A SunCam online continuing education course

Mechanically Stabilized Earth Structures – Part 3

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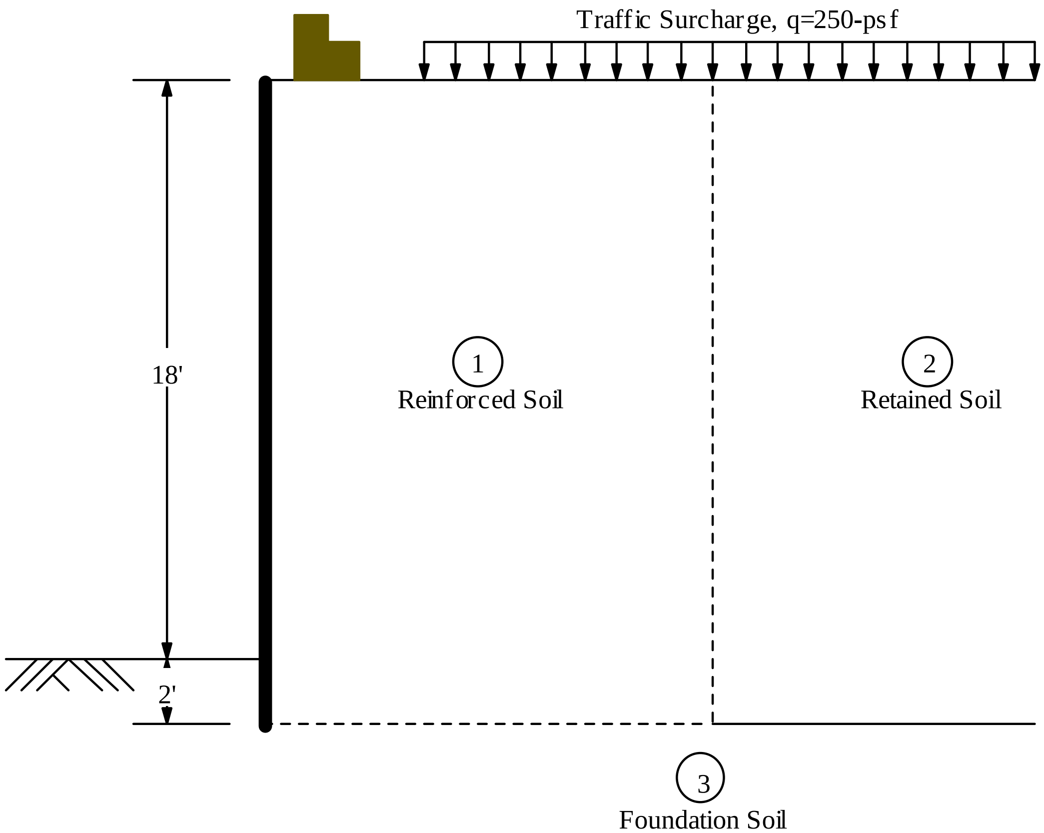
<u>Sample Design Calculation</u>

Sample Design Calculation based on AASHTO/FHWA Allowable Stress Design as noted in the "NHI-00-043, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Design and Construction Guidelines", March 2001.

- Design a 20-ft tall MSE wall with the following geometry and loading conditions
  - Level toe with 2-ft embedment depth below final grade
  - Level backfill, β=0-degrees
  - Live load traffic surcharge of q=250-lb/ft^2
- Segmental retaining wall block for this sample calculation has a mechanical connection and high strength geotextile reinforcement will be used to reinforce the soil.

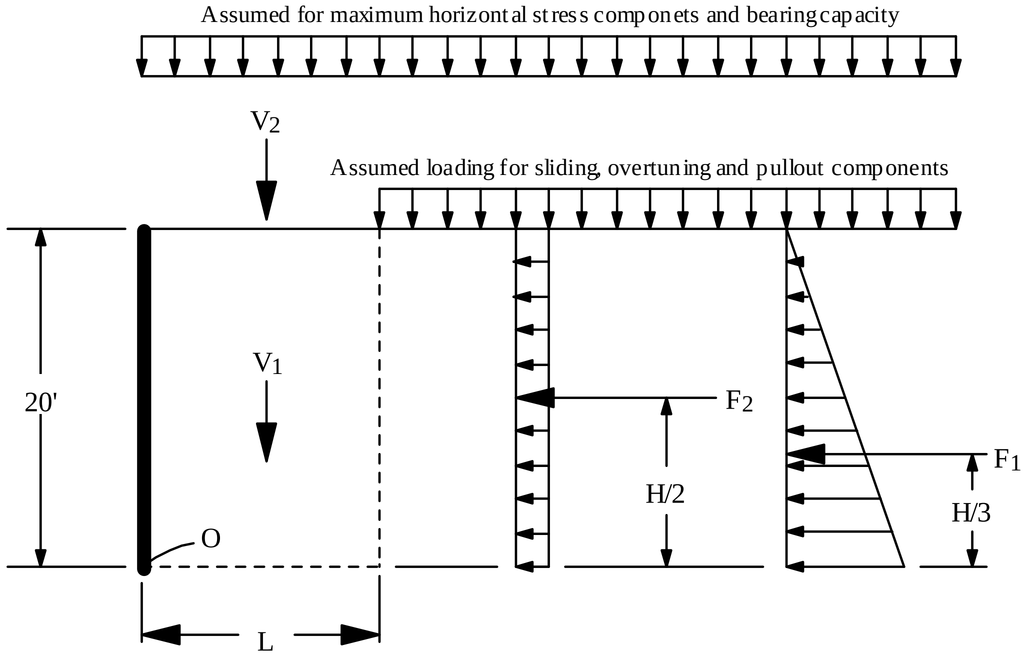
Figure 1 – General cross section of MSE wall.



- **Soil Design Parameters**
  1. Reinforced Soil ϕ'=35-degrees c'=0-lb/ft^2 γ =125-lb/ft^3
  2. Retained Soil ϕ'_r =28-degrees c'_r=0-lb/ft^2 γ_r=110-lb/ft^3
  3. Foundation Soil ϕ'_f=28-degrees c'_f =0-lb/ft^2 γ_f =110-lb/ft^3

External Stability Calculations

Figure 2 – Free Body Diagram of MSE wall.



Determine Active Earth Pressure Coefficient ($K_{a\,(ext)}$) based on Rankine Earth Pressure Theory and Horizontal and Vertical Forces acting on the MSE wall.

$$K_{a(ext)} = \cos\beta\left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi_r}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi_r}}\right]$$ **Eq. 1**

$$K_{a(ext)} = \cos(0)\left[\frac{\cos(0) - \sqrt{\cos^2 0 - \cos^2 28}}{\cos(0) + \sqrt{\cos^2 0 - \cos^2 28}}\right] = 0.361$$

$F_1 = ½\, \gamma_r\, H^2\, K_{a(ext)}$	= ½ (110 lb/ft^3) (20-ft)^2 (0.361)	= 7,943 lb/ft	**Eq. 2**
$F_2 = q\, H\, K_{a(ext)}$	= (250 lb/ft^2) (20-ft) (0.361)	= 1,805 lb/ft	**Eq. 3**
$L = 0.7\, H$	= 0.7 (20-ft)	= 14-ft	**Eq. 4**
$V_1 = \gamma\, H\, L$	= (125 lb/ft^3) (20-ft) (14-ft)	= 35,000 lb/ft	**Eq. 5**
$V_2 = q\, L$	= (250 lb/ft^2) (14-ft)	= 3,500 lb/ft	**Eq. 6**

~ External Factors of Safety for Overturning, Sliding and Bearing Capacity

Factor of Safety for Overturning - FS_{OT}

- Moments are taken about Point "O" (see Figure 1)
- Resisting moment does not include traffic surcharge, q

$$\mathbf{FS_{OT}} = \frac{\sum Moments \cdot \text{Resisting}}{\sum Moments \cdot Overeturning} = \frac{\sum M_R}{\sum M_O} = \frac{V_1\left(\frac{L}{2}\right)}{F_1\left(\frac{H}{3}\right) + F_2\left(\frac{H}{2}\right)} \geq 2.0 \qquad \textbf{Eq. 7}$$

$$M_R = \frac{1}{2} V_1 L = \frac{1}{2} (35{,}000 \text{ lb/ft}) (14\text{-ft}) = 245{,}000 \text{ lb-ft}$$

$$M_O = \frac{1}{3} F_1 H + \frac{1}{2} F_2 H = \frac{1}{3} (7{,}943 \text{ lb/ft}) (20\text{-ft}) + \frac{1}{2} (1{,}805 \text{ lb/ft}) (20\text{-ft}) = 71{,}003 \text{ lb-ft}$$

$$\mathbf{FS_{OT}} = \frac{\sum M_R}{\sum M_O} = \frac{245{,}000 \text{ lb-ft}}{71{,}003 \text{ lb-ft}} = 3.45 \geq 2.0 \text{ therefore meets requirement.}$$

Factor of Safety for Sliding – FS_{SL}

- Resisting force does not include traffic surcharge, q

$$\mathbf{FS_{SL}} = \frac{\sum Horizontal \cdot \text{Resisting} \cdot Force}{\sum Horizontal \cdot Driving \cdot Force} = \frac{\sum F_R}{\sum F_D} = \frac{V_1 \cdot (\tan \phi_f)}{F_1 + F_2} \geq 1.5 \qquad \textbf{Eq. 8}$$

$$\mathbf{FS_{SL}} = \frac{\sum F_R}{\sum F_D} = \frac{35{,}000 \text{ lb/ft} \cdot (\tan 28^o)}{7{,}943 \text{ lb/ft} + 1{,}805 \text{ lb/ft}} = 1.91 \geq 1.5 \text{ therefore meets requirement.}$$

Factor of Safety for Bearing Capacity – FS_{BC}

- R_{BC} = Resultant of Vertical Forces
- Resisting moment for bearing capacity ($M_{R(BC)}$) includes traffic surcharge, q
- e = eccentricity (feet)
- B' = effective foundation width
- σ'_v = vertical overburden stress (lb/ft^2)
- q_{ult} = ultimate bearing capacity (lb/ft^2)

R_{BC} = Resultant of Vertical Forces = $V_1 + V_2$ = 35,000 lb/ft + 3,500 lb/ft = 38,500 lb/ft **Eq. 9**

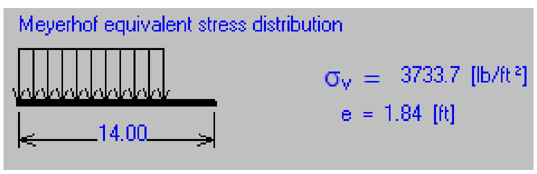
$$M_{R\,(BC)} = \frac{1}{2}(R_{BC})(L) = \frac{1}{2}(38{,}500 \text{ lb/ft})(14\text{-ft}) = 269{,}500 \text{ lb/ft} \quad \textbf{Eq. 10}$$

$$e = \frac{L}{2} - \frac{M_R - M_O}{R_{BC}} \le \frac{L}{6} \;\ldots\ldots\; e = \frac{14-ft}{2} - \frac{269{,}500 lb\cdot ft - 71{,}003 lb\cdot ft}{38{,}500\cdot lb/ft} = 1.84\text{-ft} \quad \textbf{Eq. 11}$$

$\frac{L}{6} = \frac{14-ft}{6}$ = 2.33-ft ……… $e \le \frac{L}{6}$ ………1.84-ft $\le$ 2.33-ft (therefore eccentricity is okay)

B' = L - 2e = 14-ft – 2 (1.84-ft) = 10.31-ft **Eq. 12**

$$\sigma'_v = \frac{V_1 + V_2 + F_1 \sin\beta}{L - 2e} = \frac{35{,}000 \text{ lb/ft} + 3{,}500 \text{ lb/ft} + 7{,}943 \text{ lb/ft}(\sin 0^\circ)}{14\text{-ft} - 2(1.84\text{-ft})} = 3{,}734 \text{ lb/ft}^2 \quad \textbf{Eq. 13}$$



MSEW Screen Shot of Meyerhof stress distribution, vertical overburden stress and eccentricity.

