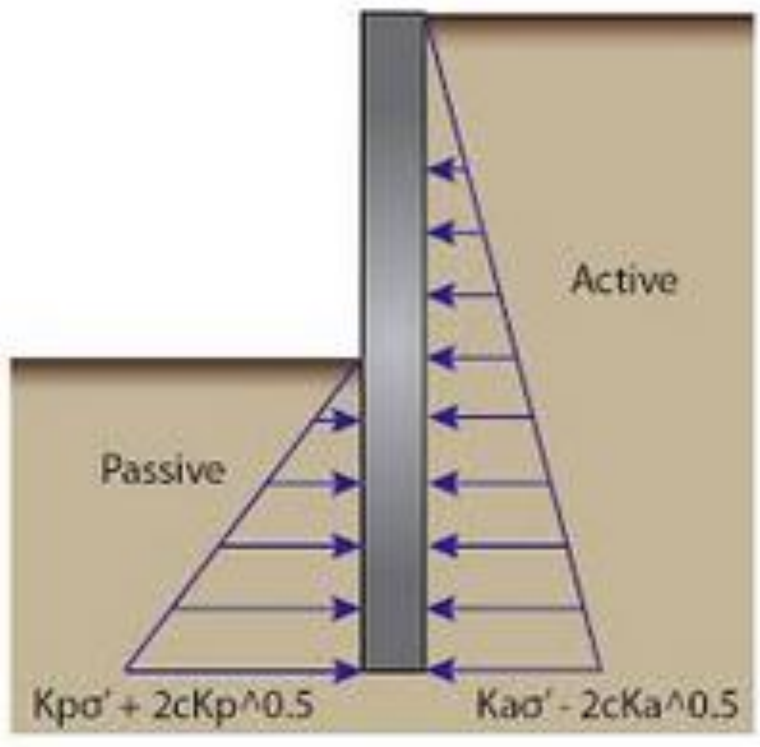
Lateral Earth Pressure

CHAPTER 16



Omitted parts:

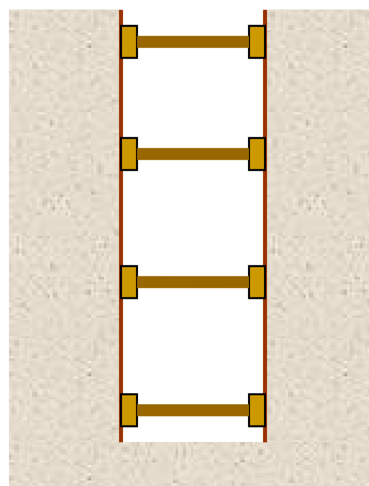
Sections 16.9, 16.10, 16.16 , 16.17

INTRODUCTION

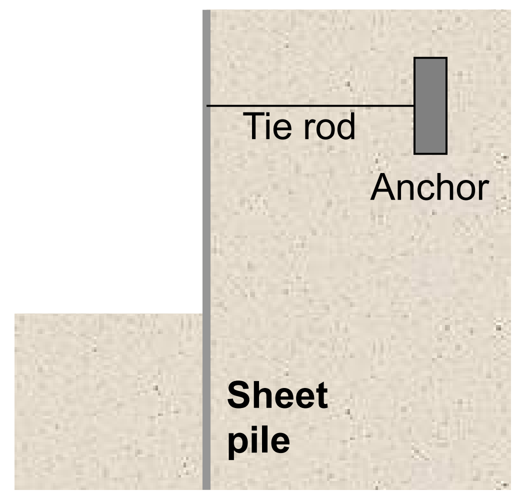
- Proper design and construction of many structures such as:
  - Retaining walls (basements walls, highways and railroads, platforms, landscaping, and erosion controls)
  - Braced excavations
  - Anchored bulkheads
  - Grain pressure on silo walls and bins

require a thorough knowledge of the lateral forces that act between the retaining structures and the soil masses being retained.

Cantilever retaining wall



Braced excavation



Anchored sheet pile

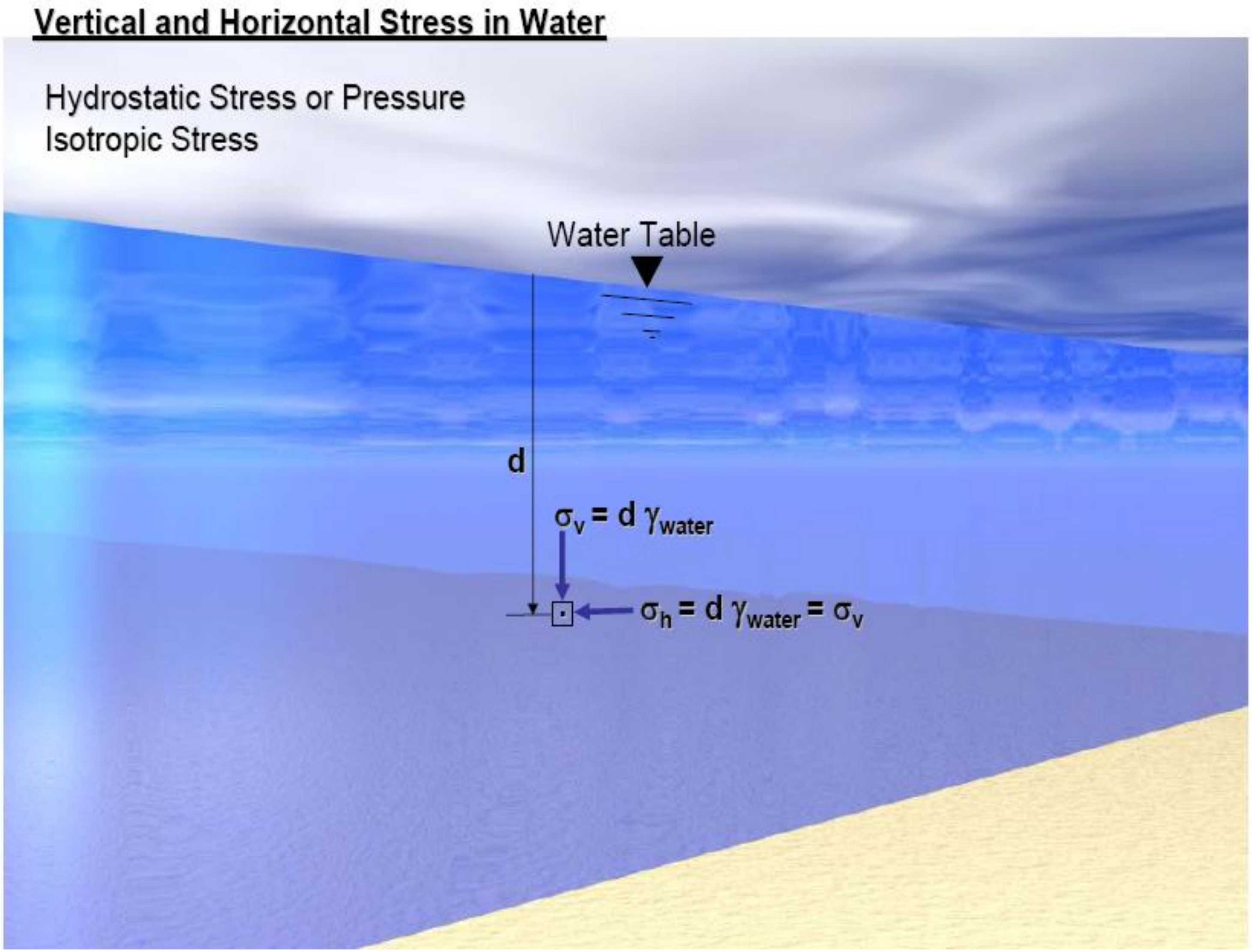
INTRODUCTION

- These lateral forces are caused by lateral earth pressure.
- We have to estimate the lateral soil pressures acting on these structures, to be able to design them.

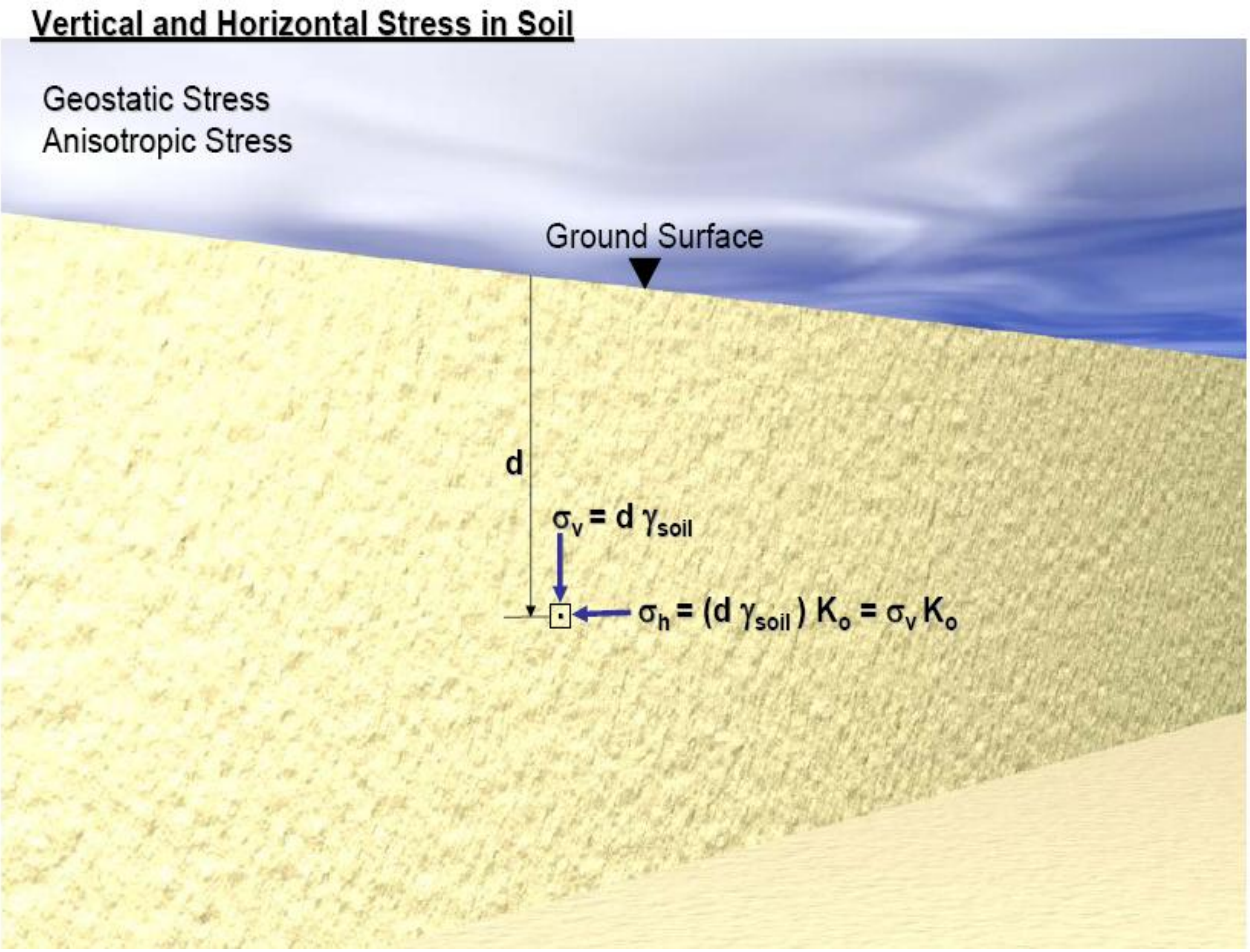
The magnitude and distribution of lateral earth pressure depends on many factors, such as:

- The shear strength parameters of the soil being retained,
- The inclination of the surface of the backfill,
- The height and inclination of the retaining wall at the wall–backfill interface,
- The nature of wall movement under lateral pressure,
- The adhesion and friction angle at the wall–backfill interface.

Vertical and Horizontal Stress in Water

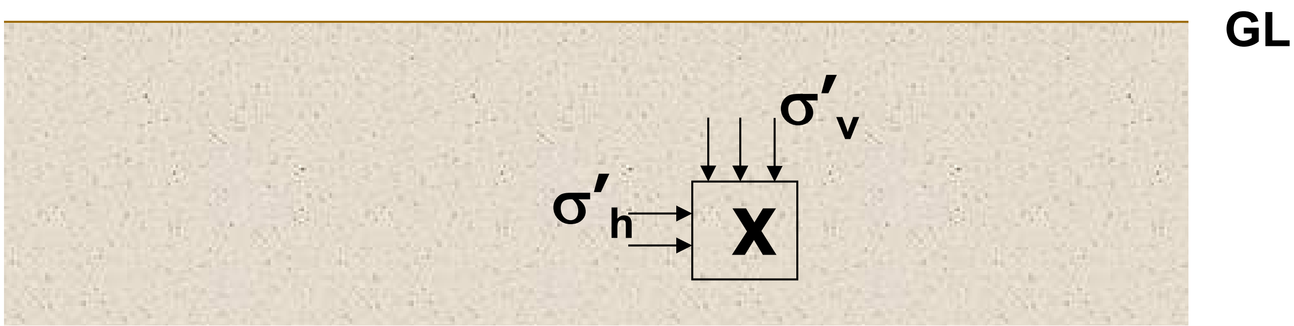


Vertical and Horizontal Stress in Soil



Coefficient of Lateral Earth Pressure

In a homogeneous natural soil deposit,



The ratio σ_h'/σ_v' is a constant known as coefficient of lateral earth pressure.

