BERKEY
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Project Description

Building Width = 36 ft

Building Length = 36 ft

Eave Ht_1 = 16 ft

Pole Building Design for

Todd Mills

Criteria

2015 IBC

2000 Post Frame Design Manual

2010 NDS

ASCE 7 - 10

30 psf Snow Load

110 mph Wind, Exp. B

Seismic Zone : D2

Concrete :

f_c' = 2500

Soil :

S_v = 1500 psf, Vertical Soil Pressure

S = 100 psf/ft, Lateral Soil Pressure

CHANGES
MUST Be Approved Prior
To Performing Work

Subject To Field Inspection

Reviewed for code compliance
with IRC 2015
Kitsap County Building Department
jbowling@co.kitsap.wa.us
06/03/2020

CLIFTON WADE BERKEY
STATE OF WASHINGTON
37708
REGISTERED
PROFESSIONAL ENGINEER
5/12/20

EXPIRES 08/18/21

Permit Number: 20-01902

2

Snow Loads

P_g = 30 psf

I_s = 1.0 *TABLE 1.5-2, ASCE 7-10*

C_e = 1.2 *TABLE 7-2, ASCE 7-10*

C_t = 1.2 *TABLE 7-3, ASCE 7-10*

Roof Pitch *4 /12* θ = 18.43 degrees

0.322 radians

Exp = B

0.316 sin θ

0.949 cos θ

$P_f = P_g(0.7)(I_s)(C_e)(C_t)$ 0.900 $\cos^2\theta$

= 30.2 psf

SLOPE FACTOR

C_S = 0.94 *FIGURE 7-2, ASCE 7-10*

$P_s = (P_f)(C_s)$

= 28.36 psf

P_{min} = 20 psf

28 psf Controls

Wind Loads

Basic Wind Speed = 110 mph

Exposure = B

I = 1.00

λ = 1.00

h = 19.00

a = 3.60

2a = 7.20

Horizontal Pressures

FIGURE 28.6-1, ASCE 7-10

A = 24.1

B = 8

C = 16

D = 4.6

Vertical Pressures

E = 23.1

F = 15.1

G = 16

H = 11.5

Overhangs

E_{OH} = 32.3

G_{OH} = 25.3

Lateral Control - Wind

$L =$ 36 ft

Overhang $= x =$ 1.5 ft

Overhang $= y =$ 1.5 ft

span $= s =$ 36 ft

pitch $= m =$ 4 /12

$Ht =$ 16 ft

$A_{eave\ wall} = (Ht-0.5)(L)$

$=$ 558 sf

$A_{roof} = (L + 2x)(m(s/2)) =$ 234 sf

$ht_{roof} = m(s/2) =$ 6.00 ft

Wind Forces

$V_A = (24.1)(Ht)(2a) =$ 2776 lbs

$V_B = (8)(m(s/2)(2a+x) =$ 418 lbs

$V_C = (16)(Ht)(L-2a) =$ 7373 lbs

$V_D = (4.6)m(s/2)(L+x-2a) =$ 836 lbs

$\sum V_{WIND} =$ 11403 lbs

Seismic

$DL_{roof} =$ 5 psf

$DL_{walls} =$ 5 psf

Area of Roof:

$A_R = (s+2y)(L+2x) =$ 1521 sf

Area of Walls:

$A_w = 2[(L)(Ht) + ((s)(Ht)+(m(s^2/4))] =$ 2520 sf

$Wt_{roof} = A_R(DL_{roof}) =$ 7605 lbs

$Wt_{walls} = A_W(DL_{walls}) =$ 12600 lbs

$Wt = Wt_{roof} + Wt_{walls} =$ 20205 lbs

Seismic Parameters:

$I_e =$	1.0	*Table 1.5-2, ASCE 7-10*
$R =$	2.5	*Table 12.2-1, ASCE 7-10*
$S_s =$	1.35	*USGS Seismic Hazard Curves*
$F_a =$	1	*Table 11.4-1, ASCE 7-10*
$S_{MS} = (S_s)(F_a) =$	1.4	*Eq 11.4-1, ASCE 7-10*
$S_{DS} = (2/3)S_{ms} =$	0.9	*Eq 11.4-3, ASCE 7-10*
$Cs = (S_{DS})(I_e)/R =$	0.4	*Eq 12.8-2, ASCE 7-10*

