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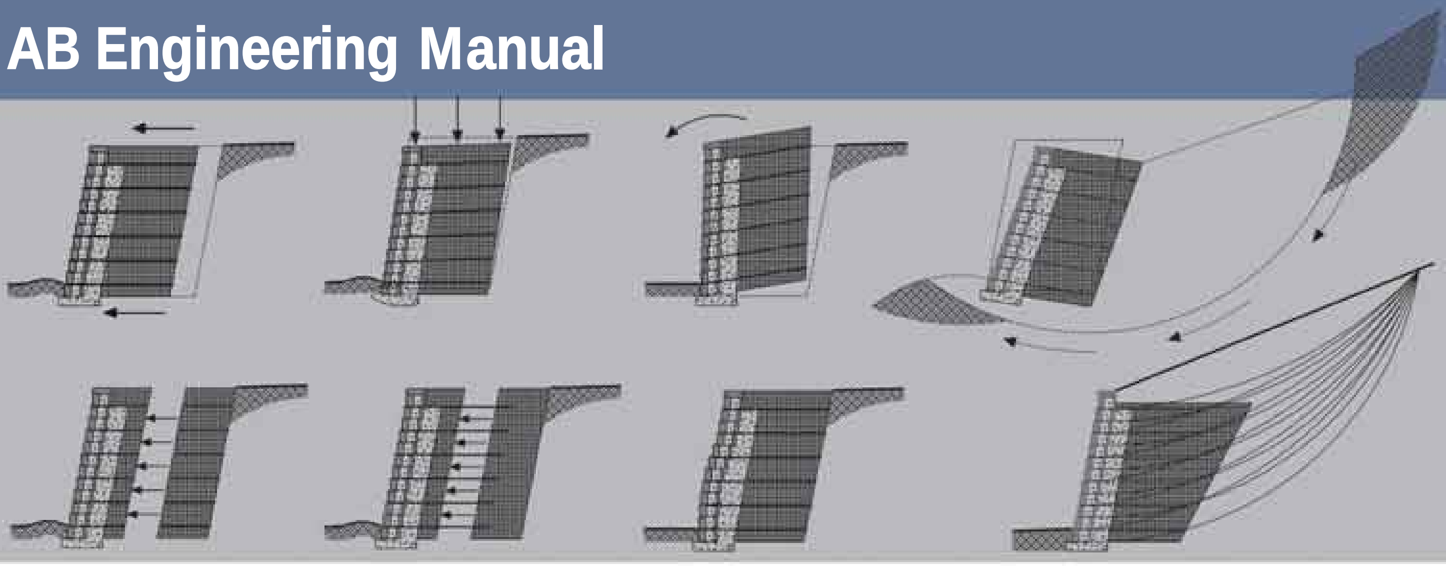
AB Engineering Manual



Allan Block® Retaining Walls

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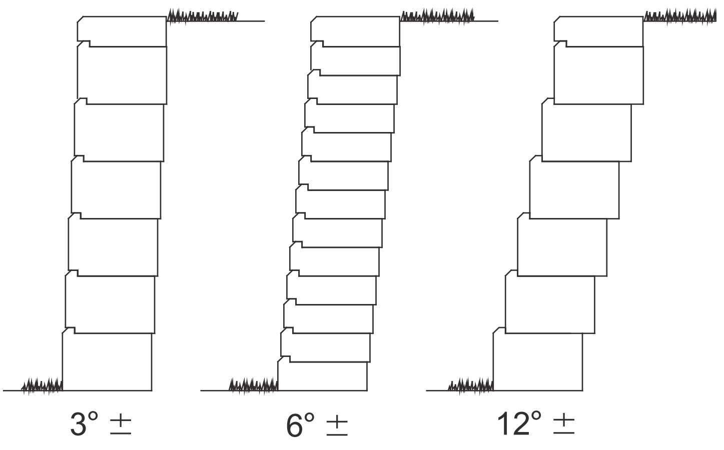
FOREWORD

This manual presents the techniques used by Allan Block in our engineering practice to design retaining walls. It is not intended as a textbook of soil mechanics or geotechnical engineering. The methods we use are based on time tested soil mechanics and the principles of dry stacked block which have existed for thousands of years. Manufactured segmental retaining walls have evolved over the course of over twenty years and continue to evolve as our knowledge and experience grows.

The intended users of this manual are practicing engineers. When writing it, we assumed that the reader would already be familiar with the basic principles of statics and soil mechanics. We encourage others to contact a qualified engineer for help with the design of geogrid reinforced retaining walls.

The example problems in this manual are based on walls constructed with Allan Block Retaining Wall System's AB Stones. The AB Stones provide a nominal setback of twelve degrees from vertical. We believe that a twelve degree setback maximizes the leverage achieved by a battered wall, while providing a finished retaining wall that fulfills the goal of more useable flat land. Allan Block also has developed products with three and six degree nominal setbacks. The equations that follow can be used for each product by selecting the appropriate β angle (β = 90 - Wall Batter).



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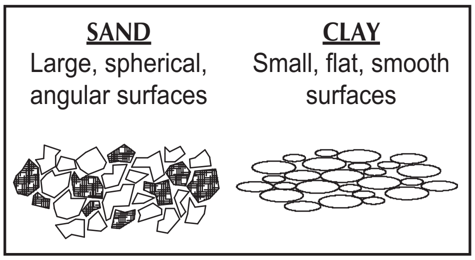
CHAPTER ONE

Concepts & Definitions

Soil Characteristics

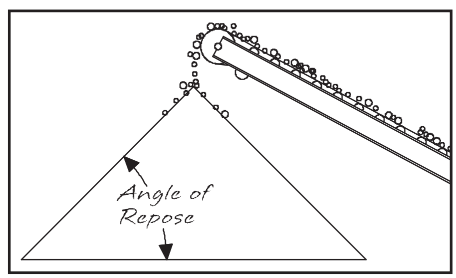
Soil can be described in many different ways. One way to describe it is by the average size of the particles that make up a soil sample. Sandy soil consists of relatively large particles, while clay soil consists mainly of smaller particles. Another way to describe soil is by the tendency of the particles to stick together -- a property called cohesion. Sand, such as is found at the beach, has very low cohesion. Even when it is wet, you can pick up a handful of sand and it will pour out of your hand as individual particles. Clay, on the other hand, is much more cohesive than sand. A wet clay soil can be molded into a ball or rolled into a thread that resists being pulled apart.



Still another way to describe a soil is by its natural tendency to resist movement. This property can be expressed by a number known as the effective friction angle, or simply, the friction angle (PHI ϕ). It should be noted that the design methodology outlined in this manual uses the effective friction angle without the addition of cohesion to increase the design strength of the soil. At the discretion of the engineer of record, cohesion may be used when calculating the ultimate bearing capacity even though it is typically ignored.

If you take a dry soil sample and pour it out onto a flat surface, it will form a cone-shaped pile. The angle formed by the base of the cone and its sides is known as the angle of repose. The angle of repose of a soil is always smaller than the friction angle for the same soil. However, the difference between the two angles is small and for the design of retaining walls the angle of repose can be used to approximate the friction angle. The larger the friction angle the steeper the stable slope that can be formed using that soil.

Soil that consists mainly of sand has a larger friction angle than soil composed mainly of clay. This is due to the fact that sand particles are roughly spherical with irregular surfaces, while clay particles are flat and smooth. When subjected to external pressure, the clay particles tend to slide past one another. The surface irregularities of the sand particles tend to interlock and resist movement.



Clay soil has some characteristics that make it undesirable for use as backfill for a retaining wall. First of all, clay soil is not readily permeable and retains the water that filters into it. The added weight of the retained water increases the force on the retaining wall. Secondly, once the clay becomes saturated, its cohesion decreases almost to zero. The shear strength of the soil is the sum of the frictional resistance to movement and the cohesion of the soil. Once the cohesion is lost due to soil saturation, the full force of the weight of water and most of the weight of the soil is applied to the wall. For these reasons, clay soil is not a good choice for retaining wall backfill.

The preferred soil for backfill behind retaining walls is soil that contains a high percentage of sand and gravel. Such a soil is referred to as a granular soil and has a friction angle of approximately 32° to 36°, depending on the degree of compaction of the soil. The main reason for preferring a granular soil for backfill is that it allows water to pass through it more readily than a nongranular, or clayey soil does. Also, the shear strength of a granular soil doesn't vary with moisture content and therefore its shear strength is more predictable.

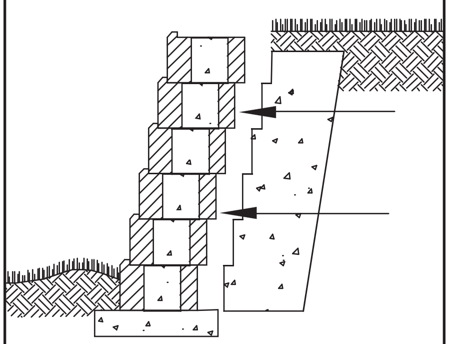
Infill material shall be site excavated soils when approved by the on-site soils engineer unless otherwise specified in the drawings. Unsuitable soils for backfill (heavy clays or organic soils) shall not be used in the reinforced soil mass. Fine grained cohesive soils (f<31) may be used in wall construction, but additional backfilling, compaction and water management efforts are required. Poorly graded sands, expansive clays and/or soils with a plasticity index (PI) >20 or a liquid limit (LL) >40 should not be used in wall construction.

Typical Soil Properties

Soil Groups	Cohesion Compacted	Cohesion Saturated	Soil Friction Angle
Clean Gravel-Sand Mix	0	0	36°
Sand-Silt Clay Mix	1050 PSF (50 KPA)	300 PSF (14 KPA)	32°
Inorganic Clays	1800 PSF (86 KPA)	270 PSF (13 KPA)	27°

Retaining Wall Failure

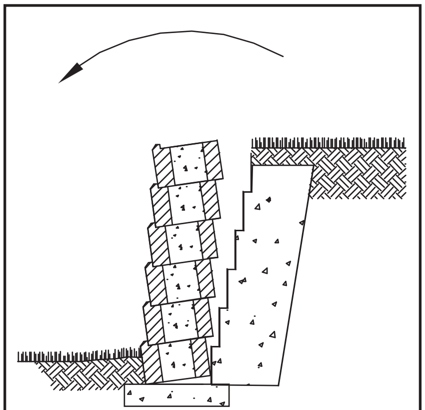
There are two primary modes of retaining wall failure. The wall can fail by sliding too far forward and encroaching on the space it was designed to protect. It can also fail by overturning -- by rotating forward onto its face.



Sliding

Sliding Failure

Sliding failure is evident when the wall moves forward, and occurs when the horizontal forces tending to cause sliding are greater than the horizontal forces resisting sliding. Generally, this will occur when either the driving force is underestimated or the resisting force is overestimated. Underestimating the driving force is the most common mistake and usually results from: 1) neglecting surcharge forces from other walls, 2) designing for level backfill when the backfill is in fact sloped, 3) using cohesive soils for backfill.



Overturning

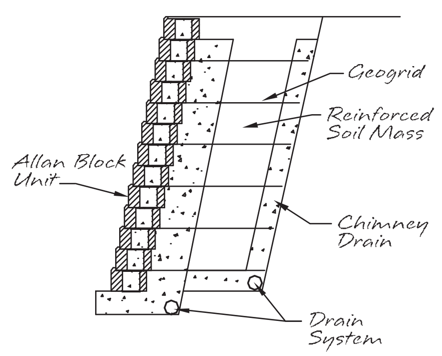
Overturning Failure

Overturning failure is evident when the wall rotates about its bottom front edge (also called the toe of the wall). This occurs when the sum of the moments tending to cause overturning is greater than the sum of the moments resisting overturning. As with sliding failures, overturning failures usually result from underestimating the driving forces.

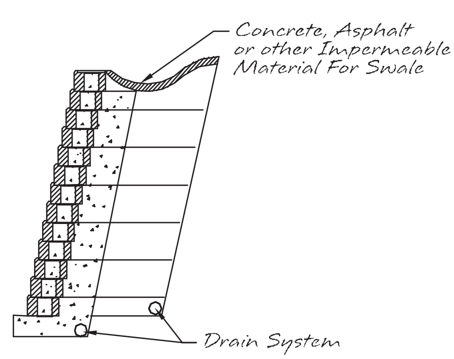
Effects of Water on Wall Stability

Perhaps the single most important factor in wall failure is water. Water contributes to wall failure in several different ways. If the soil used for backfill is not a free-draining granular soil, it will retain most of the water that filters into it. The force on a wall due to water can be greater than the force due to soil. Walls with greater setbacks have a larger natural resistance to overturning.

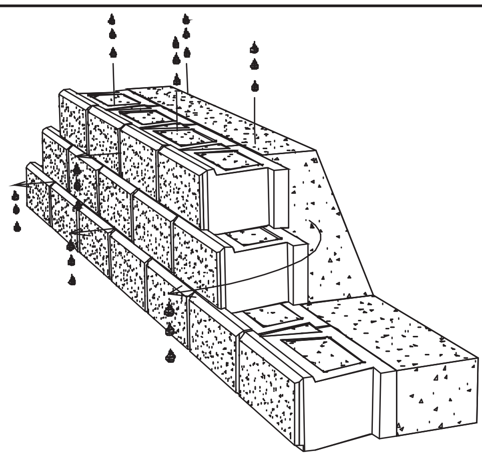
As the moisture content of the soil increases, the unit weight of the soil increases also, resulting in greater force on the wall. When the soil becomes saturated, the unit weight of the soil is reduced because of the buoyant force of the water on the soil particles. However, the water exerts hydrostatic pressure on the wall. Therefore, the total force on the wall is greater than it is for unsaturated soil, because the force on the wall is the sum of the force exerted by the soil and the force exerted by the water. The problem is even greater if the soil contains a high percentage of clay. Saturated, high-clay-content soil loses its cohesion and the force on the wall increases. Good drainage is essential for proper wall design.



Chimney Drain



Concrete Swale

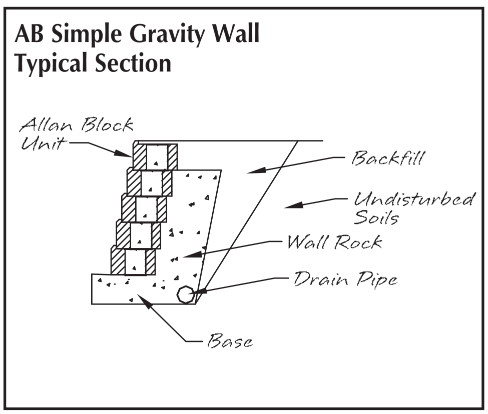


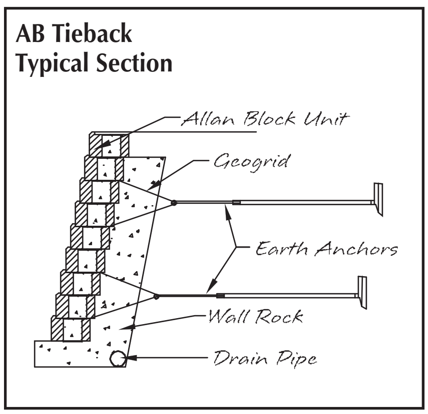
Drainage

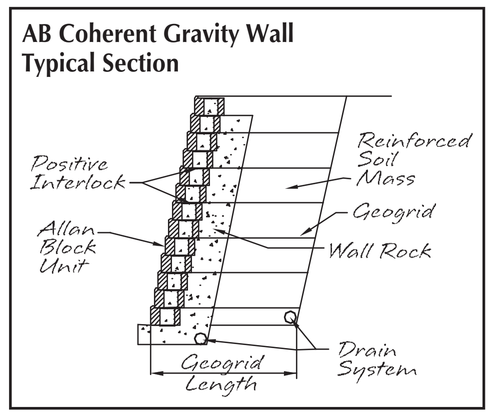
Some clay soils exhibit the characteristic of expanding when wet. This expansion, coupled with contraction when the soil dries, can work to weaken the soil mass and cause failure.

Another way in which water contributes to wall failure is by the action of the freeze-thaw cycle. Water trapped in the soil expands when it freezes causing increased pressure on the wall. Water in contact with the wall itself can also cause failure of the concrete within the block.

Several things can be done to reduce the likelihood of wall failure due to water. First, use a free-draining granular material for the backfill. Second, create a drain field in and around the block cores and 12 inches (300 mm) deep behind the wall using a material with large individual particles, such as gravel. Third, install a drain pipe at the bottom rear of the base and provide outlets as needed. Finally, direct water away from the top and bottom of the wall using swales as required. All these measures will ensure that excess water is removed from behind the wall before it can build up or freeze and cause damage.







Types of Retaining Walls

• Simple Gravity

A wall that relies solely on its weight to prevent failure is called a gravity wall. For a gravity wall, the primary factor affecting the wall's resistance to overturning is the horizontal distance from the toe of the wall to the center of gravity of the wall. The greater this distance is, the less likely it is that the wall will overturn. For example, a wall four feet high and two feet thick will have a lower resistance to overturning than a wall two feet high and four feet thick, even if the weights are equal. **Battering the retaining wall also enhances stability by moving the center of gravity back from the toe of the wall and reducing the load applied to the wall from the soil**.

• Tieback

Anchor reinforced walls rely on mechanical devices embedded in the backfill to provide the force necessary to resist sliding and overturning. **Battering an anchor reinforced wall will shift its center of gravity and enhance its stability. Examples of tieback walls will include: earth anchors and soil nails.**

• Coherent Gravity

Coherent gravity walls, also known as Geogrid Reinforced walls, combine the mass of the wall facing with the mass of the soil behind into one coherent mass that together resists sliding and overturning. Coherent Gravity walls use a flexible synthetic mesh (geogrid) to stabilize the soil. Studies have shown that retaining walls reinforced with several layers of geogrid act as giant gravity walls. **"Geogrid reinforced soil masses create the same effect as having an extremely thick wall with the center of gravity located well back from the toe of the wall."** For this reason, reinforced soil walls are more likely to fail by sliding than by overturning.

Forces Acting on Retaining Walls

The forces that act on a retaining wall can be divided into two groups:

- Those forces that tend to cause the wall to move
- Those forces that oppose movement of the wall (see Figure 1-1)

Included in the first group are the weight of the soil behind the retaining wall and any surcharge on the backfill. Typical surcharges include driveways, roads, buildings, and other retaining walls. Forces that oppose movement of the wall include the frictional resistance to sliding due to the weight of the wall, the passive resistance of the soil in front of the wall, and the force provided by mechanical restraining devices. When the forces that tend to cause the wall to move become greater than the forces resisting movement, the wall will not be stable.

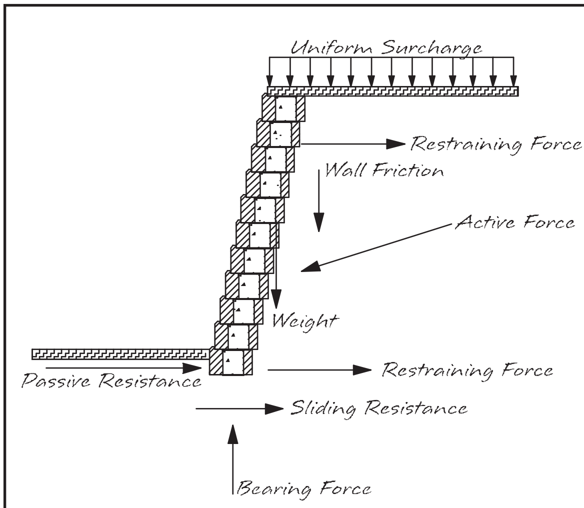


Figure 1-1. Forces Acting on the Retaining Walls

Soil States

The soil behind a retaining wall exists in one of three states:

1) the active state, **2)** the passive state, **3)** the at-rest state.

When a wall is built and soil is placed behind it and compacted, the soil is in the *at-rest state*. If the pressure on the wall due to the soil is too great, the wall will move forward. As the wall moves forward, the soil settles into a new equilibrium condition called the *active state*. The pressure on the wall due to the soil is lower in the active state than it is in the at-rest state (see Figure 1-2).

The *passive state* is achieved when a wall is pushed back into the soil. This could occur by building the retaining wall, placing and compacting the soil, and then somehow forcing the retaining wall to move into the backfill. Usually, the passive state occurs at the toe of the wall when the wall moves forward. The movement of the wall causes a horizontal pressure on the soil in front of the wall. This passive resistance of the soil in front of the wall helps keep the wall from sliding. However, the magnitude of the passive resistance at the toe of the wall is so low that it is usually neglected in determining the stability of the wall.

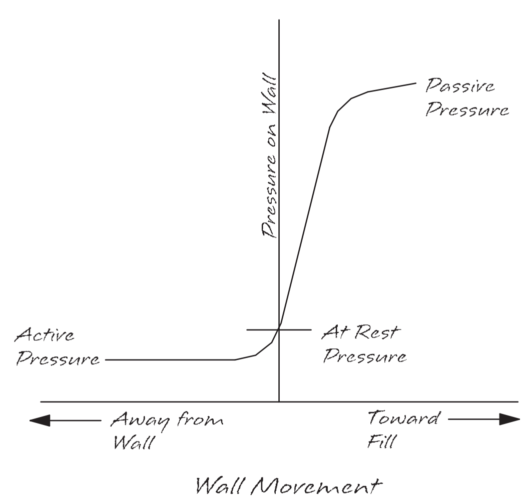


Figure 1-2. Relative Pressures for the Three Soil States

The occurrence of the passive state behind a retaining wall is extremely rare and it will most likely never be encountered behind an Allan Block wall. The *at-rest condition* occurs whenever a retaining wall is built. Some designers may prefer to take a conservative approach and design for the higher at-rest pressure rather than the active pressure. However, this is not necessary since the amount of wall movement required to cause the pressure to decrease from the at-rest level to the active level is very small. Studies of soil pressure on retaining walls have shown that the top of a retaining wall needs to move only 0.001 times the height of the wall in order for the pressure to drop to the active value.

There are some applications where the wall cannot be allowed to move. These include bridge abutments and walls that are rigidly connected to buildings. In cases such as these, the design should be based on the higher at-rest pressure; otherwise, the lower active pressure can be used. Designing on the basis of the active pressure will reduce the cost of the wall and give a more accurate model of the actual behavior of most retaining walls.

Active and Passive Zones

When the wall moves forward, a certain portion of the soil behind the wall moves forward also. The area containing the soil that moves with the wall is referred to as the *active zone*. The area behind the active zone is called the *passive zone*. The line that divides the two zones is called Theoretical Failure Surface or the Line of Maximum Tension. This can be estimated by drawing a line that begins at the bottom rear edge of the wall and extends into the backfill at an angle of 45° plus one-half the friction angle of the soil (45° + ϕ/2) and intersects a vertical line 0.3 times the height of the wall (H x 0.3), Figure 1-3.

The active zone for a geogrid reinforced soil mass includes the entire reinforcement zone and the area included in the theoretical failure surface. The origin of the theoretical failure surface is located at the back bottom of the reinforced zone.

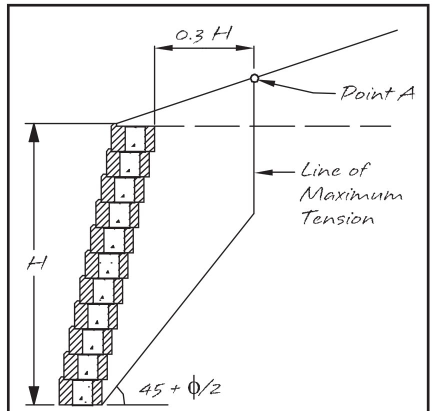


Figure 1-3. Theoretical Failure Surface

Pressure Coefficients

The horizontal stress (σ_h) on a retaining wall due to the retained soil is directly proportional to the vertical stress (σ_v) on the soil at the same depth. The ratio of the two stresses is a constant called the *pressure coefficient*:

$$K = \frac{(\sigma_h)}{(\sigma_v)}$$

The pressure coefficient for the at-rest state can be calculated using the formula:

$$K_o = 1 - \sin(\phi)$$

The active pressure coefficient can be calculated using an equation that was derived by Coulomb in 1776. This equation takes into account the slope of the backfill, the batter of the retaining wall, and the effects of friction between the retained soil and the surface of the retaining wall. Figure 1-4 illustrates the various terms of Coulomb's equation.

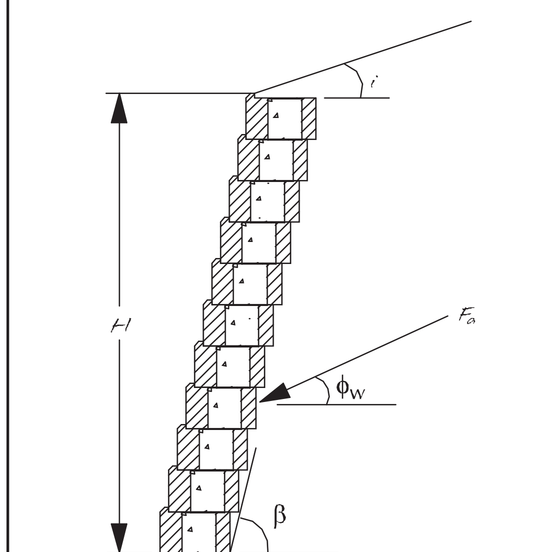


Figure 1-4. Definition for Active Force for Coulomb's Equation

where:

ϕ = the friction angle of the soil.

F_a = the active force on the retaining wall; it is the resultant force of the active pressure on the retaining wall

H = distance from the bottom of the wall to the top of the wall

γ = unit weight of the soil

β = angle between the horizontal and the sloped back face of the wall

i = slope of the top of the retained soil

ϕ_w = angle between a line perpendicular to the wall face and the line of action of the active force

K_a = the active pressure coefficient

$$K_a = \left[\frac{\csc(\beta)\sin(\beta - \phi)}{\sqrt{\sin(\beta + \phi_w)} + \sqrt{\dfrac{\sin(\phi + \phi_w)\sin(\phi - i)}{\sin(\beta - i)}}} \right]^2$$

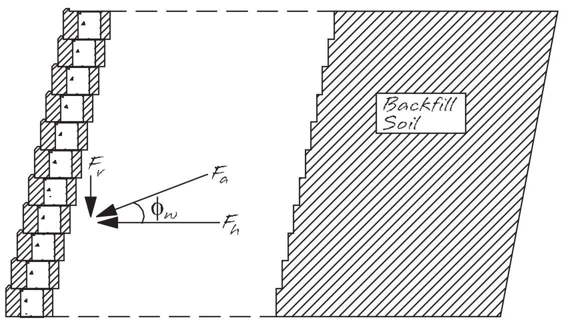


Figure 1-5. Effect of Wall Friction on Active Force

As the wall moves forward slightly, the soil enters the active state by moving forward and downward. At the interface of the soil and the wall, this downward movement of the wall is resisted by the friction between the soil and the wall. Figure 1-5 shows the resultant active force on a retaining wall and the effect of wall friction on the direction of the force.

The magnitude of ϕ_w varies depending on the compaction level of the backfill. For a loose backfill, ϕ_w is approximately equal to ϕ. For a dense back-fill, however, $\phi_w < \phi$. Since retaining wall backfill is thoroughly compacted, the design method in this manual assumes that $\phi_w = (0.66)\ \phi$.

Active Force on the Wall

Once the active pressure coefficient has been determined, the active force on the wall can be determined. Figure 1-6 shows the active pressure distribution on a retaining wall. The active pressure distribution is triangular, which reflects the fact that soil pressure increases linearly with soil depth. The vertical pressure at any depth is given by:

$$P_v = (\gamma)\ (H)$$

Where:

γ = the unit weight of the soil

H = the depth from the top of the retained soil mass.

As discussed previously, the horizontal pressure (P_h) is related to the vertical pressure (P_v) by the active pressure coefficient:

$$K_a = \frac{(P_h)}{(P_v)}$$

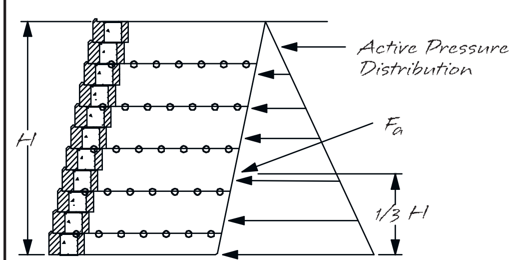


Figure 1-6. Active Pressure Distribution on a Retaining Wall

Since K_a and γ are constants, the horizontal pressure increases linearly as the depth increases and the resulting pressure distribution is triangular. The magnitude of the resultant force of a triangular pressure distribution is equal to the area of the triangle. The pressure at the base of the triangle is given by:

$$P_h = (K_a)\ (\gamma)\ (H)$$

The magnitude of the active force is:

$$\begin{aligned} F_a &= \text{(area of the triangle)} \\ &= (0.5)\ (\text{base})\ (\text{height}) \\ &= (0.5)\ (P_{hb})\ (H) \\ &= (0.5)\ (\gamma)\ (K_a)\ (H)\ (H) \\ &= (0.5)\ (\gamma)\ (K_a)\ (H)^2 \end{aligned}$$

Therefore, the equation for the active force on a retaining wall is:

$$F_a = (0.5)\ (\gamma)\ (K_a)\ (H)^2$$

The resultant force acts at a point above the base equal to one-third of the height of the triangle. Adding a surcharge or slope above the wall has the effect of adding a rectangular pressure distribution. The resultant force of a rectangular pressure distribution acts at a point above the base equal to one-half of the height of the rectangle.

Two-Dimensional Analysis

A retaining wall is a three-dimensional object. It has height, length, and depth. In order to simplify the analysis, the length of the wall is taken to be one foot (or one meter) and the wall is analyzed as a two-dimensional system. Because of this, the units for forces will always be *pounds per foot* (lb/ft) (newtons per meter (N/m)), and the units for moments will be *foot-pounds per foot* (ft-lb/ft) (newton-meters per meter (N-m/m)).

Calculating the Effective Unit Weight of the Wall Facing

The effective unit weight of the wall facing is often needed for wall design. Allan Block's unit weight is the sum of the blocks plus the wall rock material and is calculated below. Concrete usually weighs more than soil. A typical unit weight for concrete is 135 lb/ft^3 (2,163 kg/m^3) while a typical unit weight for soil is 120 lb/ft^3 (1,923 kg/m^3). Depending on the size of the wall, this difference may be significant, and the design engineer should know how to calculate the weight of the wall facing.

The weight of a AB Stone unit is approximately 72 lbs (33 kg). The unit weight of the concrete is 135 lb/ft^3 (2,163 kg/m^3). The block dimensions used are: Length (l) = 1.5 ft (0.46 m), Height (h) = 0.635 ft (0.19 m) and Depth (t) = 0.97 ft (0.3 m). From these values, the volume of concrete for each Allan Block unit can be calculated:

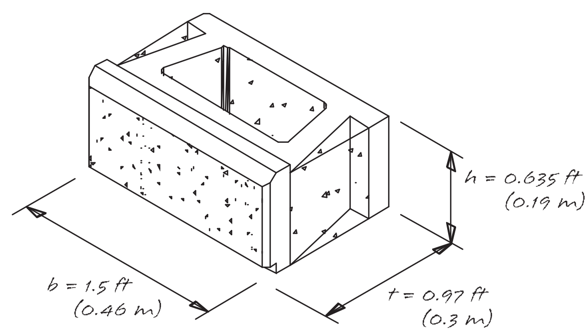
$$V_c = \frac{(72 \text{ lb})}{(135 \text{ lb/ft}^3)} = 0.53 \text{ ft}^3$$

$$= \frac{(33 \text{ kg})}{(2{,}163 \text{ kg/m}^3)} = 0.015 \text{ m}^3$$

The total volume occupied by each standard Allan Block unit, including the voids, is:

$$V_{tot} = (1.5 \text{ ft})(0.635 \text{ ft})(0.97 \text{ ft})$$
$$= 0.92 \text{ ft}^3$$

$$= (0.46 \text{ m})(0.19 \text{ m})(0.3 \text{ m})$$
$$= 0.026 \text{ m}^3$$



Total Unit Weight

Therefore, the volume of the voids is:

$$V_v = V_{tot} - V_c$$
$$= 0.92 \text{ ft}^3 - 0.53 \text{ ft}^3$$
$$= 0.39 \text{ ft}^3$$

$$= 0.026 \text{ m}^3 - 0.015 \text{ m}^3$$
$$= 0.011 \text{ m}^3$$

The unit weight of the wall facing can now be calculated. Assuming that the voids are filled with wall rock with a unit weight of 120 lb/ft^3 (1,923 kg/m^3), the unit weight of the wall facing is:

$$\gamma_{wall} = \frac{(\text{weight of concrete}) + (\text{weight of wall rock})}{(\text{volume of block})}$$

$$= \frac{(\text{weight of concrete}) + (\text{weight of wall rock})}{(V_{tot})}$$

$$= \frac{(0.53 \text{ ft}^3)(135 \text{ lb/ft}^3) + (0.39 \text{ ft}^3)(120 \text{ lb/ft}^3)}{(0.92 \text{ ft}^3)} = 129 \text{ lb/ft}^3$$

$$= \frac{(0.015 \text{ m}^3)(2{,}163 \text{ kg/m}^3) + (0.011 \text{ m}^3)(1{,}923 \text{ kg/m}^3)}{0.026 \text{ m}^3} = 2{,}061 \text{ kg/m}^3$$

Once the unit weight of the wall facing is known, it is a simple matter to calculate the weight per linear foot of wall:

$$
\begin{aligned}
W_f &= \text{(unit weight of wall) (volume of wall)} \\
&= \text{(unit weight of wall) (wall height) (facing depth)} \\
&= (\gamma_{wall})(V_w) \\
&= (\gamma_{wall})(H)(t)
\end{aligned}
$$

For a wall 5.72 feet (1.74 m) tall with a facing depth of 0.97 foot (0.3 m), the weight of the facing would be:

$$
\begin{aligned}
W_f &= (129\ \text{lb/ft}^3)\ (5.72\ \text{ft})\ (0.97\ \text{ft}) &&= (2{,}061\ \text{kg/m}^3)\ (1.74\ \text{m})\ (0.3\ \text{m}) \\
&= 716\ \text{lb/ft} &&= 1{,}076\ \text{kg/m}
\end{aligned}
$$

In general, the weight of the facing is:

$$
W_f = (125\ \text{lb/ft}^2)\ \text{(wall height)} \qquad = (610\ \text{kg/m}^2)\ \text{(wall height)}
$$

Safety Factors

The safety factors used in this design manual conform to the guidelines of the Federal Highway Administration, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Design and Construction Guidelines. They recommend using the following safety factors:

Sliding > 1.5
Overturning > 2.0
Internal Compound Stability > 1.3
Global Stability > 1.3

These are the same values recommended by most governmental agencies. However, you should check your state and local building codes to make sure these safety factors are sufficient.

CHAPTER TWO
Basic Wall Design Techniques

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Gravity Wall

Tieback Wall

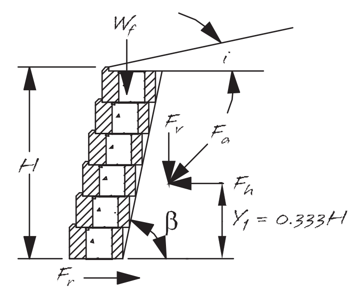
Coherent Gravity Wall

Introduction

One way to classify retaining walls is by the type of reinforcement the walls require. If a wall is stable without reinforcement, it is referred to as a simple gravity wall. When the forces behind a wall are greater than a simple gravity system can provide, a tieback wall can often be built using anchors to tie the wall to the soil or a coherent gravity wall can be built by using two or more layers of geogrid to stabilize the soil mass.

Simple Gravity Walls

Simple gravity walls rely on the weight of the wall to counteract the force of the retained soil. Figure 2-1 is a diagram showing the forces acting on a simple gravity wall. Two modes of failure must be analyzed, sliding and overturning.



Free body Ex. 2-1

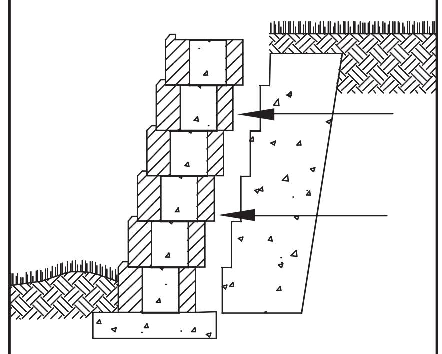
Sliding Failure

A simple gravity wall will not fail in sliding if the force resisting sliding, F_r, is greater than or equal to the force causing sliding, F_h. The force resisting sliding is the frictional resistance at the wall base. The minimum safety factor for sliding failure is 1.5. Therefore, F_r, must be greater than or equal to (1.5) F_h. The following example illustrates the procedure for analyzing stability in sliding.

<u>Example 2-1A</u>: (6 course wall)

Given:

$\phi = 30°$	$H = 3.81$ ft	(1.16 m)
$i = 0°$	$\gamma = 120$ lb/ft^3	(1,923 kg/m^3)
$\beta = 78°$	$\phi_w = (0.666)(\phi) = 20°$	
$K_a = 0.2197$	$\gamma_{wall} = 130$ lb/ft^3	(2,061 kg/m^3)



Sliding

Find: The safety factor against sliding, SFS.

The first step is to determine the total active force exerted by the soil on the wall:

$$F_a = (0.5)(\gamma)(K_a)(H)^2$$

$$= (0.5)(120 \text{ lb/ft}^3)(0.2197)(3.81 \text{ ft})^2 = 191 \text{ lb/ft}$$

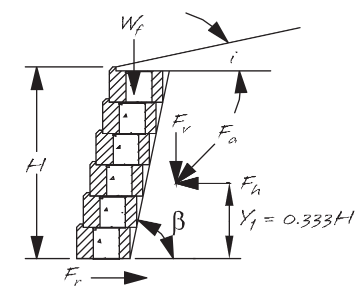
$$= (0.5)(1{,}923 \text{ kg/m}^3)(9.81 \text{m/sec}^2)(0.2197)(1.16\text{m})^2 = 2{,}788 \text{ N/m}$$

As explained in Chapter One, because of the effects of friction between the soil and the wall, the active force acts at an angle to a line perpendicular to the face of the wall. The active force can be resolved into a component perpendicular to the wall and a component parallel to the wall.

The degree of the angle between the active force and a line perpendicular to the face of the wall is ϕ_w. ϕ_w varies according to the compaction level of the soil. For very loose soil, ϕ_w approaches ϕ; for compacted soil, ϕ_w can be as low as (0.666) ϕ. Since our wall designs involve compacting the backfill soil, we use the more conservative value of ϕ_w = (0.666) ϕ. Thus, the horizontal component of the active force is:

$$
\begin{aligned}
F_h &= (F_a)\cos(\phi_w) \\
&= (F_a)\cos[(0.666)(\phi)] \\
&= (191\ \text{lb/ft})\cos(20°) \\
&= 179\ \text{lb/ft}
\end{aligned}
$$

$$
\begin{aligned}
&= (2{,}788\ \text{N/m})\cos(20°) \\
&= 2{,}620\ \text{N/m}
\end{aligned}
$$



Free body Ex. 2-1

Similarly, the vertical component of the active force is:

$$
\begin{aligned}
F_v &= (F_a)\sin(\phi_w) \\
&= (F_a)\sin[(0.666)(\phi)] \\
&= (191\ \text{lb/ft})\sin(20°) \\
&= 65\ \text{lb/ft}
\end{aligned}
$$

$$
\begin{aligned}
&= (2{,}788\ \text{N/m})\sin(20°) \\
&= 954\ \text{N/m}
\end{aligned}
$$

The weight of the wall facing must be determined before the frictional resistance to sliding can be calculated:

$$
\begin{aligned}
W_f &= (\gamma_{wall})(H)(t) \\
&= (130\ \text{lb/ft}^3)(3.81\ \text{ft})(0.97\ \text{ft}) \\
&= 480\ \text{lb/ft}
\end{aligned}
$$

$$
\begin{aligned}
&= (2061\ \text{kg/m}^3)(1.16\ \text{m})(0.3\ \text{m})(9.81\ \text{m/sec}^3) \\
&= 7{,}036\ \text{N/m}
\end{aligned}
$$

The maximum frictional resistance to sliding, F_r is calculated by multiplying the total vertical force, V_t , by the coefficient of friction. The total vertical force is the sum of the weight of the wall and the vertical component of the active force. The coefficient of friction, C_f , is assumed to be equal to tan (ϕ). Thus, the maximum frictional resistance is:

$$
\begin{aligned}
F_r &= (V_t)(C_f) \\
&= (V_t)\tan(\phi) \\
&= (W_f + F_v)\tan(\phi) \\
&= (480\ \text{lb/ft} + 65\ \text{lb/ft})\tan(30°) \\
&= 315\ \text{lb/ft}
\end{aligned}
$$

$$
\begin{aligned}
&= (7{,}036\ \text{N/m} + 954\ \text{N/m})\tan(30°) \\
&= 4{,}613\ \text{N/m}
\end{aligned}
$$

Finally, the safety factor against sliding can be calculated:

$$
\begin{aligned}
SFS &= \frac{(\text{Force resisting sliding})}{(\text{Force causing sliding})} = \frac{F_r}{F_h} \\
&= \frac{(315\ \text{lb/ft})}{(179\ \text{lb/ft})} = \mathit{1.8 \geq 1.5}\ \text{OK}
\end{aligned}
$$

$$
= \frac{(4{,}613\ \text{N/m})}{(2{,}620\ \text{N/m})} = \mathit{1.8 \geq 1.5}\ \text{OK}
$$

The safety factor against sliding is greater than 1.5. Therefore, the wall is stable and doesn't require reinforcement to prevent sliding failure. However, the wall must still be analyzed for overturning failure.

