METTEMEYER ENGINEERING LLC

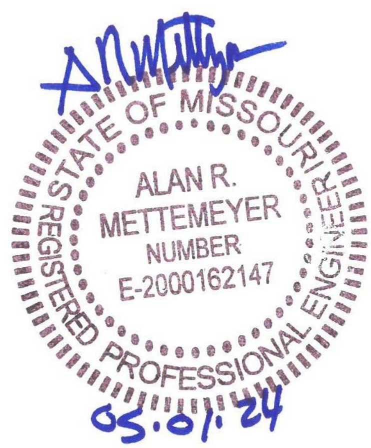
Andy's Frozen Custard #204

Lee's Summit, MO

Structural Calculations

Respectfully Submitted,

Mettemeyer Engineering, LLC



Joshua Farrar
Project Engineer

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Principal Structural Engineer

METTEMEYER ENGINEERING LLC

Project Number: 24-0121
Project Name: AFC - Lee's Summit

Date: 4/24/2024 Sheet 1 of 2

Design Criteria

Project Location: 700 NW Ward Road, Lee's Summit, MO 64086

Building Code: 2018 International Building Code

Loading - Typical To All Buildings

Dead Load

Roof -

Roofing - 3 psf
Sheathing - 3 psf
Insulation - 2 psf
Wood Trusses - 5 psf
Ceiling - 2 psf
MEP - 3 psf

Total - 18 psf, Use 20 psf

Walls

Gyp Walls - 8 psf (includes sheathing, studs, insulation)

Live Load

Roof - 20 psf

Snow

Ground Snow Load - 20 psf

Frost Depth - 36"

METTEMEYER ENGINEERING LLC

Project Number: 24-0121
Project Name: AFC - Lee's Summit

Date: 4/24/2024 Sheet 2 of 2

Design Criteria Cont.

Loading Cont.

Wind Load

ATC Hazard Website - See Attached

Risk Category 2

Wind Speed - 109 mph (Ultimate), 84 mph (Service)

Seismic

ATC Hazard Website - See Attached

Risk Category = 2

Site Class = C

Ss = 0.099
S1 = 0.068
SMS = 0.159
SM1 = 0.106

S_{DS} = 0.106
S_{D1} = 0.109

Seismic Design Category = B

Seismic Force-Resisting System

Main Building - Use: Light frame (wood) walls sheathed with wood structural panels rated for shear resistance

R = 6.5
Overstrength = 3
Defl. Amplification, C_d = 4

Canopy - Use: Cantilevered Columns

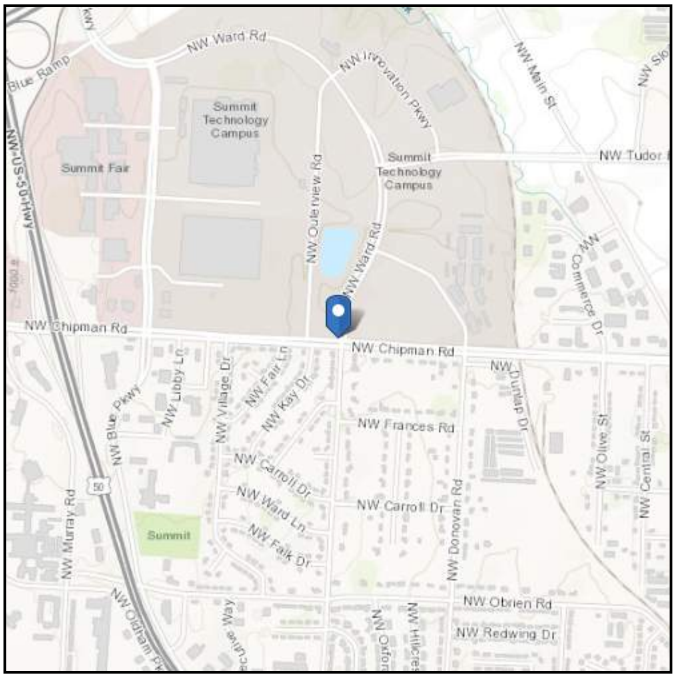
R = 1.25
Overstrength = 1.25
Defl. Amplification, Cd = 1.25

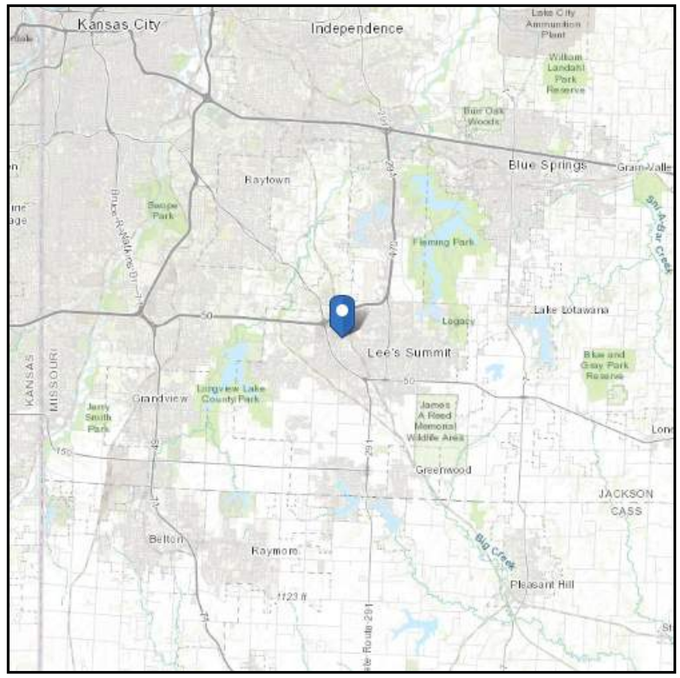
ASCE Hazards Report

Address:
700 NW Ward Rd
Lees Summit, Missouri
64086

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

Latitude: 38.925682
Longitude: -94.394104
Elevation: 998.2993978543723 ft (NAVD 88)





Wind

Results:

Wind Speed	109 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:	Thu Apr 25 2024

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.

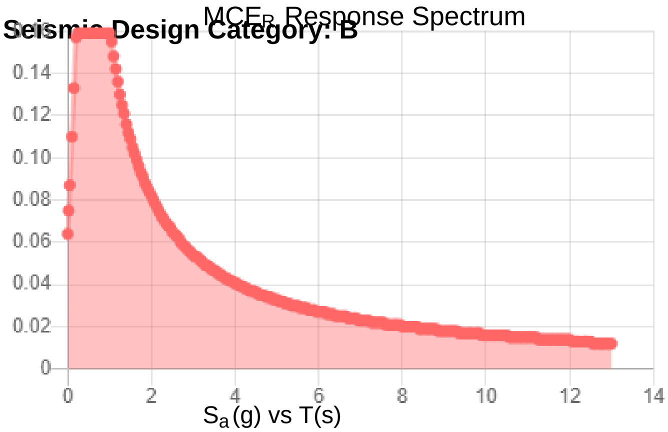
Seismic

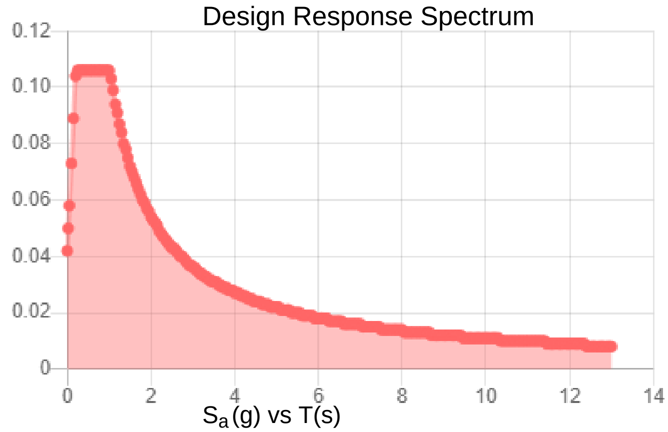
Site Soil Class: D - Stiff Soil

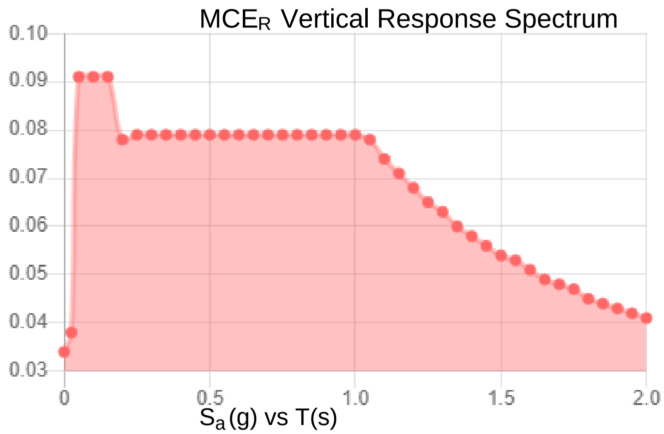
Results:

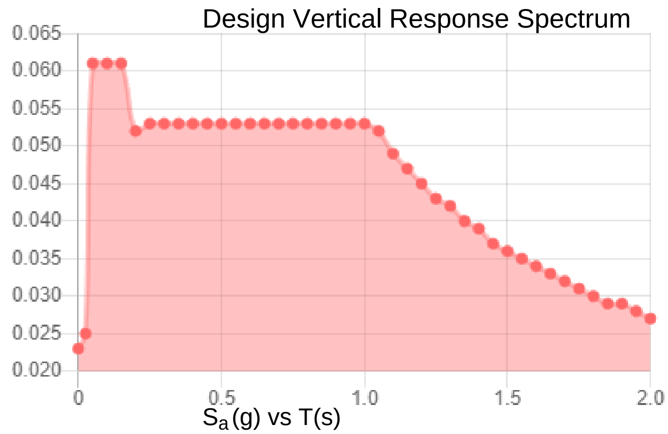
S_S :	0.099	S_{D1} :	0.109
S_1 :	0.068	T_L :	12
F_a :	1.6	PGA :	0.047
F_v :	2.4	PGA_M :	0.075
S_{MS} :	0.159	F_{PGA} :	1.6
S_{M1} :	0.163	I_e :	1
S_{DS} :	0.106	C_v :	0.7

Seismic Design Category: B









Data Accessed: **Thu Apr 25 2024**

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.

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Snow

Results:

Ground Snow Load, p_g :	20 lb/ft^2
Mapped Elevation:	998.3 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Thu Apr 25 2024

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.

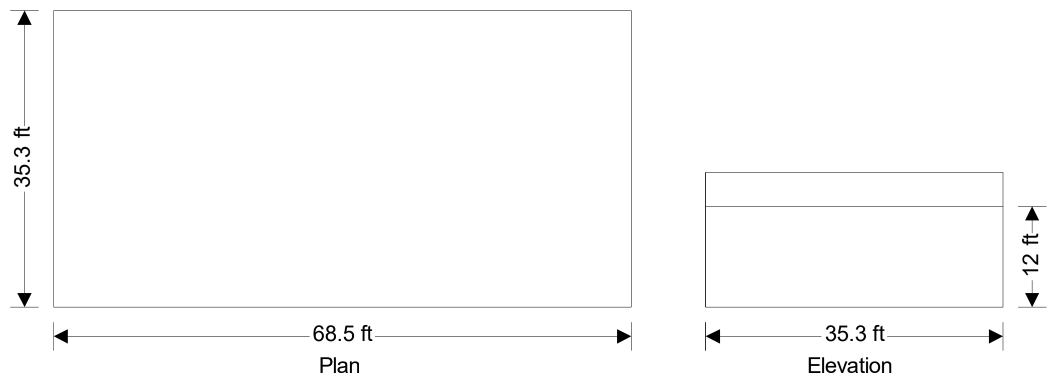
Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported ‘elevation’ and ‘mapped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

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.14



Building data

Type of roof	Flat
Length of building	b = **68.50** ft
Width of building	d = **35.25** ft
Height to eaves	H = **12.00** ft
Height of parapet	h_p = **4.00** ft
Mean height	h = **12.00** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.1.5)	p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
12.00	0.85	21.98
16.00	0.86	22.23

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **21.98** psf

Parapet pressures and forces

Velocity pressure at top of parapet — q_p = **22.23** psf

Combined net pressure coefficient, leeward — GC_{pnl} = **-1.0**

Combined net parapet pressure, leeward — $p_{pl} = q_p \times GC_{pnl}$ = **-22.23** psf

Combined net pressure coefficient, windward — GC_{pnw} = **1.5**

Combined net parapet pressure, windward — $p_{pw} = q_p \times GC_{pnw}$ = **33.35** psf

Wind direction 0 deg:

Leeward parapet force — $F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-6.1** kips

Windward parapet force — $F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **9.1** kips

Wind direction 90 deg:

Leeward parapet force — $F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-3.1** kips

Windward parapet force — $F_{w,wpw\_90} = p_{pw} \times h_p \times d$ = **4.7** kips

Pressures and forces

Net pressure — $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force — $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	12.00	-0.90	21.98	-20.77	411.00	-8.54
B (-ve)	12.00	-0.90	21.98	-20.77	411.00	-8.54
C (-ve)	12.00	-0.50	21.98	-13.29	822.00	-10.93
D (-ve)	12.00	-0.30	21.98	-9.56	770.63	-7.37

Total vertical net force — $F_{w,v}$ = **-35.37** kips

Total horizontal net force — $F_{w,h}$ = **0.00** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.00	0.80	21.98	10.99	822.00	9.03
B	12.00	-0.50	21.98	-13.29	822.00	-10.93
C	12.00	-0.70	21.98	-17.03	423.00	-7.20
D	12.00	-0.70	21.98	-17.03	423.00	-7.20

Overall loading

Projected vertical plan area of wall — $A_{vert\_w\_0} = b \times (H + h_p)$ = **1096.00** ft^2

Projected vertical area of roof — $A_{vert\_r\_0}$ = **0.00** ft^2

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **17.54** kips

Leeward net force — $F_l = F_{w,wB} + F_{w,wpl\_0}$ = **-17.0** kips

Windward net force — $F_w = F_{w,wA} + F_{w,wpw\_0}$ = **18.2** kips

Overall horizontal loading — $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **35.2** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	12.00	-0.18	21.98	0.59	411.00	0.24
B (+ve)	12.00	-0.18	21.98	0.59	411.00	0.24
C (+ve)	12.00	-0.18	21.98	0.59	822.00	0.49
D (+ve)	12.00	-0.18	21.98	0.59	770.63	0.46

Total vertical net force $F_{w,v}$ = **1.43** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.00	0.80	21.98	18.90	822.00	15.53
B	12.00	-0.50	21.98	-5.38	822.00	-4.43
C	12.00	-0.70	21.98	-9.12	423.00	-3.86
D	12.00	-0.70	21.98	-9.12	423.00	-3.86

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times (H + h_p)$ = **1096.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0}$ = **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **17.54** kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_0}$ = **-10.5** kips

Windward net force $F_w = F_{w,wA} + F_{w,wpw\_0}$ = **24.7** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **35.2** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	12.00	-0.90	21.98	-20.77	211.50	-4.39
B (-ve)	12.00	-0.90	21.98	-20.77	211.50	-4.39
C (-ve)	12.00	-0.50	21.98	-13.29	423.00	-5.62
D (-ve)	12.00	-0.30	21.98	-9.56	1568.63	-14.99

Total vertical net force $F_{w,v}$ = **-29.40** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.00	0.80	21.98	10.99	423.00	4.65
B	12.00	-0.31	21.98	-9.77	423.00	-4.13

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
C	12.00	-0.70	21.98	-17.03	822.00	-14.00
D	12.00	-0.70	21.98	-17.03	822.00	-14.00

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times (H + h_p) =$ **564.00** ft^2

Projected vertical area of roof $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **9.02** kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_90} =$ **-7.3** kips

Windward net force $F_w = F_{w,wA} + F_{w,wpw\_90} =$ **9.4** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **16.6** kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, +c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	12.00	-0.18	21.98	0.59	211.50	0.13
B (+ve)	12.00	-0.18	21.98	0.59	211.50	0.13
C (+ve)	12.00	-0.18	21.98	0.59	423.00	0.25
D (+ve)	12.00	-0.18	21.98	0.59	1568.63	0.93

Total vertical net force $F_{w,v} =$ **1.43** kips

Total horizontal net force $F_{w,h} =$ **0.00** kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, +c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.00	0.80	21.98	18.90	423.00	7.99
B	12.00	-0.31	21.98	-1.86	423.00	-0.79
C	12.00	-0.70	21.98	-9.12	822.00	-7.50
D	12.00	-0.70	21.98	-9.12	822.00	-7.50

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times (H + h_p) =$ **564.00** ft^2

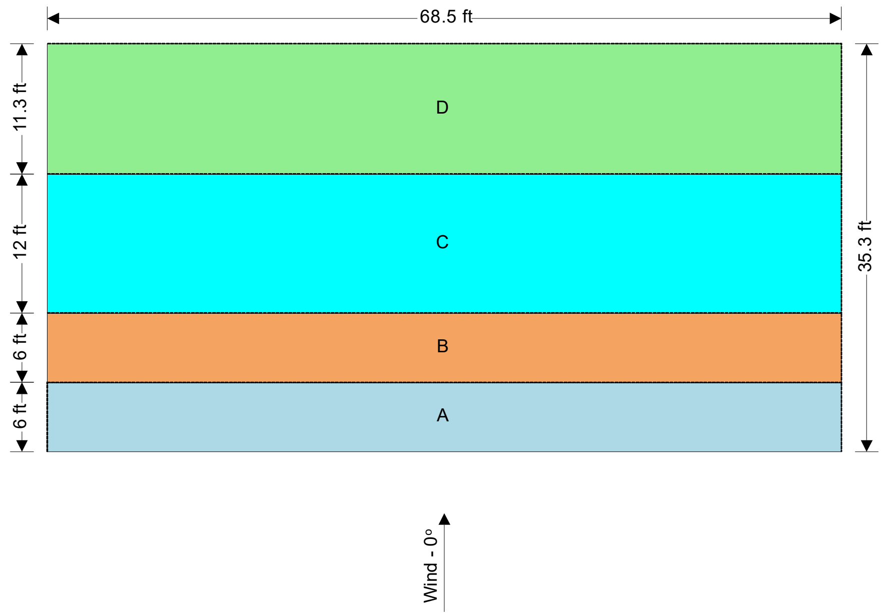
Projected vertical area of roof $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **9.02** kips

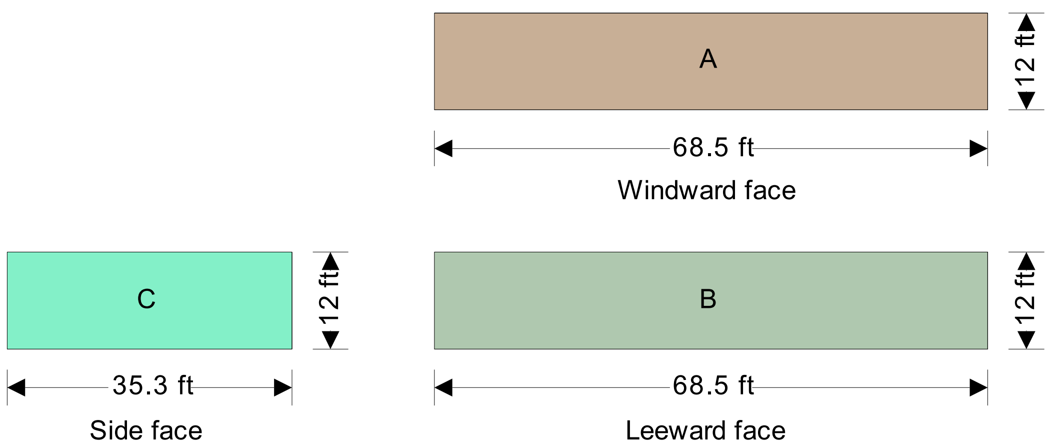
Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_90} =$ **-3.9** kips

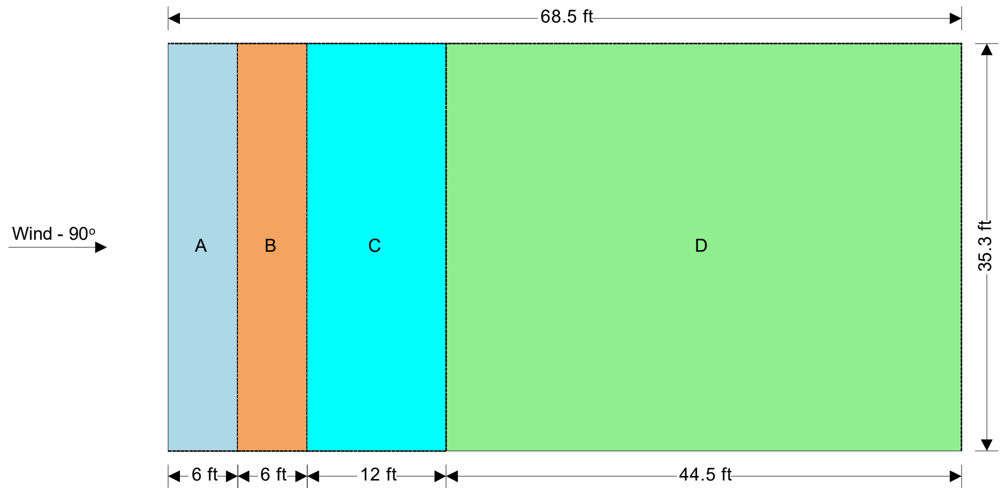
Windward net force $F_w = F_{w,wA} + F_{w,wpw\_90} =$ **12.7** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **16.6** kips

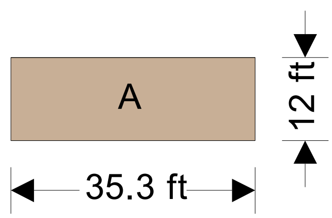


Plan view - Flat roof

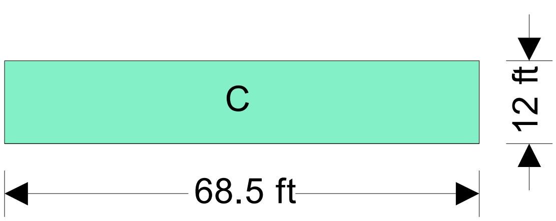




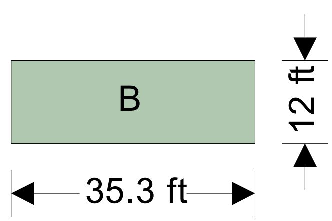
Plan view - Flat roof



Windward face



Side face



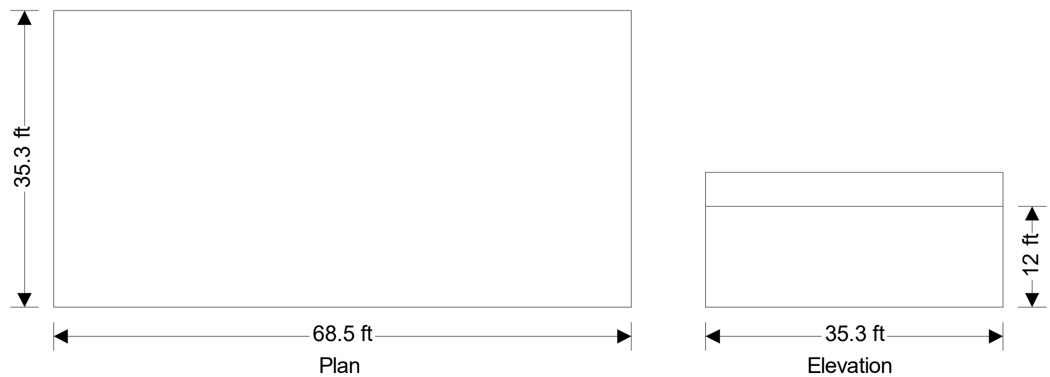
Leeward face

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.14



Building data

Type of roof	Flat
Length of building	b = **68.50** ft
Width of building	d = **35.25** ft
Height to eaves	H = **12.00** ft
Height of parapet	h_p = **4.00** ft
Mean height	h = **12.00** ft
End zone width	$a = \max(\min(0.1\times\min(b, d), 0.4\times h), 0.04\times\min(b, d), 3ft)$ = **3.53** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1ft)$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Parapet internal pressure coef +ve (Table 26.11-1)	GC_{pi_pp} = **0.18**
Parapet internal pressure coef –ve (Table 26.11-1)	GC_{pi_np} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1psf/mph^2$ = **22.0** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.86**

Velocity pressure $q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2 =$ **22.2** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_i =$ **21.98** psf

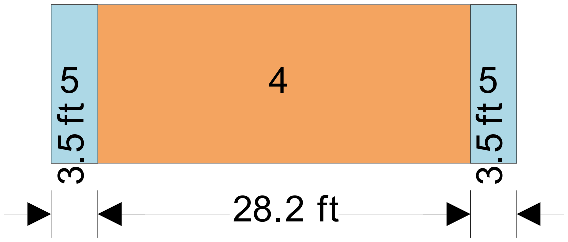
Equations used in tables

Net pressure $p = q_h \times [GC_p - GC_{pi}]$

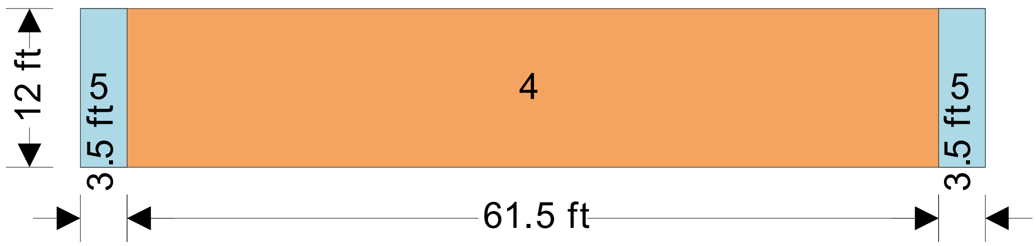
Parapet net pressure $p = q_p \text{ x } [GC_p - GC_{pi\_p}]$

Components and cladding pressures - Wall (Table 30.3-1 and (Figure 30.3-2A))

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	+GC_p	-GC_p	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	0.90	-0.99	23.7	-25.7
20 sf	4	-	-	20.0	0.85	-0.94	22.7	-24.7
50 sf	4	-	-	50.0	0.79	-0.88	21.3	-23.3
>100 sf	4	-	-	100.1	0.74	-0.83	20.2	-22.2
<=10 sf	5	-	-	10.0	0.90	-1.26	23.7	-31.6
20 sf	5	-	-	20.0	0.85	-1.16	22.7	-29.5
50 sf	5	-	-	50.0	0.79	-1.04	21.3	-26.8
>100 sf	5	-	-	100.1	0.74	-0.94	20.2	-24.7
10 sf (W)	4p	-	-	10.0	0.90	-2.30	24.0	-55.1
20 sf (W)	4p	-	-	20.0	0.85	-2.14	22.9	-51.6
50 sf (W)	4p	-	-	50.0	0.79	-1.93	21.5	-46.9
100 sf (W)	4p	-	-	100.0	0.74	-1.77	20.5	-43.4
10 sf (W)	5p	-	-	10.0	0.90	-2.30	24.0	-55.1
20 sf (W)	5p	-	-	20.0	0.85	-2.14	22.9	-51.6
50 sf (W)	5p	-	-	50.0	0.79	-1.93	21.5	-46.9
100 sf (W)	5p	-	-	100.0	0.74	-1.77	20.5	-43.4



Elevation of gable wall

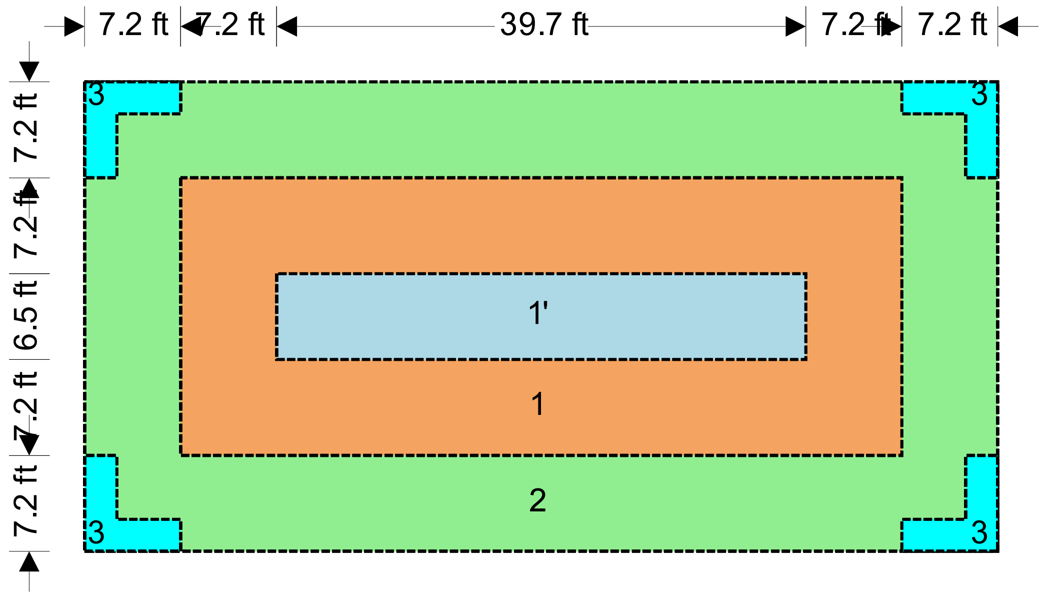


Elevation of side wall

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

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.30	-1.70	10.5 #	-41.3
20 sf	1	-	-	20.0	0.27	-1.58	9.9 #	-38.6
50 sf	1	-	-	50.0	0.23	-1.41	9.0 #	-35.0
>100 sf	1	-	-	100.1	0.20	-1.29	8.4 #	-32.3
<=10 sf	1'	-	-	10.0	0.30	-0.90	10.5 #	-23.7
20 sf	1'	-	-	20.0	0.27	-0.90	9.9 #	-23.7
50 sf	1'	-	-	50.0	0.23	-0.90	9.0 #	-23.7
>100 sf	1'	-	-	100.1	0.20	-0.90	8.4 #	-23.7
<=10 sf	2	-	-	10.0	0.90	-2.30	23.7	-54.5
20 sf	2	-	-	20.0	0.85	-2.14	22.7	-51.0
50 sf	2	-	-	50.0	0.79	-1.93	21.3	-46.4
>100 sf	2	-	-	100.1	0.74	-1.77	20.2	-42.9
<=10 sf	3	-	-	10.0	0.90	-2.30	23.7	-54.5
20 sf	3	-	-	20.0	0.85	-2.14	22.7	-51.0
50 sf	3	-	-	50.0	0.79	-1.93	21.3	-46.4
>100 sf	3	-	-	100.1	0.74	-1.77	20.2	-42.9

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



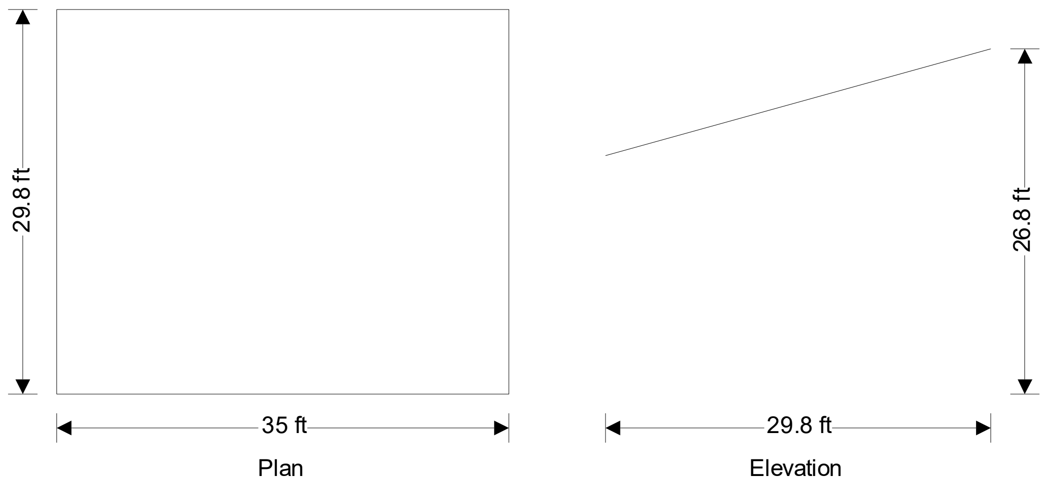
Plan on roof

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.14



Building data

Type of roof	Monoslope free
Length of building	b = **35.00** ft
Width of building	d = **29.83** ft
Height to eaves	H = **18.50** ft
Pitch of roof	α_0 = **15.5** deg
Mean height	h = **22.64** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **3.00** ft
Wind flow	Obstructed

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1ft)$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Open buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.00**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **0.00**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.92**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1psf/mph^2$ = **23.8** psf

Peak velocity pressure for internal pressure

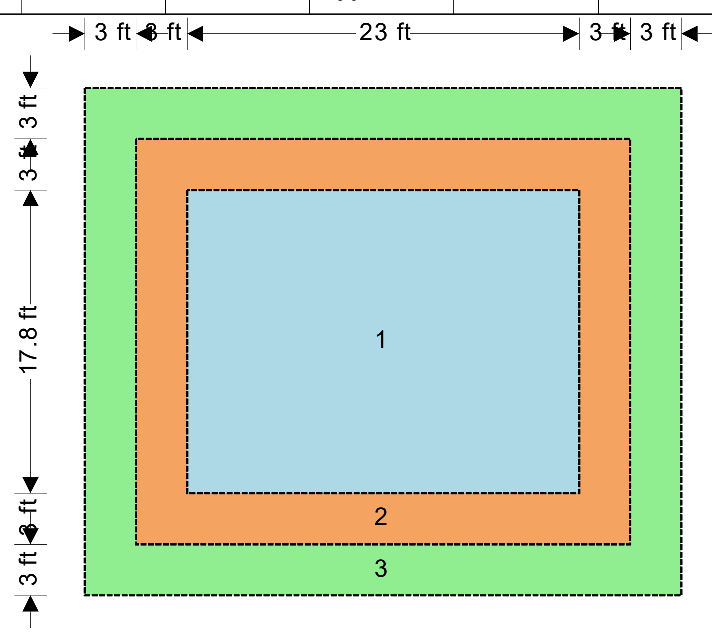
Peak velocity pressure – internal (as roof press.) q_i = **23.81** psf

Equations used in tables

Net pressure $p = q_h \times [GC_p - GC_{pi}]$

Components and cladding pressures - Roof (Figure 30.7-1)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft²)	+C_N	-C_N	Pres (+ve) (psf)	Pres (-ve) (psf)
<=9 sf	1	-	-	9.0	1.21	-2.11	24.6	-42.6
18 sf	1	-	-	18.0	1.21	-2.11	24.6	-42.6
>36 sf	1	-	-	36.1	1.21	-2.11	24.6	-42.6
<=9 sf	2	-	-	9.0	1.82	-3.21	36.8	-65.0
18 sf	2	-	-	18.0	1.82	-3.21	36.8	-65.0
>36 sf	2	-	-	36.1	1.21	-2.11	24.6	-42.6
<=9 sf	3	-	-	9.0	2.43	-4.21	49.1	-85.3
18 sf	3	-	-	18.0	1.82	-3.21	36.8	-65.0
>36 sf	3	-	-	36.1	1.21	-2.11	24.6	-42.6



Plan on roof

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.14

Wall/sign data

Length of wall/sign	B = **13.50** ft
Height of wall/sign	s = **8.00** ft
Height to top of sign	h = **8.00** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **1.00**
Exposure category (cl 26.7.3)	C
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.1.5)	p_{min_r} = **8** lb/ft^2

Topography

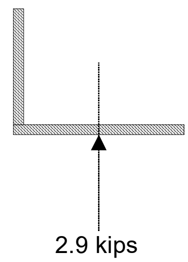
Topography factor not significant	K_{zt} = 1.0

Velocity pressure

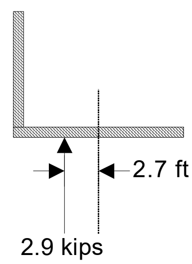
Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **22.0** psf
Area of sign	$A_f = B \times s$ = **108** ft^2
Ratio of solid area to gross area	ε = **1.00**

Wall/sign forces – Case A and B

Force coefficient (Figure 29.3-1)	C_{f_A} = **1.42**
Resultant force	$F_A = \max(16\text{psf}, q_h \times G_f \times C_{f\_A}) \times A_f$ = **2.9** kips



Plan - Case A



Plan - Case B

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.12

Building details

Roof type	Flat
Width of roof	$b = \mathbf{34.25}$ ft

Ground snow load

Ground snow load (Figure 7.2-1)	$p_g = \mathbf{20.00}$ lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3) = \mathbf{16.60}$ lb/ft^3
Surface roughness category (Sect. 26.7)	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	$C_e = \mathbf{1.00}$
Thermal condition (Table 7.3-2)	Structures kept just above freezing
Thermal factor (Table 7.3-2)	$C_t = \mathbf{1.10}$
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	$I_s = \mathbf{1.00}$
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g = \mathbf{20.00}$ lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = \mathbf{15.40}$ lb/ft^2

Left parapet

Balanced snow load height	$h_b = p_f / \gamma = \mathbf{0.93}$ ft
Height of left parapet	$h_{pptL} = \mathbf{4.00}$ ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b = \mathbf{3.07}$ ft
Length of roof - left parapet	$l_{u\_pptL} = b = \mathbf{34.25}$ ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}) = \mathbf{1.33}$ ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b) = \mathbf{1.33}$ ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b) = \mathbf{5.30}$ ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma = \mathbf{22.01}$ lb/ft^2

Right parapet

Height of right parapet	$h_{pptR} = \mathbf{4.00}$ ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b = \mathbf{3.07}$ ft
Length of roof - right parapet	$l_{u\_pptR} = b = \mathbf{34.25}$ ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}) = \mathbf{1.33}$ ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b) = \mathbf{1.33}$ ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b) = \mathbf{5.30}$ ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma = \mathbf{22.01}$ lb/ft^2

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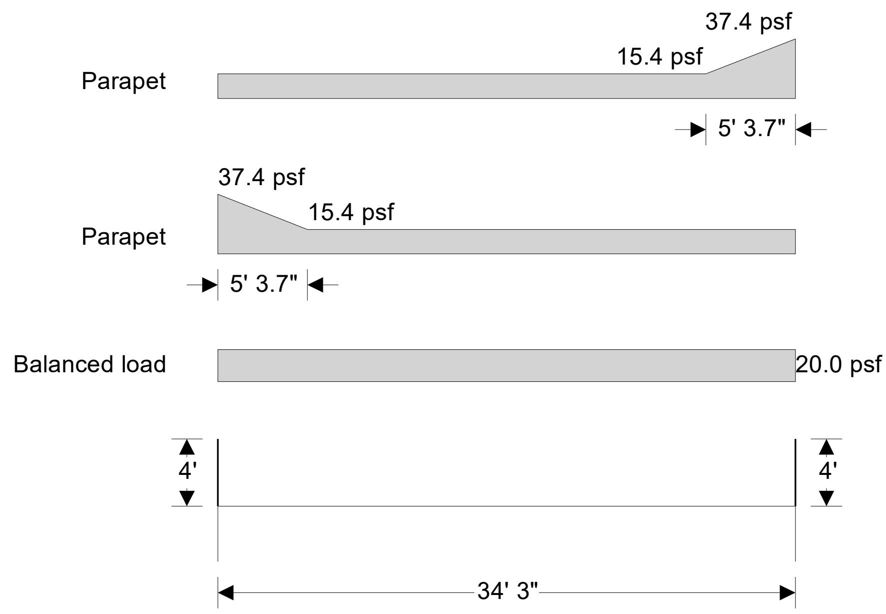
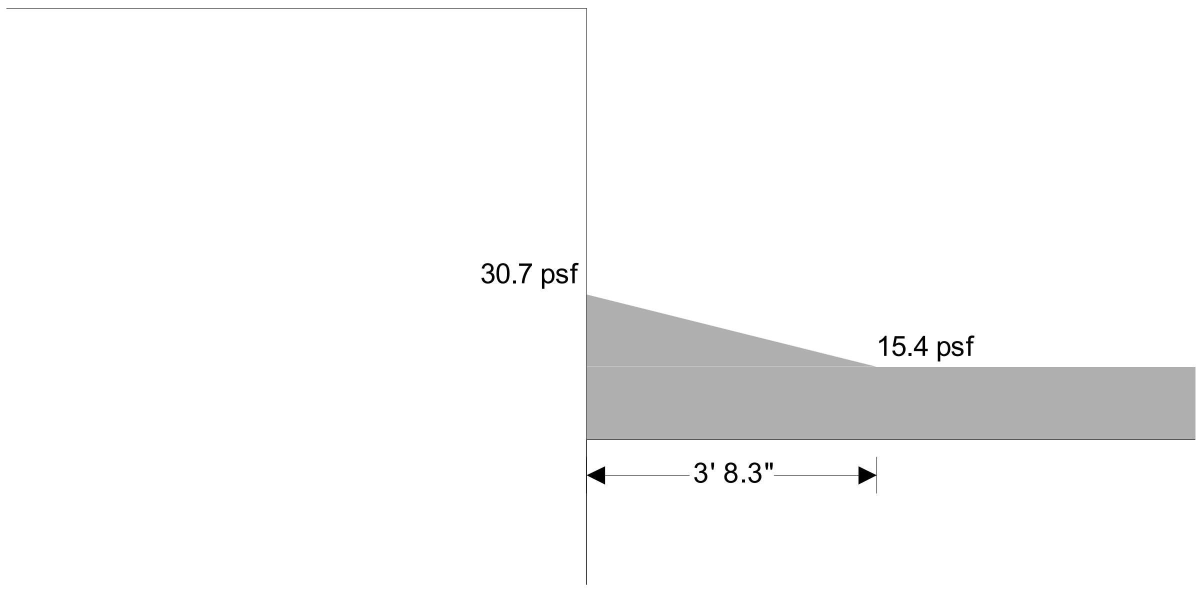


Roof elevation

Drift calculations

Balanced snow load height	$h_b = p_f / \gamma =$ **0.93** ft
Length of upper roof	$l_u =$ **0.50** ft
Length of lower roof	$l_l =$ **7.75** ft
Height diff between uppper and lower roofs	$h_{diff} =$ **5.50** ft
Height from balance load to top of upper roof	$h_c = h_{diff} - h_b =$ **4.57** ft
Drift height leeward drift	$h_{d\_l} = \min(\sqrt{(I_s)} \times (0.43 \times (\max(20\text{ ft}, l_u) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}), 0.6 \times l_l, \sqrt{(I_s \times p_g \times l_u / (4 \times \gamma))}) =$ **0.39** ft
Drift height windward drift	$h_{d\_w} = \min(0.75 \times \sqrt{(I_s)} \times (0.43 \times (\max(20\text{ ft}, l_l) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}), \sqrt{(I_s \times p_g \times l_l / (4 \times \gamma))}) =$ **0.92** ft
Maximum lw/ww drift height	$h_{d\_max} = \max(h_{d\_w}, h_{d\_l}) =$ **0.92** ft
Drift height	$h_d = \min(h_{d\_max}, h_c) =$ **0.92** ft
Drift width	$W_d = \min(4 \times h_{d\_max}, 8 \times h_c) =$ **3.69** ft
Drift surcharge load	$p_d = h_d \times \gamma =$ **15.33** lb/ft^2



Elevation on snow drift

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.10

Building details

Roof type	Flat
Width of roof	b = **10.25** ft

Ground snow load

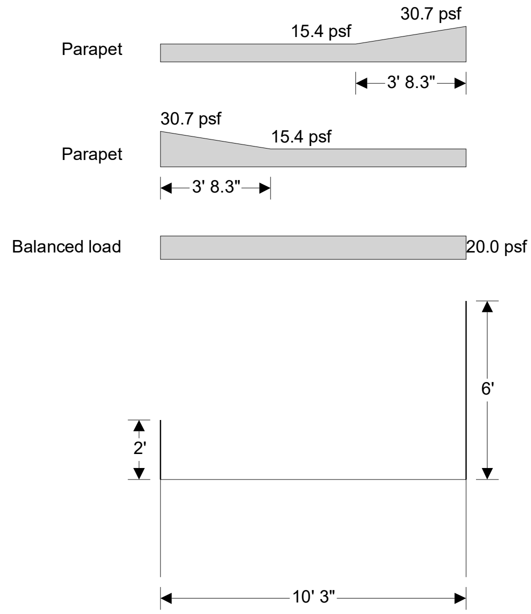
Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	γ = min(0.13 × p_g / 1ft + 14lb/ft^3, 30lb/ft^3) = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal condition (Table 7.3-2)	Structures kept just above freezing
Thermal factor (Table 7.3-2)	C_t = **1.10**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	p_{f_min} = I_s × p_g = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3)	p_f = 0.7 × C_e × C_t × I_s × p_g = **15.40** lb/ft^2

Left parapet

Balanced snow load height	h_b = p_f / γ = **0.93** ft
Height of left parapet	h_{pptL} = **2.00** ft
Height from balance load to top of left parapet	h_{c_pptL} = h_{pptL} - h_b = **1.07** ft
Length of roof - left parapet	l_{u_pptL} = b = **10.25** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL}$ = min($\sqrt{(I_s)}$ × 0.75 × (0.43 × (max(20 ft, l_{u_pptL}) × 1ft^2)$^{1/3}$ × (p_g / 1lb/ft^2 + 10)$^{1/4}$ - 1.5ft), $\sqrt{(I_s \times p_g \times l_{u\_pptL} / (4 \times \gamma))}$) = **0.92** ft
Drift height - left parapet	h_{d_pptL} = min($h_{d\_l\_pptL}$, h_{pptL} - h_b) = **0.92** ft
Drift width	W_{d_pptL} = min(4 × $h_{d\_l\_pptL}$, 8 × (h_{pptL} - h_b), b) = **3.69** ft
Drift surcharge load - left parapet	p_{d_pptL} = h_{d_pptL} × γ = **15.33** lb/ft^2

Right parapet

Height of right parapet	h_{pptR} = **6.00** ft
Height from balance load to top of right parapet	h_{c_pptR} = h_{pptR} - h_b = **5.07** ft
Length of roof - right parapet	l_{u_pptR} = b = **10.25** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR}$ = min($\sqrt{(I_s)}$ × 0.75 × (0.43 × (max(20 ft, l_{u_pptR}) × 1ft^2)$^{1/3}$ × (p_g / 1lb/ft^2 + 10)$^{1/4}$ - 1.5ft), $\sqrt{(I_s \times p_g \times l_{u\_pptR} / (4 \times \gamma))}$) = **0.92** ft
Drift height - right parapet	h_{d_pptR} = min($h_{d\_l\_pptR}$, h_{pptR} - h_b) = **0.92** ft
Drift width	W_{d_pptR} = min(4 × $h_{d\_l\_pptR}$, 8 × (h_{pptR} - h_b), b) = **3.69** ft
Drift surcharge load - right parapet	p_{d_pptR} = h_{d_pptR} × γ = **15.33** lb/ft^2



Roof elevation

SNOW LOADING

In accordance with ASCE7-16

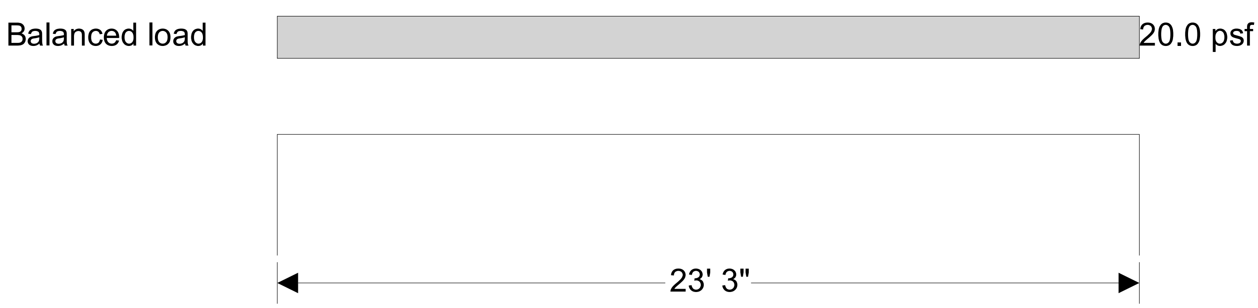
Tedds calculation version 1.0.10

Building details

Roof type	Flat
Width of roof	b = **23.25** ft

Ground snow load

Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1ft + 14lb/ft^3, 30lb/ft^3)$ = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal condition (Table 7.3-2)	Unheated structures
Thermal factor (Table 7.3-2)	C_t = **1.20**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g$ = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **16.80** lb/ft^2



Roof elevation

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SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.10

Building details

Roof type	Monopitch
Width of roof	b = **23.25** ft
Slope of roof 1	α = **20.00** deg

Ground snow load

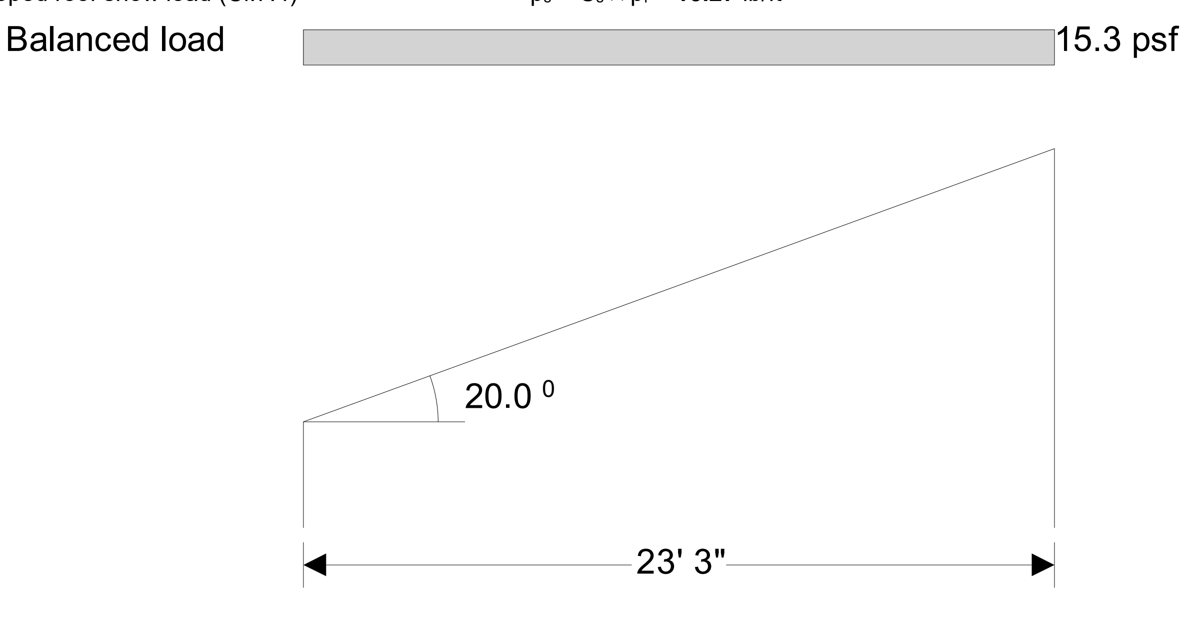
Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal condition (Table 7.3-2)	Unheated structures
Thermal factor (Table 7.3-2)	C_t = **1.20**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **16.80** lb/ft^2

Cold roof slope factor (C_t > 1.0)

Roof surface type	Slippery
Ventilation	Ventilated
Thermal resistance (R-value)	R = **30.00** °F h ft^2 / Btu
Roof slope factor Fig 7.4-1c (dashed line)	C_s = **0.91**

Monoslope

Sloped roof snow load (Cl.7.4)	$p_s = C_s \times p_f$ = **15.27** lb/ft^2



Roof elevation

SEISMIC FORCES

In accordance with ASCE 7-16

Tedds calculation version 3.1.04

Site parameters

Site class	D
Mapped acceleration parameters (Section 11.4.2)	
at short period	S_S = **0.099**
at 1 sec period	S_1 = **0.068**
Site coefficientat short period (Table 11.4-1)	F_a = **1.600**
at 1 sec period (Table 11.4-2)	F_v = **2.400**

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S$ = **0.158**
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1$ = **0.163**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS}$ = **0.106**
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1}$ = **0.109**

Seismic design category

Occupancy category (Table 1-1) II

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

A

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

B

Seismic design category B

Approximate fundamental period

Height above base to highest level of building h_n = **12** ft

From Table 12.8-2:

Structure type	All other systems
Building period parameter C_t	C_t = **0.02**
Building period parameter x	x = **0.75**

Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$ = **0.129** sec
Building fundamental period (Sect 12.8.2)	$T = T_a$ = **0.129** sec
Long-period transition period	T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	A. Bearing_Wall_Systems
	15. Light-frame (wood) walls sheathed with wood structural panels
Response modification factor (Table 12.2-1)	R = **6.5**
Seismic importance factor (Table 1.5-2)	I_e = **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2)	$C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.0162**
Maximum (Eq 12.8-3)	$C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e))$ = **0.1298**

Minimum (Eq.12.8-5) $C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) = \mathbf{0.0100}$

Seismic response coefficient $C_s = \mathbf{0.0162}$

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure W = **108.9** kips

Seismic response coefficient $C_s = \mathbf{0.0162}$

Seismic base shear (Eq 12.8-1) $V = C_s \times W = \mathbf{1.8}$ kips

SEISMIC FORCES

In accordance with ASCE 7-16

Tedds calculation version 3.1.04

Site parameters

Site class D

Mapped acceleration parameters (Section 11.4.2)

at short period S_S = **0.099**

at 1 sec period S_1 = **0.068**

Site coefficientat short period (Table 11.4-1) F_a = **1.600**

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

Spectral response acceleration parameters

at short period (Eq. 11.4-1) $S_{MS} = F_a \times S_S$ = **0.158**

at 1 sec period (Eq. 11.4-2) $S_{M1} = F_v \times S_1$ = **0.163**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) $S_{DS} = 2 / 3 \times S_{MS}$ = **0.106**

at 1 sec period (Eq. 11.4-4) $S_{D1} = 2 / 3 \times S_{M1}$ = **0.109**

Seismic design category

Occupancy category (Table 1-1) II

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

A

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

B

Seismic design category B

Approximate fundamental period

Height above base to highest level of building h_n = **27** ft

From Table 12.8-2:

Structure type All other systems

Building period parameter C_t C_t = **0.02**

Building period parameter x x = **0.75**

Approximate fundamental period (Eq 12.8-7) $T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$= **0.237** sec

Building fundamental period (Sect 12.8.2) $T = T_a$ = **0.237** sec

Long-period transition period T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1) G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIRE

2. Steel ordinary cantilever column systems

Response modification factor (Table 12.2-1) R = **1.25**

Seismic importance factor (Table 1.5-2) I_e = **1.000**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2) $C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.0845**

Maximum (Eq 12.8-3)	$C_{s\_max} = S_{D1} / ((T / 1 \text{ sec}) \times (R / I_e)) =$ **0.3674**
Minimum (Eq.12.8-5)	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) =$ **0.0100**
Seismic response coefficient	$C_s =$ **0.0845**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure	$W =$ **26.1** kips
Seismic response coefficient	$C_s =$ **0.0845**
Seismic base shear (Eq 12.8-1)	$V = C_s \times W =$ **2.2** kips