Bearing capacity factors can be found in most Foundation Engineering text books or by applying the following formulas, in this example for ϕ'_f=28-degrees:

- $N_q = \tan^2(45 + \frac{\phi}{2})\, e^{\pi \tan\phi}$ = 14.72 **Eq. 14**
- $N_c = (N_q - 1)\cot\phi$ = 25.80 **Eq. 15**
- $N_\gamma = 2(N_q + 1)\tan\phi$ = 16.72 **Eq. 16**

$q_{ult} = c_f N_c + \frac{1}{2}\gamma_f B' N_\gamma$ (this is the ultimate bearing capacity) **Eq. 17**

$$q_{ult} = (0\text{-psf})(25.80) + \frac{1}{2}(110\text{-lb/ft}^3)(10.31\text{-ft})(16.72) = 9{,}481 \text{ lb/ft}^2$$

$FS_{BC} = \frac{q_{ult}}{\sigma'_v} = \frac{9{,}481 \text{ lb/ft2}}{3{,}734 \text{ lb/ft2}}$ = 2.54 $\ge$ 2.0 therefore meets requirement. **Eq. 18**

$$\frac{e}{L} = \frac{1.84-ft}{14-ft} = 0.132 \quad \textbf{Eq. 19}$$

~ Internal Factors of Safety for Sliding, Overstress, Pullout and Connection

Determine the reinforcement Long Term Design Strength of a High Strength Geotextile

T_{ult} = Ultimate Geosynthetic Strength (from manufacturer) = 7,200 lb/ft

RF_{CR} = Creep Reduction Factor (from manufacturer) = 1.68

RF_{ID} = Installation Damage Reduction Factor (from manufacturer) = 1.10

RF_{D} = Durability Reduction Factor (from manufacturer) = 1.10

$$LTDS = \frac{Tult}{RFcr \times RFid \times RFd} = \frac{7{,}200 \text{ lb/ft}}{1.68 \times 1.10 \times 1.10} = 3{,}274 \text{ lb/ft} \qquad \textbf{Eq. 20}$$

Determine Internal Active Earth Pressure Coefficient

$$K_{a(int)} = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 35}{1+\sin 35} = 0.271 \qquad \textbf{Eq. 21}$$

Determine Reinforcement Pullout Properties

F^* = $\tan\phi = \tan 35^o = 0.700$ Pullout Resistant Factor **Eq. 22**

R_c = 1.0 100% Geosynthetic Coverage

C_i = 0.9 Coefficient of Interaction (from manufacturer)

C_{ds} = 0.8 Coefficient of Direct Sliding (from manufacturer)

C = 2.0 (for geotextile or geogrid) Reinforcement effective unit parameter

α = 1.0 Scale correction factor (from manufacturer)

Calculate maximum tensile force in each reinforcement layer. We need to determine horizontal stress (σ_H) along the potential failure line from the weight of the reinforced fill (γZ) plus, if present uniform surcharge loads (q) and concentrated surcharge loads $\Delta\sigma_v$ and $\Delta\sigma_H$.

Z = distance from top of wall to reinforcement layer

σ_2 = surcharge load due to sloping backfill

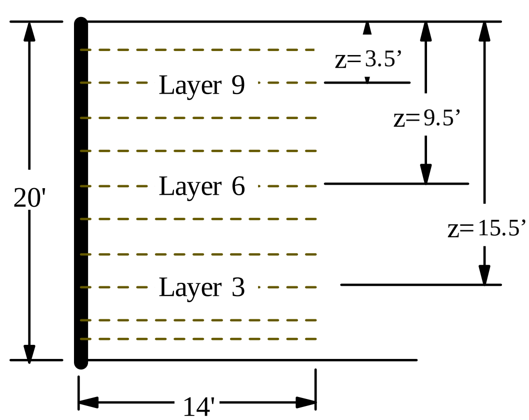
$\Delta\sigma_H$ = horizontal surcharge load due to footing

$\Delta\sigma_v$ = vertical surcharge load due to footing

q = traffic surcharge = 250-lb/ft^2

σ_H = horizontal stress = $K_{a(int)}\,\sigma_v + \Delta\sigma_H$

σ_v = vertical stress = $\gamma Z + \sigma_2 + q + \Delta\sigma_v$



Given no slopes or footing loads: σ_2, $\Delta\sigma_H$ and $\Delta\sigma_v$ = 0-psf

For this sample problem we will calculate the vertical and horozontal stress at three geosynthetic layers located at a depth of 3.5-ft, 9.5-ft and 15.5-ft from the top of wall.

$$\sigma_{v(n)} = \gamma Z_{(n)} + \sigma_2 + q + \Delta\sigma_v \qquad \textbf{Eq. 23}$$

$$\sigma_{v(3)} = (125\ lb/ft^3)(15.5\text{-}ft) + 0\text{-}lb/ft^2 + 250\text{-}lb/ft^2 + 0\text{-}lb/ft^2 = 2{,}188\ lb/ft^2$$

$$\sigma_{v(6)} = (125\ lb/ft^3)(9.5\text{-}ft) + 0\text{-}lb/ft^2 + 250\text{-}lb/ft^2 + 0\text{-}lb/ft^2 = 1{,}438\ lb/ft^2$$

$$\sigma_{v(9)} = (125\ lb/ft^3)(3.5\text{-}ft) + 0\text{-}lb/ft^2 + 250\text{-}lb/ft^2 + 0\text{-}lb/ft^2 = 688\ lb/ft^2$$

$$\sigma_{h(n)} = K_{a(int)}\,\sigma_{v(n)} + \Delta\sigma_H \qquad \textbf{Eq. 24}$$

$$\sigma_{h(3)} = (0.271)(2{,}188\ lb/ft^2) + 0\text{-}lb/ft^2 = 593\ lb/ft^2$$

$$\sigma_{h(6)} = (0.271)(1{,}438\ lb/ft^2) + 0\text{-}lb/ft^2 = 390\ lb/ft^2$$

$$\sigma_{h(9)} = (0.271)(688\ lb/ft^2) + 0\text{-}lb/ft^2 = 186\ lb/ft^2$$

Calculate the maximum tension (T_{max}) in each reinforcement layer. The maximum vertical spacing (S_v) between reinforcement layers is limited to 2.0-ft.

$$T_{max(n)} = (\sigma_{h(n)})(S_v) \qquad \textbf{Eq. 25}$$

$$T_{max(3)} = (593\ lb/ft^2)(2.0\text{-}ft) = 1{,}186\ lb/ft$$

$$T_{max(6)} = (390\ lb/ft^2)(2.0\text{-}ft) = 779\ lb/ft$$

$$T_{max(9)} = (186\ lb/ft^2)(2.0\text{-}ft) = 373\ lb/ft$$

Calculate the Factor of Safety for Reinforcement Overstress in each reinforcement layer.

$$FS_{OS(n)} = \frac{LTDS_{(n)}}{T_{\max(n)}} \qquad \textbf{Eq. 26}$$

$$FS_{OS(3)} = \frac{3{,}274\ lb/ft}{1{,}186\ lb/ft} = 2.762 >> 1.50 \therefore \text{ ok}$$

$$FS_{OS(6)} = \frac{3{,}274\ lb/ft}{779\ lb/ft} = 4.202 >> 1.50 \therefore \text{ ok}$$

$$FS_{OS(9)} = \frac{3{,}274\ lb/ft}{373\ lb/ft} = 8.787 >> 1.50 \therefore \text{ ok}$$

Observation: The factor of safety for reinforcement overstress increases in the upper portion of the wall due to the lower stress level. In this example we are using one type of reinforcement for design, however in a real application two or three types of reinforcement types may be used with highest strength reinforcement in the lower portion of the wall where the stress is high and lower strength reinforcement in the upper portion of the wall where horizontal stresses are lower.

Stability with respect to reinforcements' pullout requires the following criteria be satisfied…

$$T_{max} = \frac{1}{FS_{po}} F^* \gamma Z_p L_e C R_c \alpha$$ **Eq. 27**

where: FS_{PO} = Safety factor against pullout $\geq$ 1.5; Rc = Coverage ratio

T_{max} = Maximum reinforcement tension; α = Scale correction factor

C = 2 for geogrid or geotextile; F* = Pullout resistance factor

γZ_p = Overburden pressure, including distributed dead load surcharges, neglecting traffic loads (see Figure 30 in NHI-00-043).

Le = Length of embedment in the resisting zone. Note that the boundary between the resisting and active zones may be modified by concentrated loadings.

Required embedment length in reinforced zone beyond the potential failure surface is determined from:

$$L_{e(n)} > \frac{1.5\left(T_{\max(n)}\right)}{C \cdot F^* \cdot c_i \cdot \gamma \cdot Z_{(n)} \cdot R_c \cdot \alpha}$$ **Eq. 28**

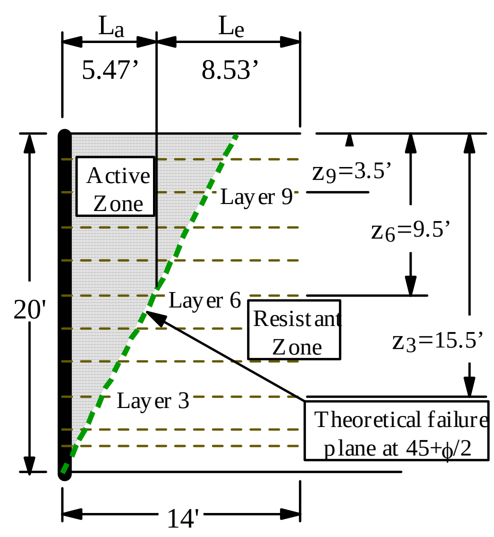
Determine the active zone (L_a) for each reinforcement layer.

