

Retaining Wall Design Report

Wall Info

Project Name: my first project

Wall Designation: my wall

Date:

Input Data

Wall Data:

Wall Height Above Soil (Ht) = 1 ft

Foundation Height (hf) = 1.5 ft

Heel Length (Le) = 6 ft

Top Thickness (tt) = 1 ft

Back Slope (i) = 0°

Materials:

Steel Yield Strength f_y = 60000 psi

Soil Density γ = 110 pcf

Toe Rebar Size = #4

Toe Rebar Clearance = 3 in

Heel Bar Spacing = 12 in o/c

Wall Rebar Size = #4

Wall Rebar Clearance = 2 in

Aggregate Correction Factor (K1) = 1

Equivalent Passive Fluid Pressure = 300 pcf

Foundation Friction Factor (f_r) = 0.4

Wall Height (H) = 6 ft

Soil Height in Front (H') = 1 ft

Foundation Length (Lf) = 8 ft

Base Thickness (t) = 1 ft

Key (Hk $\times$ tk @ from toe) = 1 $\times$ 1 ft @ 2 ft

Concrete Compressive Strength f'_c = 4000 psi

Concrete Density γ_c = 150 pcf

Steel Elastic Modulus Es = 29000 ksi

Toe Bar Spacing = 12 in o/c

Heel Rebar Size = #4

Heel Rebar Clearance = 3 in

Wall Bar Spacing = 12 in o/c

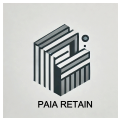
Wall Vertical Bar Anchorage = Standard 90-degree tension hook (assumed)

AASHTO Crack-Control Class = Class 1 (γ_e = 1.00)

Soil:

Bearing Pressure at Strength Limit (q_{ua}) = 4500 psf

Equivalent Uniform Surcharge Height (H_e) = 2 ft



Retaining Wall Design Report

Bearing Condition = Soil Bearing

Passive Ignored Height = 0 ft

Soil Friction $\phi = 30^\circ$

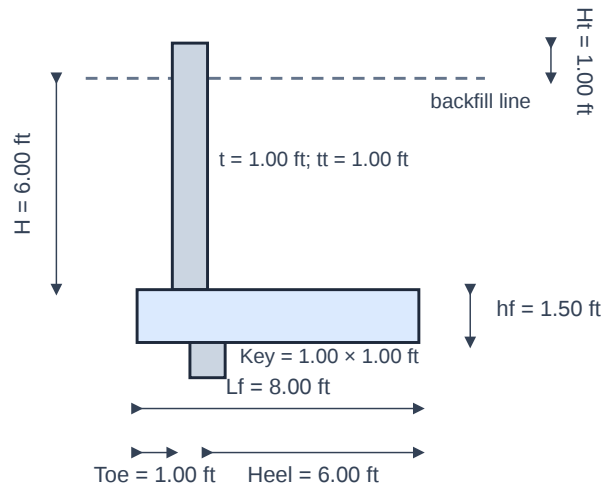
Wall Batter $\beta = 0^\circ$

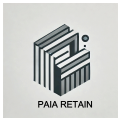
Passive Resistance Coefficient = 0.5

Active Pressure Method = Coulomb

Wall-Soil Friction $\delta = 15^\circ$

Wall Geometry

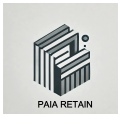




Design Summary

OVERALL DESIGN STATUS: PASS

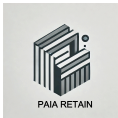
Check	Status
External stability — eccentricity	PASS
External stability — bearing	PASS
External stability — sliding	PASS
Toe flexure	PASS
Heel flexure	PASS
Toe crack control	PASS
Heel crack control	PASS
Toe shear	PASS
Heel shear	PASS
Wall stem flexure	PASS
Wall stem crack control	PASS
Wall vertical hooked-bar anchorage	PASS
Wall horizontal shrinkage/temperature	PASS
Wall stem shear	PASS



Retaining Wall Design Report

Coulomb Earth Pressure Calculation (Step-by-step)

Parameter	Value
phi_deg	30.00
phi_rad	0.52
delta_deg	15.00
delta_rad	0.26
wall_from_vertical_beta_deg	0.00
wall_from_vertical_beta_rad	0.00
backfill_slope_i_deg	0.00
backfill_slope_i_rad	0.00
alpha_deg	90.00
alpha_rad	1.57
beta_deg	0.00
beta_rad	0.00
sin_alpha	0.87
sin_alpha_minus_delta	1.00
sin_alpha_plus_beta	0.97
num_sin(alpha+phi)^2	0.75
den1_sin(alpha)^2*sin(alpha-delta)	1.00
r_top_sin(phi+delta)*sin(phi-beta)	0.35
r_bot_sin(alpha-delta)*sin(alpha+beta)	0.97
ratio_r_top_over_r_bot	0.37
sqrt_ratio	0.61
inner_1_plus_sqrt	1.61
KA	0.30
gamma_pcf	110.00
Pa_pcf	33.16
H_eff_ft	7.50
Ha_lb_per_ft	932.51
warnings	[]



Retaining Wall Design Report

Mononobe-Okabe (K_{AE}) Calculation

$$A1_e = \cos(\varphi - \beta - \tan^{-1}(kh)) = 0.931$$

$$B1_e = \cos(\beta) = 1.000$$

$$C1_e = \cos(\delta + \beta + \tan^{-1}(kh)) = 0.917$$

$$D1_e = \sin(\varphi + \delta) = 0.707$$

$$E1_e = \sin(\varphi - i_1 - \tan^{-1}(kh)) = 0.366$$

$$F1_e = \cos(\delta + \beta + \tan^{-1}(kh)) = 0.917$$

$$G1_e = \cos(i_1 - \beta) = 1.000$$

$$H1_e = \cos(\tan^{-1}(kh)) = 0.989$$

$$\text{denom_e} = (H1_e * B1_e^{**2} * C1_e) / (\tan^{-1}(kh) * \cos(\beta)^{**2} * \cos(\delta + \beta + \tan^{-1}(kh))) = 0.907$$

$$\text{inner_e} = 1.0 + \text{math.sqrt}(\text{abs}((D1_e * E1_e) / (F1_e * G1_e))) = 1.0 + \text{math.sqrt}(\text{abs}((\sin(\varphi + \delta) * \cos(\delta + \beta + \tan^{-1}(kh))) / (\cos(\delta + \beta + \tan^{-1}(kh)) * \cos(i_1 - \beta)))) = 1.531$$

$$KAE = (\cos(\varphi - \beta - \tan^{-1}(kh))^{**2}) / (\text{denom_e} * (\text{inner_e}^{**2})) = 0.407$$

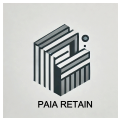
$$K_{AE} = (\cos(\varphi - \beta - \tan^{-1}(kh))^2) / (\text{denom_e} * (\text{inner_e}^2))$$

$$K_{AE} = 0.407$$

$$KE = KAE - KA = 0.106$$

Term	Value
A1e	0.931
B1e	1.000
C1e	0.917
D1e	0.707
E1e	0.366
F1e	0.917
G1e	1.000
H1e	0.989
denom_e	0.907
inner_e	1.531
KAE	0.407
KE = KAE - KA	0.106

Additional Loads:**Additional Vertical Dead Load (VDL) = 0 lbs****Additional Vertical Live Load (VLL) = 0 lbs****Eccentricity of DL + LL = 0 ft**



Calculation Details:

0. Effective Retained Height for External Stability (H_{stab})

$$\begin{aligned} H_{eff} &= H + hf + \tan(i) \cdot Le \\ &= 6.00 + 1.50 + \tan(0.00^\circ) \cdot 6.00 \\ &= 7.50 \text{ ft} \end{aligned}$$

External stability considers retained soil down to the bottom of the footing. Wall-stem structural design uses the retained height above the footing.

1. Active Earth Pressure

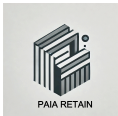
$$\begin{aligned} \text{Active pressure method} &= \text{Coulomb} \\ k &= 0.3014 \\ P_a &= k \cdot \gamma = 0.3014 \cdot 110.00 = 33.16 \text{ pcf} \\ H_a &= 0.5 \cdot P_a \cdot H_{eff}^2 \cdot \cos(\delta) = 900.73 \text{ lb/ft} \\ H_{av} &= 0.5 \cdot P_a \cdot H_{eff}^2 \cdot \sin(\delta) = 241.35 \text{ lb/ft} \end{aligned}$$

2. Passive Earth Pressure

$$\begin{aligned} H_{p_tot} &= H' + hf + H_k = 1.00 + 1.50 + 1.00 = 3.50 \text{ ft} \\ H_p &= 0.5 \cdot P_p \cdot H_{p_tot}^2 = 0.5 \cdot 300.00 \cdot 3.50^2 = 1837.50 \text{ lb/ft} \end{aligned}$$

3. Surcharge Pressure

$$H_{sur} = k \cdot \gamma \cdot H_e \cdot H_{eff} \cdot \cos(\delta) = 480.39 \text{ lb/ft}; H_{surv} = k \cdot \gamma \cdot H_e \cdot H_{eff} \cdot \sin(\delta) = 128.72 \text{ lb/ft}$$



Retaining Wall Design Report

4. Dead Loads (Weights)

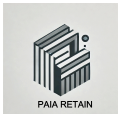
Component	Equation
W1	$W1 = 0.5 \cdot (H+Ht) \cdot (t+tt) \cdot \gamma_c =$ $0.5 \cdot (6.00+1.00) \cdot (1.00+1.00) \cdot 150.00 = 1050.00$ lb/ft
W2	$W2 = Lf \cdot hf \cdot \gamma_c = 8.00 \cdot 1.50 \cdot 150.00 = 1800.00$ lb/ft
W3	$W3 = [Le \cdot H + 0.5 \cdot Le^2 \cdot \tan(i) + (t-tt) \cdot H/2] \cdot \gamma =$ $[6.00 \cdot 6.00 + 0.5 \cdot 6.00^2 \cdot \tan(0.00^\circ) +$ $(1.00-1.00) \cdot 6.00/2] \cdot 110.00 = 3960.00 \text{ lb/ft}$
Wtoe	$Wtoe = toe \cdot H_{\text{front}} \cdot \gamma = 1.00 \cdot 1.00 \cdot 110.00 =$ 110.00 lb/ft
W4	$W4 = tk \cdot Hk \cdot \gamma_c = 1.00 \cdot 1.00 \cdot 150.00 = 150.00$ lb/ft
Wsur	$Wsur = \gamma \cdot He \cdot Le = 110.00 \cdot 2.00 \cdot 6.00 = 1320.00$ lb/ft

5. Seismic Loads

Seismic method = Mononobe-Okabe (M-O)

