WILLSCOT

Williams Scotsman Inc.
901 S Bond St Ste 600,
Baltimore, MD 21231

Structural Calculations

Party City Classroom

495 W. Jefferson Avenue

Hayden, CO 81639

Date: 9/06/2019

Thierry R. LeBoulch PE

WILLSCOT Williams Scotsman Inc. 901 S. Bond Street Suite 600 Baltimore, MD 21231	Project Party City Classroom				Job Ref.	
	Section (2) 12 x 60				Sheet no./rev. 2	
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Contents

Thierry Leboulch

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9/06/2019

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LOAD DESIGN

Length of building; bl = 60 ft
Width of building; bw = 24 ft
Height of building; h = 13 ft

Snow Load
b = bl
Pg = 87 psf
Ce= 1.00
Ct = 1.10
Is = 1.00
Roof_Exposure = "Partially exposed"
Risk_category = "II"
Exposure = "C"

Gravity
Roof Dead Load; Rdl = 10 psf;
Roof Live Load ; Rll = 20 psf
Floor Dead Load; Fdl = 10 psf;
Floor Live Load; Fll = 50 psf;
Pier Spacing; Sp = 6.4 ft;
Unit width; Wunit = 11.75 ft;
Roof trib at int. column; Ltrib_int = 30 ft;
Roof trib at ext. column; Ltrib_ext = 15 ft;
Wall Dead Load; Wdl = 60 plf;

Wind
Wind speed (ultimate); Wind= 115 mph
Exposure; Wind_exp = "C"
Risk Category; Risk_category= "II"

Seismic
Risk Category; Risk_category= "II"
Soil Site Class; Soil_site_class = "D"
SS= .271
S1= .075
Dead_load= 58 kips

Allowable soil Bearing Capacity
Sall = 2500 psf;

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WIND LOADING (ASCE7-10)

V=Wind

_wld.Exposure = Wind_exp

_wld.Risk = Risk_category

b = bl

d = bw

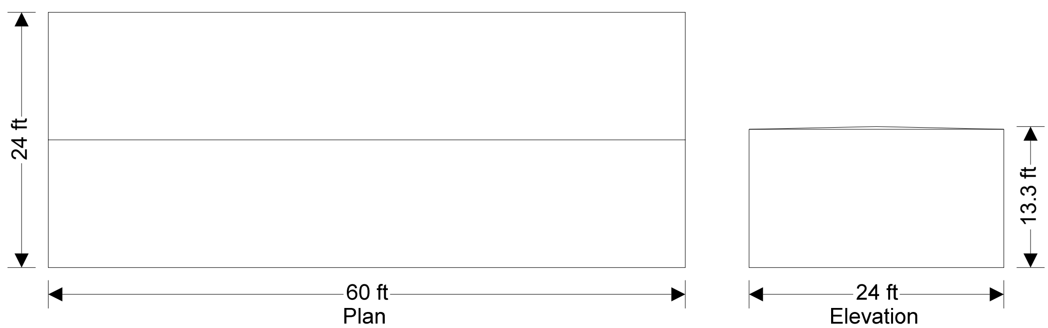
h = h

WIND LOADING

In accordance with ASCE7-10

Using the envelope design method

Tedds calculation version 2.1.05



Building data

Type of roof;	Gable
Length of building;	b = **60.00** ft
Width of building;	d = **24.00** ft
Height to eaves;	H = **13.00** ft
Pitch of roof;	α_0 = **1.2** deg
Mean height;	h = **13.00** ft
End zone width;	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **3.00** ft
Plan length of Zone 2/2E when GC_{pf} negative;	L_{Z2} = min(0.5 × d, 2.5 × H) = **12.00** 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 = **115.0** mph
Risk category;	II
Velocity pressure exponent coef (Table 26.6-1);	K_d = **0.85**
Exposure category (cl 26.7.3);	C
Enclosure classification (cl.26.10);	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1);	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1);	GC_{pi_n} = **-0.18**

Topography

Topography factor not significant;	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (T.28.3-1);	K_z = **0.85**

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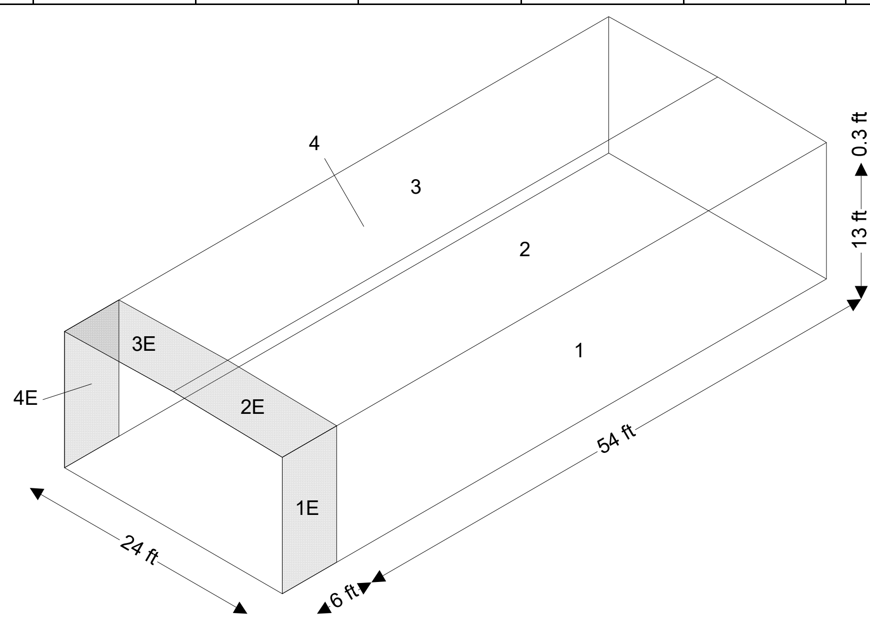
Velocity pressure; $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1psf/mph^2$ = **24.5** psf

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	5.4	14.2	702	3.8	10.0
2	-0.69	-21.3	-12.5	648	-13.8	-8.1
3	-0.37	-13.5	-4.6	648	-8.7	-3.0
4	-0.29	-11.5	-2.7	702	-8.1	-1.9
1E	0.61	10.5	19.3	78	0.8	1.5
2E	-1.07	-30.6	-21.8	72	-2.2	-1.6
3E	-0.53	-17.4	-8.6	72	-1.3	-0.6
4E	-0.43	-14.9	-6.1	78	-1.2	-0.5

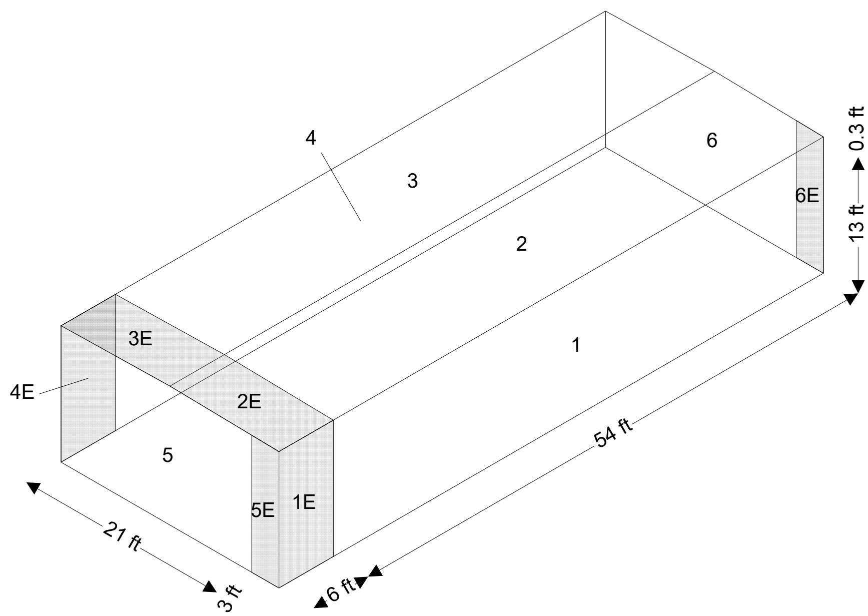


Loadcase A

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	-15.4	-6.6	702	-10.8	-4.6
2	-0.69	-21.3	-12.5	648	-13.8	-8.1
3	-0.37	-13.5	-4.6	648	-8.7	-3.0
4	-0.45	-15.4	-6.6	702	-10.8	-4.6
5	0.40	5.4	14.2	276	1.5	3.9

6	-0.29	-11.5	-2.7	276	-3.2	-0.7
1E	-0.48	-16.1	-7.3	78	-1.3	-0.6
2E	-1.07	-30.6	-21.8	72	-2.2	-1.6
3E	-0.53	-17.4	-8.6	72	-1.3	-0.6
4E	-0.48	-16.1	-7.3	78	-1.3	-0.6
5E	0.61	10.5	19.3	39	0.4	0.8
6E	-0.43	-14.9	-6.1	39	-0.6	-0.2