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

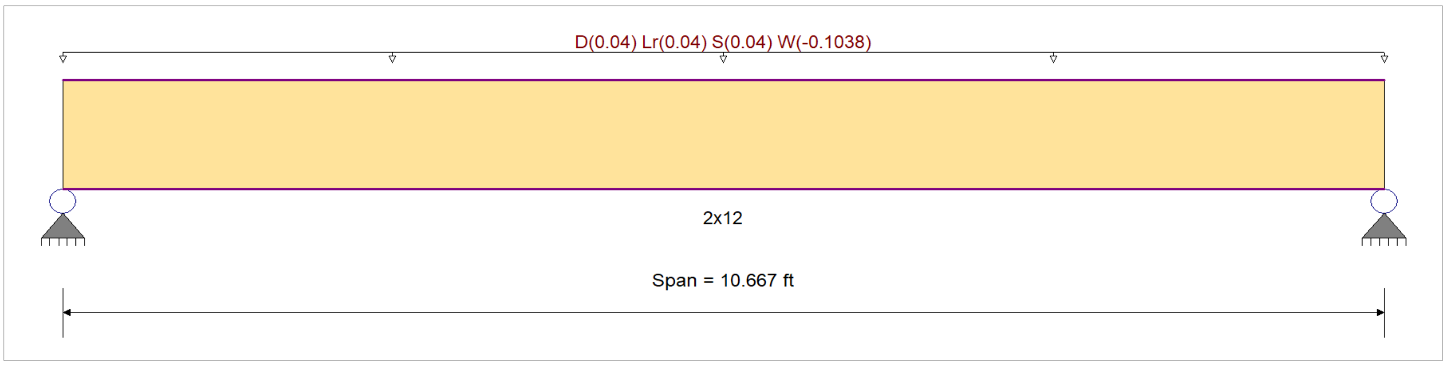
DESCRIPTION: Roof Joist - Freezer / Low Roof

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05190 ksf, Tributary Width = 2.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.417** : 1	Maximum Shear Stress Ratio	=	**0.151** : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	431.54 psi	fv: Actual	=	31.28 psi
F'b	=	1,035.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.334 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.041 in	Ratio =	3110	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.107 in	Ratio =	1198	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.082 in	Ratio =	1555	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.039 in	Ratio =	3249	>=180	Span: 1 : +0.60D+0.60W

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.266	0.097	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.57	215.8	810.0	0.18	15.6	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.384	0.139	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.14	431.5	1,125.0	0.35	31.3	225.0
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.417	0.151	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.14	431.5	1,035.0	0.35	31.3	207.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.336	0.122	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.00	377.6	1,125.0	0.31	27.4	225.0
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.365	0.132	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.00	377.6	1,035.0	0.31	27.4	207.0

Wood Beam

DESCRIPTION: Roof Joist - Freezer / Low Roof

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.083	0.030	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.32	120.2	1,440.0	0.10	8.7	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.087	0.032	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.33	125.6	1,440.0	0.10	9.1	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.087	0.032	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.33	125.6	1,440.0	0.10	9.1	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.143	0.052	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.54	206.5	1,440.0	0.17	15.0	288.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.090	0.033	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.34	129.5	1,440.0	0.11	9.4	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.1068	5.372

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.427	0.427
Max Upward from Load Combinations	0.427	0.427
Max Upward from Load Cases	0.213	0.213
Max Downward from all Load Conditio	-0.554	-0.554
Max Downward from Load Combinations	-0.204	-0.204
Max Downward from Load Cases (Resis	-0.554	-0.554
D Only	0.213	0.213
+D+Lr	0.427	0.427
+D+S	0.427	0.427
+D+0.750Lr	0.373	0.373
+D+0.750S	0.373	0.373
+D+0.60W	-0.119	-0.119
+D+0.750Lr+0.450W	0.124	0.124
+D+0.750S+0.450W	0.124	0.124
+0.60D+0.60W	-0.204	-0.204
+0.60D	0.128	0.128
Lr Only	0.213	0.213
S Only	0.213	0.213
W Only	-0.554	-0.554

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

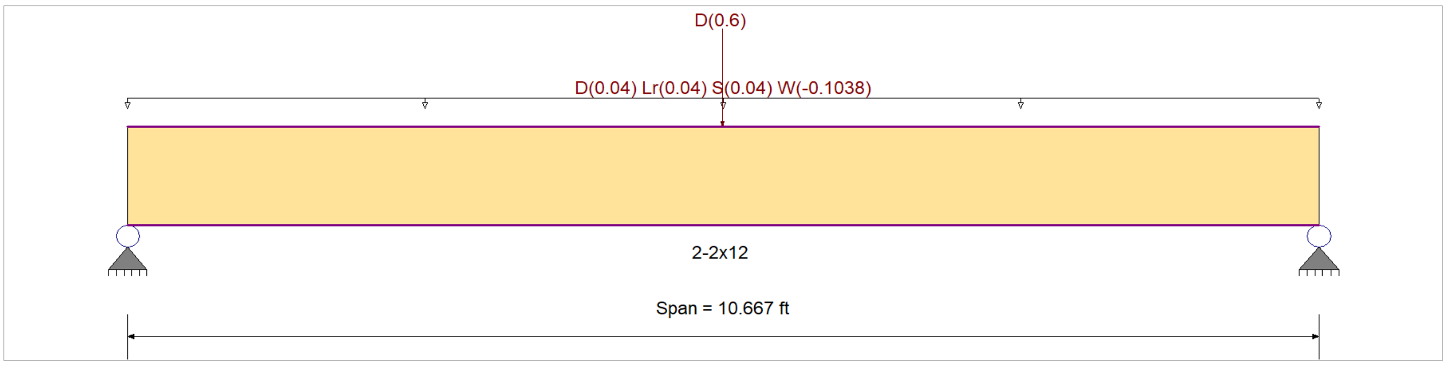
DESCRIPTION: Roof Joist - Freezer / Low Roof w/ RTU Load

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*		
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi	
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi	
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi			
Wood Grade :	No.2	Fv	180.0 psi			
		Ft	575.0 psi	Density	31.210 pcf	
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling					



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05190 ksf, Tributary Width = 2.0 ft, (Roof)
Point Load : D = 0.60 k @ 5.330 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.508** : 1	Maximum Shear Stress Ratio	=	**0.140** : 1
Section used for this span		**2-2x12**	Section used for this span		**2-2x12**
fb: Actual	=	411.10 psi	fv: Actual	=	28.98 psi
F'b	=	810.00 psi	F'v	=	207.00 psi
Load Combination		D Only	Load Combination		+D+S
Location of maximum on span	=	5.334 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.021 in	Ratio =	6220	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.053 in	Ratio =	2396	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.087 in	Ratio =	1463	>=180	Span: 1 : +D+Lr
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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 10.667 ft	1	0.508	0.131	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.17	411.1	810.0	0.48	21.2	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	1	0.461	0.129	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.74	519.0	1,125.0	0.65	29.0	225.0
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	1	0.501	0.140	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.74	519.0	1,035.0	0.65	29.0	207.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	1	0.437	0.120	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.59	492.0	1,125.0	0.61	27.0	225.0
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Roof Joist - Freezer / Low Roof w/ RTU Load

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 10.667 ft	**1**	0.475	0.131	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.59	492.0	1,035.0	0.61	27.0	207.0
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.169	0.046	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.28	243.1	1,440.0	0.30	13.3	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.254	0.062	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.93	366.0	1,440.0	0.40	17.9	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.254	0.062	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.93	366.0	1,440.0	0.40	17.9	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.055	0.028	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.41	78.7	1,440.0	0.18	8.0	288.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.667 ft	**1**	0.171	0.044	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.30	246.7	1,440.0	0.29	12.7	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0874	5.334		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 Conditions	0.727	0.726
Max Upward from Load Combinations	0.727	0.726
Max Upward from Load Cases	0.514	0.513
Max Downward from all Load Conditio	-0.554	-0.554
Max Downward from Load Combinations	-0.024	-0.024
Max Downward from Load Cases (Resis	-0.554	-0.554
D Only	0.514	0.513
+D+Lr	0.727	0.726
+D+S	0.727	0.726
+D+0.750Lr	0.674	0.673
+D+0.750S	0.674	0.673
+D+0.60W	0.181	0.181
+D+0.750Lr+0.450W	0.424	0.424
+D+0.750S+0.450W	0.424	0.424
+0.60D+0.60W	-0.024	-0.024
+0.60D	0.308	0.308
Lr Only	0.213	0.213
S Only	0.213	0.213
W Only	-0.554	-0.554

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

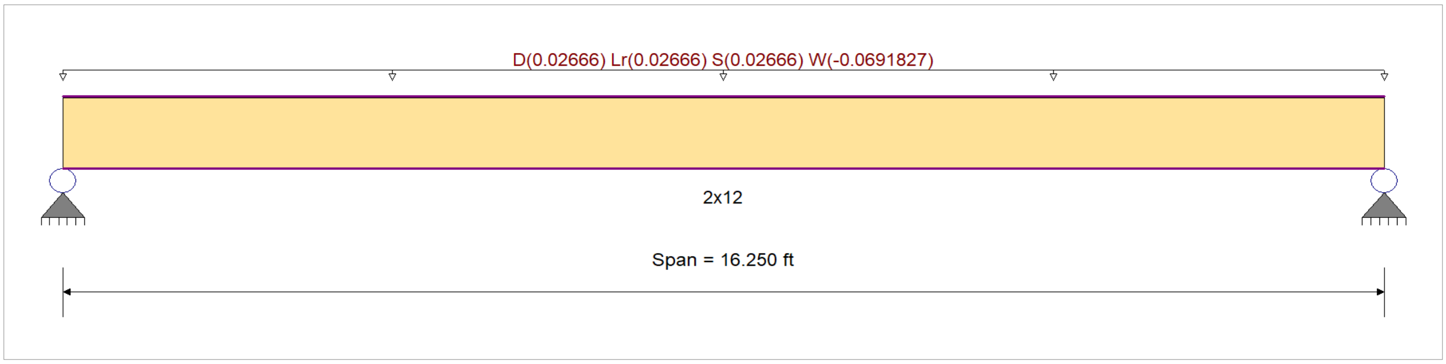
DESCRIPTION: Roof Joist - Center / Back of House

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05190 ksf, Tributary Width = 1.333 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.645** : 1	Maximum Shear Stress Ratio	=	**0.166** : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	667.49 psi	fv: Actual	=	34.29 psi
F'b	=	1,035.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.125 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.148 in	Ratio =	1319	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.383 in	Ratio =	508	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.295 in	Ratio =	659	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.141 in	Ratio =	1379	>=180	Span: 1 : +0.60D+0.60W

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.412	0.106	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.88	333.7	810.0	0.19	17.1	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.593	0.152	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.76	667.5	1,125.0	0.39	34.3	225.0
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.645	0.166	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.76	667.5	1,035.0	0.39	34.3	207.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.519	0.133	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.54	584.1	1,125.0	0.34	30.0	225.0
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.564	0.145	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.54	584.1	1,035.0	0.34	30.0	207.0

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Roof Joist - Center / Back of House

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.129	0.033	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.49	185.9	1,440.0	0.11	9.6	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.135	0.035	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	194.3	1,440.0	0.11	10.0	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.135	0.035	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	194.3	1,440.0	0.11	10.0	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.222	0.057	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.84	319.4	1,440.0	0.18	16.4	288.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.250 ft	**1**	0.139	0.036	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.53	200.2	1,440.0	0.12	10.3	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.3834	8.184

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.433	0.433
Max Upward from Load Combinations	0.433	0.433
Max Upward from Load Cases	0.217	0.217
Max Downward from all Load Conditio	-0.562	-0.562
Max Downward from Load Combinations	-0.207	-0.207
Max Downward from Load Cases (Resis	-0.562	-0.562
D Only	0.217	0.217
+D+Lr	0.433	0.433
+D+S	0.433	0.433
+D+0.750Lr	0.379	0.379
+D+0.750S	0.379	0.379
+D+0.60W	-0.121	-0.121
+D+0.750Lr+0.450W	0.126	0.126
+D+0.750S+0.450W	0.126	0.126
+0.60D+0.60W	-0.207	-0.207
+0.60D	0.130	0.130
Lr Only	0.217	0.217
S Only	0.217	0.217
W Only	-0.562	-0.562

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

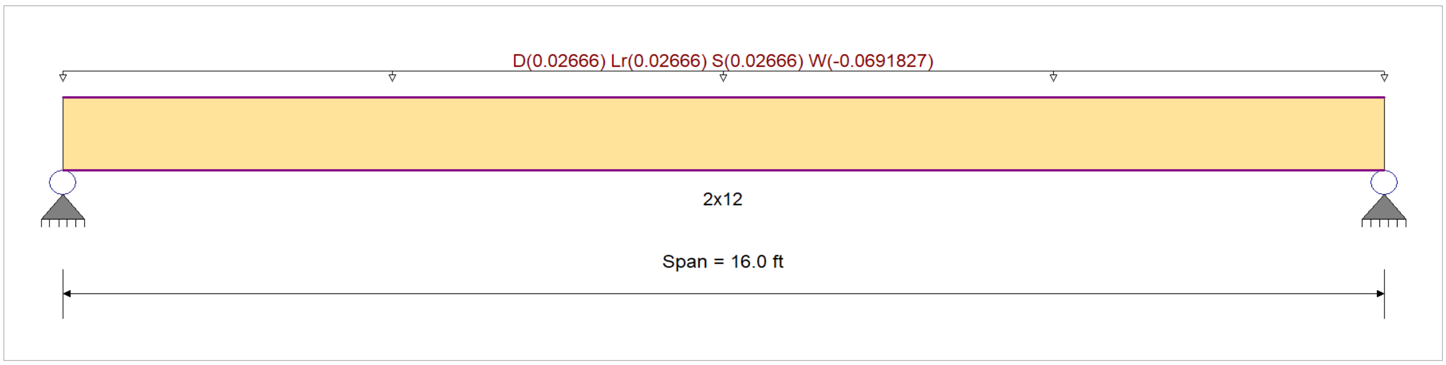
DESCRIPTION: Roof Joist - Office

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05190 ksf, Tributary Width = 1.333 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.625 : 1	Maximum Shear Stress Ratio	=	0.162 : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	647.11 psi	fv: Actual	=	33.49 psi
F'b	=	1,035.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	15.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.139 in	Ratio =	1382	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.360 in	Ratio =	532	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.278 in	Ratio =	691	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.133 in	Ratio =	1444	>=180	Span: 1 : +0.60D+0.60W

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.399	0.103	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.85	323.6	810.0	0.19	16.7	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.575	0.149	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.71	647.1	1,125.0	0.38	33.5	225.0
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.625	0.162	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.71	647.1	1,035.0	0.38	33.5	207.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.503	0.130	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.49	566.2	1,125.0	0.33	29.3	225.0
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.547	0.142	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.49	566.2	1,035.0	0.33	29.3	207.0

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Roof Joist - Office

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.125	0.032	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.48	180.2	1,440.0	0.10	9.3	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.131	0.034	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.50	188.4	1,440.0	0.11	9.7	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.131	0.034	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.50	188.4	1,440.0	0.11	9.7	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.215	0.056	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.82	309.6	1,440.0	0.18	16.0	288.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.135	0.035	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	194.1	1,440.0	0.11	10.0	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.3603	8.058

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.427	0.427
Max Upward from Load Combinations	0.427	0.427
Max Upward from Load Cases	0.213	0.213
Max Downward from all Load Conditio	-0.553	-0.553
Max Downward from Load Combinations	-0.204	-0.204
Max Downward from Load Cases (Resis	-0.553	-0.553
D Only	0.213	0.213
+D+Lr	0.427	0.427
+D+S	0.427	0.427
+D+0.750Lr	0.373	0.373
+D+0.750S	0.373	0.373
+D+0.60W	-0.119	-0.119
+D+0.750Lr+0.450W	0.124	0.124
+D+0.750S+0.450W	0.124	0.124
+0.60D+0.60W	-0.204	-0.204
+0.60D	0.128	0.128
Lr Only	0.213	0.213
S Only	0.213	0.213
W Only	-0.553	-0.553

Wood Beam

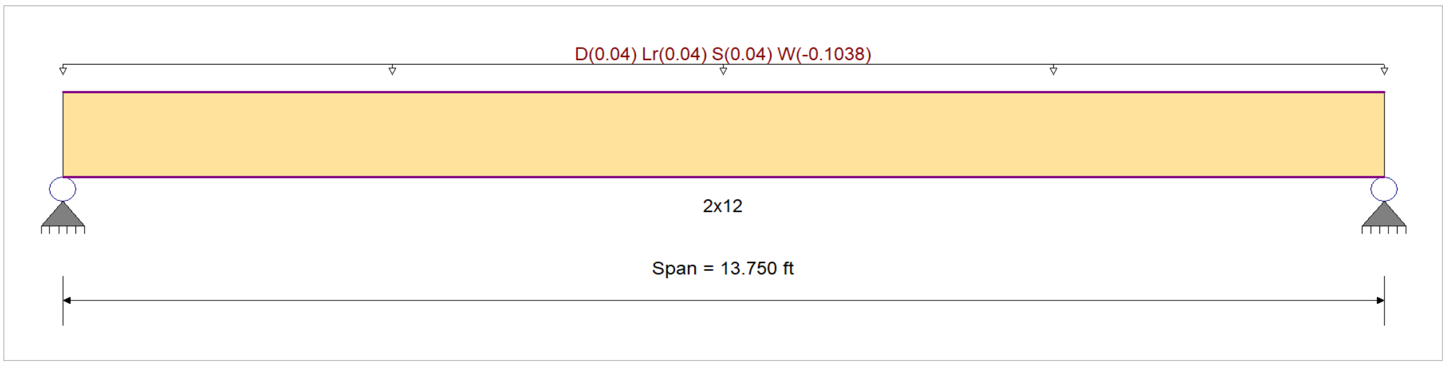
DESCRIPTION: Roof Joist - Left / Baking

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05190 ksf, Tributary Width = 2.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.693 : 1	Maximum Shear Stress Ratio	=	0.205 : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	717.04 psi	fv: Actual	=	42.47 psi
F'b	=	1,035.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.875 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.114 in	Ratio =	1452	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.295 in	Ratio =	559	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.227 in	Ratio =	726	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.109 in	Ratio =	1517	>=180	Span: 1 : +0.60D+0.60W

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.443	0.131	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.95	358.5	810.0	0.24	21.2	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.637	0.189	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.89	717.0	1,125.0	0.48	42.5	225.0
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.693	0.205	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.89	717.0	1,035.0	0.48	42.5	207.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.558	0.165	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.65	627.4	1,125.0	0.42	37.2	225.0
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.606	0.180	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.65	627.4	1,035.0	0.42	37.2	207.0

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Roof Joist - Left / Baking

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.139	0.041	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.53	199.7	1,440.0	0.13	11.8	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.145	0.043	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.55	208.7	1,440.0	0.14	12.4	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.145	0.043	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.55	208.7	1,440.0	0.14	12.4	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.238	0.071	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.90	343.1	1,440.0	0.23	20.3	288.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.750 ft	**1**	0.149	0.044	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.57	215.1	1,440.0	0.14	12.7	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.2949	6.925

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.550	0.550
Max Upward from Load Combinations	0.550	0.550
Max Upward from Load Cases	0.275	0.275
Max Downward from all Load Conditio	-0.714	-0.714
Max Downward from Load Combinations	-0.263	-0.263
Max Downward from Load Cases (Resis	-0.714	-0.714
D Only	0.275	0.275
+D+Lr	0.550	0.550
+D+S	0.550	0.550
+D+0.750Lr	0.481	0.481
+D+0.750S	0.481	0.481
+D+0.60W	-0.153	-0.153
+D+0.750Lr+0.450W	0.160	0.160
+D+0.750S+0.450W	0.160	0.160
+0.60D+0.60W	-0.263	-0.263
+0.60D	0.165	0.165
Lr Only	0.275	0.275
S Only	0.275	0.275
W Only	-0.714	-0.714

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

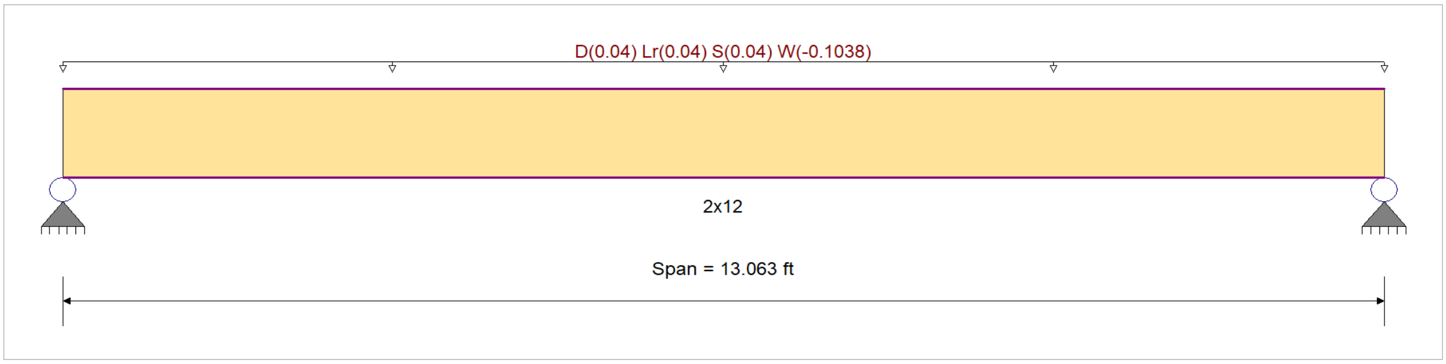
DESCRIPTION: Roof Joist - Front of House

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05190 ksf, Tributary Width = 2.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.625** : 1	Maximum Shear Stress Ratio	=	**0.193** : 1
Section used for this span		**2x12**	Section used for this span		**2x12**
fb: Actual	=	647.13 psi	fv: Actual	=	40.00 psi
F'b	=	1,035.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.531 ft	Location of maximum on span	=	12.157 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.093 in	Ratio =	1693	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.240 in	Ratio =	652	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.185 in	Ratio =	846	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.089 in	Ratio =	1769	>=180	Span: 1 : +0.60D+0.60W

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 13.063 ft	1	0.399	0.123	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.85	323.6	810.0	0.23	20.0	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.063 ft	1	0.575	0.178	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.71	647.1	1,125.0	0.45	40.0	225.0
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.063 ft	1	0.625	0.193	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.71	647.1	1,035.0	0.45	40.0	207.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.063 ft	1	0.503	0.156	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.49	566.2	1,125.0	0.39	35.0	225.0
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.063 ft	1	0.547	0.169	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.49	566.2	1,035.0	0.39	35.0	207.0

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Roof Joist - Front of House

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.063 ft	**1**	0.125	0.039	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.48	180.2	1,440.0	0.13	11.1	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.063 ft	**1**	0.131	0.040	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.50	188.4	1,440.0	0.13	11.6	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.063 ft	**1**	0.131	0.040	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.50	188.4	1,440.0	0.13	11.6	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.063 ft	**1**	0.215	0.066	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.82	309.6	1,440.0	0.22	19.1	288.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.063 ft	**1**	0.135	0.042	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	194.1	1,440.0	0.14	12.0	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.2402	6.579

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.523	0.523
Max Upward from Load Combinations	0.523	0.523
Max Upward from Load Cases	0.261	0.261
Max Downward from all Load Conditio	-0.678	-0.678
Max Downward from Load Combinations	-0.250	-0.250
Max Downward from Load Cases (Resis	-0.678	-0.678
D Only	0.261	0.261
+D+Lr	0.523	0.523
+D+S	0.523	0.523
+D+0.750Lr	0.457	0.457
+D+0.750S	0.457	0.457
+D+0.60W	-0.146	-0.146
+D+0.750Lr+0.450W	0.152	0.152
+D+0.750S+0.450W	0.152	0.152
+0.60D+0.60W	-0.250	-0.250
+0.60D	0.157	0.157
Lr Only	0.261	0.261
S Only	0.261	0.261
W Only	-0.678	-0.678

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

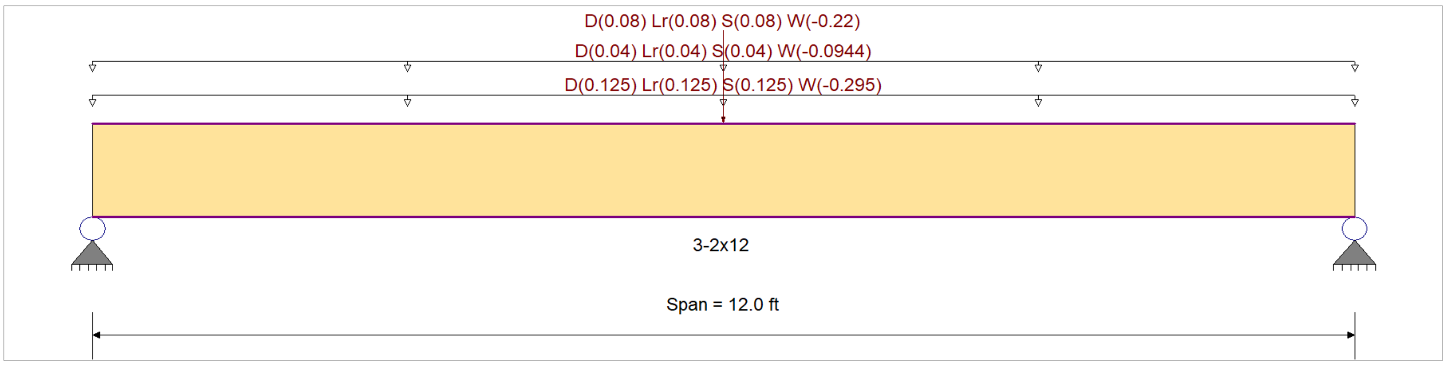
DESCRIPTION: Upset Beam @ RTU Opening - Perpendicular to Joist

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.04720 ksf, Tributary Width = 6.250 ft, (Roof Left)
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.04720 ksf, Tributary Width = 2.0 ft, (Roof Right)
Point Load : D = 0.080, Lr = 0.080, S = 0.080, W = -0.220 k @ 6.0 ft, (Upset Beam)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.784** : 1	Maximum Shear Stress Ratio	=	**0.251** : 1
Section used for this span		**3-2x12**	Section used for this span		**3-2x12**
fb: Actual	=	811.61 psi	fv: Actual	=	52.04 psi
F'b	=	1,035.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	6.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.096 in	Ratio =	1492 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.230 in	Ratio =	626 >=360	Span: 1 : W Only
Max Downward Total Deflection	0.193 in	Ratio =	746 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.080 in	Ratio =	1797 >=180	Span: 1 : +0.60D+0.60W

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 12.0 ft	1	0.501	0.161	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.21	405.8	810.0	0.88	26.0	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.0 ft	1	0.721	0.231	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.42	811.6	1,125.0	1.76	52.0	225.0
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.0 ft	1	0.784	0.251	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.42	811.6	1,035.0	1.76	52.0	207.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.0 ft	1	0.631	0.202	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.62	710.2	1,125.0	1.54	45.5	225.0
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Upset Beam @ RTU Opening - Perpendicular to Joist

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 12.0 ft	**1**	0.686	0.220	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.62	710.2	1,035.0	1.54	45.5	207.0
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.0 ft	**1**	0.122	0.039	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.39	175.9	1,440.0	0.37	11.1	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.0 ft	**1**	0.190	0.061	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.17	273.9	1,440.0	0.60	17.7	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.0 ft	**1**	0.190	0.061	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.17	273.9	1,440.0	0.60	17.7	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.0 ft	**1**	0.235	0.075	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.68	338.2	1,440.0	0.73	21.5	288.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.0 ft	**1**	0.169	0.054	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.93	243.5	1,440.0	0.53	15.6	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.2300	6.044

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.060	2.060
Max Upward from Load Combinations	2.060	2.060
Max Upward from Load Cases	1.030	1.030
Max Downward from all Load Conditio	-2.446	-2.446
Max Downward from Load Combinations	-0.850	-0.850
Max Downward from Load Cases (Resis	-2.446	-2.446
D Only	1.030	1.030
+D+Lr	2.060	2.060
+D+S	2.060	2.060
+D+0.750Lr	1.803	1.803
+D+0.750S	1.803	1.803
+D+0.60W	-0.438	-0.438
+D+0.750Lr+0.450W	0.702	0.702
+D+0.750S+0.450W	0.702	0.702
+0.60D+0.60W	-0.850	-0.850
+0.60D	0.618	0.618
Lr Only	1.030	1.030
S Only	1.030	1.030
W Only	-2.446	-2.446

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

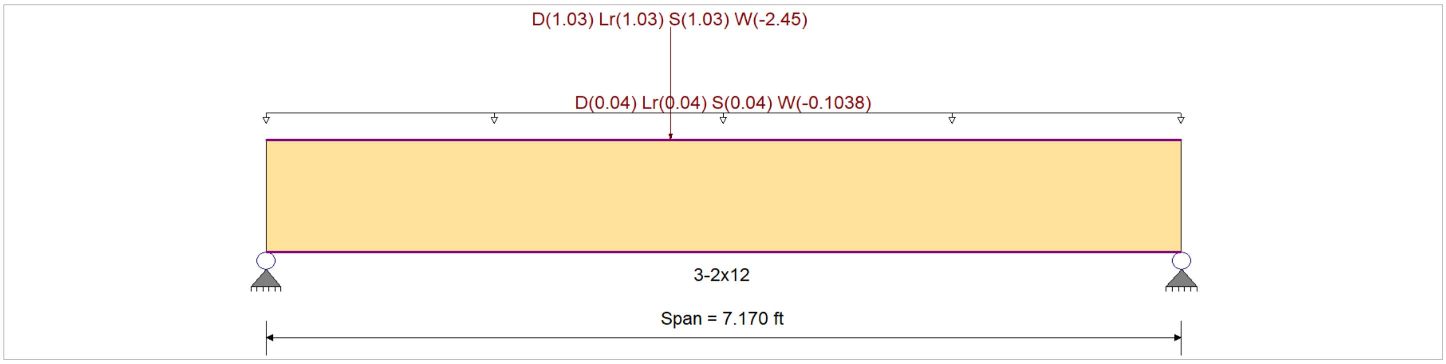
DESCRIPTION: Upset Beam @ RTU Opening - Parallel to Joist w/ mid span beam load

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	850.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,400.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch (North)	Fc - Perp	625.0 psi		
Wood Grade :	No. 1/No. 2	Fv	180.0 psi		
		Ft	500.0 psi	Density	30.590 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05190 ksf, Tributary Width = 2.0 ft, (Roof)
Point Load : D = 1.030, Lr = 1.030, S = 1.030, W = -2.450 k @ 3.170 ft, (Beam North)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.536** : 1	Maximum Shear Stress Ratio	=	**0.195** : 1
Section used for this span		**3-2x12**	Section used for this span		**3-2x12**
fb: Actual	=	524.12 psi	fv: Actual	=	40.38 psi
F'b	=	977.50 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.166 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.019 in	Ratio =	4624	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.045 in	Ratio =	1918	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.037 in	Ratio =	2312	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.016 in	Ratio =	5462	>=180	Span: 1 : +0.60D+0.60W

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 7.170 ft	1	0.343	0.125	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.07	262.1	765.0	0.68	20.2	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.170 ft	1	0.493	0.179	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.15	524.1	1,062.5	1.36	40.4	225.0
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.170 ft	1	0.536	0.195	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.15	524.1	977.5	1.36	40.4	207.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.170 ft	1	0.432	0.157	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.63	458.6	1,062.5	1.19	35.3	225.0
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Upset Beam @ RTU Opening - Parallel to Joist w/ mid span beam load

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 7.170 ft	**1**	0.469	0.171	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.63	458.6	977.5	1.19	35.3	207.0
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.170 ft	**1**	0.085	0.031	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.92	116.1	1,360.0	0.30	9.0	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.170 ft	**1**	0.129	0.047	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.38	175.0	1,360.0	0.45	13.4	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.170 ft	**1**	0.129	0.047	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.38	175.0	1,360.0	0.45	13.4	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.170 ft	**1**	0.162	0.059	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.75	220.9	1,360.0	0.58	17.1	288.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 7.170 ft	**1**	0.116	0.042	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.24	157.2	1,360.0	0.41	12.1	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.0449	3.480

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.436	1.198
Max Upward from Load Combinations	1.436	1.198
Max Upward from Load Cases	0.718	0.599
Max Downward from all Load Conditio	-1.739	-1.455
Max Downward from Load Combinations	-0.613	-0.514
Max Downward from Load Cases (Resis	-1.739	-1.455
D Only	0.718	0.599
+D+Lr	1.436	1.198
+D+S	1.436	1.198
+D+0.750Lr	1.257	1.048
+D+0.750S	1.257	1.048
+D+0.60W	-0.325	-0.274
+D+0.750Lr+0.450W	0.474	0.393
+D+0.750S+0.450W	0.474	0.393
+0.60D+0.60W	-0.613	-0.514
+0.60D	0.431	0.359
Lr Only	0.718	0.599
S Only	0.718	0.599
W Only	-1.739	-1.455

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

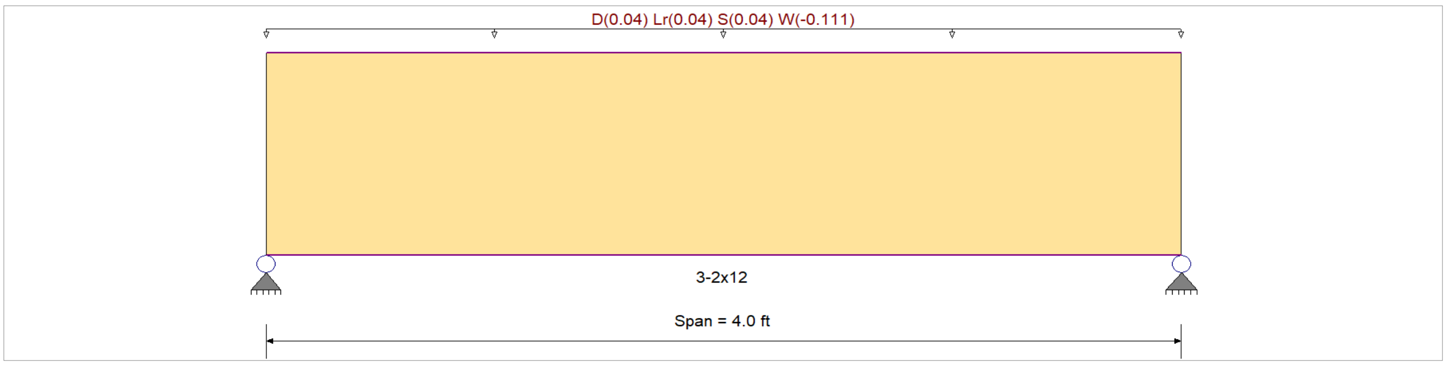
DESCRIPTION: Upset Beam @ RTU Opening - Parallel to Joist

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	850.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,400.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch (North)	Fc - Perp	625.0 psi		
Wood Grade :	No. 1/No. 2	Fv	180.0 psi		
		Ft	500.0 psi	Density	30.590 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05550 ksf, Tributary Width = 2.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.021**: 1	Maximum Shear Stress Ratio	=	**0.012** : 1
Section used for this span		**3-2x12**	Section used for this span		**3-2x12**
fb: Actual	=	20.23 psi	fv: Actual	=	2.53 psi
F'b	=	977.50 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.000 ft	Location of maximum on span	=	3.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.001 in	Ratio =	88473 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.000 in	Ratio =	166147 >=180	Span: 1 : +0.60D+0.60W

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.013	0.008	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.08	10.1	765.0	0.04	1.3	162.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.019	0.011	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.16	20.2	1,062.5	0.09	2.5	225.0
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.021	0.012	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.16	20.2	977.5	0.09	2.5	207.0
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.017	0.010	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.14	17.7	1,062.5	0.07	2.2	225.0
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.018	0.011	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.14	17.7	977.5	0.07	2.2	207.0

Wood Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Upset Beam @ RTU Opening - Parallel to Joist

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	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.005	0.003	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.05	6.7	1,360.0	0.03	0.8	288.0
+D+0.750Lr+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.004	0.002	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.04	5.1	1,360.0	0.02	0.6	288.0
+D+0.750S+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.004	0.002	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.04	5.1	1,360.0	0.02	0.6	288.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.008	0.005	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.09	10.8	1,360.0	0.05	1.3	288.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.004	0.003	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.05	6.1	1,360.0	0.03	0.8	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.0008	2.015

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.160	0.160
Max Upward from Load Combinations	0.160	0.160
Max Upward from Load Cases	0.080	0.080
Max Downward from all Load Conditio	-0.222	-0.222
Max Downward from Load Combinations	-0.085	-0.085
Max Downward from Load Cases (Resis	-0.222	-0.222
D Only	0.080	0.080
+D+Lr	0.160	0.160
+D+S	0.160	0.160
+D+0.750Lr	0.140	0.140
+D+0.750S	0.140	0.140
+D+0.60W	-0.053	-0.053
+D+0.750Lr+0.450W	0.040	0.040
+D+0.750S+0.450W	0.040	0.040
+0.60D+0.60W	-0.085	-0.085
+0.60D	0.048	0.048
Lr Only	0.080	0.080
S Only	0.080	0.080
W Only	-0.222	-0.222

Wood Column

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Upset Beam Posts

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Analysis Method	Allowable Stress Design	
End Fixities	Top & Bottom Pinned	
Overall Column Height		14.25 ft
(Used for non-slender calculations)		
Wood Species	Douglas Fir-Larch	
Wood Grade	No.2	

Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 psi	Density	31.210 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name	**3-2x6**
Wood Grading/Manuf.	Graded Lumber
Wood Member Type	Sawn
Exact Width	**4.50** in
Exact Depth	**5.50** in
Area	24.750 in^2
Ix	62.391 in^4
Iy	41.766 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.30
Cf or Cv for Compressio	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Fact	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0
Use Cr : Repetitive ?	No

Column Buckling Condition:
ABOUT X-X Axis: Lux = 14.25 ft, Kx = 1.0
ABOUT Y-Y Axis: Luy = 4.0 ft, Ky = 1.0

Applied Loads

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

Column self weight included : 76.440 lbs * Dead Load Factor
AXIAL LOADS . . .
Upset Beam Gravity Load: Axial Load at 14.250 ft, D = 0.720, Lr = 0.720, S = 0.720 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.1334 : 1**
Load Combination	+D+S
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are .	
Applied Axial	1.516 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	459.399 psi
PASS Maximum Shear Stress Ratio =	**0.0 : 1**
Load Combination	+0.60D
Location of max.above base	14.250 ft
Applied Design Shear	0.0 psi
Allowable Shear	288.0 psi

Maximum SERVICE Lateral Load Reactions . .

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

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.335	0.07183	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+Lr	1.250	0.249	0.1325	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+S	1.150	0.269	0.1334	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+0.750Lr	1.250	0.249	0.1167	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+0.750S	1.150	0.269	0.1175	PASS	0.0 ft	0.0	PASS	14.250 ft
+0.60D	1.600	0.198	0.04108	PASS	0.0 ft	0.0	PASS	14.250 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only					0.796				

Wood Column

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Upset Beam Posts

Maximum Reactions

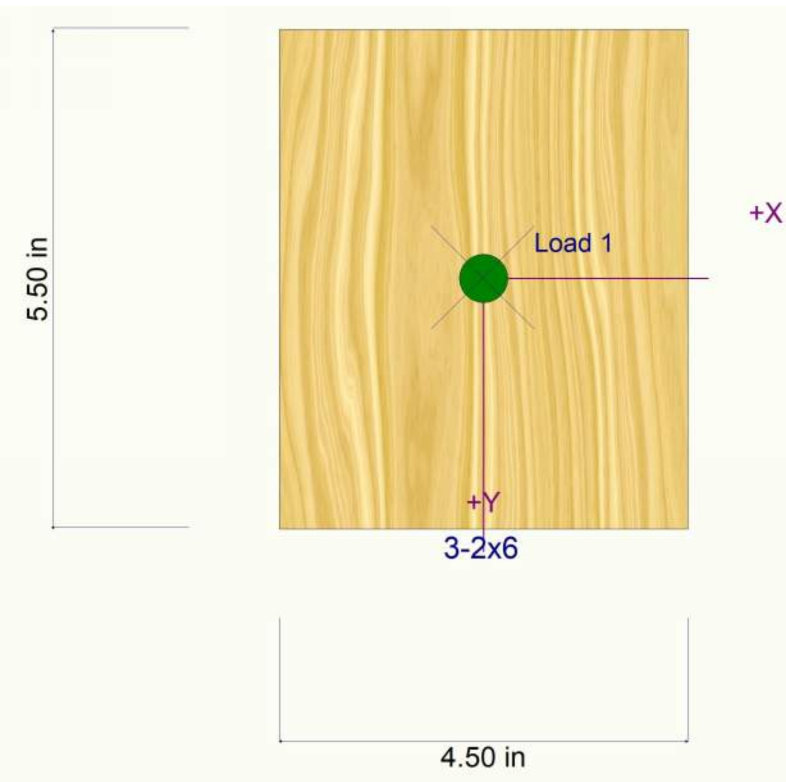
Note: Only non-zero reactions are listed.

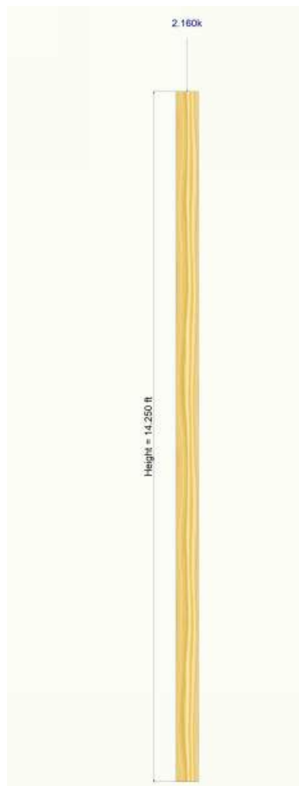
Load Combination	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	@ Top	Mx - End Moments @ Base	@ Top
+D+Lr					1.516				
+D+S					1.516				
+D+0.750Lr					1.336				
+D+0.750S					1.336				
+0.60D					0.478				
Lr Only					0.720				
S Only					0.720				

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches





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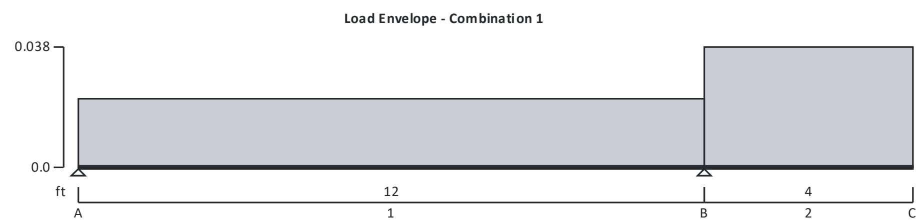
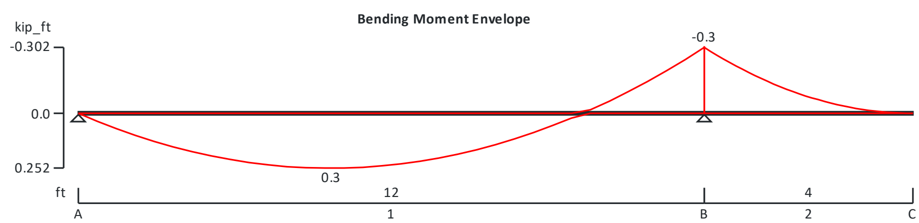
	Section				Sheet no./rev. 1	
	Calc. by jcf	Date 4/25/2024	Chk'd by	Date	App'd by	Date

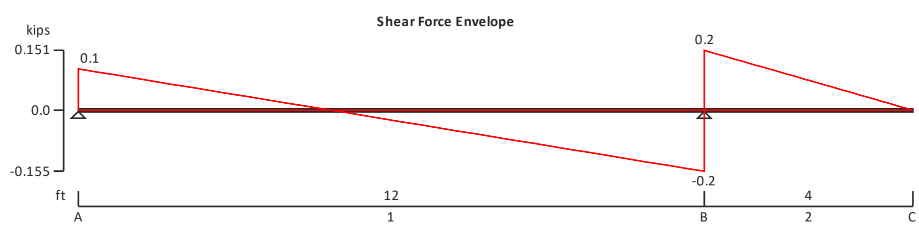
STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10







Applied loading

Beam loads

Wind	Wind partial UDL 36 lb/ft from 0.00 in to 144.00 in
Wind	Wind partial UDL 63 lb/ft from 144.00 in to 192.00 in
Axial load	Factored axial compression 0.400 kips

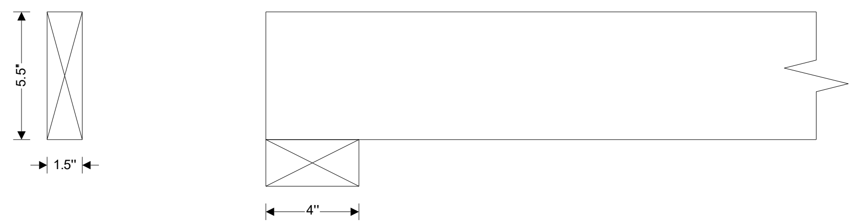
Load combinations

Load combination 1	Support A	Dead × 1.00
		Roof Live × 1.00
		Wind × 0.60
	Span 1	Dead × 1.00
		Roof Live × 1.00
		Wind × 0.60
	Support B	Dead × 1.00
		Roof Live × 1.00

		Wind × 0.60
	Span 2	Dead × 1.00
		Roof Live × 1.00
		Wind × 0.60
	Support C	Dead × 1.00
		Roof Live × 1.00
		Wind × 0.60

Analysis results

Maximum moment	M_{max} = **252** lb_ft	M_{min} = **-302** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **302** lb_ft	
Maximum shear	F_{max} = **151** lb	F_{min} = **-155** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **155** lb	
Total load on member	W_{tot} = **410** lb	
Reaction at support A	R_{A_max} = **104** lb	R_{A_min} = **104** lb
Unfactored wind load reaction at support A	R_{A_Wind} = **174** lb	
Reaction at support B	R_{B_max} = **306** lb	R_{B_min} = **306** lb
Unfactored wind load reaction at support B	R_{B_Wind} = **510** lb	
Reaction at support C	R_{C_max} = **0** lb	R_{C_min} = **0** lb
Unfactored wind load reaction at support C	R_{C_Wind} = **0** lb	



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**

Length of span 1 L_{s1} = **12** ft
Length of span 2 L_{s2} = **4** ft
Length of bearing L_b = **4** in
Load duration **Ten years**
Unbraced length in x-axis L_x = **12** ft
Effective length factor in x-axis K_x = **1**
Effective length in x-axis $L_{ex} = L_x \times K_x$ = **12** ft
Unbraced length in y-axis L_y = **4** ft
Effective length factor in y-axis K_y = **1**
Effective length in y-axis $L_{ey} = L_y \times K_y$ = **4** ft
The beam is one of three or more repetitive members

Section properties

Cross sectional area of member $A = N \times b \times d$ = **8.25** in^2
Section modulus $S_x = N \times b \times d^2 / 6$ = **7.56** in^3
$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area $I_x = N \times b \times d^3 / 12$ = **20.80** in^4
$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**
Temperature factor - Table 2.3.3 C_t = **1.00**
Size factor for bending - Table 4A C_{Fb} = **1.30**
Size factor for tension - Table 4A C_{Ft} = **1.30**
Size factor for compression - Table 4A C_{Fc} = **1.10**
Flat use factor - Table 4A C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8
C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8
C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8
C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9 C_r = **1.15**
Bearing area factor - cl.3.10.4 $C_b = (L_b + 0.375 \text{ in}) / L_b$ = **1.09**
Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2
Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ey} / b)^2$ = **466** lb/in^2
c = **0.80**
Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} = \mathbf{0.29}$$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**
- Beam is fully restrained

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

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_{ic\_perp} \times C_b$ = **684** lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{B\_max} / (N \times b \times L_b) =$ **51** lb/in^2

$f_{c\_perp} / F_{c\_perp}' =$ **0.075**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r =$ **1346** lb/in^2

Actual bending stress $f_b = M / S_x =$ **480** lb/in^2

$f_b / F_b' =$ **0.357**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P =$ **430** lb/in^2

Applied compressive stress $f_c = P / A =$ **48** lb/in^2

$f_c / F_c' =$ **0.113**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2 =$ **696** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_b / (F_b' \times [1 - (f_c / F_{cE1})]) =$ **0.396 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i =$ **180** lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) =$ **28** lb/in^2

$f_v / F_v' =$ **0.156**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE} =$ **1600000** lb/in^2

Design deflection $\delta_{adm} = 0.003 \times L_{s1} =$ **0.432** in

Total deflection $\delta_{b\_s1} =$ **0.273** in

$\delta_{b\_s1} / \delta_{adm} =$ **0.633**

PASS - Total deflection is less than design deflection

Mettemeyer Engineering	Project Andy's Frozen Custard Lee's Summit				Job Ref. 24-0121	
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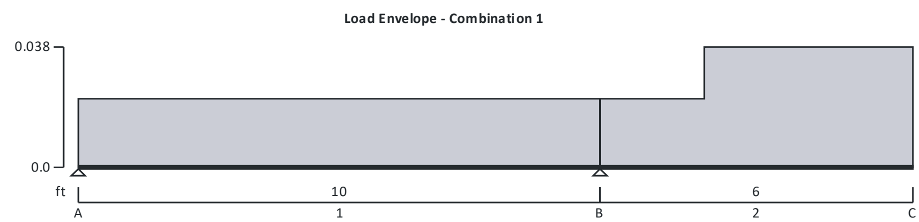
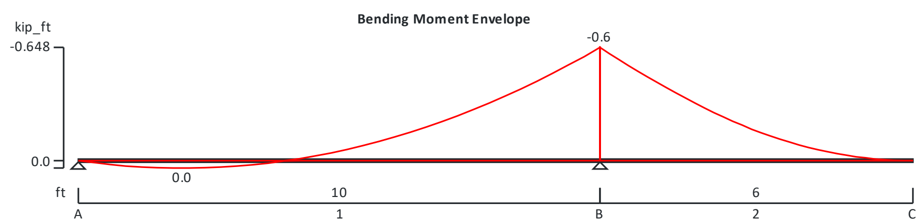
	Calc. by jcf	Date 4/25/2024	Chk'd by	Date	App'd by	Date

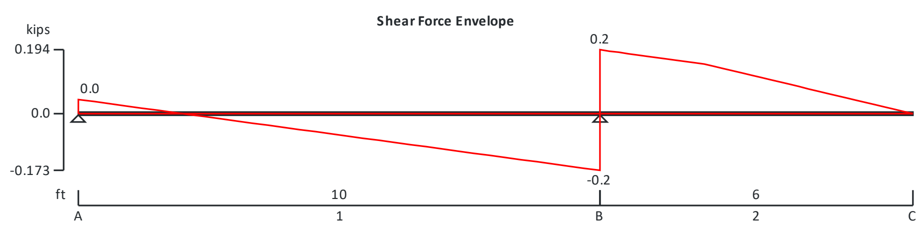
STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10







Applied loading

Beam loads

Wind	Wind partial UDL 36 lb/ft from 0.00 in to 144.00 in
Wind	Wind partial UDL 63 lb/ft from 144.00 in to 192.00 in
Axial load	Factored axial compression 0.400 kips

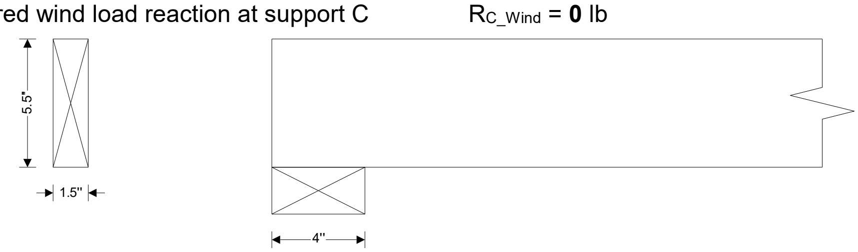
Load combinations

Load combination 1	Support A	Dead × 1.00
		Roof Live × 1.00
		Wind × 0.60
	Span 1	Dead × 1.00
		Roof Live × 1.00
		Wind × 0.60
	Support B	Dead × 1.00
		Roof Live × 1.00

		Wind × 0.60
	Span 2	Dead × 1.00
		Roof Live × 1.00
		Wind × 0.60
	Support C	Dead × 1.00
		Roof Live × 1.00
		Wind × 0.60

Analysis results

Maximum moment	M_{max} = **43** lb_ft	M_{min} = **-648** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **648** lb_ft	
Maximum shear	F_{max} = **194** lb	F_{min} = **-173** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **194** lb	
Total load on member	W_{tot} = **410** lb	
Reaction at support A	R_{A_max} = **43** lb	R_{A_min} = **43** lb
Unfactored wind load reaction at support A	R_{A_Wind} = **72** lb	
Reaction at support B	R_{B_max} = **367** lb	R_{B_min} = **367** lb
Unfactored wind load reaction at support B	R_{B_Wind} = **612** lb	
Reaction at support C	R_{C_max} = **0** lb	R_{C_min} = **0** lb
Unfactored wind load reaction at support C	R_{C_Wind} = **0** lb	



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**

Length of span 1	L_{s1} = **10** ft
Length of span 2	L_{s2} = **6** ft
Length of bearing	L_b = **4** in
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **10** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **10** ft
Unbraced length in y-axis	L_y = **4** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **4** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.30**
Size factor for tension - Table 4A	C_{Ft} = **1.30**
Size factor for compression - Table 4A	C_{Fc} = **1.10**
Flat use factor - Table 4A	C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.00**
Bearing area factor - cl.3.10.4	$C_b = (L_b + 0.375 \text{ in}) / L_b$ = **1.09**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ey} / b)^2$ = **466** lb/in^2
	c = **0.80**
Column stability factor - eq.3.7-1	

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} = \mathbf{0.29}$$

Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **3.00**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_{ic\_perp} \times C_b$ = **684** lb/in^2
Applied compression stress perpendicular to grain	$f_{c\_perp} = R_{B\_max} / (N \times b \times L_b)$ = **61** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.090**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1170** lb/in^2

Actual bending stress $f_b = M / S_x$ = **1028** lb/in^2

f_b / F_b' = **0.879**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **430** lb/in^2

Applied compressive stress $f_c = P / A$ = **48** lb/in^2

f_c / F_c' = **0.113**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1002** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_b / (F_b' \times [1 - (f_c / F_{cE1})])$ = **0.936 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i$ = **180** lb/in^2

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

f_v / F_v' = **0.196**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1600000** lb/in^2

Design deflection $\delta_{adm} = 0.004 \times 2 \times L_{s2}$ = **0.576** in

Total deflection δ_{b_s2} = **1.174** in

$\delta_{b\_s2} / \delta_{adm}$ = **2.038**

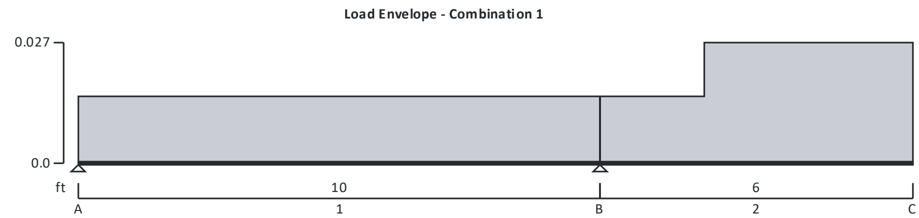
FAIL - Total deflection exceeds design deflection

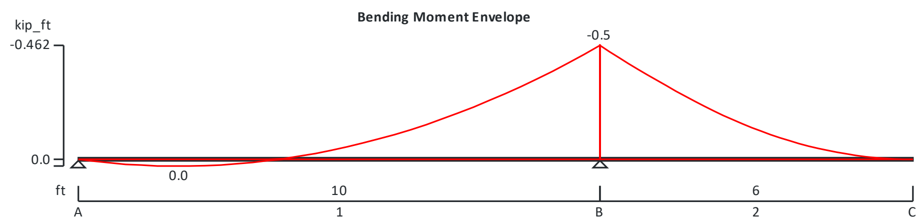
Per 2018 IBC Table 1604.3 (note f), total deflection due to wind is permitted to be taken as 0.42 times the "components and cladding" loads. Please see following calculation with reduced loading for deflection calculation.

