STRUCTURAL CALCULATIONS

FOR

WM 6958 Cameron, NC OPD

2800 NC 24-87

NORTH CAROLINA
PROFESSIONAL
SEAL
051941
ENGINEER
BRIAN W. MAKOVEC

1/7/25

JOHNSTON
BURKHOLDER
ASSOCIATES
consulting structural engineers

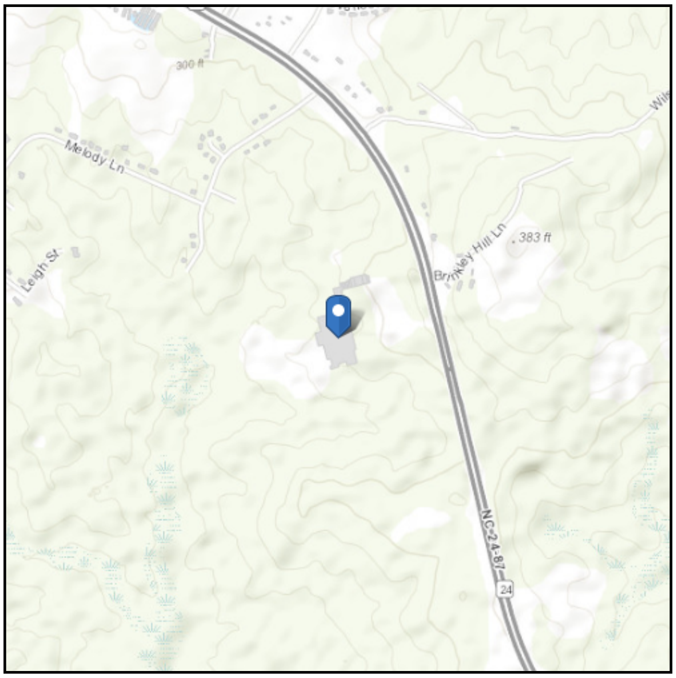
DESIGN LOADS

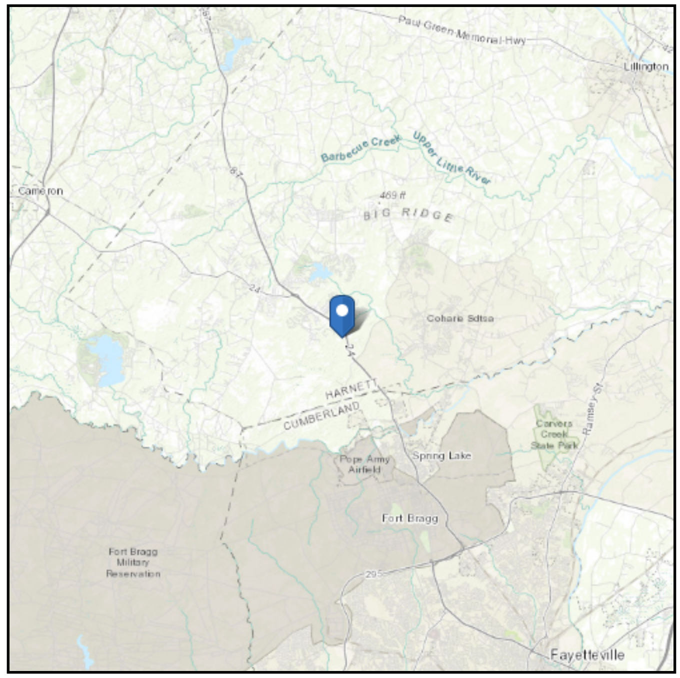
ASCE Hazards Report

Address:
2800 Nc 24-87
Cameron, North Carolina
28326

Standard: ASCE/SEI 7-10
Risk Category: II
Soil Class: D - Stiff Soil

Latitude: 35.244068
Longitude: -79.026188
Elevation: 277.48959845971933 ft (NAVD 88)





Wind

Results:

Wind Speed	118 Vmph
10-year MRI	76 Vmph
25-year MRI	84 Vmph
50-year MRI	90 Vmph
100-year MRI	96 Vmph

Data Source:	ASCE/SEI 7-10, Fig. 26.5-1A and Figs. CC-1–CC-4, and Section 26.5.2, incorporating errata of March 12, 2014
Date Accessed:	Sat Oct 19 2024

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-10 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is in a hurricane-prone region as defined in ASCE/SEI 7-10 Section 26.2. Glazed openings need not be protected against wind-borne debris.

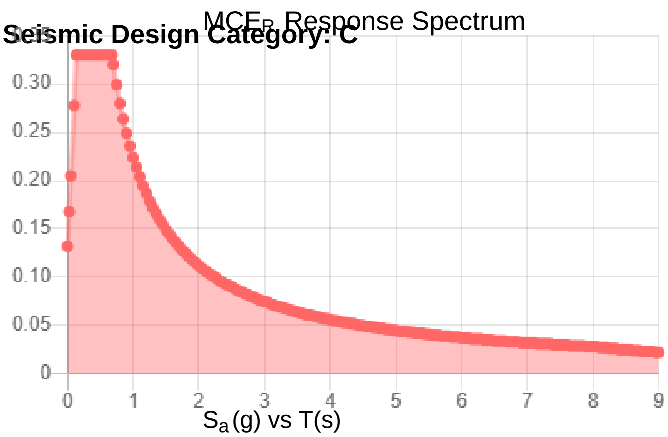
Seismic

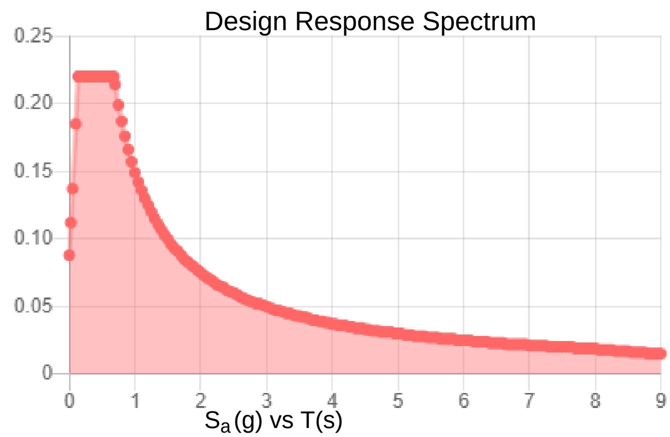
Site Soil Class: D - Stiff Soil

Results:

S_S :	0.206	S_{D1} :	0.149
S_1 :	0.093	T_L :	8
F_a :	1.6	PGA :	0.102
F_v :	2.4	PGA_M :	0.163
S_{MS} :	0.33	F_{PGA} :	1.596
S_{M1} :	0.224	I_e :	1
S_{DS} :	0.22		

Seismic Design Category: C





Data Accessed: **Sat Oct 19 2024**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-10, incorporating Supplement 1 and errata of March 31, 2013, and ASCE/SEI 7-10 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-10 Ch. 21 are available from USGS.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Snow

Results:

Ground Snow Load, p_g :	10 lb/ft^2
Mapped Elevation:	277.5 ft
Data Source:	ASCE/SEI 7-10, Fig. 7-1.
Date Accessed:	Sat Oct 19 2024

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

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

Rain

Results:

15-minute Precipitation Intensity:	6.62 in./h
60-minute Precipitation Intensity:	3.49 in./h

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

Date Accessed: Sat Oct 19 2024

Code Search

Code: ASCE 7 - 10

Occupancy:

Occupancy Group = B Business

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)	0.25 / 12	1.2 deg
Building length	156.0 ft	
Least width	28.7 ft	
Mean Roof Ht (h)	17.3 ft	
Parapet ht above grd	20.7 ft	
Minimum parapet ht	3.4 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

Partitions N/A

Johnston Burkholder Associates
930 Central St
Kansas City, MO
(816) 421-4200

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JOB NO.	2431906958	**SHEET NO.**	
CALCULATED BY	DTR	**DATE**	
CHECKED BY		**DATE**	

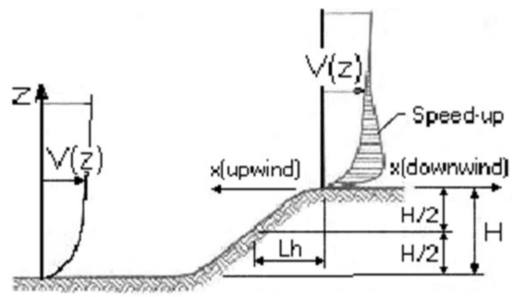
Wind Loads : ASCE 7- 10

Ultimate Wind Speed	120 mph
Nominal Wind Speed	93 mph
Risk Category	II
Exposure Category	B
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.701
Kh case 2	0.598
Type of roof	Monoslope

Topographic Factor (Kzt)

Topography	Flat
Hill Height (H)	1.0 ft
Half Hill Length (Lh)	1.0 ft
Actual H/Lh =	0.00
Use H/Lh =	0.00
Modified Lh =	1.0 ft
From top of crest: x =	1.0 ft
Bldg up/down wind?	downwind

H< 60ft;exp B
∴ Kzt=1.0

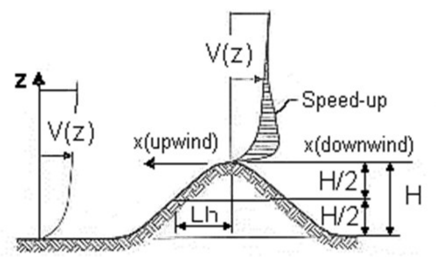


ESCARPMENT

H/Lh=	0.00	K_1 =	0.000
x/Lh =	1.00	K_2 =	0.333
z/Lh =	17.25	K_3 =	1.000

At Mean Roof Ht:

$Kzt = (1+K_1K_2K_3)^2 =$ 1.00



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h =	17.3 ft
B =	28.7 ft
/z (0.6h) =	30.0 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 = 0.60 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure

ē =	0.33
ℓ =	320 ft
z_{min} =	30 ft
c =	0.30
g_Q, g_v =	3.4
L_z =	310.0 ft
Q =	0.92
I_z =	0.30
G =	**0.88** use G = 0.85

Flexible or Dynamically Sensitive Structure

Natural Frequency (η_1) =	0.0 Hz				
Damping ratio (β) =	0				
/b =	0.45				
/α =	0.25				
Vz =	77.3				
N_1 =	0.00				
R_n =	0.000				
R_h =	28.282	η =	0.000	h =	17.3 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				

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CHECKED BY		DATE	

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

Kz = Kh (case 1) =	0.70	Edge Strip (a) =	3.0 ft
Base pressure (qh) =	**22.0 psf**	End Zone (2a) =	6.0 ft
GCpi =	+/-0.18	Zone 2 length =	14.3 ft

Wind Pressure Coefficients

	CASE A				CASE B		
		θ = 1.2 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	12.7	4.8		-5.9	-13.8
2	-11.2	-19.1		-11.2	-19.1
3	-4.2	-12.1		-4.2	-12.1
4	-2.4	-10.3		-5.9	-13.8
5				12.7	4.8
6				-2.4	-10.3
1E	17.3	9.4		-6.6	-14.5
2E	-19.5	-27.4	MWFRS ROOF BEAM	-19.5	-27.4
3E	-7.7	-15.6	UPLIFT FOR RISA	-7.7	-15.6
4E	-5.5	-13.4		-6.6	-14.5
5E				17.3	9.4
6E				-5.5	-13.4

Parapet

Windward parapet = 32.9 psf (GCpn = +1.5)
Leeward parapet = -22.0 psf (GCpn = -1.0)

Windward roof overhangs = 15.4 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

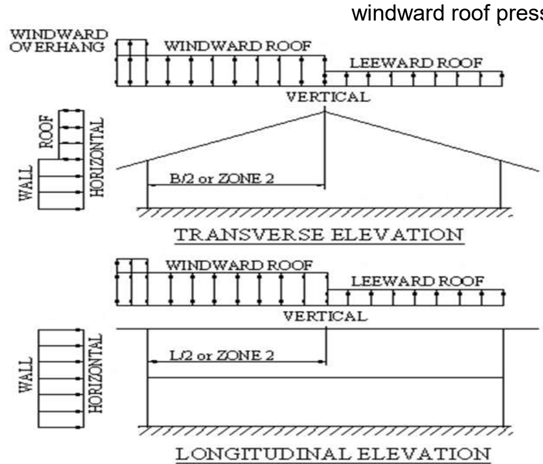
Interior Zone:	Wall	15.1 psf	
	Roof	-7.0 psf	**
End Zone:	Wall	22.8 psf	
	Roof	-11.9 psf	**

Longitudinal direction (parallel to L)

Interior Zone:	Wall	15.1 psf
End Zone:	Wall	22.8 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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Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

Kz = Kh (case 1) = 0.70
Base pressure (qh) = **13.2 psf**
GCpi = +/-0.18

Edge Strip (a) = 3.0 ft
End Zone (2a) = 6.0 ft
Zone 2 length = 14.3 ft

Wind Pressure Coefficients

Surface	CASE A GCpf	θ = 1.2 deg w/-GCpi	w/+GCpi		CASE B 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

Nominal Wind Surface Pressures (psf)

1		7.6	2.9			-3.6	-8.3
2		-6.7	-11.5			-6.7	-11.5
3		-2.5	-7.2			-2.5	-7.2
4		-1.4	-6.2			-3.6	-8.3
5						7.6	2.9
6						-1.4	-6.2
1E		10.4	5.7			-4.0	-8.7
2E		-11.7	-16.5			-11.7	-16.5
3E		-4.6	-9.4	GIRDER UPLIFT		-4.6	-9.4
4E		-3.3	-8.0	16.5 PSF - 6 PSF RELIABLE DL = 10.5 PSF		-4.0	-8.7
5E						10.4	5.7
6E						-3.3	-8.0

Parapet

Windward parapet = 19.8 psf (GCpn = +1.5)
Leeward parapet = -13.2 psf (GCpn = -1.0)

Windward roof overhangs = 9.2 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

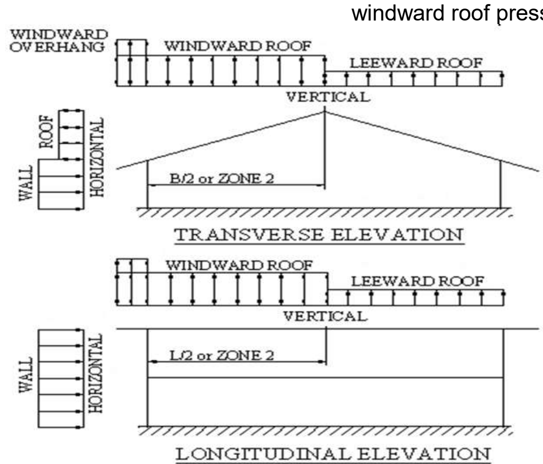
Interior Zone: Wall 9.1 psf
Roof -4.2 psf **
End Zone: Wall 13.7 psf
Roof -7.1 psf **

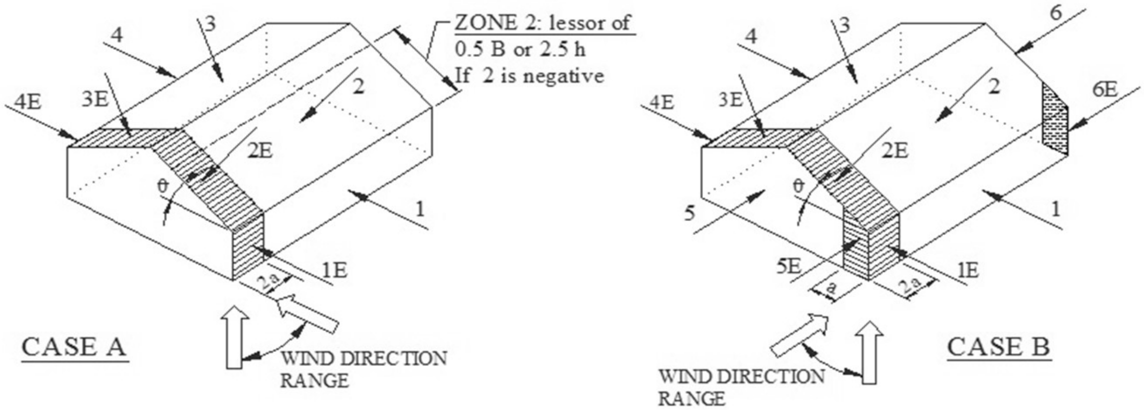
Longitudinal direction (parallel to L)

Interior Zone: Wall 9.1 psf
End Zone: Wall 13.7 psf

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

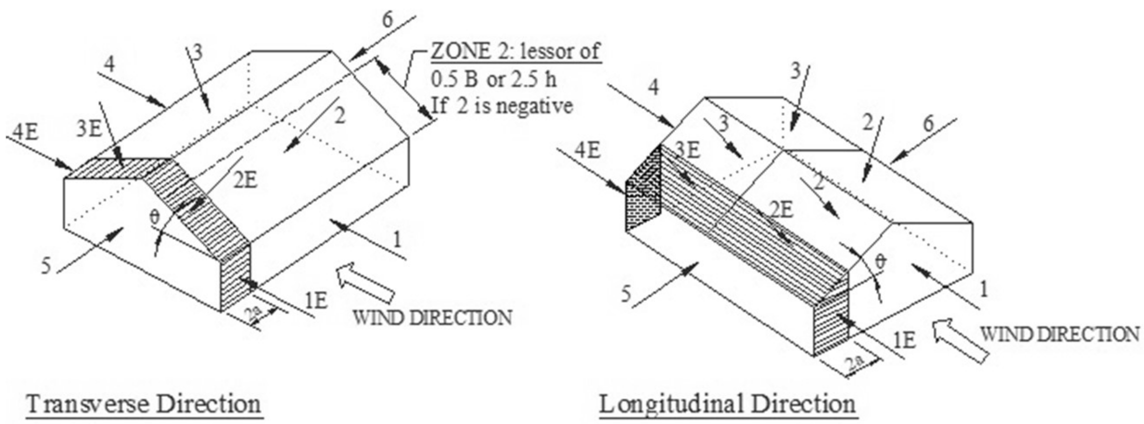
The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.





NOTE: Torsional loads are 25% of zones 1 - 6. See code for loading diagram.
Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

ASCE 7-98 & ASCE 7-10 (& later) - MWFRS wind pressure zones



NOTE: Torsional loads are 25% of zones 1 - 4. See code for loading diagram.
Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

ASCE 7-02 and ASCE 7-05 - MWFRS wind pressure zones

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Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) =	0.70	h =	17.3 ft
Base pressure (qh) =	**22.0 psf**	a =	3.0 ft
Minimum parapet ht =	3.4 ft	GCpi =	+/-0.18
Roof Angle (θ) =	1.2 deg		
Type of roof =	Monoslope		

C&C ROOF BEAM UPLIFT FOR RISA

Roof

	GCp +/- GCpi				Surface Pressure (psf)				User input	
Area	10 sf	50 sf	100 sf	500 sf	10 sf	50 sf	100 sf	500 sf	75 sf	150 sf
Negative Zone 1	-1.18	-1.11	-1.08	-1.08	-25.9	-24.4	-23.7	-23.7	-24.0	-23.7
Negative Zone 2	-1.98	-1.49	-1.28	-1.28	-43.5	-32.7	$\boxed{-28.1}$	-28.1	-30.0	-28.1
Negative Zone 3	-1.98	-1.49	-1.28	-1.28	-43.5	-32.7	-28.1	-28.1	-30.0	-28.1
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	23.7	21.3	20.2	17.8	20.7	19.6
Overhang Zone 1&2	-1.7	-1.63	-1.6	-1.1	-37.3	-35.8	-35.1	-24.1	-35.4	-32.4
Overhang Zone 3	-1.7	-1.63	-1.6	-1.1	-37.3	-35.8	-35.1	-24.1	-35.4	-32.4

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 psf)

Parapet

qp = 22.0 psf

		Surface Pressure (psf)						User input
Solid Parapet Pressure		10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	50 sf
CASE A:	Zone 2 :	59.3	53.6	46.1	40.4	39.4	38.0	46.1
	Zone 3 :	59.3	53.6	46.1	40.4	39.4	38.0	46.1
CASE B:	Edge zones 2 :	-41.5	-39.4	-36.6	-34.5	-32.4	-29.6	-36.6
	Corner zones 3 :	-47.4	-44.3	-40.1	-37.0	-33.8	-29.6	-40.1

Walls

	GCp +/- GCpi				Surface Pressure (psf)				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	10 sf	55 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-25.7	-22.2	-21.1	-19.8	-25.7	-23.1
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-31.6	-24.6	-22.5	-19.8	-31.6	-26.4
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	23.7	20.2	19.2	17.8	23.7	21.1

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

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Nominal Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) = 0.70
Base pressure (qh) = **13.2 psf**
Minimum parapet ht = 3.4 ft
Roof Angle (θ) = 1.2 deg
Type of roof = Monoslope

h = 17.3 ft
a = 3.0 ft 2a = 6.0'
GCpi = +/-0.18

WIDTH OF EDGE ZONE FOR GIRDERS = 2h = 35'-0" (FULL WIDTH OF EXPANSION)

JOIST UPLIFT (INTERIOR)
16.9 PSF - 6 PSF RELIABLE DL = 10.9 PSF
JOIST UPLIFT (EDGE)
14.2 PSF - 6 PSF RELIABLE DL = 8.2 PSF

Roof

	GCp +/- GCpi				Surface Pressure (psf)				User input	
Area	10 sf	50 sf	100 sf	500 sf	10 sf	50 sf	100 sf	500 sf	75 sf	150 sf
Negative Zone 1	-1.18	-1.11	-1.08	-1.08	-15.5	-14.6	-14.2	-14.2	-14.4	-14.2
Negative Zone 2	-1.98	-1.49	-1.28	-1.28	-26.1	-19.6	-16.9	-16.9	-18.0	-16.9
Negative Zone 3	-1.98	-1.49	-1.28	-1.28	-26.1	-19.6	-16.9	-16.9	-18.0	-16.9
Positive Zone 1	0.48	0.41	0.38	0.38	10.0	10.0	10.0	10.0	10.0	10.0
Positive Zones 2 & 3	1.08	0.97	0.92	0.81	14.2	12.8	12.1	10.7	12.4	11.8
Overhang Zone 1&2	-1.7	-1.63	-1.6	-1.1	-22.4	-21.5	-21.1	-14.5	-21.2	-19.4
Overhang Zone 3	-1.7	-1.63	-1.6	-1.1	-22.4	-21.5	-21.1	-14.5	-21.2	-19.4

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 2.4 psf)

