10/2/2024

STRUCTURAL CALCULATIONS

Bull Run Filtration

(3)12x60

36800 SE Lusted Rd

Boring, OR 97009

WILLSCOT™

WILLIAMS SCOTSMAN INC.
4646 East Van Buren Street,
Suite 400, Phoenix, AZ 85008

Aman Dhakal

Thierry R. LeBoulch P.E.

WILLSCOT WILLIAMS SCOTSMAN INC. 4646 East Van Buren Street, Suite 400, Phoenix, AZ 85008	Project Bull Run Filtration				Job Ref.	
	Section (3) 12 x 60				Sheet no./rev. 1	
	Calc. by AD	Date 10/2/2024	Chk'd by	Date	App'd by	Date

Contents

LOAD DESIGN 2
WIND LOADING (ASCE7) 3
WIND LOADING COMPONENT AND CLADDING (ASCE7) 8
SEISMIC FORCES (ASCE7) 11
CHECK WIND AND SEISMIC 13
ANCHOR 13
OVERTURNING LOAD 15
GRAVITY LOAD 16
MATELINE WOOD BEAM REACTIONS 16
CONT EXTERIOR BEAM REACTION 20
CONT INTERIOR BEAM REACTION 25
PIER LOAD 30
ABS BEARING PAD 33

Thierry Leboulch

Digitally signed by Thierry Leboulch
Date: 2024.10.07 08:34:14 -05'00'

10/3/2024

REGISTERED PROFESSIONAL ENGINEER
87081PE
OREGON
SEPTEMBER 11, 2012
THIERRY R. LEBOULCH

EXPIRES 12/31/2024

WILLSCOT WILLIAMS SCOTSMAN INC. 4646 East Van Buren Street, Suite 400, Phoenix, AZ 85008	Project Bull Run Filtration				Job Ref.	
	Section (3) 12 x 60				Sheet no./rev. 2	
	Calc. by AD	Date 10/2/2024	Chk'd by	Date	App'd by	Date

LOAD DESIGN

Length of building; bl = 60 ft

Width of building; bw = 35.4 ft

Height of building; h = 13.5 ft

Wind Analysis; If(h>min(bl,bw),"DIRECTIONAL PROCEDURE","ENVELOPE PROCEDURE") = **"ENVELOPE PROCEDURE"**

Snow Load + Rain-on-Snow Surcharge

Pf = 25 psf

Gravity

Roof Dead Load; R_dl = 10 psf;

Roof Live Load ; R_ll = 20 psf

Floor Dead Load; F_dl = 10 psf;

Floor Live Load; F_ll = 50 psf;

Pier Spacing; Sp = 8.57 ft;

Unit width; Wunit = 11.75 ft;

Roof trib at int. column; Ltrib_int = 20 ft;

Roof trib at ext. column; Ltrib_ext = 10 ft;

Wall Dead Load; Wdl = 60 plf;

Wind

Wind speed (ultimate); Wind= 98 mph

Exposure; Wind_exp = "C"

Risk Category; Risk_category= "II"

Seismic

Risk Category; Risk_category= "II"

Soil Site Class; Soil_site_class = "D"

SS= 0.754; S1= 0.33

Dead_load= (bl × bw) × (R_dl+F_dl) + Wdl × 2 × (bw+bl) = **53.928** kips

Allowable soil Bearing Capacity; Sall = 2500 psf;

WILLSCOT WILLIAMS SCOTSMAN INC. 4646 East Van Buren Street, Suite 400, Phoenix, AZ 85008	Project Bull Run Filtration				Job Ref.	
	Section (3) 12 x 60				Sheet no./rev. 3	
	Calc. by AD	Date 10/2/2024	Chk'd by	Date	App'd by	Date

WIND LOADING (ASCE7)

V=Wind

_wld.Exposure = Wind_exp

_wld.Risk = Risk_category

b = bl

d = bw

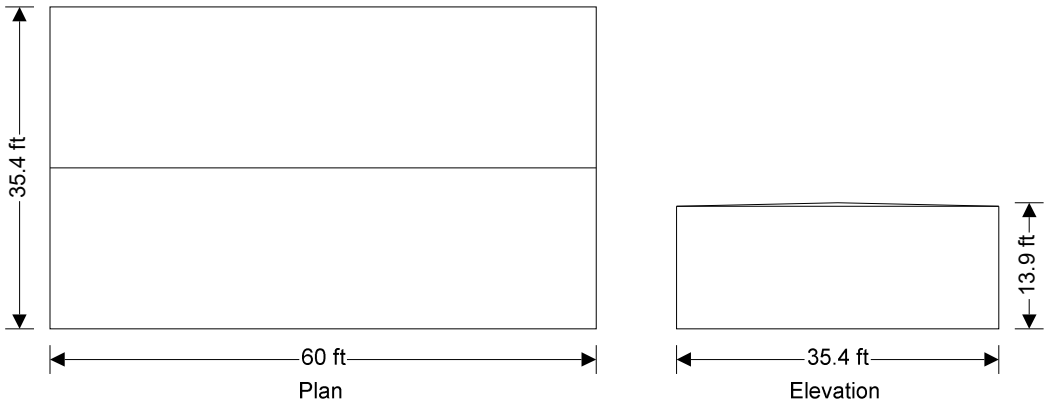
h = h

WIND LOADING

In accordance with ASCE7-16

Using the envelope design method

Tedds calculation version 2.1.16



Building data

Type of roof;	Gable
Length of building;	b = **60.00** ft
Width of building;	d = **35.40** ft
Height to eaves;	H = **13.50** ft
Pitch of roof;	α_0 = **1.2** deg
Mean height;	h = **13.50** ft
End zone width;	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **3.54** ft
Plan length of Zone 2/2E when GC_{pf} negative;	L_{Z2} = min(0.5 × d, 2.5 × H) = **17.70** ft
Plan length of Zone 3/3E encroachment on zone 2;	L_{Z3} = max(0 ft,0.5 × d - L_{Z2}) = **0.00** ft

General wind load requirements

Basic wind speed;	V = **98.0** mph
Risk category;	II
Velocity pressure exponent coef (Table 26.6-1);	K_d = **0.85**
Ground elevation above sea level;	z_{gl} = **497** ft
Ground elevation factor;	K_e = exp(-0.0000362 × z_{gl}/1ft) = **0.98**
Exposure category (cl 26.7.3);	C
Enclosure classification (cl.26.12);	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1);	GC_{pi_p} = **0.18**

Internal pressure coef –ve (Table 26.13-1); GC_{pi_n} = **-0.18**

Topography

Topography factor not significant; K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1); K_z = **0.85**

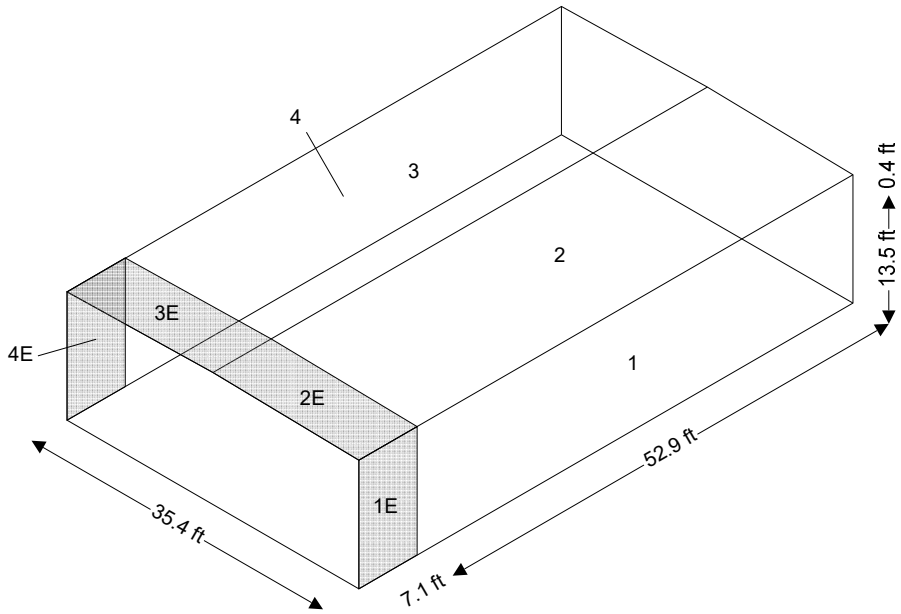
Velocity pressure; $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1psf/mph^2$ = **17.4** psf

