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Gravity Wall Design - ASD	20004.00	6/2/20

GRAVITY WALL DESIGN - ASD

STONE STRONG PRECAST MODULAR BLOCK

This engineering section presents information for design of Stone Strong retaining walls in a gravity configuration using conventional procedures with safety factors.

The design methodologies presented conform substantially to AASHTO specifications (Standard Specifications for Highway Bridges - 2002). This section includes the following documents:

Gravity Wall Design Methodology (15 pages)
Example Gravity Wall Calculations (9 pages)
Example Spreadsheet Output (12 pages)

The example calculations and example spreadsheet output match identical design conditions and are intended as verification of the spreadsheet method. Note that the Gravity Analysis Spreadsheet is available on the Stone Strong website.

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GRAVITY WALL DESIGN METHODOLOGY (ASD)
STONE STRONG PRECAST MODULAR BLOCK

Evaluate gravity retaining wall using allowable stress design approach following AASHTO and NCMA analytical techniques. Additional requirements, analytical methods, and theories are taken from the International Building Code, other AASHTO versions, and FHWA publications. Refer to:

AASHTO Standard Specifications for Highway Bridges 2002, 17th Edition

NCMA Design Manual for Segmental Retaining Wall, 3rd Edition

FHWA Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, NHI-00-043

International Building Code

Properties of Soil/Aggregate

Soil and material properties should be determined for the specific materials to be used:

unit fill - γ_u = 110 pcf (17.3 kN/m^3) max (see AASHTO 5.9.2) & ϕ_u

leveling base –γ_b & ϕ_b for typical aggregate base (or concrete base may be substituted)

retained soil - γ & ϕ by site conditions (where select backfill is used, select material must encompass entire retained soil influence zone)

foundation soil - γ ϕ & c by site conditions

interface angle (see AASHTO 5.9.2)

For stepped modules, when the block width varies within a vertical section, $\delta = ¾\ \phi$

For cases where all blocks are substantially uniform width, $\delta = ½\ \phi$

Note: infill weight is reduced to account for infill material not engaged by modular units in overturning. Only 80% of the weight of aggregate is included in the overturning calculations, W' (see AASHTO 5.9.2)

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Precast Modular Unit Geometric Properties

(not all units available from all dealers, verify availability)

Block Library – Imperial Units

(not all units available from all dealers, verify availability)

Block Type	Description	Conc. Wt. (lbs)	Void Vol. (ft³)	Length (ft)	Height (ft)	Unit Width (in)	Conc. Cen. of Gravity x_b (in)	Void Cen. of Gravity x_a (in)
6-28	6SF-28 unit (6 square feet)	950	6.65	4	1.50	28	12.8	14.0
6SF	6SF unit (6 square feet)	1,500	10.95	4	1.50	44	21.0	23.5
24SF	24SF unit (24 square feet)	6,000	43.21	8	3.00	44	21.2	24.8
24-ME	24SF Mass Extender unit	10,000	44.94	8	3.00	56	32.7	25.8
24-62	24SF-62 unit	6,800	76.05	8	3.00	62	29.1	33.0
24-86	24SF-86 unit	7,600	117.90	8	3.00	86	40.0	45.1

dimensions are for battered units - for vertical face 24SF units, the width and center of gravity dimensions are all reduced by 1 inch

Block Library – Metric Units

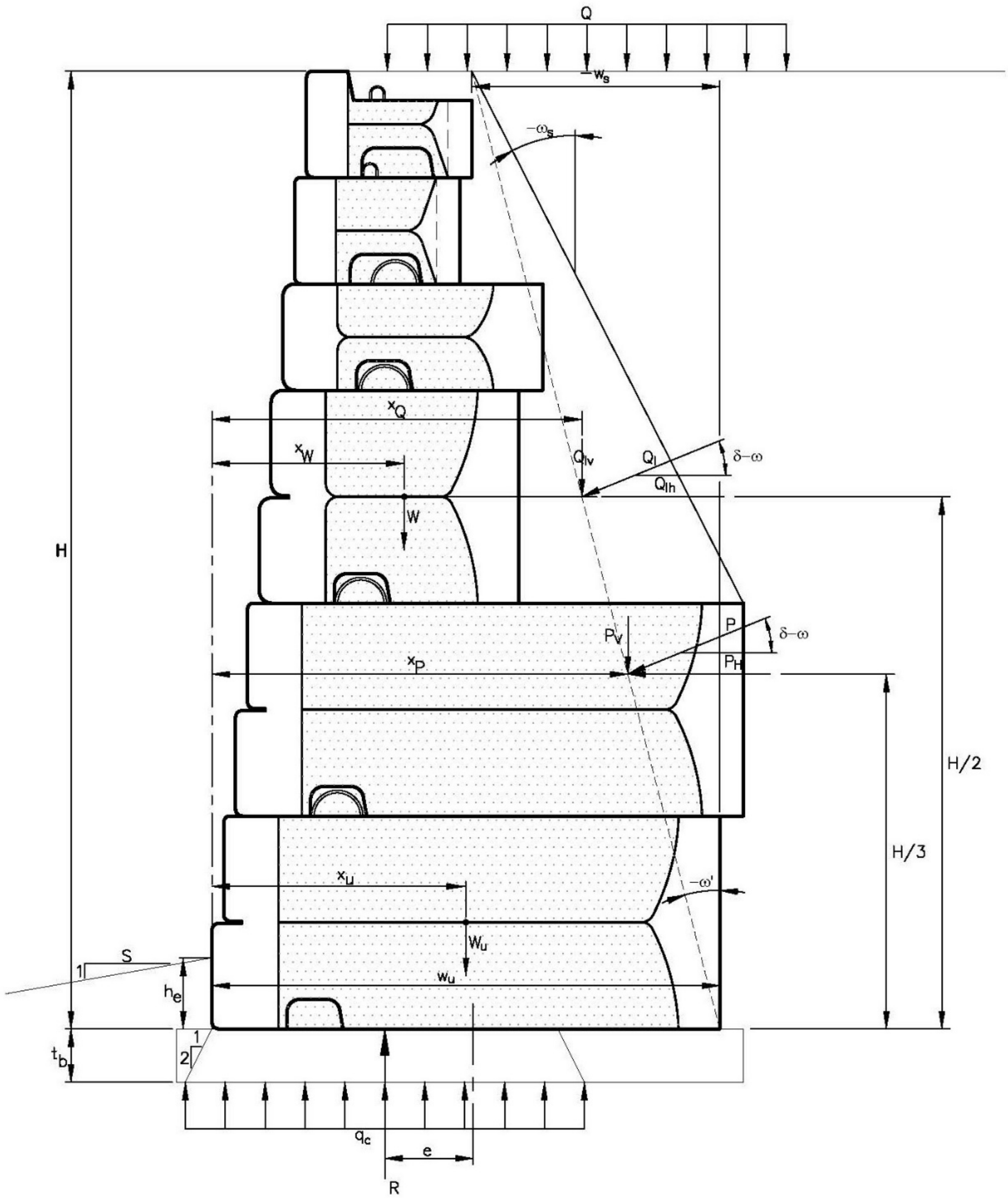
(not all units available from all dealers, verify availability)

Block Type	Description	Conc. Wt. (kN)	Void Vol. (m³)	Length (m)	Height (m)	Unit Width (mm)	Conc. Cen. of Gravity x_b (mm)	Void Cen. of Gravity x_a (mm)
6-28	6SF-28 unit (6 square feet)	4.23	0.19	1.22	0.46	711	324	356
6SF	6SF unit (6 square feet)	6.67	0.31	1.22	0.46	1,118	533	597
24SF	24SF unit (24 square feet)	26.69	1.22	2.44	0.91	1,118	538	630
24-ME	24SF Mass Extender unit	44.48	1.28	2.44	0.91	1,422	831	655
24-62	24SF-62 unit	30.25	2.16	2.44	0.91	1,575	739	838
24-86	24SF-86 unit	33.8	3.35	2.44	0.91	2,184	1,016	1,146

dimensions are for battered units - for vertical f 24SF units, the width and center of gravity dimensions are all reduced by 25 mm

Wall stability calculations are performed per unit length of wall, so all weights and forces are expressed per foot or m of wall length.

Typical gravity wall configuration, variables, and nomenclature:



Note that surcharge loads over the top of the wall are a stabilizing force and are neglected as conservative.

