



NODAL STRUCTURES, INC.

7725 SW 114 ST

By: heritage Homes

Rational Analysis and Engineering Design Calculations

**Miami-Dade County, Florida
2020 Florida Building Code**



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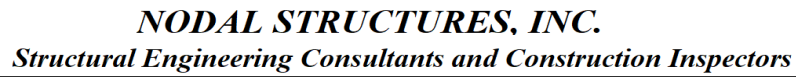
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NODAL STRUCTURES, INC.
STRUCTURAL ENGINEERS
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SUNRISE, FLORIDA 33351



<u>DEAD LOADS</u>	5.0	12.0	a =	22.6
Wood truss system				4.0 Lbs/ft ²
Superimposed loads				21.0
				<u>25.0 Lbs/ft²</u>

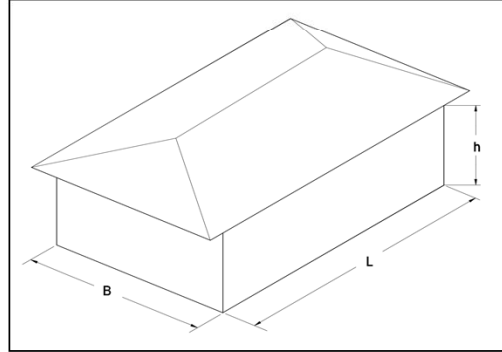
LIVE LOADS 20.0 Lbs/ft²

Dead loads	25.0 Lbs/ft ²	x	1.2	30.0
Live loads	<u>20.0</u>	x	1.6	<u>32.0</u>
	45.0 Lbs/ft ²			62.0 Lbs/ft²

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)

$$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2$$

$$P = q \times GC_p - q_h \times GC_{pi}$$

$$L/B = 1.27$$

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	



$P = q_h \times [GC_p - (GC_{pi})]$
 GC_p = External Pressure Coefficients
 GC_{pi} = Internal Pressure Coefficients

$GC_{pi} = -0.18 \quad \text{or} \quad 0.18$

| Internal Pressure, $q_h * GC_{pi} = -6.48 \text{ psf} \quad \text{or} \quad 6.48 \text{ psf}$

DESIGN WALL PRESSURES - MWFRS

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

L/B = 1.27

			Wall Pressures			
H	Wind Pressure		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	= q_h	-27.89	30.95	-21.77	-52.72
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