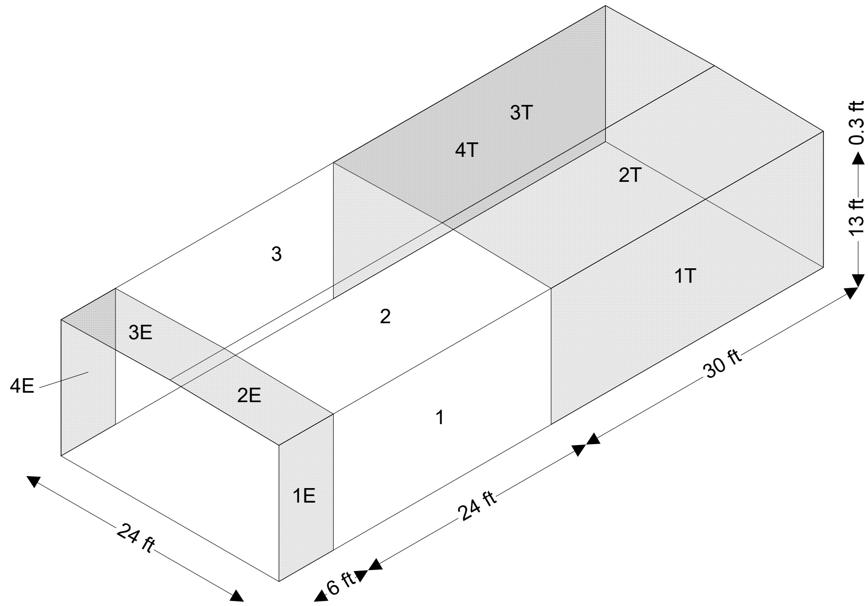


Loadcase B

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	5.4	14.2	312	1.7	4.4
2	-0.69	-21.3	-12.5	288	-6.1	-3.6
3	-0.37	-13.5	-4.6	288	-3.9	-1.3
4	-0.29	-11.5	-2.7	312	-3.6	-0.8
1E	0.61	10.5	19.3	78	0.8	1.5
2E	-1.07	-30.6	-21.8	72	-2.2	-1.6
3E	-0.53	-17.4	-8.6	72	-1.3	-0.6
4E	-0.43	-14.9	-6.1	78	-1.2	-0.5
1T	-	1.3	3.5	390	0.5	1.4
2T	-	-5.3	-3.1	360	-1.9	-1.1
3T	-	-3.4	-1.2	360	-1.2	-0.4

4T	-	-2.9	-0.7	390	-1.1	-0.3



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	-15.4	-6.6	702	-10.8	-4.6
2	-0.69	-21.3	-12.5	648	-13.8	-8.1
3	-0.37	-13.5	-4.6	648	-8.7	-3.0
4	-0.45	-15.4	-6.6	702	-10.8	-4.6
5	0.40	5.4	14.2	138	0.7	2.0
6	-0.29	-11.5	-2.7	138	-1.6	-0.4
1E	-0.48	-16.1	-7.3	78	-1.3	-0.6
2E	-1.07	-30.6	-21.8	72	-2.2	-1.6
3E	-0.53	-17.4	-8.6	72	-1.3	-0.6
4E	-0.48	-16.1	-7.3	78	-1.3	-0.6
5E	0.61	10.5	19.3	20	0.2	0.4
6E	-0.43	-14.9	-6.1	39	-0.6	-0.2
5T	-	1.3	3.5	158	0.2	0.6
6T	-	-2.9	-0.7	158	-0.5	-0.1

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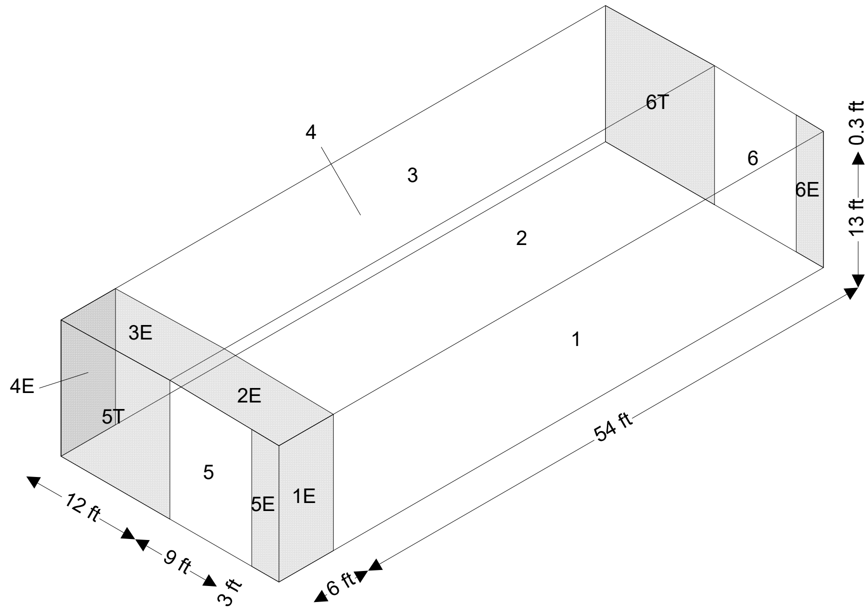

Loadcase BT

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WIND LOADING COMPONENT AND CLADDING (ASCE7-10)

V=Wind

_wld.Exposure = Wind_exp

_wld.Risk = Risk_category

b = bl

d = bw

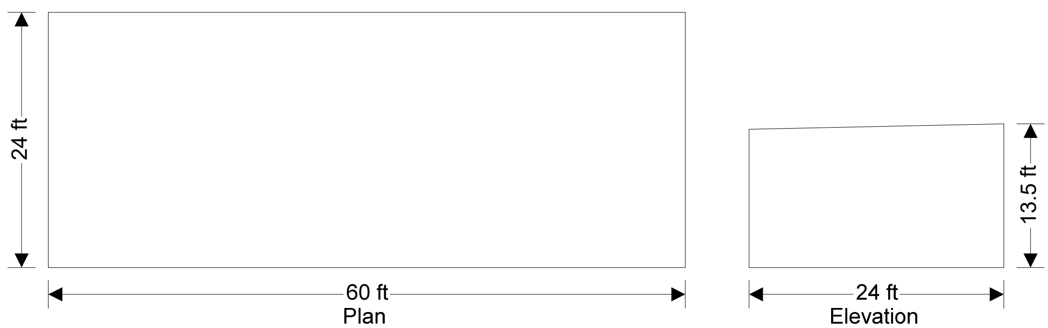
h = h;

WIND LOADING

In accordance with ASCE7-10

Using the components and cladding design method

Tedds calculation version 2.1.05



Building data

Type of roof;	Monoslope
Length of building;	b = **60.00** ft
Width of building;	d = **24.00** ft
Height to eaves;	H = **13.00** ft
Pitch of roof;	α_0 = **1.2** deg
Mean height;	h = **13.00** ft

General wind load requirements

Basic wind speed;	V = **115.0** mph
Risk category;	II
Velocity pressure exponent coef (Table 26.6-1);	K_d = **0.85**
Exposure category (cl 26.7.3);	C
Enclosure classification (cl.26.10);	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1);	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1);	GC_{pi_n} = **-0.18**
Gust effect factor;	G_f = **0.85**

Topography

Topography factor not significant;	K_{zt} = 1.0

Velocity pressure

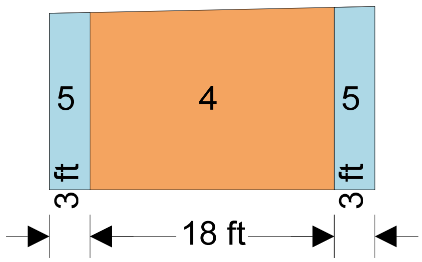
Velocity pressure coefficient (T.30.3-1);	K_z = **0.85**
Velocity pressure;	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$ = **24.5** psf

Peak velocity pressure for internal pressure

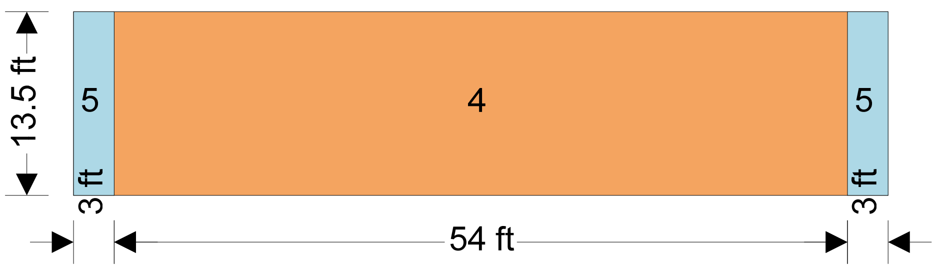
Peak velocity pressure – internal (as roof press.); q_i = **24.46** psf

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 30.4-2A)

