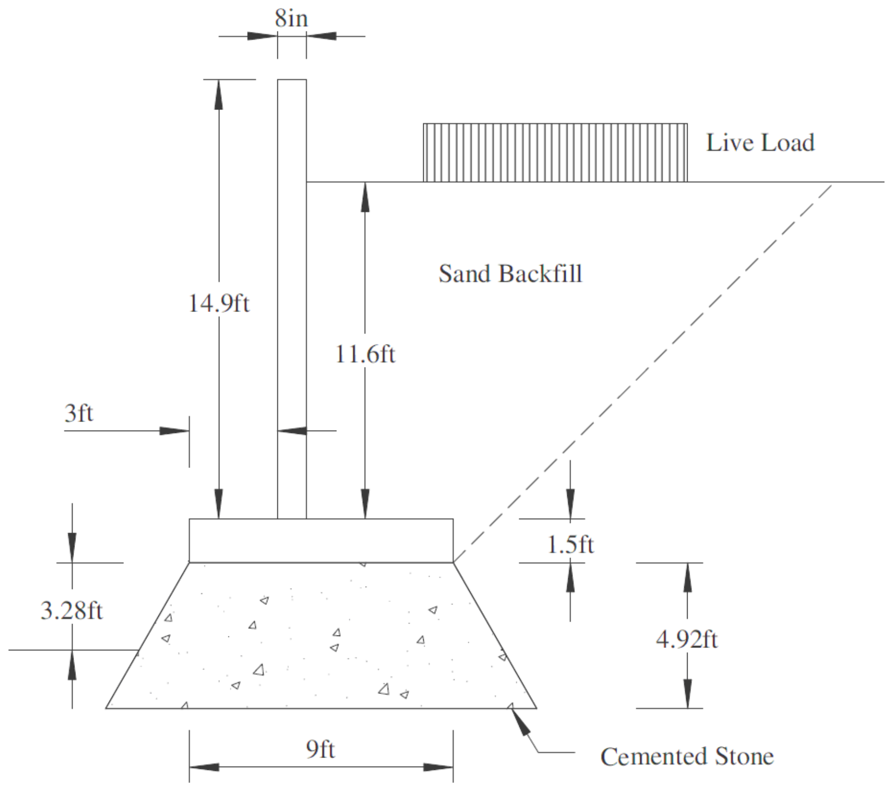
Retaining Wall:



Seismic coefficients from El Salvador's code:

$k_h := .16$ $k_v := 0$

seismic rotation angle for use with Mononobe-Okabe equations:

$\psi := \operatorname{atan}\left(\dfrac{k_h}{1 - k_v}\right)$ $\psi = 0.16$

Active pressure:

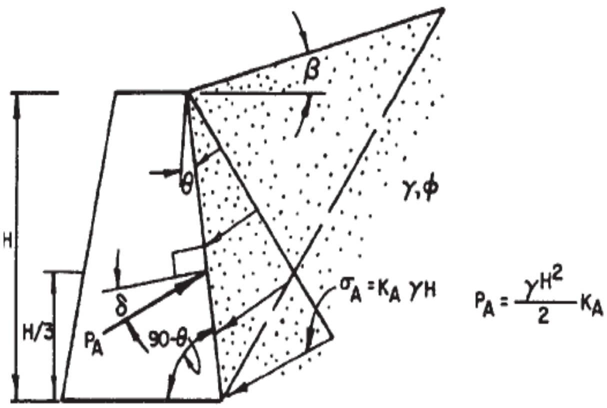
note: Soil properties are assumed based on typical values, since no geotechnical reports are available.

wall and soil properties:

