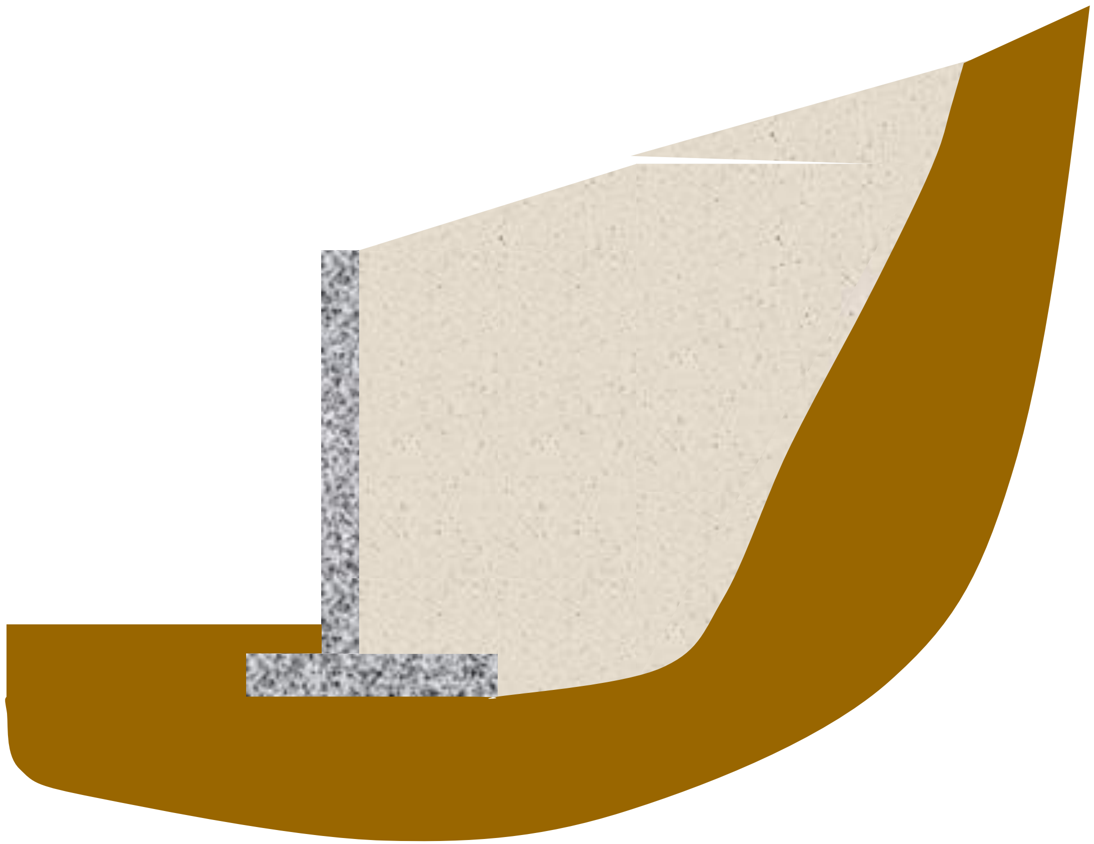
RETAINING WALLS

CHAPTER 13



Omitted parts:

Section 13.9 13.15-13.17

INTRODUCTION

Retaining walls are structures that restrain soil or other materials at locations having an abrupt change in elevation.

In the preceding chapter, you were introduced to various theories of lateral earth pressure. Those theories will be used in this chapter to design various types of retaining walls.

In general, retaining walls can be divided into two major categories:
(a) Conventional retaining walls
(b) Mechanically stabilized earth walls.

When a retaining wall is used to support the end of a bridge span as well as retaining the earth backfill, it is called an abutment.

Bridge abutments differ in two major respects from the usual retaining wall in:

1) The carry end reactions from the bridge span

2) They are restrained at the top so that an active earth pressure is unlikely to develop.

COMMON TYPES OF RETAINING WALLS

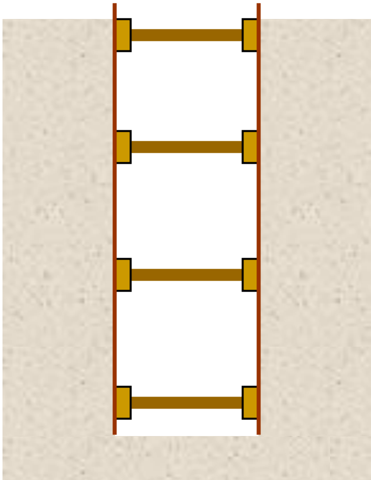
Conventional retaining walls can generally be classified into four varieties:

1. **Gravity retaining walls**
2. **Semigravity retaining walls**
3. **Cantilever retaining walls**
4. **Counterfort retaining walls**

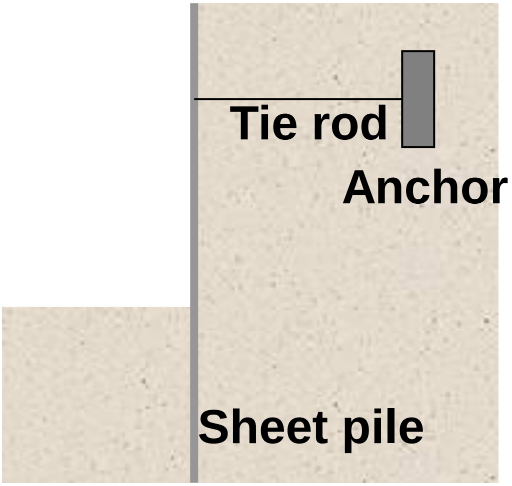
- **Most of these types are permanent.**
- **Some types of the embedded retaining walls (such as sheet Pile wall and braced cut) are used temporarily during construction.**
- **The temporary retaining work is called "shoring".**

INTRODUCTION

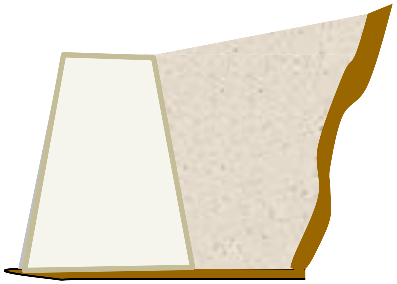
Cantilever retaining wall



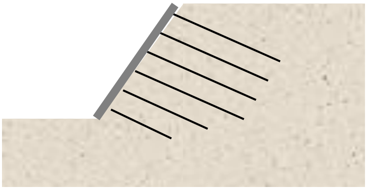
Braced excavation



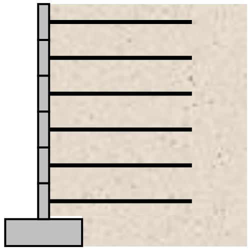
Anchored sheet pile



Gravity Retaining wall



Soil nailing



Reinforced earth wall

- **We have to estimate the lateral soil pressures acting on these structures, to be able to design them.**

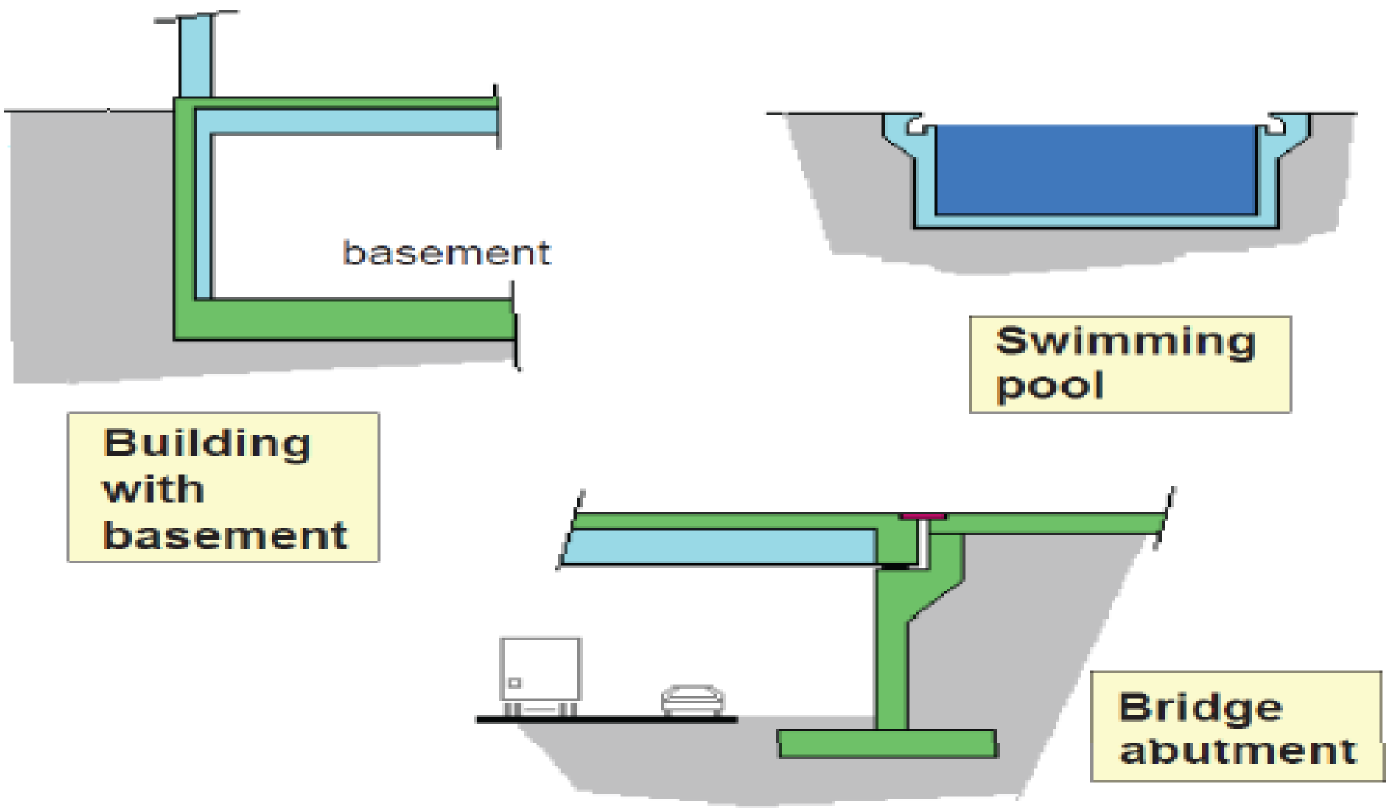
APPLICATIONS OF RETAINING WALLS

Different forms

Different sizes

Different loadings

Different failure concerns



APPLICATIONS OF RETAINING WALLS

APPLICATIONS OF RETAINING WALLS

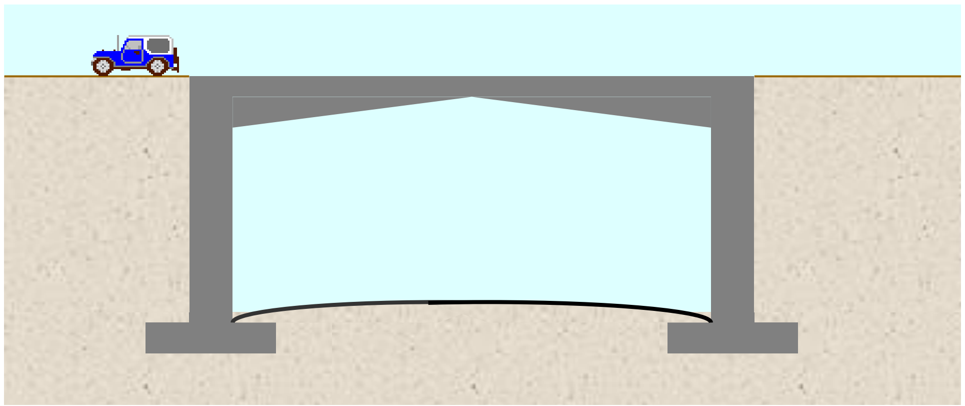
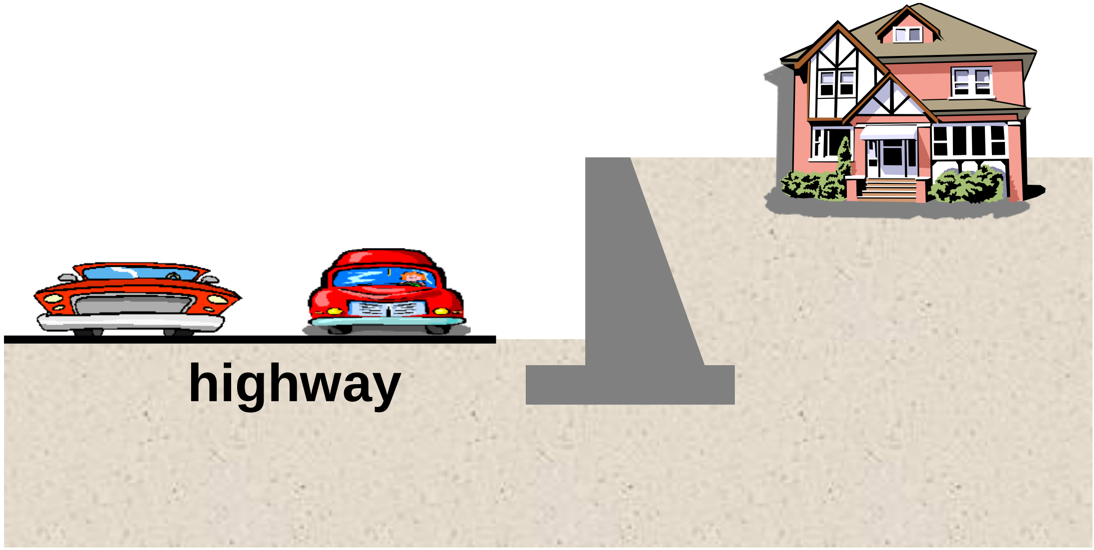


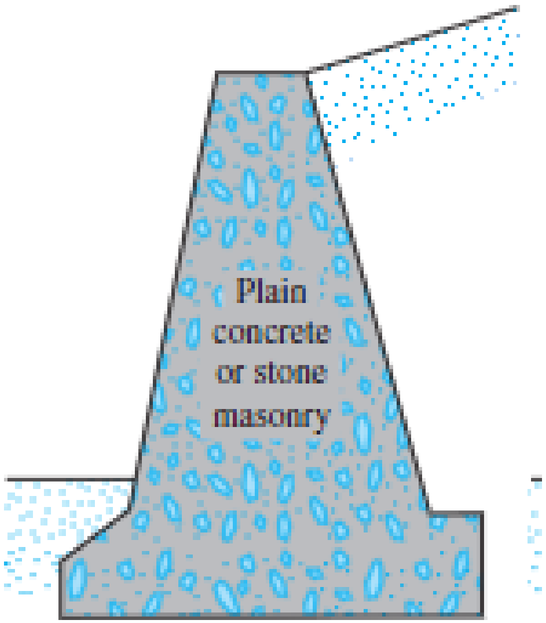
Photo: John Ricard, Maguire Group

APPLICATIONS OF RETAINING WALLS



GRAVITY RETAINING WALLS

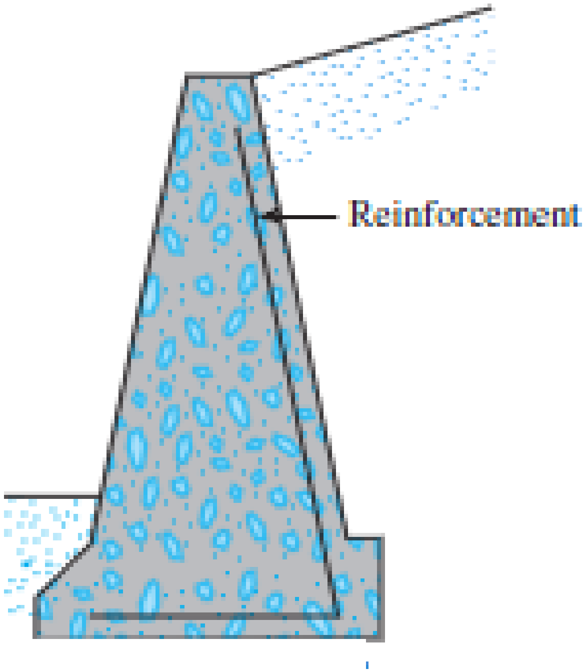
- This type of wall has relatively huge size and weight, and **not economical for high walls.**
- **They rely on their self weight to support the backfill** and achieve stability against failures.
- The following are the main types of wall:
  - Masonry gravity walls
  - Concrete gravity walls



MASONRY WALLS

SEMIGRAVITY RETAINING WALLS

In many cases, a small amount of steel may be used for the construction of gravity walls, thereby minimizing the size of wall sections



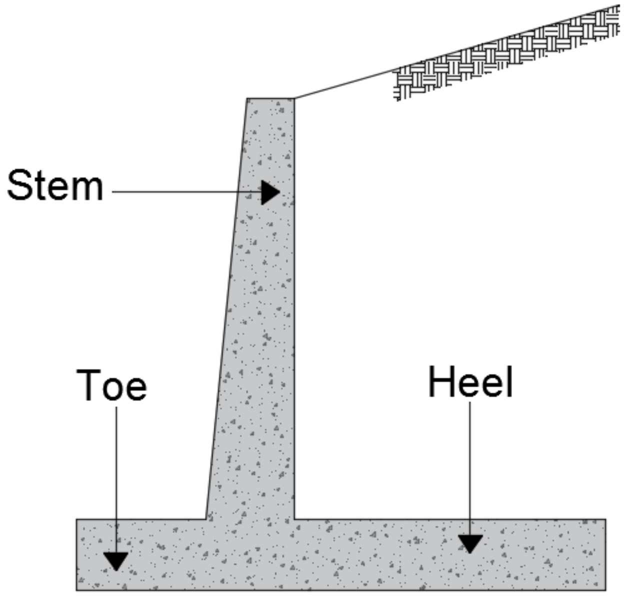
CANTILEVER RETAINING WALLS

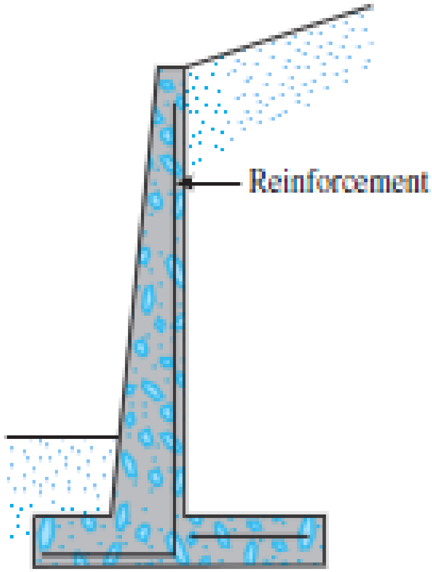
- **Cantilever retaining walls are made of reinforced concrete that consists of a thin stem and a base slab.**

- **This type of wall is economical to a height of about 8 m.**

- **The cantilever wall utilizes cantilever action to retain the mass behind the wall from assuming a natural slope.**

- **Stability of this wall is partially achieved from the weight of soil on the heel portion of the base slab.**



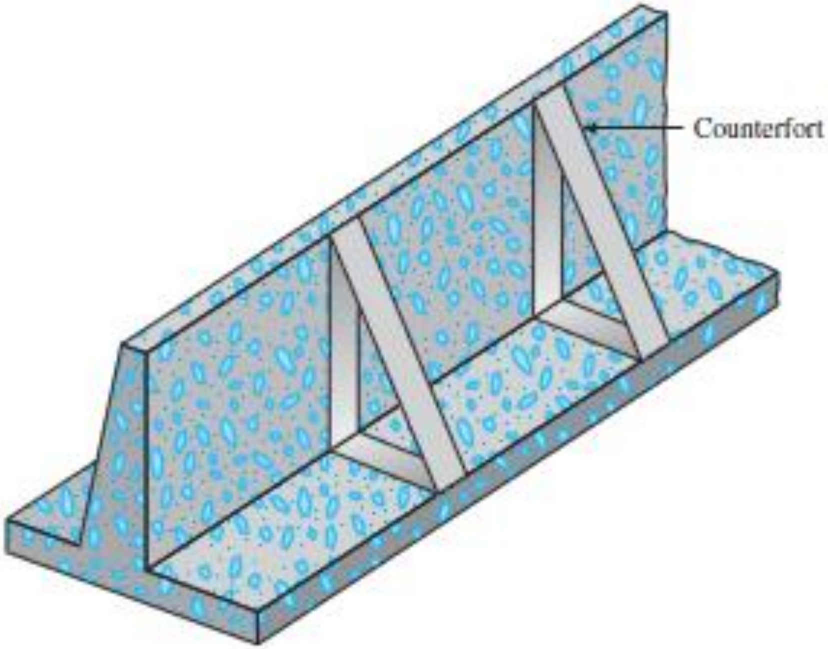


CANTILEVER WALLS

Made of reinforced concrete that consists of a thin stem and a base slab. This type of wall is economical to a height of about 8 m.

