	3,479	109	24,354	762	180	0.60	**3**	
2nd Floor	B-D/E	**W**			42.000	42.000	1.000	248.5	14,600	S	5,219	124	54,797	1,305	215	0.58	**3**	
2nd Floor	E-F	**W**			6.750	16.500	0.409	248.5	14,600	C	1,677	102	5,661	344	180	0.56	**3**	
					58.750													
Main Floor	B-E	**W**			42.000	42.000	1.000	245.2	10,300	S	5,150	123	54,075	1,288	215	0.57	**3**	
					42.000													
Level	Bay	Load			L (ft)	W (ft)	L/W	w_{dist} (plf)	F_d (lbf)	Supp.	V_d (lbf)	v_d (plf)	M_d (ft-lbf)	T_d=C_d (lbf)	v_{Dcap} (plf)	INT_D	Case	Remarks
Perpendicular to Wide Face																		
Roof	2-3	**W**			42.000	42.000	1.000	319.0	13,400	S	6,700	160	70,350	1,675	180	0.89	**3**	
					42.000													
Upper Floor	1-2	**W**			10.500	30.000	0.350	228.4	19,300	S	1,199	40	3,148	105	180	0.22	**3**	
Upper Floor	2-3	**W**			42.000	42.000	1.000	228.4	19,300	S	4,796	114	50,363	1,200	215	0.53	**3**	
Upper Floor	4-5	**W**			32.000	28.000	1.143	228.4	19,300	S	3,654	131	29,236	1,045	180	0.73	**3**	
					84.500													
Main Floor	1-2	**W**			10.500	30.000	0.350	293.3	15,400	S	1,540	51	4,043	135	215	0.24	**3**	
Main Floor	2-3	**W**			42.000	42.000	1.000	293.3	15,400	S	6,160	147	64,680	1,540	215	0.68	**3**	
					52.500													

S = Simply Supported; C = Cantilever

Level	Bay	Load	EL (psf)	A_{tr} (ft^2)	L (ft)	W (ft)	L/W	w_{dist} (plf)	F_d (lbf)	Supp.	V_d (lbf)	v_d (plf)	M_d (ft-lbf)	T_d=C_d (lbf)	v_{Dcap} (plf)	INT_D	Case	Remarks
Perpendicular to Narrow Face																		
Roof	B-E	**E**	3.96	2,116	42.000	42.000	1.000	199.5	8,379	S	4,190	100	43,992	1,048	180	0.55	**3**	
				2,116	42.000				8,379									
2nd Floor	A-B	**E**	4.29	408	10.000	30.000	0.333	175.0	1,750	C	1,750	58	8,752	292	180	0.32	**3**	
2nd Floor	C-D/E	**E**	4.29	1,088	28.000	32.000	0.875	166.7	4,668	S	2,334	73	16,336	511	180	0.41	**3**	
2nd Floor	B-D/E	**E**	4.29	2,207	42.000	42.000	1.000	225.4	9,468	S	4,734	113	49,707	1,184	215	0.52	**3**	
2nd Floor	E-F	**E**	4.29	160	6.750	16.500	0.409	101.7	686	C	686	42	2,317	141	180	0.23	**3**	
				3,863	58.750				16,572									
Main Floor	B-E	**E**	5.43	2,055	42.000	42.000	1.000	265.7	11,159	S	5,579	133	58,583	1,395	215	0.62	**3**	
				2,055	42.000				11,159									
Level	Bay	Load	EL (psf)	A_{tr} (ft^2)	L (ft)	W (ft)	L/W	w_{dist} (plf)	F_d (lbf)	Supp.	V_d (lbf)	v_d (plf)	M_d (ft-lbf)	T_d=C_d (lbf)	v_{Dcap} (plf)	INT_D	Case	Remarks
Perpendicular to Wide Face																		
Roof	2-3	**E**	3.96	2,116	42.000	42.000	1.000	199.5	8,379	S	4,190	100	43,992	1,048	180	0.55	**3**	
				2,116	42.000				8,379									
2nd Floor	1-2	**E**	4.29	443	10.500	30.000	0.350	181.0	1,900	S	950	32	2,494	84	180	0.18	**3**	
2nd Floor	2-3	**E**	4.29	2,332	42.000	42.000	1.000	238.2	10,004	S	5,002	119	52,522	1,251	215	0.55	**3**	
2nd Floor	3-4	**E**	4.29	1,088	32.000	28.000	1.143	145.9	4,668	S	2,334	83	18,670	667	180	0.46	**3**	
				3,863	84.500				16,572									
Main Floor	1-2	**E**	5.43	443	10.500	30.000	0.350	229.1	2,405	S	1,203	40	3,157	106	215	0.19	**3**	
Main Floor	2-3	**E**	5.43	1,612	42.000	42.000	1.000	208.4	8,753	S	4,377	104	45,954	1,095	215	0.48	**3**	
				2,055	52.500				11,159									

WALL LINE LOADS

Level	Wall	Load	F_w (lbf)	F_{prev} (lbf)	F_{w_tot} (lbf)	L_d (ft)	v_{w_d} (plf)	b_{s_tot} (ft)	v_{w_sw} (plf)	γ_p	ρ_o*F_e (lbf)	F_{prev} (lbf)	F_{sw_tot} (lbf)	Remarks
Perpendicular to Narrow Face														
2nd Floor	B	**W**	6,550	0	6,550	42.000	156	16.500	397			0	6,550	
2nd Floor	E	**W**	6,550	0	6,550	42.000	156	16.000	409			0	6,550	
			13,100										13,100	
2nd Floor	B	**E**	4,190	0	4,190	42.000	100	16.500	254	1.000	5,447	0	5,447	
2nd Floor	E	**E**	4,190	0	4,190	42.000	100	16.000	262	1.000	5,447	0	5,447	
			8,379								10,893		10,893	
Main Floor	B	**W**	7,704	0	7,704	42.000	183	16.333	472			6,550	14,254	
Main Floor	C	**W**	3,479	0	3,479	32.000	109	27.000	129			0	3,479	
Main Floor	D/E	**W**	10,375	0	10,375	74.000	140	16.000	648			6,550	16,925	
			14,600										34,658	
Main Floor	B	**E**	6,484	0	6,484	42.000	154	16.333	397	1.128	7,473	5,447	12,920	
Main Floor	C	**E**	2,334	0	2,334	32.000	73	27.000	86	1.128	2,690	0	2,690	
Main Floor	D/E	**E**	7,754	0	7,754	74.000	105	16.000	485	1.128	8,937	5,447	14,383	
			16,572								19,099		29,992	
Base-ment	B	**W**	5,150	0	5,150	42.000	123	42.000	123			14,254	19,404	***RW=Retaining Wall***
Base-ment	E	**W**	5,150	0	5,150	42.000	123	42.000	123			16,925	22,075	***RW=Retaining Wall***
			10,300										41,479	
Base-ment	B	**E**	5,579	0	5,579	42.000	133	42.000	133	2.436	2,977	12,920	15,897	***RW=Retaining Wall***
Base-ment	E	**E**	5,579	0	5,579	42.000	133	42.000	133	2.436	2,977	14,383	17,361	***RW=Retaining Wall***
			11,159								5,955		33,258	

WALL LINE LOADS

Level	Wall	Load	F_w (lbf)	F_{prev} (lbf)	F_{w_tot} (lbf)	L_d (ft)	v_{w_d} (plf)	b_{s_tot} (ft)	v_{w_sw} (plf)	γ_p	ρ_o*F_e (lbf)	F_{prev} (lbf)	F_{sw_tot} (lbf)	Remarks
Perpendicular to Wide Face														
2nd Floor	2	**W**	6,700	0	6,700	40.000	168	36.000	186			0	6,700	
2nd Floor	3	**W**	6,700	0	6,700	40.000	168	25.667	261			0	6,700	
			13,400										13,400	
2nd Floor	2	**E**	4,190	0	4,190	40.000	105	36.000	116	1.000	5,447	0	5,447	
2nd Floor	3	**E**	4,190	0	4,190	40.000	105	25.667	163	1.000	5,447	0	5,447	
			8,379								10,893		10,893	
Main Floor	1	**W**	1,199	0	1,199	30.000	40	22.000	55			0	1,199	
Main Floor	2	**W**	5,996	0	5,996	42.000	143	12.000	500			6,700	12,696	
Main Floor	3	**W**	8,451	0	8,451	42.000	201	39.000	217			6,700	15,151	
Main Floor	4	**W**	3,654	0	3,654	28.000	131	20.000	183			0	3,654	
			19,300										32,700	
Main Floor	1	**E**	950	0	950	30.000	32	22.000	43	1.128	1,095	0	1,095	
Main Floor	2	**E**	5,952	0	5,952	42.000	142	12.000	496	1.128	6,860	5,447	12,307	
Main Floor	3	**E**	7,336	0	7,336	42.000	175	39.000	188	1.128	8,454	5,447	13,901	
Main Floor	4	**E**	2,334	0	2,334	28.000	83	20.000	117	1.128	2,690	0	2,690	
			16,572								19,099		29,992	
Base-ment	1	**W**	1,540	0	1,540	30.000	51	24.750	62			1,199	2,739	
Base-ment	2	**W**	7,700	0	7,700	42.000	183	12.000	642			12,696	20,396	
Base-ment	3	**W**	6,160	0	6,160	42.000	147	42.000	147			15,151	21,311	
			15,400										44,446	
Base-ment	1	**E**	1,203	0	1,203	30.000	40	30.000	40	2.436	642	1,095	1,737	***RW=Retaining Wall***
Base-ment	2	**E**	5,579	0	5,579	42.000	133	42.000	133	2.436	2,977	12,307	15,284	***RW=Retaining Wall***
Base-ment	3	**E**	4,377	0	4,377	42.000	104	42.000	104	2.436	2,336	13,901	16,237	***RW=Retaining Wall***
			11,159								5,955		33,258	

SHEAR WALL / OVERTURNING LOADS

Indicates increased shear wall loads due to wall height to width ratio greater than 2:1, per SDPWS-15, Sect. 4.3.4.2. ARF = 1.25-0.125 H / b_s .

Level	Wall	Wall Segments	b_{s_min} (ft)	H/b_s	ARF	b_{s_tot} (ft)	ΣF_w (lbf)	v_{sw} (plf)	Wall ID	v_{SWcap} (plf)	INT_{SW}	v_{otw} (plf)	H (ft)	F_{g_ot} (lbf)	Remarks
Perpendicular to Narrow Face															
2nd Floor	B	(2) 3'-6" + (2) 7'-6"	3.500	2.60	1.298	16.500	6,550	515	4	640	0.80	397	9.083	3,606	
2nd Floor	E	(4) 4'-0"	4.000	2.27	1.135	16.000	6,550	465	3	490	0.95	409	9.083	3,718	
Main Floor	B	3'-10" + (2) 4'-0" + 3'-6"	3.500	2.60	1.298	16.333	14,254	1,132	8	1,280	0.88	873	9.083	7,927	
Main Floor	C	27'-0"	27.000	0.34	1.000	27.000	3,479	129	1	260	0.50	129	9.083	1,170	
Main Floor	E	(4) 4'-0"	4.000	2.27	1.135	16.000	16,925	1,201	8	1,280	0.94	1,058	9.083	9,609	
Lower Floor	B	RW				42.000	19,404							0	
Lower Floor	E	RW				42.000	22,075							0	

SHEAR WALL / OVERTURNING LOADS

Indicates increased shear wall loads due to wall height to width ratio greater than 2:1, per SDPWS-15, Sect. 4.3.3.4.1 ARF = 2b_s/h.

Level	Wall	Wall Segments	b_{s_min} (ft)	H/b_s	ARF	b_{s_tot} (ft)	ΣF_w (lbf)	v_{sw} (plf)	Wall ID	v_{SWcap} (plf)	INT_{SW}	v_{otw} (plf)	H (ft)	F_{g_ot} (lbf)	Remarks
Perpendicular to Wide Face															
2nd Floor	2	11'-0" + 6'-7" + 7'-5" + 11'-0"	6.583	1.38	1.000	36.000	6,700	186	1	260	0.72	186	9.083	1,691	
2nd Floor	3	10'-6" + 7'-0" + 8'-2"	7.000	1.30	1.000	25.667	6,700	261	2	380	0.69	261	9.083	2,371	
Main Floor	1	16'-0" + 6'-0"	6.000	1.51	1.000	22.000	1,199	55	1	260	0.21	55	9.083	495	
Main Floor	2	4'-0" + 8'-0"	4.000	2.27	1.135	12.000	12,696	1,201	8	1,280	0.94	1,058	9.083	9,610	
Main Floor	3	20'-7" + 18'-5"	18.417	0.49	1.000	39.000	15,151	388	3	490	0.79	388	9.083	3,529	
Main Floor	4	6'-0" + 8'-0" + 6'-0"	6.000	1.51	1.000	20.000	3,654	183	1	260	0.70	183	9.083	1,660	
Base-ment	2	RW				24.750	2,739								
Base-ment	3	RW				12.000	20,396								
Base-ment	4	RW				42.000	21,311								

OVERTURNING DESIGN

Level	Wall	bs (ft)	DL_{pl} (psf)	b_{pl} (ft)	w_{pl} (plf)	DL_w (psf)	w_w (plf)	w_{tot} (plf)	P_{dl} (lbf)	P_r (lbf)	F_{g_ot} (lbf)	F_{n_ot} (lbf)	F_{pr_ot} (lbf)	F_{tot_ot} (lbf)	Strap/ Holdown	F_{HDcap} (lbf)	INT_{AB}	Remarks
Perpendicular to Narrow Face																		
2nd Floor	B	3.500	17	4.000	68	12	109	177	310	186	3,606	3,420	0	3,420	MST48	4,205	0.81	
		7.500	17	23.000	391	12	109	500	1,875	1,125	3,606	2,481	0	2,481	MST48	4,205	0.59	
														3,420	HDU4	4,565	0.75	***Max. Total Uplift Down to Fndn.***
2nd Floor	E	4.000	17	4.000	68	12	109	177	354	212	3,718	3,506	0	3,506	MST48	4,205	0.83	
Main Floor	B	3.833	15	14.750	221	12	109	330	633	380	7,927	7,547	3,420	10,967	HHDQ11	11,810	0.93	***Stacked Side***
											7,927	7,927	0	7,927	HDQ8	9,230	0.86	***Non-Stacked Side***
		4.000	15	14.750	221	12	109	330	661	396	7,927	7,531	0	7,531	HDQ8	9,230	0.82	
		3.500	15	14.750	221	12	109	330	578	347	7,927	7,580	0	7,580	HDQ8	9,230	0.82	***Non-Stacked Side***
											7,927	7,927	2,481	10,408	HHDQ11	11,810	0.88	***Stacked Side***
Main Floor	C	27.000	17	16.000	272	12	109	381	5,144	3,086	1,170	-1,916	0	-1,916	N/A	500	0.00	
Main Floor	E	4.000	15	10.583	159	12	109	268	536	321	9,609	9,287	3,506	12,793	HDU14	14,445	0.89	

OVERTURNING DESIGN

Level	Wall	bs (ft)	DL_{pl} (psf)	b_{pl} (ft)	w_{pl} (plf)	DL_w (psf)	w_w (plf)	w_{tot} (plf)	P_{dl} (lbf)	P_r (lbf)	F_{g_ot} (lbf)	F_{n_ot} (lbf)	F_{pr_ot} (lbf)	F_{tot_ot} (lbf)	Strap/ Holdown	F_{HDcap} (lbf)	INT_{AB}	Remarks
Perpendicular to Wide Face																		
2nd Floor	2	11.000	17	5.000	85	12	109	194	1,067	640	1,691	1,050	0	1,050	MST37	2,710	0.39	
		6.583	17	5.000	85	12	109	194	639	383	2,371	1,988	0	1,988	MST37	2,710	0.73	
		7.417	17	5.000	85	12	109	194	719	432	2,371	1,939	0	1,939	MST37	2,710	0.72	
		11.000	17	5.000	85	12	109	194	1,067	640	2,371	1,731	0	1,731	MST37	2,710	0.64	
														1,988	HDU2	3,075	0.65	***Max. Total Uplift Down to Fndn.***
2nd Floor	3	10.500	17	5.000	85	12	109	194	1,019	611	2,371	1,760	0	1,760	MST37	2,710	0.65	
		7.000	17	5.000	85	12	109	194	679	407	2,371	1,964	0	1,964	MST37	2,710	0.72	
		8.167	17	5.000	85	12	109	194	792	475	2,371	1,896	0	1,896	MST37	2,710	0.70	
														1,964	HDU2	3,075	0.64	***Max. Total Uplift Down to Fndn.***
Main Floor	1	16.000	17	7.000	119	15	136	255	2,042	1,225	495	-730	0	-730	N/A	500	0.00	
		6.000	17	7.000	119	15	136	255	766	459	495	36	0	36	N/A	500	0.07	
Main Floor	2	4.000	15	1.000	15	12	109	124	248	149	9,610	9,461	1,050	10,511	HHDQ11	11,810	0.89	***Stacked Side***
											9,610	9,610	0	9,610	HHDQ11	11,810	0.81	***Non-Stacked Side***
		8.000	15	1.000	15	12	109	124	496	298	9,610	9,312	0	9,312	HHDQ11	11,810	0.79	***Non-Stacked Side***
											9,610	9,610	1,731	11,341	HHDQ11	11,810	0.96	***Stacked Side***
Main Floor	3	20.583	15	1.000	15	12	109	124	1,276	766	3,529	2,763	1,760	4,523	HDU5	5,645	0.80	***Stacked Side***
											3,529	3,529	0	3,529	HDU4	4,565	0.77	***Non-Stacked Side***
		18.417	15	1.000	15	12	109	124	1,142	685	3,529	2,844	0	2,844	HDU4	4,565	0.62	***Non-Stacked Side***
											3,529	3,529	1,896	5,425	HDU5	5,645	0.96	***Stacked Side***
Main Floor	4	6.000	17	3.000	51	12	109	160	480	288	1,660	1,372	0	1,372	LTT20B	1,500	0.91	
		8.000	17	3.000	51	12	109	160	640	384	1,660	1,276	0	1,276	LTT20B	1,500	0.85	

COLLECTOR FORCES

Level	Wall	L_d (ft)	v_{w_d} (plf)	b_{s_tot} (ft)	v_{w_sw} (plf)	L_c (ft)	v_c (plf)	IR_f	F_c (lbf)	Strap/Clips (if req'd)	F_{Ccap}	INT_{col}	Governing Load	F_{LRFD}	Remarks
Perpendicular to Narrow Face															
2nd Floor	B	42.000	156	16.500	397	25.500	-241	1.00	3,977	N/A	0	0.00	W	6,628	
2nd Floor	E	42.000	156	16.000	409	26.000	-253	1.00	4,055	N/A	0	0.00	W	6,758	
Main Floor	B	42.000	183	16.333	472	25.667	-288	1.00	4,708	N/A	0	0.00	W	7,846	
Main Floor	C	32.000	109	27.000	129	5.000	-20	1.00	3,479	CMSTC16x13'-0" w/16d Sinkers All Holes	4,690	0.74	W	5,799	
Main Floor	D/E	74.000	140	16.000	648	58.000	-508	1.25	5,608	CMST14x20'-0" w/10d's All Holes	6,475	0.87	W	9,347	
Level	Wall	L_d (ft)	v_{w_d} (plf)	b_{s_tot} (ft)	v_{w_sw} (plf)	L_c (ft)	v_c (plf)	IR_f	F_c (lbf)	Strap/Clips (if req'd)	F_{Ccap}	INT_{col}	Governing Load	F_{LRFD}	Remarks
Perpendicular to Wide Face															
2nd Floor	2	40.000	168	36.000	186	4.000	-19	1.00	670	N/A	0	0.00	W	1,117	
2nd Floor	3	40.000	168	25.667	261	14.333	-94	1.00	2,401	N/A	0	0.00	W	4,001	
Main Floor	1	30.000	40	22.000	55	8.000	-15	1.00	320	N/A	0	0.00	W	533	
Main Floor	2	42.000	143	12.000	500	30.000	-357	1.00	4,283	N/A	0	0.00	W	7,138	
Main Floor	3	42.000	201	39.000	217	3.000	-15	1.00	604	N/A	0	0.00	W	1,006	
Main Floor	4	28.000	131	20.000	183	8.000	-52	1.00	1,044	N/A	0	0.00	W	1,740	

DIAPHRAGM CLIP / ANCHOR BOLT

Level	Wall	ΣF_w (lbf)	L_w (ft)	v_w (plf)	Clip	F_{clip} (lbf)	s_{clip} (in)	v_{clip} (plf)	INT_{clip}	Remarks	L_w (ft)	v_w (plf)	F_{ABcap} (lbf)	s_{AB} (in)	n_{AB}	v_{ABcap} (plf)	INT_{AB}	Remarks
											5/8" Dia. AB in 1-1/2" HF Plate, NDS-15, Table 11E, Adjusted: Z' = 1,370 lbf							
Perpendicular to Narrow Face																		
2nd Floor	B	6,550	42.000	156	RBC	445	24	223	0.70									
2nd Floor	E	6,550	42.000	156	RBC	445	24	223	0.70									
Main Floor	B	14,254	42.000	339	A35	650	24	325	1.04	***4% Over Acceptable***	16.333	873	1,370	16	10.4	1,028	0.85	
Main Floor	C	3,479	32.000	109	A35	650	24	325	0.33		27.000	129	1,370	48	2.5	343	0.38	
Main Floor	D/E	16,925	74.000	229	A35	650	24	325	0.70		16.000	1,058	1,370	16	12.4	1,028	1.03	***3% Over Acceptable***
Level	Wall	ΣF_w (lbf)	L_w (ft)	v_w (plf)	Clip	F_{clip} (lbf)	s_{clip} (in)	v_{clip} (plf)	INT_{clip}	Remarks	L_w (ft)	v_w (plf)	F_{ABcap} (lbf)	s_{AB} (in)	n_{AB}	v_{ABcap} (plf)	INT_{AB}	Remarks
											5/8" Dia. AB in 1-1/2" HF Plate, NDS-15, Table 11E, Adjusted: Z' = 1,370 lbf							
Perpendicular to Wide Face																		
2nd Floor	2	6,700	40.000	168	RBC	445	24	223	0.75									
2nd Floor	3	6,700	40.000	168	RBC	445	24	223	0.75									
Main Floor	1	1,199	30.000	40	A35	650	24	325	0.12		22.000	55	1,370	48	0.9	343	0.16	
Main Floor	2	12,696	42.000	302	A35	650	24	325	0.93		12.000	1,058	1,370	16	9.3	1,028	1.03	***3% Over Acceptable***
Main Floor	3	12,696	42.000	302	A35	650	24	325	0.93		39.000	326	1,370	48	9.3	343	0.95	
Main Floor	4	3,654	28.000	131	A35	650	24	325	0.40		20.000	183	1,370	48	2.7	343	0.53	

COLLECTOR / DRAG STRUT LOADS							
Wall D/E at Roof/2nd Floor							
			IF =	1.25			
L (ft) =	74.000		v_d (plf) =	140			
b_s (ft) =	16.000		v_{sw} (plf) =	648			
			v_c (plf) =	-508			
	Lw			v (plf)	A (lbf)	F (lbf)	
	ft	in	ft				
SW	4	0	4.000	-508	-2,033.0	-2,033.0	
C	7	0	7.000	140	981.4	-1,051.6	
SW	4	0	4.000	-508	-2,033.0	-3,084.6	
C	12	0	12.000	140	1,682.5	-1,402.1	
SW	4	0	4.000	-508	-2,033.0	-3,435.1	
C	7	0	7.000	140	981.4	-2,453.6	
SW	4	0	4.000	-508	-2,033.0	-4,486.6	
C	32	0	32.000	140	4,486.6	0.0	
							w/IF
L_{tot} (ft)	74.000		74.000		MAX	0.0	0.0
b_{tot} (ft)					MIN	-4,486.6	-5,608.3

VERTICAL DESIGN

Dead Loads:

Concrete density: $\gamma_c := 150 \cdot pcf$

Roof dead load: $DL_r := 17 \cdot psf$

Floor dead load: $DL_f := 15 \cdot psf$

Exterior wall dead load: $DL_{ew} := 15 \cdot psf$

Partition dead load: $DL_{iw} := 12 \cdot psf$

Dead load for members supporting 2 floors,

$DL_{2f} := 2 \cdot DL_f$ $DL_{2f} = 30 psf$

Live Loads:

Define wall live load: $LL_w := 5 \cdot psf$

Roof live load: $LL_r := 20 \cdot psf$

Floor live load: $LL_f := 40 \cdot psf$

Deck live load: $LL_d := 60 \cdot psf$

Main floor / slab on grade: $LL_m := 100 \cdot psf$

Define live load element factor for beams [ASCE 7-10, Table 4-2]: $K_{LL} := 2$

Live load for members supporting 2 floors,

$LL_{2f} := 2 \cdot LL_f$ $LL_{2f} = 80 psf$

Snow Loads:

Define sloped snow: $SL_s := 25 \cdot psf$

Wind Loads:

Wind C&C LRFD downward load: $WL_d := 18.0 \cdot psf$

Wind C&C LRFD uplift load: $WL_u := -74.0 \cdot psf$

Wind C&C LRFD absolute horizontal load: $WL_h := 42.0 \cdot psf$

Seismic Loads:

Short seismic spectral response coefficient: $S_{DS} := 0.996$

Seismic LRFD acceleration: $C_s := 0.153$

Redundancy factor: $\rho_o := 1.3$

Sheathing Design:

Note: Positive values are gravity and negative values are uplift.

Wall Sheathing

Define span rating of sheathing & corresponding thickness: $Span_w := \text{"24/16"}$

Define maximum stud spacing: $s_{st\_max} := 16.0 \cdot in$

Define sheathing capacity [2001 NDS, Table 7.1]: $U_{cap\_w} := 128 \cdot psf$

Maximum wall dead load,

$DL_w := C_s \cdot max(DL_{ew}, DL_{iw})$ $\qquad DL_w = 2.30\,psf$

Governing roof sheathing load combination [ASCE 7-10, Sect. 2.4],

$$U_{app\_w} := \left| \begin{array}{l} A2 \leftarrow \dfrac{LL_w}{1.00} \\ A5a \leftarrow \dfrac{0.6 \cdot WL_h}{1.60} \\ A5b \leftarrow \dfrac{\rho_o \cdot 0.7 \cdot DL_w}{1.60} \\ max(A2, A5a, A5b) \end{array} \right.$$

$U_{app\_w} = 15.75 \cdot psf$ $\qquad \dfrac{U_{app\_w}}{U_{cap\_w}} = 0.12$

Floor Sheathing

Define span rating of sheathing & corresponding thickness: $Span_f := \text{"48/24"}$

Define maximum joist spacing: $s_{jt\_max} := 16.0 \cdot in$

Define sheathing capacity [Ref. 2001 NDS, Table 7.1]: $U_{cap\_f} := 345 \cdot psf$

Governing roof sheathing load combination [ASCE 7-10, Sect. 2.4],

$$U_{app\_f} := \left| \begin{array}{l} A1 \leftarrow \dfrac{DL_f}{0.90} \\ A2 \leftarrow \dfrac{DL_f + LL_m}{1.00} \\ A5 \leftarrow \dfrac{(1.0 + 0.14 \cdot S_{DS}) \cdot DL_f}{1.60} \\ A6 \leftarrow \dfrac{(1.0 + 0.10 \cdot S_{DS}) \cdot DL_f + 0.75 \cdot LL_m}{1.60} \\ max(A1, A2, A5, A6) \end{array} \right.$$

$U_{app\_f} = 115.00 \cdot psf$ $\qquad \dfrac{U_{app\_f}}{U_{cap\_f}} = 0.33$

Roof Sheathing

Define span rating of sheathing & corresponding thickness: $Span_r := \text{"24/16"}$

Define maximum truss spacing: $s_{tr\_max} := 24.0 \cdot in$

Define sheathing capacity [Ref. 2001 NDS, Table 7.1]: $U_{cap\_r} := 51 \cdot psf$

Governing roof sheathing load combination [ASCE 7-10, Sect. 2.4],

$$U_{rs} := \left| \begin{array}{l} A1 \leftarrow \dfrac{DL_r}{0.90} \\ A3a \leftarrow \dfrac{DL_r + LL_r}{1.25} \\ A3b \leftarrow \dfrac{DL_r + SL_s}{1.15} \\ A5a \leftarrow \dfrac{DL_r + 0.6 \cdot WL_d}{1.60} \\ A5b \leftarrow \dfrac{(1.0 + 0.14 \cdot S_{DS}) \cdot DL_r}{1.60} \\ A6a \leftarrow \dfrac{DL_r + 0.75 \cdot (0.6 \cdot WL_d) + 0.75 \cdot max(LL_r, SL_s)}{1.60} \\ A6b \leftarrow \dfrac{(1.0 + 0.10 \cdot S_{DS}) \cdot DL_r + 0.75 \cdot max(LL_r, SL_s)}{1.60} \\ A7 \leftarrow \dfrac{0.6 \cdot DL_r + 0.6 \cdot WL_u}{1.60} \\ A8 \leftarrow \dfrac{(0.6 - 0.14 \cdot S_{DS}) \cdot DL_r}{1.60} \\ Xg \leftarrow max(A1, A3a, A3b, A5a, A5b, A6a, A6b, A7, A8) \\ Xu \leftarrow min(A1, A3a, A3b, A5a, A5b, A6a, A6b, A7, A8) \\ stack(Xg, Xu) \end{array} \right.$$

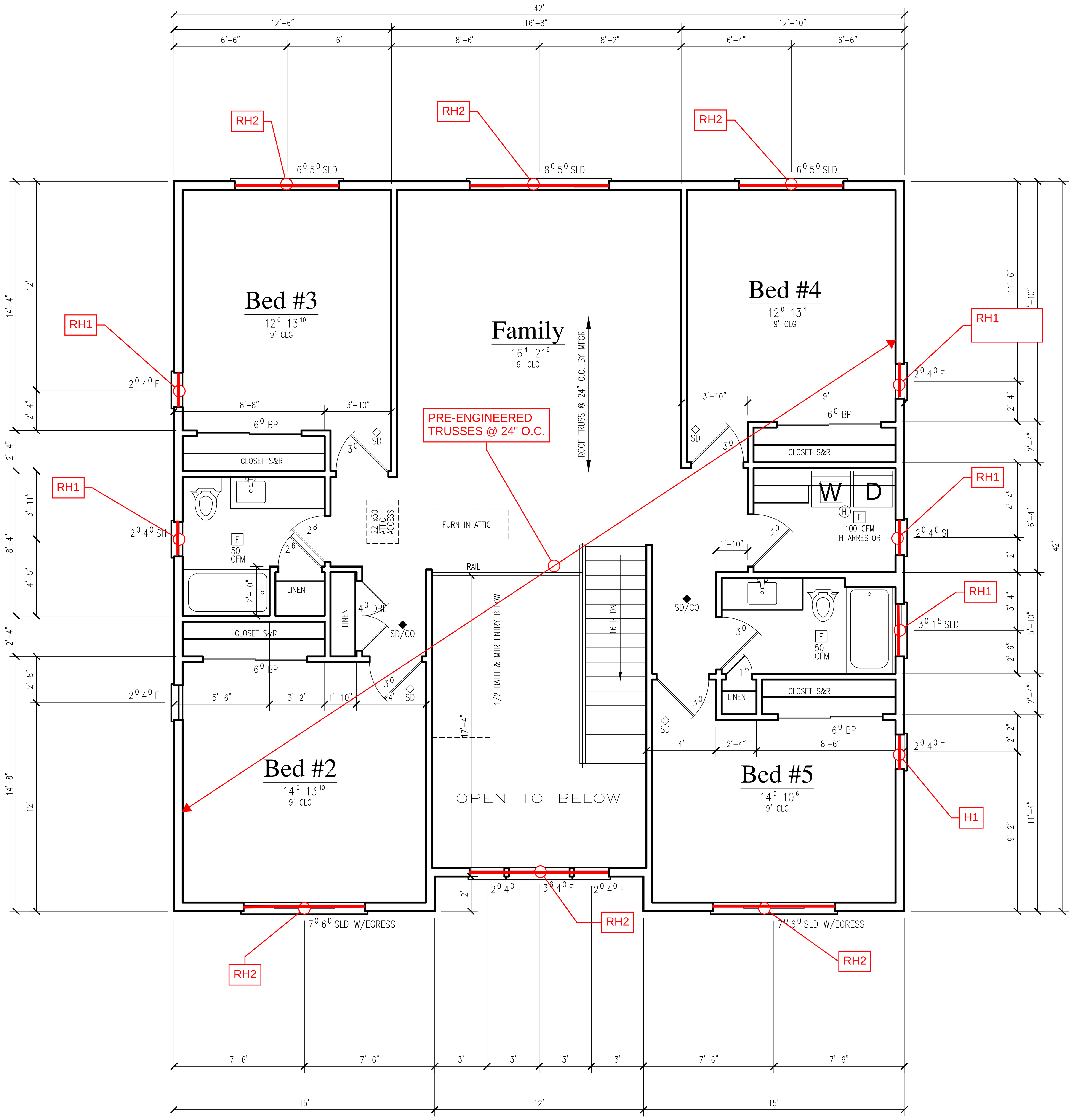
$$U_{rs} = \begin{pmatrix} 36.52 \\ -21.38 \end{pmatrix} \cdot psf$$

Applied load for panel,

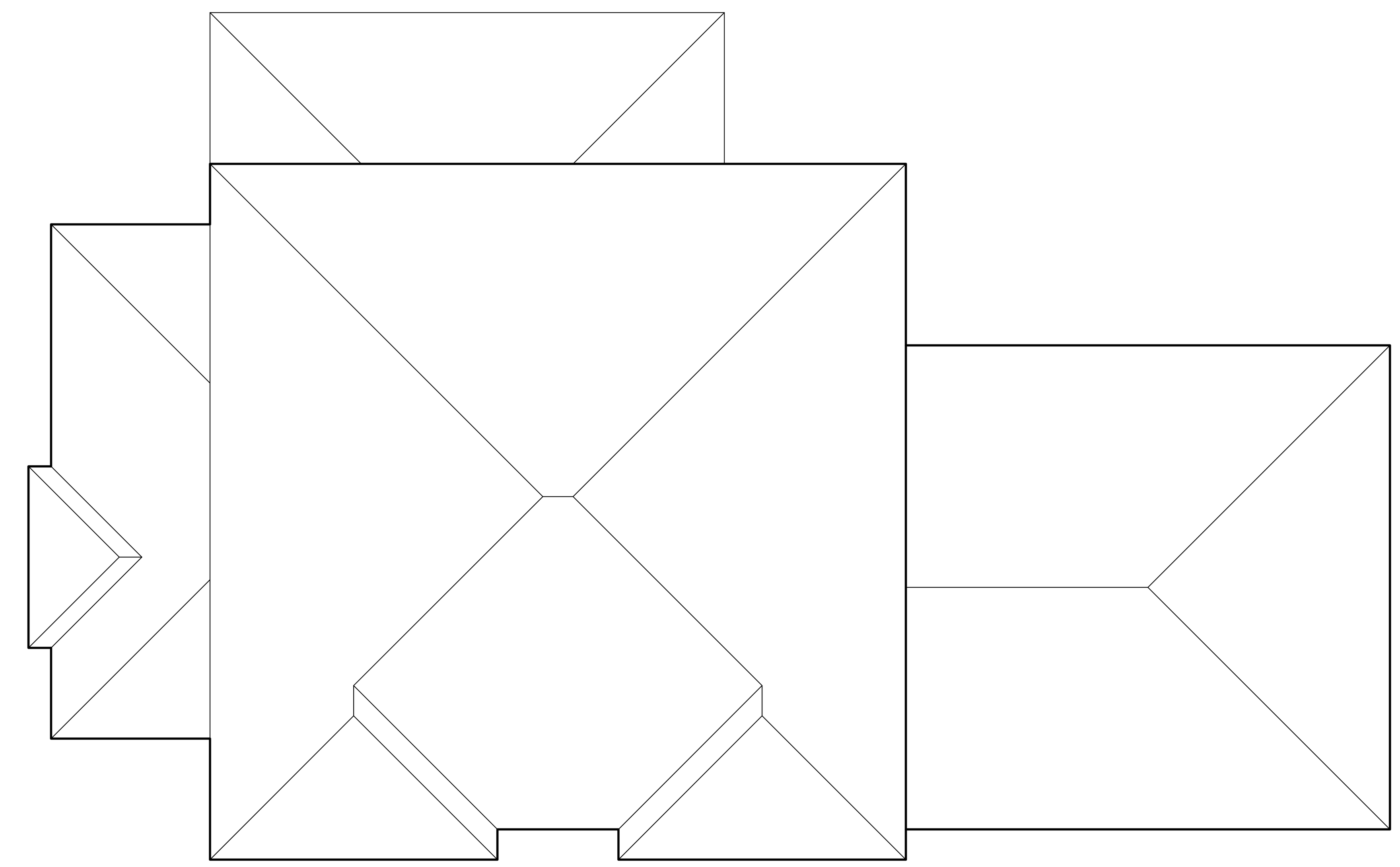
$$U_{app\_r} := max\left(U_{rs}, \left|min\left(U_{rs}\right)\right|\right)$$

$$U_{app\_r} = 36.52 \cdot psf$$

$$\frac{U_{app\_r}}{U_{cap\_r}} = 0.72$$



Upper Floor Plan



Roof Truss Providers
PLEASE PROVIDE YOUR PLANS TO THIS OFFICE VIA EMAIL (mwcm01@gmail.com).

