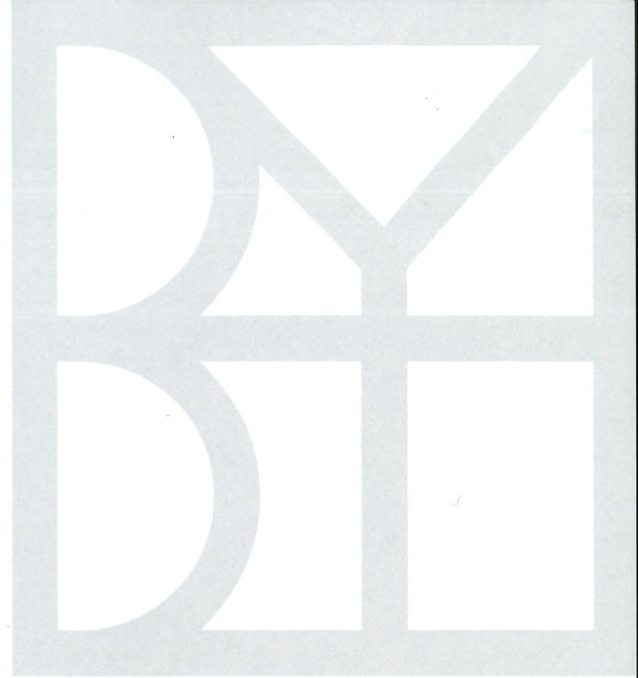




STRUCTURAL ENGINEERING CALCULATIONS

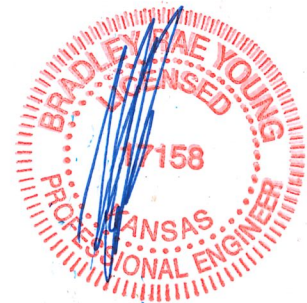
(Revision 4 – Metal Stud Framing)

MCDONALD'S RESTAURANT
2320 METROPOLITAN AVE
KANSAS CITY, KS 66106

BYA# 251023
August 5, 2026

Prepared For:

LINGLE DESIGN GROUP
158 West Main St.
Lena, IL 61048



BRAD YOUNG & ASSOCIATES, INC.
STRUCTURAL ENGINEERING

McDonald's Restaurant

Design Parameters

Kansas City, Kansas

Revision #4 - Change Wall Framing to Metal Stud

Code	= 2018 International Building Code (IBC)
Wind	= 110 mph, Exposure B
Seismic	= Risk Category II
	$S_s := 0.095$ $S_1 := 0.069$
	$I = 1.0$
	$R = 6.5$ (Metal Stud Shearwalls)
	$R = 3.25$ (Ordinary Braced Frame)
Steel f_y (W.F.) A992	= 50 ksi, UNO
Steel f_y (Tube) A500 Grade B	= 46 ksi, UNO
Steel f_y (Pipe) A53 Grade B	= 35 ksi, UNO
Steel f_y (Misc.) A36	= 36 ksi, UNO
Light gage studs f_y	= 33 ksi minimum, UNO
High Strength Bolts	= A325N
Typical Bolts	= A307
Concrete f'_c (Foundation)	= 3000 psi, UNO
Concrete f'_c (walls)	= 3000 psi, UNO
Concrete f'_c (Columns)	= 3000 psi, UNO
Concrete f'_c (Floor Decks)	= 3000 psi, UNO
Rebar #3 bars	= Gr. 40, $f_y = 40,000$ psi, UNO
Rebar #4 and larger bars	= Gr. 60, $f_y = 60,000$ psi, UNO
Welded Rebar	= A706
Dimensioned Lumber	= DF#2, UNO
Glulam Beams	= 24F-V4 @ simple spans, UNO
	= 24F-V8 @ Cantilever spans, UNO
Wood Connectors	= Simpson Strong Tie, UNO (Manufacturer)
(Equivalent may be used)	
Masonry f_m	= 1500 psi, UNO
Allowable Soil Bearing Pressure	= 2000 psf DL+LL/SL (per Geotechnical Report)

Note: The above material properties are typical properties and apply unless noted otherwise within this calculation package. The materials listed are not necessarily used in this design. Please see the following calculations for the actual types of materials used for this specific design.

BRAD YOUNG & ASSOCIATES **STRUCTURAL ENGINEERING**

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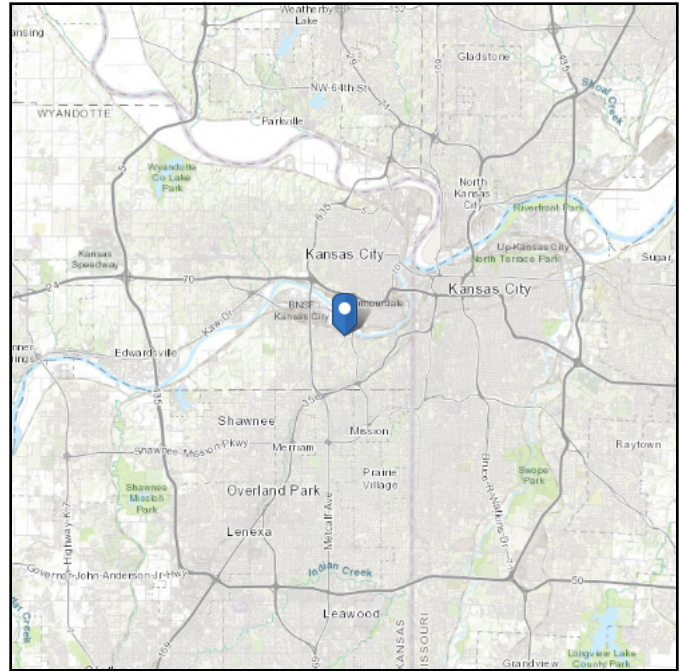
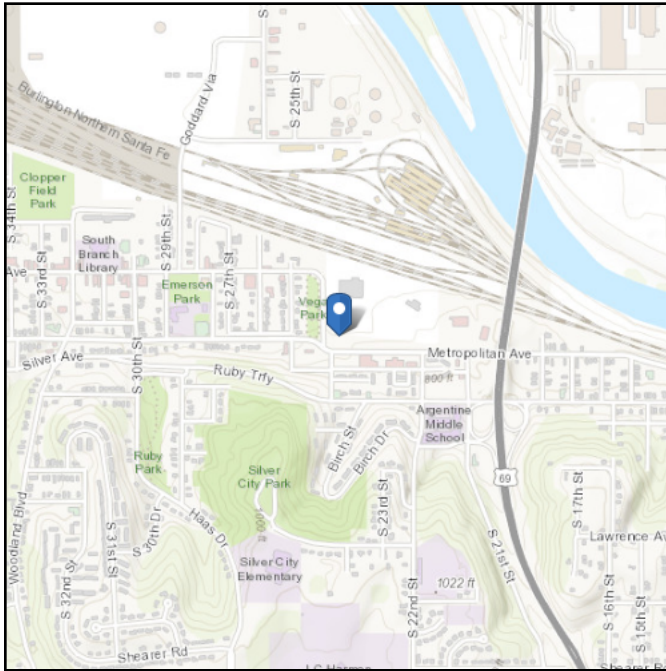
Copyright 2026 Brad Young & Associates
Print Date: 8/3/2026
File Name: 251023 - Metal Stud.xmcd
Engineer: RV

ASCE Hazards Report

Address:
2320 Metropolitan Ave
Kansas City, Kansas
66106

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Stiff Soil

Latitude: 39.073348
Longitude: -94.656376
Elevation: 761.7949181372792 ft
(NAVD 88)



Wind

Results:

Wind Speed	110 Vmph
10-year MRI	76 Vmph
25-year MRI	83 Vmph
50-year MRI	88 Vmph
100-year MRI	94 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2

Date Accessed: Tue Sep 30 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

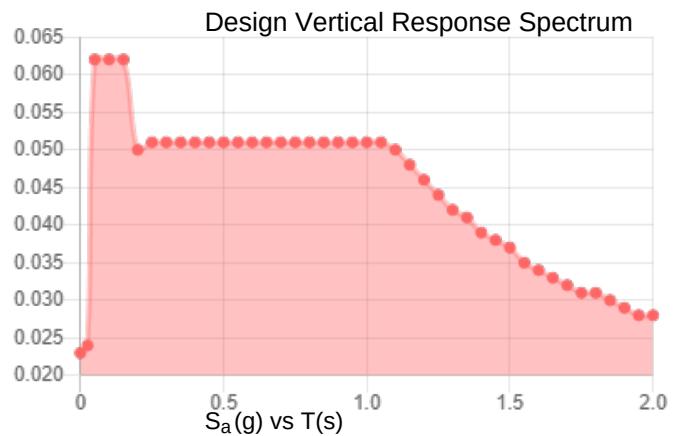
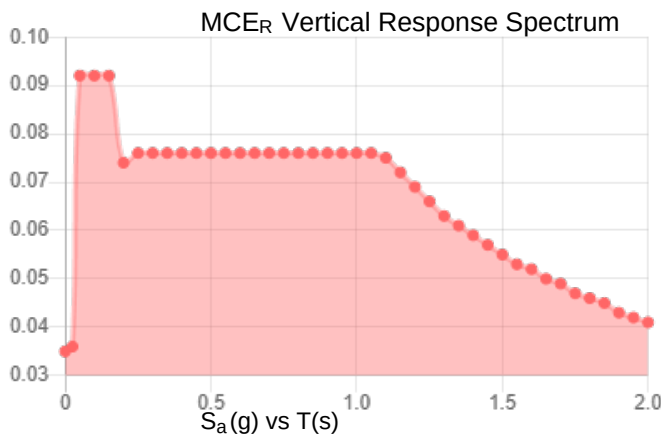
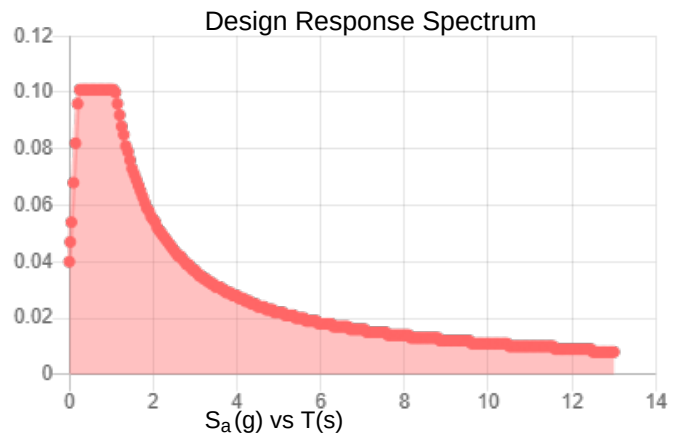
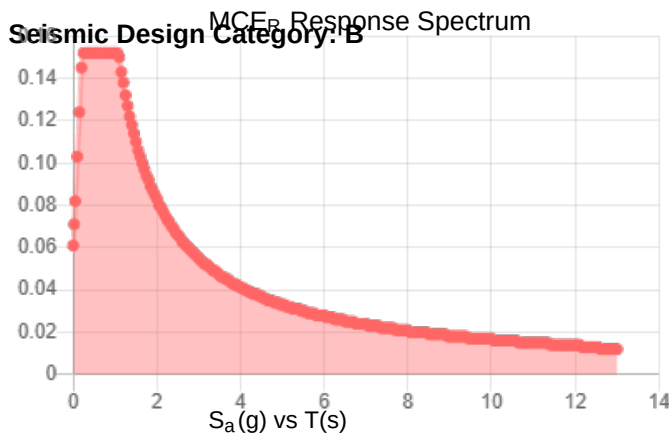
Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Site Soil Class: D - Stiff Soil

Results:

S_S :	0.095
S_1 :	0.069
F_a :	1.6
F_v :	2.4
S_{MS} :	0.152
S_{M1} :	0.165
S_{DS} :	0.101

S_{D1} :	0.11
T_L :	12
PGA :	0.045
PGA _M :	0.072
F_{PGA} :	1.6
I_e :	1
C_v :	0.7



Data Accessed: Tue Sep 30 2025

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

Results:

Ground Snow Load, p_g : 20 lb/ft²
Mapped Elevation: 761.8 ft

Data Source: ASCE/SEI 7-16, Table 7.2-8

Date Accessed: Tue Sep 30 2025

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.

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Loading

Main Roof (Flat)

$\begin{pmatrix} DL_{\text{roof}} \\ LL_{\text{roof}} \end{pmatrix} :=$			
	Roofing & Re-roofing	4.0	PSF
	Rigid Insulation	1.5	
	22 Gauge Metal Deck	1.6	
	Roof Truss & Blocking	4.5	
	Suspended Ceiling	1.8	
	MEP	2.0	
	Miscellaneous	4.6	
	Total Dead Load	20	
	Roof Live Load	20	

$$DL_{\text{roof}} := DL_{\text{roof}} \text{ psf}$$

$$DL_{\text{roof}} = 20 \cdot \text{psf}$$

$$LL_{\text{roof}} := LL_{\text{roof}} \text{ psf}$$

$$LL_{\text{roof}} = 20 \cdot \text{psf}$$

Walls

$$DL_{\text{wall.metal}} := 15 \cdot \text{psf}$$

Metal stud wall with siding

$$DL_{\text{wall.veneer}} := 40 \cdot \text{psf}$$

Masonry veneer w/ exterior stud framing

Awnings

$$DL_{\text{trellis}} := 5 \cdot \text{psf}$$

Trellis is aluminum framed with
metal deck - designed by others

Snow Load

$$p_g := 20 \cdot \text{psf}$$

Ground snow load per 2018 IBC

$$C_e := 1.0 \quad C_t := 1.0 \quad I_s := 1.0$$

$$p_f := 0.70 \cdot C_e \cdot C_t \cdot I_s \cdot p_g$$

$$p_f = 14 \cdot \text{psf}$$

$$C_s := 1.0$$

$$p_s := C_s \cdot p_f$$

$$p_s = 14 \cdot \text{psf}$$

$$p_m := \text{if}(p_g > 20 \text{psf}, 20 \text{psf} \cdot I_s, I_s \cdot p_g)$$

$$p_m = 20 \cdot \text{psf}$$

$$LL_{\text{snow}} := \max(p_s, p_m)$$

$$LL_{\text{snow}} = 20 \cdot \text{psf}$$

Snow Drift Calculation

The snow drift can accumulate in the plan NS direction or plan EW direction. The max wall height is ~ 6 ft above the building roof height. Length of roof upwind is about 6" (use 20 ft). Length of roof is a maximum of 97ft 0-1/4in (NS direction).

Snow Loads - Plan NS direction snow drift

$$p_g = 20 \cdot \text{psf}$$

$$p_s = 14 \cdot \text{psf}$$

Snow Drift - per ASCE 7-16 Sections 7.7.1 & 7.8

$$\gamma := \min\left(0.13 \cdot p_g \cdot \frac{1}{\text{ft}} + 14 \cdot \text{pcf}, 30 \cdot \text{pcf}\right)$$

$$\gamma = 16.6 \cdot \text{pcf}$$

$$h_p := 6 \cdot \text{ft}$$

wall/parapet height

$$h_b := \frac{p_s}{\gamma}$$

$$h_b = 0.843 \text{ ft}$$

non drift snow height

$$h_c := h_p - h_b$$

$$h_c = 5.157 \text{ ft}$$

max drift height

$$l_{u,\text{lee}} := 20 \cdot \text{ft}$$

minimum Lu value for leeward case

$$h_{d,\text{lee}} := \left(0.43 \cdot \sqrt[3]{l_{u,\text{lee}} \cdot \frac{1}{\text{ft}}} \cdot \sqrt[4]{p_g \cdot \frac{1}{\text{psf}} + 10 - 1.5}\right) \cdot \text{ft}$$

$$h_{d,\text{lee}} = 1.232 \text{ ft}$$

leeward drift height

$$l_{u,\text{wind}} := (97\text{ft} + 3\text{in}) = 97.25 \text{ ft}$$

Low Roof Length for windward case

$$h_{d,\text{wind}} := \left(0.43 \cdot \sqrt[3]{l_{u,\text{wind}} \cdot \frac{1}{\text{ft}}} \cdot \sqrt[4]{p_g \cdot \frac{1}{\text{psf}} + 10 - 1.5}\right) \cdot \text{ft}$$

$$h_{d,\text{wind}} = 3.128 \text{ ft}$$

windward drift height

$$h_d := \min\left(h_c, \max\left(h_{d,\text{lee}}, \frac{3}{4} \cdot h_{d,\text{wind}}\right), 60\% \cdot l_{u,\text{wind}}\right)$$

$$h_d = 2.346 \text{ ft}$$

$$p_{d1} := \gamma \cdot h_d$$

$$p_{d1} = 38.942 \cdot \text{psf}$$

maximum intensity of drift surcharge load

$$w1 := \min(4 \cdot h_d, 8 \cdot h_c)$$

$$w1 = 9.384 \text{ ft}$$

drift width

The snow drift can accumulate in the plan NS direction or plan EW direction. The max wall height is ~6 ft above the building roof height. Length of roof upwind is about 6" (use 20 ft). Length of roof is a maximum of 45ft 8in (EW direction).

Snow Loads - Plan EW direction snow drift

$$p_g = 20 \cdot \text{psf}$$

$$p_s = 14 \cdot \text{psf}$$

Snow Drift - per ASCE 7-16 Sections 7.7.1 & 7.8

$$\gamma := \min\left(0.13 \cdot p_g \cdot \frac{1}{\text{ft}} + 14 \cdot \text{pcf}, 30 \cdot \text{pcf}\right)$$

$$\gamma = 16.6 \cdot \text{pcf}$$

$$h_p := 6 \cdot \text{ft}$$

wall/parapet height

$$h_b := \frac{p_s}{\gamma}$$

$$h_b = 0.843 \text{ ft}$$

non drift snow height

$$h_c := h_p - h_b$$

$$h_c = 5.157 \text{ ft}$$

max drift height

$$l_{u,lee} := 20 \cdot \text{ft}$$

minimum Lu value for leeward case

$$h_{d,lee} := \left(0.43 \cdot \sqrt[3]{l_{u,lee} \cdot \frac{1}{\text{ft}}} \cdot \sqrt[4]{p_g \cdot \frac{1}{\text{psf}} + 10 - 1.5}\right) \cdot \text{ft}$$

$$h_{d,lee} = 1.232 \text{ ft}$$

leeward drift height

$$l_{u,wind} := (45 \text{ ft} + 7 \text{ in}) = 45.583 \text{ ft}$$

Low Roof Length for windward case

$$h_{d,wind} := \left(0.43 \cdot \sqrt[3]{l_{u,wind} \cdot \frac{1}{\text{ft}}} \cdot \sqrt[4]{p_g \cdot \frac{1}{\text{psf}} + 10 - 1.5}\right) \cdot \text{ft}$$

$$h_{d,wind} = 2.095 \text{ ft}$$

windward drift height

$$h_d := \min\left(h_c, \max\left(h_{d,lee}, \frac{3}{4} \cdot h_{d,wind}\right)\right)$$

$$h_d = 1.571 \text{ ft}$$

$$p_{d2} := \gamma \cdot h_d$$

$$p_{d2} = 26.081 \cdot \text{psf}$$

maximum intensity of drift surcharge load

$$w_2 := \min(4 \cdot h_d, 8 \cdot h_c)$$

$$w_2 = 6.285 \text{ ft}$$

drift width

Wind Uplift

Determine the wind uplift forces on roof. Note the wind uplift forces on the drawings for truss manufacturer to account for this in truss design.

Exposure B

$$V := 110$$

$$K_{zt} := 1.0$$

$$K_z := 0.57$$

$$K_d := 0.85$$

$$z_g := 761 \cdot \text{ft}$$

$$K_e := \exp\left(-0.0000362 \cdot \frac{z_g}{\text{ft}}\right) \quad K_e = 0.973$$

$$q_h := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 \cdot \text{psf} \quad (q_h) = 14.6 \cdot \text{psf}$$

$$A_{\text{truss}} := \max\left(44.417 \text{ft} \cdot 6 \text{ft}, 44.417 \text{ft} \cdot \frac{44.417 \text{ft}}{3}\right) \quad A_{\text{truss}} = 658 \text{ft}^2 \quad \text{Tributary Area at each truss}$$

$$p = q_h[(GC_p) - (GC_{pi})] \quad \text{ASCE 7-16: Section 30.4 (Components and Cladding Low Rise Buildings)}$$

$$GC_{p,1'} := -0.5 \quad \text{External Pressure Coefficient (Roof Zone 1)}$$

$$GC_{p,1} := -1.0 \quad \text{External Pressure Coefficient (Roof Zone 1)}$$

$$GC_{p,2} := -1.4 \quad \text{External Pressure Coefficient (Roof Zone 2)}$$

$$GC_{p,3} := -1.4 \quad \text{External Pressure Coefficient (Roof Zone 3)}$$

$$GC_{pi} := 0.18 \quad \text{Internal Pressure Coefficient}$$

Wind uplift pressure

$$p_{\text{Up},1'} := q_h \cdot (GC_{p,1'} - GC_{pi}) \quad p_{\text{Up},1'} = -9.928 \cdot \text{psf}$$

$$p_{\text{Up},1} := q_h \cdot (GC_{p,1} - GC_{pi}) \quad p_{\text{Up},1} = -17.228 \cdot \text{psf}$$

$$p_{\text{Up},2} := q_h \cdot (GC_{p,2} - GC_{pi}) \quad p_{\text{Up},2} = -23.068 \cdot \text{psf}$$

$$h := 14.50 \text{ft}$$

$$a := \max(\min(10\% \cdot 46.167 \text{ft}, 0.4 \cdot h), 4\% \cdot 46.167 \text{ft}, 3 \text{ft}) \quad a = 4.617 \text{ft}$$

$$0.6 \cdot \min(p_{\text{Up},1}, p_{\text{Up},2}) + 0.6 \cdot DL_{\text{roof}} = -1.8 \cdot \text{psf} \quad \text{Net Uplift Force for Joists}$$

Wind on walls

$$q_h = 14.6 \cdot \text{psf}$$

$$A_{\text{stud}} := \max\left(14.50\text{ft} \cdot 16\text{in}, 14.50\text{ft} \cdot \frac{14.50\text{ft}}{3}\right) \quad A_{\text{stud}} = 70\text{ft}^2$$

$$p = q_h[(GC_p) - (GC_{pi})] \quad 30.3.1 \text{ (Components and Cladding Low Rise Buildings)}$$

$$GC_{p1} := 0.85 \quad \text{Positive Pressure Coefficient (Wall Zone 4)}$$

$$GC_{p2} := -0.95 \quad \text{Negative Pressure Coefficient (Wall Zone 4)}$$

$$GC_{pi} := 0.18 \quad \text{Internal Pressure Coefficient}$$

Wall pressures

$$p_{\text{wall.1}} := q_h \cdot (GC_{p1} + GC_{pi}) \quad p_{\text{wall.1}} = 15.038 \cdot \text{psf}$$

$$p_{\text{wall.2}} := q_h \cdot (GC_{p2} - GC_{pi}) \quad p_{\text{wall.2}} = -16.498 \cdot \text{psf}$$

Wind on Parapet

$$q_h = 14.6 \cdot \text{psf}$$

$$A_{\text{parp}} := \max\left(5.50\text{ft} \cdot 16\text{in}, 5.50\text{ft} \cdot \frac{5.50\text{ft}}{3}\right) \quad A_{\text{parp}} = 10\text{ft}^2 \quad (\text{Conservatively Use Smaller Height})$$

$$p = q_p[(GC_p) - (GC_{pi})] \quad \text{EQ 30.8-1 (Components and Cladding Low Rise Buildings)}$$

$$GC_{p1} := 1.0 \cdot (0.9) \quad \text{Positive Pressure Coefficient (Wall Zone 4)}$$

$$GC_{p2} := -2.3 \quad \text{Negative Pressure Coefficient (Roof Zone 2)}$$

$$GC_{p3} := 1.0 \cdot (0.9) \quad \text{Positive Pressure Coefficient (Wall Zone 4)}$$

$$GC_{p4} := -1.1 \cdot (0.9) \quad \text{Negative Pressure Coefficient (Wall Zone 4)}$$

$$GC_{pi} := 0.0$$

Parapet pressures

$$p_{\text{parp.1}} := q_h \cdot (GC_{p1} - GC_{p2}) \quad p_{\text{parp.1}} = 46.72 \cdot \text{psf}$$

$$p_{\text{parp.2}} := q_h \cdot (GC_{p4} - GC_{p3}) \quad p_{\text{parp.2}} = -27.594 \cdot \text{psf}$$

Parapet Kickers - Wall Perpendicular to Joists

Check the wall studs at the load bearing walls for out of plane wind forces acting on the parapet wall. There will be parapet braces kicked back to the roof trusses to support the tall parapet. Determine the wind force acting on the roof joist.

NOTE: There are three different parapet heights. The main wall, the DT windows and the tall parapet.

Wind pressure on walls

$$p_{\text{wall}.1} = 15.038 \cdot \text{psf} \quad \text{positive wind force acting on wall}$$

$$p_{\text{wall}.2} = -16.498 \cdot \text{psf} \quad \text{negative wind force acting on wall}$$

$$p_{\text{parp}.1} = 46.72 \cdot \text{psf} \quad \text{positive wind force acting on parapet}$$

$$p_{\text{parp}.2} = -27.594 \cdot \text{psf} \quad \text{negative wind force acting on parapet}$$

$$H_{\text{plate}} := 13.333 \cdot \text{ft} \quad \text{Wall top plate/track height}$$

$$H_{\text{seat}} := 5 \cdot \text{in} \quad \text{Roof joist seat and blocking depth}$$

$$H_{\text{roofing}} := \left(\frac{1}{2} \cdot \text{in} \right) + (8 \cdot \text{in}) + \left(\frac{1}{2} \cdot \text{in} \right) \quad \text{Roof plywood, insulation and densdeck}$$

$$H_{\text{roof}} := H_{\text{plate}} + H_{\text{seat}} + H_{\text{roofing}}$$

$$H_{\text{roof}} = 14.50 \text{ ft} \quad \text{Height of the roofing}$$

$$H_{\text{parp}.1} := 20.125 \cdot \text{ft} \quad \text{Height of the Main parapet}$$

$$H_{\text{parp}.2} := 21.50 \cdot \text{ft} \quad \text{Height of the Drive Through (DT) parapet}$$

$$H_{\text{parp}.3} := 23.50 \cdot \text{ft} \quad \text{Height of the Tall parapet}$$

$$H_{\text{conn}} := 19.0 \cdot \text{ft} \quad \text{Height of the parapet brace connection}$$

Case 1 - Main Parapet W/ Kicker

Check the wind uplift forces acting on the roof framing

$$L_{C1.1} := H_{\text{conn}} - H_{\text{roof}} = 4.50 \text{ ft} \quad \text{length from roof to brace connection (vertical)}$$

$$L_{C1.2} := H_{\text{parp.1}} - H_{\text{conn}} = 1.125 \text{ ft} \quad \text{length from brace to top of parapet (vertical)}$$

$$L_{C1.3} := 4 \cdot \text{ft} \quad \text{distance brace connection to wall (horizontal)}$$

$$\text{stud}_{\text{spacing}} := 16 \text{ in}$$

Wind pressure on parapet walls (Use Maximum Load)

$$p_{\text{parp.1}} = 46.72 \cdot \text{psf} \quad p_{\text{wall.1}} = 15.038 \cdot \text{psf}$$

$$p_{\text{parp.2}} = -27.594 \cdot \text{psf} \quad p_{\text{wall.2}} = -16.498 \cdot \text{psf}$$

Wind pressure on parapet walls (Factored Reaction Loads)

$$M_{\text{max.C1}} := 83 \cdot \text{lb} \cdot \text{ft} \quad @ L = 2.112 \text{ ft} \quad V_{\text{max.C1}} := 89.33 \cdot \text{lb} \quad @ L = 4.50 \text{ ft}$$

$$R_{C1.\text{support.1}} := 79 \cdot \text{lb} \quad R_{C1.\text{support.2}} := 131 \cdot \text{lb}$$

See attached parapet wall stud design for wind pressure forces

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Case 1 - Main Parapet

Code References

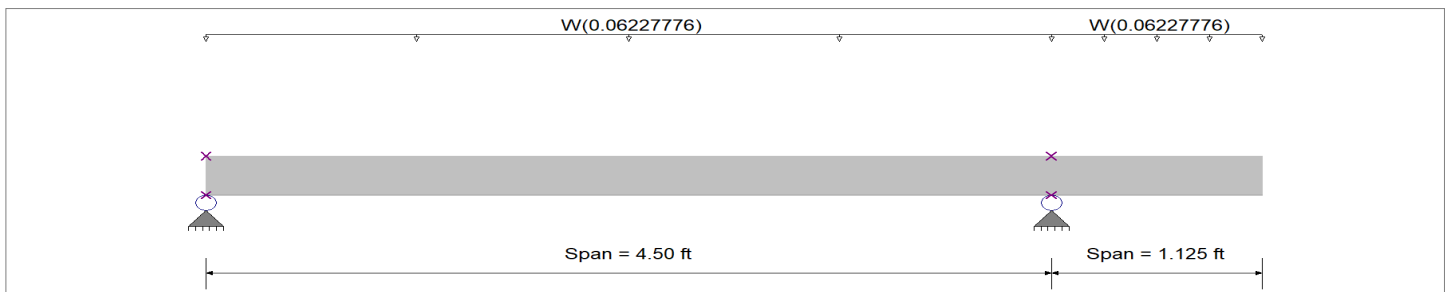
Governing Code : IBC 2018, CBC 2019

Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Beam Properties

Elastic Modulus	29,000.0 ksi				
Span #1	Span Length =	4.50 ft	Area =	0.3068 in^2	Moment of Inertia = 2.799 in^4
Span #2	Span Length =	1.125 ft	Area =	0.3068 in^2	Moment of Inertia = 2.799 in^4



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1

Uniform Load : $W = 0.04672$ ksf, Tributary Width = 1.333 ft, (Wind Pressure)

Load for Span Number 2

Uniform Load : $W = 0.04672$ ksf, Tributary Width = 1.333 ft, (Wind Pressure)

DESIGN SUMMARY

Maximum Bending =	0.083 k-ft	Maximum Shear =	0.08933 k
Load Combination		Load Combination	
+0.60W		+0.60W	
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	2.112 ft	Location of maximum on span	4.500 ft
Maximum Deflection	Deflection	Ratio	Span
Max Downward Transient Deflection	0.006 in	8855	Span #1
Max Upward Transient Deflection	-0.004 in	6778	Span #2
Max Downward Total Deflection	0.000 in	0	Span #2
Max Upward Total Deflection	-0.002 in	15062	Span #2
			Load Combo
			W Only
			W Only
			Span #2
			+0.450W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)						Shear Values (k)	
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega Cb	Rm	Va Max	Vnx/Vnx/Omega
Overall MAXimum Envelope											
Dsgn. L = 4.50 ft	1			0.08	-0.02	0.08				0.09	
Dsgn. L = 1.13 ft	2				-0.02	0.02				0.04	
Dsgn. L = 4.50 ft	1									-0.00	
Dsgn. L = 1.13 ft	2									-0.00	
+0.60W											
Dsgn. L = 4.50 ft	1			0.08	-0.02	0.08				0.09	
Dsgn. L = 1.13 ft	2				-0.02	0.02				0.04	
+0.450W											
Dsgn. L = 4.50 ft	1			0.06	-0.02	0.06				0.07	
Dsgn. L = 1.13 ft	2				-0.02	0.02				0.03	

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.0061	2.215	W Only	0.0000	0.000
	2	0.0000	2.215		-0.0040	1.125

Vertical Reactions

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Cases	0.131	0.219	

Support notation : Far left is #1

Values in KIPS

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Case 1 - Main Parapet

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from Load Combination	0.079	0.131	
Max Upward from Load Cases	0.131	0.219	
Max Downward from all Load Cases			
Max Downward from Load Combination			
Max Downward from Load Cases			
+0.60W	0.079	0.131	
+0.450W	0.059	0.099	
W Only	0.131	0.219	

Vertical Reactions for Load Cases

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from Load Cases	0.131	0.219	
W Only	0.131	0.219	

Vertical Reactions for Load Combinations

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from Load Combination	0.079	0.131	
+0.60W	0.079	0.131	
+0.450W	0.059	0.099	

Case 2 - DT Parapet W/ Kicker

Check the wind uplift forces acting on the roof framing

$$L_{C2.1} := H_{\text{conn}} - H_{\text{roof}} = 4.50 \text{ ft} \quad \text{length from roof to brace connection (vertical)}$$

$$L_{C2.2} := H_{\text{parp.2}} - H_{\text{conn}} = 2.5 \text{ ft} \quad \text{length from brace to top of parapet (vertical)}$$

$$L_{C2.3} := 4 \cdot \text{ft} \quad \text{distance brace connection to wall (horizontal)}$$

$$\text{stud}_{\text{spacing}} := 16 \text{ in}$$

Wind pressure on parapet walls (Use Maximum Load)

$$p_{\text{parp.1}} = 46.72 \cdot \text{psf} \quad p_{\text{wall.1}} = 15.038 \cdot \text{psf}$$

$$p_{\text{parp.2}} = -27.594 \cdot \text{psf} \quad p_{\text{wall.2}} = -16.498 \cdot \text{psf}$$

Wind pressure on parapet walls (Factored Reaction Loads)

$$M_{\text{max.C2}} := 117 \cdot \text{lb} \cdot \text{ft} \quad @ L = 2.112 \text{ ft} \quad V_{\text{max.C2}} := 110 \cdot \text{lb} \quad @ L = 4.50 \text{ ft}$$

$$R_{C2.\text{support.1}} := 58 \cdot \text{lb} \quad R_{C2.\text{support.2}} := 203 \cdot \text{lb}$$

See attached parapet wall stud design for wind pressure forces

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Case 2 - Drive Through Parapet

Code References

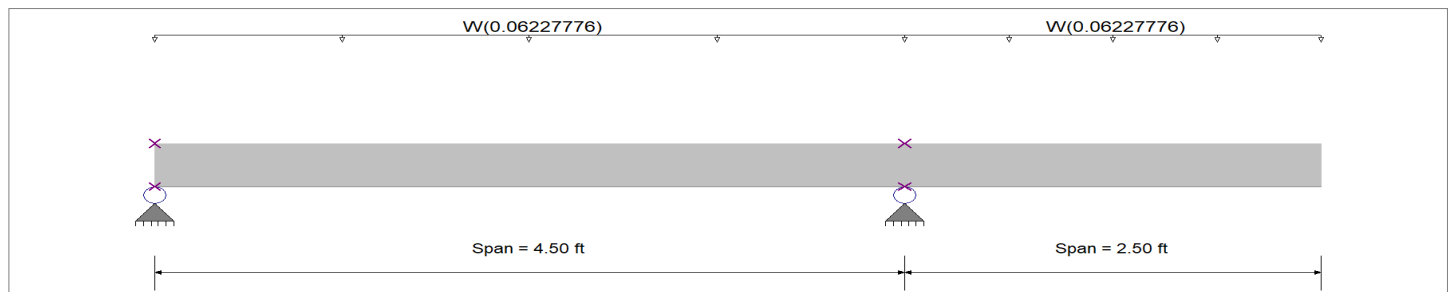
Governing Code : IBC 2018, CBC 2019

Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Beam Properties

Elastic Modulus	29,000.0 ksi				
Span #1	Span Length =	4.50 ft	Area =	0.3068 in^2	Moment of Inertia = 2.799 in^4
Span #2	Span Length =	2.50 ft	Area =	0.3068 in^2	Moment of Inertia = 2.799 in^4



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1

Uniform Load : $W = 0.04672$ ksf, Tributary Width = 1.333 ft, (Wind Pressure)

Load for Span Number 2

Uniform Load : $W = 0.04672$ ksf, Tributary Width = 1.333 ft, (Wind Pressure)

DESIGN SUMMARY

Maximum Bending =	0.117 k-ft	Maximum Shear =	0.110 k
Load Combination		Load Combination	
+0.60W		+0.60W	
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	4.500 ft	Location of maximum on span	4.500 ft
Maximum Deflection			
Max Downward Transient Deflection	Deflection	Ratio	Span
Max Upward Transient Deflection	0.009 in	6388	Span #2
Max Downward Total Deflection	0.000 in	0	Span #2
Max Upward Total Deflection	0.004 in	14194	Span #2
	-0.000 in	787293	Span #1
			Load Combo
			W Only
			+0.450W
			+0.450W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)					Shear Values (k)	
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx Mnx/Omega Cb	Rm	Va Max	VnxVnx/Omega
Overall MAXimum Envelope										
Dsgn. L = 4.50 ft	1			0.05	-0.12	0.12			0.11	
Dsgn. L = 2.50 ft	2				-0.12	0.12			0.09	
Dsgn. L = 4.50 ft	1								-0.00	
Dsgn. L = 2.50 ft	2								-0.00	
+0.60W										
Dsgn. L = 4.50 ft	1			0.05	-0.12	0.12			0.11	
Dsgn. L = 2.50 ft	2				-0.12	0.12			0.09	
+0.450W										
Dsgn. L = 4.50 ft	1			0.03	-0.09	0.09			0.08	
Dsgn. L = 2.50 ft	2				-0.09	0.09			0.07	

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.0021	1.731	W Only	-0.0002	4.188
W Only	2	0.0094	2.500		0.0000	4.188

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Cases	0.097	0.339	

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Case 2 - Drive Through Parapet

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from Load Combination	0.058	0.203	
Max Upward from Load Cases	0.097	0.339	
Max Downward from all Load Cases			
Max Downward from Load Combination			
Max Downward from Load Cases			
+0.60W	0.058	0.203	
+0.450W	0.044	0.153	
W Only	0.097	0.339	

Vertical Reactions for Load Cases

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from Load Cases	0.097	0.339	
W Only	0.097	0.339	

Vertical Reactions for Load Combinations

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from Load Combination	0.058	0.203	
+0.60W	0.058	0.203	
+0.450W	0.044	0.153	

Case 3 - Tall Parapet W/ Kicker

Check the wind uplift forces acting on the roof framing

$$L_{C3.1} := H_{\text{conn}} - H_{\text{roof}} = 4.50 \text{ ft} \quad \text{length from roof to brace connection (vertical)}$$

$$L_{C3.2} := H_{\text{parp.3}} - H_{\text{conn}} = 4.50 \text{ ft} \quad \text{length from brace to top of parapet (vertical)}$$

$$L_{C3.3} := 4 \cdot \text{ft} \quad \text{distance brace connection to wall (horizontal)}$$

$$\text{stud}_{\text{spacing}} := 16 \text{ in}$$

Wind pressure on parapet walls (Use Maximum Load)

$$p_{\text{parp.1}} = 46.72 \cdot \text{psf} \quad p_{\text{wall.1}} = 15.038 \cdot \text{psf}$$

$$p_{\text{parp.2}} = -27.594 \cdot \text{psf} \quad p_{\text{wall.2}} = -16.498 \cdot \text{psf}$$

Wind pressure on parapet walls (Factored Reaction Loads)

$$M_{\text{max.C3}} := 378 \cdot \text{lb} \cdot \text{ft} \quad @ L = 2.112 \text{ ft} \quad V_{\text{max.C3}} := 168.20 \cdot \text{lb} \quad @ L = 4.50 \text{ ft}$$

$$R_{C3.\text{support.1}} := 0 \cdot \text{lb} \quad R_{C3.\text{support.2}} := 336 \cdot \text{lb}$$

See attached parapet wall stud design for wind pressure forces

NOTE: Check Metal Stud (600s162-54) for moment and shear per enercalc report for the parapet wall span. Case 3 is the worst case, thus, if Case 3 passes then both Case 1 & Case 2 are ok by inspection.



Section Inputs

Material: A1003 ST Grade 50H

Apply cold work of forming strength increase.

No inelastic reserve strength increase.

Modulus of Elasticity, E 29500000 psi

Yield Strength, Fy 50000 psi

Tensile Strength, Fu 65000 psi

Min Elongation in 2 inches 10 %

Torsion Constant Override, J 0 in⁴

Warping Constant Override, Cw 0 in⁶

Hole Length 4

Hole Spacing 24

Stud, Thickness 0.0566 in (54 mil)

Placement of Part from Origin:

X to center of gravity 0 in

Y to center of gravity 0 in

Outside dimensions, Open shape

	Length (in)	Angle (deg)	Radius (in)	Web	k Coef.	Hole Size (in)	Distance (in)
1	0.5000	270.000	0.084900	None	0.000	0.0000	0.2500
2	1.6250	180.000	0.084900	Single	0.000	0.0000	0.8125
3	6.0000	90.000	0.084900	Cee	0.000	1.5000	3.0000
4	1.6250	0.000	0.084900	Single	0.000	0.0000	0.8125
5	0.5000	-90.000	0.084900	None	0.000	0.0000	0.2500

Member Check - AISI S100-24, US, ASD

Material Type: A1003 ST Grade 50H, Fy=50000 psi

Design Parameters:

Lx	4.5000 ft	Ly	0.0000 ft	Lt	0.0000 ft
Kx	1.0000	Ky	1.0000	Kt	1.0000
Cbx	1.0000	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: None		kφ	0 lb		
Red. Factor, R: 0		Lm	9.0000 ft		

Section: 600S162-54.cfss

R.V.

600S162-54, 50 ksi Stud

Brad Young & Associates, Inc.

SSMA Library

Rev. Date: 7/21/2025 8:15:00 AM

By: RSG Software

Printed: 7/29/2026 1:08:43 PM

Loads:	P (lb)	Mx (lb-ft)	Vy (lb)	My (lb-ft)	Vx (lb)
Entered	0.0	378.0	169.0	0.0	0.0
Applied	0.0	378.0	169.0	0.0	0.0
Strength	7890.3	1964.8	1803.2	402.8	2729.0

Interaction Equations

Eq. H1.2-1 (P, Mx, My) $0.000 + 0.192 + 0.000 = 0.192 \leq 1.0$ Eq. H2-1 (Mx, Vy) $\text{Sqrt}(0.027 + 0.009) = 0.189 \leq 1.0$ Eq. H2-1 (My, Vx) $\text{Sqrt}(0.000 + 0.000) = 0.000 \leq 1.0$ Calculation of Mndx used $\alpha_s=1$ from DSM inputs.Calculation of Mndy used $\alpha_s=0$ from DSM inputs.

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Case 3 - Tall Parapet

Code References

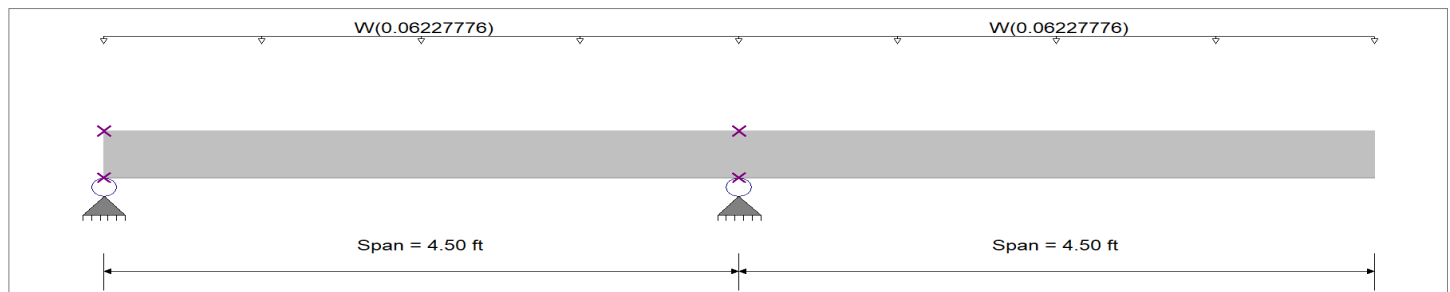
Governing Code : IBC 2018, CBC 2019

Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Beam Properties

Elastic Modulus	29,000.0 ksi				
Span #1	Span Length =	4.50 ft	Area =	0.3068 in ²	Moment of Inertia = 2.799 in ⁴
Span #2	Span Length =	4.50 ft	Area =	0.3068 in ²	Moment of Inertia = 2.799 in ⁴



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1

Uniform Load : $W = 0.04672$ ksf, Tributary Width = 1.333 ft, (Wind Pressure)

Load for Span Number 2

Uniform Load : $W = 0.04672$ ksf, Tributary Width = 1.333 ft, (Wind Pressure)

DESIGN SUMMARY

Maximum Bending =	0.378 k-ft	Maximum Shear =	0.1682 k
Load Combination		Load Combination	
+0.60W		+0.60W	
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	4.500 ft	Location of maximum on span	4.500 ft
Maximum Deflection	Deflection	Ratio	Span
Max Downward Transient Deflection	0.136 in	796	Span #2
Max Upward Transient Deflection	-0.011 in	5005	Span #1
Max Downward Total Deflection	0.000 in	0	Span #1
Max Upward Total Deflection	-0.005 in	11122	Span #1
			Load Combo
			W Only
			W Only
			Span #1
			+0.450W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)					Shear Values (k)	
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx Mnx/Omega Cb	Rm	Va Max	VnxVnx/Omega
Overall MAXimum Envelope										
Dsgn. L = 4.50 ft	1				-0.38	0.38			0.17	
Dsgn. L = 4.50 ft	2				-0.38	0.38			0.17	
Dsgn. L = 4.50 ft	1								-0.00	
Dsgn. L = 4.50 ft	2								-0.00	
+0.60W										
Dsgn. L = 4.50 ft	1				-0.38	0.38			0.17	
Dsgn. L = 4.50 ft	2				-0.38	0.38			0.17	
+0.450W										
Dsgn. L = 4.50 ft	1				-0.28	0.28			0.13	
Dsgn. L = 4.50 ft	2				-0.28	0.28			0.13	

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.0000	0.000	W Only	-0.0108	2.838
	2	0.1356	4.500		0.0000	2.838

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Cases	0.000	0.560	

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

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DESCRIPTION: Case 3 - Tall Parapet

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from Load Combination	0.000	0.336	
Max Upward from Load Cases	0.000	0.560	
Max Downward from all Load Cases			
Max Downward from Load Combination			
Max Downward from Load Cases			
+0.60W	0.000	0.336	
+0.450W	0.000	0.252	
W Only	0.000	0.560	

Vertical Reactions for Load Cases

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from Load Cases	0.000	0.560	
W Only	0.000	0.560	

Vertical Reactions for Load Combinations

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from Load Combination	0.000	0.336	
+0.60W	0.000	0.336	
+0.450W	0.000	0.252	

Windward Force

$$w_{\text{down.C1}} := \frac{p_{\text{parp.1}}}{2 \cdot (L_{\text{C1.1}})} \cdot (L_{\text{C1.1}} + L_{\text{C1.2}})^2$$

$$w_{\text{down.C1}} = 164 \cdot \text{plf}$$

$$w_{\text{down.C2}} := \frac{p_{\text{parp.1}}}{2 \cdot (L_{\text{C2.1}})} \cdot (L_{\text{C2.1}} + L_{\text{C2.2}})^2$$

$$w_{\text{down.C2}} = 254 \cdot \text{plf}$$

$$w_{\text{down.C3}} := \frac{p_{\text{parp.1}}}{2 \cdot (L_{\text{C3.1}})} \cdot (L_{\text{C3.1}} + L_{\text{C3.2}})^2$$

$$w_{\text{down.C3}} = 420 \cdot \text{plf}$$

Unfactored Downward Force per brace

The parapet will land 4'-0" away from the wall on roof joists.

$$P_{\text{down.C1}} := (w_{\text{down.C1}} \cdot 32\text{in}) \cdot \left(\frac{L_{\text{C1.1}}}{L_{\text{C1.3}}} \right)$$

$$P_{\text{down.C1}} = 493 \text{ lb}$$

$$P_{\text{down.C2}} := (w_{\text{down.C2}} \cdot 32\text{in}) \cdot \left(\frac{L_{\text{C2.1}}}{L_{\text{C2.3}}} \right)$$

$$P_{\text{down.C2}} = 763 \text{ lb}$$

$$P_{\text{down.C3}} := (w_{\text{down.C3}} \cdot 32\text{in}) \cdot \left(\frac{L_{\text{C3.1}}}{L_{\text{C3.3}}} \right)$$

$$P_{\text{down.C3}} = 1262 \text{ lb}$$

Leeward Force

$$w_{\text{uplift.C1}} := \frac{|p_{\text{parp.2}}|}{2 \cdot L_{\text{C1.1}}} \cdot (L_{\text{C1.1}} + L_{\text{C1.2}})^2$$

$$w_{\text{uplift.C1}} = 97 \cdot \text{plf}$$

$$w_{\text{uplift.C2}} := \frac{|p_{\text{parp.2}}|}{2 \cdot L_{\text{C2.1}}} \cdot (L_{\text{C2.1}} + L_{\text{C2.2}})^2$$

$$w_{\text{uplift.C2}} = 150 \cdot \text{plf}$$

$$w_{\text{uplift.C3}} := \frac{|p_{\text{parp.2}}|}{2 \cdot L_{\text{C3.1}}} \cdot (L_{\text{C3.1}} + L_{\text{C3.2}})^2$$

$$w_{\text{uplift.C3}} = 248 \cdot \text{plf}$$

Unfactored Uplift Force per brace

The parapet will land 4'-0" away from the wall on roof joists.

$$P_{\text{uplift.C1}} := (w_{\text{uplift.C1}} \cdot 32\text{in}) \cdot \left(\frac{L_{\text{C1.1}}}{L_{\text{C1.3}}} \right)$$

$$P_{\text{uplift.C1}} = 291 \text{ lb}$$

$$P_{\text{uplift.C2}} := (w_{\text{uplift.C2}} \cdot 32\text{in}) \cdot \left(\frac{L_{\text{C2.1}}}{L_{\text{C2.3}}} \right)$$

$$P_{\text{uplift.C2}} = 451 \text{ lb}$$

$$P_{\text{uplift.C3}} := (w_{\text{uplift.C3}} \cdot 32\text{in}) \cdot \left(\frac{L_{\text{C3.1}}}{L_{\text{C3.3}}} \right)$$

$$P_{\text{uplift.C3}} = 745 \text{ lb}$$

Factored Uplift Force per kicker connection

$$w_{\text{uplift.C1}} = 97 \cdot \text{plf}$$

$$w_{\text{uplift.C2}} = 150 \cdot \text{plf}$$

$$w_{\text{uplift.C3}} = 248 \cdot \text{plf}$$

$$P_{\text{uplift.C1.kick}} := 0.6 \cdot \left[(w_{\text{uplift.C1}} \cdot 16 \text{ in}) \cdot \left(\frac{L_{\text{C1.1}}}{L_{\text{C1.3}}} \right) \right]$$

$$P_{\text{uplift.C1.kick}} = 87.3 \text{ lb}$$

$$P_{\text{uplift.C2.kick}} := 0.6 \cdot \left[(w_{\text{uplift.C2}} \cdot 16 \text{ in}) \cdot \left(\frac{L_{\text{C2.1}}}{L_{\text{C2.3}}} \right) \right]$$

$$P_{\text{uplift.C2.kick}} = 135.2 \text{ lb}$$

$$P_{\text{uplift.C3.kick}} := 0.6 \cdot \left[(w_{\text{uplift.C3}} \cdot 16 \text{ in}) \cdot \left(\frac{L_{\text{C3.1}}}{L_{\text{C3.3}}} \right) \right]$$

$$P_{\text{uplift.C3.kick}} = 223.5 \text{ lb}$$

BRAD YOUNG & ASSOCIATES

STRUCTURAL ENGINEERING

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Print Date: 8/3/2026

File Name: 251023 - Metal Stud.xmcd

Engineer: RV

Wall Stud Analysis:

Check the wall studs for the various loading conditions around the building.

