

Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project Herrera Res.				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 05/12/2025	Chk'd by Z.T.B	Date	App'd by	Date

STRUCTURAL CALCULATIONS FOR HERRERA RESIDENCE

6825 SW 132nd St,
Miami, FL 33156

May 12th, 2025

Zvonimir T. Belfranin, PE

FL PE# 33074



THIS DOCUMENT HAS BEEN DIGITALLY SIGNED
AND IS ALREADY A DIGITAL SIGNATURE.
PRINTED COPIES OF THIS DOCUMENT ARE NOT
AUTHENTICATED BY THE STATE OF FLORIDA AND THE
AUTHENTICITY OF ANY COPIES MUST BE VERIFIED ON
ANY ELECTRONIC COPIES.

Digitally signed by Zvonimir Belfranin
DN: c=US, o=Zvonimir Belfranin,
dnQualifier=A01410D000001926C393
A9E00006B45, cn=Zvonimir Belfranin
Date: 2025.05.14 07:11:09 -04'00'

Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project Herrera Res.				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 05/12/2025	Chk'd by Z.T.B	Date	App'd by	Date

Table of Contents

Topic	Page
Design Criteria	3-4
Wind Pressure Design	5-10
CMU Wall Design	11-17
Conc. Beam Design	18-35
Concrete Column Design	36-44
Conc. Footing Design	45-52



THIS DOCUMENT HAS BEEN DIGITALLY SIGNED
AND SEALED USING A DIGITAL SIGNATURE.
PRINTED COPIES OF THIS DOCUMENT ARE NOT
CONSIDERED SIGNED AND SEALED AND THE
AUTHENTICATION CODE MUST BE VERIFIED ON
ANY ELECTRONIC COPIES.

Digitally signed by Zvonimir
Belfranin
DN: c=US, o=Zvonimir Belfranin,
dnQualifier=A01410D00000192
6C393A9E00006B45,
cn=Zvonimir Belfranin
Date: 2025.05.14 07:11:30 -04'00'

Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project Herrera Res.				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 05/12/2025	Chk'd by Z.T.B	Date	App'd by	Date

- 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)

Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project Herrera Res.				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 05/12/2025	Chk'd by Z.T.B	Date	App'd by	Date

- Wind Design Load
 - Wind Velocity 175 MPH
 - Building Exposure “C”
 - Risk Category “ II”
 - Coefficient of Internal pressure ± 0.18
 - Building Type “Enclosed”
 - Building Height 25’

MecaWind v2482

Developed by Meca Enterprises Inc., www.mecaenterprises.com, Copyright © 2025

Calculations Prepared by:

Date: May 12, 2025

Designer: Z.T.B

File Location: C:\Users\User\ASM Eng Dropbox\2025-ASM\Hossein\HERRERA\Calc's\
Mecawind\Hipped Wind-7-22.wnd

Calculations Prepared For:

Location: Miami, FL

Description: Herrera Res.

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	= 4.0 :12
θ = Slope of Roof	= 18.43 Deg	R_{Ht} = Ridge Height	= 31.333 ft
E_{Ht} = Eave Height	= 25.000 ft	W = Building Width	= 32.000 ft
L = Building Length	= 92.000 ft	HipLen = Ridge Hipped Length	= 60.000 ft
EndSlope = Hipped End Slope of Roof	= 18.43 Deg	OH = Type of Overhang	= All Soffit
Par = Parapet	= None	HT_{over} = Override Mean Roof Height	= False
Ht_{man} = Mean Roof Height	= 28.167 ft	RA_{over} = Override Roof Area	= False
GC_{pi_o} = Override GC_{pi} value	= False	IsElev = Building is Elevated	= False

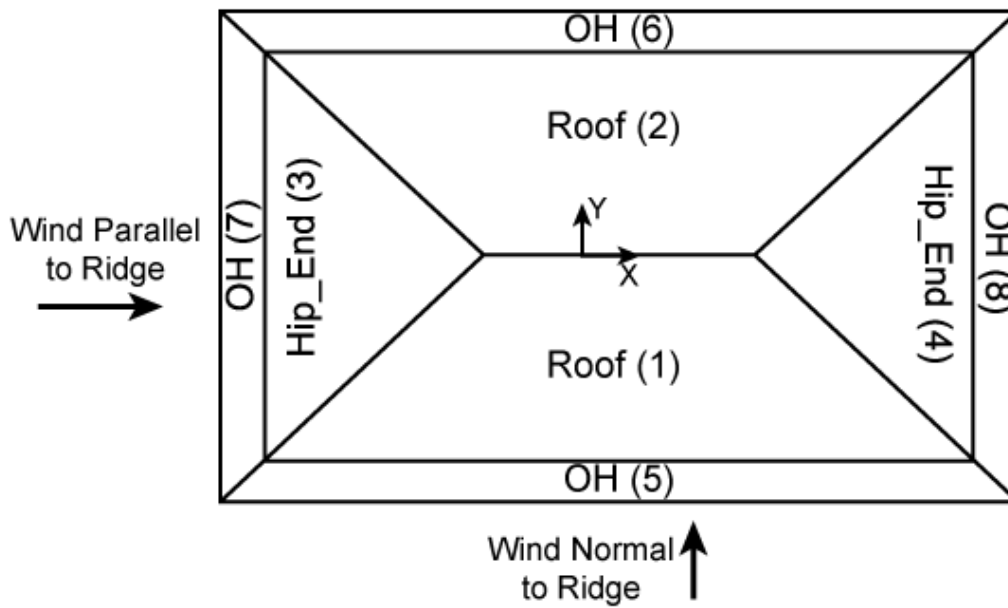
Exposure Constants [Tbl 26.11-1]:

α = 3-s Gust-speed exponent	= 9.800	Z_g = Nominal Ht of Boundary Layer	= 2460.000 ft
$\hat{\alpha}$ = 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

Overhang Inputs:

Std	= Overhangs on all sides are the same	= True
OHType	= Type of Roof Wall Intersections	= Soffit
OH	= Overhang of Roof Beyond Wall	= 3.000 ft

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



h	= Mean structure height	= 28.167 ft
K _h	= $2.41 \cdot (Z/Z_g)^{2/\alpha}$ [Tbl 26.10-1]	= 0.968
K _{zt}	= Topographic: $(1+K_1 \cdot K_2 \cdot K_3)^{1/2}$ [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	= $0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 \cdot LF$ [Eq 26.10-1]	= 45.53 psf
RA	= Roof Area	= 3324.61 ft ²
Q _h	= $0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 \cdot LF$ [Eq 26.10-1]	= 45.53 psf
Q _{in}	= Negative Internal Pressure: $q_h \cdot LF$	= 45.53 psf
Q _{ip}	= Positive Internal Pressure: $q_h \cdot LF$	= 45.53 psf

MWFRS Wind Loads [Normal to Ridge]

h	= Mean Roof Height Of Building	= 28.167 ft
RHt	= Ridge Height Of Roof	= 31.333 ft
B	= Horizontal Dimension Of Building Normal To Wind Direction	= 92.000 ft
L	= Horizontal Dimension Of building Parallel To Wind Direction	= 32.000 ft
L/B	= Ratio Of L/B used For Cp determination	= 0.348
h/L	= Ratio Of h/L used For Cp determination	= 0.880
Slope	= Slope Of Main Roof	= 18.43 Deg
HipSlope	= Slope Of Hipped Ends of Roof	= 18.43 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: $\text{Max}(0.6 \cdot h, Z_{\min})$	= 16.900 ft
I _{zm}	= Turbulence Intensity: $c \cdot (33/Z_m)^{1/6}$ [Eq 26.11-7]	= 0.224
L _{zm}	= Turbulence Integral Length Scale: $l \cdot (Z_m/33)^e$ [Eq 26.11-9]	= 437.366 ft
B	= Building Width Width Normal to Wind Direction	= 92.000 ft
Q	= $[1/(1+0.63 \cdot [(B+h)/L_{zm}]^{0.63})]^{0.5}$ [Eq 26.11-8]	= 0.884
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]	= 0.865

Gust Factor Used in Analysis

G	= Gust Factor: $\text{Min}(G_1, G_2)$	= 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
25.000	+0.18	45.53	0.945	1.000	44.44	18.72	-23.42	-29.99	42.13	9.60

25.000	-0.18	45.53	0.945	1.000	44.44	32.65	-9.48	-16.06	42.13	9.60
--------	-------	-------	-------	-------	-------	-------	-------	--------	-------	------

$$K_z = 2.41 \cdot (Z/Z_g)^{2/\alpha}$$

$$GC_{pi} = \text{Enclosed Internal Pressure Tbl } 26.13-1$$

$$q_{ip} = \text{Positive Internal Pressure: } q_h \cdot LF$$

$$\text{Side} = q_h \cdot K_d \cdot G \cdot C_{psw} - q_{ip} \cdot K_d \cdot (+GC_{pi}) \text{ Eq 27.3-1}$$

$$\text{Windward} = q_h \cdot K_d \cdot G \cdot C_{pww} - q_{ip} \cdot K_d \cdot (+GC_{pi}) \text{ Eq 27.3-1}$$

$$K_{zt} = \text{Topographic: } (1+K_1 \cdot K_2 \cdot K_3)^{-2} \text{ [Eq 26.8-1]}$$

$$q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_e \cdot V^2 \cdot LF \text{ [Eq 26.10-1]}$$

$$q_{in} = \text{Negative Internal Pressure: } q_h \cdot LF$$

$$\text{Leeward} = q_h \cdot K_d \cdot G \cdot C_{plw} - q_{ip} \cdot K_d \cdot (+GC_{pi}) \text{ Eq 27.3-1}$$

$$\text{Total} = \text{Windward} - \text{Leeward}$$

• 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	3.000	14.083	+0.18	-1.006	-0.180	-40.08	-12.89	4.80
Hip End	Hipped End (h/2 to h)	3,4	14.083	28.167	+0.18	-0.748	-0.180	-31.57	-12.89	4.80
Hip End	Hipped End (h to 2*h)	3,4	28.167	35.000	+0.18	-0.652	-0.180	-28.42	-12.89	4.80
OH Top	Overhang Top	5	All	All	+0.18	-0.150	-0.722	-11.91	-30.72	4.80
OH Top	Overhang Top (0 to h/2)	7,8	0.000	14.083	+0.18	-1.006	-0.180	-40.08	-12.89	4.80
OH Top	Overhang Top (h/2 to h)	7,8	14.083	28.167	+0.18	-0.748	-0.180	-31.57	-12.89	4.80
OH Top	Overhang Top (h to 2*h)	7,8	28.167	38.000	+0.18	-0.652	-0.180	-28.42	-12.89	4.80
OH Top	Overhang Leeward	7,8	All	All	+0.18	-0.593	-0.593	-26.46	-26.46	4.80
Roof LW	Roof Leeward	2	All	All	+0.18	-0.593	-0.593	-26.46	-26.46	4.80
Roof WW	Roof Windward	1	All	All	+0.18	-0.150	-0.722	-11.91	-30.72	4.80
Soffit	Soffit Bottom	5	All	All	+0.18	0.800	0.800	19.35	19.35	4.80
Hip End	Hipped End (0 to h/2)	3,4	3.000	14.083	-0.18	-1.006	-0.180	-26.14	1.04	4.80
Hip End	Hipped End (h/2 to h)	3,4	14.083	28.167	-0.18	-0.748	-0.180	-17.64	1.04	4.80
Hip End	Hipped End (h to 2*h)	3,4	28.167	35.000	-0.18	-0.652	-0.180	-14.49	1.04	4.80
OH Top	Overhang Top	5	All	All	-0.18	-0.150	-0.722	2.02	-16.79	4.80
OH Top	Overhang Top (0 to h/2)	7,8	0.000	14.083	-0.18	-1.006	-0.180	-26.14	1.04	4.80
OH Top	Overhang Top (h/2 to h)	7,8	14.083	28.167	-0.18	-0.748	-0.180	-17.64	1.04	4.80
OH Top	Overhang Top (h to 2*h)	7,8	28.167	38.000	-0.18	-0.652	-0.180	-14.49	1.04	4.80
OH Top	Overhang Leeward	7,8	All	All	-0.18	-0.593	-0.593	-12.53	-12.53	4.80
Roof LW	Roof Leeward	2	All	All	-0.18	-0.593	-0.593	-12.53	-12.53	4.80
Roof WW	Roof Windward	1	All	All	-0.18	-0.150	-0.722	2.02	-16.79	4.80
Soffit	Soffit Bottom	5	All	All	-0.18	0.800	0.800	33.28	33.28	4.80

Roof Pressures based upon Ch 27:

Component = The building component for pressures

Start = Start Dist from Windward Edge

C_{pMin} = Smallest 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]

Location = Reference Graphic in Output for Values

End = End Dist from Windward Edge

C_{pMax} = Largest Coefficient Magnitude

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	= 28.167 ft
RHt	= Ridge Height Of Roof	= 31.333 ft
B	= Horizontal Dimension Of Building Normal To Wind Direction	= 32.000 ft
L	= Horizontal Dimension Of building Parallel To Wind Direction	= 92.000 ft
L/B	= Ratio Of L/B used For Cp determination	= 2.875
h/L	= Ratio Of h/L used For Cp determination	= 0.306
Slope	= Slope Of Main Roof	= 18.43 Deg
HipSlope	= Slope Of Hipped Ends of Roof	= 18.43 Deg

Gust Factor Calculation for Wind: [Parallel to Ridge]

Gust Factor Category I Rigid Structures - Simplified Method

G₁ = 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•h, Z_{min}) = 16.900 ft

I_{zm} = Turbulence Intensity: $c \cdot (33/Z_m)^{1/6}$ [Eq 26.11-7] = 0.224

L_{zm} = Turbulence Integral Length Scale: $l \cdot (Z_m/33)^c$ [Eq 26.11-9] = 437.366 ft

B = Building Width Normal to Wind Direction = 32.000 ft

Q = $[1/(1+0.63 \cdot [(B+h)/L_{zm}]^{0.63})]^{0.5}$ [Eq 26.11-8] = 0.920

G₂ = Detailed: $0.925 \cdot [(1+1.7 \cdot q_q \cdot I_{zm} \cdot Q)/(1+1.7 \cdot q_v \cdot I_{zm})]$ [Eq 26.11-6] = 0.883

Gust Factor Used in Analysis

G = Gust Factor: Min(G₁, G₂) = 0.850

$C_{p_{WW}}$ = Windward Wall Coefficient (All L/B Values) = 0.800
 $C_{p_{LW}}$ = Leeward Wall Coefficient using L/B = -0.256
 $C_{p_{SW}}$ = 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
25.000	+0.18	45.53	0.945	1.000	44.44	18.72	-15.40	-29.99	34.11	9.60
25.000	-0.18	45.53	0.945	1.000	44.44	32.65	-1.46	-16.06	34.11	9.60

$K_z = 2.41 \cdot (Z/Z_g)^{2/\alpha}$
 GC_{pi} = Enclosed Internal Pressure Tbl 26.13-1
 q_{ip} = Positive Internal Pressure: $q_h \cdot LF$
 Side = $q_h \cdot K_d \cdot G \cdot C_{p_{SW}} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eq 27.3-1
 Windward = $q_z \cdot K_d \cdot G \cdot C_{p_{WW}} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eq 27.3-1
 q_{in} = Negative Internal Pressure: $q_h \cdot LF$
 Leeward = $q_h \cdot K_d \cdot G \cdot C_{p_{LW}} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ 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

K_{zt} = Topographic: $(1+K_1 \cdot K_2 \cdot K_3)^{-2}$ [Eq 26.8-1]
 $q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot LF$ [Eq 26.10-1]

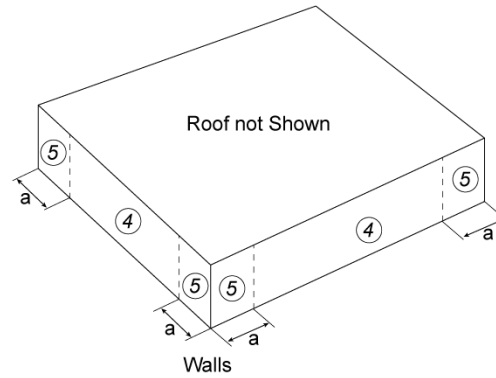
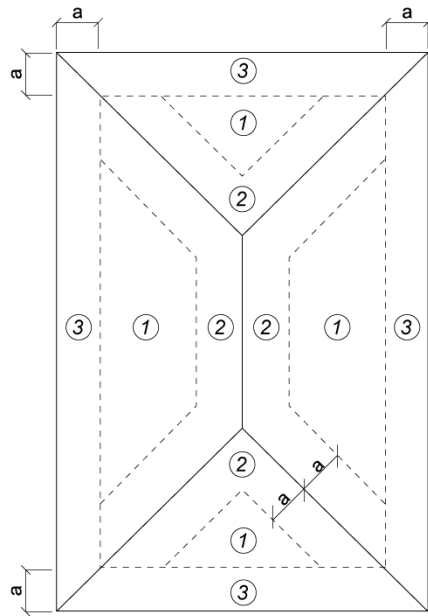
Roof Wind Pressures [Parallel to Ridge]
All wind pressures include a Load Factor (LF) of 0.6

