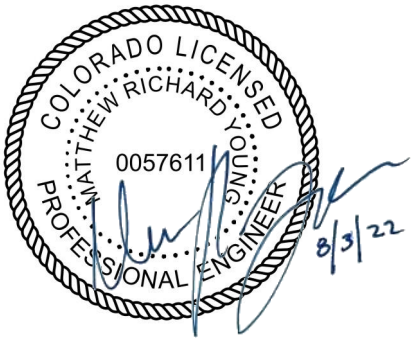
STRUCTURAL CALCULATIONS

FOR

Murphy Oil - Parker, CO

Parker, CO

Frey-Moss Structures
1801 Rockdale Industrial Blvd
Conyers, GA 30012
FMS Job No G20MU00117



BAKER, INGRAM, & ASSOCIATES
STRUCTURAL ENGINEERS
TWO WHITE HORSE PIKE HADDON HEIGHTS, NJ 08035
(856) 310-1491 fax: (856) 310-1829

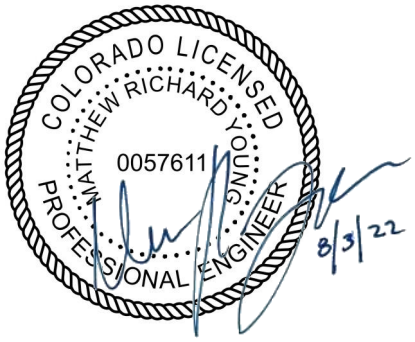


Table of Contents

Code Search

Code: International Building Code 2018

Occupancy:

Occupancy Group = M Mercantile

Risk Category & Importance Factors:

Risk Category =	II
Wind factor =	1.00
Snow factor =	1.00
Seismic factor =	1.00

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	0.00 / 12	0.0 deg
Building length (L)	64.8 ft	
Least width (B)	43.8 ft	
Mean Roof Ht (h)	12.1 ft	
Parapet ht above grd	18.5 ft	
Minimum parapet ht	6.4 ft	

Live Loads:

Roof

0 to 200 sf:	20 psf
200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
over 600 sf:	12 psf

Floor:

Typical Floor	100 psf
Partitions	N/A

Baker, Ingram & Associates
2 White Horse Pike
Haddon Heights, NJ
(856) 310-1491

JOB TITLE	Murphy Oil - Parker, CO 2,824 sf Modular Convenience Store		
JOB NO.	H13157-20	**SHEET NO.**	
CALCULATED BY	GAS	**DATE**	10/8/20
CHECKED BY		**DATE**	

Wind Loads : ASCE 7- 16

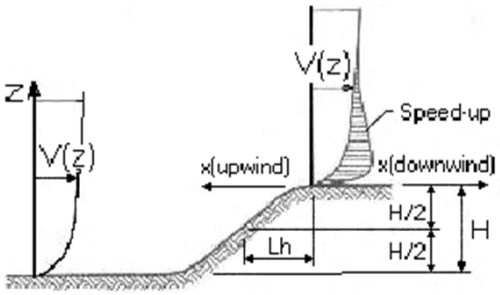
Ultimate Wind Speed	115 mph
Nominal Wind Speed	89.1 mph
Risk Category	II
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.866
Kh case 2	0.866
Type of roof	Monoslope

Topographic Factor (Kzt)

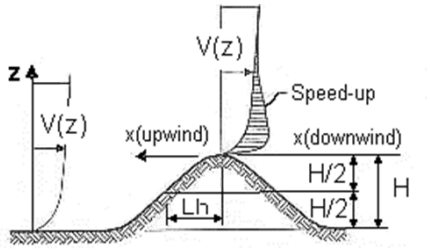
Topography		Flat
Hill Height	(H)	80.0 ft
Half Hill Length (Lh)		100.0 ft
Actual H/Lh	=	0.80
Use H/Lh	=	0.50
Modified Lh	=	160.0 ft
From top of crest: x =		50.0 ft
Bldg up/down wind?		downwind

H/Lh= 0.50 K_1 = 0.000
x/Lh = 0.31 K_2 = 0.792
z/Lh = 0.10 K_3 = 1.000

At Mean Roof Ht:
Kzt = $(1+K_1K_2K_3)^2$ = 1.00



ESCARPMENT



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h = 16.5 ft
B = 43.8 ft
/z (0.6h) = 15.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 0.38 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure

ē =	0.20	
ℓ =	500 ft	
z_{min} =	15 ft	
c =	0.20	
g_Q, g_v =	3.4	
L_z =	427.1 ft	
Q =	0.92	
I_z =	0.23	
G =	0.88	use G = 0.85

Flexible or Dynamically Sensitive Structure

34 ncy (η_1) =	0.0 Hz			
Damping ratio (β) =	0			
/b =	0.65			
/α =	0.15			
Vz =	97.1			
N_1 =	0.00			
R_n =	0.000			
R_h =	28.282	η =	0.000	h = 16.5 ft
R_B =	28.282	η =	0.000	
R_L =	28.282	η =	0.000	
g_R =	0.000			
R =	0.000			
Gf =	0.000			

Enclosure Classification

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

Kz = Kh (case 1) = 0.85
Base pressure (qh) = **24.4 psf**
GCpi = +/-0.18

Edge Strip (a) = 4.4 ft
End Zone (2a) = 8.8 ft
Zone 2 length = 21.9 ft

Wind Pressure Coefficients

	CASE A				CASE B		
		θ = 0 deg					
Surface	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 = 38.3 psf (GCpn = +1.5)
Leeward parapet = -25.5 psf (GCpn = -1.0)

Windward roof overhangs = 17.1 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

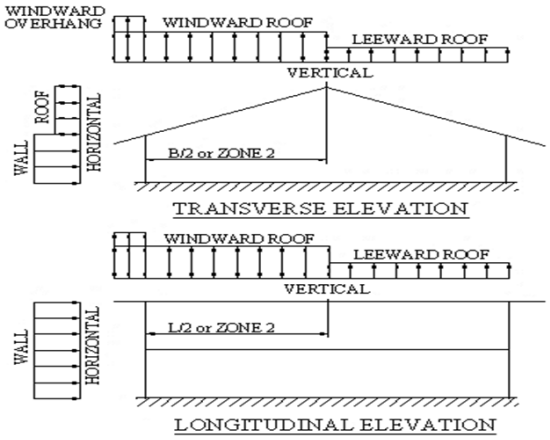
Interior Zone: Wall 16.9 psf
Roof -7.8 psf **
End Zone: Wall 25.4 psf
Roof -13.2 psf **

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.



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Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) =	0.87	h =	16.5 ft	0.2h = 3.3 ft
Base pressure (qh) =	**24.9 psf**	0.6h =	9.9 ft	
Minimum parapet ht =	0.0 ft	GCpi =	+/-0.18	
Roof Angle (θ) =	0.0 deg	qi = qh =	24.9 psf	
Type of roof =	Monoslope			

Roof

	Surface Pressure (psf)								User input	
Area	10 sf	20 sf	50 sf	100 sf	200 sf	350 sf	500 sf	1000 sf	10 sf	50 sf
Negative Zone 1	-46.9	-43.8	-39.7	-36.6	-33.5	-31.0	-29.4	-29.4	-46.9	-39.7
Negative Zone 1'	-26.9	-26.9	-26.9	-26.9	-23.2	-20.1	-18.2	-16.0	-26.9	-26.9
Negative Zone 2	-61.8	-57.8	-52.6	-48.6	-44.6	-41.4	-39.4	-39.4	-61.8	-52.6
Negative Zone 3	-84.2	-76.3	-65.8	-57.8	-49.9	-43.5	-39.4	-39.4	-84.2	-65.8
Positive All Zones	16	16	16	16	16.0	16.0	16.0	16.0	16.0	16.0
Overhang Zone 1&1'	-42.4	-41.6	-40.6	-39.9	-33.4	-28.2	-24.9	-24.9	-42.4	-40.6
Overhang Zone 2	-57.3	-52	-45	-39.7	-34.4	-30.1	-27.4	-27.4	-57.3	-45.0
Overhang Zone 3	-79.8	-70.5	-58.2	-49	-39.7	-32.2	-27.4	-27.4	-79.8	-58.2

