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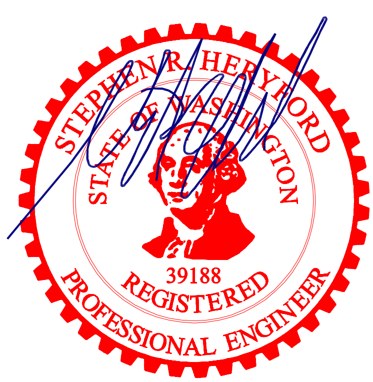
Project No.
11911042

Revision 1
Calculations for
Geoff Sweet
310th Ave. SE
Ravensdale, WA. 98038

Date
7/8/2022

Engineer



POST FRAME BUILDING SUMMARY SHEET

Owner: Geoff Sweet
Date: 7/8/2022
Building location: 310th Ave. SE
Ravensdale, WA. 98038
Project No.: 11911042
Revision: 1
Building Description: Private shop
Building Codes: 2018 IBC, ASCE 7-16

Building dimensions:

Width:	40	ft.
Length:	40	ft.
Height:	16	ft.
Eave overhang:	1.5	ft.
Gable overhang:	1.5	ft.
Roof pitch:	4	/12
Greatest bay spacing:	14	ft.
Greatest post tributary width:	14	ft.
Concrete Slab:	Yes	

Environmental information:

Wind speed:	110	MPH
Wind exposure:	B	
Seismic design category:	D	
S_s:	1.19	
S_1:	0.41	
Ground snow load:	46	psf.
Design Roof Snow Load:	37	psf.
Roof dead load:	3	psf. (incl. ceiling load if any)
Soil bearing capacity:	1,500	psf.
Risk Category:	II	Per Table 1.5-1 ASCE 7-16

Post & posthole information:

Eave wall posts:

Size:	6x8	
Grade:	#2 H-F	
Type:	RS*	
Posthole diameter:	30	in
Posthole depth**:	5.00	ft.
Post Constraint/backfill:	Slab w/ granular backfill	

*Rough Sawn
**To bottom of footing

Gable wall posts:

Size:	6x8	
Grade:	#2 H-F	
Type:	RS*	
Posthole diameter:	24	in
Posthole depth**:	5.00	ft.
Post Constraint/backfill:	Slab w/ granular backfill	

*Rough Sawn
**To bottom of footing

Purlin & girt information:

Purlins

Size:	2x8	
Grade:	#2 D-F	
Spacing:	18	in. o.c.

Girts

Size:	2x6	
Grade:	#2 D-F	
Spacing:	24	in. o.c.
Orientation:	Flat W/ 2x4 Strongbacks @ 48" o.c.	

Sheathing information:

Roof: 29 ga. Metal only

Walls:
Left gable wall is 29 ga. metal only
Right gable wall is 29 ga. metal only
Front eave wall is metal over wood sheathing
Rear eave wall is 29 ga. metal only

Snow Load Calculations

Snow load calculations per ASCE 7-16 Chapter 7

p_g:	46	psf - Ground Snow Load
C_e:	1.0	Exposure Factor from ASCE Table 7-2
C_t:	1.2	Thermal Factor from ASCE Table 7-3
I_s:	1.0	Importance Factor from ASCE Table 1.5-2

Flat Roof Snow Load, $p_f = 0.7 \times p_{g\,x} C_e \times C_t \times I_s$

p_f:	38.6	psf - Flat Roof Snow Load
C_s:	0.95	Figure 7.4-1 based on C_t, roof slope and surface
p_s:	36.7	psf-Sloped roof snow load
p_{design}:	**37**	**psf-Design Roof Snow Load**

Wind Pressure Calculations

Wind calculations per ASCE 7-16 Chapters 26, 28 and 30

Roof Pitch:	4	/12	Design Wind Speed, V:	110	MPH		
Eave Height:	16	ft.	Wind Exposure:	B		Risk Category:	II

Velocity pressures q_h per equation 26.10-1

$$q_h = 0.00256 \times K_h \times K_{zt} \times K_d \times K_e \times V^2 \text{ at mean roof height h}$$