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	3.000	19.000	+0.18	-0.900	-0.180	-36.57	-12.89	4.80
Hip_End	Hipped End (>= 2*h)	4	79.000	95.000	+0.18	-0.300	-0.180	-16.84	-12.89	4.80
OH_Top	Overhang Top (0 to h)	5,6,7	0.000	28.167	+0.18	-0.900	-0.180	-36.57	-12.89	4.80
OH_Top	Overhang Top (h to 2*h)	5,6	28.167	56.333	+0.18	-0.500	-0.180	-23.42	-12.89	4.80
OH_Top	Overhang Top (>= 2*h)	5,6,8	56.333	98.000	+0.18	-0.300	-0.180	-16.84	-12.89	4.80
Roof	Roof (0 to h)	5,6	3.000	28.167	+0.18	-0.900	-0.180	-36.57	-12.89	4.80
Roof	Roof (h to 2*h)	5,6	28.167	56.333	+0.18	-0.500	-0.180	-23.42	-12.89	4.80
Roof	Roof (>= 2*h)	5,6	56.333	95.000	+0.18	-0.300	-0.180	-16.84	-12.89	4.80
Soffit	Soffit Bottom	7	All	All	+0.18	0.800	0.800	19.35	19.35	4.80
Hip_End	Hipped End (0 to h)	3	3.000	19.000	-0.18	-0.900	-0.180	-22.64	1.04	4.80
Hip_End	Hipped End (>= 2*h)	4	79.000	95.000	-0.18	-0.300	-0.180	-2.90	1.04	4.80
OH_Top	Overhang Top (0 to h)	5,6,7	0.000	28.167	-0.18	-0.900	-0.180	-22.64	1.04	4.80
OH_Top	Overhang Top (h to 2*h)	5,6	28.167	56.333	-0.18	-0.500	-0.180	-9.48	1.04	4.80
OH_Top	Overhang Top (>= 2*h)	5,6,8	56.333	98.000	-0.18	-0.300	-0.180	-2.90	1.04	4.80
Roof	Roof (0 to h)	5,6	3.000	28.167	-0.18	-0.900	-0.180	-22.64	1.04	4.80
Roof	Roof (h to 2*h)	5,6	28.167	56.333	-0.18	-0.500	-0.180	-9.48	1.04	4.80
Roof	Roof (>= 2*h)	5,6	56.333	95.000	-0.18	-0.300	-0.180	-2.90	1.04	4.80
Soffit	Soffit Bottom	7	All	All	-0.18	0.800	0.800	33.28	33.28	4.80

Roof Pressures based upon Ch 27:
 Component = The building component for pressures
 Start = Start Dist from Windward Edge
 C_{pMin} = Smallest 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_{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

Location = Reference Graphic in Output for Values
 End = End Dist from Windward Edge
 C_{pMax} = Largest Coefficient Magnitude
 $P_{CpMax} = q_h \cdot K_d \cdot G \cdot C_{pMax} - q_{in} \cdot K_d \cdot GC_{pi}$ [Eq 27.3-1]

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



h/W	=	Ratio of mean roof height to building width	=	0.880
h/L	=	Ratio of mean roof height to building length	=	0.306
h	=	Mean structure height	=	28.167 ft
K_h	=	$2.41 \cdot (Z/Z_g)^{2/\alpha}$	=	0.968
K_{zt}	=	Topographic: $(1+K_1 \cdot K_2 \cdot K_3)^2$ [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	=	$0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 \cdot LF$ [Eq 26.10-1]	=	45.53 psf
LHD	=	Least Horizontal Dimension: Min(B, L)	=	32.000 ft
a_1	=	Min(0.1 • LHD, 0.4 • h)	=	3.200 ft
a	=	Max(a_1 , 0.04 • LHD, 3 ft [0.9 m])	=	3.200 ft
h/B	=	Ratio of mean roof height to least horizontal dim: h/B	=	0.880

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 ft ²	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	30.3-2E	0.700	-1.800	34.06	-76.63
Zone 2e- 10 SF	2	2.000	5.000	10.00	No	30.3-2E	0.700	-2.400	34.06	-99.85
Zone 2r- 10 SF	2	2.000	5.000	10.00	No	30.3-2E	0.700	-2.400	34.06	-99.85
Zone 3- 10 SF	3	2.000	5.000	10.00	No	30.3-2E	0.700	-2.600	34.06	-107.59
Zone 4- 10 SF	4	2.000	5.000	10.00	No	30.3-1	1.000	-1.100	45.67	-49.54
Zone 5- 10 SF	5	2.000	5.000	10.00	No	30.3-1	1.000	-1.400	45.67	-61.15

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

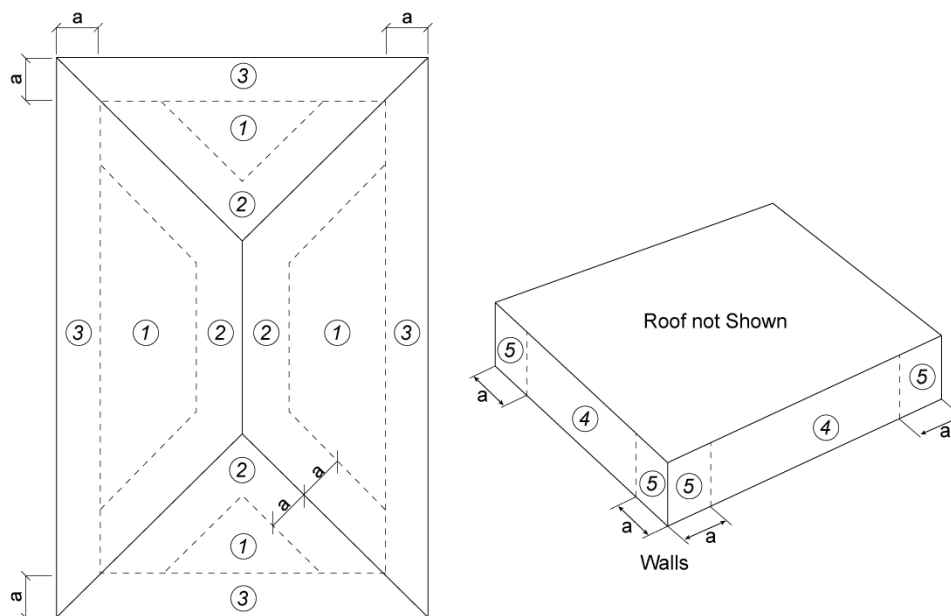
p = Wind Pressure: $q_h \cdot K_d \cdot [GC_p - GC_{pi}]$ [Eq 30.3-1]

* 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	=	0.880
h/L	=	Ratio of mean roof height to building length	=	0.306
h	=	Mean structure height	=	28.167 ft
K_h	=	$2.41 \cdot (Z/Z_g)^{2/\alpha}$	=	0.968
K_{zt}	=	Topographic: $(1+K_1 \cdot K_2 \cdot K_3)^2$ [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	=	$0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 \cdot LF$ [Eq 26.10-1]	=	45.53 psf
LHD	=	Least Horizontal Dimension: Min(B, L)	=	32.000 ft
a_1	=	Min($0.1 \cdot LHD$, $0.4 \cdot h$)	=	3.200 ft
a	=	Max(a_1 , $0.04 \cdot LHD$, 3 ft [0.9 m])	=	3.200 ft
h/B	=	Ratio of mean roof height to least horizontal dim: h/B	=	0.880



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 ft ² psf	Neg A ≤ 10 ft ² psf	Pos A = 20 ft ² psf	Neg A = 20 ft ² psf	Pos A = 50 ft ² psf	Neg A = 50 ft ² psf
1	30.3-2E	34.06	-76.63	29.40	-67.68	23.24	-55.84
2	30.3-2E	34.06	-99.85	29.40	-90.00	23.24	-76.98
3	30.3-2E	34.06	-107.59	29.40	-96.85	23.24	-82.64
4	30.3-1	45.67	-49.54	43.61	-47.48	40.89	-44.76
5	30.3-1	45.67	-61.15	43.61	-57.04	40.89	-51.60
1_OHS	30.3-2E/30.3-1	9.60	-115.33	9.60	-104.32	9.60	-89.76
2_OHS	30.3-2E/30.3-1	9.60	-138.56	9.60	-126.65	9.60	-110.91
3_OHS	30.3-2E/30.3-1	9.60	-146.30	9.60	-133.49	9.60	-116.57

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 ft ² psf	Neg A = 100 ft ² psf	Pos A = 200 ft ² psf	Neg A = 200 ft ² psf	Pos A > 500 ft ² psf	Neg A > 500 ft ² psf
1	30.3-2E	18.58	-46.88	18.58	-37.93	18.58	-37.93
2	30.3-2E	18.58	-67.13	18.58	-57.28	18.58	-57.28
3	30.3-2E	18.58	-71.90	18.58	-61.15	18.58	-61.15
4	30.3-1	38.84	-42.71	36.78	-40.65	34.06	-37.93
5	30.3-1	38.84	-47.48	36.78	-43.37	34.06	-37.93
1_OHS	30.3-2E/30.3-1	9.60	-78.75	9.60	-67.74	9.60	-65.02
2_OHS	30.3-2E/30.3-1	9.60	-99.00	9.60	-87.09	9.60	-84.37
3_OHS	30.3-2E/30.3-1	9.60	-103.76	9.60	-90.96	9.60	-88.24

* 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.

Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project Herrera Res.				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 05/12/2025	Chk'd by Z.T.B	Date	App'd by	Date

- CMU Wall Design

Height of CMU wall is 15'(First Floor Height)-2'(Beam Depth)=13'.

Based on Mecawind calculation, we have:

These loads are based on area A=50 sf.

$$P_w = 51.6 \text{ psf}(\text{zone } 5)$$

$$F_{Ultimate} = \frac{51.6}{0.6} = 86 \text{ psf}$$

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

Use #5 @ 24" O.C for zone 4 & 5

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 $L = 20$ ft

Panel height $h = 13$ ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) $I_e = 1$

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

$S_{DS} = 1$

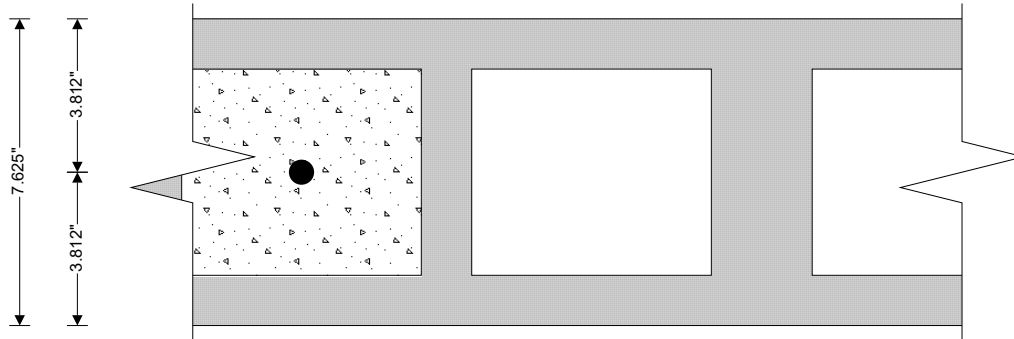
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

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

Construction details

Wall thickness $t = 7.625$ 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

$$f_{cu} = 1900 \text{ psi}$$

Density of masonry units

$$\gamma_{block} = 115 \text{ lb/ft}^3$$

Height of masonry units

$$h_b = 7.625 \text{ in}$$

Length of masonry units

$$l_b = 15.625 \text{ in}$$

Number of internal webs

$$N_{web} = 1$$

Number of end webs

$$N_{end} = 2$$

Internal web thickness

$$t_{bw} = 1.25 \text{ in}$$

Face shell thickness

$$t_{bf} = 1.25 \text{ in}$$

End web thickness

$$t_{be} = 1.25 \text{ in}$$

Area of block

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

Area of grout

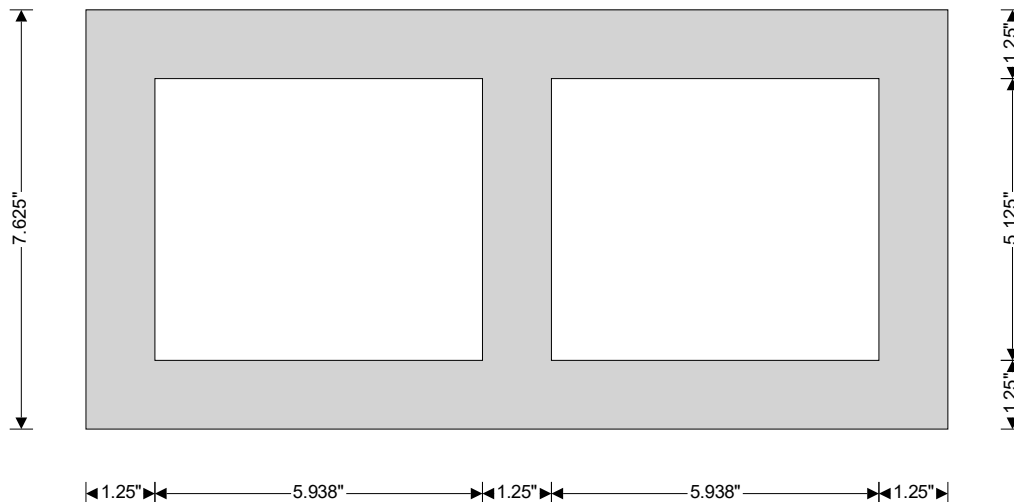
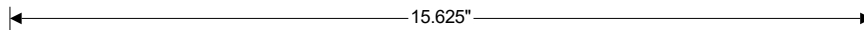
$$A_{grout} = [0.33 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b = 15.42 \text{ in}^2/\text{ft}$$

Density of grout

$$\gamma_{grout} = 140 \text{ lb/ft}^3$$

Self weight of wall

$$WSW = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout} = 50.74 \text{ psf}$$



 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section CMU Wall Design				Sheet no./rev. 3	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

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 $A_s = \pi \times Dia^2 / (4 \times s) = 0.15$ in²/ft

Lateral out-of-plane loads

Wind load on panel $W = 86$ 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 seismic 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)

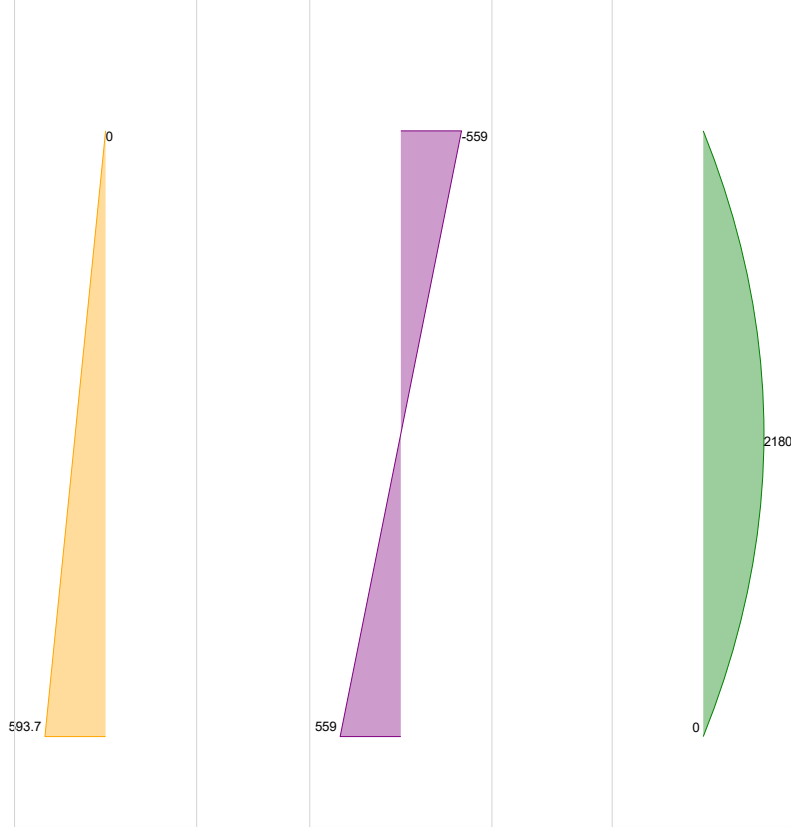
Load combination no.1 $1.4 \times DL$ (0.018)
Load combination no.4 $1.2 \times DL + 1.6 \times (LL_r \text{ or } SL \text{ or } RL) + 0.5 \times W$ (0.355)
Load combination no.5 $1.2 \times DL + W + LL + 0.5 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.717)
Load combination no.7 $0.9 \times DL + W$ (0.723)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b = 60.2$ in²/ft
Properties for walls loaded out-of-plane:
Moment of inertia $I = t^3 / 12 - 0.67 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) = 374.8$ in⁴/ft
Section modulus $S = I / c = 98.3$ in³/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

Axial force, out of plane - Combination No.7 - lbs/ft Shear force, out of plane - Combination No.7 - lbs/ft Moment force, out of plane - Combination No.7 - lb_in