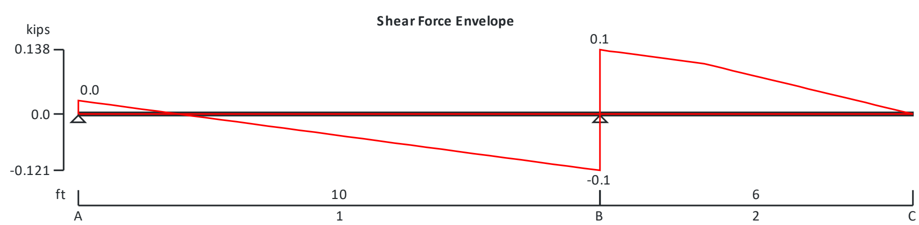
STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10







Applied loading

Beam loads

Wind	Wind partial UDL 15 lb/ft from 0.00 in to 144.00 in
Wind	Wind partial UDL 27 lb/ft from 144.00 in to 192.00 in
Axial load	Factored axial compression 0.400 kips

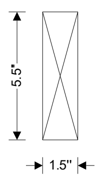
Load combinations

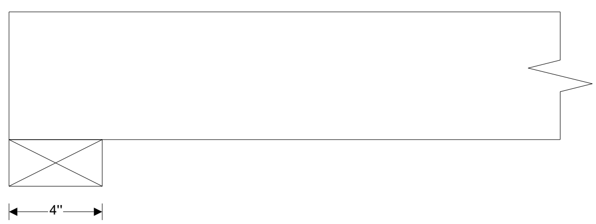
Load combination 1	Support A	Dead × 1.00
		Roof Live × 1.00
		Wind × 1.00
	Span 1	Dead × 1.00
		Roof Live × 1.00
		Wind × 1.00
	Support B	Dead × 1.00
		Roof Live × 1.00

		Wind × 1.00
	Span 2	Dead × 1.00
		Roof Live × 1.00
		Wind × 1.00
	Support C	Dead × 1.00
		Roof Live × 1.00
		Wind × 1.00

Analysis results

Maximum moment	M_{max} = **28** lb_ft	M_{min} = **-462** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **462** lb_ft	
Maximum shear	F_{max} = **138** lb	F_{min} = **-121** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **138** lb	
Total load on member	W_{tot} = **288** lb	
Reaction at support A	R_{A_max} = **29** lb	R_{A_min} = **29** lb
Unfactored wind load reaction at support A	R_{A_Wind} = **29** lb	
Reaction at support B	R_{B_max} = **259** lb	R_{B_min} = **259** lb
Unfactored wind load reaction at support B	R_{B_Wind} = **259** lb	
Reaction at support C	R_{C_max} = **0** lb	R_{C_min} = **0** lb
Unfactored wind load reaction at support C	R_{C_Wind} = **0** lb	





Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**

Length of span 1	L_{s1} = **10** ft
Length of span 2	L_{s2} = **6** ft
Length of bearing	L_b = **4** in
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **10** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **10** ft
Unbraced length in y-axis	L_y = **4** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **4** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.30**
Size factor for tension - Table 4A	C_{Ft} = **1.30**
Size factor for compression - Table 4A	C_{Fc} = **1.10**
Flat use factor - Table 4A	C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.00**
Bearing area factor - cl.3.10.4	$C_b = (L_b + 0.375\text{ in}) / L_b$ = **1.09**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ey} / b)^2$ = **466** lb/in^2
	c = **0.80**
Column stability factor - eq.3.7-1	

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} = \mathbf{0.29}$$

Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **3.00**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_{ic\_perp} \times C_b$ = **684** lb/in^2
Applied compression stress perpendicular to grain	$f_{c\_perp} = R_{B\_max} / (N \times b \times L_b)$ = **43** lb/in^2

$f_{c\_perp} / F_{c\_perp}' = \mathbf{0.063}$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1170}$ lb/in^2

Actual bending stress $f_b = M / S_x = \mathbf{733}$ lb/in^2

$f_b / F_b' = \mathbf{0.627}$

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P = \mathbf{430}$ lb/in^2

Applied compressive stress $f_c = P / A = \mathbf{48}$ lb/in^2

$f_c / F_c' = \mathbf{0.113}$

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2 = \mathbf{1002}$ lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_b / (F_b' \times [1 - (f_c / F_{cE1})]) = \mathbf{0.671} \ \mathbf{< 1}$

PASS - Combined compressive and bending stresses are within permissible limits

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{180}$ lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A) = \mathbf{25}$ lb/in^2

$f_v / F_v' = \mathbf{0.139}$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE} = \mathbf{1600000}$ lb/in^2

Design deflection $\delta_{adm} = 0.004 \times 2 \times L_{s2} = \mathbf{0.576}$ in

Total deflection $\delta_{b\_s2} = \mathbf{0.508}$ in

$\delta_{b\_s2} / \delta_{adm} = \mathbf{0.881}$

PASS - Total deflection is less than design deflection

Steel Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

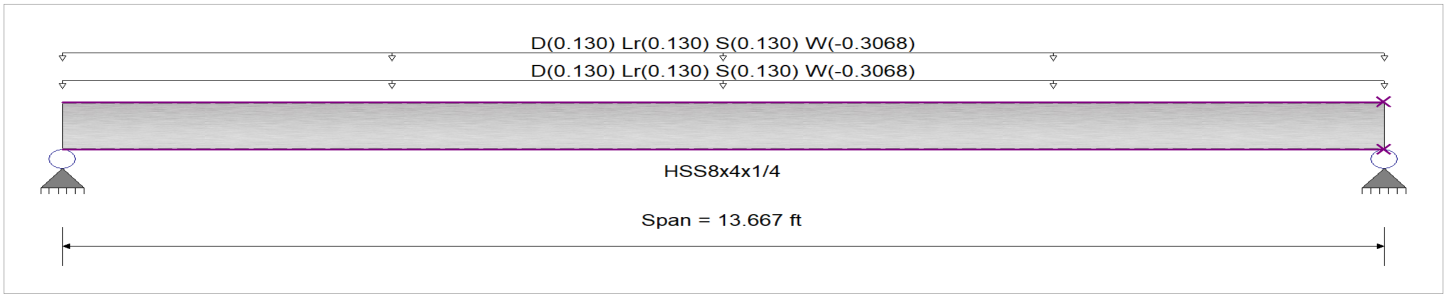
DESCRIPTION: Front of House - Grid 7

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.04720 ksf, Tributary Width = 6.50 ft, (Roof Left)

Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.04720 ksf, Tributary Width = 6.50 ft, (Roof Right)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.379** : 1	Maximum Shear Stress Ratio =	**0.060** : 1
Section used for this span	**HSS8x4x1/4**	Section used for this span	**HSS8x4x1/4**
Ma : Applied	12.585 k-ft	Va : Applied	3.683 k
Mn / Omega : Allowable	33.184 k-ft	Vn/Omega : Allowable	61.119 k
Load Combination	+D+Lr	Load Combination	+D+Lr
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.166 in Ratio =	985	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.392 in Ratio =	417	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.345 in Ratio =	476	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.128 in Ratio =	1277	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 13.67 ft	1	0.196	0.031	6.51		6.51	55.42	33.18	1.00	1.00	1.91	102.07	61.12
+D+Lr													
Dsgn. L = 13.67 ft	1	0.379	0.060	12.59		12.59	55.42	33.18	1.00	1.00	3.68	102.07	61.12
+D+S													
Dsgn. L = 13.67 ft	1	0.379	0.060	12.59		12.59	55.42	33.18	1.00	1.00	3.68	102.07	61.12
+D+0.750Lr													
Dsgn. L = 13.67 ft	1	0.334	0.053	11.07		11.07	55.42	33.18	1.00	1.00	3.24	102.07	61.12
+D+0.750S													
Dsgn. L = 13.67 ft	1	0.334	0.053	11.07		11.07	55.42	33.18	1.00	1.00	3.24	102.07	61.12
+D+0.60W													
Dsgn. L = 13.67 ft	1	0.063	0.010		-2.08	2.08	55.42	33.18	1.00	1.00	0.61	102.07	61.12
+D+0.750Lr+0.450W													
Dsgn. L = 13.67 ft	1	0.139	0.022	4.62		4.62	55.42	33.18	1.00	1.00	1.35	102.07	61.12
+D+0.750S+0.450W													
Dsgn. L = 13.67 ft	1	0.139	0.022	4.62		4.62	55.42	33.18	1.00	1.00	1.35	102.07	61.12
+0.60D+0.60W													
Dsgn. L = 13.67 ft	1	0.141	0.022		-4.69	4.69	55.42	33.18	1.00	1.00	1.37	102.07	61.12
+0.60D													
Dsgn. L = 13.67 ft	1	0.118	0.019	3.91		3.91	55.42	33.18	1.00	1.00	1.14	102.07	61.12

Steel Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Front of House - Grid 7

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.3926	6.873

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.683	3.683
Max Upward from Load Combinations	3.683	3.683
Max Upward from Load Cases	1.907	1.907
Max Downward from all Load Conditions (Resis	-4.193	-4.193
Max Downward from Load Combinations (Resi:	-1.372	-1.372
Max Downward from Load Cases (Resisting Up	-4.193	-4.193
D Only	1.907	1.907
+D+Lr	3.683	3.683
+D+S	3.683	3.683
+D+0.750Lr	3.239	3.239
+D+0.750S	3.239	3.239
+D+0.60W	-0.609	-0.609
+D+0.750Lr+0.450W	1.352	1.352
+D+0.750S+0.450W	1.352	1.352
+0.60D+0.60W	-1.372	-1.372
+0.60D	1.144	1.144
Lr Only	1.777	1.777
S Only	1.777	1.777
W Only	-4.193	-4.193

Steel Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

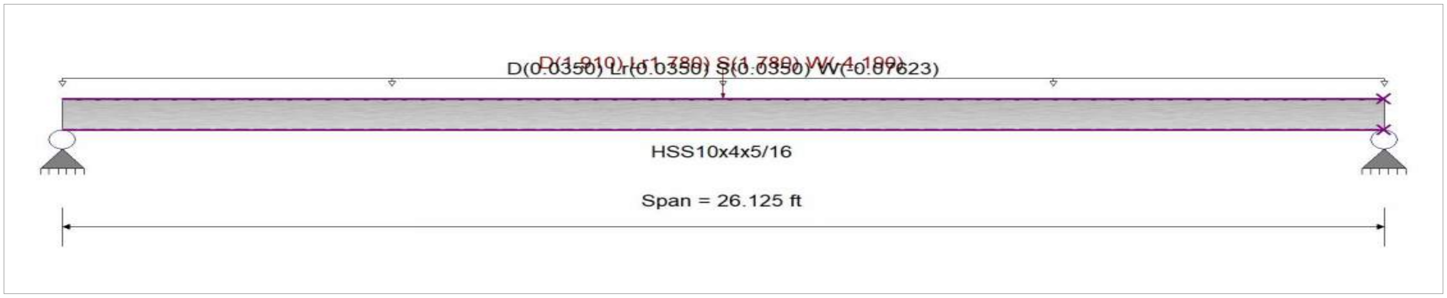
DESCRIPTION: Front of House - Curtain Wall Grids B & D

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.04356 ksf, Tributary Width = 1.750 ft, (Roof)

Point Load : D = 1.910, Lr = 1.780, S = 1.780, W = -4.190 k @ 13.060 ft, (B1)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.563 : 1	Maximum Shear Stress Ratio =	0.033 : 1
Section used for this span	**HSS10x4x5/16**	Section used for this span	**HSS10x4x5/16**
Ma : Applied	32.422 k-ft	Va : Applied	3.120 k
Mn / Omega : Allowable	57.635 k-ft	Vn/Omega : Allowable	95.424 k
Load Combination	+D+Lr	Load Combination	+D+Lr
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.580 in Ratio =	540	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.804 in Ratio =	389	>=360	Span: 1 : +0.60W
Max Downward Total Deflection	1.304 in Ratio =	240	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.370 in Ratio =	846	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 26.13 ft	1	0.309	0.019	17.81		17.81	96.25	57.63	1.00	1.00	1.77	159.36	95.42
+D+Lr													
Dsgn. L = 26.13 ft	1	0.563	0.033	32.42		32.42	96.25	57.63	1.00	1.00	3.12	159.36	95.42
+D+S													
Dsgn. L = 26.13 ft	1	0.563	0.033	32.42		32.42	96.25	57.63	1.00	1.00	3.12	159.36	95.42
+D+0.750Lr													
Dsgn. L = 26.13 ft	1	0.499	0.029	28.77		28.77	96.25	57.63	1.00	1.00	2.78	159.36	95.42
+D+0.750S													
Dsgn. L = 26.13 ft	1	0.499	0.029	28.77		28.77	96.25	57.63	1.00	1.00	2.78	159.36	95.42
+D+0.60W													
Dsgn. L = 26.13 ft	1	0.043	0.003		-2.51	2.51	96.25	57.63	1.00	1.00	0.30	159.36	95.42
+D+0.750Lr+0.450W													
Dsgn. L = 26.13 ft	1	0.235	0.015	13.53		13.53	96.25	57.63	1.00	1.00	1.39	159.36	95.42
+D+0.750S+0.450W													
Dsgn. L = 26.13 ft	1	0.235	0.015	13.53		13.53	96.25	57.63	1.00	1.00	1.39	159.36	95.42
+0.60D+0.60W													
Dsgn. L = 26.13 ft	1	0.167	0.008		-9.63	9.63	96.25	57.63	1.00	1.00	0.79	159.36	95.42
+0.60D													
Dsgn. L = 26.13 ft	1	0.185	0.011	10.69		10.69	96.25	57.63	1.00	1.00	1.06	159.36	95.42

Steel Beam

DESCRIPTION: Front of House - Curtain Wall Grids B & D

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	1.3036	13.063		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.120	3.119
Max Upward from Load Combinations	3.120	3.119
Max Upward from Load Cases	1.773	1.772
Max Downward from all Load Conditions (Resis	-3.091	-3.090
Max Downward from Load Combinations (Resi:	-0.791	-0.791
Max Downward from Load Cases (Resisting Up	-3.091	-3.090
D Only	1.773	1.772
+D+Lr	3.120	3.119
+D+S	3.120	3.119
+D+0.750Lr	2.783	2.783
+D+0.750S	2.783	2.783
+D+0.60W	-0.082	-0.082
+D+0.750Lr+0.450W	1.392	1.392
+D+0.750S+0.450W	1.392	1.392
+0.60D+0.60W	-0.791	-0.791
+0.60D	1.064	1.063
Lr Only	1.347	1.347
S Only	1.347	1.347
W Only	-3.091	-3.090

Steel Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

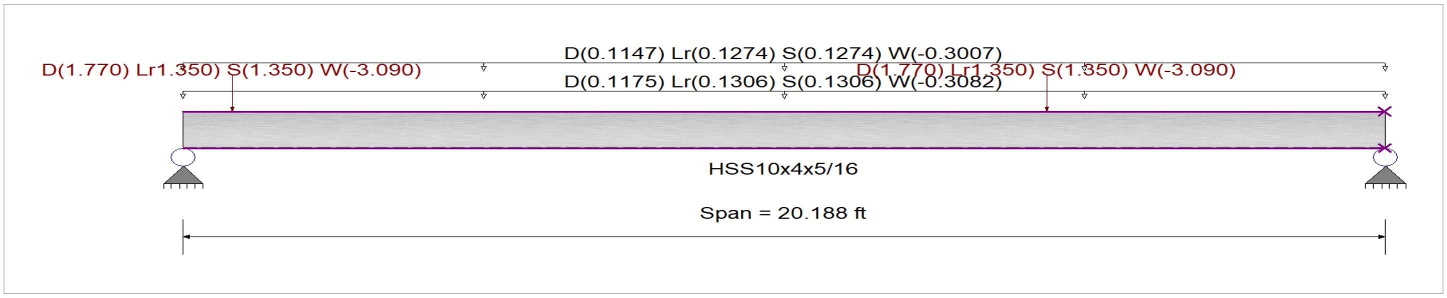
DESCRIPTION: Front of House - Transition Area Grid 5

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.0180, Lr = 0.020, S = 0.020, W = -0.04720 ksf, Tributary Width = 6.530 ft, (Roof East)

Uniform Load : D = 0.0180, Lr = 0.020, S = 0.020, W = -0.04720 ksf, Tributary Width = 6.370 ft, (Roof East)

Point Load : D = 1.770, Lr = 1.350, S = 1.350, W = -3.090 k @ 0.830 ft, (Curtain Wall Beam)

Point Load : D = 1.770, Lr = 1.350, S = 1.350, W = -3.090 k @ 14.50 ft, (B2)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.644** : 1	Maximum Shear Stress Ratio =	**0.095** : 1
Section used for this span	**HSS10x4x5/16**	Section used for this span	**HSS10x4x5/16**
Ma : Applied	37.089 k-ft	Va : Applied	9.097 k
Mn / Omega : Allowable	57.635 k-ft	Vn/Omega : Allowable	95.424 k
Load Combination	+D+Lr	Load Combination	+D+Lr
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.506 in Ratio =	478	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.497 in Ratio =	486	>=360	Span: 1 : +0.420W
Max Downward Total Deflection	1.057 in Ratio =	229	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.380 in Ratio =	637	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 20.19 ft	1	0.336	0.050	19.35		19.35	96.25	57.63	1.00	1.00	4.82	159.36	95.42
+D+Lr													
Dsgn. L = 20.19 ft	1	0.644	0.095	37.09		37.09	96.25	57.63	1.00	1.00	9.10	159.36	95.42
+D+S													
Dsgn. L = 20.19 ft	1	0.644	0.095	37.09		37.09	96.25	57.63	1.00	1.00	9.10	159.36	95.42
+D+0.750Lr													
Dsgn. L = 20.19 ft	1	0.567	0.084	32.65		32.65	96.25	57.63	1.00	1.00	8.03	159.36	95.42
+D+0.750S													
Dsgn. L = 20.19 ft	1	0.567	0.084	32.65		32.65	96.25	57.63	1.00	1.00	8.03	159.36	95.42
+D+0.60W													
Dsgn. L = 20.19 ft	1	0.098	0.012		-5.65	5.65	96.25	57.63	1.00	1.00	1.17	159.36	95.42
+D+0.750Lr+0.450W													
Dsgn. L = 20.19 ft	1	0.243	0.037	13.98		13.98	96.25	57.63	1.00	1.00	3.54	159.36	95.42

Steel Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Front of House - Transition Area Grid 5

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+0.750S+0.450W													
Dsgn. L = 20.19 ft	1	0.243	0.037	13.98		13.98	96.25	57.63	1.00	1.00	3.54	159.36	95.42
+0.60D+0.60W													
Dsgn. L = 20.19 ft	1	0.231	0.032		-13.34	13.34	96.25	57.63	1.00	1.00	3.10	159.36	95.42
+0.60D													
Dsgn. L = 20.19 ft	1	0.201	0.030	11.61		11.61	96.25	57.63	1.00	1.00	2.89	159.36	95.42

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	1.0567	10.324		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	9.097	7.596
Max Upward from Load Combinations	9.097	7.596
Max Upward from Load Cases	4.818	3.966
Max Downward from all Load Conditions (Resis	-9.979	-8.492
Max Downward from Load Combinations (Resi:	-3.097	-2.716
Max Downward from Load Cases (Resisting Up	-9.979	-8.492
D Only	4.818	3.966
+D+Lr	9.097	7.596
+D+S	9.097	7.596
+D+0.750Lr	8.027	6.688
+D+0.750S	8.027	6.688
+D+0.60W	-1.169	-1.129
+D+0.750Lr+0.450W	3.537	2.867
+D+0.750S+0.450W	3.537	2.867
+0.60D+0.60W	-3.097	-2.716
+0.60D	2.891	2.380
Lr Only	4.279	3.629
S Only	4.279	3.629
W Only	-9.979	-8.492

Steel Beam

Project File: 24-0121 - afc lee's summit.ec6

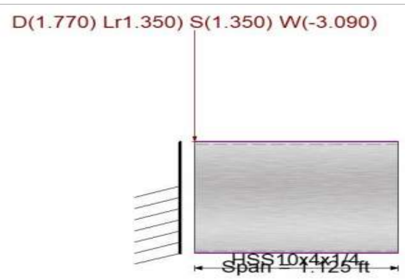
DESCRIPTION: Front of House - Cantilever Support Beam Grid 8

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load(s) for Span Number 1
Point Load : D = 1.770, Lr = 1.350, S = 1.350, W = -3.090 k @ 0.0 ft, (Grid B Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.000** : 1	Maximum Shear Stress Ratio =	**0.000** : 1
Section used for this span	**HSS10x4x1/4**	Section used for this span	**HSS10x4x1/4**
Ma : Applied	0.014 k-ft	Va : Applied	0.02522 k
Mn / Omega : Allowable	47.405 k-ft	Vn/Omega : Allowable	77.861 k
Load Combination	D Only	Load Combination	+D+0.60W
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in Ratio =	0	<360	n/a
Max Upward Transient Deflection	0 in Ratio =	0	<360	n/a
Max Downward Total Deflection	0 in Ratio =	0	<180	n/a
Max Upward Total Deflection	0 in Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination			Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length		Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.03	130.03	77.86
+D+Lr														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.03	130.03	77.86
+D+S														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.03	130.03	77.86
+D+0.750Lr														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.03	130.03	77.86
+D+0.750S														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.03	130.03	77.86
+D+0.60W														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.03	130.03	77.86
+D+0.750Lr+0.450W														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.03	130.03	77.86
+D+0.750S+0.450W														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.03	130.03	77.86
+0.60D+0.60W														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.02	130.03	77.86
+0.60D														
Dsgn. L =	1.13 ft	1	0.000	0.000		-0.01	0.01	79.17	47.41	1.00	1.00	0.02	130.03	77.86

Steel Beam

DESCRIPTION: Front of House - Cantilever Support Beam Grid 8

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000		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 Conditions	3.145	
Max Upward from Load Combinations	3.145	
Max Upward from Load Cases	1.795	
Max Downward from all Load Conditions (Resis	-3.090	
Max Downward from Load Combinations (Resi:	-0.777	
Max Downward from Load Cases (Resisting Up	-3.090	
D Only	1.795	
+D+Lr	3.145	
+D+S	3.145	
+D+0.750Lr	2.808	
+D+0.750S	2.808	
+D+0.60W	-0.059	
+D+0.750Lr+0.450W	1.417	
+D+0.750S+0.450W	1.417	
+0.60D+0.60W	-0.777	
+0.60D	1.077	
Lr Only	1.350	
S Only	1.350	
W Only	-3.090	

Steel Column

Project File: 24-0121 - afc lee's summit.ec6

DESCRIPTION: Front of House - Column @ F-5

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name :	**HSS5-1/2x5-1/2x3/8**	Overall Column Height	15.875 ft
Analysis Method :	Allowable Strength	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade		Brace condition :	
Fy : Steel Yield	36.0 ksi		
E : Elastic Bending Modulus	29,000.0 ksi		

Unbraced Length for buckling ABOUT X-X Axis = 15.875 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 15.875 ft, K = 2.1

Applied Loads

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

Column self weight included : 395.764 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 12.0 ft, D = 3.970, LR = 3.630, S = 3.630, W = -8.490 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.2860**	: 1
Load Combination	+D+Lr	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pa : Axial	7.996	k
Pn / Omega : Allowabl	27.956	k
Ma-x : Applied	0.0	k-ft
Mn-x / Omega : Allowable	23.533	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	23.533	k-ft
PASS Maximum Shear Stress Rati	**0.0**	: 1
Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	k

Maximum Load Reactions . .

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

Maximum Load Deflections . . .

Along Y-Y	0.0 in	at	0.0ft	above base
for load combination :				
Along X-X	0.0 in	at	0.0ft	above base
for load combination :				

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.156	PASS	0.00 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft
+D+Lr	0.286	PASS	0.00 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft
+D+S	0.286	PASS	0.00 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft
+D+0.750Lr	0.254	PASS	0.00 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft
+D+0.750S	0.254	PASS	0.00 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft
+D+0.60W	0.014	PASS	12.04 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft
+D+0.750Lr+0.450W	0.117	PASS	0.00 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft
+D+0.750S+0.450W	0.117	PASS	0.00 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft
+0.60D+0.60W	0.017	PASS	0.00 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft
+0.60D	0.094	PASS	0.00 ft	1.00	1.00	192.33	192.33	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	4.366								
+D+Lr	7.996								
+D+S	7.996								
+D+0.750Lr	7.088								
+D+0.750S	7.088								
+D+0.60W	-0.728								

Steel Column

DESCRIPTION: Front of House - Column @ F-5

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
+D+0.750Lr+0.450W	3.268								
+D+0.750S+0.450W	3.268								
+0.60D+0.60W	-2.475								
+0.60D	2.619								
Lr Only	3.630								
S Only	3.630								
W Only	-8.490								

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	7.996								
"	Minimum	-8.490								
Reaction, X-X Axis Base	Maximum	4.366								
"	Minimum	4.366								
Reaction, Y-Y Axis Base	Maximum	4.366								
"	Minimum	4.366								
Reaction, X-X Axis Top	Maximum	4.366								
"	Minimum	4.366								
Reaction, Y-Y Axis Top	Maximum	4.366								
"	Minimum	4.366								
Moment, X-X Axis Base	Maximum	4.366								
"	Minimum	4.366								
Moment, Y-Y Axis Base	Maximum	4.366								
"	Minimum	4.366								
Moment, X-X Axis Top	Maximum	4.366								
"	Minimum	4.366								
Moment, Y-Y Axis Top	Maximum	4.366								
"	Minimum	4.366								

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	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : HSS5-1/2x5-1/2x3/8

Steel Section Properties : HSS5-1/2x5-1/2x3/8

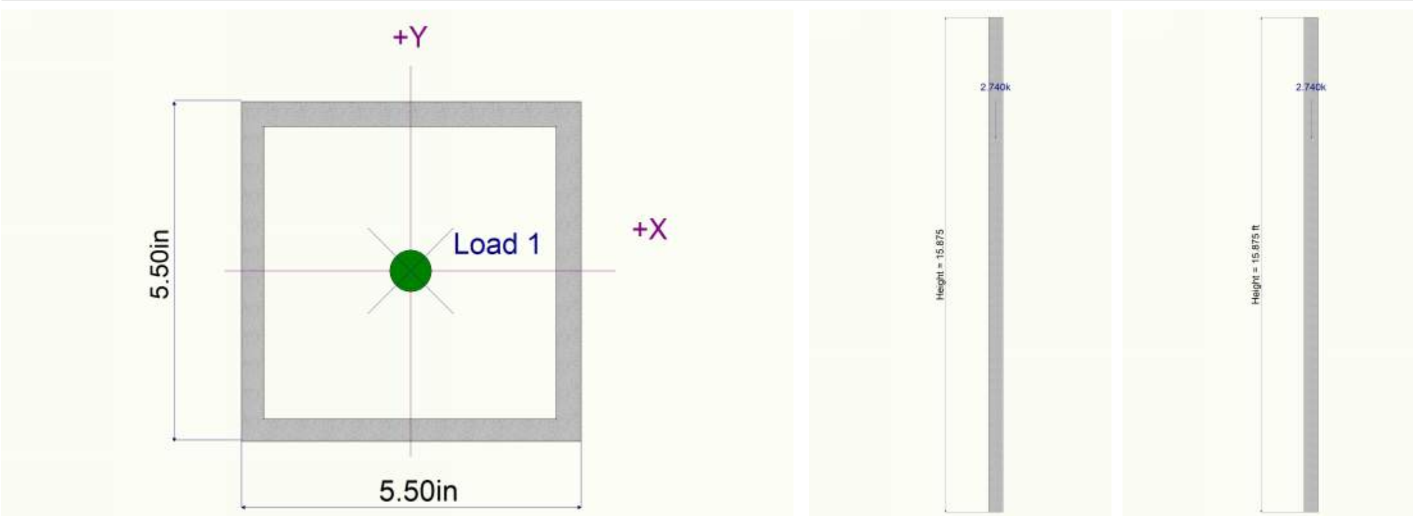
Steel Column

DESCRIPTION: Front of House - Column @ F-5

Depth	=	5.500 in	I xx	=	29.70 in^4	J	=	49.000 in^4
Design Thick	=	0.349 in	S xx	=	10.80 in^3			
Width	=	5.500 in	R xx	=	2.080 in			
Wall Thick	=	0.375 in	Zx	=	13.100 in^3			
Area	=	6.880 in^2	I yy	=	29.700 in^4	C	=	18.400 in^3
Weight	=	24.930 plf	S yy	=	10.800 in^3			
			R yy	=	2.080 in			

Ycg	=	0.000 in

Sketches



Steel Beam

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

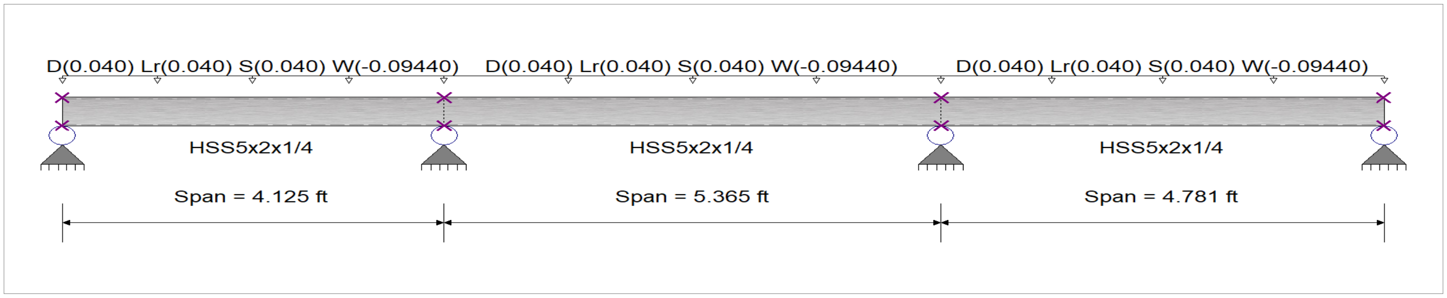
DESCRIPTION: Drive-Thru Cantilever Canopy - Edge Beam Grid A

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.04720 ksf, Tributary Width = 2.0 ft, (Roof)

Load for Span Number 2
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.04720 ksf, Tributary Width = 2.0 ft, (Roof)

Load for Span Number 3
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.04720 ksf, Tributary Width = 2.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.023** : 1	Maximum Shear Stress Ratio =	**0.007** : 1
Section used for this span	**HSS5x2x1/4**	Section used for this span	**HSS5x2x1/4**
Ma : Applied	0.241 k-ft	Va : Applied	0.2668 k
Mn / Omega : Allowable	10.654 k-ft	Vn/Omega : Allowable	36.005 k
Load Combination	+D+Lr	Load Combination	+D+Lr
		Location of maximum on span	5.365 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Upward Transient Deflection	-0.002 in	Ratio =	26,679	>=360	Span: 3 : W Only
Max Downward Total Deflection	0.002 in	Ratio =	27826	>=180	Span: 3 : +D+Lr
Max Upward Total Deflection	-0.001 in	Ratio =	95637	>=180	Span: 3 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 4.13 ft	1	0.010	0.004	0.06	-0.11	0.11	17.79	10.65	2.13	1.00	0.13	60.13	36.00
Dsgn. L = 5.37 ft	2	0.013	0.004	0.06	-0.13	0.13	17.79	10.65	2.61	1.00	0.15	60.13	36.00
Dsgn. L = 4.78 ft	3	0.013	0.004	0.08	-0.13	0.13	17.79	10.65	1.90	1.00	0.15	60.13	36.00
+D+Lr													
Dsgn. L = 4.13 ft	1	0.019	0.007	0.11	-0.20	0.20	17.79	10.65	2.13	1.00	0.24	60.13	36.00
Dsgn. L = 5.37 ft	2	0.023	0.007	0.11	-0.24	0.24	17.79	10.65	2.61	1.00	0.27	60.13	36.00
Dsgn. L = 4.78 ft	3	0.023	0.007	0.15	-0.24	0.24	17.79	10.65	1.90	1.00	0.27	60.13	36.00
+D+S													
Dsgn. L = 4.13 ft	1	0.019	0.007	0.11	-0.20	0.20	17.79	10.65	2.13	1.00	0.24	60.13	36.00
Dsgn. L = 5.37 ft	2	0.023	0.007	0.11	-0.24	0.24	17.79	10.65	2.61	1.00	0.27	60.13	36.00
Dsgn. L = 4.78 ft	3	0.023	0.007	0.15	-0.24	0.24	17.79	10.65	1.90	1.00	0.27	60.13	36.00
+D+0.750Lr													
Dsgn. L = 4.13 ft	1	0.017	0.006	0.09	-0.18	0.18	17.79	10.65	2.13	1.00	0.21	60.13	36.00

Steel Beam

DESCRIPTION: Drive-Thru Cantilever Canopy - Edge Beam Grid A

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 5.37 ft	2	0.020	0.007	0.09	-0.21	0.21	17.79	10.65	2.61	1.00	0.24	60.13	36.00
Dsgn. L = 4.78 ft	3	0.020	0.007	0.14	-0.21	0.21	17.79	10.65	1.90	1.00	0.24	60.13	36.00
+D+0.750S													
Dsgn. L = 4.13 ft	1	0.017	0.006	0.09	-0.18	0.18	17.79	10.65	2.13	1.00	0.21	60.13	36.00
Dsgn. L = 5.37 ft	2	0.020	0.007	0.09	-0.21	0.21	17.79	10.65	2.61	1.00	0.24	60.13	36.00
Dsgn. L = 4.78 ft	3	0.020	0.007	0.14	-0.21	0.21	17.79	10.65	1.90	1.00	0.24	60.13	36.00
+D+0.60W													
Dsgn. L = 4.13 ft	1	0.001	0.000	0.01	-0.01	0.01	17.79	10.65	2.13	1.00	0.02	60.13	36.00
Dsgn. L = 5.37 ft	2	0.002	0.001	0.02	-0.01	0.02	17.79	10.65	2.61	1.00	0.02	60.13	36.00
Dsgn. L = 4.78 ft	3	0.002	0.001	0.02	-0.01	0.02	17.79	10.65	1.90	1.00	0.02	60.13	36.00
+D+0.750Lr+0.450W													
Dsgn. L = 4.13 ft	1	0.008	0.003	0.04	-0.08	0.08	17.79	10.65	2.13	1.00	0.10	60.13	36.00
Dsgn. L = 5.37 ft	2	0.010	0.003	0.04	-0.10	0.10	17.79	10.65	2.61	1.00	0.11	60.13	36.00
Dsgn. L = 4.78 ft	3	0.010	0.003	0.06	-0.10	0.10	17.79	10.65	1.90	1.00	0.11	60.13	36.00
+D+0.750S+0.450W													
Dsgn. L = 4.13 ft	1	0.008	0.003	0.04	-0.08	0.08	17.79	10.65	2.13	1.00	0.10	60.13	36.00
Dsgn. L = 5.37 ft	2	0.010	0.003	0.04	-0.10	0.10	17.79	10.65	2.61	1.00	0.11	60.13	36.00
Dsgn. L = 4.78 ft	3	0.010	0.003	0.06	-0.10	0.10	17.79	10.65	1.90	1.00	0.11	60.13	36.00
+0.60D+0.60W													
Dsgn. L = 4.13 ft	1	0.005	0.002	0.06	-0.03	0.06	17.79	10.65	2.13	1.00	0.07	60.13	36.00
Dsgn. L = 5.37 ft	2	0.007	0.002	0.07	-0.03	0.07	17.79	10.65	2.61	1.00	0.08	60.13	36.00
Dsgn. L = 4.78 ft	3	0.007	0.002	0.07	-0.04	0.07	17.79	10.65	1.90	1.00	0.08	60.13	36.00
+0.60D													
Dsgn. L = 4.13 ft	1	0.006	0.002	0.04	-0.07	0.07	17.79	10.65	2.13	1.00	0.08	60.13	36.00
Dsgn. L = 5.37 ft	2	0.008	0.002	0.04	-0.08	0.08	17.79	10.65	2.61	1.00	0.09	60.13	36.00
Dsgn. L = 4.78 ft	3	0.008	0.002	0.05	-0.08	0.08	17.79	10.65	1.90	1.00	0.09	60.13	36.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.0011	1.733
W Only	2	0.0000	5.258	W Only	-0.0014	2.611
	3	0.0000	5.258	W Only	-0.0022	2.741

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions	0.138	0.470	0.517	0.166
Max Upward from Load Combinations	0.138	0.470	0.517	0.166
Max Upward from Load Cases	0.077	0.262	0.289	0.093
Max Downward from all Load Conditions (Resis	-0.144	-0.490	-0.540	-0.173
Max Downward from Load Combinations (Resi:	-0.040	-0.137	-0.151	-0.048
Max Downward from Load Cases (Resisting Up	-0.144	-0.490	-0.540	-0.173
D Only	0.077	0.262	0.289	0.093
+D+Lr	0.138	0.470	0.517	0.166
+D+S	0.138	0.470	0.517	0.166
+D+0.750Lr	0.123	0.418	0.460	0.148
+D+0.750S	0.123	0.418	0.460	0.148
+D+0.60W	-0.009	-0.032	-0.035	-0.011
+D+0.750Lr+0.450W	0.058	0.198	0.217	0.070
+D+0.750S+0.450W	0.058	0.198	0.217	0.070
+0.60D+0.60W	-0.040	-0.137	-0.151	-0.048
+0.60D	0.046	0.157	0.173	0.056
Lr Only	0.061	0.208	0.229	0.073
S Only	0.061	0.208	0.229	0.073
W Only	-0.144	-0.490	-0.540	-0.173

Steel Beam

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

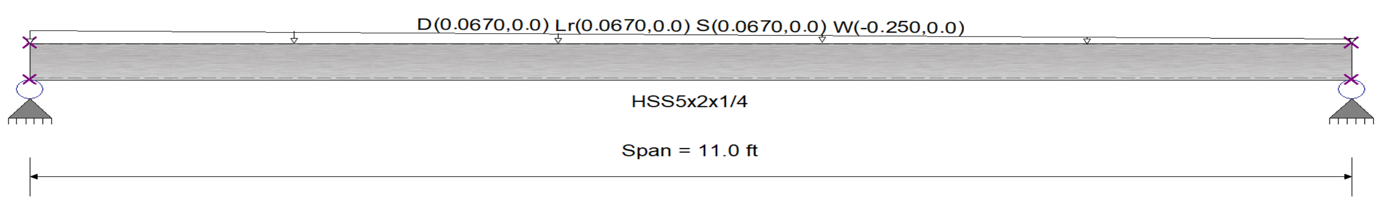
DESCRIPTION: Drive-Thru Cantilever Canopy - Edge Beam Angled

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Varying Uniform Load : D= 0.0670->0.0, Lr= 0.0670->0.0, S= 0.0670->0.0, W= -0.250->0.0 k/ft, Extent = 0.0 -->> 11.0 ft, Trib Width = 1.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.112** : 1	Maximum Shear Stress Ratio =	**0.015** : 1
Section used for this span	**HSS5x2x1/4**	Section used for this span	**HSS5x2x1/4**
Ma : Applied	1.196 k-ft	Va : Applied	0.5491 k
Mn / Omega : Allowable	10.654 k-ft	Vn/Omega : Allowable	36.005 k
Load Combination	+D+Lr	Load Combination	+D+Lr
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.047 in	Ratio = 2,784	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.177 in	Ratio = 746	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.110 in	Ratio = 1204	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.069 in	Ratio = 1918	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 11.00 ft	1	0.063	0.008	0.68		0.68	17.79	10.65	1.15	1.00	0.30	60.13	36.00
+D+Lr													
Dsgn. L = 11.00 ft	1	0.112	0.015	1.20		1.20	17.79	10.65	1.15	1.00	0.55	60.13	36.00
+D+S													
Dsgn. L = 11.00 ft	1	0.112	0.015	1.20		1.20	17.79	10.65	1.15	1.00	0.55	60.13	36.00
+D+0.750Lr													
Dsgn. L = 11.00 ft	1	0.100	0.014	1.07		1.07	17.79	10.65	1.15	1.00	0.49	60.13	36.00
+D+0.750S													
Dsgn. L = 11.00 ft	1	0.100	0.014	1.07		1.07	17.79	10.65	1.15	1.00	0.49	60.13	36.00
+D+0.60W													
Dsgn. L = 11.00 ft	1	0.046	0.007		-0.49	0.49	17.79	10.65	1.18	1.00	0.25	60.13	36.00
+D+0.750Lr+0.450W													
Dsgn. L = 11.00 ft	1	0.018	0.002	0.20		0.20	17.79	10.65	1.14	1.00	0.08	60.13	36.00
+D+0.750S+0.450W													
Dsgn. L = 11.00 ft	1	0.018	0.002	0.20		0.20	17.79	10.65	1.14	1.00	0.08	60.13	36.00
+0.60D+0.60W													
Dsgn. L = 11.00 ft	1	0.071	0.010		-0.76	0.76	17.79	10.65	1.17	1.00	0.37	60.13	36.00
+0.60D													
Dsgn. L = 11.00 ft	1	0.038	0.005	0.41		0.41	17.79	10.65	1.15	1.00	0.18	60.13	36.00

Steel Beam

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Drive-Thru Cantilever Canopy - Edge Beam Angled

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.1769	5.311

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2		
Max Upward from all Load Conditions	0.549	0.303	-0.540	-0.173
Max Upward from Load Combinations	0.549	0.303	-0.540	-0.173
Max Upward from Load Cases	0.303	0.181	-0.540	-0.173
Max Downward from all Load Conditions (Resis	-0.917	-0.458	-0.540	-0.173
Max Downward from Load Combinations (Resi:	-0.368	-0.167	-0.540	-0.173
Max Downward from Load Cases (Resisting Up	-0.917	-0.458	-0.540	-0.173
D Only	0.303	0.181	-0.540	-0.173
+D+Lr	0.549	0.303	-0.540	-0.173
+D+S	0.549	0.303	-0.540	-0.173
+D+0.750Lr	0.488	0.273	-0.540	-0.173
+D+0.750S	0.488	0.273	-0.540	-0.173
+D+0.60W	-0.247	-0.094	-0.540	-0.173
+D+0.750Lr+0.450W	0.075	0.067	-0.540	-0.173
+D+0.750S+0.450W	0.075	0.067	-0.540	-0.173
+0.60D+0.60W	-0.368	-0.167	-0.540	-0.173
+0.60D	0.182	0.108	-0.540	-0.173
Lr Only	0.246	0.123	-0.540	-0.173
S Only	0.246	0.123	-0.540	-0.173
W Only	-0.917	-0.458	-0.540	-0.173

Steel Beam

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

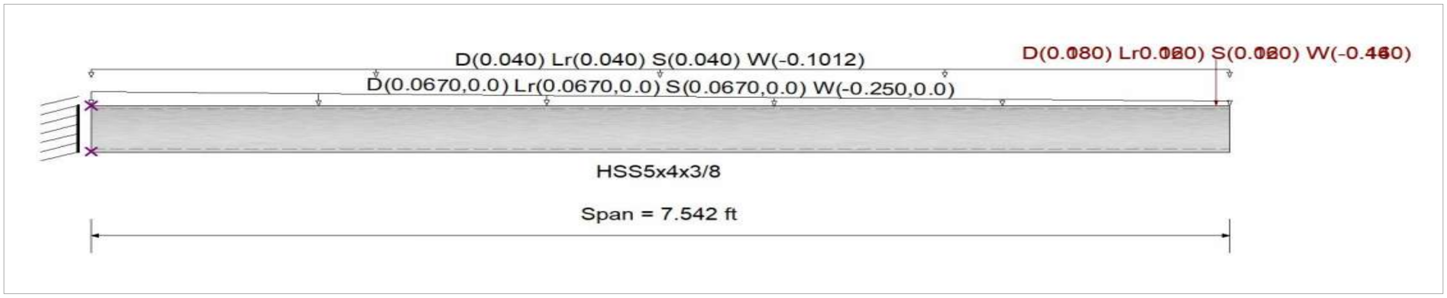
DESCRIPTION: Drive-Thru Cantilever Canopy - Cantilever Beam Grid 2

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1

Varying Uniform Load : D= 0.0670->0.0, Lr= 0.0670->0.0, S= 0.0670->0.0, W= -0.250->0.0 k/ft, Extent = 0.0 -->> 7.542 ft, Trib Width = 1.0 ft, (Roof)

Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05060 ksf, Tributary Width = 2.0 ft, (Roof)

Point Load : D = 0.180, Lr = 0.120, S = 0.120, W = -0.460 k @ 7.452 ft, (Angled Beam)

Point Load : D = 0.080, Lr = 0.060, S = 0.060, W = -0.140 k @ 7.452 ft, (Edge Beam)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.435 : 1	Maximum Shear Stress Ratio =	0.047 : 1
Section used for this span	HSS5x4x3/8	Section used for this span	HSS5x4x3/8
Ma : Applied	9.719 k-ft	Va : Applied	2.306 k
Mn / Omega : Allowable	22.355 k-ft	Vn/Omega : Allowable	49.566 k
Load Combination	W Only	Load Combination	W Only
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.162 in	Ratio =	1,119	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.387 in	Ratio =	467	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.168 in	Ratio =	1078	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination			Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length		Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only														
Dsgn. L =	7.54 ft	1	0.191	0.019		-4.27	4.27	37.33	22.36	1.00	1.00	0.96	82.78	49.57
+D+Lr														
Dsgn. L =	7.54 ft	1	0.330	0.034		-7.39	7.39	37.33	22.36	1.00	1.00	1.70	82.78	49.57
+D+S														
Dsgn. L =	7.54 ft	1	0.330	0.034		-7.39	7.39	37.33	22.36	1.00	1.00	1.70	82.78	49.57
+D+0.750Lr														
Dsgn. L =	7.54 ft	1	0.296	0.031		-6.61	6.61	37.33	22.36	1.00	1.00	1.51	82.78	49.57
+D+0.750S														
Dsgn. L =	7.54 ft	1	0.296	0.031		-6.61	6.61	37.33	22.36	1.00	1.00	1.51	82.78	49.57
+D+0.60W														
Dsgn. L =	7.54 ft	1	0.070	0.008	1.56		1.56	37.33	22.36	1.00	1.00	0.42	82.78	49.57
+D+0.750Lr+0.450W														

DESCRIPTION: Drive-Thru Cantilever Canopy - Cantilever Beam Grid 2

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 7.54 ft	1	0.100	0.010		-2.24	2.24	37.33	22.36	1.00	1.00	0.48	82.78	49.57
+D+0.750S+0.450W													
Dsgn. L = 7.54 ft	1	0.100	0.010		-2.24	2.24	37.33	22.36	1.00	1.00	0.48	82.78	49.57
+0.60D+0.60W													
Dsgn. L = 7.54 ft	1	0.146	0.016	3.27		3.27	37.33	22.36	1.00	1.00	0.81	82.78	49.57
+0.60D													
Dsgn. L = 7.54 ft	1	0.115	0.012		-2.56	2.56	37.33	22.36	1.00	1.00	0.58	82.78	49.57
Lr Only													
Dsgn. L = 7.54 ft	1	0.139	0.015		-3.11	3.11	37.33	22.36	1.00	1.00	0.73	82.78	49.57
W Only													
Dsgn. L = 7.54 ft	1	0.435	0.047	9.72		9.72	37.33	22.36	1.00	1.00	2.31	82.78	49.57

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.3874	7.542		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	
Max Upward from all Load Conditions	1.698	-0.540	-0.173
Max Upward from Load Combinations	1.698	-0.540	-0.173
Max Upward from Load Cases	0.964	-0.540	-0.173
Max Downward from all Load Conditions (Resis	-2.306	-0.540	-0.173
Max Downward from Load Combinations (Resi:	-0.805	-0.540	-0.173
Max Downward from Load Cases (Resisting Up	-2.306	-0.540	-0.173
D Only	0.964	-0.540	-0.173
+D+Lr	1.698	-0.540	-0.173
+D+S	1.698	-0.540	-0.173
+D+0.750Lr	1.515	-0.540	-0.173
+D+0.750S	1.515	-0.540	-0.173
+D+0.60W	-0.420	-0.540	-0.173
+D+0.750Lr+0.450W	0.477	-0.540	-0.173
+D+0.750S+0.450W	0.477	-0.540	-0.173
+0.60D+0.60W	-0.805	-0.540	-0.173
+0.60D	0.578	-0.540	-0.173
Lr Only	0.734	-0.540	-0.173
S Only	0.734	-0.540	-0.173
W Only	-2.306	-0.540	-0.173

Steel Beam

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

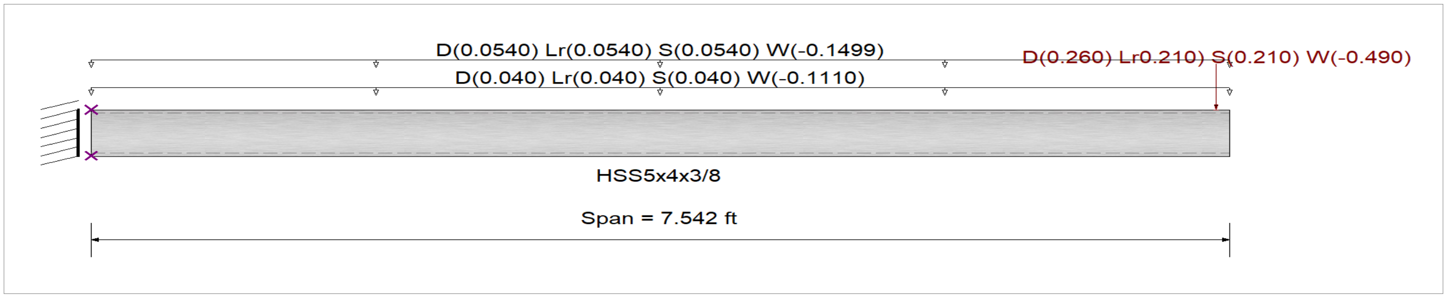
DESCRIPTION: Drive-Thru Cantilever Canopy - Cantilever Beam Grid 3

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05550 ksf, Tributary Width = 2.0 ft, (Roof)

Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05550 ksf, Tributary Width = 2.70 ft, (Roof)

Point Load : D = 0.260, Lr = 0.210, S = 0.210, W = -0.490 k @ 7.452 ft, (Edge Beam)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.495 : 1	Maximum Shear Stress Ratio =	0.050 : 1
Section used for this span	HSS5x4x3/8	Section used for this span	HSS5x4x3/8
Ma : Applied	11.070 k-ft	Va : Applied	2.457 k
Mn / Omega : Allowable	22.355 k-ft	Vn/Omega : Allowable	49.566 k
Load Combination	W Only	Load Combination	W Only
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.224 in	Ratio =	807	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.499 in	Ratio =	363	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.183 in	Ratio =	990	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 7.54 ft	1	0.231	0.023		-5.17	5.17	37.33	22.36	1.00	1.00	1.12	82.78	49.57
+D+Lr													
Dsgn. L = 7.54 ft	1	0.421	0.041		-9.41	9.41	37.33	22.36	1.00	1.00	2.04	82.78	49.57
+D+S													
Dsgn. L = 7.54 ft	1	0.421	0.041		-9.41	9.41	37.33	22.36	1.00	1.00	2.04	82.78	49.57
+D+0.750Lr													
Dsgn. L = 7.54 ft	1	0.374	0.036		-8.35	8.35	37.33	22.36	1.00	1.00	1.81	82.78	49.57
+D+0.750S													
Dsgn. L = 7.54 ft	1	0.374	0.036		-8.35	8.35	37.33	22.36	1.00	1.00	1.81	82.78	49.57
+D+0.60W													
Dsgn. L = 7.54 ft	1	0.066	0.007	1.47		1.47	37.33	22.36	1.00	1.00	0.36	82.78	49.57
+D+0.750Lr+0.450W													
Dsgn. L = 7.54 ft	1	0.151	0.014		-3.37	3.37	37.33	22.36	1.00	1.00	0.70	82.78	49.57
+D+0.750S+0.450W													
Dsgn. L = 7.54 ft	1	0.151	0.014		-3.37	3.37	37.33	22.36	1.00	1.00	0.70	82.78	49.57
+0.60D+0.60W													

Steel Beam

DESCRIPTION: Drive-Thru Cantilever Canopy - Cantilever Beam Grid 3

Maximum Forces & Stresses for Load Combinations

Load Combination			Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length		Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L =	7.54 ft	1	0.158	0.016	3.54		3.54	37.33	22.36	1.00	1.00	0.80	82.78	49.57
+0.60D														
Dsgn. L =	7.54 ft	1	0.139	0.014		-3.10	3.10	37.33	22.36	1.00	1.00	0.67	82.78	49.57
Lr Only														
Dsgn. L =	7.54 ft	1	0.190	0.019		-4.24	4.24	37.33	22.36	1.00	1.00	0.92	82.78	49.57
W Only														
Dsgn. L =	7.54 ft	1	0.495	0.050	11.07		11.07	37.33	22.36	1.00	1.00	2.46	82.78	49.57

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.4985	7.542		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2		
Max Upward from all Load Conditions	2.037		-0.540	-0.173
Max Upward from Load Combinations	2.037		-0.540	-0.173
Max Upward from Load Cases	1.118		-0.540	-0.173
Max Downward from all Load Conditions (Resis	-2.457		-0.540	-0.173
Max Downward from Load Combinations (Resi:	-0.803		-0.540	-0.173
Max Downward from Load Cases (Resisting Up	-2.457		-0.540	-0.173
D Only	1.118		-0.540	-0.173
+D+Lr	2.037		-0.540	-0.173
+D+S	2.037		-0.540	-0.173
+D+0.750Lr	1.808		-0.540	-0.173
+D+0.750S	1.808		-0.540	-0.173
+D+0.60W	-0.356		-0.540	-0.173
+D+0.750Lr+0.450W	0.702		-0.540	-0.173
+D+0.750S+0.450W	0.702		-0.540	-0.173
+0.60D+0.60W	-0.803		-0.540	-0.173
+0.60D	0.671		-0.540	-0.173
Lr Only	0.919		-0.540	-0.173
S Only	0.919		-0.540	-0.173
W Only	-2.457		-0.540	-0.173

Steel Beam

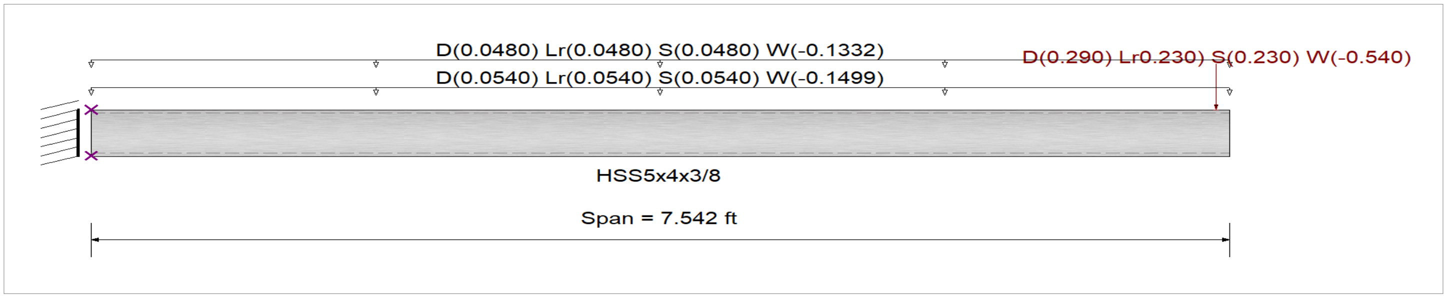
DESCRIPTION: Drive-Thru Cantilever Canopy - Cantilever Beam Grid 4