$kh = 0.150$, $\theta = \text{atan}(kh) = 8.53^\circ$

$Heq = 0.5 \cdot \Delta K \cdot \gamma \cdot H_{\text{eff}}^2 = 0.5 \cdot 0.106 \cdot 110.00 \cdot 7.50^2 = 327.70 \text{ lb/ft}$

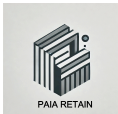


Retaining Wall Design Report

6. Moments about the Toe

Resisting Moments

Moment	Equation
M1	$M1 = w1 \cdot \text{arm} = 1050.00 \cdot ((8.00 - 6.00) - 0.5 \cdot (1.00 + 1.00)/2) = 1575.00 \text{ lb-ft/ft}$
M2	$M2 = w2 \cdot (Lf/2) = 1800.00 \cdot (8.00/2) = 7200.00 \text{ lb-ft/ft}$
M3	$M3 = [Le \cdot H \cdot (Le/2 + (Lf - Le)) + 0.5 \cdot Le^2 \cdot \tan(i) \cdot ((Lf - Le) + Le \cdot 2/3) + (t - tt) \cdot H/2 \cdot ((Lf - Le) + (t - tt)/3)] \cdot \gamma = [6.00 \cdot 6.00 \cdot (6.00/2 + (8.00 - 6.00)) + 0.5 \cdot 6.00^2 \cdot \tan(0.00^\circ) \cdot ((8.00 - 6.00) + 6.00 \cdot 2/3) + (1.00 - 1.00) \cdot 6.00/2 \cdot ((8.00 - 6.00) + (1.00 - 1.00)/3)] \cdot 110.00 = 19800.00 \text{ lb-ft/ft}$
Mtoe	$Mtoe = wtoe \cdot toe/2 = 110.00 \cdot 1.00/2 = 55.00 \text{ lb-ft/ft}$
M4	$M4 = w4 \cdot (KLoc + tk/2) = 150.00 \cdot (2.00 + 1.00/2) = 375.00 \text{ lb-ft/ft}$
Msur	$Msur = wsur \cdot (Le/2 + (Lf - Le)) = 1320.00 \cdot (6.00/2 + (8.00 - 6.00)) = 6600.00 \text{ lb-ft/ft}$
Mhav	$Mhav = Hav \cdot Lf = 241.35 \cdot 8.00 = 1930.81 \text{ lb-ft/ft}$
Msurv	$Msurv = Hsurv \cdot Lf = 128.72 \cdot 8.00 = 1029.76 \text{ lb-ft/ft}$
MVDL	$MVDL = VDL \cdot ((Lf - Le) - t/2 + ED) = 0.00 \cdot ((8.00 - 6.00) - 1.00/2 + 0.00) = 0.00 \text{ lb-ft/ft}$
MVLL	$MVLL = VLL \cdot ((Lf - Le) - t/2 + ED) = 0.00 \cdot ((8.00 - 6.00) - 1.00/2 + 0.00) = 0.00 \text{ lb-ft/ft}$



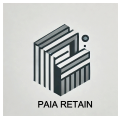
Retaining Wall Design Report

Overturning Moments

Moment	Equation
Mh1	$Mh1 = Ha \cdot (H_{eff}/3) = 900.73 \cdot (7.50/3) = 2251.83 \text{ lb-ft/ft}$
Mh2	$Mh2 = H_{sur} \cdot (H_{eff}/2) = 480.39 \cdot (7.50/2) = 1801.47 \text{ lb-ft/ft}$
Mh4	$\begin{aligned} Mh4 &= H_{eq} \cdot (SeisH \cdot H_{eff}) + \\ &kh \cdot W1 \cdot (0.5 \cdot (H+Ht)+hf) + \\ &kh \cdot W3 \cdot (H+hf+\tan(i) \cdot Le)/2 \\ &= 327.70 \cdot (0.50 \cdot 7.50) + \\ &0.150 \cdot 1050.00 \cdot (0.5 \cdot (6.00+1.00)+1.50) + \\ &0.150 \cdot 3960.00 \cdot (6.00+1.50+\tan(0.00^\circ) \cdot 6.00)/2 = \\ &4243.88 \text{ lb-ft/ft} \end{aligned}$

Unfactored Loads

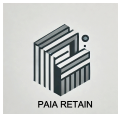
Quantity	Equation	Result
Mvs	$\begin{aligned} Mvs &= M1 + M2 + M3 + M_{toe} + \\ &M4 + M_{sur} + M_{hav} + M_{surv} + \\ &MVDL + MVLL = 1575.00 + \\ &7200.00 + 19800.00 + 55.00 + \\ &375.00 + 6600.00 + 1930.81 + \\ &1029.76 + 0.00 + 0.00 = \\ &38565.57 \text{ lb-ft/ft} \end{aligned}$	38565.57
Mhts	$Mhts = Mh1 + Mh2 = 2251.83 + 1801.47 = 4053.30 \text{ lb-ft/ft}$	4053.30
Vds	$\begin{aligned} Vds &= W1 + W2 + W3 + W_{toe} + \\ &W4 + W_{sur} + H_{av} + H_{surv} + VDL \\ &+ VLL = 1050.00 + 1800.00 + \\ &3960.00 + 110.00 + 150.00 + \\ &1320.00 + 241.35 + 128.72 + \\ &0.00 + 0.00 = 8760.07 \text{ lb/ft} \end{aligned}$	8760.07
Hts	$Hts = Ha + H_{sur} = 900.73 + 480.39 = 1381.12 \text{ lb/ft}$	1381.12



Factored Load Combinations

Strength Ia

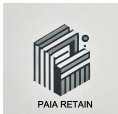
Type	Equation	Result
Factored Vertical Load (Vt)	$\begin{aligned} Vt &= 0.90*(W1+W2+W4+VDL) + \\ &1.00*(W3+Wtoe) + 1.50*Hav + \\ &0.00*VLL + 0.00*Wsur + \\ &1.75*Hsurv \\ &= 0.90* \\ &(1050.00+1800.00+150.00+0.00) \\ &+ 1.00*(3960.00+110.00) + \\ &1.50*241.35 + 0.00*0.00 + \\ &0.00*1320.00 + 1.75*128.72 = \\ &7357.29 \text{ lb/ft} \end{aligned}$	7357.29 lb/ft
Factored Horizontal Load (Ht)	$\begin{aligned} Ht &= 1.50*Ha + 1.75*Hsur + \\ &0.00*Heq \\ &= 1.50*900.73 + \\ &1.75*480.39 + 0.00*327.70 = \\ &2191.78 \text{ lb/ft} \end{aligned}$	2191.78 lb/ft
Resisting Moment (MVt)	$\begin{aligned} MVt &= 0.90*(M1+M2+M4+MVDL) + \\ &1.00*(M3+Mtoe) + 1.50*Mhav + \\ &0.00*MVLL + 0.00*Msur + \\ &1.75*Msurv \\ &= 0.90*(9150.00) + \\ &1.00*(19800.00+55.00) + \\ &1.50*1930.81 + 0.00*0.00 + \\ &0.00*6600.00 + 1.75*1029.76 = \\ &32788.29 \text{ lb-ft/ft} \end{aligned}$	32788.29 lb-ft/ft
Overturning Moment (MHt)	$\begin{aligned} MHt &= 1.50*Mh1 + 1.75*Mh2 + \\ &0.00*Mh4 \\ &= 1.50*2251.83 + \\ &1.75*1801.47 + 0.00*4243.88 = \\ &6530.32 \text{ lb-ft/ft} \end{aligned}$	6530.32 lb-ft/ft



Retaining Wall Design Report

Strength lb

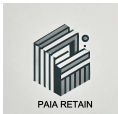
Type	Equation	Result
Factored Vertical Load (Vt)	$\begin{aligned} Vt &= 1.25*(W1+W2+W4+VDL) + \\ &1.35*(W3+Wtoe) + 1.50*Hav + \\ &1.75*VLL + 1.75*Wsur + \\ &1.75*Hsurv \\ &= 1.25* \\ &(1050.00+1800.00+150.00+0.00) \\ &+ 1.35*(3960.00+110.00) + \\ &1.50*241.35 + 1.75*0.00 + \\ &1.75*1320.00 + 1.75*128.72 = \\ &12141.79 \text{ lb/ft} \end{aligned}$	12141.79 lb/ft
Factored Horizontal Load (Ht)	$\begin{aligned} Ht &= 1.50*Ha + 1.75*Hsur + \\ &0.00*Heq \\ &= 1.50*900.73 + \\ &1.75*480.39 + 0.00*327.70 = \\ &2191.78 \text{ lb/ft} \end{aligned}$	2191.78 lb/ft
Resisting Moment (MVt)	$\begin{aligned} MVt &= 1.25*(M1+M2+M4+MVDL) + \\ &1.35*(M3+Mtoe) + 1.50*Mhav + \\ &1.75*MVLL + 1.75*Msur + \\ &1.75*Msurv \\ &= 1.25*(9150.00) + \\ &1.35*(19800.00+55.00) + \\ &1.50*1930.81 + 1.75*0.00 + \\ &1.75*6600.00 + 1.75*1029.76 = \\ &54490.04 \text{ lb-ft/ft} \end{aligned}$	54490.04 lb-ft/ft
Overturing Moment (Mht)	$\begin{aligned} Mht &= 1.50*Mh1 + 1.75*Mh2 + \\ &0.00*Mh4 \\ &= 1.50*2251.83 + \\ &1.75*1801.47 + 0.00*4243.88 = \\ &6530.32 \text{ lb-ft/ft} \end{aligned}$	6530.32 lb-ft/ft



Retaining Wall Design Report

Strength IV

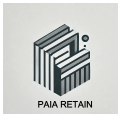
Type	Equation	Result
Factored Vertical Load (Vt)	$\begin{aligned} Vt &= 1.50*(W1+W2+W4+VDL) + \\ &1.35*(W3+Wtoe) + 1.50*Hav + \\ &0.00*VLL + 0.00*Wsur + \\ &0.00*Hsurv \\ &= 1.50* \\ &(1050.00+1800.00+150.00+0.00) \\ &+ 1.35*(3960.00+110.00) + \\ &1.50*241.35 + 0.00*0.00 + \\ &0.00*1320.00 + 0.00*128.72 = \\ &10356.53 \text{ lb/ft} \end{aligned}$	10356.53 lb/ft
Factored Horizontal Load (Ht)	$\begin{aligned} Ht &= 1.50*Ha + 0.00*Hsur + \\ &0.00*Heq \\ &= 1.50*900.73 + \\ &0.00*480.39 + 0.00*327.70 = \\ &1351.10 \text{ lb/ft} \end{aligned}$	1351.10 lb/ft
Resisting Moment (MVt)	$\begin{aligned} MVt &= 1.50*(M1+M2+M4+MVDL) + \\ &1.35*(M3+Mtoe) + 1.50*Mhav + \\ &0.00*MVLL + 0.00*Msur + \\ &0.00*Msurv \\ &= 1.50*(9150.00) + \\ &1.35*(19800.00+55.00) + \\ &1.50*1930.81 + 0.00*0.00 + \\ &0.00*6600.00 + 0.00*1029.76 = \\ &43425.46 \text{ lb-ft/ft} \end{aligned}$	43425.46 lb-ft/ft
Overturning Moment (Mht)	$\begin{aligned} Mht &= 1.50*Mh1 + 0.00*Mh2 + \\ &0.00*Mh4 \\ &= 1.50*2251.83 + \\ &0.00*1801.47 + 0.00*4243.88 = \\ &3377.75 \text{ lb-ft/ft} \end{aligned}$	3377.75 lb-ft/ft