$L_{a(n)}$ = $(H - z_{(n)}) \tan(45-\phi/2)$ **Eq. 29**

$L_{a(3)}$ = (20-ft – 15.5-ft) tan(45-35°/2) = 2.34-ft

$L_{a(6)}$ = (20-ft – 9.5-ft) tan(45-35°/2) = 5.47-ft

$L_{a(9)}$ = (20-ft – 3.5-ft) tan(45-35°/2) = 8.59-ft



Determine the actual design embedment length for (L_e) for each reinforcement layer.

$L_{e(n)}$ = $L - L_{a(n)}$ **Eq. 30**

$L_{e(3)}$ = 14-ft – 2.34-ft = 11.66-ft

$L_{e(6)}$ = 14-ft – 5.47-ft = 8.53-ft

$L_{e(9)}$ = 14-ft – 8.59-ft = 5.41-ft

Determine the Factor of Safety for Pullout for each reinforcement layer.

$$FS_{po(n)} = \frac{C \cdot F^* \cdot c_i \cdot \gamma \cdot Z_{(n)} \cdot L_{e(n)} \cdot R_c \cdot \alpha}{T_{\max(n)}} \qquad \textbf{Eq. 31}$$

$$FS_{po(3)} = \frac{(2.0)(0.7)(0.9)(125 \cdot lb/ft^3)(15.5-ft)(11.66-ft)(1.0)(1.0)}{1{,}186-lb/ft} = 24.009 > 1.50 \therefore \text{ ok}$$

$$FS_{po(6)} = \frac{(2.0)(0.7)(0.9)(125 \cdot lb/ft^3)(9.5-ft)(8.53-ft)(1.0)(1.0)}{779-lb/ft} = 16.389 > 1.50 \therefore \text{ ok}$$

$$FS_{po(9)} = \frac{(2.0)(0.7)(0.9)(125 \cdot lb/ft^3)(3.5-ft)(5.41-ft)(1.0)(1.0)}{373-lb/ft} = 8.001 > 1.50 \therefore \text{ ok}$$

Stability analysis with respect to Internal Sliding along individual reinforcement layers.

$$FS_{SL(n)} = \frac{\sum F_R \tan\rho}{\sum F_D} = \frac{V_{1(n)} \tan\rho}{F_{1(3)} + F_{2(3)}} \qquad \textbf{Eq. 32}$$

$$\rho = \tan^{-1}(C_{ds} * \tan\phi) = \tan^{-1}(0.8 \tan 35^o) = 29.256^o \qquad \textbf{Eq. 33}$$

Previously defined or calculated terms/values to be used in internal sliding calculations:

$K_{a(ext)}$ = 0.361	γ = 125-lb/ft^3	$z_{(3)}$ = 15.5-ft	$z_{(9)}$ = 3.5-ft	ρ = 29.256^o
$q_{traffic}$ = 250-lb/ft^2	γ_r = 110-lb/ft^3	$z_{(6)}$ = 9.5-ft		L = 14-ft

Internal Factor of Safety for Sliding on Layer 3

$F_{1(3)}$ = ½ $\gamma_r\, z^2_{(3)}\, K_{a(ext)}$

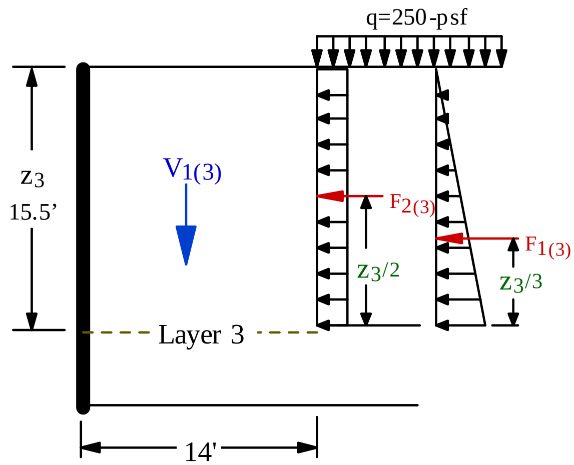
$F_{1(3)}$ = ½ (110 lb/ft^3) (15.5-ft)^2 (0.361) = 4,771 lb/ft

$F_{2(3)}$ = $q\, z_{(3)}\, K_{a(ext)}$

$F_{2(3)}$ = (250 lb/ft^2) (15.5-ft) (0.361) = 1,399 lb/ft

$V_{1(3)}$ = $\gamma\, z_{(3)}\, L$

$V_{1(3)}$ = (125 lb/ft^3) (15.5-ft) (14-ft) = 27,125 lb/ft



$$FS_{SL(3)} = \frac{V_{1(n)} \tan 29.256}{F_{1(3)} + F_{2(3)}}$$

$$FS_{SL(3)} = \frac{27{,}125\text{ lb/ft }(\tan 29.256)}{4{,}771\text{ lb/ft} + 1{,}399\text{ lb/ft}} = 2.463 > 1.50 \therefore \text{ ok}$$

Internal Factor of Safety for Sliding on Layer 6

$F_{1(6)} \;=\; \frac{1}{2}\,\gamma_r\, z^2_{(6)}\, K_{a(ext)}$

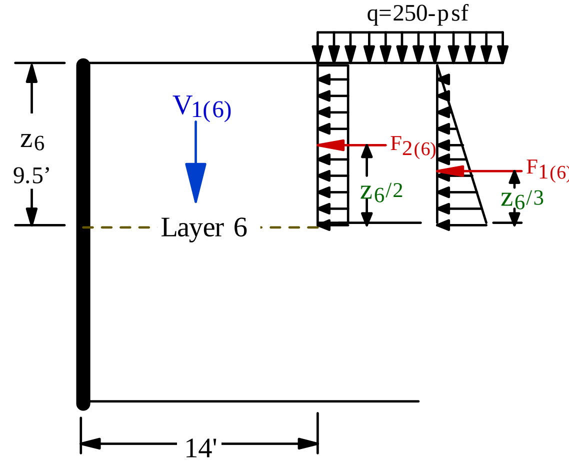
$F_{1(6)} \;=\; \frac{1}{2}\,(110\ \text{lb/ft}^3)\,(9.5\text{-ft})^2\,(0.361) \;=\; 1{,}792\ \text{lb/ft}$

$F_{2(6)} \;=\; q\, z_{(6)}\, K_{a(ext)}$

$F_{2(6)} \;=\; (250\ \text{lb/ft}^2)\,(9.5\text{-ft})\,(0.361) \;=\; 857\ \text{lb/ft}$

$V_{1(6)} \;=\; \gamma\, z_{(6)}\, L$

$V_{1(6)} \;=\; (125\ \text{lb/ft}^3)\,(9.5\text{-ft})\,(14\text{-ft}) \;=\; 16{,}625\ \text{lb/ft}$



$$FS_{SL(6)} = \frac{V_{1(n)}\tan 29.256}{F_{1(6)} + F_{2(6)}}$$

$$FS_{SL(6)} = \frac{16{,}625\ \text{lb/ft}\,(\tan 29.256)}{1{,}792\ \text{lb/ft} + 857\ \text{lb/ft}} = 3.515 > 1.50\ \therefore\ \text{ok}$$

Internal Factor of Safety for Sliding on Layer 9

$F_{1(9)} \;=\; \frac{1}{2}\,\gamma_r\, z^2_{(9)}\, K_{a(ext)}$

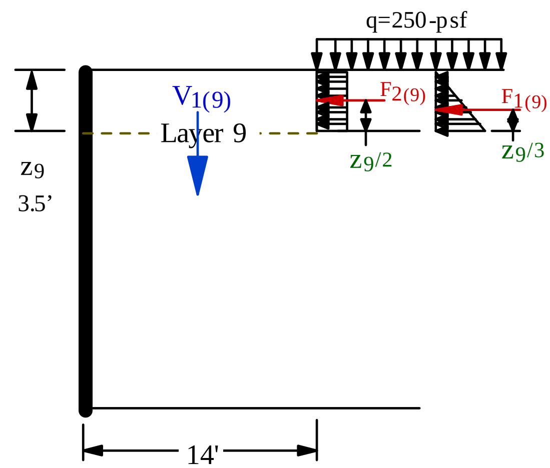
$F_{1(6)} \;=\; \frac{1}{2}\,(110\ \text{lb/ft}^3)\,(3.5\text{-ft})^2\,(0.361) \;=\; 243\ \text{lb/ft}$

$F_{2(9)} \;=\; q\, z_{(9)}\, K_{a(ext)}$

$F_{2(9)} \;=\; (250\ \text{lb/ft}^2)\,(3.5\text{-ft})\,(0.361) \;=\; 316\ \text{lb/ft}$

$V_{1(9)} \;=\; \gamma\, z_{(9)}\, L$

$V_{1(9)} \;=\; (125\ \text{lb/ft}^3)\,(3.5\text{-ft})\,(14\text{-ft}) \;=\; 6{,}125\ \text{lb/ft}$

