STRUCTURAL CALCULATIONS

FOR

Club Car Wash
Lee´s Summit, Missouri

Date:
August 20, 2025

8/20/25

GREGORY L. LINNEMAN - PE
MO LICENSE - 2005001013

PREPARED BY:

CROCKETT
ENGINEERING CONSULTANTS

1000 W Nifong Boulevard, Bldg 1 • Columbia, Missouri 65203

DESIGN DATA

<u>2018 INTERNATIONAL BUILDING CODE / ASCE 7-16</u>

<u>BUILDING OCCUPANCY CATEGORY</u>   II

<u>ROOF LOAD DATA</u>

   LIVE LOAD   20

   ROOFING MEMBRANE & INSULATION   8

   MECHANICAL ALLOWANCE/SOLAR   5

   STEEL FRAMING   2

   TOTAL TO WALLS   35 lbs/sq.ft

<u>ROOF SNOW LOAD DATA*</u>   (*UNBALANCED & DRIFTING SNOW TO BE DETERMINED
                                 IN ADDITION TO UNIFORM LOAD, WHERE APPLICABLE)

   $pg =$ 20 lbs/sq.ft

   $C_e =$ 1.0

   $I_s =$ 1.0

   $C_t =$ 1.0

   $pr =$ 14 lbs/sq.ft

<u>WIND DESIGN DATA</u>

   $V_{ult} =$ 109 M.P.H. (3-SECOND GUST)

   RISK CATEGORY   II

   EXPOSURE   C

   INTERNAL PRESSURE COEFFICIENT =   ± 0.18

   DIRECTIONAL PROCEDURE (MWFRS - ASCE 7-10, CH 27; C&C - ASCE 7-10, CH 30, PART 4)

   MAXIMUM COMPONENTS & CLADDING WIND   +40/-40 lbs/sq.ft

<u>EARTHQUAKE DESIGN DATA</u>

   RISK CATEGORY   II

   $I_E =$ 1.0

   $S_S =$ 0.1

   $S_1 =$ 0.068

   SITE CLASS   C

   $S_{DS} =$ 0.087

   $S_{D1} =$ 0.068

   SEISMIC DESIGN CATEGORY   B

   BASIC SEISMIC-FORCE-RESISTING SYSTEM =

       INTERMEDIATE REINFORCED MASONRY SHEAR BEARING WALLS

   $R =$ 3.50

   $\Omega_o =$ 2.50

   $C_d =$ 2.25

   DESIGN BASE SHEAR   0.025W

   EQUIVALENT LATERAL FORCE PROCEDURE
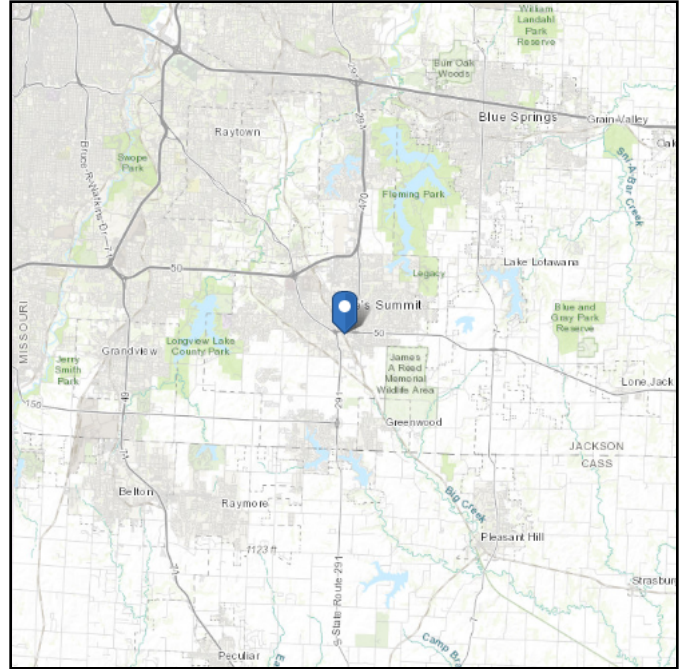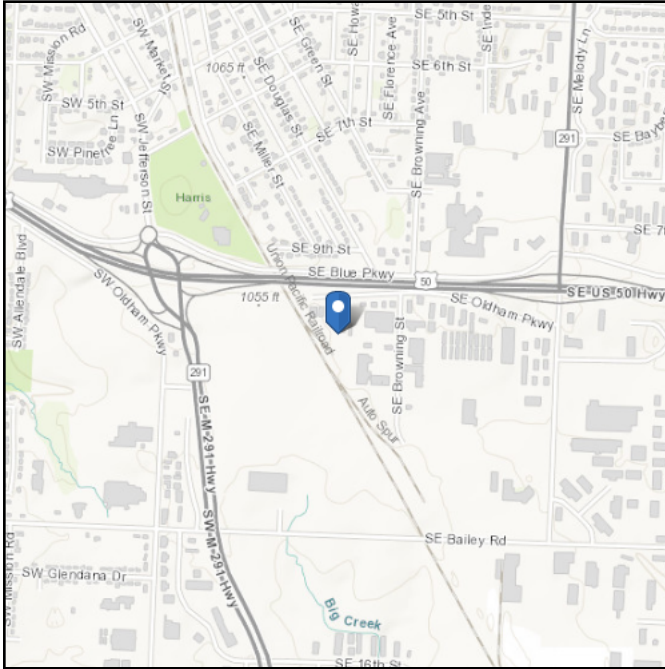
<u>NET ALLOWABLE SOIL BEARING</u>   2,000 STRIP/2,400 PAD lbs/sq.ft

ASCE Hazards Report

Address:
9 SE Oldham Pkwy
Lees Summit, Missouri
64081

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: C - Very Dense Soil and Soft Rock

Latitude: 38.901544
Longitude: -94.370978
Elevation: 1038.8334111101221 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: Wed Aug 20 2025

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

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

Crockett Engineering
1000 W Nifong Blvd
Columbia Missouri 65203
573-447-0292

JOB TITLE Club Wash

JOB NO. _____ SHEET NO. _____
CALCULATED BY _____ DATE _____
CHECKED BY _____ DATE _____

Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

$Kz = Kh$ (case 1) =	0.85	Edge Strip (a) =	3.7 ft
Base pressure (qh) =	**24.4 psf**	End Zone (2a) =	7.5 ft
GCpi =	+/-0.18	Zone 2 length =	18.7 ft

Wind Pressure Coefficients

Surface	CASE A	θ = 1.2 deg			CASE B		
	GCpf	w/-GCpi	w/+GCpi		GCpf	w/-GCpi	w/+GCpi
1	0.40	0.58	0.22		-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87		-0.69	-0.51	-0.87
3	-0.37	-0.19	-0.55		-0.37	-0.19	-0.55
4	-0.29	-0.11	-0.47		-0.45	-0.27	-0.63
5					0.40	0.58	0.22
6					-0.29	-0.11	-0.47
1E	0.61	0.79	0.43		-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25		-1.07	-0.89	-1.25
3E	-0.53	-0.35	-0.71		-0.53	-0.35	-0.71
4E	-0.43	-0.25	-0.61		-0.48	-0.30	-0.66
5E					0.61	0.79	0.43
6E					-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

1	14.2	5.4			-6.6	-15.4
2	-12.5	-21.3			-12.5	-21.3
3	-4.6	-13.4			-4.6	-13.4
4	-2.7	-11.5			-6.6	-15.4
5					14.2	5.4
6					-2.7	-11.5
1E	19.3	10.5			-7.3	-16.1
2E	-21.7	-30.5			-21.7	-30.5
3E	-8.6	-17.3			-8.6	-17.3
4E	-6.1	-14.9			-7.3	-16.1
5E					19.3	10.5
6E					-6.1	-14.9

Parapet

Windward parapet = 38.1 psf (GCpn = +1.5)   Windward roof
Leeward parapet = -25.4 psf (GCpn = -1.0)   overhangs = 17.1 psf (upward) add to
windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)
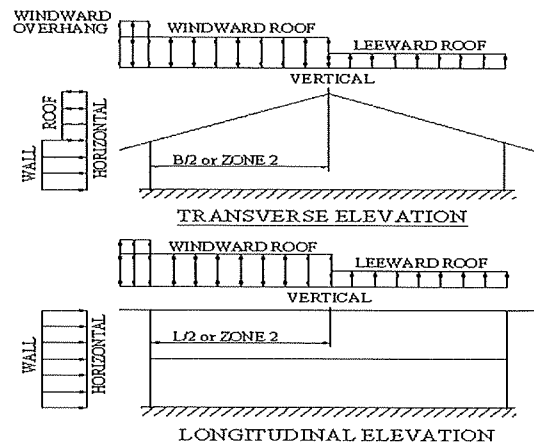Interior Zone: Wall 16.9 psf
Roof -7.8 psf **
End Zone: Wall 25.4 psf
Roof -13.2 psf **

Longitudinal direction (parallel to L)
Interior Zone: Wall 16.9 psf
End Zone: Wall 25.4 psf

** NOTE: Total horiz force shall not be less than that determined
by neglecting roof forces (except for MWFRS moment frames).

The code requires the MWFRS be designed for a min ultimate
force of 16 psf multiplied by the wall area plus an 8 psf force
applied to the vertical projection of the roof.

WINDWARD OVERHANG — WINDWARD ROOF — LEEWARD ROOF
VERTICAL
WALL / ROOF / HORIZONTAL
B/2 or ZONE 2
TRANSVERSE ELEVATION

WINDWARD ROOF — LEEWARD ROOF
VERTICAL
WALL / HORIZONTAL
L/2 or ZONE 2
LONGITUDINAL ELEVATION

Crockett Engineering
1000 W Nifong Blvd
Columbia Missouri 65203
573-447-0292

JOB TITLE Club Wash

JOB NO. _____   SHEET NO. _____
CALCULATED BY _____   DATE _____
CHECKED BY _____   DATE _____

Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

$Kz = Kh$ (case 1) =	0.96	Edge Strip (a) =	3.7 ft
Base pressure (qh) =	**27.5 psf**	End Zone (2a) =	7.5 ft
GCpi =	+/-0.18	Zone 2 length =	18.7 ft

Wind Pressure Coefficients

Surface	CASE A θ = 7.1 deg GCpf	w/-GCpi	w/+GCpi		CASE B GCpf	w/-GCpi	w/+GCpi
1	0.42	0.60	0.24		-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87		-0.69	-0.51	-0.87
3	-0.39	-0.21	-0.57		-0.37	-0.19	-0.55
4	-0.31	-0.13	-0.49		-0.45	-0.27	-0.63
5					0.40	0.58	0.22
6					-0.29	-0.11	-0.47
1E	0.64	0.82	0.46		-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25		-1.07	-0.89	-1.25
3E	-0.55	-0.37	-0.73		-0.53	-0.35	-0.71
4E	-0.46	-0.28	-0.64		-0.48	-0.30	-0.66
5E					0.61	0.79	0.43
6E					-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

1	16.5	6.6		-7.4	-17.3	
2	-14.0	-24.0		-14.0	-24.0	
3	-5.7	-15.6		-5.2	-15.1	
4	-3.6	-13.5		-7.4	-17.3	
5				16.0	6.1	
6				-3.0	-12.9	
1E	22.5	12.6		-8.3	-18.2	
2E	-24.5	-34.4		-24.5	-34.4	
3E	-10.3	-20.2		-9.6	-19.6	
4E	-7.7	-17.6		-8.3	-18.2	
5E				21.8	11.8	
6E				-6.9	-16.8	

Parapet

Windward parapet =	0.0 psf	(GCpn = +1.5)
Leeward parapet =	0.0 psf	(GCpn = -1.0)

Windward roof
overhangs = 19.3 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

Interior Zone:	Wall	20.1 psf
	Roof	-8.4 psf **
End Zone:	Wall	30.2 psf
	Roof	-14.2 psf **

Longitudinal direction (parallel to L)

Interior Zone:	Wall	19.0 psf
End Zone:	Wall	28.6 psf

** NOTE: Total horiz force shall not be less than that determined by neglecting roof forces (except for MWFRS moment frames).

The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.
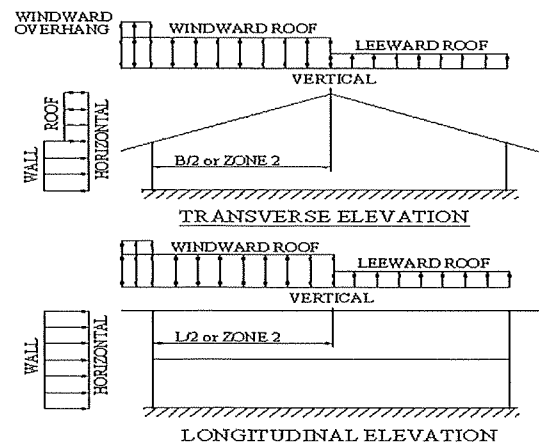
Crockett Engineering
1000 W Nifong Blvd
Columbia Missouri 65203
573-447-0292

JOB TITLE Club Wash

JOB NO.   SHEET NO.
CALCULATED BY   , DATE
CHECKED BY   DATE

Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) =	0.85	h =	15.0 ft
Base pressure (qh) =	**24.4 psf**	a =	3.7 ft
Minimum parapet ht =	3.0 ft	GCpi =	+/-0.18
Roof Angle (θ) =	1.2 deg		
Type of roof =	Monoslope		

Roof

Area	GCp +/- GCpi				Surface Pressure (psf)				User input	
	10 sf	50 sf	100 sf	500 sf	10 sf	50 sf	100 sf	500 sf	50 sf	100 sf
Negative Zone 1	-1.18	-1.11	-1.08	-1.08	-28.8	-27.1	-26.4	-26.4	-27.1	-26.4
Negative Zone 2	-1.98	-1.49	-1.28	-1.28	-48.4	-36.4	-31.3	-31.3	-36.4	-31.3
Negative Zone 3	-1.98	-1.49	-1.28	-1.28	-48.4	-36.4	-31.3	-31.3	-36.4	-31.3
Positive Zone 1	0.48	0.41	0.38	0.38	16.0	16.0	16.0	16.0	16.0	16.0
Positive Zones 2 & 3	1.08	0.97	0.92	0.81	26.4	23.7	22.5	19.8	23.7	22.5
Overhang Zone 1&2	-1.7	-1.63	-1.6	-1.1	-41.5	-39.8	-39.1	-26.9	-39.8	-39.1
Overhang Zone 3	-1.7	-1.63	-1.6	-1.1	-41.5	-39.8	-39.1	-26.9	-39.8	-39.1

Negative zone 3 = zone 2, since parapet >= 3ft.
Overhang pressures in the table above assume an internal pressure coefficient (Gcpi) of 0.0
Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 4.4 psf)

Parapet

qp = 25.4 psf

Solid Parapet Pressure	Surface Pressure (psf)						User input
	10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	50 sf
CASE A: Zone 2 :	68.5	62.0	53.3	46.7	45.5	43.9	53.3
Zone 3 :	68.5	62.0	53.3	46.7	45.5	43.9	53.3
CASE B: Edge zones 2 :	-48.0	-45.5	-42.3	-39.9	-37.5	-34.3	-42.3
Corner zones 3 :	-54.8	-51.2	-46.4	-42.7	-39.1	-34.3	-46.4

Walls

Area	GCp +/- GCpi				Surface Pressure (psf)				User input	
	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-28.6	-24.7	-23.5	-22.0	-28.6	-24.7
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-35.2	-27.4	-25.1	-22.0	-35.2	-27.4
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	26.4	22.5	21.3	19.8	26.4	22.5

Note: GCp reduced by 10% due to roof angle <= 10 deg.

JOB TITLE	Club Wash
JOB NO.	SHEET NO.
CALCULATED BY	DATE
CHECKED BY	DATE

Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

K_h (case 1) =	0.96	h =	26.5 ft
Base pressure (q_h) =	**27.5 psf**	a =	3.7 ft
Minimum parapet ht =	0.0 ft	GCpi =	+/-0.18
Roof Angle (θ) =	7.1 deg		
Type of roof =	Monoslope		

towers

Roof

Area	GCp +/- GCpi				Surface Pressure (psf)				User input	
	10 sf	20 sf	50 sf	100 sf	10 sf	20 sf	50 sf	100 sf	50 sf	100 sf
Negative Zone 1	-1.28	-1.28	-1.28	-1.28	-35.2	-35.2	-35.2	-35.2	-35.2	-35.2
Negative Zone 2	-1.48	-1.45	-1.41	-1.38	-40.8	-39.9	-38.8	-38.0	-38.8	-38.0
Negative Zone 3	-1.98	-1.8	-1.56	-1.38	-54.5	-49.6	-43.0	-38.0	-43.0	-38.0
Positive All Zones	0.48	0.45	0.41	0.38	16.0	16.0	16.0	16.0	16.0	16.0
Negative Zone 2'	-1.78	-1.75	-1.71	-1.68	-49.0	-48.2	-47.1	-46.3	-47.1	-46.3
Negative Zone 3'	-2.78	-2.48	-2.08	-1.78	-76.6	-68.3	-57.3	-49.0	-57.3	-49.0

Parapet

q_p = 0.0 psf

Solid Parapet Pressure		Surface Pressure (psf)						User input
		10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	50 sf
CASE A:	Zone 2 :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 2' :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 3' :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CASE B: Edge zones 2 :		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Corner zones 3 :		0.0	0.0	0.0	0.0	0.0	0.0	0.0

Walls

| Area	GCp +/- GCpi				Surface Pressure (psf)				User input	
	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-32.2	-27.8	-26.5	-24.8	-32.2	-27.8
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-39.7	-30.9	-28.3	-24.8	-39.7	-30.9
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	29.7	25.4	24.0	22.3	29.7	25.4

Note: GCp reduced by 10% due to roof angle <= 10 deg.