Retaining Wall Design Report

Extreme I

Type	Equation	Result
Factored Vertical Load (Vt)	$\begin{aligned} Vt &= 1.00*(W1+W2+W4+VDL) + \\ &1.00*(W3+Wtoe) + 1.00*Hav + \\ &0.00*VLL + 0.00*Wsur + \\ &0.00*Hsurv \\ &= 1.00* \\ &(1050.00+1800.00+150.00+0.00) \\ &+ 1.00*(3960.00+110.00) + \\ &1.00*241.35 + 0.00*0.00 + \\ &0.00*1320.00 + 0.00*128.72 = \\ &7311.35 \text{ lb/ft} \end{aligned}$	7311.35 lb/ft
Factored Horizontal Load (Ht)	$\begin{aligned} Ht &= 1.00*Ha + 0.00*Hsur + \\ &1.00*Heq \\ &= 1.00*900.73 + \\ &0.00*480.39 + 1.00*327.70 = \\ &1228.43 \text{ lb/ft} \end{aligned}$	1228.43 lb/ft
Resisting Moment (MVt)	$\begin{aligned} MVt &= 1.00*(M1+M2+M4+MVDL) + \\ &1.00*(M3+Mtoe) + 1.00*Mhav + \\ &0.00*MVLL + 0.00*Msur + \\ &0.00*Msurv \\ &= 1.00*(9150.00) + \\ &1.00*(19800.00+55.00) + \\ &1.00*1930.81 + 0.00*0.00 + \\ &0.00*6600.00 + 0.00*1029.76 = \\ &30935.81 \text{ lb-ft/ft} \end{aligned}$	30935.81 lb-ft/ft
Overturning Moment (Mht)	$\begin{aligned} Mht &= 1.00*Mh1 + 0.00*Mh2 + \\ &1.00*Mh4 \\ &= 1.00*2251.83 + \\ &0.00*1801.47 + 1.00*4243.88 = \\ &6495.71 \text{ lb-ft/ft} \end{aligned}$	6495.71 lb-ft/ft



Retaining Wall Design Report

Service I

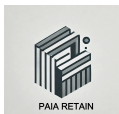
Type	Equation	Result
Factored Vertical Load (Vt)	$\begin{aligned} Vt &= 1.00*(W1+W2+W4+VDL) + \\ &1.00*(W3+Wtoe) + 1.00*Hav + \\ &1.00*VLL + 1.00*Wsur + \\ &1.00*Hsurv \\ &= 1.00* \\ &(1050.00+1800.00+150.00+0.00) \\ &+ 1.00*(3960.00+110.00) + \\ &1.00*241.35 + 1.00*0.00 + \\ &1.00*1320.00 + 1.00*128.72 = \\ &8760.07 \text{ lb/ft} \end{aligned}$	8760.07 lb/ft
Factored Horizontal Load (Ht)	$\begin{aligned} Ht &= 1.00*Ha + 1.00*Hsur + \\ &0.00*Heq \\ &= 1.00*900.73 + \\ &1.00*480.39 + 0.00*327.70 = \\ &1381.12 \text{ lb/ft} \end{aligned}$	1381.12 lb/ft
Resisting Moment (MVt)	$\begin{aligned} MVt &= 1.00*(M1+M2+M4+MVDL) + \\ &1.00*(M3+Mtoe) + 1.00*Mhav + \\ &1.00*MVLL + 1.00*Msur + \\ &1.00*Msurv \\ &= 1.00*(9150.00) + \\ &1.00*(19800.00+55.00) + \\ &1.00*1930.81 + 1.00*0.00 + \\ &1.00*6600.00 + 1.00*1029.76 = \\ &38565.57 \text{ lb-ft/ft} \end{aligned}$	38565.57 lb-ft/ft
Overturning Moment (Mht)	$\begin{aligned} Mht &= 1.00*Mh1 + 1.00*Mh2 + \\ &0.00*Mh4 \\ &= 1.00*2251.83 + \\ &1.00*1801.47 + 0.00*4243.88 = \\ &4053.30 \text{ lb-ft/ft} \end{aligned}$	4053.30 lb-ft/ft

Stability Check

•

Eccentricity (Overturning) Check

Formula: $\Sigma M = \Sigma MV - \Sigma MH$; $x_R = \Sigma M / \Sigma V$; $e_{actual} = |B/2 - x_R|$; allowable $e = B/4$ (strength) or $B/3$ (extreme).



Retaining Wall Design Report

Load Combo	ΣMV (kip-ft/ft)	ΣMH (kip-ft/ft)	ΣV (kip/ft)	B (ft)	e_{allow} (ft)	e_{actual} (ft)	Status
Strength Ia	32.79	6.53	7.36	8.00	2.00	0.43	OK
Strength Ib	54.49	6.53	12.14	8.00	2.00	0.05	OK
Strength IV	43.43	3.38	10.36	8.00	2.00	0.13	OK
Extreme I	30.94	6.50	7.31	8.00	2.67	0.66	OK
Service I	38.57	4.05	8.76	8.00	2.00	0.06	OK

Bearing Resistance Check

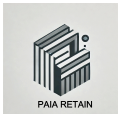
Formula: $L_{per} = B - 2|e|$; $q_{max} = q_{min} = \Sigma V / L_{per}$.

Load Combo	ΣMV (lb-ft/ft)	ΣMH (lb-ft/ft)	ΣV (lb/ft)	B (ft)	e_{actual} (ft)	L_{per} (ft)	q_{toe} (ksf)	q_{heel} (ksf)	q_{max} (ksf)	q_{min} (ksf)	Max at	q_{allow} (ksf)	Status
Strength Ia	32.79	6.53	7.36	8.00	0.43	7.14	1.03	1.03	1.03	1.03	Toe and Heel	4.50	OK
Strength Ib	54.49	6.53	12.14	8.00	0.05	7.90	1.54	1.54	1.54	1.54	Toe and Heel	4.50	OK
Strength IV	43.43	3.38	10.36	8.00	0.13	7.73	1.34	1.34	1.34	1.34	Toe and Heel	4.50	OK
Extreme I	30.94	6.50	7.31	8.00	0.66	6.69	1.09	1.09	1.09	1.09	Toe and Heel	9.00	OK
Service I	38.57	4.05	8.76	8.00	0.06	7.88	1.11	1.11	1.11	1.11	Toe and Heel	4.50	OK

Sliding Check

Formula: $R_{friction} = \mu_s \Sigma V$; $R_{total} = R_{friction} + R_{passive}$; $F.S. = R_{total} / \Sigma H$.

Load Combo	ΣH (kip/ft)	ΣV (kip/ft)	μ_s	C	$R_{friction}$ (kip/ft)	$R_{passive}$ (kip/ft)	R_{total} (kip/ft)	F.S.	Status
Strength Ia	2.19	7.36	0.400	1.00	2.94	0.92	3.86	1.76	OK
Strength Ib	2.19	12.14	0.400	1.00	4.86	0.92	5.78	2.64	OK
Strength IV	1.35	10.36	0.400	1.00	4.14	0.92	5.06	3.75	OK
Extreme I	1.23	7.31	0.450	1.00	3.29	0.92	4.21	3.43	OK
Service I	1.38	8.76	0.400	1.00	3.50	0.92	4.42	3.20	OK



Retaining Wall Design Report

Structural Design

Foundation Design Moments

Load Combo	eb (ft)	qb (ksf)	Lper (ft)	Mtoe (lb-ft/ft)	Location	Mheel (lb-ft/ft)	Location	Vtoe (lb/ft)	Vheel (lb/ft)
Strength Ia	0.43	1.03	7.14	359.11	Bottom	-1920.14	Top	718.23	120.83
Strength Ib	0.05	1.54	7.90	553.60	Bottom	-1280.45	Top	1107.20	-275.60
Strength IV	0.13	1.34	7.73	426.56	Bottom	-100.10	Top	853.12	307.28
Extreme I	0.66	1.09	6.69	379.30	Bottom	-3925.38	Top	758.61	-185.87
Service I	0.06	1.11	7.88	388.38	Bottom	-674.35	Top	776.76	-93.45

Foundation Design Equations (Soil Bearing)

Strength Ia

Detailed calculation trail

$$e_b = 0.5 \cdot L_f - (MV_t - M H_t) / V_t = 0.5 \cdot 8.00 - (32788.29 - 6530.32) / 7357.29 = 0.431 \text{ ft}$$

$$L_{per} = L_f - 2 \cdot |e_b| = 8.00 - 2 \cdot 0.43 = 7.138 \text{ ft}$$

$$q_b = V_t / L_{per} = 7357.29 / 7.14 = 1030.73 \text{ psf}$$

Toe-Loaded Condition:

$$M_{u1} = 359.11 \text{ lbs-ft/ft}$$

$$V_{u1} = 718.23 \text{ lbs/ft}$$

$$M_{u2} = -1920.14 \text{ lbs-ft/ft}$$

$$V_{u2} = 120.83 \text{ lbs/ft}$$

Reinforcement location:

Toe $M_{u1} = 359.11 \text{ lb-ft/ft}$ → rebar at the bottom of footing.Heel $M_{u2} = -1920.14 \text{ lb-ft/ft}$ → rebar at the top of footing.Toe — M_{u1} and V_{u1}

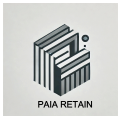
$$\begin{aligned} M_{u1} &= 1030.73 \cdot 1.00^2 / 2 - \\ &0.90 \cdot 1.50 \cdot 1.00^2 / 2 \cdot 150.00 - \\ &1.00 \cdot 110.00 \cdot 1.00 / 2 = 359.11 \text{ lb-ft/ft} \\ V_{u1} &= 1030.73 \cdot 1.00 - 0.90 \cdot 1.50 \cdot 1.00 \cdot 150.00 \\ &- 1.00 \cdot 110.00 = 718.23 \text{ lb/ft} \\ \text{Factored toe soil} &= 1.00 \cdot 110.00 = 110.00 \\ \text{lb/ft; toe-soil moment} &= \\ &1.00 \cdot 110.00 \cdot 1.00 / 2 = 55.00 \text{ lb-ft/ft} \end{aligned}$$

Parameters: $q_b = 1.03 \text{ ksf}$; $L_{per} = 7.14 \text{ ft}$; $l_{toe} = L_f - t - L_e = 1.00 \text{ ft}$; $t_f = h_f = 1.50 \text{ ft}$.