In other words, it is the ratio of the effective horizontal stress (σ_h') to the effective vertical stress (σ_v'); then

$$\boxed{K = \frac{\sigma_h'}{\sigma_v'}}$$

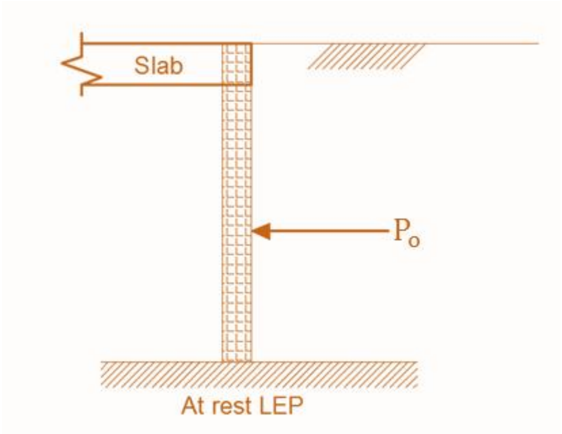
Or in terms of total stresses

$$\boxed{K = \frac{\sigma_h}{\sigma_v}}$$

Types of Lateral Earth Pressures

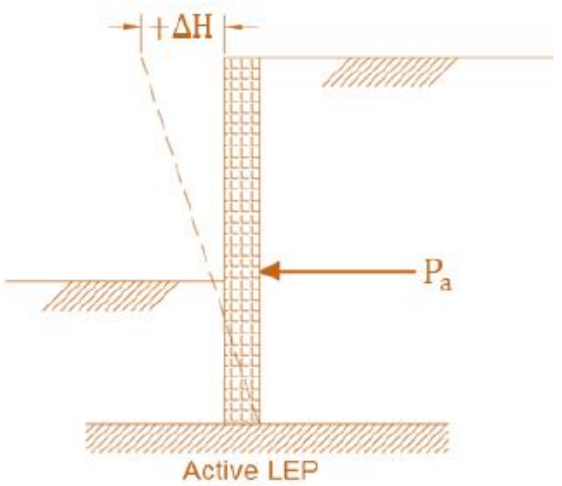
1. At Rest Lateral Earth Pressure:

The wall may be restrained from moving, for example; basement wall is restrained to move due to slab of the basement and the lateral earth force in this case can be termed as" P_o".



2. Active Lateral Earth Pressure:

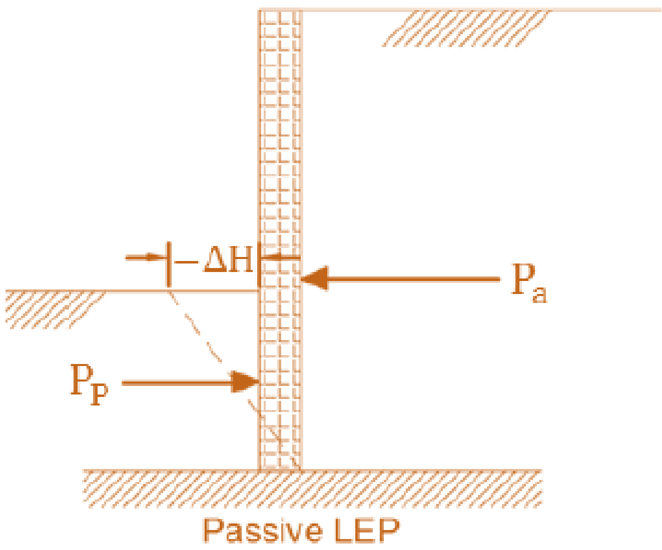
In case of the wall is free from its upper edge (retaining wall), the wall may move away from the soil that is retained with distance "+ΔH " (i.e. the soil pushes the wall away) this means the soil is active and the force of this pushing is called active force and termed by " P_a".



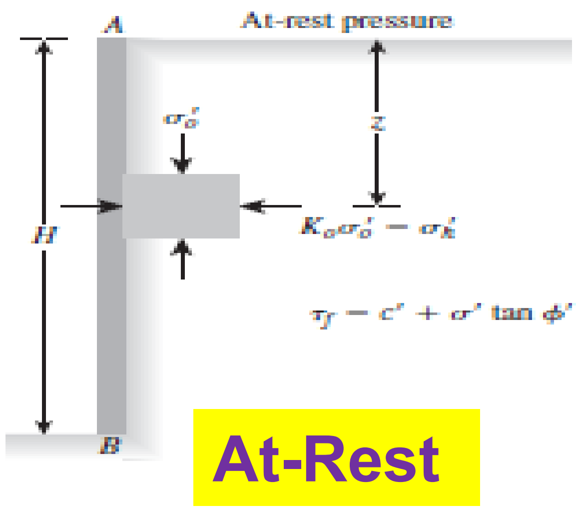
Types of Lateral Earth Pressures

3. Passive Lateral Earth Pressure:

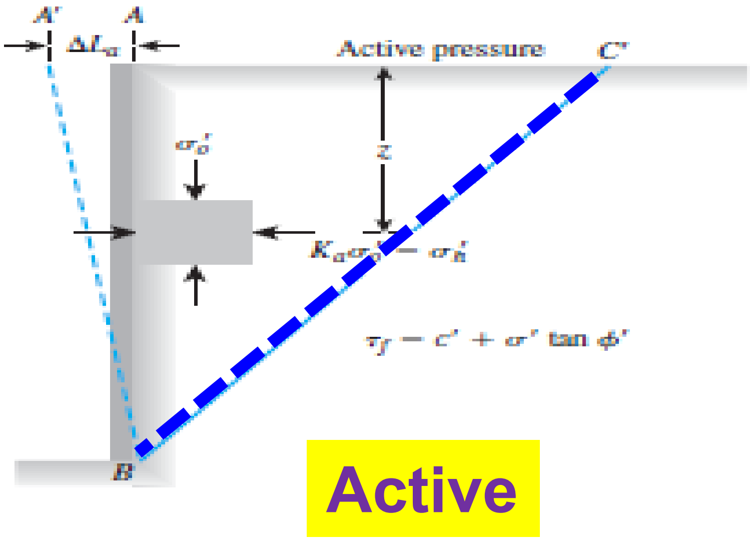
For the wall (retaining wall) in the left side there exist a soil with height less than the soil in the right and as mentioned above the right soil will pushes the wall away, so the wall will be pushed into the left soil (i.e. soil compresses the left soil) this means the soil has a passive effect and the force in this case is called passive force and termed by " P_P".



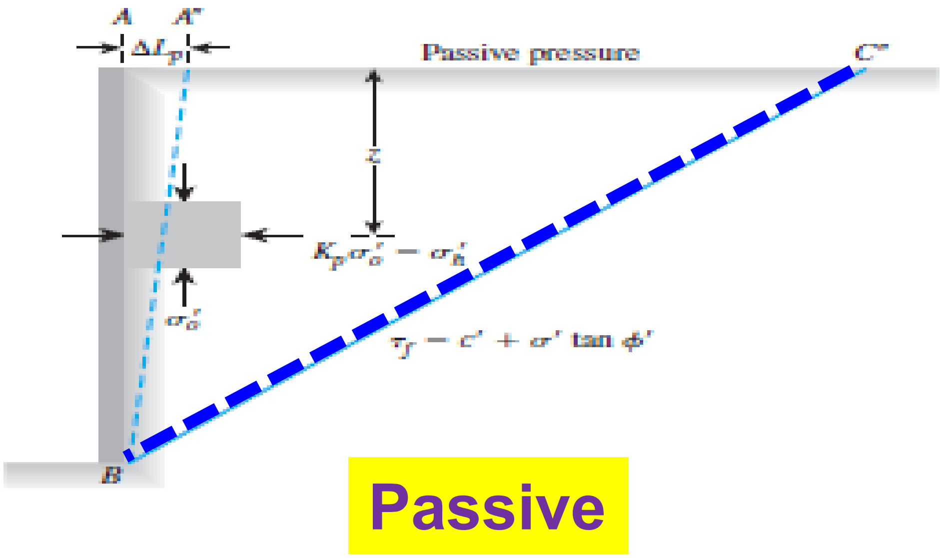
CASES



A
At-rest pressure
σ'_o
z
H
$K_o\sigma'_o = \sigma'_h$
$\tau_f = c' + \sigma' \tan \phi'$
B
At-Rest

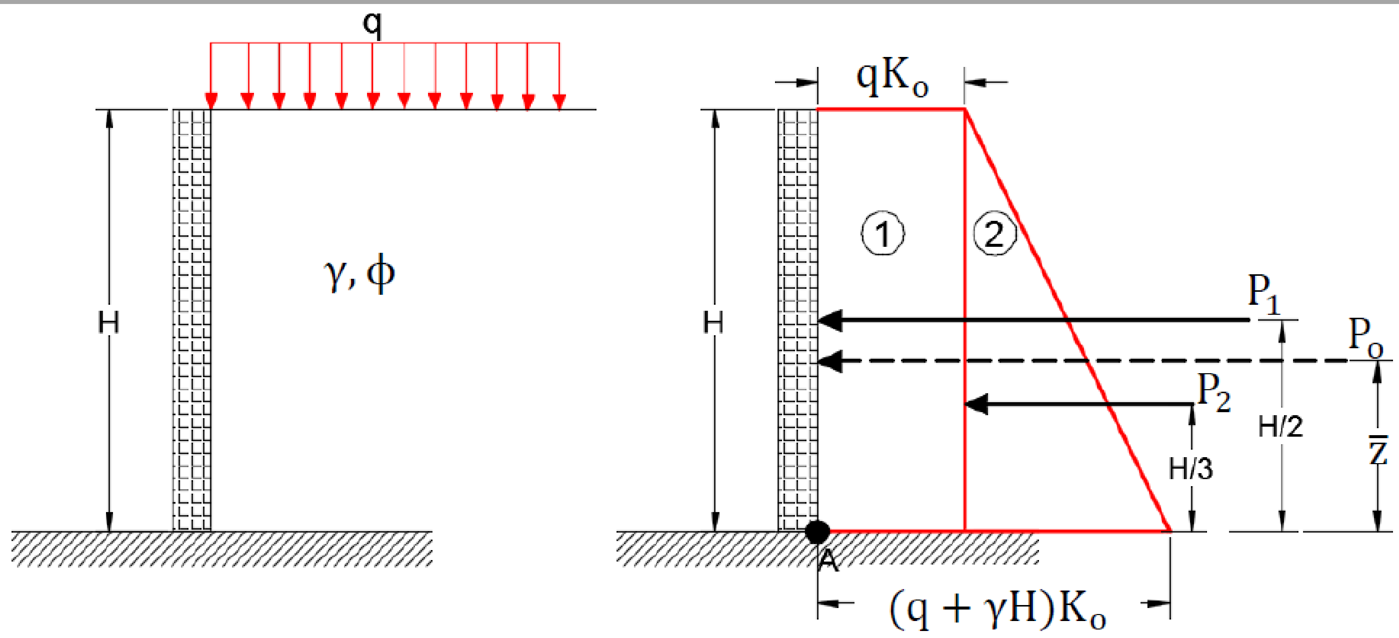


A'
A
ΔL_a
Active pressure
C'
σ'_o
z
$K_a\sigma'_o = \sigma'_h$
$\tau_f = c' + \sigma' \tan \phi'$
B
Active



A
A"
ΔL_p
Passive pressure
C"
z
$K_p\sigma'_o = \sigma'_h$
σ'_o
$\tau_f = c' + \sigma' \tan \phi'$
B
Passive

Lateral Earth Pressure at Rest



$$P_o = P_1 + P_2 = qK_oH + \tfrac{1}{2}\gamma H^2 K_o$$

where

P_1 = area of rectangle 1
P_2 = area of triangle 2

$$\bar{z} = \frac{P_1\left(\frac{H}{2}\right) + P_2\left(\frac{H}{3}\right)}{P_o}$$

Coefficient of Lateral Earth Pressure K_0

Jaky formula

For normally consolidated clays and loose sand.

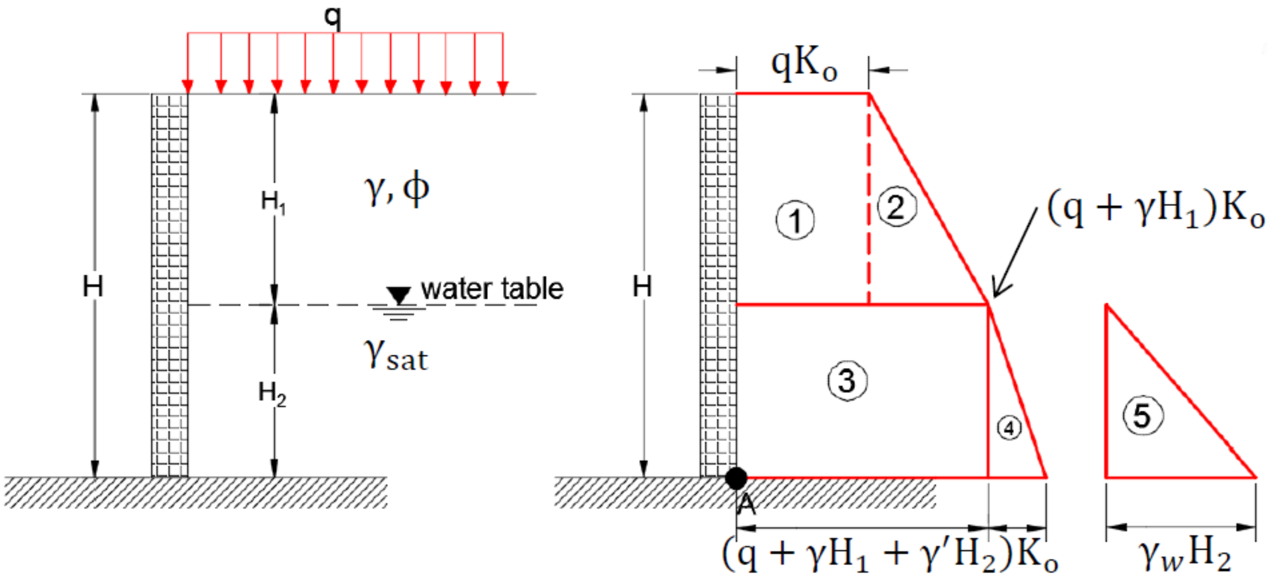
$$K_o \approx 1 - \sin \phi'$$

Mayne and Kulhawy

For Overconsolidated clays

$$K_o = (1 - \sin \phi')\,\mathrm{OCR}^{\sin \phi'}$$

Lateral Earth Pressure at Rest with Water



at $z = 0$, $\quad \sigma'_h = K_o\sigma'_o = K_o q$

at $z = H_1$, $\quad \sigma'_h = K_o\sigma'_o = K_o(q + \gamma H_1)$

and

at $z = H_2$, $\quad \sigma'_h = K_o\sigma'_o = K_o(q + \gamma H_1 + \gamma' H_2)$

$$P_o = A_1 + A_2 + A_3 + A_4 + A_5$$

where A = area of the pressure diagram.