Lateral Design

Wind Design/Lateral

Main Roof
Int = 16.9
Ext = 25.4
Parapet = 63.5

R Roof Int = 360 plf
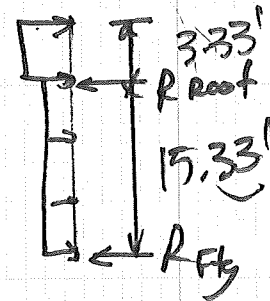
Tower Roof
Int = 20.1 wall   -8.4 Roof
Ext = 30.2 wall   -14.2 Roof
Roof =

R Roof Int = 485.2 plf
R Roof Ext = 7026 plf
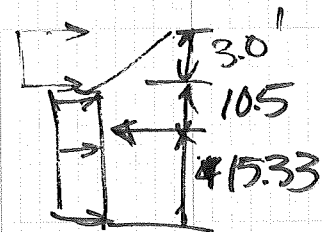
$R_2 = 31.09$ M = 1005.5 K·ft

Shear = 31.09/25.33' = 1228 plf

Chords = 1005.5/36' = 27.93 K

L3+3+1/4 @ 4'0" unbraced
= 30.7 K → yes OK

Conservative Approach

![ASCE - American Society of Civil Engineers logo]
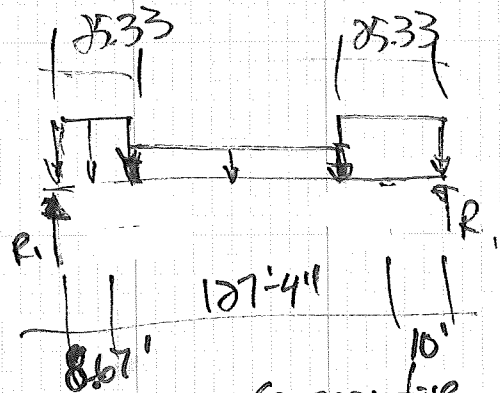
Seismic

Site Soil Class: C - Very Dense Soil and Soft Rock

Results:

S_S :	0.1	S_{D1} :	0.068
S_1 :	0.068	T_L :	12
F_a :	1.3	PGA :	0.047
F_v :	1.5	PGA_M :	0.062
S_{MS} :	0.13	F_{PGA} :	1.3
S_{M1} :	0.102	I_e :	1
S_{DS} :	0.087	C_v :	0.7

Seismic Design Category: B

MCE_R Response Spectrum

S_a(g) vs T(s)
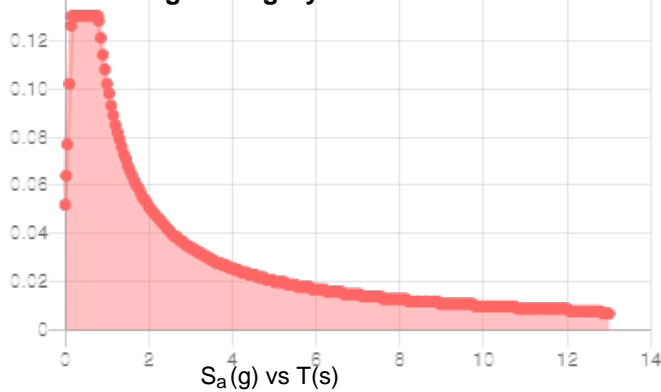
Design Response Spectrum

S_a(g) vs T(s)
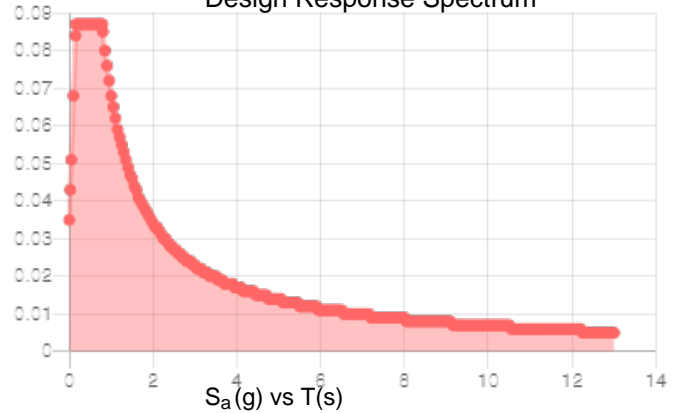
MCE_R Vertical Response Spectrum

S_a(g) vs T(s)
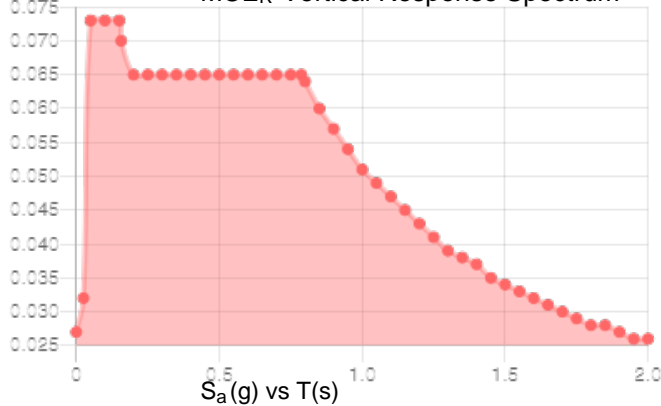
Design Vertical Response Spectrum

S_a(g) vs T(s)
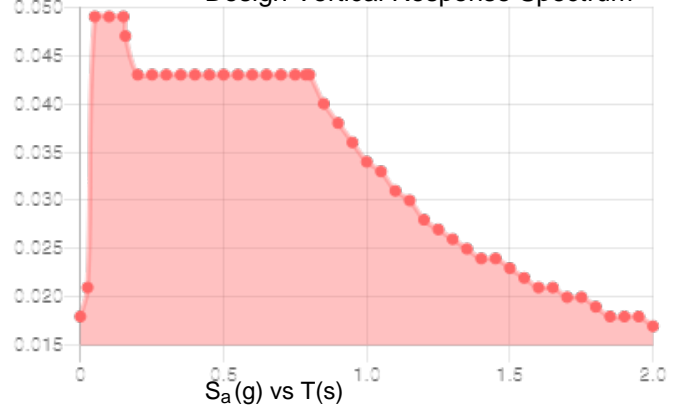
Data Accessed: Wed Aug 20 2025

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

Seismic Lateral

$$C_s = \frac{0.087}{3.5} = 0.025 \ W$$

$$Roof = 37.333' \ (15) \ (0.025) = 14.0 \ plf$$

$$Walls = 15' \ (68 ps) \ (3 \ walls) (0.025) = 77.0 \ plf$$

$$\underline{91 \ plf} \ << \ Wind$$

Thus Wind
Control
Lateral
in Long Dir

General Beam Analysis

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: Diaphragm load

General Beam Properties

Elastic Modulus	29,000.0 ksi				
Span #1	Span Length =	146.0 ft	Area =	10.0 in^2	Moment of Inertia = 100.0 in^4

Span = 146.0 ft

Applied Loads

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

Uniform Load : W = 0.360 k/ft, Tributary Width = 1.0 ft

Uniform Load : W = 0.1252 k/ft, Extent = 7.50 -->> 25.333 ft, Tributary Width = 1.0 ft

Uniform Load : W = 0.1252 k/ft, Extent = 120.670 -->> 138.50 ft, Tributary Width = 1.0 ft

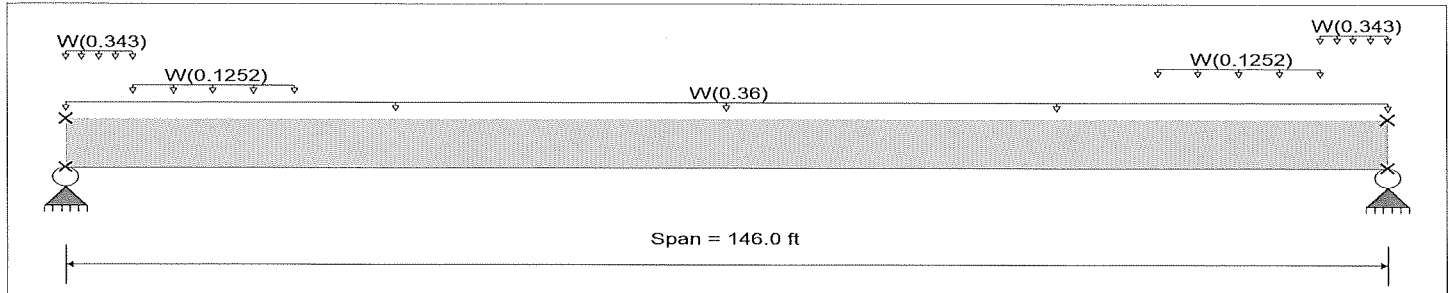Uniform Load : W = 0.3430 k/ft, Extent = 0.0 -->> 7.50 ft, Tributary Width = 1.0 ft

Uniform Load : W = 0.3430 k/ft, Extent = 138.50 -->> 146.0 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Maximum Bending =	1,005.515 k-ft	Maximum Shear =	31.085 k
Load Combination	W Only	Load Combination	W Only
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	73.000 ft	Location of maximum on span	0.000 ft

Maximum Deflection		
Max Downward Transient Deflection	0.000 in	0
Max Upward Transient Deflection	0.000 in	0
Max Downward Total Deflection	1352.130 in	1
Max Upward Total Deflection	16.216 in	108

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)						Shear Values (k)	
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega Cb	Rm	Va Max	Vnx Vnx/Omega
Overall MAXimum Envelope											
Dsgn. L = 146.00 ft	1			1,005.52		1,005.52				31.09	
Dsgn. L = 146.00 ft	1									-0.00	
W Only											
Dsgn. L = 146.00 ft	1			1,005.52		1,005.52				31.09	
+0.750W											
Dsgn. L = 146.00 ft	1			754.14		754.14				23.31	

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	###.####	73.730		0.0000	0.000

Vertical Reactions

Support notation : Far left is #·   Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	31.085	31.085
Overall MINimum		
W Only	31.085	31.085
+0.750W	23.314	23.314

36/7 DIAPHRAGM SHEAR STRENGTH:
1.5" Decking (WR or NR), 3-Span Continuous — Revised per 3rd Edition SDI Diaphragm Design Manual

PATTERN: A   1

DECK DATA:

Gage (18,20,22)	20	I [in^4/ft] 0.198
Type (WR, NR)	WR	d/s 0.6857143

Wind or Earthquake (W,E): W
MWFRS Wind Uplift (psf): 17.1

FASTENERS:

	SIZE	BRAND/TYPE
Support	12	Tek
Sidelap	#12	Tek

DECK GA. (IN.)	JOIST SPCG	QS [k]	QF [k]	ALPHAS (QS/QF)	SUM XSQ	NO. SIDELAPS	B	LAMBDA	SUM Xe / SUM Xp	N	2.2-2 SU1	2.2-4 SU2	2.2-5 SU3	2.3-3 SC	LRFD DESIGN STRENGTH S [plf]	CONN. SPACING (in.)
0.0358	4.000	0.870	1.233	0.705	6.222	0	6.222	0.868	2.000 / 2.000	2.000	0.822	0.612	0.619	4.516	429	12
0.0358	4.000	0.870	1.233	0.705	6.222	1	8.339	0.868	2.000 / 2.000	2.000	1.130	0.830	0.809	4.516	567	12
0.0358	4.000	0.870	1.233	0.705	6.222	2	10.455	0.868	2.000 / 2.000	2.000	1.439	1.047	0.985	4.516	689	12
0.0358	4.000	0.870	1.233	0.705	6.222	3	12.572	0.868	2.000 / 2.000	2.000	1.747	1.265	1.144	4.516	801	12
0.0358	4.000	0.870	1.233	0.705	6.222	4	14.688	0.868	2.000 / 2.000	2.000	2.055	1.482	1.287	4.516	901	9
0.0358	4.000	0.870	1.233	0.705	6.222	5	16.805	0.868	2.000 / 2.000	2.000	2.363	1.700	1.415	4.516	990	8
0.0358	4.000	0.870	1.233	0.705	6.222	6	18.921	0.868	2.000 / 2.000	2.000	2.672	1.917	1.527	4.516	1,069	6
0.0358	4.000	0.870	1.233	0.705	6.222	7	21.038	0.868	2.000 / 2.000	2.000	2.980	2.135	1.626	4.516	1,138	6
0.0358	4.000	0.870	1.233	0.705	6.222	8	23.154	0.868	2.000 / 2.000	2.000	3.288	2.352	1.712	4.516	1,199	5
0.0358	4.000	0.870	1.233	0.705	6.222	9	25.271	0.868	2.000 / 2.000	2.000	3.597	2.570	1.788	4.516	1,252	4
0.0358	4.000	0.870	1.233	0.705	6.222	10	27.387	0.868	2.000 / 2.000	2.000	3.905	2.787	1.855	4.516	1,298	4
0.0358	4.000	0.870	1.233	0.705	6.222	11	29.504	0.868	2.000 / 2.000	2.000	4.213	3.005	1.913	4.516	1,339	4

NOTES:

Loads to be calculated per ASCE7-02. 1.6W, 1.0E for LRFD 1.0W, 0.7E for ASD
QS == strength of sidelap fasteners (see Section 4 of SDI Manual) as function of deck thickness
QF == strength of support fasteners (see Section 4 of SDI Manual) as function of deck thickness
SUM XSQ == depends on fastener layout (i.e. 36/7) and # of interior purlins "np" (i.e. # of spans continuous)
B == depends upon total number of fasteners over deck length (therefore, depends on number of spans continuous)
LAMBDA == depends on panel depth (i.e. 1.5"), purlin/joist spacing, and deck thickness
SUM X == depends on fastener layout.
A == 1 for single edge fasteners as in 36/7 pattern

SU1 == limiting strength of edge fasteners (EQ. 2.2-2)
SU2 == limiting strength of interior panels (EQ. 2.2-4)
SU3 == limiting strength of corner fasteners (EQ. 2.2-5)
SC == stability / buckling check (EQ. 2.3-3)

36/7 Pattern with 3-Span Continuous:

deck width	36
flute width	6

Buildex Tek Screws (size 12 or 14):
QF [K] 1.233

	xp	xe
fastener		
1	18	18
2	12	12
3	6	6
4	0	0
5	-6	-6
6	-12	-12
7	-18	-18

NF >>>>>

Np	2	
SUM X	72	72
SUM SQRD	1008	1008
SUM XSQ	6.2222222	

wind uplift (plf)	75.24
screw tension capacity (lb)	482
Beta	2
screw tension capacity (plf)	964
LRFD (Tu/phiTn)	0.12488
LRFD reduction factor	1

36/7 DIAPHRAGM SHEAR STRENGTH:
1.5" Decking (WR or NR), 3-Span Continuous

Revised per 3rd Edition SDI Diaphragm Design Manual

PATTERN: A

DECK DATA:

Gage (18,20,2	22
Type (WR, NR	WR

I [in^4/ft]	0.152
d/s	0.6857143

Wind or Earthquake (W,E)	W
MWFRS Wind Uplift (psf)	17.1

FASTENERS:

	SIZE	BRAND/TYPE
Support	12	Tek
Sidelap	#12	TEK

Handwritten note: "@ Deck Transition"

Handwritten note: "0.53/36 = SUB p/t" with "36 = SUB p/t"

DECK GA. (IN.)	JOIST SPCG	QS [k]	QF [k]	ALPHAS (QS/QF)	SUM XSQ	NO. SIDELAPS	B	LAMBDA	SUM Xe / SUM Xp	N	2.2-2 SU1	2.2-4 SU2	2.2-5 SU3	2.3-3 SC	LRFD DESIGN STRENGTH S [plf]	CONN. SPACING (in.)
0.0295	4.000	0.717	1.014	0.707	6.222	0	6.222	0.854	2.000 2.000	2.000	0.676	0.501	0.509	3.203	351	12
0.0295	4.000	0.717	1.014	0.707	6.222	1	8.343	0.854	2.000 2.000	2.000	0.930	0.680	0.666	3.203	466	12
0.0295	4.000	0.717	1.014	0.707	6.222	2	10.463	0.854	2.000 2.000	2.000	1.183	0.860	0.811	3.203	567	12
0.0295	4.000	0.717	1.014	0.707	6.222	3	12.584	0.854	2.000 2.000	2.000	1.437	1.039	0.942	3.203	659	12
0.0295	4.000	0.717	1.014	0.707	6.222	4	14.704	0.854	2.000 2.000	2.000	1.690	1.218	1.060	3.203	742	9
0.0295	4.000	0.717	1.014	0.707	6.222	5	16.825	0.854	2.000 2.000	2.000	1.944	1.397	1.164	3.203	815	8
0.0295	4.000	0.717	1.014	0.707	6.222	6	18.945	0.854	2.000 2.000	2.000	2.197	1.577	1.257	3.203	880	6
0.0295	4.000	0.717	1.014	0.707	6.222	7	21.066	0.854	2.000 2.000	2.000	2.451	1.756	1.338	3.203	937	6
0.0295	4.000	0.717	1.014	0.707	6.222	8	23.186	0.854	2.000 2.000	2.000	2.704	1.935	1.409	3.203	987	5
0.0295	4.000	0.717	1.014	0.707	6.222	9	25.307	0.854	2.000 2.000	2.000	2.958	2.114	1.472	3.203	1,030	4
0.0295	4.000	0.717	1.014	0.707	6.222	10	27.428	0.854	2.000 2.000	2.000	3.211	2.293	1.526	3.203	1,069	4
0.0295	4.000	0.717	1.014	0.707	6.222	11	29.548	0.854	2.000 2.000	2.000	3.465	2.473	1.574	3.203	1,102	4

NOTES:
Loads to be calculated per ASCE7-02. 1.6W, 1.0E for LRFD. 1.0W, 0.7E for ASD

QS == strength of sidelap fasteners (see Section 4 of SDI Manual) as function of deck thickness
QF == strength of support fasteners (see Section 4 of SDI Manual) as function of deck thickness
SUM XSQ == depends on fastener layout (i.e. 36/7) and # of interior purlins "np" (i.e. # of spans continuous)
B == depends upon total number of fasteners over deck length (therefore, depends on number of spans continuous)
LAMBDA == depends on panel depth (i.e. 1.5"), purlin/joist spacing, and deck thickness
SUM X == depends on fastener layout
A == 1 for single edge fasteners as in 36/7 pattern

SU1 == limiting strength of edge fasteners (EQ. 2.2-2)
SU2 == limiting strength of interior panels (EQ. 2.2-4)
SU3 == limiting strength of corner fasteners (EQ. 2.2-5)
SC == stability / buckling check (EQ. 2.3-3)

36/7 Pattern with 3 Span Continuous:

deck width	36	
flute width	6	

			wind uplift (plf)	75.24

Buildex Tek Screws (size 12 or 14):

fastener	xp	xe
1	18	18
2	12	12
3	6	6
4	0	0
5	-6	-6
6	-12	-12
7	-18	-18

QF [k]	1.016		screw tension capacity (lb)	397

			Beta	2
			screw tension capacity (plf)	794

Np	2		LRFD (Tu/phiTn)	0.151617
SUM X	72	72	LRFD reduction factor	0.998097
SUM SQRD	1008	1008		
SUM XSQ	6.2222222			

Masonry Shear Wall	Project File: Tiger Moline.ec6
LIC# : KW-06015690, Build:20.22.3.31 CROCKETT ENGINEERING CONSULTANTS	(c) ENERCALC INC 1983-2022

DESCRIPTION: 4.67' pilaster

Code References

LOAD COMBINATIONS USED: IBC 2018

General Information

Wall Material MASONRY		f'm	2.0 ksi	Block Class	
Total Wall Height	22.0 ft	Fy - Rebar	60.0 ksi	Concrete Density	150.0 pcf
Base Wall Length	4.670 ft	Fy - HJR	70.0 ksi	Min. Bending As %	0.00180
R: Resp. Mod Factor		Em	3,120.0 ksi		
Ie: Seismic Import. Factor	1.0	Phi - Shear	0.80	Phi : Axial & Flexure	0.90

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	4.670 ft
Effective Length 'd'	48.040 in
Nominal Block Thickness	8 in
Solid Grout?	Partial Groute