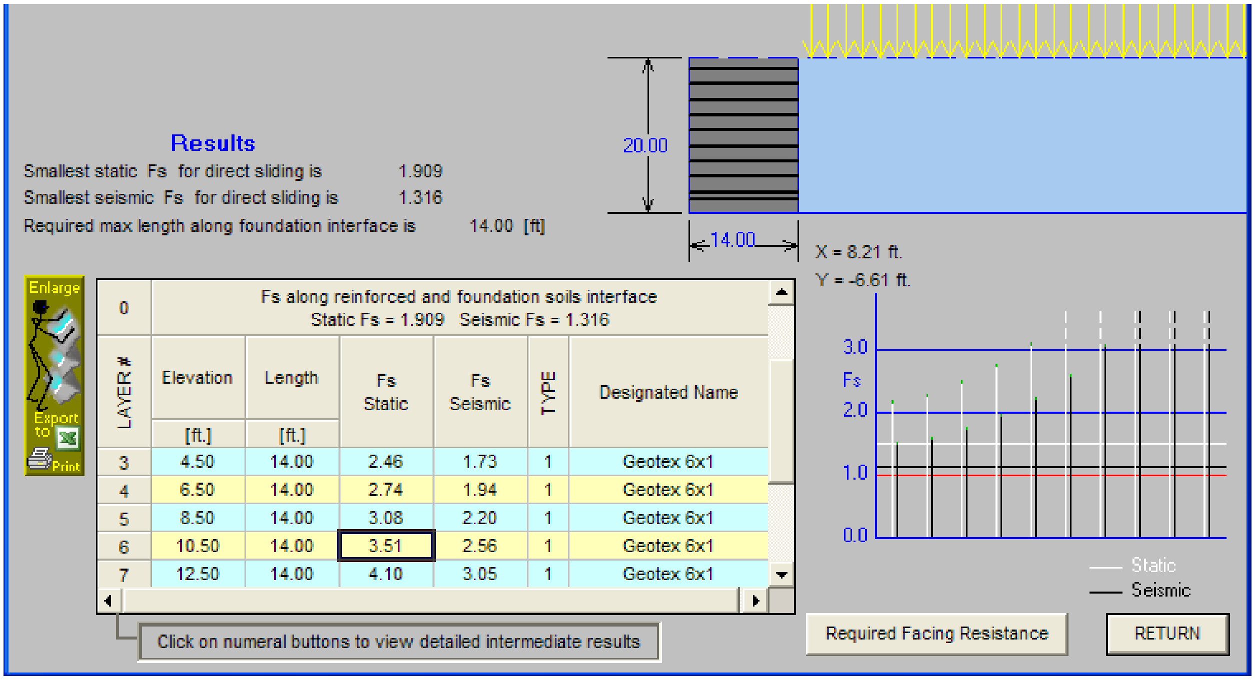


$$FS_{SL(9)} = \frac{V_{1(n)}\tan 29.256}{F_{1(9)} + F_{2(9)}}$$

$$FS_{SL(9)} = \frac{6{,}125\ \text{lb/ft}\,(\tan 29.256)}{243\ \text{lb/ft} + 316\ \text{lb/ft}} = 6.136 > 1.50\ \therefore\ \text{ok}$$

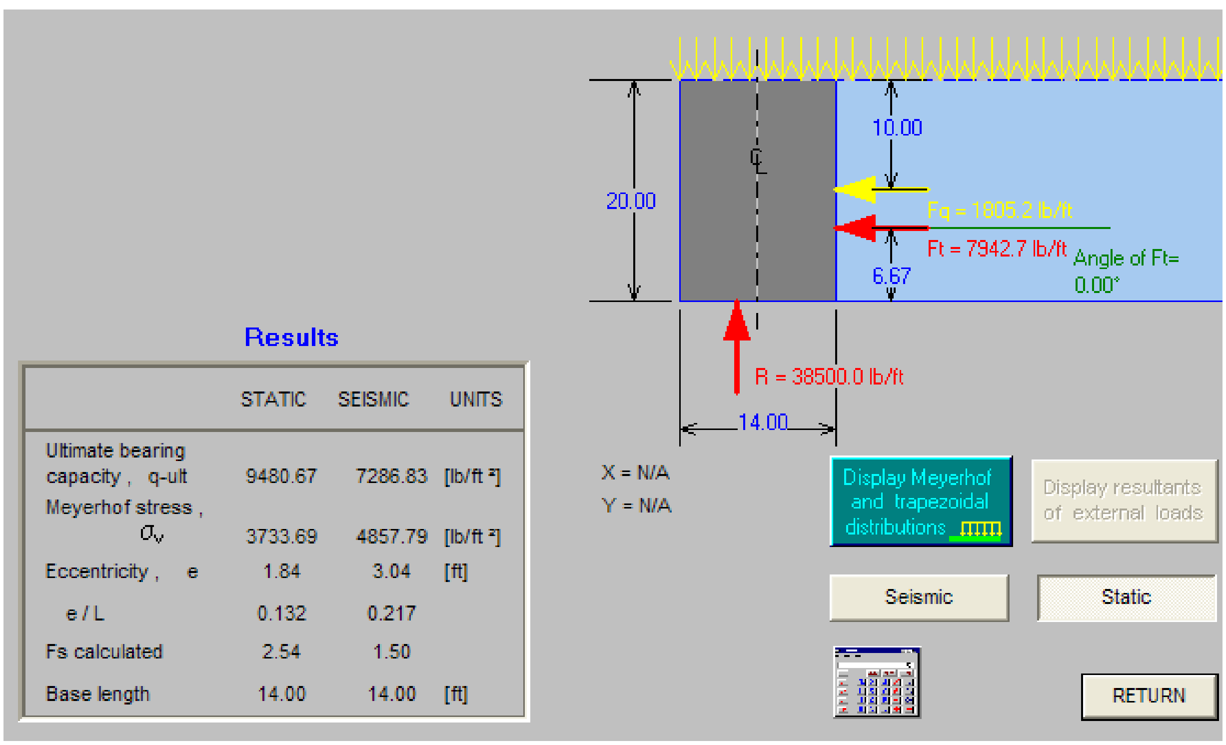
Observation: The factor of safety for internal direct sliding increases as you move from the bottom of wall to the top of wall. This is due to the fact that the reinforcement length remains constant at L-14-ft throughout the wall height while the horizontal force on a given reinforcement layer decreases as reinforcement elevation increases. In short internal direct sliding controls the design lengths at the bottom of the wall, whereas pullout failure controls the design lengths at the top of wall as previously calculated.

Screen shot from program MSEW 3.0 with respect to internal sliding and external sliding.



LAYER #	Elevation [ft.]	Length [ft.]	Fs Static	Fs Seismic	TYPE	Designated Name
3	4.50	14.00	2.46	1.73	1	Geotex 6x1
4	6.50	14.00	2.74	1.94	1	Geotex 6x1
5	8.50	14.00	3.08	2.20	1	Geotex 6x1
6	10.50	14.00	3.51	2.56	1	Geotex 6x1
7	12.50	14.00	4.10	3.05	1	Geotex 6x1

Screen shot from program MSEW 3.0 with respect to bearing capacity.



	STATIC	SEISMIC	UNITS
Ultimate bearing capacity, q-ult	9480.67	7286.83	[lb/ft ²]
Meyerhof stress, σ_v	3733.69	4857.79	[lb/ft ²]
Eccentricity, e	1.84	3.04	[ft]
e / L	0.132	0.217	
Fs calculated	2.54	1.50	
Base length	14.00	14.00	[ft]

Internal eccentricity calculations for each reinforcement layer. Note ASSHTO/FHWA does not require an evaluation of e/L, the calculation provide here is for information only.

Reinforcement Layer 3

$$M_{R(3)} = \frac{1}{2} V_{1(3)} L = \frac{1}{2} (27{,}125 \text{ lb/ft}) (14\text{-ft}) = 189{,}875 \text{ lb-ft}$$

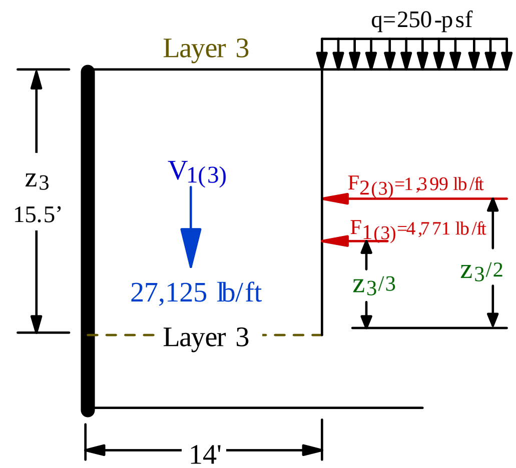
$$M_{O(3)} = \frac{1}{3} F_{1(3)} z_{(3)} + \frac{1}{2} F_{2(3)} z_{(3)} = \frac{1}{3}(4{,}471 \text{ lb/ft})(15.5\text{-ft}) + \frac{1}{2}(1{,}399 \text{ lb/ft})(15.5\text{-ft}) = 35{,}492 \text{ lb-ft}$$

$$\mathbf{FS_{OT(3)}} = \frac{\sum M_{R(3)}}{\sum M_{O(3)}} = \frac{189{,}875 \text{ lb-ft}}{35{,}492 \text{ lb-ft}} = 3.45 \geq 2.0$$

$$e = \frac{L}{2} - \frac{M_{R(3)} - M_{O(3)}}{R_{BC(3)}} \leq \frac{L}{6}$$

$$e = \frac{14\text{-ft}}{2} - \frac{189{,}875 \text{ lb-ft} - 35{,}492 \text{ lb-ft}}{27{,}125 \text{ lb/ft}} = 1.308\text{-ft}$$

$$\frac{e}{L} = \frac{1.308\text{-ft}}{14\text{-ft}} = 0.0935$$



Reinforcement Layer 6

$$M_{R(6)} = \frac{1}{2} V_{1(6)} L = \frac{1}{2} (16{,}625 \text{ lb/ft}) (14\text{-ft}) = 116{,}375 \text{ lb-ft}$$

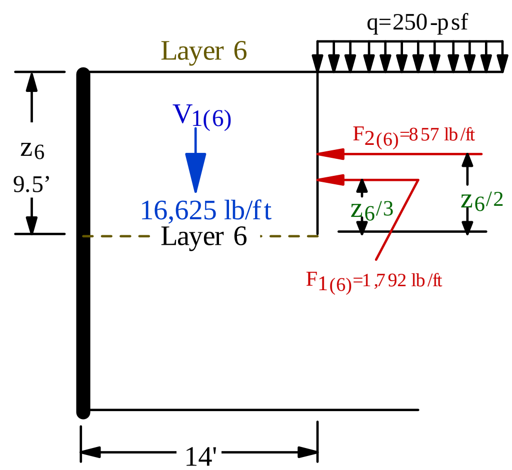
$$M_{O(6)} = \frac{1}{3} F_{1(3)} z_{(6)} + \frac{1}{2} F_{2(3)} z_{(6)} = \frac{1}{3}(1{,}792 \text{ lb/ft})(15.5\text{-ft}) + \frac{1}{2}(857 \text{ lb/ft})(15.5\text{-ft}) = 9{,}745 \text{ lb-ft}$$

$$\mathbf{FS_{OT(3)}} = \frac{\sum M_{R(6)}}{\sum M_{O(6)}} = \frac{116{,}375 \text{ lb-ft}}{9{,}745 \text{ lb-ft}} = 11.94 \geq 2.0$$

$$e = \frac{L}{2} - \frac{M_{R(3)} - M_{O(6)}}{R_{BC(6)}} \leq \frac{L}{6}$$

$$e = \frac{14\text{-ft}}{2} - \frac{116{,}375 \text{ lb-ft} - 9{,}745 \text{ lb-ft}}{16{,}625 \text{ lb/ft}} = 0.586\text{-ft}$$

$$\frac{e}{L} = \frac{0.586\text{-ft}}{14\text{-ft}} = 0.0419$$



Reinforcement Layer 9

$$M_{R(9)} = \frac{1}{2}\, V_{1(9)}\, L \;=\; \frac{1}{2}\,(6{,}125 \text{ lb/ft})\,(14\text{-ft}) = 42{,}875 \text{ lb-ft}$$