So,

$$P_o = K_o q H_1 + \tfrac{1}{2}K_o\gamma H_1^2 + K_o(q + \gamma H_1)H_2 + \tfrac{1}{2}K_o\gamma' H_2^2 + \tfrac{1}{2}\gamma_w H_2^2$$

EXAMPLE 16.1

For the retaining wall shown in Figure 16.6a, determine the lateral earth force at rest per unit length of the wall. Also determine the location of the resultant force. Assume OCR = 1.

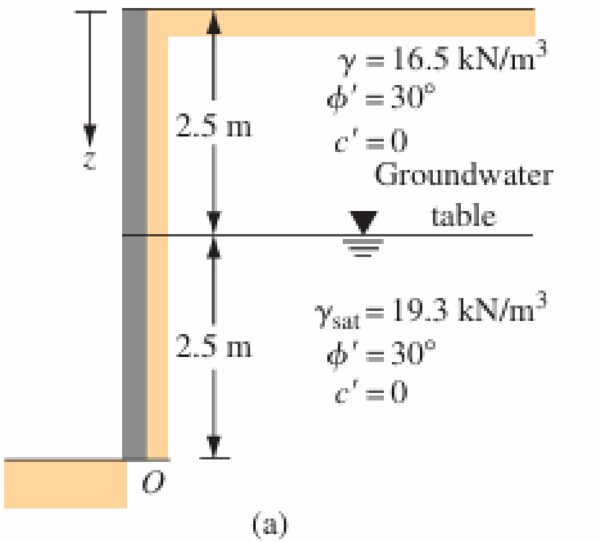


FIGURE 16.6

EXAMPLE 16.1

SOLUTION

$$K_o = 1 - \sin \phi' = 1 - \sin 30° = 0.5$$

At $z = 0$, $\sigma_o' = 0$; $\sigma_h' = 0$

At $z = 2.5$ m, $\sigma_o' = (16.5)(2.5) = 41.25$ kN/m^2;

$$\sigma_h' = K_o\sigma_o' = (0.5)(41.25) = 20.63 \text{ kN/m}^2$$

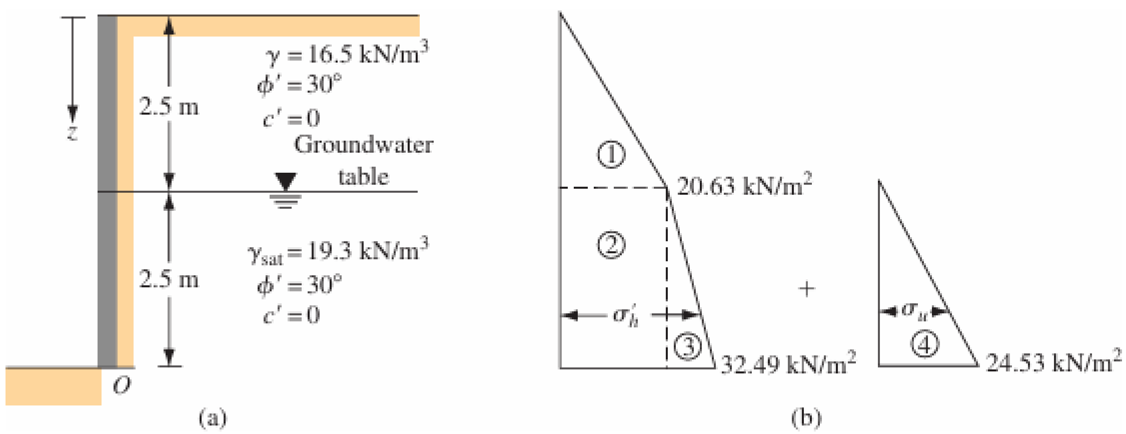
At $z = 5$ m, $\sigma_o' = (16.5)(2.5) + (19.3 - 9.81)2.5 = 64.98$ kN/m^2;

$$\sigma_h' = K_o\sigma_o' = (0.5)(64.98) = 32.49 \text{ kN/m}^2$$

The hydrostatic pressure distribution is as follows:

From $z = 0$ to $z = 2.5$ m, $u = 0$. At $z = 5$ m, $u = \gamma_w(2.5) = (9.81)(2.5) = 24.53$ kN/m^2.

The pressure distribution for the wall is shown in Figure 16.6b.



EXAMPLE 16.1

The total force per unit length of the wall can be determined from the area of the pressure diagram, or

$$P_o = \text{Area } 1 + \text{Area } 2 + \text{Area } 3 + \text{Area } 4$$
$$= \tfrac{1}{2}(2.5)(20.63) + (2.5)(20.63) + \tfrac{1}{2}(2.5)(32.49 - 20.63)$$
$$+ \tfrac{1}{2}(2.5)(24.53) = \mathbf{122.85\ kN/m}$$

The location of the center of pressure measured from the bottom of the wall (point O) =

$$\bar{z} = \frac{(\text{Area } 1)\left(2.5 + \frac{2.5}{3}\right) + (\text{Area } 2)\left(\frac{2.5}{2}\right) + (\text{Area } 3 + \text{Area } 4)\left(\frac{2.5}{3}\right)}{P_o}$$

$$= \frac{(25.788)(3.33) + (51.575)(1.25) + (14.825 + 30.663)(0.833)}{122.85}$$

$$= \frac{85.87 + 64.47 + 37.89}{122.85} = \mathbf{1.53\ m}$$

NOTES

- **If the lateral strain in the soil is ZERO the corresponding lateral pressure is called the earth pressure at-rest. This is the case before construction.**

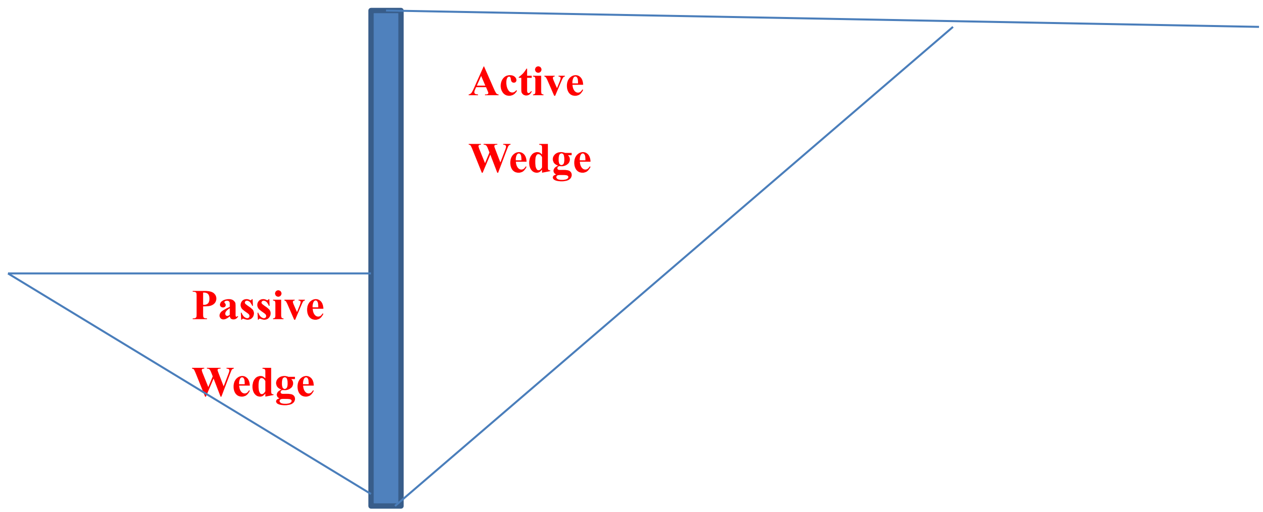
- **In the case of active case the soil is the actuating element and in the case of passive the wall is the actuating element.**

- **For either the active or passive states to develop, the wall must MOVE. If the wall does not move, an intermediate stress state exists called earth pressure at rest. (i.e. zero lateral strain).**

- **For greatest economy, retaining structures are designed only sufficiently strong to resist ACTIVE PRESSURE. They therefore must be allowed to move.**

- **It may at first seem unlikely that a wall ever would be built to PUSH into the soil and mobilize passive earth pressure.**

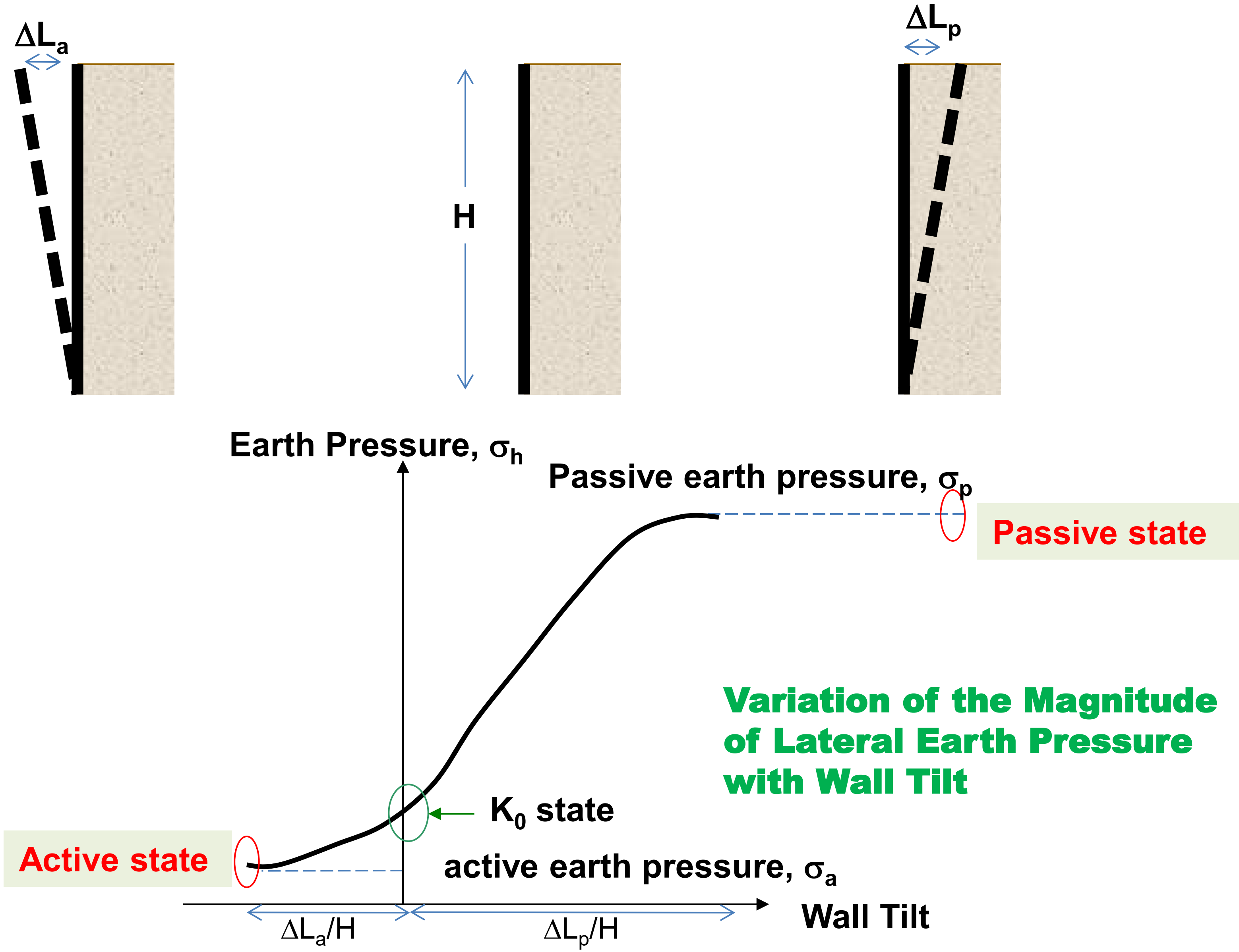
NOTES



- Typically passive earth pressure is developed by anchor plates or blocks, embedded in the soil and where the anchor rod or cable tension pulls the anchor into/against the soil to develop passive resistance. Walls are seldom designed for passive pressure.