Heel — M_{u2} and V_{u2}

$$\begin{aligned} M_{u2} &= 1030.73 / 2 \cdot (7.14 - (1.00 + 1.00))^2 - \\ &0.90 \cdot 1.50 \cdot 6.00^2 / 2 \cdot 150.00 - \\ &1.00 \cdot 3960.00 \cdot 6.00 / 2 - 0.00 \cdot 1320.00 \cdot 6.00 / 2 \\ &= -1920.14 \text{ lb-ft/ft} \\ V_{u2} &= 1030.73 \cdot (7.14 - (1.00 + 1.00)) - \\ &0.90 \cdot 1.50 \cdot 6.00 \cdot 150.00 - 1.00 \cdot 3960.00 - \\ &0.00 \cdot 1320.00 = 120.83 \text{ lb/ft} \end{aligned}$$

Parameters: $L_e = 6.00 \text{ ft}$; $W_3 = 3960.00 \text{ lb/ft}$; $W_{sur} = 1320.00 \text{ lb/ft}$.



Retaining Wall Design Report

Strength Ib

Detailed calculation trail

$$e_b = 0.5 \cdot L_f - (MV_t - MH_t)/V_t = 0.5 \cdot 8.00 - (54490.04 - 6530.32)/12141.79 = 0.050 \text{ ft}$$

$$L_{per} = L_f - 2 \cdot |e_b| = 8.00 - 2 \cdot 0.05 = 7.900 \text{ ft}$$

$$q_b = V_t / L_{per} = 12141.79 / 7.90 = 1536.95 \text{ psf}$$

Toe-Loaded Condition:

$$Mu_1 = 553.60 \text{ lbs-ft/ft}$$

$$Vu_1 = 1107.20 \text{ lbs/ft}$$

$$Mu_2 = -1280.45 \text{ lbs-ft/ft}$$

$$Vu_2 = -275.60 \text{ lbs/ft}$$

Reinforcement location:

Toe $Mu_1 = 553.60 \text{ lb-ft/ft}$ → rebar at the bottom of footing.Heel $Mu_2 = -1280.45 \text{ lb-ft/ft}$ → rebar at the top of footing.Toe — Mu_1 and Vu_1

$$\begin{aligned} Mu_1 &= 1536.95 \cdot 1.00^2/2 - \\ &1.25 \cdot 1.50 \cdot 1.00^2/2 \cdot 150.00 - \\ &1.35 \cdot 110.00 \cdot 1.00/2 = 553.60 \text{ lb-ft/ft} \\ Vu_1 &= 1536.95 \cdot 1.00 - 1.25 \cdot 1.50 \cdot 1.00 \cdot 150.00 \\ &- 1.35 \cdot 110.00 = 1107.20 \text{ lb/ft} \\ \text{Factored toe soil} &= 1.35 \cdot 110.00 = 148.50 \\ \text{lb/ft; toe-soil moment} &= \\ &1.35 \cdot 110.00 \cdot 1.00/2 = 74.25 \text{ lb-ft/ft} \end{aligned}$$

Parameters: $q_b = 1.54 \text{ ksf}$; $L_{per} = 7.90 \text{ ft}$; $l_{toe} = L_f - t - L_e = 1.00 \text{ ft}$; $t_f = h_f = 1.50 \text{ ft}$.

Heel — Mu_2 and Vu_2

$$\begin{aligned} Mu_2 &= 1536.95/2 \cdot (7.90 - (1.00 + 1.00))^2 - \\ &1.25 \cdot 1.50 \cdot 6.00^2/2 \cdot 150.00 - \\ &1.35 \cdot 3960.00 \cdot 6.00/2 - 1.75 \cdot 1320.00 \cdot 6.00/2 \\ &= -1280.45 \text{ lb-ft/ft} \\ Vu_2 &= 1536.95 \cdot (7.90 - (1.00 + 1.00)) - \\ &1.25 \cdot 1.50 \cdot 6.00 \cdot 150.00 - 1.35 \cdot 3960.00 - \\ &1.75 \cdot 1320.00 = -275.60 \text{ lb/ft} \end{aligned}$$

Parameters: $L_e = 6.00 \text{ ft}$; $W_3 = 3960.00 \text{ lb/ft}$; $W_{sur} = 1320.00 \text{ lb/ft}$.

Strength IV

Detailed calculation trail

$$e_b = 0.5 \cdot L_f - (MV_t - MH_t)/V_t = 0.5 \cdot 8.00 - (43425.46 - 3377.75)/10356.53 = 0.133 \text{ ft}$$

$$L_{per} = L_f - 2 \cdot |e_b| = 8.00 - 2 \cdot 0.13 = 7.734 \text{ ft}$$

$$q_b = V_t / L_{per} = 10356.53 / 7.73 = 1339.12 \text{ psf}$$

Toe-Loaded Condition:

$$Mu_1 = 426.56 \text{ lbs-ft/ft}$$

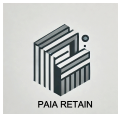
$$Vu_1 = 853.12 \text{ lbs/ft}$$

$$Mu_2 = -100.10 \text{ lbs-ft/ft}$$

$$Vu_2 = 307.28 \text{ lbs/ft}$$

Reinforcement location:

Toe $Mu_1 = 426.56 \text{ lb-ft/ft}$ → rebar at the bottom of footing.Heel $Mu_2 = -100.10 \text{ lb-ft/ft}$ → rebar at the top of footing.



Retaining Wall Design Report

Toe — Mu1 and Vu1

$$\begin{aligned} \text{Mu1} &= 1339.12 \cdot 1.00^2 / 2 - 1.50 \cdot 1.50 \cdot 1.00^2 / 2 \cdot 150.00 - 1.35 \cdot 110.00 \cdot 1.00 / 2 = 426.56 \text{ lb-ft/ft} \\ \text{Vu1} &= 1339.12 \cdot 1.00 - 1.50 \cdot 1.50 \cdot 1.00 \cdot 150.00 - 1.35 \cdot 110.00 = 853.12 \text{ lb/ft} \\ \text{Factored toe soil} &= 1.35 \cdot 110.00 = 148.50 \text{ lb/ft; toe-soil moment} = 1.35 \cdot 110.00 \cdot 1.00 / 2 = 74.25 \text{ lb-ft/ft} \end{aligned}$$

Parameters: $q_b = 1.34 \text{ ksf}$; $L_{\text{per}} = 7.73 \text{ ft}$; $l_{\text{toe}} = L_f - t - L_e = 1.00 \text{ ft}$; $t_f = h_f = 1.50 \text{ ft}$.

Heel — Mu2 and Vu2

$$\begin{aligned} \text{Mu2} &= 1339.12 / 2 \cdot (7.73 - (1.00 + 1.00))^2 - 1.50 \cdot 1.50 \cdot 6.00^2 / 2 \cdot 150.00 - 1.35 \cdot 3960.00 \cdot 6.00 / 2 - 0.00 \cdot 1320.00 \cdot 6.00 / 2 = -100.10 \text{ lb-ft/ft} \\ \text{Vu2} &= 1339.12 \cdot (7.73 - (1.00 + 1.00)) - 1.50 \cdot 1.50 \cdot 6.00 \cdot 150.00 - 1.35 \cdot 3960.00 - 0.00 \cdot 1320.00 = 307.28 \text{ lb/ft} \end{aligned}$$

Parameters: $L_e = 6.00 \text{ ft}$; $W_3 = 3960.00 \text{ lb/ft}$; $W_{\text{sur}} = 1320.00 \text{ lb/ft}$.

Extreme I**Detailed calculation trail**

$$\begin{aligned} e_b &= 0.5 \cdot L_f - (M_{Vt} - M_{Ht}) / V_t = 0.5 \cdot 8.00 - (30935.81 - 6495.71) / 7311.35 = 0.657 \text{ ft} \\ L_{\text{per}} &= L_f - 2 \cdot |e_b| = 8.00 - 2 \cdot 0.66 = 6.686 \text{ ft} \\ q_b &= V_t / L_{\text{per}} = 7311.35 / 6.69 = 1093.61 \text{ psf} \\ \text{Toe-Loaded Condition:} \\ \text{Mu1} &= 379.30 \text{ lbs-ft/ft} \\ \text{Vu1} &= 758.61 \text{ lbs/ft} \\ \text{Mu2} &= -3925.38 \text{ lbs-ft/ft} \\ \text{Vu2} &= -185.87 \text{ lbs/ft} \end{aligned}$$

Reinforcement location:

Toe Mu1 = 379.30 lb-ft/ft → rebar at the bottom of footing.
Heel Mu2 = -3925.38 lb-ft/ft → rebar at the top of footing.

Toe — Mu1 and Vu1

$$\begin{aligned} \text{Mu1} &= 1093.61 \cdot 1.00^2 / 2 - 1.00 \cdot 1.50 \cdot 1.00^2 / 2 \cdot 150.00 - 1.00 \cdot 110.00 \cdot 1.00 / 2 = 379.30 \text{ lb-ft/ft} \\ \text{Vu1} &= 1093.61 \cdot 1.00 - 1.00 \cdot 1.50 \cdot 1.00 \cdot 150.00 - 1.00 \cdot 110.00 = 758.61 \text{ lb/ft} \\ \text{Factored toe soil} &= 1.00 \cdot 110.00 = 110.00 \text{ lb/ft; toe-soil moment} = 1.00 \cdot 110.00 \cdot 1.00 / 2 = 55.00 \text{ lb-ft/ft} \end{aligned}$$

Parameters: $q_b = 1.09 \text{ ksf}$; $L_{\text{per}} = 6.69 \text{ ft}$; $l_{\text{toe}} = L_f - t - L_e = 1.00 \text{ ft}$; $t_f = h_f = 1.50 \text{ ft}$.

Heel — Mu2 and Vu2

$$\begin{aligned} \text{Mu2} &= 1093.61 / 2 \cdot (6.69 - (1.00 + 1.00))^2 - 1.00 \cdot 1.50 \cdot 6.00^2 / 2 \cdot 150.00 - 1.00 \cdot 3960.00 \cdot 6.00 / 2 - 0.00 \cdot 1320.00 \cdot 6.00 / 2 = -3925.38 \text{ lb-ft/ft} \\ \text{Vu2} &= 1093.61 \cdot (6.69 - (1.00 + 1.00)) - 1.00 \cdot 1.50 \cdot 6.00 \cdot 150.00 - 1.00 \cdot 3960.00 - 0.00 \cdot 1320.00 = -185.87 \text{ lb/ft} \end{aligned}$$

Parameters: $L_e = 6.00 \text{ ft}$; $W_3 = 3960.00 \text{ lb/ft}$; $W_{\text{sur}} = 1320.00 \text{ lb/ft}$.