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 4.5 psf)

Parapet

qp = 0.0 psf

		Surface Pressure (psf)						User input
Solid Parapet Pressure		10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	12 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
CASE B :	Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Walls

	GCp +/- GCpi				Surface Pressure at h				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	10 sf	50 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-29.2	-25.2	-24.0	-22.4	-29.2	-26.4
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-35.9	-28.0	-25.6	-22.4	-35.9	-30.4
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	26.9	23.0	21.8	20.2	26.9	24.1

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

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Snow Loads : ASCE 7- 16

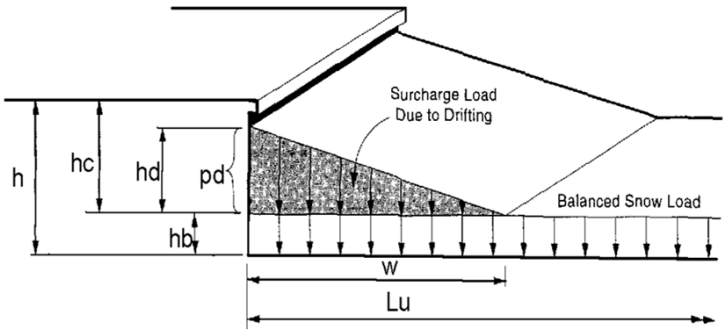
Nominal Snow Forces

Roof slope		=	0.0 deg
Horiz. eave to ridge dist (W)		=	43.8 ft
Roof length parallel to ridge (L)		=	64.8 ft
Type of Roof			Monoslope
Ground Snow Load	Pg	=	35.0 psf
Risk Category		=	II
Importance Factor	I	=	1.0
Thermal Factor	Ct	=	1.00
Exposure Factor	Ce	=	1.0
Pf = 0.7*Ce*Ct*I*Pg		=	30.0 psf
Unobstructed Slippery Surface			yes
Sloped-roof Factor	Cs	=	1.00
Balanced Snow Load		=	**30.0 psf**
Rain on Snow Surcharge Angle			0.88 deg
Code Maximum Rain Surcharge			5.0 psf
Rain on Snow Surcharge		=	0.0 psf
Ps plus rain surcharge		=	30.0 psf
Minimum Snow Load	Pm	=	20.0 psf
Uniform Roof Design Snow Load		=	**30.0 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

Upwind fetch	lu	=	64.8 ft
Projection height	h	=	5.2 ft
Snow density	g	=	19.6 pcf
Balanced snow height	hb	=	1.53 ft
	hd	=	2.37 ft
	hc	=	3.67 ft
hc/hb >0.2 = 2.4	**Therefore, design for drift**		
Drift height (hd)		=	2.37 ft
Drift width	w	=	9.47 ft
Surcharge load:	pd = γ*hd	=	**46.3 psf**
Balanced Snow load:		=	30.0 psf
			76.3 psf



Note: If bottom of projection is at least 2 feet above hb then snow drift is not required.

Windward Snow Drifts 2 - Against walls, parapets, etc

Upwind fetch	lu	=	160.0 ft
Projection height	h	=	4.0 ft
Snow density	g	=	19.6 pcf
Balanced snow height	hb	=	1.53 ft
	hd	=	3.59 ft
	hc	=	2.47 ft
hc/hb >0.2 = 1.6	**Therefore, design for drift**		
Drift height (hc)		=	2.47 ft
Drift width	w	=	19.75 ft
Surcharge load:	pd = γ*hd	=	**48.3 psf**
Balanced Snow load:		=	30.0 psf
			78.3 psf

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Seismic Loads: IBC 2018 Strength Level Forces

Risk Category : II
Importance Factor (I) : 1.00
Site Class : D

Ss (0.2 sec) = 20.10 %g
S1 (1.0 sec) = 5.60 %g

Fa =	1.600		Sms =	0.322	S_{DS} =	0.214	Design Category =	B
Fv =	2.400		Sm1 =	0.134	S_{D1} =	0.090	Design Category =	B

Seismic Design Category = **B**

Redundancy Coefficient ρ = 1.30
Number of Stories: 1
Structure Type: Light Frame
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: Yes
Building System: **Building Frame Systems**
Seismic resisting system: **Light frame walls with shear panels-all other materials**
System Structural Height Limit: **Height not limited**
Actual Structural Height (hn) = 16.5 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 2.5
Over-Strength Factor (Ωo) = 2
Deflection Amplification Factor (Cd) = 2.5
S_{DS} = 0.214
S_{D1} = 0.090

Seismic Load Effect (E) = Eh +/-Ev = ρ Q_E +/- 0.2S_{DS} D = 1.3Qe +/- 0.000D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = Ωo Q_E +/- 0.2S_{DS} D = 2Qe +/- 0.043D D = dead load

PERMITTED ANALYTICAL PROCEDURES

Simplified Analysis - Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted

Building period coef. (C_T) =	0.020				Cu = 1.70
Approx fundamental period (Ta) =	$C_T h_n^x$ =	0.164	sec	x= 0.75	Tmax = CuTa = 0.278
User calculated fundamental period (T) =			sec		Use T = 0.164
Long Period Transition Period (TL) =	ASCE7 map =	4			
Seismic response coef. (Cs) =	S_{DS}I/R =	0.086			
need not exceed Cs =	Sd1 I /RT =	0.219			
but not less than Cs =		0.010			
USE Cs =		0.086			

Design Base Shear V = 0.086W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures

Allowable story drift Δa = 0.020hsx where hsx is the story height below level x

ORDINANCE NO. 4.92.5, Series of 2018

TITLE: A BILL FOR AN ORDINANCE TO AMEND SECTIONS 11.03.020, 11.03.050, 11.03.060 AND 11.03.080 OF THE PARKER MUNICIPAL CODE CONCERNING THE PARKER BUILDING CODE

NOW, THEREFORE, THE TOWN COUNCIL OF THE TOWN OF PARKER, COLORADO, ORDAINS:

Section 1. Section 11.03.020 of the Parker Municipal Code is amended to read as follows:

11.03.020 International Building Code adopted.

Pursuant to Section 7.7 of the Town of Parker Home Rule Charter, the International Building Code, 2018 Edition, as published by the International Code Council (ICC), 500 New Jersey Avenue, NW, 6th Floor, Washington, DC 20001 is adopted by reference and incorporated into this Chapter as though fully set forth herein. Except as otherwise provided hereafter, such code is adopted in full, including the outline of contents, index and appendices contained therein.

Section 2. Section 11.03.050 of the Parker Municipal Code is repealed in its entirety and readopted to read as follows:

11.03.050 Amendments.

The International Building Code, as adopted by this Chapter, is amended as follows (section numbers correspond with those in the International Building Code):

(1) Delete all Appendix Chapters.

(2) Amend Section 101.1, Title, to read:

"**101.1 Title**. These regulations shall be known as the Parker Building Code, hereinafter referred to as the 'code.'"

(3) Sections 101.2, 101.2.1 and 101.3 remain unchanged.

(4) The remainder of Chapter 1 of the code, entitled "Scope and Administration," is deleted in its entirety (*see* Parker Administrative Code contained in Chapter 11.01 of the Parker Municipal Code)

(5) Amend Section 1608.2, Ground snow loads, to read:

"**1608.2 Ground snow loads**. The ground snow load within the Town of Parker shall be a minimum of 30 pounds per square foot for calculating roof

drifting. Snow load for roofs shall be 30 pounds per square foot minimum, plus drifting."

(6) Amend Section1609.3, Basic wind speed, by the addition of the following:

Figure 1609.3(1) equals 110 miles per hour

Figure 1609.3(2) equals 115 miles per hour

Figure 1609.3(3) equals 120 miles per hour

Figure 1609.3(4) equals 105 miles per hour

(7) Amend Subsection 1609.4.3, Exposure categories, Exposure C, to read:

"Exposure C shall be used for the design of all structures in the Town of Parker."

