Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project GS JPD 10635 Residence				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 11/27/2024	Chk'd by Z.T.B	Date	App'd by	Date

STRUCTURAL CALCULATIONS
FOR

GS JPD 10635 RESIDENCE

10635 SW 63rd Ave,

Miami, FL 33156

November 27th, 2024

Zvonimir T. Belfranin, PE

FL PE# 33074

ZVONIMIR BELFRANIN
LICENSE
No 33074
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

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Table of Contents

Design Criteria

- Design Codes
  - Florida Building Code 2023, 8th Edition
  - ASCE 07-22
  - ACI 318-19, ACI 530-14, ACI 530.1-14 & ACI 421.1R-20
  - A.I.C.S. 16TH EDITION
  - TMS 402/602

- Materials
  - Concrete Strength at 28 days – 4000-5000 PSI (U.O.N)
  - Concrete Masonry Strength at 28 days – 2000 PSI (U.O.N)
  - Steel Tube A500 Grade B
  - Steel W Shape A992
  - All other Steel A36
  - Anchor Bolt A307
  - All other Bolt A325

- Gravity Design Load
  - Supper Imposed DL: 35 PSF (First & Second Floor)
  - Live load: 40 PSF (First & Second Floor)
  - Live load: 100 PSF (Stairs)
  - Live load: 60 PSF (Balconies)
  - Supper Imposed DL: 25 PSF (Roof Floor)
  - Live load: 30 PSF (Roof Floor)

- Wind Design Load
  - Wind Velocity 175 MPH
  - Building Exposure "C"
  - Risk Category " II"
  - Coefficient of Internal pressure ±0.18
  - Building Type "Enclosed"
  - Building Height 26.2'

MecaWind v2482

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Calculations Prepared by:
Date: Nov 27, 2024
Designer: Z.T.B

Calculations Prepared For:
Location: Miami, FL
Description: GS JPD 10635 Residence

File Location: C:\Users\User\ASM Eng Dropbox\2024-ASM\TOM\Hossein\MCDONOUGHT\Calc's\Mecawind\Hipped Wind-7-22.wnd

General:

Wind Load Standard	=	ASCE 7-22	Basic Wind Speed	=	175.0 mph
Exposure Classification	=	C	Risk Category	=	II
Structure Type	=	Building	Design Basis for Wind Pressures	=	ASD
MWFRS Analysis Method	=	Ch 27	C&C Analysis Method	=	Ch 30 Pt 1
Dynamic Type of Structure	=	Rigid	Show Advanced Options	=	True
Reset Advanced Options to Default Values	=	Defaults	Simple Diaphragm Building	=	False
Show Base Reactions in Output	=	0	MWFRS Pressure Elevations	=	Mean Ht
Topographic Effects	=	None	Override Directionality Factor K_d	=	False
Override the Gust Factor G	=	False	Override Minimum Pressure	=	False

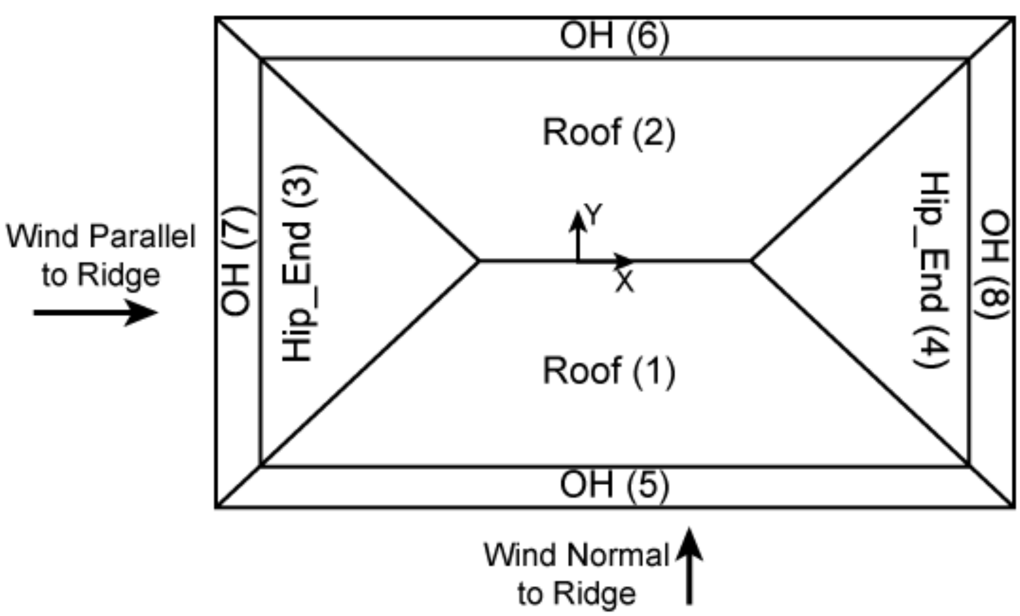
Building:

Roof	=	Roof Type	=	Hipped	Encl	=	Enclosure Classification	=	Enclosed
Help	=	Help on Building Roof Type	=	Help	Pitch	=	Pitch of Roof	=	7.0 :12
θ	=	Slope of Roof	=	30.26 Deg	R_{Ht}	=	Ridge Height	=	36.992 ft
E_{Ht}	=	Eave Height	=	26.200 ft	W	=	Building Width	=	53.500 ft
L	=	Building Length	=	87.000 ft	HipLen	=	Ridge Hipped Length	=	5.000 ft
EndSlope	=	Hipped End Slope of Roof	=	14.75 Deg	OH	=	Type of Overhang	=	None
Par	=	Parapet	=	None	HT_{over}	=	Override Mean Roof Height	=	False
Ht_{man}	=	Mean Roof Height	=	31.596 ft	RA_{over}	=	Override Roof Area	=	False
GC_{pi_o}	=	Override GC_{pi} value	=	False	IsElev	=	Building is Elevated	=	False

Exposure Constants [Tbl 26.11-1]:

α = 3-s Gust-speed exponent	=	9.800	Z_g = Nominal Ht of Boundary Layer	=	2460.000 ft
â = Reciprocal of α	=	0.102	b = 3 sec gust speed factor	=	1.000
α_m = Mean hourly Wind-Speed Exponent	=	0.156	b_m = Mean hourly Windspeed Exponent	=	0.660
c = Turbulence Intensity Factor	=	0.200	ε = Integral Length Scale Exponent	=	0.2000

Main Wind Force Resisting System (MWFRS) Wind Calculations per Ch 27



h	=	Mean structure height	=	31.596 ft
K_h	=	$2.41 \cdot (Z/Z_g)^{2/\alpha}$ [Tbl 26.10-1]	=	0.991
K_{zt}	=	Topographic: $(1+K_1 \cdot K_2 \cdot K_3)^{\wedge 2}$ [Eq 26.8-1]	=	1.000
K_d	=	Wind Directionality Factor per Tbl 26.6-1	=	0.85

$+GC_{pi}$	=	Enclosed Positive Internal Pressure Tbl 26.13-1	=	+0.18
$-GC_{pi}$	=	Enclosed Negative Internal Pressure Tbl 26.13-1	=	-0.18
LF	=	Load Factor based upon ASD Design	=	0.60
K_e	=	Ground Elev Factor [Tbl 26.10-1]	=	1.000
q_h	=	$0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 * LF$ [Eq 26.10-1]	=	46.61 psf
RA	=	Roof Area	=	4921.93 ft^2
q_h	=	$0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 * LF$ [Eq 26.10-1]	=	46.61 psf
q_{in}	=	Negative Internal Pressure: $q_h \cdot LF$	=	46.61 psf
q_{ip}	=	Positive Internal Pressure: $q_h \cdot LF$	=	46.61 psf

MWFRS Wind Loads [Normal to Ridge]

h	=	Mean Roof Height Of Building	=	31.596 ft
RHt	=	Ridge Height Of Roof	=	36.992 ft
B	=	Horizontal Dimension Of Building Normal To Wind Direction	=	87.000 ft
L	=	Horizontal Dimension Of building Parallel To Wind Direction	=	53.500 ft
L/B	=	Ratio Of L/B used For Cp determination	=	0.615
h/L	=	Ratio Of h/L used For Cp determination	=	0.591
Slope	=	Slope Of Main Roof	=	30.26 Deg
HipSlope	=	Slope Of Hipped Ends of Roof	=	14.75 Deg

Gust Factor Calculation for Wind: [Normal to Ridge]

Gust Factor Category I Rigid Structures - Simplified Method

G_1	=	Simplified: For Rigid Structures can use 0.85	=	0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Z_m	=	Equiv Struc Height: $Max(0.6 \cdot h, Z_{min})$	=	18.958 ft
I_{zm}	=	Turbulence Intensity: $c \cdot (33/Z_m)^{1/6}$ [Eq 26.11-7]	=	0.219
L_{zm}	=	Turbulence Integral Length Scale: $\ell \cdot (Z_m/33)^{\varepsilon}$ [Eq 26.11-9]	=	447.531 ft
B	=	Building Width Width Normal to Wind Direction	=	87.000 ft
Q	=	$[1/(1+0.63 \cdot [(B+h)/L_{zm}]^{0.63})]^{0.5}$ [Eq 26.11-8]	=	0.886
G_2	=	Detailed: $0.925 \cdot [(1+1.7 \cdot g_q \cdot I_{zm} \cdot Q)/(1+1.7 \cdot g_v \cdot I_{zm})]$ [Eq 26.11-6]	=	0.866

Gust Factor Used in Analysis

G	=	Gust Factor: $Min(G_1, G_2)$	=	0.850
Cp_{WW}	=	Windward Wall Coefficient (All L/B Values)	=	0.800
Cp_{LW}	=	Leeward Wall Coefficient using L/B	=	-0.500
Cp_{SW}	=	Side Wall Coefficient (All L/B values)	=	-0.700

Wind Pressures [Normal to Ridge]

All wind pressures include a Load Factor (LF) of 0.6

Elev ft	GC_{pi}	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
26.200	+0.18	46.61	0.954	1.000	44.86	18.80	-23.97	-30.71	42.77	9.60
26.200	-0.18	46.61	0.954	1.000	44.86	33.06	-9.71	-16.44	42.77	9.60