Overturning Failure

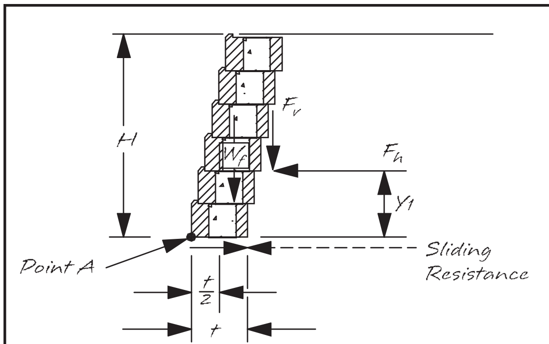


Figure 2-1. Schematic Diagram of Simple Gravity Retaining Wall

Overturning failure occurs when the forces acting on the wall cause it to rotate about the bottom front corner of the wall (Point A in Figure 2-1). For stability, the moments resisting overturning, M_r, must be greater than or equal to the moments causing overturning, M_o. The minimum safety factor for overturning is 2.0. Therefore, M_r must be greater than or equal to (2.0) M_o.

<u>**Example 2-1B**</u>:
Find the safety factor against overturning, SFO, for Example 2-1.

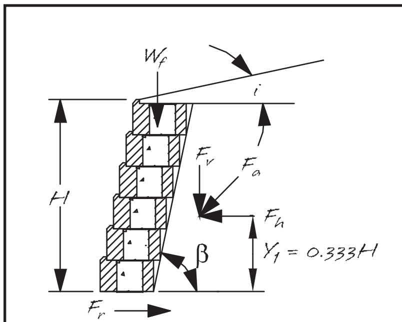
Two forces contribute to the moment resisting overturning of the wall. These are the weight of the wall and the vertical component of the active force on the wall. Summing these moments about Point A:

$$M_r = (W_f)\,[(t/2) + (0.5)\,(H)\tan(90° - \beta)] + (F_v)\,[(t) + (0.333)\,(H)\tan(90° - \beta)]$$

$$= (480\text{ lb/ft})\,[(0.49\text{ ft}) + (0.5)\,(3.81\text{ ft})\tan(90° - 78°)]$$
$$+\ (65\text{ lb/ft})\,[(0.97\text{ ft}) + (0.333)\,(3.81\text{ ft})\tan(90° - 78°)]$$
$$= 510\text{ ft-lb/ft}$$

$$= (7{,}036\text{ N/m})\,[(0.149\text{ m}) + (0.5)\,(1.16\text{ m})\tan(90° - 78°)]$$
$$+\ (954\text{ N/m})\,[(0.3\text{ m}) + (0.333)\,(1.16\text{ m})\tan(90° - 78°)]$$
$$= 2{,}280\text{ N-m/m}$$

(**NOTE**: The quantities (0.5) (H) tan (90° − β) and (0.333) (H) tan (90° − β) account for the distance added to the moment arms because the wall is not vertical.)



Free body Ex. 2-1

The horizontal component of the active force is the only force that contributes to the overturning moment. The active force is the resultant of the active pressure distribution, which is triangular. For triangular pressure distributions, the vertical centroid is located at one-third the height of the triangle. Therefore, the horizontal component of the active force acts on the wall (0.333) H from the bottom of the wall, where $\gamma_1 = 1/3H$. The moment causing overturning is given by:

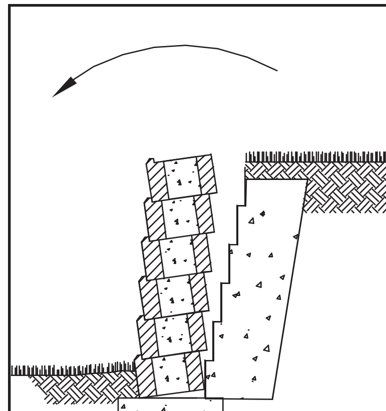
$$M_o = (F_h)\,(y_1) = (F_h)\,(0.333)\,(H)$$
$$= (179\text{ lb/ft})\,(0.333)\,(3.81\text{ ft}) = 227\text{ ft-lb/ft}$$
$$= (2{,}620\text{ N/m})\,(0.333)\,(1.16\text{ m}) = 1{,}012\text{ N-m/m}$$

The safety factor against overturning is:

$$SFO = \frac{(\text{Moment resisting overturning})}{(\text{Moment causing overturning})} = \frac{M_r}{M_o}$$

$$= \frac{(510\text{ ft-lb/ft})}{(227\text{ ft-lb/ft})} = 2.2 \geq 2.0\ \ OK$$

$$= \frac{(2{,}280\text{ N-m/m})}{(1{,}012\text{ N-m/m})} = 2.2 \geq 2.0\ \ OK$$



Overturning

The safety factor against overturning is greater than 2.0. Therefore, the wall is stable and doesn't require geogrid reinforcement to prevent overturning. As calculated previously, the safety factor against sliding is also greater than 1.5 for this wall. This wall is adequate in both sliding and overturning and no geogrid reinforcement is required.

Tieback Walls

A simple gravity wall may be analyzed and found to be unstable in either sliding or overturning. When this occurs, one possible solution is to analyze the wall with soil nails or earth anchors behind it. The soil nail or earth anchor is treated as a restraining device or anchor. The force on the wall due to the weight of the retained soil is calculated exactly as it was in the simple gravity wall analysis. However, the forces resisting failure in this instance are the frictional resistance due to the weight of the wall plus the friction force due to the weight of the soil on the grid or restraining force of the anchor. Figure 2-2 is a schematic diagram of a tieback wall.

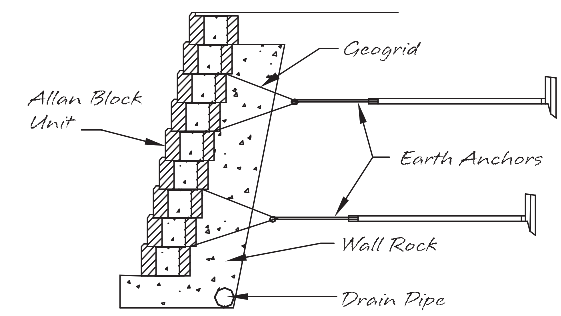


Figure 2-2. Diagram of Retaining Wall for Tieback Analysis

Example 2-2: **(9 course wall)**

Given:

$\phi = 30°$	$\phi_w = 20°$			
$K_a = 0.2197$	$H = 5.72$ ft	(1.74 m)		
$\beta = 78°$	$\gamma = 120$ lb/ft³	(1,923 kg/m³)		
$i = 0°$	$\gamma_{wall} = 130$ lb/ft³	(2,061 kg/m³)		

Find: The safety factors against sliding, SFS, and overturning, SFO.

The first step is to analyze this wall without grid:

W_f = (5.72 ft) (0.97 ft) (130 lb/ft³) = 721 lb/ft

= (1.74 m) (0.3 m) (2,061 kg/m³) (9.81 m/sec²) = 10,554 N/m

Next, the active force of the soil on the wall is calculated:

F_a = (0.5) (120 lb/ft³) (0.2197) (5.72 ft)² = 431 lb/ft

= (0.5) (1,923 kg/m³) (0.2197) (1.74 m)² (9.81 m/sec²) = 6,074 N/m

The horizontal and vertical components of the active force are:

F_h = (431 lb/ft) cos (20°) = 405 lb/ft

= (6,274 N/m) cos (20°) = 5,896 N/m

F_v = (431 lb/ft) sin (20°) = 147 lb/ft

= (6,274 N/m) sin (20°) = 2,146 N/m

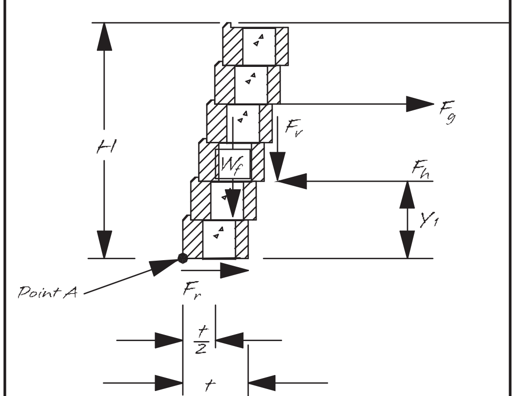


Figure 2-3. Freebody Diagram of Retaining Wall for Tieback Analysis

The total vertical force due to the weight of the wall and the vertical component of the active force is:

V_t = $W_f + F_v$

= 721 lb/ft + 147 lb/ft

= 868 lb/ft

= 10,554 N/m + 2,146 N/m

= 12,700 N/m

The force that resists sliding of the wall because of friction between the wall and the soil is:

$$F_r = (V_t)(C_f)$$
$$= (868 \text{ lb/ft}) \tan(30°)$$
$$= 501 \text{ lb/ft}$$

$$= (12{,}700 \text{ N/m}) \tan(30°)$$
$$= 7{,}332 \text{ N/m}$$

The safety factor against sliding is:

$$SFS = \frac{F_r}{F_h} = \frac{(501 \text{ lb/ft})}{(405 \text{ lb/ft})} = 1.24 \leq 1.5$$

$$= \frac{F_r}{F_h} = \frac{(7{,}332 \text{ N/m})}{(5{,}896 \text{ N/m})} = 1.24 \leq 1.5$$

The safety factor against overturning is:

$$M_r = (W_f)[(t/2) + (0.5)(H)\tan(90° - \beta)] + (F_v)[(t) + (0.333)(H)\tan(90° - \beta)]$$
$$= (721 \text{ lb/ft})[(0.49 \text{ ft}) + (0.5)(5.72 \text{ ft})\tan(90° - 78°)]$$
$$+ (147 \text{ lb/ ft})[(0.97 \text{ ft}) + (0.333)(5.72 \text{ ft})\tan(90° - 78°)]$$
$$= 994 \text{ ft-lb/ft}$$

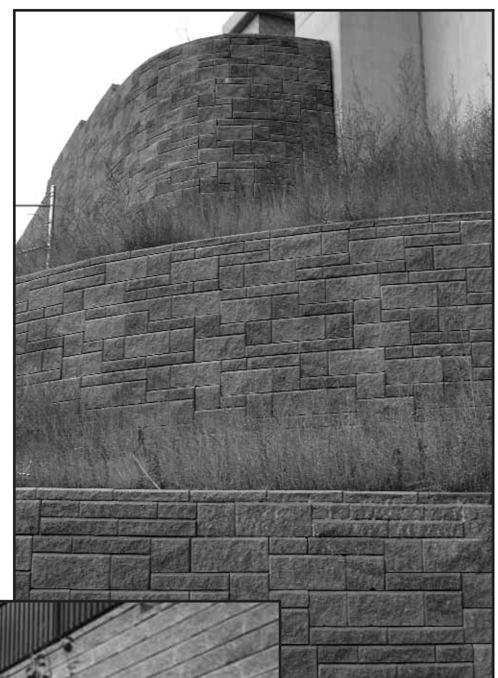
$$= (10{,}554 \text{ N/m})[(0.149 \text{ m}) + (0.5)(1.74 \text{ m})\tan(90° - 78°)]$$
$$+ (2{,}146 \text{ N/m})[(0.3 \text{ m}) + (0.333)(1.74 \text{ m})\tan(90° - 78°)]$$
$$= 4{,}432 \text{ N-m/m}$$

$$M_o = (F_h)(Y_1)$$
$$= (405 \text{ lb/ft})(0.333)(5.72 \text{ ft})$$
$$= 771 \text{ ft-lb/ft}$$

$$= (5{,}896 \text{ N/m})(0.333)(1.74 \text{ m})$$
$$= 3{,}416 \text{ N-m/m}$$

$$SFO = \frac{M_r}{M_o} = \frac{(994 \text{ ft-lb/ft})}{(771 \text{ ft-lb/ft})} = 1.29 \leq 2.0$$

$$= \frac{M_r}{M_o} = \frac{(4{,}432 \text{ N-m/m})}{(3{,}416 \text{ N-m/m})} = 1.29 \leq 2.0$$



Without reinforcement, this wall is not adequate with respect to either sliding failure or overturning failure. Therefore, a tieback wall will be required. A good rule of thumb is to place the reinforcement as close as possible to halfway between the top and bottom of the wall.

Earth Anchors as a Tieback

A single row of earth anchors can be utilized to provide the additional tieback resistance. The earth anchors extend beyond the line of maximum tension and provide additional resistance to overturning and sliding. This additional force can be utilized in our calculations as follows:

F_e = 10,500 lbs. (4,763 kg)

where:

F_e = Preloaded value of installed earth anchor.

For design purposes, we will use a weighted value (0.67 F_e) and correction for horizontal anchor spacing. For this example we will specify spacing of anchors on 8 foot (2.44 m) centers and Fortrac 35/-20-20 geogrid (Diagram Ex.2-2). Therefore the additional force resisting sliding is:

F_r = $(W_f + F_v)$ tan (30°)
= (721 lb/ft + 147 lb/ft) tan (30°) = 501 lb/ft   = (10,554 N/m + 2,146 N/m) tan (30°) = 7,332 N/m

F_{we} = (0.67) F_e ÷ 8 ft = 879 lb/ft   = (0.67) F_e ÷ (2.44 m) = 12,830 N/m

F_{gr} = LTADS = 1,322 lb/ft   = (19,300 N/m)

F_{pa} = 1,313 lb/ft + tan (8°) x N
= 1,313 lb/ft + [0.141 x 1.9 ft (0.97 ft)(130 lb/ft^3)] = 1,347 lb/ft
= 19,160 N/m + tan (8°) x N
= 19,160 N/m + [0.141 x 0.58 m (0.3m)(2,082 N/m^3)] = 19,211 N/m

F_h = 405 lb/ft   = (5,896 N/m)

where:

F_r = The maximum frictional resistance to sliding.
F_{we} = Weighted design value of anchor.
F_{gr} = Restraining strength of the geogrid = LTADS.
F_{pa} = Pullout grid capacity. (See Table B-1, page 68)
N = Weight of facing above anchor location.
F_{ga} = The least of F_{we}, F_{gr}, or F_{pa}.

The resulting factor of safety with one row of earth anchors is:

$$SFS = \frac{F_r + F_{ga}}{F_h} = \frac{(501 \text{ lb/ft} + 879 \text{ lb/ft})}{405 \text{ lb/ft}}$$

$= 3.41 \geq 1.5$ *OK*

$$= \frac{(7{,}332 \text{ N/m} + 12{,}830 \text{ N/m})}{5{,}896 \text{ N/m}}$$

$= 3.41 \geq 1.5$ *OK*

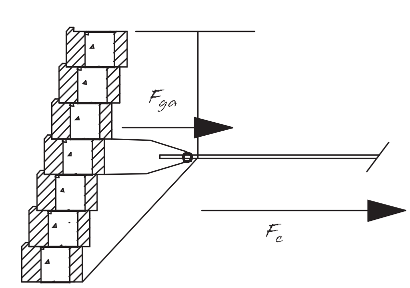


Diagram Ex. 2-2

The safety factor against overturning is:

$$M_r = (W_f)\,[(t/2) + (0.5)\,(H)\tan(90° - \beta)] + (F_v)\,[(t) + (0.333)\,(H)\tan(90° - \beta)] + F_{ga}\,(H/2)$$
$$= (721\text{ lb/ft})\,[(0.49\text{ ft}) + (0.5)\,(5.72\text{ ft})\tan(90° - 78°)]$$
$$+ (147\text{ lb/ft})\,[(0.97\text{ ft}) + (0.333)\,(5.72\text{ ft})\tan(90° - 78°)] + (879\text{ lb/ft})\,(2.86\text{ ft})$$
$$= 3{,}507\text{ ft-lb/ft}$$

$$= (10{,}554\text{ N/m})\,[(0.149\text{ m}) + (0.5)\,(1.72\text{ m})\tan(90° - 78°)]$$
$$+ (2{,}146\text{ N/m})\,[(0.296\text{ m}) + (0.333)\,(1.72\text{ m})\tan(90° - 78°)] + (12{,}830\text{ N/m})\,(0.86\text{ m})$$
$$= 15{,}432\text{ N-m/m}$$

$$SFO = \frac{M_r}{M_o} = \frac{(3{,}507\text{ ft-lb/ft})}{(771\text{ ft-lb/ft})} = 4.5 \geq \mathit{2.0\ OK}$$

$$= \frac{(15{,}432\text{ N-m/m})}{(3{,}416\text{ N-m/m})} = 4.5 > \mathit{2.0\ OK}$$

The anchor length requires a 3 ft (0.9 m) embedment into the passive zone. (Past the line of maximum tension)

$$L_t = L_a + 3\text{ ft}$$
$$= (5.72\text{ ft} - 2.5\text{ ft})\ [\tan(30°) - \tan(12°)] + 3.0\text{ ft} = 4.2\text{ ft}$$
$$= (1.74\text{ m} - 0.8\text{ m})\,[\tan(30°) - \tan(12°)] + 0.9\text{ m} = 1.24\text{ m}$$

Where:

L_a = length of geogrid in the active zone

See page 27 for further discussion.

Check to determine if the F_{we} or the grid pullout from the block or rupture is the determining factor.

NOTE: The pullout from the block can be eliminated as the governing factor by bonding the block to grid interface with a grouted connection. However, the geogrid type will need to be specified to resist the high alkaline content of the concrete grout. See page 23 for further discussion of grid to block connection.

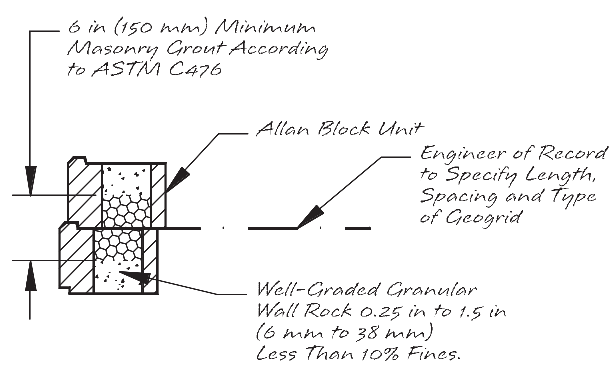


Figure 2-4. Grouted Connection

Coherent Gravity Walls

The theory behind coherent gravity walls is that two or more layers of geogrid make the reinforced soil mass behave as a single unit. The wall facing and reinforced soil mass are then treated as a unit and analyzed as a large simple gravity wall. The wall must be analyzed for stability in sliding and overturning. In addition, the number of layers of geogrid required, and their spacing, must be determined. Finally, the bearing pressure of such a large gravity wall must be checked to ensure that it doesn't exceed the allowable bearing capacity of the soil.

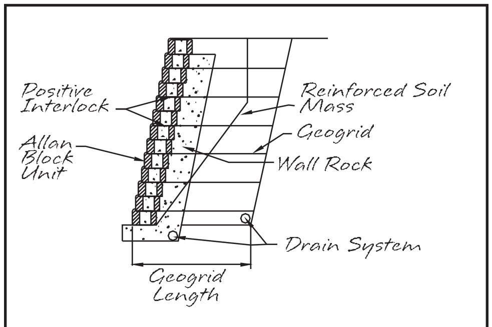


Figure 2-5. Typical Coherent Gravity Wall

Example 2-3:

Figure 2-6 is a schematic diagram of a coherent gravity wall with seven layers of geogrid. Figure 2-8 is a freebody diagram of the same wall. The subscripts r and i refer to the retained soil and the infill soil, respectively. The values shown in Figure 2-6 will be used to analyze the stability of the wall. For this example use 6 ft (1.83 m) geogrid lengths (L_g).

Given: (15 course wall)

i = 0° (Slope above wall)

ϕ_{wi} = 20° ϕ_r = 27°

ϕ_i = 30° β = 78°

K_{ai} = 0.2197 K_{ar} = 0.2561

H = 9.52 ft (2.9 m) γ_r = 120 lb/ft^3 (1,923 kg/m^3)

ϕ_{wr} = 18° γ_i = 125 lb/ft^3 (2,002 kg/m^3)

L_s = Equivalent lip thickness of 12° AB Unit

L_t = $L_g + L_s$

L_t = 6.0 + 0.13 = 6.13 ft (1.87 m)

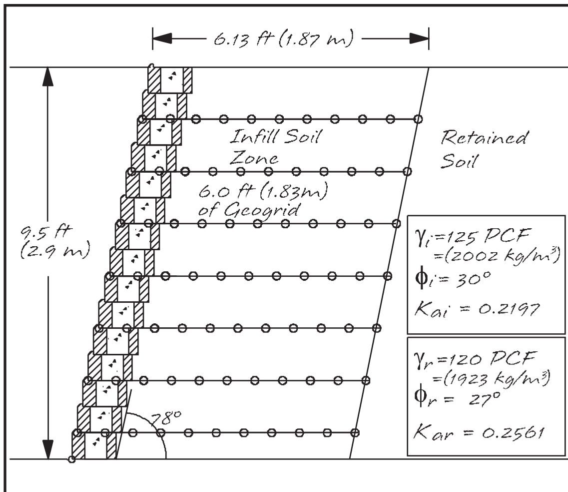


Figure 2-6. Coherent Gravity Wall for Example 2-3

Find: The safety factors against sliding, SFS, and overturning, SFO.

Length of Geogrid

Typically, the first step in analyzing the stability of the wall is to estimate the length of geogrid required. A rule of thumb is that the minimum reinforcement length is <u>60% of the wall height</u>.

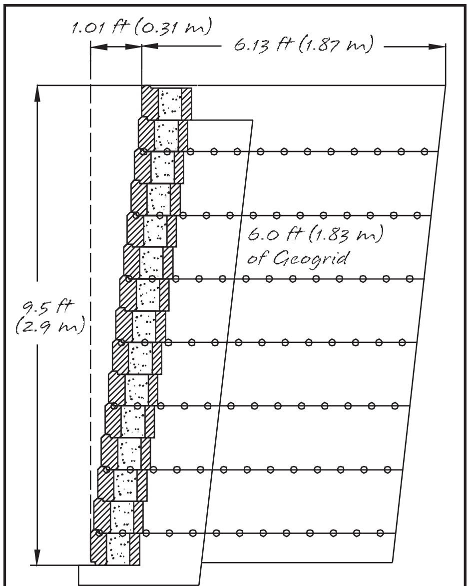


Figure 2-7. Typical Geogrid Reinforcement Layout

External Stability

Once the length of the geogrid is known, the weight of the coherent gravity wall can be calculated. The weight of the structure is the sum of the weights of the wall facing and the reinforced soil mass. The weight of the wall facing is equal to the unit weight of the wall facing times the height times the depth:

W_f = (130 lb/ft^3) (9.52 ft) (0.97 ft) = 1,200 lb/ft

= (2,061 kg/m^3) (2.9 m) (0.3 m) (9.81 m/sec^2) = 17,590 N/m

The weight of the reinforced soil mass is equal to the unit weight of the backfill soil, times the height of the reinforced soil mass, times the depth (measured from back face of wall to the end of the geogrid):

W_s = (125 lb/ft^3) (9.52 ft) (6.0 ft − 0.84 ft) = 6,140 lb/ft

= (2,002 kg/m^3) (2.9 m) (1.83 m − 0.256 m) (9.81 m/sec^2) = 89,647 N/m

The total weight of the coherent gravity wall is:

W_w = W_f + W_s

= (1,200 lb/ft) + (6,140 lb/ft) = 7,340 lb/ft

= (17,590 N/m) + (89,647 N/m) = 107,237 N/m

The next step is to calculate the active force on the gravity wall. The properties of the retained soil are used to calculated the active force since it acts at the back of the reinforced soil zone. The active force is given by the equation:

F_a = (0.5) (γ_r) (K_{ar}) (H)^2

= (0.5) (120 lb/ft^3) (0.2561) (9.52 ft)^2

= 1,393 lb/ft

= (0.5) (1,923 kg/m^3) (0.2561) (2.9 m)^2 (9.81 m/sec^2)

= 20,315 N/m

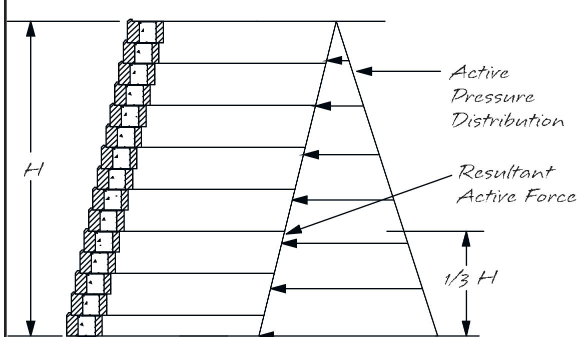


Diagram Ex. 2-3

The horizontal and vertical components of the active force are:

F_h = (F_a) cos (ϕ_{wr})

= (1,393 lb/ft) cos (18°) = (20,315 N/m) cos (18°)

= 1,325 lb/ft = 19,321 N/m

F_v = (F_a) sin (ϕ_{wr})

= (1,393 lb/ft) sin (18°) = (20,315 N/m) sin (18°)

= 430 lb/ft = 6,278 N/m

Next, the total vertical force is calculated:

V_t = W_w + F_v

= (7,340 lb/ft) + (430 lb/ft) = 7,770 lb/ft

= (107,237 N/m) + (6,278 N/m) = 113,515 N/m

The force resisting sliding is calculated by multiplying the total vertical force by the coefficient of friction between the reinforced soil mass and the underlying soil:

F_r = (V_t) (C_f)

= (7,770 lb/ft) tan (30°) = 4,486 lb/ft

= (113,515 N/m) tan (30°) = 65,538 N/m

The safety factor against sliding is:

$$SFS = \frac{F_r}{F_h} = \frac{(4{,}486 \text{ lb/ft})}{(1{,}325 \text{ lb/ft})} = \mathit{3.45 \geq 1.5 \;\; OK}$$

$$= \frac{F_r}{F_h} = \frac{(65{,}538 \text{ N/m})}{(19{,}321 \text{ N/m})} = \mathit{3.4 \geq 1.5 \;\; OK}$$

The safety factor against overturning is:

(**NOTE**: All moments are taken about Point A in Figure 2-8.)

$$\begin{aligned}
\Sigma M_r &= (W_f)\,[(0.5)\,(t) + (0.5)\,(H) \tan(90° - \beta)] \\
&\quad + (W_s)\,[(0.5)\,(L_t - t) + (t) + (0.5)\,(H) \tan(90° - \beta)] \\
&\quad + (F_v)\,[(L_t) + (0.333)\,(H) \tan(90° - \beta)] \\
&= (1{,}200 \text{ lb/ft})\,[(0.5)\,(0.97 \text{ ft}) + (0.5)\,(9.52 \text{ ft}) \tan(90° - 78°)] \\
&\quad + (6{,}140 \text{ lb/ft})\,[(0.5)\,(6.13 \text{ ft} - 0.97 \text{ ft}) + (0.97 \text{ ft}) + (0.5)\,(9.52 \text{ ft}) \tan(90° - 78°)] \\
&\quad + (430 \text{ lb/ft})\,[(6.13 \text{ ft}) + (0.333)\,(9.52 \text{ ft}) \tan(90° - 78°)] \\
&= 32{,}731 \text{ ft-lb/ft}
\end{aligned}$$

$$\begin{aligned}
&= (17{,}590 \text{ N/m})\,[(0.5)\,(0.3 \text{ m}) + (0.5)\,(2.9 \text{ m}) \tan(90° - 78°)] \\
&\quad + (89{,}647 \text{ N/m})\,[(0.5)\,(1.87 \text{ m} - 0.3 \text{ m}) + (0.3 \text{ m}) + (0.5)\,(2.9 \text{ m}) \tan(90° - 78°)] \\
&\quad + (6{,}278 \text{ N/m})\,[(1.87 \text{ m}) + (0.333)\,(2.9 \text{ m}) \tan(90° - 78°)] \\
&= 145{,}985 \text{ N-m/m}
\end{aligned}$$

$$\begin{aligned}
\Sigma M_o &= (F_h)\,(0.333)\,(H) \\
&= (1{,}325 \text{ lb/ft})\,(0.333)\,(9.52 \text{ ft}) \\
&= 4{,}200 \text{ ft-lb/ft}
\end{aligned}$$

$$\begin{aligned}
&= (19{,}321 \text{ N/m})\,(0.333)\,(2.9 \text{ m}) \\
&= 18{,}658 \text{ N-m/m}
\end{aligned}$$

$$SFO = \frac{\Sigma M_r}{\Sigma M_o} = \frac{(32{,}731 \text{ ft-lb/ft})}{(4{,}200 \text{ ft-lb/ft})} = \mathit{7.8 \geq 2.0 \;\; OK}$$

$$= \frac{\Sigma M_r}{\Sigma M_o} = \frac{(145{,}985 \text{ N-m/m})}{(18{,}658 \text{ N-m/m})} = \mathit{7.8 \geq 2.0 \;\; OK}$$

The minimum recommended safety factors for geogrid reinforced retaining walls are 1.5 for sliding failure and 2.0 for overturning failure. Since both safety factors for this wall exceed the minimum values, the wall is adequate with respect to sliding and overturning. In cases where either of the safety factors is lower than required, the length of geogrid is increased and the analysis is repeated. The process ends when both safety factors exceed the minimum recommended values.

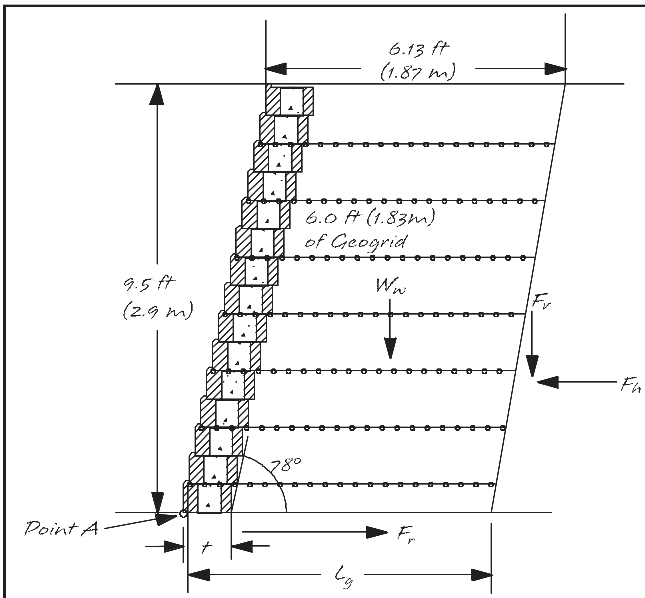


Figure 2-8. Freebody Diagram of a Coherent Wall

Bearing Pressure on the Underlying Soil

Another consideration in the design of a coherent gravity wall is the ability of the underlying soil to support the weight of a giant gravity wall. Most undisturbed soils can withstand pressures between 2,500 (120 kPa) and 4,000 (192 kPa) pounds per square foot.

Figure 2-9 is a freebody diagram of the coherent gravity wall in Example 2-3. It shows the forces acting on the wall. With this information, the maximum bearing pressure can be calculated and compared to the allowable bearing pressure.

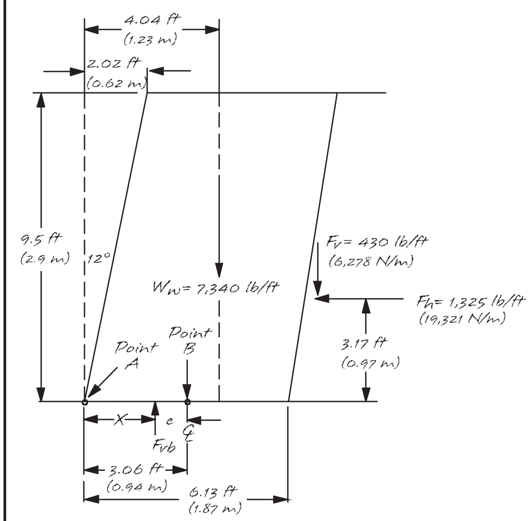


Figure 2-9. Freebody Diagram for Bearing Pressure Analysis

The first step is to calculate the resultant vertical resisting force, F_{vb}, exerted on the gravity wall by the soil:

$$F_{vb} = \Sigma F_y = W_w + F_v$$
$$= (7{,}340 \text{ lb/ft} + 430 \text{ lb/ft}) \qquad = 107{,}237 \text{ N/m} + 6{,}278 \text{ N/m}$$
$$= 7{,}770 \text{ lb/ft} \qquad = 113{,}515 \text{ N/m}$$

The next step is to locate the point of application of the resultant force. This is done by summing moments around Point A, setting the result equal to zero, and solving for X.

$$\Sigma M_A = (F_{vb})(X) + (F_h)(1/3 \text{ H}) - W_w (4.04 \text{ ft}) - F_v (6.13 \text{ ft})$$
$$= (7{,}770 \text{ lb/ft})(X) + (1{,}325 \text{ lb/ft})(3.17 \text{ ft})$$
$$- (7{,}340 \text{ lb/ft})(4.04 \text{ ft}) - (430 \text{ lb/ft})(6.13 \text{ ft})$$

$$X = \frac{(29{,}654 \text{ ft-lb/ft}) + (2{,}636 \text{ ft-lb/ft}) - (4{,}200 \text{ ft-lb/ft})}{(7{,}770 \text{ lb/ft})} = 3.62 \text{ ft}$$

$$= (113{,}515 \text{ N/m})(X) + (19{,}321 \text{ N/m})(0.966 \text{ m})$$
$$- (107{,}237 \text{ N/m})(1.23 \text{ m}) - (6{,}278 \text{ N/m})(1.87 \text{ m})$$
$$= \frac{(131{,}902 \text{ N-m/m}) + (11{,}740 \text{ N-m/m}) - (18{,}664 \text{ N-m/m})}{(113{,}515 \text{ N/m})} = 1.10 \text{ m}$$

The eccentricity, e, of the resultant vertical force, is the distance from the centerline of bearing of the gravity wall to the point of application of the resultant force, F_{vb}. In this case:

$$e = (0.5)(L_t) - X$$
$$= (0.5)(6.13 \text{ ft}) - X \qquad = (0.5)(1.87 \text{ m}) - X$$
$$= (0.5)(6.13 \text{ ft}) - 3.62 \text{ ft} = -0.56 \text{ ft} \qquad = (0.5)(1.87 \text{ m}) - 1.10 \text{ m} = -0.165 \text{ m}$$

In this case the eccentricity is negative. A negative eccentricity means that the wall mass is rolling backwards, thus causing a decrease in bearing pressure at the toe. Since this is not practical, “e” shall always be conservatively taken as greater than or equal to zero.

$$e = 0 \text{ ft} \qquad = 0 \text{ m}$$

Assuming a linear bearing pressure distribution, the average bearing pressure occurs at the centerline of the wall. Its magnitude is:

$$\sigma_{avg} = \frac{F_{vb}}{L_t} = \frac{(7{,}770 \text{ lb/ft})}{(6.13 \text{ ft})} = 1{,}268 \text{ lb/sq ft} \qquad = \frac{F_{vb}}{L_t} = \frac{113{,}515 \text{ N/m}}{1.87 \text{ m}\ (1000)} = 61 \text{ kPa}$$

Next, the bearing pressure due to the moment about the centerline of bearing is calculated. This is done by finding the moment due to the resultant vertical force about the centerline of bearing (Point B) and dividing it by the section modulus of a horizontal section through gravity wall. The moment due to the eccentricity of the resultant vertical force is:

$M_B = (F_{vb})(e)$
$= (7{,}770 \text{ lb/ft})(0 \text{ ft}) \qquad = (113{,}515 \text{ N/m})(0 \text{ m})$
$= 0 \text{ ft-lb/ft} \qquad = 0 \text{ N-m/m}$

The section modulus of a 1-foot or 1-meter wide section of the wall is given by:

$$S = \frac{(l)(L_t)^2}{6}$$

Where:

l = the width of the section = 1.0 ft or 1 m
L_t = the depth of the section = L_t = 6.13 ft (1.87 m)

$$S = \frac{(1 \text{ ft})(6.13 \text{ ft})^2}{6} \qquad = \frac{(1.0 \text{ m})(1.87 \text{ m})^2}{6}$$
$$= 6.26 \text{ ft}^3 \qquad = 0.583 \text{ m}^3$$

The difference in stress due to the eccentricity is:

$$\sigma_{mom} = \frac{M_B}{S}$$
$$= \frac{(0 \text{ ft-lb/ft})}{(6.26 \text{ ft}^3)} \qquad = \frac{(0 \text{ N-m/m})}{(0.583 \text{ m}^3)(1000)}$$
$$= 0 \text{ lb/ft}^2 \qquad = 0 \text{ kPa}$$

Finally, the maximum and minimum bearing pressures are calculated:

$\sigma = \sigma_{avg} \pm \sigma_{mom}$

$\sigma_{max} = \sigma_{avg} + \sigma_{mom}$
$= (1{,}268 \text{ lb/sq ft}) + (0 \text{ lb/sq ft}) \qquad = (61 \text{ kPa}) + (0 \text{ kPa})$
$= 1{,}268 \text{ lb/sq ft} \qquad = 61 \text{ kPa} = 6{,}100 \text{ kg/m}^2$

$\sigma_{min} = \sigma_{avg} - \sigma_{mom}$
$= (1{,}268 \text{ lb/sq ft}) - (0 \text{ lb/sq ft}) \qquad = (61 \text{ kPa}) - (0 \text{ kPa})$
$= 1{,}268 \text{ lb/sq ft} \qquad = 61 \text{ kPa} = 6{,}100 \text{ kg/m}^2$

If the maximum bearing pressure was greater than the allowable bearing pressure of 2,500 lb/sq ft (120 kPa), the wall would be unstable with respect to the allowable bearing capacity of the underlying soil.

The procedure outlined above can be simplified by rearranging the equations as follows:

$$\sigma = \sigma_{avg} \pm \sigma_{mom}$$

$$\sigma = \frac{F_{vb}}{L_t} \pm \frac{M_b}{S} = \frac{F_{vb}}{L_t} \pm \frac{(6)M_b}{L_t^2} = \frac{F_{vb}}{L_t} \pm \frac{(6)(F_{vb})(e)}{L_t^2}$$

When the maximum bearing pressure is greater than the allowable bearing pressure the underlying soil is not stable. Stabilizing the soil under the wall is accomplished by spreading the forces of the wall over a larger area. Engineers use this concept in designing spread footings.

Once the σ_{max} is determined, compare it to ultimate bearing capacity (q_f) as defined by Terzaghi:

$$q_f = (½)(\gamma_f)(B_b)(N_\gamma) + (c)(N_c) + (\gamma_f)(D)(N_q)$$

(Craig, p. 303, Soil Mechanics, Fifth Edition)

Where:

N_q = Contribution due to entire pressure (Terzaghi's value)
N_c = Contribution due to constant component of shear strength (Terzaghi's value)
N_γ = Contribution from self weight of the soil (Meyerhof's value)

N_q = $\exp(\pi \tan \phi_f) \tan^2 (45 + \phi_f/2)$
N_c = $(N_q - 1) \cot \phi_f$
N_γ = $(N_q - 1) \tan (1.4\phi_f)$

γ_f = Unit weight of foundation soils
D = Depth of wall embedment
= Buried block + footing thickness (d_b).
c = Cohesion of foundation soils
B_b = Width of the foundation
ϕ_f = Friction angle of foundation soils

NOTE: The Terzaghi values do not take into account the rectangular footing and eccentric loads. Using the Meyerhof equations to modify these parameters will include these affects.

The ultimate bearing (q_f) should be designed to a factor of safety of 2.0

If $SFB = \dfrac{q_f}{\sigma_{max}} < 2.0,$ then increase the size of the base.

The material in the base will usually be a select gravel, ϕ_B = 36°. However, the foundation soil below the base material is native soil and assume for this example to be ϕ_f = 30°.