(8) Amend Chapter 31 to add a new Section 3114, Manufactured housing, to read:

"**3114 Manufactured housing.** Manufactured housing constructed in accordance with standards other than those set forth in this ordinance may be erected in approved locations. The site constructed foundation, porches, decks, utilities and other functions must meet the standards set forth in this and other related ordinances. Permits may be issued for these elements of the building; however, a certificate of occupancy will not be issued, as the conformance with the Town's standards of the manufactured home is not known to the department. No additions shall be made to a manufactured home, unless said addition meets the standards set forth in this ordinance."

(9) Amend Chapter 35, ASME standards, as follows:

ASME A 18.1: Replace the 2014 Safety Standard for Platform Lifts and Stairway Chair Lifts with the 2011 Safety Standard for Platform Lifts and Stairway Chair Lifts.

The remaining ASME standards remain unchanged.

Section 3. Section 11.03.060 of the Parker Municipal Code is amended to read as follows:

11.03.060 Violation; penalty.

In addition to the penalty provisions contained in the Parker Building Code, any person who violates any of the provisions of this Chapter shall be guilty of a misdemeanor and, upon conviction thereof, shall be fined the sum of not more than four hundred ninety-nine dollars ($499.00) for each such violation. Each day that a violation continues after due notice has been served shall be deemed a separate offense.

Hazards by Location

Search Information

Address: 12045 S Parker Rd, Parker, CO 80134, USA

Coordinates: 39.50214620000001, -104.7622508

Elevation: 5862 ft

Timestamp: 2020-09-18T15:56:56.179Z

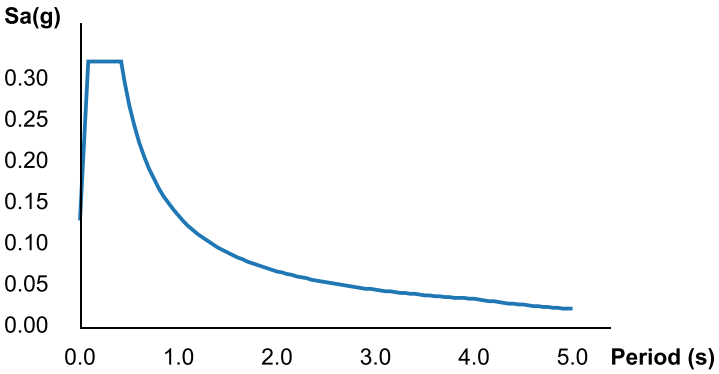
Hazard Type: Seismic

Reference Document: ASCE7-16

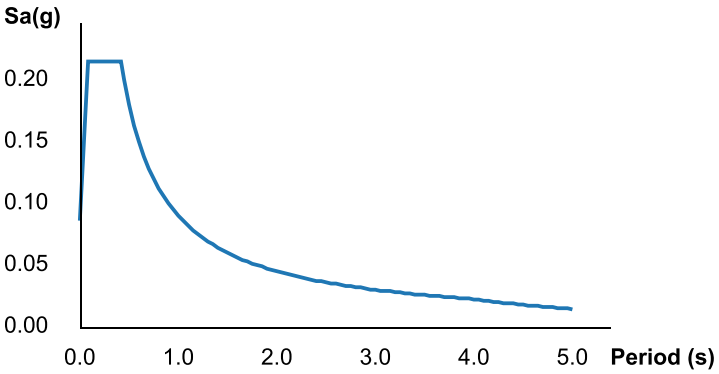
Risk Category: II

Site Class: D-default

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.201	MCE_R ground motion (period=0.2s)
S_1	0.056	MCE_R ground motion (period=1.0s)
S_{MS}	0.322	Site-modified spectral acceleration value
S_{M1}	0.135	Site-modified spectral acceleration value
S_{DS}	0.215	Numeric seismic design value at 0.2s SA
S_{D1}	0.09	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s

CR_S	0.95	Coefficient of risk (0.2s)
CR_1	0.926	Coefficient of risk (1.0s)
PGA	0.11	MCE_G peak ground acceleration
F_{PGA}	1.58	Site amplification factor at PGA
PGA_M	0.174	Site modified peak ground acceleration
T_L	4	Long-period transition period (s)
SsRT	0.201	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.212	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.056	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.061	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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STRUCTURAL ENGINEERS
TWO WHITE HORSE PIKE HADDON HEIGHTS, NJ 08035
(856) 310-1491 fax: (856) 310-1829

Roof Deck Design

Deck Properties: (FMS roof deck used as basis of design)

Gage =	20		Span =	3.50	feet (max)
Fy =	50	ksi	Width =	16	inches
Fb =	30	ksi	Sx =	0.313	in3
			Ix =	0.448	in4

Loads:

Dead =	12	psf
Live =	20	psf
Snow =	30	psf
Max drift =	76.3	psf
Wind =	16.0	psf (positive)
	-39.7	psf (suction)

Load Combinations:

DL + LL =	32.0	psf
DL + SL =	88.3	psf
DL + .75(SL+0.6WL) =	76.4	psf
DL + .75(LL+0.6WL) =	34.2	psf
0.6DL + 0.6WL =	-16.6	psf
Controlling Load =	88.3	psf

Deck Design - Flexure:

w(tl) =	117.7333	plf	M(actual) =	0.180279	ft-kip
w(ll) =	40	plf	M(allow) =	0.7825	ft-kip OK

Deck Design - Deflection:

Δ(tl) =	0.013	in	L/	3307	OK
Δ(ll) =	0.004	in	L/	9732	OK

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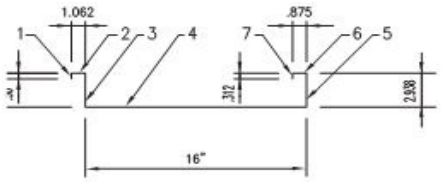
Roof Deck Properties

20 GA Ceiling/ Wall Deck - 3" Tall



Fy = 50,000
Fb = 30,000
t = .036 in

Calculation Taken from From "AISI Cold Formed Steel Design" 2002 Edition - Section 3.5 & 3.6

	POSITIVE BENDING					NEGATIVE BENDING				
Member	A	Y	AY	AY2	I	A	Y	AY	AY2	I
1	0.500	2.750	1.375	3.781	0.010	0.500	2.750	1.375	3.781	0.010
2	0.990	2.982	2.952	8.803	0.000	0.990	2.982	2.952	8.803	0.000
3	3.000	1.500	4.500	6.750	2.250	3.000	1.500	4.500	6.750	2.250
4	15.928	0.018	0.287	0.005	0.002	1.611	0.018	0.029	0.001	0.000
5	2.938	1.469	4.316	6.340	2.113	2.938	1.469	4.316	6.340	2.113
6	0.803	2.920	2.345	6.847	0.000	0.803	2.920	2.345	6.847	0.000
7	0.312	2.782	0.868	2.415	0.003	0.312	2.782	0.868	2.415	0.003
Totals	24.471		16.643	34.941	4.378	10.154		16.385	34.937	4.377

Determine Effective Width Using Equation B2.1-1 through B2.1-5
From 2001 AISI Standard with 2004 Supplement

Check Compression Members
w/t = 0.990/0.036 = 28 < 30 OK

w/t = 15.928/0.036 = 442 > 30

Calculate Effective Width of Compression Flange
b = p X w = 0.101 X 0.036 = 1.611 in

Ybar = 16.643/24.471 = 0.680
C = 3.000 - 0.680 = 2.320
I = 4.378 + 34.941 - 24.471(0.680)^2
I = 28.001 x t = 28.001 x 0.036
I = 1.008 in^4
S = 1.008 / 2.320
S = 0.435 in^3

Ybar = 16.385/10.154 = 1.614
C = 3.000 - 1.614 = 1.386
I = 4.377 + 34.937 - 10.154(1.614)^2
I = 12.874 x t = 12.874 x 0.036
I = 0.463 in^4
S = 0.463 / 1.386
S = 0.334 in^3

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Roof Framing Design

Loads:

Dead =	12	psf
Live =	20	psf
Snow =	30	psf
Max drift =	76.3	psf
Wind =	16.0	psf (positive)
	-39.7	psf (suction)

Load Combinations:

DL + LL =	32.0	psf
DL + SL =	88.3	psf
DL + .75(SL+0.6WL) =	76.4	psf
DL + .75(LL+0.6WL) =	34.2	psf
0.6DL + 0.6WL =	-16.6	psf
Controlling Load =	88.3	psf

Purlin Design:

Span =	13.00	feet	**C5x6.7**		
Trib Width =	4	feet	Sx =	2.99	in3
w(tl) =	353	plf	Ix =	7.48	in4
w(ll) =	80	plf	Fy =	36	ksi
			Fb =	21.6	ksi

Flexure:

M(actual) =	4.46	ft-kip	(from finite elements model)
M(allow) =	5.38	ft-kip	OK

Deflection:

Δ(tl) =	0.616	in	L/	253	OK	(finite elements)
Δ(ll) =	0.237	in	L/	658	OK	

Exterior Girder Design:

Span =	10.75	feet	**C5x6.7**		
Trib Width =	6.5	feet	Sx =	2.99	in3
w(tl) =	574	plf	Ix =	7.48	in4
w(ll) =	130	plf	Fy =	36	ksi
			Fb =	21.6	ksi

Flexure:

M(actual) =	5.14	ft-kip	(from finite elements)
M(allow) =	5.38	ft-kip	OK

Deflection:

Δ(tl) =	0.159	in	L/	811	OK	
Δ(ll) =	0.036	in	L/	3582	OK	

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Roof Framing Design

Interior Girder Design:

Span =	34.80	feet		**C12x30**		
Trib Width =	6.54	feet		Sx =	27	in3
w(tl) =	577	plf		Ix =	126	in4
w(ll) =	131	plf		Fy =	36	ksi
				Fb =	21.6	ksi

Flexure:

M(actual) =	45.85	ft-kip	(from finite elements model)
M(allow) =	48.60	ft-kip	OK

Deflection:

Δ(tl) =	1.340	in	L/	312	OK	(from FE)
Δ(ll) =	0.490	in	L/	852	OK	
					(using 2 span continuous)	

Interior Girder Design:

Span =	29.50	feet		**C10x30**		
Trib Width =	6.54	feet		Sx =	20.7	in3
w(tl) =	577	plf		Ix =	103	in4
w(ll) =	131	plf		Fy =	36	ksi
				Fb =	21.6	ksi

Flexure:

M(actual) =	30.40	ft-kip	(from finite elements model)
M(allow) =	37.26	ft-kip	OK

Deflection:

Δ(tl) =	1.368	in	L/	259	OK	
Δ(ll) =	0.310	in	L/	1143	OK	
					(using 2 span continuous)	

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Floor Deck Design

Deck Properties:

Gage =	22		Span =	3.50	feet (max)	
Fy =	33	ksi	Width =	12	inches	
Fb =	19.8	ksi	Sx =	0.145	in3	
			Ix =	0.154	in4	

Loads:

Dead =	30	psf
Live =	100	psf

Load Combinations:

DL + LL =	130	psf

Deck Design - Flexure:

w(tl) =	130	plf	M(actual) =	0.199	ft-kip	
w(ll) =	100	plf	M(allow) =	0.239	ft-kip	OK

Deck Design - Deflection:

Δ(tl) =	0.041	in	L/	1029	OK
Δ(ll) =	0.031	in	L/	1338	OK

ROOF DECK DATA BASE

ATTRIBUTE		TYPE B DECK (B, BI, BA, BIA)				TYPE F DECK			TYPE N DECK (NS, NI, NSA, NIA)			
Note	Gage	22	20	18	16	22	20	18	22	20	18	16
	Thickness	.0295	.0358	.0474	.0598	.0295	.0358	.0474	.0295	.0358	.0474	.0598
	Weight, psf	1.7	2.1	2.8	3.5	1.6	2.0	2.6	2.1	2.5	3.3	4.1
1	I_p, in.^{4}	0.17	0.22	0.31	0.40	0.13	0.17	0.24	0.64	0.82	1.19	1.62
1	I_n, in.^{4}	0.20	0.24	0.32	0.40	0.15	0.19	0.25	0.85	1.04	1.38	1.75
1	S_p, in.^{3}	0.19	0.25	0.34	0.44	0.13	0.16	0.22	0.37	0.49	0.68	0.88
1	S_n, in.^{3}	0.20	0.26	0.36	0.45	0.14	0.17	0.23	0.42	0.54	0.74	0.93
2	Ext.R, lbs.	450	620	1010	1860	440	610	1000	320	450	760	1410
3	Ext.R, lbs.	540	730	1160	2100	540	720	1140	390	530	870	1590
4	Int.R, lbs.	1270	1830	3120	4670	1250	1800	3070	940	1370	2370	3800
5	Int.R, lbs.	1320	1880	3200	4750	1320	1880	3190	1090	1580	2700	4020
6	V, lbs.	1920	2300	3000	3780	1970	2360	3120	2350	3390	4960	6180
7	Max.1 span	5'10"	6'8"	8'0"	9'1"	5'2"	5'11"	7'0"	11'5"	13'0"	15'8"	18'3"
8	Max.2 span	6'11"	7'10"	9'5"	10'9"	6'1"	7'0"	8'4"	13'5"	15'3"	18'5"	21'6"
9	Max. Cant.	1'11"	2'4"	2'10"	3'3"	1'2"	1'5"	1'10"	3'6"	4'0"	4'10"	5'5"
10	FM span	6'0"	6'6"	7'5"		4'11"	5'5"	6'3"				

F, Intermediate Rib Deck

Uniform Total Load / Load that Produces l/240 Deflection, psf

Span Condition	Gage	Span 4'0"	4'6"	5'0"	5'6"	6'0"	6'6"	7'0"	7'6"	8'0"	8'6"
SINGLE	22	170 / 133	134 / 94	109 / 68	90 / 51						
	20	209 / 174	165 / 122	134 / 89	111 / 67	93 / 52	79 / 41				
	18	287 / 246	227 / 173	184 / 126	152 / 95	128 / 73	109 / 57	94 / 46			
DOUBLE	22	180 / 321	143 / 226	116 / 164	96 / 124	81 / 95	69 / 75				
	20	219 / 420	173 / 295	141 / 215	117 / 162	98 / 124	84 / 98	72 / 78			
	18	296 / 593	234 / 416	190 / 303	158 / 228	133 / 176	113 / 138	98 / 111	85 / 90	75 / 74	66 / 62
TRIPLE	22	224 / 251	178 / 176	144 / 129	120 / 97	101 / 74	86 / 59				
	20	272 / 329	216 / 231	175 / 168	145 / 126	122 / 97	104 / 77	90 / 61			
	18	367 / 464	292 / 326	237 / 238	196 / 178	165 / 137	141 / 108	122 / 87	106 / 70	93 / 58	83 / 48

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Floor Framing Design

Loads:

Dead =	30	psf
Live =	100	psf

Load Combinations:

DL + LL = 130 psf

Floor Beam Design:

Span = 13.00 feet
Trib Width = 3.5 feet
w(tl) = 455 plf
w(ll) = 350 plf

W6x9

Sx = 5.56 in3
Ix = 16.4 in4
Fy = 36 ksi
Fb = 24 ksi

Flexure:

M(actual) = 9.61 ft-kip
M(allow) = 11.11 ft-kip OK

Deflection:

Δ(tl) = 0.615 in L/ 254 OK
Δ(ll) = 0.473 in L/ 330 OK - some fixity provided @ supports

Side Channel Design:

Span = 13.00 feet
Trib Width = 2 feet
w(tl) = 260 plf
w(ll) = 200 plf

C5x6.7 + L4x3x1/4

Sx = 4.82 in3 composite section
Ix = 23.0 in4 properties
Fy = 36 ksi
Fb = 21.6 ksi

Flexure:

M(actual) = 5.49 ft-kip
M(allow) = 8.67 ft-kip OK

Deflection:

Δ(tl) = 0.251 in L/ 622 OK
Δ(ll) = 0.193 in L/ 809 OK

Interior Girder Design:

Span = 14.75 feet
Trib Width = 6.5 feet
w(tl) = 845 plf
w(ll) = 650 plf

C10x15.3

Sx = 13.5 in3
Ix = 67.3 in4
Fy = 36 ksi
Fb = 21.6 ksi

Flexure:

M(actual) = 22.98 ft-kip
M(allow) = 24.30 ft-kip OK

Deflection:

Δ(tl) = 0.461 in L/ 384 OK
Δ(ll) = 0.355 in L/ 499 OK

Current Date: 10/9/2020 8:49 AM
Units system: English
File name: H:\Projects\01 Active Projects\00.0 Murphy Oil Projects\13157-2020 Murphy OII - Parker, CO (Parker and Pine)\Design\Calculations\Building\Elements\exterior floor girder (1).retx

Steel Code Check

Report: Summary - Group by member

Load conditions to be included in design :

D1=DL
D2=DL+LL
D3=DL+RL
D4=DL+SL
D5=DL+0.75LL
D6=DL+0.75RL
D7=DL+0.75SL
D8=DL+0.75LL+0.75RL
D9=DL+0.75LL+0.75SL
D10=DL+0.6WL
D11=DL+0.75LL+0.45WL+0.75RL
D12=DL+0.75LL+0.45WL+0.75SL
D13=DL+0.75LL+0.45WL
D14=DL+0.45WL+0.75RL
D15=DL+0.45WL+0.75SL
D16=DL+0.75LL+0.75SL
D17=DL+0.75SL
D18=0.6DL+0.6WL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	C 10X15.3	**1**	D12 at 100.00%	0.55	OK	Sec. F1
		2	D12 at 0.00%	**0.59**	**OK**	Sec. F1
		3	D12 at 0.00%	0.34	OK	Sec. F1
		4	D12 at 0.00%	0.17	OK	Sec. F1

Current Date: 10/9/2020 8:55 AM
Units system: English
File name: H:\Projects\01 Active Projects\00.0 Murphy Oil Projects\13157-2020 Murphy OII - Parker, CO (Parker and Pine)\Design\Calculations\Building\Elements\exterior floor girder (6).retx

Steel Code Check

Report: Summary - Group by member

Load conditions to be included in design :

D1=DL
D2=DL+LL
D3=DL+RL
D4=DL+SL
D5=DL+0.75LL
D6=DL+0.75RL
D7=DL+0.75SL
D8=DL+0.75LL+0.75RL
D9=DL+0.75LL+0.75SL
D10=DL+0.6WL
D11=DL+0.75LL+0.45WL+0.75RL
D12=DL+0.75LL+0.45WL+0.75SL
D13=DL+0.75LL+0.45WL
D14=DL+0.45WL+0.75RL
D15=DL+0.45WL+0.75SL
D16=DL+0.75LL+0.75SL
D17=DL+0.75SL
D18=0.6DL+0.6WL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	C 10X15.3	**1**	D12 at 100.00%	0.58	OK	Sec. F1
		2	D12 at 0.00%	**0.60**	**OK**	Sec. F1
		3	D12 at 0.00%	0.37	OK	Sec. F1

Current Date: 10/9/2020 8:58 AM
Units system: English
File name: H:\Projects\01 Active Projects\00.0_Murphy Oil Projects\13157-2020 Murphy OII - Parker, CO (Parker and Pine)\Design\Calculations\Building\Elements\interior floor girder (2).retx

Steel Code Check

Report: Summary - Group by member

Load conditions to be included in design :

D1=DL
D2=DL+LL
D3=DL+RL
D4=DL+SL
D5=DL+0.75LL
D6=DL+0.75RL
D7=DL+0.75SL
D8=DL+0.75LL+0.75RL
D9=DL+0.75LL+0.75SL
D10=DL+0.6WL
D11=DL+0.75LL+0.45WL+0.75RL
D12=DL+0.75LL+0.45WL+0.75SL
D13=DL+0.75LL+0.45WL
D14=DL+0.45WL+0.75RL
D15=DL+0.45WL+0.75SL
D16=DL+0.75LL+0.75SL
D17=DL+0.75SL
D18=0.6DL+0.6WL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	C 10X15.3	**1**	D2 at 100.00%	0.48	OK	Sec. F1
		2	D2 at 0.00%	**0.52**	**OK**	Sec. F1
		3	D12 at 62.50%	0.36	OK	Sec. F1
		4	D2 at 0.00%	0.20	OK	Sec. F1

Current Date: 10/9/2020 9:01 AM
Units system: English
File name: H:\Projects\01 Active Projects\00.0 Murphy Oil Projects\13157-2020 Murphy OII - Parker, CO (Parker and Pine)\Design\Calculations\Building\Elements\interior floor girder (3, 4).retx

Steel Code Check

Report: Summary - Group by member

Load conditions to be included in design :

D1=DL
D2=DL+LL
D3=DL+RL
D4=DL+SL
D5=DL+0.75LL
D6=DL+0.75RL
D7=DL+0.75SL
D8=DL+0.75LL+0.75RL
D9=DL+0.75LL+0.75SL
D10=DL+0.6WL
D11=DL+0.75LL+0.45WL+0.75RL
D12=DL+0.75LL+0.45WL+0.75SL
D13=DL+0.75LL+0.45WL
D14=DL+0.45WL+0.75RL
D15=DL+0.45WL+0.75SL
D16=DL+0.75LL+0.75SL
D17=DL+0.75SL
D18=0.6DL+0.6WL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	C 10X15.3	**1**	D2 at 100.00%	0.53	OK	Sec. F1
		2	D2 at 0.00%	**0.54**	**OK**	Sec. F1
		3	D2 at 0.00%	0.31	OK	Sec. F1
		4	D2 at 0.00%	0.14	OK	Sec. F1

Current Date: 10/9/2020 9:03 AM
Units system: English
File name: H:\Projects\01 Active Projects\00.0_Murphy Oil Projects\13157-2020 Murphy OII - Parker, CO (Parker and Pine)\Design\Calculations\Building\Elements\interior floor girder (5).retx

Steel Code Check

Report: Summary - Group by member

Load conditions to be included in design :

D1=DL
D2=DL+LL
D3=DL+RL
D4=DL+SL
D5=DL+0.75LL
D6=DL+0.75RL
D7=DL+0.75SL
D8=DL+0.75LL+0.75RL
D9=DL+0.75LL+0.75SL
D10=DL+0.6WL
D11=DL+0.75LL+0.45WL+0.75RL
D12=DL+0.75LL+0.45WL+0.75SL
D13=DL+0.75LL+0.45WL
D14=DL+0.45WL+0.75RL
D15=DL+0.45WL+0.75SL
D16=DL+0.75LL+0.75SL
D17=DL+0.75SL
D18=0.6DL+0.6WL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	C 10X15.3	**1**	D2 at 100.00%	0.47	OK	Sec. F1
		2	D2 at 0.00%	**0.51**	**OK**	Sec. F1
		3	D2 at 0.00%	0.35	OK	Sec. F1

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Column Design

Section: **HSS3"x3"x1/4"**

Ax =	2.44	in2	L =	11.5	ft	
Ix =	3.02	in4	K =	1.2		
Sx =	2.01	in3	KL/r =	149		
rx =	1.11	in	E =	29000	ksi	
Fe =	12.86	ksi				
Fcr =	11.28	ksi				
Fa =	6.77	ksi	P(allow) =	16.51	kips	

Loads:

Dead =	12	psf
Live =	20	psf
Snow =	30	psf
Wind =	16	psf (positive)

Load Combinations:

DL + LL =	32.0	psf
DL + SL =	42.0	psf
DL + .75(SL+0.6WL) =	41.7	psf
DL + .75(LL+0.6WL) =	34.2	psf
Controlling Load =	42.0	psf

Column:

Trib Area =	140	sf				
Column Load =	5.88	kips	<	Fa =	16.51	kips **OK**

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Column Design

Section: **HSS3"x3"x3/16"**

Ax =	1.89	in2	L =	11.5	ft	
Ix =	2.46	in4	K =	1.2		
Sx =	1.64	in3	KL/r =	145		
rx =	1.14	in	E =	29000	ksi	
			Fy =	36	ksi	
			Fb =	21.6	ksi	
Fe =	13.56	ksi				
Fcr =	11.90	ksi				
Fa =	7.14	ksi	P(allow) =	13.49	kips	