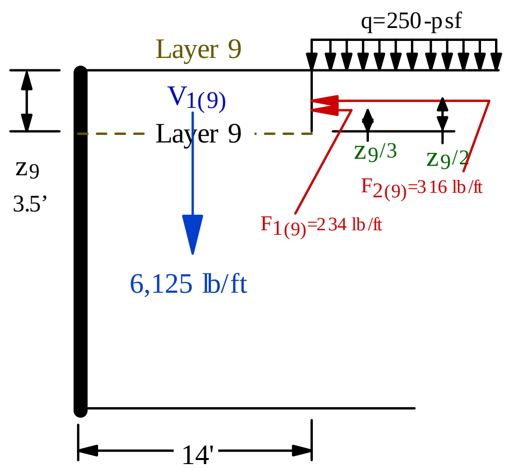
$$M_{O(9)} = \frac{1}{3} F_{1(3)} z_{(9)} + \frac{1}{2} F_{2(3)} z_{(9)} = \frac{1}{3}(234 \text{ lb/ft})(15.5\text{-ft}) + \frac{1}{2}(316 \text{ lb/ft})(15.5\text{-ft}) = 836 \text{ lb-ft}$$

$$\mathbf{FS_{OT(3)}} = \frac{\sum M_{R(6)}}{\sum M_{O(6)}} = \frac{42{,}875 \text{ lb-ft}}{836 \text{ lb-ft}} = 51.25 \;\geq 2.0$$

$$e = \frac{L}{2} - \frac{M_{R(9)} - M_{O(9)}}{R_{BC(9)}} \leq \frac{L}{6}$$

$$e = \frac{14\text{-ft}}{2} - \frac{42{,}875 \text{ lb-ft} - 836 \text{ lb-ft}}{6{,}125 \text{ lb/ft}} = 0.137\text{-ft}$$

$$\frac{e}{L} = \frac{0.137\text{-ft}}{14\text{-ft}} = 0.0098$$

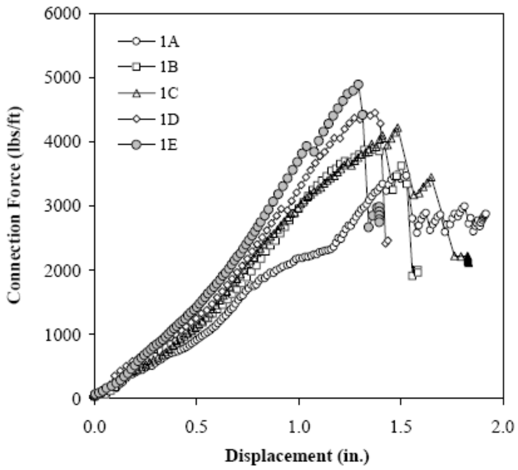


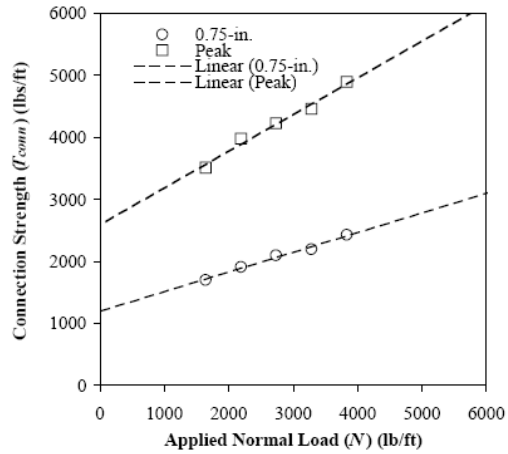
Observation: The factor of safety for overturning increases dramatically as you move from the bottom of wall to the top of wall. Resisting and overturning moments both decrease as you move from the bottom of wall to the top wall with overturning moments decreasing at a much faster rate than the resisting moments; this is due to the fact that the reinforcement length remains constant at L-14-ft throughout the wall height. In actual design overturning for external or internal stability is performed to determine the eccentricity needed for bearing capacity analysis. Furthermore overturning never controls the design reinforcement length that is typically controlled by either sliding at the bottom of the wall pullout at the top of wall or overall global stability.

Stability analysis with respect to Connection Capacity

Connection capacity results for the high strength geotextile indicate the connection capacity is

$T_{conn(n)}$ = 2,585-lb/ft + N tan 31^{o} **Eq. 34**





Test No.	Test Specimen Width (in.)	Test Normal Stress (psi)	Equivalent Normal Load (lb/ft)	Approx. No. of Blocks	Approx. Height (ft)	0.75-in. Strength (lb/ft)	Peak Strength (lb/ft)	Connection Strength Equations (T_{conn})
1A	32.0	11.4	1643	15	15.0	1699	3504	
1B	32.0	15.2	2190	20	20.0	1909	3972	$T_{0.75\text{-in.}}$ = 1195 + (N) tan (18°)
1C	32.0	19.0	2738	25	25.0	2095	4223	
1D	32.0	22.8	3285	30	30.0	2195	4452	T_{peak} = 2585 + (N) tan (31°)
1E	32.0	26.6	3833	35	35.0	2425	4889	

$N_{(n)}$ = $z_{(n)}\,\gamma_{block}$ (this is the normal load on the connection) **Eq. 35**

γ_{block} = 112-lb/ft^3 (provided by block manufacturer based on connection test data)

RF_D = 1.10 (reduction factor for durability at the connection)

RF_{CR} = 1.45 (reduction factor for creep at the connection)

The mode of failure should still be considered to be pullout if longitudinal ribs in geogrids do not rupture, with longitudinal being defined as the direction of the applied load, or for geotextiles if significant ripping of the geotextile perpendicular to the direction of loading does not occur.

- T_{sc} is defined as the peak load per unit reinforcement width obtained in the connection strength test, where pullout is known to be the mode of failure, or the load at which the end of the reinforcement between the facing blocks deflects 15 mm.
- $T_{ultconn}$ is defined as the peak load per unit reinforcement width where rupture is the mode of failure in the connection strength test.
- T_{Lot} is the ultimate wide width tensile strength (ASTM D-4595) for the reinforcement material lot used for the connection strength testing.

$$CR_{u(n)} = \frac{T_{ult\,conn\,(n)}}{T_{Lot\,(n)}}$$ **Eq. 36**

$$CR_{s(n)} = \frac{T_{sc\,(n)}}{T_{Lot\,(n)}}$$ **Eq. 37**

$$T_{ac(rup)} = \frac{T_{Lot} \cdot CR_u}{RF_D \cdot RF_{CR}}$$ (connection capacity based of reinforcement rupture or break) **Eq. 38**

$$T_{ac\,(po)} = (T_{lot})\,(CR_S)$$ (connection capacity based of reinforcement pullout) **Eq. 39**

$$FS_{\,(rup)} = \frac{T_{ac\,(rup)}}{T_{max}}$$ (connection factor of safety based of reinforcement rupture or break) **Eq. 40**

$$FS_{\,(po)} = \frac{T_{ac\,(po)}}{T_{max}}$$ (connection factor of safety based of reinforcement pullout) **Eq. 41**

Reinforcement Layer 3

$T_{max(3)}$ = 1,186-lb/ft (previously calculated when determining FS_{OS})

$z_{(3)}$ = 15.5-ft

$N_{(3)}$ = (15.5-ft) (112-lb/ft^3) = 1,736-lb/ft

$T_{conn(3)}$ = 2,585-lb/ft + 1,736-lb/ft (tan 31^o) = 3,628-lb/ft

$$CR_{u(3)} = \frac{3{,}628\text{-lb/ft}}{7{,}200\text{-lb/ft}} = 0.5039$$

$$CR_{s(3)} = \frac{3{,}628\text{-lb/ft}}{7{,}200\text{-lb/ft}} = 0.5039$$

$$T_{ac(rup)} = \frac{(7{,}200\text{-lb/ft}) \cdot (0.5039)}{(1.10) \cdot (1.45)} = 2{,}275\text{-lb/ft}$$

$$T_{ac(po)} = (7{,}200\text{-lb/ft})\,(0.5039) = 3{,}628\text{-lb/ft}$$

$$FS_{(rup)} = \frac{2{,}275\text{-lb/ft}}{1{,}186\text{-lb/ft}} = 1.92 > 1.50 \;\therefore\; \text{ok}$$

$$FS_{(po)} = \frac{3{,}628\text{-lb/ft}}{1{,}186\text{-lb/ft}} = 3.06 > 1.50 \;\therefore\; \text{ok}$$

Reinforcement Layer 6

$T_{max(6)}$ = 779-lb/ft (previously calculated when determining FS_{OS})

$z_{(6)}$ = 9.5-ft

$N_{(6)} = (9.5\text{-ft})\ (112\text{-lb/ft}^3) = 1{,}064\text{-lb/ft}$

$T_{conn(6)} = 2{,}585\text{-lb/ft} + 1{,}064\text{-lb/ft}\ (\tan 31^o) = 3{,}224\text{-lb/ft}$

$$CR_{u(6)} = \frac{3{,}224\text{ - lb/ft}}{7{,}200\text{ - lb/ft}} = 0.4478$$

$$CR_{s(6)} = \frac{3{,}224\text{ - lb/ft}}{7{,}200\text{ - lb/ft}} = 0.4478$$

$$T_{ac(rup)} = \frac{(7{,}200\text{ - lb/ft}) \cdot (0.4478)}{(1.10) \cdot (1.45)} = 2{,}022\text{-lb/ft}$$

$$T_{ac(po)} = (7{,}200\text{ - lb/ft})\ (0.\ 4478) = 3{,}224\text{-lb/ft}$$

$$FS_{(rup)} = \frac{2{,}022\text{ - lb/ft}}{779\text{ - lb/ft}} = 2.59 > 1.50 \ \therefore \text{ ok}$$

$$FS_{(po)} = \frac{3{,}224\text{ - lb/ft}}{779\text{ - lb/ft}} = 4.14 > 1.50 \ \therefore \text{ ok}$$

Reinforcement Layer 9

$T_{max(9)}$ = 373-lb/ft (previously calculated when determining FS_{OS})

$z_{(9)}$ = 3.5-ft

$N_{(9)} = (3.5\text{-ft})\ (112\text{-lb/ft}^3) = 392\text{-lb/ft}$

$T_{conn(9)} = 2{,}585\text{-lb/ft} + 392\text{-lb/ft}\ (\tan 31^o) = 2{,}821\text{-lb/ft}$

$$CR_{u(9)} = \frac{2{,}821\text{ - lb/ft}}{7{,}200\text{ - lb/ft}} = 0.3917$$

$$CR_{s(9)} = \frac{2{,}821\text{ - lb/ft}}{7{,}200\text{ - lb/ft}} = 0.3917$$

$$T_{ac(rup)} = \frac{(7{,}200\text{ - lb/ft}) \cdot (0.3917)}{(1.10) \cdot (1.45)} = 1{,}768\text{-lb/ft}$$

$$T_{ac(po)} = (7{,}200\text{ - lb/ft})\ (0.3917) = 2{,}821\text{-lb/ft}$$

$$FS_{(rup)} = \frac{1{,}768\text{ - lb/ft}}{373\text{ - lb/ft}} = 4.75 > 1.50 \ \therefore \text{ ok}$$

$$FS_{(po)} = \frac{2{,}821\text{ - lb/ft}}{373\text{ - lb/ft}} = 7.57 > 1.50 \ \therefore \text{ ok}$$

Global Stability Analysis

A global stability analyses is presented for the wall by modeling the same wall height (H=20-ft), live load traffic loading conditions of q=250-psf and soil strength/unit weight properties. The geosynthetic-reinforcement length and vertical spacing modeled in the global stability analyses was determined by hand calculation and verified with program MSEW (3.0) using the FHWA NHI-00-043 methodology as shown in Part 2 of this course.