NOTE:
TRUSS MFGR TO VERIFY BEARING POINTS. IF NEW BEARING IS NEEDED, MFGR MUST INFORM THIS DESIGNER (509) 528-2884

Roof Plan

SCALE: 1/8" = 1'-0"

Engineering Required

ALL POSTS, SHEAR WALLS, BEAMS, FOUNDATION, FOOTINGS, & OTHER STRUCTURAL MEMBERS TO BE FULLY ENGINEERED AS NEEDED.

ALL ENGINEERING DOCUMENTATION, FLOORING, AND ROOF PACKAGES SUPERCEDED THESE DRAWINGS.

MCLEOD HOME DESIGNS
www.mcleodhomedesigns.com
1900 Fowler Street, Suite F
Richland, WA 99352
509-528-2884

Altman's East Lot
APN 3020459151

Building Information:

Main Floor SQ FT:	2055
Second Floor SQ FT:	1527
Basement SQ FT:	1304
TOTAL SQ FT:	4886
Unfinished SQ FT:	723
Garage SQ FT:	896
Covered Area SQ FT:	0

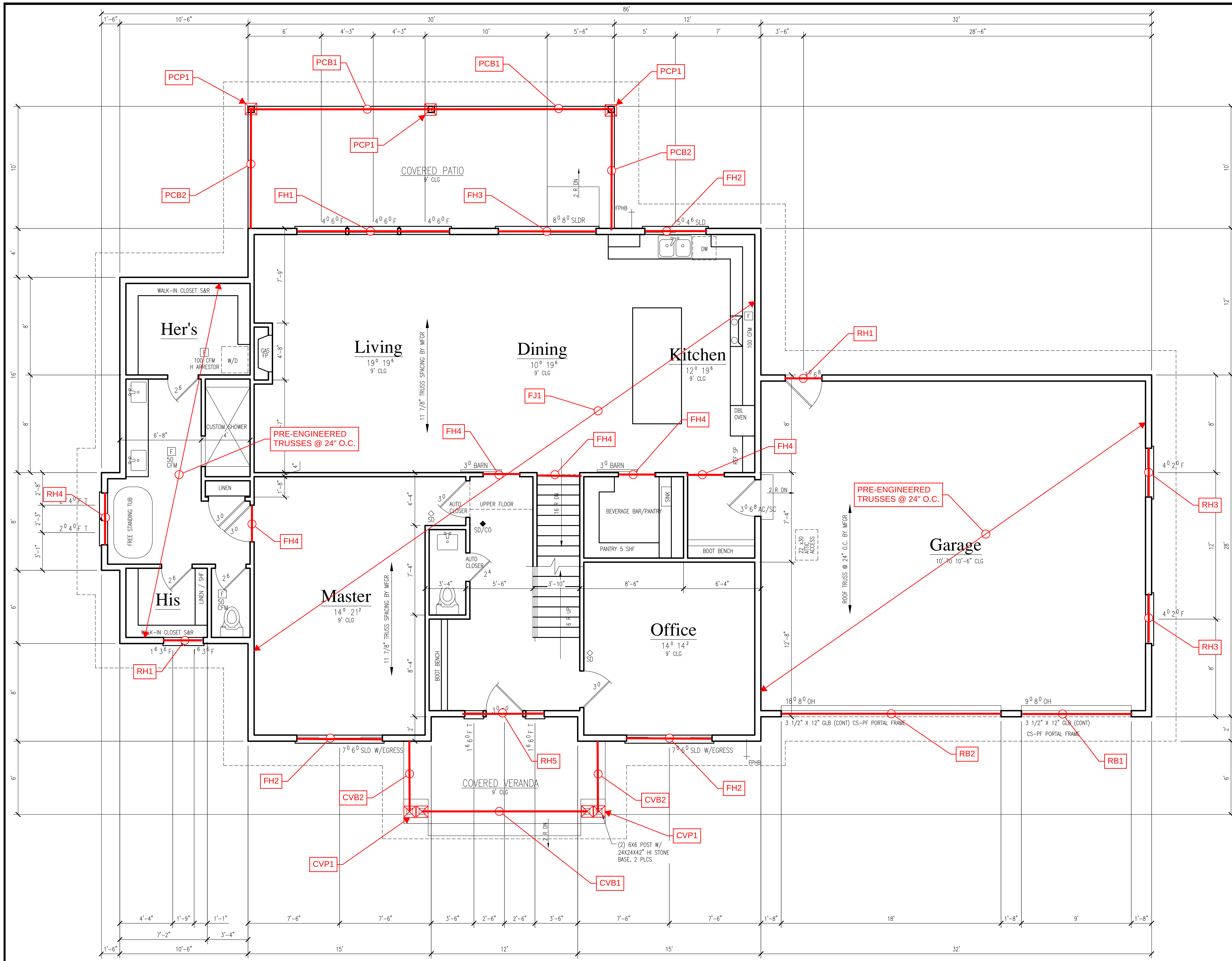
Upper Floor Plan

Altman's East Lot
4886 SF 2-Story
BUILDING ADDRESS: N/A

DWG	t4886x0a east lot.dwg
Date	4/8/20 5:30:PM
By:	Mark McLeod
Scale	1/4" = 1'
Approved	

3b

REV: 0 4/8/20



Main Floor Plan

Engineering Required

ALL POSTS, SHEAR WALLS, BEAMS, FOUNDATION, FOOTINGS, & OTHER STRUCTURAL MEMBERS TO BE FULLY ENGINEERED AS NEEDED.

ALL ENGINEERING DOCUMENTATION, FLOORING, AND ROOF PACKAGES SUPERCEDED THESE DRAWINGS.

ROOF TRUSS PROVIDERS

PLEASE PROVIDE YOUR PLANS TO THIS OFFICE VIA EMAIL (mwcmo1@gmail.com).

FLOOR TRUSS PROVIDERS

PLEASE PROVIDE YOUR PLANS TO THIS OFFICE VIA EMAIL (mwcmo1@gmail.com).

Braced Wall Schedule

CONTINOUS SHEATHING CONDITION (SEISMIC D , WIND 85)

ABW	PER DETAIL SH 4 (IF NEEDED)
CS-PF	PER DETAIL SH 4
CS-WSP	8d COMMON – 6" EDGE 12" FIELD
GB	1 3/8 (13 GA) GB SCREW – 7" EDGE 7" FIELD

LEGEND

SYMBOL	DESCRIPTION
H	HAMMER ARRESTOR
F	FAN VENTED TO EXTERIOR
SD SD/CO	SMOKE / CARBON MONOXIDE DETECTOR (NOTE 15)
FPHB	FROST PROOF HOSE BIB
SC/AC	SOLID CORE / AUTO CLOSER
T	SAFETY OR TEMPERED GLASS

ENERGY CREDITS

#	DESCRIPTION	CREDITS
2a	AIR LEAKAGE CONTROL AND EFFICIENT VENTILATION 2a Compliance based on on R402.4.1.2: Reduce the test air leakage to 3.0 air changes per hour maximum and All whole house ventilation requirements as determined by section M1507.3 of the international Residential Code shall be met with a high efficiency fan (maximum 0.35 including an ECM motor are allowed, provided that they are controlled to operate at low speed in ventilation only mode.	0.5
3b	HIGH EFFICIENCY HVAC EQUIPMENT 3b: Air-source heat pump with minimum HSPF of 9.0	1
5a	EFFICIENT WATER HEATING 5a. All showerhead and kitchen sink fauscets installed in the house shall be rated at 1.75 GPM or less. All other lavatory faucets shall be rated in 1.0 GPM or less.	0.5
5c	EFFICIENT WATER HEATING 5c. Electric heat pump water heater with a minimum EF of 2.0	1.5
	TOTAL	3.5

BUILDING INFORMATION

MAIN FLOOR SF:	2055
SECOND FLOOR SF:	1527
BASEMENT FLOOR SF	1304
TOTAL CONDITIONED SF:	**4886**
TOTAL UNCONDITIONED SF:	
UNFINISHED SF:	723
GARAGE SF:	896
COVERED AREA SF:	0

Builders Responsibility

THESE DRAWINGS ARE IN PART DIAGRAMMICAL AND DO NOT SHOW IN DETAIL HOW WORKMANSHIP, MATERIAL AND INSTALLATION OF MATERIAL ARE TO BE BROUGHT TOGETHER TO COMPLETE THE WHOLE STRUCTURE. IT IS THE RESPONSIBILITY OF THE BUILDER TO BUILD THE STRUCTURE TO COMPLY WITH ALL APPLICABLE FEDERAL, STATE, COUNTY, CITY CODES AS THEY APPLY TO EACH COMPONENT.

General Notes:

1. PROVIDE 30" RANGE AND HOOD W/ 100 CFM FAN VENTED TO EXTERIOR.
2. PROVIDE WATER RESISTANT GYPSUM BOARD IN TUB OR SHOWER RECESS.
3. PROVIDE 50 GALLON (MIN) WATER HEATER W/ ASME RATED TEMPERATURE AND PRESSURE RELIEF VALVE W/ 3/4" COPPER DRIP
4. BUILDER TO VERIFY ALL ASPECTS AND DIMENSIONS OF THESE DRAWINGS. ANY PROBLEMS WITH THESE DRAWINGS ARE TO BE BROUGHT TO THE IMMEDIATE ATTENTION OF THIS DESIGNER, MARK McLEOD (509) 528-2884.
5. DO NOT SCALE THESE DRAWINGS.
6. EXTERIOR WALLS OF HOUSE ARE TO BE 2 X 6, UNLESS OTHERWISE SPECIFIED.
7. INTERIOR WALL OF HOUSE ARE TO BE 2 X 4, UNLESS OTHERWISE SPECIFIED.
8. EXTERIOR WALLS OF GARAGE ARE TO BE 2 X 6, UNLESS OTHERWISE SPECIFIED.
9. HOUSE INSULATION AS NOTED BELOW:
   EXTERIOR WALLS = R-21 BATT INSULATION
   EXTERIOR CEILING = R-49 BLOWN INSULATION
   EXTERIOR FLOORS = R-30 BATT INSULATION
10. ALL FINISH GRADE WORK SHALL BE NO CLOSER THAN 6" TO FINISH SIDING.
11. ALL HEADER MATERIAL FOR BEARING WALLS TO BE 3 ½" x 9" G.L. HEADER STOCK UNLESS OTHERWISE NOTED.
12. DIMENSIONING FORMAT AS FOLLOWS:
    OVER ALL DIMENSIONS SHALL BE FROM EXTERIOR TO EXTERIOR OF BUILDING.
    BREAKS OR JOGS IN BUILDING SHALL BE DIMENSIONED FROM EXTERIOR OF BUILDING.
    INTERIOR WALL DIMENSIONS:
    VERTICALLY SHALL BE TAKEN FROM THE TOP SIDE OF THE WALL.
    HORIZONTAL WALLS SHALL BE TAKEN FROM THE LEFT SIDE OF WALL.
    OPENINGS SHALL BE DIMENSIONED FROM CENTER (EXCEPT GARAGE OPENINGS)
13. ANGULAR WALLS ARE ON A 45 DEGREE ANGLE, UNLESS OTHERWISE NOTED.
14. PROVIDE GAS FIREPLACE PER IRC 302.13 (per plan)
15. NOTE ALL SMOKE DETECTORS ARE ELECTRICALLY HARDWIRED.
16. ALL WINDOWS ARE TO BE .3 U FACTOR MAX.

MCLEOD HOME DESIGNS
www.mcleodhomedesigns.com
1900 Fowler Street, Suite F
Richland, WA 99352
509-528-2884

Altman's East Lot
APN 3020459151

Building Information:

Main Floor SQ FT:	2055
Second Floor SQ FT:	1527
Basement SQ FT:	1304
TOTAL SQ FT:	4886
Unfinished SQ FT:	723
Garage SQ FT:	896
Covered Area SQ FT:	0

Copyright Disclaimer

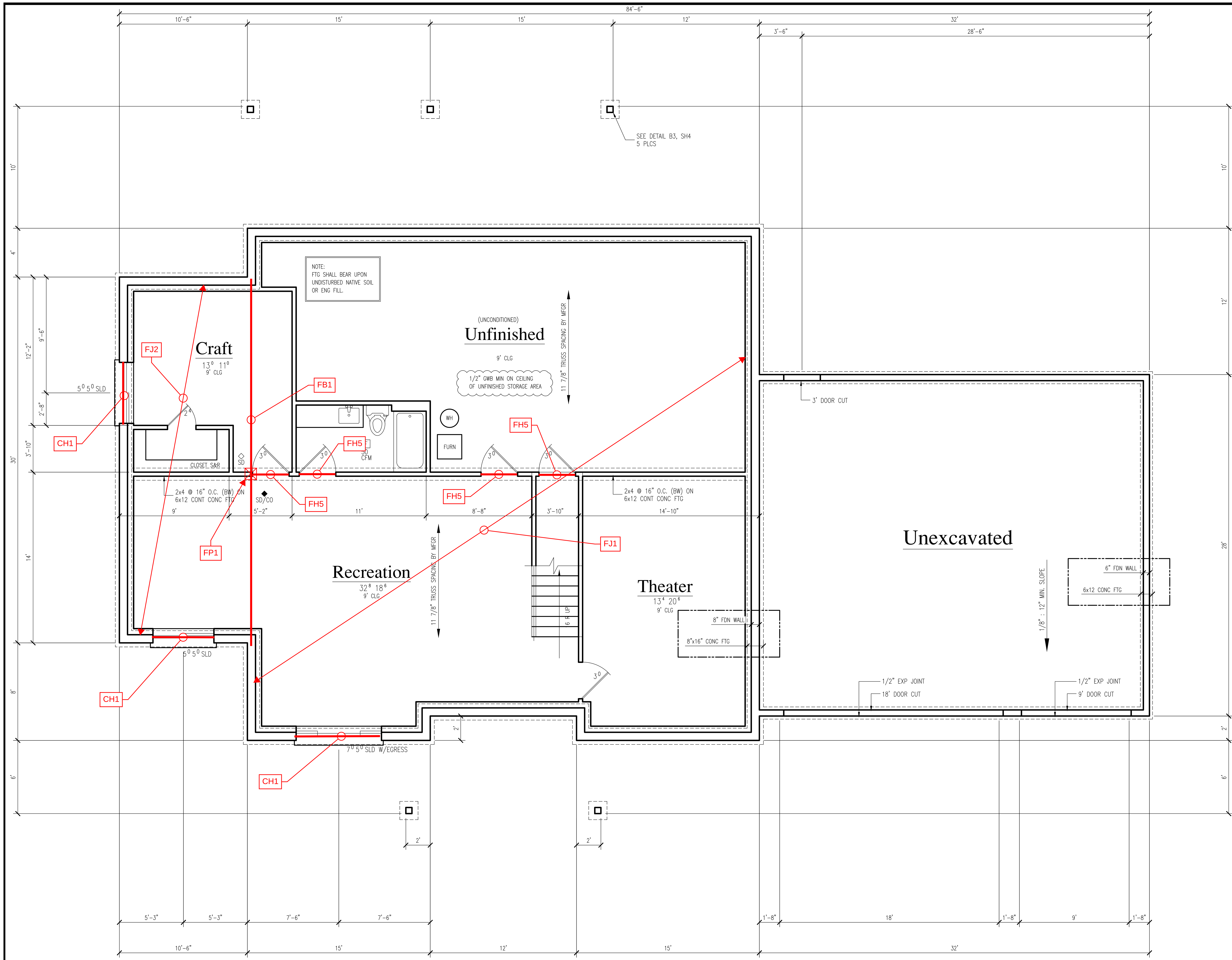
Main Floor Plans

Altman's East Lot
4886 SF **2-Story**
BUILDING ADDRESS: N/A

DWG	t4886x0a east lot.dwg
Date	4/8/20 5:30:PM
By:	Mark McLeod
Scale	1/4" = 1'
Approved	

3a

REV: 0	4/8/20



Basement Floor Plan
Footing & Foundation Plan

Engineering Required

ALL POSTS, SHEAR WALLS, BEAMS, FOUNDATION, FOOTINGS, & OTHER STRUCTURAL MEMBERS TO BE FULLY ENGINEERED AS NEEDED.

ALL ENGINEERING DOCUMENTATION, FLOORING, AND ROOF PACKAGES SUPERCEDED THESE DRAWINGS.

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Altman's East Lot
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TOTAL SQ FT:	4886
Unfinished SQ FT:	723
Garage SQ FT:	896
Covered Area SQ FT:	0

Ftg / Fdn / Roof Plan

Altman's East Lot
4886 SF 2-Story
BUILDING ADDRESS: N/A

DWG	t4886x0a east lot.dwg
Date	4/8/20 5:30:PM
By:	Mark McLeod
Scale	1/4" = 1'
Approved	

2

REV: 0 4/8/20

Vertical Members:

Joists

Floor Joists FJ1

Define span length: $L_{FJ1} := 21.167 \cdot ft$

Define tributary width: $b_{FJ1f} := 16.0 \cdot in$

Dead load, $DL_f = 15\, psf$

Live load, $LL_f = 40\, psf$

Floor Joist FJ2

Define span length: $L_{FJ2} := 12.00 \cdot ft$

Define overhang length: $a_{FJ2} := 1.50 \cdot ft$

Define floor tributary width: $b_{FJ2f} := 16.0 \cdot in$

Dead load, $DL_f = 15\, psf$

Live load, $LL_f = 40\, psf$

Define roof tributary length: $l_{FJ2r} := 8.0 \cdot ft$

Define wall tributary height: $h_{FJ2r} := 8.5 \cdot ft$

Roof dead load at tip of overhang,

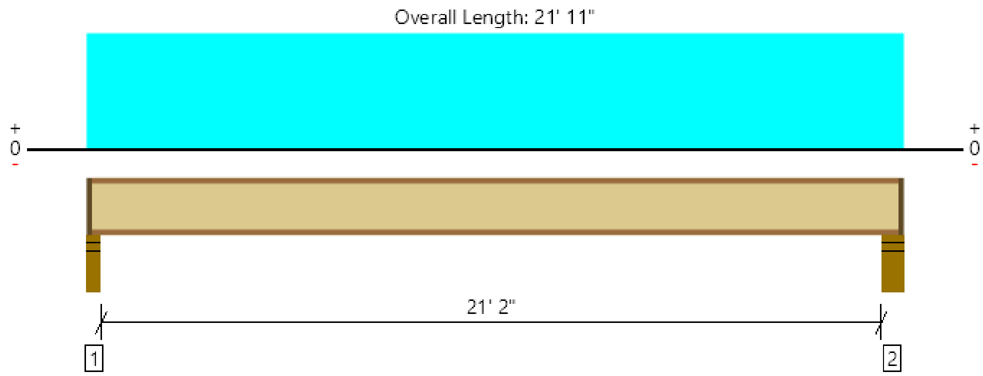
$P_{DL\_FJ\_mb} := DL_r \cdot s_{tr\_max} \cdot l_{FJ2r} + DL_{ew} \cdot s_{st\_max} \cdot h_{FJ2r}$ $\qquad P_{DL\_FJ\_mb} = 442.00 \cdot lbf$

Roof snow load at tip of overhang,

$P_{SL\_FJ\_mb} := SL_s \cdot s_{tr\_max} \cdot l_{FJ2r}$ $\qquad P_{SL\_FJ\_mb} = 400.00 \cdot lbf$

Level, 2nd Floor Joists

1 piece(s) 11 7/8" TJI® 560 @ 12" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	592 @ 2 1/2"	1396 (2.25")	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	582 @ 3 1/2"	2050	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3129 @ 10' 10 1/2"	9500	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.300 @ 10' 10 1/2"	0.533	Passed (L/853)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.413 @ 10' 10 1/2"	0.711	Passed (L/620)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	45	45	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/360).
- Top Edge Bracing (Lu): Top compression edge must be braced at 10' 1" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 21' 9" o/c based on loads applied, unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - DF	3.50"	2.25"	1.75"	163	435	598	1 1/4" Rim Board
2 - Stud wall - DF	5.50"	4.25"	1.75"	166	442	608	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 21' 11"	12"	15.0	40.0	

Weyerhaeuser Notes
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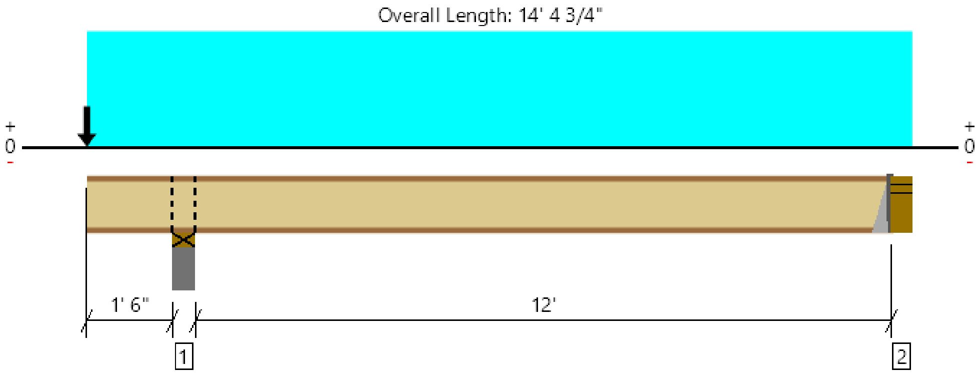
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Jesse Chase MC Squared, Inc. (360) 754-9339 jessec@mc2-inc.com	

Level, Master Bathroom Floor Joist

1 piece(s) 11 7/8" TJI® 110 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1334 @ 1' 8 3/4"	2703 (5.25")	Passed (49%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	880 @ 1' 6"	1794	Passed (49%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	-1500 @ 1' 8 3/4"	3634	Passed (41%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.098 @ 7' 10 1/8"	0.306	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.075 @ 0	0.200	Passed (2L/550)	--	1.0 D + 1.0 S (All Spans)
TJ-Pro™ Rating	56	45	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/360).
- Overhang deflection criteria: LL (2L/480) and TL (2L/0.2").
- Top Edge Bracing (Lu): Top compression edge must be braced at 5' 9" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 4' 8" o/c based on loads applied, unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Plate on concrete - DF	5.50"	5.50"	3.50"	673	425	457	1555	Blocking
2 - Hanger on DF studWall	5.25"	Hanger[1]	1.75" / - [2]	65	349	-57	414/-57	See note [1]

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Top Mount Hanger	ITS1.81/11.88	2.00"	4-10d	2-10d	2-Strong-Grip	

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 14' 4 3/4"	16"	15.0	40.0	-	
2 - Point (lb)	0	N/A	450	-	400	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Jesse Chase MC Squared, Inc. (360) 754-9339 jessec@mc2-inc.com	

Headers

Roof Header RH1

Define span length: $L_{RH1} := 3.00 \cdot ft$

Define roof tributary width: $b_{RH1r} := 23.0 \cdot ft$

Dead load, $DL_r = 17\,psf$

Snow load, $SL_s = 25\,psf$

Roof Header RH2

Define span length: $L_{RH2} := 8.00 \cdot ft$

Define tributary roof width: $b_{RH2r} := 23.00 \cdot ft$

Dead load, $DL_r = 17\,psf$

Snow load, $SL_s = 25\,psf$

Roof Header RH3

Define span length: $L_{RH3} := 4.00 \cdot ft$

Define tributary roof width: $b_{RH3r} := 3.00 \cdot ft$

Dead load, $DL_r = 17\,psf$

Snow load, $SL_s = 25\,psf$

Roof Header RH4

Define span length: $L_{RH4} := 4.50 \cdot ft$

Define tributary roof width: $b_{RH4r} := 8.00 \cdot ft$

Dead load, $DL_r = 17\,psf$

Snow load, $SL_s = 25\,psf$

Roof Header RH5

Define span length: $L_{RH5} := 7.00 \cdot ft$

Define upper tributary roof width: $b_{RH5uf} := 23.00 \cdot ft$

Define lower roof tributary width: $b_{RH5lr} := 4.00 \cdot ft$

Define wall tributary height: $h_{RH5} := 9.00 \cdot ft$

Dead load, $DL_r = 17\,psf$ $\quad DL_{ew} = 15\,psf$

Snow load, $SL_s = 25\,psf$

Multiple Simple Beam

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

Description : Roof Headers

Wood Beam Design : RH1

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **2-2x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

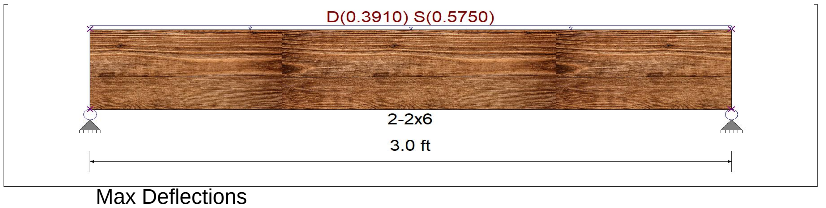
Wood Species : Douglas Fir - Larch | Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.20 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 23.0 ft

Design Summary



Max fb/Fb Ratio =	**0.644** : 1		
fb : Actual :	862.21 psi	at 1.500 ft	in Span # 1
Fb : Allowable :	1,339.10 psi		
Load Comb :	+D+S		
Max fv/FvRatio =	**0.445** : 1		
fv : Actual :	92.21 psi	at 2.550 ft	in Span # 1
Fv : Allowable :	207.00 psi		
Load Comb :	+D+S		

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.59			0.86			
Right Support	0.59			0.86			

Max Deflections

Transient Downward	0.016 in		Total Downward	0.027 in	
Ratio	2274	>480	Ratio	1353	>360
	LC: S Only			LC: +D+S	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

Wood Beam Design : RH2

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **3.5x9, GLB, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

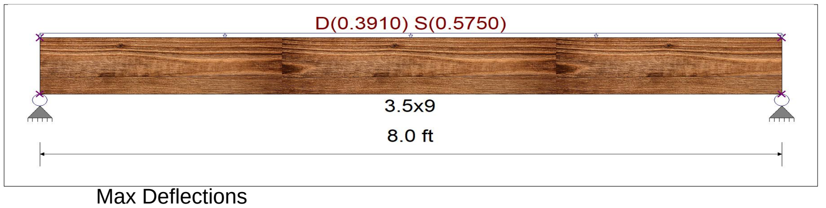
Wood Species : DF/DF | Wood Grade : 24F-V4

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 23.0 ft

Design Summary



Max fb/Fb Ratio =	**0.728** : 1		
fb : Actual :	1,962.67 psi	at 4.000 ft	in Span # 1
Fb : Allowable :	2,696.87 psi		
Load Comb :	+D+S		
Max fv/FvRatio =	**0.491** : 1		
fv : Actual :	149.65 psi	at 0.000 ft	in Span # 1
Fv : Allowable :	304.75 psi		
Load Comb :	+D+S		

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.56			2.30			
Right Support	1.56			2.30			

Max Deflections

Transient Downward	0.139 in		Total Downward	0.234 in	
Ratio	689	>480	Ratio	410	>360
	LC: S Only			LC: +D+S	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

Multiple Simple Beam

Wood Beam Design : RH3

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **2-2x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

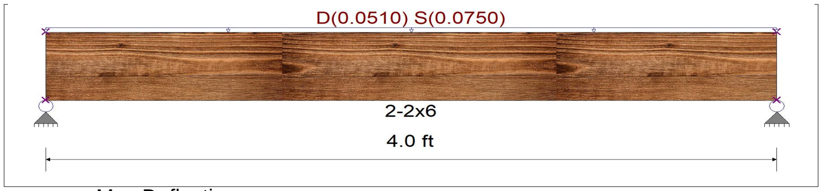
Wood Species : Douglas Fir-Larch Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 3.0 ft

Design Summary



Max fb/Fb Ratio = **0.150** : 1
fb : Actual : 199.93 psi at 2.000 ft in Span # 1
Fb : Allowable : 1,337.13 psi
Load Comb : +D+S

Max fv/FvRatio = **0.086** : 1
fv : Actual : 17.72 psi at 0.000 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.10			0.15			
Right Support	0.10			0.15			

Max Deflections

Transient Downward	0.007 in		Total Downward	0.011 in	
Ratio	7355	>480	Ratio	4378	>360
	LC: S Only			LC: +D+S	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

Wood Beam Design : RH4

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **2-2x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

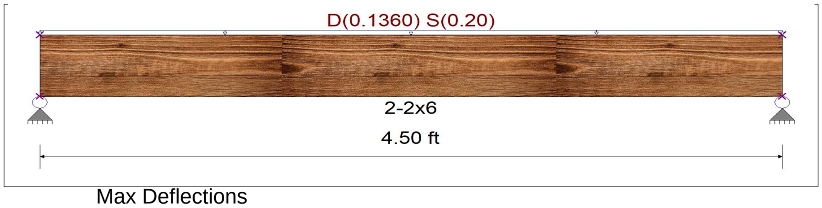
Wood Species : Douglas Fir-Larch Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 8.0 ft