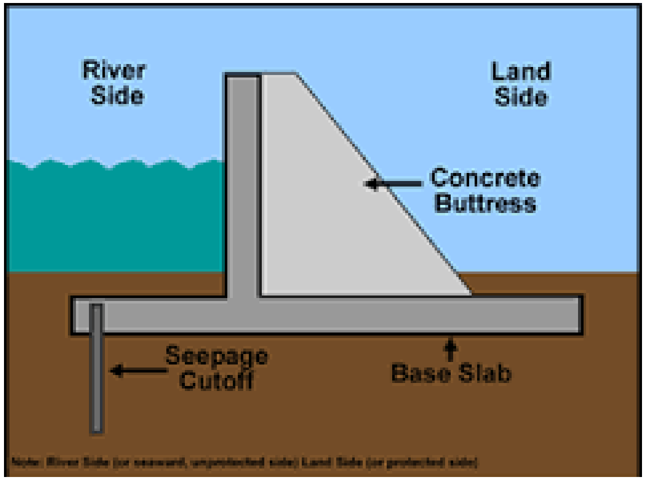
COUNTERFORT RETAINING WALLS

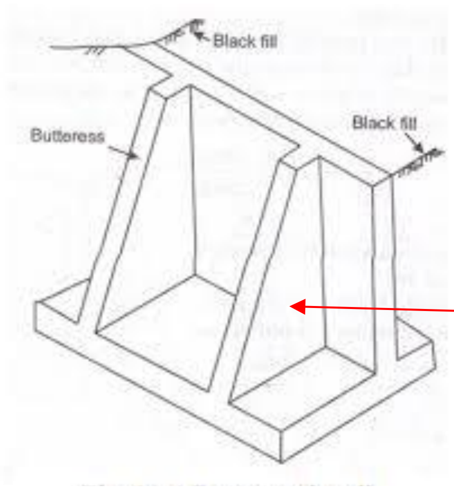
- Counterfort retaining walls are similar to cantilever walls.
- At regular intervals, however, they have thin vertical concrete slabs known as counterforts that tie the wall and the base slab together.
- The purpose of the counterforts is to reduce the shear and the bending.
- The counterfort is behind the wall and subjected to tensile forces.

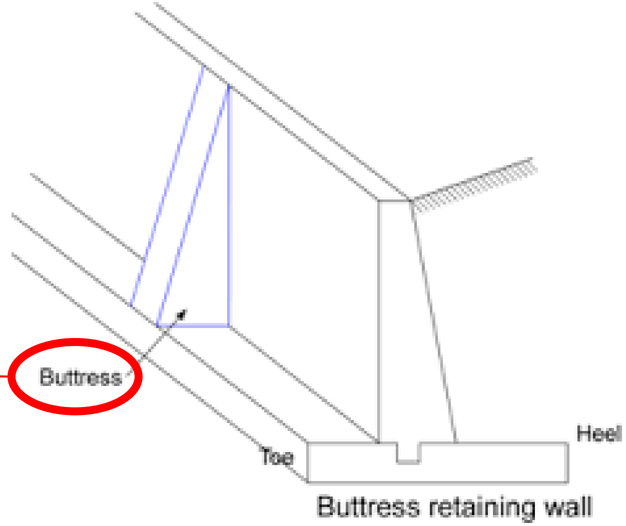


BUTTRESSED RETAINING WALLS

A buttressed retaining wall is similar to a counterfort wall, except that the bracing is in the front of the wall and is in compression instead of tension.



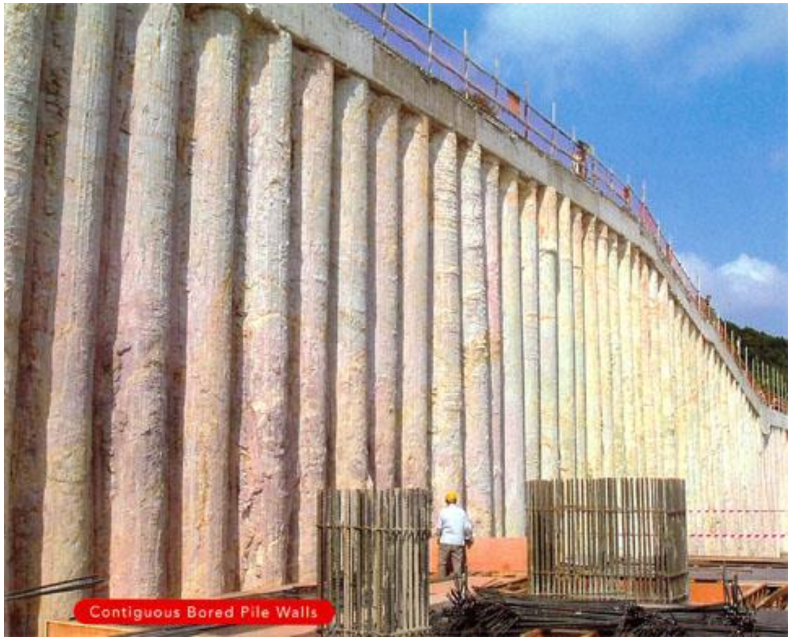




SHEET PILE WALLS

- Steel sheet pile walls are constructed by driving steel sheets into a slope or excavation up to the required depth.
- Their most common use is within temporary deep excavations.
- It cannot resist very high pressure.

PILE WALLS



SOIL NAILING

PHASES OF DESIGN

There are two phases in the design of a conventional retaining wall.

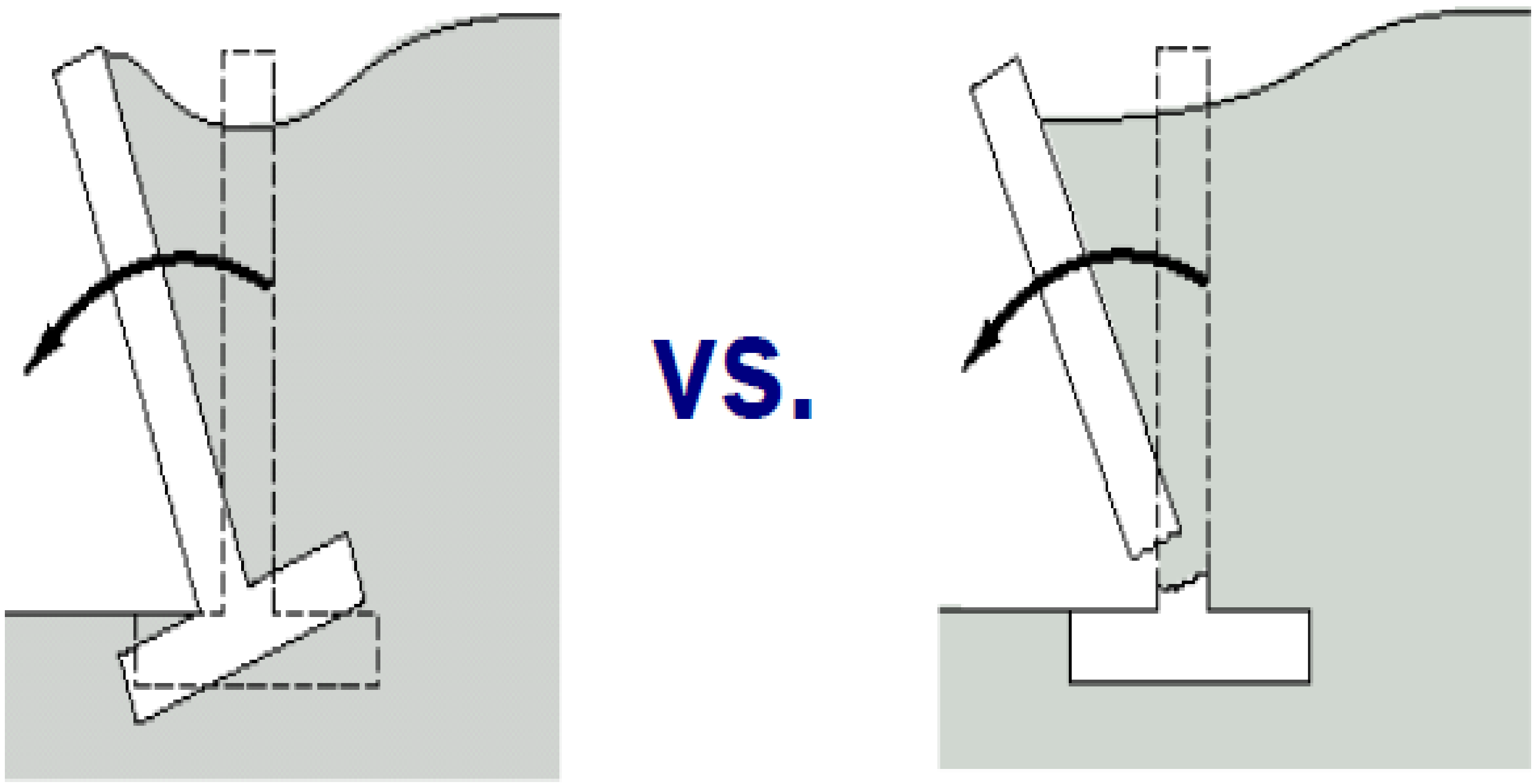
I. With the lateral earth pressure known, the structure as a whole is checked for stability. The structure is examined for possible *overturning*, *sliding*, *bearing capacity,* *and overall (deep-seated)* failures.

II. Each component of the structure is checked for strength, and the steel reinforcement of each component is determined.

- **In this course we will consider the procedures for determining the stability of the retaining wall. Checks for strength can be found in any textbook on reinforced concrete.**

EXTERNAL AND INTERNAL STABILITY

For design of retaining walls we need to consider external and internal stability



EXTERNAL STABILITY

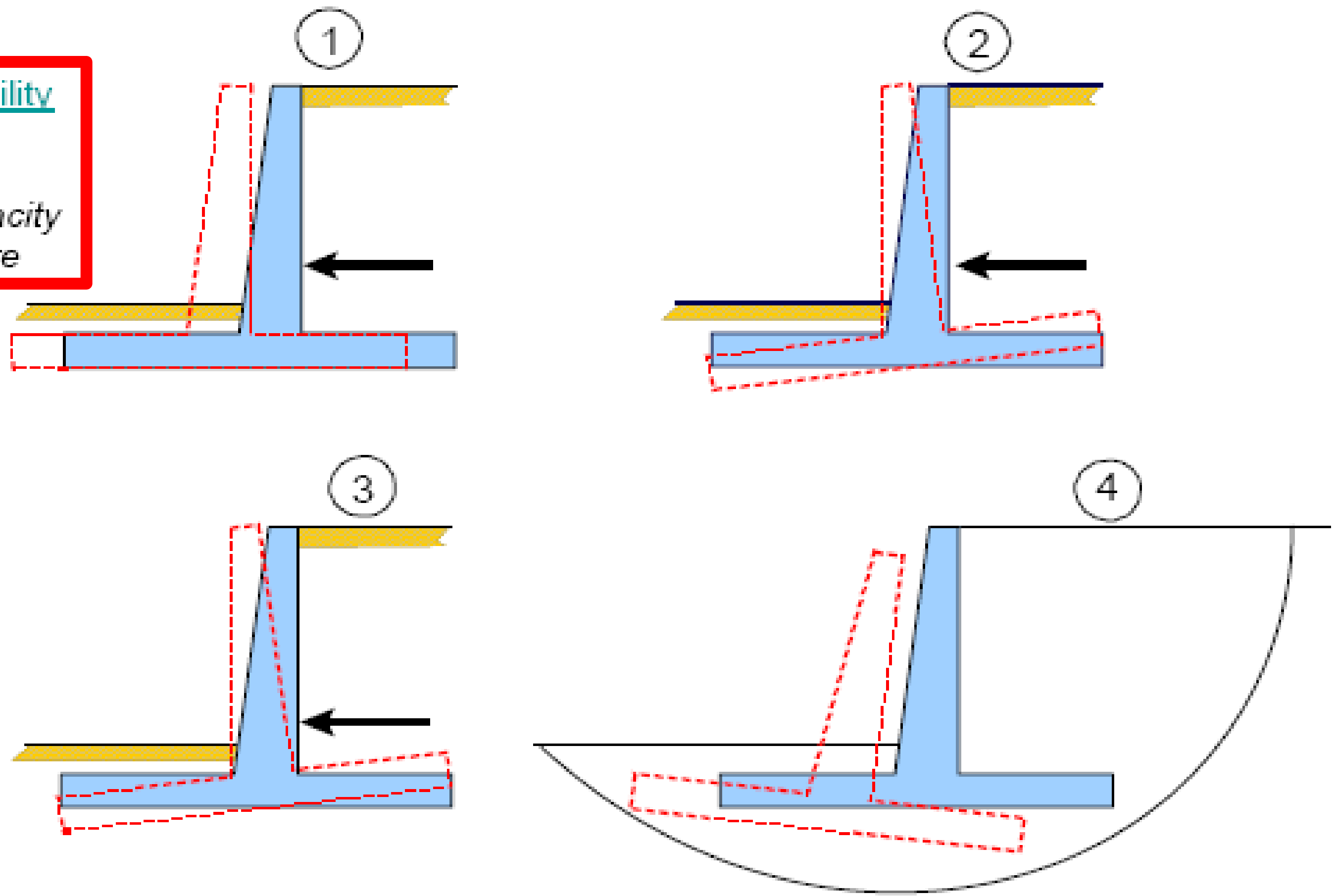
1. External Stability

1- Sliding

2- Overturning

3- Bearing Capacity

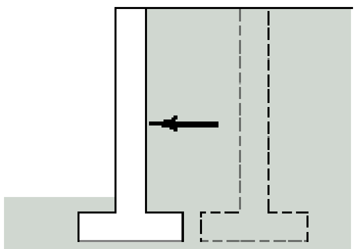
4- Overall Failure



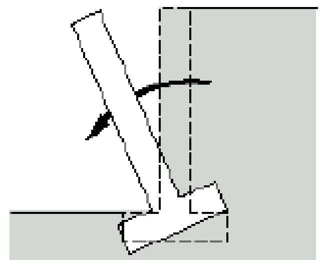
EXTERNAL STABILITY

A retaining wall may fail in any of the following ways:

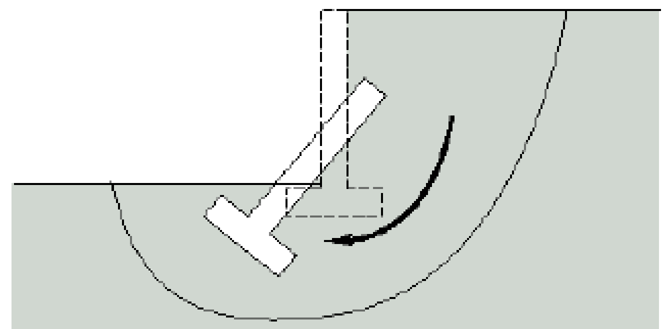
- **It may *overturn* about its toe.**
- **It may *slide* along its base.**
- **It may fail due to the loss of *bearing capacity* of the soil supporting the base.**
- **It may undergo deep-seated shear failure.**
- **It may go through excessive settlement.**



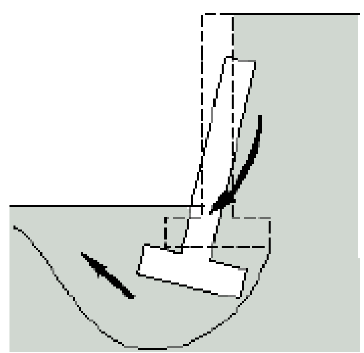
Sliding Failure



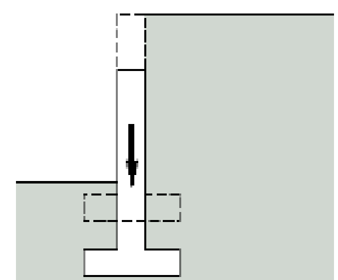
Overturning Failure



Deep Seated Shear Failure



Bearing Capacity



Excessive Settlement

EXTERNAL STABILITY

Failure by sliding

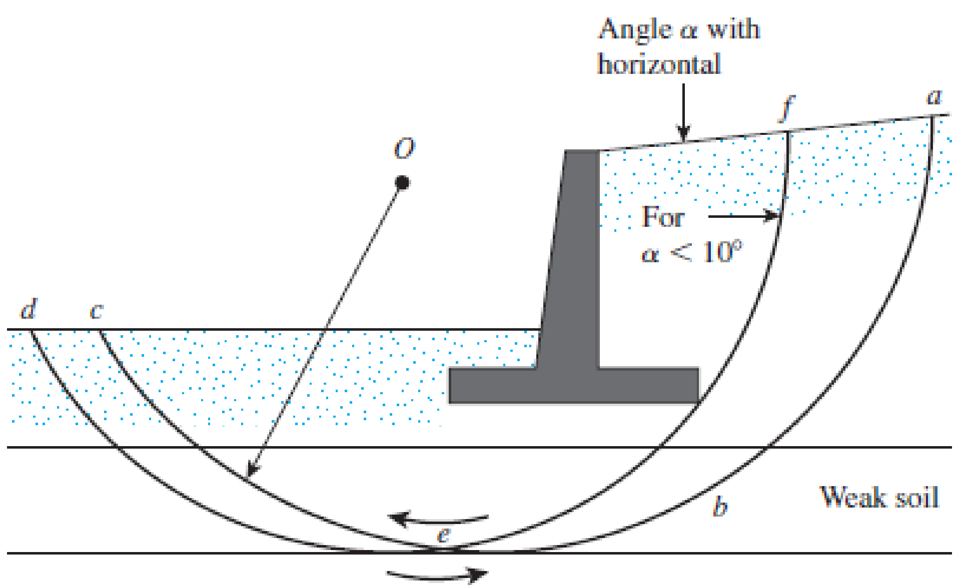
Failure by settlement

DEEP SHEAR FAILURE

- ***Deep shear failure*** **can occur along a cylindrical surface, such as *abc* shown in the figure below, as a result of the existence of a weak layer of soil underneath the wall at a depth of about 1.5 times the width of the base slab of the retaining wall.**

- **In such cases, the critical cylindrical failure surface *abc* has to be determined by trial and error, using various centers such as O.**

- **The failure surface along which the minimum factor of safety is obtained is the critical surface of sliding.**



