GRAVITY WALL DESIGN METHODOLOGY
STONE STRONG PRECAST MODULAR BLOCK

Evaluate according to industry practice following AASHTO and NCMA analytical techniques – refer to:

AASHTO Standard Specifications for Highway Bridges 2002, 17th Addition
NCMA Design Manual for Segmental Retaining Wall, Second Edition

Additional analytical methods and theories are taken from other AASHTO versions and other FHWA guidelines – refer to:

Mechanically Stabilized Earth Walls and Reinforced Slopes design and Construction Guidelines, NHI-00-043

Properties of Soil/Aggregate

soil and material properties should be determined for the specific materials to be used.

unit fill - $\gamma_a = 110$ pcf max, (see AASHTO 2002 5.9.2) & $\phi_u$
leveling base – aggregate base typical $\gamma_b$ & $\phi_b$ (or concrete base may be substituted)
retained soil - $\gamma$ & $\phi$ by site conditions
foundation soil - $\gamma$ $\phi$ & c by site conditions
interface angle - $\delta = \frac{1}{2} \phi$ (see AASHTO 2002 5.9.2)

Geometric Properties

Effective weight of unit
block weight
24 SF unit – 750 lb/ft of wall
6 SF unit – 450 lb/ft of wall

weight of aggregate
24 SF unit – 596 lb/ft of wall
6 SF unit – 296 lb/ft of wall

Only 80% of the weight of aggregate and soil is included in the overturning calculations, $W'$ (see AASHTO 2002 5.9.2).
Typical gravity wall configuration:
Unit Width/Center of Mass

The nominal unit width is 44 inches for both 24 SF and 6 SF blocks. The combined center of mass of the concrete block and the unit fill is at 22.7 inches from the face. These values may be reduced by up to 2 inches to account for the rounding of the face.

\[ w_u = 3.50 \text{ ft} \]
\[ x_u = 1.73 \text{ ft} \]

Wall batter

The wall system is based around the 24 SF block that is 36 inches of height. The next block atop a 24 SF block will batter back 4 inches. The 6 SF block is 18 inches tall, and the next block atop a 6 SF block will batter 2 inches.

- 4 in. setback per 24 SF block (36 in. tall)
- 2 in. setback per 6 SF block (18 in. tall)

\[ \omega = \tan^{-1}(4/36) = 6.34^\circ \]
\[ \omega' = \tan^{-1}(4/36) = 6.34^\circ \text{ (batter along back face matches the batter along the front)} \]

Base Thickness/Embedment

The type and thickness of wall base or leveling pad and depth of embedment can vary by site requirements. A granular base with a thickness of 9 inches is commonly used, but the thickness can be adjusted to reduce the contact pressure. A concrete leveling pad or footing can also be used. The required embedment to the top of the base is related to the exposed height of the wall and by the slope at the toe, as well as other factors. The required embedment can be calculated for slopes steeper than 6H:1V using the following equation:

\[ h_e = \frac{H'}{20*S/6} \]

where \( S \) is the run of the toe slope per unit fall and \( H' \) is the exposed height

A minimum embedment of 6 to 9 inches is recommended for private projects. A minimum embedment of 20 inches or more may be required for highway applications.
Tail Extension Adjustments

The gravity wall capability can be increased by using a precast Mass Extender block (limited to approximately 12 additional inches, for a total block width of 56 inches) or a cast-in-place tail extension (width is not limited – recommend height be at least 2 times the width to provide shear through the tail openings).

If tail extensions are used, the following adjustments are made:

Wall batter

Wall batter is recalculated along the back of the wall from the rear of the tail extension to the rear of the top of the wall. Use $\omega'$ in Coulomb equation and earth pressure component calculations. To calculate $\omega'$ it is necessary to know the effective setback width, $w_s$, which is the horizontal distance between the back edge of the top block and the back edge of the mass extender at the bottom. $w_s$ is the batter of the front face minus the length of tail extension, $w_{te}$. $w_s$ is negative when the mass extender projects further than the back of the top block. Knowing this distance and the height of wall:

$$\omega' = \arctan\left(\frac{w_s}{H_w}\right)$$

Interface Angle

$$\delta = \frac{3}{4} \phi \quad \text{(see AASHTO 2002 5.9.2)}$$

Weight of Wall

The weight of the wall includes the contributions of the mass extender and the soil wedge atop the mass extender. A typical concrete unit weight is 145 pcf. Use the soil unit weight for the soil wedge.

$$W_{te} = (w_{te} \times H_{te}) \times 145 \text{pcf}$$

where $w_{te}$ is the width of the tail extension and $H_{te}$ is the height of the extension (both in ft)

The weight of the soil triangle is calculated using the following equation:

$$W_s = (H - H_{te}) \times \gamma \times \frac{w_{te}}{2}$$

Note: The soil wedge is defined by the limit of the tail extension and not by the simplified batter of the back of the wall. The simplified batter is used in the earth pressure analysis. Since the minimum width of the tail extension is typically maintained, it may project beyond the extension at the first course.
Typical gravity wall configuration with tail extension:
Calculate Forces

Coulomb active earth pressure coefficient (see AASHTO 2002 5.5.2-1)

\[
K_a = \frac{\cos^2(\phi + \omega')}{\cos^2(\omega')\cos(\omega'-\delta)\left[1 + \frac{\sin(\phi + \delta)\sin(\phi - \beta)}{\cos(\omega'-\delta)\cos(\omega'+\beta)}\right]^2}
\]

Earth load components  (see AASHTO 2002 5.5.2-1)

Vertical Forces:

\[P_v = 0.5 \ K_a \gamma H^2 \sin(\delta - \omega')\]
\[Q_{dv} = K_a Q*H*\sin(\delta - \omega')\text{ where } Q\text{ is the effective surcharge in psf}\]

Horizontal Forces:

\[P_h = 0.5 \ K_a \gamma H^2 \cos(\delta - \omega')\]
\[Q_{dh} = K_a Q*H*\cos(\delta - \omega')\text{ where } Q\text{ is the effective surcharge in psf}\]
\[Q_{lh} = K_a Q*H*\cos(\delta - \omega')\text{ where } Q\text{ is the effective surcharge in psf}\]

Note: Surcharge loads may be divided into dead and live load components. The vertical component of the live load \(Q_{lv}\) is a stabilizing force and should be neglected as conservative.