$\phi := 30\ \text{deg}$ $H := 13.1 \cdot \text{ft}$ $\theta := 0\ \text{deg}$

$\beta := 0\ \text{deg}$ $\gamma := 105 \cdot \dfrac{\text{lb}}{\text{ft}^3}$ $\delta := \dfrac{2}{3} \times \phi$

calculate Coulomb's active pressure coefficient:



$$K_a := \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\delta + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \beta)}{\cos(\delta + \theta) \cdot \cos(\theta - \beta)}}\right)^2} \qquad K_a = 0.3$$

calculate active force:

$$P_a := \frac{1}{2} \cdot \gamma \cdot H^2 \cdot K_a \cdot (1 \cdot \text{ft}) \qquad P_a = 2.68\,\text{kip}$$

calculate active pressure with seismic per Mononobe-Okabe equation:

$$K_{ae} := \frac{\cos(\phi - \theta - \psi)^2}{\cos(\psi) \cdot \cos(\theta)^2 \cdot \cos(\delta + \theta + \psi) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi) \cdot \cos(\theta - \beta)}}\right)^2} \qquad K_{ae} = 0.42$$

calculate active force with seismic:

$$P_{ae} := \frac{1}{2} \cdot \gamma \cdot H^2 \cdot K_{ae} \cdot \left(1 - k_v\right) \cdot (1 \cdot \text{ft}) \qquad P_{ae} = 3.75\,\text{kip}$$

calculate location of force with seismic:

$$z := \frac{P_a \cdot \frac{H}{3} + (P_{ae} - P_a) \cdot \frac{2 \cdot H}{3}}{P_{ae}}$$ $z = 5.61\,ft$

Live Load:

note: (per AASHTO recommend practice) assume vehicle live load equal to 2 ft of soil on back side of wall.

$P_{al} := 2 \cdot ft \cdot \gamma \cdot K_{ae} \cdot H \cdot 1 \cdot ft$ $P_{al} = 1.14\,kip$

Check for overturning:

calculate the overturning moment:

$$OTM := P_{ae} \cdot \cos(\delta) \cdot z + P_{al} \cdot \frac{H}{2}$$ $OTM = 27.25\,kip \cdot ft$

calculate the restoring moment:

try a 2.75 m (9 ft) wide base with a 0.45 m (1.5 ft) thickness for the base and stem

	weight (kip)	dist. (ft)	moment
backfill	6.11	6.51	39.79
wall	1.75	3.51	6.15
base	2.03	4.51	9.16
live	1.05	6.51	6.86
	9.89		55.10

$RM := 55.10 \cdot kip \cdot ft$

calculate the safety factor against overturning (1.2 allowed per El Salvador code):

$$SF := \frac{RM}{OTM}$$ $SF = 2.02$ O.K.

Check for sliding:

assume a friction coefficient typical for foundations...

$\mu := \tan(\phi)$ $\quad\quad\quad\quad$ $\mu = 0.58$

calculate the sliding resistance:

$wt := 9.89 \cdot kip$

$R_S := \mu \cdot (wt)$ $\quad\quad\quad\quad$ $R_S = 5.71\, kip$

calculate the safety factor against sliding (1.2 allowed per El Salvador code):

$SF_S := \dfrac{R_S}{P_{ae} + P_{al}}$ $\quad\quad\quad\quad$ $SF_S = 1.17$ $\quad\quad$ close say O.K..

Check for soil bearing:

conservatively assume the following bearing pressures as a guideline, since no geotechnical reports are available for this site (by engineering judgement).

2000 psf	for static case
3000 psf	for transient seismic case

calculate the eccentricity about the base centroid:

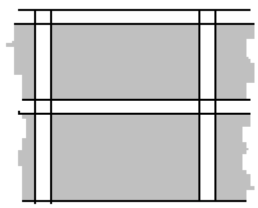
$ec := \dfrac{L}{2} - \dfrac{(RM - OTM)}{wt}$ $\quad\quad\quad\quad$ $ec = 1.7\, ft$ $\quad\quad$ > $\quad\quad$ $\dfrac{L}{6} = 1.5\, ft$

calculate the maximum soil bearing:

$SB_{max} := \dfrac{2 \cdot (wt)}{3 \cdot (1 \cdot ft)\left(\dfrac{L}{2} - ec\right)}$ $\quad\quad\quad\quad$ $SB_{max} = 2.34\, ksf$ $\quad\quad$ O.K.

