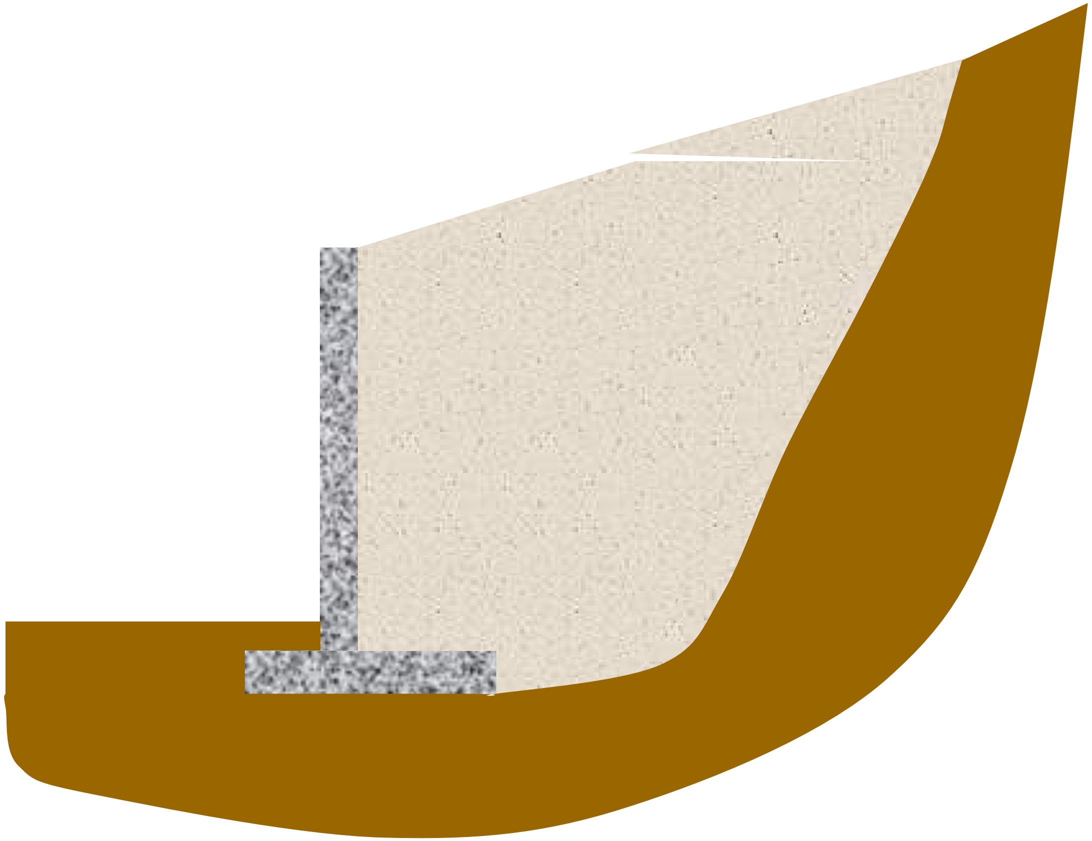
RETAINING WALLS

CHAPTER 17



Omitted parts:

Sections 17.9 -17.18

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.

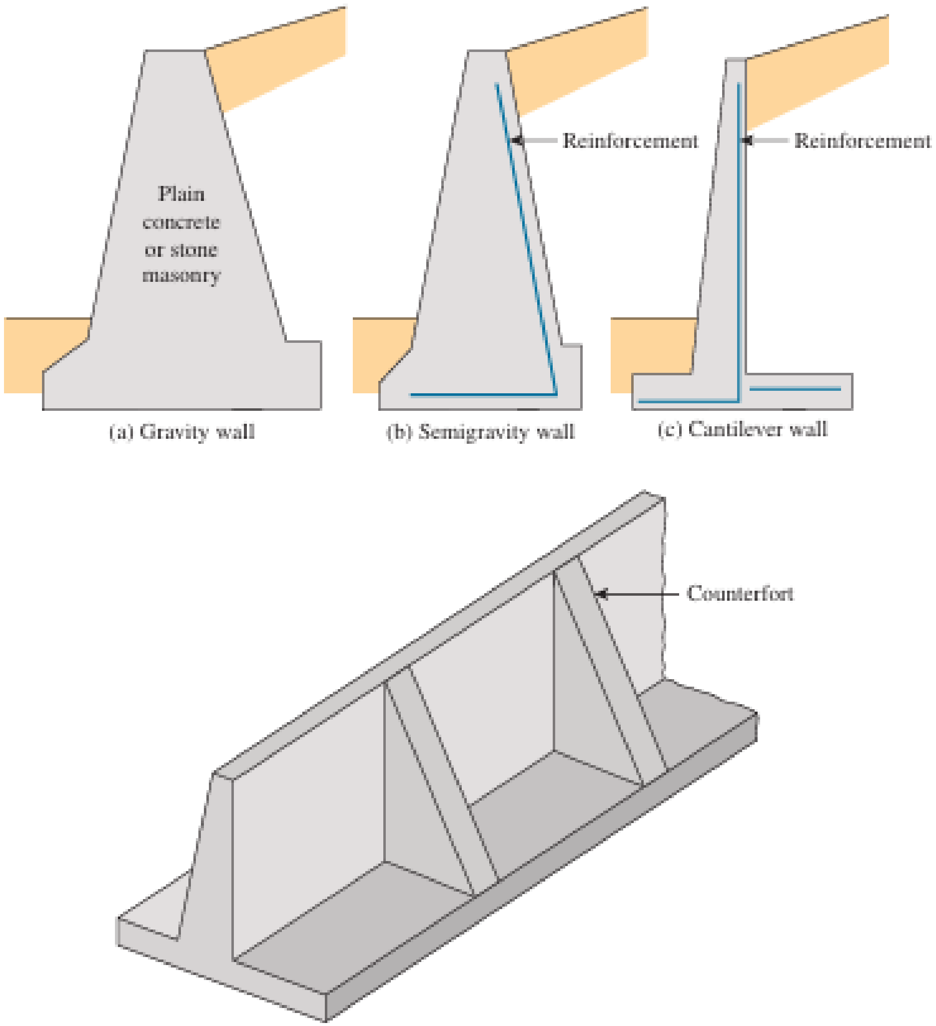
COMMON TYPES OF RETAINING WALLS

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".**

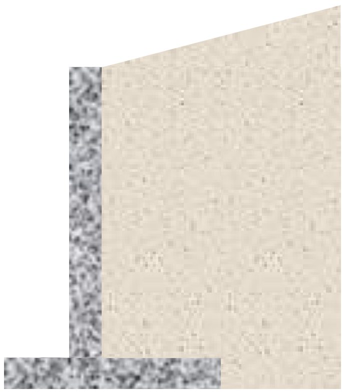
COMMON TYPES OF RETAINING WALLS



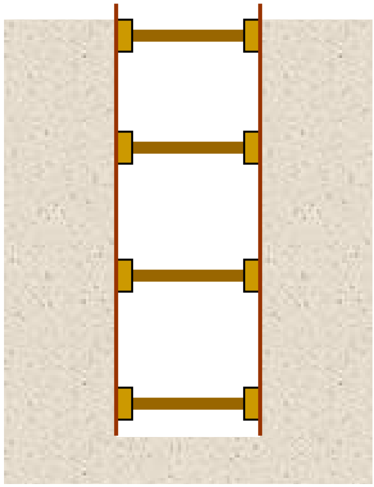
(a) Gravity wall (b) Semigravity wall (c) Cantilever wall