= q_h

DESIGN ROOF PRESSURES - MWFRS

SLOPE = 5 | 12 Angle = 22.62 degrees

h/L = 0.12

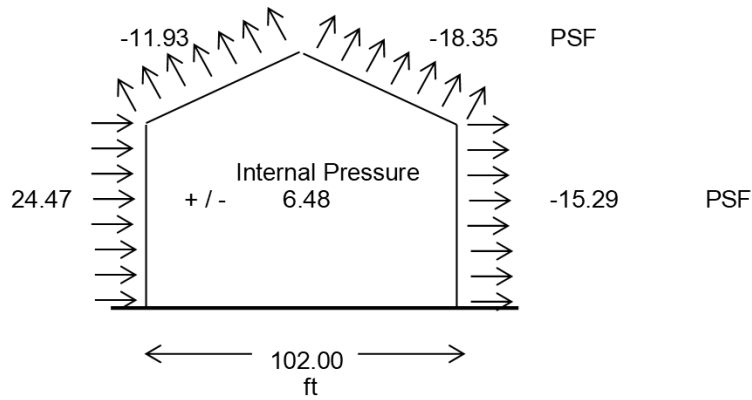
$q_h = 35.99 \text{ psf}$

Normal to ridge

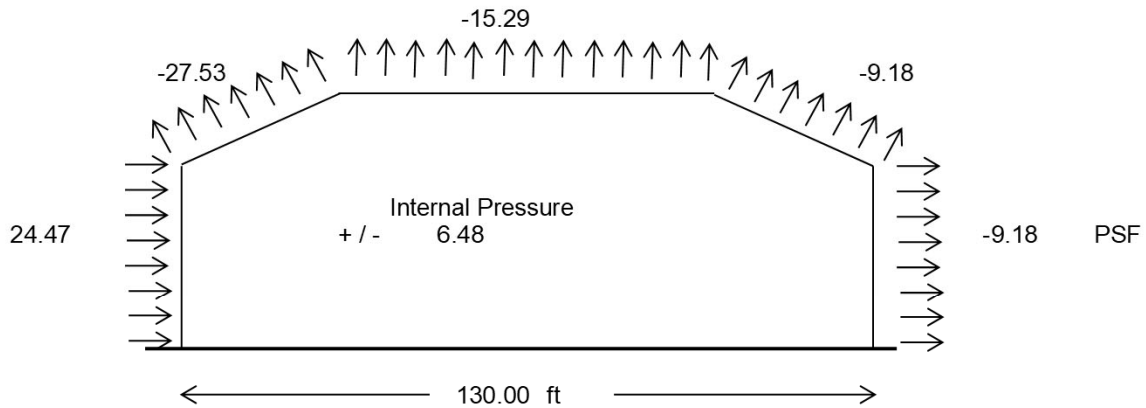
		Wind Pressure, P		
Wall	C_p	psf		
Windward	-0.39	-11.93	+ / -	6.48
Leeward	-0.60	-18.35	+ / -	6.48
Parallel to ridge		0 to h/2	-0.90	-27.53 + / - 6.48
		h to 2h	-0.50	-15.29 + / - 6.48
		> 2h	-0.30	-9.18 + / - 6.48

SUMMARY OF THE MWFRS WIND PRESSURES

TRANSVERSE WIND (Normal to ridge); PSF



LONGITUDINAL WIND (Parallel to ridge); PSF

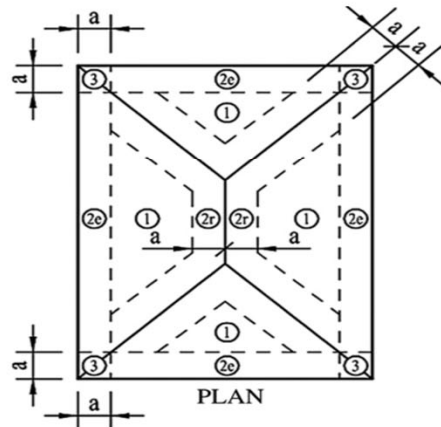




B.) FOR THE DESIGN OF COMPONENTS AND CLADDING

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 ft²

Zones		WIND PRESS	
		GCp	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

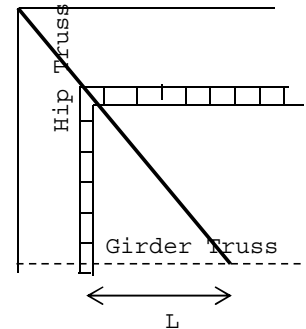
Tributary Area = 24.35 ft²

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

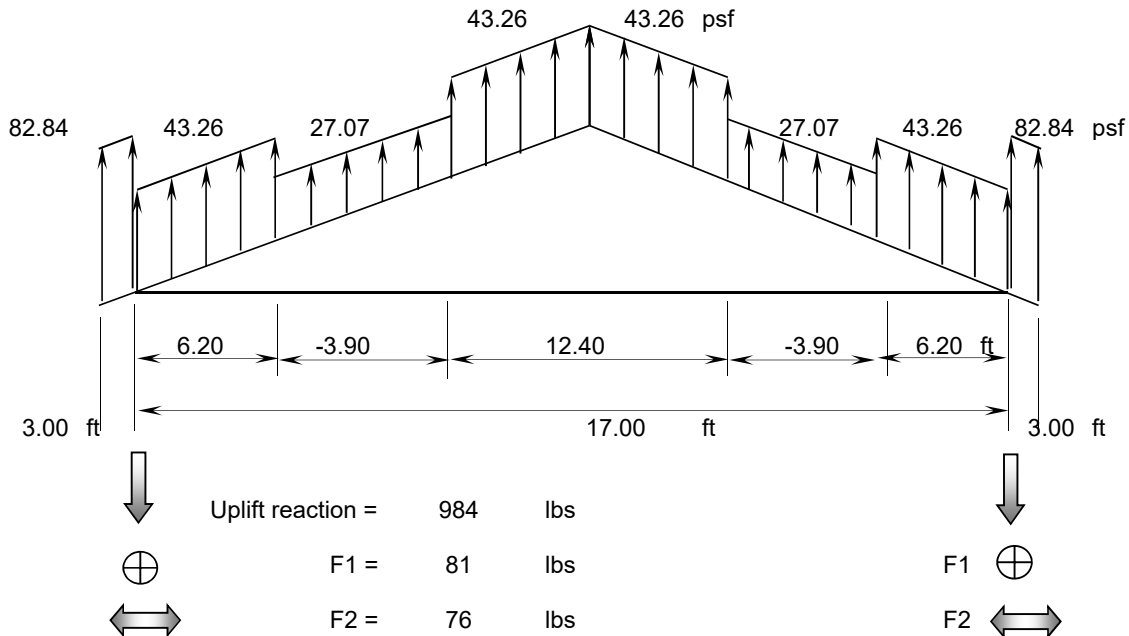
1870 Lbs > 951 O.K.