Parapet

qp = 13.2 psf

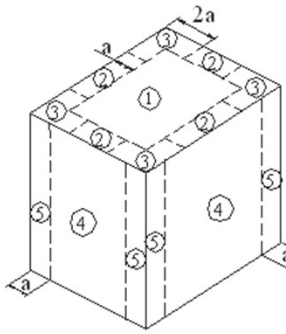
		Surface Pressure (psf)						User input
Solid Parapet Pressure		10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	50 sf
CASE A:	Zone 2 :	35.6	32.2	27.7	24.2	23.6	22.8	27.7
	Zone 3 :	35.6	32.2	27.7	24.2	23.6	22.8	27.7
CASE B:	Edge zones 2 :	-24.9	-23.6	-22.0	-20.7	-19.4	-17.8	-22.0
	Corner zones 3 :	-28.5	-26.6	-24.1	-22.2	-20.3	-17.8	-24.1

Walls

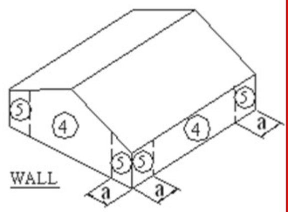
	GCp +/- GCpi				Surface Pressure (psf)				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	10 sf	55 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-15.4	-13.3	-12.7	-11.9	-15.4	-13.9
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-19.0	-14.8	-13.5	-11.9	-19.0	-15.9
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	14.2	12.1	11.5	10.7	14.2	12.7

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

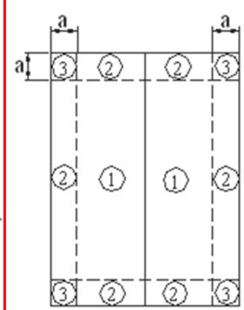
Location of C&C Wind Pressure Zones - ASCE 7-10 & earlier



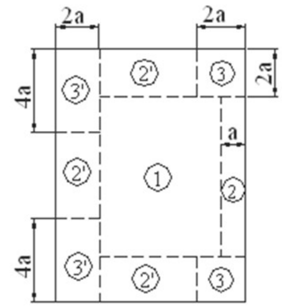
Roofs w/ θ ≤ 10°
and all walls
h > 60'



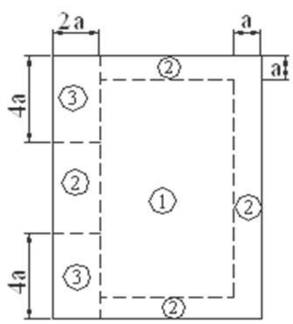
Walls h ≤ 60'
& alt design h<90'



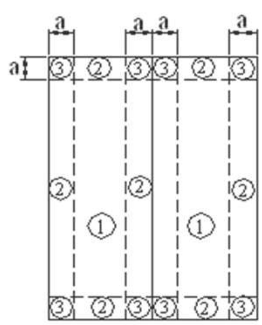
Gable, Sawtooth and
Multispan Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



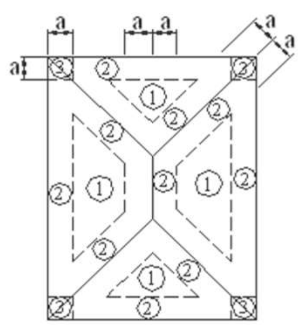
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



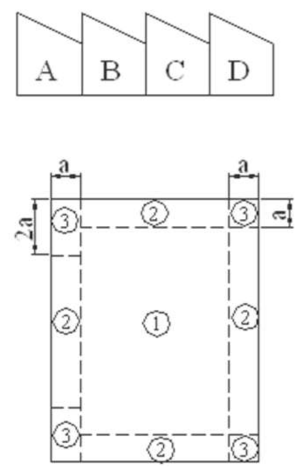
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'



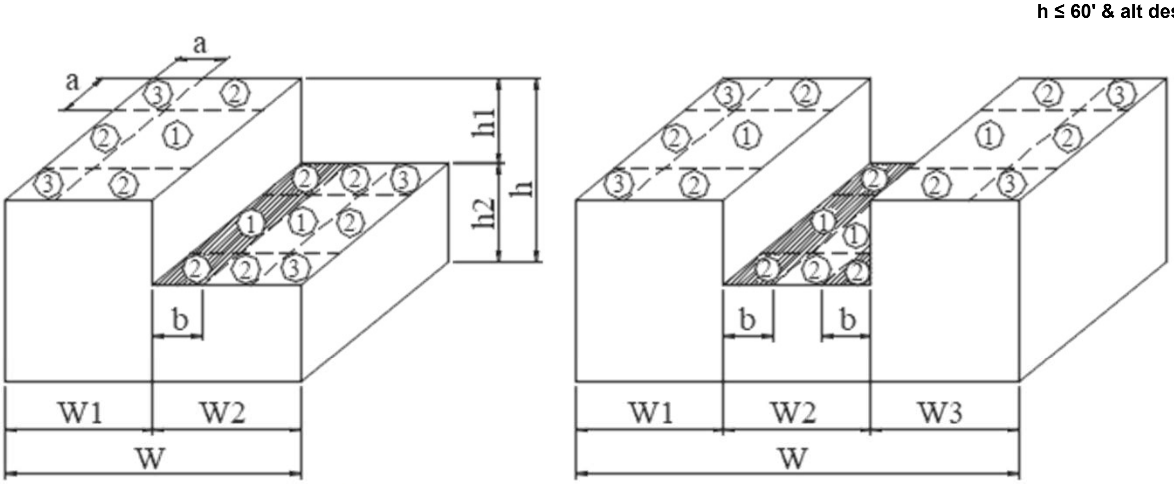
Multispan Gable &
Gable 7° < θ ≤ 45°



Hip 7° < θ ≤ 27°



Sawtooth 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'



Stepped roofs θ ≤ 3°
h ≤ 60' & alt design h<90'

Seismic Loads: ASCE 7- 10

Strength Level Forces

Risk Category : II
Importance Factor (Ie) : 1.00

Site Class : D

Ss (0.2 sec) = 20.60 %g
S1 (1.0 sec) = 9.30 %g

Site specific ground motion analysis performed:

Fa =	1.600	Sms =	0.330	S_{DS} =	0.220	Design Category =	B
Fv =	2.400	Sm1 =	0.223	S_{D1} =	0.149	Design Category =	C

Seismic Design Category = **C**
Redundancy Coefficient ρ = 1.00
Number of Stories: 1
Structure Type: All other building systems
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: Yes
Building System: **Bearing Wall Systems**
Seismic resisting system: **Intermediate reinforced masonry shear walls**
System Structural Height Limit: **Height not limited**
Actual Structural Height (hn) = 17.3 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 3.5
Over-Strength Factor (Ωo) = 2
Deflection Amplification Factor (Cd) = 2.25
S_{DS} = 0.220
S_{D1} = 0.149

Seismic Load Effect (E) = Eh +/-Ev = ρQ_E +/- $0.2S_{DS}$ D = Qe +/- 0.044D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = $\Omega o\ Q_E$ +/- $0.2S_{DS}$ D = 2Qe +/- 0.044D 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 = 1.60
Approx fundamental period (Ta) = $C_T h_n$^ = 0.169 sec x= 0.75 Tmax = CuTa = 0.271 sec
User calculated fundamental period = T = 0.169 sec
Long Period Transition Period (TL) = ASCE7 map = 8 sec
Seismic response coef. (Cs) = SdsI/R = 0.063
need not exceed Cs = Sd1 I /RT = 0.251
but not less than Cs = 0.010
USE Cs = 0.063
Design Base Shear **V = 0.063W** 0.7*0.063 = 0.044W

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

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CHECKED BY		DATE	

Seismic Loads - cont. :

Strength Level Forces

Seismic Design Category (SDC)= C
Ie = 1.00
Sds = 0.220

CONNECTIONS

Force to connect smaller portions of structure to remainder of structure

$F_p = 0.133Sdsw_p =$ 0.029 w_p
or $F_p = 0.05w_p =$ 0.05 w_p Use Fp = 0.05 w_p w_p = weight of smaller portion

Beam, girder or truss connection for resisting horizontal force parallel to member

F_P = no less than 0.05 times dead plus live load vertical reaction

Anchorage of Structural Walls to elements providing lateral support

Fp = not less than 0.2KaIeWp
Flexible diaphragm span Lf = 28.00 ft
Fp =0.4SdsKaIeWp =0.113Wp, but not less than 0.256Wp (flexible diaphragm) ka=1.28 Fp = 0.256 Wp
Fp =0.4SdskaIeWp = 0.088 Wp, but not less than 0.2Wp (rigid diaphragm) ka= 1 Fp = 0.200 Wp
but Fp shall not be less than 5 psf

0.7*0.256=0.180W

MEMBER DESIGN

Bearing Walls and Shear Walls (out of plane force)

Fp = 0.4SdsIeWw = 0.088 w_w
but not less than 0.10 w_w Use Fp = 0.10 w_w 0.7*0.100=0.070W

Diaphragms

Fp = 0.2ISdsWp + Vpx = 0.044 Wp + Vpx

ARCHITECTURAL COMPONENTS SEISMIC COEFFICIENTS

Architectural Component : Cantilever Elements (Unbraced or Braced to Structural Frame Below Its Center of Mass): Parapets and cantilever interior nonstructural walls

Importance Factor (Ip) : 1.0
Component Amplification Factor (a_p) = 2.5 h= 17.3 feet
Comp Response Modification Factor (R_p) = 2.5 z= 17.3 feet z/h = 1.00

$Fp = 0.4a_pSdsIpWp(1+2z/h)/Rp =$ 0.264 Wp
not greater than Fp = 1.6SdsIpWp = 0.352 Wp
but not less than Fp = 0.3SdsIpWp = 0.066 Wp use Fp = 0.264 Wp 0.7*0.264=0.185W

MECH AND ELEC COMPONENTS SEISMIC COEFFICIENTS

Seismic Design Category C & Ip=)1.0, therefore not required

Mech or Electrical Component : Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, etc plus other mechanical components constructed of high-deformability materials.

Importance Factor (Ip) : 1.0
Component Amplification Factor (a_p) = 1 h= 17.3 feet
Comp Response Modification Factor (R_p) = 2.5 z= 17.3 feet z/h = 1.00

$Fp = 0.4a_pSdsIpWp(1+2z/h)/Rp =$ 0.105 Wp
not greater than Fp = 1.6SdsIpWp = 0.352 Wp
but not less than Fp = 0.3SdsIpWp = 0.066 Wp use Fp = 0.105 Wp

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Rain Loads : ASCE 7- 10

Rain Intensity i =	6.62	in/hr
Static Head ds =	2.00	inches
Tributary Roof Area A =	15340	SF

FROM EXISTING BLDG
174.00*256.17 = 44574 SQ FT
FROM EXPANSION
154.33*28.00 = 1440 SQ FT
(44574+1440)/3 = 15338 SQ FT

Flow Rate Q = 1056.1 gal/min

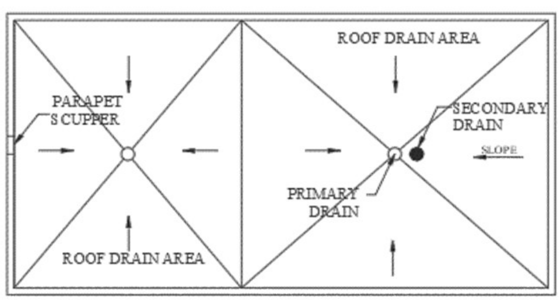
Type of overflow device: **Rectangular Closed Scupper 6" high** width = 24.0 in

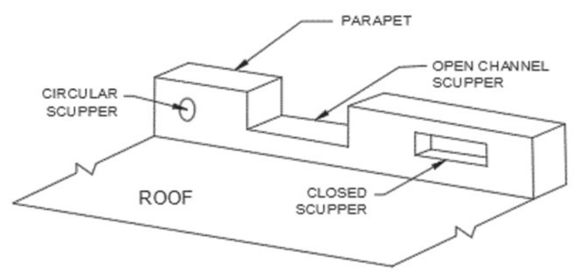
Hydraulic Head dh = 6.19 inches

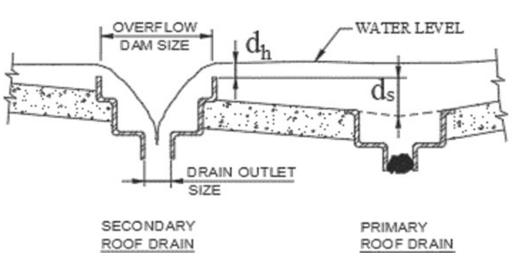
Design Rain Load R = 5.2(ds + dh) = $\boxed{\textbf{42.6 psf}}$ at primary drain

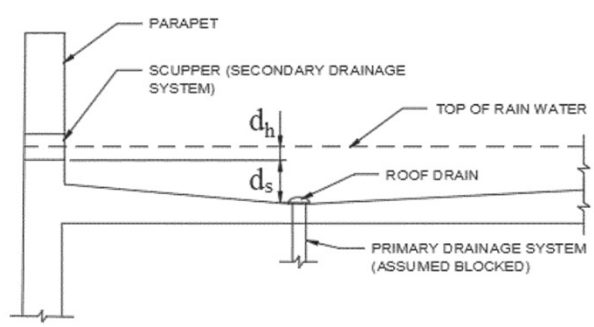
PONDING LOAD LENGTH
42.6/5.2 = 8.19
8.19*4 = 32.8'

TRUNCATED PONDING AT 28'-0"
(42.6/32.8)*(32.8-28.0)=6.3 PSF









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Snow Loads : ASCE 7-10

Nominal Snow Forces

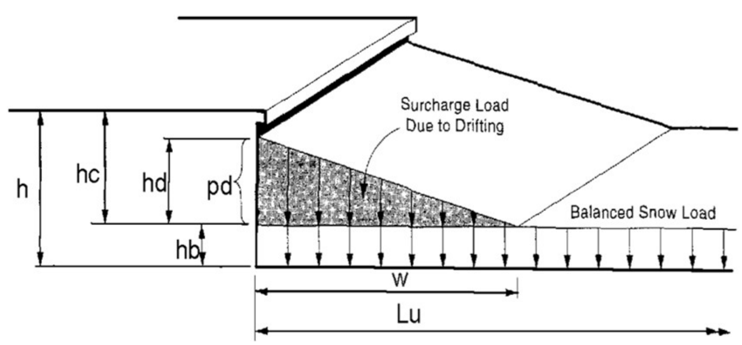
Roof slope		=	1.2 deg
Horiz. eave to ridge dist (W)		=	28.7 ft
Roof length parallel to ridge (L)		=	156.0 ft
Type of Roof			Monoslope
Ground Snow Load	Pg	=	10.0 psf
Risk Category		=	II
Importance Factor	I	=	1.0
Thermal Factor	Ct	=	1.00
Exposure Factor	Ce	=	1.0
Pf = 0.7*Ce*Ct*I*Pg		=	7.0 psf
Unobstructed Slippery Surface			no
Sloped-roof Factor	Cs	=	1.00
Balanced Snow Load		**=**	**7.0 psf**
Rain on Snow Surcharge Angle			0.57 deg
Code Maximum Rain Surcharge			5.0 psf
Rain on Snow Surcharge		=	0.0 psf
Ps plus rain surcharge		=	7.0 psf
Minimum Snow Load	Pm	=	10.0 psf
Uniform Roof Design Snow Load		=	**10.0 psf**

Near ground level surface balanced snow load = **10.0 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	lu	=	
Projection height	h	=	
Projection width/length	lp	=	
Snow density	g	=	15.3 pcf
Balanced snow height	hb	=	0.46 ft
	hd	=	0.73 ft
	hc	=	-0.46 ft
hc/hb <0.2 = -1.0	**lp <15', drift not req'd**		
Drift height (hc)		=	0.00 ft
Drift width	w	=	-4.61 ft
Surcharge load:	pd = γ*hd	=	**0.0 psf**
Balanced Snow load:		=	7.0 psf
			7.0 psf



Windward Snow Drifts 2 - Against walls, parapets, etc

Up or downwind fetch	lu	=	
Projection height	h	=	
Projection width/length	lp	=	
Snow density	g	=	15.3 pcf
Balanced snow height	hb	=	0.46 ft
	hd	=	0.73 ft
	hc	=	-0.46 ft
hc/hb <0.2 = -1.0	**lp <15', drift not req'd**		
Drift height (hc)		=	0.00 ft
Drift width	w	=	-4.61 ft
Surcharge load:	pd = γ*hd	=	**0.0 psf**
Balanced Snow load:		=	7.0 psf
			7.0 psf

Parapet Snow Drift

DL + CLL=		psf
LL =		psf
Uplift		psf
Ground Snow =	10	psf
C_e =	1.0	
C_t =	1.0	
I =	1.0	
h roofing + jst seat	0	in
hb =	0.78	ft
Roof Snow, SL =	12.0	psf[1]
Density =	15.3	pcf

[1] rain on snow added automatically

assumes roof slope <= 1/2:12

	Snow Drift Zone	L_u ft	TOS elev	Parapet elev	h_d ft	h_r ft	h_c ft	Final hd ft	W_d ft	P_{dr} psf	Load Slope psf/ft	Drift Diagram Data: Snow Drift Zone	Drift Diagram Data: DL+SL+P_{dr} psf	Drift Diagram Data: SL+P_{dr} psf
Parapet Drift Only - $(0.75*h_d)$	OP F/R L	154.33	17.25	20.67	2.53	3.42	2.64	2.53	10.13	38.8	3.83	OP F/R L	51	51
	OP F/R R	154.33	16.67	20.67	2.53	4.00	3.22	2.53	10.13	38.8	3.83	OP F/R R	51	51
	OP side L	28	17.25	27.33	0.95	10.08	9.30	0.95	3.78	14.5	3.83	OP side L	26	26
	OP side R	28	16.67	20.67	0.95	4.00	3.22	0.95	3.78	14.5	3.83	OP side R	26	26
					-1.13	0.00	-0.78	0.00	0.00	0.0	0.00	0	12	12
					-1.13	0.00	-0.78	0.00	0.00	0.0	0.00	0	12	12
	Comp 1	574	17.25	27.33	4.54	10.08	9.30	4.54	18.17	69.5	3.83	Comp 1	82	82
					-1.13	0.00	-0.78	0.00	0.00	0.0	0.00	0	12	12
					-1.13	0.00	-0.78	0.00	0.00	0.0	0.00	0	12	12
	None	0	0	0	0	0	0	0	0	0	0	None	12	0
hi/lo Roof Only	Existing at grid 1	574	17.25	27.33	6.06	10.08	9.30	6.06	24.23	92.7	3.83		105	105
					-1.50	0.00	-0.78	0.00	0.00	0.0	0.00		12	12

DRIFT FROM MAIN BUILDING ROOF ALONG GRID 1

Loads on Joists

JST. Spac. in ft.=>	Snow Drift Load at Joist									Total Snow Drift Load at Joist								
Total Dist. =>	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Zone	psf	psf	psf	psf	psf	psf	psf	psf	psf	plf	plf	plf	plf	plf	plf	plf	plf	plf
OP F/R L	38.8	38.8	38.8	38.8	38.8	38.8	38.8	38.8	38.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
OP F/R R	38.8	38.8	38.8	38.8	38.8	38.8	38.8	38.8	38.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
OP side L	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
OP side R	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Loads on Joists

	Max. Snow Load or Live Load on Joist									Total Load on Joist								
Dist. =>	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Zone	plf	plf	plf	plf	plf	plf	plf	plf	plf	plf	plf	plf	plf	plf	plf	plf	plf	plf
OP F/R L	0	0	0	0	0	0	0	0		0	0	0	0	0	0	0	0	
OP F/R R	0	0	0	0	0	0	0	0		0	0	0	0	0	0	0	0	
OP side L	0	0	0	0	0	0	0	0		0	0	0	0	0	0	0	0	
OP side R	0	0	0	0	0	0	0	0		0	0	0	0	0	0	0	0	
0	0	0	0	0	0	0	0	0		0	0	0	0	0	0	0	0	

Project:
Number:

Engineer:
Date:

METAL ROOF DECK ASD VERTICAL GRAVITY LOAD TABLE

80 KSI		5'-0"	5'-3"	5'-6"	5'-9"	6'-0"	6'-3"	6'-6"	6'-9"	7'-0"
3 Span	22	154 PSF	135 PSF	116 PSF	103 PSF	89 PSF	80 PSF	70 PSF	63 PSF	56 PSF
	20	197 PSF	173 PSF	148 PSF	131 PSF	114 PSF	102 PSF	90 PSF	81 PSF	72 PSF
	18	273 PSF	240 PSF	207 PSF	183 PSF	158 PSF	142 PSF	125 PSF	113 PSF	100 PSF
	16	254 PSF	235 PSF	216 PSF	196 PSF	177 PSF	166 PSF	154 PSF	141 PSF	128 PSF
2 Span	22	162 PSF	149 PSF	135 PSF	124 PSF	113 PSF	101 PSF	89 PSF	80 PSF	71 PSF
	20	208 PSF	192 PSF	175 PSF	160 PSF	145 PSF	130 PSF	114 PSF	103 PSF	91 PSF
	18	292 PSF	270 PSF	248 PSF	225 PSF	202 PSF	181 PSF	160 PSF	144 PSF	127 PSF
	16	205 PSF	190 PSF	174 PSF	159 PSF	143 PSF	134 PSF	124 PSF	115 PSF	105 PSF
1 Span	22	72 PSF	63 PSF	54 PSF	48 PSF	42 PSF	38 PSF	33 PSF	30 PSF	26 PSF
	20	94 PSF	82 PSF	70 PSF	62 PSF	54 PSF	49 PSF	43 PSF	39 PSF	34 PSF
	18	138 PSF	121 PSF	103 PSF	92 PSF	80 PSF	71 PSF	62 PSF	56 PSF	50 PSF
	16	186 PSF	166 PSF	146 PSF	127 PSF	108 PSF	98 PSF	88 PSF	78 PSF	68 PSF

NOTES:

1. Loads tables based on a minimum bending yield stength of Fy = 36 KSI for 22, 20, and 18 gage and 20 KSI for 16 gage.
2. Values shown based on manufacturer supplied load tables for Vulcraft and New Millenium.
3. Loads shown provide a maximum live load deflection of L/240.
4. This table is based on 1 1/2" x 36" Type "B" wide rib metal roof deck.
5. Values do not include self weight of metal deck.