Metal Wall Stud (Case 1) - Non-Truss Bearing

Check the 2x6 studs aligned under the roof truss taking the full weight of the truss

$$L_1 := (14 \cdot \text{ft} + 6 \cdot \text{in}) + (4 \cdot \text{ft} + 6 \cdot \text{in}) + (4 \cdot \text{ft} + 6 \cdot \text{in}) = 23.50 \text{ ft} \quad \text{Span from ground to top of parapet}$$

$$w_{\text{trib}} := \frac{16 \text{ in} + 16 \text{ in}}{2} \quad w_{\text{trib}} = 1.333 \text{ ft}$$

Axial Loads on Wall Studs:

$$P_{\text{DL}} := (w_{\text{trib}}) \cdot \left(\text{DL}_{\text{roof}} \cdot \frac{22 \text{ in}}{2} \right) + [w_{\text{trib}} \cdot \text{DL}_{\text{wall.metal}} \cdot (L_1)] \quad P_{\text{DL}} = 494 \text{ lb}$$

$$P_{\text{LL}} := (w_{\text{trib}}) \cdot \left[\text{LL}_{\text{roof}} \cdot \left(\frac{22 \text{ in}}{2} \right) \right] \quad P_{\text{LL}} = 24 \text{ lb}$$

$$P_{\text{SL}} := (w_{\text{trib}}) \cdot \left[\text{LL}_{\text{snow}} \cdot \left(\frac{22 \text{ in}}{2} \right) + \frac{1}{2} \cdot \text{DL}_{\text{wall.metal}} \cdot (w_1) \right] \quad P_{\text{SL}} = 118 \text{ lb}$$

$$\text{Total}_{\text{max.C1}} := P_{\text{DL}} + \max(P_{\text{LL}}, P_{\text{SL}}) \quad \text{Total}_{\text{max.C1}} = 613 \text{ lb}$$

Wind Loads on Studs:

$$p_{\text{wall.1}} = 15.038 \cdot \text{psf} \quad p_{\text{parp.1}} = 46.72 \cdot \text{psf}$$

$$p_{\text{wall.2}} = -16.498 \cdot \text{psf} \quad p_{\text{parp.2}} = -27.594 \cdot \text{psf}$$

$$w_{\text{wall}} := \max[(p_{\text{wall.1}} \cdot w_{\text{trib}}), (|p_{\text{wall.2}}| \cdot w_{\text{trib}})] \quad w_{\text{wall}} = 21.997 \cdot \text{plf}$$

$$w_{\text{parapet}} := \max[(p_{\text{parp.1}} \cdot w_{\text{trib}}), (|p_{\text{parp.2}}| \cdot w_{\text{trib}})] \quad w_{\text{parapet}} = 62.294 \cdot \text{plf}$$

See attached metal wall stud design for axial and wind pressure forces:

$$M_{\text{max}} := 236 \cdot \text{lb} \cdot \text{ft} \quad @ L = 4.50 \text{ ft} \quad V_{\text{max}} := 110 \cdot \text{lb} \quad @ L = 4.50 \text{ ft} \quad (\text{For Span 1 Only which is } 14.50 \text{ ft})$$

$$R_{\text{support.1}} := 79 \cdot \text{lb} \quad R_{\text{support.2}} := 125 \cdot \text{lb} \quad R_{\text{support.3}} := 212 \cdot \text{lb}$$

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

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DESCRIPTION: Case 1 - Non-Truss Bearing

Code References

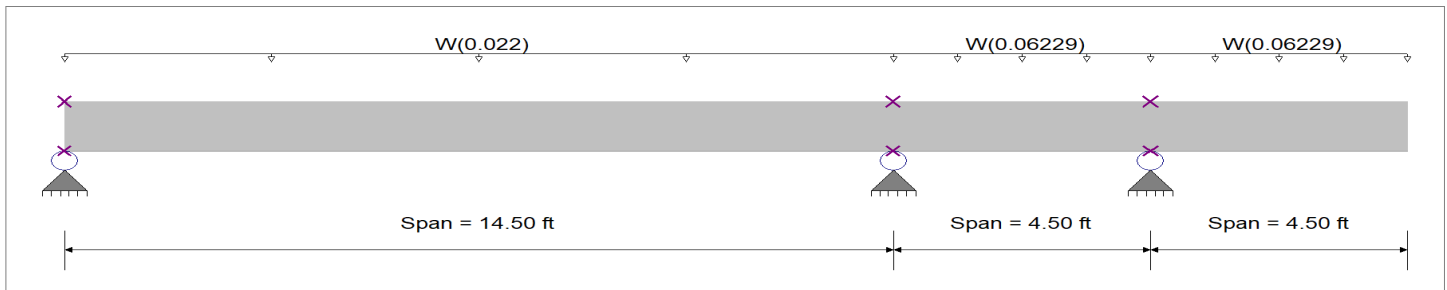
Governing Code : IBC 2018, CBC 2019

Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Beam Properties

Elastic Modulus	29,000.0 ksi				
Span #1	Span Length =	14.50 ft	Area =	0.3068 in ²	Moment of Inertia = 2.799 in ⁴
Span #2	Span Length =	4.50 ft	Area =	0.3068 in ²	Moment of Inertia = 2.799 in ⁴
Span #3	Span Length =	4.50 ft	Area =	0.3068 in ²	Moment of Inertia = 2.799 in ⁴



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1

Uniform Load : $W = 0.0220$ k/ft, Tributary Width = 1.0 ft, (Wind Pressure)

Load for Span Number 2

Uniform Load : $W = 0.06229$ k/ft, Tributary Width = 1.0 ft, (Wind Pressure)

Load for Span Number 3

Uniform Load : $W = 0.06229$ k/ft, Tributary Width = 1.0 ft, (Wind Pressure)

DESIGN SUMMARY

Maximum Bending =	0.378 k-ft	Maximum Shear =	0.1682 k
Load Combination		Load Combination	
+0.60W		+0.60W	
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2
Location of maximum on span	4.500 ft	Location of maximum on span	4.500 ft
Maximum Deflection	Deflection	Ratio	Span Load Combo
Max Downward Transient Deflection	0.164 in	656	Span #3 W Only
Max Upward Transient Deflection	-0.021 in	2527	Span #2 W Only
Max Downward Total Deflection	0.000 in	0	Span #2
Max Upward Total Deflection	-0.010 in	5617	Span #2 +0.450W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)					Shear Values (k)	
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega Cb	Rm	Va Max
Overall MAXimum Envelope										
Dsgn. L = 14.32 ft	1			0.24	-0.22	0.24				0.11
Dsgn. L = 4.68 ft	2			-0.00	-0.38	0.38				0.17
Dsgn. L = 4.50 ft	3				-0.38	0.38				0.17
Dsgn. L = 14.32 ft	1									-0.00
Dsgn. L = 4.68 ft	2									-0.00
Dsgn. L = 4.50 ft	3									-0.00
+0.60W										
Dsgn. L = 14.32 ft	1			0.24	-0.22	0.24				0.11
Dsgn. L = 4.68 ft	2			-0.00	-0.38	0.38				0.17
Dsgn. L = 4.50 ft	3				-0.38	0.38				0.17
+0.450W										
Dsgn. L = 14.32 ft	1			0.18	-0.17	0.18				0.08
Dsgn. L = 4.68 ft	2			-0.00	-0.28	0.28				0.13
Dsgn. L = 4.50 ft	3				-0.28	0.28				0.13

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Case 1 - Non-Truss Bearing

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.1618	6.791		0.0000	0.000
	2	0.0000	6.791	W Only	-0.0214	2.392
W Only	3	0.1644	4.500		0.0000	2.392

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Cases	0.132	0.277	0.471	
Max Upward from Load Combination	0.079	0.166	0.283	
Max Upward from Load Cases	0.132	0.277	0.471	
Max Downward from all Load Cases				
Max Downward from Load Combination				
Max Downward from Load Cases				
+0.60W	0.079	0.166	0.283	
+0.450W	0.059	0.125	0.212	
W Only	0.132	0.277	0.471	

Vertical Reactions for Load Cases

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from Load Cases	0.132	0.277	0.471	
W Only	0.132	0.277	0.471	

Vertical Reactions for Load Combinations

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from Load Combination	0.079	0.166	0.283	
+0.60W	0.079	0.166	0.283	
+0.450W	0.059	0.125	0.212	



Section Inputs

Material: A1003 ST Grade 50H

Apply cold work of forming strength increase.

No inelastic reserve strength increase.

Modulus of Elasticity, E 29500000 psi

Yield Strength, Fy 50000 psi

Tensile Strength, Fu 65000 psi

Min Elongation in 2 inches 10 %

Torsion Constant Override, J 0 in⁴

Warping Constant Override, Cw 0 in⁶

Hole Length 4

Hole Spacing 24

Stud, Thickness 0.0566 in (54 mil)

Placement of Part from Origin:

X to center of gravity 0 in

Y to center of gravity 0 in

Outside dimensions, Open shape

	Length (in)	Angle (deg)	Radius (in)	Web	k Coef.	Hole Size (in)	Distance (in)
1	0.5000	270.000	0.084900	None	0.000	0.0000	0.2500
2	1.6250	180.000	0.084900	Single	0.000	0.0000	0.8125
3	6.0000	90.000	0.084900	Cee	0.000	1.5000	3.0000
4	1.6250	0.000	0.084900	Single	0.000	0.0000	0.8125
5	0.5000	-90.000	0.084900	None	0.000	0.0000	0.2500

Member Check - AISI S100-24, US, ASD

Material Type: A1003 ST Grade 50H, Fy=50000 psi

Design Parameters:

Lx	14.500 ft	Ly	0.000 ft	Lt	0.000 ft
Kx	1.0000	Ky	1.0000	Kt	1.0000
Cbx	1.0000	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: None		k ϕ	0 lb		
Red. Factor, R: 0		Lm	14.500 ft		

Section: 600S162-54.cfss

R.V.

600S162-54, 50 ksi Stud

Brad Young & Associates, Inc.

SSMA Library

Rev. Date: 7/21/2025 8:15:00 AM

By: RSG Software

Printed: 7/29/2026 2:28:44 PM

Loads:	P (lb)	Mx (lb-ft)	Vy (lb)	My (lb-ft)	Vx (lb)
Entered	613.0	236.0	110.0	0.0	0.0
Applied	613.0	244.7	110.0	0.0	0.0
Strength	6526.7	1964.8	1803.2	402.8	2729.0

Interaction Equations

Eq. H1.2-1 (P, Mx, My) $0.094 + 0.125 + 0.000 = 0.218 \leq 1.0$ Eq. H2-1 (Mx, Vy) $\text{Sqrt}(0.011 + 0.004) = 0.123 \leq 1.0$ Eq. H2-1 (My, Vx) $\text{Sqrt}(0.000 + 0.000) = 0.000 \leq 1.0$ Calculation of Mndx used $\alpha_s=1$ from DSM inputs.Calculation of Mndy used $\alpha_s=0$ from DSM inputs.

Metal Wall Stud (Case 2) - Truss Bearing

Check the metal wall studs aligned under the roof truss taking the full weight of the truss

$$L_2 := (13 \cdot \text{ft} + 4 \cdot \text{in}) = 13.333 \text{ ft} \quad \text{Span from ground to bottom of joists}$$

$$w_{\text{trib}} := \frac{16 \text{ in} + 16 \text{ in}}{2} \quad w_{\text{trib}} = 1.333 \text{ ft}$$

$$RTU_1 := 3230 \cdot \text{lb} \quad RTU_2 := 1041 \cdot \text{lb}$$

Axial Loads on Wall Studs:

$$P_{DL} := 40 \text{ in} \cdot \left(DL_{\text{roof}} \cdot \frac{44.417 \text{ ft}}{2} \right) + \left[w_{\text{trib}} \cdot DL_{\text{wall.metal}} \cdot (23.50 \text{ ft}) \right] \dots \quad P_{DL} = 2567 \text{ lb}$$
$$+ \left[\left(\frac{RTU_1}{4} \right) \cdot \left(\frac{44.417 \text{ ft} - 14.333 \text{ ft}}{44.417 \cdot \text{ft}} \right) \right] + \left[\left(\frac{RTU_2}{4} \right) \cdot \left(\frac{44.417 \text{ ft} - 32.50 \text{ ft}}{44.417 \cdot \text{ft}} \right) \right]$$

$$P_{LL} := 40 \text{ in} \cdot \left[LL_{\text{roof}} \cdot \left(\frac{44.417 \text{ ft}}{2} \right) \right] \quad P_{LL} = 1481 \text{ lb}$$

$$P_{SL} := 40 \text{ in} \cdot \left[LL_{\text{snow}} \cdot \left(\frac{44.417 \text{ ft}}{2} \right) + \frac{1}{2} \cdot DL_{\text{wall.metal}} \cdot (w_2) \right] \quad P_{SL} = 1638 \text{ lb}$$

$$Total_{\text{max.C1}} := P_{DL} + \max(P_{LL}, P_{SL}) \quad Total_{\text{max.C1}} = 4205 \text{ lb}$$

Wind Loads on Studs:

$$p_{\text{wall.1}} = 15.038 \cdot \text{psf} \quad w_{\text{wall}} := \max \left[(p_{\text{wall.1}} \cdot w_{\text{trib}}), (|p_{\text{wall.2}}| \cdot w_{\text{trib}}) \right] \quad w_{\text{wall}} = 21.997 \cdot \text{plf}$$

$$p_{\text{wall.2}} = -16.498 \cdot \text{psf}$$

$$V_{\text{stud.WL}} := \frac{0.6 \cdot (w_{\text{wall}} \cdot L_2)}{2} \quad V_{\text{stud.WL}} = 87.99 \text{ lb}$$

$$M_{\text{stud.WL}} := \frac{0.6 \cdot (w_{\text{wall}} \cdot L_2^2)}{8} \quad M_{\text{stud.WL}} = 293.30 \text{ lb} \cdot \text{ft}$$

See attached metal wall stud design for axial and wind pressure forces:

BRAD YOUNG & ASSOCIATES

STRUCTURAL ENGINEERING

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Print Date: 8/3/2026

File Name: 251023 - Metal Stud.xmcd

Engineer: RV

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Case 2 - Truss Bearing

Code References

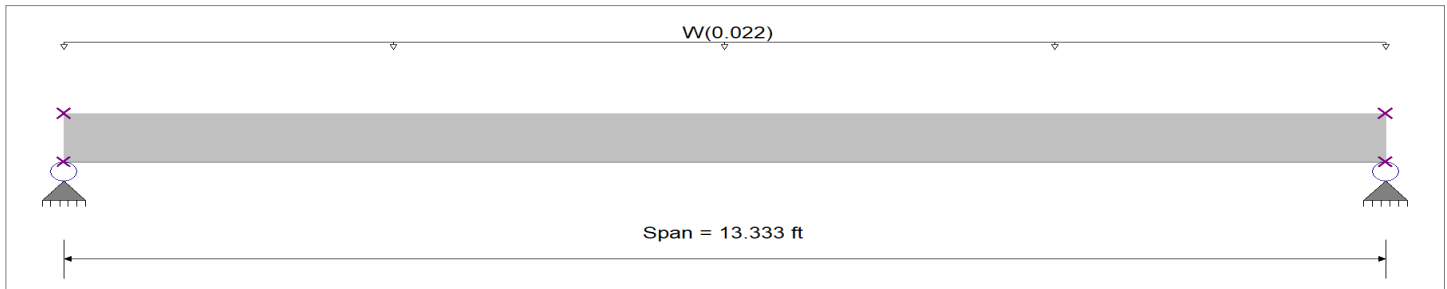
Governing Code : IBC 2018, CBC 2019

Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Beam Properties

Elastic Modulus 29,000.0 ksi
Span #1 Span Length = 13.333 ft Area = 0.3068 in^2 Moment of Inertia = 2.799 in^4



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Uniform Load : $W = 0.0220$ k/ft, Tributary Width = 1.0 ft, (Wind Pressure)

DESIGN SUMMARY

Maximum Bending =		0.293 k-ft	Maximum Shear =		0.0880 k
Load Combination			Load Combination		
+0.60W			+0.60W		
Span # where maximum occurs		Span # 1	Span # where maximum occurs		Span # 1
Location of maximum on span		6.667 ft	Location of maximum on span		13.333 ft
Maximum Deflection		Deflection	Ratio	Span	Load Combo
Max Downward Transient Deflection		0.194 in	823	Span #1	W Only
Max Upward Transient Deflection		0.000 in	0	Span #2	
Max Downward Total Deflection		0.087 in	1830	Span #1	+0.450W
Max Upward Total Deflection		0.000 in	0	Span #2	

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)					Shear Values (k)	
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega Cb	Rm	Va Max
Overall MAXimum Envelope										
Dsgn. L = 13.33 ft	1			0.29		0.29				0.09
Dsgn. L = 13.33 ft	1									-0.00
+0.60W										
Dsgn. L = 13.33 ft	1			0.29		0.29				0.09
+0.450W										
Dsgn. L = 13.33 ft	1			0.22		0.22				0.07

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.1943	6.733		0.0000	0.000

Vertical Reactions

Load Combination	Support 1	Support 2
Max Upward from all Load Cases	0.147	0.147
Max Upward from Load Combination	0.088	0.088
Max Upward from Load Cases	0.147	0.147
Max Downward from all Load Cases		
Max Downward from Load Combination		
Max Downward from Load Cases		
+0.60W	0.088	0.088
+0.450W	0.066	0.066
W Only	0.147	0.147

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

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DESCRIPTION: Case 2 - Truss Bearing

Vertical Reactions for Load Cases

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Cases	0.147	0.147
W Only	0.147	0.147

Vertical Reactions for Load Combinations

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Combinations	0.088	0.088
+0.60W	0.088	0.088
+0.450W	0.066	0.066



Section Inputs

Material: A1003 ST Grade 50H

Apply cold work of forming strength increase.

No inelastic reserve strength increase.

Modulus of Elasticity, E 29500000 psi

Yield Strength, Fy 50000 psi

Tensile Strength, Fu 65000 psi

Min Elongation in 2 inches 10 %

Torsion Constant Override, J 0 in⁴

Warping Constant Override, Cw 0 in⁶

Hole Length 4

Hole Spacing 24

Stud, Thickness 0.0566 in (54 mil)

Placement of Part from Origin:

X to center of gravity 0 in

Y to center of gravity 0 in

Outside dimensions, Open shape

	Length (in)	Angle (deg)	Radius (in)	Web	k Coef.	Hole Size (in)	Distance (in)
1	0.5000	270.000	0.084900	None	0.000	0.0000	0.2500
2	1.6250	180.000	0.084900	Single	0.000	0.0000	0.8125
3	6.0000	90.000	0.084900	Cee	0.000	1.5000	3.0000
4	1.6250	0.000	0.084900	Single	0.000	0.0000	0.8125
5	0.5000	-90.000	0.084900	None	0.000	0.0000	0.2500

Member Check - AISI S100-24, US, ASD

Material Type: A1003 ST Grade 50H, Fy=50000 psi

Design Parameters:

Lx	13.333 ft	Ly	0.000 ft	Lt	0.000 ft
Kx	1.0000	Ky	1.0000	Kt	1.0000
Cbx	1.0000	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: None		kφ	0 lb		
Red. Factor, R: 0		Lm	13.333 ft		

Section: 600S162-54.cfss

600S162-54, 50 ksi Stud

SSMA Library

Rev. Date: 7/21/2025 8:15:00 AM

By: RSG Software

Printed: 7/29/2026 2:30:27 PM

R.V.

Brad Young & Associates, Inc.

Loads:	P (lb)	Mx (lb-ft)	Vy (lb)	My (lb-ft)	Vx (lb)
Entered	4205.0	88.0	294.0	0.0	0.0
Applied	4205.0	111.0	294.0	0.0	0.0
Strength	6847.5	1964.8	1803.2	402.8	2729.0

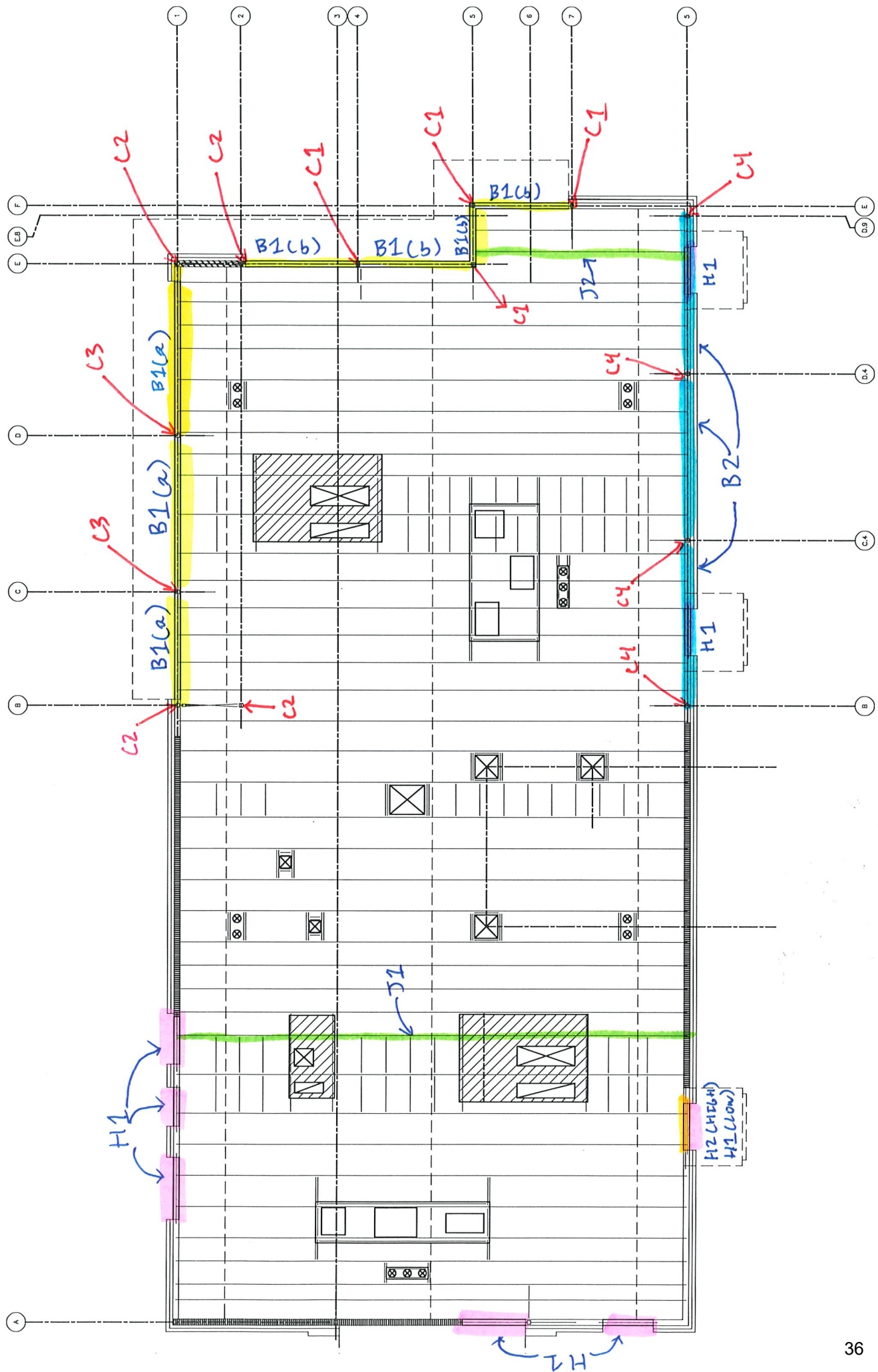
Interaction Equations

Eq. H1.2-1 (P, Mx, My) $0.614 + 0.056 + 0.000 = 0.671 \leq 1.0$ Eq. H2-1 (Mx, Vy) $\text{Sqrt}(0.002 + 0.027) = 0.170 \leq 1.0$ Eq. H2-1 (My, Vx) $\text{Sqrt}(0.000 + 0.000) = 0.000 \leq 1.0$ Calculation of Mndx used $\alpha_s=1$ from DSM inputs.Calculation of Mndy used $\alpha_s=0$ from DSM inputs.

Roof Framing

For the roof framing there will be pre-manufactured open web roof joists by others. The roof plan shows all locations of the mechanical equipment which will be supported by the roof framing members. The roof plans will be submitted to the joist manufacturer in order for all loads to be accounted for in the joist design.

ROOF FRAMING KEY PLAN



Member J1

Check the typ. worst case main roof joists and account for the weight of mechanical equipment.

$$L := (44 \cdot \text{ft} + 5 \cdot \text{in}) = 44.417 \text{ ft} \quad \text{max length}$$

$$\text{Trib}_{\text{roof}} := \frac{32 \text{ in} + 32 \text{ in}}{2} \quad \text{Tributary width} \quad \text{Trib}_{\text{roof}} = 2.667 \text{ ft}$$

$$F := 0.375 \quad \text{Roof slope (inches rise per foot of run)}$$

$$A_t := L \cdot (\text{Trib}_{\text{roof}}) \quad A_t = 118 \text{ ft}^2$$

$$L_0 := LL_{\text{roof}}$$

$$R_1 := \text{if} \left(A_t \leq 200 \cdot \text{ft}^2, 1, \text{if} \left(A_t < 600 \cdot \text{ft}^2, 1.2 - .001 \cdot \frac{A_t}{\text{ft}^2}, .6 \right) \right) \quad R_1 = 1$$

$$R_2 := \text{if}(F \leq 4, 1, \text{if}(F < 12, 1.2 - .05 \cdot F, .6)) \quad R_2 = 1$$

$$L_r := \text{if}(R_1 \cdot R_2 \leq 0.6, 0.6 \cdot L_0, R_1 \cdot R_2 \cdot L_0) \quad L_r = 20 \cdot \text{psf}$$

$$W_{DL} := DL_{\text{roof}} \quad W_{DL} = 20 \cdot \text{psf}$$

$$W_{LL} := L_r \quad W_{LL} = 20 \cdot \text{psf}$$

$$W_{SL} := LL_{\text{snow}} \quad W_{SL} = 20 \cdot \text{psf}$$

$$W_{SL.\text{drift}} := p_{d2} \cdot (\text{Trib}_{\text{roof}}) \quad \begin{array}{l} \text{at } L = 0 \text{ to } 6.285 \text{ ft \& } \\ 38.132 \text{ ft to } 44.417 \text{ ft} \end{array} \quad W_{SL.\text{drift}} = 70 \cdot \text{plf}$$

Add loads from HVAC:

$$P_{DL.RTU.1} := \frac{1041 \cdot \text{lb}}{3} \quad \text{at } L = 11.417 \text{ ft} \quad P_{DL.RTU.1} = 347 \text{ lb}$$

$$P_{DL.RTU.2} := \frac{3230 \cdot \text{lb}}{4} \quad \text{at } L = 29.833 \text{ ft} \quad P_{DL.RTU.2} = 808 \text{ lb}$$

Unfactored Wind Force per kicker

$$P_{\text{down}} := \max(P_{\text{down}.C1}, P_{\text{down}.C2}, P_{\text{down}.C3}) = 1262 \text{ lb} \quad \text{at } L = 4 \text{ ft}$$

$$P_{\text{uplift}} := \max(P_{\text{uplift}.C1}, P_{\text{uplift}.C2}, P_{\text{uplift}.C3}) = 745 \text{ lb} \quad \text{at } L = 40.417 \text{ ft}$$

Use 28"/36"/28" Red-S Pitched Open Web Truss @ 32" O.C.

See attached



RedSpec™ by RedBuilt™
v7.1.20

Project: McDonald's
Location: Kansas City, Kansas
Folder: Roof Joists
Date: 7/30/26 10:28 AM
Designer: R.V.
Comment:

Type: J1

PITCHED 28-36-28" Red-S™ @ 32" o.c.

This product meets or exceeds the set design controls for the application and loads listed

This truss design is feasible. The finished design shall be produced by RedBuilt Engineering. All open-web trusses are custom designed to carry the specific design loads for each project. Actual truss capacity when fabricated is limited to that required to resist the specific loads. Do not use this analysis to verify the capacity of existing trusses.

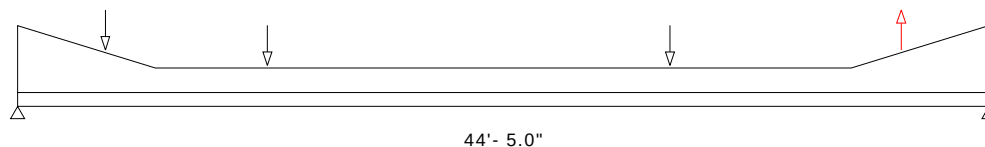
DEFLECTIONS (in)	%	Design	Allow.	Design	Allow.	Pass / Fail
Span Live	34%	0.755	2.221	L / 706	L / 240	PASS
Span Total	65%	1.924	2.961	L / 277	L / 180	PASS

SUPPORTS	Support 1	Support 2
Live Reaction (lb) (DOL %)	1551 (160)	1420 (115)
Dead Reaction (lb)	1723	1832
Total Reaction (lb) (DOL %)	3275 (160)	3252 (115)
Bearing	Top Chord	Top Chord
Support	Beam	Wall
Bearing Clip	(Red-S) S-Clip	(Red-S) S-Clip
Approx. Clip Height	3.5"	3.5"
Approx. Clip Width	5.5"	5.5"
Assumed Bearing Width	3.5"	3.5"

SPANS AND LOADS

Dimensions represent horizontal clear span.

Top Chord Slope: 0.36/12



APPLICATION LOADS

Type	Units	DOL	Live	Dead	Partition	Roaming	Tributary	Member Type
Uniform	psf	Snow(115%)	20	20	0	0 lb	32"	Snow Roof Joist

ADDITIONAL LOADS

Type	Units	DOL	Live	Dead	Location from left	Application	Comment
Point	lb	Snow(115%)	0	347	11'-5.0"	Adds To	RTU-1
Point	lb	Snow(115%)	0	808	29'-10.0"	Adds To	RTU-2
Point	lb	Wind(160%)	1262	0	4'-0.0"	Adds To	Kicker (Down)
Point	lb	Wind(160%)	-745	0	40'-5.0"	Adds To	Kicker (Uplift)
Tapered	plf	Snow(115%)	70 to 0	0 to 0	0'-0.0" to 6'-3.42"	Adds To	Snow Drift
Tapered	plf	Snow(115%)	0 to 70	0 to 0	38'-1.584" to 44'-5.0"	Adds To	Snow Drift

NOTES

- Building code and design methodology: 2018 IBC ASD (US).
- No repetitive member increase applied in design.
- Beveled plate required at left support.
- Beveled plate required at right support.
- Truss design includes consideration for partial span application live load.
- Continuous lateral support required at top edge. Lateral support at bottom edge shall be per RedBuilt recommendations.
- Pricing Load (plf) = 145
- Pricing Index (plf) = 182

O:\drawings\2025\251023\DWG\Calcs\251023.red

7/30/2026 10:28:25 AM

McDonald's : Roof Joists : J1

Page 1 of 1

The products noted are intended for interior, untreated, non-corrosive applications with normal temperatures and dry conditions of use, and must be installed in accordance with local building code requirements and RedBuilt™ recommendations. The loads, spans, and spacing have been provided by others and must be approved for the specific application by the design professional for the project. Unless otherwise noted, this output has not been reviewed by a RedBuilt™ associate. PRODUCT SUBSTITUTION VOIDS THIS ANALYSIS.

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Member J2

Check the shorter length roof joists.

$$L := 18.75 \cdot \text{ft} \quad \text{max length}$$

$$\text{Trib}_{\text{roof}} := \frac{32\text{in} + 32\text{in}}{2} \quad \text{Tributary width} \quad \text{Trib}_{\text{roof}} = 2.667 \text{ ft}$$

$$F := 0.375 \quad \text{Roof slope (inches rise per foot of run)}$$

$$A_t := L \cdot (\text{Trib}_{\text{roof}}) \quad A_t = 50 \text{ ft}^2$$

$$L_0 := LL_{\text{roof}}$$

$$R_1 := \text{if} \left(A_t \leq 200 \cdot \text{ft}^2, 1, \text{if} \left(A_t < 600 \cdot \text{ft}^2, 1.2 - .001 \cdot \frac{A_t}{\text{ft}^2}, .6 \right) \right) \quad R_1 = 1$$

$$R_2 := \text{if}(F \leq 4, 1, \text{if}(F < 12, 1.2 - .05 \cdot F, .6)) \quad R_2 = 1$$

$$L_r := \text{if}(R_1 \cdot R_2 \leq 0.6, 0.6 \cdot L_0, R_1 \cdot R_2 \cdot L_0) \quad L_r = 20 \cdot \text{psf}$$

$$W_{DL} := DL_{\text{roof}} \quad W_{DL} = 20 \cdot \text{psf}$$

$$W_{LL} := L_r \quad W_{LL} = 20 \cdot \text{psf}$$

$$W_{SL} := LL_{\text{snow}} \quad W_{SL} = 20 \cdot \text{psf}$$

$$W_{SL.\text{drift}} := p_{d2} \cdot (\text{Trib}_{\text{roof}}) \quad \text{at } L = 0 \text{ to } 6.285 \text{ ft \& } 12.465 \text{ ft to } 18.75 \text{ ft} \quad W_{SL.\text{drift}} = 70 \cdot \text{plf}$$

Unfactored Wind Force per kicker

$$P_{\text{down}} := \max(P_{\text{down.C1}}, P_{\text{down.C2}}, P_{\text{down.C3}}) = 1262 \text{ lb} \quad \text{at } L = 4 \text{ ft}$$

$$P_{\text{uplift}} := \max(P_{\text{uplift.C1}}, P_{\text{uplift.C2}}, P_{\text{uplift.C3}}) = 745 \text{ lb} \quad \text{at } L = 14.75 \text{ ft}$$

Use 28"/ 35" Red-S Tapered Open Web Truss @ 32" O.C.

See attached



RedSpec™ by RedBuilt™
v7.1.20

Project: McDonald's
Location: Kansas City, Kansas
Folder: Roof Joists
Date: 7/30/26 10:25 AM
Designer: R.V.
Comment:

Type: J2

TAPERED 28-35" Red-S™ @ 32" o.c.

This product meets or exceeds the set design controls for the application and loads listed

This truss design is feasible. The finished design shall be produced by RedBuilt Engineering. All open-web trusses are custom designed to carry the specific design loads for each project. Actual truss capacity when fabricated is limited to that required to resist the specific loads. Do not use this analysis to verify the capacity of existing trusses.

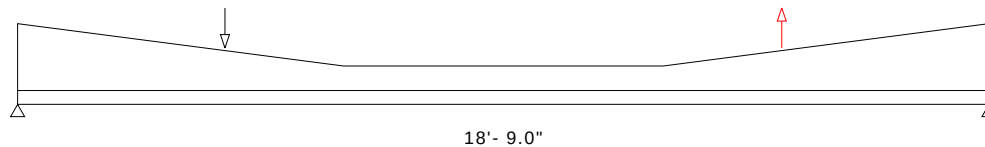
DEFLECTIONS (in)	%	Design	Allow.	Design	Allow.	Pass/Fail
Span Live	5%	0.050	0.938	L / 999+	L / 240	PASS
Span Total	7%	0.087	1.250	L / 999+	L / 180	PASS

SUPPORTS	Support 1	Support 2
Live Reaction (lb) (DOL %)	926 (160)	736 (115)
Dead Reaction (lb)	515	516
Total Reaction (lb) (DOL %)	1442 (160)	1252 (115)
Bearing Support	Top Chord Wall	Top Chord Beam
Bearing Clip	(Red-S) S-Clip	(Red-S) S-Clip
Approx. Clip Height	3.5"	3.5"
Approx. Clip Width	5.5"	5.5"
Assumed Bearing Width	3.5"	3.5"

SPANS AND LOADS

Dimensions represent horizontal clear span.

Top Chord Slope: 0.375/12 i



APPLICATION LOADS

Type	Units	DOL	Live	Dead	Partition	Roaming	Tributary	Member Type
Uniform	psf	Snow(115%)	20	20	0	0 lb	32"	Snow Roof Joist

ADDITIONAL LOADS

Type	Units	DOL	Live	Dead	Location from left	Application	Comment
Tapered	plf	Snow(115%)	70 to 0	0 to 0	0'-0.0" to 6'-3.42"	Adds To	Snow Drift
Tapered	plf	Snow(115%)	0 to 70	0 to 0	12'-5.58" to 18'-9.0"	Adds To	Snow Drift
Point	lb	Wind(160%)	1262	0	4'-0.0"	Adds To	Kicker (Down)
Point	lb	Wind(160%)	-745	0	14'-9.0"	Adds To	Kicker (Uplift)

NOTES

- Building code and design methodology: 2018 IBC ASD (US).
- No repetitive member increase applied in design.
- Beveled plate required at left support.
- Beveled plate required at right support.
- Truss design includes consideration for partial span application live load.
- Continuous lateral support required at top edge. Lateral support at bottom edge shall be per RedBuilt recommendations.
- Pricing Load (plf) = 139
- Pricing Index (plf) = 162

O:\drawings\2025\251023\DWG\Calcs\251023.red

7/30/2026 10:25:34 AM

McDonald's : Roof Joists : J2

Page 1 of 1

The products noted are intended for interior, untreated, non-corrosive applications with normal temperatures and dry conditions of use, and must be installed in accordance with local building code requirements and RedBuilt™ recommendations. The loads, spans, and spacing have been provided by others and must be approved for the specific application by the design professional for the project. Unless otherwise noted, this output has not been reviewed by a RedBuilt™ associate. PRODUCT SUBSTITUTION VOIDS THIS ANALYSIS.

RedBuilt™, RedSpec™, Red-I™, Red-I45™, Red-I45L™, Red-I58™, Red-I65™, Red-I90™, Red-I90H™, Red-I90HS™, Red-L™, Red-W™, Red-S™, Red-M™, Red-H™, RedLam™, FloorChoice™ are trademarks of RedBuilt LLC, Boise ID, USA. Copyright © 2010-2026 RedBuilt LLC. All rights reserved.

Member H1

$$L := (5\text{ft} + 4\text{in}) = 5.333\text{ft} \quad \text{max length}$$

$$\text{Trib}_{\text{roof}} := \frac{45\text{ft} + 5\text{in}}{2} \quad \text{Tributary width} \quad \text{Trib}_{\text{roof}} = 22.708\text{ft}$$

$$F := 0.375 \quad \text{Roof slope (inches rise per foot of run)}$$

$$A_t := L \cdot (\text{Trib}_{\text{roof}} + 4.333\text{ft}) \quad A_t = 144\text{ft}^2$$

$$L_0 := LL_{\text{roof}}$$

$$R_1 := \text{if} \left(A_t \leq 200 \cdot \text{ft}^2, 1, \text{if} \left(A_t < 600 \cdot \text{ft}^2, 1.2 - .001 \cdot \frac{A_t}{\text{ft}^2}, .6 \right) \right) \quad R_1 = 1$$

$$R_2 := \text{if}(F \leq 4, 1, \text{if}(F < 12, 1.2 - .05 \cdot F, .6)) \quad R_2 = 1$$

$$L_r := \text{if}(R_1 \cdot R_2 \leq 0.6, 0.6 \cdot L_0, R_1 \cdot R_2 \cdot L_0) \quad L_r = 20 \cdot \text{psf}$$

$$W_{DL} := DL_{\text{roof}} \cdot \text{Trib}_{\text{roof}} + 4.333\text{ft} \cdot DL_{\text{trellis}} + (DL_{\text{wall.metal}} + DL_{\text{wall.veneer}}) \cdot (21.50 \cdot \text{ft} - 7.333 \cdot \text{ft}) \quad W_{DL} = 1255 \cdot \text{plf}$$

$$W_{LL} := L_r \cdot (\text{Trib}_{\text{roof}} + 4.333\text{ft}) \quad W_{LL} = 541 \cdot \text{plf}$$

$$W_{SL} := LL_{\text{snow}} \cdot (\text{Trib}_{\text{roof}} + 4.333\text{ft}) + \left(p_{d2} \cdot \frac{w_2}{2} \right) \quad W_{SL} = 623 \cdot \text{plf}$$

RTU-1 & RTU-2 units are supported by 3 joists, 1 of those are supported by H1. Divide mechanical loads by 3 and multiply the load by the fraction to determine the load onto H1.

$$W_{\text{RTU.1}} := \left(\frac{1041\text{lb}}{3} \right) \cdot \left[\frac{(45\text{ft} + 5\text{in}) - (11\text{ft} + 5\text{in})}{(45\text{ft} + 5\text{in})} \right] \quad W_{\text{RTU.1}} = 260\text{lb} \quad L = 2\text{ft } 8\text{in}$$

$$W_{\text{RTU.2}} := \left(\frac{3230\text{lb}}{4} \right) \cdot \left[\frac{(45\text{ft} + 5\text{in}) - (29\text{ft} + 10\text{in})}{(45\text{ft} + 5\text{in})} \right] \quad W_{\text{RTU.2}} = 277\text{lb} \quad L = 2\text{ft } 8\text{in}$$

Unfactored Wind Force per kicker

$$P_{\text{down}} := \max(P_{\text{down.C1}}, P_{\text{down.C2}}, P_{\text{down.C3}}) = 1262\text{lb} \quad \text{at } L = 2\text{ft } 8\text{in}$$

Use (N) 5-1/4 x 9-1/2 LVL Header

See attached

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - H1

Code References

Governing Code : IBC 2018, CBC 2019

Referenced Design Standard(s) : NDS 2018

Load Combination Set : ASCE 7-16

Rho (or Omega) = 1.00

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : ASCE 7-16

Wood Species : RedBuilt

Wood Grade : RedLam LVL Beam/Joist

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb + 2,900.0 psi

Fb - 2,900.0 psi

Fc - Prll 2,635.0 psi

Fc - Perp 750.0 psi

Fv 285.0 psi

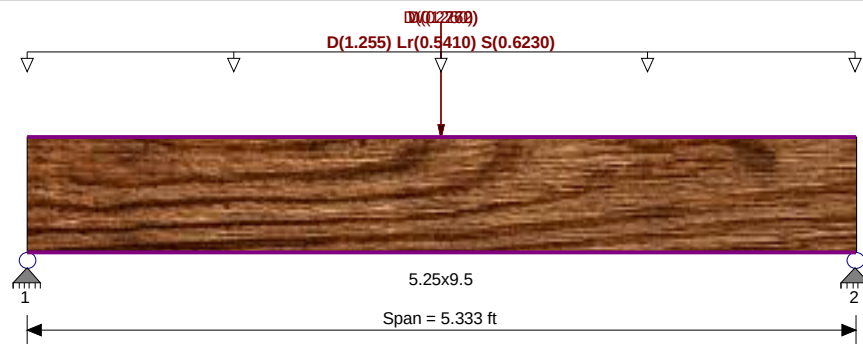
Ft 1,660.0 psi

E : Modulus of Elasticity

Ebend- xx 2,000.0ksi

Eminbend - xx 1,036.83ksi

Density 42.010pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading

Uniform Load : D = 1.255, Lr = 0.5410, S = 0.6230, Tributary Width = 1.0 ft, (Total Load)

Point Load : D = 0.260 k @ 2.667 ft, (RTU-1)

Point Load : D = 0.2770 k @ 2.667 ft, (RTU-2)

Point Load : W = 1.262 k @ 2.667 ft, (Kicker Load)

DESIGN SUMMARY

Bending about major axis (x-x)

Design OK

Maximum Bending Stress Ratio = 0.329 : 1

Section used for this span 5.25x9.5

NDS2018

(Major Axis - Strong Direction)

fb: Actual = 1,131.19psi

F'b = 3,442.66psi

Load Combination

+D+S

Location of maximum on span = 2.667ft

Span # where maximum occurs = Span # 1

Maximum Shear Stress Ratio = 0.353 : 1

Section used for this span 5.25x9.5

NDS2018

fv: Actual = 115.54 psi

F'v = 327.75 psi

Load Combination

+D+S

Location of maximum on span = 4.554 ft

Span # where maximum occurs = Span # 1

Maximum Deflection

Max Downward Transient Deflection 0.015 in Ratio = 4209 >=240 Span: 1 : S Only

Max Upward Transient Deflection 0 in Ratio = 0 >=240 n/a

Max Downward Total Deflection 0.050 in Ratio = 1268 >=180 Span: 1 : +D+0.750S+0.450W

Max Upward Total Deflection 0 in Ratio = 0 >=180 n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 5.333 ft	1	0.295	0.313	0.90	1.00	1.00	1.00	1.032	1.00	1.00	1.00	5.23	794.6	2,694.3	2.67	80.2	256.5
+D+Lr														0.0	0.00	0.0	0.0
Length = 5.333 ft	1	0.290	0.311	1.25	1.00	1.00	1.00	1.032	1.00	1.00	1.00	7.15	1,086.9	3,742.0	3.69	110.9	356.3

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - H1

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+S						1.00	1.00	1.00	1.032	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.333 ft	1		0.329	0.353	1.15	1.00	1.00	1.00	1.032	1.00	1.00	1.00	7.44	1,131.2	3,442.7	3.84	115.5	327.8
+D+0.750Lr						1.00	1.00	1.00	1.032	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.333 ft	1		0.271	0.290	1.25	1.00	1.00	1.00	1.032	1.00	1.00	1.00	6.67	1,013.8	3,742.0	3.43	103.2	356.3
+D+0.750S						1.00	1.00	1.00	1.032	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.333 ft	1		0.304	0.326	1.15	1.00	1.00	1.00	1.032	1.00	1.00	1.00	6.89	1,047.0	3,442.7	3.55	106.7	327.8
+D+0.60W						1.00	1.00	1.00	1.032	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.333 ft	1		0.198	0.201	1.60	1.00	1.00	1.00	1.032	1.00	1.00	1.00	6.24	948.0	4,789.8	3.04	91.6	456.0
+D+0.750Lr+0.450W						1.00	1.00	1.00	1.032	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.333 ft	1		0.236	0.245	1.60	1.00	1.00	1.00	1.032	1.00	1.00	1.00	7.43	1,128.9	4,789.8	3.72	111.7	456.0
+D+0.750S+0.450W						1.00	1.00	1.00	1.032	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.333 ft	1		0.243	0.253	1.60	1.00	1.00	1.00	1.032	1.00	1.00	1.00	7.65	1,162.1	4,789.8	3.83	115.2	456.0
+0.60D+0.60W						1.00	1.00	1.00	1.032	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.333 ft	1		0.132	0.130	1.60	1.00	1.00	1.00	1.032	1.00	1.00	1.00	4.15	630.2	4,789.8	1.98	59.5	456.0
+0.60D						1.00	1.00	1.00	1.032	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.333 ft	1		0.100	0.105	1.60	1.00	1.00	1.00	1.032	1.00	1.00	1.00	3.14	476.8	4,789.8	1.60	48.1	456.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+0.750S+0.450W	0.0505	2.686		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Cases & Combinations	5.315	5.315
Max Upward from Load Combinations	5.315	5.315
Max Upward from Load Cases	3.654	3.654
D Only	3.654	3.654
+D+Lr	5.096	5.096
+D+S	5.315	5.315
+D+0.750Lr	4.736	4.736
+D+0.750S	4.900	4.900
+D+0.60W	4.032	4.032
+D+0.750Lr+0.450W	5.020	5.020
+D+0.750S+0.450W	5.184	5.184
+0.60D+0.60W	2.571	2.571
+0.60D	2.192	2.192
Lr Only	1.443	1.443
S Only	1.661	1.661
W Only	0.631	0.631

