12-200133RS

BUILDING SUPPLY

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POST FRAME BUILDING STRUCTURAL CALCULATION

(This structure has been analyzed and designed for structural adequacy only.)

PROJECT No.
MW12185

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11-12-12

REFERENCES:

1. 2009 Edition of the International Building Code

2. ASCE 7-05 - Minimum Design Loads for Buildings and Other Structures
American Society of Civil Engineers, 2006

3. 2005 Edition, National Design Specification (NDS) Supplement For Wood
Construction, American Wood Counsel

4. ASABE EP486.1 - Shallow-post Foundation Design
American Society of Agricultural and Biological Engineers, 2000

DESIGN INPUT VALUES:

Building Dimensions

$W_{bldg} := 30$ ft Width of Building

$L_{bldg} := 36$ ft Length of Building

$H_{bldg} := 14$ ft Eave Height of Building

$O_{verhang} := 0$ in Length of Eave Overhang

$R_{pitch} := 3$ / 12 Roof pitch

$B_{ay} := 12$ ft Greatest nominal spacing between eavewall posts

$W_{gableopenings} := 4$ ft Total width of openings in one gable wall

$W_{eaveopenings} := 16$ ft Total width of openings in one eave wall

Design Loads for Building:

Occ_Category := "II"

Wind Design Values:

Fastest wind speed (3 second gust)

$V_{wind} := 95$ MPH

Wind Exposure:

$E_{xposure} :=$ "B"

Roof Load Design Values:

$p_g := 25$ lbs Ground snow load

$p_d := 3$ lbs Roof dead load

$p_{d2} := 0$ lbs Additional truss bottom chord dead load (if applicable)

Seismic Design Values:

Site_class := "D"

$S_s := 1.000$ Mapped spectral acceleration for short period

$S_1 := 0.339$ Mapped spectral acceleration for 1 second period

$I_E = 1.00$ Importance factor

$R_s := 7$ Response modification factor

DESIGN INPUT VALUES (Continued):

Structural Members for Building:

Post Properties:

$P_{width} := 6$ in Post width y-axis

$P_{depth} := 6$ in Post depth x-axis

POST SIZE (Solid rough-sawn Hem-Fir post unless otherwise specified)

Grade := "2" Grade of Post (2, 1, or SS = Select Structural)

Purlin Properties:

$P_{urlin\_spacing} := 24$ in

$S_{purlin} := S_{x26}$

$F_{purlin} := F_{bMSR1650}$

Girt Properties:

$G_{irt\_spacing} = 22.6$ in

Sgirt = "Sy26"

$F_{girt} := F_{bMSR1650}$

Footing and Post Hole Design Values:

$q_{soil} := 1500$ psf Assumed soil vertical bearing capacity

$S_{soil} = 150$ psf Assumed soil lateral bearing capacity

$d_{ia\_footing} := 2$ ft Main truss post footing diameter

Slab and backfill information

Concrete_slab = "Optional (may be installed but not required for post constraint)"

Concrete_backfill = "Yes" Backfill in main posts

(GO TO LAST PAGE FOR SUMMARY OF RESULTS)

SNOW LOAD ANALYSIS:

Design per ASCE 7-05
For roof slopes greater than 5 degrees, and less than 70 degrees.

$p_g = 25$ psf Ground Snow Load (from above)

$C_e := 1.0$ Exposure factor

$C_t := 1.0$ Thermal Factor

$C_s := 1$ Roof slope factor

$I_s = 1.00$ Importance factor

$p_f =$ Flat roof snow load, psf (see analysis below)

$p_s =$ Sloped roof snow load, psf (see analysis below)

1. Determine p_f

$p_f := .7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g$ Equation 1

$p_f = 17.5$ psf

$p_s := p_f \cdot C_s$ Equation 2

$p_s = 17.5$ psf This is the balanced snow load on the roof (used for seismic design only).