Wall reinforcing:

A reinforced masonry wall will not span the 5 m (16.4 ft) height alone. Design concrete beams, columns and a butress system to be able to span the height.



Space columns at every 3 m (9.84 ft) with collector beams on top and mid height of the wall.

Assume 20 x 20 x 40 (8"x8"x16") CMU size

Calculate shear and moments at base of wall

$$V_{stem} := P_{ae} \cdot \cos(\delta) \cdot \frac{(H - 1.5 \cdot ft)^2}{H^2} + P_{al} \cdot \frac{(H - 1.5 \cdot ft)}{H} \qquad V_{stem} = 3.77\,kip$$

$$M_{stem} := P_{ae} \cdot \cos(\delta) \cdot (z - 1.5 \cdot ft) + P_{al} \cdot \left(\frac{H}{2} - 1.5 \cdot ft\right) \qquad M_{stem} = 20.26\,kip \cdot ft$$

Column design:

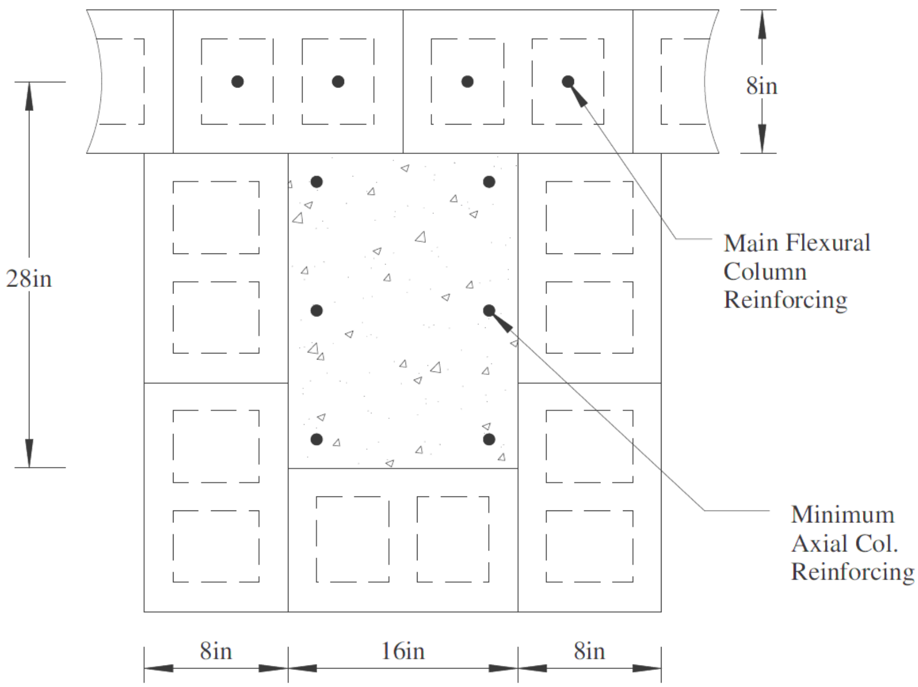
calculate ultimate moment at base:

$$\frac{spa}{ft} = 9.84$$

$$M_u := 1.6 \cdot M_{stem} \cdot \frac{spa}{ft} \qquad M_u = 318.99\,kip \cdot ft$$

Treat column as a flexural member since not being used for vertical loading.