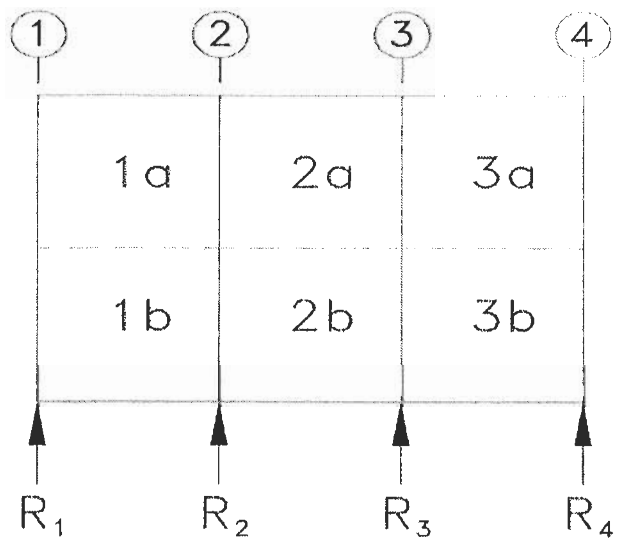
$V_{seismic} = C_sW =$ 7274 lbs *Eq 12.8-1, ASCE 7-10*

7.27 Kip < 11.40 Kip

Wind Controls ←

Lateral Model

3 Bays



Bay	1	2	3
Width	12 ft	12 ft	12 ft

Diaphragm Stiffness

Assembly # 9

29 ga. Steel (11) Overlap Purlins w/o st

Shear Stiffness c = 4700 lbs/in

Shear Modulus G = 4700 lbs/in

Allowable Shear v_a = 160 plf

$C_p = (G(b_h))/(s(\cos\theta)) =$ 8051 lbs/in

s = 12.00 ft b_h = 19.50 ft

Stiffness Properties

Frame 1: K_p = 1214 lbs/in

Frame 2: K_p = 290 lbs/in

Frame 3: K_p = 290 lbs/in

Frame 4: K_p = 10219 lbs/in

Frame Loads

Frame 1: R_1 = 1228 lbs

Frame 2: R_1 = 1566 lbs

Frame 3: R_1 = 1483 lbs

Frame 4: R_1 = 742 lbs

Frame Stiffness

Frame 2-3 — Cantilevered Post

Post Size 6x 8 N = 2

E = 1100 ksi I = 256 in^4

H_p = 180 in

$K_p = N(3EI/Hp^3)$ = 290 lbs/in

Frame 4 — 29 ga. Steel w/ Stitch

C = 4400 lbs/in G = 4400 lbs/in

V_a = 160 plf w = 36 ft

Ht = 16 ft

$K_p = (Gw)/Ht$ = 10219 lbs/in

Frame 1 — Cantilevered Post

Post Size 6x 6 N = 4

E = 1100 ksi I = 108 in^4

H_p = 120 in

$K_p = N(3EI/Hp^3)$ = 825 lbs/in

Frame 1 — Cantilevered Post

Post Size 6x 6 N = 2

E = 1100 ksi I = 108 in^4

H_p = 144 in

$K_p = N(3EI/Hp^3)$ = 239 lbs/in

Frame 1 — Cantilevered Post

Post Size 6x 6 N = 2

E = 1100 ksi I = 108 in^4

H_p = 168 in

$K_p = N(3EI/Hp^3)$ = 150 lbs/in

DAFI Input

Number of Bays 3

Roof Stiffness

Bay 1 14491
Bay 2 14491
Bay 3 14491

Frame	Stiffness	Loads
1	1214 lbs/in	1228 lbs
2	290 lbs/in	1566 lbs
3	290 lbs/in	1483 lbs
4	10219 lbs/in	742 lbs

7

16759

FRAME NUMBER	FRAME STIFFNESS	APPLIED LOAD	HORIZONTAL DISPLACEMENT	LOAD RESISTED BY FRAME	FRACTION OF APPLIED LOAD
1	1214.00	1228.0	.7212328	875.6	.7130
2	290.00	1566.0	.6969126	202.1	.1291
3	290.00	1483.0	.5784723	167.8	.1131
4	10219.00	742.0	.3692692	3773.6	5.0857

DIAPHRAGM NUMBER	DIAPHRAGM STIFFNESS	SHEAR DISPLACEMENT	SHEAR LOAD
1	14491.00	.0243201	352.4
2	14491.00	.1184403	1716.3
3	14491.00	.2092031	3031.6

