AB Walls 15 Reinforced Retaining Wall Hand Calculations

WALL NUMBER: Sample Project
CROSS SECTION: 4

These hand calculations are designed to match the output results from AB Walls. The users of these calculations are responsible for the correctness of the input and the results. The user is free to make any changes to any of the equation to alter the design methodology built into AB Walls. To match AB Walls, the user must input all design variables shown in all highlighted boxes.

INPUT INFORMATION

ALLAN BLOCK PARAMETERS

- block height: \( h := 8 \cdot \text{in} \)
- block depth: \( t := 11.875 \cdot \text{in} \)
- block length: \( l := 17.628 \cdot \text{in} \)
- unit percent concrete: \( c := 60 \cdot \% \)
- unit percent voids: \( v := 40 \cdot \% \)
- block setback: \( \omega := 6.42 \text{deg} \)

ASHLAR BLEND (Reduction for Abby Blend included in Europa)

- ASHLAR := 2
  - 1=YES  2=NO

WALL PARAMETERS

- number of block courses: \( n := 9 \)
- total wall height: \( h := n \cdot h = 6 \text{ft} \)
- embedment depth in courses: \( e := 1.245 \)
- total embedment depth: \( D := e \cdot h = 7.47 \text{ft} \)
- geogrid length: \( g := 4 \text{ft} \)
  - top layers
  - typical layers

BASE DIMENSIONS

- footing width: \( L_{\text{width}} := 2.00 \cdot \text{ft} \)
- footing depth: \( L_{\text{depth}} := 0.5 \cdot \text{ft} \)
- toe extension: \( L_{\text{toe}} := -0.5 \cdot \text{ft} \)
- geogrid length: \( L_{\text{grid}} := 0.0 \cdot \text{ft} \)

TUMBLE EUROPA COLLECTION

- TUMBLED := 2
  - 1=YES  2=NO

Preliminary design calculations unless reviewed and certified by a local professional engineer.
SURCHARGE PARAMETERS

surcharge: \[ q = 100.0 \text{ psi} \]
\[ q_x = 7.5 \text{ ft} \]
surcharge type: \[ x_q = 1 \]

Surcharge Type:
1 = Live Load
2 = Dead Load

LINE LOAD PARAMETERS

line load: \[ P = 0 \text{ psi} \]
surcharge type: \[ S_{type} = 1 \]

Contact area boundaries from toe of wall:
starting point: \[ x_1 = 10 \text{ ft} \]
ending point: \[ x_2 = 20 \text{ ft} \]

SURCHARGE PARAMETERS

backslope angle: \[ i = 18.4 \text{ deg} \]
backslope height: \[ h_i = 2 \text{ ft} \]

Designers Notes:
1) \( h_i \) is measured vertically from the top of the top block to the crest of the broken slope.
2) Typical backslopes above walls will not exceed a 2 to 1 horizontal to vertical ratio. The steeper the backslope, the worse affects that are placed on the wall. Applying a broken back slope to your wall design will greatly reduce the pressures compared to a continuous slope above.

USE TRIAL WEDGE METHOD FOR EXTERNAL STABILITY CALCULATIONS

SEISMIC FORCE ANALYSIS METHOD (SFAM) for INTERNAL CALCULATIONS:

\[ TW = 2 \]
1 = YES
2 = NO

Designers Note: Unreinforced slopes above cannot exceed the friction angle of the soil under static conditions. Under seismic conditions, the slope cannot exceed the friction angle of the soil minus the seismic inertial angle. See page 7 for calculations.

If the slope above needs to exceed these maximums, the designer can choose to run the external wall calculations using the Column's Trial Wedge method. If using Trial Wedge under seismic loading the designer must also run the internal calculations using the Trapezoidal Wedge method due to limitations in the Active Wedge weight method. See page 8.
SOIL PARAMETERS: USED IN EXTERNAL, INTERNAL AND BEARING CALCULATIONS

<table>
<thead>
<tr>
<th>INFILL SOIL</th>
<th>RETAINED SOIL</th>
<th>FOUNDATION SOIL (Standard Method)</th>
<th>LEVELING PAD SOIL</th>
</tr>
</thead>
<tbody>
<tr>
<td>friction angle: $\phi_i := 30 \cdot \text{deg}$</td>
<td>friction angle: $\phi_r := 30 \cdot \text{deg}$</td>
<td>friction angle: $\phi_f := 30 \cdot \text{deg}$</td>
<td>friction angle: $\phi_p := 36 \cdot \text{deg}$</td>
</tr>
<tr>
<td>unit weight: $\gamma_i := 120 \cdot \text{pcf}$</td>
<td>unit weight: $\gamma_r := 120 \cdot \text{pcf}$</td>
<td>unit weight: $\gamma_f := 120 \cdot \text{pcf}$</td>
<td>cohesion $c_f := 0 \cdot \text{psi}$</td>
</tr>
</tbody>
</table>

MULTIPLE SOIL TYPE DESIGN PARAMETERS: USED IN INTERNAL COMPOUND STABILITY (ICS) CALCULATIONS ONLY

**Designers Note:** Modeling multiple soil types within the infill mass and the retained soils allows the designer the freedom to more accurately model the actual site conditions. As an example, using this option, the designer could model the lower half of the mass with No-Fines Concrete and the upper half with site soils.

Because this option is only available in the ICS portion of AB Walls, the user should input the lowest friction angle of the three possible in for the correct friction angle box above, used for external, internal and bearings. If the designer uses only the soil parameters above, all the parameters input below should match those above.

**Infill Soils TOP (I 3)**
- friction angle: $\phi_i := 30 \cdot \text{deg}$
- unit weight: $\gamma_i := 120 \cdot \text{pcf}$

**Infill Soils MIDDLE (I 2)**
- friction angle: $\phi_i := 30 \cdot \text{deg}$
- unit weight: $\gamma_i := 120 \cdot \text{pcf}$

**Retained Soils MIDDLE (R 2)**
- friction angle: $\phi_r := 30 \cdot \text{deg}$
- unit weight: $\gamma_r := 120 \cdot \text{pcf}$

**Top of Soil I 2 Height:** $L_2 := 0.6H$  \( L_2 = 3.6 \text{ ft} \)

**Infill Soils BOTTOM (I 1)**
- friction angle: $\phi_i := 30 \cdot \text{deg}$
- unit weight: $\gamma_i := 120 \cdot \text{pcf}$

**Retained Soils BOTTOM (R 1)**
- friction angle: $\phi_r := 30 \cdot \text{deg}$
- unit weight: $\gamma_r := 120 \cdot \text{pcf}$

**Top of Soil R 1 Height:** $R_1 := 0.3H$  \( R_1 = 1.80 \text{ ft} \)

**Retained Soils BOTTOM (R 1)**
- friction angle: $\phi_r := 30 \cdot \text{deg}$
- unit weight: $\gamma_r := 120 \cdot \text{pcf}$

**Top of Soil R 1 Height:** $R_1 := 0.3H$  \( R_1 = 1.80 \text{ ft} \)

**Internal Compound Stability Input Values from AB Walls:**
- course := 0

<table>
<thead>
<tr>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>FSI := 1.60</td>
<td>FSI seismic := 1.60</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Xc := 0.78ft</td>
<td>Yc := 11.87ft</td>
</tr>
<tr>
<td>X2 := 0.99ft</td>
<td>Y2 := 0ft</td>
</tr>
</tbody>
</table>

Preliminary design calculations unless reviewed and certified by a local professional engineer.
**GEOGRID PARAMETERS**

<table>
<thead>
<tr>
<th>Geogrid Type</th>
<th>Long Term Allowable Design Strength</th>
<th>Reduction Factor for Long Term Creep</th>
</tr>
</thead>
<tbody>
<tr>
<td>A: &quot;Strata 200&quot;</td>
<td>LTDS_A = 1613 · pf</td>
<td>RFcr_A = 1.61</td>
</tr>
<tr>
<td>B: &quot;Strata 350&quot;</td>
<td>LTDS_B = 2259 · pf</td>
<td>RFcr_B = 1.61</td>
</tr>
</tbody>
</table>

Factor of safety geogrid overstress (Static): \( F_{S\text{os}_s} = 1.3 \)

Factor of safety geogrid overstress (Seismic): \( F_{S\text{os}_d} = 1.1 \)

**Geogrid Parameters for Pullout of soil:**

\( C_l = 0.7 \)

\( a_{\text{pullout}} = 1.0 \)

**CONNECTION STRENGTH PARAMETERS**

**PEAK CONNECTION CAPACITY**, in the form of a linear equation, \( y = Mx + B \)

where: \( y = \) connection strength and \( x = \) normal load

**GEOGRID TYPE A**

- **Segment #1**
  - Intercept: \( B_{1a} = 1383 \text{pf} \)
  - Slope: \( M_{1a} = \tan(17.7966\text{ - deg}) \)

- **Segment #2**
  - Intercept: \( B_{2a} = 1383 \text{ pf} \)
  - Slope: \( M_{2a} = \tan(17.7966\text{ - deg}) \)

**Intersecting Normal Load**

\( N_{inta} = \frac{B_{2a} - B_{1a}}{M_{1a} - M_{2a}} \)

\( N_{inta} = 0 \cdot \text{lb} \)

\( N_{inta} = 0 \cdot \text{ft} \)

**Maximum tested value:**

\( Max_A = 2087\text{pf} \)

**GEOGRID TYPE B**

- **Segment #1**
  - Intercept: \( B_{1b} = 1257 \text{ · pf} \)
  - Slope: \( M_{1b} = \tan(12.1886\text{ - deg}) \)

- **Segment #2**
  - Intercept: \( B_{2b} = 1257 \text{· pf} \)
  - Slope: \( M_{2b} = \tan(12.1886\text{ - deg}) \)

**Intersecting Normal Load**

\( N_{intb} = \frac{B_{2b} - B_{1b}}{M_{1b} - M_{2b}} \)

\( N_{intb} = 0 \cdot \text{pf} \)

**Maximum tested value:**

\( Max_B = 1979\text{pf} \)

**BLOCK SHEAR PARAMETERS**

**NOTE:** Block - Grid - Block AND Block - Block Shear Results are the same for block with a nominal 6 degree setback or greater.

**SRW UNIT INTERFACE SHEAR DATA (Block - Block)**

- Apparent minimum ultimate shear capacity between segmental units:
  - \( a_u = 2671 \text{ · pf} \)
  - \( a_u\_max = 4706\text{pf} \)
  - \( a_u = 1018 \text{ · pf} \)
  - \( a_u\_max = 6218\text{pf} \)

- Apparent angle of friction between segmental units for peak shear capacity:
  - \( \mu_u = 38\text{ - deg} \)
  - \( \mu_u = 61\text{ - deg} \)

**GEOSYNTHETIC-SRW UNIT INTERFACE SHEAR DATA (Block - Grid - Block)**

- Apparent minimum ultimate service state shear capacity:
  - \( a_{u'} = 2671\text{pf} \)
  - \( a_{u'}\_max = 4706\text{pf} \)
  - \( a_{u'} = 1150\text{pf} \)
  - \( a_{u'}\_max = 5585\text{pf} \)

- Apparent angle of friction between segmental units for service state shear capacity:
  - \( \mu_{u'} = 38\text{ - deg} \)
  - \( \mu_{u'} = 50\text{ - deg} \)

**Note:** Shear Capacity Percentage is used only in the Internal Compound Stability Calculations. This value reduces the allowable face shear: \( \text{Shear Capacity} = 100\% \) of tested values.
SELECT BLOCK LAYERS

<table>
<thead>
<tr>
<th>number of geogrid layers:</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>range of block layers:</td>
<td>k := n..0</td>
</tr>
<tr>
<td>unit_k := k</td>
<td>Elev_k := (unit_k, h)</td>
</tr>
</tbody>
</table>

SRW Course #:

Course Elevation:

NOTE: Course #1 represents the top of leveling pad.

<table>
<thead>
<tr>
<th>unit_k</th>
<th>Elev_k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>6 ft</td>
</tr>
<tr>
<td>8</td>
<td>5.333</td>
</tr>
<tr>
<td>7</td>
<td>4.667</td>
</tr>
<tr>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>3.333</td>
</tr>
<tr>
<td>4</td>
<td>2.667</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>1.333</td>
</tr>
<tr>
<td>1</td>
<td>0.667</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

GEOGRID LAYOUT PARAMETERS

range of geogrid layers: j := g..1

<table>
<thead>
<tr>
<th>geogrid course:</th>
<th>type:</th>
<th>length:</th>
<th>geogrid coursing:</th>
</tr>
</thead>
<tbody>
<tr>
<td>grid_j :=</td>
<td>type_j :=</td>
<td>length_j :=</td>
<td>Ee_j := grid_j \cdot h</td>
</tr>
<tr>
<td>6</td>
<td>A</td>
<td>Ltop</td>
<td>5.333 ft</td>
</tr>
<tr>
<td>5</td>
<td>A</td>
<td>L</td>
<td>4.000 ft</td>
</tr>
<tr>
<td>4</td>
<td>A</td>
<td>L</td>
<td>2.667 ft</td>
</tr>
<tr>
<td>2</td>
<td>A</td>
<td>L</td>
<td>1.333 ft</td>
</tr>
</tbody>
</table>

GEOGRID LAYERS ABOVE THE WALL

Are there Geogrid layers above the wall? Grid_Above := 2

1 for Yes  2 for No

How far above the top block to the first layer of grid: Sabove := 1.0 ft

Spacing between layers: Spacing := 1.50 ft

How many layers above wall are required: Gabove := 3

NOTE: This spreadsheet is set up to have a maximum number of grid above the wall equal to three. If you have less than three, simply input 0 (zero) for the grid length in the Lga array.

Length of Grid and Type: Lgrid_above := 6.5 ft

Lga_ga := type_GA_ga :=

Lgrid_above  A
Lgrid_above  A
Lgrid_above  A

P#: 5

Preliminary design calculations unless reviewed and certified by a local professional engineer.
EFFECTIVE WALL HEIGHT AND BROKEN BACK SLOPE DETERMINATION

\[ s_{\text{w}} := \text{if}\{\omega > 5\text{deg}, 0.1829\text{ft}, 0.1412\text{ft}\} = 0.183 \text{ ft} \]

effective wall height
\[ H_e := \text{if}\{H + h_i < [H + (L - (t - s)) \cdot \tan(i)], H + h_i, H + (L - (t - s)) \cdot \tan(i)\} = 7.062 \text{ ft} \]

BROKEN BACK SLOPE CALCULATIONS, \(i'\):

Determine the effective backslope angle:
Internal Calculations:
\[ i_{\text{int}} := \tan^{-1}\left(\frac{h_i}{2H}\right) = 9.462 \cdot \text{deg} \]
\[ i_{\text{int}} := \text{if}(i_{\text{int}} \geq i, i, i_{\text{int}}) = 9.462 \cdot \text{deg} \]

External Calculations:
\[ H_{\text{max}} := H_e + H_e \cdot \tan(i) = 9.412 \text{ ft} \]
\[ i_{\text{ext}} := \text{if}\{H + h_i < H + (L - (t - s)) \cdot \tan(i), 0\text{deg}, \text{if}\{H + h_i > H_{\text{max}}, i, \tan^{-1}\left(\frac{h_i - (H_e - H)}{H_e}\right)\}\} \]
\[ i_{\text{ext}} := \text{if}(i_{\text{ext}} \geq i, i, i_{\text{ext}}) = 7.563 \cdot \text{deg} \]

CALCULATION OF STATIC AND DYNAMIC EARTH PRESSURE COEFFICIENTS

weighted friction angle: \[ \phi_{wi} := \frac{2}{3} \cdot \phi i \]
\[ \phi_{wi} = 20 \cdot \text{deg} \]
\[ \phi_{wr} := \frac{2}{3} \cdot \phi r \]
\[ \phi_{wr} = 20 \cdot \text{deg} \]
wedge batter:
\[ \beta := 90 \cdot \text{deg} - \omega \]
\[ \beta = 83.58 \cdot \text{deg} \]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
STATIC:  NOTE: IF USING TRIAL WEDGE METHOD FOR CALCULATING EXTERNAL STABILITY
Kar AND Kaer WILL BE CALCULATED AS Kar_TW and Kaer_TW IN THE NEXT SECTION.

