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CALCULATION SUMMARY

Design Criteria

Design Code	2015 IBC	Wind Design	115 mph 3 second gust
Occupancy Type	U	Exposure Category	B
Construction Type	V-B	Seismic Classification	D
Risk Category	II	Seismic Design Category	D
Importance Factor	1.00	Soil Bearing Capacity	3000 psf
Building Height	13 ft	Roof Dead Load:	15 psf
Stories	1	Ground Snow Load:	52 psf
		Floor Dead Load:	12 psf
		Floor Live Load:	50 psf

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19-JUL-2018
STATE OF UTAH

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SNOW LOAD CALCULATION (2012 IBC 1608.1 & ASCE-7 CH. 7

Adapted From Utah Code 1608.1.2

$$p_g = \sqrt{{p_o}^2 + S^2(A - A_0)^2}$$

		Po	43 psf
County	Davis	S	63 psf
Elevation	4958 ft	Ao	4.5 psf
Terrain Category	B	$p_g =$	52 psf

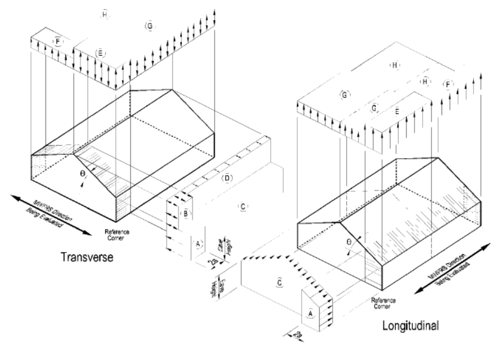
$$p_f = 0.7 C_e C_t I_s p_g$$

Flat Roof Snow load (pf)	43 psf	Sloped Snow Load (ps)	43 psf
Roof Pitch	6 on 12		
Roof Covering	Metal		
Thermal Factor (Ct)	1.0		
Exposure Factor (Ce)	1.2		
Risk Category	II		
Importance Factor (Is)	1.0		
Roof Slope Factor (Cs)	1.0		

Unbalanced Roof Snow Load:	N/A
Drifts on Lower Roofs:	N/A
Roof Projections and Parapets:	N/A
Sliding Snow:	N/A
Rain on Snow Surcharge:	N/A
Ponding Instability	N/A

WIND LOAD DESIGN WITH ASCE 7-10 MWFRS ENVELOPE PROCEDURE

Basic Wind Speed	115 mph	(IBC Fig 1609A)
Wind Exposure	B	
Risk Category	II	
Importance Factor	1.00	



Mean Roof Height

Roof Pitch	6 :12
Roof Angle	27 deg
Building Length	20 ft
Building Width	12 ft
Wall Height	8 ft
Ridge Height (Above Wall)	3.000 ft

Mean Roof Height = 10.00 ft

$\lambda = Height\ \&\ Exposure\ Adjustment\ Coefficient$ $\lambda =$ 1.21

$K_{zt} = Topographic\ Factor$ $K_{zt} =$ 1.00

$P_{g30} = Simplified\ Design\ Wind\ Pressure$

$P_s = \lambda K_{zt} I P_{g30}$

	Horizontal Pressures				Vertical Pressures			
	A	B	C	D	E	F	G	H
$P_{g30} =$	26.3	4.2	19.1	4.3	-11.7	-15.9	-8.5	-12.8
$P_s =$	31.8	5.1	23.1	5.2	-14.2	-19.2	-7.4	-11.1
$Trans.$	764 lb	46 lb	3143 lb	265 lb	-255 lb	-346 lb	-754 lb	-1136 lb
$Long.$	835 lb	N/A	2028 lb	N/A	-425 lb	-577 lb	-666 lb	-1002 lb

Width 2a

0.10(Smallest L)	1.2 ft	
0.40(h)	3.2 ft	
Minimum =		1.2 ft
But not less than:		
.04(Smallest L)	0.48 ft	
or 3 feet	3 ft	3 ft

Width 2a **3 ft**

End Wall Lateral Forces at Roof Boundary	**2109 lb**	13.18 psf
Side Wall Lateral Forces at Roof Boundary	**1432 lb**	14.91 psf
End Wall Vertical Forces at Roof Boundary	**-1246 lb**	-5.19 psf
Side Wall Vertical Forces at Roof Boundary	**-1335 lb**	-5.56 psf

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SEISMIC LOADS ASCE 7-10 EQUIVALENT LATERAL FORCE PROCEDURE

Site Class	D	(ASCE 7-10 Table 20.3-1)
Latitude	41.250271	
Longitude	-111.83903	
Risk Category	I	

$S_{DS} =$	0.702 g	(From USGS Spectral Response Calculator)
$R =$	6.50	(ASCE 7-10 Table 12.2-1)
$I_s =$	1.00	

Design Category D

$S_{DS} = design\ spectral\ response\ acceleration$
$R = Response\ Modification\ Factor$
$I_s = Seismic\ Importance\ Factor$

Building Weight

Roof

Dead Load	15 psf
Snow Load	43 psf
Roof Area	286 sf
$W =$	16588 ft

$V = C_s W$ $\quad W = D_L + S_L$

$C_s = \dfrac{S_{DS}}{\left(\frac{R}{I}\right)}$ $\quad V = S_{Ds}\dfrac{WI}{R}$

$V = Seismic\ Base\ Shear$
$C_s = Seismic\ response\ coefficient$
$W = Effective\ Seismic\ Weight$

$\boldsymbol{V} =$ **1792 lb**

Seismic Load Effect

$\rho =$	1.3	
$\boldsymbol{E_h} =$	**1792 lb**	6.26 psf
$\boldsymbol{E_v} =$	**602 lb**	2.11 psf
$\boldsymbol{E} =$	**2394 lb**	8.37 psf

$E = E_h + E_v$
$E_h = \rho Q_E$
$E_v = 0.2 S_{DS} D$
$\rho = redundancy\ factor$

$Q_E = effects\ of\ horizontal\ seismic\ forces\ from\ V$

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MASONRY LINTEL DESIGN - ASD

RB-1 Masonry Lintel

Lintel Size Data

Wall Width	$b =$	7.625 in	$E_m =$	1350 ksi			
Lintel Depth	$h =$	16 in	$E_s =$	29000 ksi			
Span	$L =$	3 ft					
	$f'_m =$	1500 psi					
	$f_y =$	60000 psi					

Loading

$w_{DL} =$	24	Lb/ft
$w_{LL} =$	86	Lb/ft
$w_{u,max} =$	0.18	k/ft

Moment

$$M_u = \frac{wL^2}{8} =$$ 0.2025 k-ft

Effective Depth $$d = h - \frac{b}{2} =$$ 12.19 in^2

Transformation Factor

$$n = \frac{E_s}{E_m} =$$ 21.48

Steel Selection

Try 2 # 5 bars $A_s =$ 0.61 in^2

Reinforcement Ratio

$$\rho = \frac{A_s}{bd} =$$ 0.007

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n =$$ 0.41

$$j = 1 - \frac{k}{3} =$$ 0.86

Tensile Strength for Steel

$$f_s = \frac{M_u}{A_s jd} =$$ 0.38 ksi $F_{s,allowed} =$ 24000 ksi **OK**

Masonry Stress

$$f_c = \frac{2M_u}{bd^2jk} =$$ 12.14 psi $$F_c = \frac{f'_m}{3} =$$ 500 psi **OK**