(d) Counterfort wall

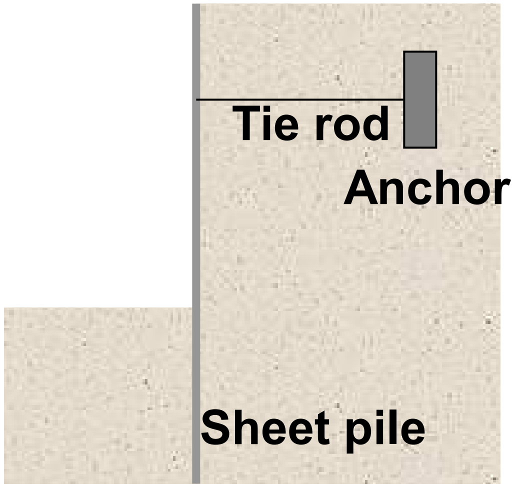
INTRODUCTION



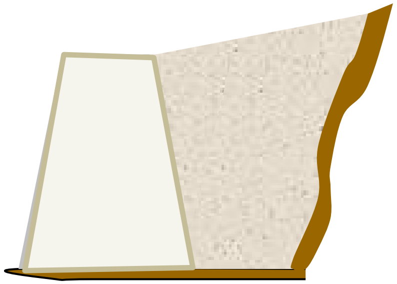
Cantilever retaining wall



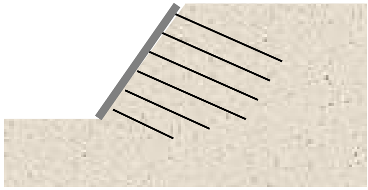
Braced excavation



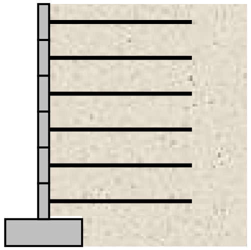
Anchored sheet pile



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.**

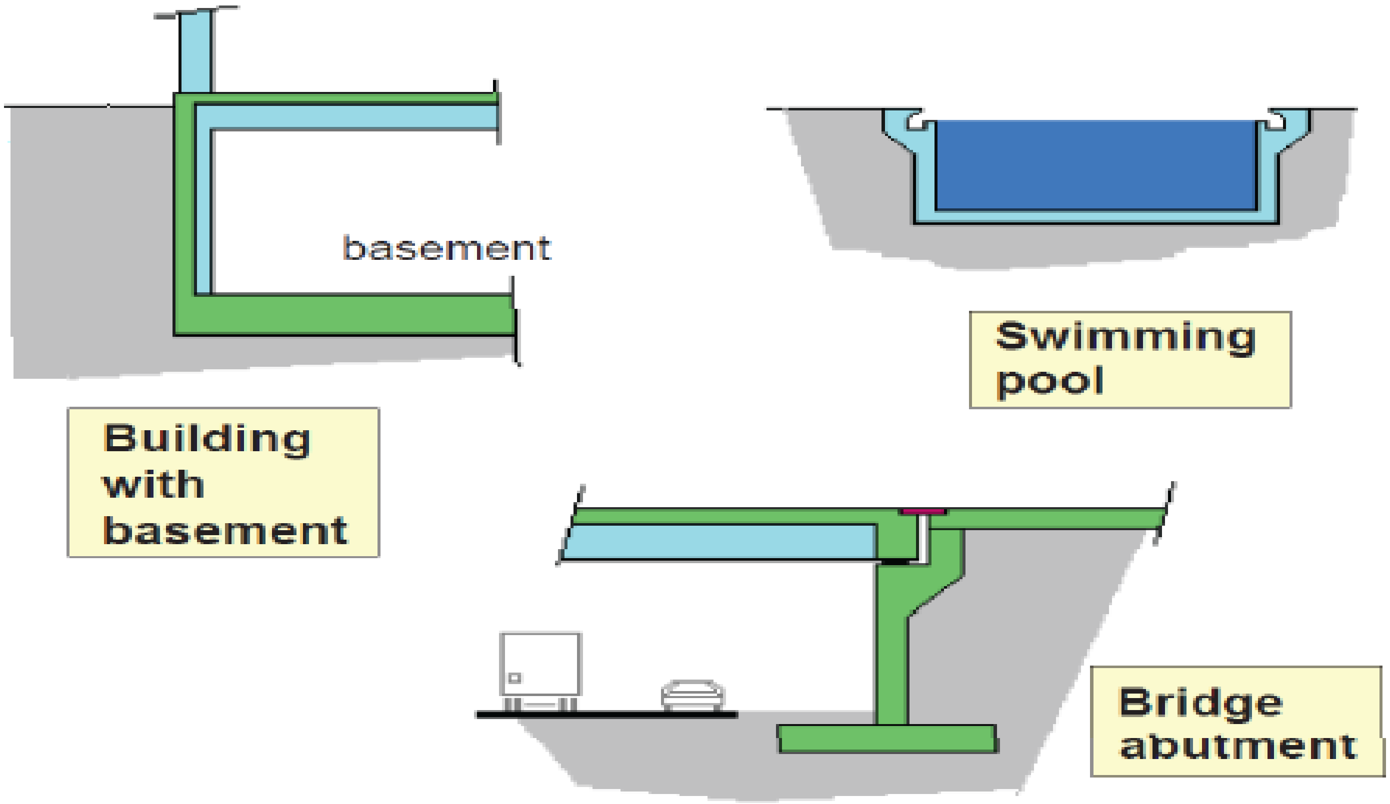
APPLICATIONS OF RETAINING WALLS

Different forms

Different sizes

Different loadings

Different failure concerns



APPLICATIONS OF RETAINING WALLS

APPLICATIONS OF RETAINING WALLS

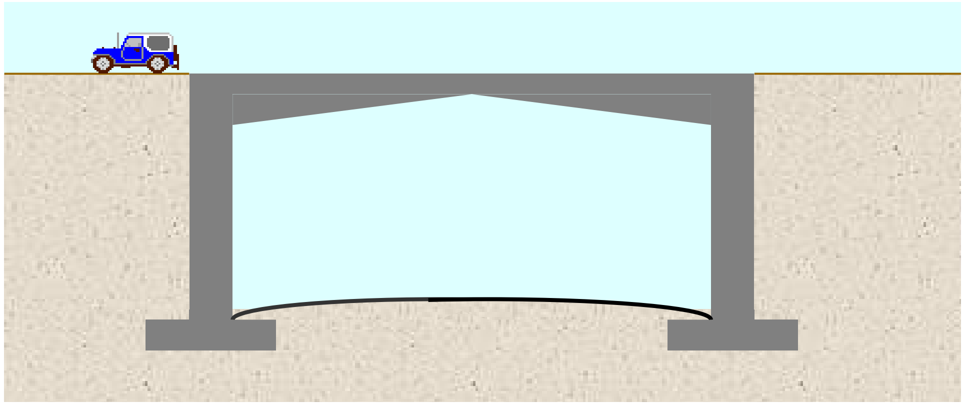
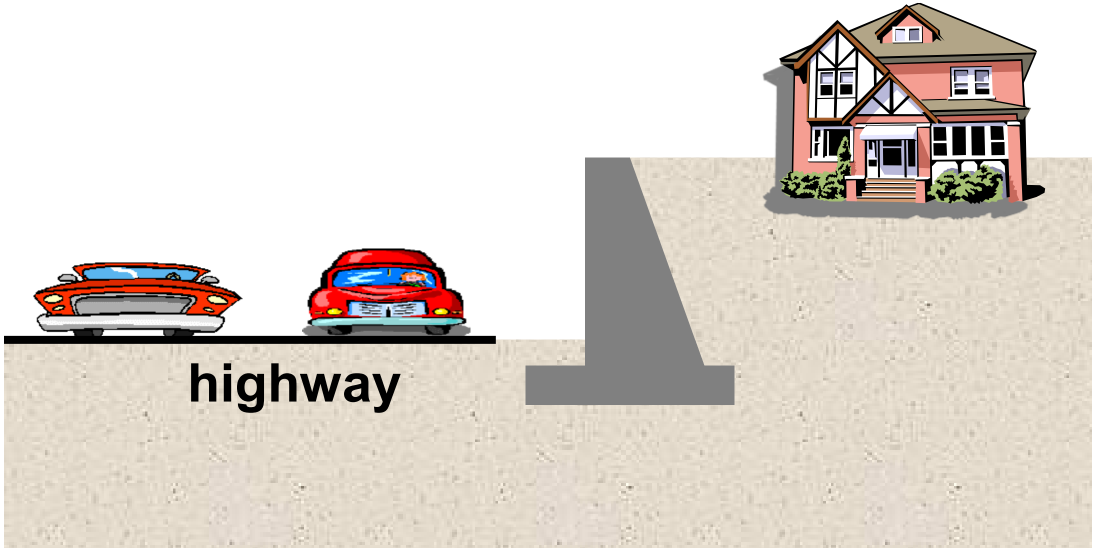
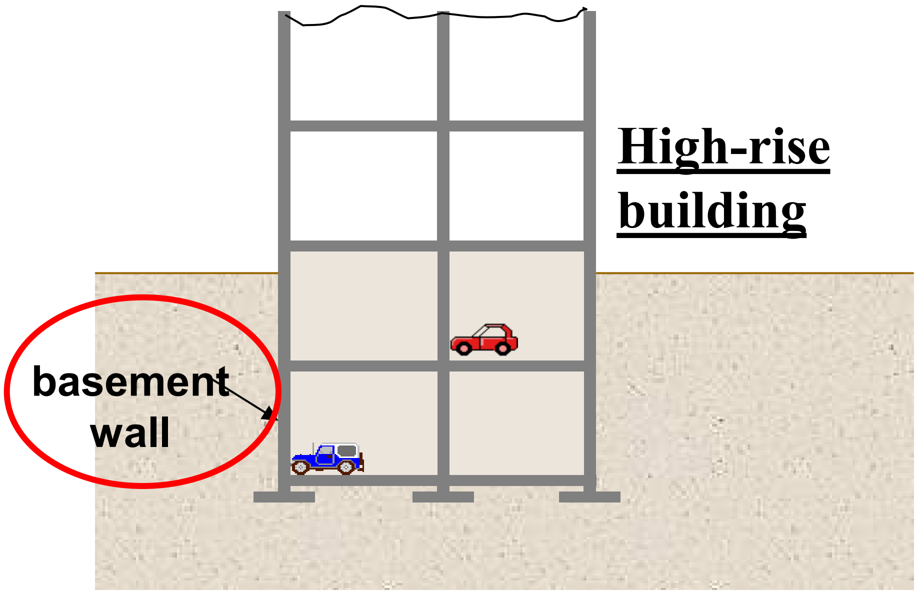


Photo: John Ricard, Maguire Group

APPLICATIONS OF RETAINING WALLS



APPLICATIONS OF RETAINING WALLS



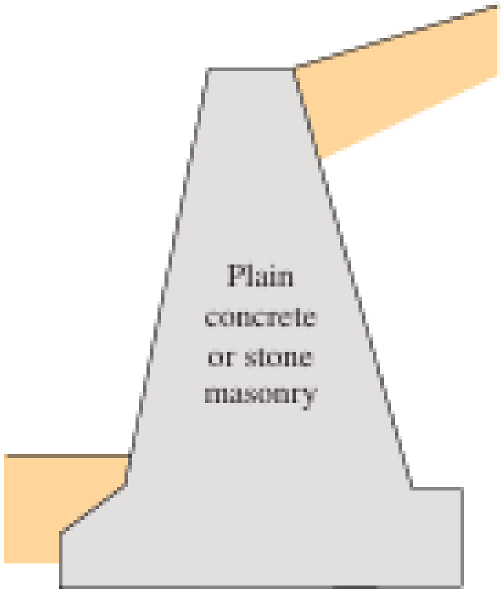
- Not all retaining walls are above ground. Here is an underground wall that contains an automatic car parking system (the Trevis SPA, Milano, Italy).

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.

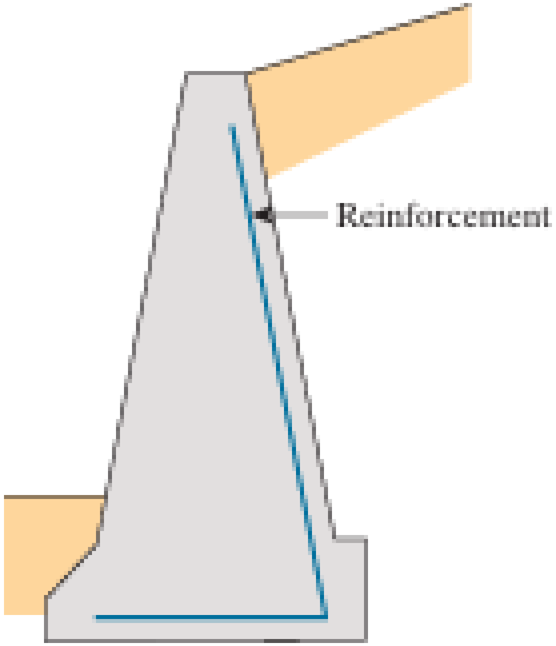
- The following are the main types of wall:
  - Masonry gravity walls
  - Concrete gravity walls



MASONRY WALLS

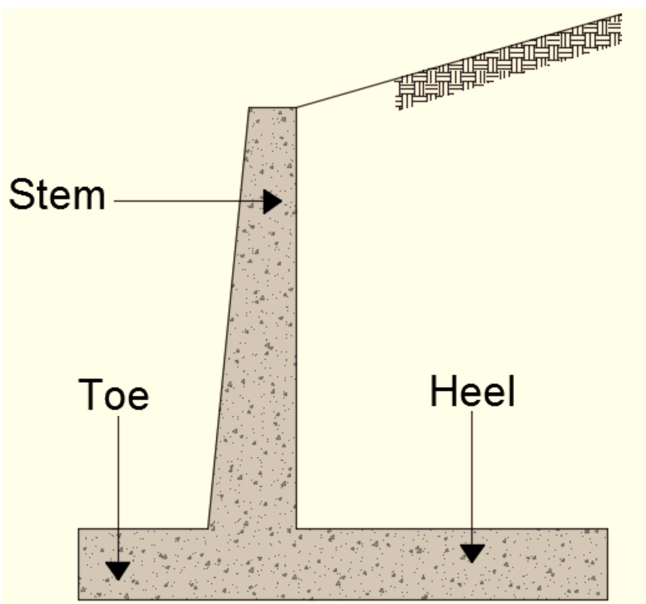
SEMIGRAVITY RETAINING WALLS

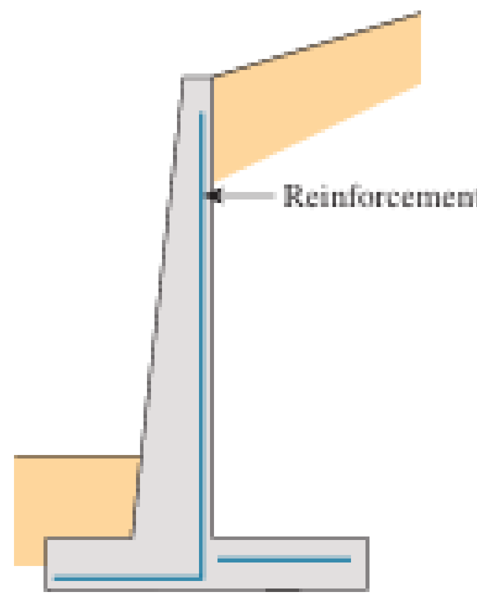
In many cases, a small amount of steel may be used for the construction of gravity walls, thereby minimizing the size of wall sections



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.**



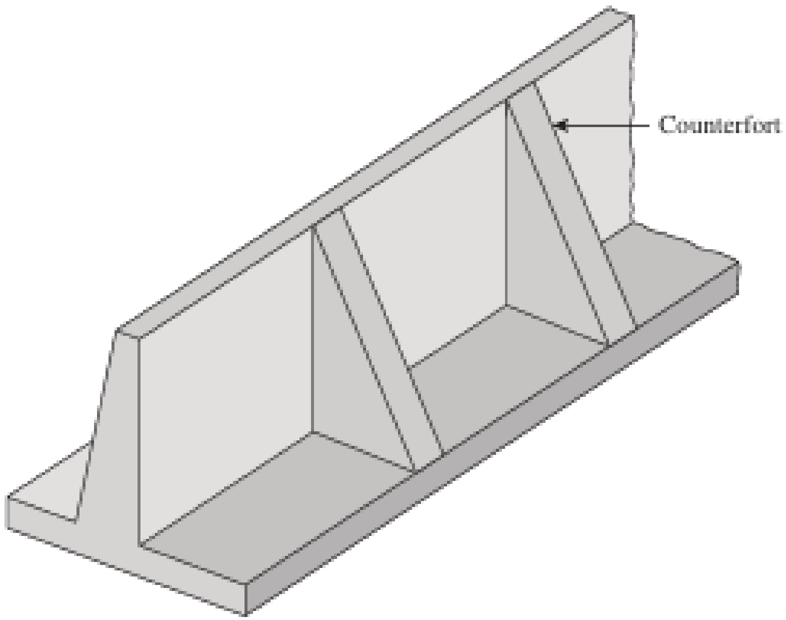


CANTILEVER WALLS

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.

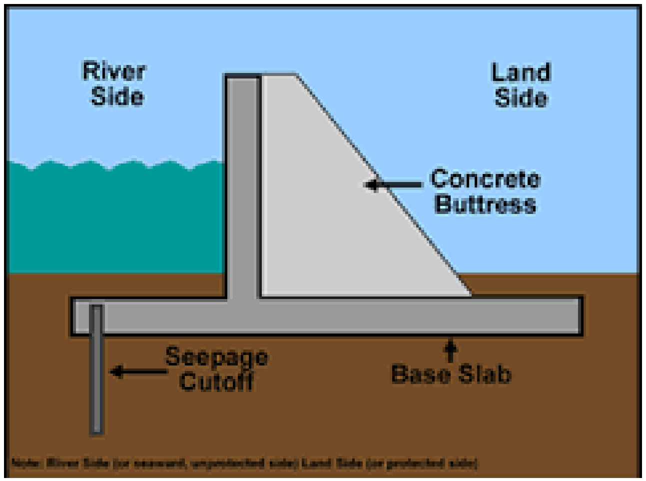
COUNTERFORT RETAINING WALLS

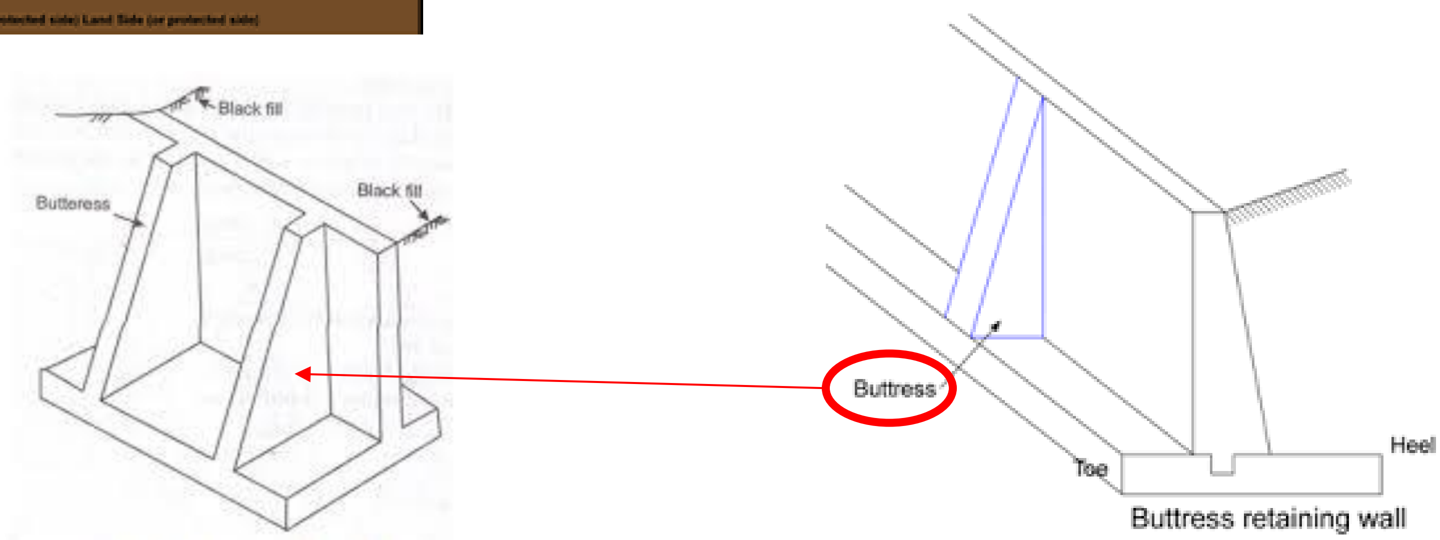
- Counterfort retaining walls are similar to cantilever walls.
- At regular intervals, however, they have thin vertical concrete slabs known as counterforts that tie the wall and the base slab together.
- The purpose of the counterforts is to reduce the shear and the bending.
- The counterfort is behind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

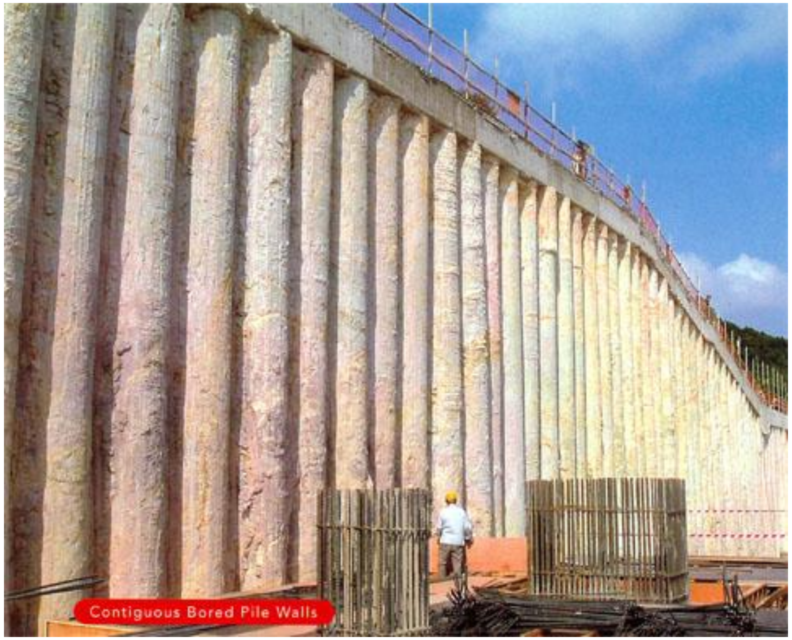




SHEET PILE WALLS

- **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.