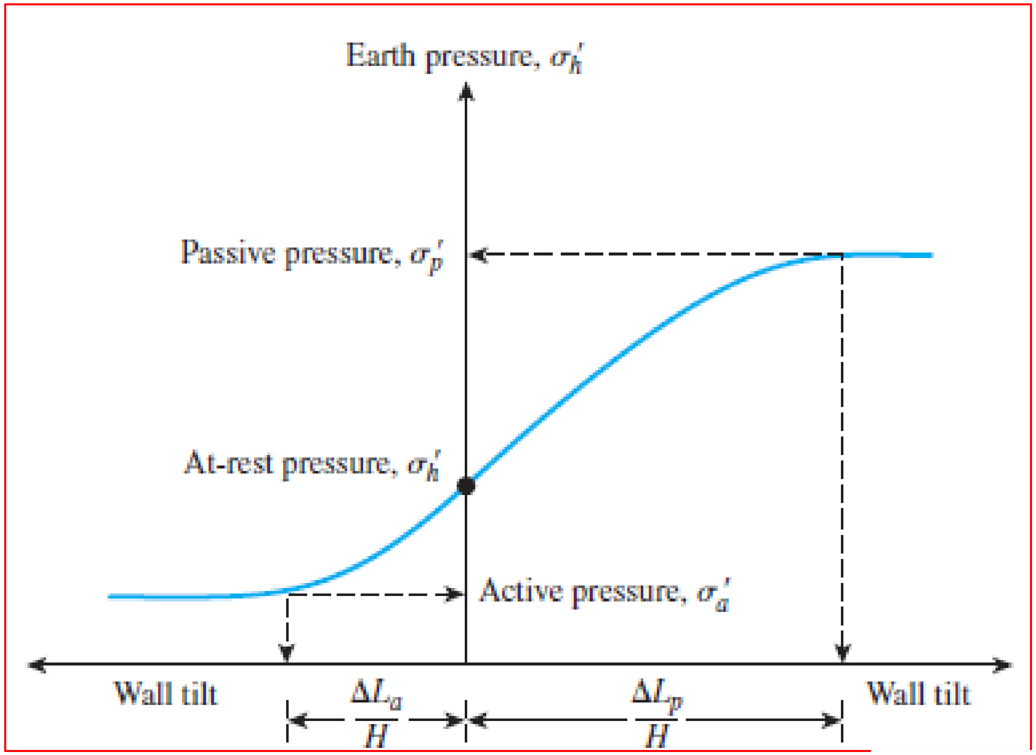
- In most retaining walls of limited height, movement may occur by simple translation or, more frequently, by rotation about the bottom.

NOTES



Variation of the Magnitude of Lateral Earth Pressure with Wall Tilt

NOTES



Active or passive condition will only be reached if the wall is allowed to yield sufficiently. The amount of wall necessary depends on:-

- **Soil type (sand vs. clay)**
- **Soil density (Loose vs. dense)**
- **Pressure (Active vs. passive)**

Typical Values of $\Delta L_a/H$ and $\Delta L_p/H$

Soil type	$\Delta L_a/H$	$\Delta L_p/H$
Loose sand	0.001–0.002	0.01
Dense sand	0.0005–0.001	0.005
Soft clay	0.02	0.04
Stiff clay	0.01	0.02

Lateral Earth Pressure Theories

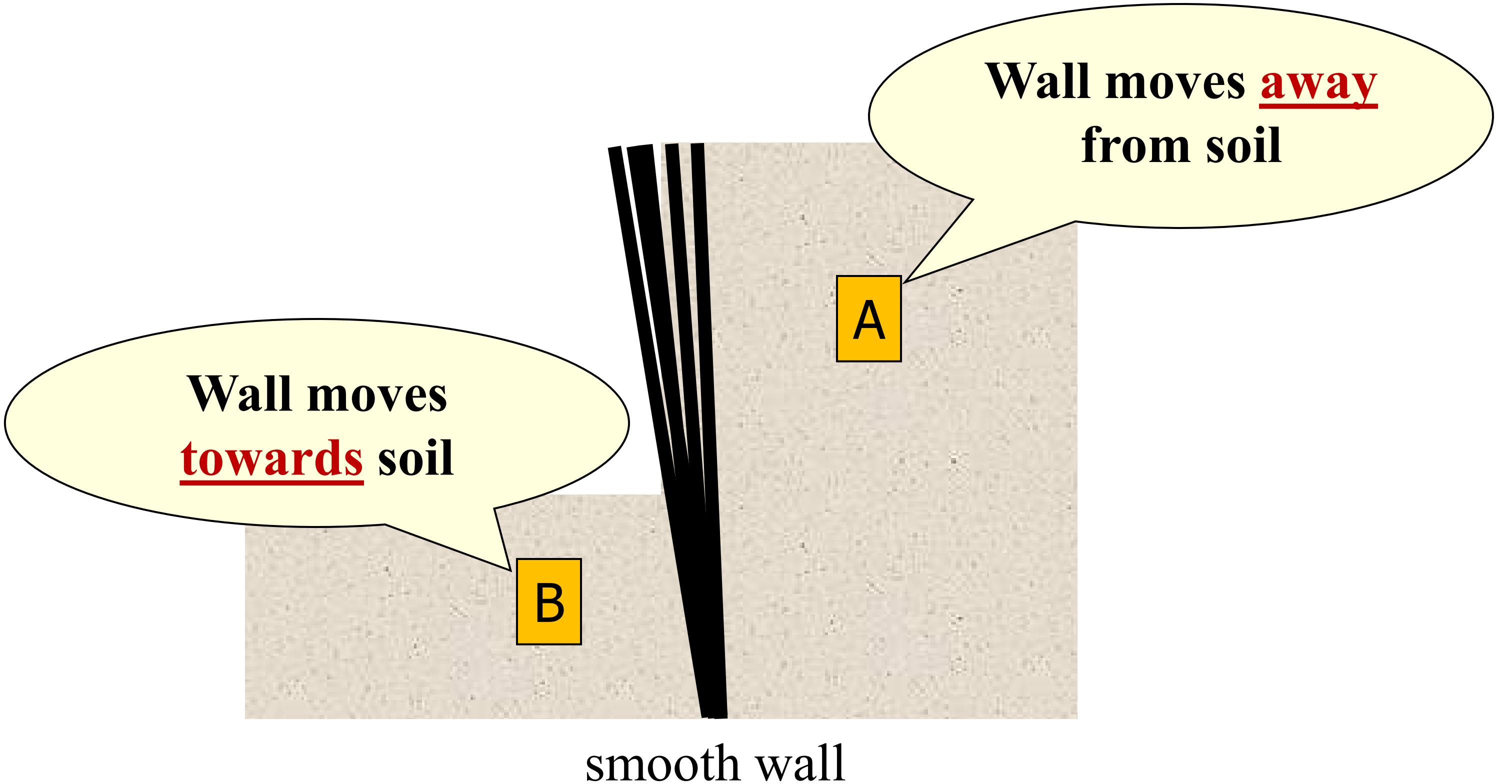
- **Since late 17th century many theories of earth of earth pressure have been proposed by various investigators. Of the theories the following two are the most popular and used for computation of active and passive earth pressures:.**

  1. **Rankine's Theory (No wall friction)**
  2. **Coulomb's Theory (With wall friction)**

- **Those are usually called the classical lateral earth pressure theories.**

- **In both theories it is required that the soil mass, or at least certain parts of the mass, is in a state of PLASTIC EQUILIBRIUM. The soil mass is on verge of failure. Failure here is defined to be the state of stress which satisfies the Mohr-Coulomb criterion.**

Active vs. Passive Earth Pressures



Let's look at the soil elements A and B during the wall movement.

- **In most retaining walls of limited height, movement may occur by simple translation or, more frequently, by rotation about the bottom.**

Rankine's Earth Pressure Theory

- Rankine (1857) investigated the stress condition in a soil at a state of PLASTIC EQUILIBRIUM.
- Developed based on semi infinite "loose granular" soil mass for which the soil movement is uniform.
- Used stress states of soil mass to determine lateral pressures on a frictionless wall

Assumptions:

- Vertical wall
- Smooth retaining wall
- Horizontal ground surface
- Homogeneous soil

Active Earth Pressure

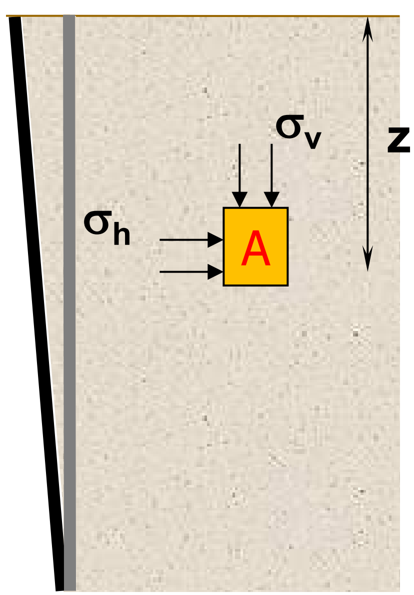
Active earth pressure

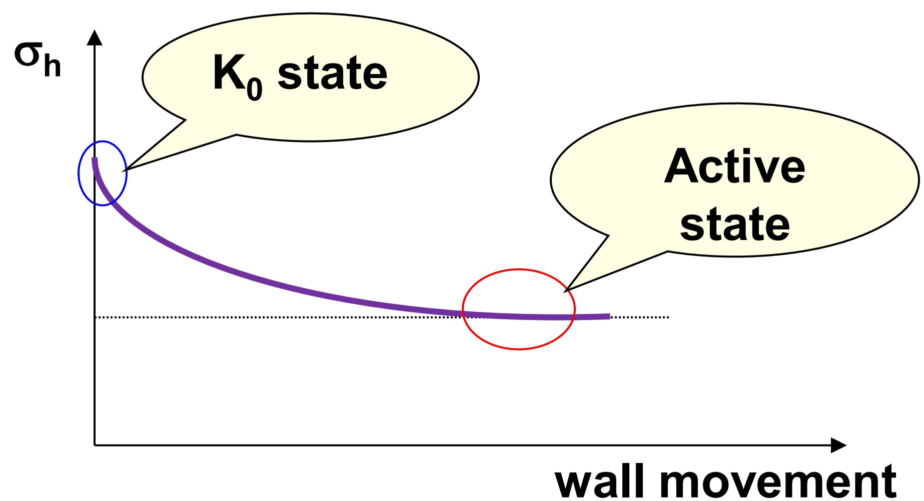
- $\sigma_v = \gamma z$
- Initially, there is no lateral movement.

  $\therefore \sigma_h = K_0 \sigma_v = K_0 \gamma z$
- As the wall moves away from the soil,
- σ_v remains the same; and
- σ_h **decreases** till failure occurs.

Active state

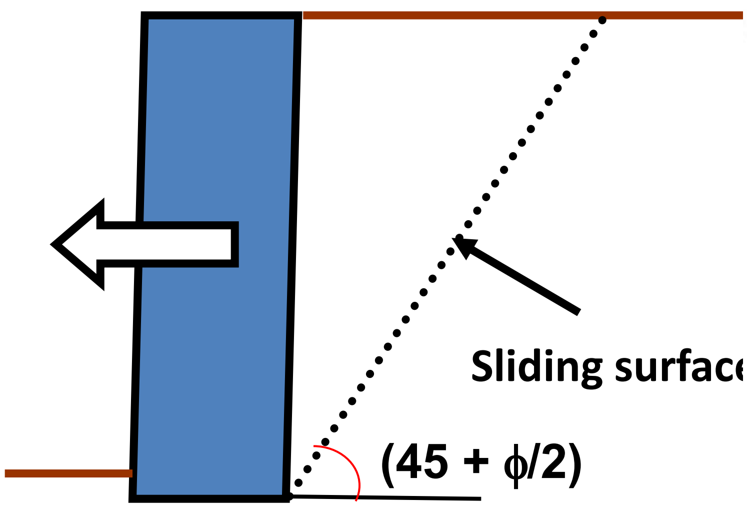
$\sigma_h \rightarrow \sigma_a$

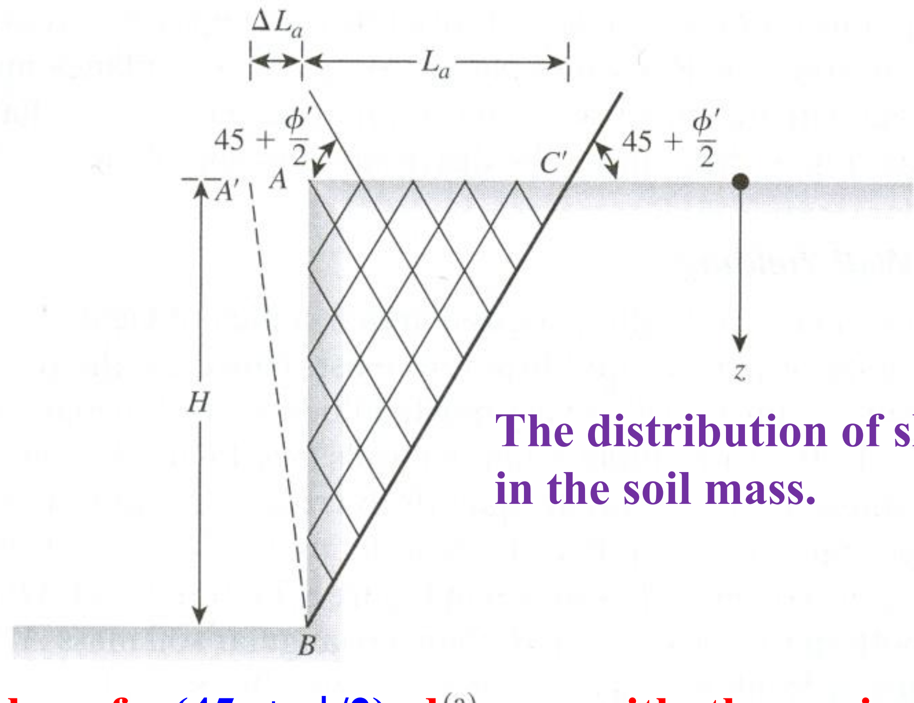




Orientation of Failure Planes

- From Mohr Circle the failure planes in the soil make ± (45 + ϕ/2)-degree angles with the direction of the major principal plane—that is, the horizontal.
- These are called potential ***slip planes.***