Shear Stress

$$V = \frac{wL}{2} =$$ 0.27 k $$f_v = \frac{V}{bd} =$$ 2.91 psi

$$F_{v,allowed} = \sqrt{f'_m} =$$ 38.73 psi **OK**

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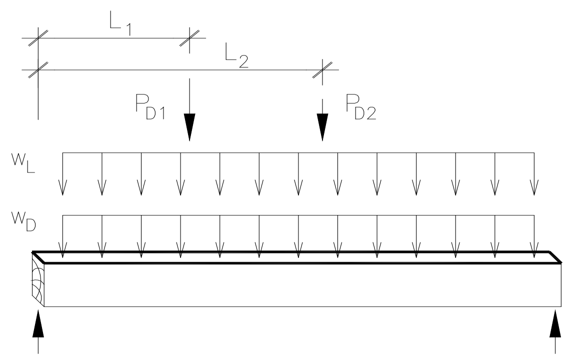
WOOD BEAM DESIGN BASED ON NDS 2012

RB-2

INPUT DATA & DESIGN SUMMARY

Member Size		(2)2 x 8
Member Span	$L =$	4.5 ft
Uniform Dead Load	$w_D =$	36 lb/ft
Uniform Live Load	$w_L =$	129 lb/ft
Concentrated Dead Load	$P_{D1} =$	0 lb
(0 for no concentrated load)	$L_1 =$	0 ft
	$P_{D2} =$	0 lb
	$L_2 =$	0 ft
	$\Delta_{max,LL} = l/$	120
	$\Delta_{k_{cr}D+L} = l/$	240

No. 2, Douglas Fir-Larch



Camber => 0.00 inch

THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

$w_{Self\ Wt} =$ 4.71 lb/ft $\quad R_{left} =$ 0.38 kips $\quad R_{Right} =$ 0.38 kips

$V_{max} =$ 0.28 kips, at 7.25 inch from left end

$M_{max} =$ 0.43 ft-kips, at 2.25 ft from left end

SECTION PROPERTIES& ALLOWABLE STRESSES

$b =$	3.00 in	$E'_{min} =$	N/A	$E = E_x =$	1300 ksi	$F = \frac{F_{bE}}{F_b^*} =$	N/A
$d =$	7.25 in	$F_{bE} =$	N/A	$F_b =$	875 psi		
$A =$	21.8 in^2	$I =$	95 in^4	$F_v =$	170 psi	$F_b' =$	1138 psi
$S_x =$	26.3 in^3	$R_B =$	N/A	$E' =$	1300 ksi	$F_v' =$	170 psi
$I_E =$	N/A			$F_b^* =$	N/A		

C_D	C_M	C_t	C_i	C_L	C_F	C_v	C_r
1.00	1.00	1.00	1.00	1.00	1.30	1.00	1.00

BENDING AND SHEAR CAPACITIES

$f_b = \frac{M_{max}}{S_x} =$ 196 psi $<$ $F_b =$ 1138 psi **OK**

$f_v' = \frac{1.5V_{max}}{A} =$ 19 psi $<$ $F_v' =$ 170 psi **OK**

CHECK DEFLECTIONS

$\Delta_{,LL} =$ 0.01 in, at 2.250 ft from left end, $<$ $\Delta_{max,LL} = l/$ 120 **OK**

$\Delta_{k_{cr}D+L} =$ 0.01 in, at 2.250 ft from left end $<$ $\Delta_{k_{cr}D+L} = l/$ 240 **OK**

$Where\ K_{cr} =$ 1.50 , (NDS 3.5.2)

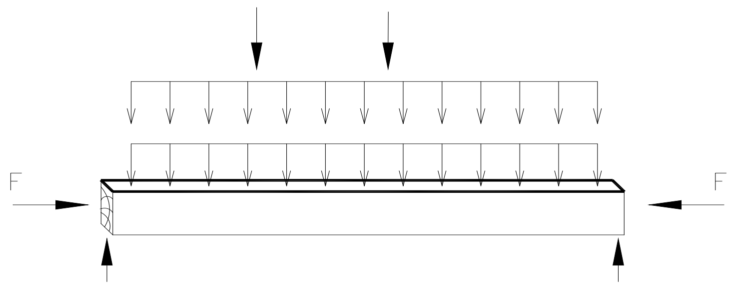
DETERMINE CAMBER AT 1.5 (DEAD + SELF WEIGHT)

$\Delta_{1.5D,max} =$ 0.00 in, at 2.250 ft from left end

CHECK THE BEAM CAPACITY WITH AXIAL LOAD

AXIAL LOAD

$F =$ 0 kips



THE ALLOWABLE COMPRESSIVE STRESS IS

$F_c' = F_c C_D C_p C_f =$ 618 psi

$Where\ F_c =$ 600 psi

$C_D =$ 1.00

$C_F =$ 1.05 (Lumber only)

$$C_p = \frac{1+F}{2c} - \sqrt{\left(\frac{1+F}{2c}\right)^2 - \frac{F}{c}} =$$ 0.981

$F_c^* = F_c C_D C_F =$ 630 psi

$L_e = K_e L = 1.0L =$ 54 in

$d =$ 7.25 in

$SF = Slenderness\ Ratio =$ 7.4 < 50 **OK (NDS Sec. 3.7.1.4)**

$$F_{cE} = \frac{0.822 E_{min}}{SF^2} =$$ 6964 psi

$E_{min} =$ 470 ksi

$$F = \frac{F_{cE}}{F_c'} =$$ 11.054 $c =$ 0.8

THE ACTUAL COMPRESSIVE STRESS IS

$$f_c = \frac{F}{A} =$$ 0 psi < $F_c' =$ 618 psi **OK**

THE ALLOWABLE FLEXURAL STRESS IS

$F_b' =$ 1138 psi

THE ACTUAL FLEXURAL STRESS IS

$$f_b = \frac{M + F_e}{S_x} =$$ 196 psi < F_b' **OK**

COMBINED STRESS [NDS Sec. 3.9.2]

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_b}{F_b'\left(1 - \frac{f_c}{F_{cE}}\right)} =$$ 0.172 < 1 **OK**

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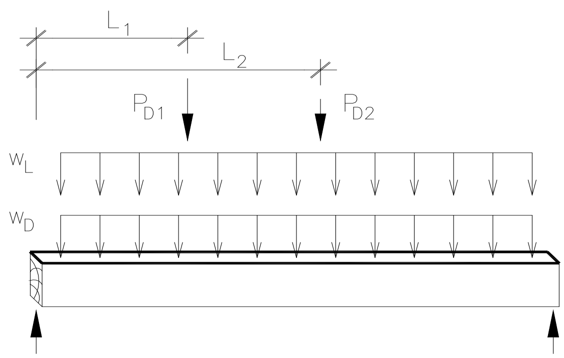
WOOD BEAM DESIGN BASED ON NDS 2012

L-1

INPUT DATA & DESIGN SUMMARY

Member Size 2 x 8 **No. 2, Douglas Fir-Larch**

Member Span	$L =$	2 ft
Uniform Dead Load	$w_D =$	24 lb/ft
Uniform Live Load	$w_L =$	86 lb/ft
Concentrated Dead Load	$P_{D1} =$	0 lb
(0 for no concentrated load)	$L_1 =$	0 ft
	$P_{D2} =$	0 lb
	$L_2 =$	0 ft
	$\Delta_{max,LL} = l/$	120
	$\Delta_{k_{cr}D+L} = l/$	240



Camber => 0.00 inch

THE BEAM DESIGN IS ADEQUATE.