Global stability analyses are sensitive to soil strength parameters therefore changes, differences or variances between assumed strength parameters that may have been made in stability analyses and actual site specific strength parameters can significantly affect the type and length of the geosynthetic-reinforcement required.

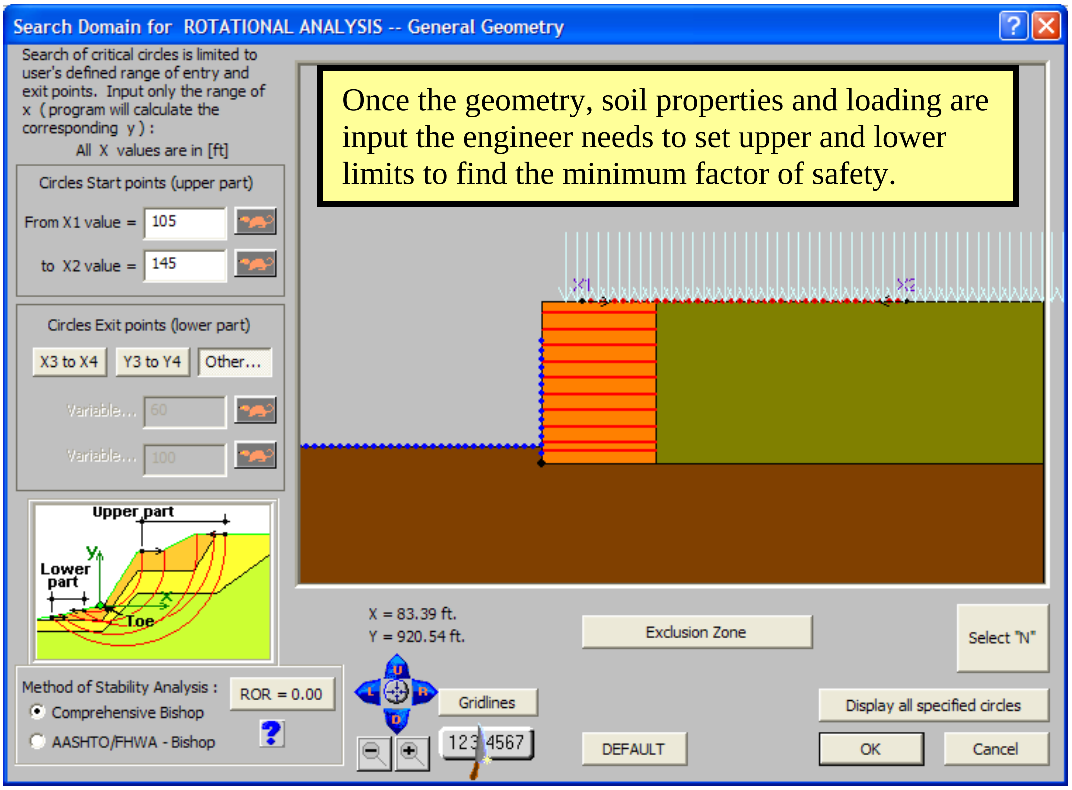
Commentary on Cohesion and Factor of Safety in Global Stability Analyses

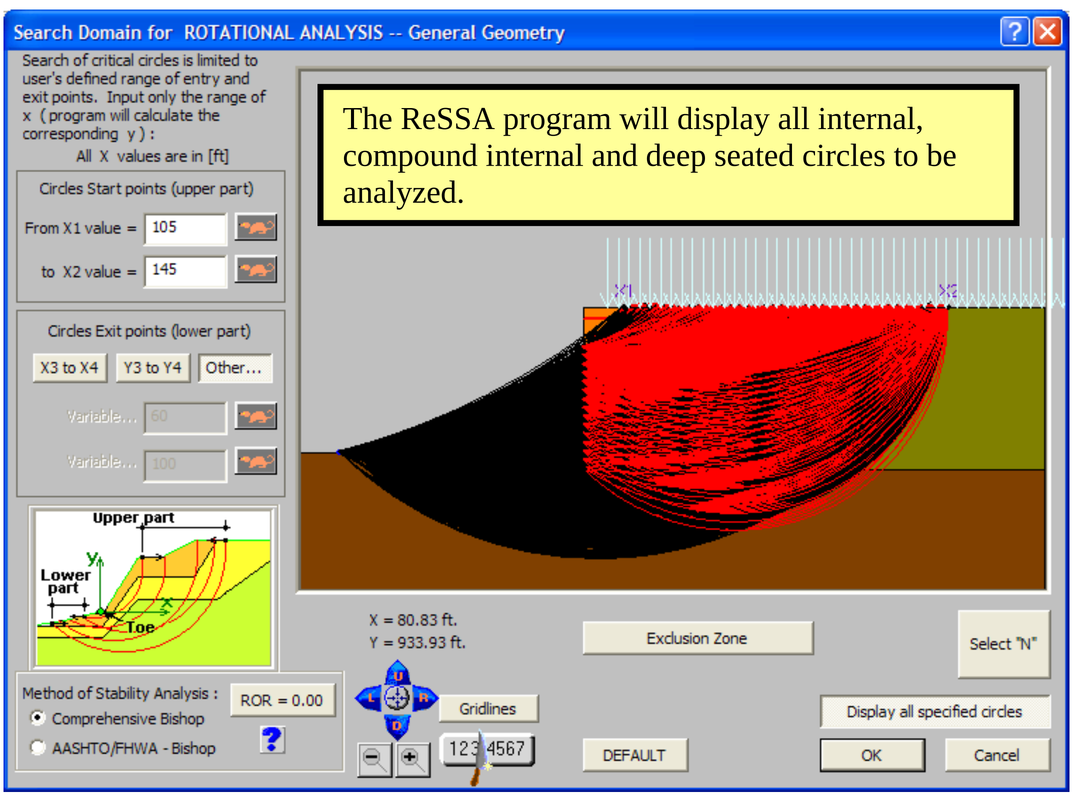
Cohesion has major effects on stability and the long-term effective strength value of cohesion is not always certain. Furthermore, cohesive backfill in made-made embankments (if at all used) is likely to be normally consolidated. Consequently, it is often assumed in practice that for design purposes the apparent cohesion is zero (c=0-psf), especially under drained loading conditions.

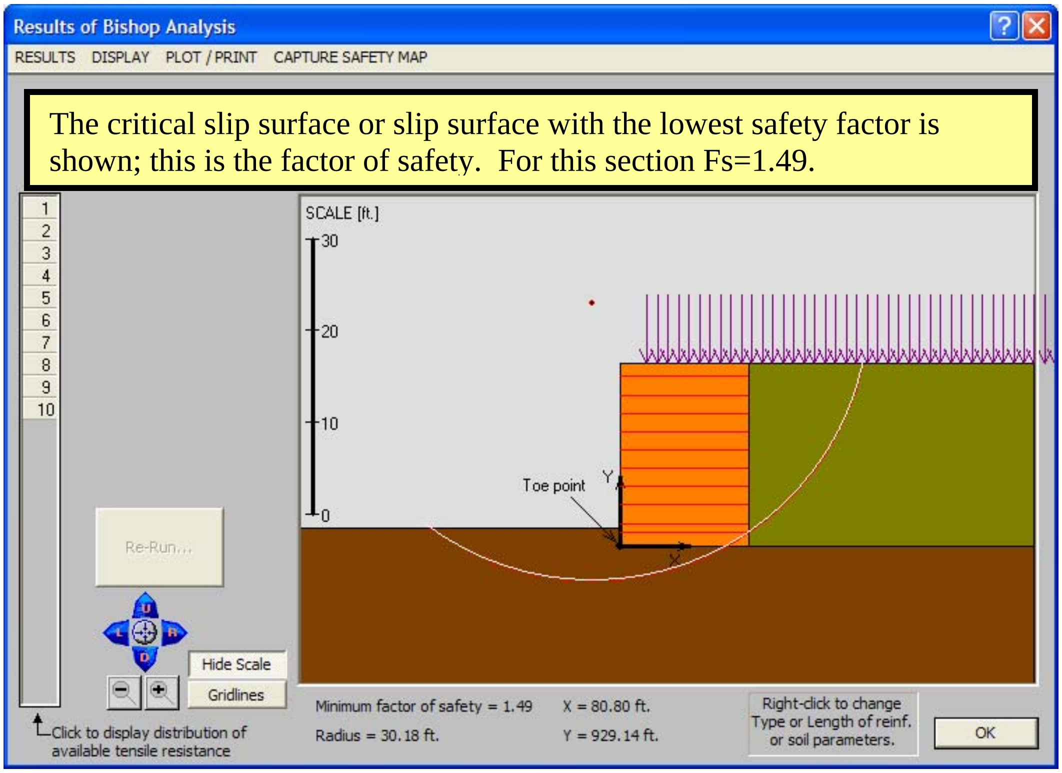
If cohesive fill is used, extreme care should be used when specifying the cohesion value. Cohesion has significant effects on stability and thus the required reinforcement strength. In fact, a small value of cohesion will indicate that no reinforcement at all is needed at the upper portion of a MSE wall. However, over the long-run cohesion of manmade structures tends to drop and nearly diminish. Since long-term stability of MSE walls is of major concern, it is perhaps wise to ignore the cohesion altogether. It is therefore recommended to limit the design value of cohesion to 100-psf but only in residual soil and no cohesion in fill soil.