Diaphragm Shear

$V_p = (c_{h,i}/C_h)(V_h)/(\cos\theta_i)$

$= 1598$ lbs

$V_h = 3031.6$ lbs

$l = b_h/\cos\theta =$ 20.55 ft

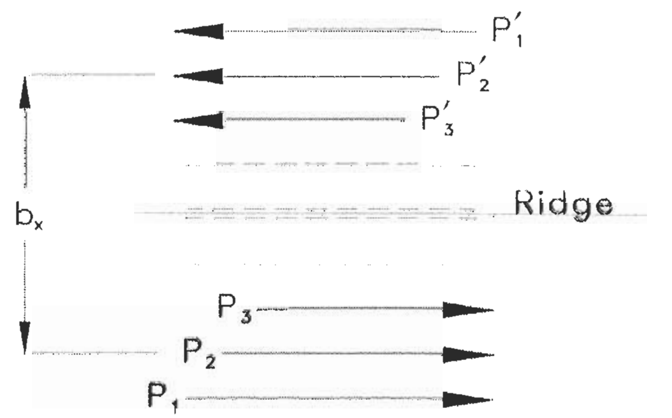
$\upsilon_p = V_p/l =$ 77.7 plf

Diaphragm Chords

$M_d = (V_h)(L)/4 =$ 27284 ft-lbs

Chord Force

$\sum P_e = M_d/b_x$



Use only the outer 3 purlins

Purlin Spacing = z = 24.0 in   W = 36 ft

$b_x = W - 2(z/12) =$ 32.00 ft

$\sum P_e = M_d/b_x =$ 853 lbs

a = 1.5 in   b = 5.5 in $A_{purlin} = ab =$ 8.25 in^2

$P_1 = (W/b_x)(\sum P_e)/3 =$ 320 lbs

$(f_T)_{max} = P_1/A_{purlin} =$ 38.8 psi

Post Design

Out of Plane Forces

H_p = 180 in   s = 12.00 ft

w = 192 plf

$M_{actual} = w(H_p)^2/8$ = 5400 ft-lbs

Post Size 6x 8

E = 1100 ksi   I = 256 in^4

S = 64 in^3 F_b = 675 psi

C_D = 1.6 F_b' = 1080 psi

M_{max} = 5760 ft-lbs

5760 ft-lbs > 5400 ft-lbs   **OK**

End-to-End Shearwalls

Lateral wind force: Reaction for fixed-pinned posts on gable ends = ΣR

$\Sigma R_1 = 3/8(V_{w(EE)})$ =

A_{EE} = 684 sf A_A = 137 sf

F_{EE} = 4847

Distribute Force to 2 Walls

Total length of controlling shear element:

L_{EE} = 25 ft   Trib = 18 ft

Span = s = 36 ft

$V_{EE} = (Trib/s)F_{EE}$ = 2424 lbs

$\upsilon_{EE} = V_{EE}/L_{EE}$ = 96.9 plf

Shearwall Type = 29 ga. Steel (11)

Allowable Shear = 110 plf

96.9 plf < 110.0 plf   OK

SHEAR WALL CHOICES

29 ga w/o stitch screws	110 plf
29 ga with stitch screws	160 plf
7/16" OSB with staples	395 plf

Constrained Post Embedded in Earth

Base Soil Lateral Bearing = S = 100 pcf

Embedded Element Width = b = 2.00 ft

Required spacing for increase in PFDM 8.3.11
6b = 12.0 ft

USE INCREASE   w = 192 plf

S' = 266 pcf   L = 15.00 ft

M_a = 5400 ft-lbs

Actual d = $(4M_a/S'b)^{1/3}$

Actual d = 3.44 ft

Use d = 4.00 ft

Uplift

$Trib_1$ = 12.00 ft

$Trib_2$ = 19.50 ft

Area = A = 234 sf

Uplift Pressure = G = 16.0 psf

Uplift = P_{up} = 3744 lbs

Soil Cone

d_T = 3.50 ft φ = 35 °   0.61 rad

r = 1.00 ft A_p = 0.33 sf

$V_s = ((\pi(d_T\tan\varphi + r)^2(d_T + r/\tan\varphi))/3) - (\pi r^3/3\tan\varphi) - d_T A_p$

= 59 cf

γ_{soil} = 110 pcf

HD = $V_s(\gamma_{soil})$ = 6.47 kip

3.74 kip < 6.47 kip   OK

Frame 4 Shearwall

Total length of controlling shear element:

L = 36 ft

V = 3773.6

v = V/L = 104.8 plf

Shearwall Type = 29 ga. Steel w/ Stitch

Allowable Shear = 160 plf

104.8 plf < 160.0 plf   OK

JOIST & RAFTER CALCULATOR — PURLINS

BEAM PARAMETERS:

Horiz. Span:	12	ft
Slope (V:H):	0	:12
True Span =	10.25	ft
Beam Type:	#2 DF-L	
S4S lumber?	YES	
No. of Pieces:	**1**	
Spacing:	**24**	in
Width:	**2**	in
Depth:	**6**	in.