ROOF TRUSS DESIGN

Truss length = 17.00 ft
 Afferent Area = 34.00 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
 Poverhang = -82.84 psf

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



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 $\frac{984}{2430} + \frac{81}{1215} + \frac{76}{1310} = 0.53$

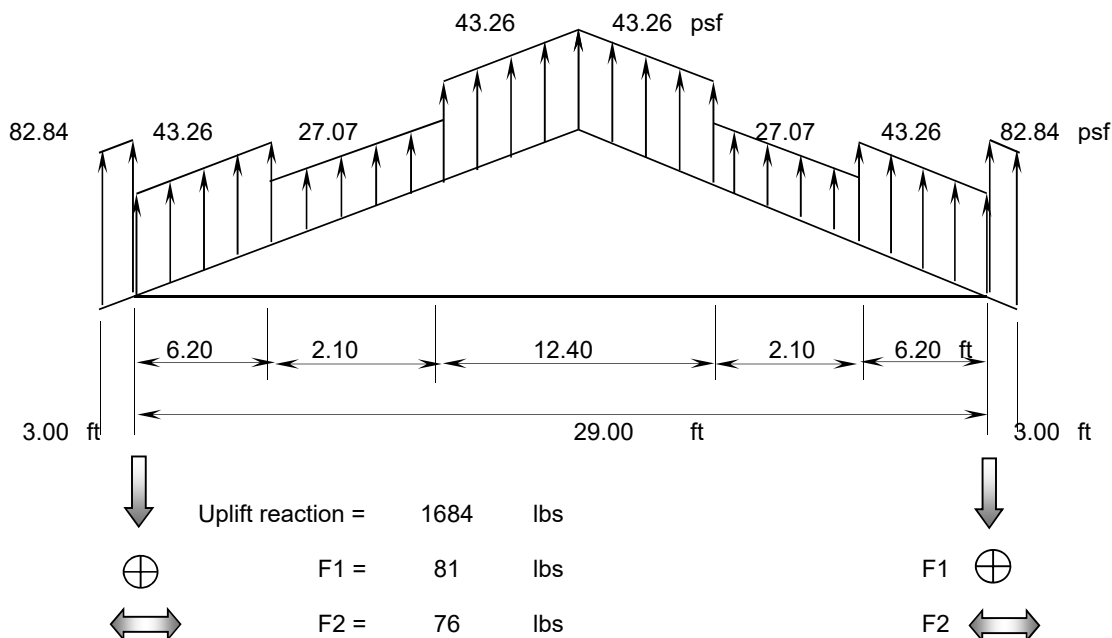
$\frac{\text{UPLIFT LOAD}}{\text{UPLIFT CAPACITY}} + \frac{\text{LATERAL LOAD}}{\text{LATERAL CAP.}} + \frac{\text{PERPEND. LOAD}}{\text{PERPEND. CAP.}}$



ROOF TRUSS DESIGN

Truss length = 29.00 ft
 Afferent Area = 58.00 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
 Poverhang = -82.84 psf

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



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 $\frac{1684}{2430} + \frac{81}{1215} + \frac{76}{1310} = 0.82$

