NSI

NODAL STRUCTURES, INC.

7725 SW 114 ST

By: heritage Homes

Rational Analysis and Engineering Design Calculations

Miami-Dade County, Florida
2020 Florida Building Code

10316 N.W. 55th Street, Sunrise, FLORIDA 33351 PHONE (954) 741-2572 FAX (954) 741-8635

INDEX

Carlos Nodal

Digitally signed by Carlos Nodal
DN: c=US, st=Florida, l=Sunrise, o=Carlos Nodal, cn=Carlos Nodal, email=Nodalco@yahoo.com
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NSI

NODAL STRUCTURES, INC.
STRUCTURAL ENGINEERS
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10316 N.W. 55TH STREET
SUNRISE, FLORIDA 33351

LOAD ANALYSIS

ROOF

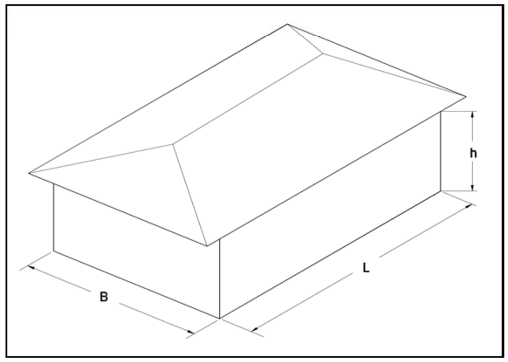
DEAD LOADS	5.0	12.0	a =	22.6	
Wood truss system				4.0	Lbs/ft^2
Superimposed loads				21.0	
				25.0	**Lbs/ft^2**
LIVE LOADS				**20.0**	**Lbs/ft^2**

Dead loads	25.0	Lbs/ft^2	x	1.2	30.0	
Live loads	20.0		x	1.6	32.0	
	45.0	Lbs/ft^2			**62.0**	**Lbs/ft^2**

WIND ANALYSIS AND DESIGN - ASCE 7-16

INPUT DATA:

Classification=	Enclosed Buildings	
Exposure=	C	
Risk Category=	II	
Factor=	1	
Wind Speed=	175.00	mph
Eave Height, h=	12.00	ft
Mean Roof Height, H=	15.50	ft
Gust Effect Factor, G=	0.85	
Wind Direct. Factor,Kd =	0.85	
Topogra. Factor,Kzt =	1.00	
Ground elev. Factor,Ke =	1.00	
a =	6.20	



Roof:

Building Length L=	130.00	ft
Building Width B=	102.00	ft

2nd Floor:

Building Length L=	-	ft
Building Width B=	-	ft

MAIN WIND FORCE RESISTING SYSTEM (MWFRS)

$q_h = 0.00256 \times Kz \times Kzt \times Kd \times Ke \times V^2$ L/B = 1.27

$P = q \times GCp - qh \times GCpi$

H	Exposure Coefficient,
ft	Kz (ft)
0 to 15	0.85
15 to 20	0.90
20 to 25	0.94
25 to 30	0.98

Wall	Wall Pressure Coefficient, Cp	
Windward Walls	0.80	
Leeward Walls	-0.50	-0.30
Side Walls	-0.70	

P = qh x [GCp - (GCpi)]
GCp = External Pressure Coefficients
GCpi = Internal Pressure Coefficients

GCpi = -0.18 or 0.18

Internal Pressure, qh * Gcpi = -6.48 psf or 6.48 psf

DESIGN WALL PRESSURES - MWFRS

A.) FOR THE DESIGN OF SHEAR WALLS AND ROOF DIAPHRAGM

L/B = 1.27

H	Wind Pressure		Wall Presures				
			Side walls	Windward	Leeward	Pressures	
ft	psf					psf	
0 to 15	33.99	-	-26.34	29.23	-20.56	-49.79	
15 to 20	35.99	= qh	-27.89	30.95	-21.77	-52.72	= qh
20 to 25	37.58	-	-29.13	32.32	-22.74	-55.06	
25 to 30	39.18	-	-30.37	33.70	-23.71	-57.41	