Vertical Reactions for Load Cases

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Cases	3.654	3.654
D Only	3.654	3.654
Lr Only	1.443	1.443
S Only	1.661	1.661
W Only	0.631	0.631

Vertical Reactions for Load Combinations

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Combinations	5.315	5.315
+D+Lr	5.096	5.096
+D+S	5.315	5.315
+D+0.750Lr	4.736	4.736
+D+0.750S	4.900	4.900

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - H1

Vertical Reactions for Load Combinations

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.60W	4.032	4.032
+D+0.750Lr+0.450W	5.020	5.020
+D+0.750S+0.450W	5.184	5.184
+0.60D+0.60W	2.571	2.571
+0.60D	2.192	2.192

Extreme Reactions

Values in KIPS

Support	D	Lr	L	S	W	E	H
Support #1	3.654	1.443		1.661	0.631		
Support #2	3.654	1.443		1.661	0.631		

Member H2 (High)

Check the header above H1 at the drive thru supporting the canopy.

$$L := (6\text{ft} + 8\text{in}) = 6.667\text{ft} \quad \text{max length}$$

$$\text{Trib}_{\text{roof}} := \frac{45\text{ft} + 5\text{in}}{2} \quad \text{Tributary width} \quad \text{Trib}_{\text{roof}} = 22.708\text{ft}$$

$$F := 0.375 \quad \text{Roof slope (inches rise per foot of run)}$$

$$A_t := L \cdot (\text{Trib}_{\text{roof}} + 4.333\text{ft}) \quad A_t = 180\text{ft}^2$$

$$L_0 := LL_{\text{roof}}$$

$$R_1 := \text{if} \left(A_t \leq 200 \cdot \text{ft}^2, 1, \text{if} \left(A_t < 600 \cdot \text{ft}^2, 1.2 - .001 \cdot \frac{A_t}{\text{ft}^2}, .6 \right) \right) \quad R_1 = 1$$

$$R_2 := \text{if}(F \leq 4, 1, \text{if}(F < 12, 1.2 - .05 \cdot F, .6)) \quad R_2 = 1$$

$$L_r := \text{if}(R_1 \cdot R_2 \leq 0.6, 0.6 \cdot L_0, R_1 \cdot R_2 \cdot L_0) \quad L_r = 20 \cdot \text{psf}$$

$$W_{\text{DL.H2}} := [DL_{\text{roof}}(\text{Trib}_{\text{roof}} + 4.333\text{ft})] + DL_{\text{trellis}} \cdot (4.333\text{ft}) \dots \quad W_{\text{DL.H2}} = 1342 \cdot \text{plf}$$
$$+ (DL_{\text{wall.metal}} + DL_{\text{wall.veneer}}) \cdot (21.50\text{ft} - 7.333\text{ft})$$

$$W_{\text{LL.H2}} := L_r \cdot (\text{Trib}_{\text{roof}} + 4.333\text{ft}) \quad W_{\text{LL.H2}} = 541 \cdot \text{plf}$$

$$W_{\text{SL.H2}} := [LL_{\text{snow}} \cdot (\text{Trib}_{\text{roof}} + 4.33\text{ft})] + \left(\frac{1}{2} \cdot p_{d2} \cdot w_2 \right) \quad W_{\text{SL.H2}} = 623 \cdot \text{plf}$$

RTU-1 & RTU-2 units are supported by 3 joists, 1 of those are supported by H1. Divide mechanical loads by 3 and multiply the load by the fraction to determine the load onto H1.

$$W_{\text{RTU.1}} := \left(\frac{1041\text{lb}}{3} \right) \cdot \left[\frac{(45\text{ft} + 5\text{in}) - (29\text{ft} + 10\text{in})}{(45\text{ft} + 5\text{in})} \right] \quad W_{\text{RTU.1}} = 119\text{lb} \quad L = 3\text{ft } 4\text{in}$$

$$W_{\text{RTU.2}} := \left(\frac{3230\text{lb}}{4} \right) \cdot \left[\frac{(45\text{ft} + 5\text{in}) - (11\text{ft} + 5\text{in})}{(45\text{ft} + 5\text{in})} \right] \quad W_{\text{RTU.2}} = 605\text{lb} \quad L = 3\text{ft } 4\text{in}$$

Unfactored Wind Force per kicker

$$P_{\text{down}} := \max(P_{\text{down.C1}}, P_{\text{down.C2}}, P_{\text{down.C3}}) = 1262\text{lb} \quad \text{at } L = 3\text{ft } 4\text{in}$$

Use (N) 5-1/4 x 16 LVL Header

See attached

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - H2 (High)

Code References

Governing Code : IBC 2018, CBC 2019

Referenced Design Standard(s) : NDS 2018

Load Combination Set : ASCE 7-16

Rho (or Omega) = 1.00

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : ASCE 7-16

Wood Species : RedBuilt

Wood Grade : RedLam LVL Beam/Joist

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

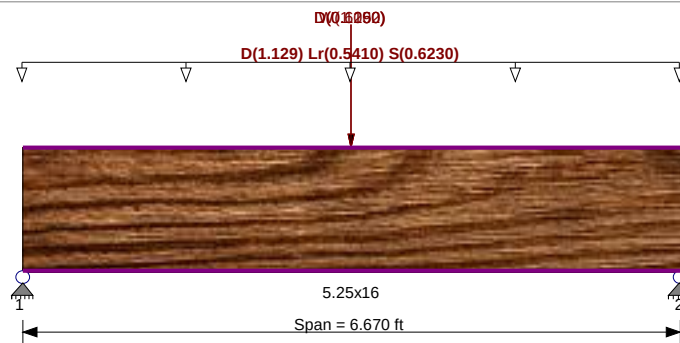
Fb + 2,900.0 psi
 Fb - 2,900.0 psi
 Fc - Prll 2,635.0 psi
 Fc - Perp 750.0 psi
 Fv 285.0 psi
 Ft 1,660.0 psi

E : Modulus of Elasticity

Ebend- xx 2,000.0ksi

Eminbend - xx 1,036.83ksi

Density 42.010pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading

Uniform Load : D = 1.129, Lr = 0.5410, S = 0.6230, Tributary Width = 1.0 ft, (Total Load)

Point Load : D = 0.1190 k @ 3.333 ft, (RTU-1)

Point Load : D = 0.6050 k @ 3.333 ft, (RTU-2)

Point Load : W = 1.262 k @ 3.333 ft, (Kicker (Down))

DESIGN SUMMARY

Bending about major axis (x-x)

Design OK

Maximum Bending Stress Ratio = 0.185 : 1

Section used for this span

5.25x16

NDS2018

(Major Axis - Strong Direction)

fb: Actual = 593.89psi

F'b = 3,207.04psi

Load Combination

+D+S

Location of maximum on span = 3.335ft

Span # where maximum occurs = Span # 1

Maximum Shear Stress Ratio = 0.215 : 1

Section used for this span

5.25x16

NDS2018

fv: Actual = 70.56 psi

F'v = 327.75 psi

Load Combination

+D+S

Location of maximum on span = 0.000 ft

Span # where maximum occurs = Span # 1

Maximum Deflection

Max Downward Transient Deflection 0.008 in Ratio = 10279 >=240 Span: 1 : S Only

Max Upward Transient Deflection 0 in Ratio = 0 >=240 n/a

Max Downward Total Deflection 0.024 in Ratio = 3283 >=180 Span: 1 : +D+S

Max Upward Total Deflection 0 in Ratio = 0 >=180 n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 6.670 ft	1	0.163	0.187	0.90	1.00	1.00	1.00	0.962	1.00	1.00	1.00	7.62	408.3	2,509.9	2.69	48.1	256.5
+D+Lr														0.0	0.00	0.0	0.0
Length = 6.670 ft	1	0.163	0.190	1.25	1.00	1.00	1.00	0.962	1.00	1.00	1.00	10.63	569.5	3,485.9	3.79	67.6	356.3

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Member - H2 (High)

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios									Moment Values			Shear Values			
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+S						1.00	1.00	1.00	0.962	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.670 ft	1		0.185	0.215	1.15	1.00	1.00	1.00	0.962	1.00	1.00	1.00	11.09	593.9	3,207.0	3.95	70.6	327.8
+D+0.750Lr						1.00	1.00	1.00	0.962	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.670 ft	1		0.152	0.176	1.25	1.00	1.00	1.00	0.962	1.00	1.00	1.00	9.88	529.2	3,485.9	3.51	62.7	356.3
+D+0.750S						1.00	1.00	1.00	0.962	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.670 ft	1		0.171	0.198	1.15	1.00	1.00	1.00	0.962	1.00	1.00	1.00	10.22	547.5	3,207.0	3.64	64.9	327.8
+D+0.60W						1.00	1.00	1.00	0.962	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.670 ft	1		0.107	0.120	1.60	1.00	1.00	1.00	0.962	1.00	1.00	1.00	8.88	475.9	4,462.0	3.07	54.9	456.0
+D+0.750Lr+0.450W						1.00	1.00	1.00	0.962	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.670 ft	1		0.130	0.149	1.60	1.00	1.00	1.00	0.962	1.00	1.00	1.00	10.82	579.9	4,462.0	3.80	67.8	456.0
+D+0.750S+0.450W						1.00	1.00	1.00	0.962	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.670 ft	1		0.134	0.154	1.60	1.00	1.00	1.00	0.962	1.00	1.00	1.00	11.17	598.2	4,462.0	3.92	70.0	456.0
+0.60D+0.60W						1.00	1.00	1.00	0.962	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.670 ft	1		0.070	0.078	1.60	1.00	1.00	1.00	0.962	1.00	1.00	1.00	5.83	312.6	4,462.0	1.99	35.6	456.0
+0.60D						1.00	1.00	1.00	0.962	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.670 ft	1		0.055	0.063	1.60	1.00	1.00	1.00	0.962	1.00	1.00	1.00	4.57	245.0	4,462.0	1.62	28.9	456.0

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+S	0.0244	3.335		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Cases & Combinations	6.287	6.286
Max Upward from Load Combinations	6.287	6.286
Max Upward from Load Cases	4.209	4.209
D Only	4.209	4.209
+D+Lr	6.013	6.013
+D+S	6.287	6.286
+D+0.750Lr	5.562	5.562
+D+0.750S	5.767	5.767
+D+0.60W	4.588	4.587
+D+0.750Lr+0.450W	5.846	5.846
+D+0.750S+0.450W	6.052	6.051
+0.60D+0.60W	2.904	2.904
+0.60D	2.525	2.525
Lr Only	1.804	1.804
S Only	2.078	2.078
W Only	0.631	0.631

Vertical Reactions for Load Cases

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Cases	4.209	4.209
D Only	4.209	4.209
Lr Only	1.804	1.804
S Only	2.078	2.078
W Only	0.631	0.631

Vertical Reactions for Load Combinations

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Combinations	6.287	6.286
+D+Lr	6.013	6.013
+D+S	6.287	6.286
+D+0.750Lr	5.562	5.562
+D+0.750S	5.767	5.767

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - H2 (High)

Vertical Reactions for Load Combinations

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.60W	4.588	4.587
+D+0.750Lr+0.450W	5.846	5.846
+D+0.750S+0.450W	6.052	6.051
+0.60D+0.60W	2.904	2.904
+0.60D	2.525	2.525

Extreme Reactions

Values in KIPS

Support	D	Lr	L	S	W	E	H
Support #1	4.209	1.804		2.078	0.631		
Support #2	4.209	1.804		2.078	0.631		

Member B1(a)

$$L := (14\text{ft} + 8\text{in}) = 14.667\text{ ft} \quad \text{max length}$$

$$\text{Trib}_{\text{roof}} := \frac{44\text{ft} + 5\text{in}}{2} \quad \text{Tributary width} \quad \text{Trib}_{\text{roof}} = 22.208\text{ ft}$$

$$F := 0.375 \quad \text{Roof slope (inches rise per foot of run)}$$

$$A_t := L \cdot (\text{Trib}_{\text{roof}}) \quad A_t = 326\text{ ft}^2$$

$$L_0 := LL_{\text{roof}}$$

$$R_1 := \text{if} \left(A_t \leq 200 \cdot \text{ft}^2, 1, \text{if} \left(A_t < 600 \cdot \text{ft}^2, 1.2 - .001 \cdot \frac{A_t}{\text{ft}^2}, .6 \right) \right) \quad R_1 = 0.874$$

$$R_2 := \text{if}(F \leq 4, 1, \text{if}(F < 12, 1.2 - .05 \cdot F, .6)) \quad R_2 = 1$$

$$L_r := \text{if}(R_1 \cdot R_2 \leq 0.6, 0.6 \cdot L_0, R_1 \cdot R_2 \cdot L_0) \quad L_r = 17.486 \cdot \text{psf}$$

$$W_{\text{DL}.1} := DL_{\text{roof}} \cdot \text{Trib}_{\text{roof}} + DL_{\text{wall.metal}} \cdot (23.50 \cdot \text{ft} - 10.667 \cdot \text{ft}) \quad W_{\text{DL}.1} = 637 \cdot \text{plf}$$

$$W_{\text{DL}.2} := DL_{\text{wall.veneer}} \cdot (23.50 \cdot \text{ft} - 10.667 \cdot \text{ft}) \quad e := 6\text{in} \quad W_{\text{DL}.2} = 513 \cdot \text{plf}$$

$$W_{\text{DL}.3} := DL_{\text{trellis}} \cdot 3\text{ft} \quad e := 2\text{ft} \quad W_{\text{DL}.3} = 15 \cdot \text{plf}$$

$$W_{\text{LL}.1} := (L_r \cdot \text{Trib}_{\text{roof}}) \cdot R_1 \quad W_{\text{LL}.1} = 340 \cdot \text{plf}$$

$$W_{\text{LL}.3} := (L_r \cdot 3\text{ft}) \cdot R_1 \quad e := 2\text{ft} \quad W_{\text{LL}.3} = 46 \cdot \text{plf}$$

$$W_{\text{SL}.1} := LL_{\text{snow}} \cdot \text{Trib}_{\text{roof}} + \left(p_{d2} \cdot \frac{w2}{2} \right) \quad W_{\text{SL}.1} = 526 \cdot \text{plf}$$

$$W_{\text{SL}.3} := LL_{\text{snow}} \cdot 3\text{ft} \quad e := 2\text{ft} \quad W_{\text{SL}.3} = 60 \cdot \text{plf}$$

HVAC unit are each supported by 4 joists, 2, 2, and 2 of those joists, respectively, are supported by B1. Multiply the load by the fraction to determine the load onto B1.

$$W_{\text{RTU}.3} := \left(\frac{2655\text{lb}}{L} \right) \cdot \left[\frac{(45\text{ft} + 5\text{in}) - (12\text{ft} + 1\text{in})}{(45\text{ft} + 5\text{in})} \right] \quad W_{\text{RTU}.3} = 133 \cdot \text{plf}$$

$$W_{\text{Cond}.3} := \left[\frac{(260\text{lb} + 260\text{lb} + 100 \cdot \text{lb})}{L} \right] \cdot \left[\frac{(45\text{ft} + 5\text{in}) - (28\text{ft} + 4\text{in})}{(45\text{ft} + 5\text{in})} \right] \quad W_{\text{Cond}.3} = 16 \cdot \text{plf}$$

Use HSS 16 x 4 x 1/4"
OK See Attached

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam with Torsional Loads

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - B1(a) W/ Torsion

Code References

Governing Code : IBC 2018, CBC 2019

Referenced Design Standard(s) : AISC 360-16

Load Combination Set : ASCE 7-16

Rho (or Omega) = 1.00

Analysis Settings

Analysis Method : Allowable Strength Design

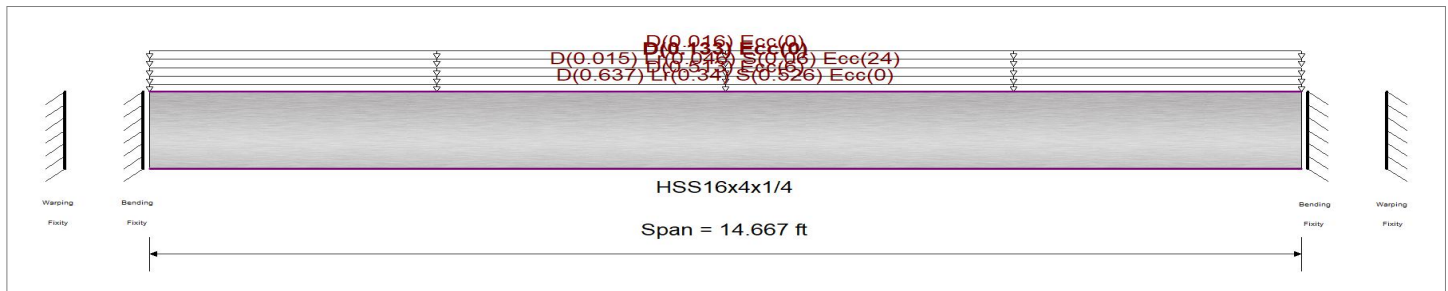
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Bending Axis : Major Axis Bending

Load Combination ASCE 7-16

Fy : Steel Yield : 46.0 ksi

E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads

Load for Span Number 0

Uniform Load : D = 0.6370, Lr = 0.340, S = 0.5260 k/ft, Tributary Width = 1.0 ft, (Roof Load + Wall)

Uniform Load : D = 0.5130 k/ft, Tributary Width = 1.0 ft, Tors Ecc. = 6.0 in, (Veneer)

Uniform Load : D = 0.0150, Lr = 0.0460, S = 0.060 k/ft, Tributary Width = 1.0 ft, Tors Ecc. = 24.0 in, (Trellis)

Uniform Load : D = 0.1330 k/ft, Tributary Width = 1.0 ft, (RTU-3)

Uniform Load : D = 0.0160 k/ft, Tributary Width = 1.0 ft, (Condensers)

DESIGN SUMMARY

Design OK

Max. Flange Normal Stress Ratio =		0.367 : 1	Maximum Shear Stress Ratio =		0.078 : 1
Flange Normal Stress		13.16 ksi	Flange Shear Stress		1.29 ksi
Fb / Ω : Flange Normal Stress		35.82 ksi	Web Shear Stress		3.20 ksi
[Mn/(Ω Sxx)]			Vn/Ω : Allowable		16.53 ksi
Max Mu : Applied		34.65 k-ft			
Section used for this span		HSS16x4x1/4	Section used for this span		HSS16x4x1/4
Load Combination			Load Combination		+D+6D+S
			Maximum Rotation =		0.0945 deg at 7.16 ft
Maximum Deflection					
Max Download Transient Load Deflection		0.017 in	Max Downward Total Deflection		0.055 in
Ratio =		10581	Ratio =		3208
Max Upward Transient Load Deflection		0.000 in	Max Upward Total Deflection		0.000 in
Ratio =		0 <360	Ratio =		0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination	Max Stress		Bending Max		Flange Normal (ksi)			Flange Shear (ksi)			Web Shear (ksi)		
	Ratio	Mu	(kft)	Vu (k)	Total	Allow	Ratio	Total	Allow	Ratio	Total	Allow	Ratio
D Only	0.256	24.14	9.88		9.168	35.823	0.256	0.913	16.527	0.055	2.237	16.527	0.135
+D+Lr	0.329	31.06	12.71		11.795	35.823	0.329	1.206	16.527	0.073	2.910	16.527	0.176
+D+S	0.367	34.65	14.17		13.157	35.823	0.367	1.295	16.527	0.078	3.196	16.527	0.193
+D+0.750Lr	0.311	29.33	12.00		11.138	35.823	0.311	1.132	16.527	0.069	2.742	16.527	0.166
+D+0.750S	0.339	32.02	13.10		12.159	35.823	0.339	1.199	16.527	0.073	2.956	16.527	0.179
+0.60D	0.154	14.48	5.93		5.501	35.823	0.154	0.548	16.527	0.033	1.342	16.527	0.081

Maximum Upward Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
span_1	1	0.0549	7.392		0.0000	0.000

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam with Torsional Loads

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - B1(a) W/ Torsion

Vertical Reactions - Unfactored

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	14.173	14.173
Max Upward from Load Combination	14.173	14.173
Max Upward from Load Cases	9.876	9.876
D Only	9.876	9.876
+D+Lr	12.706	12.706
+D+S	14.173	14.173
+D+0.750Lr	11.999	11.999
+D+0.750S	13.099	13.099
+0.60D	5.925	5.925
S Only	4.297	4.297

Member B1(b)

$$L := (10\text{ft} + 1\text{in}) = 10.083\text{ ft} \quad \text{max length}$$

$$\text{Trib}_{\text{roof}} := \frac{2\text{ft} + 3\text{in}}{2} \quad \text{Tributary width} \quad \text{Trib}_{\text{roof}} = 1.125\text{ ft}$$

$$F := 0.375 \quad \text{Roof slope (inches rise per foot of run)}$$

$$A_t := L \cdot (\text{Trib}_{\text{roof}}) \quad A_t = 11\text{ ft}^2$$

$$L_0 := LL_{\text{roof}}$$

$$R_1 := \text{if} \left(A_t \leq 200 \cdot \text{ft}^2, 1, \text{if} \left(A_t < 600 \cdot \text{ft}^2, 1.2 - .001 \cdot \frac{A_t}{\text{ft}^2}, .6 \right) \right) \quad R_1 = 1$$

$$R_2 := \text{if}(F \leq 4, 1, \text{if}(F < 12, 1.2 - .05 \cdot F, .6)) \quad R_2 = 1$$

$$L_r := \text{if}(R_1 \cdot R_2 \leq 0.6, 0.6 \cdot L_0, R_1 \cdot R_2 \cdot L_0) \quad L_r = 20 \cdot \text{psf}$$

$$W_{DL.1} := DL_{\text{roof}} \cdot \text{Trib}_{\text{roof}} + DL_{\text{wall.metal}} \cdot (23.50 \cdot \text{ft} - 10.667 \cdot \text{ft}) \quad W_{DL.1} = 215 \cdot \text{plf}$$

$$W_{DL.2} := DL_{\text{wall.veneer}} \cdot (23.50 \cdot \text{ft} - 10.667 \cdot \text{ft}) \quad e := 6\text{in} \quad W_{DL.2} = 513 \cdot \text{plf}$$

$$W_{DL.3} := DL_{\text{trellis}} \cdot 3\text{ft} \quad e := 2\text{ft} \quad W_{DL.3} = 15 \cdot \text{plf}$$

$$W_{LL.1} := (L_r \cdot \text{Trib}_{\text{roof}}) \cdot R_1 \quad W_{LL.1} = 22 \cdot \text{plf}$$

$$W_{LL.3} := (L_r \cdot 3\text{ft}) \cdot R_1 \quad e := 2\text{ft} \quad W_{LL.3} = 60 \cdot \text{plf}$$

$$W_{SL.1} := LL_{\text{snow}} \cdot \text{Trib}_{\text{roof}} + \left(p_{d1} \cdot \frac{w1}{2} \right) \quad W_{SL.1} = 205 \cdot \text{plf}$$

$$W_{SL.3} := LL_{\text{snow}} \cdot 3\text{ft} \quad e := 2\text{ft} \quad W_{SL.3} = 60 \cdot \text{plf}$$

Use HSS 16 x 4 x 1/4"
OK See Attached

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam with Torsional Loads

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - B1(b) W/ Torsion

Code References

Governing Code : IBC 2018, CBC 2019

Referenced Design Standard(s) : AISC 360-16

Load Combination Set : ASCE 7-16

Rho (or Omega) = 1.00

Analysis Settings

Analysis Method : Allowable Strength Design

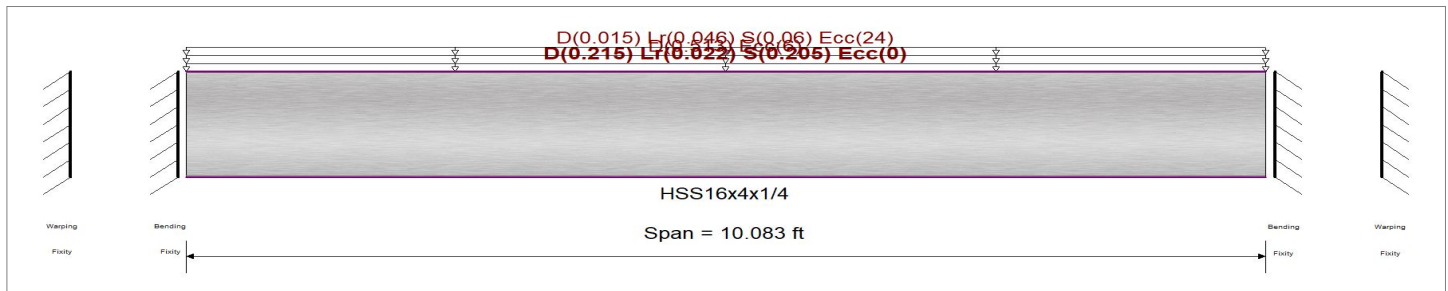
Fy : Steel Yield : 46.0 ksi

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending

Load Combination ASCE 7-16



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads

Load for Span Number 0

Uniform Load : D = 0.2150, Lr = 0.0220, S = 0.2050 k/ft, Tributary Width = 1.0 ft, (Roof Load + Wall)

Uniform Load : D = 0.5130 k/ft, Tributary Width = 1.0 ft, Tors Ecc. = 6.0 in, (Veneer)

Uniform Load : D = 0.0150, Lr = 0.0460, S = 0.060 k/ft, Tributary Width = 1.0 ft, Tors Ecc. = 24.0 in, (Trellis)

DESIGN SUMMARY

Design OK

Max. Flange Normal Stress Ratio = 0.093 : 1

Maximum Shear Stress Ratio = 0.054 : 1

Flange Normal Stress 3.35 ksi

Flange Shear Stress 0.89 ksi

Fb / Ω : Flange Normal Stress 35.82 ksi

Web Shear Stress 1.59 ksi

$[Mn/(\Omega Sxx)]$

Vn / Ω : Allowable 16.53 ksi

Max Mu : Applied 8.82 k-ft

Section used for this span HSS16x4x1/4

Section used for this span

HSS16x4x1/4

Load Combination

Load Combination

+D+6D+S

Maximum Deflection

Maximum Rotation = 0.0447 deg at 5.04 ft

Max Download Transient Load Deflection 0.002 in

Max Downward Total Deflection 0.007 in

Ratio = 72022

Ratio = 18340

Max Upward Transient Load Deflection 0.000 in

Max Upward Total Deflection 0.000 in

Ratio = 0 < 360

Ratio = 0 < 180

Maximum Forces & Stresses for Load Combinations

Load Combination	Max Stress		Bending Max		Flange Normal (ksi)			Flange Shear (ksi)			Web Shear (ksi)		
	Ratio	Mu	(kft)	Vu (k)	Total	Allow	Ratio	Total	Allow	Ratio	Total	Allow	Ratio
D Only	0.070		6.57	3.91	2.496	35.823	0.070	0.627	16.527	0.038	1.152	16.527	0.070
+D+Lr	0.085		7.15	4.25	2.714	35.823	0.076	0.829	16.527	0.050	1.399	16.527	0.085
+D+S	0.096		8.82	5.25	3.348	35.823	0.093	0.890	16.527	0.054	1.594	16.527	0.096
+D+0.750Lr	0.081		7.00	4.17	2.660	35.823	0.074	0.778	16.527	0.047	1.337	16.527	0.081
+D+0.750S	0.090		8.26	4.91	3.135	35.823	0.088	0.824	16.527	0.050	1.483	16.527	0.090
+0.60D	0.042		3.94	2.35	1.497	35.823	0.042	0.376	16.527	0.023	0.691	16.527	0.042

Maximum Upward Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
span_1	1	0.0066	5.082		0.0000	0.000

Vertical Reactions - Unfactored

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Condi	5.246	5.246

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DESCRIPTION: Member - B1(b) W/ Torsion

Vertical Reactions - Unfactored

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Combinat	5.246	5.246
Max Upward from Load Cases	3.910	3.910
D Only	3.910	3.910
+D+Lr	4.253	4.253
+D+S	5.246	5.246
+D+0.750Lr	4.168	4.168
+D+0.750S	4.912	4.912
+0.60D	2.346	2.346
S Only	1.336	1.336

Member B2 (Conservatively assume Trellis throughout beam)

$$L := (14\text{ft} + 3\text{in}) = 14.25\text{ft} \quad \text{max length}$$

$$\text{Trib}_{\text{roof}} := \frac{44\text{ft} + 5\text{in}}{2} \quad \text{Tributary width} \quad \text{Trib}_{\text{roof}} = 22.208\text{ft}$$

$$F := 0.375 \quad \text{Roof slope (inches rise per foot of run)}$$

$$A_t := L \cdot (\text{Trib}_{\text{roof}}) \quad A_t = 316\text{ft}^2$$

$$L_0 := LL_{\text{roof}}$$

$$R_1 := \text{if} \left(A_t \leq 200 \cdot \text{ft}^2, 1, \text{if} \left(A_t < 600 \cdot \text{ft}^2, 1.2 - .001 \cdot \frac{A_t}{\text{ft}^2}, .6 \right) \right) \quad R_1 = 0.884$$

$$R_2 := \text{if}(F \leq 4, 1, \text{if}(F < 12, 1.2 - .05 \cdot F, .6)) \quad R_2 = 1$$

$$L_r := \text{if}(R_1 \cdot R_2 \leq 0.6, 0.6 \cdot L_0, R_1 \cdot R_2 \cdot L_0) \quad L_r = 17.671 \cdot \text{psf}$$

$$W_{\text{DL}.1} := DL_{\text{roof}} \cdot \text{Trib}_{\text{roof}} + DL_{\text{wall.metal}} \cdot (23.50\text{ft} - 10.667\text{ft}) \quad W_{\text{DL}.1} = 637 \cdot \text{plf}$$

$$W_{\text{DL}.2} := DL_{\text{wall.veneer}} \cdot (23.50\text{ft} - 10.667\text{ft}) \quad e := 6\text{in} \quad W_{\text{DL}.2} = 513 \cdot \text{plf}$$

$$W_{\text{DL}.3} := DL_{\text{trellis}} \cdot 4.333\text{ft} \quad e := 3\text{ft} \quad W_{\text{DL}.3} = 22 \cdot \text{plf}$$

$$W_{\text{LL}.1} := (L_r \cdot \text{Trib}_{\text{roof}}) \cdot R_1 \quad W_{\text{LL}.1} = 347 \cdot \text{plf}$$

$$W_{\text{LL}.3} := (L_r \cdot 4.333\text{ft}) \cdot R_1 \quad e := 3\text{ft} \quad W_{\text{LL}.3} = 68 \cdot \text{plf}$$

$$W_{\text{SL}.1} := LL_{\text{snow}} \cdot \text{Trib}_{\text{roof}} + \left(p_{d2} \cdot \frac{w2}{2} \right) \quad W_{\text{SL}.1} = 526 \cdot \text{plf}$$

$$W_{\text{SL}.3} := LL_{\text{snow}} \cdot 4.333\text{ft} \quad e := 3\text{ft} \quad W_{\text{SL}.3} = 87 \cdot \text{plf}$$

HVAC unit are each supported by 4 joists, 2, 2, and 2 of those joists, respectively, are supported by B1. Multiply the load by the fraction to determine the load onto B2.

$$W_{\text{RTU}.3} := \left(\frac{2655\text{lb}}{L} \right) \cdot \left[\frac{(45\text{ft} + 5\text{in}) - (28\text{ft} + 4\text{in})}{(45\text{ft} + 5\text{in})} \right] \quad W_{\text{RTU}.3} = 70 \cdot \text{plf}$$

$$W_{\text{Cond}.3} := \left[\frac{(260\text{lb} + 260\text{lb} + 100 \cdot \text{lb})}{L} \right] \cdot \left[\frac{(45\text{ft} + 5\text{in}) - (12\text{ft} + 1\text{in})}{(45\text{ft} + 5\text{in})} \right] \quad W_{\text{Cond}.3} = 32 \cdot \text{plf}$$

Use HSS 12 x 4 x 1/4"
OK See Attached

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam with Torsional Loads

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - B2 W/ Torsion

Code References

Governing Code : IBC 2018, CBC 2019

Referenced Design Standard(s) : AISC 360-16

Load Combination Set : ASCE 7-16

Rho (or Omega) = 1.00

Analysis Settings

Analysis Method : Allowable Strength Design

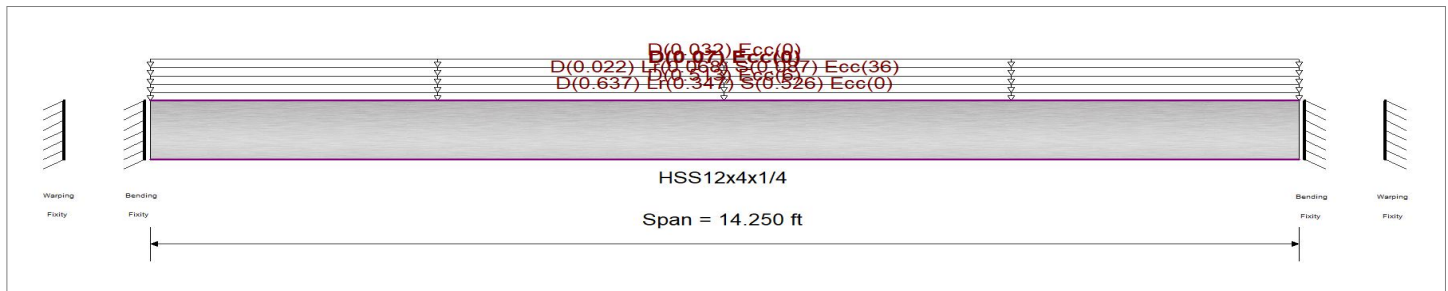
Fy : Steel Yield : 46.0 ksi

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending

Load Combination ASCE 7-16



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads

Load for Span Number 0

Uniform Load : D = 0.6370, Lr = 0.3470, S = 0.5260 k/ft, Tributary Width = 1.0 ft, (Roof Load + Wall)

Uniform Load : D = 0.5130 k/ft, Tributary Width = 1.0 ft, Tors Ecc. = 6.0 in, (Veneer)

Uniform Load : D = 0.0220, Lr = 0.0680, S = 0.0870 k/ft, Tributary Width = 1.0 ft, Tors Ecc. = 36.0 in, (Trellis)

Uniform Load : D = 0.070 k/ft, Tributary Width = 1.0 ft, (RTU-3)

Uniform Load : D = 0.0320 k/ft, Tributary Width = 1.0 ft, (Condensers)

DESIGN SUMMARY

Design OK

Max. Flange Normal Stress Ratio =	0.551 : 1	Maximum Shear Stress Ratio =	0.146 : 1
Flange Normal Stress	19.52 ksi	Flange Shear Stress	2.42 ksi
Fb / Ω : Flange Normal Stress	35.43 ksi	Web Shear Stress	4.86 ksi
$[Mn/(\Omega S_{xx})]$		Vn/ Ω : Allowable	16.53 ksi
Max Mu : Applied	32.37 k-ft		
Section used for this span	HSS12x4x1/4	Section used for this span	HSS12x4x1/4
Load Combination		Load Combination	+D+0.60D+S
Maximum Deflection		Maximum Rotation =	0.1825 deg at 7.13 ft
Max Download Transient Load Deflection	0.033 in	Max Downward Total Deflection	0.103 in
Ratio =	5188	Ratio =	1662
Max Upward Transient Load Deflection	0.000 in	Max Upward Total Deflection	0.000 in
Ratio =	0 < 360	Ratio =	0 < 180

Maximum Forces & Stresses for Load Combinations

Load Combination	Max Stress		Bending Max		Flange Normal (ksi)			Flange Shear (ksi)			Web Shear (ksi)		
	Ratio	Mu	(kft)	Vu (k)	Total	Allow	Ratio	Total	Allow	Ratio	Total	Allow	Ratio
D Only	0.374	22.00	9.26		13.264	35.435	0.374	1.338	16.527	0.081	2.994	16.527	0.181
+D+Lr	0.494	29.02	12.22		17.498	35.435	0.494	2.184	16.527	0.132	4.369	16.527	0.264
+D+S	0.551	32.37	13.63		19.519	35.435	0.551	2.421	16.527	0.146	4.858	16.527	0.294
+D+0.750Lr	0.464	27.26	11.48		16.440	35.435	0.464	1.973	16.527	0.119	4.026	16.527	0.244
+D+0.750S	0.507	29.78	12.54		17.955	35.435	0.507	2.150	16.527	0.130	4.392	16.527	0.266
+0.60D	0.225	13.20	5.56		7.958	35.435	0.225	0.803	16.527	0.049	1.797	16.527	0.109

Maximum Upward Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
span_1	1	0.1028	7.182		0.0000	0.000

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam with Torsional Loads

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member - B2 W/ Torsion

Vertical Reactions - Unfactored

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	13.629	13.629
Max Upward from Load Combination	13.629	13.629
Max Upward from Load Cases	9.261	9.261
D Only	9.261	9.261
+D+Lr	12.218	12.218
+D+S	13.629	13.629
+D+0.750Lr	11.479	11.479
+D+0.750S	12.537	12.537
+0.60D	5.557	5.557
S Only	4.368	4.368

Member C1

This column extends to the roof and supports members B1(b) (each side) at front wall at gridline - E & F. Include out of plane wind reaction on column.

$L := 13 \cdot \text{ft}$ member length

Loads from member B1(b): (1 on each sides of column)

$P_{DL.B1b} := 3.910\text{k}$ Torsion $_{DL.B1b} := 1.44\text{k} \cdot \text{ft}$ at $L = 10\text{ft}$

$P_{LL.B1b} := 0.343\text{k}$ Torsion $_{LL.B1b} := 0.47\text{k} \cdot \text{ft}$ at $L = 10\text{ft}$

$P_{SL.B1b} := 1.336\text{k}$ Torsion $_{SL.B1b} := 0.61\text{k} \cdot \text{ft}$ at $L = 10\text{ft}$

$$V_{WL.B1b} := \left[\max(p_{\text{wall}.1}, |p_{\text{wall}.2}|) \cdot \left[\frac{(10 \cdot \text{ft}) + (4.5 \cdot \text{ft})}{2} \right] \cdot \left[\frac{(9 \cdot \text{ft} + 11 \cdot \text{in}) + (10 \cdot \text{ft} + 1 \cdot \text{in})}{2} \right] \right]$$

$V_{WL.B1b} = 1.196 \cdot \text{k}$ O.O.P wind load at $L = 10\text{ft}$

Column reactions at the base

$P_{DL.C1} := 7.979\text{k}$ $V_{DL.C1} := 0.222\text{k}$

$P_{LL.C1} := 0.686\text{k}$ $V_{LL.C1} := 0.072\text{k}$ $V_{WL.C1} := 0.276\text{k}$

$P_{SL.C1} := 2.672\text{k}$ $V_{SL.C1} := 0.094\text{k}$

**Use HSS 4 x 4 x 1/4
OK See Attached**

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Column

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member C1

Code References

Governing Code : IBC 2018, CBC 2019
 Referenced Design Standard(s) : AISC 360-16
 Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Information

Steel Section Name : HSS4x4x1/4	Overall Column Height	14.0 ft
Analysis Method : Allowable Strength	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade	Brace condition :	
Fy : Steel Yield 46.0 ksi	Unbraced Length for buckling ABOUT X-X Axis = 14.0 ft, K = 1.0	
E : Elastic Bending Modulus 29,000.0 ksi	Unbraced Length for buckling ABOUT Y-Y Axis = 14.0 ft, K = 1.0	

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 170.940 lbs * Dead Load Factor

AXIAL LOADS . . .

Member B1(b): Axial Load at 10.0 ft, Xecc = -2.0 in, D = 4.284, LR = 0.3420, S = 1.336 k

Member B1(b): Axial Load at 10.0 ft, Xecc = 2.0 in, D = 4.284, LR = 0.3420, S = 1.336 k

BENDING LOADS . . .

Member B1(a): Moment acting about X-X axis at 10.0 ft, D = 1.580, LR = 0.460, S = 0.610 k-ft

Member B1(a): Moment acting about X-X axis at 10.0 ft, D = 1.580, LR = 0.460, S = 0.610 k-ft

Wind: Lat. Point Load at 10.0 ft creating Mx-x, W = 1.196 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.6291 : 1	Maximum Load Reactions . .	
Load Combination	+D+0.750S+0.450W	Top along X-X	0.0 k
Location of max.above base	9.960 ft	Bottom along X-X	0.0 k
At maximum location values are . . .		Top along Y-Y	0.8543 k
Pa : Axial	10.743 k	Bottom along Y-Y	0.4448 k
Pn / Omega : Allowable	40.812 k	Maximum Load Deflections . . .	
Ma-x : Applied	4.431 k-ft	Along Y-Y 0.4659 in at	7.705 ft above base
Mn-x / Omega : Allowable	10.765 k-ft	for load combination : +Lr+W	L/ 360.0
Ma-y : Applied	0.0 k-ft	Along X-X 0.0 in at	0.0ft above base
Mn-y / Omega : Allowable	10.765 k-ft	for load combination :	L/
PASS Maximum Shear Stress Ratio	0.01750 : 1		
Load Combination	+D+0.750S+0.450W		
Location of max.above base	0.0 ft		
At maximum location values are . . .			
Va : Applied	0.4448 k		
Vn / Omega : Allowable	25.423 k		

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios			Cb _x	Cb _y	K _x L _x /R _x	K _y L _y /R _y	Maximum Shear Ratios		
	Stress Ratio	Status	Location					Stress Ratio	Status	Location
D Only	0.400	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.009	PASS	0.00 ft
+D+Lr	0.471	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.011	PASS	0.00 ft
+D+S	0.537	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.012	PASS	0.00 ft
+D+0.750Lr	0.453	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.011	PASS	0.00 ft
+D+0.750S	0.503	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.011	PASS	0.00 ft
+D+0.60W	0.568	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.017	PASS	0.00 ft
+D+0.750Lr+0.450W	0.579	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.017	PASS	0.00 ft
+D+0.750S+0.450W	0.629	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.017	PASS	0.00 ft
+0.60D+0.60W	0.379	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.015	PASS	10.05 ft
+0.60D	0.190	PASS	9.96 ft	1.72	1.00	110.53	110.53	0.005	PASS	0.00 ft

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Column

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member C1

Reactions for Load Cases

Note: Only non-zero reactions are listed.

Load Combination	Axial (k)	Rx (k)		Ry (k)		Mx (k-ft)		My (k-ft)	
	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
D Only	8.739			-0.226	0.226				
Lr Only	0.684			-0.066	0.066				
S Only	2.672			-0.087	0.087				
W Only				-0.342	-0.854				

Reactions for Load Combinations

Note: Only non-zero reactions are listed.

Load Combination	Axial (k)	Rx (k)		Ry (k)		Mx (k-ft)		My (k-ft)	
	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
+D+Lr	9.423			-0.291	0.291				
+D+S	11.411			-0.313	0.313				
+D+0.750Lr	9.252			-0.275	0.275				
+D+0.750S	10.743			-0.291	0.291				
+D+0.60W	8.739			-0.431	-0.287				
+D+0.750Lr+0.450W	9.252			-0.429	-0.109				
+D+0.750S+0.450W	10.743			-0.445	-0.093				
+0.60D+0.60W	5.243			-0.340	-0.377				
+0.60D	5.243			-0.135	0.135				

Extreme Reactions

Item	Extreme Value	Axial (k)	Rx (k)		Ry (k)		Mx (k-ft)		My (k-ft)	
		@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
Axial, Base Max		11.411								
Axial, Base Min										
Rx, Base Max										
Rx, Base Min										
Rx, Top Max										
Rx, Top Min										
Ry, Base Max					-0.066					
Ry, Base Min					-0.445					
Ry, Top Max						0.313				
Ry, Top Min						-0.854				

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir		Distance	Max. Deflection in Y dir		Distance
D Only	0.0000	in	0.000 ft	0.201	in	7.047 ft
+D+Lr	0.0000	in	0.000 ft	0.260	in	7.047 ft
+D+S	0.0000	in	0.000 ft	0.279	in	7.047 ft
+D+0.750Lr	0.0000	in	0.000 ft	0.245	in	7.047 ft
+D+0.750S	0.0000	in	0.000 ft	0.259	in	7.047 ft
+D+0.60W	0.0000	in	0.000 ft	0.444	in	7.423 ft
+D+0.750Lr+0.450W	0.0000	in	0.000 ft	0.427	in	7.329 ft
+D+0.750S+0.450W	0.0000	in	0.000 ft	0.442	in	7.329 ft
+0.60D+0.60W	0.0000	in	0.000 ft	0.364	in	7.517 ft
+0.60D	0.0000	in	0.000 ft	0.121	in	7.047 ft
+Lr+0.60W	0.0000	in	0.000 ft	0.303	in	7.611 ft
+0.420W	0.0000	in	0.000 ft	0.171	in	7.799 ft
+Lr+W	0.0000	in	0.000 ft	0.466	in	7.705 ft
S Only	0.0000	in	0.000 ft	0.078	in	7.047 ft
W Only	0.0000	in	0.000 ft	0.408	in	7.799 ft

Steel Section Properties : HSS4x4x1/4

Steel Section Properties : HSS4x4x1/4

Steel Column

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Member C1

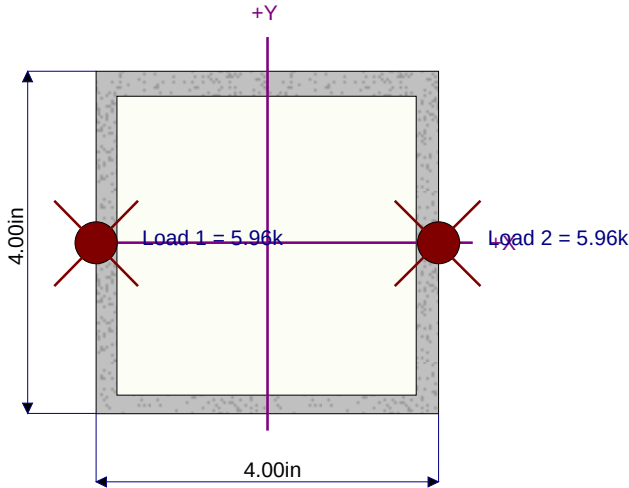
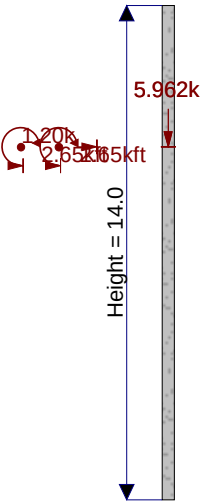
Depth	=	4.000 in	I xx	=	7.80 in^4	J	=	12.800 in^4
Design Thick	=	0.233 in	S xx	=	3.90 in^3			
Width	=	4.000 in	R xx	=	1.520 in			
Wall Thick	=	0.250 in	Zx	=	4.690 in^3			
Area	=	3.370 in^2	I yy	=	7.800 in^4	C	=	6.560 in^3
Weight	=	12.210 plf	S yy	=	3.900 in^3			
			R yy	=	1.520 in			
Ycg	=	1.150 in						

Sketches

My Loads - Looking along Y-Y Axis



Mx Loads - Looking along X-X Axis



Member C2

Check the RISA-3D braced frame analysis

Use HSS 4 x 4 x 5/16
see attached

BRAD YOUNG & ASSOCIATES
STRUCTURAL ENGINEERING

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Print Date: 8/3/2026

File Name: 251023 - Metal Stud.xmcd

Engineer: RV

Member C3

This column extends to the roof and supports members B1(a) (each side) at front wall at gridline - 1. Include out of plane wind reaction on column.

$L := 13 \cdot \text{ft}$ member length

Loads from member B1(a): (1 on each sides of column) - Conservative as the doubling loads on each side of column is doubling the mechanical unit loads.

$P_{DL.B1a} := 9.876k$ Torsion $_{DL.B1a} := 2.10k \cdot \text{ft}$ at $L = 10\text{ft}$

$P_{LL.B1a} := 2.830k$ Torsion $_{LL.B1a} := 0.68k \cdot \text{ft}$ at $L = 10\text{ft}$

$P_{SL.B1a} := 4.297k$ Torsion $_{SL.B1a} := 0.88k \cdot \text{ft}$ at $L = 10\text{ft}$

$$V_{WL.B1a} := \left[\max(p_{\text{wall}.1}, |p_{\text{wall}.2}|) \cdot \left[\frac{(10 \cdot \text{ft}) + (4.5 \cdot \text{ft})}{2} \right] \cdot \left[\frac{(13 \cdot \text{ft} + 5 \cdot \text{in}) + (14 \cdot \text{ft} + 8 \cdot \text{in})}{2} \right] \right]$$

$V_{WL.B1a} = 1.68 \cdot k$ O.O.P wind load at $L = 10\text{ft}$

Column reactions at the base

$P_{DL.C3} := 19.945k$ $V_{DL.C3} := 0.323k$

$P_{LL.C3} := 5.660k$ $V_{LL.C3} := 0.105k$ $V_{WL.C3} := 0.388k$

$P_{SL.C3} := 8.594k$ $V_{SL.C3} := 0.135k$

Use HSS 4 x 4 x 5/16"
OK See Attached

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Column

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Member C3

Code References

Governing Code : IBC 2018, CBC 2019
 Referenced Design Standard(s) : AISC 360-16
 Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Information

Steel Section Name : HSS4x4x5/16	Overall Column Height	13.0 ft
Analysis Method : Allowable Strength	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade	Brace condition :	
Fy : Steel Yield 46.0 ksi	Unbraced Length for buckling ABOUT X-X Axis = 13.0 ft, K = 1.0	
E : Elastic Bending Modulus 29,000.0 ksi	Unbraced Length for buckling ABOUT Y-Y Axis = 13.0 ft, K = 1.0	

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 192.790 lbs * Dead Load Factor

AXIAL LOADS . . .