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05550 ksf, Tributary Width = 2.70 ft, (Roof)

Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05550 ksf, Tributary Width = 2.40 ft, (Roof)

Point Load : D = 0.290, Lr = 0.230, S = 0.230, W = -0.540 k @ 7.452 ft, (Edge Beam)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.540** : 1	Maximum Shear Stress Ratio =	**0.054** : 1
Section used for this span	**HSS5x4x3/8**	Section used for this span	**HSS5x4x3/8**
Ma : Applied	12.074 k-ft	Va : Applied	2.675 k
Mn / Omega : Allowable	22.355 k-ft	Vn/Omega : Allowable	49.566 k
Load Combination	W Only	Load Combination	W Only
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.244 in	Ratio =	740	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.543 in	Ratio =	333	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.200 in	Ratio =	905	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 7.54 ft	1	0.252	0.024		-5.63	5.63	37.33	22.36	1.00	1.00	1.21	82.78	49.57
+D+Lr													
Dsgn. L = 7.54 ft	1	0.458	0.045		-10.24	10.24	37.33	22.36	1.00	1.00	2.21	82.78	49.57
+D+S													
Dsgn. L = 7.54 ft	1	0.458	0.045		-10.24	10.24	37.33	22.36	1.00	1.00	2.21	82.78	49.57
+D+0.750Lr													
Dsgn. L = 7.54 ft	1	0.406	0.040		-9.09	9.09	37.33	22.36	1.00	1.00	1.96	82.78	49.57
+D+0.750S													
Dsgn. L = 7.54 ft	1	0.406	0.040		-9.09	9.09	37.33	22.36	1.00	1.00	1.96	82.78	49.57
+D+0.60W													
Dsgn. L = 7.54 ft	1	0.072	0.008	1.62		1.62	37.33	22.36	1.00	1.00	0.40	82.78	49.57
+D+0.750Lr+0.450W													
Dsgn. L = 7.54 ft	1	0.163	0.015		-3.65	3.65	37.33	22.36	1.00	1.00	0.75	82.78	49.57
+D+0.750S+0.450W													
Dsgn. L = 7.54 ft	1	0.163	0.015		-3.65	3.65	37.33	22.36	1.00	1.00	0.75	82.78	49.57
+0.60D+0.60W													

Steel Beam

DESCRIPTION: Drive-Thru Cantilever Canopy - Cantilever Beam Grid 4

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length		Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
Dsgn. L =	7.54 ft	1	0.173	0.018	3.87		3.87	37.33	22.36	1.00	1.00	0.88	82.78	49.57
+0.60D														
Dsgn. L =	7.54 ft	1	0.151	0.015		-3.38	3.38	37.33	22.36	1.00	1.00	0.73	82.78	49.57
Lr Only														
Dsgn. L =	7.54 ft	1	0.206	0.020		-4.61	4.61	37.33	22.36	1.00	1.00	1.00	82.78	49.57
W Only														
Dsgn. L =	7.54 ft	1	0.540	0.054	12.07		12.07	37.33	22.36	1.00	1.00	2.67	82.78	49.57

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.5433	7.542		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	
Max Upward from all Load Conditions	2.208	-0.540	-0.173
Max Upward from Load Combinations	2.208	-0.540	-0.173
Max Upward from Load Cases	1.209	-0.540	-0.173
Max Downward from all Load Conditions (Resis	-2.675	-0.540	-0.173
Max Downward from Load Combinations (Resi:	-0.880	-0.540	-0.173
Max Downward from Load Cases (Resisting Up	-2.675	-0.540	-0.173
D Only	1.209	-0.540	-0.173
+D+Lr	2.208	-0.540	-0.173
+D+S	2.208	-0.540	-0.173
+D+0.750Lr	1.958	-0.540	-0.173
+D+0.750S	1.958	-0.540	-0.173
+D+0.60W	-0.396	-0.540	-0.173
+D+0.750Lr+0.450W	0.755	-0.540	-0.173
+D+0.750S+0.450W	0.755	-0.540	-0.173
+0.60D+0.60W	-0.880	-0.540	-0.173
+0.60D	0.725	-0.540	-0.173
Lr Only	0.999	-0.540	-0.173
S Only	0.999	-0.540	-0.173
W Only	-2.675	-0.540	-0.173

Steel Beam

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

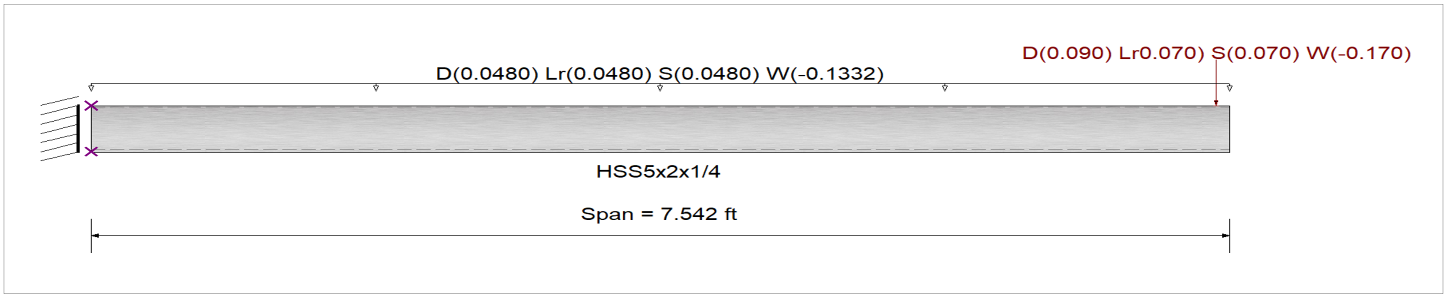
DESCRIPTION: Drive-Thru Cantilever Canopy - Cantilever Beam Grid 5,

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020, W = -0.05550 ksf, Tributary Width = 2.40 ft, (Roof)

Point Load : D = 0.090, Lr = 0.070, S = 0.070, W = -0.170 k @ 7.452 ft, (Edge Beam)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.474 : 1	Maximum Shear Stress Ratio =	0.033 : 1
Section used for this span	HSS5x2x1/4	Section used for this span	HSS5x2x1/4
Ma : Applied	5.055 k-ft	Va : Applied	1.175 k
Mn / Omega : Allowable	10.654 k-ft	Vn/Omega : Allowable	36.005 k
Load Combination	W Only	Load Combination	W Only
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.215 in	Ratio = 840	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio = 0	<360	n/a
Max Downward Total Deflection	0.482 in	Ratio = 375	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.183 in	Ratio = 989	>=180	Span: 1 : +0.60D+0.60W

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length		Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
D Only														
Dsgn. L =	7.54 ft	1	0.219	0.015		-2.33	2.33	17.79	10.65	1.00	1.00	0.53	60.13	36.00
+D+Lr														
Dsgn. L =	7.54 ft	1	0.396	0.027		-4.22	4.22	17.79	10.65	1.00	1.00	0.96	60.13	36.00
+D+S														
Dsgn. L =	7.54 ft	1	0.396	0.027		-4.22	4.22	17.79	10.65	1.00	1.00	0.96	60.13	36.00
+D+0.750Lr														
Dsgn. L =	7.54 ft	1	0.352	0.024		-3.75	3.75	17.79	10.65	1.00	1.00	0.86	60.13	36.00
+D+0.750S														
Dsgn. L =	7.54 ft	1	0.352	0.024		-3.75	3.75	17.79	10.65	1.00	1.00	0.86	60.13	36.00
+D+0.60W														
Dsgn. L =	7.54 ft	1	0.066	0.005	0.70		0.70	17.79	10.65	1.00	1.00	0.17	60.13	36.00
+D+0.750Lr+0.450W														
Dsgn. L =	7.54 ft	1	0.138	0.009		-1.48	1.48	17.79	10.65	1.00	1.00	0.33	60.13	36.00
+D+0.750S+0.450W														
Dsgn. L =	7.54 ft	1	0.138	0.009		-1.48	1.48	17.79	10.65	1.00	1.00	0.33	60.13	36.00
+0.60D+0.60W														
Dsgn. L =	7.54 ft	1	0.153	0.011	1.63		1.63	17.79	10.65	1.00	1.00	0.39	60.13	36.00
+0.60D														
Dsgn. L =	7.54 ft	1	0.131	0.009		-1.40	1.40	17.79	10.65	1.00	1.00	0.32	60.13	36.00

Steel Beam

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Drive-Thru Cantilever Canopy - Cantilever Beam Grid 5,

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Lr Only													
Dsgn. L = 7.54 ft	1	0.177	0.012		-1.89	1.89	17.79	10.65	1.00	1.00	0.43	60.13	36.00
W Only													
Dsgn. L = 7.54 ft	1	0.474	0.033	5.06		5.06	17.79	10.65	1.00	1.00	1.17	60.13	36.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.4824	7.542		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	
Max Upward from all Load Conditions	0.963	-0.540	-0.173
Max Upward from Load Combinations	0.963	-0.540	-0.173
Max Upward from Load Cases	0.531	-0.540	-0.173
Max Downward from all Load Conditions (Resis	-1.175	-0.540	-0.173
Max Downward from Load Combinations (Resi:	-0.386	-0.540	-0.173
Max Downward from Load Cases (Resisting Up	-1.175	-0.540	-0.173
D Only	0.531	-0.540	-0.173
+D+Lr	0.963	-0.540	-0.173
+D+S	0.963	-0.540	-0.173
+D+0.750Lr	0.855	-0.540	-0.173
+D+0.750S	0.855	-0.540	-0.173
+D+0.60W	-0.173	-0.540	-0.173
+D+0.750Lr+0.450W	0.327	-0.540	-0.173
+D+0.750S+0.450W	0.327	-0.540	-0.173
+0.60D+0.60W	-0.386	-0.540	-0.173
+0.60D	0.319	-0.540	-0.173
Lr Only	0.432	-0.540	-0.173
S Only	0.432	-0.540	-0.173
W Only	-1.175	-0.540	-0.173

Steel Column

Project File: 24-0121 - afc lee's summit.ec6

DESCRIPTION: Drive-Thru Cantilever Canopy - Column @ B-5 w/ Front of House Beam

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS5-1/2x5-1/2x3/8**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 15.875 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 15.875 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 15.875 ft, K = 2.1

Applied Loads

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

Column self weight included : 395.764 lbs * Dead Load Factor
AXIAL LOADS . . .
Canopy Beam Grid 5: Axial Load at 10.50 ft, D = 0.530, LR = 0.430, S = 0.430, W = -1.170 k
Front of House Beam Grid 5: Axial Load at 12.0 ft, D = 4.820, LR = 4.280, S = 4.280, W = -9.980 k
BENDING LOADS . . .
Beam Grid 5: Moment acting about X-X axis at 10.50 ft, D = -2.340, LR = -1.90, S = -1.90, W = 5.10 k-ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.4794**	: 1
Load Combination	+D+Lr	
Location of max.above base	10.441	ft
At maximum location values are . . .		
Pa : Axial	10.456	k
Pn / Omega : Allowabl	27.956	k
Ma-x : Applied	-2.789	k-ft
Mn-x / Omega : Allowable	23.533	k-ft
Ma-y : Applied	0.0	k-ft
Mn-y / Omega : Allowable	23.533	k-ft
PASS Maximum Shear Stress Rati	**0.006644**	: 1
Load Combination	+D+Lr	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.2671	k
Vn / Omega : Allowable	40.202	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.3213 k
Bottom along Y-Y	0.3213 k

Maximum Load Deflections . . .

Along Y-Y 0.08886 in at 7.458ft above base
for load combination : W Only

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.264	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.004	PASS	0.00 ft
+D+Lr	0.479	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.007	PASS	0.00 ft
+D+S	0.479	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.007	PASS	0.00 ft
+D+0.750Lr	0.425	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.006	PASS	0.00 ft
+D+0.750S	0.425	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.006	PASS	0.00 ft
+D+0.60W	0.037	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.001	PASS	0.00 ft
+D+0.750Lr+0.450W	0.152	PASS	0.00 ft	1.60	1.00	192.33	192.33	0.002	PASS	0.00 ft
+D+0.750S+0.450W	0.152	PASS	0.00 ft	1.60	1.00	192.33	192.33	0.002	PASS	0.00 ft
+0.60D+0.60W	0.104	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.003	PASS	0.00 ft
+0.60D	0.123	PASS	0.00 ft	1.60	1.00	192.33	192.33	0.002	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X @ Top	Y-Y Axis Reaction @ Base	Y-Y @ Top	Mx - End Moments k-ft @ Base	Mx @ Top	My - End Moments @ Base	My @ Top
D Only	5.746			-0.147	0.147				
+D+Lr	10.456			-0.267	0.267				
+D+S	10.456			-0.267	0.267				

Steel Column

Project File: 24-0121 - afc lee's summit.ec6

DESCRIPTION: Drive-Thru Cantilever Canopy - Column @ B-5 w/ Front of House Beam

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top (k)	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top (k-ft)	My - End Moments @ Base	My - End Moments @ Top
+D+0.750Lr	9.278			-0.237	0.237				
+D+0.750S	9.278			-0.237	0.237				
+D+0.60W	-0.944			0.045	-0.045				
+D+0.750Lr+0.450W	4.261			-0.093	0.093				
+D+0.750S+0.450W	4.261			-0.093	0.093				
+0.60D+0.60W	-3.243			0.104	-0.104				
+0.60D	3.447			-0.088	0.088				
Lr Only	4.710			-0.120	0.120				
S Only	4.710			-0.120	0.120				
W Only	-11.150			0.321	-0.321				

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top (k)	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top (k-ft)	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	10.456			-0.267	0.267				
"	Minimum	-11.150			0.321	-0.321				
Reaction, X-X Axis Base	Maximum	5.746			-0.147	0.147				
"	Minimum	5.746			-0.147	0.147				
Reaction, Y-Y Axis Base	Maximum	-11.150			0.321	-0.321				
"	Minimum	10.456			-0.267	0.267				
Reaction, X-X Axis Top	Maximum	5.746			-0.147	0.147				
"	Minimum	5.746			-0.147	0.147				
Reaction, Y-Y Axis Top	Maximum	5.746			-0.147	0.147				
"	Minimum	-0.944			0.045	-0.045				
Moment, X-X Axis Base	Maximum	5.746			-0.147	0.147				
"	Minimum	5.746			-0.147	0.147				
Moment, Y-Y Axis Base	Maximum	5.746			-0.147	0.147				
"	Minimum	5.746			-0.147	0.147				
Moment, X-X Axis Top	Maximum	5.746			-0.147	0.147				
"	Minimum	5.746			-0.147	0.147				
Moment, Y-Y Axis Top	Maximum	5.746			-0.147	0.147				
"	Minimum	5.746			-0.147	0.147				

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	-0.041 in	7.458 ft
+D+Lr	0.0000 in	0.000 ft	-0.074 in	7.458 ft
+D+S	0.0000 in	0.000 ft	-0.074 in	7.458 ft
+D+0.750Lr	0.0000 in	0.000 ft	-0.066 in	7.458 ft
+D+0.750S	0.0000 in	0.000 ft	-0.066 in	7.458 ft
+D+0.60W	0.0000 in	0.000 ft	0.013 in	7.458 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	-0.026 in	7.458 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	-0.026 in	7.458 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.029 in	7.458 ft
+0.60D	0.0000 in	0.000 ft	-0.024 in	7.458 ft
Lr Only	0.0000 in	0.000 ft	-0.033 in	7.458 ft
S Only	0.0000 in	0.000 ft	-0.033 in	7.458 ft
W Only	0.0000 in	0.000 ft	0.089 in	7.458 ft

Steel Section Properties : HSS5-1/2x5-1/2x3/8

Steel Section Properties : HSS5-1/2x5-1/2x3/8

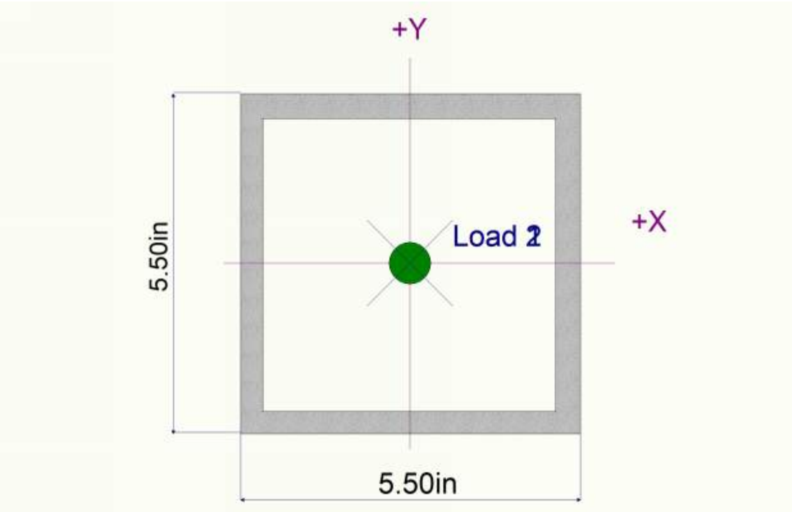
Steel Column

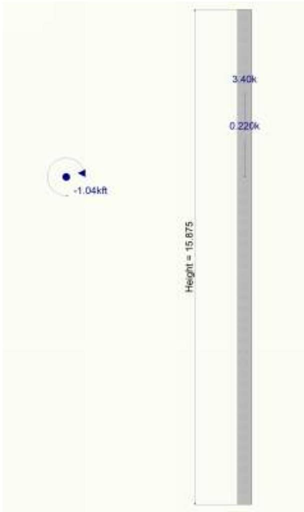
DESCRIPTION: Drive-Thru Cantilever Canopy - Column @ B-5 w/ Front of House Beam

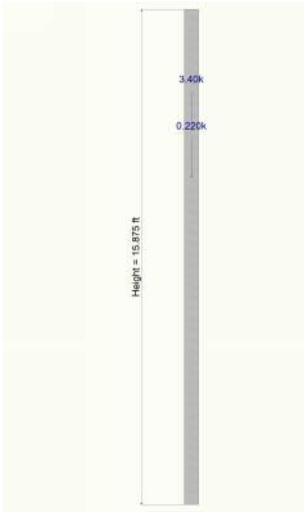
Depth	=	5.500 in	I xx	=	29.70 in^4	J	=	49.000 in^4
Design Thick	=	0.349 in	S xx	=	10.80 in^3			
Width	=	5.500 in	R xx	=	2.080 in			
Wall Thick	=	0.375 in	Zx	=	13.100 in^3			
Area	=	6.880 in^2	I yy	=	29.700 in^4	C	=	18.400 in^3
Weight	=	24.930 plf	S yy	=	10.800 in^3			
			R yy	=	2.080 in			

Ycg = 0.000 in

Sketches







Steel Column

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Drive-Thru Cantilever Canopy - Column B-4 (Max load)

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS5-1/2x5-1/2x3/8**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 15.875 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 15.875 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 15.875 ft, K = 2.1

Applied Loads

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

Column self weight included : 395.764 lbs * Dead Load Factor
AXIAL LOADS . . .
Canopy Beam Grid 3: Axial Load at 10.50 ft, D = 1.210, LR = 1.0, S = 1.0, W = -2.670 k
BENDING LOADS . . .
Beam Grid 4: Moment acting about X-X axis at 10.50 ft, D = -5.630, LR = -4.620, S = -4.620, W = 12.10 k-ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.3859	: 1
	Load Combination	W Only	
	Location of max.above base	10.441	ft
	At maximum location values are . . .		
	Pa : Axial	-2.670	k
	Pn / Omega : Allowabl	27.956	k
	Ma-x : Applied	7.958	k-ft
	Mn-x / Omega : Allowable	23.533	k-ft
	Ma-y : Applied	0.0	k-ft
	Mn-y / Omega : Allowable	23.533	k-ft
PASS	Maximum Shear Stress Rati	0.01896	: 1
	Load Combination	W Only	
	Location of max.above base	0.0	ft
	At maximum location values are . . .		
	Va : Applied	0.7622	k
	Vn / Omega : Allowable	40.202	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.7622 k
Bottom along Y-Y	0.7622 k

Maximum Load Deflections . . .

Along Y-Y 0.2108 in at 7.458ft above base
for load combination : W Only

Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.186	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.009	PASS	0.00 ft
+D+Lr	0.333	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.016	PASS	0.00 ft
+D+S	0.333	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.016	PASS	0.00 ft
+D+0.750Lr	0.296	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.014	PASS	0.00 ft
+D+0.750S	0.296	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.014	PASS	0.00 ft
+D+0.60W	0.046	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.003	PASS	0.00 ft
+D+0.750Lr+0.450W	0.123	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.006	PASS	0.00 ft
+D+0.750S+0.450W	0.123	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.006	PASS	0.00 ft
+0.60D+0.60W	0.120	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.006	PASS	0.00 ft
+0.60D	0.112	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.005	PASS	0.00 ft
Lr Only	0.147	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.007	PASS	0.00 ft
W Only	0.386	PASS	10.44 ft	1.60	1.00	192.33	192.33	0.019	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	1.606			-0.355	0.355				
+D+Lr	2.606			-0.646	0.646				

Steel Column

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Drive-Thru Cantilever Canopy - Column B-4 (Max load)

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments (k-ft) @ Base	@ Top	My - End Moments @ Base	@ Top
+D+S	2.606			-0.646	0.646				
+D+0.750Lr	2.356			-0.573	0.573				
+D+0.750S	2.356			-0.573	0.573				
+D+0.60W	0.004			0.103	-0.103				
+D+0.750Lr+0.450W	1.154			-0.230	0.230				
+D+0.750S+0.450W	1.154			-0.230	0.230				
+0.60D+0.60W	-0.639			0.245	-0.245				
+0.60D	0.963			-0.213	0.213				
Lr Only	1.000			-0.291	0.291				
S Only	1.000			-0.291	0.291				
W Only	-2.670			0.762	-0.762				

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments (k-ft) @ Base	@ Top	My - End Moments @ Base	@ Top
Axial @ Base	Maximum	2.606			-0.646	0.646				
"	Minimum	-2.670			0.762	-0.762				
Reaction, X-X Axis Base	Maximum	1.606			-0.355	0.355				
"	Minimum	1.606			-0.355	0.355				
Reaction, Y-Y Axis Base	Maximum	-2.670			0.762	-0.762				
"	Minimum	2.606			-0.646	0.646				
Reaction, X-X Axis Top	Maximum	1.606			-0.355	0.355				
"	Minimum	1.606			-0.355	0.355				
Reaction, Y-Y Axis Top	Maximum	1.606			-0.355	0.355				
"	Minimum	0.004			0.103	-0.103				
Moment, X-X Axis Base	Maximum	1.606			-0.355	0.355				
"	Minimum	1.606			-0.355	0.355				
Moment, Y-Y Axis Base	Maximum	1.606			-0.355	0.355				
"	Minimum	1.606			-0.355	0.355				
Moment, X-X Axis Top	Maximum	1.606			-0.355	0.355				
"	Minimum	1.606			-0.355	0.355				
Moment, Y-Y Axis Top	Maximum	1.606			-0.355	0.355				
"	Minimum	1.606			-0.355	0.355				

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	-0.098 in	7.458 ft
+D+Lr	0.0000 in	0.000 ft	-0.179 in	7.458 ft
+D+S	0.0000 in	0.000 ft	-0.179 in	7.458 ft
+D+0.750Lr	0.0000 in	0.000 ft	-0.158 in	7.458 ft
+D+0.750S	0.0000 in	0.000 ft	-0.158 in	7.458 ft
+D+0.60W	0.0000 in	0.000 ft	0.028 in	7.458 ft
+D+0.750Lr+0.450W	0.0000 in	0.000 ft	-0.064 in	7.458 ft
+D+0.750S+0.450W	0.0000 in	0.000 ft	-0.064 in	7.458 ft
+0.60D+0.60W	0.0000 in	0.000 ft	0.068 in	7.458 ft
+0.60D	0.0000 in	0.000 ft	-0.059 in	7.458 ft
Lr Only	0.0000 in	0.000 ft	-0.080 in	7.458 ft
S Only	0.0000 in	0.000 ft	-0.080 in	7.458 ft
W Only	0.0000 in	0.000 ft	0.211 in	7.458 ft

Steel Section Properties : HSS5-1/2x5-1/2x3/8

Steel Section Properties : HSS5-1/2x5-1/2x3/8

Steel Column

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Drive-Thru Cantilever Canopy - Column B-4 (Max load)

Depth	=	5.500 in	I xx	=	29.70 in^4	J	=	49.000 in^4
Design Thick	=	0.349 in	S xx	=	10.80 in^3			
Width	=	5.500 in	R xx	=	2.080 in			
Wall Thick	=	0.375 in	Zx	=	13.100 in^3			
Area	=	6.880 in^2	I yy	=	29.700 in^4	C	=	18.400 in^3
Weight	=	24.930 plf	S yy	=	10.800 in^3			
			R yy	=	2.080 in			

Ycg	=	0.000 in

Sketches

Drive Thru Cantilever Canopy - Light Gauge Framing

CFS Version 14.0.0

Analysis: Drive Thru Canopy.cfsa jfarrar
8.3 ft Span Simple Beam METT-ENGR

Rev. Date: 2/23/2024 11:38:32 AM
By: jfarrar
Printed: 2/23/2024 11:38:58 AM

Member Check - AISI S100-16/S1-18, US, ASD

```
Load Combination: D+Lr
Design Parameters at 4.1500 ft:
Lx        8.3000 ft      Ly        8.3000 ft      Lt        8.3000 ft
Kx        1.0000         Ky        1.0000         Kt        1.0000

Section: 400S162-54 Single.cfss
Material Type: A653 SS Grade 33, Fy=33 ksi
Cbx       1.1364         Cby       1.0000         ex        0.0000 in
Cmx       1.0000         Cmy       1.0000         ey        0.0000 in
Braced Flange: Top       kϕ             0 k
Red. Factor, R: 0        Lm        8.3000 ft

Loads:          P         Mx         Vy         My         Vx
               (k)      (k-in)       (k)      (k-in)       (k)
Total       0.0000      8.422     0.0000      0.000     0.0000
Applied     0.0000      8.422     0.0000      0.000     0.0000
Strength    4.1984     10.849     2.6035      2.254     1.8799

Interaction Equations
Eq. H1.2-1    (P, Mx, My)  0.000 + 0.776 + 0.000 = 0.776 <= 1.0
Eq. H2-1         (Mx, Vy)     Sqrt(0.603 + 0.000)= 0.776 <= 1.0
Eq. H2-1         (My, Vx)     Sqrt(0.000 + 0.000)= 0.000 <= 1.0
```

Web Crippling Check - AISI S100-16/S1-18, US, ASD

```
Load Combination: D+Lr
Parameters at 8.3000 ft:
Total Load:       0.33825 k on bottom flange
Total Moment:      0.0000 k-in
Bearing:           2.0000 in
Flange fastened to bearing surface: Yes
Distance from edge of bearing to edge of opposite load: 8.3000 ft

Section: 400S162-54 Single.cfss
Material Type: A653 SS Grade 33, Fy=33 ksi
Applied Load:       0.33825 k on bottom flange
Applied Moment:       0.000 k-in
Distance from edge of bearing to end of member: 0.0000 ft

Part Elem Calculation Type       Pa (k)     Pay (k) Notes
  1    3  Cee, FS-EOF           0.51679     0.51679
                 Web Crippling Strength     0.51679

Web Crippling Check:             0.33825 k      <=    0.51679 k
```

```
Moment Check:                          0.000 k-in   <=     10.849 k-in
Interaction Equations
Eq. H3-1a    (P, M)  0.340 + 0.000 = 0.340 <= 0.782
```

Calculation Details - AISI S100-16/S1-18, US, ASD

Stud element 3

$t=0.0566$ in, $C=4$, $\theta=90$ deg

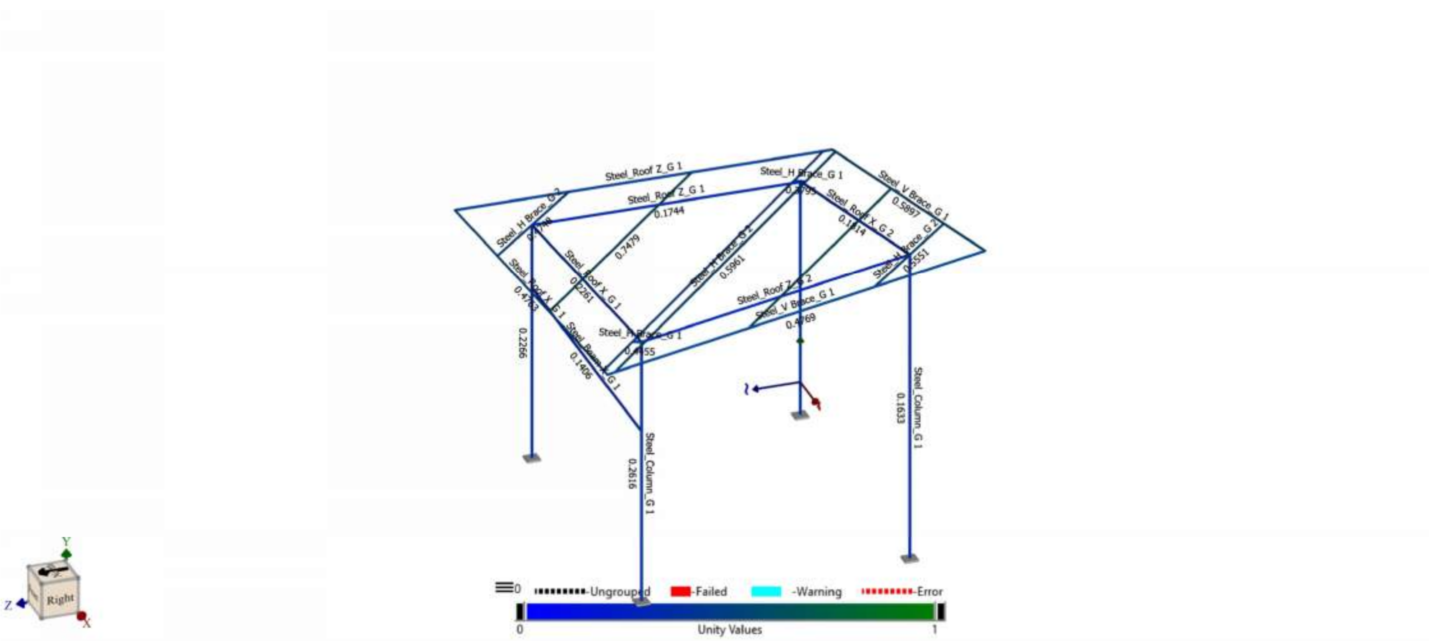
$R=0.0849$ in, $C_r=0.14$

$N=2$ in, $C_n=0.35$

$h=3.717$ in, $C_h=0.02$

$P_n=0.90438$ k — Eq. G5-1

$\Omega_w=1.75$, $\phi_w=0.85$



Bill of Materials: Members

Material	Section	Count	Total Length ft	Total Volume in^3	Total Weight K
ASTM A500 Grade B (Fy = 46ksi)	HSS6X6X.250	1	25.1670	1582.5010	0.4494
ASTM A500 Grade B (Fy = 46ksi)	HSS10X10X.500	4	87.7552	18112.6733	5.1440
ASTM A500 Grade B (Fy = 46ksi)	HSS12X4X.375	11	281.7313	35160.0624	9.9855
ASTM A500 Grade B (Fy = 46ksi)	HSS12X4X.500	3	119.4132	19344.9376	5.4940

Bill of Materials: Members (continued)

Material	Section	Count	Total Length ft	Total Volume in^3	Total Weight K
ASTM A992 Grade 50	W10X19	2	2.0415	137.6772	0.0391

Total Member Weight = 21.112 K

Member:COL001

1. Member Properties	2. Connections	3. Applied Loads
Length: 19.92 ft Shape: HSS10X10X.500 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.1677 K Framing: Column	Start: N005 @ (0, 17.087, 0 ft) - Free End: N001 @ (0, -2.8333, 0 ft) - Fixed	Unif.: (W+X) Force X 0.0169 K/ft (W-X) Force X -0.0169 K/ft (W+Z) Force Z 0.0169 K/ft (W-Z) Force Z -0.0169 K/ft
4. Extreme Forces Axial: -38.18 K (42) -> 8.8292 K (6) My: -17.749 K-ft (60) -> 17.204 K-ft (6) Mz: -29.993 K-ft (6) -> 30.734 K-ft (60) Vy: -1.5057 K (6) -> 1.5429 K (60) Vz: -0.95233 K (62) -> 0.925 K (8) Torsion: -1.0801 K-ft (43) -> 0.37157 K-ft (9)	5. Extreme Deflections (all cases) Total Dy: -0.92342 in (6) -> 0.94624 in (60) Total Dz: -0.54644 in (60) -> 0.52967 in (6) Beam Dy: 0.00508 in (2) -> 0.94624 in (60) Beam Dz: 0.00003 in (58) -> 0.54644 in (60)	

Member Design Checks:COL001

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0653	Pass	38.18 K	585.1 K	3. 1.2D+1.6S+0.5W »-Y	E3-2
Combined Check	0.2563	Pass	0.25632	1	4. 1.2D+W+L+0.5S »-Y	H1-1b
Strong Flexure Check	0.1468	Pass	30.734 K-ft	209.42 K-ft	4. 1.2D+W+L+0.5S »-Y	F7-1
Strong Shear Check	0.0078	Pass	1.5429 K	198.79 K	4. 1.2D+W+L+0.5S »-Y	G4-1
Torsion Shear Check	0.0062	Pass	0.154 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »+Z	H3-1
Weak Flexure Check	0.0848	Pass	-17.749 K-ft	209.42 K-ft	4. 1.2D+W+L+0.5S »-Y	F7-1

Member:COL002

1. Member Properties	2. Connections	3. Applied Loads
Length: 19.851 ft Shape: HSS10X10X.500 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.1636 K Framing: Column	Start: N006 @ (0, 17.018, 21.5 ft) - Free End: N002 @ (0, -2.8333, 21.5 ft) - Fixed	Unif.: (W+X) Force X 0.0169 K/ft (W-X) Force X -0.0169 K/ft (W+Z) Force Z 0.0169 K/ft (W-Z) Force Z -0.0169 K/ft
4. Extreme Forces Axial: -24.081 K (42) -> 7.1701 K (6) My: -17.795 K-ft (60) -> 17.282 K-ft (6) Mz: -26.56 K-ft (59) -> 26.163 K-ft (7) Vy: -3.1979 K (59) -> 3.1231 K (7) Vz: -1.0201 K (62) -> 0.9926 K (8) Torsion: -0.69192 K-ft (6) -> 3.1609 K-ft (42)	5. Extreme Deflections (all cases) Total Dy: -1.0052 in (59) -> 0.9874 in (7) Total Dz: -0.54364 in (60) -> 0.52834 in (6) Beam Dy: 0.00407 in (2) -> 1.0052 in (59) Beam Dz: 0.00025 in (46) -> 0.54364 in (60)	

Member Design Checks:COL002

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0411	Pass	24.081 K	585.9 K	3. 1.2D+1.6S+0.5W »-Y	E3-2
Combined Check	0.2266	Pass	0.22665	1	4. 1.2D+W+L+0.5S »-Y	H1-1b
Strong Flexure Check	0.1268	Pass	-26.56 K-ft	209.42 K-ft	4. 1.2D+W+L+0.5S »+Y	F7-1
Strong Shear Check	0.0161	Pass	-3.1979 K	198.79 K	4. 1.2D+W+L+0.5S »+Y	G4-1
Torsion Shear Check	0.0181	Pass	0.45068 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.0850	Pass	-17.795 K-ft	209.42 K-ft	4. 1.2D+W+L+0.5S »-Y	F7-1

Member:COL003

1. Member Properties	2. Connections	3. Applied Loads
Length: 22.082 ft Shape: HSS10X10X.500 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.2944 K Framing: Column	Start: N008 @ (25.167, 19.249, 21.5 ft) - Free End: N004 @ (25.167, -2.8333, 21.5 ft) - Fixed	Unif.: (W+X) Force X 0.0169 K/ft (W-X) Force X -0.0169 K/ft (W+Z) Force Z 0.0169 K/ft (W-Z) Force Z -0.0169 K/ft
4. Extreme Forces Axial: -38.857 K (42) -> 8.9104 K (6) My: -16.535 K-ft (9) -> 17.277 K-ft (61) Mz: -35.116 K-ft (59) -> 34.455 K-ft (7) Vy: -2.4646 K (59) -> 2.4089 K (7) Vz: -0.99204 K (9) -> 1.0271 K (61) Torsion: -0.85027 K-ft (6) -> 4.516 K-ft (42)	5. Extreme Deflections (all cases) Total Dy: -1.0064 in (59) -> 0.99085 in (7) Total Dz: -0.61129 in (7) -> 0.63895 in (59) Beam Dy: 0.00331 in (2) -> 1.0064 in (59) Beam Dz: 0.00066 in (58) -> 0.63895 in (59)	

Member Design Checks:COL003

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0695	Pass	38.857 K	559.39 K	3. 1.2D+1.6S+0.5W »-Y	E3-2
Combined Check	0.2616	Pass	0.26155	1	4. 1.2D+W+L+0.5S »-Y	H1-1b
Strong Flexure Check	0.1677	Pass	-35.116 K-ft	209.42 K-ft	4. 1.2D+W+L+0.5S »+Y	F7-1
Strong Shear Check	0.0124	Pass	-2.4646 K	198.79 K	4. 1.2D+W+L+0.5S »+Y	G4-1
Torsion Shear Check	0.0259	Pass	0.64389 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.0825	Pass	17.277 K-ft	209.42 K-ft	4. 1.2D+W+L+0.5S »+Z	F7-1

Member:COL004

1. Member Properties	2. Connections	3. Applied Loads
Length: 25.901 ft Shape: HSS10X10X.500 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.5183 K Framing: Column	Start: N007 @ (25.167, 23.068, 0 ft) - Free End: N003 @ (25.167, -2.8333, 0 ft) - Fixed	Unif.: (W+X) Force X 0.0169 K/ft (W-X) Force X -0.0169 K/ft (W+Z) Force Z 0.0169 K/ft (W-Z) Force Z -0.0169 K/ft
4. Extreme Forces Axial: -24.386 K (35) -> 7.7939 K (6) My: -12.301 K-ft (9) -> 12.813 K-ft (61) Mz: -17.777 K-ft (6) -> 18.251 K-ft (60) Vy: -0.68635 K (6) -> 0.70464 K (60) Vz: -0.69378 K (9) -> 0.71355 K (61) Torsion: -1.192 K-ft (43) -> 0.27777 K-ft (9)	5. Extreme Deflections (all cases) Total Dy: -0.92532 in (6) -> 0.94998 in (60) Total Dz: -0.61249 in (7) -> 0.63914 in (59) Beam Dy: 0.00592 in (2) -> 0.94998 in (60) Beam Dz: 0.00066 in (46) -> 0.63914 in (59)	

Member Design Checks:COL004

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0477	Pass	24.386 K	510.88 K	3. 1.2D+1.6Lr+0.5W »-Y	E3-2
Combined Check	0.1633	Pass	0.16327	1	4. 1.2D+W+L+0.5Lr »-Y	H1-1b
Strong Flexure Check	0.0872	Pass	18.251 K-ft	209.42 K-ft	4. 1.2D+W+L+0.5S »-Y	F7-1
Torsion Shear Check	0.0068	Pass	0.16996 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »+Z	H3-1
Weak Flexure Check	0.0612	Pass	12.813 K-ft	209.42 K-ft	4. 1.2D+W+L+0.5S »+Z	F7-1
Weak Shear Check	0.0036	Pass	0.71355 K	198.79 K	4. 1.2D+W+L+0.5S »+Z	G4-1

Member:RBX002

1. Member Properties Length: 35.127 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.245 K Framing: Bracing	2. Connections Start: N019 @ (-5, 16.604, 25.917 ft) - Free End: N020 @ (30, 19.583, 25.917 ft) - Free	3. Applied Loads Unif.: (W+Z) Force Z 0.02532 K/ft (W-Z) Force Z -0.02532 K/ft Linear: (W+Y) Force Y 0.005 -> 0.015 K/ft (W+Y) Force Y 0.015 -> 0.005 K/ft (W-Y) Force Y -0.005 -> -0.015 K/ft (W-Y) Force Y -0.015 -> -0.005 K/ft
4. Extreme Forces Axial: -3.8352 K (42) -> 0.8955 K (6) My: -7.4473 K-ft (42) -> 6.8847 K-ft (42) Mz: -10.899 K-ft (6) -> 52.112 K-ft (42) Vy: -4.7518 K (42) -> 5.6151 K (42) Vz: -1.6868 K (40) -> 0.93874 K (43) Torsion: -0.92748 K-ft (42) -> 0.63387 K-ft (42)	5. Extreme Deflections (all cases) Total Dy: -1.2844 in (42) -> 0.50147 in (41) Total Dz: -0.62042 in (9) -> 0.76587 in (61) Beam Dy: 0.01757 in (8) -> 0.54621 in (42) Beam Dz: 0.0002 in (14) -> 0.32102 in (43)	

Member Design Checks:RBX002

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0773	Pass	3.8352 K	49.642 K	3. 1.2D+1.6S+0.5W »-Y	E3-3
Combined Check	0.4763	Pass	0.47631	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.4116	Pass	52.112 K-ft	126.62 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Torsion Shear Check	0.0152	Pass	0.37693 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.1384	Pass	-7.4473 K-ft	53.824 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-2
Weak Shear Check	0.0329	Pass	-1.6868 K	51.2 K	3. 1.2D+1.6S+0.5W »-X	G4-1

Member:V002

1. Member Properties Length: 35.993 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.2757 K Framing: Bracing	2. Connections Start: N018 @ (30, 25, -4.333 ft) - Free End: N017 @ (-5, 16.604, -4.333 ft) - Free	3. Applied Loads Unif.: (W+Z) Force Z 0.02532 K/ft (W-Z) Force Z -0.02532 K/ft Linear: (W+Y) Force Y 0.005 -> 0.015 K/ft (W+Y) Force Y 0.015 -> 0.005 K/ft (W-Y) Force Y -0.005 -> -0.015 K/ft (W-Y) Force Y -0.015 -> -0.005 K/ft

Member:V002 (continued)

4. Extreme Forces	5. Extreme Deflections (all cases)	
Axial: -5.0195 K (42) -> 0.78259 K (6) My: -4.0192 K-ft (42) -> 5.2078 K-ft (42) Mz: -15.57 K-ft (6) -> 60.164 K-ft (42) Vy: -5.2273 K (42) -> 5.7153 K (42) Vz: -2.5873 K (42) -> 0.75983 K (44) Torsion: -0.76577 K-ft (42) -> 0.82368 K-ft (44)	Total Dy: -1.5263 in (42) -> 0.61099 in (40) Total Dz: -0.8267 in (61) -> 0.77618 in (60) Beam Dy: 0.01631 in (4) -> 0.45116 in (42) Beam Dz: 0.0052 in (92) -> 0.43066 in (43)	

Member Design Checks:V002

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.1066	Pass	5.0195 K	47.067 K	3. 1.2D+1.6S+0.5W »-Y	E3-3
Combined Check	0.5897	Pass	0.58971	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.4752	Pass	60.164 K-ft	126.62 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Torsion Shear Check	0.0135	Pass	0.33475 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Z	H3-1
Weak Flexure Check	0.0968	Pass	5.2078 K-ft	53.824 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-2
Weak Shear Check	0.0505	Pass	-2.5873 K	51.2 K	3. 1.2D+1.6S+0.5W »-Y	G4-1

Member:RBZ003

1. Member Properties	2. Connections	3. Applied Loads
Length: 30.25 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.0721 K Framing: Bracing	Start: N017 @ (-5, 16.604, -4.333 ft) - Free End: N019 @ (-5, 16.604, 25.917 ft) - Free	Unif.: (W+X) Force X 0.02532 K/ft (W-X) Force X -0.02532 K/ft Linear: (W+Y) Force Y 0.005 -> 0.015 K/ft (W+Y) Force Y 0.015 -> 0.005 K/ft (W-Y) Force Y -0.005 -> -0.015 K/ft (W-Y) Force Y -0.015 -> -0.005 K/ft
4. Extreme Forces Axial: -3.5674 K (42) -> 0.82753 K (6) My: -7.4742 K-ft (42) -> 3.6525 K-ft (40) Mz: -8.4081 K-ft (6) -> 42.561 K-ft (42) Vy: -5.4801 K (42) -> 4.5675 K (42) Vz: -1.185 K (40) -> 1.1154 K (55) Torsion: -0.37033 K-ft (42) -> 0.06154 K-ft (6)	**5. Extreme Deflections (all cases)** Total Dy: -0.51463 in (42) -> 0.47005 in (42) Total Dz: -0.9945 in (7) -> 1.111 in (59) Beam Dy: 0.00884 in (2) -> 0.31275 in (41) Beam Dz: 0.00069 in (64) -> 0.24467 in (43)	

Member Design Checks:RBZ003

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0720	Pass	3.5674 K	49.549 K	3. 1.2D+1.6S+0.5W »-Y	E3-3
Combined Check	0.4033	Pass	0.40331	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.3361	Pass	42.561 K-ft	126.62 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Strong Shear Check	0.0289	Pass	-5.4801 K	189.91 K	3. 1.2D+1.6S+0.5W »-Y	G4-1
Torsion Shear Check	0.0061	Pass	0.15051 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.1389	Pass	-7.4742 K-ft	53.824 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-2

Member:H001

1. Member Properties	2. Connections	3. Applied Loads
Length: 12.966 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 0.45957 K Framing: Beam	Start: N024 @ (20.55, 22.733, -4.333 ft) - Free End: N026 @ (30, 23.417, 4.519 ft) - Free	Unif.: (S) Force Y -0.134 K/ft
4. Extreme Forces Axial: -1.3988 K (35) -> 0.65905 K (6) My: -1.6733 K-ft (9) -> 9.2797 K-ft (43) Mz: -49.316 K-ft (42) -> 16.282 K-ft (6) Vy: -8.661 K (42) -> 8.7396 K (42) Vz: -1.93 K (43) -> 1.7348 K (43) Torsion: -2.6996 K-ft (42) -> 1.7734 K-ft (42)	5. Extreme Deflections (all cases) Total Dy: -0.87596 in (60) -> 0.62595 in (6) Total Dz: -0.95411 in (7) -> 1.1316 in (59) Beam Dy: 0.00857 in (8) -> 0.43007 in (42) Beam Dz: 0.00185 in (16) -> 0.17851 in (43)	

Member Design Checks:H001

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0058	Pass	1.3988 K	239.61 K	3. 1.2D+1.6Lr+0.5W »-Y	E3-2
Combined Check	0.5551	Pass	0.55509	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.3895	Pass	-49.316 K-ft	126.62 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Strong Shear Check	0.0460	Pass	8.7396 K	189.91 K	3. 1.2D+1.6S+0.5W »-Y	G4-1
Torsion Shear Check	0.0442	Pass	1.0971 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.1724	Pass	9.2797 K-ft	53.824 K-ft	3. 1.2D+1.6S+0.5W »+Z	F7-2

Member:H002

1. Member Properties	2. Connections	3. Applied Loads
Length: 13.335 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 0.47264 K Framing: Beam	Start: N027 @ (-5, 16.604, 16.798 ft) - Free End: N028 @ (4.697, 17.406, 25.917 ft) - Free	Unif.: (S) Force Y -0.134 K/ft
4. Extreme Forces Axial: -1.4423 K (33) -> 0.82858 K (44) My: -1.7624 K-ft (9) -> 3.1896 K-ft (40) Mz: -55.301 K-ft (42) -> 15.003 K-ft (6) Vy: -9.2345 K (42) -> 9.0621 K (42) Vz: -0.47249 K (61) -> 0.73603 K (40) Torsion: -2.1398 K-ft (42) -> 0.34457 K-ft (6)	5. Extreme Deflections (all cases) Total Dy: -0.47658 in (41) -> 0.26196 in (60) Total Dz: -1.0828 in (7) -> 1.1225 in (59) Beam Dy: 0.0003 in (4) -> 0.37305 in (42) Beam Dz: 0.0008 in (94) -> 0.05367 in (61)	

Member Design Checks:H002

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0062	Pass	1.4423 K	231.65 K	3. 1.2D+1.6Lr+0.5W »-X	E3-2
Combined Check	0.4748	Pass	0.47475	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.4368	Pass	-55.301 K-ft	126.62 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Strong Shear Check	0.0486	Pass	-9.2345 K	189.91 K	3. 1.2D+1.6S+0.5W »-Y	G4-1
Torsion Shear Check	0.0350	Pass	0.86964 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.0593	Pass	3.1896 K-ft	53.824 K-ft	3. 1.2D+1.6S+0.5W »-X	F7-2

Member:H005

1. Member Properties	2. Connections	3. Applied Loads
Length: 28.859 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.0229 K Framing: Beam	Start: N031 @ (-5, 16.605, 6.929 ft) - Free End: N029 @ (16.655, 18.447, 25.917 ft) - Free	Unif.: (S) Force Y -0.228 K/ft
4. Extreme Forces Axial: -1.8733 K (42) -> 0.3038 K (6) My: -3.8583 K-ft (6) -> 15.51 K-ft (42) Mz: -11.731 K-ft (6) -> 56.55 K-ft (42) Vy: -7.4162 K (42) -> 7.3314 K (42) Vz: -2.0271 K (42) -> 2.0024 K (42) Torsion: -0.03451 K-ft (6) -> 0.259 K-ft (42)	5. Extreme Deflections (all cases) Total Dy: -2.4819 in (42) -> 0.28332 in (6) Total Dz: -3.4018 in (42) -> 1.7487 in (6) Beam Dy: 0.01346 in (4) -> 0.72628 in (42) Beam Dz: 0.00043 in (71) -> 0.22541 in (62)	

Member Design Checks:H005

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0344	Pass	1.8733 K	54.438 K	3. 1.2D+1.6S+0.5W »-Y	E3-3
Combined Check	0.7479	Pass	0.74793	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.4466	Pass	56.55 K-ft	126.62 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Torsion Shear Check	0.0042	Pass	0.10526 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.2882	Pass	15.51 K-ft	53.824 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-2
Weak Shear Check	0.0396	Pass	-2.0271 K	51.2 K	3. 1.2D+1.6S+0.5W »-Y	G4-1

Member:H006

1. Member Properties	2. Connections	3. Applied Loads
Length: 28.741 ft Shape: HSS12X4X.500 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.3223 K Framing: Beam	Start: N030 @ (8.45, 19.831, -4.333 ft) - Free End: N032 @ (30, 21.618, 14.6 ft) - Free	Unif.: (S) Force Y -0.228 K/ft
4. Extreme Forces Axial: -2.1727 K (41) -> 0.3869 K (7) My: -27.651 K-ft (42) -> 3.1878 K-ft (6) Mz: -16.86 K-ft (6) -> 59.569 K-ft (42) Vy: -7.8169 K (42) -> 7.7645 K (42) Vz: -3.6479 K (42) -> 3.6679 K (42) Torsion: -0.09146 K-ft (8) -> 0.13287 K-ft (62)	5. Extreme Deflections (all cases) Total Dy: -2.7374 in (42) -> 1.1847 in (6) Total Dz: -0.91713 in (7) -> 4.3123 in (41) Beam Dy: 0.0129 in (4) -> 0.76363 in (42) Beam Dz: 0.00145 in (16) -> 0.28511 in (43)	

Member Design Checks:H006

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0324	Pass	2.1727 K	67.041 K	3. 1.2D+1.6S+0.5W »+Y	E3-3
Combined Check	0.7630	Pass	0.76301	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.3697	Pass	59.569 K-ft	161.12 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Torsion Shear Check	0.0017	Pass	0.04248 Ksi	24.84 Ksi	4. 1.2D+W+L+0.5S »-Z	H3-1
Weak Flexure Check	0.3835	Pass	-27.651 K-ft	72.105 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Weak Shear Check	0.0609	Pass	3.6679 K	60.179 K	3. 1.2D+1.6S+0.5W »-Y	G4-1

Member:H007

1. Member Properties Length: 1.0194 ft Shape: W10X19 Material: ASTM A992 Grade 50 Weight: 0.01953 K Framing: Bracing	2. Connections Start: N034 @ (0.329, 17.165, -0.329 ft) - Free End: N033 @ (-0.382, 16.997, 0.382 ft) - Free	4. Extreme Forces Axial: -2.7993 K (41) -> 2.5545 K (42) My: -0.6328 K-ft (6) -> 1.4996 K-ft (42) Mz: -8.6046 K-ft (42) -> 1.7744 K-ft (6) Vy: -17.101 K (42) -> 16.048 K (42) Vz: -2.7943 K (42) -> 2.3518 K (54) Torsion: -0.75477 K-ft (42) -> 0.45725 K-ft (42)
5. Extreme Deflections (all cases) Total Dy: -0.34821 in (42) -> 0.2401 in (41) Total Dz: -0.35817 in (61) -> 0.35509 in (9) Beam Dy: 0.00656 in (8) -> 0.44026 in (42) Beam Dz: 0.00016 in (67) -> 0.01355 in (42)		

Member Design Checks:H007

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0112	Pass	2.7993 K	249.3 K	3. 1.2D+1.6S+0.5W »+Y	E3-2
Combined Check	0.2307	Pass	0.23065	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Combined Torsion Check	0.3795	Pass	0.37954	1	3. 1.2D+1.6S+0.5W »-Y	H3-7 and H3-8
Strong Flexure Check	0.1062	Pass	-8.6046 K-ft	81 K-ft	3. 1.2D+1.6S+0.5W »-Y	F2-1
Strong Shear Check	0.2235	Pass	-17.101 K	76.5 K	3. 1.2D+1.6S+0.5W »-Y	G2-1
Weak Flexure Check	0.1194	Pass	1.4996 K-ft	12.563 K-ft	3. 1.2D+1.6S+0.5W »-Y	F6-1

Member:H008

1. Member Properties Length: 1.022 ft Shape: W10X19 Material: ASTM A992 Grade 50 Weight: 0.01957 K Framing: Bracing	2. Connections Start: N036 @ (25.343, 19.291, 21.309 ft) - Free End: N035 @ (24.658, 19.129, 22.05 ft) - Free	4. Extreme Forces Axial: -2.8134 K (41) -> 1.8544 K (42) My: -3.9741 K-ft (42) -> 1.0743 K-ft (6) Mz: -10.168 K-ft (42) -> 1.9149 K-ft (6) Vy: -18.683 K (42) -> 13.694 K (42) Vz: -1.5695 K (54) -> 5.3436 K (42) Torsion: -0.74333 K-ft (42) -> 0.37166 K-ft (42)
5. Extreme Deflections (all cases) Total Dy: -0.6402 in (42) -> 0.26467 in (6) Total Dz: -0.36494 in (61) -> 0.35001 in (9) Beam Dy: 0.00333 in (8) -> 0.72796 in (42) Beam Dz: 0.00032 in (13) -> 0.02879 in (42)		

Member Design Checks:H008

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0113	Pass	2.8134 K	249.28 K	3. 1.2D+1.6S+0.5W »+Y	E3-2
Combined Check	0.4455	Pass	0.44555	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Combined Torsion Check	0.2824	Pass	0.28241	1	3. 1.2D+1.6S+0.5W »-Y	H3-7 and H3-8
Strong Flexure Check	0.1255	Pass	-10.168 K-ft	81 K-ft	3. 1.2D+1.6S+0.5W »-Y	F2-1
Strong Shear Check	0.2442	Pass	-18.683 K	76.5 K	3. 1.2D+1.6S+0.5W »-Y	G2-1
Weak Flexure Check	0.3163	Pass	-3.9741 K-ft	12.563 K-ft	3. 1.2D+1.6S+0.5W »-Y	F6-1