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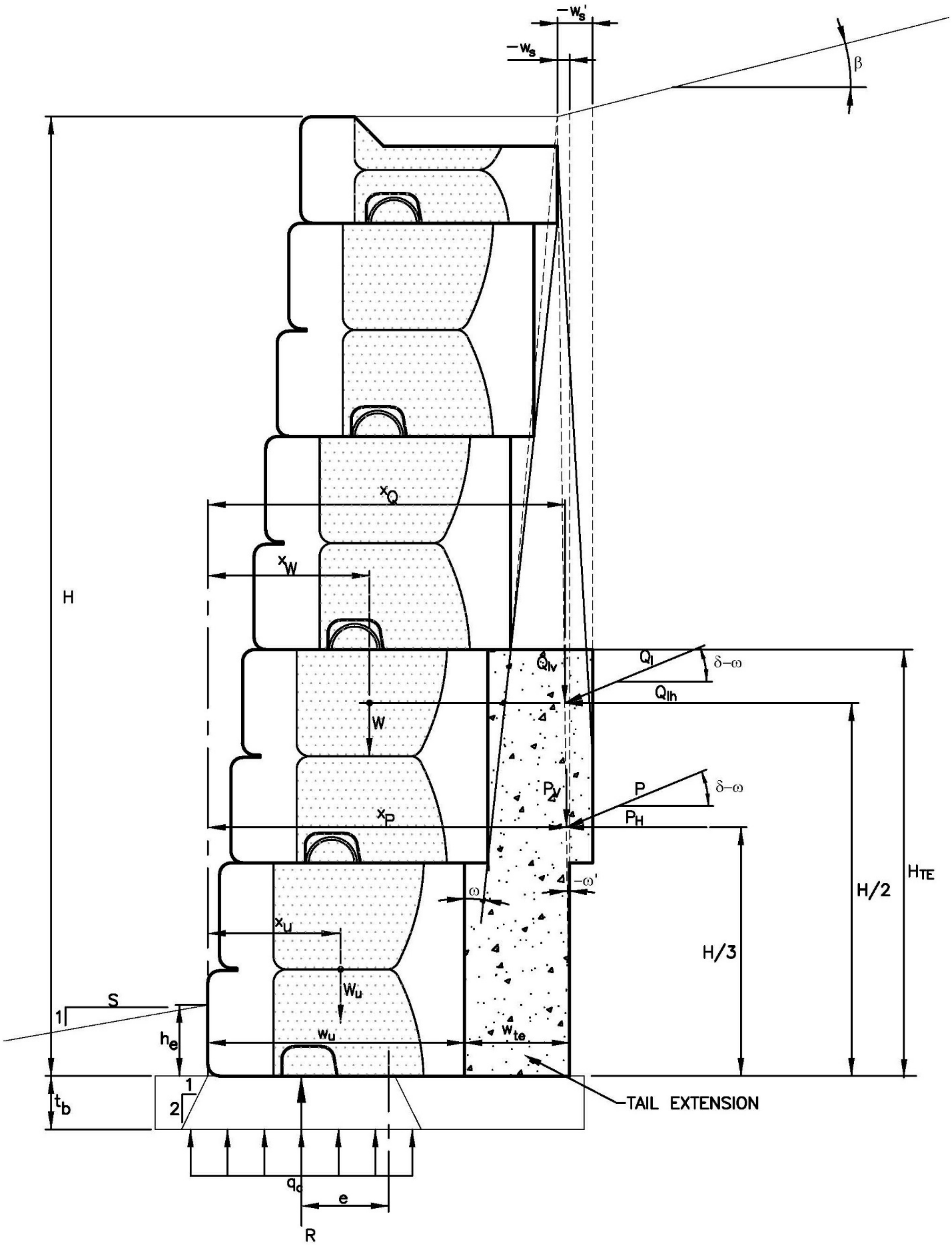
Typical gravity wall configuration, variables, and nomenclature:



Wall units that vary in width are referred to as "stepped" modules. Wider wall units are typically placed at the bottom of the wall. In addition to using wider precast units, the stability of a gravity wall can be improved by using cast-in-place tail extensions to increase the width of the units. The width of the CIP extension is not limited, but it is recommend that the height be at least 2 times the width to provide shear through the tail openings (unless connecting with reinforcing steel).

Wall batter

In common applications, the block units are installed in a battered configuration. In a typical batter, the 24 SF, 24-62, 24-86, and 24-ME units are 36 inches (914 mm) high and the next block atop a 24 SF block will batter back 4 inches (102 mm). The 6 SF and 6-28 units are 18 inches (457 mm) tall, and the next block atop a 6 SF block will batter 2 inches (51 mm). These blocks may be interchanged within a wall stack, but the batter is determined by the height of the unit below.

4 in. setback per 24 SF block (36 in. tall) — 102 mm setback per 24SF block (914 mm tall)

2 in. setback per 6 SF block (18 in. tall) — 51 mm setback per 6SF block (457 mm tall)

The face batter is calculated as:

$\omega = \arctan(4/36) = 6.34°$ — $\omega = \arctan(102/914) = 6.34°$

or $\omega = \arctan(2/18) = 6.34°$ — $\omega = \arctan(51/457) = 6.34°$

In some applications, the units may be installed with no batter to create a vertical face, $\omega = 0°$

For uniform modules, the batter of the back face matches the batter of the front face. For stepped modules, the batter is recalculated along the back of the wall from the rear of the bottom unit to the rear of the top of the wall. Use ω' in Coulomb equation and earth pressure component calculations. To calculate ω' 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 lower unit including any tail extension. 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:

$$\omega' = \arctan(w_s / H_w)$$

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 (225 mm) 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:

$$h_e = H'/(20*S/6)$$

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

A minimum embedment of 6 to 9 inches (150 to 225 mm) is recommended for private projects. A minimum embedment of 20 inches (500 mm) or more may be required for roadway applications

Weight of Wall

The weight of the wall includes the contributions of the blocks, the aggregate unit fill, the tail extension, and the soil wedge atop extended modules or tail extension

The weight of the tail extension is calculated:

$W_{te} = (w_{te} * H_{te}) * 145$ pcf (22.8 kN/m^3) (typical unit weight for concrete)

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

The angle of the batter (from vertical) of the soil wedge above the tail extension, ω_s, is calculated:

$\omega_s = \arctan(-w_s'/H_{wedge})$

The weight of soil in the wedge above the tail extension is calculated for the trapezoidal area of the wedge that lies behind each block

h_s = height of the soil trapezoid behind the block (may differ from height of the block)

w_u = width of the block

h_1 = dist. from the top of wall to top of the soil trapezoid behind the block

h_2 = dist. from the top of wall to bottom of the soil trapezoid behind the block

s = dist. from the face of wall to face of the block

s_u = dist. from the face of wall to back of the block = $s + w_u$

s_T = dist. from the face of wall to the back of top-most block of wall

b_1 = length of top side of trapezoid of soil behind block = $h_1 * \tan(\omega_s)+(s_T - s_u)$

b_2 = length of bottom side of trapezoid of soil behind block = $h_2 * \tan(\omega_s)+(s_T - s_u)$

The weight of the soil wedge above the tail extension behind each block, W_s, is calculated as the trapezoidal area multiplied by the lesser of the unit weight of the retained soil or the unit fill:

$W_s = [h_s * (b_1+b_2)/2] *$ (min of γ_{ret} or γ_u)

The center of gravity of the trapezoidal wedge behind each block, measured from the face of the wall at the bottom course, is calculated:

$x_s = [(b_1*b_2+(b_2^2-2*b_1*b_2+b_1^2)/3)/(b_1+b_2)]+ s + w_u$

$y_s = [h_s/3*(2b_1+b_2)/(b_1+b_2)]+ H - h_2$

W_s is treated as aggregate infill subject to 80% limitations for overturning calculations (conservative)

Static Forces

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

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

As an alternate, a trial wedge technique may be used to determine the earth pressure forces acting on the modular wall.

Earth Load Components (see AASHTO 5.5.2-1)

Vertical forces:

$P_v = 0.5\ K_a\ \gamma\ H^2 * \sin(\delta - \omega')$

$Q_{lv} = K_a\ Q\ H * \sin(\delta - \omega')$ where Q is the effective surcharge in psf (kPa)

Horizontal forces:

$P_h = 0.5\ K_a\ \gamma\ H^2 * \cos(\delta - \omega')$

$Q_{lh} = K_a\ Q\ H * \cos(\delta - \omega')$ where Q is the effective surcharge in psf (kPa)

Resultants of earth load components:

$y_P = H/3$

$x_P = (H/3) * \tan(\omega') + w_u$

$y_{Ql} = H/2$

$x_{Ql} = (H/2) * \tan(\omega') + w_u$

where w_u is the width of the bottom unit, including any tail extension (w_{te})

Weight Components

Vertical forces:

W_b – Weight of wall units

W_{te} – Weight of concrete tail extension, if used

W_a – Weight of infill aggregate (use 80% aggregate weight for overturning)

W_s – Weight of soil atop tail extension (use 80% aggregate weight for overturning)

$W_b = \sum(W_{b1} + W_{b2} + \cdots + W_{bn})$

$W_{te} = \sum(W_{te1} + W_{te2} + \cdots + W_{te})$

$W_a = \sum(W_{a1} + W_{a2} + \cdots + W_{an})$

$W_s = \sum(W_{s1} + W_{s2} + \cdots + W_{sn})$

Resultants of weight components:

The center of mass of the stack of blocks is calculated as:

$x_b = \sum(W_{b1}*x_{b1} + W_{b2}*x_{b2} + \cdots + W_{bn}*x_{bn}) / \sum(W_{b1} + W_{b2} + \cdots + W_{bn})$

$y_b = \sum(W_{b1}*y_{b1} + W_{b2}*y_{b2} + \cdots + W_{bn}*y_{bn}) / \sum(W_{b1} + W_{b2} + \cdots + W_{bn})$

The center of mass of the aggregate fill is:

$x_a = \sum(W_{a1}*x_{a1} + W_{a2}*x_{a2} + \cdots + W_{an}*x_{an}) / \sum(W_{a1} + W_{a2} + \cdots + W_{an})$

$y_a = \sum(W_{a1}*y_{a1} + W_{a2}*y_{a2} + \cdots + W_{an}*y_{an}) / \sum(W_{a1} + W_{a2} + \cdots + W_{an})$

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

$x_s = \sum(W_{s1}*x_{s1} + W_{s2}*x_{s2} + \cdots + W_{sn}*x_{sn}) / \sum(W_{s1} + W_{s2} + \cdots + W_{sn})$

$y_s = \sum(W_{s1}*y_{s1} + W_{s2}*y_{s2} + \cdots + W_{sn}*y_{sn}) / \sum(W_{s1} + W_{s2} + \cdots + W_{sn})$

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

$x_{te} = \sum(W_{te1}*x_{te1} + W_{te2}*x_{te2} + \cdots + W_{ten}*x_{ten}) / \sum(W_{te1} + W_{te2} + \cdots + W_{te})$

$y_{te} = \sum(W_{te1}*y_{te1} + W_{te2}*y_{te2} + \cdots + W_{ten}*y_{ten}) / \sum(W_{te1} + W_{te2} + \cdots + W_{te})$

The overall adjusted center of mass of the blocks and tail extension:

$x_{b+te} = (W_b*x_b + W_{te}*x_{te}) / (W_b + W_{te})$

$y_{b+te} = (W_b*y_b + W_{te}*y_{te}) / (W_b + W_{te})$

The overall adjusted center of mass of the aggregate and the soil above the tail is:

$x_{a+s} = (W_a*x_a + W_s*x_s) / (W_a + W_s)$

$y_{a+s} = (W_a*y_a + W_s*y_s) / (W_a + W_s)$

Seismic Loads

Seismic components of force are calculated according to the procedures in FHWA 4.2h.