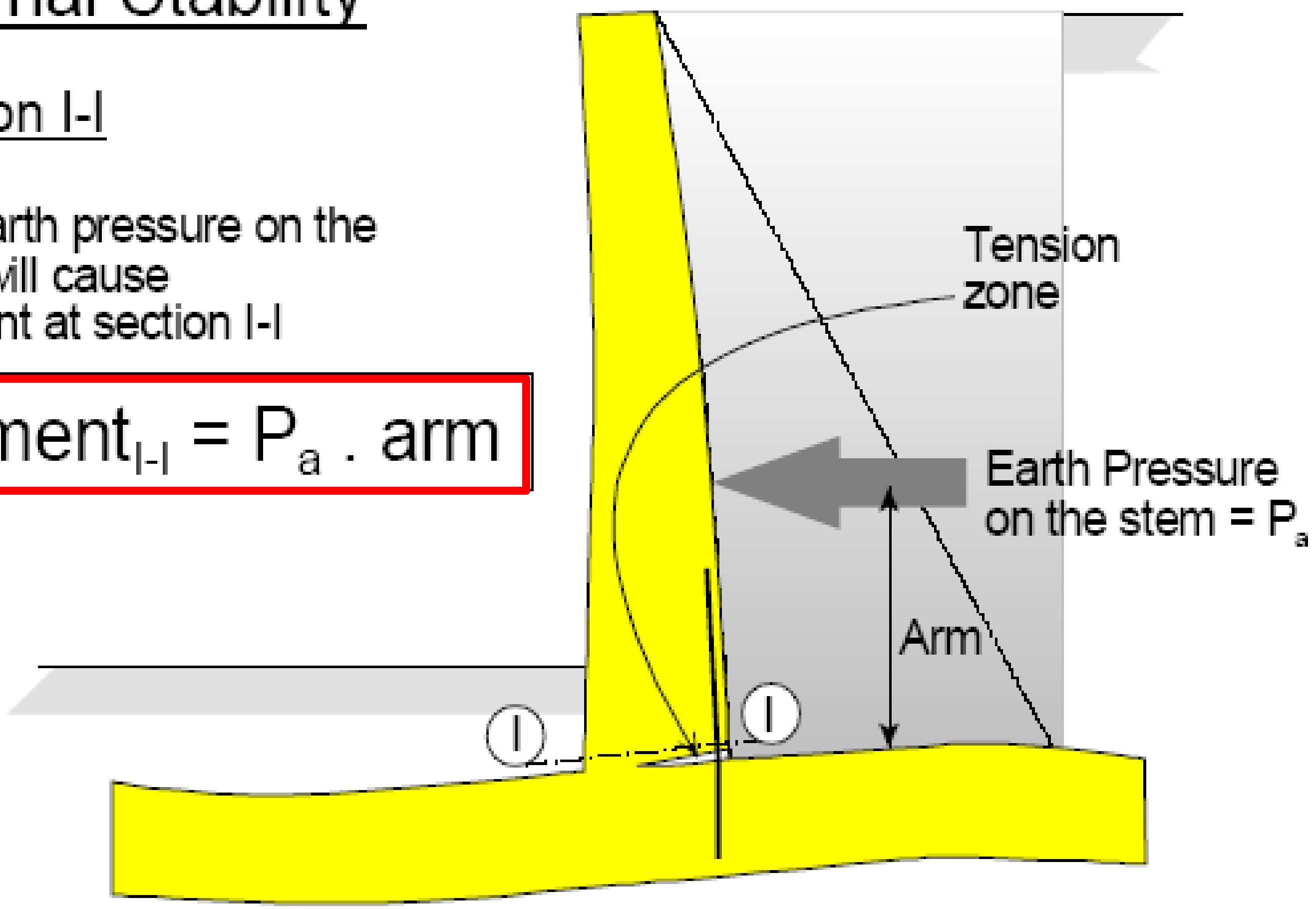
INTERNAL STABILITY

Internal Stability

Section I-I

The earth pressure on the stem will cause moment at section I-I

$$\boxed{\text{Moment}_{I\text{-}I} = P_a \cdot \text{arm}}$$



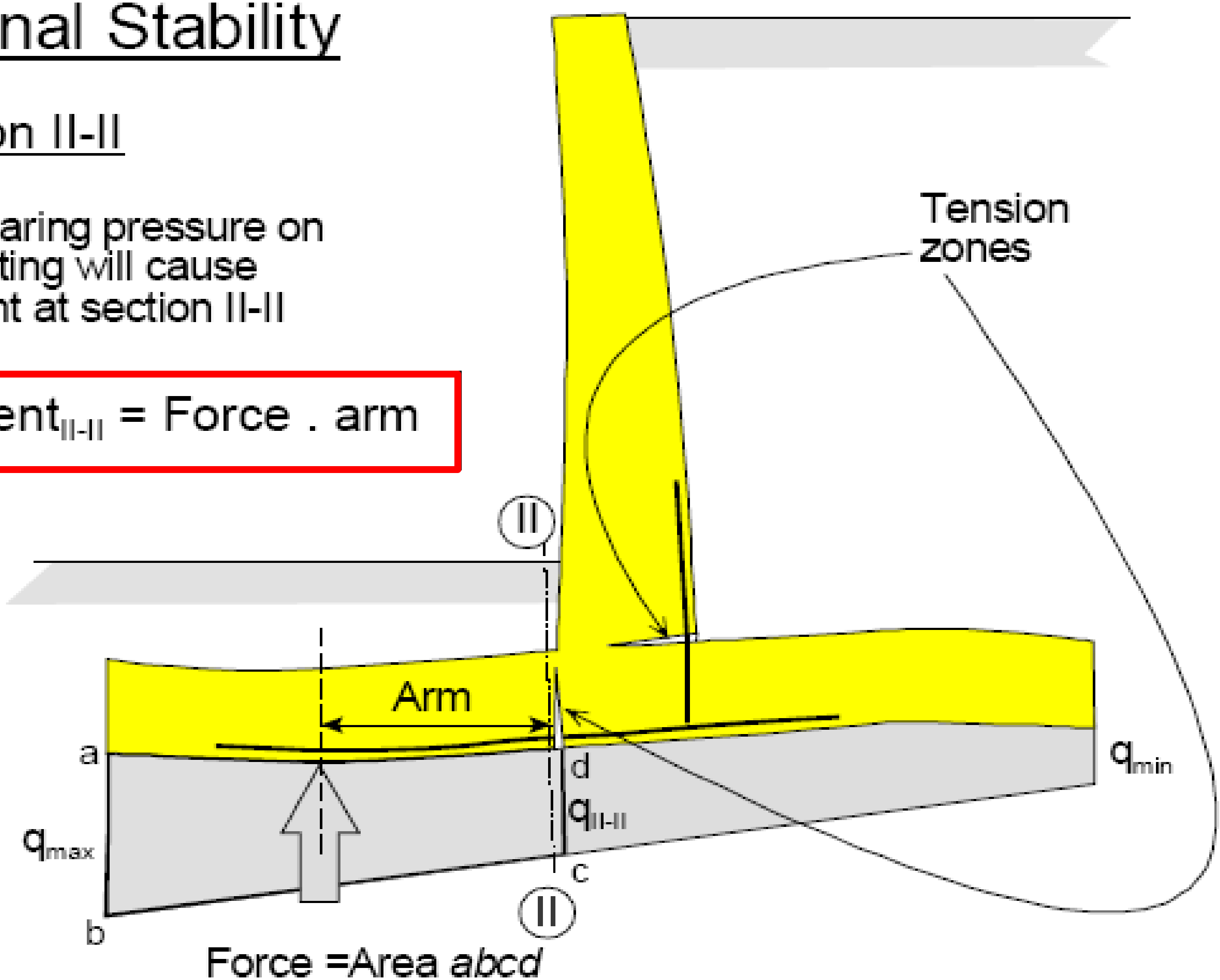
INTERNAL STABILITY

Internal Stability

Section II-II

The bearing pressure on the footing will cause moment at section II-II

$$\text{Moment}_{II\text{-}II} = \text{Force} \cdot \text{arm}$$



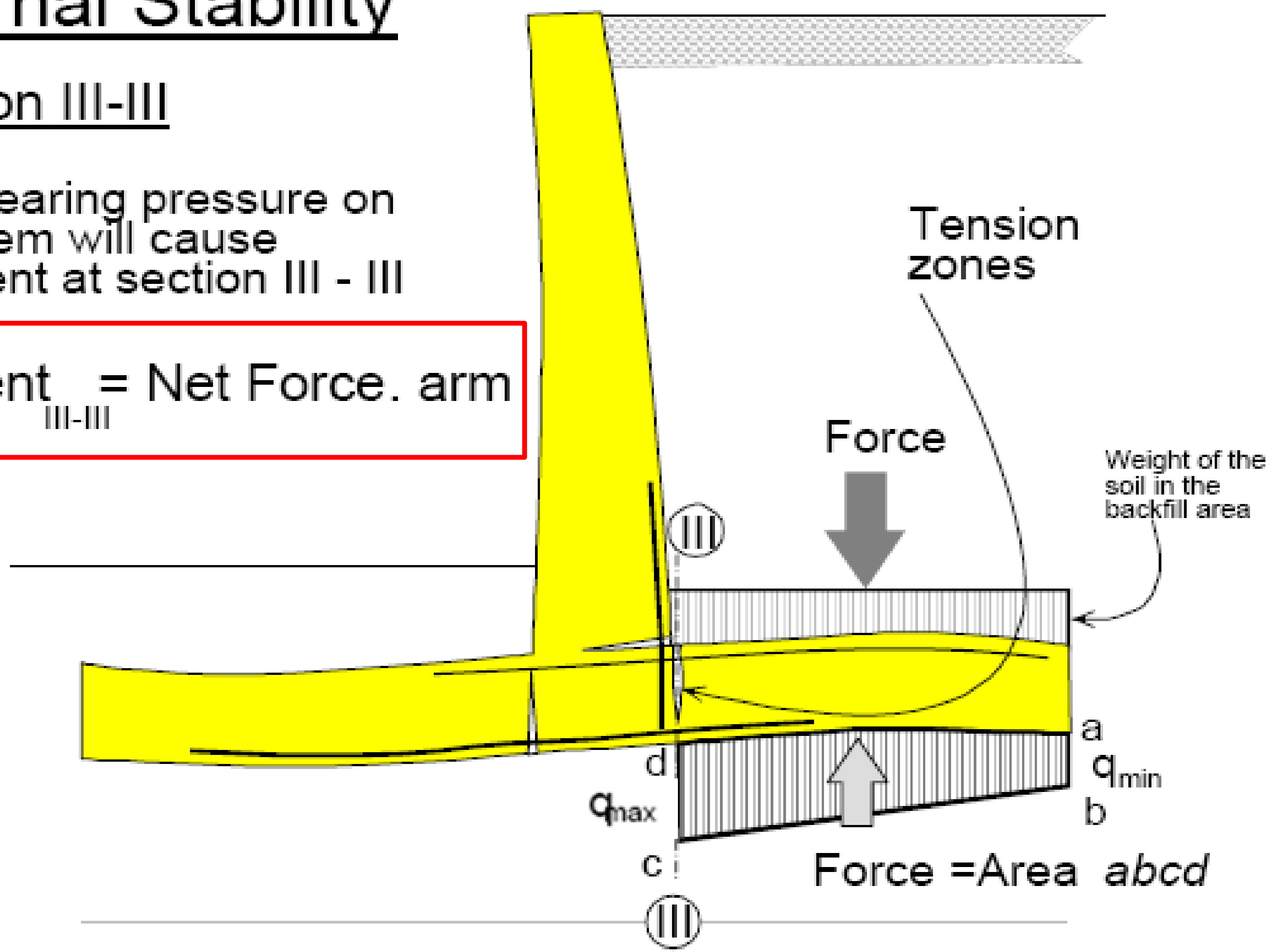
INTERNAL STABILITY

Internal Stability

Section III-III

The bearing pressure on the stem will cause moment at section III - III

$$\text{Moment}_{III\text{-}III} = \text{Net Force. arm}$$



DESIGN OF RETAINING WALLS

The Four Primary Concerns for the Design of Nearly any Retaining Wall are:

1. **It has an acceptable Factor of Safety with respect to overturning.**

2. **It has an acceptable Factor of Safety with respect to sliding.**

3. **The allowable soil bearing pressures are not exceeded.**

4. **The stresses within the components (stem and footing) are within code allowable limits to adequately resist imposed vertical and lateral loads. It is equally important that it is constructed according to the design.**

DESIGN OF RETAINING WALLS

Phase I

1. Select trial dimensions
2. Estimate horizontal soil force on stem
3. Check for sliding and overturning
4. Check sub-soil pressures

This Course

Phase II

5. Estimate critical bending and shear
6. Select concrete grade and rebar cover
7. Select rebar, observing minimum requirements

Reinforced Concrete Course

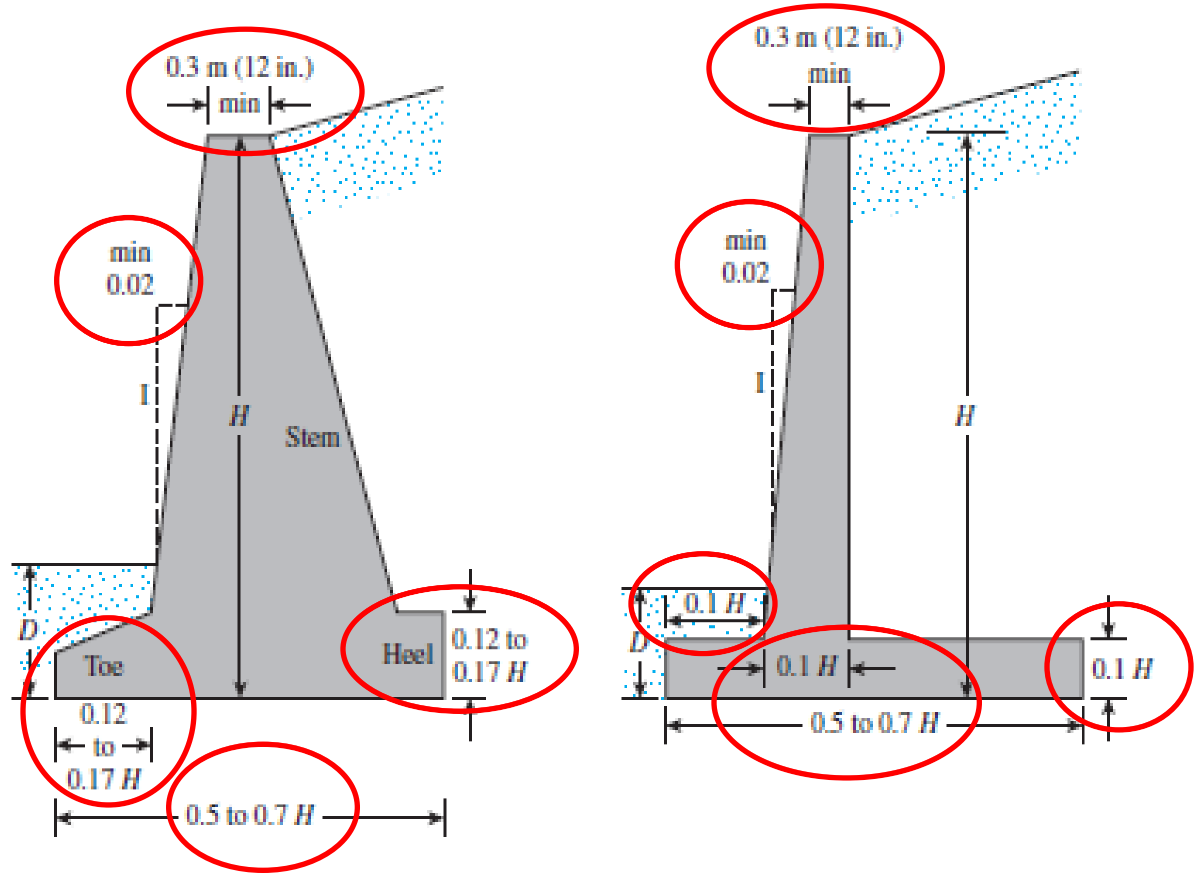
Proportioning Retaining Walls

Gravity and Cantilever Walls

- In designing retaining walls, an engineer must assume some of their dimensions. Called ***proportioning,*** such assumptions allow the engineer to check trial sections of the walls for stability.
- If the stability checks yield undesirable results, the sections can be changed and rechecked.
- Figure shows the general proportions of various retaining-wall components that can be used for initial checks.
- The top of the stem of any retaining wall should not be less than about **0.3 m** for proper placement of concrete. The depth, ***D,*** to the bottom of the base slab should be a minimum of **0.6 m**. However, the bottom of the base slab should be positioned below the seasonal frost line.
- For counterfort retaining walls, the general proportion of the stem and the base slab is the same as for cantilever walls. However, the counterfort slabs may be about **0.3 m** thick and spaced at center-to-center distances of **0.3H to 0.7H**.

Proportioning Retaining Walls

- First, approximate dimensions are chosen for the retaining wall.
- Then, stability of wall is checked for these dimensions.
- Section is changed if its undesirable from the stability or economy point of view.



Gravity wall

Cantilever wall

Design of Retaining Walls

Application of Lateral Earth Pressure Theories to Design

Rankine Theory

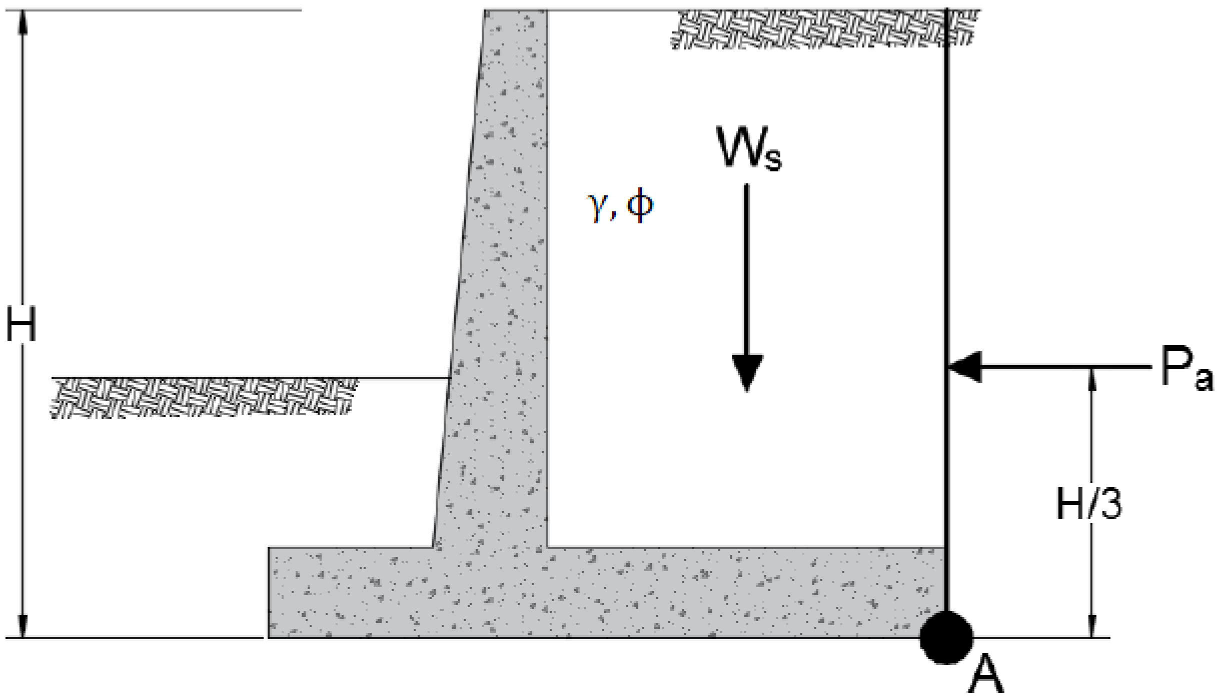
- Rankine theory is modified to be suitable for designing a retaining walls.
- This modification is drawing a vertical line from the lowest-right corner till intersection with the line of backfill, and then considering the force of soil acting on this vertical line.
- The soil between the wall and vertical line is not considered in the value of P_a, so take this soil in consideration as a vertical weight applied on the stem of the retaining wall as will explained later.

Design of Retaining Walls

The wall is vertical and backfill is horizontal

$$P_a = \frac{1}{2}\gamma H^2 K_a$$

$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right)$$



Design of Retaining Walls

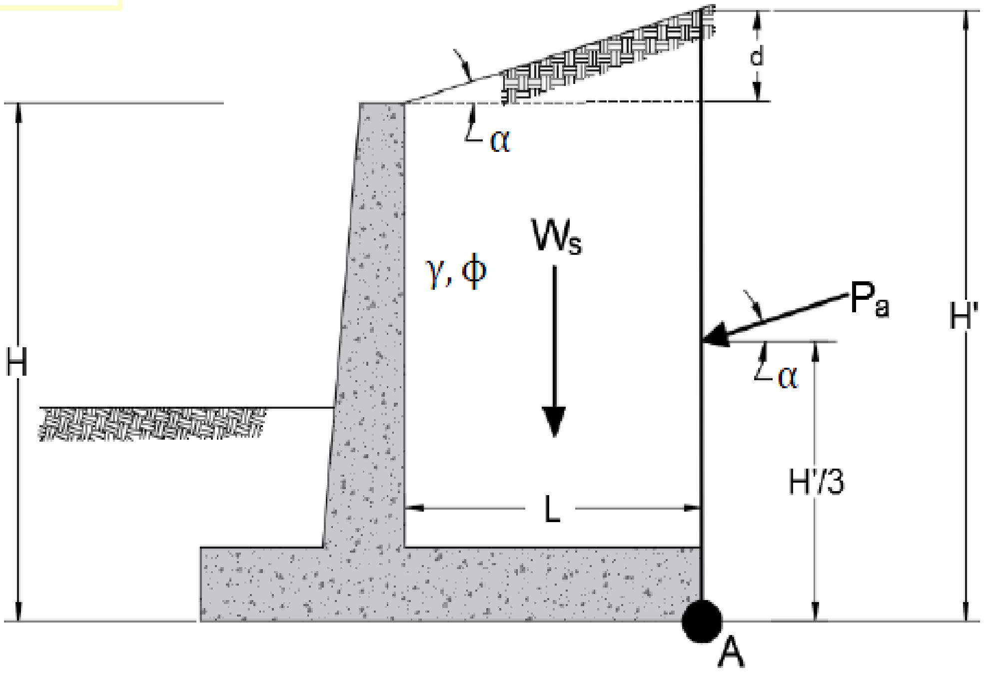
The wall is vertical and the backfill is inclined with horizontal by angle (α)

$P_a = 1/2 \gamma H'^2 K_a$

$H' = H + d \rightarrow \rightarrow d = L \tan\alpha$

$P_{a,h} = P_a \cos(\alpha)$

$P_{a,v} = P_a \sin(\alpha)$



Design of Retaining Walls

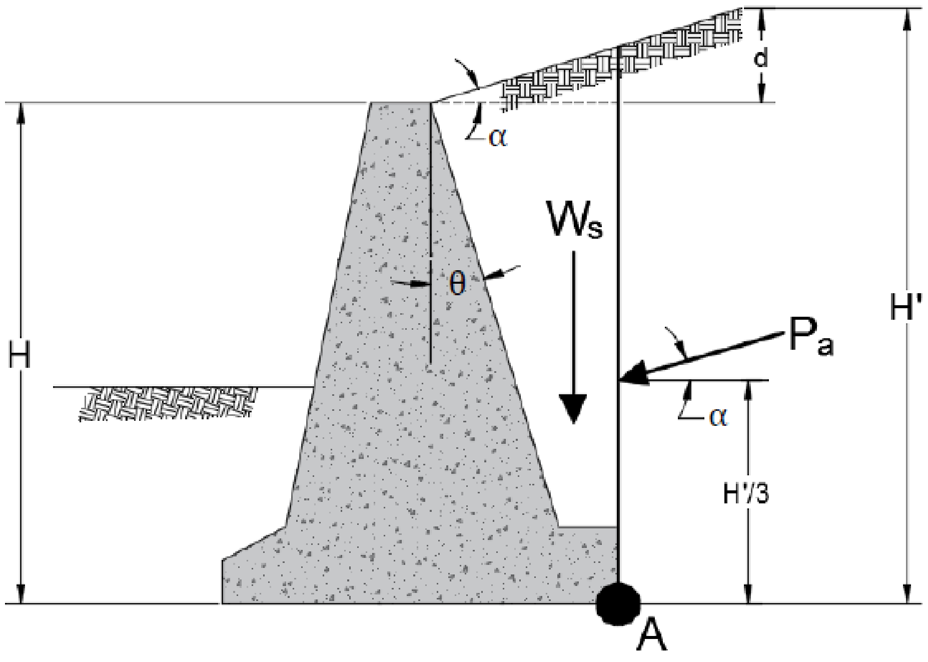
The wall is inclined by angle (θ)with vertical and the backfill is inclined with horizontal by angle (α):

$$P_a = 1/2\gamma H'^2 K_a$$

$$K_a = \frac{\cos(\alpha-\theta)\sqrt{1+\sin^2\phi - 2\sin\phi\cos\psi_a}}{\cos^2\theta\left(\cos\alpha + \sqrt{\sin^2\phi - \sin^2\alpha}\right)}$$

$$\psi_a = \sin^{-1}\left(\frac{\sin\alpha}{\sin\phi}\right) - \alpha + 2\theta$$

$$P_{a,h} = P_a\cos(\alpha) \quad , \quad P_{a,v} = P_a\sin(\alpha)$$



Design of Retaining Walls

Application of Lateral Earth Pressure Theories to Design

Coulomb's Theory

- Coulomb's theory will remains unchanged (without any modifications).
- The force P_a is applied directly on the wall, so whole soil retained by the wall will be considered in P_a
- the weight of soil will not apply on the heel of the wall.