Member:H003

1. Member Properties	2. Connections	3. Applied Loads
Length: 45.225 ft Shape: HSS12X4X.500 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 2.0807 K Framing: Beam	Start: N023 @ (29.129, 19.509, 25.917 ft) - Free End: N021 @ (-5, 16.604, -3.6149 ft) - Free	Unif.: (S) Force Y -0.2 K/ft
4. Extreme Forces Axial: -2.1358 K (42) -> 3.4422 K (42) My: -9.3301 K-ft (42) -> 11.209 K-ft (42) Mz: -35.452 K-ft (42) -> 38.04 K-ft (42) Vy: -8.3773 K (42) -> 7.3407 K (42) Vz: -2.1254 K (42) -> 2.3172 K (42) Torsion: -0.0685 K-ft (6) -> 0.36771 K-ft (42)	5. Extreme Deflections (all cases) Total Dy: -1.3165 in (43) -> 0.27168 in (60) Total Dz: -1.3577 in (6) -> 2.1019 in (60) Beam Dy: 0.01444 in (4) -> 0.34814 in (43) Beam Dz: 0.00054 in (35) -> 0.28382 in (61)	

Member Design Checks:H003

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0789	Pass	2.1358 K	27.076 K	3. 1.2D+1.6S+0.5W »-Y	E3-3
Combined Check	0.4149	Pass	0.41494	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.2361	Pass	38.04 K-ft	161.12 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Torsion Shear Check	0.0047	Pass	0.11756 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.1555	Pass	11.209 K-ft	72.105 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Weak Shear Check	0.0385	Pass	2.3172 K	60.179 K	3. 1.2D+1.6S+0.5W »-Y	G4-1

Member:H004

1. Member Properties	2. Connections	3. Applied Loads
Length: 45.447 ft Shape: HSS12X4X.500 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 2.0909 K Framing: Beam	Start: N022 @ (-4.3014, 16.772, -4.333 ft) - Free End: N025 @ (30, 19.687, 25.337 ft) - Free	Unif.: (S) Force Y -0.2 K/ft
4. Extreme Forces Axial: -0.87737 K (6) -> 5.2917 K (42) My: -11.959 K-ft (42) -> 21.394 K-ft (42) Mz: -47.772 K-ft (42) -> 24.162 K-ft (42) Vy: -8.0645 K (42) -> 8.917 K (42) Vz: -4.0483 K (42) -> 3.7871 K (42) Torsion: -0.10445 K-ft (8) -> 0.36455 K-ft (44)	5. Extreme Deflections (all cases) Total Dy: -0.7299 in (60) -> 0.626 in (6) Total Dz: -0.99072 in (7) -> 1.7481 in (41) Beam Dy: 0.00098 in (75) -> 0.11807 in (62) Beam Dz: 0.00641 in (50) -> 0.42336 in (43)	

Member Design Checks:H004

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0095	Pass	5.2917 K	558.9 K	3. 1.2D+1.6S+0.5W »-Y	D2-1
Combined Check	0.5961	Pass	0.59612	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.2965	Pass	-47.772 K-ft	161.12 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Torsion Shear Check	0.0047	Pass	0.11655 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Z	H3-1
Weak Flexure Check	0.2967	Pass	21.394 K-ft	72.105 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Weak Shear Check	0.0673	Pass	-4.0483 K	60.179 K	3. 1.2D+1.6S+0.5W »-Y	G4-1

Member:V001

1. Member Properties	2. Connections	3. Applied Loads
Length: 30.731 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 1.0892 K Framing: Bracing	Start: N020 @ (30, 19.583, 25.917 ft) - Free End: N018 @ (30, 25, -4.333 ft) - Free	Unif.: (W+X) Force X 0.02532 K/ft (W-X) Force X -0.02532 K/ft Linear: (W+Y) Force Y 0.015 -> 0.005 K/ft (W+Y) Force Y 0.005 -> 0.015 K/ft (W-Y) Force Y -0.015 -> -0.005 K/ft (W-Y) Force Y -0.005 -> -0.015 K/ft
4. Extreme Forces Axial: -3.9114 K (42) -> 0.655 K (6) My: -3.2315 K-ft (42) -> 6.1558 K-ft (42) Mz: -12.517 K-ft (6) -> 48.59 K-ft (42) Vy: -5.5165 K (42) -> 5.1081 K (42) Vz: -0.85808 K (42) -> 1.505 K (35) Torsion: -1.2103 K-ft (42) -> 0.57868 K-ft (42)	5. Extreme Deflections (all cases) Total Dy: -0.81119 in (42) -> 0.65085 in (43) Total Dz: -1.0998 in (59) -> 0.99598 in (7) Beam Dy: 0.01024 in (4) -> 0.7036 in (42) Beam Dz: 0.0003 in (28) -> 0.15641 in (61)	

Member Design Checks:V001

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0815	Pass	3.9114 K	48.009 K	3. 1.2D+1.6S+0.5W »-Y	E3-3
Combined Check	0.4769	Pass	0.47685	1	3. 1.2D+1.6S+0.5W »-Y	H1-1b
Strong Flexure Check	0.3838	Pass	48.59 K-ft	126.62 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-1
Torsion Shear Check	0.0198	Pass	0.49188 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.1144	Pass	6.1558 K-ft	53.824 K-ft	3. 1.2D+1.6S+0.5W »-Y	F7-2
Weak Shear Check	0.0294	Pass	1.505 K	51.2 K	3. 1.2D+1.6Lr+0.5W »-Y	G4-1

Member:BmX001

1. Member Properties	2. Connections	3. Applied Loads
Length: 25.167 ft Shape: HSS6X6X.250 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 0.44943 K Framing: Beam	Start: N009 @ (0, 11.667, 21.5 ft) - Free End: N010 @ (25.167, 11.667, 21.5 ft) - Free	Point: (D) Shear y -0.12 K (D) Shear y -0.12 K (D) Shear y -0.12 K (D) Shear y -0.12 K Unif.: (S) Force Y -0.006 K/ft (W+Z) Force Z 0.0127 K/ft (W-Z) Force Z -0.0127 K/ft
4. Extreme Forces Axial: -2.4714 K (7) -> 2.5463 K (59) My: -1.0055 K-ft (8) -> 1.0055 K-ft (9) Mz: 0 K-ft (38) -> 4.911 K-ft (38) Vy: -0.67416 K (38) -> 0.68275 K (38) Vz: -0.15981 K (56) -> 0.15981 K (55) Torsion: -0.03902 K-ft (9) -> 0.08175 K-ft (43)	5. Extreme Deflections (all cases) Total Dy: -0.68131 in (42) -> 0.00311 in (6) Total Dz: -0.42096 in (9) -> 0.42393 in (61) Beam Dy: 0.00002 in (8) -> 0.00515 in (42) Beam Dz: 0.00066 in (62) -> 0.05771 in (39)	

Member Design Checks:BmX001

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0344	Pass	2.4356 K	70.84 K	5. 0.9D+W »-Y	E3-3
Combined Check	0.1406	Pass	0.14055	1	3. 1.2D+1.6S+0.5W »+Z	H1-1b
Strong Flexure Check	0.1271	Pass	4.911 K-ft	38.64 K-ft	3. 1.2D+1.6S+L	F7-1
Strong Shear Check	0.0111	Pass	0.68275 K	61.361 K	3. 1.2D+1.6S+0.5W »-Z	G4-1
Torsion Shear Check	0.0026	Pass	0.0635 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »+Z	H3-1

Member Design Checks:BmX001 (continued)

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Weak Flexure Check	0.0260	Pass	1.0055 K-ft	38.64 K-ft	4. 1.2D+W+L+0.5Lr »-Z	F7-1

Member:RBZ001

1. Member Properties	2. Connections	4. Extreme Forces
Length: 21.837 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 0.77396 K Framing: Bracing	Start: N007 @ (25.167, 23.068, 0 ft) - Free End: N008 @ (25.167, 19.249, 21.5 ft) - Free	Axial: -1.3354 K (6) -> 3.2313 K (35) My: -1.2525 K-ft (6) -> 1.2525 K-ft (7) Mz: -5.3045 K-ft (6) -> 15.434 K-ft (35) Vy: -2.7523 K (35) -> 2.764 K (35) Vz: -0.22279 K (6) -> 0.22279 K (7) Torsion: -0.78609 K-ft (6) -> 5.0728 K-ft (42)
5. Extreme Deflections (all cases) Total Dy: -0.37223 in (54) -> 0.20851 in (6) Total Dz: -1.0958 in (60) -> 1.0929 in (6) Beam Dy: 0 in (32) -> 0.00562 in (41) Beam Dz: 0.00082 in (58) -> 0.14931 in (61)		

Member Design Checks:RBZ001

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0075	Pass	3.2313 K	430.56 K	3. 1.2D+1.6Lr+0.5W »-Y	D2-1
Combined Check	0.1374	Pass	0.13736	1	4. 1.2D+W+L+0.5Lr »-Y	H1-1b
Strong Flexure Check	0.1219	Pass	15.434 K-ft	126.62 K-ft	3. 1.2D+1.6Lr+0.5W »-Y	F7-1
Strong Shear Check	0.0146	Pass	2.764 K	189.91 K	3. 1.2D+1.6Lr+0.5W »-Y	G4-1
Torsion Shear Check	0.0830	Pass	2.0616 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.0233	Pass	1.2525 K-ft	53.824 K-ft	4. 1.2D+W+L+0.5Lr »-Y	F7-2

Member:BmX002

1. Member Properties	2. Connections	4. Extreme Forces
Length: 25.266 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 0.8955 K Framing: Bracing	Start: N008 @ (25.167, 19.249, 21.5 ft) - Free End: N006 @ (0, 17.018, 21.5 ft) - Free	Axial: -3.3132 K (6) -> 5.3944 K (60) My: -0.00007 K-ft (7) -> 0.00007 K-ft (6) Mz: -10.204 K-ft (6) -> 28.018 K-ft (35) Vy: -4.0266 K (35) -> 3.926 K (35) Vz: 0 K (6) -> 0 K (7) Torsion: -0.48496 K-ft (6) -> 2.833 K-ft (42)
5. Extreme Deflections (all cases) Total Dy: -0.70998 in (35) -> 0.32957 in (6) Total Dz: -0.63895 in (59) -> 0.61129 in (7) Beam Dy: 0 in (8) -> 0.00948 in (42) Beam Dz: 0.00149 in (78) -> 0.1703 in (61)		

Member Design Checks:BmX002

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0330	Pass	2.3443 K	71.025 K	5. 0.9D+W »+Y	E3-3
Combined Check	0.2261	Pass	0.22612	1	3. 1.2D+1.6Lr+0.5W »-Y	H1-1b
Strong Flexure Check	0.2213	Pass	28.018 K-ft	126.62 K-ft	3. 1.2D+1.6Lr+0.5W »-Y	F7-1
Strong Shear Check	0.0212	Pass	-4.0266 K	189.91 K	3. 1.2D+1.6Lr+0.5W »-Y	G4-1
Torsion Shear Check	0.0464	Pass	1.1513 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.0000	Pass	0.00007 K-ft	53.824 K-ft	4. 1.2D+W+L+0.5Lr »+Y	F7-2

Member:BmZ001

1. Member Properties	2. Connections	4. Extreme Forces
Length: 21.5 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 0.76203 K Framing: Beam	Start: N006 @ (0, 17.018, 21.5 ft) - Free End: N005 @ (0, 17.087, 0 ft) - Free	Axial: -1.5654 K (6) -> 3.3809 K (42) My: -0.64616 K-ft (7) -> 0.64616 K-ft (59) Mz: -7.5916 K-ft (6) -> 20.885 K-ft (35) Vy: -3.5852 K (35) -> 3.4893 K (35) Vz: -0.10959 K (6) -> 0.10959 K (54) Torsion: -0.08433 K-ft (6) -> 0.40761 K-ft (42)
5. Extreme Deflections (all cases) Total Dy: -0.36921 in (35) -> 0.13489 in (6) Total Dz: -1.0258 in (59) -> 1.0226 in (7) Beam Dy: 0 in (8) -> 0.00689 in (42) Beam Dz: 0.00154 in (88) -> 0.1493 in (61)		

Member Design Checks:BmZ001

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0079	Pass	3.3809 K	430.56 K	3. 1.2D+1.6S+0.5W »-Y	D2-1
Combined Check	0.1744	Pass	0.17443	1	3. 1.2D+1.6Lr+0.5W »-Y	H1-1b
Strong Flexure Check	0.1649	Pass	20.885 K-ft	126.62 K-ft	3. 1.2D+1.6Lr+0.5W »-Y	F7-1
Strong Shear Check	0.0189	Pass	-3.5852 K	189.91 K	3. 1.2D+1.6Lr+0.5W »-Y	G4-1
Torsion Shear Check	0.0067	Pass	0.16566 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »-Y	H3-1
Weak Flexure Check	0.0120	Pass	0.64616 K-ft	53.824 K-ft	4. 1.2D+W+L+0.5S »+Y	F7-2

Member:RBX001

1. Member Properties	2. Connections	4. Extreme Forces
Length: 25.868 ft Shape: HSS12X4X.375 Material: ASTM A500 Grade B (Fy = 46ksi) Weight: 0.91684 K Framing: Bracing	Start: N005 @ (0, 17.087, 0 ft) - Free End: N007 @ (25.167, 23.068, 0 ft) - Free	Axial: -1.6565 K (6) -> 4.1022 K (35) My: -1.271 K-ft (6) -> 1.271 K-ft (7) Mz: -7.2988 K-ft (6) -> 20.964 K-ft (35) Vy: -3.1832 K (35) -> 3.0679 K (35) Vz: -0.19225 K (7) -> 0.19225 K (6) Torsion: -1.4731 K-ft (43) -> 0.10148 K-ft (9)
5. Extreme Deflections (all cases) Total Dy: -0.70429 in (54) -> 0.39867 in (6) Total Dz: -0.75566 in (7) -> 0.76315 in (59) Beam Dy: 0.00001 in (57) -> 0.00374 in (41) Beam Dz: 0.00171 in (58) -> 0.17166 in (61)		

Member Design Checks:RBX001

Type	Unity Check	Status	Demand	Capacity	Result Case	Code Reference
Axial Check	0.0120	Pass	0.81074 K	67.757 K	5. 0.9D+W »+Y	E3-3
Combined Check	0.1814	Pass	0.18143	1	3. 1.2D+1.6Lr+0.5W »-Y	H1-1b
Strong Flexure Check	0.1656	Pass	20.964 K-ft	126.62 K-ft	3. 1.2D+1.6Lr+0.5W »-Y	F7-1
Strong Shear Check	0.0168	Pass	-3.1832 K	189.91 K	3. 1.2D+1.6Lr+0.5W »-Y	G4-1
Torsion Shear Check	0.0241	Pass	0.59867 Ksi	24.84 Ksi	3. 1.2D+1.6S+0.5W »+Z	H3-1
Weak Flexure Check	0.0236	Pass	1.271 K-ft	53.824 K-ft	4. 1.2D+W+L+0.5Lr »-Y	F7-2

Result Case Legend

(1) D	(10) D+L
(3) S	(30) 2. 1.2D+1.6L+0.5R

Result Case Legend (continued)

(5) W-X	(50) 3. 1.2D+1.6R+0.5W »-Z
(7) W-Y	(70) 4. 1.2D+0.5W+L+0.5Lr+Fa »-X
(9) W-Z	(90) 5. 0.9D+0.5W+Fa »-Y
(11) D+0.75(L+W) »+X	(12) D+0.75(L+W) »-X
(13) D+0.75(L+W) »+Y	(14) D+0.75(L+W) »-Y
(15) D+0.75(L+W) »+Z	(16) D+0.75(L+W) »-Z
(17) D+S	(18) D+Lr
(19) Live	(2) Lr
(20) Snow	(21) Wind »+X
(22) Wind »-X	(23) Wind »+Y
(24) Wind »-Y	(25) Wind »+Z
(26) Wind »-Z	(27) 1. 1.4D
(28) 2. 1.2D+1.6L+0.5Lr	(29) 2. 1.2D+1.6L+0.5S
(31) 3. 1.2D+1.6Lr+L	(32) 3. 1.2D+1.6Lr+0.5W »+X
(33) 3. 1.2D+1.6Lr+0.5W »-X	(34) 3. 1.2D+1.6Lr+0.5W »+Y
(35) 3. 1.2D+1.6Lr+0.5W »-Y	(36) 3. 1.2D+1.6Lr+0.5W »+Z
(37) 3. 1.2D+1.6Lr+0.5W »-Z	(38) 3. 1.2D+1.6S+L
(39) 3. 1.2D+1.6S+0.5W »+X	(4) W+X
(40) 3. 1.2D+1.6S+0.5W »-X	(41) 3. 1.2D+1.6S+0.5W »+Y
(42) 3. 1.2D+1.6S+0.5W »-Y	(43) 3. 1.2D+1.6S+0.5W »+Z
(44) 3. 1.2D+1.6S+0.5W »-Z	(45) 3. 1.2D+1.6R+0.5W »+X
(46) 3. 1.2D+1.6R+0.5W »-X	(47) 3. 1.2D+1.6R+0.5W »+Y
(48) 3. 1.2D+1.6R+0.5W »-Y	(49) 3. 1.2D+1.6R+0.5W »+Z
(51) 4. 1.2D+W+L+0.5Lr »+X	(52) 4. 1.2D+W+L+0.5Lr »-X
(53) 4. 1.2D+W+L+0.5Lr »+Y	(54) 4. 1.2D+W+L+0.5Lr »-Y
(55) 4. 1.2D+W+L+0.5Lr »+Z	(56) 4. 1.2D+W+L+0.5Lr »-Z
(57) 4. 1.2D+W+L+0.5S »+X	(58) 4. 1.2D+W+L+0.5S »-X
(59) 4. 1.2D+W+L+0.5S »+Y	(6) W+Y
(60) 4. 1.2D+W+L+0.5S »-Y	(61) 4. 1.2D+W+L+0.5S »+Z
(62) 4. 1.2D+W+L+0.5S »-Z	(63) 4. 1.2D+W+L+0.5R »+X
(64) 4. 1.2D+W+L+0.5R »-X	(65) 4. 1.2D+W+L+0.5R »+Y
(66) 4. 1.2D+W+L+0.5R »-Y	(67) 4. 1.2D+W+L+0.5R »+Z
(68) 4. 1.2D+W+L+0.5R »-Z	(69) 4. 1.2D+0.5W+L+0.5Lr+Fa »+X
(71) 4. 1.2D+0.5W+L+0.5Lr+Fa »+Y	(72) 4. 1.2D+0.5W+L+0.5Lr+Fa »-Y
(73) 4. 1.2D+0.5W+L+0.5Lr+Fa »+Z	(74) 4. 1.2D+0.5W+L+0.5Lr+Fa »-Z
(75) 4. 1.2D+0.5W+L+0.5S+Fa »+X	(76) 4. 1.2D+0.5W+L+0.5S+Fa »-X
(77) 4. 1.2D+0.5W+L+0.5S+Fa »+Y	(78) 4. 1.2D+0.5W+L+0.5S+Fa »-Y
(79) 4. 1.2D+0.5W+L+0.5S+Fa »+Z	(8) W+Z
(80) 4. 1.2D+0.5W+L+0.5S+Fa »-Z	(81) 5. 0.9D+W »+X
(82) 5. 0.9D+W »-X	(83) 5. 0.9D+W »+Y
(84) 5. 0.9D+W »-Y	(85) 5. 0.9D+W »+Z
(86) 5. 0.9D+W »-Z	(87) 5. 0.9D+0.5W+Fa »+X

Result Case Legend (continued)

(88) 5. 0.9D+0.5W+Fa »-X	(89) 5. 0.9D+0.5W+Fa »+Y
(91) 5. 0.9D+0.5W+Fa »+Z	(92) 5. 0.9D+0.5W+Fa »-Z
(93) 5. 0.9D+Di+Wi	(94) 6. 1.2D+E+L+0.2S

Project Settings

Building Code Load Combinations:
ASCE 7-16 LRFD
Deflection Checks
General Settings:
Vertical Direction: Y
North Axis: Plus Z
Ground Elevation: 0 ft
Occupancy Risk Category: II
Seismic Data:
Seismic Design Category: B
Spectral Acceleration SDs: 0.081
Overstrength (Omega) X: 1, Y: 1, Z: 1
Redundancy (Rho) X: 1 , Y: 1, Z: 1
Wind Data:
Wind Speed (mph): 110
Mean Roof Height: 21.3 ft
Ground Elevation: 0 ft
Gust Factor: 0.85
Analysis Data:
Analysis Method: FirstOrder
Performance: Auto
Force Tolerance: 0.1
Absolute Force Tolerance: 0.5 K
Displacement Tolerance: 0.01
Load Stepping Points: 31

Service Case, Wind Parameters

Name	Source	Projected Width ft	Side Length ft	Kd	Exposure	Enclosure	Internal Pressure	Kzt(z=0)	Roof Value	Large Volume	Topo Hill
W +X	Wind Pos X Loads	10.0000	10.0000	0.8500	Category C	Enclosed	Outward	1.0000	First Value	No	-NA-
W +Y	Wind Pos Y Loads	10.0000	10.0000	0.8500	Category C	Enclosed	Outward	1.0000	First Value	No	-NA-
W +Z	Wind Pos Z Loads	10.0000	10.0000	0.8500	Category C	Enclosed	Outward	1.0000	First Value	No	-NA-
W-X	Wind Neg X Loads	10.0000	10.0000	0.8500	Category C	Enclosed	Outward	1.0000	First Value	No	-NA-
W-Y	Wind Neg Y Loads	10.0000	10.0000	0.8500	Category C	Enclosed	Outward	1.0000	First Value	No	-NA-
W-Z	Wind Neg Z Loads	10.0000	10.0000	0.8500	Category C	Enclosed	Outward	1.0000	First Value	No	-NA-

Patio Canopy - Light Gauge Framing

CFS Version 14.0.0
Analysis: Canopy.cfsa
7.92 ft Span Simple Beam

jfarrar
METT-ENGR

Rev. Date: 2/23/2024 11:41:27 AM
By: jfarrar
Printed: 2/23/2024 11:41:52 AM

Member Check - AISI S100-16/S1-18, US, ASD

```
Load Combination: 0.6D+0.6W
Design Parameters at 3.9600 ft:
Lx         7.9200 ft      Ly         7.9200 ft      Lt         7.9200 ft
Kx         1.0000         Ky         1.0000         Kt         1.0000

Section: 1200S162-54 Single.cfss
Material Type: A653 SS Grade 50/1, Fy=50 ksi
Cbx        1.1364         Cby        1.0000         ex         0.0000 in
Cmx        1.0000         Cmy        1.0000         ey         0.0000 in
Braced Flange: Top        kφ              0 k
Red. Factor, R: 0         Lm         7.9200 ft

Loads:            P          Mx          Vy          My          Vx
                 (k)       (k-in)        (k)       (k-in)        (k)
Total         0.0000      -7.957      0.0000      0.000      0.0000
Applied       0.0000      -7.957      0.0000      0.000      0.0000
Strength      3.0471      19.074      1.1016      4.124      2.8484

Interaction Equations
Eq. H1.2-1    (P, Mx, My)  0.000 + 0.417 + 0.000 = 0.417 <= 1.0
Eq. H2-1         (Mx, Vy)     Sqrt(0.028 + 0.000)= 0.166 <= 1.0
Eq. H2-1         (My, Vx)     Sqrt(0.000 + 0.000)= 0.000 <= 1.0

Stud element 3 h/t exceeds 200.
```

Web Crippling Check - AISI S100-16/S1-18, US, ASD

```
Load Combination: 0.6D+0.6W
Parameters at 0.0000 ft:
Total Load:       0.33491 k on top flange
Total Moment:      0.0000 k-in
Bearing:          0.75000 in
Flange fastened to bearing surface: Yes
Distance from edge of bearing to edge of opposite load: 7.9200 ft

Section: 1200S162-54 Single.cfss
Material Type: A653 SS Grade 50/1, Fy=50 ksi
Applied Load:       0.33491 k on top flange
Applied Moment:       0.000 k-in
Distance from edge of bearing to end of member: 0.0000 ft

Part Elem Calculation Type        Pa (k)     Pay (k) Notes
  1    3  Cee, FS-EOF           0.42991     0.42991 h/t>200
                 Web Crippling Strength     0.42991
```

```
Web Crippling Check:              0.33491 k      <=    0.42991 k
Moment Check:                       0.000 k-in   <=     47.903 k-in
Interaction Equations
Eq. H3-1a    (P, M)  0.354 + 0.000 = 0.354 <= 0.665

Stud element 3 h/t exceeds 200.
```

Calculation Details - AISI S100-16/S1-18, US, ASD

Stud element 3

t=0.0566 in, C=4, θ=90 deg

R=0.0849 in, Cr=0.14

N=0.75 in, Cn=0.35

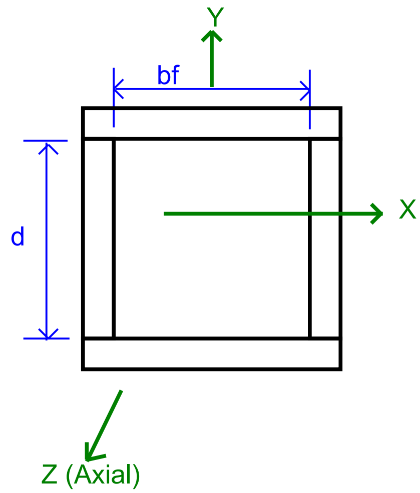
h=11.717 in, Ch=0.02

Pn=0.85981 k Eq. G5-1

Ωw=2, ϕw=0.8 (rational analysis)

Welds-16, Version 0

Canopy Girder Beams to Ring Beam
(Max shear and max moment used)



Dimensions:
****

Weld size = 0.19 in (fillet weld)
Base metal thickness = 0.38 in
d = 12.00 in
bf = 4.00 in

Welds:
******

Stress, No-Factors =1.18 ksi
Stress, J2-10(b) Factors = 0.84 ksi
Allowable Weld Stress, J2.4 = 18.00 ksi
DCR = 0.047 OK

Base Metal
****

Direct Pull-Out Stress = 0.19 ksi
Allowable Stress, Sec. G = 14.00 ksi
DCR = 0.014 OK

Additional Weld Properties:
****

Total Weld Area:

A = 4.34 in^2

Moment of Inertia:

Ix = 80.59 in^4
Iy = 15.18 in^4
Iz = 95.78 in^4

Section Modulus:

Sx = 13.33 in^3
Sy = 7.42 in^3
Sz = 15.00 in^3

J2 Stress Adjustment Factors:

For loads parallel to weld = .85
For loads perp. to weld = 1.5
For Mz loads or angled welds = 1.175

Act. stress is divided by the above factors to reflect the increase or decrease in allowable stress.

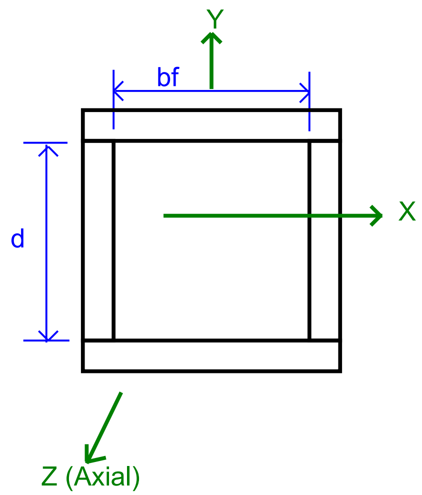
Loads:
****

Px = 0.00 kips
Py = 1.78 kips
Pz = 0.00 kips
Mx = 14.70 in-kips
My = 0.00 in-kips
Mz = 0.00 in-kips

Weld Rod Strength = 60.00 Ksi
Base Metal Yield Strength = 36.00 Ksi

Welds-16, Version 0

Canopy Ring Beam to Valley Beam Weld



Dimensions:
****

Weld size = 0.19 in (fillet weld)
Base metal thickness = 0.38 in
d = 12.00 in
bf = 4.00 in

Welds:
******

Stress, No-Factors =0.32 ksi
Stress, J2-10(b) Factors = 0.32 ksi
Allowable Weld Stress, J2.4 = 18.00 ksi
DCR = 0.018 OK

Base Metal
****

Direct Pull-Out Stress = 0.00 ksi
Allowable Stress, Sec. G = 14.00 ksi
DCR = 0.000 OK

Additional Weld Properties:
****

Total Weld Area:

A = 4.34 in^2

Moment of Inertia:

Ix = 80.59 in^4
Iy = 15.18 in^4
Iz = 95.78 in^4

Section Modulus:

Sx = 13.33 in^3
Sy = 7.42 in^3
Sz = 15.00 in^3

J2 Stress Adjustment Factors:

For loads parallel to weld = .85
For loads perp. to weld = 1.5
For Mz loads or angled welds = 1.175

Act. stress is divided by the above factors to reflect the increase or decrease in allowable stress.

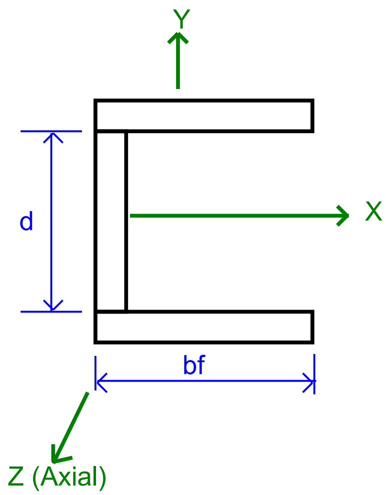
Loads:
****

Px = 0.00 kips
Py = 1.41 kips
Pz = 0.00 kips
Mx = 0.00 in-kips
My = 0.00 in-kips
Mz = 0.00 in-kips

Weld Rod Strength =   60.00 Ksi
Base Metal Yield Strength =   36.00 Ksi

Welds-16, Version 0

Canopy Valley Beam Angle Weld



Dimensions:
****

Weld size = 0.19 in (fillet weld)
Base metal thickness = 0.40 in
d = 8.00 in
bf = 3.00 in

Welds:
******

Stress, No-Factors =3.27 ksi
Stress, J2-10(b) Factors = 2.80 ksi
Allowable Weld Stress, J2.4 = 18.00 ksi
DCR = 0.156 OK

Base Metal
****

Direct Pull-Out Stress = 0.00 ksi
Allowable Stress, Sec. G = 14.00 ksi
DCR = 0.000 OK

Additional Weld Properties:
****

Total Weld Area:

A = 1.86 in^2

Moment of Inertia:

Ix = 18.68 in^4
Iy = 1.44 in^4
Iz = 20.12 in^4

Section Modulus:

Sx = 4.62 in^3
Sy = 0.63 in^3
Sz = 4.33 in^3

J2 Stress Adjustment Factors:

For loads parallel to weld = .85
For loads perp. to weld = 1.5
For Mz loads or angled welds = 1.175

Act. stress is divided by the above factors to reflect the increase or decrease in allowable stress.

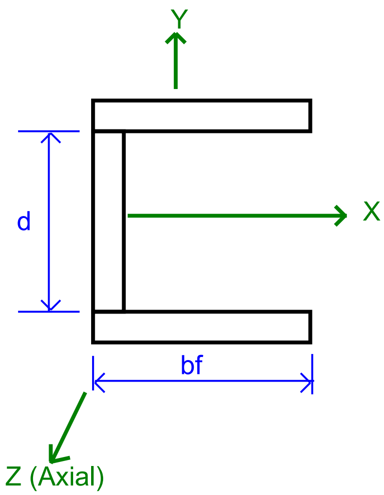
Loads:
****

Px = 0.00 kips
Py = 2.37 kips
Pz = 0.00 kips
Mx = 0.00 in-kips
My = 0.00 in-kips
Mz = 13.05 in-kips

Weld Rod Strength = 60.00 Ksi
Base Metal Yield Strength = 36.00 Ksi

Welds-16, Version 0

Canopy Valley Beam to W10x19 Weld



Dimensions:
****

Weld size = 0.19 in (fillet weld)

Base metal thickness = 0.40 in

d = 8.00 in

bf = 3.00 in

Welds:
******

Stress, No-Factors =3.10 ksi
Stress, J2-10(b) Factors = 2.20 ksi
Allowable Weld Stress, J2.4 = 18.00 ksi
DCR = 0.122 OK

Base Metal
****

Direct Pull-Out Stress = 0.47 ksi
Allowable Stress, Sec. G = 14.00 ksi
DCR = 0.034 OK

Additional Weld Properties:
****

Total Weld Area:

A = 1.86 in^2

Moment of Inertia:

Ix = 18.68 in^4
Iy = 1.44 in^4
Iz = 20.12 in^4

Section Modulus:

Sx = 4.62 in^3
Sy = 0.63 in^3
Sz = 4.33 in^3

J2 Stress Adjustment Factors:

For loads parallel to weld = .85
For loads perp. to weld = 1.5
For Mz loads or angled welds = 1.175

Act. stress is divided by the above factors to reflect the increase or decrease in allowable stress.

Loads:
****

Px = 0.00 kips

Py = 2.37 kips

Pz = 0.00 kips

Mx = 13.05 in-kips

My = 0.00 in-kips

Mz = 0.00 in-kips

Weld Rod Strength = 60.00 Ksi

Base Metal Yield Strength = 36.00 Ksi

2D Frame

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Moment Frame Front of House

Joints...

Joint Label	Joint Coordinates X ft	Joint Coordinates Y ft	X Translational Restraint	Y Translational Restraint	Z Rotational Restraint	Joint Temp deg F
1	0.0	0.0	Fixed	Fixed	Fixed	0
2	0.0	14.420				0
3	11.50	14.420				0
4	11.50	0.0	Fixed	Fixed	Fixed	0

Members...

Member Label	Property Label	Endpoint Joints I Joint	Endpoint Joints J Joint	Member Length ft	I End x	I End y	I End z (rotation)	J End x	J End y	J End z (rotation)
					Releases Specify Connectivity of Member Ends to Joints					
A	Column	1	2	14.420	Fixed	Fixed	Pinned	Fixed	Fixed	Fixed
B	Beam	2	3	11.500	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
C	Column	3	4	14.420	Fixed	Fixed	Fixed	Fixed	Fixed	Pinned

Member Stress Check Data...

Member Label	Unbraced Lengths Lu : z ft	Unbraced Lengths Lu : y	Slenderness Factors K : z	Slenderness Factors K : y	AISC Bending & Stability Factors Cm	AISC Bending & Stability Factors Cb
A	14.420	14.420	1.00	1.00	Internal	Internal
B	11.500	11.500	1.00	1.00	Internal	Internal
C	14.420	14.420	1.00	1.00	Internal	Internal

Materials...

Member Label	Youngs ksi	Density kcf	Thermal in/degr	Yield ksi
Default	1.00	0.000	0.000000	1.00
Steel	29,000.00	0.490	0.000007	50.00
Wood	1,800.00	0.035	0.000000	0.00

Wood Material Data...

Wood, Not Defined, Density= 35.0pcf, FbT= 1000psi, FbC= 1000psi, Fv= 1000psi, Ft= 1000, Fc= 400psi, E Bend XX= 1800ksi, E BendMin XX= 1800ksi, E Beny YY= 1800ksi, E BendMin YY= 1800ksi, E Axial= 1800ksi, Species= , Grade= Any, Class=

Member Sections...

Prop Label	Group Tag	Material	Area	Depth	Width	Ixx	Iyy
Default	Group	Default	1.0 in^2	0.0 in	0.0 in	1.0 in^4	1.0 in^4
HSS5x5x3/8	Column	Steel	6.180 in^2	5.0 in	5.0 in	21.70 in^4	21.70 in^4
HSS8x4x1/4	Beam	Steel	5.240 in^2	8.0 in	4.0 in	42.50 in^4	14.40 in^4

Joint Loads....

Note: Loads labeled "Global Y" act downward (in "-Y" direction)

Joint Label	Load Direction	Load Magnitude Dead	Roof Live	Live	Snow	Seismic	Wind	Earth	
2	Global X					0.30	1.140		k
2	Global Y	1.80	1.350		1.350		-3.090		k
3	Global Y	1.80	1.350		1.350		-3.090		k

Member Distributed Loads....

Note: Loads labeled "Global Y" act downward (in "-Y" direction)

Member Label	Load Direction	Load Extents Start / End ft		Load Magnitude Dead	Roof Live	Live	Snow	Seismic	Wind	Earth	
B	Global Y	0.0	Start Mag :	0.130	0.130		0.130		-0.2930		k/ft
		11.50	End Mag :	0.130	0.130		0.130		-0.2930		k/ft

Stress/Strength Load Combinations

ASCE 7-16

Load Combination Description	Cd	Load Combination Factors Dead	0.2*Sds* Seismic	Roof Live	Live	Snow	Wind	Seismic	Rho	Earth
+D+H	0.9	1.0								1.0

2D Frame

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Moment Frame Front of House

Stress/Strength Load Combinations

ASCE 7-16

Load Combination Description	Cd	Load Combination Factors								
		Dead	0.2*Sds* Seismic	Roof Live	Live	Snow	Wind	Seismic	Rho	Earth
+D+L+H	1	1.0			1.0					1.0
+D+Lr+H	1.25	1.0		1.0						1.0
+D+S+H	1.15	1.0				1.0				1.0
+D+0.750Lr+0.750L+H	1.25	1.0		0.750	0.750					1.0
+D+0.750L+0.750S+H	1.15	1.0			0.750	0.750				1.0
+D+0.60W+H	1.6	1.0					0.60			1.0
+D+0.750Lr+0.750L+0.450W+H	1.6	1.0		0.750	0.750		0.450			1.0
+D+0.750L+0.750S+0.450W+H	1.6	1.0			0.750	0.750	0.450			1.0
+0.60D+0.60W+0.60H	1.6	0.60					0.60			0.60
+D+0.70E+0.60H	0.9	1.0						0.70		0.60
+D+0.750L+0.750S+0.5250E+H	1.15	1.0			0.750	0.750		0.5250		1.0
+0.60D+0.70E+H	0.9	0.60						0.70		1.0

Reaction Load Combinations

ASCE 7-16

Load Combination Description	Load Combination Factors						
	Dead	Roof Live	Live	Snow	Wind	Seismic	Earth
+D+H	1.0						1.0
+D+L+H	1.0		1.0				1.0
+D+Lr+H	1.0	1.0					1.0
+D+S+H	1.0			1.0			1.0
+D+0.750Lr+0.750L+H	1.0	0.750	0.750				1.0
+D+0.750L+0.750S+H	1.0		0.750	0.750			1.0
+D+0.60W+H	1.0				0.60		1.0
+D+0.750Lr+0.750L+0.450W+H	1.0	0.750	0.750		0.450		1.0
+D+0.750L+0.750S+0.450W+H	1.0		0.750	0.750	0.450		1.0
+0.60D+0.60W+0.60H	0.60				0.60		0.60
+D+0.70E+0.60H	1.0					0.70	0.60
+D+0.750L+0.750S+0.5250E+H	1.0		0.750	0.750		0.5250	1.0
+0.60D+0.70E+H	0.60					0.70	1.0
D Only	1.0						
Lr Only		1.0					
L Only			1.0				
S Only				1.0			
W Only					1.0		
E Only						1.0	
H Only							1.0

Deflection Load Combinations

ASCE 7-16

Load Combination Description	Load Combination Factors						
	Dead	Roof Live	Live	Snow	Wind	Seismic	Earth
+D+H	1.0						1.0
+D+L+H	1.0		1.0				1.0
+D+Lr+H	1.0	1.0					1.0
+D+S+H	1.0			1.0			1.0
+D+0.750Lr+0.750L+H	1.0	0.750	0.750				1.0
+D+0.750L+0.750S+H	1.0		0.750	0.750			1.0
+D+0.60W+H	1.0				0.60		1.0
+D+0.750Lr+0.750L+0.450W+H	1.0	0.750	0.750		0.450		1.0
+D+0.750L+0.750S+0.450W+H	1.0		0.750	0.750	0.450		1.0
+0.60D+0.60W+0.60H	0.60				0.60		0.60
+D+0.70E+0.60H	1.0					0.70	0.60
+D+0.750L+0.750S+0.5250E+H	1.0		0.750	0.750		0.5250	1.0
+0.60D+0.70E+H	0.60					0.70	1.0
D Only	1.0						
Lr Only		1.0					
L Only			1.0				
S Only				1.0			
W Only					1.0		
E Only						1.0	
H Only							1.0

2D Frame

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Moment Frame Front of House

Extreme Joint Displacements

Only Load Combinations giving maximum values are listed

Joint Label		Joint Displacements X in	Y in	Z Radians
1	Max	0.0 W Only	0.0 W Only	0.0
1	Min	0.0 +D+Lr	0.0 +D+Lr	0.0
2	Max	1.887 W Only	0.005990 W Only	-0.000490 E Only
2	Min	.0000170 Lr Only	-0.004485 +D+Lr	-0.001303 +D+0.750S+0.5250E
3	Max	1.886 W Only	0.003230 W Only	0.001195 +D+Lr
3	Min	-.0000340 +D+Lr	-0.004485 +D+Lr	-0.003207 W Only
4	Max	0.0 W Only	0.0 W Only	0.0
4	Min	0.0 D Only	0.0 +D+Lr	0.0

Extreme Joint Reactions

Only Load Combinations giving maximum values are listed

Joint Label		Joint Reactions X k	Y k	Z k-ft
1	Max	0.07534 +D+Lr	4.645 +D+Lr	
	Min	-0.6550 W Only	-6.204 W Only	
2	Max			
	Min			
3	Max			
	Min			
4	Max	-0.03767 D Only	4.645 +D+Lr	
	Min	-0.4850 W Only	-3.345 W Only	

Extreme Member End Forces

Only Load Combinations giving maximum values are listed

Member Label	Member " I " End Forces Axial k	Shear k	Moment k-ft	Member " J " End Forces Axial k	Shear k	Moment k-ft
A *Max*	4.645 +D+Lr	0.3704 +0.60D+0.60W	0.0 D Only	2.194 +0.60D+0.60W	0.07534 +D+Lr	5.341 +0.60D+0.60W
A *Min*	-2.194 +0.60D+0.60W	-0.07534 +D+Lr	0.0 D Only	-4.645 +D+Lr	-0.3704 +0.60D+0.60W	-1.086 +D+Lr
B *Max*	0.3287 +D+0.60W	1.495 +D+Lr	1.086 +D+Lr	-0.03767 D Only	1.506 +D+0.750S+0.5250E	-0.5432 D Only
B *Min*	0.03767 D Only	-1.420 +0.60D+0.60W	-5.341 +0.60D+0.60W	-0.3287 +D+0.60W	0.2953 +0.60D+0.60W	-4.740 +D+0.60W
C *Max*	4.645 +D+Lr	0.3287 +D+0.60W	4.740 +D+0.60W	0.4787 +0.60D+0.60W	-0.03767 D Only	0.0 D Only
C *Min*	-0.4787 +0.60D+0.60W	0.03767 D Only	0.5432 D Only	-4.645 +D+Lr	-0.3287 +D+0.60W	0.0 D Only

2D Frame

Project File: 24-0121 - afc lee's summit.ec6

DESCRIPTION: Moment Frame Front of House

Extreme Member Forces

Only Load Combinations giving maximum values are listed

Mmbr Label	Axial	Dist from "I" Joint	Moment	Dist from "I" Joint	Shear	Dist from "I" Joint
A	2.194 k	0.0 ft	1.086 k-ft	14.420 ft	0.3704 k	0.0 ft
Max	*+0.60D+0.60W*		*+D+Lr*		*+0.60D+0.60W*	
A	-4.645 k	0.0 ft	-5.341 k-ft	14.420 ft	-0.07534 k	0.0 ft
Min	*+D+Lr*		*+0.60D+0.60W*		*+D+Lr*	
B	-0.03767 k	0.0 ft	4.740 k-ft	11.50 ft	1.495 k	0.0 ft
Max	*D Only*		*+D+0.60W*		*+D+Lr*	
B	-0.3287 k	0.0 ft	-5.341 k-ft	0.0 ft	-1.506 k	11.50 ft
Min	*+D+0.60W*		*+0.60D+0.60W*		*+D+0.750S+0.5250E*	
C	0.4787 k	0.0 ft	4.740 k-ft	0.0 ft	0.3287 k	0.0 ft
Max	*+0.60D+0.60W*		*+D+0.60W*		*+D+0.60W*	
C	-4.645 k	0.0 ft	0.01109 k-ft	14.126 ft	0.03767 k	0.0 ft
Min	*+D+Lr*		*D Only*		*D Only*	

Member Stress Checks...

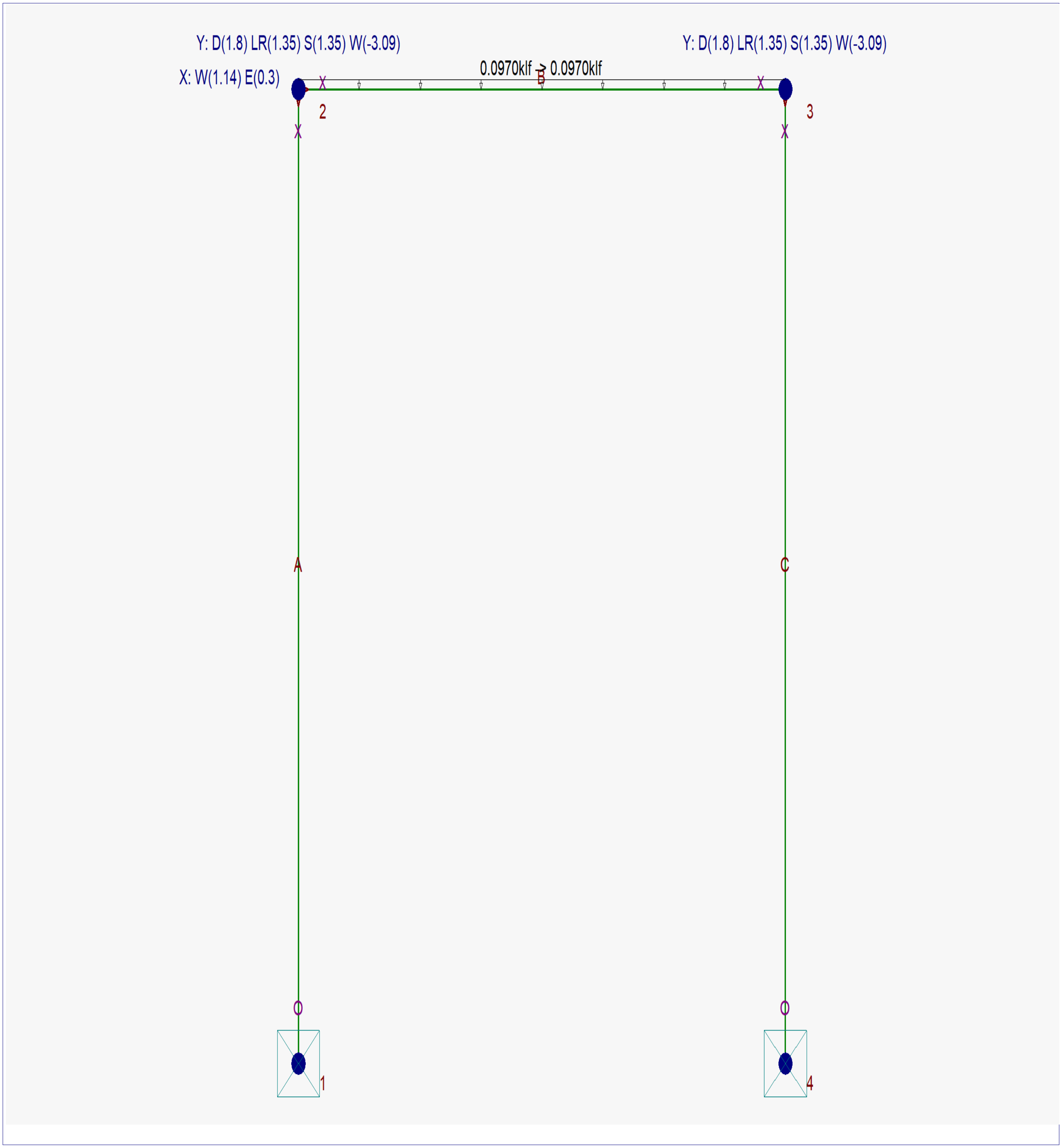
Stress Checks per AISC 360-16 & NDS 2018

Member Label	Section Label	Material	Max. Axial + Bending Stress Ratios				Max. Shear Stress Ratios			
			Load Combination	Ratio	Status	Dist (ft)	Load Combination	Ratio	Status	Dist (ft)
A	Column	Steel	+0.60D+0.60W	0.208	PASS	14.42	+0.60D+0.60W	0.007	PASS	0.00
B	Beam	Steel	+0.60D+0.60W	0.163	PASS	0.00	+D+0.750S+0.5250E	0.025	PASS	11.50
C	Column	Steel	+D+0.60W	0.182	PASS	0.00	+D+0.60W	0.007	PASS	0.00

2D Frame

Project File: 24-0121 - afc lee's summit.ec6

DESCRIPTION: Moment Frame Front of House



WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8578	3420	0.399	PASS
Chord capacity	lb/in^2	647	113	0.181	PASS
Deflection	in	0.360	0.241	0.669	PASS

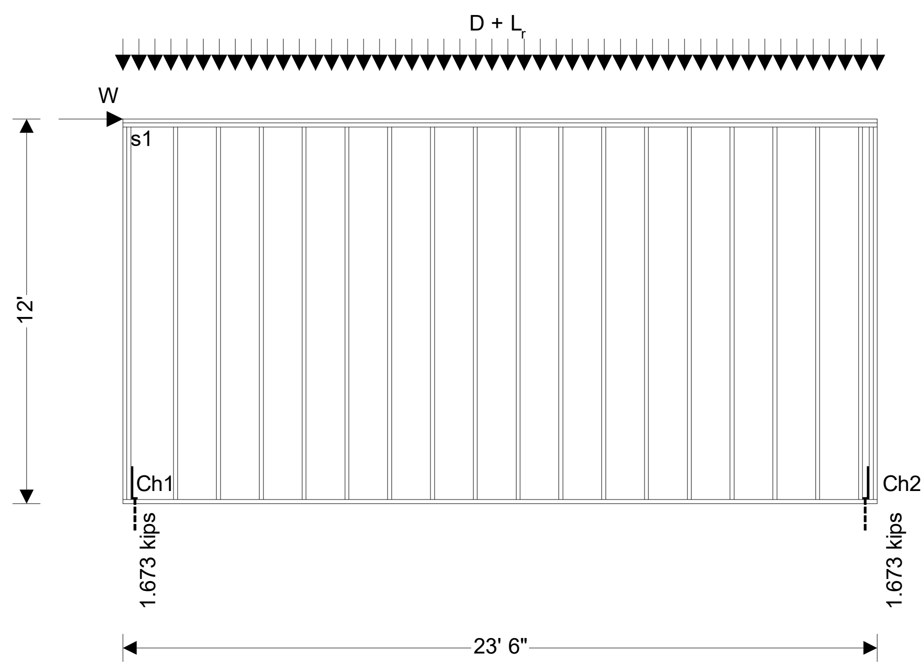
Panel details

Structural wood panel sheathing on one side

Panel height h = **12** ft

Panel length b = **23.5** ft

Total area of wall A = h × b = **282** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **34943** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	G = **0.50**
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Minimum modulus of elasticity	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = **520** lb/ft
Nominal unit shear capacity for wind design	v_w = **730** lb/ft
Apparent shear wall shear stiffness	G_a = **15** kips/in

Loading details

Dead load acting on top of panel	D = **40** lb/ft
Roof live load acting on top of panel	L_r = **40** lb/ft
Self weight of panel	S_{wt} = **12** lb/ft^2
In plane wind load acting at head of panel	W = **5700** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**

Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Bearing area factor - cl. 3.10.4	C_b = **1.0**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **696** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.27**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	b = **23.5** ft
Shear wall aspect ratio	h / b = **0.511**

Segmented shear wall capacity

Maximum shear force under wind loading	$V_{w\_max} = 0.6 \times W$ = **3.42** kips
Shear capacity for wind loading	$V_w = v_w \times b / 2$ = **8.578** kips
	V_{w_max} / V_w = **0.399**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio	h / b = **0.511**
Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W$ = **3.42** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times s / 2$ = **0.074** kips
Maximum tensile force in chord	$T = V \times h / b - P$ = **1.673** kips
Maximum applied tensile stress	$f_t = T / A_{en}$ = **124** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2
	f_t / F_t' = **0.104**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W$ = **3.42** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2$ = **0.123** kips
Maximum compressive force in chord	$C = V \times h / b + P$ = **1.869** kips
Maximum applied compressive stress	$f_c = C / A_e$ = **113** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **647** lb/in^2
	f_c / F_c' = **0.175**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate	$F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2
	f_c / F_{c_perp}' = **0.181**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1	T_1 = **1.673** kips
Chord 2	T_2 = **1.673** kips

Wind load deflection

Design shear force	$V_{\delta w} = f_{Wserv} \times W =$ **5.7** kips
Deflection limit	$\Delta_{w\_allow} = h / 400 =$ **0.36** in
Induced unit shear	$v_{\delta w} = V_{\delta w} / b =$ **242.55** lb/ft
Anchor tension force	$T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2) =$ **2.837** kips
Vertical elongation at anchor	$\Delta_a = T_{\delta} / k_a =$ **0.081** in
Shear wall deflection – Eqn. 4.3-1	$\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.241** in
	$\delta_{sww} / \Delta_{w\_allow} =$ **0.669**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	5703	1591	0.279	PASS
Chord capacity	lb/in^2	647	81	0.130	PASS
Deflection	in	0.360	0.185	0.513	PASS

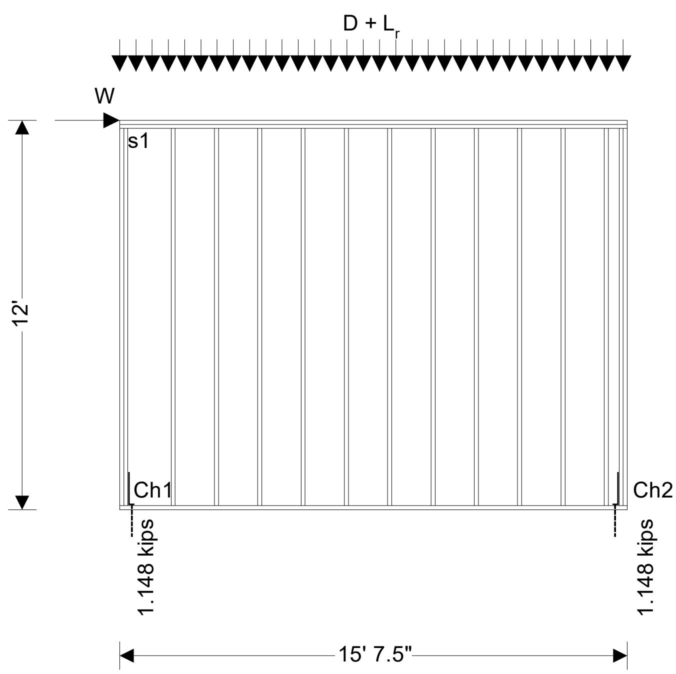
Panel details

Structural wood panel sheathing on one side

Panel height — h = **12** ft

Panel length — b = **15.625** ft

Total area of wall — A = h × b = **187.5** ft^2



Panel construction

Nominal stud size — 2" x 6"

Dressed stud size — 1.5" x 5.5"

Cross-sectional area of studs — A_s = **8.25** in^2

Stud spacing — s = **16** in

Nominal end post size — 2 x 2" x 6"

Dressed end post size — 2 x 1.5" x 5.5"

Cross-sectional area of end posts — A_e = **16.5** in^2

Hole diameter — Dia = **1** in

Net cross-sectional area of end posts — A_{en} = **13.5** in^2

Nominal collector size — 2 x 2" x 6"

Dressed collector size — 2 x 1.5" x 5.5"

Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **34943** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	G = **0.50**
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Minimum modulus of elasticity	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = **520** lb/ft
Nominal unit shear capacity for wind design	v_w = **730** lb/ft
Apparent shear wall shear stiffness	G_a = **15** kips/in

Loading details

Dead load acting on top of panel	D = **40** lb/ft
Roof live load acting on top of panel	L_r = **40** lb/ft
Self weight of panel	S_{wt} = **12** lb/ft^2
In plane wind load acting at head of panel	W = **2651** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Bearing area factor - cl. 3.10.4 C_b = **1.0**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **696** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.27**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **15.625** ft

Shear wall aspect ratio h / b = **0.768**

Segmented shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **1.591** kips

Shear capacity for wind loading $V_w = v_w \times b / 2$ = **5.703** kips

V_{w_max} / V_w = **0.279**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.768**

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **1.591** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2$ = **0.074** kips

Maximum tensile force in chord $T = V \times h / b - P$ = **1.148** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **85** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.071**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W$ = **1.591** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.123** kips

Maximum compressive force in chord $C = V \times h / b + P$ = **1.344** kips

Maximum applied compressive stress $f_c = C / A_e$ = **81** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **647** lb/in^2

f_c / F_c' = **0.126**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.130**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1 T_1 = **1.148** kips

Chord 2 T_2 = **1.148** kips

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W = \mathbf{2.651}$ kips

Deflection limit $\Delta_{w\_allow} = h / 400 = \mathbf{0.36}$ in

Induced unit shear $v_{\delta w} = V_{\delta w} / b = \mathbf{169.66}$ lb/ft

Anchor tension force $T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2) = \mathbf{1.962}$ kips

Vertical elongation at anchor $\Delta_a = T_{\delta} / k_a = \mathbf{0.056}$ in

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.185}$ in

$\delta_{sww} / \Delta_{w\_allow} = \mathbf{0.513}$

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	10158	60	0.006	PASS
Chord capacity	lb/in^2	647	14	0.023	PASS
Deflection	in	0.360	0.003	0.008	PASS

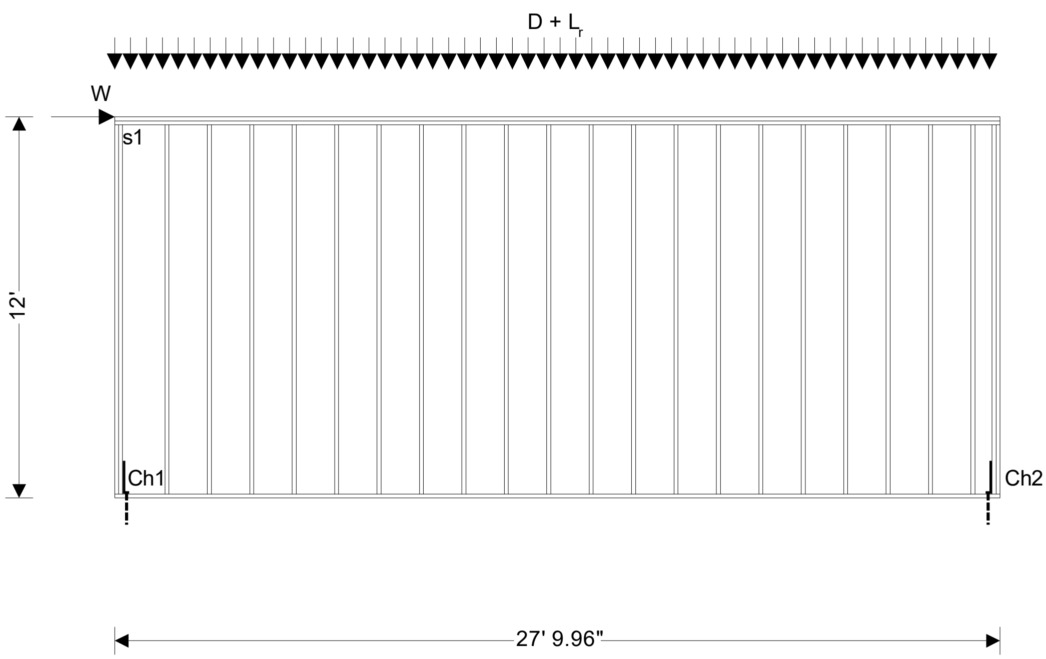
Panel details

Structural wood panel sheathing on one side

Panel height h = **12** ft

Panel length b = **27.83** ft

Total area of wall A = h × b = **333.96** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **34943** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	G = **0.50**
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Minimum modulus of elasticity	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = **520** lb/ft
Nominal unit shear capacity for wind design	v_w = **730** lb/ft
Apparent shear wall shear stiffness	G_a = **15** kips/in

Loading details

Dead load acting on top of panel	D = **100** lb/ft
Roof live load acting on top of panel	L_r = **100** lb/ft
Self weight of panel	S_{wt} = **12** lb/ft^2
In plane wind load acting at head of panel	W = **100** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Bearing area factor - cl. 3.10.4 C_b = **1.0**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **696** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.27**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **27.83** ft

Shear wall aspect ratio h / b = **0.431**

Segmented shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **0.06** kips

Shear capacity for wind loading $V_w = v_w \times b / 2$ = **10.158** kips

V_{w_max} / V_w = **0.006**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.431**

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **0.06** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2$ = **0.098** kips

Maximum tensile force in chord $T = V \times h / b - P$ = **-0.072** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **-5** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.004**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression $V = 0.45 \times W$ = **0.045** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2$ = **0.213** kips

Maximum compressive force in chord $C = V \times h / b + P$ = **0.232** kips

Maximum applied compressive stress $f_c = C / A_e$ = **14** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **647** lb/in^2

f_c / F_c' = **0.022**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.023**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W$ = **0.1** kips

Deflection limit $\Delta_{w\_allow} = h / 400$ = **0.36** in

Induced unit shear $v_{\delta w} = V_{\delta w} / b =$ **3.59** lb/ft

Anchor tension force $T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2) =$ **0.000** kips

Vertical elongation at anchor $\Delta_a = T_{\delta} / k_a =$ **0.000** in

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.003** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.008**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8150	4620	0.567	PASS
Chord capacity	lb/in^2	647	163	0.260	PASS
Deflection	in	0.360	0.346	0.960	PASS

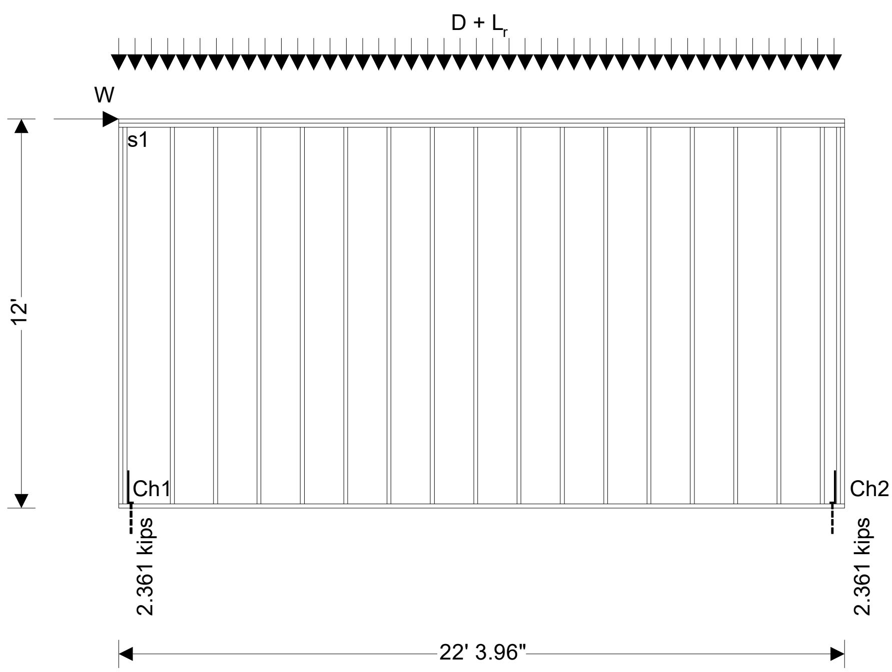
Panel details

Structural wood panel sheathing on one side

Panel height — h = **12** ft

Panel length — b = **22.33** ft

Total area of wall — A = h × b = **267.96** ft^2



Panel construction

Nominal stud size — 2" x 6"

Dressed stud size — 1.5" x 5.5"

Cross-sectional area of studs — A_s = **8.25** in^2

Stud spacing — s = **16** in

Nominal end post size — 2 x 2" x 6"

Dressed end post size — 2 x 1.5" x 5.5"

Cross-sectional area of end posts — A_e = **16.5** in^2

Hole diameter — Dia = **1** in

Net cross-sectional area of end posts — A_{en} = **13.5** in^2

Nominal collector size — 2 x 2" x 6"

Dressed collector size — 2 x 1.5" x 5.5"

Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **34943** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	G = **0.50**
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Minimum modulus of elasticity	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = **520** lb/ft
Nominal unit shear capacity for wind design	v_w = **730** lb/ft
Apparent shear wall shear stiffness	G_a = **15** kips/in

Loading details

Dead load acting on top of panel	D = **160** lb/ft
Roof live load acting on top of panel	L_r = **160** lb/ft
Self weight of panel	S_{wt} = **12** lb/ft^2
In plane wind load acting at head of panel	W = **7700** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	
	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	
	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	
	C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Bearing area factor - cl. 3.10.4 C_b = **1.0**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **696** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.27**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **22.33** ft

Shear wall aspect ratio h / b = **0.537**

Segmented shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **4.62** kips

Shear capacity for wind loading $V_w = v_w \times b / 2$ = **8.15** kips

V_{w_max} / V_w = **0.567**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.537**

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **4.62** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2$ = **0.122** kips

Maximum tensile force in chord $T = V \times h / b - P$ = **2.361** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **175** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.146**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W$ = **4.62** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.203** kips

Maximum compressive force in chord $C = V \times h / b + P$ = **2.685** kips

Maximum applied compressive stress $f_c = C / A_e$ = **163** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **647** lb/in^2

f_c / F_c' = **0.252**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.260**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1 T_1 = **2.361** kips

Chord 2 T_2 = **2.361** kips

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **7.7** kips

Deflection limit $\Delta_{w\_allow} = h / 400 =$ **0.36** in

Induced unit shear $v_{\delta w} = V_{\delta w} / b =$ **344.83** lb/ft

Anchor tension force $T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2) =$ **4.016** kips

Vertical elongation at anchor $\Delta_a = T_{\delta} / k_a =$ **0.115** in

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.346** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.96**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9933	4086	0.411	PASS
Chord capacity	lb/in^2	647	217	0.348	PASS
Deflection	in	0.360	0.292	0.812	PASS

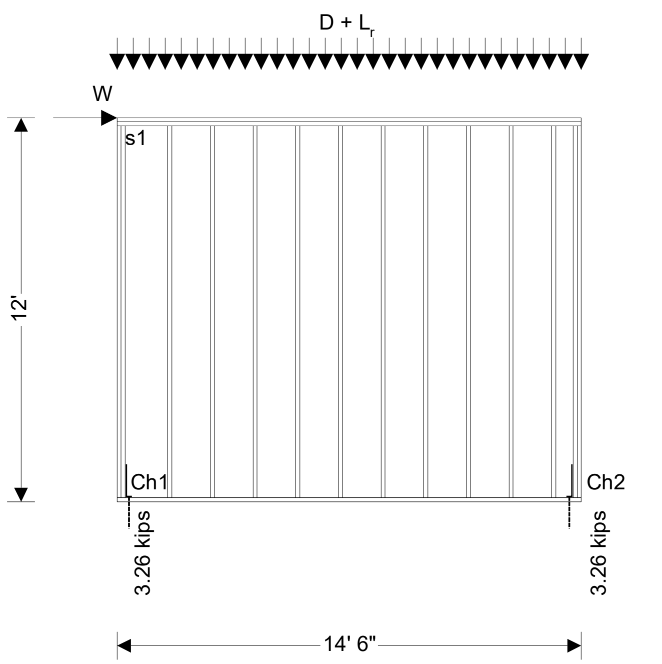
Panel details

Structural wood panel sheathing on one side

Panel height h = **12** ft

Panel length b = **14.5** ft

Total area of wall A = h × b = **174** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **61500** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	G = **0.50**
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Minimum modulus of elasticity	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = **980** lb/ft
Nominal unit shear capacity for wind design	v_w = **1370** lb/ft
Apparent shear wall shear stiffness	G_a = **28** kips/in

Loading details

Dead load acting on top of panel	D = **160** lb/ft
Roof live load acting on top of panel	L_r = **160** lb/ft
Self weight of panel	S_{wt} = **12** lb/ft^2
In plane wind load acting at head of panel	W = **6810** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Bearing area factor - cl. 3.10.4 C_b = **1.0**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **696** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.27**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Shear wall length b = **14.5** ft

Shear wall aspect ratio h / b = **0.828**

Segmented shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **4.086** kips

Shear capacity for wind loading $V_w = v_w \times b / 2$ = **9.933** kips

V_{w_max} / V_w = **0.411**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **0.828**

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **4.086** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2$ = **0.122** kips

Maximum tensile force in chord $T = V \times h / b - P$ = **3.260** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **241** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.202**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W$ = **4.086** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.203** kips

Maximum compressive force in chord $C = V \times h / b + P$ = **3.584** kips

Maximum applied compressive stress $f_c = C / A_e$ = **217** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **647** lb/in^2

f_c / F_c' = **0.336**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.348**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1 T_1 = **3.26** kips

Chord 2 T_2 = **3.26** kips

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **6.81** kips

Deflection limit $\Delta_{w\_allow} = h / 400 =$ **0.36** in

Induced unit shear $v_{\delta w} = V_{\delta w} / b =$ **469.66** lb/ft

Anchor tension force $T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2) =$ **5.514** kips

Vertical elongation at anchor $\Delta_a = T_{\delta} / k_a =$ **0.090** in

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.292** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.812**

PASS - Shear wall deflection is less than deflection limit

PFD
Post-to-Foundation Designer

Input

Condition

Design Post & Holdown	Design Anchorage

Load Demand

Uplift Load (ASD)	Load Type
2361	Wind and SDC A&B

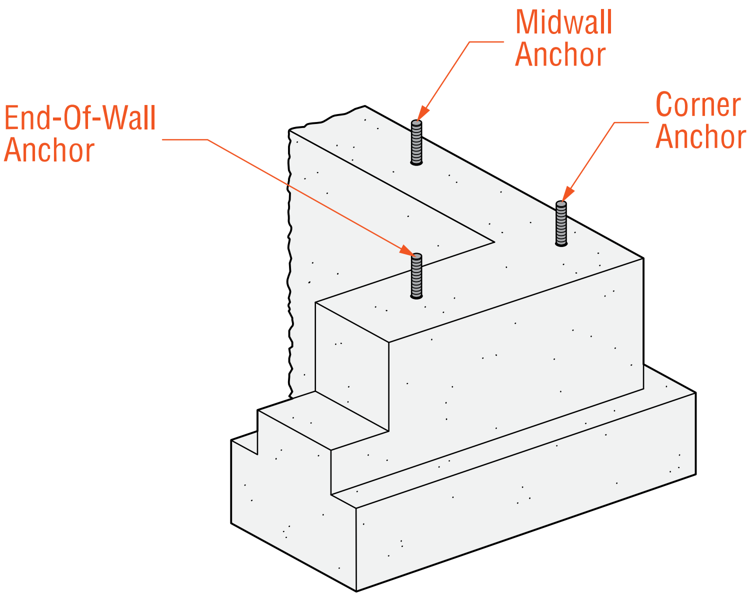
Post & Holdown Design Criteria

Holdown Subtype	Holdown Fasteners	Holdown Fasteners Size	Lumber Species	Wall Thickness
Embedded Anchor	Any	Any	Douglas Fir-Larch (DF)	6"

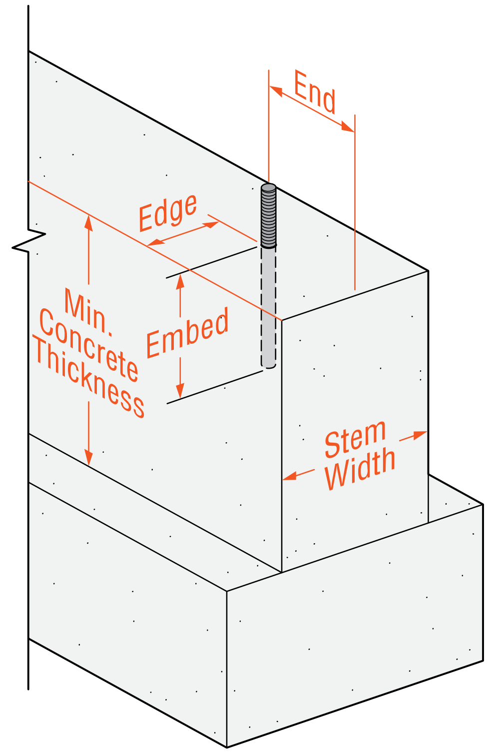
Anchorage Design Criteria

Material Type	Compressive Strength	Anchor Type	Foundation Type	Application
Concrete	4,000 psi	Cast-in-Place Anchors	Stemwall	End-of-Wall Anchor

Foundation Type Details



Anchor in a Stemwall Foundation



Anchor in a Stemwall Foundation

Output

Summary

Search: HDU2

Show Optimized Models: No

Post Size (min)	Holdown Model	Holdown Allowable Load (lbs)	Deflection at Demand Load (in)	Holdown Fasteners	Anchor Product	Anchor Steel Grade	Anchor Diameter (in)	Anchor Allowable Load (lbs)	Min Embedment Depth (in)	Min Edge Distance (in)	Min End Distance (in)	Min Concrete Thickness (in)	Min Stemwall Width (in)	Installed Cost
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	2865	4.00	2.75	6.00	6.125	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	2695	4.00	1.75	6.00	6.125	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	2695	4.00	1.75	6.00	6.125	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3820	4.00	2.75	6.00	6.125	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3820	4.00	2.75	6.00	6.125	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	2865	4.00	2.75	6.00	6.125	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3570	4.25	1.75	6.00	6.375	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3570	4.25	1.75	6.00	6.375	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	2575	6.25	2.75	6.00	8.375	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	2470	6.25	1.75	6.00	8.375	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3435	6.25	2.75	6.00	8.375	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3295	6.25	1.75	6.00	8.375	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3415	12.50	1.75	12.00	14.625	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	2750	12.50	1.75	6.00	14.625	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	4555	12.50	1.75	12.00	14.625	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	2810	12.50	2.75	6.00	14.625	6.00	+19.00%

Post Size (min)	Holdown Model	Holdown Allowable Load (lbs)	Deflection at Demand Load (in)	Holdown Fasteners	Anchor Product	Anchor Steel Grade	Anchor Diameter (in)	Anchor Allowable Load (lbs)	Min Embedment Depth (in)	Min Edge Distance (in)	Min End Distance (in)	Min Concrete Thickness (in)	Min Stemwall Width (in)	Installed Cost
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3745	12.50	2.75	6.00	14.625	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	4655	12.50	2.75	12.00	14.625	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3665	12.50	1.75	6.00	14.625	8.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	PAB5-36	F1554_GR36	0.625	3490	12.50	2.75	12.00	14.625	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	SSTB16	F1554_GR36	0.625	3465	12.625	1.75	5.00	15.625	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	SSTB20	F1554_GR36	0.625	3880	16.625	1.75	5.00	19.625	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	SB5/8X24	F1554_GR36	0.625	6550	18.00	1.75	4.25	21.00	6.00	+19.00%
(2) 2x4	HDU2-SDS2.5	3,075	0.068	(6) ¼" x 2½" SDS Screw	SSTB24	F1554_GR36	0.625	4295	20.625	1.75	5.00	23.625	6.00	+19.00%

Table Notes

- Refer to General Notes for Holdowns and Tension Ties for installation instructions, best practices and recommendations by Simpson Strong-Tie.
- Foundation dimensions are for anchorage only. Foundation design (size and reinforcement) must be performed separately by designer.
- Where loads are governed by anchor steel, the design provisions from AISC 360 are used to determine the tensile steel limit.
- SSTB is suitable for monolithic and two-pour concrete applications.
- Nuts and washers for holdown attachment are not supplied with the SSTB; install standard nuts, couplers and/or washers as required.
- On HDG SSTB anchors, chase the threads to use standard nuts or couplers or use overtapped products in accordance with ASTM A563, for example Simpson Strong-Tie NUT5/8-OST, NUT7/8-OST, CNW5/8-OST, CNW7/8-OST.
- Install SSTB before the concrete pour using AnchorMate® anchor bolt holders. Install the SSTB per the plan view detail.
- Order SSTBL models (example: SSTB16L) for longer thread length (16L = 5 1/2", 20L = 6 1/2", 24L = 6", 28L = 6 1/2"). SSTB and SSTBL load values are the same. SSTB34 and SSTB36 feature 4 1/2" and 6 1/2" of thread respectively and are not available in "L" versions.
- For installations in severe corrosion environments, refer to General Corrosion Risks for additional considerations.
- HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).

- Allowable load for HTT5 with 0.162"x1 1/2" fasteners and a BP5/8-2 bearing-plate washer installed in the seat of the holdown is 5,295 lb. for DF/SP and 4,555 lb. for SPF/HF.
- To achieve published loads, machine bolts shall be installed with the nut on the opposite side of the holdown. If this orientation is reversed, the Designer shall reduce the allowable loads shown per NDS requirements when bolt threads are in the shear plane.
- The HTT5-¾ is designed to use a ¾"-diameter anchor bolt.
- HTT5KT is sold as a kit with the holdown, bearing plate washer and Strong-Drive SD Connector screws.
- Lag or carriage bolts are not permitted.
- Anchorage design is based on the strength design provision of ACI 318-14/-11/-08 assuming dry normal weight concrete, periodic inspection and short-term loading.
- Post-Installed anchorage shall be designed to satisfy the ductility requirements of ACI 318-14 Section 17.2.3.4.3 for structures assigned to Seismic Design Category C, D, E, or F when earthquake force exceeds 20 percent of the total anchor force. If ductility requirements are not met, an overstrength factor (Ω_0) per ASCE 7 table 12.2-1 shall be applied.
- Minimum Concrete Thickness is assumed to be the bottom of the stemwall, in situations where a cold joint exists
- Installed Cost Index (ICI) does not account for the cost of the anchor or its install, only the holdown.

PFD
Post-to-Foundation Designer

Input

Condition

Design Post & Holdown	Design Anchorage

Load Demand

Uplift Load (ASD)	Load Type
3260	Wind and SDC A&B

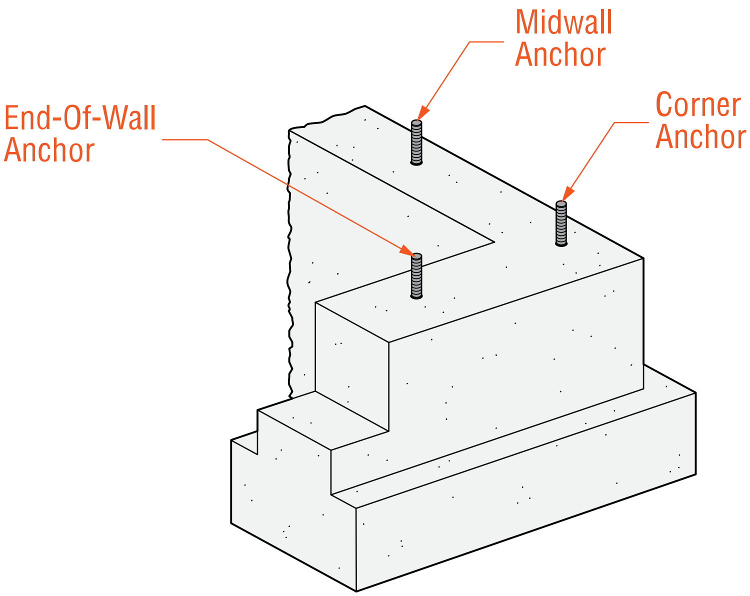
Post & Holdown Design Criteria

Holdown Subtype	Holdown Fasteners	Holdown Fasteners Size	Lumber Species	Wall Thickness
Embedded Anchor	Any	Any	Douglas Fir-Larch (DF)	6"

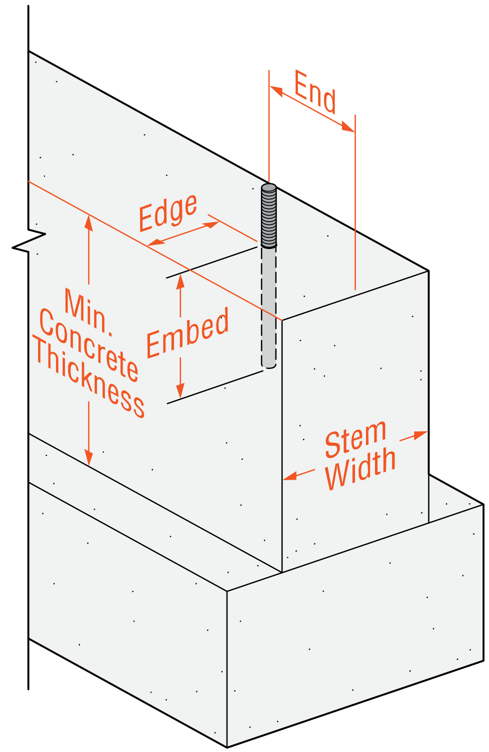
Anchorage Design Criteria

Material Type	Compressive Strength	Anchor Type	Foundation Type	Application
Concrete	4,000 psi	Cast-in-Place Anchors	Stemwall	End-of-Wall Anchor

Foundation Type Details



Anchor in a Stemwall Foundation



Anchor in a Stemwall Foundation

Output

Summary

Search: HDU8

Show Optimized Models: No

Post Size (min)	Holdown Model	Holdown Allowable Load (lbs)	Deflection at Demand Load (in)	Holdown Fasteners	Anchor Product	Anchor Steel Grade	Anchor Diameter (in)	Anchor Allowable Load (lbs)	Min Embedment Depth (in)	Min Edge Distance (in)	Min End Distance (in)	Min Concrete Thickness (in)	Min Stemwall Width (in)	Installed Cost
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3390	8.75	1.75	6.00	11.125	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3495	8.75	2.75	6.00	11.125	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	4290	17.50	2.75	6.00	19.875	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	5010	17.50	1.75	12.00	19.875	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	4225	17.50	1.75	6.00	19.875	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	5090	17.50	2.75	12.00	19.875	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3755	17.50	1.75	12.00	19.875	6.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3815	17.50	2.75	12.00	19.875	6.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	SB7/8X24	F1554_GR36	0.875	6550	18.00	1.75	4.25	21.00	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	SB7/8X24	F1554_GR36	0.875	6935	18.00	1.75	16.00	21.00	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	SSTB28	F1554_GR36	0.875	7310	24.875	1.75	5.00	27.875	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	SSTB28	F1554_GR36	0.875	6735	24.875	1.75	4.00	27.875	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	SSTB36	F1554_GR36	0.875	7310	28.875	1.75	5.00	31.875	8.00	+137.00%
(1) 4x4	HDU8-SDS2.5	6,970	0.054	(20) ¼" x 2½" SDS Screw	SSTB34	F1554_GR36	0.875	7310	28.875	1.75	5.00	31.875	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3390	8.75	1.75	6.00	11.125	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3495	8.75	2.75	6.00	11.125	8.00	+137.00%

Post Size (min)	Holdown Model	Holdown Allowable Load (lbs)	Deflection at Demand Load (in)	Holdown Fasteners	Anchor Product	Anchor Steel Grade	Anchor Diameter (in)	Anchor Allowable Load (lbs)	Min Embedment Depth (in)	Min Edge Distance (in)	Min End Distance (in)	Min Concrete Thickness (in)	Min Stemwall Width (in)	Installed Cost
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	4290	17.50	2.75	6.00	19.875	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	5010	17.50	1.75	12.00	19.875	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	4225	17.50	1.75	6.00	19.875	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	5090	17.50	2.75	12.00	19.875	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3755	17.50	1.75	12.00	19.875	6.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3815	17.50	2.75	12.00	19.875	6.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	SB7/8X24	F1554_GR36	0.875	6550	18.00	1.75	4.25	21.00	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	SB7/8X24	F1554_GR36	0.875	6935	18.00	1.75	16.00	21.00	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	SSTB28	F1554_GR36	0.875	7310	24.875	1.75	5.00	27.875	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	SSTB28	F1554_GR36	0.875	6735	24.875	1.75	4.00	27.875	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	SSTB36	F1554_GR36	0.875	7310	28.875	1.75	5.00	31.875	8.00	+137.00%
(2) 2x4	HDU8-SDS2.5	6,765	0.053	(20) ¼" x 2½" SDS Screw	SSTB34	F1554_GR36	0.875	7310	28.875	1.75	5.00	31.875	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3390	8.75	1.75	6.00	11.125	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3495	8.75	2.75	6.00	11.125	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	4290	17.50	2.75	6.00	19.875	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	5010	17.50	1.75	12.00	19.875	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	4225	17.50	1.75	6.00	19.875	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	5090	17.50	2.75	12.00	19.875	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3755	17.50	1.75	12.00	19.875	6.00	+137.00%

Post Size (min)	Holdown Model	Holdown Allowable Load (lbs)	Deflection at Demand Load (in)	Holdown Fasteners	Anchor Product	Anchor Steel Grade	Anchor Diameter (in)	Anchor Allowable Load (lbs)	Min Embedment Depth (in)	Min Edge Distance (in)	Min End Distance (in)	Min Concrete Thickness (in)	Min Stemwall Width (in)	Installed Cost
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	PAB7-36	F1554_GR36	0.875	3815	17.50	2.75	12.00	19.875	6.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	SB7/8X24	F1554_GR36	0.875	6550	18.00	1.75	4.25	21.00	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	SB7/8X24	F1554_GR36	0.875	6935	18.00	1.75	16.00	21.00	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	SSTB28	F1554_GR36	0.875	7310	24.875	1.75	5.00	27.875	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	SSTB28	F1554_GR36	0.875	6735	24.875	1.75	4.00	27.875	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	SSTB36	F1554_GR36	0.875	7310	28.875	1.75	5.00	31.875	8.00	+137.00%
(3) 2x4	HDU8-SDS2.5	7,870	0.047	(20) ¼" x 2½" SDS Screw	SSTB34	F1554_GR36	0.875	7310	28.875	1.75	5.00	31.875	8.00	+137.00%

Table Notes

- Refer to <u>General Notes for Holdowns and Tension Ties</u> for installation instructions, best practices and recommendations by Simpson Strong-Tie.
- Foundation dimensions are for anchorage only. Foundation design (size and reinforcement) must be performed separately by designer.
- Where loads are governed by anchor steel, the design provisions from AISC 360 are used to determine the tensile steel limit.
- SSTB is suitable for monolithic and two-pour concrete applications.
- Nuts and washers for holdown attachment are not supplied with the SSTB; install standard nuts, couplers and/or washers as required.
- On HDG SSTB anchors, chase the threads to use standard nuts or couplers or use overtapped products in accordance with ASTM A563, for example Simpson Strong-Tie NUT5/8-OST, NUT7/8-OST, CNW5/8-OST, CNW7/8-OST.
- Install SSTB before the concrete pour using AnchorMate® anchor bolt holders. Install the SSTB per the plan view detail.
- Order SSTBL models (example: SSTB16L) for longer thread length (16L = 5 1/2", 20L = 6 1/2", 24L = 6", 28L = 6 1/2"). SSTB and SSTBL load values are the same. SSTB34 and SSTB36 feature 4 1/2" and 6 1/2" of thread respectively and are not available in "L" versions.
- For installations in severe corrosion environments, refer to <u>General Corrosion Risks</u> for additional considerations.
- HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).
- Allowable load for HTT5 with 0.162"x1 1/2" fasteners and a BP5/8-2 bearing-plate washer installed in the seat of the holdown is 5,295 lb. for DF/SP and 4,555 lb. for SPF/HF.
- The HTT5-¾ is designed to use a ¾"-diameter anchor bolt.

- To achieve published loads, machine bolts shall be installed with the nut on the opposite side of the holdown. If this orientation is reversed, the Designer shall reduce the allowable loads shown per NDS requirements when bolt threads are in the shear plane.
- Lag or carriage bolts are not permitted.
- HTT5KT is sold as a kit with the holdown, bearing plate washer and Strong-Drive SD Connector screws.
- Anchorage design is based on the strength design provision of ACI 318-14/-11/-08 assuming dry normal weight concrete, periodic inspection and short-term loading.
- Post-Installed anchorage shall be designed to satisfy the ductility requirements of ACI 318-14 Section 17.2.3.4.3 for structures assigned to Seismic Design Category C, D, E, or F when earthquake force exceeds 20 percent of the total anchor force. If ductility requirements are not met, an overstrength factor (Ω_0) per ASCE 7 table 12.2-1 shall be applied.
- Minimum Concrete Thickness is assumed to be the bottom of the stemwall, in situations where a cold joint exists
- Installed Cost Index (ICI) does not account for the cost of the anchor or its install, only the holdown.

Company:		Date:	4/26/2024
Engineer:	JCF	Page:	1/6
Project:	AFC - Lee's Summit		
Address:			
Phone:			
E-mail:			

Anchor Designer™
Software
Version 3.2.2311.2

1.Project information

Project description: Anchor Rods for Patio Canopy Colums (Max loads) / Footing P6.5
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.250
Effective Embedment depth, h_{ef} (inch): 12.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 15.00
C_{min} (inch): 7.50
S_{min} (inch): 7.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 22.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.4
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 20.00 x 20.00 x 1.25

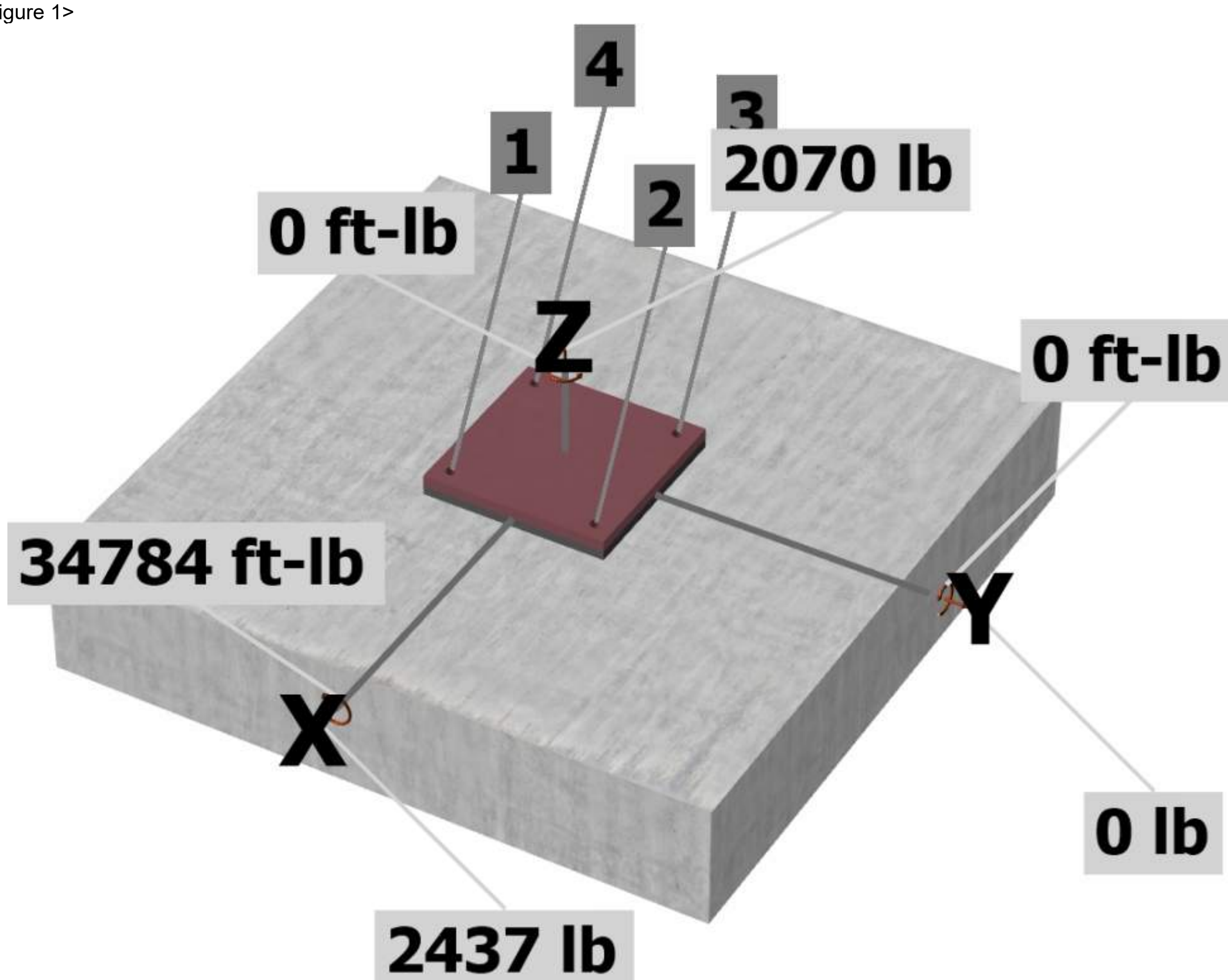
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB10 (1 1/4"Ø)

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

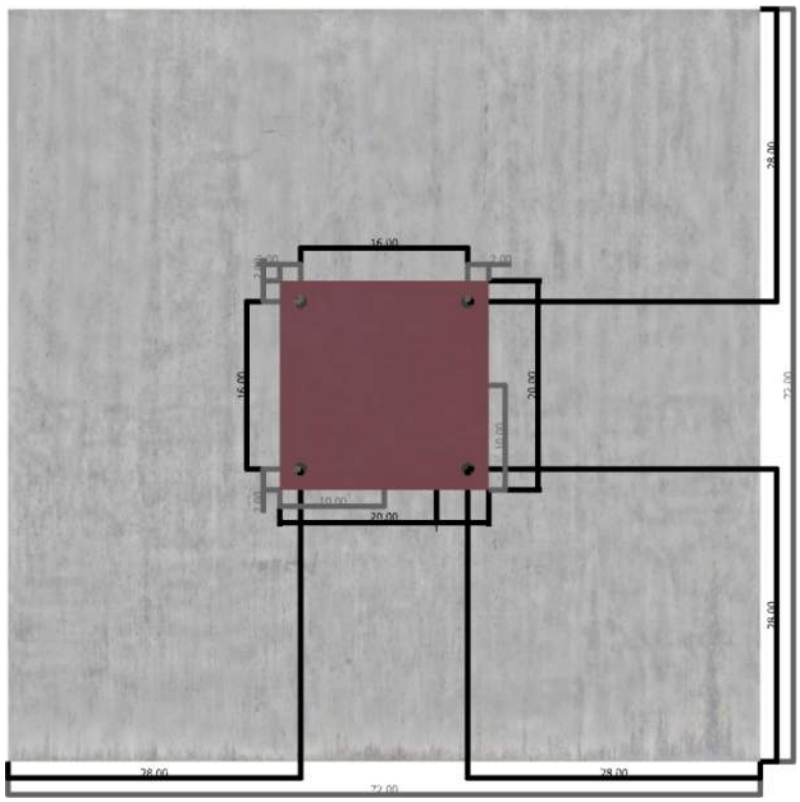
Strength level loads:

N_{ua} [lb]: -2070
V_{uax} [lb]: 2437
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 34784
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>

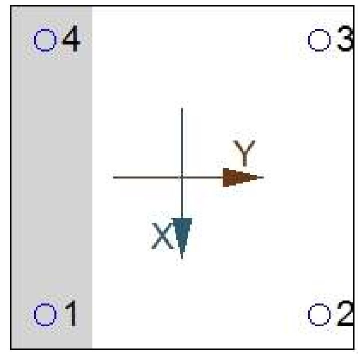


3. Resulting Anchor Forces

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	0.0	609.3	0.0	609.3
2	12178.2	609.3	0.0	609.3
3	12178.2	609.3	0.0	609.3
4	0.0	609.3	0.0	609.3
Sum	24356.4	2437.0	0.0	2437.0

Maximum concrete compression strain (‰): 0.13
Maximum concrete compression stress (psi): 558
Resultant tension force (lb): 24356
Resultant compression force (lb): 26426
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>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
56200	0.75	42150

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.4.2.2b)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	4000	12.000	63648

$\phi N_{cbg} = \phi\ (A_{Nc} / A_{Nco})\ \Psi_{ec,N}\ \Psi_{ed,N}\ \Psi_{c,N}\ \Psi_{cp,N} N_b$ (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)
2192.25	1296.00	28.00	1.000	1.000	1.25	1.000	63648	0.70	94206

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (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)
1.4	8.39	4000	0.70	263236

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
33720	0.8	0.65	17534

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (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)
10.00	1.250	1.00	4000	18.67	45906

$\phi V_{cbgx} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & 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_{cbgx} (lb)
1584.00	1568.00	1.000	1.000	1.400	1.128	45906	0.70	51271

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (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)
10.00	1.250	1.00	4000	18.67	45906

$\phi V_{cbgy} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (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)
1584.00	1568.00	1.000	1.000	1.400	1.128	45906	0.70	102543

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (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_{cpg} (lb)
2.0	3080.25	1296.00	1.000	1.000	1.250	1.000	63648	0.70	264731

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	**12178**	**42150**	**0.29**	**Pass (Governs)**
Concrete breakout	24356	94206	0.26	Pass
Pullout	12178	263236	0.05	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	609	17534	0.03	Pass
T Concrete breakout x+	**2437**	**51271**	**0.05**	**Pass (Governs)**
|| Concrete breakout y+	**1219**	**102543**	**0.01**	**Pass (Governs)**
Pryout	2437	264731	0.01	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6.1	0.29	0.00	28.9%	1.0	Pass

PAB10 (1 1/4"Ø) with hef = 12.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

Anchor Designer™
Software
Version 3.2.2311.2

Company:		Date:	4/26/2024
Engineer:	JCF	Page:	1/5
Project:	AFC - Lee's Summit		
Address:			
Phone:			
E-mail:			

1.Project information

Project description: Anchor Rods for Colums B1 / Footing P3
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 9.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.25
C_{min} (inch): 4.50
S_{min} (inch): 4.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 22.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 12.00 x 12.00 x 1.00

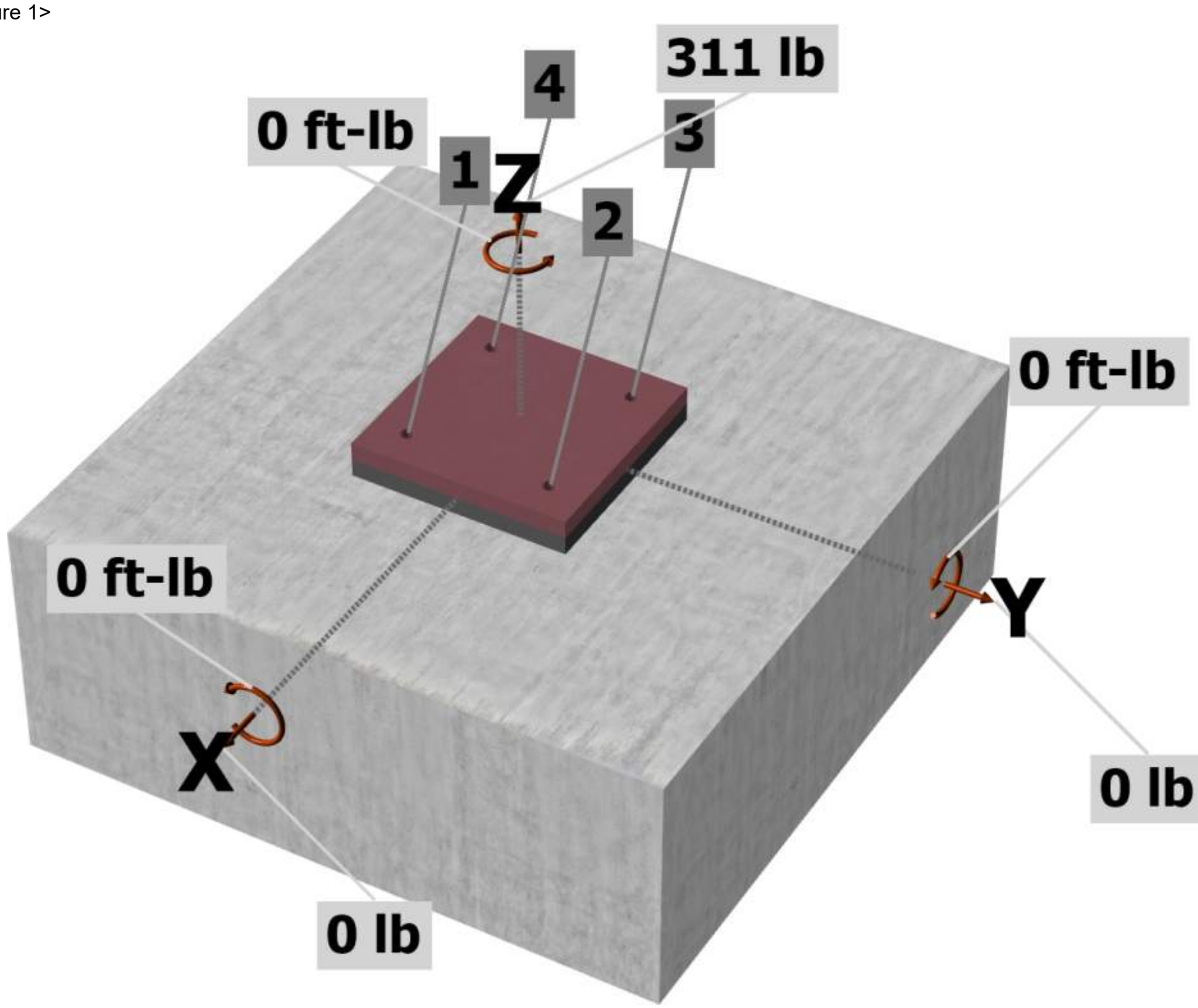
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB6 (3/4"Ø)

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:

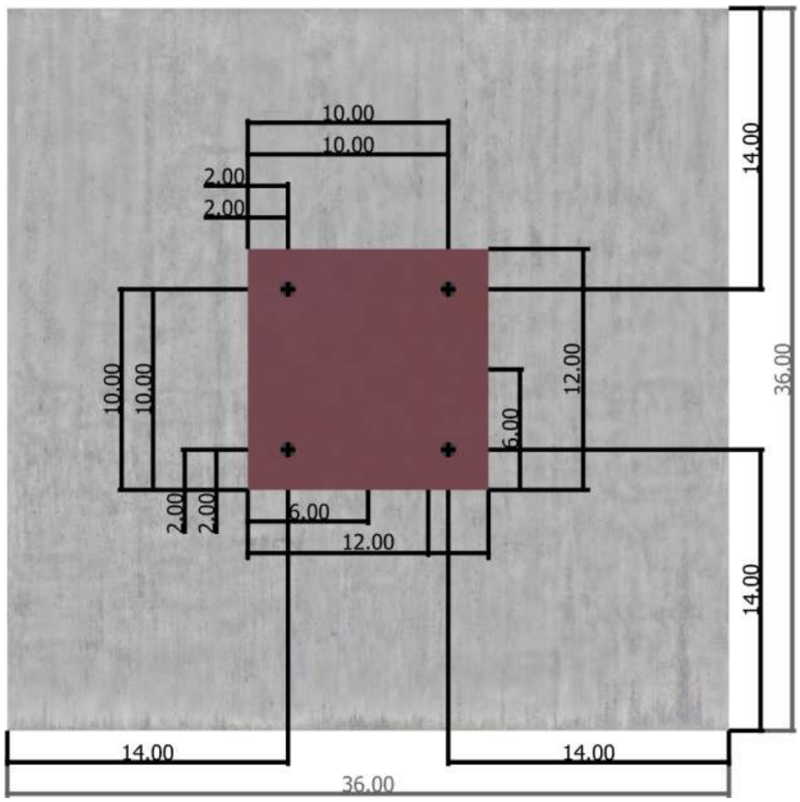
N_{ua} [lb]: 311
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



Company:		Date:	4/26/2024
Engineer:	JCF	Page:	3/5
Project:	AFC - Lee's Summit		
Address:			
Phone:			
E-mail:			

<Figure 2>

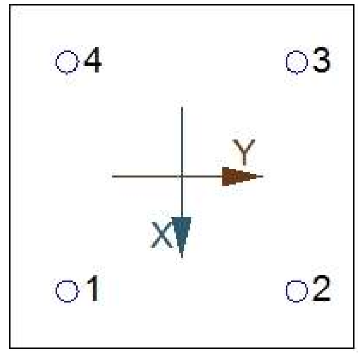


3. Resulting Anchor Forces

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	77.8	0.0	0.0	0.0
2	77.8	0.0	0.0	0.0
3	77.8	0.0	0.0	0.0
4	77.8	0.0	0.0	0.0
Sum	311.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 311
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

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	9.000	40983

$\phi N_{cbg} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (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)
1296.00	729.00	14.00	1.000	1.000	1.25	1.000	40983	0.70	63752

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (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)
1.4	3.53	4000	0.70	110826

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	**78**	**14528**	**0.01**	**Pass (Governs)**
Concrete breakout	311	63752	0.00	Pass
Pullout	78	110826	0.00	Pass

PAB6 (3/4"Ø) with hef = 9.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

Anchor Designer™
Software
Version 3.2.2311.2

Company:		Date:	4/26/2024
Engineer:	JCF	Page:	1/6
Project:	AFC - Lee's Summit		
Address:			
Phone:			
E-mail:			

1.Project information

Project description: Anchor Rods for Column F3 / Footing P3.5
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 9.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.25
C_{min} (inch): 4.50
S_{min} (inch): 4.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 22.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.4
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 12.00 x 12.00 x 0.75

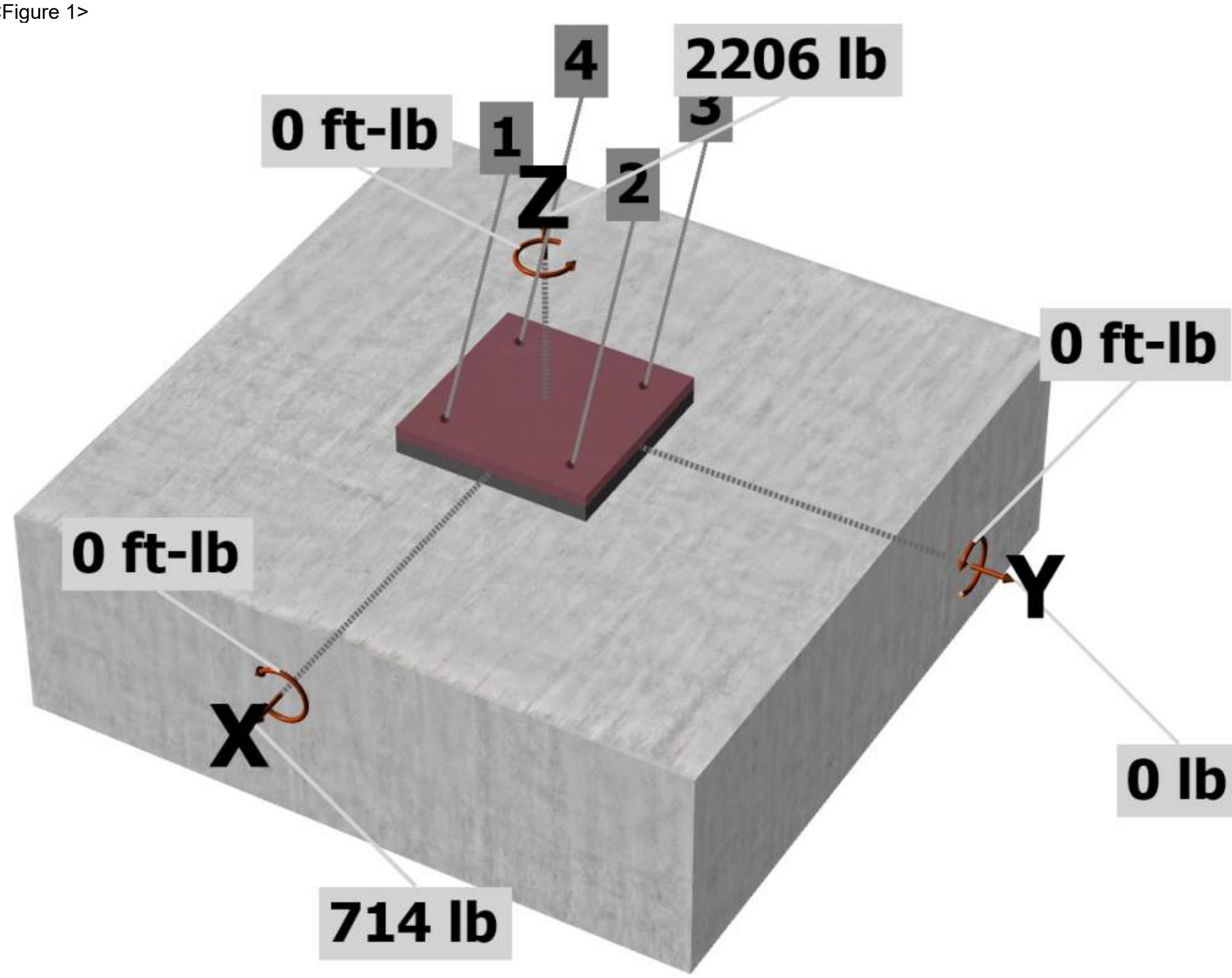
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB6 (3/4"Ø)

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]: 2206
V_{uax} [lb]: 714
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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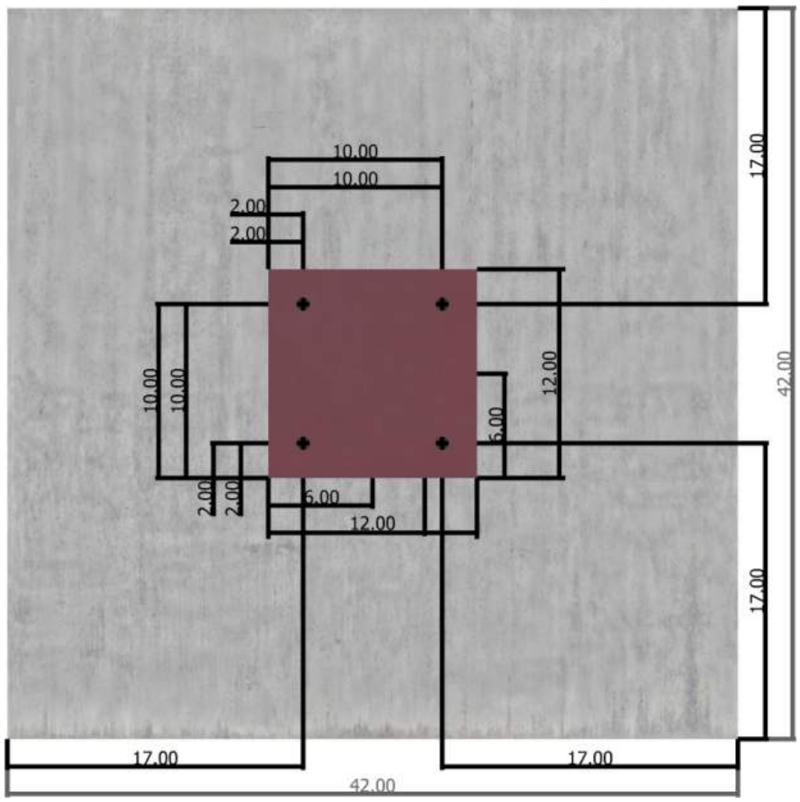
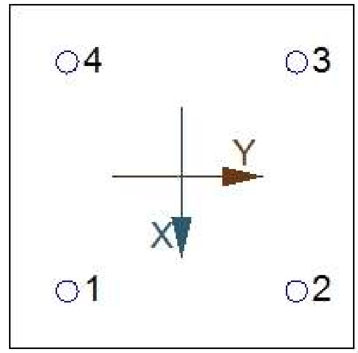