Active earth pressure coefficient:
Infill Soil
\[
\Delta_{i_{\text{static}}} = \frac{\sin(\phi_i + \phi_w) \cdot \sin(\phi_i - i\_int)}{\sin(\beta - i\_int)}
\]
\[\Delta_{i_{\text{static}}} = 0.279\]

\[
K_i := \left( \frac{\csc(\beta) \cdot \sin(\beta - \phi_i)}{\sqrt{\sin(\beta + \phi_w)} + \sqrt{\sin(\beta - \phi_i) \cdot \sin(\phi_r - i\_ext)}} \right)^2
\]
\[K_i = 0.286\]

Retained Soil
\[
K_r := \left( \frac{\csc(\beta) \cdot \sin(\beta - \phi_r)}{\sin(\beta + \phi_w) \cdot \sqrt{\sin(\phi_r + \phi_w) \cdot \sin(\phi_r - i\_ext)}} \right)^2
\]
\[K_r = 0.278\]

DYNAMIC: Seismic Coefficients:

<table>
<thead>
<tr>
<th>Internal Stability</th>
<th>External Stability</th>
</tr>
</thead>
<tbody>
<tr>
<td>[K_v = 0]</td>
<td>For: (d_i = 0) in</td>
</tr>
<tr>
<td>[K_{h1} = (1.45 - A_o) \cdot A_o]</td>
<td>For: (d_r = 0) in</td>
</tr>
<tr>
<td>[K_{h1} = 0]</td>
<td>[K_{hr1} := \frac{A_o}{2}]</td>
</tr>
</tbody>
</table>

For \(d_i \geq 1\) in

\[K_{h2} := \left[ \frac{A_o - 0.74 \cdot \text{in}}{d_i} \right]^{0.25}\]

For \(d_r \geq 1\) in

\[K_{hr2} := \left[ \frac{A_o - 0.74 \cdot \text{in}}{d_r} \right]^{0.25}\]

Seismic inertial angle:

<table>
<thead>
<tr>
<th>Internal Stability</th>
<th>External Stability</th>
</tr>
</thead>
<tbody>
<tr>
<td>[\theta_i := \frac{K_{h1}}{1 + K_v} = 0 \cdot \text{deg}]</td>
<td>[\theta_r := \frac{K_{hr1}}{1 + K_v} = 0 \cdot \text{deg}]</td>
</tr>
</tbody>
</table>

Maximum Allowable Slopes in Seismic Conditions

When designing a wall subject to seismic or static loading the designer should understand that there are limitations to the steepness of unreinforced slopes that can be designed and built above any wall.

In static designs, the maximum unreinforced slope above any wall is limited to the internal friction angle of the soil. For seismic designs, the Mononobe-Okabe (M-O) soil mechanics theory gives designers the seismic earth pressure coefficient \(K_{ae}\) to apply to their retaining wall by combining the effects of soil strength \(\phi_r\), slopes above the wall \(i\), wall setback \(\omega\), and seismic inertia angle \(\theta_r\). This equation becomes limited by its mathematics when low strength soils, steep slopes, and high seismic accelerations are combined. This may be translated to say that for specific combinations of slope angles, soil strength and seismic acceleration the project changes from a segmental retaining wall design to a slope stability problem. With a closer look at these three limiting variables the maximum allowable slope in seismic conditions is:

\[i_{\text{max}} := \phi_r - \theta_r \quad i_{\text{max}} = 30 \cdot \text{deg}\]

Note = "Entered slope above does not exceed allowable unreinforced slope"
NOTE: $\Delta l$ and $\Delta r$ are calculated separately to assure that the denominator for the $K_{aei}$ and $K_{aer}$ equations do not go negative under the square root bracket. This only happens when high seismic loads are combined with steep slopes above and poor soils.

\[
\Delta l_{\text{dyn}} := \frac{\sin(\phi_i + \phi w) \cdot \sin(\phi_i - \phi_{\text{l_int}} - \theta_i))}{\cos(\phi w - \omega + \phi_i) \cdot \cos(\omega + \phi_{\text{l_int}})} = 0.287
\]

\[
\Delta r_{\text{dyn}} := \frac{\sin(\phi r + \phi w r) \cdot \sin(\phi r - \phi_{\text{l_ext}} - \theta r)}{\cos(\phi w r - \omega + \phi r) \cdot \cos(\omega + \phi_{\text{l_ext}})} = 0.31
\]

Dynamic earth pressure coefficient:

Infill Soil

\[
K_{aei} := \left(\frac{\cos(\phi_i - \omega + \phi_i)}{\cos(\phi_i) \cdot \cos(\omega) \cdot \cos(\phi w - \omega + \phi_i)}\right)^2 \times \left(1 + \frac{\Delta l_{\text{dyn}} < 0, 0, \sqrt{\Delta l_{\text{dyn}}}}{\Delta l_{\text{dyn}}}ight)
\]

Retained Soil

\[
K_{aer} := \left(\frac{\cos(\phi r - \omega + \phi r)}{\cos(\phi r) \cdot \cos(\omega) \cdot \cos(\phi w r - \omega + \phi r)}\right)^2 \times \left(1 + \frac{\Delta r_{\text{dyn}} < 0, 0, \sqrt{\Delta r_{\text{dyn}}}}{\Delta r_{\text{dyn}}}ight)
\]

\[
K_{aei} := \text{if}(A_o = 0, 0, K_{aei}) \quad K_{aer} := \text{if}(A_o = 0, 0, K_{aer})
\]

When a designer needs to design walls with slopes above steeper than the maximum allowed, they have the option of using the Coulomb Trial Wedge method. This method will provide the active earth force and pressure coefficient to allow the designer to complete the wall design. However, the maximum unreinforced slope described above still holds true. Therefore, if the geometry of the slope exceeds this maximum, they must strongly consider reinforcing the slope above using layers of geogrid and they must review the slope using a global stability program such as ReSSA from ADAMA Engineering (reslope.com), to determine the appropriate length, strength and spacing of the geogrid used to reinforce the slope above.

**Trial Wedge Method of Determining Active Earth Pressure**

The typical seismic design methodology described in this chapter adopts a pseudo-static approach and is generally based on the Mononobe - Okabe (M-O) method to calculate dynamic earth pressures. As described above in the maximum slope above calculation, there is a very distinct limitation to the M-O method. When the designer inputs a slope above the wall that has an incline angle above that exceeds the Internal Friction Angle of the soil minus the seismic inertial angle, the M-O equation for $K_{aei}$ becomes imaginary due to the denominator outputting a negative value. Therefore the maximum unreinforced stable slope above is relative to the magnitude of the seismic coefficient and the strength of soil used in the slope.

The Coulomb Trial Wedge method dates back to 1776 when Coulomb first presented his theory on Active Earth pressures and then again in 1875, when Cullum developed a graphical solution to Coulomb's theory. The Trial Wedge Method has similarities to global stability modeling in that you determine the weight above an inclined wedge behind the wall. By determining the worst case combination of weight and slope angle, the active earth forces for static and seismic conditions can be determined.

![Trial Wedge Method Diagram](image)

**Static and Dynamic Trial Wedge Force Equations:**

\[
P_{ae} = \frac{\text{Weight of Wedge}}{\sin(90^\circ - \omega + \phi_{ae} - \phi_i)}
\]

\[
P_{ae} = \frac{\text{Weight of Wedge}}{\sin(90^\circ - \omega + \phi_{ae} - \phi_i)}
\]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
The Trial Wedge method however, does not have limitation due to slope steepness, soil strength or the magnitude of the seismic coefficient. The trial wedge calculations will provide lateral earth pressure forces no matter the geometry. With this in mind, when using the trial wedge method for walls that exceed the M-O maximum slope, it is mandatory that the user analyze the stability of the slope above the wall in a global stability modeling program. It is strongly recommended that the slope above be reinforced with layers of geogrid similar to those in the reinforced mass, with similar spacing and lengths.

The design process is straightforward using a computer program that allows rapid iterations of calculations to determine the maximum pressure, $P_a$ (static) or $P_{ae}$ (seismic). Similar to a global stability analysis, determining the area of the wedges is the first step. The weight of each wedge is determined and applied downward onto the associated inclined wedge plane to determine the forward pressure. As the wedge weights increase and the inclined plane angle continues to rotate, the combination of weight and angle will combine to find a maximum forward force.

For external sliding, overturning and bearing safety factor equations, the Trial Wedge determined forces will replace those calculated by the standard Coulomb and M-O methods. Please note that the calculated Seismic Inertial Force ($P_{ir}$) is calculated independently of the force method used. This means that $P_{ir}$ is additive to both M-O and Trial Wedge pressure results.

As in the standard Coulomb and M-O methods, the Trial Wedge pressures are applied to the back of the reinforced mass and divided into their horizontal and vertical components. Each are then applied at moment arm locations equal to $1/3^\circ He$ for static and $1/2^\circ He$ for seismic.

Preliminary design calculations unless reviewed and certified by a local professional engineer.
Active Earth Force by Trial Wedge Method:

The first exercise is to calculate the Total Area of each wedge and then each subsequent wedge thereafter. The total wedge area can then be multiplied by the unit weight of soil to determine the weight of wedge (W_Wedge_Area). The surcharge loading is additive to the wedge weight and no distinction is made between Live or Dead Load. The combined weights is given by W_Wedge:

<table>
<thead>
<tr>
<th>Total_Area_2</th>
<th>W_Wedge_area_2</th>
<th>W_Wedge_2</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.894 ft^2</td>
<td>107.23 lb/ft</td>
<td>107.234 lb/ft</td>
</tr>
<tr>
<td>1.805</td>
<td>216.63</td>
<td>216.634</td>
</tr>
<tr>
<td>2.738</td>
<td>328.54</td>
<td>328.54</td>
</tr>
<tr>
<td>3.694</td>
<td>443.33</td>
<td>443.331</td>
</tr>
<tr>
<td>4.678</td>
<td>561.4</td>
<td>561.402</td>
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The Static Active Wedge Pressure Equation:

\[
P_{a,TW} = \begin{cases} 
0 \text{ lbf/ft} & \text{if } W_{\text{Wedge}} \cdot \frac{\sin(\text{APrime}_{z} - \phi_{r})}{\sin(90 \text{deg} - \omega + \phi_{w} - \text{APrime}_{z} + \phi_{r})} < 0 \\
W_{\text{Wedge}} \cdot \frac{\sin(\text{APrime}_{z} - \phi_{r})}{\sin(90 \text{deg} - \omega + \phi_{w} - \text{APrime}_{z} + \phi_{r})} & \text{otherwise}
\end{cases}
\]

Determine the Maximum forward Static Force:

\[P_{a,TW} := \max(P_{a,TW})\]

\[P_{a,TW} = 944.37 \text{ plf}\]

Kar can be back calculated by solving the typical active earth pressure equation for Kar and using the trial wedge determined active force:

\[K_{a,TW} := \frac{P_{a,TW}}{0.5 \cdot \gamma_{r} \cdot H_{e}^2} \quad K_{a,TW} = 0.316\]

Divide the full Static force into horizontal and vertical components to be used in the Sliding and Overturning Safety Factor Equations:

\[P_{a,TW,h} := P_{a,TW} \cdot \cos(\phi_{w}) = 887.418 \text{ plf}\]

\[P_{a,TW,v} := P_{a,TW} \cdot \sin(\phi_{w}) = 322.994 \text{ plf}\]

Dynamic Earth Force by Trial Wedge Method is calculated in the same manner with the inclusion of the Seismic Inertial Angle:

\[
P_{a,TW} = \begin{cases} 
0 \text{ lbf/ft} & \text{if } W_{\text{Wedge}} \cdot \frac{\sin(\text{APrime}_{z} + \theta_{r} - \phi_{r})}{\cos(\theta_{r})} < 0 \\
W_{\text{Wedge}} \cdot \frac{\sin(\text{APrime}_{z} + \theta_{r} - \phi_{r})}{\cos(\theta_{r})} & \text{otherwise}
\end{cases}
\]

Determine the Maximum forward Static Force:

\[P_{a,TW} := \max(P_{a,TW})\]

\[P_{a,TW} = 944.37 \text{ plf}\]

Kaar can be back calculated by solving the typical active earth pressure equation for Kaar and using the trial wedge determined active force:

\[K_{a,TW} := \frac{P_{a,TW}}{0.5 \cdot \gamma_{r} \cdot H_{e}^2} \quad K_{a,TW} = 0.316\]

Subtract static force from the dynamic force to work with separate forces:

\[P_{a,TW} := P_{a,TW} - P_{a,TW} = 0 \text{ plf}\]

Divide the full Seismic force into horizontal and vertical components to be used in the Sliding and Overturning Safety Factor Equations:

\[P_{a,TW,h} := P_{a,TW} \cdot \cos(\phi_{w}) = 0 \text{ plf}\]

\[P_{a,TW,v} := P_{a,TW} \cdot \sin(\phi_{w}) = 0 \text{ plf}\]
EXTERNAL STABILITY

Free Body Diagram
Where:
He=Effective Wall Height
H=Total Wall Height
Wi=Weight of the Backslope
WQ=Infill Surcharge Dead Load
WF=Weight of the Allan Block Facing
WG=Weight of the Geogrid Reinforced Soil Mass
P=Seismic Inertial Force for each Gravity Force
Hr=Resultant Vertical Location
P=Point Load Surcharge
Qpt=Translated Point Load
DFdy=Dynamic Earth Force
FQ=Surcharge Force
FQpt=Point Load Force
Ypt=Translated Point Load Vertical Location
Fa=Active Earth Force

concrete unit weight: \( \gamma_c := 135 \cdot \text{pcf} \)
unit fill unit weight: \( \gamma_f := 120 \cdot \text{pcf} \)

DRIVING FORCE CALCULATIONS

ACTIVE EARTH FORCE:

\[
\begin{align*}
F_a &= \frac{1}{2} \cdot K_a \cdot \gamma_c \cdot H_e^2 \\
F_{ah} &= F_a \cdot \cos(\phi_w) \\
F_{av} &= F_a \cdot \sin(\phi_w)
\end{align*}
\]

\( Fa = 832.988 \cdot \text{plf} \)
\( F_{ah} = 782.753 \cdot \text{plf} \)
\( F_{av} = 284.899 \cdot \text{plf} \)

MOMENT ARMS:

\( Fa = \frac{1}{3} \cdot 832.988 = 2.354 \text{ ft} \)

\( Fa = L + s + \frac{1}{3} \cdot H_e \cdot \tan(\omega) \)

\( Fa = 4.448 \text{ ft} \)

DYNAMIC EARTH FORCE:

\[
\begin{align*}
F_{ae} &= \frac{1}{2} \cdot K_v \cdot K_a \cdot \gamma_c \cdot H_e^2 \\
D_{f_{dy}} &= \begin{cases} 0 & \text{if } A_o = 0, 0 \text{ lb} \text{ ft} \\
D_{f_{dy}} &= \begin{cases} 0 & \text{plf} \\
D_{f_{dy}} &= \begin{cases} 0 & \text{plf} \\
D_{f_{dy}} &= \begin{cases} 0 & \text{plf}
\end{align*}
\]

\( F_{ae} = 0 \cdot \text{plf} \)

Subtract static force from the dynamic force to work with separate forces:

\( DF_{dy} = F_a - Fa = -832.988 \cdot \text{plf} \)

MOMENT ARMS:

\( DF_{dy} = 3.531 \text{ ft} \)

\( DF_{dy} = L + 0.5 \cdot H_e \cdot \tan(\omega) \)

\( DF_{dy} = 4.58 \text{ ft} \)

Preliminary design calculations unless reviewed and certified by a local professional engineer.
Determine the furthest point back from the toe of the wall that ANY surcharge will apply force to the wall (MaxPoint):

\[ ss1 := \frac{H}{\tan\left(45 \cdot \text{deg} + \frac{\phi_r}{2}\right)} \]
\[ ss2 := \frac{(ss1 + L + s - t - H \cdot \tan(\omega)) \cdot \tan(i_{\text{ext}}) \cdot \sin(90 \cdot \text{deg} + i_{\text{ext}})}{\sin\left(45 \cdot \text{deg} + \frac{\phi_r}{2} - i_{\text{ext}}\right)} \cdot \cos\left(45 \cdot \text{deg} + \frac{\phi_r}{2}\right) \]

MaxPoint := L + s + ss1 + ss2

\[ ss1 = 3.464 \text{ ft} \]
\[ ss2 = 0.497 \text{ ft} \]
\[ \text{MaxPoint} = 8.144 \text{ ft} \]

If the surcharge is behind the mass determine the distance from the back of the mass to the face of the square foot surcharge \((qx1)\):

\[ qx1 := [qx - (t + H \cdot \tan(\omega))] \cdot \tan(i_{\text{ext}}) \quad qx1 = 0.775 \text{ ft} \]

Determine the effective height of the square foot surcharge if the force is behind the mass \((Yq_{\text{sf}})\):

\[ Yq_{\text{sf}} := \left[(H + qx1) - (qx - L - s) \cdot \left(\tan\left(45 \cdot \text{deg} + \frac{\phi_r}{2}\right)\right)\right] \cdot \left[1 + \sin\left(45 \cdot \text{deg} + \frac{\phi_r}{2}\right) \cdot \frac{\sin(90 \cdot \text{deg} + \omega) \cdot \tan(\omega)}{\sin\left(45 \cdot \text{deg} + \frac{\phi_r}{2} - \omega\right)}\right] \]

Determine the end of grid at the top of the wall:

\[ Yq_{\text{sf}} = 1.153 \text{ ft} \]
\[ \text{Endg} := L + s + H \cdot \tan(\omega) \quad \text{Endg} = 4.858 \text{ ft} \]

