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.

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| | Section: (3) 12 x 60 | | | | Sheet no./rev.: 1 | |
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Thierry Leboulch

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

10/3/2024

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

EXPIRES: 12/31/2024

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# 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;

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# WIND LOADING (ASCE7)

V=Wind

_wld.Exposure = Wind_exp

_wld.Risk = Risk_category

b = bl

d = bw

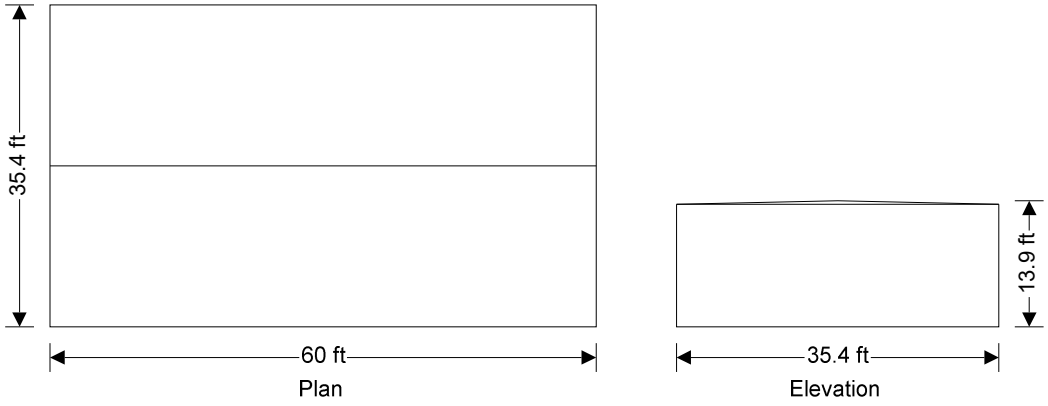
h = h

## 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; | $\alpha_0$ = **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**

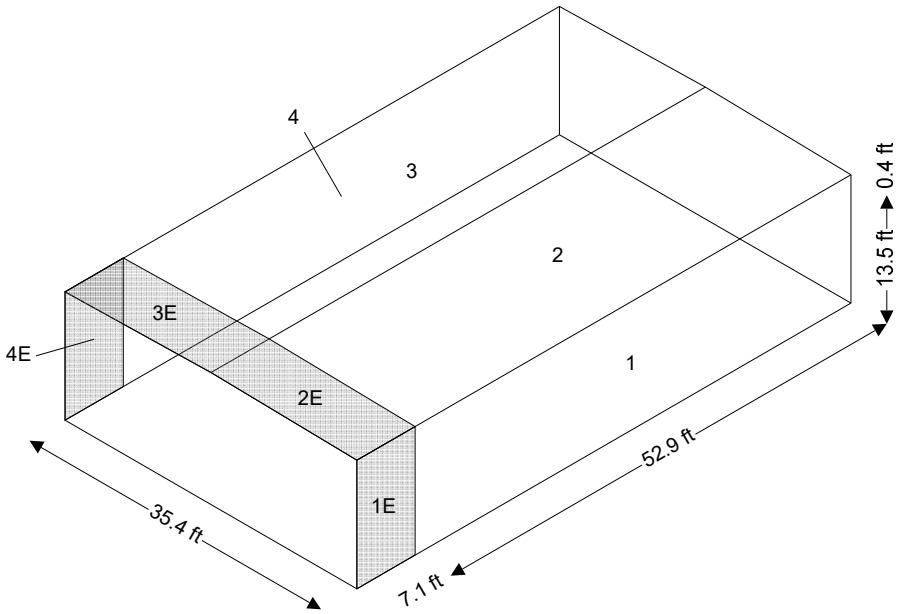
Velocity pressure; $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1psf/mph^2$ = **17.4** psf