Design wind pressures

Design wind pressure equation; $p = q_h \times ((GC_{pf}) - (GC_{pi}))$

Design wind pressures – Loadcase A

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	3.8	10.1	714	2.7	7.2
2	-0.69	-15.2	-8.9	937	-14.2	-8.3
3	-0.37	-9.6	-3.3	937	-9.0	-3.1
4	-0.29	-8.2	-1.9	714	-5.9	-1.4
1E	0.61	7.5	13.8	96	0.7	1.3
2E	-1.07	-21.8	-15.5	125	-2.7	-1.9
3E	-0.53	-12.4	-6.1	125	-1.6	-0.8
4E	-0.43	-10.6	-4.4	96	-1.0	-0.4

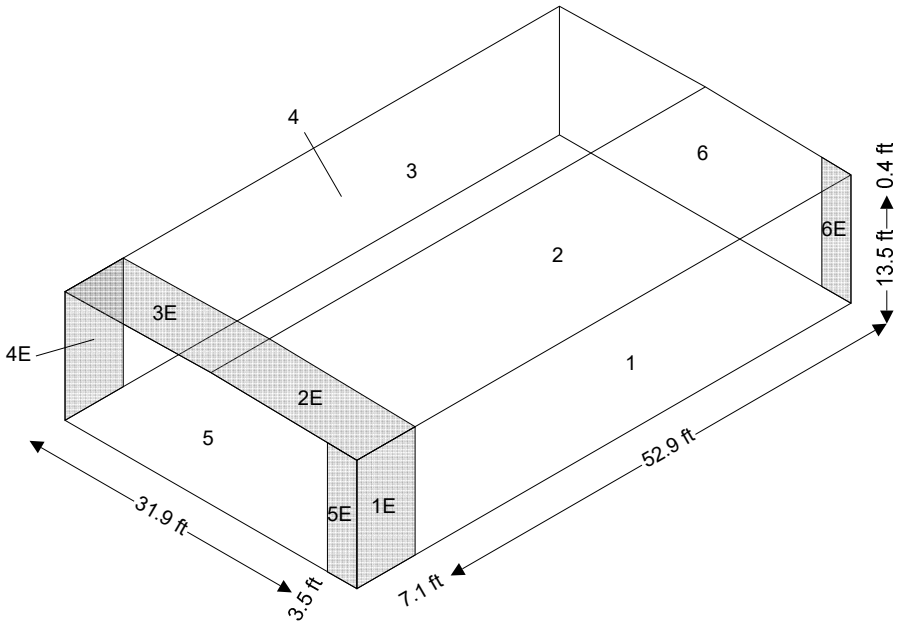


Loadcase A

Design wind pressures – Loadcase B

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-11.0	-4.7	714	-7.9	-3.4

2	-0.69	-15.2	-8.9	937	-14.2	-8.3
3	-0.37	-9.6	-3.3	937	-9.0	-3.1
4	-0.45	-11.0	-4.7	714	-7.9	-3.4
5	0.40	3.8	10.1	437	1.7	4.4
6	-0.29	-8.2	-1.9	437	-3.6	-0.8
1E	-0.48	-11.5	-5.2	96	-1.1	-0.5
2E	-1.07	-21.8	-15.5	125	-2.7	-1.9
3E	-0.53	-12.4	-6.1	125	-1.6	-0.8
4E	-0.48	-11.5	-5.2	96	-1.1	-0.5
5E	0.61	7.5	13.8	48	0.4	0.7
6E	-0.43	-10.6	-4.4	48	-0.5	-0.2

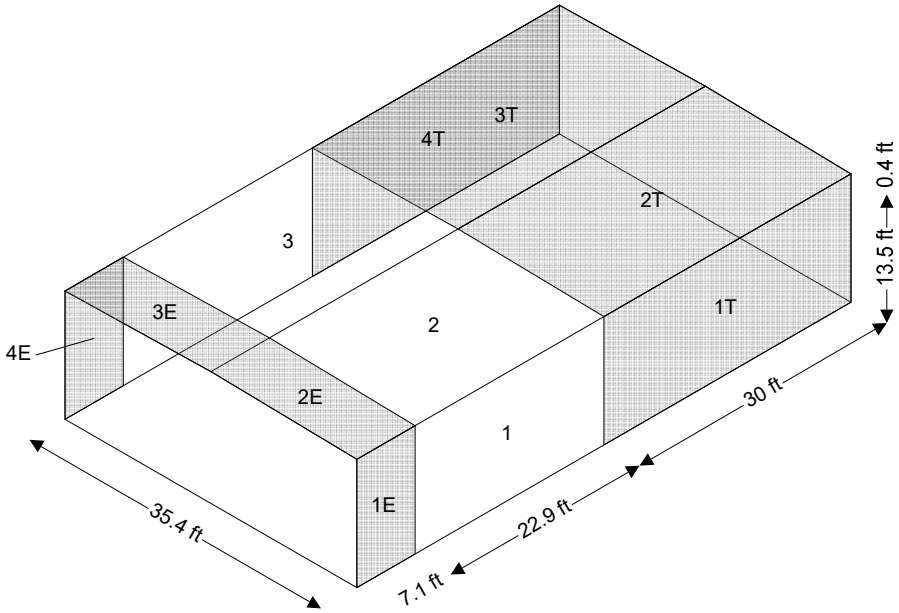


Loadcase B

Design wind pressures – Loadcase AT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	3.8	10.1	309	1.2	3.1
2	-0.69	-15.2	-8.9	406	-6.2	-3.6
3	-0.37	-9.6	-3.3	406	-3.9	-1.3
4	-0.29	-8.2	-1.9	309	-2.5	-0.6
1E	0.61	7.5	13.8	96	0.7	1.3
2E	-1.07	-21.8	-15.5	125	-2.7	-1.9
3E	-0.53	-12.4	-6.1	125	-1.6	-0.8

4E	-0.43	-10.6	-4.4	96	-1.0	-0.4
1T	-	1.0	2.5	405	0.4	1.0
2T	-	-3.8	-2.2	531	-2.0	-1.2
3T	-	-2.4	-0.8	531	-1.3	-0.4
4T	-	-2.1	-0.5	405	-0.8	-0.2



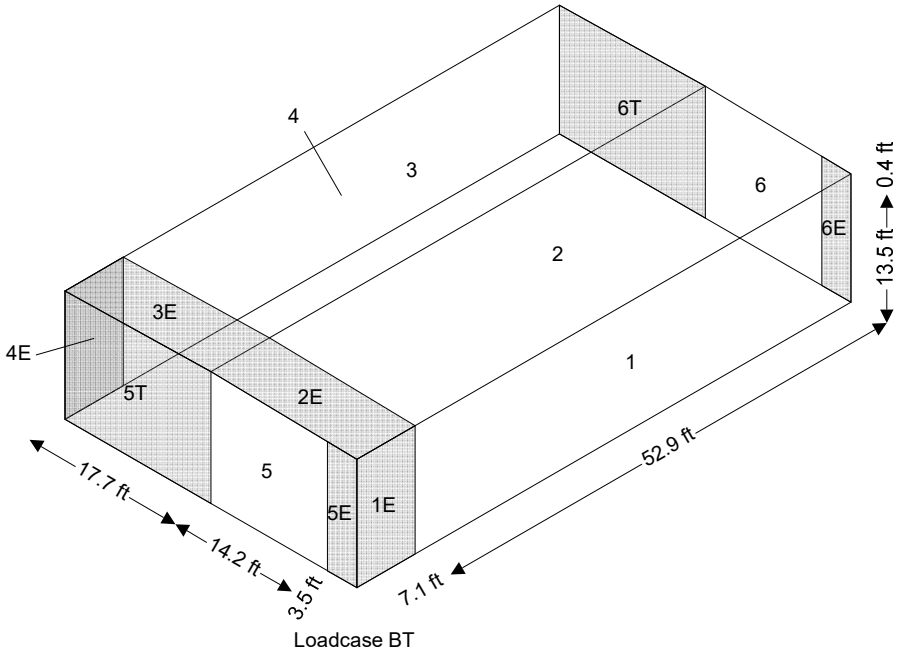
Loadcase AT

Design wind pressures – Loadcase BT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-11.0	-4.7	714	-7.9	-3.4
2	-0.69	-15.2	-8.9	937	-14.2	-8.3
3	-0.37	-9.6	-3.3	937	-9.0	-3.1
4	-0.45	-11.0	-4.7	714	-7.9	-3.4
5	0.40	3.8	10.1	218	0.8	2.2
6	-0.29	-8.2	-1.9	218	-1.8	-0.4
1E	-0.48	-11.5	-5.2	96	-1.1	-0.5
2E	-1.07	-21.8	-15.5	125	-2.7	-1.9
3E	-0.53	-12.4	-6.1	125	-1.6	-0.8
4E	-0.48	-11.5	-5.2	96	-1.1	-0.5
5E	0.61	7.5	13.8	24	0.2	0.3
6E	-0.43	-10.6	-4.4	48	-0.5	-0.2
5T	-	1.0	2.5	242	0.2	0.6