$\tan (45 - \phi/2) = 0.5$ ft/W	$\tan (45 - \phi/2) = 0.15$ m/W
W = 0.5 ft / tan (45 − 30°/2)	W = 0.15 m / tan (45 − 30°/2)
W = 0.87 ft use 1.0 ft	W = 0.26 m use 0.3 m

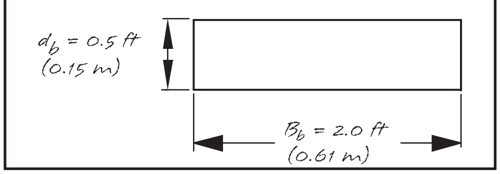
Therefore, the incremental base size is:

Depth (d_i) = (d_b − 1) + 0.5 ft = (d_b − 1) + 0.15 m

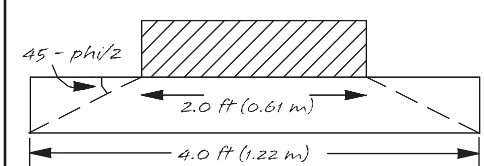
Width (B_i) = (B_b − 1) + (2) (W)
= (B_b − 1) + (2) (1 ft) = (B_B − 1) + (2) (0.3 m)

The toe extension will be equal to the footing depth.

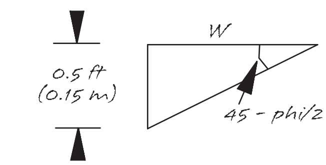
Minimum Base Size:

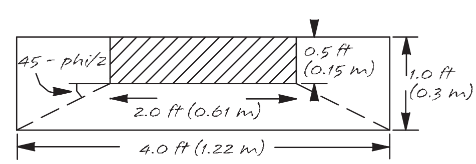


Increment to next size:

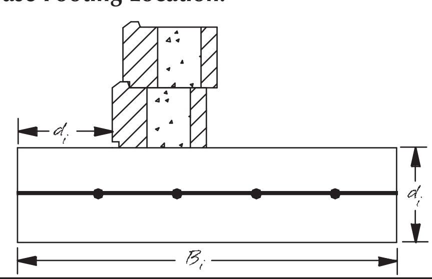


Increase Width by:





Base Footing Location:



Internal Stability

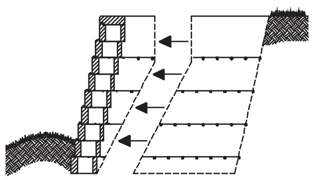
Allan Block recommends no more than 2-course spacing -16 in. (406 mm) - between each layer of geogrid reinforcement for any Allan Block system to ensure that the wall acts as a coherent mass.

The load on each layer of geogrid is equal to the average pressure on the wall section, P_{avg}, multiplied by the height of the section, d_h, (Figure 2-10). The pressure at any depth is given by:

$$P_v = (\gamma_i)\,(\text{depth})\,(K_{ai})\cos(\phi_{wi})$$

Internal Stability

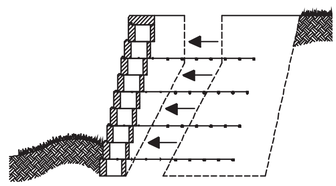
Internal stability is the ability of the reinforcement combined with the internal strength of the soil to hold the soil mass together and work as a single unit.



Grid Rupture

Rupture occurs when excessive forces from the retained soil mass exceed the ultimate tensile strength of the geogrid.

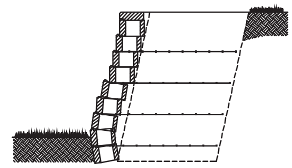
Increase grid strength



Pullout

Pullout results when grid layers are not embedded a sufficient distance beyond the line of maximum tension.

Increase embedment length



Bulging

Bulging occurs when horizontal forces between the geogrid layers causes localized rotation of the wall. Refer to Chapter Six for detailed analysis.

Increase number of grid layers

The load on each layer of grid is given by:

$$F_g = (P_{avg})\,(d_h)$$

where:

$$\begin{aligned} P_{avg} &= (0.5)\,(P_{base} + P_{top}) \\ &= (0.5)\,[(\gamma_i)\,(d_1)\,(K_{ai})\cos(\phi_{wi}) \\ &\quad + (\gamma_i)\,(d_2)\,(K_{ai})\cos(\phi_{wi})] \\ &= (0.5)\,(\gamma_i)\,(K_{ai})\cos(\phi_{wi})\,(d_1 + d_2) \end{aligned}$$

$d_h = d_1 - d_2$

d_1 = distance from the top of the backfill to the bottom of the zone supported by the layer of geogrid.

d_2 = distance from the top of the backfill to the top of the zone supported by the layer of geogrid.

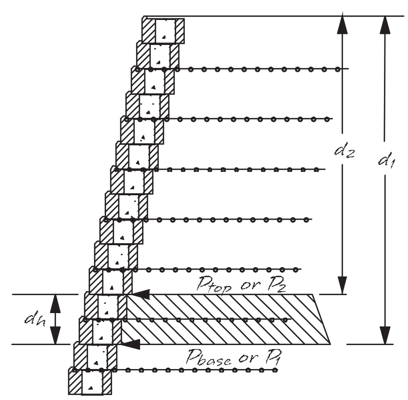
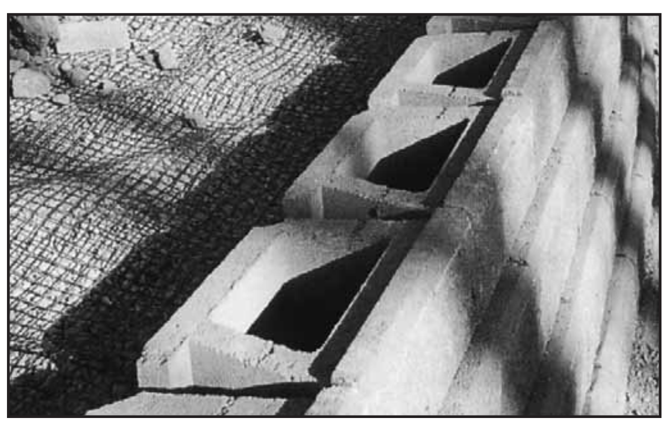


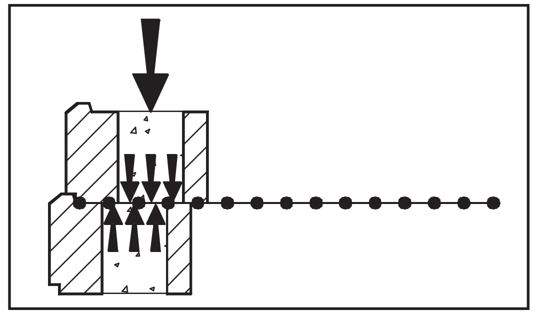
Figure 2-10. Load Distribution on Specific Grid Layers

Geogrid can only be placed between the blocks forming the wall facing. That means that the geogrid can only be placed at heights evenly divisible by the block height, this example is 7.62 inches or 0.635 ft (194 mm).



Attachment of the Geogrid to the Wall Facing

A logical question to ask is: What keeps the geogrid from slipping out from between the courses of Allan Block? The answer is that the weight of the Allan Blocks sitting on top of the geogrid creates friction between the blocks and the geogrid. In addition, some of the material used to fill the voids in the Allan Blocks becomes wedged in the apertures of the geogrid. This is called *Rock-Lock* and results in additional resistance to sliding.



Allan Block's original pullout tests were conducted in 1989 at the University of Wisconsin-Platteville by Kliethermes, et al. Two sets of tests were run. In the first set, the voids of the Allan Blocks were filled with gravel. In the second set, the voids were left empty.

When the voids were filled with gravel, there was an apparent *coefficient of friction* (ACF) of about 3.0 between the geogrid and the Allan Blocks. When the voids were left empty, the ACF was about 0.88. The surprising magnitude of the ACF for gravel is due to a significant amount of interlocking between the gravel and the geogrid.

The hollow core, pinless design of Allan Block raises questions on how the geogrid is attached to the wall facing. Allan Block's gravel filled hollow core provides a multi-point interlock with the grid. As wall heights increase, our exclusive "rock lock" connection, combined with the weight of the Allan Block units, provides a more uniform block-to-grid interlock than any system on the market.

Allan Block had additional pullout tests conducted at the University of Washington in 1993-1994. A total of ten geogrids and two geofabrics were tested. Each product was tested three times under four loading conditions; 500 lbs. (226.8 kg), 1000 lbs. (453.6 kg), 1500 lbs. (680.4 kg), and 2000 lbs. (907.18 kg) vertical load per lineal foot of wall. The data compiled was consistent. From a total of 144 pullout tests, the results exhibited a uniform behavior based on grid strengths and normal loads applied. The test values increased with added vertical loads. A typical pullout equation for service and ultimate loads takes the form X + Y * N. The variables X and Y are constant values as determined by testing. The normal (vertical) load N, is load applied to the block. The location of the block to grid connection will be the determining factor for the amount of normal (vertical) load applied. Appendix B has a thorough discussion on the current ASTM connection methodology and a complete table of current tested connection values with a large variety of geogrids.

The maximum force in the geogrid occurs at the Line of Maximum Tension - the boundary between the active and passive zones of the retained soil. The force on the geogrid decreases as the horizontal distance from the failure plane increases. At the back of the wall, the force on the geogrid is reduced to about two-thirds of the maximum force (McKittrick, 1979). As a result there is a 0.667 reduction factor for the load at the face (RF_{LF}).

The static geogrid/block connection capacity factor of safety is determined by comparing the peak connection strength, which is a function of the normal load, to the applied load on each layer of geogrid. Find the factors of safety for the static geogrid/block connection capacity:

$$SF_{conn} = \frac{F_{cs}}{(F_{gTopLayer})(RF_{LF})} \geq 1.5$$

The peak connection strength (F_{cs}) is an equation of a line generated by comparing the maximum pullout force under various normal loads. The numbers in this example are based on testing done with Allan Block and Fortrac 35/20-20 geogrid. The resulting equation for F_{cs} is:

$$F_{cs} = 1{,}313 \text{ lb/ft} + \tan(8°)(N) \qquad = 19{,}204 \text{ N/m} + \tan(8°)(N)$$

Where the normal load (N) is:

$$N = (H - \text{grid elev})(\gamma_{wall})(t)$$

$$= (9.52 \text{ ft} - 6.35 \text{ ft})(130 \text{ lb/ft}^3)(0.97 \text{ ft}) \qquad = (2.9 \text{ m} - 1.94 \text{ m})(2{,}061 \text{ kg/m}^3)(0.30 \text{ m})(9.81)$$

$$= 400 \text{ lb/ft} \qquad = (5{,}822 \text{ N/m})$$

Therefore, the peak connection strength (F_{cs}) is:

$$F_{cs} = 1{,}313 \text{ lb/ft} + \tan(8°)(400 \text{ lb/ft})$$

$$= 1{,}313 \text{ lb/ft} + 0.140 (85 \text{ lb/ft}) = 1{,}369 \text{ lb/ft} \qquad = 19{,}204 \text{ N/m} + 0.140 (5{,}822 \text{ N/m}) = 20{,}019 \text{ N/m}$$

$$SF_{conn} = \frac{1{,}369 \text{ lb/ft}}{360 \text{ lb/ft} (0.667)} = 5.7 \geq 1.5 \qquad = \frac{20{,}019 \text{ N/m}}{5{,}822 \text{ N/m}} = 5.7 \geq 1.5$$

Example 2-5a

Let's analyze the wall of Example 2-3 for geogrid pullout from blocks. Diagram Ex. 2-5a shows the wall and some of the dimensions that will be needed in the calculations. Calculate the horizontal force on the bottom layer of geogrid:

$$Ph_1 = (\gamma_i)\,(K_{ai})\,(d_1)\,(\cos\phi_{wi})$$
$$= (125\ \text{lb/ft}^3)\,(0.2197)\,(9.52\ \text{ft})\,(0.940)$$
$$= 246\ \text{lb/ft}^2$$

$$= (2{,}002\ \text{kg/m}^3)\,(0.2197)\,(2.9\ \text{m})\,(0.940)$$
$$= 1{,}200\ \text{kg/m}^2$$

$$Ph_2 = (\gamma_i)\,(K_{ai})\,(d_2)\,(\cos\phi_{wi})$$
$$= (125\ \text{lb/ft}^3)\,(0.2197)\,(8.25\ \text{ft})\,(0.940)$$
$$= 213\ \text{lb/ft}^2$$

$$= (2{,}002\ \text{kg/m}^3)\,(0.2197)\,(2.51\ \text{m})\,(0.940)$$
$$= 1{,}038\ \text{kg/m}^2$$

$$P_{avg} = (0.5)\,(246\ \text{lb/ft}^2 + 213\ \text{lb/ft}^2)$$
$$= 230\ \text{lb/ft}^2$$

$$= (0.5)\,(1{,}200\ \text{kg/m}^2 + 1{,}038\ \text{kg/m}^2)$$
$$= 1{,}119\ \text{kg/m}^2$$

$$F_1 = P_{avg}\,(d_h)$$
$$= (230\ \text{lb/ft}^2)\,(1.27\ \text{ft})$$
$$= 292\ \text{lb/ft}$$

$$= (1{,}119\ \text{kg/m}^2)\,(0.39\ \text{m})\,(9.81\ \text{m/sec}^2)$$
$$= 4{,}281\ \text{N/m}$$

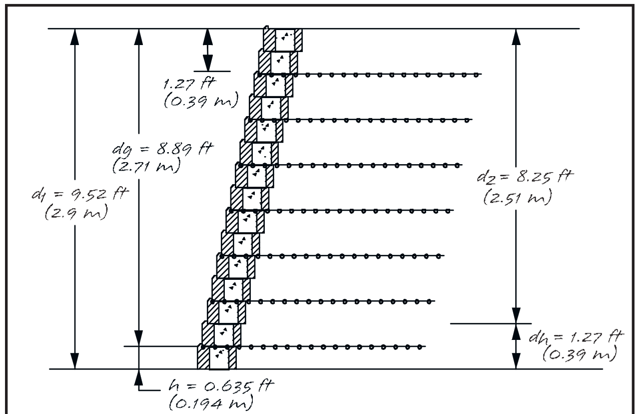


Diagram Ex. 2-5a

The force on the geogrid at the back face of the wall will be approximately two-thirds of F_1:

$$F_w = (0.667)\,(F_1) = (0.667)\,(292\ \text{lb/ft})$$
$$= 195\ \text{lb/ft}$$

$$= (0.667)\,(F_1) = (0.667)\,(4{,}281\ \text{N/m})$$
$$= 2{,}854\ \text{N/m}$$

The normal load is:

$$N_1 = (130\ \text{lb/ft}^3)\,(0.97\ \text{ft})\,(8.89\ \text{ft}) = 1{,}121\ \text{lb/ft}$$
$$= (2{,}082\ \text{kg/m}^3)\,(0.3\ \text{m})\,(2.71\ \text{m})\,(9.81\ \text{m/sec}^2) = 16{,}605\ \text{N/m}$$

Using Fortrac 35/20-20 equation from Table B-1, Page 68 for pullout of block resistance:

$$F_{cs} = 1{,}313\ \text{lb/ft} + \tan(8°)\,(1{,}121\ \text{lb/ft})$$
$$= 1{,}313\ \text{lb/ft} + 0.140\,(1{,}121\ \text{lb/ft})$$
$$= 1{,}470\ \text{lb/ft}$$

$$= 19{,}204\ \text{N/m} + 0.140\,(16{,}605\ \text{N/m})$$
$$= 21{,}529\ \text{N/m}$$

The safety factor against pullout of block for the bottom layer of geogrid is:

$$SF_{conn} = \frac{F_{cs}}{F_w} = \frac{(1{,}470\ \text{lb/ft})}{195\ \text{lb/ft})} = 7.5 \geq 1.5$$

$$= \frac{(21{,}529\ \text{N/m})}{(2{,}854\ \text{N/m})} = 7.5 \geq 1.5$$

Example 2-5b

The horizontal force on the top layer of geogrid is:

$P_{h4} = (\gamma)\ (K_a)\ (d_4)\ (\cos \phi_{wi}) = (125\ \text{lb/ft}^3)\ (0.2197)\ (1.91\ \text{ft})\ (0.940) = 49\ \text{lb/ft}^2$

$= (\gamma)\ (K_a)\ (d_4)\ (\cos \phi_{wi}) = (2{,}002\ \text{kg/m}^3)\ (0.2197)\ (0.58\ \text{m})\ (0.940) = 240\ \text{kg/m}^2$

$P_{h5} = (\gamma)\ (K_a)\ (d_5)\ (\cos \phi_{wi}) = (125\ \text{lb/ft}^3)\ (0.2197)\ (0\ \text{ft})\ (0.940) = 0\ \text{lb/ft}^2$

$= (\gamma)\ (K_a)\ (d_5)\ (\cos \phi_{wi}) = (2{,}002\ \text{kg/m}^3)\ (0.2197)\ (0\ \text{m})\ (0.940) = 0\ \text{kg/m}^2$

$P_{avg} = (0.5)\ (49\ \text{lb/ft}^2 + 0\ \text{lb/ft}^2) = 25\ \text{lb/ft}^2$ $= (0.5)\ (240\ \text{kg/m}^2 + 0\ \text{kg/m}^2) = 120\ \text{kg/m}^2$

$F_4 = (P_{avg})\ (d_h) = (25\ \text{lb/ft}^2)\ (1.91\ \text{ft}) = 47\ \text{lb/ft}$

$= (P_{avg})\ (d_h) = (120\ \text{kg/m}^2)\ (0.58\ \text{m})\ (9.81\ \text{m/sec}^2) = 683\ \text{N/m}$

The force on the geogrid at the back face of the wall will be approximately two-thirds of F_4:

$F_w = (0.667)\ (F_4) = (0.667)\ (47\ \text{lb/ft}) = 31\ \text{lb/ft}$ $= (0.667)\ (F_4) = (0.667)\ (683\ \text{N/m}) = 455\ \text{N/m}$

The force resisting pullout, caused by the weight of the aggregate filled blocks above the top geogrid layer, is:

$N_4 = (130\ \text{lb/ft}^3)\ (0.97\ \text{ft})\ (1.27\ \text{ft}) = 160\ \text{lb/ft}$ $= (2{,}082\ \text{kg/m}^3)\ (0.3\ \text{m})\ (0.39\ \text{m})\ (9.81\ \text{m/sec}^2) = 2{,}390\ \text{N/m}$

$F_{cs} = 1{,}313\ \text{lb/ft} + 0.140\ (160\ \text{lb/ft}) = 1{,}335\ \text{lb/ft}$ $= 19{,}204\ \text{N/m} + 0.140\ (2{,}390\ \text{N/m}) = 19{,}539\ \text{N/m}$

The safety factor against pullout of block for the top layer of geogrid is:

$$SF_{conn} = \frac{F_{cs}}{F_w} = \frac{(1{,}335\ \text{lb/ft})}{(31\ \text{lb/ft})} = 43.0 \geq 1.5$$

$$= \frac{(19{,}539\ \text{N/m})}{(455\ \text{N/m})} = 43.0 \geq 1.5$$

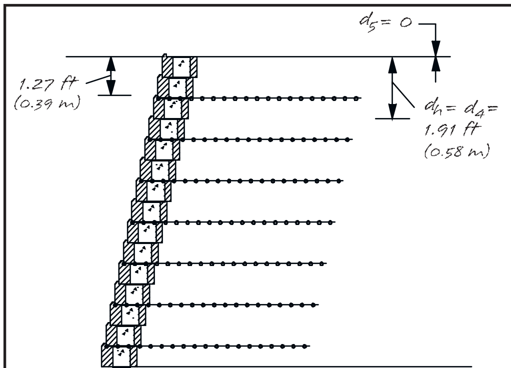


Diagram Ex. 2-5b

At a certain depth, the force holding the geogrid between the blocks will be equal to or greater than the long-term allowable design load of the geogrid. Any layer of geogrid located below this depth may be controlled by tensile overstress and not connection. The depth will be different for each wall depending on the type of soil, the slope of the backfill, and the presence of surcharges, if any.

Mechanical Connection

A grouted / mechanical connection may be desirable in special circumstances such as for geogrid layers under high seismic loading or when barriers are attached. The hollow cores of the Allan Block provide for a cell to encapsulate the geogrid placed between block courses. When a grouted connection is specified, a minimum of 3 inches (7.6 cm) of grout above and below the grid layers is required. Factors of safety for this connection are determined by comparing the long-term allowable design strength (LTADS) of the geogrid to the applied load at the face. Please note that designers using a grouted connection should verify with the geogrid manufacturer that their grids are allowed in areas of high alkaline content.

$$SF_{mech} = \frac{\text{LTADS}}{(\text{Applied Load})\ (RF_{LF})}$$

Example 2-4: **(15 course wall)**

Given:

$H = 9.52$ ft (2.9 m)	LTADS $= 1322$ lb/ft (19,300 N/m)
$\phi = 30°$	$L_t = 6.13$ ft (1.87 m)
$\gamma = 120$ lb/ft^3 (1,923 kg/m^3)	Geogrid Courses = 3, 6, 9, 12
$\gamma_{wall} = 130$ lb/ft^3 (2,061 kg/m^3)	$\phi_{wi} = 20°$

From Example 2-3: $F_{is} = F_{gTopLayer} = P_{avg}\ (d_h)$ for this example $F_{gTopLayer} = 360$ lb/ft (5,256 N/m).

Geogrid Pullout from the Soil

Geogrid extends into the backfill soil and the frictional resistance due to the weight of the soil on top of the geogrid provides the restraining force. The relationship can be expressed as follows:

F_{gr} = (Unit weight of soil) x (Depth to grid)
x (2) x (Length of the grid in the passive zone)
x (Coefficient of friction)

The following equation can be used to calculate the maximum potential restraining force:

$$F_{gr} = (2)\,(d_g)\,(\gamma_i)\,(L_e)\,(C_i)\tan(\phi)$$

where:

d_g = the depth from the top of the infill to the layer of geogrid.

γ_i = the unit weight of the infill soil.

L_e = the length of geogrid embedded in the passive zone of the soil.

C_i = the coefficient of interaction between the soil and the geogrid, a measure of the ability of the soil to hold the geogrid when a force is applied to it. Typical values of C_i are 0.9 for gravelly soil, 0.85 for sand or silty sands, and 0.75 for silts and clays.

$\tan(\phi)$ = the coefficient of friction (shear strength) between adjacent layers of soil.

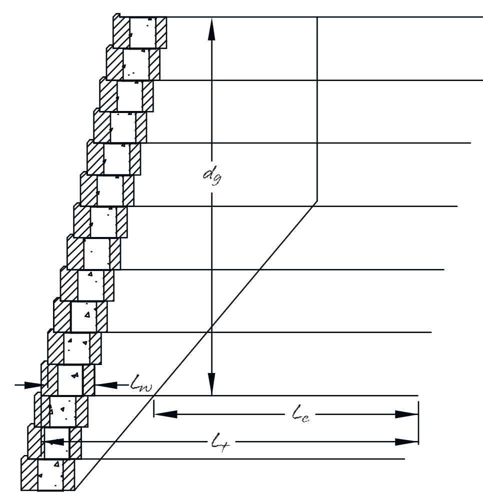


Diagram Ex. 2-5c

The factor 2 is used because both the top and the bottom of the geogrid interact with the soil.

NOTE: Typically a designer will use a grid length of 60% of wall height, run the Safety Factor for Pullout of Soil calculations, and lengthen the grid if necessary. The following steps can be taken as a check to find the minimum grid lengths required to meet the pullout of soil requirements.

First, the depth to the geogrid, d_g, must be specified. To complete Example 2-5a, let d_g = 8.89 ft (2.71 m). Another important assumption is that the geogrid will extend far enough into the passive zone to develop the full allowable design strength of the geogrid. In this case an average strength geogrid will be used, the full long-term allowable load is 1,322 lb/ft (19,300 N/m). A safety factor of 1.5 is applied to this value. The embedment length required to generate that force can be calculated as follows:

$$L_e = \frac{\text{LTADS}}{(F_{gr})\,(SF_{pulloutsoil})}$$

$$F_{gr} = (2)\,(d_g)\,(\gamma_i)\,(L_e)\,(C_i)\tan(\phi)$$

$$L_e = \frac{\text{LTADS}}{(2)\,(d_g)\,(\gamma_i)\,(L_e)\,(C_i)\tan(\phi)\,(SF_{pulloutsoil})}$$

$$= \sqrt{\frac{(1{,}322\ \text{lb/ft})}{(2)\,(8.89\ \text{ft})\,(120\ \text{lb/ft}^3)\,(0.85)\tan(30°)\,(1.5)}} = 0.92\ \text{ft}$$

$$= \sqrt{\frac{(19{,}300\ \text{N/m})}{(2)\,(2.71\ \text{m})\,(1{,}923\ \text{kg/m}^3)\,(9.81\ \text{m/sec}^2)\,(0.85)\tan(30°)\,(1.5)}} = 0.28\ \text{m}$$

The total length of geogrid required per linear foot of wall is:

$$L_t = L_w + L_a + L_e$$

where:

L_w = length of geogrid inside the Allan Block unit = 0.84 ft (0.26 m)
= Block thickness - Equivalent lip thickness

L_a = length of geogrid in the active zone

$$= (H - d_g)\left[\tan(45° - \phi/2) - \tan(90° - \beta)\right] = 0.23 \text{ ft} \quad (0.07 \text{ m})$$

L_e = length of geogrid embedded in the passive zone.

The estimated total length of geogrid required for the wall in Example 2-5 is:

$$L_t = 0.84 \text{ ft} + 0.23 \text{ ft} + 0.84 \text{ ft} = 1.91 \text{ ft}$$

$$= 0.26 \text{ m} + 0.07 \text{ m} + 0.26 \text{ m} = 0.59 \text{ m}$$

Standard practice for design is to use a minimum geogrid length of 60% of the wall height. For this example, L_t = 6.0 ft (1.83 m).

With a total geogrid length of 6.0 ft (1.83 m) the actual embedment length is:

$$L_e = L_t - L_w - L_a$$

$$= 6.0 \text{ ft} - 0.84 \text{ ft} - 0.23 \text{ ft} = 4.93 \text{ ft}$$

$$= 1.83 \text{ m} - 0.26 \text{ m} - 0.07 \text{ m} = 1.5 \text{ m}$$

The maximum potential restraining force on the geogrid for an embedment length of 4.93 feet (1.50 m) is:

$$F_{gr} = (2)(8.89 \text{ ft})(120 \text{ lb/ft}^3)(4.93 \text{ ft})(0.85)\tan(30°) = 5{,}162 \text{ lb/ft}$$

$$= (2)(2.71 \text{ m})(1{,}923 \text{ kg/m}^3)(1.50 \text{ m})(0.85)\tan(30°)(9.81 \text{ m/sec}^2) = 75{,}266 \text{ N/m}$$

However, the long-term allowable design load (LTADS) of the grid specified is only 1,322 lb/ft (19,300 N/m). The maximum restraining force must be less than or equal to the LTADS. Therefore, F_{gr} is limited to LTADS.

Studies have shown that the line of maximum tension for the soil inside the reinforced soil mass is not well represented by a straight line at an angle of 45° + ϕ/2 to the horizontal. Instead, the line of maximum tension looks more like the one depicted in Figure 2-11. It begins at the bottom rear edge of the wall facing and extends upward at an angle of 45° + ϕ/2 to the horizontal. The failure surface continues upward at that angle until it intersects a vertical line located behind the wall facing a distance equal to 0.3 the height of the wall.

When analyzing the loads on an individual layer of geogrid, the effective depth (dg) of grid is measured from the grid layer up to the geometric vertical center of the slope above. The geometric vertical center is easily calculated for both continuous and broken back slopes above the wall. If there is no slope above, it is measured to the top of the wall.

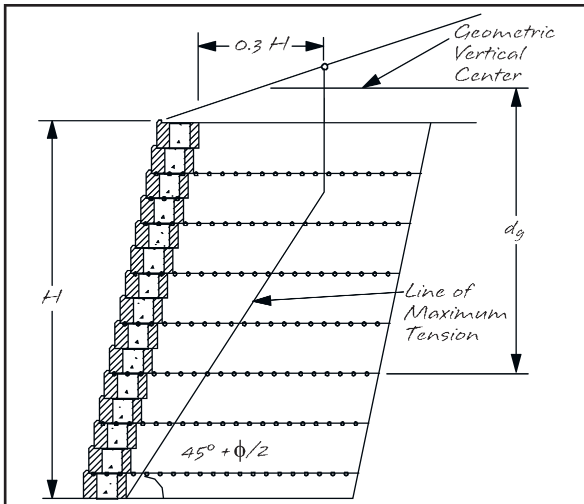


Figure 2-11. Line of Maximum Tension in a Coherent Gravity Wall

Chapter Three

Surcharges

Introduction

A *surcharge (q)* is an external load applied to the retained soil. Typical surcharges include: sidewalks, driveways, roads, buildings, and other retaining walls. Retaining walls as surcharges will be dealt with in a separate section entitled "Tiered Walls." In this chapter, we will show how to apply the force due to surcharges on simple gravity walls and coherent gravity walls.

The effect a surcharge has on a wall depends on the magnitude of the surcharge and the location of the surcharge relative to the wall. A surcharge located directly behind a wall will have a much greater effect than one located ten or twenty feet behind the wall. Generally, in good soil if the distance from the back of the wall to the surcharge is greater than twice the height of the wall, the effect of the surcharge will be insignificant. *Keep in mind that the back of a coherent gravity wall is located at the end of the geogrid furthest from the wall facing.*

In order to properly determine the effects of a surcharge load, it is necessary to determine how the stress within the soil varies with vertical and horizontal distance from the surcharge. There are several theories about how to calculate the stress at some point within the soil and they range from relatively simple to extremely complex. The one that we have chosen to use is illustrated in Figure 3-1. We assume that the force due to a surcharge load on the retained soil is transmitted downward through the soil at an angle of $45° + \phi/2$ to the horizontal. (ϕ is the friction angle of the soil.) The plane of influence can be approximated by drawing a line up from the bottom rear edge of the wall at an angle of $45° + \phi/2$ until it intersects the top of the backfill. Any surcharge located between the front of the wall and the point of intersection will have a measurable effect on the wall. Surcharges located beyond the point of intersection will have a minimal effect on the wall and will be neglected.

The nature of a surcharge can be defined as a live load or a dead load. Essentially, a live load is that which is transient in its influence on the wall structure and a dead load is that which is taken as a permanent influence on the wall structure. In our calculations for stability, a conservative approach is followed that does not include the presence of the vertical live load weight and vertical forces on the resistance side of the equation.

The location of the live or dead load surcharge, be it the retained soil or the infill soil, affects individual forces on the wall resulting in increased or decreased stability factors of safety. For example, a coherent gravity wall with a live load surcharge on the infill soil will act to decrease FOS overstress and also decrease FOS for sliding and overturning. If the live load surcharge is acting on the retained soil, we see decreases in FOS for sliding and overturning. As for a coherent gravity wall with a dead load surcharge on the infill soils, we see a decrease in FOS for overstress and an increase in FOS for sliding and overturning. If the dead load is on the retained soil, we see an increase in FOS for sliding and overturning.

Another assumption we make in analyzing a surcharge load is that the stress within the soil due to the surcharge is constant with depth. This assumption is fairly accurate for surcharges covering a large area and will result in an error on the conservative side while greatly simplifying the analysis. More exact methods of analysis are available and can be used if desired.

Assumptions:

1. Stress in Soil Due to Surcharge Does Not Vary with Depth.
2. Wall Friction is Neglected in this example.

where:

X_L = **the distance from the front of the top AB Unit to the surcharge.**

L_{PI} = **the distance from the front of the top AB Unit to the plane of influence.**

P_q = **the pressure due to the surcharge**

q = **the surcharge**

H_q = **height of wall effected by the surcharge**

Table 3-1. Effect of Uniform Surcharge on a Retaining Wall

CASE 1

$X_L = 0$

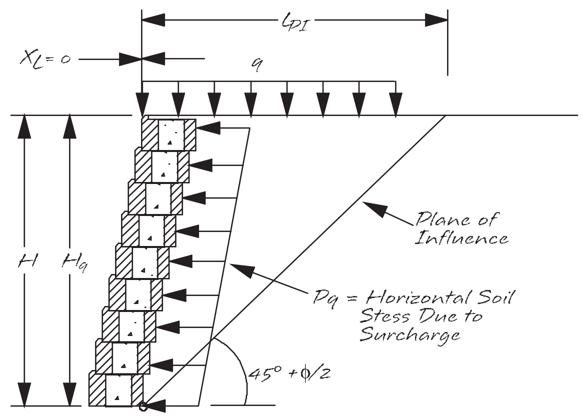
$P_q = (q)(K_a)$

Sliding Force:

$F_s = (P_q)(H)\cos(\phi_w)$

Overturning Moment:

$M_q = (0.5)(H)(F_s)$



CASE 2

$0 < X_L < L_{PI}$

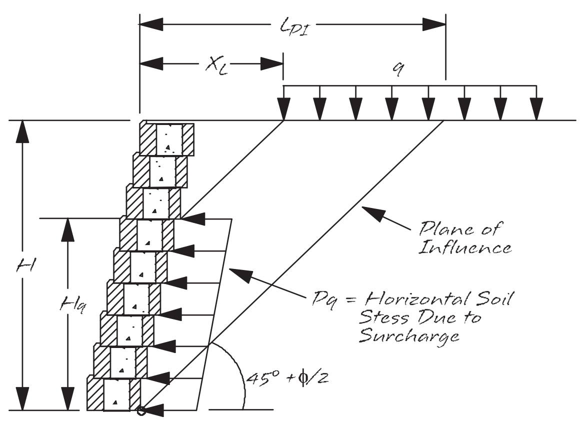
$P_q = (q)(K_a)$

Sliding Force:

$F_s = (P_q)(H_q)\cos(\phi_w)$

Overturning Moment:

$M_q = (0.5)(H_q)(F_s)$



CASE 3

$X_L \geq L_{PI}$

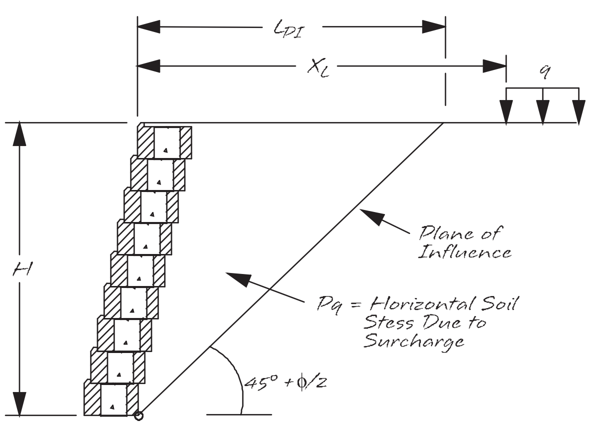
$P_q = 0$

Sliding Force:

$F_s = 0$

Overturning Moment:

$M_q = 0$



Surcharges on Simple Gravity Walls

Example 3-1:

Figure 3-1 shows the simple gravity wall of Example 2-1 with a uniform dead load surcharge *(q)* of 120 lb/ft^2 (6 kPa) behind it. The surcharge is 4 feet wide (1.22 m) and is located right next to the back of the wall. The first step in the analysis is to calculate the pressure on the retaining wall due to the surcharge:

P_q	$= (q)(K_a)$	
	$= (120\ lb/ft^2)(0.2197)$	$= (6\ kPa)(0.2197)$
	$= 26\ lb/ft^2$	$= 1.32\ kPa$

Again, because of the effects of friction between the wall and the soil, the pressure due to the surcharge has both a horizontal component and a vertical component. Therefore, the next step in the analysis is to calculate the horizontal and vertical components of the pressure:

P_{qh}	$= (P_q)\cos(\phi_w)$	
	$= (26\ lb/ft^2)\cos(20°)$	$= (1.32\ kPa)\cos(20°)$
	$= 24\ lb/ft^2$	$= 1.24\ kPa$
P_{qv}	$= (P_q)\sin(\phi_w)$	
	$= (26\ lb/ft^2)\sin(20°)$	$= (1.32\ kPa)\sin(20°)$
	$= 9\ lb/ft^2$	$= 0.45\ kPa$

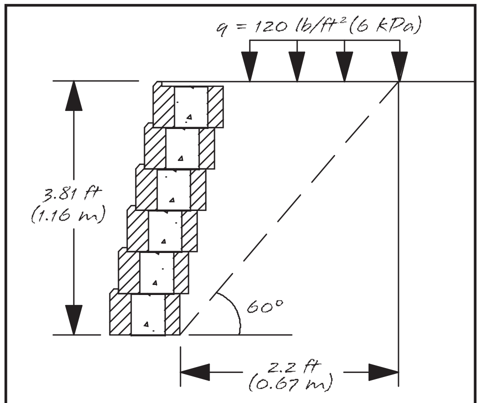


Figure 3-1. Simple Gravity Retaining Wall with Surcharge

Finally, the total surcharge forces on the wall are calculated:

F_{qh}	$= (P_{qh})(H)$	
	$= (24\ lb/ft^2)(3.81\ ft)$	$= (1.24\ kPa)(1.16\ m)$
	$= 91\ lb/ft$	$= 1.44\ kN/m$
F_{qv}	$= (P_{qv})(H)$	
	$= (9\ lb/ft^2)(3.81\ ft)$	$= (0.45\ kPa)(1.16\ m)$
	$= 34\ lb/ft$	$= 0.52\ kN/m$

Figure 3-2 is a freebody diagram showing the active forces on the wall. Now that the force and pressure distribution due to the surcharge are known, the wall can be analyzed as described in Chapter Two. (The rest of the forces have already been calculated in Example 2-1.) For a simple gravity wall, the horizontal force due to the surcharge is a force that tends to cause both sliding and overturning. Therefore, it must be added to those forces when the safety factors are calculated.

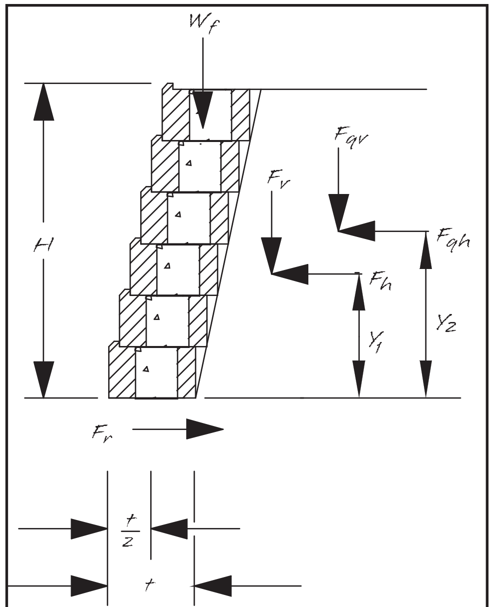


Figure 3-2. Freebody Diagram of a Simple Gravity Wall with Surcharge

The safety factor against sliding is:

$$\mathrm{SFS} = \frac{F_r + (F_{qv})(C_f)}{F_h + F_{qh}}$$

$$= \frac{(315\ \mathrm{lb/ft}) + (34\ \mathrm{lb/ft})\tan(30°)}{(179\ \mathrm{lb/ft}) + (91\ \mathrm{lb/ft})} = 1.23$$

$$= \frac{(4{,}613\ \mathrm{N/m}) + (520\ \mathrm{N/m})\tan(30°)}{(2{,}620\ \mathrm{N/m}) + (1{,}440\ \mathrm{N/m})} = 1.23$$

(**NOTE:** F_r and F_h were calculated in Example 2-1).