Design Summary



Max fb/Fb Ratio = **0.505** : 1
fb : Actual : 674.78 psi at 2.250 ft in Span # 1
Fb : Allowable : 1,336.16 psi
Load Comb : +D+S

Max fv/FvRatio = **0.266** : 1
fv : Actual : 54.98 psi at 0.000 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.31			0.45			
Right Support	0.31			0.45			

Max Deflections

Transient Downward	0.028 in		Total Downward	0.047 in	
Ratio	1937	>480	Ratio	1153	>360
	LC: S Only			LC: +D+S	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

Multiple Simple Beam

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

Wood Beam Design : RH5

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **3.5x9, GLB, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

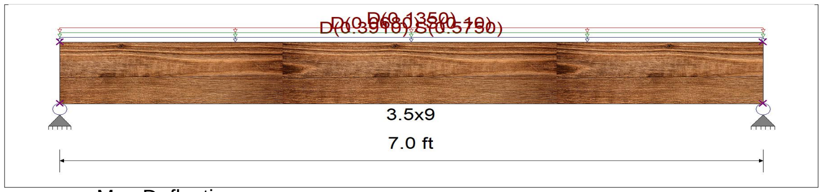
Wood Species : DF/DF Wood Grade : 24F-V4

Fb - Tension	2400 psi	Fc - Prll	1650 psi	Fv	265 psi	Ebend- xx	1800 ksi	Density	31.21 pcf
Fb - Compr	1850 psi	Fc - Perp	650 psi	Ft	1100 psi	Eminbend - xx	950 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 23.0 ft
Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 4.0 ft
Unif Load: D = 0.0150 k/ft, Trib= 9.0 ft

Design Summary



Max fb/Fb Ratio = **0.729** : 1
fb : Actual : 1,974.00 psi at 3.500 ft in Span # 1
Fb : Allowable : 2,706.05 psi
Load Comb : +D+S

Max fv/FvRatio = **0.546** : 1
fv : Actual : 166.38 psi at 6.253 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	2.08			2.36			
Right Support	2.08			2.36			

Max Deflections

Transient Downward	0.096 in		Total Downward	0.180 in	
Ratio	876	>480	Ratio	466	>360
	LC: S Only			LC: +D+S	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

Floor Header FH1

Define span length: $L_{FH1} := 12.50 \cdot ft$

Define upper roof tributary width: $b_{FH1ur} := 23.00 \cdot ft$

Define lower roof tributary width: $b_{FH1lr} := 5.00 \cdot ft$

Define roof tributary width: $b_{FH1f} := 9.75 \cdot ft$

Define wall tributary height: $h_{FH1} := 9.00 \cdot ft$

Dead load, $DL_r = 17\,psf$ $\quad DL_f = 15\,psf$ $\quad DL_{ew} = 15\,psf$

Live load, $LL_f = 40\,psf$

Snow load, $SL_s = 25\,psf$

Floor Header FH2

Define span length: $L_{FH2} := 7.00 \cdot ft$

Define tributary roof width: $b_{FH2r} := 23.00 \cdot ft$

Define tributary floor width: $b_{FH2f} := 10.667 \cdot ft$

Define wall tributary height: $h_{FH2} := 9.00 \cdot ft$

Dead load, $DL_r = 17\,psf$ $\quad DL_f = 15\,psf$ $\quad DL_{ew} = 15\,psf$

Live load, $LL_f = 40\,psf$

Snow load, $SL_s = 25\,psf$

Floor Header FH3

Define span length: $L_{FH3} := 8.00 \cdot ft$

Define upper roof tributary width: $b_{FH3ur} := 23.00 \cdot ft$

Define lower roof tributary width: $b_{FH3lr} := 5.00 \cdot ft$

Define tributary floor width: $b_{FH3f} := 9.75 \cdot ft$

Define wall tributary height: $h_{FH3} := 9.00 \cdot ft$

Dead load, $DL_r = 17\,psf$ $\quad DL_f = 15\,psf$ $\quad DL_{ew} = 15\,psf$

Live load, $LL_f = 40\,psf$

Snow load, $SL_s = 25\,psf$

Floor Header FH4

Define span length: $L_{FH4} := 3.00 \cdot ft$

Define tributary floor width: $b_{FH4f} := 20.5 \cdot ft$

Dead load, $DL_f = 15\,psf$

Live load, $LL_f = 40\,psf$

Floor Header FH5

Define span length: $L_{FH6} := 3.00 \cdot ft$

Define tributary floor width: $b_{FH6f} := 20.5 \cdot ft$

Define wall tributary height: $h_{FH6} := 9.00 \cdot ft$

Dead load, $DL_{2f} = 30\,psf$ $\quad DL_{iw} = 12\,psf$

Live load, $LL_{2f} = 80\,psf$

Multiple Simple Beam

Description : Floor Headers

Wood Beam Design : FH1

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **5.5x15, GLB, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

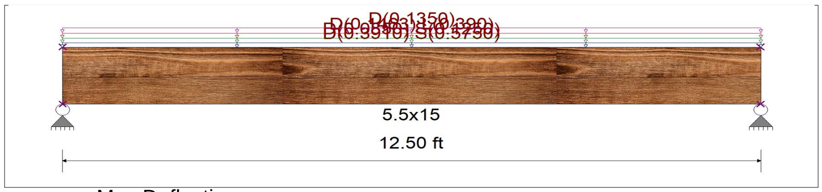
Wood Species : DF/DF Wood Grade : 24F-V4

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 23.0 ft
Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 5.0 ft
Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 9.750 ft
Unif Load: D = 0.0150 k/ft, Trib= 9.0 ft

Design Summary



Max fb/Fb Ratio = **0.665** : 1
fb : Actual : 1,789.49 psi at 6.250 ft in Span # 1
Fb : Allowable : 2,691.03 psi
Load Comb : +D+0.750L+0.750S
Max fv/FvRatio = **0.474** : 1
fv : Actual : 144.35 psi at 11.292 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	4.73	2.44		4.38			
Right Support	4.73	2.44		4.38			

Max Deflections

Transient Downward	0.139 in		Total Downward	0.312 in
Ratio	1080 >480		Ratio	480 >360
	LC: S Only			LC: +D+0.750L+0.750S
Transient Upward	0.000 in		Total Upward	0.000 in
Ratio	9999		Ratio	9999
	LC:			LC:

Wood Beam Design : FH2

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **3.5x9, GLB, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

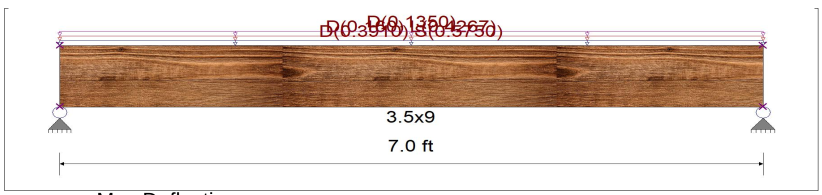
Wood Species : DF/DF Wood Grade : 24F-V4

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 23.0 ft
Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 10.667 ft
Unif Load: D = 0.0150 k/ft, Trib= 9.0 ft

Design Summary



Max fb/Fb Ratio = **0.826** : 1
fb : Actual : 2,235.75 psi at 3.500 ft in Span # 1
Fb : Allowable : 2,706.05 psi
Load Comb : +D+0.750L+0.750S
Max fv/FvRatio = **0.618** : 1
fv : Actual : 188.44 psi at 6.253 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	2.40	1.49		2.01			
Right Support	2.40	1.49		2.01			

Max Deflections

Transient Downward	0.082 in		Total Downward	0.204 in
Ratio	1029 >480		Ratio	411 >360
	LC: S Only			LC: +D+0.750L+0.750S
Transient Upward	0.000 in		Total Upward	0.000 in
Ratio	9999		Ratio	9999
	LC:			LC:

Multiple Simple Beam

Wood Beam Design : FH3

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **3.5x10.5, GLB, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : DF/DF   Wood Grade : 24F-V4

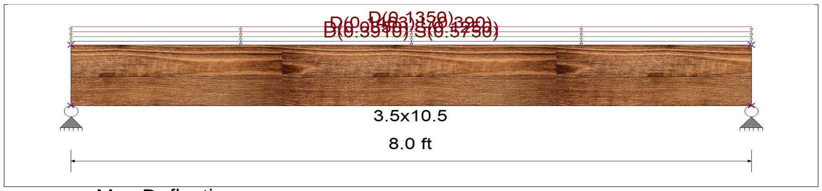
Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 23.0 ft
Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 5.0 ft
Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 9.750 ft
Unif Load: D = 0.0150 k/ft, Trib= 9.0 ft

Design Summary

Max fb/Fb Ratio = **0.878** : 1
fb : Actual : 2,350.65 psi at 4.000 ft in Span # 1
Fb : Allowable : 2,678.42 psi
Load Comb : +D+0.750L+0.750S
Max fv/FvRatio = **0.664** : 1
fv : Actual : 202.25 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+0.750L+0.750S



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	3.03	1.56		2.80			
Right Support	3.03	1.56		2.80			

Max Deflections

Transient Downward	0.107 in		Total Downward	0.240	in
Ratio	899	>480	Ratio	399	>360
	LC: S Only			LC: +D+0.750L+0.750S	
Transient Upward	0.000 in		Total Upward	0.000	in
Ratio	9999		Ratio	9999	
	LC:			LC:	

Wood Beam Design : FH4

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **2-2x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch   Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.0150, L = 0.040 k/ft, Trib= 20.50 ft

Design Summary

Max fb/Fb Ratio = **0.864** : 1
fb : Actual : 1,006.36 psi at 1.500 ft in Span # 1
Fb : Allowable : 1,165.22 psi
Load Comb : +D+L
Max fv/FvRatio = **0.598** : 1
fv : Actual : 107.63 psi at 2.550 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.46	1.23					
Right Support	0.46	1.23					

Max Deflections

Transient Downward	0.023 in		Total Downward	0.031	in
Ratio	1594	>480	Ratio	1159	>360
	LC: L Only			LC: +D+L	
Transient Upward	0.000 in		Total Upward	0.000	in
Ratio	9999		Ratio	9999	
	LC:			LC:	

Multiple Simple Beam

Wood Beam Design : FH5

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **4x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir-Larch   Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.030, L = 0.080 k/ft, Trib= 20.50 ft
Unif Load: D = 0.0120 k/ft, Trib= 9.0 ft

Design Summary

Max fb/Fb Ratio = **0.595** : 1
fb : Actual : 639.14 psi at 1.500 ft in Span # 1
Fb : Allowable : 1,074.91 psi
Load Comb : +D+L

Max fv/FvRatio = **0.444** : 1
fv : Actual : 79.92 psi at 2.230 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.08	2.46					
Right Support	1.08	2.46					

Max Deflections

Transient Downward	0.008 in		Total Downward	0.012 in
Ratio	4424	>480	Ratio	3071 >360
	LC: L Only			LC: +D+L
Transient Upward	0.000 in		Total Upward	0.000 in
Ratio	9999		Ratio	9999
	LC:			LC:

Beams

Roof Beam RB1

Define span length: $L_{RB1} := 9.00 \cdot ft$

Define tributary upper roof width: $b_{RB1r} := 16.00 \cdot ft$

Dead load, $DL_r = 17\,psf$

Snow load, $SL_s = 25\,psf$

Roof Beam RB2

Define span length: $L_{RB2} := 18.00 \cdot ft$

Define tributary upper roof width: $b_{RB2r} := 16.00 \cdot ft$

Dead load, $DL_r = 17\,psf$

Snow load, $SL_s = 25\,psf$

Patio Cover Beam PCB1

Define span length: $L_{PCB1} := 14.75 \cdot ft$

Define tributary roof width: $b_{UFB1r} := 7.00 \cdot ft$

Dead load, $DL_r = 17\,psf$

Live load, $SL_s = 25\,psf$

Patio Cover Beam PCB2

Define span length: $L_{PCB2} := 9.75 \cdot ft$

Define tributary roof width: $b_{UFB2r} := 4.00 \cdot ft$

Dead load, $DL_r = 17\,psf$

Live load, $SL_s = 25\,psf$

Covered Veranda Beam CVB1

Define span length: $L_{CVB1} := 15.50 \cdot ft$

Define tributary roof width: $b_{CVBr} := 6.00 \cdot ft$

Dead load, $DL_r = 17\,psf$

Live load, $SL_s = 25\,psf$

Covered Veranda Beam CVB2

Define span length: $L_{CVB2} := 6.00 \cdot ft$

Define tributary roof width: $b_{CV2r} := 4.00 \cdot ft$

Dead load, $DL_r = 17\,psf$

Live load, $SL_s = 25\,psf$

Floor Beam FB1 **Note: Check shear at depth of 11-7/8" at supports.**

Define span length 1: $L_{FB1\_1} := 16.00 \cdot ft$

Define span length 2: $L_{FB1\_2} := 13.667 \cdot ft$

Beam overall length,

$L_{FB1\_o} := L_{FB1\_1} + L_{FB1\_2}$ $L_{FB1\_o} = 29.67\,ft$

Define tributary upper roof width: $b_{FB1ur} := 6.00 \cdot ft$

Define tributary lower roof width: $b_{FB1lr} := 6.00 \cdot ft$

Define tributary upper floor width: $b_{FB1uf} := 1.00 \cdot ft$

Define tributary low floor width: $b_{FB1lf} := 7.00 \cdot ft$

Define wall tributary height: $h_{FB1} := 18.00 \cdot ft$

Dead load, $DL_r = 17\,psf$ $DL_f = 15\,psf$ $DL_{ew} = 15\,psf$

Live load, $LL_f = 40\,psf$

Snow load, $SL_s = 25\,psf$

Distributed dead load,

$w_{DL\_FB1} := DL_r \cdot (b_{FB1ur} + b_{FB1lr}) + DL_f \cdot (b_{FB1uf} + b_{FB1lf}) \ldots + DL_{ew} \cdot (h_{FB1})$ $w_{DL\_FB1} = 0.59 \cdot klf$

Distributed live load,

$w_{LL\_FB1} := LL_f \cdot (b_{FB1uf} + b_{FB1lf})$ $w_{LL\_FB1} = 0.32 \cdot klf$

Distributed snow load,

$w_{SL\_FB1} := SL_s \cdot (b_{FB1ur} + b_{FB1lr})$ $w_{SL\_FB1} = 0.30 \cdot klf$

Define gross overturning force and source: $OT\_FB1 :=$ "W" $P_{OT\_FB1} := 2.9 \cdot kip$

Define overall locations from start of beam: Location from start of beam segment,

$x1_{OT\_FB1} := 7.000 \cdot ft$ $x1_{OT\_FB1} = 7.000\,ft$

$x2_{OT\_FB1} := 9.000 \cdot ft$ $x2_{OT\_FB1} = 9.000\,ft$

$x3_{OT\_FB1} := 15.583 \cdot ft$ $x3_{OT\_FB1} = 15.583\,ft$

$x4_{OT\_FB1} := 17.583 \cdot ft$ $x4_{OT\_FB1} - L_{FB1\_1} = 1.583\,ft$

$x5_{OT\_FB1} := 25.000 \cdot ft$ $x5_{OT\_FB1} - L_{FB1\_1} = 9.000\,ft$

$x6_{OT\_FB1} := 27.000 \cdot ft$ $x6_{OT\_FB1} - L_{FB1\_1} = 11.000\,ft$

Note: Net uplift from overturning force is less than zero, therefore no special support requried.

Multiple Simple Beam

Description : Roof & Floor Beams

Wood Beam Design : RB1

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **3.5x9, GLB, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

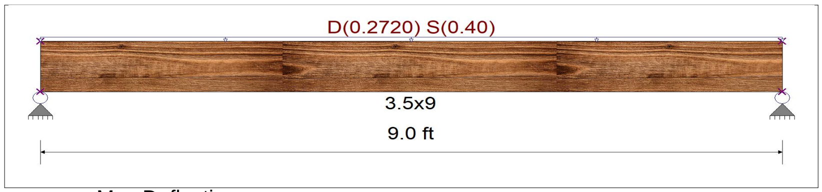
Wood Species : DF/DF   Wood Grade : 24F-V4

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 16.0 ft

Design Summary



Max fb/Fb Ratio = **0.643** : 1
fb : Actual : 1,728.00 psi at 4.500 ft in Span # 1
Fb : Allowable : 2,687.14 psi
Load Comb : +D+S

Max fv/FvRatio = **0.397** : 1
fv : Actual : 120.96 psi at 8.280 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.22			1.80			
Right Support	1.22			1.80			

Max Deflections

Transient Downward	0.155 in		Total Downward	0.261 in	
Ratio	696	>480	Ratio	414	>360
	LC: S Only			LC: +D+S	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

Wood Beam Design : RB2

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **5.5x15, GLB, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

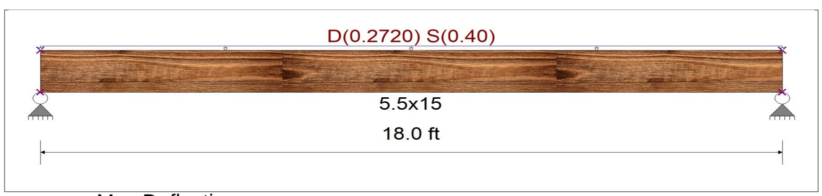
Wood Species : DF/DF   Wood Grade : 24F-V4

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	1,850.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 16.0 ft

Design Summary



Max fb/Fb Ratio = **0.598** : 1
fb : Actual : 1,583.48 psi at 9.000 ft in Span # 1
Fb : Allowable : 2,648.20 psi
Load Comb : +D+S

Max fv/FvRatio = **0.313** : 1
fv : Actual : 95.30 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	2.45			3.60			
Right Support	2.45			3.60			

Max Deflections

Transient Downward	0.341 in		Total Downward	0.573 in	
Ratio	633	>480	Ratio	376	>360
	LC: S Only			LC: +D+S	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

Multiple Simple Beam

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Wood Beam Design : PCB1

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

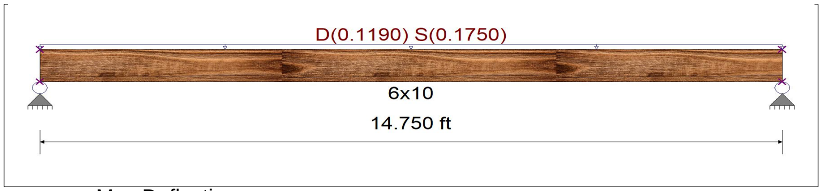
BEAM Size : **6x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch			Wood Grade :	No.1				
Fb - Tension	1,200.0 psi	Fc - Prll	1,000.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	1,200.0 psi	Fc - Perp	625.0 psi	Ft	825.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 7.0 ft

Design Summary



Max fb/Fb Ratio =	**0.851** : 1		
fb : Actual :	1,159.75 psi	at 7.375 ft	in Span # 1
Fb : Allowable :	1,362.93 psi		
Load Comb :	+D+S		
Max fv/FvRatio =	**0.284** : 1		
fv : Actual :	55.61 psi	at 13.963 ft	in Span # 1
Fv : Allowable :	195.50 psi		
Load Comb :	+D+S		

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.88			1.29			
Right Support	0.88			1.29			

Max Deflections

Transient Downward	0.298 in		Total Downward	0.501 in
Ratio	593	>360	Ratio	353 >240
	LC: S Only			LC: +D+S
Transient Upward	0.000 in		Total Upward	0.000 in
Ratio	9999		Ratio	9999
	LC:			LC:

Wood Beam Design : PCB2

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

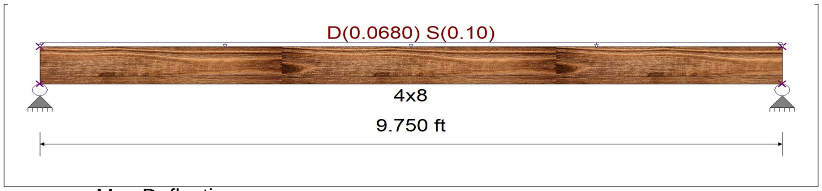
BEAM Size : **4x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir-Larch			Wood Grade :	No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 4.0 ft

Design Summary



Max fb/Fb Ratio =	**0.590** : 1		
fb : Actual :	781.30 psi	at 4.875 ft	in Span # 1
Fb : Allowable :	1,324.31 psi		
Load Comb :	+D+S		
Max fv/FvRatio =	**0.206** : 1		
fv : Actual :	42.60 psi	at 9.165 ft	in Span # 1
Fv : Allowable :	207.00 psi		
Load Comb :	+D+S		

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.33			0.49			
Right Support	0.33			0.49			

Max Deflections

Transient Downward	0.115 in		Total Downward	0.193 in
Ratio	1017	>360	Ratio	605 >240
	LC: S Only			LC: +D+S
Transient Upward	0.000 in		Total Upward	0.000 in
Ratio	9999		Ratio	9999
	LC:			LC:

Multiple Simple Beam

File = C:\Users\JesseC\Desktop\Enercalc\2020-0196-Vertical Design.ec6 .
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Wood Beam Design : CVB1

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **6x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir-Larch — Wood Grade : No.1

Fb - Tension	1,200.0 psi	Fc - Prll	1,000.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	1,200.0 psi	Fc - Perp	625.0 psi	Ft	825.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

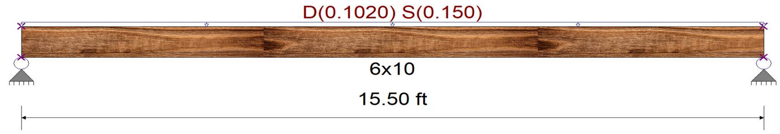
Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 6.0 ft

Design Summary

Max fb/Fb Ratio = **0.806** : 1
fb : Actual : 1,097.73 psi at 7.750 ft in Span # 1
Fb : Allowable : 1,361.87 psi
Load Comb : +D+S

Max fv/FvRatio = **0.258** : 1
fv : Actual : 50.46 psi at 14.725 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.79			1.16			
Right Support	0.79			1.16			



Max Deflections

Transient Downward	0.311 in		Total Downward	0.523 in
Ratio	597 >360		Ratio	355 >240
	LC: S Only			LC: +D+S
Transient Upward	0.000 in		Total Upward	0.000 in
Ratio	9999		Ratio	9999
	LC:			LC:

Wood Beam Design : CVB2

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **4x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir-Larch — Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

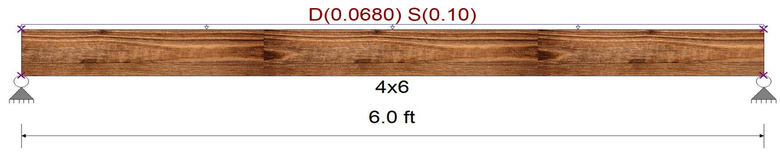
Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 4.0 ft

Design Summary

Max fb/Fb Ratio = **0.385** : 1
fb : Actual : 514.12 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,336.78 psi
Load Comb : +D+S

Max fv/FvRatio = **0.162** : 1
fv : Actual : 33.51 psi at 5.560 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.20			0.30			
Right Support	0.20			0.30			



Max Deflections

Transient Downward	0.038 in		Total Downward	0.063 in
Ratio	1906 >360		Ratio	1135 >240
	LC: S Only			LC: +D+S
Transient Upward	0.000 in		Total Upward	0.000 in
Ratio	9999		Ratio	9999
	LC:			LC:

Wood Beam

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

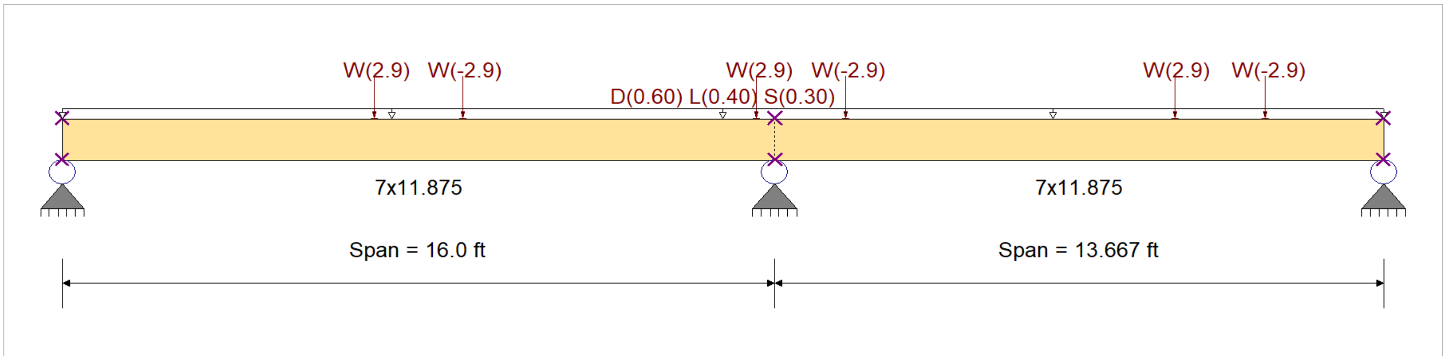
Description : FB1

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-10	Fb -	2,900.0 psi	Ebend- xx	2,200.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,118.19 ksi
Wood Species	: iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade	: Parallam PSL 2.2E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing	: Completely Unbraced				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Loads on all spans...
Uniform Load on ALL spans : D = 0.60, L = 0.40, S = 0.30 k/ft
Load for Span Number 1
Point Load : W = 2.90 k @ 7.0 ft, (Overturning Force)
Point Load : W = -2.90 k @ 9.0 ft, (Overturning Force)
Point Load : W = 2.90 k @ 15.583 ft, (Overturning Force)
Load for Span Number 2
Point Load : W = -2.90 k @ 1.583 ft, (Overturning Force)
Point Load : W = 2.90 k @ 9.0 ft, (Overturning Force)
Point Load : W = -2.90 k @ 11.0 ft, (Overturning Force)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.731**: 1	Maximum Shear Stress Ratio	=	**0.560** : 1
Section used for this span		**7x11.875**	Section used for this span		**7x11.875**
fb : Actual	=	2,096.54 psi	fv : Actual	=	162.33 psi
FB : Allowable	=	2,867.88 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H, LL Comb Run (LL)	Load Combination		+D+L+H, LL Comb Run (LL)
Location of maximum on span	=	16.000 ft	Location of maximum on span	=	15.017 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.188 in	Ratio =	1019 >=480
Max Upward Transient Deflection	-0.067 in	Ratio =	2430 >=480
Max Downward Total Deflection	0.439 in	Ratio =	437 >=360
Max Upward Total Deflection	-0.042 in	Ratio =	3918 >=360

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.450W+H, LL Co	1	0.4387	7.151		0.0000	0.000
+D+L+H, LL Comb Run (*L)	2	0.1747	7.788	+D+L+H, LL Comb Run (L*)	-0.0394	1.909

Wood Beam

Description : FB1

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	7.605	21.564	6.211
Overall MINimum	-0.468	-0.257	0.725
+D+H	3.912	11.665	2.995
+D+L+H, LL Comb Run (*L)	3.643	14.982	5.413
+D+L+H, LL Comb Run (L*)	6.681	15.802	2.490
+D+L+H, LL Comb Run (LL)	6.412	19.119	4.908
+D+Lr+H, LL Comb Run (*L)	3.912	11.665	2.995
+D+Lr+H, LL Comb Run (L*)	3.912	11.665	2.995
+D+Lr+H, LL Comb Run (LL)	3.912	11.665	2.995
+D+S+H	5.787	17.256	4.430
+D+0.750Lr+0.750L+H, LL Comb Run (*	3.710	14.153	4.809
+D+0.750Lr+0.750L+H, LL Comb Run (L	5.988	14.768	2.616
+D+0.750Lr+0.750L+H, LL Comb Run (L	5.787	17.256	4.430
+D+0.750L+0.750S+H, LL Comb Run (*L	5.116	18.346	5.885
+D+0.750L+0.750S+H, LL Comb Run (L*	7.395	18.960	3.692
+D+0.750L+0.750S+H, LL Comb Run (LL	7.193	21.448	5.506
+D+0.60W+H	4.193	11.820	2.560
+D-0.60W+H	3.631	11.511	3.430
+D+0.70E+H	3.912	11.665	2.995
+D+0.750Lr+0.750L+0.450W+H, LL Comb	3.921	14.269	4.482
+D+0.750Lr+0.750L+0.450W+H, LL Comb	6.199	14.883	2.290
+D+0.750Lr+0.750L+0.450W+H, LL Comb	5.997	17.371	4.103
+D+0.750Lr+0.750L-0.450W+H, LL Comb	3.500	14.037	5.135
+D+0.750Lr+0.750L-0.450W+H, LL Comb	5.778	14.652	2.942
+D+0.750Lr+0.750L-0.450W+H, LL Comb	5.576	17.140	4.756
+D+0.750L+0.750S+0.450W+H, LL Comb	5.327	18.462	5.559
+D+0.750L+0.750S+0.450W+H, LL Comb	7.605	19.076	3.366
+D+0.750L+0.750S+0.450W+H, LL Comb	7.403	21.564	5.180
+D+0.750L+0.750S-0.450W+H, LL Comb	4.906	18.230	6.211
+D+0.750L+0.750S-0.450W+H, LL Comb	7.184	18.845	4.019
+D+0.750L+0.750S-0.450W+H, LL Comb	6.982	21.333	5.832
+D+0.750L+0.750S+0.5250E+H, LL Comb	5.116	18.346	5.885
+D+0.750L+0.750S+0.5250E+H, LL Comb	7.395	18.960	3.692
+D+0.750L+0.750S+0.5250E+H, LL Comb	7.193	21.448	5.506
+0.60D+0.60W+0.60H	2.628	7.153	1.362
+0.60D-0.60W+0.60H	2.066	6.845	2.232
+0.60D+0.70E+0.60H	2.347	6.999	1.797
D Only	3.912	11.665	2.995
Lr Only, LL Comb Run (*L)			
Lr Only, LL Comb Run (L*)			
Lr Only, LL Comb Run (LL)			
L Only, LL Comb Run (*L)	-0.269	3.317	2.419
L Only, LL Comb Run (L*)	2.769	4.137	-0.505
L Only, LL Comb Run (LL)	2.500	7.454	1.913
S Only	1.875	5.590	1.435
W Only	0.468	0.257	-0.725
-W	-0.468	-0.257	0.725
E Only			
H Only			

Concrete Headers

Concrete Header CH1

Define span length:	$L_{CH1} := 7.00 \cdot ft$
Define tributary roof width:	$b_{CH1r} := 23.00 \cdot ft$
Define tributary floor width:	$b_{CH1f} := 20.667 \cdot ft$
Define wall tributary height:	$h_{CH1} := 18.00 \cdot ft$
Dead load,	$DL_r = 17\,psf$ $\quad DL_{2f} = 30\,psf$ $\quad DL_{ew} = 15\,psf$
Live load,	$LL_{2f} = 80\,psf$
Snow load,	$SL_s = 25\,psf$

Multiple Simple Beam

Description : Concrete Header

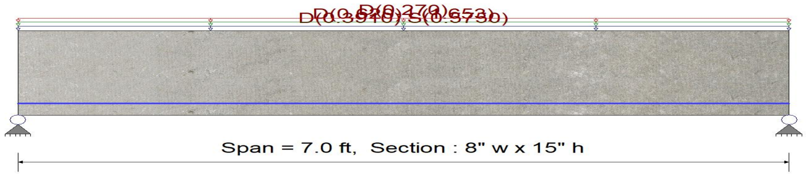
Concrete Beam Design : CH1

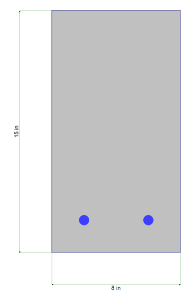
Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10

Rectangular Beam : 8.0 in wide x 15.0 in high

Using Ultimate Strength Design with ASCE 7-10 Load Combinations, Major Axis Bending

f'c =	3.0 ksi	fy Main Stl =	60.0 ksi	E Main Stl =	29,000.0 ksi	Density	150.0 pcf
E Conc =	3,122.0 ksi	fy Stirrups =	60.0 ksi	E Stirrups =	29,000.0 ksi	ϕ Values Bending	0.90
fr =	410.792 ksi	β =	0.850			Shear	0.750





Cross Section & Reinforcing Details

2-#5 at 2.0 in from Bottom, from 0.0 to 7.0 ft in this span

Shear Stirrup Requirements

Stirrup Bar Size = # 4   Number of Resisting Legs Per Stirrup = 2

#4 stirrups (2 legs) at 6.50 in o/c from 0.00 to 1.40 ft along span, Condition : PhiVc < Vu
#4 stirrups (2 legs) at 6.50 in o/c from 1.44 to 2.45 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
No Stirrups Required from 2.49 to 4.51 ft along span, Condition : Vu < PhiVc/2
#4 stirrups (2 legs) at 6.50 in o/c from 4.55 to 5.56 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
#4 stirrups (2 legs) at 6.50 in o/c from 5.60 to 7.00 ft along span, Condition : PhiVc < Vu

Applied Loads

Unif Load: D = 0.0170, S = 0.0250 k/ft, Trib= 23.0 ft
Unif Load: D = 0.030, L = 0.080 k/ft, Trib= 20.667 ft
Unif Load: D = 0.0150 k/ft, Trib= 18.0 ft

Design Summary

Max fb/Fb Ratio = **0.812** : 1
Mu : Applied 27.379 k-ft at 3.500 ft in Span # 1
Mn * Phi : Allowable 33.726 k-ft
Load Comb : +1.20D+1.60L+0.50S

Reactions (k)	D	Lr	L	S	W	E	H
Left Support	4.48		5.79	2.01			
Right Support	4.48		5.79	2.01			

Max Deflections

Transient Downward	0.013 in	Total Downward	0.054 in
Ratio	6646	Ratio	1551
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Ledger Design:

Envelope design by using dimensions off of smaller cantilevered roof and loads off of larger cantilever roof.

