ZFA STRUCTURAL ENGINEERS

san francisco
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Morgan Hill - Butterfield FS
Structural Calculations

Butterfield, CA

ZFA Project Number: 22342

Permit Submittal

March 14, 2023

Prepared For:

COAR Design

Santa Rosa, California

Prepared By:

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Santa Rosa, California

REGISTERED PROFESSIONAL ENGINEER
LUKE S. WILSON
No. 5933
STRUCTURAL
STATE OF CALIFORNIA

03/14/2023

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STRUCTURAL NARRATIVE

The project is a new fire station that is approximately 7,300 square foot one-story 2 drive through apparatus bay station located at 17285 Butterfield Boulevard, Morgan Hill, CA. The building will be designed as a risk category IV essential service building which requires designing to higher seismic and wind loading criteria. The gravity system consists of manufactured roof trusses for the typical roof and LVL rafters spanning between Glulam Beams for the Apparatus Bay roof. The lateral system will be a plywood diaphragm and wood shear walls with holdowns on each end.

The building will be designed to the current 2022 California Building Code (CBC), the 1986 Essential Service Act and ASCE 7-16. All calculations on the following pages are in conformance with these codes.

DESIGN CRITERIA

Material (unless noted otherwise)

Concrete:	f'c = 3000 psi @ 28 days (Foundation)		
	f'c = 3000 psi @ 28 days (Beams and Columns)		
	f'c = 3000 psi @ 28 days (Walls)		
Masonry:	f'm = 2000 psi @ 28 days, ASTM C90		
Reinf. Steel:	fy = 60 ksi, ASTM A615, Grade 60		
Lumber:	W.C.D.F Grades as follows:		
	Joists & Plates	Fb = 900 psi	Douglas Fir #2
	Studs	Fb = 900 psi	Douglas Fir #2
	Beams & Headers	Fb = 1350 psi	Douglas Fir #1
	Plywood / OSB	PS1 / PS2	
	Glued-Laminated Beam (GLB)	Fb = 2400 psi	24F-V4 (DF/DF) simple span, 24F-V8 (DF/DF) continuous span
	Exterior Glued-Laminated Beam (GLB - EXT)	Fb = 2000 psi	20F-V12 (AC/AC) simple span, 20F-V13 (AC/AC) continuous span
Structural Steel:	WF Shapes	fy = 50 ksi	ASTM A992 or A572 Grade 50
	C and L Shapes, and Plates	fy = 36 ksi	ASTM A36 or A572 Grade 50
	Pipes	fy = 35 ksi	ASTM A53, Grade B
	Round HSS	fy = 46 ksi	ASTM A500, Grade C
	Square and Rectangular HSS	fy = 50 ksi	ASTM A500, Grade C
Cold Formed:	33 & 43 mils (20 & 18 ga)	fy = 33 ksi	ASTM A1003, Grade ST33H or ST33L
	54, 68, & 97 mils (16, 14, & 12 ga)	fy = 50 ksi	ASTM A1003, Grade ST50H or ST50L

Stamp

DESIGN LOADING (Typical)

			ROOF*	ROOF*	FLOOR	FLOOR	WALLS	WALLS
LIVE LOADS (PSF)								
Partitions (Office Building)?		NO			0.0	0.0		
DEAD LOADS (PSF)								
Roofing**	Asphalt Shingles		3.0	3.0				
Fin. Floor								
Sheathing	1/2 in. OSB		1.7	1.7				
Joists/Truss	MFR WD Trusses @ 24" oc		3.0	3.0				
Beams								
Ceiling	Suspension System (w/ tile)		1.3	1.3				
Insulation	Fill, Blown-In, & Batt t = 9"		1.8	1.8				
HVAC	Typical		2.0	2.0				
Sprinklers	Sprinkler System (typical)		1.5	1.5				
Misc.	Solar		3.0	3.0				
Misc.			1.7	1.7				
DEAD LOADS (PSF)			19.0	19.0	0.0	0.0	10.0	21.5
SLOPE (X:12)			0.0					
SLOPE ADJUSTED DEAD LOADS (PSF)			19.0	19.0	0.0	0.0		
Partition Mass (Include in Seismic)					0.0	0.0		
TOTAL LOADS (PSF)			19.0	19.0	0.0	0.0		

*25% stress increase for load duration on wood roof member and connections. **For DSA: wt. Includes 1 re-roof.

DESIGN LOADING (App Bay)

			SLOPED ROOF*	FLAT ROOF*	FLOOR	FLOOR	INTERIOR WALLS	EXTERIOR WALLS
LIVE LOADS (PSF)								
Partitions (Office Building)?		NO			0.0	0.0		
DEAD LOADS (PSF)								
Roofing**	Roofing & 1/2' Cover Board		3.0	3.0				
Fin. Floor			0.0	0.0				
Sheathing	5/8 in. OSB		2.0	2.0				
Joists/Truss	Glulams		3.5	3.5				
Beams								
Ceiling	5/8 in. Gypsum Board		2.8	2.8				
Insulation	e Fill, Blown-In, & Batt t =		1.8	1.8				
HVAC	Plymovent Capture System		2.0	2.0				
Sprinklers	Sprinkler System (typical)		1.5	1.5				
Misc.	Solar		3.0	3.0				
Misc.			1.4	1.4				
DEAD LOADS (PSF)			21.0	21.0	0.0	0.0	10.0	20.0
SLOPE (X:12)			0.0					
SLOPE ADJUSTED DEAD LOADS (PSF)			21.0	21.0	0.0	0.0		
Partition Mass (Include in Seismic)					0.0	0.0		
TOTAL LOADS (PSF)			21.0	21.0	0.0	0.0		

*25% stress increase for load duration on wood roof member and connections. **For DSA: wt. Includes 1 re-roof.

Design Code: **2019 California Building Code** — Risk Category = **IV**

Wind Exposure = **C** — Basic Wind Speed (3sec. gust) = **102 mph**

Design Category = **D** — $R_{(N\text{-}S)}$ = **6.50** — $\Omega_{0\,(N\text{-}S)}$ = **2.50** — $I_{SEISMIC}$ = **1.50**

Soil Site Class = **D** — $R_{(E\text{-}W)}$ = **6.50** — $\Omega_{0\,(E\text{-}W)}$ = **2.50**

S_s = **1.582 g** — S_1 = **0.600 g**

Seismic Coefficients:

Foundation:

Design Dead + Live Bearing = **3000** **psf**

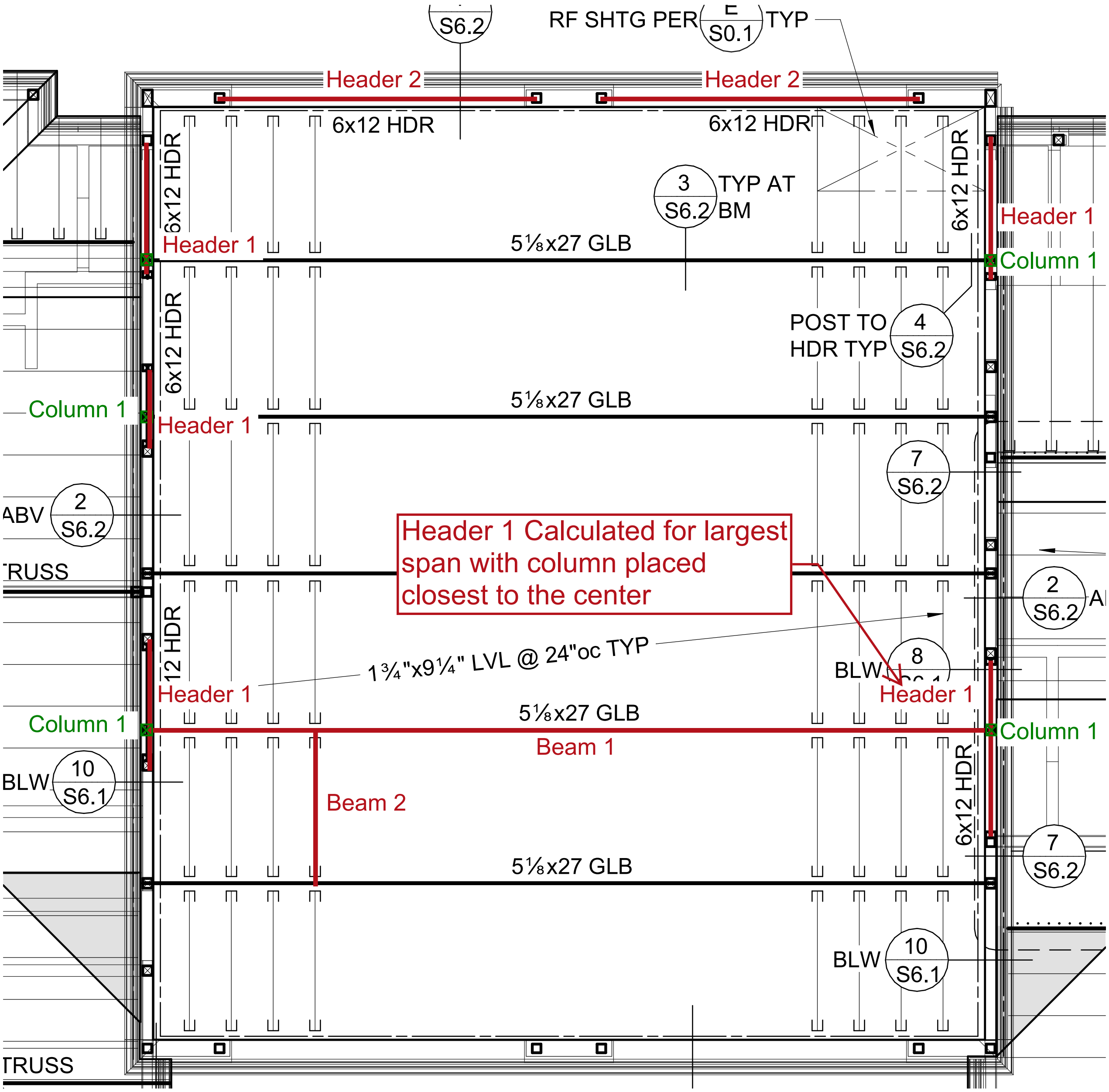
Design Dead + Wind/Seismic Bearing = **4000** **psf**

F_a = **1.000**
F_v = **1.700**
S_{MS} = **0.600**
S_{M1} = **1.020**
S_{DS} = **1.055**
S_{D1} = **0.680**

Geotechnical Report by: Geocon Consultants, Inc.
E9291-04-01
5/18/2022

Gravity & Out-of-Plane Analysis & Design

Revised design for 2022 CBC does not affect gravity and out-of-plane analysis



Manufactured roof trusses to be designed by others

Wood Beam

Project File: morgan hill.ec6

LIC# : KW-06015331, Build:20.22.10.25 ZFA STRUCTURAL ENGINEERS (c) ENERCALC INC 1983-2022

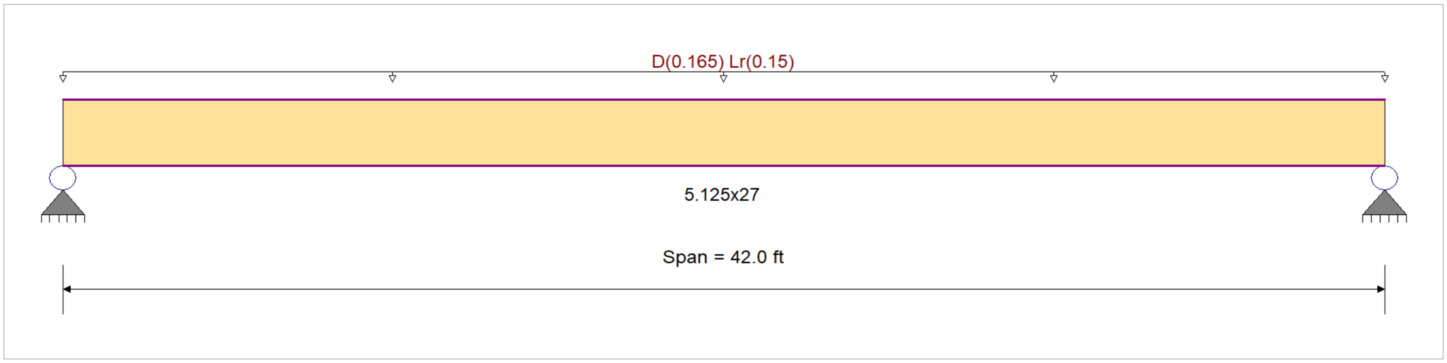
DESCRIPTION: Beam 1

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade :	24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.0220, Lr = 0.020 ksf, Tributary Width = 7.50 ft, (Roof Loads)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.519** : 1	Maximum Shear Stress Ratio	=	**0.194** : 1
Section used for this span		**5.125x27**	Section used for this span		**5.125x27**
fb: Actual	=	1,338.54 psi	fv: Actual	=	64.38 psi
F'b	=	2,581.07 psi	F'v	=	331.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	21.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.698 in	Ratio =	721 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	1.466 in	Ratio =	343 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Wood Beam

Project File: morgan hill.ec6

LIC# : KW-06015331, Build:20.22.10.25 ZFA STRUCTURAL ENGINEERS (c) ENERCALC INC 1983-2022

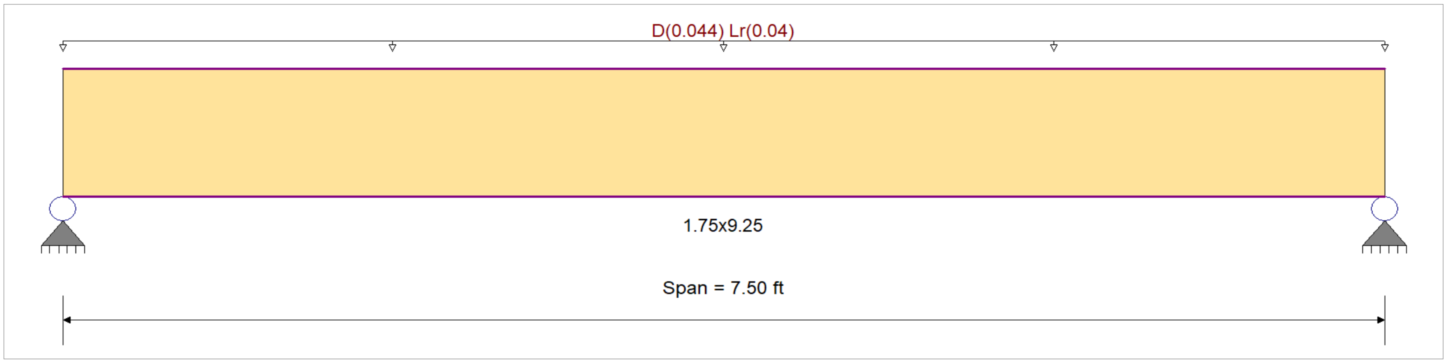
DESCRIPTION: Beam 2

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasticity*		
Load Combination :	ASCE 7-16	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi	
		Fc - Prll	2,635.0 psi	Eminbend - xx	1,036.83 ksi	
Wood Species :	RedBuilt	Fc - Perp	750.0 psi			
Wood Grade :	RedLam LVL Beam/Joist	Fv	285.0 psi			
		Ft	1,660.0 psi	Density	42.010 pcf	
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling					



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Uniform Load : D = 0.0220, Lr = 0.020 ksf, Tributary Width = 2.0 ft, (Roof Loads)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.080**: 1	Maximum Shear Stress Ratio	=	**0.069** : 1
Section used for this span		**1.75x9.25**	Section used for this span		**1.75x9.25**
fb: Actual	=	299.97 psi	fv: Actual	=	24.53 psi
F'b	=	3,755.62 psi	F'v	=	356.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	3.750 ft	Location of maximum on span	=	6.734 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.012 in	Ratio =	7253	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.028 in	Ratio =	3270	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Wood Column

Project File: morgan hill.ec6

LIC# : KW-06015331, Build:20.22.10.25 ZFA STRUCTURAL ENGINEERS (c) ENERCALC INC 1983-2022

DESCRIPTION: Column 1

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Analysis Method: Allowable Stress Design
End Fixities: Top & Bottom Pinned
Overall Column Height: 4 ft
(Used for non-slender calculations)
Wood Species: Douglas Fir-Larch
Wood Grade: No.1

Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.210 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name: **6x6**
Wood Grading/Manuf.: Graded Lumber
Wood Member Type: Sawn

Exact Width: **5.50** in
Exact Depth: **5.50** in
Area: 30.250 in^2
Ix: 76.255 in^4
Iy: 76.255 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compressio	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Fact	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 4 ft, K
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 4 ft, K

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 26.225 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 4.0 ft, D = 3.30, Lr = 3.0 k
BENDING LOADS . . .
Lat. Uniform Load creating Mx-x, L = 0.020 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.1752 : 1**
	Load Combination	+D+Lr
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are .	
	Applied Axial	6.326 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	1,193.74 psi
PASS	Maximum Shear Stress Ratio =	**0.01167 : 1**
	Load Combination	+D+L
	Location of max.above base	0.0 ft
	Applied Design Shear	1.983 psi
	Allowable Shear	170.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.040 k	Bottom along Y-Y	0.040 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.000954 in at 2.013 ft above base
for load combination : +D+L
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.969	0.1261	PASS	0.0 ft	0.0	PASS	4.0 ft
+D+L	1.000	0.965	0.1140	PASS	1.987 ft	0.01167	PASS	0.0 ft
+D+Lr	1.250	0.955	0.1752	PASS	0.0 ft	0.0	PASS	4.0 ft
+D+0.750Lr+0.750L	1.250	0.955	0.1544	PASS	1.987 ft	0.0070	PASS	0.0 ft
+D+0.750L	1.150	0.959	0.09971	PASS	1.987 ft	0.007609	PASS	0.0 ft
+0.60D	1.600	0.940	0.04384	PASS	0.0 ft	0.0	PASS	4.0 ft

Wood Column

Project File: morgan hill.ec6

LIC# : KW-06015331, Build:20.22.10.25 ZFA STRUCTURAL ENGINEERS (c) ENERCALC INC 1983-2022

DESCRIPTION: Column 1

Maximum Reactions

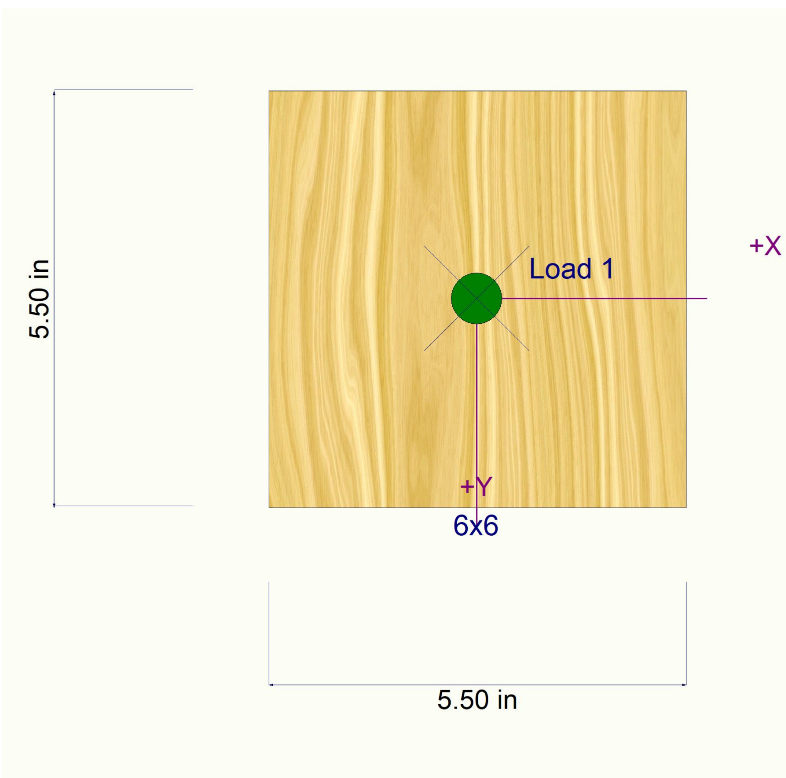
Note: Only non-zero reactions are listed.

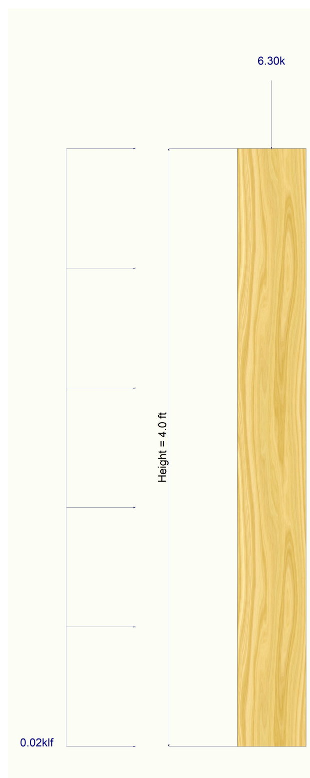
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
	k					k-ft			
D Only					3.326				
+D+L			0.040	0.040	3.326				
+D+Lr					6.326				
+D+0.750Lr+0.750L			0.030	0.030	5.576				
+D+0.750L			0.030	0.030	3.326				
+0.60D					1.996				
Lr Only					3.000				
L Only			0.040	0.040					

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.001 in	2.013 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L	0.0000 in	0.000 ft	0.001 in	2.013 ft
+D+0.750L	0.0000 in	0.000 ft	0.001 in	2.013 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.001 in	2.013 ft

Sketches





Wood Beam

Project File: morgan hill.ec6

LIC# : KW-06015331, Build:20.22.10.25 ZFA STRUCTURAL ENGINEERS

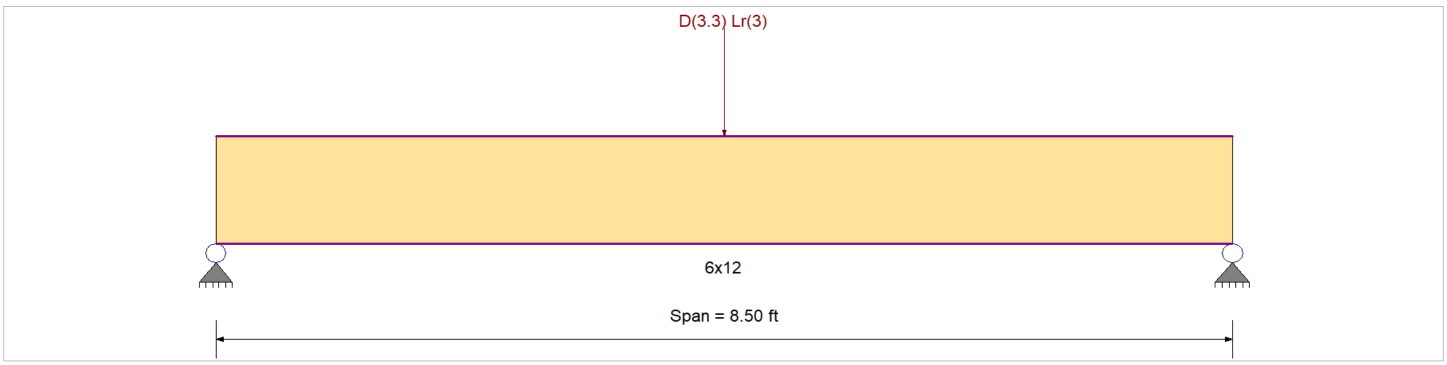
DESCRIPTION: Header 1

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,350.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	1,350.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	925.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1	Fv	170.0 psi		
		Ft	675.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Point Load : D = 3.30, Lr = 3.0 k @ 4.250 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.793** : 1	Maximum Shear Stress Ratio	=	**0.357** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
fb: Actual	=	1,337.43 psi	fv: Actual	=	75.78 psi
F'b	=	1,687.50 psi	F'v	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	4.250 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.060 in	Ratio =	1705	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.127 in	Ratio =	803	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Wood Beam

Project File: morgan hill.ec6

LIC# : KW-06015331, Build:20.22.10.25 ZFA STRUCTURAL ENGINEERS (c) ENERCALC INC 1983-2022

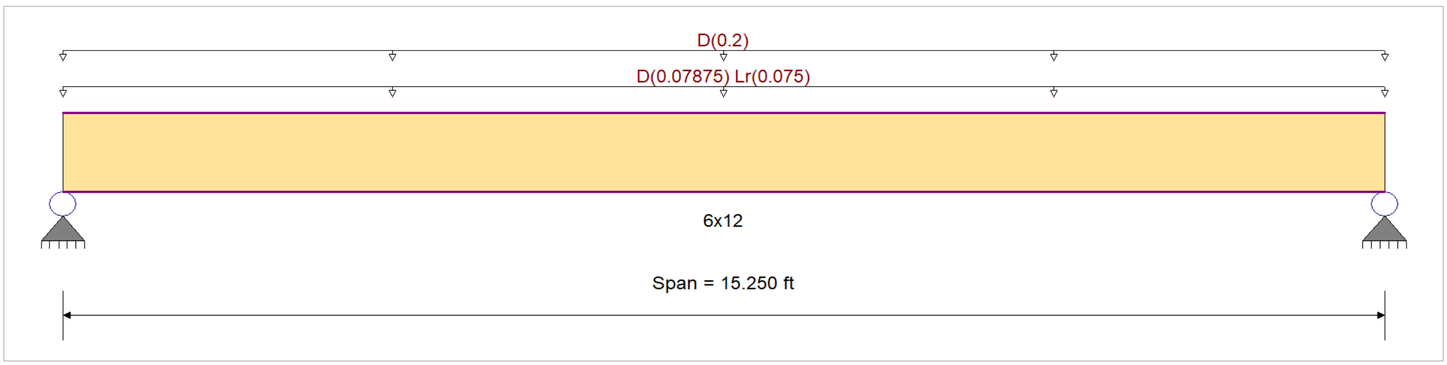
DESCRIPTION: Header 2

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,350.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	1,350.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	925.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1	Fv	170.0 psi		
		Ft	675.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Uniform Load : D = 0.0210, Lr = 0.020 ksf, Tributary Width = 3.750 ft, (Roof)
Uniform Load : D = 0.20 , Tributary Width = 1.0 ft, (Wall)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.693** : 1	Maximum Shear Stress Ratio	=	**0.303** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
fb: Actual	=	841.57 psi	fv: Actual	=	46.32 psi
F'b	=	1,215.00 psi	F'v	=	153.00 psi
Load Combination		D Only	Load Combination		D Only
Location of maximum on span	=	7.625 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.082 in	Ratio =	2223	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.403 in	Ratio =	453	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Lateral
Analysis & Design

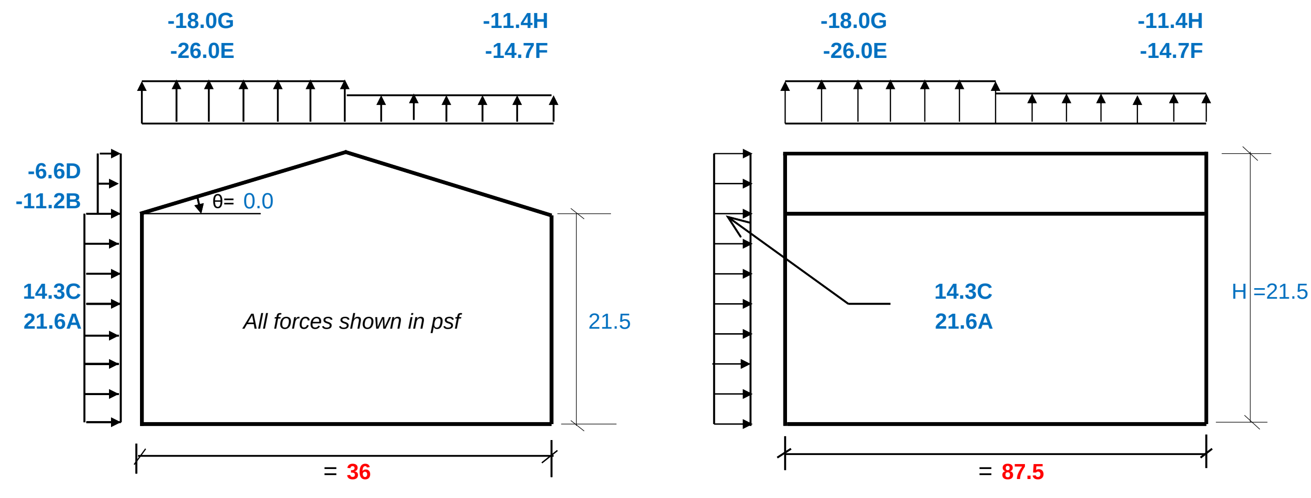
Revised design for 2022 CBC does not affect Wind analysis

ASCE 7-16 (CBC 2019) WIND: BUILDING DATA:

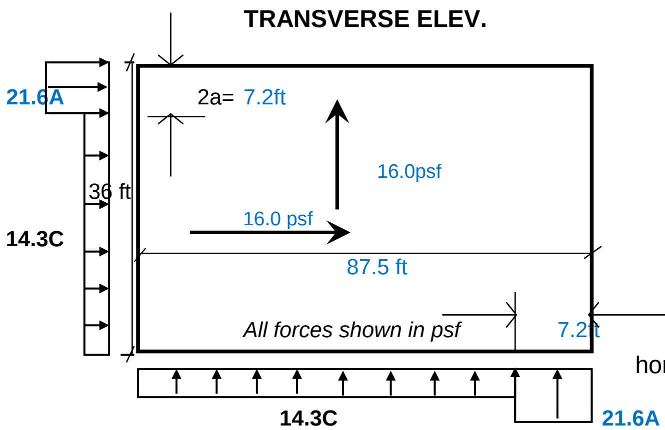
Basic wind speed (3 sec gust) = **102** MPH
Exposure C
Roof Pitch = **0.00** :12
Mean Roof Height h = **21.5** ft

28.5 ENVELOPE PROCEDURE PART 2- SIMPLIFIED PROCEDURE (LOW-RISE, 60 FT)

Height Adjustment factor λ = 1.31 Fig 28.5-1



TRANSVERSE ELEV. **LONGITUDINAL ELEV.**



PLAN VIEW

10 % of least dimension= 3.6 ft
40 % of the eave height = 8.6 ft
4 % of least dimension or 3 ft= 3.0 ft
therefore a = 3.6 ft
Example: $p_s = \lambda K_{zt} p_{s30}$ (28.5-1)
Topography factor K_{zt} = **1.00** 26.8
horizontal load at end zone p_{s30} = 16.5 Fig 28.5-1
Height Adjustment factor λ = 1.31 X Fig 28.5-1
21.6psf

FIGURE 28.5-1, Main Wind Force System

MWFRS ***Min total load per §28.5.4 governs!***

Load Direction	Roof Angle	Horizontal Loads				Vertical Loads					
		End Zone		Interior zone		End Zone		Interior zone		Overhang	
		Wall (A)	Roof (B)	Wall (C)	Roof (D)	WW (E)	LW (F)	WW (G)	LW (H)	E_{OH}	G_{OH}
Transverse	0.0	**21.6**	**-11.2**	**14.3**	**-6.6**	**-26.0**	**-14.7**	**-18.0**	**-11.4**	**-36.3**	**-28.5**
Longitudinal	All	**21.6**	**-11.2**	**14.3**	**-6.6**	**-26.0**	**-14.7**	**-18.0**	**-11.4**	**-36.3**	**-28.5**

* If roof pressure under horizontal loads is less than zero, use zero
Plus and minus signs signify pressures acting toward and away from projected surfaces, respectively.
For the design of the longitudinal MWFRS use Ө = 0°, and locate the zone E/F, G/H boundary at the mid-length of the building

CBC 2022, ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

DSA Project?	No			
Soil Site Class:	D	Table 20.3-1		
Response Spectral Acc. (0.2 sec) Ss =	1.582g	= 158.2%g		Figure 22-1, 22-3, 22-5, and 22-6
Response Spectral Acc. (1.0 sec) S1 =	0.600g	= 60.0%g		Figure 22-2, 22-4, 22-5, and 22-6
Site Coefficient F_a =	1.00			Table 11.4-1
Site Coefficient F_v =	1.70			Table 11.4-2
Max Considered Earthquake Acc. S_{MS} =	$F_a(S_s)$	= 1.582		(11.4-1)
Max Considered Earthquake Acc. S_{M1} =	$F_v(S_1)$	= 1.530		(11.4-2) with Exception per §11.4.8
S_{DS} =	$2/3(S_{MS})$	= 1.055	(at 5% Damped Design)	(11.4-3)
S_{D1} =	$2/3(S_{M1})$	= 0.680		(11.4-4)
T_S =	0.645 seconds			
Building Risk Category:	IV	Essential		Table 1.5-1
Redundancy Factor ρ =	1.0			Section 12.3.4
Design Category Consideration:	Flexible Diaphragm			Section 12.3
Seismic Design Category for 0.1sec	D			Table 11.6-1
Seismic Design Category for 1.0sec	D			Table 11.6-2
S1 < .75g	NA			Section 11.6
Since Ta < .8Ts (see below), SDC =	D	exception of Section 11.6 does not apply		

CBC - Comply with Seismic Design Category D

12.8 Equivalent Lateral Force Procedure

Seismic Force Resisting System: A. BEARING WALL SYSTEMS — T-12.2-1

15. Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance

C_t =	0.02	x = 0.75		T-12.8-2
Building Height H_n =	22 ft	Limited Building Height (ft) = 65		
C_u =	1.4	for S_{D1} of	0.680g	Table 12.8-1
Approx Fundamental Period, T_a =	$C_t(h_n)^x$	= 0.200	12.8-7	T_L = 12 Sec
Calculated T shall not exceed ≤	Cu*Ta	= 0.280		Use T = 0.200 Sec
0.8Ts =	$0.8(S_{D1}/S_{DS})$	= 0.516	exception of Section 11.6 does not apply	
Is structure Regular & ≤ 5 stories?	No			12.8.1.3
S_{DS} to determine C_s & E_v =	1.055g			11.4-3
Response Modification Coef. R =	6.5			Table-12.2-1
Overstrength Factor Ω_o =	2.5			Table-12.2-1 Footnote b
Deflection Amplification Factor C_d =	4			Table-12.2-1
Seismic Importance Factor I_e =	1.50			Table 1.5-1