The safety factor against overturning is:

$$\Sigma M_r = (W_f)[(t/2) + (0.5)(H)\tan(90° - \beta)] + (F_v)[(t) + (0.333)(H)\tan(90° - \beta)] + (F_{qv})[(t) + (0.5)(H)\tan(90° - \beta)]$$

$$= (480\ \mathrm{lb/ft})[(0.49\ \mathrm{ft}) + (0.5)(3.81\ \mathrm{ft})\tan(90° - 78°)] + (65\ \mathrm{lb/ft})[(0.97\ \mathrm{ft}) + (0.333)(3.81\ \mathrm{ft})\tan(90° - 78°)] + (34\ \mathrm{lb/ft})[(0.97\ \mathrm{ft}) + (0.5)(3.81\ \mathrm{ft})\tan(90° - 78°)]$$

$$= 557\ \mathrm{ft\text{-}lb/ft}$$

$$= (7{,}036\ \mathrm{N/m})[(0.149\ \mathrm{m}) + (0.5)(1.16\ \mathrm{m})\tan(90° - 78°)] + (984\ \mathrm{N/m})[(0.3\ \mathrm{m}) + (0.333)(1.16\ \mathrm{m})\tan(90° - 78°)] + (520\ \mathrm{N/m})[(0.3\ \mathrm{m}) + (0.5)(1.16\ \mathrm{m})\tan(90° - 78°)]$$

$$= 2{,}512\ \mathrm{N\text{-}m/m}$$

$$\Sigma M_o = (F_h)(0.333)(H) + (F_{qh})(0.5)(H)$$

$$= (179\ \mathrm{lb/ft})(0.333)(3.81\ \mathrm{ft}) + (91\ \mathrm{lb/ft})(0.5)(3.81\ \mathrm{ft})$$

$$= 400\ \mathrm{ft\text{-}lb/ft}$$

$$= (2{,}620\ \mathrm{N/m})(0.333)(1.16\ \mathrm{m}) + (1{,}440\ \mathrm{N/m})(0.5)(1.16\ \mathrm{m})$$

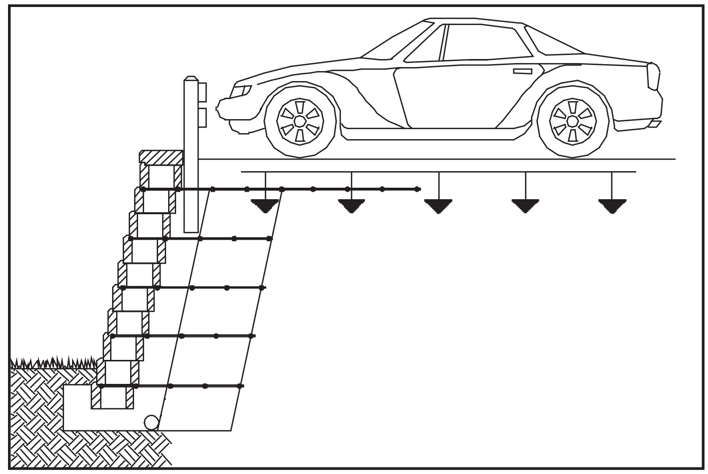
$$= 1{,}847\ \mathrm{N\text{-}m/m}$$

$$\mathrm{SFO} = \frac{\Sigma M_r}{\Sigma M_o} = \frac{(557\ \mathrm{ft\text{-}lb/ft})}{(400\ \mathrm{ft\text{-}lb/ft})} = 1.4 \qquad \frac{(2{,}512\ \mathrm{N\text{-}m/m})}{(1{,}847\ \mathrm{N\text{-}m/m})} = 1.4$$

Notice that with the surcharge on the backfill the safety factors are much lower than the recommended minimum values of 1.5 for sliding and 2.0 for overturning. This illustrates that a surcharge can make the difference between a stable wall and an unstable one.

Surcharges on Coherent Gravity Walls

Analyzing the effects of a surcharge on a coherent gravity wall is a two-part problem. First, the effect on the entire reinforced soil mass (external stability) must be analyzed. The surcharge will have an effect on both sliding failure and overturning failure. Second, the effect of the surcharge on the individual layers of geogrid (internal stability) must be analyzed. The surcharge will affect the stress in each layer of geogrid and will influence the spacing of the layers.



External Stability

The effect of a surcharge on the external stability of a coherent gravity retaining wall is nearly identical to the effect on a simple gravity wall and depends on the location of the surcharge. Recall that the back of a coherent gravity wall is located at the end of the geogrid farthest from the wall facing.

Figure 3-3 shows three possible locations of a dead load surcharge. The surcharge in Location A contributes to the forces resisting both sliding and overturning. Surcharges at location B contribute to the forces causing sliding and overturning. In Location C, the surcharge contributes partly to the forces causing sliding and partly to the forces resisting sliding. In the same manner, it also contributes both to the forces causing overturning and the forces resisting overturning.

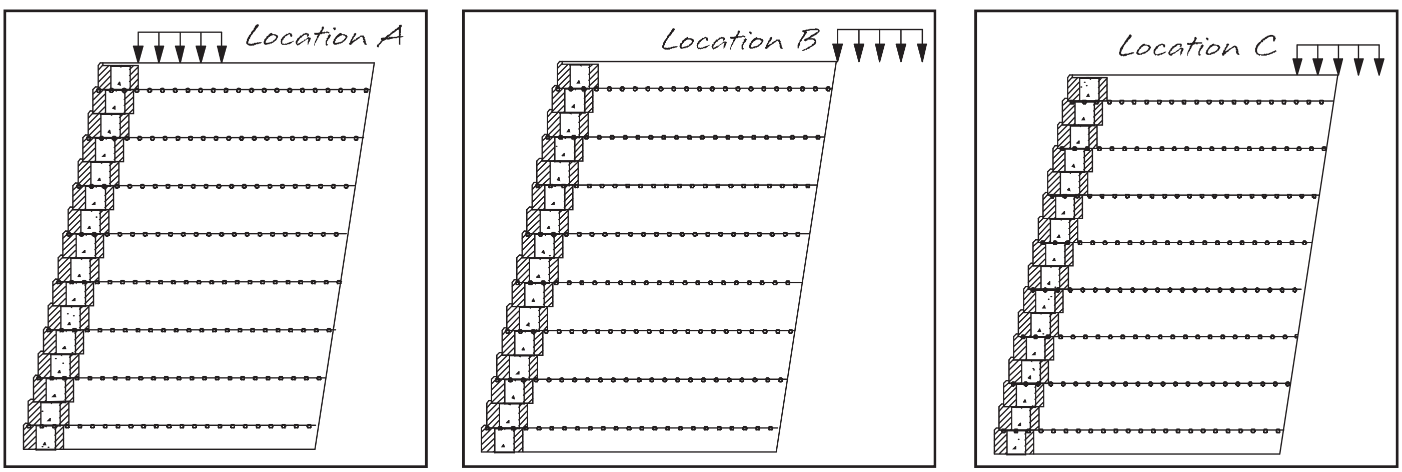


Figure 3-3. Locations of Surcharge on Coherent Gravity Walls

Example 3-3:

Consider the coherent gravity wall analyzed in Example 2-3, but with a three-foot-wide dead load surcharge of 120 lb/ft^2 (6 kPa). Analyze the external stability of the wall with the surcharge in the three locations shown in Figure 3-3.

Location A:

The surcharge can be resolved into an equivalent vertical force, Q, of 360 lb/ft (5,256 N/m) that is located 2.5 ft (0.762 m) from the front face of the wall and acts at the center of the uniform surcharge. This force can be added to the forces resisting sliding when calculating F_r:

F_r = $(W_w + F_v + Q)(C_f)$

= [(7,340 lb/ft) + (430 lb/ft) + (360 lb/ft)] tan (30°) = 4,694 lb/ft

= [(107,237 N/m) + (6,278 N/m) + (5,256 N/m)] tan (30°) = 68,572 N/m

The new safety factor against sliding is:

$$SFS = \frac{F_r}{F_h} = \frac{(4{,}694 \text{ lb/ft})}{(1{,}325 \text{ lb/ft})} = 3.54$$

$$= \frac{(68{,}572 \text{ N/m})}{(19{,}321 \text{ N/m})} = 3.54$$

Q can also be added to the moments of the forces resisting overturning:

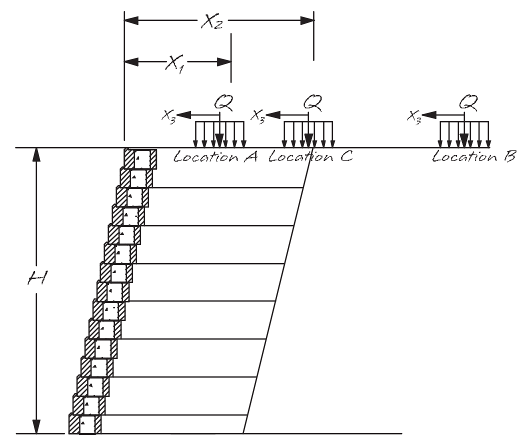


Diagram Ex. 3-3

where:

X_1 = distance to the center line of the reinforced mass
X_2 = distance to the back of the reinforced mass
X_3 = distance to the center line of the surcharge

$$\begin{aligned}
\Sigma M_r &= (W_w)\,[(X_1) + (0.5)\,(H)\tan(90° - \beta)] \\
&+ (F_v)\,[(X_2) + (0.333)\,(H)\tan(90° - \beta)] \\
&+ (Q)\,[(X_3) + (H)\tan(90° - \beta)] \\
&= (7{,}340 \text{ lb/ft})\,[(3.0 \text{ ft}) + (0.5)\,(9.52 \text{ ft})\tan(90° - 78°)] \\
&+ (430 \text{ lb/ft})\,[(6.\ 13 \text{ ft}) + (0.\ 333)\,(9.52 \text{ ft})\tan(90° - 78°)] \\
&+ (360 \text{ lb/ft})\,[(2.5 \text{ ft}) + (9.52 \text{ ft})\tan(90° - 78°)] \\
&= 34{,}000 \text{ ft-lb/ft}
\end{aligned}$$

$$\begin{aligned}
&= (107{,}237 \text{ N/m})\,[(0.91 \text{ m}) + (0.5)\,(2.9 \text{ m})\tan(90° - 78°)] \\
&+ (6{,}278 \text{ N/m})\,[(1.87 \text{ m}) + (0.\ 333)\,(2.9 \text{ m})\tan(90° - 78°)] \\
&+ (5{,}256 \text{ N/m})\,[(0.762 \text{ m}) + (2.9 \text{ m})\tan(90° - 78°)] \\
&= 150{,}912 \text{ N-m/m}
\end{aligned}$$

$$\begin{aligned}
\Sigma M_o &= (F_h)\,(0.333)\,(H) \\
&= (1{,}325 \text{ lb/ft})\,(0.333)\,(9.52 \text{ ft}) = 4{,}200 \text{ ft-lb/ft}
\end{aligned}$$

$$= (19{,}321 \text{ N/m})\,(0.333)\,(2.9 \text{ m}) = 18{,}658 \text{ N-m/m}$$

The new safety factor against overturning is:

$$SFO = \frac{\Sigma M_r}{\Sigma M_o} = \frac{(34{,}000 \text{ ft-lb/ft})}{(4{,}200 \text{ ft-lb/ft})} = 8.09$$

$$= \frac{(150{,}912 \text{ N-m/m})}{(18{,}658 \text{ N-m/m})} = 8.09$$

Thus, the effect of a surcharge in Location A is to make the wall slightly more stable with respect to sliding and overturning. However, such a surcharge can have a detrimental effect on the internal stability of the wall. Also, the added force due to the surcharge must be taken into account when calculating the bearing pressure on the underlying soil.

Location B:

A surcharge in this location has the same effect on the external stability of a coherent gravity wall as on a simple gravity wall. In this case, the surcharge results in a horizontal force with its point of application located at H/2 on the back of the reinforced soil mass. The magnitude of the force is:

$$\begin{aligned}
F_q &= (q)\,(K_a)\,(H) \\
&= (120 \text{ lb/ft}^2)\,(0.2561)\,(9.52 \text{ ft}) = 293 \text{ lb/ft}
\end{aligned}$$

$$= (5{,}748 \text{ Pa})\,(0.2561)\,(2.9 \text{ m}) = 4{,}269 \text{ N/m}$$

The horizontal and vertical components of the force on the reinforced soil mass due to the surcharge are:

$$\begin{aligned}
F_{qh} &= (F_q)\cos(\phi_{wr}) \\
&= (293 \text{ lb/ft})\cos(18°) = 279 \text{ lb/ft}
\end{aligned}$$

$$= (4{,}269 \text{ N/m})\cos(18°) = 4{,}060 \text{ N/m}$$

$$\begin{aligned}
F_{qv} &= (F_q)\sin(\phi_{wr}) \\
&= (293 \text{ lb/ft})\sin(18°) = 91 \text{ lb/ft}
\end{aligned}$$

$$= (4{,}269 \text{ N/m})\sin(18°) = 1{,}319 \text{ N/m}$$

Notice that the pressure coefficient for the onsite soil is used. This is because the surcharge is located entirely outside the reinforced soil zone and the surcharge force is transmitted through the onsite soil.

For Location B, the safety factors against sliding and overturning are:

$$SFS = \frac{F_r + (F_{qv})(C_f)}{F_h + F_{qh}}$$

$$= \frac{4{,}486 \text{ lb/ft} + 91 \text{ lb/ft} \tan 27°}{1{,}325 \text{ lb/ft} + 279 \text{ lb/ft}} = 2.83$$

$$= \frac{65{,}538 \text{ N/m} + 1{,}319 \text{ N/m} \tan 27°}{19{,}321 \text{ N/m} + 4{,}060 \text{ N/m}} = 2.83$$

$$SFO = \frac{\Sigma M_r + (F_{qv})[(X_2) + (0.5)(H)\tan(90° - \beta)]}{\Sigma M_o + (F_{qh})(0.5)(H)}$$

$$= \frac{34{,}000 \text{ ft-lb/ft} + 91 \text{ lb/ft}\,[6.13 \text{ ft} + (0.5)(9.52 \text{ ft})\tan(90° - 78°)]}{4{,}200 \text{ ft-lb/ft} + (279 \text{ lb/ft})(0.5)(9.52 \text{ ft})}$$

$$= 6.27$$

$$= \frac{150{,}912 \text{ N-m/m} + 1{,}319 \text{ N/m}\,[(1.87 \text{ m}) + (0.5)(2.9 \text{ m})\tan(90° - 78°)]}{18{,}658 \text{ N-m/m} + (4{,}060 \text{ N/m})(0.5)(2.9 \text{ m})}$$

$$= 6.27$$

Location C:

With the surcharge at Location C, half of the surcharge is over the reinforced soil zone and half is not. Therefore, the effects on the coherent gravity wall are a combination of the effects of a surcharge at Location A and a surcharge at Location B. The part of the surcharge over the geogrid will contribute to the stability of the wall with respect to sliding and overturning. The horizontal and vertical components of the force on the reinforced soil mass due to the surcharge are:

$$F_q = (q)(H)(K_a)$$

$$= (120 \text{ lb/ft}^2)(9.52 \text{ ft})(0.2561) = 293 \text{ lb/ft} \qquad = (5{,}748 \text{ Pa})(2.9 \text{ m})(0.2561) = 4{,}269 \text{ N/m}$$

$$F_{qh} = (F_q)\cos(\phi_{wr})$$

$$= (293 \text{ lb/ft})\cos(18°) = 279 \text{ lb/ft} \qquad = (4{,}269 \text{ N/m})\cos(18°) = 4{,}060 \text{ N/m}$$

$$F_{qv} = (F_q)\sin(\phi_{wr})$$

$$= (293 \text{ lb/ft})\sin(18°) = 91 \text{ lb/ft} \qquad = (4{,}269 \text{ N/m})\sin(18°) = 1{,}319 \text{ N/m}$$

The force resisting sliding is:

$$F_r = [W_w + F_v + 0.5(Q) + F_{qv}](C_f)$$

$$= [7{,}340 \text{ lb/ft} + 430 \text{ lb/ft} + 180 \text{ lb/ft} + 91 \text{ lb/ft}]\tan 30° = 4{,}642 \text{ lb/ft}$$

$$= [107{,}237 \text{ N/m} + 6{,}278 \text{ N/m} + 2{,}628 \text{ N/m} + 1{,}319 \text{ N/m}]\tan 30° = 67{,}817 \text{ N/m}$$

The force causing sliding is:

$$F_s = F_h + F_{qh}$$

$$= 1{,}325 \text{ lb/ft} + 279 \text{ lb/ft} = 1{,}604 \text{ lb/ft} \qquad = 19{,}321 \text{ N/m} + 4{,}060 \text{ N/m} = 23{,}381 \text{ N/m}$$

The safety factor against sliding is:

$$SFS = \frac{(4{,}642 \text{ lb/ft})}{(1{,}604 \text{ lb/ft})} = 2.9$$

$$= \frac{(67{,}817 \text{ N/m})}{(23{,}381 \text{ N/m})} = 2.9$$

The sum of the moments resisting overturning is:

$$\begin{aligned}
\Sigma M_r &= (W_w)\,[(X_1) + (0.5)\,(H)\tan(90° - \beta)] \\
&+ (F_v)\,[(X_2) + (0.333)\,(H)\tan(90° - \beta)] \\
&+ (F_{qv})\,[(X_2) + (0.5)\,(H)\tan(90° - \beta)] \\
&+ (0.5)\,(Q)\,[(X_3) + (H)\tan(90° - \beta)] \\
&= (7{,}340 \text{ lb/ft})\,[(3.0 \text{ ft}) + (0.5)\,(9.52 \text{ ft})\tan(90° - 78°)] \\
&+ (430 \text{ lb/ft})\,[(6.13 \text{ ft}) + (0.333)\,(9.52 \text{ ft})\tan(90° - 78°)] \\
&+ (91 \text{ lb/ft})\,[(6.13 \text{ ft}) + (0.5)\,(9.52 \text{ ft})\tan(90° - 78°)] \\
&+ (0.5)\,(360 \text{ lb/ft})\,[(5.38 \text{ ft}) + (9.52 \text{ ft})\tan(90° - 78°)] \\
&= 34{,}354 \text{ ft-lb/ft}
\end{aligned}$$

$$\begin{aligned}
&= (107{,}237 \text{ N/m})\,[(0.91 \text{ m}) + (0.5)\,(2.9 \text{ m})\tan(90° - 78°)] \\
&+ (6{,}278 \text{ N/m})\,[(1.87 \text{ m}) + (0.333)\,(2.9 \text{ m})\tan(90° - 78°)] \\
&+ (1{,}319 \text{ N/m})\,[(1.87 \text{ m}) + (0.5)\,(2.9 \text{ m})\tan(90° - 78°)] \\
&+ (0.5)\,(5{,}256 \text{ N/m})\,[(1.64 \text{ m}) + (2.9 \text{ m})\tan(90° - 78°)] \\
&= 152{,}469 \text{ N-m/m}
\end{aligned}$$

The sum of the moments causing overturning is:

$$\begin{aligned}
\Sigma M_o &= (F_h)\,(0.333)\,(H) + (F_{qh})\,(0.5)\,(H) \\
&= (1{,}325 \text{ lb/ft})\,(0.333)\,(9.52 \text{ ft}) + (279 \text{ lb/ft})\,(0.5)\,(9.52 \text{ ft}) \\
&= 5{,}529 \text{ ft-lb/ft}
\end{aligned}$$

$$\begin{aligned}
&= (19{,}321 \text{ N/m})\,(0.333)\,(2.9 \text{ m}) + (4{,}060 \text{ N/m})\,(0.5)\,(2.9 \text{ m}) \\
&= 24{,}545 \text{ N-m/m}
\end{aligned}$$

The safety factor against overturning is:

$$SFO = \frac{(34{,}354 \text{ ft-lb/ft})}{(5{,}529 \text{ ft-lb/ft})} = 6.21 \qquad = \frac{(152{,}469 \text{ N-m/m})}{(24{,}545 \text{ N-m/m})} = 6.21$$

If the surcharge was considered as a live load (ie: traffic), only the component of the surcharge force driving failure would be included.

Internal Stability

In addition to its effects on sliding and overturning failure, a surcharge can also have an impact on the spacing of the geogrid layers. It does so by putting an additional load on some or all of the layers of geogrid.

The first step in analyzing the effects of a surcharge on internal stability is to determine the horizontal soil stress within the reinforced soil zone. Once again, we will use the wall of Example 2-3 with a surcharge of 120 lb/sq ft (5,747 Pa), located as shown in Figure 3-4. The surcharge is 2 ft (0.61 m) wide.

Notice the diagonal lines connected to the beginning and end of the surcharge pressure diagram. These lines are drawn at an angle of 45° + ϕ/2 to the horizontal and mark the limits of the zone of influence of the surcharge within the soil. The horizontal stress due to the surcharge will act only on the portion of the retaining wall located in the area labeled “ZONE OF INFLUENCE.”

The magnitude of the horizontal surcharge stress is:

$$P_{qh} = (q)\,(K_{ai})\cos(\phi_{wi})$$
$$= (120\ \text{lb/ft}^2)\,(0.2197)\cos(20°)$$
$$= 25\ \text{lb/ft}^2$$

$$= (5{,}747\ \text{Pa})\,(0.2197)\cos(20°)$$
$$= 1{,}186\ \text{Pa}$$

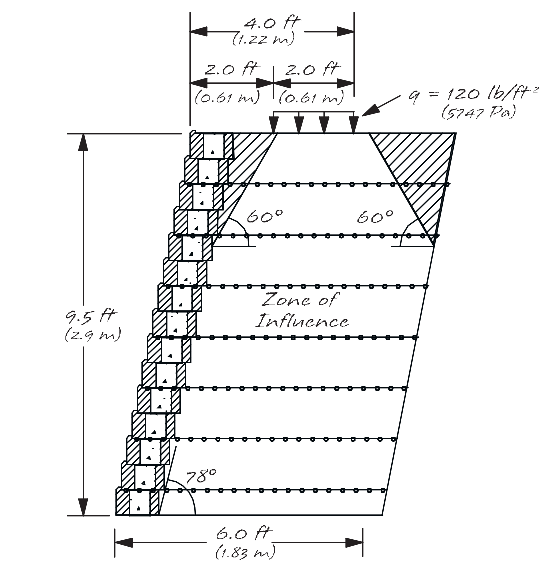


Figure 3-4. Coherent Gravity Wall with Surcharge

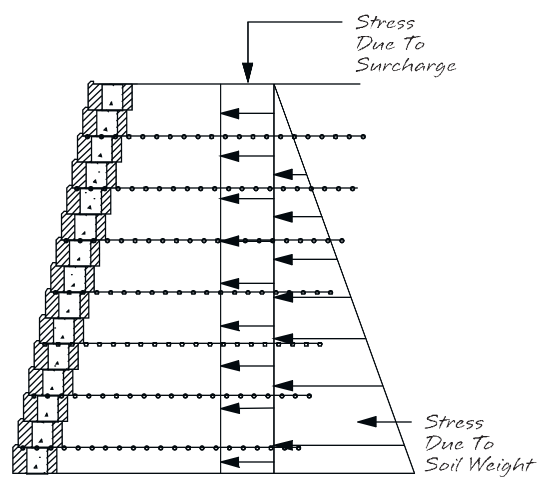


Figure 3-5. Pressure Distributions Due to the Weight and Surcharge

Figure 3-5 shows the wall facing with the two pressure distributions that affect it - one due to the soil weight and one due to the surcharge. The rectangular pressure distribution represents the effect of the surcharge on the wall facing.

Example 3-4:

Given the wall shown in Figure 3-4 and using the data of Example 2-3, determine the force acting on the first layer of grid.

$$F_g = (P_{avg})\,(d_h)$$

Where:

$$P_{avg} = (0.5)\,(\gamma_i)\,(K_{ai})\cos(\phi_{wi})\,(d_1 + d_2)$$
$$d_h = (d_1 - d_2)$$

Since the pressure from the surcharge remains constant, add P_{qh} to P_{avg}. So:

$$F_g = [(0.5)\,(\gamma_i)\,(K_{ai})\cos(\phi_{wi})\,(d_1 + d_2) + (q)\,(K_{ai})\cos(\phi_{wi})]\,(d_1 - d_2)$$

For the first layer of grid:

$$d_1 = 9.53\ \text{ft}\ (2.93\ \text{m})$$
$$d_2 = 8.26\ \text{ft}\ (2.5\ \text{m})$$
$$F_{g1} = [(0.5)\,(125\ \text{lb/ft}^3)\,(0.2197)\cos(20°)\,(9.53\ \text{ft} + 8.26\ \text{ft}) + (120\ \text{lb/ft}^2)\,(0.2197)\cos(20°)]\,(9.53\ \text{ft} - 8.26\ \text{ft})]$$
$$= 291.5\ \text{lb/ft}$$

$$= [(0.5)\,(2{,}002\ \text{kg/m}^3)\,(0.2197)\cos(20°)\,(2.9\ \text{m} + 2.5\ \text{m}) + (5{,}800\ \text{N/m}^2)\,(0.2197)\cos(20°)]\,(2.9\ \text{m} - 2.5\ \text{m})$$
$$= 4.256\ \text{kN/m}$$

Tiered Walls

Sometimes it is desirable to build two or more smaller walls at different elevations rather than one very tall wall. Such an arrangement is called a tiered wall and an example is pictured in Figure 3-6. The analysis of tiered walls can become very complicated. We have decided upon a design method that we feel comfortable with and will briefly describe it below. However, you as an engineer must use your own judgement. If you are not comfortable with this design method, use your best engineering judgement or seek advice from a local expert.

You should also be aware that, as the number and walls increase, the threat of global instability increases. A tiered wall consisting of three 5 ft (1.52 m) walls can have as great an impact on the underlying soil as a single 15 ft (4.6 m) wall. You should do a global stability analysis or have someone do one for you for tiered wall applications.

The first step in designing a tiered wall is to decide what the total height of all the walls will be, how many tiers there will be and the height of each tier. Each wall should be designed using a minimum grid length based on the total height of all the walls. Please note that the design grid lengths for the lower wall are often longer than the calculated minimum due to global stability. Then, using the design procedures presented earlier, design the top retaining wall. Next, find the average bearing stress of the top wall on the underlying soil. This average bearing stress is then applied as a uniform surcharge to the retained soil mass of the second wall from the top. (See Figure 3-7) The second wall is then analyzed using the procedures described earlier in this chapter.

The process is repeated until all of the tiers have been analyzed. As a final step, check the maximum soil bearing pressure of the bottom wall to make sure it doesn't exceed the allowable bearing pressure of the onsite soil. The need for a full global analysis should be conducted with tiered wall applications.

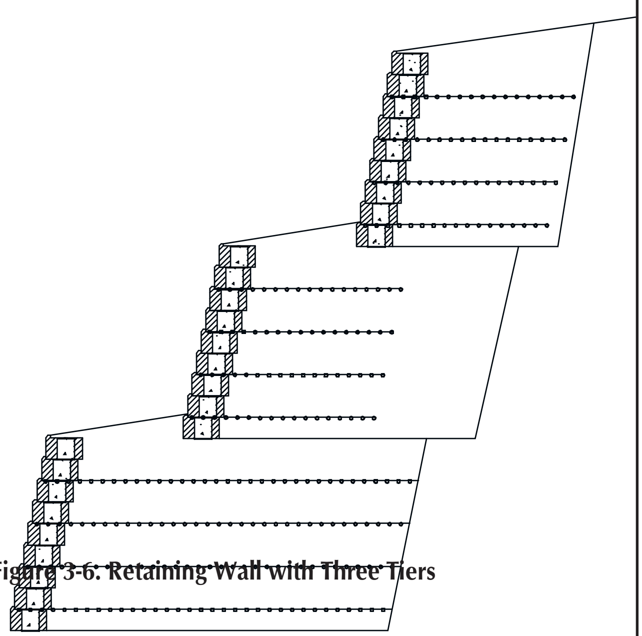


Figure 3-6. Retaining Wall with Three Tiers

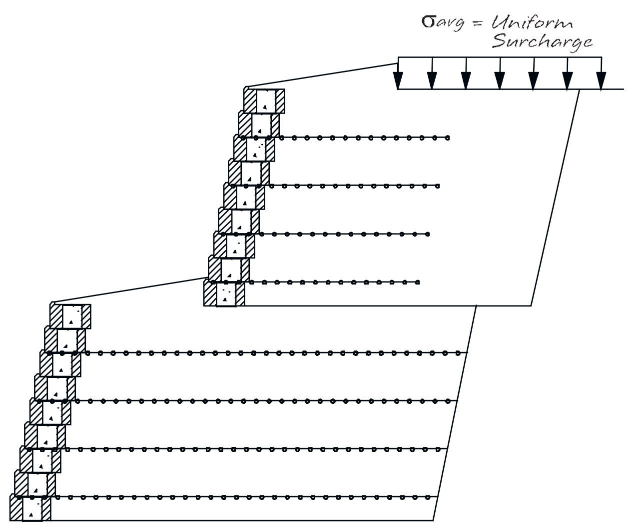
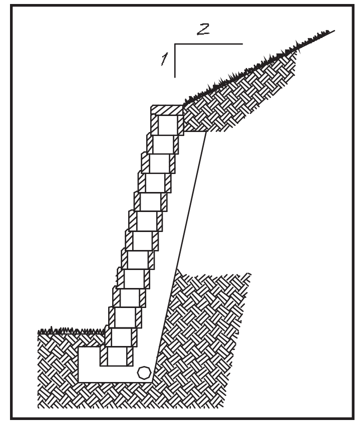


Figure 3-7. Average Bearing Stress of Top Wall Applied as Surcharge to Second Wall

CHAPTER FOUR

Sloped Backfill

Introduction



Sometimes it is not feasible or desirable to build a retaining wall that is tall enough to allow for a flat backfill. In that case, the backfill must be sloped. Sloped backfill is one of the most significant factors contributing to the active force on the wall. The slope of the backfill must be taken into account when designing a geogrid-reinforced retaining wall. Also, it should be noted that the slope of the backfill cannot exceed the friction angle of the soil. (This is not true if the cohesion of the soil is taken into account. However, the design procedures in this manual are based on the assumption that cohesion is not used in the methods outlined.)

Simple Gravity Walls With Sloped Backfill

As discussed in Chapter One, Coulomb's equation for the active force on the wall includes a term that changes the magnitude of the pressure coefficient as the slope of the backfill changes. The active pressure coefficient of Coulomb's equation is given by:

where: i = the slope of the backfill.

$$K_a = \left[\frac{\csc(\beta)\sin(\beta - \phi)}{\sqrt{\sin(\beta + \phi_w)} + \sqrt{\dfrac{\sin(\phi + \phi_w)\sin(\phi - i)}{\sin(\beta - i)}}} \right]^2$$

Let's look at the wall in Example 2-1 and see what effect changing the backfill slope has on the active force.

Example 4-1:

Given:

ϕ_w = 20°
ϕ = 30°
γ = 120 lb/ft³ (1,923 kg/m³)
γ_{wall} = 130 lb/ft³ (2,061 kg/m³)
β = 78°
H = 3.81 ft (1.16 m)

The table below shows the effect increasing the backfill slope has on the active pressure coefficient and the active force.

i (degrees)	K_a	F_a 1 lb/ft (1 N/m)
0	0.2197	191 (2,788)
18	0.2847	248 (3,613)
26	0.3662	319 (4,648)

Changing the slope of the backfill from 0° to 26° increased the active force by 67%. The wall in Example 2-1 would not be stable if the back-fill had a slope of 26°. For simple gravity walls, the effect of the sloping backfill is automatically taken into account by using Coulomb's equation to calculate the active force.

Coherent Gravity Walls With Sloped Backfill

One effect of a sloped backfill on a coherent gravity wall is to increase the weight of the wall and consequently, the resistance to sliding. The increased weight is due to the backfill soil that is located above the wall facing and over the reinforced soil mass. In Figure 4-1, the area designated W_i contains the soil that contributes the extra weight. The total weight of the wall can be calculated by adding the weight of the rectangular section, W_r to the weight of the triangular section, W_i:

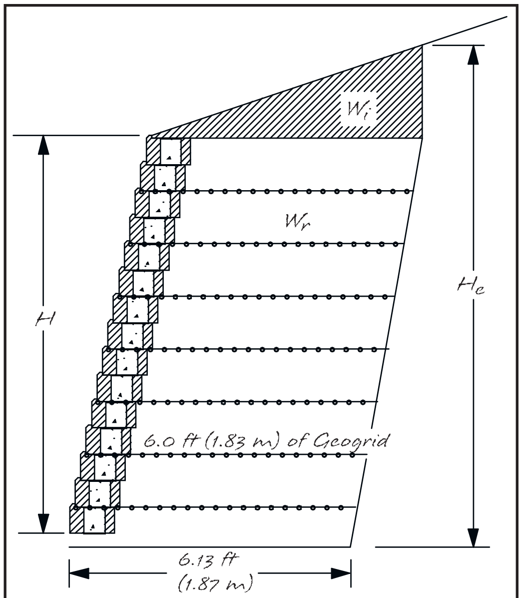


Figure 4-1. Coherent Gravity Wall with Sloped Backfill

W_r = (130 lb/ft³) (9.52 ft) (0.97 ft)
+ (125 lb/ft³) (9.52 ft) (6.0 ft − 0.97 ft)
= 7,186 lb/ft

= (2,061 kg/m³) (2.9 m) (0.3 m)
+ (2,002 kg/m³) (2.9 m) (1.83 m − 0.3 m)
= 104,731 N/m

W_i = (0.5) (6.0 ft) [(6.0 ft) tan (18°)] (125 lb/ft³)
= 731 lb/ft

= (0.5) (1.83 m) [(1.83 m) tan (18°)] (2,002 kg/m³)
= 10,685 N/m

W_w = (W_r) + (W_i)
= (7,186 lb/ft) + (731 lb/ft) = 7,917 lb/ft

= (104,731 N/m) + (10,685 N/m) = 115,416 N/m

External Stability

The external stability of the wall can be calculated as it was in Example 2-3, with three differences. First, the weight of the wall is greater, as shown above. Second, the height of the retaining wall is taken to be the height at the back of the reinforced soil mass, H_e. Third, the active force on the retained soil mass is greater because of the sloping backfill. The increase in the active force is automatically accounted for by using Coulomb's equation to calculate the active force. Calculate the safety factors for sliding and overturning of the wall in Figure 4-1. Compare these values to the safety factors in Example 2-3.

Example 4-3:

Given:

ϕ_i = 30°		i = 18°		H = 9.52 ft	(2.9 m)
ϕ_{wi} = 20°		β = 78°		γ_r = 120 lb/ft³	(1,923 kg/m³)
ϕ_r = 27°		K_{ar} = 0.3440		γ_i = 125 lb/ft³	(2,002 kg/m³)
ϕ_{wr} = 18°		K_{ai} = 0.2847			

The first step is to calculate the effective height, H_e at the rear of the coherent gravity wall:

H_e = (H) + (L_g) tan (i)
= (9.52 ft) + (6.0 ft) tan (18°) = 11.47 ft

= (2.9 m) + (1.83 m) tan (18°) = 3.49 m

Next, the active force on the coherent gravity wall is calculated:

F_a = (0.5) (γ_r) (K_{ar}) (H_e)²
= (0.5) (120 lb/ft³) (0.344) (11.47 ft)² = 2,636 lb/ft

= (0.5) (1,923 kg/m³) (0.3440) (3.49 m)² = 38,372 N/m

The horizontal component of the active force is:

F_h = (F_a) cos (ϕ_{wr})
= (2,636 lb/ft) cos (18°) = 2,507 lb/ft

= (38,372 N/m) cos (18°) = 36,494 N/m

The vertical component of the active force is:

$$F_v = (F_a) \sin(\phi_{wr})$$
$$= (2{,}636 \text{ lb/ft}) \sin(18°) = 815 \text{ lb/ft}$$

$$= (38{,}372 \text{ N/m}) \sin(18°) = 11{,}858 \text{ N/m}$$

The force resisting sliding is:

$$F_r = (W_w + F_v)(C_f)$$
$$= (7{,}917 \text{ lb/ft} + 815 \text{ lb/ft}) \tan(30°) = 5{,}041 \text{ lb/ft}$$

$$= (115{,}416 \text{ N/m} + 11{,}858 \text{ N/m}) \tan(30°) = 73{,}482 \text{ N/m}$$

The safety factor against sliding is:

$$SFS = \frac{F_r}{F_h} = \frac{(5{,}041 \text{ lb/ft})}{(2{,}507 \text{ lb/ft})} = 2.01$$

$$= \frac{(73{,}482 \text{ N/m})}{(36{,}494 \text{ N/m})} = 2.01$$

The moment resisting overturning is:

where:

X_1 = distance to the center line AB block
X_2 = distance to the center line of the reinforced mass
X_3 = distance to the centroid of the backslope
X_4 = distance to the back of the reinforced mass

$$\begin{aligned}
\Sigma M_r &= (W_f)[(X_1) + (0.5)(H)\tan(90° - \beta)] + (W_r)[(X_2) + (0.5)(H)\tan(90° - \beta)] \\
&+ (W_i)[(X_3) + (H)\tan(90° - \beta)] + (F_v)[(X_4) + (0.333)(H_e)\tan(90° - \beta)] \\
&= (1{,}142 \text{ lb/ft})[(0.49 \text{ ft}) + (0.5)(9.52 \text{ ft})\tan(90° - 78°)] \\
&+ (7{,}186 \text{ lb/ft})[(3.47 \text{ ft}) + (0.5)(9.52 \text{ ft})\tan(90° - 78°)] \\
&+ (731 \text{ lb/ft})[(4.08 \text{ ft}) + (9.52 \text{ ft})\tan(90° - 78°)] \\
&+ (815 \text{ lb/ft})[(6.13 \text{ ft}) + (0.333)(11.47 \text{ ft})\tan(90° - 78°)] \\
&= 43{,}876 \text{ ft-lb/ft}
\end{aligned}$$

$$\begin{aligned}
&= (16{,}673 \text{ N/m})[(0.149 \text{ m}) + (0.5)(2.9 \text{ m})\tan(90° - 78°)] \\
&+ (104{,}731 \text{ N/m})[(1.05 \text{ m}) + (0.5)(2.9 \text{ m})\tan(90° - 78°)] \\
&+ (10{,}685 \text{ N/m})[(1.21 \text{ m}) + (2.9 \text{ m})\tan(90° - 78°)] \\
&+ (11{,}858 \text{ N/m})[(1.82 \text{ m}) + (0.333)(3.49 \text{ m})\tan(90° - 78°)] \\
&= 193{,}895 \text{ N-m/m}
\end{aligned}$$

The moment causing overturning is:

$$M_o = (F_h)(0.333)(H_e)$$
$$= (2{,}507 \text{ lb/ft})(0.333)(11.47 \text{ ft}) = 9{,}576 \text{ ft-lb/ft}$$

$$= (36{,}494 \text{ N/m})(0.333)(3.49 \text{ m}) = 42{,}412 \text{ N-m/m}$$

The safety factor against overturning is:

$$SFO = \frac{\Sigma M_r}{\Sigma M_o} = \frac{(43{,}876 \text{ ft-lb/ft})}{(9{,}576 \text{ ft-lb/ft})} = 4.58$$

$$= \frac{\Sigma M_r}{\Sigma M_o} = \frac{(193{,}895 \text{ N-m/m})}{(42{,}412 \text{ N-m/m})} = 4.58$$

As calculated in Example 2-3, the same wall with a flat backfill had a safety factor against sliding of 3.4 and a safety factor against overturning of 7.8. Sloping the backfill cut the safety factors by 41% for sliding and 42% for overturning.

Internal Stability

Let's examine the effect of sloping backfill on the bottom layer of geogrid in the wall shown in Figure 4-3. The load on a layer of geogrid is given by:

$$F_g = (P_{avg})\,(d_h)$$

Suppose the wall in Figure 4-3 had a flat backfill, the load on the bottom layer of geogrid would be:

$$\begin{aligned} F_1 &= (P_{avg})\,(d_h) \\ &= (0.5)\,(P_1 + P_2)\,(d_1 - d_2) \\ &= (0.5)\,[(\gamma_i)\,(K_{ai})\,(d_1)\cos(\phi_{wi}) + (\gamma_i)\,(K_{ai})\,(d_2)\cos(\phi_{wi})]\,(d_1 - d_2) \\ &= (0.5)\,[(125\ \text{lb/ft}^3)\,(0.2197)\,(9.52\ \text{ft})\cos(20°) \\ &\quad + (125\ \text{lb/ft}^3)\,(0.2197)\,(8.25\ \text{ft})\cos(20°)]\,(9.52\ \text{ft} - 8.25\ \text{ft}) = 291\ \text{lb/ft} \end{aligned}$$

$$\begin{aligned} &= (0.5)\,[(2{,}002\ \text{kg/m}^3)\,(0.2197)\,(2.9\ \text{m})\cos(20°) \\ &\quad + (2{,}002\ \text{kg/m}^3)\,(0.2197)\,(2.51\ \text{m})\cos(20°)]\,(2.9\ \text{m} - 2.51\ \text{m})\,(9.81\ \text{m/sec}^2) = 4{,}237\ \text{N/m} \end{aligned}$$

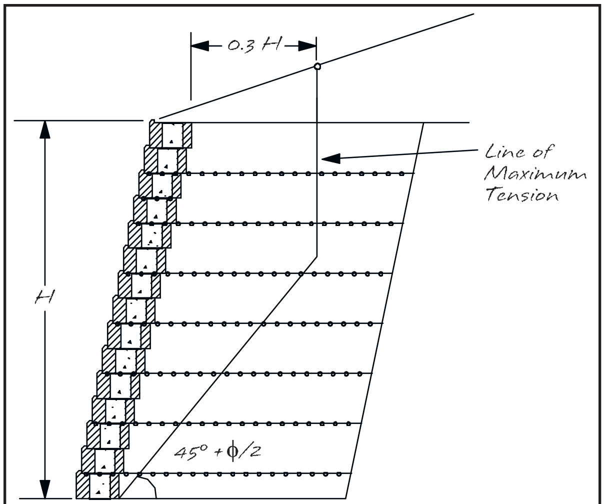


Figure 4-2. Line of Maximum Tension in a Coherent Gravity Wall

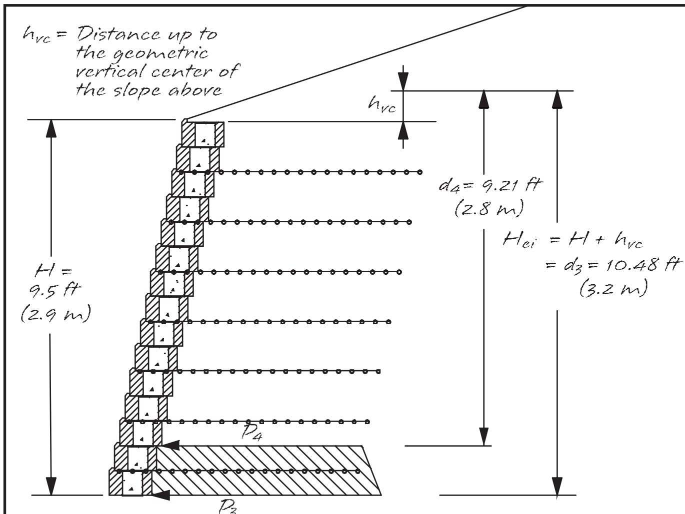


Figure 4-3. Effect of Sloped Backfill on Spacing of Geogrid Layers.