Define screw: $Screw_l :=$ "Simpsons 1/4" Dia. x 3-1/2" SDS Screw"

Define screw shear capacity [Simpson SDS Screws]: $Z_l := 1.6\cdot(405\cdot lbf)$ $\quad Z_l = 648.00\cdot lbf$

Define screw withdrawal capacity [Simpson SDS Screws]: $W_l := 1.6\cdot(590\cdot lbf)$ $\quad W_l = 944.00\cdot lbf$

Define edge distance (assumed): $d_{ed\_h} := 2.0\cdot in$

Define width of hanger (assumed): $b_{hg\_v} := 3.0\cdot in$ $\quad$ Define edge distance: $d_{ed\_v} := 1.5\cdot in$

Define screw horizontal spacing: $s_h := 16\cdot in$ $\quad$ Define vert. screw spacing: $s_v := 6.25\cdot in$

Define number of screws in tension: $n_{t\_v} := 1$ $\quad$ Define number of screws in shear: $n_{v\_v} := 2$

Define thickness of ledger: $b_l := 1.50\cdot in$ $\quad$ Define depth of ledger: $d_l := 5.5\cdot in$

Define diaph. length (parallel to load): $L_d := 16.5\cdot ft$ $\quad$ Define diaph. width (perp. to load): $B_d := 10.0\cdot ft$

Number of screw groups,

$$n_{sg} := Floor\left(\frac{L_d}{s_h} + 1, 1\right) \qquad n_{sg} = 13$$

Ledger Vertical Loads

Distributed dead load,

$$w_{dl\_v} := DL_r\cdot b_{UFB1r} \qquad w_{dl\_v} = 119.00\,plf$$

Distributed transient load,

$$w_{tr\_v} := SL_s\cdot b_{UFB1r} \qquad w_{tr\_v} = 175.00\,plf$$

Distributed total load,

$$w_{TL\_v} := w_{dl\_v} + w_{tr\_v} \qquad w_{TL\_v} = 294.00\,plf$$

Shear force on screw per screw due to vertical loads,,

$$V_v := \frac{w_{TL\_v}\cdot s_h}{n_{v\_v}} \qquad V_v = 196.00\cdot lbf$$

Moment on ledger,

$$M_v := \left(b_l + 0.5\cdot b_{hg\_v}\right)\cdot\left(w_{TL\_v}\cdot s_h\right) \qquad M_v = 98.00\cdot ft\cdot lbf$$

Moment arm on tension screw,

$$d_{m\_v} := \left(d_l - d_{ed\_v}\right) - \frac{1}{2}\cdot\frac{1}{3}\cdot\left(d_l - d_{ed\_v}\right) \qquad d_{m\_v} = 3.33\cdot in$$

Tension force due to vertical loads,

$$T_v := \frac{M_v}{d_{m\_v}} \qquad T_v = 352.80\cdot lbf$$

Ledger Horizontal Loads

Define horizontal shear force: $V_H := 2500 \cdot lbf$

Shear force on screw due to horizontal loads,

$$V_h := \frac{V_H}{n_{v\_v} \cdot n_{sg}} \qquad V_h = 96.15 \cdot lbf$$

Moment on ledger,

$$M_h := V_H \cdot (0.5 \cdot B_d) \qquad M_h = 12500.00 \cdot ft \cdot lbf$$

Moment arm on tension screw,

$$d_{m\_h} := (L_d - d_{ed\_h}) - \frac{1}{2} \cdot \frac{1}{3} \cdot (L_d - d_{ed\_h}) \qquad d_{m\_h} = 163.33 \cdot in$$

Tension force due to horizontal loads,

$$T_h := \frac{M_h}{n_{v\_v} \cdot d_{m\_h}} \qquad T_h = 459.18 \cdot lbf$$

Ledger Design

Resultant screw shear force,

$$V_s := \sqrt{V_v^2 + V_h^2} \qquad V_s = 218.32 \cdot lbf \qquad \frac{V_s}{Z_l} = 0.34$$

Total tension force on screw,

$$T_s := T_v + T_h \qquad T_s = 811.98 \cdot lbf \qquad \frac{T_s}{W_l} = 0.86$$

Resultant force on screw,

$$R_s := \sqrt{V_s^2 + T_s^2} \qquad R_s = 840.82 \cdot lbf$$

Hankinsons shear capacity,

$$\alpha_s := atan\left(\frac{T_s}{V_s}\right) \qquad \alpha_s = 74.95 \cdot deg$$

$$Z'_{\alpha\_l} := \frac{W_l \cdot Z_l}{W_l \cdot cos(\alpha_s)^2 + Z_l \cdot sin(\alpha_s)^2} \qquad Z'_{\alpha\_l} = 915.80 \cdot lbf \qquad \frac{R_s}{Z'_{\alpha\_l}} = 0.92$$

Trimmer Stud Design:

Define typical stud width: $b_{st} := 1.5 \cdot in$

Define 2x4 stud depth: $d_{2x4} := 3.5 \cdot in$

Define 2x6 stud depth: $d_{2x6} := 5.5 \cdot in$

Nominal compr. perp. stress for DF [NDS-15 Suppl., Table 4A]: $F_{cL\_H} := 405 \cdot psi$

Nominal compr. perp. stress for DF [NDS-15 Suppl., Table 4A]: $F_{cL\_D} := 625 \cdot psi$

Nominal compr. perp. stress for GLB [NDS-15 Suppl., Table 5A]: $F_{cL\_G} := 650 \cdot psi$

Nominal compr. perp. stress for LVL [Trus Joist]: $F_{cL\_L} := 750 \cdot psi$

Define stress for Simpson Base support (large number of calcs): $F_{cL\_B} := 1000 \cdot psi$

Define stud heights: $H_s := \begin{pmatrix} 9 \\ 9 \\ 9 \end{pmatrix} \cdot ft + \begin{pmatrix} 1.125 \\ 1.125 \\ 1.125 \end{pmatrix} \cdot in$

Define levels: $Level := \begin{pmatrix} \text{"R"} \\ \text{"2F"} \\ \text{"B"} \end{pmatrix}$

$x := length(Level)$ $\quad x = 3$

$j := 1 .. x$

Maximum Trimmer Reaction

Define eccentricity: $e_P := 1.2 \cdot in$

Define load duration factor [NDS-15, Table 2.3.2]: $C_D := 0.90$

Define column buckling factor [NDS-15, Sect. 3.7.1.5]: $c_P := 0.8$

Define #1 DF post bending stress: $F_b := 1200 \cdot psi$

Define #1 DF post compression stress: $F_c := 1000 \cdot psi$

Define #2 DF sill plate comp. perp. stress: $F_{cL} := 625 \cdot psi$

Define #1 DF post min. modulus of elasticity: $E_{min} := 580 \cdot ksi$

Adjusted comp. perp. stress [NDS-15, Sect. 4.2.6],

$F'_{cL} := 0.73 \cdot F_{cL}$ $\quad F'_{cL} = 456.25 \cdot psi$

Compressive stress capacity less column stability factor [NDS-15, Sect. 4.3.1 & Sect. 3.7.1],

$F_{cF} := C_D \cdot F_c$ $\quad F_{cF} = 900.00 \cdot psi$

Flexural bending stress capacity less beam stability factor [NDS-15, Sect. 4.3.1 & Sect. 3.7.1],

$F_{bL} := C_D \cdot F_b$ $\quad F_{bL} = 1080.00 \cdot psi$

Maximum allowable axial force based on sill plate compr. perp. (Ignore C_b factor),

$P_{max} := F'_{cL} \cdot b_{st} \cdot d_{2x6}$ $\quad P_{max} = 3.76\,kip$ $\quad P_{max} \cdot e_P = 0.38\,ft \cdot kip$

Define maximum trimmer height (Wall Height - Dbl Top Plate - 2x6 Hdr Min. Size - Hdr Plate - Sill Plate):

$H_{ts} := (9 \cdot ft + 1.25 \cdot in) - 2 \cdot (1.5 \cdot in) - (5.5 \cdot in) - (1.5 \cdot in) - (1.5 \cdot in)$ $\quad H_{ts} = 8.15\,ft$

Applied compression stress,

$$f_c := \frac{P_{max}}{b_{st} \cdot d_{2x6}} \qquad f_c = 456.25 \cdot psi$$

Compressive stress capacity [NDS-15, Sect. 4.3.1 & Sect. 3.7.1], $\quad 1.0 \cdot H_{ts} = 8.15\,ft$

$$K_C := \frac{1.0 \cdot H_{ts}}{d_{2x6}} \qquad K_C = 17.77$$

$$F_{cE} := \frac{0.822 \cdot E_{min}}{K_C^{\,2}} \qquad F_{cE} = 1509.36 \cdot psi$$

$$C_P := \frac{1 + \frac{F_{cE}}{F_{cF}}}{2 \cdot c_P} - \sqrt{\left(\frac{1 + \frac{F_{cE}}{F_{cF}}}{2 \cdot c_P}\right)^2 - \frac{\frac{F_{cE}}{F_{cF}}}{c_P}} \qquad C_P = 0.835$$

$$F'_c := C_P \cdot F_{cF} \qquad F'_c = 751.16 \cdot psi \qquad \frac{f_c}{F'_c} = 0.61$$

Flexural bending stress capacity [NDS-15, Sect. 4.3.1 & Sect. 3.7.1],

$$F'_b := F_{bL} \qquad F'_b = 1080.00 \cdot psi$$

Combined stress interaction [NDS-15, Sect. 15.4.1 (b)],

$$INT_{ts} := \left(\frac{f_c}{F'_b}\right)^2 + \frac{f_c \cdot \left(\frac{6 \cdot e_P}{d_{2x6}}\right) \cdot \left[1 + 0.234 \cdot \left(\frac{f_c}{F_{cE}}\right)\right]}{F'_b \cdot \left[1 - \left(\frac{f_c}{F_{cE}}\right)\right]} \qquad INT_{ts} = 1.03$$

3% Over Acceptable

Header Trimmer Stud Design

Define headers & vector data:

$Header := augment("RH1", "RH2", "RH3", "RH4", "RH5", "FH1", "FH2", "FH3", "FH4", "FH5")$

$h := length\left(Header^T\right)$ $\quad h = 10 \quad$ $ts := 1 .. h$

Define header beam width:

$$b_{h\_X} := \left| \begin{array}{l} V1 \leftarrow stack(3.0, 0.0, 0.0) \cdot in \\ V2 \leftarrow stack(3.5, 0.0, 0.0) \cdot in \\ V3 \leftarrow stack(3.0, 0.0, 0.0) \cdot in \\ V4 \leftarrow stack(3.0, 0.0, 0.0) \cdot in \\ V5 \leftarrow stack(3.5, 0.0, 0.0) \cdot in \\ V6 \leftarrow stack(0.0, 5.5, 0.0) \cdot in \\ V7 \leftarrow stack(0.0, 3.5, 0.0) \cdot in \\ V8 \leftarrow stack(0.0, 3.5, 0.0) \cdot in \\ V9 \leftarrow stack(0.0, 3.0, 0.0) \cdot in \\ V10 \leftarrow stack(0.0, 0.0, 3.5) \cdot in \\ augment(V1, V2, V3, V4, V5, V6, V7, V8, V9, V10) \end{array} \right.$$

$$b_h := \left| \begin{array}{l} X \leftarrow augment\left(Level, b_{h\_X}\right) \\ stack(augment("Level", Header), X) \end{array} \right.$$

$$b_h = \begin{pmatrix} "Level" & "RH1" & "RH2" & "RH3" & "RH4" & "RH5" & "FH1" & "FH2" & "FH3" & "FH4" & "FH5" \\ "R" & 3.00 & 3.50 & 3.00 & 3.00 & 3.50 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ "2F" & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 5.50 & 3.50 & 3.50 & 3.00 & 0.00 \\ "B" & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 3.50 \end{pmatrix} \cdot in$$

Define trimmer studs dead load:

$$P_{DLts\_X} := \left| \begin{array}{l} V1 \leftarrow stack(0.6, 0.0, 0.0) \cdot kip \\ V2 \leftarrow stack(1.6, 0.0, 0.0) \cdot kip \\ V3 \leftarrow stack(0.1, 0.0, 0.0) \cdot kip \\ V4 \leftarrow stack(0.4, 0.0, 0.0) \cdot kip \\ V5 \leftarrow stack(2.1, 0.0, 0.0) \cdot kip \\ V6 \leftarrow stack(0.0, 4.8, 0.0) \cdot kip \\ V7 \leftarrow stack(0.0, 1.6 + 2.4, 0.0) \cdot kip \\ V8 \leftarrow stack(0.0, 3.1, 0.0) \cdot kip \\ V9 \leftarrow stack(0.0, 0.5, 0.0) \cdot kip \\ V10 \leftarrow stack(0.2, 0.0, 1.1) \cdot kip \\ augment(V1, V2, V3, V4, V5, V6, V7, V8, V9, V10) \end{array} \right.$$

$$P_{DLts} := \left| \begin{array}{l} X \leftarrow augment\left(Level, P_{DLts\_X}\right) \\ stack(augment("Level", Header), X) \end{array} \right.$$

$$P_{DLts} = \begin{pmatrix} "Level" & "RH1" & "RH2" & "RH3" & "RH4" & "RH5" & "FH1" & "FH2" & "FH3" & "FH4" & "FH5" \\ "R" & 0.60 & 1.60 & 0.10 & 0.40 & 2.10 & 0.00 & 0.00 & 0.00 & 0.00 & 0.20 \\ "2F" & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 4.80 & 4.00 & 3.10 & 0.50 & 0.00 \\ "B" & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 1.10 \end{pmatrix} kip$$

Define trimmer studs live load,

$$P_{LLts\_X} := \begin{array}{|l} V1 \leftarrow stack(0.0, 0.0, 0.0) \cdot kip \\ V2 \leftarrow stack(0.0, 0.0, 0.0) \cdot kip \\ V3 \leftarrow stack(0.0, 0.0, 0.0) \cdot kip \\ V4 \leftarrow stack(0.0, 0.0, 0.0) \cdot kip \\ V5 \leftarrow stack(0.0, 0.0, 0.0) \cdot kip \\ V6 \leftarrow stack(0.0, 2.5, 0.0) \cdot kip \\ V7 \leftarrow stack(0.0, 1.5, 0.0) \cdot kip \\ V8 \leftarrow stack(0.0, 1.6, 0.0) \cdot kip \\ V9 \leftarrow stack(0.0, 1.3, 0.0) \cdot kip \\ V10 \leftarrow stack(0.0, 0.0, 2.5) \cdot kip \\ augment(V1, V2, V3, V4, V5, V6, V7, V8, V9, V10) \end{array}$$

$$P_{LLts} := \begin{array}{|l} X \leftarrow augment(Level, P_{LLts\_X}) \\ stack(augment(\text{"Level"}, Header), X) \end{array}$$

$$P_{LLts} = \begin{pmatrix} \text{"Level"} & \text{"RH1"} & \text{"RH2"} & \text{"RH3"} & \text{"RH4"} & \text{"RH5"} & \text{"FH1"} & \text{"FH2"} & \text{"FH3"} & \text{"FH4"} & \text{"FH5"} \\ \text{"R"} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ \text{"2F"} & 0 & 0 & 0 & 0 & 0 & 2.5 & 1.5 & 1.6 & 1.3 & 0 \\ \text{"B"} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 2.5 \end{pmatrix} kip$$

Define trimmer studs snow load,

$$P_{SLts\_X} := \begin{array}{|l} V1 \leftarrow stack(0.9, 0.0, 0.0) \cdot kip \\ V2 \leftarrow stack(2.3, 0.0, 0.0) \cdot kip \\ V3 \leftarrow stack(0.2, 0.0, 0.0) \cdot kip \\ V4 \leftarrow stack(0.5, 0.0, 0.0) \cdot kip \\ V5 \leftarrow stack(2.4, 0.0, 0.0) \cdot kip \\ V6 \leftarrow stack(0.0, 4.4, 0.0) \cdot kip \\ V7 \leftarrow stack(0.0, 2.3 + 2.1, 0.0) \cdot kip \\ V8 \leftarrow stack(0.0, 2.8, 0.0) \cdot kip \\ V9 \leftarrow stack(0.0, 0.0, 0.0) \cdot kip \\ V10 \leftarrow stack(0.0, 0.0, 0.0) \cdot kip \\ augment(V1, V2, V3, V4, V5, V6, V7, V8, V9, V10) \end{array}$$

$$P_{SLts} := \begin{array}{|l} X \leftarrow augment(Level, P_{SLts\_X}) \\ stack(augment(\text{"Level"}, Header), X) \end{array}$$

$$P_{SLts} = \begin{pmatrix} \text{"Level"} & \text{"RH1"} & \text{"RH2"} & \text{"RH3"} & \text{"RH4"} & \text{"RH5"} & \text{"FH1"} & \text{"FH2"} & \text{"FH3"} & \text{"FH4"} & \text{"FH5"} \\ \text{"R"} & 0.9 & 2.3 & 0.2 & 0.5 & 2.4 & 0 & 0 & 0 & 0 & 0 \\ \text{"2F"} & 0 & 0 & 0 & 0 & 0 & 4.4 & 4.4 & 2.8 & 0 & 0 \\ \text{"B"} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \end{pmatrix} kip$$

Trimmer stud axial total load,

$$P_{TLts\_X_{j,ts}} := P_{DLts\_X_{j,ts}} + \left| \begin{array}{l} Y1_{j,ts} \leftarrow max\left(P_{LLts\_X_{j,ts}}, P_{SLts\_X_{j,ts}}\right) \\ Y2_{j,ts} \leftarrow 0.75 \cdot P_{LLts\_X_{j,ts}} + 0.75 \cdot P_{SLts\_X_{j,ts}} \\ max\left(Y1_{j,ts}, Y2_{j,ts}\right) \end{array} \right.$$

$$P_{TLts} := \left| \begin{array}{l} X \leftarrow augment\left(Level, P_{TLts\_X}\right) \\ stack(augment(\text{"Level"}, Header), X) \end{array} \right.$$

$$P_{TLts} = \begin{pmatrix} \text{"Level"} & \text{"RH1"} & \text{"RH2"} & \text{"RH3"} & \text{"RH4"} & \text{"RH5"} & \text{"FH1"} & \text{"FH2"} & \text{"FH3"} & \text{"FH4"} & \text{"FH5"} \\ \text{"R"} & 1.50 & 3.90 & 0.30 & 0.90 & 4.50 & 0.00 & 0.00 & 0.00 & 0.00 & 0.20 \\ \text{"2F"} & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 9.97 & 8.42 & 6.40 & 1.80 & 0.00 \\ \text{"B"} & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 3.60 \end{pmatrix} kip$$

Number of trimmer studs required at top,

$$n_{ts\_tX_{j,ts}} := \left| \begin{array}{l} A1 \leftarrow F_{cL\_D} \cdot \left(b_{h\_X} \cdot b_{st}\right) \\ A2 \leftarrow Ceil\left(\dfrac{P_{TLts\_X_{j,ts}}}{if\left(A1_{j,ts} = 0, kip, A1_{j,ts}\right)}, 1\right) \end{array} \right.$$

$$n_{ts\_t} := \left| \begin{array}{l} X \leftarrow augment\left(Level, n_{ts\_tX}\right) \\ stack(augment(\text{"Level"}, Header), X) \end{array} \right.$$

$$n_{ts\_t} = \begin{pmatrix} \text{"Level"} & \text{"RH1"} & \text{"RH2"} & \text{"RH3"} & \text{"RH4"} & \text{"RH5"} & \text{"FH1"} & \text{"FH2"} & \text{"FH3"} & \text{"FH4"} & \text{"FH5"} \\ \text{"R"} & 1 & 2 & 1 & 1 & 2 & 0 & 0 & 0 & 0 & 1 \\ \text{"2F"} & 0 & 0 & 0 & 0 & 0 & 2 & 3 & 2 & 1 & 0 \\ \text{"B"} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 2 \end{pmatrix}$$

Number of trimmer studs required at bottom,

$$n_{ts\_bX_{j,ts}} := \left| \begin{array}{l} A1 \leftarrow F_{cL\_H} \cdot \left(d_{2x6} \cdot b_{st}\right) \\ A2 \leftarrow Ceil\left(\dfrac{P_{TLts\_X_{j,ts}}}{if(A1 = 0, kip, A1)}, 1\right) \end{array} \right.$$

$$n_{ts\_b} := \left| \begin{array}{l} X \leftarrow augment\left(Level, n_{ts\_bX}\right) \\ stack(augment(\text{"Level"}, Header), X) \end{array} \right.$$

$$n_{ts\_b} = \begin{pmatrix} \text{"Level"} & \text{"RH1"} & \text{"RH2"} & \text{"RH3"} & \text{"RH4"} & \text{"RH5"} & \text{"FH1"} & \text{"FH2"} & \text{"FH3"} & \text{"FH4"} & \text{"FH5"} \\ \text{"R"} & 1 & 2 & 1 & 1 & 2 & 0 & 0 & 0 & 0 & 1 \\ \text{"2F"} & 0 & 0 & 0 & 0 & 0 & 3 & 3 & 2 & 1 & 0 \\ \text{"B"} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 2 \end{pmatrix}$$

Post Design

Define beam & vector data:

$Beam := stack("RB1", "RB2", "PCB1", "PCB2", "CVB1", "CVB1", "FB1\_a", "FB1\_b", "FB1\_c")$

$\boxed{b := 1 .. length(Beam)}$

Define post size:

$Post\_X := stack("(1)2x6", "(2)2x6", "6x6", "(1)2x6", "6x6", "(1)2x6", "Mud Sill", "6x8", "Mud Sill")$

$Post := augment(Beam, Post\_X)$

$$Post = \begin{pmatrix} "RB1" & "(1)2x6" \\ "RB2" & "(2)2x6" \\ "PCB1" & "6x6" \\ "PCB2" & "(1)2x6" \\ "CVB1" & "6x6" \\ "CVB1" & "(1)2x6" \\ "FB1\_a" & "Mud Sill" \\ "FB1\_b" & "6x8" \\ "FB1\_c" & "Mud Sill" \end{pmatrix}$$

Define effective beam width:

$b_{b\_X} := stack(3.5, 5.5, 5.5, 3.5, 5.5, 3.5, 7, 7, 7) \cdot in$

$b_b := augment(Beam, Post\_X, b_{b\_X})$

$$b_b = \begin{pmatrix} "RB1" & "(1)2x6" & 3.50 \\ "RB2" & "(2)2x6" & 5.50 \\ "PCB1" & "6x6" & 5.50 \\ "PCB2" & "(1)2x6" & 3.50 \\ "CVB1" & "6x6" & 5.50 \\ "CVB1" & "(1)2x6" & 3.50 \\ "FB1\_a" & "Mud Sill" & 7.00 \\ "FB1\_b" & "6x8" & 7.00 \\ "FB1\_c" & "Mud Sill" & 7.00 \end{pmatrix} \cdot in$$

Define post bearing width at top:

$b_{p\_tX} := stack\left[1.5, 2\cdot(1.5), \frac{5.5}{2}, 1.5, \frac{5.5}{2}, 1.5, 3, 5.5, 3\right] \cdot in$

$b_{p\_t} := augment(Beam, b_{p\_tX})$

$$b_{p\_t} = \begin{pmatrix} "RB1" & 1.50 \\ "RB2" & 3.00 \\ "PCB1" & 2.75 \\ "PCB2" & 1.50 \\ "CVB1" & 2.75 \\ "CVB1" & 1.50 \\ "FB1\_a" & 3.00 \\ "FB1\_b" & 5.50 \\ "FB1\_c" & 3.00 \end{pmatrix} \cdot in$$

Define post bearing width at bottom:

$b_{p\_bX} := stack(5.5, 5.5, 5.5, 5.5, 5.5, 5.5, 7, 7.5, 7) \cdot in$

$b_{p\_b} := augment(Beam, Post\_X, b_{p\_bX})$

$$b_{p\_b} = \begin{pmatrix} "RB1" & "(1)2x6" & 5.50 \\ "RB2" & "(2)2x6" & 5.50 \\ "PCB1" & "6x6" & 5.50 \\ "PCB2" & "(1)2x6" & 5.50 \\ "CVB1" & "6x6" & 5.50 \\ "CVB1" & "(1)2x6" & 5.50 \\ "FB1\_a" & "Mud Sill" & 7.00 \\ "FB1\_b" & "6x8" & 7.50 \\ "FB1\_c" & "Mud Sill" & 7.00 \end{pmatrix} \cdot in$$

Define posts dead load:

$P_{DLp\_X} := stack(1.3, 2.5, 0.9, 0.4, 0.8, 0.2, 3.9, 11.5, 3.0) \cdot kip$

$P_{DLp} := augment(Beam, Post\_X, P_{DLp\_X})$

$$P_{DLp} = \begin{pmatrix} \text{"RB1"} & \text{"(1)2x6"} & 1.3 \\ \text{"RB2"} & \text{"(2)2x6"} & 2.5 \\ \text{"PCB1"} & \text{"6x6"} & 0.9 \\ \text{"PCB2"} & \text{"(1)2x6"} & 0.4 \\ \text{"CVB1"} & \text{"6x6"} & 0.8 \\ \text{"CVB1"} & \text{"(1)2x6"} & 0.2 \\ \text{"FB1\_a"} & \text{"Mud Sill"} & 3.9 \\ \text{"FB1\_b"} & \text{"6x8"} & 11.5 \\ \text{"FB1\_c"} & \text{"Mud Sill"} & 3.0 \end{pmatrix} kip$$

Define posts studs live load,

$P_{LLp\_X} := stack(0, 0, 0, 0, 0, 0, 2.5, 7.4, 1.9) \cdot kip$

$P_{LLp} := augment(Beam, Post\_X, P_{LLp\_X})$

$$P_{LLp} = \begin{pmatrix} \text{"RB1"} & \text{"(1)2x6"} & 0.0 \\ \text{"RB2"} & \text{"(2)2x6"} & 0.0 \\ \text{"PCB1"} & \text{"6x6"} & 0.0 \\ \text{"PCB2"} & \text{"(1)2x6"} & 0.0 \\ \text{"CVB1"} & \text{"6x6"} & 0.0 \\ \text{"CVB1"} & \text{"(1)2x6"} & 0.0 \\ \text{"FB1\_a"} & \text{"Mud Sill"} & 2.5 \\ \text{"FB1\_b"} & \text{"6x8"} & 7.4 \\ \text{"FB1\_c"} & \text{"Mud Sill"} & 1.9 \end{pmatrix} kip$$

Define posts studs snow load:

$P_{SLp\_X} := stack(1.8, 3.6, 1.3, 0.5, 1.2, 0.3, 1.9, 5.6, 1.5) \cdot kip$

$P_{SLp} := augment(Beam, Post\_X, P_{SLp\_X})$

$$P_{SLp} = \begin{pmatrix} \text{"RB1"} & \text{"(1)2x6"} & 1.8 \\ \text{"RB2"} & \text{"(2)2x6"} & 3.6 \\ \text{"PCB1"} & \text{"6x6"} & 1.3 \\ \text{"PCB2"} & \text{"(1)2x6"} & 0.5 \\ \text{"CVB1"} & \text{"6x6"} & 1.2 \\ \text{"CVB1"} & \text{"(1)2x6"} & 0.3 \\ \text{"FB1\_a"} & \text{"Mud Sill"} & 1.9 \\ \text{"FB1\_b"} & \text{"6x8"} & 5.6 \\ \text{"FB1\_c"} & \text{"Mud Sill"} & 1.5 \end{pmatrix} kip$$

Define posts studs overturning load:

$P_{OTp\_X} := stack(0, 0, 0, 0, 0, 0, 0, 0, 0) \cdot kip$

$P_{OTp} := augment(Beam, Post\_X, P_{OTp\_X})$

$$P_{OTp} = \begin{pmatrix} \text{"RB1"} & \text{"(1)2x6"} & 0.0 \\ \text{"RB2"} & \text{"(2)2x6"} & 0.0 \\ \text{"PCB1"} & \text{"6x6"} & 0.0 \\ \text{"PCB2"} & \text{"(1)2x6"} & 0.0 \\ \text{"CVB1"} & \text{"6x6"} & 0.0 \\ \text{"CVB1"} & \text{"(1)2x6"} & 0.0 \\ \text{"FB1\_a"} & \text{"Mud Sill"} & 0.0 \\ \text{"FB1\_b"} & \text{"6x8"} & 0.0 \\ \text{"FB1\_c"} & \text{"Mud Sill"} & 0.0 \end{pmatrix}$$

Trimmer stud axial total load,

$$P_{TLp\_X_b} := P_{DLp\_X_b} + \left| \begin{array}{l} A1_b \leftarrow max\left(P_{LLp\_X_b}, P_{SLp\_X_b}\right) \\ A2_b \leftarrow 0.75 \cdot \left(P_{LLp\_X_b} + P_{SLp\_X_b}\right) \\ A3_b \leftarrow P_{OTp\_X_b} + 0.75 \cdot \left(P_{LLp\_X_b} + P_{SLp\_X_b}\right) \\ max\left(A1_b, A2_b, A3_b\right) \end{array} \right.$$

$P_{TLp} := augment(Beam, Post\_X, P_{TLp\_X})$

$$P_{TLp} = \begin{pmatrix} \text{"RB1"} & \text{"(1)2x6"} & 3.10 \\ \text{"RB2"} & \text{"(2)2x6"} & 6.10 \\ \text{"PCB1"} & \text{"6x6"} & 2.20 \\ \text{"PCB2"} & \text{"(1)2x6"} & 0.90 \\ \text{"CVB1"} & \text{"6x6"} & 2.00 \\ \text{"CVB1"} & \text{"(1)2x6"} & 0.50 \\ \text{"FB1\_a"} & \text{"Mud Sill"} & 7.20 \\ \text{"FB1\_b"} & \text{"6x8"} & 21.25 \\ \text{"FB1\_c"} & \text{"Mud Sill"} & 5.55 \end{pmatrix} kip$$

Applied compressive perp. stress at top of post,

$$f_{cL\_p\_tX_b} := \frac{P_{TLp\_X_b}}{b_{b\_X_b} \cdot b_{p\_tX_b}}$$

$$f_{cL\_p\_t} := augment(Beam, f_{cL\_p\_tX})$$

$$f_{cL\_p\_t} = \begin{pmatrix} \text{"RB1"} & 590.5 \\ \text{"RB2"} & 369.7 \\ \text{"PCB1"} & 145.5 \\ \text{"PCB2"} & 171.4 \\ \text{"CVB1"} & 132.2 \\ \text{"CVB1"} & 95.2 \\ \text{"FB1\_a"} & 342.9 \\ \text{"FB1\_b"} & 551.9 \\ \text{"FB1\_c"} & 264.3 \end{pmatrix} \cdot psi$$

Adjusted compressive perp. stress at top [NDS-15, Table 4.3.1 & Sect. 4.2.6],

$$F'_{cL\_p\_tX} := stack(F_{cL\_G}, F_{cL\_G}, F_{cL\_D}, F_{cL\_D}, F_{cL\_D}, F_{cL\_D}, F_{cL\_L}, F_{cL\_L}, F_{cL\_L})$$

$$F'_{cL\_p\_t} := augment(Beam, F'_{cL\_p\_tX})$$

$$F'_{cL\_p\_t} = \begin{pmatrix} \text{"RB1"} & 650.00 \\ \text{"RB2"} & 650.00 \\ \text{"PCB1"} & 625.00 \\ \text{"PCB2"} & 625.00 \\ \text{"CVB1"} & 625.00 \\ \text{"CVB1"} & 625.00 \\ \text{"FB1\_a"} & 750.00 \\ \text{"FB1\_b"} & 750.00 \\ \text{"FB1\_c"} & 750.00 \end{pmatrix} \cdot psi$$

Compressive perp. stress interaction at top,

$$INT_{cL\_p\_tX_b} := if\left(Post\_X_b = \text{"HSS"}, \text{"OK"}, \frac{f_{cL\_p\_tX_b}}{F'_{cL\_p\_tX_b}}\right)$$

$$INT_{cL\_p\_t} := augment(Beam, Post\_X, INT_{cL\_p\_tX})$$

$$INT_{cL\_p\_t} = \begin{pmatrix} \text{"RB1"} & \text{"(1)2x6"} & 0.91 \\ \text{"RB2"} & \text{"(2)2x6"} & 0.57 \\ \text{"PCB1"} & \text{"6x6"} & 0.23 \\ \text{"PCB2"} & \text{"(1)2x6"} & 0.27 \\ \text{"CVB1"} & \text{"6x6"} & 0.21 \\ \text{"CVB1"} & \text{"(1)2x6"} & 0.15 \\ \text{"FB1\_a"} & \text{"Mud Sill"} & 0.46 \\ \text{"FB1\_b"} & \text{"6x8"} & 0.74 \\ \text{"FB1\_c"} & \text{"Mud Sill"} & 0.35 \end{pmatrix}$$