**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 |

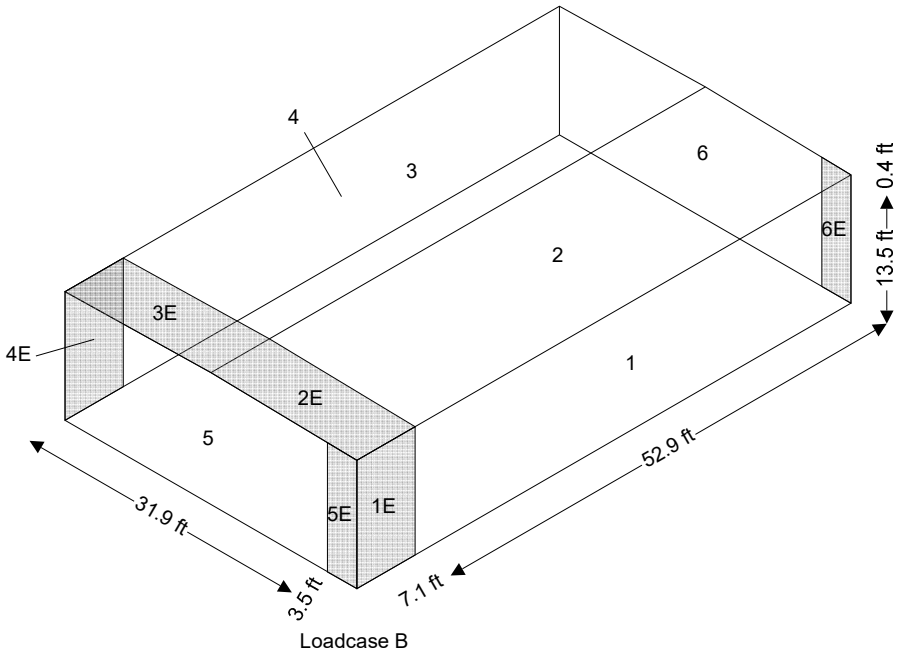



Loadcase A

**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 |

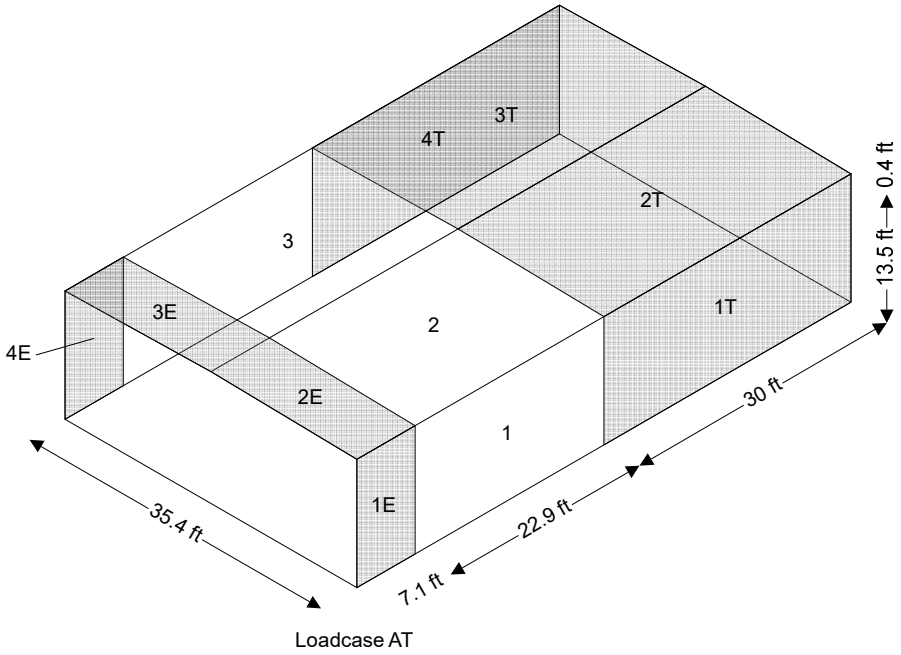


Loadcase B

**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 |

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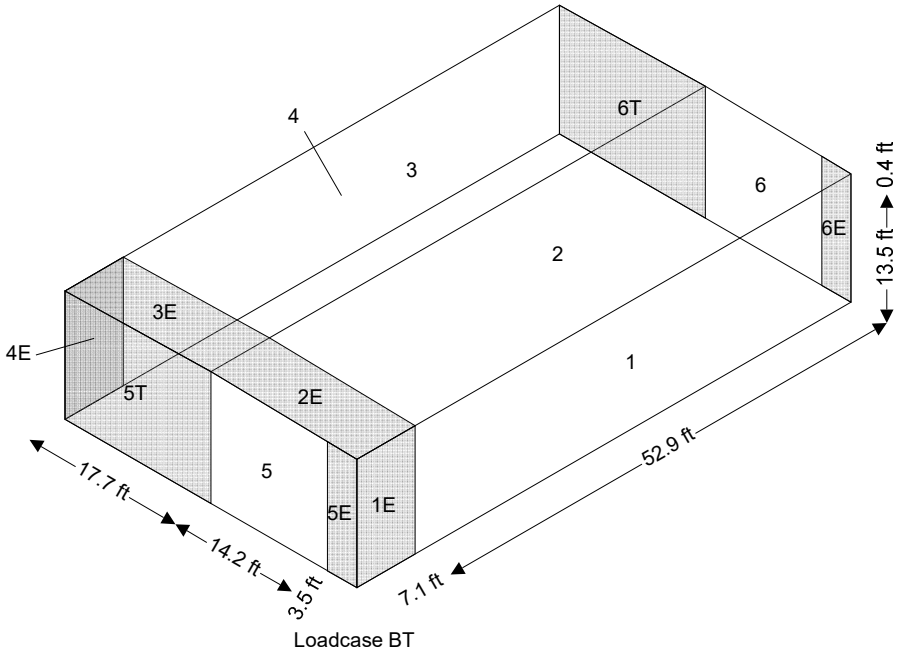
| | | | | | | |
|---|---|---|---|---|---|---|
| 6T | - | -2.1 | -0.5 | 242 | -0.5 | -0.1 |




Loadcase BT

# WIND LOADING COMPONENT AND CLADDING (ASCE7)

V=Wind

_wld.Exposure = Wind_exp