WILLSCOT WILLIAMS SCOTSMAN INC. 4646 East Van Buren Street, Suite 400, Phoenix, AZ 85008	Project Bull Run Filtration				Job Ref.	
	Section (3) 12 x 60				Sheet no./rev. 7	
	Calc. by AD	Date 10/2/2024	Chk'd by	Date	App'd by	Date

6T	-	-2.1	-0.5	242	-0.5	-0.1



Loadcase BT

WIND LOADING COMPONENT AND CLADDING (ASCE7)

V=Wind

_wld.Exposure = Wind_exp

_wld.Risk = Risk_category

b = bl

d = bw

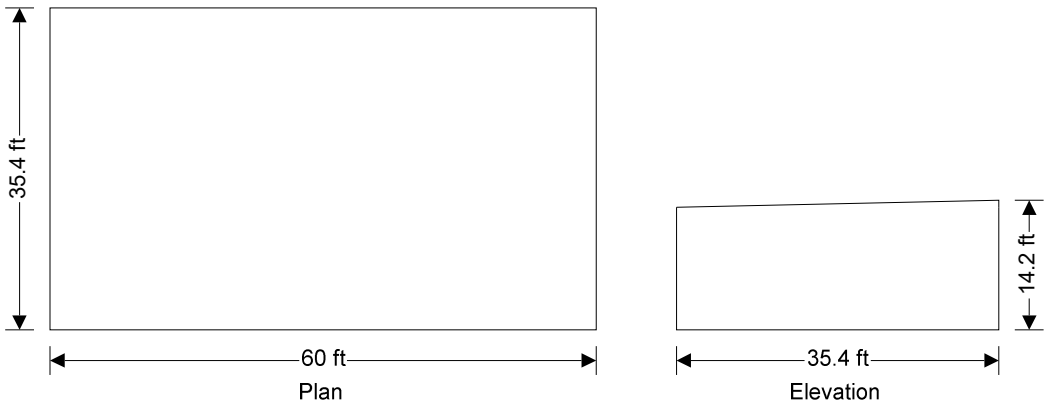
h = h;

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.16



Building data

Type of roof;	Monoslope
Length of building;	b = **60.00** ft
Width of building;	d = **35.40** ft
Height to eaves;	H = **13.50** ft
Pitch of roof;	α_0 = **1.2** deg
Mean height;	h = **13.50** ft
End zone width;	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **3.54** ft

General wind load requirements

Basic wind speed;	V = **98.0** mph
Risk category;	II
Velocity pressure exponent coef (Table 26.6-1);	K_d = **0.85**
Ground elevation above sea level;	z_{gl} = **497** ft
Ground elevation factor;	$K_e = \exp(-0.0000362 \times z_{gl}/1ft)$ = **0.98**
Exposure category (cl 26.7.3);	C
Enclosure classification (cl.26.12);	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1);	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1);	GC_{pi_n} = **-0.18**

Gust effect factor; G_f = **0.85**

Topography

Topography factor not significant; K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1); K_z = **0.85**

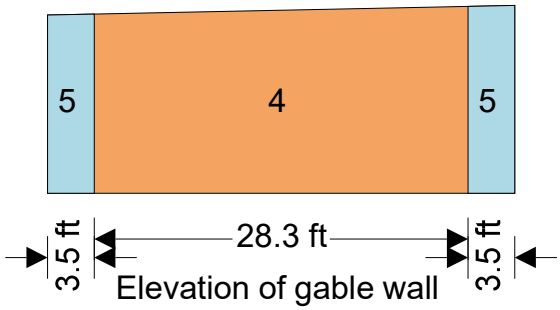
Velocity pressure; $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **17.4** psf

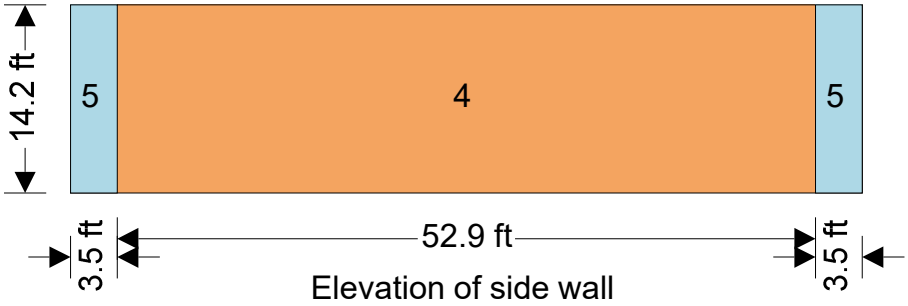
Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); q_i = **17.45** psf

Equations used in tables

Net pressure; $p = q_h \times (GC_p - GC_{pi})$





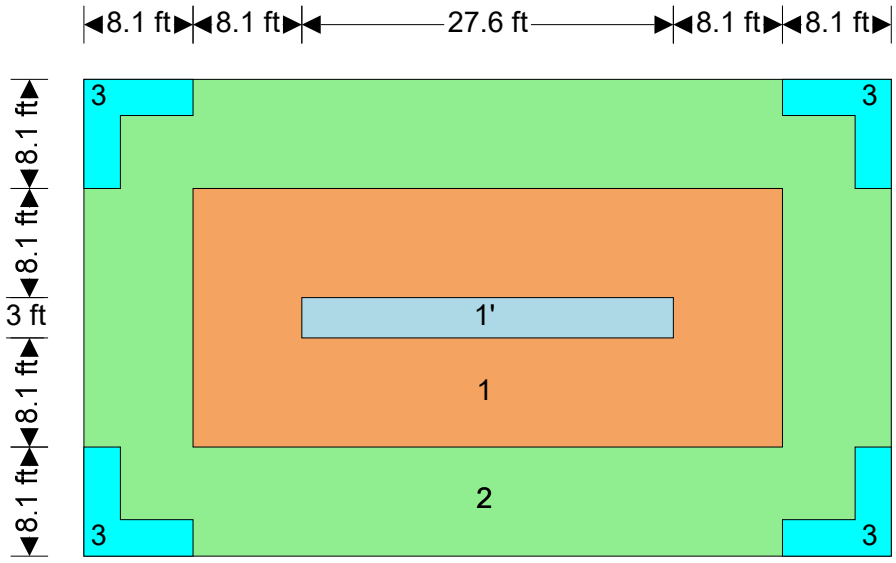
Components and cladding pressures - Roof (Figure 30.3-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10 sf	1	-	-	10.0	0.30	-1.70	8.4 #	-32.8
25 sf	1	-	-	25.0	0.26	-1.54	7.7 #	-29.9
50 sf	1	-	-	50.0	0.23	-1.41	7.2 #	-27.8
>100 sf	1	-	-	100.0	0.20	-1.29	6.6 #	-25.6
<10 sf	2	-	-	10.0	0.30	-2.30	8.4 #	-43.3
25 sf	2	-	-	25.0	0.26	-2.09	7.7 #	-39.6
50 sf	2	-	-	50.0	0.23	-1.93	7.2 #	-36.8
>100 sf	2	-	-	100.0	0.20	-1.77	6.6 #	-34.0

WILLSCOT WILLIAMS SCOTSMAN INC. 4646 East Van Buren Street, Suite 400, Phoenix, AZ 85008	Project Bull Run Filtration				Job Ref.	
	Section (3) 12 x 60				Sheet no./rev. 10	
	Calc. by AD	Date 10/2/2024	Chk'd by	Date	App'd by	Date

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10 sf	3	-	-	10.0	0.30	-3.20	8.4 #	-59.0
25 sf	3	-	-	25.0	0.26	-2.78	7.7 #	-51.6
50 sf	3	-	-	50.0	0.23	-2.46	7.2 #	-46.1
>100 sf	3	-	-	100.0	0.20	-2.14	6.6 #	-40.5

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction



Plan on roof

SEISMIC FORCES (ASCE7)

Risk_category = Risk_category

Site_Class = Soil_site_class

S_S = SS

S_1= S1

h_n= h

w_{1} = Dead_load

w_{1} = Dead_load

W = Dead_load

SEISMIC FORCES

In accordance with ASCE 7-16

Tedds calculation version 3.1.05

Site parameters

Site class;	D, Soil properties not known
Mapped acceleration parameters (Section 11.4.2)	
at short period;	S_S = **0.754**
at 1 sec period;	S_1 = **0.33**
Site coefficient;	
at short period (Table 11.4-1);	F_a = **1.200**
at 1 sec period (Table 11.4-2);	F_v = **1.970**