MAX BEAM SPACE FOR
DRIFT FROM MAIN BUILDING

METAL ROOF DECK ALLOWABLE SNOW DRIFT LOAD TABLE

Reliable Dead Load = 6.0 PSF
Roof Snow Load = 7.0 PSF

ALLOWABLE DRIFT FOR 5'-8" SPACING
(X-90)/(103-90)=(5.667-5.75)/(5.5-5.75)
X = (5.667-5.75)/(5.5-5.75)*(103-90)+90 = 94.3 PSF

80 KSI		5'-0"	5'-3"	5'-6"	5'-9"	6'-0"	6'-3"	6'-6"	6'-9"	7'-0"
3 Span	22	141 PSF	122 PSF	103 PSF	90 PSF	76 PSF	67 PSF	57 PSF	50 PSF	43 PSF
	20	184 PSF	160 PSF	135 PSF	118 PSF	101 PSF	89 PSF	77 PSF	68 PSF	59 PSF
	18	260 PSF	227 PSF	194 PSF	170 PSF	145 PSF	129 PSF	112 PSF	100 PSF	87 PSF
	16	241 PSF	222 PSF	203 PSF	183 PSF	164 PSF	153 PSF	141 PSF	128 PSF	115 PSF
2 Span	22	149 PSF	136 PSF	122 PSF	111 PSF	100 PSF	88 PSF	76 PSF	67 PSF	58 PSF
	20	195 PSF	179 PSF	162 PSF	147 PSF	132 PSF	117 PSF	101 PSF	90 PSF	78 PSF
	18	279 PSF	257 PSF	235 PSF	212 PSF	189 PSF	168 PSF	147 PSF	131 PSF	114 PSF
	16	192 PSF	177 PSF	161 PSF	146 PSF	130 PSF	121 PSF	111 PSF	102 PSF	92 PSF
1 Span	22	59 PSF	50 PSF	41 PSF	35 PSF	29 PSF	25 PSF	20 PSF	17 PSF	13 PSF
	20	81 PSF	69 PSF	57 PSF	49 PSF	41 PSF	36 PSF	30 PSF	26 PSF	21 PSF
	18	125 PSF	108 PSF	90 PSF	79 PSF	67 PSF	58 PSF	49 PSF	43 PSF	37 PSF
	16	173 PSF	153 PSF	133 PSF	114 PSF	95 PSF	85 PSF	75 PSF	65 PSF	55 PSF

Code Search

Code: ASCE 7 - 10

Occupancy:

Occupancy Group = B Business

Risk Category & Importance Factors:

Risk Category =	III		
Wind factor =	1.00	use 0.60	NOTE: Output will be nominal wind pressures
Snow factor =	1.10		
Seismic factor =	1.25		

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	0.25 / 12	1.2 deg
Building length	573.3 ft	
Least width	322.0 ft	
Mean Roof Ht (h)	23.5 ft	
Parapet ht above grd	28.0 ft	
Minimum parapet ht	1.8 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

Partitions N/A

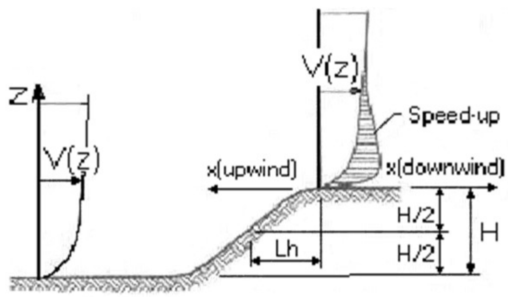
Wind Loads : ASCE 7- 10

Ultimate Wind Speed	120 mph
Nominal Wind Speed	93 mph
Risk Category	III
Exposure Category	B
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.701
Kh case 2	0.653
Type of roof	Monoslope

Topographic Factor (Kzt)

Topography	Flat
Hill Height (H)	1.0 ft
Half Hill Length (Lh)	1.0 ft
Actual H/Lh =	0.00
Use H/Lh =	0.00
Modified Lh =	1.0 ft
From top of crest: x =	1.0 ft
Bldg up/down wind?	downwind

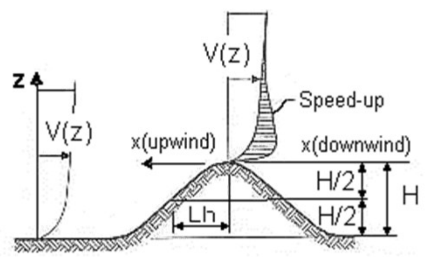
H< 60ft;exp B
∴ Kzt=1.0



ESCARPMENT

H/Lh= 0.00 K_1 = 0.000
x/Lh = 1.00 K_2 = 0.333
z/Lh = 23.50 K_3 = 1.000

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



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h =	23.5 ft
B =	322.0 ft
/z (0.6h) =	30.0 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 = 0.07 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure

ē =	0.33
ℓ =	320 ft
z_{min} =	30 ft
c =	0.30
g_Q, g_v =	3.4
L_z =	310.0 ft
Q =	0.77
I_z =	0.30
G =	**0.79**

Flexible or Dynamically Sensitive Structure

Natural Frequency (η_1) =	0.0 Hz				
Damping ratio (β) =	0				
/b =	0.45				
/α =	0.25				
Vz =	77.3				
N_1 =	0.00				
R_n =	0.000				
R_h =	28.282	η =	0.000	h =	23.5 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				

Johnston Burkholder Associates
930 Central St
Kansas City, MO
(816) 421-4200

JOB TITLE	WM 6958 Cameron, NC Existing Building		
JOB NO.	2431906958	**SHEET NO.**	
CALCULATED BY	DTR	**DATE**	
CHECKED BY		**DATE**	

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

Kz = Kh (case 1) = 0.70
Base pressure (qh) = **13.2 psf**
GCpi = +/-0.18

Edge Strip (a) = 12.9 ft
End Zone (2a) = 25.8 ft
Zone 2 length = 58.8 ft

Wind Pressure Coefficients

Surface	CASE A GCpf	θ = 1.2 deg w/-GCpi	w/+GCpi		CASE B 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

Nominal Wind Surface Pressures (psf)

Surface	CASE A w/-GCpi	CASE A w/+GCpi		CASE B w/-GCpi	CASE B w/+GCpi
1	7.6	2.9		-3.6	-8.3
2	-6.7	-11.5		-6.7	-11.5
3	-2.5	-7.2		-2.5	-7.2
4	-1.4	-6.2		-3.6	-8.3
5				7.6	2.9
6				-1.4	-6.2
1E	10.4	5.7		-4.0	-8.7
2E	-11.7	-16.5		-11.7	-16.5
3E	-4.6	-9.4		-4.6	-9.4
4E	-3.3	-8.0		-4.0	-8.7
5E				10.4	5.7
6E				-3.3	-8.0

Parapet

Windward parapet = 19.8 psf (GCpn = +1.5)
Leeward parapet = -13.2 psf (GCpn = -1.0)

Windward roof overhangs = 9.2 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

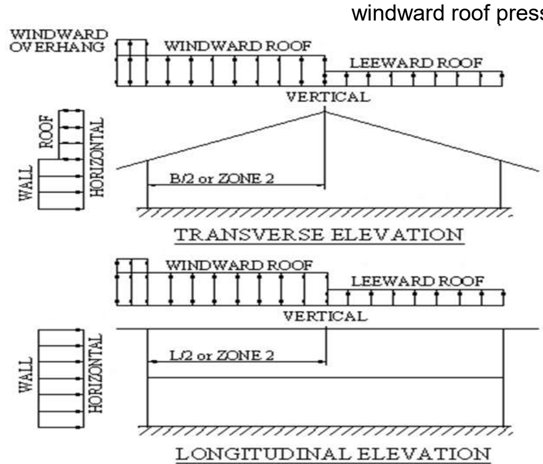
Interior Zone: Wall 9.1 psf
Roof -4.2 psf **
End Zone: Wall 13.7 psf
Roof -7.1 psf **

Longitudinal direction (parallel to L)

Interior Zone: Wall 9.1 psf
End Zone: Wall 13.7 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.



Nominal Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) =	0.70	h =	23.5 ft
Base pressure (qh) =	**13.2 psf**	a =	12.9 ft
Minimum parapet ht =	1.8 ft	GCpi =	+/-0.18
Roof Angle (θ) =	1.2 deg		
Type of roof =	Monoslope		

Roof

	GCp +/- GCpi				Surface Pressure (psf)				User input	
Area	10 sf	50 sf	100 sf	500 sf	10 sf	50 sf	100 sf	500 sf	75 sf	150 sf
Negative Zone 1	-1.18	-1.11	-1.08	-1.08	-15.5	-14.6	-14.2	-14.2	-14.4	-14.2
Negative Zone 2	-1.98	-1.49	-1.28	-1.28	-26.1	-19.6	-16.9	-16.9	-18.0	-16.9
Negative Zone 3	-2.98	-1.79	-1.28	-1.28	-39.3	-23.6	-16.9	-16.9	-19.7	-16.9
Positive All Zones	0.48	0.41	0.38	0.38	10.0	10.0	10.0	10.0	10.0	10.0
Overhang Zone 1&2	-1.7	-1.63	-1.6	-1.1	-22.4	-21.5	-21.1	-14.5	-21.2	-19.4
Overhang Zone 3	-2.8	-1.4	-0.8	-0.8	-36.9	-18.4	-10.5	-10.5	-13.8	-10.5

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 2.4 psf)

Parapet

qp = 13.2 psf

Solid Parapet Pressure	Surface Pressure (psf) 10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	User input 50 sf
CASE A: Zone 2 :	35.6	32.2	27.7	24.2	23.6	22.8	27.7
Zone 3 :	48.7	41.4	31.6	24.2	23.6	22.8	31.6
CASE B: Edge zones 2 :	-24.9	-23.6	-22.0	-20.7	-19.4	-17.8	-22.0
Corner zones 3 :	-28.5	-26.6	-24.1	-22.2	-20.3	-17.8	-24.1

Walls

	GCp +/- GCpi				Surface Pressure (psf)				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	10 sf	55 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-15.4	-13.3	-12.7	-11.9	-15.4	-13.9
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-19.0	-14.8	-13.5	-11.9	-19.0	-15.9
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	14.2	12.1	11.5	10.7	14.2	12.7

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

Seismic Loads: ASCE 7- 10

Strength Level Forces

Risk Category : III
Importance Factor (Ie) : 1.25

Site Class : D

Ss (0.2 sec) = 20.60 %g
S1 (1.0 sec) = 9.30 %g

Site specific ground motion analysis performed:

Fa =	1.600	Sms =	0.330	S_{DS} =	0.220	Design Category =	B
Fv =	2.400	Sm1 =	0.223	S_{D1} =	0.149	Design Category =	C

Seismic Design Category = **C**
Redundancy Coefficient ρ = 1.00
Number of Stories: 1
Structure Type: All other building systems
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: Yes
Building System: **Bearing Wall Systems**
Seismic resisting system: **Intermediate reinforced masonry shear walls**
System Structural Height Limit: **Height not limited**
Actual Structural Height (hn) = 23.5 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 3.5
Over-Strength Factor (Ωo) = 2
Deflection Amplification Factor (Cd) = 2.25
S_{DS} = 0.220
S_{D1} = 0.149

Seismic Load Effect (E) = Eh +/-Ev = ρ Q_E +/- 0.2S_{DS} D = Qe +/- 0.044D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = Ωo Q_E +/- 0.2S_{DS} D = 2Qe +/- 0.044D 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 =	1.60
Approx fundamental period (Ta) =	$C_T h_n$^ =	0.213 sec x= 0.75	Tmax = CuTa =	0.342 sec
User calculated fundamental period =			T =	0.213 sec
Long Period Transition Period (TL) =	ASCE7 map =	8 sec		
Seismic response coef. (Cs) =	SdsI/R =	0.078		
need not exceed Cs =	Sd1 I /RT =	0.249		
but not less than Cs =	0.044SdsI =	0.012		
USE Cs =		0.078		

Design Base Shear V = 0.078W 0.7*0.078 = 0.055W

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

ALLOWABLE STORY DRIFT

Structure Type: All other structures

Allowable story drift Δa = 0.015hsx where hsx is the story height below level x

JOHNSTON BURKHOLDER ASSOCIATES
consulting structural engineers

LATERAL ANALYSIS AND DESIGN

NEW ADDITION LEANS ON EXISTING BUILDING - CHECK EXISTING WALL WITH NEW BUILDING REACTION

930 CENTRAL ST • KANSAS CITY, MO 64105
(816) 421-4200 • WWW.JBAENGR.COM

JOHNSTON BURKHOLDER ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
930 CENTRAL - KANSAS CITY, MO 64105
(P) 816.421.4200 (F) 816.421.4381

[6958 Cameron, NC OPD and Existing Building - LATERAL ANALY

PROJECT INFORMATION:

Project Name:	WM 6958 Cameron, NC OPD and Existing Building
Project Number:	2431906958
Proto:	
Building Code:	
Engineer:	
Date:	

WIND LOADS: Main Wind Force Resisting System (MWFRS)

Windward Wall =	W_w =	7.6	psf
Windward Parapet =	W_{wp} =	19.8	psf
Leeward Wall =	W_L =	1.4	psf
Leeward Parapet =	W_{Lp} =	13.2	psf

Components & Cladding Pressures(100 sf)

Max Wall Suction =	W_s =	14.8	psf
Max Parapet Pressure =	W_{sp} =	24.2	psf

SEISMIC LOADS: **Service Loads**

Base Shear Coefficient =	Transverse =>	SIf_{Bt} =	0.0550	*W
	Longitudinal =>	SIf_{Bl} =	0.0550	*W
Diaphgram Shear Coefficient =	Transverse =>	SIf_{Dt} =	0.0550	*W
	Longitudinal =>	SIf_{Dl} =	0.0550	*W
Collector Shear Coefficient =	Transverse =>	SIf_{Ct} =	0.0550	*W
	Longitudinal =>	SIf_{Cl} =	0.0550	*W
Out-of-Plane Anchorage Coefficient =				
	Transverse =>	SIf_{Ot} =	0.2250	*Wwall
	Longitudinal =>	SIf_{Ol} =	0.2250	*Wwall
	Seismic Dead Load =	SDL =	15.0	psf

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(P) 816.421.4200 (F) 816.421.4381

NEW WIND ADDED TO EXISTING LATERAL (NEW PARAPET ONLY) 156-63=93 PLF

WM 6958 Cameron, NC OPD and Existing Building
Project # 2431906958

Date: 1/0/1900
Engineer: 0

WIND LOADS

Wall No.	Wall Height (JBE + 5") (ft)	TOM (ft)	Parapet Height (ft)	Roof Length (Seismic) (ft)	Diaphgram Wind Loads (Windward at front)		Wind Loads (Leeward at front)
	Right wall						
1)	H_{w1} = 16.50	20.67	H_{p1} = 4.17	L_1 = 28.00	$w=W_w\frac{H_w}{2}+W_{wp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_1 = 156 plf	w_1 = 74 plf
	Right wall seismic (no roof)						
2)	H_{w2} = 16.50	20.67	H_{p2} = 4.17	L_2 = 0.00	$w=W_w\frac{H_w}{2}+W_{wp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_2 = 156 plf	w_2 = 74 plf
	New right wall without parapet						
3)	H_{w3} = 16.50	16.50	H_{p3} = 0.00	L_3 = 0.00	$w=W_w\frac{H_w}{2}+W_{wp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_3 = 63 plf	w_3 = 12 plf
	Front wall at right						
4)	H_{w4} = 17.08	20.67	H_{p4} = 3.59	L_4 = 154.67	$w=W_w\frac{H_w}{2}+W_{wp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_4 = 144 plf	w_4 = 65 plf
	Front wall at left						
5)	H_{w5} = 16.50	20.67	H_{p5} = 4.17	L_5 = 154.67	$w=W_w\frac{H_w}{2}+W_{wp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_5 = 156 plf	w_5 = 74 plf
	Rear wall at right						
6)	H_{w6} = 16.50	20.67	H_{p6} = 4.17	L_6 = 0.00	$w=W_L\frac{H_w}{2}+W_{Lp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_6 = 74 plf	w_6 = 156 plf
	Rear wall at left						
7)	H_{w7} = 17.08	20.67	H_{p7} = 3.59	L_7 = 0.00	$w=W_L\frac{H_w}{2}+W_{Lp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_7 = 65 plf	w_7 = 144 plf
8)	H_{w8} =		H_{p8} = 0.00	L_8 =	$w=W_w\frac{H_w}{2}+W_{wp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_8 = #DIV/0! plf	w_8 = #### plf
	Existing grid 1 to exp joint						
9)	H_{w9} = 17.75	25.33	H_{p9} = 7.58	L_9 = 256.17	$w=W_w\frac{H_w}{2}+W_{wp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_9 = 250 plf	w_9 = 134 plf
	Existing grid 1 to exp joint (raised)						
10)	H_{w10} = 17.75	27.67	H_{p10} = 9.92	L_{10} = 256.17	$w=W_w\frac{H_w}{2}+W_{wp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_{10} = 319 plf	w_{10} = 180 plf
	Existing grid 1 to exp joint (raised 12" CMU)						
11)	H_{w11} = 17.75	27.67	H_{p11} = 9.92	L_{11} = 256.17	$w=W_w\frac{H_w}{2}+W_{wp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_{11} = 319 plf	w_{11} = 180 plf
	Existing grid 12/13 to exp joint						
12)	H_{w12} = 17.75	25.33	H_{p12} = 7.58	L_{12} = 317.83	$w=W_L\frac{H_w}{2}+W_{Lp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_{12} = 134 plf	w_{12} = 250 plf
13)	H_{w13} =		H_{p13} = 0.00	L_{13} =	$w=W_L\frac{H_w}{2}+W_{Lp}H_p\frac{(2H_w+H_p)}{2H_w}$	w_{13} = #DIV/0! plf	w_{13} = #### plf

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CONSULTING STRUCTURAL ENGINEERS
930 CENTRAL - KANSAS CITY, MO 64105
(P) 816.421.4200 (F) 816.421.4381

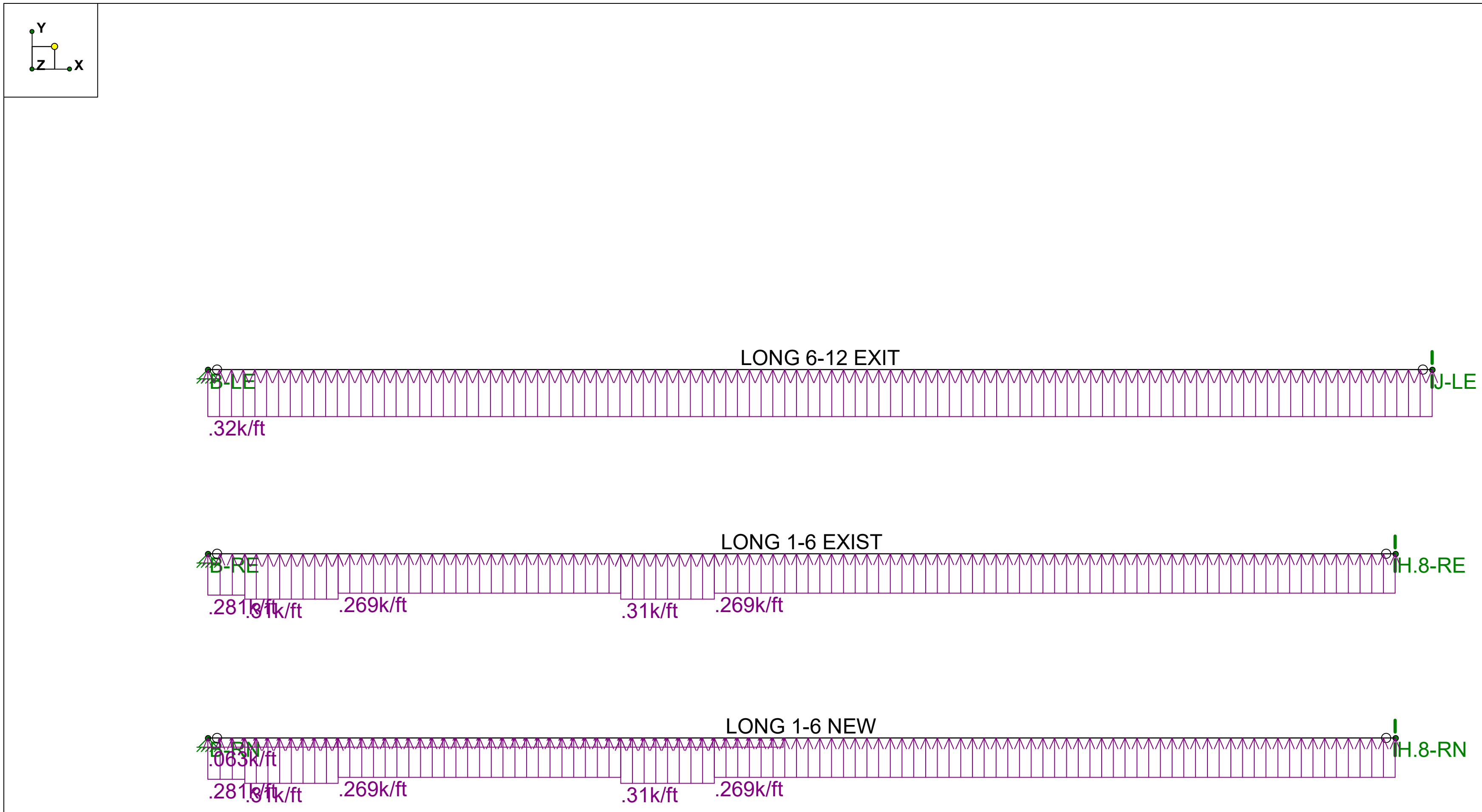
NEW SEISMIC ON EXISTING
ANCHORAGE =
63*0.225/0.055=258 PLF
258/0.7*4' = 1475# PER ANCHOR