Seismic Base Shear V = C_sW

$$C_s = \frac{S_{DS}}{R/I_e} = 0.243 \quad (12.8\text{-}2)$$

or need not to exceed, $$C_s = \frac{S_{D1}}{(R/I_e)T} = 0.786 \quad \text{For } T \le T_L \quad (12.8\text{-}3)$$

or $$C_s = \frac{S_{D1}T_L}{T^2(R/I_e)} = \text{N/A} \quad \text{For } T > T_L \quad (12.8\text{-}4)$$

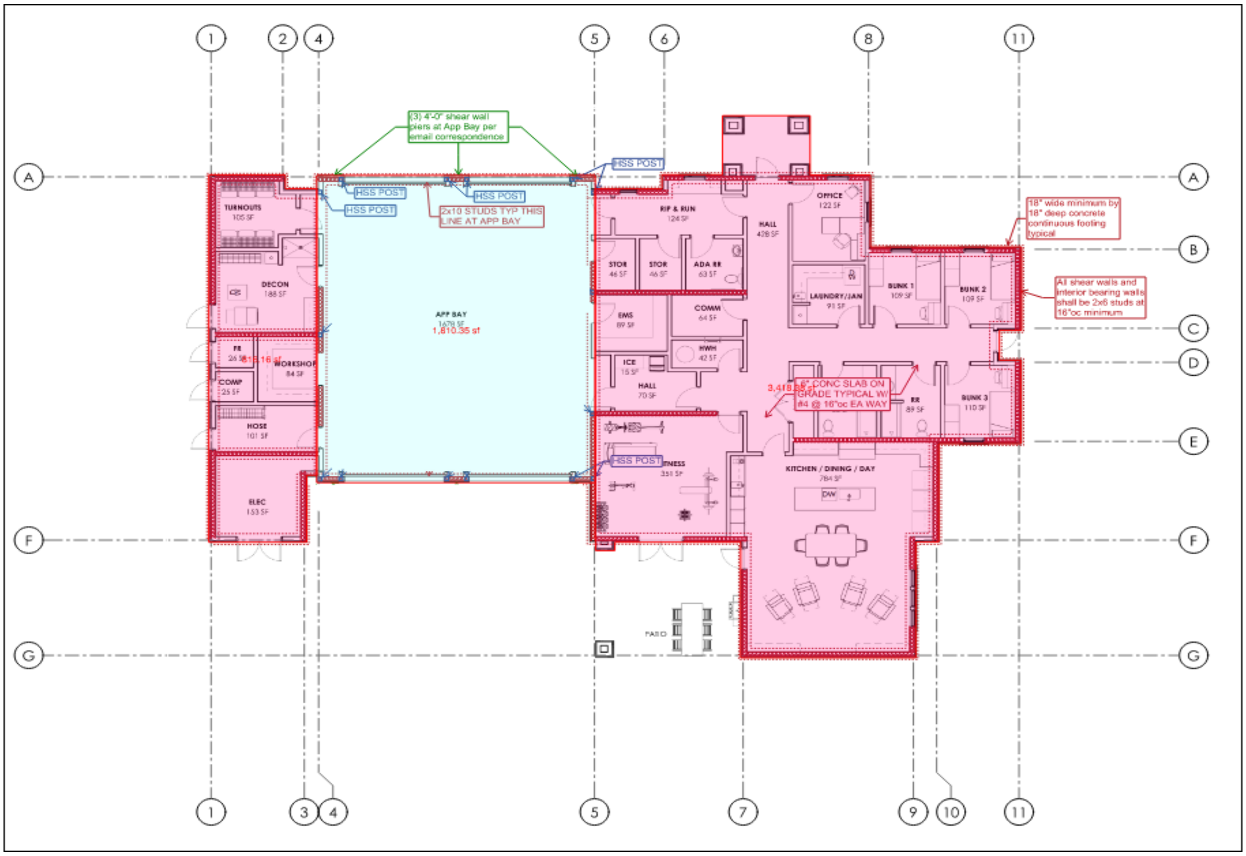
C_s shall not be less than…	0.070	≥ 0.01		(12.8-5)
Min C_s =	$0.5S_1I_e/R$	= 0.069	For S_1 ≥ 0.6g	(12.8-6)
Use C_s =	0.243			

Design Base Shear V (ULT) = 0.243 W
Design Base Shear V (ASD) = 0.170 W

By observation, seismic design forces governs over wind design forces in lateral force resisting systems

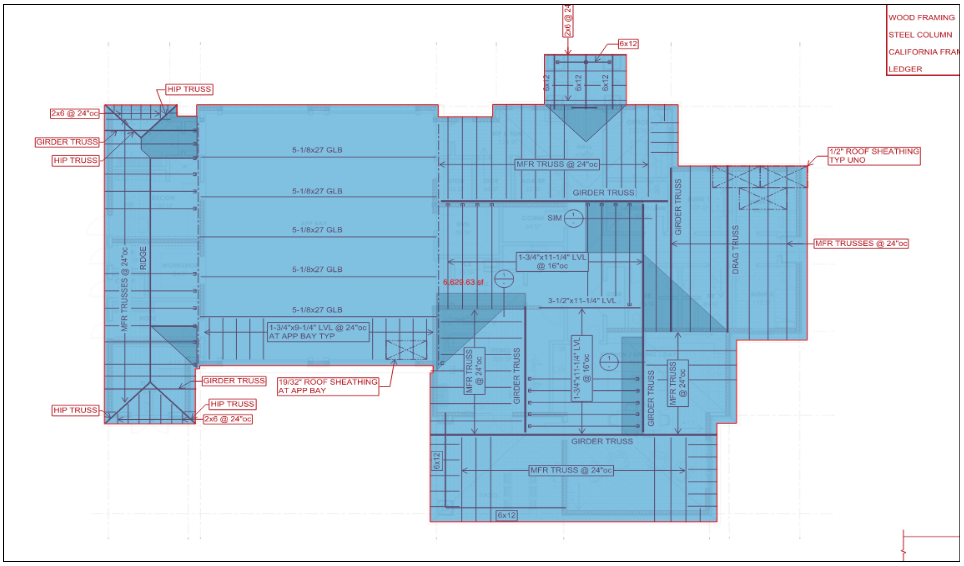
Seismic Weight Takeoff

Level	Diaphragm			Exterior Walls					Interior Walls					Level Weight (kips)
	Area (ft^2)	DL (psf)	Weight (kips)	DL (psf)	Trib Ht (ft)	Length (ft)	Area (ft^2)	Weight (kips)	DL (psf)	Trib Ht (ft)	Length (ft)	Area (ft^2)	Weight (kips)	
Apparatus Bay	4811	21	**101**	20	12.5	168	2100	**42**						**143**
Typical	1818	19	**35**	22	5.0	328	1640	**35**	10	5.0	461	2305	**23**	**93**
Σ	**6629**		**136**					**77**					**23**	**236**



Seismic Base Shear

Roof Plan:



Seismic Weight:

$w_{D, App Bay}$=	21 psf	Typical Roof Area:	4811 sf
$w_{D, TYP}$=	19 psf	App Bay Roof Area:	1818 sf
$W_{D, int wall}$=	10 psf	Total Roof Area:	6629 sf
$W_{D, App Bay ext wall}$=	20 psf		
$W_{D, TYP ext wall}$=	22 psf		
		Ext Wall Length (App Bay)	168 ft
TYP Wall height:	5 ft	Ext Wall Length (TYP)	328 ft
App Bay Wall height:	12.5 ft	Int Wall Length	461 ft

Seismic Weight: 230.7 kips

Seismic Base Shear:

Seismic Weight: 230.7 kips
Cs = 0.24 ULT

Cs Min	0.07

V =	56.15 ULT, Kips

39.30708

v =	8.47 psf, ULT
v =	5.93 psf, ASD

Seismic Diaphragm Loading:

Fpmin = 0.2SDS I Wx, ULT = 73.00 kips
Fpmax = 0.4SDS I Wx, ULT = 146.00 kips

Fpx = 56.15 kips

Fpmin Governs

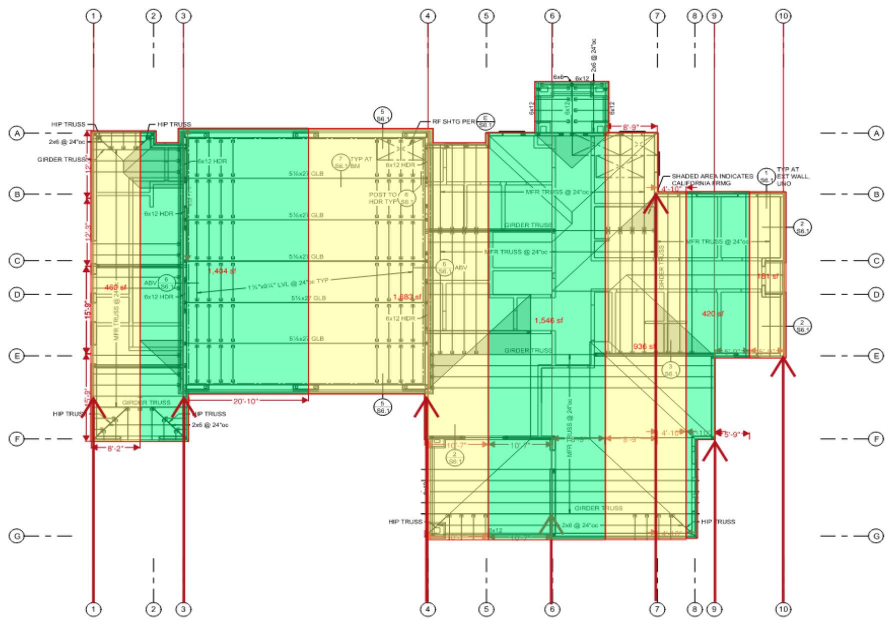
Fp =	73.00 kips

Factor of Fp/Fx = 1.3 *apply ot all collector and diaphragm forces

Line Load Distribution

v = 8.47 psf, ULT   Fp/Fx = 1.3

Plan North-South Loading



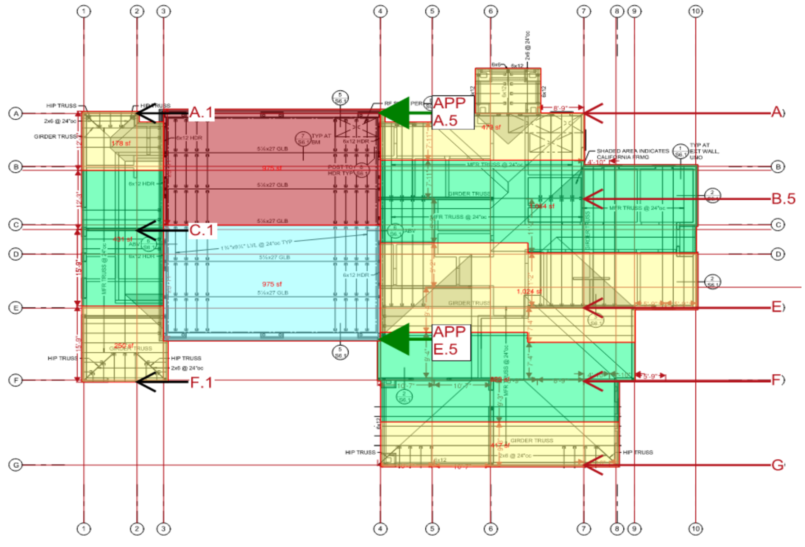
ROOF FRAMING PLAN

Line	Trib Area (SF)	Line Load, Kips, ULT	Line Load, Kips,ASD	Diaphragm Force, Kips, ULT	Diaphragm Force, Kips, ASD
1	460	3.90	2.73	5.07	3.55
3	1404	11.89	8.33	15.46	10.82
4	1683	14.26	9.98	18.53	12.97
6	1546	13.10	9.17	17.02	11.92
7	936	7.93	5.55	10.31	7.22
8&9	420	3.56	2.49	4.63	3.24
10	181	1.53	1.07	1.99	1.40
Total:	6630	56.16	39.31	73.01	51.11

Line Load Shear Wall 8 1.34 kips Separation of lateral load for the strapped shear wall along line 8

Line	Trib Area (SF)	Line Load, Kips, ULT	Line Load, Kips,ASD	Diaphragm Force, Kips, ULT	Diaphragm Force, Kips, ASD
3	1015.64	8.60	6.02	11.18	7.83
4	1015.64	8.60	6.02	11.18	7.83

Plan East-West Loading

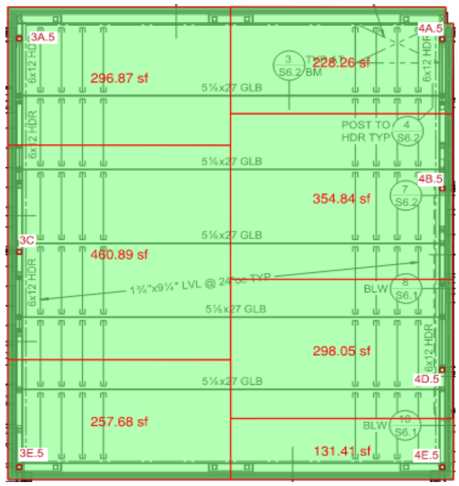


Line	Trib Area (SF)	Line Load, Kips, ULT	Line Load, Kips,ASD	Diaphragm Force, Kips, ULT	Diaphragm Force, Kips, ASD
A	475	4.02	2.82	5.23	3.66
B.5	1065	9.02	6.31	11.73	8.21
E	1025	8.68	6.08	11.29	7.90
F	825	6.99	4.89	9.08	6.36
G	420	3.56	2.49	4.63	3.24
APP A.5	975	8.26	5.78	10.74	7.52
APP E.5	975	8.26	5.78	10.74	7.52
A.1	180	1.52	1.07	1.98	1.39
C.1	435	3.68	2.58	4.79	3.35
F.1	255	2.16	1.51	2.81	1.97
Total:	6630	56.16	39.31	73.01	51.11

Line	Trib Area (SF)	Line Load, Kips, ULT	Line Load, Kips,ASD	Diaphragm Force, Kips, ULT	Diaphragm Force, Kips, ASD
A	475	5.73	4.01	7.45	5.21
B.5	1065	11.39	7.97	14.81	10.37
E	1025	10.67	7.47	13.87	9.71
A.1	180	3.51	2.46	4.56	3.19
C.1	435	6.78	4.75	8.81	6.17

Lower table indicates additional load being added by cantilevered columns shown in the column loading table below

Column Loading



Column	Trib Area (SF)	Roof Weight (lbs)	Line Load, Kips, ULT	Line Load Min, Kips ULT	Line Load, Kips,ASD
3A.5	298	6266.9	1.53	1.98	1.39
3C	465	9779.0	2.38	3.09	2.17
4E.5	256	5383.7	1.31	1.70	1.19
4A.5	230	4836.9	1.18	1.53	1.07
4B.5	356	7486.7	1.82	2.37	1.66
4D.5	298	6266.9	1.53	1.98	1.39
4E.5	131	2754.9	0.67	0.87	0.61

DIRECTION: N/S **WALL LINE: 1** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo N/S 2.5

Total Wall Line Shear (ASD)= V (lb) = **2728** ρ N/S 1

Wall Lengths = L (ft) =	**18.5**	**13**							

Total Wall Length = ΣL (ft) = **31.5**
Minimum Wall Length = L_{min} (ft) = **13**
Wall Height = h (ft) = **10**
Sheathing Grade = **CD / OSB**
Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **87**
HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 7**

Use SW CD / OSB w/ 10d@ 6"o.c. **(Capacity = 285 plf)** **DCR=** 0.30

Tributary Dead Load = DL (psf) =	**19**	**19**							
Tributary Width = TW (ft) =	**8**	**8**							
Wall Dead Load = WDL (psf) =	**21.5**	**21.5**							
Total Dead Load at Wall = w_{DL} (plf) =	367	367							
ASD Uplift Force =	0	0							
Minimum Holdown =	No Uplift	No Uplift							
LRFD Uplift Force =	0	0							
Overstrength LRFD Uplift Force =	796	1569							

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **0 lb**
Holdown Type = **HDU**

Use: No Uplift **(Capacity = 0 lb)** **DCR =** 0.00

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **0 lb**
Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **1569 lb**
Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **3604 lb**

DIRECTION: N/S **WALL LINE: 3** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo N/S 2.5

Total Wall Line Shear (ASD)= V (lb) = **8325** ρ N/S 1

Wall Lengths = L (ft) =	**5.5**	**9**	**10**						

Total Wall Length = ΣL (ft) = **24.5**
Minimum Wall Length = L_{min} (ft) = **5.5** **←H:W ratio below 2:1, reduce shear by 2w/h per SDPWS 4.3.3.4.1**
Wall Height = h (ft) = **14**
Sheathing Grade = **CD / OSB** Capacity Reduction (SDPWS Table 4.3.4.1) = **0.932**
Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **340**
HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 8**

Use SW CD / OSB w/ 10d@ 4"o.c. **(Capacity = 423 plf)** **DCR=** 0.80
Use SW CD / OSB w/ 10d@ 3" oc @ low ratio wall **(Red. Cap.= 434 plf)** **DCR=** 0.78

CALC MARK: 9

Tributary Dead Load = DL (psf) =	**19**	**19**	**19**						
Tributary Width = TW (ft) =	**26**	**25**	**6.5**						
Wall Dead Load = WDL (psf) =	**20**	**20**	**21.5**						
Total Dead Load at Wall = w_{DL} (plf) =	774	755	424.5						
ASD Uplift Force =	4638	3623	4219						
Minimum Holdown =	HDU5	HDU4	HDU4						
LRFD Uplift Force =	6514	5012	5926						
Overstrength LRFD Uplift Force =	18973	16480	17253						

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **4638 lb**
Holdown Type = **HDU**

Use: HDU5 **(Capacity = 5645 lb)** **DCR =** 0.82

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **6514 lb**
Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **18973 lb**
Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **7200 lb**

DIRECTION: N/S **WALL LINE: 4** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo N/S 2.5

Total Wall Line Shear (ASD)= V (lb) = 9979 ρ N/S 1

Wall Lengths = L (ft) =	5.5	19	5.5						

Total Wall Length = ΣL (ft) = 30

Minimum Wall Length = L_{min} (ft) = 5.5 ←H:W ratio below 2:1, reduce shear by 2w/h per SDPWS 4.3.3.4.1

Wall Height = h (ft) = 14

Sheathing Grade = CD / OSB Capacity Reduction (SDPWS Table 4.3.4.1) = 0.932

Nail Size = 10d Wall shear per lineal foot = v (plf) = V/L = 333

HD's Inside Post? YES ...shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 8**

Use SW CD / OSB w/ 10d@ 4"o.c. (Capacity = 423 plf) DCR= 0.79

Use SW CD / OSB w/ 10d@ 3" oc @ low ratio wall (Red. Cap.= 434 plf) DCR= 0.77

CALC MARK: 9

Tributary Dead Load = DL (psf) =	19	19	19						
Tributary Width = TW (ft) =	20	20	20						
Wall Dead Load = WDL (psf) =	20	20	20						
Total Dead Load at Wall = w_{DL} (plf) =	660	660	660						
ASD Uplift Force =	4689	1922	4689						
Minimum Holdown =	HDU5	HDU2	HDU5						
LRFD Uplift Force =	6603	2462	6603						
Overstrength LRFD Uplift Force =	18800	12996	18800						

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **4689 lb**

Holdown Type = HDU

Use: HDU5 (Capacity = 5645 lb) DCR = 0.83

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **6603 lb**

Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **18800 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **6740 lb**

DIRECTION: N/S **WALL LINE: 6** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo N/S 2.5

Total Wall Line Shear (ASD)= V (lb) = 9167 ρ N/S 1

Wall Lengths = L (ft) =	11.5								

Total Wall Length = ΣL (ft) = 11.5

Minimum Wall Length = L_{min} (ft) = 11.5

Wall Height = h (ft) = 10

Sheathing Grade = CD / OSB

Nail Size = 10d Wall shear per lineal foot = v (plf) = V/L = 797

HD's Inside Post? YES ...shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 11**

Use SW CD / OSB w/ 10d@ 3 both sides"o.c. (Capacity = 1104 plf) DCR= 0.72

Tributary Dead Load = DL (psf) =	19								
Tributary Width = TW (ft) =	2								
Wall Dead Load = WDL (psf) =	21.5								
Total Dead Load at Wall = w_{DL} (plf) =	253								
ASD Uplift Force =	8010								
Minimum Holdown =	HDU11								
LRFD Uplift Force =	11374								
Overstrength LRFD Uplift Force =	30083								

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **8010 lb**

Holdown Type = HDU

Use: HDU11 (Capacity = 9335 lb) DCR = 0.86

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **11374 lb**

Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **30083 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **9641 lb**

DIRECTION: N/S **WALL LINE: 8&9** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo N/S 2.5

Total Wall Line Shear (ASD)= V (lb) = **2490** ρ N/S 1

Wall Lengths = L (ft) = **14** | **17.5**

Total Wall Length = ΣL (ft) = **31.5**

Minimum Wall Length = L_{min} (ft) = **14**

Wall Height = h (ft) = **10**

Sheathing Grade = **CD / OSB**

Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **79**

HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 7**

Use SW **CD / OSB w/ 10d@ 3 both sides"o.c.** **(Capacity = 1104 plf)** **DCR=** 0.07

Tributary Dead Load =	DL (psf) =	**19**	**19**
Tributary Width =	TW (ft) =	**2**	**2**
Wall Dead Load =	WDL (psf) =	**21.5**	**21.5**
Total Dead Load at Wall =	w_{DL} (plf) =	253	253
ASD Uplift Force =		0	0
Minimum Holdown =		No Uplift	No Uplift
LRFD Uplift Force =		0	0
Overstrength LRFD Uplift Force =		1727	1377

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **0 lb**

Holdown Type = **HDU**

Use: No Uplift (Capacity = 0 lb) DCR = 0.00

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **0 lb**

Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **1727 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **2823 lb**

DIRECTION: N/S **WALL LINE: 10** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo N/S 2.5

Total Wall Line Shear (ASD)= V (lb) = **1073** ρ N/S 1

Wall Lengths =	L (ft) =	**12**	**12**							

Total Wall Length = ΣL (ft) = **24**

Minimum Wall Length = L_{min} (ft) = **12**

Wall Height = h (ft) = **10**

Sheathing Grade = **CD / OSB**

Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **45**

HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 7**

Use SW CD / OSB w/ 10d@ 6"o.c. (Capacity = 285 plf) DCR= 0.16

Tributary Dead Load =	DL (psf) =	**19.0**	**19.0**							
Tributary Width =	TW (ft) =	**2**	**3**							
Wall Dead Load =	WDL (psf) =	**21.5**	**21.5**							
Total Dead Load at Wall =	w_{DL} (plf) =	253.1	272.1							
ASD Uplift Force =		0	0							
Minimum Holdown =		No Uplift	No Uplift							
LRFD Uplift Force =		0	0							
Overstrength LRFD Uplift Force =		601	515							

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **0 lb**

Holdown Type = **HDU**

Use: No Uplift (Capacity = 0 lb) DCR = 0.00

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **0 lb**

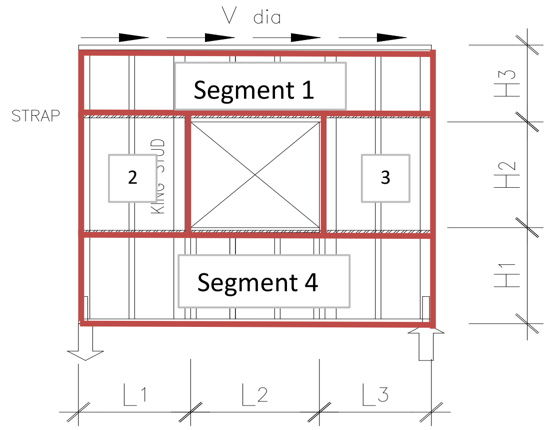
Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **601 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **2190 lb**

Strapped Shear Wall Line 7

L_1 = 4 ft , L_2 = 3 ft , L_3 = 4 ft
H_1 = 3 ft , H_2 = 5 ft , H_3 = 2 ft

Total Trib Width = 1



∑ forces = Seismic Force/Total SW Length * (Length Segment 2+3)

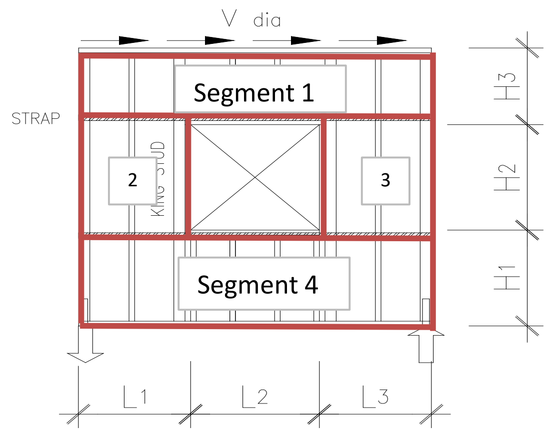
Story force = 5.6 k
Critical Length of SW = Segments 2 + 3
= 8 ft
Unit shear = Shear/L_{sw}
= 694 plf

Shear wall nailing = 2" o.c.
= 770 plf
HD's inside Post = YES
Reduced Capacity = 708.4 plf
DCR = 0.98

Strapped Shear Wall Line 7

L_1 = 4 ft , L_2 = 3 ft , L_3 = 4 ft
H_1 = 3 ft , H_2 = 5 ft , H_3 = 2 ft

Total Trib Width = 1



∑ forces = Seismic Force/Total SW Length * (Length Segment 2+3)

Story force =	5.6	k
Length of SW at foundation =	Segments 2 + 3	
	8	ft
Unit shear =	Shear/L_{sw}	
=	694	plf
Segment 1 Overturning Ht	4	ft
Shear	5.6	k
M_{OT}	22.20	k-ft
Wall DL	21.5	psf
Wall DL Trib	4	ft
Roof DL	19	psf
Roof DL Trib	4	ft
DL	162.04	plf
$0.6\text{-}0.14S_{DS}$*DL	73.30	plf
Segment Length	11	ft
Resisting Moment	4.43	k-ft
Uplift	1.78	k
Segment 2 Overturning Ht	5	ft
Shear	2.8	k
M_{OT}	13.88	k-ft
Wall DL	21.5	psf
Wall DL Trib	5	ft
Roof DL	19	psf
Roof DL Trib	0	ft
DL	107.55	plf
$0.6\text{-}0.14S_{DS}$*DL	48.65	plf
Segment Length	4	ft
Resisting Moment	0.39	k-ft
Uplift	4.50	k

Segment 3 Overturning Ht	5	ft
Shear	2.8	k
M_{OT}	13.88	k-ft
Wall DL	21.5	psf
Wall DL Trib	5	ft
Roof DL	19	psf
Roof DL Trib	0	ft
DL	107.55	plf
$0.6-0.14S_{DS}$*DL	48.65	plf
Segment Length	4	ft
Resisting Moment	0.39	k-ft
Uplift	4.50	k

Segment 4 Overturning Ht	3	ft
Shear	5.6	k
M_{OT}	16.65	k-ft
Wall DL	21.5	psf
Wall DL Trib	3	ft
Roof DL	0	psf
Roof DL Trib	0	ft
DL	64.53	plf
$0.6-0.14S_{DS}$*DL	29.19	plf
Segment Length	11	ft
Resisting Moment	1.77	k-ft
Uplift	1.49	k

Uplift Summary

Segment 1+2+3 Uplift	7.8	k
HD	HDU8	
Capacity	7.9	k
DCR =	99%	

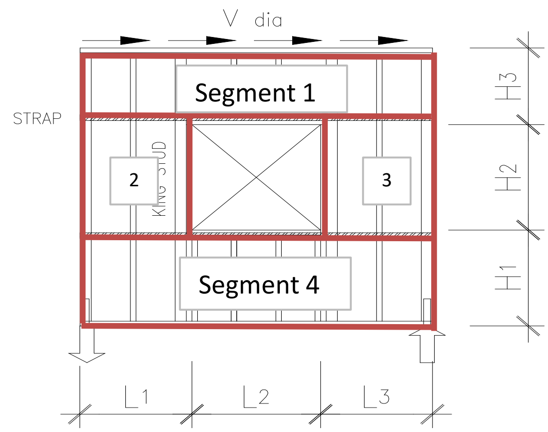
Segment 1+3+4 Uplift	7.8	k
HD	HDU8	
Capacity	7.9	k
DCR =	99%	

Segment 2/3+4 Uplift	6.0	k
HD	HDU8	
Capacity	7.9	k
DCR =	76%	

Strapped Shear Wall Line 8

L_1 = 4 L_2 = 9 L_3 = 4 ft
H_1 = 3 H_2 = 5 H_3 = 2 ft

Total Trib Width = 1



∑ forces = Seismic Force/Total SW Length * (Length Segment 2+3)
Story force = 2.5 k
Total Length of SW on Lines 8&9 = 31.5 ft
Unit Shear = Shear/L_{sw}
= 79 plf

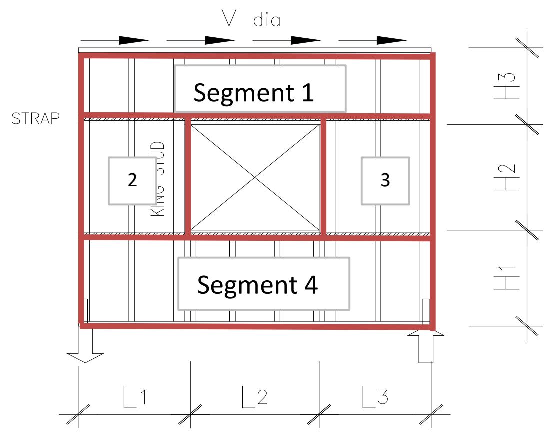
Shear on Line 8 = Unit Shear * Legth of Strapped Shear Wall
= 1.4 k
Unit Shear on Wall Line 8 = Shear on Line 8 / (L_1 + L_3)
= 173 plf

Shear wall nailing = 6" o.c.
= 310 plf
HD's inside Post = YES
Reduced Capacity = 285.2 plf
DCR = 0.61

Strapped Shear Wall Line 7

L_1 = 4 ft , L_2 = 3 ft , L_3 = 4 ft
H_1 = 3 ft , H_2 = 5 ft , H_3 = 2 ft

Total Trib Width = 1



∑ forces = Seismic Force/Total SW Length * (Length Segment 2+3)

Story force =	2.5	k
Total Length of SW on Lines 8&9 =	31.5	ft
Unit Shear =	Shear/L_{sw}	
=	79	plf
Shear on Line 8 =	Unit Shear * Legth of Strapped Shear Wall	
=	1.4	k
Unit Shear on Wall Line 8 =	Shear on Line 8 / (L_1 + L_3)	
=	173	plf

∑ forces = Seismic Force/Total SW Length * (Length Segment 2+3)

Story force =	5.6	k
Length of SW at foundation =	Segments 2 + 3	
	8	ft
Unit shear =	Shear/L_{sw}	
=	694	plf

Segment 1 Overturning Ht	4	ft
Shear	2.5	k
M_{OT}	9.96	k-ft
Wall DL	21.5	psf
Wall DL Trib	4	ft
Roof DL	19	psf
Roof DL Trib	4	ft
DL	162.04	plf
0.6-0.14S_{DS}*DL	73.30	plf
Segment Length	11	ft
Resisting Moment	4.43	k-ft
Uplift	0.55	k

Segment 2 Overturning Ht	5	ft
Shear	0.7	k
M_{OT}	3.46	k-ft
Wall DL	21.5	psf
Wall DL Trib	5	ft
Roof DL	19	psf
Roof DL Trib	0	ft
DL	107.55	plf
$0.6-0.14S_{DS}$*DL	48.65	plf
Segment Length	4	ft
Resisting Moment	0.39	k-ft
Uplift	1.02	k

Segment 3 Overturning Ht	5	ft
Shear	0.7	k
M_{OT}	3.46	k-ft
Wall DL	21.5	psf
Wall DL Trib	5	ft
Roof DL	19	psf
Roof DL Trib	0	ft
DL	107.55	plf
$0.6-0.14S_{DS}$*DL	48.65	plf
Segment Length	4	ft
Resisting Moment	0.39	k-ft
Uplift	1.02	k

Segment 4 Overturning Ht	3	ft
Shear	2.5	k
M_{OT}	7.47	k-ft
Wall DL	21.5	psf
Wall DL Trib	3	ft
Roof DL	0	psf
Roof DL Trib	0	ft
DL	64.53	plf
$0.6-0.14S_{DS}$*DL	29.19	plf
Segment Length	11	ft
Resisting Moment	1.77	k-ft
Uplift	0.57	k

Uplift Summary

Segment 1+2+3 Uplift	2.1	k
HD	HDU2	
Capacity	3.1	k
DCR =	70%	
Segment 1+3+4 Uplift	2.1	k
HD	HDU2	
Capacity	3.1	k
DCR =	70%	
Segment 2/3+4 Uplift	1.6	k
HD	HDU2	
Capacity	3.1	k
DCR =	52%	

DIRECTION: E/W **WALL LINE: A** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = **4009** ρ E/W 1

Wall Lengths =	L (ft) =	**3**	**3**	**6**	**6**					

Total Wall Length = ΣL (ft) = **18**

Minimum Wall Length = L_{min} (ft) = **3** **←H:W ratio below 2:1, reduce shear by 2b/h per SDPWS 4.3.3.4.1**

Wall Height = h (ft) = **10**

Sheathing Grade = **CD / OSB** Capacity Reduction (SDPWS Table 4.3.4.1) = **0.833**

Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **223**

HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 7**

Use SW **CD / OSB w/ 10d@ 6"o.c.** **(Capacity = 285 plf)** **DCR=** 0.78

Use SW **CD / OSB w/ 10d@ 4" o.c. @ low ratio wall** **(Red. Cap.= 254 plf)** **DCR=** 0.88

Tributary Dead Load =	DL (psf) =	**19**	**19**	**19**	**19**					
Tributary Width =	TW (ft) =	**8**	**8**	**9**	**9**					
Wall Dead Load =	WDL (psf) =	**21.5**	**21.5**	**21.5**	**21.5**					
Total Dead Load at Wall =	w_{DL} (plf) =	367.1	367.1	386.1	386.1					
	ASD Uplift Force =	2967	2967	2044	2044					
	Minimum Holdown =	HDU2	HDU2	HDU2	HDU2					
	LRFD Uplift Force =	4203	4203	2860	2860					
	Overstrength LRFD Uplift Force =	11362	11362	8587	8587					