Maximum moment location

6.5 ft

Axial load at mid-height of panel

$P = 297$ lb/ft

Slenderness ratio

$(K \times h) / r = 62.514 < 99$

Nominal axial strength

$P_n = 0.8 \times (0.8 \times f_m \times (A - A_s) + A_s \times 0 \text{ ksi}) \times [1 - ((K \times h) / (140 \times r))^2] = 58443$ lb/ft

Strength reduction factor

$\phi = 0.9$

Design axial strength

$\phi \times P_n = 52598$ lb/ft

$P / (\phi \times P_n) = 0.006$

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

$c_{cr} = (A_s \times f_y + P) / (0.64 \times f_m) = 0.651$ in

Moment of inertia of cracked section

$I_{cr} = n \times (A_s \times P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3 = 27.9$ in⁴/ft

By iteration

$M_{u0} = M = 21801$ lb_in/ft

$\delta_{u0} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I) + 5 \times (M_{u0} - M_{cr}) \times h^2 / (48 \times E_m \times I_{cr}) = 0.624$ in

$M_{u1} = M_{u0} + P \times \delta_{u0} = 21986$ lb_in/ft

Bending moment at mid-height of panel

Depth of reinforcement

Effective width per bar

Strength reduction factor

Tensile strain in reinforcement

Maximum usable compressive strain of masonry

Fiber of max.compressive strain to neutral axis

Tensile force at balance point

Compressive force at balance point

Design axial force at balance point

Design moment at balance point

Maximum design moment from interaction diagram

$$\delta_{u1} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I) + 5 \times (M_{u1} - M_{cr}) \times h^2 / (48 \times E_m \times I_{cr}) = \mathbf{0.634 \text{ in}}$$

$$M = M_{u0} + P \times \delta_{u1} = \mathbf{21989 \text{ lb_in/ft}}$$

$$d = \mathbf{3.813 \text{ in}}$$

$$b_{eff} = \min(s, 6 \times t_{nom}, 72 \text{ in}) = \mathbf{24 \text{ in}}$$

$$\phi = \mathbf{0.9}$$

$$\epsilon_s = f_y / E_s = \mathbf{0.0021}$$

$$\epsilon_{mu} = \mathbf{0.0025}$$

$$C_{bal} = \epsilon_{mu} \times d / (\epsilon_{mu} + \epsilon_s) = \mathbf{2.086 \text{ in}}$$

$$T_{bal} = A_s \times f_y = \mathbf{9204 \text{ lb/ft}}$$

$$\beta_1 = \mathbf{0.8}$$

$$C_{bal} = 0.8 \times f'_m \times \beta_1 \times C_{bal} = \mathbf{30440 \text{ lb/ft}}$$

$$P_{bal} = \phi \times (C_{bal} - T_{bal}) = \mathbf{19113 \text{ lb/ft}}$$

$$M_{bal} = \phi \times [T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - \beta_1 \times C_{bal} / 2)] = \mathbf{81588 \text{ lb_in/ft}}$$

$$M_c = \mathbf{30423 \text{ lb_in/ft}}$$

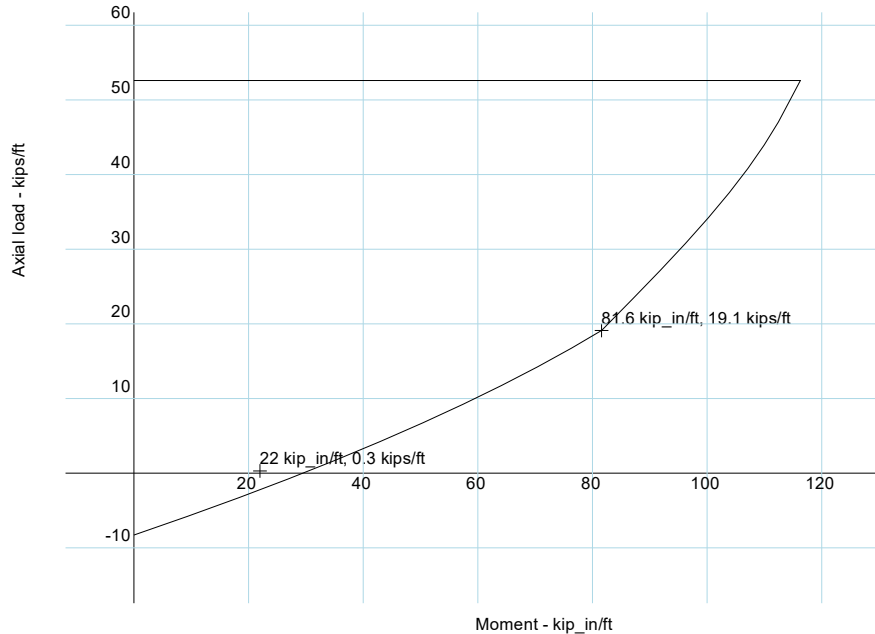
$$M / M_c = \mathbf{0.723}$$

PASS - Combination of applied axial load and flexure is acceptable

Maximum area of tensile reinforcement (9.3.3.2)

$$A_{s_max} = 0.64 \times f'_m \times [\epsilon_{mu} / (\epsilon_{mu} + 1.5 \times \epsilon_s)] \times d / f_y = \mathbf{0.414 \text{ in}^2/\text{ft}}$$

PASS - Area of reinforcement provided is less than maximum allowable



Strength interaction diagram

Consider wall at top under load combination no.7

Shear force

$$V = \mathbf{559 \text{ lb/ft}}$$

Compressive force

$$N_u = \mathbf{0 \text{ lb/ft}}$$

Net shear area

$$A_{nv} = d \times I_b / ((N_{web} + 1) \times S_{grout}) = \mathbf{14.893 \text{ in}^2/\text{ft}}$$

Project Herrera Res.				Job Ref.	
Section CMU Wall Design				Sheet no./rev. 6	
Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Nominal shear strength

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

Strength reduction factor

$$\phi_v = \mathbf{0.8}$$

Design shear strength

$$\phi_v \times V_n = \mathbf{2077 \text{ lb/ft}}$$

$$V / (\phi_v \times V_n) = \mathbf{0.269}$$

PASS - Design shear strength exceeds applied shear strength

Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project Herrera Res.				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 05/12/2025	Chk'd by Z.T.B	Date	App'd by	Date

- Concrete Beam Design (2B-27):

$$\begin{aligned}
W_{DL} &= 150lb/ft \times 3 \times \frac{8''}{12} \times (8.5') (8'' \text{ conc. slab weight at 2nd floor}) \\
&\quad + 35psf \times 8.5' (\text{superimposed dead load at 2nd floor}) \\
&\quad + 75 psf \times 10' (CMU wall at 2nd floor) \\
&\quad + 25 psf \times 16' (\text{superimposed dead load at roof}) = 2297.5 plf \\
&= 2.3 k/ft
\end{aligned}$$

$$\begin{aligned}
W_{LL} &= 40 psf \times 8.5' (\text{Live load at 2nd Floor}) + 30 psf \times 16' (\text{Live load at Roof}) \\
&= 820 plf = 0.82 k/ft
\end{aligned}$$

Use 16"x34" with 4#8 T&B, 2#5 E.F and #3 ties @ 8" O.C.

Please see next page for calculation.

RC BEAM ANALYSIS & DESIGN (ACI318-2019)

In accordance with ACI318-2019

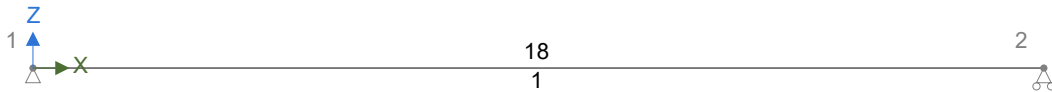
Tedds calculation version 3.3.16

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Concrete (5000 150) - R 16x34



Span	Length (ft)	Section	Start Support	End Support
1	18	R 16x34	Pinned	Roller Pin X

R 16x34: Area 544 in², Inertia Major 52405 in⁴, Inertia Minor 11605 in⁴, Shear area parallel to Minor 453 in², Shear area parallel to Major 453 in²

Concrete (5000 150): Density 150 lbm/ft³, 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	2.3 kips/ft
Beam	Live	UDL	GlobalZ	0.82 kips/ft

Results

Reactions

Load case: Self Weight

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	5.1	0
2	0	5.1	0

Load case: Dead

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	20.7	0
2	0	20.7	0

Load case: Live

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	7.38	0
2	0	7.38	0

Load combination: 1.2D + 1.6L (Strength)

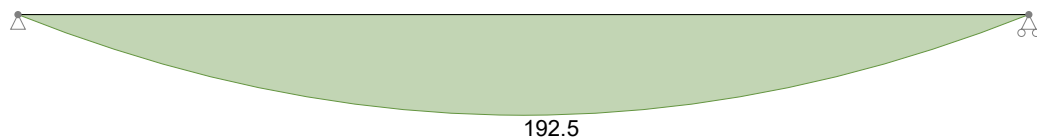
Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	42.768	0
2	0	42.768	0

Load combination: 1.0D + 1.0L (Service)

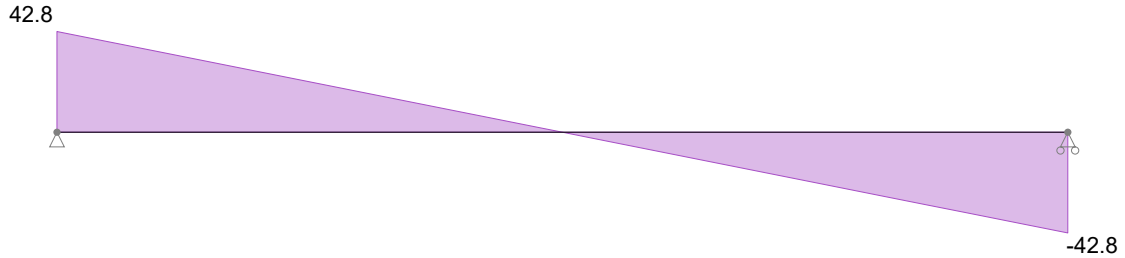
Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	33.18	0
2	0	33.18	0

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 / ft³

Concrete type

Normal weight

Modulus of elasticity of concrete (cl.19.2.2.1)

$E = (w_c / 1 \text{ lb/ft}^3)^{1.5} \times 33 \text{ psi} \times (f'_c / 1 \text{ psi})^{0.5} = 4286826$ psi

Strength reduction factor for shear

$\phi_s = 0.75$

Reinforcement details

Yield strength of reinforcement

$f_y = 60000$ psi

Compression-controlled strain limit (cl.21.2.2.1)

$\epsilon_{ty} = 0.00200$

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

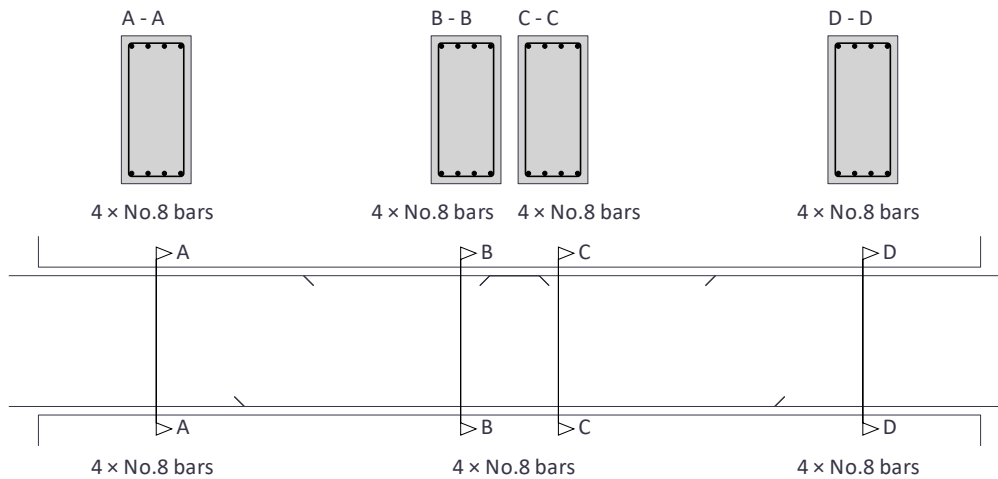
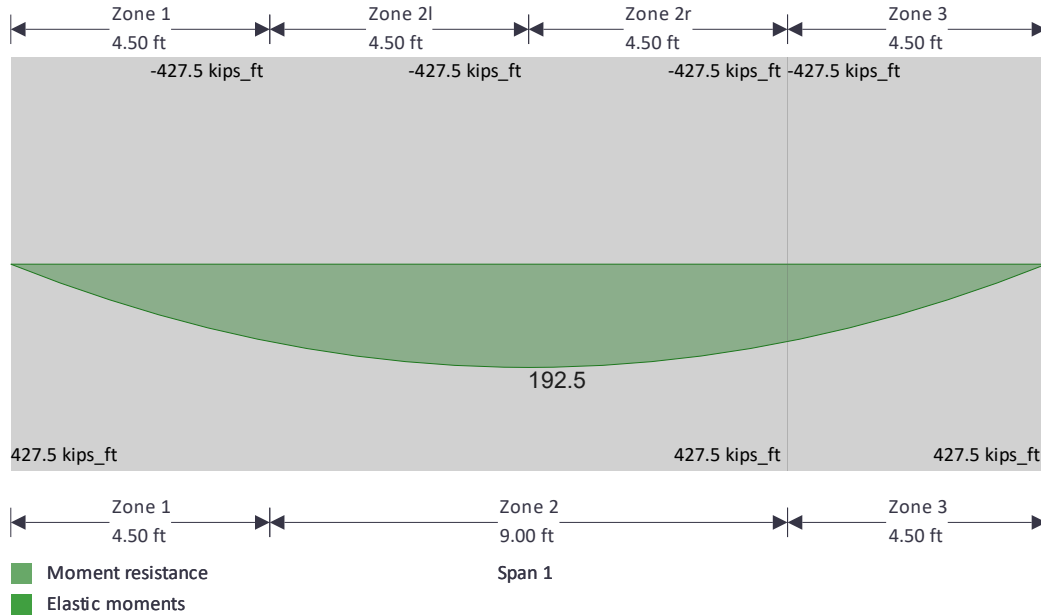
Section width

$b = 16$ in

Section depth

$h = 34$ in

Moment design



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

Factored bending moment at section

$$M_u = \text{abs}(M_{m1_s1_z1_max_red}) = 144.342 \text{ kip_ft}$$

Effective depth to tension reinforcement

$$d = 31.625 \text{ in}$$

Tension reinforcement provided

$$4 \times \text{No.8 bars}$$

Area of tension reinforcement provided

$$A_{s,prov} = 3.142 \text{ in}^2$$

Minimum area of reinforcement (cl.9.6.1.2)

$$A_{s,min} = \max(3 \text{ psi} \times \sqrt{f'_c / 1 \text{ psi}}, 200 \text{ psi}) \times b \times d / f_y = 1.789 \text{ in}^2$$

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

$$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) = 2.772 \text{ in}$$

Depth to neutral axis

$$c = a / \beta_1 = 3.465 \text{ in}$$

Net tensile strain in extreme tension fibers

$$\epsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) = 0.02438$$

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section 2B-27				Sheet no./rev. 5	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1) $\phi_r = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.90}$

Nominal moment strength $M_n = A_{s,prov} \times f_y \times (d - a / 2) = \mathbf{474.993 \text{ kip_ft}}$

Design moment strength $\phi M_n = M_n \times \phi_r = \mathbf{427.494 \text{ kip_ft}}$

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf. $S_{b,max} = S_{bot} + \phi_{m1_s1_z1_b_L1} = \mathbf{3.750 \text{ in}}$

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

Clear cover of reinforcement $C_c = C_{nom_b} + \phi_v = \mathbf{1.875 \text{ in}}$

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

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing $S_{top} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_t_L1} \times N_{m1_s1_z1_t_L1} + \phi_{m1_s1_z2_t_L1} \times N_{m1_s1_z2_t_L1})) / ((N_{m1_s1_z1_t_L1} + N_{m1_s1_z2_t_L1}) - 1) = \mathbf{2.750 \text{ in}}$

Min. allowable top bar clear spacing (cl.25.2.1) $S_{top,min} = \mathbf{1.000 \text{ in}}$

Bottom bar clear spacing $S_{bot} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / (N_{m1_s1_z1_b_L1} - 1) = \mathbf{2.750 \text{ in}}$

Min. allowable bottom bar clear spacing (cl.25.2.1) $S_{bot,min} = \mathbf{1.000 \text{ in}}$

PASS - Actual bar spacing exceeds minimum allowable

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

Factored bending moment at section $M_u = \text{abs}(M_{m1_s1_z2_max_red}) = \mathbf{192.456 \text{ kip_ft}}$

Effective depth to tension reinforcement $d = \mathbf{31.625 \text{ in}}$

Tension reinforcement provided $4 \times \text{No.8 bars}$

Area of tension reinforcement provided $A_{s,prov} = \mathbf{3.142 \text{ in}^2}$

Minimum area of reinforcement (cl.9.6.1.2) $A_{s,min} = \max(3 \text{ psi} \times \sqrt{f'_c / 1 \text{ psi}}, 200 \text{ psi}) \times b \times d / f_y = \mathbf{1.789 \text{ in}^2}$