_wld.Risk = Risk_category

b = bl

d = bw

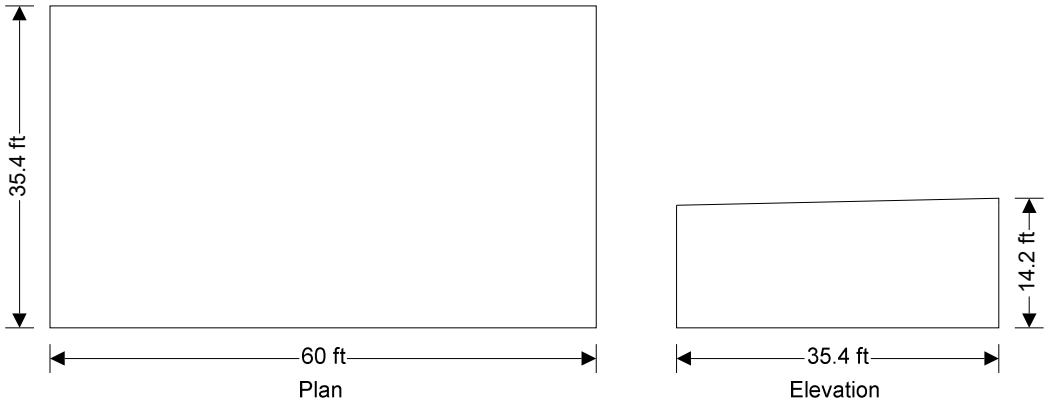
h = h;

## 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; | $\alpha_0$ = **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 × $z_{gl}$/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**

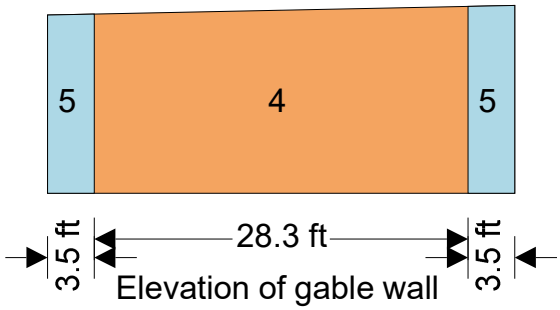
Velocity pressure; $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **17.4** psf

**Peak velocity pressure for internal pressure**

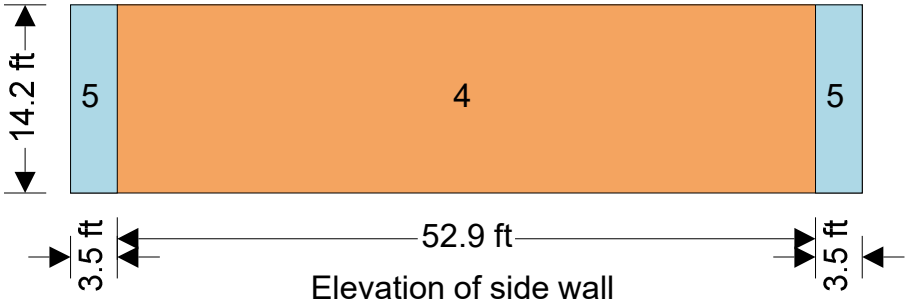
Peak velocity pressure – internal (as roof press.); $q_i$ = **17.45** psf