Angle:	18.43	°
K_h:	0.70	Velocity pressure coefficient at roof ht. h from Table 26.10-1
K_{zt}:	1.00	Topographic effect-assume no ridges or escarpments
K_d:	0.85	Wind Directionality Factor, Table 26.6-1
K_e:	1.00	Ground Elevation Factor, Table 26.9-1

Velocity Pressures: q_h= 18.43 psf

Determine Velocity Pressure Coefficients & Wind Pressures per ASCE 7-16 Figure 28.3-1 for MWFRS

MWFRS

1. Windward Eave Wall Pressure

   GC_{pfww}: 0.52

   q_{ww}: 9.52 psf

2. Leeward Eave Wall:

   GC_{pfwr}: -0.42

   q_{lw}: -7.66 psf

3. Windward Eave Roof Pressure

   GC_{pfwr}: -0.69

   q_{wr}: -12.72 psf

4. Leeward Eave Roof:

   GC_{pflr}: -0.47

   q_{lr}: -8.64 psf

5. Windward Gable Wall:

   GC_{pfwg}: 0.40

   q_{lw}: 7.37 psf

6. Leeward Gable Wall:

   C_{pfwg}: -0.29

   q_{lw}: -5.34 psf

Components & Cladding

GC_{pi}: 0.18 Internal pressure per Table 26.13-1

7. Roof elements

   GC_{pr}: -0.81

   q_{er}: 18.32 psf Roof elements per Figure 30.3-2A thru I

8. Wall elements:

   GCpw: -0.94

   q_{er}: 20.66 psf Wall elements per Figure 30.3-1

Seismic Design Parameters

Calculate seismic building loads from ASCE 7-16 Chapter's 11 & 12

Seismic Parameters

S_s=	**1.19**	**S_1=**	**0.41**	
F_a=	1.02	F_v=	1.89	per Tables 11.4-1 & 11.4-2
S_{MS}=	1.22	S_{M1}=	0.77	Calculated per Section 11.4.3
S_{DS}=	**0.81**	**S_{D1}=**	0.52	Calculated per Section 11.4.4

Seismic Design Category=	**D**	From Section 11.6	Importance factor:	1.00
F=	1.0	for 1 story building		

Response Mod. Factor R:

Roof:	**2.5**	From Table 12.14-1, Section B-24
Left gable wall:	**2.5**	From Table 12.14-1, Section B-24
Right gable wall:	**2.5**	From Table 12.14-1, Section B-24
Front eave wall:	**7**	From Table 12.14-1, Section B-22
Rear eave wall:	**2.5**	From Table 12.14-1, Section B-24

Calculate building weights, W, for seismic forces

Building width=	40	ft.	Building length=	40	ft.	Building height=	16	ft.
Roof area=	1,849	sf	Gable wall area=	453	sf	Eave wall area=	320	sf
Roof + ceiling DL=	3	psf	Snow LL (if appliable)=	7.728	psf	Roof W=	19,836	lbs
Loft (y/n):	n		Loft dead load:	N/A	psf	Full or partial loft:	N/A	

Wall Areas			Building dead loads			Loft dead loads		
Left gable wall:	453	SF	Left gable wall:	3	psf	Left gable wall:	0	lbs
Right gable wall:	453	SF	Right gable wall:	3	psf	Right gable wall:	0	lbs
Front eave wall:	320	SF	Front eave wall:	5	psf	Front eave wall:	0	lbs
Rear eave wall:	320	SF	Rear eave wall:	3	psf	Rear eave wall:	0	lbs

Calculate Seismic Base Shear, V per Section 12.14.8

$V=[(FxS_{DS})/R]xW$ (Eqn. 12.14-12)

Total dead loads,W (incl roof, loft)					
Roof:	19,836	lbs	**V_{roof}=**	**6,452**	**lbs base shear for roof diaphragm**
Left gable wall:	1,360	lbs	**V_{LGW}=**	**3,642**	**lbs base shear for wall diaphragm**
Right gable wall:	1,360	lbs	**V_{RGW}=**	**3,642**	**lbs base shear for wall diaphragm**
Front eave wall:	1,600	lbs	**V_{FEW}=**	**1,310**	**lbs base shear for wall diaphragm**
Rear eave wall:	960	lbs	**V_{REW}=**	**3,668**	**lbs base shear for wall diaphragm**

Diaphragm Stiffness Calculation

The diaphragm stiffness will be calculated based on the methodology from "Post Frame Building Design", by John N. Walker and Frank E. Woeste. This method is widely accepted in the post frame industry for determining metal diaphragm stiffness.