Determine the effective height of the square foot surcharge to use if the force is behind the mass \((He_{\text{q}})\):

\[ He_{\text{q}} := \text{if}(qx < \text{MaxPoint}, \text{if}(qx < \text{Endg}, \text{He}, Yq_{\text{sf}}), 0) \quad He_{\text{q}} = 1.153 \text{ ft} \]

Determine the end of grid at the effective height of the square foot surcharge:

\[ \text{Endg}Yq_{\text{sf}} := L + s + He_{\text{q}} \cdot \tan(\omega) \quad \text{Endg}Yq_{\text{sf}} = 4.313 \text{ ft} \]

**SQUARE FOOT SURCHARGE INFLUENCE:**

(If using Trial Wedge Method Ignore this section)

If the square foot surcharge acts above the mass the applied load is the \(q\) as input above. If the surcharge is applied only behind the mass the load is translated down into the soil to a point at which the force lines intersect the back of the mass. This translation through the soil causes the load to be distributed over a larger footprint. Because the square foot surcharge does not have an ending point like the \(x2\) in the point load calculations the applied load is truncated at the Maxpoint location. The following, \(q_{sfi}\), equation calculates the translated square foot surcharge.

\[ q_{sfi} := \frac{q \cdot (\text{MaxPoint} - qx)}{[(qx - \text{Endg}Yq_{\text{sf}}) \cdot 2 + (\text{MaxPoint} - qx)]} \quad q_{sfi} = 9.171 \cdot \text{psf} \]

Surcharge based on its position relative to the reinforced mass:

\[ q_{sf} := \text{if}(qx < \text{MaxPoint}, \text{if}(qx < \text{Endg}, q, q_{sfi}), 0) \quad q_{sf} = 9.171 \cdot \text{psf} \]
SQUARE FOOT SURCHARGE FORCE:
\[
F_q = \text{if}(qx < \text{MaxPoint}, \text{if}(qx < \text{Endg}, q \cdot \text{Kar}, \text{He}, q_{sf_l} \cdot \text{Kar} \cdot \text{He}_q), 0 \text{pf}) \quad F_q = 2.944 \cdot \text{pf}
\]

\[
F_{qh} = F_q \cdot \cos(\phi \text{wr}) \quad F_{qh} = 2.767 \cdot \text{pf}
\]

\[
F_{qAr} := 0.5 \cdot \text{He} \cdot q \quad F_{qAr} = 0.577 \text{ ft}
\]

\[
F_{vq} := \text{if}(x_q = 2, F_q \cdot \sin(\phi \text{wr}), 0 \text{pf}) \quad F_{vq} = 0 \cdot \text{pf}
\]

\[
F_{qArm} := L + s + 0.5 \cdot \text{He} \cdot \tan(\omega) \quad F_{qArm} = 4.248 \text{ ft}
\]

LINE LOAD SURCHARGE:

(If using trial wedge method ignore this section)

If the surcharge is behind the mass determine the distance from the back of the mass to the face of the square foot surcharge (Qx1):

\[
Qx_1 := [x_1 - (L + \text{H} \cdot \tan(\omega))] \cdot \tan(i\_ext)
\]

Determine the effective height of the square foot surcharge if the force is behind the mass (Q\_pt):

\[
Q_{\_pt} := \left( H + Qx_1 \right) - (x_1 - L - s) \cdot \left( \tan\left( 45^\circ \cdot \text{deg} + \frac{\phi r}{2} \right) \right) \quad 1 + \sin\left( 45^\circ \cdot \text{deg} + \frac{\phi r}{2} \right) \cdot \left( \frac{\sin(90^\circ \cdot \text{deg} + \omega) \cdot \tan(\omega)}{\sin(45^\circ \cdot \text{deg} + \frac{\phi r}{2} - \omega)} \right)
\]

\[
Q_{\_pt} = -3.326 \text{ ft}
\]

Determine the effective height of the line load surcharge to use if the force is behind the mass (He\_Q):

\[
He\_Q := \text{if}(x_1 < \text{MaxPoint}, \text{if}(x_1 < \text{Endg}, He, Q_{\_pt}), 0\text{ft}) \quad He\_Q = 0 \text{ ft}
\]

Location of the end of the grid at the YQ\_pt elevation:

\[
\text{Endg}\_YQ\_pt := L + s + He\_Q \cdot \tan(\omega) \quad \text{Endg}\_YQ\_pt = 4.183 \text{ ft}
\]

If the ending position of the line load surcharge (x2) is beyond the
MaxPoint of influence the load is truncated at the MaxPoint location:

\[
x_2 := \text{if}(x_2 > \text{MaxPoint}, \text{MaxPoint}, x_2) = 8.144 \text{ ft}
\]

If the line load surcharge acts above the mass the applied load is the P as input above. If the surcharge is applied only
behind the mass the load is translated down into the soil to a point at which the force lines intersect the back of the mass.
This translation through the soil causes the load to be distributed over a larger footprint. The following, Q\_pti, equation calculates the translated square foot surcharge.

\[
Q_{\_pti} := \frac{P \cdot (x_2 - x_1)}{(x_2 - x_1)} \quad Q_{\_pti} = 0 \cdot \text{psf}
\]

Point Load Surcharge Influence

If the point load contacts only with the reinforced mass it will add stability to the wall structure, therefore the loads are only considered in the internal stability calculations.

\[
Q_p := \text{if}(x_2 \geq \text{Endg}, Q_{\_pt}, 0\text{lb}) \quad Q_p = 0 \cdot \text{psf}
\]

If the point load contacts in beyond the reinforced mass and its influence zone buffer it will only affect the external stability. If it overlaps both the influence zone and retained soil it will affect both internal and external stability.

\[
Q_{\_pt} := \text{if}(x_1 \geq \text{Endg}, Q_{\_pt}, Q_{\_pt}) \quad Q_{\_pt} = 0 \cdot \text{psf}
\]

\[
Q_{\_pt} := \text{if}(x_1 < \text{MaxPoint}, Q_{\_pt}, 0\text{psf}) \quad Q_{\_pt} = 0 \cdot \text{psf}
\]

Note:
Q\_pt is the translated distributed point load surcharge used to determine the point load force that will be influencing the external stability of the retaining wall structure. Q\_pt is a function of the location of the contact area with respect to the geogrid reinforcement. Q\_pt will be used to calculate the point load surcharge if it acts directly on top of the reinforced soil. No translation calculations are necessary for Q\_p because its applications area is on top of the reinforced mass and its influence zone buffer.
POINT LOAD SURCHARGE FORCE:

\[ F_{Qpt} := Qpt \cdot \text{Kar} \cdot \text{He}_Q = 0 \cdot \text{plf} \]

\[ F_{Qpt} := F_{Qpt} \cdot \cos(\phi \text{w}) = 0 \cdot \text{plf} \]

\[ F_{Qpt} := \text{if} (\text{Stype} = 2, F_{Qpt} \cdot \sin(\phi \text{w}), 0 \text{plf}) = 0 \cdot \text{plf} \]

POINT LOAD SURCHARGE WEIGHT:

\[ W_{Qpt1} := \text{Qpi} \cdot (x_2 - x_1) = 0 \cdot \text{plf} \]

\[ W_{Qpt2} := \text{Qpi} \cdot (\text{Endg} - x_1) = 0 \cdot \text{plf} \]

\[ W_{Qpt} := \text{if}(x_2 \leq \text{Endg}, W_{Qpt1}, W_{Qpt2}) \]

\[ W_{Qpt} := \text{if}(x_1 > \text{Endg}, 0 \text{plf}, W_{Qpt}) \]

\[ W_{Qpt} = 0 \cdot \text{plf} \]

RESISTING FORCE CALCULATIONS:

WEIGHT OF THE BACKSLOPE:

\[ W_i := 0.5 \cdot \gamma_r \cdot (\text{He}_H \cdot [L - (t - s)]) \]

\[ W_i = 203.531 \cdot \text{plf} \]

Determine the position of the square foot surcharge (qp):

\[ q_p := \text{if}(x_q < \text{Endg}, \text{if}(x_q > H \cdot \tan(\omega) + t, (\text{Endg} - q_x, L - (t - s)), 0 \text{ft}) \]

\[ q_p = 0 \text{ ft} \]

MOMENT ARM for Weight of Dead Load Surcharge:

\[ W_{Qpt} := \text{if}(x_q < \text{Endg}, \text{if}(x_q > H \cdot \tan(\omega) + t, \frac{1}{2} q_p + q_x, \frac{1}{2} [L - (t - s)] + H \cdot \tan(\omega) + t, 0 \text{ft}) \]

\[ W_{Qpt} = 0 \text{ ft} \]

WEIGHT OF THE DEAD LOAD SURCHARGE:

\[ W_q := \text{if}(x_q = 2, (q_p \cdot q, 0 \text{plf}) \]

\[ W_q = 0 \cdot \text{plf} \]

WEIGHT OF THE FACING:

\[ W_f := H \cdot t \cdot (c \cdot \gamma_c + v \cdot \gamma_u) \]

\[ W_f = 765.937 \cdot \text{plf} \]

WEIGHT OF THE REINFORCED SOIL MASS:

\[ W_r := H \cdot [L - (t - s)] \cdot \gamma_i \]

\[ W_r = 2299.188 \cdot \text{plf} \]

TOTAL WEIGHT:

\[ W_t := W_f + W_r \]

\[ W_t = 3065.125 \cdot \text{plf} \]

MOMENT:

\[ W_{Qpt} := 0.5 \cdot (L + s) + 0.5 \cdot H \cdot \tan(\omega) \]

\[ W_{Qpt} = 2.429 \text{ ft} \]
If Trial Wedge calculations are used, the active force calculated must replace the Cullomb active forces.

\[
\text{aa} = "\text{Not Using Trail Wedge}"
\]

\[
\text{Sliding\_Force\_Static} := \text{if}(T W = 1, P_{a\_TW\_v}, F_{av})
\]

\[
\text{Sliding\_Force\_Seismic} := \text{if}(T W = 1, P_{a\_TW\_v} + P_{ae\_TW\_v}, F_{av} + D_{Fdynv})
\]

The sliding calculations should use the less of the infill or foundation soils to calculated sliding resistance:

\[
\text{SLIDING RESISTANCE:}
\]

\[
\text{Fr\_static} := (\text{Sliding\_Force\_Static + Fqv} + F_{Qptv} + W_{l} + W_{q} + W_{f} + W_{s} + W_{Qp}) \cdot \tan(\text{Sliding\_Friction\_Angle})
\]

\[
\text{Fr\_static} = 2051.646 \cdot \text{plf}
\]

\[
\text{Fr\_seismic} := (\text{Sliding\_Force\_Seismic + Fqv} + F_{Qptv} + W_{l} + W_{q} + W_{f} + W_{s} + W_{Qp}) \cdot \tan(\text{Sliding\_Friction\_Angle})
\]

\[
\text{Fr\_seismic} = 2051.646 \cdot \text{plf}
\]

\[
\text{SEISMIC INERTIAL FORCE:}
\]

The weight of each component of the wall structure has a horizontal inertial force acting at its centroid during a seismic event. The three components that have this inertial force are the block facing the reinforced soil mass and the back slope soil. The resultant \( \text{Pir} \) is the sum of all three. The weight of the reinforced soil mass and the back slope soil is based on a reinforcement length of 0.5H.

- weight of the block face: \( W_{f} = 765.937 \cdot \text{plf} \)
- weight of the reinforced soil mass: \( W_{s}' := [0.5 \cdot H - (t - s)] \cdot \gamma_{i} \cdot H \) \( W_{s}' = 1579.188 \cdot \text{plf} \)
- weight of the back slope soil: \( W_{f}' := \frac{1}{2} \cdot [0.5 \cdot H - (t - s)] ^ {2} \cdot \gamma_{r} \cdot \tan(i) \) \( W_{f}' = 96.017 \cdot \text{plf} \)

\[
\text{SEISMIC INERTIAL FORCE:}
\]

\[
\text{Pir} := K_{hr} \cdot (W_{f} + W_{s}' + W_{f}') \quad \text{Pir} = 0 \cdot \text{plf}
\]

\[
\text{MOIEMENT ARM:}
\]

\[
H_{r} := \frac{K_{hr} \cdot W_{f}' \cdot \frac{H}{2} + K_{hr} \cdot W_{s}' \cdot \frac{H}{2} + K_{hr} \cdot W_{f}' \cdot [H + \frac{1}{3} \cdot [0.5 \cdot H - (t - s)] \cdot \tan(i)]}{\text{Pir}}
\]

\[
H_{r} = 0
\]

\[
\text{EXTERNAL STABILITY FACTORS OF SAFETY}
\]

\[
\text{STATIC HORIZONTAL FORCE:}
\]

\[
\text{aa} = "\text{Not Using Trail Wedge}"
\]

\[
P_{a\_h} := \text{if}(T W = 1, P_{a\_TW\_h}, F_{ah} + F_{qh} + F_{Qpth}) = 785.519 \cdot \text{plf}
\]

\[
P_{ae\_h} := \text{if}(T W = 1, P_{a\_TW\_h} + P_{ae\_TW\_h} + \text{Pir}, F_{ah} + D_{Fdynh} + F_{qh} + F_{Qpth} + \text{Pir}) = 785.519 \cdot \text{plf}
\]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
FACTOR OF SAFETY FOR SLIDING:

Static Conditions: \( FS_{\text{staticsliding}} \geq 1.5 \)

\[
FS_{\text{staticsliding}} := \frac{Fr_{\text{static}}}{Pa_h} \quad \text{FS}_{\text{staticsliding}} = 2.61
\]

Seismic Conditions: \( FS_{\text{seismicsliding}} \geq 1.1 \)

\[
FS_{\text{seismicsliding}} := \frac{Fr_{\text{seismic}}}{Pa_{ae_h}} \quad \text{FS}_{\text{seismicsliding}} = 2.61
\]

FACTOR OF SAFETY FOR OVERTURNING:

NOTE For overturning calculations, we use the same moment arms for both M_O and Trial Wedge method. This is possible because we separated the static and seismic forces in the Trial Wedge calculations above.

Static Conditions: \( FS_{\text{staticoverturning}} \geq 2.0 \)

\[
\text{STATIC OVERTURNING MOMENT:} \quad \text{aa = "Not Using Trail Wedge"}
\]

\[
M_P_a := \text{if}(TW = 1, P_{a\_TW\_h} \cdot Fa_{Armh}, Fa_h \cdot Fa_{Armh} + Fq_h \cdot Fq_{Armh} + FQpth \cdot FQptArmh) \]

\[
www := \text{if}(TW = 1, P_{a\_TW\_v} \cdot Fa_{Armv}, Fa_v \cdot Fa_{Armv} + Fq_v \cdot Fq_{Armv} + FQptv \cdot FQptArmv)
\]

\[
FS_{\text{staticoverturning}} := \frac{Wt \cdot Wi_{Arm} + Wi \cdot Wi_{Arm} + Wq \cdot Wq_{Arm} + WQp \cdot WQpt_{Arm} + www}{M_P_a}
\]

\[FS_{\text{staticoverturning}} = 5.143\]

Seismic Conditions: \( FS_{\text{seismicoverturning}} \geq 1.5 \)

\[
\text{SEISMIC OVERTURNING MOMENT:} \quad \text{aa = "Not Using Trail Wedge"}
\]

\[
\text{False := Fa}_{h} \cdot Fa_{Armh} + DF_{dynh} \cdot DF_{dynArmh} + Fq_{h} \cdot Fq_{Armh} + FQpth \cdot FQptArmh + Pir \cdot Hir \]

\[
M_P_{ae} := \text{if}(TW = 1, P_{a\_TW\_h} \cdot Fa_{Armh} + P_{ae\_TW\_h} \cdot DF_{dynArmh} + Pir \cdot Hir, False) \]

\[
\text{False := Fa}_{v} \cdot Fa_{Armv} + DF_{dynv} \cdot DF_{dynArmv} + Fq_{v} \cdot Fq_{Armv} + FQptv \cdot FQptArmv \]

\[
www := \text{if}(TW = 1, P_{a\_TW\_v} \cdot Fa_{Armv} + P_{ae\_TW\_v} \cdot DF_{dynArmv}, False)
\]

\[
FS_{\text{seismicoverturning}} := \frac{Wt \cdot Wi_{Arm} + Wi \cdot Wi_{Arm} + Wq \cdot Wq_{Arm} + WQp \cdot WQpt_{Arm} + www}{M_P_{ae}}
\]

\[FS_{\text{seismicoverturning}} = 5.143\]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
BEARING CAPACITY CALCULATIONS: Standard Method

Vertical Force Resultant Using M_O Determined Forces:

\[ R_{M,O} := Wf + Ws + Wi + Wq + F_{av} + DF_{dynv} + F_{qv} + FQ_{ptv} + WQpt \]