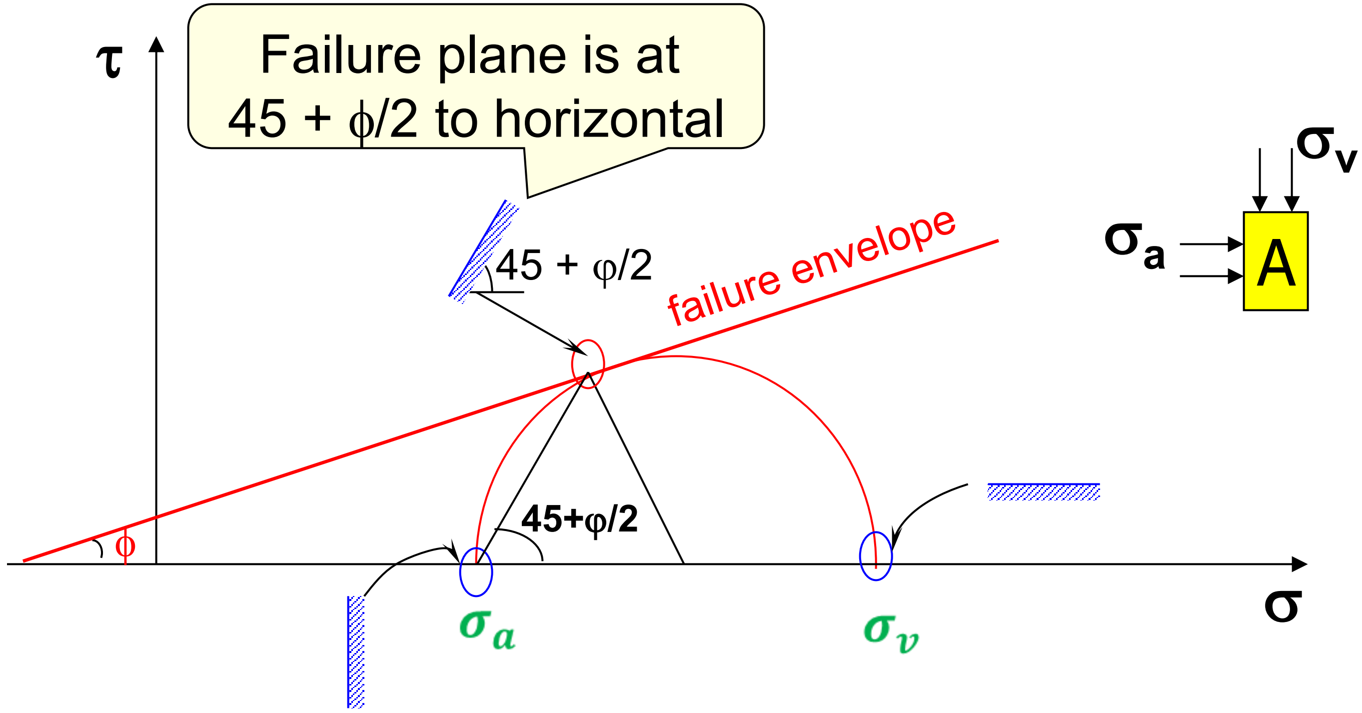




The distribution of slip plan in the soil mass.

- **Because the slip planes make angles of (45 + ϕ/2) degrees with the major principal plane, the soil mass in the state of plastic equilibrium is bounded by the plane *BC′*. The soil inside the zone *ABC′* undergoes the same unit deformation in the horizontal direction everywhere, which is equal to $\Delta L_a/L_a$.**

Active Earth Pressure



Active Earth Pressure

Retaining wall with a vertical back and a horizontal backfill

Mohr–Coulomb failure envelope defined by the equation

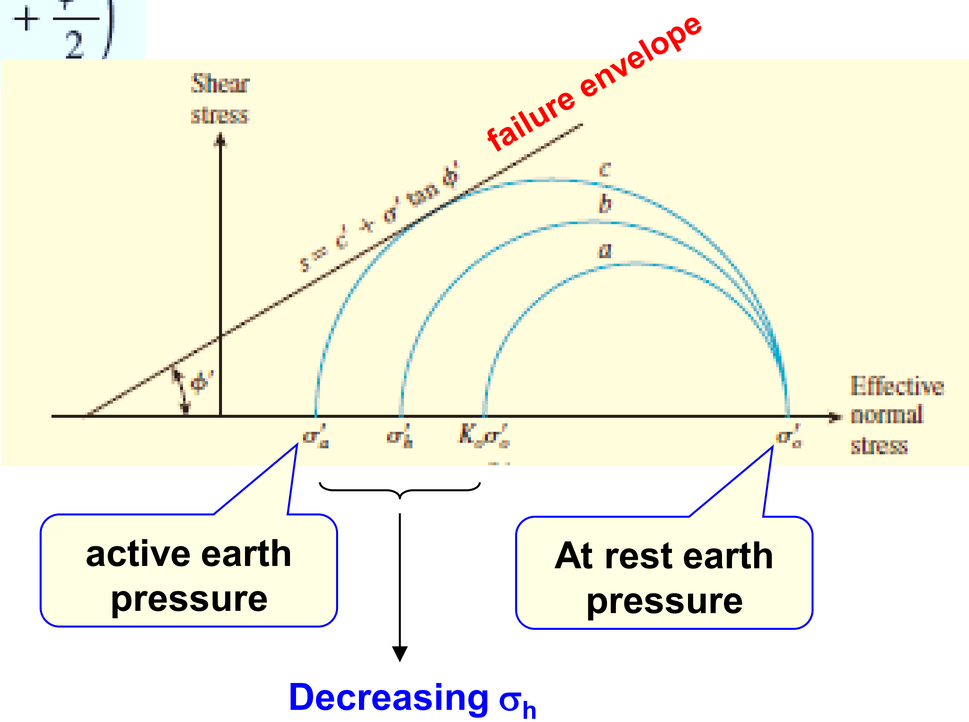
$$s = c' + \sigma' \tan \phi'$$

The principal stresses for a Mohr's circle that touches the Mohr–Coulomb failure envelope:

$$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

$$\sigma_o' = \sigma_a' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

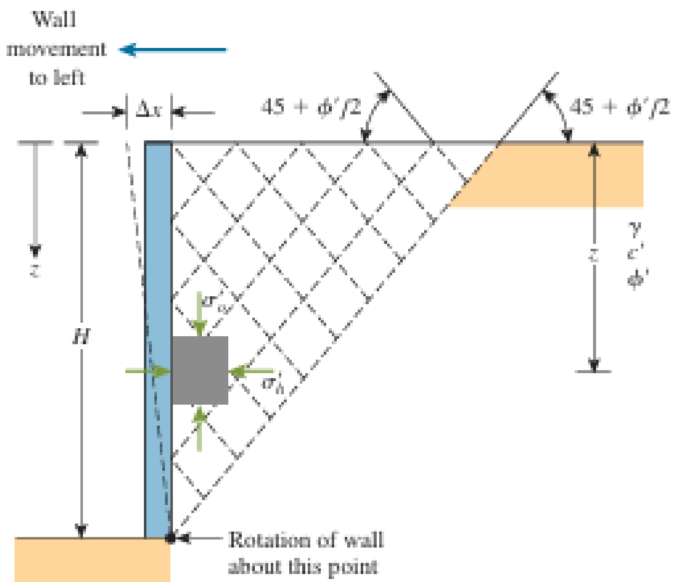
$$\sigma_a' = \frac{\sigma_o'}{\tan^2\left(45 + \frac{\phi'}{2}\right)} - \frac{2c'}{\tan\left(45 + \frac{\phi'}{2}\right)}$$

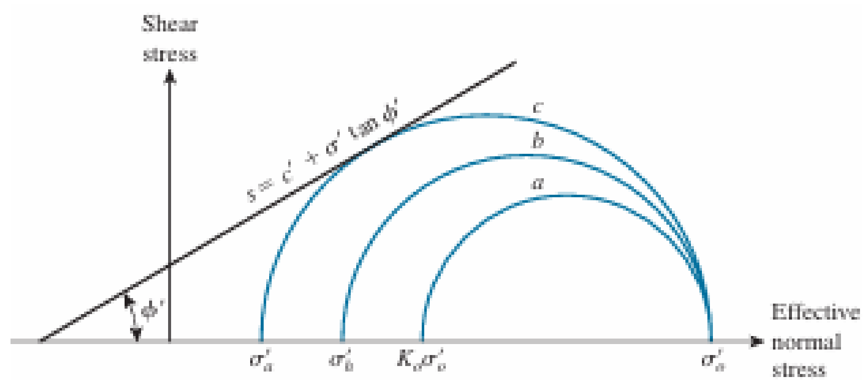


$$\sigma_a' = \sigma_o' \tan^2\left(45 - \frac{\phi'}{2}\right) - 2c' \tan\left(45 - \frac{\phi'}{2}\right)$$
$$= \sigma_o' K_a - 2c'\sqrt{K_a}$$

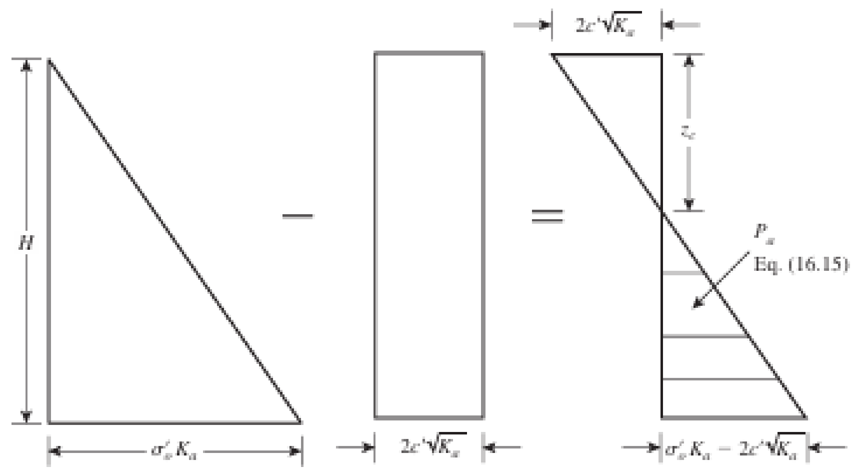
$K_a = \tan^2(45 - \phi'/2) =$ Rankine active-pressure coefficient.

Active Earth Pressure





$$
\begin{aligned}
\sigma'_a &= \sigma'_o \tan^2\left(45 - \frac{\phi'}{2}\right) - 2c' \tan\left(45 - \frac{\phi'}{2}\right) \\
&= \sigma'_o K_a - 2c'\sqrt{K_a}
\end{aligned}
$$