Moment at column base is large; therefore make into a butress as shown on sketch



design reinforcing for column

$f_y := 60000 \cdot psi$ $\qquad f'_c := 3000 \cdot psi$ $\qquad \phi := .9$

$b := 16 \cdot in$ $\qquad d := 28 \cdot in$

$$K_u := \frac{M_u}{\phi \cdot b \cdot d^2} \qquad \rho := 0.85 \cdot f'_c \cdot \frac{\left(1 - \sqrt{1 - \frac{2 \cdot K_u}{0.85 \cdot f'_c}}\right)}{f_y} \qquad \rho = 0.0061$$

$A_s := \rho \cdot b \cdot d$ $\qquad A_s = 2.73\,in^2$

Use 8 # 6 Bars in bundles of 2 (area provided 3.52 sq inches)

Check for shear:

$\phi_s := .85$

$V_u := 1.6 \cdot V_{stem} \cdot \frac{spa}{ft}$ $\quad V_u = 59.43\,kip$

$\phi V_c := 2 \cdot \phi_s \cdot b \cdot d \cdot \sqrt{f'_c} \cdot psi$ $\quad \phi V_c = 41.71\,kip$ Need shear reinforcing

$\phi V_s := V_u - \phi V_c$ $\quad \phi V_s = 17.72\,kip$

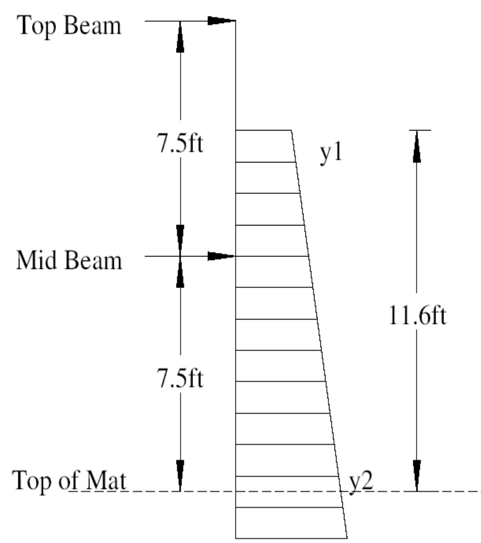
assume spacing of $\quad s := 16 \cdot in$

$A_v := \frac{\phi V_s \cdot s}{f_y \cdot d}$ $\quad A_v = 0.17\,in^2$

Use 3 # Ties @ 12 inches (area provided 0.22 sq inches)

Design Beams:

Find equivalent distributed load...



Given

$$\frac{y_1 + y_2}{2} = \frac{3.77}{11.6}$$

$$\frac{y_1 \cdot 11.6^2}{2} + \frac{(y_2 - y_1) \cdot 11.6^2}{6} = 20.26$$

$$\mathrm{Find}(y_1, y_2) = \begin{pmatrix} 0.25 \\ 0.4 \end{pmatrix}$$

Distributed loads at top and mid beams...

$P_1 := .3 \cdot \frac{kip}{ft}$ $\quad P_2 := 2.08 \frac{kip}{ft}$

Mid height collector beam reinforcing:

$$M_u := 1.6 \cdot \left[\frac{P_2 \cdot (spa)^2}{8}\right]$$

$$M_u = 40.3\,ft\,kip$$

$b := 16 \cdot in$ $\qquad d := 6.5 \cdot in$

$$K_u := \frac{M_u}{\phi \cdot b \cdot d^2} \qquad \rho := 0.85 \cdot f'_c \cdot \frac{\left(1 - \sqrt{1 - \frac{2 \cdot K_u}{0.85 \cdot f'_c}}\right)}{f_y} \qquad \rho = 0.0164$$

$A_s := \rho \cdot b \cdot d$ $\qquad A_s = 1.71\,in^2$

Use 4 # 6 Bars (area provided 1.76 sq inches)

Check for shear:

$$\phi_s := .85$$

$$V_u := 1.6 \cdot P_2 \cdot \frac{spa}{2} \qquad V_u = 16.38\,kip$$

$$\phi V_c := 2 \cdot \phi_s \cdot b \cdot d \cdot \sqrt{f'_{c.}} \cdot psi \qquad \phi V_c = 9.68\,kip$$