Service I**Detailed calculation trail**



Retaining Wall Design Report

$$e_b = 0.5 \cdot L_f - (MV_t - M H_t)/V_t = 0.5 \cdot 8.00 - (38565.57 - 4053.30)/8760.07 = 0.060 \text{ ft}$$

$$L_{per} = L_f - 2 \cdot |e_b| = 8.00 - 2 \cdot 0.06 = 7.879 \text{ ft}$$

$$q_b = V_t / L_{per} = 8760.07 / 7.88 = 1111.76 \text{ psf}$$

Toe-Loaded Condition:

$$Mu_1 = 388.38 \text{ lbs-ft/ft}$$

$$Vu_1 = 776.76 \text{ lbs/ft}$$

$$Mu_2 = -674.35 \text{ lbs-ft/ft}$$

$$Vu_2 = -93.45 \text{ lbs/ft}$$

Reinforcement location:

Toe $Mu_1 = 388.38 \text{ lb-ft/ft}$ → rebar at the bottom of footing.Heel $Mu_2 = -674.35 \text{ lb-ft/ft}$ → rebar at the top of footing.**Toe — Mu_1 and Vu_1**

$$Mu_1 = 1111.76 \cdot 1.00^2/2 -$$

$$1.00 \cdot 1.50 \cdot 1.00^2/2 \cdot 150.00 -$$

$$1.00 \cdot 110.00 \cdot 1.00/2 = 388.38 \text{ lb-ft/ft}$$

$$Vu_1 = 1111.76 \cdot 1.00 - 1.00 \cdot 1.50 \cdot 1.00 \cdot 150.00$$

$$- 1.00 \cdot 110.00 = 776.76 \text{ lb/ft}$$

$$\text{Factored toe soil} = 1.00 \cdot 110.00 = 110.00$$

$$\text{lb/ft; toe-soil moment} =$$

$$1.00 \cdot 110.00 \cdot 1.00/2 = 55.00 \text{ lb-ft/ft}$$

Parameters: $q_b = 1.11 \text{ ksf}$; $L_{per} = 7.88 \text{ ft}$; $l_{toe} = L_f - t - L_e = 1.00 \text{ ft}$; $t_f = h_f = 1.50 \text{ ft}$.

Heel — Mu_2 and Vu_2

$$Mu_2 = 1111.76/2 \cdot (7.88 - (1.00 + 1.00))^2 -$$

$$1.00 \cdot 1.50 \cdot 6.00^2/2 \cdot 150.00 -$$

$$1.00 \cdot 3960.00 \cdot 6.00/2 - 1.00 \cdot 1320.00 \cdot 6.00/2$$

$$= -674.35 \text{ lb-ft/ft}$$

$$Vu_2 = 1111.76 \cdot (7.88 - (1.00 + 1.00)) -$$

$$1.00 \cdot 1.50 \cdot 6.00 \cdot 150.00 - 1.00 \cdot 3960.00 -$$

$$1.00 \cdot 1320.00 = -93.45 \text{ lb/ft}$$

Parameters: $L_e = 6.00 \text{ ft}$; $W_3 = 3960.00 \text{ lb/ft}$; $W_{sur} = 1320.00 \text{ lb/ft}$.

Maximum Foundation Moments & Shears

Type	Max Value	Load Combo	Rebar Location
Toe Moment (Mu_1)	553.60 lbs-ft/ft	Strength Ib	Bottom
Heel Moment (Mu_2)	3925.38 lbs-ft/ft	Extreme I	Top
Toe Shear (Vu_1)	1107.20 lbs/ft	Strength Ib	—
Heel Shear (Vu_2)	307.28 lbs/ft	Strength IV	—

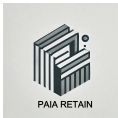
Toe Flexural Design — Step-by-Step**1) Effective Depth (d)**

$$d = h - c_{lr} - 0.5 \cdot d_b$$

$$= 18.00 - 3.00 - 0.5 \times 0.505$$

$$= 14.748$$

$$\mathbf{d = 14.750 \text{ in}}$$



2) Steel Area (A_s)

$$\begin{aligned}A_s &= n \cdot A_b \\&= 1.00 \times 0.200 \\&= 0.200\end{aligned}$$

$$A_s = 0.200 \text{ in}^2$$

3) Whitney Stress Block & β_1

$$\begin{aligned}\beta_1 &= 0.850 \\a &= (A_s \cdot f_y) / (0.85 \cdot f'_c \cdot b) \\&= (0.200 \times 60000.00) / (0.85 \times 4000.00 \times 12.00) \\&= 0.294\end{aligned}$$

$$a = 0.294 \text{ in}$$

$$\begin{aligned}c &= a / \beta_1 \\&= 0.294 / 0.850 \\&= 0.346\end{aligned}$$

$$c = 0.346 \text{ in}$$

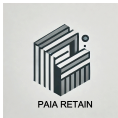
$$\begin{aligned}\epsilon_t &= 0.003 \cdot (d - c) / c \\&= 0.003 \cdot (14.750 - 0.346) / 0.346 \\&= 0.125\end{aligned}$$

$$\epsilon_t = 0.125$$

4) Nominal Flexural Strength (M_n)

$$\begin{aligned}M_n &= A_s \cdot f_y \cdot (d - a/2) \\&= 0.200 \times 60000.00 \times (14.750 - 0.147) \\&= 14.603\end{aligned}$$

$$M_n = 14.603 \text{ kip-ft}$$

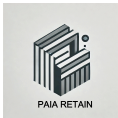


5) Strength Reduction Factor (ϕ)

$$\phi = 0.900$$

$$\phi M_n = \phi \times M_n = 0.900 \times 14.603$$

$$\phi M_n = 13.143 \text{ kip-ft}$$



6) Cracking Moment (M_{cr})

$$S = b \cdot h^2 / 6$$

$$= 12.00 \cdot (18.00^2) / 6$$

$$= 648.00 \text{ in}^3$$

$$f_r = 7.5 \cdot \sqrt{f'_c} \text{ (psi)}$$

$$= 7.5 \cdot \sqrt{4000.00}$$

$$= 474.34 \text{ psi (= 0.474 ksi)}$$

$$M_{cr} = \gamma_1 \cdot \gamma_3 \cdot f_r \cdot S$$

$$= 1.60 \cdot 0.67 \cdot 474.34 \cdot 648.00$$

$$= 329504.27 \text{ lb-in}$$

$$M_{cr}(\text{kip-ft}) = M_{cr}(\text{lb-in}) / (1000 \cdot 12)$$

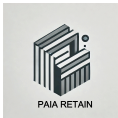
$$= 329504.27 / 12000$$

$$= 27.459 \text{ kip-ft}$$

$$M_{min} = \min(M_{cr}, 1.33 \cdot M_u)$$

$$1.33 \cdot M_u = 1.33 \cdot 0.554 = 0.736 \text{ kip-ft}$$

$$\mathbf{M_{cr} = 27.459 \text{ kip-ft; } M_{min} = 0.736 \text{ kip-ft}}$$



Toe — Flexural Crack-Control and Detailing

$$d_c = c_{lr} + 0.5 \cdot d_b = 3.25 \text{ in}$$

$$s_{\max, \text{crack}} = (700 \cdot \gamma_e) / (\beta_s \cdot f_{ss}) - 2 \cdot d_c$$

Crack-control upper bound = 316.74 in

Actual reinforcement spacing = 12.00 in c/c

Clear bar spacing = 11.50 in

PASS — actual spacing $\leq$ crack-control upper bound

$$A_{s, \text{shrink/temp}} = 0.110 \text{ in}^2/\text{ft}$$

Article 5.10.6 maximum spacing = 18.00 in

The crack-control upper bound is not the selected reinforcement spacing. Article 5.10.6 shrinkage/temperature reinforcement is a separate requirement. Cover is assumed to be measured to the main flexural bars.

7) Utilization

$$U = M_u / (\phi M_n)$$

$$= 0.554 / 13.143$$

$$= 0.042$$

OK — Section adequate

Heel Flexural Design — Step-By-Step

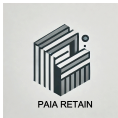
1) Effective Depth (d)

$$d = h - c_{lr} - 0.5 \cdot d_b$$

$$= 18.00 - 3.00 - 0.5 \times 0.500$$

$$= 14.750$$

d = 14.750 in



2) Steel Area (A_s)

$$\begin{aligned}A_s &= n \cdot A_b \\&= 1.00 \times 0.200 \\&= 0.200\end{aligned}$$

$$A_s = 0.200 \text{ in}^2$$

3) Whitney Block & β_1

$$\beta_1 = 0.85 - 0.05(f_c' - 4.0), \text{ min } 0.65$$

$$\beta_1 = 0.850$$

$$\begin{aligned}a &= (A_s \cdot f_y) / (0.85 \cdot f_c' \cdot b) \\&= (0.200 \times 60000.00) / (0.85 \times 4000.00 \times 12.00) \\&= 0.294\end{aligned}$$

$$a = 0.294 \text{ in}$$

$$\begin{aligned}c &= a / \beta_1 \\&= 0.294 / 0.850 \\&= 0.346\end{aligned}$$

$$c = 0.346 \text{ in}$$

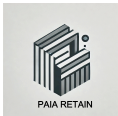
$$\begin{aligned}\epsilon_t &= 0.003 \cdot (d - c) / c \\&= 0.003 \cdot (14.750 - 0.346) / 0.346 \\&= 0.125\end{aligned}$$

$$\epsilon_t = 0.125$$

4) Nominal Flexural Strength (M_n)

$$\begin{aligned}M_n &= A_s \cdot f_y \cdot (d - a/2) \\&= 0.200 \times 60000.00 \times (14.750 - 0.147) \\&= 14.603\end{aligned}$$

$$M_n = 14.603 \text{ kip-ft}$$



6) Cracking Moment (M_{cr})

$$S = b \cdot h^2 / 6$$

$$= 12.00 \cdot (18.00^2) / 6$$

$$= 648.00 \text{ in}^3$$

$$f_r = 7.5 \cdot \sqrt{f'_c} \text{ (psi)}$$

$$= 7.5 \cdot \sqrt{4000.00}$$

$$= 474.34 \text{ psi (= 0.474 ksi)}$$

$$M_{cr} = \gamma_1 \cdot \gamma_3 \cdot f_r \cdot S$$

$$= 1.60 \cdot 0.67 \cdot 474.34 \cdot 648.00$$

$$= 329504.27 \text{ lb-in}$$

$$M_{cr}(\text{kip-ft}) = M_{cr}(\text{lb-in}) / (1000 \cdot 12)$$

$$= 329504.27 / 12000$$

$$= 27.459 \text{ kip-ft}$$

$$M_{min} = \min(M_{cr}, 1.33 \cdot M_u)$$

$$1.33 \cdot M_u = 1.33 \cdot 3.925 = 5.221 \text{ kip-ft}$$

$$\mathbf{M_{cr} = 27.459 \text{ kip-ft; } M_{min} = 5.221 \text{ kip-ft}}$$

Heel — Flexural Crack-Control and Detailing

$$d_c = c_{lr} + 0.5 \cdot d_b = 3.25 \text{ in}$$

$$s_{\max, \text{crack}} = (700 \cdot \gamma_e) / (\beta_s \cdot f_{ss}) - 2 \cdot d_c$$

Crack-control upper bound = 179.67 in

Actual reinforcement spacing = 12.00 in c/c

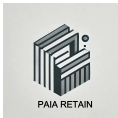
Clear bar spacing = 11.50 in

PASS — actual spacing $\leq$ crack-control upper bound

$$A_{s, \text{shrink/temp}} = 0.110 \text{ in}^2/\text{ft}$$

Article 5.10.6 maximum spacing = 18.00 in

The crack-control upper bound is not the selected reinforcement spacing. Article 5.10.6 shrinkage/temperature reinforcement is a separate requirement. Cover is assumed to be measured to the main flexural bars.