Applied compressive perp. stress at bottom of post,

$$f_{cL\_p\_bX_b} := \frac{P_{TLp\_X_b}}{b_{p\_tX_b} \cdot b_{p\_bX_b}}$$

$$f_{cL\_p\_b} := augment\left(Beam, f_{cL\_p\_bX}\right)$$

$$f_{cL\_p\_b} = \begin{pmatrix} \text{"RB1"} & 375.8 \\ \text{"RB2"} & 369.7 \\ \text{"PCB1"} & 145.5 \\ \text{"PCB2"} & 109.1 \\ \text{"CVB1"} & 132.2 \\ \text{"CVB1"} & 60.6 \\ \text{"FB1\_a"} & 342.9 \\ \text{"FB1\_b"} & 515.2 \\ \text{"FB1\_c"} & 264.3 \end{pmatrix} \cdot psi$$

Adjusted compressive perp. stress at bottom [NDS-15, Table 4.3.1 & Sect. 4.2.6],

$$F'_{cL\_p\_bX} := stack\left(F_{cL\_H}, F_{cL\_H}, F_{cL\_B}, F_{cL\_B}, F_{cL\_B}, F_{cL\_B}, F_{cL\_H}, F_{cL\_B}, F_{cL\_H}\right)$$

$$F'_{cL\_p\_b} := augment\left(Beam, F'_{cL\_p\_bX}\right)$$

$$F'_{cL\_p\_b} = \begin{pmatrix} \text{"RB1"} & 405.00 \\ \text{"RB2"} & 405.00 \\ \text{"PCB1"} & 1000.00 \\ \text{"PCB2"} & 1000.00 \\ \text{"CVB1"} & 1000.00 \\ \text{"CVB1"} & 1000.00 \\ \text{"FB1\_a"} & 405.00 \\ \text{"FB1\_b"} & 1000.00 \\ \text{"FB1\_c"} & 405.00 \end{pmatrix} \cdot psi$$

Compressive perp. stress interaction at bottom,

$$INT_{cL\_p\_bX_b} := if\left(Post\_X_b = \text{"HSS"}, \text{"OK"}, \frac{f_{cL\_p\_bX_b}}{F'_{cL\_p\_bX_b}}\right)$$

$$INT_{cL\_p\_b} := augment\left(Beam, Post\_X, INT_{cL\_p\_bX}\right)$$

$$INT_{cL\_p\_b} = \begin{pmatrix} \text{"RB1"} & \text{"(1)2x6"} & 0.93 \\ \text{"RB2"} & \text{"(2)2x6"} & 0.91 \\ \text{"PCB1"} & \text{"6x6"} & 0.15 \\ \text{"PCB2"} & \text{"(1)2x6"} & 0.11 \\ \text{"CVB1"} & \text{"6x6"} & 0.13 \\ \text{"CVB1"} & \text{"(1)2x6"} & 0.06 \\ \text{"FB1\_a"} & \text{"Mud Sill"} & 0.85 \\ \text{"FB1\_b"} & \text{"6x8"} & 0.52 \\ \text{"FB1\_c"} & \text{"Mud Sill"} & 0.65 \end{pmatrix}$$

Wood Column

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

Description : Post FB1

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 9 ft
(Used for non-slender calculations)

Wood Species Douglas Fir-Larch
Wood Grade No.1

Fb +	1200 psi	Fv	170 psi
Fb -	1200 psi	Ft	825 psi
Fc - Prll	1000 psi	Density	31.21 pcf
Fc - Perp	625 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1600	1600	1600 ksi
Minimum	580	580	

Wood Section Name 6x8
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	5.50 in
Exact Depth	7.50 in
Area	41.250 in^2
Ix	193.359 in^4
Iy	103.984 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 9 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 80.463 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = 1.20 in, D = 11.50, L = 7.40, S = 5.60 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8567** : 1

Load Combination	+D+0.750L+0.750S
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	8.940 ft
At maximum location values are . . .	
Applied Axial	21.330 k
Applied Mx	-2.111 k-ft
Applied My	0.0 k-ft
Fc : Allowable	822.75 psi

PASS Maximum Shear Stress Ratio = **0.04492** : 1

Load Combination	+D+L
Location of max.above base	9.0 ft
Applied Design Shear	7.636 psi
Allowable Shear	170.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.2361 k	Bottom along Y-Y	0.2361 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y -0.06221 in at 5.255 ft above base
for load combination : +D+0.750L+0.750S
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.788	0.4382	PASS	8.940 ft	0.03037	PASS	9.0 ft
+D+L	1.000	0.759	0.8249	PASS	8.940 ft	0.04492	PASS	9.0 ft
+D+S	1.150	0.715	0.6080	PASS	8.940 ft	0.03534	PASS	9.0 ft
+D+0.750L	1.250	0.687	0.5563	PASS	8.940 ft	0.03242	PASS	9.0 ft
+D+0.750L+0.750S	1.150	0.715	0.8567	PASS	8.940 ft	0.04392	PASS	9.0 ft
+0.60D	1.600	0.596	0.1766	PASS	8.940 ft	0.01025	PASS	9.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	My - End Moments (k-ft) @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only			-0.128	0.128	11.580				

Wood Column

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

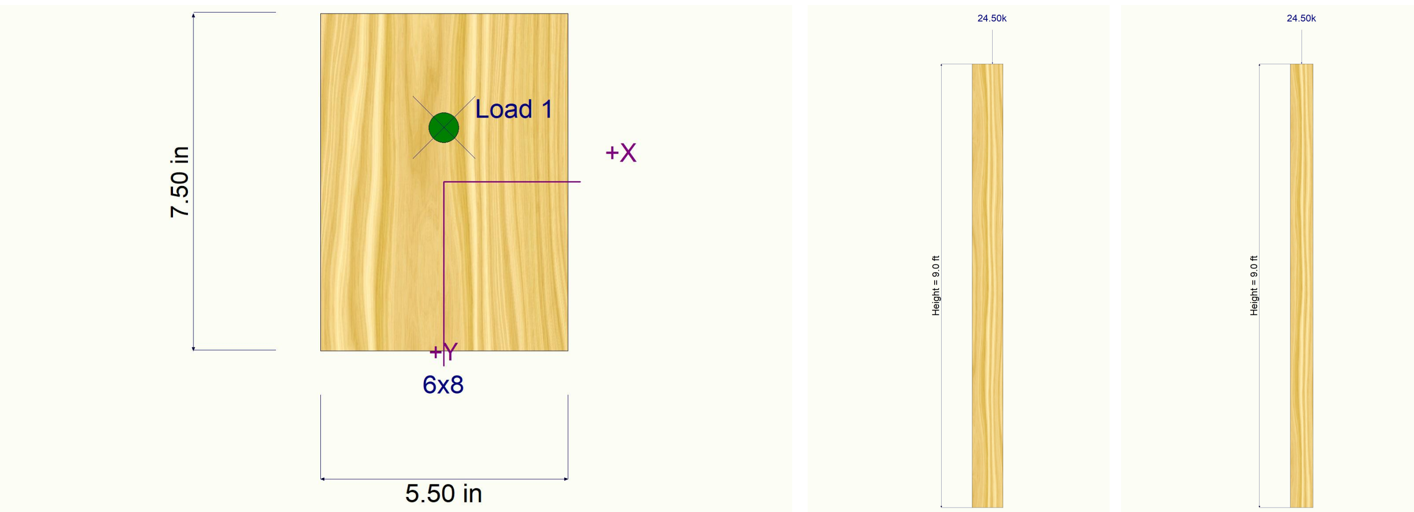
Description : Post FB1

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	k @ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments @ Base	k-ft @ Top	Mx - End Moments @ Base	@ Top
+D+L			-0.210	0.210	18.980				
+D+S			-0.190	0.190	17.180				
+D+0.750L			-0.189	0.189	17.130				
+D+0.750L+0.750S			-0.236	0.236	21.330				
+0.60D			-0.077	0.077	6.948				
L Only			-0.082	0.082	7.400				
S Only			-0.062	0.062	5.600				

Sketches



Wood Column

Description : PC/CV Posts

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities: Top & Bottom Pinned
Overall Column Height 9.0 ft
(Used for non-slender calculations)

Wood Species: Hem-Fir
Wood Grade: No.1

Fb +	975 psi	Fv	140 psi
Fb -	975 psi	Ft	650 psi
Fc - Prll	850 psi	Density	26.84 pcf
Fc - Perp	405 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1300	1300	1300 ksi
Minimum	470	470	

Wood Section Name: **6x6**
Wood Grading/Manuf.: Graded Lumber
Wood Member Type: Sawn

Exact Width	**5.50** in
Exact Depth	**5.50** in
Area	30.250 in^2
Ix	76.255 in^4
Iy	76.255 in^4
Incising Factors :	
for Bending	0.80
for Elastic Modulus	0.95

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 9.0 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 50.744 lbs * Dead Load Factor
AXIAL LOADS . . .
PCB1 Rxn: Axial Load at 9.0 ft, Yecc = 1.20 in, D = 0.90, S = 1.30 k
PCB1 Rxn: Axial Load at 9.0 ft, Yecc = 1.20 in, D = 0.90, S = 1.30 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3070** : 1

Load Combination	+D+S
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	8.940 ft
At maximum location values are . . .	
Applied Axial	4.451 k
Applied Mx	-0.4370 k-ft
Applied My	0.0 k-ft
Fc : Allowable	601.40 psi

PASS Maximum Shear Stress Ratio = **0.01882** : 1

Load Combination	+D+S
Location of max.above base	9.0 ft
Applied Design Shear	2.424 psi
Allowable Shear	128.80 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.04889 k	Bottom along Y-Y	0.04889 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y -0.04232 in at 5.255 ft above base
for load combination : +D+S
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
D Only	0.900	0.830	0.1319	PASS	8.940 ft	0.009839	PASS	9.0 ft
+D+S	1.150	0.769	0.3070	PASS	8.940 ft	0.01882	PASS	9.0 ft
+D+0.750S	1.150	0.769	0.2491	PASS	8.940 ft	0.01604	PASS	9.0 ft
+0.60D	1.600	0.662	0.05099	PASS	8.940 ft	0.003321	PASS	9.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only			-0.020	0.020	1.851				
+D+S			-0.049	0.049	4.451				

Wood Column

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

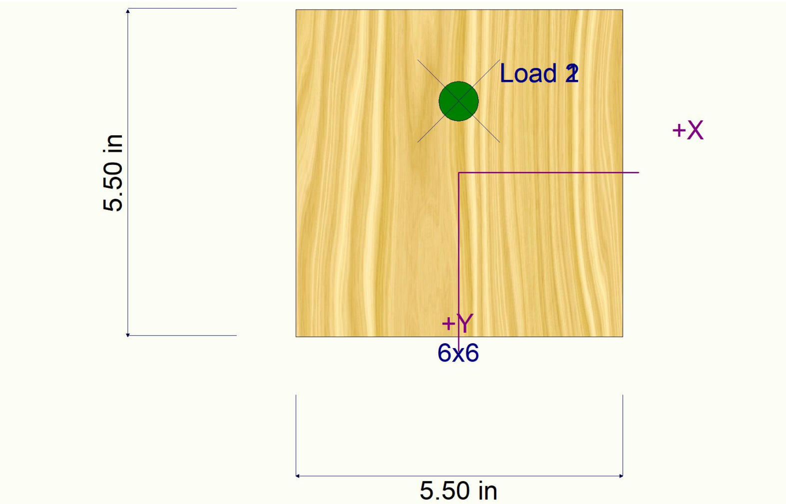
Description : PC/CV Posts

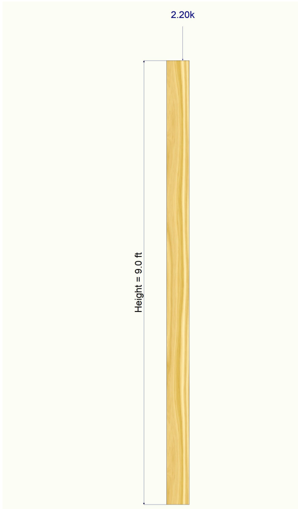
Maximum Reactions

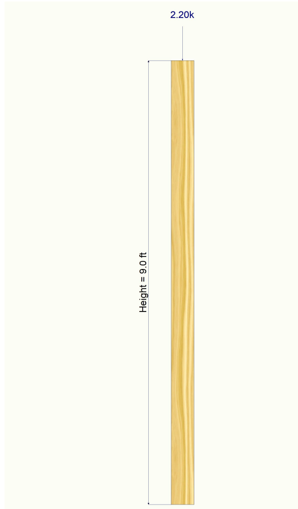
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+0.750S			-0.042	0.042	3.801				
+0.60D			-0.012	0.012	1.110				
S Only			-0.029	0.029	2.600				

Sketches







Guard Rail Post Design:

Define lateral applied post force:

$P_p := max[200 \cdot lbf, (50 \cdot plf) \cdot 4 \cdot ft]$ $\quad P_p = 200.00 \cdot lbf$

Define support depth: $d_j := 10.0 \cdot in$

Define tension bolt edge distance: $d_{eb} := 2.0 \cdot in$

Define post height (perp. to force): $h_p := 3.5 \cdot in$

Define post width (parallel to force): $b_p := 5.5 \cdot in$

Define force height: $H_p := 37.5 \cdot in$

Post section modulus,

$$S_p := \frac{b_p}{6} \cdot h_p^{\,2} \qquad S_p = 11.23 \cdot in^3$$

Define tension capacity of bolts [Simpson DTT2Z in 1.5" Member]: $T_{cap} := 1825 \cdot lbf$

Define nominal bending strength of post [#2 PT HF]: $F_{bp} := 850 \cdot psi$

Define factors:

$C_{D\_p} := 1.0$	$C_{M\_p} := 1.0$	$C_{t\_p} := 1.0$
$C_{L\_p} := 1.0$	$C_{F\_p} := 1.3$	$C_{fu\_p} := 1.05$
$C_{i\_p} := 0.80$	$C_{r\_p} := 1.0$	

Adjusted bending stress,

$$F'_{bp} := F_{bp} \cdot C_{D\_p} \cdot C_{M\_p} \cdot C_{t\_p} \cdot C_{L\_p} \cdot C_{F\_p} \cdot C_{fu\_p} \cdot C_{i\_p} \cdot C_{r\_p} \qquad F'_{bp} = 928.20 \cdot psi$$

Applied post moment,

$$M_p := P_p \cdot \left(H_p + d_{eb}\right) \qquad M_p = 658.33\, ft \cdot lbf$$

Applied bending stress,

$$f_{bp} := \frac{M_p}{S_p} \qquad f_{bp} = 703.53 \cdot psi \qquad \frac{f_{bp}}{F'_{bp}} = 0.76$$

Applied bolt moment,

$$M_b := P_p \cdot \left(H_p + 0.5 \cdot d_j\right) \qquad M_b = 708.33\, ft \cdot lbf$$

Bolt bending depth,

$$d_b := \left(d_j - d_{eb}\right) - \frac{1}{2} \cdot \left[\frac{1}{3} \cdot \left(d_j - d_{eb}\right)\right] \qquad d_b = 6.67 \cdot in$$

Applied bolt tension force,

$$T_b := \frac{M_b}{d_b} \qquad T_b = 1275.00 \cdot lbf \qquad \frac{T_b}{T_{cap}} = 0.70$$

Concrete shear load,

$$V_{uc} := 1.6 \cdot P_p \qquad V_{uc} = 320.00 \cdot lbf$$

Concrete tension load,

$$T_{uc} := 1.6 \cdot P_p \qquad T_{uc} = 320.00 \cdot lbf$$

Concrete moment,

$$M_{uc} := 1.6 \cdot M_b \qquad M_{uc} = 1133.33\, ft \cdot lbf$$

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Anchor Designer™ Software
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Company:		Date:	5/4/2020
Engineer:		Page:	1/6
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

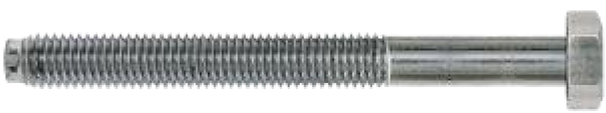
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 0.500
Effective Embedment depth, h_{ef} (inch): 4.250
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 5.50
C_{min} (inch): 1.01
S_{min} (inch): 2.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
State: Cracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 5.50 x 10.00 x 3.50

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 1/2"Ø Heavy Hex Bolt, F1554 Gr. 36

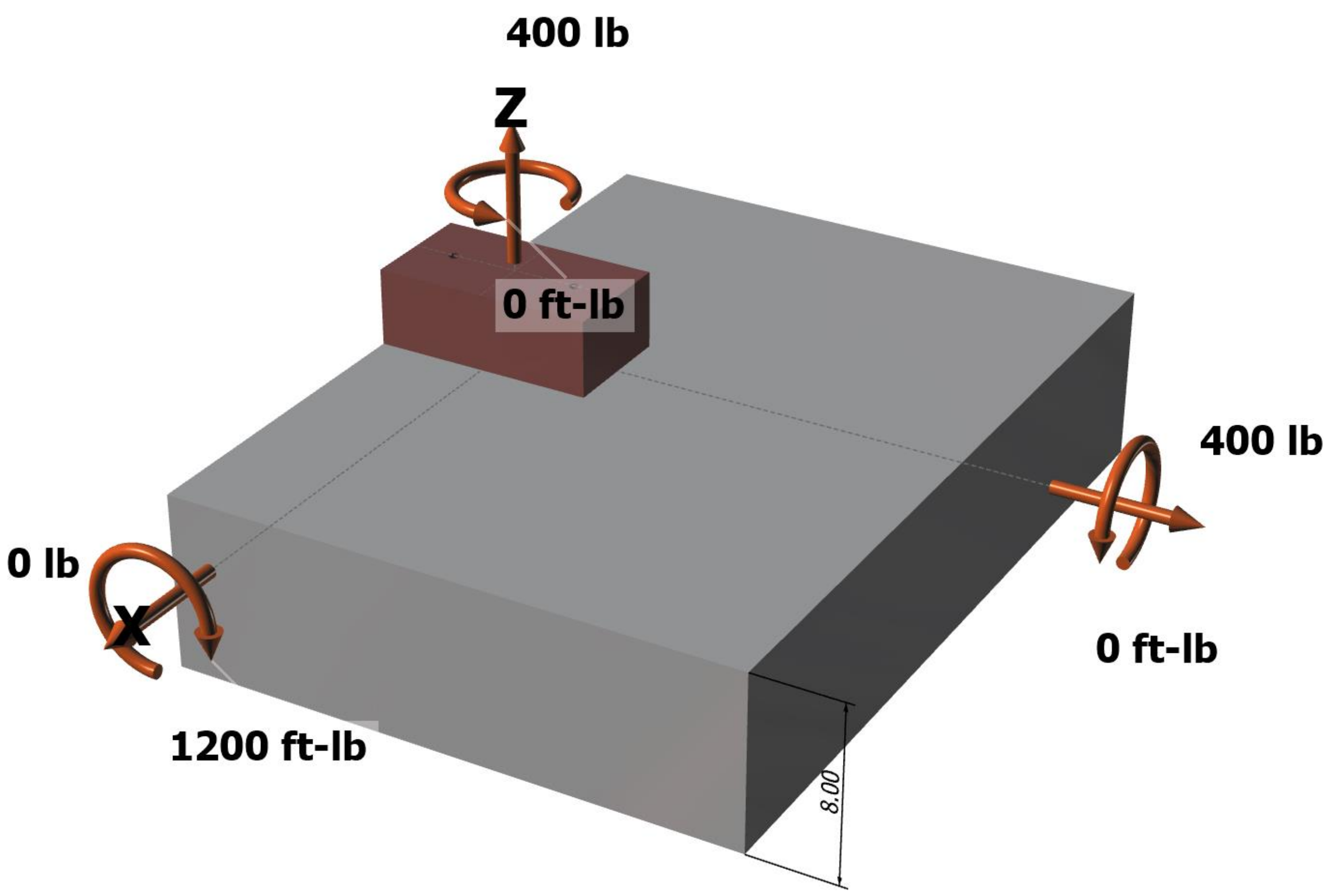


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 400
V_{uax} [lb]: 0
V_{uay} [lb]: 400
M_{ux} [ft-lb]: -1200
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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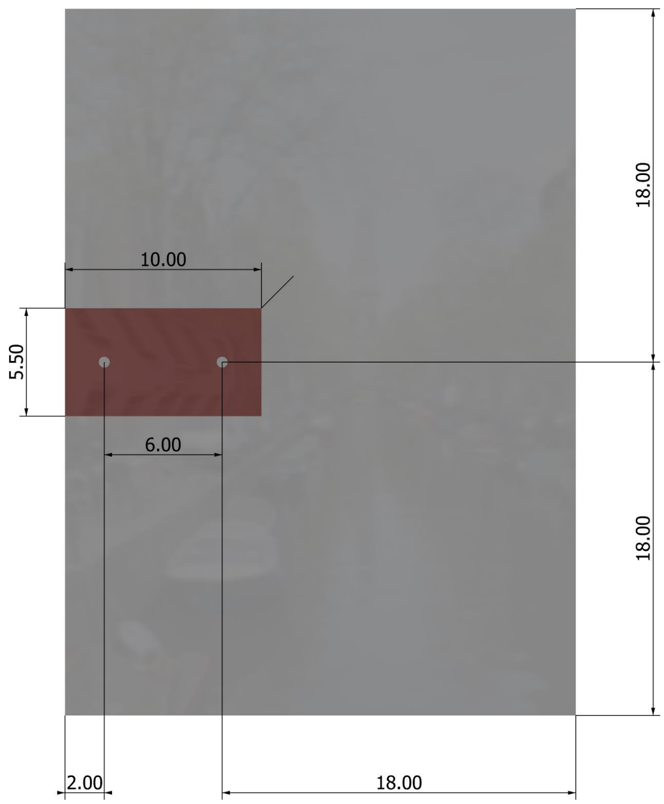
Company:		Date:	5/4/2020
Engineer:		Page:	3/6
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>



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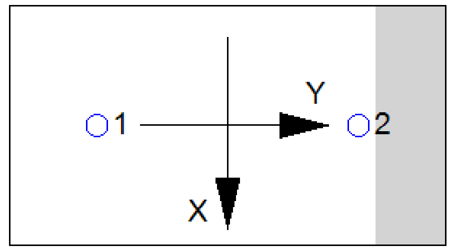
Company:		Date:	5/4/2020
Engineer:		Page:	4/6
Project:			
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	2145.5	0.0	200.0	200.0
2	124.1	0.0	200.0	200.0
Sum	2269.5	0.0	400.0	400.0

Maximum concrete compression strain (‰): 0.10
Maximum concrete compression stress (psi): 417
Resultant tension force (lb): 2270
Resultant compression force (lb): 1870
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 2.67
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
8235	0.75	6176

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	3000	4.250	11517

$\phi N_{cbg} = \phi\, (A_{Nc} / A_{Nco})\, \Psi_{ec,N}\, \Psi_{ed,N}\, \Psi_{c,N}\, \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
183.28	162.56	2.00	0.705	0.794	1.00	1.000	11517	0.70	5086

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.47	3000	0.70	7846

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Company:		Date:	5/4/2020
Engineer:		Page:	5/6
Project:			
Address:			
Phone:			
E-mail:			

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
4940	1.0	0.65	3211

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
4.00	0.500	1.00	3000	12.00	17082

$\phi V_{cby} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cby} (lb)
288.00	648.00	1.000	1.000	1.500	17082	0.70	7972

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
4.00	0.500	1.00	3000	12.00	17082

$\phi V_{cbgy} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
208.00	648.00	1.000	1.000	1.000	1.500	17082	0.70	11514

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	183.28	162.56	1.000	0.794	1.000	1.000	11517	0.70	14437

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	2145	6176	0.35	Pass
Concrete breakout	**2270**	**5086**	**0.45**	**Pass (Governs)**
Pullout	2145	7846	0.27	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	**200**	**3211**	**0.06**	**Pass (Governs)**
T Concrete breakout y+	400	7972	0.05	Pass
|| Concrete breakout x-	400	11514	0.03	Pass
Pryout	400	14437	0.03	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.45	0.00	44.6%	1.0	Pass

1/2"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 4.250 inch meets the selected design criteria.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Designer must exercise own judgement to determine if this design is suitable.

STUD DESIGN

Nominal compr. perp. stress for DF [NDS-15 Suppl., Table 4A]: $F_{cL\_H} := 405 \cdot psi$

Nominal compr. perp. stress for DF [NDS-15 Suppl., Table 4A]: $F_{cL\_D} := 625 \cdot psi$

Nominal compr. perp. stress for GLB [NDS-15 Suppl., Table 5A]: $F_{cL\_G} := 650 \cdot psi$

Nominal compr. perp. stress for LVL [Trus Joist]: $F_{cL\_L} := 750 \cdot psi$

Define double top plate to sill plate heights: $H_s := \begin{pmatrix} 9 \\ 9 \\ 9 \end{pmatrix} \cdot ft + \begin{pmatrix} 1.125 \\ 1.125 \\ 1.125 \end{pmatrix} \cdot in$

Dead Loads:

Roof dead load: $DL_r := 17 \cdot psf$

Floor dead load: $DL_f := 15 \cdot psf$

Exterior wall dead load: $DL_{ew} := 15 \cdot psf$

Interior wall dead load: $DL_{iw} := 12 \cdot psf$

Live Loads:

Define wall live load: $LL_w := 5 \cdot psf$

Roof live load: $LL_r := 20 \cdot psf$

Floor live load: $LL_f := 40 \cdot psf$

Main floor / slab on grade: $LL_m := 100 \cdot psf$

Snow Loads:

Define sloped snow: $SL_s := 25 \cdot psf$

Wind Loads:

Define LRFD horizontal C&C wind loads: $WL_{st} := 42\ psf$

Wall Studs:

Define walls & vector data:

$Wall := augment("2F\ 2x6\ Ext.\ Wall", "MF\ 2x6\ Ext.\ Wall", "MF\ 2x4\ Int.\ Wall", "B\ 2x4\ Int.\ Wall", "Garage\ Wall")$

$\boxed{w := length\left(Wall^T\right)}$ $w = 5$ $\boxed{ws := 1 .. w}$

Define typical stud width: $b_{st} := 1.5 \cdot in$

Define 2x4 stud depth: $d_{2x4} := 3.5 \cdot in$

Define 2x6 stud depth: $d_{2x6} := 5.5 \cdot in$

Define studs roof tributary area:

$A_{Tws\_rX} := augment[(24 \cdot in) \cdot 23.0 \cdot ft, (24 \cdot in) \cdot (23.0 \cdot ft + 5.0 \cdot ft), 0, 0, (24 \cdot in) \cdot 16.0 \cdot ft]$

$A_{Tws\_r} := stack\left(Wall, A_{Tws\_rX}\right)$

$$A_{Tws\_r} = \begin{pmatrix} "2F\ 2x6\ Ext.\ Wall" & "MF\ 2x6\ Ext.\ Wall" & "MF\ 2x4\ Int.\ Wall" & "B\ 2x4\ Int.\ Wall" & "Garage\ Wall" \\ 46.00 & 56.00 & 0.00 & 0.00 & 32.00 \end{pmatrix} \cdot ft^2$$

Define studs floor tributary area:

$A_{Tws\_fX} := augment[0, 2 \cdot (16 \cdot in) \cdot 10.667 \cdot ft, 1 \cdot (16 \cdot in) \cdot 20.5 \cdot ft, 2 \cdot (16 \cdot in) \cdot 20.5 \cdot ft, 0]$

$A_{Tws\_f} := stack\left(Wall, A_{Tws\_fX}\right)$

$$A_{Tws\_f} = \begin{pmatrix} "2F\ 2x6\ Ext.\ Wall" & "MF\ 2x6\ Ext.\ Wall" & "MF\ 2x4\ Int.\ Wall" & "B\ 2x4\ Int.\ Wall" & "Garage\ Wall" \\ 0.00 & 28.45 & 27.33 & 54.67 & 0.00 \end{pmatrix} \cdot ft^2$$

Define studs wall tributary area:

$A_{Tws\_wX} := augment[1 \cdot (16 \cdot in) \cdot 9.0 \cdot ft, 2 \cdot (16 \cdot in) \cdot 9.0 \cdot ft, 1 \cdot (16 \cdot in) \cdot 9.0 \cdot ft, 2 \cdot (16 \cdot in) \cdot 9.0 \cdot ft, 1 \cdot (16 \cdot in) \cdot 9.0 \cdot ft]$

$A_{Tws\_w} := stack\left(Wall, A_{Tws\_wX}\right)$

$$A_{Tws\_w} = \begin{pmatrix} "2F\ 2x6\ Ext.\ Wall" & "MF\ 2x6\ Ext.\ Wall" & "MF\ 2x4\ Int.\ Wall" & "B\ 2x4\ Int.\ Wall" & "Garage\ Wall" \\ 12.00 & 24.00 & 12.00 & 24.00 & 12.00 \end{pmatrix} \cdot ft^2$$

Define studs cross sectional area for 1 stud:

$A_{z\_stX} := augment(8.25, 8.25, 5.25, 5.25, 8.25) \cdot in^2$

$A_{z\_st} := stack\left(Wall, A_{z\_stX}\right)$

$$A_{z\_st} = \begin{pmatrix} "2F\ 2x6\ Ext.\ Wall" & "MF\ 2x6\ Ext.\ Wall" & "MF\ 2x4\ Int.\ Wall" & "B\ 2x4\ Int.\ Wall" & "Garage\ Wall" \\ 8.25 & 8.25 & 5.25 & 5.25 & 8.25 \end{pmatrix} \cdot in^2$$

Define number of stacked studs:

$n_{ws\_X} := augment(1, 1, 2, 3, 1)$

$n_{ws} := stack\left(Wall, n_{ws\_X}\right)$

$$n_{ws} = \begin{pmatrix} "2F\ 2x6\ Ext.\ Wall" & "MF\ 2x6\ Ext.\ Wall" & "MF\ 2x4\ Int.\ Wall" & "B\ 2x4\ Int.\ Wall" & "Garage\ Wall" \\ 1 & 1 & 2 & 3 & 1 \end{pmatrix}$$

Typical distributed wind load for studs,

$w_{WL\_st} := WL_{st} \cdot (16 \cdot in)$ $\qquad w_{WL\_st} = 0.06 \cdot klf$

Wall stud distributed live load,

$w_{LL\_st} := LL_{w} \cdot (16 \cdot in)$ $\qquad w_{LL\_st} = 0.01 \cdot klf$

Define wall dead load:

$DL_{w\_X} := augment(DL_{ew}, DL_{ew}, DL_{iw}, DL_{iw}, DL_{ew})$

$DL_{w} := stack(Wall, DL_{w\_X})$

$$DL_{w} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 15 & 15 & 12 & 12 & 15 \end{pmatrix} psf$$

Dead axial force on studs,

$P_{DLwsX_{1,ws}} := DL_{r} \cdot A_{Tws\_rX_{1,ws}} + DL_{f} \cdot A_{Tws\_fX_{1,ws}} + DL_{w\_X_{1,ws}} \cdot A_{Tws\_wX_{1,ws}}$

$P_{DLws} := stack(Wall, P_{DLwsX})$

$$P_{DLws} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 0.96 & 1.74 & 0.55 & 1.11 & 0.72 \end{pmatrix} kip$$

Live axial force on studs,

$P_{LLwsX} := LL_{f} \cdot A_{Tws\_fX}$

$P_{LLws} := stack(Wall, P_{LLwsX})$

$$P_{LLws} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 0.00 & 1.14 & 1.09 & 2.19 & 0.00 \end{pmatrix} kip$$

Snow axial force on studs,

$P_{SLwsX} := SL_{s} \cdot A_{Tws\_rX}$

$P_{SLws} := stack(Wall, P_{SLwsX})$

$$P_{SLws} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 1.15 & 1.40 & 0.00 & 0.00 & 0.80 \end{pmatrix} kip$$

Total cumulative axial force on studs,

$$P_{TLwsX_{1,ws}} := P_{DLwsX_{1,ws}} + \left| \begin{array}{l} Y1_{1,ws} \leftarrow max\left(P_{LLwsX_{1,ws}}, P_{SLwsX_{1,ws}}\right) \\ Y2_{1,ws} \leftarrow 0.75 \cdot \left(P_{LLwsX_{1,ws}} + P_{SLwsX_{1,ws}}\right) \\ max\left(Y1_{1,ws}, Y2_{1,ws}\right) \end{array} \right.$$

$P_{TLws} := stack(Wall, P_{TLwsX})$

$$P_{TLws} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 2.11 & 3.64 & 1.65 & 3.29 & 1.52 \end{pmatrix} kip$$