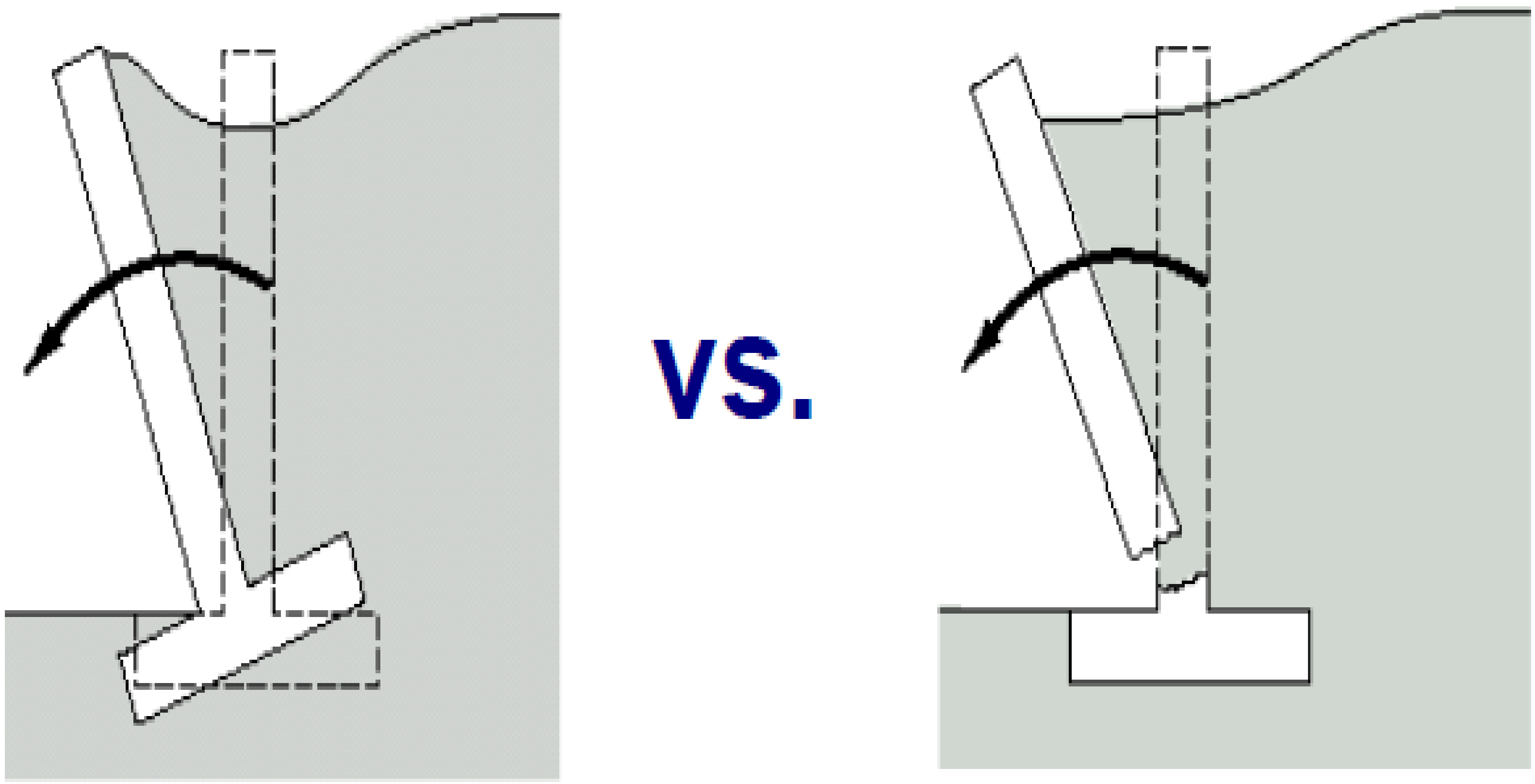
I. With the lateral earth pressure known, the structure as a whole is checked for stability. The structure is examined for possible *overturning*, *sliding*, *bearing capacity*, *and overall* (*deep-seated*) failures.

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

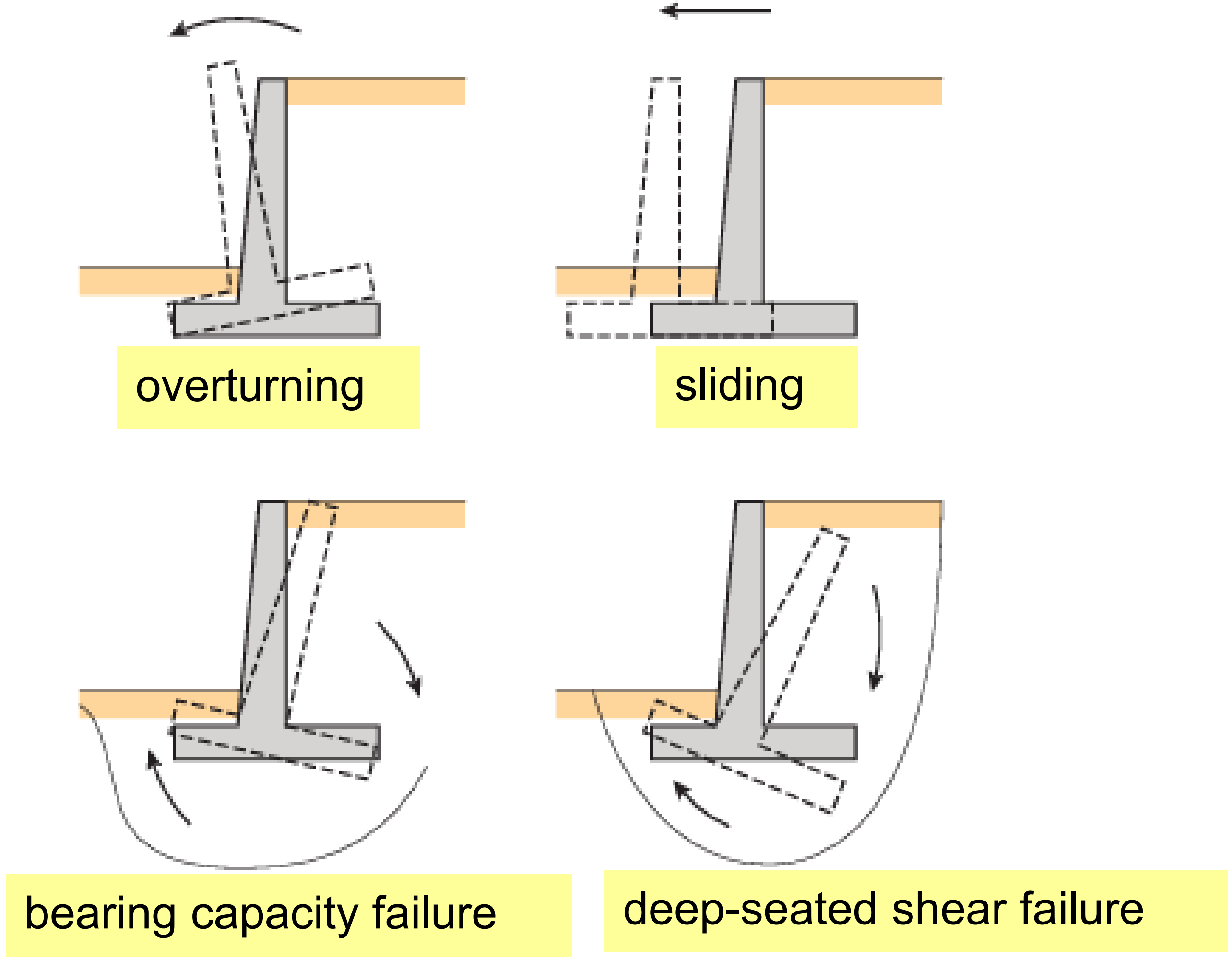
- In this course we will consider the procedures for determining the stability of the retaining wall. Checks for strength can be found in any textbook on reinforced concrete.

EXTERNAL AND INTERNAL STABILITY

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



EXTERNAL STABILITY

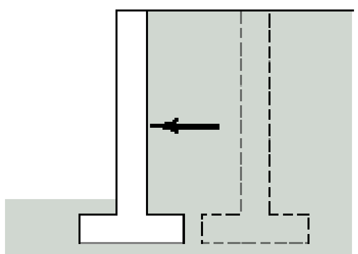


Failure of retaining wall

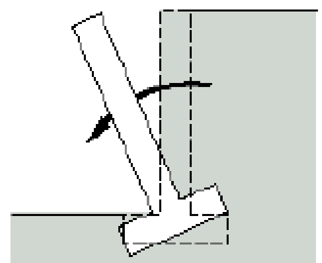
EXTERNAL STABILITY

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

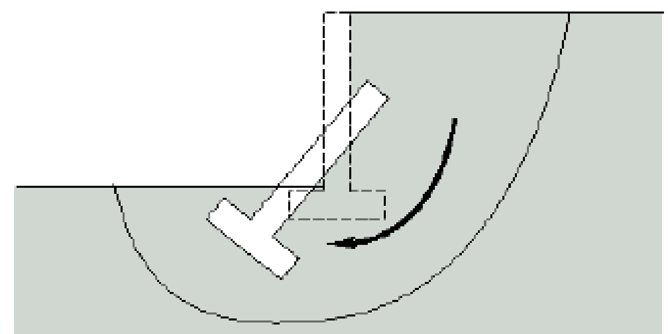
- **It may *overturn* about its toe.**
- **It may *slide* along its base.**
- **It may fail due to the loss of *bearing capacity* of the soil supporting the base.**
- **It may undergo deep-seated shear failure.**
- **It may go through excessive settlement.**



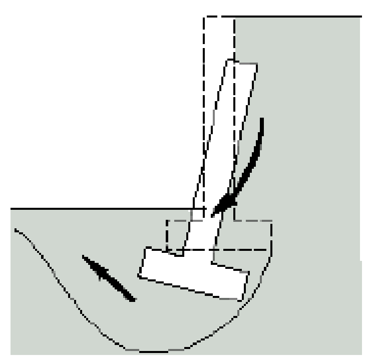
Sliding Failure



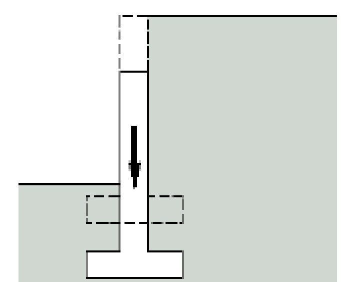
Overturning Failure



Deep Seated Shear Failure



Bearing Capacity



Excessive Settlement

EXTERNAL STABILITY

Failure by sliding

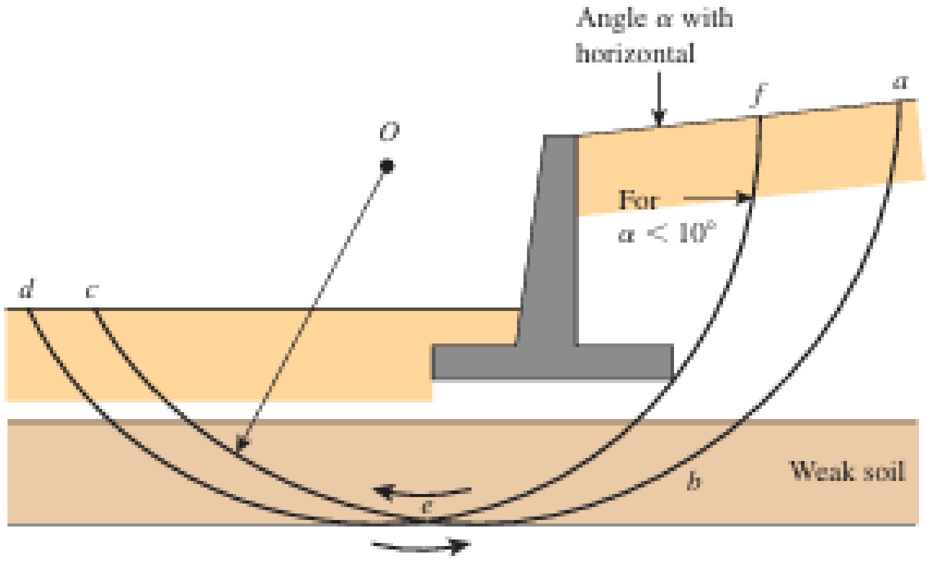
Failure by settlement

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.