WM 6958 Cameron, NC OPD and Existing Building
Project #: 2431906958

Date: 1/0/1900
Engineer: 0

SEISMIC LOADS

	Wall Height (JBE + 5") (ft)	Parapet Height (ft)	Roof Length (Seismic) (ft)	Wall Weight (psf)	Diaphgram Loads		Base	Diaphragm	Collector	
	Right wall									
1)	H_{w1} = 16.50	H_{p1} = 4.17	L_1 = 28.00	Wt_1 = 55.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_1 =	63	63	63	plf
	Right wall seismic (no roof)									
2)	H_{w2} = 16.50	H_{p2} = 4.17	L_2 = 0.00	Wt_2 = 55.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_2 =	40	40	40	plf
	New right wall without parapet									
3)	H_{w3} = 16.50	H_{p3} = 0.00	L_3 = 0.00	Wt_3 = 55.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_3 =	25	25	25	plf
	Front wall at right									
4)	H_{w4} = 17.08	H_{p4} = 3.59	L_4 = 154.67	Wt_4 = 55.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_4 =	166	166	166	plf
	Front wall at left									
5)	H_{w5} = 16.50	H_{p5} = 4.17	L_5 = 154.67	Wt_5 = 55.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_5 =	167	167	167	plf
	Rear wall at right									
6)	H_{w6} = 16.50	H_{p6} = 4.17	L_6 = 0.00	Wt_6 = 55.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_6 =	40	40	40	plf
	Rear wall at left									
7)	H_{w7} = 17.08	H_{p7} = 3.59	L_7 = 0.00	Wt_7 = 55.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_7 =	38	38	38	plf
	0									
8)	H_{w8} = 0.00	H_{p8} = 0.00	L_8 = 0.00	Wt_8 =	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_8 =	####	####	####	plf
	Existing grid 1 to exp joint									
9)	H_{w9} = 17.75	H_{p9} = 7.58	L_9 = 256.17	Wt_9 = 58.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_9 =	269	269	269	plf
	Existing grid 1 to exp joint (raised)									
10)	H_{w10} = 17.75	H_{p10} = 9.92	L_{10} = 256.17	Wt_{10} = 58.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_{10} =	281	281	281	plf
	Existing grid 1 to exp joint (raised 12" CMU)									
11)	H_{w11} = 17.75	H_{p11} = 9.92	L_{11} = 256.17	Wt_{11} = 83.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_{11} =	310	310	310	plf
	Existing grid 12/13 to exp joint									
12)	H_{w12} = 17.75	H_{p12} = 7.58	L_{12} = 317.83	Wt_{12} = 58.00	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_{12} =	320	320	320	plf
	0									
13)	H_{w13} = 0.00	H_{p13} = 0.00	L_{13} = 0.00	Wt_{12} =	$s=(slf_b*Wt)\frac{H_w}{2}+(slf_b*Wt)H_p\frac{(2H_w+H_p)}{2H_w}+slf_b*SDS*L$	s_{13} =	####	####	####	plf



Loads: BLC 2, Seismic (Right wall)

Johnston Burkholder Associates		
		Dec 30, 2024 at 10:20 PM
		6958 - Lateral Forces.r2d

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distributed
1	Wind (Windward-Right wall)	None					11
2	Seismic (Right wall)	None					12

Member Distributed Loads (BLC 1 : Wind (Windward-Right wall))

	Member Label	Direction	Start Magnitude[k/ft,F,ksf]	End Magnitude[k/ft,...	Start Location[ft,%]	End Location[ft,%]
1	LONG 1-6 NEW	Y	.319	.319	0	10
2	LONG 1-6 NEW	Y	.319	.319	10	35.33
3	LONG 1-6 NEW	Y	.25	.25	35.33	112
4	LONG 1-6 NEW	Y	.319	.319	112	137.33
5	LONG 1-6 NEW	Y	.25	.25	137.33	322
6	LONG 1-6 NEW	Y	.093	.093	0	156
7	LONG 1-6 EXIST	Y	.319	.319	0	10
8	LONG 1-6 EXIST	Y	.319	.319	10	35.33
9	LONG 1-6 EXIST	Y	.25	.25	35.33	112
10	LONG 1-6 EXIST	Y	.319	.319	112	137.33
11	LONG 1-6 EXIST	Y	.25	.25	137.33	322

Member Distributed Loads (BLC 2 : Seismic (Right wall))

	Member Label	Direction	Start Magnitude[k/ft,F,ksf]	End Magnitude[k/ft,...	Start Location[ft,%]	End Location[ft,%]
1	LONG 1-6 NEW	Y	.281	.281	0	10
2	LONG 1-6 NEW	Y	.31	.31	10	35.33
3	LONG 1-6 NEW	Y	.269	.269	35.33	112
4	LONG 1-6 NEW	Y	.31	.31	112	137.33
5	LONG 1-6 NEW	Y	.269	.269	137.33	322
6	LONG 1-6 NEW	Y	.063	.063	0	156
7	LONG 6-12 EXIT	Y	.32	.32	0	0
8	LONG 1-6 EXIST	Y	.281	.281	0	10
9	LONG 1-6 EXIST	Y	.31	.31	10	35.33
10	LONG 1-6 EXIST	Y	.269	.269	35.33	112
11	LONG 1-6 EXIST	Y	.31	.31	112	137.33
12	LONG 1-6 EXIST	Y	.269	.269	137.33	322

Load Combinations

	Description	So..	P...	S...	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac...
1	Wind (Windward-Left)	Yes			1	1																		
2	Seismic	Yes			2	1																		

Joint Reactions

	LC	Joint Label	X [k]	Y [k]	MZ [k-ft]
1	1	B-RN	0	-54.619	0
2	1	H.8-RN	0	-44.575	0
3	1	B-LE	0	0	0
4	1	J-LE	0	0	0
5	1	B-RE	0	-43.625	0
6	1	H.8-RE	0	-41.06	0
7	1	Totals:	0	-183.879	
8	1	COG (ft):	X: 149.96	Y: -26.972	
9	2	B-RN	0	-52.476	0
10	2	H.8-RN	0	-46.167	0
11	2	B-LE	0	-53.12	0
12	2	J-LE	0	-53.12	0

Joint Reactions (Continued)

	LC	Joint Label	X [k]	Y [k]	MZ [k-ft]
13	2	B-RE	0	-45.029	0
14	2	H.8-RE	0	-43.786	0
15	2	Totals:	0	-293.698	
16	2	COG (ft):	X: 158.668	Y: 1.293	

Member: ***LONG 1-6 NEW***

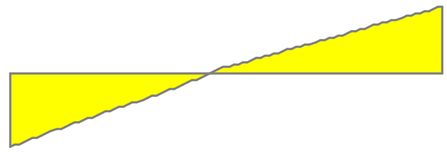
Shape:
Material: **RIGID**
Length: **322 ft**
I Joint: **B-RN**
J Joint: **H.8-RN**

Code Check: **No Calc**
Report Based On 96 Sections

A **k**

fa **ksi**

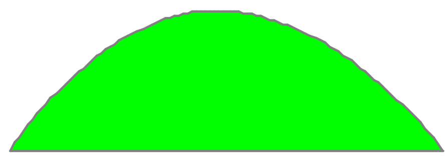
46.167 at 322 ft

V  **k**

-52.476 at 0 ft

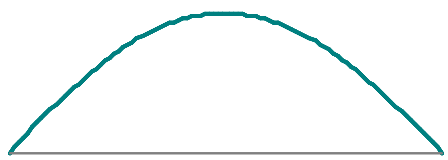
fc **ksi**

3960.648 at 152.526 ft

M  **k-ft**

ft **ksi**

0 at 159.305 ft

D  **in**

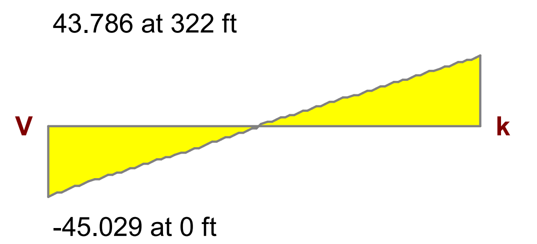
Member: ***LONG 1-6 EXIST***

Shape:
Material: **RIGID**
Length: **322 ft**
I Joint: **B-RE**
J Joint: **H.8-RE**

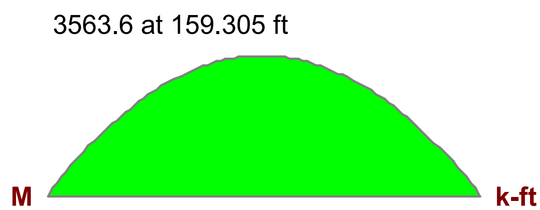
Code Check: **No Calc**
Report Based On 96 Sections

A **k**

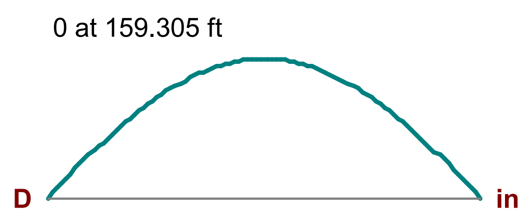
fa **ksi**



fc **ksi**



ft **ksi**



Member: ***LONG 6-12 EXIT***

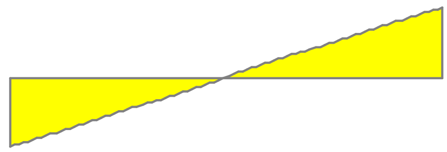
Shape:
Material: **RIGID**
Length: **332 ft**
I Joint: **B-LE**
J Joint: **J-LE**

Code Check: **No Calc**
Report Based On 96 Sections

A **k**

fa **ksi**

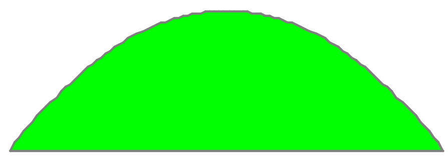
53.12 at 332 ft

V

k

-53.12 at 0 ft

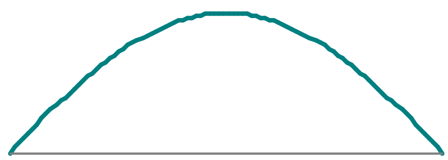
fc **ksi**

4408.471 at 164.253 ft

M

k-ft

ft **ksi**

0 at 164.253 ft

D

in

LATERAL ANALYSIS

Overall Building Dimensions --	Length =	156.00 ft	Width =	28.67 ft	F&B wall thickness = 8.00 in
					Side wall thickness = 8.00 in

FRONT TO BACK ANALYSIS

Shear walls each side

Diaphragm width =	28.67 ft		Diaphragm depth =	154.67	
WIND			SEISMIC		
Wall reaction at roof (WW front) =	221 plf		Sesimic reaction at roof =	206 plf	(service)
Wall reaction at roof (WW rear) =	221 plf		Diaphragm Seismic Reaction =	2.95 k	
Controlling wall reaction at roof =	221 plf	(service)	Base shear =	19 plf	
Diaphragm Wind Reaction =	3.17 k		Diaphragm shear =	19 plf	
Shear =	20 plf		Moment =	21.2 k-ft	
Moment =	22.7 k-ft		Chord T/C =	0.14 k	
Chord T/C =	0.15 k				

SIDE TO SIDE ANALYSIS

Leans on existing building

NEW WIND ON EXISTING FOR WALL ANCHORAGE = 236/0.6*4' = 1573# PER ANCHOR

WALL ANCHORAGE

C&C wall wind =	14.8	(factored)	Roof ht =	16.50 ft	Wall reaction at roof =	236 plf
C&C paraepet wind =	24.2	(factored)	TOW =	20.67 ft		
			Parapet ht =	4.17 ft		

Seismic wall anchorage Fp =	0.225 W	(factored)	Wall seismic load =	12.4 psf
			Roof seismic load =	3.4 psf

JOIST AXIAL LOAD FOR WALL ANCHORAGE (FROM WALL SEISMIC ONLY)

Side to Side - Use each joist for the continuous tie/sub-diaphragms

Side wall anchorage reaction at roof =	164 plf			
Joist spacing =	5.50 ft	Seismic subdiaphragm axial load =	0.90 k	Each joist
			1.29 k	(Ultimate for RISA)

Front to back - Use deck ribs

Max wall reaction at roof =	164 plf
Embed spacing =	4.00 ft
Reaction at each embed =	0.65 k

EXISTING BUILDING SIDE TO SIDE ANALYSIS WITH ADDITION (GRID 1 TO EXP JOINT)

NEW SEISMIC

Existing Diaphragm width =	322.00 ft		Diaphragm depth =	256.17	
WIND			SEISMIC		
Diaphragm Reaction Grid B-R =	54.62 k	(from RISA)	Diaphragm Reaction Grid B-R =	52.48 k	(from RISA)
Diaphragm Reaction Grid H.8-R =	44.58 k	(from RISA)	Diaphragm Reaction Grid H.8-R =	46.17 k	(from RISA)
Diaphragm shear Grid B =	213 plf		Diaphragm shear Grid B =	205 plf	
Diaphragm shear Grid H.8 =	174 plf		Diaphragm shear Grid H.8 =	180 plf	
Max Moment =	3968.4 k-ft	(from RISA)	Max Moment =	3960.7 k-ft	(from RISA)
Chord T/C =	15.49 k		Chord T/C =	15.46 k	

EXISTING BUILDING SIDE TO SIDE ANALYSIS WITHOUT ADDITION (GRID 1 TO EXP JOINT)

SEISMIC		
Diaphragm Reaction Grid B-L =	45.03 k	(from RISA)
Diaphragm Reaction Grid J-L =	43.79 k	(from RISA)
Diaphragm shear Grid B =	176 plf	
Diaphragm shear Grid H.8 =	171 plf	
Max Moment =	3563.6 k-ft	(from RISA)
Chord T/C =	13.91 k	

EXISTING BUILDING SIDE TO SIDE ANALYSIS FOR SEISMIC DEFLECTION (EXP JOINT TO GRID 12/13)

Existing Diaphragm width =	332.00 ft		Diaphragm depth =	317.83	
			SEISMIC		
			Diaphragm Reaction Grid B-L =	53.12 k	(from RISA)
			Diaphragm Reaction Grid J-L =	53.12 k	(from RISA)
			Diaphragm shear Grid B =	167 plf	
			Diaphragm shear Grid H.8 =	167 plf	
			Max Moment =	4408.5 k-ft	(from RISA)
			Chord T/C =	13.87 k	

Project:		Date:	
Number:		Engineer:	

DIAPHRAGM DESIGN SUMMARY (EXISTING BUILDING)

Wind Uplift & Maximum Deck Shears (from lateral analysis):

Transverse Wind =		Longitudinal Wind =	213 plf
Transverse Seismic =		Longitudinal Seismic =	205 plf

Include Tension Check =	Yes	
MWFRS Uplift (Service) =	14.3 psf	Positive Interior Pressure, Edge Zone
C&C (<10 ft^2) (Service) =	28.4 psf	Positive Interior Pressure, Edge Zone

Diaphragm Design Input: <-- Based on 1 1/2" x 36" Type "B" Wide Rib Metal Roof Deck -->

Deck Gage =	22 GA
Fastener Pattern =	36-7
Sidelap Fastener =	#10 Screws - Proprietary
Deck Grade =	Grade 80
Average Support Spacing =	6.00 ft
Edge Support Fastener Spacing =	6.00 in
Maximum Wind Shear =	213 plf
Maximum Seismic Shear =	205 plf

ASTM A1008
F_y = 60 ksi
F_u = 62 ksi
F_{xx} = 60 ksi

	Screw/PAF Factor of Safety	Weld Factor of Safety
Wind =	2.35	2.35
Seismic =	2.50	3.00

Diaphragm Design Results:

EXISTING DECK FASTENERS REQUIRED FOR NEW WIND/SEISMIC

	SUPPORT FASTENER TYPE	SIDELAP FASTENER SPACING	ALLOWABLE DIAPHRAGM SHEAR - WIND	ALLOWABLE DIAPHRAGM SHEAR - SEISMIC
WELDS	5/8" VISIBLE DIAMETER PUDDLE WELDS	(1) AT (2) EQ SPACES	372 PLF	292 PLF
	3/4" VISIBLE DIAMETER PUDDLE WELDS	(1) AT (2) EQ SPACES	440 PLF	345 PLF
SCREWS	#12-14 SELF DRILLING SCREWS	(1) AT (2) EQ SPACES	251 PLF	240 PLF
	#12-24 SELF DRILLING SCREWS	(1) AT (2) EQ SPACES	255 PLF	240 PLF
	SIMPSON XL (0.125")	(1) AT (2) EQ SPACES	270 PLF	258 PLF
	SIMPSON XL (0.250")	(1) AT (2) EQ SPACES	300 PLF	282 PLF
PAF	HILTI X-HSN-24	(1) AT (2) EQ SPACES	215 PLF	232 PLF
	HILTI X-ENP-19-L15	(1) AT (2) EQ SPACES	230 PLF	248 PLF
	PNEUTEK SDK-61075	(1) AT (2) EQ SPACES	219 PLF	238 PLF
	PNEUTEK SDK-63075	(1) AT (2) EQ SPACES	243 PLF	262 PLF
	PNEUTEK K-66062	(1) AT (2) EQ SPACES	253 PLF	274 PLF

Design Notes:

1. All calculations are based on the AISI S100 and the SDI 4th Edition Diaphragm Design Manual (DDM04).
2. All diaphragm shears listed above include the applicable factor of safety.
3. All support fasteners shall be one of the fasteners listed above.
4. Maximum sidelap spacing is 36" for support spans greater than 5'-0".
5. All sidelap fasteners shall be self drilling, self tapping screws manufactured by Buildex, Elco, Hilti, or Simpson.

Project:		Engineer:	
Number:		Date:	

STEEL ROOF DECK DIAPHRAGM DEFLECTION CALCULATION - EXISTING LEFT

INPUT - DIAPHRAGM STIFFNESS

Deck Gage =	22 GA	
Support Fastener =	0.625 in Arc Welds	
Sidelap Fastener =	#10 Screws - Proprietary	
Fastener Pattern =	36-7	
Deck Grade =	Grade 80	
Avg. Support Spacing =	6.00 ft	= L_{span}
Number of Sidelaps =	2	

ASTM A1008	
F_y =	60 ksi
F_u =	62 ksi
F_{xx} =	60 ksi

GENERAL PARAMETERS

ASTM Steel Type =	A1008		Yield and Tensile Strength
E =	29,000	ksi	Modulus of Elasticity
D_d =	1.5	in	Depth of Wide Rib Deck Profile
t =	0.0295	in	Thickness of metal deck
w =	36.00	in	Deck panel width
Number of Spans =	3		Assume 3 span condition
L_{sheet} =	18.00	ft	Min length of deck sheet
n_p =	2		Number of interior supports
n_s =	6		Total number of sidelap fasteners
α =	2.00		exterior support fastener group distribution
d_{wr} =	6.00	in	Corrugation pitch for wide rib deck
s =	8.18	in	Developed width of corrugation per pitch d_{wr}
D =	1237	in	Warping Coefficient
ρ =	0.900		Assume 3 span condition

SDI Stiffness Equations

$$C = E * t * \frac{S_f}{w} * \frac{2}{2 * \alpha + n_p * \alpha + 2 * n_s * \frac{S_f}{S_s}} \qquad K_1 = \frac{C}{3 * L_{span}} * (L_{sheet} * 12)$$

$$G' = \frac{E * t}{2.6 * \frac{s}{d} + \rho * \frac{D}{L_{sheet} * 12} + C * L_{sheet} * 12}$$

DIAPHRAGM STIFFNESS RESULTS

Calculated Per SDI Criteria:

S_f =	0.0067	in/kip	C =	0.0253	1/in
S_s =	0.0175	in/kip	K_1 =	0.3031	1/ft
			G' =	60.44	kip/in

Final Design: SDI Criteria

G' =	**60.44**	**kip/in**
F =	16.54	10^6 in/lbs
Diaphragm =	Semi-Flexible	

Project:	0	Engineer:	0
Number:	0	Date:	1/0/1900

INPUT - DIAPHRAGM DEFLECTION

Diaphragm Span L_d =	332.00	ft
Diaphragm Depth B_d =	317.83	ft
Front Steel Chord Area A_{cf} =	1.94	in^2
Rear Steel Chord Area A_c =	1.94	in^2
Wind Max Deck Shear (Service) =		plf
Wind Max Diaphragm Moment (Service) =		kip*ft
Seismic Max Deck Shear (Service) =	167	plf
Seismic Max Diaphragm Moment (Service) =	4,409	kip*ft
Diaphragm Support Deflection =	0.125	in
Roof Height at Maximum Deflection H =	17.75	ft

Wind Factors	
Deflection =	0.70

Seismic Factors	
Cd =	2.25
I =	1.25
ρ =	1.00
ASD =	0.70

Estimated Moment of Inertia		
I_{chord} =	1.41E+07	in^4
I_{deck} =	1.36E+08	in^4

DIAPHRAGM DEFLECTION - WIND

Shear Deflection (Service)

V_{wind} =	0	plf	Deck Shear
G' =	60.44	kip/in	Shear Modulus
Δ_s =	0.0000	in	Shear Deflection

Bending Deflection (Service)

$q_{avg.w}$ =	0	plf	Based on Moment
Δ_b =	0.0000	in	Bending Deflection

Total Deflection (Service)

$\delta_{t.wind}$ = 0.13 in

DIAPHRAGM DEFLECTION - SEISMIC

Shear Deflection (Ultimate)

V_{eq} =	191	plf	Deck Shear
G' =	60.44	kip/in	Shear Modulus
Δ_s =	0.2621	in	Shear Deflection

Bending Deflection (Ultimate)

$q_{avg.eq}$ =	366	plf	Based on Moment
Δ_b =	0.0229	in	Bending Deflection

Total Deflection (with Seismic Amplification)

$\delta_{t.eq}*C_d$ = 0.92 in

ALLOWABLE STORY DRIFT

δ =	2.13	in	0.010*H for masonry walls

MINIMUM BUILDING SEPARATION

δ_m =	0.92	in	Min Property Line Offset
$\Delta_{t.bldg1}$ =	0.92	in	
$\Delta_{t.bldg2}$ =		in	
δ_{mt} =	0.92	in	Min Expansion Joint Width

EXISTING DIAPHRAGM DEFLECTION
LEFT SIDE OF EXPANSION JOINT

EQUATIONS

$$I_{chord} = A_C * \left(\frac{B_d * 12}{2}\right)^2$$

$$I_{deck} = \frac{1}{12} * t * (B_d * 12)^3$$

$$\Delta_s = \frac{Vmax/_{1000} * 0.5 * L_d}{2 * G'}$$

$$\Delta_b = \frac{5 * {}^{q_{avg}}/_{12} * (L_d * 12)^4}{384 * E * I * 1000}$$

$$\delta_{mt} = \sqrt{\Delta_{t.bldg1}{}^2 + \Delta_{t.bldg2}{}^2}$$

1. All calculations are based on the AISI S100 and the SDI 4th Edition Diaphragm Design Manual (DDM04).
2. All sidelap fasteners shall be self drilling, self tapping screws manufactured by Buildex, Elco, Hilti, or Simpson.