DESIGN ROOF PRESSURES - MWFRS

SLOPE = 5 | 12 Angle = 22.62 degrees

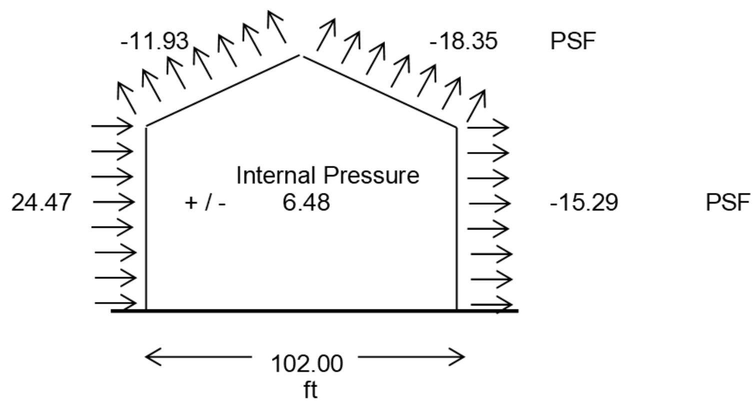
h/L = 0.12

q_h = 35.99 psf

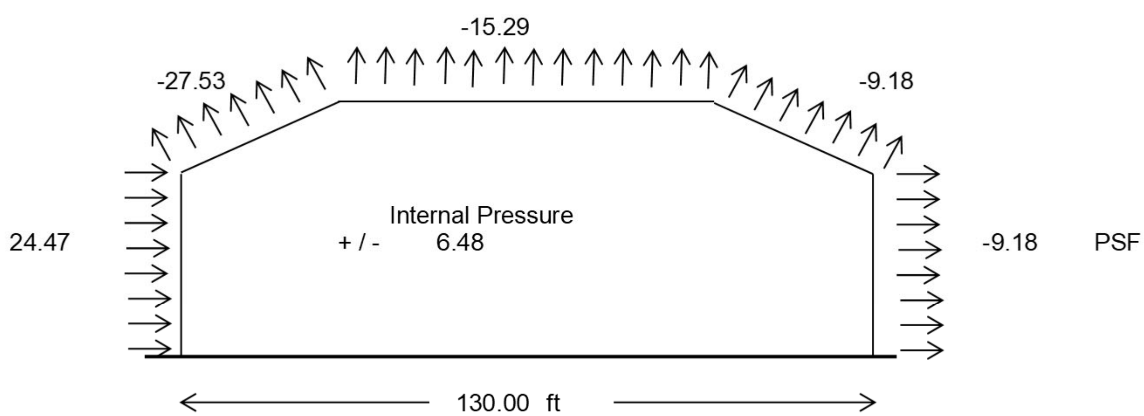
	Wall	Cp	Wind Pressure, P		
			psf		
Normal to ridge	Windward	-0.39	-11.93	+ / -	6.48
	Leeward	-0.60	-18.35	+ / -	6.48
Parallel to ridge	0 to h/2	-0.90	-27.53	+ / -	6.48
	h to 2h	-0.50	-15.29	+ / -	6.48
	> 2h	-0.30	-9.18	+ / -	6.48

SUMMARY OF THE MWFRS WIND PRESSURES

TRANSVERSE WIND (Normal to ridge); PSF



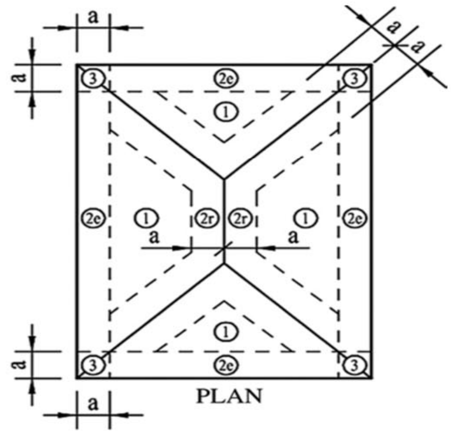
LONGITUDINAL WIND (Parallel to ridge); PSF



B.) FOR THE DESIGN OF COMPONENTS AND CLADDING

AREA	INTERIOR	END ZONE	POSITIVE	GCp	GCp	GCp
	Zone 4	**Zone 5**	**Zone 4 & 5**	Zone 4	Zone 5	Zone 4 & 5
10.00	-46.06	-56.86	42.46	-1.10	-1.40	1.00
15.00	-45.34	-55.06	41.38	-1.08	-1.35	0.97
20.00	-44.62	-53.26	40.66	-1.06	-1.30	0.95
25.00	-43.54	-52.54	40.30	-1.03	-1.28	0.94
30.00	-42.46	-51.82	39.94	-1.00	-1.26	0.93
35.00	-41.74	-50.38	39.58	-0.98	-1.22	0.92
40.00	-40.66	-49.66	39.22	-0.95	-1.20	0.91
45.00	-39.58	-48.58	38.86	-0.92	-1.17	0.90
50.00	-38.50	-47.86	38.14	-0.89	-1.15	0.88
55.00	-38.50	-47.50	37.78	-0.89	-1.14	0.87
60.00	-38.14	-47.14	37.43	-0.88	-1.13	0.86
65.00	-38.14	-46.78	37.07	-0.88	-1.12	0.85
70.00	-38.14	-46.42	36.71	-0.88	-1.11	0.84
75.00	-37.78	-46.06	36.35	-0.87	-1.10	0.83
80.00	-37.78	-45.70	35.99	-0.87	-1.09	0.82
85.00	-37.78	-45.34	35.99	-0.87	-1.08	0.82
90.00	-37.43	-44.98	35.63	-0.86	-1.07	0.81
95.00	-37.43	-44.62	35.63	-0.86	-1.06	0.81
100.00	-37.43	-44.26	35.27	-0.86	-1.05	0.80
105.00	-37.07	-43.90	35.27	-0.85	-1.04	0.80
110.00	-36.71	-43.90	35.27	-0.84	-1.04	0.80
115.00	-36.71	-44.26	35.27	-0.84	-1.05	0.80
120.00	-36.35	-44.26	35.27	-0.83	-1.05	0.80

DESIGN ROOF WIND PRESSURES C & C



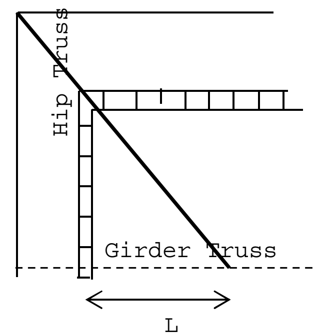
Hip roof

h/B= 0.12

Effective wind area = 34 ft^2

Zones		GCp	WIND PRESS psf
Roof	1	-0.85	**-37.07**
Roof	2e, 2r & 3	-1.30	**-53.26**
Overhang	2e, 2r	-2.40	**-92.84**
Overhang	3	-2.80	**-107.24**

ROOF TRUSS DESIGN



HIP TRUSS

For zone 2 pressures - 5 psf (dead load)		-48.26	psf
For zone 3 pressures - 5 psf (dead load)		-102.24	psf
L =	7.00	ft	
Tributary Area =	24.35	ft2	
Overhang =	3.00	ft	
Hip reaction =	-470	Lbs	
Overhang at zone 3 =	-818	Lbs	
	-1288	Lbs	