Applied stress perpendicular to top of studs,

$$f_{cL\_ws\_tX_{1,ws}} := \frac{P_{TLwsX_{1,ws}}}{A_{z\_stX_{1,ws}} \cdot n_{ws\_X_{1,ws}}}$$

$$f_{cL\_ws\_t} := stack(Wall, f_{cL\_ws\_tX})$$

$$f_{cL\_ws\_t} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 256.0 & 441.5 & 156.9 & 209.2 & 184.7 \end{pmatrix} \cdot psi$$

Adjusted compressive perp. stress at top of stud [NDS-15, Table 4.3.1 & Sect. 4.2.6],

$$F'_{cL\_ws\_tX} := 0.73 \cdot augment(F_{cL\_D}, F_{cL\_D}, F_{cL\_D}, F_{cL\_D}, F_{cL\_D})$$

$$F'_{cL\_ws\_t} := stack(Wall, F'_{cL\_ws\_tX})$$

$$F'_{cL\_ws\_t} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 456.25 & 456.25 & 456.25 & 456.25 & 456.25 \end{pmatrix} \cdot psi$$

Compressive perp. stress interaction at top of stud,

$$INT_{cL\_ws\_tX} := \frac{f_{cL\_ws\_tX}}{F'_{cL\_ws\_tX}}$$

$$INT_{cL\_ws\_t} := stack(Wall, INT_{cL\_ws\_tX})$$

$$INT_{cL\_ws\_t} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 0.56 & 0.97 & 0.34 & 0.46 & 0.40 \end{pmatrix}$$

Applied stress perpendicular to bottom of studs,

$$f_{cL\_ws\_bX_{1,ws}} := \frac{P_{TLwsX_{1,ws}}}{A_{z\_stX_{1,ws}} \cdot n_{ws\_X_{1,ws}}}$$

$$f_{cL\_ws\_b} := stack(Wall, f_{cL\_ws\_bX})$$

$$f_{cL\_ws\_b} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 256.0 & 441.5 & 156.9 & 209.2 & 184.7 \end{pmatrix} \cdot psi$$

Adjusted compressive perp. stress at bottom of stud [NDS-15, Table 4.3.1 & Sect. 4.2.6],

$$F'_{cL\_ws\_bX} := 0.73 \cdot augment(F_{cL\_H}, F_{cL\_H}, F_{cL\_D}, F_{cL\_H}, F_{cL\_H})$$

$$F'_{cL\_ws\_b} := stack(Wall, F'_{cL\_ws\_bX})$$

$$F'_{cL\_ws\_b} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 295.65 & 295.65 & 456.25 & 295.65 & 295.65 \end{pmatrix} \cdot psi$$

Compressive perp. stress interaction at bottom of stud,

$$INT_{cL\_ws\_bX} := \frac{f_{cL\_ws\_bX}}{F'_{cL\_ws\_bX}}$$

$$INT_{cL\_ws\_b} := stack(Wall, INT_{cL\_ws\_bX})$$

$$INT_{cL\_ws\_b} = \begin{pmatrix} \text{"2F 2x6 Ext. Wall"} & \text{"MF 2x6 Ext. Wall"} & \text{"MF 2x4 Int. Wall"} & \text{"B 2x4 Int. Wall"} & \text{"Garage Wall"} \\ 0.87 & 1.49 & 0.34 & 0.71 & 0.62 \end{pmatrix}$$

King Studs:

Define headers & vector data:

$King := augment(\text{"Roof Hdr<=6ft"}, \text{"Roof Hdr>6ft"}, \text{"Floor Hdr"}, \text{"RB1"}, \text{"RB2"})$

$\boxed{ks := length(King^T)}$ $\qquad ks = 5$

Define length of header:

$L_{hX} := augment(6.0, 8.0, 14.0, 9.0, 18.0)\cdot ft$

$L_h := stack(King, L_{hX})$

$$L_h = \begin{pmatrix} \text{"Roof Hdr<=6ft"} & \text{"Roof Hdr>6ft"} & \text{"Floor Hdr"} & \text{"RB1"} & \text{"RB2"} \\ 6.00 & 8.00 & 14.00 & 9.00 & 18.00 \end{pmatrix} ft$$

Distributed wind load on king studs,

$w_{WL\_ksX} := (WL_{st})\cdot[0.5\cdot(16\cdot in) + 0.5\cdot L_{hX}]$

$w_{WL\_ks} := stack(King, w_{WL\_ksX})$

$$w_{WL\_ks} = \begin{pmatrix} \text{"Roof Hdr<=6ft"} & \text{"Roof Hdr>6ft"} & \text{"Floor Hdr"} & \text{"RB1"} & \text{"RB2"} \\ 0.154 & 0.196 & 0.322 & 0.217 & 0.406 \end{pmatrix}\cdot klf$$

Define number of trimmer studs:

$n_{ks\_X} := augment(1, 2, 2, 2, 3)$

$n_{ks} := stack(King, n_{ks\_X})$

$$n_{ks} = \begin{pmatrix} \text{"Roof Hdr<=6ft"} & \text{"Roof Hdr>6ft"} & \text{"Floor Hdr"} & \text{"RB1"} & \text{"RB2"} \\ 1 & 2 & 2 & 2 & 3 \end{pmatrix}$$

King stud dead load,

$P_{DLksX} := augment(P_{DLwsX_{1,1}}, P_{DLwsX_{1,1}}, P_{DLwsX_{1,2}}, P_{DLwsX_{1,5}}, P_{DLwsX_{1,5}})$

$P_{DLks} := stack(King, P_{DLksX})$

$$P_{DLks} = \begin{pmatrix} \text{"Roof Hdr<=6ft"} & \text{"Roof Hdr>6ft"} & \text{"Floor Hdr"} & \text{"RB1"} & \text{"RB2"} \\ 0.96 & 0.96 & 1.74 & 0.72 & 0.72 \end{pmatrix} kip$$

King stud live load,

$P_{LLksX} := augment(P_{LLwsX_{1,1}}, P_{LLwsX_{1,1}}, P_{LLwsX_{1,2}}, P_{LLwsX_{1,5}}, P_{LLwsX_{1,5}})$

$P_{LLks} := stack(King, P_{LLksX})$

$$P_{LLks} = \begin{pmatrix} \text{"Roof Hdr<=6ft"} & \text{"Roof Hdr>6ft"} & \text{"Floor Hdr"} & \text{"RB1"} & \text{"RB2"} \\ 0.00 & 0.00 & 1.14 & 0.00 & 0.00 \end{pmatrix} kip$$

King stud snow load,

$P_{SLksX} := augment(P_{SLwsX_{1,1}}, P_{SLwsX_{1,1}}, P_{SLwsX_{1,2}}, P_{SLwsX_{1,5}}, P_{SLwsX_{1,5}})$

$P_{SLks} := stack(King, P_{SLksX})$

$$P_{SLks} = \begin{pmatrix} \text{"Roof Hdr<=6ft"} & \text{"Roof Hdr>6ft"} & \text{"Floor Hdr"} & \text{"RB1"} & \text{"RB2"} \\ 1.15 & 1.15 & 1.40 & 0.80 & 0.80 \end{pmatrix} kip$$

Wood Column

File = C:\Users\JesseC\Desktop\Enercalc\2020-0196-Stud Design.ec6 .
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Description : 2x6 Walls - Enveloping Design

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 9.0 ft
(Used for non-slender calculations)
Wood Species Douglas Fir - Larch
Wood Grade No.2

Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name **2x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**1.50** in
Exact Depth	**5.50** in
Area	8.250 in^2
Ix	20.797 in^4
Iy	1.547 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	Yes

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling about X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 16.088 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = -1.20 in, D = 1.80, L = 1.20, S = 1.40 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, L = 0.010, W = 0.060 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.8276** : 1
	Load Combination	+D+0.750L+0.750S
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
	Location of max.above base	8.940 ft
	At maximum location values are . . .	
	Applied Axial	3.766 k
	Applied Mx	0.3745 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	976.05 psi
PASS	Maximum Shear Stress Ratio =	**0.1243** : 1
	Load Combination	+D+0.750L+0.750S+0.450W
	Location of max.above base	0.0 ft
	Applied Design Shear	35.803 psi
	Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.270 k	Bottom along Y-Y	0.270 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.2691 in at 4.530 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.663	0.3467	PASS	8.940 ft	0.02245	PASS	9.0 ft
+D+L	1.000	0.625	0.6690	PASS	7.852 ft	0.07913	PASS	0.0 ft
+D+S	1.150	0.572	0.6356	PASS	8.940 ft	0.03123	PASS	9.0 ft
+D+0.750L	1.250	0.540	0.4564	PASS	8.517 ft	0.05152	PASS	0.0 ft
+D+0.750L+0.750S	1.150	0.572	0.8276	PASS	8.940 ft	0.06624	PASS	0.0 ft
+D+0.60W	1.600	0.448	0.3942	PASS	5.074 ft	0.1149	PASS	0.0 ft
+D+0.750L+0.450W	1.600	0.448	0.5214	PASS	5.376 ft	0.1170	PASS	0.0 ft
+D+0.750L+0.750S+0.450W	1.600	0.448	0.7435	PASS	5.678 ft	0.1243	PASS	0.0 ft
+0.60D+0.60W	1.600	0.448	0.3110	PASS	4.832 ft	0.1098	PASS	0.0 ft

Wood Column

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Description : 2x6 Walls - Enveloping Design

Load Combination Results

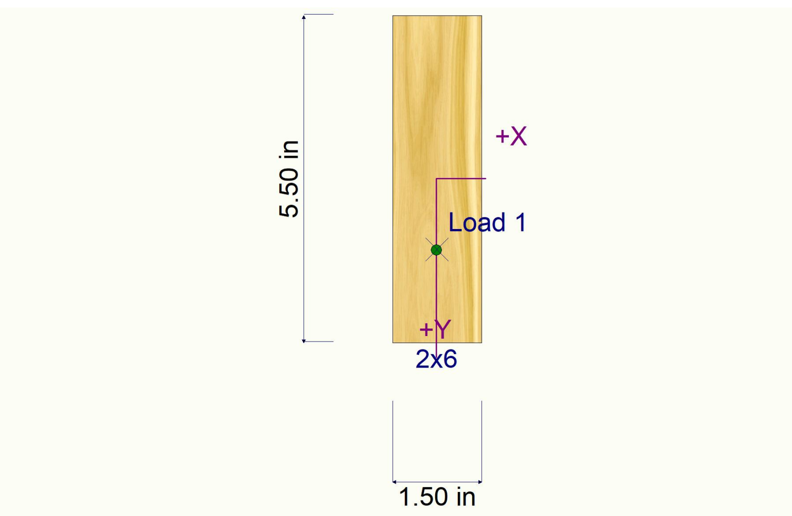
Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.448	0.1241	PASS	0.0 ft	0.007576	PASS	9.0 ft

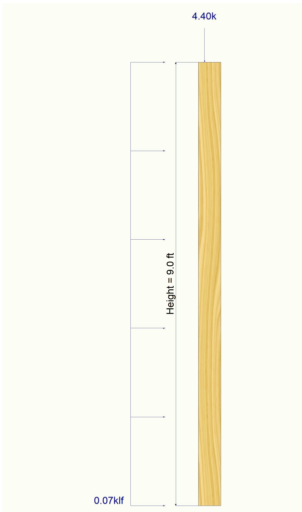
Maximum Reactions

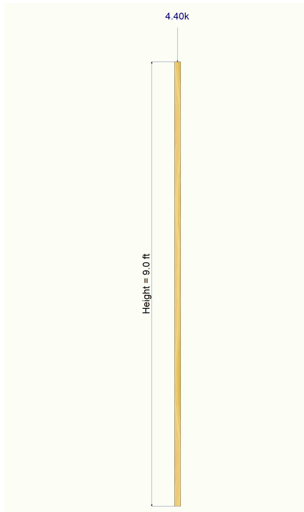
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only			0.020	-0.020	1.816				
+D+L			0.078	0.012	3.016				
+D+S			0.036	-0.036	3.216				
+D+0.750L			0.064	0.004	2.716				
+D+0.750L+0.750S			0.075	-0.008	3.766				
+D+0.60W			0.182	0.142	1.816				
+D+0.750L+0.450W			0.185	0.125	2.716				
+D+0.750L+0.750S+0.450W			0.197	0.114	3.766				
+0.60D+0.60W			0.174	0.150	1.090				
+0.60D			0.012	-0.012	1.090				
L Only			0.058	0.032	1.200				
S Only			0.016	-0.016	1.400				
W Only			0.270	0.270					

Sketches







Wood Column

Description : 2x6 Walls - Main Floor

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design	Wood Section Name	**2x6**
End Fixities	Top & Bottom Pinned	Wood Grading/Manuf.	Graded Lumber
Overall Column Height	9.0 ft	Wood Member Type	Sawn

(Used for non-slender calculations)

Wood Species	Douglas Fir - Larch		
Wood Grade	No.2		
Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Exact Width	**1.50** in
Exact Depth	**5.50** in
Area	8.250 in^2
Ix	20.797 in^4
Iy	1.547 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	Yes

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling about X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 16.088 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = 1.20 in, D = 0.60, L = 1.10 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, L = 0.010 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.2859** : 1
	Load Combination	+D+L
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
	Location of max.above base	8.940 ft
	At maximum location values are . . .	
	Applied Axial	1.716 k
	Applied Mx	-0.1662 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	927.65 psi
PASS	Maximum Shear Stress Ratio =	**0.06453** : 1
	Load Combination	+D+L
	Location of max.above base	9.0 ft
	Applied Design Shear	11.616 psi
	Allowable Shear	180.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.06389 k	Bottom along Y-Y	0.03278 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.01656 in at 3.685 ft above base
for load combination : L Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.663	0.09021	PASS	8.940 ft	0.007482	PASS	9.0 ft
+D+L	1.000	0.625	0.2859	PASS	8.940 ft	0.06453	PASS	9.0 ft
+D+0.750L	1.250	0.540	0.1837	PASS	8.940 ft	0.04007	PASS	9.0 ft
+0.60D	1.600	0.448	0.04212	PASS	0.0 ft	0.002525	PASS	9.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only			-0.007	0.007	0.616				

Wood Column

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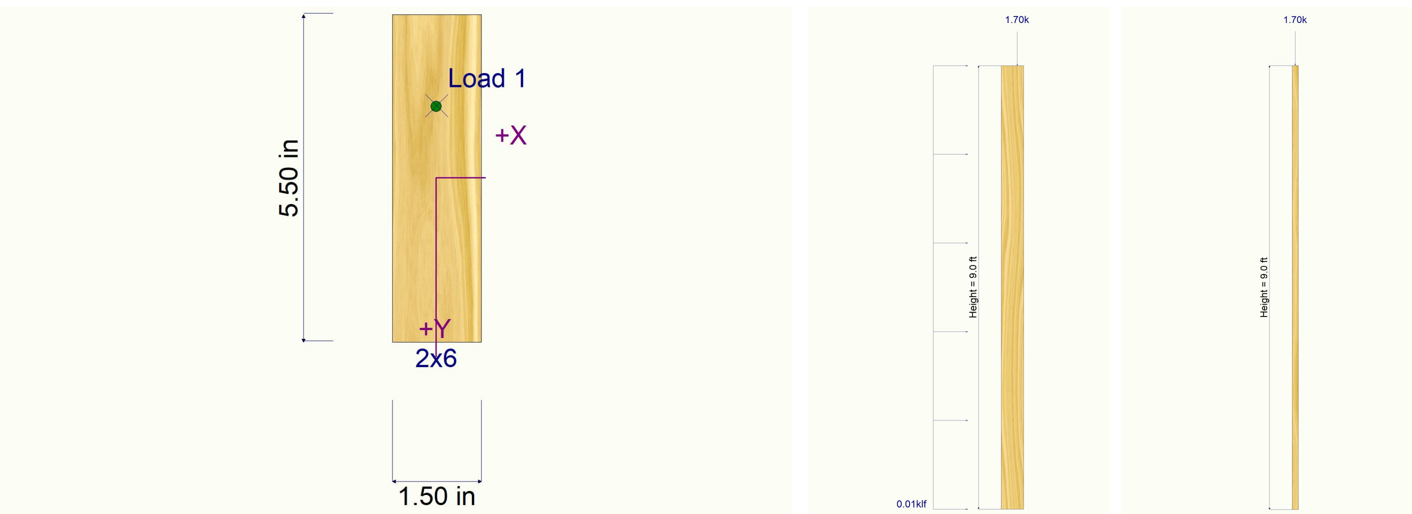
Description : 2x6 Walls - Main Floor

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	k @ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments @ Base	k-ft @ Top	Mx - End Moments @ Base	@ Top
+D+L			0.026	0.064	1.716				
+D+0.750L			0.018	0.050	1.441				
+0.60D			-0.004	0.004	0.370				
L Only			0.033	0.057	1.100				

Sketches



Wood Column

Description : 2x6 Walls - Daylight Basement

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 9.0 ft
(Used for non-slender calculations)
Wood Species Douglas Fir - Larch
Wood Grade No.2

Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name **2x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**1.50** in
Exact Depth	**5.50** in
Area	8.250 in^2
Ix	20.797 in^4
Iy	1.547 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	Yes

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling about X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 16.088 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = 1.20 in, D = 1.10, L = 2.20 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, L = 0.010 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.7558** : 1
	Load Combination	+D+L
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
	Location of max.above base	8.940 ft
	At maximum location values are . . .	
	Applied Axial	3.316 k
	Applied Mx	-0.3251 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	927.65 psi
PASS	Maximum Shear Stress Ratio =	**0.08249** : 1
	Load Combination	+D+L
	Location of max.above base	9.0 ft
	Applied Design Shear	14.848 psi
	Allowable Shear	180.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.08167 k	Bottom along Y-Y	0.02056 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y -0.04750 in at 5.859 ft above base
for load combination : +D+L
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.663	0.1840	PASS	8.940 ft	0.01372	PASS	9.0 ft
+D+L	1.000	0.625	0.7558	PASS	8.940 ft	0.08249	PASS	9.0 ft
+D+0.750L	1.250	0.540	0.4630	PASS	8.940 ft	0.05196	PASS	9.0 ft
+0.60D	1.600	0.448	0.07629	PASS	0.0 ft	0.004630	PASS	9.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only			-0.012	0.012	1.116				

Wood Column

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

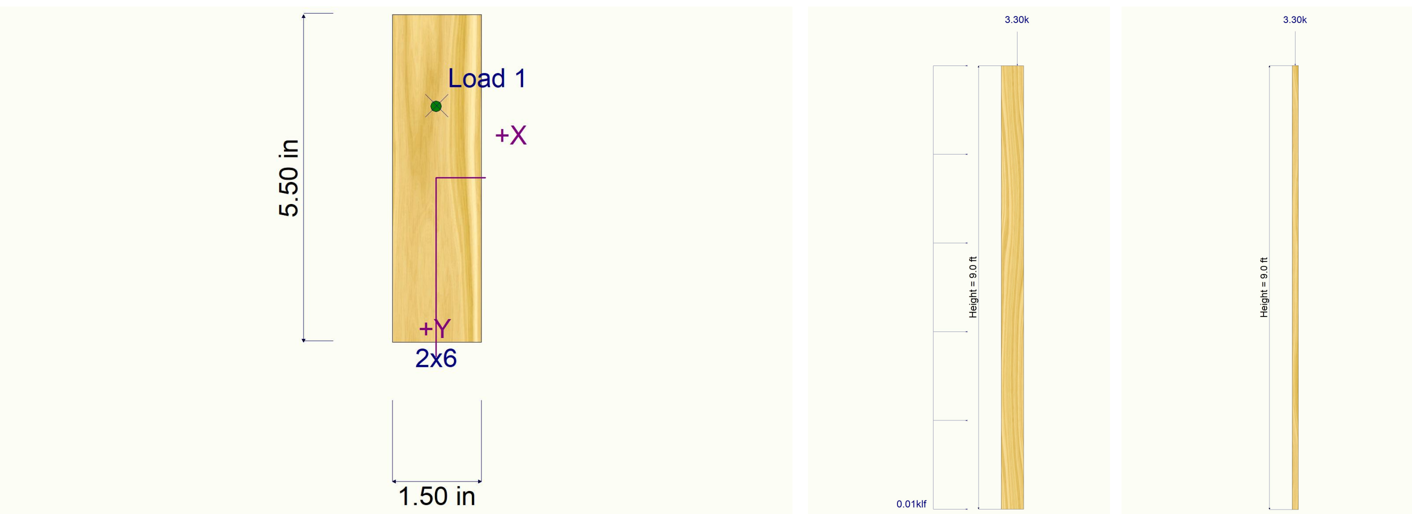
Description : 2x6 Walls - Daylight Basement

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	k @ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments @ Base	k-ft @ Top	Mx - End Moments @ Base	@ Top
+D+L			0.008	0.082	3.316				
+D+0.750L			0.003	0.064	2.766				
+0.60D			-0.007	0.007	0.670				
L Only			0.021	0.069	2.200				

Sketches



Wood Column

Description : 2nd Floor King Stud Enveloping Design - Header <= 6ft

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design	Wood Section Name	**2x6**
End Fixities	Top & Bottom Pinned	Wood Grading/Manuf.	Graded Lumber
Overall Column Height	9.0 ft	Wood Member Type	Sawn

(Used for non-slender calculations)

Wood Species	Douglas Fir - Larch		
Wood Grade	No.2		
Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Exact Width	**1.50** in
Exact Depth	**5.50** in
Area	8.250 in^2
Ix	20.797 in^4
Iy	1.547 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling about X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 16.088 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = -1.20 in, D = 1.0, S = 1.20 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, L = 0.010, W = 0.1540 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.9614** : 1
Load Combination	+D+0.750L+0.750S+0.450W
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	4.772 ft
At maximum location values are . . .	
Applied Axial	1.916 k
Applied Mx	0.8755 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,063.92 psi
PASS Maximum Shear Stress Ratio =	**0.2695** : 1
Load Combination	+D+0.60W
Location of max.above base	0.0 ft
Applied Design Shear	77.620 psi
Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.6930 k	Bottom along Y-Y	0.6930 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.6906 in at 4.530 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.663	0.1855	PASS	8.940 ft	0.01247	PASS	9.0 ft
+D+L	1.000	0.625	0.2547	PASS	5.617 ft	0.05668	PASS	0.0 ft
+D+S	1.150	0.572	0.4050	PASS	8.940 ft	0.02147	PASS	9.0 ft
+D+0.750L	1.250	0.540	0.1768	PASS	5.980 ft	0.03625	PASS	0.0 ft
+D+0.750L+0.750S	1.150	0.572	0.3480	PASS	7.309 ft	0.04819	PASS	0.0 ft
+D+0.60W	1.600	0.448	0.9418	PASS	4.591 ft	0.2695	PASS	0.0 ft
+D+0.750L+0.450W	1.600	0.448	0.7933	PASS	4.651 ft	0.2252	PASS	0.0 ft
+D+0.750L+0.750S+0.450W	1.600	0.448	0.9614	PASS	4.772 ft	0.2315	PASS	0.0 ft
+0.60D+0.60W	1.600	0.448	0.8755	PASS	4.591 ft	0.2667	PASS	0.0 ft

Wood Column

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

Description : 2nd Floor King Stud Enveloping Design - Header <= 6ft

Load Combination Results

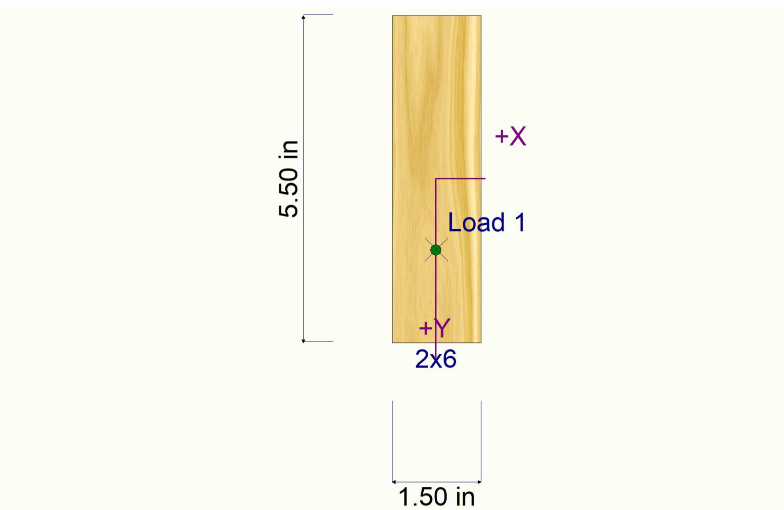
Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.448	0.06946	PASS	0.0 ft	0.004209	PASS	9.0 ft

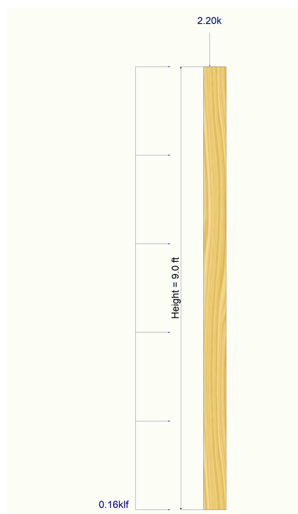
Maximum Reactions

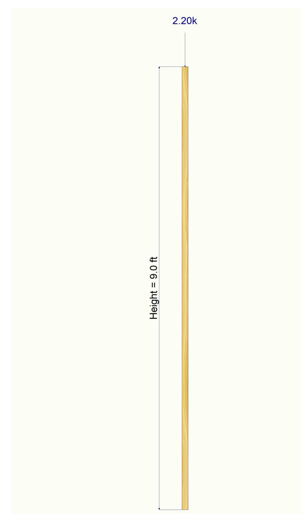
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only			0.011	-0.011	1.016				
+D+L			0.056	0.034	1.016				
+D+S			0.024	-0.024	2.216				
+D+0.750L			0.045	0.023	1.016				
+D+0.750L+0.750S			0.055	0.013	1.916				
+D+0.60W			0.427	0.405	1.016				
+D+0.750L+0.450W			0.357	0.334	1.016				
+D+0.750L+0.750S+0.450W			0.367	0.324	1.916				
+0.60D+0.60W			0.422	0.409	0.610				
+0.60D			0.007	-0.007	0.610				
L Only			0.045	0.045					
S Only			0.013	-0.013	1.200				
W Only			0.693	0.693					

Sketches







Wood Column

Description : 2nd Floor King Stud Enveloping Design - Headers > 6ft.

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height 9.0 ft
(Used for non-slender calculations)
Wood Species : Douglas Fir - Larch
Wood Grade : No.2

Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name: **2-2x6**
Wood Grading/Manuf.: Graded Lumber
Wood Member Type: Sawn

Exact Width	**3.0** in
Exact Depth	**5.50** in
Area	16.50 in^2
Ix	41.594 in^4
Iy	12.375 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling about X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 32.175 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = -1.20 in, D = 1.0, S = 1.20 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, L = 0.010, W = 0.1960 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.5575** : 1

Load Combination	+D+0.60W
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	4.591 ft
At maximum location values are . . .	
Applied Axial	1.032 k
Applied Mx	1.241 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,063.92 psi

PASS Maximum Shear Stress Ratio = **0.1706** : 1

Load Combination	+D+0.60W
Location of max.above base	0.0 ft
Applied Design Shear	49.119 psi
Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.8820 k	Bottom along Y-Y	0.8820 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.4395 in at 4.530 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.663	0.08381	PASS	8.940 ft	0.006235	PASS	9.0 ft
+D+L	1.000	0.625	0.1170	PASS	5.617 ft	0.02834	PASS	0.0 ft
+D+S	1.150	0.572	0.1639	PASS	8.940 ft	0.01074	PASS	9.0 ft
+D+0.750L	1.250	0.540	0.08056	PASS	5.980 ft	0.01813	PASS	0.0 ft
+D+0.750L+0.750S	1.150	0.572	0.1451	PASS	7.309 ft	0.02409	PASS	0.0 ft
+D+0.60W	1.600	0.448	0.5575	PASS	4.591 ft	0.1706	PASS	0.0 ft
+D+0.750L+0.450W	1.600	0.448	0.4586	PASS	4.591 ft	0.1394	PASS	0.0 ft
+D+0.750L+0.750S+0.450W	1.600	0.448	0.5113	PASS	4.711 ft	0.1426	PASS	0.0 ft
+0.60D+0.60W	1.600	0.448	0.5349	PASS	4.530 ft	0.1692	PASS	0.0 ft

Wood Column

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

Description : 2nd Floor King Stud Enveloping Design - Headers > 6ft.

Load Combination Results

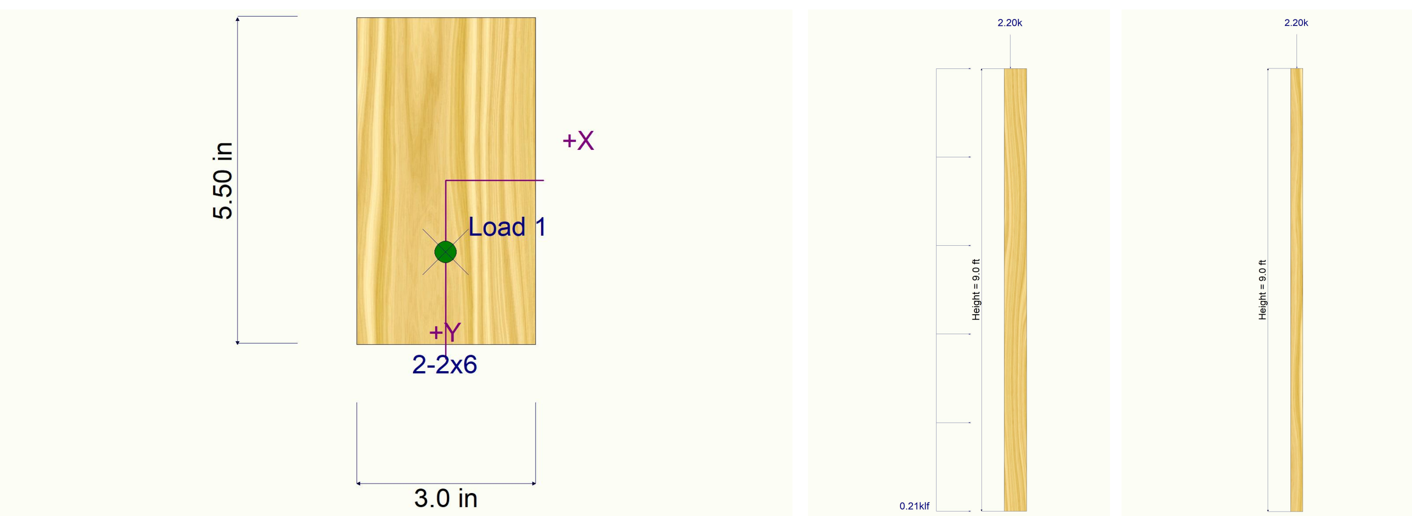
Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.448	0.03528	PASS	0.06040 ft	0.002104	PASS	9.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only			0.011	-0.011	1.032				
+D+L			0.056	0.034	1.032				
+D+S			0.024	-0.024	2.232				
+D+0.750L			0.045	0.023	1.032				
+D+0.750L+0.750S			0.055	0.013	1.932				
+D+0.60W			0.540	0.518	1.032				
+D+0.750L+0.450W			0.442	0.420	1.032				
+D+0.750L+0.750S+0.450W			0.452	0.410	1.932				
+0.60D+0.60W			0.536	0.523	0.619				
+0.60D			0.007	-0.007	0.619				
L Only			0.045	0.045					
S Only			0.013	-0.013	1.200				
W Only			0.882	0.882					

Sketches



Wood Column

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Software copyright ENERCALC, INC. 1983-2018, Build:10.18.12.13 .
Lic. # : KW-06005122
Licensee : MC SQUARED, INC.