K_z	= $2.41 \cdot (Z/Z_g)^{2/\alpha}$		K_{zt}	= Topographic: $(1+K_1 \cdot K_2 \cdot K_3)^{2}$ [Eq 26.8-1]
GC_{pi}	= Enclosed Internal Pressure Tbl 26.13-1		q_z	= $0.00256 \cdot K_z \cdot K_{zt} \cdot K_e \cdot V^2 * LF$ [Eq 26.10-1]
q_{ip}	= Positive Internal Pressure: $q_h \cdot LF$		q_{in}	= Negative Internal Pressure: $q_h \cdot LF$
Side	= $q_h \cdot K_d \cdot G \cdot Cp_{SW} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eq 27.3-1		*Leeward*	= $q_h \cdot K_d \cdot G \cdot Cp_{LW} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eq 27.3-1
Windward	= $q_z \cdot K_d \cdot G \cdot Cp_{WW} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eq 27.3-1		*Total*	= Windward - Leeward

- *Minimum Pressure: § 27.1.5 no less than 9.60 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures [Normal to Ridge]

All wind pressures include a Load Factor (LF) of 0.6

Component	Description	Location	Start ft	End ft	GC_{pi}	C_{pMin}	C_{pMax}	P_{CpMin} psf	P_{CpMax} psf	P_{min} psf
Hip_End	Hipped End (0 to h/2)	3,4	0.000	15.798	+0.18	-0.925	-0.180	-38.30	-13.19	4.80
Hip_End	Hipped End (h/2 to h)	3,4	15.798	31.596	+0.18	-0.864	-0.180	-36.22	-13.19	4.80
Hip_End	Hipped End (h to 2*h)	3,4	31.596	53.500	+0.18	-0.536	-0.180	-25.19	-13.19	4.80
Roof_LW	Roof Leeward	2	All	All	+0.18	-0.600	-0.600	-27.34	-27.34	4.80
Roof_WW	Roof Windward	1	All	All	+0.18	0.204	-0.217	-0.25	-14.45	4.80
Hip_End	Hipped End (0 to h/2)	3,4	0.000	15.798	-0.18	-0.925	-0.180	-24.03	1.07	4.80
Hip_End	Hipped End (h/2 to h)	3,4	15.798	31.596	-0.18	-0.864	-0.180	-21.96	1.07	4.80
Hip_End	Hipped End (h to 2*h)	3,4	31.596	53.500	-0.18	-0.536	-0.180	-10.93	1.07	4.80
Roof_LW	Roof Leeward	2	All	All	-0.18	-0.600	-0.600	-13.07	-13.07	4.80
Roof_WW	Roof Windward	1	All	All	-0.18	0.204	-0.217	14.01	-0.18	4.80

Roof Pressures based upon Ch 27:

Component	= *The building component for pressures*	*Location*	= *Reference Graphic in Output for Values*
Start	= *Start Dist from Windward Edge*	*End*	= *End Dist from Windward Edge*
C_{pMin}	= *Smallest Coefficient Magnitude*	C_{pMax}	= *Largest Coefficient Magnitude*
P_{CpMin}	= $q_h \cdot K_d \cdot G \cdot C_{pMin} - q_{ip} \cdot K_d \cdot GC_{pi}$ *[Eq 27.3-1]*	P_{CpMax}	= $q_h \cdot K_d \cdot G \cdot C_{pMax} - q_{in} \cdot K_d \cdot GC_{pi}$ *[Eq 27.3-1]*

P_{min} = *Min Press projected on vertical plane [§ 27.1.5]*

- *0.800 Reduction Factor applied for h/L>=1 & 10<=Slope<=15 Deg*
- *The smaller uplift pressures due to C_{pMin} can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

MWFRS Wind Loads [Parallel to Ridge]

h	=	Mean Roof Height Of Building	=	31.596 ft
RHt	=	Ridge Height Of Roof	=	36.992 ft
B	=	Horizontal Dimension Of Building Normal To Wind Direction	=	53.500 ft
L	=	Horizontal Dimension Of building Parallel To Wind Direction	=	87.000 ft
L/B	=	Ratio Of L/B used For Cp determination	=	1.626
h/L	=	Ratio Of h/L used For Cp determination	=	0.363
Slope	=	Slope Of Main Roof	=	30.26 Deg
HipSlope	=	Slope Of Hipped Ends of Roof	=	14.75 Deg

Gust Factor Calculation for Wind: [Parallel to Ridge]

Gust Factor Category I Rigid Structures - Simplified Method

G_1	=	Simplified: For Rigid Structures can use 0.85	=	0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Z_m	=	Equiv Struc Height: Max$(0.6\cdot h,\ Z_{min})$	=	18.958 ft
I_{zm}	=	Turbulence Intensity: $c\cdot(33/Z_m)^{1/6}$ [Eq 26.11-7]	=	0.219
L_{zm}	=	Turbulence Integral Length Scale: $\ell\cdot(Z_m/33)^{\varepsilon}$ [Eq 26.11-9]	=	447.531 ft
B	=	Building Width Width Normal to Wind Direction	=	53.500 ft
Q	=	$[1/(1+0.63\cdot[(B+h)/L_{zm}]^{0.63})]^{0.5}$ [Eq 26.11-8]	=	0.905
G_2	=	Detailed: $0.925\cdot[(1+1.7\cdot g_q\cdot I_{zm}\cdot Q)/(1+1.7\cdot g_v\cdot I_{zm})]$ [Eq 26.11-6]	=	0.876

Gust Factor Used in Analysis

G	=	Gust Factor: Min$(G_1,\ G_2)$	=	0.850

Cp_{WW}	=	Windward Wall Coefficient (All L/B Values)	=	0.800
Cp_{LW}	=	Leeward Wall Coefficient using L/B	=	-0.375
Cp_{SW}	=	Side Wall Coefficient (All L/B values)	=	-0.700

Wind Pressures [Parallel to Ridge]
All wind pressures include a Load Factor (LF) of 0.6

Elev ft	GC_{pi}	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
26.200	+0.18	46.61	0.954	1.000	44.86	18.80	-19.75	-30.71	38.55	9.60
26.200	-0.18	46.61	0.954	1.000	44.86	33.06	-5.49	-16.44	38.55	9.60

K_z	= $2.41\cdot(Z/Z_g)^{2/\alpha}$	K_{zt}	= *Topographic:* $(1+K_1\cdot K_2\cdot K_3)^{2}$ *[Eq 26.8-1]*
GC_{pi}	= *Enclosed Internal Pressure Tbl 26.13-1*	q_z	= $0.00256\cdot K_z\cdot K_{zt}\cdot K_e\cdot V^2 * LF$ *[Eq 26.10-1]*
q_{ip}	= *Positive Internal Pressure:* $q_h\cdot LF$	q_{in}	= *Negative Internal Pressure:* $q_h\cdot LF$
Side	= $q_h\cdot K_d\cdot G\cdot Cp_{SW}-q_{ip}\cdot K_d\cdot(+GC_{pi})$ *Eq 27.3-1*	*Leeward*	= $q_h\cdot K_d\cdot G\cdot Cp_{LW}-q_{ip}\cdot K_d\cdot(+GC_{pi})$ *Eq 27.3-1*
Windward	= $q_z\cdot K_d\cdot G\cdot Cp_{WW}-q_{ip}\cdot K_d\cdot(+GC_{pi})$ *Eq 27.3-1*	*Total*	= *Windward - Leeward*

- *Minimum Pressure: § 27.1.5 no less than 9.60 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures [Parallel to Ridge]
All wind pressures include a Load Factor (LF) of 0.6

Component	Description	Location	Start ft	End ft	GC_{pi}	C_{pMin}	C_{pMax}	P_{CpMin} psf	P_{CpMax} psf	P_{min} psf
Hip_End	Hipped End (0 to h)	3	0.000	31.596	+0.18	-0.900	-0.180	-37.44	-13.19	4.80
Hip_End	Hipped End (h to 2*h)	3,4	31.596	63.192	+0.18	-0.500	-0.180	-23.97	-13.19	4.80
Hip_End	Hipped End (>= 2*h)	4	63.192	87.000	+0.18	-0.300	-0.180	-17.24	-13.19	4.80
Roof	Roof (0 to h)	5,6	0.000	31.596	+0.18	-0.900	-0.180	-37.44	-13.19	4.80
Roof	Roof (h to 2*h)	5,6	31.596	63.192	+0.18	-0.500	-0.180	-23.97	-13.19	4.80
Roof	Roof (>= 2*h)	5,6	63.192	87.000	+0.18	-0.300	-0.180	-17.24	-13.19	4.80
Hip_End	Hipped End (0 to h)	3	0.000	31.596	-0.18	-0.900	-0.180	-23.18	1.07	4.80
Hip_End	Hipped End (h to 2*h)	3,4	31.596	63.192	-0.18	-0.500	-0.180	-9.71	1.07	4.80
Hip_End	Hipped End (>= 2*h)	4	63.192	87.000	-0.18	-0.300	-0.180	-2.97	1.07	4.80
Roof	Roof (0 to h)	5,6	0.000	31.596	-0.18	-0.900	-0.180	-23.18	1.07	4.80
Roof	Roof (h to 2*h)	5,6	31.596	63.192	-0.18	-0.500	-0.180	-9.71	1.07	4.80
Roof	Roof (>= 2*h)	5,6	63.192	87.000	-0.18	-0.300	-0.180	-2.97	1.07	4.80

Roof Pressures based upon Ch 27:

Component = The building component for pressures | *Location = Reference Graphic in Output for Values*