Resisting forces

Vertical Forces:

\[W_b - \text{ Weight of wall units}\]
\[W_{te} - \text{ Weight of concrete tail extension, if used}\]
\[W_a - \text{ Weight of infill aggregate (use 80% aggregate weight for overturning)}\]
\[W_s - \text{ Weight of soil atop tail extension (use 80% weight for overturning)}\]

The center of gravity of the components of the wall can be calculated by laying out the components of the wall and taking a weighted average of their weight and distance from the hinge point of the block (see AASHTO 2002 5.9.2). Alternately, the center of mass can be calculated using the following equations:
The center of mass of the stack of blocks is calculated as:

\[ x_b = x_u + \frac{(H - h_u)}{2} \cdot \tan(\omega) \]

The center of mass of the soil triangle over the tail is:

\[ x_s = w_u + (H_{te} - h_u) \cdot \tan(\omega) + \frac{2 \cdot w_{te}}{3} - \frac{w_s}{3} \]

The center of mass of the tail extension can be calculated with the following equation:

\[ x_{te} = w_u + \frac{w_{te}}{2} \]

This leads to an overall adjusted center of mass of:

\[ x_w = \left[ x_u + \frac{(H - h_u)}{2} \cdot \tan(\omega) \right] \cdot (W_b + W_a) + x_{te} \cdot W_{te} + x_s \cdot W_s \]

Note: the height of unit, \( h_u \), is taken as 3 ft. based on the 24 SF unit instead of 1.5 ft. based on the 6 SF unit to produce the more conservative result (units can be stacked with either unit as the bottom course).

The resultants of the earth load components are calculated as follows:

\[ x_{Pv} = \frac{H}{3} \cdot \tan(\omega') + w_u + w_{te} \]
\[ x_{Qdv} = \frac{H}{2} \cdot \tan(\omega') + w_u + w_{te} \]
\[ x_{Ph} = \frac{H}{3} \]
\[ x_{Qdh} = \frac{H}{2} \]
\[ x_{Qlh} = \frac{H}{2} \]

### Table of Forces & Moments

<table>
<thead>
<tr>
<th>Force</th>
<th>x</th>
<th>Moment about toe</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(lb)  (ft)   (lb*ft)</td>
</tr>
<tr>
<td><strong>Vertical Forces</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>weight of wall</td>
<td>( W_b + W_a + W_{te} + W_s )</td>
<td>( x_w )</td>
</tr>
<tr>
<td>modified weight</td>
<td>( W_b + 0.8<em>W_a + W_{te} + 0.8</em>W_s )</td>
<td>( x_w )</td>
</tr>
<tr>
<td>earth pressure</td>
<td>( P_v )</td>
<td>( x_{Pv} )</td>
</tr>
<tr>
<td>DL surcharge</td>
<td>( Q_{dv} )</td>
<td>( x_{Qdv} )</td>
</tr>
</tbody>
</table>

| **Horizontal Forces** |   |                   |
| earth pressure | \( P_h \) | \( x_{Ph} \) | \( P_h \cdot x_{Ph} \) |
| DL surcharge | \( Q_{dh} \) | \( x_{Qdh} \) | \( Q_{dh} \cdot x_{Qdh} \) |
| LL surcharge | \( Q_{lh} \) | \( x_{Qlh} \) | \( Q_{lh} \cdot x_{Qlh} \) |
Overturning

For overturning, the modified weights using 80% of the aggregate weight (including the soil over the tail extension) are used for all overturning calculations.

\[
\begin{align*}
M'_V & \quad \Sigma \text{ moments from vertical forces (using 80\% \( W_s \) & \( W_a \))} \\
M_H & \quad \Sigma \text{ moments from horizontal forces} \\
FS & \quad \frac{M'_V}{M_H}
\end{align*}
\]

The overturning safety factor should be greater than 1.5 for private projects (NCMA 4.3 and ICBO 2006 1806.1). A minimum safety factor of 2.0 may be required for highway applications (AASHTO 2002 5.5.5).

Check that \( FS > 1.5 \)

Sliding

Friction on the base of the wall is used to resist sliding failure. Frictional resistance must be determined both between the wall assembly and the base and between the base and the foundation soil (or through the foundation soil).

The sliding resistance is calculated as the smaller result of the following equations:

For base to foundation soil failure, use:

\[
R_{s(foundation \ soil)} (W + P_v + Q_{dv}) \tan \phi + B_w \cdot c
\]

\[
B_w = w_t + w_{te} + t_b
\]

where \( \phi \) represents foundation soils, \( B_w \) is base width (block width plus \( \frac{1}{2}H:1V \) distribution through base), and \( c \) represents foundation soil cohesion

For block to base material sliding, use:

\[
R_{s(footing)} = \mu_b (W + P_v + Q_{dv})
\]

where \( \mu_b \) represents a composite coefficient of friction for the base

The composite friction coefficient is calculated using contributory areas. The base of the standard Stone Strong 24 SF unit is 80 percent open and 20 percent concrete. On a unit width basis, the contributory area is 0.73 sf of concrete and 2.94 sf of aggregate.