7) Minimum Strength Requirement

$$\phi M_n \geq M_{\min} = \min(M_{cr}, 1.33 \cdot M_u)$$

$$M_{\min} = \min(27.459, 5.221) = 5.221$$

$$\phi M_n = 13.143 \text{ kip-ft}$$

$$M_{cr} = 27.459 \text{ kip-ft}$$

$$M_{\min} = 5.221 \text{ kip-ft}$$

$$1.33 \cdot M_u = 5.221 \text{ kip-ft}$$

8) Utilization

$$U = M_u / (\phi M_n)$$

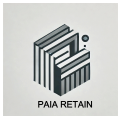
$$= 3.925 / 13.143$$

$$= 0.299$$

$$U = 0.299$$

Result

OK — Heel flexure capacity adequate.



Toe Shear Design — Step-by-Step

1) Compute a , β_1 , c

$$\begin{aligned} a &= (A_s \cdot f_y) / (0.85 f'_c b) \\ &= (0.200 \times 60000.00) / (0.85 \times 4000.00 \times 12.00) \\ &= 0.294 \end{aligned}$$

$$a = 0.294 \text{ in}$$

$$\beta_1 = 0.85 (f'_c \leq 4 \text{ ksi}) \text{ else } \max(0.65, 0.85 - 0.05(f'_c - 4))$$

$$f'_c = 4.00 \text{ ksi} \rightarrow \beta_1 = 0.850$$

$$\beta_1 = 0.850$$

$$\begin{aligned} c &= a / \beta_1 \\ &= 0.294 / 0.850 \\ &= 0.346 \end{aligned}$$

$$c = 0.346 \text{ in}$$

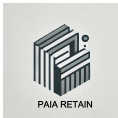
2) Compute d_c and d_v

$$\begin{aligned} d_c &= h - c_{lr} \\ &= 18.00 - 3.00 \\ &= 15.000 \end{aligned}$$

$$d_c = 15.000 \text{ in}$$

$$\begin{aligned} d_v &= \max(0.9 d, 0.72 h, d_c - a/2) \\ &= \max(0.9 \times 14.75, 0.72 \times 18.00, 15.000 - 0.147) \\ &= \max(13.275, 12.960, 14.853) \end{aligned}$$

$$d_v = 14.853 \text{ in}$$



3) Longitudinal strain ϵ_s (MCFT logic) & θ

$$M_u(\text{lb-in}) = |M_u(\text{kip-ft})| \cdot 1000 \cdot 12$$

$$= |0.55| \times 12000 = 6643.17$$

$$V_u(\text{lb}) = |V_u(\text{kip})| \cdot 1000$$

$$= |1.11| \times 1000 = 1107.20$$

$$\epsilon_{s, \text{trial}} = (M_u/d_v + V_u) / (E_s \cdot 1000 \cdot A_s)$$

$$= (6643.17/14.853 + 1107.20) / (29000.00 \times 1000 \times 0.200)$$

$$\epsilon_{s, \text{trial}} = 0.000$$

$$\text{Tension case: denom} = E_s \cdot 1000 \cdot A_s$$

$$= 29000.00 \times 1000 \times 0.200$$

$$\text{denom} = 5800000.00$$

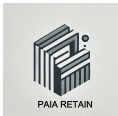
$$\epsilon_s = \text{clamp}(|\epsilon_s|, 0, 0.006)$$

$$\epsilon_s = 0.000$$

$$\theta(\text{deg}) = 29 + 3500 \cdot \epsilon_s$$

$$= 29 + 3500 \times 0.000 = 29.94$$

$$\theta = 29.94^\circ$$



4) β (base) and β (final path selection)

$$\begin{aligned}\beta_{\text{base}} &= 4.8 / (1 + 750 \cdot \epsilon_s) \\ &= 4.8 / (1 + 750 \times 0.000) = 3.997\end{aligned}$$

$$\beta_{\text{base}} = 3.997$$

$$V_{c,\text{trial}} = b \cdot d_v \cdot 0.0316 \cdot \beta_{\text{base}} \cdot \sqrt{f'_c} \quad (\text{with } f'_c \text{ in ksi})$$

$$\phi V_{c,\text{trial}} = \phi \cdot V_{c,\text{trial}}, \text{ with } \phi = 0.90$$

$$\phi V_{c,\text{trial}} = 40.52$$

$$\text{Requires minimum transverse reinforcement?} \quad V_u > 0.5 \cdot \phi V_{c,\text{trial}}$$

$$1.11 > 0.5 \times 40.52 \rightarrow \text{NO}$$

$$\begin{aligned}A_{v,\text{min}} &= 0.0316 \cdot \sqrt{f'_c} \cdot s \cdot b / f_y \\ &= 0.0316 \times \sqrt{4.00} \times 0.00 \times 12.00 / 60.00\end{aligned}$$

$$A_{v,\text{min}} = 0.000 \text{ in}^2$$

$$\text{Path B } (A_v \leq A_{v,\text{min}}): s_x = \min(d_v, s)$$

$$= \min(14.853, 0.00) = 14.853$$

$$s_{x\text{e,raw}} = s_x \cdot (1.38 / (0.63 + a_g)) \quad a_g = 0.75$$

$$= 14.853 \times (1.38 / 1.38) = 14.853$$

$$s_{x\text{e}} = \text{clamp}(s_{x\text{e,raw}}, 12, 80)$$

$$= \text{clamp}(14.853, 12, 80) = 14.853$$

$$\beta = \beta_{\text{base}} \cdot (51 / (39 + s_{x\text{e}}))$$

$$= 3.997 \times (51 / (39 + 14.853)) = 3.785$$

$$\beta = 3.785$$

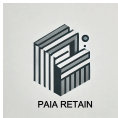
5) Concrete Shear Contribution

$$V_c = b \cdot d_v \cdot 0.0316 \cdot \beta \cdot \sqrt{f'_c} \quad (f'_c \text{ in ksi})$$

$$\phi V_c = \phi \cdot V_c, \text{ with } \phi = 0.90$$

$$= 0.90 \times (12.00 \times 14.853 \times 0.0316 \times 3.785 \times \sqrt{4.00})$$

$$\phi V_c = 38.37 \text{ kip}$$



6) Stirrup Shear Contribution

No transverse reinforcement provided: $A_v = 0$, therefore $V_s = 0$.

$$\phi V_s = 0.00 \text{ kip}$$

7) Total Shear Capacity

$$\phi V_n = \phi V_c + \phi V_s$$

$$= 38.37 + 0.00$$

$$\phi V_n = 38.37 \text{ kip}$$

8) Shear Demand

$$V_u = 1.11 \text{ kip}$$

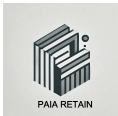
9) Utilization

$$U = V_u / \phi V_n$$

$$= 1.11 / 38.37$$

$$U = 0.029$$

OK — $\phi V_n \geq V_u$



Heel Shear Design — Step-by-Step

1) Compute a , β_1 , c

$$\begin{aligned} a &= (A_s \cdot f_y) / (0.85 f'_c b) \\ &= (0.200 \times 60000.00) / (0.85 \times 4000.00 \times 12.00) \\ &= 0.294 \end{aligned}$$

$$a = 0.294 \text{ in}$$

$$\beta_1 = 0.85 (f'_c \leq 4 \text{ ksi}) \text{ else } \max(0.65, 0.85 - 0.05(f'_c - 4))$$

$$f'_c = 4.00 \text{ ksi} \rightarrow \beta_1 = 0.850$$

$$\beta_1 = 0.850$$

$$\begin{aligned} c &= a / \beta_1 \\ &= 0.294 / 0.850 \\ &= 0.346 \end{aligned}$$

$$c = 0.346 \text{ in}$$

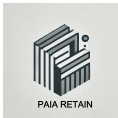
2) Compute d_c and d_v

$$\begin{aligned} d_c &= h - c_{lr} \\ &= 18.00 - 3.00 \\ &= 15.000 \end{aligned}$$

$$d_c = 15.000 \text{ in}$$

$$\begin{aligned} d_v &= \max(0.9 d, 0.72 h, d_c - a/2) \\ &= \max(0.9 \times 14.75, 0.72 \times 18.00, 15.000 - 0.147) \\ &= \max(13.275, 12.960, 14.853) \end{aligned}$$

$$d_v = 14.853 \text{ in}$$



3) Longitudinal strain ϵ_s (MCFT logic) & θ

$$M_u(\text{lb-in}) = |M_u(\text{kip-ft})| \cdot 1000 \cdot 12$$

$$= |3.93| \times 12000 = 47104.61$$

$$V_u(\text{lb}) = |V_u(\text{kip})| \cdot 1000$$

$$= |0.31| \times 1000 = 307.28$$

$$\epsilon_{s, \text{trial}} = (M_u/d_v + V_u) / (E_s \cdot 1000 \cdot A_s)$$

$$= (47104.61/14.853 + 307.28) / (29000.00 \times 1000 \times 0.200)$$

$$\epsilon_{s, \text{trial}} = 0.001$$

$$\text{Tension case denom} = E_s \cdot 1000 \cdot A_s$$

$$= 29000.00 \times 1000 \times 0.200$$

$$\text{denom} = 5800000.00$$

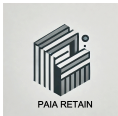
$$\epsilon_s = \text{clamp}(|\epsilon_s|, 0, 0.006)$$