Start = Start Dist from Windward Edge | *End = End Dist from Windward Edge*

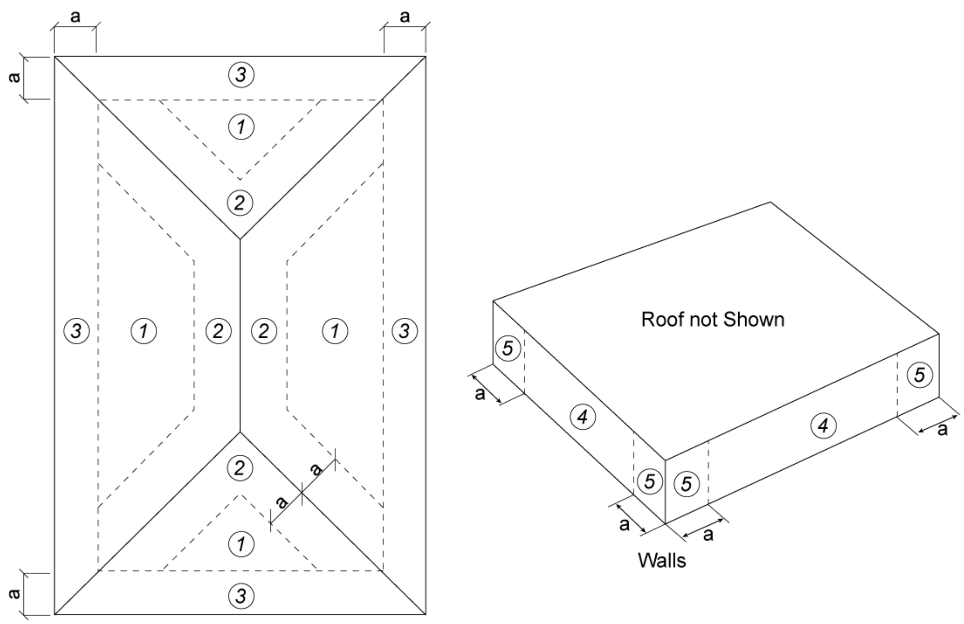
C_{pMin} = Smallest Coefficient Magnitude | *C_{pMax} = Largest Coefficient Magnitude*

P_{CpMin} = $q_h \cdot K_d \cdot G \cdot C_{pMin} - q_{ip} \cdot K_d \cdot GC_{pi}$ [Eq 27.3-1] | *P_{CpMax} = $q_h \cdot K_d \cdot G \cdot C_{pMax} - q_{in} \cdot K_d \cdot GC_{pi}$ [Eq 27.3-1]*

P_{min} = Min Press projected on vertical plane [§ 27.1.5]

- *No reduction factor was applicable*
- *The smaller uplift pressures due to C_{pMin} can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Components and Cladding (C&C) Wind Loads per Ch 30 Part 1 Roof & Wall



h/W	=	Ratio of mean roof height to building width	=	0.591
h/L	=	Ratio of mean roof height to building length	=	0.363
h	=	Mean structure height	=	31.596 ft
K_h	=	$2.41 \cdot (Z/Z_g)^{2/\alpha}$	=	0.991
K_{zt}	=	Topographic: $(1+K_1 \cdot K_2 \cdot K_3)^{\wedge 2}$ [Eq 26.8-1]	=	1.000
K_d	=	Wind Directionality Factor per Tbl 26.6-1	=	0.85
$+GC_{pi}$	=	Enclosed Positive Internal Pressure Tbl 26.13-1	=	+0.18
$-GC_{pi}$	=	Enclosed Negative Internal Pressure Tbl 26.13-1	=	-0.18
LF	=	Load Factor based upon ASD Design	=	0.60
K_e	=	Ground Elev Factor [Tbl 26.10-1]	=	1.000
q_h	=	$0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 * LF$ [Eq 26.10-1]	=	46.61 psf
LHD	=	Least Horizontal Dimension: Min(B, L)	=	53.500 ft
a_1	=	Min(0.1•LHD, 0.4•h)	=	5.350 ft
a	=	Max(a_1, 0.04•LHD, 3 ft [0.9 m])	=	5.350 ft
h/B	=	Ratio of mean roof height to least horizontal dim: h/B	=	0.591

Wind Pressures for C&C Ch 30 Pt 1 Roof & Wall
All wind pressures include a Load Factor (LF) of 0.6

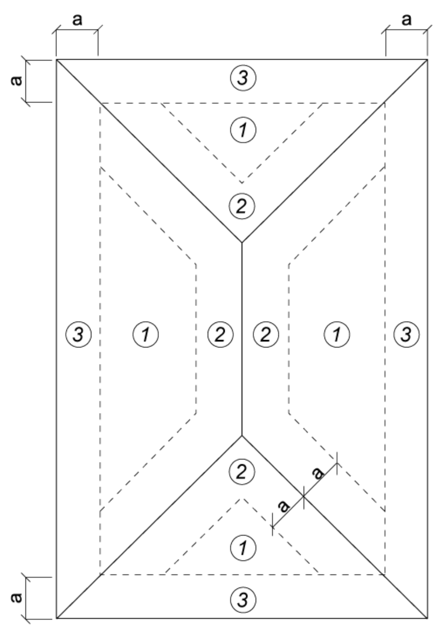
Description	Zone	Width ft	Span ft	Area ft²	1/3 Rule	Figure	GCp Max	GCp Min	p Max psf	p Min psf
Zone 1- 10 SF	1	2.000	5.000	10.00	No	30.3-2G	0.700	-1.418	34.87	-63.32
Zone 2e- 10 SF	2	2.000	5.000	10.00	No	30.3-2G	0.700	-1.964	34.87	-84.94
Zone 2r- 10 SF	2	2.000	5.000	10.00	No	30.3-2G	0.700	-1.964	34.87	-84.94
Zone 3- 10 SF	3	2.000	5.000	10.00	No	30.3-2G	0.700	-2.072	34.87	-89.24
Zone 4- 10 SF	4	2.000	5.000	10.00	No	30.3-1	1.000	-1.100	46.75	-50.71
Zone 5- 10 SF	5	2.000	5.000	10.00	No	30.3-1	1.000	-1.400	46.75	-62.60

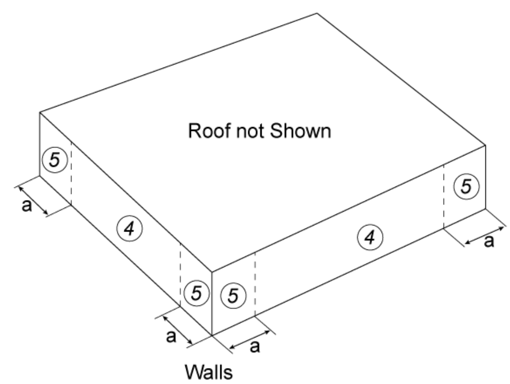
Area = Span Length x Effective Width
1/3 Rule = Effective width need not be less than 1/3 of the span length
GCp = External Pressure Coefficients taken from Figures 30.3-1 through 30.3-7
p = Wind Pressure: $q_h \cdot K_d \cdot [GC_p - GC_{pi}]$ [Eq 30.3-1]

** Per § 30.2.2 the Minimum Pressure for C&C is 9.60 psf [0.460 kPa] {Includes LF}*

Components and Cladding (C&C) Wind Roof & Wall Summary per Ch 30 Pt 1:

h/W	=	Ratio of mean roof height to building width	=	0.591
h/L	=	Ratio of mean roof height to building length	=	0.363
h	=	Mean structure height	=	31.596 ft
K_h	=	$2.41 \bullet (Z/Z_g)^{2/\alpha}$	=	0.991
K_{zt}	=	Topographic: $(1+K_1 \bullet K_2 \bullet K_3)^{\wedge 2}$ [Eq 26.8-1]	=	1.000
K_d	=	Wind Directionality Factor per Tbl 26.6-1	=	0.85
$+GC_{pi}$	=	Enclosed Positive Internal Pressure Tbl 26.13-1	=	+0.18
$-GC_{pi}$	=	Enclosed Negative Internal Pressure Tbl 26.13-1	=	-0.18
LF	=	Load Factor based upon ASD Design	=	0.60
K_e	=	Ground Elev Factor [Tbl 26.10-1]	=	1.000
q_h	=	$0.00256 \bullet K_h \bullet K_{zt} \bullet K_e \bullet V^2 * LF$ [Eq 26.10-1]	=	46.61 psf
LHD	=	Least Horizontal Dimension: Min(B, L)	=	53.500 ft
a_1	=	Min(0.1•LHD, 0.4•h)	=	5.350 ft
a	=	Max(a_1, 0.04•LHD, 3 ft [0.9 m])	=	5.350 ft
h/B	=	Ratio of mean roof height to least horizontal dim: h/B	=	0.591





Wind Pressure Summary for C&C Roof & Wall based Upon Areas Ch 30 Pt 1 (Table 1 of 2)
All wind pressures include a Load Factor (LF) of 0.6

Zone	Figure	Pos A ≤ 10 ft² psf	Neg A ≤ 10 ft² psf	Pos A = 20 ft² psf	Neg A = 20 ft² psf	Pos A = 50 ft² psf	Neg A = 50 ft² psf
1	30.3-2G	34.87	-63.32	30.10	-55.73	23.79	-45.70
2	30.3-2G	34.87	-84.94	30.10	-73.01	23.79	-57.25
3	30.3-2G	34.87	-89.24	30.10	-76.45	23.79	-59.54
4	30.3-1	46.75	-50.71	44.65	-48.61	41.86	-45.82
5	30.3-1	46.75	-62.60	44.65	-58.39	41.86	-52.82

Wind Pressure Summary for C&C Roof & Wall based Upon Areas Ch 30 Pt 1 (Table 2 of 2)
All wind pressures include a Load Factor (LF) of 0.6

Zone	Figure	Pos A = 100 ft² psf	Neg A = 100 ft² psf	Pos A = 200 ft² psf	Neg A = 200 ft² psf	Pos A > 500 ft² psf	Neg A > 500 ft² psf
1	30.3-2G	19.02	-38.11	19.02	-38.11	19.02	-38.11
2	30.3-2G	19.02	-45.32	19.02	-45.32	19.02	-45.32
3	30.3-2G	19.02	-46.75	19.02	-46.75	19.02	-46.75
4	30.3-1	39.76	-43.72	37.65	-41.61	34.87	-38.83
5	30.3-1	39.76	-48.61	37.65	-44.40	34.87	-38.83

* *A is effective wind area for C&C: Span Length * Effective Width*
* *Effective width need not be less than 1/3 of the span length*
* *Maximum and minimum values of pressure shown.*
* *+ Pressures acting toward surface, - Pressures acting away from surface*
* *Per § 30.2.2 the Minimum Pressure for C&C is 9.60 psf [0.460 kPa] {Includes LF}*
* *Interpolation can be used for values of A that are between those values shown.*

- ## <u>CMU Wall Design</u>

<u>Height</u> of CMU wall is <u>14.5'(First Floor Height)-2'(Beam Depth)=12.5'.</u>

Based on Mecawind calculation, we have:

These loads are based on area <u>A=50 sf.</u>

$P_w = 52.82\ psf(zone\ 5)$

$$F_{Ultimate} = \frac{52.82}{0.6} = 88.0834\ psf \approx 88\,psf$$

We don't consider gravity load (safety side).

<u>Use #5 @ 24" O.C for zone 4 & 5</u>

Please see next page for calculation.