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



ROOF GIRDER TRUSS DESIGN

Girder length = 17.00 ft
Afferent Area = 76.50 ft²
a = 6.20 ft

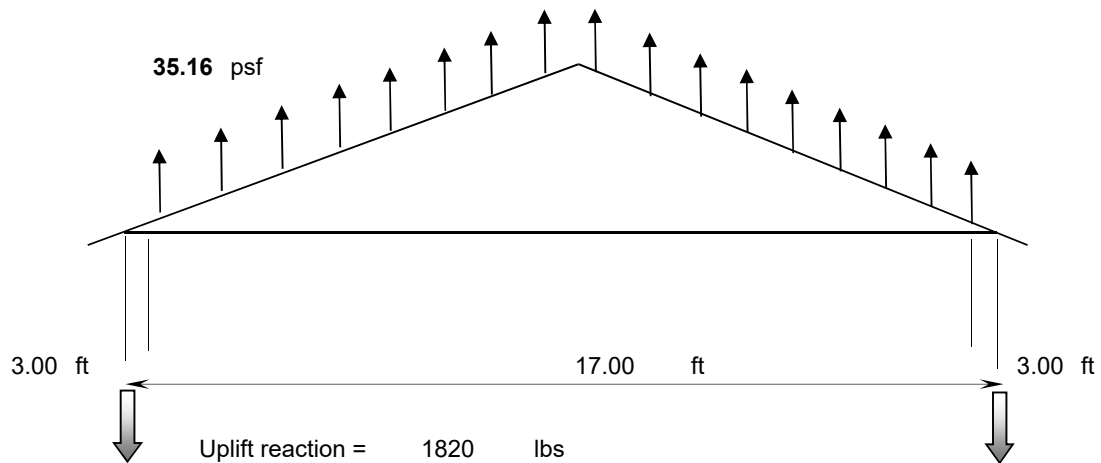
4.50 Girder afference

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

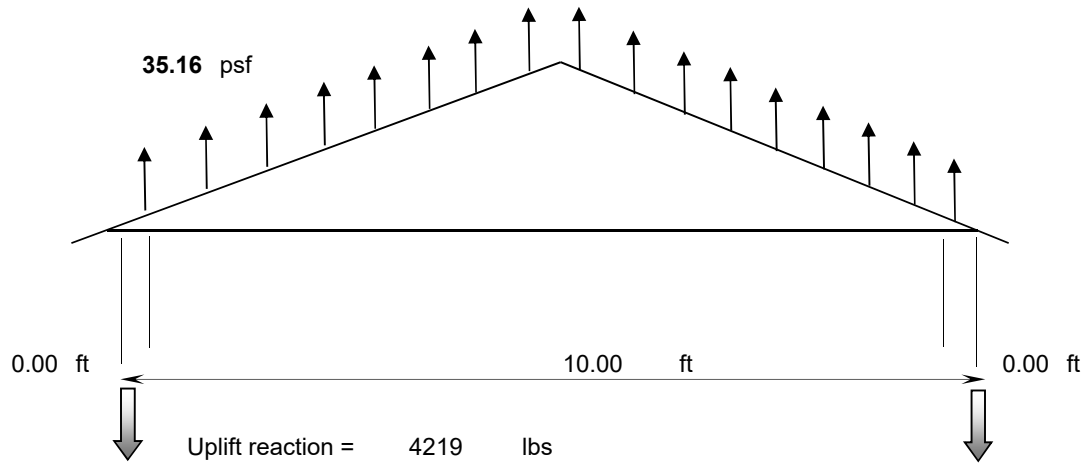




ROOF GIRDER TRUSS DESIGN

Girder length = 10.00 ft
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

24.00 Girder afference





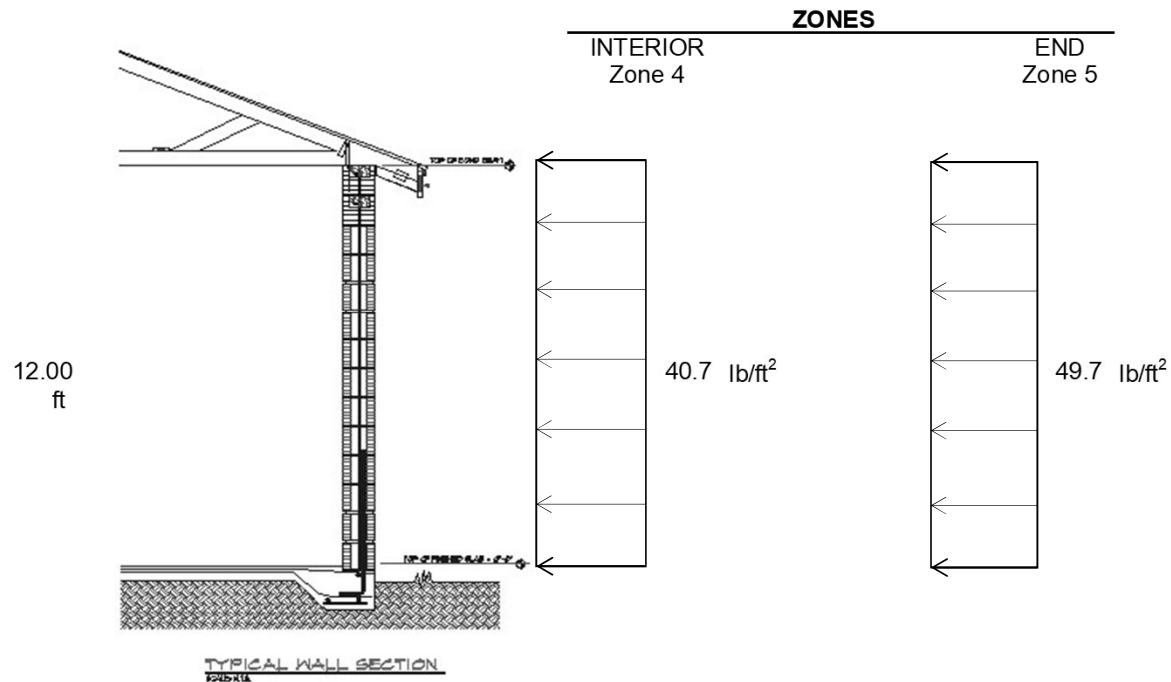
MASONRY WALLS LATERAL LOADS

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/ft²

Design pressures end zone 5 : 49.66 lb/ft²



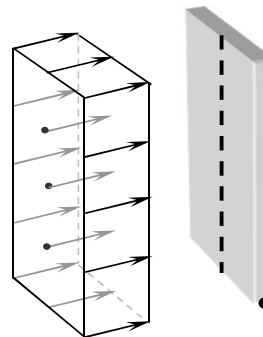
MASONRY WALL DESIGN

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 in²

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

Ms = As Fs j d

Ma < Ms **O.K.**

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