Vertical Force Resultant Using Trial Wedge Determined Forces:

\[ R_{TW} := Wf + Ws + Wi + Wq + P_{a-TW.v} + P_{ae-TW.v} + F_{qv} + FQ_{ptv} + WQpt \]

\[ R := \text{if} \left( TW = 1, R_{TW}, R_{M,O} \right) \quad R = 3553.555 \cdot \text{plf} \]

Location of the Resultant Force:

\[ wwww := \text{if} \left( TW = 1, P_{a-TW,v} \cdot FaArmv + P_{ae-TW,v} \cdot DF_{dynArmv}, Fav \cdot FaArmv + DF_{dyv} \cdot DF_{dynArmv} \right) \]

positive := Wt \cdot WtArm + Wi \cdot WtArm + Wq \cdot WqArm + WQpt \cdot WQptArm + Fqv \cdot FqArmV + FQptv \cdot FQptArm + wwww

\[ \text{positive} = 9484.51 \text{ lb} \]

\\[ \text{False} := FaAv \cdot FaArmh + DF_{dynh} \cdot DF_{dynArmh} + Fqh \cdot FqArmh + FQpth \cdot FQptArmh \]

\\[ wwww := \text{if} \left( TW = 1, P_{a-TW,h} \cdot FaArmh + P_{ae-TW,h} \cdot DF_{dynArmh}, \text{False} \right) \]

negative := Pir \cdot Hir + wwww

\[ \text{negative} = 1844.27 \text{ lb} \]

\[ x := \frac{\text{positive} - \text{negative}}{R} \quad x = 2.15 \text{ ft} \]

Determine the eccentricity, E, of the resultant vertical force. If the eccentricity is negative the maximum bearing pressure occurs at the heel of the mass. Therefore, a negative eccentricity causes a decrease in pressure at the toe. For conservative calculations E will always be considered greater than or equal to zero.

\[ E := 0.5 \cdot (L + s) - x \quad E = -0.059 \text{ ft} \]

\[ E1 := \text{if} (E < 0 \text{ ft}, 0 \text{ ft}, E) \quad E1 = 0 \text{ ft} \]

Determine the average bearing pressure acting at the centerline of the wall.

\[ \sigma_{avg} := \frac{R}{(L + s)} \quad \sigma_{avg} = 849.543 \cdot \text{psf} \]

Determine the moment about the centerline of the wall due to the resultant bearing load.

\[ Mcl := R \cdot E1 \quad Mcl = 0 \text{ lb} \cdot \text{ft} \]

Section Modulus:

\[ S := \frac{(1.0 \cdot \text{ft}) \cdot (L + s)^2}{6} \quad S = 2.916 \text{ ft}^3 \]

Differenced in bearing pressure due to the eccentric loading.

\[ \sigma_{mom} := \frac{Mcl \cdot 1 \cdot \text{ft}}{S} \quad \sigma_{mom} = 0 \cdot \text{psf} \]

\[ \text{therefore:} \]

\[ \sigma_{max} := \sigma_{avg} + |\sigma_{mom}| \quad \sigma_{max} = 849.543 \cdot \text{psf} \]

\[ \sigma_{min} := \sigma_{avg} - |\sigma_{mom}| \quad \sigma_{min} = 849.543 \cdot \text{psf} \]

ALLEAN BLOCK BEARING PRESSURE ANALYSIS
ULTIMATE BEARING CAPACITY CALCULATION:

Meyerhof bearing capacity equation:

$$\sigma_{ult} = 1/2 \cdot \gamma_f \cdot L \cdot N_q + cf \cdot N_c + \gamma_f \cdot (L \cdot D) \cdot N_q$$

Where:

$$N_q := \exp(\pi \cdot \tan(\phi f)) \cdot \left(\tan\left(\frac{45 \cdot \deg + \phi f}{2}\right)\right)^2$$

$$N_q = 18.401$$

$$N_c := (N_q - 1) \cdot \cot(\phi f)$$

$$N_c = 30.14$$

$$N_\gamma := (N_q - 1) \cdot \tan(1.4 \cdot \phi f)$$

$$N_\gamma = 15.668$$

Therefore:

$$\sigma_{ult \_ abc} := \frac{1}{2} \cdot \gamma_f \cdot L \cdot \gamma \cdot N_\gamma + cf \cdot N_c + \gamma_f \cdot (L \cdot D) \cdot N_q = 4816.984 \cdot \text{psf}$$

$$\text{FSbearing\_abc := } \frac{\sigma_{ult\_abc}}{\sigma_{max}}$$

$$\text{FSbearing\_abc = 5.67}$$

**Meyerhof Method as used by the NCMA:**

Note: The NCMA bearing capacity method is less conservative than the Modified Meyerhof method utilized by Allan Block. The NCMA distributes the entire bearing load over the geogrid footprint and does not focus it on the size of the leveling pad. Therefore if the user chooses to use the Meyerhof NCMA method the $$\sigma_{ult}$$ equation simply uses L for the bearing width. Please note that the NCMA uses the Vesic equation for $$N_\gamma$$.

Vesic $$N_\gamma$$ for NCMA Methodology:

$$N_\gamma\_ves := 2 \cdot (N_q + 1) \cdot \tan(\phi f)$$

$$N_\gamma\_ves = 22.402$$

Determine the effective length of the bearing pad

$$b := (L + s) - 2E1$$

$$b = 4.183 \text{ ft}$$

Determine the applied load on the bearing pad

$$Q_a := \frac{(Wf + Ws + Wl + Wq + Wqpt)}{b}$$

$$Q_a = 781.433 \cdot \text{psf}$$

$$\sigma_{ult\_ncma} := \frac{1}{2} \cdot \gamma_f \cdot b \cdot N_\gamma\_ves + cf \cdot N_c + \gamma_f \cdot D \cdot N_q$$

$$\sigma_{ult\_ncma} = 7455.193 \cdot \text{psf}$$

$$\text{FSbearing\_ncma := } \frac{\sigma_{ult\_ncma}}{Q_a}$$

$$\text{FSbearing\_ncma = 9.54}$$

$$\sigma_{ult} := \text{if(bearing = 1, } \sigma_{ult\_abc}, \sigma_{ult\_ncma})$$

$$\sigma_{max} := \text{if(bearing = 1, } \sigma_{max}, Q_a)$$

Factor of safety:

$$\text{FSbearing := } \frac{\sigma_{ult}}{\sigma_{max}}$$

$$\text{FSbearing = 5.67}$$

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NCMA BEARING PRESSURE ANALYSIS

Preliminary design calculations unless reviewed and certified by a local professional engineer.
INTERNAL STABILITY

Free Body Diagram

Where:
Gj = Depth to each geogrid layer
Acj = Influence area of each geogrid layer
He,i = Effective wall height for internal stability
h_vc = Height above wall to the geometric vertical center of the slope

Area_a

\[ a := \frac{1}{3} (H - H_e) = 0.354 \text{ ft} \]

Area_b

\[ b := \frac{1}{2} (H - H_e) = 0.531 \text{ ft} \]

Note:
For internal stability calculations sample calculations will be shown for grid layer #1. All other grid layers will be shown through tabular calculations at the end of this section.

DETERMINATION OF THE FORCE ACTING ON EACH GRID LAYER

STATIC LOADS, use the subscript “s”

\[ a_g := \frac{H \text{ + Sabove} + \text{grid}_j \cdot h}{2} \]
\[ b_g := \frac{\text{grid}_j \cdot h + \text{grid}_{j-1} \cdot h}{2} \]

a_g = 6.167 ft  H = 6 ft  b_g = 4.667 ft
Sabove = 1 ft  He,i = 6.354 ft  Weak := if(a_g < He,i, a_g - b_g, He,i - b_g)

P#: 20
influence area:

\[ A_{ij} := \begin{cases} 1, & \text{if } g < 2, (\text{He}_i - \text{grid}_j \cdot h + \text{grid}_j \cdot h) \cdot (\text{He}_i - \text{grid}_j \cdot h) \cdot (\text{He}_i - \text{grid}_j \cdot h) \cdot (\text{He}_i - \text{grid}_j \cdot h) \\ 0, & \text{otherwise} \end{cases} \]

active earth pressure per grid layer:

\[ R_{ij} := \begin{cases} \text{g, if } (\text{Grid\_Above} = 1, \text{He}_i - \text{a}_j + \text{He}_i - \text{b}_j, \text{He}_i - \text{b}_j) \end{cases} \]

\[ G_{ij} := \begin{cases} 0.5 \cdot \text{g, if } (\text{ij} = 1), \text{He}_i - \frac{(\text{grid}_j \cdot h + \text{grid}_j \cdot h)}{2} + \text{He}_i, \text{He}_i - \frac{(\text{grid}_j \cdot h + \text{grid}_j \cdot h)}{2} + \text{He}_i - \text{b}_j \end{cases} \]

\[ F_{ai} := \text{Kai} \cdot \cos(\phi \text{wi}) \cdot \gamma_i \cdot A_{ij} \cdot G_{ij} \]

\[
\begin{align*}
\text{Static Loads: Internal Force} \\
\text{Grid Layers:} & \\
\text{Fai}_{ij} = & \\
\text{G}_{ij} = & \\
\end{align*}
\]

surcharge pressure:

\[ F_{ai} := \begin{cases} \text{qg, if } (\text{qg} > \text{Endg}, 0 \text{plf}, q \cdot \text{Kai} \cdot \cos(\phi \text{wi}) \cdot A_{ij}) \end{cases} \]

point load surcharge pressure:

\[ F_{opt} := \begin{cases} \text{q_1, if } (x_1 > \text{Endg}, 0 \text{plf}, q_1 \cdot (\text{Kai} \cdot \cos(\phi \text{wi})) \cdot A_{ij}) \end{cases} \]

SEISMIC (DYNAMIC) LOADS: use the subscript, "d"

Indication of Coulomb failure surface for internal stability (\(\alpha_i\)):

\[ \alpha_i := \arctan \left[ \frac{-\tan(\omega) + \sqrt{[\tan(\omega) \cdot (\tan(\phi + \omega) \cdot \cot(\phi + \omega)) \cdot (1 + \tan(\phi + \omega) \cdot \tan(\phi + \omega))]} + \phi}{1 + \tan(\phi + \omega) \cdot (\tan(\phi + \omega) \cdot \tan(\phi + \omega))} \right] \]

Weight of the active wedge in the infill zone:

\[ W_{Ai} := \frac{1}{2} \cdot \gamma_i \cdot H^2 \cdot \frac{\sin(90 \deg - \omega - \alpha_i)}{\sin(\alpha_i) \cdot \cos(\omega)} \]

Weight of the active wedge in the backslope:

\[ D_1 := \frac{H \cdot \sin(90 \ deg - \omega - \alpha_i)}{\cos(\omega) \cdot \sin(\alpha_i)} \]

\[ D_2 := \frac{D_1 \cdot \sin(i) \cdot \sin(\alpha_i)}{\sin(\alpha_i - i)} \]

\[ W_{As} := \begin{cases} \text{if } (i > 0, 1 \cdot D_1 \cdot D_2 \cdot \gamma_i \cdot 0 \text{plf}) \end{cases} \]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
dynamic earth pressure based on Active Wedge theory:

\[
DF_{dyni\_SW_j} := K_{hi} \cdot (W_{Ai} + W_{As}) \cdot \frac{A_{ci}}{H_{ei}}
\]

\[
DF_{dyni\_SW_j} = 0 \cdot \text{pf}
\]

\[
\sum DF_{dyni\_SW} = 0 \cdot \text{pf}
\]

**Active Wedge theory:**

**Trapezoidal theory:**

\[
DF_{dyni\_Trap_j} := (0.5) \cdot (K_{aei} - K_{ai}) \cdot \cos(\phi_{wi}) \cdot \gamma_i \cdot H_{ei} \cdot A_{ci}
\]

**Trapezoidal theory:**

\[
DF_{dyni\_Trap_j} =
\begin{align*}
&-172.816 \\
&-136.552 \\
&-136.552 \\
&-204.828 \\
\end{align*}
\]

\[
false_j := \text{if} \left( \sum DF_{dyni\_Trap} > \sum DF_{dyni\_SW, DF_{dyni\_Trap}, DF_{dyni\_SW_j}} \right)
\]

\[
DF_{dyni_j} := \text{if} \left( SFAM = 1, DF_{dyni\_Trap_j}, \text{if} (SFAM = 2, DF_{dyni\_SW_j}, false_j) \right)
\]

\[
DF_{dyni_1} = 0 \cdot \text{pf}
\]

seismic inertial force:

\[
P_{ri_j} := K_{hi} \cdot t \cdot (c \cdot \gamma c + v \cdot \gamma uf) \cdot A_{ci}
\]

\[
P_{ri_1} = 0 \cdot \text{pf}
\]

**TENSILE FORCE ON EACH GRID:**

**STATIC:**

\[
F_{ij} := F_{aij} + F_{qij} + F_{qptij}
\]

\[
F_{is_1} = 345.185 \cdot \text{pf}
\]

**SEISMIC:**

\[
F_{id_j} := F_{aij} + F_{qij} + F_{qptij} + DF_{dyni_j} + P_{ri_j}
\]

\[
F_{id_1} = 345.185 \cdot \text{pf}
\]
GEOTEXTILE TENSILE OVERSTRESS

\( \text{GEOGRID/BLOCK CONNECTION CAPACITY} \)

\( \text{normal load:} \quad N_{j} := \left( H - \text{grid}_{j} \cdot h \right) \cdot \left( c \cdot \gamma c + \gamma v \cdot \gamma uf \right) \cdot t \quad N_{s} = 595.729 \cdot \text{plf} \)

\( \text{peak connection strength:} \quad F_{cs,j} := \begin{cases} A, & \text{if} \left( N_{j} < \text{Ninta}, B1a + M1a - N_{j}, B2a + M2a \cdot N_{j} \right), \text{if} \left( N_{j} < \text{Nintb}, B1b + M1b - N_{j}, B2b + M2b \cdot N_{j} \right) \end{cases} \)

\( \text{Does calculated value exceed that maximum tested?} \)

\( F_{cs,j} := \begin{cases} A, & \text{if} \left( F_{cs,j} < \text{Max}_{A}, F_{cs,j}, \text{Max}_{A} \right), \text{if} \left( F_{cs,j} < \text{Max}_{B}, F_{cs,j}, \text{Max}_{B} \right) \end{cases} \)

\( F_{cs} = 1574.229 \cdot \text{plf} \)

TUMBLED REDUCTION FACTOR

\( \text{TRF} := \begin{cases} \text{TUMBLED} = 1, 0.7, 1.0 \end{cases} \quad \text{TRF} = 1 \)

FACTOR OF SAFETY CONNECTION STRENGTH, Static:

\( F_{\text{conn},s_{j}} := \frac{(\text{TRF} \cdot \text{ARF}) \cdot F_{cs,j}}{F_{s,j}} \quad F_{\text{conn},s_{1}} = 6.837 \)

ASHLAR REDUCTION FACTOR

\( \text{ARF} := \begin{cases} \text{ASHLAR} = 1, 0.9, 1.0 \end{cases} \quad \text{ARF} = 1 \)

FACTOR OF SAFETY CONNECTION STRENGTH, Seismic:

\( F_{\text{conn},d_{j}} := \frac{(\text{TRF} \cdot \text{ARF}) \cdot F_{cs,j}}{F_{d,j}} \quad F_{\text{conn},d_{1}} = 6.837 \)

Preliminary design calculations unless reviewed and certified by a local professional engineer.
GEOGRID PULLOUT FROM THE SOIL:

Equations for each segment of the line of maximum tension:
segment #1: \( y_1 = \tan(45^\circ - \phi/2) \cdot (x-t) \)
segment #2: \( x = H \cdot (0.3 + \tan(\omega)) + t \)
where: \( x \) = distance to the line of maximum tension

Setting these two equations equal to each other yields the elevation of their intersection point:
\[
y_{\text{int}} = \tan \left( \frac{45^\circ \cdot \deg + \frac{\phi}{2}}{2} \right) \cdot H \cdot (0.3 + \tan(\omega))
\]
\[
y_{\text{int}} = 4.287 \text{ ft}
\]

Therefore, the length of geogrid embedded beyond the line of maximum tension is the following:
End of Geogrid Location \( L_{\text{EG}} := \text{length}_j + s + \tan(\omega) \cdot (\text{grid}_j \cdot h) \)

Line of Maximum Tension for Bi-Linear - Static:
For geogrid elevation < \( y_{\text{int}} \)
\[
S_{\text{MT1}} := \frac{\text{grid}_j \cdot h}{\tan \left( \frac{45^\circ \cdot \deg + \frac{\phi}{2}}{2} \right)} + t
\]
For geogrid elevations > \( y_{\text{int}} \)
\[
S_{\text{MT2}} := H \cdot (0.3 + \tan(\omega)) + t
\]
\[
S_{\text{MT1}} := \text{if} \left( \text{grid}_j \cdot h < y_{\text{int}}, S_{\text{MT1}}, S_{\text{MT2}} \right)
\]
\[
S_{\text{MT1}} = 1.759 \text{ ft}
\]