MASONRY WALL PANEL DESIGN (TMS 402/602-16)

In accordance with strength design method

Tedds calculation version 2.2.11

Masonry wall panel details

Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length	$L = \mathbf{20}$ ft

Panel height	$h = \mathbf{12.5}$ ft

Seismic properties

Seismic design category	A

Seismic importance factor (ASCE7 Table 1.5-2)	$I_e = \mathbf{1}$

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

$S_{DS} = \mathbf{1}$

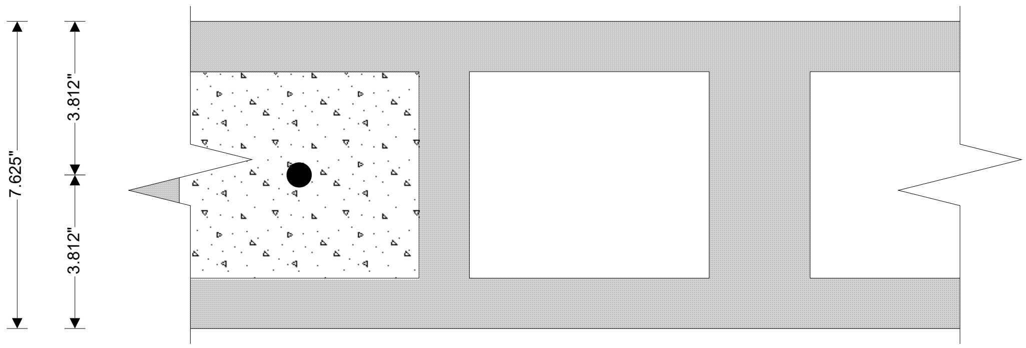
Seismic wall classification	Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load	$\rho_E = \mathbf{1.0}$

Construction details

Wall thickness	$t = \mathbf{7.625}$ in



Masonry details

Hollow concrete units grouted at 24 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **1900** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Length of masonry units l_b = **15.625** in

Number of internal webs N_{web} = **1**

Number of end webs N_{end} = **2**

Internal web thickness t_{bw} = **1.25** in

Face shell thickness t_{bf} = **1.25** in

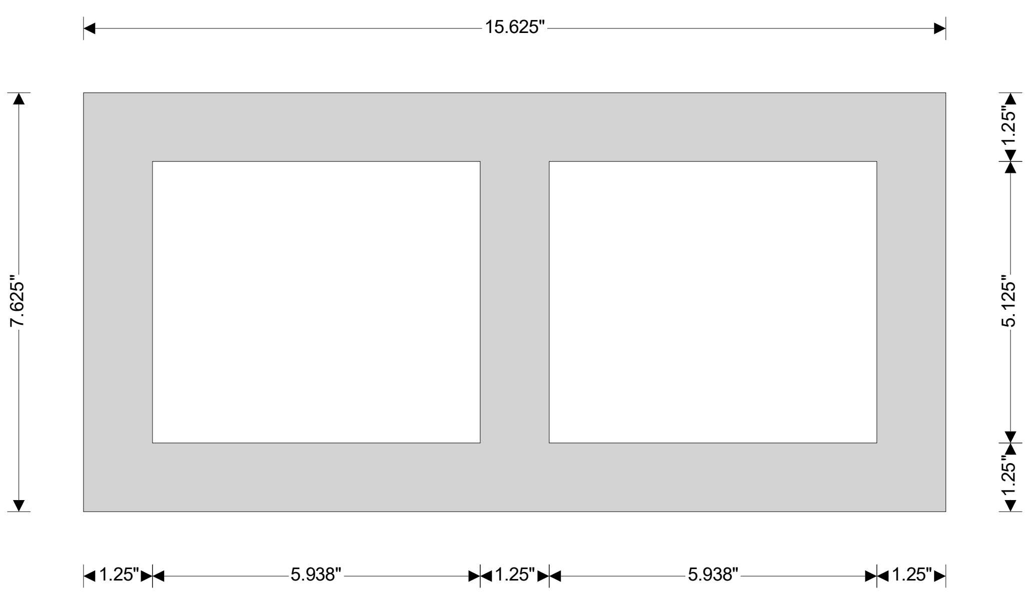
End web thickness t_{be} = **1.25** in

Area of block $A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft

Area of grout $A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **15.42** in^2/ft

Density of grout γ_{grout} = **140** lb/ft^3

Self weight of wall $w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **50.74** psf



From TMS 602-16 Table 2 - Compressive strength of masonry

Net compressive strength of masonry f'_m = **1900** psi

Modulus of elasticity for masonry $E_m = 900 \times f'_m$ = **1710000** psi

Shear modulus of masonry $G_v = 0.4 \times E_m$ = **684000** psi

From TMS 402 -16 Table 9.1.9.2 - Modulus of rupture

Modulus of rupture normal to bed f_{r_norm} = **110** psi

Modulus of rupture parallel to bed f_{r_para} = **167** psi

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Allowable tensile stress in reinforcement F_s = **32000** psi

Modulus of elasticity for reinforcement E_s = **29000000** psi

Vertical reinforcement provided No.5 bars at 24 in centers

Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s)$ = **0.15** in^2/ft

Lateral out-of-plane loads

Wind load on panel W = **88** psf

Wind load on parapet W_p = **18** psf

Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**

Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **20.3** psf

Additional seimic load E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add}$ = **20.3** psf

Lateral in-plane loads

Vertical loading details

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.2**

From ASCE 7-16 cl.2.3 - Combining factored loads using strength design (Utilization)

Load combination no.1 $1.4 \times DL$ (0.017)

Load combination no.4 $1.2 \times DL + 1.6 \times (LL_r$ or SL or RL$) + 0.5 \times W$ (0.336)

Load combination no.5 $1.2 \times DL + W + LL + 0.5 \times (LL_r$ or SL or RL$)$ (0.678)

Load combination no.7 $0.9 \times DL + W$ (0.684)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **60.2** in^2/ft

Properties for walls loaded out-of-plane:

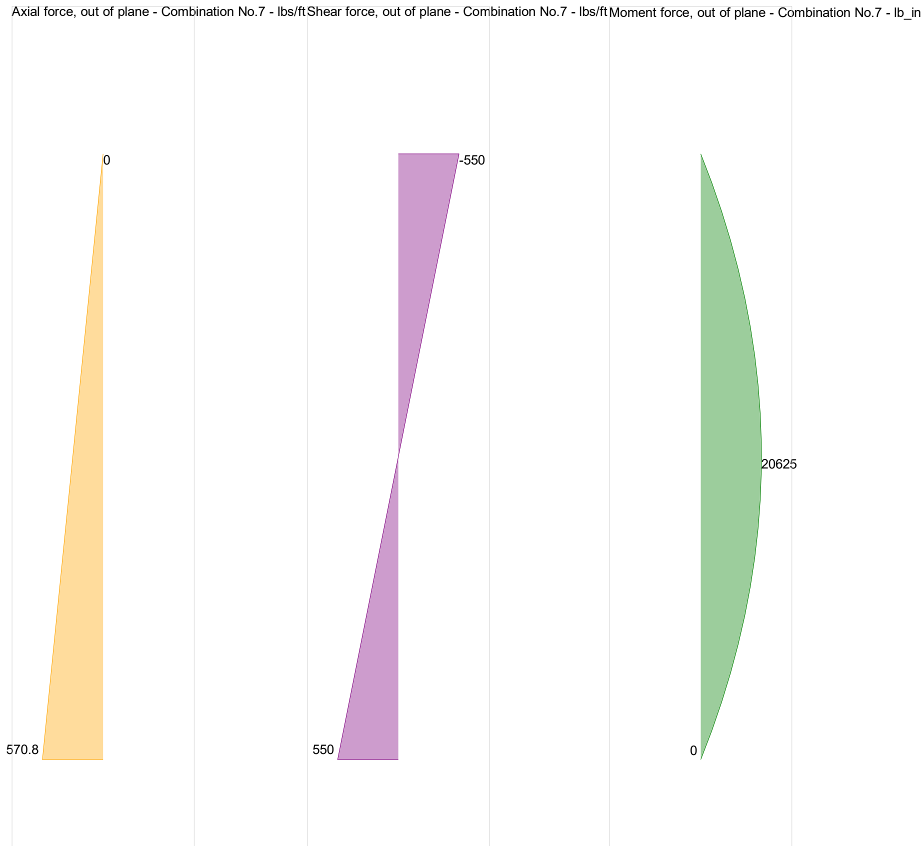
Moment of inertia $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **374.8** in^4/ft

Section modulus $S = I / c$ = **98.3** in^3/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.495** in

Effective height factor K = **1**

Consider wall at maximum moment location under load combination no.7



Maximum moment location	6.25 ft
Axial load at mid-height of panel	P = **285** lb/ft
Slenderness ratio	$(K \times h) / r$ = **60.110** < 99
Nominal axial strength	$P_n = 0.8 \times (0.8 \times f'_m \times (A - A_s) + A_s \times 0 \text{ ksi}) \times [1 - ((K \times h) / (140 \times r))^2]$ = **59541** lb/ft
Strength reduction factor	ϕ = **0.9**
Design axial strength	$\phi \times P_n$ = **53587** lb/ft
	$P / (\phi \times P_n)$ = **0.005**

PASS - Nominal axial strength exceeds axial load

Factored axial stress	P / t = **3** psi
Factored axial stress limit	$0.2 \times f'_m$ = **380** psi

PASS - Allowable stress under out of plane loads exceeds factored axial stress

Nominal cracking moment strength	$M_{cr} = S \times f_{r\_norm}$ = **10846** lb_in/ft
Modular ratio	$n = E_s / E_m$ = **16.959**
Distance to neutral axis	$c_{cr} = (A_s \times f_y + P) / (0.64 \times f'_m)$ = **0.65** in
Moment of inertia of cracked section	$I_{cr} = n \times (A_s + P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3$ = **27.9** in^4/ft
By iteration	$M_{u0} = M$ = **20625** lb_in/ft
	$\delta_{u0} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I) + 5 \times (M_{u0} - M_{cr}) \times h^2 / (48 \times E_m \times I_{cr})$ = **0.520** in
	$M_{u1} = M_{u0} + P \times \delta_{u0}$ = **20773** lb_in/ft

$\delta_{u1} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I) + 5 \times (M_{u1} - M_{cr}) \times h^2 / (48 \times E_m \times I_{cr})$ = **0.527** in