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

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

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

Depth to neutral axis $c = a / \beta_1 = \mathbf{3.465 \text{ in}}$

Net tensile strain in extreme tension fibers $\epsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) = \mathbf{0.02438}$

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1) $\phi_r = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.90}$

Nominal moment strength $M_n = A_{s,prov} \times f_y \times (d - a / 2) = \mathbf{474.993 \text{ kip_ft}}$

Design moment strength $\phi M_n = M_n \times \phi_r = \mathbf{427.494 \text{ kip_ft}}$

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf. $S_{b,max} = S_{bot} + \phi_{m1_s1_z2_b_L1} = \mathbf{3.750 \text{ in}}$

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

Clear cover of reinforcement $C_c = C_{nom_b} + \phi_v = \mathbf{1.875 \text{ in}}$

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

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section 2B-27				Sheet no./rev. 6	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Control of deflections (Section 24.2)

Concrete density factor

$$K_w = 1.00$$

Reinforcement yield strength factor

$$K_f = 0.4 + f_y / 100000 \text{ psi} = 1.00$$

Minimum thickness of beam (Table 9.3.1.1)

$$h_{min} = (L_{m1_s1} / 16.0) \times K_w \times K_f = 13.5 \text{ in}$$

PASS - Thickness of beam exceeds minimum thickness

Spacing limits for reinforcement

Top bar clear spacing

$$S_{top} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2r_t_L1} \times N_{m1_s1_z2r_t_L1})) / (N_{m1_s1_z2r_t_L1} - 1) = 2.750 \text{ in}$$

Min. allowable top bar clear spacing (cl.25.2.1)

$$S_{top,min} = 1.000 \text{ in}$$

Bottom bar clear spacing

$$S_{bot} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2_b_L1} \times N_{m1_s1_z2_b_L1} + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / ((N_{m1_s1_z2_b_L1} + N_{m1_s1_z1_b_L1}) - 1) = 2.750 \text{ in}$$

Min. allowable bottom bar clear spacing (cl.25.2.1)

$$S_{bot,min} = 1.000 \text{ in}$$

PASS - Actual bar spacing exceeds minimum allowable

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

Factored bending moment at section

$$M_u = \text{abs}(M_{m1_s1_z3_max_red}) = 144.342 \text{ kip_ft}$$

Effective depth to tension reinforcement

$$d = 31.625 \text{ in}$$

Tension reinforcement provided

$$4 \times \text{No.8 bars}$$

Area of tension reinforcement provided

$$A_{s,prov} = 3.142 \text{ in}^2$$

Minimum area of reinforcement (cl.9.6.1.2)

$$A_{s,min} = \max(3 \text{ psi} \times \sqrt{f_c} / 1 \text{ psi}, 200 \text{ psi}) \times b \times d / f_y = 1.789 \text{ in}^2$$

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

$$a = A_{s,prov} \times f_y / (0.85 \times f_c \times b) = 2.772 \text{ in}$$

Depth to neutral axis

$$c = a / \beta_1 = 3.465 \text{ in}$$

Net tensile strain in extreme tension fibers

$$\epsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) = 0.02438$$

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1)

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = 0.90$$

Nominal moment strength

$$M_n = A_{s,prov} \times f_y \times (d - a / 2) = 474.993 \text{ kip_ft}$$

Design moment strength

$$\phi M_n = M_n \times \phi_f = 427.494 \text{ kip_ft}$$

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.

$$S_{b,max} = S_{bot} + \phi_{m1_s1_z3_b_L1} = 3.750 \text{ in}$$

Service load stress in reinforcement (cl.24.3.2)

$$f_s = 2/3 \times f_y = 40000 \text{ psi}$$

Clear cover of reinforcement

$$C_c = C_{nom_b} + \phi_v = 1.875 \text{ in}$$

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

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing

$$S_{top} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_t_L1} \times N_{m1_s1_z3_t_L1} + \phi_{m1_s1_z2r_t_L1} \times N_{m1_s1_z2r_t_L1})) / ((N_{m1_s1_z3_t_L1} + N_{m1_s1_z2r_t_L1}) - 1) = 2.750 \text{ in}$$

Min. allowable top bar clear spacing (cl.25.2.1)

$$S_{top,min} = 1.000 \text{ in}$$

Bottom bar clear spacing

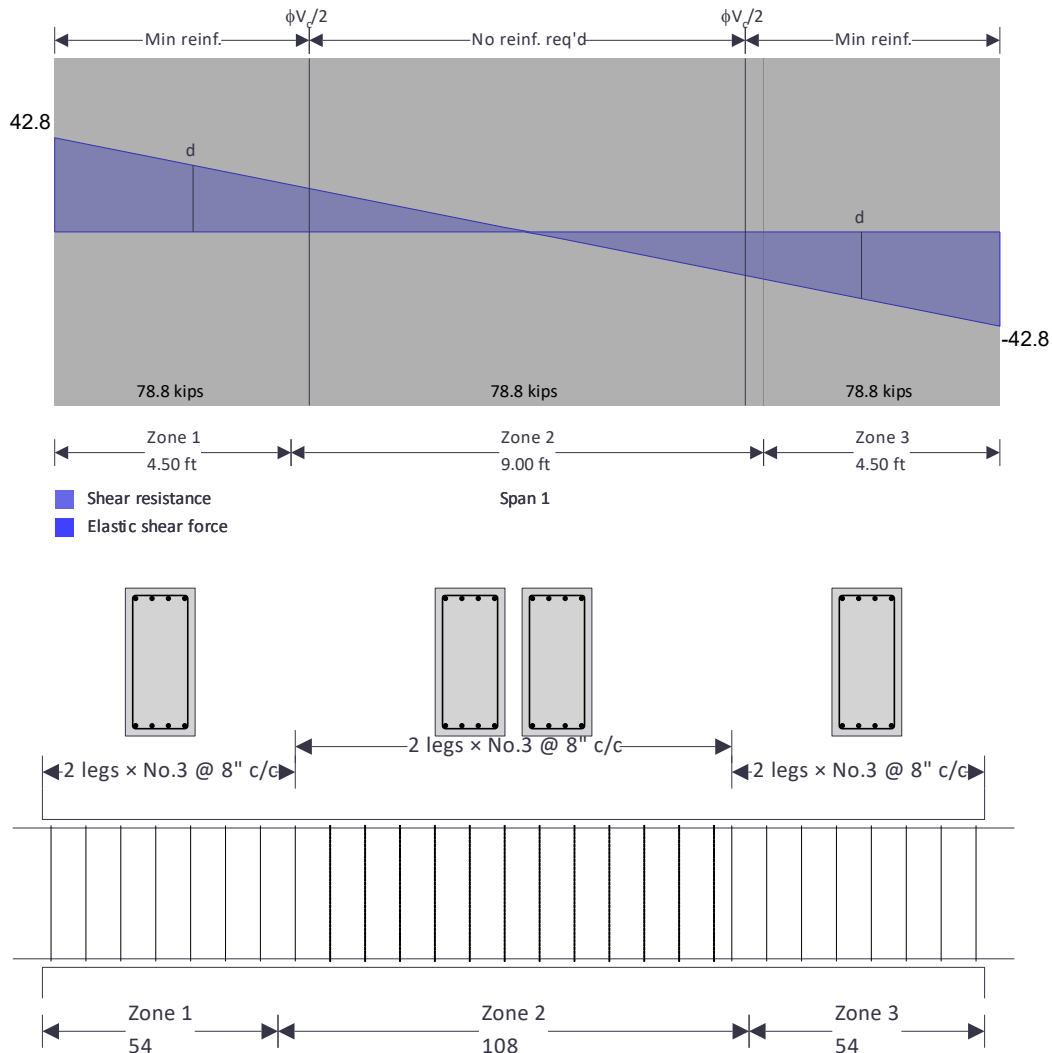
$$S_{bot} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_b_L1} \times N_{m1_s1_z3_b_L1})) / (N_{m1_s1_z3_b_L1} - 1) = 2.750 \text{ in}$$

Min. allowable bottom bar clear spacing (cl.25.2.1)

$$S_{bot,min} = 1.000 \text{ in}$$

PASS - Actual bar spacing exceeds minimum allowable

Shear design



Rectangular section in shear

Concrete weight modification factor

$\lambda = 1.00$

Location where min. reinf. is req'd (V_u less ϕV_c)

Between 0.70 ft and 17.30 ft

Location where no reinf. is req'd (V_u less $\phi V_c / 2$)

Between 4.85 ft and 13.15 ft

Maximum reinforcement shear strength

$\phi V_{s,max} = \phi_s \times 8 \text{ psi} \times \sqrt{(\min(f'_c, 10000 \text{ psi}) / 1 \text{ psi})} \times b \times d = \mathbf{214.678 \text{ kips}}$

Minimum area of shear reinf. (Table 9.6.3.4)

$A_{sv,min} = \max(50 \text{ psi}, 0.75 \text{ psi} \times \sqrt{(f'_c / 1 \text{ psi})}) \times b / \min(f_y, 60000 \text{ psi}) = \mathbf{0.170 \text{ in}^2/\text{ft}}$

Zone 1

Effective depth of long. reinf. used for shear zone

$d = \mathbf{31.625 \text{ in}}$

Size effect factor (cl. 22.5.5.1.3)

$\lambda_s = \min(\sqrt{2 / (1 + (d / 1 \text{ in}) / 10)}), 1.0) = \mathbf{0.69}$

Area of longitudinal reinforcement

$A_{st} = \mathbf{3.142 \text{ in}^2}$

Ratio of longitudinal reinforcement

$\rho_w = A_{st} / (b \times d) = \mathbf{0.006}$

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section 2B-27				Sheet no./rev. 8	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Concrete shear strength (Table 22.5.5.1) $\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{f'_c \times 1 \text{ psi}} \times b \times d = \mathbf{39.457 \text{ kips}}$
Design shear force at 31.6in from support $V_u = \mathbf{30.245 \text{ kips}}$
Reinf. shear strength required (eqn. 22.5.1.1) $\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips}) = \mathbf{0.000 \text{ kips}}$
Area of design shear reinf. req'd (eqn. 22.5.8.5.3) $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d) = \mathbf{0.000 \text{ in}^2/\text{ft}}$
Area of shear reinforcement required $A_{sv,req} = A_{sv,min} = \mathbf{0.170 \text{ in}^2/\text{ft}}$
Shear reinforcement provided $2 \text{ legs} \times \text{No.3 @ } 8" \text{ c/c}$
Area of shear reinforcement provided $A_{sv,prov} = \mathbf{0.331 \text{ in}^2/\text{ft}}$

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

Maximum longitudinal spacing (Table 9.7.6.2.2) $s_{vl,max} = \min(d / 2, 24 \text{ in}) = \mathbf{15.813 \text{ in}}$

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

Zone 2

Effective depth of long. reinf. used for shear zone $d = \mathbf{31.625 \text{ in}}$
Size effect factor (cl. 22.5.5.1.3) $\lambda_s = \min(\sqrt{2 / (1 + (d / 1 \text{ in}) / 10)}), 1.0) = \mathbf{0.69}$
Area of longitudinal reinforcement $A_{st} = \mathbf{3.142 \text{ in}^2}$
Ratio of longitudinal reinforcement $\rho_w = A_{st} / (b \times d) = \mathbf{0.006}$
Concrete shear strength (Table 22.5.5.1) $\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{f'_c \times 1 \text{ psi}} \times b \times d = \mathbf{39.457 \text{ kips}}$
Design shear force within zone $V_u = \mathbf{21.384 \text{ kips}}$
Reinf. shear strength required (eqn. 22.5.1.1) $\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips}) = \mathbf{0.000 \text{ kips}}$
Area of design shear reinf. req'd (eqn. 22.5.8.5.3) $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d) = \mathbf{0.000 \text{ in}^2/\text{ft}}$
Area of shear reinforcement required $A_{sv,req} = A_{sv,min} = \mathbf{0.170 \text{ in}^2/\text{ft}}$
Shear reinforcement provided $2 \text{ legs} \times \text{No.3 @ } 8" \text{ c/c}$
Area of shear reinforcement provided $A_{sv,prov} = \mathbf{0.331 \text{ in}^2/\text{ft}}$

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

Maximum longitudinal spacing (Table 9.7.6.2.2) $s_{vl,max} = \min(d / 2, 24 \text{ in}) = \mathbf{15.813 \text{ in}}$

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

Zone 3

Effective depth of long. reinf. used for shear zone $d = \mathbf{31.625 \text{ in}}$
Size effect factor (cl. 22.5.5.1.3) $\lambda_s = \min(\sqrt{2 / (1 + (d / 1 \text{ in}) / 10)}), 1.0) = \mathbf{0.69}$
Area of longitudinal reinforcement $A_{st} = \mathbf{3.142 \text{ in}^2}$
Ratio of longitudinal reinforcement $\rho_w = A_{st} / (b \times d) = \mathbf{0.006}$
Concrete shear strength (Table 22.5.5.1) $\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{f'_c \times 1 \text{ psi}} \times b \times d = \mathbf{39.457 \text{ kips}}$
Design shear force at 31.6in from support $V_u = \mathbf{30.245 \text{ kips}}$
Reinf. shear strength required (eqn. 22.5.1.1) $\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips}) = \mathbf{0.000 \text{ kips}}$
Area of design shear reinf. req'd (eqn. 22.5.8.5.3) $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d) = \mathbf{0.000 \text{ in}^2/\text{ft}}$
Area of shear reinforcement required $A_{sv,req} = A_{sv,min} = \mathbf{0.170 \text{ in}^2/\text{ft}}$
Shear reinforcement provided $2 \text{ legs} \times \text{No.3 @ } 8" \text{ c/c}$
Area of shear reinforcement provided $A_{sv,prov} = \mathbf{0.331 \text{ in}^2/\text{ft}}$

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

Maximum longitudinal spacing (Table 9.7.6.2.2) $s_{vl,max} = \min(d / 2, 24 \text{ in}) = \mathbf{15.813 \text{ in}}$

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

Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project Herrera Res.				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 05/12/2025	Chk'd by Z.T.B	Date	App'd by	Date

- Concrete Beam Design (2B-29):

$$\begin{aligned}
W_{DL} &= 150lb/ft \times \frac{8"}{12} \times (4') (8" \text{ conc. slab weight at 2nd floor}) \\
&\quad + 35 \text{ psf} \times 4' (\text{superimposed dead load at 2nd floor}) = 540 \text{ plf} \\
&= 0.54 \text{ k/ft}
\end{aligned}$$

$$W_{LL} = 40 \text{ psf} \times 4' = 160 \text{ plf} = 0.16 \text{ k/ft}$$

Use 8"x16" with 2#8 T&B and #3 ties @ 6" O.C.

Please see next page for calculation.