Spectral response acceleration parameters

at short period (Eq. 11.4-1);	$S_{MS} = F_a \times S_S$ = **0.905**
at 1 sec period (Eq. 11.4-2);	$S_{M1} = F_v \times S_1$ = **0.650**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3);	$S_{DS} = 2 / 3 \times S_{MS}$ = **0.603**
at 1 sec period (Eq. 11.4-4);	$S_{D1} = 2 / 3 \times S_{M1}$ = **0.433**

Seismic design category

Risk category (Table 1.5-1);	II
Seismic design category based on short period response acceleration (Table 11.6-1)	D
Seismic design category based on 1 sec period response acceleration (Table 11.6-2)	D
Seismic design category;	D

Approximate fundamental period

Height above base to highest level of building;	h_n = **13** ft
From Table 12.8-2:	
Structure type;	All other systems

Building period parameter C_t;	C_t = **0.02**
Building period parameter x;	x = **0.75**
Approximate fundamental period (Eq 12.8-7);	$T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$= **0.137** sec
Building fundamental period (Sect 12.8.2);	$T = T_a$ = **0.137** sec
Long-period transition period;	T_L = **16** sec
Limiting period;	$T_S = S_{D1} / S_{DS} \times 1$ sec = **0.719** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1);	A. Bearing_Wall_Systems 15. Light-frame (wood) walls sheathed with wood structural panels
Response modification factor (Table 12.2-1);	R = **6.5**
Seismic importance factor (Table 1.5-2);	I_e = **1.000**
Seismic response coefficient (Sect 11.4.8)	
Calculated (Eq 12.8-2);	$C_{s_calc} = S_{DS} / (R / I_e)$ = **0.0928**
Minimum (Eq.12.8-5);	$C_{s_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.0265**
Seismic response coefficient;	C_s = **0.0928**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure;	W = **53.9** kips
Seismic response coefficient;	C_s = **0.0928**
Seismic base shear (Eq 12.8-1);	$V = C_s \times W$ = **5.0** kips

;

CHECK WIND AND SEISMIC

h=**13.500** ft

Pseismic_factored= getsectionvar(6,"V","undefined")= **5.005** kips

Pmax = 16 psf;

Pwind_length_code = (Pmax × bl × h) ×.6 = **7.776** kips

Pwind_length_calcs = (h × (bl-(4 ×a)) × (GC_{pi_1_A_n}+ abs(GC_{pi_4_A_n})) + (4 ×a×h) × (GC_{pi_1E_A_p} +abs(GC_{pi_4E_A_p}))) × .6 = **6.551** kips

Pwind_length = max(Pwind_length_code, Pwind_length_calcs) = **7.776** kips

Pwind_width_code = (Pmax × bw × h) ×.6 = **4.588** kip

Pwind_width_calcs = (h × (bw-(2 ×a)) × (GC_{pi_1_A_n}+ abs(GC_{pi_4_A_n})) + (2 ×a×h) × (GC_{pi_1E_A_p} +abs(GC_{pi_4E_A_p}))) × .6 = **3.802** kips

Pwind_width = max(Pwind_width_code, Pwind_width_calcs) = **4.588** kips

Pseismic = Pseismic_factored/ 1.4 × 1.3 = **4.647** kips; Increasing 30% orthogonally as per ASCE.

if(Pseismic>Pwind_length,"Seismic Control", "Wind Control") = **"Wind Control"**

if(Pseismic>Pwind_width,"Seismic Control", "Wind Control") = **"Seismic Control"**

ANCHOR

Minute Man Anchor Capacity; Anc_cap = 3150 lbs

Anc_all = Anc_cap/√(2) = **2227.386** lbs

CASE A

Sp = Sp = **8.570** ft

Wunit = Wunit = **11.750** ft

L_caseB =bw= **35.400** ft

From Roof Load

L_caseA =bl= **60.000** ft

h_roof = 5.25 ft;

r = h_roof/h= **0.389**

Proof_caseA = max(Pwind_length, Pseismic) × r = **3.024** kips

Spacing between end unit anf first strap; S_1= 2 ft;

Spacing between first and second strap; S2 = 14 ft;

From floor load

h_floor = 8.25 ft;

r = h_floor/h= **0.611**

Pfloor_caseA = max(Pwind_length, Pseismic) × r = **4.752** kips

Pup = (abs(GC_{pi_2_B_p}) +abs(GC_{pi_3_A_p}))/2 ×.6× Wunit/2 × L_caseA – ((R_dl +F_dl) × L_caseA × Wunit/2) ×.6 = **-1.610** kips

Wfloor_caseA=(Pfloor_caseA)/L_caseA = **79.200** plf

Max Spacing; Spmax = Anc_all/ Wfloor_caseA= **28.124** ft

Nfloor= (Pfloor_caseA) /Anc_all =**2.133**

WcaseA = Proof_caseA/2 + Wfloor_caseA × (S_1 +S2/2) = **2.225** kips

Nroof = WcaseA /Anc_all = **0.999**

CASE B

From Roof Load

L_caseB =bw= **35.400** ft

h_roof = 5.25 ft;

r = h_roof/h= **0.389**

Proof_caseB = max(Pwind_width, Pseismic) × r = **1.807** kips

WcaseB = Proof_caseB/2 = **0.904** kips

From floor load

h_floor = 8.25 ft;

r = h_floor/h= **0.611**

Pfloor_caseB = max(Pwind_width, Pseismic) × r = **2.840** kips

Nfloor= Pfloor_caseB /Anc_all =**1.275**

Wfloor_caseB= Pfloor_caseB/L_caseB = **80.222** plf

Max Spacing; Spmax = Anc_all/ Wfloor_caseB= **27.765** ft

Spacing between end unit anf first strap; S3= 1.73 ft;

Spacing between first and second strap; S4 = 11.75 ft;

Proof_caseB = Proof_caseB/2 + Wfloor_caseB × (S3 +S4/2) = **1.514** kips

Nroof = Proof_caseB /Anc_all = **0.680**

Pcorner anchor perpandicular to main i beam

Wup = (abs(GC_{pi_2E_A_p}) + abs(GC_{pi_3E_A_p}))/2 ×.6 = **10.259** psf

Anchor spacing on long side; a_sp = (S_1 +S2/2) = **9.000** ft;

Pup= Wup × a_sp × Wunit/2- ((R_dl +F_dl) × a_sp× Wunit/2) ×.6 = **-0.092** kips

N = (Pup+ Proof_caseA/2)/Anc_all = **0.637**

OVERTURNING LOAD

Roof

hover = 11 ft;

Beam Spacing; BeamSp = 32 ft;

Case A

Pover_caseA = Proof_caseA/2 = **1.512** kips

Proof_caseA_over = Pover_caseA × hover/BeamSp = **0.520** kips

Case B

Pover_caseB = WcaseB = **0.904** kips

Proof_caseB_over = Pover_caseB × h/L_caseA = **0.203** kips

Floor

Case A

hfloor = 3 ft;

Wunit = Wunit = **11.750** ft

Sp = Sp = **8.570** ft

Pfloor_caseA_over = Sp× Wfloor_caseA×hfloor/L_caseB = **0.058** kips

Case B

Pfloor_caseB_over = Wunit × Wfloor_caseB×hfloor/L_caseA = **0.047** kips

Down from overturning force

P_corner_caseA = Pfloor_caseA_over +Proof_caseA_over = **0.577** kips

P_corner_caseB = Pfloor_caseB_over +Proof_caseB_over = **0.250** kips

P_corner_down = max(P_corner_caseA, P_corner_caseB) = **0.577** kips

Check anchor at corner for shear and uplift:

N_roof_tension = 1

N_roof_shear = 1

Vstrap = WcaseA/N_roof_shear =**2.225** kips

Tstrap = P_corner_down/N_roof_tension = **0.577** kips

If(Vstrap/sin(45) <Anc_cap,"OK","NG") = **"OK"**

If(Tstrap/cos(45) <Anc_cap,"OK","NG") = **"OK"**

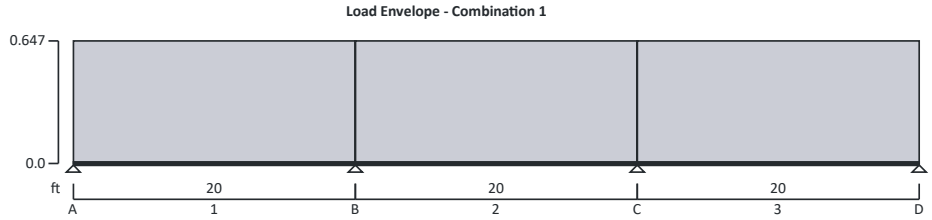
GRAVITY LOAD

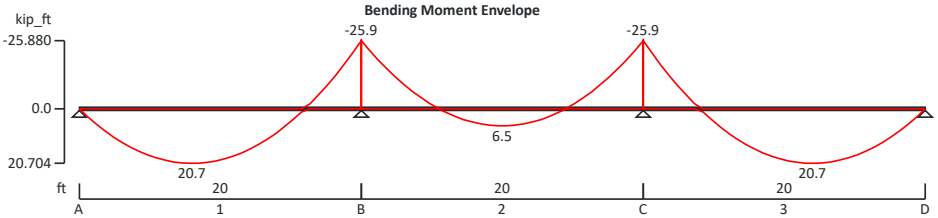
MATELINE WOOD BEAM REACTIONS

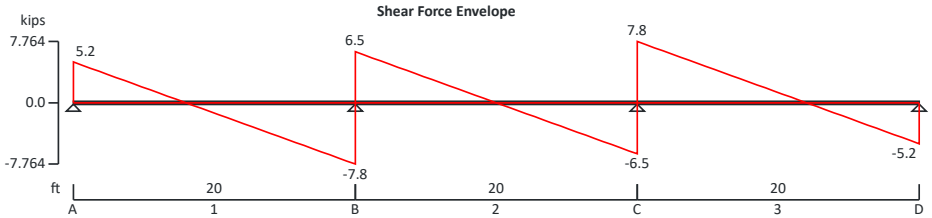
STRUCTURAL COMPOSITE LUMBER MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2015 using the ASD method

Tedds calculation version 1.7.10







Applied loading

Beam loads

Dead full UDL 118 lb/ft
Roof Live full UDL 235 lb/ft
Snow full UDL 294 lb/ft

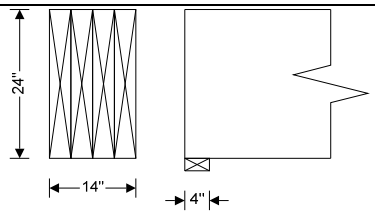
Load combinations

Load combination 1	Support A	Dead × 1.00
		Roof Live × 1.00

		Snow × 1.00
	Span 1	Dead × 1.00
		Roof Live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Roof Live × 1.00
		Snow × 1.00
	Span 2	Dead × 1.00
		Roof Live × 1.00
		Snow × 1.00
	Support C	Dead × 1.00
		Roof Live × 1.00
		Snow × 1.00
	Span 3	Dead × 1.00
		Roof Live × 1.00
		Snow × 1.00
	Support D	Dead × 1.00
		Roof Live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **20704** lb_ft;	M_{min} = **-25880** lb_ft
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **25880** lb_ft	
Maximum shear;	F_{max} = **7764** lb;	F_{min} = **-7764** lb
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **7764** lb	
Total load on member;	W_{tot} = **38820** lb	
Reaction at support A;	R_{A_max} = **5176** lb;	R_{A_min} = **5176** lb
Unfactored dead load reaction at support A;	R_{A_Dead} = **944** lb	
Unfactored roof live load reaction at support A;	$R_{A_Roof\ Live}$ = **1880** lb	
Unfactored snow load reaction at support A;	R_{A_Snow} = **2352** lb	
Reaction at support B;	R_{B_max} = **14234** lb;	R_{B_min} = **14234** lb
Unfactored dead load reaction at support B;	R_{B_Dead} = **2596** lb	
Unfactored roof live load reaction at support B;	$R_{B_Roof\ Live}$ = **5170** lb	
Unfactored snow load reaction at support B;	R_{B_Snow} = **6468** lb	
Reaction at support C;	R_{C_max} = **14234** lb;	R_{C_min} = **14234** lb
Unfactored dead load reaction at support C;	R_{C_Dead} = **2596** lb	
Unfactored roof live load reaction at support C;	$R_{C_Roof\ Live}$ = **5170** lb	
Unfactored snow load reaction at support C;	R_{C_Snow} = **6468** lb	
Reaction at support D;	R_{D_max} = **5176** lb;	R_{D_min} = **5176** lb
Unfactored dead load reaction at support D;	R_{D_Dead} = **944** lb	
Unfactored roof live load reaction at support D;	$R_{D_Roof\ Live}$ = **1880** lb	
Unfactored snow load reaction at support D;	R_{D_Snow} = **2352** lb	



Composite section details

Breadth of composite section;	b = **3.5** in
Depth of composite section;	d = **24** in
Number of composite sections in member;	N = **4**
Overall breadth of composite member;	$b_b = N \times b$ = **14** in
Composite type and grade;	Parallam PSL, 2.0E-2900Fb grade
Bending parallel to grain;	F_b = **2900** lb/in^2
Tension parallel to grain;	F_t = **2025** lb/in^2
Compression parallel to grain;	F_c = **2900** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Shear parallel to grain;	F_v = **290** lb/in^2
Modulus of elasticity;	E = **2000000** lb/in^2
Modulus of elasticity, stability calculations;	E_{min} = **1017000** lb/in^2
Mean shear modulus;	$G_{def} = E / 16$ = **125000** lb/in^2
Average density;	ρ = **45** lb/ft^3

Member details

Service condition;	**Dry**
Length of span 1;	L_{s1} = **20** ft
Length of span 2;	L_{s2} = **20** ft
Length of span 3;	L_{s3} = **20** ft
Length of bearing;	L_b = **4** in
Load duration;	**Ten years**

Section properties

Cross sectional area of member;	$A = N \times b \times d$ = **336.00** in^2
Section modulus;	$S_x = N \times b \times d^2 / 6$ = **1344.00** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **784.00** in^3
Second moment of area;	$I_x = N \times b \times d^3 / 12$ = **16128.00** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **5488.00** in^4

Adjustment factors

Load duration factor - Table 2.3.2;	C_D = **1.00**
Temperature factor - Table 2.3.3;	C_t = **1.00**
Volume factor;	$C_V = (12 \text{ in} / \max(d, 3.5 \text{ in}))^{0.111}$ = **0.93**
Repetitive member factor - cl.8.3.7;	C_r = **1.00**
Length factor;	$C_{Len} = (4 \text{ ft} / (L_{s1} + L_{s2} + L_{s3}))^{0.056}$ = **0.86**
Bearing area factor - cl.3.10.4;	$C_b = (L_b + 0.375 \text{ in}) / L_b$ = **1.09**
Depth-to-breadth ratio;	$d / (N \times b)$ = **1.71**

- Beam is fully restrained

Beam stability factor - cl.3.3.3; C_L = **1.00**

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain; $F_{c_perp}' = F_{c_perp} \times C_t \times C_b$ = **684** lb/in^2

Applied compression stress perpendicular to grain; $f_{c_perp} = R_{B_max} / (N \times b \times L_b)$ = **254** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.372**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress; $F_b' = F_b \times C_D \times C_t \times \min(C_L, C_V) \times C_r$ = **2685** lb/in^2

Actual bending stress; $f_b = M / S_x$ = **231** lb/in^2

f_b / F_b' = **0.086**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress; $F_v' = F_v \times C_D \times C_t$ = **290** lb/in^2

Actual shear stress - eq.3.4-2; $f_v = 3 \times F / (2 \times A)$ = **35** lb/in^2

f_v / F_v' = **0.120**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection; $E' = E \times C_M \times C_t$ = **2000000** lb/in^2

Design deflection; $\delta_{adm} = 0.0044 \times L_{s3}$ = **1.056** in

Total deflection; δ_{b_s3} = **0.038** in

$\delta_{b_s3} / \delta_{adm}$ = **0.036**

PASS - Total deflection is less than design deflection

CONT EXTERIOR BEAM REACTION

STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

In accordance with AISC360-10 using the ASD method

Tedds calculation version 3.0.15

Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free
Support C	Vertically restrained
	Rotationally free
Support D	Vertically restrained
	Rotationally free
Support E	Vertically restrained
	Rotationally free
Support F	Vertically restrained
	Rotationally free
Support G	Vertically restrained
	Rotationally free
Support H	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead full UDL 0.178 kips/ft
	Live full UDL 0.294 kips/ft
	Roof live full UDL 0.118 kips/ft
	Snow full UDL 0.147 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
		Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00

	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support C	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support D	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support E	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support F	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support G	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00

		Roof live × 1.00
		Snow × 1.00
	Support H	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00