Line of Maximum Tension for Linear Plane - dynamic:
\[
D_{\text{MTj}} := t + \text{grid}_j \cdot h \cdot \tan(90^\circ - \alpha i)
\]
\[
D_{\text{MT1}} = 2.191 \text{ ft}
\]

geogrid embedment length within infill zone - Static:
\[
L_{\text{ei}} := \lfloor \text{length}_j > L, \text{EG}_j - (\text{length}_j - L) - S_{\text{MT1}}, \text{EG}_j - S_{\text{MT1}} \rfloor
\]
\[
L_{\text{ei}} = 2.574 \text{ ft}
\]

geogrid embedment length within retained zone - Static:
\[
L_{\text{er}} := \lfloor \text{length}_j > L, (\text{length}_j - L), 0 \rfloor
\]
\[
L_{\text{er}} = 0 \text{ ft}
\]

geogrid embedment length within infill zone - dynamic:
\[
L_{\text{ei}} := \lfloor \text{length}_j > L, \text{EG}_j - (\text{length}_j - L) - D_{\text{MT1}}, \text{EG}_j - D_{\text{MT1}} \rfloor
\]
\[
L_{\text{ei}} = 2.142 \text{ ft}
\]

geogrid embedment length within retained zone - dynamic:
\[
L_{\text{er}} := \lfloor \text{length}_j > L, (\text{length}_j - L), 0 \rfloor
\]
\[
L_{\text{er}} = 0 \text{ ft}
\]

geogrid length affected by surcharge within infill zone - Static:
\[
L_{\text{qi}} := \lfloor q_x < \text{EG}_j - (\text{length}_j - L), \lfloor q_x > S_{\text{MT1}}, \text{EG}_j - q_x - (\text{length}_j - L), \text{EG}_j - S_{\text{MT1}} - (\text{length}_j - L) \rfloor, 0 \text{ft} \rfloor
\]
\[
L_{\text{qi}} = 0 \text{ ft}
\]

geogrid length affected by surcharge within retained zone - Static:
\[
L_{\text{qr}} := \lfloor q_x < \text{EG}_j, \lfloor q_x > \text{EG}_j - (\text{length}_j - L), \text{EG}_j - q_x - (\text{length}_j - L) \rfloor, 0 \text{ft} \rfloor
\]
\[
L_{\text{qr}} = 0 \text{ ft}
\]

geogrid length affected by surcharge within infill zone - dynamic:
\[
L_{\text{qi}} := \lfloor q_x < \text{EG}_j - (\text{length}_j - L), \lfloor q_x > D_{\text{MT1}}, \text{EG}_j - q_x - (\text{length}_j - L), \text{EG}_j - D_{\text{MT1}} - (\text{length}_j - L) \rfloor, 0 \text{ft} \rfloor
\]
\[
L_{\text{qi}} = 0 \text{ ft}
\]

geogrid length affected by surcharge within retained zone - dynamic:
\[
L_{\text{qr}} := \lfloor q_x < \text{EG}_j, \lfloor q_x > \text{EG}_j - (\text{length}_j - L), \text{EG}_j - q_x - (\text{length}_j - L) \rfloor, 0 \text{ft} \rfloor
\]
\[
L_{\text{qr}} = 0 \text{ ft}
\]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
geogrid length affected by a point load within the infill zone - Static:

For $x_1 <$ the line of maximum tension
\[ \text{LiQpt1}_s_j := \begin{cases} 1 & \text{if} \left[ x_2 < EG_j - (\text{length}_j - L), 0 \leq x_2 - x_2 - \text{S}_j \right] \end{cases} \]

For $x_1 >$ the line of maximum tension and $x_1 <$ the end of the infill zone
\[ \text{LiQpt2}_s_j := \begin{cases} 1 & \text{if} \left[ x_2 < EG_j - (\text{length}_j - L), x_2 - x_1, EG_j - x_1 - (\text{length}_j - L) \right] \end{cases} \]

For $x_1 <$ the end of the infill zone
\[ \text{LiQpt3}_s_j := 0 \cdot ft \]

\[ \text{LiQpt}_s_j := \begin{cases} 1 & \text{if} \left[ S_{type} = 1, 0 \cdot ft, \text{LiQpt}_s_j \right] \end{cases} \]

\[ \text{LiQpt}_s_j = 0 \cdot ft \]

point load retained geogrid length - Static:

For $x_1 <$ the infill zone
\[ \text{LrQpt1}_s_j := \begin{cases} 1 & \text{if} \left[ x_2 < EG_j, 0 \leq x_2 - [EG_j - (\text{length}_j - L)], 0 \cdot ft \right] \end{cases} \]

For $x_1 >$ the infill zone and $x_1 <$ the end of the geogrid
\[ \text{LrQpt2}_s_j := \begin{cases} 1 & \text{if} \left[ x_2 < EG_j, x_2 - x_1, EG_j - x_1 - (\text{length}_j - L) \right] \end{cases} \]

For $x_1 <$ the end of the geogrid
\[ \text{LrQpt3}_s_j := 0 \cdot ft \]

\[ \text{LrQpt}_s_j := \begin{cases} 1 & \text{if} \left[ S_{type} = 1, 0 \cdot ft, \text{LrQpt}_s_j \right] \end{cases} \]

\[ \text{LrQpt}_s_j = 0 \cdot ft \]

geogrid length affected by a point load within the infill zone - Dynamic:

For $x_1 <$ the line of maximum tension
\[ \text{LiQpt1}_d_j := \begin{cases} 1 & \text{if} \left[ x_2 < EG_j - (\text{length}_j - L), 0 \leq x_2 - \text{D}_j \right] \end{cases} \]

For $x_1 >$ the line of maximum tension and $x_1 <$ the end of the infill zone
\[ \text{LiQpt2}_d_j := \begin{cases} 1 & \text{if} \left[ x_2 < EG_j - (\text{length}_j - L), x_2 - x_1, EG_j - x_1 - (\text{length}_j - L) \right] \end{cases} \]

For $x_1 <$ the end of the infill zone
\[ \text{LiQpt3}_d_j := 0 \cdot ft \]

\[ \text{LiQpt}_d_j := \begin{cases} 1 & \text{if} \left[ S_{type} = 1, 0 \cdot ft, \text{LiQpt}_d_j \right] \end{cases} \]

\[ \text{LiQpt}_d_j = 0 \cdot ft \]

point load retained geogrid length - Static:

For $x_1 <$ the infill zone
\[ \text{LrQpt1}_d_j := \begin{cases} 1 & \text{if} \left[ x_2 < EG_j, 0 \leq x_2 - [EG_j - (\text{length}_j - L)], 0 \cdot ft \right] \end{cases} \]

For $x_1 >$ the infill zone and $x_1 <$ the end of the geogrid
\[ \text{LrQpt2}_d_j := \begin{cases} 1 & \text{if} \left[ x_2 < EG_j, x_2 - x_1, EG_j - x_1 - (\text{length}_j - L) \right] \end{cases} \]

For $x_1 =$ the end of the geogrid
\[ \text{LrQpt3}_d_j := 0 \cdot ft \]

\[ \text{LrQpt}_d_j := \begin{cases} 1 & \text{if} \left[ S_{type} = 1, 0 \cdot ft, \text{LrQpt}_d_j \right] \end{cases} \]

\[ \text{LrQpt}_d_j = 0 \cdot ft \]
Determine the distance down to each layer of geogrid:

\[ G_m := \begin{cases} 1 & \text{if } g < 2, \ He_i - \sum \text{Elev_Grid}, He_i - \text{grid}_j, h \end{cases} \]

\[ G_1 = 5.021 \text{ ft} \]

pullout capacity - Static:

\[ F_{p0_s_j} := 2 \cdot C_l \cdot \tan(\phi) \cdot \left[ G_j \cdot \gamma_i \cdot \text{Lei}_j + q \cdot \left( Lq_i_s_j + Qpi \cdot \text{LiQpt}_s_j \right) \right] \]

\[ F_{p0_s_j} := 2 \cdot C_l \cdot \tan(\psi) \cdot \left[ G_j \cdot \gamma_r \cdot \text{Ler}_j + q \cdot \left( Lq_r_s_j + Qpi \cdot \text{LrQpt}_s_j \right) \right] \]

\[ F_{po_s_j} := F_{p0_s_j} + Fr_{po_s_j} \]

\[ F_{po_s_1} = 1253.285 \cdot \text{ plf} \]

pullout capacity - dynamic:

\[ F_{p0_d_j} := 2 \cdot C_l \cdot tan(\phi) \cdot \left[ G_j \cdot \gamma_i \cdot \text{Lei}_p_d_j + q \cdot \left( Lq_i_d_j + Qpi \cdot \text{LiQpt}_d_j \right) \right] \]

\[ F_{p0_d_j} := 2 \cdot C_l \cdot \tan(\psi) \cdot \left[ G_j \cdot \gamma_r \cdot \text{Ler}_p_d_j + q \cdot \left( Lq_r_d_j + Qpi \cdot \text{LrQpt}_d_j \right) \right] \]

\[ F_{p0_d_1} := F_{p0_d_1} + Fr_{p0_d_1} \]

\[ F_{po_d_1} = 1043.075 \cdot \text{ plf} \]

**FACTOR OF SAFETY GEOGRID PULLOUT, static:**

\[ FS_{pullout_s_j} := \frac{F_{po_s_j}}{F_{lsj}} \]

\[ FS_{pullout_s_1} = 3.631 \]

**FACTOR OF SAFETY GEOGRID PULLOUT, dynamic:**

\[ FS_{pullout_d_j} := \frac{F_{po_d_j}}{F_{ldj}} \]

\[ FS_{pullout_d_1} = 3.022 \]

**GEOGRID EFFICIENCY**

Static Conditions:

\[ e_{f_{f_j}} := \frac{F_{ls_j}}{LTDS \cdot \frac{1}{FS_{os_s}}} \cdot 100 \]

\[ e_{f_{f_1}} = 32.1 \]

Seismic Conditions:

\[ e_{f_{f_d_j}} := \frac{F_{ld_j}}{RTDS \cdot \frac{1}{FS_{os_d}}} \cdot 100 \]

\[ e_{f_{f_d_1}} = 14.621 \]
LOCALIZED STABILITY, TOP OF THE WALL STABILITY

LOCAL WALL PARAMETERS:

\[
\text{unreinforced height: } H_{\text{top}} = H - \text{gridg} \cdot h \quad H_{\text{top}} = 0.667 \text{ ft}
\]

\[
\text{local weight of facing: } W_{\text{t}} = H_{\text{top}} \cdot t \cdot (c \cdot \gamma_c + v \cdot \gamma_u) \quad W_{\text{t}} = 85.104 \cdot \text{ plf}
\]

SOIL AND SURCHARGE FORCES:

active force:

\[
F_{a_{\text{t},s}} = \frac{1}{2} \cdot K_{a_{\text{t}}} \cdot \gamma_{\text{i}} \cdot H_{\text{top}}^2 
\quad F_{a_{\text{t},s}} = 7.623 \cdot \text{ plf}
\]

\[
F_{a_{\text{v},s}} = F_{a_{\text{t},s}} \cdot \sin(\phi_{\text{wi}}) 
\quad F_{a_{\text{v},s}} = 2.607 \cdot \text{ plf}
\]

\[
F_{a_{\text{h},s}} = F_{a_{\text{t},s}} \cdot \cos(\phi_{\text{wi}}) 
\quad F_{a_{\text{h},s}} = 7.163 \cdot \text{ plf}
\]

dynamic force:

\[
F_{a_{\text{t},d}} = \frac{1}{2} \cdot (1 + K_v) \cdot K_{a_{\text{t}}} \cdot \gamma_{\text{i}} \cdot H_{\text{top}}^2 
\quad F_{a_{\text{t},d}} = 0 \cdot \text{ plf}
\]

\[
D_{F_{\text{dyn},s}} = F_{a_{\text{t},d}} - F_{a_{\text{t},s}} 
\quad D_{F_{\text{dyn},s}} = -7.623 \cdot \text{ plf}
\]

\[
D_{F_{\text{dyn},v}} = D_{F_{\text{dyn},s}} \cdot \sin(\phi_{\text{wi}}) 
\quad D_{F_{\text{dyn},v}} = 0 \cdot \text{ plf}
\]

\[
D_{F_{\text{dyn},h}} = D_{F_{\text{dyn},s}} \cdot \cos(\phi_{\text{wi}}) 
\quad D_{F_{\text{dyn},h}} = 0 \cdot \text{ plf}
\]

seismic inertial force: \( P_{\text{i},s} = K_{\text{hi}} \cdot (W_{\text{t}}) \)

\[
P_{\text{i},s} = 0 \cdot \text{ plf}
\]

Determine the maximum point back where the any surcharge will not effect the wall:

\[
ss_{\text{top}} = \frac{H_{\text{top}}}{\tan\left(45^\circ + \phi/2\right)} 
\quad ss_{\text{top}} = 0.385 \text{ ft}
\]

surcharge force:

\[
F_{q_{\text{top}}} := \begin{cases} q_x \cdot (H \cdot \tan(\omega) + t) & \text{if } (q_x \cdot (H \cdot \tan(\omega) + t) < ss_{\text{top}}, q \cdot K_{a_{\text{t}}} \cdot H_{\text{top}}, 0 \text{ lb/ft}) \quad \text{F}_{q_{\text{top}}} = 0 \cdot \text{ plf} \\
F_{q_{\text{h},s}} = F_{q_{\text{top}}} \cdot \cos(\phi_{\text{wi}}) 
\quad F_{q_{\text{h},s}} = 0 \cdot \text{ plf}
\end{cases}
\]

\[
F_{q_{v,y}} := \begin{cases} 
q_x = 2, F_{q_{\text{top}}} \cdot \sin(\phi_{\text{wi}}), 0 \text{ plf} \\
F_{q_{v,y}} = 0 \cdot \text{ plf}
\end{cases}
\]

point load surcharge:

\[
F_{Q_{\text{pt},s}} := \begin{cases} [x_1 \cdot (H \cdot \tan(\omega) + t)] < ss_{\text{top}}, Q_{\text{pt}} \cdot K_{a_{\text{t}}} \cdot H_{\text{top}}, 0 \text{ lb/ft} \\
F_{Q_{\text{pt},s}} = 0 \cdot \text{ plf}
\end{cases}
\]

\[
F_{Q_{\text{pth},s}} = F_{Q_{\text{pt},s}} \cdot \cos(\phi_{\text{wi}}) 
\quad F_{Q_{\text{pth},s}} = 0 \cdot \text{ plf}
\]

\[
F_{Q_{\text{ptv},s}} := \begin{cases} 
Q_{\text{type}} = 2, F_{Q_{\text{pt},s}} \cdot \sin(\phi_{\text{wi}}), 0 \text{ plf} \\
F_{Q_{\text{ptv},s}} = 0 \cdot \text{ plf}
\end{cases}
\]
LOCAL SLIDING RESISTANCE:
Total weight acting to resist sliding of the top of wall:
\[ W_{\text{total static}} := W_t \cdot \text{top} + F_{\text{av, top}} + F_{\text{qv, top}} + F_{\text{qptv, top}} \quad W_{\text{total static}} = 87.711 \cdot \text{plf} \]
\[ W_{\text{total seismic}} := W_t \cdot \text{top} + F_{\text{av, top}} + F_{\text{qv, top}} + F_{\text{qptv, top}} \quad W_{\text{total seismic}} = 87.711 \cdot \text{plf} \]
local sliding resistance:
\[ F_{t, \text{static}} := \text{if}(\omega < 6 \text{deg}, au + W_{\text{total static}} \cdot \tan(\lambda u3), au + W_{\text{total static}} \cdot \tan(\lambda u3)) \quad F_{t, \text{static}} = 1176.236 \cdot \text{plf} \]
\[ F_{t, \text{seismic}} := \text{if}(\omega < 6 \text{deg}, au + W_{\text{total seismic}} \cdot \tan(\lambda u3), au + W_{\text{total seismic}} \cdot \tan(\lambda u3)) \quad F_{t, \text{seismic}} = 1176.236 \cdot \text{plf} \]
FACTOR OF SAFETY LOCAL SLIDING, Static:
\[ F_{\text{s sliding, s top}} := \frac{F_{t, \text{static}}}{(F_{a, \text{top}} + F_{q, \text{top}} + F_{\text{qpt, top}}) \cdot \cos(\phi w)} \quad F_{\text{s sliding, s top}} = 164.199 \]
FACTOR OF SAFETY LOCAL SLIDING, Seismic:
\[ F_{\text{s sliding, d top}} := \frac{F_{t, \text{seismic}}}{(F_{a, \text{top}} + D_{\text{dyn, top}} + F_{q, \text{top}} + F_{\text{qpt, top}} + P_{\text{ir, top}}) \cdot \cos(\phi w)} \quad F_{\text{s sliding, d top}} = 164.199 \]
FACTOR OF SAFETY LOCAL OVERTURNING, Static:
\[ \text{num1} := W_t \cdot \text{top} \cdot \left[ \frac{H_t \cdot \tan(\omega)}{2} + \frac{t}{2} \right] + F_{\text{av, top}} \cdot \left( \frac{H_t \cdot \tan(\omega)}{3} + t \right) + (F_{\text{qv, top}} + F_{\text{qptv, top}}) \cdot \left( \frac{H_t \cdot \tan(\omega)}{2} + t \right) \]
\[ F_{\text{s overturning, s top}} := \frac{\text{num1}}{F_{a, \text{top}} \cdot \left( \frac{H_t}{3} \right) + F_{q, \text{top}} \cdot \left( \frac{H_t}{2} \right) + F_{\text{qpt, top}} \cdot \left( \frac{H_t}{2} \right)} \quad F_{\text{s overturning, s top}} = 30.119 \]
FACTOR OF SAFETY LOCAL OVERTURNING, Seismic:
\[ \text{num2} := \text{num1} + D_{\text{dyn, top}} \cdot (0.6 \cdot H_t + t) \quad \text{Den1} := F_{\text{qpt, top}} \cdot \left( \frac{H_t}{2} \right) + P_{\text{ir, top}} \cdot \left( \frac{H_t}{2} \right) \]
\[ F_{\text{s overturning, d top}} := \frac{\text{num2}}{F_{a, \text{top}} \cdot \left( \frac{H_t}{3} \right) + D_{\text{dyn, top}} \cdot (0.6 \cdot H_t + t) + F_{q, \text{top}} \cdot \left( \frac{H_t}{2} \right) + \text{Den1}} \quad F_{\text{s overturning, d top}} = 30.119 \]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
COMPOUND STABILITY CALCULATIONS