RC BEAM ANALYSIS & DESIGN (ACI318-2019)

In accordance with ACI318-2019

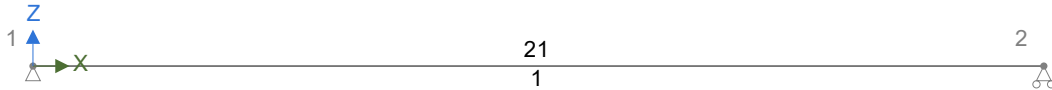
Tedds calculation version 3.3.16

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Concrete (5000 150) - R 8x16



Span	Length (ft)	Section	Start Support	End Support
1	21	R 8x16	Pinned	Roller Pin X
R 8x16: Area 128 in ² , Inertia Major 2731 in ⁴ , Inertia Minor 683 in ⁴ , Shear area parallel to Minor 107 in ² , Shear area parallel to Major 107 in ²				
Concrete (5000 150): Density 150 lbm/ft ³ , Youngs 4287 ksi, Shear 1750 ksi, Thermal 0.00001 °C ⁻¹				

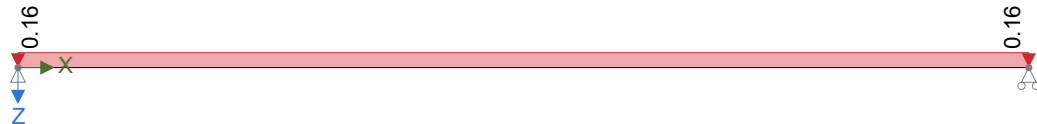
Loading

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	0.54 kips/ft
Beam	Live	UDL	GlobalZ	0.16 kips/ft

Results

Reactions

Load case: Self Weight

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	1.4	0
2	0	1.4	0

Load case: Dead

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	5.67	0
2	0	5.67	0

Load case: Live

Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	1.68	0
2	0	1.68	0

Load combination: 1.2D + 1.6L (Strength)

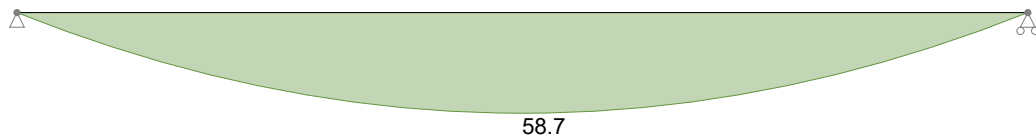
Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	11.172	0
2	0	11.172	0

Load combination: 1.0D + 1.0L (Service)

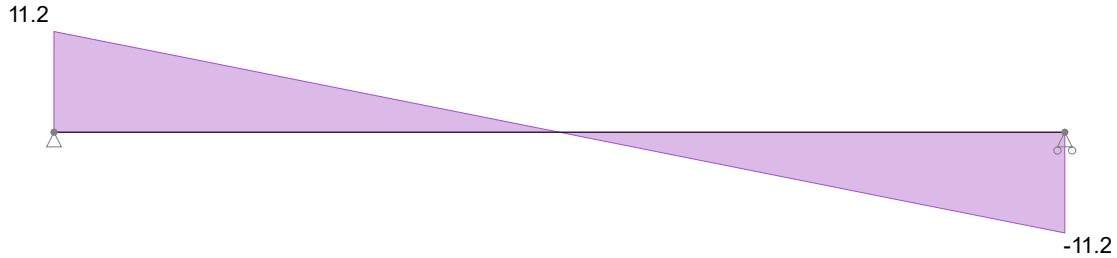
Node	Force		Moment My (kip_ft)
	Fx (kips)	Fz (kips)	
1	0	8.75	0
2	0	8.75	0

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 / ft³

Concrete type

Normal weight

Modulus of elasticity of concrete (cl.19.2.2.1)

$E = (w_c / 1 \text{ lb/ft}^3)^{1.5} \times 33 \text{ psi} \times (f'_c / 1 \text{ psi})^{0.5} = 4286826$ psi

Strength reduction factor for shear

$\phi_s = 0.75$

Reinforcement details

Yield strength of reinforcement

$f_y = 60000$ psi

Compression-controlled strain limit (cl.21.2.2.1)

$\epsilon_{ty} = 0.00200$

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

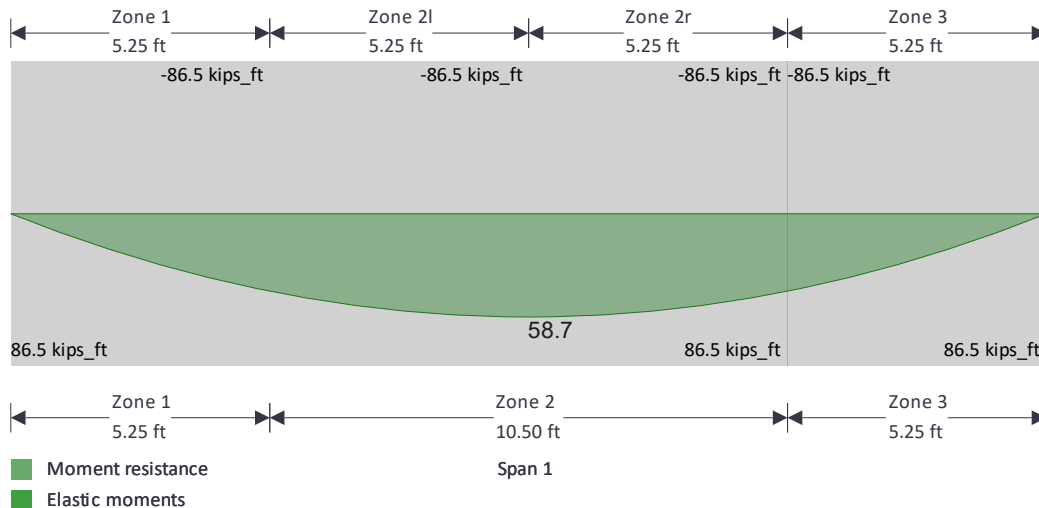
Section width

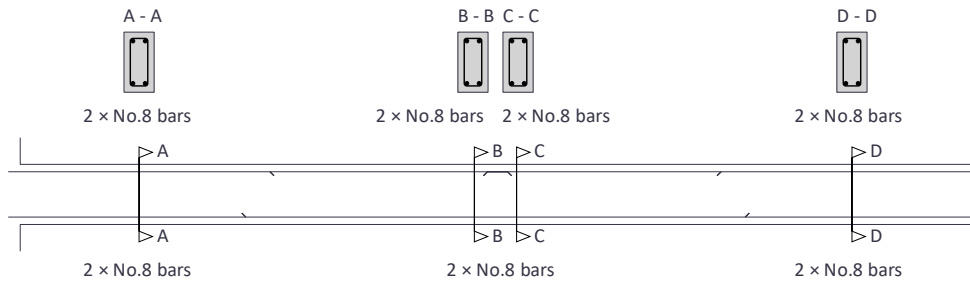
$b = 8$ in

Section depth

$h = 16$ in

Moment design





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

Factored bending moment at section

$$M_u = \text{abs}(M_{m1_s1_z1_max_red}) = 43.990 \text{ kip_ft}$$

Effective depth to tension reinforcement

$$d = 13.625 \text{ in}$$

Tension reinforcement provided

$$2 \times \text{No.8 bars}$$

Area of tension reinforcement provided

$$A_{s,prov} = 1.571 \text{ in}^2$$

Minimum area of reinforcement (cl.9.6.1.2)

$$A_{s,min} = \max(3 \text{ psi} \times \sqrt{f'_c} / 1 \text{ psi}), 200 \text{ psi}) \times b \times d / f_y = 0.385 \text{ in}^2$$

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

$$a = A_{s,prov} \times f_y / (0.85 \times f'_c \times b) = 2.772 \text{ in}$$

Depth to neutral axis

$$c = a / \beta_1 = 3.465 \text{ in}$$

Net tensile strain in extreme tension fibers

$$\epsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) = 0.0088$$

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1)

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = 0.90$$

Nominal moment strength

$$M_n = A_{s,prov} \times f_y \times (d - a / 2) = 96.125 \text{ kip_ft}$$

Design moment strength

$$\phi M_n = M_n \times \phi_f = 86.512 \text{ kip_ft}$$

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.

$$S_{b,max} = S_{bot} + \phi_{m1_s1_z1_b_L1} = 3.250 \text{ in}$$

Service load stress in reinforcement (cl.24.3.2)

$$f_s = 2/3 \times f_y = 40000 \text{ psi}$$

Clear cover of reinforcement

$$C_c = C_{nom_b} + \phi_v = 1.875 \text{ in}$$

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

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing

$$S_{top} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_t_L1} \times N_{m1_s1_z1_t_L1} + \phi_{m1_s1_z2_t_L1} \times N_{m1_s1_z2_t_L1})) / ((N_{m1_s1_z1_t_L1} + N_{m1_s1_z2_t_L1}) - 1) = 2.250 \text{ in}$$

Min. allowable top bar clear spacing (cl.25.2.1)

$$S_{top,min} = 1.000 \text{ in}$$

Bottom bar clear spacing

$$S_{bot} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / (N_{m1_s1_z1_b_L1} - 1) = 2.250 \text{ in}$$

Min. allowable bottom bar clear spacing (cl.25.2.1)

$$S_{bot,min} = 1.000 \text{ in}$$

PASS - Actual bar spacing exceeds minimum allowable

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

Factored bending moment at section

$$M_u = \text{abs}(M_{m1_s1_z2_max_red}) = 58.653 \text{ kip_ft}$$

Effective depth to tension reinforcement

$$d = 13.625 \text{ in}$$

Tension reinforcement provided

$$2 \times \text{No.8 bars}$$

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section 2B-29				Sheet no./rev. 5	
	Calc. by H.J	Date 5/13/2025	Chk'd by	Date	App'd by	Date

Area of tension reinforcement provided	$A_{s,prov} = 1.571 \text{ in}^2$
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = \max(3 \text{ psi} \times \sqrt{f_c / 1 \text{ psi}}, 200 \text{ psi}) \times b \times d / f_y = 0.385 \text{ in}^2$
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	$a = A_{s,prov} \times f_y / (0.85 \times f_c \times b) = 2.772 \text{ in}$
Depth to neutral axis	$c = a / \beta_1 = 3.465 \text{ in}$
Net tensile strain in extreme tension fibers	$\epsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) = 0.0088$
Net tensile strain in tension controlled zone	
Strength reduction factor (cl.21.2.1)	$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = 0.90$
Nominal moment strength	$M_n = A_{s,prov} \times f_y \times (d - a / 2) = 96.125 \text{ kip_ft}$
Design moment strength	$\phi M_n = M_n \times \phi_f = 86.512 \text{ kip_ft}$
PASS - Required moment strength is less than design moment strength	
Flexural cracking	
Max. center to center spacing of tension reinf.	$S_{b,max} = S_{bot} + \phi_{m1_s1_z2_b_L1} = 3.250 \text{ in}$
Service load stress in reinforcement (cl.24.3.2)	$f_s = 2/3 \times f_y = 40000 \text{ psi}$
Clear cover of reinforcement	$C_c = C_{nom_b} + \phi_v = 1.875 \text{ in}$
Maximum allowable bot bar spacing (Table 24.3.2)	$s_{max} = \min(15 \text{ in} \times 40000 \text{ psi} / f_s - 2.5 \times C_c, 12 \text{ in} \times 40000 \text{ psi} / f_s) = 10.313 \text{ in}$
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	$K_f = 0.4 + f_y / 100000 \text{ psi} = 1.00$
Minimum thickness of beam (Table 9.3.1.1)	$h_{min} = (L_{m1_s1} / 16.0) \times K_w \times K_f = 15.75 \text{ in}$
PASS - Thickness of beam exceeds minimum thickness	
Spacing limits for reinforcement	
Top bar clear spacing	$Stop = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2r_t_L1} \times N_{m1_s1_z2r_t_L1})) / (N_{m1_s1_z2r_t_L1} - 1) = 2.250 \text{ in}$
Min. allowable top bar clear spacing (cl.25.2.1)	$Stop_{min} = 1.000 \text{ in}$
Bottom bar clear spacing	$S_{bot} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2_b_L1} \times N_{m1_s1_z2_b_L1} + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / ((N_{m1_s1_z2_b_L1} + N_{m1_s1_z1_b_L1}) - 1) = 2.250 \text{ in}$
Min. allowable bottom bar clear spacing (cl.25.2.1)	$S_{bot,min} = 1.000 \text{ in}$
PASS - Actual bar spacing exceeds minimum allowable	
Zone 3 - Positive moment. Rectangular section in flexure (Section 9.5.2)	
Factored bending moment at section	$M_u = \text{abs}(M_{m1_s1_z3_max_red}) = 43.990 \text{ kip_ft}$
Effective depth to tension reinforcement	$d = 13.625 \text{ in}$
Tension reinforcement provided	$2 \times \text{No.8 bars}$
Area of tension reinforcement provided	$A_{s,prov} = 1.571 \text{ in}^2$
Minimum area of reinforcement (cl.9.6.1.2)	$A_{s,min} = \max(3 \text{ psi} \times \sqrt{f_c / 1 \text{ psi}}, 200 \text{ psi}) \times b \times d / f_y = 0.385 \text{ in}^2$
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	$a = A_{s,prov} \times f_y / (0.85 \times f_c \times b) = 2.772 \text{ in}$
Depth to neutral axis	$c = a / \beta_1 = 3.465 \text{ in}$
Net tensile strain in extreme tension fibers	$\epsilon_t = 0.003 \times (d_o - c) / \max(c, 0.001 \text{ in}) = 0.0088$

Net tensile strain in tension controlled zone

Strength reduction factor (cl.21.2.1)

$$\phi_r = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.90}$$

Nominal moment strength

$$M_n = A_{s,prov} \times f_y \times (d - a / 2) = \mathbf{96.125 \text{ kip_ft}}$$

Design moment strength

$$\phi M_n = M_n \times \phi_r = \mathbf{86.512 \text{ kip_ft}}$$

PASS - Required moment strength is less than design moment strength

Flexural cracking

Max. center to center spacing of tension reinf.

$$S_{b,max} = S_{bot} + \phi_{m1_s1_z3_b_L1} = \mathbf{3.250 \text{ in}}$$

Service load stress in reinforcement (cl.24.3.2)

$$f_s = 2/3 \times f_y = \mathbf{40000 \text{ psi}}$$

Clear cover of reinforcement

$$C_c = C_{nom_b} + \phi_v = \mathbf{1.875 \text{ in}}$$

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

PASS - Maximum allowable tension reinforcement spacing exceeds actual spacing

Spacing limits for reinforcement

Top bar clear spacing

$$S_{top} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_t_L1} \times N_{m1_s1_z3_t_L1} + \phi_{m1_s1_z2r_t_L1} \times N_{m1_s1_z2r_t_L1})) / ((N_{m1_s1_z3_t_L1} + N_{m1_s1_z2r_t_L1}) - 1) = \mathbf{2.250 \text{ in}}$$

Min. allowable top bar clear spacing (cl.25.2.1)

$$S_{top,min} = \mathbf{1.000 \text{ in}}$$

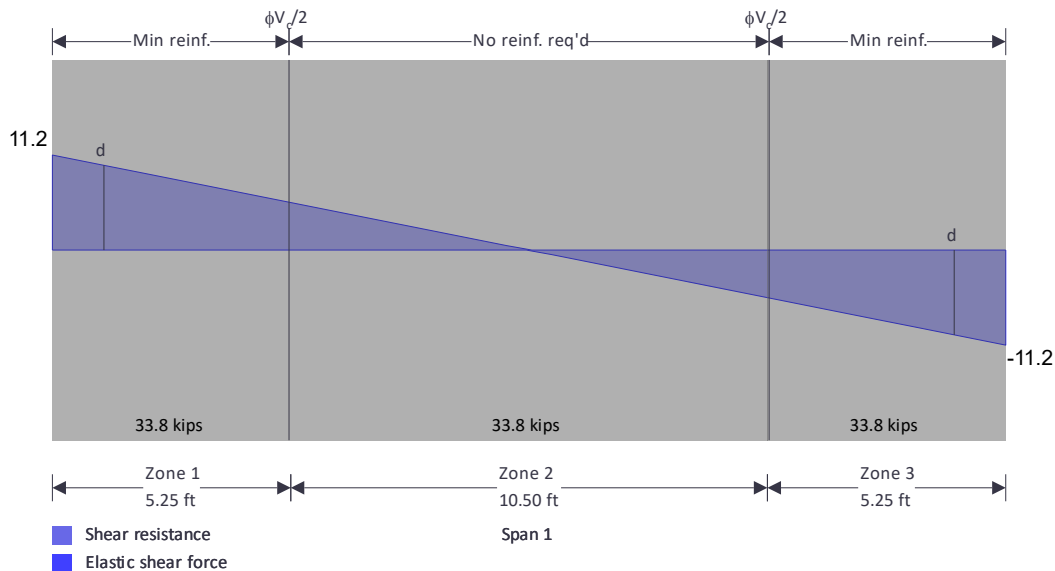
Bottom bar clear spacing

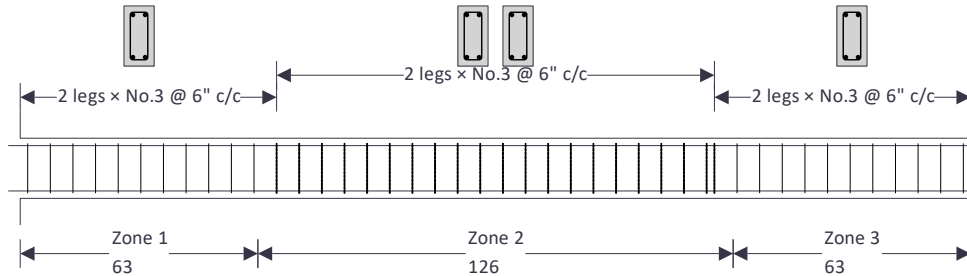
$$S_{bot} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_b_L1} \times N_{m1_s1_z3_b_L1})) / (N_{m1_s1_z3_b_L1} - 1) = \mathbf{2.250 \text{ in}}$$

Min. allowable bottom bar clear spacing (cl.25.2.1) $S_{bot,min} = \mathbf{1.000 \text{ in}}$

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Rectangular section in shear

Concrete weight modification factor

$$\lambda = 1.00$$

Location where min. reinf. is req'd (V_u less ϕV_c)

N/A no reinf. required along full length

Location where no reinf. is req'd (V_u less $\phi V_c / 2$)

Between 5.21 ft and 15.79 ft

Maximum reinforcement shear strength

$$\phi V_{s,max} = \phi_s \times 8 \text{ psi} \times \sqrt{(\min(f_c, 10000 \text{ psi}) / 1 \text{ psi})} \times b \times d = \mathbf{46.245 \text{ kips}}$$

Minimum area of shear reinf. (Table 9.6.3.4)

$$A_{sv,min} = \max(50 \text{ psi}, 0.75 \text{ psi} \times \sqrt{(f_c / 1 \text{ psi})}) \times b / \min(f_y, 60000 \text{ psi}) = \mathbf{0.085 \text{ in}^2/\text{ft}}$$

Zone 1

Effective depth of long. reinf. used for shear zone

$$d = \mathbf{13.625 \text{ in}}$$

Size effect factor (cl. 22.5.5.1.3)

$$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1 \text{ in}) / 10))}, 1.0) = \mathbf{0.92}$$