Member B1(a): Axial Load at 10.0 ft, Xecc = -2.0 in, D = 9.876, LR = 2.830, S = 4.297 k

Member B1(a): Axial Load at 10.0 ft, Xecc = 2.0 in, D = 9.876, LR = 2.830, S = 4.297 k

BENDING LOADS . . .

Member B1(a): Moment acting about X-X axis at 10.0 ft, D = 2.10, LR = 0.680, S = 0.880 k-ft

Member B1(a): Moment acting about X-X axis at 10.0 ft, D = 2.10, LR = 0.680, S = 0.880 k-ft

Wind: Lat. Point Load at 10.0 ft creating Mx-x, W = 1.680 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9013** : 1
 Load Combination +D+0.750S+0.450W
 Location of max.above base 9.946 ft
 At maximum location values are . . .
 Pa : Axial 26.390 k
 Pn / Omega : Allowable 54.024 k
 Ma-x : Applied 5.959 k-ft
 Mn-x / Omega : Allowable 12.831 k-ft
 Ma-y : Applied 0.0 k-ft
 Mn-y / Omega : Allowable 12.831 k-ft

PASS Maximum Shear Stress Ratio = **0.01992** : 1
 Load Combination +D+0.750S+0.450W
 Location of max.above base 0.0 ft
 At maximum location values are . . .
 Va : Applied 0.5991 k
 Vn / Omega : Allowable 30.078 k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	1.292 k
Bottom along Y-Y	0.5991 k

Maximum Load Deflections . . .

Along Y-Y	0.4518 in at	7.067 ft above base
for load combination : +D+0.750S+0.450W		L/ 345.0
Along X-X	0.0 in at	0.0 ft above base
for load combination :		L/

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios				Maximum Shear Ratios					
	Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Stress Ratio	Status	Location
D Only	0.592	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.011	PASS	0.00 ft
+D+Lr	0.769	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.014	PASS	0.00 ft
+D+S	0.844	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.015	PASS	0.00 ft
+D+0.750Lr	0.724	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.013	PASS	0.00 ft
+D+0.750S	0.781	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.014	PASS	0.00 ft
+D+0.60W	0.752	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.018	PASS	0.00 ft
+D+0.750Lr+0.450W	0.845	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.019	PASS	0.00 ft
+D+0.750S+0.450W	0.901	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.020	PASS	0.00 ft
+0.60D+0.60W	0.515	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.019	PASS	10.03 ft
+0.60D	0.355	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.006	PASS	0.00 ft

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Column

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Member C3

Reactions for Load Cases

Note: Only non-zero reactions are listed.

Load Combination	Axial (k)	Rx (k)		Ry (k)		Mx (k-ft)		My (k-ft)	
	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
D Only	19.945			-0.323	0.323				
Lr Only	5.660			-0.105	0.105				
S Only	8.594			-0.135	0.135				
W Only				-0.388	-1.292				

Reactions for Load Combinations

Note: Only non-zero reactions are listed.

Load Combination	Axial (k)	Rx (k)		Ry (k)		Mx (k-ft)		My (k-ft)	
	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
+D+Lr	25.605			-0.428	0.428				
+D+S	28.539			-0.458	0.458				
+D+0.750Lr	24.190			-0.402	0.402				
+D+0.750S	26.390			-0.425	0.425				
+D+0.60W	19.945			-0.556	-0.452				
+D+0.750Lr+0.450W	24.190			-0.576	-0.180				
+D+0.750S+0.450W	26.390			-0.599	-0.157				
+0.60D+0.60W	11.967			-0.426	-0.582				
+0.60D	11.967			-0.194	0.194				

Extreme Reactions

Item	Extreme Value	Axial (k)	Rx (k)		Ry (k)		Mx (k-ft)		My (k-ft)	
		@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
Axial, Base Max		28.539								
Axial, Base Min										
Rx, Base Max										
Rx, Base Min										
Rx, Top Max										
Rx, Top Min										
Ry, Base Max					-0.105					
Ry, Base Min					-0.599					
Ry, Top Max						0.458				
Ry, Top Min						-1.292				

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir		Distance	Max. Deflection in Y dir		Distance
D Only	0.0000	in	0.000 ft	0.231	in	6.893 ft
+D+Lr	0.0000	in	0.000 ft	0.306	in	6.893 ft
+D+S	0.0000	in	0.000 ft	0.328	in	6.893 ft
+D+0.750Lr	0.0000	in	0.000 ft	0.287	in	6.893 ft
+D+0.750S	0.0000	in	0.000 ft	0.303	in	6.893 ft
+D+0.60W	0.0000	in	0.000 ft	0.429	in	7.067 ft
+D+0.750Lr+0.450W	0.0000	in	0.000 ft	0.435	in	7.067 ft
+D+0.750S+0.450W	0.0000	in	0.000 ft	0.452	in	7.067 ft
+0.60D+0.60W	0.0000	in	0.000 ft	0.337	in	7.154 ft
+0.60D	0.0000	in	0.000 ft	0.138	in	6.893 ft
+Lr+0.60W	0.0000	in	0.000 ft	0.273	in	7.242 ft
+0.420W	0.0000	in	0.000 ft	0.139	in	7.329 ft
+Lr+W	0.0000	in	0.000 ft	0.405	in	7.242 ft
S Only	0.0000	in	0.000 ft	0.097	in	6.893 ft
W Only	0.0000	in	0.000 ft	0.331	in	7.329 ft

Steel Section Properties : HSS4x4x5/16

Steel Section Properties : HSS4x4x5/16

Steel Column

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

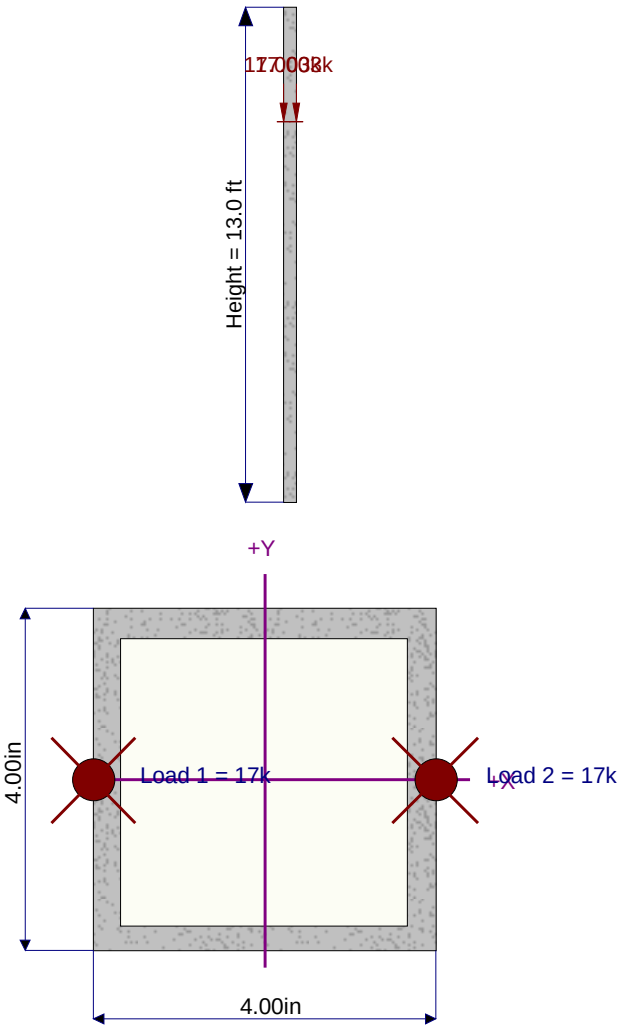
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DESCRIPTION: Member C3

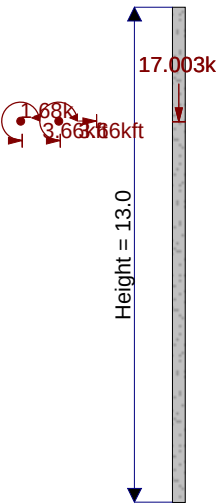
Depth	=	4.000 in	I xx	=	9.14 in^4	J	=	15.300 in^4
Design Thick	=	0.291 in	S xx	=	4.57 in^3			
Width	=	4.000 in	R xx	=	1.490 in			
Wall Thick	=	0.313 in	Zx	=	5.590 in^3			
Area	=	4.100 in^2	I yy	=	9.140 in^4	C	=	7.910 in^3
Weight	=	14.830 plf	S yy	=	4.570 in^3			
			R yy	=	1.490 in			
Ycg	=	1.150 in						

Sketches

My Loads - Looking along Y-Y Axis



Mx Loads - Looking along X-X Axis



Member C4

This column extends to the roof and supports members B2 (each side) at front wall at gridline - 5. Include out of plane wind reaction on column.

$L := 13 \cdot \text{ft}$ member length

Loads from member B2: (1 on each sides of column) - Conservative as the doubling loads on each side of column is doubling the mechanical unit loads. Additionally, this was taken with trellis loads that was conservatively assumed is carrying throughout the entire span for each beam member.

$P_{DL.B2} := 9.261\text{k}$ $\text{Torsion}_{DL.B2} := 2.30\text{k} \cdot \text{ft}$ at $L = 10\text{ft}$

$P_{LL.B2} := 2.957\text{k}$ $\text{Torsion}_{LL.B2} := 1.45\text{k} \cdot \text{ft}$ at $L = 10\text{ft}$

$P_{SL.B2} := 4.368\text{k}$ $\text{Torsion}_{SL.B2} := 1.86\text{k} \cdot \text{ft}$ at $L = 10\text{ft}$

$$V_{WL.B2} := \left[\max(p_{\text{wall}.1}, |p_{\text{wall}.2}|) \cdot \left[\frac{(10 \cdot \text{ft}) + (4.5 \cdot \text{ft})}{2} \right] \cdot \left[\frac{(14 \cdot \text{ft} + 2 \cdot \text{in}) + (14 \cdot \text{ft} + 3 \cdot \text{in})}{2} \right] \right]$$

$V_{WL.B2} = 1.699 \cdot \text{k}$ O.O.P wind load at $L = 10\text{ft}$

Column reactions at the base

$P_{DL.C4} := 18.715\text{k}$ $V_{DL.C4} := 0.354\text{k}$

$P_{LL.C4} := 5.914\text{k}$ $V_{LL.C4} := 0.223\text{k}$ $V_{WL.C4} := 0.392\text{k}$

$P_{SL.C4} := 8.736\text{k}$ $V_{SL.C4} := 0.286\text{k}$

Use HSS 4 x 4 x 5/16"
OK See Attached

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Column

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Member C4

Code References

Governing Code : IBC 2018, CBC 2019
 Referenced Design Standard(s) : AISC 360-16
 Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Information

Steel Section Name : HSS4x4x5/16	Overall Column Height	13.0 ft
Analysis Method : Allowable Strength	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade	Brace condition :	
Fy : Steel Yield 46.0 ksi	Unbraced Length for buckling ABOUT X-X Axis = 13.0 ft, K = 1.0	
E : Elastic Bending Modulus 29,000.0 ksi	Unbraced Length for buckling ABOUT Y-Y Axis = 13.0 ft, K = 1.0	

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 192.790 lbs * Dead Load Factor

AXIAL LOADS . . .

Member B2: Axial Load at 10.0 ft, Xecc = -2.0 in, D = 9.261, LR = 2.957, S = 4.368 k

Member B2: Axial Load at 10.0 ft, Xecc = 2.0 in, D = 9.261, LR = 2.957, S = 4.368 k

BENDING LOADS . . .

Member B2: Moment acting about X-X axis at 10.0 ft, D = 2.30, LR = 1.450, S = 1.860 k-ft

Member B2: Moment acting about X-X axis at 10.0 ft, D = 2.30, LR = 1.450, S = 1.860 k-ft

Wind: Lat. Point Load at 10.0 ft creating Mx-x, W = 1.699 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9809** : 1
 Load Combination +D+0.750S+0.450W
 Location of max.above base 9.946 ft
 At maximum location values are . . .
 Pa : Axial 25.267 k
 Pn / Omega : Allowable 54.024 k
 Ma-x : Applied 7.409 k-ft
 Mn-x / Omega : Allowable 12.831 k-ft
 Ma-y : Applied 0.0 k-ft
 Mn-y / Omega : Allowable 12.831 k-ft

PASS Maximum Shear Stress Ratio = **0.02477** : 1
 Load Combination +D+0.750S+0.450W
 Location of max.above base 0.0 ft
 At maximum location values are . . .
 Va : Applied 0.7449 k
 Vn / Omega : Allowable 30.078 k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	1.307 k
Bottom along Y-Y	0.7449 k

Maximum Load Deflections . . .

Along Y-Y	0.5562 in at	7.067 ft above base
for load combination : +D+0.750S+0.450W		L/ 280.0
Along X-X	0.0 in at	0.0ft above base
for load combination :		L/

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios				Maximum Shear Ratios					
	Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Stress Ratio	Status	Location
D Only	0.590	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.012	PASS	0.00 ft
+D+Lr	0.853	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.019	PASS	0.00 ft
+D+S	0.949	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.021	PASS	0.00 ft
+D+0.750Lr	0.788	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.017	PASS	0.00 ft
+D+0.750S	0.859	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.019	PASS	0.00 ft
+D+0.60W	0.752	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.020	PASS	0.00 ft
+D+0.750Lr+0.450W	0.909	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.023	PASS	0.00 ft
+D+0.750S+0.450W	0.981	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.025	PASS	0.00 ft
+0.60D+0.60W	0.516	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.019	PASS	10.03 ft
+0.60D	0.354	PASS	9.95 ft	1.38	1.00	104.70	104.70	0.007	PASS	0.00 ft

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Column

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Member C4

Reactions for Load Cases

Note: Only non-zero reactions are listed.

Load Combination	Axial (k)	Rx (k)		Ry (k)		Mx (k-ft)		My (k-ft)	
	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
D Only	18.715			-0.354	0.354				
Lr Only	5.914			-0.223	0.223				
S Only	8.736			-0.286	0.286				
W Only				-0.392	-1.307				

Reactions for Load Combinations

Note: Only non-zero reactions are listed.

Load Combination	Axial (k)	Rx (k)		Ry (k)		Mx (k-ft)		My (k-ft)	
	@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
+D+Lr	24.629			-0.577	0.577				
+D+S	27.451			-0.640	0.640				
+D+0.750Lr	23.150			-0.521	0.521				
+D+0.750S	25.267			-0.568	0.568				
+D+0.60W	18.715			-0.589	-0.430				
+D+0.750Lr+0.450W	23.150			-0.698	-0.067				
+D+0.750S+0.450W	25.267			-0.745	-0.020				
+0.60D+0.60W	11.229			-0.448	-0.572				
+0.60D	11.229			-0.212	0.212				

Extreme Reactions

Item	Extreme Value	Axial (k)	Rx (k)		Ry (k)		Mx (k-ft)		My (k-ft)	
		@ Base	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top	@ Base	@ Top
Axial, Base Max		27.451								
Axial, Base Min										
Rx, Base Max										
Rx, Base Min										
Rx, Top Max										
Rx, Top Min										
Ry, Base Max					-0.212					
Ry, Base Min					-0.745					
Ry, Top Max						0.640				
Ry, Top Min						-1.307				

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir		Distance	Max. Deflection in Y dir		Distance
D Only	0.0000	in	0.000 ft	0.253	in	6.893 ft
+D+Lr	0.0000	in	0.000 ft	0.412	in	6.893 ft
+D+S	0.0000	in	0.000 ft	0.457	in	6.893 ft
+D+0.750Lr	0.0000	in	0.000 ft	0.372	in	6.893 ft
+D+0.750S	0.0000	in	0.000 ft	0.406	in	6.893 ft
+D+0.60W	0.0000	in	0.000 ft	0.453	in	7.067 ft
+D+0.750Lr+0.450W	0.0000	in	0.000 ft	0.522	in	7.067 ft
+D+0.750S+0.450W	0.0000	in	0.000 ft	0.556	in	7.067 ft
+0.60D+0.60W	0.0000	in	0.000 ft	0.352	in	7.154 ft
+0.60D	0.0000	in	0.000 ft	0.152	in	6.893 ft
+Lr+0.60W	0.0000	in	0.000 ft	0.360	in	7.154 ft
+0.420W	0.0000	in	0.000 ft	0.141	in	7.329 ft
+Lr+W	0.0000	in	0.000 ft	0.494	in	7.154 ft
S Only	0.0000	in	0.000 ft	0.204	in	6.893 ft
W Only	0.0000	in	0.000 ft	0.335	in	7.329 ft

Steel Section Properties : HSS4x4x5/16

Steel Section Properties : HSS4x4x5/16

Steel Column

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

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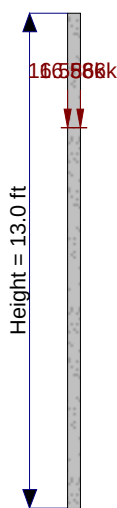
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DESCRIPTION: Member C4

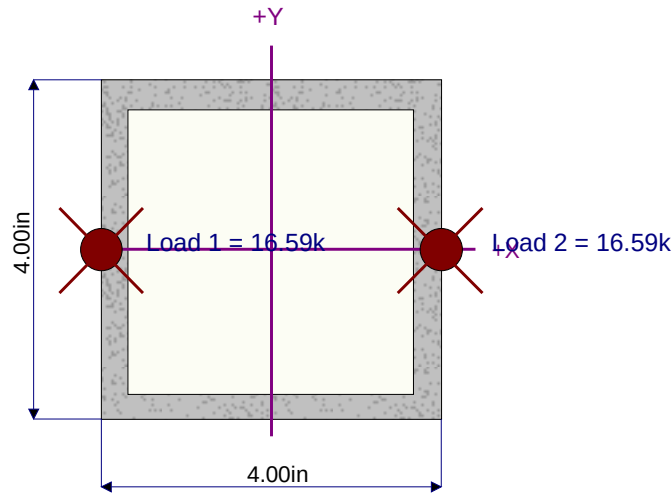
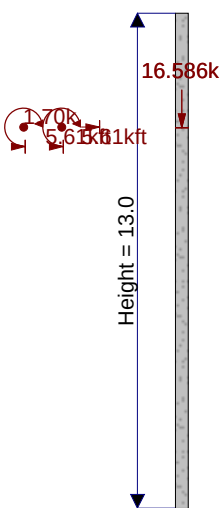
Depth	=	4.000 in	I xx	=	9.14 in^4	J	=	15.300 in^4
Design Thick	=	0.291 in	S xx	=	4.57 in^3			
Width	=	4.000 in	R xx	=	1.490 in			
Wall Thick	=	0.313 in	Zx	=	5.590 in^3			
Area	=	4.100 in^2	I yy	=	9.140 in^4	C	=	7.910 in^3
Weight	=	14.830 plf	S yy	=	4.570 in^3			
			R yy	=	1.490 in			
Ycg	=	1.150 in						

Sketches

My Loads - Looking along Y-Y Axis



Mx Loads - Looking along X-X Axis



Lateral Analysis - Seismic

Site Class - D

Occupancy Category - II

$I := 1.0$

Importance Factor

Seismic Loads

$S_s = 0.095$

$S_1 = 0.069$

From 2018 IBC Code

$F_a := 1.6$

$F_v := 2.4$

$S_{ds} := \frac{2}{3} \cdot F_a \cdot S_s$

$S_{ds} = 0.101$

$S_{d1} := \frac{2}{3} \cdot F_v \cdot S_1$

$S_{d1} = 0.110$

Seismic Design Category - B

$0.067 < S_{d1} < 0.133$

$\rho := 1.0$

Redundancy Factor

For the shear resisting systems, there will be a combination of Ordinary CMU shearwalls at the perimeter of the building. Use the C_d and Ω values associated with the R used for determining the seismic lateral forces (ASCE 7-16 section 12.2.3.3)

$R := 3.0$

Steel System not Specifically
Detailed for Seismic Resistance

$\Omega := 3.0$

Over strength Factor

$C_d := 3.0$

Amplification Factor

$C_t := 0.02$

$h_n := 14.50$

Building Height in feet

$T_a := C_t \cdot h_n^{0.75}$

Approx. Period

$T_a = 0.149$

$< T_L := 12$

$$C_{s,max} := \frac{S_{d1}}{T_a \cdot \left(\frac{R}{I}\right)} \quad C_{s,max} = 0.248$$

$$C_{s,frame} := \frac{S_{ds}}{\left(\frac{R}{I}\right)} \quad \text{Seismic Response Coefficient} \quad C_{s,frame} = 0.034$$

$$C_{s,min} := \max(0.01, 0.044 \cdot S_{ds} \cdot I) \quad C_{s,min} = 0.01$$

$$C_{s,frame} = 0.034 \quad \text{Seismic Response Coefficient (Frame)}$$

Seismic forces in longitudinal direction

$$R_{cold.form} := 6.5 \quad \text{Cold-form steel w/ framed shearwall}$$

$$\Omega_{cold.form} := 3.0 \quad \text{Over strength Factor}$$

$$C_{d,cold.form} := 4.0 \quad \text{Amplification Factor}$$

$$C_{s,cold.form} := \frac{S_{ds}}{\left(\frac{R_{cold.form}}{I}\right)} \quad \text{Seismic Response Coefficient} \quad C_{s,cold.form} = 0.016$$

$$C_{s,min} = 0.01$$

$$C_{s,cold.form} = 0.016 \quad \text{Seismic Response Coefficient (Cold-Form)}$$

Weight of Structure

$$A_{\text{roof}} := (97.25 \cdot \text{ft}) \cdot (45.583 \cdot \text{ft})$$

$$A_{\text{roof}} = 4433 \text{ ft}^2$$

$$A_{\text{walls}} := 2 \cdot (97.25 \cdot \text{ft} + 45.417 \cdot \text{ft}) \cdot \left(\frac{14.50 \cdot \text{ft}}{2} + 7.25 \cdot \text{ft} \right)$$

$$A_{\text{walls}} = 4137 \text{ ft}^2$$

$$A_{\text{trellis}} := 3 \cdot (4.333 \cdot \text{ft} \cdot 6.667 \text{ft}) + 3 \cdot \text{ft} \cdot (11.667 \text{ft} + 37.25 \cdot \text{ft} + 26 \text{ft} + 16 \text{ft})$$

$$A_{\text{trellis}} = 359 \text{ ft}^2$$

$$A_{\text{veneer}} := 2 \cdot (97.25 \cdot \text{ft} + 45.583 \cdot \text{ft}) \cdot \left(\frac{14.50 \cdot \text{ft}}{2} + 7.25 \cdot \text{ft} \right)$$

$$A_{\text{veneer}} = 4142 \text{ ft}^2$$

$$WT_{\text{roof}} := A_{\text{roof}} \cdot DL_{\text{roof}}$$

$$WT_{\text{roof}} = 88659 \text{ lb}$$

$$WT_{\text{walls}} := A_{\text{walls}} \cdot DL_{\text{wall.metal}}$$

$$WT_{\text{walls}} = 62060 \text{ lb}$$

$$WT_{\text{trellis}} := A_{\text{trellis}} \cdot DL_{\text{trellis}}$$

$$WT_{\text{trellis}} = 1797 \text{ lb}$$

$$WT_{\text{veneer}} := A_{\text{veneer}} \cdot DL_{\text{wall.veneer}}$$

$$WT_{\text{veneer}} = 165686 \text{ lb}$$

$$WT_{\text{mech}} := 3230 \cdot \text{lb} + 2655 \text{lb} + 1041 \text{lb} + 2 \cdot (100 \text{lb}) + 3 \cdot (150 \text{lb}) + 3 \cdot (260 \text{lb}) + 350 \text{lb} + 100 \text{lb} + 2 \cdot (100 \text{lb})$$

$$WT_{\text{mech}} = 9006 \text{ lb}$$

$$WT_{\text{total}} := WT_{\text{roof}} + WT_{\text{walls}} + WT_{\text{trellis}} + WT_{\text{mech}} + WT_{\text{veneer}}$$

$$WT_{\text{total}} = 327208 \text{ lb}$$

Calculated Seismic Force:

$$V_{\text{eq.trans.frame}} := C_{\text{s.frame}} \cdot (WT_{\text{total}})$$

$$V_{\text{eq.trans.frame}} = 11052 \text{ lb}$$

$$E_{\text{eq.trans.frame}} := \rho \cdot V_{\text{eq.trans.frame}}$$

$$E_{\text{eq.trans.frame}} = 11052 \text{ lb}$$

$$V_{\text{eq.trans.cold.form}} := C_{\text{s.cold.form}} \cdot (WT_{\text{total}})$$

$$V_{\text{eq.trans.cold.form}} = 5101 \text{ lb}$$

$$E_{\text{eq.trans.cold.form}} := \rho \cdot V_{\text{eq.trans.cold.form}}$$

$$E_{\text{eq.trans.cold.form}} = 5101 \text{ lb}$$

$$V_{\text{eq.long.cold.form}} := C_{\text{s.cold.form}} \cdot (WT_{\text{total}})$$

$$V_{\text{eq.long.cold.form}} = 5101 \text{ lb}$$

$$E_{\text{eq.long.cold.form}} := \rho \cdot V_{\text{eq.long.cold.form}}$$

$$E_{\text{eq.long.cold.form}} = 5101 \text{ lb}$$

Lateral Analysis - Wind

$$L_{\text{long}} := 97\text{ft} + 3\text{in}$$

max horizontal dimension

$$L_{\text{trans}} := 45\text{ft} + 7\text{in}$$

least horizontal dimension

$$H_{\text{ave}} := 14.50\text{ft}$$

Average roof height

$$P_{\text{ave}} := 7.25\text{ft}$$

Average parapet height

$$P_{\text{ave.2}} := 3.25\text{ft}$$

Hlgh parapet height (over main parapet)

$$a_1 := 0.1 \cdot L_{\text{trans}}$$

$$a_1 = 4.558\text{ft}$$

10% of least horizontal dimension

$$a_2 := 0.4 \cdot H_{\text{ave}}$$

$$a_2 = 5.8\text{ft}$$

.4 times mean roof height

$$a_3 := 0.04 \cdot L_{\text{trans}}$$

$$a_3 = 1.823\text{ft}$$

At least 4% of least horizontal dimension

$$a_4 := 3\text{ft}$$

$$a_4 = 3\text{ft}$$

At least 3'

$$a := \max(\min(a_1, a_2), a_3, a_4)$$

$$a = 4.558\text{ft}$$

Exposure B

$$V := 110$$

$$K_{zt} := 1.0$$

$$K_z := 0.70$$

$$K_d := 0.85$$

$$z_g := 761.80\text{ft}$$

$$K_e := \exp\left(-0.0000362 \cdot \frac{z_g}{\text{ft}}\right)$$

$$K_e = 0.973$$

$$q_h := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 \cdot \text{psf}$$

$$(q_h) = 17.929 \cdot \text{psf}$$

$$GC_{pi} := 0.18$$

Internal Pressure Coefficient

Interior Zone

$$GC_{pf.1} := 0.40$$

External Pressure Coefficient (Positive Face)

$$GC_{pf.2} := -0.29$$

External Pressure Coefficient (Negative Face)

$$p_{iz.1} := q_h \cdot [(GC_{pf.1} - GC_{pi}) - (GC_{pf.2} - GC_{pi})]$$

$$p_{iz.1} = 12.371 \cdot \text{psf}$$

$$p_{iz} := \max(p_{iz.1}, 25\text{psf})$$

$$p_{iz} = 25 \cdot \text{psf}$$

End Zone

$$GC_{pf.1} := 0.61 \quad \text{External Pressure Coefficient (Positive Face)}$$

$$GC_{pf.2} := -0.43 \quad \text{External Pressure Coefficient (Negative Face)}$$

$$p_{ez.1} := q_h \cdot [(GC_{pf.1} - GC_{pi}) - (GC_{pf.2} - GC_{pi})] \quad p_{ez.1} = 18.647 \cdot \text{psf}$$

$$p_{ez} := \max(p_{ez.1}, 25 \text{psf}) \quad p_{ez} = 25 \cdot \text{psf}$$

Parapet

$$GC_{pn.ww} := 1.5 \quad \text{External Pressure Coefficient (Positive Face)}$$

$$GC_{pn.lw} := -1.0 \quad \text{External Pressure Coefficient (Negative Face)}$$

$$p_p := q_h \cdot (GC_{pn.ww} - GC_{pn.lw}) \quad p_p = 44.823 \cdot \text{psf}$$

Wind force in the transverse direction of the building

$$WL_{trans} := [p_{iz} \cdot (L_{long} - 2 \cdot a) + p_{ez} \cdot (2 \cdot a)] \cdot \left(\frac{H_{ave}}{2} \right) + p_p \cdot (P_{ave} \cdot L_{long} + P_{ave.2} \cdot 45.583 \cdot \text{ft})$$

$$WL_{trans} = 55870 \text{ lb}$$

Wind force in the longitudinal direction of the building

$$WL_{long} := [p_{iz} \cdot (L_{trans} - a) + p_{ez} \cdot (a)] \cdot \left(\frac{H_{ave}}{2} \right) + p_p \cdot (P_{ave} + P_{ave.2}) \cdot L_{trans}$$

$$WL_{long} = 29716 \text{ lb}$$

Lateral Forces

ASD Load combinations are being used, check forces at design levels:
(0.7*E) for seismic force
(0.6*W) for wind force

Unfactored Loads

$$E_{eq.trans.cold.form} = 5101 \text{ lb}$$

$$E_{eq.trans.frame} = 11052 \text{ lb}$$

$$E_{eq.long.cold.form} = 5101 \text{ lb}$$

$$WL_{trans} = 55870 \text{ lb}$$

$$WL_{long} = 29716 \text{ lb}$$

Transverse Direction

$$0.7 \cdot E_{eq.trans.cold.form} = 3571 \text{ lb} \quad \text{Seismic Force}$$

$$0.7 \cdot E_{eq.trans.frame} = 7737 \text{ lb} \quad \text{Seismic Force}$$

$$0.6 \cdot WL_{trans} = 33522 \text{ lb} \quad \text{Wind Force} \quad \text{Wind Forces Govern}$$

Longitudinal Direction

$$0.7 \cdot E_{eq.long.cold.form} = 3571 \text{ lb} \quad \text{Seismic Force}$$

$$0.6 \cdot WL_{long} = 17829 \text{ lb} \quad \text{Wind Force} \quad \text{Wind Forces Govern}$$

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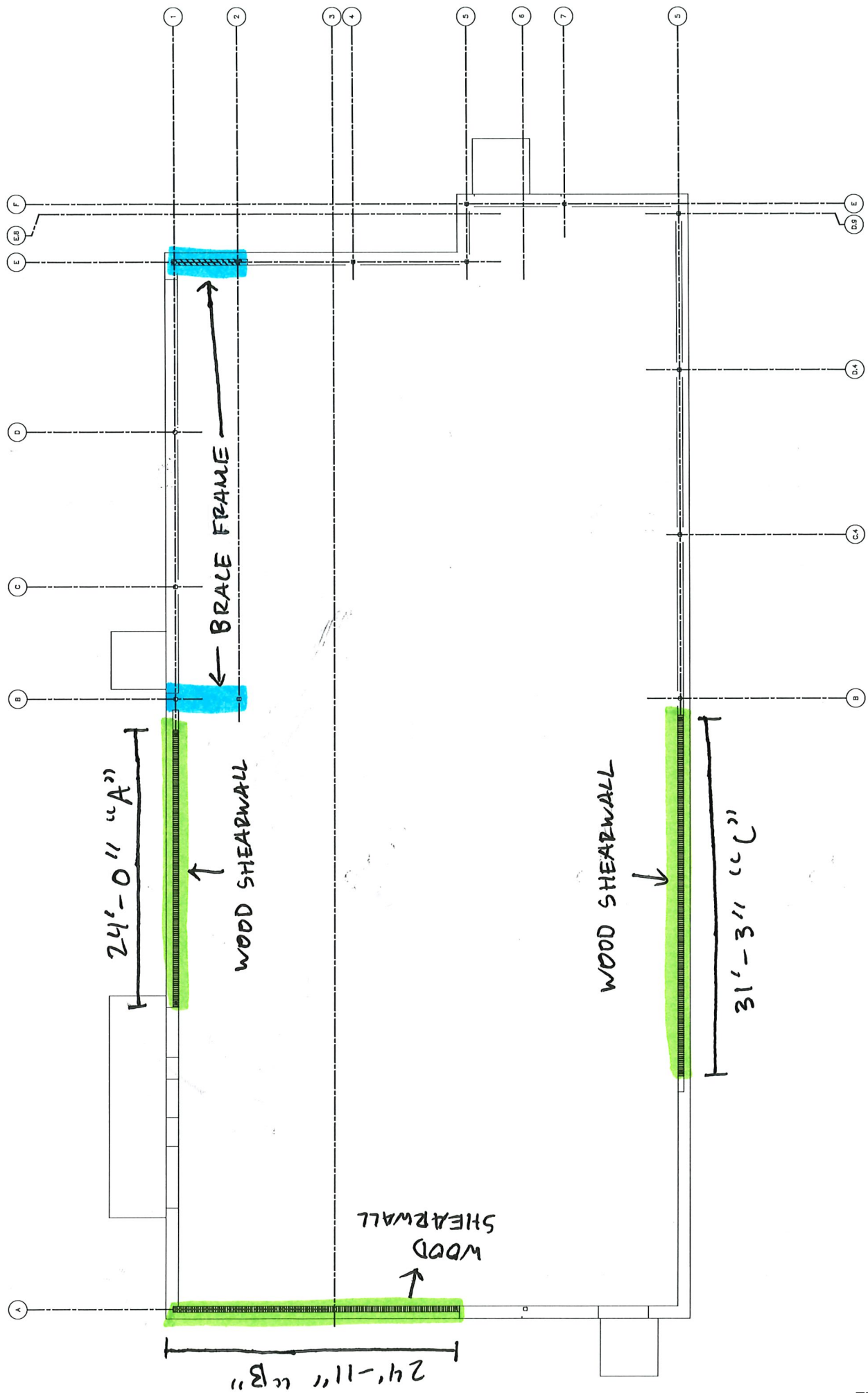
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Print Date: 8/3/2026

File Name: 251023 - Metal Stud.xmcd

Engineer: RV

LATERAL KEY PLAN



Transverse Direction

In the transverse direction, there will be metal stud shearwall along Line A, additionally, there will be braced frames along Line B and Line E. The transverse direction is governed by **Wind** forces. Determine the lateral forces acting at each line.

$$W_{L_{trans}} = 55870 \text{ lb} \quad \text{Unfactored wind force}$$

$$E_{eq.trans.cold.form} = 5101 \text{ lb} \quad \text{Unfactored seismic force (Shearwall)}$$

$$E_{eq.trans.frame} = 11052 \text{ lb} \quad \text{Unfactored seismic force (Frame)}$$

$$L_{long} = 97.25 \text{ ft} \quad \text{Max horizontal dimension}$$

$$L_{trans} = 45.583 \text{ ft} \quad \text{Max horizontal dimension}$$

$$w_{trans.WL} := \frac{W_{L_{trans}}}{L_{long}} \quad w_{trans.WL} = 575 \cdot \text{plf}$$

$$w_{trans.EQ.cold.form} := \frac{E_{eq.trans.cold.form}}{L_{long}} \quad w_{trans.EQ.cold.form} = 52 \cdot \text{plf}$$

$$w_{trans.EQ.frame} := \frac{E_{eq.trans.frame}}{L_{long}} \quad w_{trans.EQ.frame} = 114 \cdot \text{plf}$$

Metal stud shearwall

$$\Omega_V := 2.0 \quad \text{ASD Safety Factor (B5.2.3)}$$

Line A - Metal Stud Shearwall

$$L_{\text{wall.A}} := (24\text{ft} + 11\text{in}) = 24.917\text{ ft}$$

$$H_{\text{wall}} := (13\text{ft} + 4\text{in}) = 13.333\text{ ft}$$

$$\text{Trib}_A := \frac{(53\text{ft} + 0.75\text{in})}{2} = 26.531\text{ ft}$$

Check Type I Shearwall Requirements (AIS S240-2015):

$$\text{Condition}_a := \text{if} \left[\left(\frac{H_{\text{wall}}}{L_{\text{wall.A}}} \right) < 4.0, \text{"Satisfied"}, \text{"Not Satisfied"} \right] \quad \text{Condition}_a = \text{"Satisfied"}$$

$$\text{Condition}_b := \text{if} \left[(L_{\text{wall.A}}) > 24\text{in}, \text{"Satisfied"}, \text{"Not Satisfied"} \right] \quad \text{Condition}_b = \text{"Satisfied"}$$

Note: Since there are no wall openings, condition c or d do not apply.

Shearwall Forces:

$$V_{A.WL} := (0.6 \cdot w_{\text{trans.WL}}) \cdot \text{Trib}_A$$

$$V_{A.WL} = 9145\text{ lb}$$

$$v_{A.WL} := \frac{V_{A.WL}}{L_{\text{wall.A}}}$$

$$v_{A.WL} = 367 \cdot \text{plf} \quad \text{ASD}$$

Type I Shearwall Design (AIS S240-2015):

$$\frac{H_{\text{wall}}}{L_{\text{wall.A}}} = 0.535 \quad < 2.0, \text{ Use EQ. B5.2.2-1}$$

$$v_{\text{nom}} := 910 \cdot \text{plf} \quad \text{Table B5.2.2.3-2}$$

$$v_{\text{available}} := \frac{v_{\text{nom}}}{\Omega_V}$$

$$v_{\text{available}} = 455 \cdot \text{plf}$$

Note: Prototype uses 15/32" APA plywood, but Table B5.2.2.3-2 shows capacity of 15/32" Struct. 1 plywood. Since the applied shear force is very small, OK to check capacity using 7/16" OSB.

Check shearwall capacity:

$$\frac{v_{A.WL}}{v_{\text{available}}} = 0.807 \quad < 1.0, \text{ Therefore OK}$$

Use: 15/32" APA Plywood
w/ #8 screws @ 6/12 @ E/F
Type B Shearwall

Line B - Braced Frame

Check the brace frame for the wind loading.

$$\text{Trib}_B := \frac{(53\text{ft} + 0.75\text{in}) + (38\text{ft} + 0.25\text{in})}{2}$$

$$\text{Trib}_B = 45.542 \text{ ft}$$

$$V_{B.WL} := w_{\text{trans.WL}} \cdot \text{Trib}_B$$

$$V_{B.WL} = 26164 \text{ lb} \quad (\text{Wind Governs})$$

$$V_{B.EQ} := w_{\text{trans.EQ.frame}} \cdot \text{Trib}_B$$

$$V_{B.EQ} = 5176 \text{ lb}$$

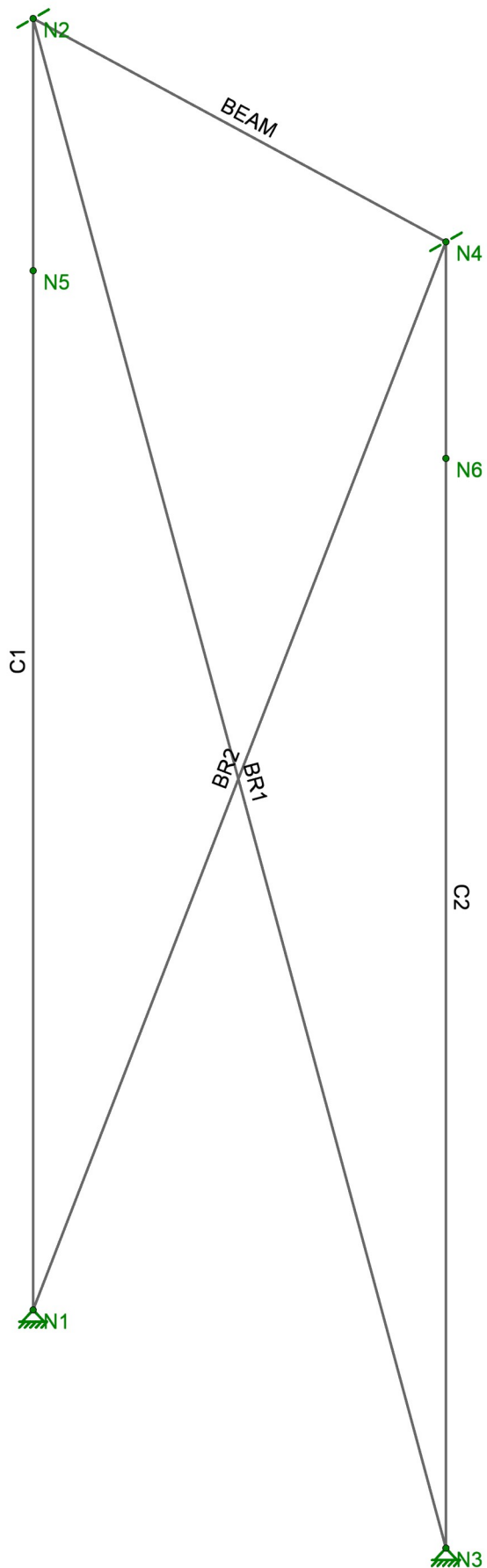
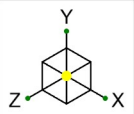
Vertical Loads acting at column along gridline 1

$$P_{DL.B1a} = 9.876 \cdot \text{k}$$

$$P_{LL.B1a} = 2.830 \cdot \text{k}$$

$$P_{SL.B1a} = 4.297 \cdot \text{k}$$

See attached RISA3D output



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251023

Braced Frame - Line B

SK-1

Aug 03, 2026 at 01:18 PM

251023 - Braced Frame Line ...

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	-1.33	0	
2	N2	0	13.578	0	
3	N3	5.5	-1.33	0	
4	N4	5.5	13.75	0	
5	N5	0	10.667	0	
6	N6	5.5	11.25	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]
1	N1	Reaction	Reaction	Reaction
2	N3	Reaction	Reaction	Reaction
3	N2			Reaction
4	N4			Reaction

Basic Load Cases

	BLC Description	Category	Y Gravity	Nodal
1	DL	DL	-1	1
2	RLL	RLL		1
3	SL	SL		1
4	Wind	WL		1
5	EL	EL		1

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor
1	IBC 18/ASCE Strength 1	Yes	Y	DL	1.4				
2	IBC 18/ASCE Strength 3 (a)	Yes	Y	DL	1.2	RLL	1.6		
3	IBC 18/ASCE Strength 3 (c)	Yes	Y	DL	1.2	SL	1.6		
4	IBC 18/ASCE Strength 3 (b)	Yes	Y	DL	1.2	RLL	1.6	WL	0.5
5	IBC 18/ASCE Strength 3 (d)	Yes	Y	DL	1.2	SL	1.6	WL	0.5
6	IBC 18/ASCE Strength 4 (a)	Yes	Y	DL	1.2	WL	1	RLL	0.5
7	IBC 18/ASCE Strength 4 (b)	Yes	Y	DL	1.2	WL	1	SL	0.5
8	IBC 18/ASCE Strength 5	Yes	Y	DL	0.9	WL	1		
9	Dead Load		Y	DL	1				
10	Roof Live Load		Y	RLL	1				
11	Snow Load		Y	SL	1				
12	Wind Load		Y	WL	1				

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule Area [in ²]	I _{yy} [in ⁴]	I _{zz} [in ⁴]	J [in ⁴]
1	COL	HSS4X4X5	Column	Tube	A500 Gr.46	Typical	4.1	9.14	9.14
2	Beam-1	HSS4X4X5	Beam	Tube	A500 Gr.46	Typical	4.1	9.14	9.14
3	Brace	HSS4X4X5	HBrace	Tube	A500 Gr.46	Typical	4.1	9.14	9.14
4	Beam-2	HSS8X4X4	Beam	Tube	A500 Gr.46	Typical	5.24	14.4	42.5

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	C1	N1	N2	COL	Column	Tube	A500 Gr.46	Typical
2	BEAM	N2	N4	Beam-1	Beam	Tube	A500 Gr.46	Typical

Member Primary Data (Continued)

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
3	C2	N4	N3	COL	Column	Tube	A500 Gr.46	Typical
4	BR1	N3	N2	Brace	HBrace	Tube	A500 Gr.46	Typical
5	BR2	N1	N4	Brace	HBrace	Tube	A500 Gr.46	Typical

Member Advanced Data

	Label	Col-Wall Vert Release	Physical	Deflection Ratio Options	Seismic DR
1	C1		Yes	** NA **	None
2	BEAM		Yes	Default	None
3	C2		Yes	** NA **	None
4	BR1		Yes	** NA **	None
5	BR2		Yes	** NA **	None

Member Area Loads

No Data to Print...

Member Distributed Loads

No Data to Print...

Member Point Loads

No Data to Print...