1. The diaphragm stiffness, $c' = (Ext) / [2x(1+\upsilon) \times (g/p) + (K_2/(bxt)^2)$

Where:	$c'=$	3130	lbs/in = Diaphragm stiffness of the test panel (1992 Fabral Test for Grandrib III)
	$E=$	2.75E+07	psi = Modulus of elasticity for metal sheathing
	$t=$	0.017	in = Steel thickness for 29 ga metal sheathing
	$\upsilon=$	0.3	= Poisson's ratio for steel
	$g/p=$	1.085	= Ratio of steel corrugation pitch to steel sheet width
	$b=$	144	in. = Length of test panel
	$K_2=$	-	= Sheet edge purlin fastening constant (unknown)

2. The diaphragm for the same metal for a different length b can be calculated with the above above equation once the constant K_2 is known. Solving for K_2 yields:

$K_2 = [((Ext)x(bxt^2))/c]-[2x(1+\upsilon)x(bxt)^2x(g/p)$ $K_2 =$ 878 in^4

3. The stiffness of the acutal panel will be calculated from equation in 1. above, based on its actual length, b'

Roof pitch= 4 /12 Building width= 40 ft $\theta=$ 18.43 ° roof angle

b'= 252.98 in = length of steel roof panel at the given angle for 1/2 of the roof

c= 9294 lbs/in - stiffness of actual roof diaphragm

4. Calculate the equivalent horizontal roof stiffness, c_h for the entire roof

$c_h = 2xcx(\cos^2\theta)x(b'/a)$ $c_h=$ 25,192 lb/in a= 168 in. post spacing

5. Calculate the stiffness, k, of the post frame, which is the load required for the top of the frame a distance, d

For d=1", $k=P=(6xdxE_pxI_p)/L^3$

d=	1	in-deflection used to establish k	$I_p=$	256	in^4 - Momentof inertia of post
$E_p=$	1.10E+06	psi - Modulus of elasticity of post	L=	180	in - Bending length of post
k=	290	lbs/in			

6. Determine the side sway force, mD from tables based on k/c_h verses number of frames.

$N_F=$	4	frames in building (including end walls)	$k/c_h=$	0.0115
mD=	0.99	= calculated stiffness of metal roof diaphragm		

Since roof sheathing is metal, mD used for calculations is $\boxed{\textbf{0.99}}$

Post Wind Load Calculation

Determine the bending stress on the post from the wind load

Windward wall wind pressure =	9.52	psf
Leeward wall wind pressure =	-7.66	psf
Total wind pressure =	17.17	psf
Total wall pressure to use =	17.17	psf (10 psf min. per code)

L=	180	in	Bending length of the post
w=	20.04	pli	Distributed wind load on the post
M_{pc}=	40,574	lbf-in	Moment as a propped cantilever (w x L^2) / (2 x 8)
$f_{b\text{-}pc}$=	634	psi	Stress on the post from the distirbuted wall wind, = M_{pc} / S_x
R=	1,352	lbf	Total side sway force = 3 x w x (L/8)
mD=	0.99		Stiffness coefficient from diaphragm stiffness calculation, or 1.0 if wood sheathing in roof
Q=	1,339	lbf	Side sway force resisted by the roof diaphragm = mD x R
w_R=	19.8	pli	The total distributed wind load resisted by the roof diaphragm = 8 x ((Q/(3 x L))
w_{post}=	0.20	pli	The total distributed wind load NOT resisted by the roof diaphragm for which the post must resist. W_{post} = w - w_R
M_{cant}=	3,246	lbf-in	The moment in the post as a simple cantilever = w_{post} x ((L^2)/2) (This value is 0 if roof is a wood diaphragm)
f_{cant}=	25	psi	The fiber stress in the post from simple cantilever stress = M_{cant}/(2 x S_x) (This value is 0 if roof is a wood diaphragm)
M_{post}=	43,414	lbf-in	The total moment in the post = (mD x M_{pc}) + M_{cant}
$f_{b\text{-}post}$=	653	psi	The total bending stress on the post = (mD x $f_{b\text{-}pc}$) + f_{cant}