ANALYSIS

$w_{Self\ Wt} =$ 2.36 lb/ft $R_{left} =$ 0.11 kips $R_{Right} =$ 0.11 kips

$V_{max} =$ 0.04 kips, at 7.25 inch from left end

$M_{max} =$ 0.06 ft-kips, at 1.00 ft from left end

SECTION PROPERTIES& ALLOWABLE STRESSES

$b =$	1.50 in	$E'_{min} =$	N/A	$E = E_x =$	1600 ksi	$F = \frac{F_{bE}}{F_b^*} =$	N/A
$d =$	7.25 in	$F_{bE} =$	N/A	$F_b =$	900 psi		
$A =$	10.9 in^2	$I =$	48 in^4	$F_v =$	180 psi	$F_b' =$	1080 psi
$S_x =$	13.1 in^3	$R_B =$	N/A	$E' =$	1600 ksi	$F_v' =$	180 psi
$I_E =$	N/A			$F_b^* =$	N/A		

C_D	C_M	C_t	C_i	C_L	C_F	C_v	C_r
1.00	1.00	1.00	1.00	1.00	1.20	1.00	1.00

BENDING AND SHEAR CAPACITIES

$f_b = \frac{M_{max}}{S_x} =$ 51 psi < $F_b =$ 1080 psi **OK**

$f_v' = \frac{1.5V_{max}}{A} =$ 6 psi < $F_v' =$ 180 psi **OK**

CHECK DEFLECTIONS

$\Delta_{,LL} =$ 0.00 in, at 1.000 ft from left end, < $\Delta_{max,LL} = l/$ 120 **OK**

$\Delta_{k_{cr}D+L} =$ 0.00 in, at 1.000 ft from left end < $\Delta_{k_{cr}D+L} = l/$ 240 **OK**

$Where\ K_{cr} =$ 1.50 , (NDS 3.5.2)

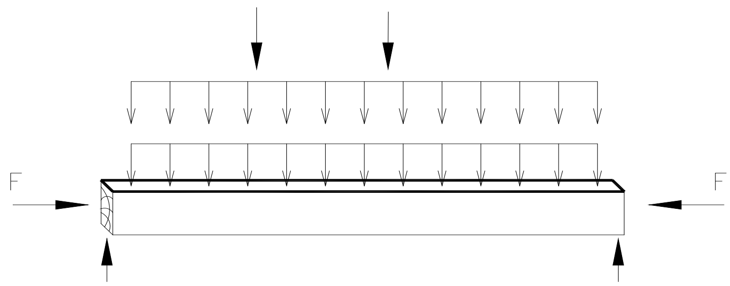
DETERMINE CAMBER AT 1.5 (DEAD + SELF WEIGHT)

$\Delta_{1.5D,max} =$ 0.00 in, at 1.000 ft from left end

CHECK THE BEAM CAPACITY WITH AXIAL LOAD

AXIAL LOAD

$F =$ 0 kips



THE ALLOWABLE COMPRESSIVE STRESS IS

$F_c{}' = F_c C_D C_p C_f =$ 1408 psi

$Where\ F_c =$ 1350 psi

$C_D =$ 1.00

$C_F =$ 1.05 (Lumber only)

$$C_p = \frac{1+F}{2c} - \sqrt{\left(\frac{1+F}{2c}\right)^2 - \frac{F}{c}} =$$ 0.993

$F_c{}^* = F_c C_D C_F =$ 1418 psi

$L_e = K_e L = 1.0L =$ 24 in

$d =$ 7.25 in

$SF = Slenderness\ Ratio =$ 3.3 < 50 **OK (NDS Sec. 3.7.1.4)**

$$F_{cE} = \frac{0.822 E_{min}}{SF^2} =$$ 43506 psi

$E_{min} =$ 580 ksi

$$F = \frac{F_{cE}}{F_c{}'} =$$ 30.692 $c =$ 0.8

THE ACTUAL COMPRESSIVE STRESS IS

$$f_c = \frac{F}{A} =$$ 0 psi < $F_c{}' =$ 1408 psi **OK**

THE ALLOWABLE FLEXURAL STRESS IS

$F_b{}' =$ 1080 psi

THE ACTUAL FLEXURAL STRESS IS

$$f_b = \frac{M + F_e}{S_x} =$$ 51 psi < $F_b{}'$ **OK**

COMBINED STRESS [NDS Sec. 3.9.2]

$$\left(\frac{f_c}{F_c{}'}\right)^2 + \frac{f_b}{F_b{}'\left(1 - \frac{f_c}{F_{cE}}\right)} =$$ 0.048 < 1 **OK**

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WOOD COLUMN DESIGN BASED ON NDS 2012

C-1

Length	$L =$	8 ft		Type	#2 DF
Width	$w =$	3.5 in		$F_b =$	900 psi
Depth	$d =$	3.5 in		$F_c =$	1350 psi
				$E =$	1600000 psi

Compression Calculations

Axial Load $P =$ 380 lb

Slenderness Ratio $\frac{L_e}{d} =$ 27.43

Euler Buckling Coefficient $F_{cE} = \frac{K_{cE} E}{\left(\frac{L_e}{d}\right)^2} =$ 638 psi

$K_{cE} =$ 0.3
$c =$ 0.8

$\Phi = \frac{F_{cE}}{F_c} =$ 0.47

Stability Factor $C_p = \frac{1+\Phi}{2c} \sqrt{\left(\frac{1+\Phi}{2c}\right)^2 - \frac{\Phi}{c}} =$ 0.41

$P_{allow} = F_c C_p A =$ 6848 lb **OK**

Length Calculations

$C_{p1} = \frac{P}{F_c w d} =$ 0.02

$\Phi_1 =$ 0.36

$C_{p2} = \frac{1+\Phi_1}{2c} \sqrt{\left(\frac{1+\Phi_1}{2cc}\right)^2 - \frac{\Phi_1}{c}} =$ 0.33

Max. Length

$L_{max} = \sqrt{\frac{K_{cE} E}{\Phi_1 F_c}} d =$ 9.17 ft **OK**

WOOD COLUMN DESIGN BASED ON NDS 2012

C-2

Length	$L =$	8 ft	Type		#2 DF
Width	$w =$	3 in	$F_b =$		900 psi
Depth	$d =$	5.5 in	$F_c =$		1350 psi
			$E =$		1600000 psi

Compression Calculations

Axial Load $P =$ 380 lb

Slenderness Ratio $\frac{L_e}{d} =$ 17.45

Euler Buckling Coefficient $F_{cE} = \frac{K_{cE}E}{\left(\frac{L_e}{d}\right)^2} =$ 1576 psi

$K_{cE} =$ 0.3

$c =$ 0.8

$\Phi = \frac{F_{cE}}{F_c} =$ 1.17

Stability Factor $C_p = \frac{1+\Phi}{2c} \sqrt{\left(\frac{1+\Phi}{2c}\right)^2 - \frac{\Phi}{c}} =$ 0.74

$P_{allow} = F_c C_p A =$ 16518 lb **OK**

Length Calculations

$C_{p1} = \frac{P}{F_c wd} =$ 0.02

$\Phi_1 =$ 0.36

$C_{p2} = \frac{1+\Phi_1}{2c} \sqrt{\left(\frac{1+\Phi_1}{2cc}\right)^2 - \frac{\Phi_1}{c}} =$ 0.33

Max. Length

$L_{max} = \sqrt{\frac{K_{cE}E}{\Phi_1 F_c}}\, d =$ 14.40 ft **OK**

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CONCRETE SHORT COLUMN DESIGN WITH ACI 318 Appendix C

C-3

Column Size Data

$$\phi P_n = \phi 0.80\left(0.85 f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right)$$

$\phi P_n = P_{u,max} =$ 0.60 kips

$\phi =$ 0.65 (for tied columns)

$f'_c =$ 4000 psi

$f_y =$ 60000 psi

Loading

$P_{u,max}$	0.6	kips
$P_{u,min}$	-0.5	kips
Column Height	6	ft

Steel must be between 1% and 8% of the concrete area:

$A_{st} = 0.01 A_g$<=$.08 A_g$

			Width		Depth	
$A_{g,req} =$	0.25 in^2	Use	12 in	x	12 in	
				$A_g =$	144 in^2	OK
$A_{st,req} =$	1.44	<=	11.52			

Longitudinal Steel

Use	6	# 5 bars	$A_{st} =$	1.84 in^2	OK
					OK
$\phi P_n =$	**308.77 k**	**>**		**0.60 kips**	**OK**

Tie Design

Use # 3 bars

Min. size is #3 for longitudinal bars #10 or smaller.
Min. size is #4 for longitudinal bars #11 or larger.