Project:		Engineer:	
Number:		Date:	

STEEL ROOF DECK DIAPHRAGM DEFLECTION CALCULATION - EXISTING RIGHT

INPUT - DIAPHRAGM STIFFNESS

Deck Gage =	22 GA	
Support Fastener =	0.625 in Arc Welds	
Sidelap Fastener =	#10 Screws - Proprietary	
Fastener Pattern =	36-7	
Deck Grade =	Grade 80	
Avg. Support Spacing =	6.00 ft	$= L_{span}$
Number of Sidelaps =	2	

ASTM A1008	
F_y =	60 ksi
F_u =	62 ksi
F_{xx} =	60 ksi

GENERAL PARAMETERS

ASTM Steel Type =	A1008		Yield and Tensile Strength
E =	29,000	ksi	Modulus of Elasticity
D_d =	1.5	in	Depth of Wide Rib Deck Profile
t =	0.0295	in	Thickness of metal deck
w =	36.00	in	Deck panel width
Number of Spans =	3		Assume 3 span condition
L_{sheet} =	18.00	ft	Min length of deck sheet
n_p =	2		Number of interior supports
n_s =	6		Total number of sidelap fasteners
α =	2.00		exterior support fastener group distribution
d_{wr} =	6.00	in	Corrugation pitch for wide rib deck
s =	8.18	in	Developed width of corrugation per pitch d_{wr}
D =	1237	in	Warping Coefficient
ρ =	0.900		Assume 3 span condition

SDI Stiffness Equations

$$C = E * t * \frac{S_f}{w} * \frac{2}{2 * \alpha + n_p * \alpha + 2 * n_s * \frac{S_f}{S_s}} \qquad K_1 = \frac{C}{3 * L_{span}} * (L_{sheet} * 12)$$

$$G' = \frac{E * t}{2.6 * \frac{s}{d} + \rho * \frac{D}{L_{sheet} * 12} + C * L_{sheet} * 12}$$

DIAPHRAGM STIFFNESS RESULTS

Calculated Per SDI Criteria:

S_f =	0.0067	in/kip	C =	0.0253	1/in
S_s =	0.0175	in/kip	K_1 =	0.3031	1/ft
			G' =	60.44	kip/in

Final Design: SDI Criteria

G' =	**60.44**	**kip/in**
F =	16.54	10^6 in/lbs
Diaphragm =	Semi-Flexible	

Project:	0	Engineer:	0
Number:	0	Date:	1/0/1900

INPUT - DIAPHRAGM DEFLECTION

Diaphragm Span L_d =	322.00	ft
Diaphragm Depth B_d =	256.17	ft
Front Steel Chord Area A_{cf} =	1.94	in^2
Rear Steel Chord Area A_c =	1.94	in^2
Wind Max Deck Shear (Service) =		plf
Wind Max Diaphragm Moment (Service) =		kip*ft
Seismic Max Deck Shear (Service) =	176	plf
Seismic Max Diaphragm Moment (Service) =	3,564	kip*ft
Diaphragm Support Deflection =	0.125	in
Roof Height at Maximum Deflection H =	17.75	ft

Wind Factors	
Deflection =	0.70

Seismic Factors	
Cd =	2.25
I =	1.25
ρ =	1.00
ASD =	0.70

Estimated Moment of Inertia		
I_{chord} =	9.17E+06	in^4
I_{deck} =	7.14E+07	in^4

DIAPHRAGM DEFLECTION - WIND

Shear Deflection (Service)

V_{wind} =	0	plf	Deck Shear
G' =	60.44	kip/in	Shear Modulus
Δ_s =	0.0000	in	Shear Deflection

Bending Deflection (Service)

$q_{avg.w}$ =	0	plf	Based on Moment
Δ_b =	0.0000	in	Bending Deflection

Total Deflection (Service)

$\delta_{t.wind}$ = 0.13 in

DIAPHRAGM DEFLECTION - SEISMIC

Shear Deflection (Ultimate)

V_{eq} =	201	plf	Deck Shear
G' =	60.44	kip/in	Shear Modulus
Δ_s =	0.2679	in	Shear Deflection

Bending Deflection (Ultimate)

$q_{avg.eq}$ =	314	plf	Based on Moment
Δ_b =	0.0325	in	Bending Deflection

Total Deflection (with Seismic Amplification)

$\delta_{t.eq} * C_d$ = 0.96 in

ALLOWABLE STORY DRIFT

δ =	2.13	in	0.010*H for masonry walls

MINIMUM BUILDING SEPARATION

δ_m =	0.96	in	Min Property Line Offset
$\Delta_{t.bldg1}$ =	0.96	in	
$\Delta_{t.bldg2}$ =	0.92	in	
δ_{mt} =	1.33	in	Min Expansion Joint Width

EXISTING DIAPHRAGM DEFLECTION
RIGHT SIDE OF EXPANSION JOINT

EXISTING TOTAL DEFLECTION

EQUATIONS

$$I_{chord} = A_C * \left(\frac{B_d * 12}{2}\right)^2$$

$$I_{deck} = \frac{1}{12} * t * (B_d * 12)^3$$

$$\Delta_s = \frac{{}^{Vmax}/_{1000} * 0.5 * L_d}{2 * G'}$$

$$\Delta_b = \frac{5 * {}^{q_{avg}}/_{12} * (L_d * 12)^4}{384 * E * I * 1000}$$

$$\delta_{mt} = \sqrt{\Delta_{t.bldg1}{}^2 + \Delta_{t.bldg2}{}^2}$$

1. All calculations are based on the AISI S100 and the SDI 4th Edition Diaphragm Design Manual (DDM04).
2. All sidelap fasteners shall be self drilling, self tapping screws manufactured by Buildex, Elco, Hilti, or Simpson.

Project:		Engineer:	
Number:		Date:	

STEEL ROOF DECK DIAPHRAGM DEFLECTION CALCULATION - NEW RIGHT

INPUT - DIAPHRAGM STIFFNESS

Deck Gage =	22 GA	
Support Fastener =	0.625 in Arc Welds	
Sidelap Fastener =	#10 Screws - Proprietary	
Fastener Pattern =	36-7	
Deck Grade =	Grade 80	
Avg. Support Spacing =	6.00 ft	$= L_{span}$
Number of Sidelaps =	2	

ASTM A1008	
F_y =	60 ksi
F_u =	62 ksi
F_{xx} =	60 ksi

GENERAL PARAMETERS

ASTM Steel Type =	A1008		Yield and Tensile Strength
E =	29,000	ksi	Modulus of Elasticity
D_d =	1.5	in	Depth of Wide Rib Deck Profile
t =	0.0295	in	Thickness of metal deck
w =	36.00	in	Deck panel width
Number of Spans =	3		Assume 3 span condition
L_{sheet} =	18.00	ft	Min length of deck sheet
n_p =	2		Number of interior supports
n_s =	6		Total number of sidelap fasteners
α =	2.00		exterior support fastener group distribution
d_{wr} =	6.00	in	Corrugation pitch for wide rib deck
s =	8.18	in	Developed width of corrugation per pitch d_{wr}
D =	1237	in	Warping Coefficient
ρ =	0.900		Assume 3 span condition

SDI Stiffness Equations

$$C = E * t * \frac{S_f}{w} * \frac{2}{2 * \alpha + n_p * \alpha + 2 * n_s * \frac{S_f}{S_s}} \qquad K_1 = \frac{C}{3 * L_{span}} * (L_{sheet} * 12)$$

$$G' = \frac{E * t}{2.6 * \frac{s}{d} + \rho * \frac{D}{L_{sheet} * 12} + C * L_{sheet} * 12}$$

DIAPHRAGM STIFFNESS RESULTS

Calculated Per SDI Criteria:

S_f =	0.0067	in/kip	C =	0.0253	1/in
S_s =	0.0175	in/kip	K_1 =	0.3031	1/ft
			G' =	60.44	kip/in

Final Design: SDI Criteria

G' =	**60.44**	**kip/in**
F =	16.54	10^6 in/lbs
Diaphragm =	Semi-Flexible	

Project:	0	Engineer:	0
Number:	0	Date:	1/0/1900

INPUT - DIAPHRAGM DEFLECTION

Diaphragm Span L_d =	322.00	ft
Diaphragm Depth B_d =	256.17	ft
Front Steel Chord Area A_{cf} =	1.94	in^2
Rear Steel Chord Area A_c =	1.94	in^2
Wind Max Deck Shear (Service) =	213	plf
Wind Max Diaphragm Moment (Service) =	3,968	kip*ft
Seismic Max Deck Shear (Service) =	205	plf
Seismic Max Diaphragm Moment (Service) =	3,962	kip*ft
Diaphragm Support Deflection =	0.125	in
Roof Height at Maximum Deflection H =	17.75	ft

Wind Factors	
Deflection =	0.70

Seismic Factors	
Cd =	2.25
I =	1.25
ρ =	1.00
ASD =	0.70

Estimated Moment of Inertia		
I_{chord} =	9.17E+06	in^4
I_{deck} =	7.14E+07	in^4

DIAPHRAGM DEFLECTION - WIND

Shear Deflection (Service)

V_{wind} =	149	plf	Deck Shear
G' =	60.44	kip/in	Shear Modulus
Δ_s =	0.1986	in	Shear Deflection

Bending Deflection (Service)

$q_{avg.w}$ =	214	plf	Based on Moment
Δ_b =	0.0222	in	Bending Deflection

Total Deflection (Service)

$\delta_{t.wind}$ = 0.35 in

DIAPHRAGM DEFLECTION - SEISMIC

Shear Deflection (Ultimate)

V_{eq} =	234	plf	Deck Shear
G' =	60.44	kip/in	Shear Modulus
Δ_s =	0.3120	in	Shear Deflection

Bending Deflection (Ultimate)

$q_{avg.eq}$ =	349	plf	Based on Moment
Δ_b =	0.0362	in	Bending Deflection

Total Deflection (with Seismic Amplification)

$\delta_{t.eq}*C_d$ = 1.06 in

ALLOWABLE STORY DRIFT

δ =	2.13	in	0.010*H for masonry walls

MINIMUM BUILDING SEPARATION

δ_m =	1.06	in	Min Property Line Offset
$\Delta_{t.bldg1}$ =	1.06	in	
$\Delta_{t.bldg2}$ =	0.92	in	
δ_{mt} =	1.41	in	Min Expansion Joint Width

NEW DIAPHRAGM DEFLECTION
RIGHT SIDE OF EXPANSION JOINT

NEW TOTAL DEFLECTION
(ONLY INCREASES 0.08")

EQUATIONS

$$I_{chord} = A_C * \left(\frac{B_d * 12}{2}\right)^2$$

$$I_{deck} = \frac{1}{12} * t * (B_d * 12)^3$$

$$\Delta_s = \frac{Vmax/_{1000} * 0.5 * L_d}{2 * G'}$$

$$\Delta_b = \frac{5 * {}^{q_{avg}}/_{12} * (L_d * 12)^4}{384 * E * I * 1000}$$

$$\delta_{mt} = \sqrt{\Delta_{t.bldg1}{}^2 + \Delta_{t.bldg2}{}^2}$$

1. All calculations are based on the AISI S100 and the SDI 4th Edition Diaphragm Design Manual (DDM04).
2. All sidelap fasteners shall be self drilling, self tapping screws manufactured by Buildex, Elco, Hilti, or Simpson.

WM 6958 Cameron, NC OPD and Existing Building
2431906958

Date: 1/0/1900
Engineer: 0

Type II Embed Plate Capacity (ACI 530-13)

Plate Thickness =>	t_{pl} =	0.375	in
Stud Diameter =>	dia =	0.5	in
Stud Length =>	L =	5	in
Effective Length = >	L_b =	4.5	in
Stud Spacing or Edge Distance =>	L_{be} =	7	in
Masonry Strength =>	f'm =	2000	psi
Stud Yield Stress =>	f_y =	50	ksi

Shear Capacity (ACI 530 Section 2.1.4.3.2) -

$$B_{vc} = 350\left[f_m * \pi\left(\frac{dia}{2}\right)^2\right]^{0.25}$$

(Equation 2-7) B_{vc} = 1558 lbs (Masonry Crushing)

$$B_{vs} = 0.36 * \pi * \left(\frac{dia}{2}\right)^2 * f_y * 1000$$

(Equation 2-9) B_{vs} = 3534 lbs (Steel Yeilding)

$$B_{vb} = 1.25 * (\pi * l._{be} \wedge 2 * 0.5) * \sqrt{f'm}$$

(Equation 2-6) B_{vb} = 4303 lbs (Masonry Breakout)

$$B_{vpry} = 2.5 * (\pi * l._{b} \wedge 2) * \sqrt{f'm} * Rf$$

(Equation 2-8) B_{vpry} = 6259 lbs (Anchor Pryout)

Tension Capacity (ACI 530 Section 2.1.4.3.1.1) -

$$B_{ab} = 1.25 * \pi * L_b^2 * f'm^{0.5} * Rf$$

(Equation 2-1) B_{ab} = 3130 lbs (Masonry Breakout)

$$B_{as} = 0.60 * \pi * \left(\frac{dia}{2}\right)^2 * f_y * 1000$$

(Equation 2-2) B_{as} = 5890 lbs (Steel Yeilding)

Tension Reduction Factor - (Based on Geometry) Rf = 0.88

Single Stud Shear Capacity Bv =	**1558**	**lbs**
Single Stud Tension Capacity Ba =	**3130**	**lbs**

Combined Axial Tensions & Shear (ACI 530 Section 2.1.4.3.3) -

$$\frac{b_v}{B_v} + \frac{b_a}{B_a} \leq 1.0$$

(Equation 2-10)

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Roof anchor to CMU wall	Date:	1/7/2025
Fastening point:			

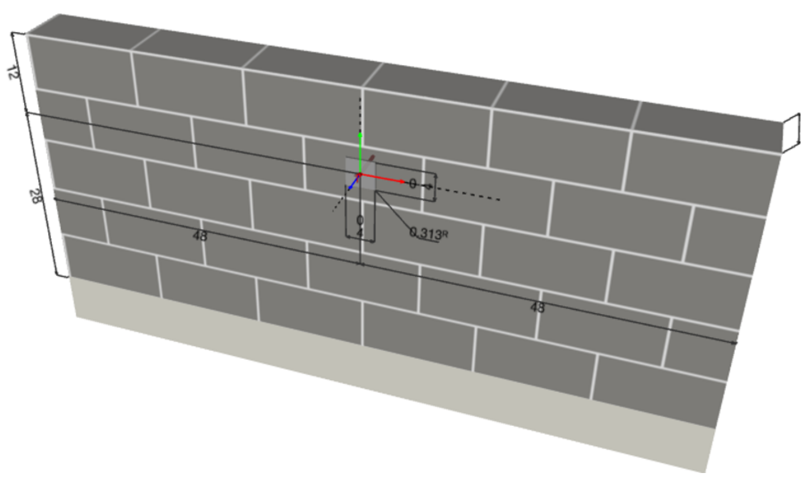
Specifier's comments:

1 Input data

Anchor type and diameter:	**HY 270 + threaded rod A36 1/2**
Item number:	2198022 HAS-V-36 1/2"x6-1/2" (element) / 2194247 HIT-HY 270 (adhesive)
Specification text:	Hilti ∅ 1/2 HY 270 + threaded rod A36 with 4.5 in nominal embedment depth per Technical data , Hammer drilled installation per MPII
Effective embedment depth:	h_{ef} = 4.500 in.
Material:	ASTM A 36
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ASD Masonry
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.313 in.
Anchor plate[R] :	l_x x l_y x t = 4.000 in. x 4.000 in. x 0.313 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.;
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
	Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	no

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.]



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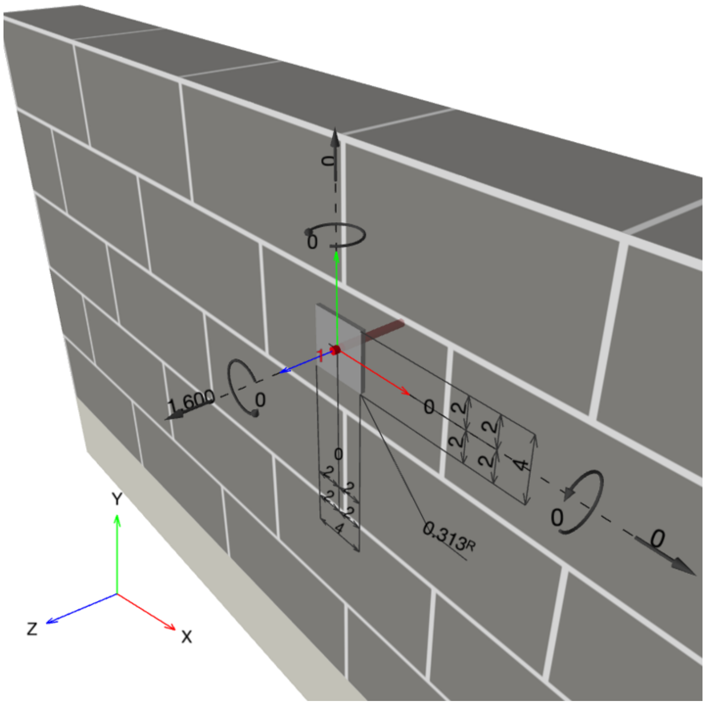
Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Roof anchor to CMU wall	Date:	1/7/2025
Fastening point:			

Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	WL (Anchors at 48" OC)	N = 1,600; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	90

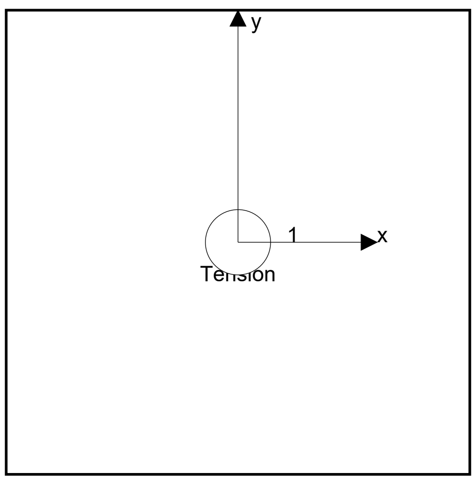
2 Load case/Resulting anchor forces

Load case: Service loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,600	0	0	0

max. compressive strain: - [‰]
max. compressive stress: - [psi]
Resulting tension force in (x/y)=(0.000/0.000): 1,600 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]



Anchor forces are calculated based on the assumption of a rigid anchor plate.

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Roof anchor to CMU wall	Date:	1/7/2025
Fastening point:			

3 Tension load (Most utilized anchor 1)

	Load P_s [lb]	Capacity P_t [lb]	Utilization $\beta_P = P_s/P_t$ [%]	Status
Steel strength	1,600	3,755	43	OK
Bond strength	1,600	1,791	90	OK

3.1 Steel strength

$P_{t,s}$ = Value refer to Hilti Technical Data

$P_{t,s} \geq P_s$

Results

$P_{t,s}$ [lb]	P_s [lb]
3,755	1,600

3.2 Bond strength

$P_{t,b,Base}$ = Value refer to Hilti Technical Data

$P_{t,b} = P_{t,b,Base} \cdot f_{red,E} \cdot f_{red,s} \cdot f_{red,Temp} \cdot f_{red,Bedjoint}$

$P_{t,b} \geq P_s$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	20.000	4.000	18.000	68

Results

$P_{t,b}$ [lb]	$P_{t,b,Base}$ [lb]	P_s [lb]	$f_{red,E}$	$f_{red,S}$	$f_{red,Temp}$	$f_{red,Bedjoint}$
1,791	2,035	1,600	0.880	1.000	1.000	1.000

4 Shear load (Most utilized anchor 1)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Overall strength	N/A	N/A	N/A	N/A

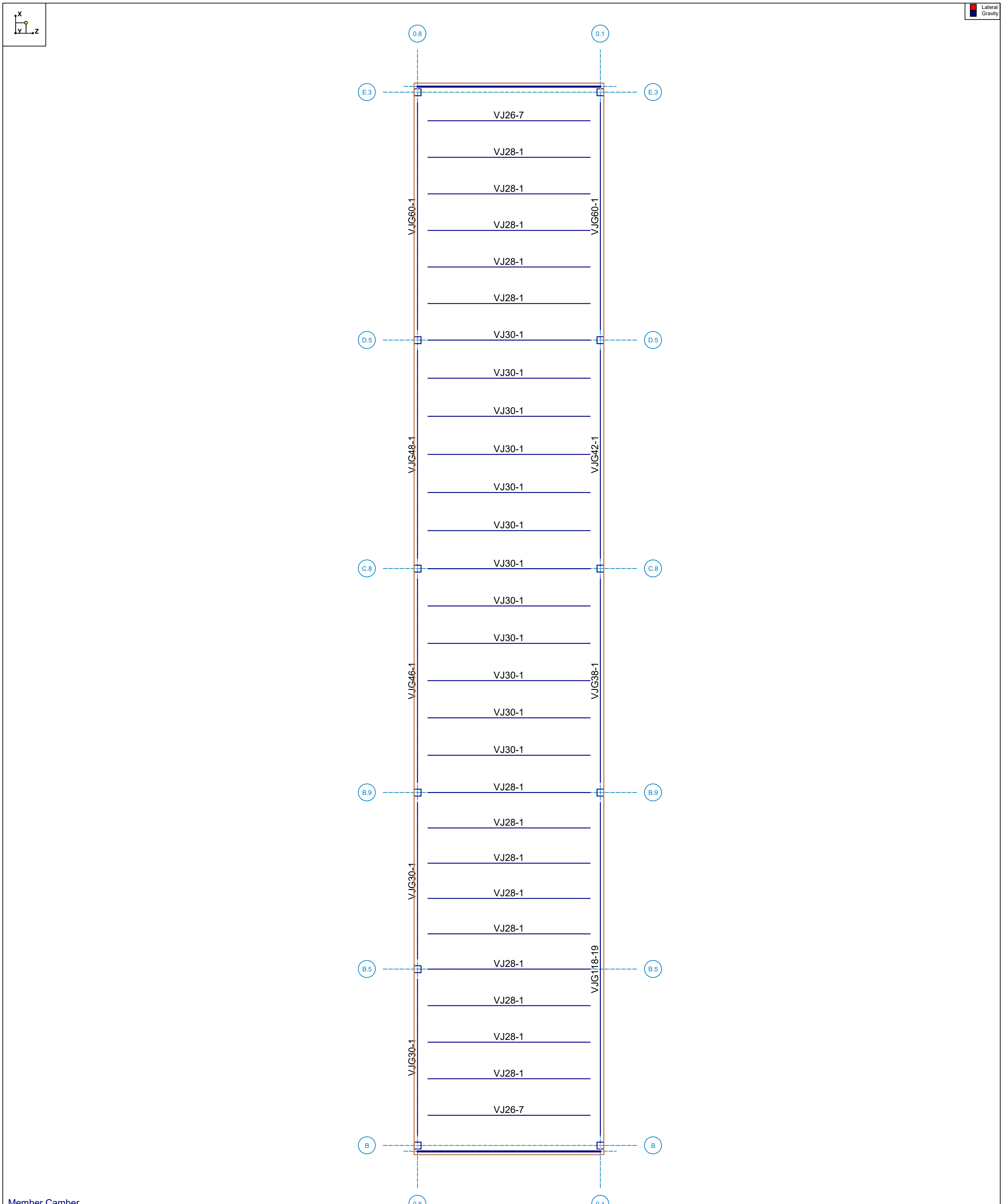
5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry needs to be built in a regular way in accordance with state-of the art guidelines!