Bending moment at mid-height of panel — $M = M_{u0} + P \times \delta_{u1}$ = **20775** lb_in/ft

Depth of reinforcement — d = **3.813** in

Effective width per bar — $b_{eff} = \min(s, 6 \times t_{nom}, 72 \text{ in})$ = **24** in

Strength reduction factor — ϕ = **0.9**

Tensile strain in reinforcement — $\varepsilon_s = f_y / E_s$ = **0.0021**

Maximum usable compressive strain of masonry — ε_{mu} = **0.0025**

Fiber of max.compressive strain to neutral axis — $c_{bal} = \varepsilon_{mu} \times d / (\varepsilon_{mu} + \varepsilon_s)$ = **2.086** in

Tensile force at balance point — $T_{bal} = A_s \times f_y$ = **9204** lb/ft

β_1 = **0.8**

Compressive force at balance point — $C_{bal} = 0.8 \times f'_m \times \beta_1 \times c_{bal}$ = **30440** lb/ft

Design axial force at balance point — $P_{bal} = \phi \times (C_{bal} - T_{bal})$ = **19113** lb/ft

Design moment at balance point — $M_{bal} = \phi \times [T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - \beta_1 \times c_{bal} / 2)]$ = **81588** lb_in/ft

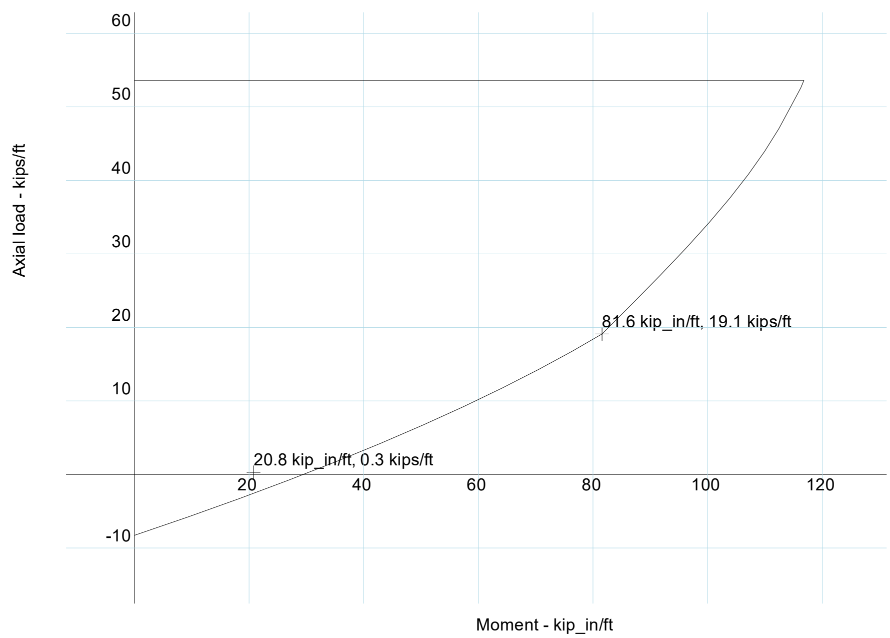
Maximum design moment from interaction diagram — M_c = **30386** lb_in/ft

M / M_c = **0.684**

PASS - Combination of applied axial load and flexure is acceptable

Maximum area of tensile reinforcement (9.3.3.2) — $A_{s\_max} = 0.64 \times f'_m \times [\varepsilon_{mu} / (\varepsilon_{mu} + 1.5 \times \varepsilon_s)] \times d / f_y$ = **0.414** in^2/ft

PASS - Area of reinforcement provided is less than maximum allowable



Strength interaction diagram

Consider wall at top under load combination no.7

Shear force — V = **550** lb/ft

Compressive force — N_u = **0** lb/ft

Net shear area — $A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout})$ = **14.893** in^2/ft

Nominal shear strength $V_n = \min([4 - 1.75 \times \min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_m \times 1\text{ psi})} + 0.25 \times N_u, 6 \times A_{nv} \times \sqrt{(f'_m \times 1\text{ psi})})$ = **2597** lb/ft

Strength reduction factor ϕ_v = **0.8**

Design shear strength $\phi_v \times V_n$ = **2077** lb/ft

$V / (\phi_v \times V_n)$ = **0.265**

PASS - Design shear strength exceeds applied shear strength

- Concrete Beam Design (2B-15):

$$W_{DL} = \frac{150lb}{ft3} \times \frac{9"}{12} \times 12.0'(9"\ conc.slab\ at\ 2nd\ floor) + 35psf\ \times 12.0' = 1770\ plf \approx 1.8\ k/ft$$

$$W_{LL} = 40psf \times 12.0' = 480\ plf = 0.48\ k/ft$$

Use 8"x26" with 2#7 Top, 2#8 Bottom, 1#5 E.F and #3 ties @ 6" O.C.

Please see next page for calculation.

RC BEAM ANALYSIS & DESIGN (ACI318-2019)

In accordance with ACI318-2019

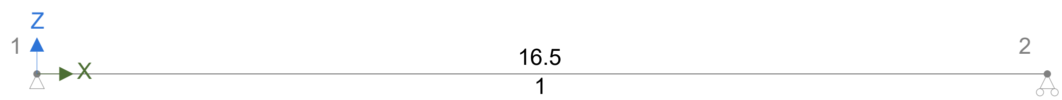
Tedds calculation version 3.3.16

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Concrete (5000 150) - R 8x26

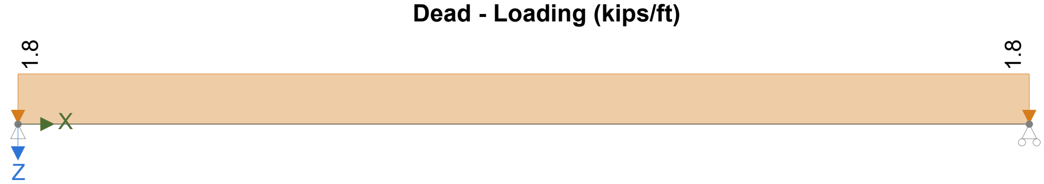


Span	Length (ft)	Section	Start Support	End Support
1	16.5	R 8x26	Pinned	Roller Pin X
R 8x26: Area 208 in^2, Inertia Major 11717 in^4, Inertia Minor 1109 in^4, Shear area parallel to Minor 173 in^2, Shear area parallel to Major 173 in^2				
Concrete (5000 150): Density 150 lbm/ft^3, Youngs 4287 ksi, Shear 1750 ksi, Thermal 0.00001 °C^{-1}				

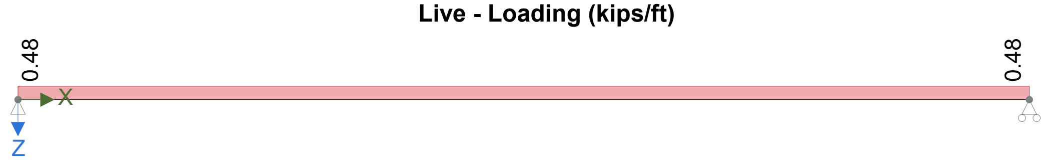
Loading

Self weight included

Dead - Loading (kips/ft)



Live - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	1.8 kips/ft
Beam	Live	UDL	GlobalZ	0.48 kips/ft

Results

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	1.788	0
2	0	1.788	0

Load case: Dead

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	14.85	0
2	0	14.85	0

Load case: Live

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	3.96	0
2	0	3.96	0

Load combination: 1.2D + 1.6L (Strength)

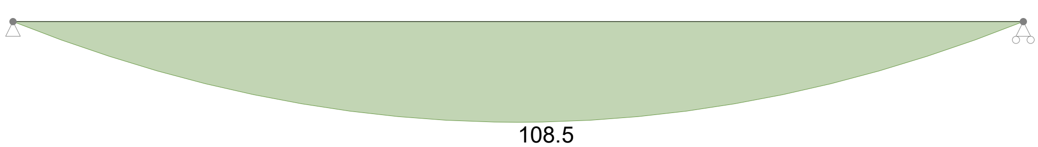
Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	26.301	0
2	0	26.301	0

Load combination: 1.0D + 1.0L (Service)

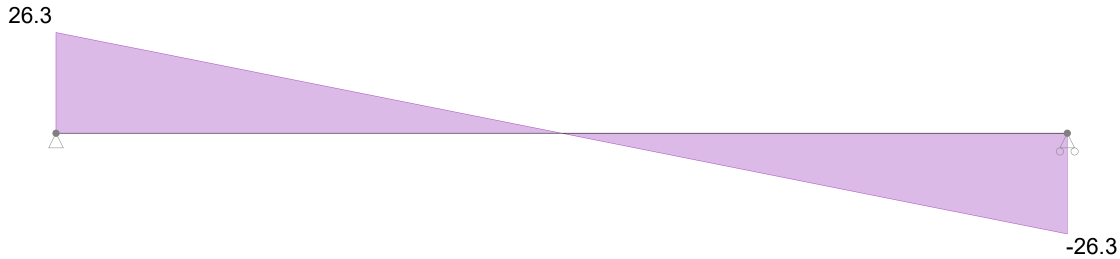
Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	20.598	0
2	0	20.598	0

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Concrete details

Compressive strength of concrete f'_c = **5000** psi

Density of reinforced concrete w_c = **150** lb / ft^3

Concrete type Normal weight

Modulus of elasticity of concrete (cl.19.2.2.1) $E = (w_c / 1\ lb/ft^3)^{1.5} \times 33\ psi \times (f'_c / 1\ psi)^{0.5}$ = **4286826** psi

Strength reduction factor for shear ϕ_s = **0.75**

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Compression-controlled strain limit (cl.21.2.2.1) ε_{ty} = **0.00200**