Post Design

Determine the allowable bending and compression stresses for the eave wall posts per NDS

Nominal Design Values (allowable)			Adjustment factors per Table 4.3.1		
F_b:	575	psi-bending	C_D for snow	1.15	LDF for snow
F_c:	575	psi-compression	C_D for wind/seismic	1.6	LDF for wind/seismic
			C_D for post	1.0	Size factor for posts $\leq$ 12" in depth
Final Design Values			Cp=	0.82	Column stability factor per Section 3.7

F_{b_design}:	920	psi final allowable bending stress
F_{c_design}:	539	psi final allowable compression stress

Combined Bending And Compressive (CBAC) Post Loads by Load Case

Determine the maximum Combined Bending And Compressive stresses in the eave wall post per NDS 3.9.2 using applicable load cases from ASCE 7-16 Section 2.4.

Load Case 1 - Dead Load + Snow

F_{b_design}:	920	psi	Final allowable bending stress		
F_{c_design}:	539	psi	Final allowable compression stress		
P_{dead}=	903	lbs	Dead load		
P_{snow}=	11047	lbs	Snow load		
A=	48	sq-in	Cross-sectional area of post	F_{cE}=	1,015 psi

f_b= 0 psi=0 f_c= 249 psi= $(P_{snow} + P_{dead})/A$

CBAC1= 0.21 $=((f_c/F_{c\_design})^2)+((f_b/(F_{b\_design}(1-(f_c/F_{cE})))))$

Load Case 2 - Dead Load + 0.6Wind

F_{b_design}:	920	psi	Final allowable bending stress		
F_{c_design}:	539	psi	Final allowable compression stress		
P_{dead}=	903	lbs	Dead load		
P_{snow}=	11047	lbs	Snow load		
A=	48	sq-in	Cross-sectional area of post	F_{cE}=	1,015 psi

f_b= 392 psi=0.6 x $f_{b\text{-}post}$ f_c= 19 psi= P_{dead}/A

CBAC2= 0.44 $=((f_c/F_{c\_design})^2)+((f_b/(F_{b\_design}(1-(f_c/F_{cE})))))$

Load Case 3 - Dead Load + 0.75(0.6Wind) + 0.75Snow

F_{b_design}:	920	psi	Final allowable bending stress		
F_{c_design}:	539	psi	Final allowable compression stress		
P_{dead}=	903	lbs	Dead load		
P_{snow}=	11047	lbs	Snow load		
A=	48	sq-in	Cross-sectional area of post	F_{cE}=	1,015 psi

f_b= 294 psi=.75 x (0.6 x $f_{b\text{-}post}$) f_c= 191 psi = $((.75 \times P_{snow}) + P_{dead})/A$

CBAC3= 0.52 $=((f_c/F_{c\_design})^2)+((f_b/(F_{b\_design}(1-(f_c/F_{cE})))))$

Max. CBAC= 52% >> Maximum post usage < 100% OK

Post Embedment Calculation

Determine the minimum posthole diameter and embedment depth for the eave wall posts per ASAE EP486.1

Since there is a slab, the post will be considered constrained at the top.
The backfill will be compacted gravel or sand full depth unless otherwise required for shear wall uplift.

Design Criteria:

S_y=	1500	psf-vertical soil bearing capacity
S=	150	psf-lateral soil bearing capacity
M_{post}=	2,171	ft-lbs - Moment at top of one posthole
V_a=	687	lbs-Lateral load on post at top of posthole
Posthole dia.=	2.5	ft.
b=	0.50	ft - maximum width of post in soil
A_{ftg}=	4.91	ft^2 - area of footing
d=	-	ft - depth of footing to be determined below

Per Sections 4.2.2.1 and 4.2.2.2, allowable lateral soil bearing capacities may be increased by 2 for isolated posts (spaced at least 3 ft. apart), and by 1.33 for wind loading