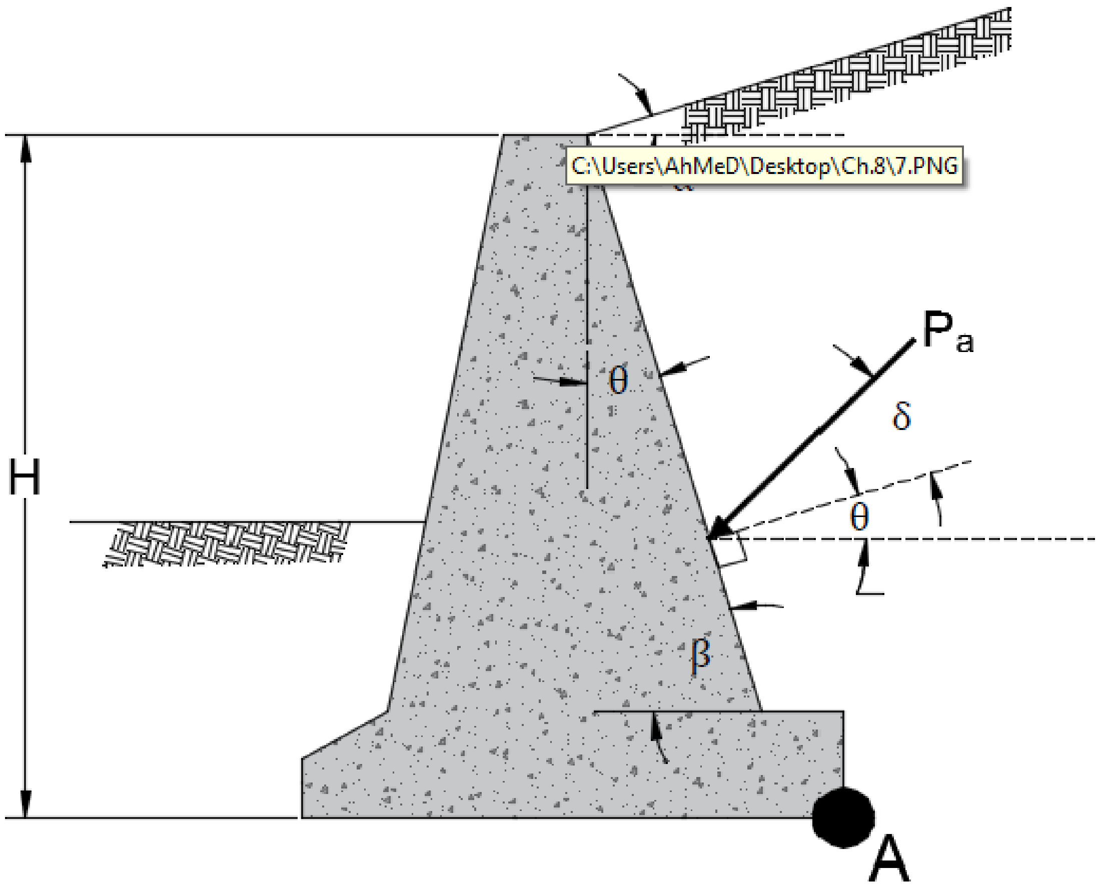
Design of Retaining Walls

$P_a = 1/2 \gamma H^2 K_a$

$P_{a,h} = P_a \cos(\delta + \theta)$

$P_{a,v} = P_a \sin(\delta + \theta)$

Backfill material	Range of δ' (deg)
Gravel	27–30
Coarse sand	20–28
Fine sand	15–25
Stiff clay	15–20
Silty clay	12–16



STABILITY FOR OVERTURNING

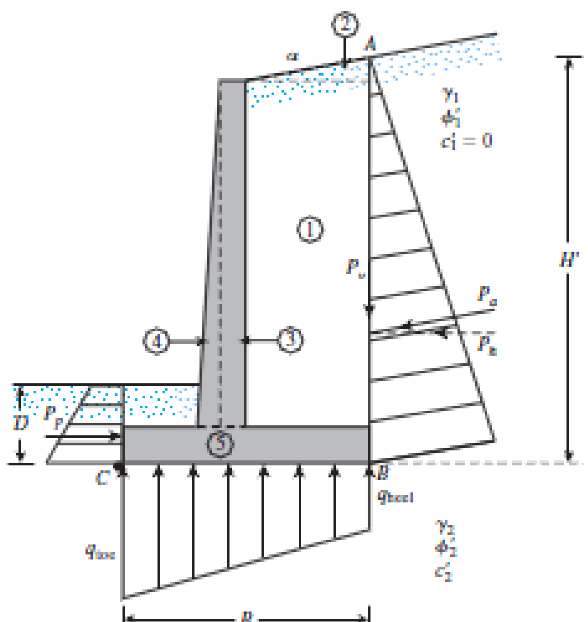
$$\text{FS}_{(\text{overturning})} = \frac{\Sigma M_R}{\Sigma M_o}$$

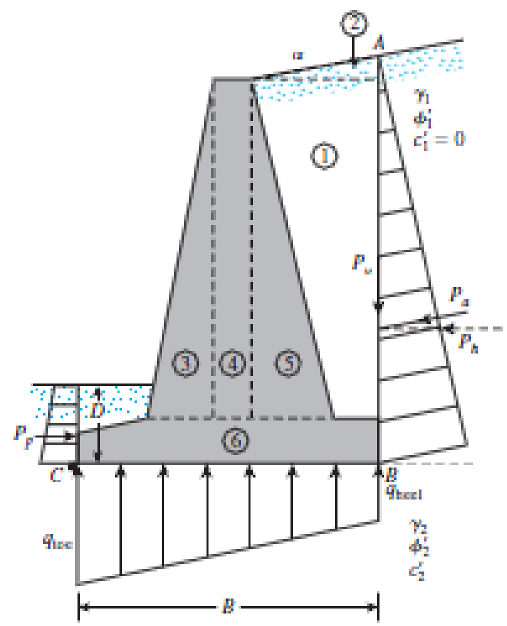
where

ΣM_o = sum of the moments of forces tending to overturn about point C
ΣM_R = sum of the moments of forces tending to resist overturning about point C

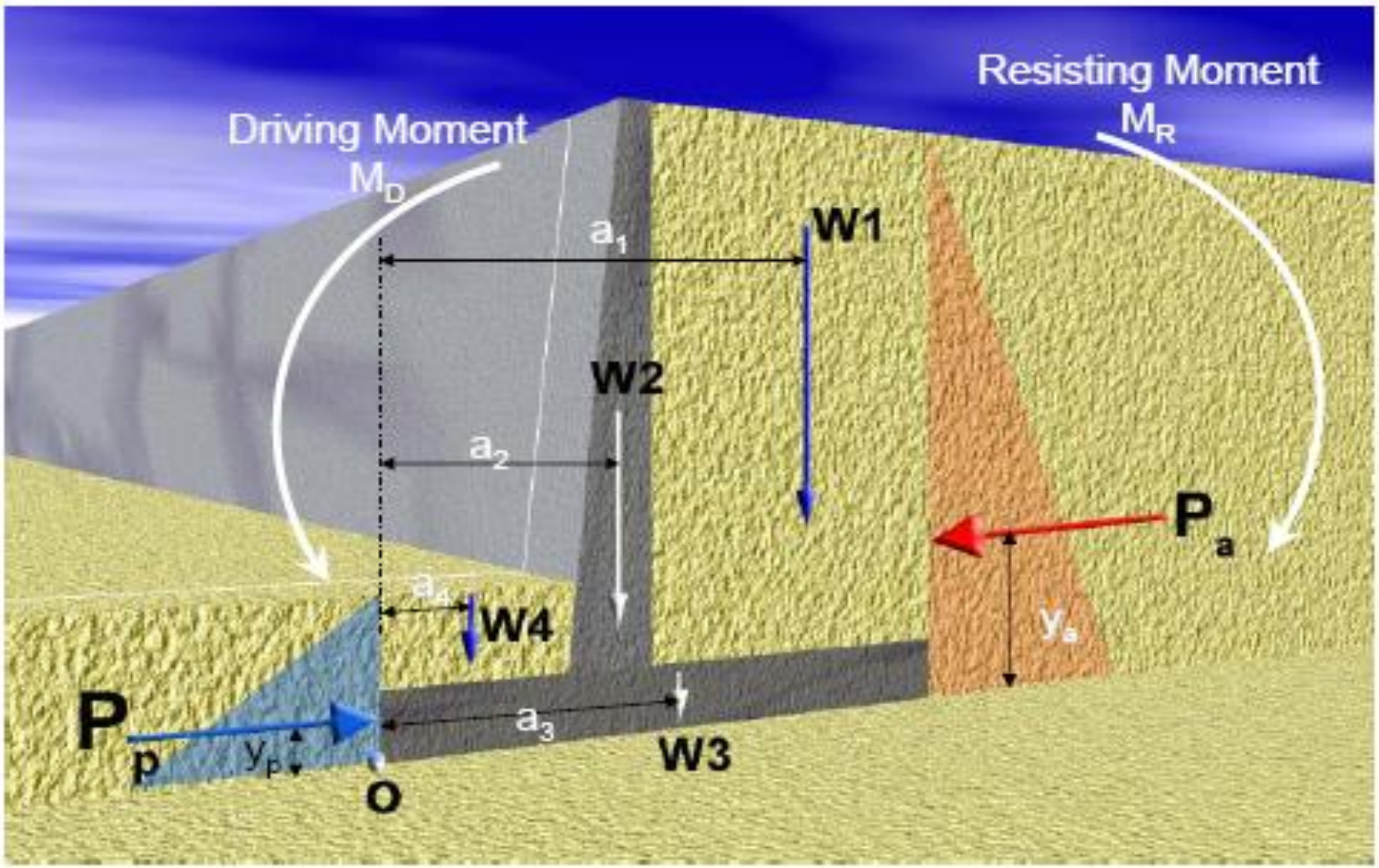
$$\Sigma M_o = P_h\left(\frac{H'}{3}\right)$$

where $P_h = P_a \cos\alpha$
$P_v = P_a \sin\alpha$





STABILITY FOR OVERTURNING



Moment About o

$M_D = P_a \cdot y_a$

$M_R = P_p \cdot y_p + W_1 a_1 + W_2 a_2 + W_3 a_3 + W_4 a_4$

STABILITY FOR OVERTURNING

$$FS_{(overturning)} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_v}{P_a \cos\alpha (H'/3)}$$

Table 13.1 Procedure for Calculating ΣM_R

Section (1)	Area (2)	Weight/unit length of wall (3)	Moment arm measured from *C* (4)	Moment about *C* (5)
1	A_1	$W_1 = \gamma_1 \times A_1$	X_1	M_1
2	A_2	$W_2 = \gamma_1 \times A_2$	X_2	M_2
3	A_3	$W_3 = \gamma_c \times A_3$	X_3	M_3
4	A_4	$W_4 = \gamma_c \times A_4$	X_4	M_4
5	A_5	$W_5 = \gamma_c \times A_5$	X_5	M_5
6	A_6	$W_6 = \gamma_c \times A_6$	X_6	M_6
		P_v	B	M_v
		ΣV		ΣM_R

(*Note:* γ_1 = unit weight of backfill
γ_c = unit weight of concrete
X_i = horizontal distance between C and the centroid of the section)

Remark

Some designers prefer to determine the factor of safety against overturning with the formula

$$FS_{(overturning)} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6}{P_a \cos\alpha (H'/3) - M_v}$$

EXAMPLE 13.1

Example 13.1

The cross section of a cantilever retaining wall is shown in Figure 13.12. Calculate the factors of safety with respect to overturning.

OVERTURNING

Solution

From the figure,

$$H' = H_1 + H_2 + H_3 = 2.6 \tan 10^\circ + 6 + 0.7$$
$$= 0.458 + 6 + 0.7 = 7.158 \text{ m}$$

The Rankine active force per unit length of wall $= P_p = \frac{1}{2}\gamma_1 H'^2 K_a$. For $\phi'_1 = 30^\circ$ and $\alpha = 10^\circ$, K_a is equal to 0.3495. (See Table 12.1.) Thus,

$$P_a = \tfrac{1}{2}(18)(7.158)^2(0.3495) = 161.2 \text{ kN/m}$$

$$P_v = P_a \sin 10^\circ = 161.2\,(\sin 10^\circ) = 28.0 \text{ kN/m}$$

and

$$P_h = P_a \cos 10^\circ = 161.2\,(\cos 10^\circ) = 158.75 \text{ kN/m}$$

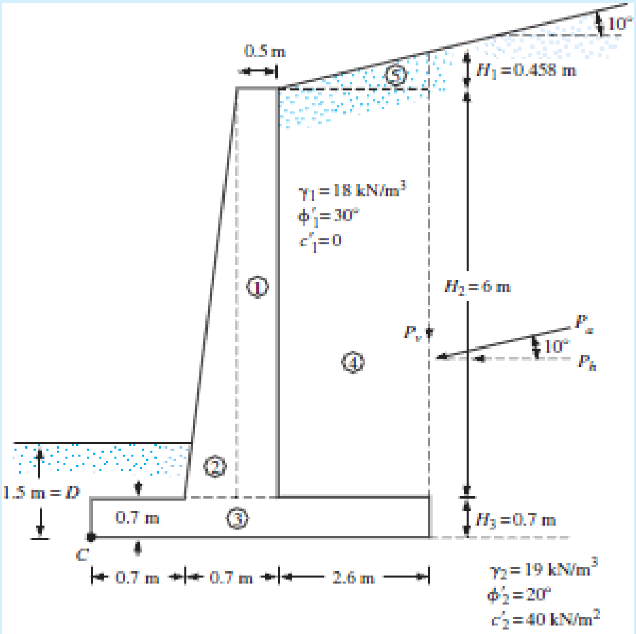


Figure 13.12 Calculation of stability of a retaining wall

Section no.[a]	Area (m²)	Weight/unit length (kN/m)	Moment arm from point C (m)	Moment (kN-m/m)
1	$6 \times 0.5 = 3$	70.74	1.15	81.35
2	$\frac{1}{2}(0.2)6 = 0.6$	14.15	0.833	11.79
3	$4 \times 0.7 = 2.8$	66.02	2.0	132.04
4	$6 \times 2.6 = 15.6$	280.80	2.7	758.16
5	$\frac{1}{2}(2.6)(0.458) = 0.595$	10.71	3.13	33.52
		$P_v = 28.0$	4.0	112.0
		$\Sigma V = 470.42$		$1128.86 = \Sigma M_R$

[a]For section numbers, refer to Figure 13.12

$\gamma_{concrete} = 23.58$ kN/m³

$$M_o = P_h\left(\frac{H'}{3}\right) = 158.75\left(\frac{7.158}{3}\right) = 378.78 \text{ kN-m/m}$$

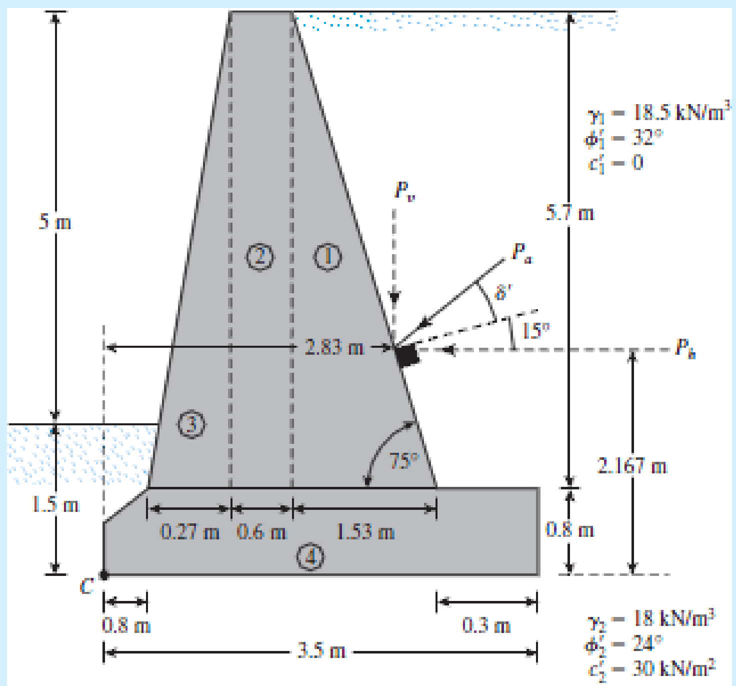
$$FS_{(overturning)} = \frac{\Sigma M_R}{M_o} = \frac{1128.86}{378.78} = \mathbf{2.98 > 2,\ OK}$$

EXAMPLE 13.2

Example 13.2

A gravity retaining wall is shown in Figure 13.13. Use $\delta' = 2/3\phi_1'$ and Coulomb's active earth pressure theory. Determine

OVERTURNING



Solution

The height

$$H' = 5 + 1.5 = 6.5 \text{ m}$$

Coulomb's active force is

$$P_a = \frac{1}{2}\gamma_1 H'^2 K_a$$

With $\alpha = 0°$, $\beta = 75°$, $\delta' = 2/3\phi_1'$, and $\phi_1' = 32°$, $K_a = 0.4023$. (See Table 12.6.) So,

$$P_a = \tfrac{1}{2}(18.5)(6.5)^2(0.4023) = 157.22 \text{ kN/m}$$

$$P_h = P_a \cos(15 + \tfrac{2}{3}\phi_1') = 157.22 \cos 36.33 = 126.65 \text{ kN/m}$$

and

$$P_v = P_a \sin(15 + \tfrac{2}{3}\phi_1') = 157.22 \sin 36.33 = 93.14 \text{ kN/m}$$

Area no.	Area (m²)	Weight* (kN/m)	Moment arm from C (m)	Moment (kN-m/m)
1	$\frac{1}{2}(5.7)(1.53) = 4.36$	102.81	2.18	224.13
2	$(0.6)(5.7) = 3.42$	80.64	1.37	110.48
3	$\frac{1}{2}(0.27)(5.7) = 0.77$	18.16	0.98	17.80
4	$\approx (3.5)(0.8) = 2.8$	66.02	1.75	115.54
		$P_v = 93.14$	2.83	263.59
		$\Sigma V = 360.77$ kN/m		$\Sigma M_R = 731.54$ kN-m/m

*$\gamma_{concrete} = 23.58$ kN/m³

Note that the weight of the soil above the back face of the wall is not taken into account in the preceding table. We have