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **2967 lb**

Holdown Type = **HDU**

Use: HDU2 **(Capacity = 3075 lb)** **DCR =** 0.96

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **4203 lb**

Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **11362 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **2859 lb**

DIRECTION: E/W **WALL LINE: B.5** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = **7973** ρ E/W 1

Wall Lengths =	L (ft) =	**20.5**								

Total Wall Length = ΣL (ft) = **20.5**

Minimum Wall Length = L_{min} (ft) = **20.5**

Wall Height = h (ft) = **14**

Sheathing Grade = **CD / OSB**

Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **389**

HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 8**

Use SW **CD / OSB w/ 10d@ 4"o.c.** **(Capacity = 423 plf)** **DCR=** 0.92

Tributary Dead Load =	DL (psf) =	**21**								
Tributary Width =	TW (ft) =	**4.5**								
Wall Dead Load =	WDL (psf) =	**20**								
Total Dead Load at Wall =	w_{DL} (plf) =	374.6								
	ASD Uplift Force =	3898								
	Minimum Holdown =	HDU4								
	LRFD Uplift Force =	5396								
	Overstrength LRFD Uplift Force =	17662								

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **3898 lb**

Holdown Type = **HDU**

Use: HDU4 **(Capacity = 4565 lb)** **DCR =** 0.85

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **5396 lb**

Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **17662 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **9852 lb**

DIRECTION: E/W **WALL LINE: E** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = **7466** ρ E/W 1

Wall Lengths =	L (ft) =	17	23							

Total Wall Length = ΣL (ft) = **40**
Minimum Wall Length = L_{min} (ft) = **17**
Wall Height = h (ft) = **10**
Sheathing Grade = **CD / OSB**
Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **187**
HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 7**

Use SW CD / OSB w/ 10d@ 6"o.c. **(Capacity = 285 plf)** **DCR=** 0.65

Tributary Dead Load = DL (psf) = **19** **19**
Tributary Width = TW (ft) = **17** **18**
Wall Dead Load = WDL (psf) = **15** **21.5**
Total Dead Load at Wall = w_{DL} (plf) = 473 557.1

ASD Uplift Force =	51	0								
Minimum Holdown =	Neglect	No Uplift								
LRFD Uplift Force =	0	0								
Overstrength LRFD Uplift Force =	4139	2354								

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **51 lb**
Holdown Type = **HDU**

Use: Neglect **(Capacity = 0 lb)** **DCR =** 0.00

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **0 lb**
Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **4139 lb**
Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **6481 lb**

DIRECTION: E/W **WALL LINE: F** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = **4892** ρ E/W 1

Wall Lengths =	L (ft) =	6	8							

Total Wall Length = ΣL (ft) = **14**
Minimum Wall Length = L_{min} (ft) = **6**
Wall Height = h (ft) = **10**
Sheathing Grade = **CD / OSB**
Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **349**
HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 8**

Use SW CD / OSB w/ 10d@ 4"o.c. **(Capacity = 423 plf)** **DCR=** 0.83

Tributary Dead Load = DL (psf) = **19** **19**
Tributary Width = TW (ft) = **2** **2**
Wall Dead Load = WDL (psf) = **21.5** **21.5**
Total Dead Load at Wall = w_{DL} (plf) = 253.1 253.1

ASD Uplift Force =	3781	3470								
Minimum Holdown =	HDU4	HDU4								
LRFD Uplift Force =	5362	4908								
Overstrength LRFD Uplift Force =	14347	13465								

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **3781 lb**
Holdown Type = **HDU**

Use: HDU4 **(Capacity = 4565 lb)** **DCR =** 0.83

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **5362 lb**
Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **14347 lb**
Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **4366 lb**

DIRECTION: E/W **WALL LINE: G** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = **2490** ρ E/W 1

Wall Lengths = L (ft) = **24**

Total Wall Length = ΣL (ft) = **24**

Minimum Wall Length = L_{min} (ft) = **24**

Wall Height = h (ft) = **10**

Sheathing Grade = **CD / OSB**

Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **104**

HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 7**

Use SW CD / OSB w/ 10d@ 6"o.c. **(Capacity = 285 plf)** **DCR=** 0.36

Tributary Dead Load = DL (psf) = **19.0**

Tributary Width = TW (ft) = **10**

Wall Dead Load = WDL (psf) = **15.0**

Total Dead Load at Wall = w_{DL} (plf) = 340

ASD Uplift Force =	0							
Minimum Holdown =	No Uplift							
LRFD Uplift Force =	0							
Overstrength LRFD Uplift Force =	933							

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **0 lb**

Holdown Type = **HDU**

Use: No Uplift **(Capacity = 0 lb)** **DCR =** 0.00

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **0 lb**

Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **933 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **5720 lb**

DIRECTION: E/W **WALL LINE: APP A.5** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = **5781** ρ E/W 1

Wall Lengths = L (ft) = **4** | **4** | **4**

Total Wall Length = ΣL (ft) = **12**

Minimum Wall Length = L_{min} (ft) = **4** **←H:W ratio below 2:1, reduce shear by 2w/h per SDPWS 4.3.3.4.1**

Wall Height = h (ft) = **14**

Sheathing Grade = **CD / OSB** Capacity Reduction (SDPWS Table 4.3.4.1) = **0.813**

Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **482**

HD's Inside Post? **NO** …no shear capacity reduction (SDPWS Table4.3A Note 10) **CALC MARK: 9**

Use SW CD / OSB w/ 10d@ 3"o.c. **(Capacity = 600 plf)** **DCR=** 0.80

Use SW CD / OSB w/ 10d@ 3" both sides oc **(Red. Cap.= 526 plf)** **DCR=** 0.92

CALC MARK: 12

Tributary Dead Load =	DL (psf) =	**19.0**	**19.0**	**19.0**					
Tributary Width =	TW (ft) =	**2**	**2**	**2**					
Wall Dead Load =	WDL (psf) =	**21.5**	**21.5**	**21.5**					
Total Dead Load at Wall =	w_{DL} (plf) =	339.1	339.1	339.1					
	ASD Uplift Force =	8584	8584	8584					
	Minimum Holdown =	HDU11	HDU11	HDU11					
	LRFD Uplift Force =	12224	12224	12224					
	Overstrength LRFD Uplift Force =	31495	31495	31495					

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **8584 lb**

Holdown Type = **HDU**

Use: HDU11 **(Capacity = 9335 lb)** **DCR =** 0.92

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **12224 lb**

Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **31495 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **7523 lb**

DIRECTION: E/W **WALL LINE: APP E.5** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = 5781 ρ E/W 1

Wall Lengths = L (ft) =	4	4	4						

Total Wall Length = ΣL (ft) = 12

Minimum Wall Length = L_{min} (ft) = 4 ←**H:W ratio below 2:1, reduce shear by 2w/h per SDPWS 4.3.3.4.1**

Wall Height = h (ft) = 14

Sheathing Grade = CD / OSB Capacity Reduction (SDPWS Table 4.3.4.1) = 0.813

Nail Size = 10d Wall shear per lineal foot = v (plf) = V/L = 482

HD's Inside Post? NO ...no shear capacity reduction (SDPWS Table4.3A Note 10) **CALC MARK: 9**

Use SW CD / OSB w/ 10d@ 3"o.c. **(Capacity = 600 plf)** **DCR=** 0.80

Use SW CD / OSB w/ 10d@ 3" both sides oc **(Red. Cap.= 526 plf)** **DCR=** 0.92

CALC MARK: 9

Tributary Dead Load = DL (psf) =	19.0	19.0	19.0						
Tributary Width = TW (ft) =	5.5	5.5	5.5						
Wall Dead Load = WDL (psf) =	20.0	20.0	20.0						
Total Dead Load at Wall = w_{DL} (plf) =	384.5	384.5	384.5						
ASD Uplift Force =	8529	8529	8529						
Minimum Holdown =	HDU11	HDU11	HDU11						
LRFD Uplift Force =	12141	12141	12141						
Overstrength LRFD Uplift Force =	31412	31412	31412						

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **8529 lb**

Holdown Type = HDU

Use: HDU11 (Capacity = 9335 lb) DCR = 0.91

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **12141 lb**

Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **31412 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **7627 lb**

DIRECTION: E/W **WALL LINE: A.1** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = 2455 ρ E/W 1

Wall Lengths = L (ft) =	9.5								

Total Wall Length = ΣL (ft) = 9.5

Minimum Wall Length = L_{min} (ft) = 9.5

Wall Height = h (ft) = 10

Sheathing Grade = CD / OSB

Nail Size = 10d Wall shear per lineal foot = v (plf) = V/L = 258

HD's Inside Post? YES ...shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 7**

Use SW CD / OSB w/ 10d@ 6"o.c. **(Capacity = 285 plf)** **DCR=** 0.91

Tributary Dead Load = DL (psf) =	19.0								
Tributary Width = TW (ft) =	2								
Wall Dead Load = WDL (psf) =	21.5								
Total Dead Load at Wall = w_{DL} (plf) =	253.1								
ASD Uplift Force =	2281								
Minimum Holdown =	HDU2								
LRFD Uplift Force =	3201								
Overstrength LRFD Uplift Force =	9391								

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **2281 lb**

Holdown Type = HDU

Use: HDU2 (Capacity = 3075 lb) DCR = 0.74

Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **3201 lb**

Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **9391 lb**

Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **3964 lb**

DIRECTION: E/W **WALL LINE: C.1** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = **4745** ρ E/W 1

Wall Lengths =	L (ft) =	**14**							

Total Wall Length = ΣL (ft) = **14**
Minimum Wall Length = L_{min} (ft) = **14**
Wall Height = h (ft) = **10**
Sheathing Grade = **CD / OSB**
Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **339**
HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 8**

Use SW CD / OSB w/ 10d@ 4"o.c. **(Capacity = 423 plf)** **DCR=** 0.80

Tributary Dead Load = DL (psf) = **19.0**
Tributary Width = TW (ft) = **2**
Wall Dead Load = WDL (psf) = **21.5**
Total Dead Load at Wall = w_{DL} (plf) = 253.1

ASD Uplift Force =	2787								
Minimum Holdown =	HDU2								
LRFD Uplift Force =	3900								
Overstrength LRFD Uplift Force =	11722								

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **2787 lb**
Holdown Type = **HDU**
Use: HDU2 **(Capacity = 3075 lb)** **DCR =** 0.91
Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **3900 lb**
Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **11722 lb**
Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **5423 lb**

DIRECTION: E/W **WALL LINE: F.1** **SHEAR LOAD: SEISMIC** Sds= 1.05 Ωo E/W 2.5

Total Wall Line Shear (ASD)= V (lb) = **1512** ρ E/W 1

Wall Lengths =	L (ft) =	**4**	**4**						

Total Wall Length = ΣL (ft) = **8**
Minimum Wall Length = L_{min} (ft) = **4** **←H:W ratio below 2:1, reduce shear by 2w/h per SDPWS 4.3.3.4.1**
Wall Height = h (ft) = **10**
Sheathing Grade = **CD / OSB** Capacity Reduction (SDPWS Table 4.3.4.1) = **0.938**
Nail Size = **10d** Wall shear per lineal foot = v (plf) = V/L = **189**
HD's Inside Post? **YES** …shear capacity reduction factor = 0.92 (SDPWS Table4.3A Note 10) **CALC MARK: 7**

Use SW CD / OSB w/ 10d@ 6"o.c. **(Capacity = 285 plf)** **DCR=** 0.66
Use SW CD / OSB w/ 10d@ 6"o.c. @ low ratio wall **(Red. Cap.= 228 plf)** **DCR=** 0.83

CALC MARK: 7

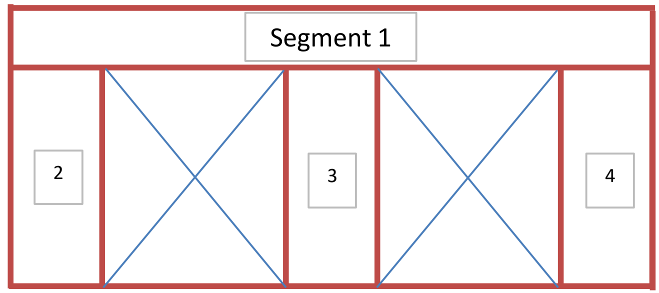
Tributary Dead Load = DL (psf) = **19.0** **19.0**
Tributary Width = TW (ft) = **2** **3**
Wall Dead Load = WDL (psf) = **21.5** **21.5**
Total Dead Load at Wall = w_{DL} (plf) = 253.1 272.1

ASD Uplift Force =	2215	2192							
Minimum Holdown =	HDU2	HDU2							
LRFD Uplift Force =	3135	3100							
Overstrength LRFD Uplift Force =	8535	8500							

Max. Uplift Force at Tie Down = {(v*h*L) - [(0.6-.14Sds)*wDL*L2/2]} / (L-1') = U_{ASD} = **2215 lb**
Holdown Type = **HDU**
Use: HDU2 **(Capacity = 3075 lb)** **DCR =** 0.72
Max. LRFD Uplift Force = {(v/.7*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = U_{LRFD} = **3135 lb**
Max. Overstrength Uplift Force= {(Ωo*v/.7/ρ*h*L) - [(.9-.2Sds)*wDL*L2/2]} / (L-1') = $\Omega_o U_{LRFD}$ = **8535 lb**
Max. Comp Force at End Post = {(v*h*L) + [(1.0+.14Sds)*wDL*L2/2]} / (L) = C_{ASD} = **2471 lb**

Strapped Shear Wall App Bay A.5 and E.5

L_1 = 40 ft , H_1 = 4 ft
L_2 = 4 ft , $H_{2,3,4}$= 14 ft
L_3 = 4 ft
L_4 = 4 ft



∑ forces = Seismic Force/Total SW Length * (Length Segment 2+3+4)

Story force =	5.8	k
Length of SW at foundation =	Segments 2 + 3 + 4	
	12	ft
Unit shear =	Shear/L_{sw}	
=	481	plf
Segment 1 Overturning Ht	4	ft
Shear	5.8	k
M_{OT}	23.08	k-ft
Wall DL	20.0	psf
Wall DL Trib	4	ft
Roof DL	21	psf
Roof DL Trib	0	ft
DL	80	plf
$0.6-0.14S_{DS}$*DL	36.19	plf
Segment Length	40	ft
Resisting Moment	28.95	k-ft
Uplift	-0.15	k
Segment 2 Overturning Ht	14	ft
Shear	1.9	k
M_{OT}	26.92	k-ft
Wall DL	20.0	psf
Wall DL Trib	14	ft
Roof DL	21	psf
Roof DL Trib	4	ft
DL	364.12	plf
$0.6-0.14S_{DS}$*DL	164.71	plf
Segment Length	4	ft
Resisting Moment	1.32	k-ft
Uplift	8.54	k

Segment 3 Overturning Ht	14	ft
Shear	1.9	k
M_{OT}	26.92	k-ft
Wall DL	20.0	psf
Wall DL Trib	14	ft
Roof DL	21	psf
Roof DL Trib	4	ft
DL	364.12	plf
$0.6\text{-}0.14S_{DS}$*DL	164.71	plf
Segment Length	4	ft
Resisting Moment	1.32	k-ft
Uplift	8.54	k

Segment 4 Overturning Ht	14	ft
Shear	1.9	k
M_{OT}	26.92	k-ft
Wall DL	20.0	psf
Wall DL Trib	14	ft
Roof DL	21	psf
Roof DL Trib	4	ft
DL	364.12	plf
$0.6\text{-}0.14S_{DS}$*DL	164.71	plf
Segment Length	4	ft
Resisting Moment	1.32	k-ft
Uplift	8.54	k

Uplift Summary

Segment 2/3/4 Uplift	8.4	k
HD	HDU11	
Capacity	11.2	k
DCR =	75%	

Collector Force Diagram

LOCATION (Grid): 1

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 3.55 kips
EQ/Wind force (Cd=1.6)? Yes
NUMBER OF SEGMENTS n = 3
Irregularity Increase per ASCE 7 §12.3.3.4? no
Wall Height = 10 ft
Fp, asd (Collector) 3.55 kips

Segment	1	2	3									
Length, ft	18.5	22.5	13									
Shear Wall ?	Yes	No	Yes									

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 54 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 31.5 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = 66 plf

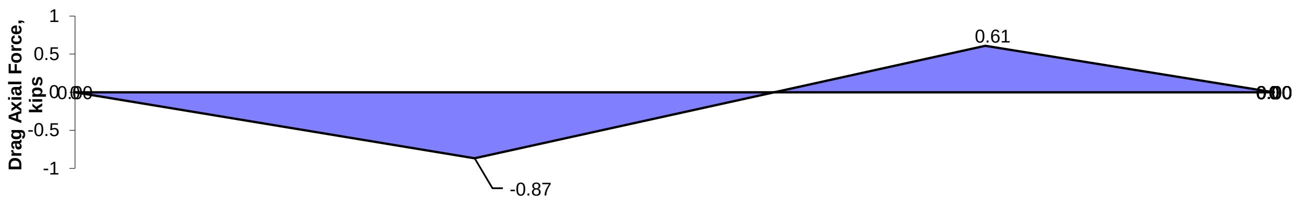
Section Point	0	1	2	3
Distance, ft	0	18.5	41	54
Axial Force	0	**-0.87**	**0.61**	**0.00**
		A	A	A
		A	A	A

Double Top Plate Splice:
1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (12) 16d PER 4' MIN LAP or MSTC28

Ledger Splice:
1 MSTA24
2 MSTA24
3 MSTA24

Conservatively use A **For ALL top plate splices on this line.**
Conservatively use A **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM



SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): 3 (Ledger at loof roof)

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p, ASD}$ = 10.82 kips   EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 6   Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft   Fp, asd (Collector) 10.82 kips

Segment	1	2	3	4	5	6						
Length, ft	17	9.5	5	5.5	4	10						
Shear Wall ?	no	Yes	no	yes	no	yes						

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 51 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 25 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **212** plf

Section Point	0	1	2	3	4	5	6
Distance, ft	0	17	26.5	31.5	37	41	51
Axial Force	0	**3.61**	**1.51**	**2.57**	**1.36**	**2.21**	**0.00**
		B	A	B	A	A	A
		C	A	C	A	C	A

Double Top Plate Splice:

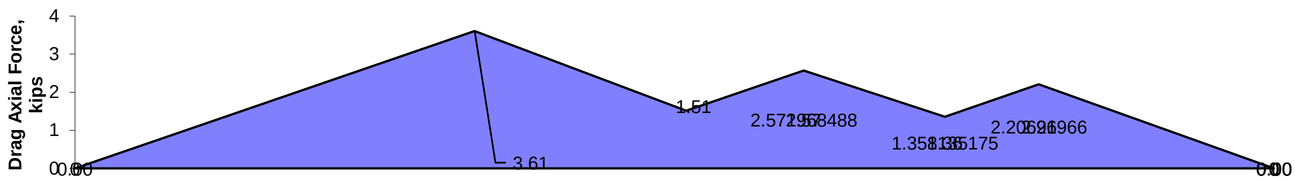
1 (22) 16d PER 4' MIN LAP or MSTC40
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (22) 16d PER 4' MIN LAP or MSTC40
4 (12) 16d PER 4' MIN LAP or MSTC28
5 (12) 16d PER 4' MIN LAP or MSTC28
6 (12) 16d PER 4' MIN LAP or MSTC28

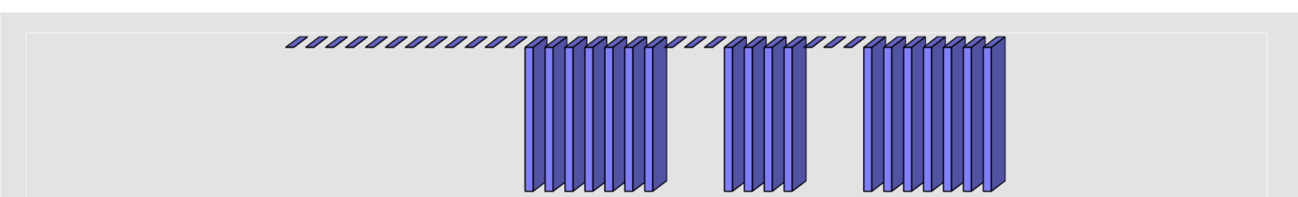
Ledger Splice:

1 MSTI48
2 MSTA24
3 MSTI48
4 MSTA24
5 MSTI48
6 MSTA24

Conservatively use B For ALL top plate splices on this line.
Conservatively use C For ALL ledger splices on this line.

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): 3 APP (Ledger at High App Bay I

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 7.83 kips EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 6 Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 14 ft Fp, asd (Collector) 7.83 kips

Segment	1	2	3	4	5	6						
Length, ft	17	9.5	5	5.5	4	10						
Shear Wall ?	no	Yes	no	yes	no	yes						

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 51 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 25 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **154** plf

Section Point	0	1	2	3	4	5	6
Distance, ft	0	17	26.5	31.5	37	41	51
Axial Force	0	**2.61**	**0.95**	**1.71**	**1.14**	**1.75**	**0.00**
		B	A	A	A	A	A
		C	A	B	A	B	A

Double Top Plate Splice:

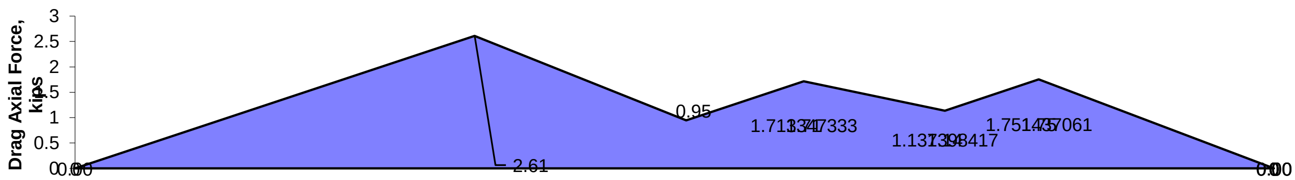
1 (22) 16d PER 4' MIN LAP or MSTC40
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (12) 16d PER 4' MIN LAP or MSTC28
4 (12) 16d PER 4' MIN LAP or MSTC28
5 (12) 16d PER 4' MIN LAP or MSTC28
6 (12) 16d PER 4' MIN LAP or MSTC28

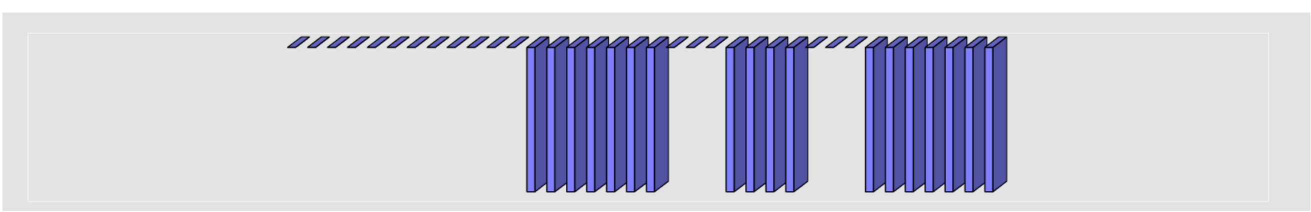
Ledger Splice:

1 MSTI48
2 MSTA24
3 MSTA30
4 MSTA24
5 MSTA30
6 MSTA24

Conservatively use B For ALL top plate splices on this line.
Conservatively use C For ALL ledger splices on this line.

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): 4

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 12.97 kips

EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 6

Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft

Fp, asd (Collector) 12.97 kips

Segment	1	2	3	4	5	6
Length, ft	13	5.5	3.5	5	8	19
Shear Wall ?	no	Yes	no	yes	no	yes

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 54 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 29.5 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **240** plf

Section Point	0	1	2	3	4	5	6
Distance, ft	0	13	18.5	22	27	35	54
Axial Force	0	**3.12**	**2.03**	**2.87**	**1.87**	**3.79**	**0.00**
		B	A	B	A	B	A
		C	B	C	B	C	A

Double Top Plate Splice:

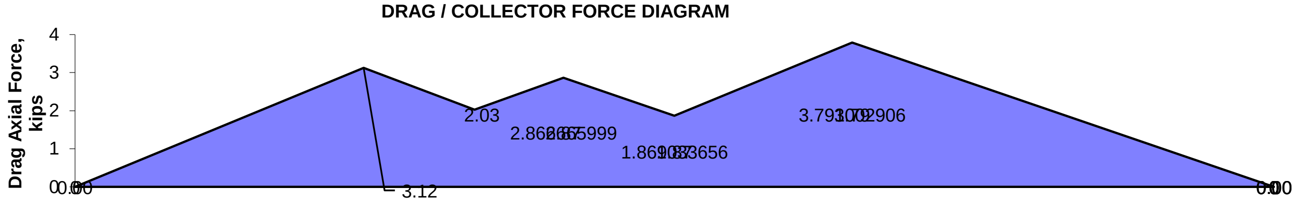
1 (22) 16d PER 4' MIN LAP or MSTC40
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (22) 16d PER 4' MIN LAP or MSTC40
4 (12) 16d PER 4' MIN LAP or MSTC28
5 (22) 16d PER 4' MIN LAP or MSTC40
6 (12) 16d PER 4' MIN LAP or MSTC28

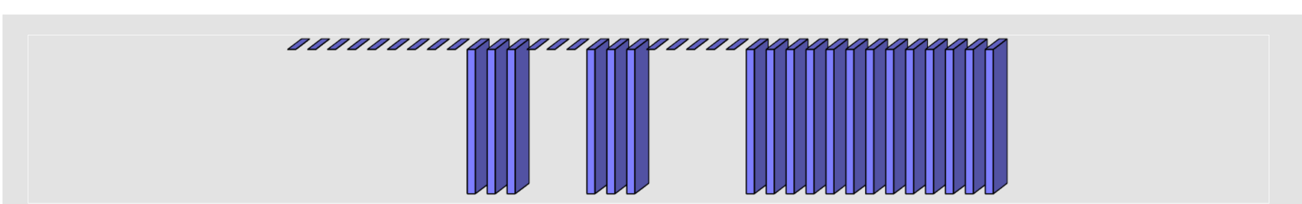
Ledger Splice:

1 MSTI48
2 MSTA30
3 MSTI48
4 MSTA30
5 MSTI48
6 MSTA24

Conservatively use B For ALL top plate splices on this line.
Conservatively use C For ALL ledger splices on this line.

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): 4 APP (Ledger at High App Bay I

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 7.83 kips   EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 6   Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 14 ft   Fp, asd (Collector) 7.83 kips

Segment	1	2	3	4	5	6						
Length, ft	13	5.5	3.5	5	8	19						
Shear Wall ?	no	Yes	no	yes	no	yes						

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 54 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 29.5 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = 145 plf

Section Point	0	1	2	3	4	5	6
Distance, ft	0	13	18.5	22	27	35	54
Axial Force	0	**1.88**	**1.42**	**1.93**	**1.62**	**2.78**	**0.00**
		A	A	A	A	B	A
		B	A	B	A	C	A

Double Top Plate Splice:

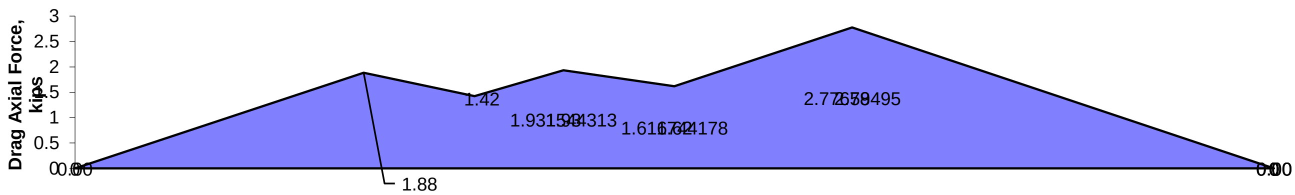
1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (12) 16d PER 4' MIN LAP or MSTC28
4 (12) 16d PER 4' MIN LAP or MSTC28
5 (22) 16d PER 4' MIN LAP or MSTC40
6 (12) 16d PER 4' MIN LAP or MSTC28

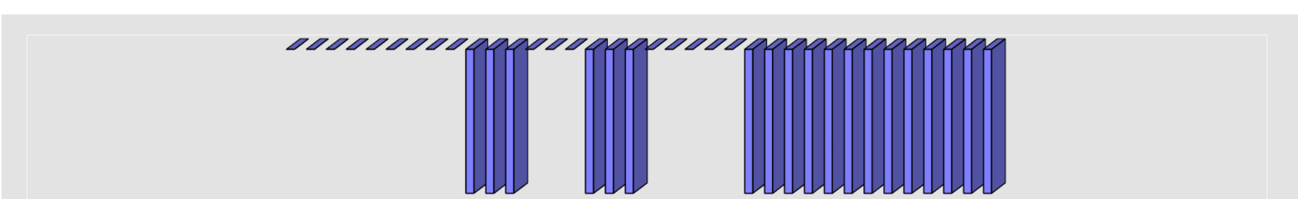
Ledger Splice:

1 MSTA30
2 MSTA24
3 MSTA30
4 MSTA24
5 MSTI48
6 MSTA24

Conservatively use B **For ALL top plate splices on this line.**
Conservatively use C **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): 6

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 11.92 kips

EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 2

Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft

Fp, asd (Collector) 11.92 kips

Segment	1	2										
Length, ft	60.5	12.5										
Shear Wall ?	no	Yes										

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 73 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 12.5 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **163** plf

Section Point	0	1	2
Distance, ft	0	60.5	73
Axial Force	0	**9.88**	**0.00**
		H	A
		F	A

Double Top Plate Splice:

1 MSTC66 EA SIDE
2 (12) 16d PER 4' MIN LAP or MSTC28

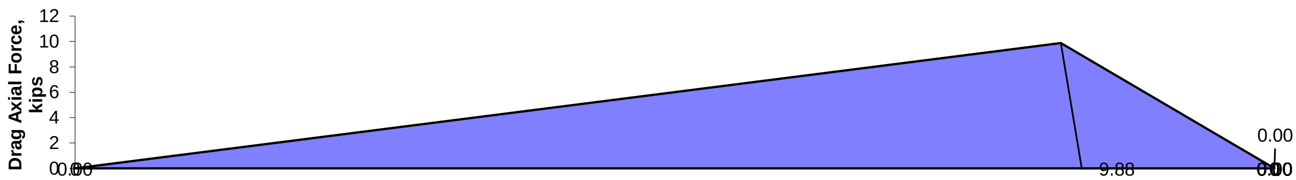
Ledger Splice:

1 (1) 1/4" PL w/ (6) 1" Dia MB EE at 2x
2 MSTA24

Top plates will not be spliced as indicated on plan, therefore top plate splice not required

Conservatively use H **For ALL top plate splices on this line.**
Conservatively use F **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM



SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): 7

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 7.22 kips — EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 2 — Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft — Fp, asd (Collector) 7.22 kips

Segment	1	2										
Length, ft	11	61										
Shear Wall ?	Yes	No										

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 72 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 11 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **100** plf

Section Point	0	1	2
Distance, ft	0	11	72
Axial Force	0	**-6.11**	**0.00**
		E	A
		D	A

Double Top Plate Splice:

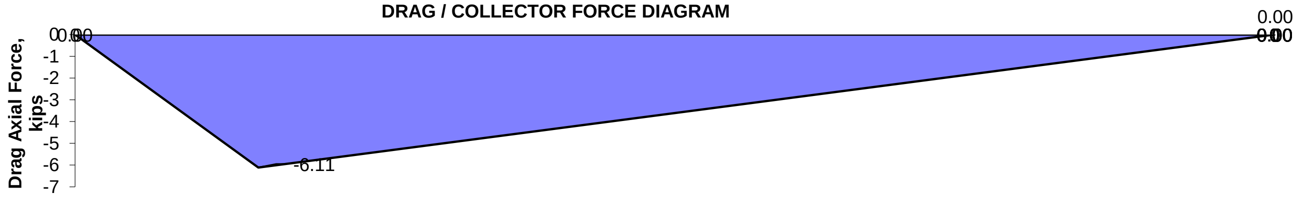
1 (36) 16d PER 8' MIN LAP or MSTC28 EA SIDE
2 (12) 16d PER 4' MIN LAP or MSTC28

Ledger Splice:

1 CMST14x5'-6"
2 MSTA24

Conservatively use E **For ALL top plate splices on this line.**
Conservatively use D **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM



SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): 8&9

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 3.24 kips

EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 2

Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft

Fp, asd (Collector) 3.24 kips

Segment	1	2										
Length, ft	29	32.5										
Shear Wall ?	no	Yes										

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 61.5 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 32.5 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **53** plf

Section Point	0	1	2
Distance, ft	0	29	61.5
Axial Force	0	**1.53**	**0.00**
		A	A
		A	A

Double Top Plate Splice:

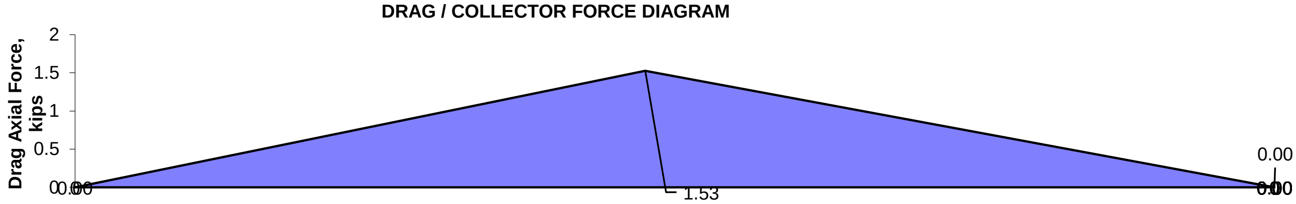
1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28

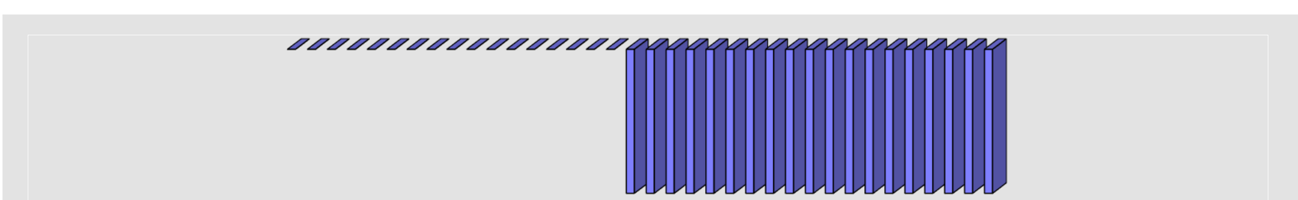
Ledger Splice:

1 MSTA24
2 MSTA24

Conservatively use $\boxed{A}$ **For ALL top plate splices on this line.**
Conservatively use A **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): 10

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 1.40 kips
EQ/Wind force (Cd=1.6)? Yes
NUMBER OF SEGMENTS n = 3
Irregularity Increase per ASCE 7 §12.3.3.4? no
Wall Height = 10 ft
Fp, asd (Collector) 1.40 kips

Segment	1	2	3									
Length, ft	12	6	12									
Shear Wall ?	Yes	No	Yes									

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 30 ft
TOTAL SHEAR WALL LENGTH L_{wall} = 24 ft
DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = 47 plf

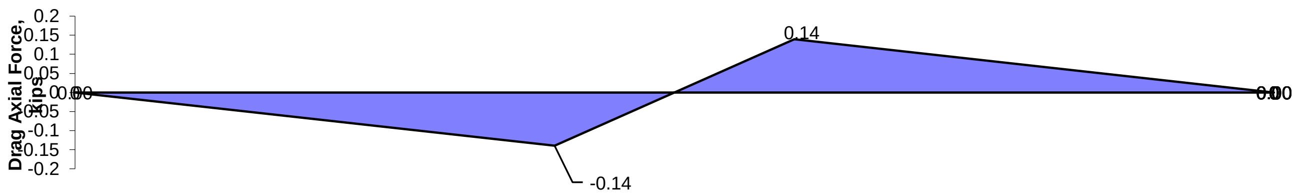
Section Point	0	1	2	3
Distance, ft	0	12	18	30
Axial Force	0	**-0.14**	**0.14**	**0.00**
		A	A	A
		A	A	A

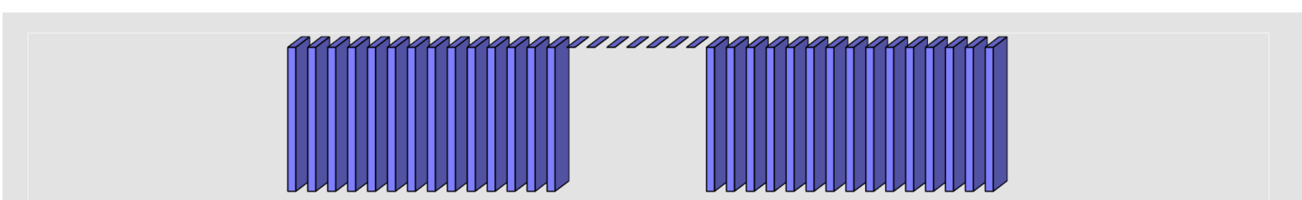
Double Top Plate Splice:
1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (12) 16d PER 4' MIN LAP or MSTC28

Ledger Splice:
1 MSTA24
2 MSTA24
3 MSTA24

Conservatively use A **For ALL top plate splices on this line.**
Conservatively use A **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): A

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 3.66 kips | EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 8 | Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft | Fp, asd (Collector) 3.66 kips

Segment	1	2	3	4	5	6	7	8				
Length, ft	3	3	3	6	6	3.5	6	6				
Shear Wall ?	Yes	No	Yes	no	yes	no	yes	no				

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 36.5 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 18 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = 100 plf

Section Point	0	1	2	3	4	5	6	7	8				
Distance, ft	0	3	6	9	15	21	24.5	30.5	36.5				
Axial Force	0	**-0.12**	**0.18**	**0.06**	**0.66**	**-0.15**	**0.20**	**-0.60**	**0.00**				
		A	A	A	A	A	A	A	A				
		A	A	A	A	A	A	A	A				

Double Top Plate Splice:

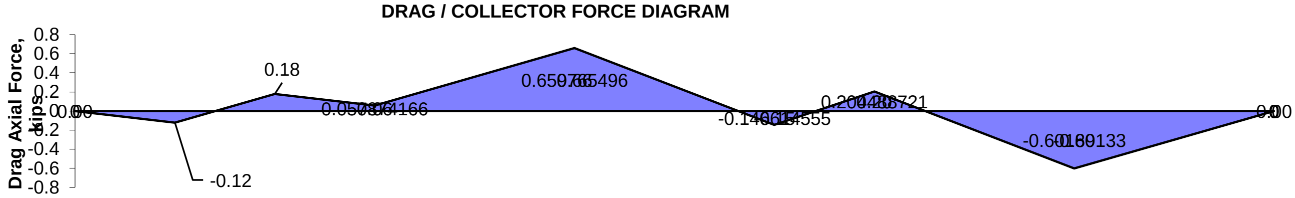
1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (12) 16d PER 4' MIN LAP or MSTC28
4 (12) 16d PER 4' MIN LAP or MSTC28
5 (12) 16d PER 4' MIN LAP or MSTC28
6 (12) 16d PER 4' MIN LAP or MSTC28
7 (12) 16d PER 4' MIN LAP or MSTC28
8 (12) 16d PER 4' MIN LAP or MSTC28

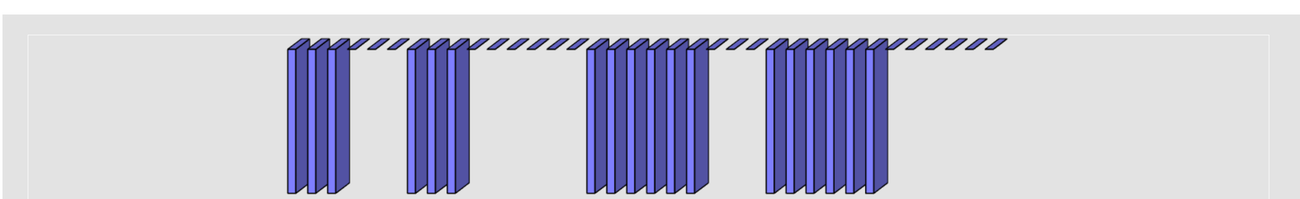
Ledger Splice:

1 MSTA24
2 MSTA24
3 MSTA24
4 MSTA24
5 MSTA24
6 MSTA24
7 MSTA24
8 MSTA24

Conservatively use A **For ALL top plate splices on this line.**
Conservatively use A **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): B.5

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\ ASD}$ = 8.21 kips

EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 2

Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft

Fp, asd (Collector) 8.21 kips

Segment	1	2										
Length, ft	38	20.5										
Shear Wall ?	no	Yes										

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 58.5 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 20.5 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **140** plf

Section Point	0	1	2
Distance, ft	0	38	58.5
Axial Force	0	**5.33**	**0.00**
		D	A
		D	A

Double Top Plate Splice:

1 (32) 16d PER 6' MIN LAP or MSTC66

2 (12) 16d PER 4' MIN LAP or MSTC28

Ledger Splice:

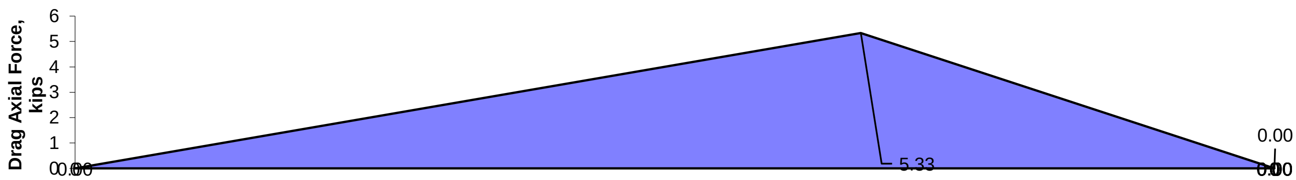
1 CMST14x5'-6"

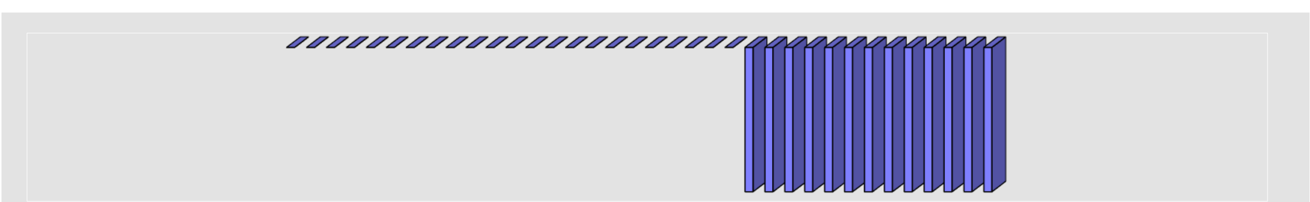
2 MSTA24

Conservatively use D **For ALL top plate splices on this line.**

Conservatively use D **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): E

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 7.90 kips

EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 3

Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft

Fp, asd (Collector) 7.90 kips

Segment	1	2	3									
Length, ft	17	10.5	31.5									
Shear Wall ?	Yes	No	Yes									

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 59 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 48.5 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **134** plf

Section Point	0	1	2	3
Distance, ft	0	17	27.5	59
Axial Force	0	**-0.49**	**0.91**	**0.00**
		A	A	A
		A	A	A

Double Top Plate Splice:

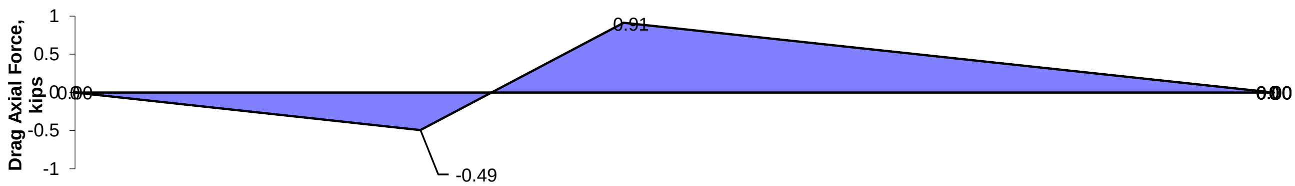
1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (12) 16d PER 4' MIN LAP or MSTC28

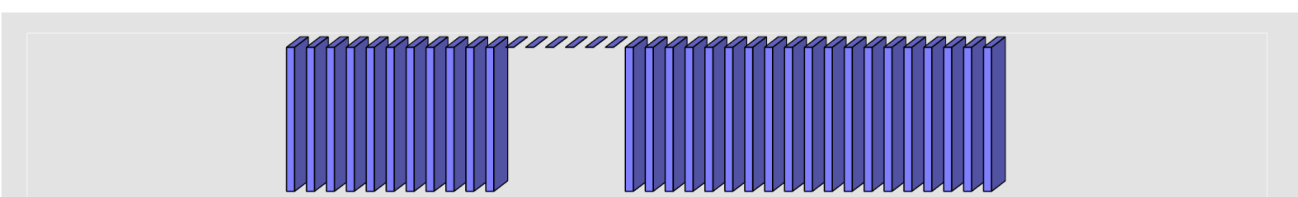
Ledger Splice:

1 MSTA24
2 MSTA24
3 MSTA24

Conservatively use A For ALL top plate splices on this line.
Conservatively use A For ALL ledger splices on this line.

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): F

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 6.36 kips

EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 4

Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft

Fp, asd (Collector) 6.36 kips

Segment	1	2	3	4								
Length, ft	27.5	8	6.5	6								
Shear Wall ?	no	Yes	no	yes								

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 48 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 14 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = 132 plf

Section Point	0	1	2	3	4
Distance, ft	0	27.5	35.5	42	48
Axial Force	0	3.64	1.07	1.93	0.00
		B	A	A	A
		C	A	B	A

Double Top Plate Splice:

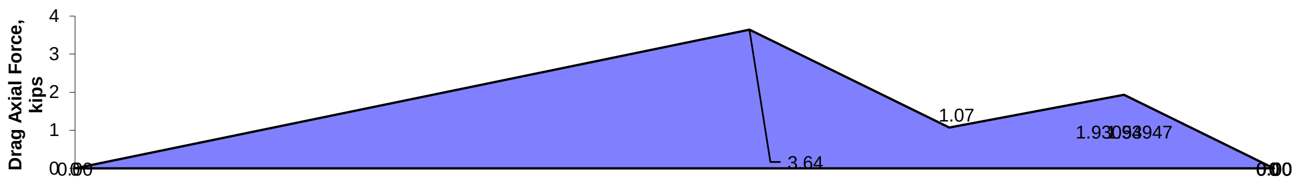
1 (22) 16d PER 4' MIN LAP or MSTC40
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (12) 16d PER 4' MIN LAP or MSTC28
4 (12) 16d PER 4' MIN LAP or MSTC28

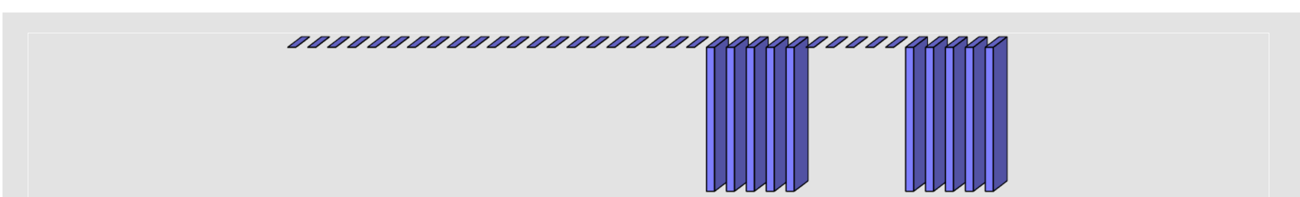
Ledger Splice:

1 MSTI48
2 MSTA24
3 MSTA30
4 MSTA24

Conservatively use B **For ALL top plate splices on this line.**
Conservatively use C **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): G

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p, ASD}$ = 3.24 kips

EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 2

Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft

Fp, asd (Collector) 3.24 kips

Segment	1	2										
Length, ft	24	24										
Shear Wall ?	no	Yes										

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 48 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 24 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **67** plf

Section Point	0	1	2
Distance, ft	0	24	48
Axial Force	0	**1.62**	**0.00**
		A	A
		A	A

Double Top Plate Splice:

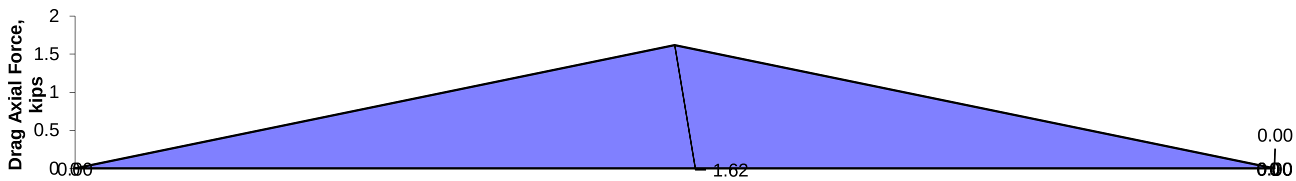
1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28

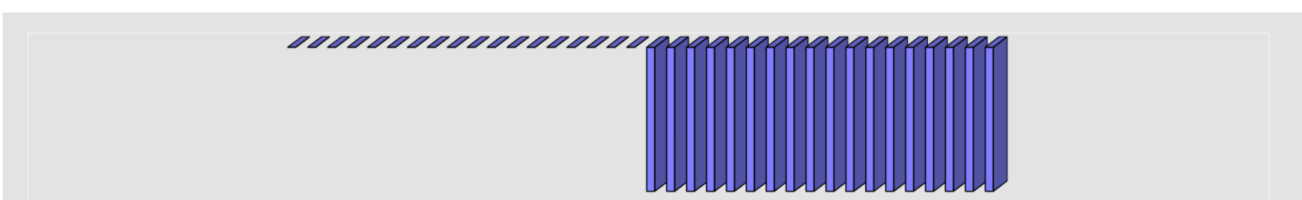
Ledger Splice:

1 MSTA24
2 MSTA24

Conservatively use A **For ALL top plate splices on this line.**
Conservatively use A **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): APP A.5

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 7.52 kips

EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 5

Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 14 ft

Fp, asd (Collector) 7.52 kips

Segment	1	2	3	4	5							
Length, ft	4	14	4	14	4							
Shear Wall ?	Yes	No	Yes	no	yes							

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 40 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 12 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **188** plf

Section Point	0	1	2	3	4	5
Distance, ft	0	4	18	22	36	40
Axial Force	0	**-1.75**	**0.88**	**-0.88**	**1.75**	**0.00**
		A	A	A	A	A
		B	A	A	B	A

Double Top Plate Splice:

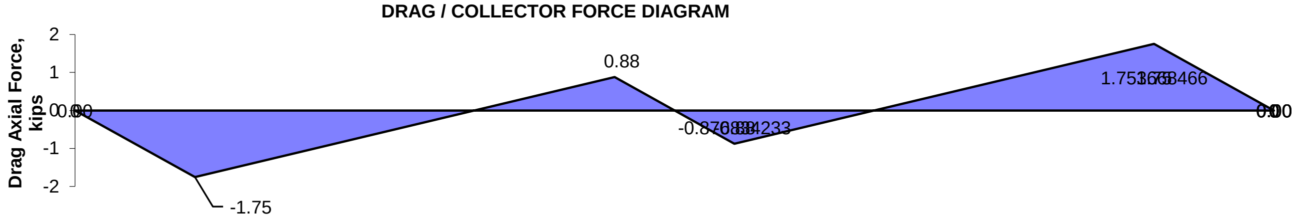
1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (12) 16d PER 4' MIN LAP or MSTC28
4 (12) 16d PER 4' MIN LAP or MSTC28
5 (12) 16d PER 4' MIN LAP or MSTC28

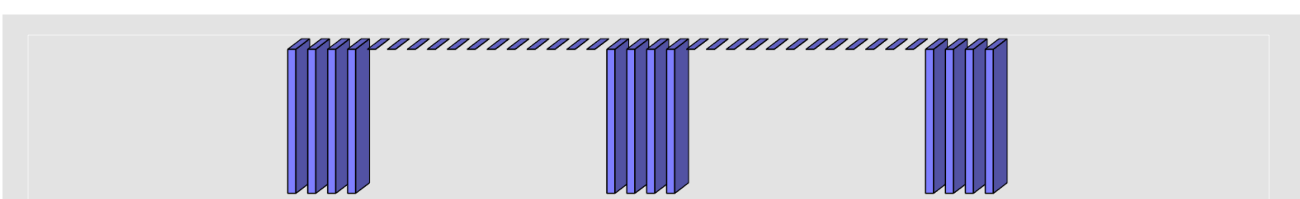
Ledger Splice:

1 MSTA30
2 MSTA24
3 MSTA24
4 MSTA30
5 MSTA24

Conservatively use A **For ALL top plate splices on this line.**
Conservatively use B **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): A.1

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 1.39 kips
EQ/Wind force (Cd=1.6)? Yes
NUMBER OF SEGMENTS n = 2
Irregularity Increase per ASCE 7 §12.3.3.4? no
Wall Height = 10 ft
Fp, asd (Collector) 1.39 kips

Segment	1	2										
Length, ft	9.5	4.5										
Shear Wall ?	Yes	No										

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 14 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 9.5 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = 99 plf

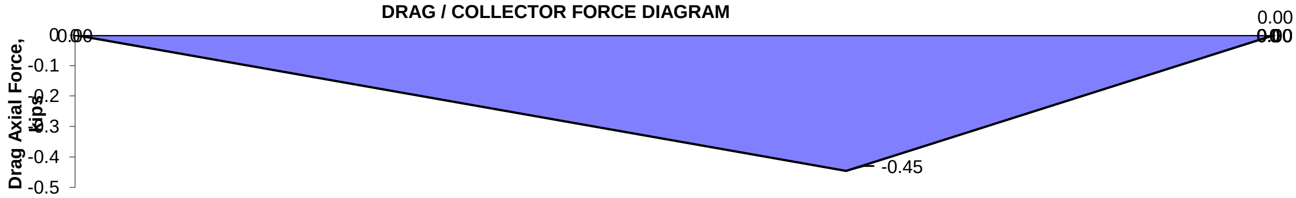
Section Point	0	1	2
Distance, ft	0	9.5	14
Axial Force	0	**-0.45**	**0.00**
		A	A
		A	A

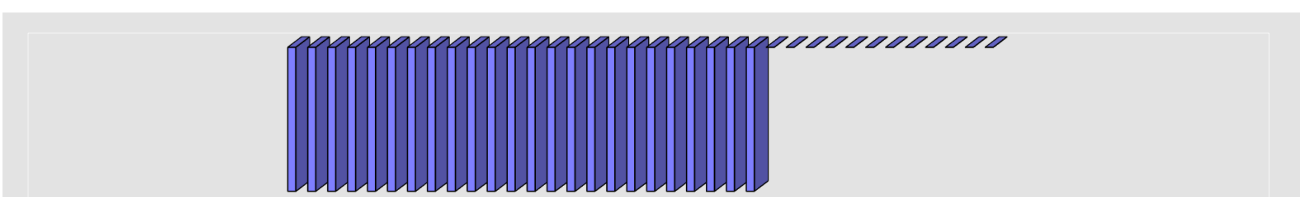
Double Top Plate Splice:
1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28

Ledger Splice:
1 MSTA24
2 MSTA24

Conservatively use $\boxed{A}$ **For ALL top plate splices on this line.**
Conservatively use A **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): C.1

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 3.35 kips

EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 1

Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft

Fp, asd (Collector) 3.35 kips

Segment	1											
Length, ft	14											
Shear Wall ?	Yes											

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 14 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 14 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **240** plf

Section Point	0	1
Distance, ft	0	14
Axial Force	0	**0.00**

A

A

Double Top Plate Splice:

1 (12) 16d PER 4' MIN LAP or MSTC28

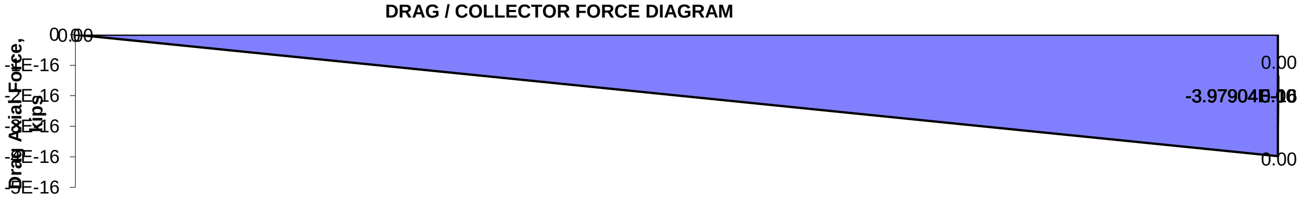
Ledger Splice:

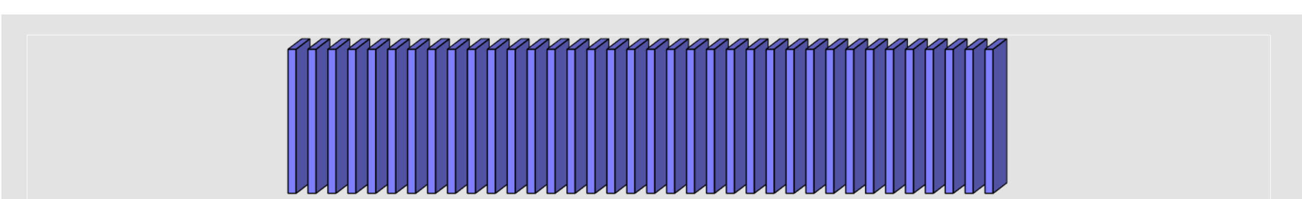
1 MSTA24

Conservatively use A **For ALL top plate splices on this line.**

Conservatively use A **For ALL ledger splices on this line.**

DRAG / COLLECTOR FORCE DIAGRAM





SHEAR WALL & DRAG SCHEMATIC ELEVATION

Collector Force Diagram

LOCATION (Grid): F.1

Diaphragm Force per ASCE 7 §12.10.1.1 (ASD) $F_{p,\,ASD}$ = 1.97 kips — EQ/Wind force (Cd=1.6)? Yes

NUMBER OF SEGMENTS n = 3 — Irregularity Increase per ASCE 7 §12.3.3.4? no

Wall Height = 10 ft — Fp, asd (Collector) 1.97 kips

Segment	1	2	3									
Length, ft	4	6.3	4									
Shear Wall ?	Yes	No	Yes									

ANALYSIS

TOTAL DRAG LENGTH L_{drag} = 14.3 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 8 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = **137** plf

Section Point	0	1	2	3
Distance, ft	0	4	10.3	14.3
Axial Force	0	**-0.43**	**0.43**	**0.00**
		A	A	A
		A	A	A

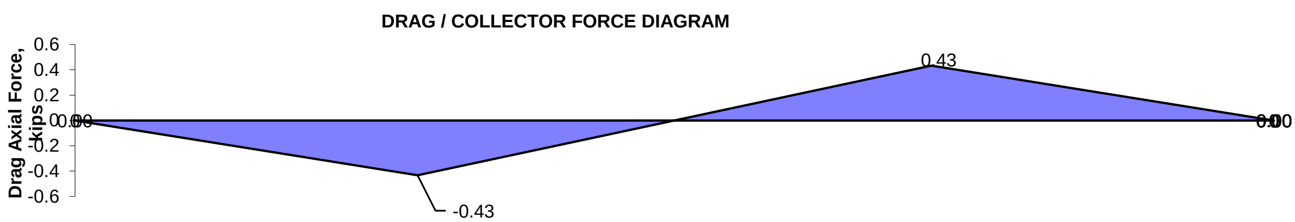
Double Top Plate Splice:

1 (12) 16d PER 4' MIN LAP or MSTC28
2 (12) 16d PER 4' MIN LAP or MSTC28
3 (12) 16d PER 4' MIN LAP or MSTC28

Ledger Splice:

1 MSTA24
2 MSTA24
3 MSTA24

Conservatively use $\boxed{A}$ **For ALL top plate splices on this line.**
Conservatively use A **For ALL ledger splices on this line.**



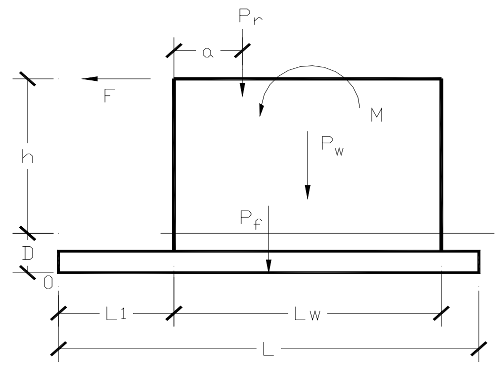
SHEAR WALL & DRAG SCHEMATIC ELEVATION

Foundation
Analysis & Design

Revised design for 2022 CBC does not affect foundation analysis

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: 1



WALL LENGTH	L_w =	13	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	17	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	1.976	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	6.5	ft
WALL SELF WEIGHT	P_w =	2.80	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	1.12	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi

Demands Summary

Stability	0.19	Bearing	0.14
Flexure-Top Steel	0.00	Flexure-Bot Steel	0.09
Shear	0.06	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL		2	#	5				
BOTTOM BARS, LONGITUDINAL		2	#	5				
SHEAR REINFORCEMENT	#	3	@	24	in o.c.	2	legs	**← Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 8.01 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 5.55 kips (footing self weight)

$M_o = 0.75\,[F\,(h + D) + M] =$ 11 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 88 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 10.32 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 88 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 1.06 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

= 0.56 ksf **<** q_b

[Satisfactory]

Where e = 1.06 ft, < (L / 6)

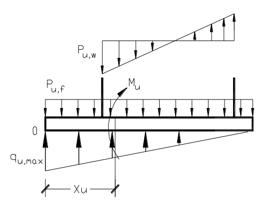
CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 105 ft-kips

$M_{u,o} =$ $1.4\,[F(h + D) + M] =$ 20 ft-kips

$P_u =$ $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 12 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 1.65 ft

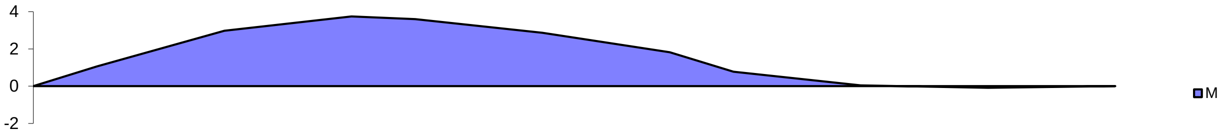


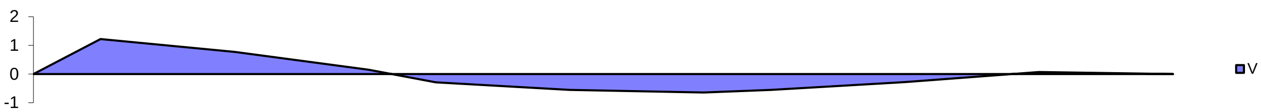
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

0.77 ksf

BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	1.70	3.40	5.10	6.80	8.50	10.20	11.90	13.60	15.30	17.00
$P_{u,w}$ (klf)	0.0	0.0	1.0	0.8	0.6	0.4	0.3	0.1	-0.1	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	-1	-5	-11	-20	-29	-39	-49	-59	-69
$V_{u,w}$ (kips)	0	0	-2	-3	-4	-5	-6	-6	-6	-6	-6
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-1	-2	-5	-9	-14	-20	-28	-36	-46	-57
$V_{u,f}$ (kips)	0	-1	-1	-2	-3	-3	-4	-5	-5	-6	-7
q_u (ksf)	-0.8	-0.7	-0.7	-0.6	-0.5	-0.5	-0.4	-0.4	-0.3	-0.3	-0.2
$M_{u,q}$ (ft-k)	0	2	6	14	24	37	51	68	86	105	126
$V_{u,q}$ (kips)	0	2	4	5	7	8	9	10	11	12	12
ΣM_u (ft-k)	**0**	**1**	**3**	**4**	**4**	**3**	**2**	**1**	**0**	**0**	**0**
ΣV_u (kips)	**0**	**1**	**1**	**0**	**0**	**-1**	**-1**	**-1**	**0**	**0**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	14.31	0.0000	0.0025	1	kips	21	kips
Bottom Longitudinal	4	ft-k	14.31	0.0002	0.0025	1	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where

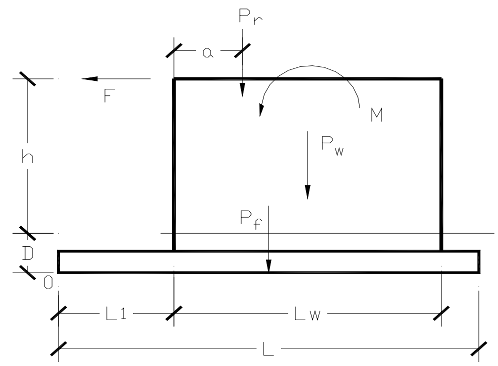
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\ max}$ =	85	kips	$(8\ b\ d\ f_c'^{0.5})$
ϕV_s =	5.9	kips	$(A_v\ f_{yt}\ d\ /\ s)$
ϕV_n =	21	kips	$(\phi V_s + \phi V_c)$
$A_{v\ min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\ provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14



INPUT DATA: 3

WALL LENGTH	L_w =	5.5	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	9.5	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	3	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.68	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	2.75	ft
WALL SELF WEIGHT	P_w =	1.1	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	1.87	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi
TOP BARS, LONGITUDINAL		4	# 5
BOTTOM BARS, LONGITUDINAL		4	# 5
SHEAR REINFORCEMENT	#	3	@ 24 in o.c. 2 legs ← Not Required

Demands Summary

Stability	0.75	Bearing	0.18
Flexure-Top Steel	0.02	Flexure-Bot Steel	0.13
Shear	0.14	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 2.08 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 6.20 kips (footing self weight)

$Mo = 0.75 [F (h + D) + M] =$ 18 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 38 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 7.98 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 38 ft-kips (resisting moment with live load)

$e = 0.5 L - (M_R - M_O) / P =$ 2.28 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \frac{P\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{2P}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases}$$

= 0.72 ksf **<** q_b **[Satisfactory]**

Where e = 2.28 ft, > (L / 6)

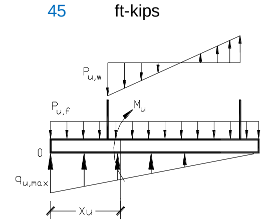
CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ 1.2 $[P_{r,DL} (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)] + 0.5 P_{r,LL}(L_1 + a) =$ 45 ft-kips

$M_{u,o} =$ 1.4 [F(h + D) + M] = 34 ft-kips

$P_u =$ 1.2 $(P_{r,DL} + P_f + P_w) + 0.5 P_{r,LL} =$ 10 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 3.55 ft

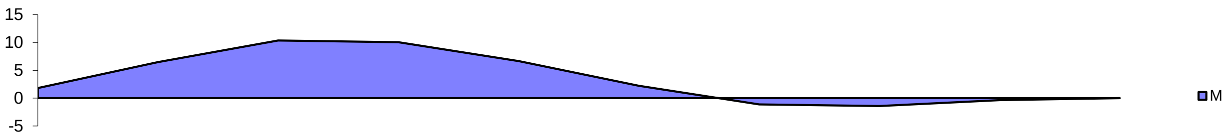


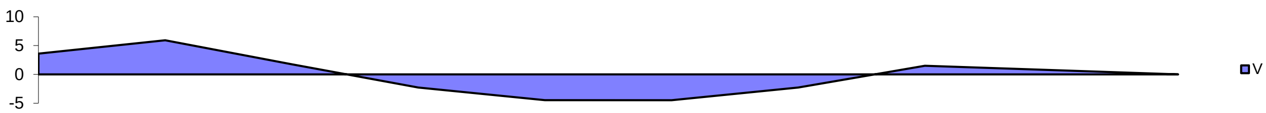
$$q_{u,MAX} = \begin{cases} \frac{P_u\left(1+\frac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \frac{L}{6} \\ \frac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \frac{L}{6} \end{cases}$$