The maximum acceleration A_m = (1.45 - A)*A where A is the peak horizontal ground acceleration.

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

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

where $\xi = \arctan[k_h/(1 - k_v)]$

The trial wedge technique is recommended in high seismicity regions to determine the dynamic thrust forces acting on the modular wall.

<u>Seismic Earth load components</u>

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:

$$k_h = 1.66 * A_m * [A_m/(d*C)]^{0.25}$$

d is the maximum horizontal displacement, and is typically set at 2 inches (50 mm) as conservative. Note that this equation has embedded units of mm, and C is a conversion factor (25.4 when d is in units of inches, and 1 when d is in units of mm).

$$A_m = (1.45\text{-}PGA)*PGA$$

Note that when PGA is not provided, it can be calculated from seismic response values provided in the International Building Code. IBC 1802.2.7 allows for PGA to be taken as $S_{DS}/2.5$. Following IBC Eq. 16-37 and 16-39:

$$PGA = 0.267 * S_s F_a$$

The horizontal inertial force P_{ir} is calculated as follows:

$$P_{ir} = (W_b+W_{te}+W_a+W_s)*k_h$$

The seismic thrust is calculated as follows:

$$\Delta P_{ae} = 0.5 * \gamma * H^2 * (K_{ae} - K_a)$$

$$\Delta P_{aeh} = 0.5 * \gamma * H^2 * (K_{ae} - K_a) * \cos(\delta - \omega')$$

$$\Delta P_{aev} = 0.5 * \gamma * H^2 * (K_{ae} - K_a) * \sin(\delta - \omega')$$

<u>Resultants of Seismic Earth load components</u>

In overturning analysis, the inertial force is applied at the vertical center of gravity of the wall, while the seismic thrust is applied at 60% of the wall height.

$y_{Pae} = 0.6*H$

$x_{Pae} = 0.6*H*\tan(\omega') + w_u$

$y_{Pir} = (W_b*y_b + W_{te}*y_{te} + W_a*y_a + W_s*y_s) / (W_b + W_{te} + W_a + W_s)$

Since the inertial and thrust forces are generally not in sync and do not peak simultaneously, the full inertial force is applied along with 50% of the seismic thrust (FHWA 4.2h).

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

Base Friction

Friction across 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:

$R_{s(foundation\ soil)} = (W_b + W_{te} + W_a + W_s + P_v + w_u*t_b*\gamma_b) \tan\phi + B_w*c$

where ϕ represents foundation soil friction angle, B_w is base width (bottom block width including any tail extension plus ½H:1V distribution through base), and c represents foundation soil cohesion. The weight of the base is included in the wall weight.

For block to base material sliding, use:

$R_{s(footing)} = \mu_b (W_b + W_{te} + W_a + W_s + P_v)$

where μ_b represents a composite coefficient of friction for the base

The composite friction coefficient is calculated using contributory areas. The base of a Stone Strong unit consists of a percentage of open void space to be filled with aggregate and a percentage of concrete. These percentages are calculated as follows:

$\%_{void} = V_{void}/(V_{void}+V_{concrete})$

$\%_{concrete} = V_{concrete}/(V_{void}+V_{concrete})$

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:

$\mu_b = (\%_{void}*w_{u(bottom)}*\mu_{p\text{ - unit fill/base}} + \%_{concrete}*w_{u(bottom)}*\mu_{p\text{ - block/base}} + w_{te}*\mu_{p\text{ - extension/base}})/(w_{u(bottom)} + w_{te})$

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Partial friction coefficients can be interpreted from the following table:

	Coefficient of Friction
Block to Aggregate Base formed precast surface on compacted aggregate surface (includes Mass Extender)	0.8*tan ϕ_b
Unit Fill to Aggregate Base screened aggregate (loose to moderate relative density - dumped) on compacted aggregate surface	lower tan ϕ_b or tan ϕ_u
Block to Concrete Base formed precast surface on floated concrete surface (includes Mass Extender)	0.60
Unit Fill Aggregate to Concrete Base screened aggregate (loose to moderate relative density - dumped) on floated concrete surface	0.8*tan ϕ_u
Concrete Tail Extension to Aggregate Base cast in place concrete on aggregate surface	tan ϕ_b
Concrete Tail Extension to Concrete Base cast in place concrete on floated concrete surface	0.75
Concrete Tail Extension Directly on Foundation Soil (Sand) cast in place concrete on granular soil	tan ϕ_f
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 and infill aggregates:

	Friction Angle (degrees)		
	Well Graded, Aggregate, Densely Compacted	Screened Aggregate, Compacted	Screened Aggregate, Loose to Moderate Relative Density
Crushed Hard Aggregate >75% w/ 2 fractured faces, hard natural rock	42	40	36
Crushed Aggregate >75% w/ 2 fractured faces, medium natural rock or recycled concrete	40	38	35
Cracked Gravel >90% w/ 1 fractured face	36	35	32
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.			

Table of Forces & Moments

	Force (lb) or (kN)	Arm (ft) or (m)	Moment about toe (lb*ft) or (kN *m)
Vertical Forces			
weight of blocks	$W_b + W_{te}$	x_{b+te}	$(W_b + W_{te})* x_{b+te}$
weight of agg. & soil over tail	$W_a + W_s$	x_{a+s}	$(W_a + W_s) * x_{a+s}$
modified weight of a & s (80%)	$0.8*(W_a + W_s)$	x_{a+s}	$0.8*(W_a + W_s) * x_{a+s}$
earth pressure	P_v	x_P	P_v*x_P
seismic thrust	$P_{aev}/2$	x_{Pae}	$P_{aeh}/2 * x_{Pae}$
LL surcharge	Q_{lv}	x_{Ql}	$Q_{lv}*x_{Ql}$
Horizontal Forces			
earth pressure	P_h	y_{Ph}	P_h*y_{Ph}
seismic thrust	$\Delta P_{aeh}/2$	y_{Pae}	$\Delta P_{aeh}/2 * y_{Pae}$
inertial force	P_{ir}	y_{Pir}	$P_{ir} * y_{Pir}$
LL surcharge	Q_{lh}	y_{Ql}	$Q_{lh}*y_{Ql}$

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.

M'_V	Σ moments from vertical forces (using 80% W_s & W_a)
M_H	Σ moments from horizontal forces
FS	M'_V / M_H

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

check that FS > 1.5 with static earth pressure loads and surcharge loads

Safety factors are typically reduced 25% for seismic loading due to the extreme nature of these events. Surcharge loads are generally not applied concurrent with seismic loads.

check that FS > 1.13 with static earth pressure loads and seismic loads

Sliding

The minimum value for sliding resistance is calculated as follows:

F_H	Σ horizontal forces
F_V	Σ vertical forces (using 100% W_s & W_a)
$R_{s\ (footing)}$	$\mu_b F_V$
$R_{s\ (foundation\ soil)}$	$(F_V + t_b{*}w_b{*}\gamma_b){*}\tan(\phi) + B_w{*}c$
min R_s	smaller of $R_{s\ (footing)}$ or $R_{s\ (foundation\ soil)}$
FS	min R'_s / F_H

The safety factor for sliding should be greater than 1.5

check that FS > 1.5 with static earth pressure loads and surcharge loads

check that FS > 1.13 with static earth pressure loads and seismic loads

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Bearing/Eccentricity

B_f' is the equivalent bearing area. This is the base block width adjusted for eccentricity, and including a ½H: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)

F_V	Σ vertical forces (using 100% W_s & W_a)
weight of base	$t_b * \gamma_b$
M_v	Σ moments from vertical forces (using 100% W_s & W_a)
M_H	Σ moments from horizontal forces
e	$(w_u + w_{te})/2 - (M_V - M_H)/F_V$
B_f' (granular base)	$w_u + w_{te} + t_b - 2*e$
B_f' (concrete base)	$w_u + w_{te} + 2*t_b - 2*e$
contact pressure q_c	$F_V / B_f' + t_b*\gamma_b$
bearing resistance q_{ut}	$[c*N_c + (h_e + t_b)*\gamma_{found}*N_q+0.5*\gamma_{found}*B_f'*N_\gamma]$
FS	q_{ult} / q_c

The safety factor for bearing should be greater than 2

check that FS > 2.0 with static earth pressure loads and surcharge loads

check that FS > 1.5 with static earth pressure loads and seismic loads

Internal Analysis

Internal stability analysis is conducted for each section above the wall base. Since bearing conditions are addressed in the external stability analysis, only topping and shear failures are evaluated.

Toppling is evaluated similarly to external overturning analysis, except that the overturning point is set in 1 inch (25 mm) to account for face rounding.

$FS = M'_V / M_H$

check that FS > 1.5 with static earth pressure loads and surcharge loads

check that FS > 1.13 with static earth pressure loads and seismic loads

Shear, or sliding, resistance is calculated based on the interface shear test (see interaction test reports for complete test data)

$R_s = [S_i + (W + P_v + Q_{lv})^* \tan(35.2°)]$

where S_i = 362 lb/ft or 5.28 kN/m

$FS = R_s / F_H$

check that FS > 1.5 with static earth pressure loads and surcharge loads

check that FS > 1.13 with static earth pressure loads and seismic loads

At a minimum, internal stability should be evaluated at each change in block width (including any tail extension), at the base of any dual-face units, and for the top course(s) if a surcharge or lateral load is applied.