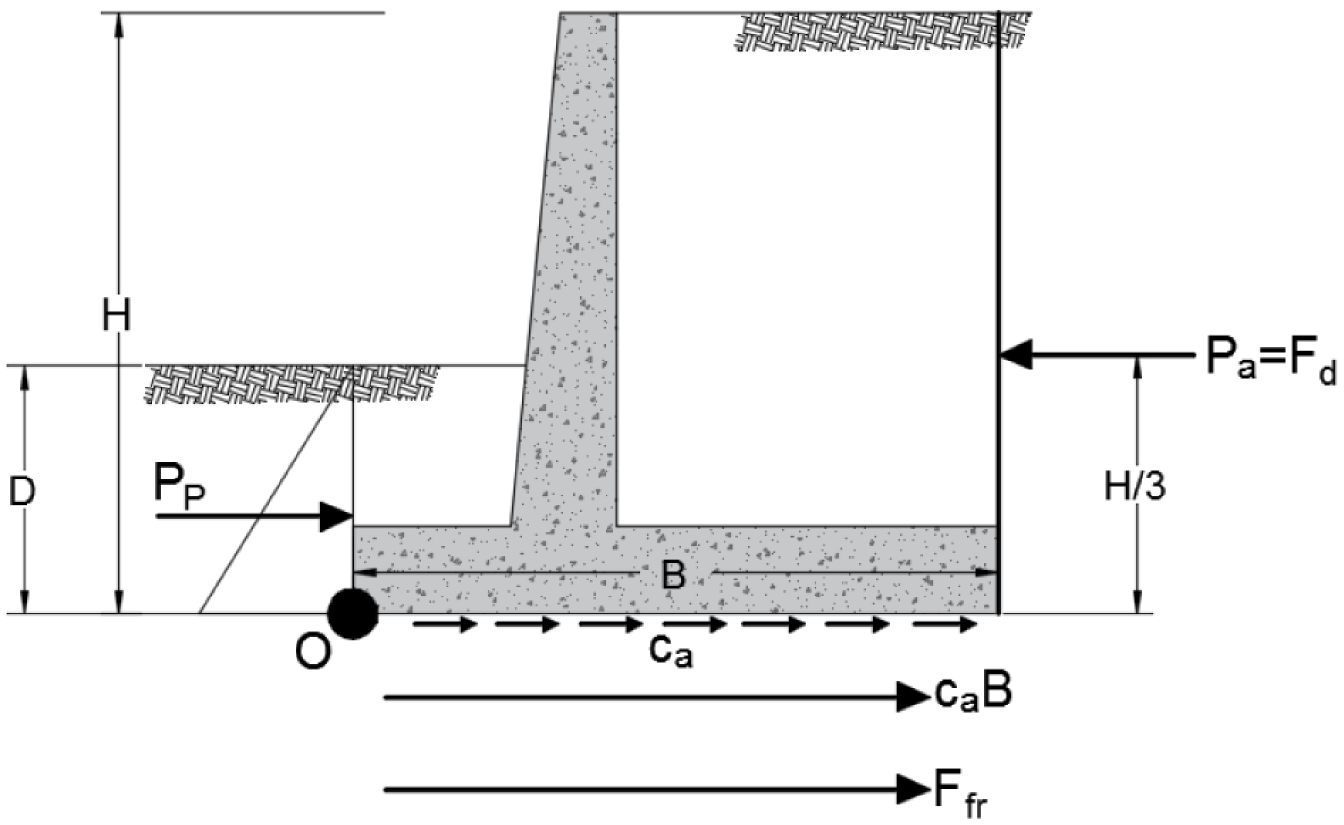
$$\text{Overturning moment} = M_o = P_h\left(\frac{H'}{3}\right) = 126.65(2.167) = 274.45 \text{ kN-m/m}$$

$$FS_{(overturning)} = \frac{\Sigma M_R}{\Sigma M_o} = \frac{731.54}{274.45} = \mathbf{2.67 > 2, OK}$$

STABILITY FOR SLIDING ALONG THE BASE

The horizontal component of active force may causes movement of the wall in horizontal direction (i.e. causes sliding for the wall), this force is called driving force

$$F_d = P_{a,h}$$



STABILITY FOR SLIDING ALONG THE BASE

The driving force will be resisted by the following forces:

1. Adhesion between the soil (under the base) and the base of retaining wall:

c_a = adhesion along the base of retaining wall (KN/m)

$C_a = c_a \times B$ = adhesion force under the base of retaining wall (KN)

c_a can be calculated from the following relation:

$$c_a = K_2 c_2$$

where c_2=cohesion of soil under the base

So adhesion force is:

$$C_a = K_2 c_2 B$$

STABILITY FOR SLIDING ALONG THE BASE

2. Friction force due to the friction between the soil and the base of retaining wall :

Friction force is calculated from the following relation:

$$F_{fr} = \mu_s N$$

where N is the sum of vertical forces calculated in the table of the first check (overturning) $\rightarrow N = \Sigma V$ (including the vertical component of active force)

μ_s = coefficient of friction (related to the friction between soil and base)

$\mu_s = \tan(\delta_2)$

$\delta_2 = K_1 \phi_2$

$\mu_s = \tan(K_1 \phi_2)$

ϕ_2 = friction angle of the soil under the base.

$F_{fr} = \Sigma V \times \tan(K_1 \phi_2)$

Note: $K_1 = K_2 = (1/2 \rightarrow 2/3)$ if you are not given them $\rightarrow$take $K_1 = K_2 = 2/3$

STABILITY FOR SLIDING ALONG THE BASE

3. Passive force P_P

The total resisting force FR can be calculated as following:

$F_R = \Sigma V \times \tan(K_1 \phi_2) + K_2 c_2 B + P_P$

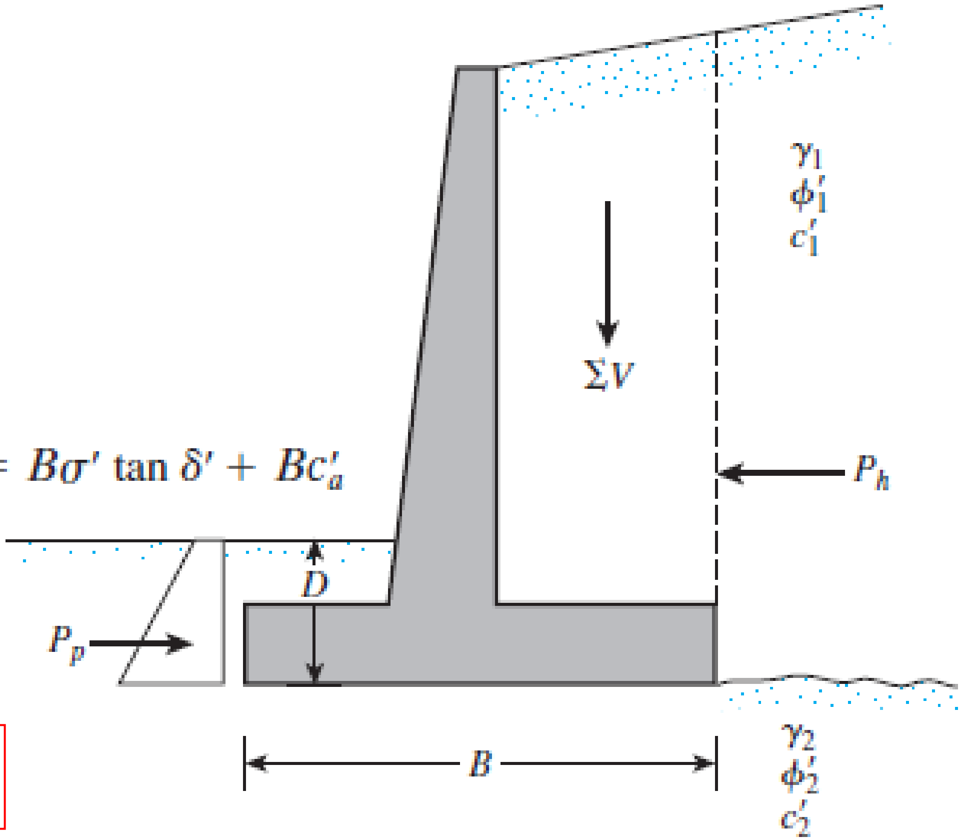
Factor of safety against sliding

$$FS_S = \frac{F_R}{F_d} \geq 2 \quad \text{(if we consider } P_P \text{ in } F_R\text{)}$$

$$FS_S = \frac{F_R}{F_d} \geq 1.5 \quad \text{(if we dont consider } P_P \text{ in } F_R\text{)}$$

STABILITY FOR SLIDING ALONG THE BASE

$$\boxed{FS_{(sliding)} = \frac{\sum F_{R'}}{\sum F_d}}$$



Shear strength along the base

$$s = \sigma' \tan \delta' + c'_a$$

$$R' = s(\text{area of cross section}) = s(B \times 1) = B\sigma' \tan \delta' + Bc'_a$$

$$R' = (\Sigma V) \tan \delta' + Bc'_a$$

$$\boxed{\Sigma F_{R'} = (\Sigma V) \tan \delta' + Bc'_a + P_p}$$

$$\boxed{\Sigma F_d = P_a \cos \alpha}$$

$$\boxed{FS_{(sliding)} = \frac{(\Sigma V) \tan \delta' + Bc'_a + P_p}{P_a \cos \alpha}}$$

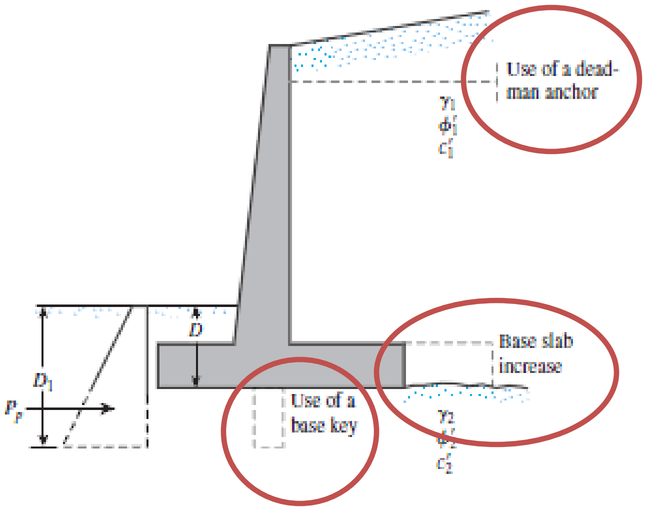
STABILITY FOR SLIDING ALONG THE BASE

If the desired value of $FS_{sliding}$ is not achieved, several alternatives may be investigated:

1. Increase the width of the base slab (i.e., the heel of the footing).
2. Use a key to the base slab. If a key is included, the passive force per unit length of the wall becomes

$$P_p = \frac{1}{2}\gamma_2 D_1^2 K_p + 2c_2' D_1 \sqrt{K_p}$$

where $K_p = \tan^2\left(45 + \frac{\phi_2'}{2}\right)$.



3. Use a deadman anchor at the stem of the retaining wall.
4. Another possible way to increase the value of F_S(sliding) is to consider reducing the value of P_a.

STABILITY FOR SLIDING ALONG THE BASE

4. Another possible way to increase the value of F_S(sliding) is to consider reducing the value of *Pa*. One possible way to do so is to use the method developed by Elman and Terry (1988) for the case in which the retaining wall has a horizontal granular backfill.

The active force, P_a, is horizontal ($\alpha = 0$) so that

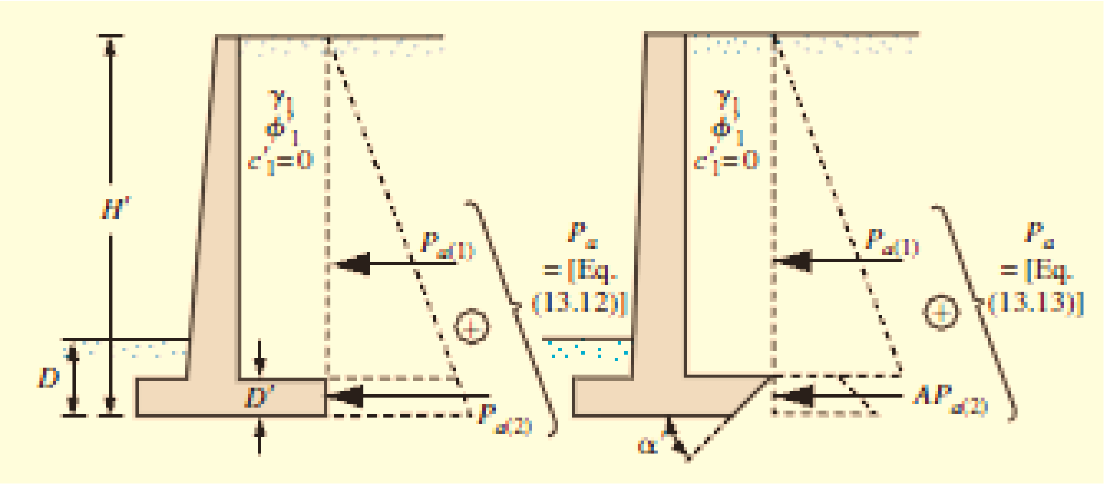
$P_a \cos \alpha = P_h = P_a$

and

$P_a \sin \alpha = P_v = 0$

However,

$P_a = P_{a(1)} + P_{a(2)}$



The magnitude of $P_{a(2)}$ can be reduced if the heel of the retaining wall is sloped. For this case,

$P_a = P_{a(1)} + AP_{a(2)}$

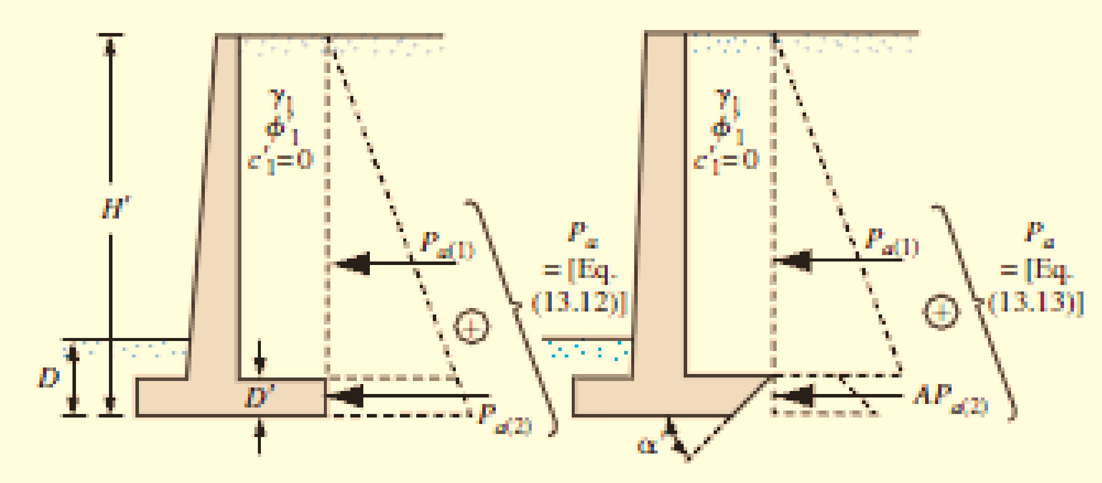
STABILITY FOR SLIDING ALONG THE BASE

$$P_a = P_{a(1)} + AP_{a(2)}$$

$$P_{a(1)} = \frac{1}{2}\gamma_1 K_a (H' - D')^2$$

$$P_a = \frac{1}{2}\gamma_1 K_a H'^2$$

$$P_{a(2)} = \frac{1}{2}\gamma_1 K_a [H'^2 - (H' - D')^2]$$



$$P_a = \frac{1}{2}\gamma_1 K_a (H' - D')^2 + \frac{A}{2}\gamma_1 K_a [H'^2 - (H' - D')^2]$$

Table 13.2 Variation of A with ϕ'_1 (for $\alpha' = 45°$)

Soil friction angle, ϕ'_1 (deg)	A
20	0.28
25	0.14
30	0.06
35	0.03
40	0.018

EXAMPLE 13.1

Example 13.1

The cross section of a cantilever retaining wall is shown in Figure 13.12. Calculate the factors of safety with respect to

SLIDING

Solution

From the figure,

$$H' = H_1 + H_2 + H_3 = 2.6 \tan 10^\circ + 6 + 0.7$$
$$= 0.458 + 6 + 0.7 = 7.158 \text{ m}$$

The Rankine active force per unit length of wall $= P_p = \frac{1}{2}\gamma_1 H'^2 K_a$. For $\phi'_1 = 30^\circ$ and $\alpha = 10^\circ$, K_a is equal to 0.3495. (See Table 12.1.) Thus,

$$P_a = \tfrac{1}{2}(18)(7.158)^2(0.3495) = 161.2 \text{ kN/m}$$

$$P_v = P_a \sin 10^\circ = 161.2 (\sin 10^\circ) = 28.0 \text{ kN/m}$$

and

$$P_h = P_a \cos 10^\circ = 161.2 (\cos 10^\circ) = 158.75 \text{ kN/m}$$

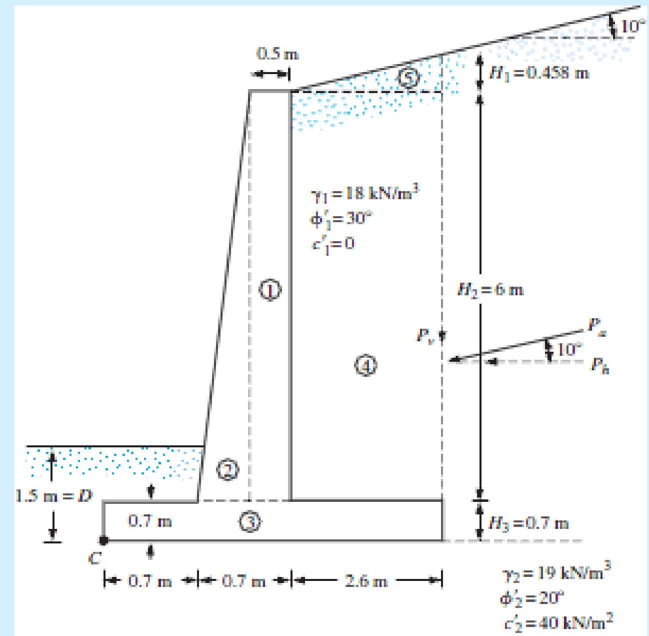


Figure 13.12 Calculation of stability of a retaining wall

Section no.[a]	Area (m²)	Weight/unit length (kN/m)	Moment arm from point *C* (m)	Moment (kN-m/m)
1	$6 \times 0.5 = 3$	70.74	1.15	81.35
2	$\frac{1}{2}(0.2)6 = 0.6$	14.15	0.833	11.79
3	$4 \times 0.7 = 2.8$	66.02	2.0	132.04
4	$6 \times 2.6 = 15.6$	280.80	2.7	758.16
5	$\frac{1}{2}(2.6)(0.458) = 0.595$	10.71	3.13	33.52
		$P_v = 28.0$	4.0	112.0
		$\Sigma V = 470.42$		$1128.86 = \Sigma M_R$

[a]For section numbers, refer to Figure 13.12

$\gamma_{concrete} = 23.58$ kN/m³

EXAMPLE 13.1

$$\text{FS}_{\text{(sliding)}} = \frac{(\Sigma V)\tan(k_1\phi_2') + Bk_2c_2' + P_p}{P_a\cos\alpha}$$

Let $k_1 = k_2 = \frac{2}{3}$. Also,

$$P_p = \frac{1}{2}K_p\gamma_2D^2 + 2c_2'\sqrt{K_p}D$$

$$K_p = \tan^2\left(45 + \frac{\phi_2'}{2}\right) = \tan^2(45 + 10) = 2.04$$

and

$$D = 1.5 \text{ m}$$

So

$$P_p = \frac{1}{2}(2.04)(19)(1.5)^2 + 2(40)(\sqrt{2.04})(1.5)$$
$$= 43.61 + 171.39 = 215 \text{ kN/m}$$

Hence,

$$\text{FS}_{\text{(sliding)}} = \frac{(470.42)\tan\left(\frac{2\times 20}{3}\right) + (4)\left(\frac{2}{3}\right)(40) + 215}{158.75}$$

$$= \frac{111.49 + 106.67 + 215}{158.75} = \mathbf{2.73 > 1.5, OK}$$

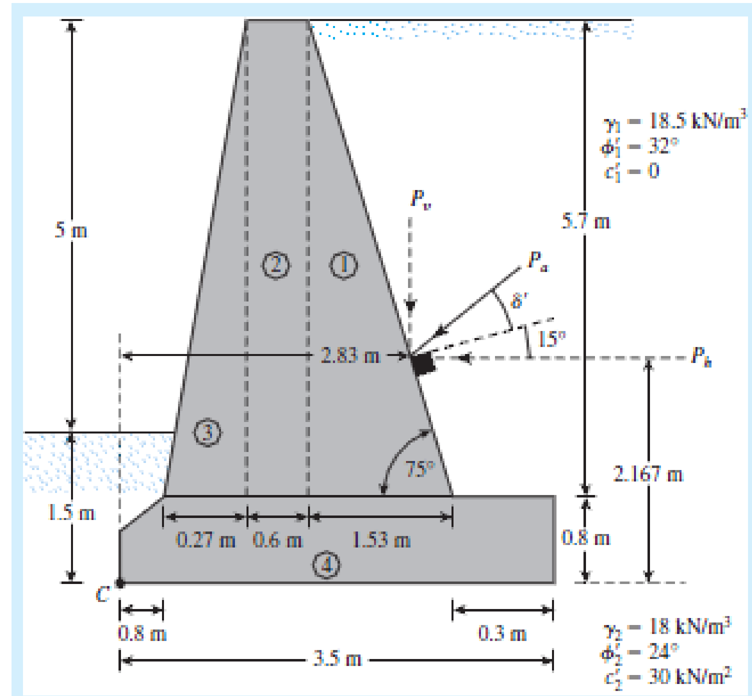
Note: For some designs, the depth *D* in a passive pressure calculation may be taken to be *equal to the thickness of the base slab*.