**Equations used in tables**

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




Elevation of gable wall




Elevation of side wall

**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 |

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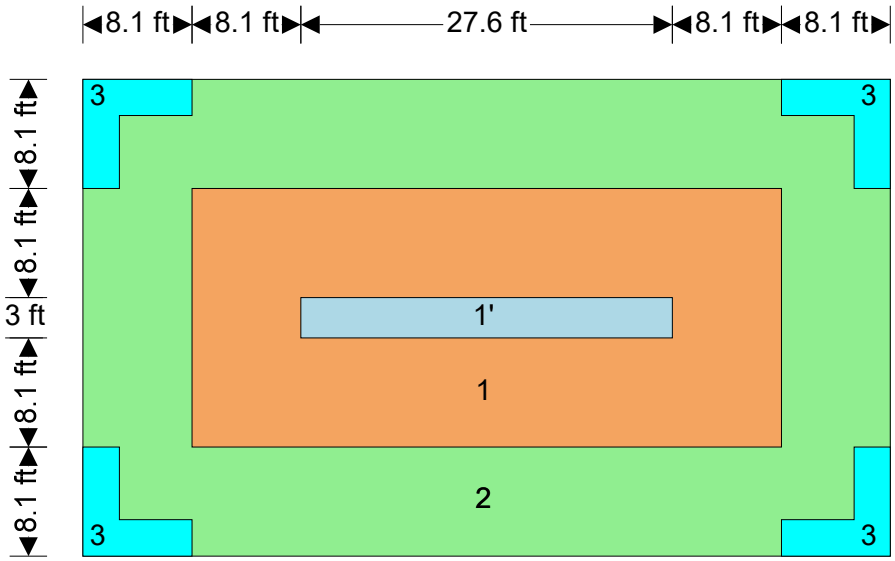
| Component | Zone | Length (ft) | Width (ft) | Eff. area ($ft^2$) | +$GC_p$ | -$GC_p$ | Pres (+ve) (psf) | Pres (-ve) (psf) |
|---|---|---|---|---|---|---|---|---|
| <10 sf | 3 | - | - | 10.0 | 0.30 | -3.20 | 8.4 # | -59.0 |
| 25 sf | 3 | - | - | 25.0 | 0.26 | -2.78 | 7.7 # | -51.6 |
| 50 sf | 3 | - | - | 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

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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
;

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## 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**

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| Section | | | | Sheet no./rev. | |
| (3) 12 x 60 | | | | 15 | |
| Calc. by | Date | Chk'd by | Date | App'd by | Date |
| AD | 10/2/2024 | | | | |

## 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"**

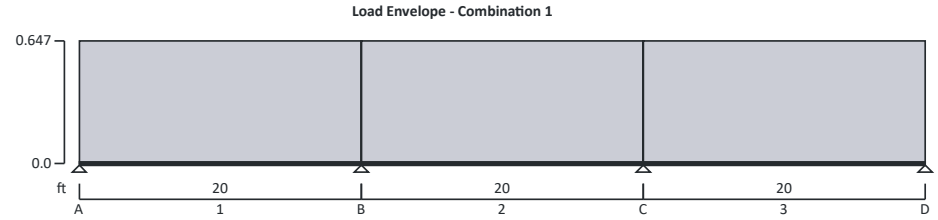
# GRAVITY LOAD

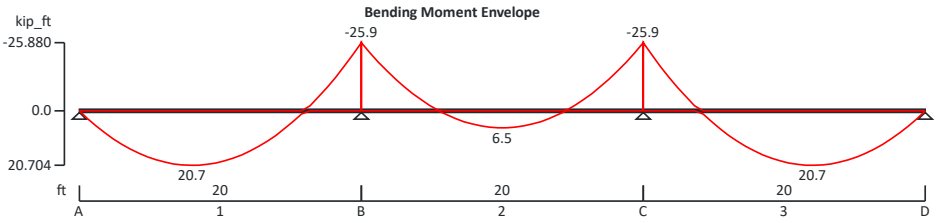
## MATELINE WOOD BEAM REACTIONS

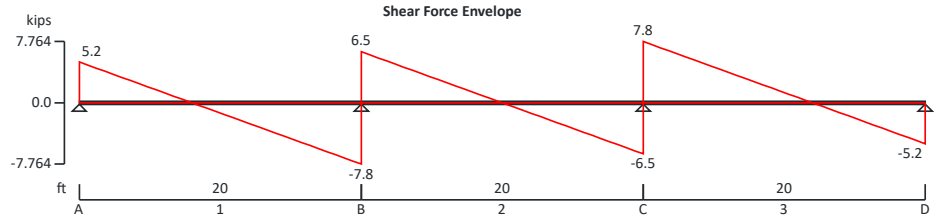
### STRUCTURAL COMPOSITE LUMBER MEMBER ANALYSIS & DESIGN (NDS)