If a tail extension is used, the area of the tail extension must also be calculated and the total area is also increased accordingly. Thus, the equation for composite friction coefficient across the base becomes:

\[
\mu_b = \frac{(2.94 \cdot \mu_p - \text{unit fill/base} + 0.73 \cdot \mu_p - \text{block/base} + w_{te} \cdot \mu_p - \text{extension/base})}{(3.67 + w_{te})}
\]

where \( \mu_p \) is the partial friction coefficient for the indicated materials (dimensions in ft)
Partial friction coefficients can be interpreted from the following table:

<table>
<thead>
<tr>
<th>Block to Aggregate Base</th>
<th>Coefficient of Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>formed precast surface on compacted aggregate surface (includes Mass Extender)</td>
<td>0.8*tan $\phi_b$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Unit Fill to Aggregate Base</th>
<th>Coefficient of Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>screened aggregate (loose to moderate relative density - dumped) on compacted aggregate surface</td>
<td>lower tan $\phi_b$ or tan $\phi_u$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Block to Concrete Base</th>
<th>Coefficient of Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>formed precast surface on floated concrete surface (includes Mass Extender)</td>
<td>0.60</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Unit Fill Aggregate to Concrete Base</th>
<th>Coefficient of Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>screened aggregate (loose to moderate relative density - dumped) on floated concrete surface</td>
<td>0.8*tan $\phi_u$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete Tail Extension to Aggregate Base</th>
<th>Coefficient of Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>cast in place concrete on aggregate surface</td>
<td>tan $\phi_b$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete Tail Extension to Concrete Base</th>
<th>Coefficient of Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>cast in place concrete on floated concrete surface</td>
<td>0.75</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete Tail Extension Directly on Foundation Soil (Sand)</th>
<th>Coefficient of Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>cast in place concrete on granular soil</td>
<td>tan $\phi_f$</td>
</tr>
</tbody>
</table>

Note: These typical values may be used for evaluation of base sliding at the discretion of the user. The licensed engineer of record is responsible for all design input and for evaluating the reasonableness of calculation output based upon his/her knowledge of local materials and practices and on the specific design details.

Since the unit fill aggregate is typically placed to a moderately loose state, the friction angle for the screened unit fill aggregate typically controls for the interface between the unit fill and the base aggregate.

If actual test data for the project specific materials is not available, or for preliminary design, the following conservative friction angles are suggested for base material:

<table>
<thead>
<tr>
<th>Friction Angle (degrees)</th>
<th>Well Graded, Densely Compacted</th>
<th>Screened Aggregate, Compacted</th>
<th>Screened Aggregate, Loose to Moderate Relative Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed Hard Aggregate</td>
<td>&gt;75% w/ 2 fractured faces, hard natural rock</td>
<td>42</td>
<td>40</td>
</tr>
<tr>
<td>Crushed Aggregate</td>
<td>&gt;75% w/ 2 fractured faces, medium natural rock or recycled concrete</td>
<td>40</td>
<td>38</td>
</tr>
<tr>
<td>Cracked Gravel</td>
<td>&gt;90% w/ 1 fractured face</td>
<td>36</td>
<td>35</td>
</tr>
</tbody>
</table>

Note: Physical testing of specific aggregates is recommended. When test data is not available, these typical values may be used at the discretion of the user. The licensed engineer of record is responsible for all design input and for evaluating the reasonableness of calculation output based upon his/her knowledge of local materials and practices and on the specific design details.
The minimum value for sliding resistance is calculated as follows:

<table>
<thead>
<tr>
<th></th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_H$</td>
<td>$\Sigma$ horizontal forces</td>
</tr>
<tr>
<td>$F_V$</td>
<td>$\Sigma$ vertical forces (using 100% $W_s$ &amp; $W_a$)</td>
</tr>
<tr>
<td>$R'_s$ (footing)</td>
<td>$\mu_b F_V$</td>
</tr>
<tr>
<td>$R'_s$ (foundation soil)</td>
<td>$[F_V \tan(\phi) + B_w c]$</td>
</tr>
<tr>
<td>min $R'_s$</td>
<td>smaller of $R'_s$ (footing) or $R'_s$ (foundation soil)</td>
</tr>
<tr>
<td>$FS$</td>
<td>$\text{min } R'_s / F_H$</td>
</tr>
</tbody>
</table>

The safety factor for sliding should be greater than 1.5

check that $FS > 1.5$

**Bearing/Eccentricity**

$B'_f$ is the equivalent bearing area. This is the base block width adjusted for eccentricity, and including a $\frac{1}{2}H:1V$ distribution through granular base or $1H:1V$ distribution through concrete base.

$$B'_f = w_u + w_{te} + t_b - 2*e$$

or

$$B'_f = w_u + w_{te} + 2*t_b - 2*e$$ (for concrete base)

<table>
<thead>
<tr>
<th></th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_V$</td>
<td>$\Sigma$ vertical forces (using 100% $W_s$ &amp; $W_a$)</td>
</tr>
<tr>
<td>weight of base</td>
<td>$t_b \gamma_b$</td>
</tr>
<tr>
<td>$M_v$</td>
<td>$\Sigma$ moments from vertical forces (using 100% $W_s$ &amp; $W_a$)</td>
</tr>
<tr>
<td>$M_H$</td>
<td>$\Sigma$ moments from horizontal forces</td>
</tr>
<tr>
<td>$e$</td>
<td>$(w_u + w_{te})/2 - (M_V - M_H)/F_V$</td>
</tr>
<tr>
<td>$B'_f$ (granular base)</td>
<td>$w_u + w_{te} + t_b - 2*e$</td>
</tr>
<tr>
<td>$B'_f$ (concrete base)</td>
<td>$w_u + w_{te} + 2<em>t_b - 2</em>e$</td>
</tr>
<tr>
<td>contact pressure $q_c$</td>
<td>$F_V / B'_f + t_b \gamma_b$</td>
</tr>
<tr>
<td>bearing resistance $q_b$</td>
<td>$[c*N_c + (h_o + t_b)\gamma_{found}<em>N_q + 0.5</em>\gamma_{found}<em>B'_f</em>N_f]$</td>
</tr>
<tr>
<td>$FS$</td>
<td>$q_b / q_c$</td>
</tr>
</tbody>
</table>

The safety factor for bearing should be greater than 2

Check that $FS > 2.0$
Seismic Design

Seismic components of force are calculated according to the procedures in FHWA 4.2h. The maximum acceleration \( A_m = (1.45 - A) \times A \) where \( A \) is the peak horizontal ground acceleration.