$$\epsilon_s = 0.001$$

$$\theta(\text{deg}) = 29 + 3500 \cdot \epsilon_s$$

$$= 29 + 3500 \times 0.001 = 31.10$$

$$\theta = 31.10^\circ$$



4) β (base) and β (final path selection)

$$\begin{aligned}\beta_{\text{base}} &= 4.8 / (1 + 750 \cdot \epsilon_s) \\ &= 4.8 / (1 + 750 \times 0.001) = 3.311\end{aligned}$$

$$\beta_{\text{base}} = 3.311$$

$$V_{c,\text{trial}} = b \cdot d_v \cdot 0.0316 \cdot \beta_{\text{base}} \cdot \sqrt{f'_c} \quad (f'_c \text{ in ksi})$$

$$\phi V_{c,\text{trial}} = \phi \cdot V_{c,\text{trial}}, \text{ with } \phi = 0.90$$

$$\phi V_{c,\text{trial}} = 33.56$$

$$\text{Requires minimum transverse reinforcement?} \quad V_u > 0.5 \cdot \phi V_{c,\text{trial}}$$

$$0.31 > 0.5 \times 33.56 \rightarrow \text{NO}$$

$$\begin{aligned}A_{v,\text{min}} &= 0.0316 \cdot \sqrt{f'_c} \cdot s \cdot b / f_y \\ &= 0.0316 \times \sqrt{4.00} \times 0.00 \times 12.00 / 60.00\end{aligned}$$

$$A_{v,\text{min}} = 0.000 \text{ in}^2$$

$$\text{Path B } (A_v \leq A_{v,\text{min}}): s_x = \min(d_v, s)$$

$$= \min(14.853, 0.00) = 14.853$$

$$s_{xe,\text{raw}} = s_x \cdot (1.38 / (0.63 + 0.75))$$

$$= 14.853 \times (1.38 / 1.38) = 14.853$$

$$s_{xe} = \text{clamp}(s_{xe,\text{raw}}, 12, 80)$$

$$= \text{clamp}(14.853, 12, 80) = 14.853$$

$$\beta = \beta_{\text{base}} \cdot (51 / (39 + s_{xe}))$$

$$= 3.311 \times (51 / (39 + 14.853)) = 3.135$$

$$\beta = 3.135$$

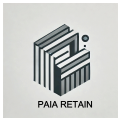
5) Concrete Shear Contribution

$$V_c = b \cdot d_v \cdot 0.0316 \cdot \beta \cdot \sqrt{f'_c} \quad (f'_c \text{ in ksi})$$

$$\phi V_c = \phi \cdot V_c, \text{ with } \phi = 0.90$$

$$= 0.90 \times (12.00 \times 14.853 \times 0.0316 \times 3.135 \times \sqrt{4.00})$$

$$\phi V_c = 31.79 \text{ kip}$$



6) Stirrup Shear Contribution

No transverse reinforcement provided: $A_v = 0$, therefore $V_s = 0$.

$$\phi V_s = 0.00 \text{ kip}$$

7) Total Shear Capacity

$$\phi V_n = \phi V_c + \phi V_s$$

$$= 31.79 + 0.00$$

$$\phi V_n = 31.79 \text{ kip}$$

8) Shear Demand

$$V_u = 0.31 \text{ kip}$$

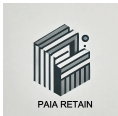
9) Utilization

$$U = V_u / \phi V_n$$

$$= 0.31 / 31.79$$

$$U = 0.010$$

OK — $\phi V_n \geq V_u$



Wall Lateral Load Design — Step-by-Step

1) Structural Stem Retained Height (H_{stem})

$$\begin{aligned} H_{\text{stem}} &= H + \tan(i) \cdot L_e \\ &= 6.00 + \tan(0.00^\circ) \cdot 6.00 \\ &= 6.00 \end{aligned}$$

$$H_{\text{stem}} = 6.00 \text{ ft}$$

Stem flexural and shear design considers lateral pressure acting above the top of footing; external stability uses H_{stab} through the footing depth.

2) Active Earth Pressure Resultant

Coulomb components use $\delta = 15.00^\circ$

$$\begin{aligned} H_{a, \text{total}} &= P_a \cdot H_{\text{eff}}^2 / 2 \\ &= 33.16 \cdot (6.00^2) / 2 \\ &= 576.47 \end{aligned}$$

$$H_a = 576.47 \text{ lb/ft}$$

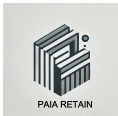
$$H_a = H_{a, \text{total}} \cdot \cos(\delta)$$

$$H_{av} = H_{a, \text{total}} \cdot \sin(\delta)$$

$$H_{av} = 0.00 \text{ lb/ft}$$

$$\begin{aligned} M_{h1} &= H_a \cdot (H_{\text{eff}}/3) \\ &= 576.47 \cdot (6.00/3) \\ &= 1152.94 \end{aligned}$$

$$M_{h1} = 1152.94 \text{ lb-ft/ft}$$



3) Surcharge Resultant

Coulomb components use $\delta = 15.00^\circ$

$$\begin{aligned}H_{\text{sur,total}} &= k \cdot \gamma \cdot H_e \cdot H_{\text{eff}} \\&= 0.301 \cdot 110.00 \cdot 2.00 \cdot 6.00 \\&= 384.31\end{aligned}$$

$$H_{\text{sur}} = 384.31 \text{ lb/ft}$$

$$H_{\text{sur}} = H_{\text{sur,total}} \cdot \cos(\delta)$$

$$H_{\text{surv}} = H_{\text{sur,total}} \cdot \sin(\delta)$$

$$H_{\text{surv}} = 0.00 \text{ lb/ft}$$

$$\begin{aligned}M_{h2} &= H_{\text{sur}} \cdot (H_{\text{eff}}/2) \\&= 384.31 \cdot (6.00/2) \\&= 1152.94\end{aligned}$$

$$M_{h2} = 1152.94 \text{ lb-ft/ft}$$

4) Seismic Earth Pressure Resultant

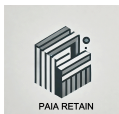
Method: Mononobe-Okabe (M-O)

$$\begin{aligned}H_{\text{eq}} &= 0.5 \cdot K_E \cdot \gamma \cdot H_{\text{eff}}^2 \\&= 0.5 \cdot 0.106 \cdot 110.00 \cdot (6.00^2) \\&= 209.73\end{aligned}$$

$$H_{\text{eq}} = 209.73 \text{ lb/ft}$$

$$\begin{aligned}M_{h4} &= H_{\text{eq}}(\text{SeisH} \cdot H_{\text{eff}}) + k_h W_1(0.5(H+H_t)) + k_h W_3(H + \tan(i)L_e) \cdot 0.5 \\&= 209.73 \cdot (0.500 \cdot 6.00) + 0.150 \cdot 1050.00 \cdot (0.5 \cdot (6.00+1.00)) + \\&\quad 0.150 \cdot 3960.00 \cdot (6.00 + \tan(0.00^\circ) \cdot 6.00) \cdot 0.5 \\&= 2962.43\end{aligned}$$

$$M_{h4} = 2962.43 \text{ lb-ft/ft}$$



5) Factored Shear & Moment by Load Combination

$$H_{\text{factored}} = Y_{EH}H_1 + Y_{LS,H}(H_2+H_3) + Y_{CT}H_4$$

$$M_{\text{factored}} = Y_{EH}M_{h1} + Y_{LS,H}M_{h2} + Y_{CT}M_{h4}$$

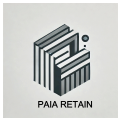
Combo	$Y_{LS,H}$	Y_{EH}	H_{factored} (lb/ft)	M_{factored} (lb-ft/ft)
Strength Ia	1.75	1.50	1537.25	3747.05
$H_{\text{factored}} = 1.50 \cdot 576.47 + 1.75 \cdot (384.31 + 0.00) + 0.00 \cdot 209.73 = 1537.25$ $M_{\text{factored}} = 1.50 \cdot 1152.94 + 1.75 \cdot 1152.94 + 0.00 \cdot 2962.43 = 3747.05$				
Strength Ib	1.75	1.50	1537.25	3747.05
$H_{\text{factored}} = 1.50 \cdot 576.47 + 1.75 \cdot (384.31 + 0.00) + 0.00 \cdot 209.73 = 1537.25$ $M_{\text{factored}} = 1.50 \cdot 1152.94 + 1.75 \cdot 1152.94 + 0.00 \cdot 2962.43 = 3747.05$				
Strength IV	0.00	1.50	864.70	1729.41
$H_{\text{factored}} = 1.50 \cdot 576.47 + 0.00 \cdot (384.31 + 0.00) + 0.00 \cdot 209.73 = 864.70$ $M_{\text{factored}} = 1.50 \cdot 1152.94 + 0.00 \cdot 1152.94 + 0.00 \cdot 2962.43 = 1729.41$				
Extreme I	0.00	1.00	786.20	4115.37
$H_{\text{factored}} = 1.00 \cdot 576.47 + 0.00 \cdot (384.31 + 0.00) + 1.00 \cdot 209.73 = 786.20$ $M_{\text{factored}} = 1.00 \cdot 1152.94 + 0.00 \cdot 1152.94 + 1.00 \cdot 2962.43 = 4115.37$				
Service I	1.00	1.00	960.78	2305.88
$H_{\text{factored}} = 1.00 \cdot 576.47 + 1.00 \cdot (384.31 + 0.00) + 0.00 \cdot 209.73 = 960.78$ $M_{\text{factored}} = 1.00 \cdot 1152.94 + 1.00 \cdot 1152.94 + 0.00 \cdot 2962.43 = 2305.88$				