Fastening meets the design criteria!

JOHNSTON
BURKHOLDER
ASSOCIATES
consulting structural engineers

GRAVITY DESIGN



Member Camber
Loads: Envelope Results

JBA	Roof	
DTR	6958 - Cameron, NC OPD	Jan 3, 2025 at 10:27 AM
		6958 - Cameron, NC OPD.rfl

OPJ1 LOAD DIAGRAM (D+S)

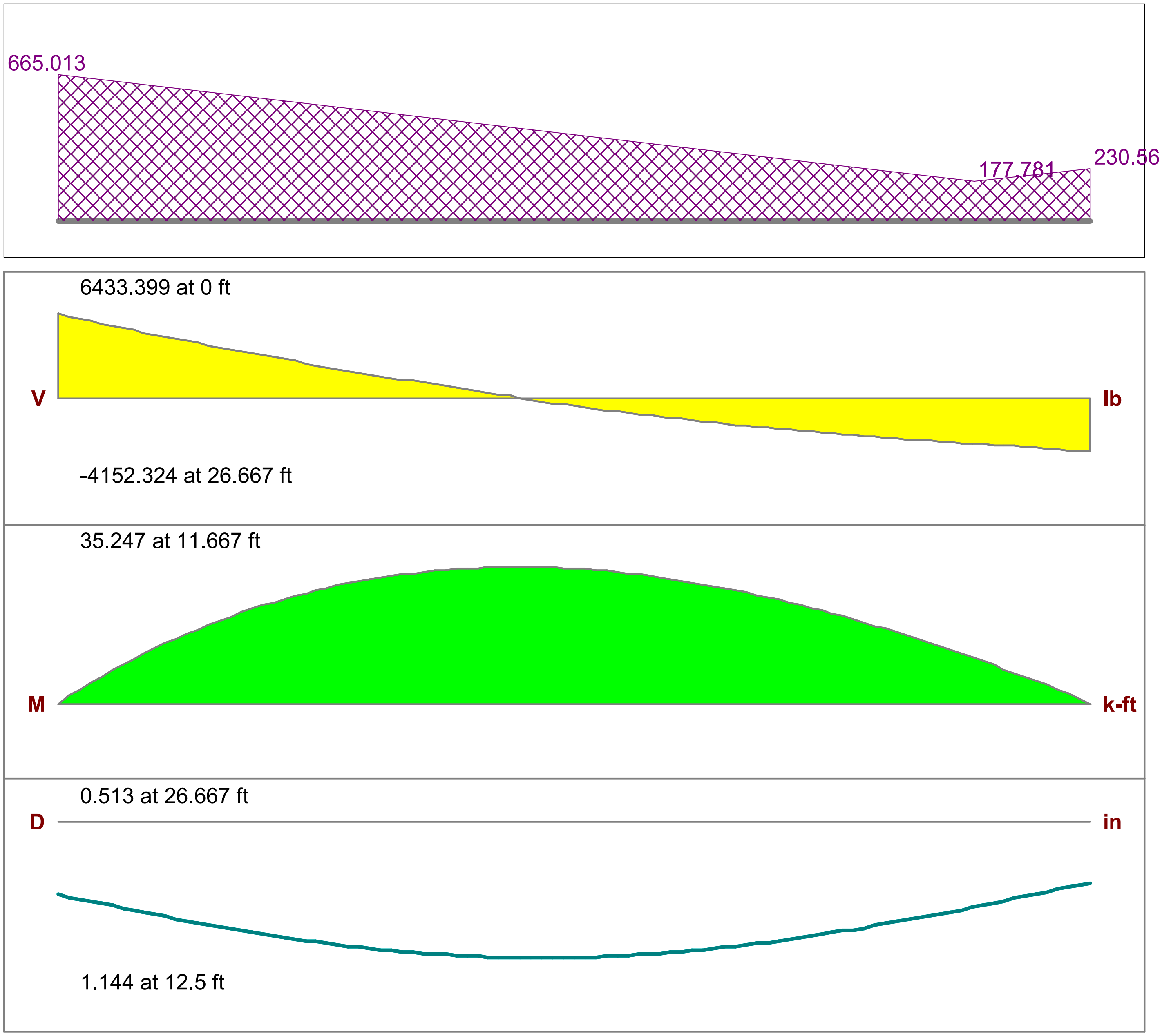
Beam:	**J33**	Shape Group:	**Virtual Joist**	Code:	**AISC 15th (360-16): ASD**
Floor:	**Roof**	Span:	**Single**		
Size:	**VJ30-1**	Fixity:	**Pinned-Pinned**		
Material:	**Virtual Joist (Fy = 50ksi)**	Bending:	**Strong Axis**		
Function:	**Gravity**				

Geometry: Length = 26.667ft

Points (Start to End): N41 to N42 (-0,107.556,0) to (26.667,107.556,0) Angle: 0 degrees

Diagrams for Load Combination 2 : DL+SL

Load Diagram (Live Load Reduction not shown): Distributed Loads (lb/ft), Point Loads (lb)



OPJG1 LOAD DIAGRAM (D+RLL)

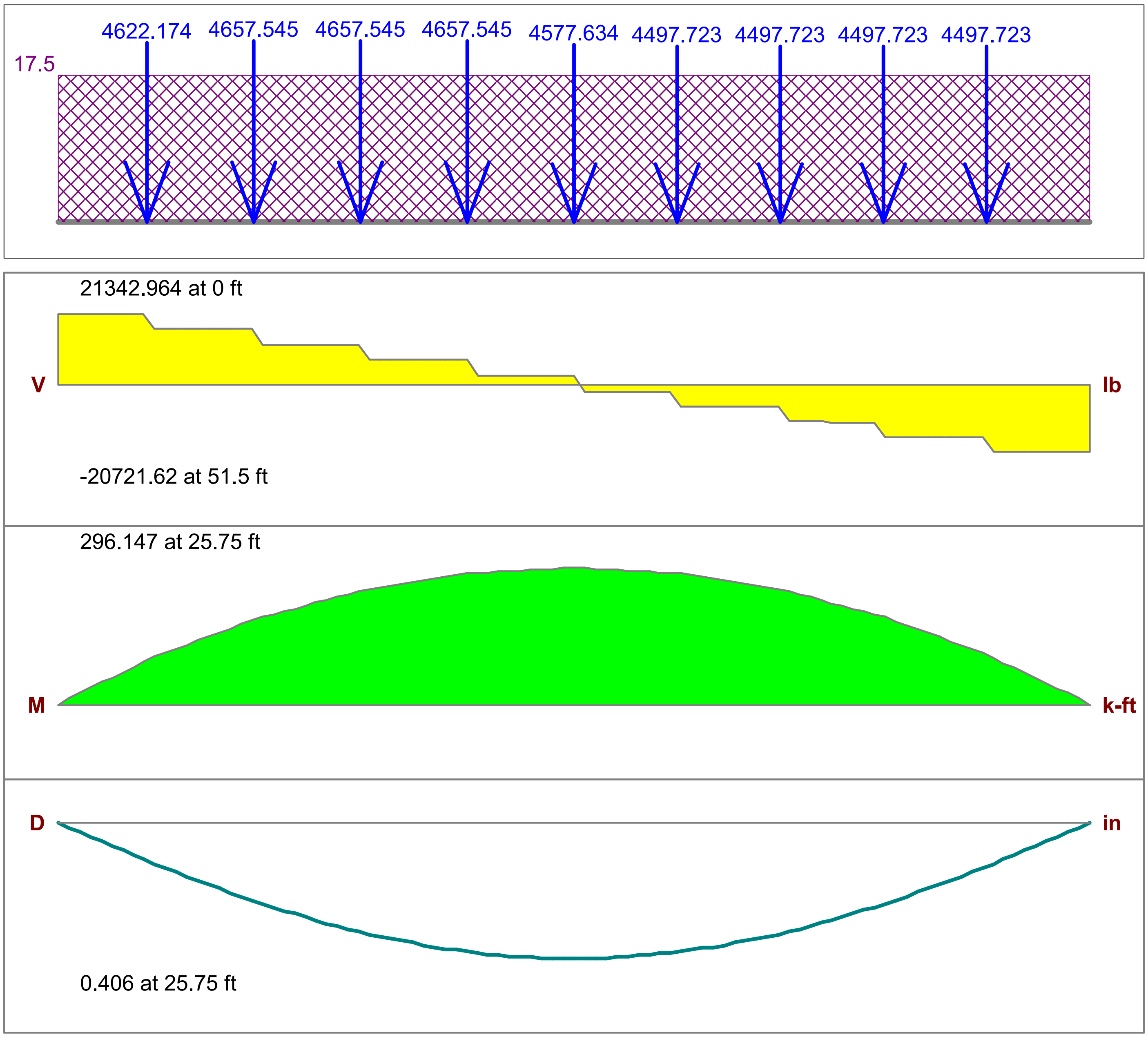
Beam:	**JG2**	Shape Group:	**Virtual Joist Girder**	Code:	**AISC 15th (360-16): ASD**
Floor:	**Roof**	Span:	**Single**		
Size:	**VJG118-19**	Fixity:	**Pinned-Pinned**		
Material:	**Virtual Joist (Fy = 50ksi)**	Bending:	**Strong Axis**		
Function:	**Gravity**				

Geometry: Length = 51.5ft

Points (Start to End): N9 to N6 (26.667,1.167,0) to (26.667,52.667,0)   Angle: 90 degrees

Diagrams for Load Combination 1 : DL+RLL

Load Diagram (Live Load Reduction not shown): Distributed Loads (lb/ft), Point Loads (lb)

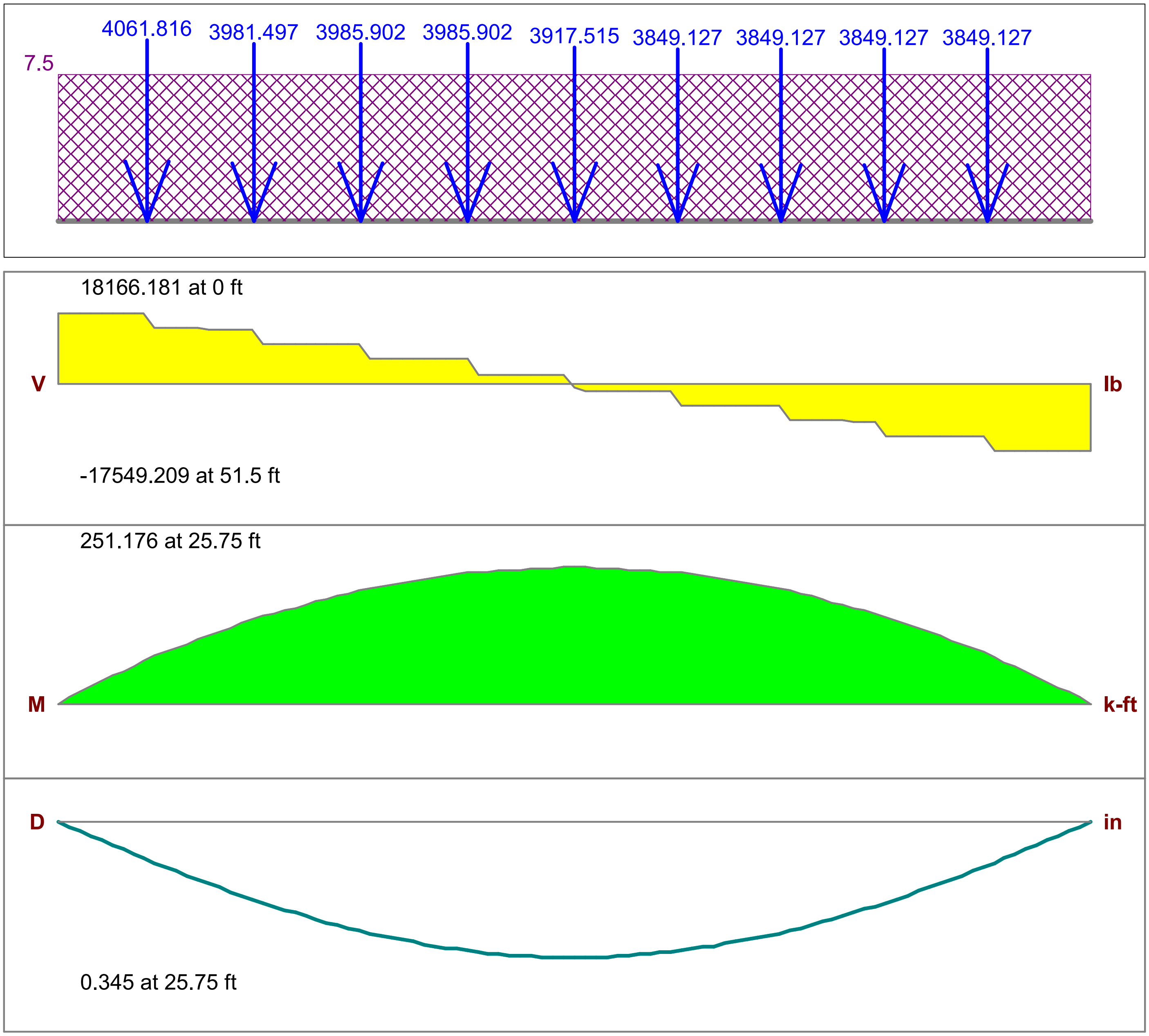


OPJG1 LOAD DIAGRAM (D+S)

Beam:	**JG2**	Shape Group:	**Virtual Joist Girder**	Code:	**AISC 15th (360-16): ASD**
Floor:	**Roof**	Span:	**Single**		
Size:	**VJG118-19**	Fixity:	**Pinned-Pinned**		
Material:	**Virtual Joist (Fy = 50ksi)**	Bending:	**Strong Axis**		
Function:	**Gravity**				

Geometry: Length = 51.5ft
Points (Start to End): N9 to N6 (26.667,1.167,0) to (26.667,52.667,0)   Angle: 90 degrees

Diagrams for Load Combination 2 : DL+SL
Load Diagram (Live Load Reduction not shown): Distributed Loads (lb/ft), Point Loads (lb)



Combinations

	Label	Sol...	Cat...	Fa...	Cat...	Fa...	Cat...	Fa...	Cat...	Fa...	Cat...	Fact...	Cat...	Fact...	Cat...	Fact...	Cat...	Fact...	Cat...	Fact...	Cat...	Fact...
1	DL+RLL	Yes	DL	1			RLL	1														
2	DL+SL	Yes	DL	1			SL	1														
3	MWFRS - 0....	Yes	DL	0.6					OL1	0.6												
4	C&C - 0.6DL...		DL	0.4					OL2	0.6												

Hot Rolled Steel Column Code Checks

	Stack	Lift	Shape	Code C...	Elev[ft]	LC	Shear ...	Elev[ft]	Dir	LC	Pnc/om...	Pnt/om ...	Mnyy/o...	Mnzz/o...	Cb	AISC...
1	(B.9-0.1)	1	HSS6X6X4	0.466	16.667	1	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1a*
2	(C.8-0.1)	1	HSS6X6X4	0.366	16.667	1	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1a*
3	(D.5-0.1)	1	HSS6X6X4	0.386	16.667	1	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1a*
4	(B-0.1)	1	HSS6X6X4	0.267	16.667	1	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1a*
5	(E.3-0.1)	1	HSS6X6X4	0.184	16.667	1	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1b*
6	(B.5-0.8)	1	HSS6X6X4	0.374	16.667	2	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1a*
7	(B.9-0.8)	1	HSS6X6X4	0.423	16.667	2	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1a*
8	(C.8-0.8)	1	HSS6X6X4	0.478	16.667	2	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1a*
9	(D.5-0.8)	1	HSS6X6X4	0.503	16.667	2	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1a*
10	(B-0.8)	1	HSS6X6X4	0.16	16.667	2	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1b*
11	(E.3-0.8)	1	HSS6X6X4	0.236	16.667	2	0	16.667		3	81216....	144335...	25.709	25.709	1	H1-1a*

MAX PERIMETER

Column Forces/Moments, Dead & Other Categories : Axial Force

	Column ...	Lift ...	Floor La...	Coordin...	Max Base Reactio...	Max Base Reaction LC	DLPre[...	DL[lb]	OL1[lb]	OL2[lb]	OL3[lb]	OL4[lb]
1	(B.9-0.1)	1	Roof	26.667,5...	37865.826	1	345.185	9084.667	-15964...	0	0	0
2	(C.8-0.1)	1	Roof	26.667,8...	29743.045	1	345.185	7192.767	-12508...	0	0	0
3	(D.5-0.1)	1	Roof	26.667,1...	31343.994	1	345.185	7565.508	-13189...	0	0	0
4	(B-0.1)	1	Roof	26.667,1...	21696.144	1	345.882	5326.639	-9098....	0	0	0
5	(E.3-0.1)	1	Roof	26.667,1...	14964.48	1	345.882	3758.261	-6233....	0	0	0
6	(B.5-0.8)	1	Roof	0,26.917	30393.215	2	345.185	5702.684	-9786....	0	0	0
7	(B.9-0.8)	1	Roof	0,52.667	34386.776	2	345.185	6405.918	-11070...	0	0	0
8	(C.8-0.8)	1	Roof	0,85.333	38806.309	2	345.185	7192.767	-12508...	0	0	0
9	(D.5-0.8)	1	Roof	0,118.667	40868.947	2	345.185	7565.508	-13189...	0	0	0
10	(B-0.8)	1	Roof	0,1.167	12981.045	2	345.882	2647.89	-4205....	0	0	0
11	(E.3-0.8)	1	Roof	0,154.833	19197.983	2	345.882	3758.261	-6233....	0	0	0

COL AT EXISTING FTG

Column Forces/Moments, Dead & Other Categories : Shear z-

	Column ...	Lift ...	Floor La...	Coordin...	Max Base Reactio...	Max Base Reaction LC	DL					
1	(B.9-0.1)	1	Roof	26.667,5...	0	1						
2	(C.8-0.1)	1	Roof	26.667,8...	0	1						
3	(D.5-0.1)	1	Roof	26.667,1...	0	1						
4	(B-0.1)	1	Roof	26.667,1...	0	1						
5	(E.3-0.1)	1	Roof	26.667,1...	0	1						
6	(B.5-0.8)	1	Roof	0,26.917	0	1						
7	(B.9-0.8)	1	Roof	0,52.667	0	1						
8	(C.8-0.8)	1	Roof	0,85.333	0	1						
9	(D.5-0.8)	1	Roof	0,118.667	0	1	0	0	0	0	0	0
10	(B-0.8)	1	Roof	0,1.167	0	1	0	0	0	0	0	0
11	(E.3-0.8)	1	Roof	0,154.833	0	1	0	0	0	0	0	0

COL FOOTING AT EXISTING WALL (GRID 0.8)
MAX COL REACTION = 40.9K
ALLOWABLE SOIL BEARING = 2500 PSF

LOAD ON EXISTING FTG
EXISTING WALL = 58 PSF*(25.33+1.33') = 1550 PLF
EXISTING ROOF LOAD (D+S) = 1420 PLF
TOTAL = 2970 PLF

EXISTING WALL FOOTING = 2.0' WIDE
ASSUME NEW FTG = 5.5' WIDE ---- 40.9K+(5.5')*(3.0)KLF = 57.4K
57.4K/2.5KSF = 23.0 SQ FT
TOTAL FOOTING WITH (OLD+NEW) = 23.0/5.5' = 4.18'
ADDED FOOTING WIDTH REQUIRED = 4.18-2.00 = 2.18'

Column Forces/Moments, Dead & Other Categories : Shear y-y

	Column ...	Lift ...	Floor La...	Coordin...	Max Base Reactio...	Max Base Reaction LC	DLPre[...	DL[lb]	OL1[lb]	OL2[lb]	OL3[lb]	OL4[lb]
1	(B.9-0.1)	1	Roof	26.667,5...	0	1	0	0	0	0	0	0
2	(C.8-0.1)	1	Roof	26.667,8...	0	1	0	0	0	0	0	0
3	(D.5-0.1)	1	Roof	26.667,1...	0	1	0	0	0	0	0	0
4	(B-0.1)	1	Roof	26.667,1...	0	1	0	0	0	0	0	0
5	(E.3-0.1)	1	Roof	26.667,1...	0	1	0	0	0	0	0	0
6	(B.5-0.8)	1	Roof	0,26.917	0	1	0	0	0	0	0	0

Column Forces/Moments, Dead & Other Categories : Moment y-y (Bot) (Continued)

	Column ...	Lift ...	Floor La...	Coordin...	Max Base Reactio...	Max Base Reaction LC	DLPre[...	DL[k-ft]	OL1[k-ft]	OL2[k-ft]	OL3[k-ft]	OL4[k-ft]
6	(B.5-0.8)	1	Roof	0,26.917	0	1	0	0	0	0	0	0
7	(B.9-0.8)	1	Roof	0,52.667	0	1	0	0	0	0	0	0
8	(C.8-0.8)	1	Roof	0,85.333	0	1	0	0	0	0	0	0
9	(D.5-0.8)	1	Roof	0,118.667	0	1	0	0	0	0	0	0
10	(B-0.8)	1	Roof	0,1.167	0	1	0	0	0	0	0	0
11	(E.3-0.8)	1	Roof	0,154.833	0	1	0	0	0	0	0	0