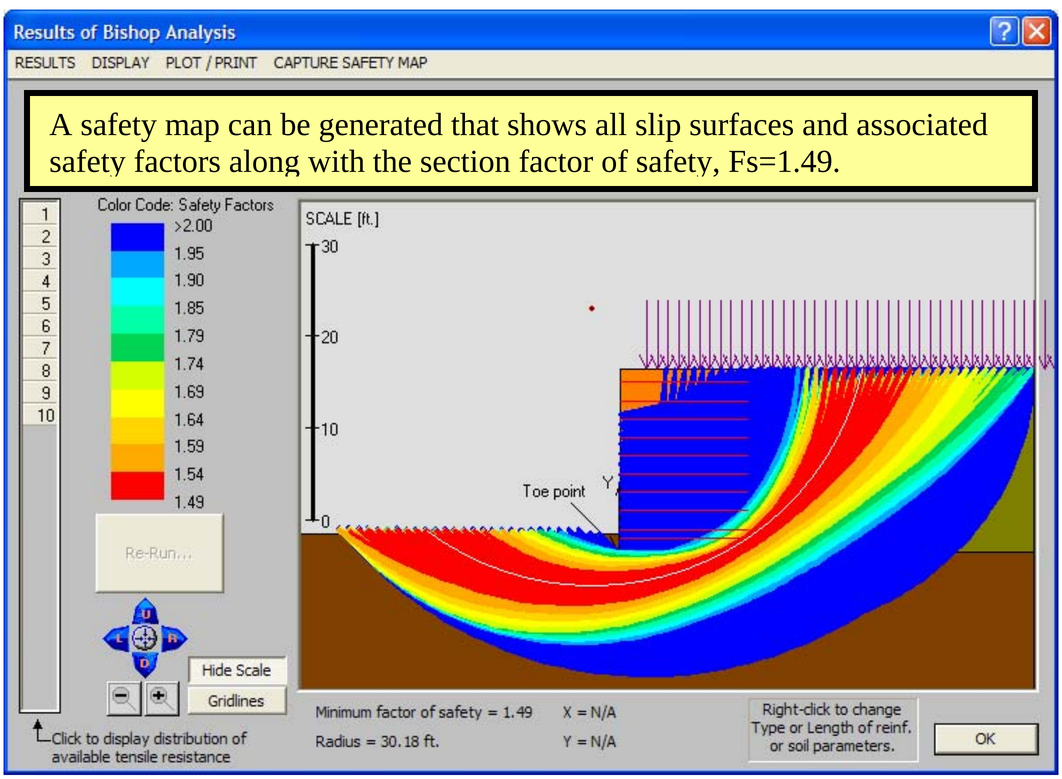
Global stability for this example is analyzed using the commercially available two-dimensional slope stability program ReSSA (v3.0), screen shots provided from program ReSSA by permission of the software developer Adama Engineering (www.geoprograms.com).

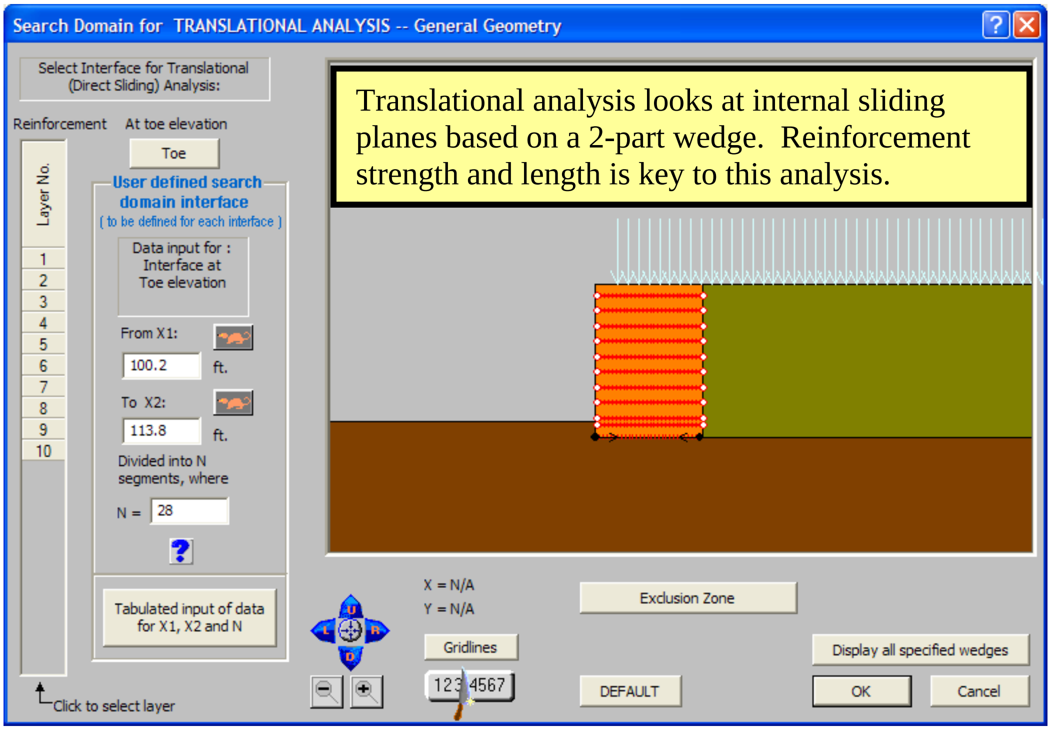
Global stability analyses were performed using Bishop's method of slices (Bishop, 1955), which accounts for moment equilibrium used for circular searches; and Spencer's method (Spencer, 1973), which accounts for force and moment equilibrium used for circular and non-circular searches. Spencer’s factor of safety best represents the most precise factor of safety against global stability for non-circular slip surfaces, although factor of safety results for circular and non-circular slip surfaces are often similar using Bishop, Janbu, or Spencer's method if the slip surface is uniform as in the case of circular surfaces. It is generally accepted to use Bishop's method of slices for circular searches.

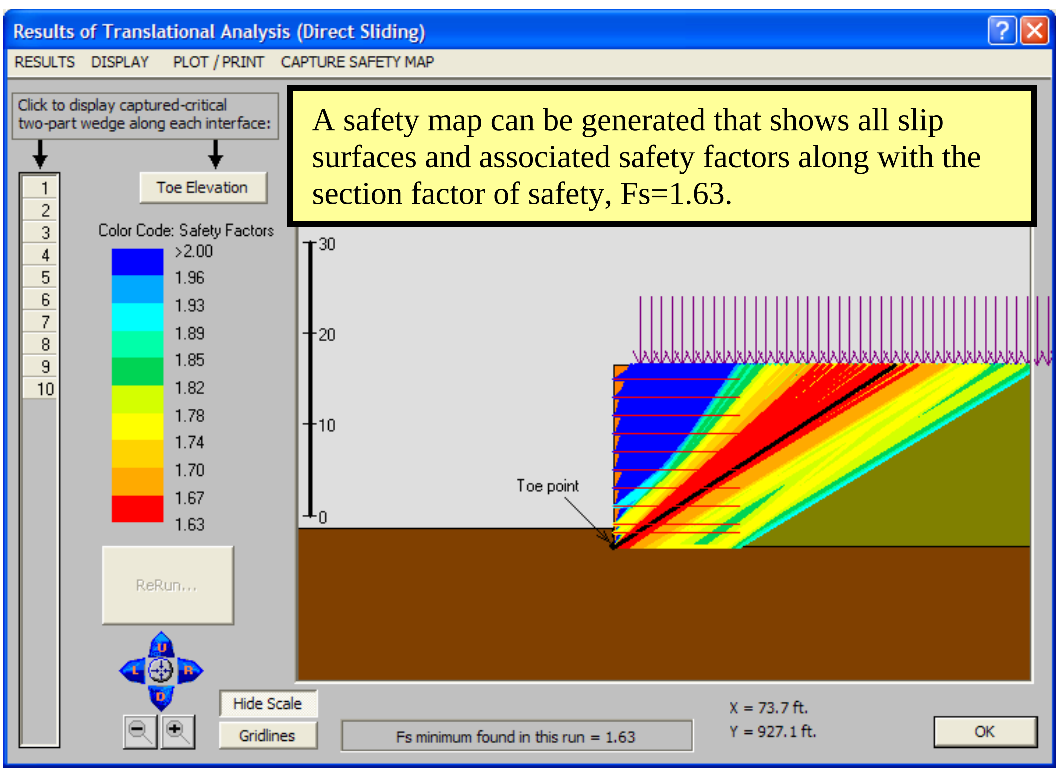

Search Domain for ROTATIONAL ANALYSIS -- General Geometry
Search of critical circles is limited to user's defined range of entry and exit points. Input only the range of x (program will calculate the corresponding y):
All X values are in [ft]
Once the geometry, soil properties and loading are input the engineer needs to set upper and lower limits to find the minimum factor of safety.
Circles Start points (upper part)
From X1 value = 105
to X2 value = 145
Circles Exit points (lower part)
X3 to X4
Y3 to Y4
Other...
Variable... 60
Variable... 100
Upper part
Lower part
Toe
X = 83.39 ft.
Y = 920.54 ft.
Exclusion Zone
Select "N"
Method of Stability Analysis :
Comprehensive Bishop
AASHTO/FHWA - Bishop
ROR = 0.00
Gridlines
Display all specified circles
DEFAULT
OK
Cancel

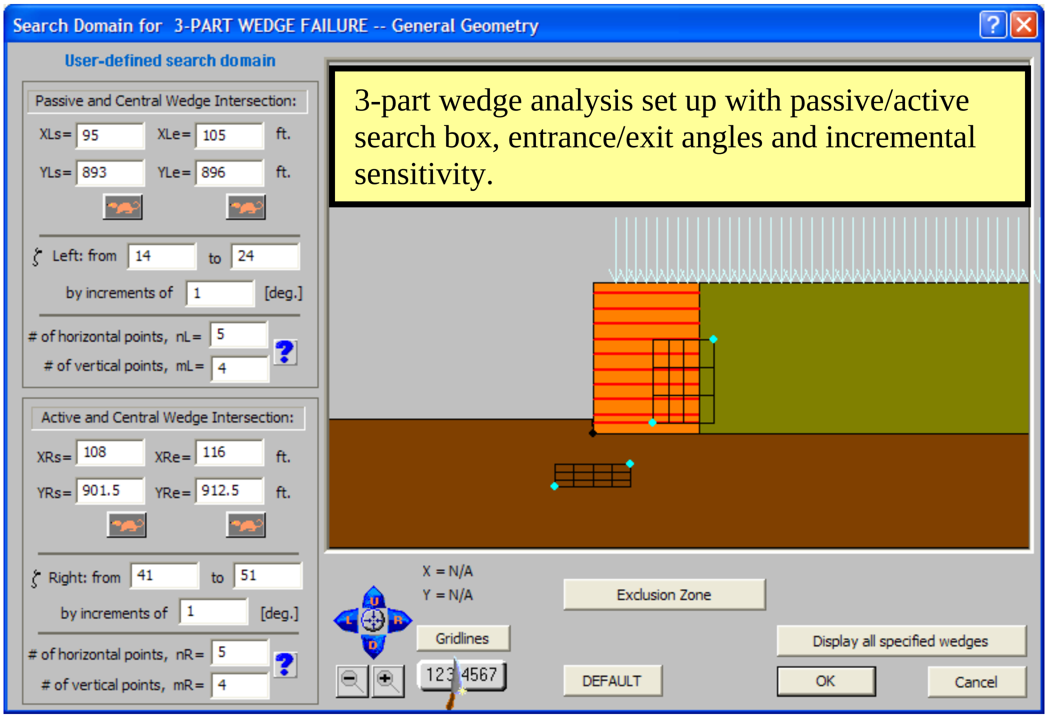

Search Domain for ROTATIONAL ANALYSIS -- General Geometry
Search of critical circles is limited to user's defined range of entry and exit points. Input only the range of x (program will calculate the corresponding y):
All X values are in [ft]
The ReSSA program will display all internal, compound internal and deep seated circles to be analyzed.
Circles Start points (upper part)
From X1 value = 105
to X2 value = 145
Circles Exit points (lower part)
X3 to X4
Y3 to Y4
Other...
Variable... 60
Variable... 100
Upper part
Lower part
Toe
X = 80.83 ft.
Y = 933.93 ft.
Exclusion Zone
Select "N"
Method of Stability Analysis :
Comprehensive Bishop
AASHTO/FHWA - Bishop
ROR = 0.00
Gridlines
Display all specified circles
DEFAULT
OK
Cancel