ENTRANCE POINTS
(xe, ye)

EXIT POINTS
or COURSES COORD.
(xo, yo)

2 * H
or He + L

He
H

(xo_N, yo_N)

(x_c, y_c)

RADIUS

(x1, y1)

(x2H, y2H)

Soil Wedges

Slip Arc

(xw, ywt)

(xw, ywb)
### COURSING COORDINATES

Range of courses: courses := 0 .. n

\[ x_0 \text{courses} := t \cdot \text{courses} \cdot h \cdot \tan(\omega) - h \cdot \tan(\omega) \]

<table>
<thead>
<tr>
<th>Block Course</th>
<th>Course coord.</th>
<th>Course Coord</th>
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<tr>
<td>Course := 0</td>
<td>x0 = 0.915</td>
<td>y0 = 0.000</td>
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<tr>
<td>1</td>
<td>0.990</td>
<td>0.667</td>
</tr>
<tr>
<td>2</td>
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<td>2.000</td>
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</tr>
<tr>
<td>8</td>
<td>1.515</td>
<td>6.000</td>
</tr>
<tr>
<td>9</td>
<td>1.590</td>
<td></td>
</tr>
</tbody>
</table>

### Working Point at Top of Facing (PT):

\[ y_0 \text{courses} := \text{courses} \cdot h \]

\[ x_0 = 1.59 \text{ ft} \]

\[ y_0 = 6 \text{ ft} \]

\[ x_g := x_0 + (L - t + s) \]

\[ y_g := y_0 + (L - t + s) \cdot \tan(i) \]

\[ y_\text{g} := \text{if}(y_g > H + hi, H + hi, y_g) \]

\[ x_g = 4.783 \text{ ft} \]

### Working Point at Back of Reinforced Mass (PTG):

\[ x_2H_1 := 2 \cdot H \]

\[ x2H1 = 12 \text{ ft} \]

\[ y_2H_1 := y_0 + (x_2H1 - x_0) \cdot \tan(i) \]

\[ y2H1 := \text{if}(y_2H1 > H + hi, H + hi, y_2H1) \]

\[ y2H1 = 8 \text{ ft} \]

### He + grid length - block embedment:

\[ x_{H \text{g}} := L + He \]

\[ y_{H \text{g}} := y_0 + (x_{H \text{g}} - x_0) \cdot \tan(i) \]

\[ y_{H \text{g}} := \text{if}(y_{H \text{g}} > H + hi, H + hi, y_{H \text{g}}) \]

\[ x_{2H} := (x_2H1 < x_{H \text{g}}, x_{H \text{g}}, x_{2H1}) \]

\[ x_{2H} := \text{if}(x_{2H1} < x_{H \text{g}}, y_{H \text{g}}, y_{2H1}) \]

### Coordinates of Entrance Points (Equal to # of Courses):

Determine twenty equal divisions between back of reinforced mass and the horizontal limit:

\[ \text{division} := \frac{(x_{2H} - x_g)}{n} \]

\[ \text{division} = 0.802 \text{ ft} \]

\[ x_e \text{courses} := x_{2H} - \text{division} \cdot \text{courses} \]

\[ y_e \text{courses} := y_0 + (x_e \text{courses} - x_0) \cdot \tan(i) \]

\[ y_e \text{courses} := \text{if}(y_e \text{courses} > H + hi, H + hi, y_e \text{courses}) \]

### Input Values from AB Walls:

\[ \text{course} = 0 \quad \text{FSI} = 1.69 \]

\[ X_c = 0.78 \text{ ft} \quad Y_c = 11.87 \text{ ft} \quad \text{Radius} = 11.87 \text{ ft} \]

\[ x_{\text{course}} = 0.915 \text{ ft} \quad y_{\text{course}} = 0 \text{ ft} \]

\[ X_1 = 12 \text{ ft} \quad Y_1 = 8 \text{ ft} \]

### Chord Geometry:

\[ \text{chord} := \left( (X_1 - x_{\text{course}})^2 + (Y_1 - y_{\text{course}})^2 \right)^{0.5} = 13.671 \text{ ft} \]

\[ \text{chordslope} := \frac{Y_1 - y_{\text{course}}}{X_1 - x_{\text{course}}} \quad \text{anglechord} := \text{atan} (\text{chordslope}) \]

\[ \text{chordslope} = 0.722 \quad \text{anglechord} = 35.817 \text{ deg} \]

---

P#: 30

Preliminary design calculations unless reviewed and certified by a local professional engineer.
Wedge Thicknesses Relative to Slip Arc:

\[ w := 20 \]

NoWedges := \[ 0 \ldots w \]

\[ \text{wedge\_thick} := \frac{(X_{1} - x_{0\_course})}{w} \]

This is the thickness of each wedge Relative to the selected Slip Arc length:

\[ \text{wedge\_thick} = 0.554 \text{ ft} \]

\[ \text{Elev}_{\text{course}} = 0 \text{ ft} \]

\[ \text{wedge\_thick} = 0.554 \text{ ft} \]

\[ x_{w\_\text{NoWedges}} := (\text{wedge\_thick} \cdot \text{NoWedges}) + x_{0\_\text{course}} \]

\[ \text{Radius} = 11.87 \text{ ft} \]

\[ y_{w\_\text{NoWedges}} := y_{c} - \left[ \frac{\text{Radius}^{2} - (x_{w\_\text{NoWedges}} - x_{c})^{2}}{0.5} \right] \]

\[ y_{w\_\text{NoWedges}} := y_{0} + \left( x_{w\_\text{NoWedges}} - x_{0} \right) \cdot \tan(i) \]

\[ y_{w\_\text{NoWedges}} := \text{if}(y_{w\_\text{NoWedges}} > H + h_i, H + h_i, y_{w\_\text{NoWedges}}) \]

Coordinates of intersection points of Arcs and Vertical Wedges:

\[
\begin{array}{c|c|c|c}
\text{y}_{\text{wNoWedges}} & \text{y}_{\text{w1NoWedges}} & \text{x}_{\text{wNoWedges}} \\
0.001 & 5.775 & 0.915 \\
0.02 & 5.96 & 1.469 \\
0.065 & 6.144 & 2.023 \\
0.137 & 6.329 & 2.577 \\
0.235 & 6.513 & 3.132 \\
0.361 & 6.697 & 3.686 \\
0.516 & 6.882 & 4.24 \\
0.699 & 7.066 & 4.794 \\
0.914 & 7.25 & 5.349 \\
1.162 & 7.435 & 5.903 \\
1.446 & 7.619 & 6.457 \\
1.767 & 7.804 & 7.012 \\
2.131 & 7.988 & 7.566 \\
2.542 & 8 & 8.12 \\
3.006 & 8 & 8.674 \\
... & ... & 9.229 \\
\end{array}
\]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
Area of each of the 10 Wedges Relative to the chosen Arc Number:

- \( \text{Area}_\text{Wedge}_0 = 1.36 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_1 = 3.263 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_2 = 3.401 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_3 = 3.456 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_4 = 3.496 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_5 = 3.52 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_6 = 3.529 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_7 = 3.52 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_8 = 3.494 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_9 = 3.449 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{10} = 3.384 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{11} = 3.296 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{12} = 3.136 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{13} = 2.897 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{14} = 2.622 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{15} = 2.31 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{16} = 1.95 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{17} = 1.529 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{18} = 1.022 \text{ ft}^2 \)
- \( \text{Area}_\text{Wedge}_{19} = 0.372 \text{ ft}^2 \)

Wedge Properties:

\( \alpha = \text{Angle from Horizontal to bottom of each wedge.} \)
\( \theta = \text{Angle from Horizontal to relative Geogrid placement.} \)
\( \psi = \text{Internal friction angle of either infill or retained soils.} \)
\( \gamma_i = \text{Unit weight of infill soil will be used for all Wedge weights.} \)

Where: \( m_\alpha = \cos(\alpha) + [\sin(\alpha) \times \tan(\psi)] / \text{FS} \)

**POINT LOAD SURCHARGE PARAMETERS**

\( P = 0 \cdot \text{psf} \)

\( \text{Weight of point load surcharge per wedge:} \)
\( x_1 = 10 \text{ ft} \)
\( x_2 = 8.144 \text{ ft} \)
\( W_{\text{pt}} = Qpi \times \text{wedge thick} \)
\( W_{\text{pt}3} = 0 \cdot \text{plf} \)

**SQUARE FOOT SURCHARGE PARAMETERS**

\( q = 100 \cdot \text{psf} \)

\( \text{Weight of square foot surcharge per wedge:} \)
\( q_x = 7.5 \text{ ft} \)
\( W_{\text{Sf}} = q \times \text{wedge thick} \)
\( W_{\text{Sf}3} = 0 \cdot \text{plf} \)

Total weight of surcharges:
\( W_{\text{Sur}} := W_{\text{Sf}} + W_{\text{pt}} \)

**SOIL WEDGE PARAMETERS**

<table>
<thead>
<tr>
<th>\text{Area}_\text{Wedge}1</th>
<th>\text{Area}_\text{Wedge}2</th>
<th>\text{Area}_\text{Wedge}3</th>
<th>\gamma_{\text{1bb}}</th>
<th>\gamma_{\text{2bb}}</th>
<th>\gamma_{\text{3bb}}</th>
<th>W_{\text{Wedge}}</th>
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<td>0.81 ft^2</td>
<td>0.451 ft^2</td>
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## SOIL WEDGE PARAMETERS

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<th>( \alpha_{\text{bb}} )</th>
<th>( \theta )</th>
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<td>6</td>
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<td>18.358</td>
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<td>1.024</td>
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<td>1.007</td>
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<tr>
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<tr>
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<tr>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
</tbody>
</table>

\[
\sum \text{Wt\_Wedge} = 6600.652 \cdot \text{plf} \quad \sum \text{Wt\_Sur} = 450 \cdot \text{plf}
\]

### Sliding Resistance Due to Soil Weight, Surcharges and Soil Frictional Interaction:

\[
F_{r_{\text{bb}}} = \left( \frac{\text{Wt\_Wedge}_{\text{bb}-1} + \text{Wt\_Sur}_{\text{bb}-1}}{m_{\alpha_{1\text{bb}}}} \right) \cdot \tan(\phi_{\text{bb}})
\]

\[
F_{r_{\text{seismic,bb}}} = \left( \frac{\text{Wt\_Wedge}_{\text{bb}-1} + \text{Wt\_Sur}_{\text{bb}-1}}{m_{\alpha_{1\text{seismic}}}} \right) \cdot \tan(\phi_{\text{bb}})
\]

\[
\begin{align*}
F_{r_{\text{bb}}} &= 93.16 \cdot \text{plf} \\
 220.70 \\
 227.52 \\
 229.26 \\
 230.48 \\
 231.18 \\
 231.36 \\
 230.99 \\
 230.05 \\
 228.48 \\
 226.20 \\
 226.83 \\
 247.56 \\
 236.00 \\
 222.51 \\
 206.74 \\
 \ldots
\end{align*}
\]

\[
\sum F_r = 4084.77 \cdot \text{plf}
\]

\[
\begin{align*}
F_{r_{\text{seismic,bb}}} &= 93.16 \cdot \text{plf} \\
 220.70 \\
 227.52 \\
 229.26 \\
 230.48 \\
 231.18 \\
 231.36 \\
 230.99 \\
 230.05 \\
 228.48 \\
 226.20 \\
 226.83 \\
 247.56 \\
 236.00 \\
 222.51 \\
 \ldots
\end{align*}
\]

\[
\sum F_r_{\text{seismic}} = 4084.77 \cdot \text{plf}
\]

---

Preliminary design calculations unless reviewed and certified by a local professional engineer.
Lateral Sliding Force:

$$F_{s_{bbb}} = (Wt\_Wedge_{bbb-1} + Wt\_Sur_{bbb-1}) \cdot \sin(\alpha_{bbb})$$

$$\sum F_{s} = 3126.939 \cdot \text{plf}$$

SEISMIC PARAMETERS

$$\text{Dyn}_{-}\text{CS} := \sum F_{s} \cdot K_{hr}$$

$$\text{Dyn}_{-}\text{CS} = 0 \cdot \text{plf}$$

Sum of Lateral Sliding Forces:

$$\sum F_{s} + \text{Dyn}_{-}\text{CS} = 3126.94 \cdot \text{plf}$$

GEOGRID INTERACTION

$$x_{grid1_{k}} := \text{if} \begin{cases} \text{Elev}_{-}\text{Grid}_{k} \leq \text{Elev}_{-}\text{course}, & 0\text{ft}, Xc + \left(\frac{\text{Radius}}{2} - \left(\text{Elev}_{k} - Y_{0}\right)\right)^{0.5} \\ \end{cases}$$

Note: geo course #0 represents the top of leveling pad.

$$y_{grid1_{k}} := \text{if} \begin{cases} \text{geo}_{k} > 0, & \text{if} \left(x_{grid1_{k}} \leq 0\text{ft}, 0\text{ft}, \text{Elev}_{k}\right), 0\text{ft} \\ \end{cases}$$

$$x_{grid2_{k}} := \text{if} \begin{cases} \text{geo}_{k} > 0, & \text{if} \left(x_{grid1_{k}} \leq 0\text{ft}, 0\text{ft}, x_{grid1_{k}}\right), 0\text{ft} \\ \end{cases}$$

Preliminary design calculations unless reviewed and certified by a local professional engineer.
Horizontal resistance Forces due to Geogrid layers at intersection with Slip Arc:

Note: The designer should determine the least amount of resisting force provided by each grid layer by calculating the resistance from both sides of the Slip Arc. The resisting force from the retained side is the embedment length (Le) combined with the confining pressure of the soil above. Similarly, the sliding wedge side is figured by combining the connection strength of that layer with the confining soil pressure above the effected grid length.