The seismic earth pressure coefficient is calculated with the following equation:

\[
K_{ae} = \frac{\cos^2 \left( \phi + \omega - \xi \right)}{\cos(\xi) \cos^2 \left( -\omega \right) \cos(\delta - \omega + \xi) \left[ 1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \xi - \beta)}{\cos(\delta - \omega + \xi) \cos(\omega + \beta)}} \right]^2}
\]

where \( \xi = \arctan \left( \frac{K_h}{1 - K_v} \right) \). \( K_v \) is generally taken as 0. \( K_h \) is the maximum horizontal acceleration of the wall, and is a function of the maximum allowable displacement of the wall during a seismic event. It is calculated with the following equation:

\[
K_h = 1.66 \times A_m \times [A_m/(d \times 25.4)]^{0.25}
\]

with \( d = 2 \) inches, the conservatively assumed maximum horizontal displacement

The horizontal inertia force \( P_{ir} \) is calculated as follows:

\[
P_{ir} = 0.5 \times K_h \times \gamma \times H_2 \times H + 0.125 \times K_h \times \gamma \times H_2 \times \tan(\beta)
\]

where \( H_2 \) is the height of backfill at the back of the block.

The seismic thrust is calculated as follows:

\[
P_{ae} = 0.5 \times \gamma \times H_2 \times (K_{ae} - K_a)
\]
\[
P_{aeh} = 0.5 \times \gamma \times H_2 \times (K_{ae} - K_a) \times \cos(\delta - \omega)
\]

In overturning analysis, the inertial force is applied at half the height of the wall, while the seismic thrust is applied at 60% of the wall height. By AASHTO requirements, the full inertial force is applied along with 50% of the seismic thrust (FHWA 4.2h).
The total overturning moment is increased as shown in the following equation:

$$M_H + P_{ir} \cdot H/2 + (P_{aeh}/2) \cdot (0.6 \cdot H)$$

The total horizontal sliding force is increased as shown in the following equation:

$$F_H + P_{ir} + (P_{aeh}/2)$$

Seismic load conditions should be verified for sliding, overturning/eccentricity, and bearing. Live loads are typically excluded from seismic analysis.

**Internal Analysis**

Internal stability analysis is conducted for each segment of block. Since bearing conditions are addressed in the external stability analysis, only overturning and shearing failures are possible.

Overturning is evaluated identically to external stability analysis.

Sliding resistance is calculated based on the interface shear test (see interaction test reports for complete test data)

$$R'_s = [362 + (W + P_v + Q_{dv}) \cdot \tan (35.2^\circ)]$$

For each load case, the sliding safety factor must be greater than 1.5:

$$FS = R'_s / F_H$$

check that $FS > 1.5$

At a minimum, internal stability should be evaluated at each change in block width (i.e. immediately above the mass extender), any change in mass extender size and at the base of any dual-face units.
Note: Examples to demonstrate method of analysis only - not intended to conform w/ AASHTO safety factors

**Example section – 9 ft tall unreinforced wall, 4H:1V backslope, sand backfill**

Uniform soil (sand) - $\gamma = 125$ pcf $\phi = 30^\circ$ $c = 0$ psf

Wall is composed of three 24 SF blocks

$$\omega' = \arctan\left(\frac{3\times4^\prime}{9\text{ ft}\times12^\prime/\text{ft}}\right) = 6.34^\circ \quad \delta = \frac{1}{2}\times30^\circ = 15^\circ$$

Granular base aggregate – $\phi = 40^\circ$

Unit fill aggregate – $\phi = 35^\circ$

**Weight of Wall**

$$W_b = \frac{3\times6,000 \text{ lb}}{8 \text{ ft}} = 2,250 \text{ lb/ft block}$$

$$W_a = \frac{3\times43.32 \text{ ft}^3\times110 \text{ pcf}}{8 \text{ ft}} = 1,787 \text{ lb/ft aggregate fill}$$

Total Wall Weight = 2,250 + 1,787 = 4,037 lb/ft

$$W' = 1,787 \text{ lb/ft}\times0.80 + 2,250 \text{ lb/ft} = 3,680 \text{ lb/ft}$$

**Forces/Geometric Properties**

**Center of Gravity**

$$x_{block} = 22.7^\prime \text{ from face, with 2 additional inches removed due to rounding – 1.73’ total}$$

$$x_w = \left[\frac{(1.73+0.5\times(9 \text{ ft}-3 \text{ ft})\times\tan(6.34^\circ))\times(2,250 \text{ lb} + 1,787 \text{ lb})}{4,037 \text{ lb}}\right] = 2.06 \text{ ft}$$

$$w_u = \frac{44 \text{ in-2 in}}{12} = 3.5 \text{ ft}$$

**Soil force components**

$$K_a = \frac{\cos^2(30^\circ + 6.34^\circ)}{\cos^2(6.34^\circ)\cos(6.34^\circ - 15^\circ)\left[1 + \frac{\sin(30^\circ + 15^\circ)\sin(30^\circ - 14.0^\circ)}{\cos(6.34^\circ - 15^\circ)\cos(6.34^\circ + 14.0^\circ)}\right]^2} = 0.313$$

$$P_h = 0.5\times(0.313)\times125\text{pcf}\times(9 \text{ ft})^2\times\cos(15^\circ - 6.34^\circ) = 1,564 \text{ lb/ft}$$

$$P_v = 0.5\times(0.313)\times125\text{pcf}\times(9 \text{ ft})^2\times\sin(15^\circ - 6.34^\circ) = 238 \text{ lb/ft}$$
Overturning