Types
#2 DF-L
#2 HF
LVL
PARALLAM

STRESS CALCULATIONS:

Moment @ midspan =	*824*	*ft-lbs*
Moment @ Point #1 =	*0*	*ft-lbs*
User Calc'd Moment =	*0*	*ft-lbs*
Shear @ 0' end =	*306*	*lbs*
Shear @ 10.25' end =	*306*	*lbs*
User Calc'd Shear =	*0*	*lbs*
Reaction @ 0' end =	*321*	*lbs*
Reaction @ 10.25' end	*321*	*lbs*

LOADS:

	VERT. LOAD			PERP. LOAD	
Uniform Dead:	3	psf	=	3	psf
Uniform Live:	28	psf	=	28	psf
Point #1:	0	lbs	@	0	ft

fb =	***1307***		**TEN. BENDING OK**
fv =	***58***		**SHEAR OK**
LL δ allow.=	*0.68*	in	
TL δ allow.=	*1.03*	in	
Live Load δ =	***0.49***		**LIVE DEFLECTION OK**
Total Load δ =	***0.54***		**TOTAL DEFLECTION OK**

Requires 1.50 in. bearing surface

BASE DESIGN VALUES:

Allow. Live Load δ:	L/180			
Allow. Total Load δ:	L/120			
Fb =	900 psi		*F'b =*	*1547 psi*
Fv =	180 psi		*I =*	*18 in^4*
Fc_{perp} =	625 psi			
E =	1,600,000 psi			

ADJUSTMENT FACTORS:

Cd =	*1.15*
Cf =	*1.30*
Cr =	*1.15*

SPECIFICATION:

2 x 6 #2 DF-L @ 24 in. o/c

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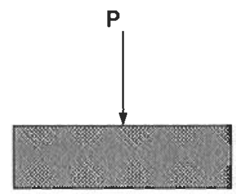
POST FOOTING DESIGN



MATERIAL PARAMETERS:

Base Allow. Soil Press.=	1500	psf
Adjust. Allow. Soil Press. =	2700	
Concrete Strength: f_c' =	2500	psi
Post Width =	6	in
Post Depth =	8	in

FOOTING PARAMETERS:

Diameter = b =	**2.5**	ft
Thickness = t =	6	in
Column Width = h_1 =	6	in
Depth to Base =	48	in
Column Depth = h_2 =	8	in

AXIAL LOADS

Dead Load = DL =	1080	lbs
Live Load = LL =	6126	lbs

CALCULATIONS:

Actual Soil Pressure = q = $(DL+LL)/(\pi b^2/4)$
= *1469* *psf* **OK**

Punching Shear (Vu 2) = $q[(\pi(b/2)^2) - ((h_1+h_2)/12)^2]$
= *7711* *lbs*

Allow. Shear $(\phi Vc) = (.85)(4)SQRT(F_c')(4)(h_1+h_2)(h_2)/2$
= *24480* *lbs* **OK**

Actual Conc. Bearing = (1.4DL +1.7LL)/(h1·h2)
= *331* *psi*

Allow. Conc. Bearing = (0.85)(0.7fc'SQRT2)
= *2975* *psi* **OK**

CONSTRUCTION SPECIFICATIONS:

2.5 ft. dia. x 6 in. CONCRETE FOOTING
2 ft. dia. x 48 in. DEEP HOLE (BELL OUT AS REQ'D)

2500 psi CONCRETE

BEAM-COLUMN

AXIAL LOADS & STRESSES:

P1= 7206 LBS
P2= 0 LBS
P3= 5404 LBS
D + S, fc= 150.12 psi — **Load Case (16-9)**
D + W, fc= 0.00 psi — **Load Case (16-11)**
D+0.75(S+W), fc= 112.59 psi — **Load Case (16-10)**

BENDING LOADS & STRESSES:

Max Moment(M1)= 0 lb-ft
Max Moment(M2)= 5400 lb-ft
Max Moment(M3)= 4050 lb-ft
D + S, fb= 0.00 psi
D + W, fb= 1012.50 psi
D+0.75(S+W), fb= 759.38 psi

TRUSS MEMBER PARAMETERS:

Length: 15.00 ft
Ke: 0.8
Effect. Length = 12.0

S= 64.0 in^3
I= 144.0 in^4
A= 48.0 in^2

SPECS:

GRADE: #2 HF
Width: 8 in.
Depth: 6 in.
d = 8 in.
Fc = 575 psi
Fb = 675 psi
E = 1100000 psi

CALCULATIONS

Le/d =	*18.0*
Fc =*	*920*
F_{cE} =	*1019*
K_{cE} =	*0.30*
c' =	*0.8*
C_P =	*0.725*
F_b' =	1080

ADJUSTMENT FACTORS:

wind C_D = 1.60
C_F = 1.00
snow C_D = 1.15

LOAD CASES:		INTERACTION EQUATION $[f_c / F'_c]^2 + [f_{b1} / (F_{b1}'(1-f_c / F_{cE1})] \le 1.0$			STATUS
CASES (16-7&12)		By Inspection			**OK**
CASE (16-9)	D+S	0.098	0.000	0.098	**OK**
CASE (16-11)	D+W	0.000	0.938	0.938	**OK**
CASE (16-10)	D+0.75(S+W)	0.028	0.791	0.819	**OK**

SPECIFICATION:

USE 8 in x 6 in #2 HF

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