2. Determine final snow load, p_{su}

$p_{su} := \max(p_g \cdot I_s, 25)$

$p_{su} = 25$ psf **This is the snow load to be used for design of all structural components except for seismic loads.**

WIND ANALYSIS:

Design per ASCE 7-05
Method 2 - Analytical Procedure

$V_{wind} = 95$ Basic Wind Speed

$k_d := .85$ Wind Directionality Factor

$k_{zt} = 1.0$ Topographic Factor

$k_z = 0.701$ Wind Exposure Factor

$I_w = 1.00$ Importance factor

$q_h := .00256 \cdot k_z \cdot k_{zt} \cdot k_d \cdot V_{wind}^2 \cdot I_w$ Velocity Pressure

$q_h = 13.76$ psf

Calculated Wind Pressures:

Windward Eave Wall:

$q_{ww} := q_h \cdot GC_{pfww}$

$q_{ww} = 6.58$ psf

Leeward Eave Wall:

$q_{lw} := q_h \cdot GC_{pflw}$

$q_{lw} = -5.15$ psf

Windward Gable Wall:

$q_{wwg} := q_h \cdot GC_{pfwwg}$

$q_{wwg} = 5.50$ psf

Leeward Gable Wall:

$q_{lwg} := q_h \cdot GC_{pflwg}$

$q_{lwg} = -3.99$ psf

Windward Roof:

$q_{wr} := q_h \cdot GC_{pfwr}$

$q_{wr} = -9.49$ psf

Leeward Roof:

$q_{lr} := q_h \cdot GC_{pflr}$

$q_{lr} = -6.00$ psf

Wall Elements:

$q_{we} := q_h \cdot GC_{pfw}$

$q_{we} = -13.35$ psf

Roof Elements:

$q_r := q_h \cdot GC_{pfr}$

$q_r = -18.57$ psf

Internal Wind Pressure (+/-):

$q_i := q_h \cdot GC_{pi}$

$q_i = 2.48$ psf

BUILDING MODEL:

STEP 1: DETERMINE THE SHEAR STIFFNESS OF THE TEST PANEL

This procedure relies on tests conducted by the National Frame Builders Association.

The test was conducted using 29 gauge ribbed steel panels. These ribbed steel panels are similar to Strongpanel, Norclad, and Delta-Rib which are in common use by builders in this area. The material and section properties for the test panels are thus reasonable and will be used throughout.

The stiffness of the test panel was calculated to be: c = 2166 lb/in

STEP 2: CALCULATED ROOF DIAPHRAGM STIFFNESS OF THE TEST PANEL

c' = (E X t) / (2 X (1+V) X (g/p) + (K_2 / (b' X t)^2))

Where:	E =	27.5x10^6 psi (modulus of elasticity for steel)
	t =	0.017" (thickness of 29 gauge steel)
	V =	0.3 (Poisson's Ratio for steel)
	g/p =	1.139 ratio of sheathing corrugation length to corrugation pitch
	b' =	144" (12'-0" length of test panel)

STEP 2.1

This equation was set equal to the stiffness of the test panel (2166 lb/in) and the unknown value (K_2) was solved for.

K_2 = 1275 in^4 sheet edge purlin fastening constant

STEP 2.2:

Use new building width to determine stiffness of new roof diaphragm (c_h

$$b_{new} := \frac{\frac{W_{bldg} \cdot 12}{2}}{\cos(\Theta)}$$

$K_2 := 1275$ lbf / ft

$t := 0.017$ in $\Theta = 14.036\,deg$ (Angle of roof pitch from horizontal)

$b_{new} = 186$ in

$E := 27500000$

$$c := \frac{E \cdot t}{2.961 + \frac{K_2}{(b_{new} \cdot t)^2}}$$

$c = 3566$ lbf / in

STEP 2.3 & 2.4:

Calculate the equivalent horizontal roof stiffness (c_h) for the full roof:

Since c_h is for the full roof, the roof length must be ratioed by the aspect ratio of the roof panel (b / a) where "a" is the truss spacing in inches.

$a := B_{ay} \cdot 12$

$$c_h := 2 \cdot c \cdot \cos(\Theta)^2 \cdot \frac{b_{new}}{a}$$

$a = 144$ in

$c_h = 8648$ lbf / in

STEP 3: DETERMINE THE STIFFNESS OF THE POST FRAME (k):
Since the connection between the posts and the rafters can be assumed to be a pinned joint, the model for the post frame can be assumed to be the sum of two cantilevers (the posts) that act in parallel. The stiffness of the post frame can be calculated from the amount of force required to deflect the system one inch. The spring constant (k) in pounds per inch of deflection results directly.