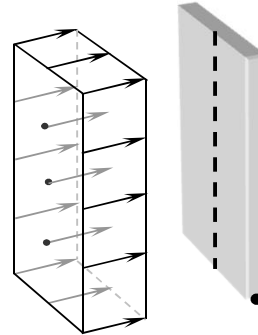
MASONRY WALL DESIGN

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 in²

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

Ms = As Fs j d

Ma < Ms **O.K.**

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

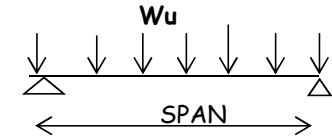


CONCRETE BEAMS DESIGN

INPUT DATA	
$f'_c =$	3
$f_y =$	60
$f_y =$	40
$\phi =$	0.90
$\phi =$	0.75
$\beta_1 =$	0.85
$\rho_{min} =$	0.0033
$d' =$	2.5

ksi
ksi steel bars
ksi ties
Axial tension & bending
Shear & Torsion

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	0.75ρ _{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
$f_y =$	60
$f_y =$	40
$\phi =$	0.90
$\phi =$	0.75
$\beta_1 =$	0.85
$\rho_{min} =$	0.0033
$d' =$	2.5

ksi
ksi steel bars
ksi ties
Axial tension & bending
Shear & Torsion

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	ϕV_c		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