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	-1.00	11.7 #	-28.9
25 sf	1	-	-	25.0	0.26	-0.96	10.8 #	-27.9
50 sf	1	-	-	50.0	0.23	-0.93	10.0 #	-27.2
>100 sf	1	-	-	100.0	0.20	-0.90	9.3 #	-26.4
<10 sf	2	-	-	10.0	0.30	-1.80	11.7 #	-48.4
25 sf	2	-	-	25.0	0.26	-1.52	10.8 #	-41.6
50 sf	2	-	-	50.0	0.23	-1.31	10.0 #	-36.5
>100 sf	2	-	-	100.0	0.20	-1.10	9.3 #	-31.3
<10 sf	3	-	-	10.0	0.30	-2.80	11.7 #	-72.9
25 sf	3	-	-	25.0	0.26	-2.12	10.8 #	-56.3
50 sf	3	-	-	50.0	0.23	-1.61	10.0 #	-43.8
>100 sf	3	-	-	100.0	0.20	-1.10	9.3 #	-31.3

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

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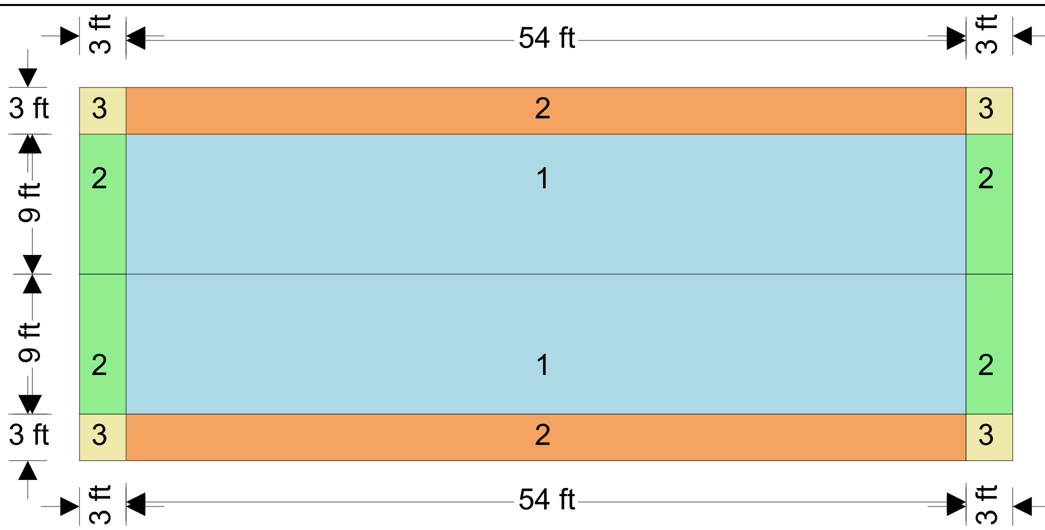

Plan on roof

; Calculation of 2a:

Minimum dimension; Dim_min = 3 ft;

a = max (min (.4 × h, .1× min (B, L)), Dim_min) = **3.000** ft
End_Zone = 2 × a = **6.000** ft

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 (ASCE 7-10)

Tedds calculation version 3.1.00

Site parameters

Site class; D

Mapped acceleration parameters (Section 11.4.1)

at short period; S_S = **0.271**

at 1 sec period; S_1 = **0.075**

Site coefficientat short period (Table 11.4-1); F_a = **1.583**

at 1 sec period (Table 11.4-2); F_v = **2.400**

Spectral response acceleration parameters

at short period (Eq. 11.4-1); $S_{MS} = F_a \times S_S$ = **0.429**

at 1 sec period (Eq. 11.4-2); $S_{M1} = F_v \times S_1$ = **0.180**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3); $S_{DS} = 2 / 3 \times S_{MS}$ = **0.286**

at 1 sec period (Eq. 11.4-4); $S_{D1} = 2 / 3 \times S_{M1}$ = **0.120**

Seismic design category

Risk category (Table 1.5-1); II

Seismic design category based on short period response acceleration (Table 11.6-1)

B

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

B

Seismic design category; B

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); $T_a = C_t \times (h_n)^x \times 1sec / (1ft)^x$= **0.137** sec

Building fundamental period (Sect 12.8.2); $T = T_a =$ **0.137** sec

Long-period transition period; $T_L =$ **12** 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 12.8.1.1)

Calculated (Eq 12.8-3); $C_{s\_calc} = S_{DS} / (R / I_e) =$ **0.0440**

Maximum (Eq 12.8-3); $C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e)) =$ **0.1348**

Minimum (Eq 12.8-5); $C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) =$ **0.0126**

Seismic response coefficient; $C_s =$ **0.0440**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure; $W =$ **58.0** kips

Seismic response coefficient; $C_s =$ **0.0440**

Seismic base shear (Eq 12.8-1); $V = C_s \times W =$ **2.6** kips

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SNOW LOADING (ASCE7)

b = bl

p_g = Pg

C_e = Ce

C_t = Ct

I_s = Is

_snl.RoofExp = Roof_Exposure

_snl.ImportanceCat = Risk_category

_snl.TerrainCat = Exposure

SNOW LOADING

In accordance with ASCE7-10

Tedds calculation version 1.0.09

Building details

Roof type;	Monopitch
Width of roof;	b = **60.00** ft
Slope of roof 1;	α = **1.20** deg

Ground snow load

Ground snow load (Figure 7-1);	p_g = **87.00** lb/ft^2
Density of snow;	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **25.31** lb/ft^3
Terrain typeSect. 26.7;	C
Exposure condition (Table 7-2);	Partially exposed
Exposure factor (Table 7-2);	C_e = **1.00**
Thermal condition (Table 7-3);	Structures kept just above freezing
Thermal factor (Table 7-3);	C_t = **1.10**
Importance category (Table 1.5-1);	II
Importance factor (Table 1.5-2);	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4);	$p_{f\_min} = I_s \times 20$ lb/ft^2 = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3);	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **66.99** lb/ft^2

Cold roof slope factor (C_t > 1.0)

Roof surface type;	Non slippery
Ventilation;	Ventilated
Thermal resistance (R-value);	R = **30.00**;°F h ft^2 / Btu
Roof slope factor Fig 7-2b (solid line);	C_s = **1.00**

Monoslope

Sloped roof snow load (Cl.7.4);	$p_s = \max(C_s \times p_f, p_{f\_min})$ = **66.99** lb/ft^2

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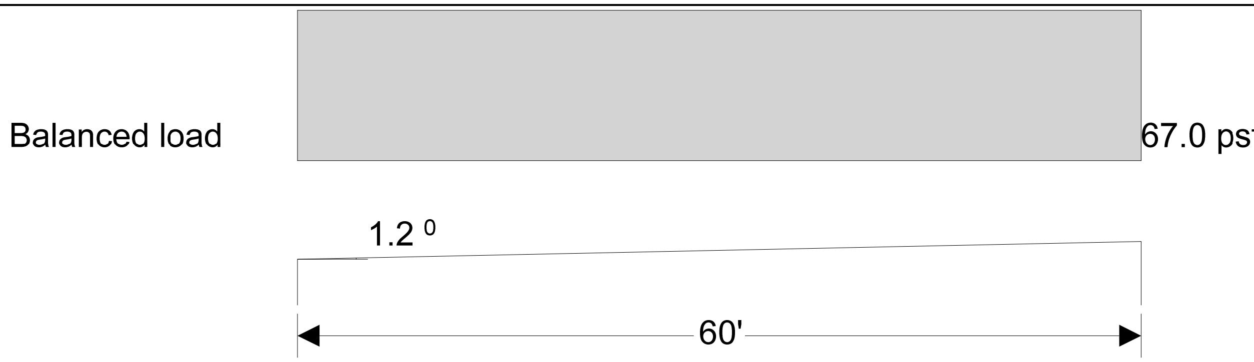

Roof elevation

CHECK WIND AND SEISMIC

Pseismic_factored= V= **2.552** kips

Pwind_length = P = (F_{wi_1_A_n} + F_{wi_1E_A_n} + abs(F_{wi_4_A_n}) + abs(F_{wi_4E_A_n})) × .6= **8.300** kips

Pwind_width = (F_{wi_5_B_n} + abs(F_{wi_6_B_n}) + abs(F_{wi_6E_B_n}) + F_{wi_5E_B_n}) ×.6 = **3.391** kips

Pseismic = Pseismic_factored/ 1.4 = **1.823** kips

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

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

ANCHOR

Minute Man Anchor Capacity; Anc_cap = 1800 lbs

Anc_all = Anc_cap/√(2) = **1272.792** lbs

CASE A

Sp = Sp = **6.400** ft

Wunit = Wunit = **11.750** ft

L_caseB =bw= **24.000** ft

From Roof Load

L_caseA =bl= **60.000** ft

h_roof = 5 ft;

r = h_roof/h= **0.385**

Proof_caseA = Pwind_length × r = **3.192** kips

WcaseA = Proof_caseA/2 = **1.596** kips

N = WcaseA /Anc_all = **1.254**

From floor load

h_floor = 8 ft;

r = h_floor/h= **0.615**

Pfloor_caseA = Pwind_length × r = **5.107** kips

Roof uplift:

Wup = abs(GC_{pi_2_B_p})×.6 = **12.769** psf

Pup = Wup × L_caseA × L_caseB/2 – (Rdl × L_caseA × L_caseB/2 + Fdl ×L_caseA ×L_caseB/2) ×.6 = **0.553** kips