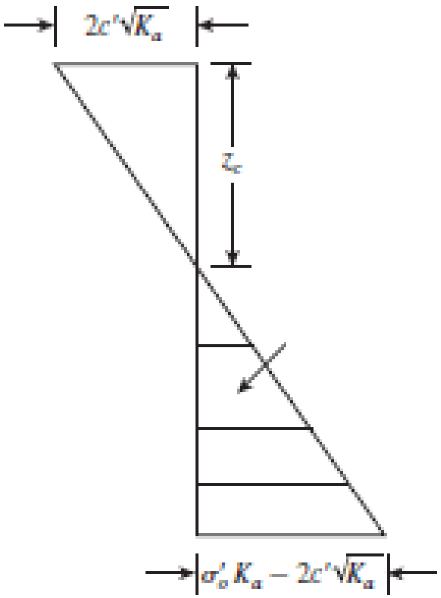
Active Earth Pressure

Tensile stress in the soil will cause a crack along the soil–wall interface

The *depth of tensile crack*

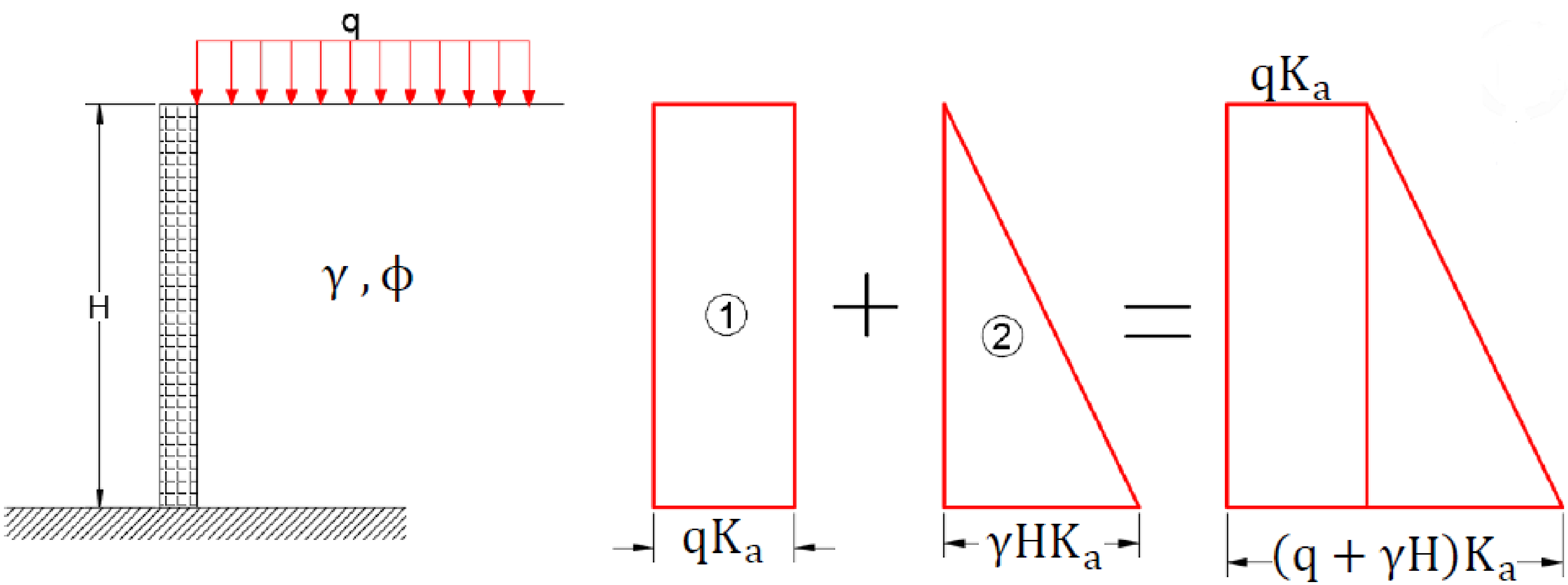
$$\gamma z_c K_a - 2c'\sqrt{K_a} = 0$$

$$z_c = \frac{2c'}{\gamma\sqrt{K_a}}$$



Active Earth Pressure

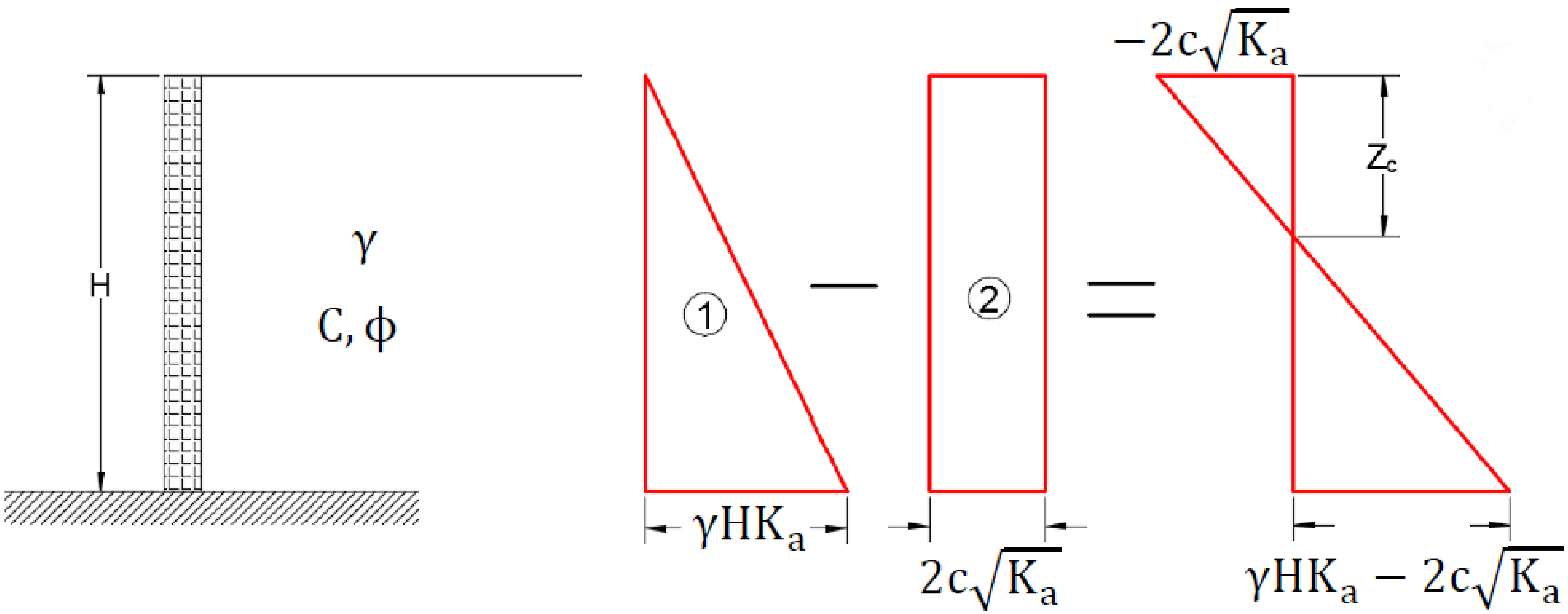
In case of granular soil (pure sand):



$$P_a = P_1 + P_2$$

Active Earth Pressure

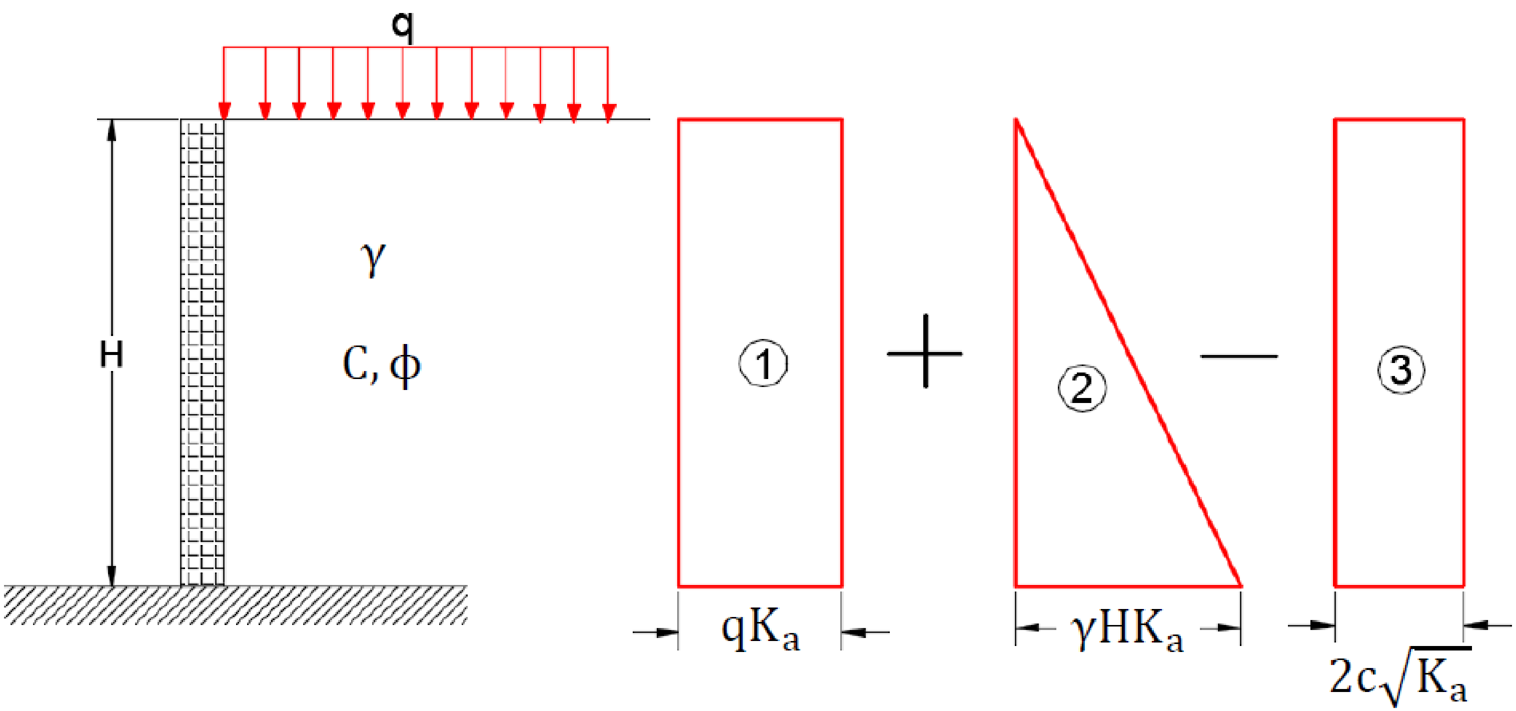
If the soil is $C - \phi$ soil:



Active Earth Pressure

If the soil is C – ϕ soil:

If there exist surcharge:



$$\sigma_{h,a} = (q + \gamma H)K_a - 2c\sqrt{K_a}$$

EXAMPLE 16.2

EXAMPLE 16.2

A 6 m high retaining wall is to support a soil with unit weight $\gamma = 17.4$ kN/m^3, soil friction angle $\phi' = 26°$, and cohesion $c' = 5$ kN/m^2. Determine the Rankine active force per unit length of the wall both before and after the tensile crack occurs, and determine the line of action of the resultant in both cases.

SOLUTION

For $\phi' = 26°$,

$$K_a = \tan^2\left(45 - \frac{\phi'}{2}\right) = \tan^2(45 - 13) = 0.39$$

$$\sqrt{K_a} = 0.625$$

$$\sigma'_a = \gamma H K_a - 2c'\sqrt{K_a}$$

From Figure 16.7c, at $z = 0$,

$$\sigma'_a = -2c'\sqrt{K_a} = -2(5)(0.625) = -6.25 \text{ kN/m}^2$$

and at $z = 6$ m,

$$\sigma'_a = (17.4)(6)(0.39) - 2(5)(0.625)$$

$$= 40.72 - 6.25 = 34.47 \text{ kN/m}^2$$

EXAMPLE 16.2

Active Force Before the Tensile Crack Appeared: Eq. (16.13)

$$P_a = \tfrac{1}{2}\gamma H^2 K_a - 2c'H\sqrt{K_a}$$
$$= \tfrac{1}{2}(6)(40.72) - (6)(6.25) = 122.16 - 37.5 = \mathbf{84.66\ kN/m}$$

The line of action of the resultant can be determined by taking the moment of the area of the pressure diagrams about the bottom of the wall, or

$$P_a\bar{z} = (122.16)\left(\frac{6}{3}\right) - (37.5)\left(\frac{6}{2}\right)$$

Thus,

$$\bar{z} = \frac{244.32 - 112.5}{84.66} = \mathbf{1.56\ m}$$

EXAMPLE 16.2

Active Force After the Tensile Crack Appeared: Eq. (16.12)

$$z_c = \frac{2c'}{\gamma\sqrt{K_a}} = \frac{2(5.0)}{(17.4)(0.625)} = 0.92 \text{ m}$$

Using Eq. (16.14) gives

$$P_a = \tfrac{1}{2}(H - z_c)(\gamma H K_a - 2c'\sqrt{K_a}) = \tfrac{1}{2}(6 - 0.92)(34.47) = \mathbf{87.55\ kN/m}$$

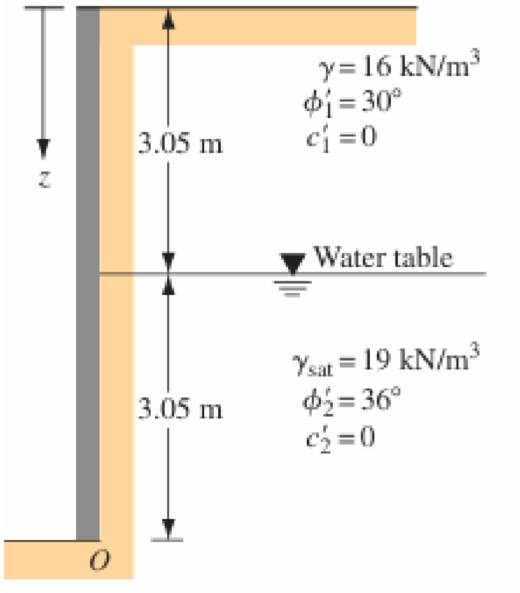
Figure 16.7c indicates that the force $P_a = 87.55$ kN/m is the area of the hatched triangle. Hence, the line of action of the resultant will be located at a height $\bar{z} = (H - z_c)/3$ above the bottom of the wall, or

$$\bar{z} = \frac{6 - 0.92}{3} = \mathbf{1.69\ m}$$

EXAMPLE 16.3

EXAMPLE 16.3

Assume that the retaining wall shown in Figure 16.8a can yield sufficiently to develop an active state. Determine the Rankine active force per unit length of the wall and the location of the resultant line of action.