For the wall in Figure 4-3 with a backfill slope of 26°, $K_{ai} = 0.3662$ and the load on the bottom layer of geogrid is:

$$\begin{aligned} F_1 &= (P_{avg})\,(d_h) \\ &= (0.5)\,(P_3 + P_4)\,(d_3 - d_4) \\ &= (0.5)\,[(\gamma_i)\,(K_{ai})\,(d_3)\cos(\phi_{wi}) + (\gamma_i)\,(K_{ai})\,(d_4)\cos(\phi_{wi})]\,(d_3 - d_4) \\ &= (0.5)\,[(125\ \text{lb/ft}^3)\,(0.3662)\,(10.48\ \text{ft})\cos(20°) \\ &\quad + (125\ \text{lb/ft}^3)\,(0.3662)\,(9.21\text{ft})\cos(20°)]\,(10.48\ \text{ft} - 9.21\ \text{ft}) \\ &= 538\ \text{lb/ft} \end{aligned}$$

$$\begin{aligned} &= (0.5)\,[(2{,}002\ \text{kg/m}^3)\,(0.3662)\,(3.2\ \text{m})\cos(20°) \\ &\quad + (2{,}002\ \text{kg/m}^3)\,(0.3662)\,(2.8\ \text{m})\cos(20°)]\,(3.2\ \text{m} - 2.8\ \text{m})\,(9.81\ \text{m/sec}^2) \\ &= 8{,}110\ \text{N/m} \end{aligned}$$

Increasing the slope of the backfill from 0° to 26° increased the load on the bottom layer of geogrid by nearly 100%. If the calculated load at any given layer exceeded the allowable design load of the grid, the strength of the grid or additional layers of grid would need to be considered.

When designing a wall with a sloping backfill, start from the bottom of the wall and calculate the maximum d_h as in Example 2-3. But this time, use the depth from the geometric vertical center of the slope above the reinforced soil mass rather than the depth from the top of the wall facing.

Coherent Gravity Walls with Broken Back Slopes

Broken back slopes are very simply non-continuous slopes. They are modeled to more accurately describe a specific site condition. Broken back slopes provide much less force to a wall design than does a full continuous slope because of the greatly reduced soil mass above the wall. Figure 4-4 and Figure 4-5 shows the variation in the effective slope (i) and the effective wall height terms, H_e for external and H_{ei} for internal calculations. In broken back slope calculations an effective slope (i) is calculated based on the height of the slope over a distance of H_e. The designer should use these effective terms in all equations relating to slope calculations. Figure 4-6 shows the effective slopes for broken back slopes that crest over the reinforced mass.

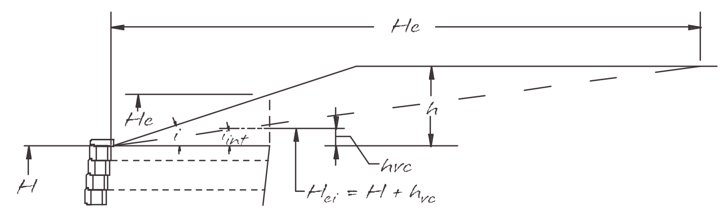


Figure 4-4. Effective Slope for Internal Calculations

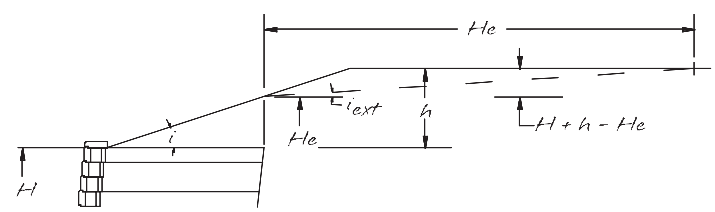


Figure 4-5. Effective Slope for External Calculations

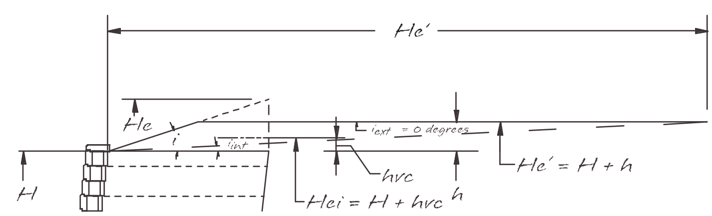


Figure 4-6. Effective Slope for Short Broken Back Slopes

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CHAPTER FIVE

Seismic Analysis

Introduction

In seismic design we take a dynamic force and analyze it as a temporary static load. The forces from seismic activity yield both a vertical and a horizontal acceleration. For our calculations, the vertical acceleration is assumed to be zero (Bathurst, 1998, NCMA Segmental Retaining Walls - Seismic Design Manual, 1998). Due to the temporary nature of the loading, the minimum recommended factors of safety for design in seismic conditions are 75% of the values recommended for static design.

The wall performance during the Northridge earthquake in Los Angeles, California and the Kobe earthquake in Japan proves that a soil mass reinforced with geogrid, which is flexible in nature, performs better than rigid structures in real life seismic situations (Columbia University in Cooperation with Allan Block Corporation and Huesker Geosynthetics. “Executive Summary - Seismic Testing - Geogrid Reinforced Soil Structures Faced with Segmental Retaining Wall Block”, Sandri, Dean, 1994, "Retaining Walls Stand Up to the Northridge Earthquake").

The following design uses the earth pressure coefficient method derived by Mononobe-Okabe (M-O) to quantify the loads placed on the reinforced mass and the internal components of the structure. Since the nature of segmental retaining walls is flexible, an allowable deflection can be accepted resulting in a more efficient design while remaining within accepted factors of safety.

Pressure Coefficients

The calculation of the dynamic earth pressure coefficient is similar to the static earth pressure coefficient derived by Coulomb, with the addition by Mononobe-Okabe of a seismic inertia angle (θ).

$$K_{ae} = \frac{\left[\dfrac{\cos^2(\phi + \omega - \theta)}{\cos(\theta)\cos^2(\omega)\cos(\phi_w - \omega + \theta)}\right]}{\left[1 + \sqrt{\dfrac{\sin(\phi + \phi_w)\sin(\phi - i - \theta)}{\cos(\phi_w - \omega + \theta)\cos(\omega + i)}}\right]^2}$$

Where:

ϕ = peak soil friction angle

ω = block setback

ϕ_w = angle between the horizontal and the sloped back face of the wall

i = back slope angle

θ = seismic inertia angle

The seismic inertia angle (θ) is a function of the vertical and horizontal acceleration coefficients:

$$\theta = \operatorname{atan}\left(\frac{K_h}{1 + K_v}\right)$$

Where:

K_v = vertical acceleration coefficient

K_h = horizontal acceleration coefficient

The vertical acceleration coefficient (K_v) is taken to be zero based on the assumption that a vertical and horizontal peak acceleration will not occur simultaneously during a seismic event (Bathurst et al.). The horizontal acceleration coefficient (K_h) is based on the acceleration coefficient (A_o) and the allowable deflection (d) of the wall system. (See equations below) The acceleration coefficient (A_o) typically varies from 0 to 0.4 in our calculations and is defined as the fraction of the gravitational constant g experienced during a seismic event. AASHTO provides recommendations for the acceleration coefficient based on the seismic zone that the retaining wall is being designed for. The allowable deflection (d) represents the lateral deflection that the retaining wall can be designed to withstand during a seismic event. The amount of deflection allowed in the design is based on engineering judgement. An approximation of the allowable deflection is 10 (A_o) in inches or 254 (A_o) for millimeters. However, the typical allowable deflection (d) is approximately 3 in. (76 mm). The equation used to determine the horizontal acceleration coefficient (K_h) varies depending on the amount of deflection allowed and whether it is calculated for the infill soils or the retained soils.

For **Infill** soils:

If d $= 0$, then

$$K_h = (1.45 - A_o)\,A_o$$

This equation, proposed by Segrestin and Bastic, is used in AASHTO / FHWA guidelines. It is assumed to be constant at all locations in the wall.

If d > 0, then

$$K_h = 0.74\,A_o\left(\frac{(A_o)\,(1\text{ in})}{d}\right)^{0.25}$$

$$K_h = 0.74\,A_o\left(\frac{(A_o)\,(25.4\text{ mm})}{d}\right)^{0.25}$$

This is a standard equation for the horizontal acceleration coefficient based on the Mononobe-Okabe methodology (Mononobe, 1929; Okabe, 1926).

For **Retained** soils if:

If d ≤ 1, then

$$K_h = \frac{A_o}{2}$$

If d > 1, then

$$K_h = 0.74\,A_o\left(\frac{(A_o)\,(1\text{ in})}{d}\right)^{0.25}$$

$$K_h = 0.74\,A_o\left(\frac{(A_o)\,(25.4\text{ mm})}{d}\right)^{0.25}$$

The following example illustrates the calculation of the dynamic earth pressure coefficient for the infill and retained soils with a typical allowable deflection of 3 in. (76 mm).

Example 5-1

Given:

ϕ_i	=	34°	ϕ_r	=	28°
ϕ_{wi}	=	2/3(34°) = 23°	ϕ_{wr}	=	2/3(28°) = 19°
d	=	3 in (76 mm)	ω	=	12°
i	=	0°	A_o	=	0.4

Find:

The dynamic earth pressure coefficients (Kae_i, Kae_r) for the infill and retained soils.

$$Kae_i = \frac{\left[\dfrac{\cos^2(\phi + \omega - \theta)}{\cos(\theta)\cos^2(\omega)\cos(\phi_w - \omega + \theta)}\right]}{\left[1+\sqrt{\dfrac{\sin(\phi + \phi_w)\sin(\phi - i - \theta)}{\cos(\phi_w - \omega + \theta)\cos(\omega + i)}}\right]^2}$$

The first step is to calculate the acceleration coefficients.

K_v = 0, based on the assumption that a vertical and horizontal peak acceleration will not occur simultaneously during a seismic event.

To determine K_h, we must look at the allowable deflection (d). Since the allowable deflection is greater than zero, the following equation is used:

$$K_h = 0.74\,A_o\left(\frac{(A_o)(1\text{ in})}{d}\right)^{0.25} \qquad K_h = 0.74\,A_o\left(\frac{(A_o)(25.4\text{ mm})}{d}\right)^{0.25}$$

$$K_h = 0.74\,(0.4)\left(\frac{(0.4)(1\text{ in})}{3\text{ in}}\right)^{0.25} = 0.179 \qquad K_h = 0.74\,(0.4)\left(\frac{(0.4)(25.4\text{ mm})}{76\text{ mm}}\right)^{0.25} = 0.179$$

The seismic inertia angle (θ) is:

$$\theta = \text{atan}\left(\frac{K_h}{1 + K_v}\right) = \text{atan}\left(\frac{0.179}{1 + 0}\right) = 10.1°$$

Finally, the dynamic earth pressure coefficient for the infill is:

$$Kae_i = \frac{\left[\dfrac{\cos^2(34 + 12 - 10.1)}{\cos(10.1)\cos^2(12)\cos(23 - 12 + 10.1)}\right]}{\left[1+\sqrt{\dfrac{\sin(34 + 23)\sin(34 - 0 - 10.1)}{\cos(23 - 12 + 10.1)\cos(12 + 0)}}\right]^2} = 0.289$$

The same process is followed in determining the dynamic earth pressure coefficient for the retained soil. Here again, the vertical acceleration coefficient (K_v) is equal to zero. With the allowable deflection greater than 1 inch (25 mm), the horizontal acceleration coefficient is the following:

$$K_h = 0.74\,A_o \left(\frac{(A_o)\,(1\text{ in})}{d}\right)^{0.25} \qquad K_h = 0.74\,A_o \left(\frac{(A_o)\,(25.4\text{ mm})}{d}\right)^{0.25}$$

$$K_h = 0.74\,(0.4) \left(\frac{(0.4)\,(1\text{ in})}{3\text{ in}}\right)^{0.25} = 0.179 \qquad K_h = 0.74\,(0.4) \left(\frac{(0.4)\,(25.4\text{ mm})}{76\text{ mm}}\right)^{0.25} = 0.179$$

Next, the seismic inertia angle (θ) can be calculated:

$$\theta = \operatorname{atan}\left(\frac{K_h}{1 + K_v}\right) = \operatorname{atan}\left(\frac{0.179}{1 + 0}\right) = 10.1°$$

The dynamic earth pressure coefficient for the retained soil is:

$$Kae_r = \frac{\left[\dfrac{\cos^2(28 + 12 - 10.1)}{\cos(10.1)\cos^2(12)\cos(19 - 12 + 10.1)}\right]}{\left[1 + \sqrt{\dfrac{\sin(28 + 19)\sin(28 - 0 - 10.1)}{\cos(19 - 12 + 10.1)\cos(12 + 0)}}\right]^2} = 0.377$$

Dynamic Earth Force of the Wall

The dynamic earth force is based on a pseudo-static approach using the Mononobe-Okabe (M-O) method. Figures 5-1 and 5-2 illustrate the pressure distributions for the active force, dynamic earth force increment, and the dynamic earth force. The magnitude of the dynamic earth force is:

$$\text{DFdyn} = F_{ae} - F_a$$

Where:

$$F_a = (0.5)\,(K_a)\,(\gamma)\,(H)^2$$
$$F_{ae} = (0.5)\,(1 + K_v)\,(K_{ae})\,(\gamma)\,(H)^2$$

The magnitude of the resultant force (F_a) acts at 1/3 of the height of the wall. Based on full scale seismic testing DFdyn has been found to act at 1/2 the height of the wall. Based on a rectangular pressure distribution.

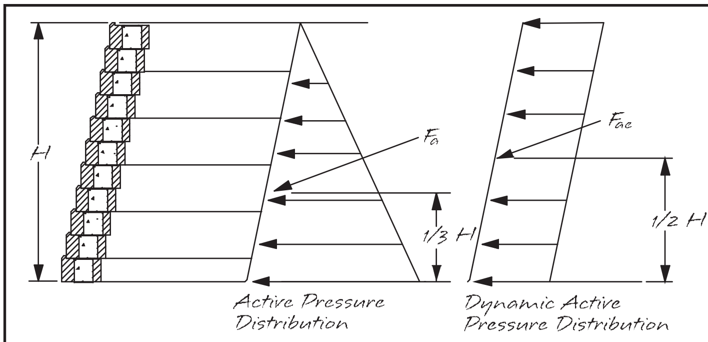

Figure 5-1. Dynamic and Static Active Pressure Distribution

Safety Factors

The minimum accepted factors of safety for seismic design are taken to be 75% of the values recommended for static design.

Sliding > 1.1

Overturning > 1.5

NOTE: The values 1.1 and 1.5 are based on 75% of the recommended minimum factors of safety for design of conventional segmental retaining walls. (Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guide Lines, FHWA NHI-00-043).

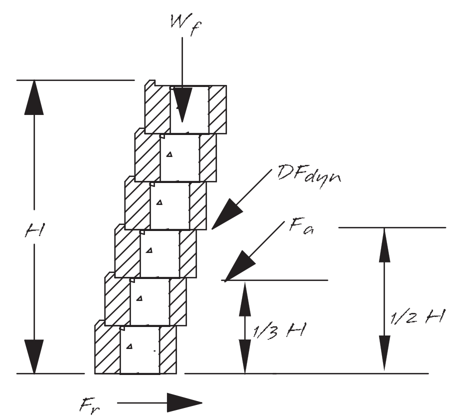

Figure 5-2. Free Body Diagram of Simple Gravity Wall Under Seismic Influence

Simple Gravity Wall with Seismic Influence

In seismic analysis, the weight of a simple gravity wall must counteract the static and temporary dynamic forces of the retained soil. Figure 5-2 illustrates the forces on a simple gravity wall during a seismic event. In the following example, the same equilibrium principles apply as in a static gravity wall analysis with additional consideration for the seismic earth force and the allowed reductions in required factors of safety for sliding and overturning.

Example 5-2:

Given:

ϕ_i	$= \phi_r = 30°$	β	$= 78°$
ω	$= (90 - \beta) = 12°$	i	$= 0°$
A_o	$= 0.4$	d	= 2 in. (51 mm)
Ka_i	$= 0.2197$	H	= 2.54 ft (0.77 m)
Ka_r	$= 0.2197$	ϕ	$= \phi_{wi} = \phi_{wr} = 2/3(\phi) = 20°$
γ_{wall}	= 130 lb/ft^3 (2,061 kg/m^3)	γ	$= \gamma_i = \gamma_r =$ 120 lb/ft^3 (1,923 kg/m^3)
Kae_i	$= 0.362$	Kae_r	$= 0.362$

Find:
The safety factor against sliding (SFS) and overturning (SFO).

NOTE: The dynamic earth pressure coefficients Kae_i and Kae_r were determined by following the allowable deflection criteria established at the beginning of the section.

The first step is to determine the driving forces exerted by the soil on the wall:

Active earth force:

$F_a \quad = (0.5)\,(K_a)\,(\gamma)\,(H)^2$

$\quad\quad = (0.5)\,(0.2197)\,(120\ \text{lb/ft}^3)\,(2.54\ \text{ft})^2 = 85\ \text{lb/ft}$

$\quad\quad = (0.5)\,(0.2197)\,(1{,}923\ \text{kg/m}^3)\,(0.67\ \text{m})^2$

$\quad\quad = (95\ \text{kg/m})\,(9.81\ \text{m/sec}^2) = 1{,}229\ \text{N/m}$

Dynamic earth force:

$F_{ae} \quad = (0.5)\,(1 + K_v)\,(K_{ae})\,(\gamma)\,(H)^2$

$\quad\quad = (0.5)\,(1 + 0)\,(0.362)\,(120\ \text{lb/ft}^3)\,(2.54)^2 = 140\ \text{lb/ft}$

$\quad\quad = (0.5)\,(1 + 0)\,(0.362)\,(1{,}923\ \text{kg/m}^3)\,(0.77)^2 = 2{,}024.5\ \text{N/m}$

Dynamic earth force increment:

$DFdyn = F_{ae} - F_a$

$\quad\quad = 140\ \text{lb/ft} - 85\ \text{lb/ft} = 55\ \text{lb/ft} \qquad = 2{,}024.5\ \text{N/m} - 1{,}229\ \text{N/m} = 795.5\ \text{N/m}$

Resolving the active earth force and the dynamic earth force increment into horizontal and vertical components:

$F_{ah} \quad = (F_a)\cos(\phi_w)$

$\quad\quad = (85\ \text{lb/ft})\cos(20°) = 80\ \text{lb/ft} \qquad = (1{,}229\ \text{N/m})\cos(20°) = 1{,}155\ \text{N/m}$

$F_{av} \quad = (F_a)\sin(\phi_w)$

$\quad\quad = (85\ \text{lb/ft})\sin(20°) = 29\ \text{lb/ft} \qquad = (1{,}229\ \text{N/m})\sin(20°) = 420\ \text{N/m}$

$DFdyn_h = (DFdyn)\cos(\phi_w)$

$\quad\quad = (55\ \text{lb/ft})\cos(20°) = 51.7\ \text{lb/ft} \qquad = (795.5\ \text{N/m})\cos(20°) = 747.5\ \text{N/m}$

$DFdyn_v = (DFdyn)\sin(\phi_w)$

$\quad\quad = (55\ \text{lb/ft})\sin(20°) = 18.8\ \text{lb/ft} \qquad = (795.5\ \text{N/m})\sin(20°) = 272.1\ \text{N/m}$

The next step is to determine the resisting forces:

Sliding Analysis

Weight of the wall facing:

$W_f \quad = (\gamma_{wall})(H)(d)$

$\quad\quad = (130\ \text{lb/ft}^3)\,(2.54\ \text{ft})\,(0.97\ \text{ft}) = 320\ \text{lb/ft} \qquad = (2{,}061\ \text{kg/m}^3)\,(0.77\ \text{m})\,(0.296\text{m}) = 4{,}608\ \text{N/m}$

Maximum frictional resistance to sliding:

$F_r \quad = (W_f + F_{av} + DFdyn_v)\tan(\phi)$

$\quad\quad = (320\ \text{lb/ft} + 29\ \text{lb/ft} + 18.8\ \text{lb/ft})\tan(30°) = 212.3\ \text{lb/ft} \qquad = (4{,}608\ \text{N/m} + 420\ \text{N/m} + 272.1\ \text{N/m})\tan(30°) = 3{,}060\ \text{N/m}$

Safety factor against sliding (SFS):

$$SFS_{seismic} = \frac{\text{(Force resisting sliding)}}{\text{(Force driving sliding)}} = \frac{F_r}{F_{ah} + DFdyn_h}$$

$$= \frac{(223 \text{ lb/ft})}{(80 \text{ lb/ft} + 51.7 \text{ lb/ft})} = 1.70 \geq 1.1 \text{ ok}$$

$$= \frac{(3{,}241 \text{ N/m})}{(1{,}155 \text{ N/m} + 747.5 \text{ N/m})} = 1.70 \geq 1.1 \text{ ok}$$

The factor of safety of 1.21 shows that an AB gravity wall during an earthquake in a seismic zone 4 is stable and does not require reinforcement to prevent sliding. As a comparison, the factor of safety in a static condition is the following:

$$SFS_{static} = \frac{\text{(Force resisting sliding)}}{\text{(Force driving sliding)}} = \frac{F_r}{F_{ah}} = \frac{(W_f + F_{av}) \tan \phi}{F_{ah}}$$

$$= \frac{(320 \text{ lb/ft} + 29 \text{ lb/ft}) \tan (30)}{(80 \text{ lb/ft})} = 2.52 \geq 1.5 \text{ ok}$$

$$= \frac{(4{,}608 \text{ N/m} + 420 \text{ N/m}) \tan (30)}{(1{,}155 \text{ N/m})} = 2.52 \geq 1.5 \text{ ok}$$

Overturning Failure Analysis

In seismic analysis, the moments resisting overturning (M_r) must be greater than or equal to 75% of the static requirement for overturning times the moments causing overturning (M_o).

The moments resisting overturning (M_r):

The weight of the wall, the vertical component of the active force, and the vertical component of the dynamic earth increment force contribute to the moment resisting overturning failure of the wall.

$$M_r = (W_f)\,(Wfarm) + (F_{av})\,(Faarm_v) + (DFdyn_v)\,(DFdynarm_v)$$
$$= (W_f)\,[(X_1) + (0.5)\,(H) \tan(\omega)] + F_{av}\,[(L + s) + (0.333)\,(H) \tan(\omega)]$$
$$+ DFdyn_v\,[(L + s) + (0.5)\,(H) \tan(\omega)]$$
$$= (320 \text{ lb/ft})\,[(0.49 \text{ ft}) + (0.5)\,(2.54) \tan(12)] + (29 \text{ lb/ft})\,[(0) + (0.171 \text{ ft})$$
$$+ (0.333)\,(2.54) \tan(12°)] + (18.8 \text{ lb/ft})\,[(0) + (0.171 \text{ ft}) + (0.5)\,(2.54) \tan(12°)]$$
$$= 261.5 \text{ ft-lb/ft}$$
$$= (4{,}608 \text{ N/m})\,[(0.149 \text{ m}) + (0.5)\,(0.77 \text{ m}) \tan(12°)] + (420 \text{ N/m})\,[(0) + (0.053 \text{ m})$$
$$+ (0.333)\,(0.77 \text{ m}) \tan(12°)] + (272.1 \text{ N/m})\,[(0) + (0.053 \text{ m}) + (0.5)\,(0.77 \text{ m}) \tan(12°)]$$
$$= 1{,}147.7 \text{ N-m/m}$$

NOTE: (s = setback per block, L = length of geogrid, X_1 = half the block depth)

The moments causing overturning (M_o):

The horizontal components of the active and dynamic forces contribute to the moment causing overturning failure of the wall.

$$
\begin{aligned}
M_o &= (F_h)\,(Faarm_h) + (DFdyn_h)\,(DFdynarm_h)\\
&= (F_h)\,(0.333)(H) + (DFdyn_h)\,(0.5)(H)\\
&= (80\ \text{lb/ft})\,(0.333)\,(2.54\ \text{ft}) + (51.7\ \text{lb/ft})\,(0.5)\,(2.54\ \text{ft})\\
&= 133.3\ \text{ft-lb/ft}
\end{aligned}
$$

$$
\begin{aligned}
&= (1{,}155\ \text{N/m})\,(0.333)\,(0.77\ \text{m}) + (747.5\ \text{N/m})\,(0.5)\,(0.77\ \text{m})\\
&= 584.2\ \text{N-m/m}
\end{aligned}
$$

Safety Factor Against Overturning (SFO):

$$SFS_{seismic} = \frac{\text{(Moments resisting overturning)}}{\text{(Moments driving overturning)}} = \frac{M_r}{M_o} \geq 1.5$$

$$= \frac{(261.5\ \text{ft-lb/ft})}{(133.3\ \text{ft-lb/ft})} = 1.96 > 1.5,\quad \text{ok}$$

$$= \frac{(1{,}147.7\ \text{N-m/m})}{(584.2\ \text{N-m/m})} = 1.96 > 1.5,\quad \text{ok}$$

This shows that the gravity wall is adequate with respect to overturning failure. However, if the safety factors were not met, geogrid reinforcement for this wall would be needed to achieve proper factor of safety. Evaluating the wall under static conditions we see that the required factors of safety are also met.

$$
\begin{aligned}
M_r &= (W_f)\,(Wfarm) + (F_v)\,(Faarm_v)\\
&= (W_f)\,[(X_1) + (0.5)\,(H)\tan(\omega)] + (F_v)\,[(L + s + (0.333)\,(H)\tan(\omega)]\\
&= (320\ \text{lb/ft})\,[(0.49\ \text{ft}) + (0.5)\,(2.54)\tan(12°)] + (29\ \text{lb/ft})\,[(0) + (0.171\ \text{ft})\\
&+ (0.333)\,(2.54)\tan(12°)]\\
&= 253\ \text{ft-lb/ft}
\end{aligned}
$$

$$
\begin{aligned}
&= (4{,}608\ \text{N/m})\,[(0.149\ \text{m}) + (0.5)\,(0.77\ \text{m})\tan(12°)] + (420\ \text{N/m})\,[(0) + (0.053\ \text{m})\\
&+ (0.333)\,(0.77\ \text{m})\tan(12°)]\\
&= 1{,}108\ \text{N-m/m}
\end{aligned}
$$

$$
\begin{aligned}
M_o &= (F_h)\,(Faarm_h)\\
&= (F_h)\,(0.333)\,(H)\\
&= (80\ \text{lb/ft})\,(0.333)\,(2.54\ \text{ft})\\
&= 68\ \text{ft-lb/ft}
\end{aligned}
$$

$$
\begin{aligned}
&= (1{,}155\ \text{N/m})\,(0.333)\,(0.77\ \text{m})\\
&= 296\ \text{N-m/m}
\end{aligned}
$$

$$SFO_{static} = \frac{\text{(Moments resisting overturning)}}{\text{(Moments driving overturning)}}$$

$$= \frac{M_r}{M_o} \geq 2.0$$

$$= \frac{(253\ \text{ft-lb/ft})}{(68\ \text{ft-lb/ft})} \qquad = \frac{(1{,}108\ \text{N-m/m})}{(296\ \text{N-m/m})}$$

$$= 3.72 \geq 2.0\ \text{ok} \qquad = 3.72 \geq 2.0\ \text{ok}$$

COHERENT GRAVITY WALL WITH SEISMIC INFLUENCE

Seismic inertial force (P_{ir})

In the external stability analysis of a geogrid reinforced retaining wall during a seismic event, a seismic inertial force (P_{ir}) is introduced. The seismic inertial force is the sum of the weight components that exert a horizontal inertial force within a reinforced soil mass during a seismic event. The three components exerting this inertial force are the block facing, the reinforced soil mass, and the backslope.

$$P_{ir} = K_{hr} (W_f + W_s + W_i)$$

This force along with the dynamic earth increment force combine with the static earth forces from the retained soil and the weight forces from the wall structure to create the conditions during an earthquake.

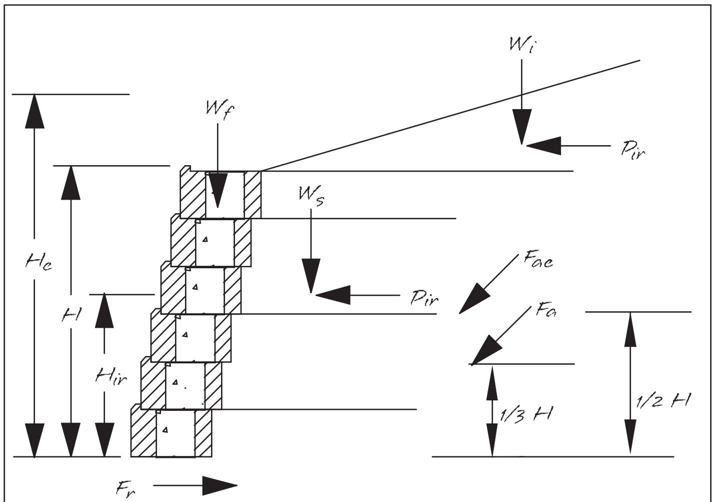

Figure 5-3. Free Body Diagram of a Coherent Gravity Wall Under Seismic Influence

Factor of Safety against Sliding

Calculating the Factor of Safety against Sliding for a coherent gravity wall follows the same stability criteria as a simple gravity wall. The principle being that the forces resisting sliding must be 1.1 times the forces causing sliding (75% of static Factor of Safety). As can be seen below, the formula for calculating the Factor of Safety against Sliding is the same as the gravity wall analysis with the addition of the seismic inertial force (P_{ir}) and the weight of the reinforced soil (W_s).

$$SFS_{seismic} = \frac{Fr_{seismic}}{F_{ah} + DFdyn_h + P_{ir}} \geq 1.1$$

Where:

$$Fr_{seismic} = (F_{av} + DFdyn_v + W_f + W_s) \tan (\phi_i)$$

Factor of Safety against Overturning

The Factor of Safety against Overturning is computed in the same way as a simple gravity wall with the addition of the seismic inertial force (P_{ir}) and the weight of the reinforced soil (W_s). The minimum $SFO_{seismic}$ can be defined as 75% of SFO_{static}.

$$SFO_{seismic} = \frac{M_r}{M_o} = \frac{(W_f)(Wfarm) + (W_s)(Wsarm) + (F_{av})(Faarm_v) + (DFdyn_v)(DFdynarm_v)}{(F_{ah})(Faarm_h) + (DFdyn_h)(DFdynarm_h) + (P_{ir})(H_{ir})} \geq 1.5$$

Example 5-3:

Given:

ϕ_i	=	ϕ_r = 30°	β	=	78°	F_a	=	1,362 lb/ft (19,884 N/m)
W_i	=	0 lb/ft	ω	=	$(90 - \beta)$ = 12°	DFdyn	=	879 lb/ft (12,850 N/m)
i	=	0°	Ka_i	=	0.2197	W_f	=	1,243 lb/ft (18,147 N/m)
d	=	2 in (51 mm)	Ka_r	=	0.2197	W_s	=	6,345 lb/ft (92,632 N/m)
A_o	=	0.4	Kae_i	=	0.362	W_s'	=	5,219 lb/ft (76,269 N/m)
H	=	10.16 ft (3.10 m)	Kae_r	=	0.362	γ_{wall}	=	130 lb/ft^3 (2,061 kg/m^3)
ϕ_w	=	$\phi_{wi} = \phi_{wr} = 2/3(\phi) = 20°$	Grid Lengths	=	6 ft (1.82 m)	H_{ir}	=	5.08 ft (1.548 m)
γ	=	$\gamma_i = \gamma_r$ = 120 lb/ft^3 (1,923 kg/m^3)						

Find:

The safety factor against sliding and overturning.

Factor of Safety Against Sliding Analysis

Based on the given information, we must first determine the frictional resistance to sliding (F_r).

$$F_r = (F_{av} + DFdyn_v + W_f + W_s) \tan(\phi)$$
$$= [(1{,}362 \text{ lb/ft}) \sin(20°) + (879 \text{ lb/ft}) \sin(20°) + 1{,}243 \text{ lb/ft} + 6{,}345 \text{ lb/ft}] \tan(30°)$$
$$= 4{,}823 \text{ lb/ft}$$

$$= [(19{,}884 \text{ N/m}) \sin(20°) + (12{,}850 \text{ N/m}) \sin(20°) + 18{,}147 \text{ N/m} + 92{,}632 \text{ N/m}] \tan(30°)$$
$$= 70{,}437 \text{ N/m}$$

Next, the seismic inertial force is calculated:

$$P_{ir} = K_{hr} (W_f + W_s' + W_i)$$

Since,

d = 2 in (51 mm)

$$K_{hr} = (0.74)(A_o)\left(\frac{(A_o)(1 \text{ in})}{d}\right)^{0.25} \qquad = (0.74)(A_o)\left(\frac{(A_o)(25.4 \text{ mm})}{d}\right)^{0.25}$$

$$= (0.74)(0.4)\left(\frac{(0.4)(1 \text{ in})}{2 \text{ in}}\right)^{0.25} \qquad = (0.74)(0.4)\left(\frac{(0.4)(25.4 \text{ mm})}{51 \text{ mm}}\right)^{0.25}$$

$$= 0.198 \qquad = 0.198$$

$$P_{ir} = 0.198 (1{,}243 \text{ lb/ft} + 5{,}219 \text{ lb/ft} + 0) = 1{,}279 \text{ lb/ft}$$

$$= 0.198 (18{,}147 \text{ N/m} + 76{,}269 \text{ N/m} + 0) = 18{,}694 \text{ N/m}$$

Finally, the safety factor against sliding can be calculated:

$$SFS_{seismic} = \frac{\text{(Forces resisting sliding)}}{\text{(Forces driving sliding)}} = \frac{F_r}{F_{ah} + DFdyn_h + P_{ir}} \geq 1.1$$

$$= \frac{(4{,}823 \text{ lb/ft})}{(1{,}362 \text{ lb/ft}) \cos 20° + (879 \text{ lb/ft}) \cos 20° + 1{,}279 \text{ lb/ft}} = 1.42 \geq 1.1 \text{ ok}$$

$$= \frac{(70{,}437 \text{ N/m})}{(19{,}884 \text{ N/m}) \cos 20° + (12{,}850 \text{ N/m}) \cos 20° + 18{,}694 \text{ N/m}} = 1.42 \geq 1.1 \text{ ok}$$

Comparing the seismic SFS to the static SFS below, we again see much higher safety values for static.

$$SFS_{static} = \frac{\text{(Forces resisting sliding)}}{\text{(Forces driving sliding)}} = \frac{F_r}{F_{ah}} = \frac{F_r - (DFdyn_v)\tan\phi}{(F_a)\cos(\phi_w)}$$

$$= \frac{(4{,}823\ \text{lb/ft}) - (173.6\ \text{lb/ft})}{(1{,}362\ \text{lb/ft})\cos 20°} = 3.63 \geq 1.5\ \text{ok}$$

$$= \frac{(70{,}437\ \text{N/m}) - (2{,}537\ \text{N/m})}{(19{,}889\ \text{N/m})\cos 20°} = 3.63 \geq 1.5\ \text{ok}$$

Factor of Safety Against Overturning Analysis

The safety factor against overturning is equal to the moments resisting overturning divided by the moments driving overturning (M_r / M_o) and must be greater than or equal to 1.5 (75% of SFO_{static}).

The moments resisting overturning (M_r):

$$M_r = (W_t)(Wt_{arm}) + (F_{av})(Faarm_v) + (DFdyn_v)(DFdynarm_v)$$

Where: $W_t = W_s + W_f$

$$= (W_t)[0.5 (L + s) + (0.5)(H)\tan(\omega)] + F_{av}[(L + s) + (0.333)(H)\tan(\omega)] + DFdyn_v[(L + s) + (0.5)(H)\tan(\omega)]$$

$$\begin{aligned} &= (7{,}588\ \text{lb/ft})[0.5 (6.0\ \text{ft} + 0.171\ \text{ft}) + (0.5)(10.16\ \text{ft})\tan(12°)] \\ &+ [(1{,}362\ \text{lb/ft})\sin 20°]\ [6.0\ \text{ft} + 0.171\ \text{ft} + (0.333)(10.16\ \text{ft})\tan(12°)] \\ &+ [(879\ \text{lb/ft})\sin(20°)]\ [6.0\ \text{ft} + 0.171\ \text{ft} + (0.5)(10.16\ \text{ft})\tan(12°)] \\ &= 37{,}002\ \text{ft-lb/ft} \end{aligned}$$

$$\begin{aligned} &= (110{,}778\ \text{N/m})[0.5 (1.82\ \text{m} + 0.053\ \text{m}) + (0.5)(3.10\ \text{m})\tan(12°)] \\ &+ [(19{,}884\ \text{N/m})\sin 20°][1.82\ \text{m} + 0.053\ \text{m} + (0.333)(3.10\ \text{m})\tan(12°)] \\ &+ [(12{,}850\ \text{N/m})\sin(20°)][1.82\ \text{m} + 0.053\ \text{m} + (0.5)(3.10\ \text{m})\tan(12°)] \\ &= 164{,}788\ \text{N-m/m} \end{aligned}$$

The moments driving overturning (M_o):

$$M_o = (F_{ah})(Faarm_h) + (DFdyn_h)(DFdynarm_h) + (P_{ir})(H_{ir})$$

$$= (F_{ah})(0.333)(H) + (DFdyn_h)(0.5)(H) + (P_{ir})(5.08\ \text{ft})$$

$$\begin{aligned} &= [(1{,}362\ \text{lb/ft})\cos(20°)](0.333)(10.16\ \text{ft}) + [(879\ \text{lb/ft})\cos(20°)](0.5)(10.16\ \text{ft}) + 1{,}279\ \text{lb/ft}\ (5.08\ \text{ft}) \\ &= 15{,}023\ \text{ft-lb/ft} \end{aligned}$$

$$\begin{aligned} &= [(19{,}884\ \text{N/m})\cos(20°)](0.333)(3.10\ \text{m}) + [(12{,}850\ \text{N/m})\cos(20°)](0.5)(3.10\ \text{m}) + 18{,}694\ \text{N/m}\ (1.548\ \text{m}) \\ &= 66{,}943\ \text{N-m/m} \end{aligned}$$

Safety Factor Against Overturning (SFO):

$$SFO_{seismic} = \frac{\text{(Moments resisting overturning)}}{\text{(Moments driving overturning)}} = \frac{M_r}{M_o} \geq 1.5$$

$$= \frac{(37{,}002\ \text{ft-lb /ft})}{(15{,}023\ \text{ft-lb/ft})} = 2.46 \geq 1.5\ \text{ok}$$

$$= \frac{(164{,}788\ \text{N-m/m})}{(66{,}943\ \text{N-m/m})} = 2.46 \geq 1.5\ \text{ok}$$

Comparing the seismic (SFO) to the below static (SFO):

$M_r = (W_t)(Wt_{arm}) + (F_{av})(F_{aarm})$

Where: $W_t = W_s + W_f$

$= (W_t)\,[0.5\,(L + s) + (0.5)\,(H)\tan(\omega)] + (F_{av})\,[(L + s) + (0.333)\,(H)\tan(\omega)]$

$= (7{,}588\text{ lb/ft})\,[0.5\,(6.0\text{ ft} + 0.171\text{ ft}) + (0.5)\,(10.16\text{ ft})\tan(12°)]$
$+ \;[(1{,}362\text{ lb/ft})\sin 20°]\;[(6.0\text{ ft} + 0.171\text{ ft}) + (0.333)\,(10.16\text{ ft})\tan(12°)]$
$= 34{,}821$ ft-lb/ft

$= (110{,}778\text{ N/m})\,[0.5\,(1.82\text{ m} + 0.053\text{ m}) + (0.5)\,(3.10\text{ m})\tan(12°)]$
$+ \;[(19{,}884\text{ N/m})\sin 20°]\,[(1.82\text{ m} + 0.053\text{ m}) + (0.333)\,(3.10\text{ m})\tan(12°)]$
$= 145{,}909$ N-m/m

$M_o = (F_{ah})\,(Faarm_h)$
$= (F_{ah})\,(0.333)\,(H)$
$= [(1{,}362\text{ lb/ft})\cos(20°)]\,(0.333)\,(10.16\text{ ft})$
$= 4{,}334$ ft-lb/ft

$= [(19{,}884\text{ N/m})\cos(20°)]\,(0.333)\,(3.10\text{ m})$
$= 18{,}161$ N-m/m

$$SFO_{static} = \frac{(\text{Moments resisting overturning})}{(\text{Moments driving overturning})} = \frac{M_r}{M_o}$$

$$= \frac{(34{,}821\text{ ft-lb/ft})}{(4{,}334\text{ ft-lb/ft})} = 8.0 \geq 2.0 \text{ ok}$$

$$= \frac{(145{,}909\text{ N-m/m})}{(18{,}161\text{ N-m/m})} = 8.0 \geq 2.0 \text{ ok}$$

Internal Stability

The factor of safety checks for the internal stability of a geogrid reinforced retaining wall under seismic conditions include the geogrid overstress, geogrid / block connection strength, geogrid pullout from the soil, and localized or top of the wall stability. These calculations are identical to those for a static stability analysis with the exception of the seismic forces introduced which affect the tensile loading on the geogrid.