N= (Pfloor_caseA + Pup) /Anc_all =**4.448**

Wfloor_caseA = (Pfloor_caseA + Pup)/L_caseA = **94.348** plf

S_caseA = Anc_all / Wfloor_caseA = **13.490** ft

CASE B

From Roof Load

L_caseB =bw= **24.000** ft

h_roof = 5 ft;

r = h_roof/h= **0.385**

Proof_caseB = Pwind_width× r = **1.304** kips

WcaseB = Proof_caseB/2 = **0.652** kips

Pfloor_caseB = Pwind_width × r = **1.304** kips

Wfloor = Pfloor_caseB/ L_caseB= **54.342** plf

N = (WcaseB+Wfloor ×Wunit/2) /Anc_all = **0.763**

From floor load

h_floor = 8 ft;

r = h_floor/h= **0.615**

Roof uplift:

Wup = abs(GC_{pi_2_B_p})×.6= **12.769** psf

Pup = Wup × L_caseA/2 × L_caseB – (Rdl × L_caseB × L_caseA/2 + Fdl ×L_caseB ×L_caseA/2) ×.6 = **0.553** kips

N= (Pfloor_caseB + Pup) /Anc_all =**1.459**

Wfloor_caseB= (Pfloor_caseB + Pup)/L_caseB = **77.401** plf

Pcorner anchor perpandicular to main i beam

Wup = abs(GC_{pi_2E_A_p}) ×.6 = **18.346** psf

Anchor spacing on long side; a_sp = 5 ft;

Plateral = WcaseA + Wup × a_sp × h_floor = **2.330** kips

If(Plateral<Anc_all,"OK","NG") = **"NG"**

Use 2 anchors

If(Plateral<Anc_all×2,"OK","NG") = **"OK"**

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OVERTURNING LOAD:

Roof

Case A

Pover_caseA = WcaseA = **1.596** kips

h= h = **13.000** ft

Proof_caseA = Pover_caseA × h/L_caseB = **0.865** kips

Case B

Pover_caseB = WcaseB = **0.652** kips

Proof_caseB = Pover_caseB × h/L_caseA = **0.141** kips

Floor

Case A

hfloor = 3 ft;

Wunit = Wunit = **11.750** ft

Sp = Sp = **6.400** ft

Pfloor_caseA = Sp × Wfloor_caseA×hfloor/L_caseB = **0.075** kips

Case B

Pfloor_caseB = Wunit × Wfloor_caseB×hfloor/L_caseA = **0.045** kips

Down from overturning force

P_corner_caseA = Pfloor_caseA +Proof_caseA = **0.940** kips

P_corner_caseB = Pfloor_caseB +Proof_caseB = **0.187** kips

P_corner_down = max(P_corner_caseA, P_corner_caseB) = **0.940** kips

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GRAVITY LOAD

FOUNDATION WITH ABS PADS

Roof Dead Load; Rdl = Rdl = **10.000** psf;
Roof Live Load ; Rll = Rll = **20.000** psf
Floor Dead Load; Fdl = Fdl = **10.000** psf;
Floor Live Load; Fll = Fll = **50.000** psf;
Pier Spacing; Sp = Sp = **6.400** ft;
Unit width; Wunit = Wunit = **11.750** ft;
Roof trib at int. column; Ltrib_int = Ltrib_int= **30.000** ft;
Roof trib at ext. column; Ltrib_ext = Ltrib_ext= **15.000** ft;
Wall Dead Load; Wdl = Wdl = **60.000** plf;

Load Exterior pier:
DL_ext_pier = (Rdl + Fdl) × Sp × Wunit/2 + Wdl × Sp = **1.136** kips
LL_ext_pier = (max(Rll, p_{f}) +Fll) × Sp × Wunit/2 = **4.399** kips
Ptotal_ext = DL_ext_pier + LL_ext_pier + Pfloor_caseA = **5.610** kips

Load Interior pier without roof load:
DL_int_pier = (Fdl) × Sp × Wunit/2 = **0.376** kips
LL_int_pier = (Fll) × Sp × Wunit/2 = **1.880** kips
Ptotal_int = DL_int_pier + LL_int_pier = **2.256** kips

Corner Pier
DL_cor = (Rdl + Fdl) × Sp/2 × Wunit/2 + Wdl × Sp = **0.760** kips
LL_cor = (max(Rll, p_{f}) +Fll) × Sp/2 × Wunit/2 = **2.199** kips
Ptotal_cor = DL_cor + LL_cor + P_corner_down = **3.899** kips

End Pier
DL_end = Fdl × Sp/2 × Wunit + Wdl × Sp = **0.760** kips
LL_end = Fll × Sp/2 × Wunit = **1.880** kips
Ptotal_end = DL_end + LL_end + Pfloor_caseB = **2.685** kips

Int column Use 2 piers
DL_col_int = Rdl × Ltrib_int× Wunit = **3.525** kips
LL_col_int = (max(Rll, p_{f})) × Ltrib_int× Wunit = **23.614** kips
Ptotal_col_int = (DL_col_int + LL_col_int)/2 = **13.569** kips

Int. column uplift
Puplift = abs(p_{cc_n_r4})×.6 = **15.851** psf
Pup = Puplift × Ltrib_int× Wunit = **5.587** kips
DL = (Rdl +Fdl) × Ltrib_int× Wunit × .6 = **4.230** kips

Pdown_dl = DL = **4.230** kips

Pnet = Pup – Pdown_dl = **1.357** kips

If(Anc_all >Pnet,"OK", "NG") = **"NG"**

Ext column use 2 piers

DL_col_ext = Rdl × Ltrib_ext× Wunit = **1.763** kips

LL_col_ext = (max(Rll, p_{f})) × Ltrib_ext× Wunit = **11.807** kips

Ptotal_col_ext = (DL_col_ext + LL_col_ext)/2 = **6.785** kips

Int. column uplift

Puplift = abs(p_{cc_n_r4})×.6 = **15.851** psf

Pup = Puplift × Ltrib_ext× Wunit = **2.794** kips

DL = (Rdl +Fdl) × Ltrib_ext× Wunit × .6 = **2.115** kips

Pdown_dl = DL = **2.115** kips

Pnet = Pup – Pdown_dl = **0.679** kips

If(Anc_all >Pnet,"OK", "NG") = **"OK"**

ABS BEARING PAD

Lpad_20 = 20 in;

Wpad_20 = 20 in;

Wpad_24 = 24 in;

Lpad_24 = 24 in;

Lpad_40 = 40 in;

Lpad_48 = 48 in;

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

ABS pad 20" x 20"

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

P = max(Ptotal_int, Ptotal_cor, Ptotal_ext,Ptotal_end, Ptotal_col_ext) = **6.785** kips

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

(3) ABS pad 24" x 24"

Pmax = Lpad_48 × Wpad_24 × Sall = **20.000** kips

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

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FOUNDATION WITH CONCRETE PIERS

Load Exterior pier:

DL_ext_pier = (Rdl + Fdl) × Sp × Wunit/2 + Wdl × Sp = **1.136** kips

LL_ext_pier = (max(Rll, p_{f}) +Fll) × Sp × Wunit/2 = **4.399** kips

Ptotal_ext = DL_ext_pier + LL_ext_pier + Pfloor_caseA = **5.610** kips

Load Interior pier without roof load:

DL_int_pier = (Fdl) × Sp × Wunit/2 = **0.376** kips

LL_int_pier = (Fll) × Sp × Wunit/2 = **1.880** kips

Ptotal_int = DL_int_pier + LL_int_pier = **2.256** kips

Corner Pier

DL_cor = (Rdl + Fdl) × Sp/2 × Wunit/2 + Wdl × Sp = **0.760** kips

LL_cor = (max(Rll, p_{f}) +Fll) × Sp/2 × Wunit/2 = **2.199** kips

Ptotal_cor = DL_cor + LL_cor + P_corner_down = **3.899** kips

End Pier

DL_end = Fdl × Sp/2 × Wunit + Wdl × Sp = **0.760** kips

LL_end = Fll × Sp/2 × Wunit = **1.880** kips

Ptotal_end = DL_end + LL_end + Pfloor_caseB = **2.685** kips

Int column

DL_col_int = Rdl × Ltrib_int× Wunit = **3.525** kips

LL_col_int = (max(Rll, p_{f})) × Ltrib_int× Wunit = **23.614** kips

Ptotal_col_int = DL_col_int + LL_col_int = **27.139** kips

Int. column uplift

Puplift = abs(p_{cc_n_r4})×.6 = **15.851** psf

Pup = Puplift × Ltrib_int× Wunit = **5.587** kips

DL = (Rdl +Fdl) × Ltrib_int× Wunit × .6 = **4.230** kips

Pdown_dl = DL = **4.230** kips

Pnet = Pup – Pdown_dl = **1.357** kips

If(Anc_all >Pnet,"OK", "NG") = **"NG"**

Ext column

DL_col_ext = Rdl × Ltrib_ext× Wunit = **1.763** kips

LL_col_ext = (max(Rll, p_{f})) × Ltrib_ext× Wunit = **11.807** kips

Ptotal_col_ext = DL_col_ext + LL_col_ext= **13.569** kips

Int. column uplift

Puplift = abs(p_{cc_n_r4})×.6 = **15.851** psf

Pup = Puplift × Ltrib_ext× Wunit = **2.794** kips

DL = (Rdl +Fdl) × Ltrib_ext× Wunit × .6 = **2.115** kips

Pdown_dl = DL = **2.115** kips

Pnet = Pup – Pdown_dl = **0.679** kips

If(Anc_all >Pnet,"OK", "NG") = **"OK"**

PIER CAPACITY

Bearing Capacity; Bear = 2500 psf

24" pier

Pier Diameter; d = 24 in;