S_{LAT}= 450 psf-factored lateral soil bearing capacity

Minimum embedment depth required for lateral load, constrained at the top, gravel backfill, per Section 6.5

d_{min}= [(4 x Mpost)/(SLAT x b)]^1/3

d_{min_L}= 3.38 ft.-minimum depth requried for lateral load

Allowable vertical soil bearing pressure for gravity loads

$S_v = S_y \times A_{ftg} \times (1+(0.2 \times (d-1))$

S_y=	1500	psf-vertical soil bearing capacity
A_{ftg}=	4.91	ft^2-area of footing
d=	minimum depth for vertical bearing requirements	

Maximum vertical load on footing from gravity load $P_{footing}$= 11,950 lbs-vertical load on footing

Posthole depth for this building = 5.00 ft-minimum depth to bottom of footing

Vertical capacity for footing P_{allow}= 13,254 lbs - > Pfooting - OK

Roof and Gable Wall Shear Loads and Diaphragm Design

Roof

Roof width=	40	ft.
H_{roof}=	6.67	ft.
Total roof wind pres., 0.6 x P_r=	-2.45	psf (0.6 x P_r)
Total roof wind pressure to use=	4.80	psf - use 0 if $P_r < 0$
Total wall wind pressure=	10.30	psf (0.6 x (q_{ww} - q_{lr}))
Total wall wind pressure to use=	10.30	psf - use 0.6 x 16 = 9.6 psf minimum
Diaphragm seismic load=	2,258	lbs-(V_{roof}/2) x 0.7
Diaphragm wind load=	1,864	lbs
Diaphragm load to use=	**2,258**	**lbs - Seismic load controls**
Roof shear=	56	plf
Sheathing=	29 ga. Metal only	
Allowable shear=	113	plf > Roof shear - OK
Sheathing fastening=	#9 screws	at 9" o.c.

Gable walls

Left Gable Wall

Left gable wall shear $V_{seismic}$=	2,549	lbs-V_{LGW} x 0.7
Left gable wall shear V_{wind}=	1,864	lbs-from Diaphragm wind load above
Diaphragm load to use=	**2,549**	**lbs-Seismic controls**
Left Gable wall=	69	plf
Allowable shear=	113	plf > Wall shear - OK
Sheathing fastening=	#9 screws	at 9" o.c.

Right Gable Wall

Right gable wall shear $V_{seismic}$=	2,549	lbs-V_{RGW} x 0.7
Right gable wall shear V_{wind}=	1,864	lbs-from diaphragm wind load above
Diaphragm load to use=	**2,549**	**lbs-Seismic controls**
Right Gable wall=	64	plf
Allowable shear=	113	plf > Wall shear - OK
Sheathing fastening=	#9 screws	at 9" o.c.

Eave Wall Shear Loads and Diaphragm Design

Eave walls

Building Length=	40	ft.
Gable wall wind pressure=	9.60	psf - use 0.6 x 16 = 9.6 psf minimum
Diaphragm wind load=	1,378	lbs

Front Eave Wall

Front eave wall shear $V_{seismic}$=	917	lbs-V_{FEW} x 0.7					
Front eave wall shear V_{wind}=	1,378	lbs-from diaphragm wind load above					
Diaphragm load to use=	**1,378**	**lbs-Wind controls**					
Front eave wall=	115	plf					
Allowable shear=	335	plf > Wall shear - OK					
Sheathing fastening=	8d nails @		6 in. o.c. edges				
Block all panel edges			12 in. o.c. field				
Net shear panel uplift=	1,097	lbs	Uplift resistance=	5,046	lbs - >	1,097	lbs - OK

Backfill posthole with concrete or install uplift cleats

Rear Eave Wall

Rear eave wall shear $V_{seismic}$=	2,568	lbs-V_{REW} x 0.7	
Rear eave wall shear V_{wind}=	1,378	lbs-from diaphragm wind load above	
Diaphragm load to use=	**2,568**	**lbs-Seismic controls**	
Rear eave wall=	64		
Allowable shear=	113	plf > Wall shear - OK	
Sheathing fastening=	#9 screws		at 9" o.c.