Factor of Safety Geogrid Tensile Overstress

In order to calculate the Factor of Safety for Geogrid Tensile Overstress, the tensile force on each grid must first be determined. In a seismic event, the sum of the active force (F_a), the dynamic earth force increment ($DFdyn_i$), and the seismic inertial force (P_{ir}) represent the tensile force on each layer of geogrid.

$$F_{id_i} = F_{a_i} + DFdyn_i + P_{ir_i}$$

Where:

$$F_{a_i} = (K_a)\cos(\phi_w)\,(\gamma)\,(Ac_i)\,(0.5)$$

$$DFdyn_i = (0.5)(H_{ei})(K_{ae} - K_a)\cos(\phi_w)\,(\gamma)\,(Ac_i)$$

NOTE: This equation comes directly from the NCMA SRW Design Manual (3rd Edition) and can be referred to as the trapezoidal method.

Ac_i = The tributary influence area on each grid layer.

and

$$P_{ir_i} = (K_h)\,(\gamma)\,(Ac_i)$$

AASHTO or FHWA projects often use the active wedge method to determine DFdyn.

$$DFdyn_i = (K_h)\,(WA)\left(\frac{Ac_i}{H_e}\right)$$

AB Walls 2007 allows the user to choose either method but is defaulted to use the greater of the two.

We have used full scale seismic testing to determine that the internal seismic pressure closely matches a rectangle shape where the load is evenly distributed between the grid layers relative to their tributary area. This gives values that are not only more accurate, but are easier to design with. This load value is determined by the soil weight based on either the trapezoidal method shown in Figure 5-4 or by the active wedge method shown in Figure 5-5.

The angle of inclination (αi) of the Coulomb failure surface for the active wedge method:

$$\alpha i = \operatorname{atan}\left[\frac{-\tan(\phi i - i) + \sqrt{[\tan(\phi i - i)\,(\tan(\phi i - i) + \cot(\phi i + \omega))\,(1 + \tan(\phi_w - \omega)\cot(\phi i + \omega))]}}{1 + \tan(\phi_w - \omega)\,(\tan(\phi i - i) + \cot(\phi i + \omega))}\right] + \phi i$$

Determine the Factor of Safety against Tensile Overstress:

$$FS_{overstress} = \frac{(LTADS)(RF_{cr})}{F_{id}}$$

In the calculation of the Factor of Safety Geogrid Tensile Overstress for a seismic event, we do not take a reduction of the geogrid ultimate strength for long-term creep. This is due to the short-term loading during a seismic event.

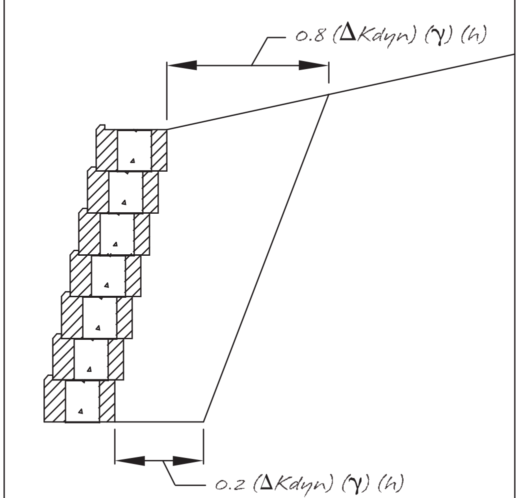

Figure 5-4. Trapezoidal Method

Geogrid / Block Connection Capacity

The Factor of Safety for Connection Strength is equal to the peak connection strength divided by the tensile force on that layer of grid multiplied by 0.666. We take the reduction on the tensile force due to the reality that some of the tensile force is absorbed by the soil in the influence area.

$$FS_{conn} = \frac{F_{cs}}{F_{id}\,(0.667)} \geq 1.1$$

Geogrid Pullout from the Soil

The Factor of Safety for Geogrid Pullout from the Soil is:

$$FS_{pullout} = \frac{F_{gr}}{F_{id}} \geq 1.1$$

where,

$$F_{gr} = 2\,(d_g)\,(\gamma)\,(L_e)\,(C_i)\tan(\phi)$$

The above pullout capacity equation takes into account the geogrid interaction coefficient (C_i) and is calculated based on the length of geogrid embedded beyond the line of maximum tension (L_e).

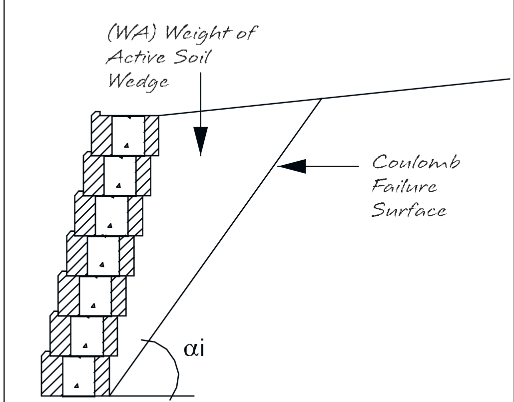

Figure 5-5. Active Wedge Method

Localized Stability, Top of the Wall

To determine local or top of the wall stability (SFS and SFO), the wall parameters and soils forces in the unreinforced portion of the retaining wall are focused on. The unreinforced height of the wall (H_t) is simply the total height of the wall minus the elevation at which the last grid layer is placed. The local weight of the facing is:

$$W_f = (H_t)\,(t)\,(\gamma_{wall})$$

The local sliding resistance (F_r) is an equation based on the Allan Block shear strength, which was developed through empirical test data and is a function of the normal load acting at that point and is the following:

$$F_r = 2{,}671 \text{ lb/ft} + (W_f)\tan(38°) \qquad = 38{,}900 \text{ N/m} + (W_f)\tan(38°)$$

The soil and surcharge forces are as follows:

Active Force: $F_a = (0.5)(K_a)(\gamma)(H_t)^2$

Dynamic Force: $F_{ae} = (0.5)(1 + K_v)(K_{ae})(\gamma)(H_t)^2$

Dynamic Earth Force Infrement: $DFdyn = F_{ae} - F_a$

Seismic Inertial Force: $P_{ir} = (K_h)(W_f)$

Finally, the safety factor equations are:

$$SFS_{localstatic} = \frac{F_r}{(F_a)\cos(\phi_w)} \geq 1.5$$

$$SFS_{localseismic} = \frac{F_r}{(F_a + DFdyn + P_{ir})\cos(\phi_w)} \geq 1.1$$

$$SFO_{localstatic} = \frac{W_f\,[(H_t/2)\tan\omega + t/2] + (F_a)\sin(\phi_w)\,[(H_t/3)\tan\omega + t]}{(F_a)\cos(\phi_w)(H_t/3)} \geq 2.0$$

$$SFO_{localseismic} = \frac{W_f\,[(H_t/2)\tan\omega + t/2] + (F_a)\sin(\phi_w)\,[(H_t/3)\tan\omega + t] + (DFdyn)\sin(\phi_w)(0.5\,H_t + t)}{(F_a)\cos(\phi_w)(H_t/3) + (DFdyn)\cos(\phi_w)(0.5\,H_t) + P_{ir}(H_t/2)}$$

$$\geq 1.5$$

NOTE: Verify local requirements for static and seismic Factors of Safety.

Maximum Allowable Slopes in Seismic Conditions

The Mononobe-Okabe 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 (ϕ_r), weighted friction angle (ϕ_w), slope above the wall (i), wall setback (ω), and seismic inertia angle (θ_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:

$$i_{max} = \phi_r - \theta_r$$

See reference: Engineering Implications of the Relation between Static and Pseudo-Static Slope Stability Analysis, Robert Shukha, Rafael Baker and Dov Leshchinsky, EJGE Paper 2005-616)

PHI	A_o	Maximum Allowable Slope
34	0.2	30.1
34	0.4	24.7
32	0.2	28.1
32	0.4	22.7
30	0.2	26.1
30	0.4	20.7
28	0.2	24.1
28	0.4	18.7

For slopes that are planned to exceed the maximum allowable value, the M-O equation does not provide for accurate loading values and therefore does not accurately evaluate pseudo-static loading. We recommend consulting a geotechnical engineer for a more in depth analysis and possible solutions.

CHAPTER SIX

Internal Compound Stability

Introduction

Wall designs have typically been limited to internal stability, external stability and bearing analysis by the site civil engineer or the wall design engineer. Additionally, the overall stability of the site is the responsibility of the owner and should be addressed by the owner, by contracting with a geotechnical engineering firm. The geotechnical engineering firm should provide a full global analysis of the entire site including the effects of the segmental retaining walls.

As the design roles become more defined it has become more customary for an Internal Compound Stability (ICS) analysis to be performed. ICS calculations determine the factors of safety for potential slip surfaces which pass through the unreinforced retained soil, the reinforced soil mass and the wall facing within the wall design envelope.

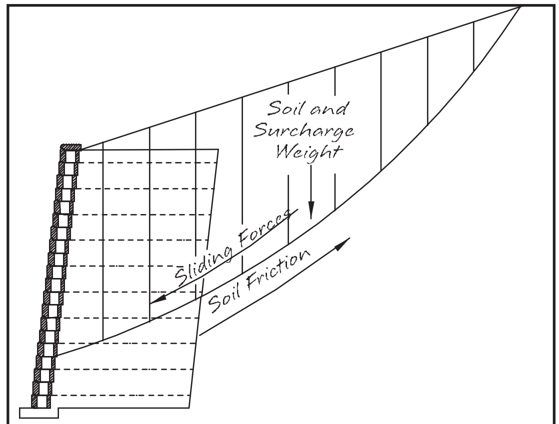

Figure 6-1. Internal Compound Analysis

Internal compound stability calculations are limited to a wall design envelope above the base material and back no further than 2 (H) or He + L, whichever is greater. This evaluation zone models the slip surface through the wall facing. The slip surface slices the affected grid layers and shears or bulges the SRW facing units. The designers performing ICS calculations can now model the entire wall design envelope in one comprehensive calculation. These calculations include the effects of the infill and retained soil strength, the individual grid layer strengths and spacing and the shear and connection strength the SRW facing brings to the system.

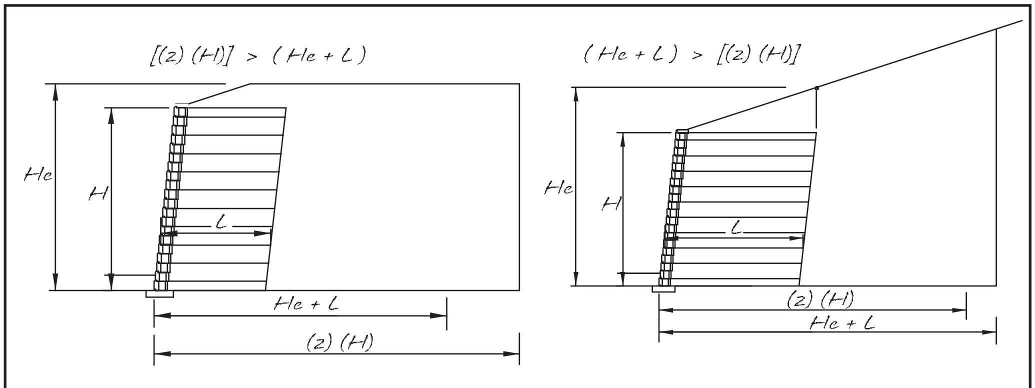

Figure 6-2. Internal Compound Stability Design Envelope

The distinctions between an ICS analysis and a global stability analysis form a clear line of design responsibility. A site civil or wall designer should review the ICS above the base material and through the wall facing within the design envelop for each wall designed on a site. For the larger site stability design, the owner through their geotechnical engineer should be responsible for the global stability of the entire site including the soils below the base material of all walls and structures designed on the project site.

Design Methodology

The Simplified Bishop Method of Slices (see References) is one of the most common analysis methods used in global stability modeling of reinforced slopes. In this method the volume, or weight, of the soil above a slip surface is divided into vertical wedges. The weight of soil is used to calculate the forward sliding forces as well as the sliding resistance due to the frictional interaction with the soil along the slip surface. In the ICS calculations we use the same process of evaluating the soil interaction, but additionally, the ICS analysis combines the resisting forces developed by geogrid layers intersecting the slip arc and the contribution from the SRW facing. Current slope stability modeling either ignores the facing or tries to mimic it by exaggerating a thin semi-vertical soil layer. Internal compound stability calculations analyze both the facing shear capacity and the facing connection capacities to formulate a reasonable facing contribution to the resistance side of the equation. By combining these multiple sliding and resisting forces along the slip surface, a safety factor equation is formed by a ratio of resisting forces to the sliding forces. The end result determines if there is an equilibrium of forces along a particular slip surface.

The following equation calculates the Factor of Safety of Internal Compound Stability.

Safety Factor of ICS

$$= (\Sigma F_r + \Sigma \text{Facing} + \Sigma F_{gr}) / (\Sigma F_s + \Sigma F_{dyn})$$

Where:

Where:	ΣF_r	= sum of soil resisting forces
	Σ Facing	= sum of facing contribution
	ΣF_{gr}	= sum of geogrid contribution
	ΣF_s	= sum of sliding force
	ΣF_{dyn}	= sum of sliding forces due to seismic loading

Soil Sliding and Resisting Forces

As mentioned earlier, the Simplified Bishop Method of Slices is used to determine first the weight of the soil above the slip surface and then the sliding and resisting forces due to that soil weight along the slip surface. Figure 6-3 shows a typical section through the evaluation zone for ICS calculations. The vertical slices in the soil above the slip arc represent the individual portions of soil analyzed using Bishops theory. We will determine the weights and forces relative to one soil slice or wedge as an example. For a complete Simplified Bishop Method of Slices the designer would follow the same calculations for each individual soil wedge and at the end, sum them all together.

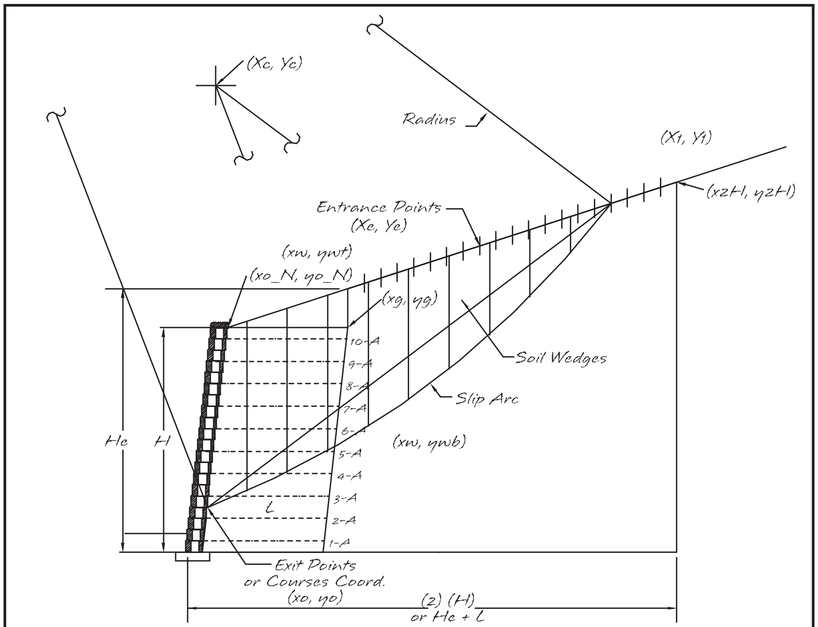

Figure 6-3. Internal Compound Stability Diagram

In Bishop modeling the soil wedges can be calculated as individual parts due mainly to Bishop's assumption that the vertical frictional forces between soil wedges are neglected, meaning that for design purposes there is no interaction between individual soil wedges. Therefore, the individual soil wedge weight (W) is determined simply by multiplying the volume of soil in that wedge by the unit weight of the soil. To determine the individual wedge volumes the designer must determine the exact geometry of the wall section and the slip arc to be evaluated. This is complex geometry that varies for every slip arc so it is a very difficult calculation to perform by hand. Please note that the thinner the wedge slice is the less the loss of weight is in the calculations. That is, the bottom of each wedge is considered a straight chord, not an arc, for ease of calculations. The lost soil weight is the area below the bottom chord and arc, and is negligible when the wedges are thinner.

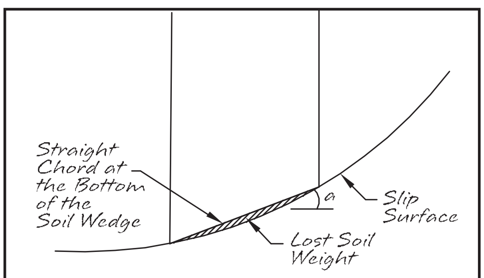

Figure 6-4. Lost Soil Weight

Once the wedge weight is determined the forward sliding force (F_s) is calculated by multiplying it by the sine of the angle below the soil wedge (α), where α is defined as the angle between horizontal and the bottom chord of each soil wedge; α is different for each wedge due to the relative location of each wedge along the slip surface.

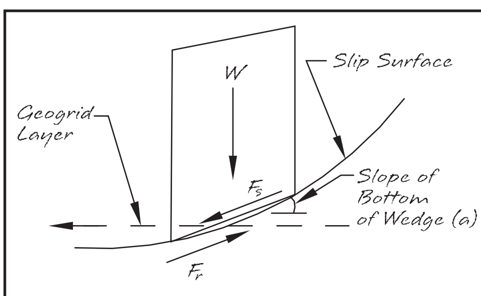

Figure 6-5. Wedge Force Diagram

Sliding Force:

F_s = (Weight Wedge) sin (α)

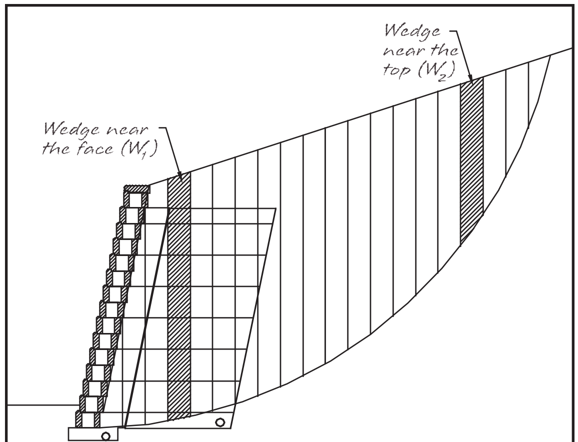

Figure 6-6. Wedge Weight

Compare for a moment two wedges, W_1 = 1000 lb/ft (14.6 kN/m) and W_2 = 100 lb/ft (1.46 kN/m). The first (W_1) is near the bottom of the slip arc where the arc ends near the facing and is relatively flat and therefore the α angle is relatively small, say 10 degrees. The other (W_2) is near the top of the slip arc where the arc is steeper and therefore the α angle is steeper, say 60 degrees. The sine (α) term acts as a percentage of forward movement, i.e. the flatter the α angle the smaller percentage:

Fs_1 = (W_1) sin (10 degrees) = 1000 lb/ft (0.174)
17.4% of (1000 lb/ft) = 174 lb/ft (2.54 kN/m)

Fs_2 = (W_2) sin (60 degrees) = 100 lb/ft (0.866)
86.6% of (100 lb/ft) = 86.6 lb/ft (1.26 kN/m)

The sliding resisting force (F_r) is calculated by multiplying the wedge weight by tangent of the internal friction angle of soil, which is commonly used for the soil frictional interaction coefficient. However, Bishop's method then divides this term by a geometric equation called m_α; m_α is a relationship between the strength of the soil and the relative angle of slip (α) for each wedge and is more clearly defined in global stability text books or global stability modeling programs such as ReSSa.

Sliding Resisting Force (F_r):

F_r = (Weight Wedge) tan (ϕ) / m_α

Where:

m_α = cos (α) + [sin (α) tan (ϕ)] / FS_i

And FS_i is the initial safety factor used to start the iteration process.

Generally, the Simplified Bishop procedure is more accurate than the Ordinary Method of Slices, but it does require an iterative, trial-and-error solution for the safety factor. Therefore, the designer needs to approximate what the safety factor will be for the final resulting slip surface. The closer the initial approximation is to the actual safety factor, the less iteration that will be required. This iteration process is standard for a Bishops calculation and again stresses the point that it is difficult to do hand calculations.

Surcharges and Seismic Forces

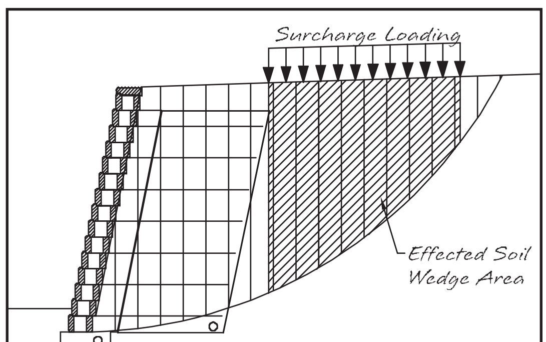

Figure 6-7. Effects of Surcharge Loading

Surcharge and seismic forces are calculated very similarly in a Bishops model. Surcharges, whether live or dead are simply added to the weights of the individual soil wedges. It should be noted that in an ICS calculations there is no distinction between live and dead load. By handling it in this manner the wedge weight term is increased by the relative weight of the surcharge and is than carried through the Sliding Force (F_s) and the Sliding Resisting Force (F_r) calculations. The designer should be careful to analyze where the surcharges are applied so they add that weight to only the effected soil wedges.

Therefore, the Sliding Forces and Sliding Resisting Force equations are redefined as:

Sliding Force: F_s = (Weight Wedge + Weight Surcharge) sin (α)

Sliding Resisting Force: F_r = (Weight Wedge + Weight Surcharge) tan (ϕ) / m_α

The Seismic Force (F_{dyn}) for a particular slip surface is additive to the Sliding Force (F_s) and is calculated by multiplying F_s by the horizontal acceleration coefficient (k_h); k_h is defined in **Chapter 5, Seismic Analysis**.

F_{dyn} = (F_s) (k_h) or for all wedges: $\Sigma F_{dyn} = \Sigma F_s (k_h)$

Geogrid Contribution (F_{gr}):

It would stand to reason that if a layer of geogrid is passed though by a slip arc, that the geogrid strength would increase the safety factor or stability of that slip surface. Therefore the relative geogrid interaction (F_{gr}) will be directly added to the resisting side of the equilibrium equation. The grid interaction in this calculation is directly effected by the geogrid spacing. If grid layers are closer together there is a higher likelihood of grid layers being passed through by the slip surface, thus providing more geogrid interaction. The greater the grid spacing the greater possibility of the slip surface falling between grid layers and thus not increasing the slip surfaces stability.

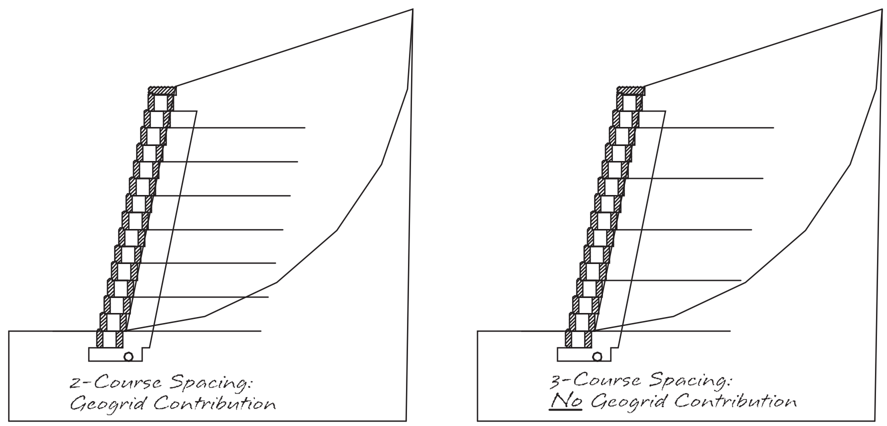

Figure 6-8. Geogrid Contribution at the Slip Arc

The horizontal resistance forces due to geogrid layers that intersect the slip arc are determined by the lesser of either the pullout of soil strength or the long term allowable load strength (LTADS) of the geogrid. Both are defined in the Internal Stability section of Chapter 2. The pullout of soil is calculated by determining the embedment length (L_e) on either side of the slip surface and combining it with the confining pressure, or normal load, from the soil above.

The designer should consider that there are two sides of the slip arc to consider when calculating the geogrid contribution. If the slip arc breaks free from the soil resistance along the slip surface, it will engage the affected geogrid layers. The grid layers can fail in three ways. First the grid can be pulled out from the soil on the retained side of the slip surface. Second, the geogrid layer can be pulled out from the soil on the sliding side of the slip surface. But on this side, the designer must take into account that the end of the grid is connected to the facing. Therefore the total pullout strength on the sliding wedge side is the connection strength plus the pullout of soil. This is a very unlikely way for the grid to fail because this combination will most always be greater than the rupture strength of the grid (limited to the LTADS). Third, the grid can rupture if the pullout of soil strengths exceeds the LTADS of any affected layer.

Calculations show that it is most likely that if a slip occurs some layers will pullout from the retained side and at the same time some layers will rupture.

The designer should analize each layer of effected geogrid for the three failure modes to determine the lesser for each layer, and then the sum of these lesser amounts becomes the Σ F_{gr} value.

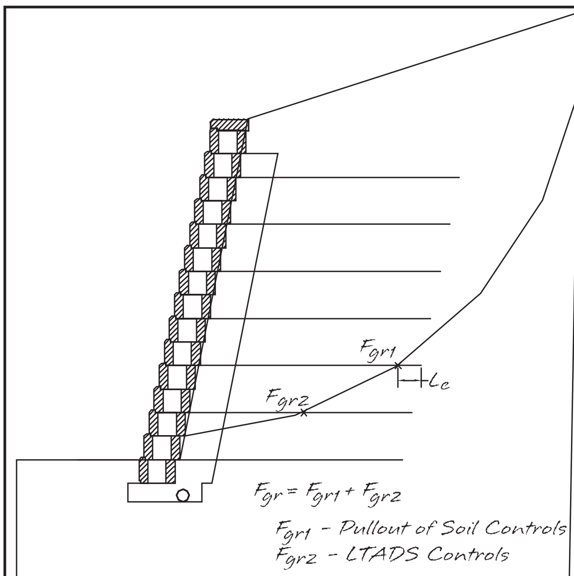

Figure 6-9. Grid Force

Wall Facing Contribution (Facing):

One element of the ICS calculations is the inclusion of facial stability to add to the sliding resistance. The stability of the wall facing has typically been ignored in global modeling due to the complexity of modeling a segmental retaining wall into a slope stability computer program.

Wall facing stability is provided by the interlocking shear between block and by the connection capacity between block and geogrid. Both are directly related to the spacing of the geogrid layers and the amount of normal load above the area in question. The closer together the reinforcement layers are, the more stable the facing becomes in both shear and connection strength. The maximum spacing between grid layers that can be found within the industry is around 32 in. (812 mm). However, past experience has shown that retaining walls that have geogrid layers spaced too far apart do not yield the best design for a wall. Problems associated with excess settlement, deflection and bulging may be experienced. Allan Block recommends a geogrid spacing of 16 in. (406 mm) or less. Closer spacing of lower strength reinforcement is a more efficient way of distributing the loads throughout the mass, which creates a more coherent structure.

Please note that the designer must evaluate both the stability provided by the geogrid connection and the shear strength of the block units, but can only use the lesser of the two in the ICS safety factor equation. Understanding that these two stabilizing forces are interconnected is a benefit to the designer of reinforced segmental retaining walls.

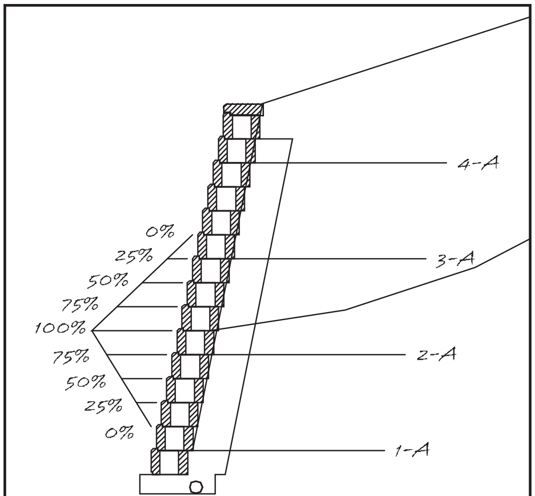

Figure 6-10. Geogrid Contribution to the Wall Face Case A

Facing Stability from Geogrid Connections

In the internal compound stability analysis, when the slip arc travels through the wall face at a grid layer we can safely assume that the full connection capacity is available to resist the sliding. However, the grid layers at the face that are above and below the slip arc will also provide some resistance and increase stability. Using a maximum influence distance of 32 in. (812 mm) from the slip arc, a percentage of the grid connection is used in calculating the contribution from block to grid connections when evaluating facial stability. Here are a few examples showing different spacing and slip arc locations.

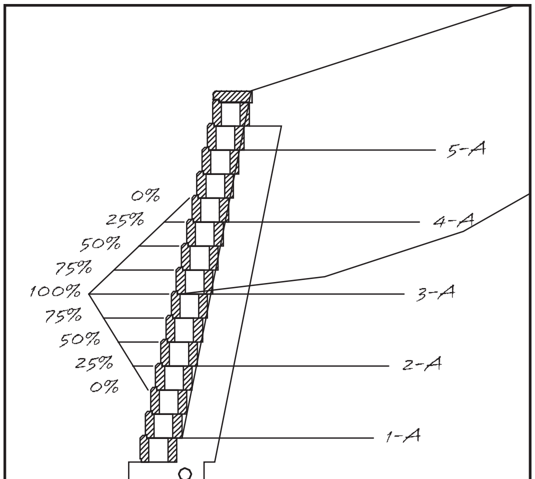

Figure 6-11. Geogrid Contribution to the Wall Face Case B

In **Case A** the slip arc is directly above a layer of geogrid and there are two layers that fall within the influence zone of 32 in. (812 mm) on either side of the slip arc. Looking at how the percentages are distributed, 75% of Grid 2A and 25% of Grid 3A connection strength capacities can be in the analysis of the wall facing. Assuming a full 8 in. (200 mm) tall unit.

Case B has three course spacing between grids and the slip arc intersecting the wall face at a geogrid layer. Therefore 100% of Grid 3A and 25% of Grids 2A and 4A connection strength capacities can be included.

Case C illustrates the boundary layers. The slip arc is towards the bottom of the wall, which means the bottom portion of the influence zone actually includes the bottom of the wall. Grid connection strength capacities are easily identified at 25% of Grid 3A and 75% of Grids 1A and 2A. However, because the slip arc is located towards the bottom of the wall we can also include 50% of the frictional sliding resistance between the Allan Block unit and the gravel base.

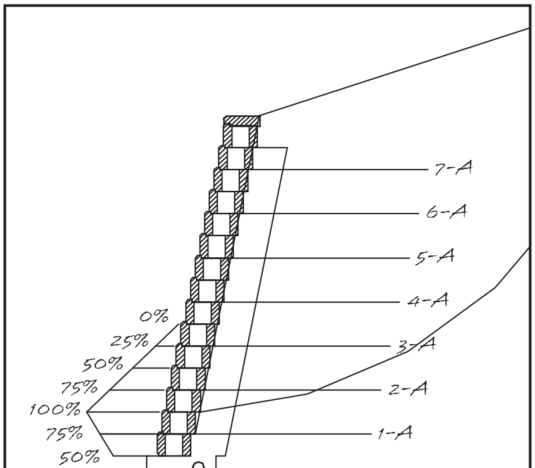

Figure 6-12. Geogrid Contribution to the Wall Face Case C

Facing Stability from Block Shear Strength

Shear interaction between units is easily calculated by understanding that the greater the normal load above a particular joint, the greater the block-to-block shear strength becomes. The tested shear strength equation comes from each SRW manufacture in the form of an ASTM D6916 test (also known as SRW-2 and is included in the appendices), which determines the block-grid-block shear resistance and block-block shear resistance relative to the normal load above that joint.

The first thing a designer should do is determine if the slip surface in question passes through the facing at a geogrid layer. If it does the assumption is made that the facing is 100% stable due to the connection strength with the geogrid and thus the designer can consider adding the tested block-grid-block shear strength of that joint in the analysis of the wall facing.

If the slip surface passes through the facing between grid layers a rotational moment develops between grid layers, with the lower grid layer forming a pivot point for the potential wall facing bulge. Summing the moments about this pivot point the designer can determine if the normal load at that joint is substantial enough to resist the upward rotational effect caused by the sliding forces. If there is sufficient normal load to resist the rotational effect the block will not uplift and the designer can consider adding the full block-block shear strength into the sliding resistance. However, if the normal load is overcome by the rotational uplift, the wall facing will pivot forward and the shear strength of the block cannot be added to the resistance.

Ultimately, this forward rotation will engage the geogrid connection strength from the grid layer above which will act to restrain the facing. If the wall continues to rotate, more uplift will occur and a forward bulge will form between layers and eventually a localized wall failure will occur.

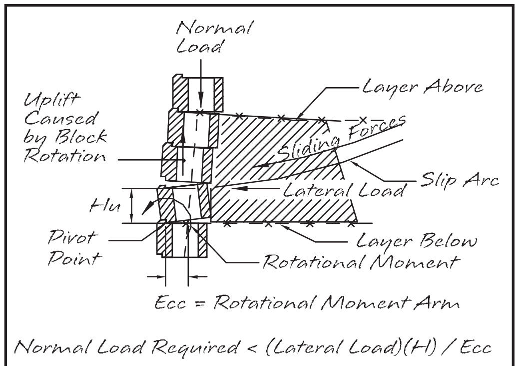

Figure 6-13. Facing Instability

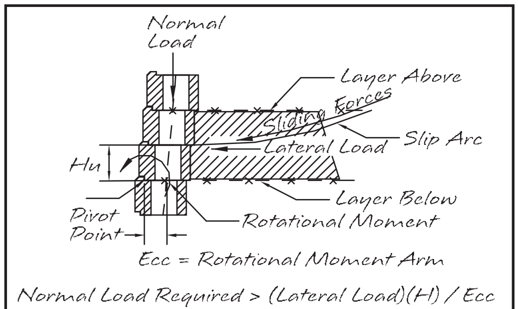

Figure 6-14. Facing Stability

Contribution from the Wall Face

As mentioned earlier, the designer cannot take both the facing stability from the geogrid connection and block shear when totaling up the resisting force. Only one will need to fail before instability of the wall face occurs. Therefore, the one with the least resisting force is the controlling face contribution and is used in the ICS safety factor calculation. The basis of this approach relies on a simple theory that as reinforcement layers are placed closer together, the facing becomes more rigid. The more rigid the facing is made by the connection contribution, the more likely that the shear strength at the evaluated course will control. Likewise, as the geogrid spacing is increased, the connection contribution is lessened thus causing the connection contribution to control.

The following is an example of evaluating ICS for a give set of site and soil conditions. Please note that a full global stability review should be obtained by the owner. These types of calculations require hundreds of thousands of iterations, while evaluating tens of thousands of slip arcs.

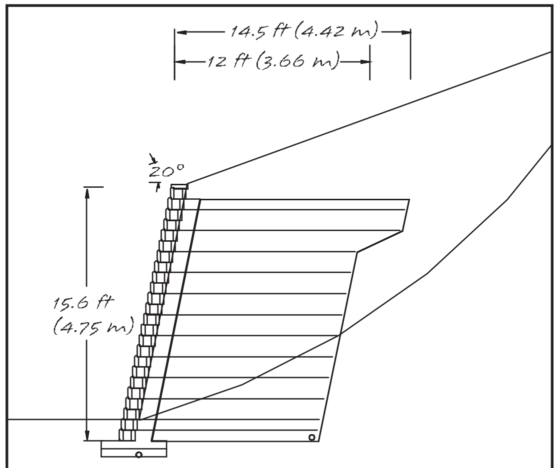

Diagram Ex. 6-1

<u>Example 6-1:</u>

Looking at Diagram Ex. 6-1 and given the following:

$\beta = 78°$	$\gamma = 120\ \text{lb/ft}^3$ (19 kN/m^3)
$\phi_i = 30°$	$A_o = 0.25$
$\phi_r = 28°$	

Geogrid is spaced 2 courses apart and a minimum length of 12 ft (3.66 m). The LTADS for this example is approximately 1,008 lb/ft (14.7 kN/m).

Reviewing the full ICS analysis, it is determined that the minimum Factor of Safety for ICS occurs between the 2nd and 3rd course of blocks.

The following summarizes the results for the slip arc with the minimum Factor of Safety for ICS:

ΣF_r = sum of soil resisting forces
= 18,156 lb/ft (265 kN/m)

Σ Facing = sum of facing contribution (either geogrid connection or shear)

ΣV_u = sum of block shear = 4,082 lb/ft (59.6 kN/m)

Σ Conn = sum of connection = 4,819 lb/ft (70.4 kN/m)

Σ Facing = 4,082 lb/ft (minimum of the shear and connection) (59.6 kN/m)

ΣF_{gr} = sum of geogrid contribution
= 2,791 lb/ft (40.7 kN/m)

ΣF_s = sum of sliding force
= 17,608 lb/ft (257 kN/m)

ΣF_{dyn} = sum of sliding forces due to seismic loading
= 1,585 lb/ft (23.1 kN/m)

Safety Factor of ICS

$$= (\Sigma F_r + \Sigma \text{Facing} + \Sigma F_{gr}) / (\Sigma F_s + \Sigma F_{dyn})$$

$$= \frac{(18{,}156 \text{ lb/ft} + 4{,}082 \text{ lb/ft} + 2{,}791 \text{ lb/ft})}{(17{,}608 \text{ lb/ft} + 1{,}585 \text{ lb/ft})}$$

$$= 1.304$$

$$= \frac{(265 \text{ kN/m} + 59.6 \text{ kN/m} + 40.7 \text{ kN/m})}{(257 \text{ kN/m} + 23.1 \text{ kN/m})}$$

$$= 1.304$$

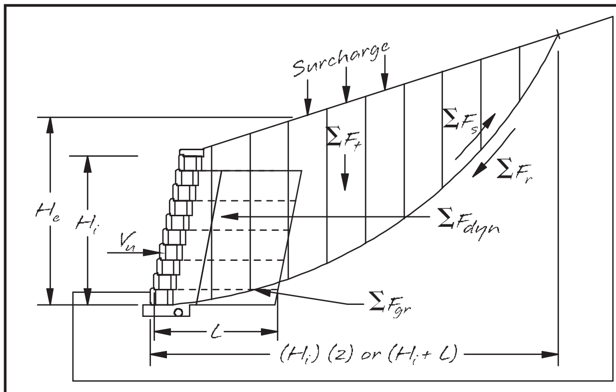

Figure 6-15. ICS Force Summary

Safety Factors and Design Approach

The minimum safety factor for Internal Compound Stability is 1.3 for static conditions and 1.1 for seismic. If after completing the analysis the safety factors are below these standards, the wall design will need to be revised. Please note that to provide a conservative expanded review for a geogrid reinforced retaining wall when analyzing ICS, cohesion is not considered in the methodology presented. Most global stability computer programs provide for the engineer to include a value for cohesion, which would dramatically change the final numbers. Additionally most global stability programs have not provided a detailed approach to contributions from the wall facing and therefore the exact results will be difficult to duplicate when trying to run a comparative review with off the shelf GS software. The following provides a few design options to increase factors of safety for Internal Compound Stability:

1. Use select backfill: It has been well documented that using select soils with higher internal strength as backfill in the infill area results in a better wall with increased stability and performance. This will also improve the internal compound stability as well and should be one of the first recommendations.
2. Additional geogrid reinforcement layers: Decreasing the spacing between the geogrid reinforcement will force the slip surface to intersect more geogrid layers which will increase the safety factor. The wall facing stability will also improve and will have a direct enhancement in the internal compound stability analysis.
3. Lengthen the geogrid reinforcement: Lengthening the geogrid will, again, force the slip surface to intersect more layers of geogrid and ultimately force the slip surface deeper into the evaluation zone. However, this will require additional excavation, and out of the three design options will typically cost the most.
4. Addition of geogrid in the slope above the wall: For slopes above the wall, adding geogrid reinforcement within the slope may improve Internal Compound Stability. The length and spacing of these grids will depend on the site conditions and should be done in cooperation with the geotechnical engineer of record.