Pier area; $A = \pi \times (d/2)^2$ = **452.389** in^2;

Pcap_24 = Bear × A = **7.854** kips

Steel reinforcement. Use ½ percent of the pier area

Asteel = .5/100 × A = **2.262** in^2

#5 steel area; A#5 = .31 in^2

#6 steel area; A#6 = .44 in^2

#7 steel area; A#7 = .60 in^2

Number of vertical rebar; N#5 = Asteel/A#5 =**7.297** ; Ceiling (N#5,1) = **8**

Number of vertical rebar; N#6 = Asteel/A#6 =**5.141** ; Ceiling (N#6,1) = **6**

Number of vertical rebar; N#7 = Asteel/A#7 =**3.770** ; Ceiling (N#7,1) = **4**

42" pier

Pier Diameter; d = 42 in;

Pier area; $A = \pi \times (d/2)^2$ = **1385.442** in^2;

Pcap_42 = Bear × A = **24.053** kips

Steel reinforcement. Use ½ percent of the pier area

Asteel = .5/100 × A = **6.927** in^2

#5 steel area; A#5 = .31 in^2

#6 steel area; A#6 = .44 in^2

#7 steel area; A#7 = .60 in^2

Number of vertical rebar; N#5 = Asteel/A#5 =**22.346** ; Ceiling (N#5,1) = **23**

Number of vertical rebar; N#6 = Asteel/A#6 =**15.744** ; Ceiling (N#6,1) = **16**

Number of vertical rebar; N#7 = Asteel/A#7 =**11.545** ; Ceiling (N#7,1) = **12**

48" pier

Pier Diameter; d = 48 in;

Pier area; $A = \pi \times (d/2)^2$ = **1809.557** in^2;

Pcap_48 = Bear × A = **31.416** kips

Steel reinforcement. Use ½ percent of the pier area

Asteel = .5/100 × A = **9.048** in^2

#5 steel area; A#5 = .31 in^2

#6 steel area; A#6 = .44 in^2

#7 steel area; A#7 = .60 in^2

Number of vertical rebar; N#5 = Asteel/A#5 =**29.186** ; Ceiling (N#5,1) = **30**

Number of vertical rebar; N#6 = Asteel/A#6 =**20.563** ; Ceiling (N#6,1) = **21**

Number of vertical rebar; N#7 = Asteel/A#7 =**15.080** ; Ceiling (N#7,1) = **16**

RC BEAM ANALYSIS AND DESIGN (ACI318)

RC BEAM ANALYSIS & DESIGN (ACI318-2011)

In accordance with ACI318

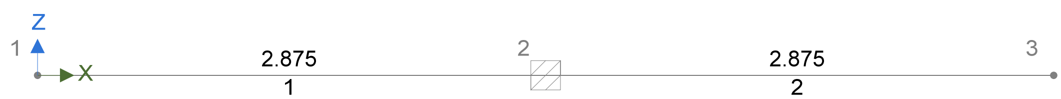
Tedds calculation version 3.3.00

ANALYSIS

Tedds calculation version 1.0.28

Geometry

Geometry (ft) - Concrete (4000 150) - R 24x10

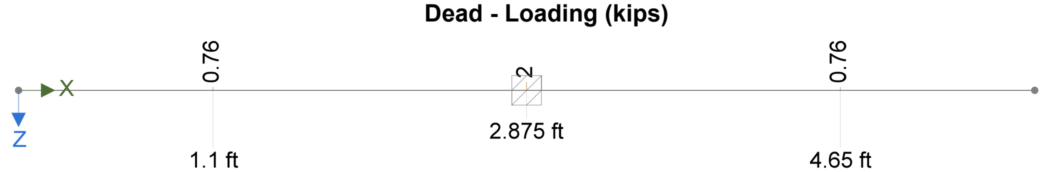


Span	Length (ft)	Section	Start Support	End Support
1	2.88	R 24x10	Free	Fixed
2	2.88	R 24x10	Fixed	Free
R 24x10: Area 240 in^2, Inertia Major 2000 in^4, Inertia Minor 11520 in^4, Shear area parallel to Minor 200 in^2, Shear area parallel to Major 200 in^2				
Concrete (4000 150): Density 150 lbm/ft^3, Youngs 3834 ksi, Shear 1750 ksi, Thermal 0.00001 °C^{-1}				

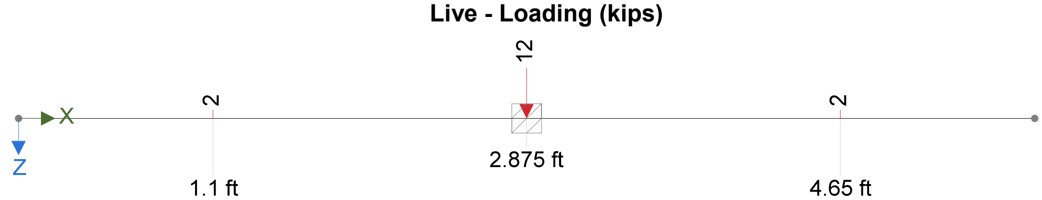
Loading

Self weight included

Dead - Loading (kips)



Live - Loading (kips)



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Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	*1.20*	*1.20*	*1.60*
1.0D + 1.0L (Service)	*1.00*	*1.00*	*1.00*

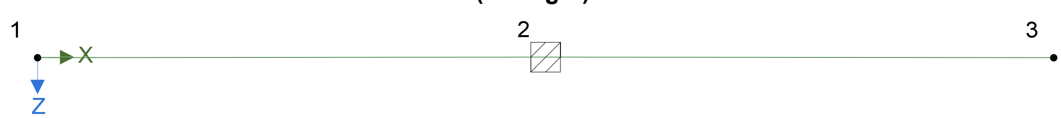
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	Point load	GlobalZ	0.76 kips at 1.1 ft
Beam	Dead	Point load	GlobalZ	2 kips at 2.875 ft
Beam	Dead	Point load	GlobalZ	0.76 kips at 4.65 ft
Beam	Live	Point load	GlobalZ	2 kips at 1.1 ft
Beam	Live	Point load	GlobalZ	12 kips at 2.875 ft
Beam	Live	Point load	GlobalZ	2 kips at 4.65 ft

Results

Total deflection

1.2D + 1.6L (Strength) - Total deflection

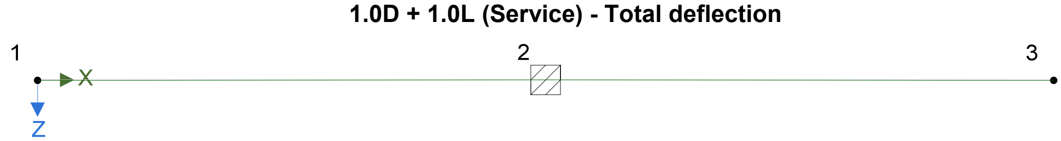


Member results

Load combination: 1.2D + 1.6L (Strength)

Member	Position (ft)	Deflection (in)		Axial deflection (in)	
Beam	0	0 (max)		0	
	2.88	0		0	
	5.75	0 (max)		0	

1.0D + 1.0L (Service) - Total deflection



Member results

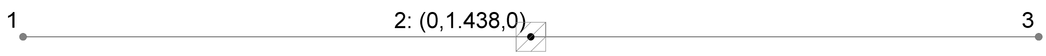
Load combination: 1.0D + 1.0L (Service)

Member	Position (ft)	Deflection (in)		Axial deflection (in)	
Beam	0	0 (max)		0	
	2.88	0		0	

Member	Position (ft)	Deflection (in)		Axial deflection (in)	
	5.75	0 (max)		0	

Reactions

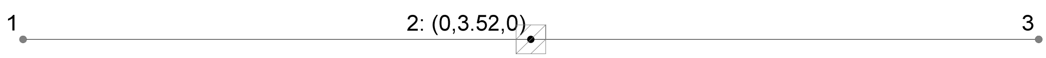
Self Weight - Local node reactions - Node: (Horiz (kips), Vert (kips), Mom (kips_ft))



Load case: Self Weight

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	1.438	0

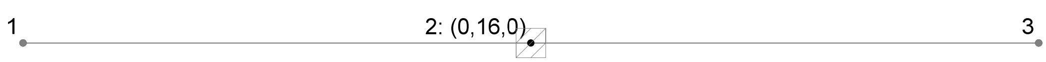
Dead - Local node reactions - Node: (Horiz (kips), Vert (kips), Mom (kips_ft))



Load case: Dead

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	3.52	0

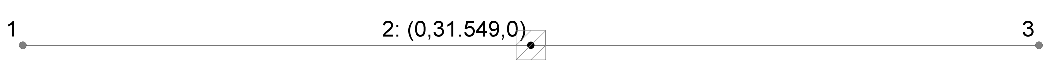
Live - Local node reactions - Node: (Horiz (kips), Vert (kips), Mom (kips_ft))