10.00
10.00
2.00
2.00
17.00
10.00
10.00
12.00
6.00
42.00
2.00
2.00
6.00
12.00
17.00
17.00
17.00
42.00

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	551.5	178.5	0.0	178.5
2	551.5	178.5	0.0	178.5
3	551.5	178.5	0.0	178.5
4	551.5	178.5	0.0	178.5
Sum	2206.0	714.0	0.0	714.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 2206
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>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	9.000	40983

$\phi N_{cbg} = \phi\, (A_{Nc} / A_{Nco})\, \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (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)
1387.56	729.00	17.00	1.000	1.000	1.25	1.000	40983	0.70	68256

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (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)
1.4	3.53	4000	0.70	110826

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
11625	0.8	0.65	6045

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (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)
6.00	0.750	1.00	4000	14.67	31972

$\phi V_{cbgx} = \phi\ (A_{Vc} / A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & 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_{cbgx} (lb)
924.00	968.00	1.000	0.932	1.400	1.000	31972	0.70	27869

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (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)
6.00	0.750	1.00	4000	14.67	31972

$\phi V_{cbgy} = \phi\ (2)(A_{Vc} / A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (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)
924.00	968.00	1.000	1.000	1.400	1.000	31972	0.70	59817

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (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_{cpg} (lb)
2.0	1387.56	729.00	1.000	1.000	1.250	1.000	40983	0.70	136511

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	**552**	**14528**	**0.04**	**Pass (Governs)**
Concrete breakout	2206	68256	0.03	Pass
Pullout	552	110826	0.00	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	**179**	**6045**	**0.03**	**Pass (Governs)**
T Concrete breakout x+	714	27869	0.03	Pass
|| Concrete breakout y+	357	59817	0.01	Pass
Pryout	714	136511	0.01	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6.1	0.04	0.00	3.8%	1.0	Pass

PAB6 (3/4"Ø) with hef = 9.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

1.Project information

Project description: Anchor Rods for Column B5 (Max loading for main building columns) / Footing P3.5
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 9.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.25
C_{min} (inch): 4.50
S_{min} (inch): 4.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 22.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.4
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 12.00 x 12.00 x 0.75

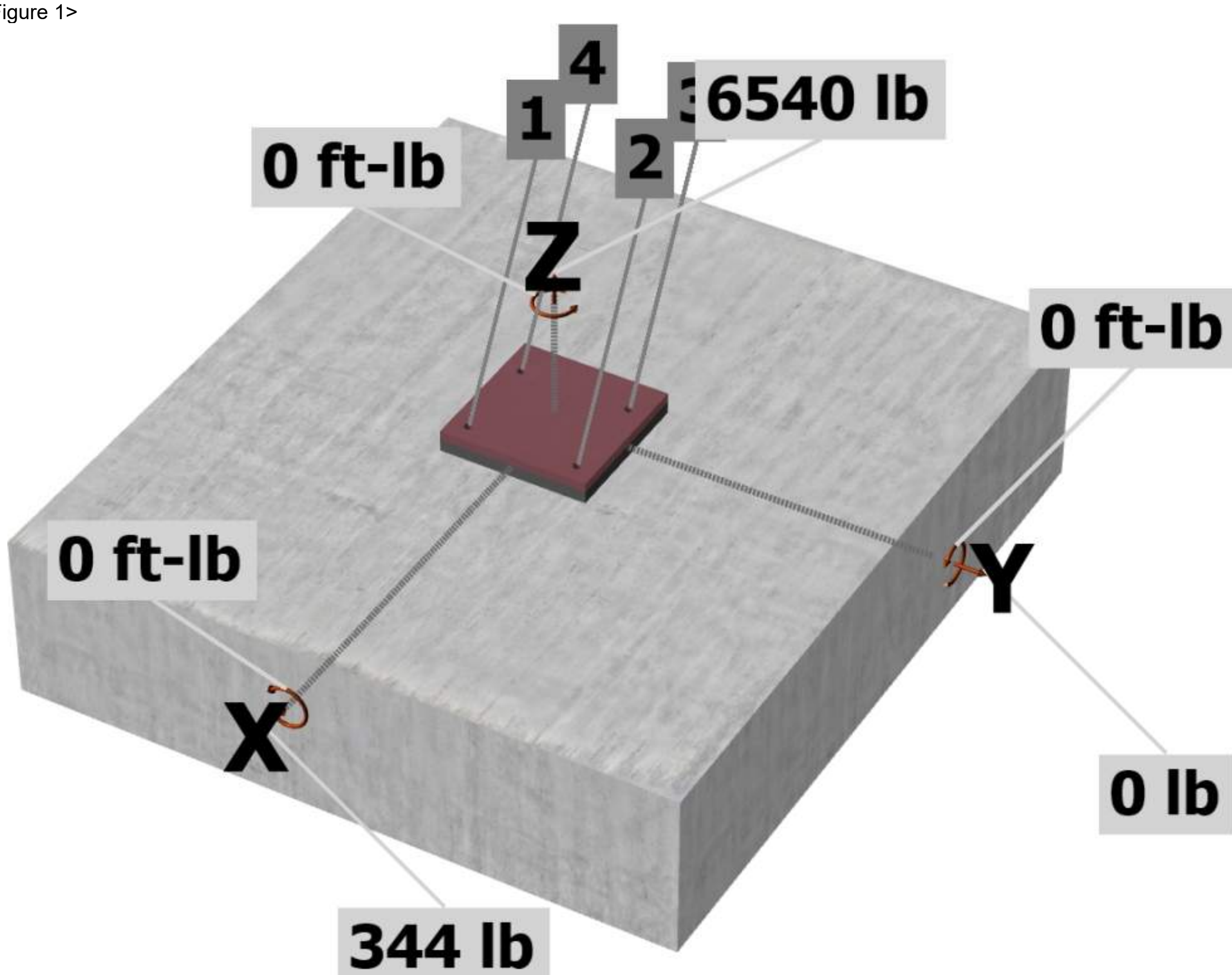
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB6 (3/4"Ø)

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

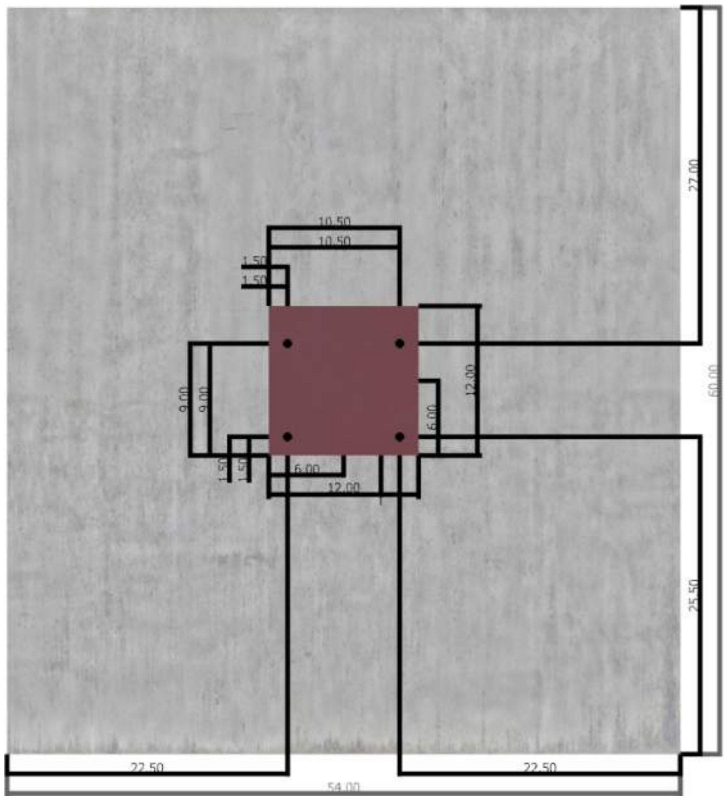
Strength level loads:

N_{ua} [lb]: 6540
V_{uax} [lb]: -344
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>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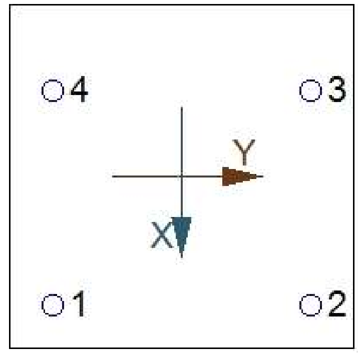
1.50
27.00
60.00
12.00
9.00
9.00
6.00
6.00
12.00
1.50
1.50
25.50
22.50
22.50
54.00

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	1308.5	-86.0	0.0	86.0
2	1308.5	-86.0	0.0	86.0
3	1961.5	-86.0	0.0	86.0
4	1961.5	-86.0	0.0	86.0
Sum	6540.0	-344.0	0.0	344.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 6540
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): 0.75
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>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	9.000	40983

$\phi N_{cbg} = \phi (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (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)
1405.69	729.00	22.50	0.947	1.000	1.25	1.000	40983	0.70	65513

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (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)
1.4	3.53	4000	0.70	110826

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
11625	0.8	0.65	6045

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (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)
6.00	0.750	1.00	4000	15.00	33068

$\phi V_{cbgx} = \phi\ (A_{Vc} / A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & 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_{cbgx} (lb)
1188.00	1012.50	1.000	1.000	1.400	1.011	33068	0.70	38454

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (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)
6.00	0.750	1.00	4000	18.00	43469

$\phi V_{cbgy} = \phi\ (2)(A_{Vc} / A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (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)
1320.00	1458.00	1.000	1.000	1.400	1.108	43469	0.70	85452

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (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_{cpg} (lb)
2.0	1405.69	729.00	1.000	1.000	1.250	1.000	40983	0.70	138294

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	**1961**	**14528**	**0.14**	**Pass (Governs)**
Concrete breakout	6540	65513	0.10	Pass
Pullout	1961	110826	0.02	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	**86**	**6045**	**0.01**	**Pass (Governs)**
T Concrete breakout x-	344	38454	0.01	Pass
|| Concrete breakout y+	172	85452	0.00	Pass
Pryout	344	138294	0.00	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6.1	0.14	0.00	13.5%	1.0	Pass

PAB6 (3/4"Ø) with hef = 9.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

SIMPSON Strong-Tie®

Anchor Designer™ Software
Version 3.2.2311.2

Company:		Date:	2/22/2023
Engineer:	JCF	Page:	1/6
Project:	AFC - Lee's Summit		
Address:			
Phone:			
E-mail:			

1.Project information

Project description: Anchor Rods for Colums C8 and D8 / Footing P4
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 12.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 14.63
C_{min} (inch): 6.00
S_{min} (inch): 6.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 22.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.4
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 12.00 x 12.00 x 1.00

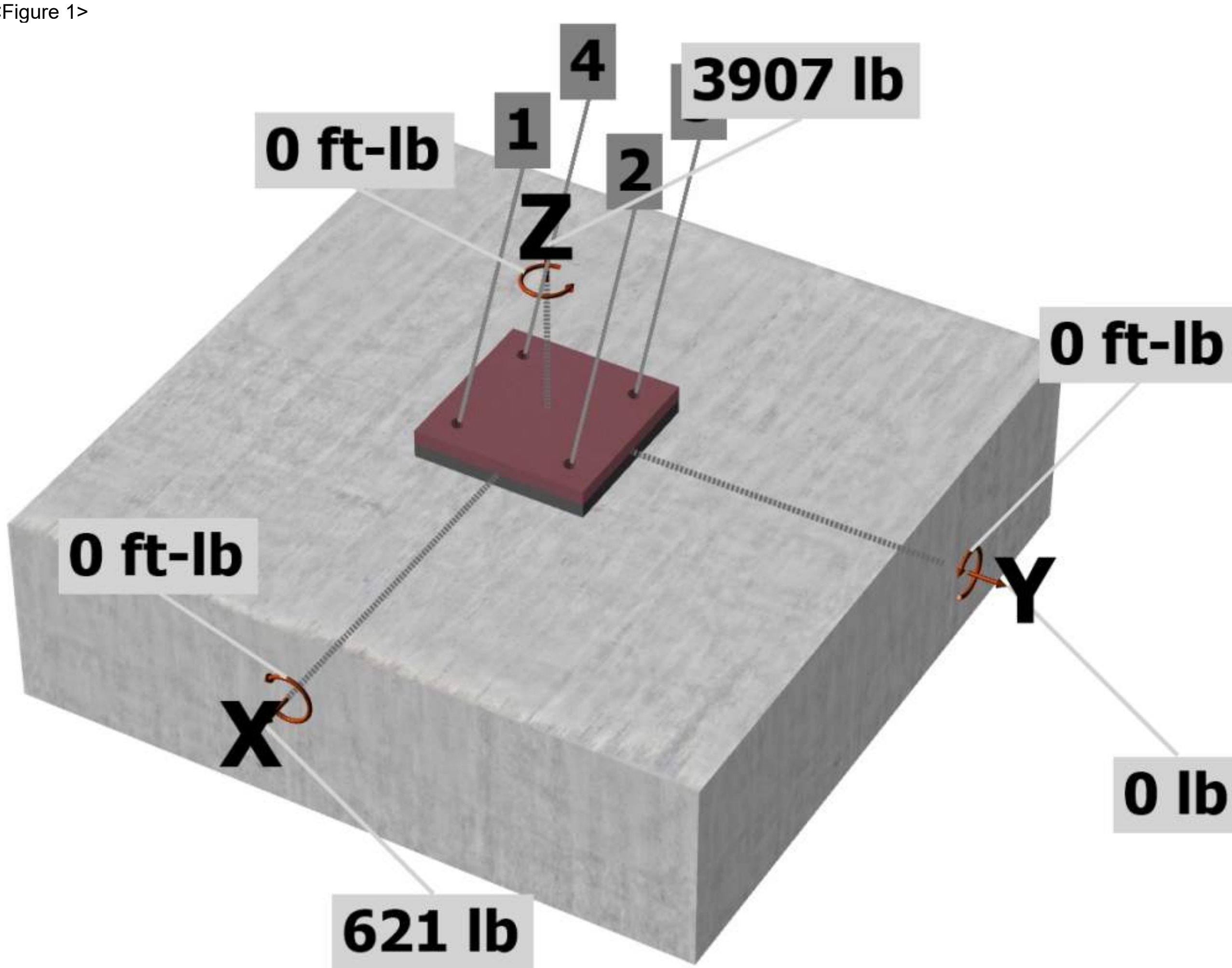
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8 (1"Ø)

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]: 3907
V_{uax} [lb]: 621
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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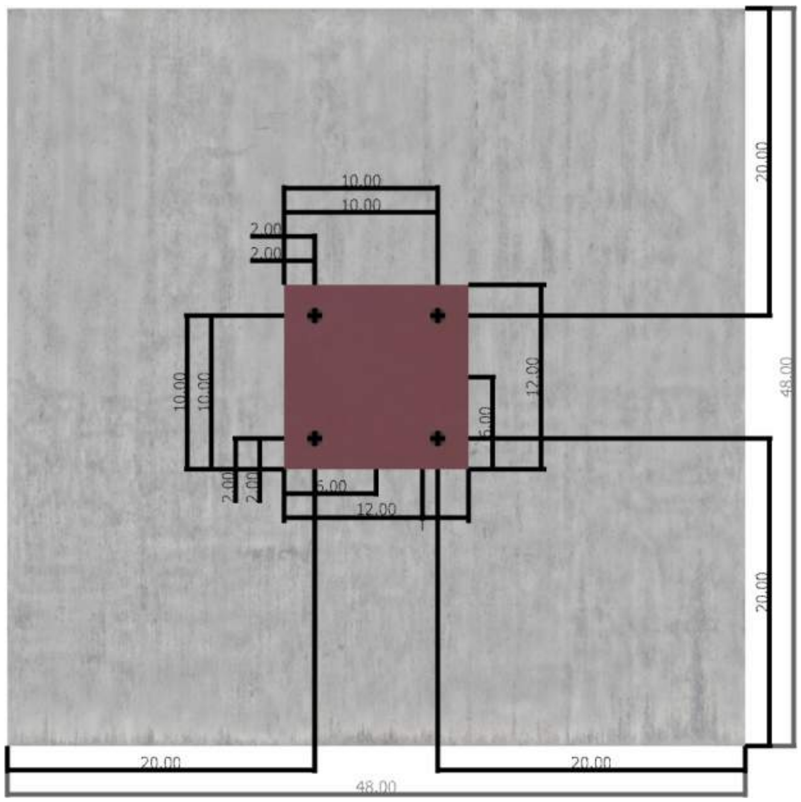
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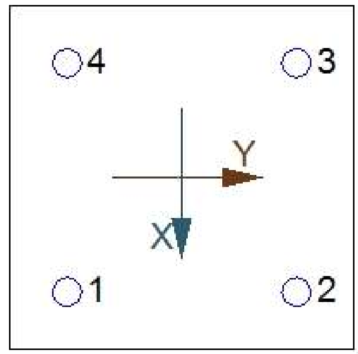
12.00
20.00
20.00
20.00
48.00

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	976.8	155.3	0.0	155.3
2	976.8	155.3	0.0	155.3
3	976.8	155.3	0.0	155.3
4	976.8	155.3	0.0	155.3
Sum	3907.0	621.0	0.0	621.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 3907
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>



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)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.4.2.2b)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	4000	12.000	63648

$\phi N_{cbg} = \phi\,(A_{Nc}/A_{Nco})\,\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (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)
2185.56	1296.00	20.00	1.000	1.000	1.25	1.000	63648	0.70	93919

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi\Psi_{c,P}N_p = \phi\Psi_{c,P}8A_{brg}f'_c$ (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)
1.4	5.15	4000	0.70	161629

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
21090	0.8	0.65	10967

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (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	4000	14.67	31972

$\phi V_{cbgx} = \phi\ (A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & 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_{cbgx} (lb)
1056.00	968.00	1.000	0.973	1.400	1.000	31972	0.70	33249

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (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	4000	14.67	31972

$\phi V_{cbgy} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (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)
1056.00	968.00	1.000	1.000	1.400	1.000	31972	0.70	68362

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (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_{cpg} (lb)
2.0	2185.56	1296.00	1.000	1.000	1.250	1.000	63648	0.70	187837

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	977	26363	0.04	Pass
Concrete breakout	**3907**	**93919**	**0.04**	**Pass (Governs)**
Pullout	977	161629	0.01	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	155	10967	0.01	Pass
T Concrete breakout x+	**621**	**33249**	**0.02**	**Pass (Governs)**
|| Concrete breakout y+	**311**	**68362**	**0.00**	**Pass (Governs)**
Pryout	621	187837	0.00	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6.1	0.04	0.00	4.2%	1.0	Pass

PAB8 (1"Ø) with hef = 12.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

Anchor Designer™
Software
Version 3.2.2311.2

Company:		Date:	4/26/2024
Engineer:	JCF	Page:	1/5
Project:	AFC - Lee's Summit		
Address:			
Phone:			
E-mail:			

1.Project information

Project description: Anchor Rods for Column F5 (Max loading for main building columns) / Footing P4
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 9.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.25
C_{min} (inch): 4.50
S_{min} (inch): 4.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 22.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 12.00 x 12.00 x 0.75

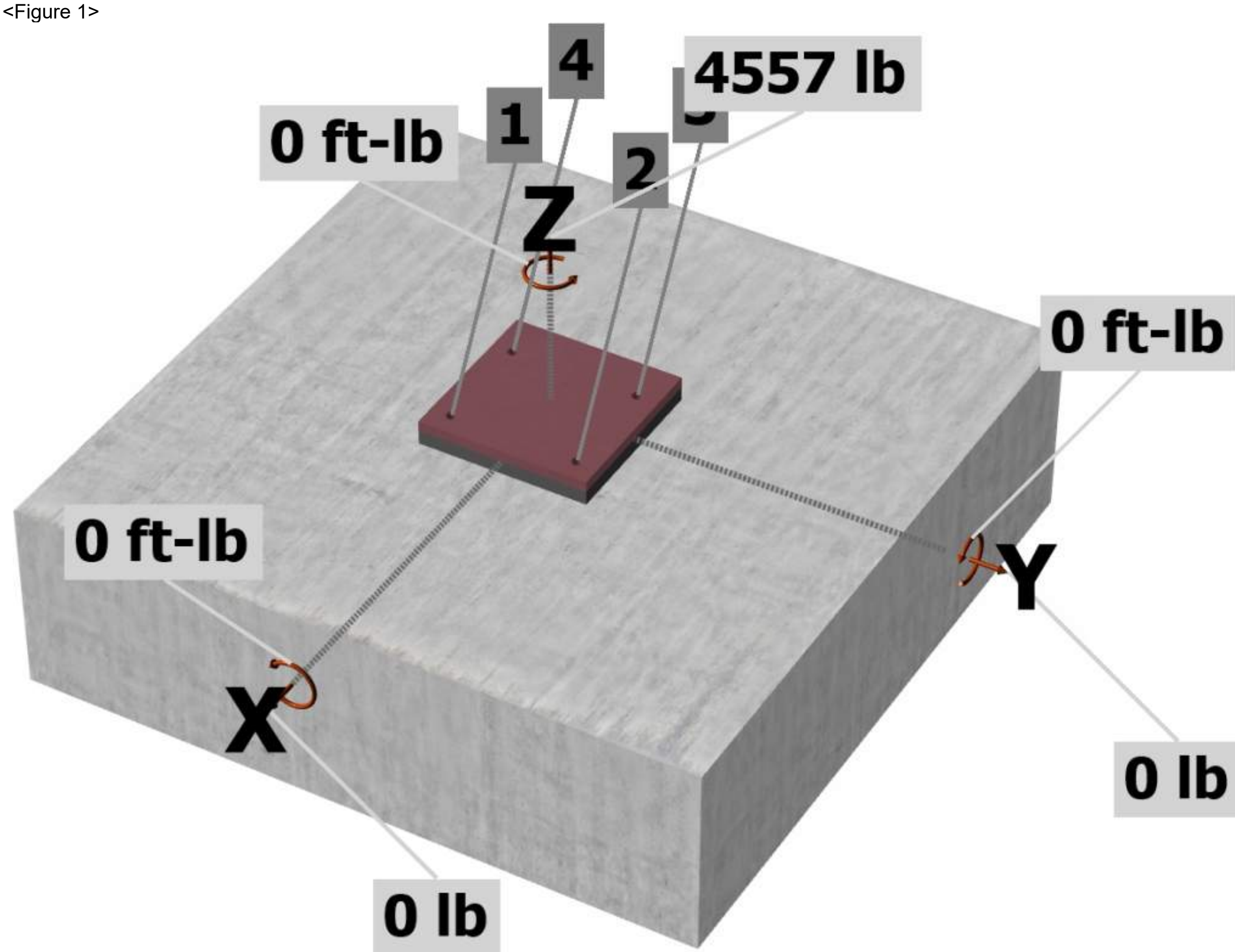
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB6 (3/4"Ø)

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]: 4557
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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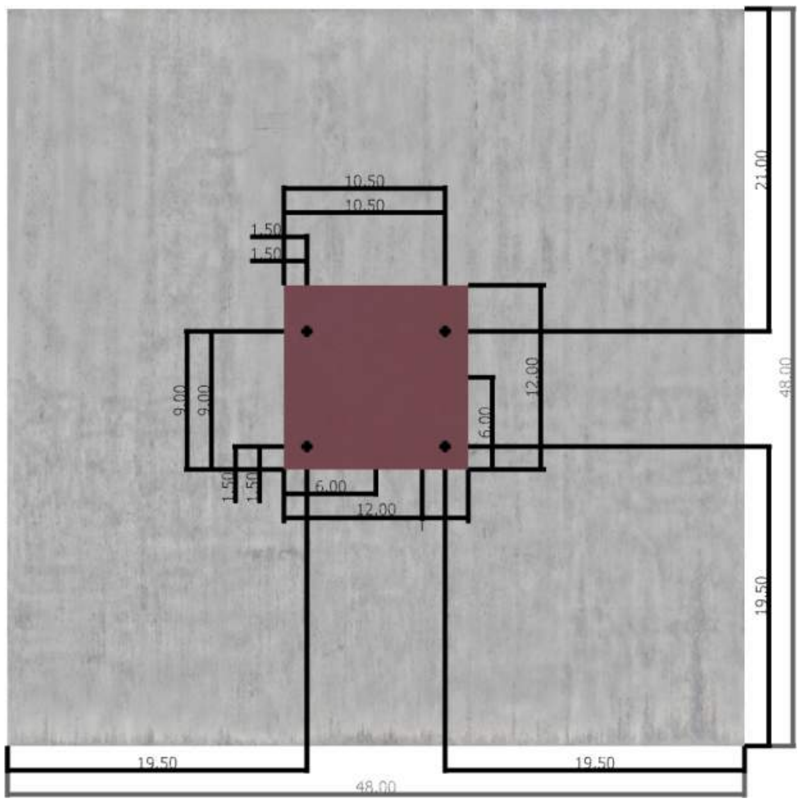
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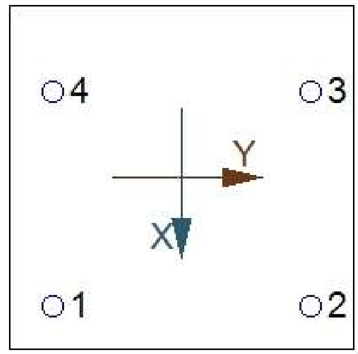
12.00
19.50
19.50
19.50
48.00

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	911.9	0.0	0.0	0.0
2	911.9	0.0	0.0	0.0
3	1366.4	0.0	0.0	0.0
4	1366.4	0.0	0.0	0.0
Sum	4556.6	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 4557
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): 0.75

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	9.000	40983

$\phi N_{cbg} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (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)
1405.69	729.00	19.50	0.947	1.000	1.25	1.000	40983	0.70	65516

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (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)
1.4	3.53	4000	0.70	110826

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	**1366**	**14528**	**0.09**	**Pass (Governs)**
Concrete breakout	4557	65516	0.07	Pass
Pullout	1366	110826	0.01	Pass

PAB6 (3/4"Ø) with hef = 9.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

Masonry Slender Wall

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Dumpster Wall

Code References

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

Construction Type : Grouted Hollow Concrete Masonry

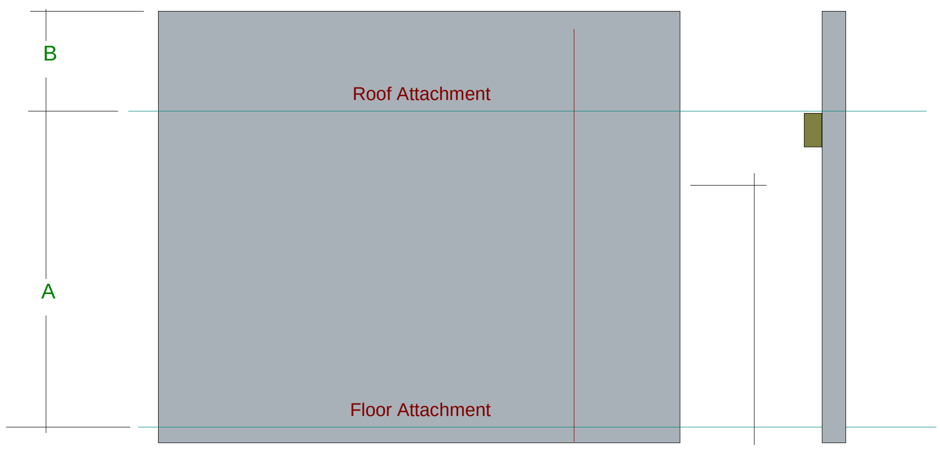
F'm	=	1.50 ksi	Nom. Wall Thickness	8 in	Temp Diff across thickness	=	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness	7.625 in	Min Allow Out-of-plane Defl Ra	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance	3.8125 in			
Em = f'm *	=	900.0	Lower Level Rebar . . .		Minimum Vertical Steel %	=	0.0020
Max % of ρ bal.	=	0.007134	Bar Size #	5			
Grout Density	=	140 pcf	Bar Spacing	32 in			
Block Weight		Normal Weight					
Wall Weight	=	51.0 psf					

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A	Clear Height	=	8.0 ft
B	Parapet height	=	ft

Wall Support Condition Top Free, Bottom Fix



Lateral Loads

Wind Loads :

Full area WIND load 22.40 psf

Seismic Loads :

Wall Weight Seismic Load Input Method : ASCE seismic factors entered

SDS Value per ASCE 12.11.1 S_{DS} * I = 0.0810

Fp = Wall Wt. * 0.03240 = 1.652 psf

Masonry Slender Wall

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Dumpster Wall

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check	Maximum Bending Stress Rat	**0.4010**		
	+0.90D+W	Max Mu	-0.7186 k-ft	Phi * Mn	1.792 k-ft
PASS	Service Deflection Check	Actual Defl. Ratio L/	1,298	Allowable Defl. Ratio	150.0
	W Only	Max. Deflection	0.1480 in		/2 for Cantilever
PASS	Axial Load Check	Max Pu / Ag	10.652 psi	Max. Allow. Defl.	1.280 in
	+1.20D+W	Location	0.1333 ft	0.2 * f'm	300.0 psi
	Reinforcing Limit Check				
		Actual As/bd	0.002541	Max Allow As/bd	0.007134

Maximum Reactions	*for Load Combination...*	
Top Horizontal		0.0 k
Base Horizontal	W Only	0.1792 k
Vertical Reaction	+D+0.60W	0.4080 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Axial Load 0.2*f'm*b*t k	Mcr k-ft	Mu k-ft	Phi	Phi Mn k-ft	As in^2	As Ratio	0.6 * rho bal	Bar 'd'
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
+1.20D+0.50W at 0.00 to 0.27	0.490	13.788	0.46	0.36	0.90	1.82	0.116	0.0025	0.0070	0.00
+1.20D+W at 0.00 to 0.27	0.490	13.788	0.46	0.72	0.90	1.82	0.116	0.0025	0.0070	0.00
+0.90D+W at 0.00 to 0.27	0.367	13.788	0.46	0.72	0.90	1.79	0.116	0.0025	0.0070	0.00
+1.20D+E at 0.00 to 0.27	0.490	13.788	0.46	0.05	0.90	1.82	0.116	0.0025	0.0070	0.00
+0.90D+E at 0.00 to 0.27	0.367	13.788	0.46	0.05	0.90	1.79	0.116	0.0025	0.0070	0.00

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values Mcr k-ft	Moment Values Mactual k-ft	Stiffness I gross in^4	Stiffness I cracked in^4	Stiffness I effective in^4	Deflections Deflection in	Deflections Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.60W at 7.73 to 8.00	0.014	0.46	0.00	342.40	23.72	342.400	0.025	7,627.3
+D+0.450W at 7.73 to 8.00	0.014	0.46	0.00	342.40	23.72	342.400	0.019	10,169.8
+0.60D+0.60W at 7.73 to 8.00	0.008	0.46	0.00	342.40	23.70	342.400	0.025	7,630.3
+D+0.70E at 7.73 to 8.00	0.014	0.46	0.00	342.40	23.72	342.400	0.002	88,625.3
+D+0.5250E at 7.73 to 8.00	0.014	0.46	0.00	342.40	23.72	342.400	0.002	118167.0
+0.60D+0.70E at 7.73 to 8.00	0.008	0.46	0.00	342.40	23.70	342.400	0.002	88,659.9
W Only at 7.73 to 8.00	0.000	0.46	0.00	342.40	23.68	342.400	0.148	1,297.7
E Only at 7.73 to 8.00	0.000	0.46	0.00	342.40	23.68	342.400	0.003	62,098.2

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	0.408 k
+D+0.60W	0.1 k	0.00 k	0.408 k
+D+0.450W	0.1 k	0.00 k	0.408 k
+0.60D+0.60W	0.1 k	0.00 k	0.245 k
+D+0.70E	0.0 k	0.00 k	0.408 k
+D+0.5250E	0.0 k	0.00 k	0.408 k
+0.60D+0.70E	0.0 k	0.00 k	0.245 k
W Only	0.2 k	0.00 k	0.000 k

DESCRIPTION: Dumpster Wall

E Only	0.0 k	0.00 k	0.000 k

Masonry Beam

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Dumpster Wall Wind Reinforcement

Code References

Calculations per TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

f'm	1,500.0 psi	Clear Span	13.330 ft	Rebar Size	5.0
Fs	24,000.0 psi	Beam Depth	1.334 ft	# Bars @ Locations	1
Em = f'm *	900.0	Thickness	8 in	Top Clear	6.0 in
Wall Wt Mult.	1.0	End Fixity	Pin-Pin	Btm Clear	6.0 in
Block Type	Normal Wt	Equiv. Solid Thick	7.60 in	# Bar Sets	1
Lateral Wind Load	22.40 psf	Wall Weight	86.0 psf	Bar Spacing	5.0 in
Beam is Fully Braced ?	No	E	1,350.0 ksi		
Lateral Wall Weight Seismic Factor	0.250	n	21.481		
Calculate vertical beam weight ?	Yes				

DESIGN SUMMARY

Design OK

Maximum Stress Ratios...

	Vertical	Lateral	SRSS Combination
fb/Fb	0.0	0.2152	**0.2152** : 1.00
fv/Fv	0.0	0.05661	**0.05661** : 1.00

Minimum Mn = 1.3 * Fcr * S = 3.528 k-ft

Maximum Moment

	Actual	Allowable k-ft
Vertical Loads	0.0 k-ft	0.0 k-ft
for Load Combination :		
Lateral Loads	0.4457 k-ft	2.072
for Load Combination E Only * 0.70		

Maximum Shear

	Actual	Allowable
Vertical Loads	0.0 psi	0.0 psi
for Load Combination :		
Lateral Loads	2.193 psi	38.730 psi
for Load Combination E Only * 0.70		

Vertical Strength

As	0.310 in^2
rho	0.004062
np	0.08727
k : ((np)^2+2np)^.5-np	0.3395
j = 1 - k/3	0.8868
M:mas=Fb k j b d^2/2	6.467 k-ft
M:Stl = Fs As j d	5.503 k-ft

Lateral Strength (Checking lateral bending for span)

As	0.310 in^2
rho	0.005079
np	0.1091
k : (np^2+2np)^.5-np	0.3706
j = 1 - k/3	0.8765
M:mas=Fb k j b d^2/2	2.126 k-ft
M:Stl = Fs As j d	2.072 k-ft

Detailed Load Combination Results

Load Combinatic	Vertical				Lateral			
	Mmax k-ft	Mallow k-ft	fv : Vert psi	Fv : Vert psi	Mactual k-ft	Mallow k-ft	fv psi	Fv psi
	0.00	5.50	0.00	77.46	0.00	2.07	0.00	38.73
+0.60W	0.00	5.50	0.00	77.46	0.40	2.07	1.96	38.73
+0.450W	0.00	5.50	0.00	77.46	0.30	2.07	1.47	38.73

DESCRIPTION: Dumpster Wall Wind Reinforcement

Detailed Load Combination Results

Load Combinatic	Vertical				Lateral			
	Mmax k-ft	Mallow k-ft	fv : Vert psi	Fv : Vert psi	Mactual k-ft	Mallow k-ft	fv psi	Fv psi
E Only * 0.70	0.00	5.50	0.00	77.46	0.45	2.07	2.19	38.73
E Only * 0.5250	0.00	5.50	0.00	77.46	0.33	2.07	1.64	38.73

Project Title:
Engineer:
Project ID:
Project Descr:

Beam on Elastic Foundation

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: Typical Wall Footing

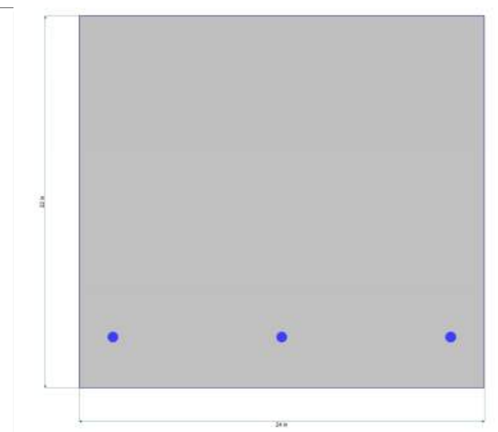
CODE REFERENCES

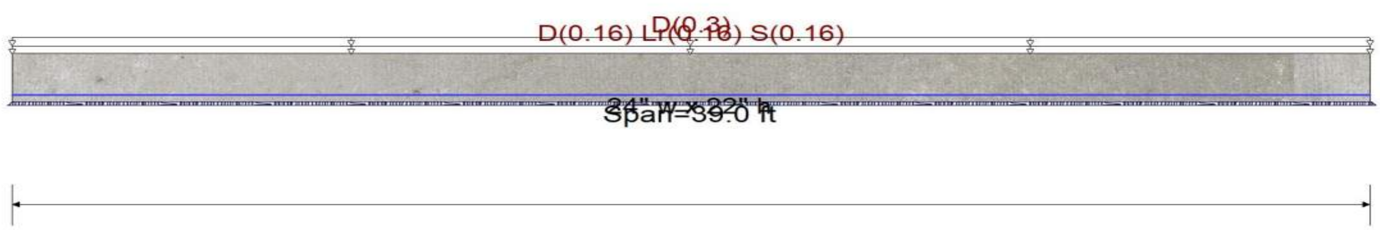
Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

Material Properties

f'c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ Lt Wt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi			
Soil Subgrade Modulus	=	250.0 psi / (inch deflection)			
Load Combination	ASCE 7-16				
fy - Main Rebar	=	60.0 ksi	Fy - Stirrups	=	40.0 ksi
E - Main Rebar	=	29,000.0 ksi	E - Stirrups	=	29,000.0 ksi
			Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup		2

Beam is supported on an elastic foundation.





Cross Section & Reinforcing Details

Rectangular Section, Width = 24.0 in, Height = 22.0 in
Span #1 Reinforcing....
3-#5 at 3.0 in from Bottom, from 0.0 to 39.0 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.020, Lr = 0.020, S = 0.020 ksf, Tributary Width = 8.0 ft, (Roof Load Max)
Uniform Load : D = 0.30 k/ft, Tributary Width = 1.0 ft, (Wall)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.002**: 1	Maximum Deflection	
Section used for this span	**Typical Section**	Max Downward L+Lr+S Deflection	0.000 in
Mu : Applied	-0.02535 k-ft	Max Upward L+Lr+S Deflection	0.000 in
Mn * Phi : Allowable	11.124 k-ft	Max Downward Total Deflection	0.016 in
Load Combination	+1.20D+1.60Lr	Max Upward Total Deflection	0.007 in
Location of maximum on span	19.271 ft		
Span # where maximum occurs	Span # 1		

Maximum Soil Pressure = **0.576** ksf at 10.83 ft LdComb: +D+Lr
Allowable Soil Pressure = **1.80** ksf **OK**

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combination

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum Bending Envelope					
Span # 1	1	38.541	-0.00	78.08	0.00
+1.40D					
Span # 1	1	38.541	-0.00	78.08	0.00

Beam on Elastic Foundation

DESCRIPTION: Typical Wall Footing

Load Combination		Location (ft)	Bending Stress Results (k-ft		
Segment Length	Span #	in Span	Mu : Max	Phi*Mnx	Stress Ratio
+1.20D+0.50Lr					
Span # 1	1	38.541	-0.00	78.08	0.00
+1.20D+0.50S					
Span # 1	1	38.541	-0.00	78.08	0.00
+1.20D+1.60Lr					
Span # 1	1	38.541	-0.00	78.08	0.00
+1.20D+1.60S					
Span # 1	1	38.541	-0.00	78.08	0.00
+0.90D					
Span # 1	1	38.541	-0.00	78.08	0.00
+1.20D+0.20S					
Span # 1	1	38.541	-0.00	78.08	0.00

Overall Maximum Deflections - Unfactored Lo

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Span 1	1	0.0160	10.833		0.0000	0.000

General Footing

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: P3 - Drive Thru B-4 (Max loading)

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0	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.	=	0.00180
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	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	1.80	ksf
Soil Density	=	110.0	pcf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

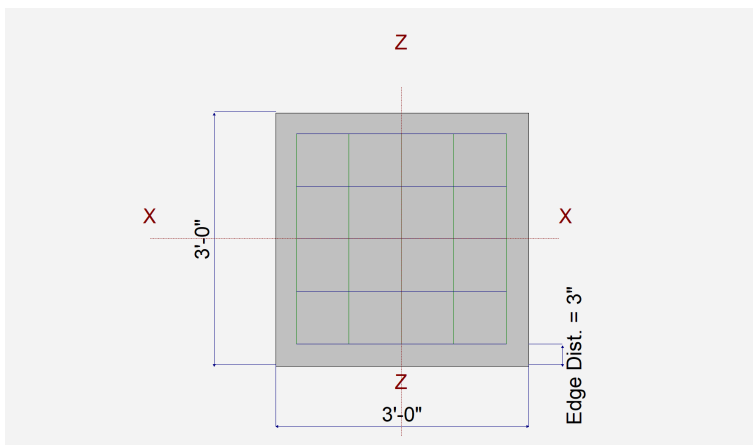
Footing base depth below soil surface	=	2.0	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

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	=	22.0	in



Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height	=		in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in

Reinforcing

Bars parallel to X-X Axis

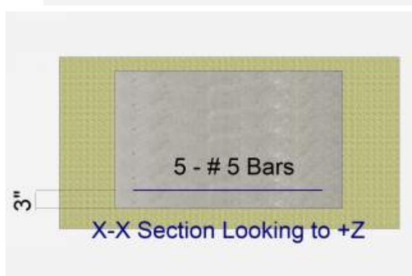
Number of Bars	=		5.0
Reinforcing Bar Size	=	#	5

Bars parallel to Z-Z Axis

Number of Bars	=		5.0
Reinforcing Bar Size	=	#	5

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



Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	1.610	1.0		1.0	-2.670			k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=	0.3550	0.2910		0.2910	-0.7620			k
V-z	=								k

General Footing

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: P3 - Drive Thru B-4 (Max loading)

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.4637	Soil Bearing	0.8347 ksf	1.80 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	1.278	Overturning - Z-Z	3.241 k-ft	4.141 k-ft	+0.60D+0.60W
PASS	4.706	Sliding - X-X	0.6460 k	3.040 k	+D+Lr
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	1.561	Uplift	-1.602 k	2.501 k	+0.60D+0.60W
PASS	0.01649	Z Flexure (+X)	0.7139 k-ft/ft	43.292 k-ft/ft	+1.20D+1.60Lr
PASS	0.003906	Z Flexure (-X)	0.1691 k-ft/ft	43.292 k-ft/ft	+1.20D+1.60Lr
PASS	0.01020	X Flexure (+Z)	0.4415 k-ft/ft	43.292 k-ft/ft	+1.20D+1.60Lr
PASS	0.01020	X Flexure (-Z)	0.4415 k-ft/ft	43.292 k-ft/ft	+1.20D+1.60Lr
PASS	n/a	1-way Shear (+X)	0.0 psi	94.868 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a
PASS	n/a	1-way Shear (+Z)	0.0 psi	94.868 psi	n/a
PASS	n/a	1-way Shear (-Z)	0.0 psi	94.868 psi	n/a
PASS	n/a	2-way Punching	1.785 psi	94.868 psi	+1.20D+1.60Lr

Top reinforcing mat required (see 'Bending' tab).

Hand check required for anchor pullout.

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	1.80	n/a	0.0	0.4631	0.4631	n/a	n/a	0.257
X-X, +D+Lr	1.80	n/a	0.0	0.5742	0.5742	n/a	n/a	0.319
X-X, +D+S	1.80	n/a	0.0	0.5742	0.5742	n/a	n/a	0.319
X-X, +D+0.750Lr	1.80	n/a	0.0	0.5464	0.5464	n/a	n/a	0.304
X-X, +D+0.750S	1.80	n/a	0.0	0.5464	0.5464	n/a	n/a	0.304
X-X, +D+0.60W	1.80	n/a	0.0	0.2851	0.2851	n/a	n/a	0.158
X-X, +D+0.750Lr+0.450W	1.80	n/a	0.0	0.4129	0.4129	n/a	n/a	0.229
X-X, +D+0.750S+0.450W	1.80	n/a	0.0	0.4129	0.4129	n/a	n/a	0.229
X-X, +0.60D+0.60W	1.80	n/a	0.0	0.09983	0.09983	n/a	n/a	0.055
X-X, +0.60D	1.80	n/a	0.0	0.2778	0.2778	n/a	n/a	0.154
Z-Z, D Only	1.80	1.874	n/a	n/a	n/a	0.3199	0.6062	0.337
Z-Z, +D+Lr	1.80	2.750	n/a	n/a	n/a	0.3136	0.8347	0.464
Z-Z, +D+S	1.80	2.750	n/a	n/a	n/a	0.3136	0.8347	0.464
Z-Z, +D+0.750Lr	1.80	2.565	n/a	n/a	n/a	0.3152	0.7776	0.432
Z-Z, +D+0.750S	1.80	2.565	n/a	n/a	n/a	0.3152	0.7776	0.432
Z-Z, +D+0.60W	1.80	-0.8764	n/a	n/a	n/a	0.3263	0.2438	0.181
Z-Z, +D+0.750Lr+0.450W	1.80	1.364	n/a	n/a	n/a	0.320	0.5058	0.281
Z-Z, +D+0.750S+0.450W	1.80	1.364	n/a	n/a	n/a	0.320	0.5058	0.281
Z-Z, +0.60D+0.60W	1.80	-5.979	n/a	n/a	n/a	0.1983	0.001339	0.110
Z-Z, +0.60D	1.80	1.874	n/a	n/a	n/a	0.1919	0.3637	0.202

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	None	0.0 k-ft	Infinity	OK
X-X, +D+Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+S	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750S	None	0.0 k-ft	Infinity	OK
X-X, +D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750S+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +0.60D	None	0.0 k-ft	Infinity	OK
Z-Z, D Only	0.6508 k-ft	6.251 k-ft	9.605	OK
Z-Z, +D+Lr	1.184 k-ft	7.751 k-ft	6.545	OK

General Footing

DESCRIPTION: P3 - Drive Thru B-4 (Max loading)

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Z-Z, +D+S	1.184 k-ft	7.751 k-ft	6.545	OK
Z-Z, +D+0.750Lr	1.051 k-ft	7.376 k-ft	7.019	OK
Z-Z, +D+0.750S	1.051 k-ft	7.376 k-ft	7.019	OK
Z-Z, +D+0.60W	3.241 k-ft	6.902 k-ft	2.129	OK
Z-Z, +D+0.750Lr+0.450W	2.853 k-ft	8.005 k-ft	2.806	OK
Z-Z, +D+0.750S+0.450W	2.853 k-ft	8.005 k-ft	2.806	OK
Z-Z, +0.60D+0.60W	3.241 k-ft	4.141 k-ft	1.278	OK
Z-Z, +0.60D	0.3905 k-ft	3.751 k-ft	9.605	OK

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
X-X, D Only	0.3550 k	2.740 k	7.718	OK
X-X, +D+Lr	0.6460 k	3.040 k	4.706	OK
X-X, +D+S	0.6460 k	3.040 k	4.706	OK
X-X, +D+0.750Lr	0.5733 k	2.965 k	5.172	OK
X-X, +D+0.750S	0.5733 k	2.965 k	5.172	OK
X-X, +D+0.60W	-0.1022 k	2.259 k	22.106	OK
X-X, +D+0.750Lr+0.450W	0.2304 k	2.604 k	11.306	OK
X-X, +D+0.750S+0.450W	0.2304 k	2.604 k	11.306	OK
X-X, +0.60D+0.60W	-0.2442 k	1.759 k	7.204	OK
X-X, +0.60D	0.2130 k	2.240 k	10.515	OK
Z-Z, D Only	0.0 k	2.740 k	No Sliding	OK
Z-Z, +D+Lr	0.0 k	3.040 k	No Sliding	OK
Z-Z, +D+S	0.0 k	3.040 k	No Sliding	OK
Z-Z, +D+0.750Lr	0.0 k	2.965 k	No Sliding	OK
Z-Z, +D+0.750S+0.450W	0.0 k	2.604 k	No Sliding	OK
Z-Z, +0.60D+0.60W	0.0 k	1.759 k	No Sliding	OK
Z-Z, +0.60D	0.0 k	2.240 k	No Sliding	OK
Z-Z, +D+0.750S	0.0 k	2.965 k	No Sliding	OK
Z-Z, +D+0.60W	0.0 k	2.259 k	No Sliding	OK
Z-Z, +D+0.750Lr+0.450W	0.0 k	2.604 k	No Sliding	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	0.2818	+Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.40D	0.2818	-Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.50Lr	0.3040	+Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.50Lr	0.3040	-Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.50S	0.3040	+Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.50S	0.3040	-Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+1.60Lr	0.4415	+Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+1.60Lr	0.4415	-Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+1.60Lr+0.50W	0.2746	+Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+1.60Lr+0.50W	0.2746	-Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+1.60S	0.4415	+Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+1.60S	0.4415	-Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+1.60S+0.50W	0.2746	+Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+1.60S+0.50W	0.2746	-Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.50Lr+W	0.02975	+Z	Top	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.50Lr+W	0.02975	-Z	Top	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.50S+W	0.02975	+Z	Top	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.50S+W	0.02975	-Z	Top	0.4752	AsMin	0.5167	43.292	OK
X-X, +0.90D+W	0.1526	+Z	Top	0.4752	AsMin	0.5167	43.292	OK
X-X, +0.90D+W	0.1526	-Z	Top	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.20S	0.2665	+Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +1.20D+0.20S	0.2665	-Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +0.90D	0.1811	+Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
X-X, +0.90D	0.1811	-Z	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.40D	0.1299	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.40D	0.4336	+X	Bottom	0.4752	AsMin	0.5167	43.292	OK

General Footing

Project File: 24-0121 - afc lee's summit.ec6

DESCRIPTION: P3 - Drive Thru B-4 (Max loading)

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+0.50Lr	0.1294	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+0.50Lr	0.4786	+X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+0.50S	0.1294	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+0.50S	0.4786	+X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+1.60Lr	0.1691	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+1.60Lr	0.7139	+X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.1186	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.4306	+X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+1.60S	0.1691	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+1.60S	0.7139	+X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+1.60S+0.50W	0.1186	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+1.60S+0.50W	0.4306	+X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+0.50Lr+W	0.02845	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+0.50Lr+W	0.08795	+X	Top	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+0.50S+W	0.02845	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+0.50S+W	0.08795	+X	Top	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +0.90D+W	0.007371	-X	Top	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +0.90D+W	0.2778	+X	Top	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+0.20S	0.1186	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +1.20D+0.20S	0.4144	+X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +0.90D	0.08351	-X	Bottom	0.4752	AsMin	0.5167	43.292	OK
Z-Z, +0.90D	0.2787	+X	Bottom	0.4752	AsMin	0.5167	43.292	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+1.20D+0.50Lr	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+1.20D+0.50S	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+1.20D+1.60Lr	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+1.20D+1.60Lr+0.50W	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+1.20D+1.60S	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+1.20D+1.60S+0.50W	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+1.20D+0.50Lr+W	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+1.20D+0.50S+W	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+0.90D+W	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+1.20D+0.20S	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK
+0.90D	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	94.87 psi	0.00	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	1.14 psi	189.74 psi	0.006002	OK
+1.20D+0.50Lr	1.23 psi	189.74 psi	0.006476	OK
+1.20D+0.50S	1.23 psi	189.74 psi	0.006476	OK
+1.20D+1.60Lr	1.79 psi	189.74 psi	0.009406	OK
+1.20D+1.60Lr+0.50W	1.11 psi	189.74 psi	0.005851	OK
+1.20D+1.60S	1.79 psi	189.74 psi	0.009406	OK
+1.20D+1.60S+0.50W	1.11 psi	189.74 psi	0.005851	OK
+1.20D+0.50Lr+W	0.12 psi	189.74 psi	0.000634	OK
+1.20D+0.50S+W	0.12 psi	189.74 psi	0.000634	OK
+0.90D+W	0.62 psi	189.74 psi	0.003251	OK
+1.20D+0.20S	1.08 psi	189.74 psi	0.005677	OK
+0.90D	0.73 psi	189.74 psi	0.003859	OK

General Footing

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: P4 - Max Load Front of House Moment Frame

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0	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.	=	0.00180	
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	:	No	
Use Pedestal wt for stability, mom & shear	:	No	

Soil Design Values

Allowable Soil Bearing	=	1.80	ksf
Soil Density	=	110.0	pcf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

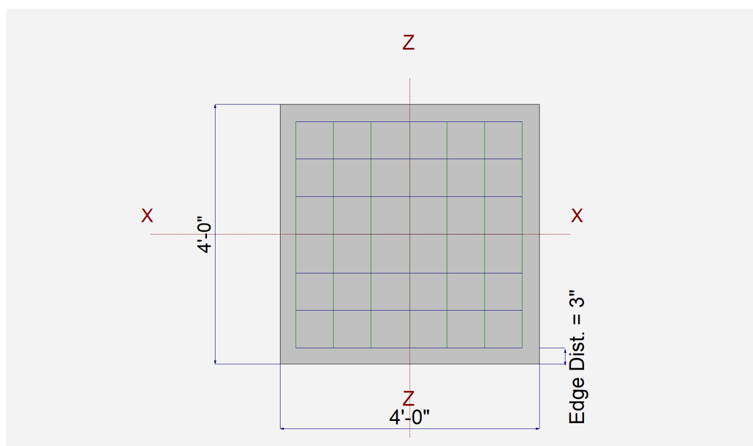
Footing base depth below soil surface	=	2.0	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

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	=	4.0	ft
Length parallel to Z-Z Axis	=	4.0	ft
Footing Thickness	=	22.0	in



Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height	=		in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in

Reinforcing

Bars parallel to X-X Axis

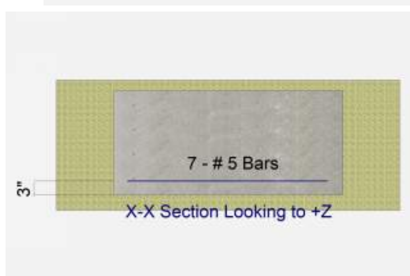
Number of Bars	=		7.0
Reinforcing Bar Size	=	#	5

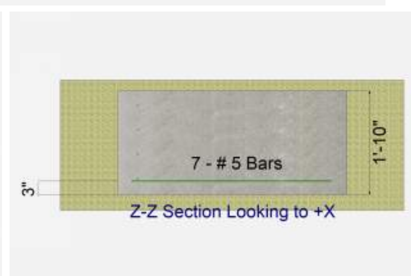
Bars parallel to Z-Z Axis

Number of Bars	=		7.0
Reinforcing Bar Size	=	#	5

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





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	2.548	2.10		2.10	-6.20			k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: P4 - Max Load Front of House Moment Frame

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.3193	Soil Bearing	0.5747 ksf	1.80 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	1.144	Uplift	-3.720 k	4.257 k	+0.60D+0.60W
PASS	0.01767	Z Flexure (+X)	0.8022 k-ft/ft	45.410 k-ft/ft	+1.20D+1.60Lr
PASS	0.01767	Z Flexure (-X)	0.8022 k-ft/ft	45.410 k-ft/ft	+1.20D+1.60Lr
PASS	0.01767	X Flexure (+Z)	0.8022 k-ft/ft	45.410 k-ft/ft	+1.20D+1.60Lr
PASS	0.01767	X Flexure (-Z)	0.8022 k-ft/ft	45.410 k-ft/ft	+1.20D+1.60Lr
PASS	0.007417	1-way Shear (+X)	0.7037 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.007417	1-way Shear (-X)	0.7037 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.007417	1-way Shear (+Z)	0.7037 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.007417	1-way Shear (-Z)	0.7037 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.01968	2-way Punching	3.733 psi	189.737 psi	+1.20D+1.60Lr

Top reinforcing mat required (see 'Bending' tab).