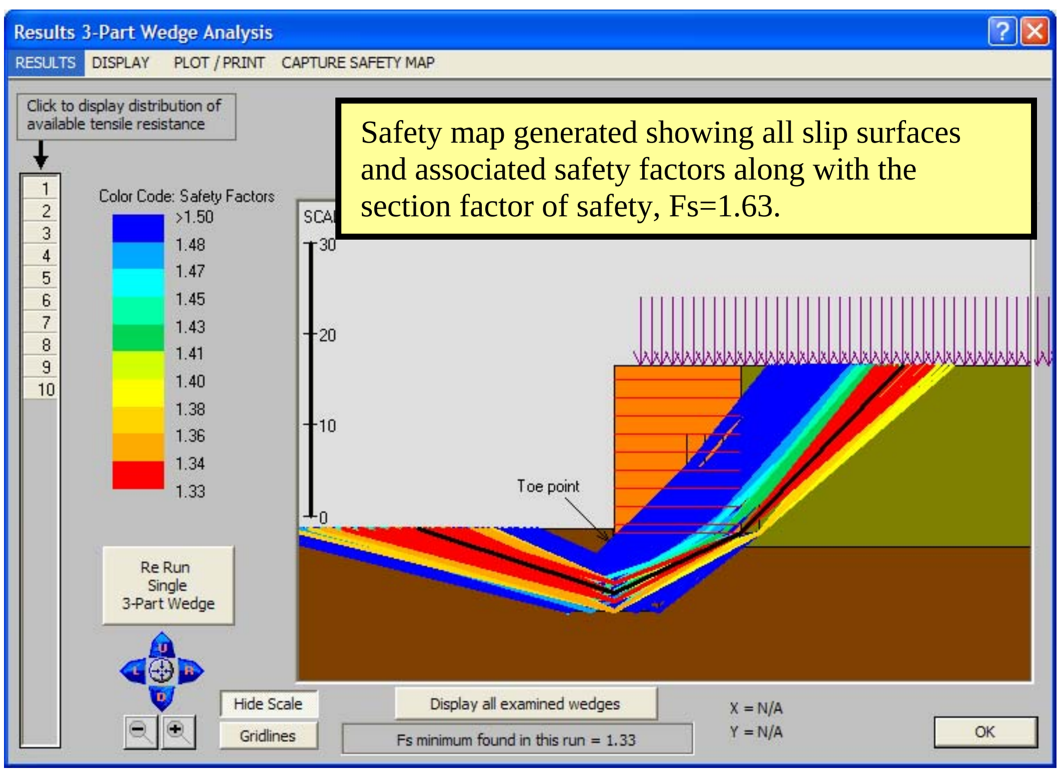

Results of Bishop Analysis
RESULTS DISPLAY PLOT / PRINT CAPTURE SAFETY MAP
The critical slip surface or slip surface with the lowest safety factor is shown; this is the factor of safety. For this section Fs=1.49.
SCALE [ft.]
30
20
10
0
Toe point
Re-Run...
Hide Scale
Gridlines
Click to display distribution of available tensile resistance
Minimum factor of safety = 1.49
Radius = 30.18 ft.
X = 80.80 ft.
Y = 929.14 ft.
Right-click to change Type or Length of reinf. or soil parameters.
OK

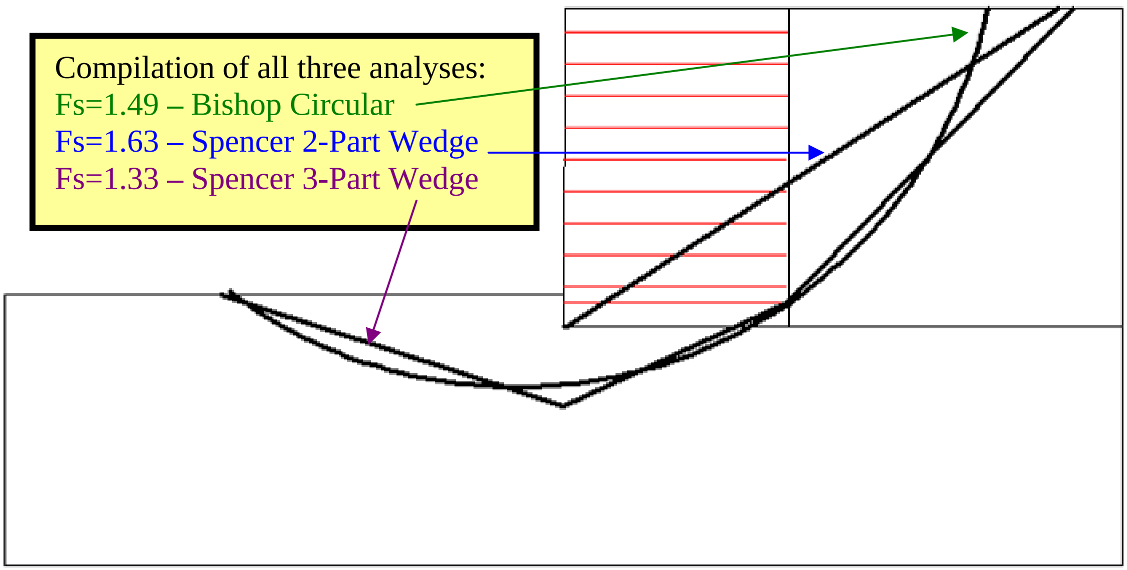

Results of Bishop Analysis
RESULTS DISPLAY PLOT / PRINT CAPTURE SAFETY MAP
A safety map can be generated that shows all slip surfaces and associated safety factors along with the section factor of safety, Fs=1.49.
Color Code: Safety Factors
>2.00
1.95
1.90
1.85
1.79
1.74
1.69
1.64
1.59
1.54
1.49
SCALE [ft.]
30
20
10
0
Toe point
Re-Run...
Hide Scale
Gridlines
Click to display distribution of available tensile resistance
Minimum factor of safety = 1.49
Radius = 30.18 ft.
X = N/A
Y = N/A
Right-click to change Type or Length of reinf. or soil parameters.
OK


Search Domain for TRANSLATIONAL ANALYSIS -- General Geometry
Translational analysis looks at internal sliding planes based on a 2-part wedge. Reinforcement strength and length is key to this analysis.
Select Interface for Translational (Direct Sliding) Analysis:
Reinforcement
At toe elevation
Toe
Layer No.
1
2
3
4
5
6
7
8
9
10
User defined search domain interface
(to be defined for each interface)
Data input for : Interface at Toe elevation
From X1:
100.2
ft.
To X2:
113.8
ft.
Divided into N segments, where
N = 28
Tabulated input of data for X1, X2 and N
Click to select layer
X = N/A
Y = N/A
Gridlines
Exclusion Zone
Display all specified wedges
DEFAULT
OK
Cancel


Results of Translational Analysis (Direct Sliding)
RESULTS
DISPLAY
PLOT / PRINT
CAPTURE SAFETY MAP
A safety map can be generated that shows all slip surfaces and associated safety factors along with the section factor of safety, Fs=1.63.
Click to display captured-critical two-part wedge along each interface:
Toe Elevation
1
2
3
4
5
6
7
8
9
10
Color Code: Safety Factors
>2.00
1.96
1.93
1.89
1.85
1.82
1.78
1.74
1.70
1.67
1.63
30
20
10
0
Toe point
ReRun...
Hide Scale
Gridlines
Fs minimum found in this run = 1.63
X = 73.7 ft.
Y = 927.1 ft.
OK


Search Domain for 3-PART WEDGE FAILURE -- General Geometry
User-defined search domain
Passive and Central Wedge Intersection:
XLs= 95
XLe= 105
ft.
YLs= 893
YLe= 896
ft.
Left: from 14 to 24
by increments of 1 [deg.]
# of horizontal points, nL= 5
# of vertical points, mL= 4
Active and Central Wedge Intersection:
XRs= 108
XRe= 116
ft.
YRs= 901.5
YRe= 912.5
ft.
Right: from 41 to 51
by increments of 1 [deg.]
# of horizontal points, nR= 5
# of vertical points, mR= 4
3-part wedge analysis set up with passive/active search box, entrance/exit angles and incremental sensitivity.
X = N/A
Y = N/A
Exclusion Zone
Gridlines
Display all specified wedges
DEFAULT
OK
Cancel


Results 3-Part Wedge Analysis
RESULTS
DISPLAY
PLOT / PRINT
CAPTURE SAFETY MAP
Click to display distribution of available tensile resistance
1
2
3
4
5
6
7
8
9
10
Color Code: Safety Factors
>1.50
1.48
1.47
1.45
1.43
1.41
1.40
1.38
1.36
1.34
1.33
Safety map generated showing all slip surfaces and associated safety factors along with the section factor of safety, Fs=1.63.
30
20
10
0
Toe point
Re Run Single 3-Part Wedge
Hide Scale
Gridlines
Display all examined wedges
Fs minimum found in this run = 1.33
X = N/A
Y = N/A
OK



Global stability results indicate the most critical slip surface passes below and behind the reinforced zone. Safety maps for all three analyses (circular, 2-part wedge and 3-part wedge) indicate safety factors within the reinforced zone are greater than 1.50. The lowest factors of safety is Fs=1.33, which would meet the minimum requirement for most jurisdictions unless otherwise specified to be greater than Fs=1.30.

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