Analysis results

Maximum moment;	M_{max} = **4.2** kips_ft;	M_{min} = **-5.7** kips_ft
Maximum moment span 1;	M_{s1_max} = **4.2** kips_ft;	M_{s1_min} = **-5.7** kips_ft
Maximum moment span 2;	M_{s2_max} = **1.8** kips_ft;	M_{s2_min} = **-5.7** kips_ft
Maximum moment span 3;	M_{s3_max} = **2.4** kips_ft;	M_{s3_min} = **-4.6** kips_ft
Maximum moment span 4;	M_{s4_max} = **2.2** kips_ft;	M_{s4_min} = **-4.6** kips_ft
Maximum moment span 5;	M_{s5_max} = **2.4** kips_ft;	M_{s5_min} = **-4.6** kips_ft
Maximum moment span 6;	M_{s6_max} = **1.8** kips_ft;	M_{s6_min} = **-5.7** kips_ft
Maximum moment span 7;	M_{s7_max} = **4.2** kips_ft;	M_{s7_min} = **-5.7** kips_ft
Maximum shear;	V_{max} = **3.8** kips;	V_{min} = **-3.8** kips
Maximum shear span 1;	V_{s1_max} = **2.5** kips;	V_{s1_min} = **-3.8** kips
Maximum shear span 2;	V_{s2_max} = **3.3** kips;	V_{s2_min} = **-3** kips
Maximum shear span 3;	V_{s3_max} = **3.1** kips;	V_{s3_min} = **-3.2** kips
Maximum shear span 4;	V_{s4_max} = **3.2** kips;	V_{s4_min} = **-3.2** kips
Maximum shear span 5;	V_{s5_max} = **3.2** kips;	V_{s5_min} = **-3.1** kips
Maximum shear span 6;	V_{s6_max} = **3** kips;	V_{s6_min} = **-3.3** kips
Maximum shear span 7;	V_{s7_max} = **3.8** kips;	V_{s7_min} = **-2.5** kips
Deflection;	δ_{max} = **0** in;	δ_{min} = **0** in
Deflection span 1;	δ_{s1_max} = **0** in;	δ_{s1_min} = **0** in
Deflection span 2;	δ_{s2_max} = **0** in;	δ_{s2_min} = **0** in
Deflection span 3;	δ_{s3_max} = **0** in;	δ_{s3_min} = **0** in
Deflection span 4;	δ_{s4_max} = **0** in;	δ_{s4_min} = **0** in
Deflection span 5;	δ_{s5_max} = **0** in;	δ_{s5_min} = **0** in
Deflection span 6;	δ_{s6_max} = **0** in;	δ_{s6_min} = **0** in
Deflection span 7;	δ_{s7_max} = **0** in;	δ_{s7_min} = **0** in
Maximum reaction at support A;	R_{A_max} = **2.5** kips;	R_{A_min} = **2.5** kips
Unfactored dead load reaction at support A;	R_{A_Dead} = **0.6** kips	
Unfactored live load reaction at support A;	R_{A_Live} = **1** kips	
Unfactored roof live load reaction at support A;	$R_{A_Roof\ live}$ = **0.4** kips	
Unfactored snow load reaction at support A;	R_{A_Snow} = **0.5** kips	
Maximum reaction at support B;	R_{B_max} = **7.2** kips;	R_{B_min} = **7.2** kips
Unfactored dead load reaction at support B;	R_{B_Dead} = **1.7** kips	
Unfactored live load reaction at support B;	R_{B_Live} = **2.9** kips	
Unfactored roof live load reaction at support B;	$R_{B_Roof\ live}$ = **1.1** kips	
Unfactored snow load reaction at support B;	R_{B_Snow} = **1.4** kips	
Maximum reaction at support C;	R_{C_max} = **6.1** kips;	R_{C_min} = **6.1** kips
Unfactored dead load reaction at support C;	R_{C_Dead} = **1.5** kips	

Unfactored live load reaction at support C;	R_{C_Live} = **2.4** kips	
Unfactored roof live load reaction at support C;	$R_{C_Roof\ live}$ = **1** kips	
Unfactored snow load reaction at support C;	R_{C_Snow} = **1.2** kips	
Maximum reaction at support D;	R_{D_max} = **6.4** kips;	R_{D_min} = **6.4** kips
Unfactored dead load reaction at support D;	R_{D_Dead} = **1.5** kips	
Unfactored live load reaction at support D;	R_{D_Live} = **2.5** kips	
Unfactored roof live load reaction at support D;	$R_{D_Roof\ live}$ = **1** kips	
Unfactored snow load reaction at support D;	R_{D_Snow} = **1.3** kips	
Maximum reaction at support E;	R_{E_max} = **6.4** kips;	R_{E_min} = **6.4** kips
Unfactored dead load reaction at support E;	R_{E_Dead} = **1.5** kips	
Unfactored live load reaction at support E;	R_{E_Live} = **2.5** kips	
Unfactored roof live load reaction at support E;	$R_{E_Roof\ live}$ = **1** kips	
Unfactored snow load reaction at support E;	R_{E_Snow} = **1.3** kips	
Maximum reaction at support F;	R_{F_max} = **6.1** kips;	R_{F_min} = **6.1** kips
Unfactored dead load reaction at support F;	R_{F_Dead} = **1.5** kips	
Unfactored live load reaction at support F;	R_{F_Live} = **2.4** kips	
Unfactored roof live load reaction at support F;	$R_{F_Roof\ live}$ = **1** kips	
Unfactored snow load reaction at support F;	R_{F_Snow} = **1.2** kips	
Maximum reaction at support G;	R_{G_max} = **7.2** kips;	R_{G_min} = **7.2** kips
Unfactored dead load reaction at support G;	R_{G_Dead} = **1.7** kips	
Unfactored live load reaction at support G;	R_{G_Live} = **2.9** kips	
Unfactored roof live load reaction at support G;	$R_{G_Roof\ live}$ = **1.1** kips	
Unfactored snow load reaction at support G;	R_{G_Snow} = **1.4** kips	
Maximum reaction at support H;	R_{H_max} = **2.5** kips;	R_{H_min} = **2.5** kips
Unfactored dead load reaction at support H;	R_{H_Dead} = **0.6** kips	
Unfactored live load reaction at support H;	R_{H_Live} = **1** kips	
Unfactored roof live load reaction at support H;	$R_{H_Roof\ live}$ = **0.4** kips	
Unfactored snow load reaction at support H;	R_{H_Snow} = **0.5** kips	

Section details

Section type;	**M 12x11.8 (AISC 15th Edn (v15.0))**
ASTM steel designation;	**A992**
Steel yield stress;	F_y = **50** ksi
Steel tensile stress;	F_u = **65** ksi
Modulus of elasticity;	E = **29000** ksi

Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**
Safety factor for compression	Ω_c = **1.67**
Safety factor for flexure	Ω_b = **1.67**

Lateral bracing

Span 1 has continuous lateral bracing
Span 2 has continuous lateral bracing
Span 3 has continuous lateral bracing

Span 4 has continuous lateral bracing
Span 5 has continuous lateral bracing
Span 6 has continuous lateral bracing
Span 7 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio; $b_f / (2 \times t_f) =$ **6.82**

Limiting ratio for compact section; $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]} =$ **9.15**

Limiting ratio for non-compact section; $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]} =$ **24.08**; Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio; $(d - 2 \times k) / t_w =$ **61.44**

Limiting ratio for compact section; $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]} =$ **90.55**

Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]} =$ **137.27**; Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **3.825** kips

Web area $A_w = d \times t_w =$ **2.124** in^2

Web plate buckling coefficient $k_v =$ **5**

Web shear coefficient - eq G2-4 $C_v = 1.10 \times \sqrt{(k_v \times E / F_y)} / ((d - 2 \times k) / t_w) =$ **0.964**

Nominal shear strength – eq G2-1 $V_n = 0.6 \times F_y \times A_w \times C_v =$ **61.440** kips

Safety factor for shear $\Omega_v =$ **1.67**

Allowable shear strength $V_c = V_n / \Omega_v =$ **36.790** kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis at span 1 - Chapter F

Required flexural strength; $M_r = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) =$ **5.718** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1; $M_{nyld} = M_p = F_y \times Z_x =$ **59.583** kips_ft

Nominal flexural strength; $M_n = M_{nyld} =$ **59.583** kips_ft

Allowable flexural strength; $M_c = M_n / \Omega_b =$ **35.679** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live and roof live loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 =$ **0.286** in