Loads:

Dead =	12	psf
Live =	20	psf
Snow =	30	psf
Wind =	16	psf (positive)

Load Combinations:

DL + LL =	32.0	psf
DL + SL =	42.0	psf
DL + .75(SL+0.6WL) =	41.7	psf
DL + .75(LL+0.6WL) =	34.2	psf
Controlling Load =	42.0	psf

Column:

Trib Area =	100	sf
Column Load =	4.20	kips

Bending Moment =

Header =	0.29	ft-kips			
Wind Load =	0.22	ft-kips			
Total =	0.51	ft-kips	M(allow) =	2.95	ft-kips

Interaction Eqn =	P/P(allow)	+	0.889*(M/M(allow)	≤	1.0	
Interaction Eqn =	0.31	+	0.15	=	0.46	**OK**

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Lateral Analysis

Building Dimensions:

Length =	64.80	ft	End Zone (a) =	4.50	ft
Width =	43.8	ft	End Zone =	25.4	psf
Height (T/o W) =	12.00	ft	Interior Zone =	16.9	psf
Parapet Height =	3.75	ft			
Cornice Height =	2.67	ft	Parapet (WW+LW) =	63.8	psf

Wind Loads (transverse):

Location				Trib Width				
End Zone =	15.2	psf	x	18.00	ft	=	274	plf
Interior Zone =	10.14	psf	x	46.80	ft	=	475	plf
							749	plf
				(half height)		x	6.00	ft
							4493	pounds
Parapet =	38.28	psf	x	64.80	ft	=	2481	plf
						x	3.75	ft
							9302	pounds
Tower =	38.28	psf	x	24.00	ft	=	919	plf
						x	3	ft
							2450	pounds
				walls + parapet + tower =			16245	pounds

Wind Loads (longitudinal):

Location				Trib Width				
End Zone =	15.2	psf	x	18.00	ft	=	274	plf
Interior Zone =	10.14	psf	x	25.80	ft	=	262	plf
							536	plf
				(half height)		x	6.00	ft
							3216	pounds
Parapet =	38.28	psf	x	43.80	ft	=	1677	plf
						x	3.75	ft
							6287	pounds
Tower =	38.28	psf	x	27.00	ft	=	1034	plf
						x	3	ft
							2757	pounds
				walls + parapet + tower =			12260	pounds

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Seismic Loads:

Seismic Base Shear = Cs * W * 0.7 (load factor)

Seismic Coeff (Cs) = 0.086

Building Mass (W) =

roof =	12	psf	x	1	x	2838	sf	=	34059	pounds
walls =	12	psf	x	0.5	(half ht) x	2606	sf	=	15638	pounds
									49697	pounds

Seismic Base Shear = 2992 pounds

Lateral Load Comparison:

Transverse Direction

Max Base Shear = 16245 pounds Wind Loads Govern

Longitudinal Direction

Max Base Shear = 12260 pounds Wind Loads Govern

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Shear Wall Design

Transverse Direction: (short walls 40.5')

Total Shear Force =	16245	pounds
x	0.5	(2 walls resist shear)
	8123	pounds
/	36.5	(wall length)
	222.5	plf

Longitudinal Direction: (long walls) Note: seismic force @ roof < wind load

Total Shear Force =	12260	pounds
x	0.5	(2 walls resist shear)
	6130	pounds
	0	(shear from canopy)
	6130	pounds
/	9	(net wall length)
	681.1	plf

Shear Capacity of Metal Wall Panels: (FMS wall panel used as basis of design)

Dimensions:

Width =	16	in
Depth =	3	in
Thickness =	0.036	in (20 gage)
Fy =	50	ksi
h/t =	444	
kv =	5.34	(for unreinforced web)
sqrt(E*kv/Fy) =	55.7	

h/t > 1.51 * sqrt(E*kv/Fy) = 84.0 Fv = 0.904*E*kv/(h/t)^2 = 0.71 ksi

F.S. = 1.6

v(allow) = 0.44 ksi

Allowable Shear =	191	plf <	681.1	NG use FSB
	191	plf <	222.5	OK w/ add'l strap bracing

Shear Capacity of #10 Screws: (20 gage material)

Shear Capacity =	565	pounds	x	1.44	=	814 pounds (increase for Fu = 65 ksi)
F.S. =	3.0					
Allowable Shear =	271	pounds				
Screw Spacing =	16	inches				
Shear Capacity =	204	plf <		681.1		NG use FSB
	204	plf <		222.5		OK w/ add'l strap bracing

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Flat Strap Bracing Wall Design - Front Wall

Allowable Tension Strap Bracing

Width =	6	in	
Gauge =	0.0713	in	(14 Ga.)
Area =	0.4278	in^2	
Fy =	50	ksi	
T_{allow} =	**12.834**	**kip**	

Transverse Direction:

Front Wall:

Panel Length 1 =	8.75	feet	

Strap Tension From Elements Model:

Front Wall:

Tension Length 1 =	**10.14**	**kip**	**OK**

Strap Connection to Sill & Posts:

Weld Capacity:

Fillet Weld Size =	0.125	in	
Total Weld Length =	7.0	in	
Weld Capacity =	13.0	kip	**OK**

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Flat Strap Bracing Wall Design - Left Wall

Allowable Tension Strap Bracing

Width =	2	in	
Gauge =	0.0713	in	(14 Ga.)
Area =	0.1426	in^2	
Fy =	50	ksi	
T_{allow} =	**4.278**	**kip**	

Transverse Direction:

Both Walls:

Panel Length 1 =	5.5	feet
Panel Length 2 =	5.5000	feet

Strap Tension From Elements Model:

Both Walls:

Tension Length 1 =	**2.5**	**kip**	**OK**
Tension Length 2 =	**2.50**	**kip**	**OK**

Strap Connection to Sill & Posts:

Weld Capacity:

Fillet Weld Size =	0.125	in	
Total Weld Length =	4.0	in	
Weld Capacity =	7.4	kip	**OK**

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Foundation Design

Location	Footing No	Design Load (k)	Square Footing Size			
			Bearing	Minimum	Rounded	Ftg Mk
Exterior Wall (1)	1	5.14	1.6	2.0	2.0	**F4.0**
	2	15.75	2.8	2.8	3.0	**F4.0**
	3	12.39	2.5	2.5	2.5	**F4.0**
	4	8.28	2.0	2.0	2.5	**F4.0**
Interior Wall (2)	1	9.13	2.1	2.1	2.5	**F4.0**
	2	27.79	3.7	3.7	4.0	**F4.0**
	3	23.06	3.3	3.3	3.5	**F4.0**
	4	19.09	3.0	3.0	3.5	**F4.0**
Interior Wall (3)	1	9.42	2.1	2.1	2.5	**F4.0**
	2	28.57	3.7	3.7	4.0	**F4.0**
	3	29.44	3.8	3.8	4.0	**F4.0**
	4	13.96	2.6	2.6	3.0	**F4.0**

Notes:

1. Allowable Net Bearing Pressure = 3000 psf
2. Design Loads from 1st Floor Beam FE models

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Foundation Design

Location	Footing No	Design Load (k)	Square Footing Size			
			Bearing	Minimum	Rounded	Ftg Mk
Interior Wall (4)	1	9.42	2.1	2.1	2.5	**F4.0**
	2	28.57	3.7	3.7	4.0	**F4.0**
	3	29.44	3.8	3.8	4.0	**F4.0**
	4	13.96	2.6	2.6	3.0	**F4.0**
Interior Wall (5)	1	9.18	2.1	2.1	2.5	**F4.0**
	2	27.47	3.7	3.7	4.0	**F4.0**
	3	30.73	3.9	3.9	4.0	**F4.0**
	4	7.01	1.8	2.0	2.0	**F4.0**
Exterior Wall (6)	1	5.36	1.6	2.0	2.0	**F4.0**
	2	15.90	2.8	2.8	3.0	**F4.0**
	3	13.06	2.5	2.5	3.0	**F4.0**
	4	3.76	1.4	2.0	2.0	**F4.0**

Notes:

1. Allowable Net Bearing Pressure = 3000 psf
2. Design Loads from 1st Floor Beam FE models

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Foundation Design

Check Lateral Loads on Piers

Transverse Direction: (short walls)

Total Shear Force =	4320	lbs
Shear Force / Wall =	2160	lbs

Four piers available to resist shear in each short wall
Assume end piers each resist 16.667% and center pier resists 33.3%

End Pier Shear =	360	lbs
Center Pier Shear =	719	lbs

Longitudinal Direction: (long walls)

Building Shear Force =	3149	lbs
Canopy Shear Force =	0	lbs
Total Shear Force =	3149	lbs
Shear Force / Wall =	1574	lbs

Six piers available to resist shear in each long wall
Assume end piers each resist 10% and interior piers each resist 20%

End Pier Shear =	157	lbs
Interior Pier Shear =	315	lbs

Controlling load condition occurs for piers beneath short walls

Check corner piers:

V =	360.054	lbs	D =	3	feet (pier depth)
=	0.5 * D * B * f(max)		Ø =	4	feet (pier width)
f(max) =	60	psf	f(max) =	max passive pressure	
			Ftg. Wt. =	5466	lbs

Max corner pier load = 8280

Max Allowable Passive Pressure

=	Kp * ϒ * D		Kp =	3.0	(passive coeff)
=	990	psf OK	ϒ =	110	pcf (soil unit weight)

Check bearing pressure with overturning

P =	13746	lbs	F(allow) =	3000	psf
M =	720	ft-lbs			
e =	0.05	ft			
kern =	0.67	ft OK			

Max Bearing Press =	P/A	+	M/S	
=	1093.9	+	115	
=	1209	psf	OK	

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Foundation Design

Check Lateral Loads on Piers

Check center piers:

V =	719.244	lbs		D =	3	feet (pier depth)
=	0.5 * D * B * f(max)			Ø =	4	feet (pier width)
f(max) =	120	psf		f(max) =	max passive pressure	
				Ftg wt. =	5466	lbs
Min center pier load =	19090	lbs				

Max Allowable Passive Pressure

=	Kp * ϒ * D			Kp =	3.0	(passive coeff)
=	990	psf	OK	ϒ =	110	pcf (soil unit weight)

Check bearing pressure with overturning

P =	24556	lbs		F(allow) =	3000	psf
M =	1439	ft-lbs				
e =	0.06	ft				
kern =	0.67	ft	OK			

Max Bearing Press =	P/A		+	M/S	
=	1954		+	229	
=	2183	psf		OK	

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Check Brick Veneer Anchorage

Option A

1/2" OSB sheathing attached to 20 Ga. wall panels.

Max suction pressure (corners) = 19.1 psf
Trib. Area = 32.0 sf

Nominal Panel Thickness (in)	Span Rating	Spacing of Supports, center-to-center (in)	Allowable Load (psf)		Ratio of Strength Axis Perpendicular Load/Strength Axis Parallel Load
			Strength Axis Perpendicular to Supports[2]	Strength Axis Parallel to Supports[3]	
7/16	24/16	16	110	50	2.2
15/32	32/16	24	80	25	3.2
½				30	2.67
19/32	40/20		140	50[4]	2.8
5/8				55[4]	2.54
23/32 & 3/4	48/24		185	65[4]	2.85

1. Assumes panels are continuous over two or more spans and at least 24" wide.
2. Support of panel edges perpendicular to the framing not required for the panel to support the uniformly distributed load with the corresponding framing spacing.
3. Support of panel edges perpendicular to the framing required. Panel edge support may include tongue-and-groove edges, panel edge clips (one between each support, except two between supports 48 inches on center), lumber blocking or other.
4. Reduce load by 15 psf for composite and four-ply plywood.
5. Allowable loads per Tables 10 & 11 of TECO's *Design & Application Guides*.

Capacity of 1/2" sheathing applied to 18 Ga studs @ 16" o/c w/ #6 screws = 30.0 psf
OK

Check Capacity of #6 Screws

(sheathing connection to metal wall panels)

Base Metal =	0.072	in	14 GA (0.075")
Tensile Capacity =	395	lb	ultimate
Factor of Safety =	3		
Allowable Tension =	131.67	lb	
Trib Area/ screw =	0.89	sf	(8" o/c)
Max Suction Pressure =	148.13	psf	OK

#6 screws @ 8" o/c will be attached where 20 GA wall panels overlap. Equivalent thickness of (2) 20 GA panels is greater than 14 GA stud thickness - OK

Check Glen-Gary Panels Connection

(Use #10 screws at 8" o/c vertically and 16" o/c horizontally)

Base Material =	0.5	in	cement sheathing
Specific Gravity =	0.42		
#10 wood screw =	0.19	in	shank diameter
Allowable Tension =	47.76	lb	
Trib Area/ screw =	0.89	sf	
Max Suction Pressure =	53.73	psf	OK

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Check Brick Veneer Anchorage

Option B

1/2" cement board attached to 20 Ga. wall panels.

Max suction pressure (corners) = 19.14 psf
Trib. Area = 32.0 sf

UNIFORM LOAD —1/2" USG DUROCK® BRAND CEMENT BOARD

Stud Spacing	Fastener Spacing	Design Wind Load (l/240)
12" o.c.	8" o.c.	45 psf
	6" o.c.	60 psf
16" o.c.	8" o.c.	33 psf
	6" o.c.	45 psf
24" o.c.(for shaft wall assemblies only)	8" o.c.	13 psf
	6" o.c.	13 psf

Capacity of 1/2" cement board applied to 18 Ga studs @ 16" o/c w/ #6 screws = 33.0 psf

19.14 psf < 33.0 psf OK

Check Capacity of #6 Screws

(sheathing connection to metal wall panels)

Base Metal =	0.072	in	14 GA (0.075")
Tensile Capacity =	395	lb	ultimate
Factor of Safety =	3		
Allowable Tension =	131.67	lb	
Trib Area/ screw =	0.89	sf	(8" o/c)
Max Suction Pressure =	148.13	psf	OK

#6 screws @ 8" o/c will be attached where 20 GA wall panels overlap. Equivalent thickness of (2) 20 GA panels is greater than 14 GA stud thickness - OK

Brick Veneer Anchorage

Max suction pressure (corners) = 21.5 psf
Trib. Area = 10.0 sf

Bond strength of MVIS Hi-Bond veneer mortar = 230 psi (min) as per technical bond strength data

Factor of Safety = 5

Allowable Bond Strength = 6624 psf OK

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Check Skin Panel Fastening System

Max Suction Pressure (corners) =	21.54	psf
Trib area =	10	sf

Check capacity of #10 Screws

(skin panel connection to meal wall panels)

Base Metal =	0.072	in	14 GA (0.075")
Tensile Capacity =	545	lb	ultimate
Factor of Safety =	3		
Allowable Tension =	181.67	lb	
Trib Area/ screw =	4.00	sf	(24" o/c)
Max Suction Capacity =	45.42	psf	OK

#10 screws @ 24" o/c will be attached where (2) 20 GA wall panels overlap. Equivalent thickness of (2) 20 GA panels is greater than 14 GA stud thickness - OK

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Check Awning Anchorage

Max suction pressure (corners) =	21.54	psf
Trib Area =	10	sf

Check capacity of #14 Screws (Upper Connection)

Base Metal =	0.075	in	14 GA (0.075")
Tensile Capacity =	715	lb	ultimate
Factor of Safety =	3		
Allowable Tension =	238.33	lb	
Trib Area/ screw =	1.33	sf	(16" o/c)
Max Suction Capacity =	178.79	psf	OK

#14 screws @ 16" o/c will attach awning to 14 GA hat channel at upper connection.

Check capacity of #14 Screws (Lower Connection)

Base Metal =	0.075	in	14 GA (0.075")
Tensile Capacity =	715	lb	ultimate
Factor of Safety =	3		
Allowable Tension =	238.33	lb	((4) screws @ 5' o/c)
Trib Area/ screw =	3.13	sf	
Max Suction Capacity =	76.27	psf	OK

(4) #14 screws @ 5' o/c will attach awning supports to 14 GA hat channel at lower connection.