FS = \[3,680 \text{ lb/ft} \times 2.06 \text{ ft} + 238 \text{ lb} \times (3.5 \text{ ft} + 9 \text{ ft} \times 3 \times \tan(6.34^\circ))\] / [1,564 \text{ lb} \times 9 \text{ ft} / 3]

= 1.81 > 1.5  \quad \text{OK!}

Sliding

\[\mu_b = \frac{[(0.8 \times 3.67 \text{ ft} \times \tan(35^\circ)) + (0.2 \times 3.67 \text{ ft} \times 0.8 \times \tan(40^\circ))]}{3.67 \text{ ft}} = 0.69\]

Use the smaller of the following:

\[R_s = 0.69 \times (4,037 \text{ lb/ft} + 238 \text{ lb/ft}) = 2,950 \text{ lb/ft}\]

\[R_s = (4,037 \text{ lb/ft} + 238 \text{ lb/ft}) \times \tan(30^\circ) + 0 = 2,468 \text{ lb/ft}\]

FS = 2,468 lb/ft / 1,564 lb/ft = 1.58 > 1.5  \quad \text{OK!}

Bearing

\[N_q = e^{\pi \times \tan(30^\circ)} \times (\tan(45^\circ + 30^\circ / 2))^2 = 18.40\]

\[N_c = (18.40 - 1) / \tan(30^\circ) = 30.14\]

\[N_{\gamma} = 2 \times (18.40 + 1) \times \tan(30^\circ) = 22.40\]

\[e = \frac{1,564 \text{ lb/ft} \times 9 \text{ ft} - 4,037 \text{ lb/ft} \times (2.06 \text{ ft} - 3.5 \text{ ft} / 2) - 238 \text{ lb/ft} \times (3.5 \text{ ft} / 2 + 9 \text{ ft} / 2 \times \tan(6.34^\circ))}{4,037 \text{ lb/ft} + 238 \text{ lb/ft}} = 0.69 \text{ ft}\]

\[B_f' = 3.5 \text{ ft} + 0.75 \text{ ft} - 2 \times 0.69 \text{ ft} = 2.88 \text{ ft}\]

\[q_c = (4,037 \text{ lb} + 238 \text{ lb/ft}) / 2.88 \text{ ft} + 0.75 \text{ ft} \times 125 \text{ pcf} = 1,580 \text{ psf}\]

\[q_b = 0 \times 30.14 + (9'' + 9'') / 12 \times 125 \text{ pcf} = 18.40 + 0.5 \times 125 \text{ pcf} = 2.88 \text{ ft} \times 22.40 = 7,479 \text{ psf}\]

FS = 7,479 psf / 1,580 psf = 4.73 > 2.0  \quad \text{OK!}
Internal Analysis, Upper 2 Courses

Weight of Wall

\[ W_b = \frac{(2*6,000 \text{ lb})}{8 \text{ ft}} = 1,500 \text{ lb/ft block} \]
\[ W_a = \frac{(2*43.32 \text{ ft}^3*110 \text{ pcf})}{8 \text{ ft}} = 1,191 \text{ lb/ft aggregate fill} \]

Total Wall Weight = 1,500 lb/ft + 1,191 lb/ft = 2,691 lb/ft

\[ W' = 1,191 \text{ lb/ft}*0.80+1,500 \text{ lb/ft} = 2,453 \text{ lb/ft} \]

Forces/Geometric Properties

Center of Gravity

\[ x_{\text{block}} = 22.7" \text{ from face, with 2 additional inches removed due to rounding} - 1.73' \text{ total} \]
\[ x_w = \left[ \frac{(1.73+0.5*(6 \text{ ft}-3 \text{ ft})*\tan(6.34°))*(1,500 \text{ lb/ft} + 1,191 \text{ lb/ft})}{2,153 \text{ lb/ft}} \right] = 1.90 \text{ ft} \]
\[ w_u = \frac{(44 \text{ in}-2 \text{ in})}{12} = 3.5 \text{ ft} \]

Soil force components

\[ K_a = 0.313 \]
\[ P_h = 0.5*(0.313)*125\text{pcf}*(6 \text{ ft})^2*cos(15° - 6.34°) = 695 \text{ lb/ft} \]
\[ P_v = 0.5*(0.313)*125\text{pcf}*(6 \text{ ft})^2*sin(15° - 6.34°) = 106 \text{ lb/ft} \]

Overturning

\[ \text{FS} = \frac{2,453*1.90 \text{ ft} +106 \text{ lb}*(3.5 \text{ ft}+6 \text{ ft} / 3*\tan(6.34°))}{695 \text{ lb}*6 \text{ ft}/3} \]
\[ = 3.63 > 1.5 \quad \text{OK!} \]

Interface Shear

\[ R_s = 362 \text{ lb/ft} + \frac{(1,500 \text{ lb/ft} + 1,191 \text{ lb/ft} + 106 \text{ lb/ft})*\tan(35.2°)}{2,335 \text{ lb/ft}} \]

\[ \text{FS} = \frac{2,335 \text{ lb/ft} / 695 \text{ lb/ft}}{3.36 > 1.5} \quad \text{OK!} \]
Internal Analysis, Top Course

Weight of Wall
\[ W_b = \frac{6,000 \text{ lb}}{8 \text{ ft}} = 750 \text{ lb/ft block} \]
\[ W_a = \frac{43.32 \text{ ft}^3 \times 110 \text{pcf}}{8 \text{ ft}} = 596 \text{ lb/ft aggregate fill} \]
Total Wall Weight = 750 lb/ft + 596 lb/ft = 1,346 lb/ft
\[ W' = 596 \text{ lb/ft} \times 0.80 + 750 \text{ lb/ft} = 1,227 \text{ lb/ft} \]