1.77 ksf

BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	0.95	1.90	2.85	3.80	4.75	5.70	6.65	7.60	8.55	9.50
$P_{u,w}$ (klf)	0.0	0.0	0.0	5.0	2.7	0.4	-1.9	-4.3	0.0	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	0	-2	-9	-18	-28	-36	-40	-42	-44
$V_{u,w}$ (kips)	0	0	0	-5	-9	-10	-10	-7	-2	-2	-2
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	0	-1	-3	-6	-9	-13	-17	-23	-29	-35
$V_{u,f}$ (kips)	0	-1	-1	-2	-3	-4	-4	-5	-6	-7	-7
q_u (ksf)	-1.8	-1.3	-0.8	-0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	2	8	16	25	34	43	52	61	70	79
$V_{u,q}$ (kips)	0	4	7	9	10	10	10	10	10	10	10
ΣM_u (ft-k)	**0**	**2**	**6**	**10**	**10**	**7**	**2**	**-1**	**-1**	**0**	**0**
ΣV_u (kips)	**0**	**4**	**6**	**2**	**-2**	**-4**	**-4**	**-2**	**1**	**1**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f'_c)^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0000	0.0025	6	kips	42	kips
Bottom Longitudinal	10	ft-k	14.31	0.0003	0.0025	6	kips	42	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \dfrac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	169	kips	$(8\, b\, d\, f_c'^{0.5})$
ϕV_s =	5.9	kips	$(A_v\, f_{yt}\, d\, /\, s)$
ϕV_n =	42	kips	$(\phi V_s + \phi V_c)$
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

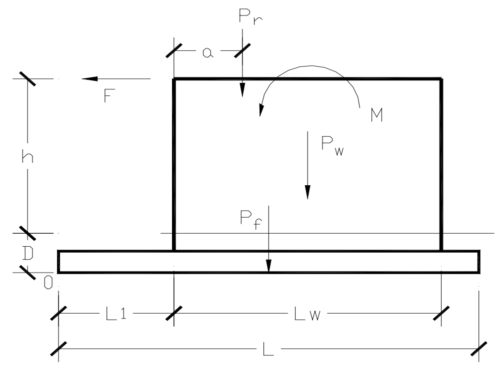
ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: 4

WALL LENGTH	L_w =	5.5	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	9.5	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	3	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	2.09	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	2.75	ft
WALL SELF WEIGHT	P_w =	1.1	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	1.86	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi



Demands Summary

Stability	0.63	Bearing	0.18
Flexure-Top Steel	0.02	Flexure-Bot Steel	0.14
Shear	0.12	T&S Steel	0.95

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL		3	#	5				
BOTTOM BARS, LONGITUDINAL		3	#	5				
SHEAR REINFORCEMENT	#	3	@	24	in o.c.	2	legs	← Not Required

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 2.46 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 6.20 kips (footing self weight)

$Mo = 0.75\,[F\,(h + D) + M] =$ 18 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 45 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 9.39 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 45 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 1.93 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\quad e > \dfrac{L}{6} \end{cases}$$

= 0.74 ksf **<** q_b **[Satisfactory]**

Where e = 1.93 ft, > (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 54 ft-kips

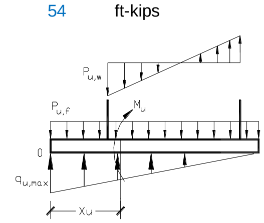
$M_{u,o} =$ $1.4\,[F(h + D) + M] =$ 34 ft-kips

$P_u =$ $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 11 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 3.00 ft

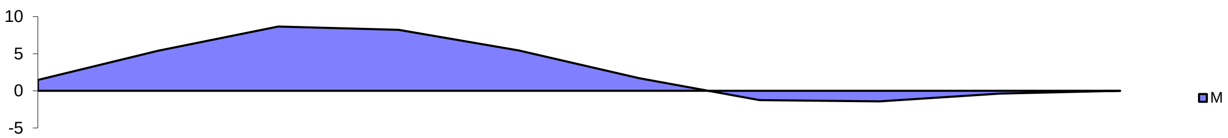
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\quad e_u > \dfrac{L}{6} \end{cases}$$

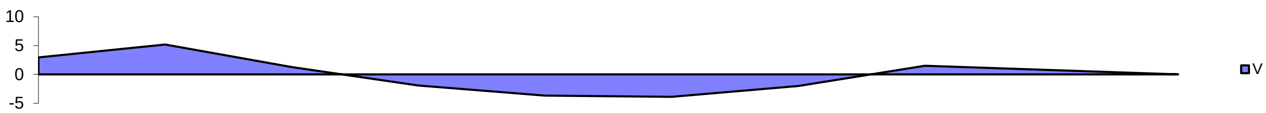
1.43 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	0.95	1.90	2.85	3.80	4.75	5.70	6.65	7.60	8.55	9.50
$P_{u,w}$ (klf)	0.0	0.0	0.0	5.3	3.0	0.7	-1.6	-3.9	0.0	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	0	-2	-10	-20	-30	-39	-45	-48	-52
$V_{u,w}$ (kips)	0	0	0	-5	-9	-11	-11	-8	-4	-4	-4
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	0	-1	-3	-6	-9	-13	-17	-23	-29	-35
$V_{u,f}$ (kips)	0	-1	-1	-2	-3	-4	-4	-5	-6	-7	-7
q_u (ksf)	-1.4	-1.2	-0.9	-0.7	-0.4	-0.1	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	2	7	14	24	34	44	55	66	77	87
$V_{u,q}$ (kips)	0	4	7	9	10	11	11	11	11	11	11
ΣM_u (ft-k)	**0**	**1**	**5**	**9**	**8**	**5**	**2**	**-1**	**-1**	**0**	**0**
ΣV_u (kips)	**0**	**3**	**5**	**1**	**-2**	**-4**	**-4**	**-2**	**1**	**1**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0000	0.0019	5	kips	42	kips
Bottom Longitudinal	9	ft-k	14.31	0.0003	0.0019	5	kips	42	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where $$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	169	kips	($8\, b\, d\, f_c'^{0.5}$)
ϕV_s =	5.9	kips	($A_v\, f_{yt}\, d$ / s)
ϕV_n =	42	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0019		

ϕ = **0.75** ACI 318-14 21.2.1

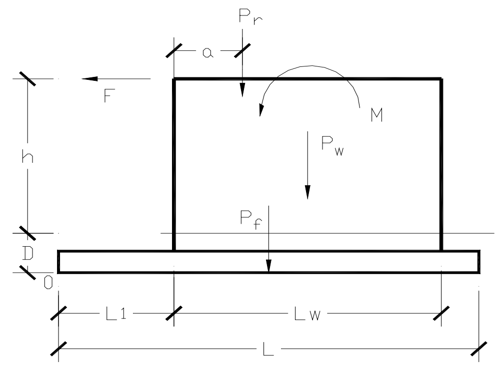
$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: 6

WALL LENGTH	L_w =	12.5	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	16.5	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	2	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	3.5625	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	6.25	ft
WALL SELF WEIGHT	P_w =	2.69	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	9.15	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi
TOP BARS, LONGITUDINAL		3	# 5
BOTTOM BARS, LONGITUDINAL		3	# 5
SHEAR REINFORCEMENT	#	3	@ 8 in o.c. 2 legs

s needs to be less tl



Demands Summary

Stability	0.89	Bearing	0.45
Flexure-Top Steel	0.02	Flexure-Bot Steel	0.91
Shear	0.63	T&S Steel	0.64

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 1.75 > 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 12.62 kips (footing self weight)

$M_o = 0.75\,[F\,(h + D) + M] =$ 89 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 156 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 18.87 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 156 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 4.73 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\ \ e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\ \ e > \dfrac{L}{6} \end{cases}$$

= 1.79 ksf < q_b **[Satisfactory]**

Highlighted region indicates addition of slab weight to increase stability of the footing

Where e = 4.73 ft, > (L / 6)

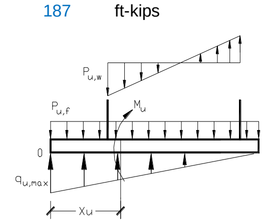
CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 187 ft-kips

$M_{u,o} =$ $1.4\,[F(h + D) + M] =$ 166 ft-kips

$P_u =$ $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 23 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 7.35 ft

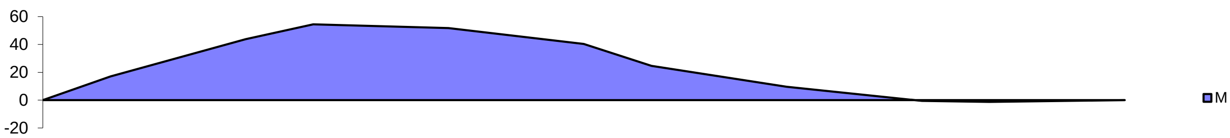


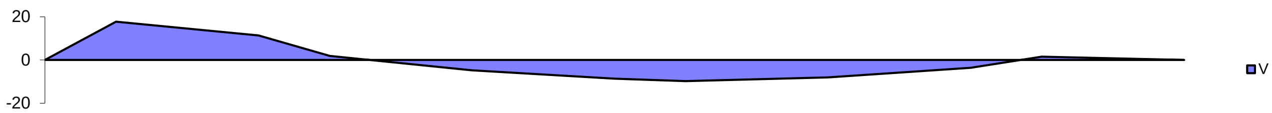
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\ \ e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\ \ e_u > \dfrac{L}{6} \end{cases}$$

8.42 ksf

BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	1.65	3.30	4.95	6.60	8.25	9.90	11.55	13.20	14.85	16.50
$P_{u,w}$ (klf)	0.0	0.0	5.7	4.0	2.3	0.6	-1.1	-2.8	-4.5	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	-6	-26	-57	-95	-134	-170	-199	-216	-228
$V_{u,w}$ (kips)	0	0	-8	-16	-21	-24	-23	-20	-14	-8	-8
$P_{u,f}$ (ksf)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
$M_{u,f}$ (ft-k)	0	-1	-5	-11	-20	-31	-45	-61	-80	-101	-125
$V_{u,f}$ (kips)	0	-2	-3	-5	-6	-8	-9	-11	-12	-14	-15
q_u (ksf)	-8.4	-3.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	18	54	92	129	166	204	241	279	316	353
$V_{u,q}$ (kips)	0	19	23	23	23	23	23	23	23	23	23
ΣM_u (ft-k)	**0**	**17**	**44**	**54**	**52**	**40**	**25**	**10**	**0**	**-1**	**0**
ΣV_u (kips)	**0**	**18**	**11**	**2**	**-5**	**-9**	**-10**	**-8**	**-4**	**2**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0001	0.0028	18	kips	28	kips
Bottom Longitudinal	54	ft-k	14.31	0.0025	0.0028	18	kips	28	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

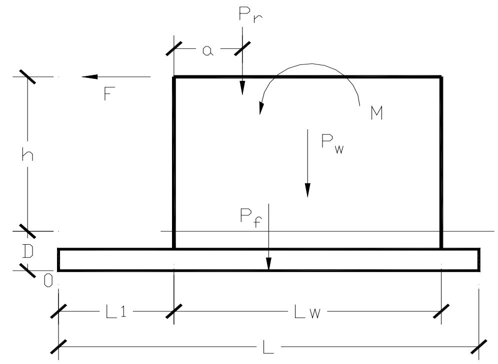
$\phi V_{s\,max}$ =	113	kips	($8\, b\, d\, f_c'^{0.5}$)
ϕV_s =	17.7	kips	($A_v\, f_{yt}\, d / s$)
ϕV_n =	28	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0017	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0275	in^2/in	**[Satisfactory]**
Min shear tie spacing =	7.4	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0028		

ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: 7



WALL LENGTH	L_w =	11	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	5.5	in
FOOTING LENGTH	L =	13	ft
	L_1 =	1	ft
FOOTING WIDTH	B =	2	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	2.75	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.418	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	5.5	ft
WALL SELF WEIGHT	P_w =	2.37	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	5.54	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi

Demands Summary

Stability	0.96	Bearing	0.44
Flexure-Top Steel	0.01	Flexure-Bot Steel	0.53
Shear	0.46	T&S Steel	0.64

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL	3	#	5			
BOTTOM BARS, LONGITUDINAL	3	#	5			
SHEAR REINFORCEMENT #	3	@	8	in o.c.	2 legs	← **Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O$ = 1.62 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 10.44 kips (footing self weight)

$M_o = 0.75\,[F\,(h + D) + M]$ = 53 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)$ = 86 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f$ = 13.22 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)$ = 86 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P$ = 4.01 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$ = 1.77 ksf < q_b **[Satisfactory]**

Highlighted region indicates addition of slab weight to increase stability of the footing

Where e = 4.01 ft, > (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

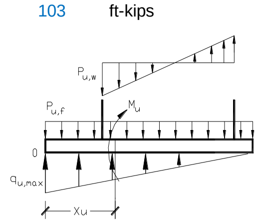
$M_{u,R}$ = $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a)$ = 103 ft-kips

$M_{u,o}$ = $1.4\,[F(h + D) + M]$ = 99 ft-kips

P_u = $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL}$ = 16 kips

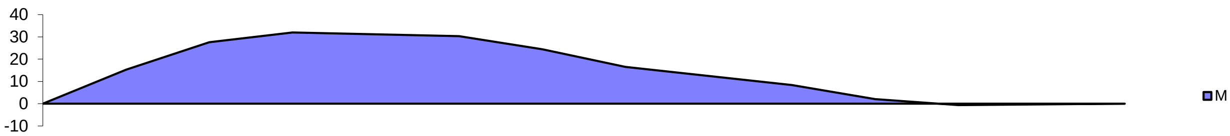
$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u$ = 6.23 ft

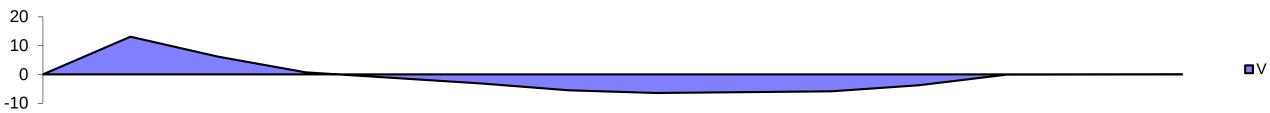
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$ = 19.59 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	1.30	2.60	3.90	5.20	6.50	7.80	9.10	10.40	11.70	13.00
$P_{u,w}$ (klf)	0.0	4.9	3.8	2.6	1.5	0.3	-0.9	-2.0	-3.2	-4.3	0.0
$M_{u,w}$ (ft-k)	0	0	-6	-18	-35	-54	-74	-92	-107	-116	-121
$V_{u,w}$ (kips)	0	-2	-7	-11	-14	-15	-15	-13	-10	-5	-3
$P_{u,f}$ (ksf)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
$M_{u,f}$ (ft-k)	0	-1	-3	-7	-13	-20	-29	-40	-52	-66	-81
$V_{u,f}$ (kips)	0	-1	-3	-4	-5	-6	-8	-9	-10	-11	-13
q_u (ksf)	-19.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	16	37	58	78	99	119	140	161	181	202
$V_{u,q}$ (kips)	0	16	16	16	16	16	16	16	16	16	16
ΣM_u (ft-k)	**0**	**15**	**28**	**32**	**30**	**24**	**17**	**8**	**2**	**-1**	**0**
ΣV_u (kips)	**0**	**13**	**6**	**1**	**-3**	**-6**	**-6**	**-6**	**-4**	**0**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0000	0.0028	13	kips	28	kips
Bottom Longitudinal	32	ft-k	14.31	0.0015	0.0028	13	kips	28	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f_c' \left(1 - \sqrt{1 - \dfrac{M_u}{0.383 b d^2 f_c'}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	113	kips	$(8\, b\, d\, f_c'^{0.5})$
ϕV_s =	17.7	kips	$(A_v\, f_{yt}\, d\, /\, s)$
ϕV_n =	28	kips	$(\phi V_s + \phi V_c)$
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0275	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0028		

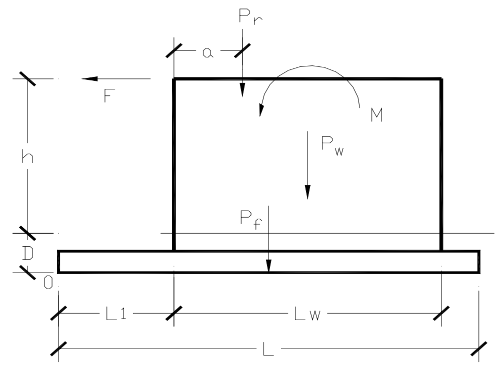
ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85\beta_1 f_c'}{f_y}\frac{\varepsilon u}{\varepsilon u + \varepsilon t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: 8&9

WALL LENGTH	L_w =	14	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	18	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	3.99	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	7	ft
WALL SELF WEIGHT	P_w =	3.01	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	1.58	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi



Demands Summary

Stability	0.21	Bearing	0.17
Flexure-Top Steel	0.00	Flexure-Bot Steel	0.13
Shear	0.08	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL 2 # 5
BOTTOM BARS, LONGITUDINAL 2 # 5
SHEAR REINFORCEMENT # 3 @ 24 in o.c. 2 legs ← **Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 7.51 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 5.87 kips (footing self weight)

$Mo = 0.75 [F (h + D) + M] =$ 15 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 116 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 12.87 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 116 ft-kips (resisting moment with live load)

$e = 0.5 L - (M_R - M_O) / P =$ 1.20 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

= 0.67 ksf < q_b
[Satisfactory]

Where e = 1.20 ft, < (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} = 1.2 [P_{r,DL} (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)] + 0.5 P_{r, LL}(L_1 + a) =$ 139 ft-kips

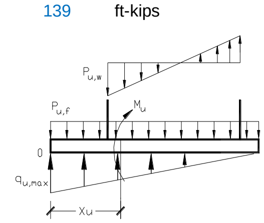
$M_{u,o} = 1.4 [F(h + D) + M] =$ 29 ft-kips

$P_u = 1.2 (P_{r,DL} + P_f + P_w) + 0.5 P_{r, LL} =$ 15 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 1.86 ft

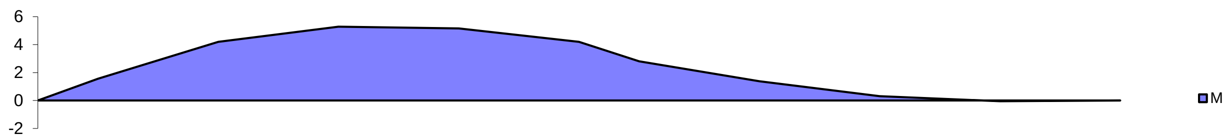
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

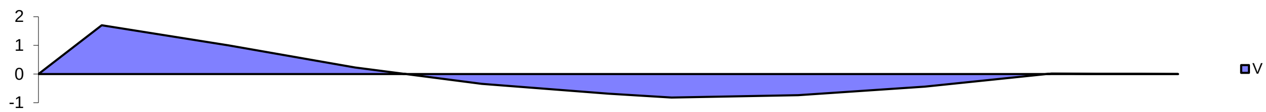
0.93 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	1.80	3.60	5.40	7.20	9.00	10.80	12.60	14.40	16.20	18.00
$P_{u,w}$ (klf)	0.0	0.0	1.3	1.1	0.8	0.6	0.4	0.1	-0.1	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	-2	-8	-17	-29	-43	-58	-74	-89	-104
$V_{u,w}$ (kips)	0	0	-2	-4	-6	-7	-8	-9	-9	-8	-8
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-1	-3	-6	-10	-16	-23	-31	-41	-51	-63
$V_{u,f}$ (kips)	0	-1	-1	-2	-3	-4	-4	-5	-6	-6	-7
q_u (ksf)	-0.9	-0.9	-0.8	-0.7	-0.6	-0.6	-0.5	-0.4	-0.4	-0.3	-0.2
$M_{u,q}$ (ft-k)	0	2	9	19	32	49	69	91	115	141	168
$V_{u,q}$ (kips)	0	2	5	7	8	10	12	13	14	15	15
ΣM_u (ft-k)	**0**	**2**	**4**	**5**	**5**	**4**	**3**	**1**	**0**	**0**	**0**
ΣV_u (kips)	**0**	**2**	**1**	**0**	**0**	**-1**	**-1**	**-1**	**0**	**0**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	14.31	0.0000	0.0025	2	kips	21	kips
Bottom Longitudinal	5	ft-k	14.31	0.0003	0.0025	2	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \dfrac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	85	kips	($8\, b\, d\, f_c'^{0.5}$)
ϕV_s =	5.9	kips	($A_v f_{yt} d / s$)
ϕV_n =	21	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

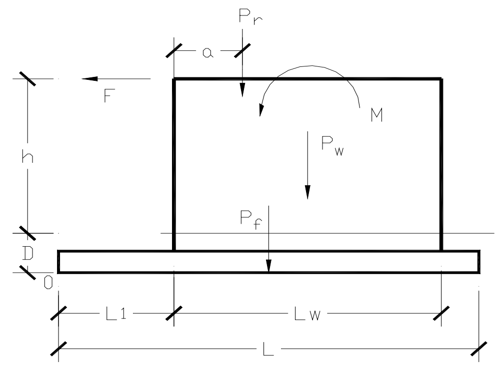
ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: 10

WALL LENGTH	L_w =	12	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	16	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.456	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	6	ft
WALL SELF WEIGHT	P_w =	2.581	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	0.536	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi



Demands Summary

Stability	0.12	Bearing	0.11
Flexure-Top Steel	0.00	Flexure-Bot Steel	0.05
Shear	0.03	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL		2	#	5				← **Not Required**
BOTTOM BARS, LONGITUDINAL		2	#	5				
SHEAR REINFORCEMENT	#	3	@	24	in o.c.	2	legs	← **Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 12.65 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 5.22 kips (footing self weight)

$M_o = 0.75\,[F\,(h + D) + M] =$ 5 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 66 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 8.26 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 66 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 0.63 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\ \ e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\ \ e > \dfrac{L}{6} \end{cases}$$

= 0.43 ksf < q_b

[Satisfactory]

Where e = 0.63 ft, < (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ 1.2 $[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 79 ft-kips

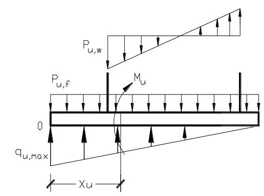
$M_{u,o} =$ 1.4 $[F(h + D) + M] =$ 10 ft-kips

$P_u =$ 1.2 $(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 10 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 0.98 ft

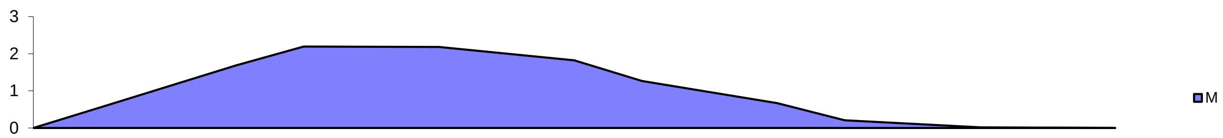
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\ \ e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\ \ e_u > \dfrac{L}{6} \end{cases}$$

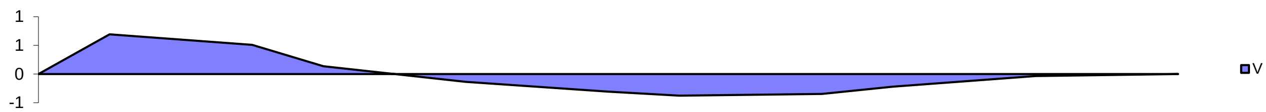
0.57 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	1.60	3.20	4.80	6.40	8.00	9.60	11.20	12.80	14.40	16.00
$P_{u,w}$ (klf)	0.0	0.0	0.6	0.5	0.4	0.3	0.2	0.1	0.0	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	0	-3	-6	-10	-16	-21	-27	-33	-39
$V_{u,w}$ (kips)	0	0	-1	-2	-2	-3	-3	-4	-4	-4	-4
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-1	-2	-5	-8	-13	-18	-25	-32	-41	-50
$V_{u,f}$ (kips)	0	-1	-1	-2	-3	-3	-4	-4	-5	-6	-6
q_u (ksf)	-0.6	-0.5	-0.5	-0.5	-0.4	-0.4	-0.4	-0.4	-0.3	-0.3	-0.3
$M_{u,q}$ (ft-k)	0	1	4	9	16	25	35	46	59	74	89
$V_{u,q}$ (kips)	0	1	3	4	5	6	7	8	9	9	10
ΣM_u (ft-k)	**0**	**1**	**2**	**2**	**2**	**2**	**1**	**1**	**0**	**0**	**0**
ΣV_u (kips)	**0**	**1**	**1**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	14.31	0.0000	0.0025	1	kips	21	kips
Bottom Longitudinal	2	ft-k	14.31	0.0001	0.0025	1	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

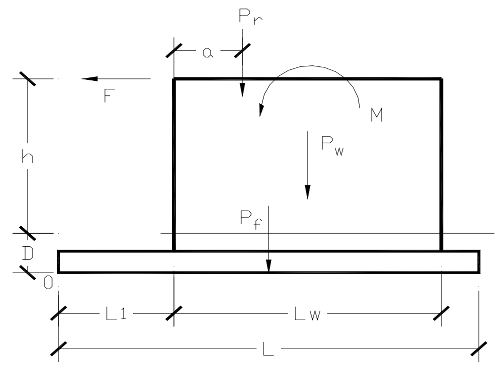
$\phi V_{s\,max}$ =	85	kips	($8\, b\, d\, f_c'^{0.5}$)
ϕV_s =	5.9	kips	($A_v f_{yt} d / s$)
ϕV_n =	21	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: A



WALL LENGTH	L_w =	3	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	5.5	in
FOOTING LENGTH	L =	7	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.57	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	1.5	ft
WALL SELF WEIGHT	P_w =	0.65	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	0.67	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi
TOP BARS, LONGITUDINAL		2	# 5
BOTTOM BARS, LONGITUDINAL		2	# 5
SHEAR REINFORCEMENT	#	3	@ 24 in o.c. 2 legs ← **Not Required**

Demands Summary

Stability	0.82	Bearing	0.24
Flexure-Top Steel	0.02	Flexure-Bot Steel	0.13
Shear	0.19	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 1.89 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 2.28 kips (footing self weight)

$M_o = 0.75\,[F\,(h + D) + M] =$ 6 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 12 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 3.50 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 12 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 1.85 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\quad e > \dfrac{L}{6} \end{cases}$$

= 0.94 ksf **<** q_b **[Satisfactory]**

Where e = 1.85 ft, > (L / 6)

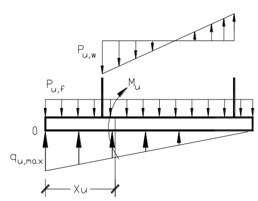
CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 15 ft-kips

$M_{u,o} =$ $1.4\,[F(h + D) + M] =$ 12 ft-kips

$P_u =$ $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 4 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 2.88 ft

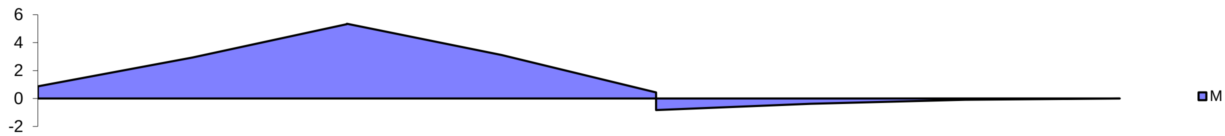


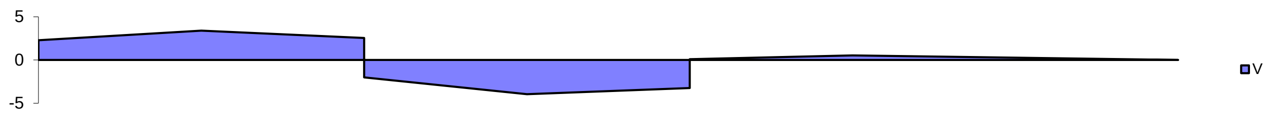
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\quad e_u > \dfrac{L}{6} \end{cases}$$

3.03 ksf

BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	0.70	1.40	2.10	2.80	3.50	4.20	4.90	5.60	6.30	7.00
$P_{u,w}$ (klf)	0.0	0.0	0.0	8.0	4.3	0.5	-3.3	-7.0	0.0	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	0	0	-2	-7	-11	-14	-15	-16	-17
$V_{u,w}$ (kips)	0	0	0	-1	-5	-7	-6	-2	-1	-1	-1
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	0	0	-1	-2	-2	-3	-5	-6	-8	-10
$V_{u,f}$ (kips)	0	0	-1	-1	-1	-1	-2	-2	-2	-2	-3
q_u (ksf)	-3.0	-1.9	-0.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	1	3	6	9	12	15	18	21	24	27
$V_{u,q}$ (kips)	0	3	4	4	4	4	4	4	4	4	4
ΣM_u (ft-k)	**0**	**1**	**3**	**5**	**5**	**3**	**0**	**-1**	**0**	**0**	**0**
ΣV_u (kips)	**0**	**2**	**3**	**3**	**-2**	**-4**	**-3**	**0**	**1**	**0**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0000	0.0025	4	kips	21	kips
Bottom Longitudinal	5	ft-k	14.31	0.0003	0.0025	4	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	85	kips	$(8\, b\, d\, f_c'^{0.5})$
ϕV_s =	5.9	kips	$(A_v\, f_{yt}\, d / s)$
ϕV_n =	21	kips	$(\phi V_s + \phi V_c)$
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

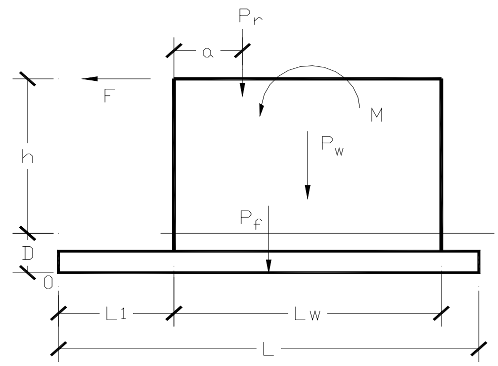
ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: **B.5**

WALL LENGTH	L_W =	20.5	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	24.5	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	8.62	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	10.25	ft
WALL SELF WEIGHT	P_W =	4.1	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	7.95	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi



Demands Summary

Stability	0.48	Bearing	0.27
Flexure-Top Steel	0.03	Flexure-Bot Steel	0.37
Shear	0.18	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL		2	#	5				
BOTTOM BARS, LONGITUDINAL		2	#	5				
SHEAR REINFORCEMENT	#	3	@	24	in o.c.	2	legs	← **Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 3.27 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 7.99 kips (footing self weight)

$M_O = 0.75\,[F\,(h + D) + M] =$ 78 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_W\,(L_1 + 0.5L_W) =$ 254 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_W + P_f =$ 20.72 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_W\,(L_1 + 0.5L_W) =$ 254 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 3.74 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

= 1.08 ksf **<** q_b

[Satisfactory]

Where e = 3.74 ft, < (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_W\,(L_1 + 0.5L_W)] + 0.5\,P_{r,LL}(L_1 + a) =$ 305 ft-kips

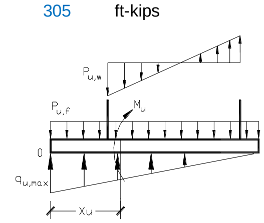
$M_{u,O} =$ $1.4\,[F(h + D) + M] =$ 145 ft-kips

$P_u =$ $1.2\,(P_{r,DL} + P_f + P_W) + 0.5\,P_{r,LL} =$ 25 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 5.82 ft

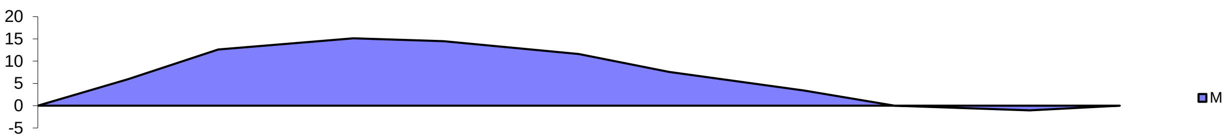
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

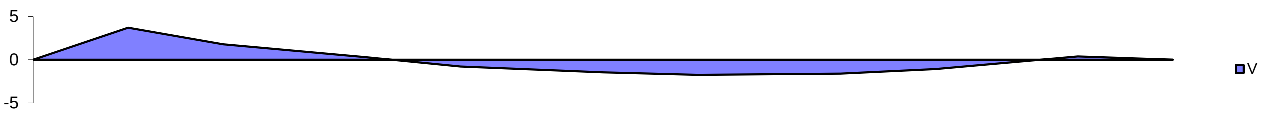
1.72 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	2.45	4.90	7.35	9.80	12.25	14.70	17.15	19.60	22.05	24.50
$P_{u,w}$ (klf)	0.0	2.7	2.2	1.7	1.2	0.7	0.3	-0.2	-0.7	-1.2	0.0
$M_{u,w}$ (ft-k)	0	0	-11	-35	-70	-112	-158	-206	-252	-294	-332
$V_{u,w}$ (kips)	0	-1	-7	-12	-16	-18	-19	-19	-18	-16	-15
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-1	-5	-11	-19	-29	-42	-58	-75	-95	-117
$V_{u,f}$ (kips)	0	-1	-2	-3	-4	-5	-6	-7	-8	-9	-10
q_u (ksf)	-1.7	-1.5	-1.3	-1.1	-0.8	-0.6	-0.4	-0.2	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	7	28	61	103	153	208	267	327	388	449
$V_{u,q}$ (kips)	0	6	11	15	19	22	23	25	25	25	25
ΣM_u (ft-k)	**0**	**6**	**13**	**15**	**14**	**12**	**8**	**3**	**0**	**-1**	**0**
ΣV_u (kips)	**0**	**4**	**2**	**0**	**-1**	**-1**	**-2**	**-2**	**-1**	**0**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\phi b d (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0001	0.0025	4	kips	21	kips
Bottom Longitudinal	15	ft-k	14.31	0.0009	0.0025	4	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

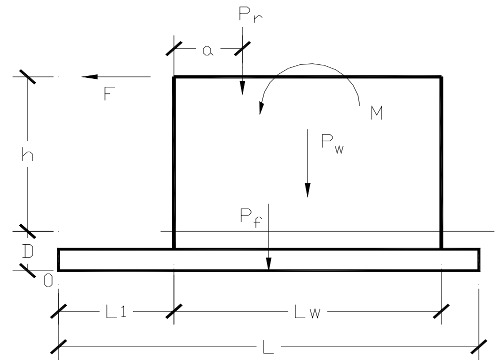
$\phi V_{s\,max}$ =	85	kips	$(8\, b\, d\, f_c'^{0.5})$
ϕV_s =	5.9	kips	$(A_v f_{yt} d / s)$
ϕV_n =	21	kips	$(\phi V_s + \phi V_c)$
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: E