**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 | |

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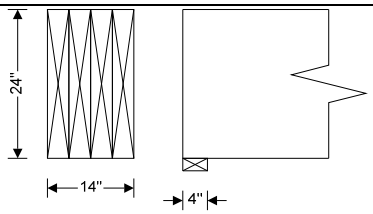
| | Calc. by | Date | Chk'd by | Date | App'd by | Date |
| | AD | 10/2/2024 | | | | |




**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; | $\rho$ = **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; $\delta_{b\_s3}$ = **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; | $\delta_{max}$ = **0** in; | $\delta_{min}$ = **0** in |
| Deflection span 1; | $\delta_{s1\_max}$ = **0** in; | $\delta_{s1\_min}$ = **0** in |
| Deflection span 2; | $\delta_{s2\_max}$ = **0** in; | $\delta_{s2\_min}$ = **0** in |
| Deflection span 3; | $\delta_{s3\_max}$ = **0** in; | $\delta_{s3\_min}$ = **0** in |
| Deflection span 4; | $\delta_{s4\_max}$ = **0** in; | $\delta_{s4\_min}$ = **0** in |
| Deflection span 5; | $\delta_{s5\_max}$ = **0** in; | $\delta_{s5\_min}$ = **0** in |
| Deflection span 6; | $\delta_{s6\_max}$ = **0** in; | $\delta_{s6\_min}$ = **0** in |
| Deflection span 7; | $\delta_{s7\_max}$ = **0** in; | $\delta_{s7\_min}$ = **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 | $\Omega_{ty}$ = **1.67** |
| Safety factor for tensile rupture | $\Omega_{tr}$ = **2.00** |
| Safety factor for compression | $\Omega_c$ = **1.67** |
| Safety factor for flexure | $\Omega_b$ = **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<br>Roof live × 1.00<br>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; | $\delta_{max}$ = **0** in; | $\delta_{min}$ = **0** in |
| Deflection span 1; | $\delta_{s1\_max}$ = **0** in; | $\delta_{s1\_min}$ = **0** in |
| Deflection span 2; | $\delta_{s2\_max}$ = **0** in; | $\delta_{s2\_min}$ = **0** in |
| Deflection span 3; | $\delta_{s3\_max}$ = **0** in; | $\delta_{s3\_min}$ = **0** in |
| Deflection span 4; | $\delta_{s4\_max}$ = **0** in; | $\delta_{s4\_min}$ = **0** in |
| Deflection span 5; | $\delta_{s5\_max}$ = **0** in; | $\delta_{s5\_min}$ = **0** in |
| Deflection span 6; | $\delta_{s6\_max}$ = **0** in; | $\delta_{s6\_min}$ = **0** in |
| Deflection span 7; | $\delta_{s7\_max}$ = **0** in; | $\delta_{s7\_min}$ = **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 | $\Omega_{ty}$ = **1.67** |
| Safety factor for tensile rupture | $\Omega_{tr}$ = **2.00** |
| Safety factor for compression | $\Omega_c$ = **1.67** |
| Safety factor for flexure | $\Omega_b$ = **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***

| WILLIAMS SCOTSMAN INC.<br>4646 East Van Buren Street,<br>Suite 400, Phoenix, AZ 85008 | Project: Bull Run Filtration | | | | Job Ref. | |
|---|---|---|---|---|---|---|
| | Section: (3) 12 x 60 | | | | Sheet no./rev.: 30 | |
| | Calc. by: AD | Date: 10/2/2024 | Chk'd by | Date | App'd by | Date |

## 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

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

| Project | | | | Job Ref. | |
|---|---|---|---|---|---|
| Bull Run Filtration | | | | | |
| Section | | | | Sheet no./rev. | |
| (3) 12 x 60 | | | | 33 | |
| Calc. by | Date | Chk'd by | Date | App'd by | Date |
| AD | 10/2/2024 | | | | |

## 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"**