Forces/Geometric Properties

Center of Gravity
\[ x_{\text{block}} = 22.7'' \text{ from face, with 2 additional inches removed due to rounding} = 1.73 \text{ ft total} \]
\[ w_u = \frac{44 \text{ in} - 2 \text{ in}}{12} = 3.5 \text{ ft} \]

Soil force components
\[ K_a = 0.313 \]
\[ P_h = 0.5 \times (0.313) \times 125 \text{pcf} \times (3 \text{ ft})^2 \times \cos(15^\circ - 6.34^\circ) = 174 \text{ lb/ft} \]
\[ P_v = 0.5 \times (0.13) \times 125 \text{pcf} \times (3 \text{ ft})^2 \times \sin(15^\circ - 6.34^\circ) = 23 \text{ lb/ft} \]

Overturning
\[ FS = \frac{[1,227 \text{ lb/ft} \times 1.73 \text{ ft} + 26 \text{ lb}(3.5 \text{ ft} + 3 \text{ ft} / 3 \times \tan(6.34^\circ))]}{[174 \text{ lb/ft} \times 3 \text{ ft} / 3]} \]
\[ = 12.75 > 1.5 \quad \text{OK!} \]

Interface Shear
\[ R_s = 362 \text{ lb/ft} + (750 \text{ lb/ft} + 596 \text{ lb/ft} + 26 \text{ lb/ft}) \times \tan(35.2^\circ) = 1,330 \text{ lb/ft} \]
\[ FS = \frac{1,330 \text{ lb/ft}}{174 \text{ lb/ft}} = 7.65 > 1.5 \quad \text{OK!} \]
Example Gravity Calculation

**Project #**

08110.00

**Date**

6/28/09

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**Example section – 10.5 ft tall wall, 150 psf surcharge, 18”x3’ tail extension, clay backfill**

Uniform soil (sand) - $\gamma = 120$ pcf $\phi = 26^\circ$ $c = 150$ psf

Wall is composed of three 24 SF blocks and one 6 SF block

$\omega = \arctangent(-8 \text{ in}/10.5 \text{ ft}) = -3.63^\circ$ $\delta = 3/4 \times 26^\circ = 19.5^\circ$

Granular base aggregate – $\phi = 40^\circ$

Unit fill aggregate – $\phi = 35^\circ$

**Weight of Wall**

$W_b = (3 \times 6,000 \text{ lb})/8 \text{ ft} + 1,600 \text{ lb}/4 \text{ ft} = 2,650 \text{ lb}/\text{ft block}$

$W_a = (3 \times 43.32 \text{ ft}^3 \times 110 \text{ pcf})/8 \text{ ft} + (10.75 \text{ ft}^3 \times 110 \text{ pcf})/4 \text{ ft} = 2,083 \text{ lb}/\text{ft aggregate fill}$

$W_{te} = 18 \text{ in}/12 \times 3 \text{ ft} \times 145 \text{ pcf} = 653 \text{ lb}/\text{ft}$

$W_{sot} = (10.5 \text{ ft} - 3 \text{ ft}) \times 18 \text{ in}/12/2 \times 120 \text{ pcf} = 675 \text{ lb}/\text{ft}$

Total Wall Weight = 2,650 + 2,083 + 653 + 675 = 6,061 lb/ft

$W' = (2,033 \text{ lb}/\text{ft} + 675 \text{ lb}/\text{ft}) \times 0.80 + 2,700 \text{ lb}/\text{ft} + 653 \text{ lb}/\text{ft} = 5,509 \text{ lb}/\text{ft}$

**Forces/Geometric Properties**

**Center of Gravity**

$x_{block} = 22.7''$ from face, with 2 additional inches removed due to rounding – 1.73’ total

$w_u = (44 \text{ in}-2 \text{ in})/12 = 3.5 \text{ ft}$

$w_s = 2 \times 4 \text{ in} + 2 \text{ in}-18 \text{ in} = -8 \text{ in}$

$x_w = [(1.73 + 0.5 \times (10.5 \text{ ft} - 3 \text{ ft}) \times \tan(6.34^\circ)) \times (2,650 \text{ lb} + 2,083 \text{ lb}) + (3.5 \text{ ft} + 18 \text{ in}/12) / 2 \times 653 \text{ lb}/\text{ft}$

$+ (3.5 \text{ ft} + 2 \times 3 \times 18 \text{ in}/12 + 1/3 \times (-8 \text{ in})/12) \times 675 \text{ lb}/\text{ft}] / 6,061 \text{ lb} = 2.61 \text{ ft}$

**Soil force components**

$K_a = \frac{\cos^2(26^\circ + -3.63^\circ)}{\cos^2(-3.63^\circ) \cos(-3.63^\circ - 19.5^\circ) \left[ 1 + \frac{\sin(26^\circ + 19.5^\circ) \sin(26^\circ - 0^\circ)}{\cos(-3.63^\circ - 19.5^\circ) \cos(-3.63^\circ + 0^\circ)} \right]^2} = 0.372$

$P_h = 0.5 \times (0.372) \times 120 \text{ pcf} \times (10.5 \text{ ft})^2 \times \cos(19.5^\circ - (-3.63^\circ)) = 2,265 \text{ lb}/\text{ft}$

$P_v = 0.5 \times (0.372) \times 120 \text{ pcf} \times (10.5 \text{ ft})^2 \times \sin(19.5^\circ - (-3.63^\circ)) = 967 \text{ lb}/\text{ft}$

$Q_h = 0.372 \times 150 \text{ psf} \times 10.5 \text{ ft} \times \cos(19.5^\circ - (-3.63^\circ)) = 539 \text{ lb}/\text{ft}$
Overturning