CONNECTOR: **1-USP HTA16R w/ 10-10d NAILS TO TRUSS**

Uplift capacity = 1870 Lbs

1870 Lbs > 1288 **O.K.**

JACK TRUSS

For zone 2 pressures - 5 psf (dead load)		-48.26	psf
For zone 2 overhang - 5 psf (dead load)		-102.24	psf
Spacing @	2.00	ft	
L =	7.00	ft	
Tributary Area =	7.00	ft2	
Overhang =	3.00	ft	
Jack reaction=	338	Lbs	
Overhang at zone 3 =	613	Lbs	
Uplift	951	Lbs	

CONNECTOR: **1-USP HTA16R w/ 10-10d NAILS TO TRUSS**

Uplift capacity = 1870 Lbs

1870 Lbs > 951 **O.K.**

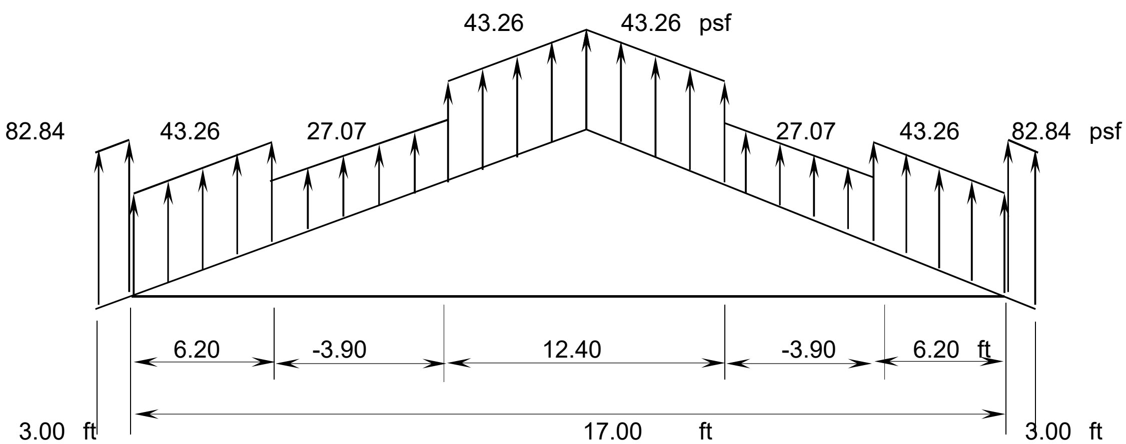
ROOF TRUSS DESIGN

Truss length =	17.00	ft
Afferent Area =	34.00	ft^2
a =	6.20	ft

Net pressure = design pressure C&C - 10 psf (dead loads)

P 1 =	-27.07	psf
P 2 =	-43.26	psf
Poverhang =	-82.84	psf

Forces parallel to ridge, F1 =	92	lbs/ft
Forces normal to ridge, F2 =	76	lbs/ft



Uplift reaction =	984	lbs
F1 =	81	lbs
F2 =	76	lbs

F1

F2

CONNECTOR: **USP DHTA20 DOUBLE ANCHOR w/ 10-10d NAILS TO EACH STRAP**

Uplift capacity =	2430	**Lbs**	**>**	984	**O.K.**
F1 =	1215	Lbs	>	81	**O.K.**
F2 =	1310	Lbs	>	76	**O.K.**

UNITY EQUATION

$$\frac{\text{UPLIFT LOAD}}{\text{UPLIFT CAPACITY}} + \frac{\text{LATERAL LOAD}}{\text{LATERAL CAP.}} + \frac{\text{PERPEND. LOAD}}{\text{PERPEND. CAP.}} = \mathbf{0.53}$$

$$\frac{984}{2430} + \frac{81}{1215} + \frac{76}{1310}$$

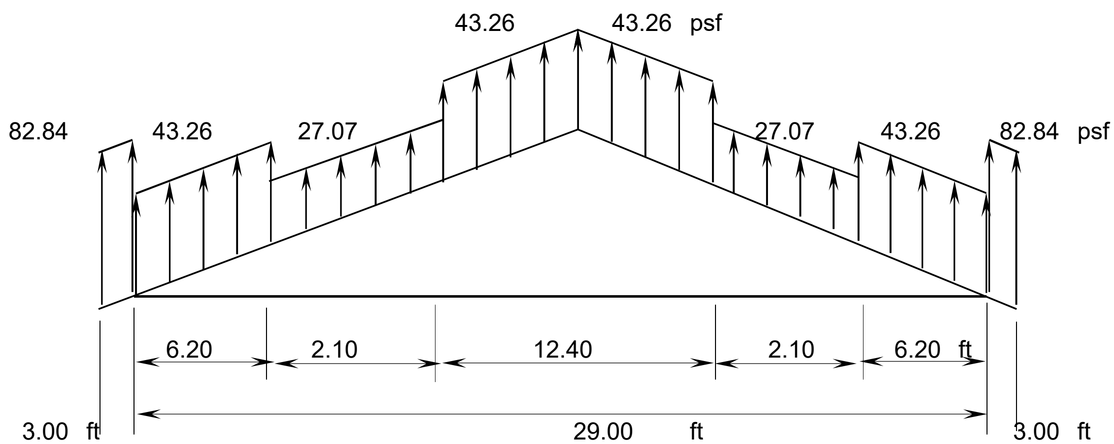
ROOF TRUSS DESIGN

Truss length =	29.00	ft
Afferent Area =	58.00	ft^2
a =	6.20	ft

Net pressure = design pressure C&C - 10 psf (dead loads)