Purlin & Girt Calculations

Purlin Calculation

Roof Pitch:	4	/12
Roof Angle:	18.4	°
Greatest purlin span:	162	in
Purlin S_x:	13.14	in^3
Live + dead load:	40	psf
Max. o.c. spacing:	18	in. o.c.
M:	15,444	in-lbf
f_b:	1,175	psi
F_b allowable:	1,428	psi-per NDS Section 4 and Design Values for Wood Construction
Purlin usage:	**82%**	**OK**

End reactions:

Snow load:	402	lbs	If joist hanging, use LU26 joist hanger w/ 10d nails or JB26 top-flange joist hanger w/ 10d nails
uplift:	231	lbs	(3) nails each side of flat purlin block

Girt Calculation

Greatest Bay Spacing:	14	ft.
O.C. Spacing:	24	in
Girt S_x:	3.59	in^3
Total wind pressure:	12.39	psf
w:	2.07	pli
Girt Span:	162	in
M:	6,777	lbf-in
f_b:	1,888	psi
F_b allowable:	2,476	psi-per NDS Section 4 and Design Values for Wood Construction
Girt usage:	**76%**	**OK**

Bearing Block Bolts In Double Shear

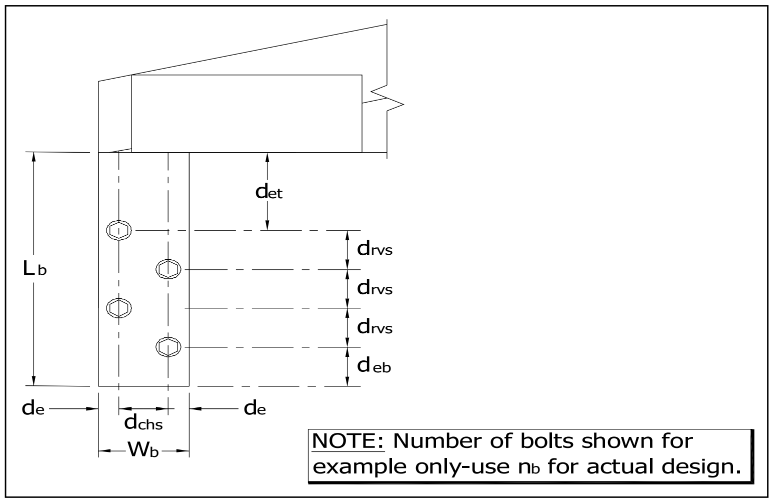
Calculate required number of bolts, and the correct bolts spacings and bearing block size for the intermediate truss bearing posts. Posts are assumed to be #2 HF; bearing blocks assumed to be #2 HF.

Total load from both trusses=	11,950	lbs		
Bolt size=	3/4	"Ø (**NOTE:** Use 5/8"Ø, 3/4"Ø 7/8"Ø or 1"Ø only)		
Main member, ℓ_m=	6	in-post width		
Post depth=	8	in-post depth		
Side member(s), ℓ_s=	3	in- total for 2 side members		
No. of fastener columns=	2			
No. of bolts required, n_b=	4.75	>> n_b=	5	Bolt(s) in block
Truss bearing block=	2x6	Verify with truss engineering		
Minimum block length, L_b=	20.25	in (no less than 12")		
Minimum block width, W_b=	5.5	in (<[(2 x d_e) + d_{chs}], < truss bearing block)		

Dimension Summary

d_{et}=	**5 1/4**	**in (min)**
d_{eb}=	**3**	**in (min)**
d_{rvs}=	**3**	**in (min)**
d_{chs}=	**1 1/8**	**in (min)**
d_e=	**1 1/8**	**in (min)**

Number of bolts, n_b=	**5**	**(min)**
Block length, L_b=	**20 1/4**	**in (min)**
Block width, W_b=	**5 1/2**	**in (min)**



<u>Gable End Rafters</u>

Calculate rafter size for the gable ends using single 2x rafters

Rafter Loads

Rafter span=	162	in
Single or double rafters=	Double	
Tributary width=	6.5	ft
Dead load=	3	psf
Live load=	37	psf
w=	21.5	pli

Rafter

Rafter size= 2x12

Rafter grade= #1 D-F

F_b=	1,150	psi-allowable bending
F_v=	207	psi-allowable shear

Rafter adequacy

f_b=	**1115**	**psi < allowable bending OK**
f_v=	**77**	**psi < allowable shear OK**