Load case: Live

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	16	0

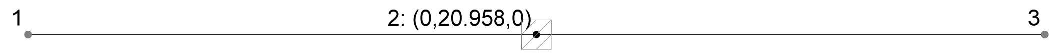
1.2D + 1.6L (Strength) - Local node reactions - Node: (Horiz (kips), Vert (kips), Mom (kips_ft))



Load combination: 1.2D + 1.6L (Strength)

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	31.549	0

1.0D + 1.0L (Service) - Local node reactions - Node: (Horiz (kips), Vert (kips), Mom (kips_ft))

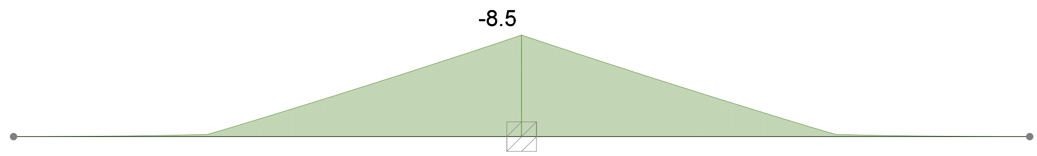


Load combination: 1.0D + 1.0L (Service)

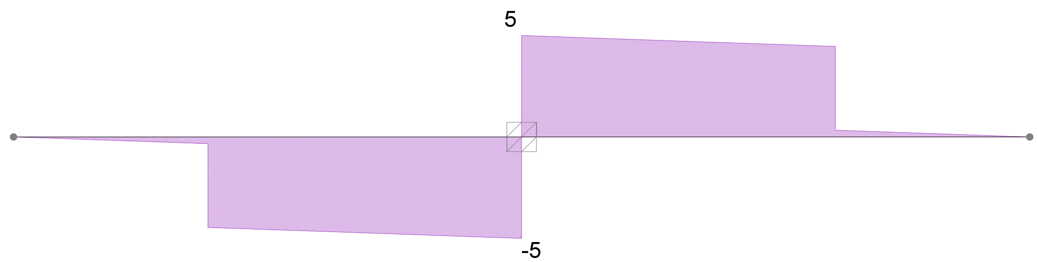
Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	20.958	0

Forces

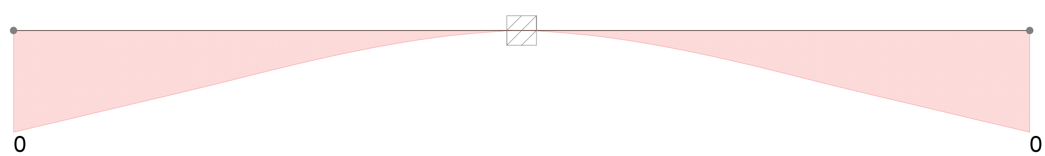
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)



Member results

Envelope - Strength combinations

Member	Position (ft)	Shear force (kips)		Moment (kip_ft)	
Beam	0	0		0 (max)	
	2.88	4.975	-4.974 (max abs)	-8.539 (min)	

Envelope - Service combinations

Member	Position (ft)	Deflection (in)	
Beam	2.88	0	
	5.75	0 (max)	

;

Concrete details

Concrete details

Compressive strength of concrete; f'_c = **4000** psi

Density of reinforced concrete; w_c = **150** lb / ft^3

Concrete type; Normal weight

Modulus of elasticity of concrete (cl.8.5.1); $E = (w_c / 1\ lb/ft^3)^{1.5} \times 33\ psi \times (f'_c / 1\ psi)^{0.5}$ = **3834254** psi

Strength reduction factor for shear; ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement; f_y = **60000** psi

Nominal cover to reinforcement

Cover to top reinforcement; c_{nom_t} = **1.5** in

Cover to bottom reinforcement; c_{nom_b} = **3** in

Cover to side reinforcement; c_{nom_s} = **3** in

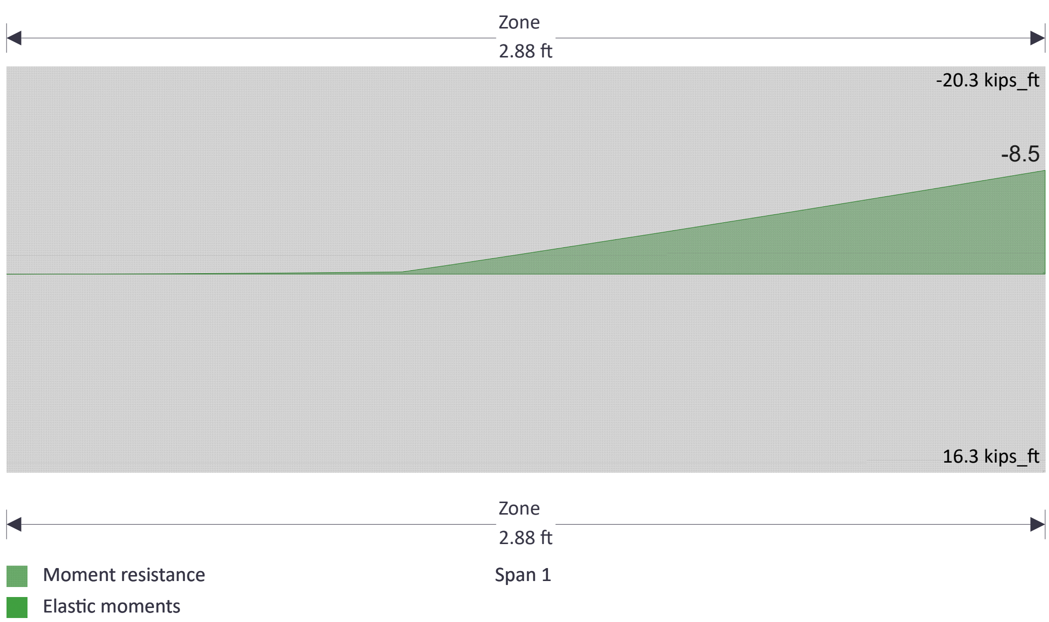
Beam - Span 1

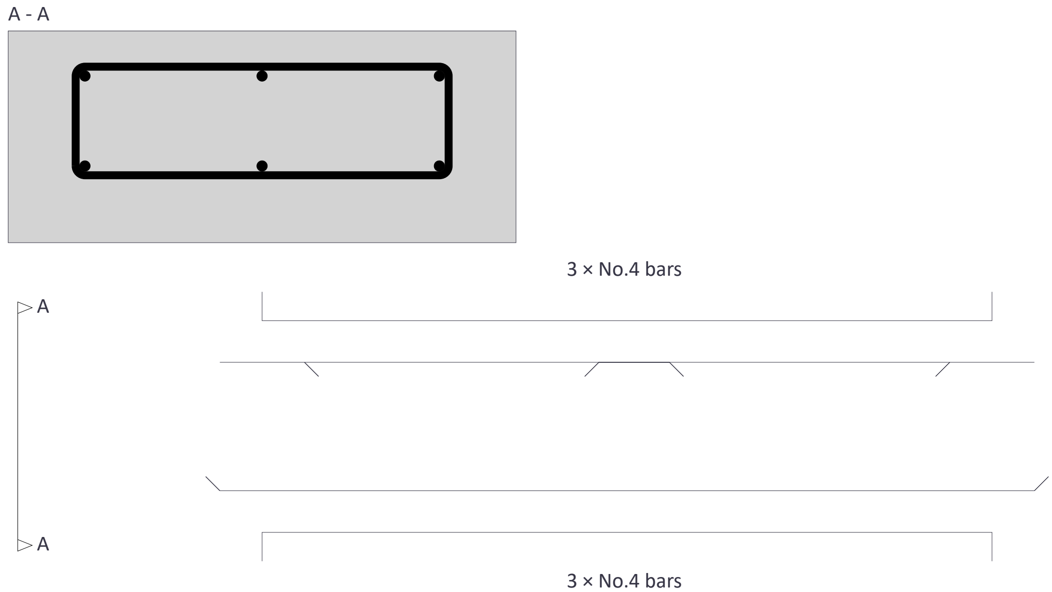
Rectangular section details

Section width; b = **24** in

Section depth; h = **10** in

Moment design





Zone 1 - Negative moment. Rectangular section in flexure (Chapter 10)

Factored bending moment at section; $M_u = abs(M_{m1\_s1\_z2\_min\_red})$ = **8.539** kip_ft

Effective depth to tension reinforcement; d = **7.875** in

Tension reinforcement provided; 3 × No.4 bars

Area of tension reinforcement provided; $A_{s,prov}$ = **0.589** in^2

Minimum area of reinforcement (eqn. 10-3); $A_{s,min} = \min(\max(3\ psi \times \sqrt{(f'_c / 1\ psi)}, 200\ psi) \times b \times d / f_y, A_{s,req})$ = **0.324** in^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.10.2.7.3); $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\ ksi) / 1\ ksi, 0.65), 0.85)$ = **0.85**

Depth of equivalent rectangular stress block; $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **0.433** in