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APPENDIX A

AB Engineering Manual Variables

A_o Seismic acceleration coefficient, pg 44

B_b Width of the foundation, pg 21

c Cohesion of foundation soils, pg 21

C_f Coefficient of friction, pg 10

C_i Coefficient of interaction between the soil and the geogrid, a measure of the ability of the soil to hold the geogrid when a force is applied to it, pg 26

D Depth of wall embedment = buried block + footing thickness, pg 21

d Allowable lateral deflection that a retaining wall can be designed to withstand during a seismic event, pg 44

d_b Footing thickness, pg 21

d_1 Distance from the top of the backfill or H_{ei} to the bottom of the zone supported by the layer of geogrid, pg 22

d_2 Distance from the top of the backfill or H_{ei} to the top of the zone supported by the layer of geogrid, pg 22

$DFdyn_i$ Dynamic earth force increment, pg 46

d_g Depth from the top of the infill or H_{ei} to the layer of geogrid, pg 26

d_h Difference between d_1 and d_2, pg 22

e Eccentricity of the resultant vertical force; the distance from the centerline of bearing of the gravity wall to the point of application of the resultant force , pg 19

F_a Active force on retaining wall; resultant force of the active pressure on the retaining wall, pg 5

F_{ae} Magnitude of dynamic earth force, pg 46

F_{cs} Peak connection strength, pg 24

F_e Preloaded value of installed earth anchor, pg 14

F_g Force applied to geogrid, pg 22

F_{gr} Maximum potential restraining force of geogrid, pg 26

F_h Horizontal component of active force, pg 10

F_{id} Dynamic internal force on geogrid, pg 54

F_{pa} Pullout grid capacity, pg 14

F_{qh} Horizontal compaction of surcharge force at the wall, pg 30

F_{qv} Vertical component of surcharge force at the wall, pg 30

F_r Maximum frictional resistance (the force that resists sliding of the wall because of friction and the soil), pg 10

$FS_{overstress}$ Geogrid overstress factor of safety, pg 55

$FS_{pullout}$ Factor of Safety for Geogrid Pullout from the Soil, pg 26

F_v Vertical component of active force, pg 10

F_{vb} The resultant vertical resisting force exerted on the wall by the soil, pg 19

F_w Force on the geogrid at the back face of the wall, pg 25

F_{we} Weighted design value of anchor, pg 14

H Distance from the bottom of the wall to the top of the wall (Depth from the top of the retained soil mass), pg 5

H_e Effective wall height of a coherent gravity wall for external calculations, pg 39

H_{ei} Effective wall height of a coherent gravity wall for internal calculations, pg 41

H_{ir} Moment arm associated with the seismic inertial force, pg 52

H_q Height of the wall affected by the surcharge, pg 29

h_{vc} Distance up to the geometric vertical center of the slope above, pg 41

i Slope of the top of the retained soil, pg 5

K Pressure coefficient, pg 5

K_a Active pressure coefficient, pg 5

K_{ae} Dynamic earth pressure coefficient, pg 43

Kae_i Dynamic earth pressure coefficient for the infill soil, pg 45

Kae_r Dynamic earth pressure coefficient for the retained soil, pg 46

K_{ai} Active earth pressure coefficient infill, pg 16

K_{ar} Active earth pressure coefficient retained, pg 16

K_h Horizontal seismic acceleration coefficient, pg 43

K_o At rest earth pressure coefficient, pg 5

K_v Vertical seismic acceleration coefficient, pg 43

l Width of the section, pg 20

L_a Length of geogrid in the active zone, pg 27

L_e Length of geogrid embedded in the passive zone of the soil, pg 26

L_s Equivalent lip thickness, pg 16

L_t Total length of geogrid required per linear foot of wall, pg 16

LTADS Long term allowable design strength of geogrid reinforcement, pg 14, 23

L_w Length of geogrid inside the Allan Block unit, pg 27

M_B Bearing capacity moment due to the eccentricity of the resultant vertical force, pg 20

AB Engineering Manual Variables

M_o — Moments causing overturning, pg 11
M_r — Moments resisting overturning, pg 11
N — Normal load from the weight of facing above grid location, pg 24
N_g — Number of geogrid layers, pg 54
N_q, N_c, N_γ — Terzaghi/Meyerhof equations from Craig p. 303, Soil Mechanics, Fifth Edition, pg 21
P_{avg} — Average soil pressure on the wall section, pg 22
P_h — The earth pressure at the base of the wall, pg 6
P_{ir} — Seismic inertial force, pg 51
P_q — The pressure due to a surcharge, pg 29
P_v — The vertical pressure at any given depth, pg 6
q — Surcharge, pg 28
q_f — Ultimate bearing capacity, pg 21
Q — Horizontal component of the surcharge force, pg 32
RF_{cr} — Reduction factor applied to geogrid for long term creep, pg 55
S — Section modulus of a 1 ft (0.3 m) wide section of the wall, pg 20
SFB — Bearing factor of safety, pg 21
SF_{conn} — Factor of Safety for the Static Geogrid/Block connection capacity, pg 24
SF_{mech} — Mechanical connection factor of safety, pg 23
SFO — Safety factor against overturning, pg 11
SFS — Safety factor against sliding, pg 10
t — Depth of block, pg 7
$\tan(\phi)$ — The coefficient of friction (shear strength) between adjacent layers of soil, pg 10, 26
T_u — Ultimate pullout resistance capacity, pg 66
V_c — Volume of concrete for each Allan Block unit, pg 7
V_t — Total vertical force, pg 12
V_{tot} — The total volume occupied by each standard Allan Block unit, including voids, pg 7
V_v — The volume of voids (difference of V_{tot} and V_c) for each Allan Block unit, pg 7
WA — Weight of soil in the active wedge, pg 54
W_f — Weight per linear foot of wall facing, pg 8
W_i — Weight of the triangular section of the sloped backfill, pg 51
W_s — Weight of the reinforced soil mass, pg 17
W_s' — Weight of soil mass based on a reinforced depth of 0.5 H, pg 52
W_w — Total weight of coherent gravity wall, pg 17
X — The point of application of the resultant bearing capacity force, pg 19
X_L — The distance from the front of the top AB unit to the uniform surcharge, pg 29
Y_1 — Moment of arm of the horizontal component of the active force, = H/3, pg 11
α_i — Angle of inclination of the Coulomb failure surface, pg 55
β — Angle between horizontal and the sloped back face of the wall, pg 5
γ — Unit weight of soil, pg 5
γ_i — The unit weight of the infill soil, pg 16
γ_r — Unit weight of retained soil, pg 16
γ_{wall} — Unit weight of the wall facing, pg 7
θ — Seismic inertia angle, pg 43
σ_{avg} — Average bearing pressure, pg 19
σ_h — Horizontal stress on retaining wall, pg 5
σ_{max} — Maximum bearing pressure, pg 20
σ_{min} — Minimum bearing pressure, pg 20
σ_{mom} — Difference in stress due to eccentricity, pg 20
σ_v — Vertical stress on soil at a given depth, pg 5
ϕ — Friction angle of soil, pg 1, 5
ϕ_B — Internal friction angle for base material, pg 21
ϕ_f — Friction angle of foundation soils, pg 21
ϕ_i — Friction angle of infill soils, pg 16
ϕ_r — Friction angle of retained soils, pg 16
ϕ_w — Angle between a line perpendicular to the wall face and the line of action of the active force, pg 5

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APPENDIX B

Allan Block Connection Tests and Shear Testing

Internal Compound Stability (ICS) allows you to consider the wall facing of the reinforced soil structure as part of the analysis. This is important to remember because the Allan Block units provide shear connection at the face and geogrid connection capacities that make a substantial difference in the stability of the wall. However, we need to understand just how the wall facing components work. The following tech sheet provides the basic understanding and Allan Block results of the two most widely used tests in the design of Segmental Retaining Walls (SRW's), SRW-1 and SRW-2. The specific test procedures are described in ASTM D6638 and D6916, respectively.

SRW-1 (ASTM D6638) Connection Testing

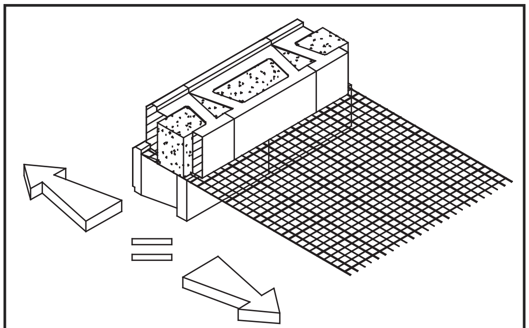

Figure B-1. Block-Grid Connection

Allan Block has always been a leader in the SRW industry by thoroughly testing our products to the highest of industry standards. SRW-1 determines the grid pull-out capacities or connection strength of a block to the geogrid reinforcement. Allan Block's patented "Rock-Lock connection" provides a continuous positive inter-locking of the geogrid to the aggregate filled cores of the Allan Block unit (See Figure 1). Allan Block has performed SRW-1 at the University of Wisconsin – Platteville, Bathurst Clarabut Geotechnical Testing (BCGT), and the National Concrete Masonry Association (NCMA) test facilities among others on many different grid families. The results in Figure 2 are for Huesker's Fortrac 35. The strength of the Rock-Lock connection allows the connection strength to well exceed the Long Term Allowable Design Strength (LTADS) as the normal loads increase. In fact, the lower strength grids perform so efficiently with the Rock-Lock connection that the ultimate connection strength nearly reaches the grids LTADS at the lowest applied normal load or the y-intercept. For these and other test summaries please contact the Allan Block Engineering Department.

Fortrac 35

Design Equations

Ultimate Connection Strength

Segment 1

Tu = 1,313 lb/ft + tan (8°)

Tu = 19.16 kN/m + tan (8°)

Maximum
= 1,686 lb/ft (24.6 kN/m)

Service Connection Strength

Segment 1

Tu = 1,056 lb/ft + tan (9°)

Tu = 15.41 kN/m + tan (9°)

Long Term Allowable Design Strength

LTADS
= 1,322 lb/ft (19.3 kN/m)

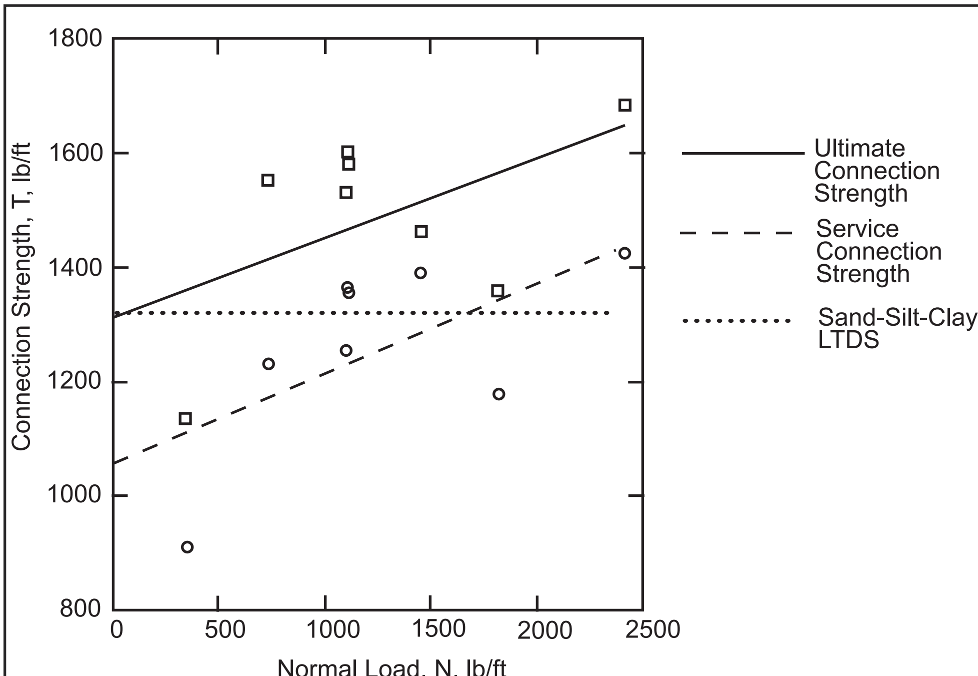

Figure B-2. Connection Strength Curve

SRW-2 (ASTM D6916) Interface Shear Strength

Shear testing has been commonly used to determine the effective internal shear resistance of one course of block relative to the next. Figure 3 shows the three pieces that together make up the total resistance, Shear Key (Upper Lip), Block-to-Block Friction and the aggregate Rock Lock. Testing was performed on AB Stones and AB Classic, AB Three and AB Rocks. The AB Rocks units, because of their larger shear lip, tested so well they did not shear under test conditions. The shear equations are shown in Figure 4. Testing with a layer of geogrid between courses is designed to be a worst-case condition as the grid acts as a slip surface reducing the contributions from Block Friction and aggregate Rock Lock. In the case of AB Stones and AB Classic the results were so great with the grid layer in place that a block-to-block test was not run.

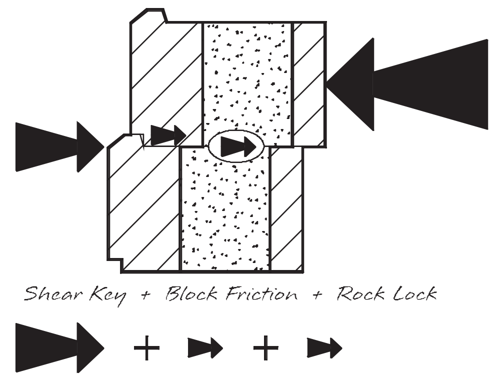

Figure B-3. Block Shear

Localized Wall Stability

New design theories such as ICS are recognizing the added benefit of a high shear and grid connection between layers of stackable block when analyzing wall stability. Careful analysis reveals that in order for geogrid to be dislodged from its position between two blocks, one of two things must happen. Either the entire wall facing must rotate forward or there must be relative movement between block courses.

The relative movement between block courses in an SRW wall can be defined as the localized wall stability. In the event that an ICS slip plane is formed though the reinforced mass (Figure 5) the grid connection and block shear will act together to resist the sliding forces. However, at some point one or the other will become the lesser and thus be the controlling factor in the wall stability. Consider a wall with single course grid spacing from bottom to top. This wall is more likely to have Shear control the localized stability than connection because the wall is ultimately as “rigid” as possible due to the continuous grid interaction. Now consider this same wall with 4 course grid spacing. It is intuitive that the wall is less “rigid” and thus more capable of bulging. In this case the shear capacity would well exceed the connection contribution from the few grid layers surrounding the slip surface and thus connection would be the lesser controlling factor.

Once a wall reinforced with geogrid has been properly constructed with well compacted soils and proper length and spaced geogrids, the reinforced mass works as a solid unit or coherent gravity mass. Therefore, in a competent coherent gravity mass and ICS slip plane will not occur and the actual stresses at the back of the facing will be minimal.

AB Stones and AB Classic w/ Geogrid Layer

$V_u = 2671 + N \tan(38°)$

$V_u = 38.9$ kN/m $+ N \tan(38°)$

AB Three Block-to-Block

$V_u = 1{,}018$ lb/ft $+ N \tan(61°)$

$V_u = 14.8$ kN/m $+ N \tan(61°)$

AB Three w/ Geogrid Layer

$V_{u1} = 743$ lb/ft $+ N \tan(71°)$

$V_{u1} = 10.8$ kN/m $+ N \tan(71°)$

$V_{u2} = 3{,}780$ lb/ft

$V_{u2} = 55.1$ kN/m

Figure B-4. Shear Test Results

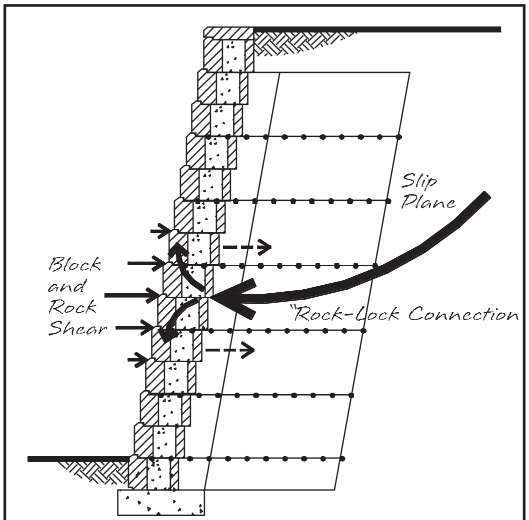

Figure B-5. Localized Wall Stability

Competitive Advantage

The raised front shear lip and granular infill in an Allan Block Wall provides a better engineering solution than the pin type interlock systems offered by many other retaining wall systems. Understanding this concept and you will understand why Allan Block retaining walls perform better than the competition.

GEOGRID SPECIFICATIONS AND CONNECTION TESTING RESULTS FOR:

AB FULL-SIZE UNITS

GEOGRID TYPE	LONG TERM ALLOWABLE DESIGN STRENGTH, LTDS, lb/ft (kN/m)			REDUCTION FACTOR CREEP, RFc	PEAK CONNECTION STRENGTH EQUATIONS, P, lb/ft (kN/m)		NORMAL LOAD INTERCEPT, lb/ft (kN/m)
	Sand-Silt-Clay	Sand-Gravel	Gravel		Segment 1:	Segment 2:	
Huesker - 11107-A South Commerce Blvd. - Charlotte, NC 28273 - (800) 942-9418							
FORTRAC 35/20-20	1322 (19.30)	1300 (19.00)	1243 (18.10)	1.572	Tu_1=728lb/ft+Ntan(18°) (Tu_1=10.6kN/m +Ntan(18°))	Tu_2=958lb/ft+Ntan(8°) (Tu_2=14.0kN/m +Ntan(18°))	1247.4 (18.20)
FORTRAC 55/30-20	1936 (28.30)	1882 (27.50)	1783 (26.00)	1.655	Tu_1=1090lb/ft+Ntan(27°) (Tu_1=15.9kN/m +Ntan(27°))	Tu_2=1906lb/ft+Ntan(4°) (Tu_2=27.8kN/m +Ntan(4°))	1856.2 (27.08)
FORTRAC 80/30-20	2815 (41.10)	2815 (41.10)	2615 (38.20)	1.655	Tu_1=680lb/ft+Ntan(57°) (Tu_1=9.92kN/m +Ntan(57°))	Tu_2=2143lb/ft+Ntan(14°) (Tu_2=31.27kN/m +Ntan(14°))	1133.6 (16.54)
Tencate Nicolon - 365 South Holland Drive - Pendergrass, Georgia 30567 - (888) 795-0808							
MIRAGRID 2XT	949 (13.80)	839 (12.20)	727 (10.6)	1.67	Tu_1=125.6lb/ft+Ntan(58.48°) (Tu_1=1.8kN/m +Ntan(58.48°))	Tu_1=1623.5lb/ft+Ntan(0°) (Tu_1=23.65kN/m +Ntan(0°))	918.6 (13.40)
MIRAGRID 3XT	15.58 (22.70)	1423 (20.80)	1309 (19.10)	1.67	Tu=1047lb/ft+Ntan(33°) (Tu=15.3kN/m +Ntan(33°))	-	-
MIRAGRID 5XT	2234 (32.60)	2040 (29.80)	1875 (27.40)	1.67	Tu_1=983lb/ft+Ntan(37°) (Tu_1=14.3kN/m +Ntan(37°))	Tu_2=1756lb/ft+Ntan(20°) (Tu_2=25.6kN/m +Ntan(20°))	1984.2 (28.95)
MIRAGRID 7XT	2961 (43.20)	2704 (39.40)	2487 (36.30)	1.67	Tu=1065.4lb/ft+Ntan(25.62°) (Tu=15.52kN/m +Ntan(25.62°))	-	-
MIRAGRID 8XT	3089 (45.07)	3007 (43.87)	2719 (39.87)	1.67	Tu_1=1063lb/ft+Ntan(40°) (Tu_1=15.51kN/m +Ntan(40°))	Tu_2=2872lb/ft (Tu_2=41.9kN/m)	21.55.9 (31.45)
MIRAGRID 10XT	4116 (60.05)	4006 (58.45)	3622 (52.84)	1.67	Tu_1=513lb/ft+Ntan(52°) (Tu_1=7.48kN/m +Ntan(52°))	Tu_2=1426lb/ft+Ntan(23°) (Tu_2=20.81kN/m +Ntan(23°))	1067.3 (15.57)
Strata Systems Inc. - 380 Dahlonega Road - Cumming, Georga 30040 - (800) 680-7750							
STRATA 150	1008 (14.70)	962 (14.10)	882 (12.90)	1.61	Tu=930lb/ft+Ntan(24°) (Tu=13.5kN/m +Ntan(24°))	-	-
STRATA 200	1613 (23.60)	1540 (22.50)	1412 (20.60)	1.61	Tu=9481lb/ft+Ntan(31°) (Tu=13.8kN/m +Ntan(31°))	-	-
STRATA 350	2259 (33.00)	2156 (31.50)	1976 (28.90)	1.61	Tu=900lb/ft+Ntan(23°) (Tu=13.1kN/m +Ntan(23°))	-	-
STRATA 500	2796 (40.80)	2669 (39.00)	2447 (35.70)	1.61	Tu=848lb/ft+Ntan(30°) (Tu=12.4kN/m +Ntan(30°))	-	-

AB FIELDSTONE UNITS

GEOGRID TYPE	LONG TERM ALLOWABLE DESIGN STRENGTH, LTDS, lb/ft (kN/m)			REDUCTION FACTOR CREEP, RFc	PEAK CONNECTION STRENGTH EQUATIONS, P, lb/ft (kN/m)		NORMAL LOAD INTERCEPT, lb/ft (kN/m)
	Sand-Silt-Clay	Sand-Gravel	Gravel		Segment 1:	Segment 2:	
Huesker - 11107-A South Commerce Blvd. - Charlotte, NC 28273 - (800) 942-9418							
FORTRAC 35/20-20	1322 (19.30)	1300 (19.00)	1243 (18.10)	1.572	Tu=769lb/ft+Ntan(6°) (Tu=11.2kN/m +Ntan(6°))	-	-
FORTRAC 55/30-20	1936 (28.30)	1882 (27.50)	1783 (26.00)	1.655	Tu=1444lb/ft+Ntan(3°) (Tu=21.0kN/m +Ntan(3°))	-	-
Tencate Nicolon - 365 South Holland Drive - Pendergrass, Georgia 30567 - (888) 795-0808							
MIRAGRID 2XT	949 (13.80)	839 (12.20)	727 (10.6)	1.67	Tu_1=893lb/ft+Ntan(31°) (Tu_1=13.0kN/m +Ntan(31°))	Tu_2=1516lb/ft+Ntan(5°) (Tu_2=22.1kN/m +Ntan(5°))	1213.5 (17.70)
MIRAGRID 3XT	15.58 (22.70)	1423 (20.80)	1309 (19.10)	1.67	Tu_1=829lb/ft+Ntan(39°) (Tu_1=12.1kN/m +Ntan(39°))	Tu_2=1715lb/ft+Ntan(6°) (Tu_2=25.0kN/m +Ntan(6°))	1257.3 (18.34)
Strata Systems Inc. - 380 Dahlonega Road - Cumming, Georga 30040 - (800) 680-7750							
STRATA 150	1008 (14.70)	962 (14.10)	882 (12.90)	1.61	Tu_1=853lb/ft+Ntan(10°) (Tu_1=12.4kN/m +Ntan(10°))	Tu_2=541lb/ft+Ntan(16°) (Tu_2=7.9kN/m +Ntan(16°))	1967.9 (28.71)
STRATA 200	1613 (23.60)	1540 (22.50)	1412 (20.60)	1.61	Tu_1=784lb/ft+Ntan(35°) (Tu_1=11.4kN/m +Ntan(35°))	Tu_2=1875lb/ft+Ntan(0°) (Tu_2=27.3kN/m +Ntan(0°))	1558.1 (22.73)

Table B-1 Pullout Resistance Equations

APPENDIX C

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Designing Balance into Your Retaining Wall Project

Engineers have the responsibility of designing cost effective structures that are safe and reliable. On the surface this task seems to be relatively straight forward and one that can easily be quantified. The questions that must be answered to achieve this design standard will determine how complicated this process will be.

What forces will be applied to the structure? What materials will be used to build the structure? Are there other elements that may affect the performance of the structure? During the construction process, what safeguards will be in place to ensure that plans and specifications are followed? What will be required after completion of the project for the continued safe, reliable performance of the structure? What has our experience told us about what can go wrong in real life?

These questions have led to a series of changes over the last twenty years in the design of segmental retaining walls. Allan Block has helped to drive the industry to ensure cost effectiveness with safety and reliability. During this time frame many things have evolved, and design refinements are producing a better final product that suits the needs of our customers.

From our field experience and full scale testing we have arrived at conclusions that change how we approach designs. The following design guidelines should be implemented to provide for a safer more reliable structure. This does not imply that the structures built over the last twenty years are not safe, but rather we have determined that with a few simple changes we can build safer yet still efficient retaining wall structures.

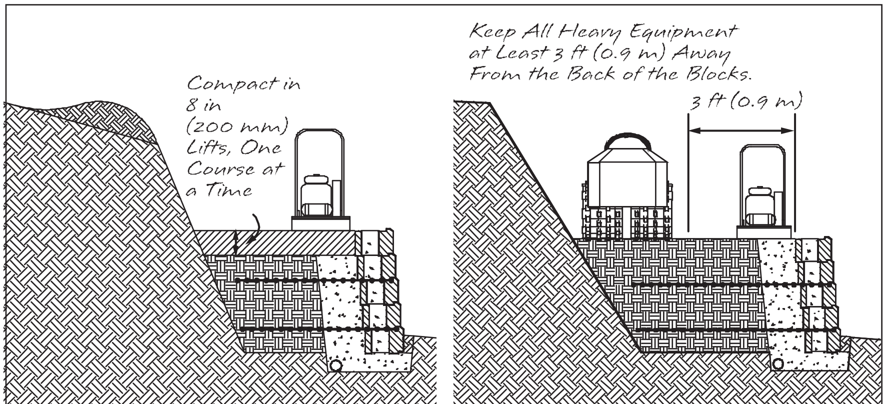

Figure C-1. Compaction

1. Compaction. Geogrid-reinforced structures are designed to perform as a composite structure. In order for them to perform in this manner, consistent compaction is mandatory. Actual installations are plagued with improper compaction due to soil lifts in excess of the maximum 8 in. (200 mm) lifts. Tighter specifications should be used on compaction and field testing requirements.

2. Geogrid Spacing. Compound failure planes may develop when the reinforced mass is constructed with geogrids that are not spaced close enough together. Allan Block recommends geogrid spacing of 16 in. (406 mm) or less. This is a more efficient way to distribute the reinforcement throughout the mass, which develops a more coherent structure. Since more layers of grid are installed, lower strength grids may be utilized and not affect the project budget, as long as all safety factors are met.

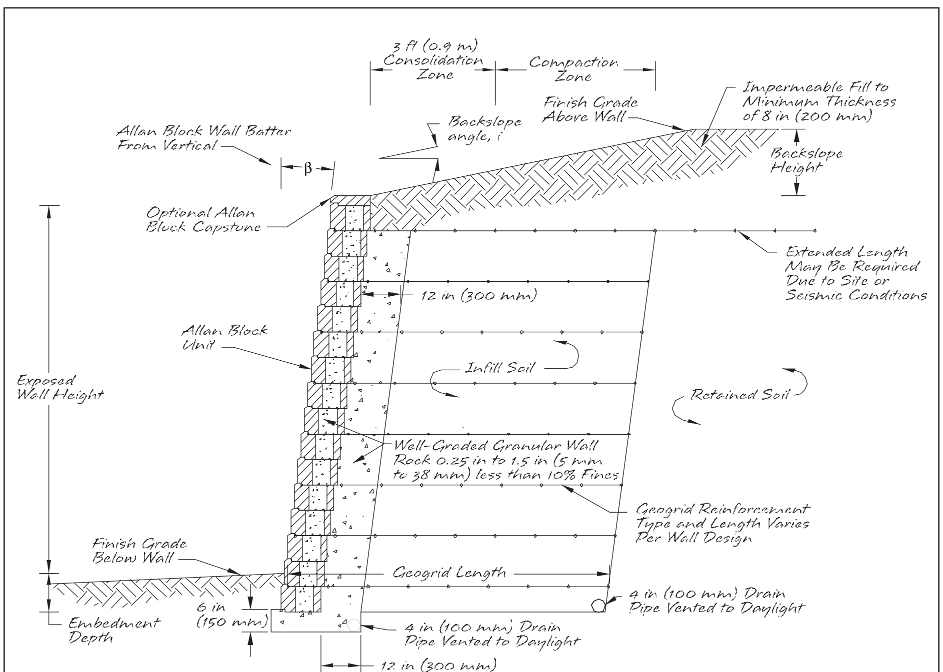

Figure C-2. Allan Block Typical Section

3. Geogrid Length. We have concluded that grid lengths between 50 and 60 percent of the wall height will provide a safe and efficient structure, but for simplicity we are recommending 60 percent as the typical grid length for a starting point. The exception is the top layer of grid which should be extended to intertwine the reinforced mass with the retained soil mass. This eliminates potential for soil cracks at the intersection of these two masses, by extending the top grid layers by approximately 3 ft (0.9 m), or to 90% of the wall height to tie the reinforced mass into the retained mass for seismic designs, walls with surcharges, or slopes above the infill mass.

4. Infill Soil. Onsite soils may be used as infill soil if they are of sufficient quality. Stay away from high plastic clays in the reinforced soil mass and use granular material whenever possible. When clay soils are used in the reinforced zone extra precautions should be employed to keep water from penetrating the mass. See Table 1 for the recommended materials for infill soil.

5. Water Management. The addition of water to the reinforced soil mass can change the soil properties dramatically. Designers need to understand and control surface and subsurface water flows. Wall rock and toe drains are intended for incidental water only, any excess surface or subsurface water should be routed away from the reinforced soil mass by using berms, swales and chimney drains.

Table 1: Inorganic USCS Soil Types:
GP, GW, SW, SP, SM meeting the following gradation as determined in accordance with ASTM D422.

Sieve Size	Percent Passing
4 inch	100 - 75
No. 4	100 - 20
No. 40	0 - 60
No. 200	0 - 35

Issues of design and construction will always be an ongoing evolutionary process. To accommodate this Allan Block has and will continue to invest in obtaining data from new experiences and full scale tests. Contact the Allan Block Engineering Department for additional assistance and visit our web site to obtain more information on designing segmental walls.

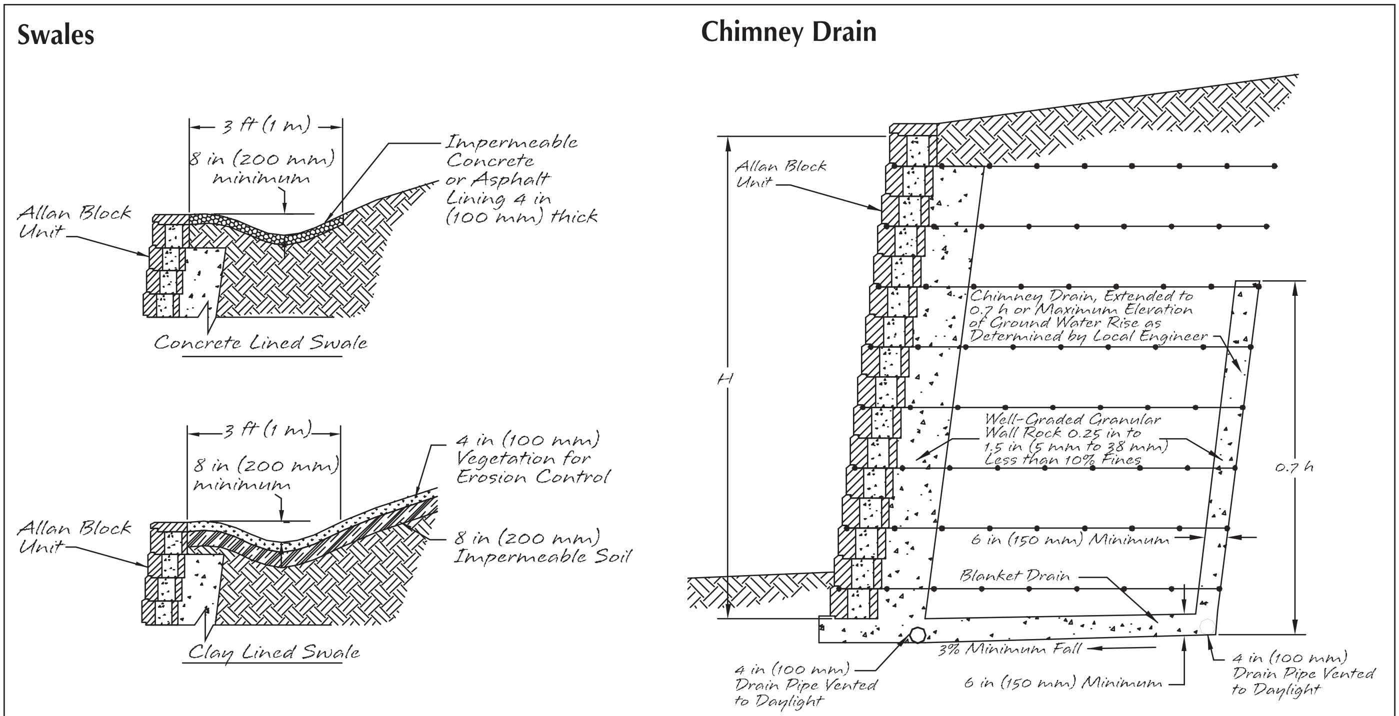

Figure C-4. Additional Water Management Options

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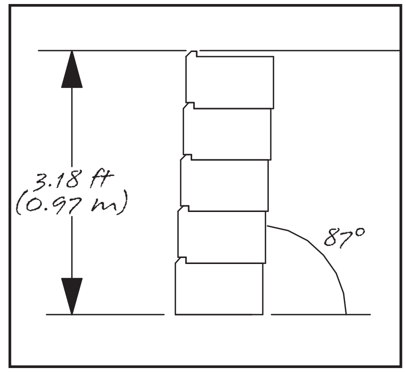
APPENDIX D

This example has been constructed following methodology outlined in this manual and the references listed on page 83.

Sample Calculations

Example S-1:

Given:



$i = 0°$

$\phi = 36°$

$\beta = 90 - 3 = 87°$

$\phi_w = (0.666)\,(36) = 24°$

$H = 3.18$ ft (0.97 m)

$\gamma = 120$ lb/ft^3 (1,923 kg/m^3)

$\gamma_{wall} = 130$ lb/ft^3 (2,061 kg/m^3)

$$K_a = \left[\frac{\csc(\beta)\sin(\beta-\phi)}{\sqrt{\sin(\beta+\phi_w)} + \sqrt{\dfrac{\sin(\phi+\phi_w)\sin(\phi-i)}{\sin(\beta-i)}}}\right]^2$$

$$K_a = \left[\frac{\csc(87)\sin(87-36)}{\sqrt{\sin(87+24)} + \sqrt{\dfrac{\sin(36+24)\sin(36-0)}{\sin(87-0)}}}\right]^2$$

$$K_a = \left[\frac{0.7782124}{0.966219657 + 0.713957656}\right]^2 = 0.2145$$

Find: The safety factor against sliding, SFS.

The first step is to determine the total active force exerted by the soil on the wall:

$F_a = (0.5)\,(\gamma)\,(K_a)\,(H)^2 = (0.5)\,(120\text{ lb/ft}^3)\,(0.2145)\,(3.18\text{ ft})^2 = 130\text{ lb/ft}$

$= (0.5)\,(\gamma)\,(K_a)\,(H)^2 = (0.5)\,(1{,}923\text{ kg/m}^3)\,(0.2145)\,(0.97\text{ m})^2 = 1{,}904\text{ N/m}$

$F_h = (F_a)\cos(\phi_w) = (130\text{ ft/lb})\cos(24°) = 119\text{ lb/ft}$

$= (F_a)\cos(\phi_w) = (1{,}904\text{ N/m})\cos(24°) = 1{,}739\text{ N/m}$

$F_v = (F_a)\sin(\phi_w) = (130\text{ ft/lb})\sin(24°) = 53\text{ lb/ft}$

$= (F_a)\sin(\phi_w) = (1{,}904\text{ N/m})\sin(24°) = 774\text{ N/m}$

$W_f = (\gamma_{wall})\,(H)\,(d) = (130\text{ lb/ft}^3)\,(3.18\text{ ft})\,(0.97\text{ ft}) = 401\text{ lb/ft}$

$= (\gamma_{wall})\,(H)\,(d) = (2{,}061\text{ kg/m}^3)\,(0.97\text{ m})\,(0.3\text{ m}) = 5{,}884\text{ N/m}$

$F_r = (V_t)\,(C_f) = (W_f + F_v)\tan(\phi) = (401\text{ lb/ft} + 53\text{ lb/ft})\tan(36°) = 330\text{ lb/ft}$

$= (V_t)\,(C_f) = (W_f + F_v)\tan(\phi) = (5{,}884\text{ N/m} + 774\text{ N/m})\tan(36°) = 4{,}837\text{ N/m}$

$SFS = \dfrac{F_r}{F_h} = \dfrac{330\text{ lb/ft}}{169\text{ lb/ft}} = 2.77 \geq 1.5$ OK $\qquad = \dfrac{F_r}{F_h} = \dfrac{4{,}837\text{ N/m}}{1{,}739\text{ N/m}} = 2.77 \geq 1.5$ OK

Find: The safety factor against overturning, SFO.

$$\Sigma M_r = (W_f)\,[(x_1) + (0.5)\,(H)\tan(90° - \beta)] + (F_V)\,[(x_2) + (0.333)\,(H)\tan(90° - \beta)]$$

$$= (401\text{ lb/ft})\,[(0.49\text{ ft}) + (0.5)\,(3.18\text{ ft})\tan(90° - 87°)] + (53\text{ lb/ft})\,[(0.97\text{ ft}) + (0.333)\,(3.18\text{ ft})\tan(90° - 87°)]$$

$$= 284\text{ ft-lb/ft}$$

$$= (5{,}884\text{ N/m})\,[(0.15\text{ m}) + (0.5)\,(0.97\text{ m})\tan(90° - 87°)] + (774\text{ N/m})\,[(0.3\text{ m}) + (0.333)\,(0.97\text{ m})\tan(90° - 87°)]$$

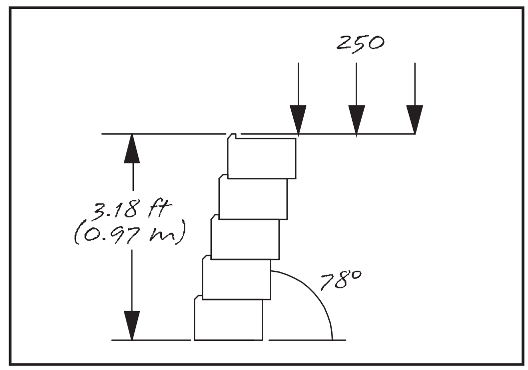
$$= 1{,}277\text{ N-m/m}$$

$$M_o = (F_h)\,(0.333)\,(H)$$

$$= (119\text{ lb/ft})\,(0.333)\,(3.18\text{ ft}) = 126\text{ ft-lb/ft} \qquad = (1{,}739\text{ N/m})\,(0.333)\,(0.97\text{ m}) = 562\text{ N-m/m}$$

$$SFO = \frac{\Sigma M_r}{\Sigma M_o} = \frac{(284\text{ ft-lb/ft})}{(176\text{ ft-lb/ft})} = 2.25 \geq 2.0 \quad OK \qquad = \frac{\Sigma M_r}{\Sigma M_o} = \frac{(1{,}277\text{ N-m/m})}{(562\text{ N-m/m})} = 2.25 \geq 2.0 \quad OK$$

Example S-2:



Given:

$\phi = 36°$	$\gamma = 120\text{ lb/ft}^3$ (1,923 kg/m³)
$H = 3.18\text{ ft}$ (0.97 m)	$\gamma_w = 130\text{ lb/ft}^3$ (2,061 kg/m³)
$\beta = 90 - 12 = 78°$	$q = 250\text{ lb/ft}^2$ (11,974 Pa)
$i = 0°$	$\phi_w = (0.666)\,(36) = 24°$

$$K_a = \left[\frac{\csc(\beta)\sin(\beta - \phi)}{\sqrt{\sin(\beta + \phi_w)} + \sqrt{\dfrac{\sin(\phi + \phi_w)\sin(\phi - i)}{\sin(\beta - i)}}}\right]^2$$

$$K_a = \left[\frac{\csc(78)\sin(78 - 36)}{\sqrt{\sin(78 + 24)} + \sqrt{\dfrac{\sin(36 + 24)\sin(36 - 0)}{\sin(78 - 0)}}}\right]^2$$

$$K_a = \left[\frac{0.684079382}{0.989013448 + 0.72139389}\right]^2 = 0.1599$$

Find: The safety factor against sliding, SFS.