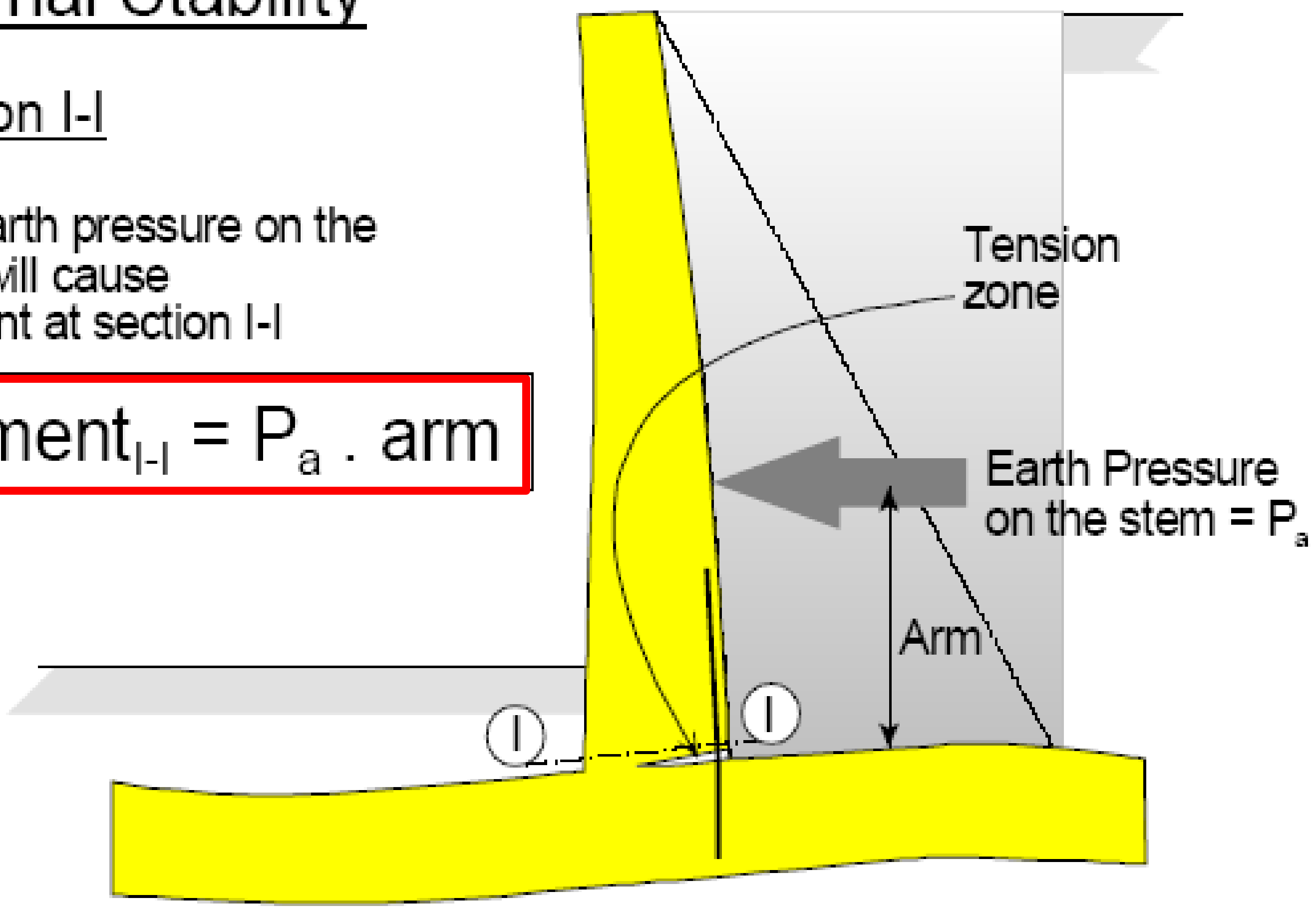
INTERNAL STABILITY

Internal Stability

Section I-I

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

$$\text{Moment}_{I\text{-}I} = P_a \,.\, \text{arm}$$



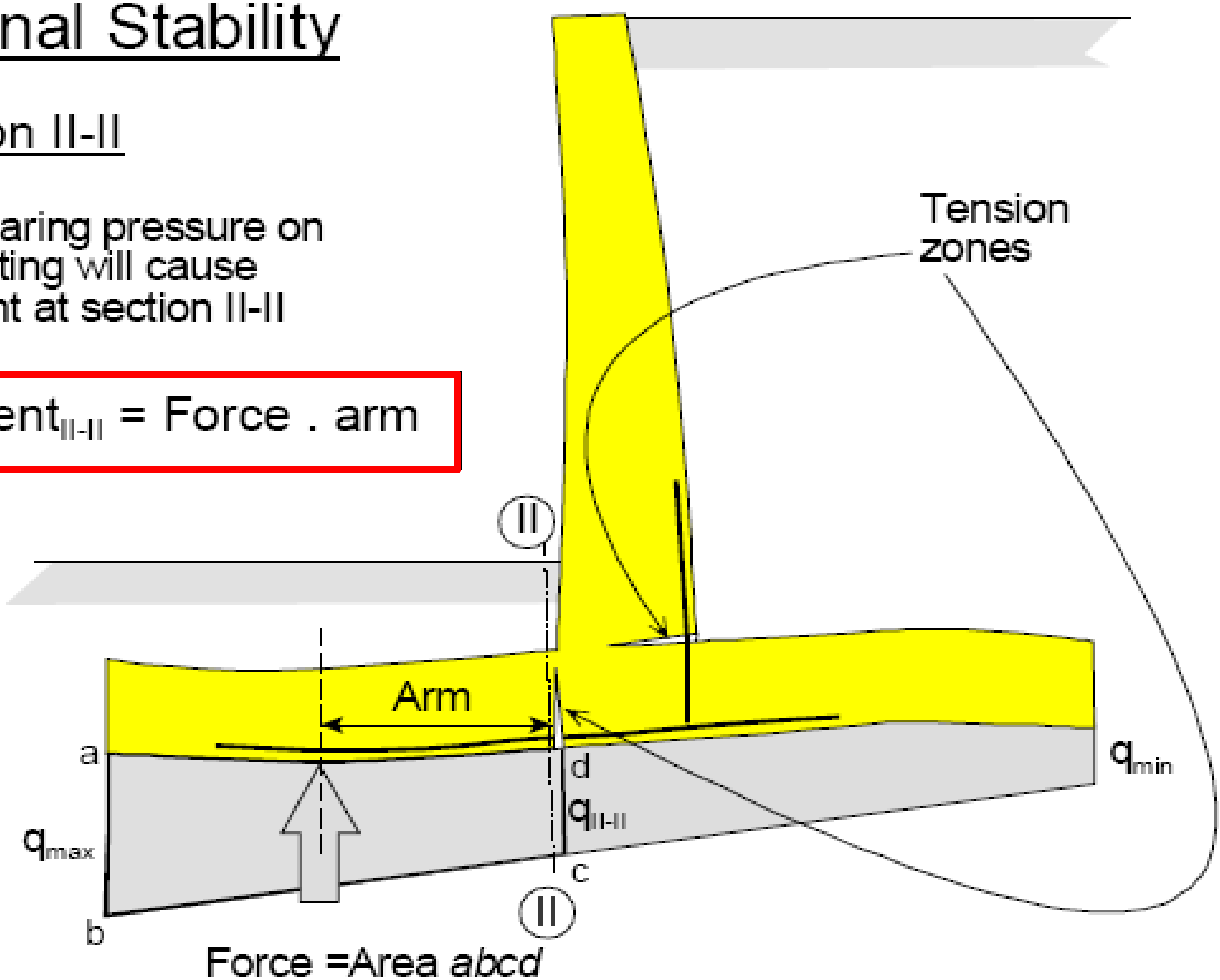
INTERNAL STABILITY

Internal Stability

Section II-II

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

$$Moment_{II-II} = Force \;.\; arm$$



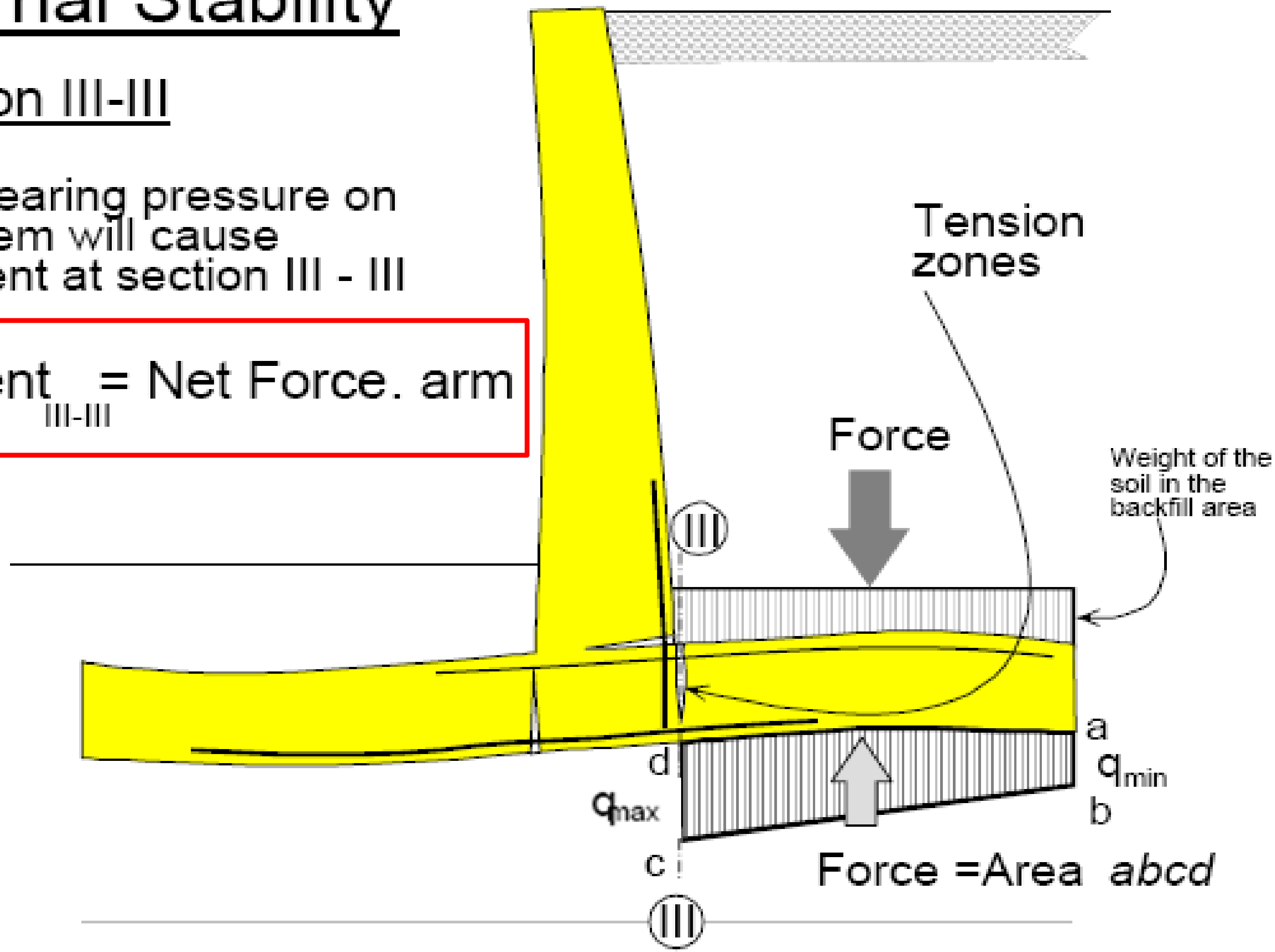
INTERNAL STABILITY

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

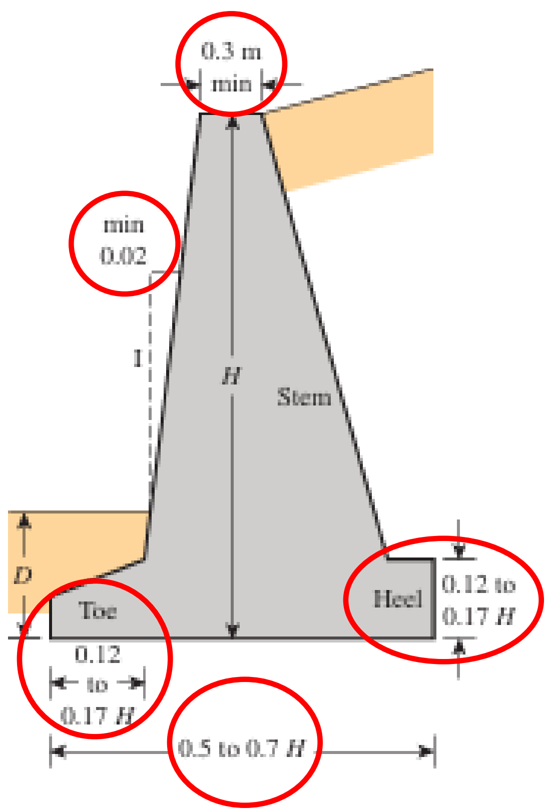
Proportioning Retaining Walls

Gravity and Cantilever Walls

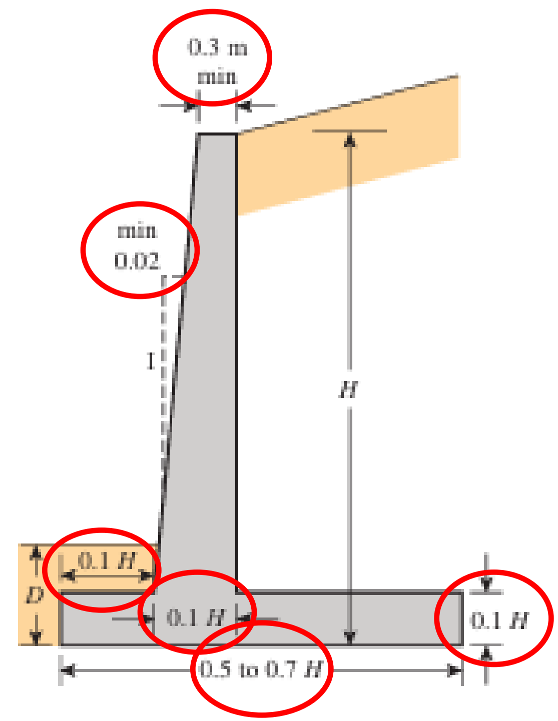
- In designing retaining walls, an engineer must assume some of their dimensions. Called ***proportioning***, such assumptions allow the engineer to check trial sections of the walls for stability.
- If the stability checks yield undesirable results, the sections can be changed and rechecked.
- Figure 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

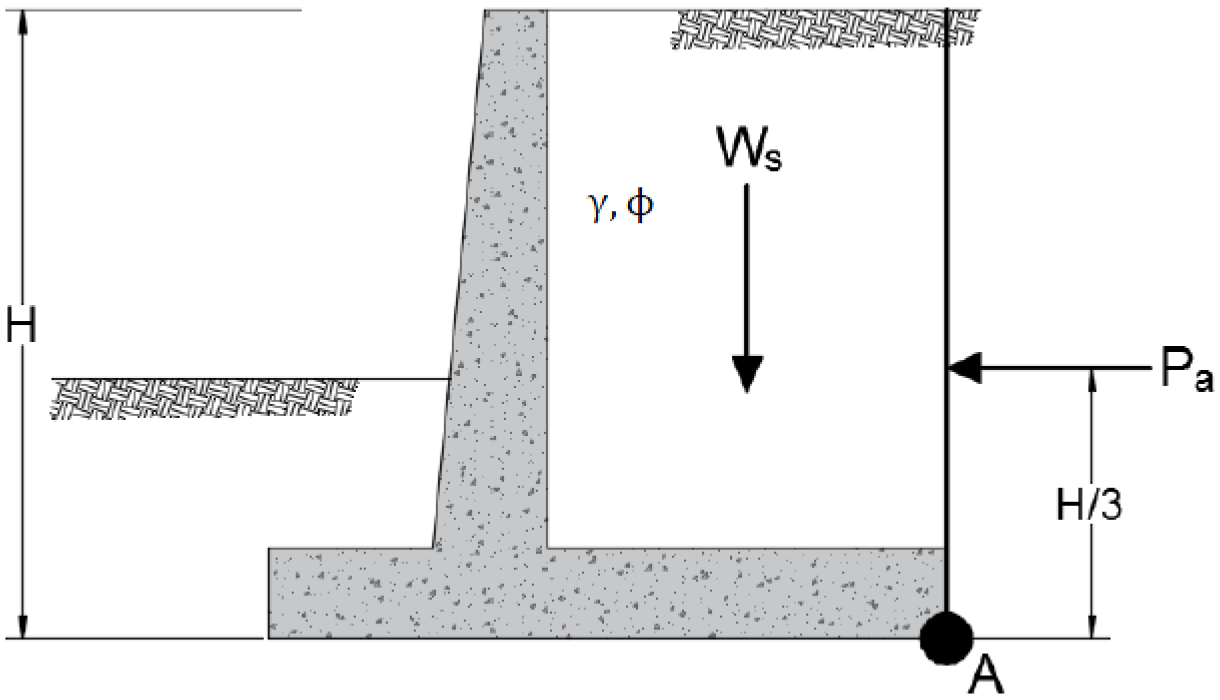
- Rankine theory is modified to be suitable for designing a retaining walls.
- This modification is drawing a vertical line from the lowest-right corner till intersection with the line of backfill, and then considering the force of soil acting on this vertical line.
- The soil between the wall and vertical 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