$k = 68$ lbf/in

STEP 4: DETERMINE THE TOTAL SIDE SWAY FORCE (R):
Apply wind loads to the walls to determine the moment, fiber stress and end reaction at prop point R.

Calculate Total Wind Pressure:

$q_e := \text{if}(q_{ww} - q_{lw} \le 10, 10, q_{ww} - q_{lw})$ $\quad q_e = 11.73$ psf

$q_{wwpost} := q_e \cdot \left(\frac{a}{12 \cdot 12}\right)$ $\quad q_{wwpost} = 11.73$ pli

$M_{wind} := q_{wwpost} \cdot \frac{L_{post\_bndg}^{2}}{8}$ $\quad M_{wind} = 35692$ in-lbf

$f_{wind} := \frac{M_{wind}}{S_{xeavepost}}$ $\quad f_{wind} = 496$ psi

$R := 3 \cdot q_{wwpost} \cdot \frac{L_{post\_bndg}}{8}$ $\quad R = 686$ lbs

STEP 5: DETERMINE THE RATIO OF THE FRAME STIFFNESS TO THE ROOF STIFFNESS:
This ratio (k/c_h) will be used to determine the side sway force modifiers.

$\frac{k}{c_h} = 0.008$

STEP 6: DETERMINE SIDE SWAY RESISTANCE FORCE:

$mD = 0.99$

STEP 7: DETERMINE THE ROOF DIAPHRAGM SIDE SWAY RESISTANCE FORCE:

$Q := mD \cdot R$ $\quad Q = 681$ lbf

Since not all of the total side sway force (R) is resisted by the roof diaphragm, some translation will occur at the top of the post. The distributed load that is not resisted by the roof diaphragm will apply additional moment and fiber stress to the post.

$M_{dfl} = 1127$ in-lbf $\quad f_{dfl} = 16$ psi

Calculate the total moment and the total fiber stress in the post.

$M_{tot} := mD \cdot M_{wind} + M_{dfl}$ $\quad M_{tot} = 36537$ in-lbf

$f_{tot} := mD \cdot f_{wind} + f_{dfl}$ $\quad f_{tot} = 507$ psi

MAIN POST DESIGN:

Calculate allowable unit compression stress, F_{cc}.

$F_{c1} = 575$ psi $\quad F_c := F_{c1} \cdot 1.15$

$F_c = 661$ psi Allowable compression stress including load factors

$L_{post\_bndg} = 156$ in Bending length of post $\quad d_{post} = 6$ in Minimum unbraced dimension of post

$K_e := 0.8 \quad c := 0.8 \quad E_{wood} = 400000$ psi

$I_e := K_e \cdot L_{post\_bndg} \quad I_e = 124.8$ in

$$F_{cE} := \frac{.822 \cdot E_{wood}}{\left(\frac{I_e}{d_{post}}\right)^2} \qquad F_{cE} = 760$$

Calculate Column Stability Factor, C_p:

$$C_p := \left[\left(\frac{1 + \frac{F_{cE}}{F_c}}{2 \cdot c}\right) - \sqrt{\left(\frac{1 + \frac{F_{cE}}{F_c}}{2 \cdot c}\right)^2 - \frac{\frac{F_{cE}}{F_c}}{c}}\right] \qquad C_p = 0.74$$

$F_{cc} := F_c \cdot C_p$

$F_{cc} = 487$ psi Allowable compression stress on the post

$W_{roof} = 28$ psf Total roof loading

$P_{snowpost} = 4500$ lbs Axial loading per post due to roof snow load

$P_{deadpost} = 540$ lbs Axial loading per post due to roof dead load

$F_b := F_{b1} \cdot 1.6$

$F_b = 920$ psi Allowable bending stress per post including load factors

Check Load Cases:

Load Case 1: Dead Load + .75 * Wind Load + .75 * Snow Load

$f_{b1} := .75 f_{tot}$ $f_{b1} = 381$ psi Actual bending stress on post

$f_c := \frac{.75 P_{snowpost} + P_{deadpost}}{A_{post}}$ $f_c = 109$ psi Actual compression stress per post

$$CCFALI1 := \left[\left(\frac{f_c}{F_{cc}} \right)^2 + \frac{f_{b1}}{F_b \cdot \left(1 - \frac{f_c}{F_{cE}} \right)} \right]$$ $CCFALI1 = 0.53$

Load Case 2: Dead Load + Wind Load

$f_{b1} := f_{tot}$ $f_{b1} = 507$ psi (Actual bending stress on post)

$f_c := \frac{P_{deadpost}}{A_{post}}$ $f_c = 15$ psi (Actual compression stress per post)

$$CCFALI2 := \left[\left(\frac{f_c}{F_{cc}} \right)^2 + \frac{f_{b1}}{F_b \cdot \left(1 - \frac{f_c}{F_{cE}} \right)} \right]$$ $CCFALI2 = 0.56$

Load Case 3: Dead Load + Snow Load

$f_{b1} := 0$ $f_{b1} = 0$ psi (Actual bending stress on post)

$f_c := \frac{P_{snowpost} + P_{deadpost}}{A_{post}}$ $f_c = 140$ psi (Actual compression stress per post)

$$CCFALI3 := \left(\frac{f_c}{F_{cc}} \right)$$ $CCFALI3 = 0.29$

$CCFALI = 0.56$ Less than or equal to 1.00 thus OK

SEISMIC CALCULATIONS:

Design per ASCE 7-05

$S_s = 1.00$ Mapped spectral acceleration for short periods (from above)

$S_1 = 0.34$ Mapped spectral acceleration for 1-second period (from above)

$I_E = 1.0$ Importance factor

W = Dead load of building

$R_s = 7$ Response modification factor (from above)

1. Determine the Seismic Design Category

a. Calculate S_{DS} and S_{D1}

For S_{DS}:

For $S_s = 1.00$

$F_a = 1.10$

$S_{MS} := S_s \cdot F_a$

$S_{MS} = 1.10$

$S_{DS} := \left(\frac{2}{3}\right) \cdot S_{MS}$

$S_{DS} = 0.73$

For S_{D1}:

For $S_1 = 0.34$

$F_v = 1.72$

$S_{M1} := S_1 \cdot F_v$

$S_{M1} = 0.58$

$S_{D1} := \left(\frac{2}{3}\right) \cdot S_{M1}$

$S_{D1} = 0.39$

Seismic_Design_Category = "D"

2. Determine the building parameters

Building dead load weight, W:

$$W := \left[\left(W_{bldg} \cdot L_{bldg}\right) \cdot \left(p_f \cdot 2\right)\right] + \left[\left(W_{bldg} \cdot L_{bldg}\right) + \left[2 \cdot \left(W_{bldg} + L_{bldg}\right) \cdot \frac{H_{bldg}}{2}\right]\right] \cdot p_d$$

$W = 6012$ lbf

Building area, A_b:

$A_b := L_{bldg} \cdot W_{bldg}$ $A_b = 1080$ ft^2

3. Determine the shear force to be applied

a. Determine the structural period, T

$T_a := .02 \cdot (H_{bldg} + H_{roof})^{.75}$ $T := T_a$ $T = 0.17$

b. Detemine the Seismic Response Coefficient, Cs:

Cs is calculated as:

$$C_{s2} := \frac{S_{DS}}{\frac{R_s}{I_E}} \qquad C_{s2} = 0.105$$

But shall not be less than:

$$C_{s1} := \text{if}\left[S_1 \geq 0.6, \left(\frac{0.5 \cdot S_1}{\frac{R_s}{I_E}}\right), 0.01\right] \qquad C_{s1} = 0.010$$

But need not exceed:

$$C_{s3} := \frac{S_{D1}}{T \cdot \left(\frac{R_s}{I_E}\right)} \qquad C_{s3} = 0.321$$