Hand check required for anchor pullout.

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	1.80	n/a	0.0	0.4434	0.4434	n/a	n/a	0.246
X-X, +D+Lr	1.80	n/a	0.0	0.5747	0.5747	n/a	n/a	0.319
X-X, +D+S	1.80	n/a	0.0	0.5747	0.5747	n/a	n/a	0.319
X-X, +D+0.750Lr	1.80	n/a	0.0	0.5419	0.5419	n/a	n/a	0.301
X-X, +D+0.750S	1.80	n/a	0.0	0.5419	0.5419	n/a	n/a	0.301
X-X, +D+0.60W	1.80	n/a	0.0	0.2109	0.2109	n/a	n/a	0.117
X-X, +D+0.750Lr+0.450W	1.80	n/a	0.0	0.3675	0.3675	n/a	n/a	0.204
X-X, +D+0.750S+0.450W	1.80	n/a	0.0	0.3675	0.3675	n/a	n/a	0.204
X-X, +0.60D+0.60W	1.80	n/a	0.0	0.03355	0.03355	n/a	n/a	0.019
X-X, +0.60D	1.80	n/a	0.0	0.2661	0.2661	n/a	n/a	0.148
Z-Z, D Only	1.80	0.0	n/a	n/a	n/a	0.4434	0.4434	0.246
Z-Z, +D+Lr	1.80	0.0	n/a	n/a	n/a	0.5747	0.5747	0.319
Z-Z, +D+S	1.80	0.0	n/a	n/a	n/a	0.5747	0.5747	0.319
Z-Z, +D+0.750Lr	1.80	0.0	n/a	n/a	n/a	0.5419	0.5419	0.301
Z-Z, +D+0.750S	1.80	0.0	n/a	n/a	n/a	0.5419	0.5419	0.301
Z-Z, +D+0.60W	1.80	0.0	n/a	n/a	n/a	0.2109	0.2109	0.117
Z-Z, +D+0.750Lr+0.450W	1.80	0.0	n/a	n/a	n/a	0.3675	0.3675	0.204
Z-Z, +D+0.750S+0.450W	1.80	0.0	n/a	n/a	n/a	0.3675	0.3675	0.204
Z-Z, +0.60D+0.60W	1.80	0.0	n/a	n/a	n/a	0.03355	0.03355	0.019
Z-Z, +0.60D	1.80	0.0	n/a	n/a	n/a	0.2661	0.2661	0.148

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

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	0.4459	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK

General Footing

DESCRIPTION: P4 - Max Load Front of House Moment Frame

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	0.4459	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50Lr	0.5135	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50Lr	0.5135	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50S	0.5135	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50S	0.5135	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60Lr	0.8022	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60Lr	0.8022	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60Lr+0.50W	0.4147	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60Lr+0.50W	0.4147	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60S	0.8022	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60S	0.8022	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60S+0.50W	0.4147	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60S+0.50W	0.4147	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50Lr+W	0.2616	+Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50Lr+W	0.2616	-Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50S+W	0.2616	+Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50S+W	0.2616	-Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.20S	0.4347	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.20S	0.4347	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +0.90D+W	0.4884	+Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +0.90D+W	0.4884	-Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +0.90D	0.2867	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +0.90D	0.2867	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.40D	0.4459	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.40D	0.4459	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50Lr	0.5135	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50Lr	0.5135	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50S	0.5135	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50S	0.5135	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60Lr	0.8022	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60Lr	0.8022	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.4147	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.4147	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60S	0.8022	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60S	0.8022	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60S+0.50W	0.4147	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60S+0.50W	0.4147	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50Lr+W	0.2616	-X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50Lr+W	0.2616	+X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50S+W	0.2616	-X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50S+W	0.2616	+X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.20S	0.4347	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.20S	0.4347	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +0.90D+W	0.4884	-X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +0.90D+W	0.4884	+X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +0.90D	0.2867	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +0.90D	0.2867	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.39 psi	0.39 psi	0.39 psi	0.39 psi	0.39 psi	94.87 psi	0.00	OK
+1.20D+0.50Lr	0.45 psi	0.45 psi	0.45 psi	0.45 psi	0.45 psi	94.87 psi	0.00	OK
+1.20D+0.50S	0.45 psi	0.45 psi	0.45 psi	0.45 psi	0.45 psi	94.87 psi	0.00	OK
+1.20D+1.60Lr	0.70 psi	0.70 psi	0.70 psi	0.70 psi	0.70 psi	94.87 psi	0.01	OK
+1.20D+1.60Lr+0.50W	0.36 psi	0.36 psi	0.36 psi	0.36 psi	0.36 psi	94.87 psi	0.00	OK
+1.20D+1.60S	0.70 psi	0.70 psi	0.70 psi	0.70 psi	0.70 psi	94.87 psi	0.01	OK
+1.20D+1.60S+0.50W	0.36 psi	0.36 psi	0.36 psi	0.36 psi	0.36 psi	94.87 psi	0.00	OK
+1.20D+0.50Lr+W	0.23 psi	0.23 psi	0.23 psi	0.23 psi	0.23 psi	94.87 psi	0.00	OK
+1.20D+0.50S+W	0.23 psi	0.23 psi	0.23 psi	0.23 psi	0.23 psi	94.87 psi	0.00	OK
+1.20D+0.20S	0.38 psi	0.38 psi	0.38 psi	0.38 psi	0.38 psi	94.87 psi	0.00	OK
+0.90D+W	0.43 psi	0.43 psi	0.43 psi	0.43 psi	0.43 psi	94.87 psi	0.00	OK
+0.90D	0.25 psi	0.25 psi	0.25 psi	0.25 psi	0.25 psi	94.87 psi	0.00	OK

General Footing

DESCRIPTION: P4 - Max Load Front of House Moment Frame

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	2.08 psi	189.74 psi	0.01094	OK
+1.20D+0.50Lr	2.39 psi	189.74 psi	0.01259	OK
+1.20D+0.50S	2.39 psi	189.74 psi	0.01259	OK
+1.20D+1.60Lr	3.73 psi	189.74 psi	0.01968	OK
+1.20D+1.60Lr+0.50W	1.93 psi	189.74 psi	0.01017	OK
+1.20D+1.60S	3.73 psi	189.74 psi	0.01968	OK
+1.20D+1.60S+0.50W	1.93 psi	189.74 psi	0.01017	OK
+1.20D+0.50Lr+W	1.22 psi	189.74 psi	0.006415	OK
+1.20D+0.50S+W	1.22 psi	189.74 psi	0.006415	OK
+1.20D+0.20S	2.02 psi	189.74 psi	0.01066	OK
+0.90D+W	2.27 psi	189.74 psi	0.01198	OK
+0.90D	1.33 psi	189.74 psi	0.007031	OK

General Footing

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 | METTEMEYER ENGINEERING LLC | (c) ENERCALC INC 1983-2023

DESCRIPTION: P4 - Front of House F-5

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0 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.	=	0.00180
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	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	1.80 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

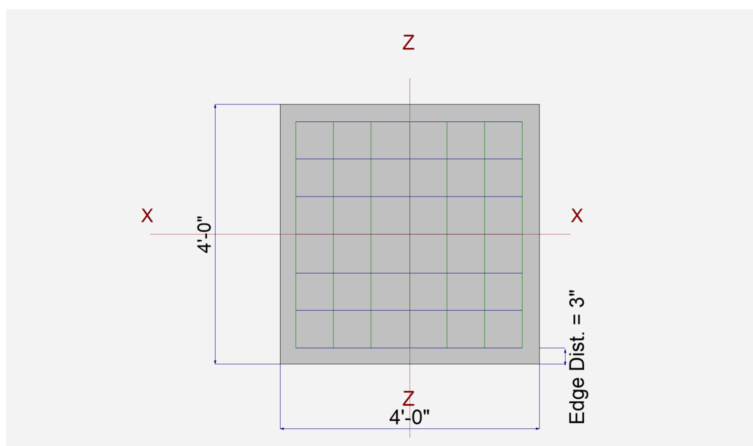
Footing base depth below soil surface	=	2.0 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

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	=	4.0 ft
Length parallel to Z-Z Axis	=	4.0 ft
Footing Thickness	=	22.0 in



Pedestal dimensions...

px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
Height	=	in

Rebar Centerline to Edge of Concrete...

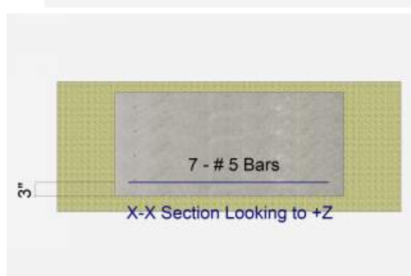
at Bottom of footing	=	3.0 in

Reinforcing

Bars parallel to X-X Axis		
Number of Bars	=	7.0
Reinforcing Bar Size	=	# 5
Bars parallel to Z-Z Axis		
Number of Bars	=	7.0
Reinforcing Bar Size	=	# 5

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



Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	4.370	3.630		3.630	-8.490			k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: P4 - Front of House F-5

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.4357	Soil Bearing	0.7842 ksf	1.80 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	1.050	Uplift	-5.094 k	5.350 k	+0.60D+0.60W
PASS	0.03042	Z Flexure (+X)	1.382 k-ft/ft	45.410 k-ft/ft	+1.20D+1.60Lr
PASS	0.03042	Z Flexure (-X)	1.382 k-ft/ft	45.410 k-ft/ft	+1.20D+1.60Lr
PASS	0.03042	X Flexure (+Z)	1.382 k-ft/ft	45.410 k-ft/ft	+1.20D+1.60Lr
PASS	0.03042	X Flexure (-Z)	1.382 k-ft/ft	45.410 k-ft/ft	+1.20D+1.60Lr
PASS	0.01277	1-way Shear (+X)	1.212 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.01277	1-way Shear (-X)	1.212 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.01277	1-way Shear (+Z)	1.212 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.01277	1-way Shear (-Z)	1.212 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.03389	2-way Punching	6.429 psi	189.737 psi	+1.20D+1.60Lr

Top reinforcing mat required (see 'Bending' tab).

Hand check required for anchor pullout.

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	1.80	n/a	0.0	0.5573	0.5573	n/a	n/a	0.310
X-X, +D+Lr	1.80	n/a	0.0	0.7842	0.7842	n/a	n/a	0.436
X-X, +D+S	1.80	n/a	0.0	0.7842	0.7842	n/a	n/a	0.436
X-X, +D+0.750Lr	1.80	n/a	0.0	0.7274	0.7274	n/a	n/a	0.404
X-X, +D+0.750S	1.80	n/a	0.0	0.7274	0.7274	n/a	n/a	0.404
X-X, +D+0.60W	1.80	n/a	0.0	0.2389	0.2389	n/a	n/a	0.133
X-X, +D+0.750Lr+0.450W	1.80	n/a	0.0	0.4887	0.4887	n/a	n/a	0.272
X-X, +D+0.750S+0.450W	1.80	n/a	0.0	0.4887	0.4887	n/a	n/a	0.272
X-X, +0.60D+0.60W	1.80	n/a	0.0	0.0160	0.0160	n/a	n/a	0.009
X-X, +0.60D	1.80	n/a	0.0	0.3344	0.3344	n/a	n/a	0.186
Z-Z, D Only	1.80	0.0	n/a	n/a	n/a	0.5573	0.5573	0.310
Z-Z, +D+Lr	1.80	0.0	n/a	n/a	n/a	0.7842	0.7842	0.436
Z-Z, +D+S	1.80	0.0	n/a	n/a	n/a	0.7842	0.7842	0.436
Z-Z, +D+0.750Lr	1.80	0.0	n/a	n/a	n/a	0.7274	0.7274	0.404
Z-Z, +D+0.750S	1.80	0.0	n/a	n/a	n/a	0.7274	0.7274	0.404
Z-Z, +D+0.60W	1.80	0.0	n/a	n/a	n/a	0.2389	0.2389	0.133
Z-Z, +D+0.750Lr+0.450W	1.80	0.0	n/a	n/a	n/a	0.4887	0.4887	0.272
Z-Z, +D+0.750S+0.450W	1.80	0.0	n/a	n/a	n/a	0.4887	0.4887	0.272
Z-Z, +0.60D+0.60W	1.80	0.0	n/a	n/a	n/a	0.0160	0.0160	0.009
Z-Z, +0.60D	1.80	0.0	n/a	n/a	n/a	0.3344	0.3344	0.186

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

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	0.7648	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK

General Footing

DESCRIPTION: P4 - Front of House F-5

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	0.7648	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50Lr	0.8824	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50Lr	0.8824	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50S	0.8824	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50S	0.8824	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60Lr	1.382	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60Lr	1.382	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60Lr+0.50W	0.8509	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60Lr+0.50W	0.8509	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60S	1.382	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60S	1.382	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60S+0.50W	0.8509	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+1.60S+0.50W	0.8509	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50Lr+W	0.1789	+Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50Lr+W	0.1789	-Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50S+W	0.1789	+Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.50S+W	0.1789	-Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.20S	0.7463	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +1.20D+0.20S	0.7463	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +0.90D+W	0.5696	+Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +0.90D+W	0.5696	-Z	Top	0.4752	AsMin	0.5425	45.410	OK
X-X, +0.90D	0.4916	+Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
X-X, +0.90D	0.4916	-Z	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.40D	0.7648	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.40D	0.7648	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50Lr	0.8824	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50Lr	0.8824	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50S	0.8824	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50S	0.8824	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60Lr	1.382	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60Lr	1.382	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.8509	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.8509	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60S	1.382	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60S	1.382	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60S+0.50W	0.8509	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+1.60S+0.50W	0.8509	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50Lr+W	0.1789	-X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50Lr+W	0.1789	+X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50S+W	0.1789	-X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.50S+W	0.1789	+X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.20S	0.7463	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +1.20D+0.20S	0.7463	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +0.90D+W	0.5696	-X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +0.90D+W	0.5696	+X	Top	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +0.90D	0.4916	-X	Bottom	0.4752	AsMin	0.5425	45.410	OK
Z-Z, +0.90D	0.4916	+X	Bottom	0.4752	AsMin	0.5425	45.410	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.67 psi	0.67 psi	0.67 psi	0.67 psi	0.67 psi	94.87 psi	0.01	OK
+1.20D+0.50Lr	0.77 psi	0.77 psi	0.77 psi	0.77 psi	0.77 psi	94.87 psi	0.01	OK
+1.20D+0.50S	0.77 psi	0.77 psi	0.77 psi	0.77 psi	0.77 psi	94.87 psi	0.01	OK
+1.20D+1.60Lr	1.21 psi	1.21 psi	1.21 psi	1.21 psi	1.21 psi	94.87 psi	0.01	OK
+1.20D+1.60Lr+0.50W	0.75 psi	0.75 psi	0.75 psi	0.75 psi	0.75 psi	94.87 psi	0.01	OK
+1.20D+1.60S	1.21 psi	1.21 psi	1.21 psi	1.21 psi	1.21 psi	94.87 psi	0.01	OK
+1.20D+1.60S+0.50W	0.75 psi	0.75 psi	0.75 psi	0.75 psi	0.75 psi	94.87 psi	0.01	OK
+1.20D+0.50Lr+W	0.16 psi	0.16 psi	0.16 psi	0.16 psi	0.16 psi	94.87 psi	0.00	OK
+1.20D+0.50S+W	0.16 psi	0.16 psi	0.16 psi	0.16 psi	0.16 psi	94.87 psi	0.00	OK
+1.20D+0.20S	0.65 psi	0.65 psi	0.65 psi	0.65 psi	0.65 psi	94.87 psi	0.01	OK
+0.90D+W	0.50 psi	0.50 psi	0.50 psi	0.50 psi	0.50 psi	94.87 psi	0.01	OK
+0.90D	0.43 psi	0.43 psi	0.43 psi	0.43 psi	0.43 psi	94.87 psi	0.00	OK

DESCRIPTION: P4 - Front of House F-5

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	3.56 psi	189.74 psi	0.01876	OK
+1.20D+0.50Lr	4.11 psi	189.74 psi	0.02164	OK
+1.20D+0.50S	4.11 psi	189.74 psi	0.02164	OK
+1.20D+1.60Lr	6.43 psi	189.74 psi	0.03389	OK
+1.20D+1.60Lr+0.50W	3.96 psi	189.74 psi	0.02087	OK
+1.20D+1.60S	6.43 psi	189.74 psi	0.03389	OK
+1.20D+1.60S+0.50W	3.96 psi	189.74 psi	0.02087	OK
+1.20D+0.50Lr+W	0.83 psi	189.74 psi	0.004387	OK
+1.20D+0.50S+W	0.83 psi	189.74 psi	0.004387	OK
+1.20D+0.20S	3.47 psi	189.74 psi	0.0183	OK
+0.90D+W	2.65 psi	189.74 psi	0.01397	OK
+0.90D	2.29 psi	189.74 psi	0.01206	OK

General Footing

Project File: 24-0121 - afc lee's summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: P5 - Drive Thru B-5

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0	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.	=	0.00180	
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	:	No	
Use Pedestal wt for stability, mom & shear	:	No	

Soil Design Values

Allowable Soil Bearing	=	1.80	ksf
Soil Density	=	110.0	pcf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

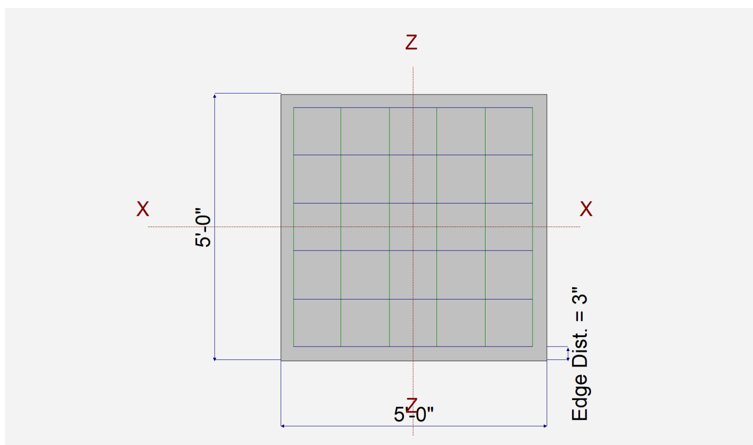
Footing base depth below soil surface	=	2.0	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

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	=	5.0	ft
Length parallel to Z-Z Axis	=	5.0	ft
Footing Thickness	=	22.0	in



Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height	=		in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in

Reinforcing

Bars parallel to X-X Axis

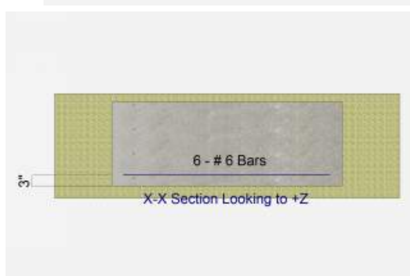
Number of Bars	=		6.0
Reinforcing Bar Size	=	#	6

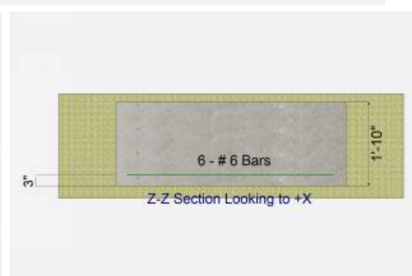
Bars parallel to Z-Z Axis

Number of Bars	=		6.0
Reinforcing Bar Size	=	#	6

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





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	5.750	4.710		4.710	-11.150			k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=	0.1470	0.120		0.120	-0.3210			k
V-z	=								k

General Footing

DESCRIPTION: P5 - Drive Thru B-5

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.4032	Soil Bearing	0.7258 ksf	1.80 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	1.138	Overturning - Z-Z	17.078 k-ft	19.443 k-ft	+0.60D+0.60W
PASS	26.718	Sliding - X-X	0.1044 k	2.789 k	+0.60D+0.60W
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	1.153	Uplift	-6.690 k	7.713 k	+0.60D+0.60W
PASS	0.04233	Z Flexure (+X)	1.872 k-ft/ft	44.222 k-ft/ft	+1.20D+1.60Lr
PASS	0.03928	Z Flexure (-X)	1.737 k-ft/ft	44.222 k-ft/ft	+1.20D+1.60Lr
PASS	0.04081	X Flexure (+Z)	1.805 k-ft/ft	44.222 k-ft/ft	+1.20D+1.60Lr
PASS	0.04081	X Flexure (-Z)	1.805 k-ft/ft	44.222 k-ft/ft	+1.20D+1.60Lr
PASS	0.02513	1-way Shear (+X)	2.384 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.02292	1-way Shear (-X)	2.174 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.02403	1-way Shear (+Z)	2.279 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.02403	1-way Shear (-Z)	2.279 psi	94.868 psi	+1.20D+1.60Lr
PASS	0.04730	2-way Punching	8.974 psi	189.737 psi	+1.20D+1.60Lr

Top reinforcing mat required (see 'Bending' tab).

Hand check required for anchor pullout.

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	1.80	n/a	0.0	0.5142	0.5142	n/a	n/a	0.286
X-X, +D+Lr	1.80	n/a	0.0	0.7026	0.7026	n/a	n/a	0.390
X-X, +D+S	1.80	n/a	0.0	0.7026	0.7026	n/a	n/a	0.390
X-X, +D+0.750Lr	1.80	n/a	0.0	0.6555	0.6555	n/a	n/a	0.364
X-X, +D+0.750S	1.80	n/a	0.0	0.6555	0.6555	n/a	n/a	0.364
X-X, +D+0.60W	1.80	n/a	0.0	0.2466	0.2466	n/a	n/a	0.137
X-X, +D+0.750Lr+0.450W	1.80	n/a	0.0	0.4548	0.4548	n/a	n/a	0.253
X-X, +D+0.750S+0.450W	1.80	n/a	0.0	0.4548	0.4548	n/a	n/a	0.253
X-X, +0.60D+0.60W	1.80	n/a	0.0	0.04090	0.04090	n/a	n/a	0.023
X-X, +0.60D	1.80	n/a	0.0	0.3085	0.3085	n/a	n/a	0.171
Z-Z, D Only	1.80	0.2516	n/a	n/a	n/a	0.5014	0.5270	0.293
Z-Z, +D+Lr	1.80	0.3344	n/a	n/a	n/a	0.6793	0.7258	0.403
Z-Z, +D+S	1.80	0.3344	n/a	n/a	n/a	0.6793	0.7258	0.403
Z-Z, +D+0.750Lr	1.80	0.3182	n/a	n/a	n/a	0.6348	0.6761	0.376
Z-Z, +D+0.750S	1.80	0.3182	n/a	n/a	n/a	0.6348	0.6761	0.376
Z-Z, +D+0.60W	1.80	-0.1627	n/a	n/a	n/a	0.2505	0.2426	0.139
Z-Z, +D+0.750Lr+0.450W	1.80	0.1791	n/a	n/a	n/a	0.4467	0.4628	0.257
Z-Z, +D+0.750S+0.450W	1.80	0.1791	n/a	n/a	n/a	0.4467	0.4628	0.257
Z-Z, +0.60D+0.60W	1.80	-2.246	n/a	n/a	n/a	0.050	0.03181	0.028
Z-Z, +0.60D	1.80	0.2516	n/a	n/a	n/a	0.3008	0.3162	0.176

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	None	0.0 k-ft	Infinity	OK
X-X, +D+Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+S	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750S	None	0.0 k-ft	Infinity	OK
X-X, +D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750S+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +0.60D	None	0.0 k-ft	Infinity	OK
Z-Z, D Only	0.2695 k-ft	32.135 k-ft	119.241	OK
Z-Z, +D+Lr	0.4895 k-ft	43.910 k-ft	89.705	OK

General Footing

DESCRIPTION: P5 - Drive Thru B-5

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Z-Z, +D+S	0.4895 k-ft	43.910 k-ft	89.705	OK
Z-Z, +D+0.750Lr	0.4345 k-ft	40.967 k-ft	94.285	OK
Z-Z, +D+0.750S	0.4345 k-ft	40.967 k-ft	94.285	OK
Z-Z, +D+0.60W	17.078 k-ft	32.405 k-ft	1.897	OK
Z-Z, +D+0.750Lr+0.450W	12.978 k-ft	41.231 k-ft	3.177	OK
Z-Z, +D+0.750S+0.450W	12.978 k-ft	41.231 k-ft	3.177	OK
Z-Z, +0.60D+0.60W	17.078 k-ft	19.443 k-ft	1.138	OK
Z-Z, +0.60D	0.1617 k-ft	19.281 k-ft	119.241	OK

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
X-X, D Only	0.1470 k	6.339 k	43.122	OK
X-X, +D+Lr	0.2670 k	7.752 k	29.033	OK
X-X, +D+S	0.2670 k	7.752 k	29.033	OK
X-X, +D+0.750Lr	0.2370 k	7.399 k	31.218	OK
X-X, +D+0.750S	0.2370 k	7.399 k	31.218	OK
X-X, +D+0.60W	-0.04560 k	4.332 k	94.998	OK
X-X, +D+0.750Lr+0.450W	0.09255 k	5.893 k	63.678	OK
X-X, +D+0.750S+0.450W	0.09255 k	5.893 k	63.678	OK
X-X, +0.60D+0.60W	-0.1044 k	2.789 k	26.718	OK
X-X, +0.60D	0.08820 k	4.796 k	54.381	OK
Z-Z, D Only	0.0 k	6.339 k	No Sliding	OK
Z-Z, +D+Lr	0.0 k	7.752 k	No Sliding	OK
Z-Z, +D+S	0.0 k	7.752 k	No Sliding	OK
Z-Z, +D+0.750Lr	0.0 k	7.399 k	No Sliding	OK
Z-Z, +D+0.750S+0.450W	0.0 k	5.893 k	No Sliding	OK
Z-Z, +0.60D+0.60W	0.0 k	2.789 k	No Sliding	OK
Z-Z, +0.60D	0.0 k	4.796 k	No Sliding	OK
Z-Z, +D+0.750S	0.0 k	7.399 k	No Sliding	OK
Z-Z, +D+0.60W	0.0 k	4.332 k	No Sliding	OK
Z-Z, +D+0.750Lr+0.450W	0.0 k	5.893 k	No Sliding	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	1.006	+Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.40D	1.006	-Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.50Lr	1.157	+Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.50Lr	1.157	-Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.50S	1.157	+Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.50S	1.157	-Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+1.60Lr	1.805	+Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+1.60Lr	1.805	-Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+1.60Lr+0.50W	1.108	+Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+1.60Lr+0.50W	1.108	-Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+1.60S	1.805	+Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+1.60S	1.805	-Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+1.60S+0.50W	1.108	+Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+1.60S+0.50W	1.108	-Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.50Lr+W	0.2369	+Z	Top	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.50Lr+W	0.2369	-Z	Top	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.50S+W	0.2369	+Z	Top	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.50S+W	0.2369	-Z	Top	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.20S	0.9803	+Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +1.20D+0.20S	0.9803	-Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +0.90D+W	0.7469	+Z	Top	0.4752	AsMin	0.5280	44.222	OK
X-X, +0.90D+W	0.7469	-Z	Top	0.4752	AsMin	0.5280	44.222	OK
X-X, +0.90D	0.6469	+Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
X-X, +0.90D	0.6469	-Z	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.40D	0.9685	-X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.40D	1.044	+X	Bottom	0.4752	AsMin	0.5280	44.222	OK

General Footing

Project File: 24-0121 - afc lee's summit.ec6

DESCRIPTION: P5 - Drive Thru B-5

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+0.50Lr	1.114	-X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+0.50Lr	1.20	+X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+0.50S	1.114	-X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+0.50S	1.20	+X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+1.60Lr	1.737	-X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+1.60Lr	1.872	+X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+1.60Lr+0.50W	1.070	-X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+1.60Lr+0.50W	1.146	+X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+1.60S	1.737	-X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+1.60S	1.872	+X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+1.60S+0.50W	1.070	-X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+1.60S+0.50W	1.146	+X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+0.50Lr+W	0.2214	-X	Top	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+0.50Lr+W	0.2524	+X	Top	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+0.50S+W	0.2214	-X	Top	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+0.50S+W	0.2524	+X	Top	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+0.20S	0.9435	-X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +1.20D+0.20S	1.017	+X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +0.90D+W	0.7123	-X	Top	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +0.90D+W	0.7815	+X	Top	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +0.90D	0.6226	-X	Bottom	0.4752	AsMin	0.5280	44.222	OK
Z-Z, +0.90D	0.6711	+X	Bottom	0.4752	AsMin	0.5280	44.222	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.21 psi	1.33 psi	1.27 psi	1.27 psi	1.33 psi	94.87 psi	0.01	OK
+1.20D+0.50Lr	1.39 psi	1.53 psi	1.46 psi	1.46 psi	1.53 psi	94.87 psi	0.02	OK
+1.20D+0.50S	1.39 psi	1.53 psi	1.46 psi	1.46 psi	1.53 psi	94.87 psi	0.02	OK
+1.20D+1.60Lr	2.17 psi	2.38 psi	2.28 psi	2.28 psi	2.38 psi	94.87 psi	0.03	OK
+1.20D+1.60Lr+0.50W	1.34 psi	1.46 psi	1.40 psi	1.40 psi	1.46 psi	94.87 psi	0.02	OK
+1.20D+1.60S	2.17 psi	2.38 psi	2.28 psi	2.28 psi	2.38 psi	94.87 psi	0.03	OK
+1.20D+1.60S+0.50W	1.34 psi	1.46 psi	1.40 psi	1.40 psi	1.46 psi	94.87 psi	0.02	OK
+1.20D+0.50Lr+W	0.28 psi	0.32 psi	0.30 psi	0.30 psi	0.32 psi	94.87 psi	0.00	OK
+1.20D+0.50S+W	0.28 psi	0.32 psi	0.30 psi	0.30 psi	0.32 psi	94.87 psi	0.00	OK
+1.20D+0.20S	1.18 psi	1.30 psi	1.24 psi	1.24 psi	1.30 psi	94.87 psi	0.01	OK
+0.90D+W	0.89 psi	1.00 psi	0.94 psi	0.94 psi	1.00 psi	94.87 psi	0.01	OK
+0.90D	0.78 psi	0.85 psi	0.82 psi	0.82 psi	0.85 psi	94.87 psi	0.01	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	5.00 psi	189.74 psi	0.02637	OK
+1.20D+0.50Lr	5.75 psi	189.74 psi	0.03032	OK
+1.20D+0.50S	5.75 psi	189.74 psi	0.03032	OK
+1.20D+1.60Lr	8.97 psi	189.74 psi	0.0473	OK
+1.20D+1.60Lr+0.50W	5.51 psi	189.74 psi	0.02903	OK
+1.20D+1.60S	8.97 psi	189.74 psi	0.0473	OK
+1.20D+1.60S+0.50W	5.51 psi	189.74 psi	0.02903	OK
+1.20D+0.50Lr+W	1.18 psi	189.74 psi	0.006208	OK
+1.20D+0.50S+W	1.18 psi	189.74 psi	0.006208	OK
+1.20D+0.20S	4.88 psi	189.74 psi	0.02569	OK
+0.90D+W	3.71 psi	189.74 psi	0.01958	OK
+0.90D	3.22 psi	189.74 psi	0.01695	OK

General Footing

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC (c) ENERCALC INC 1983-2023

DESCRIPTION: P6.5 - Canopy Footing - Max Axial Loads

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0 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.	=	0.00180
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	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	1.80 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

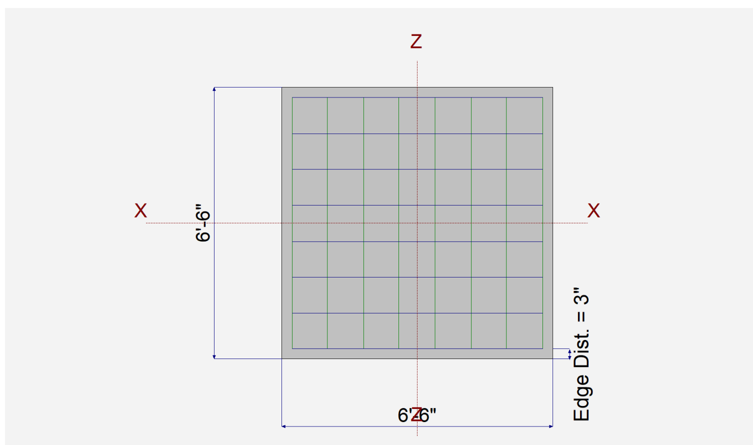
Footing base depth below soil surface	=	ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

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	=	6.50 ft
Length parallel to Z-Z Axis	=	6.50 ft
Footing Thickness	=	22.0 in



Pedestal dimensions...

px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
Height	=	in

Rebar Centerline to Edge of Concrete...

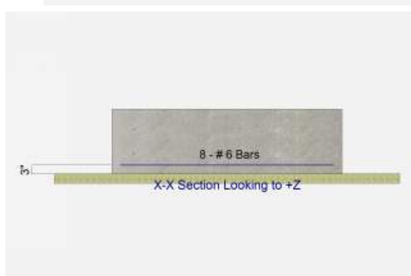
at Bottom of footing	=	3.0 in

Reinforcing

Bars parallel to X-X Axis		
Number of Bars	=	8.0
Reinforcing Bar Size	=	# 6
Bars parallel to Z-Z Axis		
Number of Bars	=	8.0
Reinforcing Bar Size	=	# 6

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



Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	12.20	6.30		12.40	-8.910			k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=	0.360	0.170		0.620	34.460			k-ft
V-x	=	0.030	0.010		0.040	2.410			k
V-z	=								k

General Footing

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: P6.5 - Canopy Footing - Max Axial Loads

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8944	Soil Bearing	1.610 ksf	1.80 ksf	+0.60D+0.60W about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	1.116	Overturning - Z-Z	40.951 k-ft	45.691 k-ft	+0.60D+0.60W
PASS	1.785	Sliding - X-X	1.464 k	2.614 k	+0.60D+0.60W
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	2.630	Uplift	-5.346 k	14.059 k	+0.60D+0.60W
PASS	0.1185	Z Flexure (+X)	5.372 k-ft/ft	45.331 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.09235	Z Flexure (-X)	4.186 k-ft/ft	45.331 k-ft/ft	+1.20D+1.60S
PASS	0.09508	X Flexure (+Z)	4.310 k-ft/ft	45.331 k-ft/ft	+1.20D+1.60S
PASS	0.09508	X Flexure (-Z)	4.310 k-ft/ft	45.331 k-ft/ft	+1.20D+1.60S
PASS	0.08211	1-way Shear (+X)	7.790 psi	94.868 psi	+1.20D+1.60S+0.50W
PASS	0.06173	1-way Shear (-X)	5.857 psi	94.868 psi	+1.20D+1.60S
PASS	0.06376	1-way Shear (+Z)	6.049 psi	94.868 psi	+1.20D+1.60S
PASS	0.06376	1-way Shear (-Z)	6.049 psi	94.868 psi	+1.20D+1.60S
PASS	0.1186	2-way Punching	22.503 psi	189.737 psi	+1.20D+1.60S

Top reinforcing mat required (see 'Bending' tab).

Hand check required for anchor pullout.

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	1.80	n/a	0.0	0.5546	0.5546	n/a	n/a	0.308
X-X, +D+Lr	1.80	n/a	0.0	0.7037	0.7037	n/a	n/a	0.391
X-X, +D+S	1.80	n/a	0.0	0.8481	0.8481	n/a	n/a	0.471
X-X, +D+0.750Lr	1.80	n/a	0.0	0.6664	0.6664	n/a	n/a	0.370
X-X, +D+0.750S	1.80	n/a	0.0	0.7747	0.7747	n/a	n/a	0.430
X-X, +D+0.60W	1.80	n/a	0.0	0.4281	0.4281	n/a	n/a	0.238
X-X, +D+0.750Lr+0.450W	1.80	n/a	0.0	0.5715	0.5715	n/a	n/a	0.318
X-X, +D+0.750S+0.450W	1.80	n/a	0.0	0.6798	0.6798	n/a	n/a	0.378
X-X, +0.60D+0.60W	1.80	n/a	0.0	0.2062	0.2062	n/a	n/a	0.115
X-X, +0.60D	1.80	n/a	0.0	0.3328	0.3328	n/a	n/a	0.185
Z-Z, D Only	1.80	0.2125	n/a	n/a	n/a	0.5456	0.5636	0.313
Z-Z, +D+Lr	1.80	0.2435	n/a	n/a	n/a	0.6907	0.7168	0.398
Z-Z, +D+S	1.80	0.3712	n/a	n/a	n/a	0.8241	0.8721	0.485
Z-Z, +D+0.750Lr	1.80	0.2371	n/a	n/a	n/a	0.6544	0.6785	0.377
Z-Z, +D+0.750S	1.80	0.3428	n/a	n/a	n/a	0.7545	0.7949	0.442
Z-Z, +D+0.60W	1.80	15.753	n/a	n/a	n/a	0.0	0.9522	0.529
Z-Z, +D+0.750Lr+0.450W	1.80	8.971	n/a	n/a	n/a	0.1811	0.9620	0.534
Z-Z, +D+0.750S+0.450W	1.80	7.70	n/a	n/a	n/a	0.2812	1.078	0.599
Z-Z, +0.60D+0.60W	1.80	32.471	n/a	n/a	n/a	0.0	1.610	0.894
Z-Z, +0.60D	1.80	0.2125	n/a	n/a	n/a	0.3274	0.3381	0.188

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	None	0.0 k-ft	Infinity	OK
X-X, +D+Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+S	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750S	None	0.0 k-ft	Infinity	OK
X-X, +D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750S+0.450W	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.60W	None	0.0 k-ft	Infinity	OK
X-X, +0.60D	None	0.0 k-ft	Infinity	OK
Z-Z, D Only	0.4150 k-ft	76.152 k-ft	183.499	OK
Z-Z, +D+Lr	0.6033 k-ft	96.627 k-ft	160.156	OK

General Footing

Project File: 24-0121 - AFC Lee's Summit.ec6

DESCRIPTION: P6.5 - Canopy Footing - Max Axial Loads

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Z-Z, +D+S	1.108 k-ft	116.452 k-ft	105.070	OK
Z-Z, +D+0.750Lr	0.5563 k-ft	91.508 k-ft	164.510	OK
Z-Z, +D+0.750S	0.9350 k-ft	106.377 k-ft	113.772	OK
Z-Z, +D+0.60W	41.117 k-ft	76.152 k-ft	1.852	OK
Z-Z, +D+0.750Lr+0.450W	31.082 k-ft	91.508 k-ft	2.944	OK
Z-Z, +D+0.750S+0.450W	31.461 k-ft	106.377 k-ft	3.381	OK
Z-Z, +0.60D+0.60W	40.951 k-ft	45.691 k-ft	1.116	OK
Z-Z, +0.60D	0.2490 k-ft	45.691 k-ft	183.499	OK

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
X-X, D Only	0.030 k	7.029 k	234.315	OK
X-X, +D+Lr	0.040 k	8.919 k	222.986	OK
X-X, +D+S	0.070 k	10.749 k	153.563	OK
X-X, +D+0.750Lr	0.03750 k	8.447 k	225.252	OK
X-X, +D+0.750S	0.060 k	9.819 k	163.657	OK
X-X, +D+0.60W	1.476 k	5.426 k	3.676	OK
X-X, +D+0.750Lr+0.450W	1.122 k	7.244 k	6.456	OK
X-X, +D+0.750S+0.450W	1.145 k	8.617 k	7.529	OK
X-X, +0.60D+0.60W	1.464 k	2.614 k	1.785	OK
X-X, +0.60D	0.0180 k	4.218 k	234.315	OK
Z-Z, D Only	0.0 k	7.029 k	No Sliding	OK
Z-Z, +D+Lr	0.0 k	8.919 k	No Sliding	OK
Z-Z, +D+S	0.0 k	10.749 k	No Sliding	OK
Z-Z, +D+0.750Lr	0.0 k	8.447 k	No Sliding	OK
Z-Z, +D+0.750S+0.450W	0.0 k	8.617 k	No Sliding	OK
Z-Z, +0.60D+0.60W	0.0 k	2.614 k	No Sliding	OK
Z-Z, +0.60D	0.0 k	4.218 k	No Sliding	OK
Z-Z, +D+0.750S	0.0 k	9.819 k	No Sliding	OK
Z-Z, +D+0.60W	0.0 k	5.426 k	No Sliding	OK
Z-Z, +D+0.750Lr+0.450W	0.0 k	7.244 k	No Sliding	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	2.135	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.40D	2.135	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.50Lr	2.224	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.50Lr	2.224	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.50S	2.605	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.50S	2.605	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+1.60Lr	3.090	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+1.60Lr	3.090	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+1.60Lr+0.50W	2.533	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+1.60Lr+0.50W	2.533	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+1.60S	4.310	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+1.60S	4.310	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+1.60S+0.50W	3.753	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+1.60S+0.50W	3.753	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.50Lr+W	1.110	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.50Lr+W	1.110	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.50S+W	1.491	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.50S+W	1.491	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +0.90D+W	0.2588	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +0.90D+W	0.2588	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.20S	2.140	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +1.20D+0.20S	2.140	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +0.90D	1.373	+Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
X-X, +0.90D	1.373	-Z	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.40D	2.090	-X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.40D	2.180	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK

General Footing

DESCRIPTION: P6.5 - Canopy Footing - Max Axial Loads

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+0.50Lr	2.178	-X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+0.50Lr	2.269	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+0.50S	2.540	-X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+0.50S	2.670	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+1.60Lr	3.029	-X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+1.60Lr	3.151	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.9765	-X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+1.60Lr+0.50W	4.090	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+1.60S	4.186	-X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+1.60S	4.434	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+1.60S+0.50W	2.134	-X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+1.60S+0.50W	5.372	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+0.50Lr+W	1.584	-X	Top	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+0.50Lr+W	4.488	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+0.50S+W	1.383	-X	Top	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+0.50S+W	4.728	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +0.90D+W	1.264	-X	Top	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +0.90D+W	4.536	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+0.20S	2.091	-X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +1.20D+0.20S	2.189	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +0.90D	1.344	-X	Bottom	0.4752	AsMin	0.5415	45.331	OK
Z-Z, +0.90D	1.401	+X	Bottom	0.4752	AsMin	0.5415	45.331	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	2.93 psi	3.07 psi	3.00 psi	3.00 psi	3.07 psi	94.87 psi	0.03	OK
+1.20D+0.50Lr	3.05 psi	3.19 psi	3.12 psi	3.12 psi	3.19 psi	94.87 psi	0.03	OK
+1.20D+0.50S	3.56 psi	3.76 psi	3.66 psi	3.66 psi	3.76 psi	94.87 psi	0.04	OK
+1.20D+1.60Lr	4.24 psi	4.43 psi	4.34 psi	4.34 psi	4.43 psi	94.87 psi	0.05	OK
+1.20D+1.60Lr+0.50W	1.13 psi	5.98 psi	3.56 psi	3.56 psi	5.98 psi	94.87 psi	0.06	OK
+1.20D+1.60S	5.86 psi	6.24 psi	6.05 psi	6.05 psi	6.24 psi	94.87 psi	0.07	OK
+1.20D+1.60S+0.50W	2.75 psi	7.79 psi	5.27 psi	5.27 psi	7.79 psi	94.87 psi	0.08	OK
+1.20D+0.50Lr+W	2.37 psi	6.91 psi	1.56 psi	1.56 psi	6.91 psi	94.87 psi	0.07	OK
+1.20D+0.50S+W	2.32 psi	7.18 psi	2.09 psi	2.09 psi	7.18 psi	94.87 psi	0.08	OK
+0.90D+W	1.77 psi	6.13 psi	0.36 psi	0.36 psi	6.13 psi	94.87 psi	0.06	OK
+1.20D+0.20S	2.93 psi	3.08 psi	3.00 psi	3.00 psi	3.08 psi	94.87 psi	0.03	OK
+0.90D	1.88 psi	1.97 psi	1.93 psi	1.93 psi	1.97 psi	94.87 psi	0.02	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	11.15 psi	189.74 psi	0.05875	OK
+1.20D+0.50Lr	11.61 psi	189.74 psi	0.06119	OK
+1.20D+0.50S	13.60 psi	189.74 psi	0.07168	OK
+1.20D+1.60Lr	16.13 psi	189.74 psi	0.08503	OK
+1.20D+1.60Lr+0.50W	13.23 psi	189.74 psi	0.06971	OK
+1.20D+1.60S	22.50 psi	189.74 psi	0.1186	OK
+1.20D+1.60S+0.50W	19.60 psi	189.74 psi	0.1033	OK
+1.20D+0.50Lr+W	5.98 psi	189.74 psi	0.03153	OK
+1.20D+0.50S+W	7.87 psi	189.74 psi	0.04147	OK
+0.90D+W	1.52 psi	189.74 psi	0.007994	OK
+1.20D+0.20S	11.17 psi	189.74 psi	0.05889	OK
+0.90D	7.17 psi	189.74 psi	0.03777	OK

Beam on Elastic Foundation

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

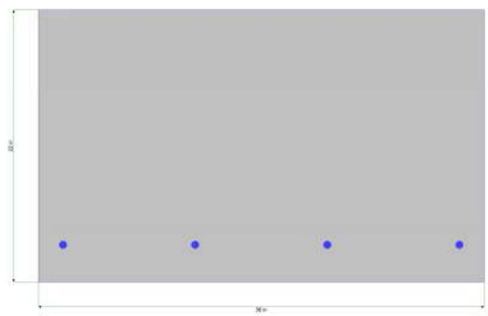
DESCRIPTION: Trash Enclosure Wall Footing

CODE REFERENCES

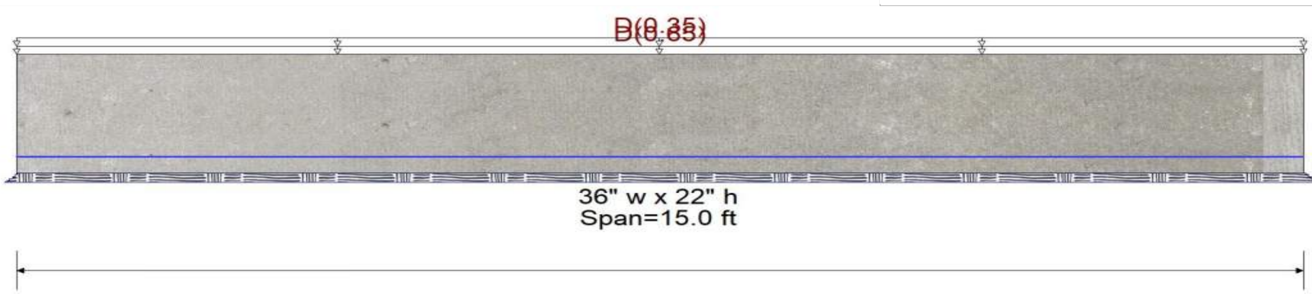
Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

Material Properties

f'c	=	3.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	410.792 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ Lt Wt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi			
Soil Subgrade Modulus	=	250.0 psi / (inch deflection)			
Load Combination		ASCE 7-16			
fy - Main Rebar	=	60.0 ksi	Fy - Stirrups	=	40.0 ksi
E - Main Rebar	=	29,000.0 ksi	E - Stirrups	=	29,000.0 ksi
			Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup		2



Beam is supported on an elastic foundation.



Cross Section & Reinforcing Details

Rectangular Section, Width = 36.0 in, Height = 22.0 in
Span #1 Reinforcing....
4-#5 at 3.0 in from Bottom, from 0.0 to 15.0 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.650 k/ft, Tributary Width = 1.0 ft, (Masonry Wall)
Uniform Load : D = 0.350 k/ft, Tributary Width = 1.0 ft, (Brick)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.000**: 1	Maximum Deflection	
Section used for this span	**Typical Section**	Max Downward L+Lr+S Deflection	0.000 in
Mu : Applied	-0.001181 k-ft	Max Upward L+Lr+S Deflection	0.000 in
Mn * Phi : Allowable	103.759 k-ft	Max Downward Total Deflection	0.017 in
Load Combination	+1.40D	Max Upward Total Deflection	0.007 in
Location of maximum on span	7.412 ft		
Span # where maximum occurs	Span # 1		

Maximum Soil Pressure = **0.599** ksf at 7.50 ft LdComb: D Only
Allowable Soil Pressure = **1.80** ksf **OK**

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combination

Load Combination		Location (ft)	Bending Stress Results (k-ft		
Segment Length	Span #	in Span	Mu : Max	Phi*Mnx	Stress Ratio
MAXimum Bending Envelope					
Span # 1	1	14.824	-0.00	103.76	0.00
+1.40D					
Span # 1	1	14.824	-0.00	103.76	0.00

Beam on Elastic Foundation

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Trash Enclosure Wall Footing

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
+1.20D					
Span # 1	1	14.824	-0.00	103.76	0.00
+0.90D					
Span # 1	1	14.824	-0.00	103.76	0.00

Overall Maximum Deflections - Unfactored Lo

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Span 1	1	0.0166	7.500		0.0000	0.000

Wall Footing

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Trash Enclosure Footing

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-10

General Information

Material Properties

f'c : Concrete 28 day strength	=	4.0	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.	=	0.00180	
Min. Overturning Safety Factor	=	1.0 : 1	
Min. Sliding Safety Factor	=	1.0 : 1	
AutoCalc Footing Weight as DL :		Yes	

Soil Design Values

Allowable Soil Bearing	=	1.80	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

Reference Depth below Surface	=		ft
Allow. Pressure Increase per foot of depth	=		ksf
when base footing is below	=		ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=		ksf
when footing is wider than	=		ft

Adjusted Allowable Bearing Pressure = 1.80 ksf

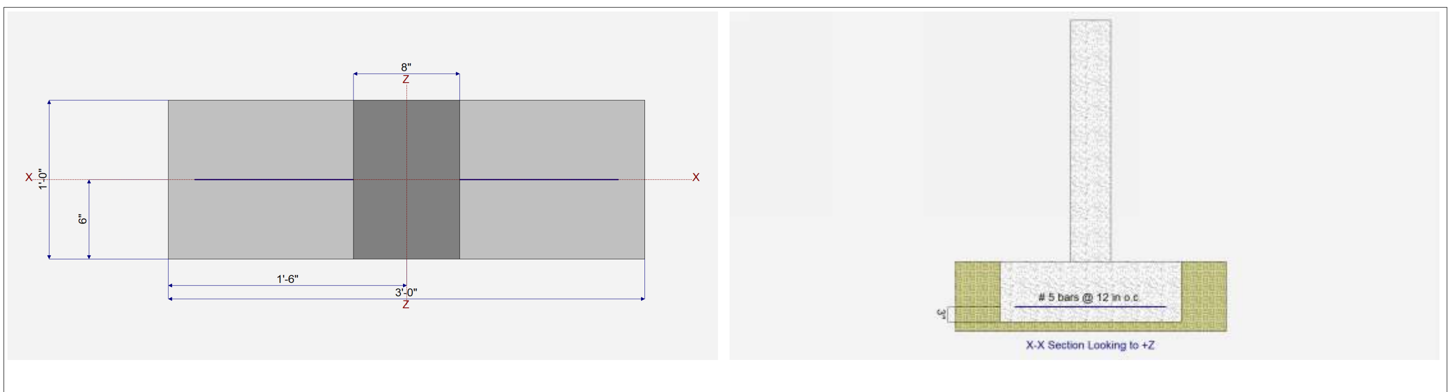
Dimensions

Footing Width	=	3.0	ft
Wall Thickness	=	8.0	in
Wall center offset from center of footing	=	0	in
Footing Thickness	=	12.0	in
Rebar Centerline to Edge of Concrete... at Bottom of footing	=	3.0	in

Reinforcing

Bars along X-X Axis

Bar spacing	=	12.00
Reinforcing Bar Size	=	# 5



Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	1.0							k
OB : Overburden	=								ksf
V-x	=					0.180			k
M-zz	=					0.720			k-ft

Vx applied = in above top of footing

Wall Footing

Project File: 24-0121 - AFC Lee's Summit.ec6

LIC# : KW-06015913, Build:20.23.08.30 METTEMEYER ENGINEERING LLC

DESCRIPTION: Trash Enclosure Footing

DESIGN SUMMARY

Design OK

	Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS	2.392	Overturning - Z-Z	0.540 k-ft	1.292 k-ft	+0.60D+0.60W
PASS	2.392	Sliding - X-X	0.1080 k	0.2583 k	+0.60D+0.60W
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift

	Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.4656	Soil Bearing	0.8381 ksf	1.80 ksf	+D+0.60W
PASS	0.05666	Z Flexure (+X)	0.6934 k-ft	12.237 k-ft	+1.20D+W
PASS	0.02394	Z Flexure (-X)	0.2930 k-ft	12.237 k-ft	+0.90D
PASS	0.02745	1-way Shear (+X)	2.604 psi	94.868 psi	+1.40D
PASS	0.02745	1-way Shear (-X)	2.604 psi	94.868 psi	+1.40D

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Actual Soil Bearing Stress -X	Actual Soil Bearing Stress +X	Actual / Allowable Ratio
, D Only	1.80 ksf	0.0 in	0.4783 ksf	0.4783 ksf	0.266
, +D+0.60W	1.80 ksf	4.516 in	0.1186 ksf	0.8381 ksf	0.466
, +D+0.450W	1.80 ksf	3.387 in	0.2085 ksf	0.7482 ksf	0.416
, +0.60D+0.60W	1.80 ksf	7.526 in	0.0 ksf	0.6574 ksf	0.365
, +0.60D	1.80 ksf	0.0 in	0.2870 ksf	0.2870 ksf	0.159

Overturning Stability

Units : k-ft

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
, D Only	None	0.0 k-ft	Infinity	OK
, +D+0.60W	0.540 k-ft	2.153 k-ft	3.986	OK
, +D+0.450W	0.4050 k-ft	2.153 k-ft	5.315	OK
, +0.60D+0.60W	0.540 k-ft	1.292 k-ft	2.392	OK
, +0.60D	None	0.0 k-ft	Infinity	OK

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status
, D Only	0.0 k	0.4305 k	No Sliding	OK
, +D+0.60W	0.1080 k	0.4305 k	3.986	OK
, +D+0.450W	0.0810 k	0.4305 k	5.315	OK
, +0.60D+0.60W	0.1080 k	0.2583 k	2.392	OK
, +0.60D	0.0 k	0.2583 k	No Sliding	OK

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Which Side ?	Tension @ Bot. or Top ?	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
, +1.40D	0.4557	-X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +1.40D	0.4557	+X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +1.20D	0.3906	-X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +1.20D	0.3906	+X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +1.20D+0.50W	0.2394	-X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +1.20D+0.50W	0.5419	+X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +1.20D+W	0.08856	-X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +1.20D+W	0.6934	+X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +0.90D+W	0.01418	-X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +0.90D+W	0.6118	+X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +0.90D	0.293	-X	Bottom	0.2592	Min Temp %	0.31	12.237	OK
, +0.90D	0.293	+X	Bottom	0.2592	Min Temp %	0.31	12.237	OK

One Way Shear

Units : k

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	2.604 psi	2.604 psi	2.604 psi	94.868 psi	0.02745	OK
+1.20D	2.232 psi	2.232 psi	2.232 psi	94.868 psi	0.02353	OK
+1.20D+0.50W	2.232 psi	2.232 psi	2.232 psi	94.868 psi	0.02353	OK
+1.20D+W	2.232 psi	2.232 psi	2.232 psi	94.868 psi	0.02353	OK

DESCRIPTION: Trash Enclosure Footing

One Way Shear

Units : k

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+0.90D+W	1.674 psi	1.674 psi	1.674 psi	94.868 psi	0.01765	OK
+0.90D	1.674 psi	1.674 psi	1.674 psi	94.868 psi	0.01765	OK