WALL LENGTH	L_w =	17	ft					
WALL HEIGHT	h =	10	ft					
WALL THICKNESS	t =	8	in					
FOOTING LENGTH	L =	21	ft					
	L_1 =	2	ft					
FOOTING WIDTH	B =	1.5	ft					
FOOTING THICKNESS	T =	1.5	ft					
FOOTING EMBEDMENT DEPTH	D =	3	ft					
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf					
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf					
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	3.23	kips					
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips					
TOP LOAD LOCATION	a =	8.5	ft	**Demands Summary**				
WALL SELF WEIGHT	P_w =	2.55	kips	Stability	0.36	Bearing	0.17	
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic	Flexure-Top Steel	0.03	Flexure-Bot Steel	0.15	
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	3.17	kips	Shear	0.10	T&S Steel	0.72	
	M =	0	ft-kips					
CONCRETE STRENGTH	f_c' =	3000	psi		THE FOOTING DESIGN IS ADEQUATE.			
REBAR YIELD STRESS	f_y =	60	ksi					
TOP BARS, LONGITUDINAL		2	#	5				
BOTTOM BARS, LONGITUDINAL		2	#	5				
SHEAR REINFORCEMENT	#	3	@	24	in o.c.	2	legs	← **Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 4.30 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 6.85 kips (footing self weight)

$M_o = 0.75\,[F\,(h + D) + M] =$ 31 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 133 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 12.63 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 133 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 2.44 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\ \ e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\ \ e > \dfrac{L}{6} \end{cases}$$

= 0.68 ksf < q_b

[Satisfactory]

Where e = 2.44 ft, < (L / 6)

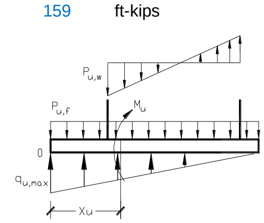
CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} = 1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 159 ft-kips

$M_{u,o} = 1.4\,[F(h + D) + M] =$ 58 ft-kips

$P_u = 1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 15 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 3.80 ft

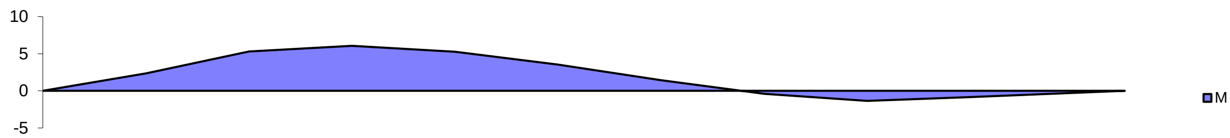


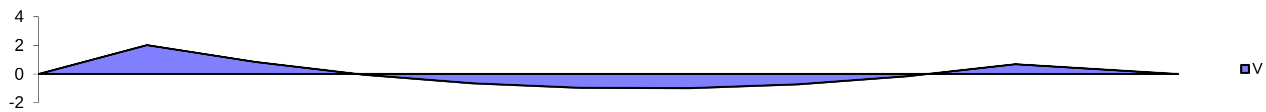
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\ \ e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\ \ e_u > \dfrac{L}{6} \end{cases}$$

1.01 ksf

BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	2.10	4.20	6.30	8.40	10.50	12.60	14.70	16.80	18.90	21.00
$P_{u,w}$ (klf)	0.0	1.6	1.3	1.0	0.7	0.4	0.1	-0.2	-0.5	-0.8	0.0
$M_{u,w}$ (ft-k)	0	0	-4	-13	-27	-44	-62	-81	-100	-116	-130
$V_{u,w}$ (kips)	0	0	-3	-6	-7	-9	-9	-9	-8	-7	-7
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-1	-3	-8	-14	-22	-31	-42	-55	-70	-86
$V_{u,f}$ (kips)	0	-1	-2	-2	-3	-4	-5	-6	-7	-7	-8
q_u (ksf)	-1.0	-0.9	-0.8	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
$M_{u,q}$ (ft-k)	0	3	12	27	46	69	95	123	154	185	217
$V_{u,q}$ (kips)	0	3	6	8	10	12	13	14	15	15	15
ΣM_u (ft-k)	**0**	**2**	**5**	**6**	**5**	**4**	**1**	**0**	**-1**	**-1**	**0**
ΣV_u (kips)	**0**	**2**	**1**	**0**	**-1**	**-1**	**-1**	**-1**	**0**	**1**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0001	0.0025	2	kips	21	kips
Bottom Longitudinal	6	ft-k	14.31	0.0004	0.0025	2	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \dfrac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

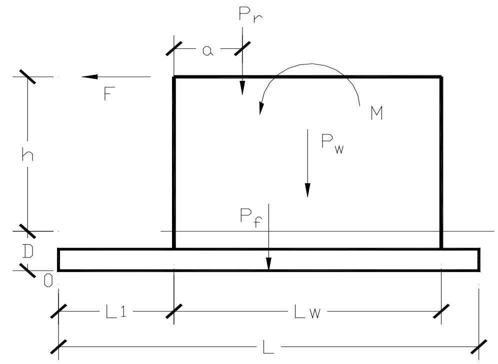
$\phi V_{s\,max}$ =	85	kips	$(8\, b\, d\, f_c'^{0.5})$
ϕV_s =	5.9	kips	$(A_v\, f_{yt}\, d\, /\, s)$
ϕV_n =	21	kips	$(\phi V_s + \phi V_c)$
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$

0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14



INPUT DATA: F

WALL LENGTH	L_w =	6	ft
WALL HEIGHT	h =	14	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	9	ft
	L_1 =	1.5	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.228	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	3	ft
WALL SELF WEIGHT	P_w =	1.81	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	1.60	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi

Demands Summary

Stability	0.95	Bearing	0.47
Flexure-Top Steel	0.01	Flexure-Bot Steel	0.25
Shear	0.39	T&S Steel	0.49

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL		3	#	5				
BOTTOM BARS, LONGITUDINAL		3	#	5				
SHEAR REINFORCEMENT	#	3	@	24	in o.c.	2	legs	**← Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 1.64 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 5.38 kips (footing self weight)

$M_o = 0.75\,[F\,(h + D) + M] =$ 20 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 33 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 7.42 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 33 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 2.74 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\ \ e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\ \ e > \dfrac{L}{6} \end{cases}$$

= 1.88 ksf < q_b **[Satisfactory]**

Highlighted region indicates addition of slab weight to increase stability of the footing

Where e = 2.74 ft, > (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} = 1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,\,LL}(L_1 + a) =$ 40 ft-kips

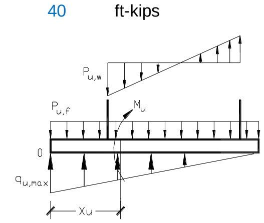
$M_{u,o} = 1.4\,[F(h + D) + M] =$ 38 ft-kips

$P_u = 1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,\,LL} =$ 9 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 4.26 ft

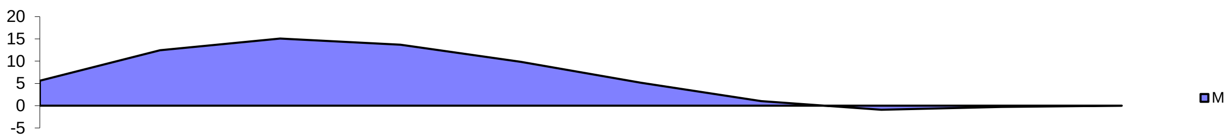
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\ \ e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\ \ e_u > \dfrac{L}{6} \end{cases}$$

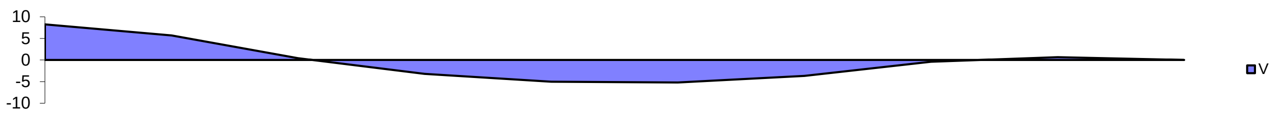
16.83 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	0.90	1.80	2.70	3.60	4.50	5.40	6.30	7.20	8.10	9.00
$P_{u,w}$ (klf)	0.0	0.0	6.1	4.2	2.3	0.4	-1.5	-3.4	-5.3	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	0	-4	-12	-21	-30	-39	-44	-47	-49
$V_{u,w}$ (kips)	0	0	-2	-7	-9	-11	-10	-8	-4	-2	-2
$P_{u,f}$ (ksf)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
$M_{u,f}$ (ft-k)	0	0	-1	-3	-5	-7	-10	-14	-19	-24	-29
$V_{u,f}$ (kips)	0	-1	-1	-2	-3	-3	-4	-5	-5	-6	-6
q_u (ksf)	-16.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	6	14	22	30	38	46	54	62	70	78
$V_{u,q}$ (kips)	0	9	9	9	9	9	9	9	9	9	9
ΣM_u (ft-k)	**0**	**6**	**12**	**15**	**14**	**10**	**5**	**1**	**-1**	**0**	**0**
ΣV_u (kips)	**0**	**8**	**6**	**0**	**-3**	**-5**	**-5**	**-4**	**0**	**1**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0001	0.0037	8	kips	21	kips
Bottom Longitudinal	15	ft-k	14.31	0.0009	0.0037	8	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \dfrac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	85	kips	($8\, b\, d\, f_c'^{0.5}$)
ϕV_s =	5.9	kips	($A_v\, f_{yt}\, d$ / s)
ϕV_n =	21	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0037		

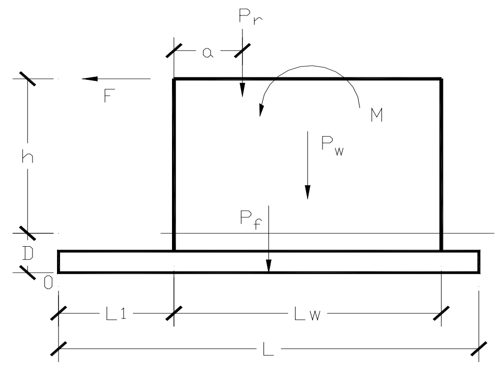
ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\,\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: G

WALL LENGTH	L_w =	24	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	28	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.91	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	12	ft
WALL SELF WEIGHT	P_w =	5.16	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	2.47	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi



Demands Summary

Stability	0.18	Bearing	0.12
Flexure-Top Steel	0.00	Flexure-Bot Steel	0.12
Shear	0.05	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL	2	#	5				
BOTTOM BARS, LONGITUDINAL	2	#	5				
SHEAR REINFORCEMENT #	3	@	24	in o.c.	2	legs	← **Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 8.85 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 9.14 kips (footing self weight)

$Mo = 0.75\,[F\,(h + D) + M] =$ 24 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 213 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 15.21 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 213 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 1.58 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\quad e > \dfrac{L}{6} \end{cases}$$

= 0.48 ksf < q_b

[Satisfactory]

Where e = 1.58 ft, < (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 255 ft-kips

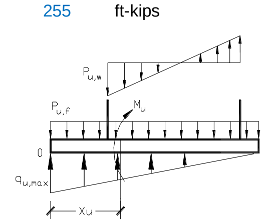
$M_{u,o} =$ $1.4\,[F(h + D) + M] =$ 45 ft-kips

$P_u =$ $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 18 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 2.46 ft

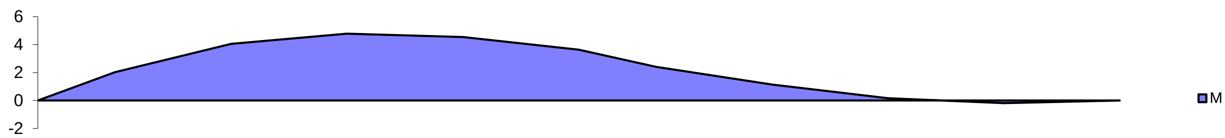
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\quad e_u > \dfrac{L}{6} \end{cases}$$

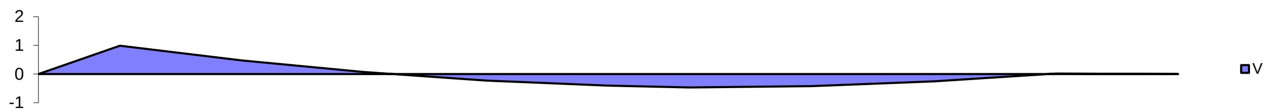
0.66 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	2.80	5.60	8.40	11.20	14.00	16.80	19.60	22.40	25.20	28.00
$P_{u,w}$ (klf)	0.0	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0	-0.1	0.0
$M_{u,w}$ (ft-k)	0	0	-5	-14	-28	-44	-63	-84	-105	-126	-147
$V_{u,w}$ (kips)	0	-1	-3	-4	-5	-6	-7	-8	-8	-7	-7
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-2	-6	-14	-25	-38	-55	-75	-98	-124	-153
$V_{u,f}$ (kips)	0	-1	-2	-3	-4	-5	-7	-8	-9	-10	-11
q_u (ksf)	-0.7	-0.6	-0.6	-0.5	-0.5	-0.4	-0.4	-0.3	-0.3	-0.3	-0.2
$M_{u,q}$ (ft-k)	0	4	15	33	57	86	121	160	204	251	300
$V_{u,q}$ (kips)	0	3	5	7	10	12	13	15	16	17	18
ΣM_u (ft-k)	**0**	**2**	**4**	**5**	**5**	**4**	**2**	**1**	**0**	**0**	**0**
ΣV_u (kips)	**0**	**1**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	14.31	0.0000	0.0025	1	kips	21	kips
Bottom Longitudinal	5	ft-k	14.31	0.0003	0.0025	1	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \dfrac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

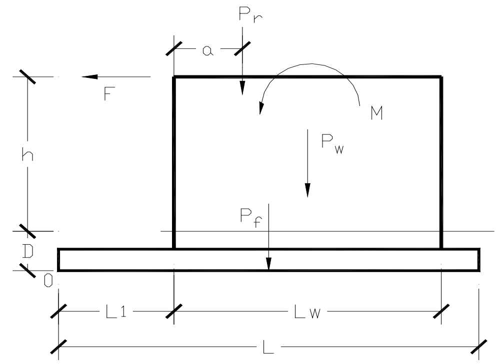
$\phi V_{s\,max}$ =	85	kips	($8\,b\,d\,f_c'^{0.5}$)
ϕV_s =	5.9	kips	($A_v f_{yt}\, d$ / s)
ϕV_n =	21	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: APP A.5



WALL LENGTH	L_w =	4	ft			
WALL HEIGHT	h =	10	ft			
WALL THICKNESS	t =	8	in			
FOOTING LENGTH	L =	8	ft			
	L_1 =	2	ft			
FOOTING WIDTH	B =	3	ft			
FOOTING THICKNESS	T =	1.5	ft			
FOOTING EMBEDMENT DEPTH	D =	3	ft			
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf			
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf			
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.34	kips			
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips			
TOP LOAD LOCATION	a =	2	ft			
WALL SELF WEIGHT	P_w =	0.8	kips			
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic			
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	1.92	kips			
	M =	0	ft-kips			
CONCRETE STRENGTH	f_c' =	3000	psi			
REBAR YIELD STRESS	f_y =	60	ksi			
TOP BARS, LONGITUDINAL		4	#	5		
BOTTOM BARS, LONGITUDINAL		4	#	5		
SHEAR REINFORCEMENT	#	3	@	24 in o.c.	2 legs	← Not Required

Demands Summary

Stability	0.90	Bearing	0.27
Flexure-Top Steel	0.02	Flexure-Bot Steel	0.19
Shear	0.20	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 1.73 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 6.96 kips (footing self weight)

$M_O = 0.75\,[F\,(h + D) + M] =$ 19 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 32 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 8.10 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 32 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 2.32 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\ \ e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\ \ e > \dfrac{L}{6} \end{cases}$$

= 1.07 ksf < q_b **[Satisfactory]**

Highlighted region indicates addition of slab weight to increase stability of the footing

Where e = 2.32 ft, > (L / 6)

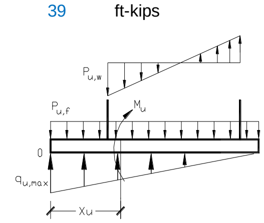
CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} = 1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 39 ft-kips

$M_{u,O} = 1.4\,[F(h + D) + M] =$ 35 ft-kips

$P_u = 1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 10 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 3.60 ft

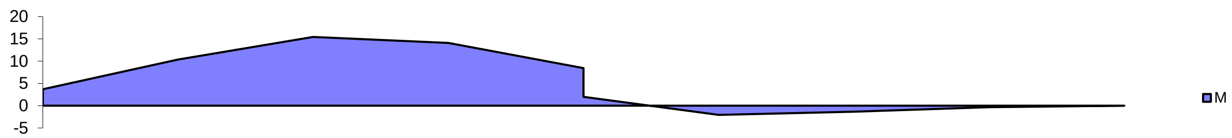


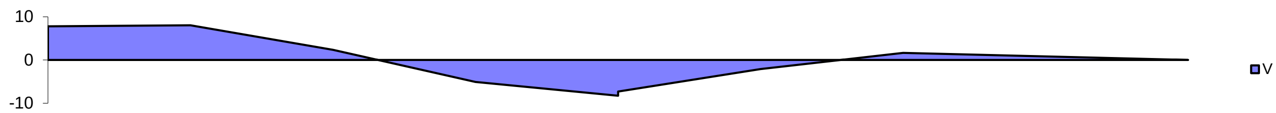
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\ \ e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\ \ e_u > \dfrac{L}{6} \end{cases}$$

5.43 ksf

BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	0.80	1.60	2.40	3.20	4.00	4.80	5.60	6.40	7.20	8.00
$P_{u,w}$ (klf)	0.0	0.0	0.0	10.8	5.6	0.3	-4.9	-10.2	0.0	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	0	-1	-8	-18	-29	-36	-38	-39	-40
$V_{u,w}$ (kips)	0	0	0	-5	-11	-14	-12	-6	-1	-1	-1
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	0	-1	-3	-5	-8	-12	-16	-21	-27	-33
$V_{u,f}$ (kips)	0	-1	-2	-3	-3	-4	-5	-6	-7	-8	-8
q_u (ksf)	-5.4	-1.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	4	12	19	27	35	43	51	58	66	74
$V_{u,q}$ (kips)	0	9	10	10	10	10	10	10	10	10	10
ΣM_u (ft-k)	**0**	**4**	**10**	**15**	**14**	**8**	**2**	**-2**	**-1**	**0**	**0**
ΣV_u (kips)	**0**	**8**	**8**	**2**	**-5**	**-8**	**-7**	**-2**	**2**	**1**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-2	ft-k	14.31	0.0001	0.0025	8	kips	42	kips
Bottom Longitudinal	15	ft-k	14.31	0.0005	0.0025	8	kips	42	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	169	kips	($8\, b\, d\, f_c'^{0.5}$)
ϕV_s =	5.9	kips	($A_v\, f_{yt}\, d$ / s)
ϕV_n =	42	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

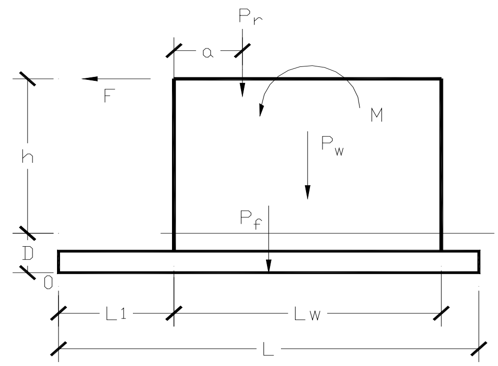
ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: APP E.5

WALL LENGTH	L_w =	4	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	8	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	4	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.34	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	2	ft
WALL SELF WEIGHT	P_w =	0.8	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	1.92	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi



Demands Summary

Stability	0.90	Bearing	0.20
Flexure-Top Steel	0.02	Flexure-Bot Steel	0.19
Shear	0.08	T&S Steel	0.95

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL		4	#	5				
BOTTOM BARS, LONGITUDINAL		4	#	5				
SHEAR REINFORCEMENT	#	4	@	6	in o.c.	2	legs	← **Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 1.73 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 6.96 kips (footing self weight)

Mo = 0.75 [F (h + D) + M] = 19 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 32 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 8.10 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 32 ft-kips (resisting moment with live load)

$e = 0.5 L - (M_R - M_O) / P =$ 2.32 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

= 0.80 ksf **<** q_b **[Satisfactory]**

Where e = 2.32 ft, > (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ 1.2 [$P_{r,DL}$ (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)] + 0.5 $P_{r,LL}$(L_1 + a) = 39 ft-kips

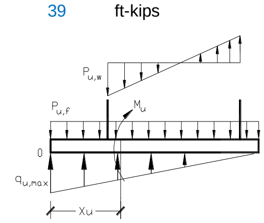
$M_{u,o} =$ 1.4 [F(h + D) + M] = 35 ft-kips

$P_u =$ 1.2 ($P_{r,DL}$ + P_f + P_w) + 0.5 $P_{r,LL}$ = 10 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 3.60 ft

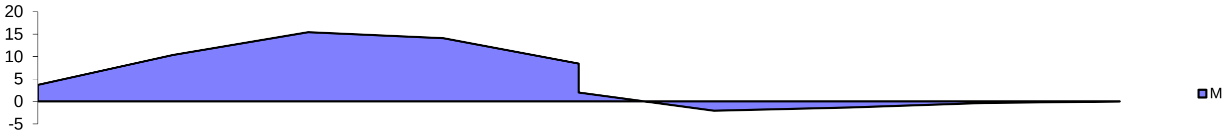
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

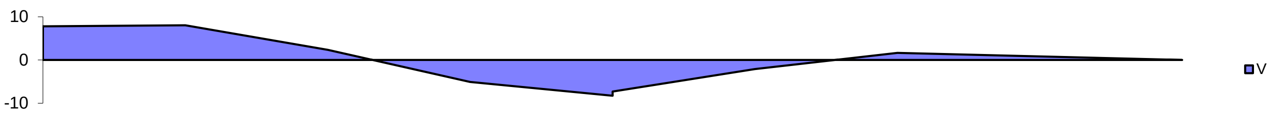
4.08 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	0.80	1.60	2.40	3.20	4.00	4.80	5.60	6.40	7.20	8.00
$P_{u,w}$ (klf)	0.0	0.0	0.0	10.8	5.6	0.3	-4.9	-10.2	0.0	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	0	-1	-8	-18	-29	-36	-38	-39	-40
$V_{u,w}$ (kips)	0	0	0	-5	-11	-14	-12	-6	-1	-1	-1
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	0	-1	-3	-5	-8	-12	-16	-21	-27	-33
$V_{u,f}$ (kips)	0	-1	-2	-3	-3	-4	-5	-6	-7	-8	-8
q_u (ksf)	-4.1	-1.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	4	12	19	27	35	43	51	58	66	74
$V_{u,q}$ (kips)	0	9	10	10	10	10	10	10	10	10	10
ΣM_u (ft-k)	**0**	**4**	**10**	**15**	**14**	**8**	**2**	**-2**	**-1**	**0**	**0**
ΣV_u (kips)	**0**	**8**	**8**	**2**	**-5**	**-8**	**-7**	**-2**	**2**	**1**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-2	ft-k	14.19	0.0000	0.0019	8	kips	56	kips
Bottom Longitudinal	15	ft-k	14.19	0.0004	0.0019	8	kips	56	kips
Bottom Transverse	0	ft-k / ft	14.75	0.0018	0.0023	1	kips / ft	15	kips / ft

Where $$\rho = \frac{0.85 f_c' \left(1 - \sqrt{1 - \dfrac{M_u}{0.383 b d^2 f_c'}}\right)}{f_y}$$

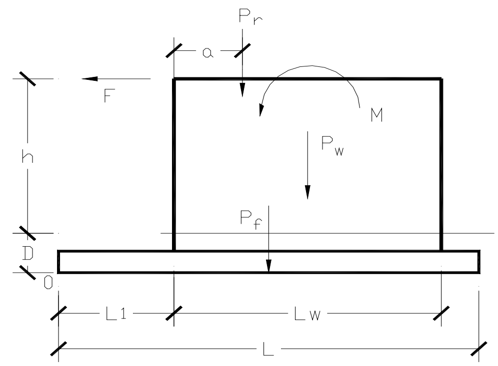
$\phi V_{s\,max}$ =	224	kips	($8\, b\, d\, f_c'^{0.5}$)
ϕV_s =	42.6	kips	($A_v\, f_{yt}\, d$ / s)
ϕV_n =	99	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0667	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0019		

ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85 \beta_1 f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: A.1



WALL LENGTH	L_w =	9.5	ft
WALL HEIGHT	h =	14	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	13.5	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.36	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	4.75	ft
WALL SELF WEIGHT	P_w =	2.86	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	2.44	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi

Demands Summary

Stability	0.94	Bearing	0.32
Flexure-Top Steel	0.02	Flexure-Bot Steel	0.55
Shear	0.41	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL		2	#	5				
BOTTOM BARS, LONGITUDINAL		2	#	5				
SHEAR REINFORCEMENT	#	3	@	24	in o.c.	2	legs	**← Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 1.65 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 4.40 kips (footing self weight)

Mo = 0.75 [F (h + D) + M] = 31 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 51 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 7.63 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 51 ft-kips (resisting moment with live load)

$e = 0.5 L - (M_R - M_O) / P =$ 4.08 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$ = 1.27 ksf **<** q_b **[Satisfactory]**

Where e = 4.08 ft, > (L / 6)

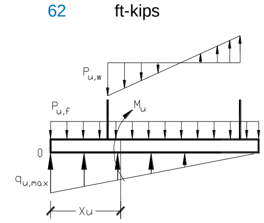
CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R}$ = $1.2 [P_{r,DL} (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)] + 0.5 P_{r,LL}(L_1 + a) =$ 62 ft-kips

$M_{u,o}$ = $1.4 [F(h + D) + M] =$ 58 ft-kips

P_u = $1.2 (P_{r,DL} + P_f + P_w) + 0.5 P_{r,LL} =$ 9 kips

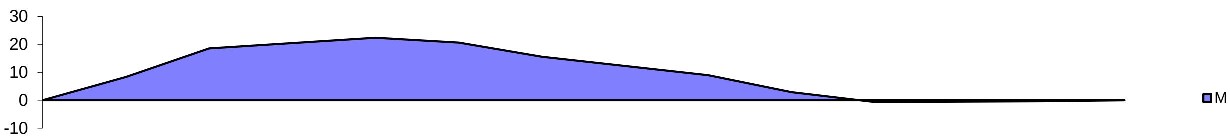
$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 6.35 ft

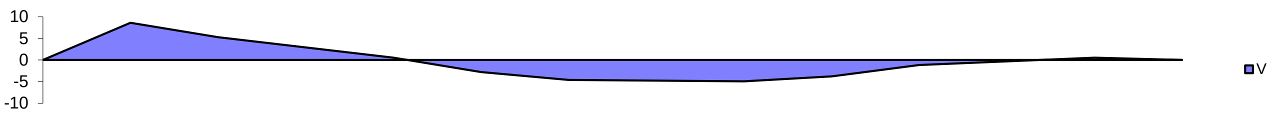


$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$ 10.14 ksf

BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	1.35	2.70	4.05	5.40	6.75	8.10	9.45	10.80	12.15	13.50
$P_{u,w}$ (klf)	0.0	0.0	3.7	2.6	1.5	0.4	-0.7	-1.8	-2.9	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	-1	-8	-19	-34	-49	-62	-73	-79	-84
$V_{u,w}$ (kips)	0	0	-3	-7	-10	-11	-11	-9	-6	-4	-4
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	0	-1	-3	-6	-9	-13	-17	-23	-29	-36
$V_{u,f}$ (kips)	0	-1	-1	-2	-2	-3	-3	-4	-4	-5	-5
q_u (ksf)	-10.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	9	21	33	46	58	70	83	95	108	120
$V_{u,q}$ (kips)	0	9	9	9	9	9	9	9	9	9	9
ΣM_u (ft-k)	**0**	**8**	**19**	**22**	**21**	**16**	**9**	**3**	**-1**	**0**	**0**
ΣV_u (kips)	**0**	**9**	**5**	**1**	**-3**	**-5**	**-5**	**-4**	**-1**	**1**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0000	0.0025	9	kips	21	kips
Bottom Longitudinal	22	ft-k	14.31	0.0014	0.0025	9	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	85	kips	($8\, b\, d\, f_c'^{0.5}$)
ϕV_s =	5.9	kips	($A_v\, f_{yt}\, d$ / s)
ϕV_n =	21	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

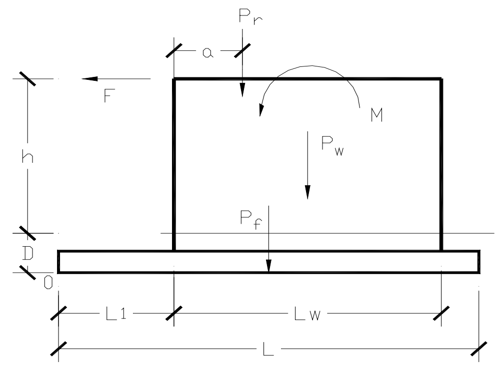
ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA: C.1

WALL LENGTH	L_w =	14	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	18	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.53	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	7	ft
WALL SELF WEIGHT	P_w =	3.01	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	4.72	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi
TOP BARS, LONGITUDINAL		2	# 5
BOTTOM BARS, LONGITUDINAL		2	# 5
SHEAR REINFORCEMENT	#	3	@ 24 in o.c. 2 legs ← Not Required



Demands Summary

Stability	0.84	Bearing	0.25
Flexure-Top Steel	0.02	Flexure-Bot Steel	0.61
Shear	0.33	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 1.84 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 5.87 kips (footing self weight)

$Mo = 0.75\,[F\,(h + D) + M] =$ 46 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 85 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 9.42 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 85 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 4.88 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

= 1.02 ksf < q_b

[Satisfactory]

Where e = 4.88 ft, > (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} =$ $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 102 ft-kips

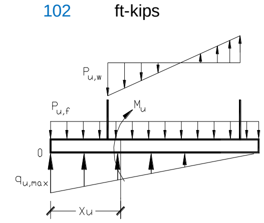
$M_{u,o} =$ $1.4\,[F(h + D) + M] =$ 86 ft-kips

$P_u =$ $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 11 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 7.60 ft

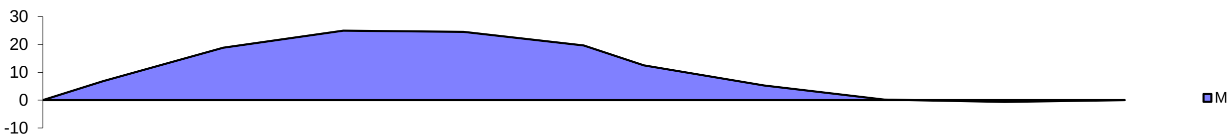
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

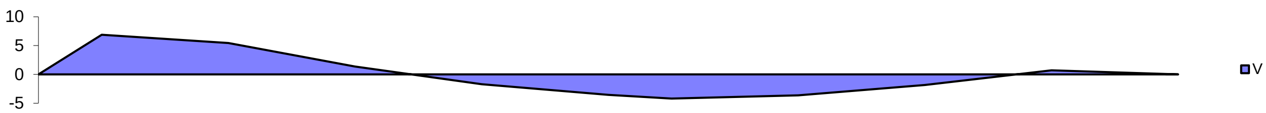
3.58 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	1.80	3.60	5.40	7.20	9.00	10.80	12.60	14.40	16.20	18.00
$P_{u,w}$ (klf)	0.0	0.0	2.3	1.7	1.0	0.3	-0.4	-1.0	-1.7	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	-3	-14	-31	-50	-71	-90	-106	-116	-124
$V_{u,w}$ (kips)	0	0	-4	-8	-10	-11	-11	-10	-7	-4	-4
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-1	-3	-6	-10	-16	-23	-31	-41	-51	-63
$V_{u,f}$ (kips)	0	-1	-1	-2	-3	-4	-4	-5	-6	-6	-7
q_u (ksf)	-3.6	-2.0	-0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	7	25	45	65	86	106	127	147	167	188
$V_{u,q}$ (kips)	0	8	11	11	11	11	11	11	11	11	11
ΣM_u (ft-k)	**0**	**7**	**19**	**25**	**25**	**20**	**12**	**5**	**0**	**-1**	**0**
ΣV_u (kips)	**0**	**7**	**5**	**1**	**-2**	**-4**	**-4**	**-4**	**-2**	**1**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0000	0.0025	7	kips	21	kips
Bottom Longitudinal	25	ft-k	14.31	0.0015	0.0025	7	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

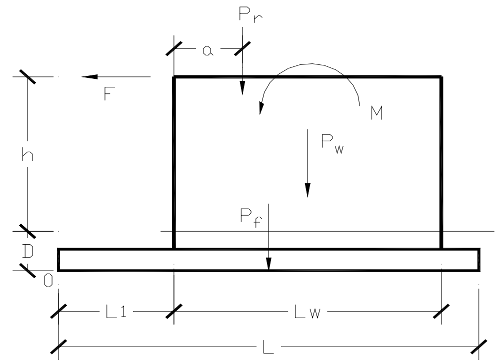
Where
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \dfrac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	85	kips	$(8\, b\, d\, f_c'^{0.5})$
ϕV_s =	5.9	kips	$(A_v\, f_{yt}\, d\, /\, s)$
ϕV_n =	21	kips	$(\phi V_s + \phi V_c)$
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\,\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Footing Design of Shear Wall Based on ACI 318-14



INPUT DATA: F.1

WALL LENGTH	L_w =	4	ft
WALL HEIGHT	h =	10	ft
WALL THICKNESS	t =	8	in
FOOTING LENGTH	L =	8	ft
	L_1 =	2	ft
FOOTING WIDTH	B =	1.5	ft
FOOTING THICKNESS	T =	1.5	ft
FOOTING EMBEDMENT DEPTH	D =	3	ft
ALLOWABLE SOIL PRESSURE (D+L)	q_a =	3000	psf
ALLOW. SOIL (D+L+0.7E) OR (D+L+0.6W)	qb =	4000	psf
UNFACTORED DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.15	kips
UNFACTORED LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips
TOP LOAD LOCATION	a =	2	ft
WALL SELF WEIGHT	P_w =	0.86	kips
LATERAL LOAD TYPE (0=wind,1=seismic)		1	seismic
SEISMIC LOADS AT TOP (E/1.4 , ASD)	F =	0.74	kips
	M =	0	ft-kips
CONCRETE STRENGTH	f_c' =	3000	psi
REBAR YIELD STRESS	f_y =	60	ksi

Demands Summary

Stability	0.77	Bearing	0.20
Flexure-Top Steel	0.02	Flexure-Bot Steel	0.12
Shear	0.14	T&S Steel	0.72

THE FOOTING DESIGN IS ADEQUATE.