Nominal cover to reinforcement

Cover to top reinforcement c_{nom_t} = **1.5** in

Cover to bottom reinforcement c_{nom_b} = **1.5** in

Cover to side reinforcement c_{nom_s} = **1.5** in

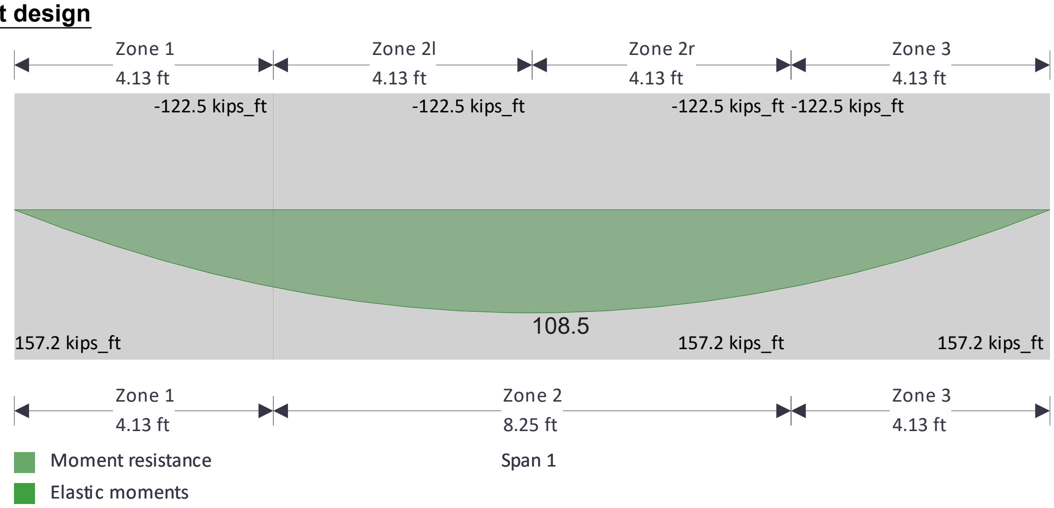
Beam - Span 1

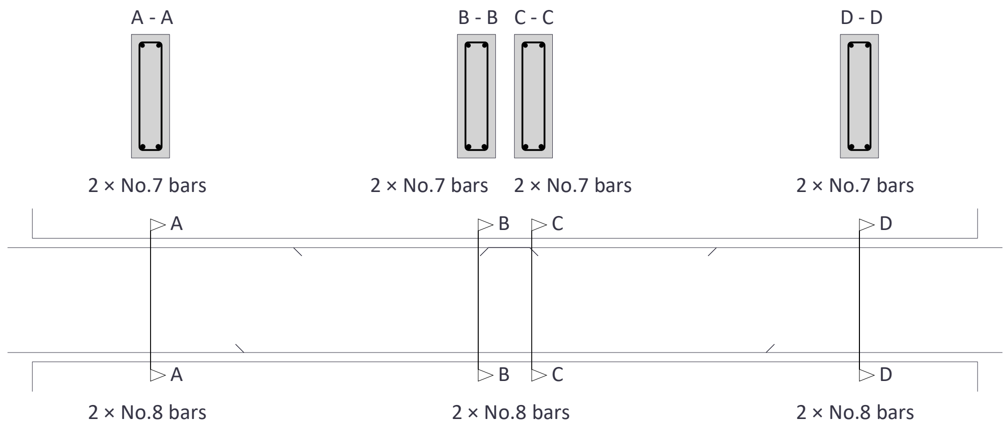
Rectangular section details

Section width b = **8** in

Section depth h = **26** in

Moment design





Zone 1 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section $M_u = abs(M_{m1\_s1\_z1\_max\_red})$ = **81.369** kip_ft

Effective depth to tension reinforcement d = **23.625** in

Tension reinforcement provided 2 × No.8 bars

Area of tension reinforcement provided $A_{s,prov}$ = **1.571** in^2

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = \max(3\text{ psi} \times \sqrt{(f'_c / 1\text{ psi})}, 200\text{ psi}) \times b \times d / f_y$ = **0.668** in^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3) $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85)$ = **0.80**

Depth of equivalent rectangular stress block $a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b)$ = **2.772** in

Depth to neutral axis $c = a / \beta_1$ = **3.465** in

Net tensile strain in extreme tension fibers $\varepsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001\text{ in})$ = **0.01745**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1) $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.90**

Nominal moment strength $M_n = A_{s,prov} \times f_y \times (d - a / 2)$ = **174.665** kip_ft

Design moment strength $\phi M_n = M_n \times \phi_f$ = **157.198** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max} = s_{bot} + \phi_{m1\_s1\_z1\_b\_L1}$ = **3.250** in

Service load stress in reinforcement (cl.24.3.2) $f_s = 2/3 \times f_y$ = **40000** psi

Clear cover of reinforcement $c_c = c_{nom\_b} + \phi_v$ = **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) $s_{max} = \min(15\text{in} \times 40000\text{psi} / f_s - 2.5 \times c_c, 12\text{in} \times 40000\text{psi} / f_s)$ = **10.313** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1} + \phi_{m1\_s1\_z2l\_t\_L1} \times N_{m1\_s1\_z2l\_t\_L1})) / ((N_{m1\_s1\_z1\_t\_L1} + N_{m1\_s1\_z2l\_t\_L1}) - 1)$ = **2.500** in

Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

Bottom bar clear spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1)$ = **2.250** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Zone 2 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section — M_u = abs($M_{m1\_s1\_z2\_max\_red}$) = **108.492** kip_ft

Effective depth to tension reinforcement — d = **23.625** in

Tension reinforcement provided — 2 × No.8 bars

Area of tension reinforcement provided — $A_{s,prov}$ = **1.571** in^2

Minimum area of reinforcement (cl.9.6.1.2) — $A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f_y = **0.668** in^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3) — β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.80**

Depth of equivalent rectangular stress block — a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **2.772** in

Depth to neutral axis — c = a / β_1 = **3.465** in

Net tensile strain in extreme tension fibers — ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.01745**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1) — ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.003), 0.65), 0.9) = **0.90**

Nominal moment strength — M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **174.665** kip_ft

Design moment strength — ϕM_n = M_n × ϕ_f = **157.198** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf. — $s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z2\_b\_L1}$ = **3.250** in

Service load stress in reinforcement (cl.24.3.2) — f_s = 2/3 × f_y = **40000** psi

Clear cover of reinforcement — c_c = c_{nom_b} + ϕ_v = **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) — s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.313** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Control of deflections (Section 24.2)

Concrete density factor — K_w = **1.00**

Reinforcement yield strength factor — K_f = 0.4 + f_y / 100000 psi = **1.00**

Minimum thickness of beam (Table 9.3.1.1) — h_{min} = (L_{m1_s1} / 16.0) × K_w × K_f = **12.375** in

PASS - Thickness of beam exceeds minimum thickness

Spacing limits for reinforcement

Top bar clear spacing — s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z2\_v}$) + $\phi_{m1\_s1\_z2r\_t\_L1}$ × $N_{m1\_s1\_z2r\_t\_L1}$)) / ($N_{m1\_s1\_z2r\_t\_L1}$ - 1) = **2.500** in

Min. allowable top bar clear spacing (cl.25.2.1) — $s_{top,min}$ = **1.000** in

Bottom bar clear spacing — s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z2\_v}$) + $\phi_{m1\_s1\_z2\_b\_L1}$ × $N_{m1\_s1\_z2\_b\_L1}$ + $\phi_{m1\_s1\_z1\_b\_L1}$ × $N_{m1\_s1\_z1\_b\_L1}$)) / (($N_{m1\_s1\_z2\_b\_L1}$ + $N_{m1\_s1\_z1\_b\_L1}$) - 1) = **2.250** in

Min. allowable bottom bar clear spacing (cl.25.2.1) — $s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)

Factored bending moment at section — M_u = abs($M_{m1\_s1\_z3\_max\_red}$) = **81.369** kip_ft

Effective depth to tension reinforcement — d = **23.625** in

Tension reinforcement provided — 2 × No.8 bars

Area of tension reinforcement provided — $A_{s,prov}$ = **1.571** in^2

Minimum area of reinforcement (cl.9.6.1.2) — $A_{s,min}$ = max(3 psi × √(f'$_c$ / 1 psi), 200 psi) × b × d / f_y = **0.668** in^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Stress block depth factor (cl.22.2.2.4.3) β_1 = min(max(0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.80**

Depth of equivalent rectangular stress block a = $A_{s,prov}$ × f_y / (0.85 × f'$_c$ × b) = **2.772** in

Depth to neutral axis c = a / β_1 = **3.465** in

Net tensile strain in extreme tension fibers ε_t = 0.003 × (d_o - c) / max(c, 0.001 in) = **0.01745**

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1) ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.003), 0.65), 0.9) = **0.90**

Nominal moment strength M_n = $A_{s,prov}$ × f_y × (d - a / 2) = **174.665** kip_ft

Design moment strength ϕM_n = M_n × ϕ_f = **157.198** kip_ft

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf. $s_{b,max}$ = s_{bot} + $\phi_{m1\_s1\_z3\_b\_L1}$ = **3.250** in

Service load stress in reinforcement (cl.24.3.2) f_s = 2/3 × f_y = **40000** psi

Clear cover of reinforcement c_c = c_{nom_b} + ϕ_v = **1.875** in

Maximum allowable bot bar spacing (Table 24.3.2) s_{max} = min(15in × 40000psi / f_s - 2.5 × c_c, 12in × 40000psi / f_s) = **10.313** in

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing s_{top} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z3\_v}$) + $\phi_{m1\_s1\_z3\_t\_L1}$ × $N_{m1\_s1\_z3\_t\_L1}$ + $\phi_{m1\_s1\_z2r\_t\_L1}$ × $N_{m1\_s1\_z2r\_t\_L1}$)) / (($N_{m1\_s1\_z3\_t\_L1}$ + $N_{m1\_s1\_z2r\_t\_L1}$) - 1) = **2.500** in

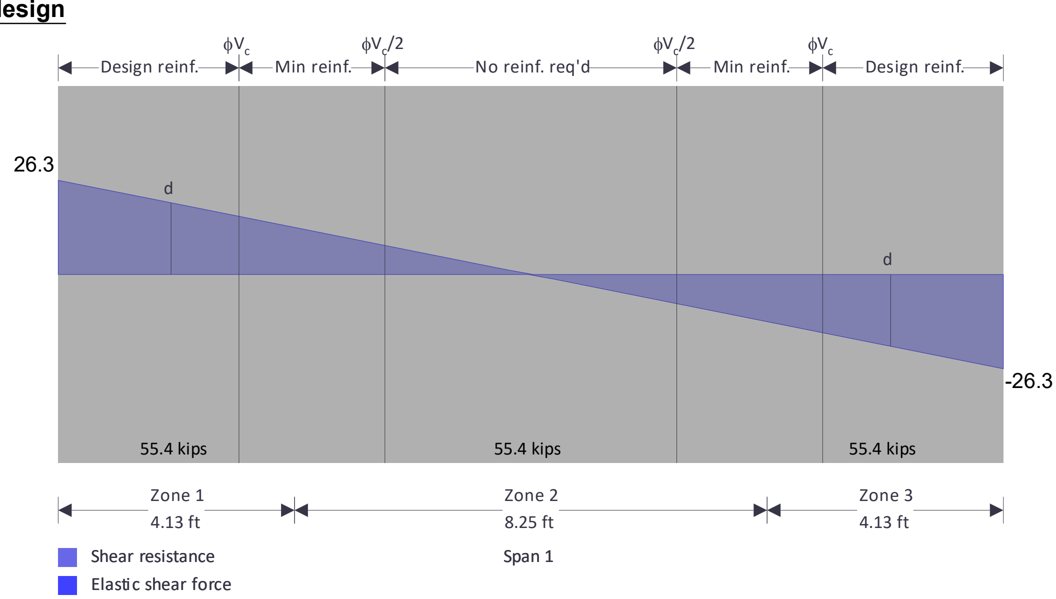
Min. allowable top bar clear spacing (cl.25.2.1) $s_{top,min}$ = **1.000** in