EXAMPLE 13.2

Example 13.2

A gravity retaining wall is shown in Figure 13.13. Use $\delta' = 2/3\phi'_1$ and Coulomb's active earth pressure theory. Determine

SLIDING



Solution

The height

$$H' = 5 + 1.5 = 6.5 \text{ m}$$

Coulomb's active force is

$$P_a = \tfrac{1}{2}\gamma_1 H'^2 K_a$$

With $\alpha = 0°$, $\beta = 75°$, $\delta' = 2/3\phi'_1$, and $\phi'_1 = 32°$, $K_a = 0.4023$. (See Table 12.6.) So,

$$P_a = \tfrac{1}{2}(18.5)(6.5)^2(0.4023) = 157.22 \text{ kN/m}$$

$$P_h = P_a \cos(15 + \tfrac{2}{3}\phi'_1) = 157.22 \cos 36.33 = 126.65 \text{ kN/m}$$

and

$$P_v = P_a \sin(15 + \tfrac{2}{3}\phi'_1) = 157.22 \sin 36.33 = 93.14 \text{ kN/m}$$

Area no.	Area (m²)	Weight* (kN/m)	Moment arm from *C* (m)	Moment (kN-m/m)
1	$\tfrac{1}{2}(5.7)(1.53) = 4.36$	102.81	2.18	224.13
2	$(0.6)(5.7) = 3.42$	80.64	1.37	110.48
3	$\tfrac{1}{2}(0.27)(5.7) = 0.77$	18.16	0.98	17.80
4	$\approx (3.5)(0.8) = 2.8$	66.02	1.75	115.54
		$P_v = 93.14$	2.83	263.59
		$\Sigma V = 360.77$ kN/m		$\Sigma M_R = 731.54$ kN-m/m

*$\gamma_{concrete} = 23.58$ kN/m³

Note that the weight of the soil above the back face of the wall is not taken into account in the preceding table. We have

EXAMPLE 13.2

$$\text{FS}_{(\text{sliding})} = \frac{(\Sigma V)\tan\left(\frac{2}{3}\phi_2'\right) + \frac{2}{3}c_2'B + P_p}{P_h}$$

$$P_p = \tfrac{1}{2}K_p\gamma_2 D^2 + 2c_2'\sqrt{K_p}D$$

$$K_p = \tan^2\left(45 + \frac{24}{2}\right) = 2.37$$

$$P_p = \tfrac{1}{2}(2.37)(18)(1.5)^2 + 2(30)(1.54)(1.5) = 186.59 \text{ kN/m}$$

$$\text{FS}_{(\text{sliding})} = \frac{360.77\tan\left(\frac{2}{3}\times 24\right) + \frac{2}{3}(30)(3.5) + 186.59}{126.65}$$

$$= \frac{103.45 + 70 + 186.59}{126.65} = \mathbf{2.84}$$

If P_p is ignored, the factor of safety is **1.37**.

STABILITY FOR BEARING CAPACITY FAILURE

$$\mathbf{R} = \Sigma\mathbf{V} + \mathbf{P}_h$$

$\mathbf{P}_h$ is $\bar{P}_a \cos\alpha$.

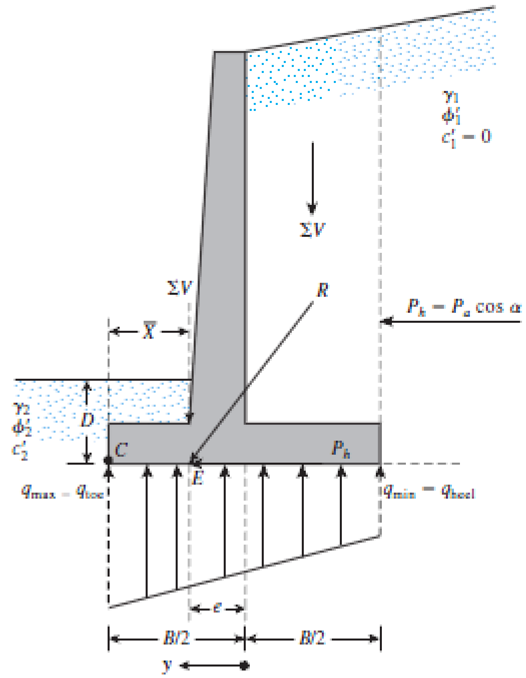
The net moment of these forces about point *C* is

$$M_{net} = \Sigma M_R - \Sigma M_o$$

Let the line of action of the resultant *R* intersect the base slab at *E*. Then the distance

$$\overline{CE} = \overline{X} = \frac{M_{net}}{\Sigma V}$$

$$e = \frac{B}{2} - \overline{CE}$$



$$q = \frac{\Sigma V}{A} \pm \frac{M_{net}y}{I}$$

$$q_{max} = q_{toe} = \frac{\Sigma V}{(B)(1)} + \frac{e(\Sigma V)\frac{B}{2}}{\left(\frac{1}{12}\right)(B^3)} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right)$$

$$q_{min} = q_{heel} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right)$$

STABILITY FOR BEARING CAPACITY FAILURE

$$q_u = c_2' N_c F_{cd} F_{ci} + q N_q F_{qd} F_{qi} + \frac{1}{2}\gamma_2 B' N_\gamma F_{\gamma d} F_{\gamma i}$$

$$q = \gamma_2 D$$

$$B' = B - 2e$$

Shape factors F_{cs} **,** F_{qs} **, and** $F_{gs} = 1$

Depth factors : (Use B not B')

cases $\frac{D}{B} \leq 1$

1. **For $\phi = 0.0$**

$$F_{cd} = 1 + 0.4\left(\frac{D_f}{B}\right)$$

$$F_{qd} = 1$$

$$F_{\gamma d} = 1$$

2. **For $\phi > 0.0$**

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi}$$

$$F_{qd} = 1 + 2\tan\phi\,(1 - \sin\phi)^2\left(\frac{D_f}{B}\right)$$

$$F_{\gamma d} = 1$$

Inclination factors

$$F_{ci} = F_{qi} = \left(1 - \frac{\psi^\circ}{90^\circ}\right)^2$$

$$F_{\gamma i} = \left(1 - \frac{\psi^\circ}{\phi_2'^\circ}\right)^2$$

$$\psi^\circ = \tan^{-1}\left(\frac{P_a \cos\alpha}{\Sigma V}\right)$$

$$FS_{(\text{bearing capacity})} = \frac{q_u}{q_{max}}$$

EXAMPLE 13.1

Example 13.1

The cross section of a cantilever retaining wall is shown in Figure 13.12. Calculate the factors of safety with respect to

Bearing Capacity Failure

Solution

From the figure,

$$H' = H_1 + H_2 + H_3 = 2.6 \tan 10° + 6 + 0.7$$
$$= 0.458 + 6 + 0.7 = 7.158 \text{ m}$$

The Rankine active force per unit length of wall $= P_p = \frac{1}{2}\gamma_1 H'^2 K_a$. For $\phi'_1 = 30°$ and $\alpha = 10°$, K_a is equal to 0.3495. (See Table 12.1.) Thus,

$$P_a = \tfrac{1}{2}(18)(7.158)^2(0.3495) = 161.2 \text{ kN/m}$$

$$P_v = P_a \sin 10° = 161.2\,(\sin 10°) = 28.0 \text{ kN/m}$$

and

$$P_h = P_a \cos 10° = 161.2\,(\cos 10°) = 158.75 \text{ kN/m}$$

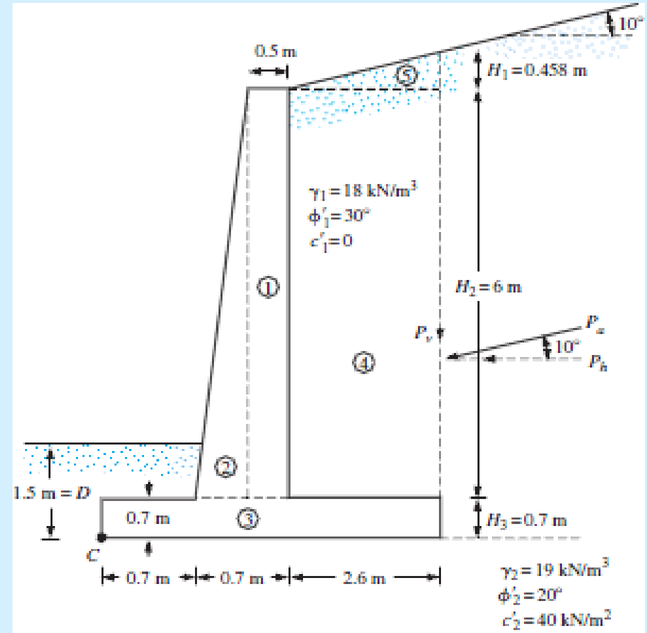


Figure 13.12 Calculation of stability of a retaining wall

Section no.[a]	Area (m²)	Weight/unit length (kN/m)	Moment arm from point *C* (m)	Moment (kN-m/m)
1	$6 \times 0.5 = 3$	70.74	1.15	81.35
2	$\frac{1}{2}(0.2)6 = 0.6$	14.15	0.833	11.79
3	$4 \times 0.7 = 2.8$	66.02	2.0	132.04
4	$6 \times 2.6 = 15.6$	280.80	2.7	758.16
5	$\frac{1}{2}(2.6)(0.458) = 0.595$	10.71	3.13	33.52
		$P_v = 28.0$	4.0	112.0
		$\Sigma V = 470.42$		$1128.86 = \Sigma M_R$

[a]For section numbers, refer to Figure 13.12

$\gamma_{concrete} = 23.58$ kN/m³

$$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_o}{\Sigma V} = \frac{4}{2} - \frac{1128.86 - 378.78}{470.42}$$

$$= 0.406 \text{ m} < \frac{B}{6} = \frac{4}{6} = 0.666 \text{ m}$$

Again, from Eqs. (13.20) and (13.21)

$$q_{\substack{toe\\heel}} = \frac{\Sigma V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{470.42}{4}\left(1 \pm \frac{6 \times 0.406}{4}\right) = 189.2 \text{ kN/m}^2 \text{ (toe)}$$

$$= 45.98 \text{ kN/m}^2 \text{ (heel)}$$

EXAMPLE 13.1

The ultimate bearing capacity of the soil can be determined from Eq. (13.22)

$$q_u = c_2' N_c F_{cd} F_{ci} + q N_q F_{qd} F_{qi} + \frac{1}{2}\gamma_2 B' N_\gamma F_{\gamma d} F_{\gamma i}$$

For $\phi_2' = 20°$ (see Table 4.2), $N_c = 14.83$, $N_q = 6.4$, and $N_\gamma = 5.39$. Also,

$$q = \gamma_2 D = (19)(1.5) = 28.5 \text{ kN/m}^2$$

$$B' = B - 2e = 4 - 2(0.406) = 3.188 \text{ m}$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi_2'} = 1.148 - \frac{1 - 1.148}{(14.83)(\tan 20)} = 1.175$$

$$F_{qd} = 1 + 2\tan\phi_2'(1 - \sin\phi_2')^2\left(\frac{D}{B'}\right) = 1 + 0.315\left(\frac{1.5}{3.188}\right) = 1.148$$

$$F_{\gamma d} = 1$$

$$F_{ci} = F_{qi} = \left(1 - \frac{\psi°}{90°}\right)^2$$

and

$$\psi = \tan^{-1}\left(\frac{P_a \cos\alpha}{\Sigma V}\right) = \tan^{-1}\left(\frac{158.75}{470.42}\right) = 18.65°$$

So

$$F_{ci} = F_{qi} = \left(1 - \frac{18.65}{90}\right)^2 = 0.628$$

and

$$F_{\gamma i} = \left(1 - \frac{\psi}{\phi_2'}\right)^2 = \left(1 - \frac{18.65}{20}\right)^2 \approx 0$$

Hence,

$$q_u = (40)(14.83)(1.175)(0.628) + (28.5)(6.4)(1.148)(0.628)$$
$$+ \tfrac{1}{2}(19)(5.93)(3.188)(1)(0)$$
$$= 437.72 + 131.5 + 0 = 569.22 \text{ kN/m}^2$$

and

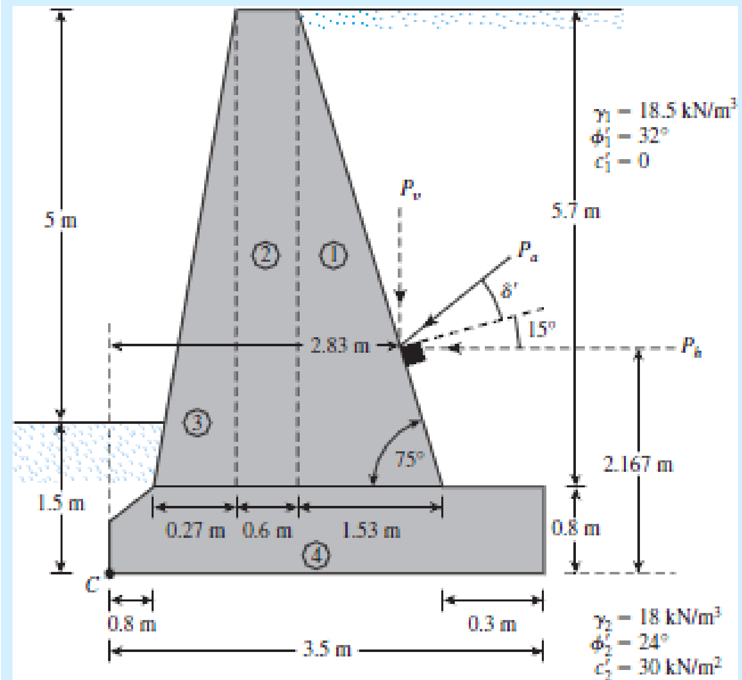
$$FS_{\text{(bearing capacity)}} = \frac{q_u}{q_{\text{toe}}} = \frac{569.22}{189.2} = \mathbf{3.0\ OK}$$

EXAMPLE 13.2

Example 13.2

A gravity retaining wall is shown in Figure 13.13. Use $\delta' = 2/3\phi'_1$ and Coulomb's active earth pressure theory. Determine

The pressure on the soil at the toe and heel



Solution

The height

$$H' = 5 + 1.5 = 6.5 \text{ m}$$

Coulomb's active force is

$$P_a = \tfrac{1}{2}\gamma_1 H'^2 K_a$$

With $\alpha = 0°$, $\beta = 75°$, $\delta' = 2/3\phi'_1$, and $\phi'_1 = 32°$, $K_a = 0.4023$. (See Table 12.6.) So,

$$P_a = \tfrac{1}{2}(18.5)(6.5)^2(0.4023) = 157.22 \text{ kN/m}$$

$$P_h = P_a \cos(15 + \tfrac{2}{3}\phi'_1) = 157.22 \cos 36.33 = 126.65 \text{ kN/m}$$

and

$$P_v = P_a \sin(15 + \tfrac{2}{3}\phi'_1) = 157.22 \sin 36.33 = 93.14 \text{ kN/m}$$

Area no.	Area (m²)	Weight* (kN/m)	Moment arm from C (m)	Moment (kN-m/m)
1	$\frac{1}{2}(5.7)(1.53) = 4.36$	102.81	2.18	224.13
2	$(0.6)(5.7) = 3.42$	80.64	1.37	110.48
3	$\frac{1}{2}(0.27)(5.7) = 0.77$	18.16	0.98	17.80
4	$\approx (3.5)(0.8) = 2.8$	66.02	1.75	115.54
		$P_v = 93.14$	2.83	263.59
		$\Sigma V = 360.77$ kN/m		$\Sigma M_R = 731.54$ kN-m/m

*$\gamma_{concrete} = 23.58$ kN/m³

Note that the weight of the soil above the back face of the wall is not taken into account in the preceding table. We have

$$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_o}{\Sigma V} = \frac{3.5}{2} - \frac{731.54 - 274.45}{360.77} = 0.483 < \frac{B}{6} = 0.583$$

$$q_{toe} = \frac{\Sigma V}{B}\left[1 + \frac{6e}{B}\right] = \frac{360.77}{3.5}\left[1 + \frac{(6)(0.483)}{3.5}\right] = \mathbf{188.43\ kN/m^2}$$

$$q_{heel} = \frac{V}{B}\left[1 - \frac{6e}{B}\right] = \frac{360.77}{3.5}\left[1 - \frac{(6)(0.483)}{3.5}\right] = \mathbf{17.73\ kN/m^2}$$

SETTLEMENT FAILURE

Generally, a factor of safety of 3 is required.

The ultimate bearing capacity of shallow foundations occurs at a settlement of about 10% of the foundation width.

In the case of retaining walls, the width B is large. Hence, the ultimate load q_u will occur at a fairly large foundation settlement.

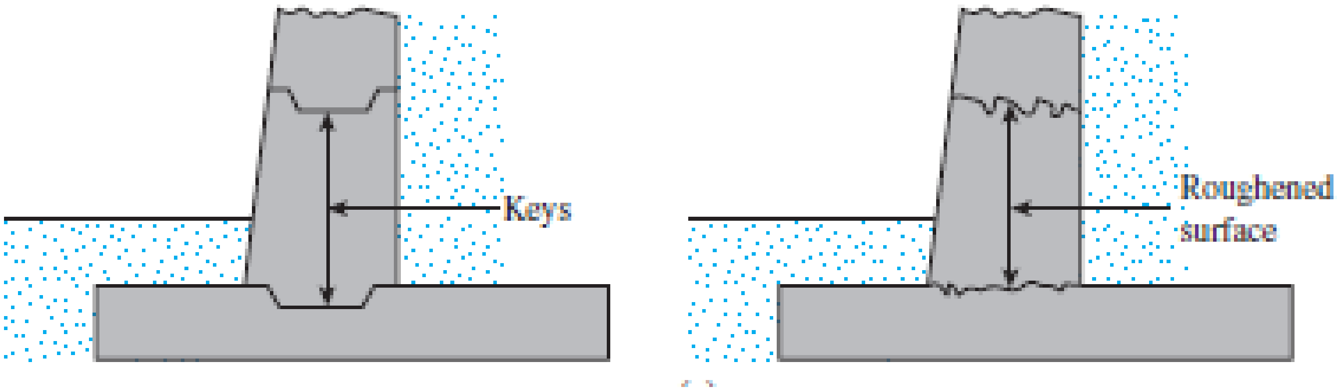
A factor of safety of 3 against bearing capacity failure may not ensure that settlement of the structure will be within the tolerable limit in all cases.

Thus, this situation needs further investigation

CONSTRUCTION JOINTS

Construction joints

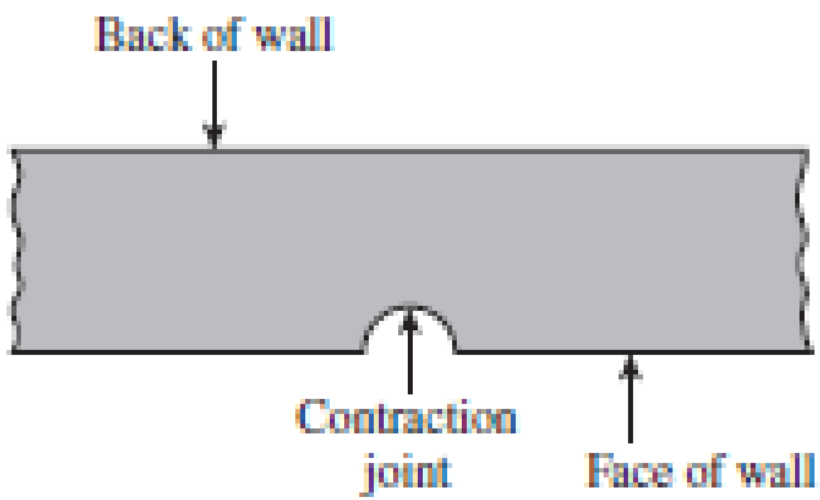
are vertical and horizontal joints that are placed between two successive pours of concrete. To increase the shear at the joints, keys may be used. If keys are not used, the surface of the first pour is cleaned and roughened before the next pour of concrete.



CONSTRUCTION JOINTS

Contraction joints

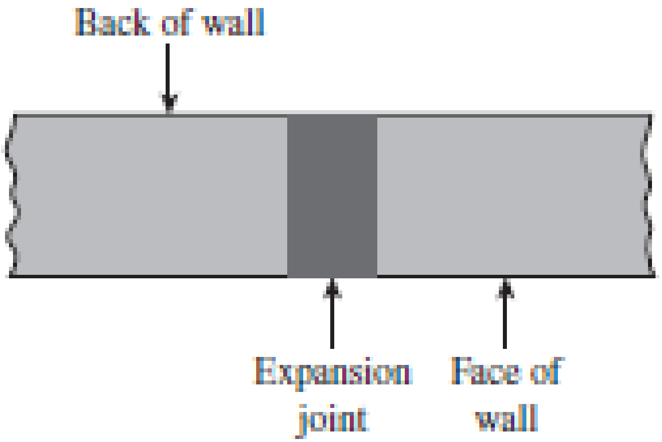
are vertical joints (grooves) placed in the face of a wall (from the top of the base slab to the top of the wall) that allow the concrete to shrink without noticeable harm. The grooves may be about 6 to 8 mm wide and 12 to 16 mm deep.



CONSTRUCTION JOINTS

Expansion joints

Allow for the expansion of concrete caused by temperature changes; vertical expansion joints from the base to the top of the wall may also be used. These joints may be filled with flexible joint fillers. In most cases, horizontal reinforcing steel bars running across the stem are continuous through all joints. The steel is greased to allow the concrete to expand.