Column Forces/Moments, Roof Load : Axial Force

	Column Stack	Lift No.	Floor Label	Coordinat...	Reducible ...	RLL Redu...	NonReduc...	RLL[lb] (u...	SL[lb]	SLN[lb]	RL[lb]
1	(B.9-0.1)	1	Roof	26.667,52...	0	1	28781.158	0	23064.127	0	0
2	(C.8-0.1)	1	Roof	26.667,85...	0	1	22550.278	0	18064.228	0	0
3	(D.5-0.1)	1	Roof	26.667,11...	0	1	23778.485	0	19060.153	0	0
4	(B-0.1)	1	Roof	26.667,1.1...	0	1	16369.504	0	13188.551	0	0
5	(E.3-0.1)	1	Roof	26.667,15...	0	1	11206.219	0	9054.427	0	0
6	(B.5-0.8)	1	Roof	0,26.917	0	1	13478.297	0	24690.531	0	0
7	(B.9-0.8)	1	Roof	0,52.667	0	1	15246.763	0	27980.858	0	0
8	(C.8-0.8)	1	Roof	0,85.333	0	1	17226.211	0	31613.541	0	0
9	(D.5-0.8)	1	Roof	0,118.667	0	1	18164.328	0	33303.438	0	0
10	(B-0.8)	1	Roof	0,1.167	0	1	5770.855	0	10333.155	0	0
11	(E.3-0.8)	1	Roof	0,154.833	0	1	8565.491	0	15439.722	0	0

Column Forces/Moments, Roof Load : Shear z-z

	Column Stack	Lift No.	Floor Label	Coordinat...	Reducible ...	RLL Redu...	NonReduc...	RLL[lb] (u...	SL[lb]	SLN[lb]	RL[lb]
1	(B.9-0.1)	1	Roof	26.667,52...	0	1	0	0	0	0	0
2	(C.8-0.1)	1	Roof	26.667,85...	0	1	0	0	0	0	0
3	(D.5-0.1)	1	Roof	26.667,11...	0	1	0	0	0	0	0
4	(B-0.1)	1	Roof	26.667,1.1...	0	1	0	0	0	0	0
5	(E.3-0.1)	1	Roof	26.667,15...	0	1	0	0	0	0	0
6	(B.5-0.8)	1	Roof	0,26.917	0	1	0	0	0	0	0
7	(B.9-0.8)	1	Roof	0,52.667	0	1	0	0	0	0	0
8	(C.8-0.8)	1	Roof	0,85.333	0	1	0	0	0	0	0
9	(D.5-0.8)	1	Roof	0,118.667	0	1	0	0	0	0	0
10	(B-0.8)	1	Roof	0,1.167	0	1	0	0	0	0	0
11	(E.3-0.8)	1	Roof	0,154.833	0	1	0	0	0	0	0

Column Forces/Moments, Roof Load : Shear y-y

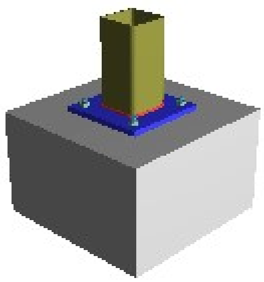
	Column Stack	Lift No.	Floor Label	Coordinat...	Reducible ...	RLL Redu...	NonReduc...	RLL[lb] (u...	SL[lb]	SLN[lb]	RL[lb]
1	(B.9-0.1)	1	Roof	26.667,52...	0	1	0	0	0	0	0
2	(C.8-0.1)	1	Roof	26.667,85...	0	1	0	0	0	0	0
3	(D.5-0.1)	1	Roof	26.667,11...	0	1	0	0	0	0	0
4	(B-0.1)	1	Roof	26.667,1.1...	0	1	0	0	0	0	0
5	(E.3-0.1)	1	Roof	26.667,15...	0	1	0	0	0	0	0
6	(B.5-0.8)	1	Roof	0,26.917	0	1	0	0	0	0	0
7	(B.9-0.8)	1	Roof	0,52.667	0	1	0	0	0	0	0
8	(C.8-0.8)	1	Roof	0,85.333	0	1	0	0	0	0	0
9	(D.5-0.8)	1	Roof	0,118.667	0	1	0	0	0	0	0
10	(B-0.8)	1	Roof	0,1.167	0	1	0	0	0	0	0
11	(E.3-0.8)	1	Roof	0,154.833	0	1	0	0	0	0	0

Column Forces/Moments, Roof Load : Moment z-z (Top)

	Column Stack	Lift No.	Floor Label	Coordinat...	Reducible ...	RLL Redu...	NonReduc...	RLL[k-ft] (...	SL[k-ft]	SLN[k-ft]	RL[k-ft]
1	(B.9-0.1)	1	Roof	26.667,52...	0	1	0	0	0	0	0
2	(C.8-0.1)	1	Roof	26.667,85...	0	1	0	0	0	0	0
3	(D.5-0.1)	1	Roof	26.667,11...	0	1	0	0	0	0	0
4	(B-0.1)	1	Roof	26.667,1.1...	0	1	0	0	0	0	0

(D.5-0.8)_L1_F1 I: Summary Report

Single Column Base Plate Connection



Material Properties:

Column	HSS6X6X4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Base Plate	P0.75x12.00x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

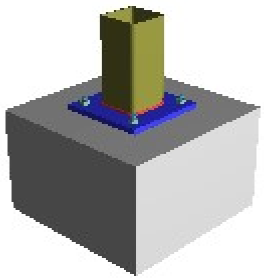
Axial	40868.95 lbs	*Axial load on the column*
Strong Axis Shear	0.00 lbs	*Shear load on the column that causes strong axis bending*
Weak Axis Shear	0.00 lbs	*Shear load on the column that causes weak axis bending*
Strong Axis Moment	0.00 kips-ft	*Column moment about the strong axis*
Weak Axis Moment	0.00 kips-ft	*Column moment about the weak axis*

Governing LC: FL - 2 - LC 2: DL+SL

Connection	Required	Max Unity Check	Result
Column/Base Plate connection	Plate Flexural Yielding(Compression)	**0.46**	**PASS**

(D.5-0.8)_L1_F1 I: Base Plate Report

Single Column Base Plate Connection



Material Properties:

Column	HSS6X6X4	A500 Gr.B Rect	F_y = 46.00 ksi	F_u = 58.00 ksi
Base Plate	P0.75x12.00x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi

Input Data:

Axial	40868.95 lbs	*Axial load on the column*
Strong Axis Shear	0.00 lbs	*Shear load on the column that causes strong axis bending*
Weak Axis Shear	0.00 lbs	*Shear load on the column that causes weak axis bending*
Strong Axis Moment	0.00 kips-ft	*Column moment about the strong axis*
Weak Axis Moment	0.00 kips-ft	*Column moment about the weak axis*

Governing LC: FL - 2 - LC 2: DL+SL

Note: Unless specified, all code references are from AISC 360-10

Limit State	Required	Available	Unity Check	Result
Geometry Restrictions				**PASS**
Concrete Bearing	0.28 ksi	2.21 ksi	**0.13**	**PASS**
Plate Flexural Yielding(Compression)	0.12 kips-ft/in	0.25 kips-ft/in	**0.46**	**PASS**
Anchor Bolt Shear	0.00 lbs	11530.63 lbs	**0.00**	**PASS**
Anchor Bolt Bearing on Base Plate	0.00 lbs	11530.63 lbs	**0.00**	**PASS**
Column Weld Limitations				**PASS**
Column Flange Weld Strength	0.00 lbs/ft	44544.00 lbs/ft	**0.00**	**PASS**
Column Web Weld Strength	0.00 lbs/ft	44544.00 lbs/ft	**0.00**	**PASS**

(D.5-0.8)_L1_F1 I: Connection Properties Report

Connection	
Connection Title	(D.5-0.8)_L1_F1 I
Connection Type	Single Column Base Plate Connection
Anchorage	
Anchorage Type	Cast-in-place
Connection Category	
Bolt Layout	Four
Plate Washers	No
Loading (ASD)	
Custom?	No
Axial	40868.947 lbs
Strong Axis Shear	0.000 lbs
Weak Axis Shear	0.000 lbs
Strong Axis Moment	0.000 kips-ft
Weak Axis Moment	0.000 kips-ft
Components	
Column Section	HSS6X6X4
Material	A500 Gr.B Rect
Base Plate	P0.75x12.00x12.00
Material	A36
Length	12.000 in
Width	12.000 in
Thickness	0.750 in
Static Friction Coefficient	0.550 Coeff
Hole Type	OVS
Concrete Support	C24.00x24.00x15.00
Length	24.000 in
Width	24.000 in
Thickness	15.000 in
Compressive Strength (f'c)	3.000 ksi
Concrete Weight	Normal Weight
Cracked Concrete	Yes
Edge Reinforcement	None or < no. 4 bar
Anchor Bolts	3/4" F1554 Gr.36-N
Material	F1554 Gr.36-N
Head Type	Hex Bolt
Torque Type	Untorqued Anchor
Diameter, in.	3/4"
Embedment depth	9.000 in
Bolt Spacing y	9.000 in
Bolt Spacing z	9.000 in
Column Weld	E70
Type	Fillet
Fillet Size	4.000 Sixteenths
Assembly	
Edge Distance y	1.500 in
Edge Distance z	1.500 in
Edge Distance +y	6.000 in
Edge Distance -y	6.000 in
Edge Distance +z	6.000 in
Edge Distance -z	6.000 in

General Footing

Project File: 6958 - Misc Calcs.ec6

DESCRIPTION: Perimeter column footing (Grid 0.1)

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019
Load Combinations Used : ASCE 7-16

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	:	Yes	
Use ftg wt for stability, moments & shears	:	Yes	
Add Pedestal Wt for Soil Pressure	:	No	
Use Pedestal wt for stability, mom & shear	:	No	

Soil Design Values

Allowable Soil Bearing	=	2.50	ksf
Soil Density	=	110.0	pcf
Increase Bearing By Footing Weight	=	Yes	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

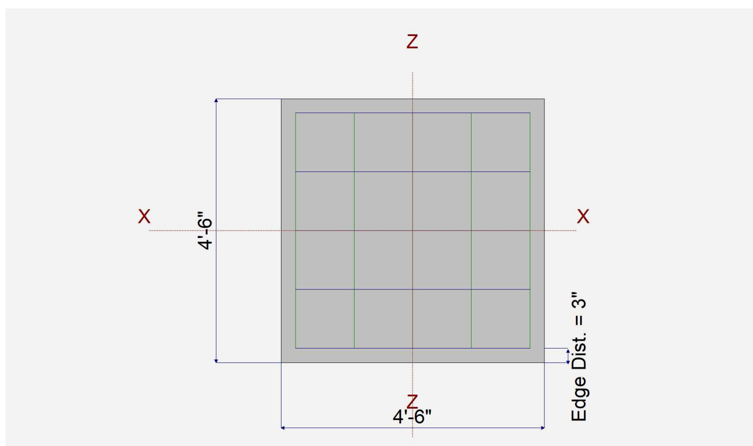
Footing base depth below soil surface	=	2.50	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	4.50	ft
Length parallel to Z-Z Axis	=	4.50	ft
Footing Thickness	=	15.0	in



Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height	=		in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in

Reinforcing

Bars parallel to X-X Axis

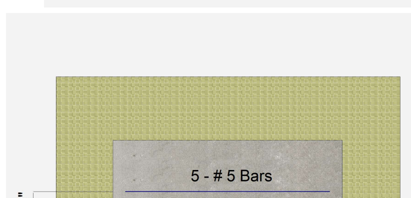
Number of Bars	=		5.0
Reinforcing Bar Size	=	#	5

Bars parallel to Z-Z Axis

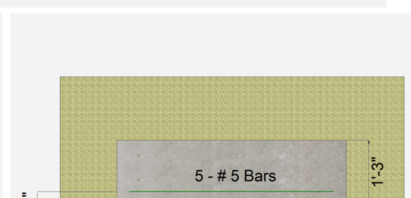
Number of Bars	=		5.0
Reinforcing Bar Size	=	#	5

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a



X-X Section Looking to +Z



Z-Z Section Looking to +X

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	10.0	29.0		24.0				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

DESCRIPTION: Perimeter column footing (Grid 0.1)

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8373	Soil Bearing	2.245 ksf	2.681 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	0.4038	Z Flexure (+X)	7.30 k-ft/ft	18.077 k-ft/ft	+1.20D+1.60Lr
PASS	0.4038	Z Flexure (-X)	7.30 k-ft/ft	18.077 k-ft/ft	+1.20D+1.60Lr
PASS	0.4038	X Flexure (+Z)	7.30 k-ft/ft	18.077 k-ft/ft	+1.20D+1.60Lr
PASS	0.4038	X Flexure (-Z)	7.30 k-ft/ft	18.077 k-ft/ft	+1.20D+1.60Lr
PASS	0.3071	1-way Shear (+X)	25.235 psi	82.158 psi	+1.20D+1.60Lr
PASS	0.3071	1-way Shear (-X)	25.235 psi	82.158 psi	+1.20D+1.60Lr
PASS	0.3071	1-way Shear (+Z)	25.235 psi	82.158 psi	+1.20D+1.60Lr
PASS	0.3071	1-way Shear (-Z)	25.235 psi	82.158 psi	+1.20D+1.60Lr
PASS	0.5872	2-way Punching	96.482 psi	164.317 psi	+1.20D+1.60Lr

Combined Footing

Project File: 6958 - Misc Calcs.ec6

LIC# : KW-06015958, Build:20.24.12.02 JOHNSTON - BURKHOLDER

DESCRIPTION: Column footing at existing (left reaction is existing wall)

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019
Load Combinations Used : ASCE 7-16

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 : Phi Values — Flexure :	0.90	
Shear :	0.750	

Analysis/Design Settings

Calculate footing weight as dead load ?	Yes
Calculate Pedestal weight as dead load ?	No
Min Steel % Bending Reinf (based on 'd')	
Min Allow % Temp Reinf (based on thick)	0.00180
Min. Overturning Safety Factor	1.0: 1
Min. Sliding Safety Factor	1.0: 1

Soil Information

Allowable Soil Bearing	2.50	ksf
Increase Bearing By Footing Weight	Yes	
Soil Passive Sliding Resistance	250.0	pcf
(Uses entry for "Footing base depth below soil surface" for force)		
Coefficient of Soil/Concrete Friction	0.30	

Soil Bearing Increase

Footing base depth below soil surface		ft
Increases based on footing Depth		
Allowable pressure increase per foot		ksf
when base of footing is below		ft
Increases based on footing Width . . .		
Allowable pressure increase per foot		ksf
when maximum length or width is greater tha		ft
Maximum Allowed Bearing Pressure	10.0	ksf
(A value of zero implies no limit)		
Adjusted Allowable Soil Bearing		ksf

(Allowable Soil Bearing adjusted for footing weight and depth & width increases as specified by user.)

Dimensions & Reinforcing

Distance Left of Column #1	=	1.0	ft
Between Columns	=	1.167	ft
Distance Right of Column #2	=	1.833	ft
Total Footing Length	=		ft
Footing Width	=	5.50	ft
Footing Thickness	=	12.0	in
Rebar Center to Concrete Edge @ Top	=	3.0	in
Rebar Center to Concrete Edge @ Bottom	=	3.0	in

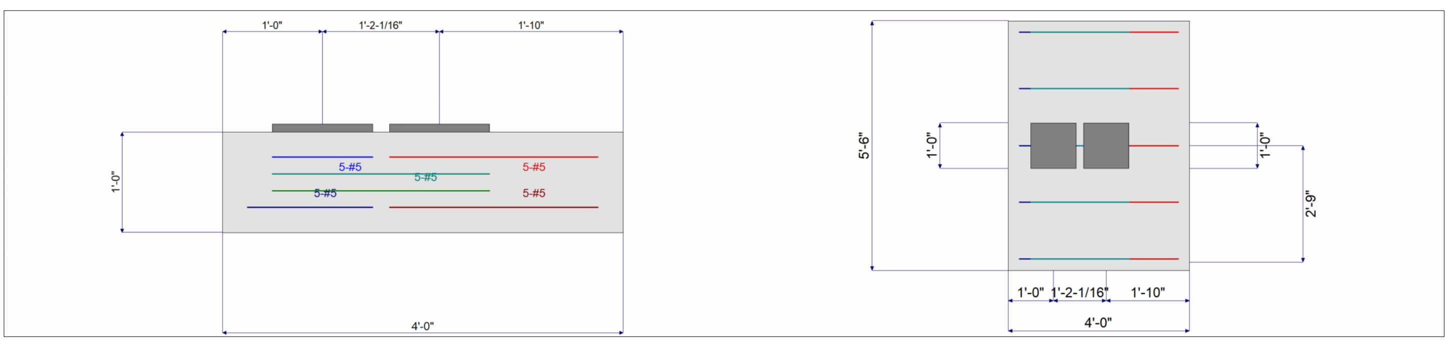
Pedestal dimensions...

		Col #1	Col #2	
Sq. Dim.	=	12.0	12.0	in
Height	=	1.0	1.0	in

	Count	Size #	As Provided	As Req'd
Bars left of Col #1				
Bottom Bars	5.0	5	1.550	1.426 in^2
Top Bars	5.0	5	1.550	0.0 in^2
Bars Btwn Cols				
Bottom Bars	5.0	5	1.550	1.426 in^2
Top Bars	5.0	5	1.550	0.0 in^2
Bars Right of Col #2				
Bottom Bars	5.0	5	1.550	1.426 in^2
Top Bars	5.0	5	1.550	0.0 in^2

Applied Loads

		D	Lr	L	S	W	E	H	
Applied @ Left Column									
Axial Load Downward	=	1.90	2.50		5.20				k
Moment (+CW)	=								k-ft
Shear (+X)	=								k
Applied @ Right Column									
Axial Load Downward	=	7.60	18.20		33.30				k
Moment (+CW)	=								k-ft
Shear (+X)	=								k
Overburden	=								



Combined Footing

Project File: 6958 - Misc Calcs.ec6

DESCRIPTION: Column footing at existing (left reaction is existing wall)

DESIGN SUMMARY

Design OK

	Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS	No OTM	Overturning	0.0 k-ft	0.0 k-ft	No OTM
PASS	No Sliding	Sliding	0.0 k	3.807 k	No Sliding
PASS	No Uplift	Uplift	0.0 k	0.0 k	No Uplift

	Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.887	Soil Bearing	2.345 ksf	2.645 ksf	+D+S
PASS	0.375	1-way Shear - Col #1	30.8 psi	82.2 psi	+1.20D+1.60S
PASS	0.375	1-way Shear - Col #2	30.8 psi	82.2 psi	+1.20D+1.60S
PASS	0.004	2-way Punching - Col #1	0.7 psi	164.3 psi	+1.20D+0.20S
PASS	0.005	2-way Punching - Col #2	0.7 psi	164.3 psi	+1.20D+0.20S
PASS	No Bending	Flexure - Left of Col #1 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS	0.036	Flexure - Left of Col #1 - Bottom	2.198 k-ft	60.848 k-ft	+1.20D+1.60S
PASS	No Bending	Flexure - Between Cols - Top	0.0 k-ft	0.0 k-ft	N/A
PASS	0.299	Flexure - Between Cols - Bottom	18.216 k-ft	60.848 k-ft	+1.20D+1.60S
PASS	No Bending	Flexure - Right of Col #2 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS	0.265	Flexure - Right of Col #2 - Bottom	16.095 k-ft	60.848 k-ft	+1.20D+1.60S

Overturning Stability

	Moments about Left Edge k-ft			Moments about Right Edge k-ft		
Load Combination...	**Overturning**	**Resisting**	**Ratio**	**Overturning**	**Resisting**	**Ratio**
D Only	0.00	0.00	999.000	0.00	0.00	999.000
+D+Lr	0.00	0.00	999.000	0.00	0.00	999.000
+D+S	0.00	0.00	999.000	0.00	0.00	999.000
+D+0.750Lr	0.00	0.00	999.000	0.00	0.00	999.000
+D+0.750S	0.00	0.00	999.000	0.00	0.00	999.000
+0.60D	0.00	0.00	999.000	0.00	0.00	999.000

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CMU WALLS

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Prototype	41

Project No.	Sheet No:	Project Name:	Made By:	Date:
2431906958		WM 6958 Cameron, NC OPD	DTR	12/30/24

CMU WALL DESIGN WORKSHEET

TMS 402-16

Version 2.0

Wall Lateral Forces

Wall Location	Wind		Seismic	
Main Wall	Wall WL, Service =	14.8 psf	OOP Wall(serv.) 0.7*EQ =	0.070
Parapet Only	Parapet WL, Service =	24.2 psf	Parapet(serv.) 0.7*EQ =	0.185
DL= 18 psf	RoofLL= 20 psf		SL= 24.5 psf	

Wall Properties

CMU Density= 135 pcf

Masonry Inspection =	Y		Steel Mod. Of Elasticity, E_s=	29,000	ksi
Masonry, f_m =	2,000	psi	Fm =	900	psi
Rebar yield strength, F_y=	60,000	psi	Fs =	32000	psi
Masonry Mod. Of Elasticity, E_m =	1,800	ksi	$n = E_s/E_m$ =	16.11	

General Wall Information												Seismic Loads		Roof Uniform Loads			Wall Properties								
Wall Location	Bearing Wall Y / N	Wind Girt Location Y / N	Top of Wall Elev. (ft)	Jst. Brg. Elev. (ft)	Roof Elev. h (ft)	Depth to T.O. Ftg h_f (ft)	Nominal Wall Thk. t (in)	Rebar Location c/e/comp	Face Shell Thick. t_f (in)	Depth to Rebar d (in)	Wall Weight Wt (psf)	Wall EQ_w (psf)	Parapet EQ_p (psf)	DL wd (plf)	RLL wl (plf)	SL ws (plf)	Steel Ratio p	Radius of Gyration r (in)	Eff. Wall Area A_w (in^2)	k_1	k_1*d (in)	j_1	k_2	k_2*d (in)	j_2
Side wall	Y	N	20.67	16.50	16.54	1.33	8	Center	1.25	3.813	54.05	3.78	10.00	260	290	1500	0.002404	2.658551	162.5888	0.24227	0.92379	0.91924	0.25226	0.96186	0.84693
Front wall at right	N	N	20.67	16.50	16.54	1.33	8	Center	1.25	3.813	54.05	3.78	10.00				0.002404	2.658551	162.5888	0.24227	0.92379	0.91924	0.25226	0.96186	0.84693
Front wall at left	N	N	20.67	17.08	17.12	1.33	8	Center	1.25	3.813	54.05	3.78	10.00				0.002404	2.658551	162.5888	0.24227	0.92379	0.91924	0.25226	0.96186	0.84693
Rear wall at right	N	N	20.67	16.50	16.54	1.33	8	Center	1.25	3.813	54.05	3.78	10.00				0.002404	2.658551	162.5888	0.24227	0.92379	0.91924	0.25226	0.96186	0.84693
Rear wall at left	N	N	20.67	17.08	17.12	1.33	8	Center	1.25	3.813	54.05	3.78	10.00				0.002404	2.658551	162.5888	0.24227	0.92379	0.91924	0.25226	0.96186	0.84693
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

CMU WALL DESIGN WORKSHEET

Wall Location	Bending Forces: Wind				Bending Forces: Seismic				Roof Loads + Self Wt.				Rectangular Section Stresses: CMU Comp. Str.		Rectangular Section Stresses: Steel Stress		T-Beam Section Stresses: CMU Comp. Str.		T-Beam Section Stresses: Steel Stress		Applied Axial Stresses: Roof Loads + Self Wt.			Allow. Axial Stress	Axial Check
	M_{span} lb-ft	$M_{parapet}$ lb-ft	M_{max} lb-ft	hm ft	M_{span} lb-ft	$M_{parapet}$ lb-ft	M_{max} lb-ft	hm ft	eccent. e in.	DL M_{roof} lb-ft	RLL M_{roof} lb-ft	SL M_{roof} lb-ft	Wind fb1 psi	EQ fb1eq psi	Wind fs1 psi	EQ fs1eq psi	Wind fb2 psi	EQ fb2eq psi	Wind fs2 psi	EQ fs2eq psi	DL fadl psi	RLL fall psi	SL fasl psi	Fa psi	
Side wall	2091	825	2091	13.49	541	341	541	13.54	2	173	193	1000	323	84	16266	4213	370	96	17655	4573	24	7	37	334	O.K.
Front wall at right	2091	825	2091	13.49	541	341	541	13.54	0	0	0	0	323	84	16266	4213	370	96	17655	4573	18	0	0	334	O.K.
Front wall at left	2309	609	2309	13	597	252	597	13	0	0	0	0	357	92	17970	4648	408	106	19504	5044	17	0	0	323	O.K.
Rear wall at right	2091	825	2091	13	541	341	541	14	0	0	0	0	323	84	16266	4213	370	96	17655	4573	18	0	0	334	O.K.
Rear wall at left	2309	609	2309	13	597	252	597	13	0	0	0	0	357	92	17970	4648	408	106	19504	5044	17	0	0	323	O.K.
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!