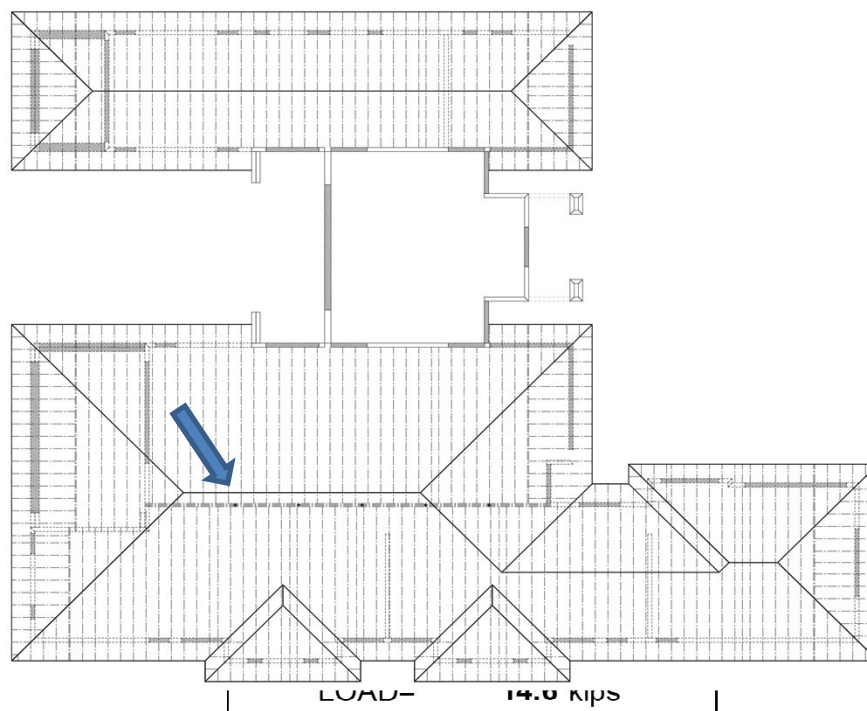


TABLE 4-4 SQUARE STRUCTURAL TUBING HSS 4" x 4" x 1/4"

$r_y =$ 46 ksi	$r_x =$ 1.52 in	$E =$ 29000 ksi
$F_u =$ 58 ksi	$r_y =$ 1.52 in	$\Omega_c =$ 1.67
$A_g =$ 3.37 in ²	$t_{design} =$ 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

$$P_n/\Omega_c = 50.8 \text{ kips} > P_a = 14.6 \text{ kips} \quad \text{OK}$$

CHECK FOR SLENDER ELEMENTS - TABLE B4.1 CASE 6

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

$$1.40(E/F_y)^{1/2} = 35.15$$

If, $b/t < 1.40(E/F_y)^{1/2}$
then, **NON-SLENDER ELEMENTS**



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

Determine Critical Stress F_{cr} :

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

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

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

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

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

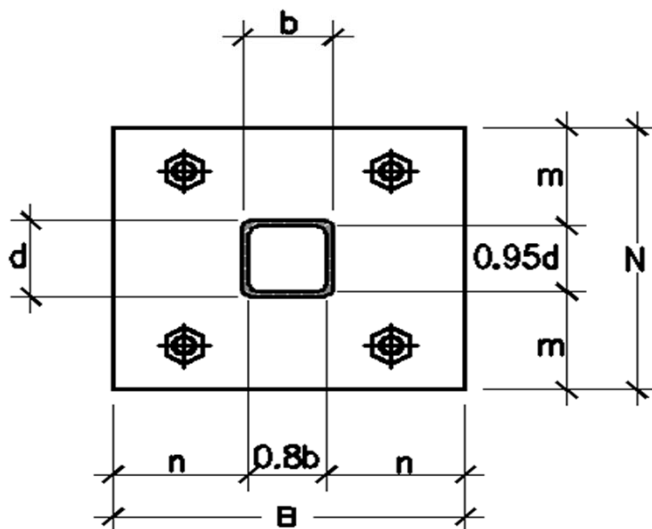
$$\text{Since } \frac{KL}{r_y} < 4.71\sqrt{E/F_y} \quad \text{then, Use Equation (E3-2) for } F_{cr}$$

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

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

$P_n/\Omega_c = 49.0 \text{ kips} > P_a = 14.6 \text{ kips} \quad \mathbf{OK}$
--