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EXAMPLE GRAVITY WALL CALCULATIONS

ALLOWABLE STRESS METHOD USING IBC SAFETY FACTORS

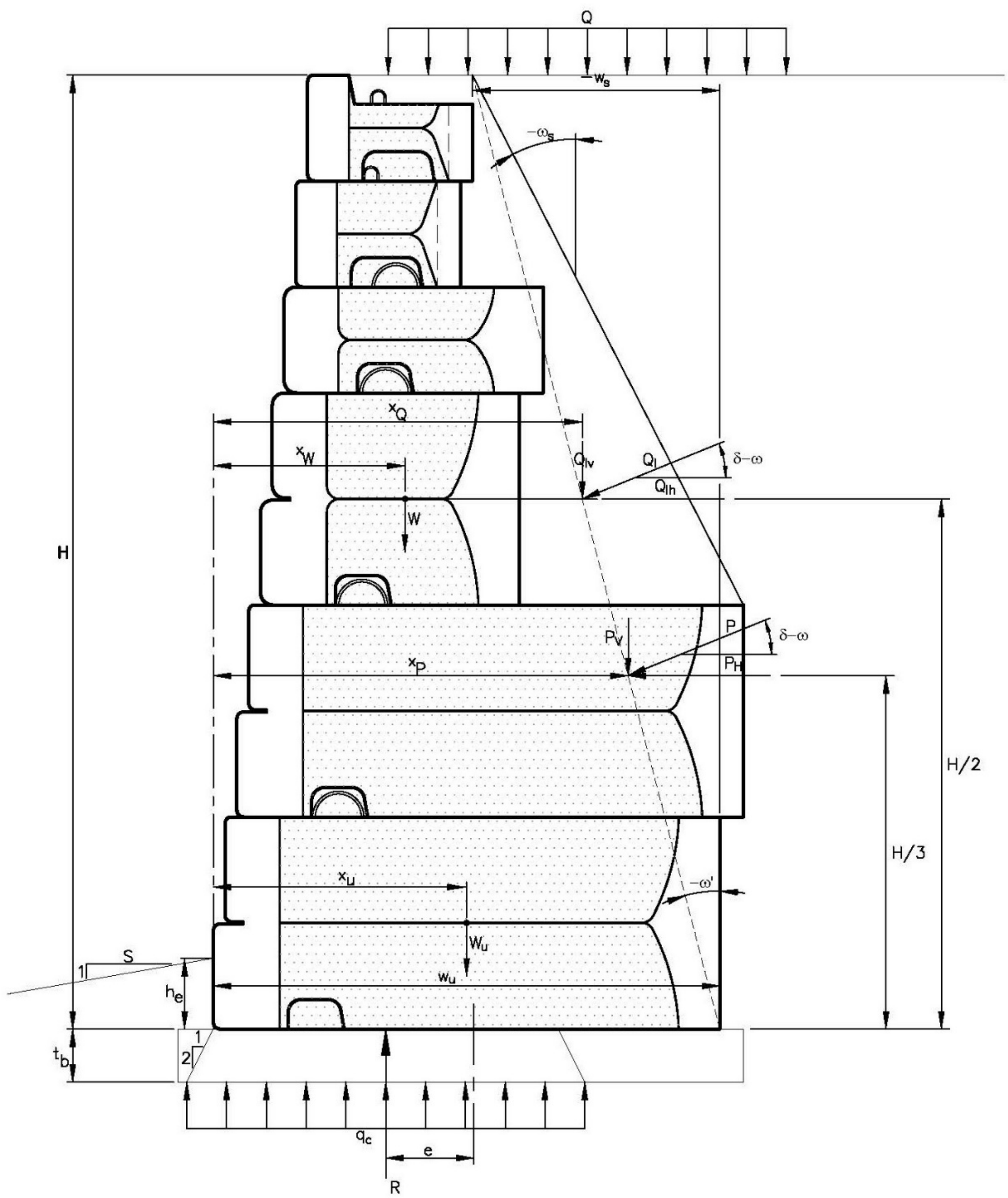
Example 1: 13.5 feet tall wall, level back slope, 150 psf parking surcharge

Retained Soil: sand with γ = 120 pcf and ϕ = 30 degrees

Foundation Soil: clay with γ = 125 pcf, ϕ = 26 degrees, and c' = 150 psf

Infill Aggregate: screened crushed aggregate with γ = 110 pcf and ϕ = 35 degrees

Base Aggregate: well graded crushed aggregate with γ = 125 pcf and ϕ = 40 degrees



Project	Project #	Date
ASD Example Calculations	20004.00	2/20/20

Wall Configuration (all weights per foot along length of wall)

External Stability Analysis

Modular Units			Setback (in)		Concrete (/ft.)		Unit Fill (/ft.)		Soil Wedge (/ft.)	
unit	w (in)	h (ft)	face	tail	W_b (lb)	x_b (in)	W_a (lb)	x_a (in)	W_s (lb)	x_s (in)
6-28	28.0	1.50	16.0	-42.0	238	28.8	183	30.0	63	47.1
6-28	28.0	1.50	14.0	-44.0	238	26.8	183	28.0	217	50.1
6	44.0	1.50	12.0	-30.0	375	33.0	301	35.5	151	61.8
24	44.0	3.00	8.0	-34.0	750	29.2	594	32.8	792	66.9
24-86	86.0	3.00	4.0	4.0	950	44.0	1,621	49.1	0	0.0
24-86	86.0	3.00	0.0	0.0	950	40.0	1,621	45.1	0	0.0

Weight and Center of Gravity of Wall Components

W_b = 950+950+750+375+238+238 = 3,500 lb/ft

W_a = 1,621+1,621+594+301+183+183 = 4,503 lb/ft

W_s = 792+151+217+63 = 1,224 lb/ft

Total Wall Weight = 3,500+4,503+1,224 = 9,227 lb/ft

x_b = (950*40.0+950*44.0+750*29.2+375*33.0+238*26.8+238*28.8) / 3,500 =36.4 in

x_a = (1,621*45.1+1,621*49.1+594*32.8+301*35.5+183*28.0+183*30.0) / 4,503 = 43.0 in

x_s = (792*66.9+151*61.8+217*50.1+62*47.1) / 1,224 = 62.3 in

Earth Pressure Components

ω' = arctan(-42/12/13.5) = -14.53° δ = 0.75*30 = 22.5°

$$K_a = \frac{\cos^2(30-14.53)}{\cos^2(-14.53)\cos(-14.53-22.5)\left[1+\sqrt{\frac{\sin(30+22.5)\sin(30-0)}{\cos(-14.53-22.5)\cos(-14.53+0)}}\right]^2}$$

K_a = 0.421

P_h = 0.5*0.421*120*$(12.0)^2$*cos(22.5 +14.53) = 3,679 lb

P_v = 0.5*0.421*120*$(12.0)^2$*sin(22.5 +14.53) = 2,776 lb

Q_{lh} = 0.421*150*12.0*cos(22.5 +14.53) = 681 lb

Q_{lv} = 0.421*150*12.0*sin(22.5 +14.53) = 514 lb

x_P = (13.5/3)*tan(-14.53)+86/12 = 6.00 ft y_P = 13.5/3 = 4.50 ft

x_{Ql} = (13.5/2)*tan(-14.53)+86/12 =5.42 ft y_{Ql} = 13.5/2 = 6.75 ft

Project	Project #	Date
ASD Example Calculations	20004.00	2/20/20

Base Friction

Use the smaller sliding resistance, R, of the following:

Determine composite friction coefficient across base:

$\%_{void}$ = (1,621/110) / (950/145+1,621/110) = 0.6922

$\%_{concrete}$ = (950/145) / (950/145+1,621/110) = 0.3078

μ_b = 0.6922*tan(35)+0.3078*0.8*tan(40) = 0.691

$R_{footing}$ = 0.691*(9,227+2,776+514) = 8,653 lb/ft

R_{soil} = (9,227+2,776+514+(86/12*9/12)*125)*tan(26)+ ((86+9)/12)*150
= 7,620 lb/ft

Factors of Safety

Overturning

FS = [3,500*(36.4/12)+0.8*4,503*(43.0/12)+0.8*1,224*(62.3/12)+2,776*6.00+514*5.42] / (3,679*(4.50)+681*6.75) = 2.27 > 1.5 **OK!!**

Sliding

FS = 7,620/(3,679+681) = 1.75 > 1.5 **OK!!**

Bearing

e = (86/12)/2-[(3,500*(36.4/12)+4,503*(43.0/12)+1,224*(62.3/12)+2,776*6.00+514*5.42) - (3,679*4.50+681*6.75)] / (3,500+4,503+1,224+2,776+514)) = 1.08 ft

B_f' = (86+9)/12 – 2*1.08 = 5.76 ft.

q_c = (9,227+2,776+514) / 5.76 + 9/12*125 = 2,266 psf

Bearing Factors (Vesic):

N_c = 22.25 N_q = 11.85 N_γ = 12.54

q_{ult} = 150*22.25+((9+9)/12)*125*11.85+0.5*125*5.76*12.54 = 10,076 psf

FS = 10,076/2,266 = 4.45 > 2.0 **OK!!**

Project	Project #	Date
ASD Example Calculations	20004.00	2/20/20

Internal Stability Analysis

Internal stability should be checked at each change in block width, at all dual-face unit, and at the top unit at a minimum. The following is taken at the first change from 24-86 to 24SF. Internal stability of the block stack above this interface is calculated as follows:

Wall Configuration (all weights per foot along length of wall)

Modular Units			Setback (in)		Concrete (/ft.)		Unit Fill (/ft.)		Soil Wedge (/ft.)	
unit	w (in)	h (ft)	face	tail	W_b (lb)	x_b (in)	W_a (lb)	x_a (in)	W_s (lb)	x_s (in)
6-28	28.0	1.50	8.0	-8.0	238	19.8	183	21.0	41	37.0
6-28	28.0	1.50	6.0	-10.0	238	17.8	183	19.0	151	38.6
6	44.0	1.50	4.0	4.0	375	24.0	301	26.5	0	0.0
24	44.0	3.00	0.0	0.0	750	20.2	594	23.8	0	0.0