Description : Main Floor King Stud Enveloping Design

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design	Wood Section Name	2-2x6
End Fixities	Top & Bottom Pinned	Wood Grading/Manuf.	Graded Lumber
Overall Column Height	9.0 ft	Wood Member Type	Sawn

(Used for non-slender calculations)

Wood Species	Douglas Fir - Larch		
Wood Grade	No.2		
Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Exact Width	3.0 in
Exact Depth	5.50 in
Area	16.50 in^2
Ix	41.594 in^4
Iy	12.375 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling about X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 32.175 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = -1.20 in, D = 1.80, L = 1.20, S = 1.40 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, L = 0.010, W = 0.3220 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.9641** : 1
	Load Combination	+D+0.60W
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
	Location of max.above base	4.591 ft
	At maximum location values are . . .	
	Applied Axial	1.832 k
	Applied Mx	2.047 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	1,063.92 psi
PASS	Maximum Shear Stress Ratio =	**0.2807** : 1
	Load Combination	+D+0.60W
	Location of max.above base	0.0 ft
	Applied Design Shear	80.855 psi
	Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.449 k	Bottom along Y-Y	1.449 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.7220 in at 4.530 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.663	0.1637	PASS	8.940 ft	0.01122	PASS	9.0 ft
+D+L	1.000	0.625	0.2836	PASS	7.852 ft	0.03956	PASS	0.0 ft
+D+S	1.150	0.572	0.2630	PASS	8.940 ft	0.01562	PASS	9.0 ft
+D+0.750L	1.250	0.540	0.1970	PASS	8.517 ft	0.02576	PASS	0.0 ft
+D+0.750L+0.750S	1.150	0.572	0.3262	PASS	8.940 ft	0.03312	PASS	0.0 ft
+D+0.60W	1.600	0.448	0.9641	PASS	4.591 ft	0.2807	PASS	0.0 ft
+D+0.750L+0.450W	1.600	0.448	0.8468	PASS	4.711 ft	0.2259	PASS	0.0 ft
+D+0.750L+0.750S+0.450W	1.600	0.448	0.9497	PASS	4.772 ft	0.2296	PASS	0.0 ft
+0.60D+0.60W	1.600	0.448	0.9045	PASS	4.591 ft	0.2782	PASS	0.0 ft

Wood Column

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Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

Description : Main Floor King Stud Enveloping Design

Load Combination Results

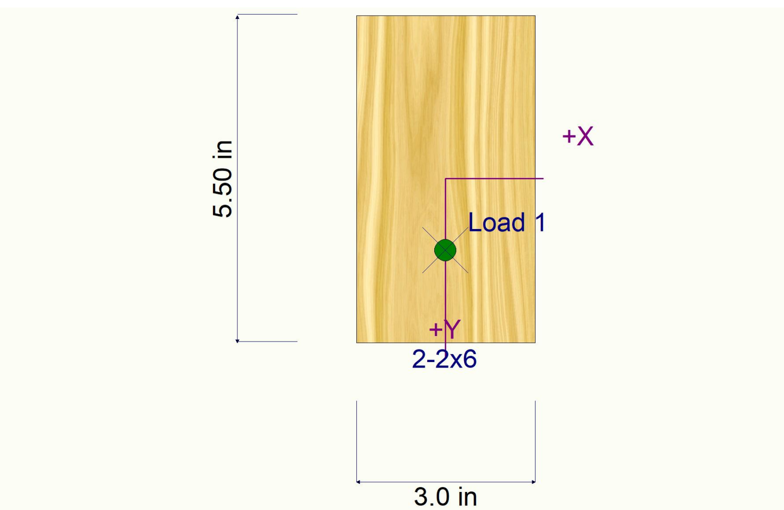
Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.448	0.06262	PASS	0.06040 ft	0.003788	PASS	9.0 ft

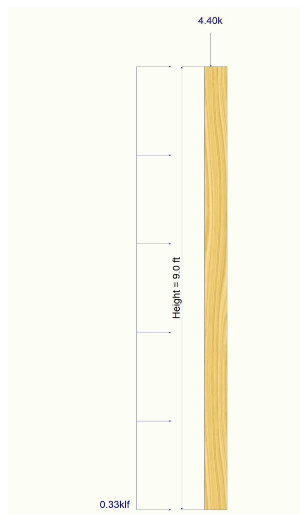
Maximum Reactions

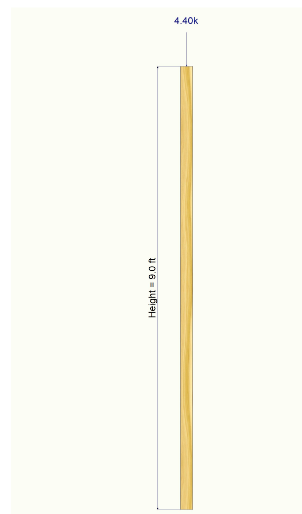
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only			0.020	-0.020	1.832				
+D+L			0.078	0.012	3.032				
+D+S			0.036	-0.036	3.232				
+D+0.750L			0.064	0.004	2.732				
+D+0.750L+0.750S			0.075	-0.008	3.782				
+D+0.60W			0.889	0.849	1.832				
+D+0.750L+0.450W			0.716	0.656	2.732				
+D+0.750L+0.750S+0.450W			0.727	0.644	3.782				
+0.60D+0.60W			0.881	0.857	1.099				
+0.60D			0.012	-0.012	1.099				
L Only			0.058	0.032	1.200				
S Only			0.016	-0.016	1.400				
W Only			1.449	1.449					

Sketches







Wood Column

Description : RB1 King Stud Enveloping Design

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height 9.0 ft
(Used for non-slender calculations)

Wood Species: Douglas Fir - Larch
Wood Grade: No.2

Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name: 2-2x6
Wood Grading/Manuf.: Graded Lumber
Wood Member Type: Sawn

Exact Width	3.0 in
Exact Depth	5.50 in
Area	16.50 in^2
Ix	41.594 in^4
Iy	12.375 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling about X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 32.175 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = -1.20 in, D = 0.80, S = 0.80 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, L = 0.010, W = 0.2170 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.6025** : 1

Load Combination	+D+0.60W
Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base	4.591 ft
At maximum location values are . . .	
Applied Axial	0.8322 k
Applied Mx	1.359 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,063.92 psi

PASS Maximum Shear Stress Ratio = **0.1877** : 1

Load Combination	+D+0.60W
Location of max.above base	0.0 ft
Applied Design Shear	54.072 psi
Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.9765 k	Bottom along Y-Y	0.9765 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.4865 in at 4.530 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.663	0.06565	PASS	8.940 ft	0.004988	PASS	9.0 ft
+D+L	1.000	0.625	0.1056	PASS	5.376 ft	0.02722	PASS	0.0 ft
+D+S	1.150	0.572	0.1121	PASS	8.940 ft	0.007808	PASS	9.0 ft
+D+0.750L	1.250	0.540	0.07108	PASS	5.678 ft	0.01723	PASS	0.0 ft
+D+0.750L+0.750S	1.150	0.572	0.1107	PASS	6.584 ft	0.02165	PASS	0.0 ft
+D+0.60W	1.600	0.448	0.6025	PASS	4.591 ft	0.1877	PASS	0.0 ft
+D+0.750L+0.450W	1.600	0.448	0.4905	PASS	4.591 ft	0.1522	PASS	0.0 ft
+D+0.750L+0.750S+0.450W	1.600	0.448	0.5244	PASS	4.651 ft	0.1543	PASS	0.0 ft
+0.60D+0.60W	1.600	0.448	0.5840	PASS	4.530 ft	0.1866	PASS	0.0 ft

Wood Column

Description : RB1 King Stud Enveloping Design

Load Combination Results

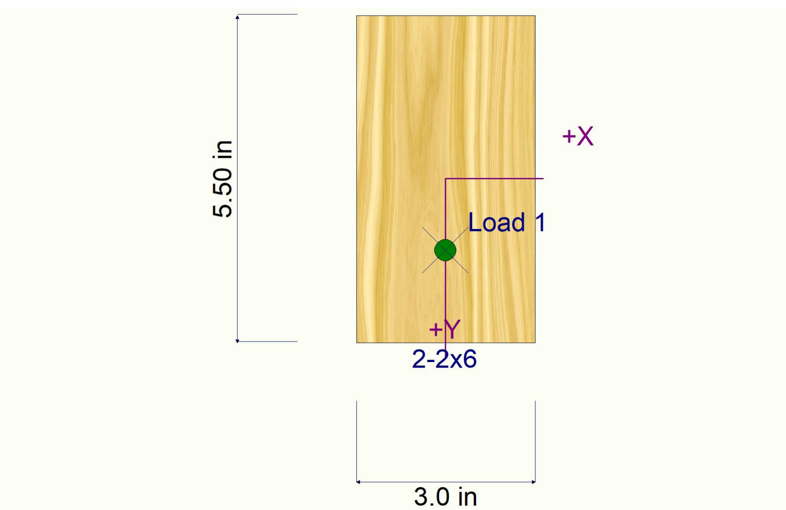
Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.448	0.02844	PASS	0.06040 ft	0.001684	PASS	9.0 ft

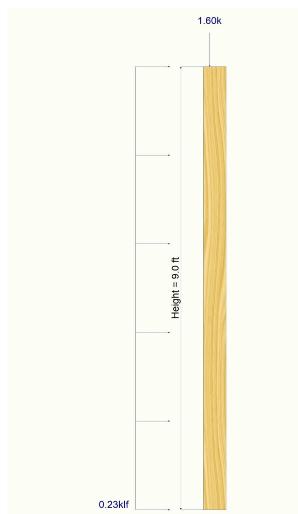
Maximum Reactions

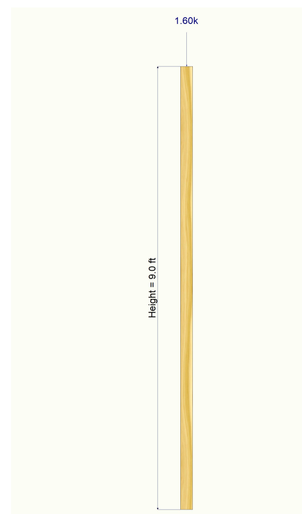
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only			0.009	-0.009	0.832				
+D+L			0.054	0.036	0.832				
+D+S			0.018	-0.018	1.632				
+D+0.750L			0.043	0.025	0.832				
+D+0.750L+0.750S			0.049	0.018	1.432				
+D+0.60W			0.595	0.577	0.832				
+D+0.750L+0.450W			0.482	0.464	0.832				
+D+0.750L+0.750S+0.450W			0.489	0.458	1.432				
+0.60D+0.60W			0.591	0.581	0.499				
+0.60D			0.005	-0.005	0.499				
L Only			0.045	0.045					
S Only			0.009	-0.009	0.800				
W Only			0.977	0.977					

Sketches







Wood Column

Description : RB2 King Stud Enveloping Design

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 9.0 ft
(Used for non-slender calculations)
Wood Species Douglas Fir - Larch
Wood Grade No.2

Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name **3-2x6**
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	**4.50** in
Exact Depth	**5.50** in
Area	24.750 in^2
Ix	62.391 in^4
Iy	41.766 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling about X-X Axis
Y-Y (depth) axis : Unbraced Length for Y-Y Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 48.263 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Yecc = -1.20 in, D = 0.80, S = 0.80 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, L = 0.010, W = 0.4060 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.7295** : 1
	Load Combination	+D+0.60W
	Governing NDS Forumla	Comp + Mxx, NDS Eq. 3.9-3
	Location of max.above base	4.530 ft
	At maximum location values are . . .	
	Applied Axial	0.8483 k
	Applied Mx	2.507 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	1,063.92 psi
PASS	Maximum Shear Stress Ratio =	**0.2326** : 1
	Load Combination	+D+0.60W
	Location of max.above base	0.0 ft
	Applied Design Shear	66.975 psi
	Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	1.827 k	Bottom along Y-Y	1.827 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.6069 in at 4.530 ft above base
for load combination : W Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.663	0.04255	PASS	8.940 ft	0.003325	PASS	9.0 ft
+D+L	1.000	0.625	0.06888	PASS	5.376 ft	0.01814	PASS	0.0 ft
+D+S	1.150	0.572	0.07069	PASS	8.940 ft	0.005205	PASS	9.0 ft
+D+0.750L	1.250	0.540	0.04626	PASS	5.678 ft	0.01149	PASS	0.0 ft
+D+0.750L+0.750S	1.150	0.572	0.07047	PASS	6.584 ft	0.01444	PASS	0.0 ft
+D+0.60W	1.600	0.448	0.7295	PASS	4.530 ft	0.2326	PASS	0.0 ft
+D+0.750L+0.450W	1.600	0.448	0.5723	PASS	4.530 ft	0.1820	PASS	0.0 ft
+D+0.750L+0.750S+0.450W	1.600	0.448	0.5951	PASS	4.591 ft	0.1834	PASS	0.0 ft
+0.60D+0.60W	1.600	0.448	0.7160	PASS	4.530 ft	0.2318	PASS	0.0 ft

Wood Column

Lic. # : KW-06005122 **Licensee : MC SQUARED, INC.**

Description : RB2 King Stud Enveloping Design

Load Combination Results

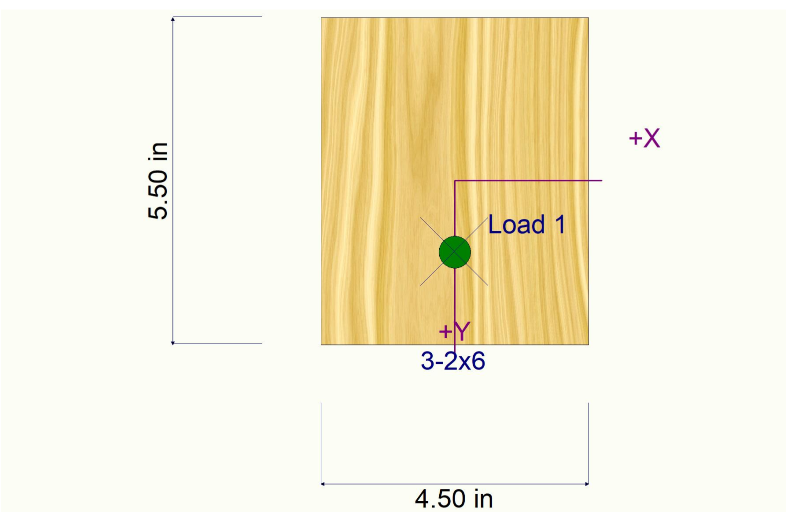
Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D	1.600	0.448	0.01933	PASS	0.06040 ft	0.001122	PASS	9.0 ft

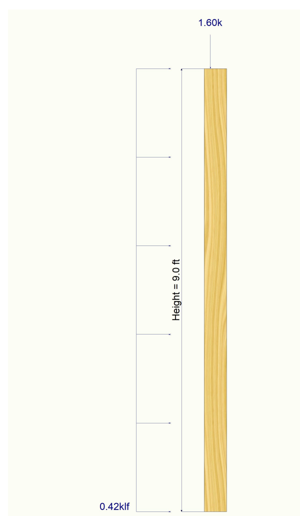
Maximum Reactions

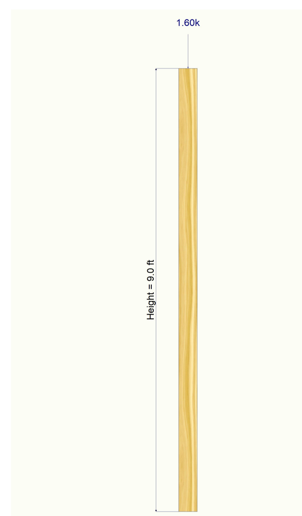
Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only			0.009	-0.009	0.848				
+D+L			0.054	0.036	0.848				
+D+S			0.018	-0.018	1.648				
+D+0.750L			0.043	0.025	0.848				
+D+0.750L+0.750S			0.049	0.018	1.448				
+D+0.60W			1.105	1.087	0.848				
+D+0.750L+0.450W			0.865	0.847	0.848				
+D+0.750L+0.750S+0.450W			0.871	0.840	1.448				
+0.60D+0.60W			1.102	1.091	0.509				
+0.60D			0.005	-0.005	0.509				
L Only			0.045	0.045					
S Only			0.009	-0.009	0.800				
W Only			1.827	1.827					

Sketches







FOUNDATION DESIGN

Soils:

Define allowable sustained vert. bearing press. [Geotech Report]: $q_{Sv} := 1500 \cdot psf$

Define soil density [Assumed]: $\gamma_g := 120 \cdot pcf$

Define active lateral earth pressure [Geotech Report]: $K_a := 35 \cdot pcf$

Define at-rest lateral earth pressure [Geotech Report]: $K_o := 50 \cdot pcf$

Define allowable passive lateral earth pressure [Geotech Report]: $K_{p\_all} := 300 \cdot pcf$

Define allowable coefficient of friction [Geotech Report]: $\mu_{q\_all} := 0.40$

Define applied seismic force [Geotech Report]: $K_e = 6 \cdot H$

Define surcharge coefficient [Geotech Report]: $\nu_q := 0.30$

Passive lateral earth pressure,

$K_p := Floor\left(1.5 \cdot K_{p\_all}, 5 \cdot pcf\right)$ $K_p = 450\, pcf$

Allowable vert. intermittent bearing press. [IBC-15, Sect. 1807.2.3, Exception],

$q_{Iv} := \frac{4}{3} \cdot q_{Sv}$ $q_{Iv} = 2000.00\, psf$

Vertical Loads:

Dead Loads:

Concrete density [ASCE 7-10, Table C3-2]: $\gamma_c := 150 \cdot pcf$

Roof dead load: $DL_r := 17 \cdot psf$

Floor dead load: $DL_f := 15 \cdot psf$

Exterior wall dead load: $DL_{ew} := 15 \cdot psf$

Partition dead load: $DL_{iw} := 12 \cdot psf$

Live Loads:

Roof live load: $LL_r := 20 \cdot psf$

Floor live load: $LL_f := 40 \cdot psf$

Main floor / slab on grade: $LL_m := 100 \cdot psf$

Snow Loads:

Define sloped snow: $SL_s := 25 \cdot psf$

Footing Design:

Define #4 rebar area: $A_{b4} := 0.20 \cdot in^2$

Define min. rebar area [ACI 318-14, Table 24.4.3.2]: $\rho_{min} := 0.0018$

Define slab on grade thickness: $t_{sog} := 4 \cdot in$

Define minimum footing depth below grade [Geotech Report]: $d_{bg\_min} := 18.0 \cdot in$

Define exterior stem thickness: $t_{st} := 8 \cdot in$

Define stem height: $h_{st} := 18 \cdot in$

Define exterior strip footing thickness: $t_{stf\_ext} := 8 \cdot in$

Define interior strip footing thickness: $t_{stf\_int} := 12 \cdot in$

Define exterior strip footing width: $b_{stf\_ext} := 18 \cdot in$

Define interior strip footing width: $b_{stf\_int} := 22 \cdot in$

Define spread footing thickness: $t_{spf} := 12 \cdot in$

Define vert. bar spacing (alt. tails): $s_{vb} := 32 \cdot in$

Footing overall depth,

$d_{ftg} := h_{st} + t_{stf\_ext}$ $\qquad d_{ftg} = 26.00 \cdot in$

Footing depth below grade,

$d_{bg} := d_{ftg} - 8 \cdot in$ $\qquad d_{bg} = 18.00 \cdot in$ $\qquad \dfrac{d_{bg\_min}}{d_{bg}} = 1.00$

Footing length for concentrated load check,

$l_{ftg} := 2 \cdot d_{ftg}$ $\qquad l_{ftg} = 4.333\, ft$ $\qquad l_{ftg} = 52.00 \cdot in$

Uniform load on footing,

$UL_{ftg} := t_{sog} \cdot \gamma_c + LL_m$ $\qquad UL_{ftg} = 150\, psf$

Exterior Strip Footing Design

Distributed wall dead load,

$w_{DL\_stf\_ext} := DL_r \cdot (23.0 \cdot ft) + 2 \cdot DL_f \cdot (10.667 \cdot ft) + 2 \cdot DL_{ew} \cdot (9.0 \cdot ft)$ $w_{DL\_stf\_ext} = 0.98 \cdot klf$

Distributed live load,

$w_{LL\_stf\_ext} := 2 \cdot LL_f \cdot (10.667 \cdot ft)$ $w_{LL\_stf\_ext} = 0.85 \cdot klf$

Distributed snow load,

$w_{SL\_stf\_ext} := SL_s \cdot (23.0 \cdot ft)$ $w_{SL\_stf\_ext} = 0.58 \cdot klf$

Total distributed load,

$$w_{TL\_stf\_ext} := w_{DL\_stf\_ext} + \left| \begin{array}{l} A1 \leftarrow max\left(w_{LL\_stf\_ext}, w_{SL\_stf\_ext}\right) \\ A2 \leftarrow 0.75 \cdot \left(w_{LL\_stf\_ext} + w_{SL\_stf\_ext}\right) \\ max(A1, A2) \end{array} \right.$$

$w_{TL\_stf\_ext} = 2052.28\, plf$

Minimum footing width,

$$b_{min\_stf\_ext} := \frac{w_{TL\_stf\_ext}}{q_{Sv} - UL_{ftg}}$$

$b_{min\_stf\_ext} = 18.24 \cdot in$ $\frac{b_{min\_stf\_ext}}{b_{stf\_ext}} = 1.01$

1% Over Acceptable

Number of bars required,

$$n_{stf\_ext} := max\left(Ceil\left(\frac{\rho_{min} \cdot t_{stf\_ext} \cdot b_{stf\_ext}}{A_{b4}}, 1\right), 2\right)$$

$n_{stf\_ext} = 2$

Interior Strip Footing Design

Distributed wall dead load,

$w_{DL\_stf\_int} := 2 \cdot DL_f \cdot (20.5 \cdot ft) + 2 \cdot DL_{iw} \cdot (9.0 \cdot ft)$ $w_{DL\_stf\_int} = 0.83 \cdot klf$

Distributed live load,

$w_{LL\_stf\_int} := 2 \cdot LL_f \cdot (20.5 \cdot ft)$ $w_{LL\_stf\_int} = 1.64 \cdot klf$

Total distributed load,

$w_{TL\_stf\_int} := w_{DL\_stf\_int} + w_{LL\_stf\_int}$ $w_{TL\_stf\_int} = 2471.00\, plf$

Minimum footing width,

$$b_{min\_stf\_int} := \frac{w_{TL\_stf\_int}}{q_{Sv} - UL_{ftg}}$$

$b_{min\_stf\_int} = 21.96 \cdot in$ $\frac{b_{min\_stf\_int}}{b_{stf\_int}} = 1.00$

Number of bars required,

$$n_{stf\_int} := max\left(Ceil\left(\frac{\rho_{min} \cdot t_{stf\_int} \cdot b_{stf\_int}}{A_{b4}}, 1\right), 2\right)$$

$n_{stf\_int} = 3$

Garage Strip Footing Design

Distributed wall dead load,

$w_{DL\_stf\_g} := DL_r \cdot (16.0 \cdot ft) + DL_{ew} \cdot (9.0 \cdot ft)$ $\quad w_{DL\_stf\_g} = 0.41 \cdot klf$

Distributed snow load,

$w_{SL\_stf\_g} := SL_s \cdot (16.0 \cdot ft)$ $\quad w_{SL\_stf\_g} = 0.40 \cdot klf$

Total distributed load,

$w_{TL\_stf\_g} := w_{DL\_stf\_g} + w_{SL\_stf\_g}$ $\quad w_{TL\_stf\_g} = 807.00\,plf$

Minimum footing width,

$b_{min\_stf\_g} := \dfrac{w_{TL\_stf\_g}}{q_{Sv} - UL_{ftg}}$ $\quad b_{min\_stf\_g} = 7.17 \cdot in$ $\quad \dfrac{b_{min\_stf\_g}}{b_{stf\_int}} = 0.33$

Number of bars required,

$n_{stf\_g} := max\left(Ceil\left(\dfrac{\rho_{min} \cdot t_{stf\_int} \cdot b_{min\_stf\_g}}{A_{b4}}, 1\right), 2\right)$ $\quad n_{stf\_g} = 2$

Check Exterior Strip Footing for Concentrated Load (Conservatively use Garage Door Headers as 1 Conc. Force)

Footing axial dead load (RB1 + RB2 Reactions),

$P_{DL\_G} := (1.3 \cdot kip) + (2.5 \cdot kip)$ $\quad P_{DL\_G} = 3.80\,kip$

Footing axial snow load (RB1 + RB2 Reactions),

$P_{SL\_G} := (1.8 \cdot kip) + (3.6 \cdot kip)$ $\quad P_{SL\_G} = 5.40\,kip$

Footing total load,

$P_{TL\_G} := P_{DL\_G} + P_{SL\_G}$ $\quad P_{TL\_G} = 9.20\,kip$

Footing width at interior wall (add in trimmer studs),

$b_{spf\_G} := \dfrac{P_{TL\_G}}{\left(q_{Sv} - UL_{ftg}\right) \cdot \left(l_{ftg}\right)}$ $\quad b_{spf\_G} = 18.87 \cdot in$ $\quad \dfrac{b_{spf\_G}}{b_{stf\_ext}} = 1.05$

5% Over Acceptable

Beam Support Spread Footing

Footing axial dead load (FB1 Reactions),

$P_{DL\_FB1} := (11.5 \cdot kip)$ $\qquad P_{DL\_FB1} = 11.50\,kip$

Footing axial live load (FB1 Reactions),

$P_{LL\_FB1} := (7.4 \cdot kip)$ $\qquad P_{LL\_FB1} = 7.40\,kip$

Footing axial snow load (FB1 Reactions),

$P_{SL\_FB1} := (5.6 \cdot kip)$ $\qquad P_{SL\_FB1} = 5.60\,kip$

Footing total load,

$$P_{TL\_FB1} := P_{DL\_FB1} + \left| \begin{array}{l} A1 \leftarrow max\left(P_{LL\_FB1}, P_{SL\_FB1}\right) \\ A2 \leftarrow 0.75 \cdot \left(P_{LL\_FB1} + P_{SL\_FB1}\right) \\ max(A1, A2) \end{array} \right. \qquad P_{TL\_FB1} = 21.25\,kip$$

Footing width at interior wall,

$$b_{spf\_FB1} := Ceil\left(\sqrt{\frac{P_{TL\_FB1}}{q_{Sv} - UL_{ftg}}}, 6.0 \cdot in\right) \qquad b_{spf\_FB1} = 4.00 \cdot ft$$

Number of bars required,

$$n_{spf\_FB1} := max\left(Ceil\left(\frac{\rho_{min} \cdot t_{spf} \cdot b_{spf\_FB1}}{A_{b4}}, 1\right), 2\right) \qquad n_{spf\_FB1} = 6$$

Patio Cover/Covered Veranda Beam Spread Footing

Footing axial dead load (Dbl CPB1 Reactions),

$P_{DL\_C} := 2 \cdot (0.9 \cdot kip)$ $\qquad P_{DL\_C} = 1.80\,kip$

Footing axial snow load (Dbl CPB1 Reactions),

$P_{SL\_C} := 2 \cdot (1.3 \cdot kip)$ $\qquad P_{SL\_C} = 2.60\,kip$

Footing total load,

$P_{TL\_C} := P_{DL\_C} + P_{SL\_C}$ $\qquad P_{TL\_C} = 4.40\,kip$

Footing width at interior wall,

$$b_{spf\_C} := Ceil\left(\sqrt{\frac{P_{TL\_C}}{q_{Sv}}}, 6.0 \cdot in\right) \qquad b_{spf\_C} = 2.00 \cdot ft$$

Number of bars required,

$$n_{spf\_C} := max\left(Ceil\left(\frac{\rho_{min} \cdot t_{spf} \cdot b_{spf\_C}}{A_{b4}}, 1\right), 2\right) \qquad n_{spf\_C} = 3$$

Check Punching Shear

Assume post is 6x6 and depth of bar is 8".

Applied load,

$V_u := 1.6 \cdot P_{TL\_FB1}$ $\quad V_u = 34.00\,kip$

Define post square width: $\quad b_{post} := 5.5 \cdot in$

Define depth of rebar: $\quad d_r := 8 \cdot in$

Define concrete strength: $\quad f'_c := 3 \cdot ksi$

Concrete perimeter,

$b_o := 4 \cdot \left(0.5 \cdot d_r + b_{post} + 0.5 \cdot d_r\right)$ $\quad b_o = 4.50\,ft$

Define length to width ratio [ACI 318-14, Sect. R22.6.5.2]: $\quad \beta_s := 1.0$

Define column location factor [ACI 318-14, Sect. 22.6.5.3]: $\quad \alpha_s := 20$

Allowable concrete stress [ACI 318-14, Table 22.6.5.2],

$v_{c1} := 4 \cdot \sqrt{f'_c \cdot psi}$ $\quad v_{c1} = 219.09 \cdot psi$

$$v_{c2} := \left(2 + \frac{4}{\beta_s}\right) \cdot \sqrt{f'_c \cdot psi} \qquad v_{c2} = 328.63 \cdot psi$$

$$v_{c3} := \left(2 + \frac{\alpha_s \cdot d_r}{b_o}\right) \cdot \sqrt{f'_c \cdot psi} \qquad v_{c3} = 271.83 \cdot psi$$

$v_c := min\left(v_{c1}, v_{c2}, v_{c3}\right)$ $\quad v_c = 219.09 \cdot psi$

Allowable shear force,

$\phi V_c := 0.75 \cdot v_c \cdot b_o \cdot d_r$ $\quad \phi V_c = 70.98\,kip$ $\quad \dfrac{V_u}{\phi V_c} = 0.48$

Retaining Wall Design:

Define retaining wall stem thickness: $t_{st\_rw} := 8 \cdot in$

Define retaining wall stem height: $h_{st\_rw} := 9 \cdot ft + 2 \cdot in$

Define retained soil height below slab: $h_{g\_rw} := 8 \cdot ft + 6 \cdot in$

Define retaining wall footing thickness: $t_{ftg\_rw} := 12 \cdot in$

Define retaining wall footing width: $b_{ftg\_rw} := 6 \cdot ft + 6 \cdot in$

Number of bars required,

$$n_{ftg\_rw} := max\left(Ceil\left(\frac{\rho_{min} \cdot t_{ftg\_rw} \cdot b_{ftg\_rw}}{A_{b4}}, 1\right), 2\right) \qquad n_{ftg\_rw} = 9$$

Equivalent soil density over heel,

$$\gamma_{g\_eq} := \frac{t_{sog} \cdot \gamma_c + h_{g\_rw} \cdot \gamma_g}{t_{sog} + h_{g\_rw}} \qquad \gamma_{g\_eq} = 121.1\,pcf$$

Retaining Wall Loads Along Floor Bearing Walls

Distributed dead load on retaining wall,

$$w_{DL\_rw\_bw} := DL_r \cdot (23.0 \cdot ft) + 2 \cdot DL_f \cdot (10.667 \cdot ft) + 2 \cdot (9.0 \cdot ft) \cdot DL_{ew} \qquad w_{DL\_rw\_bw} = 981.01\,plf$$

Distributed transient load on retaining wall,

$$w_{LL\_rw\_bw} := \left| \begin{array}{l} A1 \leftarrow max\left[2 \cdot LL_f \cdot (10.667 \cdot ft), SL_s \cdot (23.0 \cdot ft)\right] \\ A2 \leftarrow 0.75 \cdot \left[2 \cdot LL_f \cdot (10.667 \cdot ft) + SL_s \cdot (23.0 \cdot ft)\right] \\ max(A1, A2) \end{array} \right. \qquad w_{LL\_rw\_bw} = 1071.27\,plf$$

Concentrated load from Porch Cover/Covered Veranda footing, $P_{TL\_C} = 4400.00 \cdot lbf$

Footing width, $b_{spf\_C} = 2.00\,ft$

Bottom of footing to top of grade, $t_{spf} = 1.00\,ft$

Define horizontal distance from footing to retaining wall: $d_{rw\_h} := 7.75 \cdot ft$

Retaining Wall Loads Along Floor Non-Bearing Walls

Distributed dead load on retaining wall,

$$w_{DL\_rw\_nbw} := DL_r \cdot (23.0 \cdot ft) + 2 \cdot DL_f \cdot (1.0 \cdot ft) + 2 \cdot (9.0 \cdot ft) \cdot DL_{ew} \qquad w_{DL\_rw\_nbw} = 691.00\,plf$$

Distributed live load on retaining wall,

$$w_{LL\_rw\_nbw} := \left| \begin{array}{l} A1 \leftarrow max\left[2 \cdot LL_f \cdot (1.0 \cdot ft), SL_s \cdot (23.0 \cdot ft)\right] \\ A2 \leftarrow 0.75 \cdot \left[2 \cdot LL_f \cdot (1.0 \cdot ft) + SL_s \cdot (23.0 \cdot ft)\right] \\ max(A1, A2) \end{array} \right. \qquad w_{LL\_rw\_nbw} = 575.00\,plf$$

Retaining Wall Loads at Patio Cover

Distributed dead load on retaining wall,

$$w_{DL\_rw\_pc} := \frac{P_{DL\_C}}{2 \cdot \left(h_{st\_rw} + t_{ftg\_rw}\right)} \qquad w_{DL\_rw\_pc} = 88.52\,plf$$

Distributed live load on retaining wall,

$$w_{LL\_rw\_pc} := \frac{P_{SL\_C}}{2 \cdot \left(h_{st\_rw} + t_{ftg\_rw}\right)} \qquad w_{LL\_rw\_pc} = 127.87\,plf$$

This Wall in File: M:\PROJECTS\Altman, Benjamin\2020-0196 Altman East Lot\Documents\Revision 1\Found

Cantilevered Retaining Wall

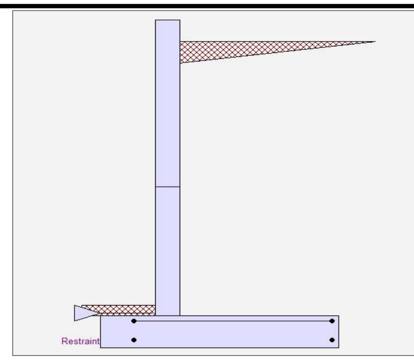
Code: IBC 2015,ACI 318-14,ACI 530-13

Criteria

Retained Height	=	8.50 ft
Wall height above soil	=	0.67 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	4.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	450.0 psf/ft
Soil Density, Heel	=	121.10 pcf
Soil Density, Toe	=	150.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel = 40.0 psf
NOT Used To Resist Sliding & Overturning
Surcharge Over Toe = 40.0 psf
NOT Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	990.0 lbs
Axial Live Load	=	1,100.0 lbs
Axial Load Eccentricity	=	1.3 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	4,400.0 lbs
Footing Width	=	1.50 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	7.75 ft
Footing Type		Line
Base Above/Below Soil at Back of Wall	=	-1.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning = 5.23 OK

Slab Resists All Sliding !