Retained side of Slip Arc Calculation:
ygrid = Elevation of Geogrid Layer at intersection with Slip Arc
Le_grid_b = Length of Geogrid beyond intersection with Slip Arc (the "_b" indicates "beyond" the Slip Arc)
Ngrid_b = The weight or confining pressure from soil above Le_grid_b

Le_grid1 := \text{Glength} - (\text{xgrid2} - \text{ygrid1} \cdot \tan(\omega))
Le_grid_b := \text{if}(\text{xgrid2} = 0 \cdot \text{ft}, 0 \cdot \text{ft}, \text{if}(\text{Le_grid1} \leq 0 \cdot \text{ft}, \text{Le_grid1}))
ygrid1 := \text{if}(\text{Le_grid_b} \leq 0\text{ft}, 0\text{ft}, \text{ygrid1})
xgrid1 := \text{if}(\text{ygrid1} = 0\text{ft}, 0\text{ft}, \text{xgrid2})

Normal load above grid:
Grid between L_2 and top of wall

\[ \text{kkk}_k := \left[ \left[ \text{ygrid} + \left( \text{xgrid1}_k - \text{x_0} \right) \cdot \tan(\text{i_int}) \right] - \text{ygrid1} \right] + \left[ \left( \text{xgrid1}_k + \text{Le_grid_b}_k \right) \cdot \tan(\text{i_int}) \right] \cdot 2 \cdot \eta_{3} \cdot \text{Le_grid_b}_k \]

Grid between L_1 and L_2

\[ \text{bbbb}_k := \left[ \left[ \text{ygrid} + \left( \text{xgrid1}_k - \text{x_0} \right) \cdot \tan(\text{i_int}) \right] - \text{L_2} \right] + \left[ \left( \text{xgrid1}_k + \text{Le_grid_b}_k \right) \cdot \tan(\text{i_int}) \right] \cdot \eta_{3} \]

\[ \text{III}_k := \text{if}(\text{ygrid1} > \text{L_1} \land \text{ygrid1} < \text{L_2}, \left[ \left( \text{L_2} - \text{ygrid1} \right) \cdot \eta_{3} + \text{bbbb}_k \right] \cdot \text{Le_grid_b}_k \), \text{kkk}_k \]

Grid between L_1 and bottom of wall

\[ \text{Ngrid_b}_k := \text{if}(\text{ygrid1} = 0 \cdot \text{ft}, 0 \cdot \text{plf}, \text{if}(\text{ygrid1} < \text{L_1}, \left[ \left( \text{L_1} - \text{ygrid1} \right) \cdot \eta_{3} + (\text{L_2} - \text{L_1}) \cdot \eta_{3} + \text{bbbb}_k \right] \cdot \text{Le_grid_b}_k \), \text{III}_k \]

\[ \text{Tgrid1}_k := \text{if}(\text{ygrid1} \leq 0\text{ft}, 0\text{plf}, \text{appliance} \cdot 2 \cdot \text{Le_grid_b}_k \cdot \text{Cl} \cdot \frac{\text{Ngrid_b}_k \cdot \tan(\phi_i)}{1 \text{ft} \cdot 1.5} \) \]

where: \( \phi_i = 30 \cdot \text{deg} \)

P# : 35
Geogrid Layer strength is limited to it's LTDS:

\[ T_{grid\_b\_k} := \text{if} \left( T_{grid1\_k} \leq 0 \text{plf}, 0 \text{plf}, \text{if} \left( T_{grid1\_k} > \text{LTDS}_{geo\_k}, \text{LTDS}_{geo\_k}, T_{grid1\_k} \right) \right) \]

### Allowable geogrid strength:

<table>
<thead>
<tr>
<th>courses</th>
<th>xgrid =</th>
<th>ygrid =</th>
<th>Le_grid_b =</th>
<th>Ngrid_b =</th>
<th>Tgrid_b =</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
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<td>0</td>
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<td>0</td>
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<tr>
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<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
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</tr>
<tr>
<td>5</td>
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<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
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<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

### Sum of Allowable grid strengths based on embedment depth beyond the Slip Arc:

\[ F_{g\_b} := T_{grid\_b} \cdot \cos(\alpha_{grid\_w}) \]

\[ F_{g\_b} = 0 \cdot \text{plf} \]

### Failure Wedge side of Slip Arc Calculation:

#### Soil resistance portion:

- \( ygrid = \) Elevation of Geogrid Layer at intersection with Slip Arc
- \( L_{grid\_f} = \) Length of Geogrid beyond intersection with Slip Arc (the "f" indicates "in front" of the Slip Arc)
- \( N_{grid\_f} = \) The weight or confining pressure from soil above \( L_{grid\_b} \)

\[ L_{grid\_2} := L - (t - s) - L_{grid\_1} \]

\[ L_{grid\_f\_k} := \text{if} \left( L_{grid2\_k} = 0 \text{ft}, 0 \text{ft}, \text{if} \left( L_{grid1\_k} \leq 0.01 \text{ft}, 0 \text{ft}, L_{grid2\_k} \right) \right) \]

### Normal load above grid:

- **Grid between L_2 and top of wall**

\[ g_{gg\_k} := \left[ \left[ y_{o\_n} + (x_{grid2\_k} - x_{o\_n}) \cdot \tan(l_{int}) \right] - y_{grid\_k} \right] - \frac{\left[ (x_{grid2\_k} - x_{o\_n}) \cdot \tan(l_{int}) \right]}{2} \cdot \gamma_{l\_3} \cdot L_{grid\_f\_k} \]

- **Grid between L_1 and L_2**

\[ b_{b\_k\_k} := \left[ \left[ y_{o\_n} + (x_{grid2\_k} - x_{o\_n}) \cdot \tan(l_{int}) \right] - L_{2} \right] - \frac{\left[ (x_{grid2\_k} - x_{o\_n}) \cdot \tan(l_{int}) \right]}{2} \cdot \gamma_{l\_3} \]

\[ f_{f\_k} := \text{if} \left[ y_{grid\_k} > L_{1} \land y_{grid\_k} < L_{2}, \left[ \left[ L_{2} - y_{grid\_k} \right] \cdot \gamma_{l\_2} + b_{b\_k\_k} \right] \cdot L_{grid\_f\_k}, g_{gg\_k} \right] \]

- **Grid between L_1 and bottom of wall**

\[ N_{grid\_f\_k} := \text{if} \left[ y_{grid\_k} = 0 \text{ ft}, 0 \text{ plf}, \text{if} \left[ y_{grid\_k} < L_{1}, \left[ \left( L_{1} - y_{grid\_k} \right) \cdot \gamma_{l\_1} + \left( L_{2} - L_{1} \right) \cdot \gamma_{l\_2} + b_{b\_k\_k} \right] \cdot L_{grid\_f\_k}, f_{f\_k} \right] \]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
\[ T_{\text{grid}_k} = \left\{ \begin{array}{l}
y_{\text{grid}_k} \leq 0 \text{ft}, 0 \cdot \text{plf}, \alpha_{\text{pullout}} \cdot 2 \cdot L_{\text{grid}_f} \cdot C_l \cdot \frac{N_{\text{grid}_f}}{1 \text{ft}} \cdot \frac{\tan(\phi_i)}{1.5} \\
\end{array} \right. \]

where: \( \phi_i = 30 \cdot \text{deg} \)

**Geogrid Layer strength is limited to it's LTDS:**

\[ T_{\text{grid}_f} := \text{if} \left( T_{\text{grid}_2k} \leq 0 \text{plf}, 0 \text{plf}, \text{if} \left( T_{\text{grid}_2k} \geq L_{\text{LTDS}_{\text{geo}}_k}, L_{\text{LTDS}_{\text{geo}}_k}, T_{\text{grid}_2k} \right) \right) \]

**Connection Capacity Portion:**

\[ F_{\text{con}_f} := \text{if} \left[ L_{\text{grid}_f} \geq 0 \text{ft}, F_{\text{CS}_{\text{geo}}_k} \cdot (\text{TRF} \cdot \text{ARF}), 0 \text{plf} \right] \]

**Note:** TRF and ARF are connection reductions for pattern walls and tumbled product.

- **Connection capacity:**
  - Courses: \(0, 1, 2, 3, 4, 5, 6, 7, 8, 9\)
  - Xgrid: \(0 \text{ ft}\)
  - Ygrid: \(0 \text{ ft}\)
  - Lgrid: \(0 \text{ ft}\)
  - Ngrid: \(0 \cdot \text{plf}\)
  - Tgrid: \(0 \cdot \text{plf}\)
  - Fcon: \(0 \cdot \text{plf}\)

**Allowable geogrid strength:**

\[ F_{g_f} := T_{\text{grid}_f} \cdot \cos(\alpha_{\text{grid}_w}) \]

\[ F_{g_f} = 0 \cdot \text{plf} \]

\[ \sum F_{\text{con}_f} = 0 \cdot \text{plf} \]

**Sum of Allowable resistance on Wedge side:**

\[ F_{g_f} + \sum F_{\text{con}_f} = 0 \cdot \text{plf} \]

\[ F_{g} := \text{if} \left( F_{g_b} \leq F_{g_f} + \sum F_{\text{con}_f}, F_{g_b}, \sum F_{g_f} + \sum F_{\text{con}_f} \right) \]

**Allowable Resisting force from Geogrid:**

\[ F_{g} = 0 \cdot \text{plf} \]
GEOGRID LAYERS ABOVE THE WALL

Are there Geogrid layers above the wall?

<table>
<thead>
<tr>
<th>Grid_Above</th>
<th>1 for Yes</th>
<th>2 for No</th>
</tr>
</thead>
</table>

How far above the top block to the first layer of grid:

<table>
<thead>
<tr>
<th>Sabove</th>
<th>1 ft</th>
</tr>
</thead>
</table>

How many layers above wall are required:

<table>
<thead>
<tr>
<th>Gabove</th>
<th>3</th>
</tr>
</thead>
</table>

Spacing between layers:

<table>
<thead>
<tr>
<th>Spacing</th>
<th>1.5 ft</th>
</tr>
</thead>
</table>

Length of Grid and Type:

<table>
<thead>
<tr>
<th>LgaGa</th>
<th>6.5 ft</th>
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</thead>
<tbody>
<tr>
<td>type_GaGa</td>
<td>&quot;Strata 200&quot;</td>
</tr>
</tbody>
</table>

Starting and ending grid coordinates:

\[
\text{Elev}_\text{GA}_{ga} = \begin{cases} \text{Grid}_\text{Above} = 1, \text{if} \left( \text{i}_\text{int} \leq 0 \text{deg}, 0 \text{ft}, \text{yo}_{ga} + \text{Sabove} + \text{ga} \cdot \text{Spacing} \right), 0 \text{ft} \end{cases}
\]

\[
Xga1_{ga} := \begin{cases} \text{Grid}_\text{Above} = 1, \text{if} \left( \text{i}_\text{int} \leq 0 \text{deg}, 0 \text{ft}, \text{xo}_{ga} + \frac{\text{Elev}_\text{GA}_{ga} - \text{yo}_{ga}}{\tan(\text{i}_\text{int})} \right), 0 \text{ft} \end{cases}
\]

\[
Xga2_{ga} := \begin{cases} \text{Grid}_\text{Above} = 1, \text{if} \left( \text{i}_\text{int} \leq 0 \text{deg}, 0 \text{ft}, Xga1_{ga} + \text{Lga}_{ga} \right), 0 \text{ft} \end{cases}
\]

Geogrid intersection point with Slip-Arc:

\[
xgrid_{ga} := \begin{cases} \text{Grid}_\text{Above} = 1, \text{XC} + \left[ \left( \text{Radius} \right)^2 - \left( \text{Elev}_\text{GA}_{ga} - \text{Yc} \right)^2 \right]^{0.5}, 0 \text{ft} \end{cases}
\]

<table>
<thead>
<tr>
<th>Elev_GAga</th>
<th>Xga1ga =</th>
<th>Xga2ga =</th>
<th>xgrid_gaga =</th>
<th>\gamma_above_gaga =</th>
<th>\phi_above_gaga =</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 ft</td>
<td>0 ft</td>
<td>0 ft</td>
<td>0 ft</td>
<td>120 \cdot \text{pcf}</td>
<td>30 \cdot \text{deg}</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>120</td>
<td>30</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>120</td>
<td>30</td>
</tr>
</tbody>
</table>

Grid Length in front of Slip-Arc:

\[
\text{Le}_{GA_{fga}} := \begin{cases} \text{if} (Xga1_{ga} \geq xgrid_{ga} - \text{gaga}, 0 \text{ft}, \text{if} (Xga2_{ga} \leq xgrid_{ga} - \text{Xga1}_{ga})) \end{cases}
\]

Grid Length behind Slip-Arc:

\[
\text{Le}_{GA_{bgm}} := \begin{cases} \text{if} (Xga1_{ga} \geq xgrid_{ga} - \text{gaga}, 0 \text{ft}, \text{if} (Xga2_{ga} \leq xgrid_{ga} - \text{Xga2}_{ga} - \text{Xga1}_{ga})) \end{cases}
\]

Normal load above grid:

\[
\text{N}_{GA_{fga}} := \begin{cases} \frac{\gamma \cdot \text{above}_{ga} \cdot \left( xgrid_{ga} - Xga1_{ga} \right) \cdot \text{Le}_{GA_{fga}} \cdot \tan(\text{i}_\text{int})}{2} \end{cases}
\]

\[
\text{Tgrid2}_{GA_{fga}} := \begin{cases} \text{if} \left( \text{Le}_{GA_{fga}} \leq 0 \text{ft}, 0 \cdot \text{plf}, \text{opullout} \cdot 2 \cdot \text{Le}_{GA_{fga}} \cdot \text{Ci} \cdot \frac{\text{N}_{GA_{fga}}}{1 \text{ft}} \cdot \tan(\phi \text{above}_{ga}) \right) \end{cases}
\]

Determine if the pullout of grid from soil is greater than the LTDS of the grid:

\[
\text{Tgrid}_{GA_{fga}} := \begin{cases} \text{if} \left( \text{Le}_{GA_{fga}} \leq 0 \text{ft}, 0 \cdot \text{plf}, \text{if} (Tgrid_{GA_{fga}} \leq \text{LTDS}_{Gabove}, \text{LTDS}_{Gabove}, \text{Tgrid}_{GA_{fga}}) \right) \end{cases}
\]

<table>
<thead>
<tr>
<th>Le_GA_fga = N_GA_fga =</th>
<th>Tgrid2_GA_fga =</th>
<th>Tgrid_GA_fga =</th>
<th>\alpha_grid_GA_fga =</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 ft</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
Allowable geogrid strength:

\[ F_{g, GA} := T_{grid, GA} \cdot f_{ga} \cdot \cos(\alpha_{grid, GA}) \]

Normal load above grid:

\[ N_{GA, bga} := \begin{cases} 0 \cdot \text{plf} & \text{if} \; \text{Le}_{GA, bga} \leq 0 \text{ft, 0 plf, } \gamma_{above, bga} \cdot \left( \frac{\text{Le}_{GA, fga} \cdot \tan(i_{int}) + \text{L}_{ga} \cdot \tan(i_{int})}{2} \right) \cdot \text{Le}_{GA, bga} \end{cases} \]

\[ T_{grid2, GA, bga} := \begin{cases} 0 \cdot \text{plf} & \text{if} \; \text{Le}_{GA, bga} \leq 0 \text{ft, 0 plf, } \alpha_{pullout} \cdot 2 \cdot \text{Le}_{GA, bga} \cdot C_i \cdot \frac{N_{GA, bga}}{1 \text{ft}} \cdot \tan(\phi_{above, bga}) \end{cases} \]

Determine if the pullout of grid from soil is greater than the LTDS of the grid:

\[ T_{grid, GA, bga} := \begin{cases} 0 \cdot \text{plf} & \text{if} \; \text{Le}_{GA, bga} \leq 0 \text{ft, 0 plf, } T_{grid2, GA, bga} \geq \text{LTDS}_{above, bga}, \text{LTDS}_{Gabove, bga}, T_{grid2, GA, bga} \end{cases} \]

\[
\begin{array}{cccc}
\text{Le}_{GA, bga} = N_{GA, bga} = & 0 \cdot \text{ft} & 0 \cdot \text{plf} & 0 \cdot \text{plf} \\
& 0 & 0 & 0 \\
& 0 & 0 & 0 \\
& 0 & 0 & 0 \\
\end{array}
\]

\[
\begin{array}{cccc}
T_{grid2, GA, bga} = T_{grid, GA, bga} \cdot \alpha_{grid, GA} = & 0 \cdot \text{plf} & 0 \cdot \text{plf} & 0 \cdot \text{plf} & 0 \cdot \text{deg} \\
& 0 & 0 & 0 & 0 \\
& 0 & 0 & 0 & 0 \\
& 0 & 0 & 0 & 0 \\
\end{array}
\]

Allowable geogrid strength:

\[ F_{g, GA} := T_{grid, GA} \cdot f_{ga} \cdot \cos(\alpha_{grid, GA}) \]

\[
\begin{array}{cccc}
F_{g, GA} = & 0 \cdot \text{plf} & \text{Allowable Resisting force from Geo\-grids placed above} & \sum F_{g, GA} = 0 \cdot \text{plf} \\
& 0 & \text{the wall:} & 0 \cdot \text{plf} \\
& 0 & & 0 \\
\end{array}
\]

\[
\begin{array}{cccc}
F_{g, GA} = & 0 \cdot \text{plf} & \text{if } F_{g, GA} < F_{g, GA}, F_{g, GA}, F_{g, GA} \\
& 0 & & 0 \\
& 0 & & 0 \\
\end{array}
\]

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WALL FACING CONTRIBUTION

The Wall facing is subject to lateral forces from the soil load and a vertical normal load from the block facing. If the Slip Arc passes through the facing at a grid layer the shear strength of the Block-Grid-Block shear tests will be considered. If the Slip Arc passes between grid layers, we will determine the applied force on the back of the wall facing from the soil pressure between the upper and lower grid layers relative to the Slip Arc position. The combination of the Normal Load and Connection Strength will help form the resisting loads.

\[
\text{Drg\_Frc} := \sum F_s - \left( \sum F_{r} + F_g + \sum F_{GA} \right)
\]

\[
\text{Drg\_Frc\_Seis} := \sum F_s - \left( \sum F_{r\text{-seismic}} + F_g + \sum F_{GA} \right)
\]

If this value is POSITIVE the driving force has exceeded the resisting force and the sliding wedge has been mobilized. Then this net driving force should be applied to the back of the wall facing in the Wall Facing Contribution Section.