Weight and Center of Gravity of Wall Components

W_b = 750+375+238+238 = 1,600 lb/ft

W_a = 594+301+183+183 = 1,261 lb/ft

W_s = 151+41 = 193 lb/ft

Total Wall Weight = 1,600+1,261+193 = 3,054 lb/ft

x_b = (750*20.2+375*24.0+238*17.8+238*19.8)/1,600 = 20.7 in

x_a = (594*23.8+301*26.5+183*19.0+183*21.0)/1,261 = 23.3 in

x_s = (151*38.6+41*37.0)/193 = 38.3 in

Earth Pressure Components

ω' = arctan(-8/12/7.5) = -5.08° δ = 0.75*30 = 22.5°

$$K_a=\frac{\cos^2(30+-5.08)}{\cos^2(-5.08)\cos(-5.08-22.5)\left[1+\sqrt{\frac{\sin(30+22.5)\sin(30-0)}{\cos(-5.08-22.5)\cos(-5.08+0)}}\right]^2}$$

K_a = 0.335

P_h = 0.5*0.335*120*(7.5)^2*cos(22.5 +5.08) = 1,003 lb

P_v = 0.5*0.335*120*(7.5)^2*sin(22.5 +5.08) = 524 lb

Q_{lh} = 0.335*150*7.5*cos(22.5 +5.08) = 334 lb

Q_{lv} = 0.335*150*7.5*sin(22.5 +5.08) = 175 lb

x_P = (7.5/3)*tan(-5.08)+43/12 = 3.36 ft y_P = 7.5/3 = 2.5 ft

x_{Ql} = (7.5/2)*tan(-5.08)+43/12 = 2.61 ft y_{Ql} = 7.5/2 = 3.75 ft

Project	Project #	Date
ASD Example Calculations	20004.00	2/20/20

Interface Shear

R_s = 362+ (3,054+524+175)*tan(35.2) = 3,009

Factors of Safety

Overturning/Toppling

FS = [1,600*(20.7/12)+0.8*1,261*(23.3/12)+0.8*193*(38.3/12)+524*3.36+175*2.61] / (1,003*2.50+334*3.75) = 2.00 > 1.5 **OK!!**

Sliding/Internal Shear

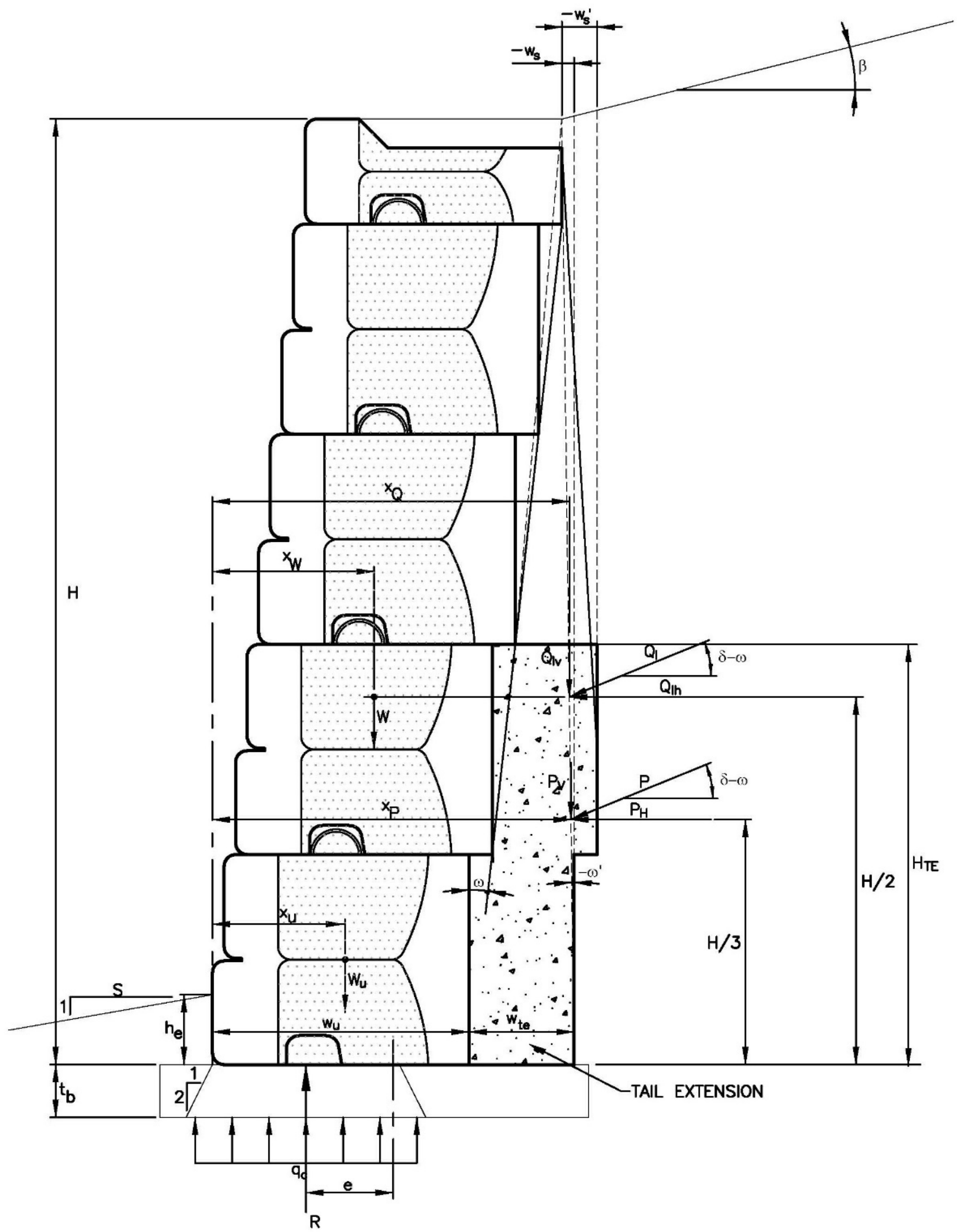
FS = 3,009/(1,003+334) = 2.25 > 1.5 **OK!!**

All other interfaces **OK!!**

Project	Project #	Date
ASD Example Calculations	20004.00	2/20/20

<u>Example 2:</u> 13.5 feet tall wall, 3H:1V back slope, CIP tail extension

Retained Soil: sand with γ = 120 pcf and ϕ = 30 degrees
Foundation Soil: clay with γ = 125 pcf, ϕ = 26 degrees, and c' = 150 psf
Infill Aggregate: screened crushed aggregate with γ = 110 pcf and ϕ = 35 degrees
Base Aggregate: well graded crushed aggregate with γ = 125 pcf and ϕ = 40 degrees
Tail Extension: 30 inches wide by 72 inches tall, placed on aggregate base



Project ASD Example Calculations	Project # 20004.00	Date 2/20/20

Wall Configuration including CIP tail extension (all weights per foot along length of wall)

Modular Units			Setback (in)		Concrete (/ft.)		Unit Fill (/ft.)		Soil Wedge (/ft.)	
unit	w (in)	h (ft)	face	tail	W_b (lb)	x_b (in)	W_a (lb)	x_a (in)	W_s (lb)	x_s (in)
6	44.0	1.50	16.0	-14.0	375	37.0	301	39.5	25	61.2
24	44.0	3.00	12.0	-18.0	750	33.2	594	36.8	308	61.8
24	44.0	3.00	8.0	-22.0	750	29.2	594	32.8	616	63.3
24	74.0	3.00	4.0	4.0	1,838	47.6	594	28.8	0	0.0
24	74.0	3.00	0.0	0.0	1,838	43.6	594	24.8	0	0.0

External Stability Analysis

Weight and Center of Gravity of Wall Components

$W_b + W_{te}$ = 750+2.5*3.0*145+750+2.5*3.0*145+750++750+375 = 5,550 lb/ft

W_a = 594+594+594+594+301 = 2,678 lb/ft

W_s = 616+308+25 = 949 lb/ft

Total Wall Weight = 5,500+2,678+949 = 9,176 lb/ft

x_{b+te} = (1,838*43.6+1,838*47.6+750*29.2+750*33.2+375*37.0) / 5,550 = 41.1 in

x_a = (594*24.8+594*28.8+594*32.8+594*36.8+301*39.5) / 2,678 = 31.8 in

x_s = (616*63.3+308*61.8+25*61.2) / 949 = 62.8 in

Earth Pressure Components

ω' = arctan(-14/12/13.5) = -4.94°

δ = 0.75*30 = 22.5°

$$K_a = \frac{\cos^2(30+4.94)}{\cos^2(-4.94)\cos(-4.94-22.5)\left[1+\sqrt{\frac{\sin(30+22.5)\sin(30-18.4)}{\cos(-4.94-22.5)\cos(-4.94+18.4)}}\right]^2}$$

K_a = 0.456

$P_h = 0.5*(.456)*120*(13.5)^2*\cos(22.5+4.94)$ = 4,425 lb

$P_v = 0.5*(.456)*120*(13.5)^2*\sin(22.5+4.94)$ = 2,298 lb

x_P = (13.5/3)*tan(-4.94)+(74/12) = 5.78 ft

y_P = (13.5/3) = 4.5

Project	Project #	Date
ASD Example Calculations	20004.00	2/20/20

Base Friction

Use the smaller sliding resistance, R, of the following:

Determine composite friction coefficient across base:

$\%_{void}$ = (1,621/110) / (750/145+2.0*3.0+1,621/110) = 0.5688

$\%_{prescast}$ = (750/145) / (750/145+2.0*3.0+1,621/110) = 0.1996

$\%_{CIP}$ = (2.0*3.0) / (750/145+2.0*3.0+1,621/110) = 0.2316

μ_b = 0.5688*tan(35)+0.1996*0.8*tan(40)+0.2316*tan(40) = 0.606

$R_{footing}$ = 0.606*(9,176+2,298) = 6,953 lb/ft

R_{soil} = (9,176+2,289+(74/12*9/12)*125)*tan(26)+((74+9)/12)*150 = 6,916 lb/ft

Factors of Safety

Overturning

FS = [(5,550*(41.1/12)+0.8*2,678*(31.8/12)+0.8*949*(62.8/12)+2,298*5.78] / (4,425*4.5)
= 2.11 > 1.5 **OK!!**

Sliding

FS = 6,916/4,425 = 1.56 > 1.5 **OK!!**

Bearing

e = (74/12)/2-[5,550*(41.1/12)+2,678*(31.8/12)+949*(62.8/12)+2,298*5.78)-
(4,425*4.5)]/(5,550+2,678+949+2,298) = 0.95

B_f' = (74+9)/12-2*0.95 = 5.01 ft.

q_c = ((5,550+2,678+949+2,298) / 5.01)+(9/12)*125 = 2,385 psf

Bearing Factors (Vesic):

N_c = 22.25 N_q = 11.85 N_γ = 12.54

q_{ult} = 150*22.25+((9+9)/12)125*11.85+0.5*125*5.01*12.54 = 9,485 psf

FS = 9,485/2,385 = 3.98 > 2.0 **OK!!**

Project	Project #	Date
ASD Example Calculations	20004.00	2/20/20

Internal Stability Analysis

Internal stability should be checked at each change in block width, at all dual-face unit, and at the top unit at a minimum. The following is taken at the change from 24SF unit with tail extension to a 24SF unit. Internal stability of the block stack above this interface is calculated as follows:

Wall Configuration (all weights per foot along length of wall)

Modular Units			Setback (in)		Concrete (/ft.)		Unit Fill (/ft.)	
unit	w (in)	h (ft)	face	tail	W_b (lb)	x_b (in)	W_a (lb)	x_a (in)
6	44.0	1.50	8.0	8.0	375	28.0	301	30.5
24	44.0	3.00	4.0	4.0	750	24.2	594	27.8
24	44.0	3.00	0.0	0.0	750	20.2	594	23.8

Weight and Center of Gravity of Wall Components

W_b = 750+750+375 = 1,875 lb/ft
W_a = 594+594+301 = 1,489 lb/ft
Total Wall Weight = 1,875+1,489 = 3,364 lb/ft

x_b= (750*20.2+750*24.2+375*28.0) / 1,875 =23.4 in
x_a = (594*248+594*28.8+296*35.5) / 1,489 = 26.8 in

Earth Pressure Components

ω' = 6.34° δ = 0.5*30 = 15.0°

$$K_a = \frac{\cos^2(30+6.34)}{\cos^2(6.34)\cos(6.34-15.0)\left[1+\sqrt{\frac{\sin(30+15.0)\sin(30-18.4)}{\cos(6.34-15.0)\cos(6.34+18.4)}}\right]^2}$$

K_a = 0.340
P_h = 0.5*0.340*120*(7.5)^2*cos(15-6.34) = 1,135 lb
P_v = 0.5*0.340*120*(7.5)^2*sin(15-6.34) = 173 lb

x_P = (7.5/3)*tan(6.34)+(44/12) = 3.94 ft y_P = 7.5/3 = 2.5 ft

Interface Shear

R_s = 362+(3,364+173)*tan(35.2) = 2,857 lb

Factors of Safety

Overturning/Toppling

FS = [1,875*(23.4/12)+0.8*1,489*(26.7/12)+173*3.94] / (1,135*2.5) = 2.46 > 1.5 **OK!!**

Sliding/Internal Shear

FS = 2,857/1,135 = 2.52 > 1.5 **OK!!**

All other interfaces **OK!!**

STONE STRONG GRAVITY CALCULATIONS - ver 6.0

Project Name:	Example Calculations
Location:	
Job#:	20004.00
Section:	Example #1
Calc by:	D Thiele

2/20/20 15:49

Notes
13.5 tall wall with extended precast units, battered face
level back slope, 150 psf parking lot surcharge
External Stability

Wall Configuration

unit	w (in)	h (ft)	setback (in) face	setback (in) tail	modular units Wb (lb)	modular units xb (in)	unit fill Wa (lb)	unit fill xa (in)	soil wedge Ws (lb)	soil wedge xs (in)	CIP Extension we (in)	CIP Extension h_t	Internal Stability FS Topple	Internal Stability FS Shear	Seismic Internal FS Topple	Seismic Internal FS Shear	
6-28	28.0	1.50	16.0	-42.0	238	28.8	183	30.0	63	47.1			6.60	6.47			OK!
6-28	28.0	1.50	14.0	-44.0	238	26.8	183	28.0	217	50.1			3.01	3.87			OK!
6	44.0	1.50	12.0	-30.0	375	33.0	301	35.5	151	61.8			3.53	3.02			OK!
24	44.0	3.00	8.0	-34.0	750	29.2	594	32.8	792	66.9			2.00	2.25			OK!
24-86	86.0	3.00	4.0	4.0	950	44.0	1,621	49.1	0	0.0			2.98	2.38			OK!
24-86	86.0	3.00	0.0	0.0	950	40.0	1,621	45.1	0	0.0							
	86.0	13.50	16.0	-42.0	3,500	36.4	4,503	43.0	1,224	62.3	9,227						

External Stability OK!

backfill height 13.50 feet | ω= 6.34 deg | interface friction angle

exposed height 12.75 feet | ω'= -14.53 deg | δ 22.5 deg

Retained Soil
γ 120 pcf
ϕ 30 deg

Foundation Soil
γ 125 pcf
ϕ 26 deg
c' 150 psf

base embedment 9 in
base thickness 9 in
base material agg
toe slope H:1V slope

allowable bearing pressure (if specified) n/a psf (net)

Aggregate Unit Fill
γ 110 pcf

composite friction coefficient μ_b 0.69

STONE STRONG GRAVITY CALCULATIONS - ver 6.0

Project Name:	**Example Calculations**
Location:	
Job#:	**20004.00**
Section:	**Example #1**
Calc by:	**D Thiele**

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Seismic Load Ss [ ] G site class (A to E or 1) **D** Fpga 1.60 Fa 1.60 k_h 0.00

Backfill Slope & Surcharge

			rise in grade		LL surcharge		tier height	
length 1	30	feet (horizontal)		ft	150	psf		ft
length 2		feet (horizontal)		ft		psf		ft
length 3		feet (horizontal)		ft		psf		ft
length 4		feet (horizontal)		ft		psf		

effective slope H:1V slope β 0.0 deg avg q 150 psf

failure plane α 60.23 deg zone of influence 14.89 ft

Analysis

		Q_{lh} =	681 lb	ΔK_{AE} =	0.000	e=	1.08 ft
K_a =	0.421	Q_{lv} =	514 lb	P_{IR} =	0 lb	B_f' =	5.76 ft
P_h =	3,679 lb	R_s =	7,620 lb	ΔP_{AEh} =	0 lb	e_{eq}=	0.82 ft
P_v =	2,776 lb	q_{ult} =	10,076 psf	ΔP_{AEv} =	0 lb	$B'_{f\,eq}$ =	6.28 ft

Results

Overturning:	Desired FS = 1.5	**Actual FS = 2.27**	***OK!***
Sliding:	Desired FS = 1.5	**Actual FS = 1.75**	***OK!***
Bearing Capacity:	Desired FS = 2	**Actual FS = 4.45**	***OK!***
	q_{all} = 5,038 psf	q_c = 2,266 psf	