Reinforcing in Field of Wall

Vertical Bar Size #	5
Vertical Bar Spacing	32 in
Horiz. joint reinf. area (HJR)	0.55 in
HJR Spacing	24 in
Bond beam reinf. area	in
Spacing of bond beams	48 in

In each chord cell:

Vertical rebar size #	7
# Chord Cells @ Each End	2.0

Masonry Shear Wall

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: 4.67' pilaster

Wall Sketch

22'-0" ft

Datum

4'-8-1/16" ft
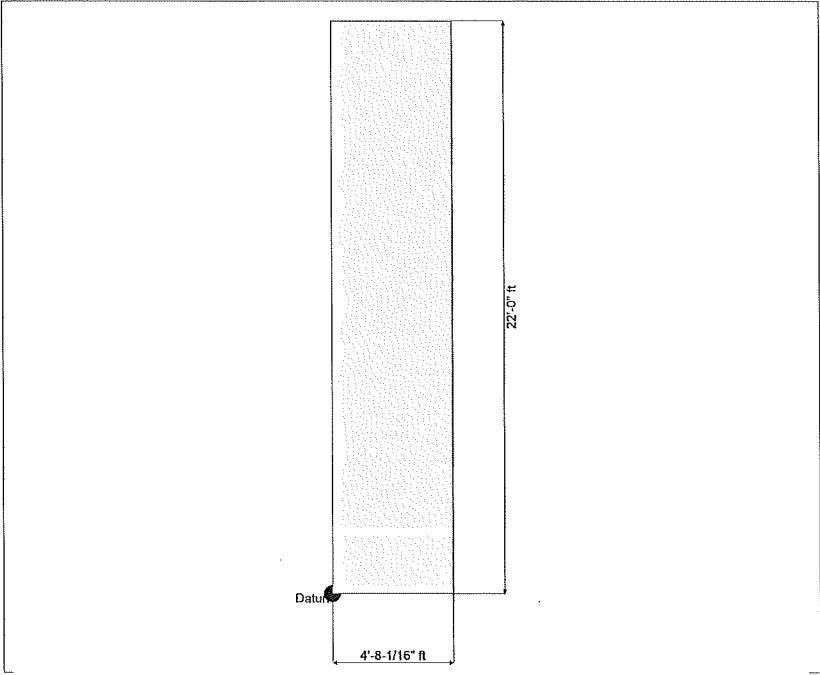
Applied Concentrated Lateral Loads

		Load Magnitude (kips)					
Load "Y" Location (ft)		Dead Load	Roof Live Load	Floor Live Load	Wind Load	Seismic Load	Earth Load
15.330		0.0	0.0	0.0	15.60	10.0	0.0

SHEAR ANALYSIS

	Bottom Level
Special Boundary Elements Req'd?	Not Req'd
Vu : Story Shear	15.60 k
for Load Combination	+1.20D+W
Controlling Mu/(Vud)	1.00
Vn Masonry	38.877 k
Vn Steel	35.959 k
Vn Masonry + Vn Steel	74.836 k
Vn Max	48.433 k
Phi Vn	38.746 k
Ratio: Vu/PhiVn (controlling)	0.4026
Vertical As >= Av/3	OK
Vertical Bar Spacing <= 96"	OK

Masonry Shear Wall

DESCRIPTION: 4.67' pilaster

AXIAL ANALYSIS

	Bottom Level
H / d Ratio	5.50
Pu	10.210 k
for Load Combination	+1.40D
Phi Pn	+1.40D k
Ratio: Pu/PhiPn (controlling)	**0.01950**

BENDING ANALYSIS

	Bottom Level
"a" : Flexural compression	5.90 in
Length of defined chord zone is >= the "a" dimension of the masonry (the compression zone)	OK
"d" : Eff depth to tension reinf	48.040
As-flex < As-max ?	200 <= 4.328
Mu	239.148 k
for Load Combination	+1.20D+W
Phi Mn	243.482 k
Ratio: Mu/PhiMn (controlling)	**0.9822**

Force Summary

Load Combination Wall Level	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
+1.40D							
Wall Level : 1			10.210				
+1.20D							
Wall Level : 1			8.752				
+1.20D+0.50W							
Wall Level : 1	7.800	119.574	8.752	13.663	0.142	25.615	25.615
+1.20D+W							
Wall Level : 1	15.600	239.148	8.752	27.326	0.071	55.483	55.483
+1.20D+E							
Wall Level : 1	10.000	153.300	8.752	17.517	0.111	34.039	34.039
+0.90D+W							
Wall Level : 1	15.600	239.148	6.564	36.435	0.071	55.483	55.483
+0.90D+E							
Wall Level : 1	10.000	153.300	6.564	23.355	0.111	34.039	34.039

Masonry Shear Wall

DESCRIPTION: 4.67' pilaster

Footing Information

Footing Dimensions

Dist. Left	8.0 ft	f'c	3.0 ksi	Rebar Cover	3.0 in
Wall Length	4.670 ft	Fy	60.0 ksi	Footing Thickness	42.0 in
Dist. Right	1.0 ft			Width	2.0 ft
Total Ftg Length	13.670 ft				

Max Factored Soil Pressures

@ Left Side of Footing   535.08 psf
    .... governing load comb +1.40D

@ Right Side of Footing   2,391,055 psf
    .... governing load comb +1.20D+0.50W

Max UNfactored Soil Pressures

@ Left Side of Footing   382.203 psf
    .... governing load comb D Only

@ Right Side of Footing   6,617.82 psf
    .... governing load comb D+0.5250E

Footing One-Way Shear Check...		Overturning Stability...	@ Left End of Ftg	@ Right End of Ftg
v_u @ Left End of Footing	8.770 psi	Overturning Moment	176.249 k-ft	176.249 k-ft
v_u @ Right End of Footing	0.0 psi	Resisting Moment	92.315 k-ft	61.684 k-ft
v_n * phi : Allowable	93.113 psi	Stability Ratio	0.5238 : 1	0.350 : 1
		.... governing load comb	+0.60D+0.60W	+0.60D+0.60W

Footing Bending Design...	@ Left End	@ Right End
Mu	48.571 k-ft	2.046 k-ft
Ru	17.741 psi	0.7474 psi
As % Req'd	0.00180 in^2	0.00180 in^2
As Req'd in Footing Width	1.814 in^2	1.814 in^2

Gravity Design

Snow

Results:

    Ground Snow Load, p_g :   20 lb/ft^2

    Mapped Elevation: 1038.8 ft

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

Date Accessed: Wed Aug 20 2025

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

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

Rain

Results:

    15-minute Precipitation Intensity: 7.49 in./h

    60-minute Precipitation Intensity: 3.51 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14 (https://www.nws.noaa.gov/oh/hdsc/)

Date Accessed: Wed Aug 20 2025
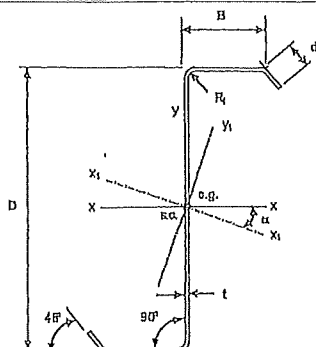
8 ½" Zee Section

$D = 8 \frac{1}{2}"$
$B = 2 \frac{1}{2}"$
$R_i = 0.3125"$ $j = 0.0"$
$F_y = 60$ ksi $m = 0.0"$
$F_u = 70$ ksi $X_o = 0.0"$

Section Properties have been derived in accordance with AISI Standard "North American Specification for the Design of Cold-Formed Steel Structural Members"
See DP 20.1 for definition of terms.

Dimensions					Full Properties						α_1 deg.
					Axis x-x			Axis y-y			
Thick in.	Blank in.	Area in²	Lip, d in.	Weight plf	I_x in⁴	S_x in³	r_x in.	I_y in⁴	S_y in³	r_y in.	
0.060	14.500	0.870	0.766	2.99	9.30	2.19	3.266	1.24	0.42	1.195	-15.76
0.068	14.500	0.986	0.782	3.38	10.52	2.48	3.263	1.42	0.47	1.198	-15.83
0.073	14.750	1.077	0.916	3.69	11.52	2.71	3.267	1.69	0.55	1.253	-16.60
0.079	14.750	1.165	0.928	3.99	12.45	2.93	3.264	1.84	0.60	1.255	-16.65
0.088	14.750	1.298	0.945	4.44	13.84	3.26	3.260	2.06	0.67	1.259	-16.72
0.098	14.750	1.446	0.964	4.94	15.37	3.62	3.256	2.31	0.75	1.263	-16.81
0.113	14.750	1.667	0.993	5.70	17.67	4.16	3.250	2.69	0.87	1.269	-16.93

	Full Properties					Distortional Buckling				
						Critical Stress			Critical Unbraced Length	
Thick in.	J in⁴	C_w in⁶	r_o in.	I_{yc} in⁴	I_{xy} in⁴	$F_{db(0.0)}$ ksi	$F_{db(2.4)}$ ksi	$F_{da(0.0)}$ ksi	$L_{crd\text{-}bend}$ in.	$L_{crd\text{-}axial}$ in.
0.060	0.00105	15.749	3.478	0.622	2.470	31.22	180.14	14.36	21.19	23.43
0.068	0.00152	17.931	3.476	0.709	2.806	36.71	154.57	16.82	20.09	22.21
0.073	0.00192	21.538	3.499	0.847	3.214	42.66	155.67	20.52	21.48	23.75
0.079	0.00243	23.382	3.497	0.920	3.484	47.19	144.56	22.65	20.76	22.95
0.088	0.00336	26.172	3.495	1.031	3.889	54.26	133.81	25.96	19.83	21.92
0.098	0.00464	29.299	3.492	1.156	4.342	62.52	127.60	29.82	18.96	20.95
0.113	0.00712	34.050	3.489	1.347	5.024	75.68	125.69	35.95	17.89	19.76

	Effective Properties Evaluated at f = 60 ksi							Deflection Properties at f = 36 ksi		Minor Principal Axis Properties	
	Axis x-x			Axis y-y							
Thick in.	I_{ex} in⁴	S_{ex} in³	M_{nxo} k-in.	I_{ey} in⁴	S_{ey} in³	M_{nyo} k-in.	$A_{e\,col}$ in²	I_{dx} in⁴	I_{dy} in⁴	I_2 in⁴	r_2 in.
0.060	8.26	1.803	108.18	1.00	0.322	19.29	0.447	9.07	1.24	0.547	0.792
0.068	9.66	2.164	129.84	1.21	0.396	23.75	0.538	10.48	1.42	0.623	0.794
0.073	10.92	2.486	149.16	1.67	0.543	32.59	0.639	11.52	1.69	0.735	0.825
0.079	12.07	2.786	167.16	1.84	0.597	35.81	0.741	12.45	1.84	0.798	0.827
0.088	13.77	3.251	195.08	2.06	0.668	40.07	0.902	13.84	2.06	0.894	0.829
0.098	15.37	3.618	217.05	2.31	0.747	44.81	1.055	15.37	2.31	1.001	0.831
0.113	17.67	4.157	249.40	2.69	0.867	51.99	1.273	17.67	2.69	1.164	0.834

Steel Beam

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: typ two span Z purlin

CODE REFERENCES

LOAD COMBINATIONS USED: IBC 2018

Material Properties

Analysis Method 'Allowable Strength Design	Fy : Steel Yield :	55.0 ksi
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis : Major Axis Bending		

Vertical Leg Up

D(0.060) Lr(0.080) S(0.10)

8.5Zx2.5-11GA   8.5Zx2.5-11GA

Span = 12.0 ft   Span = 24.0 ft

Applied Loads

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

Beam self weight NOT internally calculated and added
Loads on all spans...
    Uniform Load on ALL spans : D = 0.060, Lr = 0.080, S = 0.10 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.757 : 1	Maximum Shear Stress Ratio =	0.125 : 1
Section used for this span	8.5Zx2.5-11GA	Section used for this span	8.5Zx2.5-11GA
Ma : Applied	8.640 k-ft	Va : Applied	2.280 k
Mn / Omega : Allowable	11.417 k-ft	Vn/Omega : Allowable	18.287 k
Load Combination	+D+S	Load Combination	+D+S
		Location of maximum on span	12.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.814 in	Ratio =	353	>=240.	Span: 2 : S Only
Max Upward Transient Deflection	-0.083 in	Ratio =	1,734	>=240.	Span: 2 : S Only
Max Downward Total Deflection	1.303 in	Ratio =	221	>=180	Span: 2 : +D+S
Max Upward Total Deflection	-0.133 in	Ratio =	1084	>=180	Span: 2 : +D+S

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Max Stress Ratios M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 12.00 ft	1	0.284	0.047	0.07	-3.24	3.24	19.07	11.42	1.00	1.00	0.85	30.54	18.29
Dsgn. L = 24.00 ft	2	0.284	0.047	2.85	-3.24	3.24	19.07	11.42	1.00	1.00	0.85	30.54	18.29
+D+Lr													
Dsgn. L = 12.00 ft	1	0.662	0.109	0.16	-7.56	7.56	19.07	11.42	1.00	1.00	1.99	30.54	18.29
Dsgn. L = 24.00 ft	2	0.662	0.109	6.65	-7.56	7.56	19.07	11.42	1.00	1.00	1.99	30.54	18.29
+D+S													
Dsgn. L = 12.00 ft	1	0.757	0.125	0.18	-8.64	8.64	19.07	11.42	1.00	1.00	2.28	30.54	18.29
Dsgn. L = 24.00 ft	2	0.757	0.125	7.60	-8.64	8.64	19.07	11.42	1.00	1.00	2.28	30.54	18.29
+D+0.750Lr													
Dsgn. L = 12.00 ft	1	0.568	0.094	0.13	-6.48	6.48	19.07	11.42	1.00	1.00	1.71	30.54	18.29
Dsgn. L = 24.00 ft	2	0.568	0.094	5.70	-6.48	6.48	19.07	11.42	1.00	1.00	1.71	30.54	18.29
+D+0.750S													
Dsgn. L = 12.00 ft	1	0.639	0.105	0.15	-7.29	7.29	19.07	11.42	1.00	1.00	1.92	30.54	18.29
Dsgn. L = 24.00 ft	2	0.639	0.105	6.42	-7.29	7.29	19.07	11.42	1.00	1.00	1.92	30.54	18.29
+0.60D													
Dsgn. L = 12.00 ft	1	0.170	0.028	0.04	-1.94	1.94	19.07	11.42	1.00	1.00	0.51	30.54	18.29
Dsgn. L = 24.00 ft	2	0.170	0.028	1.71	-1.94	1.94	19.07	11.42	1.00	1.00	0.51	30.54	18.29

Steel Beam

DESCRIPTION: typ two span Z purlin

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+S	-0.1329	7.968
+D+S	2	1.3029	13.248		0.0000	7.968

Vertical Reactions

Support notation : Far left is #   Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.240	3.960	1.560
Overall MINimum	0.054	0.891	0.351
D Only	0.090	1.485	0.585
+D+Lr	0.210	3.465	1.365
+D+S	0.240	3.960	1.560
+D+0.750Lr	0.180	2.970	1.170
+D+0.750S	0.203	3.341	1.316
+0.60D	0.054	0.891	0.351
Lr Only	0.120	1.980	0.780
S Only	0.150	2.475	0.975

Crockett Engineering
1000 W Nifong Blvd
Columbia Missouri 65203
573-447-0292

JOB TITLE	Club Wash
JOB NO.	SHEET NO.
CALCULATED BY	DATE
CHECKED BY	DATE

Snow Loads :

<div align="right">Nominal Snow Forces</div>

Roof slope	=	1.2 deg
Horiz. eave to ridge dist (W) =		37.3 ft
Roof length parallel to ridge (L) =		146.0 ft

Type of Roof		Monoslope
Ground Snow Load	P_g =	20.0 psf
Risk Category	=	II
Importance Factor	I =	1.0
Thermal Factor	C_t =	1.00
Exposure Factor	C_e =	0.9

$P_f = 0.7*C_e*C_t*I*P_g$	=	12.6 psf
Unobstructed Slippery Surface		no

Sloped-roof Factor	C_s =	1.00
Balanced Snow Load	=	**12.6 psf**

Near ground level surface balanced snow load = **20.0 psf**

Rain on Snow Surcharge Angle		0.75 deg
Code Maximum Rain Surcharge		5.0 psf
Rain on Snow Surcharge	=	0.0 psf
Ps plus rain surcharge	=	12.6 psf
Minimum Snow Load	P_m =	20.0 psf

Uniform Roof Design Snow Load	=	**20.0 psf**

NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs..