Area of longitudinal reinforcement

$$A_{st} = \mathbf{1.571 \text{ in}^2}$$

Ratio of longitudinal reinforcement

$$\rho_w = A_{st} / (b \times d) = \mathbf{0.014}$$

Concrete shear strength (Table 22.5.5.1)

$$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f_c \times 1 \text{ psi})} \times b \times d = \mathbf{11.254 \text{ kips}}$$

Design shear force at 13.6in from support

$$V_u = \mathbf{9.964 \text{ kips}}$$

Reinf. shear strength required (eqn. 22.5.1.1)

$$\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips}) = \mathbf{0.000 \text{ kips}}$$

Area of design shear reinf. req'd (eqn. 22.5.8.5.3)

$$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d) = \mathbf{0.000 \text{ in}^2/\text{ft}}$$

Area of shear reinforcement required

$$A_{sv,req} = A_{sv,min} = \mathbf{0.085 \text{ in}^2/\text{ft}}$$

Shear reinforcement provided

$$2 \text{ legs} \times \text{No.3 @ } 6" \text{ c/c}$$

Area of shear reinforcement provided

$$A_{sv,prov} = \mathbf{0.442 \text{ in}^2/\text{ft}}$$

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

Maximum longitudinal spacing (Table 9.7.6.2.2)

$$s_{vl,max} = \min(d / 2, 24 \text{ in}) = \mathbf{6.813 \text{ in}}$$

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

Zone 2

Effective depth of long. reinf. used for shear zone

$$d = \mathbf{13.625 \text{ in}}$$

Size effect factor (cl. 22.5.5.1.3)

$$\lambda_s = \min(\sqrt{(2 / (1 + (d / 1 \text{ in}) / 10))}, 1.0) = \mathbf{0.92}$$

Area of longitudinal reinforcement

$$A_{st} = \mathbf{1.571 \text{ in}^2}$$

Ratio of longitudinal reinforcement

$$\rho_w = A_{st} / (b \times d) = \mathbf{0.014}$$

Concrete shear strength (Table 22.5.5.1)

$$\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{(f_c \times 1 \text{ psi})} \times b \times d = \mathbf{11.254 \text{ kips}}$$

Design shear force within zone

$$V_u = \mathbf{5.586 \text{ kips}}$$

Reinf. shear strength required (eqn. 22.5.1.1)

$$\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips}) = \mathbf{0.000 \text{ kips}}$$

Area of design shear reinf. req'd (eqn. 22.5.8.5.3)

$$A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d) = \mathbf{0.000 \text{ in}^2/\text{ft}}$$

Area of shear reinforcement required

$$A_{sv,req} = 0 \text{ mm}^2/\text{ft} = \mathbf{0.000 \text{ in}^2/\text{ft}}$$

Shear reinforcement provided

$$2 \text{ legs} \times \text{No.3 @ } 6" \text{ c/c}$$

Area of shear reinforcement provided

$$A_{sv,prov} = \mathbf{0.442 \text{ in}^2/\text{ft}}$$

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section 2B-29				Sheet no./rev. 8	
	Calc. by H.J	Date 5/13/2025	Chk'd by	Date	App'd by	Date

PASS - No shear reinforcement required ($\phi V_c/2 \geq V_u$)

Zone 3

Effective depth of long. reinf. used for shear zone $d = 13.625$ in

Size effect factor (cl. 22.5.5.1.3) $\lambda_s = \min(\sqrt{2 / (1 + (d / 1 \text{ in}) / 10)}), 1.0) = 0.92$

Area of longitudinal reinforcement $A_{st} = 1.571 \text{ in}^2$

Ratio of longitudinal reinforcement $\rho_w = A_{st} / (b \times d) = 0.014$

Concrete shear strength (Table 22.5.5.1) $\phi V_c = \phi_s \times \min(8 \times \lambda \times (\rho_w)^{1/3}, 5 \times \lambda) \times \sqrt{f'_c \times 1 \text{ psi}} \times b \times d = 11.254 \text{ kips}$

Design shear force at 13.6in from support $V_u = 9.964 \text{ kips}$

Reinf. shear strength required (eqn. 22.5.1.1) $\phi V_s = \max(V_u - \phi V_c, 0 \text{ kips}) = 0.000 \text{ kips}$

Area of design shear reinf. req'd (eqn. 22.5.8.5.3) $A_{sv,des} = \phi V_s / (\phi_s \times \min(f_y, 60000 \text{ psi}) \times d) = 0.000 \text{ in}^2/\text{ft}$

Area of shear reinforcement required $A_{sv,req} = A_{sv,min} = 0.085 \text{ in}^2/\text{ft}$

Shear reinforcement provided $2 \text{ legs} \times \text{No.3 @ } 6" \text{ c/c}$

Area of shear reinforcement provided $A_{sv,prov} = 0.442 \text{ in}^2/\text{ft}$

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

Maximum longitudinal spacing (Table 9.7.6.2.2) $s_{vl,max} = \min(d / 2, 24 \text{ in}) = 6.813 \text{ in}$

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

Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project Herrera Res.				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 05/12/2025	Chk'd by Z.T.B	Date	App'd by	Date

• Concrete Column Design (C3) at Grid line 3 & Between F, G:

$$\begin{aligned}
P_{DL} &= 150 \frac{lb}{ft^3} \times \frac{8"}{12} \times 280 sf (8" conc. slab weight at 2nd floor) \\
&\quad + 35 psf \times 280 sf (superimposed dead load at 2nd floor) \\
&\quad + 25 psf \times 210 sf (Superimposed load at Roof) \\
&\quad + 75 psf \times 10' \times 13' (CMU wall at 2nd floor) = 52800 lbs = 52.8 kips
\end{aligned}$$

$$\begin{aligned}
P_{LL} &= 40 psf \times 280 sf (live load at 2nd floor) + 30 psf \times 210 sf (live load at Roof) \\
&= 17500 lbs = 17.5 kips
\end{aligned}$$

$$P_u = 1.2P_{DL} + 1.6P_{LL} = 1.2 \times 52.8 + 1.6 \times 17.5 = 91.36 kips \approx 92 kips$$

H=13' (Height of Column)

Use 8"x18" with 6#5 Vert. bar and #3 ties @ 8" O.C.

Please see next page for calculation.

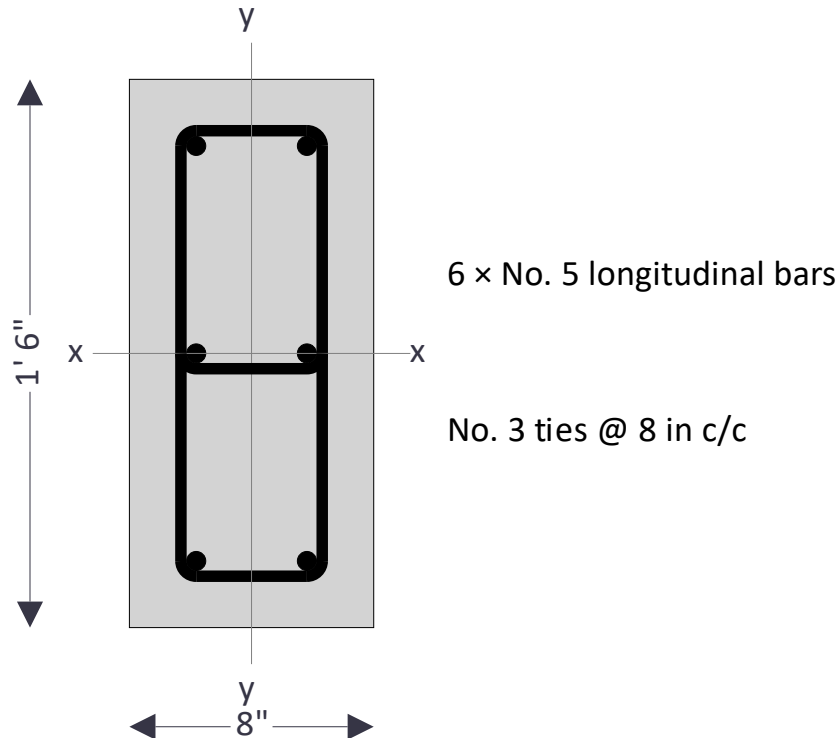
RC COLUMN DESIGN

In accordance with ACI318-19

Tedds calculation version 2.2.04

Design summary

Description	Unit	Capacity	Applied	Utilization	Result
For ultimate axial load 92.0 kips:					PASS
Bending x axis	kip_ft	108.7	29.1	0.268	PASS
Bending y axis	kip_ft	38.0	8.5	0.223	PASS



Applied loads

Ultimate axial force acting on column

$$P_{u_act} = 92 \text{ kips}$$

Ultimate smaller end moment about x axis

$$M_{1x_act} = 1 \text{ kips_ft}$$

Ultimate larger end moment about x axis

$$M_{2x_act} = 2 \text{ kips_ft}$$

Column curvature about x axis

single curvature

Ultimate smaller end moment about y axis

$$M_{1y_act} = 1 \text{ kips_ft}$$

Ultimate larger end moment about y axis

$$M_{2y_act} = 2 \text{ kips_ft}$$

Column curvature about y axis

single curvature

Ratio of DL moment to total moment

$$\beta_d = 0.600$$

Contour beta factor

$$\beta = 0.500$$

Geometry of column

Depth of column (larger dimension of column)

$$h = 18.0 \text{ in}$$

Width of column (smaller dimension of column)

$$b = 8.0 \text{ in}$$

Clear cover to reinforcement (both sides)

$$c_c = 1.5 \text{ in}$$

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section Concrete Column Design-C3				Sheet no./rev. 2	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Unsupported height of column about x axis $l_{ux} = 13.0$ ft
 Effective height factor about x axis $k_x = 1.00$
 Column state about the x axis Braced
 Unsupported height of column about y axis $l_{uy} = 13.0$ ft
 Effective height factor about y axis $k_y = 1.00$
 Column state about the y axis Braced

Check on overall column dimensions

Column dimensions are OK - $h < 4b$

Reinforcement of column

Numbers of bars of longitudinal steel $N = 6$
 Longitudinal steel bar diameter number $D_{bar_num} = 5$
 Diameter of longitudinal bar $D_{long} = 0.625$ in
 Stirrup bar diameter number $D_{stir_num} = 3$
 Diameter of stirrup bar $D_{stir} = 0.375$ in
 Specified yield strength of reinforcement $f_y = 60000$ psi
 Specified compressive strength of concrete $f'_c = 5000$ psi
 Modulus of elasticity of bar reinforcement $E_s = 29 \times 10^6$ psi
 Modulus of elasticity of concrete $E_c = 57000 \times f'_c{}^{1/2} \times (1\text{psi})^{1/2} = 4030509$ psi
 Yield strain $\epsilon_y = f_y / E_s = 0.00207$
 Ultimate design strain $\epsilon_c = 0.003$ in/in

Check for minimum area of steel - 10.6.1.1

Gross area of column $A_g = h \times b = 144.000$ in²
 Area of steel $A_{st} = N \times (\pi \times D_{long}^2) / 4 = 1.841$ in²
 Minimum area of steel required $A_{st_min} = 0.01 \times A_g = 1.440$ in²

$A_{st} > A_{st_min}$, PASS- Minimum steel check

Check for maximum area of steel - 10.6.1.1

Permissible maximum area of steel $A_{st_max} = 0.08 \times A_g = 11.520$ in²

$A_{st} < A_{st_max}$, PASS - Maximum steel check

Slenderness check about x axis

Radius of gyration $r_x = 0.3 \times h = 5.4$ in
 Actual slenderness ratio $s_{rx_act} = k_x \times l_{ux} / r_x = 28.89$
 Permissible slenderness ratio $s_{rx_perm} = \min(34 - 12 \times (M_{1x_act} / M_{2x_act}), 40) = 28$

Column is slender about the X axis

Magnified moments about x axis

Moment of inertia of section $I_{gx} = (b \times h^3) / 12 = 3888$ in⁴
 Euler's buckling load $P_{cx} = (\pi^2 / (k_x \times l_{ux})^2) \times (0.4 \times E_c \times I_{gx} / (1 + \beta_d)) = 1588.83$ kips
 Correction factor for actual to equiv. mmt.diagram $C_{mx} = 0.6 + (0.4 \times M_{1x_act} / M_{2x_act}) = 0.800$
 Moment magnifier $\delta_{nsx} = \max(C_{mx} / (1 - (P_{u_act} / (0.75 \times P_{cx}))), 1.0) = 1$
 Magnified moment about x axis $M_{cx} = \delta_{nsx} \times M_{2x_act} = 2$ kip_ft
 Minimum factored moment about x axis $M_{2x_min} = P_{u_act} \times (0.6 \text{ in} + 0.03 \times h) = 8.74$ kip_ft
 Minimum magnified moment about x axis $M_{cx_min} = \delta_{nsx} \times M_{2x_min} = 8.74$ kip_ft

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section Concrete Column Design-C3				Sheet no./rev. 3	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Slenderness check about y axis

Radius of gyration

$$r_y = 0.3 \times b = \mathbf{2.4 \text{ in}}$$

Actual slenderness ratio

$$S_{ry_act} = k_y \times l_{uy} / r_y = \mathbf{65}$$

Permissible slenderness ratio

$$S_{ry_perm} = \min(34 - 12 \times (M_{1y_act} / M_{2y_act}), 40) = \mathbf{28}$$

Column is slender about the Y axis

Magnified moments about y axis

Moment of inertia of section

$$I_{gy} = (h \times b^3) / 12 = \mathbf{768 \text{ in}^4}$$

Euler's buckling load

$$P_{cy} = (\pi^2 / (k_y \times l_{uy})^2) \times (0.4 \times E_c \times I_{gy} / (1 + \beta_d)) = \mathbf{313.84 \text{ kips}}$$

Correction factor for actual to equiv. mmt.diagram

$$C_{my} = 0.6 + (0.4 \times M_{1y_act} / M_{2y_act}) = \mathbf{0.800}$$

Moment magnifier

$$\delta_{nsy} = \max(C_{my} / (1 - (P_{u_act} / (0.75 \times P_{cy}))), 1.0) = \mathbf{1.313}$$

Magnified moment about y axis

$$M_{cy} = \delta_{nsy} \times M_{2y_act} = \mathbf{2.63 \text{ kip_ft}}$$

Minimum factored moment about y axis

$$M_{2y_min} = P_{u_act} \times (0.6 \text{ in} + 0.03 \times b) = \mathbf{6.44 \text{ kip_ft}}$$

Minimum magnified moment about y axis

$$M_{cy_min} = \delta_{nsy} \times M_{2y_min} = \mathbf{8.46 \text{ kip_ft}}$$

Axial load capacity of axially loaded column

Strength reduction factor

$$\phi = 0.65$$

Area of steel on compression face

$$A'_s = A_{st} / 2 = \mathbf{0.920 \text{ in}^2}$$

Area of steel on tension face

$$A_s = A_{st} / 2 = \mathbf{0.920 \text{ in}^2}$$

Net axial load capacity of column

$$P_n = 0.8 \times (0.85 \times f'_c \times (A_g - A_{st}) + f_y \times A_{st}) = \mathbf{571.699 \text{ kips}}$$

Ultimate axial load capacity of column

$$P_u = \phi \times P_n = \mathbf{371.604 \text{ kips}}$$

PASS : Column is safe in axial loading

Net moments for biaxial column

Assuming strength reduction factor

$$\phi = 0.65$$

Net moment about major (X) axis

$$M_{nx} = M_{cx_min} / \phi = \mathbf{13.45 \text{ kips_ft}}$$

Net moment about minor (Y) axis

$$M_{ny} = M_{cy} / \phi = \mathbf{4.04 \text{ kips_ft}}$$

Uniaxially loaded column about major axis

Details of column cross-section

c/d_t ratio

$$r_{xb} = \mathbf{0.340}$$

Effective cover to reinforcement

$$d' = c_c + D_{stir} + (D_{long}/2) = \mathbf{2.188 \text{ in}}$$

Spacing between bars

$$s = ((h - (2 \times d')) / ((N_y) - 1)) = \mathbf{6.813 \text{ in}}$$

Depth of tension steel

$$d_t = h - d' = \mathbf{15.813 \text{ in}}$$

Depth of NA from extreme compression face

$$c_x = r_{xb} \times d_t = \mathbf{5.371 \text{ in}}$$

Factor of depth of compressive stress block

$$\beta_1 = \mathbf{0.800}$$

Depth of equivalent rectangular stress block

$$a_x = \min((\beta_1 \times c_x), h) = \mathbf{4.297 \text{ in}}$$

Yield strain in steel

$$\epsilon_{sx} = f_y / E_s = \mathbf{0.002}$$

Strength reduction factor

$$\phi_x = \mathbf{0.900}$$

Details of concrete block

Force carried by concrete

Forces carried by concrete

$$P_{xcon} = 0.85 \times f'_c \times b \times a_x = \mathbf{146.082 \text{ kips}}$$

Moment carried by concrete

Moment carried by concrete

$$M_{xcon} = P_{xcon} \times ((h/2) - (a_x/2)) = \mathbf{83.410 \text{ kip_ft}}$$

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section Concrete Column Design-C3				Sheet no./rev. 4	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Details of steel layer 1