$C_s = 0.105$

c. Detemine the Seismic Base Shear:

$V_{base\_shear} := C_s \cdot W$ $V_{base\_shear} = 630$ lbf

4. Determine the seismic load on the building:

Per ASCE 7-05

Since $\text{Seismic\_Design\_Category} = \text{"D"}$, $\rho = 1.3$

$E := \rho \cdot V_{base\_shear}$

$E = 819$ lbf Seismic load on building

DETERMINE GABLE WALL SHEAR LOADS:

1. Determine the wind load on the eave wall to be resisted by the gable wall in shear:

$q_e = 11.7$ psf Eave wall wind pressure from above

$$Veave\_wind := \frac{(0.375 \cdot mD \cdot H_{bldg} \cdot L_{bldg} \cdot q_e) + (H_{roof} \cdot L_{bldg} \cdot q_{roof})}{2}$$

$Veave\_wind = 1100$ lbf

2. Determine the seismic load to be resisted by the gable wall in shear:

$Veave\_seismic := \frac{E}{2}$ $\quad Veave\_seismic = 409$ lbf

3. Determine the controlling load to be resisted by the gable wall in shear:

The *controlling* load = "Veave_wind" . Therefore, $V_{gable\_shear} = 1100$ lbf

V_{gable_shear} is the shear load that is transmitted through the roof diaphragm to each gable wall. Normalize the load to a per foot basis.

$$v_{gablewall} := \frac{V_{gable\_shear}}{W_{bldg} - W_{gableopenings}} \qquad v_{gablewall} = 42 \text{ plf}$$

The gable wall diaphragms can resist the shear loads as follows:

$v_{gablewall} < 142$ plf	Use 29 gauge metal sheathing. Install per the Typical Screw Schedule as shown on the Standard Details drawing in the engineered drawing package.

DETERMINE EAVE WALL SHEAR LOADS:

1. Determine the wind load on the gable wall to be resisted by the eave wall in shear:

$$q_g := \text{if}\left(q_{wwg} - q_{lwg} \le 10, 10, q_{wwg} - q_{lwg}\right)$$

$q_g = 10$ psf Gable wall wind pressure

$H_{roof} = 3.75\,ft$

$$Vgable\_wind := \frac{0.375 \cdot mD \cdot H_{bldg} \cdot W_{bldg} \cdot q_g + 0.5 \cdot H_{roof} \cdot W_{bldg} \cdot q_g}{2}$$

$Vgable\_wind = 1063$ lbf

2. Determine the seismic load to be resisted by the eave wall in shear:

$Vgable\_seismic := \dfrac{E}{2}$ $\quad$ $Vgable\_seismic = 409$ lbf

3. Determine the controlling load to be resisted by the eave wall in shear:

The controlling $load = \text{"Vgable\_wind"}$. Therefore, $V_{eave\_shear} = 1063$ lbf

V_{eave_shear} is the shear load that is transmitted through the roof diaphragm to each eave wall. Normalize the load to a per foot basis.

$v_{eavewall} := \dfrac{V_{eave\_shear}}{L_{bldg} - W_{eaveopenings}}$ $\quad$ $v_{eavewall} = 53$ plf

The eave wall diaphragms can resist the shear loads as follows:

$v_{eavewall} < 142$ plf	Use 29 gauge metal sheathing. Install per the Typical Screw Schedule as shown on the Standard Details drawing in the engineered drawing package.

EMBEDMENT FOR MAIN POST:

Calculate the minimum required post embedment depth for lateral loading for the main posts.

Post_is = "not constrained by a concrete slab"

$V_a = 619$ lbf Lateral shear load at the groundline

$M_a = 3045$ ft-lbf Moment at the groundline

$d_{ia\_footing} = 2$ ft. Main post footing diameter

$S_{soil} = 150$ psf Lateral capacity of soil

Trial depth = 1.5 ft.- The starting depth of the post hole depth. The final post hole depth is determined by iterating to a final depth

$d_{epth\_post} = 2.6$ ft. This is the minimum required post embedment depth for lateral loading

Gable wall uplift due to shear loading on gable wall shear panel:

Calculate uplift pullout of the gable wall posts due to shear loads on the gable walls.