Windward Snow Drifts 1 - Against walls, parapets, etc

Up or downwind fetch	l_u =	128.0 ft
Projection height	h =	3.0 ft
Projection width/length	l_p =	16.0 ft
Snow density	g =	16.6 pcf
Balanced snow height	h_b =	0.76 ft
	h_d =	2.68 ft
	h_c =	2.24 ft
h_c/h_b >0.2 = 3.0	**Therefore, design for drift**	
Drift height (h_c)	=	2.24 ft
Drift width	w =	12.81 ft
Surcharge load:	$p_d = \gamma*h_d$ =	**37.2 psf**
Balanced Snow load:	=	12.6 psf
		49.8 psf

Windward Snow Drifts 2 - Against walls, parapets, etc

Up or downwind fetch	l_u =	121.0 ft
Projection height	h =	10.0 ft
Projection width/length	l_p =	25.0 ft
Snow density	g =	16.6 pcf
Balanced snow height	h_b =	0.76 ft
	h_d =	2.61 ft
	h_c =	9.24 ft
h_c/h_b >0.2 = 12.2	**Therefore, design for drift**	
Drift height (h_d)	=	2.61 ft
Drift width	w =	10.43 ft
Surcharge load:	$p_d = \gamma*h_d$ =	**43.3 psf**
Balanced Snow load:	=	12.6 psf
		55.9 psf
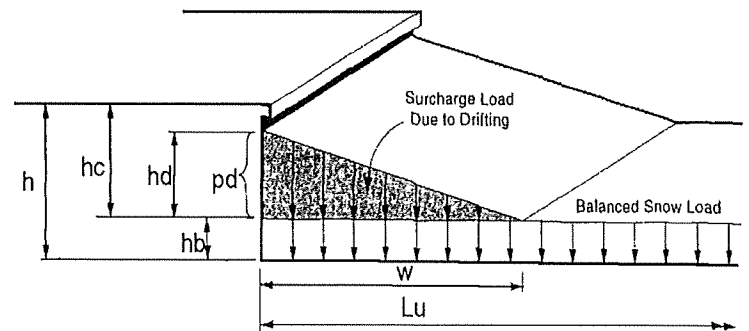
Drift @ tower

$$51.7/11.55 = 4.50x$$

First Joist

$$10.22 (4.5) = 45.98 \text{ Drift}$$
$$+ 18.9 \text{ Bal.}$$
$$\overline{64.88 \text{ psf}}$$
$$+ 15 \text{ psf DL}$$
$$\overline{80 \text{ psf}}$$
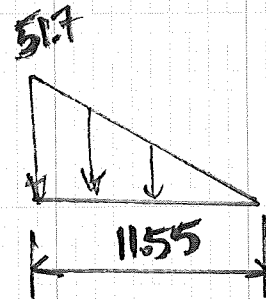
Second Joist

$$8.22 (4.5) = 37 \text{ psf} + 18.9 = 55.9 \text{ psf}$$

Third

$$6.22 (4.5) = 28 \text{ psf} + 18.9 = 46.9 \text{ psf}$$

Fourth

$$2.22 (4.5) = 9.9 + 18.9 = 28.8 \text{ psf}$$

51.7

1155

Steel Beam

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31 CROCKETT ENGINEERING CONSULTANTS (c) ENERCALC INC 1983-2022

DESCRIPTION: two span Z purlin (first at tower)

CODE REFERENCES

LOAD COMBINATIONS USED: IBC 2018

Material Properties

Analysis Method Allowable Strength Design	Fy : Steel Yield :	55.0 ksi
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis : Major Axis Bending		

Vertical Leg Up

D(0.060) Lr(0.080) S(0.10)   D(0.030) Lr(0.040) S(0.130)

8.5Zx2.5-11GA   8.5Zx2.5-11GA

Span = 12.0 ft   Span = 24.0 ft
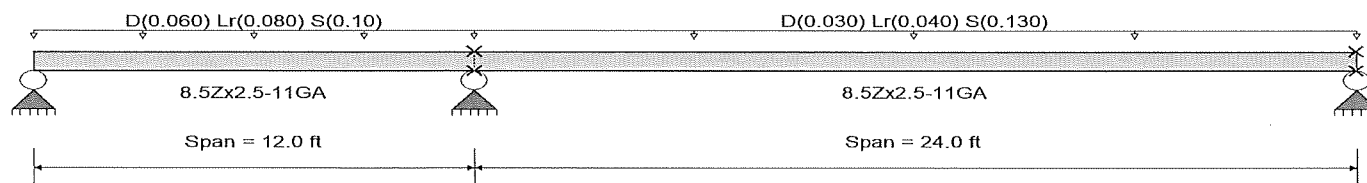
Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
    Uniform Load : D = 0.060, Lr = 0.080, S = 0.10 k/ft, Tributary Width = 1.0 ft

Load for Span Number 2
    Uniform Load : D = 0.030, Lr = 0.040, S = 0.130 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.757 : 1	Maximum Shear Stress Ratio =	0.125 : 1
Section used for this span	8.5Zx2.5-11GA	Section used for this span	8.5Zx2.5-11GA
Ma : Applied	8.640 k-ft	Va : Applied	2.280 k
Mn / Omega : Allowable	11.417 k-ft	Vn/Omega : Allowable	18.287 k
Load Combination	+D+S	Load Combination	+D+S
		Location of maximum on span	12.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	1.079 in	Ratio =	266	>=240.	Span: 2 : S Only
Max Upward Transient Deflection	-0.127 in	Ratio =	1,132	>=240.	Span: 2 : S Only
Max Downward Total Deflection	1.303 in	Ratio =	221	>=180	Span: 2 : +D+S
Max Upward Total Deflection	-0.133 in	Ratio =	1084	>=180	Span: 2 : +D+S

Maximum Forces & Stresses for Load Combinations

Load Combination Segment Length	Span #	Max Stress Ratios M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 12.00 ft	1	0.158	0.028	0.37	-1.80	1.80	19.07	11.42	1.00	1.00	0.51	30.54	18.29
Dsgn. L = 24.00 ft	2	0.158	0.024	1.35	-1.80	1.80	19.07	11.42	1.00	1.00	0.43	30.54	18.29
+D+Lr													
Dsgn. L = 12.00 ft	1	0.368	0.065	0.86	-4.20	4.20	19.07	11.42	1.00	1.00	1.19	30.54	18.29
Dsgn. L = 24.00 ft	2	0.368	0.056	3.16	-4.20	4.20	19.07	11.42	1.00	1.00	1.01	30.54	18.29
+D+S													
Dsgn. L = 12.00 ft	1	0.757	0.125	0.18	-8.64	8.64	19.07	11.42	1.00	1.00	2.28	30.54	18.29
Dsgn. L = 24.00 ft	2	0.757	0.125	7.60	-8.64	8.64	19.07	11.42	1.00	1.00	2.28	30.54	18.29
+D+0.750Lr													
Dsgn. L = 12.00 ft	1	0.315	0.056	0.73	-3.60	3.60	19.07	11.42	1.00	1.00	1.02	30.54	18.29
Dsgn. L = 24.00 ft	2	0.315	0.048	2.71	-3.60	3.60	19.07	11.42	1.00	1.00	0.87	30.54	18.29
+D+0.750S													
Dsgn. L = 12.00 ft	1	0.607	0.099	0.20	-6.93	6.93	19.07	11.42	1.00	1.00	1.82	30.54	18.29
Dsgn. L = 24.00 ft	2	0.607	0.099	6.04	-6.93	6.93	19.07	11.42	1.00	1.00	1.82	30.54	18.29
+0.60D													
Dsgn. L = 12.00 ft	1	0.095	0.017	0.22	-1.08	1.08	19.07	11.42	1.00	1.00	0.31	30.54	18.29
Dsgn. L = 24.00 ft	2	0.095	0.014	0.81	-1.08	1.08	19.07	11.42	1.00	1.00	0.26	30.54	18.29

Steel Beam

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: two span Z purlin (first at tower)

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+S	-0.1329	7.968
+D+S	2	1.3029	13.248		0.0000	7.968

Vertical Reactions

Support notation : Far left is #'   Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.490	3.960	1.560
Overall MINimum	0.030	0.567	0.171
D Only	0.210	0.945	0.285
+D+Lr	0.490	2.205	0.665
+D+S	0.240	3.960	1.560
+D+0.750Lr	0.420	1.890	0.570
+D+0.750S	0.233	3.206	1.241
+0.60D	0.126	0.567	0.171
Lr Only	0.280	1.260	0.380
S Only	0.030	3.015	1.275

Steel Beam

Project File: Tiger Moline.ec6

DESCRIPTION: simple span Z purlin (second at tower)

CODE REFERENCES

LOAD COMBINATIONS USED: IBC 2018

Material Properties

Analysis Method Allowable Strength Design	Fy : Steel Yield :	55.0 ksi
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis : Major Axis Bending		

Vertical Leg Up

D(0.030) Lr(0.040) S(0.1120)

8.5Zx2.5-11GA

Span = 24.0 ft

Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.030, Lr = 0.040, S = 0.1120 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio =	**0.895** : 1	Maximum Shear Stress Ratio =	**0.093** : 1
Section used for this span	**8.5Zx2.5-11GA**	Section used for this span	**8.5Zx2.5-11GA**
Ma : Applied	10.224 k-ft	Va : Applied	1.704 k
Mn / Omega : Allowable	11.417 k-ft	Vn/Omega : Allowable	18.287 k
Load Combination	+D+S	Load Combination	+D+S
		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	1.637 in	Ratio =	175	<240.0	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<240.0	Span: 1 : S Only
Max Downward Total Deflection	2.078 in	Ratio =	139	<180	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

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 = 24.00 ft	1	0.189	0.020	2.16		2.16	19.07	11.42	1.00	1.00	0.36	30.54	18.29
+D+Lr													
Dsgn. L = 24.00 ft	1	0.441	0.046	5.04		5.04	19.07	11.42	1.00	1.00	0.84	30.54	18.29
+D+S													
Dsgn. L = 24.00 ft	1	0.895	0.093	10.22		10.22	19.07	11.42	1.00	1.00	1.70	30.54	18.29
+D+0.750Lr													
Dsgn. L = 24.00 ft	1	0.378	0.039	4.32		4.32	19.07	11.42	1.00	1.00	0.72	30.54	18.29
+D+0.750S													
Dsgn. L = 24.00 ft	1	0.719	0.075	8.21		8.21	19.07	11.42	1.00	1.00	1.37	30.54	18.29
+0.60D													
Dsgn. L = 24.00 ft	1	0.114	0.012	1.30		1.30	19.07	11.42	1.00	1.00	0.22	30.54	18.29

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	2.0781	12.069		0.0000	0.000

Vertical Reactions

Support notation : Far left is #'   Values in KIPS

Load Combination	Support 1	Support 2	
Overall MAXimum	1.704	1.704	1.385
Overall MINimum	0.216	0.216	1.385
D Only	0.360	0.360	1.385

Steel Beam

DESCRIPTION: simple span Z purlin (second at tower)

Vertical Reactions

Support notation : Far left is #·   Values in KIPS

Load Combination	Support 1	Support 2	
+D+Lr	0.840	0.840	1.385
+D+S	1.704	1.704	1.385
+D+0.750Lr	0.720	0.720	1.385
+D+0.750S	1.368	1.368	1.385
+0.60D	0.216	0.216	1.385
Lr Only	0.480	0.480	1.385
S Only	1.344	1.344	1.385

Steel Beam

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

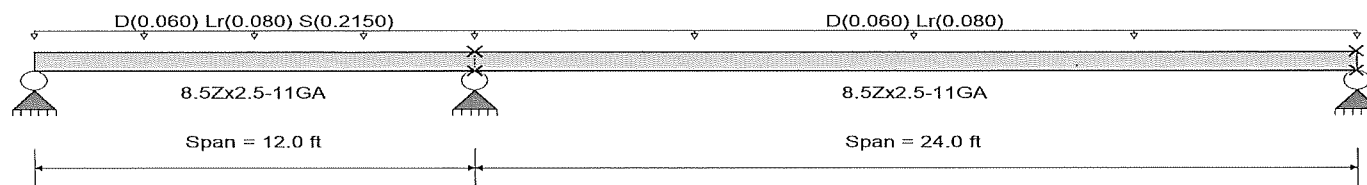**DESCRIPTION:** two span Z purlin (third at tower)

CODE REFERENCES

LOAD COMBINATIONS USED: IBC 2018

Material Properties

Analysis Method :Allowable Strength Design	Fy : Steel Yield :	55.0 ksi
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis : Major Axis Bending		

Vertical Leg Up

D(0.060) Lr(0.080) S(0.10)   D(0.0450) Lr(0.060) S(0.1410)

8.5Zx2.5-11GA   8.5Zx2.5-11GA

Span = 12.0 ft   Span = 24.0 ft

Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
     Uniform Load : D = 0.060, Lr = 0.080, S = 0.10 k/ft, Tributary Width = 1.0 ft

Load for Span Number 2
     Uniform Load : D = 0.0450, Lr = 0.060, S = 0.1410 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.866 : 1	Maximum Shear Stress Ratio =	0.145 : 1
Section used for this span	8.5Zx2.5-11GA	Section used for this span	8.5Zx2.5-11GA
Ma : Applied	9.888 k-ft	Va : Applied	2.644 k
Mn / Omega : Allowable	11.417 k-ft	Vn/Omega : Allowable	18.287 k
Load Combination	+D+S	Load Combination	+D+S
		Location of maximum on span	12.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	1.176 in	Ratio =	244	>=240.	Span: 2 : S Only
Max Upward Transient Deflection	-0.143 in	Ratio =	1,003	>=240.	Span: 2 : S Only
Max Downward Total Deflection	1.533 in	Ratio =	188	>=180	Span: 2 : +D+S
Max Upward Total Deflection	-0.171 in	Ratio =	843	>=180	Span: 2 : +D+S

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 = 12.00 ft	1	0.221	0.035	0.19	-2.52	2.52	19.07	11.42 1.00	1.00	0.64	30.54 18.29
Dsgn. L = 24.00 ft	2	0.221	0.035	2.10	-2.52	2.52	19.07	11.42 1.00	1.00	0.64	30.54 18.29
+D+Lr											
Dsgn. L = 12.00 ft	1	0.515	0.082	0.44	-5.88	5.88	19.07	11.42 1.00	1.00	1.50	30.54 18.29
Dsgn. L = 24.00 ft	2	0.515	0.082	4.91	-5.88	5.88	19.07	11.42 1.00	1.00	1.50	30.54 18.29
+D+S											
Dsgn. L = 12.00 ft	1	0.866	0.145	0.06	-9.89	9.89	19.07	11.42 1.00	1.00	2.64	30.54 18.29
Dsgn. L = 24.00 ft	2	0.866	0.145	8.90	-9.89	9.89	19.07	11.42 1.00	1.00	2.64	30.54 18.29
+D+0.750Lr											
Dsgn. L = 12.00 ft	1	0.441	0.071	0.37	-5.04	5.04	19.07	11.42 1.00	1.00	1.29	30.54 18.29
Dsgn. L = 24.00 ft	2	0.441	0.071	4.20	-5.04	5.04	19.07	11.42 1.00	1.00	1.29	30.54 18.29
+D+0.750S											
Dsgn. L = 12.00 ft	1	0.705	0.117	0.07	-8.05	8.05	19.07	11.42 1.00	1.00	2.14	30.54 18.29
Dsgn. L = 24.00 ft	2	0.705	0.117	7.20	-8.05	8.05	19.07	11.42 1.00	1.00	2.14	30.54 18.29
+0.60D											
Dsgn. L = 12.00 ft	1	0.132	0.021	0.11	-1.51	1.51	19.07	11.42 1.00	1.00	0.39	30.54 18.29
Dsgn. L = 24.00 ft	2	0.132	0.021	1.26	-1.51	1.51	19.07	11.42 1.00	1.00	0.39	30.54 18.29

Steel Beam

DESCRIPTION: two span Z purlin (third at tower)

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+S	-0.1709	7.776
+D+S	2	1.5330	13.152		0.0000	7.776

Vertical Reactions

Support notation : Far left is #'   Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.350	4.428	1.820
Overall MINimum	-0.014	0.729	0.261
D Only	0.150	1.215	0.435
+D+Lr	0.350	2.835	1.015
+D+S	0.136	4.428	1.820
+D+0.750Lr	0.300	2.430	0.870
+D+0.750S	0.140	3.625	1.474
+0.60D	0.090	0.729	0.261
Lr Only	0.200	1.620	0.580
S Only	-0.014	3.213	1.385

Steel Beam

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: two span Z purlin with drift at line 2

CODE REFERENCES

LOAD COMBINATIONS USED: IBC 2018

Material Properties

Analysis Method : Allowable Strength Design	Fy : Steel Yield : 55.0 ksi
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling	E: Modulus : 29,000.0 ksi
Bending Axis : Major Axis Bending	

Vertical Leg Up

D(0.060) Lr(0.080) S(0.2150)   D(0.060) Lr(0.080)

8.5Zx2.5-11GA   8.5Zx2.5-11GA

Span = 12.0 ft   Span = 24.0 ft

Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
    Uniform Load : D = 0.060, Lr = 0.080, S = 0.2150 k/ft, Tributary Width = 1.0 ft

Load for Span Number 2
    Uniform Load : D = 0.060, Lr = 0.080 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.662** : 1	Maximum Shear Stress Ratio =	**0.111** : 1
Section used for this span	**8.5Zx2.5-11GA**	Section used for this span	**8.5Zx2.5-11GA**
Ma : Applied	7.560 k-ft	Va : Applied	2.028 k
Mn / Omega : Allowable	11.417 k-ft	Vn/Omega : Allowable	18.287 k
Load Combination	+D+Lr	Load Combination	+D+S
		Location of maximum on span	12.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.650 in	Ratio =	442	>=240.	Span: 2 : S Only
Max Upward Transient Deflection	-0.162 in	Ratio =	1,773	>=240.	Span: 2 : Lr Only
Max Downward Total Deflection	1.140 in	Ratio =	253	>=180	Span: 2 : +D+Lr
Max Upward Total Deflection	-0.116 in	Ratio =	1239	>=180	Span: 2 : +D+Lr

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	VnxVnx/Omega
D Only											
Dsgn. L = 12.00 ft	1	0.284	0.047	0.07	-3.24	3.24	19.07	11.42 1.00	1.00	0.85	30.54 18.29
Dsgn. L = 24.00 ft	2	0.284	0.047	2.85	-3.24	3.24	19.07	11.42 1.00	1.00	0.85	30.54 18.29
+D+Lr											
Dsgn. L = 12.00 ft	1	0.662	0.109	0.16	-7.56	7.56	19.07	11.42 1.00	1.00	1.99	30.54 18.29
Dsgn. L = 24.00 ft	2	0.662	0.109	6.65	-7.56	7.56	19.07	11.42 1.00	1.00	1.99	30.54 18.29
+D+S											
Dsgn. L = 12.00 ft	1	0.397	0.111	2.94	-4.53	4.53	19.07	11.42 1.00	1.00	2.03	30.54 18.29
Dsgn. L = 24.00 ft	2	0.397	0.050	2.35	-4.53	4.53	19.07	11.42 1.00	1.00	0.91	30.54 18.29
+D+0.750Lr											
Dsgn. L = 12.00 ft	1	0.568	0.094	0.13	-6.48	6.48	19.07	11.42 1.00	1.00	1.71	30.54 18.29
Dsgn. L = 24.00 ft	2	0.568	0.094	5.70	-6.48	6.48	19.07	11.42 1.00	1.00	1.71	30.54 18.29
+D+0.750S											
Dsgn. L = 12.00 ft	1	0.369	0.092	2.16	-4.21	4.21	19.07	11.42 1.00	1.00	1.68	30.54 18.29
Dsgn. L = 24.00 ft	2	0.369	0.049	2.47	-4.21	4.21	19.07	11.42 1.00	1.00	0.90	30.54 18.29
+0.60D											
Dsgn. L = 12.00 ft	1	0.170	0.028	0.04	-1.94	1.94	19.07	11.42 1.00	1.00	0.51	30.54 18.29
Dsgn. L = 24.00 ft	2	0.170	0.028	1.71	-1.94	1.94	19.07	11.42 1.00	1.00	0.51	30.54 18.29

Steel Beam

DESCRIPTION: two span Z purlin with drift at line 2

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
S Only	1	0.1579	5.808	+D+Lr	-0.1124	8.832
+D+Lr	2	1.1400	13.248		0.0000	8.832

Vertical Reactions

Support notation : Far left is #·   Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	1.273	3.465	1.365
Overall MINimum	0.054	0.891	-0.054
D Only	0.090	1.485	0.585
+D+Lr	0.210	3.465	1.365
+D+S	1.273	2.936	0.531
+D+0.750Lr	0.180	2.970	1.170
+D+0.750S	0.977	2.573	0.545
+0.60D	0.054	0.891	0.351
Lr Only	0.120	1.980	0.780
S Only	1.183	1.451	-0.054

Steel Beam

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: 24' block lintel

CODE REFERENCES

LOAD COMBINATIONS USED: IBC 2018

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		

Vertical Leg Up

D(0.60)