Depth of layer	$x_{x1} = 2.188$ in
Strain of layer	$\epsilon_{x1} = \epsilon_c \times (1 - x_{x1} / c_x) = 0.00178$
Stress in layer	$\sigma_{x1} = \min(f_y, E_s \times \epsilon_{x1}) - 0.85 \times f'_c = 47314.39$ psi
Force carried by layer	$P_{x1} = N_x \times A_{bar} \times \sigma_{x1} = 29.032$ kips
Moment carried by steel layer	$M_{x1} = P_{x1} \times ((h / 2) - x_{x1}) = 16.482$ kip_ft

Details of steel layer 2

Depth of layer	$x_{x2} = 9.000$ in
Strain of layer	$\epsilon_{x2} = \epsilon_c \times (1 - x_{x2} / c_x) = -0.00203$
Stress in layer	$\sigma_{x2} = \max(-1 \times f_y, E_s \times \epsilon_{x2}) = -58792.21$ psi
Force carried by layer	$P_{x2} = 2 \times A_{bar} \times \sigma_{x2} = -36.074$ kips
Moment carried by steel layer	$M_{x2} = P_{x2} \times ((h / 2) - x_{x2}) = 0.000$ kip_ft

Details of steel layer 3

Depth of layer	$x_{x3} = 15.813$ in
Strain of layer	$\epsilon_{x3} = \epsilon_c \times (1 - x_{x3} / c_x) = -0.00583$
Stress in layer	$\sigma_{x3} = \max(-1 \times f_y, E_s \times \epsilon_{x3}) = -60000.00$ psi
Force carried by layer	$P_{x3} = N_x \times A_{bar} \times \sigma_{x3} = -36.816$ kips
Moment carried by steel layer	$M_{x3} = P_{x3} \times ((h / 2) - x_{x3}) = 20.900$ kip_ft

Force carried by steel

Sum of forces by steel	$P_{xs} = -43.9$ kips
------------------------	-----------------------

Total force carried by column

Nominal axial load strength	$P_{nx} = 102.224$ kips
Strength reduction factor	$\phi_x = 0.900$
Ultimate axial load carrying capacity of column	$P_{ux} = \phi_x \times P_{nx} = 92.001$ kips

Total moment carried by column

Total moment carried by column	$M_{ox} = 120.792$ kip_ft
Ultimate moment strength capacity of column	$M_{ux} = \phi_x \times M_{ox} = 108.712$ kip_ft

Equivalent required uniaxial moment about x axis

Equivalent required uniaxial nominal moment	$M_{nxe} = M_{nx} + M_{ny} \times h / b \times ((1 - \beta) / \beta) = 22.538$ kip_ft
Equivalent required uniaxial ultimate moment	$M_{uxe} = M_{nxe} \times \phi_x = 20.284$ kip_ft

Check load capacity about the x axis

Factored axial load	$P_{u_act} = 92$ kips
Ultimate axial capacity	$P_{ux} = 92$ kips

PASS - Ultimate axial capacity exceeds factored axial load

Equivalent required uniaxial factored moment	$M_{uxe} = 20.3$ kip_ft
Ultimate moment capacity about the x axis	$M_{ux} = 108.7$ kip_ft

PASS - Ultimate moment capacity exceeds factored moment about x axis

Uniaxially loaded column about minor axis

Details of column cross-section

c/d_t ratio	$r_{yb} = 0.449$
Effective cover to reinforcement	$d' = c_c + D_{stir} + (D_{long}/2) = 2.188$ in

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section Concrete Column Design-C3				Sheet no./rev. 5	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Spacing between bars

$$s = ((b - (2 \times d')) / ((N_x) - 1)) = \mathbf{3.625 \text{ in}}$$

Depth of tension steel

$$b_t = b - d' = \mathbf{5.812 \text{ in}}$$

Depth of NA from extreme compression face

$$c_y = r_{yb} \times b_t = \mathbf{2.607 \text{ in}}$$

Factor of depth of compressive stress block

$$\beta_1 = \mathbf{0.800}$$

Depth of equivalent rectangular stress block

$$a_y = \min((\beta_1 \times c_y), b) = \mathbf{2.086 \text{ in}}$$

Yield strain in steel

$$\epsilon_{sy} = f_y / E_s = \mathbf{0.002}$$

Strength reduction factor

$$\phi_y = \mathbf{0.785}$$

Details of concrete block

Force carried by concrete

Forces carried by concrete

$$P_{ycon} = 0.85 \times f'_c \times h \times a_y = \mathbf{159.546 \text{ kips}}$$

Moment carried by concrete

Moment carried by concrete

$$M_{ycon} = P_{ycon} \times ((b/2) - (a_y/2)) = \mathbf{39.318 \text{ kip_ft}}$$

Details of steel layer 1

Depth of layer

$$x_{y1} = \mathbf{2.188 \text{ in}}$$

Strain of layer

$$\epsilon_{y1} = \epsilon_c \times (1 - x_{y1} / c_y) = \mathbf{0.00048}$$

Stress in layer

$$\sigma_{y1} = \min(f_y, E_s \times \epsilon_{y1}) = \mathbf{13998.32 \text{ psi}}$$

Force carried by layer

$$P_{y1} = N_y \times A_{bar} \times \sigma_{y1} = \mathbf{12.884 \text{ kips}}$$

Moment carried by steel layer

$$M_{y1} = P_{y1} \times ((b / 2) - x_{y1}) = \mathbf{1.946 \text{ kip_ft}}$$

Details of steel layer 2

Depth of layer

$$x_{y2} = \mathbf{5.813 \text{ in}}$$

Strain of layer

$$\epsilon_{y2} = \epsilon_c \times (1 - x_{y2} / c_y) = \mathbf{-0.00369}$$

Stress in layer

$$\sigma_{y2} = \max(-1 \times f_y, E_s \times \epsilon_{y2}) = \mathbf{-60000.00 \text{ psi}}$$

Force carried by layer

$$P_{y2} = N_y \times A_{bar} \times \sigma_{y2} = \mathbf{-55.223 \text{ kips}}$$

Moment carried by steel layer

$$M_{y2} = P_{y2} \times ((b / 2) - x_{y2}) = \mathbf{8.341 \text{ kip_ft}}$$

Force carried by steel

Sum of forces by steel

$$P_{ys} = \mathbf{-42.3 \text{ kips}}$$

Total force carried by column

Nominal axial load strength

$$P_{ny} = \mathbf{117.207 \text{ kips}}$$

Strength reduction factor

$$\phi_y = \mathbf{0.785}$$

Ultimate axial load carrying capacity of column

$$P_{uy} = \phi_y \times P_{ny} = \mathbf{92.006 \text{ kips}}$$

Moment carried by biaxial column minor axis

Nominal moment strength

$$M_{oy} = \mathbf{49.605 \text{ kip_ft}}$$

Contour beta factor

Contour beta factor

$$\beta = \mathbf{0.500}$$

$$M_{nx_upon_M_{ox1}} = M_{nx} / M_{ox} = \mathbf{0.111}$$

$$M_{ny_upon_M_{oy}} = \mathbf{0.889}$$

Net moment along minor axis resisted by column

$$M_{ny1} = M_{oy} \times (M_{ny_upon_M_{oy}}) = \mathbf{44.099 \text{ kip_ft}}$$

Ultimate moment along minor axis

$$M_{uy} = M_{ny1} \times \phi_y = \mathbf{34.617 \text{ kip_ft}}$$

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section Concrete Column Design-C3				Sheet no./rev. 6	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Check load capacity about the y axis

Factored axial load

$$P_{u_act} = 92 \text{ kips}$$

Ultimate axial capacity

$$P_{uy} = 92 \text{ kips}$$

PASS - Ultimate axial capacity exceeds factored axial load

Factored moment about the y axis

$$M_{uy_max} = \phi \times M_{ny} = 2.6 \text{ kip_ft}$$

Ultimate moment capacity about the y axis

$$M_{uy} = 34.6 \text{ kip_ft}$$

PASS - Ultimate moment capacity exceeds factored moment about y axis

Net moments for biaxial column

Assuming strength reduction factor

$$\phi = 0.65$$

Net moment about major (X) axis

$$M_{nx} = M_{cx} / \phi = 3.08 \text{ kips_ft}$$

Net moment about minor (Y) axis

$$M_{ny} = M_{cy_min} / \phi = 13.01 \text{ kips_ft}$$

Uniaxially loaded column about major axis

Details of column cross-section

c/d_t ratio

$$r_{xb} = 0.340$$

Effective cover to reinforcement

$$d' = c_c + D_{stir} + (D_{long}/2) = 2.188 \text{ in}$$

Spacing between bars

$$s = ((h - (2 \times d')) / ((N_y) - 1)) = 6.813 \text{ in}$$

Depth of tension steel

$$d_t = h - d' = 15.813 \text{ in}$$

Depth of NA from extreme compression face

$$c_x = r_{xb} \times d_t = 5.371 \text{ in}$$

Factor of depth of compressive stress block

$$\beta_1 = 0.800$$

Depth of equivalent rectangular stress block

$$a_x = \min((\beta_1 \times c_x), h) = 4.297 \text{ in}$$

Yield strain in steel

$$\epsilon_{sx} = f_y / E_s = 0.002$$

Strength reduction factor

$$\phi_x = 0.900$$

Details of concrete block

Force carried by concrete

Forces carried by concrete

$$P_{xcon} = 0.85 \times f'_c \times b \times a_x = 146.082 \text{ kips}$$

Moment carried by concrete

Moment carried by concrete

$$M_{xcon} = P_{xcon} \times ((h/2) - (a_x/2)) = 83.410 \text{ kip_ft}$$

Details of steel layer 1

Depth of layer

$$x_{x1} = 2.188 \text{ in}$$

Strain of layer

$$\epsilon_{x1} = \epsilon_c \times (1 - x_{x1} / c_x) = 0.00178$$

Stress in layer

$$\sigma_{x1} = \min(f_y, E_s \times \epsilon_{x1}) - 0.85 \times f'_c = 47314.39 \text{ psi}$$

Force carried by layer

$$P_{x1} = N_x \times A_{bar} \times \sigma_{x1} = 29.032 \text{ kips}$$

Moment carried by steel layer

$$M_{x1} = P_{x1} \times ((h/2) - x_{x1}) = 16.482 \text{ kip_ft}$$

Details of steel layer 2

Depth of layer

$$x_{x2} = 9.000 \text{ in}$$

Strain of layer

$$\epsilon_{x2} = \epsilon_c \times (1 - x_{x2} / c_x) = -0.00203$$

Stress in layer

$$\sigma_{x2} = \max(-1 \times f_y, E_s \times \epsilon_{x2}) = -58792.21 \text{ psi}$$

Force carried by layer

$$P_{x2} = 2 \times A_{bar} \times \sigma_{x2} = -36.074 \text{ kips}$$

Moment carried by steel layer

$$M_{x2} = P_{x2} \times ((h/2) - x_{x2}) = 0.000 \text{ kip_ft}$$

Details of steel layer 3

Depth of layer

$$x_{x3} = 15.813 \text{ in}$$

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section Concrete Column Design-C3				Sheet no./rev. 7	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Strain of layer $\epsilon_{x3} = \epsilon_c \times (1 - x_{x3} / c_x) = -0.00583$

Stress in layer $\sigma_{x3} = \max(-1 \times f_y, E_s \times \epsilon_{x3}) = -60000.00$ psi

Force carried by layer $P_{x3} = N_x \times A_{bar} \times \sigma_{x3} = -36.816$ kips

Moment carried by steel layer $M_{x3} = P_{x3} \times ((h / 2) - x_{x3}) = 20.900$ kip_ft

Force carried by steel

Sum of forces by steel $P_{xs} = -43.9$ kips

Total force carried by column

Nominal axial load strength $P_{nx} = 102.224$ kips

Strength reduction factor $\phi_x = 0.900$

Ultimate axial load carrying capacity of column $P_{ux} = \phi_x \times P_{nx} = 92.001$ kips

Total moment carried by column

Total moment carried by column $M_{ox} = 120.792$ kip_ft

Ultimate moment strength capacity of column $M_{ux} = \phi_x \times M_{ox} = 108.712$ kip_ft

Equivalent required uniaxial moment about x axis

Equivalent required uniaxial nominal moment $M_{nxe} = M_{nx} + M_{ny} \times h / b \times ((1 - \beta) / \beta) = 32.354$ kip_ft

Equivalent required uniaxial ultimate moment $M_{uxe} = M_{nxe} \times \phi_x = 29.118$ kip_ft

Check load capacity about the x axis

Factored axial load $P_{u_act} = 92$ kips

Ultimate axial capacity $P_{ux} = 92$ kips

PASS - Ultimate axial capacity exceeds factored axial load

Equivalent required uniaxial factored moment $M_{uxe} = 29.1$ kip_ft

Ultimate moment capacity about the x axis $M_{ux} = 108.7$ kip_ft

PASS - Ultimate moment capacity exceeds factored moment about x axis

Uniaxially loaded column about minor axis

Details of column cross-section

c/d_t ratio $r_{yb} = 0.449$

Effective cover to reinforcement $d' = c_c + D_{stir} + (D_{long}/2) = 2.188$ in

Spacing between bars $s = ((b - (2 \times d')) / ((N_x) - 1)) = 3.625$ in

Depth of tension steel $b_t = b - d' = 5.812$ in

Depth of NA from extreme compression face $c_y = r_{yb} \times b_t = 2.607$ in

Factor of depth of compressive stress block $\beta_1 = 0.800$

Depth of equivalent rectangular stress block $a_y = \min((\beta_1 \times c_y), b) = 2.086$ in

Yield strain in steel $\epsilon_{sy} = f_y / E_s = 0.002$

Strength reduction factor $\phi_y = 0.785$

Details of concrete block

Force carried by concrete

Forces carried by concrete $P_{ycon} = 0.85 \times f'_c \times h \times a_y = 159.546$ kips

Moment carried by concrete

Moment carried by concrete $M_{ycon} = P_{ycon} \times ((b/2) - (a_y/2)) = 39.318$ kip_ft

Details of steel layer 1

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section Concrete Column Design-C3				Sheet no./rev. 8	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Depth of layer $x_{y1} = 2.188$ in
Strain of layer $\epsilon_{y1} = \epsilon_c \times (1 - x_{y1} / c_y) = 0.00048$
Stress in layer $\sigma_{y1} = \min(f_y, E_s \times \epsilon_{y1}) = 13998.32$ psi
Force carried by layer $P_{y1} = N_y \times A_{bar} \times \sigma_{y1} = 12.884$ kips
Moment carried by steel layer $M_{y1} = P_{y1} \times ((b / 2) - x_{y1}) = 1.946$ kip_ft

Details of steel layer 2

Depth of layer $x_{y2} = 5.813$ in
Strain of layer $\epsilon_{y2} = \epsilon_c \times (1 - x_{y2} / c_y) = -0.00369$
Stress in layer $\sigma_{y2} = \max(-1 \times f_y, E_s \times \epsilon_{y2}) = -60000.00$ psi
Force carried by layer $P_{y2} = N_y \times A_{bar} \times \sigma_{y2} = -55.223$ kips
Moment carried by steel layer $M_{y2} = P_{y2} \times ((b / 2) - x_{y2}) = 8.341$ kip_ft

Force carried by steel

Sum of forces by steel $P_{ys} = -42.3$ kips

Total force carried by column

Nominal axial load strength $P_{ny} = 117.207$ kips
Strength reduction factor $\phi_y = 0.785$
Ultimate axial load carrying capacity of column $P_{uy} = \phi_y \times P_{ny} = 92.006$ kips

Moment carried by biaxial column minor axis

Nominal moment strength $M_{oy} = 49.605$ kip_ft

Contour beta factor

Contour beta factor $\beta = 0.500$
 $M_{nx_upon_M_{ox1}} = M_{nx} / M_{ox} = 0.025$

$M_{ny_upon_M_{oy}} = 0.975$
Net moment along minor axis resisted by column $M_{ny1} = M_{oy} \times (M_{ny_upon_M_{oy}}) = 48.365$ kip_ft
Ultimate moment along minor axis $M_{uy} = M_{ny1} \times \phi_y = 37.966$ kip_ft

Check load capacity about the y axis

Factored axial load $P_{u_act} = 92$ kips
Ultimate axial capacity $P_{uy} = 92$ kips

PASS - Ultimate axial capacity exceeds factored axial load

Factored moment about the y axis $M_{uy_max} = \phi \times M_{ny} = 8.5$ kip_ft
Ultimate moment capacity about the y axis $M_{uy} = 38$ kip_ft

PASS - Ultimate moment capacity exceeds factored moment about y axis

Design of column ties - 25.7.2

Spacing of lateral ties $s_{v_ties} = 8.000$ in
16 times longitudinal bar diameter $s_{v1} = 16 \times D_{long} = 10.000$ in
48 times tie bar diameter $s_{v2} = 48 \times D_{stir} = 18.000$ in
Least column dimension $s_{v3} = \min(h, b) = 8.000$ in
Required tie spacing $s = \min(s_{v1}, s_{v2}, s_{v3}) = 8.000$ in

$s_{v_ties} < s$ PASS

Z. Belfranin 4836 SW 74Ct MIAMI, FL 33155	Project Herrera Res.				Job Ref.	
	Section Structural Calc's				Sheet no./rev.	
	Calcs. By H.J	Date 05/12/2025	Chk'd by Z.T.B	Date	App'd by	Date

- Concrete Footing Design (F-60) under Column C3:

From concrete column design C3, we have:

$$P_{DL} = 52.8 \text{ kips}$$

$$P_{LL} = 17.5 \text{ kips}$$

$$P_{DL} = \frac{150lb}{ft^3} \times \frac{8"}{12} \times \frac{18"}{12} \times 13' (Column \ C3 \ weight) = 1950 \text{ lbs} = 1.95 \text{ kips}$$

$$P_{DL(Total)} = 52.8 + 1.95 = 54.75 \text{ kips} \approx 55 \text{ kips}$$

$$q_{LL} \text{allowable capacity of soil} = 2.5 \text{ ksf}$$

Use 6'-0"x6'-0"x24" with #6@ 9" O.C Bottom E.W.