Spacing is the least of:

$S = 48 \times d_{b,ties} =$ 18 in

$S = 16 \times d_{b,longitudinal} =$ 10 in

Use # 3 ties at 10 in o.c.

Clear Space from Edges

$C_{sp} = \dfrac{b-5}{2} - d_b =$ 2.88 **>** 1.50 **OK**

Steel Percentage

$*.01 < \rho < 0.08$

$\rho = \dfrac{A_s}{A_g} =$ 0.013 in^2 0.01 **<** 0.013 **OK**

0.013 **<** 0.08 **OK**

Anchor Bolt Embedment

Bolt Diameter	$d_b =$	0.75 in	$A_s =$	0.44 in^2
	$d_h =$	1.3125	$A_s f_y =$	26.51 k
	$L_d =$	9.00 in		

$$P_d > A_s f_y$$

$$A_{cp} = \pi[\left(L_d + \frac{d_h}{2}\right)^2 - \frac{d_h}{2}^2]$$

$P_d = \phi 4\sqrt{f'_c} A_{cp} =$ 47.95 k **47.95 k** **>** **26.51 k** **OK**

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PLAIN CONCRETE WALL FOOTING DESIGN - ACI 318

F-1

Footing Size Data			**Loading**		
Footing Thickness	12	in	$w_{U,MAX} =$	0.52	Kips/ft
d =	8.5	in	$w_{U,MIN} =$	-0.02	Kips/ft
Footing Weight	145	psf	Wall Height	5.5	ft
Footing Depth below Grade	1.5	ft	Wall Width	8	in
Backfill Density	120	pcf	Wall Weight	0.550	Kips
Weight of Backfill on Footing	60.00	psf			
Allowable Bearing Capacity	1500	psf			
Effective Soil Pressure	1295.00	psf			

Required Footing Size

$$A_{req} = \frac{D_L + L_L}{q_e} =$$ **0.83** ft **= 10 in**

$$A_{req,uplift} = \frac{-P_{u,min}}{q_a - q_e} =$$ **0.10** ft **= 1 in** **Use** **24 in** **wide footing**

OK

Bearing Pressure for Strength Design

$$q_u = \frac{P_{u,max}}{B_{footing}}$$ $\boldsymbol{q_u} =$ **260 plf**

Bending Strength for 12" Width of Footing

$$M_u = \left(\frac{B}{2}\right)\left(\frac{B}{4}\right)q_u$$ $M_u =$ 130 lb-ft

$\Phi =$ 0.55
$f'_c =$ 3000 psi

$$S = \frac{b_w d^2}{6}$$ $S =$ 144.5 in^3

$$\phi M_u = \phi 5\sqrt{f'_c}S$$ $\phi M_u =$ 21765.13 Lb-ft **21765.1** **>** **130** **OK**

Check Shear Strength at dw

$d_w =$ 6 in

$$V_u = \left(\frac{B}{2} - d_w\right)q_u$$ $V_u =$ 130 Lb

$$\phi V_u = \phi\left(\frac{4}{3}\right)\sqrt{f'_c}b_w d$$ $\phi V_u =$ 4086.722 Lb **4086.72** **>** **130** **OK**

Longitudinal Temperature and Shrinkage Steel

$A_{s,req} = \rho b_w d =$ 0.259 in^2

$\boldsymbol{\rho} =$ 0.0018 **Use** **2** **#4** **Longitudinal bars**

$A_{s,furn} =$ 0.4 in^2 **OK**

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ISOLATED FOOTING DESIGN WITH ACI 318 Appendix C

F-2

Footing Size Data			**Loading**		
Footing Thickness	12	in	$P_{u,max}$	0.6	kips
Distance to centroid of rebar	3.5	in	$P_{u,min}$	-0.5	kips
d =	8.5	in	Column Height	3	ft
Footing Weight	150	psf	Column Width	12	in
Footing Depth below Grade	5.5	ft	Column Depth	12	in
Backfill Density	130	pcf			
Weight of Backfill on Footing	585.00	psf			
Allowable Bearing Capacity	1500	psf			
Effective Soil Pressure	765.00	psf			

Required Footing Size

$$A_{req} = \frac{D_L + L_L}{q_e} =$$ **0.78** sq. ft. **= 11 in X 11 in**

$$A_{req,uplift} = \frac{-P_{u,min}}{q_a - q_e} =$$ **0.68** sq ft **= 10 in X 10 in**

Use 24 in square footing

OK

Bearing Pressure for Strength Design

$$q_u = \frac{Max\ Factored\ Load}{A_{footing}}$$

$\boldsymbol{q_u =}$ **150.00 psf**

Depth Required for Two-Way Punching Shear

$b_o = 4(a + d)$ $\quad b_o =$ 82 in $\quad V_{u2} = (A - (a + d)^2)q_u$ $\quad V_{u2} =$ 162 Lb

d = The largest of the following equations:

$$d_1 = \frac{V_{u2}}{\Phi(4)(\sqrt{f'c})b_o}$$

$$d_2 = \frac{V_{u2}}{\Phi\left(\frac{\alpha d}{b_o} + 2\right)(\sqrt{f'c})b_o}$$

$\alpha =$ 40

$\Phi =$ 0.85

$f'_c =$ 3000 psi

$d_1 =$ 0.01 in

$d_2 =$ 0.01 in

0.01 < 8.5 OK

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Depth Required for One Way Shear

$$V_{u1} = b_w\left(\frac{l}{2} - \frac{a}{2} - d\right)q_u \qquad V_{u1} = \quad 150 \text{ Lb}$$

$$d = \frac{V_{u1}}{\emptyset 2\sqrt{f'c}(b_w)} \qquad d = \quad 0.07 \text{ in}$$

0.07 < 8.5 OK

Reinforcement Selection

$$M_u = \left(\frac{l}{2} - \frac{a}{2}\right)b_w q_u\left(\frac{l}{4} - \frac{a}{4}\right)$$

$l_1 =$ 6 in $\qquad \Phi =$ 0.90

$M_u =$ 0.04 k-ft $\qquad f_y =$ 60000 psi

$$R_n = \frac{M_u}{\emptyset b d^2} \qquad R_n = \quad 0.29 \text{ psi}$$

$$\rho = \frac{0.85f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}}\right) \qquad \rho = \quad 5\text{E-}06$$