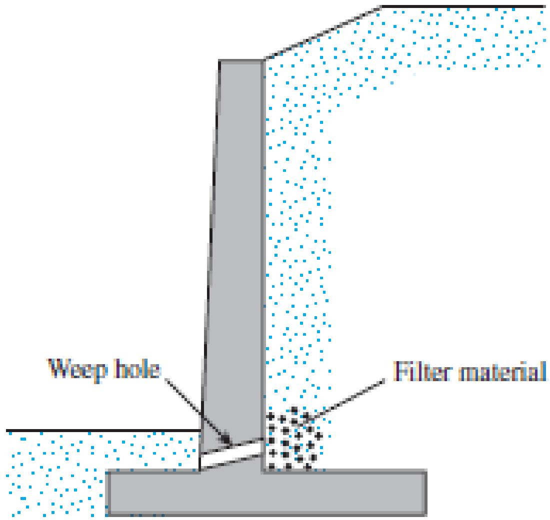


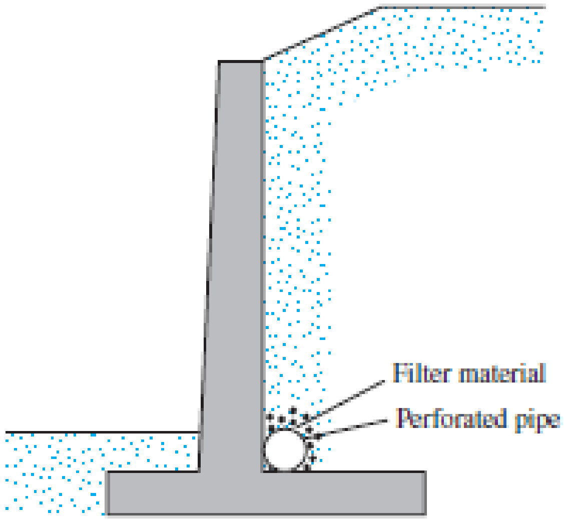
DRAINAGE FROM BACKFILL

As the result of rainfall or other wet conditions, the backfill material for a retaining wall may become saturated, thereby increasing the pressure on the wall and perhaps creating an unstable condition.

For this reason, adequate drainage must be provided by means of

1. **Weep holes**
2. **Perforated drainage pipes**





DRAINAGE FROM BACKFILL

When provided, weep holes should have a minimum diameter of about 0.1 m and be adequately spaced. Note that there is always a possibility that backfill material may be washed into weep holes or drainage pipes and ultimately clog them. Thus, a filter material needs to be placed behind the weep holes or around the drainage pipes, as the case may be; geotextiles now serve that purpose.

Two main factors influence the choice of filter material:
The grain-size distribution of the materials should be such that
(a) The soil to be protected is not washed into the filter and
(b) Excessive hydrostatic pressure head is not created in the soil with a lower hydraulic conductivity (in this case, the backfill material).

The preceding conditions can be satisfied if the following requirements are met (Terzaghi and Peck, 1967):

$$\frac{D_{15(F)}}{D_{85(B)}} < 5 \qquad \text{[to satisfy condition(a)]}$$

$$\frac{D_{15(F)}}{D_{15(B)}} > 4 \qquad \text{[to satisfy condition(b)]}$$

F : filter
B : backfill soil
D_{15} : diameter through which 15% will pass
D_{85} : diameter through which 85% will pass

DRAINAGE FROM BACKFILL

Example 13.3

Figure 13.16 shows the grain-size distribution of a backfill material. Using the conditions outlined in Section 13.8, determine the range of the grain-size distribution for the filter material.

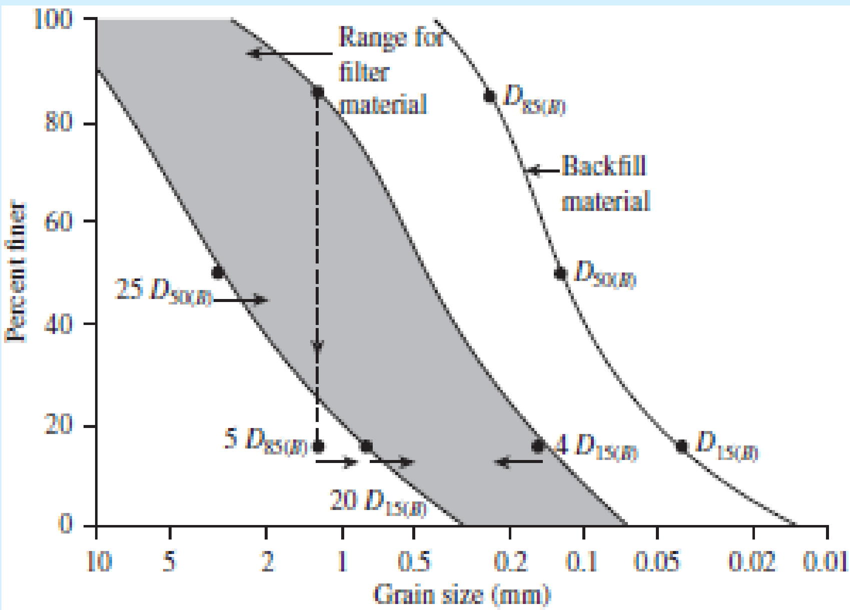


Figure 13.16 Determination of grain-size distribution of filter material

Solution

From the grain-size distribution curve given in the figure, the following values can be determined:

$$D_{15(B)} = 0.04 \text{ mm}$$
$$D_{85(B)} = 0.25 \text{ mm}$$
$$D_{50(B)} = 0.13 \text{ mm}$$

Conditions of Filter

1. $D_{15(F)}$ should be less than $5D_{85(B)}$; that is, $5 \times 0.25 = 1.25$ mm.
2. $D_{15(F)}$ should be greater than $4D_{15(B)}$; that is, $4 \times 0.04 = 0.16$ mm.
3. $D_{50(F)}$ should be less than $25D_{50(B)}$; that is, $25 \times 0.13 = 3.25$ mm.
4. $D_{15(F)}$ should be less than $20D_{15(B)}$; that is, $20 \times 0.04 = 0.8$ mm.

These limiting points are plotted in Figure 13.16. Through them, two curves can be drawn that are similar in nature to the grain-size distribution curve of the backfill material. These curves define the range of the filter material to be used. ■

Mechanically Stabilized Retaining Walls

More recently, soil reinforcement has been used in the construction and design of foundations, retaining walls, embankment slopes, and other structures. Depending on the type of construction, the reinforcements may be galvanized metal strips, geotextiles, geogrids, or geocomposites.

Reinforcement materials such as metallic strips, geotextiles, and geogrids are now being used to reinforce the backfill of retaining walls, which are generally referred to as mechanically stabilized retaining walls*.*

SOIL REINFORCEMENT

Reinforced earth is a construction material made from soil that has been strengthened by tensile elements such as metal rods or strips, non biodegradable fabrics (geotextiles), geogrids, and the like.

The first reinforced-earth retaining wall with metal strips as reinforcement in the United States was constructed in 1972 in southern California.

The beneficial effects of soil reinforcement derive from:
(a) the soil's increased tensile strength and
(b) the shear resistance developed from the friction at the soil-reinforcement interfaces.

Such reinforcement is comparable to that of concrete structures. Currently, most reinforced-earth design is done with free-draining granular soil only. Thus, the effect of pore water development in cohesive soils, which, in turn, reduces the shear strength of the soil, is avoided.

Considerations in Soil Reinforcement

Metal Strips

In most instances, galvanized steel strips are used as reinforcement in soil. However, galvanized steel is subject to corrosion. The rate of corrosion depends on several environmental factors.

Binquet and Lee (1975) suggested that the average rate of corrosion of galvanized steel strips varies between 0.025 and 0.050 mm/yr. So, in the actual design of reinforcement, allowance must be made for the rate of corrosion. Thus,

$$t_c = t_{\text{design}} + r\,(\text{life span of structure})$$

Where
t_c = actual thickness of reinforcing strips to be used in construction
t_{design} = thickness of strips determined from design calculations
r = rate of corrosion

Considerations in Soil Reinforcement

Nonbiodegradable Fabrics

Nonbiodegradable fabrics are generally referred to as geotextiles.
Geotextiles are not prepared from natural fabrics, because they decay too quickly.
Geotextiles may be woven, knitted, or nonwoven.
Woven geotextiles are made of two sets of parallel filaments or strands of yarn systematically interlaced to form a planar structure.
Knitted geotextiles are formed by interlocking a series of loops of one or more filaments or strands of yarn to form a planar structure.
Nonwoven geotextiles are formed from filaments or short fibers arranged in an oriented or random pattern in a planar structure.

These filaments or short fibers are arranged into a loose web in the beginning and then are bonded by one or a combination of the following processes:

1. Chemical bonding —by glue, rubber, latex, a cellulose derivative
2. Thermal bonding —by heat for partial melting of filaments
3. Mechanical bonding —by needle punching

Needle-punched nonwoven geotextiles are thick and have high in-plane permeability

GEOTEXTILES

Geotextiles have four primary uses in foundation engineering:

1. Drainage: The fabrics can rapidly channel water from soil to various outlets, thereby providing a higher soil shear strength and hence stability.

2. Filtration: When placed between two soil layers, one coarse grained and the other fine grained, the fabric allows free seepage of water from one layer to the other. However, it protects the fine-grained soil from being washed into the coarse-grained soil.

3. Separation: Geotextiles help keep various soil layers separate after construction and during the projected service period of the structure. For example, in the construction of highways, a clayey subgrade can be kept separate from a granular base course.

4. Reinforcement: The tensile strength of geofabrics increases the load-bearing capacity of the soil.

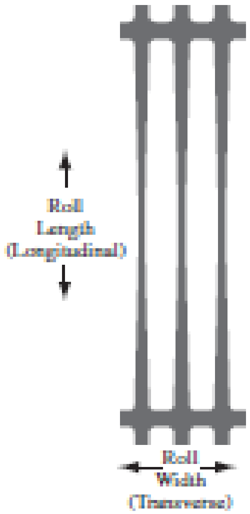
GEOGRIDS

Geogrids are high-modulus polymer materials, such as polypropylene and polyethylene, and are prepared by tensile drawing.
Geogrids generally are of two types: (a) uniaxial and (b) biaxial.
Commercially available geogrids may be categorized by manufacturing process, principally: extruded, woven, and welded.

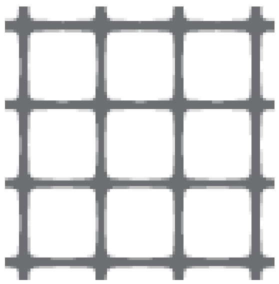
Extruded geogrids are formed using a thick sheet of polyethylene or polypropylene that is punched and drawn to create apertures and to enhance engineering properties of the resulting ribs and nodes.

Woven geogrids are manufactured by grouping polymeric—usually polyester and polypropylene—and weaving them into a mesh pattern that is then coated with a polymeric lacquer.

Welded geogrids are manufactured by fusing junctions of polymeric strips. Extruded geogrids have shown good performance when compared to other types for pavement reinforcement applications.



uniaxial



biaxial

GEOGRIDS

The commercial geogrids currently available for soil reinforcement have nominal rib thicknesses of about 0.5 to 1.5 mm and junctions of about 2.5 to 5 mm. The grids used for soil reinforcement usually have openings or apertures that are rectangular or elliptical. The dimensions of the apertures vary from about 25 to150 mm. Geogrids are manufactured so that the open areas of the grids are greater than 50% of the total area. They develop reinforcing strength at low strain levels, such as 2% (Carroll, 1988).

The major function of geogrids is reinforcement. They are relatively stiff. The apertures are large enough to allow interlocking with surrounding soil or rock to perform the function of reinforcement or segregation (or both).

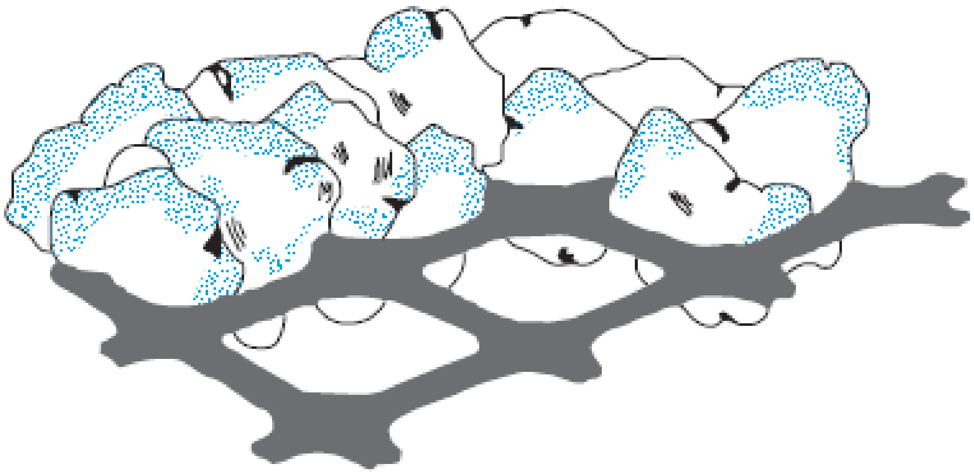


Figure 13.21 Geogrid apertures allowing interlocking with surrounding soil

GEOGRIDS

Sarsby (1985) investigated the influence of aperture size on the size of soil particles for maximum frictional occurs when

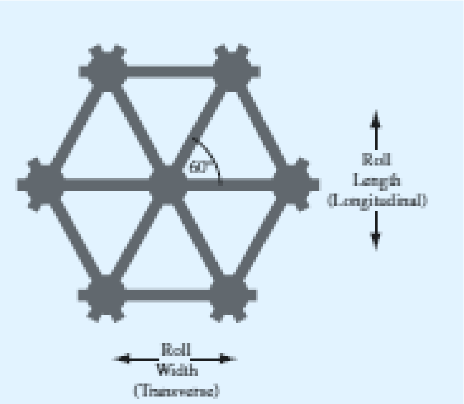
$$B_{GG} > 3.5 D_{50}$$

where

B_{GG} = minimum width of the geogrid aperture

D_{50} = the particle size through which 50% of the backfill soil passes (i.e., the average particle size)

More recently, geogrids with triangular apertures have been introduced for construction purposes. Geogrids with triangular apertures are manufactured from a punched polypropylene sheet, which is then oriented in three substantially equilateral directions so that the resulting ribs shall have a high degree of molecular orientation.

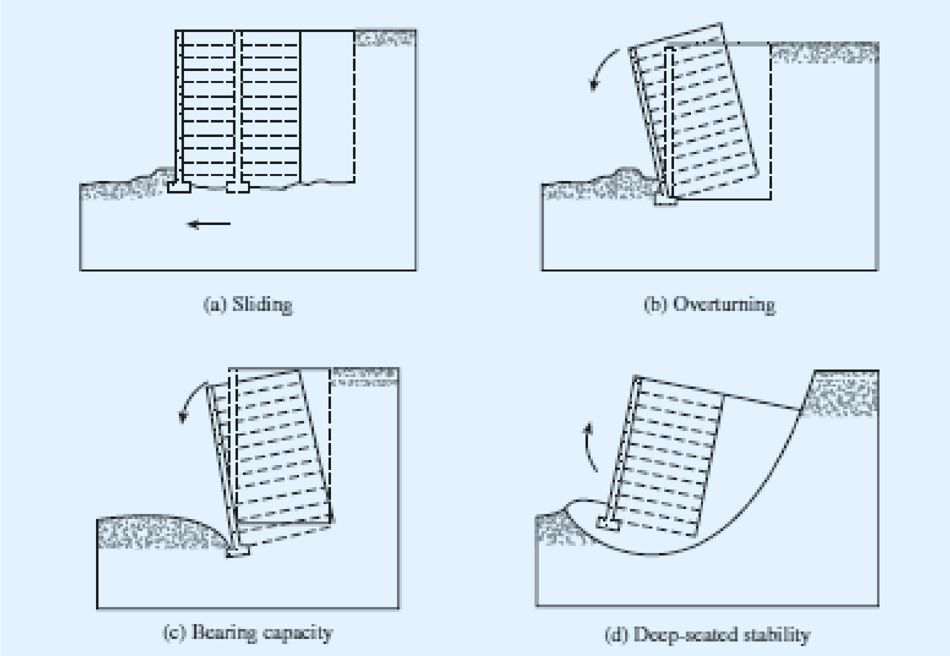


General Design Considerations

The general design procedure of any mechanically stabilized retaining wall can be divided into two parts:

1. Satisfying internal stability requirements
2. Checking the external stability of the wall

The internal stability checks involve determining tension and pullout resistance in the reinforcing elements and ascertaining the integrity of facing elements. The external stability checks include checks for overturning, sliding, and bearing capacity failure.



Retaining Walls with Metallic Strip Reinforcement

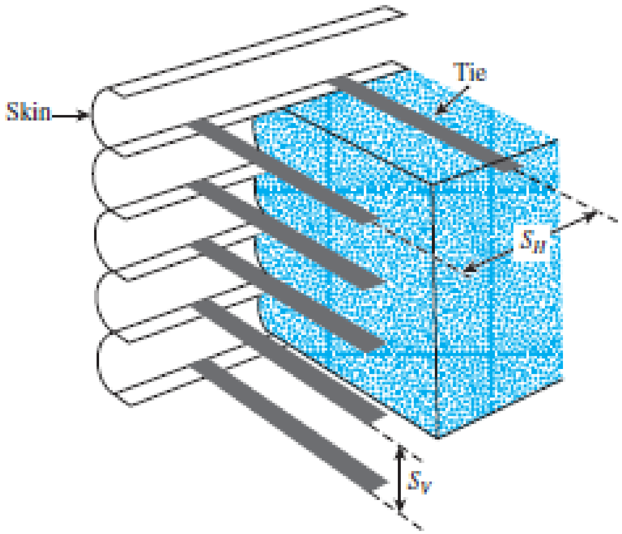
Reinforced-earth walls are flexible walls. Their main components are

1. Backfill, which is granular soil

2. Reinforcing strips, which are thin, wide strips placed at regular intervals, and

3. A cover or skin, on the front face of the wall

At any depth, the reinforcing strips or ties are placed with a horizontal spacing of S_H center to center; the vertical spacing of the strips or ties is S_V center to center. The skin can be constructed with sections of relatively flexible thin material.

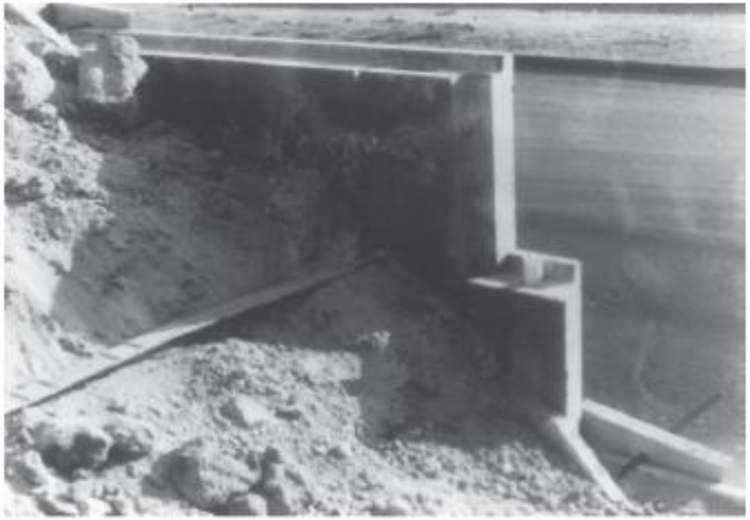


Lee et al. (1973) showed that, with a conservative design, a 5 mm-thick galvanized steel skin would be enough to hold a wall about 14 to 15 m high. In most cases, precast concrete slabs also can be used as skin. The slabs are grooved to fit into each other so that soil cannot flow out between the joints. When metal skins are used, they are bolted together, and reinforcing strips are placed between the skins.