Envelope AISC 15TH (360-16): LRFD Member Steel Code Checks

Member	Shape	Code Check	Loc[ft]	LC	Shear	Check	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
1	C1	HSS4X4X5	0.747	0	7	0	14.908	y	7	64.519	169.74	19.285	19.285	2.019	H1-1a
2	BEAM	HSS4X4X5	0.084	0	8	0.002	5.503	y	3	148.814	169.74	19.285	19.285	1	H1-1b*
3	C2	HSS4X4X5	0.231	0	7	0	2.462	y	1	63.055	169.74	19.285	19.285	2.426	H1-1a
4	BR1	HSS4X4X5	0.218	15.89	8	0.001	15.89	y	1	56.789	169.74	19.285	19.285	1.684	H1-1a
5	BR2	HSS4X4X5	0.747	0	7	0.001	0	y	1	55.652	169.74	19.285	19.285	1.759	H1-1a

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	13.892	7	86.566	7	0	8	0	8	0	8	0	8
2		min	0.998	1	14.673	1	0	1	0	1	0	1	0	1
3	N3	max	12.617	8	0.655	1	0	8	0	8	0	8	0	8
4		min	-1.309	3	-71.39	8	0	1	0	1	0	1	0	1
5	N2	max	0	8	0	8	0	8	0	8	0	8	0	8
6		min	0	1	0	1	0	1	0	1	0	1	0	1
7	N4	max	0	8	0	8	0	8	0	8	0	8	0	8
8		min	0	1	0	1	0	1	0	1	0	1	0	1
9	Totals:	max	26.164	6	20.014	3	0	8						
10		min	0	1	9.854	8	0	1						

Envelope Member End Reactions

	Member	Member End		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
1	C1	I	max	47.786	7	-0.009	3	0	8	0	8	0	8	-0.092	2
2			min	11.802	1	-0.02	7	0	1	0	1	0	1	-0.137	7
3		J	max	34.293	8	0.021	7	0	8	0	8	0	8	0.058	8
4			min	-3.393	3	-0.009	1	0	1	0	1	0	1	0.024	5
5	BEAM	I	max	12.449	8	0.031	6	0	8	0	8	0	8	-0.05	8

Envelope Member End Reactions (Continued)

Member	Member End		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
6		min	-1.289	3	0.013	3	0	1	0	1	0	1	-0.121	5
7	J	max	12.447	8	-0.039	8	0	8	0	8	0	8	0.069	3
8		min	-1.292	3	-0.08	3	0	1	0	1	0	1	-0.028	8
9	C2	max	-2.448	1	-0.002	8	0	8	0	8	0	8	-0.012	8
10		min	-38.856	7	-0.016	1	0	1	0	1	0	1	-0.128	1
11	J	max	-2.154	1	-0.002	8	0	8	0	8	0	8	0.104	1
12		min	-38.603	7	-0.015	1	0	1	0	1	0	1	0.019	8
13	BR1	max	3.854	3	0.051	1	0	8	0	8	0	8	0.104	1
14		min	-35.902	8	0.029	8	0	1	0	1	0	1	0.019	8
15	J	max	3.605	3	-0.04	8	0	8	0	8	0	8	0.151	1
16		min	-36.089	8	-0.057	1	0	1	0	1	0	1	0.108	8
17	BR2	max	41.226	7	0.055	1	0	8	0	8	0	8	0.137	7
18		min	3.036	1	0.039	8	0	1	0	1	0	1	0.092	2
19	J	max	40.974	7	-0.03	8	0	8	0	8	0	8	0.088	1
20		min	2.741	1	-0.053	1	0	1	0	1	0	1	0.04	8

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor
1	IBC 18/ASCE Strength 1		Y	DL	1.4				
2	IBC 18/ASCE Strength 3 (a)		Y	DL	1.2	RLL	1.6		
3	IBC 18/ASCE Strength 3 (c)		Y	DL	1.2	SL	1.6		
4	IBC 18/ASCE Strength 3 (b)		Y	DL	1.2	RLL	1.6	WL	0.5
5	IBC 18/ASCE Strength 3 (d)		Y	DL	1.2	SL	1.6	WL	0.5
6	IBC 18/ASCE Strength 4 (a)		Y	DL	1.2	WL	1	RLL	0.5
7	IBC 18/ASCE Strength 4 (b)		Y	DL	1.2	WL	1	SL	0.5
8	IBC 18/ASCE Strength 5		Y	DL	0.9	WL	1		
9	Dead Load	Yes	Y	DL	1				
10	Roof Live Load	Yes	Y	RLL	1				
11	Snow Load	Yes	Y	SL	1				
12	Wind Load	Yes	Y	WL	1				

Node Reactions

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	9	N1	0.713	10.48	0	0	0	0
2	9	N3	-0.713	0.469	0	0	0	0
3	9	N2	0	0	0	0	0	0
4	9	N4	0	0	0	0	0	0
5	9	Totals:	0	10.949	0			
6	9	COG (ft):	X: 0.237	Y: 10.333	Z: 0			
7	10	N1	0.187	2.83	0	0	0	0
8	10	N3	-0.187	0	0	0	0	0
9	10	N2	0	0	0	0	0	0
10	10	N4	0	0	0	0	0	0
11	10	Totals:	0	2.83	0			
12	10	COG (ft):	X: 0	Y: 10.667	Z: 0			
13	11	N1	0.284	4.297	0	0	0	0
14	11	N3	-0.284	0	0	0	0	0
15	11	N2	0	0	0	0	0	0
16	11	N4	0	0	0	0	0	0
17	11	Totals:	0	4.297	0			
18	11	COG (ft):	X: 0	Y: 10.667	Z: 0			
19	12	N1	12.923	71.764	0	0	0	0
20	12	N3	13.241	-71.764	0	0	0	0
21	12	N2	0	0	0	0	0	0
22	12	N4	0	0	0	0	0	0
23	12	Totals:	26.164	0	0			
24	12	COG (ft):	NC	NC	NC			

Line E - Braced Frame

Check the brace frame for the wind loading.

$$\text{Trib}_E := \frac{(38\text{ft} + 0.25\text{in})}{2} + (5\text{ft} + 0\text{in})$$

$$\text{Trib}_E = 24.010 \text{ ft}$$

$$V_{E.WL} := w_{\text{trans.WL}} \cdot \text{Trib}_E$$

$$V_{E.WL} = 13794 \text{ lb} \quad (\text{Wind Governs})$$

$$V_{E.EQ} := w_{\text{trans.EQ.frame}} \cdot \text{Trib}_E$$

$$V_{E.EQ} = 2729 \text{ lb}$$

Apply loads from beam B1(a) & B1(b):

$$P_{DL.B1a} = 9.876 \cdot \text{k}$$

$$P_{DL.B1b} = 3.910 \cdot \text{k}$$

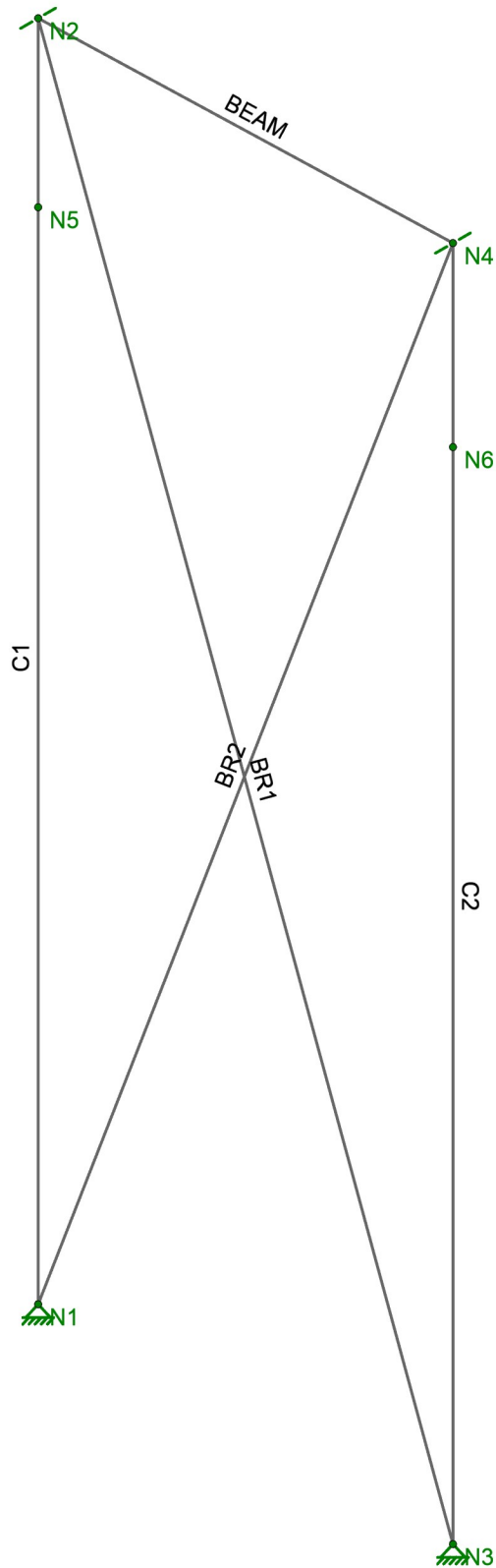
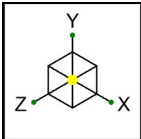
$$P_{LL.B1a} = 2.830 \cdot \text{k}$$

$$P_{LL.B1b} = 0.343 \cdot \text{k}$$

$$P_{SL.B1a} = 4.297 \cdot \text{k}$$

$$P_{SL.B1b} = 1.336 \cdot \text{k}$$

See attached RISA3D output



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Braced Frame - Line E

SK-2

Aug 03, 2026 at 01:22 PM

251023 - Braced Frame Lin...

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	-1.33	0	
2	N2	0	13.417	0	
3	N3	5.5	-1.33	0	
4	N4	5.5	13.589	0	
5	N5	0	11.25	0	
6	N6	5.5	11.25	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]
1	N1	Reaction	Reaction	Reaction
2	N3	Reaction	Reaction	Reaction
3	N2			Reaction
4	N4			Reaction

Basic Load Cases

	BLC Description	Category	Y Gravity	Nodal
1	DL	DL	-1	2
2	RLL	RLL		2
3	SL	SL		2
4	Wind	WL		1
5	EL	EL		1

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor
1	IBC 18/ASCE Strength 1	Yes	Y	DL	1.4				
2	IBC 18/ASCE Strength 3 (a)	Yes	Y	DL	1.2	RLL	1.6		
3	IBC 18/ASCE Strength 3 (c)	Yes	Y	DL	1.2	SL	1.6		
4	IBC 18/ASCE Strength 3 (b)	Yes	Y	DL	1.2	RLL	1.6	WL	0.5
5	IBC 18/ASCE Strength 3 (d)	Yes	Y	DL	1.2	SL	1.6	WL	0.5
6	IBC 18/ASCE Strength 4 (a)	Yes	Y	DL	1.2	WL	1	RLL	0.5
7	IBC 18/ASCE Strength 4 (b)	Yes	Y	DL	1.2	WL	1	SL	0.5
8	IBC 18/ASCE Strength 5	Yes	Y	DL	0.9	WL	1		
9	Dead Load		Y	DL	1				
10	Roof Live Load		Y	RLL	1				
11	Snow Load		Y	SL	1				
12	Wind Load		Y	WL	1				

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule Area [in ²]	I _{yy} [in ⁴]	I _{zz} [in ⁴]	J [in ⁴]
1	COL	HSS4X4X5	Column	Tube	A500 Gr.46	Typical	4.1	9.14	9.14
2	Beam-1	HSS4X4X5	Beam	Tube	A500 Gr.46	Typical	4.1	9.14	9.14
3	Brace	HSS4X4X5	HBrace	Tube	A500 Gr.46	Typical	4.1	9.14	9.14
4	Beam-2	HSS8X4X4	Beam	Tube	A500 Gr.46	Typical	5.24	14.4	42.5

Member Primary Data

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
1	C1	N1	N2	COL	Column	Tube	A500 Gr.46	Typical
2	BEAM	N2	N4	Beam-2	Beam	Tube	A500 Gr.46	Typical

Member Primary Data (Continued)

	Label	I Node	J Node	Section/Shape	Type	Design List	Material	Design Rule
3	C2	N4	N3	COL	Column	Tube	A500 Gr.46	Typical
4	BR1	N3	N2	Brace	HBrace	Tube	A500 Gr.46	Typical
5	BR2	N1	N4	Brace	HBrace	Tube	A500 Gr.46	Typical

Member Advanced Data

	Label	Col-Wall Vert Release	Physical	Deflection Ratio Options	Seismic DR
1	C1		Yes	** NA **	None
2	BEAM		Yes	Default	None
3	C2		Yes	** NA **	None
4	BR1		Yes	** NA **	None
5	BR2		Yes	** NA **	None

Member Area Loads

No Data to Print...

Member Distributed Loads

No Data to Print...

Member Point Loads

No Data to Print...

Envelope AISC 15TH (360-16): LRFD Member Steel Code Checks

Member	Shape	Code Check	Loc[ft]	LC	Shear	Check	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
1	C1	HSS4X4X5	0.452	0	7	0	12.49	y	7	65.935	169.74	19.285	19.285	2.222	H1-1a
2	BEAM	HSS8X4X4	0.029	0	8	0.001	5.503	y	1	209.231	216.936	26.719	45.885	1	H1-1b*
3	C2	HSS4X4X5	0.133	0	7	0	2.284	y	5	64.423	169.74	19.285	19.285	2.235	H1-1b*
4	BR1	HSS4X4X5	0.102	15.739	8	0.001	15.739	y	1	57.884	169.74	19.285	19.285	2.082	H1-1b*
5	BR2	HSS4X4X5	0.431	0	7	0.001	15.901	y	1	56.715	169.74	19.285	19.285	1.487	H1-1a

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	8.272	7	52.212	7	0	8	0	8	0	8	0	8
2		min	1.45	1	14.681	1	0	1	0	1	0	1	0	1
3	N3	max	6.028	8	7.395	3	0	8	0	8	0	8	0	8
4		min	-1.879	3	-33.512	8	0	1	0	1	0	1	0	1
5	N2	max	0	8	0	8	0	8	0	8	0	8	0	8
6		min	0	1	0	1	0	1	0	1	0	1	0	1
7	N4	max	0	8	0	8	0	8	0	8	0	8	0	8
8		min	0	1	0	1	0	1	0	1	0	1	0	1
9	Totals:	max	13.794	7	26.858	3	0	8						
10		min	0	2	13.384	8	0	1						

Envelope Member End Reactions

Member			Member End	Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
1	C1	I	max	29.487	7	-0.007	3	0	8	0	8	0	8	-0.077	8
2			min	10.618	1	-0.012	7	0	1	0	1	0	1	-0.098	7
3		J	max	16.403	8	0.007	7	0	8	0	8	0	8	0.041	8
4			min	-4.886	3	-0.007	1	0	1	0	1	0	1	0.011	3
5	BEAM	I	max	5.991	8	0.049	1	0	8	0	8	0	8	-0.078	8

Envelope Member End Reactions (Continued)

Member	Member End		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Moment[k-ft]	LC	z-z Moment[k-ft]	LC
6		min	-1.862	3	0.03	3	0	1	0	1	0	1	-0.149	5
7	J	max	5.988	8	-0.048	8	0	8	0	8	0	8	0.009	3
8		min	-1.866	3	-0.088	1	0	1	0	1	0	1	-0.056	8
9	C2	max	-3.619	1	-0.007	8	0	8	0	8	0	8	-0.02	8
10		min	-22.564	7	-0.015	5	0	1	0	1	0	1	-0.096	3
11	J	max	2.274	3	-0.003	8	0	8	0	8	0	8	0.089	1
12		min	-17.541	8	-0.012	1	0	1	0	1	0	1	0.03	8
13	BR1	max	5.451	3	0.048	1	0	8	0	8	0	8	0.089	1
14		min	-17.064	8	0.029	8	0	1	0	1	0	1	0.03	8
15	J	max	5.204	3	-0.04	8	0	8	0	8	0	8	0.171	1
16		min	-17.249	8	-0.059	1	0	1	0	1	0	1	0.119	8
17	BR2	max	24.192	7	0.052	1	0	8	0	8	0	8	0.098	7
18		min	4.311	1	0.035	8	0	1	0	1	0	1	0.077	8
19	J	max	23.943	7	-0.034	8	0	8	0	8	0	8	0.121	1
20		min	4.02	1	-0.056	1	0	1	0	1	0	1	0.076	8

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor
1	IBC 18/ASCE Strength 1		Y	DL	1.4				
2	IBC 18/ASCE Strength 3 (a)		Y	DL	1.2	RLL	1.6		
3	IBC 18/ASCE Strength 3 (c)		Y	DL	1.2	SL	1.6		
4	IBC 18/ASCE Strength 3 (b)		Y	DL	1.2	RLL	1.6	WL	0.5
5	IBC 18/ASCE Strength 3 (d)		Y	DL	1.2	SL	1.6	WL	0.5
6	IBC 18/ASCE Strength 4 (a)		Y	DL	1.2	WL	1	RLL	0.5
7	IBC 18/ASCE Strength 4 (b)		Y	DL	1.2	WL	1	SL	0.5
8	IBC 18/ASCE Strength 5		Y	DL	0.9	WL	1		
9	Dead Load	Yes	Y	DL	1				
10	Roof Live Load	Yes	Y	RLL	1				
11	Snow Load	Yes	Y	SL	1				
12	Wind Load	Yes	Y	WL	1				

Node Reactions

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	9	N1	1.036	10.486	0	0	0	0
2	9	N3	-1.036	4.386	0	0	0	0
3	9	N2	0	0	0	0	0	0
4	9	N4	0	0	0	0	0	0
5	9	Totals:	0	14.871	0			
6	9	COG (ft):	X: 1.623	Y: 10.968	Z: 0			
7	10	N1	0.224	2.83	0	0	0	0
8	10	N3	-0.224	0.343	0	0	0	0
9	10	N2	0	0	0	0	0	0
10	10	N4	0	0	0	0	0	0
11	10	Totals:	0	3.173	0			
12	10	COG (ft):	X: 0.595	Y: 11.25	Z: 0			
13	11	N1	0.398	4.297	0	0	0	0
14	11	N3	-0.398	1.336	0	0	0	0
15	11	N2	0	0	0	0	0	0
16	11	N4	0	0	0	0	0	0
17	11	Totals:	0	5.633	0			
18	11	COG (ft):	X: 1.304	Y: 11.25	Z: 0			
19	12	N1	6.84	37.424	0	0	0	0
20	12	N3	6.954	-37.424	0	0	0	0
21	12	N2	0	0	0	0	0	0
22	12	N4	0	0	0	0	0	0
23	12	Totals:	13.794	0	0			
24	12	COG (ft):	NC	NC	NC			

Longitudinal Direction

In the longitudinal direction, there will be metal stud shearwall along Line 1 and Line 5. The longitudinal direction is governed by Wind forces. Determine the lateral forces acting at each line.

$$W_{L_{long}} = 29.716 \cdot k \quad \text{Unfactored Wind Force}$$

$$E_{eq.trans.cold.form} = 5101 \text{ lb} \quad \text{Unfactored seismic force (Shearwall)}$$

$$E_{eq.trans.frame} = 11052 \text{ lb} \quad \text{Unfactored seismic force (Frame)}$$

$$L_{long} = 97.25 \text{ ft} \quad \text{Max horizontal dimension}$$

$$L_{trans} = 45.583 \text{ ft} \quad \text{Max horizontal dimension}$$

$$w_{long.WL} := \frac{W_{L_{long}}}{L_{trans}} \quad w_{long.WL} = 652 \cdot \text{plf}$$

$$w_{long.EQ.cold.form} := \frac{E_{eq.long.cold.form}}{L_{trans}} \quad w_{long.EQ.cold.form} = 112 \cdot \text{plf}$$

Metal stud shearwall

$$\Omega_V := 2.0 \quad \text{ASD Safety Factor (B5.2.3)}$$

Line 1 - Metal Stud Shearwall

$$L_{\text{wall.1}} := (24\text{ft} + 0\text{in}) = 24\text{ft}$$

$$H_{\text{wall}} := (13\text{ft} + 4\text{in}) = 13.333\text{ft}$$

$$\text{Trib}_1 := \frac{L_{\text{trans}}}{2} = 22.792\text{ft}$$

Check Type I Shearwall Requirements (AIS S240-2015):

$$\text{Condition}_a := \text{if} \left[\left(\frac{H_{\text{wall}}}{L_{\text{wall.1}}} \right) < 4.0, \text{"Satisfied"}, \text{"Not Satisfied"} \right] \quad \text{Condition}_a = \text{"Satisfied"}$$

$$\text{Condition}_b := \text{if} \left[(L_{\text{wall.1}}) > 24\text{in}, \text{"Satisfied"}, \text{"Not Satisfied"} \right] \quad \text{Condition}_b = \text{"Satisfied"}$$

Note: Since there are no wall openings, condition c or d do not apply.

Shearwall Forces:

$$V_{1.WL} := (0.6 \cdot w_{\text{long.WL}}) \cdot \text{Trib}_1$$

$$V_{1.WL} = 8915\text{lb}$$

$$v_{1.WL} := \frac{V_{1.WL}}{L_{\text{wall.1}}}$$

$$v_{1.WL} = 371 \cdot \text{plf} \quad \text{ASD}$$

Type I Shearwall Design (AIS S240-2015):

$$\frac{H_{\text{wall}}}{L_{\text{wall.1}}} = 0.556 \quad < 2.0, \text{ Use EQ. B5.2.2-1}$$

$$v_{\text{nom}} := 910 \cdot \text{plf} \quad \text{Table B5.2.2.3-2}$$

$$v_{\text{available}} := \frac{v_{\text{nom}}}{\Omega_V} \quad v_{\text{available}} = 455 \cdot \text{plf}$$

Note: Prototype uses 15/32" APA plywood, but Table B5.2.2.3-2 shows capacity of 15/32" Struct. 1 plywood. Since the applied shear force is very small, OK to check capacity using 7/16" OSB.

Check shearwall capacity:

$$\frac{v_{1.WL}}{v_{\text{available}}} = 0.816 \quad < 1.0, \text{ Therefore OK}$$

Use: 15/32" APA Plywood
w/ #8 screws @ 6/12 @ E/F
Type A Shearwall

Line 5 - Metal Stud Shearwall

$$L_{\text{wall}.5} := (31\text{ft} + 3\text{in}) = 31.25\text{ft}$$

$$H_{\text{wall}} := (13\text{ft} + 4\text{in}) = 13.333\text{ft}$$

$$\text{Trib}_5 := \frac{L_{\text{trans}}}{2} = 22.792\text{ft}$$

Check Type I Shearwall Requirements (AIS S240-2015):

$$\text{Condition}_a := \text{if} \left[\left(\frac{H_{\text{wall}}}{L_{\text{wall}.5}} \right) < 4.0, \text{"Satisfied"}, \text{"Not Satisfied"} \right] \quad \text{Condition}_a = \text{"Satisfied"}$$

$$\text{Condition}_b := \text{if} \left[(L_{\text{wall}.5}) > 24\text{in}, \text{"Satisfied"}, \text{"Not Satisfied"} \right] \quad \text{Condition}_b = \text{"Satisfied"}$$

Note: Since there are no wall openings, condition c or d do not apply.

Shearwall Forces:

$$V_{5.WL} := (0.6 \cdot w_{\text{long}.WL}) \cdot \text{Trib}_5$$

$$V_{5.WL} = 8915\text{lb}$$

$$v_{5.WL} := \frac{V_{5.WL}}{L_{\text{wall}.5}}$$

$$v_{5.WL} = 285 \cdot \text{plf} \quad \text{ASD}$$

Type I Shearwall Design (AIS S240-2015):

$$\frac{H_{\text{wall}}}{L_{\text{wall}.5}} = 0.427 \quad < 2.0, \text{ Use EQ. B5.2.2-1}$$

$$v_{\text{nom}} := 910 \cdot \text{plf} \quad \text{Table B5.2.2.3-2}$$

$$v_{\text{available}} := \frac{v_{\text{nom}}}{\Omega_V} \quad v_{\text{available}} = 455 \cdot \text{plf}$$

Note: Prototype uses 15/32" APA plywood, but Table B5.2.2.3-2 shows capacity of 15/32" Struct. 1 plywood. Since the applied shear force is very small, OK to check capacity using 7/16" OSB.

Check shearwall capacity:

$$\frac{v_{5.WL}}{v_{\text{available}}} = 0.627 \quad < 1.0, \text{ Therefore OK}$$

Use: 15/32" APA Plywood
w/ #8 screws @ 6/12 @ E/F
Type C Shearwall

Shear Anchorage Spacing with 1/2 diam. anchor bolts w/550T125-43 Sill Plate

Check sill track capacity for shear forces transferred from shear wall. This will be used to determine the anchor bolt spacing.

1/2" Dia. Simpson Titen HD Shear Design:

$$V_{\text{allow}} := 7455 \text{ lb}$$

Allowable Shear per Table 3
(ICC ESR-2713)

$$\frac{V_{\text{allow}}}{\max(v_{A.WL}, v_{1.WL}, v_{5.WL})} = 240.8 \text{ in}$$

Space anchors at 40" O.C. for all Shearwall Types

Bearing Strength Per Section J3.3.1 from AISI S100-2016:

$$d := 0.5 \text{ in} \quad \text{Diameter of bolt}$$

$$t := 0.0538 \text{ in} \quad \text{Thickness of sheet (in)}$$

$$F_u := 50 \text{ ksi} \quad \text{Tensile strength}$$

$$m_f := 1.0 \quad \text{Modification factor}$$

$$\frac{d}{t} = 9.294 \quad \text{Ratio of bolt diameter to sill thickness}$$

$$C := 4 - 0.1 \cdot \left(\frac{d}{t} \right) = 3.071 \quad \text{Bearing factor (Table J3.3.1-1)}$$

$$\Omega := 2.5 \quad \text{ASD safety factor}$$

$$P_{nb} := C \cdot m_f \cdot d \cdot t \cdot F_u \quad P_{nb} = 4130 \text{ lb}$$

$$P_{nb.\text{destrength}} := \frac{P_{nb}}{\Omega} \quad P_{nb.\text{destrength}} = 1652 \text{ lb}$$

$$S_{\text{spacing}} := 48 \text{ in}$$

$$V_{\text{anchor}} := \max(v_{A.WL}, v_{1.WL}, v_{5.WL}) \cdot S_{\text{spacing}} \quad V_{\text{anchor}} = 1485.781 \text{ lb}$$

$$\frac{V_{\text{anchor}}}{P_{nb.\text{destrength}}} = 0.899 \quad < 1.0, \text{ therefore OK}$$

Use: Simpson 1/2" Dia. Titen HD @ 48" O.C.

Shearwall Overturning Analysis

Transverse Direction

Line A - Wood Shearwall

$$L_{\text{wall.A}} = 24.917 \text{ ft}$$

Shearwall Length

$$V_{\text{A.WL}} = 9145 \cdot \text{lb}$$

Lateral Force Type - Wind

$$h_{\text{wall}} := (13\text{ft} + 4\text{in}) = 13.333 \text{ ft}$$

$$w_{\text{DL}} := DL_{\text{roof}} \cdot \left(\frac{32\text{in}}{2} \right) + (DL_{\text{wall.metal}} + DL_{\text{wall.veneer}}) \cdot [h_{\text{wall}} + (20.125\text{ft} - 14.50\text{ft})] = 1069 \cdot \text{plf}$$

Use smallest parapet
(Conservative)

Holdowns

$$l_{\text{wall}} := L_{\text{wall.A}}$$

$$OTM_{\text{WL}} := V_{\text{A.WL}} \cdot h_{\text{wall}}$$

$$OTM_{\text{WL}} = 121938 \text{ lb} \cdot \text{ft}$$

$$RM_{\text{WL}} := 0.6 \cdot w_{\text{DL}} \cdot \frac{l_{\text{wall}}^2}{2}$$

$$RM_{\text{WL}} = 199173 \text{ lb} \cdot \text{ft}$$

$$\text{Uplift}_{\text{WL}} := \frac{\max(OTM_{\text{WL}} - RM_{\text{WL}}, 0)}{l_{\text{wall}} - 1 \cdot \text{ft}}$$

$$\text{Uplift}_{\text{WL}} = 0 \text{ lb}$$

Use: Simpson S/HDU9 w/ 54 mil thick stud
w/ 7/8" Φ anchor bolt
Good For 6,750 lbs (factored)

Anchorage OK by inspection

Check anchor uplift

$$\alpha_{\text{WL}} := \frac{0.9 + 1.0}{0.6 + 0.6} = 1.583$$

ASD to LRFD Factor Conversion

$$\text{Uplift}_{\text{WL.anchor}} := \text{Uplift}_{\text{WL}} \cdot \alpha_{\text{WL}}$$

$$\text{Uplift}_{\text{WL.anchor}} = 0 \text{ lb}$$

Line 1 - Wood Shearwall

$$L_{\text{wall.1}} = 24 \text{ ft}$$

Shearwall Length

$$V_{1.WL} = 8915 \cdot \text{lb}$$

Lateral Force Type - Wind

$$h_{\text{wall}} := (13 \text{ ft} + 4 \text{ in}) = 13.333 \text{ ft}$$

$$w_{DL} := DL_{\text{roof}} \cdot \left(\frac{44.417 \text{ ft}}{2} \right) + (DL_{\text{wall.metal}} + DL_{\text{wall.veneer}}) \cdot [h_{\text{wall}} + (20.125 \text{ ft} - 14.50 \text{ ft})] = 1487 \cdot \text{plf}$$

Holdowns

Use smallest parapet
(Conservative)

$$l_{\text{wall}} := L_{\text{wall.A}}$$

$$OTM_{WL} := V_{1.WL} \cdot h_{\text{wall}}$$

$$OTM_{WL} = 118862 \text{ lb} \cdot \text{ft}$$

$$RM_{WL} := 0.6 \cdot w_{DL} \cdot \frac{l_{\text{wall}}^2}{2}$$

$$RM_{WL} = 276934 \text{ lb} \cdot \text{ft}$$

$$\text{Uplift}_{WL} := \frac{\max(OTM_{WL} - RM_{WL}, 0)}{l_{\text{wall}} - 1 \cdot \text{ft}}$$

$$\text{Uplift}_{WL} = 0 \text{ lb}$$

Use: Simpson S/HDU9 w/ 54 mil thick stud
w/ 7/8" Φ anchor bolt
Good For 6,750 lbs (factored)

Anchorage OK by inspection

Check anchor uplift

$$\alpha_{WL} := \frac{0.9 + 1.0}{0.6 + 0.6} = 1.583$$

ASD to LRFD Factor Conversion

$$\text{Uplift}_{WL.\text{anchor}} := \text{Uplift}_{WL} \cdot \alpha_{WL}$$

$$\text{Uplift}_{WL.\text{anchor}} = 0 \text{ lb}$$

Line 5 - Wood Shearwall

$$L_{\text{wall}.5} = 31.25 \text{ ft}$$

Shearwall Length

$$V_{1.WL} = 8915 \cdot \text{lb}$$

Lateral Force Type - Wind

$$h_{\text{wall}} := (13\text{ft} + 4\text{in}) = 13.333 \text{ ft}$$

$$w_{DL} := DL_{\text{roof}} \cdot \left(\frac{44.417\text{ft}}{2} \right) + (DL_{\text{wall}.metal} + DL_{\text{wall}.veneer}) \cdot [h_{\text{wall}} + (20.125\text{ft} - 14.50\text{ft})] = 1487 \cdot \text{plf}$$

Use smallest parapet
(Conservative)

Holdowns

$$l_{\text{wall}} := L_{\text{wall}.A}$$

$$OTM_{WL} := V_{5.WL} \cdot h_{\text{wall}}$$

$$OTM_{WL} = 118862 \text{ lb} \cdot \text{ft}$$

$$RM_{WL} := 0.6 \cdot w_{DL} \cdot \frac{l_{\text{wall}}^2}{2}$$

$$RM_{WL} = 276934 \text{ lb} \cdot \text{ft}$$

$$\text{Uplift}_{WL} := \frac{\max(OTM_{WL} - RM_{WL}, 0)}{l_{\text{wall}} - 1 \cdot \text{ft}}$$

$$\text{Uplift}_{WL} = 0 \text{ lb}$$

Use: Simpson S/HDU9 w/ 54 mil thick stud
w/ 7/8" Φ anchor bolt
Good For 6,750 lbs (factored)

Anchorage OK by inspection

Check anchor uplift

$$\alpha_{WL} := \frac{0.9 + 1.0}{0.6 + 0.6} = 1.583$$

ASD to LRFD Factor Conversion

$$\text{Uplift}_{WL.anchor} := \text{Uplift}_{WL} \cdot \alpha_{WL}$$

$$\text{Uplift}_{WL.anchor} = 0 \text{ lb}$$

Diaphragm Loading

Determine Seismic Diaphragm Forces

$$I := 1.0$$

$$w_{px} := 1.0 \quad \text{For single story building}$$

$$S_{ds} = 0.101 \quad \text{Ref. Seismic Factor Determination}$$

$$C_{s.frame} = 0.034 \quad \text{Ref. Seismic Factor Determination (transverse)}$$

$$C_{s.cold.form} = 0.016 \quad \text{Ref. Seismic Factor Determination (longitudinal)}$$

$$\rho := 1.0 \quad \text{For Diaphragms } \rho = 1.0$$

$$\Omega_{Seismic} := 2.0 \quad \text{ASD Reduction Factor (Seismic)}$$

$$\Omega_{Wind} := 2.0 \quad \text{ASD Reduction Factor (Wind)}$$

Check minimum and maximum diaphragm loading per ASCE 7-16 section 12.10.1.1

$$F_{px.max} := 0.4 \cdot S_{ds} \cdot I \cdot w_{px} \cdot \rho \quad F_{px.max} = 0.041 \quad (\text{Base Shear factor, max.})$$

$$F_{px.min} := 0.2 \cdot S_{ds} \cdot I \cdot w_{px} \cdot \rho \quad F_{px.min} = 0.02 \quad (\text{Base Shear factor, min.})$$

$$\text{Thus: } V_{diaph.trans} := \text{if}(C_{s.frame} > F_{px.max}, F_{px.max}, \max(C_{s.frame}, F_{px.min}))$$

$$V_{diaph.trans} = 0.034$$

$$V_{diaph.long} := \text{if}(C_{s.cold.form} > F_{px.max}, F_{px.max}, \max(C_{s.cold.form}, F_{px.min}))$$

$$V_{diaph.long} = 0.02$$

Weight of Structure (calculated previously)

$$WT_{\text{total}} = 327208 \text{ lb}$$

$$F_{\text{diaph.trans}} := V_{\text{diaph.trans}} \cdot WT_{\text{total}}$$

$$F_{\text{diaph.trans}} = 11052 \text{ lb}$$

$$WL_{\text{trans}} = 55870 \text{ lb}$$

$$F_{\text{diaph.long}} := V_{\text{diaph.long}} \cdot WT_{\text{total}}$$

$$F_{\text{diaph.long}} = 6631 \text{ lb}$$

$$WL_{\text{long}} = 29716 \text{ lb}$$

Transverse Direction

$$0.7 \cdot F_{\text{diaph.trans}} = 7737 \text{ lb} \quad \text{Seismic Force}$$

$$0.6 \cdot WL_{\text{trans}} = 33522 \text{ lb} \quad \text{Wind Force}$$

Wind Forces Govern

Longitudinal Direction

$$0.7 \cdot F_{\text{diaph.long}} = 4642 \text{ lb} \quad \text{Seismic Force}$$

$$0.6 \cdot WL_{\text{long}} = 17829 \text{ lb} \quad \text{Wind Force}$$

Wind Forces Govern

Wind force governs both directions

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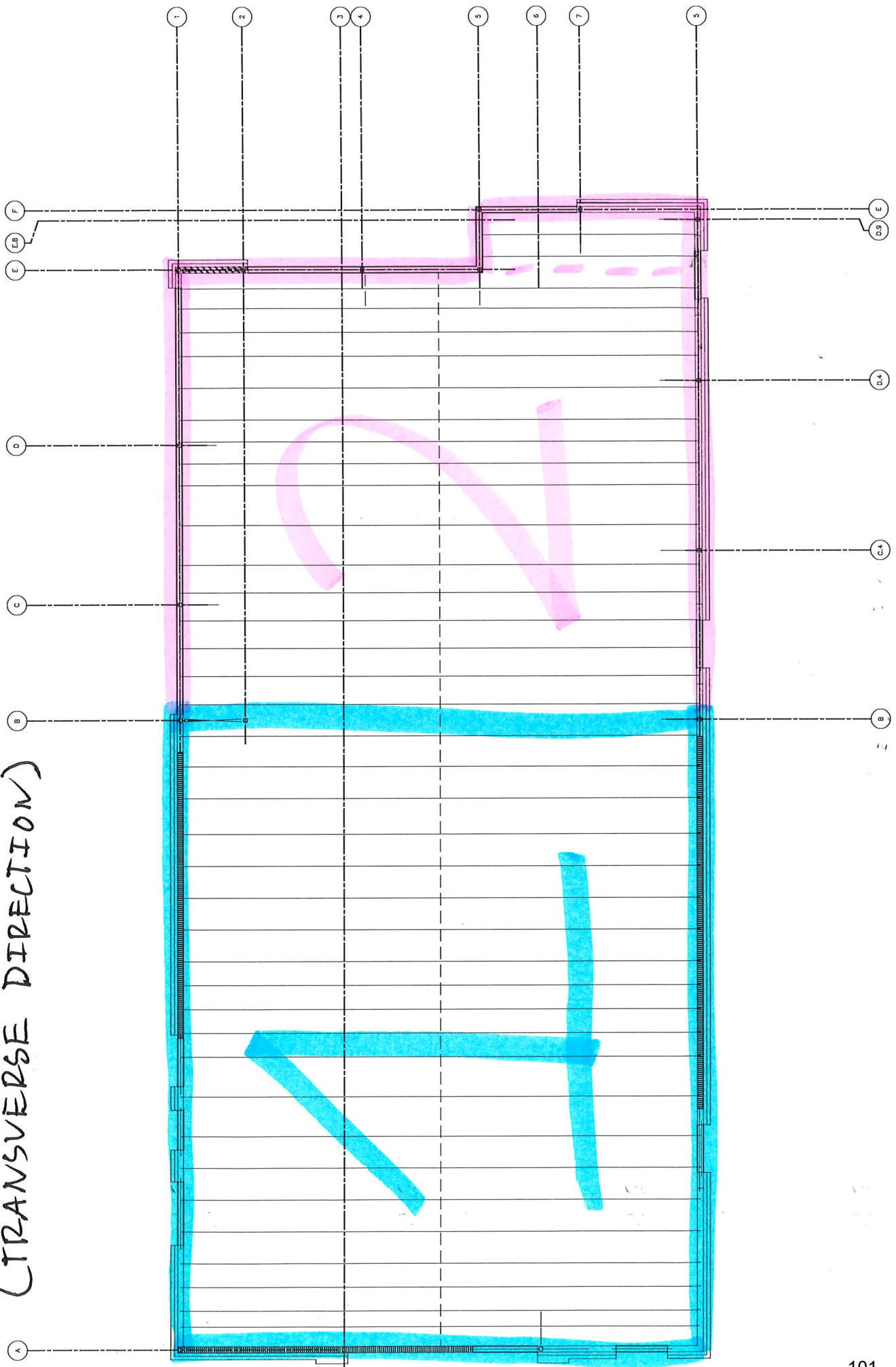
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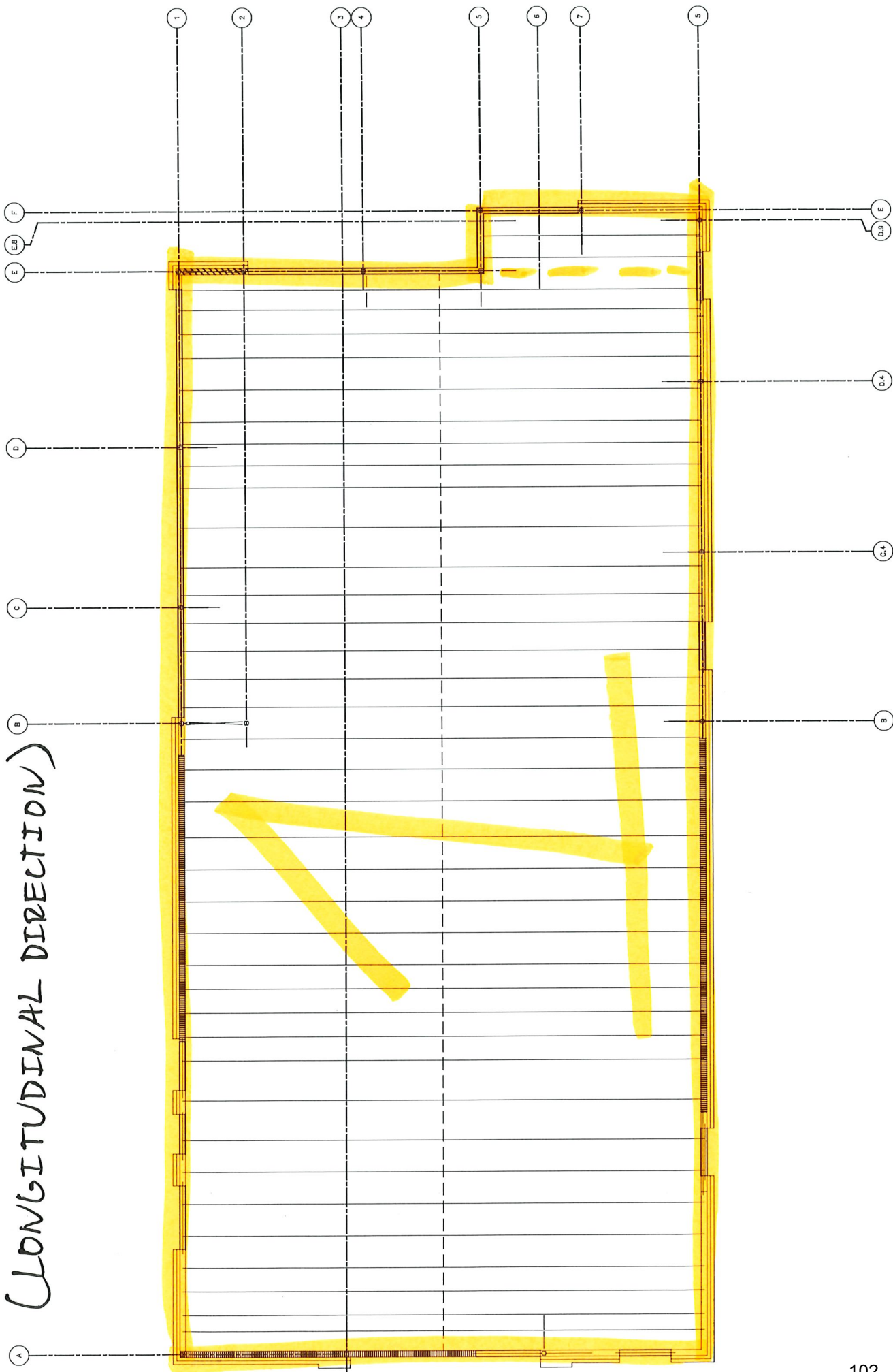
File Name: 251023 - Metal Stud.xmcd

Engineer: RV

DIAPHRAGM KEY PLAN (TRANSVERSE DIRECTION)



DIAPHRAGM KEY PLAN (LONGITUDINAL DIRECTION)



Diaphragm Loads: Transverse Direction

In the NS force direction, the main building roof diaphragm is separated into two sub-diaphragms along gridlines A, B, and F. Wind loads govern in this direction.

$$WL_{trans} = 55870 \text{ lb} \quad \text{Unfactored wind diaphragm force}$$

$$0.6 \cdot WL_{trans} = 33522 \text{ lb} \quad \text{Factored wind diaphragm force}$$

Sub-diaphragm 1:

Sub-diaphragm 1 is located along gridlines A and B

$$L_{long.1} := (53\text{ft} + 0.75\text{in}) = 53.063 \text{ ft}$$

$$L_{trans.1} := (44\text{ft} + 5\text{in}) = 44.417 \text{ ft}$$

$$diaph_{ratio} := \frac{L_{long.1}}{L_{trans.1}} = 1.195$$

diaphragm span to depth ratio, max = 3
for unblocked diaphragm

$$w_{trans.WL} := \frac{WL_{trans}}{L_{long}}$$

$$w_{trans.WL} = 574.5 \cdot \text{plf}$$

$$Trib_{diaph.AB} := \frac{L_{long.1}}{2}$$

$$Trib_{diaph.AB} = 26.531 \text{ ft}$$

$$V_{diaph.1} := 0.6 \cdot (w_{trans.WL} \cdot Trib_{diaph.AB})$$

$$V_{diaph.1} = 9145.3 \text{ lb}$$

$$v_{diaph.1} := \frac{V_{diaph.1}}{L_{trans.1}}$$

$$v_{diaph.1} = 205.9 \cdot \text{plf}$$

$$v_{diaph.capacity.1} := \left(\frac{895 \text{ plf}}{\Omega_{Wind}} \right) = 447.5 \cdot \text{plf}$$

2015 SDPWS Table 4.2C
Unblocked Diaphragm Capacity

$$\frac{v_{diaph.1}}{v_{diaph.capacity.1}} = 0.46 < 1.0 \text{ Diaphragm OK}$$

Use: 23/32" APA Rated Sheathing **Unblocked Diaphragm**
w/ 10d @ 6"/12" @ B/F GF ~ 447.5 plf (Wind)
SDPWS Table 4.2C, Case 1

Sub-diaphragm 2:

Sub-diaphragm 2 is located along gridlines B and E

$$L_{\text{long.2.a}} := (38\text{ft} + 0.25\text{in}) = 38.021\text{ ft}$$

$$L_{\text{long.2.b}} := 5\text{ft}$$

$$L_{\text{trans.2}} := (44\text{ft} + 5\text{in}) = 44.417\text{ ft}$$

$$\text{diaph}_{\text{ratio}} := \frac{(L_{\text{long.2.a}} + L_{\text{long.2.b}})}{L_{\text{trans.2}}} = 0.969$$

diaphragm span to depth ratio, max = 3
for unblocked diaphragm

$$w_{\text{trans.WL}} := \frac{WL_{\text{trans}}}{L_{\text{long}}}$$

$$w_{\text{trans.WL}} = 574.5 \cdot \text{plf}$$

$$\text{Trib}_{\text{diaph.B}} := \left(\frac{L_{\text{long.2.a}}}{2} \right)$$

$$\text{Trib}_{\text{diaph.B}} = 19.01\text{ ft}$$

$$\text{Trib}_{\text{diaph.E}} := \left(\frac{L_{\text{long.2.a}}}{2} \right) + L_{\text{long.2.b}}$$

$$\text{Trib}_{\text{diaph.E}} = 24.01\text{ ft}$$

$$V_{\text{diaph.B}} := 0.6 \cdot (w_{\text{trans.WL}} \cdot \text{Trib}_{\text{diaph.B}})$$

$$V_{\text{diaph.B}} = 6552.9\text{ lb}$$

$$V_{\text{diaph.E}} := 0.6 \cdot (w_{\text{trans.WL}} \cdot \text{Trib}_{\text{diaph.E}})$$

$$V_{\text{diaph.E}} = 8276.4\text{ lb}$$

$$v_{\text{diaph.2.a}} := \frac{V_{\text{diaph.B}}}{L_{\text{trans.2}}}$$

$$v_{\text{diaph.2.a}} = 147.5 \cdot \text{plf}$$

$$v_{\text{diaph.2.b}} := \frac{V_{\text{diaph.E}}}{L_{\text{trans.2}}}$$

$$v_{\text{diaph.2.b}} = 186.3 \cdot \text{plf}$$

$$v_{\text{diaph.capacity.2}} := \left(\frac{895\text{plf}}{\Omega_{\text{Wind}}} \right) = 447.5 \cdot \text{plf}$$

2015 SDPWS Table 4.2C
Unblocked Diaphragm Capacity

$$\frac{\max(v_{\text{diaph.2.a}}, v_{\text{diaph.2.b}})}{v_{\text{diaph.capacity.2}}} = 0.416$$

< 1.0 Diaphragm OK

Use: 23/32" APA Rated Sheathing **Unblocked Diaphragm**
w/ 10d @ 6"/12" @ B/F GF ~ 447.5 plf (Wind)
SDPWS Table 4.2C, Case 1

Diaphragm Loads: Longitudinal Direction

In the longitudinal direction, the roof diaphragm is supported by exterior cold form metal with wood shearwalls along gridlines 1 and 5. Wind loads govern in this direction.

$$WL_{\text{long}} = 29715.624 \text{ lb} \quad \text{Unfactored wind diaphragm force}$$

$$0.6 \cdot WL_{\text{long}} = 17829 \text{ lb} \quad \text{Factored wind diaphragm force}$$

$$L_{\text{long.a}} := (91\text{ft} + 1\text{in}) = 91.083 \text{ ft} \quad L_{\text{long.b}} := (96\text{ft} + 1\text{in}) = 96.083 \text{ ft} \quad L_{\text{trans}} := (44\text{ft} + 5\text{in}) = 44.417 \text{ ft}$$

$$\text{diaph}_{\text{ratio}} := \frac{L_{\text{trans}}}{L_{\text{long}}} = 0.457 \quad \text{diaphragm span to depth ratio, max} = 3 \text{ for unblocked diaphragm}$$

$$\text{Trib}_{\text{diaph}} := \frac{L_{\text{trans}}}{2} = 22.208 \text{ ft}$$

$$w_{\text{long.WL}} := \frac{WL_{\text{long}}}{L_{\text{trans}}} \quad w_{\text{long.WL}} = 669 \cdot \text{plf}$$

$$V_{\text{diaph.15}} := 0.6 \cdot (w_{\text{long.WL}} \cdot \text{Trib}_{\text{diaph}}) \quad V_{\text{diaph.15}} = 8914.7 \text{ lb}$$

$$v_{\text{diaph.1}} := \frac{V_{\text{diaph.15}}}{L_{\text{long.a}}} \quad v_{\text{diaph.1}} = 97.9 \cdot \text{plf}$$

$$v_{\text{diaph.5}} := \frac{V_{\text{diaph.15}}}{L_{\text{long.b}}} \quad v_{\text{diaph.5}} = 92.8 \cdot \text{plf}$$

$$v_{\text{diaph.capacity}} := \left(\frac{670 \text{ plf}}{\Omega_{\text{Wind}}} \right) = 335 \cdot \text{plf} \quad \begin{array}{l} \text{2015 SDPWS Table 4.2C} \\ \text{Unblocked Diaphragm Capacity} \end{array}$$

$$\frac{\max(v_{\text{diaph.1}}, v_{\text{diaph.5}})}{v_{\text{diaph.capacity}}} = 0.292 < 1.0 \text{ Diaphragm OK}$$

Use 23/32" APA Rated Sheathing **Unblocked Diaphragm**
w/ 10d @ 6/12 @ B/F
Good for 335 plf (Wind - Case 3)

Chords, Collectors and Top Plate Splice

Chord Forces: Transverse

The largest chord forces will occur on walls at Lines 1 and 5 along sub-diaphragm 1 or 2.
Determine chord force.

$$d := L_{\text{trans}} = 44.417 \text{ ft} \quad \text{depth of diaphragm from line 1 to line 5}$$

$$T_{1.1} := \frac{\left[\frac{0.6 \cdot w_{\text{trans}} \cdot WL \cdot (L_{\text{long}.1})^2}{8} \right]}{d} \quad T_{1.1} = 2.731 \cdot k$$

$$T_{1.2} := \frac{\left[\frac{0.6 \cdot w_{\text{trans}} \cdot WL \cdot (L_{\text{long}.2.a} + L_{\text{long}.2.b})^2}{8} \right]}{d} \quad T_{1.2} = 1.795 \cdot k$$

$$T_1 := \max(T_{1.1}, T_{1.2}) \quad T_1 = 2.731 \cdot k$$

$$T_5 := T_1 \quad T_5 = 2.731 \cdot k$$

Chord Forces: Longitudinal

The largest chord force will occur between lines A and E. Determine chord force

$$d := L_{\text{long}} = 97.25 \text{ ft} \quad \text{depth of diaphragm from line A to line E}$$

$$T_A := \frac{\left[\frac{0.6 \cdot w_{\text{long}} \cdot WL \cdot (L_{\text{trans}})^2}{8} \right]}{d} \quad T_A = 1.018 \cdot k$$

$$T_E := T_A \quad T_E = 1.018 \cdot k$$

Collector At Line A - Transverse

$$v_{\text{diaph.1}} = 98 \cdot \text{plf}$$

$$L_{\text{trans}} = 44.417 \text{ ft}$$

$$v_{\text{wall}} := \frac{v_{\text{diaph.1}} \cdot L_{\text{trans}}}{(24\text{ft} + 11\text{in})}$$

$$v_{\text{wall}} = 174 \cdot \text{plf}$$

Roof Collector Lengths

$$L_1 := 24\text{ft} + 11\text{in}$$

Length of roof along shearwall

$$L_2 := 19\text{ft} + 6\text{in}$$

Length of roof from shearwall to end

Roof Collector Forces

$$F_{A.1} := (v_{\text{diaph.1}} \cdot L_1) - (v_{\text{wall}} \cdot L_1)$$

$$F_{A.1} = -1909 \text{ lb}$$

$$F_{A.2} := (F_{A.1}) + (v_{\text{diaph.1}} \cdot L_2)$$

$$F_{A.2} = 0 \text{ lb}$$

$$\text{Collector}_A := \max(|F_{A.1}|, |F_{A.2}|)$$

$$\text{Collector}_A = 1909 \text{ lb}$$

Collector At Line B

At Line D there is a collector consisting of (2) 1 3/4" x 11 7/8" LVL beams laid flat and attached to the bottom of the roof plywood. The plywood is to be nailed to the LVL beams and the beams will attach to the braced frame. All forces are wind loading. Determine the collector force and its connection.