ρ_{min} = The larger of the following equations

$$\rho_{min} = \frac{200}{f_y} \qquad \rho_{min} = \quad 0.00333$$

$\boldsymbol{\rho}$ **<** $\boldsymbol{\rho_{min}}$

$$\rho_{min} = \frac{3\sqrt{f'_c}}{f_y} \qquad \rho_{min} = \quad 0.00274$$

Use $\boldsymbol{\rho =}$ **0.00333**

Area of Steel Required

$$A_{s,req} = \rho b d = \quad 0.68 \quad in^2$$

Use 2 # 4 bars each direction

$$A_{s,furn} = \quad 0.80 \quad in^2$$

OK

Development Length

K_{tr} =Transverse Reinforcement Index = 0 (ACI 12.2.4)

$$\frac{c + K_{tr}}{d_b} < 2.5$$

$c =$ 3.50 in $\qquad \alpha =$ 1.00 $\qquad \gamma =$ 0.80

$d_b =$ 0.50 in $\qquad \beta =$ 1.00 $\qquad \lambda =$ 1.00

$$\frac{c + K_{tr}}{d_b} = \quad \mathbf{7} \quad \mathbf{>} \quad \mathbf{2.5}$$

Use K of 2.5

$$\frac{l_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\frac{c + K_{tr}}{d_b}} = \quad 26.29 \text{ Bar Diameters}$$

$$\frac{l_d}{d_b}\frac{A_{s,reqd}}{A_{s,furn}} = \quad 22.35 \text{ Bar Diameters}$$

$\boldsymbol{l_d =}$ **11.17 in**

$$l_{d,available} = b_w - \frac{a}{2} - 3" = \quad \mathbf{15.00 \text{ in}}$$

11.17 < 15.00 OK

Development Length for Hooked Bars

$$l_{dh} = \frac{0.02\beta\lambda f_y}{\sqrt{f_c'}} d_b =$$ **11 in** **10.95** **<** **15.00** **N/A**

Lateral Loading

Factored Horiz. Load (V_u) 0.25 k at column base

$$V_n = \frac{V_u}{\Phi} =$$ 0.29 k

Area of steel Required

$$A_{s,req} = \frac{V_n}{f_y \mu}$$

$\mu = 0.6\lambda$ (footing not intentionally roughened)

μ =0.6

$A_{s,req} =$ 0.01 in^2

Use **1** **# 4** **bar hooked around anchor**

$A_{s,furn} =$ 0.20 in^2 **OK**

Check Development Length

$c =$ 3.50 in

$d_b =$ 0.50 in

$\frac{c + K_{tr}}{d_b} < 2.5$ **7.00** **>** **2.5**

Use K of 2.5

$$\frac{l_d}{d_b} = \frac{3}{40} \frac{f_y}{\sqrt{f'_c}} \frac{\alpha\beta\gamma\lambda}{\frac{c + K_{tr}}{d_b}}$$

$\frac{l_d}{d_b} =$ 26.29 Bar Diameters

$\frac{l_d}{d_b} \frac{A_{s,reqd}}{A_{s,furn}} =$ 1.07 Bar Diameters

$\boldsymbol{l_d} =$ **0.54 in**

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SHEAR WALL DESIGN (2012 IBC SECTION 2306)

Wood Structural Panel Shear Walls

Controlling Force		Wind
End Wall Force	F_e	2109 lb
Side Wall Force	F_s	1432 lb

$$W = Wall\ Weight$$
$$W_s = Weight\ for\ Shear = Wlh$$
$$V = Total\ Shear = \frac{F_e or F_s}{l_s}$$
$$M = Moment\ at\ Base = Vlh$$
$$F_{uplift} = Uplift\ Force = \frac{M}{l} - \frac{W_s}{2}$$

East Side

Wall H/W Ratio Must Be < 3.5

Total Shearwall Length $l_s =$ 20.00 ft $\quad W =$ 15 psf

Total Shear $V =$ 72 lb/ft

	l	h	W_s	M	F_{uplift}	Hold Down
W-1	20.00 ft	8.00 ft	2400 lb	11456 ft-lb	-627 lb	NONE

Use Masonry Wall with #4 vertical rebar at 32" o.c. and #4 horizontal rebar at 48" o.c.

West Side

Wall H/W Ratio Must Be < 3.5

Total Shearwall Length $l_s =$ 20.00 ft $\quad W =$ 100 psf

Total Shear $V =$ 72 lb/ft

	l	h	W_s	M	F_{uplift}	Hold Down
W-1	20.00 ft	8.00 ft	16000 lb	11456 ft-lb	-7427 lb	NONE

Concrete Shear Wall with #4 Rebar at 12" o.c. Vertically & Horizontally

North Side

Wall H/W Ratio Must Be < 3.5						
Total Shearwall Length	$l_s =$	12.00 ft		$W =$	15 psf	
Total Shear	$V =$	176 lb/ft				
	l	h	W_s	M	F_{uplift}	Hold Down
W-1	12.00 ft	8.00 ft	1440 lb	16872 ft-lb	686 lb	Reinforcing

Use Masonry Wall with #4 vertical rebar at 32" o.c. and #4 horizontal rebar at 48" o.c.

South Side

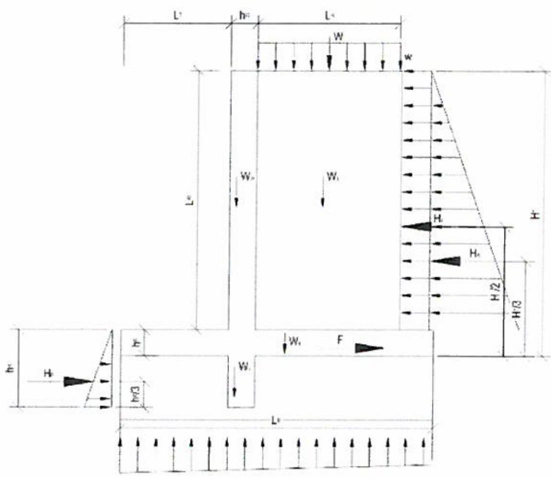
Wall H/W Ratio Must Be < 3.5						
Total Shearwall Length	$l_s =$	8.00 ft		$W =$	15 psf	
Total Shear	$V =$	264 lb/ft				
	l	h	W_s	M	F_{uplift}	Hold Down
W-1	8.00 ft	8.00 ft	960 lb	16872 ft-lb	1629 lb	Reinforcing

Use Masonry Wall with #4 vertical rebar at 32" o.c. and #4 horizontal rebar at 48" o.c.

West Building Wall

Wall Section width for Analysis (B)	1.00 ft	$f'_c =$	3000 psi
Soil Friction Angle (φ)	30 deg.	$f_y =$	60000 psi
Soil Bearing Capacity (q_{ALLOW})	1500 psf	$f'_{c\,stem} =$	4000 psi
Soil Height above heel (L_W)	7.5 ft		
Soil Height above toe (L_W)	2 ft		
Slope Over Heel (β)	30.00 deg.		
Surcharge Load (w)	0 plf		
Heel length (L_H)	4.00 ft		
Toe Length (L_T)	2.00 ft		
Wall Thickness (h_W)	0.67 ft		
Footing Thickness (h_B)	1.50 ft		
Key Length (h_K')	1.50 ft		
Weight of Soil (γ_S)	120 pcf		
Weight of Reinforced Concrete (γ_C)	150 pcf		
Frictional Resistance (μ)	0.35		
Passive Force Length (h_K)	5.00 ft		
Total Footing Length (L_B)	6.67 ft		
Total Wall Height (h_T)	9.00 ft		