(a)

EXAMPLE 16.3

SOLUTION

If the cohesion, c', is zero, then

$$\sigma'_a = \sigma'_o K_a$$

For the top layer of soil, $\phi'_1 = 30°$, so

$$K_{a(1)} = \tan^2\left(45 - \frac{\phi'_1}{2}\right) = \tan^2(45 - 15) = \frac{1}{3}$$

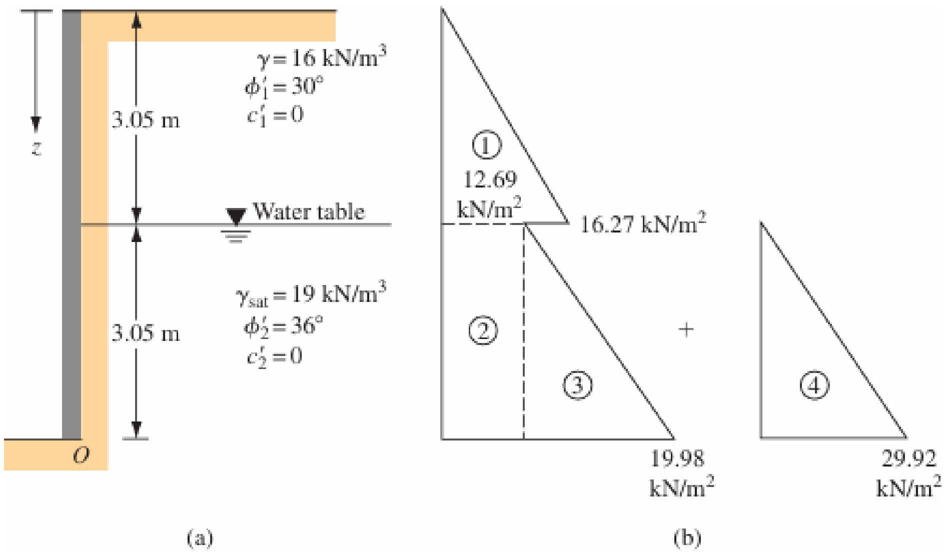
Similarly, for the bottom layer of soil, $\phi'_2 = 36°$, and it follows that

$$K_{a(2)} = \tan^2\left(45 - \frac{36}{2}\right) = 0.26$$

The following table shows the calculation of σ'_a and u at various depths below the ground surface.

Depth, z (m)	σ'_o (kN/m^2)	K_a	$\sigma'_a = K_a \sigma'_o$ (kN/m^2)	u (kN/m^2)
0	0	1/3	0	0
3.05^-	(16)(3.05) = 48.8	1/3	16.27	0
3.05^+	48.8	0.26	12.69	0
6.1	(16)(3.05) + (19 − 9.81)(3.05) = 76.83	0.26	19.98	(9.81)(3.05) = 29.92

EXAMPLE 16.3



The pressure distribution diagram is plotted in Figure 16.8b. The force per unit length is

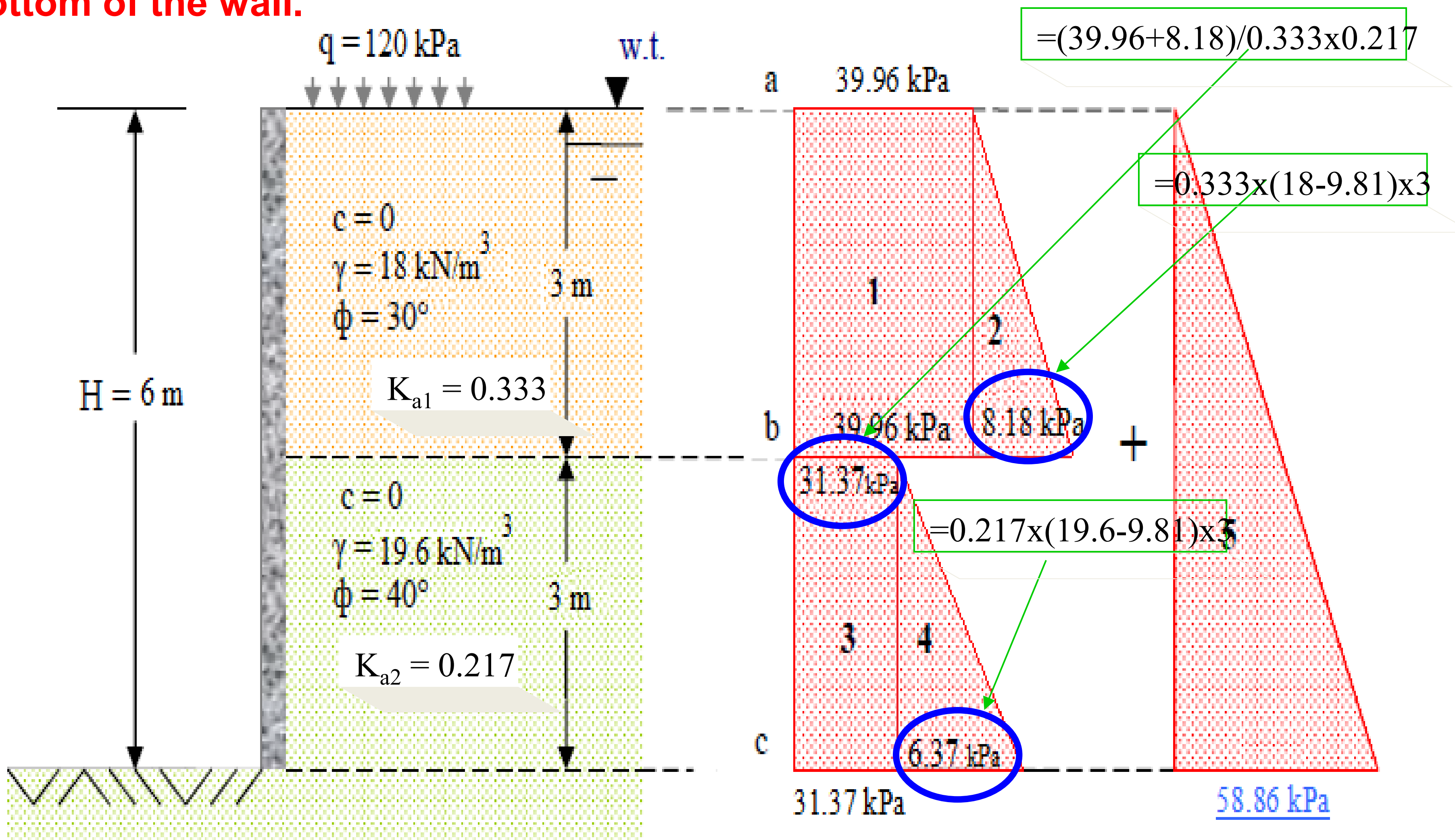
$$P_a = \text{area 1} + \text{area 2} + \text{area 3} + \text{area 4}$$
$$= \tfrac{1}{2}(3.05)(16.27) + (12.69)(3.05) + \tfrac{1}{2}(19.98 - 12.69)(3.05) + \tfrac{1}{2}(29.92)(3.05)$$
$$= 24.81 + 38.70 + 11.12 + 45.63 = \mathbf{120.26\ kN/m}$$

The distance of the line of action of the resultant force from the bottom of the wall can be determined by taking the moments about the bottom of the wall (point O in Figure 16.8a) and is

$$\bar{z} = \frac{(24.81)\left(3.05 + \frac{3.05}{3}\right) + (38.7)\left(\frac{3.05}{2}\right) + (11.12 + 45.63)\left(\frac{10}{3}\right)}{120.26} = \mathbf{1.81\ m}$$

EXAMPLE

Draw the pressure diagram on the wall in an active pressure condition, and find the total resultant F on the wall and its location with respect to the bottom of the wall.



SOLUTION

Step 1

$K_{a1} = \tan^2 (45° - 30°/2) = 0.333$

$K_{a2} = \tan^2 (45° - 40°/2) = 0.217$

Step 2

The stress on the wall at point *a* is:

$p_a = q K_{a1} = (120)(0.333) = \underline{39.96\ \text{kPa}}$

The stress at *b* (within the top stratum) is:

$p_{b-} = (q + \gamma' h) K_{a1}$

$= [120 + (18 - 9.81)(3)] [0.333] = \underline{48.14\ \text{kPa}}$

The stress at *b* (within bottom stratum) is:

$p_{b+} = (q + \gamma' h) K_{a2}$

$= [120 + (18 - 9.81)(3)] [0.217] = \underline{31.37\ \text{kPa}}$

The stress at point *c* is:

$p_{c'} = [q + (\gamma' h)_1 + (\gamma' h)_2] K_{a2}$

$= [120 + (18 - 9.81)(3) + (19.6 - 9.81)(3)] [0.217] = \underline{37.75\ \text{kPa}}$

The pressure of the water upon the wall is:

$p_w = \gamma_w h = (9.81)(6) = \underline{58.86\ \text{kPa}}$

SOLUTION

Step 3

The forces from each area:

$F_1 = (3)\ (39.96) = 119.88$ kN/m

$F_2 = ½\ (3)\ (8.18) = 12.27$ kN/m

$F_3 = (3)\ (31.37) = 94.11$ kN/m

$F_4 = ½\ (3)\ (6.37) = 9.555$ kN/m

$F_5 = ½\ (58.86)\ (6) = 176.58$ kN/m

$\mathbf{F_{total}} = \mathbf{412.395}$ kN/m

Step 4

The location of forces ŷ is at:

$\hat{y}\ \mathbf{412.395} = 119.88\ (4.5) + 12.27\ (4) + 94.11\ (1.5) + 9.555\ (1) + 176.58\ (2)$

$= 539.46 + 49.08 + 141.165 + 9.555 + 353.16 = 1092.42$ kN

ŷ = 2.65 m from bottom of wall

General Case: Inclined Wall with Inclined Backfill

Granular backfill (c' =0)

α = inclination of backfill with horizontal

θ = inclination of wall with vertical

β = inclination of P_a with the normal to the wall

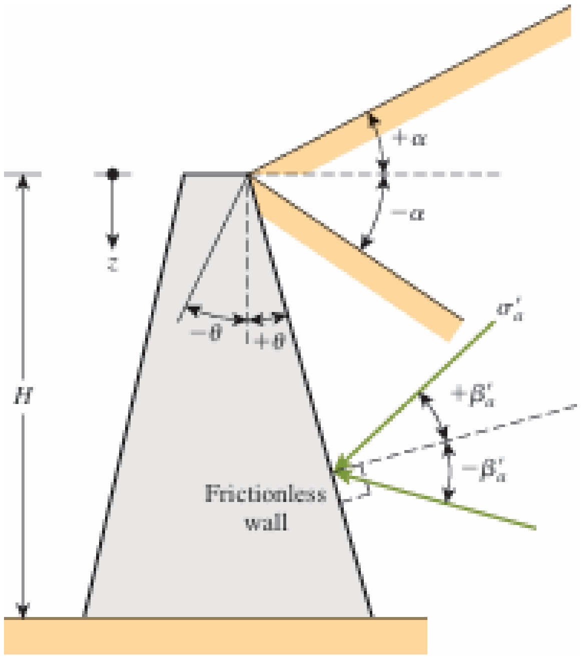
$$\sigma'_a = \frac{\gamma z \cos\alpha \sqrt{1 + \sin^2\phi' - 2\sin\phi' \cos\psi_a}}{\cos\alpha + \sqrt{\sin^2\phi' - \sin^2\alpha}}$$

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

$$\beta'_a = \tan^{-1}\left(\frac{\sin\phi' \sin\psi_a}{1 - \sin\phi' \cos\psi_a}\right)$$

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

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



General Case : Inclined Wall with Inclined Backfill

TABLE 16.1 Variation of $K_{a(R)}$ [Eq. (16.20)]

α (deg)	θ (deg)	$K_{a(R)}$ ϕ' (deg) 28	30	32	34	36	38	40
0	0	0.361	0.333	0.307	0.283	0.260	0.238	0.217
	2	0.363	0.335	0.309	0.285	0.262	0.240	0.220
	4	0.368	0.341	0.315	0.291	0.269	0.248	0.228
	6	0.376	0.350	0.325	0.302	0.280	0.260	0.242
	8	0.387	0.362	0.338	0.316	0.295	0.276	0.259
	10	0.402	0.377	0.354	0.333	0.314	0.296	0.280
	15	0.450	0.428	0.408	0.390	0.373	0.358	0.345
5	0	0.366	0.337	0.311	0.286	0.262	0.240	0.219
	2	0.373	0.344	0.317	0.292	0.269	0.247	0.226
	4	0.383	0.354	0.328	0.303	0.280	0.259	0.239
	6	0.396	0.368	0.342	0.318	0.296	0.275	0.255
	8	0.412	0.385	0.360	0.336	0.315	0.295	0.276
	10	0.431	0.405	0.380	0.358	0.337	0.318	0.300
	15	0.490	0.466	0.443	0.423	0.405	0.388	0.373
10	0	0.380	0.350	0.321	0.294	0.270	0.246	0.225
	2	0.393	0.362	0.333	0.306	0.281	0.258	0.236
	4	0.408	0.377	0.348	0.322	0.297	0.274	0.252
	6	0.426	0.395	0.367	0.341	0.316	0.294	0.273
	8	0.447	0.417	0.389	0.363	0.339	0.317	0.297
	10	0.471	0.441	0.414	0.388	0.365	0.344	0.324
	15	0.542	0.513	0.487	0.463	0.442	0.422	0.404

General Case : Inclined Wall with Inclined Backfill

TABLE 16.2 Variation of β'_a [Eq. (16.18)]