P 1 =	-27.07	psf
P 2 =	-43.26	psf
Poverhang =	-82.84	psf

Forces parallel to ridge, F1 =	92	lbs/ft
Forces normal to ridge, F2 =	76	lbs/ft



Uplift reaction =	1684	lbs
F1 =	81	lbs
F2 =	76	lbs

F1

F2

CONNECTOR: **USP DHTA20 DOUBLE ANCHOR w/ 10-10d NAILS TO EACH STRAP**

Uplift capacity =	2430	**Lbs**	**>**	1684	**O.K.**
F1 =	1215	Lbs	>	81	**O.K.**
F2 =	1310	Lbs	>	76	**O.K.**

UNITY EQUATION

$$\frac{\text{UPLIFT LOAD}}{\text{UPLIFT CAPACITY}} + \frac{\text{LATERAL LOAD}}{\text{LATERAL CAP.}} + \frac{\text{PERPEND. LOAD}}{\text{PERPEND. CAP.}} = \mathbf{0.82}$$

$$\frac{1684}{2430} + \frac{81}{1215} + \frac{76}{1310}$$

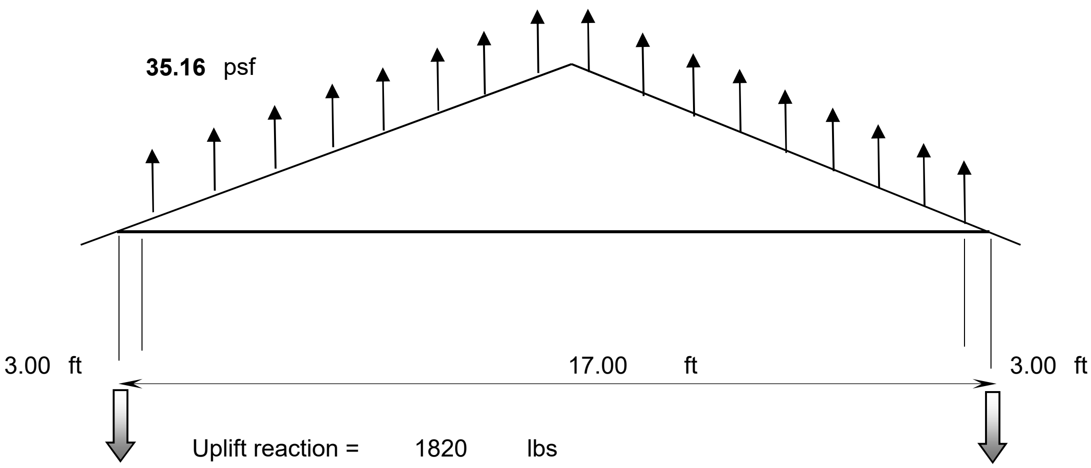
ROOF GIRDER TRUSS DESIGN

Girder length =	17.00	ft	**4.50** Girder afference
Afferent Area =	76.50	ft2	

a = 6.20 ft

Net pressure = design pressure C&C - 10 psf (dead loads)

P 1 =	-27.07	psf
P 2 =	-43.26	psf
P overha	-82.84	psf

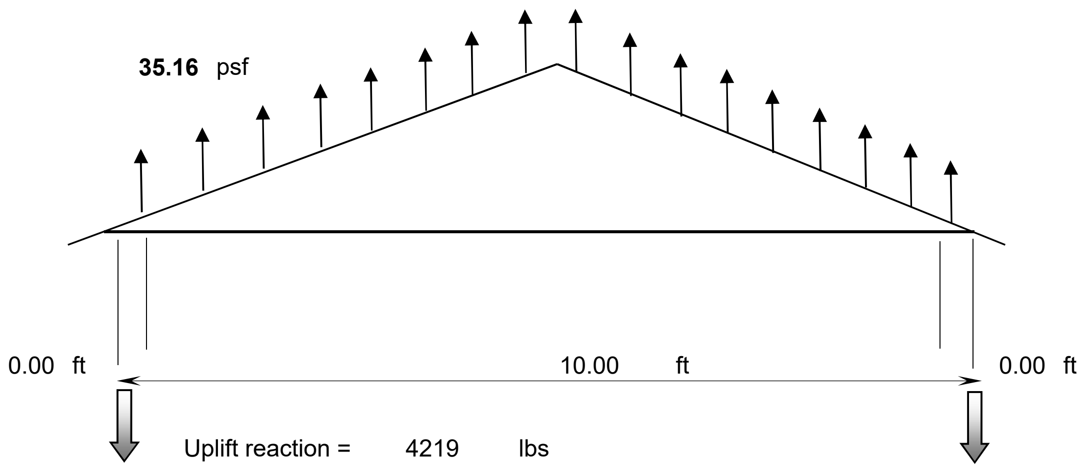


ROOF GIRDER TRUSS DESIGN

Girder length =	10.00	ft	**24.00** Girder afference
Afferent Area =	240.00	ft2	
a =	6.20	ft	

Net pressure = design pressure C&C - 10 psf (dead loads)

P 1 =	-27.07	psf
P 2 =	-43.26	psf
P overhang	-82.84	psf



MASONRY WALLS LATERAL LOADS

WIND PRESSURES FROM COMPONENTS AND CLADDING:

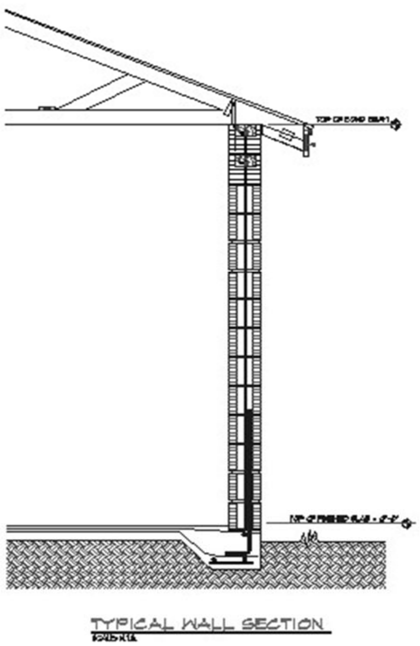
Reinf. masonry cells spacing	2.00	ft
Wall elevation	12.00	ft
Tributary area	24.00	ft^2

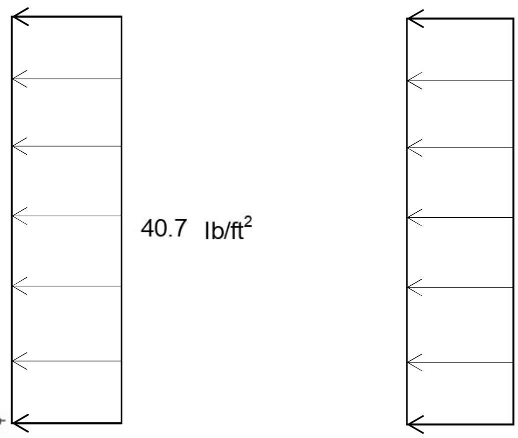
Design pressures interior zone 4 :	40.66	lb/ft^2
Design pressures end zone 5 :	49.66	lb/ft^2

ZONES

INTERIOR Zone 4 — END Zone 5

12.00 ft



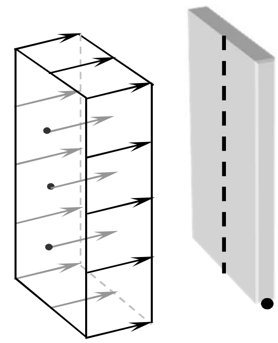


40.7 lb/ft^2

49.7 lb/ft^2

MASONRY WALL DESIGN

FIRST FLOOR	ZONE 4



DESIGN CRITERIA
TSM 402/602

Fs =	32.00	KSI
Fv =	35.00	PSI
F'm =	2000.00	PSI
Es =	29000.00	KSI
Em =	1800.00	KSI
n =	16.11	
Fb =	666.67	PSI
Fa =	368.51	PSI

OUT OF PLANE BENDING FOR MASONRY WALLS

HEIGHT	12.00	ft
B (wall) =	24	in
D =	3.80	in
rho =	0.0034	
rho*n =	0.05	
k =	0.28	
j =	0.91	

REINFORCEMENT 1#5 @ 24" o.c. = 0.31 in^2

TRIBUTARY LOADED AREA =	2.0	ft
AREA (wall plus openings)	24.0	ft^2
WIND PRESSURE =	40.7	psf
AXIAL LOAD/FT =	2.00	kips/ft
LOAD=	4.00	kips
AREA OF MASONRY =	146.40	in^2
WIND LOAD =	81.33	p/ft

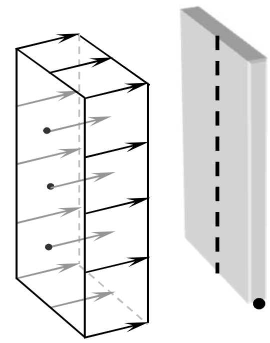
MOMENT APPLY Ma =	17.57	kip-in		
MOMENT CAPACITY Ms=	34.17	kip-in	Ma <Ms	**O.K.**

Ms = As Fs j d

fs =	16.45	ksi	<	32.00	**O.K.**
fb =	398.46	psi	<	666.67	**O.K.**
fa =	27.32	psi			

MASONRY WALL DESIGN

FIRST FLOOR	ZONE 5



DESIGN CRITERIA
TSM 402/602

Fs =	32.00	KSI
Fv =	35.00	PSI
F'm =	2000.00	PSI
Es =	29000.00	KSI
Em =	1800.00	KSI
n =	16.11	
Fb =	666.67	PSI
Fa =	368.51	PSI

OUT OF PLANE BENDING FOR MASONRY WALLS

HEIGHT =	12.00	ft
B (wall) =	24	in
D =	3.80	in
rho =	0.0034	
rho*n =	0.05	
k =	0.28	
j =	0.91	

REINFORCEMENT **1#5 @ 24" o.c. =** **0.31** **in^2**

TRIBUTARY LOADED AREA =	2.0	ft
AREA (wall plus openings)	24.0	ft^2
WIND PRESSURE =	49.7	psf
AXIAL LOAD/FT =	2.00	kips/ft
LOAD=	4.00	kips
AREA OF MASONRY =	146.40	in^2
WIND LOAD =	99.32	p/ft

MOMENT APPLY Ma =	21.45	kip-in		
MOMENT CAPACITY Ms=	34.17	kip-in	Ma <Ms	**O.K.**

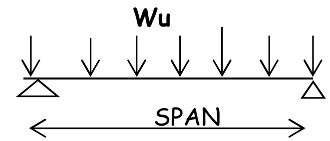
Ms = As Fs j d

fs =	20.09	ksi	<	32.00	**O.K.**
fb =	486.61	psi	<	666.67	**O.K.**
fa =	27.32	psi			

CONCRETE BEAMS DESIGN

INPUT DATA		
f'_c =	3	ksi
f_y =	60	ksi steel bars
f_y =	40	ksi ties
ϕ =	0.90	Axial tension & bending
ϕ =	0.75	Shear & Torsion
β_1 =	0.85	
ρ_{min} =	0.0033	
d'=	2.5	