Component	W (lb)	Wu (lb)
W_L	0	0
W_S	3600	4320
W_W	754	904.5
W_B	1501	1800.9
W_K	151	180.9
W_{SP}	480	576
Totals	6485	7782



$$K_A = \cos\beta \frac{\cos\beta - \sqrt{\cos\beta^2 - \cos\varphi^2}}{\cos\beta + \sqrt{\cos\beta^2 - \cos\varphi^2}} = \quad 0.87$$

$$p_A = \gamma_S K_A = \quad 104 \text{ psf}$$

$$\hbar_L = \frac{w}{\gamma_s} = \quad 0.00 \text{ ft}$$

$$H_A = \frac{p_A {h_T}^2}{2} = \quad 4209 \text{ plf}$$

$$H_L = p_A \hbar_L h_T = \quad 0 \text{ plf}$$

$$K_P = \frac{1 + \sin\varphi}{1 - \sin\varphi} = \quad 3.00$$

$$p_p = \gamma_S K_P = \quad 360 \text{ pcf}$$

$$H_P = \frac{p_P {h_K}^2}{2} = \quad 4500 \text{ lb}$$

Overturning Moment

$$M_o = H_L \frac{h_T}{2} + H_A \frac{h_T}{3} = \quad \textbf{12627 ft-lb}$$

Restoring Moment

$R_F = Floor\ resisting\ force =$ **6119 lb**

$h_F = Floor\ location\ above\ footing =$ **4.50 ft**

$$M_R = (W_S + W_L)\left(L_T + h_w + \frac{L_H}{2}\right) + W_W\left(L_T + \frac{h_W}{2}\right) + W_B\frac{L_B}{2} + R_F h_F = \quad \textbf{51113 ft-lb}$$

$FS = Factor\ of\ Safety =$ **4.0** **OK**

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Bearing Capacity

$$e' = \frac{M_R - M_O}{M_O} = \quad 3.05 \text{ ft} \qquad e = \frac{L_B}{2} - e' = \quad 0.29 \text{ ft}$$

$$\frac{L_B}{6} = \quad 1.11 \text{ ft}$$ **OK**

$$q = \frac{\sum W\left(1 \pm \frac{6e}{L_B}\right)}{BL_B} =$$

1223 psf At Toe	**<**	**1500 psf**	**OK**
721 psf At Heel	**>**	**0 psf**	**OK**

Sliding Resistance **Factor of Safety**

$$FS_S = \frac{F + H_P}{H_A + H_L} = 2270 \text{ lb} \qquad F = \mu \sum W = \quad \mathbf{1.61} \quad > \quad 1.5$$ **OK**

Footing Design

$$M_{ou} = 1.6M_o = \quad \mathbf{20203 \text{ ft-lb}}$$

$$M_{Ru} = (W_{Su} + W_{Lu})\left(L_T + h_w + \frac{L_H}{2}\right) + W_{Wu}\left(L_T + \frac{h_W}{2}\right) + W_{Bu}\frac{L_B}{2} = \quad \mathbf{28292 \text{ ft-lb}}$$

$$e_u' = \frac{M_{Ru} - M_{ou}}{M_{ou}} = \quad \mathbf{0.40 \text{ ft}} \qquad e_u = \frac{L_B}{2} - e_u' = \quad 2.93 \text{ ft}$$

$$\frac{L_B}{6} = \quad 1.11 \text{ ft}$$ **No Good**

$$q_u = \frac{\sum Wu\left(1 \pm \frac{6e_u}{L_B}\right)}{BL_B} =$$

4247 psf At Toe	q_{uF} **= 2400 psf**
-1913 psf At Heel	q_{uB} **= 1781 psf**

Toe Design

$$M_u = \frac{L_T{}^2 B(q_{uF} + 2q_{uToe})}{6} - \frac{1.2W_B L_T{}^2}{2L_B} = \quad \mathbf{7262 \text{ ft-lb}}$$

Assume Tension Controlled Section:

$$\rho_{min} = \frac{0.85f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383bd^2f'_c}}\right)}{f_y} = \quad 0.0006$$

$$\rho_t = 0.319\beta_1\frac{f'_c}{f_y} = \quad 0.0136$$ **Tension Controlled**

$$\beta_1 = \quad 0.85 \qquad A_S = \rho bd = \quad 0.11 \text{ sq. in.}$$

$$A_{Smin} = 0.0018bh = \quad 0.39 \text{ sq. in.}$$ [Governs]

Heel Design

$$M_u = \frac{L_H\left(1.2W_S + 1.6W_L + \frac{1.2W_B L_H}{L_B}\right)}{2} - \frac{{L_H}^2 B(q_{uB} + q_{uheel})}{6} =$$ **16255 ft-lb**

Assume Tension Controlled Section:

$$\rho_{min} = \frac{0.85f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383bd^2 f'_c}}\right)}{f_y} =$$ 0.0015

$$\rho_t = 0.319\beta_1 \frac{f'_c}{f_y} =$$ 0.0136 **Tension Controlled**

$\beta_1 =$ 0.85 $A_S = \rho bd =$ 0.25 sq. in.

$A_{Smin} = 0.0018bh =$ 0.39 sq. in. [Governs]

Stem Design

$$M_u = 1.6\left(\frac{p_A {L_W}^3}{6} + \frac{p_A \hbar_L {L_W}^2}{2}\right) =$$ **11691 ft-lb**

Assume Tension Controlled Section:

$$\rho_{min} = \frac{0.85f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383bd^2 f'_c}}\right)}{f_y} =$$ 0.0010

$$\rho_t = 0.319\beta_1 \frac{f'_c}{f_y} =$$ 0.0181 **Tension Controlled**

$\beta_1 =$ 0.85 $A_S = \rho bd =$ 0.07 sq. in.

$A_{Smin} = 0.0018bh =$ 0.17 sq. in. [Governs]

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MSE WALL CALCULATION

$K_A = 0.33$

$p_A = \gamma_s K_A$

$\gamma_s = 120$ pcf $\qquad p_A = (120)(0.33) = \underline{40 \text{ psf}}$

$h_L = \dfrac{\omega}{\gamma_s} \qquad \omega = 200$ plf (TRAFFIC LOAD)

$$h_L = \frac{200 \text{ plf}}{120 \text{ pcf}} = 1.67 \text{ ft/ft}$$

$$H_A = \frac{p_A h_T^2}{2} = \frac{(40 \text{ psf})(13.50)^2}{2} = \underline{3645 \text{ plf}}$$

$h_T = \underline{13.50 \text{ ft}}$

$$H_L = p_A h_L h_T = (40)(1.67)(13.50) = \underline{900 \text{ plf}}$$

TRY SPACING GEOGRID AT 24" O.C.

$$3645 \text{ plf} + 900 \text{ plf} = \underline{4545 \text{ plf}}$$

$$\frac{4545 \text{ plf}}{12 \text{ ft}} = 379 \text{ plf} \text{ (VERTICALLY}$$

$$(379 \text{ plf})(2 \text{ ft}) = 758 \text{ lb / GRID}$$

USING TENSAR UX1100 MSE, (CAPACITY = 1850 lb)

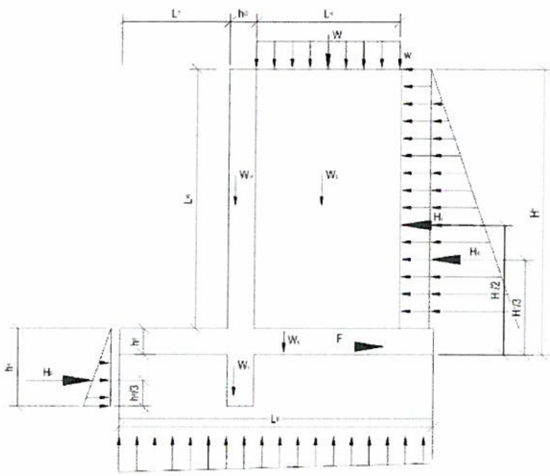
FACTOR OF SAFETY $\dfrac{1850}{758} = 2.44 > 2.0 \therefore$ OK

USING A 45° FAILURE PLANE, GRID WILL EXTEND 6', 9', 12', 12', & 15' RESPECTIVELY AT 2' INCREMENTS.