W16x31

Span = 24.0 ft

Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.60 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.337 : 1	Maximum Shear Stress Ratio =	0.087 : 1
Section used for this span	W16x31	Section used for this span	W16x31
Ma : Applied	45.432 k-ft	Va : Applied	7.572 k
Mn / Omega : Allowable	134.731 k-ft	Vn/Omega : Allowable	87.450 k
Load Combination	D Only	Load Combination	D 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	0.000 in	Ratio =	0	<360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	
Max Downward Total Deflection	0.435 in	Ratio =	662	>=180	Span: 1 : D Only
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

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 = 24.00 ft	1	0.337	0.087	45.43		45.43	225.00	134.73	1.00	1.00	7.57	131.18	87.45
+0.60D													
Dsgn. L = 24.00 ft	1	0.202	0.052	27.26		27.26	225.00	134.73	1.00	1.00	4.54	131.18	87.45

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.4351	12.069		0.0000	0.000

Vertical Reactions

Support notation : Far left is #·   Values in KIPS

Load Combination	Support 1	Support 2	
Overall MAXimum	7.572	7.572	1.385
Overall MINimum	4.543	4.543	1.385
D Only	7.572	7.572	1.385
+0.60D	4.543	4.543	1.385

Steel Beam

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31 CROCKETT ENGINEERING CONSULTANTS (c) ENERCALC INC 1983-2022

DESCRIPTION: 16' block lintel

CODE REFERENCES

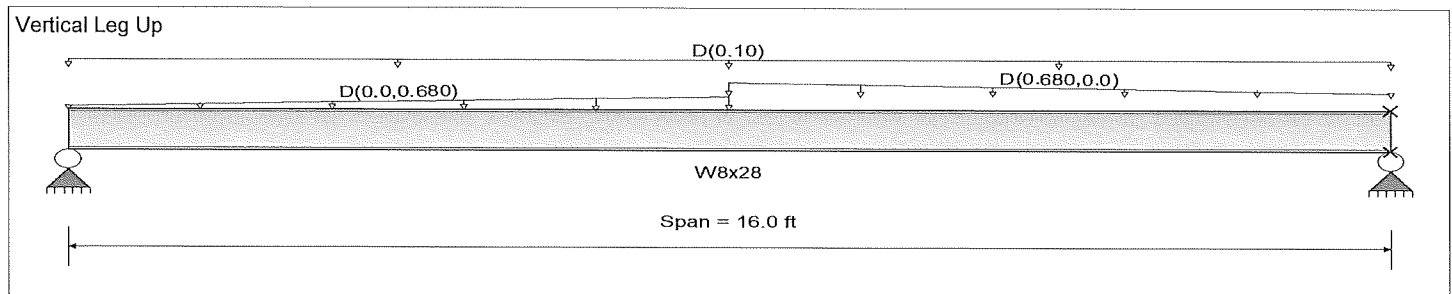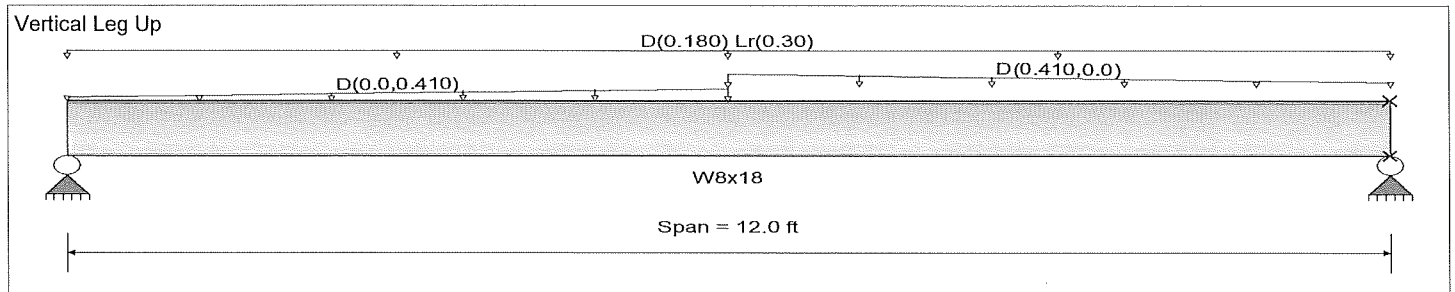## LOAD COMBINATIONS USED: IBC 2018

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			

Vertical Leg Up

D(0.10)

D(0.0,0.680)

D(0.680,0.0)

W8x28

Span = 16.0 ft

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= 0.0->0.680 k/ft, Extent = 0.0 -->> 8.0 ft, Trib Width = 1.0 ft

  Varying Uniform Load : D= 0.680->0.0 k/ft, Extent = 8.0 -->> 16.0 ft, Trib Width = 1.0 ft, (CMU)

  Uniform Load : D = 0.10 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.274 : 1	Maximum Shear Stress Ratio =		0.081 : 1
Section used for this span	W8x28	Section used for this span		W8x28
Ma : Applied	18.603 k-ft	Va : Applied		3.744 k
Mn / Omega : Allowable	67.864 k-ft	Vn/Omega : Allowable		45.942 k
Load Combination	D Only	Load Combination		D 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	0.000 in	Ratio =	0	<360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	
Max Downward Total Deflection	0.294 in	Ratio =	654	>=180	Span: 1 : D Only
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

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	VnxVnx/Omega
D Only												
Dsgn. L = 16.00 ft	1	0.274	0.081	18.60		18.60	113.33	67.86	1.00	1.00	3.74	68.91 45.94
+0.60D												
Dsgn. L = 16.00 ft	1	0.164	0.049	11.16		11.16	113.33	67.86	1.00	1.00	2.25	68.91 45.94

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.2935	8.046		0.0000	0.000

Vertical Reactions

Support notation : Far left is #·   Values in KIPS

Load Combination	Support 1	Support 2		
Overall MAXimum	3.744	3.744	1.385	
Overall MINimum	2.246	2.246	1.385	
D Only	3.744	3.744	1.385	
+0.60D	2.246	2.246	1.385	

Steel Beam

DESCRIPTION: 12' block lintel

CODE REFERENCES

LOAD COMBINATIONS USED: IBC 2018

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		

Vertical Leg Up

Beam diagram: D(0.180) Lr(0.30); D(0.0,0.410); D(0.410,0.0); W8x18; Span = 12.0 ft

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= 0.0->0.410 k/ft, Extent = 0.0 -->> 6.0 ft, Trib Width = 1.0 ft

  Varying Uniform Load : D= 0.410->0.0 k/ft, Extent = 6.0 -->> 12.0 ft, Trib Width = 1.0 ft, (CMU)

  Uniform Load : D = 0.180, Lr = 0.30 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.327 : 1	Maximum Shear Stress Ratio =	0.113 : 1
Section used for this span	W8x18	Section used for this span	W8x18
Ma : Applied	13.884 k-ft	Va : Applied	4.218 k
Mn / Omega : Allowable	42.415 k-ft	Vn/Omega : Allowable	37.444 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	0.078 in Ratio =	1,838	>=360	
Max Upward Transient Deflection	0.000 in Ratio =	0	<360	Span: 1 : Lr Only
Max Downward Total Deflection	0.199 in Ratio =	725	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0.000 in Ratio =	0	<180	

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 = 12.00 ft	1	0.200	0.065	8.48		8.48	70.83	42.42	1.00 1.00	2.42	56.17	37.44
+D+Lr												
Dsgn. L = 12.00 ft	1	0.327	0.113	13.88		13.88	70.83	42.42	1.00 1.00	4.22	56.17	37.44
+D+0.750Lr												
Dsgn. L = 12.00 ft	1	0.296	0.101	12.53		12.53	70.83	42.42	1.00 1.00	3.77	56.17	37.44
+0.60D												
Dsgn. L = 12.00 ft	1	0.120	0.039	5.09		5.09	70.83	42.42	1.00 1.00	1.45	56.17	37.44

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.1985	6.034		0.0000	0.000

Vertical Reactions

Support notation : Far left is #'   Values in KIPS

Load Combination	Support 1	Support 2	
Overall MAXimum	4.218	4.218	1.385

Steel Beam

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: 12' block lintel

Vertical Reactions

Support notation : Far left is #·   Values in KIPS

Load Combination	Support 1	Support 2	
Overall MINimum	1.451	1.451	1.385
D Only	2.418	2.418	1.385
+D+Lr	4.218	4.218	1.385
+D+0.750Lr	3.768	3.768	1.385
+0.60D	1.451	1.451	1.385
Lr Only	1.800	1.800	1.385

Steel Beam

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: 8' block lintel

CODE REFERENCES

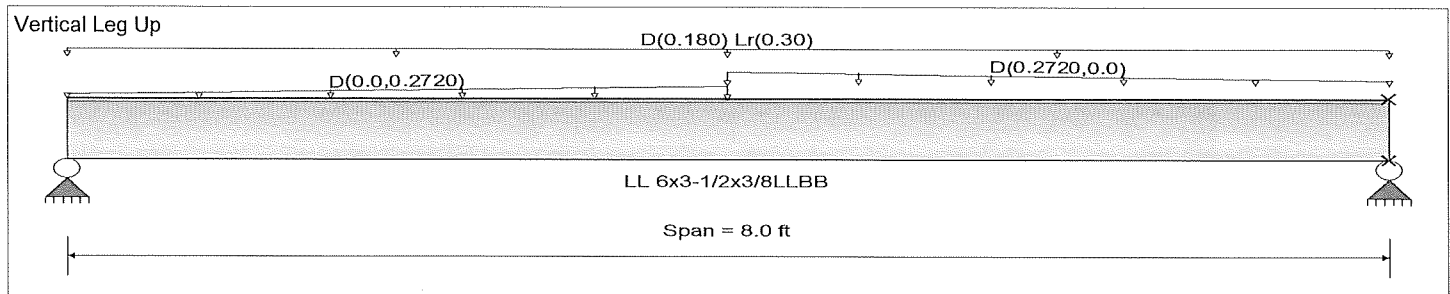## LOAD COMBINATIONS USED: IBC 2018

Material Properties

Analysis Method :Allowable Strength Design			Fy : Steel Yield :	36.0 ksi
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling			E: Modulus :	29,000.0 ksi
Bending Axis : Major Axis Bending				

Vertical Leg Up

D(0.180) Lr(0.30)

D(0.0,0.2720)

D(0.2720,0.0)

LL 6x3-1/2x3/8LLBB

Span = 8.0 ft

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= 0.0->0.2720 k/ft, Extent = 0.0 -->> 4.0 ft, Trib Width = 1.0 ft

    Varying Uniform Load : D= 0.2720->0.0 k/ft, Extent = 4.0 -->> 8.0 ft, Trib Width = 1.0 ft, (CMU)

    Uniform Load : D = 0.180, Lr = 0.30 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.322 : 1	Maximum Shear Stress Ratio =	0.044 : 1
Section used for this span	LL 6x3-1/2x3/8LLBB	Section used for this span	LL 6x3-1/2x3/8LLBB
Ma : Applied	5.478 k-ft	Va : Applied	2.558 k
Mn / Omega : Allowable	17.006 k-ft	Vn/Omega : Allowable	58.204 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	0.037 in	Ratio =	2,586	>=360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	Span: 1 : Lr Only
Max Downward Total Deflection	0.084 in	Ratio =	1145	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

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 = 8.00 ft	1	0.181	0.023	3.08		3.08	28.40	17.01 1.00	1.00	1.36	97.20	58.20
+D+Lr												
Dsgn. L = 8.00 ft	1	0.322	0.044	5.48		5.48	28.40	17.01 1.00	1.00	2.56	97.20	58.20
+D+0.750Lr												
Dsgn. L = 8.00 ft	1	0.287	0.039	4.88		4.88	28.40	17.01 1.00	1.00	2.26	97.20	58.20
+0.60D												
Dsgn. L = 8.00 ft	1	0.109	0.014	1.85		1.85	28.40	17.01 1.00	1.00	0.81	97.20	58.20

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0838	4.023		0.0000	0.000

Vertical Reactions

Support notation : Far left is #'   Values in KIPS

Load Combination	Support 1	Support 2	
Overall MAXimum	2.558	2.558	1.385

Steel Beam

CROCKETT ENGINEERING CONSULTANTS

Project File: Tiger Moline.ec6

(c) ENERCALC INC 1983-2022

DESCRIPTION: 8' block lintel

Vertical Reactions

Support notation : Far left is #   Values in KIPS

Load Combination	Support 1	Support 2	
Overall MINimum	0.815	0.815	1.385
D Only	1.358	1.358	1.385
+D+Lr	2.558	2.558	1.385
+D+0.750Lr	2.258	2.258	1.385
+0.60D	0.815	0.815	1.385
Lr Only	1.200	1.200	1.385

CROCKETT
ENGINEERING CONSULTANTS
1000 W Nifong Blvd., Bldg. 1
Columbia, Missouri 65203
(573) 447-0292

Look @ Upper Element

Conservatively use Ext wind

$$\frac{5.75'}{2} (30.2 psf) = 86.9 plf$$

$$86.9 \left(\frac{25.33}{2}\right) = 1,101 \# \quad \frac{\#}{2} = 550.5 \ per\ col.$$
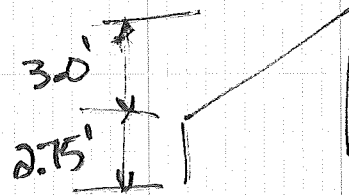
$$R_2 = \frac{551(11.75)}{6.0} = 1080\#$$

$$R_1 = 1080 - 551 = \underline{529\#}$$
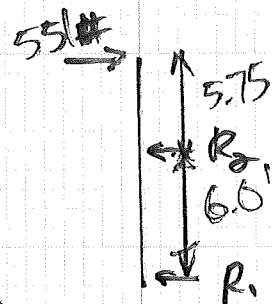
$$Gravity\ to\ single\ column = 40\left(\frac{28.33}{2}\right)\left(\frac{28.33}{2}\right) =$$

$$\underline{8.03K}$$

½"∅ A36 Threaded Rod w/ 4¼" HY200 Epoxy Embed

has 1785# Tension & 1935# Shear
per Anchor

have 1 @ Top & Bott for Lateral Then 6 for shear

$$1935(6) = 11,610\# \quad Thus \ \underline{OK}$$

Steel Column

DESCRIPTION: cant. column at upper canopy

Code References

LOAD COMBINATIONS USED: IBC 2018

General Information

Steel Section Name :	HSS4x4x1/4	Overall Column Height	6 ft
Analysis Method :	Allowable Strength	Top & Bottom Fixity	Top Free, Bottom Fixed
Steel Stress Grade		Brace condition for deflection (buckling) along columns :	
F_y : Steel Yield	46 ksi	X-X (width) axis :	
E : Elastic Bending Modulus	29,000.0 ksi	Unbraced Length for buckling ABOUT Y-Y Axis = 6 ft, K = 2.1	

Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 6 ft, K = 2.1

Applied Loads

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

Column self weight included : 73.260 lbs * Dead Load Factor
AXIAL LOADS . . .
   Axial Load at 6.0 ft, Xecc = 1.0 in, D = 3.010, LR = 4.010, S = 5.020 k
BENDING LOADS . . .
   Lat. Point Load at 6.0 ft creating Mx-x, W = 0.5510 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =   0.2624 : 1

Load Combination	+D+0.750S+0.450W	**Maximum Load Reactions . .**	
Location of max.above base	0.0 ft	Top along X-X	0.0 k
At maximum location values are . . .		Bottom along X-X	0.0 k
		Top along Y-Y	0.0 k
Pa : Axial	6.848 k	Bottom along Y-Y	0.5510 k
Pn / Omega : Allowable	47.709 k		
Ma-x : Applied	-1.488 k-ft	**Maximum Load Deflections . . .**	
Mn-x / Omega : Allowable	10.765 k-ft	Along Y-Y 0.3015 in at 6.0ft above base	
Ma-y : Applied	-0.5646 k-ft	for load combination :W Only	
Mn-y / Omega : Allowable	10.765 k-ft	Along X-X 0.09140 in at 6.0ft above base	
		for load combination :+D+S	

PASS Maximum Shear Stress Ratio   0.0130 : 1

Load Combination	+D+0.60W
Location of max.above base	0.0 ft
At maximum location values are . . .	
Va : Applied	0.3306 k
Vn / Omega : Allowable	25.423 k

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	0.056	PASS	4.67 ft	1.67	1.00	99.47	99.47	0.000	PASS	0.00 ft
+D+Lr	0.129	PASS	5.68 ft	1.67	1.00	99.47	99.47	0.000	PASS	0.00 ft
+D+S	0.147	PASS	0.00 ft	1.67	1.00	99.47	99.47	0.000	PASS	0.00 ft
+D+0.750Lr	0.110	PASS	4.67 ft	1.67	1.00	99.47	99.47	0.000	PASS	0.00 ft
+D+0.750S	0.124	PASS	5.03 ft	1.67	1.00	99.47	99.47	0.000	PASS	0.00 ft
+D+0.60W	0.240	PASS	0.00 ft	1.67	1.00	99.47	99.47	0.013	PASS	0.00 ft
+D+0.750Lr+0.450W	0.249	PASS	0.00 ft	1.67	1.00	99.47	99.47	0.010	PASS	0.00 ft
+D+0.750S+0.450W	0.262	PASS	0.00 ft	1.67	1.00	99.47	99.47	0.010	PASS	0.00 ft
+0.60D+0.60W	0.218	PASS	0.00 ft	1.67	1.00	99.47	99.47	0.013	PASS	0.00 ft
+0.60D	0.033	PASS	4.35 ft	1.67	1.00	99.47	99.47	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
D Only	3.083							-0.251	
+D+Lr	7.093							-0.585	
+D+S	8.103							-0.669	

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Column

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: cant. column at upper canopy

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
+D+0.750Lr	6.091								-0.501	
+D+0.750S	6.848								-0.565	
+D+0.60W	3.083				0.331		-1.984		-0.251	
+D+0.750Lr+0.450W	6.091				0.248		-1.488		-0.501	
+D+0.750S+0.450W	6.848				0.248		-1.488		-0.565	
+0.60D+0.60W	1.850				0.331		-1.984		-0.151	
+0.60D	1.850								-0.151	
Lr Only	4.010								-0.334	
S Only	5.020								-0.418	
W Only					0.551		-3.306			

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments k-ft @ Base	@ Top	My - End Moments @ Base	@ Top
Axial @ Base	Maximum	8.103								-0.669	
"	Minimum					0.551		-3.306			
Reaction, X-X Axis Base	Maximum	3.083								-0.251	
"	Minimum	3.083								-0.251	
Reaction, Y-Y Axis Base	Maximum					0.551		-3.306			
"	Minimum	3.083						-0.251			
Reaction, X-X Axis Top	Maximum	3.083								-0.251	
"	Minimum	3.083								-0.251	
Reaction, Y-Y Axis Top	Maximum	3.083								-0.251	
"	Minimum	3.083								-0.251	
Moment, X-X Axis Base	Maximum	3.083								-0.251	
"	Minimum		-3.306			0.551		-3.306			
Moment, Y-Y Axis Base	Maximum					0.551					
"	Minimum	8.103								-0.669	
Moment, X-X Axis Top	Maximum	3.083								-0.251	
"	Minimum	3.083								-0.251	
Moment, Y-Y Axis Top	Maximum	3.083								-0.251	
"	Minimum	3.083								-0.251	