LOADS				
Floor	Roof	Brow	Concrete	Wall
lb/ft^2	lb/ft^2	lb/ft^2	lb/ft^3	lb/ft^2
94.0	62.0	188.0	150.0	55.0



BEAM	GEOMETRY				AFFECTED AREA					Wu	FLEXURE DESIGN						
	B	H	Span	d	Floor	Roof	Wall	Slab	Self w		Mu	K	ρ	$0.75\rho_{bal}$	K_{bal}	As_{bal}	As_2
	in	in	ft	in	ft	ft	ft	ft	ft^2	Kip/ft	Kip*ft					in^2	in^2
B-1	16	24	11.00	21.5	0	9.5	0	0	480.0	1.069	16.17	26	0.0005	0.0160	703	5.52	-4.88
B-2	16	24	25.00	21.5	0	6.5	0	0	480.0	0.883	68.98	112	0.0021	0.0160	703	5.52	-4.26
B-3	8	24	13.00	21.5	0	16	0	0	240.0	1.232	26.03	84	0.0016	0.0160	703	2.76	-2.23
B-4	8	38	9.00	35.5	0	32	7.17	0	380.0	2.837	28.73	34	0.0006	0.0160	703	4.55	-3.78
B-5	8	20	6.00	17.5	0	32	7.17	0	200.0	2.657	11.96	59	0.0011	0.0160	703	2.24	-1.95
B-6	8	38	7.00	35.5	0	0	3	0	380.0	0.578	3.54	4	0.0001	0.0160	703	4.55	-3.95
B-7	12	38	21.00	35.5	0	6.5	7.17	0	570.0	1.446	79.72	63	0.0012	0.0160	703	6.83	-5.43
B-8	8	24	7.00	21.5	0	3.5	0	0	240.0	0.457	2.80	9	0.0002	0.0160	703	2.76	-2.50
B-21	8	60	17.00	57.5	0	11	0	1	600.0	1.470	53.10	24	0.0004	0.0160	703	7.38	-6.04
B-21	8	12	6.33	9.5	0	0	0	3	120.0	0.684	3.43	57	0.0011	0.0160	703	1.22	-1.23
SB-1	38	12	21.00	9.5	0	0	0	0	570.0	0.570	31.42	110	0.0021	0.0160	703	5.79	-5.38
SB-2	48	12	11.00	9.5	0	0	0	0	720.0	0.720	10.89	30	0.0006	0.0160	703	7.31	-7.71

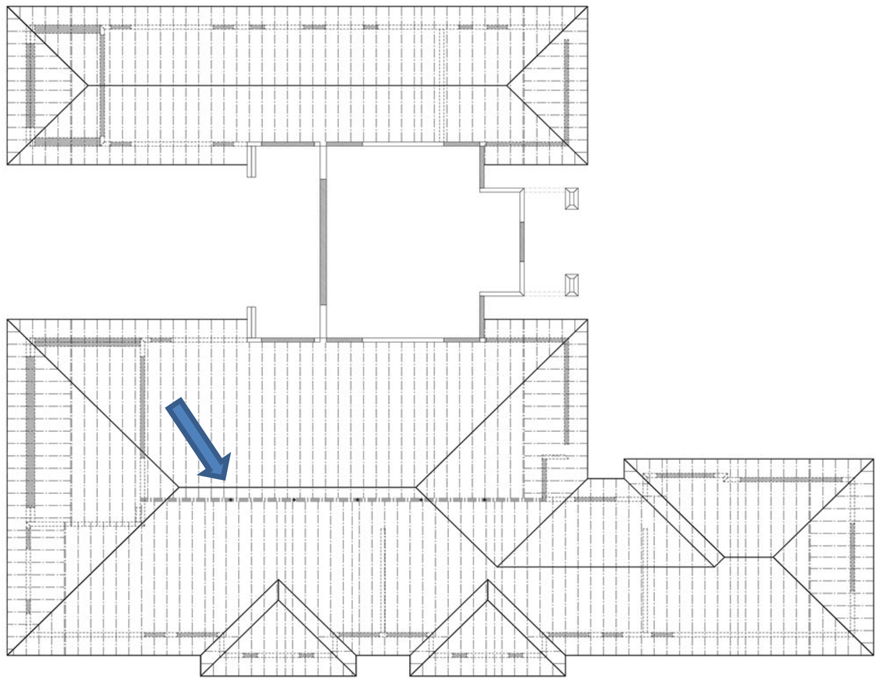
CONCRETE BEAMS DESIGN

INPUT DATA			
f'_c =	3	ksi	
f_y =	60	ksi	steel bars
f_y =	40	ksi	ties
ϕ =	0.90	Axial tension & bending	
ϕ =	0.75	Shear & Torsion	
β_1 =	0.85		
ρ_{min} =	0.0033		
d'=	2.5		

LOADS				
Floor	Roof	C. Slab	Concrete	Wall
lb/ft^2	lb/ft^2	lb/ft^2	lb/ft^3	lb/ft^2
94.0	62.0	188.0	150.0	55.0