Please see next page for calculation.

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section Concrete Footing Design-F-60				Sheet no./rev. 1	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

FOOTING ANALYSIS

In accordance with ACI318-19

Tedds calculation version 3.3.06

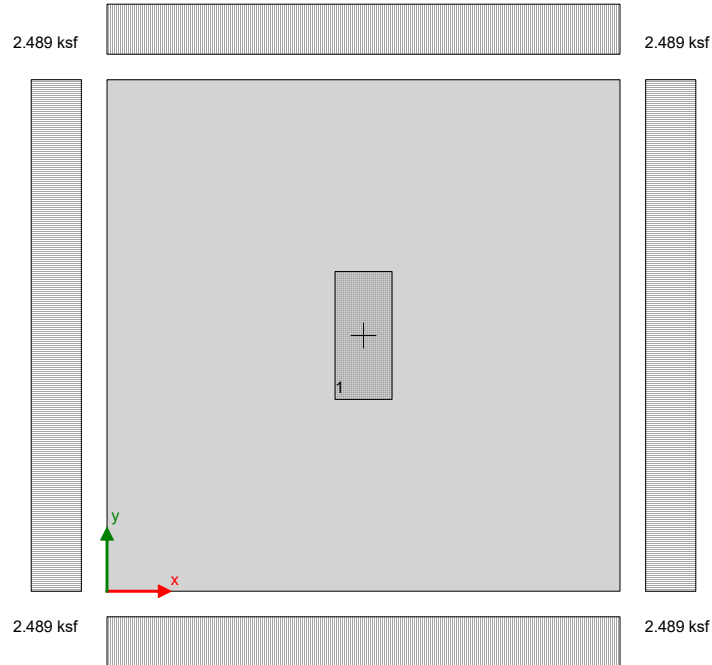
Summary results

Overall design status **PASS**
Overall design utilisation **0.996**

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	89.6			Pass
Description	Unit	Applied	Resisting	Utilization	Result
Soil bearing	ksf	2.489	2.5	0.996	Pass
Description	Unit	Required	Provided	Utilization	Result
Moment, positive, x-direction	kip_ft	55.6	319.9	0.174	Pass
Moment, positive, y-direction	kip_ft	39.6	308.0	0.129	Pass
Shear, one-way, x-direction	kips	15.8	60.7	0.260	Pass
Shear, one-way, y-direction	kips	9.3	60.0	0.155	Pass
Shear, two-way, Col 1	psi	27.483	145.707	0.189	Pass
Min.area of reinf, bot., x-direction	in ²	3.110	3.520		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	9.3		Pass
Min.area of reinf, bot., y-direction	in ²	3.110	3.520		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	9.3		Pass

Pad footing details

Length of footing $L_x = 6$ ft
Width of footing $L_y = 6$ ft
Footing area $A = L_x \times L_y = 36$ ft²
Depth of footing $h = 24$ in
Depth of soil over footing $h_{soil} = 18$ in
Density of concrete $\gamma_{conc} = 150.0$ lb/ft³



Column no.1 details

Length of column

$$l_{x1} = 8.00 \text{ in}$$

Width of column

$$l_{y1} = 18.00 \text{ in}$$

position in x-axis

$$x_1 = 36.00 \text{ in}$$

position in y-axis

$$y_1 = 36.00 \text{ in}$$

Soil properties

Gross allowable bearing pressure

$$q_{\text{allow_Gross}} = 2.5 \text{ ksf}$$

Density of soil

$$\gamma_{\text{soil}} = 120.0 \text{ lb/ft}^3$$

Angle of internal friction

$$\phi_b = 30.0 \text{ deg}$$

Design base friction angle

$$\delta_{bb} = 30.0 \text{ deg}$$

Coefficient of base friction

$$\tan(\delta_{bb}) = 0.577$$

Footing loads

Self weight

$$F_{\text{swt}} = h \times \gamma_{\text{conc}} = 300 \text{ psf}$$

Soil weight

$$F_{\text{soil}} = h_{\text{soil}} \times \gamma_{\text{soil}} = 180 \text{ psf}$$

Column no.1 loads

Dead load in z

$$F_{Dz1} = 55.0 \text{ kips}$$

Live load in z

$$F_{Lz1} = 17.5 \text{ kips}$$

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.801)

1.0D + 1.0L (0.996)

Combination 2 results: 1.0D + 1.0L

 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.				Job Ref.	
	Section Concrete Footing Design-F-60				Sheet no./rev. 3	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date	App'd by	Date

Forces on footing

Force in z-axis

$$F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{89.6 \text{ kips}}$$

Moments on footing

Moment in x-axis, about x is 0

$$M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{268.8 \text{ kip_ft}}$$

Moment in y-axis, about y is 0

$$M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{268.8 \text{ kip_ft}}$$

Uplift verification

Vertical force

$$F_{dz} = \mathbf{89.6 \text{ kips}}$$

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis

$$e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis

$$e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.489 \text{ ksf}}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.489 \text{ ksf}}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.489 \text{ ksf}}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.489 \text{ ksf}}$$

Minimum base pressure

$$q_{min} = \min(q_1, q_2, q_3, q_4) = \mathbf{2.489 \text{ ksf}}$$

Maximum base pressure

$$q_{max} = \max(q_1, q_2, q_3, q_4) = \mathbf{2.489 \text{ ksf}}$$

Allowable bearing capacity

Allowable bearing capacity

$$q_{allow} = q_{allow_Gross} = \mathbf{2.5 \text{ ksf}}$$

$$q_{max} / q_{allow} = \mathbf{0.996}$$

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

$$f'_c = \mathbf{4000 \text{ psi}}$$

Yield strength of reinforcement

$$f_y = \mathbf{60000 \text{ psi}}$$

Compression-controlled strain limit (21.2.2)

$$\epsilon_{ty} = \mathbf{0.00200}$$

Cover to top of footing

$$c_{nom_t} = \mathbf{3 \text{ in}}$$

Cover to side of footing

$$c_{nom_s} = \mathbf{3 \text{ in}}$$

Cover to bottom of footing

$$c_{nom_b} = \mathbf{3 \text{ in}}$$

Concrete type

$$\text{Normal weight}$$

Concrete modification factor

$$\lambda = \mathbf{1.00}$$

Column type

$$\text{Concrete}$$

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.213)

1.2D + 1.6L + 0.5Lr (0.260)

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

Forces on footing

Ultimate force in z-axis

$$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = 114.5 \text{ kips}$$

Moments on footing

Ultimate moment in x-axis, about x is 0

$$M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = 343.6 \text{ kip_ft}$$

Ultimate moment in y-axis, about y is 0

$$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = 343.6 \text{ kip_ft}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-axis

$$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = 0 \text{ in}$$

Eccentricity of base reaction in y-axis

$$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = 0 \text{ in}$$

Pad base pressures

$$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = 3.181 \text{ ksf}$$

$$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = 3.181 \text{ ksf}$$

$$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = 3.181 \text{ ksf}$$

$$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = 3.181 \text{ ksf}$$

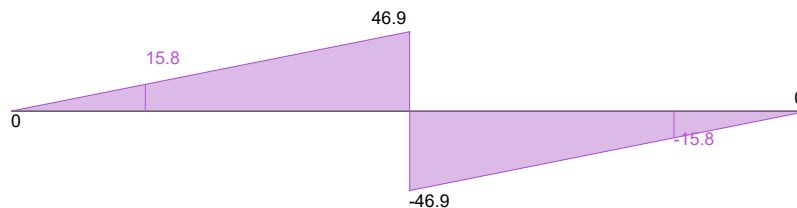
Minimum ultimate base pressure

$$q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = 3.181 \text{ ksf}$$

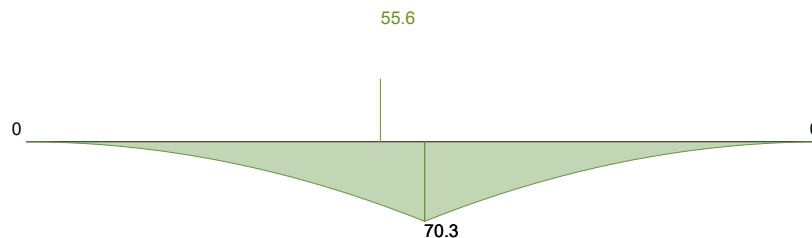
Maximum ultimate base pressure

$$q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = 3.181 \text{ ksf}$$

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment

$$M_{u,x,max} = 55.589 \text{ kip_ft}$$

Tension reinforcement provided

$$8 \text{ No.6 bottom bars (9.3 in c/c)}$$

Area of tension reinforcement provided

$$A_{sx,bot,prov} = 3.52 \text{ in}^2$$

Minimum area of reinforcement (8.6.1.1)

$$A_{s,min} = 0.0018 \times L_y \times h = 3.11 \text{ in}^2$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)

$$s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement

$$d = h - C_{nom_b} - \phi_{x,bot} / 2 = 20.625 \text{ in}$$

Depth of compression block

$$a = A_{sx,bot,prov} \times f_y / (0.85 \times f'_c \times L_y) = 0.863 \text{ in}$$

Neutral axis factor

$$\beta_1 = 0.85$$

Depth to neutral axis

$$c = a / \beta_1 = 1.015 \text{ in}$$

Strain in tensile reinforcement

$$\epsilon_t = 0.003 \times d / c - 0.003 = 0.05796$$

Minimum tensile strain(8.3.3.1)

$$\epsilon_{min} = \epsilon_{ty} + 0.003 = 0.00500$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity

$$M_n = A_{sx,bot,prov} \times f_y \times (d - a / 2) = 355.408 \text{ kip_ft}$$

Flexural strength reduction factor

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = 0.900$$

Design moment capacity

$$\phi M_n = \phi_f \times M_n = 319.867 \text{ kip_ft}$$

$$M_{u,x,max} / \phi M_n = 0.174$$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force

$$V_{u,x} = 15.799 \text{ kips}$$

Depth to reinforcement

$$d_v = h - C_{nom_b} - \phi_{x,bot} / 2 = 20.625 \text{ in}$$

Size effect factor (22.5.5.1.3)

$$\lambda_s = \min(\sqrt{2 / (1 + (d_v / 1 \text{ in}) / 10)}, 1.0) = 0.808$$

Ratio of longitudinal reinforcement

$$\rho_w = A_{sx,bot,prov} / (L_y \times d_v) = 0.00237$$

Shear strength reduction factor

$$\phi_v = 0.75$$

Nominal shear capacity (Eq. 22.5.5.1)

$$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v) = 80.958 \text{ kips}$$

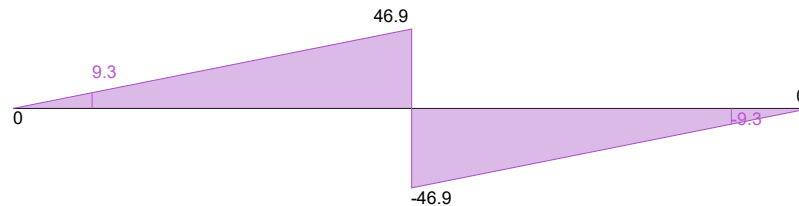
Design shear capacity

$$\phi V_n = \phi_v \times V_n = 60.719 \text{ kips}$$

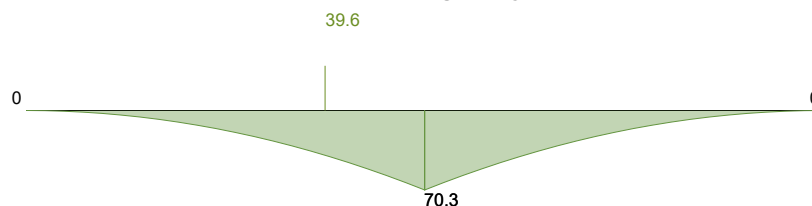
$$V_{u,x} / \phi V_n = 0.260$$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



 Z.T.B 4836 SW 74th Ct Miami FL 33155	Project Herrera Res.		Job Ref.	
	Section Concrete Footing Design-F-60		Sheet no./rev. 6	
	Calc. by H.J	Date 5/12/2025	Chk'd by	Date
			App'd by	Date

Moment design, y direction, positive moment

Ultimate bending moment $M_{u,y,max} = 39.577 \text{ kip_ft}$
 Tension reinforcement provided 8 No.6 bottom bars (9.3 in c/c)
 Area of tension reinforcement provided $A_{sy,bot,prov} = 3.52 \text{ in}^2$
 Minimum area of reinforcement (8.6.1.1) $A_{s,min} = 0.0018 \times L_x \times h = 3.11 \text{ in}^2$

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom_b} - \phi_{x,bot} - \phi_{y,bot} / 2 = 19.875 \text{ in}$
 Depth of compression block $a = A_{sy,bot,prov} \times f_y / (0.85 \times f'_c \times L_x) = 0.863 \text{ in}$
 Neutral axis factor $\beta_1 = 0.85$
 Depth to neutral axis $c = a / \beta_1 = 1.015 \text{ in}$
 Strain in tensile reinforcement $\epsilon_t = 0.003 \times d / c - 0.003 = 0.05574$
 Minimum tensile strain(8.3.3.1) $\epsilon_{min} = \epsilon_{ty} + 0.003 = 0.00500$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy,bot,prov} \times f_y \times (d - a / 2) = 342.208 \text{ kip_ft}$
 Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = 0.900$
 Design moment capacity $\phi M_n = \phi_f \times M_n = 307.987 \text{ kip_ft}$
 $M_{u,y,max} / \phi M_n = 0.129$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u,y} = 9.286 \text{ kips}$
 Depth to reinforcement $d_v = h - c_{nom_b} - \phi_{x,bot} - \phi_{y,bot} / 2 = 19.875 \text{ in}$
 Size effect factor (22.5.5.1.3) $\lambda_s = \min(\sqrt{2 / (1 + (d_v / 1 \text{ in}) / 10)}, 1.0) = 0.818$
 Ratio of longitudinal reinforcement $\rho_w = A_{sy,bot,prov} / (L_x \times d_v) = 0.00246$
 Shear strength reduction factor $\phi_v = 0.75$
 Nominal shear capacity (Eq. 22.5.5.1) $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v) = 79.969 \text{ kips}$
 Design shear capacity $\phi V_n = \phi_v \times V_n = 59.977 \text{ kips}$
 $V_{u,y} / \phi V_n = 0.155$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement $d_{v2} = 20.25 \text{ in}$
 Shear perimeter length (22.6.4) $l_{xp} = 28.250 \text{ in}$
 Shear perimeter width (22.6.4) $l_{yp} = 38.250 \text{ in}$
 Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = 133.000 \text{ in}$
 Shear area $A_p = l_{x,perim} \times l_{y,perim} = 1080.562 \text{ in}^2$
 Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1} = 936.562 \text{ in}^2$
 Ultimate bearing pressure at center of shear area $q_{up,avg} = 3.181 \text{ ksf}$
 Ultimate shear load $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up,avg} \times A_p = 74.019 \text{ kips}$
 Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = 27.483 \text{ psi}$

Column geometry factor (Table 22.6.5.2)

$$\beta = I_{y1} / I_{x1} = \mathbf{2.25}$$

Column location factor (22.6.5.3)

$$\alpha_s = \mathbf{40}$$

Size effect factor (22.5.5.1.3)

$$\lambda_s = \min(\sqrt{2 / (1 + (d_{v2} / 1 \text{ in}) / 10)}), 1.0) = \mathbf{0.813}$$

Concrete shear strength (22.6.5.2)

$$V_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = \mathbf{194.276 \text{ psi}}$$

$$V_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = \mathbf{416.048 \text{ psi}}$$

$$V_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = \mathbf{205.704 \text{ psi}}$$

$$V_{cp} = \min(V_{cpa}, V_{cpb}, V_{cpc}) = \mathbf{194.276 \text{ psi}}$$

Shear strength reduction factor

$$\phi_v = \mathbf{0.75}$$

Nominal shear stress capacity (Eq. 22.6.1.2)

$$V_n = V_{cp} = \mathbf{194.276 \text{ psi}}$$

Design shear stress capacity (8.5.1.1(d))

$$\phi V_n = \phi_v \times V_n = \mathbf{145.707 \text{ psi}}$$

$$V_{ug} / \phi V_n = \mathbf{0.189}$$

PASS - Design shear stress capacity exceeds ultimate shear stress load