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.017** in

PASS - Maximum deflection does not exceed deflection limit

CONT INTERIOR BEAM REACTION

STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

In accordance with AISC360-10 using the ASD method

Tedds calculation version 3.0.15

Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free
Support C	Vertically restrained
	Rotationally free
Support D	Vertically restrained
	Rotationally free
Support E	Vertically restrained
	Rotationally free
Support F	Vertically restrained
	Rotationally free
Support G	Vertically restrained
	Rotationally free
Support H	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead full UDL 0.06 kips/ft
	Live full UDL 0.294 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
		Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
		Dead × 1.00
		Live × 1.00
		Roof live × 1.00

	Snow × 1.00
Support C	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support D	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support E	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support F	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support G	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
	Dead × 1.00
	Live × 1.00
	Roof live × 1.00
	Snow × 1.00
Support H	Dead × 1.00

		Live × 1.00
		Roof live × 1.00
		Snow × 1.00
Analysis results		
Maximum moment;	M_{max} = **2** kips_ft;	M_{min} = **-2.7** kips_ft
Maximum moment span 1;	M_{s1_max} = **2** kips_ft;	M_{s1_min} = **-2.7** kips_ft
Maximum moment span 2;	M_{s2_max} = **0.9** kips_ft;	M_{s2_min} = **-2.7** kips_ft
Maximum moment span 3;	M_{s3_max} = **1.1** kips_ft;	M_{s3_min} = **-2.2** kips_ft
Maximum moment span 4;	M_{s4_max} = **1.1** kips_ft;	M_{s4_min} = **-2.2** kips_ft
Maximum moment span 5;	M_{s5_max} = **1.1** kips_ft;	M_{s5_min} = **-2.2** kips_ft
Maximum moment span 6;	M_{s6_max} = **0.9** kips_ft;	M_{s6_min} = **-2.7** kips_ft
Maximum moment span 7;	M_{s7_max} = **2** kips_ft;	M_{s7_min} = **-2.7** kips_ft
Maximum shear;	V_{max} = **1.8** kips;	V_{min} = **-1.8** kips
Maximum shear span 1;	V_{s1_max} = **1.2** kips;	V_{s1_min} = **-1.8** kips
Maximum shear span 2;	V_{s2_max} = **1.6** kips;	V_{s2_min} = **-1.4** kips
Maximum shear span 3;	V_{s3_max} = **1.5** kips;	V_{s3_min} = **-1.5** kips
Maximum shear span 4;	V_{s4_max} = **1.5** kips;	V_{s4_min} = **-1.5** kips
Maximum shear span 5;	V_{s5_max} = **1.5** kips;	V_{s5_min} = **-1.5** kips
Maximum shear span 6;	V_{s6_max} = **1.4** kips;	V_{s6_min} = **-1.6** kips
Maximum shear span 7;	V_{s7_max} = **1.8** kips;	V_{s7_min} = **-1.2** kips
Deflection;	δ_{max} = **0** in;	δ_{min} = **0** in
Deflection span 1;	δ_{s1_max} = **0** in;	δ_{s1_min} = **0** in
Deflection span 2;	δ_{s2_max} = **0** in;	δ_{s2_min} = **0** in
Deflection span 3;	δ_{s3_max} = **0** in;	δ_{s3_min} = **0** in
Deflection span 4;	δ_{s4_max} = **0** in;	δ_{s4_min} = **0** in
Deflection span 5;	δ_{s5_max} = **0** in;	δ_{s5_min} = **0** in
Deflection span 6;	δ_{s6_max} = **0** in;	δ_{s6_min} = **0** in
Deflection span 7;	δ_{s7_max} = **0** in;	δ_{s7_min} = **0** in
Maximum reaction at support A;	R_{A_max} = **1.2** kips;	R_{A_min} = **1.2** kips
Unfactored dead load reaction at support A;	R_{A_Dead} = **0.2** kips	
Unfactored live load reaction at support A;	R_{A_Live} = **1** kips	
Maximum reaction at support B;	R_{B_max} = **3.4** kips;	R_{B_min} = **3.4** kips
Unfactored dead load reaction at support B;	R_{B_Dead} = **0.6** kips	
Unfactored live load reaction at support B;	R_{B_Live} = **2.9** kips	
Maximum reaction at support C;	R_{C_max} = **2.9** kips;	R_{C_min} = **2.9** kips
Unfactored dead load reaction at support C;	R_{C_Dead} = **0.5** kips	
Unfactored live load reaction at support C;	R_{C_Live} = **2.4** kips	
Maximum reaction at support D;	R_{D_max} = **3.1** kips;	R_{D_min} = **3.1** kips
Unfactored dead load reaction at support D;	R_{D_Dead} = **0.5** kips	
Unfactored live load reaction at support D;	R_{D_Live} = **2.5** kips	
Maximum reaction at support E;	R_{E_max} = **3.1** kips;	R_{E_min} = **3.1** kips
Unfactored dead load reaction at support E;	R_{E_Dead} = **0.5** kips	
Unfactored live load reaction at support E;	R_{E_Live} = **2.5** kips	
Maximum reaction at support F;	R_{F_max} = **2.9** kips;	R_{F_min} = **2.9** kips

Unfactored dead load reaction at support F; R_{F_Dead} = **0.5** kips

Unfactored live load reaction at support F; R_{F_Live} = **2.4** kips

Maximum reaction at support G; R_{G_max} = **3.4** kips; R_{G_min} = **3.4** kips

Unfactored dead load reaction at support G; R_{G_Dead} = **0.6** kips

Unfactored live load reaction at support G; R_{G_Live} = **2.9** kips

Maximum reaction at support H; R_{H_max} = **1.2** kips; R_{H_min} = **1.2** kips

Unfactored dead load reaction at support H; R_{H_Dead} = **0.2** kips

Unfactored live load reaction at support H; R_{H_Live} = **1** kips

Section details

Section type; **M 12x11.8 (AISC 15th Edn (v15.0))**

ASTM steel designation; **A992**

Steel yield stress; F_y = **50** ksi

Steel tensile stress; F_u = **65** ksi

Modulus of elasticity; E = **29000** ksi

Safety factors

Safety factor for tensile yielding Ω_{ty} = **1.67**

Safety factor for tensile rupture Ω_{tr} = **2.00**

Safety factor for compression Ω_c = **1.67**

Safety factor for flexure Ω_b = **1.67**

Lateral bracing

Span 1 has continuous lateral bracing
Span 2 has continuous lateral bracing
Span 3 has continuous lateral bracing
Span 4 has continuous lateral bracing
Span 5 has continuous lateral bracing
Span 6 has continuous lateral bracing
Span 7 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio; $b_f / (2 \times t_f)$ = **6.82**

Limiting ratio for compact section; $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section; $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08**; Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio; $(d - 2 \times k) / t_w$ = **61.44**

Limiting ratio for compact section; $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **137.27**; Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **1.837** kips

Web area $A_w = d \times t_w$ = **2.124** in^2

Web plate buckling coefficient k_v = **5**

Web shear coefficient - eq G2-4 $C_v = 1.10 \times \sqrt{(k_v \times E / F_y)} / ((d - 2 \times k) / t_w)$ = **0.964**

Nominal shear strength – eq G2-1 $V_n = 0.6 \times F_y \times A_w \times C_v =$ **61.440** kips

Safety factor for shear $\Omega_v =$ **1.67**

Allowable shear strength $V_c = V_n / \Omega_v =$ **36.790** kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis at span 1 - Chapter F

Required flexural strength; $M_r = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) =$ **2.746** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1; $M_{nyld} = M_p = F_y \times Z_x =$ **59.583** kips_ft

Nominal flexural strength; $M_n = M_{nyld} =$ **59.583** kips_ft

Allowable flexural strength; $M_c = M_n / \Omega_b =$ **35.679** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and live loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 =$ **0.286** in

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.01** in

PASS - Maximum deflection does not exceed deflection limit

PIER LOAD

Roof Dead Load; Rd = R_dl = **10.000** psf;

Roof Live Load ; Rl = R_ll = **20.000** psf

Floor Dead Load;Fd= F_dl = **10.000** psf;

Floor Live Load;Fl = F_ll = **50.000** psf;

Unit width; Wunit = Wunit = **11.750** ft;

Rs = Pf = **25.000** psf

Roof trib at ext. column; Ltrib_ext = Ltrib_ext= **10.000** ft;

Roof trib at int. column; Ltrib_int = Ltrib_int= **20.000** ft;

Wall Dead Load; Wdl = Wdl = **60.000** plf;

Load Exterior pier: Second pier

DL = max(getsectionvar(18,"R_{B_Dead}","undefined"), getsectionvar(18,"R_{G_Dead}", "undefined")) = **1.730** kips

Fll = max(getsectionvar(18,"R_{B_Live}","undefined"), getsectionvar(18,"R_{G_Live}","undefined")) = **2.857** kips