BEAM	REINFORCEMENT					SHEAR DESIGN						
	As btt		**As'top**		**As mid**	**Vu**	ϕ**Vc**		**s**	**s** $_{max}$	**As ties**	USE
	in^2		in^2			Kips	Kips		in	in	in^2	
B-1	**1.15**	**3-#7**	**0.00**	**3-#7**	**2-#5**	5.9	28	**O.K.**	-6	10.8	**0.22**	**#3 @ 8" o.c.**
B-2	**1.15**	**3-#7**	**0.00**	**3-#7**	**2-#5**	11.0	28	**O.K.**	-8	10.8	**0.22**	**#3 @ 8" o.c.**
B-3	**0.57**	**2-#5**	**0.00**	**2-#5**	**2-#5**	8.0	14	**O.K.**	-23	10.8	**0.22**	**#3 @ 8" o.c.**
B-4	**0.95**	**2-#7**	**0.00**	**2-#7**	**4-#5**	12.8	23	**O.K.**	-22	17.8	**0.22**	**#3 @ 8" o.c.**
B-5	**0.47**	**2-#5**	**0.00**	**2-#5**	**2-#5**	8.0	12	**O.K.**	-33	8.8	**0.22**	**#3 @ 8" o.c.**
B-6	**0.95**	**2-#7**	**0.00**	**2-#7**	**4-#5**	2.0	23	**O.K.**	-11	17.8	**0.22**	**#3 @ 8" o.c.**
B-7	**1.42**	**3-#7**	**0.00**	**3-#7**	**4-#5**	15.2	35	**O.K.**	-12	17.8	**0.22**	**#3 @ 8" o.c.**
B-8	**0.57**	**2-#5**	**0.00**	**2-#5**	**2-#5**	1.6	14	**O.K.**	-11	10.8	**0.22**	**#3 @ 8" o.c.**
B-21	**1.53**	**2-#7**	**0.00**	**2-#7**	**6-#5**	12.5	38	**O.K.**	-15	28.8	**0.22**	**#3 @ 6" o.c.**
B-21	**0.25**	**2-#5**	**0.00**	**2-#5**	**-**	2.2	6	**O.K.**	-15	4.8	**0.22**	**#3 @ 6" o.c.**
SB-1	**1.20**	**7-#5**	**0.00**	**7-#5**	**-**	6.0	30	**O.K.**	-3	4.8	**0.22**	**#3 @ 6" o.c.**
SB-2	**1.52**	**6-#5**	**0.00**	**6-#5**	**-**	4.0	37	**O.K.**	-2	4.8	**0.22**	**#3 @ 6" o.c.**

STEEL COLUMN DESIGN



LOAD= 14.6 kips

TABLE 4-4 SQUARE STRUCTURAL TUBING HSS 4" x 4" x 1/4"

F_y=	46	ksi	r_x=	1.52	in	E=	29000	ksi
F_u=	58	ksi	r_y=	1.52	in	Ω_c =	1.67	
A_g=	3.37	in	t_{design}=	0.233	in			

EFFECTIVE LENGTH KL - TABLE C-A-7.1

Column Length	L=	12.33	ft
Effective Length Factor	K=	1.00	
Effective Length	KL=	12.33	ft

AVAILABLE STRENGTH AXIAL COMPRESSION - TABLE 4-4

P_n/Ω_c = 50.8 kips > P_a= 14.6 kips **OK**

CHECK FOR SLENDER ELEMENTS - TABLE B4.1 CASE 6

$$b/t = \frac{3.5 - 0.699}{0.233} = 12.0$$

If, b/t < $1.40(E/F_y)^{1/2}$

$1.40(E/F_y)^{1/2}$= 35.15 then, **NON-SLENDER ELEMENTS**

NOMINAL COMPRESSIVE STRENGTH $P_n=F_{cr}A_g$

Determine Critical Stress F_{cr}:

Slenderness Ratio $\frac{KL}{r_y} = \frac{148}{1.52} =$ **97.3** < 200 **OK**

$4.71\sqrt{(E/F_y)}$ = **118**

Elastic Buckling Stress $= F_e \quad \frac{\pi^2 E}{(KL/r_y)^2} =$ **30.2** ksi (E3-4)

If, $\frac{KL}{r_y} \leq 4.71\sqrt{E/F_y}$ then, $F_{cr}= [(0.658)^{Fy/Fe}]F_y$ (E3-2)

If, $\frac{KL}{r_y} > 4.71\sqrt{E/F_y}$ then, $F_{cr}= .877F_e$ (E3-3)

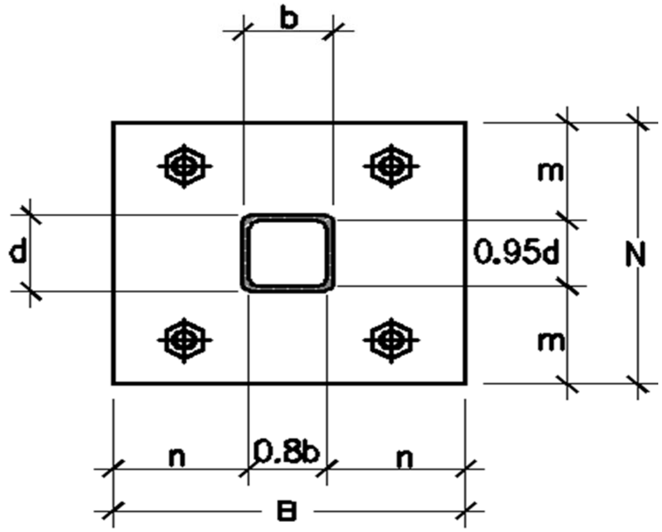
Since $\frac{KL}{r_y} < 4.71\sqrt{E/F_y}$ then, Use Equation (E3-2) for F_{cr}

Therefore, $F_{cr}=$ 24.3 ksi and $P_n=F_{cr}A_g=$ 81.9 kips (E3-1)

$\frac{P_n}{\Omega_c} = \frac{81.9}{1.67} =$ 49.0 kips

$P_n/\Omega_c =$ 49.0 kips	**>**	$P_a=$	14.6 kips	**OK**

STEEL COLUMN BASE PLATE



Properties: ASD

Ω_c= 2.50

Column

F_y= 46 ksi
F_u= 58 ksi
b= 4 in.
d= 4 in.
A_{col}= 16 in^2

Base Plate

F_y= 36 ksi
F_u= 58 ksi
t_p= 0.75 in.
N= 12 in.
B= 12 in.