Depth to neutral axis; $c = a / \beta_1$ = **0.51** in

Net tensile strain in extreme tension fibers; $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001\ in)$ = **0.04336**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.9.3.2); $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.90**

Nominal moment strength; $M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **22.556** kip_ft

Design moment strength; $\phi M_n = M_n \times \phi_f$ = **20.300** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.; $s_{t,max} = s_{top} + \phi_{m1\_s1\_z2\_t\_L1}$ = **8.375** in

Service load stress in reinforcement (cl.10.6.4); $f_s = 2/3 \times f_y$ = **40000** psi

Clear cover of reinforcement; $c_c = c_{nom\_t} + \phi_v$ = **1.875** in

Maximum allowable top bar spacing (eqn. 10-4); $s_{max} = \min(15in \times 40000psi / f_s - 2.5 \times c_c, 12in \times 40000psi / f_s)$ = **10.313** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing; $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_t\_L1} \times N_{m1\_s1\_z2\_t\_L1})) / (N_{m1\_s1\_z2\_t\_L1} - 1)$ = **7.875** in

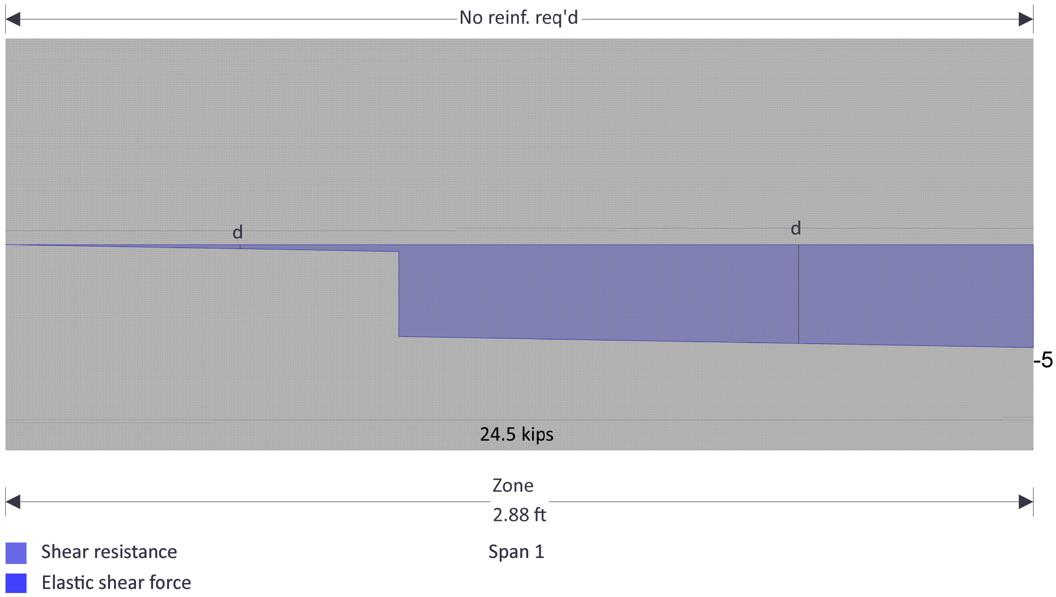
Min. allowable top bar clear spacing (cl.7.6.1); $s_{top,min}$ = **1.000** in

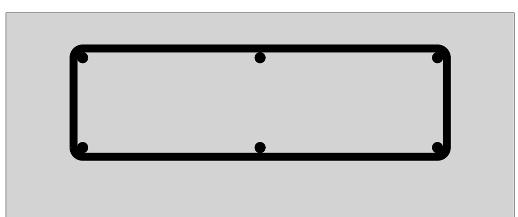
Bottom bar clear spacing; $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1})) / (N_{m1\_s1\_z2\_b\_L1} - 1)$ = **7.875** in

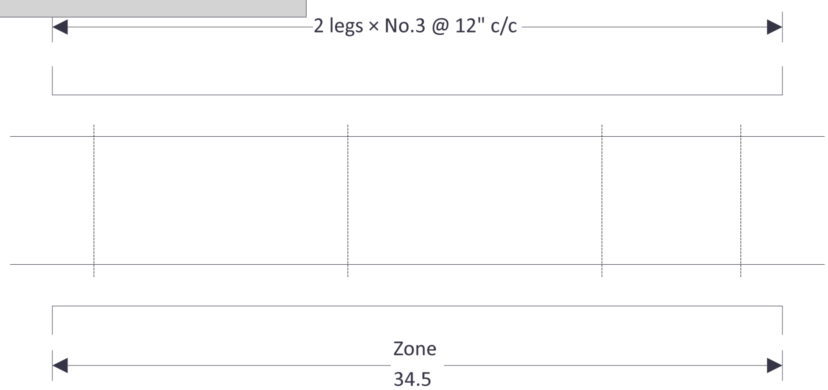
Min. allowable bottom bar clear spacing (cl.7.6.1); $s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Shear design







Rectangular section in shear

Concrete weight modification factor; λ = **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c); N/A no reinf. required along full length

Location where no reinf. is req'd (V_u less ϕV_c / 2); Between 0.00 ft and 2.88 ft

Maximum reinforcement shear strength; $\phi V_{s,max} = \phi_s \times 8\text{ psi} \times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **71.720** kips

Minimum area of shear reinf. (eqn. 11-13); $A_{sv,min}$ = max(50 psi, 0.75 psi × √(f'c / 1 psi)) × b / min(f_y, 60000 psi) = **0.240** in²/ft

Zone 1

Effective depth of long. reinf. used for shear zone; d = **7.875** in

Concrete shear strength (eqn. 11-3); $\phi V_c = \phi_s \times \lambda \times 2$ psi × √(f'c / 1 psi) × b × d = **17.930** kips

Design shear force within zone; V_u = **4.778** kips

Reinf. shear strength required (eqn. 11-2); ϕV_s = max(V_u - ϕV_c, 0 kips) = **0.000** kips

Area of design shear reinf. req'd (eqn. 11-15); $A_{sv,des} = \phi V_s / (\phi_s \times$ min(f_y, 60000 psi) × d) = **0.000** in²/ft

Area of shear reinforcement required; $A_{sv,req}$ = 0in²/ft = **0.000** in²/ft

Shear reinforcement provided; 2 legs × No.3 @ 12" c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **0.221** in²/ft

PASS - No shear reinforcement required ($\phi V_c/2 >= V_u$)