Rll = max(getsectionvar(18,"R_{B_Rooflive}","undefined"), getsectionvar(18,"R_{G_Rooflive}","undefined")) = **1.147** kips

RSL = max(getsectionvar(18,"R_{B_Snow}","undefined"), getsectionvar(18,"R_{G_Snow}","undefined")) = **1.428** kips

W = Pfloor_caseA_over = **0.058** kips

U1 =D+Fll; U1 = DL + Fll = **4.586** kips

U2 =D+RLll; U2 = DL + Rll= **2.876** kips

U3 =D+RSL; U3 = DL + RSL = **3.158** kips

U4 =D+.75(Fll+ max(Rll,RSL); U4 = DL + .75 ×(Fll +max(RSL,Rll)) = **4.943** kips

U5 =DL + W = **1.787** kips

U6 = .6 ×DL +W = **1.095** kips

U7 = DL + .75 × (Fll+ max(RSL,Rll) + W) = **4.987** kips

U_ext = Max(U1,U2,U3,U4,U5,U6,U7) = **4.987** kips

Load Exterior pier:

DL = max(getsectionvar(18,"R_{F_Dead}","undefined"), getsectionvar(18,"R_{E_Dead}", "undefined")) = **1.536** kips

Fll = max(getsectionvar(18,"R_{F_Live}","undefined"), getsectionvar(18,"R_{E_Live}","undefined")) = **2.537** kips

Rll = max(getsectionvar(18,"R_{F_Rooflive}","undefined"), getsectionvar(18,"R_{E_Rooflive}","undefined")) = **1.018** kips

RSL = max(getsectionvar(18,"R_{F_Snow}","undefined"), getsectionvar(18,"R_{E_Snow}","undefined")) = **1.269** kips

W = Pfloor_caseA_over = **0.058** kips

U1 =D+Fll; U1 = DL + Fll = **4.074** kips

U2 =D+RLll; U2 = DL + Rll= **2.555** kips

U3 =D+RSL; U3 = DL + RSL = **2.805** kips

U4 =D+.75(Fll+ max(Rll,RSL); U4 = DL + .75 ×(Fll +max(RSL,Rll)) = **4.391** kips

U5 =DL + W = **1.594** kips

U6 = .6 ×DL +W = **0.979** kips

U7 = DL + .75 × (Fll+ max(RSL,Rll) + W) = **4.434** kips

U_ext1 = Max(U1,U2,U3,U4,U5,U6,U7) = **4.434** kips

Load Interior pier without roof load second pier:

DL = max(getsectionvar(19,"R_{B_Dead}","undefined"), getsectionvar(19,"R_{G_Dead}", "undefined")) = **0.583** kips

Fll = max(getsectionvar(19,"R_{B_Live}","undefined"), getsectionvar(19,"R_{G_Live}","undefined")) = **2.857** kips

U_int =D+Fll; U_int = DL + Fll = **3.440** kips

Load Interior pier without roof load other pier:

DL = max(getsectionvar(19,"R_{E_Dead}","undefined"), getsectionvar(19,"R_{F_Dead}", "undefined")) = **0.518** kips

Fll = max(getsectionvar(19,"R_{E_Live}","undefined"), getsectionvar(19,"R_{F_Live}","undefined")) = **2.537** kips

U_int1 =D+Fll; U_int1 = DL + Fll = **3.055** kips

Load end pier without roof load:

DL = max(getsectionvar(19,"R_{A_Dead}","undefined"), getsectionvar(19,"R_{H_Dead}", "undefined")) = **0.203** kips

Fll = max(getsectionvar(19,"R_{A_Live}","undefined"), getsectionvar(19,"R_{H_Live}","undefined")) = **0.994** kips

U_end =D+Fll; U_end = DL + Fll = **1.196** kips

Corner Pier

DL = max(getsectionvar(18,"R_{A_Dead}","undefined"), getsectionvar(18,"R_{H_Dead}", "undefined")) = **0.602** kips

Fll = max(getsectionvar(18,"R_{A_Live}","undefined"), getsectionvar(18,"R_{H_Live}","undefined")) = **0.994** kips

Rll = max(getsectionvar(18,"R_{A_Rooflive}","undefined"), getsectionvar(18,"R_{H_Rooflive}","undefined")) = **0.399** kips

RSL = max(getsectionvar(18,"R_{A_Snow}","undefined"), getsectionvar(18,"R_{H_Snow}","undefined")) = **0.497** kips

W = P_corner_down = **0.577** kips

U1 =D+Fll; U1 = DL + Fll = **1.595** kips

U2 =D+RLll; U2 = DL + Rll= **1.000** kips

U3 =D+RSL; U3 = DL + RSL = **1.098** kips

U4 =D+.75(Fll+ max(Rll,RSL); U4 = DL + .75 ×(Fll +max(RSL,Rll)) = **1.719** kips

U5 =DL + W = **1.179** kips

U6 = .6 ×DL +W = **0.938** kips

U7 = DL + .75 × (Fll+ max(RSL,Rll) + W) = **2.152** kips

U_cor = Max(U1,U2,U3,U4,U5,U6,U7) = **2.152** kips

Load Interior column pier:

DL =max(getsectionvar(8,"R_{B_Dead}","undefined"), getsectionvar(8,"R_{C_Dead}","undefined")) = **2.596** kips

Rll = max(getsectionvar(8,"R_{B_RoofLive}", "undefined"), getsectionvar(8,"R_{C_RoofLive}", "undefined")) = **5.170** kips

RSL = max(getsectionvar(8,"R_{B_Snow}","undefined"), getsectionvar(8,"R_{C_Snow}","undefined")) = **6.468** kips

U2 = DL + Rll= **7.766** kips

U3 = DL + RSL= **9.064** kips

U_col = max(U2, U3) = **9.064** kips

Int column uplift:

Pmax = 16 psf;

Puplift = max(Pmax,(abs(GC_{pi_3_A_p}) +abs(GC_{pi_2_A_p}))/2)×.6 = **9.600** psf

Pup = Puplift × Ltrib_int× Wunit = **2.256** kips

DL = (Rd +Fd) × Ltrib_int× Wunit × .6 = **2.820** kips

Pdown_dl = DL = **2.820** kips

Pnet = Pup – Pdown_dl = **-0.564** kips

Load Ext column pier:

DL = Max(getsectionvar(8,"R_{A_Dead}","undefined"), getsectionvar(8,"R_{D_Dead}","undefined"), Rd × Wunit × Ltrib_ext)= **1.175** kips

Rll = Max(getsectionvar(8,"R_{A_RoofLive}","undefined"), getsectionvar(8,"R_{D_RoofLive}","undefined"), Rl × Wunit × Ltrib_ext)= **2.350** kips

RSL = max(getsectionvar(8,"R_{A_Snow}","undefined"), getsectionvar(8,"R_{D_Snow}","undefined"), Rs × Wunit × Ltrib_ext) = **2.938** kips

U2 = DL + Rll= **3.525** kips

U3 = DL + RSL= **4.113** kips

U_col1 = max(U2, U3) = **4.113** kips

Ext column uplift:

Pmax = 16 psf;

Puplift = max(Pmax,(abs(GC_{pi_3_A_p}) +abs(GC_{pi_2_A_p}))/2)×.6 = **9.600** psf

Pup = Puplift × Ltrib_ext× Wunit = **1.128** kips

DL = (Rd +Fd) × Ltrib_ext× Wunit × .6 = **1.410** kips

Pdown_dl = DL = **1.410** kips

Pnet = Pup – Pdown_dl = **-0.282** kips

ABS BEARING PAD

Wpad_16 = 16 in;

Lpad_16= 16 in;

Wpad_20 = 20 in;

Lpad_20 = 20 in;

Wpad_24 = 24 in;

Lpad_24 = 24 in;

Allowable soil bearing capacity; Sall = Sall = **2500.000** psf;

ABS pad 16" x 16"

Pmax = Lpad_16 × Wpad_16 × Sall = **4.444** kips

P = max(U_int, U_cor, U_int1, U_end, U_col1) = **4.113** kips

If(P <Pmax,"OK", "NG") = **"OK"**

20" x 20" ABS pad

Pmax = Lpad_20 × Wpad_20 × Sall = **6.944** kips

P = max(U_ext, U_ext1) = **4.987** kips

If(P <Pmax,"OK", "NG") = **"OK"**

24" x 24" ABS pad

Pmax = Lpad_24 × Wpad_24 × Sall = **10.000** kips

P =U_col = **9.064** kips

If(P <Pmax,"OK", "NG") = **"OK"**