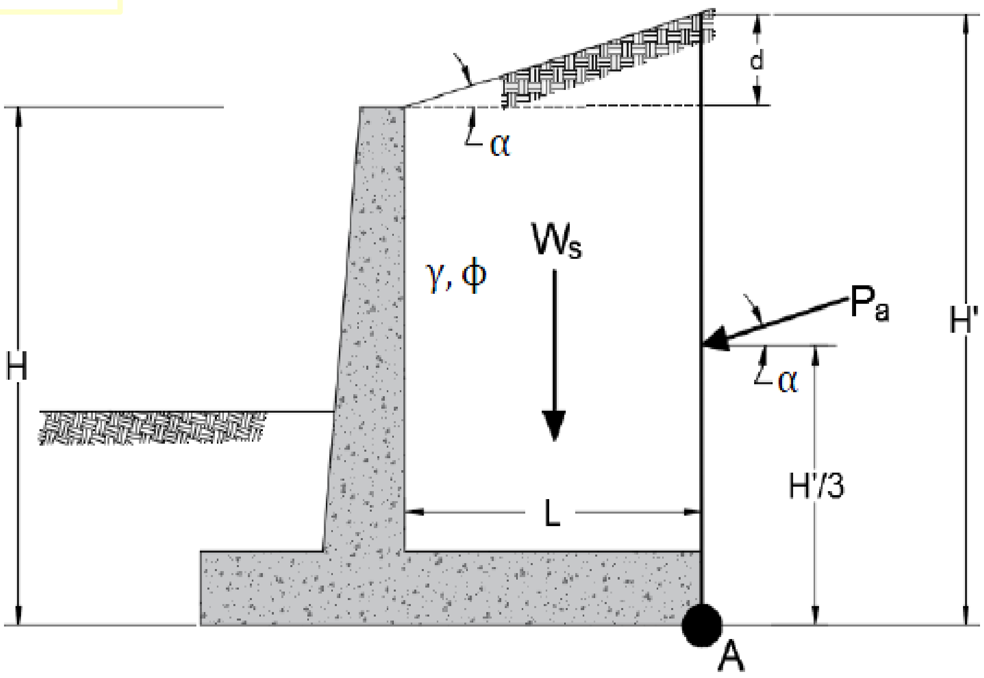
The wall is vertical and the backfill is inclined with horizontal by angle (α)

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

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

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

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



Design of Retaining Walls

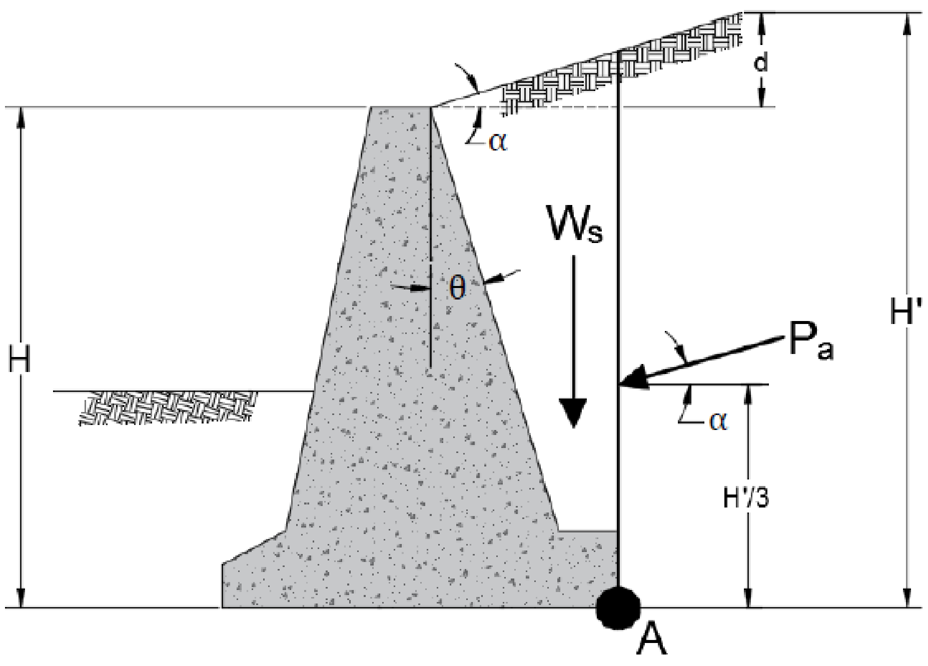
The wall is inclined by angle (θ)with vertical and the backfill is inclined with horizontal by angle (α):

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

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

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

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



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.

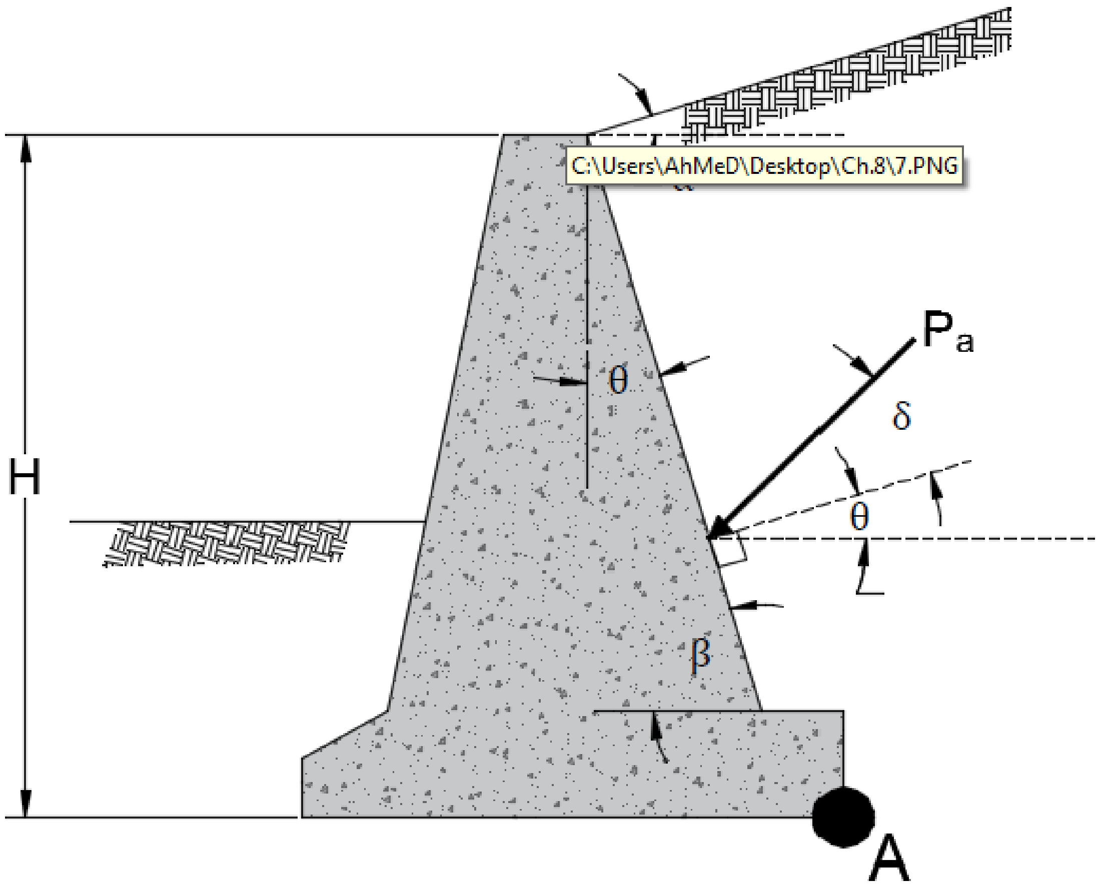
Design of Retaining Walls

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

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

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

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

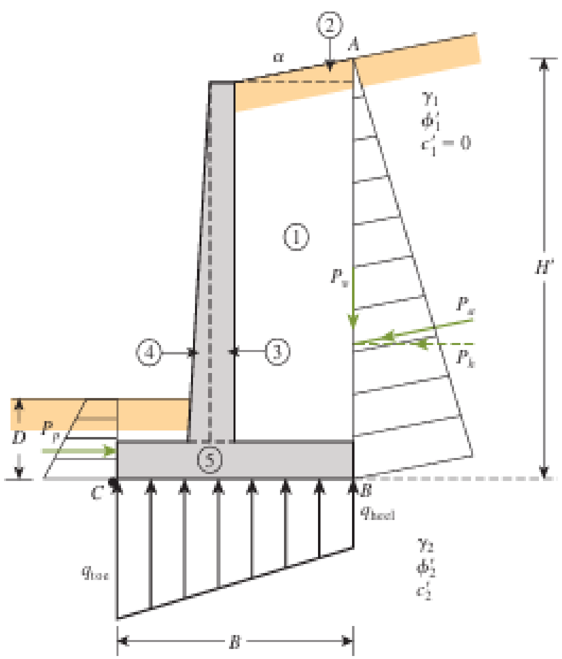
$$\text{FS}_{(\text{overturning})} = \frac{\Sigma M_R}{\Sigma M_o}$$

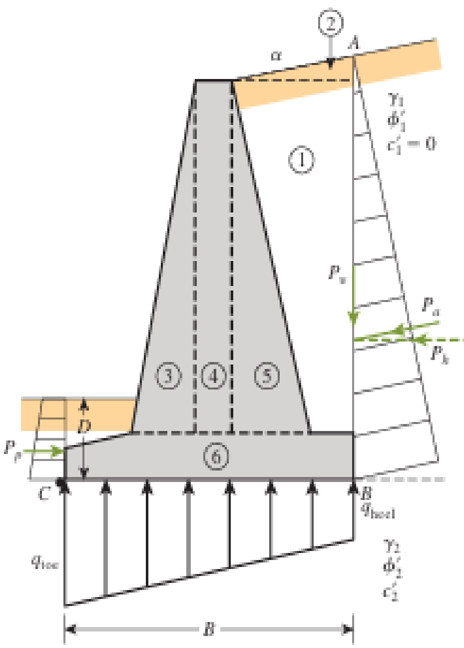
where

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

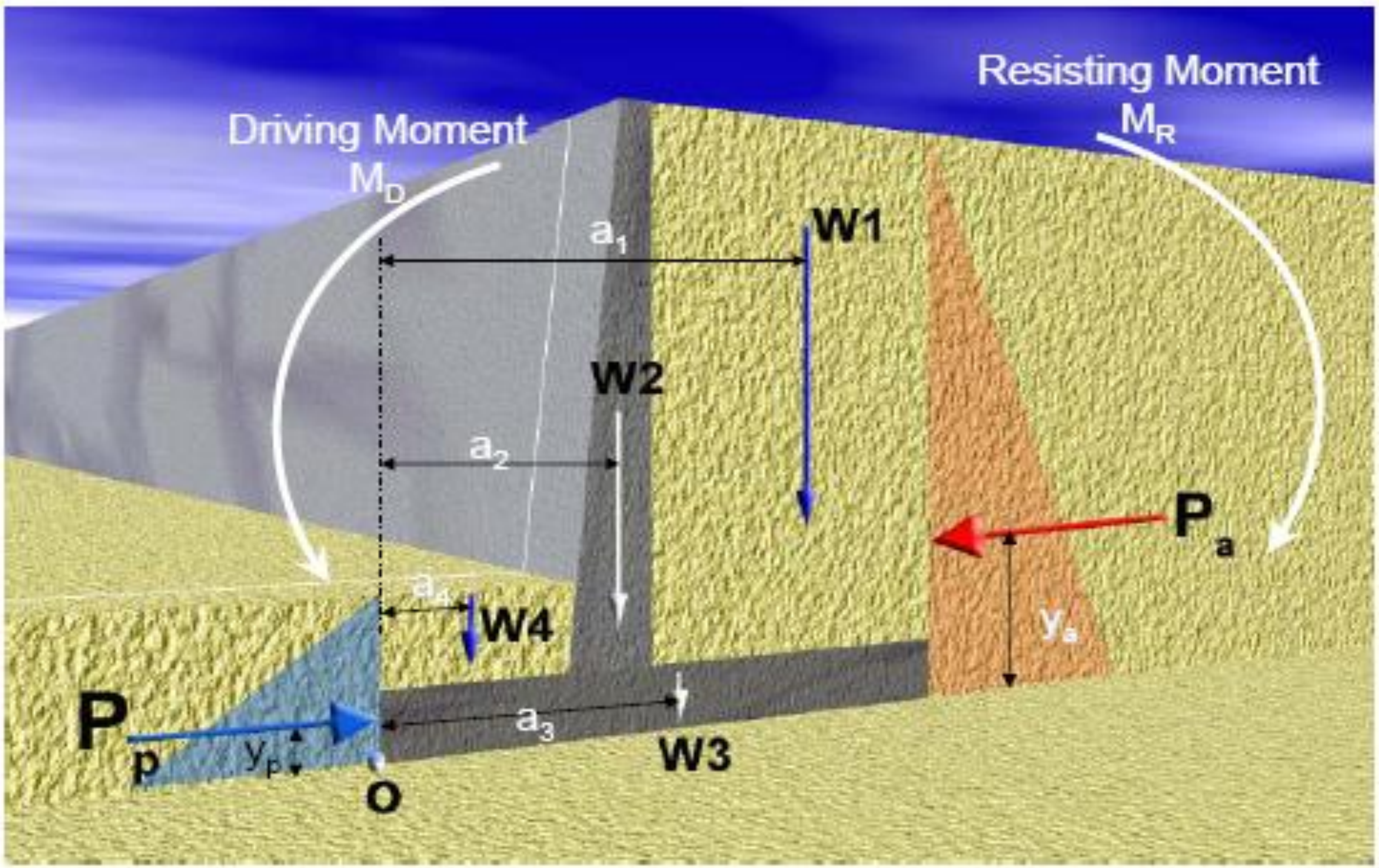
$$\Sigma M_o = P_h\left(\frac{H'}{3}\right)$$

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 17.1 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_l = 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 17.1

EXAMPLE 17.1

OVERTURNING

The cross section of a cantilever retaining wall is shown in Figure 17.12. Calculate the factors of safety with respect to overturning, sliding, and bearing capacity.