Retaining Walls with Metallic Strip Reinforcement

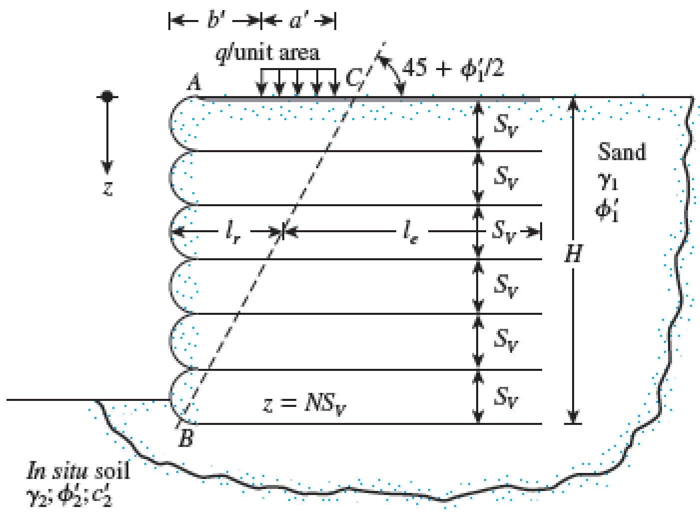
Reinforced-earth retaining wall (with metallic strip) under construction

Metallic strip attachment to the precast concrete slab used as the skin



Retaining Walls with Metallic Strip Reinforcement

Calculation of Active Horizontal and Vertical Pressure - *Rankine method*



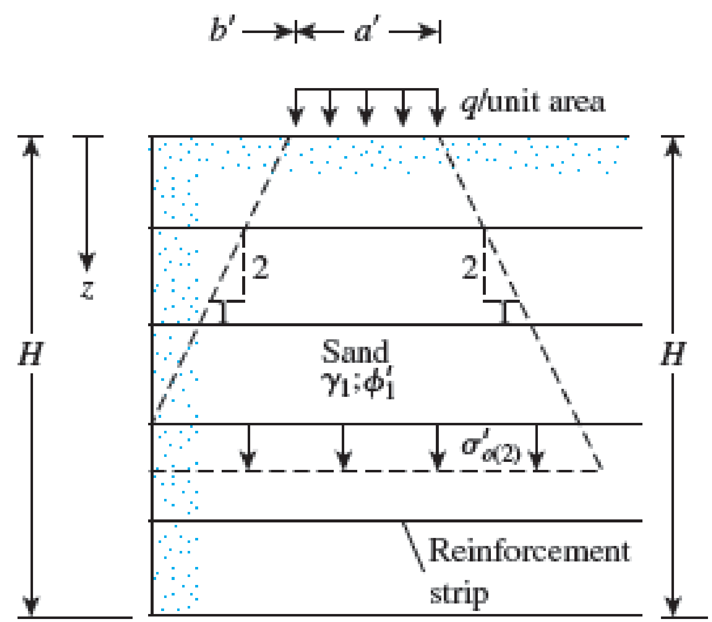
$$\sigma'_{o(2)} = \frac{qa'}{a' + z} \qquad (\text{for } z \le 2b')$$

$$\sigma'_{o(2)} = \frac{qa'}{a' + \frac{z}{2} + b'} \qquad (\text{for } z > 2b')$$

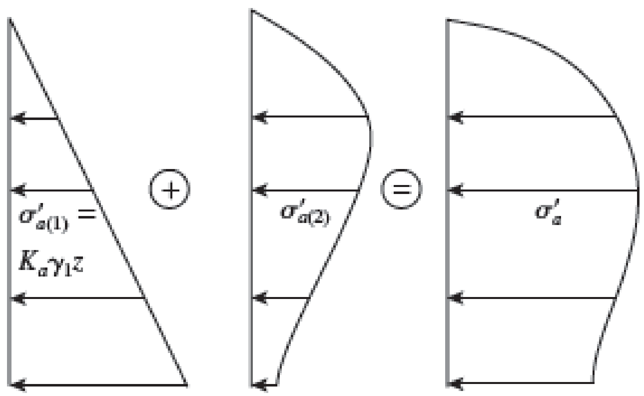
$$\sigma'_o = \sigma'_{o(1)} + \sigma'_{o(2)}$$

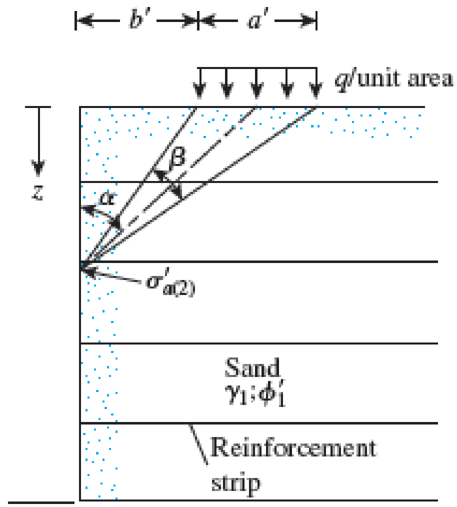
$\sigma'_{o(1)} = \gamma_1 z$ — Due to soil only

$\sigma'_{o(2)}$ — Due to the surcharge



Retaining Walls with Metallic Strip Reinforcement





$$\sigma'_a = \underbrace{\sigma'_{a(1)}}_{= K_a \gamma_1 z \text{ Due to soil only}} + \underbrace{\sigma'_{a(2)}}_{\text{Due to the surcharge}}$$

$$\sigma'_{a(2)} = M\left[\frac{2q}{\pi}\left(\underbrace{\beta}_{\text{(in radians)}} - \sin\beta\cos 2\alpha\right)\right]$$

$$M = 1.4 - \frac{0.4b'}{0.14H} \geq 1$$

Retaining Walls with Metallic Strip Reinforcement

The tie force *per unit length of the wall* developed at any depth *z* is

$$T = \text{active earth pressure at depth } z \times \text{ area of the wall to be supported by the tie} = (\sigma'_a)(S_V S_H)$$

Factor of Safety against Tie Failure (2.5 to 3 is recommended)

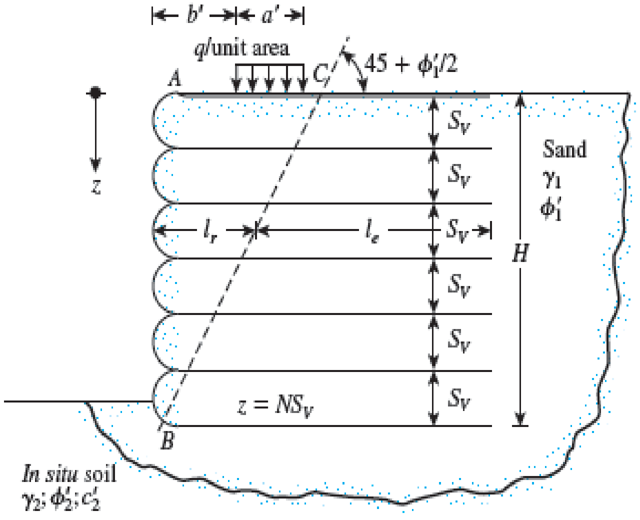
$$FS_{(B)} = \frac{\text{yield or breaking strength of each tie}}{\text{maximum force in any tie}} = \frac{w t f_y}{\sigma'_a S_V S_H}$$

w = width of each tie
t = thickness of each tie
f_y = yield or breaking strength of the tie material

Retaining Walls with Metallic Strip Reinforcement

Reinforcing ties at any depth *z* will fail by pullout if the frictional resistance developed along the surfaces of the ties is less than the force to which the ties are being subjected.

The effective length of the ties along which frictional resistance is developed may be conservatively taken as the length that extends *beyond the limits of the Rankine active failure zone*, which is the zone *ABC*. Line *BC* makes an angle of (45 + ϕ_1/2) with the horizontal.



The maximum friction force that can be realized for a tie at depth *z* is :

$$F_R = 2 l_e w \sigma'_o \tan \phi'_\mu$$

l_e = effective length
σ'_o = effective vertical pressure at a depth z
ϕ'_μ = soil–tie friction angle

Retaining Walls with Metallic Strip Reinforcement

The factor of safety against **tie pullout** at any depth z

$$FS_{(P)} = \frac{F_R}{T}$$

$$F_R = 2l_e w \sigma_o' \tan \phi_\mu'$$

$$T = (\sigma_a')(S_V S_H)$$

$$FS_{(P)} = \frac{2l_e w \sigma_o' \tan \phi_\mu'}{\sigma_a' S_V S_H}$$

Total Length of Tie

$$L = l_r + l_e$$

l_r = length within the Rankine failure zone
l_e = effective length

$$l_r = \frac{(H - z)}{\tan\left(45 + \frac{\phi_1'}{2}\right)}$$

$$l_e = \frac{FS_{(P)} \sigma_a' S_V S_H}{2w\sigma_o' \tan \phi_\mu'}$$

$$L = \frac{(H - z)}{\tan\left(45 + \frac{\phi_1'}{2}\right)} + \frac{FS_{(P)} \sigma_a' S_V S_H}{2w\sigma_o' \tan \phi_\mu'}$$

Step-by-Step-Design Procedure Using Metallic Strip Reinforcement

General

Step 1. Determine the height of the wall, H, and the properties of the granular backfill material, such as the unit weight (γ_1) and the angle of friction (ϕ_1').

Step 2. Obtain the soil–tie friction angle, ϕ_μ', and the required value of $FS_{(B)}$ and $FS_{(P)}$.

Internal Stability

Step 3. Assume values for horizontal and vertical tie spacing. Also, assume the width of reinforcing strip, w, to be used.

Step 4. Calculate σ_a' from Eqs. (13.31), (13.32), and (13.33).

Step 5. Calculate the tie forces at various levels from Eq. (12.34).

Step 6. For the known values of $FS_{(B)}$, calculate the thickness of ties, t, required to resist the tie breakout:

$$T = \sigma_a' S_V S_H = \frac{w t f_y}{FS_{(B)}}$$

$$t = \frac{(\sigma_a' S_V S_H)[FS_{(B)}]}{w f_y}$$

The convention is to keep the magnitude of t the same at all levels, so σ_a' in Eq. (13.43) should equal $\sigma_{a(\max)}'$.

Step 7. For the known values of ϕ_μ' and $FS_{(P)}$, determine the length L of the ties at various levels from Eq. (13.42).

Step 8. The magnitudes of S_V, S_H, t, w, and L may be changed to obtain the most economical design.

Step-by-Step-Design Procedure Using Metallic Strip Reinforcement

External Stability

Step 9. Check for *overturning*, using Figure 13.29 as a guide. Taking the moment about B yields the overturning moment for the unit length of the wall:

$$M_o = P_a z'$$

Here,

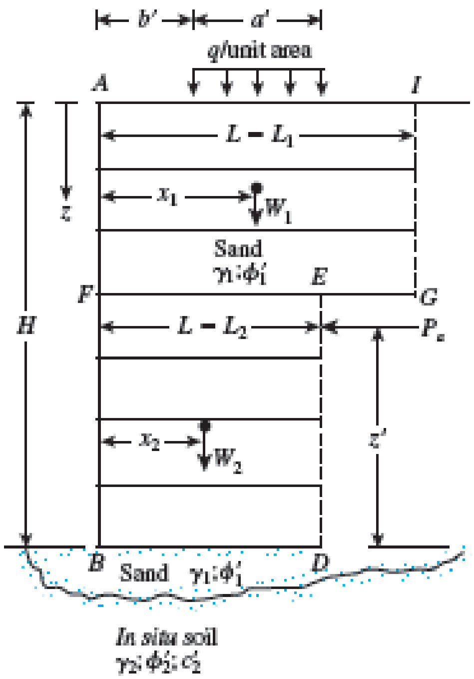
$$P_a = \text{active force} = \int_0^H \sigma'_a dz$$

The resisting moment per unit length of the wall is

$$M_R = W_1 x_1 + W_2 x_2 + \cdots + qa'\left(b' + \frac{a'}{2}\right)$$

where

$W_1 = (\text{area } AFEGI)(1)(\gamma_1)$
$W_2 = (\text{area } FBDE)(1)(\gamma_1)$
$\vdots$



$$\text{FS}_{(\text{overturning})} = \frac{M_R}{M_o}$$

$$= \frac{W_1 x_1 + W_2 x_2 + \cdots + qa'\left(b' + \frac{a'}{2}\right)}{\left(\int_0^H \sigma'_a dz\right) z'}$$

Step-by-Step-Design Procedure Using Metallic Strip Reinforcement

Step 10. The check for *sliding* can be done by using Eq. (13.11), or

$$FS_{(\text{sliding})} = \frac{(W_1 + W_2 + \cdots + qa')[\tan(k\phi_1')]}{P_a} \tag{13.47}$$

where $k \approx \frac{2}{3}$.

Step 11. Check for ultimate bearing capacity failure, which can be given as

$$q_u = c_2'N_c + \tfrac{1}{2}\gamma_2 L_2 N_\gamma \tag{13.48}$$

The bearing capacity factors N_c and N_γ correspond to the soil friction angle ϕ_2'. (See Table 4.2.)
From Eq. 13.28, the vertical stress at $z = H$ is

$$\sigma'_{o(H)} = \gamma_1 H + \sigma'_{o(2)} \tag{13.49}$$

So the factor of safety against bearing capacity failure is

$$FS_{(\text{bearing capacity})} = \frac{q_{\text{ult}}}{\sigma'_{o(H)}} \tag{13.50}$$

Generally, minimum values of $FS_{(\text{overturning})} = 3$, $FS_{(\text{sliding})} = 3$, and $FS_{(\text{bearing capacity failure})} = 3$ to 5 are recommended.

EXAMPLE 13.4

Example 13.4

A 10-m-high retaining wall with galvanized steel-strip reinforcement in a granular backfill has to be constructed. Referring to Figure 13.27, given:

Granular backfill: $\phi'_1 = 36°$
$\gamma_1 = 16.5 \text{ kN/m}^3$

Foundation soil: $\phi'_2 = 28°$
$\gamma_2 = 17.3 \text{ kN/m}^3$
$c'_2 = 50 \text{ kN/m}^2$

Galvanized steel reinforcement:

Width of strip, $w = 75$ mm
$S_V = 0.6$ m center-to-center
$S_H = 1$ m center-to-center
$f_y = 240{,}00 \text{ kN/m}^2$
$\phi'_\mu = 20°$

Required $FS_{(B)} = 3$

Required $FS_{(P)} = 3$

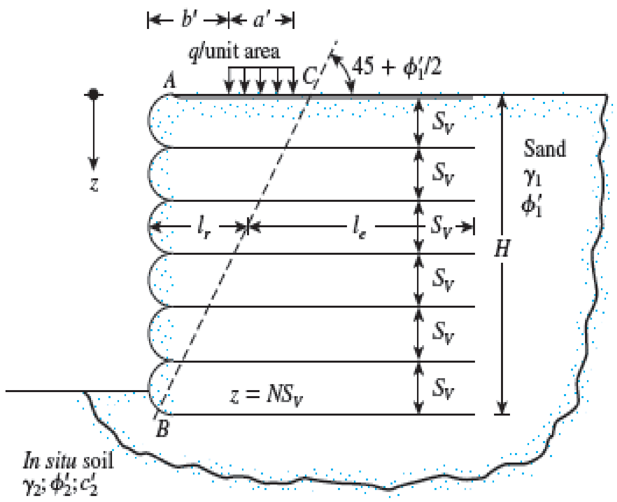
Check for the external and internal stability. Assume the corrosion rate of the galvanized steel to be 0.025 mm/year and the life span of the structure to be 50 years.

Solution

Internal Stability Check

Tie thickness: Maximum tie force, $T_{max} = \sigma'_{a(max)} S_V S_H$

$$\sigma_{a(max)} = \gamma_1 H K_a = \gamma_1 H \tan^2\left(45 - \frac{\phi'_1}{2}\right)$$



EXAMPLE 13.4

so

$$T_{max} = \gamma_1 H \tan^2\left(45 - \frac{\phi_1'}{2}\right) S_V S_H$$

From Eq. (13.43), for *tie break*,

$$t = \frac{(\sigma_a' S_V S_H)[FS_{(B)}]}{wf_y} = \frac{\left[\gamma_1 H \tan^2\left(45 - \frac{\phi_1'}{2}\right) S_V S_H\right] FS_{(B)}}{wf_y}$$

or

$$t = \frac{\left[(16.5)(10)\tan^2\left(45 - \frac{36}{2}\right)(0.6)(1)\right](3)}{(0.075 \text{ m})(240{,}000 \text{ kN/m}^2)} = 0.00428 \text{ m} = 4.28 \text{ mm}$$

If the rate of corrosion is 0.025 mm/yr and the life span of the structure is 50 yr, then the actual thickness, t, of the ties will be

$$t = 4.28 + (0.025)(50) = 5.53 \text{ mm}$$

So a **tie thickness of 6 m** would be enough.

Tie length: Refer to Eq. (13.42). For this case, $\sigma_a' = \gamma_1 z K_a$ and $\sigma_o' = \gamma_1 z$, so

$$L = \frac{(H - z)}{\tan\left(45 + \frac{\phi_1'}{2}\right)} + \frac{FS_{(P)} \gamma_1 z K_a S_V S_H}{2w\gamma_1 z \tan \phi_\mu'}$$

Now the following table can be prepared. (*Note:* $FS_{(P)} = 3$, $H = 10$ m, $w = 75$ mm, and $\phi_\mu' = 20°$.)

z (m)	Tie length L (m) [Eq. (13.42)]
2	12.65
4	11.63
6	10.61
8	9.59
10	8.57

So use a **tie length of L = 13 m.**

EXAMPLE 13.4

External Stability Check

Check for overturning: Refer to Figure 13.30. For this case, using Eq. (13.46)

$$FS_{(overturning)} = \frac{W_1 x_1}{\left[\int_0^H \sigma'_a\, dz\right] z'}$$

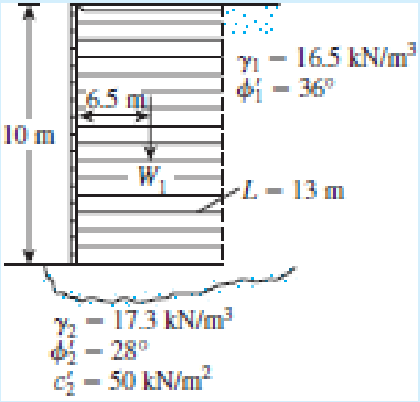


Figure 13.30 Retaining wall with galvanized steel-strip reinforcement in the backfill

$$W_1 = \gamma_1 HL = (16.5)(10)(13) = 2145 \text{ kN/m}$$

$$x_1 = 6.5 \text{ m}$$

$$P_a = \int_0^H \sigma'_a\, dz = \tfrac{1}{2}\gamma_1 K_a H^2 = (\tfrac{1}{2})(16.5)(0.26)(10)^2 = 214.5 \text{ kN/m}$$

$$z' = \frac{10}{3} = 3.33 \text{ m}$$

$$FS_{(overturning)} = \frac{(2145)(6.5)}{(214.5)(3.33)} = \mathbf{19.52 > 3—OK}$$

EXAMPLE 13.4

Check for sliding: From Eq. (13.47)

$$FS_{(\text{sliding})} = \frac{W_1 \tan(k\phi_1')}{P_a} = \frac{2145 \tan\left[\left(\frac{2}{3}\right)(36)\right]}{214.5} = \mathbf{4.45 > 3\text{—OK}}$$

Check for bearing capacity: For $\phi_2' = 28°$, $N_c = 25.8$, $N_\gamma = 16.78$ (Table 4.2). From Eq. (13.48),

$$q_{\text{ult}} = c_2' N_c + \tfrac{1}{2}\gamma_2 L\, N_\gamma$$

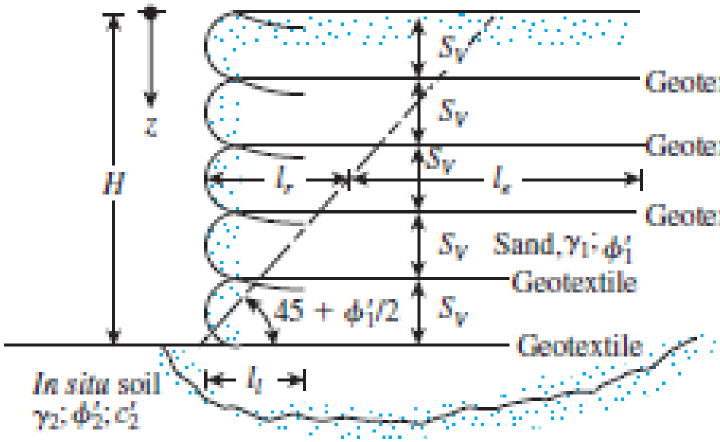
$$q_{\text{ult}} = (50)(25.8) + (\tfrac{1}{2})(17.3)(13)(16.72) = 3170.16 \text{ kN/m}^2$$

From Eq. (13.49),

$$\sigma'_{o(H)} = \gamma_1 H = (16.5)(10) = 165 \text{ kN/m}^2$$

$$FS_{(\text{bearing capacity})} = \frac{q_{\text{ult}}}{\sigma'_{o(H)}} = \frac{3170.16}{165} = \mathbf{19.2 > 5\text{—OK}}$$ ■

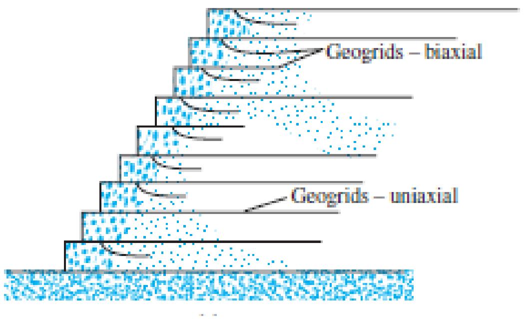
Retaining Walls with Geotextile Reinforcement



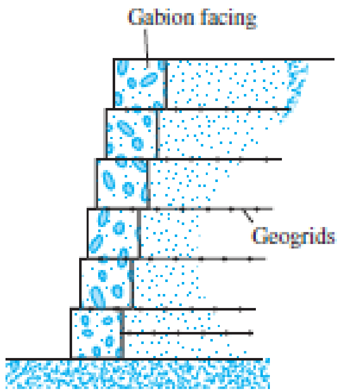
Retaining wall with geotextile reinforcement

Construction of a geotextile-reinforced retaining wall

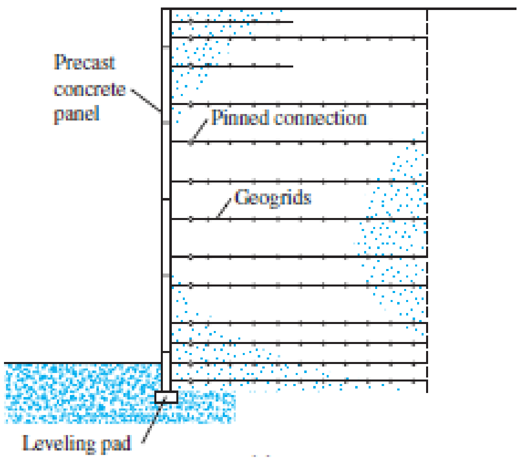
Retaining Walls with Geogrid Reinforcement



Geogrid wraparound wall



Wall with gabion facing



Concrete panel-faced wall