Footing

Dim. = 60 in. x 60 in.
f'_c= 3000 psi
A_{ped}= 3600 in^2

Total Load

P_a= 14.9 kips

Required Base Plate Area

$$A_{1(req)} = \frac{P_a \Omega_c}{0.85 f'_c} = 14.6 \quad in.^2 \quad (J8\text{-}1)$$

Verify Base Plate Dimensions

N ≥ d + 2 (3in) = 10 in **OK** Use N = 12 in.
B ≥ b + 2 (3in) = 10 in **OK** Use B = 12 in.

A_1= B x N = 144 in^2 ≥ $A_{1(req)}$ **OK**

Verify Concrete Bearing Strength

$P_p/\Omega c = \dfrac{.85f'_c A_1}{\Omega_c (A_2/A_1)^\wedge(1/2)}$ = 734 kips (J8-2)

$P_p/\Omega c$ > Pa **OK**

Verify Base PlateThickness

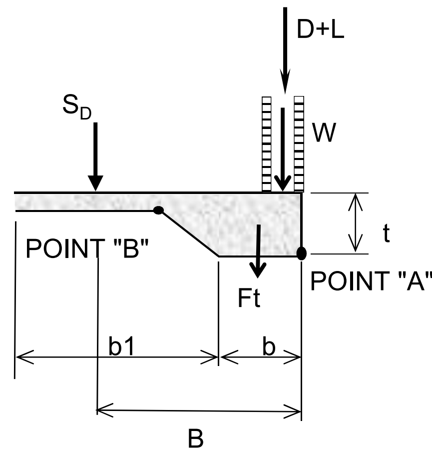
$m = \dfrac{N - (0.95)d}{2}$	=	4.10	in. (14-2)	$X = \dfrac{4bdPa\Omega c}{(b+d)^2 P_p}$	=	0.05			
$n = \dfrac{B - (0.80)b}{2}$	=	4.40	in. (14-3)	$\lambda = \dfrac{2\sqrt{X}}{1+\sqrt{1-X}}$	=	0.23	≤	1	**OK**
$n' = \dfrac{\sqrt{bd}}{4}$	=	1.00	in. (14-4)	$l = \max(m, n, \lambda n')$	=	4.4	in.		Critical Base
$t_{min} = l\sqrt{\dfrac{3.33Pa}{F_y BN}}$	=	0.430	in. (14-7)	t_p > t_{min}		**OK**			

Use Plate:	12	in.	x	12	in.	x	3/4	in.

MONOLITHIC FOOTING

FOOTING		WIDTH	DEPTH
MF	20	20	20

INPUT DATA & DESIGN SUMMARY			
Slab thick	t_S =	4.0	in
Footing width	b=	1.67	ft
Footing thick	t=	1.67	ft
Footing length	L=	1.0	ft
Concrete	f'_c =	3.0	ksi
Steel bars	f_y =	60.0	ksi



ALLOWABLE SOIL BEARING PRESSURE= **2500** lbs/ft^2

LOAD ANALYSIS:

Tributary Slab on ground S										167	lb/ft
Footing		Ft = b * t * 150 lb/ft^3							418	41.8	10%
CMU wall W		12.00	ft *	1	*	55	lb/ft^2 =		660		
Roof	Dead	28.00	ft *	0.5	*	25	lb/ft^2 =		350		
	Live	28.00	ft *	0.5	*	30	lb/ft^2 =		420		
									R =	1430	
								P tot =		1639	lb/ft

solid one way-slab in cantiliver:

$$b1 = \frac{t_S * 10}{12} = \quad 3.33 \text{ ft}$$

B = (b_1/2) + b = 3.34 ft

L= 1.0 ft Footing longitudinal dimension

Section Modulus of Footing:

$$S = \frac{L * b^2}{6} = \quad 0.4648 \text{ ft}^3$$

Applied moment @ point "A"

M= (S*B) + (Ft*(b/2)) + (W*(4/12)) + (D+L*4/12) = 1360 lbs*ft / ft

Xcg = M / Ptot = 0.83 ft Distance from point "A"

$$E = \left| \frac{b}{2} - X_{CG} \right| = \quad 0.01 \text{ ft}$$

b / 6 = 0.28 ft

Soil Pressure = 999 lbs/ft^2 Soil Pressure < Allowable soil pressure **O.K.**

TRANSFER BARS NOT REQUIRED - PROVIDE DOUBLE LAYER OF 6X6-1.4x1.4 WELDED WIRE MESH

SLAB ON GRADE DESIGN AS PER
ACI - STRUCTURAL PLAIN CONCRETE

F'c= 3000 psi

Moment due to slab weight @ point "B" M = 250 lbs*ft / ft

12 in

4.0 in

Section Modulus of slab:

$$S = \frac{bd^2}{6} = \quad 32 \ \text{in}^3$$

$$\frac{Mu}{S} \langle 5\,\phi\,\sqrt{f'c}$$

ϕ = 0.55

$$\frac{Mu}{S} = \quad 94 \ \text{psi} \quad < \quad 5\phi\sqrt{f'c} = \quad 151 \ \text{psi}$$

STEEL IS NOT REQUIRED

REINFORCING AT FOOTING	As
2 LAYERS OF 6X6-1.4x1.4 WELDED WIRE MESH	0.06 in^2/ft