8' Wall West Side

Wall Section width for Analysis (B)	1.00 ft	$f'_c =$	3000 psi
Soil Friction Angle (φ)	30 deg.	$f_y =$	60000 psi
Soil Bearing Capacity (q_{ALLOW})	3000 psf	$f'_{c\,stem} =$	4000 psi
Soil Height above heel (L_W)	8 ft		
Soil Height above toe (L_W)	2 ft		
Slope Over Heel (β)	0.00 deg.		
Surcharge Load (w)	0 plf		
Heel length (L_H)	2.50 ft		
Toe Length (L_T)	4.00 ft		
Wall Thickness (h_W)	1.00 ft		
Footing Thickness (h_B)	1.00 ft		
Key Length (h_K')	0.00 ft		
Weight of Soil (γ_S)	120 pcf		
Weight of Reinforced Concrete (γ_C)	150 pcf		
Frictional Resistance (μ)	0.35		
Passive Force Length (h_K)	3.00 ft		
Total Footing Length (L_B)	7.50 ft		
Total Wall Height (h_T)	9.00 ft		

Component	W (lb)	Wu (lb)
W_L	0	0
W_S	2400	2880
W_W	1200	1440
W_B	1125	1350
W_K	0	0
W_{SP}	960	1152
Totals	5685	6822



$$K_A = \cos\beta \frac{\cos\beta - \sqrt{\cos\beta^2 - \cos\varphi^2}}{\cos\beta + \sqrt{\cos\beta^2 - \cos\varphi^2}} = \quad 0.33$$

$$p_A = \gamma_S K_A = \quad 40 \text{ psf}$$

$$\hbar_L = \frac{w}{\gamma_S} = \quad 0.00 \text{ ft}$$

$$H_A = \frac{p_A {h_T}^2}{2} = \quad 1620 \text{ plf}$$

$$H_L = p_A \hbar_L h_T = \quad 0 \text{ plf}$$

$$K_P = \frac{1 + \sin\varphi}{1 - \sin\varphi} = \quad 3.00$$

$$p_P = \gamma_S K_P = \quad 360 \text{ pcf}$$

$$H_P = \frac{p_P {h_K}^2}{2} = \quad 1620 \text{ lb}$$

Overturning Moment

$$M_o = H_L \frac{h_T}{2} + H_A \frac{h_T}{3} = \quad \mathbf{4860\ ft\text{-}lb}$$

Restoring Moment

$$R_F = Floor\ resisting\ force = \quad \mathbf{6119\ lb}$$

$$h_F = Floor\ location\ above\ footing = \quad \mathbf{4.50\ ft}$$

$$M_R = (W_S + W_L)\left(L_T + h_w + \frac{L_H}{2}\right) + W_W\left(L_T + \frac{h_W}{2}\right) + W_B \frac{L_B}{2} + R_F h_F = \quad \mathbf{52154\ ft\text{-}lb}$$

$$FS = Factor\ of\ Safety = \quad \mathbf{10.7} \quad \mathbf{OK}$$

Bearing Capacity

$$e' = \frac{M_R - M_o}{M_o} = \quad 9.73 \text{ ft} \qquad e = \frac{L_B}{2} - e' = \quad -5.98 \text{ ft}$$

$$\frac{L_B}{6} = \quad 1.25 \text{ ft} \qquad \text{OK}$$

$$q = \frac{\sum W\left(1 \pm \frac{6e}{L_B}\right)}{BL_B} =$$

-2869 psf	At Toe	<	3000 psf	OK
4385 psf	At Heel	>	0 psf	OK

Sliding Resistance **Factor of Safety**

$$FS_S = \frac{F + H_P}{H_A + H_L} = 1990 \text{ lb} \quad F = \mu \sum W = \quad \mathbf{2.23} \quad > \quad 1.5 \quad \text{OK}$$

Footing Design

$$M_{ou} = 1.6M_o = \quad \mathbf{7776 \text{ ft-lb}}$$

$$M_{Ru} = (W_{Su} + W_{Lu})\left(L_T + h_w + \frac{L_H}{2}\right) + W_{Wu}\left(L_T + \frac{h_W}{2}\right) + W_{Bu}\frac{L_B}{2} = \quad \mathbf{29543 \text{ ft-lb}}$$

$$e_u' = \frac{M_{Ru} - M_{ou}}{M_{ou}} = \quad \mathbf{2.80 \text{ ft}} \qquad e_u = \frac{L_B}{2} - e_u' = \quad 0.95 \text{ ft}$$

$$\frac{L_B}{6} = \quad 1.25 \text{ ft} \qquad \text{OK}$$

$$q_u = \frac{\sum Wu\left(1 \pm \frac{6e_u}{L_B}\right)}{BL_B} =$$

1601 psf	**At Toe**	$q_{uF} =$	**863 psf**
218 psf	**At Heel**	$q_{uB} =$	**679 psf**

Toe Design

$$M_u = \frac{L_T{}^2 B(q_{uF} + 2q_{uToe})}{6} - \frac{1.2W_B L_T{}^2}{2L_B} = \quad \mathbf{10843 \text{ ft-lb}}$$

Assume Tension Controlled Section:

$$\rho_{min} = \frac{0.85f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383bd^2 f'_c}}\right)}{f_y} = \quad 0.0029$$

$$\rho_t = 0.319\beta_1 \frac{f'_c}{f_y} = \quad 0.0136 \qquad \textbf{Tension Controlled}$$

$$\beta_1 = \quad 0.85 \quad A_S = \rho bd = \quad 0.29 \text{ sq. in.} \quad \textbf{[Governs]}$$

$$A_{Smin} = 0.0018bh = \quad 0.26 \text{ sq. in.}$$

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Heel Design

$$M_u = \frac{L_H\left(1.2W_S + 1.6W_L + \frac{1.2W_B L_H}{L_B}\right)}{2} - \frac{{L_H}^2 B(q_{uB} + q_{uheel})}{6} = \quad \textbf{3002 ft-lb}$$

Assume Tension Controlled Section:

$$\rho_{min} = \frac{0.85f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383bd^2 f'_c}}\right)}{f_y} = \quad 0.0008$$

$$\rho_t = 0.319\beta_1 \frac{f'_c}{f_y} = \quad 0.0136$$ **Tension Controlled**

$\beta_1 =$ 0.85 $A_S = \rho bd =$ 0.08 sq. in.

$A_{Smin} = 0.0018bh =$ 0.26 sq. in. [Governs]

Stem Design

$$M_u = 1.6\left(\frac{p_A {L_W}^3}{6} + \frac{p_A \hbar_L {L_W}^2}{2}\right) = \quad \textbf{5461 ft-lb}$$

Assume Tension Controlled Section:

$$\rho_{min} = \frac{0.85f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383bd^2 f'_c}}\right)}{f_y} = \quad 0.0014$$

$$\rho_t = 0.319\beta_1 \frac{f'_c}{f_y} = \quad 0.0181$$ **Tension Controlled**

$\beta_1 =$ 0.85 $A_S = \rho bd =$ 0.16 sq. in.