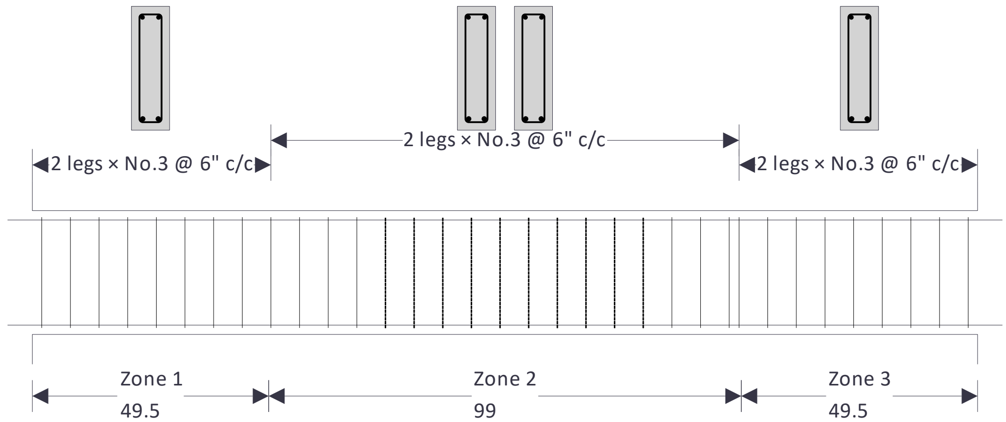
Bottom bar clear spacing s_{bot} = (b - (2 × (c_{nom_s} + $\phi_{m1\_s1\_z3\_v}$) + $\phi_{m1\_s1\_z3\_b\_L1}$ × $N_{m1\_s1\_z3\_b\_L1}$)) / ($N_{m1\_s1\_z3\_b\_L1}$ - 1) = **2.250** in

Min. allowable bottom bar clear spacing (cl.25.2.1) $s_{bot,min}$ = **1.000** in

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Rectangular section in shear

Concrete weight modification factor	λ = **1.00**
Location where min. reinf. is req'd (V_u less ϕV_c)	Between 3.16 ft and 13.34 ft
Location where no reinf. is req'd (V_u less ϕV_c / 2)	Between 5.70 ft and 10.80 ft
Maximum reinforcement shear strength	$\phi V_{s,max} = \phi_s \times 8$ psi $\times \sqrt{(\min(f'_c, 10000\text{psi}) / 1\text{ psi})} \times b \times d$ = **80.186** kips
Minimum area of shear reinf. (Table 9.6.3.4)	$A_{sv,min}$ = max(50 psi, 0.75 psi $\times \sqrt{(f'_c / 1\text{ psi})}$) $\times$ b / min(f_y, 60000 psi) = **0.085** in^2/ft

Zone 1

Effective depth of long. reinf. used for shear zone	d = **23.625** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0)$ = **0.77**
Area of longitudinal reinforcement	A_{st} = **1.571** in^2
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.008**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d$ = **16.242** kips
Design shear force at 23.6in from support	V_u = **20.025** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **3.782** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.043** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **0.085** in^2/ft
Shear reinforcement provided	2 legs $\times$ No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft

PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required

Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max}$ = d / 2 = **11.813** in

PASS - longitudinal spacing of stirrups is less than the maximum allowable

Zone 2

Effective depth of long. reinf. used for shear zone	d = **23.625** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0)$ = **0.77**
Area of longitudinal reinforcement	A_{st} = **1.571** in^2
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.008**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d$ = **16.242** kips
Design shear force within zone	V_u = **13.151** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **0.000** kips

Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.000** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = A_{sv,min}$ = **0.085** in^2/ft
Shear reinforcement provided	2 legs $\times$ No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft

PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required

Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = \min(d / 2, 24\text{ in})$ = **11.813** in

PASS - longitudinal spacing of stirrups is less than the maximum allowable

Zone 3

Effective depth of long. reinf. used for shear zone	d = **23.625** in
Size effect factor (cl. 22.5.5.1.3)	$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1\text{ in}) / 10))}, 1.0)$ = **0.77**
Area of longitudinal reinforcement	A_{st} = **1.571** in^2
Ratio of longitudinal reinforcement	$\rho_w = A_{st} / (b \times d)$ = **0.008**
Concrete shear strength (Table 22.5.5.1)	$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f'_c \times 1\text{ psi})} \times b \times d$ = **16.242** kips
Design shear force at 23.6in from support	V_u = **20.025** kips
Reinf. shear strength required (eqn. 22.5.1.1)	$\phi V_s = \max(V_u - \phi V_c, 0\text{ kips})$ = **3.782** kips
Area of design shear reinf. req'd (eqn. 22.5.8.5.3)	$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000\text{ psi}) \times d)$ = **0.043** in^2/ft
Area of shear reinforcement required	$A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **0.085** in^2/ft
Shear reinforcement provided	2 legs $\times$ No.3 @ 6" c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **0.442** in^2/ft

PASS - Area of shear reinforcement provided exceeds area of shear reinforcement required

Maximum longitudinal spacing (Table 9.7.6.2.2)	$s_{vl,max} = d / 2$ = **11.813** in

PASS - longitudinal spacing of stirrups is less than the maximum allowable

- <u>Steel Column Design (ST2):</u>

$$P_{DL} = \frac{150lb}{ft3} \times \frac{9"}{12} \times 200\ sf\,(9"\ conc. slab\ at\ 2nd\ floor) + 35psf\ \times 200\ sf\ = 29500\ lbs \approx 30\ kips$$

$$P_{LL} = 40\ psf \times 200\ sf\ = 8000\ lbs = 8\ kips$$

$$P_u = 1.2P_{DL} + 1.6P_{LL} = 1.2 \times 30 + 1.6 \times 8 = 48.8\ \ kips \approx 50\ kips$$

H=14.5' (Height of Column)

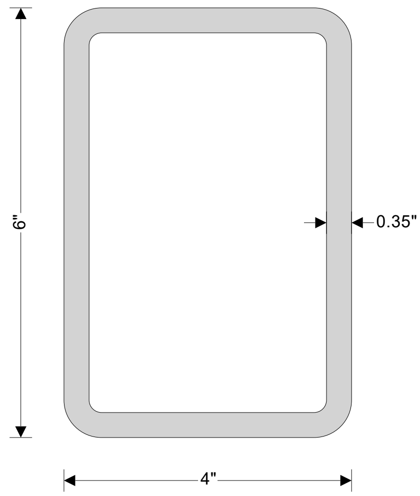
<u>Use HSS 6x4x3/8.</u>

Please see next page for calculation.

STEEL COLUMN DESIGN

In accordance with AISC360-16 and the LRFD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section **HSS 6x4x3/8**

Design loading

Required axial strength P_r = **50** kips (Compression)

Moment about x axis at end 1 M_{x1} = **0.0** kips_ft

Moment about x axis at end 2 M_{x2} = **0.0** kips_ft

Maximum moment about x axis $M_x = \max(\text{abs}(M_{x1}), \text{abs}(M_{x2}))$ = **0.0** kips_ft

Moment about y axis at end 1 M_{y1} = **0.0** kips_ft

Moment about y axis at end 2 M_{y2} = **0.0** kips_ft

Maximum moment about y axis $M_y = \max(\text{abs}(M_{y1}), \text{abs}(M_{y2}))$ = **0.0** kips_ft

Maximum shear force parallel to y axis V_{ry} = **0.0** kips

Maximum shear force parallel to x axis V_{rx} = **0.0** kips

Material details

Steel grade **A500 Gr. B**

Yield strength F_y = **46** ksi

Ultimate strength F_u = **58** ksi

Modulus of elasticity E = **29000** ksi

Shear modulus of elasticity G = **11200** ksi

Unbraced lengths

For buckling about x axis L_x = **174** in

For buckling about y axis L_y = **174** in

For torsional buckling L_z = **174** in

Effective length factors

For buckling about x axis K_x = **1.00**

For buckling about y axis K_y = **1.00**

For torsional buckling K_z = **1.00**

Effective unbraced lengths

For buckling about x axis $L_{cx} = L_x \times K_x$ = **174** in

For buckling about y axis $L_{cy} = L_y \times K_y$ = **174** in

For torsional buckling $L_{cz} = L_z \times K_z$ = **174** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f - 3 \times t$ = **2.953** in

Critical web width $h = d - 3 \times t$ = **4.953** in

Width to thickness ratio of flange (compression) $\lambda_{f\_c} = b / t$ = **8.461**

Width to thickness ratio of web (compression) $\lambda_{w\_c} = h / t$ = **14.192**

Width to thickness ratio of flange (major flexure) $\lambda_{f\_fx} = b / t$ = **8.461**

Width to thickness ratio of web (major flexure) $\lambda_{w\_fx} = h / t$ = **14.192**

Width to thickness ratio of flange (minor flexure) $\lambda_{f\_fy} = h / t$ = **14.192**

Width to thickness ratio of web (minor flexure) $\lambda_{w\_fy} = b / t$ = **8.461**

Compression

Limit for nonslender section $\lambda_{r\_c} = 1.40 \times \sqrt{(E / F_y)}$ = **35.152**

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = L_{cx} / r_x$ = **81.3**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y$ = **112.3**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 \times E / (SR_x)^2$ = **43.3** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y$ = **29.5** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} \times A$ = **182.2** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 \times E / (SR_y)^2$ = **22.7** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y$ = **19.7** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} \times A$ = **121.8** kips

Design compressive strength (cl.E1)

Resistance factor for compression ϕ_c = **0.90**

Design compressive strength $P_c = \phi_c \times \min(P_{nx}, P_{ny})$ = **109.6** kips

PASS - The design compressive strength exceeds the required compressive strength

- <u>Concrete Footing Design Under Column ST2 (F-50):</u>

From steel column design ST2, we have:

$P_{DL} = 30\ kips$

$P_{LL} = 8\ kips$

$q_{LLallowable\ capacity\ of\ soil} = 2.5\ ksf$

<u>Use 5'-0"x5'-0"x18" with 8#6 Bottom E.W.</u>

Please see next page for calculation.