The first step is to determine the total active force exerted by the soil on the wall:

$F_a = (0.5)(\gamma)(K_a)(H)^2 = (0.5)(120\ \text{lb/ft}^3)(0.1599)(3.18\ \text{ft})^2 = 97\ \text{lb/ft}$

$= (0.5)(\gamma)(K_a)(H)^2 = (0.5)(1{,}923\ \text{kg/m}^3)(0.1599)(0.97\ \text{m})^2 = 1{,}419\ \text{N/m}$

$F_h = (F_a)\cos(\phi_w) = (97\ \text{lb/ft})\cos(24°) = 89\ \text{lb/ft}$

$= (F_a)\cos(\phi_w) = (1{,}419\ \text{N/m})\cos(24°) = 1{,}296\ \text{N/m}$

$F_v = (F_a)\sin(\phi_w) = (97\ \text{lb/ft})\sin(24°) = 39\ \text{lb/ft}$

$= (F_a)\sin(\phi_w) = (1{,}419\ \text{N/m})\sin(24°) = 577\ \text{N/m}$

$W_f = (\gamma_{wall})(H)(d) = (130\ \text{lb/ft}^3)(3.18\ \text{ft})(0.97\ \text{ft}) = 401\ \text{lb/ft}$

$= (\gamma_{wall})(H)(d) = (2{,}061\ \text{kg/m}^3)(0.97\ \text{m})(0.3\ \text{m}) = 5{,}883\ \text{N/m}$

$F_r = (V_t)(C_f) = (W_f + F_v)\tan(\phi) = (401\ \text{lb/ft} + 39\ \text{lb/ft})\tan(36°) = 370\ \text{lb/ft}$

$= (V_t)(C_f) = (W_f + F_v)\tan(\phi) = (5{,}883\ \text{N/m} + 577\ \text{N/m})\tan(36°) = 4{,}693\ \text{N/m}$

$P_q = (q)(K_a) = (250\ \text{lb/ft}^2)(0.1599) = 40\ \text{lb/ft}^2$

$= (q)(K_a) = (11{,}974\ \text{N/m}^2)(0.1599) = 1{,}916\ \text{Pa}$

$P_{qh} = (P_q)\cos(\phi_w) = (40\ \text{lb/ft}^2)\cos(24°) = 37\ \text{lb/ft}^2$

$= (P_q)\cos(\phi_w) = (1{,}916\ \text{Pa})\cos(24°) = 1{,}750\ \text{Pa}$

$P_{qv} = (P_q)\sin(\phi_w) = (40\ \text{lb/ft}^2)\sin(24°) = 16\ \text{lb/ft}^2$

$= (P_q)\sin(\phi_w) = (1{,}916\ \text{Pa})\sin(24°) = 779\ \text{Pa}$

$F_{qh} = (P_{qh})(H) = (37\ \text{lb/ft}^2)(3.18\ \text{ft}) = 118\ \text{lb/ft}$

$= (P_{qh})(H) = (1{,}772\ \text{Pa})(0.97\ \text{m}) = 1{,}719\ \text{N/m}$

$F_{qv} = (P_{qv})(H) = (16\ \text{lb/ft}^2)(3.18\ \text{ft}) = 51\ \text{lb/ft}$

$= (P_{qv})(H) = (766\ \text{Pa})(0.97\ \text{m}) = 743\ \text{N/m}$

$$SFS = \frac{F_r + (F_{qv})(C_f)}{F_h + F_{qh}} = \frac{320\ \text{lb/ft} + (51\ \text{lb/ft})\tan(36°)}{89\ \text{lb/ft} + 118\ \text{lb/ft}} = 1.72 \geq 1.5 \quad \text{OK}$$

$$= \frac{F_r + (F_{qv})(C_f)}{F_h + F_{qh}} = \frac{4{,}693\ \text{N/m} + (743\ \text{N/m})\tan(36°)}{1{,}296\ \text{N/m} + 1{,}719\ \text{N/m}} = 1.72 \geq 1.5 \quad \text{OK}$$

Find: The safety factor against overturning, SFO.

$$\Sigma M_r = (W_f)\,[(X_1) + (0.5)\,(H)\tan(90° - \beta)] + (F_v)\,[(X_2) + (0.333)\,(H)\tan(90° - \beta)] + (F_{qv})\,[(X_2) + (0.5)\,(H)\tan(90° - \beta)]$$

$$\Sigma M_r = (401\text{ lb/ft})\,[(0.49\text{ ft}) + (0.5)\,(3.18\text{ ft})\tan(90° - 78°)] + (39\text{ lb/ft})\,[(0.97\text{ ft}) + (0.333)\,(3.18\text{ ft})\tan(90° - 78°)] + (51\text{ lb/ft})\,[(0.97\text{ ft}) + (0.5)\,(3.18\text{ ft})\tan(90° - 78°)] = 445\text{ ft-lb/ft}$$

$$= (5{,}883\text{ N/m})\,[(0.15\text{ m}) + (0.5)\,(0.97\text{ m})\tan(90° - 78°)] + (577\text{ N/m})\,[(0.3\text{ m}) + (0.333)\,(0.97\text{ m})\tan(90° - 78°)] + (743\text{ N/m})\,[(0.3\text{ m}) + (0.5)\,(0.97\text{ m})\tan(90° - 78°)] = 2{,}001\text{ N-m/m}$$

$$M_o = (F_h)\,(0.333)\,(H) + (F_{qh})\,(0.5)\,(H)$$

$$= (89\text{ lb/ft})\,(0.333)\,(3.18\text{ ft}) + (118\text{ lb/ft})\,(0.5)\,(3.18\text{ ft}) = 282\text{ ft-lb/ft}$$

$$= (1{,}296\text{ N/m})\,(0.333)\,(0.97\text{ m}) + (1{,}719\text{ N/m})\,(0.5)\,(0.97\text{ m}) = 1{,}252\text{ N-m/m}$$

$$SFO = \frac{\Sigma M_r}{\Sigma M_o} = \frac{(445\text{ ft-lb/ft})}{(282\text{ ft-lb/ft})} = \mathit{1.58} \geq \mathit{2.0}\quad \textit{NOT OK}$$

$$= \frac{\Sigma M_r}{\Sigma M_o} = \frac{(2{,}001\text{ N-m/m})}{(1{,}252\text{ N-m/m})} = \mathit{1.58} \geq \mathit{2.0}\quad \textit{NOT OK}$$

Example S-3:

Given:

ϕ = 27°	i = 0°	γ = 120 lb/ft³ (1,923 kg/m³)
H = 9.52 ft (2.9 m)	C_i = 0.75	γ_{wall} = 130 lb/ft³ (2,061 kg/m³)
β = 90 - 12 = 78°	ϕ_w = (0.666) (27) = 18°	q = 250 lb/ft² (11,974 Pa)

$$K_a = \left[\frac{\csc(\beta)\sin(\beta - \phi)}{\sqrt{\sin(\beta + \phi_w)} + \sqrt{\dfrac{\sin(\phi + \phi_w)\sin(\phi - i)}{\sin(\beta - i)}}}\right]^2$$

$$K_a = \left[\frac{\csc(78)\sin(78 - 27)}{\sqrt{\sin(78 + 18)} + \sqrt{\dfrac{\sin(27 + 18)\sin(27 - 0)}{\sin(78 - 0)}}}\right]^2$$

$$K_a = \left[\frac{0.794507864}{0.997257186 + 0.572880034}\right]^2 = 0.256$$

Find: The safety factor against sliding, SFS.

The first step is to determine the total active force exerted by the soil on the wall:

$F_a = (0.5)(\gamma)(K_a)(H)^2 = (0.5)(120\ lb/ft^3)(0.256)(9.52\ ft)^2 = 1,392\ lb/ft$

$= (0.5)(\gamma)(K_a)(H)^2 = (0.5)(1,923\ kg/m^3)(0.256)(2.9\ m)^2 = 20,307\ N/m$

$F_h = (F_a)\cos(\phi_w) = (1,392\ lb/ft)\cos(18°) = 1,324\ lb/ft$

$= (F_a)\cos(\phi_w) = (20,307\ N/m)\cos(18°) = 19,313\ N/m$

$F_v = (F_a)\sin(\phi_w) = (1,392\ lb/ft)\sin(18°) = 430\ lb/ft$

$= (F_a)\sin(\phi_w) = (20,307\ N/m)\sin(18°) = 6,275\ N/m$

$W_f = (\gamma_{wall})(H)(d) = (130\ lb/ft^3)(9.52\ ft)(0.97\ ft) = 1,200\ lb/ft$

$= (\gamma_{wall})(H)(d) = (2,061\ kg/m^3)(2.9\ m)(0.3\ m) = 17,590\ N/m$

$F_r = (V_t)(C_f) = (W_f + F_v)\tan(\phi) = (1,200\ lb/ft + 430\ lb/ft)\tan(27°) = 831\ lb/ft$

$= (V_t)(C_f) = (W_f + F_v)\tan(\phi) = (17,590\ N/m + 6,275\ N/m)\tan(27°) = 12,160\ N/m$

$SFS = \frac{F_r}{F_h} = \frac{831\ lb/ft}{1,324\ lb/ft} = 0.63 \geq 1.5$ NOT OK (Need Geogrid)

$= \frac{F_r}{F_h} = \frac{12,160\ N/m}{19,313\ N/m} = 0.63 \geq 1.5$ NOT OK (Need Geogrid)

Determine if a single layer of grid will work in calculation. This single grid layer example is for instructional purposes only. All actual reinforced mass designs require at least two layers of grid and most are designed using a two course spacing of geogrid from the bottom of wall to the top, regardless of the minimum grid layer calculations which follows.

$F_{gr} = 2(d_g)(\gamma)(L_e)(C_i)\tan(\phi)$

Find L_e.

$$L_e = \frac{833\ lb/ft}{2(5.08\ ft)(120\ lb/ft^3)(0.75)\tan(27°)} = 1.79\ ft$$

$$= \frac{12,161\ N/m}{2(1.55\ m)(18,865\ N/m)(0.75)\tan(27°)} = 0.544\ m$$

$L_t = L_w + L_a + L_e = 0.85 + (H - d_g)[\tan(45° - (\phi/2)) - \tan(90° - \beta)] + 1.79\ ft$
$= 0.85\ ft + (9.52\ ft - 5.08\ ft)[\tan(45° - 13.5°) - \tan(90° - 78°)] + 1.79\ ft$
$= 4.42\ ft$

$= L_w + L_a + L_e = 0.85 + (H - d_g)[\tan(45° - (\phi/2)) - \tan(90° - \beta)] + 0.544\ m$
$= 0.259\ m + (2.9\ m - 1.55\ m)[\tan(45° - 13.5°) - \tan(90° - 78°)] + 0.544\ m$
$= 1.34\ m$

Actual Embedment Length.

$L_e = (L_t - L_w - L_a)$
$= 4.42\ ft - 0.85\ ft - (9.52\ ft - 5.08\ ft)(0.4) = 1.79\ ft$
$= 1.34\ m - 0.259\ m - (2.9\ m - 1.55\ m)(0.4) = 0.544\ m$

Maximum potential restraining force with L_e = 1.79 ft (0.544 m).

$F_{gr} = 2(5.08\ ft)(120\ lb/ft^3)(1.79\ ft)(0.75)\tan(27°) = 833\ lb/ft$

$= 2(1.55\ m)(1,923\ kg/m^3)(0.541\ m)(0.75)\tan(27°) = 12,090\ N/m$

$$SFS = \frac{F_r + F_g}{F_h} = \frac{831 \text{ lb/ft} + 833 \text{ lb/ft}}{1{,}324 \text{ lb/ft}} = 1.25 \geq 1.5 \quad \text{NOT OK (Needs More Geogrid)}$$

$$= \frac{F_r + F_g}{F_h} = \frac{12{,}160 \text{ N/m} + 12{,}090 \text{ N/m}}{19{,}313 \text{ N/m}} = 1.25 \geq 1.5 \quad \text{NOT OK (Needs More Geogrid)}$$

$$L_{min} = 0.3\,(H) + 0.85 \text{ ft} + 2.4 \text{ ft} = 0.3\,(9.52 \text{ ft}) + 0.85 \text{ ft} + 1.79 \text{ ft} = 5.5 \text{ ft}$$

$$= 0.3\,(H) + 0.256 \text{ m} + 0.732 \text{ m} = 0.3\,(2.9 \text{ m}) + 0.256 \text{ m} + 0.544 \text{ m} = 1.67 \text{ m}$$

$$W_s = (\gamma_r)\,(H)\,(L_g - 0.85 \text{ ft}) = (125 \text{ lb/ft}^3)\,(9.52 \text{ ft})\,(5.5 \text{ ft} - 0.85 \text{ ft}) = 5{,}534 \text{ lb/ft}$$

$$= (\gamma_r)\,(H)\,(L_g - 0.256 \text{ m}) = (2{,}002 \text{ kg/m}^3)\,(2.9 \text{ m})\,(1.67 \text{ m} - 0.256 \text{ m}) = 80{,}534 \text{ N/m}$$

$$W_w = W_f + W_s = 1{,}200 \text{ lb/ft} + 5{,}534 \text{ lb/ft} = 6{,}734 \text{ lb/ft}$$

$$= W_f + W_s = 17{,}590 \text{ N/m} + 80{,}534 \text{ N/m} = 98{,}124 \text{ N/m}$$

Vertical Force; Solve using onsite soil

$$V_t = W_w + F_v = 6{,}734 \text{ lb/ft} + 430 \text{ lb/ft} = 7{,}164 \text{ lb/ft}$$

$$= W_w + F_v = 98{,}124 \text{ N/m} + 6{,}275 \text{ N/m} = 104{,}399 \text{ N/m}$$

$$F_r = (V_t)\,(C_f) = (7{,}164 \text{ lb/ft}) \tan(27°) = 3{,}650 \text{ lb/ft}$$

$$= (V_t)\,(C_f) = (104{,}399 \text{ N/m}) \tan(27°) = 53{,}193 \text{ N/m}$$

Pressure on the retaining wall due to the surcharge

$$P_q = (q)\,(K_a) = (250 \text{ lb/ft}^2)\,(0.256) = 64 \text{ lb/ft}^2$$

$$= (q)\,(K_a) = (11{,}974 \text{ Pa})\,(0.256) = 3{,}065 \text{ Pa}$$

Find the horizontal and vertical components of the pressure.

$$P_{qh} = (P_q) \cos(\phi_w) = (64 \text{ lb/ft}^2) \cos(18°) = 61 \text{ lb/ft}^2$$

$$= (P_q) \cos(\phi_w) = (3{,}065 \text{ Pa}) \cos(18°) = 2{,}915 \text{ Pa}$$

$$P_{qv} = (P_q) \sin(\phi_w) = (64 \text{ lb/ft}^2) \sin(18°) = 20 \text{ lb/ft}^2$$

$$= (P_q) \sin(\phi_w) = (3{,}065 \text{ Pa}) \sin(18°) = 947 \text{ Pa}$$

Finally, the total surcharge forces on the wall are calculated:

$$F_{qh} = (P_{qh})\,(H) = (61 \text{ lb/ft}^2)\,(9.52 \text{ ft}) = 581 \text{ lb/ft}$$

$$= (P_{qh})\,(H) = (2{,}915 \text{ Pa})\,(2.9 \text{ m}) = 8{,}454 \text{ N/m}$$

$$F_{qv} = (P_{qv})\,(H) = (20 \text{ lb/ft}^2)\,(9.52 \text{ ft}) = 190 \text{ lb/ft}$$

$$= (P_{qv})\,(H) = (947 \text{ Pa})\,(2.9 \text{ m}) = 2{,}746 \text{ N/m}$$

Find the safety factor against sliding:

$$SFS = \frac{F_r + (F_{qv}) \tan \phi}{F_h + F_{qh}} = \frac{3{,}650 \text{ lb/ft} + 190 \text{ lb/ft} (\tan 27°)}{1{,}324 \text{ lb/ft} + 581 \text{ lb/ft}} = 1.97 \geq 1.5 \text{ OK}$$

$$= \frac{F_r + (F_{qv})\,(F_{qv})}{F_h + F_{qh}} = \frac{53{,}193 \text{ N/m} + 2{,}746 \text{ N/m} (\tan 27°)}{19{,}313 \text{ N/m} + 8{,}454 \text{ N/m}} = 1.97 \geq 1.5 \text{ OK}$$

Find the safety factor against overturning:

$$\Sigma M_r = (W_f)[(0.5)(X_1) + (0.5)(H)\tan(90° - \beta)]$$
$$+ (W_s)[(0.5)(X_2 - X_1) + (X_1) + (0.5)(H)\tan(90° - \beta)]$$
$$+ (F_v)[(X_2) + (0.333)(H)\tan(90° - \beta)]$$
$$+ (F_{qv})[(X_2) + (0.5)(H)\tan(90° - \beta)]$$

$$= (1{,}200 \text{ lb/ft})[(0.5)(0.97 \text{ ft}) + (0.5)(9.52 \text{ ft})\tan(90° - 78°)]$$
$$+ (5{,}534 \text{ lb/ft})[(0.5)(5.62 \text{ ft} - 0.97 \text{ ft}) + (0.97 \text{ ft}) + (0.5)(9.52 \text{ ft})\tan(90° - 78°)]$$
$$+ (430 \text{ lb/ft})[(5.62 \text{ ft}) + (0.333)(9.52 \text{ ft})\tan(90° - 78°)]$$
$$+ (190 \text{ lb/ft})[(5.62 \text{ ft}) + (0.5)(9.52 \text{ ft})\tan(90° - 78°)]$$
$$= 29{,}596 \text{ ft-lb/ft}$$

$$= (17{,}590 \text{ N/m})[(0.5)(0.297 \text{ m}) + (0.5)(2.9 \text{ m})\tan(90° - 78°)]$$
$$+ (80{,}534 \text{ N/m})[(0.5)(1.71 \text{ m} - 0.297 \text{ m}) + (0.297 \text{ m}) + (0.5)(2.9 \text{ m})\tan(90° - 78°)]$$
$$+ (6{,}275 \text{ N/m})[(1.71 \text{ m}) + (0.333)(2.9 \text{ m})\tan(90° - 78°)]$$
$$+ (2{,}746 \text{ N/m})[(1.71 \text{ m}) + (0.5)(2.9 \text{ m})\tan(90° - 78°)]$$
$$= 131{,}230 \text{ N-m/m}$$

$$M_o = (F_h)(0.333)(H) + (F_{qh})(0.5)(H)$$
$$= (1{,}324 \text{ lb/ft})(0.333)(9.52 \text{ ft}) + (581 \text{ lb/ft})(0.5)(9.52 \text{ ft}) = 6{,}962 \text{ ft-lb/ft}$$
$$= (19{,}313 \text{ N/m})(0.333)(2.9 \text{ m}) + (8{,}454 \text{ N/m})(0.5)(2.9 \text{ m}) = 30{,}909 \text{ N-m/m}$$

$$SFO = \frac{\Sigma M_r}{\Sigma M_o} = \frac{(29{,}596 \text{ ft-lb/ft})}{(6{,}962 \text{ ft-lb/ft})} = \mathit{4.25 \geq 2.0 \quad OK}$$

$$= \frac{\Sigma M_r}{\Sigma M_o} = \frac{(131{,}230 \text{ N/m})}{(30{,}909 \text{ N/m})} = \mathit{4.25 \geq 2.0 \quad OK}$$

Internal Stability:

ϕ_r = 30°

γ_r = 125 lb/ft^3 (2,002 kg/m^3)

ϕ_{wr} = 0.666 (30°) = 20°

$$K_{ar} = \left[\frac{\csc(78)\sin(78 - 30)}{\sqrt{\sin(78 + 19.98)} + \sqrt{\dfrac{\sin(30 + 19.98)\sin(30 - 0)}{\sin(78 - 0)}}}\right]^2$$

$$K_{ar} = \left[\frac{0.759747}{0.995147 + 0.625671}\right]^2 = 0.2197$$

P_{qh} = $(q)(K_{ar})\cos(\phi_{wr})$ = $(250\ \text{lb/ft}^2)(0.2197)\cos(20°)$ = $52\ \text{lb/ft}^2$

= $(q)(K_{ar})\cos(\phi_{wr})$ = $(11{,}974\ \text{Pa})(0.2197)\cos(20°)$ = $2{,}472\ \text{Pa}$

Quadratic equation = $\dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

z = $(K_{ar})\cos(\phi_{wr})$ = $(0.2197)\cos(20°)$ = 0.2065

a = $(\gamma_r)(z)$ = $(125\ \text{lb/ft}^3)(0.2065)$ = $26\ \text{lb/ft}^3$

= $(\gamma_r)(z)$ = $(2{,}002\ \text{kg/m}^3)(0.2065)$ = $413\ \text{kg/m}^3$

b = $-2\,[(d_1)(a) + (q)(z)] = -2\,[(9.52\ \text{ft})(26\ \text{lb/ft}^3) + (250\ \text{lb/ft}^2)(0.2065)]$
= $-598\ \text{lb/ft}^2$

= $-2\,[(d_1)(a) + (q)(z)] = -2\,[(2.9\ \text{m})(413\ \text{kg/m}^3) + (1{,}220\ \text{kg/m}^2)(0.2065)]$
= $-2{,}899\ \text{kg/m}^2$

c = $(2)(F_{ga})$ = $(2)(833\ \text{lb/ft})$ = $1{,}666\ \text{lb/ft}$

= $(2)(F_{ga})$ = $(2)(12{,}161\ \text{N/m})$ = $24{,}322\ \text{N/m}$

d_h = $\dfrac{-(-598\ \text{lb/ft}^2) \pm \sqrt{(-598\ \text{lb/ft}^2)^2 - 4\,(26\ \text{lb/ft}^3)(1{,}666\ \text{lb/ft})}}{2\,(26\ \text{lb/ft}^3)}$

= $\dfrac{(598\ \text{lb/ft}^2) \pm (429\ \text{lb/ft}^2)}{52\ \text{lb/ft}^3}$ = 19.75 or 3.25

The wall is only 9.52 ft (2.9 m) tall so 19.75 (6.02 m) is not valid.

= $\dfrac{-(-2{,}899\ \text{kg/m}^2) \pm \sqrt{(-2{,}899\ \text{kg/m}^2)^2 - 4\,(413\ \text{kg/m}^3)(2{,}479\ \text{kg/m})}}{2\,(413\ \text{kg/m}^3)}$

= $\dfrac{(2{,}899\ \text{kg/m}^2) \pm (2{,}076\ \text{kg/m}^2)}{826\ \text{kg/m}^3}$ = 6.02 or 1.0

d_2 = $d_1 - d_h$ = $9.52\ \text{ft} - 3.25\ \text{ft}$ = $6.27\ \text{ft}$

= $d_1 - d_h$ = $2.9\ \text{m} - 1.0\ \text{m}$ = $1.9\ \text{m}$

The first layer of geogrid is placed at 1/2 dh.

h_g = $1/2\ d_h$ = $1/2\,(3.25\ \text{ft})$ = $1.625\ \text{ft}$

= $1/2\ d_h$ = $1/2\,(1.0\ \text{m})$ = $0.5\ \text{m}$

Analysis to determine if more than one additional layer of geogrid is required;

F_h = $0.5\,(\gamma_r)(K_{ar})(d_2)^2\cos(\phi_{wr})$ = $0.5\,(125\ \text{lb/ft}^3)(0.2197)(6.27\ \text{ft})^2\cos(30°)$
= $467\ \text{lb/ft}$

= $0.5\,(\gamma_r)(K_{ar})(d_2)^2\cos(f_{wr})$ = $0.5\,(2{,}002\ \text{kg/m}^3)(0.2197)(1.9\ \text{m})^2\cos(30°)$
= $6{,}745\ \text{N/m}$

Q_h = $(q)(K_{ar})(d_2 - h_g)\cos(\phi_{wr}) = (250\ \text{lb/ft}^2)(0.2197)(6.27\ \text{ft} - 1.625\ \text{ft})\cos(20°)$
= $240\ \text{lb/ft}$

= $(q)(K_{ar})(d_2 - h_g)\cos(\phi_{wr}) = (1{,}220\ \text{kg/m}^2)(0.2197)(1.9\ \text{m} - 0.5\ \text{m})\cos(20°)$
= $3{,}459\ \text{N/m}$

$F_t \quad = F_h + Q_h = 467\ \text{lb/ft} + 240\ \text{lb/ft} = 707\ \text{lb/ft}$

$\quad = F_h + Q_h = 6{,}745\ \text{N/m} + 3{,}459\ \text{N/m} = 10{,}204\ \text{N/m}$

$F_t \quad = 707\ \text{lb/ft} < 833\ \text{lb/ft}$ Only one more layer of geogrid is required.

$\quad = 10{,}204\ \text{N/m} < 12{,}161\ \text{N/m}$ Only one more layer of geogrid is required.

$h_g \quad = (H - d_2) + 0.5\ (d_h) = (9.52\ \text{ft} - 6.27\ \text{ft}) + 0.5\ (3.25) = 4.875\ \text{ft}$

$\quad = (H - d_2) + 0.5\ (d_h) = (2.9\ \text{m} - 1.9\ \text{m}) + 0.5\ (1.0\ \text{m}) = 1.5\ \text{m}$

Check number of layers of geogrid required.

$F_h \quad = 0.5\ (\gamma_r)\ (K_{ar})\ (H)^2 \cos(\phi_{wr}) = 0.5\ (125\ \text{lb/ft}^3)\ (0.2197)\ (9.52\ \text{ft})^2 \cos(20°)$
$\quad = 1{,}169\ \text{lb/ft}$

$\quad = 0.5\ (\gamma_r)\ (K_{ar})\ (H)^2 \cos(\phi_{wr}) = 0.5\ (2002\ \text{kg/m}^3)\ (0.2197)\ (2.9\ \text{m})^2 \cos(20°)$
$\quad = 17{,}050\ \text{N/m}$

$Q_h \quad = (q)\ (K_{ar})\ (H - h_g) \cos(\phi_{wr}) = (250\ \text{lb/ft}^2)\ (0.2197)\ (9.52\ \text{ft} - 1.625\ \text{ft}) \cos(20°)$
$\quad = 407\ \text{lb/ft}$

$\quad = (q)\ (K_{ar})\ (H - h_g) \cos(\phi_{wr}) = (11{,}974\ \text{N/m}^2)\ (0.2197)\ (2.9\ \text{m} - 0.5\ \text{m}) \cos(20°)$
$\quad = 5{,}933\ \text{N/m}$

$F_t \quad = F_h + Q_h = 1{,}169\ \text{lb/ft} + 407\ \text{lb/ft} = 1{,}576\ \text{lb/ft}$

$\quad = F_h + Q_h = 17{,}050\ \text{N/m} + 5{,}933\ \text{N/m} = 22{,}983\ \text{N/m}$

$$\frac{F_t}{LTADS} = N = \frac{1{,}576\ \text{lb/ft}}{833\ \text{lb/ft}} = 1.89 = 2\ \text{Layers}$$

$$= N = \frac{22{,}983\ \text{N/m}}{12{,}161\ \text{N/m}} = 1.89 = 2\ \text{Layers}$$

One layer of grid will not be sufficient for the stability of this 9.52 ft (2.9 m) tall wall. A 9.52 ft (2.9 m) tall wall will have 15 block courses. Typically a geogrid reinforced wall will be designed and constructed using geogrid on every other block course minimum. That would give this wall 7 layers of geogrid starting above the bottom course. They would also be designed with a minimum length of grid equal to 60% of the wall height and increased from there as the design requires.

APPENDIX E

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This manual uses a working Stress Approach to the analysis of segmental retaining walls. When using a working Stress Approach the final analysis should yield a Factor of Safety for Static Conditions of 1.3 to 2.0 depending on the condition being analyzed. The following examples have converted the approach outlined in this manual into a Limit States Design Approach. The main difference between a working Stress Approach and Limit States Approach is based on the introduction of load factors and reduction factors. The net result of either approach should yield similar wall designs. Final Factors of Safety for a Limit States Approach are only required to exceed 1.0, due to the fact that reductions and load factors are applied during the analysis.

Example: Limit States Design Analysis for a Gravity Wall

Given:

$\phi = 36°$	$H = 3.15$ ft	(0.96 m)
$i = 0°$	$\gamma = 120$ lb/ft^3	(1,923 kg/m^3)
$\beta = 78°$	$\gamma_{wall} = 130$ lb/ft^3	(2,061 kg/m^3)
$K_a = 0.16$		

Load Factors:

Overturning Dead Loads	Gdo = 1.5	Reflects level of knowledge/certainty of applied loads, ranges 1.5 - 2.0.
Resisting Dead Loads	Gdr = 0.95	Reflects level of knowledge/certainty of restraining loads, ranges 0.5 - 1.0.
Soil Friction Uncertainty Factor	$\phi_u = 1.0$	Recommended value of 1.0 for soils with tested friction angle values. Otherwise uncertainty factor ranges 0.6 - 1.0.
Structure Classification Factor	$\phi_n = 1.0$	Reflects effects of adjacent structures. If none then 1.0, otherwise 0.9 - 1.1.
Base Sliding Coefficient	Cds = 1.0	Interaction certainty at base. Taken as 1.0 unless geogrid is present below first block course.

Design Friction Angle

$\phi_d = \text{atan}\,[(\phi_u)(\tan\phi)] = 36°$

$\phi_w = 0.666\,\phi_d = 24°$

Horizontal Force Exerted by the soil:

$F_h = 0.5\,K_a\,[(\text{Gdo})(\gamma)]\,(H^2)\,(\cos)\,\phi_w$

$= 130$ lb/ft (1.92 kN/m)

Weight of the Facing:

$W_f = \text{Gdr}\,(\gamma_{wall})\,(H)\,(t)$

$= 377$ lb/ft (5.56 kN/m)

Sliding Failure

$F_r = (\phi_n)\,(W_f)\,(\tan)\,(\phi_d)\,(\text{Cds})$

$= 273$ lb/ft (4.04 kN/m)

$SFS = \dfrac{F_r}{F_h} = 2.1 > 1.0$ ok

Overturning Failure

$M_r = W_f\,(t/2 + (0.5\,H)\tan(90 - \beta))$

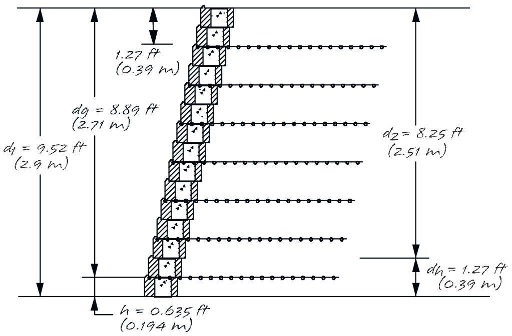
$= 309$ ft-lb/ft (1.29 kN-m/m)

$M_o = F_h\,(0.333\,h)$

$= 136$ ft-lb/ft (0.613 kN-m/m)

$SFO = \dfrac{M_r}{M_o} = 2.3 > 1.0$ ok

Example: Limit States Design Analysis for a Coherent Gravity Wall



Given:

ϕ = 27°	H = 9.52 ft	(2.9 m)
i = 0°	γ = 120 lb/ft³	(1,923 kg/m³)
β = 78°	γ_{wall} = 130 lb/ft³	(2,061 kg/m³)
K_a = 0.26	L_t = 6.0 ft	(1.87 m)
	L_s = 0.13 ft	(0.04 m)
	L_t = 0.97 ft	(0.296 m)

Overturning Dead Loads	Gdo = 1.5	Reflects level of knowledge/certainty of applied loads, ranges 1.5 - 2.0.
Resisting Dead Loads	Gdr = 0.95	Reflects level of knowledge/certainty of restraining loads, ranges 0.5 - 1.0.
Soil Friction Uncertainty Factor	ϕ_u = 1.0	Recommended value of 1.0 for soils with tested friction angle values. Otherwise uncertainty factor ranges 0.6 - 1.0.
Structure Classification Factor	ϕ_n = 1.0	Reflects effects of adjacent structures. If none then 1.0, otherwise 0.9 - 1.1.
Base Sliding Coefficient	Cds = 1.0	Interaction certainty at base. Taken as 1.0 unless geogrid is present below first block course.

Design Friction Angle

ϕ_d = atan [(ϕ_u) (tanϕ)] = 27°

ϕ_w = 0.666 ϕ_d = 18°

External Stability

Horizontal Force Exerted by the soil:

F_h = 0.5 K_a [(Gdo) (γ)] (H^2) (cos) ϕ_w
= 2,017 lb/ft (29.6 kN/m)

Weight of the Facing:

W_f = (Gdr) (γwall) (H) (t)
= 1,140 lb/ft (16.8 kN/m)

Weight of Reinforced Soil Mass:

W_s = (Gdr) (γ)(H) (L_t − t + L_s)
= 5,600 lb/ft (82.4 kN/m)

W_w = W_f + W_s = 6,740 lb/ft (99.2 kN/m)

Sliding Failure

F_r = (ϕ_n) (W_w) (tan) (ϕ_d) (Cds)
= 3,434 lb/ft (50.5 kN/m)

SFS $= \dfrac{F_r}{F_h}$ = 1.7 > 1.0 ok

Overturning Failure

M_r = W_f (t/2 + (0.5 H) tan (90 - β))
+ Ws [0.5 (L_t − t + L_s) + t + (0.5 H) tan (90 − β)]
= 27,250 ft-lb/ft (123 kN-m/m)

M_o = F_h (0.333 h)
= 6,394 ft-lb/ft (28.5 kN-m/m)

SFO $= \dfrac{M_r}{M_o}$ = 4.3 > 1.0 ok

Internal Stability

Partial Factors on Geogrid Strength:

Major geogrid manufacturers subject their materials to extensive testing to provide information for expected long term behavior. The resulting factors can vary greatly depending on geogrid material type and soil type. We suggest that specific data from a geogrid manufacturer be obtained over the give factors or ranges, which are typical values for most major manufacturers.

Service Life (in years):	SL = 100	Duration of Test (in hours):	TD = 100
Geogrid Type:	Polyester	Backfill Type (fine or coarse):	Fine

			Range of Values
Product Uncertainty Factor:	ϕup	= 1.0	0.95 - 1.0
Creep Reduction Factor:	ϕrc	= 0.6	0.17 - 0.60
Extrapolation Uncertainty Factor:	ϕue	= 0.95	0.50 - 1.0
Construction Damage Factor:	ϕri	= 0.78	0.60 - 0.90
Thickness Reduction Factor:	ϕrt	= 1.0	0.90 - 1.0
Strength Reduction Factor:	ϕrs	= 0.90	0.50 - 0.90
Temperature Reduction Factor:	ϕrst	= 1.0	
Degradation Factor:	ϕd	= 0.80	

Partial Factors on Soil/Geogrid Interaction and Geogrid Connection:

Pullout Uncertainty Factor:	$\phi upull$	= 0.80	0.75 - 0.80
Coefficient of Pullout Resistance:	Kpull	= 0.70	
Connection Uncertainty Factor:	$\phi uconn$	= 0.75	

Geogrid Properties:

Ultimate Tensile Strength:

Grid Type A = TuA = 35.04 kN/m

Design tensile Strength of Reinforcement:

TdA = (TuA) (ϕup) [(ϕrc) (ϕue)] (ϕri) [(ϕrt) (ϕrs) (ϕrst)] (ϕd) (ϕn)

TdA = 11.22 kN/m

Force on bottom grid layer:

$$F_g = Ka \left[\frac{(Gdo)\,(\gamma)\,(d_1 + d_2)}{2} \right] dh$$

= 529 lb/ft (7.72 kN/m)

Pullout resistance:

Geogrid beyond the line of maximum tension

$$L_e = Lt - \frac{h}{\tan(45 + \phi d/2)} - t + L_s$$

*Equation varies for two part maximum tension line.

Maximum Potential Restraining Force:

$$Fgr = (2)\,(Kpull)\,(L_e)\,(\phi upull)\,(Gdr)\,(dg)\,(\gamma)\tan(\phi_d)\,(\phi_n)$$

= 2,761 lb/ft (40.3 kN/m)

$$SF_{pullout} = \frac{Fgr}{Fg} = 5.2 > 1.0 \text{ ok}$$

Connection Strength:

Peak Connection Strength

$$Fcs = [acs + (\gamma_w)\,(dg)\tan(\lambda c)]\,(\phi uconn)\,(\phi_n)$$

= 1,107 lb/ft (16.15 kN/m)

$$Sf_{conn} = \frac{Fcs}{(0.666)\,(F_g)} \quad 3.14 > 1.0 \text{ ok}$$

Connection Strengths

Connection Strength Intercept:

acs = 1,313 lb/ft 19.16 kN/m

Connection Strength Slope:

λ = 8°

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REFERENCES

- Kliethermes, J., K. Buttry, E. McCullough, and R. Wetzel. "Modular Concrete Retaining Wall and Geogrid Performance and Laboratory Modeling." University of Wisconsin-Platteville, 1990.
- Leshchinsky, D. and E.B. Perry. "A Design Procedure for Geotextile Reinforced Walls." Geotechnical Fabrics Report. St. Paul: July/August, 1987.
- McKittrick, D.P. "Reinforced Earth: Application of Theory and Research to Practice." Reinforced Earth Technical Services, Report 79-1. Arlington, VA: The Reinforced Earth Company, 1978.
- Minnesota Department of Transportation. "Walls." Section 9-4.0 in Road Design Manual -- Part II. St. Paul: 1985.
- Peck, Ralph. "Earth Retaining Structures and Slopes." Chapter 13 in Soil Science, edited by T.W. Lambe and R.V. Whitman. New York: John Wiley & Sons, Inc., 1969.
- Sowers, G.B., and G.F. Sowers. "Problems in Earth Pressure." Chapter 8 in Introductory Soil mechanics and Foundations. New York: The Macmillan Company, 1970.
- R. F. Craig, Chapman & Hall, “Soil Mechanics” Fifth Edition, 1992
- Braja M. Das “Principles of Geotechnical Engineering” Third Edition, Chapter 10, 1994
- Columbia University in Cooperation with Allan Block Corporation and Huesker Geosynthetics. “Executive Summary - Seismic Testing - Geogrid Reinforced Soil Structures Faced with Segmental Retaining Wall Block”, 2003
- Ling, H, etal: “Large Scale Shaking Table Tests on Modular-Block Reinforced Soil Retaining Walls”, Journal of the Geotechnical and Geoenvironmental Engineering ASCE, April 2005
- Robert Shukha, Rafael Baker and Dov Leshchinsky, “Engineering Implications of the Relation between Static and Pseudo-Static Slope Stability Analysis”, EJGE Paper 2005-616
- Victor Elias, PE, Barry Christopher, PhD, Ryan Berg PE, “Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guide Lines”, FHWA NHI-00-043, March 2001
- American Institute of Steel Construction Inc., “Manual of Steel Construction - Load and Resistance Factor Design”, First Edition, 3rd Printing, March 1990
- Erickson Engineering Consulting Engineers - Joseph Arsenault and Brian Waters, PE
- Hoe I. Ling, et.al. Large-Scale Shaking Table tests on Modular-Block Reinforced Soil Retaining Walls; Tsukuba, Japan 2005
- Soil Strength and Slope Stability, J. Michael Duncan and Stephen G. Wright, Wiley Publications, 2005
- Slope Stability and Stabilization Methods, second edition, Lee W. Abramson, Thomas S. Lee, Sunil Sharma, Glenn M. Boyce, Wiley Publications, 2002
- Commercial Installation Manual for Allan Block Retaining Walls, 2010
- AB Spec Book, A Comprehensive Guide and Design Details for Segmental Retaining Walls, 2007

Refer to the AB Spec Book for details when applying the engineering principles outlined in this manual. The AB Spec Book addresses many common issues that should be detailed in the final approved design.

This technical specification manual will allow a wall designer to source and reference specific information for use in developing project documents. The information shown here is for use with Allan Block products only. Visit allanblock.com for the most current information.

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