$$L_{\text{long.1}} = 53.063 \text{ ft} \quad L_{\text{long.2.a}} = 38.021 \text{ ft}$$

$$F_{\text{collector.B}} := 0.6 \cdot w_{\text{trans.WL}} \cdot \left(\frac{L_{\text{long.1}} + L_{\text{long.2.a}}}{2} \right) \quad F_{\text{collector.B}} = 15698 \text{ lb}$$

Use (4) MST160 - Good for (5,070 lbs each - 20,280 lbs total) at ridge
See the following connection design at collector

$$\frac{F_{\text{collector.B}}}{0.6} = 26.164 \cdot k \quad \text{Unfactored wind load at drag strut}$$

**OK to use (2) 1.75" x 11-7/8" LVL
See attached**

Check diaphragm nailing to drag strut

$$F_{\text{collector.B}} = 15698 \text{ lb} \quad \text{collector force ASD along Line B}$$

$$Z_{\text{SDS}} := 170 \cdot \text{lb} \quad \text{per SDS Load tables for (3/4" thk - plywood) Douglas Fir}$$

$$C_d := 1.6$$

$$Z_{\text{SDS}} := Z_{\text{SDS}} \cdot C_d \quad Z_{\text{SDS}} = 272 \text{ lb}$$

$$N_{\text{nails.req}} := \text{ceil} \left(\frac{F_{\text{collector.B}}}{Z_{\text{SDS}}} \right) \quad N_{\text{nails.req}} = 58$$

OK to use (90) 1/4" SDS screws at Line B from plywood to LVL

Check the connection of the 5/8" Lag screws to the (2) LVL flat collector to BF Col

$$D := 0.625\text{in}$$

nominal screw diameter

$$F_{\text{collector.B}} = 15698\text{ lb}$$

collector force ASD along Line B, see previous analysis

$$Z_{\text{screw}} := 750\cdot\text{lb}$$

5/8" Lag Screw to 1/4" plate per NDS Table 12K

$$C_d := 1.6$$

Duration Factor

$$l_{\text{pen}} := 3.5\text{in} - 0.25\text{in}$$

$$l_{\text{pen}} = 3.25\cdot\text{in}$$

$$d_{\text{screw}} := 0.471\cdot\text{in}$$

reduced body diameter for 5/8" lag screw per NDS Table L2

$$p_{\text{en}} := \frac{l_{\text{pen}}}{8\cdot d_{\text{screw}}}$$

$$p_{\text{en}} = 0.863 \quad \text{reduced penetration factor}$$

$$F_{\text{screw}} := Z_{\text{screw}}\cdot C_d\cdot p_{\text{en}}$$

$$F_{\text{screw}} = 1035\text{ lb}$$

$$N := 22$$

Number of screws at connection

$$\frac{F_{\text{collector.B}}}{N\cdot(F_{\text{screw}})} = 0.689$$

< 1.0, LVL to PL at BF col OK

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Print Date: 8/3/2026

File Name: 251023 - Metal Stud.xmcd

Engineer: RV

Check welded connection from 1/4"x10" plate to BF Col

$$F_{y,plate} := 50 \cdot \text{ksi}$$

$$F_{y,column} := 46 \cdot \text{ksi}$$

$$F_{exx} := 70 \cdot \text{ksi}$$

$$F_{exx} = 70 \cdot \text{ksi}$$

$$F_{conn} := F_{collector.B}$$

$$F_{conn} = 15698.259 \text{ lb}$$

$$\Omega_w := 2.0$$

$$\Omega_y := 1.67$$

The welds will be 3/16" fillet welds to three sides of the HSS 4x4x5/16 column.

$$L_{weld} := 2[4 \cdot \text{in} + 2 \cdot (4 \cdot \text{in})]$$

$$L_{weld} = 24 \cdot \text{in}$$

$$A_{weld} := \frac{3}{16} \cdot \text{in} \cdot \frac{1}{\sqrt{2}} \cdot L_{weld}$$

$$A_{weld} = 3.182 \cdot \text{in}^2$$

$$A_{plate} := 0.25 \cdot \text{in} \cdot (10 \cdot \text{in})$$

$$A_{plate} = 2.5 \cdot \text{in}^2$$

$$R_{n,weld} := \frac{0.6 \cdot F_{exx} \cdot A_{weld}}{\Omega_w}$$

$$R_{n,weld} = 66.822 \cdot \text{k}$$

$$R_{n,plate} := \frac{F_{y,plate} \cdot A_{plate}}{\Omega_y}$$

$$R_{n,plate} = 74.85 \cdot \text{k}$$

$$R_n := \min(R_{n,weld}, R_{n,plate})$$

$$R_n = 66.822 \cdot \text{k}$$

$$\frac{F_{conn}}{R_n} = 0.235 \quad < 1.0 \text{ connection ok}$$

Collector At Line E

At Line G there is a collector consisting of a 1 3/4" x 7.25" LVL ledger and attached to the bottom of the roof plywood. The plywood is to be nailed to the LVL ledger and the ledger will attach to the braced frame at Line D. All forces are wind loading. Determine the collector force and its connection.

$$L_{\text{long.2.a}} = 38.021 \text{ ft} \quad L_{\text{long.2.b}} = 5 \text{ ft}$$

$$F_{\text{collector.E}} := 0.6 \cdot w_{\text{trans.WL}} \cdot \left(\frac{L_{\text{long.2.a}}}{2} + L_{\text{long.2.b}} \right) \quad F_{\text{collector.E}} = 8276 \text{ lb}$$

Use (2) MSTI60 - Good for (5,070 lbs each - 10,140 lbs total) at ridge
See the following connection design at collector

$$\frac{F_{\text{collector.E}}}{0.6} = 13.794 \cdot k \quad \text{Unfactored wind load at drag strut}$$

OK to use 1.75" x 7-1/4" LVL
See attached

Check diaphragm nailing to drag strut

$$d_1 := (44 \text{ ft} + 5 \text{ in}) \quad \text{Width of Building at Line E}$$

$$Z_{10d} := 103 \cdot \text{lb} \quad 10d \text{ nail allowable shear value}$$

$$C_d := 1.6 \quad \text{Duration Factor}$$

$$F_{10d} := Z_{10d} \cdot C_d \quad F_{10d} = 164.8 \text{ lb}$$

$$N := \text{ceil} \left(\frac{d_1 - 5.5 \text{ ft}}{6 \cdot \text{in}} \right) + 2 \quad \text{number of nails along Line D}$$

$$N = 80$$

$$\frac{F_{\text{collector.E}}}{N \cdot (F_{10d})} = 0.628 \quad < 1.0 \text{ nailing plywood to collector ok}$$

Use 10d nails at 6" o.c. spacing

Collector At Line 1 - Longitudinal

$$v_{\text{diaph.1}} = 98 \cdot \text{plf}$$

$$L_{\text{long.a}} = 91.083 \text{ ft}$$

$$v_{\text{wall}} := \frac{(v_{\text{diaph.1}}) \cdot (L_{\text{long.a}})}{(24\text{ft} + 0\text{in})}$$

$$v_{\text{wall}} = 371 \cdot \text{plf}$$

Roof Collector Lengths

$$L_1 := 26\text{ft} + 5\text{in}$$

Length of roof from beginning to shearwall

$$L_2 := 24\text{ft} + 0\text{in}$$

Length of roof along shearwall

$$L_3 := 40\text{ft} + 8\text{in}$$

Length of roof from shearwall to end

Roof Collector Forces

$$F_{1.1} := (v_{\text{diaph.1}} \cdot L_1)$$

$$F_{1.1} = 2586 \text{ lb}$$

$$F_{1.2} := (F_{1.1}) + (v_{\text{diaph.1}} \cdot L_2) - (v_{\text{wall}} \cdot L_2)$$

$$F_{1.2} = -3980 \text{ lb}$$

$$F_{1.3} := (F_{1.2}) + (v_{\text{diaph.1}} \cdot L_3)$$

$$F_{1.3} = 0 \text{ lb}$$

$$\text{Collector}_1 := \max(|F_{1.1}|, |F_{1.2}|, |F_{1.3}|)$$

$$\text{Collector}_1 = 3980 \text{ lb}$$

Collector At Line 5 - Longitudinal

$$v_{\text{diaph.5}} = 93 \cdot \text{plf}$$

$$L_{\text{long.b}} = 96.083 \text{ ft}$$

$$v_{\text{wall}} := \frac{(v_{\text{diaph.5}}) \cdot (L_{\text{long.b}})}{(31\text{ft} + 3\text{in})}$$

$$v_{\text{wall}} = 285 \cdot \text{plf}$$

Roof Collector Lengths

$$L_1 := 20\text{ft} + 4.50\text{in}$$

Length of roof from beginning to shearwall

$$L_2 := 31\text{ft} + 3\text{in}$$

Length of roof along shearwall

$$L_3 := 44\text{ft} + 5.50\text{in}$$

Length of roof from shearwall to end

Roof Collector Forces

$$F_{5.1} := (v_{\text{diaph.5}} \cdot L_1)$$

$$F_{5.1} = 1890 \text{ lb}$$

$$F_{5.2} := (F_{5.1}) + (v_{\text{diaph.5}} \cdot L_2) - (v_{\text{wall}} \cdot L_2)$$

$$F_{5.2} = -4125 \text{ lb}$$

$$F_{5.3} := (F_{5.2}) + (v_{\text{diaph.5}} \cdot L_3)$$

$$F_{5.3} = 0 \text{ lb}$$

$$\text{Collector}_5 := \max(|F_{5.1}|, |F_{5.2}|, |F_{5.3}|)$$

$$\text{Collector}_5 = 4125 \text{ lb}$$

Check Top Plate Splices:

Collector Forces

$$\text{Collector}_A = 1909 \text{ lb}$$

$$\text{Collector}_1 = 3980 \text{ lb}$$

$$\text{Collector}_5 = 4125 \text{ lb}$$

Chord Forces

$$T_{AE} := \max(T_A, T_E) = 1018 \text{ lb}$$

$$TC_{15} := \max(T_1, T_5) = 2731 \text{ lb}$$

Line 1 & 2

$$F_{15.\text{design}} := \max(\text{Collector}_1, \text{Collector}_5, TC_{15}) \quad F_{15.\text{design}} = 4125 \text{ lb}$$

Line A & B

$$F_{AE.\text{design}} := \max(\text{Collector}_A, T_{AE}, TC_{15}) \quad F_{AE.\text{design}} = 2731 \text{ lb}$$

Check Top Splice Nailing

$$Z_{16d} := 141 \cdot \text{lb}$$

$$C_d := 1.6$$

$$p_{16d} := \frac{1.5 \cdot \text{in}}{10 \cdot (0.162 \cdot \text{in})} \quad p_{16d} = 0.926$$

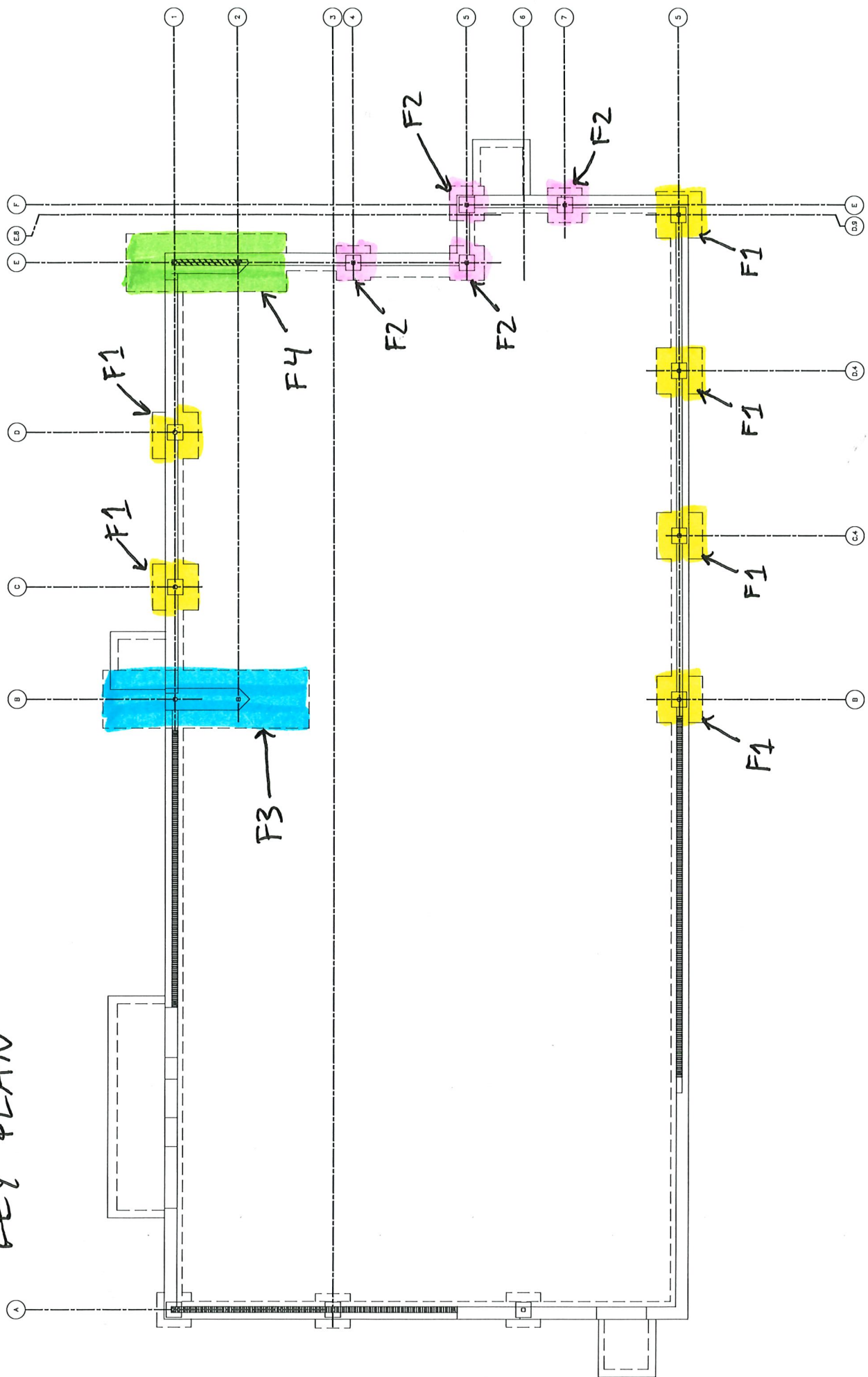
$$Z_{\text{prime}} := Z_{16d} \cdot C_d \cdot p_{16d} \quad Z_{\text{prime}} = 209 \text{ lb}$$

$$N_{\text{req}} := \max\left(\frac{F_{15.\text{design}}}{Z_{\text{prime}}}, \frac{F_{AE.\text{design}}}{Z_{\text{prime}}}\right) \quad N_{\text{req}} = 20$$

Use: 2 Rows of 22-16d Nails (44-16d Total)
at Top Plate Splices

FOUNDATION

KEY PLAN



Foundation

Soil Bearing Pressure - per IBC Minimum

$$q_{\text{base}} := 2000 \cdot \text{psf}$$

Dead + Live/Snow load

(per Geotechnical Report)

Footing F1 - Pad Footing at Columns C3 & C4

$$P_{DL} := \max(P_{DL.C3}, P_{DL.C4}) = 19945 \text{ lb}$$

$$V_{DL} := \max(V_{DL.C3}, V_{DL.C4}) = 354 \text{ lb}$$

$$P_{LL} := \max(P_{LL.C3}, P_{LL.C4}) = 5914 \text{ lb}$$

$$V_{LL} := \max(V_{LL.C3}, V_{LL.C4}) = 223 \text{ lb}$$

$$P_{SL} := \max(P_{SL.C3}, P_{SL.C4}) = 8736 \text{ lb}$$

$$V_{SL} := \max(V_{SL.C3}, V_{SL.C4}) = 286 \text{ lb}$$

$$V_{WL} := \max(V_{WL.C3}, V_{WL.C4}) = 392 \text{ lb}$$

Use: 5'-0" x 5'-0" x 1'-0" Footing (min)
w/ (5) #5 EW at Bottom

Use: 1'-4" x 1'-4" x 24" pier
See attached

Footing F2 - Pad Footing at Columns C1

$$P_{DL} := (P_{DL.C1}) = 7979 \text{ lb}$$

$$V_{DL} := (V_{DL.C1}) = 222 \text{ lb}$$

$$P_{LL} := (P_{LL.C1}) = 686 \text{ lb}$$

$$V_{LL} := (V_{LL.C1}) = 72 \text{ lb}$$

$$P_{SL} := (P_{SL.C1}) = 2672 \text{ lb}$$

$$V_{SL} := (V_{SL.C1}) = 94 \text{ lb}$$

$$V_{WL} := (V_{WL.C1}) = 276 \text{ lb}$$

Use: 3'-0" x 3'-0" x 1'-0" Footing (min)
w/ (4) #5 EW at Bottom

Use: 1'-4" x 1'-4" x 24" pier
See attached

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

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DESCRIPTION: Footing F1 - Pad Footing at Columns C3 & C4

Code References

Governing Code : IBC 2018, CBC 2019

Referenced Design Standard(s) : ACI 318-14

Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	Yes
Use Pedestal wt for stability, mom & shear	:	Yes

Soil Design Values

Allowable Soil Bearing	=	2.0 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.250

Increases based on footing Depth

Footing base depth below soil surface	=	3.0 ft
Allow press. increase per foot of depth when footing base is below	=	ksf ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	
when max. length or width is greater than	=	ksf ft

Dimensions

Width parallel to X-X Axis	=	5.0 ft
Length parallel to Z-Z Axis	=	5.0 ft
Footing Thickness	=	12.0 in

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a

Pedestal dimensions...

px : parallel to X-X Axis	=	16.0 in
pz : parallel to Z-Z Axis	=	16.0 in
Height	=	24.0 in

Bottom Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

Number of Bars	=	5.0
Reinforcing Bar Size	=	# 5

Bars parallel to Z-Z Axis (resisting X Flexure)

Number of Bars	=	# 5.0
Reinforcing Bar Size	=	# 5

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in
----------------------	---	--------

Applied Loads

		D	Lr	L	S	W	E	H
P : Vertical Load	=	19.945	5.914	0.000	8.736	0.000	0.000	0.000 k
OB : Overburden	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 ksf
M-xx	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k-ft
M-zz	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k-ft
V-x	=	0.354	0.223	0.000	0.286	0.392	0.000	0.000 k
V-z	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F1 - Pad Footing at Columns C3 & C4

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8040	Soil Bearing	1.608 ksf	2.0 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	32.612	Overturning - Z-Z	1.340 k-ft	43.790 k-ft	+0.60D+0.60W
PASS	16.192	Sliding - X-X	0.7449 k	12.062 k	+D+0.750S+0.450W
PASS	999.0	Sliding - Z-Z	0.0 k	10.424 k	D Only
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.2271	Z Flexure (+X) Bot Tens	2.755 k-ft/ft	12.131 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.1976	Z Flexure (-X) Bot Tens	2.397 k-ft/ft	12.131 k-ft/ft	+1.20D+1.60S
PASS	0.836	Min Steel X Flexure Bottom	0.259 in2/ft	0.310 in2/ft	n/a
PASS	0.2109	X Flexure (+Z) Bot Tens	2.558 k-ft/ft	12.131 k-ft/ft	+1.20D+1.60S
PASS	0.2109	X Flexure (-Z) Bot Tens	2.558 k-ft/ft	12.131 k-ft/ft	+1.20D+1.60S
PASS	0.836	Min Steel Z Flexure Bottom	0.259 in2/ft	0.310 in2/ft	n/a
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	Z Flexure (-X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel X Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel Z Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.2038	1-way Shear (+X)	16.740 psi	82.158 psi	+1.20D+1.60S+0.50W
PASS	0.1765	1-way Shear (-X)	14.497 psi	82.158 psi	+1.20D+1.60S
PASS	0.1887	1-way Shear (+Z)	15.506 psi	82.158 psi	+1.20D+1.60S
PASS	0.1887	1-way Shear (-Z)	15.506 psi	82.158 psi	+1.20D+1.60S
PASS	0.2120	2-way Punching	34.830 psi	164.317 psi	+1.20D+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc		Zecc		Actual Soil Bearing Stress @ Location				Actual / Allow Ratio
			(in)			Bottom, -Z	Top, +Z	Left, -X	Right, +X	
X-X, D Only	2.0	n/a	0.0	1.168	1.168	n/a	n/a	n/a	n/a	0.584
X-X, +D+Lr	2.0	n/a	0.0	1.404	1.404	n/a	n/a	n/a	n/a	0.702
X-X, +D+S	2.0	n/a	0.0	1.517	1.517	n/a	n/a	n/a	n/a	0.759
X-X, +D+0.750Lr	2.0	n/a	0.0	1.345	1.345	n/a	n/a	n/a	n/a	0.673
X-X, +D+0.750S	2.0	n/a	0.0	1.430	1.430	n/a	n/a	n/a	n/a	0.715
X-X, +D+0.60W	2.0	n/a	0.0	1.168	1.168	n/a	n/a	n/a	n/a	0.584
X-X, +D+0.750Lr+0.450W	2.0	n/a	0.0	1.345	1.345	n/a	n/a	n/a	n/a	0.673
X-X, +D+0.750S+0.450W	2.0	n/a	0.0	1.430	1.430	n/a	n/a	n/a	n/a	0.715
X-X, +0.60D+0.60W	2.0	n/a	0.0	0.7007	0.7007	n/a	n/a	n/a	n/a	0.350
X-X, +0.60D	2.0	n/a	0.0	0.7007	0.7007	n/a	n/a	n/a	n/a	0.350
Z-Z, D Only	2.0	0.4365	n/a	n/a	n/a	1.117	1.218			0.609
Z-Z, +D+Lr	2.0	0.5917	n/a	n/a	n/a	1.322	1.487			0.744
Z-Z, +D+S	2.0	0.6074	n/a	n/a	n/a	1.426	1.608			0.804
Z-Z, +D+0.750Lr	2.0	0.5580	n/a	n/a	n/a	1.271	1.420			0.710
Z-Z, +D+0.750S	2.0	0.5725	n/a	n/a	n/a	1.349	1.511			0.756
Z-Z, +D+0.60W	2.0	0.7265	n/a	n/a	n/a	1.084	1.252			0.626
Z-Z, +D+0.750Lr+0.450W	2.0	0.7468	n/a	n/a	n/a	1.246	1.445			0.723
Z-Z, +D+0.750S+0.450W	2.0	0.7502	n/a	n/a	n/a	1.324	1.536			0.768
Z-Z, +0.60D+0.60W	2.0	0.9199	n/a	n/a	n/a	0.6369	0.7645			0.382
Z-Z, +0.60D	2.0	0.4365	n/a	n/a	n/a	0.6704	0.7309			0.366

Overturning Stability

Rotation Axis & Load Combination...	Moment Sum About Negative Edge (k-ft)				Moment Sum About Positive Edge (k-ft)			
	Overturning	Resisting	Ratio	Status	Overturning	Resisting	Ratio	Status
X-X, D Only	None	72.99	Infinity	OK	None	72.99	Infinity	OK
X-X, +D+Lr	None	87.77	Infinity	OK	None	87.77	Infinity	OK
X-X, +D+S	None	94.83	Infinity	OK	None	94.83	Infinity	OK
X-X, +D+0.750Lr	None	84.07	Infinity	OK	None	84.07	Infinity	OK
X-X, +D+0.750S	None	89.37	Infinity	OK	None	89.37	Infinity	OK
X-X, +D+0.60W	None	72.99	Infinity	OK	None	72.99	Infinity	OK
X-X, +D+0.750Lr+0.450W	None	84.07	Infinity	OK	None	84.07	Infinity	OK
X-X, +D+0.750S+0.450W	None	89.37	Infinity	OK	None	89.37	Infinity	OK
X-X, +0.60D+0.60W	None	43.79	Infinity	OK	None	43.79	Infinity	OK
X-X, +0.60D	None	43.79	Infinity	OK	None	43.79	Infinity	OK

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F1 - Pad Footing at Columns C3 & C4

Overturning Stability

Rotation Axis & Load Combination...	Moment Sum About Negative Edge (k-ft)				Moment Sum About Positive Edge (k-ft)			
	Overturning	Resisting	Ratio	Status	Overturning	Resisting	Ratio	Status
Z-Z, D Only	None	74.05	Infinity	OK	1.06	72.99	68.725	OK
Z-Z, +D+Lr	None	89.50	Infinity	OK	1.73	87.77	50.705	OK
Z-Z, +D+S	None	96.75	Infinity	OK	1.92	94.83	49.389	OK
Z-Z, +D+0.750Lr	None	85.64	Infinity	OK	1.56	84.07	53.765	OK
Z-Z, +D+0.750S	None	91.07	Infinity	OK	1.71	89.37	52.399	OK
Z-Z, +D+0.60W	None	74.75	Infinity	OK	1.77	72.99	41.291	OK
Z-Z, +D+0.750Lr+0.450W	None	86.17	Infinity	OK	2.09	84.07	40.171	OK
Z-Z, +D+0.750S+0.450W	None	91.60	Infinity	OK	2.23	89.37	39.990	OK
Z-Z, +0.60D+0.60W	None	45.13	Infinity	OK	1.34	43.79	32.612	OK
Z-Z, +0.60D	None	44.43	Infinity	OK	0.64	43.79	68.725	OK

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
X-X, D Only	0.3540 k	10.424 k	29.445	OK
X-X, +D+Lr	0.5770 k	11.902 k	20.628	OK
X-X, +D+S	0.640 k	12.608 k	19.699	OK
X-X, +D+0.750Lr	0.5213 k	11.532 k	22.125	OK
X-X, +D+0.750S	0.5685 k	12.062 k	21.217	OK
X-X, +D+0.60W	0.5892 k	10.424 k	17.691	OK
X-X, +D+0.750Lr+0.450W	0.6977 k	11.532 k	16.530	OK
X-X, +D+0.750S+0.450W	0.7449 k	12.062 k	16.192	OK
X-X, +0.60D+0.60W	0.4476 k	7.504 k	16.765	OK
X-X, +0.60D	0.2124 k	7.504 k	35.330	OK
Z-Z, D Only	0.0 k	10.424 k	999.0	OK
Z-Z, +D+Lr	0.0 k	11.902 k	999.0	OK
Z-Z, +D+S	0.0 k	12.608 k	999.0	OK
Z-Z, +D+0.750Lr	0.0 k	11.532 k	999.0	OK
Z-Z, +D+0.750S+0.450W	0.0 k	12.062 k	999.0	OK
Z-Z, +0.60D+0.60W	0.0 k	7.504 k	999.0	OK
Z-Z, +0.60D	0.0 k	7.504 k	999.0	OK
Z-Z, +D+0.750S	0.0 k	12.062 k	999.0	OK
Z-Z, +D+0.60W	0.0 k	10.424 k	999.0	OK
Z-Z, +D+0.750Lr+0.450W	0.0 k	11.532 k	999.0	OK

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	1.889	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.40D	1.889	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.50Lr	1.818	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.50Lr	1.818	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.50S	1.912	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.50S	1.912	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+1.60Lr	2.255	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+1.60Lr	2.255	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+1.60Lr+0.50W	2.255	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+1.60Lr+0.50W	2.255	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+1.60S	2.558	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+1.60S	2.558	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+1.60S+0.50W	2.558	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+1.60S+0.50W	2.558	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.50Lr+W	1.818	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.50Lr+W	1.818	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.50S+W	1.912	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.50S+W	1.912	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +0.90D+W	1.214	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +0.90D+W	1.214	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.20S	1.736	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +1.20D+0.20S	1.736	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +0.90D	1.214	+Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
X-X, +0.90D	1.214	-Z	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.40D	1.798	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.40D	1.979	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+0.50Lr	1.720	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+0.50Lr	1.916	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F1 - Pad Footing at Columns C3 & C4

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D+0.50S	1.809	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+0.50S	2.016	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+1.60Lr	2.112	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+1.60Lr	2.398	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+1.60Lr+0.50W	2.076	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+1.60Lr+0.50W	2.434	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+1.60S	2.397	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+1.60S	2.720	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+1.60S+0.50W	2.361	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+1.60S+0.50W	2.755	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+0.50Lr+W	1.648	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+0.50Lr+W	1.987	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+0.50S+W	1.737	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+0.50S+W	2.088	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +0.90D+W	1.084	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +0.90D+W	1.344	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+0.20S	1.648	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +1.20D+0.20S	1.824	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +0.90D	1.156	-X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK
Z-Z, +0.90D	1.272	+X	Bottom	0.2592	ACI 7.6.1.1	0.310	12.131	OK

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	10.88 psi	12.01 psi	12.01 psi	82.16 psi	0.15	OK
+1.20D+0.50Lr	10.40 psi	11.63 psi	11.63 psi	82.16 psi	0.14	OK
+1.20D+0.50S	10.94 psi	12.24 psi	12.24 psi	82.16 psi	0.15	OK
+1.20D+1.60Lr	12.77 psi	14.56 psi	14.56 psi	82.16 psi	0.18	OK
+1.20D+1.60Lr+0.50W	12.55 psi	14.79 psi	14.79 psi	82.16 psi	0.18	OK
+1.20D+1.60S	14.50 psi	16.52 psi	16.52 psi	82.16 psi	0.20	OK
+1.20D+1.60S+0.50W	14.27 psi	16.74 psi	16.74 psi	82.16 psi	0.20	OK
+1.20D+0.50Lr+W	9.95 psi	12.08 psi	12.08 psi	82.16 psi	0.15	OK
+1.20D+0.50S+W	10.49 psi	12.69 psi	12.69 psi	82.16 psi	0.15	OK
+0.90D+W	6.55 psi	8.17 psi	8.17 psi	82.16 psi	0.10	OK
+1.20D+0.20S	9.97 psi	11.08 psi	11.08 psi	82.16 psi	0.13	OK
+0.90D	6.99 psi	7.72 psi	7.72 psi	82.16 psi	0.09	OK

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	11.45 psi	11.45 psi	12.01 psi	82.16 psi	0.15	OK
+1.20D+0.50Lr	11.02 psi	11.02 psi	11.63 psi	82.16 psi	0.14	OK
+1.20D+0.50S	11.59 psi	11.59 psi	12.24 psi	82.16 psi	0.15	OK
+1.20D+1.60Lr	13.67 psi	13.67 psi	14.56 psi	82.16 psi	0.18	OK
+1.20D+1.60Lr+0.50W	13.67 psi	13.67 psi	14.79 psi	82.16 psi	0.18	OK
+1.20D+1.60S	15.51 psi	15.51 psi	16.52 psi	82.16 psi	0.20	OK
+1.20D+1.60S+0.50W	15.51 psi	15.51 psi	16.74 psi	82.16 psi	0.20	OK
+1.20D+0.50Lr+W	11.02 psi	11.02 psi	12.08 psi	82.16 psi	0.15	OK
+1.20D+0.50S+W	11.59 psi	11.59 psi	12.69 psi	82.16 psi	0.15	OK
+0.90D+W	7.36 psi	7.36 psi	8.17 psi	82.16 psi	0.10	OK
+1.20D+0.20S	10.52 psi	10.52 psi	11.08 psi	82.16 psi	0.13	OK
+0.90D	7.36 psi	7.36 psi	7.72 psi	82.16 psi	0.09	OK

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	25.71 psi	164.32 psi	0.16	OK
+1.20D+0.50Lr	24.75 psi	164.32 psi	0.15	OK
+1.20D+0.50S	26.04 psi	164.32 psi	0.16	OK
+1.20D+1.60Lr	30.70 psi	164.32 psi	0.19	OK
+1.20D+1.60Lr+0.50W	30.70 psi	164.32 psi	0.19	OK
+1.20D+1.60S	34.83 psi	164.32 psi	0.21	OK
+1.20D+1.60S+0.50W	34.83 psi	164.32 psi	0.21	OK
+1.20D+0.50Lr+W	24.75 psi	164.32 psi	0.15	OK
+1.20D+0.50S+W	26.04 psi	164.32 psi	0.16	OK
+0.90D+W	16.53 psi	164.32 psi	0.10	OK
+1.20D+0.20S	23.64 psi	164.32 psi	0.14	OK
+0.90D	16.53 psi	164.32 psi	0.10	OK

All units k

General Footing

Project File: 251023.ec6

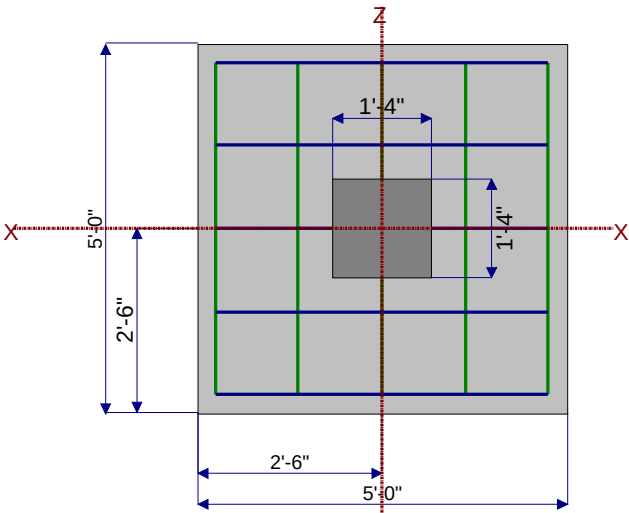
LIC# : KW-06018181, Build:20.26.07.20

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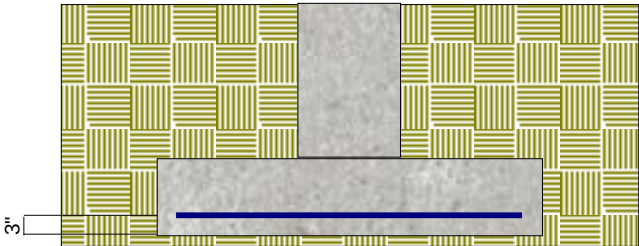
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DESCRIPTION: Footing F1 - Pad Footing at Columns C3 & C4

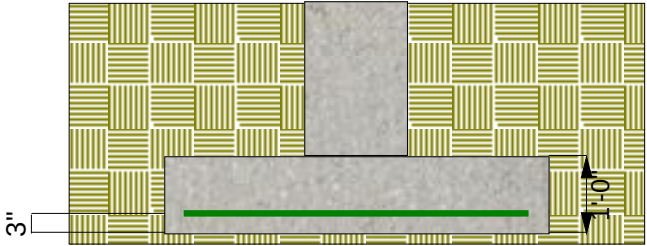
Plan and Section Views



Bottom Mat: 5#5 parallel to X-X, 5#5 parallel to Z-Z



X-X Section Looking to +Z



Z-Z Section Looking to +X

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F2 - Pad Footing at Columns C1

Code References

Governing Code : IBC 2018, CBC 2019

Referenced Design Standard(s) : ACI 318-14

Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	Yes
Use Pedestal wt for stability, mom & shear	:	Yes

Soil Design Values

Allowable Soil Bearing	=	2.0 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.250

Increases based on footing Depth

Footing base depth below soil surface	=	3.0 ft
Allow press. increase per foot of depth when footing base is below	=	ksf ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

Width parallel to X-X Axis	=	3.0 ft
Length parallel to Z-Z Axis	=	3.0 ft
Footing Thickness	=	12.0 in

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a

Pedestal dimensions...

px : parallel to X-X Axis	=	16.0 in
pz : parallel to Z-Z Axis	=	16.0 in
Height	=	24.0 in

Bottom Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

Number of Bars	=	4.0
Reinforcing Bar Size	=	# 5

Bars parallel to Z-Z Axis (resisting X Flexure)

Number of Bars	=	# 4.0
Reinforcing Bar Size	=	# 5

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in
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Applied Loads

		D	Lr	L	S	W	E	H
P : Vertical Load	=	7.979	0.686	0.000	2.672	0.000	0.000	0.000 k
OB : Overburden	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 ksf
M-xx	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k-ft
M-zz	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k-ft
V-x	=	0.220	0.072	0.000	0.094	0.276	0.000	0.000 k
V-z	=	0.000	0.000	0.000	0.000	0.000	0.000	0.000 k

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F2 - Pad Footing at Columns C1

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8850	Soil Bearing	1.770 ksf	2.0 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	11.480	Overturning - Z-Z	0.890 k-ft	10.250 k-ft	+0.60D+0.60W
PASS	12.041	Sliding - X-X	0.2976 k	3.583 k	+0.60D+0.60W
PASS	999.0	Sliding - Z-Z	0.0 k	4.722 k	D Only
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.04030	Z Flexure (+X) Bot Tens	0.6442 k-ft/ft	15.986 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.02890	Z Flexure (-X) Bot Tens	0.4619 k-ft/ft	15.986 k-ft/ft	+1.20D+1.60S
PASS	0.627	Min Steel X Flexure Bottom	0.259 in2/ft	0.413 in2/ft	n/a
PASS	0.03378	X Flexure (+Z) Bot Tens	0.5401 k-ft/ft	15.986 k-ft/ft	+1.20D+1.60S
PASS	0.03378	X Flexure (-Z) Bot Tens	0.5401 k-ft/ft	15.986 k-ft/ft	+1.20D+1.60S
PASS	0.627	Min Steel Z Flexure Bottom	0.259 in2/ft	0.413 in2/ft	n/a
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	Z Flexure (-X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel X Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel Z Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.01940	1-way Shear (+X)	1.594 psi	82.158 psi	+1.20D+1.60S+0.50W
PASS	0.01306	1-way Shear (-X)	1.073 psi	82.158 psi	+1.20D+1.60S
PASS	0.01578	1-way Shear (+Z)	1.296 psi	82.158 psi	+1.20D+1.60S
PASS	0.01578	1-way Shear (-Z)	1.296 psi	82.158 psi	+1.20D+1.60S
PASS	0.04828	2-way Punching	7.933 psi	164.317 psi	+1.20D+1.60S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc		Zecc		Actual Soil Bearing Stress @ Location				Actual / Allow Ratio
			(in)			Bottom, -Z	Top, +Z	Left, -X	Right, +X	
X-X, D Only	2.0	n/a	0.0	1.265	1.265	n/a	n/a	n/a	n/a	0.633
X-X, +D+Lr	2.0	n/a	0.0	1.342	1.342	n/a	n/a	n/a	n/a	0.671
X-X, +D+S	2.0	n/a	0.0	1.562	1.562	n/a	n/a	n/a	n/a	0.781
X-X, +D+0.750Lr	2.0	n/a	0.0	1.323	1.323	n/a	n/a	n/a	n/a	0.662
X-X, +D+0.750S	2.0	n/a	0.0	1.488	1.488	n/a	n/a	n/a	n/a	0.744
X-X, +D+0.60W	2.0	n/a	0.0	1.265	1.265	n/a	n/a	n/a	n/a	0.633
X-X, +D+0.750Lr+0.450W	2.0	n/a	0.0	1.323	1.323	n/a	n/a	n/a	n/a	0.662
X-X, +D+0.750S+0.450W	2.0	n/a	0.0	1.488	1.488	n/a	n/a	n/a	n/a	0.744
X-X, +0.60D+0.60W	2.0	n/a	0.0	0.7592	0.7592	n/a	n/a	n/a	n/a	0.380
X-X, +0.60D	2.0	n/a	0.0	0.7592	0.7592	n/a	n/a	n/a	n/a	0.380
Z-Z, D Only	2.0	0.6954	n/a	n/a	n/a	1.120	1.411			0.706
Z-Z, +D+Lr	2.0	0.8706	n/a	n/a	n/a	1.149	1.534			0.767
Z-Z, +D+S	2.0	0.8040	n/a	n/a	n/a	1.355	1.770			0.885
Z-Z, +D+0.750Lr	2.0	0.8287	n/a	n/a	n/a	1.142	1.503			0.752
Z-Z, +D+0.750S	2.0	0.7809	n/a	n/a	n/a	1.296	1.680			0.840
Z-Z, +D+0.60W	2.0	1.219	n/a	n/a	n/a	1.011	1.520			0.760
Z-Z, +D+0.750Lr+0.450W	2.0	1.204	n/a	n/a	n/a	1.060	1.585			0.793
Z-Z, +D+0.750S+0.450W	2.0	1.115	n/a	n/a	n/a	1.214	1.762			0.881
Z-Z, +0.60D+0.60W	2.0	1.568	n/a	n/a	n/a	0.5628	0.9556			0.478
Z-Z, +0.60D	2.0	0.6954	n/a	n/a	n/a	0.6721	0.8464			0.423

Overturning Stability

Rotation Axis & Load Combination...	Moment Sum About Negative Edge (k-ft)				Moment Sum About Positive Edge (k-ft)			
	Overturning	Resisting	Ratio	Status	Overturning	Resisting	Ratio	Status
X-X, D Only	None	17.08	Infinity	OK	None	17.08	Infinity	OK
X-X, +D+Lr	None	18.11	Infinity	OK	None	18.11	Infinity	OK
X-X, +D+S	None	21.09	Infinity	OK	None	21.09	Infinity	OK
X-X, +D+0.750Lr	None	17.85	Infinity	OK	None	17.85	Infinity	OK
X-X, +D+0.750S	None	20.09	Infinity	OK	None	20.09	Infinity	OK
X-X, +D+0.60W	None	17.08	Infinity	OK	None	17.08	Infinity	OK
X-X, +D+0.750Lr+0.450W	None	17.85	Infinity	OK	None	17.85	Infinity	OK
X-X, +D+0.750S+0.450W	None	20.09	Infinity	OK	None	20.09	Infinity	OK
X-X, +0.60D+0.60W	None	10.25	Infinity	OK	None	10.25	Infinity	OK
X-X, +0.60D	None	10.25	Infinity	OK	None	10.25	Infinity	OK

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General Footing

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F2 - Pad Footing at Columns C1

Overturning Stability

Rotation Axis & Load Combination...	Moment Sum About Negative Edge (k-ft)				Moment Sum About Positive Edge (k-ft)			
	Overturning	Resisting	Ratio	Status	Overturning	Resisting	Ratio	Status
Z-Z, D Only	None	17.74	Infinity	OK	0.66	17.08	25.883	OK
Z-Z, +D+Lr	None	18.99	Infinity	OK	0.88	18.11	20.675	OK
Z-Z, +D+S	None	22.03	Infinity	OK	0.94	21.09	22.389	OK
Z-Z, +D+0.750Lr	None	18.68	Infinity	OK	0.82	17.85	21.721	OK
Z-Z, +D+0.750S	None	20.96	Infinity	OK	0.87	20.09	23.051	OK
Z-Z, +D+0.60W	None	18.24	Infinity	OK	1.16	17.08	14.767	OK
Z-Z, +D+0.750Lr+0.450W	None	19.05	Infinity	OK	1.19	17.85	14.946	OK
Z-Z, +D+0.750S+0.450W	None	21.33	Infinity	OK	1.24	20.09	16.147	OK
Z-Z, +0.60D+0.60W	None	11.14	Infinity	OK	0.89	10.25	11.480	OK
Z-Z, +0.60D	None	10.65	Infinity	OK	0.40	10.25	25.883	OK

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
X-X, D Only	0.220 k	4.722 k	21.464	OK
X-X, +D+Lr	0.2920 k	4.894 k	16.759	OK
X-X, +D+S	0.3140 k	5.390 k	17.166	OK
X-X, +D+0.750Lr	0.2740 k	4.851 k	17.703	OK
X-X, +D+0.750S	0.2905 k	5.223 k	17.980	OK
X-X, +D+0.60W	0.3856 k	4.722 k	12.246	OK
X-X, +D+0.750Lr+0.450W	0.3982 k	4.851 k	12.182	OK
X-X, +D+0.750S+0.450W	0.4147 k	5.223 k	12.595	OK
X-X, +0.60D+0.60W	0.2976 k	3.583 k	12.041	OK
X-X, +0.60D	0.1320 k	3.583 k	27.146	OK
Z-Z, D Only	0.0 k	4.722 k	999.0	OK
Z-Z, +D+Lr	0.0 k	4.894 k	999.0	OK
Z-Z, +D+S	0.0 k	5.390 k	999.0	OK
Z-Z, +D+0.750Lr	0.0 k	4.851 k	999.0	OK
Z-Z, +D+0.750S+0.450W	0.0 k	5.223 k	999.0	OK
Z-Z, +0.60D+0.60W	0.0 k	3.583 k	999.0	OK
Z-Z, +0.60D	0.0 k	3.583 k	999.0	OK
Z-Z, +D+0.750S	0.0 k	5.223 k	999.0	OK
Z-Z, +D+0.60W	0.0 k	4.722 k	999.0	OK
Z-Z, +D+0.750Lr+0.450W	0.0 k	4.851 k	999.0	OK

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.4377	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.40D	0.4377	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.50Lr	0.3884	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.50Lr	0.3884	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.50S	0.4267	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.50S	0.4267	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+1.60Lr	0.4175	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+1.60Lr	0.4175	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+1.60Lr+0.50W	0.4175	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+1.60Lr+0.50W	0.4175	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+1.60S	0.5401	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+1.60S	0.5401	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+1.60S+0.50W	0.5401	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+1.60S+0.50W	0.5401	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.50Lr+W	0.3884	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.50Lr+W	0.3884	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.50S+W	0.4267	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.50S+W	0.4267	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +0.90D+W	0.2814	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +0.90D+W	0.2814	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.20S	0.3958	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +1.20D+0.20S	0.3958	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +0.90D	0.2814	+Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
X-X, +0.90D	0.2814	-Z	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.40D	0.3796	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.40D	0.4957	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+0.50Lr	0.3318	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+0.50Lr	0.4449	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK

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General Footing

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DESCRIPTION: Footing F2 - Pad Footing at Columns C1

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
Z-Z, +1.20D+0.50S	0.3680	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+0.50S	0.4853	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+1.60Lr	0.3460	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+1.60Lr	0.4890	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.3199	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.5150	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+1.60S	0.4619	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+1.60S	0.6182	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+1.60S+0.50W	0.4359	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+1.60S+0.50W	0.6442	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+0.50Lr+W	0.2797	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+0.50Lr+W	0.4970	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+0.50S+W	0.3160	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+0.50S+W	0.5374	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +0.90D+W	0.1920	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +0.90D+W	0.3707	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+0.20S	0.3424	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +1.20D+0.20S	0.4491	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +0.90D	0.2440	-X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK
Z-Z, +0.90D	0.3187	+X	Bottom	0.2592	ACI 7.6.1.1	0.4133	15.986	OK

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.88 psi	1.22 psi	1.22 psi	82.16 psi	0.01	OK
+1.20D+0.50Lr	0.77 psi	1.09 psi	1.09 psi	82.16 psi	0.01	OK
+1.20D+0.50S	0.86 psi	1.19 psi	1.19 psi	82.16 psi	0.01	OK
+1.20D+1.60Lr	0.80 psi	1.21 psi	1.21 psi	82.16 psi	0.01	OK
+1.20D+1.60Lr+0.50W	0.72 psi	1.28 psi	1.28 psi	82.16 psi	0.02	OK
+1.20D+1.60S	1.07 psi	1.52 psi	1.52 psi	82.16 psi	0.02	OK
+1.20D+1.60S+0.50W	1.00 psi	1.59 psi	1.59 psi	82.16 psi	0.02	OK
+1.20D+0.50Lr+W	0.62 psi	1.24 psi	1.24 psi	82.16 psi	0.02	OK
+1.20D+0.50S+W	0.71 psi	1.34 psi	1.34 psi	82.16 psi	0.02	OK
+0.90D+W	0.42 psi	0.93 psi	0.93 psi	82.16 psi	0.01	OK
+1.20D+0.20S	0.80 psi	1.10 psi	1.10 psi	82.16 psi	0.01	OK
+0.90D	0.57 psi	0.78 psi	0.78 psi	82.16 psi	0.01	OK

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.05 psi	1.05 psi	1.22 psi	82.16 psi	0.01	OK
+1.20D+0.50Lr	0.93 psi	0.93 psi	1.09 psi	82.16 psi	0.01	OK
+1.20D+0.50S	1.02 psi	1.02 psi	1.19 psi	82.16 psi	0.01	OK
+1.20D+1.60Lr	1.00 psi	1.00 psi	1.21 psi	82.16 psi	0.01	OK
+1.20D+1.60Lr+0.50W	1.00 psi	1.00 psi	1.28 psi	82.16 psi	0.02	OK
+1.20D+1.60S	1.30 psi	1.30 psi	1.52 psi	82.16 psi	0.02	OK
+1.20D+1.60S+0.50W	1.30 psi	1.30 psi	1.59 psi	82.16 psi	0.02	OK
+1.20D+0.50Lr+W	0.93 psi	0.93 psi	1.24 psi	82.16 psi	0.02	OK
+1.20D+0.50S+W	1.02 psi	1.02 psi	1.34 psi	82.16 psi	0.02	OK
+0.90D+W	0.68 psi	0.68 psi	0.93 psi	82.16 psi	0.01	OK
+1.20D+0.20S	0.95 psi	0.95 psi	1.10 psi	82.16 psi	0.01	OK
+0.90D	0.68 psi	0.68 psi	0.78 psi	82.16 psi	0.01	OK

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	6.43 psi	164.32 psi	0.04	OK
+1.20D+0.50Lr	5.71 psi	164.32 psi	0.03	OK
+1.20D+0.50S	6.27 psi	164.32 psi	0.04	OK
+1.20D+1.60Lr	6.13 psi	164.32 psi	0.04	OK
+1.20D+1.60Lr+0.50W	6.13 psi	164.32 psi	0.04	OK
+1.20D+1.60S	7.93 psi	164.32 psi	0.05	OK
+1.20D+1.60S+0.50W	7.93 psi	164.32 psi	0.05	OK
+1.20D+0.50Lr+W	5.71 psi	164.32 psi	0.03	OK
+1.20D+0.50S+W	6.27 psi	164.32 psi	0.04	OK
+0.90D+W	4.13 psi	164.32 psi	0.03	OK
+1.20D+0.20S	5.81 psi	164.32 psi	0.04	OK
+0.90D	4.13 psi	164.32 psi	0.03	OK

All units k

General Footing

Project File: 251023.ec6

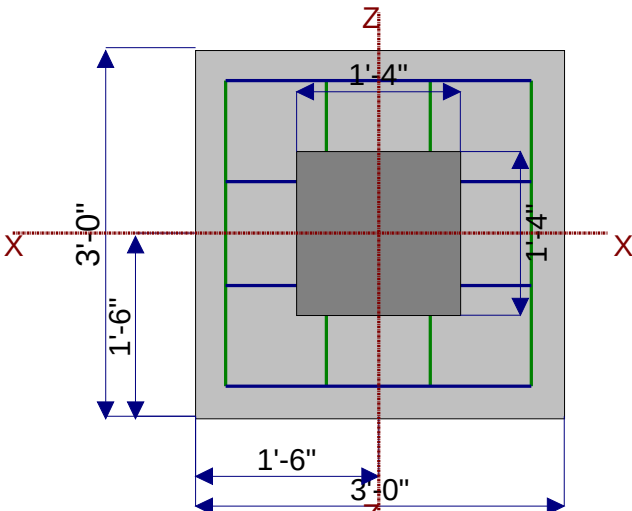
LIC# : KW-06018181, Build:20.26.07.20

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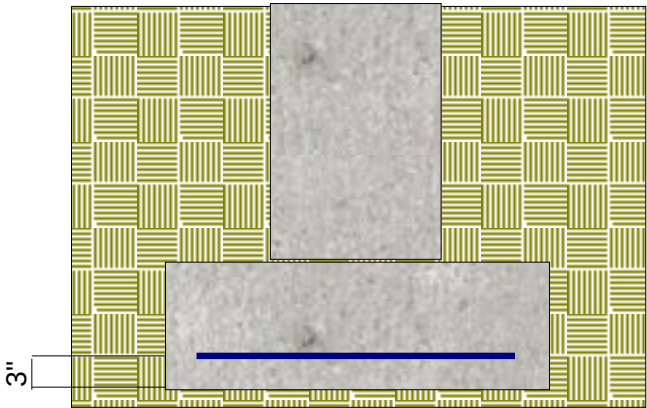
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DESCRIPTION: Footing F2 - Pad Footing at Columns C1

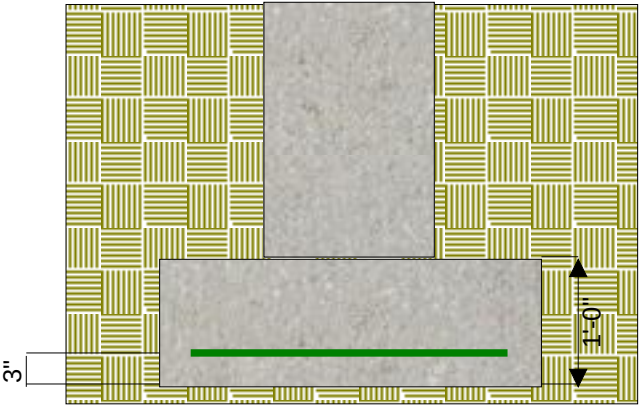
Plan and Section Views



Bottom Mat: 4#5 parallel to X-X, 4#5 parallel to Z-Z



X-X Section Looking to +Z



Z-Z Section Looking to +X

Footing F3 - Footing for Braced Frame @ Line B

Check Footing at Braced Frame at Line B

$$w_{ftg} := 5\text{ft}$$

Width of Footing

$$L_{ftg} := 18\cdot\text{ft}$$

Length of footing

$$h_{ftg} := 36\cdot\text{in}$$

Depth of Footing

$$L_{bf} := 5.50\text{ft}$$

distance between column

$$L_{ends} := \frac{L_{ftg} - L_{bf}}{2} \quad L_{ends} = 6.25\text{ft}$$

distance from column to footing end

Load at Columns from RISA3D reactions

$$P_{DL.1} := 10.418\cdot\text{k} \quad V_{DL.1} := 0.713\cdot\text{k}$$

$$P_{RLL.1} := 2.830\cdot\text{k} \quad V_{RLL.1} := 0.187\cdot\text{k}$$

From RISA output (Unfactored)
at Column 1 (Node N1)

$$P_{SL.1} := 4.297\cdot\text{k} \quad V_{SL.1} := 0.284\cdot\text{k}$$

$$P_{WL.1} := 71.764\text{k} \quad V_{WL.1} := 12.923\text{k}$$

$$P_{EQ.1} := \text{"N/A"} \quad V_{EQ.1} := \text{"N/A"}$$

$$P_{DL.2} := 0.469\cdot\text{k} \quad V_{DL.2} := -0.713\cdot\text{k}$$

$$P_{RLL.2} := 0\cdot\text{k} \quad V_{RLL.2} := -0.187\cdot\text{k}$$

From RISA output (Unfactored)
at Column 2 (Node N3)

$$P_{SL.2} := 0\cdot\text{k} \quad V_{SL.2} := -0.284\cdot\text{k}$$

$$P_{WL.2} := -71.764\text{k} \quad V_{WL.2} := 13.241\text{k}$$

$$P_{EQ.2} := \text{"N/A"} \quad V_{EQ.2} := \text{"N/A"}$$

$$A_{req.1} := 0.0018 \cdot (w_{ftg} \cdot h_{ftg})$$

$$A_{req.1} = 3.888\cdot\text{in}^2$$

$$A_7 := \left(\frac{0.875\cdot\text{in}}{2} \right)^2 \cdot \pi \quad A_7 = 0.601\cdot\text{in}^2$$

$$N_{bar.req} := \frac{A_{req.1}}{A_7} = 6.466$$

use a min of (7) #7 rebar top and bottom.