CMU WALL DESIGN WORKSHEET

Wall Location	Masonry Combined Stress Ratios Due to Axial & Bending						Steel Combined Stress Ratios Due to Axial Eccentric & Bending					
	DL+RLL	DL+SL	DL+WL	DL+EQ	DL+ 0.75WL+ .75(LorS)	DL+ 0.75EQ+ .75(LorS)	DL+RLL	DL+SL	DL+WL	DL+EQ	DL+ 0.75WL+ .75(LorS)	DL+ 0.75EQ+ .75(LorS)
Side wall	0.098	0.269	0.400	0.134	0.402	0.203	0.089	0.285	0.528	0.152	0.488	0.206
Front wall at right	0.020	0.020	0.379	0.113	0.289	0.090	0.000	0.000	0.508	0.132	0.381	0.099
Front wall at left	0.019	0.019	0.416	0.122	0.317	0.096	0.000	0.000	0.562	0.145	0.421	0.109
Rear wall at right	0.020	0.020	0.379	0.113	0.289	0.090	0.000	0.000	0.508	0.132	0.381	0.099
Rear wall at left	0.019	0.019	0.416	0.122	0.317	0.096	0.000	0.000	0.562	0.145	0.421	0.109
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Design Summary

Wall Location	Bar Spacing Sb (in)	Bar Size	Bar Area Ab (in^2)	Max CMU Stress Ratio	Max Steel Stress Ratio	Check	Does Parapet or Wall Control?
Side wall	48	6	0.44	0.40	0.53	O.K.	Wall
Front wall at right	48	6	0.44	0.38	0.51	O.K.	Wall
Front wall at left	48	6	0.44	0.42	0.56	O.K.	Wall
Rear wall at right	48	6	0.44	0.38	0.51	O.K.	Wall
Rear wall at left	48	6	0.44	0.42	0.56	O.K.	Wall
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!

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JAMB DESIGN

		CMU Wall Details								Low Roof Loads			Jamb Details							Jamb Design Values			Final Design Check		
Calc #	Wall Location	Girt	CMU Wall Thick.	Rebar Size	Bar Spacing	CMU Wall Weight	Top of Wall El.	Roof El.	Top of Ftg El below FF	Low Roof Trib Width	Low Roof Drift Only	Add'l Wall Weight	Jamb Type	Jamb Reinf Location	1st Opening Width	2nd Opening Width	Top of Opening El.	Lintel End Brg.	Pier or End Jamb Width	# of Cells Grouted	# of Tension Bars	Jamb Rebar Size	Grout, Reinf, and Stress Design Check	Max CMU Stress Ratio	Max Steel Stress Ratio
		Y/N	in	#	in	psf	(ft)	(ft)	(ft.)	(ft)	(plf)	(plf)			(ft)	(ft)	(ft)	(in)	(ft)	n_c	n_b	#			
1	Front wall at right	N	8	6	48	54.05	20.67	16.54	1.33				Jamb	Edge	12.08		8.00			2	2	6	OK	0.59	0.50
2	Front wall at right	N	8	6	48	54.05	20.67	16.54	1.33	1.00	500		Jamb	Edge	12.08		8.00			2	2	6	OK	0.60	0.50
3		0	0	0	0	0	0.00	0.00	0.00				Jamb	Edge						1	1	6			
4		0	0	0	0	0	0.00	0.00	0.00				Jamb	Edge						1	1	6			
5		0	0	0	0	0	0.00	0.00	0.00				Jamb	Edge						1	1	6			

Note: Parapet lateral trib is taken as the spacing of the wall reinforcement plus the spacing of the jamb bars, unless the height of the opening exceeds 75% of the height of the roof, in which case the lateral trib becomes half the spacing of the wall reinforcement plus the spacing of the jamb bars plus half the width of the opening

Note: Wall wind is used on the parapet when checking the wall moment, parapet wind is used on the parapet when checking the parapet moment

User Notes:
1. Number of Reinforced Cells should only include full height reinforcement.
2. Number of Grouted Cells should include grout below steel lintel bearing.
3. Pier or End Length should be the full width of masonry jamb and include any steel lintel bearing length (refer to diagrams below)
4. Jamb design assumes a uniform lateral trib on the jamb trib width.

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LINTEL DESIGN

	CMU Wall Details					Low Roof Loads			Jamb Details				Lintel Details											Final Design Check				
	Girt	CMU Wall Thick.	CMU Wall Weight	Top of Wall El.	Roof El.	Low Roof Trib Width	Low Roof Drift Only	Add'l Wall Weight	1st Opening Width	2nd Opening Width	Top of Opening El.	Lintel End Brg.	Lintel Type	Lintel Depth	Horiz Reinf Size	# of Bottom Reinf Bars	# of Top Reinf Bars	Bottom Reinf Bar Clearance	Arching Action Eligible	Include Arching Action	Check Lateral Loads	Defl. Limit Vert	Defl. Limit Horiz	CMU	Steel	Shear	Steel Deflect	Design Check
Wall Location	Y/N	in	psf	ft	ft	ft	plf	plf	ft	ft	ft	in		in	#			in	Yes/No	Yes/No	Yes/No	L/x	L/x					
Front wall at right	N	8	54.05	20.67	16.54	0.00	0	0	12.08	0.00	8	0.00	CMU	32.00	6	2	2	7.50	No	No	Yes			0.66	0.47	0.73		OK
Front wall at right	N	8	54.05	20.67	16.54	1.00	500	1	12.08	0.00	8	0.00	CMU	32.00	6	2	2	7.50	No	No	Yes			0.67	0.48	0.74		OK
													CMU	16.00	6	1	0	3.50		No	Yes							
													CMU	16.00	6	1	0	3.50		No	Yes							
													CMU	16.00	6	1	0	3.50		No	Yes							

Notes:
1. Girt uses a simple span condition equal to the wall span (TOF to Roof) multipled by a factor of 0.75.
2. Lintel lateral trib assumes 1/2 height of wall trib (TOF to Roof) trib to lintel in lateral direction.
3. Steel lintels are assumed to be braced at the mid-span for design.

Notes:
4. Deflection is only checked for steel lintels.
5. Shear is only checked for Masonry Lintels
6. Refer to page 7 for load combinations.

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PARAPET JAMB DESIGN

		CMU Wall Details						Jamb Details						Jamb Design Values			Final Design Check			Uniform Load			
																				Wind		Seismic	
Calc #	Wall Location	CMU Wall Thick.	Rebar Size	Bar Spacing	CMU Wall Weight	Top of Wall El.	Roof El.	Jamb Type	Jamb Reinf Location	1st Opening Width	2nd Opening Width	Lintel End Brg.	Pier or End Jamb Width	# of Cells Grouted	# of Tension Bars	Jamb Rebar Size	Grout, Reinf, and Stress Design Check	Max CMU Stress Ratio	Max Steel Stress Ratio	w Jamb parapet	Mwall Design	eq Jamb parapet	Mwall Design
		in	#	in	psf	(ft)	(ft)			(ft)	(ft)	(in)	(ft)	n_c	n_b	#				plf	lb-ft	plf	lb-ft
1	Side wall	8	6	48	54.05	20.67	16.54	Jamb	Edge	2.00		0.00		1	1	6	OK	0.12	0.14	80.67	687	33.33	284
2	Side wall	8	6	48	54.05	20.67	16.54	Pier	Edge	2.00	2.00	0.00	2.00	1	1	6	OK	0.16	0.16	96.80	825	40.00	341
3		0	0	0	0	0.00	0.00	Jamb	Edge			0.00		1	1	6							
4		0	0	0	0	0.00	0.00	Jamb	Edge			0.00		1	1	6							
5		0	0	0	0	0.00	0.00	Jamb	Edge			0.00		1	1	6							
6		0	0	0	0	0.00	0.00	Jamb	Edge			0.00		1	1	6							
7		0	0	0	0	0.00	0.00	Jamb	Edge			0.00		1	1	6							
8		0	0	0	0	0.00	0.00	Jamb	Edge			0.00		1	1	6							
9		0	0	0	0	0.00	0.00	Jamb	Edge			0.00		1	1	6							
10		0	0	0	0	0.00	0.00	Jamb	Edge			0.00		1	1	6							

JAMB SUPPORT DATA & DETAILS

Wall Lateral Forces

Parapet WL, Service =	24.2	psf
Parapet(serv.) 0.7*EQ =	0.185	*weight

Wall Properties

Masonry Inspection =	Y	y/n
Masonry, f_m =	2,000	psi
Rebar yield strength, F_y=	60,000	psi
Masonry Mod. Of Elasticity, E_m =	1,800	ksi
Steel Mod. Of Elasticity, E_s=	29,000	ksi

Fm =	900	psi
Fs =	32000	psi
$n = E_s/E_m$ =	16.11	

Reinforcing Bar Properties

size	area
4	0.20 in^2
5	0.31 in^2
6	0.44 in^2
7	0.60 in^2
8	0.79 in^2

Wall Thickness Properties

Unit	tf	d(edge)	d(center)	Isolid	Ihollow	Asolid	Ahollow	ta
6	1.00	2.875	2.813	178	139.3	67.5	32.2	5.625
8	1.25	4.875	3.813	443.3	334	91.5	41.5	7.625
10	1.25	6.875	4.813	891.7	606.3	115.5	48	9.625
12	1.25	8.875	5.813	1571	971.5	139.5	53.1	11.625

Parapet Jamb Schedule

Wall Thick.	Opening Width	Jamb Width
8 in	2.00 ft	8 in
8 in	2.00 ft	16 in
8 in	2.00 ft	24 in
12 in	0.00 ft	8 in
12 in	0.00 ft	16 in
12 in	0.00 ft	24 in

Load Combinations

1	1.00	DL	0.00	RLL	0.00	SL	0.00	WL	0.00	EQ
2	1.00	DL	0.00	RLL	1.00	SL	1.00	WL	0.00	EQ
3	1.00	DL	0.00	RLL	0.00	SL	0.00	WL	1.00	EQ
4	0.00	DL	0.00	RLL	0.00	SL	0.00	WL	0.00	EQ
5	0.00	DL	0.00	RLL	0.00	SL	0.00	WL	0.00	EQ
6	0.00	DL	0.00	RLL	0.00	SL	0.00	WL	0.00	EQ

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CMU WALL DESIGN WORKSHEET - EXISTING — TMS 402-16 — Version 2.0

Wall Lateral Forces

Wall Location	Wind			Seismic	
Main Wall	Wall WL, Service =	14.8 psf		OOP Wall(serv.) 0.7*EQ =	0.070
Parapet Only	Parapet WL, Service =	24.2 psf		Parapet(serv.) 0.7*EQ =	0.185
DL= 15 psf		RoofLL= 20 psf		SL= 21.5 psf	

Wall Properties

CMU Density= 135 pcf

Masonry Inspection =	Y		Steel Mod. Of Elasticity, E_s=	29,000	ksi
Masonry, f_m =	2,000	psi	Fm =	900	psi
Rebar yield strength, F_y=	60,000	psi	Fs =	32000	psi
Masonry Mod. Of Elasticity, E_m =	1,800	ksi	$n = E_s/E_m$ =	16.11	

General Wall Information												Seismic Loads		Roof Uniform Loads			Wall Properties									
Wall Location	Bearing Wall Y / N	Wind Girt Location Y / N	Top of Wall Elev. (ft)	Jst. Brg. Elev. (ft)	Roof Elev. h (ft)	Depth to T.O. Ftg h_f (ft)	Nominal Wall Thk. t (in)	Rebar Location c/e/comp	Face Shell Thick. t_f (in)	Depth to Rebar d (in)	Wall Weight Wt (psf)	Wall EQ_w (psf)	Parapet EQ_p (psf)	DL wd (plf)	RLL wl (plf)	SL ws (plf)	Steel Ratio p	Radius of Gyration r (in)	Eff. Wall Area A_w (in^2)	k_1	k_1*d (in)	j_1	k_2	k_2*d (in)	j_2
Existing wall at 1	Y	N	25.33	17.33	17.75	1.33	8	Center	1.25	3.813	58.65	4.11	10.85	380	500	1040	0.003606	2.586736	122.5888	0.28769	1.09697	0.9041	0.28978	1.10493	0.847
Existing wall at B/7	N	N	28.67	22.50	22.92	1.33	8	Center	1.25	3.813	58.65	4.11	10.85				0.003606	2.586736	122.5888	0.28769	1.09697	0.9041	0.28978	1.10493	0.847
Existing wall at B/5	N	N	31.33	22.00	22.42	1.33	12	Center	1.25	5.813	79.35	5.55	14.68				0.001892	3.937924	175.8288	0.21832	1.26908	0.92723	0.21834	1.26921	0.89683
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
						0			#N/A	#N/A	0						#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

CMU WALL DESIGN WORKSHEET

Wall Location	Bending Forces: Wind				Bending Forces: Seismic				Roof Loads + Self Wt.				Rectangular Section Stresses: CMU Comp. Str.		Rectangular Section Stresses: Steel Stress		T-Beam Section Stresses: CMU Comp. Str.		T-Beam Section Stresses: Steel Stress		Applied Axial Stresses: Roof Loads + Self Wt.			Allow. Axial Stress	Axial Check
	M_{span} lb-ft	$M_{parapet}$ lb-ft	M_{max} lb-ft	hm ft	M_{span} lb-ft	$M_{parapet}$ lb-ft	M_{max} lb-ft	hm ft	eccent. e in.	DL M_{roof} lb-ft	RLL M_{roof} lb-ft	SL M_{roof} lb-ft	Wind fb1 psi	EQ fb1eq psi	Wind fs1 psi	EQ fs1eq psi	Wind fb2 psi	EQ fb2eq psi	Wind fs2 psi	EQ fs2eq psi	DL fadl psi	RLL fall psi	SL fasl psi	Fa psi	
Existing wall at 1	1253	1856	1856	7.58	353	832	832	7.58	2	169	222	462	368	165	14680	6582	397	178	15669	7025	18	11	23	300	O.K.
Existing wall at B/7	2565	1068	2565	18.52	716	479	716	18.56	0	0	0	0	509	142	20288	5668	548	153	21656	6050	24	0	0	194	O.K.
Existing wall at B/5	2542	3204	3204	9	963	1944	1944	9	0	0	0	0	281	170	16214	9835	291	176	16763	10169	13	0	0	366	O.K.
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!
0	#DIV/0!	0	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!	#DIV/0!	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#DIV/0!

CMU WALL DESIGN WORKSHEET

Wall Location	Masonry Combined Stress Ratios Due to Axial & Bending: DL+RLL	DL+SL	DL+WL	DL+EQ	DL+ 0.75WL+ .75(LorS)	DL+ 0.75EQ+ .75(LorS)	Steel Combined Stress Ratios Due to Axial Eccentric & Bending: DL+RLL	DL+SL	DL+WL	DL+EQ	DL+ 0.75WL+ .75(LorS)	DL+ 0.75EQ+ .75(LorS)
Existing wall at 1	0.118	0.184	0.466	0.240	0.459	0.290	0.097	0.156	0.500	0.247	0.472	0.282
Existing wall at B/7	0.026	0.026	0.591	0.184	0.450	0.145	0.000	0.000	0.634	0.177	0.476	0.133
Existing wall at B/5	0.015	0.015	0.338	0.211	0.257	0.162	0.000	0.000	0.524	0.318	0.393	0.238
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Design Summary

Wall Location	Bar Spacing Sb (in)	Bar Size	Bar Area Ab (in^2)	Max CMU Stress Ratio	Max Steel Stress Ratio	Check	Does Parapet or Wall Control?
Existing wall at 1	32	6	0.44	0.47	0.50	O.K.	Parapet
Existing wall at B/7	32	6	0.44	0.59	0.63	O.K.	Wall
Existing wall at B/5	40	6	0.44	0.34	0.52	O.K.	Parapet
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!
0	0	0	#N/A	#DIV/0!	#N/A	#DIV/0!	#DIV/0!

Project No.	Project Name:	Made By:	Sheet No:
2431906958	WM 6958 Cameron, NC existing	DTR	1

JAMB DESIGN - EXISTING BUILDING

Calc #		CMU Wall Details								Low Roof Loads			Jamb Details							Jamb Design Values			Final Design Check		
		Girt	CMU Wall Thick.	Rebar Size	Bar Spacing	CMU Wall Weight	Top of Wall El.	Roof El.	Top of Ftg El below FF	Low Roof Trib Width	Low Roof Drift Only	Add'l Wall Weight	Jamb Type	Jamb Reinf Location	1st Opening Width	2nd Opening Width	Top of Opening El.	Lintel End Brg.	Pier or End Jamb Width	# of Cells Grouted	# of Tension Bars	Jamb Rebar Size	Grout, Reinf, and Stress Design Check	Max CMU Stress Ratio	Max Steel Stress Ratio
	Wall Location	Y/N	in	#	in	psf	(ft)	(ft)	(ft.)	(ft)	(plf)	(plf)			(ft)	(ft)	(ft)	(in)	(ft)	n_c	n_b	#			
1	Existing wall at 1	N	8	6	32	58.65	25.33	17.75	1.33				Jamb	Center	6.08		7.33	8.00		2	1	6	**Check Stress**	0.92	**0.86**
2	Existing wall at B/7	N	8	6	32	58.65	28.67	22.92	1.33				Jamb	Center	3.33		7.33	8.00		2	2	6	OK	0.90	0.61
3	Existing wall at B/5	N	12	6	40	79.35	31.33	22.42	1.33				Jamb	Center	3.33		7.33	8.00		1	1	6	OK	0.55	0.58
4		0	0	0	0	0	0.00	0.00	0.00				Jamb	Center				8.00		2	1	6			
5		0	0	0	0	0	0.00	0.00	0.00				Jamb	Center				8.00		2	1	6			

OK SINCE THIS IS AN INTERIOR OPENING AND DOESN'T HAVE FULL WIND PRESSURE

Note: Parapet lateral trib is taken as the spacing of the wall reinforcement plus the spacing of the jamb bars, unless the height of the opening exceeds 75% of the height of the roof, in which case the lateral trib becomes half the spacing of the wall reinforcement plus the spacing of the jamb bars plus half the width of the opening

Note: Wall wind is used on the parapet when checking the wall moment, parapet wind is used on the parapet when checking the parapet moment

User Notes:
1. Number of Reinforced Cells should only include full height reinforcement.
2. Number of Grouted Cells should include grout below steel lintel bearing.
3. Pier or End Length should be the full width of masonry jamb and include any steel lintel bearing length (refer to diagrams below)
4. Jamb design assumes a uniform lateral trib on the jamb trib width.

Project No.	Project Name:	Made By:	Sheet No:
2431906958	WM 6958 Cameron, NC existing	DTR	1

LINTEL DESIGN - EXISTING BUILDING

	CMU Wall Details					Low Roof Loads			Jamb Details				Lintel Details											Final Design Check				
Wall Location	Girt	CMU Wall Thick.	CMU Wall Weight	Top of Wall El.	Roof El.	Low Roof Trib Width	Low Roof Drift Only	Add'l Wall Weight	1st Opening Width	2nd Opening Width	Top of Opening El.	Lintel End Brg.	Lintel Type	Lintel Depth	Horiz Reinf Size	# of Bottom Reinf Bars	# of Top Reinf Bars	Bottom Reinf Bar Clearance	Arching Action Eligible	Include Arching Action	Check Lateral Loads	Defl. Limit Vert	Defl. Limit Horiz	CMU	Steel	Shear	Steel Deflect	Design Check
	Y/N	in	psf	ft	ft	ft	plf	plf	ft	ft	ft	in		in	#			in	Yes/No	Yes/No	Yes/No	L/x	L/x					
Existing wall at 1	N	8	58.65	25.33	17.75	0.00	0	0	6.08	0.00	7.33	8.00	W8X24						Yes	No	Yes	600	600	0.00	0.21	0.00	OK	OK
Existing wall at B/7	N	8	58.65	28.67	22.92	0.00	0	0	3.33	0.00	7.33	8.00	CMU	8.00	6	1		3.50	Yes	Yes	Yes	600	600	0.31	0.15	0.40		OK
Existing wall at B/5	N	12	79.35	31.33	22.42	0.00	0	0	3.33	0.00	7.33	8.00	CMU	8.00	6	2		3.50	Yes	Yes	Yes	600	600	0.18	0.07	0.30		OK
													W8X24							No	Yes	600	600					
													W8X24							No	Yes	600	600					

Notes:
1. Girt uses a simple span condition equal to the wall span (TOF to Roof) multipled by a factor of 0.75.
2. Lintel lateral trib assumes 1/2 height of wall trib (TOF to Roof) trib to lintel in lateral direction.
3. Steel lintels are assumed to be braced at the mid-span for design.

Notes:
4. Deflection is only checked for steel lintels.
5. Shear is only checked for Masonry Lintels
6. Refer to page 7 for load combinations.