$Veave\_wind = 1100$ lbf Calculated from above

$$C_{post} := \frac{Veave\_wind \cdot H_{bldg}}{W_{bldg} - W_{gableopenings}}$$

$C_{post} = 592$ lbf This is the uplift load on one gable wall post

Assume a dead load weight of roof and wall area to be 2.0 psf. The area of the roof and wall that will tend to keep the gable wall post in the ground will be as follows:

$$R_{oof} := \frac{B_{ay}}{2} \cdot W_{bldg} \cdot 2$$

$R_{oof} = 360$ lbs Dead load of roof

$$G_{able\_wall} := \left[H_{bldg} \cdot \left(W_{bldg} - W_{gableopenings} \right) + \left(H_{roof} \cdot \frac{W_{bldg}}{2} \right) + \left(H_{bldg} \cdot \frac{2 \cdot B_{ay}}{2} \right) \right] \cdot 2$$

$G_{able\_wall} = 1176.5$ lbf Dead load of gable wall

$d_{epth\_gable\_footing} = 4.0$ ft gable post embedment depth

$$P_{osts} := \left(H_{bldg} + d_{epth\_gable\_footing} \right) \cdot W_{post}$$

$P_{osts} = 158$ lbs Weight of post

$d_{ia\_gable\_footing} = 2$ ft Diameter of gable wall posthole footing

Concrete backfill in the gable end posts is = "not required" to resist gable wall panel uplift.

$Backfill = 1734$ lbs Gable post backfill weight if gable end post hole is backfilled with concrete (0 if granular or native soil backfill. Concrete backfill may or may not be required to resist gable wall panel uplift).

$$Wt_{tot} := G_{able\_wall} + R_{oof} + P_{osts} + Backfill$$

$Wt_{tot} = 3428$ lbf Total resistance for gable wall panel uplift. Since $\mathbf{Wt_{tot}}$ is greater than the gable wall panel uplift, $\mathbf{C_{post}}$, the gable wall footing is adequate.

FOOTING DESIGN FOR MAIN POST:

Determine the footing size and depth for vertical bearing for the main posts.

$$A_{footing} := \pi \cdot \left(\frac{d_{ia\_footing}^{\ 2}}{4} \right)$$

$A_{footing} = 3.14$ ft^2 Footing area

$q_{soil} = 1500$ psf Soil bearing capacity for footing

$d_{ia\_footing} = 2$ ft Footing diameter

$P_{ost\_depth} = 4$ ft Minimum required post embedment depth

$P_{footing} := A_{footing} \cdot q_{soil} \cdot d_{factor}$ $P_{footing} = 7540$ lbf End bearing capacity of footing

$P_{snow} = 5040$ lbf Total footing load

Note that the end bearing capacity ($P_{footing}$) is greater than the snow load (P_{snow}). This is OK.

GIRT DESIGN:

The girts will simple span between posts and loaded horizontally for wind. Calculate bending stress due to wind loading and determine the adequacy of the girts.

$q_{wegirt} := q_{wind\_girt} \cdot \frac{G_{irt\_spacing}}{12 \cdot 12}$ $\quad q_{wegirt} = 2.48$ pli $\quad L_{girt\_span} = 138$ in $\quad$ Orientation = "Flat"

$M_{girt} := q_{wegirt} \cdot \frac{L_{girt\_span}^{2}}{8}$ $\quad M_{girt} = 5911$ in-lbf

$f_{bgirt} := \frac{M_{girt}}{S_{girt}}$ $\quad f_{bgirt} = 2870$ psi $\quad$ Stress applied to the girt

Determine the allowable member stress including load factors.

$LDF_{wind} := 1.6$ $\quad C_{fugirt} = 1.15$ $\quad C_{Fgirt} = 1.00$ $\quad C_r := 1.15$ $\quad F_{girt} = 1650$ psi

$F_{bgirt} := LDF_{wind} \cdot C_{fugirt} \cdot C_{Fgirt} \cdot C_r \cdot F_{girt}$ $\quad F_{bgirt} = 3491$ psi > f_{bgirt} This is OK.