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0343 in	6.000 ft	0.000 in	0.000 ft
+D+Lr	0.0799 in	6.000 ft	0.000 in	0.000 ft
+D+S	0.0914 in	6.000 ft	0.000 in	0.000 ft
+D+0.750Lr	0.0685 in	6.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0771 in	6.000 ft	0.000 in	0.000 ft
+D+0.60W	0.0343 in	6.000 ft	0.181 in	6.000 ft
+D+0.750Lr+0.450W	0.0685 in	6.000 ft	0.136 in	6.000 ft
+D+0.750S+0.450W	0.0771 in	6.000 ft	0.136 in	6.000 ft
+0.60D+0.60W	0.0206 in	6.000 ft	0.181 in	6.000 ft
+0.60D	0.0206 in	6.000 ft	0.000 in	0.000 ft
Lr Only	0.0456 in	6.000 ft	0.000 in	0.000 ft
S Only	0.0571 in	6.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.298 in	5.960 ft

Steel Section Properties :   HSS4x4x1/4

Steel Section Properties :   HSS4x4x1/4

Steel Column

DESCRIPTION: cant. column at upper canopy

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

Ycg   =   0.000 in

Sketches

Steel Beam

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31   CROCKETT ENGINEERING CONSULTANTS   (c) ENERCALC INC 1983-2022

DESCRIPTION: upper roof middle beam

CODE REFERENCES

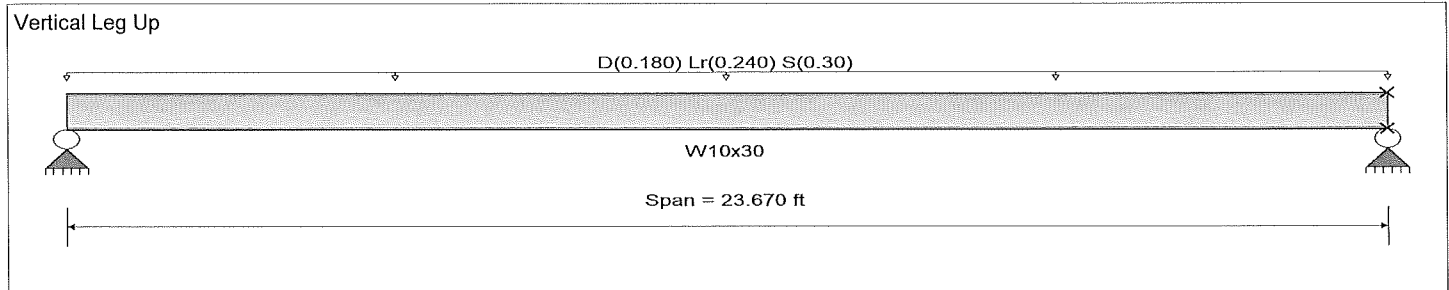## LOAD COMBINATIONS USED: IBC 2018

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	

Vertical Leg Up

D(0.180) Lr(0.240) S(0.30)

W10x30

Span = 23.670 ft

Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.180, Lr = 0.240, S = 0.30 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.368 : 1	Maximum Shear Stress Ratio =		0.090 : 1
Section used for this span	W10x30	Section used for this span		W10x30
Ma : Applied	33.616 k-ft	Va : Applied		5.681 k
Mn / Omega : Allowable	91.317 k-ft	Vn/Omega : Allowable		63.0 k
Load Combination	+D+S	Load Combination		+D+S
		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	0.432 in	Ratio =	657	>=360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	Span: 1 : S Only
Max Downward Total Deflection	0.691 in	Ratio =	411	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

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 = 23.67 ft	1	0.138	0.034	12.61		12.61	152.50	91.32	1.00	1.00	2.13	94.50	63.00
+D+Lr													
Dsgn. L = 23.67 ft	1	0.322	0.079	29.41		29.41	152.50	91.32	1.00	1.00	4.97	94.50	63.00
+D+S													
Dsgn. L = 23.67 ft	1	0.368	0.090	33.62		33.62	152.50	91.32	1.00	1.00	5.68	94.50	63.00
+D+0.750Lr													
Dsgn. L = 23.67 ft	1	0.276	0.068	25.21		25.21	152.50	91.32	1.00	1.00	4.26	94.50	63.00
+D+0.750S													
Dsgn. L = 23.67 ft	1	0.311	0.076	28.36		28.36	152.50	91.32	1.00	1.00	4.79	94.50	63.00
+0.60D													
Dsgn. L = 23.67 ft	1	0.083	0.020	7.56		7.56	152.50	91.32	1.00	1.00	1.28	94.50	63.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.6908	11.903		0.0000	0.000

Vertical Reactions

Support notation : Far left is #'   Values in KIPS

Load Combination	Support 1	Support 2		
Overall MAXimum	5.681	5.681	1.385	
Overall MINimum	1.278	1.278	1.385	
D Only	2.130	2.130	1.385	

Steel Beam

DESCRIPTION: upper roof middle beam

Vertical Reactions

Support notation : Far left is #·   Values in KIPS

Load Combination	Support 1	Support 2	
+D+Lr	4.971	4.971	1.385
+D+S	5.681	5.681	1.385
+D+0.750Lr	4.261	4.261	1.385
+D+0.750S	4.793	4.793	1.385
+0.60D	1.278	1.278	1.385
Lr Only	2.840	2.840	1.385
S Only	3.551	3.551	1.385

Steel Beam

Project File: Tiger Moline.ec6

LIC# : KW-06015690, Build:20.22.3.31	CROCKETT ENGINEERING CONSULTANTS	(c) ENERCALC INC 1983-2022

DESCRIPTION: upper roof end beam

CODE REFERENCES

LOAD COMBINATIONS USED: IBC 2018

Material Properties

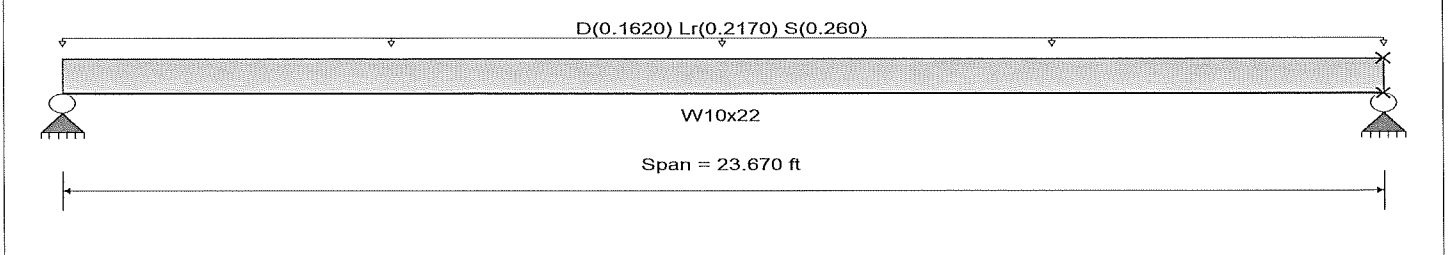| Analysis Method Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis : Major Axis Bending		

Vertical Leg Up

D(0.1620) Lr(0.2170) S(0.260)

W10x22

Span = 23.670 ft

Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.1620, Lr = 0.2170, S = 0.260 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.456 : 1	Maximum Shear Stress Ratio =	0.102 : 1
Section used for this span	W10x22	Section used for this span	W10x22
Ma : Applied	29.554 k-ft	Va : Applied	4.994 k
Mn / Omega : Allowable	64.870 k-ft	Vn/Omega : Allowable	48.960 k
Load Combination	+D+S	Load Combination	+D+S
		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	0.539 in	Ratio =	526	>=360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	Span: 1 : S Only
Max Downward Total Deflection	0.875 in	Ratio =	325	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

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 = 23.67 ft	1	0.175	0.039	11.35		11.35	108.33	64.87	1.00	1.00	1.92	73.44	48.96
+D+Lr													
Dsgn. L = 23.67 ft	1	0.409	0.092	26.54		26.54	108.33	64.87	1.00	1.00	4.49	73.44	48.96
+D+S													
Dsgn. L = 23.67 ft	1	0.456	0.102	29.55		29.55	108.33	64.87	1.00	1.00	4.99	73.44	48.96
+D+0.750Lr													
Dsgn. L = 23.67 ft	1	0.351	0.079	22.74		22.74	108.33	64.87	1.00	1.00	3.84	73.44	48.96
+D+0.750S													
Dsgn. L = 23.67 ft	1	0.385	0.086	25.00		25.00	108.33	64.87	1.00	1.00	4.23	73.44	48.96
+0.60D													
Dsgn. L = 23.67 ft	1	0.105	0.023	6.81		6.81	108.33	64.87	1.00	1.00	1.15	73.44	48.96

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.8750	11.903		0.0000	0.000

Vertical Reactions

Support notation : Far left is #'   Values in KIPS

Load Combination	Support 1	Support 2	
Overall MAXimum	4.994	4.994	1.385
Overall MINimum	1.150	1.150	1.385
D Only	1.917	1.917	1.385

Steel Beam

DESCRIPTION: upper roof end beam

Vertical Reactions

Support notation : Far left is #'   Values in KIPS

Load Combination	Support 1	Support 2	
+D+Lr	4.485	4.485	1.385
+D+S	4.994	4.994	1.385
+D+0.750Lr	3.843	3.843	1.385
+D+0.750S	4.225	4.225	1.385
+0.60D	1.150	1.150	1.385
Lr Only	2.568	2.568	1.385
S Only	3.077	3.077	1.385

Steel Beam	Project File: Tiger Moline.ec6
LIC# : KW-06015690, Build:20.22.3.31 CROCKETT ENGINEERING CONSULTANTS	(c) ENERCALC INC 1983-2022

DESCRIPTION: upper roof girder beam

CODE REFERENCES

LOAD COMBINATIONS USED: IBC 2018

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		

Vertical Leg Up

D(2.130) S(3.550)
D(0.060) Lr(0.080) S(0.10)

W10x22

Span = 23.670 ft

Applied Loads

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

Beam self weight NOT internally calculated and added
   Uniform Load : D = 0.060, Lr = 0.080, S = 0.10 k/ft, Tributary Width = 1.0 ft

   Point Load : D = 2.130, S = 3.550 k @ 12.0 ft

DESIGN SUMMARY

				Design OK
Maximum Bending Stress Ratio =	0.689 : 1	Maximum Shear Stress Ratio =		0.097 : 1
Section used for this span	W10x22	Section used for this span		W10x22
Ma : Applied	44.725 k-ft	Va : Applied		4.773 k
Mn / Omega : Allowable	64.870 k-ft	Vn/Omega : Allowable		48.960 k
Load Combination	+D+S	Load Combination		+D+S
		Location of maximum on span		23.670 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs		Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.703 in	Ratio =	403	>=360	
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360	Span: 1 : S Only
Max Downward Total Deflection	1.127 in	Ratio =	252	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0	<180	

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	VnxVnx/Omega
D Only												
Dsgn. L = 23.67 ft	1	0.259	0.037	16.77		16.77	108.33	64.87	1.00	1.00	1.79	73.44 48.96
+D+Lr												
Dsgn. L = 23.67 ft	1	0.345	0.056	22.37		22.37	108.33	64.87	1.00	1.00	2.74	73.44 48.96
+D+S												
Dsgn. L = 23.67 ft	1	0.689	0.097	44.73		44.73	108.33	64.87	1.00	1.00	4.77	73.44 48.96
+D+0.750Lr												
Dsgn. L = 23.67 ft	1	0.323	0.051	20.97		20.97	108.33	64.87	1.00	1.00	2.50	73.44 48.96
+D+0.750S												
Dsgn. L = 23.67 ft	1	0.582	0.082	37.74		37.74	108.33	64.87	1.00	1.00	4.03	73.44 48.96
+0.60D												
Dsgn. L = 23.67 ft	1	0.155	0.022	10.06		10.06	108.33	64.87	1.00	1.00	1.07	73.44 48.96

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	1.1274	11.903		0.0000	0.000

Vertical Reactions

Support notation : Far left is #   Values in KIPS

Load Combination	Support 1	Support 2	
Overall MAXimum	4.694	4.773	1.385

Steel Beam

DESCRIPTION: upper roof girder beam

Vertical Reactions

Support notation : Far left is #·   Values in KIPS

Load Combination	Support 1	Support 2	
Overall MINimum	0.947	0.947	1.385
D Only	1.760	1.790	1.385
+D+Lr	2.707	2.737	1.385
+D+S	4.694	4.773	1.385
+D+0.750Lr	2.470	2.500	1.385
+D+0.750S	3.961	4.027	1.385
+0.60D	1.056	1.074	1.385
Lr Only	0.947	0.947	1.385
S Only	2.934	2.983	1.385
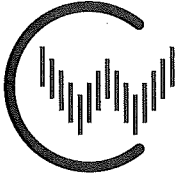
2007 North American Specification ASD
DATE: 6/8/2022

Upper Joists

SECTION DESIGNATION: 600S162-54 [50] Single

Input Properties:

Web Height =	6.000 in	Design Thickness =	0.0566 in
Top Flange =	1.625 in	Inside Corner Radius =	0.0849 in
Bottom Flange =	1.625 in	Yield Point, Fy =	50.0 ksi
Stiffening Lip =	0.500 in	Fy With Cold-Work, Fya =	55.3 ksi
Punchout Width =	1.500 in	Punchout Length =	4.000 in

Floor Solver Design Data - Simple Span

Joist Span 13.00 ft   Joist Spacing 24.0 in
Dead Load = 15.0 psf
Live Load = 25.0 psf

Check Flexure

Mmax = 1690 Ft-Lb <= Ma = 2527 Ft-Lb & Ma(distortional) = 2158 Ft-Lb
K-phi for Distortional Buckling = 0 lb*in/in

Check Deflection

Total Load Defl Limit: L/240
Total Load Defl. = 0.609 in   Total Load Defl. Ratio = L/256
Live Load Defl Limit: L/360
Live Load Defl. = 0.381 in   Live Load Defl. Ratio = L/410

Check Web Crippling

Web Crippling capacity not reduced for punchouts near ends of member
Rmax = 520 lb
End Bearing Length = 1.00 in
Ra = 599 lb >= Rmax, stiffeners not required

Check Shear

Vmax = 520 lb
Shear capacity not reduced for punchouts near ends of member
Va = 2823 lb >= Vmax

CMU

Company
Address
City
Phone
Other

JOB TITLE ....

JOB NO. .... SHEET NO. ....
CALCULATED BY .... DATE ....
CHECKED BY .... DATE ....

CMU16 Ver 2018.02.04 **CMU SLENDER WALL**

DESCRIPTION: Ext. wall section

PANEL PROPERTIES

Wall Height: h =	15.00 ft	f'm =	2,000 psi	
Parapet Height =	3.00 ft	fy =	60,000 psi	
Nominal Thickness (t) =	8.0 in			
Running Bond?	Yes	ungrouted fr =	63.0 psi	
All cells grouted?	No	interpolated fr =	79.7 psi	
Block Weight =	Normal Wt	Em factor =	900	
Grout Weight =	140 psi	Deflection limit =	0.0070 h	
		Min Defl ratio =	142.9	
Vert Bar Location :	Centered			
Cover if Bar Ea Face =	2.000 in	.20f'm =	400.0 psi	
Bar Depth: d =	3.813 in	Em =	1800.0 ksi	
		n =	16.11	
Vertical Bar Size =	# 5	Wall Weight =	55 psf	
Typical Bar Spacing =	48.0 in	Tributary width =	4.00 ft	
As =	0.077 in2/ft	Strip Width Properties		
		b =	48.0 in	
Opening Properties		Ag =	162.8 in2	
No Opening		Sg =	348.4 in3	
		Ig =	1406.8 in4	
EL Top of Opening =	0.00 ft	Mcr =	2,313 'lb	
EL Bottom Opening =	0.00 ft	ρ =	0.00168	
Opening Material Wt =	10.0 psf	ρ max =	0.00924	
# cells reinf @ opng =	2	As =	0.31 in2	
First cell reinforced? =	No	Opng Wall Wt =	0 psf	

LOADING

Lateral Loading

		plf	Ms ('lb)
Ultimate Wind Pressure =	24.7 psf	98.8	2,779
Seismic Factor =	0.034	7.5	210
Use this wind moment instead :			0
Use this seismic moment instead :			0

Vertical Loading

Sds = 0.117

Uniform Vertical Loading		eccentricity	lb	Ms ('lb)
Full ht wall wt =	578 plf	0.000 in	2,310	0
Dead Load =	180 plf	2.313 in	720	69
Floor Live Load =	0 plf	2.313 in	0	0
Roof Live Load =	300 plf	2.313 in	1,200	116
Roof Snow Load =	0 plf	2.313 in	0	0

Concentrated Vertical Loading		eccentricity		
Opening wt midht =	0.0 k	0.000 in	0	0
Dead Load =	0.0 k	2.000 in	0	0
Floor Live Load =	0.0 k	2.000 in	0	0
Roof Live Load =	0.0 k	2.000 in	0	0
Roof Snow Load =	0.0 k	2.000 in	0	0

Garage, assembly or LL>100psf: ○   Roofs that don't shed snow: ◉
All Others: ◉   All Others: ○

TYPICAL WALL REINFORCING RESULTS
LRFD Combinations

Note: Only the controlling Lr or S load combination is displayed.