FOOTING ANALYSIS

In accordance with ACI318-19

Tedds calculation version 3.3.06

Summary results

Overall design status PASS

Overall design utilisation 0.770

Description	**Unit**	**Applied**	**Resisting**	**FoS**	**Result**
Uplift verification	kips	48.1			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.924	2.5	0.770	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	24.7	223.5	0.111	Pass
Moment, positive, y-direction	kip_ft	26.5	223.5	0.119	Pass
Shear, one-way, x-direction	kips	10.7	52.9	0.202	Pass
Shear, one-way, y-direction	kips	11.5	51.1	0.225	Pass
Shear, two-way, Col 1	psi	37.911	189.737	0.200	Pass
Min.area of reinf, bot., x-direction	in^2	1.944	3.520		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	7.6		Pass
Min.area of reinf, bot., y-direction	in^2	1.944	3.520		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	7.6		Pass

Pad footing details

Length of footing L_x = **5** ft

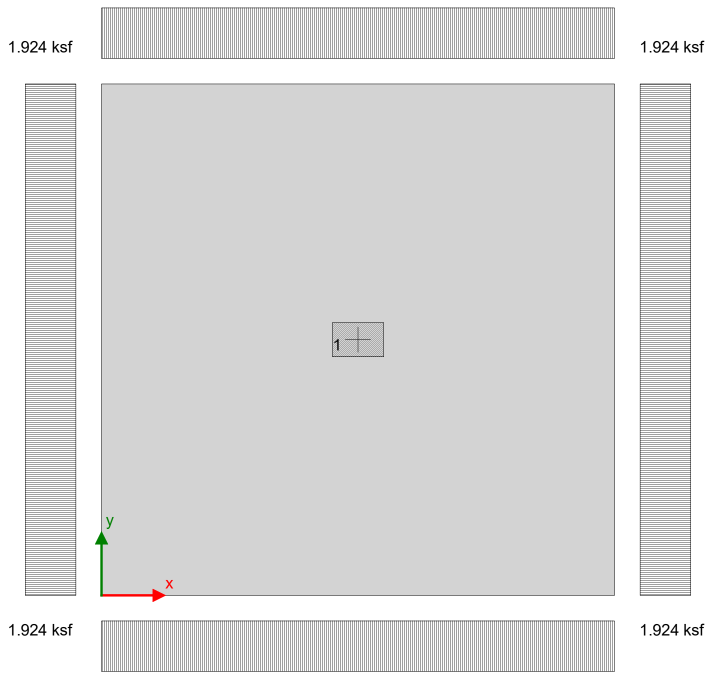
Width of footing L_y = **5** ft

Footing area $A = L_x \times L_y$ = **25** ft^2

Depth of footing h = **18** in

Depth of soil over footing h_{soil} = **18** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **6.00** in

Width of column l_{y1} = **4.00** in

position in x-axis x_1 = **30.00** in

position in y-axis y_1 = **30.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **2.5** ksf

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **225** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z F_{Dz1} = **30.0** kips

Live load in z F_{Lz1} = **8.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.642)

1.0D + 1.0L (0.770)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$ = **48.1** kips

Moments on footing

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **120.2** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **120.2** kip_ft

Uplift verification

Vertical force F_{dz} = **48.095** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.924** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.924** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.924** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.924** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **1.924** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.924** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **2.5** ksf

q_{max} / q_{allow} = **0.770**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19

Tedds calculation version 3.3.06

Material details

Compressive strength of concrete f'_c = **4000** psi

Yield strength of reinforcement f_y = **60000** psi

Compression-controlled strain limit (21.2.2) ε_{ty} = **0.00200**

Cover to top of footing c_{nom_t} = **3** in

Cover to side of footing c_{nom_s} = **3** in

Cover to bottom of footing c_{nom_b} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.193)

1.2D + 1.6L + 0.5Lr (0.225)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{60.9}$ kips

Moments on footing

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{152.3}$ kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{152.3}$ kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0}$ in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{2.437}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{2.437}$ ksf

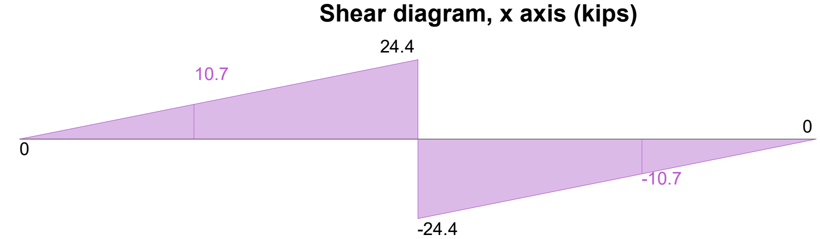
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{2.437}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{2.437}$ ksf

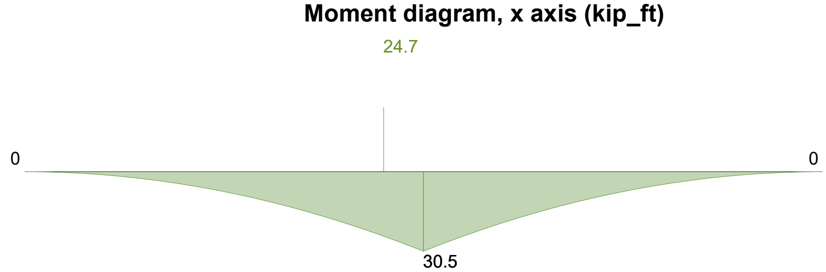
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{2.437}$ ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{2.437}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max} = \mathbf{24.701}$ kip_ft

Tension reinforcement provided 8 No.6 bottom bars (7.6 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov} = \mathbf{3.52}$ in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h =$ **1.944** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 \times h, 18\ in) =$ **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom\_b} - \phi_{y.bot} - \phi_{x.bot} / 2 =$ **13.875** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **1.035** in

Neutral axis factor $\beta_1 =$ **0.85**

Depth to neutral axis $c = a / \beta_1 =$ **1.218** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.03118**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **235.089** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **211.58** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.117**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x} =$ **10.674** kips

Depth to reinforcement $d_v = h - c_{nom\_b} - \phi_{x.bot} / 2 =$ **14.625** in

Size effect factor (22.5.5.1.3) $\lambda_s =$ **1**

Ratio of longitudinal reinforcement $\rho_w = A_{sx.bot.prov} / (L_y \times d_v) =$ **0.00401**

Shear strength reduction factor $\phi_v =$ **0.75**

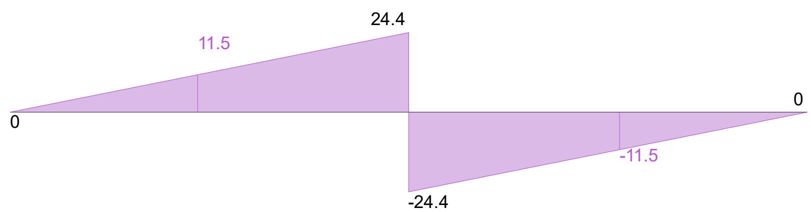
Nominal shear capacity (Eq. 22.5.5.1) $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v) =$ **70.545** kips

Design shear capacity $\phi V_n = \phi_v \times V_n =$ **52.909** kips

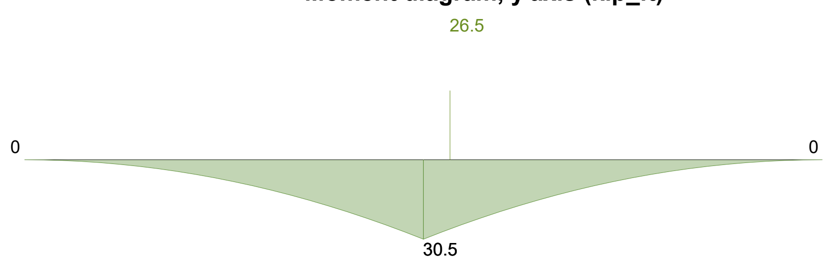
$V_{u.x} / \phi V_n =$ **0.202**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **26.549** kip_ft

Tension reinforcement provided 8 No.6 bottom bars (7.6 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **3.52** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h$ = **1.944** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 \times h, 18 \text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom\_b} - \phi_{y.bot} / 2$ = **14.625** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **1.035** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **1.218** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03302**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **248.289** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **223.46** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.119**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **11.48** kips

Depth to reinforcement $d_v = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **13.875** in

Size effect factor (22.5.5.1.3) λ_s = **1**

Ratio of longitudinal reinforcement $\rho_w = A_{sy.bot.prov} / (L_x \times d_v)$ = **0.00423**

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v)$ = **68.112** kips

Design shear capacity $\phi V_n = \phi_v \times V_n$ = **51.084** kips

$V_{u.y} / \phi V_n$ = **0.225**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement d_{v2} = **14.25** in

Shear perimeter length (22.6.4) l_{xp} = **20.250** in

Shear perimeter width (22.6.4) l_{yp} = **18.250** in

Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **77.000** in

Shear area $A_p = l_{x,perim} \times l_{y,perim}$ = **369.563** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **345.563** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg}$ = **2.437** ksf

Ultimate shear load $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **43.722** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi})$ = **39.847** psi

Column geometry factor (Table 22.6.5.2)	$\beta = I_{x1} / I_{y1} =$ **1.50**
Column location factor (22.6.5.3)	$\alpha_s =$**40**
Size effect factor (22.5.5.1.3)	$\lambda_s =$ **1**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **295.146** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **594.672** psi
	$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **252.982** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **252.982** psi
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} =$ **252.982** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n =$ **189.737** psi
	$v_{ug} / \phi v_n =$ **0.210**

PASS - Design shear stress capacity exceeds ultimate shear stress load