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 16.1.) 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

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

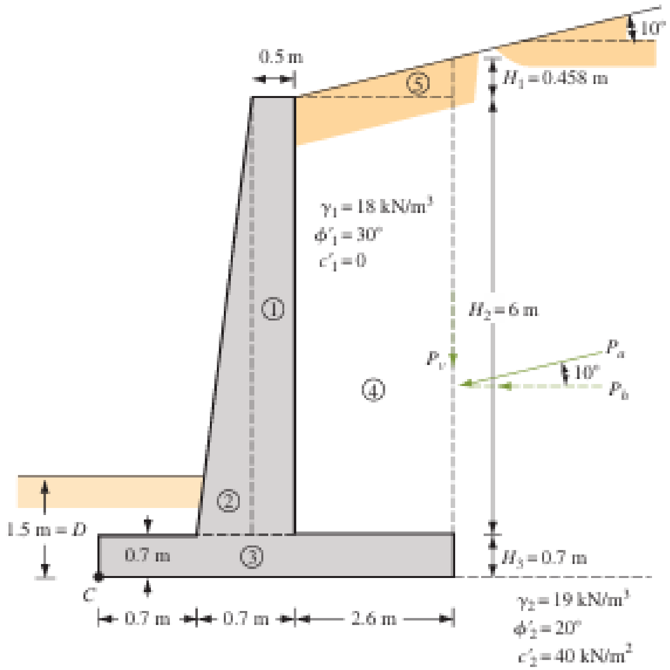


FIGURE 17.12 Calculation of stability of a retaining wall

EXAMPLE 17.1

Factor of Safety Against Overturning

The following table can now be prepared for determining the resisting moment:

Section no.[a]	Area (m^2)	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 17.12.
Note: $\gamma_{concrete} = 23.58\ kN/m^3$

The overturning moment

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

and

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

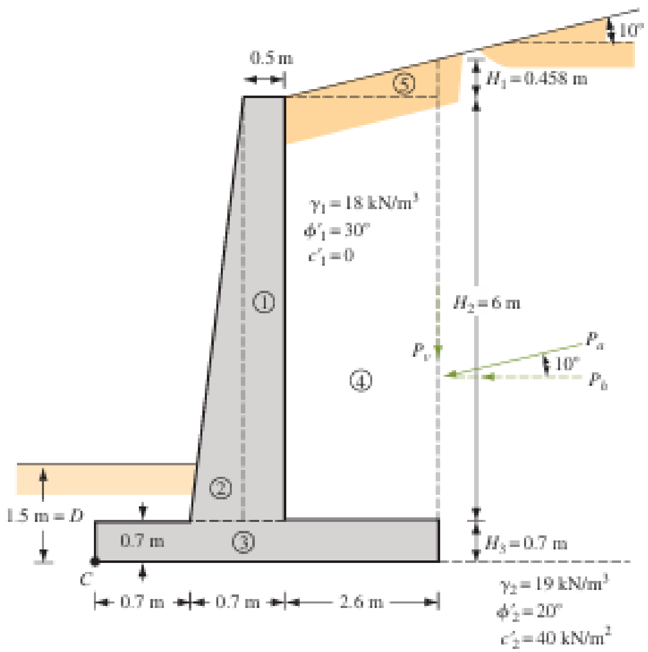


FIGURE 17.12 Calculation of stability of a retaining wall

EXAMPLE 17.2

EXAMPLE 17.2

A gravity retaining wall is shown in Figure 17.13. Use $\delta' = 2/3\phi'_1$ and Coulomb's active earth pressure theory. Determine:

a. The factor of safety against overturning

OVERTURNING

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 16.6.) 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

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

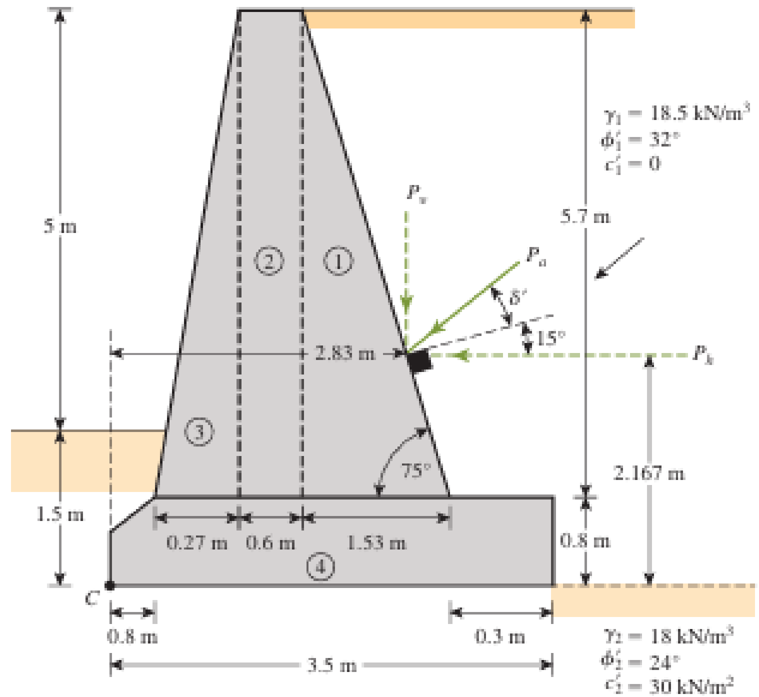


FIGURE 17.13 Gravity retaining wall (not to scale)

EXAMPLE 17.2

Part a: Factor of Safety Against Overturning

From Figure 17.13, we can prepare the following table:

Area no.	Area (m^2)	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

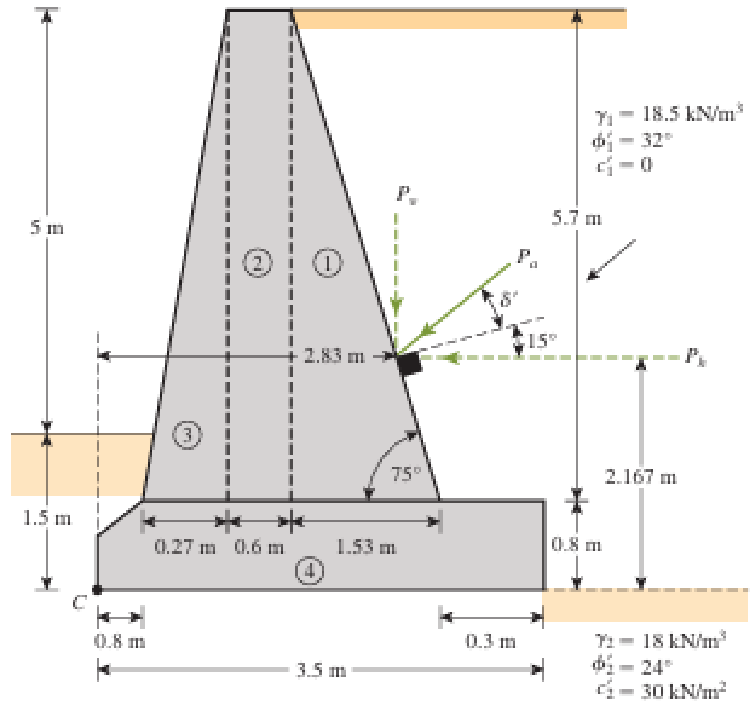
*$\gamma_{concrete} = 23.58$ kN/m^3

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

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

Hence,

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

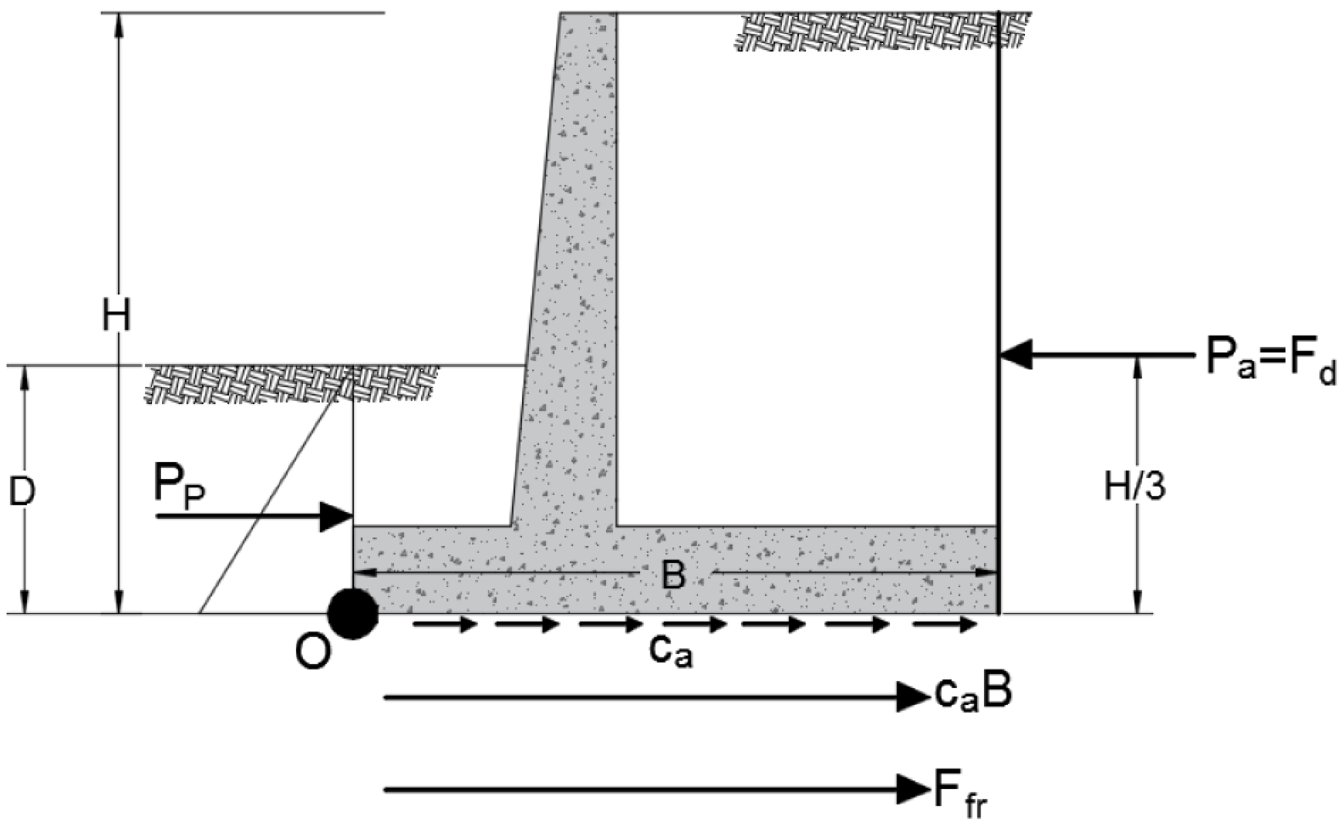


IGURE 17.13 Gravity retaining wall (not to scale)

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

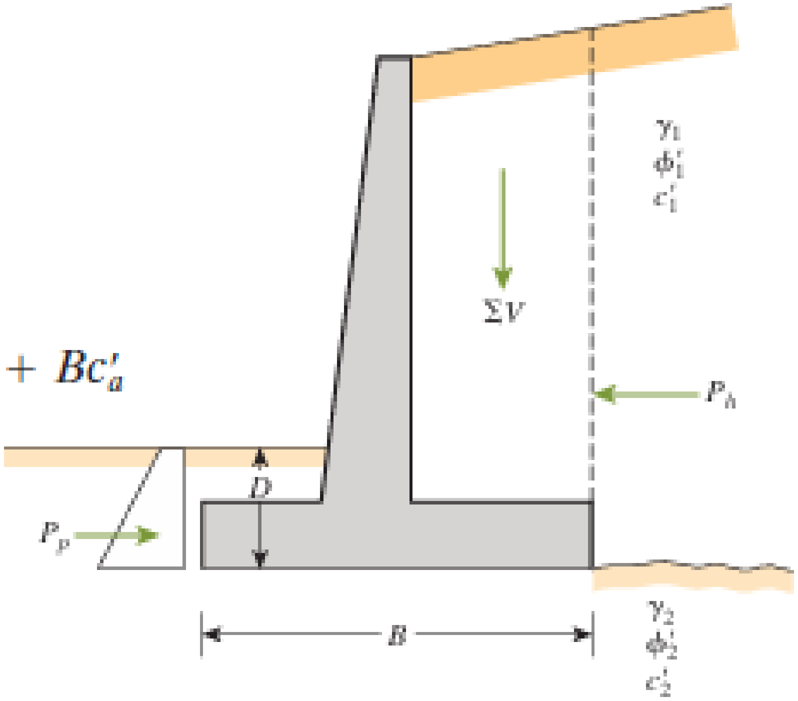
$$\mathrm{FS}_{(\text{sliding})} = \frac{\sum F_{R'}}{\sum F_d}$$

Shear strength along the base

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

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

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



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

$$\sum F_d = P_a \cos \alpha$$

$$\mathrm{FS}_{(\text{sliding})} = \frac{(\sum V) \tan \delta' + Bc'_a + P_p}{P_a \cos \alpha}$$

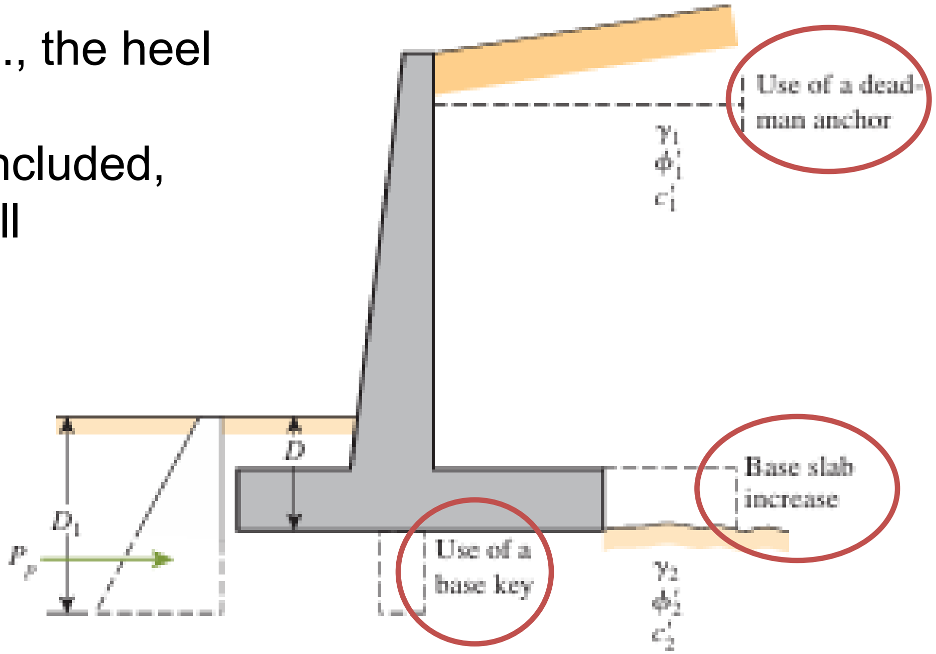
STABILITY FOR SLIDING ALONG THE BASE

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

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

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



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 (α = 0) so that

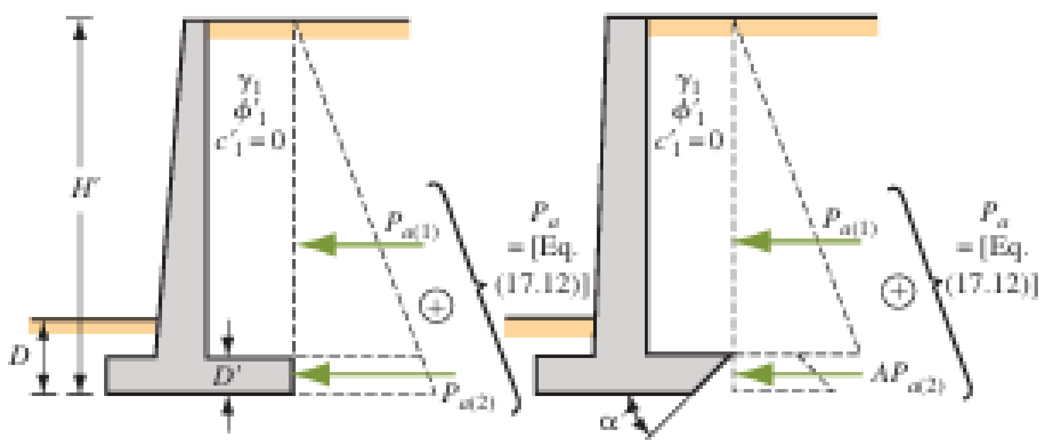
$P_a \cos \alpha = P_h = P_a$

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,

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

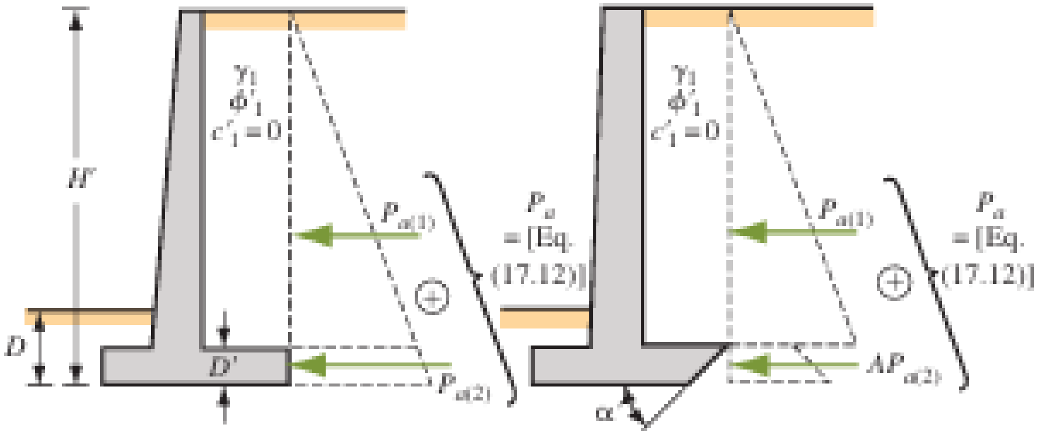
STABILITY FOR SLIDING ALONG THE BASE

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

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

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

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



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

TABLE 17.2 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 17.1

EXAMPLE 17.1

The cross section of a cantilever retaining wall is shown in Figure 17.12. 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 16.1.) 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