TOP BARS, LONGITUDINAL		2	#	5			
BOTTOM BARS, LONGITUDINAL		2	#	5			
SHEAR REINFORCEMENT	#	3	@	24	in o.c.	2 legs	← **Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (CBC 1605.2, 1808.3.1)

$F = M_R / M_O =$ 2.01 **>** 1.4 / 0.9 for seismic **[Satisfactory]**

Where P_f = 2.61 kips (footing self weight)

$M_o = 0.75\,[F\,(h + D) + M] =$ 7 ft-kips (overturning moment) ASCE 7 §12.13.4

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 14 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

$P = (P_{r,DL} + P_{r,LL}) + P_w + P_f =$ 3.62 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 14 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 1.99 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

= 0.80 ksf **<** q_b **[Satisfactory]**

Where e = 1.99 ft, > (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R} = 1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 17 ft-kips

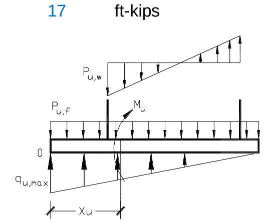
$M_{u,o} = 1.4\,[F(h + D) + M] =$ 13 ft-kips

$P_u = 1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 4 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 3.10 ft

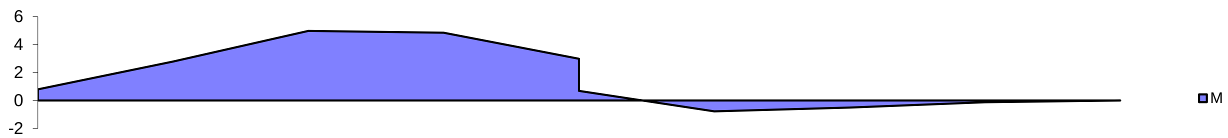
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

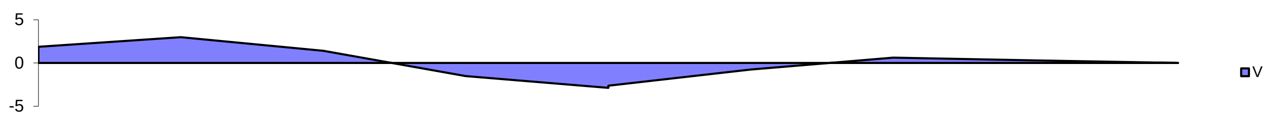
2.14 ksf



BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	0.80	1.60	2.40	3.20	4.00	4.80	5.60	6.40	7.20	8.00
$P_{u,w}$ (klf)	0.0	0.0	0.0	4.3	2.3	0.3	-1.7	-3.7	0.0	0.0	0.0
$M_{u,w}$ (ft-k)	0	0	0	0	-3	-7	-12	-15	-16	-17	-18
$V_{u,w}$ (kips)	0	0	0	-2	-5	-6	-5	-3	-1	-1	-1
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	0	-1	-1	-2	-3	-5	-6	-8	-10	-13
$V_{u,f}$ (kips)	0	0	-1	-1	-1	-2	-2	-2	-3	-3	-3
q_u (ksf)	-2.1	-1.5	-0.9	-0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	1	3	7	10	13	17	20	24	27	31
$V_{u,q}$ (kips)	0	2	4	4	4	4	4	4	4	4	4
ΣM_u (ft-k)	**0**	**1**	**3**	**5**	**5**	**3**	**1**	**-1**	**-1**	**0**	**0**
ΣV_u (kips)	**0**	**2**	**3**	**1**	**-2**	**-3**	**-3**	**-1**	**1**	**0**	**0**





Location	$M_{u,max}$		d (in)	$\rho_{analysis}$	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	14.31	0.0000	0.0025	3	kips	21	kips
Bottom Longitudinal	5	ft-k	14.31	0.0003	0.0025	3	kips	21	kips
Bottom Transverse	0	ft-k / ft	14.81	0.0000	0.0000	0	kips / ft	15	kips / ft

Where
$$\rho = \frac{0.85 f'_c \left(1-\sqrt{1-\frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

$\phi V_{s\,max}$ =	85	kips	($8\, b\, d\, f_c'^{0.5}$)
ϕV_s =	5.9	kips	($A_v f_{yt} d / s$)
ϕV_n =	21	kips	($\phi V_s + \phi V_c$)
$A_{v\,min}$ =	0.0000	in^2/in	ACI 318-14 9.6.3.3
$A_{v\,provided}$ =	0.0092	in^2/in	**[Satisfactory]**
Min shear tie spacing =	0.0	in oc	ACI 318-14 9.7.6.2.2
ρ_{min} =	0.0018		
$\rho_{min\ provided\ tension\ face}$ =	0.0025		

ϕ = **0.75** ACI 318-14 21.2.1

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\,\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} =$$ 0.0155 **[Satisfactory]**

Mechanical Anchorage

Revised design for 2022 CBC does not affect Mechanical anchorage analysis

CBC2019, ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Soil Site Class	D	Table 20.3-1	
Response Spectral Acc. (0.2 sec) Ss =	**1.582g**	= 158.20%g	Figure 22-1, 22-3, 22-5, and 22-6
Response Spectral Acc.(1.0 sec) S1 =	**0.600g**	= 60.00%g	Figure 22-2, 22-4, 22-5, and 22-6
Site Coefficient F_a =	1.000		Table 11.4-1
Site Coefficient F_v =	1.7		Table 11.4-2
Max Considered Earthquake Acc. S_{MS} =	$F_a.S_s$	= 1.582	(11.4-1)
Max Considered Earthquake Acc. S_{M1} =	$F_v.S_1$	= 1.020	(11.4-2)
@ 5% Damped Design S_{DS} =	$2/3(S_{MS})$	= 1.055	(11.4-3)
S_{D1} =	$2/3(S_{M1})$	= 0.680	(11.4-4)
Building Risk Categories	IV	Essential	Table 1.5-1
Design Category Consideration:	Flexible Diaphragm	with dist. between seismic resisting system >40ft	
Seismic Design Category for 0.1sec	D		
Seismic Design Category for 1.0sec	D		
S1 < .75g	NA		

Comply with Seismic Design Category D

13.3 Seismic Demands on Nonstructural Components

Component Name: Air Conditioning system <= Most Equipment

Component Description: Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing

$$F_p = \frac{0.4a_pS_{DS}W_p(1+2z/h)}{(R_p/I_p)}$$ (13.3-1)

S_{DS} = 1.055

a_p = **2.5** R_p = **6.0** T-13.5-1 or 13.6-1

Ω_o = **2.0** T-13.5-1 or 13.6-1

I_p = **1.0** 13.1.3

z = **0 ft** h = **21.5 ft** F_p = 0.176 Wp

Max F_p = $1.6S_{DS}I_pW_p$ = 1.687Wp (13.3-2)

Min F_p = $0.3S_{DS}I_pW_p$ = 0.316Wp (13.3-3)

F_p = **0.316 Wp**

$F_{p\ \text{Anchorage to Concrete IF using}\ \Omega_o}$ = 0.633 Wp T-13.5-1 footnote b or 13.6-1 footnote c

CBC 2019, ASCE 7-16, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = ARUM096BTE5 (1/M5.1)

Unit Weight, W_p = 550.0 lbs I_p = 1.5   13.1.3

OSHPD Project = NO

Anchored to Concrete and using Ω_o for anchorage = YES   Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = NO

Fp FORCE:

Table 13.5-1 or Table 13.6-1: a_p = 2.5 R_p = 6.0 Ω_o = 2.0

z = 15 ft h = 21.5 ft

$$F_p = \frac{0.4a_pS_{DS}W_p(1+2z/h)}{(R_p/I_p)} \quad 0.632\ Wp \qquad (13.3\text{-}1)$$

$F_{p(max)}$ = $1.6S_{DS}I_pW_p$ = 2.531 Wp   (13.3-2)

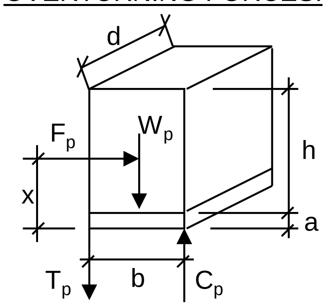
$F_{p(min)}$ = $0.3S_{DS}I_pW_p$ = 0.475 Wp   (13.3-3)

F_p = 0.632 Wp   = MAX(MIN(F_p, $F_{p(max)}$), $F_{p(min)}$)

Ωo Fp = 1.263 Wp

Ωo Fp = 0.695 kips

OVERTURNING FORCES: Method: LRFD



Distance between anchors, b = 30 in   short side of MU

Distance between anchors, d = 49 in   long side of MU

Height, h = 67 in

Frame base, a = 0 in

C.G., x = a + h/2 = 34 in

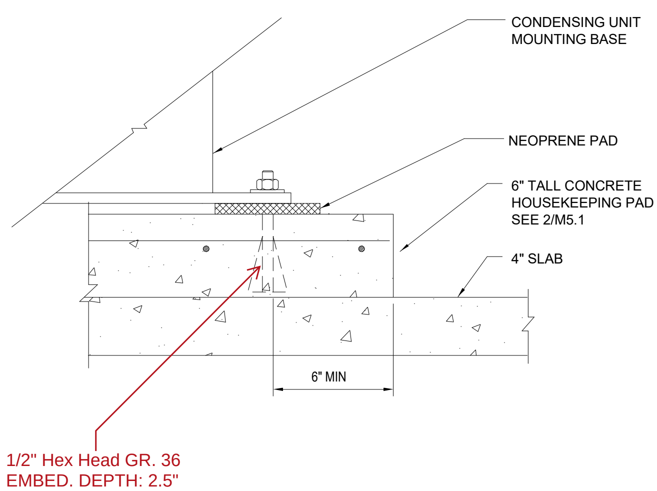
Mo = F_px = 23.3 kip-in   overturning moment

Mrb(T) = Wp(0.9-0.2SDS)b/2 = 5.7 kip-in   resisting moment in 'b' direction

Mrb(C) = Wp(1.2+0.2SDS)b/2 = 11.6 kip-in   resisting moment in 'b' direction

Tp(MAX) = (Mo-Mb(T))/b = **0.586 kips**

Cp(MAX) = (Mo+Mrb(C))/b = **1.164 kips**



GRADE MTD. CONDENSING MTG. DTL.   1 M5.1

SCALE : NONE

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Design:	Butterfield Concrete Pad	Date:	1/30/2023
Fastening point:			

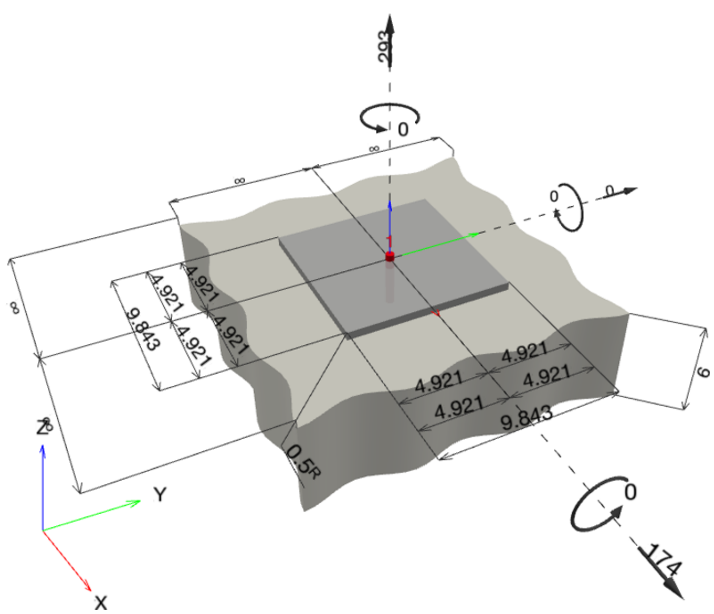
Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1/2**
Item number:	not available
Effective embedment depth:	h_{ef} = 2.500 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 9.843 in. x 9.843 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 6.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d))
	Shear load: yes (17.10.6.3 (c))

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

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



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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 293$; $V_x = 174$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	yes	12

2 Load case/Resulting anchor forces

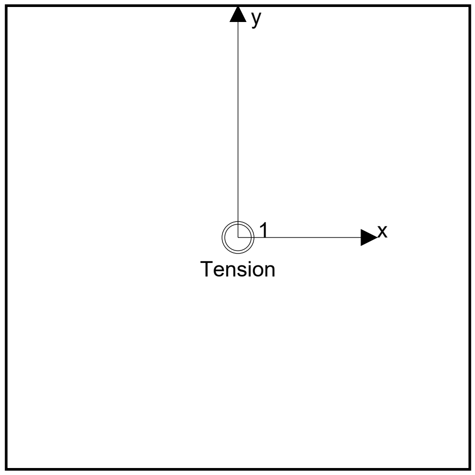
Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	293	174	174	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 293 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

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



3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	293	6,177	5	OK
Pullout Strength*	293	3,055	10	OK
Concrete Breakout Failure**	293	2,490	12	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-19 Eq. (17.6.1.2)

$\phi\, N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.14	58,000

Calculations

N_{sa} [lb]
8,236

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\, N_{sa}$ [lb]	N_{ua} [lb]
8,236	0.750	6,177	293

3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-19 Eq. (17.6.3.1)

$N_p = 8\, A_{brg}\, f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)

$\phi\, N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.29	1.000	2,500

Calculations

N_p [lb]
5,820

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\, N_{pn}$ [lb]	N_{ua} [lb]
5,820	0.700	0.750	1.000	3,055	293

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3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
2.500	∞	1.000	-	24	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
56.25	56.25	1.000	1.000	4,743

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
4,743	0.700	0.750	1.000	2,490	293

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	174	3,212	6	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	174	6,641	3	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.14	58,000

Calculations

V_{sa} [lb]
4,942

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa,eq}$ [lb]	V_{ua} [lb]
4,942	0.650	3,212	174

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4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-19 Eq. (17.7.3.1a)

$\phi\ V_{cp} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2	2.500	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
∞	24	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
56.25	56.25	1.000	1.000	4,743

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
9,487	0.700	1.000	1.000	6,641	174

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.118	0.054	5/3	4	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.

Fastening meets the design criteria!

General Footing

Project File: morgan hill.ec6

LIC# : KW-06015331, Build:20.22.10.25 ZFA STRUCTURAL ENGINEERS

DESCRIPTION: Concrete Pad ARUM096BTE5

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	Yes
Use Pedestal wt for stability, mom & shear	:	Yes

Soil Design Values

Allowable Soil Bearing	=	2.50 ksf
Soil Density	=	110.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Footing base depth below soil surface	=	1.0 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

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

Dimensions

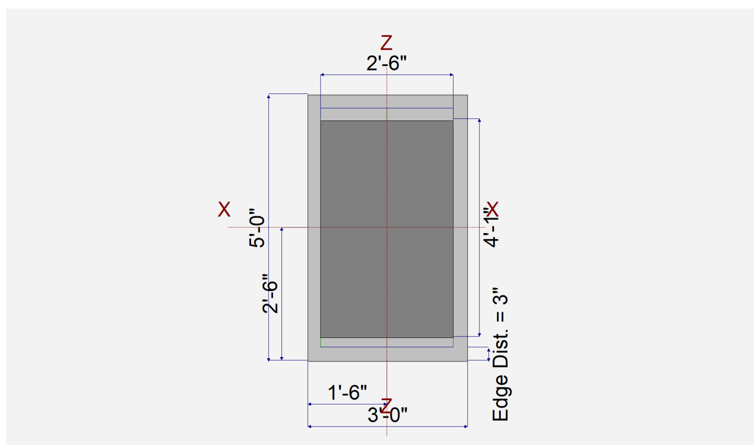
Width parallel to X-X Axis	=	3.0 ft
Length parallel to Z-Z Axis	=	5.0 ft
Footing Thickness	=	9.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	30.0 in
pz : parallel to Z-Z Axis	=	49.0 in
Height	=	67.0 in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in



Reinforcing

Bars parallel to X-X Axis

Number of Bars	=	5.0
Reinforcing Bar Size	=	# 4

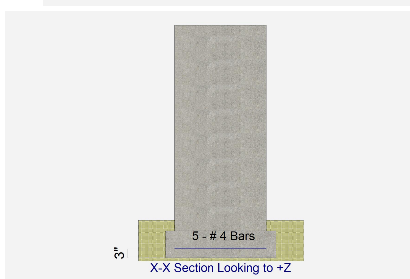
Bars parallel to Z-Z Axis

Number of Bars	=	3.0
Reinforcing Bar Size	=	# 4

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation

	Bars along X-X Axis
# Bars required within zone	75.0 %
# Bars required on each side of zone	25.0 %



X-X Section Looking to +Z

Z-Z Section Looking to +X

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	0.5550							k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=						0.6950		k
V-z	=								k

General Footing

Project File: morgan hill.ec6

LIC# : KW-06015331, Build:20.22.10.25 ZFA STRUCTURAL ENGINEERS

DESCRIPTION: Concrete Pad ARUM096BTE5

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.4448	Soil Bearing	1.112 ksf	2.50 ksf	+D+0.70E about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	3.091	Overturning - Z-Z	3.081 k-ft	9.524 k-ft	+0.60D+0.70E
PASS	5.120	Sliding - X-X	0.4865 k	2.491 k	+0.60D+0.70E
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.007390	Z Flexure (+X)	0.03860 k-ft/ft	5.224 k-ft/ft	+1.20D+E
PASS	0.004760	Z Flexure (-X)	0.02487 k-ft/ft	5.224 k-ft/ft	+1.40D
PASS	0.01602	X Flexure (+Z)	0.08368 k-ft/ft	5.224 k-ft/ft	+1.40D
PASS	0.01602	X Flexure (-Z)	0.08368 k-ft/ft	5.224 k-ft/ft	+1.40D
PASS	n/a	1-way Shear (+X)	0.0 psi	82.158 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a
PASS	n/a	1-way Shear (+Z)	0.0 psi	82.158 psi	n/a
PASS	n/a	1-way Shear (-Z)	0.0 psi	82.158 psi	n/a
PASS	n/a	2-way Punching	0.5769 psi	82.158 psi	+1.40D

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.50	n/a	0.0	0.7055	0.7055	n/a	n/a	0.282
X-X, +D+0.70E	2.50	n/a	0.0	0.7055	0.7055	n/a	n/a	0.282
X-X, +D+0.5250E	2.50	n/a	0.0	0.7055	0.7055	n/a	n/a	0.282
X-X, +0.60D	2.50	n/a	0.0	0.4233	0.4233	n/a	n/a	0.169
X-X, +0.60D+0.70E	2.50	n/a	0.0	0.4233	0.4233	n/a	n/a	0.169
Z-Z, D Only	2.50	0.0	n/a	n/a	n/a	0.7055	0.7055	0.282
Z-Z, +D+0.70E	2.50	3.494	n/a	n/a	n/a	0.2988	1.112	0.445
Z-Z, +D+0.5250E	2.50	2.620	n/a	n/a	n/a	0.4005	1.011	0.404
Z-Z, +0.60D	2.50	0.0	n/a	n/a	n/a	0.4233	0.4233	0.169
Z-Z, +0.60D+0.70E	2.50	5.823	n/a	n/a	n/a	0.01659	0.830	0.332

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, D Only	None	0.0 k-ft	Infinity	OK
X-X, +D+0.70E	None	0.0 k-ft	Infinity	OK
X-X, +D+0.5250E	None	0.0 k-ft	Infinity	OK
X-X, +0.60D	None	0.0 k-ft	Infinity	OK
X-X, +0.60D+0.70E	None	0.0 k-ft	Infinity	OK
Z-Z, D Only	None	0.0 k-ft	Infinity	OK
Z-Z, +D+0.70E	3.081 k-ft	15.874 k-ft	5.152	OK
Z-Z, +D+0.5250E	2.311 k-ft	15.874 k-ft	6.869	OK
Z-Z, +0.60D	None	0.0 k-ft	Infinity	OK
Z-Z, +0.60D+0.70E	3.081 k-ft	9.524 k-ft	3.091	OK

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
X-X, D Only	0.0 k	3.761 k	No Sliding	OK
X-X, +D+0.70E	0.4865 k	3.761 k	7.730	OK
X-X, +D+0.5250E	0.3649 k	3.761 k	10.307	OK
X-X, +0.60D	0.0 k	2.491 k	No Sliding	OK
X-X, +0.60D+0.70E	0.4865 k	2.491 k	5.120	OK
Z-Z, D Only	0.0 k	3.526 k	No Sliding	OK
Z-Z, +D+0.70E	0.0 k	3.526 k	No Sliding	OK
Z-Z, +D+0.5250E	0.0 k	3.526 k	No Sliding	OK
Z-Z, +0.60D	0.0 k	2.256 k	No Sliding	OK
Z-Z, +0.60D+0.70E	0.0 k	2.256 k	No Sliding	OK

General Footing

Project File: morgan hill.ec6

LIC# : KW-06015331, Build:20.22.10.25 ZFA STRUCTURAL ENGINEERS (c) ENERCALC INC 1983-2022

DESCRIPTION: Concrete Pad ARUM096BTE5

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.08368	+Z	Bottom	0.1944	AsMin	0.20	5.224	OK
X-X, +1.40D	0.08368	-Z	Bottom	0.1944	AsMin	0.20	5.224	OK
X-X, +1.20D	0.07173	+Z	Bottom	0.1944	AsMin	0.20	5.224	OK
X-X, +1.20D	0.07173	-Z	Bottom	0.1944	AsMin	0.20	5.224	OK
X-X, +1.20D+E	0.07173	+Z	Bottom	0.1944	AsMin	0.20	5.224	OK
X-X, +1.20D+E	0.07173	-Z	Bottom	0.1944	AsMin	0.20	5.224	OK
X-X, +0.90D	0.05379	+Z	Bottom	0.1944	AsMin	0.20	5.224	OK
X-X, +0.90D	0.05379	-Z	Bottom	0.1944	AsMin	0.20	5.224	OK
X-X, +0.90D+E	0.05379	+Z	Bottom	0.1944	AsMin	0.20	5.224	OK
X-X, +0.90D+E	0.05379	-Z	Bottom	0.1944	AsMin	0.20	5.224	OK
Z-Z, +1.40D	0.02487	-X	Bottom	0.1944	AsMin	0.20	5.224	OK
Z-Z, +1.40D	0.02487	+X	Bottom	0.1944	AsMin	0.20	5.224	OK
Z-Z, +1.20D	0.02131	-X	Bottom	0.1944	AsMin	0.20	5.224	OK
Z-Z, +1.20D	0.02131	+X	Bottom	0.1944	AsMin	0.20	5.224	OK
Z-Z, +1.20D+E	0.004023	-X	Bottom	0.1944	AsMin	0.20	5.224	OK
Z-Z, +1.20D+E	0.03860	+X	Bottom	0.1944	AsMin	0.20	5.224	OK
Z-Z, +0.90D	0.01599	-X	Bottom	0.1944	AsMin	0.20	5.224	OK
Z-Z, +0.90D	0.01599	+X	Bottom	0.1944	AsMin	0.20	5.224	OK
Z-Z, +0.90D+E	0.001305	-X	Top	0.1944	AsMin	0.20	5.224	OK
Z-Z, +0.90D+E	0.03327	+X	Bottom	0.1944	AsMin	0.20	5.224	OK

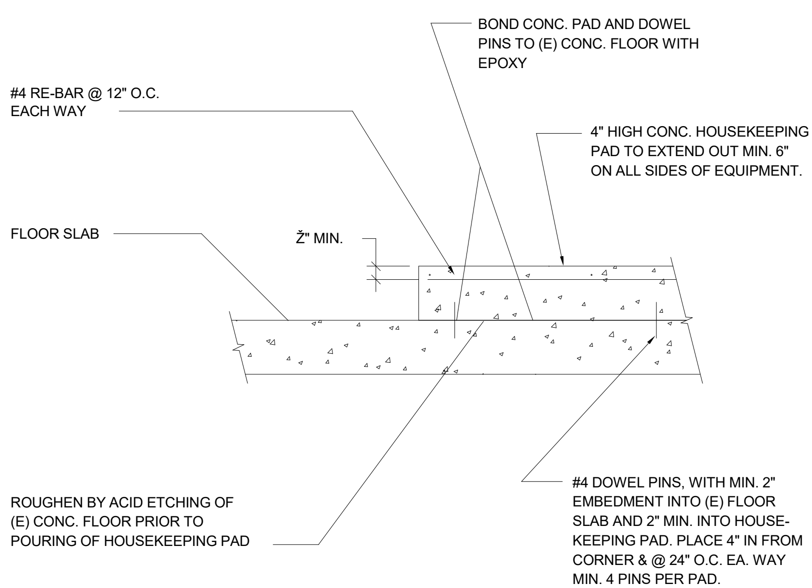
One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	82.16 psi	0.00	OK
+1.20D	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	82.16 psi	0.00	OK
+1.20D+E	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	82.16 psi	0.00	OK
+0.90D	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	82.16 psi	0.00	OK
+0.90D+E	0.00 psi	0.00 psi	0.00 psi	0.00 psi	0.00 psi	82.16 psi	0.00	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	0.58 psi	136.33 psi	0.004232	OK
+1.20D	0.58 psi	136.33 psi	0.004232	OK
+1.20D+E	0.58 psi	136.33 psi	0.004232	OK
+0.90D	0.58 psi	136.33 psi	0.004232	OK
+0.90D+E	0.58 psi	136.33 psi	0.004232	OK



CONC. EQUIPMENT PAD

SCALE : NONE

2
M5.1

CBC 2019, ASCE 7-16, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = ARNU053TRD4 (4/M5.1)

Unit Weight, W_p = 35.0 lbs I_p = 1.5 13.1.3

OSHPD Project = NO

Anchored to Concrete and using Ω_o for anchorage = NO Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = NO

Fp FORCE:

Table 13.5-1 or Table 13.6-1: a_p = 2.5 R_p = 6.0 Ω_o = 2.0

z = 15 ft h = 21.5 ft

$$F_p = \frac{0.4a_pS_{DS}W_p(1+2z/h)}{(R_p/I_p)} \quad 0.632\ Wp \qquad (13.3\text{-}1)$$

$F_{p(max)}$ = $1.6S_{DS}I_pW_p$ = 2.531 Wp (13.3-2)

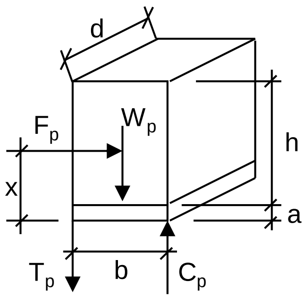
$F_{p(min)}$ = $0.3S_{DS}I_pW_p$ = 0.475 Wp (13.3-3)

F_p = 0.632 Wp = MAX(MIN(F_p, $F_{p(max)}$), $F_{p(min)}$)

Fp = 0.632 Wp

Fp = 0.022 kips

OVERTURNING FORCES: Method: LRFD



Distance between anchors, b = 20 in short side of MU

Distance between anchors, d = 20 in long side of MU

Height, h = 10 in

Frame base, a = 0 in

C.G., x = a + h/2 = 5 in

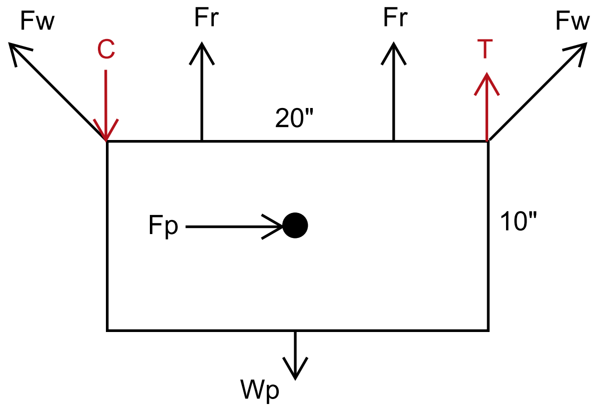
Mo = F_px = 0.1 kip-in overturning moment

Mrb(T) = Wp(0.9-0.2SDS)b/2 = 0.2 kip-in resisting moment in 'b' direction

Mrb(C) = Wp(1.2+0.2SDS)b/2 = 0.5 kip-in resisting moment in 'b' direction

Tp(MAX) = (Mo-Mb(T))/b = **-0.007 kips *NO UPLIFT***

Cp(MAX) = (Mo+Mrb(C))/b = **0.030 kips**



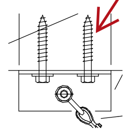
Unit Weight, W_p = 35.0 lbs

Fp = 0.022 kips

Tp(MAX) = (Mo-Mb(T))/b = **-0.007 kips** ***NO UPLIFT***

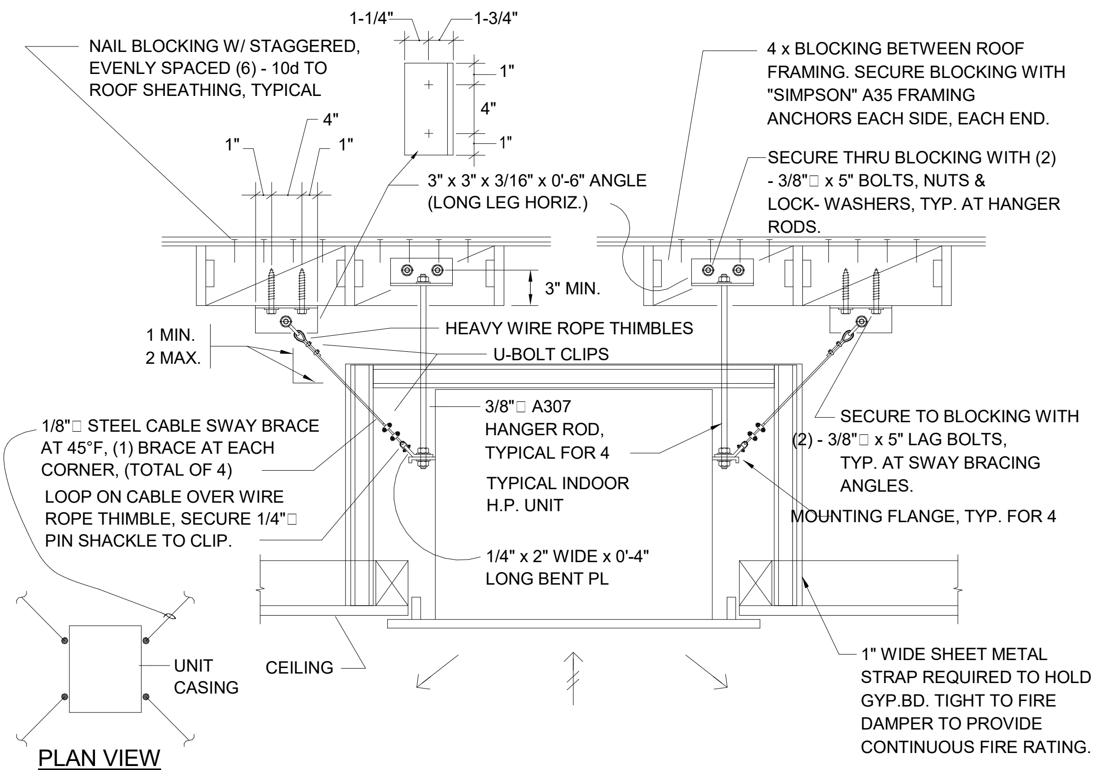
Cp(MAX) = (Mo+Mrb(C))/b = **0.030 kips**

$F_w = \frac{F_p \cdot \sqrt{2}}{2}$ = 16# <= Wire ok by observation



(2) 3/8" x 5" Lag Bolts

V = 267#/Bolt (AWC) >> 22#
T = 810#/Bolt >> 65# (30# (C) + 35# (Wp))



CLG. MTD. H.P. INDOOR UNIT MOUNTING

SCALE : NONE

4
M5.1

CBC 2019, ASCE 7-16, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = PRHR063A (7/M5.1)

Unit Weight, W_p = 75.0 lbs I_p = 1.5 13.1.3

OSHPD Project = NO

Anchored to Concrete and using Ω_o for anchorage = NO Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = YES Fp increased x2 (see Table 13.6-1 footnote b to see if Fp can be reduced)

Fp FORCE:

Table 13.5-1 or Table 13.6-1: a_p = 2.5 R_p = 6.0 Ω_o = 2.0

z = 15 ft h = 21.5 ft

$$F_p = \frac{0.4 a_p S_{DS} W_p (1+2z/h)}{(R_p/I_p)} \quad 0.632\ Wp \qquad (13.3\text{-}1)$$

$$F_{p(max)} = 1.6 S_{DS} I_p W_p = 2.531\ Wp \qquad (13.3\text{-}2)$$

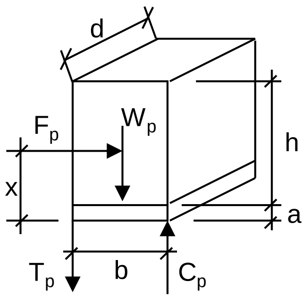
$$F_{p(min)} = 0.3 S_{DS} I_p W_p = 0.475\ Wp \qquad (13.3\text{-}3)$$

F_p = 0.632 Wp = MAX(MIN(F_p, $F_{p(max)}$), $F_{p(min)}$)