STEEL COLUMN BASE PLATE



Properties: ASD

$$\Omega_c = 2.50$$

Column

$$F_y = 46 \text{ ksi}$$

$$F_u = 58 \text{ ksi}$$

$$b = 4 \text{ in.}$$

$$d = 4 \text{ in.}$$

$$A_{col} = 16 \text{ in}^2$$

Base Plate

$$F_y = 36 \text{ ksi}$$

$$F_u = 58 \text{ ksi}$$

$$t_p = 0.75 \text{ in.}$$

$$N = 12 \text{ in.}$$

$$B = 12 \text{ in.}$$

Footing

$$\text{Dim.} = 60 \text{ in.} \times 60 \text{ in.}$$

$$f'_c = 3000 \text{ psi}$$

$$A_{ped} = 3600 \text{ in}^2$$

Total Load

$$P_a = 14.9 \text{ kips}$$

Required Base Plate Area

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

Verify Base Plate Dimensions

$$\begin{array}{llll} N \geq d + 2(3\text{in}') & = & 10 \text{ in} & \text{OK} \quad \text{Use } N = 12 \text{ in.} \\ B \geq b + 2(3\text{in}') & = & 10 \text{ in} & \text{OK} \quad \text{Use } B = 12 \text{ in.} \end{array}$$

$$A_1 = B \times N = 144 \text{ in}^2 \geq A_{1(req)} \quad \text{OK}$$



Verify Concrete Bearing Strength

$$P_p/\Omega_c = \frac{.85f_c A_1}{\Omega_c (A_2/A_1)^{1/2}} = 734 \text{ kips} \quad (\text{J8-2})$$

$$P_p/\Omega_c > P_a \quad \mathbf{OK}$$

Verify Base Plate Thickness

$$m = \frac{N - (0.95)d}{2} = 4.10 \text{ in.} \quad (14-2) \quad X = \frac{4bdPa\Omega_c}{(b+d)^2 P_p} = 0.05$$

$$n = \frac{B - (0.80)b}{2} = 4.40 \text{ in.} \quad (14-3) \quad \lambda = \frac{2\sqrt{X}}{1 + \sqrt{1-X}} = 0.23 \leq 1 \quad \mathbf{OK}$$

$$n' = \frac{\sqrt{bd}}{4} = 1.00 \text{ in.} \quad (14-4) \quad l = \max(m, n, \lambda n') = 4.4 \text{ in.} \quad \text{Critical Base}$$

$$t_{min} = l \sqrt{\frac{3.33Pa}{F_y B N}} = 0.430 \text{ in.} \quad (14-7) \quad t_p > t_{min} \quad \mathbf{OK}$$

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

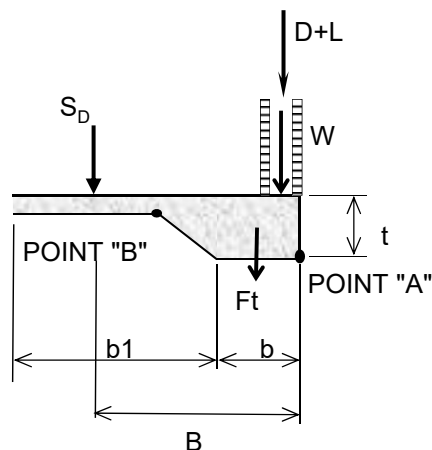


MONOLITHIC FOOTING

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/ft²**



LOAD ANALYSIS:

Tributary Slab on ground S

Footing $Ft = b * t * 150 \text{ lb/ft}^3$ 418 41.8 10% 167 lb/ft

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
							<u>R =</u>	1430

P tot = 1639 lb/ft

solid one way-slab in cantiliver:

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

$$B = (b1/2) + b = 3.34 \text{ ft}$$

L= 1.0 ft Footing longitudinal dimension

Section Modulus of Footing:

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

Applied moment @ point "A"

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

$$X_{cg} = M / P_{tot} = 0.83 \text{ ft} \quad \text{Distance from point "A"}$$

$$E = \left| \frac{b}{2} - X_{CG} \right| = 0.01 \text{ ft} \quad b/6 = 0.28 \text{ ft}$$

Soil Pressure = 999 lbs/ft² Soil Pressure < Allowable soil pressure **O.K.**

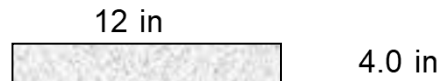
TRANSFER BARS NOT REQUIRED - PROVIDE DOUBLE LAYER OF 6X6-1.4x1.4 WELDED WIRE MESH



**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



Section Modulus of slab:

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

$$\frac{M_u}{S} < 5 \phi \sqrt{f'c}$$

$$\phi = 0.55$$

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

STEEL IS NOT REQUIRED

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