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

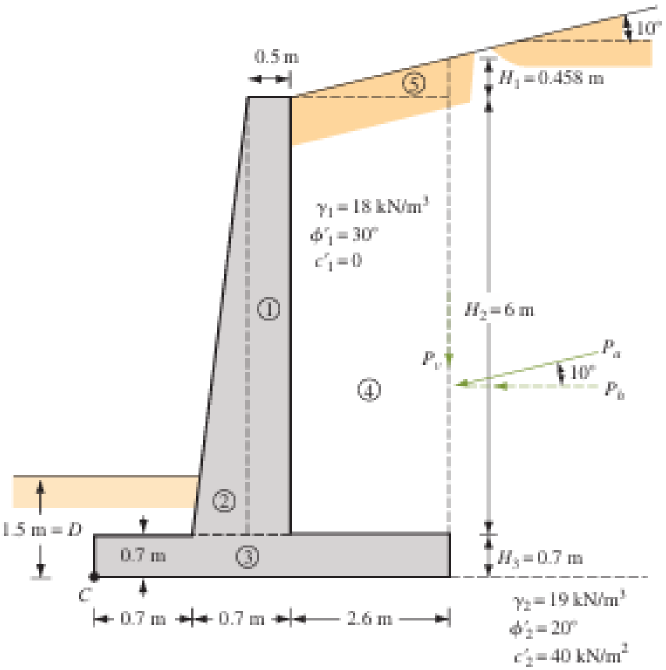


FIGURE 17.12 Calculation of stability of a retaining wall

EXAMPLE 17.1

Section no.[a]	Area (m^2)	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 17.12.

Note: $\gamma_{concrete} = 23.58\ kN/m^3$

EXAMPLE 17.1

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

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

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

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

and

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

So

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

Hence,

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

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

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 17.2

EXAMPLE 17.2

A gravity retaining wall is shown in Figure 17.13. Use $\delta' = 2/3\phi_1'$ and Coulomb's active earth pressure theory. Determine:

SLIDING

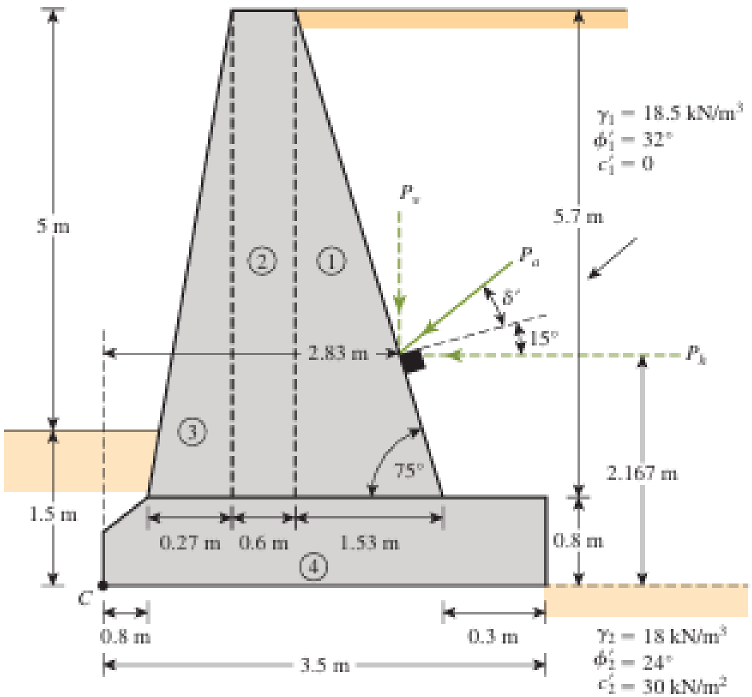


FIGURE 17.13 Gravity retaining wall (not to scale)

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 16.6.) 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

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

EXAMPLE 17.2

From Figure 17.13, we can prepare the following table:

Area no.	Area (m^2)	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/m^3

EXAMPLE 17.2

Part b: Factor of Safety Against Sliding

We have

$$FS_{(sliding)} = \frac{(\Sigma V)\tan\left(\frac{2}{3}\phi'_2\right) + \frac{2}{3}c'_2 B + P_p}{P_a}$$

$$P_p = \tfrac{1}{2}K_p \gamma_2 D^2 + 2c'_2\sqrt{K_p}D$$

and

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

Hence,

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

So

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

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

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 $\bar{P}_a \cos\alpha$.

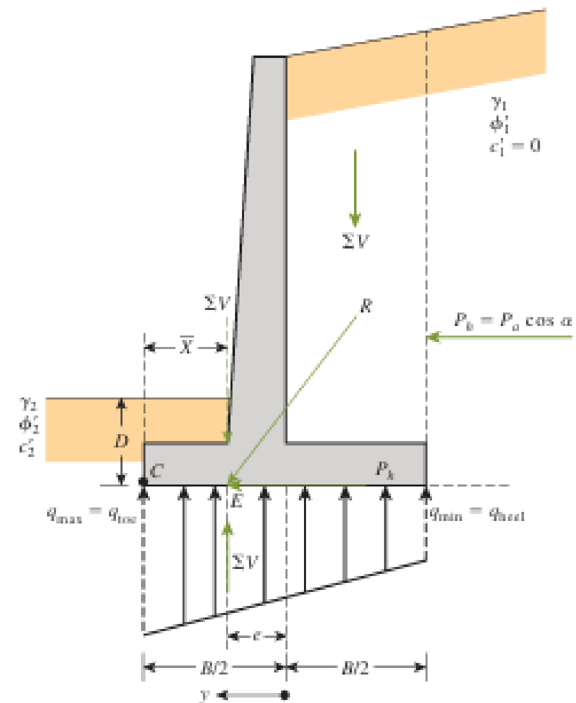
The net moment of these forces about point *C* is

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

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

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

$$q = \gamma_2 D$$

$$B' = B - 2e$$

Inclination factors

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

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

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

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 $\phi > 0.0$**

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

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

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

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

EXAMPLE 17.1

EXAMPLE 17.1

The cross section of a cantilever retaining wall is shown in Figure 17.12. Calculate the factors of safety with respect to **Bearing Capacity Failure**

SOLUTION

From the figure,

$$H' = H_1 + H_2 + H_3 = 2.6 \tan 10^\circ + 6 + 0.7$$

$$= 0.458 + 6 + 0.7 = 7.158 \text{ m}$$

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 16.1.) 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

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

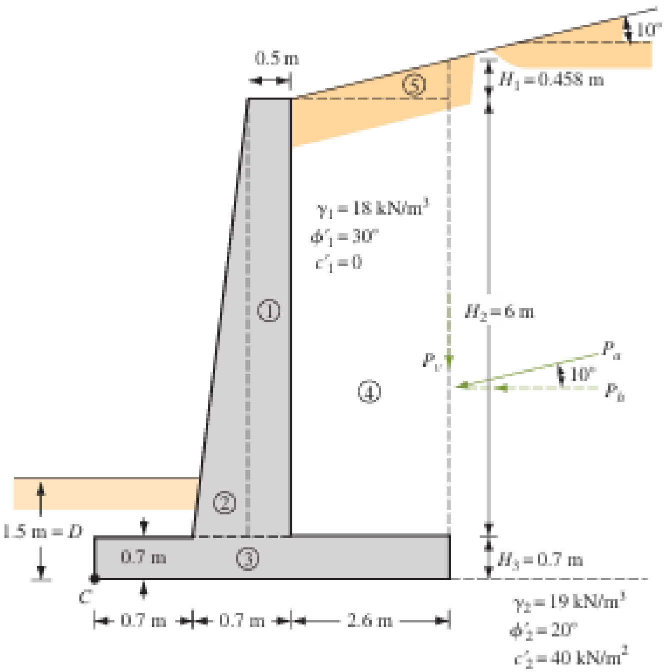


FIGURE 17.12 Calculation of stability of a retaining wall

EXAMPLE 17.1

Section no.[a]	Area (m^2)	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 17.12.

Note: $\gamma_{concrete} = 23.58\ kN/m^3$

EXAMPLE 17.1

Factor of Safety Against Bearing Capacity Failure

Combining Eqs. (17.15) and (17.16) yields

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

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

Again, from Eqs. (17.18) and (17.19),

$$q_{\text{heel}} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right) = \frac{470.42}{4}\left(1 - \frac{6 \times 0.406}{4}\right) = 45.98 \text{ kN/m}^2$$

$$q_{\text{toe}} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right) = \frac{470.42}{4}\left(1 + \frac{6 \times 0.406}{4}\right)$$

$$= 189.2 \text{ kN/m}^2$$

The ultimate bearing capacity of the soil can be determined from Eq. (17.20).

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

For $\phi_2' = 20°$ (see Table 6.2), $N_c = 14.83$, $N_q = 6.4$, and $N_\gamma = 5.39$. Also,

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

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

From Table 6.3,

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

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

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

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

EXAMPLE 17.1

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

and

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

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,

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

and

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

EXAMPLE 17.2

EXAMPLE 17.2

A gravity retaining wall is shown in Figure 17.13. 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

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

With $\alpha = 0°$, $\beta = 75°$, $\delta' = 2/3\phi'_1$, and $\phi'_1 = 32°$, $K_a = 0.4023$. (See Table 16.6.) So,

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

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

and

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

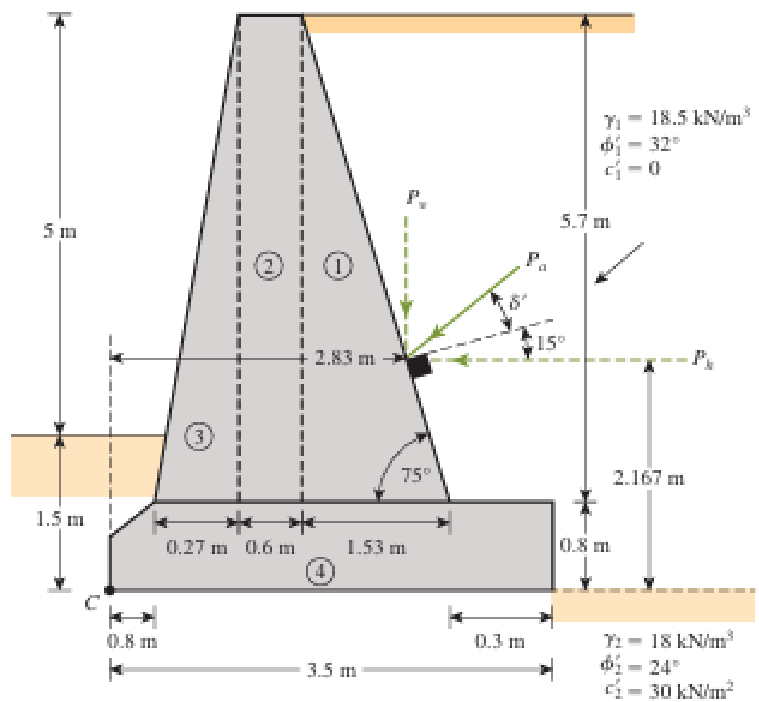


FIGURE 17.13 Gravity retaining wall (not to scale)

EXAMPLE 17.2

From Figure 17.13, we can prepare the following table:

Area no.	Area (m^2)	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

*$\gamma_{concrete} = 23.58\ kN/m^3$

Part c: Pressure on Soil at Toe and Heel

From Eqs. (17.15) and (17.16),

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

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

and

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

EXAMPLE 17.3

EXAMPLE 17.3

Refer to the gravity wall described in Example 17.2 and redo the problem using Rankine active pressure.