Need shear reinforcing

$$\phi V_s := V_u - \phi V_c \qquad \phi V_s = 6.69\,kip$$

assume spacing of $\qquad s := 12 \cdot in$

$$A_v := \frac{\phi V_s \cdot s}{f_y \cdot d} \qquad A_v = 0.21\,in^2$$

Use 3 # Ties @ 12 inches (area provided 0.22 sq inches)

Top collector beam reinforcing:

$$M_u := 1.6 \cdot \left(\frac{P_1 \cdot spa^2}{8} \right)$$

$$M_u = 5.81\,kip \cdot ft$$

$b := 12 \cdot in$ $\qquad$ $d := 6.5 \cdot in$

$$K_u := \frac{M_u}{\phi \cdot b \cdot d^2} \qquad \rho := 0.85 \cdot f'_c \cdot \frac{\left(1 - \sqrt{1 - \frac{2 \cdot K_u}{0.85 \cdot f'_c}}\right)}{f_y} \qquad \rho = 0.0026$$

$A_s := \rho \cdot b \cdot d$ $\qquad$ $A_s = 0.21\,in^2$

Use 2 # 4 Bars (area provided 0.4 sq inches)

Shear ok by inspection

Masonry reinforcing:

calculate moment on wall section between base and mid height collector beam.

$$w_1 := .25 + (.4 - .25)\left(\frac{4.1}{11.6}\right) \qquad w_1 = 0.3$$

$$w_2 := \left(\frac{.4 - w_1}{2}\right) \cdot \frac{16}{9 \cdot \sqrt{3}} \qquad w_2 = 0.05$$

$$w := (w_1 + w_2) \qquad w = 0.35$$

$$M_m := \frac{w \cdot 7.5^2}{8} \cdot kip \cdot ft \qquad M_m = 2.48\,kip \cdot ft$$

use UBC 97 masonry design (assuming no special inspection), since Salvadorean code does not specify masonry design requirements.

assumptions:

$f'_m := 1500\ \text{psi}$

$F_b := 0.5 \times 0.33 \times f'_m$ $\quad$ $F_b = 247.5\,\text{psi}$

$F_v := 0.5 \cdot \sqrt{\dfrac{f'_m}{\text{psi}}} \cdot \text{psi}$ $\quad$ $F_v = 19.36\,\text{psi}$

$F_s := 24000\ \text{psi}$

$E_m := 750 \cdot f'_m$ $\quad$ $E_m = 1.13 \times 10^6\,\text{psi}$

$E_s := 29000000\ \text{psi}$

$n := \dfrac{E_s}{E_m}$ $\quad$ $n = 25.78$

$b := 12 \cdot \text{in}$ $\quad$ $d := 6.5 \cdot \text{in}$

try # 6 bars in bundles of 2 at 8 inches $\quad$ $A_s := 1.32 \cdot \text{in}^2$

$\rho := \dfrac{A_s}{b \cdot d}$ $\quad$ $\rho = 0.0169$

$k := \sqrt{n^2 \cdot \rho^2 + 2 \cdot n \cdot \rho} - n \cdot \rho$ $\quad$ $k = 0.595$

$j := 1 - \dfrac{k}{3}$ $\quad$ $j = 0.802$

$f_b := \dfrac{2 \cdot M_m}{j \cdot k \cdot b \cdot d^2}$ $\quad$ $f_b = 246.27\,\text{psi}$ $\quad < \quad$ $F_b = 247.5\,\text{psi}$ $\quad$ OK

$f_s := \dfrac{M_m}{j \cdot d \cdot A_s}$ $\quad$ $f_s = 4.33 \times 10^3\,\text{psi}$ $\quad < \quad$ $F_s = 2.4 \times 10^4\,\text{psi}$ $\quad$ OK

Bottom reinforcing for base mat:

calculate the maximum moment for bottom reinforcement of the mat on the toe side:

using factored loads, calculate the eccentricity about the base centroid for determination of bearing pressure:

$RM = 55.1\,kip \cdot ft$ $\qquad$ $OTM = 27.25\,kip \cdot ft$

$ec := \frac{L}{2} - \frac{(1.2 \cdot RM - 1.6 \cdot OTM)}{1.2 \cdot wt}$ $\qquad$ $ec = 2.61\,ft$ $\qquad > \qquad$ $\frac{L}{6} = 1.5\,ft$

calculate the maximum soil bearing:

$SB_{max} := \frac{2 \cdot (1.2 \cdot wt)}{3 \cdot (1 \cdot ft)\left(\frac{L}{2} - ec\right)}$ $\qquad$ $SB_{max} = 4.17\,ksf$

$c := 3 \cdot \left(\frac{L}{2} - ec\right)$ $\qquad$ $c = 5.69\,ft$

$SB_{face} := \frac{\left(c - \frac{L}{3}\right) \cdot SB_{max}}{c}$ $\qquad$ $SB_{face} = 1.97\,ksf$

$M_u := \left(\frac{SB_{max} + SB_{face}}{2}\right) \cdot \left(\frac{L}{3} \cdot 1 \cdot ft\right) \cdot \frac{L \cdot (2SB_{max} + SB_{face})}{9 \cdot (SB_{max} + SB_{face})}$ $\qquad$ $M_u = 15.54\,kip \cdot ft$

reinforcing design:

$b := 12 \cdot in$ $\qquad$ $d := 14.5 \cdot in$

$K_u := \frac{M_u}{\phi \cdot b \cdot d^2}$ $\qquad$ $\rho := 0.85 \cdot f'_c \cdot \frac{\left(1 - \sqrt{1 - \frac{2 \cdot K_u}{0.85 \cdot f'_c}}\right)}{f_y}$ $\qquad$ $\rho = 0.0014$

$A_s := \rho \cdot b \cdot d$ $\qquad$ $A_s = 0.24\,in^2$

Use # 5 Bars @ 12" (area provided 0.31 sq inches / ft)

Check for shear:

$V_u := \left(\frac{SB_{max} + SB_{face}}{2}\right) \cdot \left(\frac{L}{3} \cdot 1 \cdot ft\right)$ $\quad V_u = 9.23\,kip$

$\phi V_c := 2 \cdot \phi_s \cdot b \cdot d \cdot \sqrt{f'_{c.}} \cdot psi$ $\quad \phi V_c = 16.2\,kip$ O.K.

Top reinforcing for base mat:

calculate the maximum moment for top reinforcement of the mat on the heal side:

$wt_s := 6.11 \cdot kip$ (Soil weight)

$\frac{x_1}{2} = 2.51\,ft$ (moment arm)

$M_u := 1.2 \cdot wt_s \cdot \frac{x_1}{2}$ $\quad M_u = 18.38\,kip \cdot ft$

Reinforcing design:

$b := 12 \cdot in$ $\quad d := 14.5 \cdot in$

$K_u := \frac{M_u}{\phi \cdot b \cdot d^2}$ $\quad \rho := 0.85 \cdot f'_c \cdot \frac{\left(1 - \sqrt{1 - \frac{2 \cdot K_u}{0.85 \cdot f'_c}}\right)}{f_y}$ $\quad \rho = 0.0017$

$A_s := \rho \cdot b \cdot d$ $\quad A_s = 0.29\,in^2$

Use # 5 Bars @ 12" (area provided 0.31 sq inches / ft)

Check for shear:

$V_u := 1.2 \cdot wt_s$ $\quad V_u = 7.33\,kip$

$\phi V_c := 2 \cdot \phi_s \cdot b \cdot d \cdot \sqrt{f'_{c.}} \cdot psi$ $\quad \phi V_c = 16.2\,kip$ O.K.