$A_{Smin} = 0.0018bh =$ 0.26 sq. in. [Governs]

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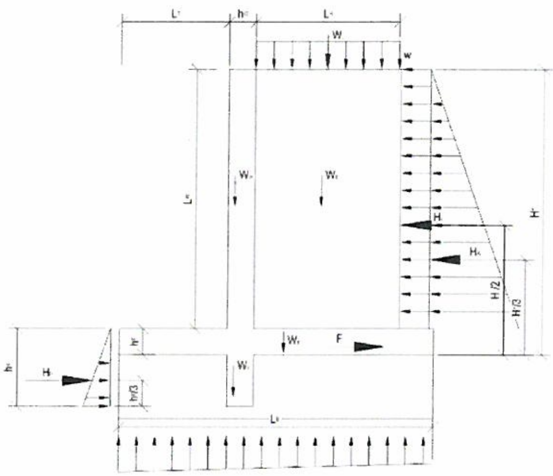
10' Wall West Side

Wall Section width for Analysis (B)	1.00 ft
Soil Friction Angle (φ)	30 deg.
Soil Bearing Capacity (q_{ALLOW})	3000 psf
Soil Height above heel (L_W)	10.5 ft
Soil Height above toe (L_W)	2 ft
Slope Over Heel (β)	0.00 deg.
Surcharge Load (w)	0 plf
Heel length (L_H)	4.00 ft
Toe Length (L_T)	6.75 ft
Wall Thickness (h_W)	1.00 ft
Footing Thickness (h_B)	1.50 ft
Key Length (h_K')	0.00 ft
Weight of Soil (γ_S)	120 pcf
Weight of Reinforced Concrete (γ_C)	150 pcf
Frictional Resistance (μ)	0.35
Passive Force Length (h_K)	3.50 ft
Total Footing Length (L_B)	11.75 ft
Total Wall Height (h_T)	12.00 ft

$f'_c =$	3000 psi
$f_y =$	60000 psi
$f'_{c\,stem} =$	4000 psi

Component	W (lb)	Wu (lb)
W_L	0	0
W_S	5040	6048
W_W	1575	1890
W_B	2644	3172.5
W_K	0	0
W_{SP}	1620	1944
Totals	10879	13055



$$K_A = \cos\beta \frac{\cos\beta - \sqrt{\cos\beta^2 - \cos\varphi^2}}{\cos\beta + \sqrt{\cos\beta^2 - \cos\varphi^2}} = \quad 0.33$$

$$p_A = \gamma_S K_A = \quad 40 \text{ psf}$$

$$\hbar_L = \frac{w}{\gamma_S} = \quad 0.00 \text{ ft}$$

$$H_A = \frac{p_A {h_T}^2}{2} = \quad 2880 \text{ plf}$$

$$H_L = p_A \hbar_L h_T = \quad 0 \text{ plf}$$

$$K_P = \frac{1 + \sin\varphi}{1 - \sin\varphi} = \quad 3.00$$

$$p_p = \gamma_S K_P = \quad 360 \text{ pcf}$$

$$H_P = \frac{p_P {h_K}^2}{2} = \quad 2205 \text{ lb}$$

Overturning Moment

$$M_o = H_L \frac{h_T}{2} + H_A \frac{h_T}{3} = \quad \textbf{11520 ft-lb}$$

Restoring Moment

$$R_F = Floor\ resisting\ force = \quad \textbf{6119 lb}$$

$$h_F = Floor\ location\ above\ footing = \quad \textbf{4.50 ft}$$

$$M_R = (W_S + W_L)\left(L_T + h_w + \frac{L_H}{2}\right) + W_W\left(L_T + \frac{h_W}{2}\right) + W_B \frac{L_B}{2} + R_F h_F = \quad \textbf{103626 ft-lb}$$

$$FS = Factor\ of\ Safety = \quad \textbf{9.0} \quad \textbf{OK}$$

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Answers to Infrastructure

Bearing Capacity

$$e' = \frac{M_R - M_o}{M_o} = \quad 8.00 \text{ ft} \qquad e = \frac{L_B}{2} - e' = \quad -2.12 \text{ ft}$$

$$\frac{L_B}{6} = \quad 1.96 \text{ ft} \qquad \text{OK}$$

$$q = \frac{\sum W\left(1 \pm \frac{6e}{L_B}\right)}{BL_B} =$$

-77 psf At Toe	<	3000 psf	OK
1928 psf At Heel	>	0 psf	OK

Sliding Resistance **Factor of Safety**

$$FS_S = \frac{F + H_P}{H_A + H_L} = 3808 \text{ lb} \qquad F = \mu \sum W = \quad \mathbf{2.09} \quad > \quad 1.5 \qquad \text{OK}$$

Footing Design

$$M_{ou} = 1.6M_o = \ \mathbf{18432 \text{ ft-lb}}$$

$$M_{Ru} = (W_{Su} + W_{Lu})\left(L_T + h_w + \frac{L_H}{2}\right) + W_{Wu}\left(L_T + \frac{h_W}{2}\right) + W_{Bu}\frac{L_B}{2} = \quad \mathbf{91309 \text{ ft-lb}}$$

$$e_u' = \frac{M_{Ru} - M_{ou}}{M_{ou}} = \quad \mathbf{3.95 \text{ ft}} \qquad e_u = \frac{L_B}{2} - e_u' = \quad 1.92 \text{ ft}$$

$$\frac{L_B}{6} = \quad 1.96 \text{ ft} \qquad \text{OK}$$

$$q_u = \frac{\sum Wu\left(1 \pm \frac{6e_u}{L_B}\right)}{BL_B} =$$

2201 psf At Toe	$q_{uF} =$	949 psf
21 psf At Heel	$q_{uB} =$	763 psf

Toe Design

$$M_u = \frac{L_T{}^2 B(q_{uF} + 2q_{uToe})}{6} - \frac{1.2W_B L_T{}^2}{2L_B} = \quad \mathbf{40629 \text{ ft-lb}}$$

Assume Tension Controlled Section:

$$\rho_{min} = \frac{0.85f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383bd^2 f'_c}}\right)}{f_y} = \quad 0.0037$$

$$\rho_t = 0.319\beta_1\frac{f'_c}{f_y} = \quad 0.0136 \qquad \textbf{Tension Controlled}$$

$$\beta_1 = \quad 0.85 \quad A_S = \rho bd = \quad 0.65 \text{ sq. in.} \quad \textbf{[Governs]}$$

$$A_{Smin} = 0.0018bh = \quad 0.39 \text{ sq. in.}$$

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Heel Design

$$M_u = \frac{L_H\left(1.2W_S + 1.6W_L + \frac{1.2W_B L_H}{L_B}\right)}{2} - \frac{{L_H}^2 B(q_{uB} + q_{uheel})}{6} = \quad \textbf{12108 ft-lb}$$

Assume Tension Controlled Section:

$$\rho_{min} = \frac{0.85f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383bd^2f'_c}}\right)}{f_y} = \quad 0.0011$$

$$\rho_t = 0.319\beta_1\frac{f'_c}{f_y} = \quad 0.0136$$ **Tension Controlled**

$\beta_1 =$ 0.85 $\quad A_S = \rho bd =$ 0.19 sq. in.

$A_{Smin} = 0.0018bh =$ 0.39 sq. in. [Governs]

Stem Design

$$M_u = 1.6\left(\frac{p_A {L_W}^3}{6} + \frac{p_A \hbar_L {L_W}^2}{2}\right) = \quad \textbf{12348 ft-lb}$$

Assume Tension Controlled Section:

$$\rho_{min} = \frac{0.85f'_c\left(1 - \sqrt{1 - \frac{M_u}{0.383bd^2f'_c}}\right)}{f_y} = \quad 0.0011$$

$$\rho_t = 0.319\beta_1\frac{f'_c}{f_y} = \quad 0.0181$$ **Tension Controlled**

$\beta_1 =$ 0.85 $\quad A_S = \rho bd =$ 0.13 sq. in.

$A_{Smin} = 0.0018bh =$ 0.26 sq. in. [Governs]

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