SOLUTION

The retaining wall is redrawn in Figure 17.14. From the figure, $H' = 5 + 1.5 = 6.5$ m.

$$K_a = \tan^2\left(45 - \frac{\phi'_1}{2}\right) = \tan^2\left(45 - \frac{32}{2}\right) = 0.307$$

$$P_a = \frac{1}{2}\gamma H'^2 K_a = \frac{1}{2}(18.5)(6.5)^2(0.307) = 119.98 \text{ kN/m} \approx 120 \text{ kN/m}$$

Section no.	Area (m²)	Weight per unit length (kN/m)	Moment arm from C (m)	Moment about C (kN · m/m)
*1	4.36	102.81	2.18	224.13
*2	3.42	80.64	1.37	110.48
*3	0.77	18.16	0.98	17.80
*4	2.8	66.02	1.75	115.54
5	(0.5)(1.53)(5.7) = 4.36	80.66	2.69	216.98
6	(5.7)(0.3) = 1.71	31.44	3.35	105.99
		$\Sigma V = 379.93$		$\Sigma M_R = 790.92$

*From Example 17.2

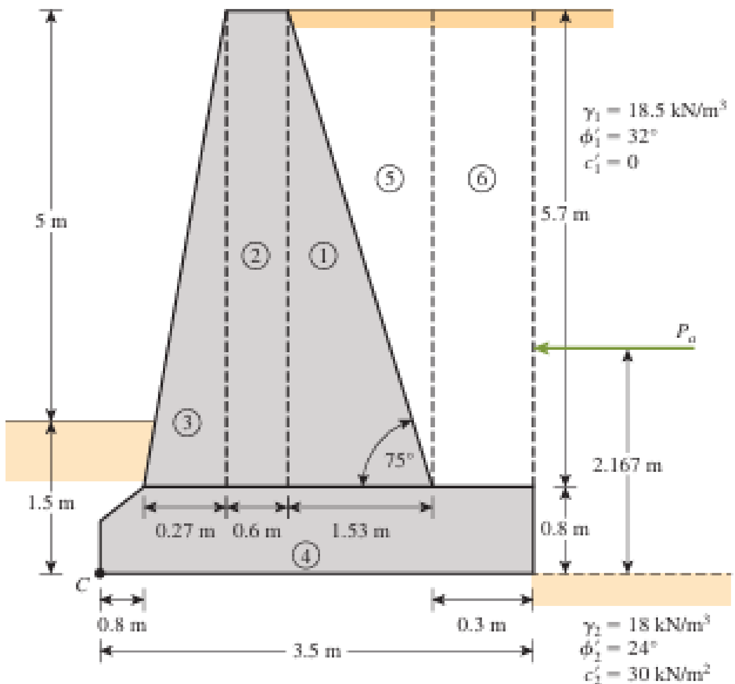


FIGURE 17.14

EXAMPLE 17.3

Part a: Factor of Safety Against Overturning

Overturning moment, $M_o = P_a\left(\frac{H'}{3}\right) = (120)(2.167) = 260.0 \text{ kN} \cdot \text{m/m}$.

$$FS_{(\text{overturning})} = \frac{\Sigma M_R}{\Sigma M_o} = \frac{790.92}{260.0} = \mathbf{3.04}$$

Part b: Factor of Safety Against Sliding

$$FS_{(\text{sliding})} = \frac{(\Sigma V)\tan\left(\frac{2}{3}\phi'_2\right) + \frac{2}{3}c'_2 + P_p}{P_h}$$

From Example 17.2, $P_p = 186.59$ kN/m. So,

$$FS_{(\text{sliding})} = \frac{(379.93)(\tan 16) + \left(\frac{2}{3}\right)(30)(3.5) + 186.59}{120} = \mathbf{3.05}$$

Part c: Pressure on Soil at Toe and Heel

$$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_o}{\Sigma V} = \frac{3.5}{2} - \frac{(790.92 - 260.0)}{379.93} = 1.75 - 1.397 = 0.353 \text{ m}$$

$$q_{\text{toe}} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right) = \frac{379.93}{3.5}\left(1 + \frac{6 \times 0.353}{3.5}\right) = \mathbf{174.24 \text{ kN/m}^2}$$

$$q_{\text{toe}} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right) = \frac{379.93}{3.5}\left(1 - \frac{6 \times 0.353}{3.5}\right) = \mathbf{42.86 \text{ kN/m}^2}$$

MID TERM EXAM

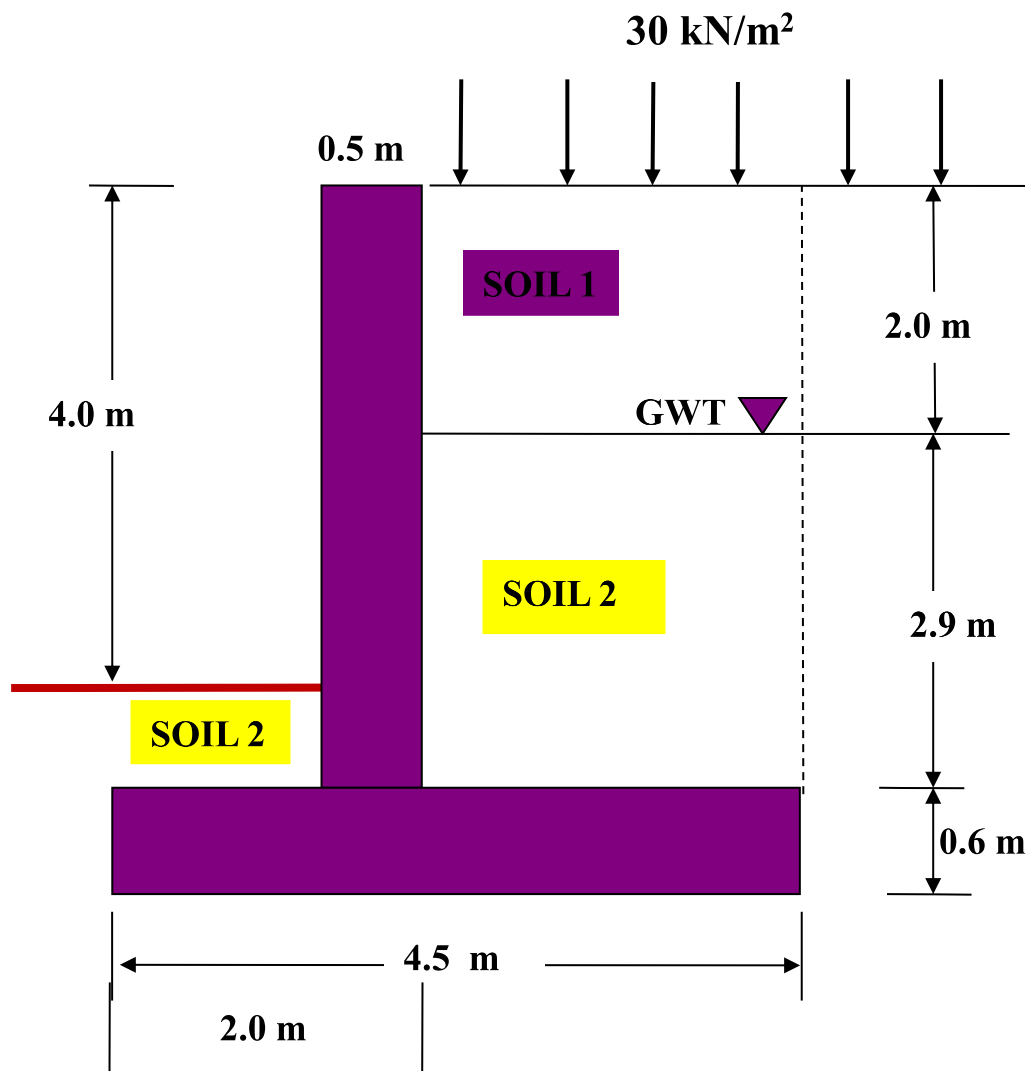
Figure below shows the cross-section of a reinforced concrete retaining structure. The retained soil behind the structure and the soil in front of it are cohesionless and has the following properties:

SOIL 1 : $\phi = 35^o$, $\gamma_d = 17$ kN/m^3,
SOIL 2 : $\phi = 30^o$, $\delta = 25^o$,
$\gamma_d = 18$ kN/m^3, $\gamma_{sat} = 20$ kN/m^3

The unit weight of concrete is 24 kN/m^3
determine

- Factor of safety against overturning
- Factor of safety against sliding
- Maximum base pressure should not exceed 150 kPa

MID TERM EXAM



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.

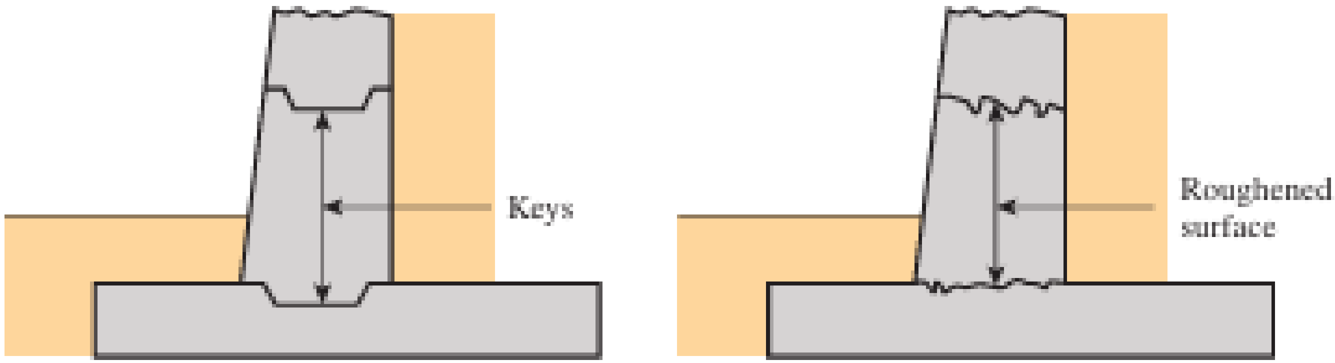
A factor of safety of 3 against bearing capacity failure may not ensure that settlement of the structure will be within the tolerable limit in all cases.

Thus, this situation needs further investigation

CONSTRUCTION JOINTS

Construction joints

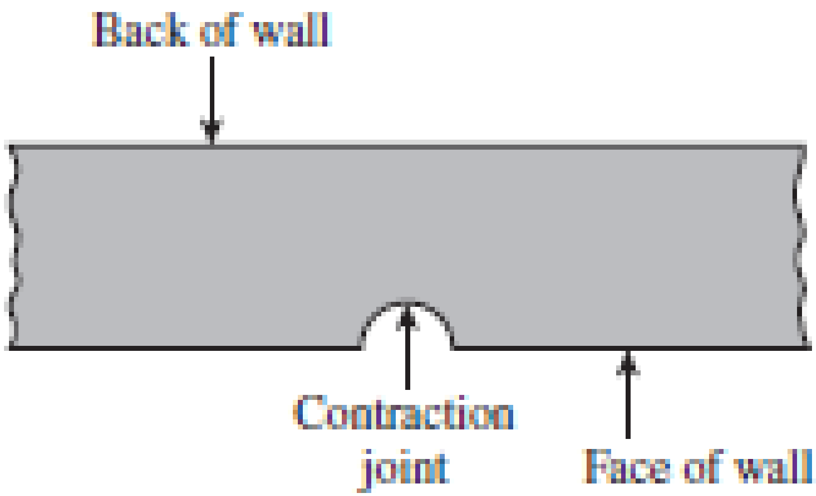
are vertical and horizontal joints that are placed between two successive pours of concrete. To increase the shear at the joints, keys may be used. If keys are not used, the surface of the first pour is cleaned and roughened before the next pour of concrete.



CONSTRUCTION JOINTS

Contraction joints

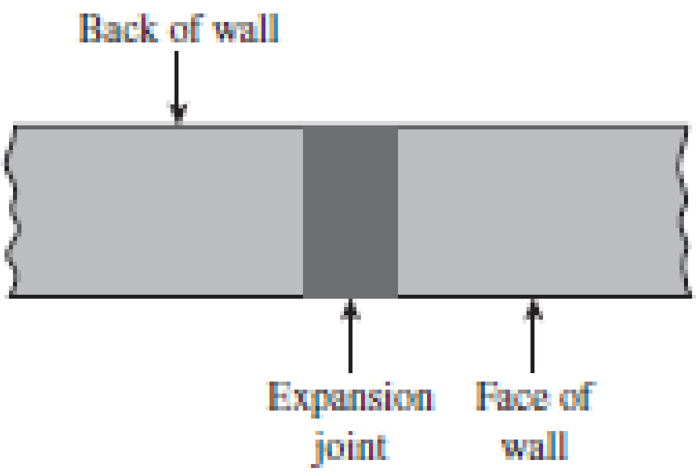
are vertical joints (grooves) placed in the face of a wall (from the top of the base slab to the top of the wall) that allow the concrete to shrink without noticeable harm. The grooves may be about 6 to 8 mm wide and 12 to 16 mm deep.



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.

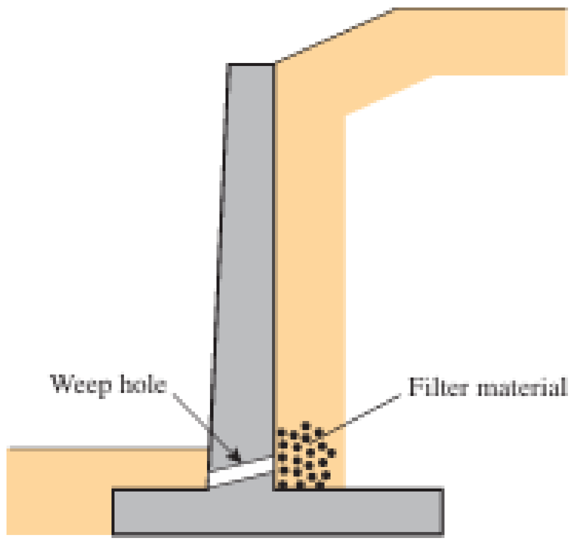


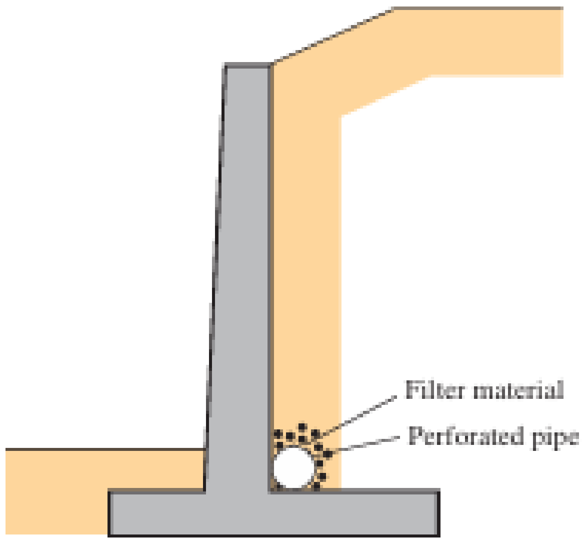
DRAINAGE FROM BACKFILL

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

EXAMPLE 17.4

Figure 17.17 shows the grain-size distribution of a backfill material. Using the conditions outlined in Section 17.8, determine the range of the grain-size distribution for the filter material.

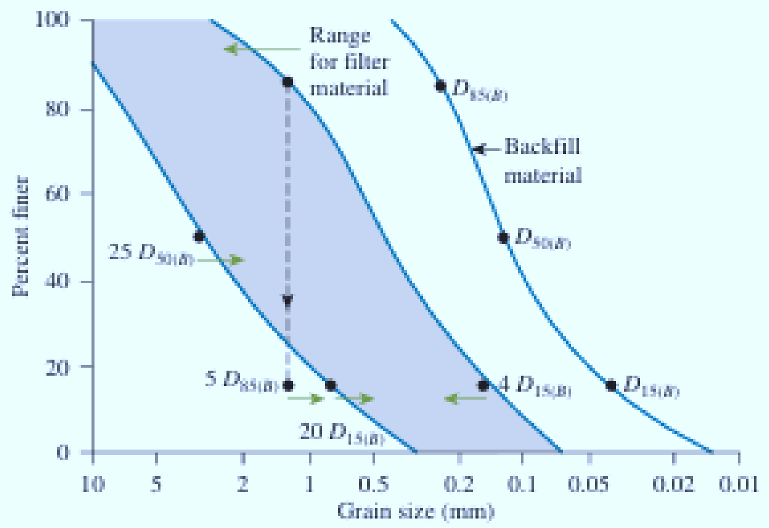


FIGURE 17.17 Determination of grain-size distribution of filter material

EXAMPLE 17.4

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 17.17. 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.

The end