	1.4D	1.2D +1.6L +0.5Lr	1.2D +0.5L +1.6Lr	1.2D +0.5W +1.6Lr	1.2D +1.0W +0.5L +0.5Lr	(1.2+.2Sds)D +1.0E +0.5L +0.7S	0.9D +1.0W	(0.9-.2Sds)D +1.0E	
Mua ('lb) =	97	141	268	1,658	2,920	295	2,841	271	
Pu (lb) =	4,242	4,236	5,556	5,556	4,236	3,707	2,727	2,656	
Pu /Ag (psi) =	26.1	26.0	34.1	34.1	26.0	22.8	16.8	16.3	COMPRESSION OKAY
Ase (in2) =	0.378	0.378	0.400	0.400	0.378	0.369	0.352	0.351	
c (in) =	0.369	0.369	0.390	0.390	0.369	0.360	0.344	0.343	
Icr (in4) =	73.0	72.9	76.4	76.4	72.9	71.6	69.0	68.8	
a (in) =	0.301	0.301	0.320	0.320	0.301	0.293	0.279	0.278	
δu (in) =	0.002	0.002	0.004	0.027	0.252	0.005	0.226	0.004	
Mu ('lb) =	98	142	270	1,670	3,009	297	2,893	272	
Capacity: φMn ('lb) =	6,353	6,352	6,737	6,737	6,352	6,197	5,909	5,888	

STRENGTH OKAY

DEFLECTION
Service Load Combinations

	D	D +L or +Lr	D +Lr or +S	D +0.6W	D+0.45W +0.75L +0.75Lr	(1+.105Sds)D +0.525E +0.75(L+S)	0.6D +0.6W	(0.6-.14Sds)D +0.7E	DEFLECTION OKAY
Msa ('lb) =	69	185	185	1,737	1,407	1,709	1,709	188	
Ps (lb) =	3,030	4,230	4,230	3,030	3,930	1,818	1,818	1,781	
M ('lb) =	70	186	186	1,744	1,414	1,713	1,713	188	
δs (in) =	0.001	0.003	0.003	0.028	0.023	0.027	0.027	0.003	
Defl Ratio =	L / 9999	L / 9999	L / 9999	L / 6454	L / 7960	L / 6570	L / 6570	L / 9999	

CMU16 Ver 2018.02.04

CMU SLENDER WALL

DESCRIPTION: typ. int wall section

PANEL PROPERTIES

Wall Height: h =	15.00 ft	f'm =	2,000 psi
Parapet Height =	0.00 ft	fy =	60,000 psi
Nominal Thickness (t) =	8.0 in		
Running Bond?	Yes	ungrouted fr =	63.0 psi
All cells grouted?	No	interpolated fr =	79.7 psi
Block Weight =	Normal Wt	Em factor =	900
Grout Weight =	140 psi	Deflection limit =	0.0070 h
		Min Defl ratio =	142.9
Vert Bar Location :	Centered		
Cover if Bar Ea Face =	2.000 in	.20f'm =	400.0 psi
Bar Depth: d =	3.813 in	Em =	1800.0 ksi
		n =	16.11
Vertical Bar Size =	# 5	Wall Weight =	55 psf
Typical Bar Spacing =	48.0 in	Tributary width =	4.00 ft
As =	0.077 in2/ft	Strip Width Properties	
		b =	48.0 in
Opening Properties		Ag =	162.8 in2
No Opening		Sg =	348.4 in3
		Ig =	1406.8 in4
EL Top of Opening =	0.00 ft	Mcr =	2,313 'lb
EL Bottom Opening =	0.00 ft	ρ =	0.00168
Opening Material Wt =	10.0 psf	ρ max =	0.00927
# cells reinf @ opng =	2	As =	0.31 in2
First cell reinforced? =	No	Opng Wall Wt =	0 psf

LOADING

Lateral Loading

		plf	Ms ('lb)
Ultimate Wind Pressure =	10.0 psf	40.0	1,125
Seismic Factor =	0.034	7.5	210
Use this wind moment instead :			0
Use this seismic moment instead :			0

Vertical Loading

Sds = 0.117

Uniform Vertical Loading		eccentricity	lb	Ms ('lb)
Full ht wall wt =	413 plf	0.000 in	1,650	0
Dead Load =	270 plf	1.000 in	1,080	45
Floor Live Load =	0 plf	1.000 in	0	0
Roof Live Load =	450 plf	1.000 in	1,800	75
Roof Snow Load =	0 plf	1.000 in	0	0

Concentrated Vertical Loading		eccentricity		
Opening wt midht =	0.0 k	0.000 in	0	0
Dead Load =	0.0 k	2.000 in	0	0
Floor Live Load =	0.0 k	2.000 in	0	0
Roof Live Load =	0.0 k	2.000 in	0	0
Roof Snow Load =	0.0 k	2.000 in	0	0

Garage, assembly or LL>100psf: ○   Roofs that don't shed snow: ◉
All Others: ◉   All Others: ○

TYPICAL WALL REINFORCING RESULTS
LRFD Combinations

Note: Only the controlling Lr or S load combination is displayed.

	1.4D	1.2D +1.6L +0.5Lr	1.2D +0.5L +1.6Lr	1.2D +0.5W +1.6Lr	1.2D +1.0W +0.5L +0.5Lr	(1.2+.2Sds)D +1.0E +0.5L +0.7S	0.9D +1.0W	(0.9-.2Sds)D +1.0E	
Mua ('lb) =	63	92	174	737	1,217	265	1,166	250	
Pu (lb) =	3,822	4,176	6,156	6,156	4,176	3,340	2,457	2,393	
Pu /Ag (psi) =	23.5	25.7	37.8	37.8	25.7	20.5	15.1	14.7	COMPRESSION OKAY
Ase (in2) =	0.371	0.377	0.410	0.410	0.377	0.363	0.348	0.347	
c (in) =	0.362	0.368	0.400	0.400	0.368	0.354	0.340	0.339	
Icr (in4) =	71.9	72.8	77.9	77.9	72.8	70.6	68.2	68.1	
a (in) =	0.295	0.300	0.329	0.329	0.300	0.288	0.275	0.274	
δu (in) =	0.001	0.001	0.003	0.012	0.020	0.004	0.019	0.004	
Mu ('lb) =	63	92	175	743	1,223	267	1,169	251	
Capacity: φMn ('lb) =	6,230	6,334	6,911	6,911	6,334	6,089	5,829	5,810	

STRENGTH OKAY

DEFLECTION
Service Load Combinations

	D	D +L or +Lr	D +Lr or +S	D +0.6W	D+0.45W +0.75L +0.75Lr	(1+.105Sds)D +0.525E +0.75(L+S)	0.6D +0.6W	(0.6-.14Sds)D +0.6W +0.7E	DEFLECTION OKAY
Msa ('lb) =	45	120	120	720	608	702	702	174	
Ps (lb) =	2,730	4,530	4,530	2,730	4,080	1,638	1,638	1,604	
M ('lb) =	45	121	121	723	611	704	704	174	
δs (in) =	0.001	0.002	0.002	0.012	0.010	0.011	0.011	0.003	
Defl Ratio =	L / 9999	L / 9999	L / 9999	L / 9999	L / 9999	L / 9999	L / 9999	L / 9999	

Company
Address
City
Phone
Other

JOB TITLE ....

JOB NO. .... SHEET NO. ....

CALCULATED BY .... DATE ....

CHECKED BY .... DATE ....

CMU16 Ver 2018.02.04   **CMU SLENDER WALL**

DESCRIPTION: Ext. wall section

PANEL PROPERTIES

Wall Height: h =	15.00 ft	f'm =	2,000 psi
Parapet Height =	3.00 ft	fy =	60,000 psi
Nominal Thickness (t) =	8.0 in		
Running Bond?	Yes	ungrouted fr =	63.0 psi
All cells grouted?	No	interpolated fr =	79.7 psi
Block Weight =	Normal Wt	Em factor =	900
Grout Weight =	140 psi	Deflection limit =	0.0070 h
		Min Defl ratio =	142.9
Vert Bar Location :	Centered		
Cover if Bar Ea Face =	2.000 in	.20f'm =	400.0 psi
Bar Depth: d =	3.813 in	Em =	1800.0 ksi
		n =	16.11
Vertical Bar Size =	# 5	Wall Weight =	55 psf
Typical Bar Spacing =	48.0 in	Tributary width =	4.00 ft
As =	0.077 in2/ft	Strip Width Properties	
		b =	48.0 in

Opening Properties

No Opening

EL Top of Opening =	0.00 ft	
EL Bottom Opening =	0.00 ft	
Opening Material Wt =	10.0 psf	
# cells reinf @ opng =	2	
First cell reinforced? =	No	

Strip Width Properties	
Ag =	162.8 in2
Sg =	348.4 in3
Ig =	1406.8 in4
Mcr =	2,313 'lb
ρ =	0.00168
ρ max =	0.00893
As =	0.31 in2
Opng Wall Wt =	0 psf

LOADING

Lateral Loading

		plf	Ms ('lb)
Ultimate Wind Pressure =	24.7 psf	98.8	2,779
Seismic Factor =	0.034	7.5	210
Use this wind moment instead :			0
Use this seismic moment instead :			0

Vertical Loading

Sds = 0.117

Uniform Vertical Loading		eccentricity	lb	Ms ('lb)
Full ht wall wt =	578 plf	0.000 in	2,310	0
Dead Load =	180 plf	2.313 in	720	69
Floor Live Load =	0 plf	2.313 in	0	0
Roof Live Load =	300 plf	2.313 in	1,200	116
Roof Snow Load =	0 plf	2.313 in	0	0

Concentrated Vertical Loading		eccentricity		
Opening wt midht =	0.0 k	0.000 in	0	0
Dead Load =	3.3 k	6.313 in	3,300	868
Floor Live Load =	0.0 k	6.313 in	0	0
Roof Live Load =	5.5 k	6.313 in	5,500	1,447
Roof Snow Load =	0.0 k	6.313 in	0	0

Garage, assembly or LL>100psf: ○   Roofs that don't shed snow: ◉
All Others: ◉   All Others: ○

TYPICAL WALL REINFORCING RESULTS
LRFD Combinations

Note: Only the controlling Lr or S load combination is displayed.

	1.4D	1.2D +1.6L +0.5Lr	1.2D +0.5L +1.6Lr	1.2D +0.5W +1.6Lr	1.2D +1.0W +0.5L +0.5Lr	(1.2+.2Sds)D +1.0E +0.5L +0.7S	0.9D +1.0W	(0.9-.2Sds)D +1.0E	
Mua ('lb) =	1,312	1,906	3,624	5,014	4,685	1,357	3,622	1,032	
Pu (lb) =	8,862	10,946	18,316	18,316	10,946	7,744	5,697	5,549	
Pu /Ag (psi) =	54.4	67.2	112.5	112.5	67.2	47.6	35.0	34.1	COMPRESSION OKAY
Ase (in2) =	0.455	0.489	0.612	0.612	0.489	0.436	0.402	0.399	
c (in) =	0.444	0.478	0.598	0.598	0.478	0.426	0.393	0.390	
Icr (in4) =	84.5	89.4	105.4	105.4	89.4	81.8	76.7	76.3	
a (in) =	0.368	0.398	0.505	0.505	0.398	0.352	0.322	0.320	
δu (in) =	0.021	0.031	0.470	0.911	0.822	0.022	0.489	0.017	
Mu ('lb) =	1,328	1,934	4,342	6,404	5,435	1,371	3,855	1,040	
Capacity: φMn ('lb) =	7,692	8,288	10,352	10,352	8,288	7,371	6,778	6,735	

STRENGTH OKAY

DEFLECTION
Service Load Combinations

	D	D +L or +Lr	D +Lr or +S	D +0.6W	D+0.45W +0.75L +0.75Lr	(1+.105Sds)D +0.525E +0.75(L+S)	0.6D +0.6W	(0.6-.14Sds)D +0.7E	DEFLECTION OKAY
Msa ('lb) =	937	2,500	2,500	2,605	3,359	2,230	2,230	698	
Ps (lb) =	6,330	13,030	13,030	6,330	11,355	3,798	3,798	3,720	
M ('lb) =	945	2,625	2,608	2,664	3,732	2,241	2,241	702	
δs (in) =	0.015	0.115	0.100	0.112	0.394	0.036	0.036	0.011	
Defl Ratio =	L / 9999	L / 1558	L / 1799	L / 1609	L / 457	L / 5022	L / 5022	L / 9999	

Company
Address
City
Phone
Other

JOB TITLE ....
JOB NO. .... SHEET NO. ....
CALCULATED BY .... DATE ....
CHECKED BY .... DATE ....

CMU16 Ver 2018.02.04

CMU SLENDER WALL

DESCRIPTION: Ext. wall 12' opening

PANEL PROPERTIES

Wall Height: h =	15.00 ft	f'm =	2,000 psi	
Parapet Height =	3.00 ft	fy =	60,000 psi	
Nominal Thickness (t) =	8.0 in			
Running Bond?	Yes	ungrouted fr =	63.0 psi	
All cells grouted?	No	interpolated fr =	79.7 psi	
Block Weight =	Normal Wt	Em factor =	900	
Grout Weight =	140 psi	Deflection limit =	0.0070 h	
		Min Defl ratio =	142.9	
Vert Bar Location :	Centered			
Cover if Bar Ea Face =	2.000 in	.20f'm =	400.0 psi	
Bar Depth: d =	3.813 in	Em =	1800.0 ksi	
		n =	16.11	
Vertical Bar Size =	# 5	Wall Weight =	55 psf	
Typical Bar Spacing =	48.0 in	Tributary width =	9.00 ft	
As =	0.077 in2/ft	**Strip Width Properties**		
		b =	36.0 in	
Opening Properties		Ag =	172.0 in2	
Single Opening Width =	12.00 ft	Sg =	290.1 in3	
		Ig =	1147.7 in4	
EL Top of Opening =	10.00 ft	Mcr =	1,926 'lb	
EL Bottom Opening =	0.00 ft	ρ =	0.00447	
Opening Material Wt =	10.0 psf	ρ max =	0.00873	
# cells reinf @ opng =	2	As =	0.61 in2	
First cell reinforced? =	Yes	Opng Wall Wt =	65 psf	

LOADING

Lateral Loading

		plf	Ms ('lb)
Ultimate Wind Pressure =	24.7 psf	222.3	6,252
Seismic Factor =	0.034	11.7	329
Use this wind moment instead :			0
Use this seismic moment instead :			0

Vertical Loading

Sds = 0.117

Uniform Vertical Loading		eccentricity	lb	Ms ('lb)
Full ht wall wt =	679 plf	0.000 in	2,037	0
Dead Load =	180 plf	2.313 in	1,620	156
Floor Live Load =	0 plf	2.313 in	0	0
Roof Live Load =	300 plf	2.313 in	2,700	260
Roof Snow Load =	0 plf	2.313 in	0	0

Concentrated Vertical Loading		eccentricity		
Opening wt midht =	2.8 k	0.000 in	2,790	0
Dead Load =	0.0 k	2.000 in	0	0
Floor Live Load =	0.0 k	2.000 in	0	0
Roof Live Load =	0.0 k	2.000 in	0	0
Roof Snow Load =	0.0 k	2.000 in	0	0

Garage, assembly or LL>100psf: ○   Roofs that don't shed snow: ◉
All Others: ◉   All Others: ○

WALL OPENING REINFORCING RESULTS
LRFD Combinations

Note: Only the controlling Lr or S load combination is displayed.

	1.4D	1.2D +1.6L +0.5Lr	1.2D +0.5L +1.6Lr	1.2D +0.5W +1.6Lr	1.2D +1.0W +0.5L +0.5Lr	(1.2+.2Sds)D +1.0E +0.5L +0.7S	0.9D +1.0W	(0.9-.2Sds)D +1.0E	
Mua ('lb) =	219	317	604	3,730	6,570	520	6,393	466	
Pu (lb) =	9,026	9,086	12,056	12,056	9,086	7,887	5,802	5,651	
Pu /Ag (psi) =	52.5	52.8	70.1	70.1	52.8	45.9	33.7	32.9	COMPRESSION OKAY
Ase (in2) =	0.764	0.765	0.815	0.815	0.765	0.745	0.711	0.708	
c (in) =	0.995	0.997	1.061	1.061	0.997	0.971	0.925	0.922	
Icr (in4) =	109.6	109.7	113.7	113.7	109.7	108.0	105.0	104.7	
a (in) =	0.814	0.815	0.872	0.872	0.815	0.792	0.752	0.749	
δu (in) =	0.004	0.006	0.012	0.493	1.173	0.010	1.111	0.009	
Mu ('lb) =	222	322	616	4,225	7,458	527	6,930	470	
Capacity: φMn ('lb) =	11,971	11,987	12,721	12,721	11,987	11,686	11,157	11,119	

STRENGTH OKAY

DEFLECTION
Service Load Combinations

	D	D +L or +Lr	D +Lr or +S	D +0.6W	D+0.45W +0.75L +0.75Lr	(1+.105Sds)D +0.525E +0.75(L+S)	0.6D +0.6W	(0.6-.14Sds)D +0.7E	
Msa ('lb) =	156	416	416	3,907	3,165	3,845	3,845	322	DEFLECTION OKAY
Ps (lb) =	6,447	9,147	9,147	6,447	8,472	3,868	3,868	3,789	
M ('lb) =	158	423	423	4,166	3,406	3,996	4,001	324	
δs (in) =	0.003	0.008	0.008	0.481	0.341	0.469	0.483	0.006	
Defl Ratio =	L / 9999	L / 9999	L / 9999	L / 374	L / 527	L / 384	L / 373	L / 9999	

Company
Address
City
Phone
Other

JOB TITLE

JOB NO.   SHEET NO.
CALCULATED BY   DATE
CHECKED BY   DATE
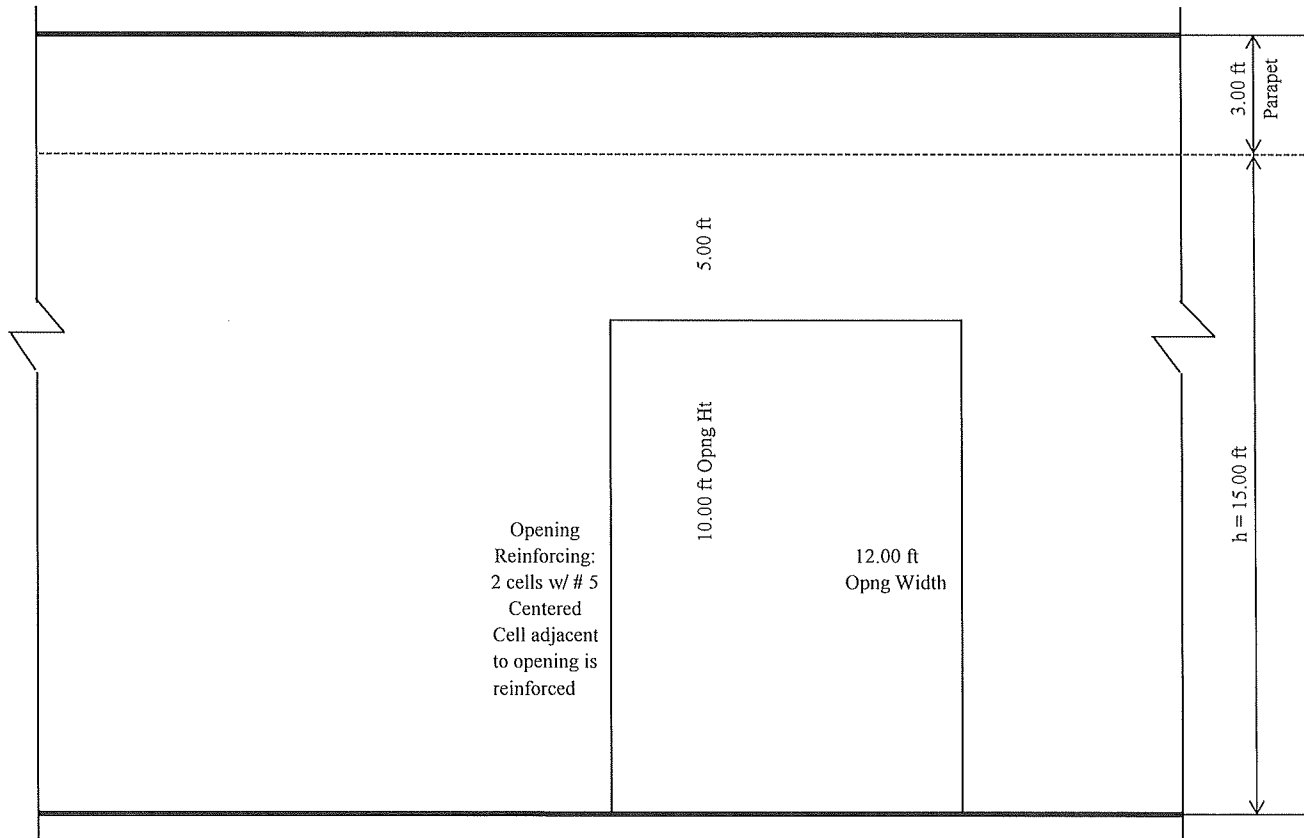
CMU16 Ver 2018.02.04   **CMU SLENDER WALL (TMS 402/ACI 530/ ASCE 5-08/11)**   www.struware.com

DESCRIPTION: Ext. wall 12' opening

3.00 ft Parapet

5.00 ft

10.00 ft Opng Ht

12.00 ft
Opng Width

h = 15.00 ft

Opening
Reinforcing:
2 cells w/ # 5
Centered
Cell adjacent
to opening is
reinforced

WALL ELEVATION

Company
Address
City
Phone
Other

JOB TITLE ....