6) Governing Load Combination

Governing Combo: Extreme I

Max Shear = 786.20 lb/ft

Max Moment = 4115.37 lb-ft/ft



Wall Flexural Design — Step-by-Step

0) Wall Moment Demand

$$M_u \text{ (lb-ft/ft)} = 4115.37$$

$$M_u \text{ (kip-ft/ft)} = 4115.37 / 1000 = 4.115$$

$$M_u \text{ (kip-in/ft)} = 4.115 \times 12 = 49.384$$

$$M_s \text{ (Service I lb-ft/ft)} = 2305.88$$

$$M_s \text{ (kip-ft/ft)} = 2305.88 / 1000 = 2.306$$

$$M_s \text{ (kip-in/ft)} = 2.306 \times 12 = 27.671$$

1) Effective Depth (d)

$$d = h - \text{clr} - 0.5 \cdot d_b$$

$$= 12.00 - 2.00 - 0.5 \times 0.500$$

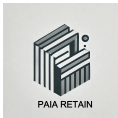
$$d = 9.750 \text{ in}$$

2) Steel Area (A_s)

$$A_s = n \cdot A_b$$

$$= 1.00 \times 0.200 = 0.200$$

$$A_s = 0.200 \text{ in}^2$$



3) Whitney Stress Block, Strain, and ϕ

$$\beta_1 = 0.850$$

$$a = (A_s \cdot f_y) / (0.85 \cdot f'_c \cdot b)$$

$$= (0.200 \times 60000.00) / (0.85 \times 4000.00 \times 12.00)$$

$$a = 0.294 \text{ in}$$

$$c = a / \beta_1$$

$$= 0.294 / 0.850$$

$$c = 0.346 \text{ in}$$

$$\epsilon_t = 0.003 \cdot (d - c) / c$$

$$= 0.003 \cdot (9.750 - 0.346) / 0.346$$

$$\epsilon_t = 0.082$$

$$\phi = 0.900$$

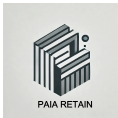
4) Nominal Flexural Strength (M_n)

$$M_n = A_s \cdot f_y \cdot (d - a/2)$$

$$= 0.200 \times 60000.00 \times (9.750 - 0.147)$$

$$M_n = 9.603 \text{ kip-ft}$$

$$\phi M_n = 8.643 \text{ kip-ft}$$



6) Cracking Moment (M_{cr})

$$S = b \cdot h^2 / 6$$

$$= 12.00 \cdot (12.00^2) / 6$$

$$= 288.00 \text{ in}^3$$

$$f_r = 7.5 \cdot \sqrt{f'_c} \text{ (psi)}$$

$$= 7.5 \cdot \sqrt{4000.00}$$

$$= 474.34 \text{ psi (= 0.474 ksi)}$$

$$M_{cr} = \gamma_1 \cdot \gamma_3 \cdot f_r \cdot S$$

$$= 1.60 \cdot 0.67 \cdot 474.34 \cdot 288.00$$

$$= 146446.34 \text{ lb-in}$$

$$M_{cr}(\text{kip-ft}) = M_{cr}(\text{lb-in}) / (1000 \cdot 12)$$

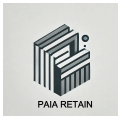
$$= 146446.34 / 12000$$

$$= 12.204 \text{ kip-ft}$$

$$M_{min} = \min(M_{cr}, 1.33 \cdot M_u)$$

$$1.33 \cdot M_u = 1.33 \cdot 4.115 = 5.473 \text{ kip-ft}$$

$$M_{cr} = 12.204 \text{ kip-ft}$$



Stem / Wall — Flexural Crack-Control and Detailing

$$d_c = c_{lr} + 0.5 \cdot d_b = 2.25 \text{ in}$$

$$s_{\max, \text{crack}} = (700 \cdot \gamma_e) / (\beta_s \cdot f_{ss}) - 2 \cdot d_c$$

Crack-control upper bound = 30.76 in

Actual reinforcement spacing = 12.00 in c/c

Clear bar spacing = 11.50 in

PASS — actual spacing $\leq$ crack-control upper bound

$$A_{s, \text{shrink/temp}} = 0.110 \text{ in}^2/\text{ft}$$

Article 5.10.6 maximum spacing = 18.00 in

The crack-control upper bound is not the selected reinforcement spacing. Article 5.10.6 shrinkage/temperature reinforcement is a separate requirement. Cover is assumed to be measured to the main flexural bars.

Wall Vertical Bar Hooked Anchorage — AASHTO LRFD 5.10.8.2.4

Anchorage assumed: Standard 90-degree tension hook (assumed). This check is for wall vertical bars developed into the footing.

$$l_{dh} = \max[(38 d_b / 60)(f_y / \sqrt{f'_c}), 8d_b, 6.0 \text{ in.}]$$

$$d_b = 0.500 \text{ in}; f_y = 60.00 \text{ ksi}; f'_c = 4.00 \text{ ksi}$$

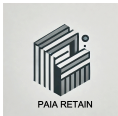
$$\text{Footing thickness} = 18.00 \text{ in}; \text{footing cover} = 3.00 \text{ in}$$

$$\text{Available embedment} = \text{footing thickness} - \text{cover} = 15.00 \text{ in}$$

$$\text{Basic hook length} = 9.50 \text{ in}; 8d_b = 4.00 \text{ in}; \text{required } l_{dh} = 9.50 \text{ in}$$

PASS — hooked anchorage length is adequate.

Available embedment is adequate for the assumed standard 90-degree hook. Standard hook bend diameter, extension, cover, and confinement requirements must also be shown and verified on the final reinforcement drawings.



Wall Horizontal Shrinkage/Temperature Reinforcement — Article 5.10.6

$$\text{Provided } A_s = A_b \cdot 12/s = 0.133 \text{ in}^2/\text{ft}$$

$$\text{Required } A_s = 0.110 \text{ in}^2/\text{ft}$$

$$\text{Provided spacing} = 18.00 \text{ in; maximum} = 18.00 \text{ in}$$

#4 @ 18.00 in horizontal, per applicable face

PASS — horizontal shrinkage/temperature reinforcement

Provide reinforcement on each applicable face and in each direction. This horizontal wall check is separate from the vertical flexural reinforcement check.

Foundation Temperature/Shrinkage Reinforcement — Article 5.10.6

$$\text{Provided } A_s = A_b \cdot 12/s = 0.133 \text{ in}^2/\text{ft}$$

$$\text{Required } A_s = 0.110 \text{ in}^2/\text{ft}$$

$$\text{Provided spacing} = 18.00 \text{ in; maximum} = 18.00 \text{ in}$$

#4 @ 18.00 in foundation reinforcement, per applicable face/direction

PASS — foundation temperature/shrinkage reinforcement

Provide #4 reinforcement on each applicable foundation face and in each applicable direction. This check is separate from toe and heel flexural reinforcement.

7) Minimum Strength Requirement

$$\phi M_n \geq M_{\min} = \min(M_{cr}, 1.33 \cdot M_u)$$

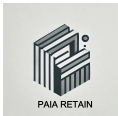
$$M_{\min} = \min(12.204, 5.473) = 5.473$$

$$\phi M_n = 8.643 \text{ kip-ft}$$

$$M_{cr} = 12.204 \text{ kip-ft}$$

$$1.33 \cdot M_u = 5.473 \text{ kip-ft}$$

OK — ϕM_n meets minimum



8) Utilization

$$\begin{aligned} U &= M_u / (\phi M_n) \\ &= 4.115 / 8.643 \\ &= 0.476 \end{aligned}$$

OK — Wall flexure capacity adequate.

Wall Shear Design — Step-by-Step

1) Compute a , β_1 , and c

$$\begin{aligned} a &= (A_s \cdot f_y) / (0.85 f'_c b) \\ &= (0.200 \times 60000.00) / (0.85 \times 4000.00 \times 12.00) \end{aligned}$$

$a = 0.294$ in

$$\beta_1 = \text{per AASHTO/ACI rule (min 0.65)}$$

$\beta_1 = 0.850$

$$\begin{aligned} c &= a / \beta_1 \\ &= 0.294 / 0.850 \end{aligned}$$

$c = 0.346$ in

2) Compute d_c and d_v

$$\begin{aligned} d_c &= h - c_{lr} \\ &= 12.00 - 2.00 \end{aligned}$$

$d_c = 10.000$ in

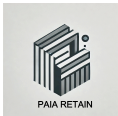
$$d_v = \max(0.9 d, 0.72 h, d_c - a/2)$$

$$0.9d = 0.9 \times 9.75 = 8.775$$

$$0.72h = 0.72 \times 12.00 = 8.640$$

$$d_c - a/2 = 10.000 - 0.147 = 9.853$$

$d_v = 9.853$ in



3) Longitudinal Strain ϵ_s , θ , and β

$$M_u(\text{lb-in}) = |M_u(\text{kip-ft})| \cdot 1000 \cdot 12$$

$$= |4.12| \cdot 12000 = 49384.47$$

$$V_u(\text{lb}) = |V_u(\text{kip})| \cdot 1000$$

$$= |0.79| \cdot 1000 = 786.20$$

$$\epsilon_{s, \text{trial}} = (M_u/d_v + V_u) / (E_s \cdot 1000 \cdot A_s)$$

$$= (49384.47/9.853 + 786.20) / (29000.00 \cdot 1000 \cdot 0.200)$$

$$\epsilon_{s, \text{trial}} = 0.001000$$

$$\text{Tension case: denom} = E_s \cdot 1000 \cdot A_s$$

$$\epsilon_s = 0.001000$$

$$\theta(\text{deg}) = 29 + 3500 \cdot \epsilon_s$$

$$= 29 + 3500 \times 0.001000 = 32.50$$

$$\theta = 32.50^\circ$$

$$\beta_{\text{base}} = 4.8 / (1 + 750 \cdot \epsilon_s)$$

$$= 4.8 / (1 + 750 \times 0.001000) = 2.743$$

Since $A_v \leq A_{v, \text{min}}$, adjust using s_{xe}

$$s_x = \min(d_v, s) \text{ (if } s > 0 \text{) else } d_v$$

$$= \min(9.853, 0.000) \rightarrow s_x = 9.853$$

$$s_{xe, \text{raw}} = s_x \cdot (1.38 / (0.63 + 0.75))$$

$$= 9.853 \cdot (1.38 / 1.38) = 9.853$$

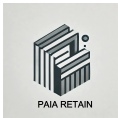
$$s_{xe} = \text{clamp}(12, s_{xe, \text{raw}}, 80)$$

$$= \text{clamp}(12, 9.853, 80) = 12.000$$

$$\beta = \beta_{\text{base}} \cdot (51 / (39 + s_{xe}))$$

$$= 2.743 \cdot (51 / (39 + 12.000)) = 2.743$$

$$\beta = 2.743$$



4) Concrete Contribution ϕV_c

$$V_c = b \cdot d_v \cdot 0.0316 \cdot \beta \cdot \sqrt{f'_c} \quad (f'_c \text{ in ksi})$$

$$= 12.00 \cdot 9.853 \cdot 0.0316 \cdot 2.743 \cdot \sqrt{4.00}$$

$$\phi V_c = \phi \cdot V_c \quad (\phi = 0.90)$$

$$\phi V_c = 18.45 \text{ kip}$$

5) Stirrup Contribution ϕV_s

No transverse reinforcement provided: $A_v = 0$, therefore $V_s = 0$.

$$\phi V_s = 0.00 \text{ kip}$$

6) Total Shear Capacity ϕV_n

$$\phi V_n = \phi V_c + \phi V_s$$

$$= 18.45 + 0.00$$

$$\phi V_n = 18.45 \text{ kip}$$

7) Factored Shear Demand V_u

$$V_u = 0.79 \text{ kip}$$

8) Shear Utilization

$$U = V_u / \phi V_n$$

$$= 0.79 / 18.45$$

$$U = 0.043$$

OK — $\phi V_n \geq V_u$