BLOCK SHEAR TEST RESULTS

Results are based on independent test lab findings.

**NOTE**: Block - Grid - Block AND Block - Block Shear Results are the same for AB Classic and AB Stones and slightly lower for AB Three, due to top lip configuration. Test values are on page 3:

- User defined Shear Capacity: \[ \text{Shear\_Capacity} = 100\% \]

\[
N_{CS\text{courses}} := (H - \text{courses} \cdot h) \cdot (c \cdot \gamma_c + v \cdot \gamma_f) \cdot t
\]

Shear at Arc Number Zero:

\[
N_{CS0} = 765.937 \cdot \text{plf}
\]

\[
V_0 := N_{CS0} \cdot \tan(\phi\text{plf})
\]

\[
V_0 = 556.486 \cdot \text{plf}
\]

Determine if the calculated shear is greater than the allowed shear:

\[
V_u_{BGB\text{course}} := \text{if}(\omega < 6\, \text{deg}, \text{if}(V_u_{BGB\text{course}} > a_u',\text{max}, a_u'_{\text{max}}, V_u_{BGB\text{course}}), \text{if}(V_u_{BGB\text{course}} > a_u_{\text{max}}, a_u'_{\text{max}}, V_u_{BGB\text{course}}))
\]

\[
V_u_{BB\text{course}} := \text{if}(\omega < 6\, \text{deg}, a_u3 + N_{CS\text{course}} \cdot \tan(\lambda u3), a_u + N_{CS\text{course}} \cdot \tan(\lambda u))
\]

Preliminary design calculations unless reviewed and certified by a local professional engineer.
Determine if the calculated shear is greater than the allowed shear:

$$\gamma_{u, BB_{course}} := \text{if}(\omega < 6 \text{deg}, \text{if}(V_{u, BB_{course}} > au_{3, max}, au_{3, max}, V_{u, BB_{course}}), \text{if}(V_{u, BB_{course}} > au_{max}, au_{max}, V_{u, BB_{course}}))$$

$$V_{u, BB_{course}} := V_{u, BB_{course}} \cdot \text{Shear\_Capacity} \quad V_{u, BB_{course}} = 3269.416 \cdot \text{pfl}$$

Determine if the Slip Arc passes thought the facing at a grid layer:

$$\text{Grid\_Layer} := \text{if}(\text{Elev\_Grid\_course} = 0 \text{ft}, "NO", "YES") \quad \text{Grid\_Layer} = "NO"$$

$$V_{u} := \text{if}(\text{Grid\_Layer} = "YES", V_{u, BGB_{course}}, V_{u, BB_{course}}) \quad V_{u} = 3269.416 \cdot \text{pfl}$$

### Determine the applied force due to soil forces:

Elevation of Slip Arc above leveling pad Elev\_course = 0 ft

Distance Below grade:

$$\text{Distance Below}: \quad H_{above} := H - \text{Layer\_Above} \quad H_{above} = 4 \text{ ft}$$

$$h = 0.667 \text{ ft} \quad \text{grid\_crs\_num\_Above} := \frac{\text{Layer\_Above}}{h} \quad \text{grid\_crs\_num\_Above} = 3$$

Elevation of grid layer below Slip Arc:

$$\text{Distance Below grade}: \quad H_{below} := H - \text{Layer\_Below} \quad H_{below} = 6 \text{ ft}$$

$$\text{grid\_crs\_num\_Below} := \frac{\text{Layer\_Below}}{h} \quad \text{grid\_crs\_num\_Below} = 0$$

$$\text{ccc} := (H_{below} - H_{above})$$

Soil Load between grid layers or driving from above if applicable:

$$\text{Soil\_Load} := \text{if}(\text{Grid\_Layer} = "YES", 0 \cdot \text{pfl}, \text{if}(\text{Drg\_FrC} > 0 \cdot \text{pfl}, \text{Drg\_FrC} \cdot \gamma_{i} \cdot \text{Kai} \cdot \frac{H_{above} + H_{below}}{2}, \text{ccc})))$$

$$\text{Soil\_Load\_Seis} := \text{if}(\text{Grid\_Layer} = "YES", 0 \cdot \text{pfl}, \text{if}(\text{Drg\_FrC\_Seis} > 0 \cdot \text{pfl}, \text{Drg\_FrC\_Seis} \cdot \gamma_{i} \cdot \text{Kai} \cdot \frac{H_{above} + H_{below}}{2}, \text{ccc})))$$

$$\text{Soil\_Load} = 343.044 \cdot \text{pfl} \quad \text{Soil\_Load\_Seis} = 343.044 \cdot \text{pfl}$$

Geogrid / Block Connection Capacity at Grid layer above Slip-Arc:

$$N_{\text{grid\_crs\_num\_Above}} := (H - \text{grid\_crs\_num\_Above} \cdot h) \cdot (c \cdot \gamma_{c} + v \cdot \gamma_{uf}) \cdot t \quad N_{\text{grid\_crs\_num\_Above}} = 510.625 \cdot \text{pfl}$$

$$\text{na} := N_{\text{grid\_crs\_num\_Above}}$$

$$\text{Fcs}_{\text{grid\_crs\_num\_Above,}i} := \text{if}((\text{type} = A, \text{if(}na < \text{Ninta, B1a + M1a} \cdot na, B2a + M2a} \cdot na), \text{if(}na < \text{Nintb, B1b} + \text{M1b} \cdot na, B2b + \text{M2b} \cdot na))$$

$$F_{\text{con}} := \text{Fcs}_{\text{grid\_crs\_num\_Above,}1} \cdot \text{TRF} \cdot \text{ARF} \quad F_{\text{con}} = 1546.91 \cdot \text{pfl} \quad \text{course} = 0$$

Normal load required to prevent overturning:

$$N_{req} := \left(\frac{\text{Soil\_Load} \cdot (\text{Elev\_course} - \text{Layer\_Below}) - \left(\frac{F_{\text{con}}}{1.5}\right) \cdot (\text{Layer\_Above} - \text{Layer\_Below})}{t - \frac{1}{2}}\right)$$

$$N_{req} = -4168.516 \cdot \text{pfl}$$

$$N_{req\_seis} := \left(\frac{\text{Soil\_Load\_Seis} \cdot (\text{Elev\_course} - \text{Layer\_Below}) - \left(\frac{F_{\text{con}}}{1.5}\right) \cdot (\text{Layer\_Above} - \text{Layer\_Below})}{t - \frac{1}{2}}\right)$$

$$N_{req\_seis} = -4168.516 \cdot \text{pfl}$$

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aaa = "Actual normal load exceeds the required, therefore the Block Shear can be used"

Therefore:

\[ V_u := \begin{cases} \text{if} (N_{CS\_course} \geq N_{req}, Vu, 0 \text{plf}) \\ Vu = 3269.416 \cdot \text{plf} \end{cases} \]

\[ V_u\_seis := \begin{cases} \text{if} (N_{CS\_course} \geq N_{req\_seis}, Vu, 0 \text{plf}) \\ Vu\_seis = 3269.416 \cdot \text{plf} \end{cases} \]

Distribution of Connection Strength at facing:

Above Slip Arc:

\[ p := 0 .. \frac{32\text{in}}{h} \]

\[ G_{-1\_p} := \begin{cases} \text{if} (\text{Elev\_Grid}_{course-p} > 0 \cdot \text{ft}, \frac{(32 \cdot \text{in} - p \cdot h)}{32\text{in}} \cdot q_{qq\_course+p}, 0 \text{plf}) \end{cases} \]

\[ G_{-1} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 393.557 \\ 0 \end{pmatrix} \cdot \text{plf} \]

\[ \sum G_{-1} = 393.557 \cdot \text{plf} \]

Below Slip Arc:

\[ p_1 := 1 .. \frac{32\text{in}}{h} \]

\[ G_{-2\_p_1} := \begin{cases} \text{if} (\text{Elev}_{course} - p_1 \cdot h \leq 0 \cdot \text{ft}, 0, \text{plf}, \text{if} (\text{Elev\_Grid}_{course-p_1} > 0 \cdot \text{ft}, \frac{(32 \cdot \text{in} - p_1 \cdot h)}{32\text{in}} \cdot q_{qq\_course-p_1}, 0 \text{plf}) \end{cases} \]

\[ G_{-2\_p_1} = \begin{cases} 0 \\ 0 \\ 0 \end{cases} \cdot \text{plf} \]

\[ \sum G_{-2} = 0 \cdot \text{plf} \]

Frictional portion of base material (Vo):

\[ V_o = 556.486 \cdot \text{plf} \]

\[ B_{\_Vo} := \begin{cases} \text{if} (\text{Elev}_{course} \geq 32 \cdot \text{in}, 0, \frac{(32 \cdot \text{in} - \text{course} \cdot h)}{32\text{in}} \cdot V_o) \end{cases} \]

\[ B_{\_Vo} = 556.486 \cdot \text{plf} \]

Sum of Connection Contribution

\[ \text{Conn} := \sum G_{-1} + \sum G_{-2} + B_{\_Vo} \]

\[ \text{Conn} = 950.043 \cdot \text{plf} \]

Determine the lesser of block Shear OR Connection Contribution:

\[ \text{Facing} := \begin{cases} \text{if} (V_u > \text{Conn}, \text{Conn}, V_u) \end{cases} \]

\[ \text{Facing} = 950.043 \cdot \text{plf} \]

\[ \text{Facing\_seis} := \begin{cases} \text{if} (V_u\_seis > \text{Conn}, \text{Conn}, V_u) \end{cases} \]

\[ \text{Facing\_seis} = 950.043 \cdot \text{plf} \]
Safety Factor against Compound Failure for Arc Number: course = 0

Note: All resisting forces are summed in the numerator and the sliding forces are summed in the denominator. This ratio is the Safety Factor for Internal Compound Stability.

STATIC RESULTS:

$$\frac{\sum F_r + \text{Facing} + F_g + \sum F_{g,GA}}{\sum F_s}$$

$$\text{SF}_{\text{slip, Arc}} = 1.61$$

SIESSIC RESULTS:

$$\frac{\sum F_r + \text{Facing} + F_g + \sum F_{g,GA}}{\sum F_s + \text{Dyn}_\text{CS}}$$

$$\text{SF}_{\text{slip, Arc, seismic}} = 1.610$$

$$\sum F_r = 4084.771 \cdot \text{pf}$$
$$\sum F_r = 4084.771 \cdot \text{pf}$$

Facing = 950.043 \cdot \text{pf} \quad \sum F_s = 3126.939 \cdot \text{pf} \quad F_g = 0 \cdot \text{pf} \quad \sum F_{g,GA} = 0 \cdot \text{pf} \quad \text{Dyn}_\text{CS} = 0 \cdot \text{pf}

Compound Stability Summary:

Relative data for analyzed Slip Arc:

- course = 0
- Elev_{course} = 0 ft
- Iterated Safety Factor for instability:
  - SF_{slip, Arc} = 1.61

Entrance coordinates:

- X1 = 12 ft
- Y1 = 8 ft

Coordinates for center of Slip Arc Circle:

- Xc = 0.78 ft
- Yc = 11.87 ft

Radius of Slip Arc Circle:

- Radius = 11.87 ft

ICS Soil Parameter Summary:

**Infill Soils TOP (I 3)**

- $\phi_i = 30 \cdot \text{deg}$
- $\gamma_i = 120 \cdot \text{pcf}$

**Infill Soils MIDDLE (I 2)**

- $\phi_i = 30 \cdot \text{deg}$
- $\gamma_i = 120 \cdot \text{pcf}$

**Infill Soils BOTTOM (I 1)**

- $\phi_i = 30 \cdot \text{deg}$
- $\gamma_i = 120 \cdot \text{pcf}$

**Retained Soils TOP (R 1)**

- $\phi_r = 30 \cdot \text{deg}$
- $\gamma_r = 120 \cdot \text{pcf}$

**Retained Soils MIDDLE (R 1)**

- $\phi_r = 30 \cdot \text{deg}$
- $\gamma_r = 120 \cdot \text{pcf}$

**Retained Soils BOTTOM (R 1)**

- $\phi_r = 30 \cdot \text{deg}$
- $\gamma_r = 120 \cdot \text{pcf}$

Initial input Safety Factor from AB Walls 10:

- FSI = 1.69

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SUMMARY OF RESULTS

DESIGN PARAMETERS:

Wall Height: \( H = 6 \text{ ft} \)
Block Setback: \( \omega = 6.42 \cdot \text{deg} \)
Backslope Angle: \( i = 18.4 \cdot \text{deg} \)
Backslope Height: \( h_i = 2 \text{ ft} \)
Surcharge Load: \( q = 100 \cdot \text{psf} \)
Line Load Surcharge: \( P = 0 \cdot \text{plf} \)
Point Load Location: \( x_1 = 10 \text{ ft} \)
Seismic Coefficient: \( A_o = 0 \)
Allowable Deflection: \( \delta_i = 0.25 \text{ ft} \), \( \delta_r = 0.25 \text{ ft} \)

SOIL PARAMETERS:

Infill Soil: \( \phi_i = 30 \cdot \text{deg} \), \( \gamma_i = 120 \cdot \text{pcf} \)
Retained Soil: \( \phi_r = 30 \cdot \text{deg} \), \( \gamma_r = 120 \cdot \text{pcf} \)
Foundation Soil: \( \phi_f = 30 \cdot \text{deg} \), \( \gamma_f = 120 \cdot \text{pcf} \)

SurType = "Retained Soil Live Load"
SurTypePoint = "Live Load"

Controlling Dynamic Earth Pressure Theory:
DynamicTheory_1 = "Active Wedge Theory"

BLOCK TYPE AND PATTERN:
BlockType = "AB COLLECTION"
BlendType = "NO PATTERN"

EXTERNAL STABILITY:

Static Conditions:
Factor of Safety for Sliding: \( F_{S\text{staticsliding}} = 2.612 \)
Factor of Safety for Overturning: \( F_{S\text{staticoverturning}} = 5.143 \)

Seismic Conditions:
Factor of Safety for Sliding: \( F_{S\text{seismicsliding}} = 2.612 \)
Factor of Safety for Overturning: \( F_{S\text{seismicoverturning}} = 5.143 \)

GEOGRID PARAMETERS:
Geogrid Type A: \( A = \text{"Strata 200"} \)
Geogrid Type B: \( B = \text{"Strata 350"} \)
Number of Layers: \( g = 4 \) Layers
Geogrid Length: \( L = 4 \text{ ft} \)
\( L_{\text{top}} = 7 \text{ ft} \)

Base Footing Dimensions:
Width of Footing: \( L_{\text{width}} = 2.0 \text{ ft} \)
Toe Extension: \( L_{\text{toe}} = -0.5 \text{ ft} \)
Depth of Footing: \( L_{\text{depth}} = 0.5 \text{ ft} \)
Width of Reinforcement: \( L_{\text{grid}} = 0 \text{ ft} \)

\text{Note:} \quad \text{When reinforcement is present it shall always be placed 6in from the bottom of the footing.}

\text{The minimum footing dimensions are 6in deep by 24in wide. If the values specifying the footing dimensions are not greater than 6in X 24in, the minimum size should be used. When geogrid reinforcement is present the minimum footing depth shall be 12in to provide 6in of minimum cover above and below the geogrid.}

INTERNAL STABILITY: Local Top of the Wall Stability

Static Conditions:
Factor of Safety for Sliding: \( F_{S\text{sliding_s_top}} = 164.2 \)
Factor of Safety for Overturning: \( F_{S\text{overturning_s_top}} = 30.12 \)

Seismic Conditions:
Factor of Safety for Sliding: \( F_{S\text{sliding_d_top}} = 164.2 \)
Factor of Safety for Overturning: \( F_{S\text{overturning_d_top}} = 30.12 \)

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## INTERNAL STABILITY:

### Static Conditions:
- **Geogrid Length:**
  - **L = 4 ft**
  - **Ltop = 7 ft**

<table>
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<th>Geogrid Number</th>
<th>Geogrid Elev.</th>
<th>Tensile Force</th>
<th>Allowable Load</th>
<th>Factor Safety Overstress</th>
<th>Factor Safety Pullout Block:</th>
<th>Factor Safety Pullout, Soil:</th>
<th>Geogrid Efficiency, %</th>
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<td>Ee&lt;sub&gt;j&lt;/sub&gt; =</td>
<td>Fis&lt;sub&gt;j&lt;/sub&gt; =</td>
<td>LTDS&lt;sub&gt;j&lt;/sub&gt; =</td>
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<td>FS&lt;sub&gt;overstress&lt;/sub&gt; &lt;sub&gt;s&lt;/sub&gt; =</td>
<td>FS&lt;sub&gt;conn&lt;/sub&gt; &lt;sub&gt;s&lt;/sub&gt; =</td>
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<td>45.894 · plf</td>
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