PURLIN DESIGN:

The purlins simply span between pairs of trusses or rafters. Determine the adequacy of the purlins.

Purlin = "2x6" $\quad P_{urlin\_spacing} = 24$ in o.c.

$L_{purlin\_span} = 135$ in $\quad$ Bending length of purlin

$w_{purlin} = 4.53$ pli $\quad$ Distributed snow load along top edge of purlin

$M_{purlin} := \frac{w_{purlin} \cdot L_{purlin\_span}^{2}}{8}$ $\quad M_{purlin} = 10314$ in-lbf $\quad$ Bending moment in the purlin

$f_{bpurlin} := \frac{M_{purlin}}{S_{purlin}}$ $\quad f_{bpurlin} = 1364$ psi $\quad$ Bending stress applied to the purlin

Determine the allowable member stress including load factors

$LDF_{snow} := 1.15$ $\quad C_{Fpurlin} = 1.00$ $\quad C_r := 1.15$ $\quad C_{fupurlin} = 1.00$ $\quad F_{purlin} = 1650$ psi

$F_{bpurlin} := LDF_{snow} \cdot C_{Fpurlin} \cdot C_r \cdot C_{fupurlin} \cdot F_{purlin}$ $\quad F_{bpurlin} = 2182$ psi > $f_{bpurlin}$ This is OK

MAIN POST CORBEL BLOCK DESIGN:

Determine the required number and size of bolts required in the main post corbel block.
Assume full snow load and dead load on the roof.

Allowable fastener shear capacities

$p_{bolt\_58} := 1590$ lbf Shear capacity for 5/8" dia. bolts

$p_{bolt\_34} := 2190$ lbf Shear capacity for 3/4" dia. bolts

$p_{20d} := 147$ lbf Shear capacity for 20d nails

$P_{snow} = 5040$ lbf Combined snow and dead load on corbels

If 5/8 dia. bolts are used:

$N_{bolts58} = 2.8$ Number of 5/8" dia. bolts required in the corbel block

If 3/4 dia. bolts are used:

$N_{bolts34} = 2.0$ Number of 3/4" dia. bolts required in the corbel block

If 20d nails are to be used:

$N_{ails20d} = 14.9$ number of 20d nails required in each corbel block.

SUMMARY OF RESULTS:

Building Dimensions

$W_{bldg} = 30$ ft (Width of Building)

$L_{bldg} = 36$ ft (Length of Building)

$H_{bldg} = 14$ ft (Eave Height of Building)

$O_{verhang} = 0$ in (Length of Eave Overhang)

$R_{pitch} = 3$ / 12 (Roof pitch)

Building Design Loads

Wind_speed = 95 MPH

Ground_snow_load = 25 psf

Wind_exposure = "B"

Roof_dead_load = 3 psf

Seismic_Design_Category = "D"

Post Details

Post_size = "6x6"

Post_grade = "No. 2 Hem-Fir"

Usage = 56 % (Combined stress usage of post)

Footing Details:

Post_is = "not constrained by a concrete slab"

Postdepth = 4.0 ft (Design Post Depth)

$d_{ia\_footing} = 2$ ft (Design Footing Diameter)

Footingusage = 67 % (Stress usage of footing)

Shear Wall Details:

$v_{gablewall} = 42$ plf (Max. shear in gable wall)

$v_{eavewall} = 53$ plf (Max. shear in eave wall)

Girt Details:

Girt_usage = "82 % (Stress usage of wall girt)"

Orientation = "Flat"

Purlin Details:

Purlin_usage = 63 % (Stress usage of roof purlin for snow loading)

Corbel Block Bolts:

$N_{bolts58} = 2.8$ Number of 5/8" dia. bolts required in the corbel block if used.

$N_{bolts34} = 2.0$ Number of 3/4" dia. bolts required in the corbel block if used.

$N_{ails20d} = 14.9$ Number of 20d nails required in each corbel block if used.

SPECIAL NOTE:
The drawings attendant to this calculation shall not be modified by the builder unless authorized in writing by the engineer. No special inspections are required. No structural observation by the design engineer is required.