JOB NO. .... SHEET NO. ....
CALCULATED BY .... DATE ....
CHECKED BY .... DATE ....

CMU16 Ver 2018.02.04

CMU SLENDER WALL

DESCRIPTION: Ext. wall 16' opening

PANEL PROPERTIES

Wall Height: h =	15.00 ft	f'm =	2,000 psi	
Parapet Height =	7.00 ft	fy =	60,000 psi	
Nominal Thickness (t) =	8.0 in			
Running Bond?	Yes	ungrouted fr =	63.0 psi	
All cells grouted?	No	interpolated fr =	79.7 psi	
Block Weight =	Normal Wt	Em factor =	900	
Grout Weight =	140 psi	Deflection limit =	0.0070 h	
		Min Defl ratio =	142.9	
Vert Bar Location :	Centered			
Cover if Bar Ea Face =	2.000 in	.20f'm =	400.0 psi	
Bar Depth: d =	3.813 in	Em =	1800.0 ksi	
		n =	16.11	
Vertical Bar Size =	# 5	Wall Weight =	55 psf	
Typical Bar Spacing =	48.0 in	Tributary width =	11.00 ft	
As =	0.077 in2/ft			

Strip Width Properties

b =	36.0 in
Ag =	172.0 in2
Sg =	290.1 in3
Ig =	1147.7 in4
Mcr =	1,926 'lb
ρ =	0.00447
ρ max =	0.00836
As =	0.61 in2
Opng Wall Wt =	65 psf

Opening Properties

Single Opening Width =	16.00 ft
EL Top of Opening =	10.00 ft
EL Bottom Opening =	0.00 ft
Opening Material Wt =	10.0 psf
# cells reinf @ opng =	2
First cell reinforced? =	Yes

LOADING

Lateral Loading

		plf	Ms ('lb)
Ultimate Wind Pressure =	24.7 psf	271.7	7,642
Seismic Factor =	0.034	13.4	377
Use this wind moment instead :			0
Use this seismic moment instead :			0

Vertical Loading

Sds = 0.117

Uniform Vertical Loading		eccentricity	lb	Ms ('lb)
Full ht wall wt =	938 plf	0.000 in	2,813	0
Dead Load =	100 plf	7.625 in	1,100	349
Floor Live Load =	0 plf	7.625 in	0	0
Roof Live Load =	100 plf	7.625 in	1,100	349
Roof Snow Load =	0 plf	7.625 in	0	0

Concentrated Vertical Loading		eccentricity		
Opening wt midht =	5.5 k	0.000 in	5,480	0
Dead Load =	0.0 k	2.000 in	0	0
Floor Live Load =	0.0 k	2.000 in	0	0
Roof Live Load =	0.0 k	2.000 in	0	0
Roof Snow Load =	0.0 k	2.000 in	0	0

Garage, assembly or LL>100psf: ○   Roofs that don't shed snow: ◉
All Others: ◉   All Others: ○

WALL OPENING REINFORCING RESULTS

LRFD Combinations

Note: Only the controlling Lr or S load combination is displayed.

	1.4D	1.2D +1.6L +0.5Lr	1.2D +0.5L +1.6Lr	1.2D +0.5W +1.6Lr	1.2D +1.0W +0.5L +0.5Lr	(1.2+.2Sds)D +1.0E +0.5L +0.7S	0.9D +1.0W	(0.9-.2Sds)D +1.0E	
Mua ('lb) =	489	594	979	4,799	8,236	804	7,956	683	
Pu (lb) =	13,150	11,822	13,032	13,032	11,822	11,491	8,454	8,234	
Pu /Ag (psi) =	76.5	68.7	75.8	75.8	68.7	66.8	49.1	47.9	COMPRESSION OKAY
Ase (in2) =	0.833	0.811	0.831	0.831	0.811	0.806	0.755	0.751	
c (in) =	1.085	1.056	1.082	1.082	1.056	1.049	0.983	0.978	
Icr (in4) =	115.2	113.4	115.0	115.0	113.4	113.0	108.8	108.5	
a (in) =	0.893	0.868	0.891	0.891	0.868	0.861	0.803	0.798	
δu (in) =	0.010	0.012	0.020	0.762	1.603	0.016	1.504	0.014	
Mu ('lb) =	500	606	1,000	5,626	9,815	820	9,016	692	
Capacity: φMn ('lb) =	12,988	12,664	12,960	12,960	12,664	12,583	11,828	11,773	

STRENGTH OKAY

DEFLECTION

Service Load Combinations

	D	D +L or +Lr	D +Lr or +S	D +0.6W	D+0.45W +0.75L +0.75Lr	(1+.105Sds)D +0.525E +0.75(L+S)	0.6D +0.6W	(0.6-.14Sds)D +0.7E	
Msa ('lb) =	349	699	699	4,934	4,050	4,795	4,795	469	DEFLECTION OKAY
Ps (lb) =	9,393	10,493	10,493	9,393	10,218	5,636	5,636	5,520	
M ('lb) =	355	711	711	5,513	4,521	5,110	5,123	473	
δs (in) =	0.007	0.014	0.014	0.739	0.553	0.672	0.699	0.009	
Defl Ratio =	L / 9999	L / 9999	L / 9999	L / 243	L / 326	L / 268	L / 257	L / 9999	

Company
Address
City
Phone
Other
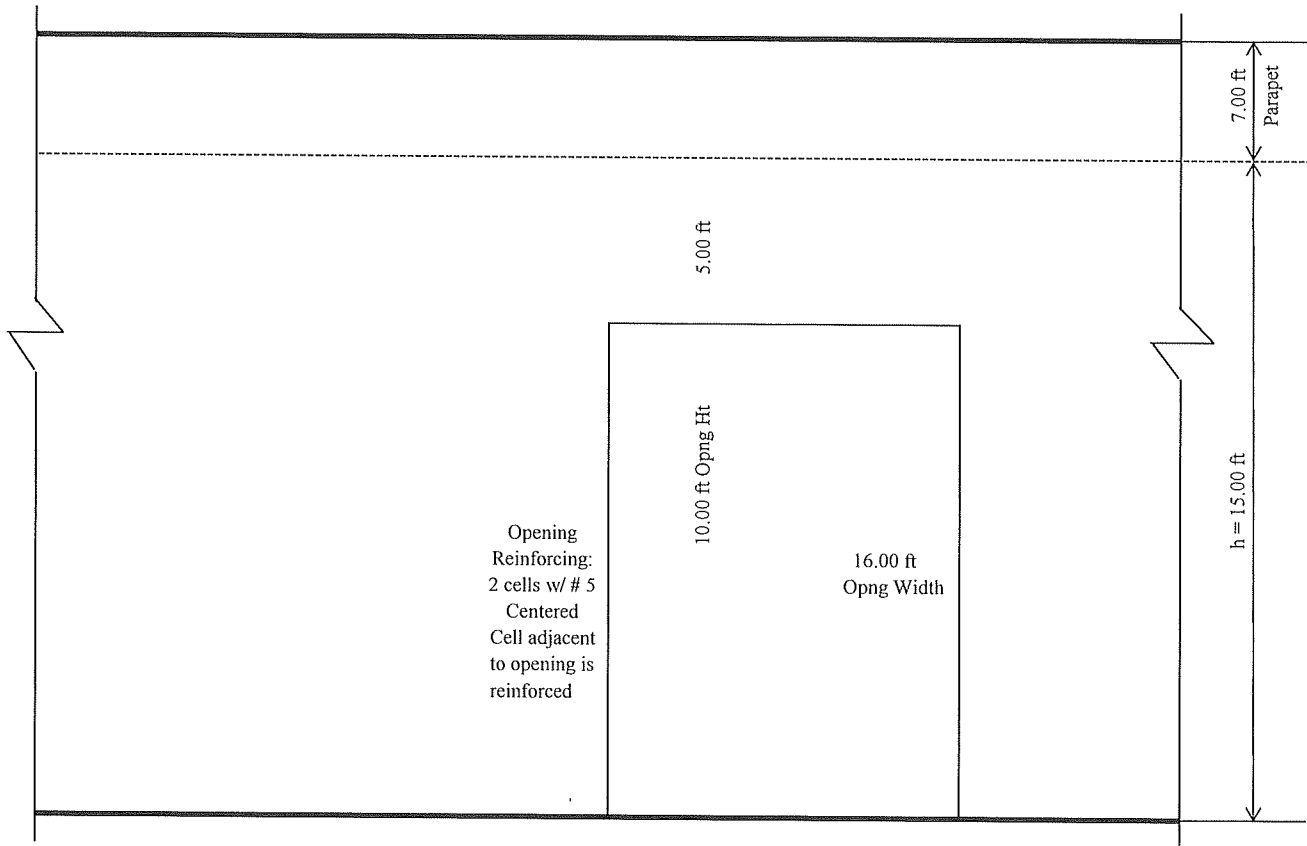
JOB TITLE

JOB NO.   SHEET NO.
CALCULATED BY   DATE
CHECKED BY   DATE

CMU16 Ver 2018.02.04   **CMU SLENDER WALL (TMS 402/ACI 530/ ASCE 5-08/11)**   www.struware.com

DESCRIPTION: Ext. wall 16' opening

7.00 ft Parapet

5.00 ft

10.00 ft Opng Ht

h = 15.00 ft

Opening Reinforcing:
2 cells w/ # 5
Centered
Cell adjacent
to opening is
reinforced

16.00 ft
Opng Width

WALL ELEVATION

Footings

Worst case Ftg

Int. Ftg   Roof $= \left(\frac{2'}{2} + \frac{24'}{2}\right)(40psf) = 720plf$

Wall $20'(55psf) = \underline{1210plf}$

$1930plf$

$\dfrac{1930plf}{1.5psf(0.8)} = 1.608'$ use $2'w \times 8"$ Deep Thickened Slab

$Rein = 24(8)(0.0018) = 0.35 \, in^2$ $2\#4 \, cont.$

Ext. Ftg

Roof $= \left(\frac{24'}{2}\right)(40) = 480plf$

Wall $20'(68) = \underline{1496plf}$

$1976 \, plf$

$\dfrac{1976}{1500(0.8)} = 1.65'$ use

$2'w \times 3.5'$ Deep

$Reinf = 2(3.5)(144)(0.0018) = 1.82 \, in^2$

$3\#5 \, cont.$

Check Flg @ Int. Post

Post Load = 8.5K
CMU Load to 6.5K
Misc.   2.0K
            17K

$\dfrac{17K}{1.5KSF} = 11.33 SF$

$11.33\%2' = 5.67' Long$ check in
                            Evercalc.

General Footing

DESCRIPTION: interior strip footing for point load

Code References

LOAD COMBINATIONS USED: IBC 2018

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	No
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	=	1.50 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	=	1.0 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	=	2.670 ft
Length parallel to Z-Z Axis	=	6.0 ft
Footing Thickness	=	12.0 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
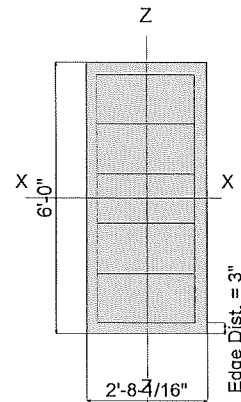
Reinforcing

Bars parallel to X-X Axis

Number of Bars	=	6.0
Reinforcing Bar Size	=	# 5

Bars parallel to Z-Z Axis

Number of Bars	=	3.0
Reinforcing Bar Size	=	# 5

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation

Bars along X-X Axis

# Bars required within zone	61.6 %
# Bars required on each side of zone	38.4 %
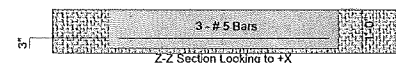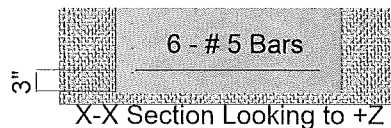
Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	14.0	3.0						k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Project File: Tiger Moline.ec6

| LIC# : KW-06015690, Build:20.22.3.31 | CROCKETT ENGINEERING CONSULTANTS | (c) ENERCALC INC 1983-2022 |

DESCRIPTION: interior strip footing for point load

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.7073	Soil Bearing	1.061 ksf	1.50 ksf	+D+Lr 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	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.09904	Z Flexure (+X)	1.202 k-ft/ft	12.131 k-ft/ft	+1.20D+1.60Lr
PASS	0.09904	Z Flexure (-X)	1.202 k-ft/ft	12.131 k-ft/ft	+1.20D+1.60Lr
PASS	0.4471	X Flexure (+Z)	6.067 k-ft/ft	13.571 k-ft/ft	+1.20D+1.60Lr
PASS	0.4471	X Flexure (-Z)	6.067 k-ft/ft	13.571 k-ft/ft	+1.20D+1.60Lr
PASS	0.08926	1-way Shear (+X)	7.333 psi	82.158 psi	+1.20D+1.60Lr
PASS	0.08926	1-way Shear (-X)	7.333 psi	82.158 psi	+1.20D+1.60Lr
PASS	0.3373	1-way Shear (+Z)	27.715 psi	82.158 psi	+1.20D+1.60Lr
PASS	0.3465	1-way Shear (-Z)	28.464 psi	82.158 psi	+1.20D+1.60Lr
PASS	0.3921	2-way Punching	64.427 psi	164.317 psi	+1.20D+1.60Lr

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	1.50	n/a	0.0	0.8739	0.8739	n/a	n/a	0.583
X-X, +D+Lr	1.50	n/a	0.0	1.061	1.061	n/a	n/a	0.707
X-X, +D+0.750Lr	1.50	n/a	0.0	1.014	1.014	n/a	n/a	0.676
X-X, +0.60D	1.50	n/a	0.0	0.5243	0.5243	n/a	n/a	0.350
Z-Z, D Only	1.50	0.0	n/a	n/a	n/a	0.8739	0.8739	0.583
Z-Z, +D+Lr	1.50	0.0	n/a	n/a	n/a	1.061	1.061	0.707
Z-Z, +D+0.750Lr	1.50	0.0	n/a	n/a	n/a	1.014	1.014	0.676
Z-Z, +0.60D	1.50	0.0	n/a	n/a	n/a	0.5243	0.5243	0.350

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	5.506	+Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
X-X, +1.40D	5.506	-Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
X-X, +1.20D+0.50Lr	5.140	+Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
X-X, +1.20D+0.50Lr	5.140	-Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
X-X, +1.20D	4.719	+Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
X-X, +1.20D	4.719	-Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
X-X, +1.20D+1.60Lr	6.067	+Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
X-X, +1.20D+1.60Lr	6.067	-Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
X-X, +0.90D	3.539	+Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
X-X, +0.90D	3.539	-Z	Bottom	0.2592	AsMin	0.3483	13.571	OK
Z-Z, +1.40D	1.090	-X	Bottom	0.2592	AsMin	0.310	12.131	OK
Z-Z, +1.40D	1.090	+X	Bottom	0.2592	AsMin	0.310	12.131	OK
Z-Z, +1.20D+0.50Lr	1.018	-X	Bottom	0.2592	AsMin	0.310	12.131	OK
Z-Z, +1.20D+0.50Lr	1.018	+X	Bottom	0.2592	AsMin	0.310	12.131	OK
Z-Z, +1.20D	0.9345	-X	Bottom	0.2592	AsMin	0.310	12.131	OK
Z-Z, +1.20D	0.9345	+X	Bottom	0.2592	AsMin	0.310	12.131	OK
Z-Z, +1.20D+1.60Lr	1.202	-X	Bottom	0.2592	AsMin	0.310	12.131	OK

General Footing

DESCRIPTION: interior strip footing for point load

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+1.60Lr	1.202	+X	Bottom	0.2592	AsMin	0.310	12.131	OK
Z-Z, +0.90D	0.7009	-X	Bottom	0.2592	AsMin	0.310	12.131	OK
Z-Z, +0.90D	0.7009	+X	Bottom	0.2592	AsMin	0.310	12.131	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	6.65 psi	6.65 psi	25.83 psi	25.15 psi	25.83 psi	82.16 psi	0.31	OK
+1.20D+0.50Lr	6.21 psi	6.21 psi	24.12 psi	23.48 psi	24.12 psi	82.16 psi	0.29	OK
+1.20D	5.70 psi	5.70 psi	22.14 psi	21.56 psi	22.14 psi	82.16 psi	0.27	OK
+1.20D+1.60Lr	7.33 psi	7.33 psi	28.46 psi	27.72 psi	28.46 psi	82.16 psi	0.35	OK
+0.90D	4.28 psi	4.28 psi	16.60 psi	16.17 psi	16.60 psi	82.16 psi	0.20	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	58.46 psi	164.32 psi	0.3558	OK
+1.20D+0.50Lr	54.58 psi	164.32 psi	0.3322	OK
+1.20D	50.11 psi	164.32 psi	0.305	OK
+1.20D+1.60Lr	64.43 psi	164.32 psi	0.3921	OK
+0.90D	37.58 psi	164.32 psi	0.2287	OK

Worst Case Conn.

Shear Loads $\qquad$ 1228 plf (0.6) = 737 plf

Conn CMU to Roof
27.8 psf (0.6) = 16.68 psf

R roof = 280 plf
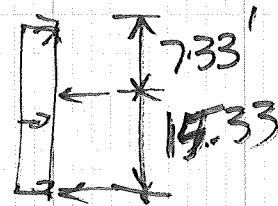
$\frac{1}{2}$"∅ Titen HD · $\qquad$ w/ 3½" Embed

has 690# tension
1385# Shear

Capacity is tight

$\frac{5}{8}$"∅ Titen HD w/ 4½" Embed has 1060# tension
2085# Shear

Ok By Inspection