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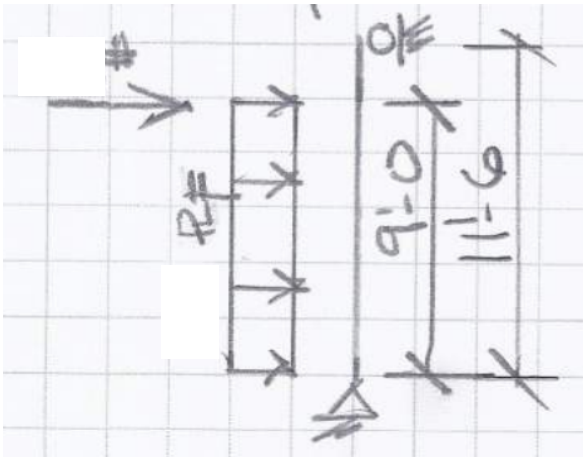
Door Jambs & Headers

Zone 5 (corner pressure) = 21.5 psf



Door Jambs are either:
= L3x3x3/8"
= HSS3x3x14 GA

Wind Load = 36.77 plf jamb
53.75 psf header

Header: HSS3x3x14 GA

Span =	4	ft	**Fbx =**	27.6	**ksi**			
Wx =	30	plf	**Fby =**	27.6	**ksi**			
Wy =	43	plf						
Mx =	60	ft-lb	**Sx =**	1.73	in^3	**fbx =**	0.42	**ksi**
My =	86	ft-lb	**Sy =**	1.73	in^3	**fby =**	0.60	**ksi**

Interation Ratio = 0.037 OK

Jamb:

Rt =	275	lb	Sx =	0.825	in^3
Rb =	342	lb	fbx =	0.3	ksi
y =	9.30	ft	Fbx =	21.6	ksi
V =	0.00	lb	**Fbx > fbx**		OK
M =	19.09	ft-lb			

Door Attachment

Drawing No. 1695; Sheet 1 of 13:

FRAME & SILL ANCHORS				
ANCHOR TYPE	OPENING TYPE (SUBSTRATE)	JAMB TO OPENING FASTENER TYPE	MINIMUM EMBED	MINIMUM EDGE DIST.
A	2X_ WOOD FRAME OR BUCK (MIN. GR. 3 & G=0.55)	NO. 14 SMS SCREW	1 1/4"	3/4"
B	MIN. 18 GA. 33 KSI METAL STUD	(2) NO. 12 SELF DRILL/TAPPING SCREW	FULL	1/2"
C	MIN. 0.22" THICK A36 STEEL	(2) 1/4-20 SELF DRILL/TAPPING SCREW	FULL	1/2"

3x3x3/8" steel angles (jambs) meet schedule C requirements
3x3x14 ga columns (studs) meet schedule B requirements

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Security Shutter Attachment

Wind Load =	21.5	psf	(zone 5, corner)
Max Opening Width =	8.0	psf	
Jamb Wind Load =	86	plf	
1/4" TEK Screw Capacity =	403	lb	(in 1/8" Aluminum)
Screw Spacing =	4.69	ft	

1/4" TEK Screw @ 6" o/c OK

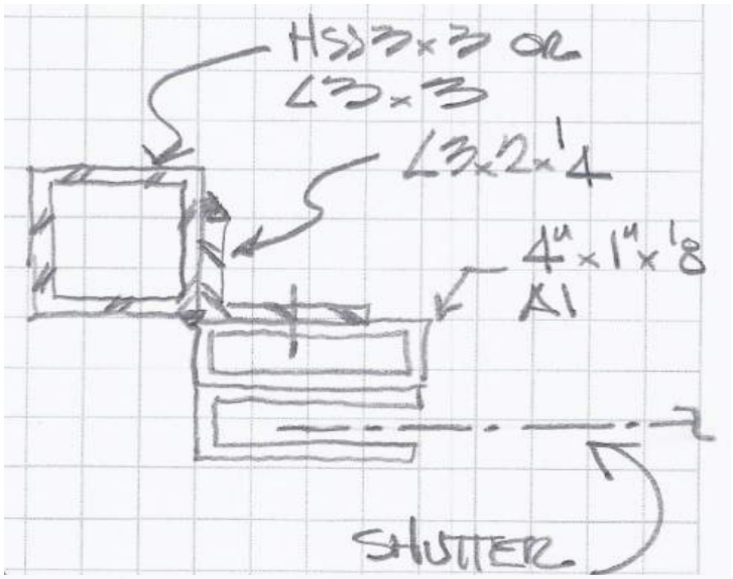

TABLE 2 – ALLOWABLE TENSILE PULL-OUT LOADS (P_{NOT}/Ω), pounds-force[1, 2, 3, 4]

Allowable Pullout Values (P/Ω) on Cold Formed Steel and Aluminum; Ω=3.0														
Screw Designation	Nominal Diameter (in.)	Steel Gauge/Thickness								Aluminum Thickness				
										6063-T52			6063-T6	
		18	16	14	12	1/8"	3/16"	1/4"	5/16"	1/8"	1/4"	3/8"	1/8"	1/4"
#10-16 HWH	0.190	140	182	211	388	404				250			377	
#12-14 HWH	0.216	138	186	230	481	496	809			278	680		434	864
#12-14 UPFH	0.216	140	218	252	473	507	837			274	642		418	812
1/4-14 HWH	0.250	170	224	274	431	582	971			283	732		403	997
1/4-20 HWH	0.250	157	231	282	427	571	1066	1422	1422	281	685	1118	396	949

1. For tension connections, the lower of the allowable pull-out, pullover, and fastener tension strength found in Table 2, 3, and 4, respectively must be used for design.
2. Nominal strengths are based on laboratory tests. Steel is compliance with AISI Manual of Cold-formed Steel Design, 2008 Edition, Part I: Dimension and Properties for Use with the 2007 North American Cold-Formed Steel Specification with minimum tensile strength 58 ksi. Aluminum is compliance with 2010 Aluminum Design Manual, Part I specification for Aluminum Structures; 6063-T52 with minimum tensile strength 22 ksi, or 6063-T6 with minimum tensile strength 30 ksi.
3. To calculate LRFD values, multiply values in table by the ASD safety factor of 3.0 and multiply again with the LRFD Φ factor of 0.5.
4. The base-metal thickness of 18 gauge steel is 0.048"; 16 gauge is 0.060"; 14 gauge is 0.075"; and 12 gauge is 0.105";

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Security Shutter Attachment (CONT.)

Check Weld:

1/8" Fillet Weld

Weld Capacity =	1.86	kip/in			
1" per 16" o/c =	1.39	kip/ft	>	0.09	kip/ft
			OK		

Bending Check in OSL of Angle:

P =	0.043	kip	
M =	0.065	in-kip	
Sx =	0.031	in^3	
fbx =	2.064	ksi	
Fbx =	27.00	ksi	OK

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Bearing Embed Plate Design

Plate Thickness:

Moment (ft-k)	0.00		
Column Load, P (kips)	29.44	Column Depth, d (in)	6
Plate Length, N (in)	6	Column Width, bf (in)	5.2
Plate Width, B (in)	6	Fy (ksi)	36
f'c (psi)	3000	Concrete Pedestal Area (sq. in)	4000

Axially Loaded w/ No Moment

Allowable Bearing Stress, Fp (ksi)	2.1		
Required Plate Area (sq. in.)	14.0		
Actual Plate Area (sq. in.)	36	OK	(Revise N & B if "NG")
Actual Bearing Stress, fp (ksi)	0.8	OK	(Revise N & B if "NG")
m (in)	0.15		
n (in)	0.92		
Required Plate Thickness, tp (in)	0.28		

Concrete Breakout Strength of Anchor in Shear:

Maximum actual shear per pier	799	lbs
Uplift on embed plate	0	lbs
Edge distance from anchor (c_{a1})	27	in
Anchor Spacing	6	in
Footing Depth	36	in
Concrete strength	3000	psi
Allowable shear per anchor group	74864	OK

6x6x1/2" EMBED PLATES w/ (2) 5/8" DIA. ANCHORS - OK