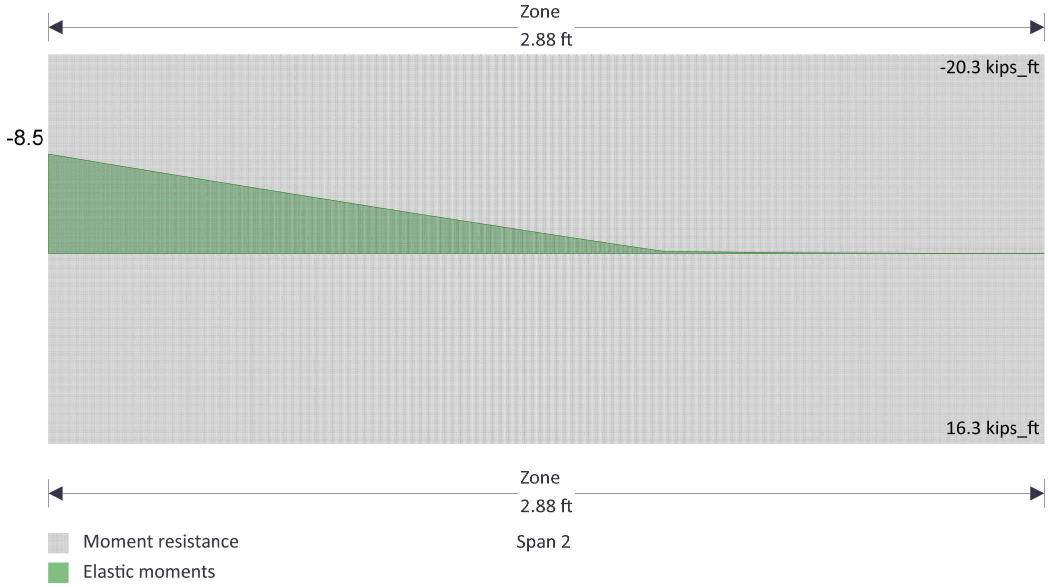
Beam - Span 2

Rectangular section details

Section width; b = **24** in

Section depth; h = **10** in

Moment design



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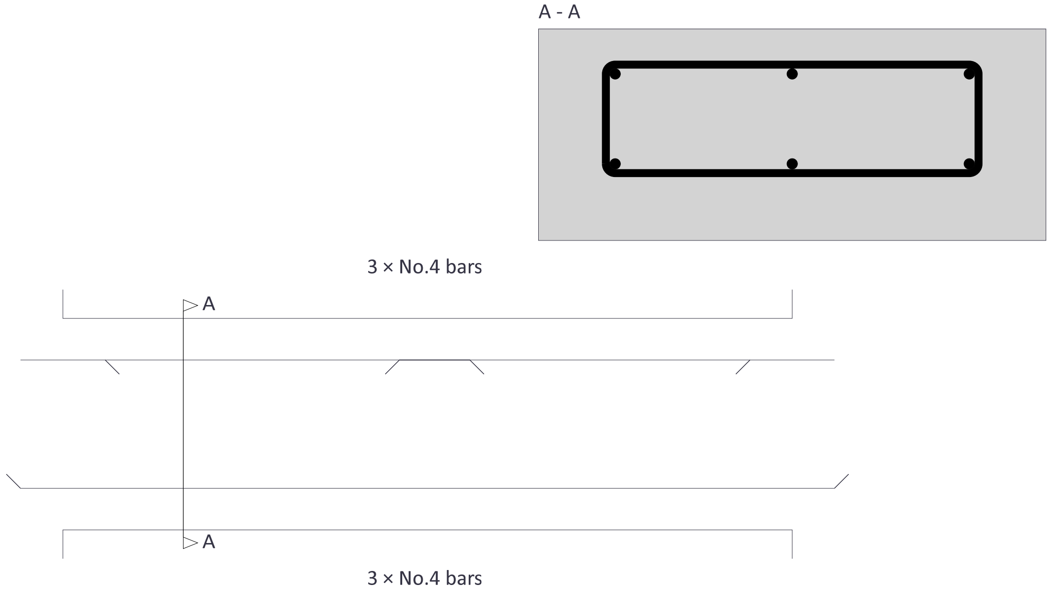


Zone 1 - Negative moment. Rectangular section in flexure (Chapter 10)

Factored bending moment at section; M_u = abs($M_{m1\_s2\_z2\_min\_red}$) = **8.539** kip_ft

Effective depth to tension reinforcement; d = **7.875** in

Tension reinforcement provided; 3 × No.4 bars

Area of tension reinforcement provided; $A_{s,prov}$ = **0.589** in^2

Minimum area of reinforcement (eqn. 10-3); $A_{s,min}$ = min(max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f_y, $A_{s,req}$) = **0.324** in^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.10.2.7.3); β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth of equivalent rectangular stress block; a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **0.433** in

Depth to neutral axis; c = a / β_1 = **0.51** in

Net tensile strain in extreme tension fibers; ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.04336**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.9.3.2); ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.90**

Nominal moment strength; M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **22.556** kip_ft

Design moment strength; ϕM_n = M_n × ϕ_f = **20.300** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.; $s_{t,max}$ = s_{top} + $\phi_{m1\_s2\_z2\_t\_L1}$ = **8.375** in

Service load stress in reinforcement (cl.10.6.4); f_s = 2/3 × f_y = **40000** psi

Clear cover of reinforcement; c_c = c_{nom_t} + ϕ_v = **1.875** in

Maximum allowable top bar spacing (eqn. 10-4); s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.313** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing; s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s2\_z2\_v}$) + $\phi_{m1\_s2\_z2\_t\_L1}$ × $N_{m1\_s2\_z2\_t\_L1}$)) / ($N_{m1\_s2\_z2\_t\_L1}$ - 1) = **7.875** in

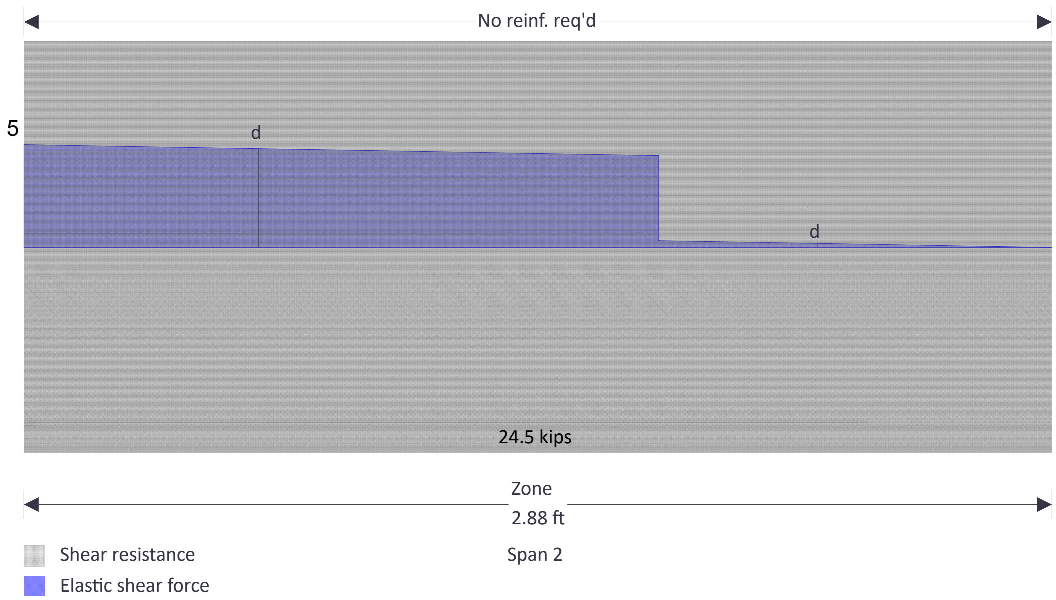
Min. allowable top bar clear spacing (cl.7.6.1); $s_{top,min}$ = **1.000** in

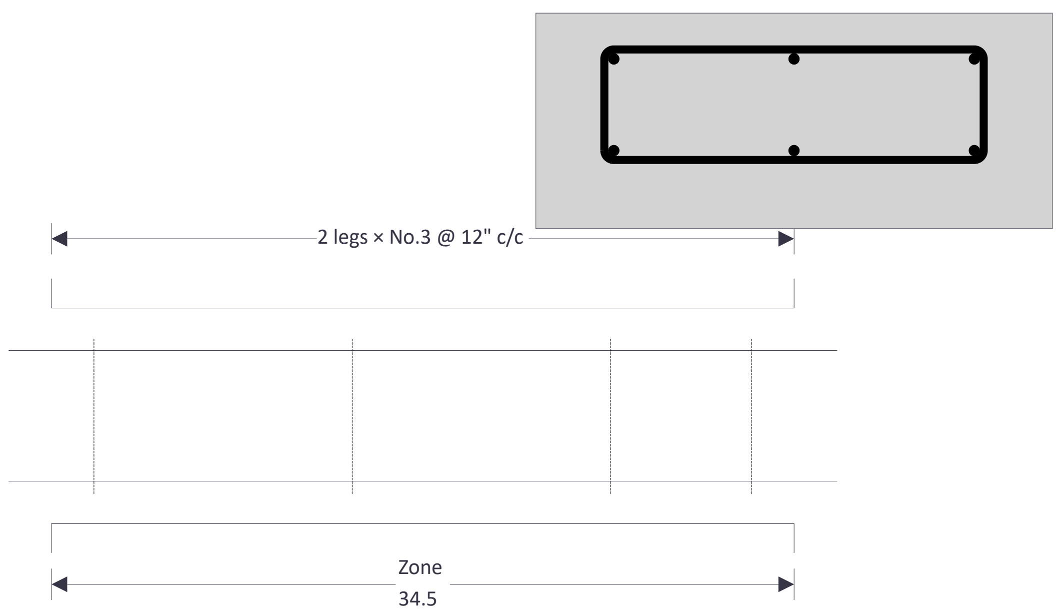
Bottom bar clear spacing; $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s2\_z2\_v}) + \phi_{m1\_s2\_z2\_b\_L1} \times N_{m1\_s2\_z2\_b\_L1})) / (N_{m1\_s2\_z2\_b\_L1} - 1)$ = **7.875** in

Min. allowable bottom bar clear spacing (cl.7.6.1); $s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Rectangular section in shear

Concrete weight modification factor; λ = **1.00**

Location where min. reinf. is req'd (V_u less ϕV_c); N/A no reinf. required along full length

Location where no reinf. is req'd (V_u less $\phi V_c / 2$); Between 0.00 ft and 2.88 ft

Maximum reinforcement shear strength; $\phi V_{s,max} = \phi_s \times 8\ psi \times \sqrt{(\min(f'_c, 10000psi) / 1\ psi)} \times b \times d$ = **71.720** kips

Minimum area of shear reinf. (eqn. 11-13); $A_{sv,min} = \max(50\ psi, 0.75\ psi \times \sqrt{(f'_c / 1\ psi)}) \times b / \min(f_y, 60000\ psi)$ = **0.240** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone; d = **7.875** in

Concrete shear strength (eqn. 11-3); $\phi V_c = \phi_s \times \lambda \times 2\ psi \times \sqrt{(f'_c / 1\ psi)} \times b \times d$ = **17.930** kips

Design shear force within zone; V_u = **4.778** kips

Reinf. shear strength required (eqn. 11-2); $\phi V_s = \max(V_u - \phi V_c, 0\ kips)$ = **0.000** kips

Area of design shear reinf. req'd (eqn. 11-15); $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\ psi) \times d)$ = **0.000** in^2/ft

Area of shear reinforcement required; $A_{sv,req} = 0in^2/ft$ = **0.000** in^2/ft

Shear reinforcement provided; 2 legs × No.3 @ 12" c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **0.221** in^2/ft

PASS - No shear reinforcement required ($\phi V_c/2 >= V_u$)

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