Fp = 0.632 Wp

Fp = 0.095 kips

OVERTURNING FORCES: Method: LRFD



Distance between anchors, b = 19 in short side of MU

Distance between anchors, d = 31 in long side of MU

Height, h = 9 in

Frame base, a = 0 in

C.G., x = a + h/2 = 5 in

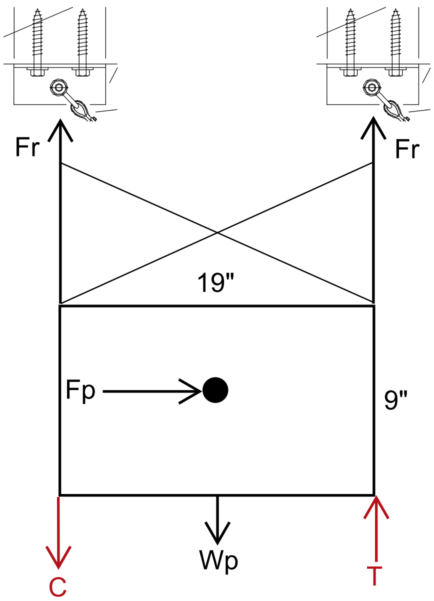
Mo = F_px = 0.4 kip-in overturning moment

Mrb(T) = Wp(0.9-0.2SDS)b/2 = 0.5 kip-in resisting moment in 'b' direction

Mrb(C) = Wp(1.2+0.2SDS)b/2 = 1.0 kip-in resisting moment in 'b' direction

Tp(MAX) = (Mo-Mb(T))/b = **-0.003 kips** ***NO UPLIFT***

Cp(MAX) = (Mo+Mrb(C))/b = **0.075 kips**



Unit Weight, W_p = 75.0 lbs

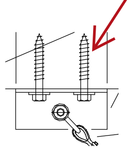
Fp = 0.095 kips

Tp(MAX) = (Mo-Mb(T))/b = **-0.003 kips** ***NO UPLIFT***

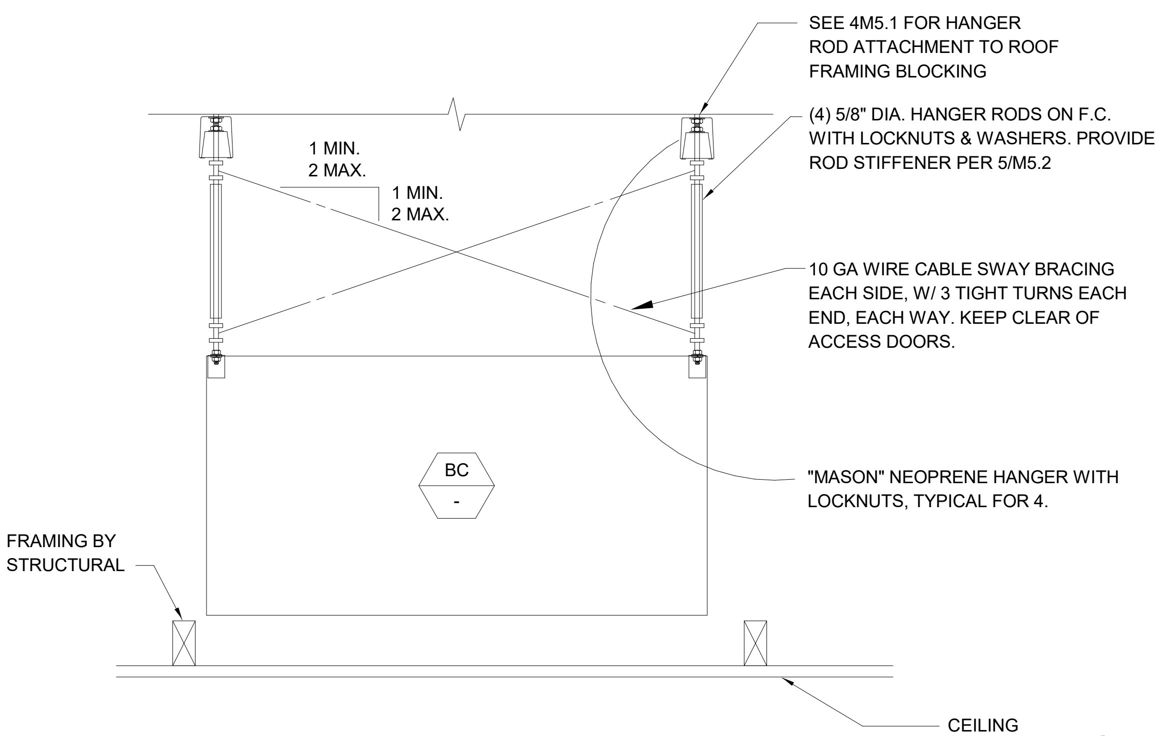
Cp(MAX) = (Mo+Mrb(C))/b = **0.075 kips**

$Fw = \frac{Fp * \sqrt{2}}{2} = 67\#$ <= Wire ok by observation

(2) 3/8" x 5" Lag Bolts



V = 267#/Bolt (AWC) >> 95#
T = 810#/Bolt >> 150# (75# (C) + 75# (Wp))



VRF BRANCH CONTROLLER MOUNTING

SCALE : NONE

7
M5.1

CBC 2019, ASCE 7-16, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = ARVU123ZFA2 (1/M5.2)

Unit Weight, W_p = 335.0 lbs I_p = 1.5 13.1.3

OSHPD Project = NO

Anchored to Concrete and using Ω_o for anchorage = NO Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = YES Fp increased x2 (see Table 13.6-1 footnote b to see if Fp can be reduced)

Fp FORCE:

Table 13.5-1 or Table 13.6-1: a_p = 2.5 R_p = 6.0 Ω_o = 2.0

z = 15 ft h = 21.5 ft

$$F_p = \frac{0.4 a_p S_{DS} W_p (1+2z/h)}{(R_p/I_p)} = 0.632\ Wp \quad (13.3\text{-}1)$$

$F_{p(max)}$ = $1.6 S_{DS} I_p W_p$ = 2.531 Wp (13.3-2)

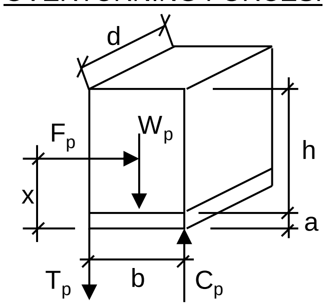
$F_{p(min)}$ = $0.3 S_{DS} I_p W_p$ = 0.475 Wp (13.3-3)

F_p = 0.632 Wp = MAX(MIN(F_p, $F_{p(max)}$), $F_{p(min)}$)

Fp = 0.632 Wp

Fp = 0.423 kips

OVERTURNING FORCES: Method: LRFD



Distance between anchors, b = 40 in short side of MU

Distance between anchors, d = 43 in long side of MU

Height, h = 29 in

Frame base, a = 4 in

C.G., x = a + h/2 = 19 in

Mo = $F_p x$ = 7.9 kip-in overturning moment

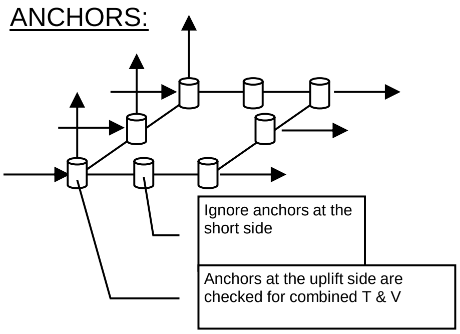
Mrb(T) = Wp(0.9-0.2SDS)b/2 = 4.6 kip-in resisting moment in 'b' direction

Mrb(C) = Wp(1.2+0.2SDS)b/2 = 9.5 kip-in resisting moment in 'b' direction

Tp(MAX) = (Mo-Mb(T))/b = **0.081 kips**

Cp(MAX) = (Mo+Mrb(C))/b = **0.433 kips**

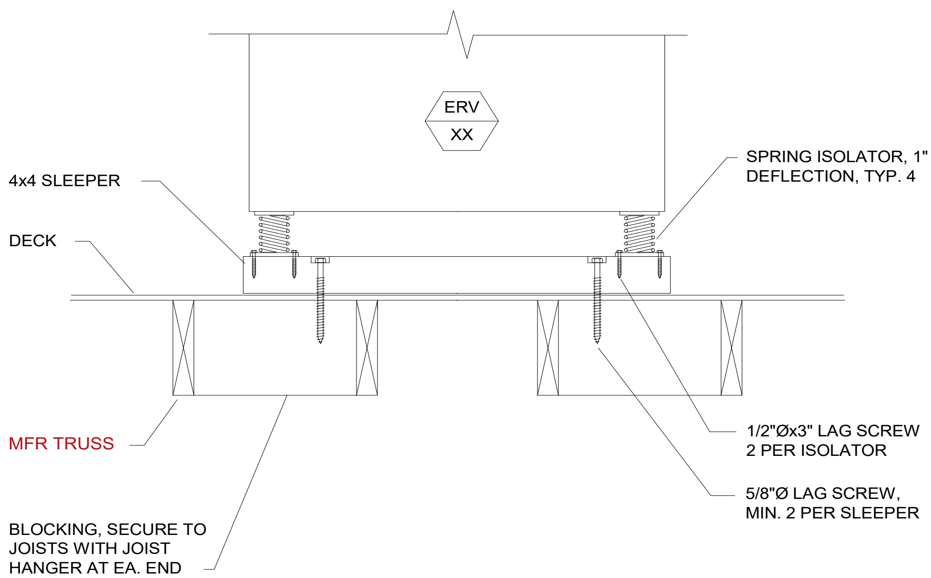
ANCHORS:



Anchor Type = 1/2"Ø 3" Lag Screw 2 per Isolator

Anchors per side, N = **2**

T = Tp/N	0.040	kips
V = Fp/2/N	0.106	kips
Tcap (LRFD) =	1.022	kips
Vcap (LRFD) =	0.264	kips
Tension D/C =	**0.040**	**< 1.0 OK**
Shear D/C =	**0.401**	**< 1.0 OK**
Combined D/C = T/Tcap+V/Vcap =	**0.440**	**< 1.2 OK**
Per ACI 318 §17.6 Combined D/C = T/Tcap + V/Vcap =	**0.440**	**< 1.2 OK**



ERV MOUNTING DETAIL

SCALE : NONE

1
M5.2

CBC 2019, ASCE 7-16, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = GRV 1&2

Unit Weight, W_p = 160.0 lbs I_p = 1.5 13.1.3

OSHPD Project = NO

Anchored to Concrete and using Ω_o for anchorage = NO Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = NO

Fp FORCE:

Table 13.5-1 or Table 13.6-1: a_p = 2.5 R_p = 6.0 Ω_o = 2.0

z = 15 ft h = 21.5 ft

$$F_p = \frac{0.4a_pS_{DS}W_p(1+2z/h)}{(R_p/I_p)}$$ 0.632 Wp (13.3-1)

$F_{p(max)}$ = $1.6S_{DS}I_pW_p$ = 2.531 Wp (13.3-2)

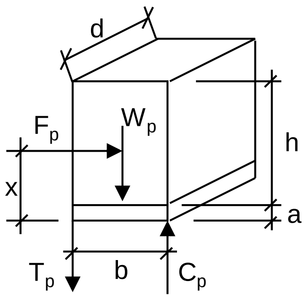
$F_{p(min)}$ = $0.3S_{DS}I_pW_p$ = 0.475 Wp (13.3-3)

F_p = 0.632 Wp = MAX(MIN(F_p, $F_{p(max)}$), $F_{p(min)}$)

Fp = 0.632 Wp

Fp = 0.101 kips

OVERTURNING FORCES: Method: LRFD



Distance between anchors, b = 20 in short side of MU

Distance between anchors, d = 20 in long side of MU

Height, h = 10 in

Frame base, a = 0 in **Need Cutsheets**

C.G., x = a + h/2 = 5 in

Mo = F_px = 0.5 kip-in overturning moment

Mrb(T) = Wp(0.9-0.2SDS)b/2 = 1.1 kip-in resisting moment in 'b' direction

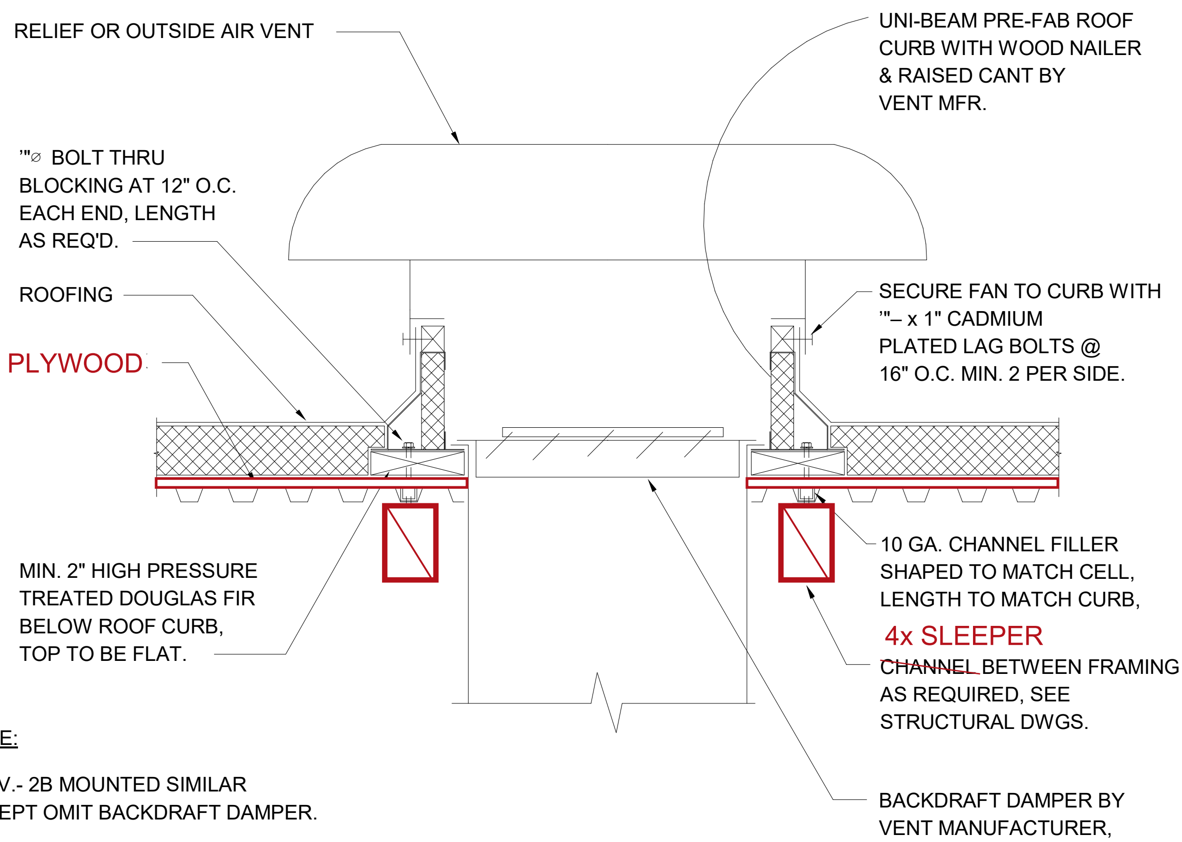
Mrb(C) = Wp(1.2+0.2SDS)b/2 = 2.3 kip-in resisting moment in 'b' direction

Tp(MAX) = (Mo-Mb(T))/b = **-0.031 kips** ***NO UPLIFT***

Cp(MAX) = (Mo+Mrb(C))/b = **0.137 kips**

GRAVITY ROOF VENTILATOR SCHEDULE

UNIT	LOCATION	"COOK" MODEL NO.	TYPE	TOTAL THROAT SIZE	CFM	S.P. (IN. W.G.)	OPER. WT. (LBS.)	MOUNTING DETAIL	
GRV 1&2	ROOF	PR	INTAKE OR OUTLET	16"	1200	0.04	160	3 M5.1	
GRV 3-6	ROOF	PR	INTAKE OR OUTLET	8"	600	0.3	30	3 M5.1	



NOTE:

O.A.V.- 2B MOUNTED SIMILAR EXCEPT OMIT BACKDRAFT DAMPER.

RELIEF/INTAKE VENT MOUNTING

SCALE : NONE

3
M5.1

CBC 2019, ASCE 7-16, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = 41-6000U (6/M5.2)

Unit Weight, W_p = 50.0 lbs I_p = 1.5   13.1.3

OSHPD Project = NO

Anchored to Concrete and using Ω_o for anchorage = NO   Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = NO

Fp FORCE:

Table 13.5-1 or Table 13.6-1: a_p = 2.5 R_p = 6.0 Ω_o = 2.0

z = 15 ft   h = 21.5 ft

$$F_p = \frac{0.4 a_p S_{DS} W_p (1+2z/h)}{(R_p/I_p)} = 0.632\ Wp \qquad (13.3\text{-}1)$$

$$F_{p(max)} = 1.6 S_{DS} I_p W_p = 2.531\ Wp \qquad (13.3\text{-}2)$$

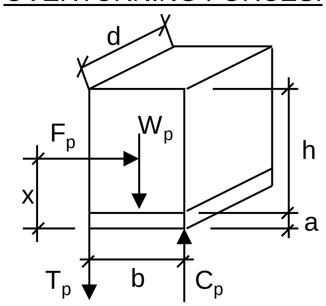
$$F_{p(min)} = 0.3 S_{DS} I_p W_p = 0.475\ Wp \qquad (13.3\text{-}3)$$

F_p = 0.632 Wp   = MAX(MIN(F_p, $F_{p(max)}$), $F_{p(min)}$)

Fp = 0.632 Wp

Fp = 0.032 kips

OVERTURNING FORCES: Method: LRFD



Distance between anchors, b = 20 in   short side of MU

Distance between anchors, d = 20 in   long side of MU

Height, h = 10 in

Frame base, a = 0 in   **Need Cutsheets**

C.G., x = a + h/2 = 5 in

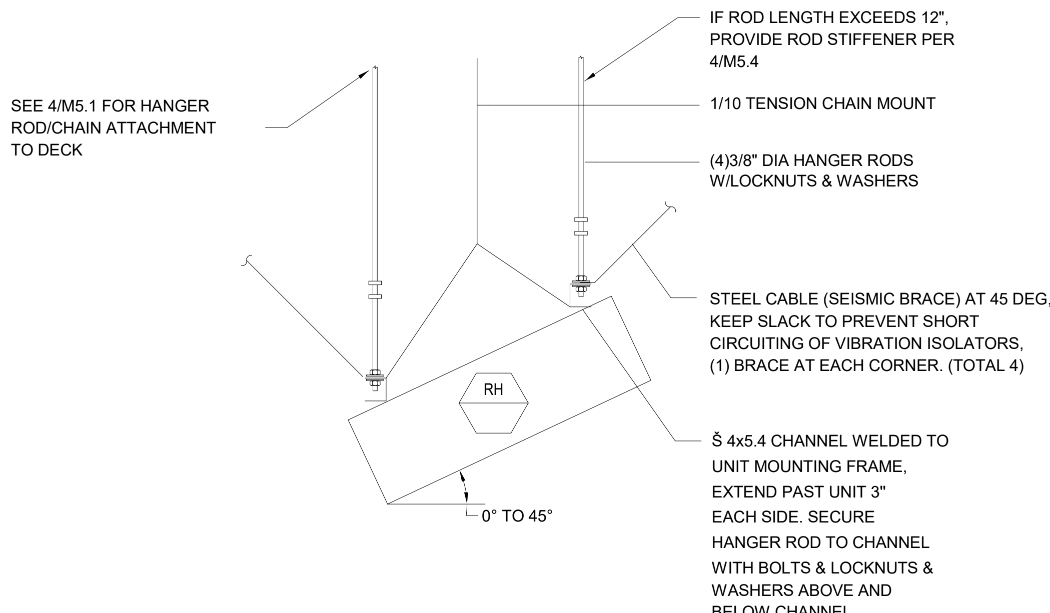
Mo = F_px = 0.1 kip-in   overturning moment

Mrb(T) = Wp(0.9-0.2SDS)b/2 = 0.3 kip-in   resisting moment in 'b' direction

Mrb(C) = Wp(1.2+0.2SDS)b/2 = 0.7 kip-in   resisting moment in 'b' direction

Tp(MAX) = (Mo-Mb(T))/b = **-0.010 kips** ***NO UPLIFT***

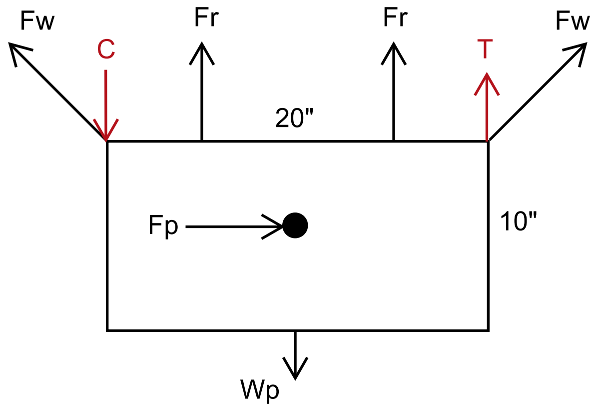
Cp(MAX) = (Mo+Mrb(C))/b = **0.043 kips**



RADIANT HEATER DETAIL

SCALE : NONE

6
M5.2



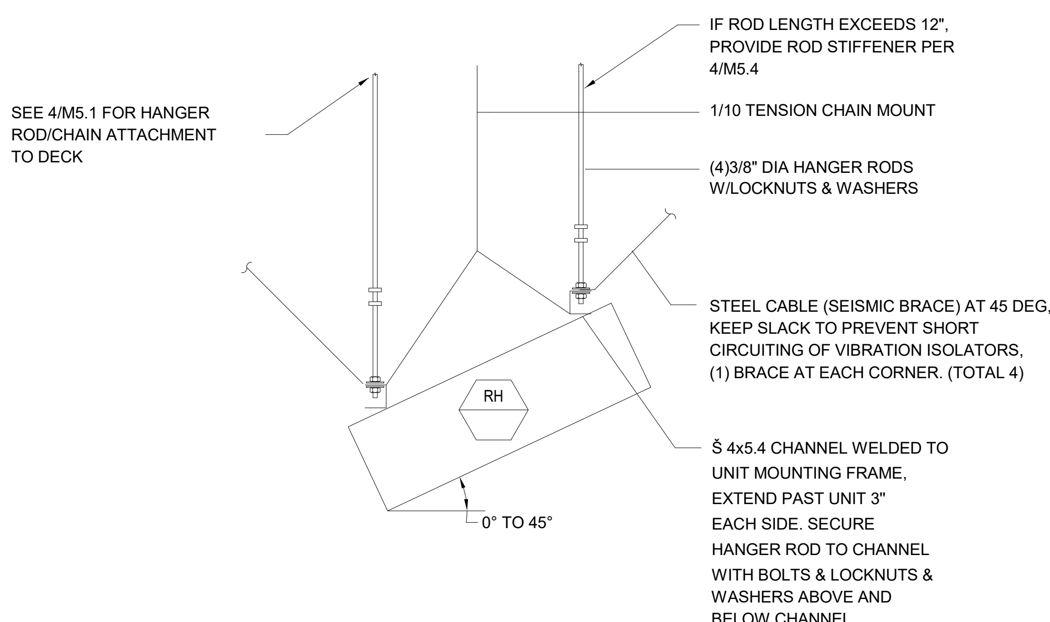
Unit Weight, W_p = 50.0 lbs

Fp = 0.032 kips <= Rod ok by observation

Tp(MAX) = (Mo-Mb(T))/b = **-0.010 kips** ***NO UPLIFT***

Cp(MAX) = (Mo+Mrb(C))/b = **0.043 kips**

$Fw = \frac{Fp * \sqrt{2}}{2}$ = 30# <= Wire ok by observation



RADIANT HEATER DETAIL

SCALE : NONE

6
M5.2

CBC 2019, ASCE 7-16, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = GC-148 (7/M5.2)

Unit Weight, W_p = 20.0 lbs I_p = 1.5   13.1.3

OSHPD Project = NO

Anchored to Concrete and using Ω_o for anchorage = NO   Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = NO

Fp FORCE:

Table 13.5-1 or Table 13.6-1: a_p = 2.5 R_p = 6.0 Ω_o = 2.0

z = 15 ft   h = 21.5 ft

$$F_p = \frac{0.4 a_p S_{DS} W_p (1+2z/h)}{(R_p/I_p)} \quad 0.632\ Wp \qquad (13.3\text{-}1)$$

$F_{p(max)}$ = $1.6 S_{DS} I_p W_p$ = 2.531 Wp   (13.3-2)

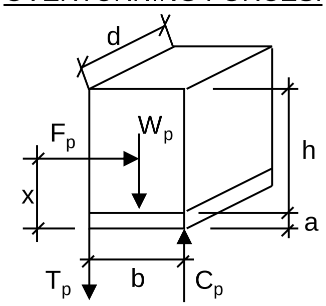
$F_{p(min)}$ = $0.3 S_{DS} I_p W_p$ = 0.475 Wp   (13.3-3)

F_p = 0.632 Wp   = MAX(MIN(F_p, $F_{p(max)}$), $F_{p(min)}$)

Fp = 0.632 Wp

Fp = 0.013 kips

OVERTURNING FORCES: Method: LRFD



Distance between anchors, b = 20 in   short side of MU

Distance between anchors, d = 20 in   long side of MU

Height, h = 10 in

Frame base, a = 0 in   **Need Cutsheets**

C.G., x = a + h/2 = 5 in

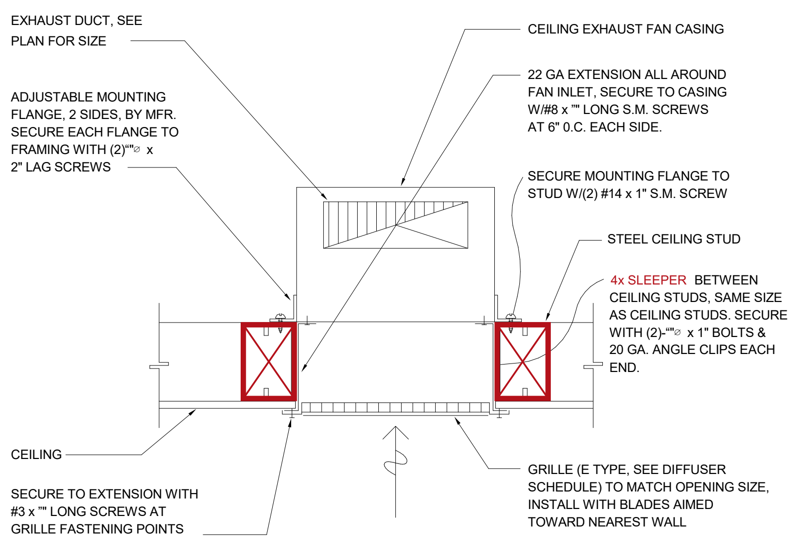
Mo = $F_p x$ = 0.1 kip-in   overturning moment

Mrb(T) = Wp(0.9-0.2SDS)b/2 = 0.1 kip-in   resisting moment in 'b' direction

Mrb(C) = Wp(1.2+0.2SDS)b/2 = 0.3 kip-in   resisting moment in 'b' direction

Tp(MAX) = (Mo-Mb(T))/b = **-0.004 kips** ***NO UPLIFT***

Cp(MAX) = (Mo+Mrb(C))/b = **0.017 kips**



CEILING EXHAUST FAN DETAIL   7 / M5.2

SCALE : NONE

CBC 2019, ASCE 7-16, §13.3 Seismic Demands on Nonstructural Components

EQUIPMENT:

Description = 100SQN17DEC (8/M5.2)

Unit Weight, W_p = 70.0 lbs — I_p = 1.5 — 13.1.3

OSHPD Project = NO

Anchored to Concrete and using Ω_o for anchorage = NO — Table 13.5-1 or Table 13.6-1 footnote C

Vibration Isolators with air gap > 0.25" = YES — Fp increased x2 (see Table 13.6-1 footnote b to see if Fp can be reduced)

Fp FORCE:

Table 13.5-1 or Table 13.6-1: a_p = 2.5 — R_p = 6.0 — Ω_o = 2.0

z = 15 ft — h = 21.5 ft

$$F_p = \frac{0.4 a_p S_{DS} W_p (1+2z/h)}{(R_p/I_p)} = 0.632\ Wp \quad (13.3\text{-}1)$$

$$F_{p(max)} = 1.6 S_{DS} I_p W_p = 2.531\ Wp \quad (13.3\text{-}2)$$

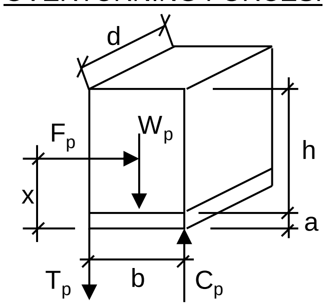
$$F_{p(min)} = 0.3 S_{DS} I_p W_p = 0.475\ Wp \quad (13.3\text{-}3)$$

F_p = 0.632 Wp = MAX(MIN(F_p, $F_{p(max)}$), $F_{p(min)}$)

Fp = 0.632 Wp

Fp = 0.088 kips

OVERTURNING FORCES: Method: LRFD



Distance between anchors, b = 20 in — short side of MU

Distance between anchors, d = 20 in — long side of MU

Height, h = 10 in

Frame base, a = 0 in — **Need Cutsheets**

C.G., x = a + h/2 = 5 in

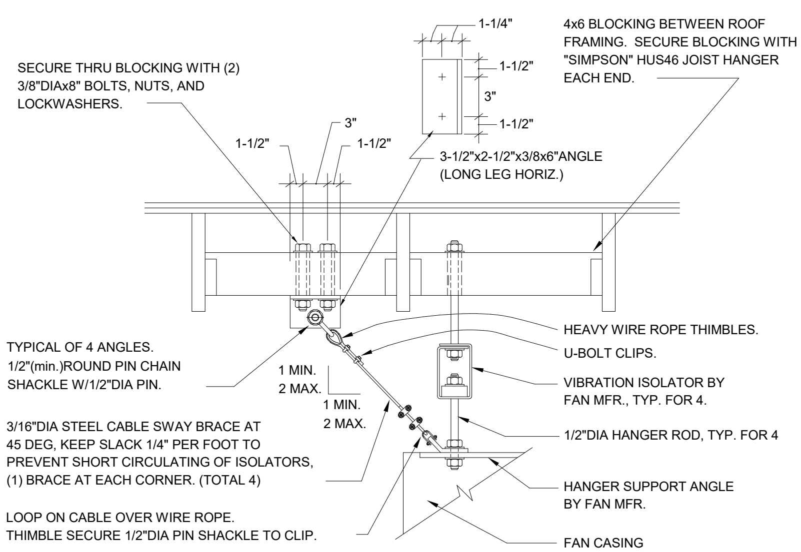
Mo = F_px = 0.4 kip-in — overturning moment

Mrb(T) = Wp(0.9-0.2SDS)b/2 = 0.5 kip-in — resisting moment in 'b' direction

Mrb(C) = Wp(1.2+0.2SDS)b/2 = 1.0 kip-in — resisting moment in 'b' direction

Tp(MAX) = (Mo-Mb(T))/b = **-0.003 kips** ***NO UPLIFT***

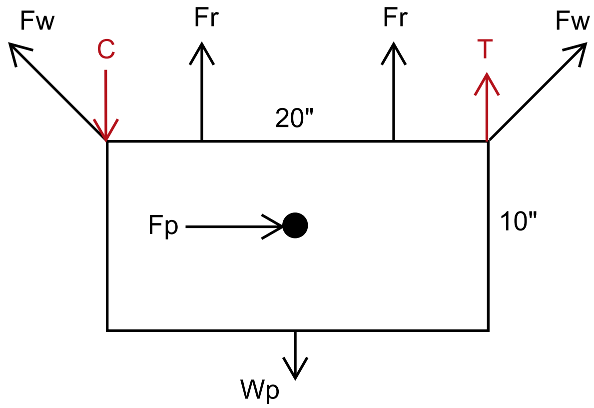
Cp(MAX) = (Mo+Mrb(C))/b = **0.070 kips**



EXHAUST FAN MOUNTING DETAIL

SCALE : NONE

8
M5.2



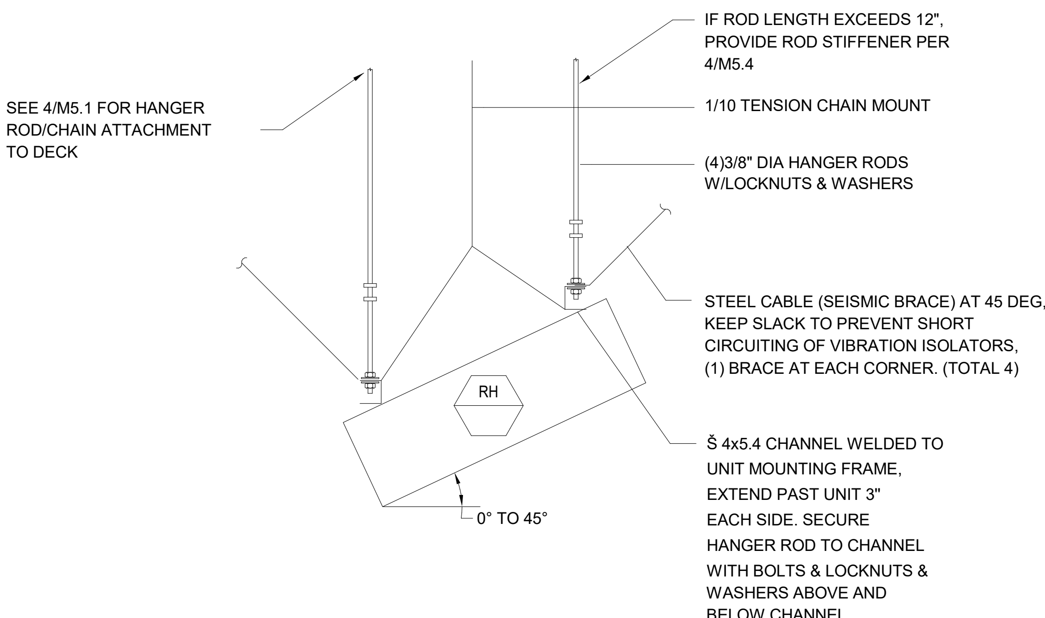
Unit Weight, W_p = 70.0 lbs

Fp = 0.088 kips <= Rod ok by observation

Tp(MAX) = (Mo-Mb(T))/b = **-0.003 kips** ***NO UPLIFT***

Cp(MAX) = (Mo+Mrb(C))/b = **0.070 kips**

$Fw = \frac{Fp * \sqrt{2}}{2}$ = 62# <= Wire ok by observation



RADIANT HEATER DETAIL

SCALE : NONE

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M5.2