FS = \[\frac{5,509 \times 2.61 \text{ ft} + 967 \text{ lb} \times (3.5 \text{ ft} + 18 \text{ in}/12 + 10.5 \text{ ft}/3 \times \tan(-3.63^\circ))}{2,265 \text{ lb} \times 10.5 \text{ ft}/3 + 539 \text{ lb} \times 10.5 \text{ ft}/2}\] = 1.77 > 1.5  \textbf{OK!}

Sliding

\[\mu_b = \frac{(0.8 \times 3.67 \text{ ft} \times \tan(35^\circ)) + (0.2 \times 3.67 \text{ ft} \times 0.8 \times \tan(40^\circ)) + 0.84 \times 18 \text{ in}/12}{3.67 \text{ ft} + 18 \text{ in}/12}\] = 0.74

Use the smaller of the following:

\[R_s = 0.74 \times (6,061 \text{ lb/ft} + 967 \text{ lb/ft}) = 5,201 \text{ lb/ft}\]
\[R_s = (6,061 \text{ lb/ft} + 967 \text{ lb/ft}) \times \tan(26^\circ) + 150 \text{ psf} \times (3.5 \text{ ft} + 18 \text{ in}/12 + 9 \text{ in}/12) = 4,290 \text{ lb/ft}\]

FS = 4,290 lb/ft / (2,265 lb/ft + 539 lb/ft) = 1.53 > 1.5  \textbf{OK!}

Bearing

\[N_q = e^{-\tan(26^\circ)} \times (\tan(45^\circ + 26^\circ/2))^2 = 11.85\]
\[N_c = (11.85 - 1)/\tan(26^\circ) = 22.25\]
\[N_\gamma = 2 \times (11.85 + 1) \times \tan(26^\circ) = 12.54\]
\[e = \frac{[2,265 \text{ lb/ft} \times 10.5 \text{ ft}/3 + 539 \text{ lb/ft} \times 10.5 \text{ ft}/2 - 6,111 \text{ lb/ft} \times (2.61 \text{ ft} - 3.5 \text{ ft}/2 - 18 \text{ in}/24)}{-967 \text{ lb/ft} \times (3.5 \text{ ft}/2 + 10.5 \text{ ft}/2 \times \tan(-3.63^\circ))} / [6,061 \text{ lb/ft} + 967 \text{ lb/ft}] = 1.12 \text{ ft}\]
\[B_f' = 3.5 \text{ ft} + 18 \text{ in}/12 + 0.75 \text{ ft} - 2 \times 1.12 \text{ ft} = 3.51 \text{ ft}\]
\[q_c = (6,061 \text{ lb} + 967 \text{ lb/ft})/3.52 \text{ ft} + 0.75 \text{ ft} \times 120 \text{ pcf} = 2,099 \text{ psf}\]
\[q_b = 150 \text{ psf} \times 22.25 + (9" + 9")/12 \times 120 \text{ pcf} \times 11.85 + 0.5 \times 120 \text{ pcf} \times 3.52 \text{ ft} \times 12.54 = 8,119 \text{ psf}\]

FS = 8,119 psf/2,099 psf = 3.86 > 2.0  \textbf{OK!}
Internal Analysis, Upper 7.5 feet

Weight of Wall

\[ \begin{align*} W_b &= \frac{(2 \times 6000 \text{ lb})}{8 \text{ ft}} + \frac{1600 \text{ lb}}{4 \text{ ft}} = 1900 \text{ lb/ft block} \\ W_a &= \frac{(2 \times 43.32 \text{ ft}^3 \times 110 \text{ pcf})}{8 \text{ ft}} + \frac{(10.75 \text{ ft}^3 \times 110 \text{ pcf})}{4 \text{ ft}} = 1487 \text{ lb/ft aggregate fill} \end{align*} \]

Total Wall Weight = \( \frac{1900 \text{ lb/ft} + 1487 \text{ lb/ft}}{3387 \text{ lb/ft}} \)

Forces/Geometric Properties

Center of Gravity

\[ \begin{align*} x_{\text{block}} &= 22.7'' \text{ from face, with 2 additional inches removed due to rounding} = 1.73' \text{ total} \\ x_w &= \frac{[(1.73+0.5*(7.5 \text{ ft}-3 \text{ ft})*\tan(6.34°))*(1900 \text{ lb/ft} + 1487 \text{ lb/ft})]}{3387 \text{ lb/ft}} = 1.98 \text{ ft} \\ w_u &= \frac{(44 \text{ in-2 in})}{12} = 3.5 \text{ ft} \\ \omega &= \arctangent(2 \text{ in}/18 \text{ in}) = 6.34° \\ \delta &= \frac{1}{2} \times 26° = 13° \end{align*} \]

Soil force components

\[ \begin{align*} K_a &= \frac{\cos^2(26° + 6.34°)}{\cos^2(6.34°) \cos(6.34° - 13°) \left[ 1 + \frac{\sin(26° + 13°) \sin(26° - 0°)}{\cos(6.34° - 13°) \cos(6.34° + 0°)} \right]^2} = 0.311 \\ P_h &= 0.5 \times (0.311) \times 120 \text{pcf} \times (7.5 \text{ ft})^2 \times \cos(13° - 6.34°) = 1043 \text{ lb/ft} \\ P_v &= 0.5 \times (0.311) \times 120 \text{pcf} \times (7.5 \text{ ft})^2 \times \sin(13° - 6.34°) = 122 \text{ lb/ft} \\ Q_h &= 0.311 \times 150 \text{ psf} \times 7.5 \text{ ft} \times \cos(13° - 6.34°) = 348 \text{ lb/ft} \end{align*} \]

Overturning

\[ \begin{align*} FS &= \frac{[(1900 \text{ lb/ft} + 0.8 \times 1487 \text{ lb/ft}) \times 1.98 \text{ ft} + 122 \text{ lb} \times (3.5 \text{ ft} + 7.5 \text{ ft} / 3 \times \tan(6.34°))]}{[1,043 \text{ lb} \times 7.5 \text{ ft} / 3 + 348 \text{ lb/ft} \times 7.5 \text{ ft} / 2]} = 1.68 > 1.5 \quad \text{OK!} \end{align*} \]