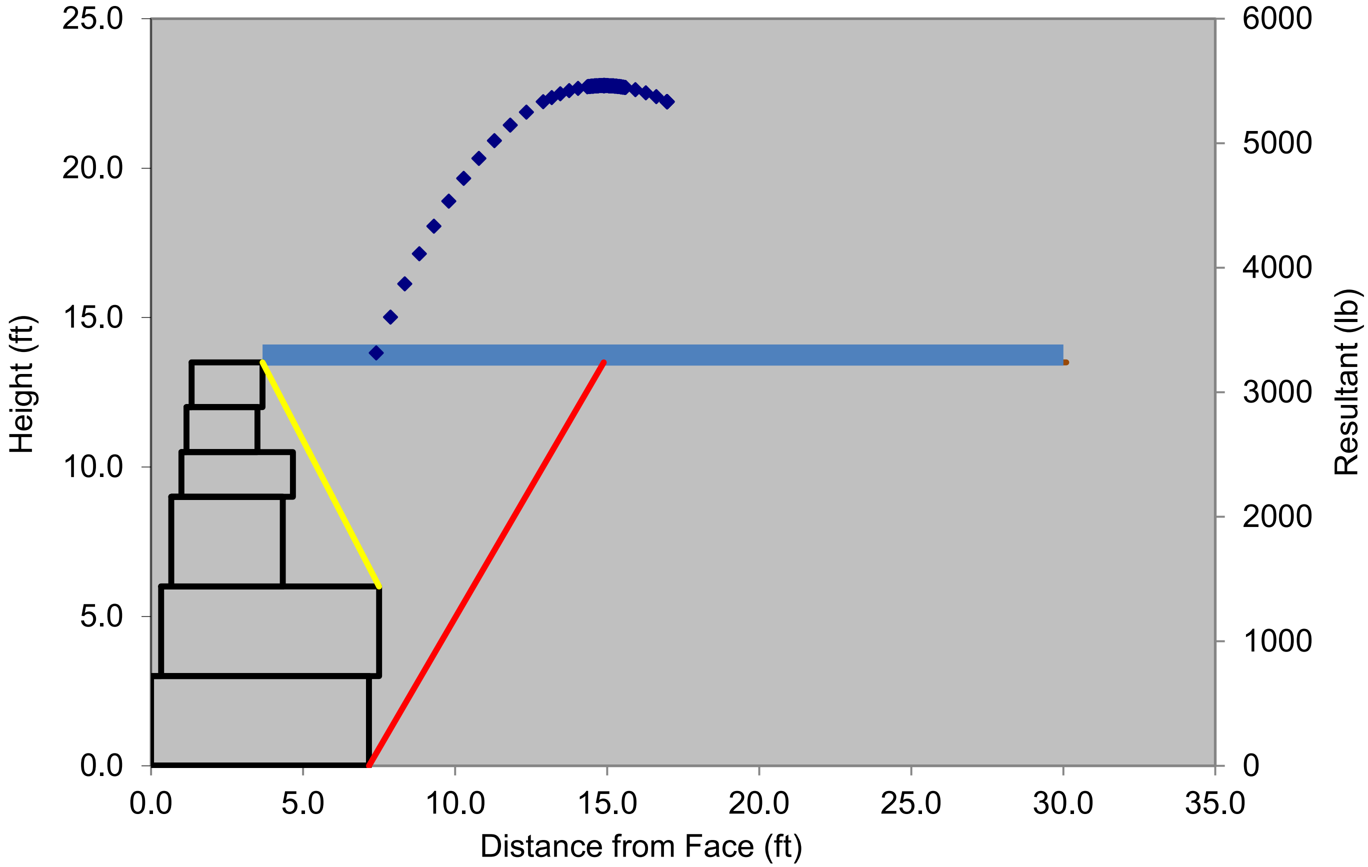
Internal Safety Factors

Desired FS = 1.5

Desired FS = 1.5

Project Name:	Example Calculations
Location:	
Job#:	20004.00
Section:	Example #1
Calc by:	D Thiele

Ground Surface & Trial Wedge Plot



STONE STRONG SYSTEMS

STONE STRONG GRAVITY CALCULATIONS - ver 6.0

Project Name: Example Calculations
Location:
Job#: 20004.00
Section: Example #1
Calc by: D Thiele

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Notes
13.5 tall wall with extended precast units, battered face
level back slope, 150 psf parking lot surcharge
Internal Stability

Wall Configuration

unit	w (in)	h (ft)	setback (in) face	setback (in) tail	modular units Wb (lb)	modular units xb (in)	unit fill Wa (lb)	unit fill xa (in)	soil wedge Ws (lb)	soil wedge xs (in)	CIP Extension we (in)	CIP Extension h_t	Internal Stability FS Topple	Internal Stability FS Shear	Seismic Internal FS Topple	Seismic Internal FS Shear	
6-28	28.0	1.50	8.0	-8.0	238	19.8	183	21.0	41	37.0			6.60	6.47			OK!
6-28	28.0	1.50	6.0	-10.0	238	17.8	183	19.0	151	38.6			3.01	3.87			OK!
6	44.0	1.50	4.0	4.0	375	24.0	301	26.5	0	0.0			3.53	3.02			OK!
24	44.0	3.00	0.0	0.0	750	20.2	594	23.8	0	0.0							
	44.0	7.50	8.0	-8.0	1,600	20.7	1,261	23.3	193	38.3	3,054						

Internal Stability OK!

backfill height 7.50 feet

ω= 6.34 deg
ω'= -5.08 deg

interface friction angle
δ 22.5 deg

Retained Soil

γ 120 pcf
ϕ 30 deg

Internal ONLY

Aggregate Unit Fill

γ 110 pcf

STONE STRONG GRAVITY CALCULATIONS - ver 6.0

Project Name: Example Calculations
Location:
Job#: 20004.00
Section: Example #1
Calc by: D Thiele

Seismic Load

Ss	G	site class (A to E or 1)	D	Fpga	1.60	Fa	1.60
k_h	0.00						

Backfill Slope & Surcharge

		rise in grade	LL surcharge	tier height
length 1	30 feet (horizontal)	ft	150 psf	ft
length 2	feet (horizontal)	ft	psf	ft
length 3	feet (horizontal)	ft	psf	ft
length 4	feet (horizontal)	ft	psf	

effective slope H:1V slope β 0.0 deg avg q 150 psf

failure plane α 57.31 deg zone of influence 8.48 ft

Analysis

		Q_{lh} =	334 lb	ΔK_{AE} =	0.000	e=	0.62 ft
K_a =	0.335	Q_{lv} =	175 lb	P_{IR} =	0 lb	B_f' =	3.09 ft
P_h =	1,003 lb	R_s =	3,009 lb	ΔP_{AEh} =	0 lb	e_{eq}=	0.37 ft
P_v =	524 lb	q_{ult} =	7,982 psf	ΔP_{AEv} =	0 lb	$B_f'{}_{eq}$ =	3.59 ft

Results

Internal Overturning:	Desired FS = 1.5	**Actual FS =**	**2.00**	***OK!***	
Interface Shear:	Desired FS = 1.5	**Actual FS =**	**2.25**	***OK!***	

Internal Safety Factors

Desired FS = 1.5

Desired FS = 1.5

STONE STRONG SYSTEMS

STONE STRONG GRAVITY CALCULATIONS - ver 6.0

Project Name:	Example Calculations
Location:	
Job#:	20004.00
Section:	Example #1
Calc by:	D Thiele

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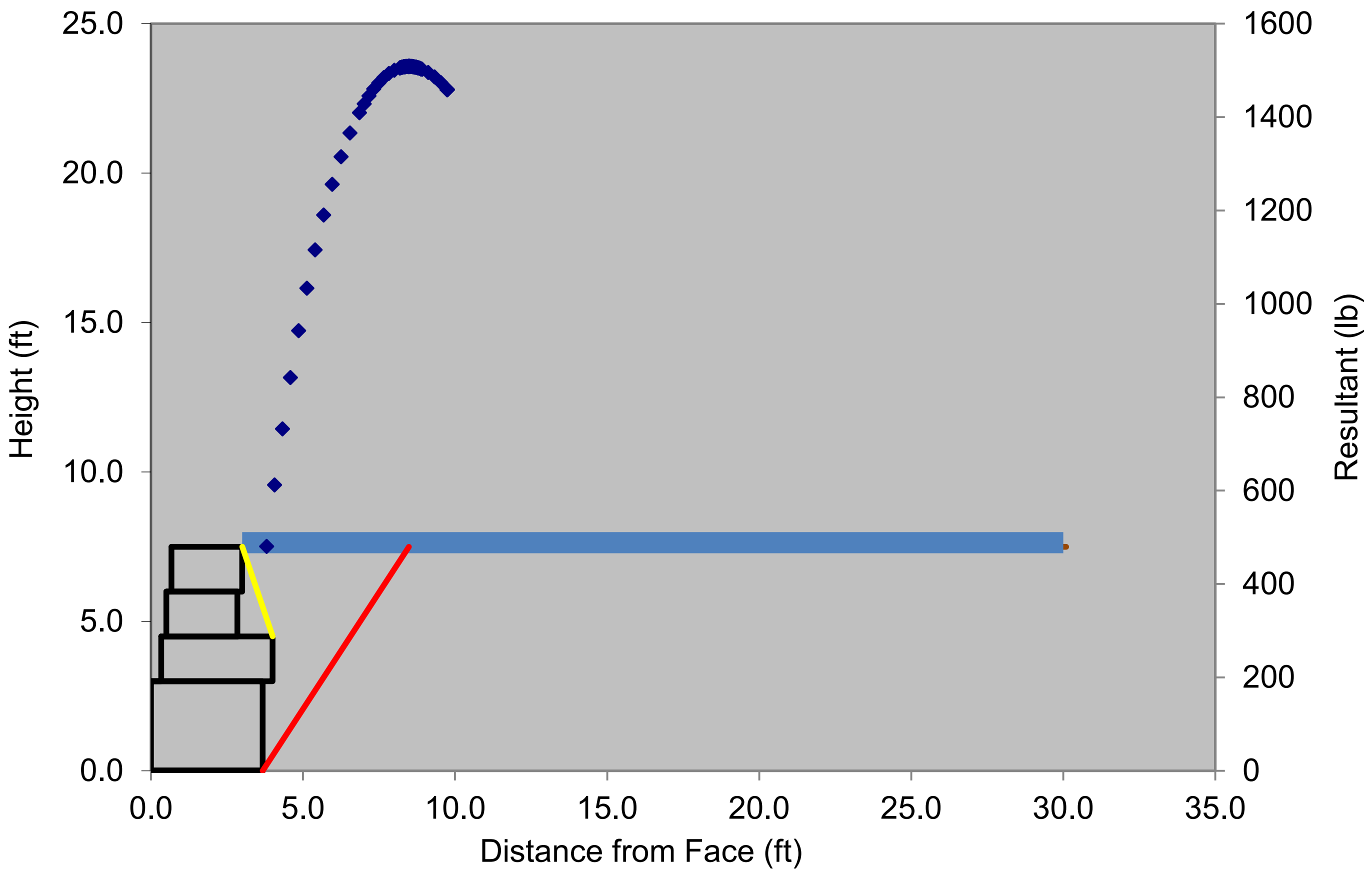
Ground Surface & Trial Wedge Plot



STONE STRONG GRAVITY CALCULATIONS - ver 6.0

Project Name: Example Calculations
Location:
Job#: 20004.00
Section: Example #2
Calc by: D Thiele

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Notes
13.5 tall wall with CIP tail extension, battered face
3H:1V backslope
External Stability