Total Bearing Load	=	9,739 lbs
...resultant ecc.	=	1.00 in
Soil Pressure @ Toe	=	1,274 psf OK
Soil Pressure @ Heel	=	1,487 psf OK
Allowable	=	1,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,829 psf
ACI Factored @ Heel	=	2,136 psf
Footing Shear @ Toe	=	10.2 psi OK
Footing Shear @ Heel	=	10.3 psi OK
Allowable	=	94.9 psi

Sliding Calcs

Lateral Sliding Force = 2,044.4 lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	IBC 2015,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		2nd	Bottom
		Stem OK	Stem OK
Design Height Above Ftg	ft =	4.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Design Method	=	LRFD	LRFD
Thickness	=	8.00	8.00
Rebar Size	=	# 5	# 6
Rebar Spacing	=	16.00	8.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	**0.216**	**0.501**
Total Force @ Section			
Service Level	lbs =		
Strength Level	lbs =	692.1	2,635.2
Moment....Actual			
Service Level	ft-# =		
Strength Level	ft-# =	1,364.5	7,647.6
Moment.....Allowable	ft-# =	6,294.3	15,260.6
Shear.....Actual			
Service Level	psi =		
Strength Level	psi =	9.3	39.0
Shear.....Allowable	psi =	94.9	94.9
Anet (Masonry)	in2 =		
Rebar Depth 'd'	in =	6.19	5.63
Masonry Data			
f'm	psi =		
Fs	psi =		
Solid Grouting	=		
Modular Ratio 'n'	=		
Wall Weight	psf =	100.0	100.0
Short Term Factor	=		
Equiv. Solid Thick.	=		
Masonry Block Type	=	Medium Weight	
Masonry Design Method	=	ASD	
Concrete Data			
f'c	psi =	4,000.0	4,000.0
Fy	psi =	60,000.0	60,000.0

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0517 in2/ft		
(4/3) * As :	0.0689 in2/ft	Min Stem T&S Reinf Area 0.992 in2	
200bd/fy : 200(12)(6.1875)/60000 :	0.2475 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.2325 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.3411 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.3203 in2/ft		
(4/3) * As :	0.427 in2/ft	Min Stem T&S Reinf Area 0.768 in2	
200bd/fy : 200(12)(5.625)/60000 :	0.225 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.3203 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.66 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.2192 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	1.50 ft
Heel Width	=	5.00
Total Footing Width	=	6.50
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ Btm.=	3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,829	2,136	psf
Mu' : Upward	=	25,015	18,577	ft-#
Mu' : Downward	=	3,948	19,210	ft-#
Mu: Design	=	1,756	633	ft-#
Actual 1-Way Shear	=	10.16	10.30	psi
Allow 1-Way Shear	=	50.60	94.87	psi
Toe Reinforcing	=	None Spec'd		
Heel Reinforcing	=	# 6 @ 16.00 in		
Key Reinforcing	=	None Spec'd		
Footing Torsion, Tu	=		0.00	ft-lbs
Footing Allow. Torsion, phi Tu	=		0.00	ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi'5'lambda'sqrt(fc)'Sm
Heel: #4@ 9.25 in, #5@ 14.35 in, #6@ 20.37 in, #7@ 27.77 in, #8@ 36.57 in, #9@ 46
Key: No key defined

Min footing T&S reinf Area	1.68	in2
Min footing T&S reinf Area per foot	0.26	in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		1,579.4	3.17	5,001.4
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=	109.8	4.75	521.7
Surcharge Over Toe	=			
Adjacent Footing Load	=	355.2	2.56	908.2
Added Lateral Load	=			
Load @ Stem Above Soil	=			
	=			
Total	=	2,044.4	**O.T.M.** =	6,431.3

Resisting/Overturning Ratio = **5.23**
Vertical Loads used for Soil Pressure = 9,738.7 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		4,460.5	4.33	19,328.9
Soil Over HL (bel. water tbl)			4.33	19,328.9
Watre Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=	454.0	5.96	2,704.9
Axial Dead Load on Stem	=	2,090.0	1.73	1,711.9
* Axial Live Load on Stem	=	1,100.0	1.73	1,902.1
Soil Over Toe	=	75.0	0.75	56.3
Surcharge Over Toe	=			
Stem Weight(s)	=	916.7	1.83	1,680.6
Earth @ Stem Transitions	=			
Footing Weight	=	975.0	3.25	3,168.8
Key Weight	=			
Vert. Component	=	767.5	6.50	4,988.9
Total =		8,638.7 lbs	**R.M.=**	33,640.1

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.080	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

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Cantilevered Retaining Wall

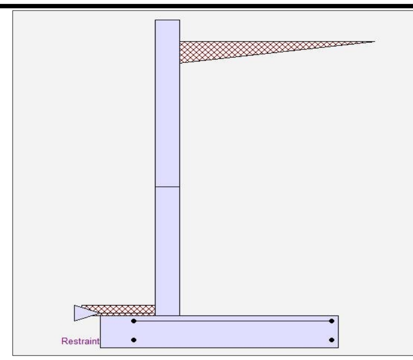
Code: IBC 2015,ACI 318-14,ACI 530-13

Criteria

Retained Height	=	8.50 ft
Wall height above soil	=	0.67 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	4.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	450.0 psf/ft
Soil Density, Heel	=	121.10 pcf
Soil Density, Toe	=	150.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel = 40.0 psf
NOT Used To Resist Sliding & Overturning
Surcharge Over Toe = 40.0 psf
NOT Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	990.0 lbs
Axial Live Load	=	1,100.0 lbs
Axial Load Eccentricity	=	1.3 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	4,400.0 lbs
Footing Width	=	1.50 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	7.75 ft
Footing Type		Line
Base Above/Below Soil at Back of Wall	=	-1.0 ft
Poisson's Ratio	=	0.300

Earth Pressure Seismic Load

Method : Mononobe-Okabe/Seed-Whitman

Design Kh = 0.275 g

Using Mononobe-Okabe / Seed-Whitman procedure

Kae for seismic earth pressure	=	0.403
Ka for static earth pressure	=	0.232
Difference: Kae - Ka	=	0.171

Added seismic base force 934.5 lbs

This Wall in File: M:\PROJECTS\Altman, Benjamin\2020-0196 Altman East Lot\Documents\Revision 1\Found

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Design Summary

Wall Stability Ratios

Overturning	=	2.86	OK
Slab Resists All Sliding !			
Total Bearing Load	=	9,739 lbs	
...resultant ecc.	=	6.12 in	
Soil Pressure @ Toe	=	2,030 psf	NG
Soil Pressure @ Heel	=	730 psf	OK
Allowable	=	2,000 psf	
Soil Pressure Exceeds Allowable!			
ACI Factored @ Toe	=	2,916 psf	
ACI Factored @ Heel	=	1,049 psf	
Footing Shear @ Toe	=	16.5 psi	OK
Footing Shear @ Heel	=	3.5 psi	OK
Allowable	=	94.9 psi	

Sliding Calcs

Lateral Sliding Force	=	2,979.0 lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	IBC 2015,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		2nd	Bottom
		Stem OK	Stem OK
Design Height Above Ftg	ft =	4.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Design Method	=	LRFD	LRFD
Thickness	=	8.00	8.00
Rebar Size	=	# 5	# 6
Rebar Spacing	=	16.00	8.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	**0.616**	**0.976**
Total Force @ Section			
Service Level	lbs =		
Strength Level	lbs =	1,624.0	4,057.6
Moment....Actual			
Service Level	ft-# =		
Strength Level	ft-# =	3,880.7	14,901.5
Moment.....Allowable	ft-# =	6,294.3	15,260.6
Shear.....Actual			
Service Level	psi =		
Strength Level	psi =	21.9	60.1
Shear.....Allowable	psi =	94.9	94.9
Anet (Masonry)	in2 =		
Rebar Depth 'd'	in =	6.19	5.63
Masonry Data			
f'm	psi =		
Fs	psi =		
Solid Grouting	=		
Modular Ratio 'n'	=		
Wall Weight	psf =	100.0	100.0
Short Term Factor	=		
Equiv. Solid Thick.	=		
Masonry Block Type	=	Medium Weight	
Masonry Design Method	=	ASD	
Concrete Data			
f'c	psi =	4,000.0	4,000.0
Fy	psi =	60,000.0	60,000.0

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.1469 in2/ft		
(4/3) * As :	0.1959 in2/ft	Min Stem T&S Reinf Area 0.992 in2	
200bd/fy : 200(12)(6.1875)/60000 :	0.2475 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1959 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.2325 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.3411 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.6241 in2/ft		
(4/3) * As :	0.8321 in2/ft	Min Stem T&S Reinf Area 0.768 in2	
200bd/fy : 200(12)(5.625)/60000 :	0.225 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.6241 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.66 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.2192 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	1.50 ft
Heel Width	=	5.00
Total Footing Width	=	6.50
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 4,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	2,916	1,049	psf
Mu' : Upward	=	37,427	13,745	ft-#
Mu' : Downward	=	3,948	19,210	ft-#
Mu: Design	=	2,790	5,465	ft-#
Actual 1-Way Shear	=	16.45	3.47	psi
Allow 1-Way Shear	=	50.60	94.87	psi

Toe Reinforcing = None Spec'd
Heel Reinforcing = # 6 @ 16.00 in
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings
Toe: phiMn = phi'5'lambda'sqrt(fc)'Sm
Heel: #4@ 9.25 in, #5@ 14.35 in, #6@ 20.37 in, #7@ 27.77 in, #8@ 36.57 in, #9@ 46
Key: No key defined

Min footing T&S reinf Area 1.68 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		1,579.4	3.17	5,001.4
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=	109.8	4.75	521.7
Surcharge Over Toe	=			
Adjacent Footing Load	=	355.2	2.56	908.2
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Seismic Earth Load	=	934.5	5.70	5,326.8
	=			
Total	=	2,979.0	**O.T.M.** =	11,758.0

Resisting/Overturning Ratio = **2.86**
Vertical Loads used for Soil Pressure = 9,738.7 lbs

If seismic is included, the OTM and sliding ratios may be 1.1 per section 1807.2.3 of IBC.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		4,460.5	4.33	19,328.9
Soil Over HL (bel. water tbl)			4.33	19,328.9
Watre Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=	454.0	5.96	2,704.9
Axial Dead Load on Stem	=	2,090.0	1.73	1,711.9
* Axial Live Load on Stem	=	1,100.0	1.73	1,902.1
Soil Over Toe	=	75.0	0.75	56.3
Surcharge Over Toe	=			
Stem Weight(s)	=	916.7	1.83	1,680.6
Earth @ Stem Transitions	=			
Footing Weight	=	975.0	3.25	3,168.8
Key Weight	=			
Vert. Component	=	767.5	6.50	4,988.9
Total =		8,638.7 lbs	**R.M.=**	33,640.1

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.080	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

This Wall in File: M:\PROJECTS\Altman, Benjamin\2020-0196 Altman East Lot\Documents\Revision 1\Found

Cantilevered Retaining Wall

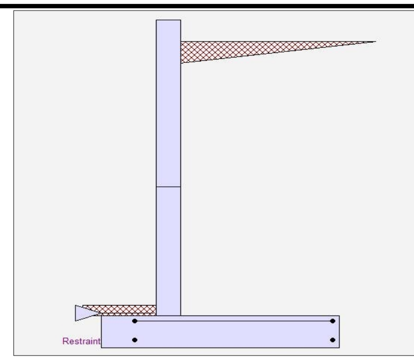
Code: IBC 2015,ACI 318-14,ACI 530-13

Criteria

Retained Height	=	8.50 ft
Wall height above soil	=	0.67 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	4.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	450.0 psf/ft
Soil Density, Heel	=	121.10 pcf
Soil Density, Toe	=	150.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel = 40.0 psf
NOT Used To Resist Sliding & Overturning
Surcharge Over Toe = 40.0 psf
NOT Used for Sliding & Overturning

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	700.0 lbs
Axial Live Load	=	580.0 lbs
Axial Load Eccentricity	=	1.3 in

Design Summary

Wall Stability Ratios

Overturning = 5.51 OK
Slab Resists All Sliding !

Total Bearing Load	=	8,475 lbs
...resultant ecc.	=	2.59 in
Soil Pressure @ Toe	=	950 psf OK
Soil Pressure @ Heel	=	1,422 psf OK
Allowable	=	1,500 psf

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,370 psf
ACI Factored @ Heel	=	2,051 psf
Footing Shear @ Toe	=	7.3 psi OK
Footing Shear @ Heel	=	2.3 psi OK
Allowable	=	94.9 psi

Sliding Calcs

Lateral Sliding Force = 1,689.2 lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	IBC 2015,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

			2nd	Bottom
			Stem OK	Stem OK
Design Height Above Ftg	ft	=	4.00	0.00
Wall Material Above "Ht"		=	Concrete	Concrete
Design Method		=	LRFD	LRFD
Thickness		=	8.00	8.00
Rebar Size		=	# 5	# 6
Rebar Spacing		=	16.00	8.00
Rebar Placed at		=	Edge	Edge
Design Data				
fb/FB + fa/Fa		=	**0.194**	**0.431**
Total Force @ Section				
Service Level	lbs	=		
Strength Level	lbs	=	650.2	2,180.2
Moment....Actual				
Service Level	ft-#	=		
Strength Level	ft-#	=	1,221.9	6,584.2
Moment.....Allowable	ft-#	=	6,294.3	15,260.6
Shear.....Actual				
Service Level	psi	=		
Strength Level	psi	=	8.8	32.3
Shear.....Allowable	psi	=	94.9	94.9
Anet (Masonry)	in2	=		
Rebar Depth 'd'	in	=	6.19	5.63
Masonry Data				
f'm	psi	=		
Fs	psi	=		
Solid Grouting		=		
Modular Ratio 'n'		=		
Wall Weight	psf	=	100.0	100.0
Short Term Factor		=		
Equiv. Solid Thick.		=		
Masonry Block Type		=	Medium Weight	
Masonry Design Method		=	ASD	
Concrete Data				
f'c	psi	=	4,000.0	4,000.0
Fy	psi	=	60,000.0	60,000.0

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0463 in2/ft		
(4/3) * As :	0.0617 in2/ft	Min Stem T&S Reinf Area 0.992 in2	
200bd/fy : 200(12)(6.1875)/60000 :	0.2475 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.2325 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.3411 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.2757 in2/ft		
(4/3) * As :	0.3677 in2/ft	Min Stem T&S Reinf Area 0.768 in2	
200bd/fy : 200(12)(5.625)/60000 :	0.225 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.2757 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.66 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.2192 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	1.50 ft
Heel Width	=	5.00
Total Footing Width	=	6.50
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ Btm.=	3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,370	2,051	psf
Mu' : Upward	=	19,201	15,976	ft-#
Mu' : Downward	=	3,948	19,210	ft-#
Mu: Design	=	1,271	3,234	ft-#
Actual 1-Way Shear	=	7.28	2.32	psi
Allow 1-Way Shear	=	50.60	94.87	psi
Toe Reinforcing	=	None Spec'd		
Heel Reinforcing	=	# 6 @ 16.00 in		
Key Reinforcing	=	None Spec'd		
Footing Torsion, Tu	=		0.00	ft-lbs
Footing Allow. Torsion, phi Tu	=		0.00	ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi'5'lambda'sqrt(fc)'Sm
Heel: #4@ 9.25 in, #5@ 14.35 in, #6@ 20.37 in, #7@ 27.77 in, #8@ 36.57 in, #9@ 46
Key: No key defined

Min footing T&S reinf Area	1.68	in2
Min footing T&S reinf Area per foot	0.26	in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		1,579.4	3.17	5,001.4
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=	109.8	4.75	521.7
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
	=			
Total	=	1,689.2	**O.T.M.** =	5,523.0

Resisting/Overturning Ratio = **5.51**
Vertical Loads used for Soil Pressure = 8,474.7 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		4,460.5	4.33	19,328.9
Soil Over HL (bel. water tbl)			4.33	19,328.9
Watre Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	1,280.0	1.73	1,210.4
* Axial Live Load on Stem	=	580.0	1.73	1,002.9
Soil Over Toe	=	75.0	0.75	56.3
Surcharge Over Toe	=			
Stem Weight(s)	=	916.7	1.83	1,680.6
Earth @ Stem Transitions	=			
Footing Weight	=	975.0	3.25	3,168.8
Key Weight	=			
Vert. Component	=	767.5	6.50	4,988.9
Total =		7,894.7 lbs	**R.M.=**	30,433.8

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.067	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe,
because the wall would then tend to rotate into the retained soil.

This Wall in File: M:\PROJECTS\Altman, Benjamin\2020-0196 Altman East Lot\Documents\Revision 1\Found

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Cantilevered Retaining Wall

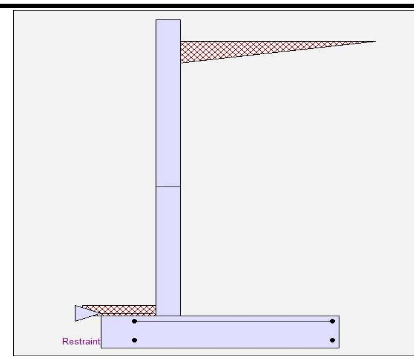
Code: IBC 2015,ACI 318-14,ACI 530-13

Criteria

Retained Height	=	8.50 ft
Wall height above soil	=	0.67 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	4.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	450.0 psf/ft
Soil Density, Heel	=	121.10 pcf
Soil Density, Toe	=	150.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel = 40.0 psf
NOT Used To Resist Sliding & Overturning
Surcharge Over Toe = 40.0 psf
NOT Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	700.0 lbs
Axial Live Load	=	580.0 lbs
Axial Load Eccentricity	=	1.3 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Earth Pressure Seismic Load

Method : Mononobe-Okabe/Seed-Whitman

Design Kh = 0.275 g

Kae for seismic earth pressure	=	0.403
Ka for static earth pressure	=	0.232
Difference: Kae - Ka	=	0.171

Added seismic base force 934.5 lbs

Using Mononobe-Okabe / Seed-Whitman procedure

This Wall in File: M:\PROJECTS\Altman, Benjamin\2020-0196 Altman East Lot\Documents\Revision 1\Found

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Design Summary

Wall Stability Ratios

Overturning	=	2.81	OK

Slab Resists All Sliding !

Total Bearing Load	=	8,475	lbs	
...resultant ecc.	=	5.71	in	
Soil Pressure @ Toe	=	1,706	psf	OK
Soil Pressure @ Heel	=	665	psf	OK
Allowable	=	2,000	psf	

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	2,461	psf	
ACI Factored @ Heel	=	959	psf	
Footing Shear @ Toe	=	13.6	psi	OK
Footing Shear @ Heel	=	11.5	psi	OK
Allowable	=	94.9	psi	

Sliding Calcs

Lateral Sliding Force	=	2,623.7	lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	IBC 2015,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		2nd	Bottom
		Stem OK	Stem OK
Design Height Above Ftg	ft =	4.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Design Method	=	LRFD	LRFD
Thickness	=	8.00	8.00
Rebar Size	=	# 5	# 6
Rebar Spacing	=	16.00	8.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	**0.593**	**0.906**
Total Force @ Section			
Service Level	lbs =		
Strength Level	lbs =	1,582.2	3,602.6
Moment....Actual			
Service Level	ft-# =		
Strength Level	ft-# =	3,738.2	13,838.1
Moment.....Allowable	ft-# =	6,294.3	15,260.6
Shear.....Actual			
Service Level	psi =		
Strength Level	psi =	21.3	53.4
Shear.....Allowable	psi =	94.9	94.9
Anet (Masonry)	in2 =		
Rebar Depth 'd'	in =	6.19	5.63
Masonry Data			
f'm	psi =		
Fs	psi =		
Solid Grouting	=		
Modular Ratio 'n'	=		
Wall Weight	psf =	100.0	100.0
Short Term Factor	=		
Equiv. Solid Thick.	=		
Masonry Block Type	=	Medium Weight	
Masonry Design Method	=	ASD	
Concrete Data			
f'c	psi =	4,000.0	4,000.0
Fy	psi =	60,000.0	60,000.0

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.1415 in2/ft		
(4/3) * As :	0.1887 in2/ft	Min Stem T&S Reinf Area 0.992 in2	
200bd/fy : 200(12)(6.1875)/60000 :	0.2475 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1887 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.2325 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.3411 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.5795 in2/ft		
(4/3) * As :	0.7727 in2/ft	Min Stem T&S Reinf Area 0.768 in2	
200bd/fy : 200(12)(5.625)/60000 :	0.225 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.5795 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.66 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.2192 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	1.50 ft
Heel Width	=	5.00
Total Footing Width	=	6.50
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 4,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	2,461	959	psf
Mu' : Upward	=	31,665	12,141	ft-#
Mu' : Downward	=	3,948	19,210	ft-#
Mu: Design	=	2,310	7,068	ft-#
Actual 1-Way Shear	=	13.60	11.50	psi
Allow 1-Way Shear	=	50.60	94.87	psi

Toe Reinforcing = None Spec'd
Heel Reinforcing = # 6 @ 16.00 in
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi'5'lambda'sqrt(fc)'Sm
Heel: #4@ 9.25 in, #5@ 14.35 in, #6@ 20.37 in, #7@ 27.77 in, #8@ 36.57 in, #9@ 46
Key: No key defined

Min footing T&S reinf Area 1.68 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		1,579.4	3.17	5,001.4
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=	109.8	4.75	521.7
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Seismic Earth Load	=	934.5	5.70	5,326.8
	=			
Total	=	2,623.7	**O.T.M.** =	10,849.8

Resisting/Overturning Ratio = **2.81**
Vertical Loads used for Soil Pressure = 8,474.7 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		4,460.5	4.33	19,328.9
Soil Over HL (bel. water tbl)			4.33	19,328.9
Watre Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	1,280.0	1.73	1,210.4
* Axial Live Load on Stem	=	580.0	1.73	1,002.9
Soil Over Toe	=	75.0	0.75	56.3
Surcharge Over Toe	=			
Stem Weight(s)	=	916.7	1.83	1,680.6
Earth @ Stem Transitions	=			
Footing Weight	=	975.0	3.25	3,168.8
Key Weight	=			
Vert. Component	=	767.5	6.50	4,988.9
Total =		7,894.7 lbs	**R.M.=**	30,433.8

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

If seismic is included, the OTM and sliding ratios may be 1.1 per section 1807.2.3 of IBC.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci

Horizontal Defl @ Top of Wall (approximate only) 0.067 in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

Cantilevered Retaining Wall

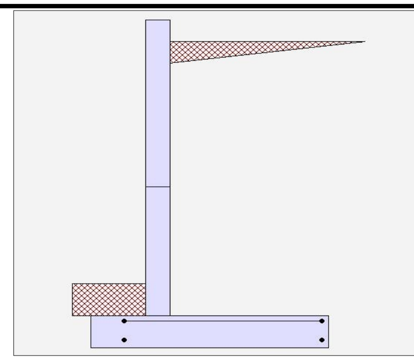
Code: IBC 2015,ACI 318-14,ACI 530-13

Criteria

Retained Height	=	8.50 ft
Wall height above soil	=	0.67 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	450.0 psf/ft
Soil Density, Heel	=	120.00 pcf
Soil Density, Toe	=	120.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel = 40.0 psf
NOT Used To Resist Sliding & Overturning
Surcharge Over Toe = 0.0 psf
NOT Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	100.0 lbs
Axial Live Load	=	130.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	50.0 #/ft
...Height to Top	=	12.00 ft
...Height to Bottom	=	11.00 ft
Load Type	=	Live Load (L) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	4.76 OK
Sliding	=	1.66 OK
Total Bearing Load	=	7,482 lbs
...resultant ecc.	=	4.03 in
Soil Pressure @ Toe	=	714 psf OK
Soil Pressure @ Heel	=	1,354 psf OK
Allowable	=	1,500 psf

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,025 psf
ACI Factored @ Heel	=	1,945 psf
Footing Shear @ Toe	=	4.7 psi OK
Footing Shear @ Heel	=	4.2 psi OK
Allowable	=	94.9 psi

Sliding Calcs

Lateral Sliding Force	=	1,740.2 lbs
less 100% Passive Force	= -	675.0 lbs
less 100% Friction Force	= -	2,205.7 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 Stability	=	0.0 lbs OK

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	IBC 2015,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		2nd	Bottom
		Stem OK	Stem OK
Design Height Above Ftg	ft =	4.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Design Method	=	LRFD	LRFD
Thickness	=	8.00	8.00
Rebar Size	=	# 5	# 6
Rebar Spacing	=	16.00	8.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	**0.260**	**0.480**
Total Force @ Section			
Service Level	lbs =		
Strength Level	lbs =	731.0	2,261.7
Moment....Actual			
Service Level	ft-# =		
Strength Level	ft-# =	1,639.5	7,326.2
Moment.....Allowable	ft-# =	6,294.3	15,260.6
Shear.....Actual			
Service Level	psi =		
Strength Level	psi =	9.8	33.5
Shear.....Allowable	psi =	94.9	94.9
Anet (Masonry)	in2 =		
Rebar Depth 'd'	in =	6.19	5.63
Masonry Data			
f'm	psi =		
Fs	psi =		
Solid Grouting	=		
Modular Ratio 'n'	=		
Wall Weight	psf =	100.0	100.0
Short Term Factor	=		
Equiv. Solid Thick.	=		
Masonry Block Type	=	Medium Weight	
Masonry Design Method	=	ASD	
Concrete Data			
f'c	psi =	4,000.0	4,000.0
Fy	psi =	60,000.0	60,000.0

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0621 in2/ft		
(4/3) * As :	0.0828 in2/ft	Min Stem T&S Reinf Area 0.992 in2	
200bd/fy : 200(12)(6.1875)/60000 :	0.2475 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.2325 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.3411 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.3068 in2/ft		
(4/3) * As :	0.4091 in2/ft	Min Stem T&S Reinf Area 0.768 in2	
200bd/fy : 200(12)(5.625)/60000 :	0.225 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.3068 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.66 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.2192 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	1.50 ft
Heel Width	=	5.00
Total Footing Width	=	6.50
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 4,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,025	1,945	psf
Mu' : Upward	=	14,794	13,833	ft-#
Mu' : Downward	=	4,374	19,056	ft-#
Mu: Design	=	868	5,223	ft-#
Actual 1-Way Shear	=	4.68	4.20	psi
Allow 1-Way Shear	=	50.60	94.87	psi

Toe Reinforcing = None Spec'd
Heel Reinforcing = # 6 @ 16.00 in
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi'5'lambda'sqrt(fc)'Sm
Heel: #4@ 9.25 in, #5@ 14.35 in, #6@ 20.37 in, #7@ 27.77 in, #8@ 36.57 in, #9@ 46
Key: No key defined

Min footing T&S reinf Area 1.68 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

This Wall in File: M:\PROJECTS\Altman, Benjamin\2020-0196 Altman East Lot\Documents\Revision 1\Found

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		1,579.4	3.17	5,001.4
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=	110.8	4.75	526.5
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=	50.0	12.50	625.0
Load @ Stem Above Soil	=			
	=			
Total	=	1,740.2	**O.T.M.** =	6,152.8

Resisting/Overturning Ratio = **4.76**
Vertical Loads used for Soil Pressure = 7,482.2 lbs

.....RESISTING.....

Item		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		4,420.0	4.33	19,153.3
Soil Over HL (bel. water tbl)			4.33	19,153.3
Watre Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	230.0	1.83	183.3
* Axial Live Load on Stem	=	130.0	1.83	238.3
Soil Over Toe	=	180.0	0.75	135.0
Surcharge Over Toe	=			
Stem Weight(s)	=	916.7	1.83	1,680.6
Earth @ Stem Transitions	=			
Footing Weight	=	975.0	3.25	3,168.8
Key Weight	=			
Vert. Component	=	760.5	6.50	4,943.5
Total =		7,352.2 lbs	**R.M.=**	29,264.6

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.067	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

This Wall in File: M:\PROJECTS\Altman, Benjamin\2020-0196 Altman East Lot\Documents\Revision 1\Found

Cantilevered Retaining Wall

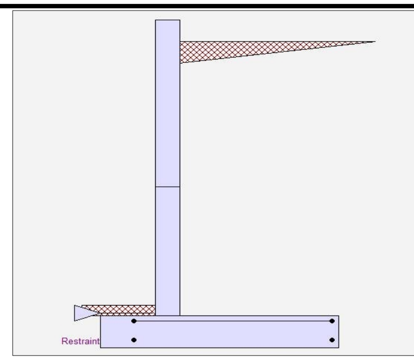
Code: IBC 2015,ACI 318-14,ACI 530-13

Criteria

Retained Height	=	8.50 ft
Wall height above soil	=	0.67 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	4.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	450.0 psf/ft
Soil Density, Heel	=	121.10 pcf
Soil Density, Toe	=	150.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel = 40.0 psf
NOT Used To Resist Sliding & Overturning
Surcharge Over Toe = 40.0 psf
NOT Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	5.29	OK

Slab Resists All Sliding !

Total Bearing Load	=	7,195 lbs	
...resultant ecc.	=	6.74 in	
Soil Pressure @ Toe	=	476 psf	OK
Soil Pressure @ Heel	=	1,501 psf	NG
Allowable	=	1,500 psf	

Soil Pressure Exceeds Allowable!

ACI Factored @ Toe	=	691 psf	
ACI Factored @ Heel	=	2,178 psf	
Footing Shear @ Toe	=	3.1 psi	OK
Footing Shear @ Heel	=	3.1 psi	OK
Allowable	=	94.9 psi	

Sliding Calcs

Lateral Sliding Force = 1,689.2 lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	IBC 2015,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		2nd	Bottom
		Stem OK	Stem OK
Design Height Above Ftg	ft =	4.00	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Design Method	=	LRFD	LRFD
Thickness	=	8.00	8.00
Rebar Size	=	# 5	# 6
Rebar Spacing	=	16.00	8.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	**0.164**	**0.419**
Total Force @ Section			
Service Level	lbs =		
Strength Level	lbs =	650.2	2,180.2
Moment....Actual			
Service Level	ft-# =		
Strength Level	ft-# =	1,037.8	6,400.0
Moment.....Allowable	ft-# =	6,294.3	15,260.6
Shear.....Actual			
Service Level	psi =		
Strength Level	psi =	8.8	32.3
Shear.....Allowable	psi =	94.9	94.9
Anet (Masonry)	in2 =		
Rebar Depth 'd'	in =	6.19	5.63
Masonry Data			
f'm	psi =		
Fs	psi =		
Solid Grouting	=		
Modular Ratio 'n'	=		
Wall Weight	psf =	100.0	100.0
Short Term Factor	=		
Equiv. Solid Thick.	=		
Masonry Block Type	=	Medium Weight	
Masonry Design Method	=	ASD	
Concrete Data			
f'c	psi =	4,000.0	4,000.0
Fy	psi =	60,000.0	60,000.0

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0393 in2/ft		
(4/3) * As :	0.0524 in2/ft	Min Stem T&S Reinf Area 0.992 in2	
200bd/fy : 200(12)(6.1875)/60000 :	0.2475 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.2325 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.3411 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.268 in2/ft		
(4/3) * As :	0.3574 in2/ft	Min Stem T&S Reinf Area 0.768 in2	
200bd/fy : 200(12)(5.625)/60000 :	0.225 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.268 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.66 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.2192 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	1.50 ft
Heel Width	=	5.00
Total Footing Width	=	6.50
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 4,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	691	2,178	psf
Mu' : Upward	=	10,875	13,290	ft-#
Mu' : Downward	=	3,948	19,210	ft-#
Mu: Design	=	577	5,920	ft-#
Actual 1-Way Shear	=	3.12	3.05	psi
Allow 1-Way Shear	=	50.60	94.87	psi
Toe Reinforcing	=	None Spec'd		
Heel Reinforcing	=	# 6 @ 16.00 in		
Key Reinforcing	=	None Spec'd		

Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi'5'lambda'sqrt(fc)'Sm
Heel: #4@ 9.25 in, #5@ 14.35 in, #6@ 20.37 in, #7@ 27.77 in, #8@ 36.57 in, #9@ 46
Key: No key defined

Min footing T&S reinf Area 1.68 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		1,579.4	3.17	5,001.4
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=	109.8	4.75	521.7
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
	=			
Total	=	1,689.2	**O.T.M.** =	5,523.0

Resisting/Overturning Ratio = **5.29**
Vertical Loads used for Soil Pressure = 7,194.7 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		4,460.5	4.33	19,328.9
Soil Over HL (bel. water tbl)			4.33	19,328.9
Watre Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	75.0	0.75	56.3
Surcharge Over Toe	=			
Stem Weight(s)	=	916.7	1.83	1,680.6
Earth @ Stem Transitions	=			
Footing Weight	=	975.0	3.25	3,168.8
Key Weight	=			
Vert. Component	=	767.5	6.50	4,988.9
Total =		7,194.7 lbs	**R.M.=**	29,223.4

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci

Horizontal Defl @ Top of Wall (approximate only) 0.080 in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.