Use: 6'-0" x 18'-0" x 3'-0" Footing
W/ (7) #7 LW T&B cont. &
#6 @ 12" SW T&B

Project Title:
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 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

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DESCRIPTION: Footing F3 - Pad Footing at Gridline B

Code References

Governing Code : IBC 2018, CBC 2019
 Referenced Design Standard(s) : ACI 318-14
 Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Information

Material Properties

f'_c : Concrete 28 day strength 3.0 ksi
 f_y : Rebar Yield 60.0 ksi
 E_c : Concrete Elastic Modulus 3,122.0 ksi
 Concrete Density 145.0 pcf
 ϕ : Phi Values Flexure : 0.90
 Shear : 0.750

Analysis/Design Settings

Calculate footing weight as dead load ? Yes
 Calculate Pedestal weight as dead load ? Yes
 Min Steel Ratio per ACI 7.6.1.1 0.0018
 (ratio=Steel Area/Concrete Area)
 Min. Overturning Safety Factor 1.0: 1
 Min. Sliding Safety Factor 1.0: 1

Soil Information

Allowable Soil Bearing 2.0 ksf
 Increase Bearing By Footing Weight No
 Soil Passive Sliding Resistance 250.0 pcf
 (Uses entry for "Footing base depth below soil surface" for force)
 Coefficient of Soil/Concrete Friction 0.250

Soil Bearing Increase

Footing base depth below soil surface 3.0 ft
 Increases based on footing Depth . . .
 Allowable pressure increase per foot ksf
 when base of footing is below ft
 Increases based on footing Width . . .
 Allowable pressure increase per foot ksf
 when maximum length or width is greater tha ft
 Maximum Allowed Bearing Pressure 10.0 ksf
 (A value of zero implies no limit)
 Adjusted Allowable Soil Bearing 2.0 ksf
 (Allowable Soil Bearing adjusted for footing weight and depth & width increases as specified by user.)

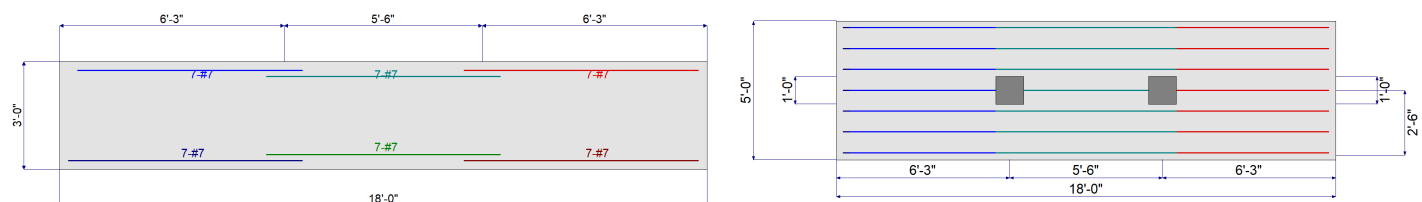
Dimensions & Reinforcing

Distance Left of Column #1	=	6.250 ft	Pedestal dimensions...									As
Between Columns	=	5.50 ft			Col #1	Col #2						Provided
Distance Right of Column #2	=	6.250 ft	Sq. Dim.	=	12.0	12.0 in						
Total Footing Length	=	18.0 ft	Height	=								
Footing Width	=	5.0 ft										
Footing Thickness	=	36.0 in										
Rebar Center to Concrete Edge @ Top	=	3.0 in										
Rebar Center to Concrete Edge @ Bottom	=	3.0 in										

Bars left of Col #1	Count	Size #	As
Bottom Bars	7.0	7	4.20
Top Bars	7.0	7	4.20
Bars Btwn Cols			
Bottom Bars	7.0	7	4.20
Top Bars	7.0	7	4.20
Bars Right of Col #2			
Bottom Bars	7.0	7	4.20
Top Bars	7.0	7	4.20

Applied Loads

Applied @ Left Column	D	Lr	L	S	W	E	H
Axial Load Downward	=	10.418	2.830		4.297	71.764	k
Moment (+CW)	=						k-ft
Shear (+X)	=	0.7130	0.1870		0.2840	12.923	k
Applied @ Right Column							
Axial Load Downward	=	0.4690			-71.764		k
Moment (+CW)	=						k-ft
Shear (+X)	=	-0.7310	-0.1870		-0.2840	13.241	k
Overburden	=						



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Combined Footing

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LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F3 - Pad Footing at Gridline B

DESIGN SUMMARY

Design N.G.

Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS 1.459	Overturning	508.13 k-ft	741.32 k-ft	+D+0.60W
PASS 1.156	Sliding	15.680 k	18.134 k	+D+0.60W
PASS 2.162	Uplift	43.058 k	93.095 k	+D+0.60W

Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS 0.7146	Soil Bearing	1.429 ksf	2.0 ksf	+D+0.60W
PASS 0.3303	1-way Shear - Col #1	27.134 psi	82.158 psi	+0.90D+W
PASS 0.3303	1-way Shear - Col #2	27.134 psi	82.158 psi	+0.90D+W
PASS 0.08877	2-way Punching - Col #1	14.587 psi	164.317 psi	+0.90D+W
PASS 0.09606	2-way Punching - Col #2	15.784 psi	164.317 psi	+1.20D+0.50S+W
PASS No Tension	Flexure - Left of Col #1 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS 0.2647	Flexure - Left of Col #1 - Bottom	160.980 k-ft	608.14 k-ft	+0.90D+W
PASS 0.09580	Flexure - Between Cols - Top	-58.262 k-ft	608.14 k-ft	+1.20D+0.50Lr+W
PASS 0.3208	Flexure - Between Cols - Bottom	195.087 k-ft	608.14 k-ft	+0.90D+W
FAIL 0.07009	Flexure - Right of Col #2 - Top	-42.624 k-ft	608.14 k-ft	+1.20D+0.50Lr+W
PASS 0.00439	Flexure - Right of Col #2 - Bottom	5.740 k-ft	608.14 k-ft	+1.20D+1.60S

Soil Bearing

CAPACITY <10% OK

Load Combination...	Total Bearing	from Ftg CL	Actual Soil Bearing Stress @ Left Edge	Actual Soil Bearing Stress @ Right Edge	Allowable	Actual / Allow Ratio
D Only	50.04 k	-0.548 ft	0.66 ksf	0.45 ksf	2.00 ksf	0.329
+D+Lr	52.87 k	-0.666 ft	0.72 ksf	0.46 ksf	2.00 ksf	0.359
+D+S	54.33 k	-0.722 ft	0.75 ksf	0.46 ksf	2.00 ksf	0.374
+D+0.750Lr	52.16 k	-0.637 ft	0.70 ksf	0.46 ksf	2.00 ksf	0.351
+D+0.750S	53.26 k	-0.681 ft	0.73 ksf	0.46 ksf	2.00 ksf	0.363
+D+0.60W	50.04 k	-4.340 ft	1.43 ksf	0.00 ksf	2.00 ksf	0.715
+D+0.750Lr+0.450W	52.16 k	-3.366 ft	1.23 ksf	0.00 ksf	2.00 ksf	0.616
+D+0.750S+0.450W	53.26 k	-3.353 ft	1.26 ksf	0.00 ksf	2.00 ksf	0.628
+0.60D+0.750S+0.450W	33.24 k	-5.042 ft	1.12 ksf	0.00 ksf	2.00 ksf	0.559
+0.60D+0.750S	33.24 k	-0.761 ft	0.46 ksf	0.28 ksf	2.00 ksf	0.231
+0.60D	30.02 k	-0.548 ft	0.39 ksf	0.27 ksf	2.00 ksf	0.197

Overturning Stability

Load Combination...	Moments about Left Edge k-ft			Moments about Right Edge k-ft		
	Overturning	Resisting	Ratio	Overturning	Resisting	Ratio
D Only	2.19	425.11	193.850	2.14	479.89	224.351
+D+Lr	2.75	443.36	160.988	2.70	513.70	190.259
+D+S	3.05	452.82	148.710	2.99	531.23	177.609
+D+0.750Lr	2.61	438.80	167.881	2.56	505.25	197.381
+D+0.750S	2.83	445.89	157.448	2.78	518.39	186.606
+D+0.60W	508.13	741.32	1.459	318.35	985.82	3.097
+D+0.750Lr+0.450W	382.07	675.96	1.769	239.72	884.70	3.691
+D+0.750S+0.450W	382.28	683.05	1.787	239.94	897.84	3.742
+0.60D+0.750S	1.95	275.85	141.113	1.92	326.44	169.807
+0.60D	1.32	255.07	193.850	1.28	287.93	224.351

Sliding Stability

Load Combination...	Sliding Force	Resisting Force	Sliding Safety Ratio
D Only	-0.02 k	18.13 k	1
+D+Lr	-0.02 k	18.84 k	1
+D+S	-0.02 k	19.21 k	1
+D+0.750Lr	-0.02 k	18.66 k	1
+D+0.750S	-0.02 k	18.94 k	1
+D+0.60W	15.68 k	18.13 k	1.156
+D+0.750Lr+0.450W	11.76 k	18.66 k	1.588
+D+0.750S+0.450W	11.76 k	18.94 k	1.611
+0.60D+0.750S	-0.01 k	13.94 k	1
+0.60D	-0.01 k	13.13 k	1

Project Title:
 Engineer:
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 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F3 - Pad Footing at Gridline B

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.60D	0.000	0.000	0	0.000	0	0.000	0.000	0.000
+0.90D+W	0.019	0.045	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.076	0.090	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.171	0.135	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.304	0.180	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.472	0.225	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+0.90D+W	0.678	0.270	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+0.90D+W	0.919	0.315	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+0.90D+W	1.195	0.360	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+0.90D+W	1.507	0.405	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+0.90D+W	1.853	0.450	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+0.90D+W	2.233	0.495	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+0.90D+W	2.646	0.540	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+0.90D+W	3.093	0.585	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+0.90D+W	3.573	0.630	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+0.90D+W	4.085	0.675	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+0.90D+W	4.628	0.720	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.008
+0.90D+W	5.204	0.765	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.009
+0.90D+W	5.810	0.810	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.010
+0.90D+W	6.446	0.855	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.011
+0.90D+W	7.113	0.900	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.012
+0.90D+W	7.809	0.945	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.013
+0.90D+W	8.535	0.990	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.014
+0.90D+W	9.289	1.035	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.015
+0.90D+W	10.072	1.080	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.017
+0.90D+W	10.882	1.125	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.018
+0.90D+W	11.720	1.170	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.019
+0.90D+W	12.585	1.215	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.021
+0.90D+W	13.476	1.260	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.022
+0.90D+W	14.393	1.305	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+0.90D+W	15.336	1.350	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.025
+0.90D+W	16.304	1.395	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.027
+0.90D+W	17.297	1.440	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.028
+0.90D+W	18.314	1.485	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.030
+0.90D+W	19.355	1.530	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+0.90D+W	20.419	1.575	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.034
+0.90D+W	21.506	1.620	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.035
+0.90D+W	22.616	1.665	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.037
+0.90D+W	23.748	1.710	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.039
+0.90D+W	24.901	1.755	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.041
+0.90D+W	26.075	1.800	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.043
+0.90D+W	27.271	1.845	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.045
+0.90D+W	28.486	1.890	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.047
+0.90D+W	29.721	1.935	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.049
+0.90D+W	30.976	1.980	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.051
+0.90D+W	32.250	2.025	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.053
+0.90D+W	33.542	2.070	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.055
+0.90D+W	34.852	2.115	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.057
+0.90D+W	36.180	2.160	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.059
+0.90D+W	37.524	2.205	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.062
+0.90D+W	38.886	2.250	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.064
+0.90D+W	40.264	2.295	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.066
+0.90D+W	41.657	2.340	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.069
+0.90D+W	43.066	2.385	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.071
+0.90D+W	44.490	2.430	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.073
+0.90D+W	45.929	2.475	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.076
+0.90D+W	47.381	2.520	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.078
+0.90D+W	48.847	2.565	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.080
+0.90D+W	50.326	2.610	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.083
+0.90D+W	51.818	2.655	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.085
+0.90D+W	53.322	2.700	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.088
+0.90D+W	54.838	2.745	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.090
+0.90D+W	56.365	2.790	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.093

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BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F3 - Pad Footing at Gridline B

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	57.903	2.835	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.095
+0.90D+W	59.451	2.880	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.098
+0.90D+W	61.010	2.925	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.100
+0.90D+W	62.578	2.970	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.103
+0.90D+W	64.155	3.015	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.105
+0.90D+W	65.741	3.060	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.108
+0.90D+W	67.335	3.105	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.111
+0.90D+W	68.937	3.150	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.113
+0.90D+W	70.547	3.195	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.116
+0.90D+W	72.163	3.240	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.119
+0.90D+W	73.785	3.285	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.121
+0.90D+W	75.414	3.330	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.124
+0.90D+W	77.048	3.375	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.127
+0.90D+W	78.687	3.420	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.129
+0.90D+W	80.331	3.465	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.132
+0.90D+W	81.980	3.510	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.135
+0.90D+W	83.632	3.555	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.138
+0.90D+W	85.287	3.600	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.140
+0.90D+W	86.945	3.645	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.143
+0.90D+W	88.606	3.690	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.146
+0.90D+W	90.268	3.735	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.148
+0.90D+W	91.932	3.780	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.151
+0.90D+W	93.597	3.825	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.154
+0.90D+W	95.263	3.870	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.157
+0.90D+W	96.929	3.915	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.159
+0.90D+W	98.595	3.960	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.162
+0.90D+W	100.260	4.005	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.165
+0.90D+W	101.925	4.050	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.168
+0.90D+W	103.587	4.095	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.170
+0.90D+W	105.247	4.140	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.173
+0.90D+W	106.905	4.185	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.176
+0.90D+W	108.560	4.230	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.179
+0.90D+W	110.212	4.275	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.181
+0.90D+W	111.860	4.320	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.184
+0.90D+W	113.504	4.365	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.187
+0.90D+W	115.144	4.410	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.189
+0.90D+W	116.780	4.455	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.192
+0.90D+W	118.412	4.500	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.195
+0.90D+W	120.040	4.545	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.197
+0.90D+W	121.665	4.590	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.200
+0.90D+W	123.285	4.635	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.203
+0.90D+W	124.901	4.680	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.205
+0.90D+W	126.513	4.725	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.208
+0.90D+W	128.122	4.770	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.211
+0.90D+W	129.726	4.815	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.213
+0.90D+W	131.326	4.860	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.216
+0.90D+W	132.923	4.905	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.219
+0.90D+W	134.515	4.950	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.221
+0.90D+W	136.104	4.995	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.224
+0.90D+W	137.688	5.040	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.226
+0.90D+W	139.269	5.085	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.229
+0.90D+W	140.845	5.130	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.232
+0.90D+W	142.418	5.175	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.234
+0.90D+W	143.987	5.220	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.237
+0.90D+W	145.551	5.265	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.239
+0.90D+W	147.112	5.310	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.242
+0.90D+W	148.669	5.355	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.244
+0.90D+W	150.222	5.400	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.247
+0.90D+W	151.770	5.445	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.250
+0.90D+W	153.315	5.490	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.252
+0.90D+W	154.856	5.535	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.255
+0.90D+W	156.393	5.580	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.257
+0.90D+W	157.926	5.625	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.260

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F3 - Pad Footing at Gridline B

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	159.455	5.670	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.262
+0.90D+W	160.980	5.715	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.265
+0.90D+W	162.497	5.760	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.267
+0.90D+W	163.896	5.805	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.270
+0.90D+W	165.126	5.850	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.272
+0.90D+W	166.188	5.895	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.273
+0.90D+W	167.082	5.940	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.275
+0.90D+W	167.807	5.985	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.276
+0.90D+W	168.364	6.030	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.277
+0.90D+W	168.753	6.075	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.277
+0.90D+W	168.973	6.120	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.278
+0.90D+W	169.026	6.165	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.278
+0.90D+W	168.910	6.210	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.278
+0.90D+W	209.320	6.255	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.344
+0.90D+W	208.867	6.300	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.343
+0.90D+W	208.246	6.345	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.342
+0.90D+W	207.457	6.390	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.341
+0.90D+W	206.500	6.435	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.340
+0.90D+W	205.374	6.480	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.338
+0.90D+W	204.080	6.525	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.336
+0.90D+W	202.618	6.570	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.333
+0.90D+W	200.988	6.615	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.330
+0.90D+W	199.189	6.660	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.328
+0.90D+W	197.222	6.705	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.324
+0.90D+W	195.087	6.750	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.321
+0.90D+W	192.865	6.795	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.317
+0.90D+W	190.640	6.840	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.313
+0.90D+W	188.411	6.885	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.310
+0.90D+W	186.177	6.930	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.306
+0.90D+W	183.940	6.975	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.302
+0.90D+W	181.699	7.020	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.299
+0.90D+W	179.454	7.065	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.295
+0.90D+W	177.205	7.110	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.291
+0.90D+W	174.952	7.155	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.288
+0.90D+W	172.694	7.200	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.284
+0.90D+W	170.433	7.245	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.280
+0.90D+W	168.168	7.290	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.277
+0.90D+W	165.900	7.335	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.273
+0.90D+W	163.627	7.380	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.269
+0.90D+W	161.350	7.425	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.265
+0.90D+W	159.069	7.470	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.262
+0.90D+W	156.784	7.515	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.258
+0.90D+W	154.495	7.560	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.254
+0.90D+W	152.203	7.605	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.250
+0.90D+W	149.906	7.650	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.247
+0.90D+W	147.605	7.695	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.243
+0.90D+W	145.301	7.740	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.239
+0.90D+W	142.992	7.785	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.235
+0.90D+W	140.679	7.830	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.231
+0.90D+W	138.363	7.875	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.228
+0.90D+W	136.042	7.920	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.224
+0.90D+W	133.718	7.965	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.220
+0.90D+W	131.390	8.010	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.216
+0.90D+W	129.057	8.055	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.212
+0.90D+W	126.721	8.100	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.208
+0.90D+W	124.381	8.145	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.205
+0.90D+W	122.036	8.190	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.201
+0.90D+W	119.688	8.235	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.197
+0.90D+W	117.336	8.280	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.193
+0.90D+W	114.980	8.325	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.189
+0.90D+W	112.620	8.370	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.185
+0.90D+W	110.256	8.415	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.181
+0.90D+W	107.888	8.460	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.177

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F3 - Pad Footing at Gridline B

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	105.516	8.505	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.174
+0.90D+W	103.140	8.550	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.170
+0.90D+W	100.760	8.595	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.166
+0.90D+W	98.376	8.640	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.162
+0.90D+W	95.988	8.685	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.158
+0.90D+W	93.596	8.730	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.154
+0.90D+W	91.200	8.775	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.150
+0.90D+W	88.800	8.820	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.146
+0.90D+W	86.397	8.865	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.142
+0.90D+W	83.989	8.910	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.138
+0.90D+W	81.577	8.955	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.134
+0.90D+W	79.162	9.000	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.130
+0.90D+W	76.742	9.045	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.126
+0.90D+W	74.319	9.090	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.122
+0.90D+W	71.891	9.135	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.118
+0.90D+W	69.460	9.180	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.114
+0.90D+W	67.024	9.225	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.110
+0.90D+W	64.585	9.270	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.106
+0.90D+W	62.142	9.315	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.102
+0.90D+W	59.694	9.360	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.098
+0.90D+W	57.243	9.405	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.094
+0.90D+W	54.788	9.450	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.090
+0.90D+W	52.328	9.495	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.086
+0.90D+W	49.865	9.540	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.082
+0.90D+W	47.398	9.585	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.078
+0.90D+W	44.927	9.630	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.074
+0.90D+W	42.452	9.675	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.070
+0.90D+W	39.973	9.720	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.066
+0.90D+W	37.490	9.765	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.062
+0.90D+W	35.003	9.810	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.058
+0.90D+W	32.512	9.855	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.053
+0.90D+W	30.017	9.900	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.049
+0.90D+W	27.518	9.945	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.045
+0.90D+W	25.015	9.990	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.041
+0.90D+W	22.509	10.035	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.037
+0.90D+W	19.998	10.080	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.033
+0.90D+W	17.483	10.125	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.029
+1.20D+1.60S	16.184	10.170	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.027
+1.20D+1.60S	16.011	10.215	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+1.20D+1.60S	15.840	10.260	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+1.20D+1.60Lr+0.50W	-16.012	10.305	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+1.20D+1.60Lr+0.50W	-17.192	10.350	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.028
+1.20D+1.60Lr+0.50W	-18.371	10.395	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.030
+1.20D+1.60Lr+0.50W	-19.550	10.440	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+1.20D+1.60Lr+0.50W	-20.728	10.485	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.034
+1.20D+0.50Lr+W	-22.766	10.530	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.037
+1.20D+0.50Lr+W	-25.096	10.575	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.041
+1.20D+0.50Lr+W	-27.430	10.620	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.045
+1.20D+0.50Lr+W	-29.770	10.665	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.049
+1.20D+0.50Lr+W	-32.116	10.710	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.053
+1.20D+0.50Lr+W	-34.466	10.755	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.057
+1.20D+0.50Lr+W	-36.822	10.800	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.061
+1.20D+0.50Lr+W	-39.183	10.845	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.064
+1.20D+0.50Lr+W	-41.549	10.890	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.068
+1.20D+0.50Lr+W	-43.921	10.935	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.072
+1.20D+0.50Lr+W	-46.298	10.980	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.076
+1.20D+0.50Lr+W	-48.680	11.025	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.080
+1.20D+0.50Lr+W	-51.067	11.070	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.084
+1.20D+0.50Lr+W	-53.460	11.115	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.088
+1.20D+0.50Lr+W	-55.858	11.160	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.092
+1.20D+0.50Lr+W	-58.261	11.205	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.096
+1.20D+0.50Lr+W	-60.670	11.250	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.100
+1.20D+0.50Lr+W	-63.012	11.295	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.104

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F3 - Pad Footing at Gridline B

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+0.50Lr+W	-65.215	11.340	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.107
+1.20D+0.50Lr+W	-67.279	11.385	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.111
+1.20D+0.50Lr+W	-69.204	11.430	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.114
+1.20D+0.50Lr+W	-70.990	11.475	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.117
+1.20D+0.50Lr+W	-72.638	11.520	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.119
+1.20D+0.50Lr+W	-74.146	11.565	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.122
+1.20D+0.50Lr+W	-75.515	11.610	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.124
+1.20D+0.50Lr+W	-76.746	11.655	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.126
+1.20D+0.50Lr+W	-77.838	11.700	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.128
+1.20D+0.50Lr+W	-78.791	11.745	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.130
+1.20D+0.50Lr+W	-42.794	11.790	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.070
+1.20D+0.50Lr+W	-43.469	11.835	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.071
+1.20D+0.50Lr+W	-44.005	11.880	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.072
+1.20D+0.50Lr+W	-44.402	11.925	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.073
+1.20D+0.50Lr+W	-44.661	11.970	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.073
+1.20D+0.50Lr+W	-44.780	12.015	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.074
+1.20D+0.50Lr+W	-44.761	12.060	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.074
+1.20D+0.50Lr+W	-44.602	12.105	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.073
+1.20D+0.50Lr+W	-44.305	12.150	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.073
+1.20D+0.50Lr+W	-43.869	12.195	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.072
+1.20D+0.50Lr+W	-43.294	12.240	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.071
+1.20D+0.50Lr+W	-42.624	12.285	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.070
+1.20D+0.50Lr+W	-41.955	12.330	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.069
+1.20D+0.50Lr+W	-41.292	12.375	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.068
+1.20D+0.50Lr+W	-40.634	12.420	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.067
+1.20D+0.50Lr+W	-39.981	12.465	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.066
+1.20D+0.50Lr+W	-39.333	12.510	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.065
+1.20D+0.50Lr+W	-38.691	12.555	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.064
+1.20D+0.50Lr+W	-38.054	12.600	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.063
+1.20D+0.50Lr+W	-37.423	12.645	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.062
+1.20D+0.50Lr+W	-36.797	12.690	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.061
+1.20D+0.50Lr+W	-36.175	12.735	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.059
+1.20D+0.50Lr+W	-35.560	12.780	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.058
+1.20D+0.50Lr+W	-34.949	12.825	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.057
+1.20D+0.50Lr+W	-34.344	12.870	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.056
+1.20D+0.50Lr+W	-33.744	12.915	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.055
+1.20D+0.50Lr+W	-33.150	12.960	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.055
+1.20D+0.50Lr+W	-32.560	13.005	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.054
+1.20D+0.50Lr+W	-31.976	13.050	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.053
+1.20D+0.50Lr+W	-31.398	13.095	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.052
+1.20D+0.50Lr+W	-30.824	13.140	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.051
+1.20D+0.50Lr+W	-30.256	13.185	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.050
+1.20D+0.50Lr+W	-29.693	13.230	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.049
+1.20D+0.50Lr+W	-29.135	13.275	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.048
+1.20D+0.50Lr+W	-28.583	13.320	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.047
+1.20D+0.50Lr+W	-28.036	13.365	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.046
+1.20D+0.50Lr+W	-27.494	13.410	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.045
+1.20D+0.50Lr+W	-26.958	13.455	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.044
+1.20D+0.50Lr+W	-26.427	13.500	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.043
+1.20D+0.50Lr+W	-25.901	13.545	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.043
+1.20D+0.50Lr+W	-25.380	13.590	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.042
+1.20D+0.50Lr+W	-24.865	13.635	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.041
+1.20D+0.50Lr+W	-24.355	13.680	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.040
+1.20D+0.50Lr+W	-23.850	13.725	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.039
+1.20D+0.50Lr+W	-23.351	13.770	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.038
+1.20D+0.50Lr+W	-22.857	13.815	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.038
+1.20D+0.50Lr+W	-22.368	13.860	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.037
+1.20D+0.50Lr+W	-21.884	13.905	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.036
+1.20D+0.50Lr+W	-21.406	13.950	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.035
+1.20D+0.50Lr+W	-20.933	13.995	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.034
+1.20D+0.50Lr+W	-20.465	14.040	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.034
+1.20D+0.50Lr+W	-20.002	14.085	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.033
+1.20D+0.50Lr+W	-19.545	14.130	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032

Project Title:
Engineer:
Project ID:
Project Descr:

Combined Footing

Project File: 251023.ec6

LIC#: KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F3 - Pad Footing at Gridline B

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+0.50Lr+W	-19.093	14.175	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.031
+1.20D+0.50Lr+W	-18.647	14.220	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.031
+1.20D+0.50Lr+W	-18.205	14.265	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.030
+1.20D+0.50Lr+W	-17.769	14.310	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.029
+1.20D+0.50Lr+W	-17.339	14.355	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.029
+1.20D+0.50Lr+W	-16.913	14.400	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.028
+1.20D+0.50Lr+W	-16.493	14.445	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.027
+1.20D+0.50Lr+W	-16.078	14.490	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+1.20D+0.50Lr+W	-15.669	14.535	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+1.20D+0.50Lr+W	-15.264	14.580	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.025
+1.20D+0.50Lr+W	-14.865	14.625	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+0.50Lr+W	-14.471	14.670	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+0.50Lr+W	-14.083	14.715	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+0.50Lr+W	-13.700	14.760	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+0.50Lr+W	-13.322	14.805	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.022
+1.20D+0.50Lr+W	-12.949	14.850	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.021
+1.20D+0.50Lr+W	-12.582	14.895	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.021
+1.20D+0.50Lr+W	-12.220	14.940	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.020
+1.20D+0.50Lr+W	-11.863	14.985	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.020
+1.20D+0.50Lr+W	-11.512	15.030	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.019
+1.20D+0.50Lr+W	-11.165	15.075	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.018
+1.20D+0.50Lr+W	-10.825	15.120	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.018
+1.20D+0.50Lr+W	-10.489	15.165	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.017
+1.20D+0.50Lr+W	-10.159	15.210	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.017
+1.20D+0.50Lr+W	-9.834	15.255	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.016
+1.20D+0.50Lr+W	-9.514	15.300	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.016
+1.20D+0.50Lr+W	-9.199	15.345	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.015
+1.20D+0.50Lr+W	-8.890	15.390	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.015
+1.20D+0.50Lr+W	-8.586	15.435	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.014
+1.20D+0.50Lr+W	-8.288	15.480	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.014
+1.20D+0.50Lr+W	-7.994	15.525	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.013
+1.20D+0.50Lr+W	-7.706	15.570	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.013
+1.20D+0.50Lr+W	-7.423	15.615	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.012
+1.20D+0.50Lr+W	-7.146	15.660	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.012
+1.20D+0.50Lr+W	-6.874	15.705	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.011
+1.20D+0.50Lr+W	-6.607	15.750	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.011
+1.20D+0.50Lr+W	-6.345	15.795	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.010
+1.20D+0.50Lr+W	-6.089	15.840	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.010
+1.20D+0.50Lr+W	-5.838	15.885	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.010
+1.20D+0.50Lr+W	-5.592	15.930	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.009
+1.20D+0.50Lr+W	-5.352	15.975	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.009
+1.20D+0.50Lr+W	-5.116	16.020	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.008
+1.20D+0.50Lr+W	-4.886	16.065	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.008
+1.20D+0.50Lr+W	-4.662	16.110	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.008
+1.20D+0.50Lr+W	-4.442	16.155	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+1.20D+0.50Lr+W	-4.228	16.200	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+1.20D+0.50Lr+W	-4.020	16.245	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+1.20D+0.50Lr+W	-3.816	16.290	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+1.20D+0.50Lr+W	-3.618	16.335	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+1.20D+0.50Lr+W	-3.425	16.380	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+1.20D+0.50Lr+W	-3.237	16.425	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+1.20D+0.50Lr+W	-3.055	16.470	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+1.20D+0.50Lr+W	-2.878	16.515	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+1.20D+0.50Lr+W	-2.706	16.560	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+1.20D+0.50Lr+W	-2.540	16.605	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+1.20D+0.50Lr+W	-2.379	16.650	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+1.20D+0.50Lr+W	-2.223	16.695	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+1.20D+0.50Lr+W	-2.072	16.740	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50Lr+W	-1.927	16.785	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50Lr+W	-1.787	16.830	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50Lr+W	-1.652	16.875	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50Lr+W	-1.522	16.920	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50Lr+W	-1.398	16.965	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002

Project Title:
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Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F3 - Pad Footing at Gridline B

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+0.50Lr+W	-1.279	17.010	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50Lr+W	-1.166	17.055	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50Lr+W	-1.057	17.100	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50Lr+W	-0.954	17.145	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50Lr+W	-0.856	17.190	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50Lr+W	-0.764	17.235	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50Lr+W	-0.677	17.280	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50Lr+W	-0.595	17.325	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50Lr+W	-0.518	17.370	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50Lr+W	-0.447	17.415	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50Lr+W	-0.381	17.460	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50Lr+W	-0.320	17.505	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50Lr+W	-0.264	17.550	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50Lr+W	-0.214	17.595	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50Lr+W	-0.169	17.640	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50Lr+W	-0.130	17.685	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50Lr+W	-0.095	17.730	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50Lr+W	-0.066	17.775	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50Lr+W	-0.042	17.820	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50Lr+W	-0.024	17.865	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50Lr+W	-0.011	17.910	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50Lr+W	0.000	17.955	0	0.000	0	0.000	0.000	0.000
+1.20D+0.50Lr+W	0.000	18.000	0	0.000	0	0.000	0.000	0.000

One Way Shear

Left of Column #1

Right of Column #1

Left of Column #2

Right of Column #2

Load Combination...	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+1.40D	2.2	82.2	0.027	1.9	82.2	0.023	0.4	82.2	0.005	1.9	82.2	0.023
+1.20D+0.50Lr	2.1	82.2	0.026	1.8	82.2	0.022	0.4	82.2	0.005	1.8	82.2	0.022
+1.20D+0.50S	2.2	82.2	0.027	1.9	82.2	0.023	0.4	82.2	0.005	1.9	82.2	0.023
+1.20D+1.60Lr	2.6	82.2	0.031	2.2	82.2	0.027	0.4	82.2	0.005	2.2	82.2	0.027
+1.20D+1.60Lr+0.50W	6.3	82.2	0.077	13.7	82.2	0.167	3.3	82.2	0.040	13.7	82.2	0.167
+1.20D+1.60S	2.9	82.2	0.036	2.6	82.2	0.031	0.5	82.2	0.006	2.6	82.2	0.031
+1.20D+1.60S+0.50W	6.6	82.2	0.081	14.0	82.2	0.171	3.2	82.2	0.039	14.0	82.2	0.171
+1.20D+0.50Lr+W	12.2	82.2	0.149	24.3	82.2	0.296	4.0	82.2	0.048	24.3	82.2	0.296
+1.20D+0.50S+W	12.3	82.2	0.150	24.3	82.2	0.296	4.0	82.2	0.048	24.3	82.2	0.296
+0.90D+W	17.8	82.2	0.216	27.1	82.2	0.330	3.0	82.2	0.036	27.1	82.2	0.330
+1.20D+0.20S	2.0	82.2	0.025	1.7	82.2	0.021	0.4	82.2	0.005	1.7	82.2	0.021
+0.90D	1.4	82.2	0.017	1.2	82.2	0.015	0.3	82.2	0.003	1.2	82.2	0.015

Two Way Shear

Column #1

Column #2

Load Combination...	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+1.40D	1.9	164.3	0.012	2.1	164.3	0.013
+1.20D+0.50Lr	1.8	164.3	0.011	2.0	164.3	0.012
+1.20D+0.50S	1.9	164.3	0.012	2.2	164.3	0.013
+1.20D+1.60Lr	2.3	164.3	0.014	2.5	164.3	0.015
+1.20D+1.60Lr+0.50W	7.9	164.3	0.048	9.0	164.3	0.055
+1.20D+1.60S	2.6	164.3	0.016	2.9	164.3	0.017
+1.20D+1.60S+0.50W	8.2	164.3	0.050	9.3	164.3	0.057
+1.20D+0.50Lr+W	13.5	164.3	0.082	15.7	164.3	0.095
+1.20D+0.50S+W	13.6	164.3	0.083	15.8	164.3	0.096
+0.90D+W	14.6	164.3	0.089	14.6	164.3	0.089
+1.20D+0.20S	1.8	164.3	0.011	2.0	164.3	0.012

Project Title:
Engineer:
Project ID:
Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F3 - Pad Footing at Gridline B

Two Way Shear	Column #1			Column #2		
	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+0.90D	1.2	164.3	0.008	1.4	164.3	0.008

Footing F4 - footing for Braced Frame @ Line G

Check Footing at Braced Frame at Line G

$$w_{ftg} := 5\text{ft}$$

Width of Footing

$$L_{ftg} := 14\cdot\text{ft}$$

Length of footing

$$h_{ftg} := 36\cdot\text{in}$$

Depth of Footing

$$L_{bf} := 5.5\text{ft}$$

distance between column

$$L_{ends} := \frac{L_{ftg} - L_{bf}}{2} \quad L_{ends} = 4.25\text{ft}$$

distance from column to footing end

Load at Columns from RISA3D reactions

$$P_{DL.1} := 10.486\cdot\text{k} \quad V_{DL.1} := 1.036\cdot\text{k}$$

$$P_{RLL.1} := 2.830\cdot\text{k} \quad V_{RLL.1} := 0.224\cdot\text{k}$$

From RISA output (Unfactored)
at Column 1 (Node N1)

$$P_{SL.1} := 4.297\cdot\text{k} \quad V_{SL.1} := 0.398\cdot\text{k}$$

$$P_{WL.1} := 37.424\text{k} \quad V_{WL.1} := 6.840\text{k}$$

$$P_{EQ.1} := \text{"N/A"} \quad V_{EQ.1} := \text{"N/A"}$$

$$P_{DL.2} := 4.386\cdot\text{k} \quad V_{DL.2} := -1.036\cdot\text{k}$$

$$P_{RLL.2} := 0.343\cdot\text{k} \quad V_{RLL.2} := -0.224\cdot\text{k}$$

From RISA output (Unfactored)
at Column 2 (Node N3)

$$P_{SL.2} := 1.336\cdot\text{k} \quad V_{SL.2} := -0.398\cdot\text{k}$$

$$P_{WL.2} := -37.424\text{k} \quad V_{WL.2} := 6.954\text{k}$$

$$P_{EQ.2} := \text{"N/A"} \quad V_{EQ.2} := \text{"N/A"}$$

$$A_{req.1} := 0.0018 \cdot (w_{ftg} \cdot h_{ftg})$$

$$A_{req.1} = 3.888 \cdot \text{in}^2$$

$$A_7 := \left(\frac{0.875 \cdot \text{in}}{2} \right)^2 \cdot \pi \quad A_7 = 0.601 \cdot \text{in}^2$$

$$N_{bar.req} := \frac{A_{req.1}}{A_7} = 6.466$$

use a min of (7) #7 rebar top and bottom.

Use: 5'-0" x 14'-0" x 3'-0" Footing
W/ (7) #7 LW T&B cont. &
#6 @ 12" SW T&B

Project Title:
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 Project Descr:

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Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

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DESCRIPTION: Footing F4 - Pad Footing at Gridline E

Code References

Governing Code : IBC 2018, CBC 2019
 Referenced Design Standard(s) : ACI 318-14
 Load Combinations Used : ASCE 7-16

Rho (or Omega) = 1.00

General Information

Material Properties

f'_c : Concrete 28 day strength 3.0 ksi
 f_y : Rebar Yield 60.0 ksi
 E_c : Concrete Elastic Modulus 3,122.0 ksi
 Concrete Density 145.0 pcf
 ϕ : Phi Values Flexure : 0.90
 Shear : 0.750

Analysis/Design Settings

Calculate footing weight as dead load ? Yes
 Calculate Pedestal weight as dead load ? Yes
 Min Steel Ratio per ACI 7.6.1.1 0.0018
 (ratio=Steel Area/Concrete Area)
 Min. Overturning Safety Factor 1.0: 1
 Min. Sliding Safety Factor 1.0: 1

Soil Information

Allowable Soil Bearing 2.0 ksf
 Increase Bearing By Footing Weight No
 Soil Passive Sliding Resistance 250.0 pcf
 (Uses entry for "Footing base depth below soil surface" for force)
 Coefficient of Soil/Concrete Friction 0.250

Soil Bearing Increase

Footing base depth below soil surface 3.0 ft
 Increases based on footing Depth . . .
 Allowable pressure increase per foot ksf
 when base of footing is below ft
 Increases based on footing Width . . .
 Allowable pressure increase per foot ksf
 when maximum length or width is greater tha ft
 Maximum Allowed Bearing Pressure 10.0 ksf
 (A value of zero implies no limit)
 Adjusted Allowable Soil Bearing 2.0 ksf
 (Allowable Soil Bearing adjusted for footing weight and depth & width increases as specified by user.)

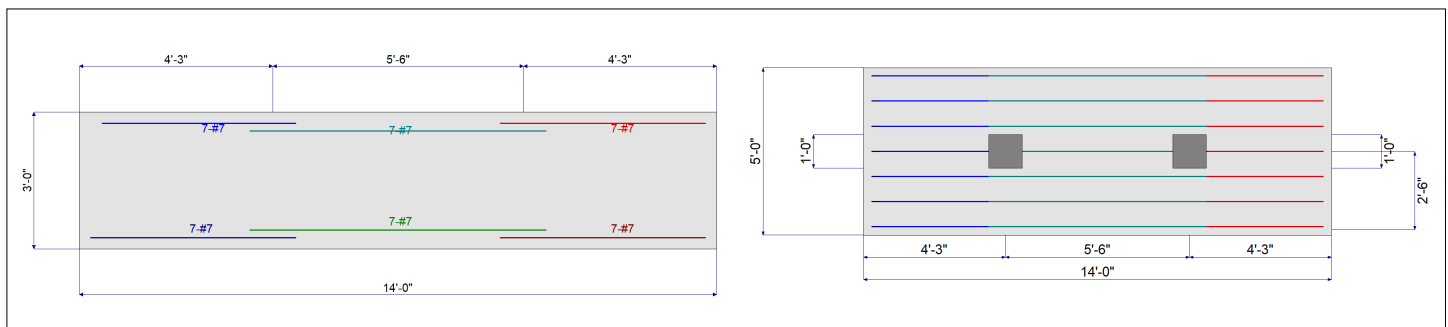
Dimensions & Reinforcing

Distance Left of Column #1	=	4.250 ft	Pedestal dimensions...									As
Between Columns	=	5.50 ft			Col #1	Col #2						Provided
Distance Right of Column #2	=	4.250 ft	Sq. Dim.	=	12.0	12.0 in						
Total Footing Length	=	14.0 ft	Height	=								
Footing Width	=	5.0 ft										
Footing Thickness	=	36.0 in										
Rebar Center to Concrete Edge @ Top	=	3.0 in										
Rebar Center to Concrete Edge @ Bottom	=	3.0 in										

Bars left of Col #1	Count	Size #	As
Bottom Bars	7.0	7	4.20
Top Bars	7.0	7	4.20
Bars Btwn Cols			
Bottom Bars	7.0	7	4.20
Top Bars	7.0	7	4.20
Bars Right of Col #2			
Bottom Bars	7.0	7	4.20
Top Bars	7.0	7	4.20

Applied Loads

Applied @ Left Column	D	Lr	L	S	W	E	H
Axial Load Downward	=	10.486	2.830		4.297	37.424	k
Moment (+CW)	=						k-ft
Shear (+X)	=	1.036	0.2240		0.3980	6.840	k
Applied @ Right Column							
Axial Load Downward	=	4.386	0.3430		1.336	-37.424	k
Moment (+CW)	=						k-ft
Shear (+X)	=	-1.036	-0.2240		-0.3980	6.594	k
Overburden	=						



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Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F4 - Pad Footing at Gridline E

DESIGN SUMMARY

Design N.G.

Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS 1.906	Overturning	222.038 k-ft	423.199 k-ft	+D+0.60W
PASS 2.104	Sliding	8.060 k	16.956 k	+D+0.60W
PASS 3.018	Uplift	22.454 k	67.776 k	+D+0.60W

Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS 0.6798	Soil Bearing	1.360 ksf	2.0 ksf	+D+0.60W
PASS 0.1285	1-way Shear - Col #1	10.559 psi	82.158 psi	+1.20D+0.50S+W
PASS 0.1285	1-way Shear - Col #2	10.559 psi	82.158 psi	+1.20D+0.50S+W
PASS 0.04263	2-way Punching - Col #1	7.005 psi	164.317 psi	+1.20D+0.50S+W
PASS 0.05733	2-way Punching - Col #2	9.420 psi	164.317 psi	+1.20D+0.50S+W
PASS No Tension	Flexure - Left of Col #1 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS 0.07963	Flexure - Left of Col #1 - Bottom	48.425 k-ft	608.14 k-ft	+0.90D+W
PASS 0.04652	Flexure - Between Cols - Top	-28.291 k-ft	608.14 k-ft	+1.20D+0.50Lr+W
PASS 0.1190	Flexure - Between Cols - Bottom	72.396 k-ft	608.14 k-ft	+0.90D+W
FAIL 0.03004	Flexure - Right of Col #2 - Top	-18.267 k-ft	608.14 k-ft	+1.20D+0.50Lr+W
PASS 0.01251	Flexure - Right of Col #2 - Bottom	7.605 k-ft	608.14 k-ft	+1.20D+1.60S

Soil Bearing

CAPACITY <10% OK

Load Combination...	Total Bearing	from Ftg CL	Actual Soil Bearing Stress @ Left Edge	Actual Soil Bearing Stress @ Right Edge	Allowable	Actual / Allow Ratio
D Only	45.32 k	-0.370 ft	0.75 ksf	0.55 ksf	2.00 ksf	0.375
+D+Lr	48.50 k	-0.487 ft	0.84 ksf	0.55 ksf	2.00 ksf	0.419
+D+S	50.96 k	-0.489 ft	0.88 ksf	0.58 ksf	2.00 ksf	0.440
+D+0.750Lr	47.70 k	-0.459 ft	0.82 ksf	0.55 ksf	2.00 ksf	0.408
+D+0.750S	49.55 k	-0.462 ft	0.85 ksf	0.57 ksf	2.00 ksf	0.424
+D+0.60W	45.32 k	-2.562 ft	1.36 ksf	0.00 ksf	2.00 ksf	0.680
+D+0.750Lr+0.450W	47.70 k	-2.021 ft	1.27 ksf	0.09 ksf	2.00 ksf	0.635
+D+0.750S+0.450W	49.55 k	-1.965 ft	1.30 ksf	0.11 ksf	2.00 ksf	0.651
+0.60D+0.750S	31.42 k	-0.515 ft	0.55 ksf	0.35 ksf	2.00 ksf	0.274
+0.60D	27.19 k	-0.370 ft	0.45 ksf	0.33 ksf	2.00 ksf	0.225

Overturning Stability

Load Combination...	Moments about Left Edge k-ft			Moments about Right Edge k-ft		
	Overturning	Resisting	Ratio	Overturning	Resisting	Ratio
D Only	3.11	303.59	97.679	3.11	337.14	108.474
+D+Lr	3.78	319.63	84.558	3.78	366.86	97.053
+D+S	4.30	336.07	78.119	4.30	385.90	89.704
+D+0.750Lr	3.61	315.62	87.381	3.61	359.43	99.510
+D+0.750S	4.00	327.95	81.915	4.00	373.71	93.347
+D+0.60W	222.04	423.20	1.906	122.72	556.07	4.531
+D+0.750Lr+0.450W	167.81	405.33	2.415	93.32	523.63	5.611
+D+0.750S+0.450W	168.20	417.66	2.483	93.71	537.91	5.740
+0.60D+0.750S	2.76	206.51	74.816	2.76	238.86	86.533
+0.60D	1.86	182.15	97.679	1.86	202.28	108.474

Sliding Stability

Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio
D Only	0.00 k	16.96 k	999
+D+Lr	0.00 k	17.75 k	999
+D+S	0.00 k	18.36 k	999
+D+0.750Lr	0.00 k	17.55 k	999
+D+0.750S	0.00 k	18.01 k	999
+D+0.60W	8.06 k	16.96 k	2.104
+D+0.750Lr+0.450W	6.05 k	17.55 k	2.903
+D+0.750S+0.450W	6.05 k	18.01 k	2.979
+0.60D+0.750S	0.00 k	13.48 k	999
+0.60D	0.00 k	12.42 k	999

Project Title:
 Engineer:
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Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F4 - Pad Footing at Gridline E

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.60D	0.000	0.000	0	0.000	0	0.000	0.000	0.000
+0.60D	0.000	0.035	0	0.000	0	0.000	0.000	0.000
+0.90D+W	0.021	0.070	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.047	0.105	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.084	0.140	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.131	0.175	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.188	0.210	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.255	0.245	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+0.90D+W	0.333	0.280	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+0.90D+W	0.420	0.315	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+0.90D+W	0.518	0.350	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+0.90D+W	0.626	0.385	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+0.90D+W	0.743	0.420	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+0.90D+W	0.870	0.455	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+0.90D+W	1.008	0.490	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+0.90D+W	1.154	0.525	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+0.90D+W	1.311	0.560	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+0.90D+W	1.477	0.595	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+0.90D+W	1.653	0.630	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+0.90D+W	1.838	0.665	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+0.90D+W	2.033	0.700	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+0.90D+W	2.237	0.735	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+0.90D+W	2.450	0.770	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+0.90D+W	2.673	0.805	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+0.90D+W	2.904	0.840	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+0.90D+W	3.145	0.875	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+0.90D+W	3.395	0.910	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+0.90D+W	3.655	0.945	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+0.90D+W	3.923	0.980	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+0.90D+W	4.200	1.015	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+0.90D+W	4.485	1.050	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+0.90D+W	4.780	1.085	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.008
+0.90D+W	5.083	1.120	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.008
+0.90D+W	5.395	1.155	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.009
+0.90D+W	5.716	1.190	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.009
+0.90D+W	6.045	1.225	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.010
+0.90D+W	6.383	1.260	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.010
+0.90D+W	6.729	1.295	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.011
+0.90D+W	7.083	1.330	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.012
+0.90D+W	7.446	1.365	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.012
+0.90D+W	7.817	1.400	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.013
+0.90D+W	8.197	1.435	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.013
+0.90D+W	8.584	1.470	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.014
+0.90D+W	8.979	1.505	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.015
+0.90D+W	9.383	1.540	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.015
+0.90D+W	9.795	1.575	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.016
+0.90D+W	10.214	1.610	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.017
+0.90D+W	10.641	1.645	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.017
+0.90D+W	11.076	1.680	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.018
+0.90D+W	11.519	1.715	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.019
+0.90D+W	11.970	1.750	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.020
+0.90D+W	12.428	1.785	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.020
+0.90D+W	12.893	1.820	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.021
+0.90D+W	13.366	1.855	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.022
+0.90D+W	13.847	1.890	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+0.90D+W	14.335	1.925	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+0.90D+W	14.830	1.960	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+0.90D+W	15.333	1.995	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.025
+0.90D+W	15.843	2.030	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+0.90D+W	16.360	2.065	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.027
+0.90D+W	16.884	2.100	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.028
+0.90D+W	17.415	2.135	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.029
+0.90D+W	17.953	2.170	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.030

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F4 - Pad Footing at Gridline E

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	18.498	2.205	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.030
+0.90D+W	19.049	2.240	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.031
+0.90D+W	19.608	2.275	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+0.90D+W	20.173	2.310	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.033
+0.90D+W	20.745	2.345	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.034
+0.90D+W	21.324	2.380	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.035
+0.90D+W	21.909	2.415	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.036
+0.90D+W	22.501	2.450	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.037
+0.90D+W	23.099	2.485	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.038
+0.90D+W	23.703	2.520	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.039
+0.90D+W	24.314	2.555	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.040
+0.90D+W	24.931	2.590	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.041
+0.90D+W	25.554	2.625	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.042
+0.90D+W	26.184	2.660	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.043
+0.90D+W	26.819	2.695	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.044
+0.90D+W	27.461	2.730	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.045
+0.90D+W	28.108	2.765	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.046
+0.90D+W	28.762	2.800	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.047
+0.90D+W	29.421	2.835	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.048
+0.90D+W	30.086	2.870	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.049
+0.90D+W	30.757	2.905	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.051
+0.90D+W	31.433	2.940	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.052
+0.90D+W	32.116	2.975	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.053
+0.90D+W	32.803	3.010	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.054
+0.90D+W	33.496	3.045	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.055
+0.90D+W	34.195	3.080	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.056
+0.90D+W	34.899	3.115	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.057
+0.90D+W	35.608	3.150	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.059
+0.90D+W	36.323	3.185	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.060
+0.90D+W	37.043	3.220	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.061
+0.90D+W	37.768	3.255	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.062
+0.90D+W	38.498	3.290	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.063
+0.90D+W	39.233	3.325	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.065
+0.90D+W	39.973	3.360	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.066
+0.90D+W	40.718	3.395	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.067
+0.90D+W	41.468	3.430	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.068
+0.90D+W	42.222	3.465	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.069
+0.90D+W	42.982	3.500	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.071
+0.90D+W	43.746	3.535	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.072
+0.90D+W	44.514	3.570	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.073
+0.90D+W	45.287	3.605	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.074
+0.90D+W	46.065	3.640	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.076
+0.90D+W	46.847	3.675	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.077
+0.90D+W	47.634	3.710	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.078
+0.90D+W	48.425	3.745	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.080
+0.90D+W	49.199	3.780	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.081
+0.90D+W	49.920	3.815	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.082
+0.90D+W	50.588	3.850	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.083
+0.90D+W	51.203	3.885	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.084
+0.90D+W	51.765	3.920	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.085
+0.90D+W	52.273	3.955	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.086
+0.90D+W	52.727	3.990	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.087
+0.90D+W	53.128	4.025	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.087
+0.90D+W	53.476	4.060	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.088
+0.90D+W	53.770	4.095	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.088
+0.90D+W	54.010	4.130	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.089
+0.90D+W	54.196	4.165	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.089
+0.90D+W	54.328	4.200	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.089
+0.90D+W	54.407	4.235	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.089
+0.90D+W	77.748	4.270	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.128
+0.90D+W	77.719	4.305	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.128
+0.90D+W	77.635	4.340	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.128
+0.90D+W	77.498	4.375	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.127

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F4 - Pad Footing at Gridline E

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	77.306	4.410	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.127
+0.90D+W	77.060	4.445	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.127
+0.90D+W	76.759	4.480	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.126
+0.90D+W	76.404	4.515	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.126
+0.90D+W	75.995	4.550	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.125
+0.90D+W	75.531	4.585	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.124
+0.90D+W	75.013	4.620	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.123
+0.90D+W	74.440	4.655	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.122
+0.90D+W	73.813	4.690	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.121
+0.90D+W	73.130	4.725	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.120
+0.90D+W	72.396	4.760	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.119
+0.90D+W	71.649	4.795	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.118
+0.90D+W	70.904	4.830	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.117
+0.90D+W	70.163	4.865	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.115
+0.90D+W	69.423	4.900	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.114
+0.90D+W	68.686	4.935	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.113
+0.90D+W	67.951	4.970	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.112
+0.90D+W	67.218	5.005	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.111
+0.90D+W	66.488	5.040	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.109
+0.90D+W	65.759	5.075	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.108
+0.90D+W	65.033	5.110	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.107
+0.90D+W	64.309	5.145	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.106
+0.90D+W	63.586	5.180	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.105
+0.90D+W	62.865	5.215	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.103
+0.90D+W	62.147	5.250	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.102
+0.90D+W	61.430	5.285	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.101
+0.90D+W	60.714	5.320	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.100
+0.90D+W	60.000	5.355	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.099
+0.90D+W	59.288	5.390	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.097
+0.90D+W	58.577	5.425	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.096
+0.90D+W	57.868	5.460	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.095
+0.90D+W	57.160	5.495	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.094
+0.90D+W	56.454	5.530	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.093
+0.90D+W	55.748	5.565	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.092
+0.90D+W	55.044	5.600	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.091
+0.90D+W	54.341	5.635	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.089
+0.90D+W	53.640	5.670	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.088
+0.90D+W	52.939	5.705	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.087
+0.90D+W	52.239	5.740	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.086
+0.90D+W	51.540	5.775	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.085
+0.90D+W	50.842	5.810	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.084
+0.90D+W	50.145	5.845	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.082
+0.90D+W	49.448	5.880	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.081
+0.90D+W	48.753	5.915	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.080
+0.90D+W	48.057	5.950	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.079
+0.90D+W	47.363	5.985	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.078
+0.90D+W	46.669	6.020	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.077
+0.90D+W	45.975	6.055	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.076
+0.90D+W	45.282	6.090	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.074
+0.90D+W	44.589	6.125	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.073
+0.90D+W	43.896	6.160	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.072
+0.90D+W	43.204	6.195	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.071
+0.90D+W	42.511	6.230	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.070
+0.90D+W	41.819	6.265	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.069
+0.90D+W	41.127	6.300	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.068
+0.90D+W	40.435	6.335	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.066
+0.90D+W	39.742	6.370	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.065
+0.90D+W	39.050	6.405	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.064
+0.90D+W	38.357	6.440	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.063
+0.90D+W	37.665	6.475	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.062
+0.90D+W	36.971	6.510	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.061
+0.90D+W	36.278	6.545	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.060
+0.90D+W	35.584	6.580	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.059

Project Title:
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 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F4 - Pad Footing at Gridline E

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+0.90D+W	34.889	6.615	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.057
+0.90D+W	34.194	6.650	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.056
+0.90D+W	33.499	6.685	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.055
+0.90D+W	32.802	6.720	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.054
+0.90D+W	32.105	6.755	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.053
+0.90D+W	31.407	6.790	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.052
+0.90D+W	30.709	6.825	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.050
+0.90D+W	30.009	6.860	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.049
+0.90D+W	29.309	6.895	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.048
+0.90D+W	28.607	6.930	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.047
+0.90D+W	27.905	6.965	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.046
+0.90D+W	27.201	7.000	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.045
+0.90D+W	26.496	7.035	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.044
+0.90D+W	25.790	7.070	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.042
+0.90D+W	25.082	7.105	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.041
+0.90D+W	24.373	7.140	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.040
+0.90D+W	23.663	7.175	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.039
+0.90D+W	22.951	7.210	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.038
+0.90D+W	22.238	7.245	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.037
+0.90D+W	21.523	7.280	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.035
+0.90D+W	20.806	7.315	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.034
+0.90D+W	20.087	7.350	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.033
+0.90D+W	19.367	7.385	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+0.90D+W	18.645	7.420	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.031
+0.90D+W	17.921	7.455	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.029
+0.90D+W	17.195	7.490	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.028
+0.90D+W	16.468	7.525	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.027
+0.90D+W	15.738	7.560	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+0.90D+W	15.005	7.595	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.025
+1.20D+1.60S	14.527	7.630	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+1.60S	14.483	7.665	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+1.60S	14.441	7.700	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+1.60S	14.401	7.735	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+1.60S	14.363	7.770	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+1.60S	14.328	7.805	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+1.60S	14.295	7.840	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+1.60S	14.264	7.875	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.235	7.910	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.209	7.945	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.184	7.980	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.162	8.015	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.142	8.050	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.124	8.085	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.108	8.120	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.095	8.155	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.083	8.190	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.074	8.225	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.067	8.260	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.062	8.295	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.059	8.330	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.058	8.365	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.059	8.400	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.063	8.435	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.068	8.470	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.076	8.505	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+1.60S	14.086	8.540	Bottom	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+0.50Lr+W	-14.636	8.575	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+0.50Lr+W	-15.347	8.610	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.025
+1.20D+0.50Lr+W	-16.059	8.645	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+1.20D+0.50Lr+W	-16.772	8.680	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.028
+1.20D+0.50Lr+W	-17.485	8.715	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.029
+1.20D+0.50Lr+W	-18.199	8.750	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.030
+1.20D+0.50Lr+W	-18.914	8.785	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.031

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F4 - Pad Footing at Gridline E

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+0.50Lr+W	-19.629	8.820	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+1.20D+0.50Lr+W	-20.346	8.855	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.033
+1.20D+0.50Lr+W	-21.063	8.890	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.035
+1.20D+0.50Lr+W	-21.781	8.925	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.036
+1.20D+0.50Lr+W	-22.500	8.960	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.037
+1.20D+0.50Lr+W	-23.220	8.995	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.038
+1.20D+0.50Lr+W	-23.941	9.030	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.039
+1.20D+0.50Lr+W	-24.663	9.065	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.041
+1.20D+0.50Lr+W	-25.386	9.100	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.042
+1.20D+0.50Lr+W	-26.111	9.135	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.043
+1.20D+0.50Lr+W	-26.836	9.170	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.044
+1.20D+0.50Lr+W	-27.563	9.205	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.045
+1.20D+0.50Lr+W	-28.291	9.240	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.047
+1.20D+0.50Lr+W	-29.010	9.275	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.048
+1.20D+0.50Lr+W	-29.693	9.310	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.049
+1.20D+0.50Lr+W	-30.339	9.345	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.050
+1.20D+0.50Lr+W	-30.946	9.380	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.051
+1.20D+0.50Lr+W	-31.516	9.415	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.052
+1.20D+0.50Lr+W	-32.048	9.450	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.053
+1.20D+0.50Lr+W	-32.543	9.485	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.054
+1.20D+0.50Lr+W	-32.999	9.520	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.054
+1.20D+0.50Lr+W	-33.419	9.555	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.055
+1.20D+0.50Lr+W	-33.801	9.590	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.056
+1.20D+0.50Lr+W	-34.145	9.625	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.056
+1.20D+0.50Lr+W	-34.452	9.660	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.057
+1.20D+0.50Lr+W	-34.722	9.695	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.057
+1.20D+0.50Lr+W	-34.954	9.730	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.057
+1.20D+0.50Lr+W	-19.432	9.765	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+1.20D+0.50Lr+W	-19.590	9.800	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+1.20D+0.50Lr+W	-19.710	9.835	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+1.20D+0.50Lr+W	-19.794	9.870	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.033
+1.20D+0.50Lr+W	-19.840	9.905	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.033
+1.20D+0.50Lr+W	-19.849	9.940	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.033
+1.20D+0.50Lr+W	-19.820	9.975	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.033
+1.20D+0.50Lr+W	-19.755	10.010	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+1.20D+0.50Lr+W	-19.653	10.045	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+1.20D+0.50Lr+W	-19.514	10.080	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+1.20D+0.50Lr+W	-19.338	10.115	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.032
+1.20D+0.50Lr+W	-19.125	10.150	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.031
+1.20D+0.50Lr+W	-18.876	10.185	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.031
+1.20D+0.50Lr+W	-18.589	10.220	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.031
+1.20D+0.50Lr+W	-18.267	10.255	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.030
+1.20D+0.50Lr+W	-17.932	10.290	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.029
+1.20D+0.50Lr+W	-17.600	10.325	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.029
+1.20D+0.50Lr+W	-17.270	10.360	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.028
+1.20D+0.50Lr+W	-16.943	10.395	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.028
+1.20D+0.50Lr+W	-16.619	10.430	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.027
+1.20D+0.50Lr+W	-16.297	10.465	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.027
+1.20D+0.50Lr+W	-15.978	10.500	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+1.20D+0.50Lr+W	-15.662	10.535	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.026
+1.20D+0.50Lr+W	-15.349	10.570	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.025
+1.20D+0.50Lr+W	-15.039	10.605	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.025
+1.20D+0.50Lr+W	-14.731	10.640	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+0.50Lr+W	-14.426	10.675	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.024
+1.20D+0.50Lr+W	-14.125	10.710	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+0.50Lr+W	-13.826	10.745	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.023
+1.20D+0.50Lr+W	-13.531	10.780	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.022
+1.20D+0.50Lr+W	-13.238	10.815	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.022
+1.20D+0.50Lr+W	-12.949	10.850	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.021
+1.20D+0.50Lr+W	-12.663	10.885	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.021
+1.20D+0.50Lr+W	-12.380	10.920	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.020
+1.20D+0.50S+W	-12.100	10.955	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.020
+1.20D+0.50S+W	-11.824	10.990	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.019

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

(c) ENERCALC, LLC 1982-2026

DESCRIPTION: Footing F4 - Pad Footing at Gridline E

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+0.50S+W	-11.550	11.025	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.019
+1.20D+0.50S+W	-11.280	11.060	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.019
+1.20D+0.50S+W	-11.013	11.095	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.018
+1.20D+0.50S+W	-10.749	11.130	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.018
+1.20D+0.50S+W	-10.489	11.165	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.017
+1.20D+0.50S+W	-10.231	11.200	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.017
+1.20D+0.50S+W	-9.977	11.235	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.016
+1.20D+0.50S+W	-9.726	11.270	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.016
+1.20D+0.50S+W	-9.478	11.305	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.016
+1.20D+0.50S+W	-9.234	11.340	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.015
+1.20D+0.50S+W	-8.992	11.375	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.015
+1.20D+0.50S+W	-8.754	11.410	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.014
+1.20D+0.50S+W	-8.519	11.445	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.014
+1.20D+0.50S+W	-8.287	11.480	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.014
+1.20D+0.50S+W	-8.059	11.515	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.013
+1.20D+0.50S+W	-7.833	11.550	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.013
+1.20D+0.50S+W	-7.611	11.585	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.013
+1.20D+0.50S+W	-7.392	11.620	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.012
+1.20D+0.50S+W	-7.176	11.655	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.012
+1.20D+0.50S+W	-6.964	11.690	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.011
+1.20D+0.50S+W	-6.754	11.725	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.011
+1.20D+0.50S+W	-6.548	11.760	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.011
+1.20D+0.50S+W	-6.345	11.795	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.010
+1.20D+0.50S+W	-6.145	11.830	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.010
+1.20D+0.50S+W	-5.949	11.865	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.010
+1.20D+0.50S+W	-5.755	11.900	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.009
+1.20D+0.50S+W	-5.565	11.935	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.009
+1.20D+0.50S+W	-5.378	11.970	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.009
+1.20D+0.50S+W	-5.194	12.005	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.009
+1.20D+0.50S+W	-5.013	12.040	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.008
+1.20D+0.50S+W	-4.836	12.075	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.008
+1.20D+0.50S+W	-4.662	12.110	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.008
+1.20D+0.50S+W	-4.491	12.145	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+1.20D+0.50S+W	-4.323	12.180	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+1.20D+0.50S+W	-4.158	12.215	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+1.20D+0.50S+W	-3.997	12.250	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.007
+1.20D+0.50S+W	-3.838	12.285	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+1.20D+0.50S+W	-3.683	12.320	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+1.20D+0.50S+W	-3.531	12.355	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+1.20D+0.50S+W	-3.383	12.390	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.006
+1.20D+0.50S+W	-3.237	12.425	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+1.20D+0.50S+W	-3.095	12.460	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+1.20D+0.50S+W	-2.956	12.495	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+1.20D+0.50S+W	-2.820	12.530	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.005
+1.20D+0.50S+W	-2.687	12.565	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+1.20D+0.50S+W	-2.558	12.600	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+1.20D+0.50S+W	-2.432	12.635	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+1.20D+0.50S+W	-2.308	12.670	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+1.20D+0.50S+W	-2.189	12.705	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.004
+1.20D+0.50S+W	-2.072	12.740	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50S+W	-1.958	12.775	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50S+W	-1.848	12.810	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50S+W	-1.741	12.845	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50S+W	-1.637	12.880	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50S+W	-1.536	12.915	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.003
+1.20D+0.50S+W	-1.439	12.950	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50S+W	-1.344	12.985	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50S+W	-1.253	13.020	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50S+W	-1.165	13.055	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50S+W	-1.081	13.090	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50S+W	-0.999	13.125	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50S+W	-0.921	13.160	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.002
+1.20D+0.50S+W	-0.846	13.195	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F4 - Pad Footing at Gridline E

Z-Axis Footing Flexure - Maximum Values for Load Combination

Load Combination...	Mu (ft-k)	Distance from left (ft)	Tension Side	As Req'd (in^2)	Governed by	Actual As (in^2)	Phi*Mn (ft-k)	Mu / PhiMn
+1.20D+0.50S+W	-0.774	13.230	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50S+W	-0.705	13.265	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50S+W	-0.639	13.300	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50S+W	-0.577	13.335	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50S+W	-0.518	13.370	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50S+W	-0.462	13.405	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50S+W	-0.409	13.440	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50S+W	-0.360	13.475	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50S+W	-0.313	13.510	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.001
+1.20D+0.50S+W	-0.270	13.545	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.230	13.580	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.193	13.615	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.160	13.650	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.129	13.685	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.102	13.720	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.078	13.755	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.058	13.790	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.040	13.825	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.026	13.860	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	-0.014	13.895	Top	3.888	ACI 7.6.1.1	4.200	608.135	0.000
+1.20D+0.50S+W	0.000	13.930	0	0.000	0	0.000	0.000	0.000
+1.20D+0.50S+W	0.000	13.965	0	0.000	0	0.000	0.000	0.000
+1.20D+0.50S+W	0.000	14.000	0	0.000	0	0.000	0.000	0.000

One Way Shear

Left of Column #1

Right of Column #1

Left of Column #2

Right of Column #2

Load Combination...	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+1.40D	1.1	82.2	0.014	0.9	82.2	0.011	0.4	82.2	0.005	0.9	82.2	0.011
+1.20D+0.50Lr	1.1	82.2	0.013	0.9	82.2	0.011	0.4	82.2	0.005	0.9	82.2	0.011
+1.20D+0.50S	1.1	82.2	0.014	0.9	82.2	0.011	0.4	82.2	0.005	0.9	82.2	0.011
+1.20D+1.60Lr	1.3	82.2	0.016	1.2	82.2	0.014	0.4	82.2	0.005	1.2	82.2	0.014
+1.20D+1.60Lr+0.50W	2.5	82.2	0.031	6.1	82.2	0.075	0.8	82.2	0.010	6.1	82.2	0.075
+1.20D+1.60S	1.5	82.2	0.018	1.3	82.2	0.015	0.5	82.2	0.006	1.3	82.2	0.015
+1.20D+1.60S+0.50W	2.7	82.2	0.033	6.2	82.2	0.076	0.7	82.2	0.008	6.2	82.2	0.076
+1.20D+0.50Lr+W	3.7	82.2	0.045	10.5	82.2	0.128	1.3	82.2	0.016	10.5	82.2	0.128
+1.20D+0.50S+W	3.7	82.2	0.046	10.6	82.2	0.129	1.3	82.2	0.016	10.6	82.2	0.129
+0.90D+W	4.1	82.2	0.050	10.2	82.2	0.124	1.0	82.2	0.012	10.2	82.2	0.124
+1.20D+0.20S	1.0	82.2	0.013	0.8	82.2	0.010	0.4	82.2	0.005	0.8	82.2	0.010
+0.90D	0.7	82.2	0.009	0.6	82.2	0.007	0.3	82.2	0.003	0.6	82.2	0.007

Two Way Shear

Column #1

Column #2

Load Combination...	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+1.40D	1.6	164.3	0.010	1.9	164.3	0.012
+1.20D+0.50Lr	1.6	164.3	0.010	1.8	164.3	0.011
+1.20D+0.50S	1.6	164.3	0.010	1.9	164.3	0.012
+1.20D+1.60Lr	1.9	164.3	0.012	2.3	164.3	0.014
+1.20D+1.60Lr+0.50W	4.6	164.3	0.028	5.9	164.3	0.036
+1.20D+1.60S	2.2	164.3	0.013	2.6	164.3	0.016
+1.20D+1.60S+0.50W	4.9	164.3	0.030	6.2	164.3	0.038
+1.20D+0.50Lr+W	6.9	164.3	0.042	9.4	164.3	0.057
+1.20D+0.50S+W	7.0	164.3	0.043	9.4	164.3	0.057
+0.90D+W	6.6	164.3	0.040	8.8	164.3	0.054
+1.20D+0.20S	1.5	164.3	0.009	1.7	164.3	0.011

Project Title:
Engineer:
Project ID:
Project Descr:

Combined Footing

Project File: 251023.ec6

LIC# : KW-06018181, Build:20.26.07.20

BRAD YOUNG & ASSOCIATES, INC

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DESCRIPTION: Footing F4 - Pad Footing at Gridline E

Two Way Shear	Column #1			Column #2		
	vu (psi)	Φ vn (psi)	vu / Φ vn	vu (psi)	Φ vn (psi)	vu / Φ vn
+0.90D	1.1	164.3	0.006	1.2	164.3	0.007

Exterior Braced Frame - Anchor Bolts along Line B

Check the anchorage of the Braced columns for the uplift forces

Check the anchor bolts per ACI 318 chapter 17. The attached RISA3D report are in LRFD load combinations. Use Simpson Strong-Tie Anchor Selector for ACI 318 to check the anchor bolt group.

$$\text{Uplift}_{\max} := 71.390 \cdot \text{k}$$

Worst Case uplift force for braced frame column

$$\text{Shear}_{\max} := 13.892 \cdot \text{k}$$

Worst Case shear force for brace frame column

Use (4) 1" dia. F1554 Gr.36 Bolts
w/ 1-1/4" 12"x12" A36 Plate for each Braced
Frame
See attached Simpson Anchor output

Exterior Braced Frame - Anchor Bolts along Line E

Check the anchorage of the Braced columns for the uplift forces

Check the anchor bolts per ACI 318 chapter 17. The attached RISA3D report are in LRFD load combinations. Use Simpson Strong-Tie Anchor Selector for ACI 318 to check the anchor bolt group.

$$\text{Uplift}_{\max} := 33.512 \cdot \text{k}$$

Worst Case uplift force for braced frame column

$$\text{Shear}_{\max} := 8.272 \cdot \text{k}$$

Worst Case shear force for brace frame column

Use (4) 1" dia. F1554 Gr.36 Bolts
w/ 1-1/4" 12"x12" A36 Plate for each Braced
Frame

PASS BY INSPECTION

Note: Both Frames are similar and designed for **worst** case scenario.



**Anchor Designer™ for
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Version 3.4.2606.1

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Engineer:	R.V.	Page:	1
Project:	#251023		
Address:			
Phone:			
E-mail:			

1. Project information

Project description:
Location: Kansas City, KS
Design name: Design

Comment:

2. Input Data & Anchor Parameters

General

Design method: ACI 318-14
Units: Imperial units

Anchor Information:

Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 24.000
Anchor category: -
Anchor ductility: Yes
 h_{min} (inch): 25.75
 C_{min} (inch): 1.44
 S_{min} (inch): 4.00

Base Material

Concrete: Normal-weight
Concrete thickness, h (inch): 36.00
State: Cracked
Compressive strength, f'_c (psi): 3000
 $\Psi_{c,v}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6d requirement: Yes
Build-up grout pad: No

Base Plate

Length x Width x Thickness (inch): 12.00 x 12.00 x 1.25

Recommended Anchor

Anchor Name: Heavy Hex Bolt - 1"Ø Heavy Hex Bolt, F1554 Gr. 36





Anchor Designer™ for Concrete Software

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Address:			
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Load and Geometry

Load factor source: ACI 318 Section 5.3

Load combination: not set

Seismic design: No

Anchors subjected to sustained tension: Not applicable

Apply entire shear load at front row: No

Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 71390

V_{uax} [lb]: 13892

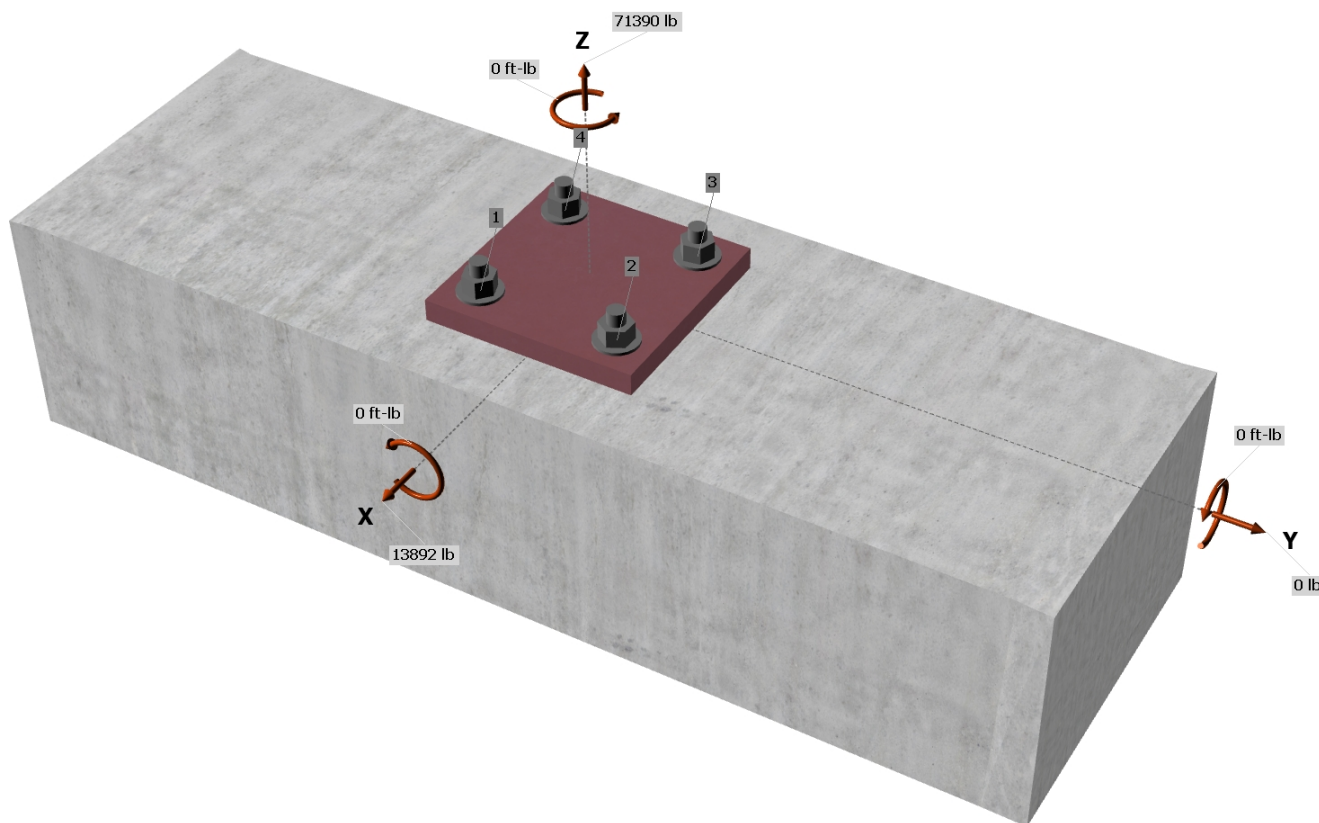
V_{uay} [lb]: 0

M_{ux} [ft-lb]: 0

M_{uy} [ft-lb]: 0

M_{uz} [ft-lb]: 0

<Figure 1>



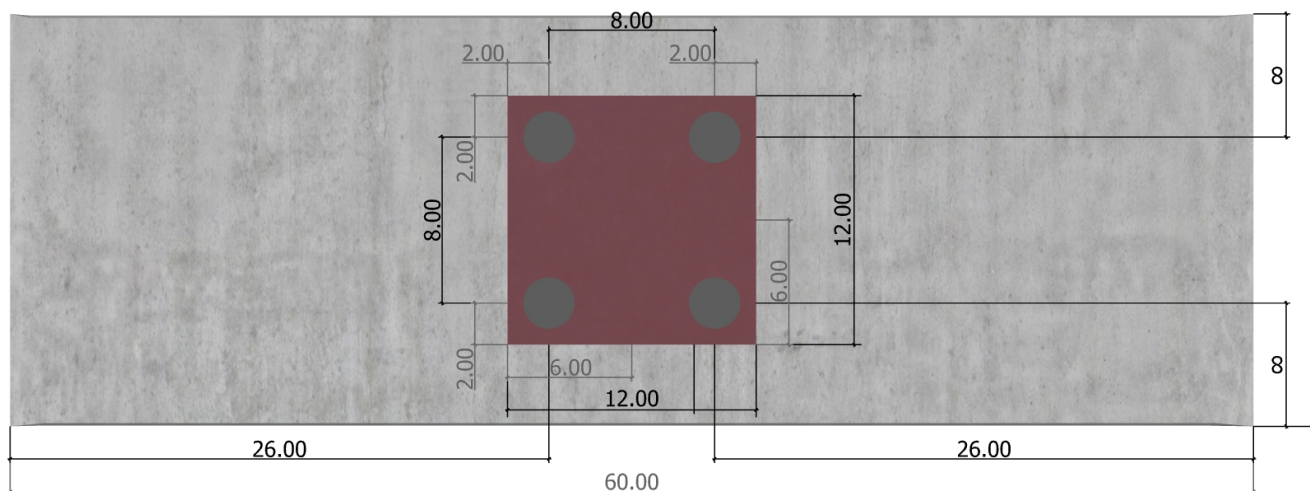
Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com



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<Figure 2>

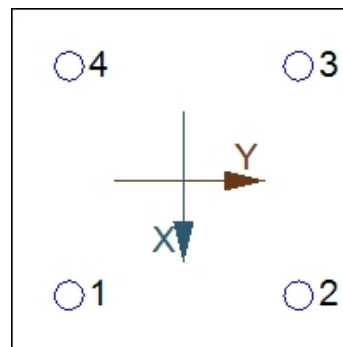


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2 + (V_{uay})^2}$ (lb)
1	17847.5	3473.0	0.0	3473.0
2	17847.5	3473.0	0.0	3473.0
3	17847.5	3473.0	0.0	3473.0
4	17847.5	3473.0	0.0	3473.0
Sum	71390.0	13892.0	0.0	13892.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 71390
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>





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4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$$N_b = 16\lambda_a \sqrt{f'_c} h_{ef}^{5/3} \text{ (Eq. 17.4.2.2b)}$$

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	3000	24.000	174998

$$\phi N_{cbg} = \phi (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \text{ (Sec. 17.3.1 & Eq. 17.4.2.1b)}$$

A_{Nc} (in ²)	A_{Nco} (in ²)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
4800.00	5184.00	26.00	1.000	0.917	1.00	1.000	174998	0.70	103972

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c \text{ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)}$$

$\Psi_{c,P}$	A_{brg} (in ²)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	1.50	3000	0.70	25217

7. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout} \phi V_{sa}$ (lb)
21090	1.0	0.65	13709

8. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear parallel to edge in y-direction:

$$V_{bx} = \min[7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5}; 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}] \text{ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)}$$

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
8.00	1.000	1.00	3000	26.00	65353

$$\phi V_{cbgy} = \phi (2)(A_{Vc} / A_{Vco}) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{bx} \text{ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)}$$

A_{Vc} (in ²)	A_{Vco} (in ²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
3096.00	3042.00	1.000	1.000	1.000	1.041	65353	0.70	96920

9. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$$\phi V_{cp} = \phi k_{cp} N_{cbg} = \phi k_{cp} (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \text{ (Sec. 17.3.1 & Eq. 17.5.3.1b)}$$

k_{cp}	A_{Nc} (in ²)	A_{Nco} (in ²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	4800.00	5184.00	1.000	0.917	1.000	1.000	174998	0.70	207945

10. Results

Interaction of Tensile and Shear Forces (Sec. R17.6)

Tension	Factored Load, N _{ua} (lb)	Design Strength, ϕN _n (lb)	Ratio	Status	
Steel	17848	26363	0.68	Pass	
Concrete breakout	71390	103972	0.69	Pass	
Pullout	17848	25217	0.71	Pass (Governs)	
Shear	Factored Load, V _{ua} (lb)	Design Strength, ϕV _n (lb)	Ratio	Status	
Steel	3473	13709	0.25	Pass (Governs)	
Concrete breakout y+	6946	96920	0.07	Pass	
Pryout	13892	207945	0.07	Pass	
Interaction check	(N _{ua} /ϕN _{ua}) ^{5/3}	(V _{ua} /ϕV _{ua}) ^{5/3}	Utilization Ratio	Permissible	Status
Sec. R17.6	0.56	0.10	66.4%	1.0	Pass

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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1"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 24.000 inch meets the selected design criteria.

11. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.
- Designer must exercise own judgment to determine if this design is suitable.

Braced Frame Connections

Brace to Col/Beam Connection

Check welded connection from braces to beam or column

$$F_{\text{conn}} := 47.786 \text{ k}$$

see RISA3D analysis

There will be flare bevel groove welds on minimum of two sides of the HSS 4x4x5/16 brace.

Note: Per the geometry of the frame (See 1/S3.1) there will be an approximate weld length of 7.125" along the brace to column and 1.625" along the brace to beam connection at the top of the frame. There will be an approximate weld length of 11.125" along the bottom brace to column connection. To be conservative, assume the weld connection in 7"

$$L_{\text{weld}} := (7 \cdot \text{in}) \cdot 2$$

$$L_{\text{weld}} = 14 \cdot \text{in}$$

$$F_{y, \text{brace}} := 46 \text{ ksi}$$

$$F_{u, \text{brace}} := 58 \text{ ksi}$$

$$F_{\text{exx}} := 70 \cdot \text{ksi}$$

$$A_{\text{brace}} := 4.10 \text{ in}^2$$

$$\phi_{\text{weld}} := 0.80$$

$$\phi_{\text{yield}} := 0.90$$

$$R := 2 \cdot \frac{5}{16} \text{ in}$$

$$R = 0.625 \cdot \text{in}$$

$$A_{\text{weld}} := \frac{3}{16} \cdot R \cdot L_{\text{weld}}$$

$$A_{\text{weld}} = 1.641 \cdot \text{in}^2$$

$$R_{n, \text{brace}} := \phi_{\text{weld}} \cdot F_{y, \text{brace}} \cdot A_{\text{brace}}$$

$$R_{n, \text{brace}} = 150.88 \cdot \text{k}$$

$$R_{n, \text{weld}} := \phi_{\text{yield}} \cdot 0.6 \cdot F_{\text{exx}} \cdot A_{\text{weld}}$$

$$R_{n, \text{weld}} = 62.016 \cdot \text{k}$$

$$R_n := \min(R_{n, \text{brace}}, R_{n, \text{weld}})$$

$$R_n = 62.016 \cdot \text{k}$$

$$\frac{F_{\text{conn}}}{R_n} = 0.771$$

< 1.0 connection ok

Column to Base plate

There will be 1/4" fillet weld from the HSS column to base plate.

$$F_{\text{conn}} := 86.566 \text{ k}$$

$$F_{y,\text{col}} := 46 \text{ ksi}$$

$$F_{u,\text{col}} := 58 \cdot \text{ksi}$$

$$L_{\text{weld}} := (4 \cdot \text{in}) \cdot 4$$

$$\phi_{\text{weld}} := 0.75$$

$$A_{\text{col}} := 4.10 \text{ in}^2$$

$$A_{\text{weld}} := \left(\frac{1}{4} \cdot \text{in} \right) \cdot \frac{1}{\sqrt{2}} \cdot L_{\text{weld}}$$

$$\phi R_{n,\text{col}} := \phi_{\text{yield}} \cdot F_{y,\text{col}} \cdot A_{\text{col}}$$

$$\phi R_{n,\text{weld}} := \phi_{\text{weld}} \cdot 0.6 \cdot F_{\text{exx}} \cdot A_{\text{weld}}$$

$$\phi R_n := \min(\phi R_{n,\text{col}}, \phi R_{n,\text{weld}})$$

$$\frac{F_{\text{conn}}}{\phi R_n} = 0.972$$

< 1.0 connection OK

HSS 4x4x5/16

$$F_{\text{exx}} := 70 \cdot \text{ksi}$$

$$L_{\text{weld}} = 16 \cdot \text{in}$$

$$\phi_{\text{yield}} := 0.9$$

$$A_{\text{weld}} = 2.828 \cdot \text{in}^2$$

$$\phi R_{n,\text{col}} = 169.74 \cdot \text{k}$$

$$\phi R_{n,\text{weld}} = 89.095 \cdot \text{k}$$

$$\phi R_n = 89.095 \cdot \text{k}$$

Thus Thicker Weld Ok

Low Brace Loading for DT Canopies

Cantilever Canopy Parameters

$$L_{\text{canopy}} := (4\text{ft} + 4\text{in}) = 4.333\text{ft}$$

$$d_{\text{header}} := 11.25\text{in}$$

depth of header

$$W_{\text{canopy}} := (6\text{ft} + 8\text{in}) = 6.667\text{ft}$$

$$A_{\text{canopy}} := (L_{\text{canopy}} \cdot W_{\text{canopy}}) = 28.889\text{ft}^2$$

Wind Calculations

$$q_h = 17.929\text{psf}$$

$$GC_{p2.ovhg} := -2.1$$

Positive Pressure Coefficient (Roof overhang Zone 2) - ASCE
7-16: FIG 30.4-2A

$$GC_{pi.ovhg} := 0.0$$

$$p_{\text{wind.ovhg}} := q_h \cdot (GC_{p2.ovhg} - GC_{pi.ovhg})$$

$$p_{\text{wind.ovhg}} = -37.652\text{psf}$$

Calculate the Moment per Loading

$$w_{DL} := DL_{\text{trellis}} \cdot 1\text{ft}$$

$$w_{DL} = 5\text{plf}$$

$$w_{LL} := LL_{\text{roof}} \cdot 1\text{ft}$$

$$w_{LL} = 20\text{plf}$$

$$w_{SL} := LL_{\text{snow}} \cdot 1\text{ft}$$

$$w_{SL} = 20\text{plf}$$

$$w_{\text{wind.up}} := p_{\text{wind.ovhg}} \cdot 1\text{ft}$$

$$w_{\text{wind.up}} = -37.652\text{plf}$$

$$M_{DL} := w_{DL} \cdot \frac{L_{\text{canopy}}^2}{2} = 47\text{lb}\cdot\text{ft}$$

$$M_{LL} := w_{LL} \cdot \frac{L_{\text{canopy}}^2}{2} = 187.778\text{lb}\cdot\text{ft}$$

$$M_S := w_{SL} \cdot \frac{L_{\text{canopy}}^2}{2} = 188\text{lb}\cdot\text{ft}$$

$$M_W := w_{\text{wind.up}} \cdot \frac{L_{\text{canopy}}^2}{2} = -354\text{lb}\cdot\text{ft}$$

Calculate the Horizontal Force acting upon the Brace from the Canopy per Loading

$$F_{p.h.D.only} := \frac{M_{DL}}{d_{header}}$$

$$F_{p.h.D.only} = 50.074 \text{ lb}$$

$$F_{p.h.Lr.only} := \frac{M_{Lr}}{d_{header}}$$

$$F_{p.h.Lr.only} = 200.296 \text{ lb}$$

$$F_{p.h.S.only} := \frac{M_S}{d_{header}}$$

$$F_{p.h.S.only} = 200.296 \text{ lb}$$

$$F_{p.h.W.only} := \frac{M_W}{d_{header}}$$

$$F_{p.h.W.only} = -377.075 \text{ lb}$$

Parameters of Brace

$$Ht_{roof} := (14\text{ft} + 6\text{in}) = 14.50\text{ft}$$

height of roof

$$Ht_{brace} := (10\text{ft} + 11\text{in}) = 10.917\text{ft}$$

height of brace above floor

$$Ly := Ht_{roof} - Ht_{brace} = 3.583\text{ft}$$

Vertical Length of Brace

$$Lx := (4\text{ft} + 0\text{in}) = 4\text{ft}$$

Horizontal Length of Brace

$$L_{brace} := \sqrt{(Lx)^2 + (Ly)^2} = 5.37\text{ft}$$

Length of the Brace

$$\text{Theta} := \text{asin}\left(\frac{Ly}{L_{brace}}\right) = 41.855 \cdot \text{deg}$$

Angle Adjacent to Canopy and Brace

Calculate the Axial Force acting upon the Brace from the Canopy per Loading

$$F_{p.axial.D.only} := \frac{F_{p.h.D.only}}{\cos(\Theta)} = 67.228 \text{ lb}$$

$$F_{p.axial.Lr.only} := \frac{F_{p.h.Lr.only}}{\cos(\Theta)} = 268.914 \text{ lb}$$

$$F_{p.axial.S.only} := \frac{F_{p.h.S.only}}{\cos(\Theta)} = 268.914 \text{ lb}$$

$$F_{p.axial.W.only} := \frac{F_{p.h.W.only}}{\cos(\Theta)} = -506.253 \text{ lb}$$

Calculate the Vertical Force acting upon the Brace from the Canopy per Loading

$$F_{p.vert.D.only} := F_{p.axial.D.only} \cdot \sin(\Theta) = 44.858 \text{ lb} \quad \text{Vertical Force at brace}$$

$$F_{p.vert.Lr.only} := F_{p.axial.Lr.only} \cdot \sin(\Theta) = 179.432 \text{ lb} \quad \text{Vertical Force at brace}$$

$$F_{p.vert.S.only} := F_{p.axial.S.only} \cdot \sin(\Theta) = 179.432 \text{ lb} \quad \text{Vertical Force at brace}$$

$$F_{p.vert.W.only} := F_{p.axial.W.only} \cdot \sin(\Theta) = -337.796 \text{ lb} \quad \text{Vertical Force at brace}$$

Convert back to a Distributed Load - ANSWER

$$W_{p.vert.D.only} := \frac{F_{p.vert.D.only}}{1\text{ft}} = 44.858 \cdot \text{plf}$$

$$W_{p.vert.Lr.only} := \frac{F_{p.vert.Lr.only}}{1\text{ft}} = 179.432 \cdot \text{plf}$$

$$W_{p.vert.S.only} := \frac{F_{p.vert.S.only}}{1\text{ft}} = 179.432 \cdot \text{plf}$$

$$W_{p.vert.W.only} := \frac{F_{p.vert.W.only}}{1\text{ft}} = -337.796 \cdot \text{plf}$$

For Drawings

Loading acting at the roof joist

$$w_{\text{uplift}} := W_{p.vert.D.only} + W_{p.vert.S.only} \quad w_{\text{uplift}} = 224.29 \frac{\text{lb}}{\text{ft}}$$

$$w_{\text{down}} := 0.6 \cdot W_{p.vert.D.only} - 0.6 W_{p.vert.W.only} \quad w_{\text{down}} = 229.593 \frac{\text{lb}}{\text{ft}}$$