Wall Configuration

			setback (in)		modular units		unit fill		soil wedge		CIP Extension		Internal Stability FS		Seismic Internal FS		
unit	w (in)	h (ft)	face	tail	Wb (lb)	xb (in)	Wa (lb)	xa (in)	Ws (lb)	xs (in)	we (in)	h_t	Topple	Shear	Topple	Shear	
6	44.0	1.50	16.0	-14.0	375	37.0	301	39.5	25	61.2			43.21	16.27			OK!
24	44.0	3.00	12.0	-18.0	750	33.2	594	36.8	308	61.8			6.07	4.48			OK!
24	44.0	3.00	8.0	-22.0	750	29.2	594	32.8	616	63.3			2.46	2.52			OK!
24	74.0	3.00	4.0	4.0	1,838	47.6	594	28.8	0	0.0	30		2.81	2.23			OK!
24	74.0	3.00	0.0	0.0	1,838	43.6	594	24.8	0	0.0	30						
	74.0	13.50	16.0	-14.0	5,550	41.1	2,678	31.8	949	62.8	9,176						

External Stability OK!

backfill height 13.50 feet
exposed height 12.75 feet

ω= 6.34 deg
ω'= -4.94 deg

interface friction angle
δ 22.5 deg

Retained Soil
γ 120 pcf
ϕ 30 deg

Foundation Soil
γ 125 pcf
ϕ 26 deg
c' 150 psf

allowable bearing pressure (if specified) n/a psf (net)

base embedment 9 in
base thickness 9 in
base material agg
toe slope H:1V slope

Aggregate Unit Fill
γ 110 pcf

composite friction coefficient μ_b 0.61

STONE STRONG SYSTEMS

STONE STRONG GRAVITY CALCULATIONS - ver 6.0

Project Name: Example Calculations
Location:
Job#: 20004.00
Section: Example #2
Calc by: D Thiele

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Seismic Load

Ss [ ] G   site class (A to E or 1) D   Fpga 1.60   Fa 1.60 k_h 0.00

Backfill Slope & Surcharge

		backslope	LL surcharge	tier height
length 1	30 feet (horizontal)	3 H:1V slope	psf	ft
length 2	feet (horizontal)	H:1V slope	psf	ft
length 3	feet (horizontal)	H:1V slope	psf	ft
length 4	feet (horizontal)	H:1V slope	psf	

effective slope 3.00 H:1V slope   β 18.4 deg   avg q 0 psf

failure plane α 49.87 deg   zone of influence 22.45 ft

Analysis

	Q_{lh} = 0 lb	ΔK_{AE} = 0.000	e= 0.95 ft
K_a = 0.456	Q_{lv} = 0 lb	P_{IR} = 0 lb	B_f' = 5.01 ft
P_h = 4,425 lb	R_s = 6,916 lb	ΔP_{AEh} = 0 lb	e_{eq}= 0.95 ft
P_v = 2,298 lb	q_{ult} = 9,485 psf	ΔP_{AEv} = 0 lb	$B'_{f\,eq}$ = 5.01 ft

Results

Overturning:	Desired FS = 1.5	**Actual FS = 2.11**	***OK!***
Sliding:	Desired FS = 1.5	**Actual FS = 1.56**	***OK!***
Bearing Capacity:	Desired FS = 2	**Actual FS = 3.98**	***OK!***
	q_{all} = 4,743 psf	q_c = 2,385 psf	

Internal Safety Factors

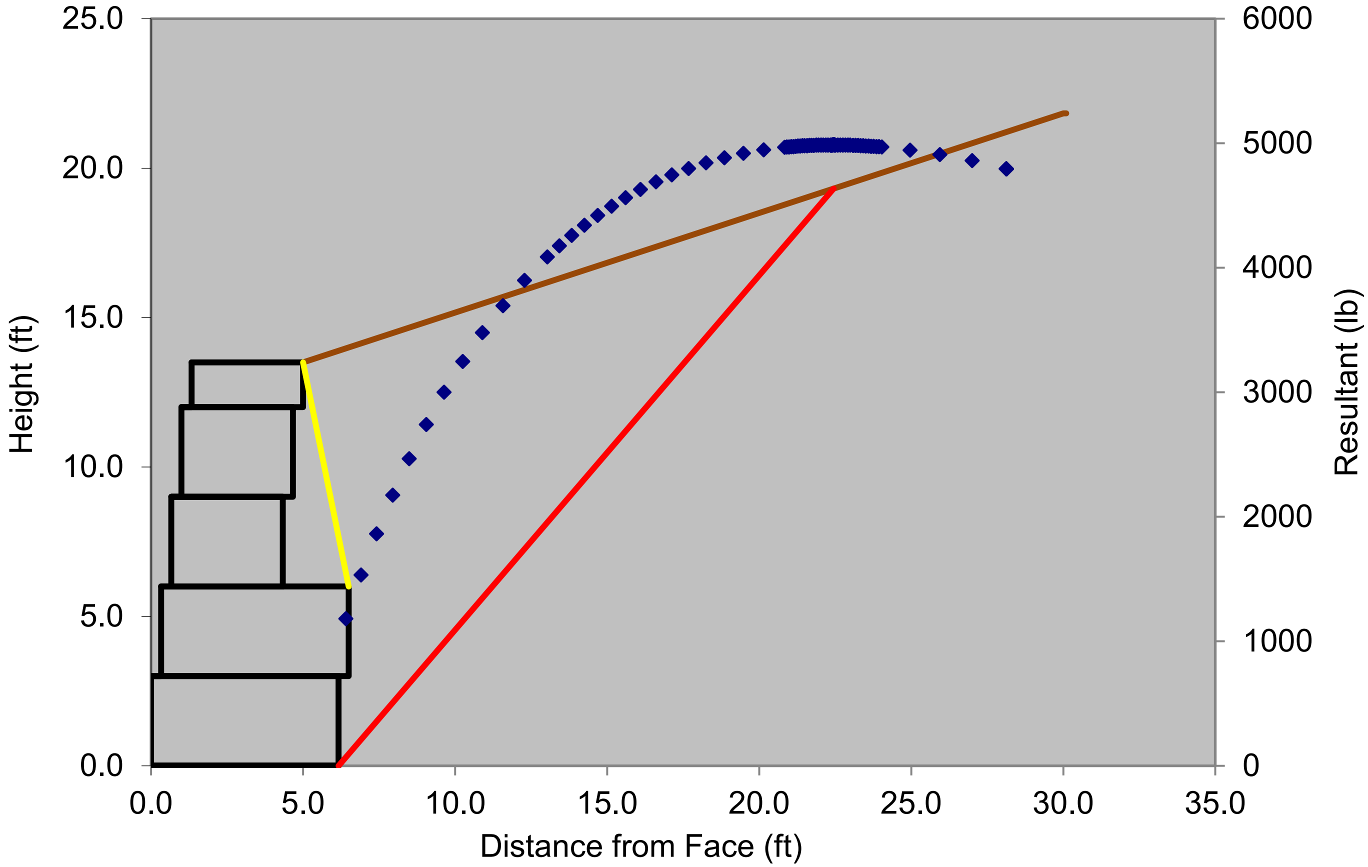
Desired FS = 1.5

Desired FS = 1.5

STONE STRONG SYSTEMS®

Project Name:	Example Calculations
Location:	
Job#:	20004.00
Section:	Example #2
Calc by:	D Thiele

Ground Surface & Trial Wedge Plot



STONE STRONG GRAVITY CALCULATIONS - ver 6.0

Project Name: Example Calculations
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Notes
13.5 tall wall with CIP tail extension, battered face
3H:1V backslope
Internal Stability

Wall Configuration

			setback (in)		modular units		unit fill		soil wedge		CIP Extension		Internal Stability FS		Seismic Internal FS		
unit	w (in)	h (ft)	face	tail	Wb (lb)	xb (in)	Wa (lb)	xa (in)	Ws (lb)	xs (in)	we (in)	h_t	Topple	Shear	Topple	Shear	
6	44.0	1.50	8.0	8.0	375	28.0	301	30.5					43.21	16.27			OK!
24	44.0	3.00	4.0	4.0	750	24.2	594	27.8					6.07	4.48			OK!
24	44.0	3.00	0.0	0.0	750	20.2	594	23.8									
	44.0	7.50	8.0	8.0	1,875	23.4	1,489	26.8	0	0.0	3,364						

Internal Stability OK!

backfill height 7.50 feet

ω= 6.34 deg
ω'= 6.34 deg

interface friction angle
δ 15.0 deg

Retained Soil
γ 120 pcf
ϕ 30 deg

Internal ONLY

Aggregate Unit Fill
γ 110 pcf

STONE STRONG SYSTEMS

STONE STRONG GRAVITY CALCULATIONS - ver 6.0

Project Name:	Example Calculations
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Seismic Load Ss [ ] G site class (A to E or 1) D Fpga 1.60 Fa 1.60 k_h 0.00

Backfill Slope & Surcharge

			backslope		LL surcharge		tier height	
length 1	30	feet (horizontal)	3	H:1V slope		psf		ft
length 2		feet (horizontal)		H:1V slope		psf		ft
length 3		feet (horizontal)		H:1V slope		psf		ft
length 4		feet (horizontal)		H:1V slope		psf		

effective slope 3.00 H:1V slope β 18.4 deg avg q 0 psf

failure plane α 48.61 deg zone of influence 12.68 ft

Analysis

		Q_{lh} =	0 lb	ΔK_{AE} =	0.000	e=	0.43 ft
K_a =	0.340	Q_{lv} =	0 lb	P_{IR} =	0 lb	B_f' =	3.46 ft
P_h =	1,135 lb	R_s =	2,857 lb	ΔP_{AEh} =	0 lb	e_{eq}=	0.43 ft
P_v =	173 lb	q_{ult} =	8,275 psf	ΔP_{AEv} =	0 lb	$B'_{f\,eq}$ =	3.46 ft

Results

						Internal Safety Factors	
Internal Overturning:	Desired FS =	1.5	**Actual FS =**	**2.46**	***OK!***	Desired FS =	1.5
Interface Shear:	Desired FS =	1.5	**Actual FS =**	**2.52**	***OK!***	Desired FS =	1.5

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Ground Surface & Trial Wedge Plot