For reference:

bar	dia	area
#3	0.38	0.11
#4	0.50	0.20
#5	0.63	0.31
#6	0.75	0.44
#7	0.88	0.60
#8	1.00	0.79
#9	1.13	1.00
#10	1.27	1.27
#11	1.41	1.56
#14	1.69	2.25
#18	2.26	4.00

Sliding Calculation Between Soil and Base of Cemented Stone:

Active pressure:

wall and soil properties:

$\phi := 30\ \text{deg}$ $\quad H := 18 \cdot \text{ft}$ $\quad \theta := 0\ \text{deg}$

$\beta := 0\ \text{deg}$ $\quad \gamma := 105 \cdot \dfrac{\text{lb}}{\text{ft}^3}$ $\quad \delta := \dfrac{2}{3} \times \phi$

calculate Coulomb's active pressure coefficient:

$$K_a := \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\delta + \theta) \cdot \left(1 + \sqrt{\dfrac{\sin(\delta + \phi) \cdot \sin(\phi - \beta)}{\cos(\delta + \theta) \cdot \cos(\theta - \beta)}}\right)^2} \qquad K_a = 0.3$$

calculate active force:

$$P_a := \frac{1}{2} \cdot \gamma \cdot H^2 \cdot K_a \cdot (1 \cdot \text{ft}) \qquad P_a = 5.06\,\text{kip}$$

calculate active pressure with seismic per Mononobe-Okabe equation:

$$K_{ae} := \frac{\cos(\phi - \theta - \psi)^2}{\cos(\psi) \cdot \cos(\theta)^2 \cdot \cos(\delta + \theta + \psi) \cdot \left(1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi) \cdot \cos(\theta - \beta)}}\right)^2} \qquad K_{ae} = 0.42$$

calculate active force with seismic:

$$P_{ae} := \frac{1}{2} \cdot \gamma \cdot H^2 \cdot K_{ae} \cdot \left(1 - k_v\right) \cdot (1 \cdot \mathrm{ft}) \qquad P_{ae} = 7.07\,\mathrm{kip}$$

calculate location of force with seismic:

$$z := \frac{P_a \cdot \frac{H}{3} + \left(P_{ae} - P_a\right) \cdot \frac{2 \cdot H}{3}}{P_{ae}} \qquad z = 7.71\,\mathrm{ft}$$

Live Load:

$$P_{al} := 2 \cdot \mathrm{ft} \cdot \gamma \cdot K_{ae} \cdot H \cdot 1 \cdot \mathrm{ft} \qquad P_{al} = 1.57\,\mathrm{kip}$$

Check for sliding:

assume a friction coefficient typical for foundations...

$\mu := .55$

calculate the weight of the foundation:

from previous calculation... $wt_{fdn} := 9.89 \cdot \mathrm{kip}$

weight of the cemented stone... $\gamma_{stone} := 135 \cdot \frac{\mathrm{lb}}{\mathrm{ft}^3}$

$$wt_{stone} := \gamma_{stone} \cdot \frac{9 + 13.92}{2} \cdot \mathrm{ft} \cdot 4.92 \cdot \mathrm{ft} \cdot 1 \cdot \mathrm{ft} \qquad wt_{stone} = 7.61\,\mathrm{kip}$$

weight of additional soil not accounted for...

$wt_{add\_soil} := \frac{13.1 + 18}{2} ft \cdot 2.84 \cdot ft \cdot 1 \cdot ft \cdot \gamma$ $\qquad wt_{add\_soil} = 4.64\, kip$

$wt := wt_{fdn} + wt_{stone} + wt_{add\_soil}$

calculate the sliding resistance:

$R_s := \mu \cdot (wt)$ $\qquad R_s = 12.18\, kip$

calculate the safety factor against sliding (1.2 allowed per El Salvador code):

$SF_s := \frac{R_s}{P_{ae} + P_{al}}$ $\qquad SF_s = 1.41$ $\qquad$ O.K..