Interface Shear

\[ \begin{align*} R_s &= 362 \text{ lb/ft} + (1900 \text{ lb/ft} + 1487 \text{ lb/ft} + 122 \text{ lb/ft}) \times \tan(35.2°) = 2837 \text{ lb/ft} \\ FS &= \frac{2837 \text{ lb/ft}}{[1,043 \text{ lb/ft} + 348 \text{ lb/ft}]} = 2.04 > 1.5 \quad \text{OK!} \end{align*} \]
Internal Analysis, Upper 4.5 feet

Weight of Wall
\[ W_b = \frac{6,000 \text{ lb}}{8 \text{ ft}} + \frac{1,600 \text{ lb}}{4 \text{ ft}} = 1,150 \text{ lb/ft block} \]
\[ W_a = \frac{43.32 \text{ ft}^3 \times 110 \text{ pcf}}{8 \text{ ft}} + \frac{10.75 \text{ ft}^3 \times 110 \text{ pcf}}{4 \text{ ft}} = 891 \text{ lb/ft aggregate fill} \]
Total Wall Weight = 1,150 lb/ft + 891 lb/ft = 2,041 lb/ft

Forces/Geometric Properties
Center of Gravity
\[ x_{block} = 22.7'' \text{ from face, with 2 additional inches removed due to rounding} - 1.73 \text{ ft total} \]
\[ x_w = \frac{[(1.73+0.5\times(4.5 \text{ ft}-3 \text{ ft})\times\tan(6.34^\circ))\times(1,150 \text{ lb/ft} + 891 \text{ lb/ft})]}{2,041 \text{ lb/ft}} = 1.81 \text{ ft} \]
\[ w_u = \frac{(44 \text{ in}-2 \text{ in})}{12} = 3.5 \text{ ft} \]

Soil force components
\[ K_a = 0.311 \]
\[ P_h = 0.5\times(0.311)\times120 \text{ pcf}\times(4.5 \text{ ft})^2\times\cos(13^\circ - 6.34^\circ) = 376 \text{ lb/ft} \]
\[ P_v = 0.5\times(0.311)\times120 \text{ pcf}\times(4.5 \text{ ft})^2\times\sin(13^\circ - 6.34^\circ) = 44 \text{ lb/ft} \]
\[ Q_h = 0.311\times150 \text{ psf}\times4.5 \text{ ft}\times\cos(13^\circ - 6.34^\circ) = 209 \text{ lb/ft} \]

Overturning
\[ \text{FS} = \frac{[(1,150 \text{ lb/ft}+0.8\times891 \text{ lb/ft})\times1.81 \text{ ft} +44 \text{ lb}\times(3.5 \text{ ft}+4.5 \text{ ft}/3\times\tan(6.34^\circ))]}{[376 \text{ lb}\times4.5 \text{ ft}/3+209 \text{ lb/ft}\times4.5 \text{ ft}/2]} = 3.42 > 1.5 \quad \text{OK!} \]

Interface Shear
\[ R_s = 362 \text{ lb/ft} + (2,041 \text{ lb/ft} + 44 \text{ lb/ft})\times\tan(35.2^\circ)= 1,833 \text{ lb/ft} \]
\[ \text{FS} = 1,833 \text{ lb/ft} /[(376 \text{ lb/ft}+209 \text{ lb/ft})] = 3.14 > 1.5 \quad \text{OK!} \]
Internal Analysis, Upper 1.5 feet

Weight of Wall

\[ W_b = 1,600 \text{ lb/ft} = 400 \text{ lb/ft block} \]
\[ W_a = (10.75 \text{ ft}^3 \times 110 \text{ pcf})/4 \text{ ft} = 296 \text{ lb/ft aggregate fill} \]
Total Wall Weight = 400 lb/ft + 296 lb/ft = 696 lb/ft

Forces/Geometric Properties

Center of Gravity

\[ x_{\text{block}} = 22.7" \text{ from face, with 2 additional inches removed due to rounding} = 1.73 \text{ ft total} \]
\[ w_u = (44 \text{ in-2 in})/12 = 3.5 \text{ ft} \]

Soil force components

\[ K_a = 0.311 \]
\[ P_h = 0.5(0.311)*120 \text{ pcf}*(1.5 \text{ ft})^2*\cos(13^\circ - 6.34^\circ) = 42 \text{ lb/ft} \]
\[ P_v = 0.5(0.311)*120 \text{ pcf}*(1.5 \text{ ft})^2*\sin(13^\circ - 6.34^\circ) = 5 \text{ lb/ft} \]
\[ Q_h = 0.311*150 \text{ psf}*(1.5 \text{ ft})^2*\cos(13^\circ - 6.34^\circ) = 70 \text{ lb/ft} \]

Overturning

\[ FS = \frac{[(400 \text{ lb/ft}+0.8*296 \text{ lb/ft})*1.73 \text{ ft} +5 \text{ lb}*(3.5 \text{ ft}+1.5 \text{ ft}/3*\tan(6.34^\circ))]}{[42 \text{ lb/ft}^2*1.5 \text{ ft}+70 \text{ lb/ft}*1.5 \text{ ft}/2]} = 15.23 > 1.5 \quad \text{OK!} \]

Interface Shear

\[ R_s = 362 \text{ lb/ft} +(696\text{lb/ft} + 5 \text{ lb/ft})\tan(35.2^\circ)= 856 \text{ lb/ft} \]

\[ FS = 856 \text{ lb/ft }/(42 \text{ lb/ft}+70 \text{ lb/ft}) = 7.64 > 1.5 \quad \text{OK!} \]