α (deg)	θ (deg)	β'_a — ϕ' (deg) 28	30	32	34	36	38	40
0	0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	2	3.525	3.981	4.484	5.041	5.661	6.351	7.124
	4	6.962	7.848	8.821	9.893	11.075	12.381	13.827
	6	10.231	11.501	12.884	14.394	16.040	17.837	19.797
	8	13.270	14.861	16.579	18.432	20.428	22.575	24.876
	10	16.031	17.878	19.850	21.951	24.184	26.547	29.039
	15	21.582	23.794	26.091	28.464	30.905	33.402	35.940
5	0	5.000	5.000	5.000	5.000	5.000	5.000	5.000
	2	8.375	8.820	9.311	9.854	10.455	11.123	11.870
	4	11.553	12.404	13.336	14.358	15.482	16.719	18.085
	6	14.478	15.679	16.983	18.401	19.942	21.618	23.441
	8	17.112	18.601	20.203	21.924	23.773	25.755	27.876
	10	19.435	21.150	22.975	24.915	26.971	29.144	31.434
	15	23.881	25.922	28.039	30.227	32.479	34.787	37.140
10	0	10.000	10.000	10.000	10.000	10.000	10.000	10.000
	2	13.057	13.491	13.967	14.491	15.070	15.712	16.426
	4	15.839	16.657	17.547	18.519	19.583	20.751	22.034
	6	18.319	19.460	20.693	22.026	23.469	25.032	26.726
	8	20.483	21.888	23.391	24.999	26.720	28.559	30.522
	10	22.335	23.946	25.653	27.460	29.370	31.385	33.504
	15	25.683	27.603	29.589	31.639	33.747	35.908	38.114

Vertical Wall with Inclined Backfill

Granular backfill (c' =0)

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

$$K_a = \cos\alpha \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}}$$

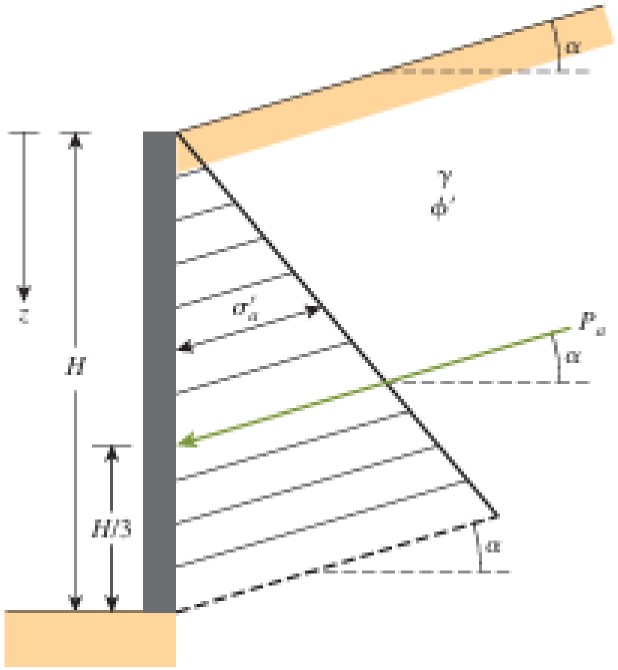


Table 16.3 gives the variation of K_a for various values of α, and ϕ'

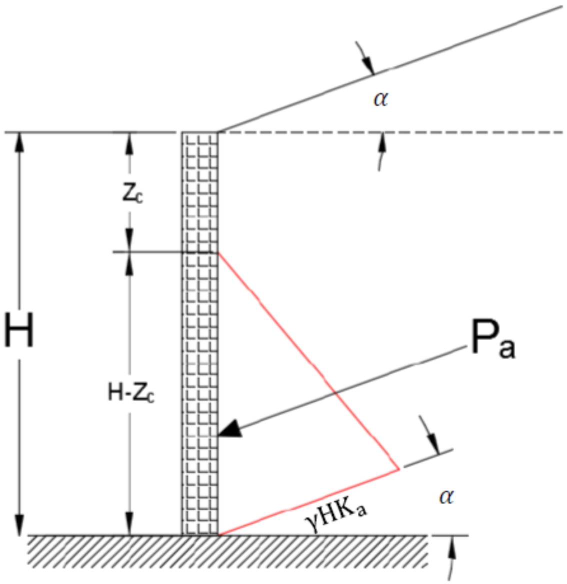
Vertical Wall with Inclined Backfill

TABLE 16.3 Values of K_a for Wall with Vertical Backface and Inclined Backfill [Eq. (16.22)]

α (deg) ↓	ϕ' (deg) → 28	29	30	31	32	33	34	35	36	37	38	39	40
0	0.3610	0.3470	0.3333	0.3201	0.3073	0.2948	0.2827	0.2710	0.2596	0.2486	0.2379	0.2275	0.2174
1	0.3612	0.3471	0.3335	0.3202	0.3074	0.2949	0.2828	0.2711	0.2597	0.2487	0.2380	0.2276	0.2175
2	0.3618	0.3476	0.3339	0.3207	0.3078	0.2953	0.2832	0.2714	0.2600	0.2489	0.2382	0.2278	0.2177
3	0.3627	0.3485	0.3347	0.3214	0.3084	0.2959	0.2837	0.2719	0.2605	0.2494	0.2386	0.2282	0.2181
4	0.3639	0.3496	0.3358	0.3224	0.3094	0.2967	0.2845	0.2726	0.2611	0.2500	0.2392	0.2287	0.2186
5	0.3656	0.3512	0.3372	0.3237	0.3105	0.2978	0.2855	0.2736	0.2620	0.2508	0.2399	0.2294	0.2192
6	0.3676	0.3531	0.3389	0.3253	0.3120	0.2992	0.2868	0.2747	0.2631	0.2518	0.2409	0.2303	0.2200
7	0.3701	0.3553	0.3410	0.3272	0.3138	0.3008	0.2883	0.2761	0.2644	0.2530	0.2420	0.2313	0.2209
8	0.3730	0.3580	0.3435	0.3294	0.3159	0.3027	0.2900	0.2778	0.2659	0.2544	0.2432	0.2325	0.2220
9	0.3764	0.3611	0.3463	0.3320	0.3182	0.3049	0.2921	0.2796	0.2676	0.2560	0.2447	0.2338	0.2233
10	0.3802	0.3646	0.3495	0.3350	0.3210	0.3074	0.2944	0.2818	0.2696	0.2578	0.2464	0.2354	0.2247
11	0.3846	0.3686	0.3532	0.3383	0.3241	0.3103	0.2970	0.2841	0.2718	0.2598	0.2482	0.2371	0.2263
12	0.3896	0.3731	0.3573	0.3421	0.3275	0.3134	0.2999	0.2868	0.2742	0.2621	0.2503	0.2390	0.2281
13	0.3952	0.3782	0.3620	0.3464	0.3314	0.3170	0.3031	0.2898	0.2770	0.2646	0.2527	0.2412	0.2301
14	0.4015	0.3839	0.3671	0.3511	0.3357	0.3209	0.3068	0.2931	0.2800	0.2674	0.2552	0.2435	0.2322
15	0.4086	0.3903	0.3729	0.3564	0.3405	0.3253	0.3108	0.2968	0.2834	0.2705	0.2581	0.2461	0.2346
16	0.4165	0.3975	0.3794	0.3622	0.3458	0.3302	0.3152	0.3008	0.2871	0.2739	0.2612	0.2490	0.2373
17	0.4255	0.4056	0.3867	0.3688	0.3518	0.3356	0.3201	0.3053	0.2911	0.2776	0.2646	0.2521	0.2401
18	0.4357	0.4146	0.3948	0.3761	0.3584	0.3415	0.3255	0.3102	0.2956	0.2817	0.2683	0.2555	0.2433
19	0.4473	0.4249	0.4039	0.3842	0.3657	0.3481	0.3315	0.3156	0.3006	0.2862	0.2724	0.2593	0.2467
20	0.4605	0.4365	0.4142	0.3934	0.3739	0.3555	0.3381	0.3216	0.3060	0.2911	0.2769	0.2634	0.2504
21	0.4758	0.4498	0.4259	0.4037	0.3830	0.3637	0.3455	0.3283	0.3120	0.2965	0.2818	0.2678	0.2545
22	0.4936	0.4651	0.4392	0.4154	0.3934	0.3729	0.3537	0.3356	0.3186	0.3025	0.2872	0.2727	0.2590
23	0.5147	0.4829	0.4545	0.4287	0.4050	0.3832	0.3628	0.3438	0.3259	0.3091	0.2932	0.2781	0.2638
24	0.5404	0.5041	0.4724	0.4440	0.4183	0.3948	0.3731	0.3529	0.3341	0.3164	0.2997	0.2840	0.2692
25	0.5727	0.5299	0.4936	0.4619	0.4336	0.4081	0.3847	0.3631	0.3431	0.3245	0.3070	0.2905	0.2750

Vertical Wall with Inclined Backfill

($c'-\phi'$) backfill



$$\sigma'_a = \gamma z K_a = \gamma z K'_a \cos\alpha$$

$$Z_c = \frac{2c}{\gamma}\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

$$P_a = \frac{1}{2} \times (\gamma H K'_a \cos\alpha) \times (H - Z_c)$$

$$K'_a = \frac{1}{\cos^2\phi'}\left\{\frac{2\cos^2\alpha + 2\left(\frac{c'}{\gamma z}\right)\cos\phi'\sin\phi' - \sqrt{\left[4\cos^2\alpha(\cos^2\alpha - \cos^2\phi') + 4\left(\frac{c'}{\gamma z}\right)^2\cos^2\phi' + 8\left(\frac{c'}{\gamma z}\right)\cos^2\alpha\sin\phi'\cos\phi'\right]}}{}\right\} - 1$$

Table 16.4 gives the variation of K_a'

Vertical Wall with Inclined Backfill

TABLE 16.4 Values of K'_a

α (deg)	ϕ' (deg)	$c'/\gamma z$						
		0	0.025	0.05	0.075	0.1	0.2	0.5
0	15	0.5888	0.5504	0.5121	0.4737	0.4353	0.2819	−0.1785
0	20	0.4903	0.4553	0.4203	0.3853	0.3502	0.2102	−0.2099
0	25	0.4059	0.3740	0.3422	0.3103	0.2784	0.1510	−0.2312
0	30	0.3333	0.3045	0.2756	0.2467	0.2179	0.1024	−0.2440
0	35	0.2710	0.2450	0.2189	0.1929	0.1669	0.0628	−0.2496
0	40	0.2174	0.1941	0.1708	0.1475	0.1242	0.0309	−0.2489
5	15	0.6069	0.5658	0.5252	0.4849	0.4449	0.2867	−0.1804
5	20	0.5015	0.4650	0.4287	0.3925	0.3565	0.2133	−0.2119
5	25	0.4133	0.3805	0.3478	0.3152	0.2826	0.1530	−0.2332
5	30	0.3385	0.3090	0.2795	0.2501	0.2207	0.1036	−0.2460
5	35	0.2746	0.2481	0.2217	0.1952	0.1688	0.0634	−0.2515
5	40	0.2200	0.1964	0.1727	0.1491	0.1255	0.0312	−0.2507
10	15	0.6738	0.6206	0.5707	0.5230	0.4769	0.3022	−0.1861
10	20	0.5394	0.4974	0.4564	0.4162	0.3767	0.2230	−0.2180
10	25	0.4376	0.4015	0.3660	0.3308	0.2960	0.1590	−0.2394
10	30	0.3549	0.3233	0.2919	0.2607	0.2297	0.1072	−0.2522
10	35	0.2861	0.2581	0.2303	0.2025	0.1749	0.0654	−0.2575
10	40	0.2282	0.2034	0.1787	0.1541	0.1296	0.0321	−0.2564
15	15	1.0000	0.7762	0.6834	0.6102	0.5464	0.3324	−0.1962
15	20	0.6241	0.5666	0.5137	0.4639	0.4165	0.2411	−0.2287
15	25	0.4860	0.4428	0.4011	0.3606	0.3211	0.1700	−0.2503
15	30	0.3861	0.3502	0.3150	0.2803	0.2462	0.1137	−0.2628
15	35	0.3073	0.2764	0.2459	0.2158	0.1860	0.0690	−0.2678
15	40	0.2429	0.2161	0.1895	0.1632	0.1370	0.0338	−0.2662
20	20	1.0000	0.7432	0.6403	0.5608	0.4927	0.2718	−0.2449
20	25	0.5820	0.5207	0.4650	0.4133	0.3645	0.1879	−0.2665
20	30	0.4408	0.3964	0.3539	0.3130	0.2734	0.1241	−0.2787
20	35	0.3423	0.3064	0.2713	0.2371	0.2035	0.0746	−0.2831
20	40	0.2665	0.2363	0.2066	0.1773	0.1484	0.0363	−0.2807

EXAMPLE 16.4

EXAMPLE 16.4

Refer to the retaining wall in Figure 16.10. The backfill is granular soil. Given:

Wall: $H = 3.05$ m

$\theta = +10°$

Backfill: $\alpha = 15°$

$\phi' = 35°$

$c' = 0$

$\gamma = 17.29$ kN/m^3

Determine the Rankine active force, P_a, and its location and direction.

SOLUTION

From Table 16.1, for $\alpha = 15°$ and $\theta = +10°$, the value of $K_{a(R)} = 0.42$. From Eq. (12.16),

$$P_a = \frac{1}{2}\gamma H^2 K_{a(R)} = \left(\frac{1}{2}\right)(17.29)(3.05)^2(0.42) = \mathbf{33.78\ kN/m}$$

Again, from Table 16.2, for $\alpha = 15°$ and $\theta = +10°$, $\beta'_a = \mathbf{30.5°}$.

The force P_a will act at a distance of $3.05/3 = 1.02$ m above the bottom of the wall and will be inclined at an angle of $+30.5°$ to the normal drawn to the backface of the wall.

EXAMPLE 16.5

EXAMPLE 16.5

For the retaining wall shown in Figure 16.11, $H = 7.5$ m, $\gamma = 18$ kN/m^3, $\phi' = 20°$, $c' = 13.5$ kN/m^2, and $\alpha = 10°$. Calculate the Rankine active force, P_a, per unit length of the wall and the location of the resultant force after the occurrence of the tensile crack.

SOLUTION

From Eq. (16.34),

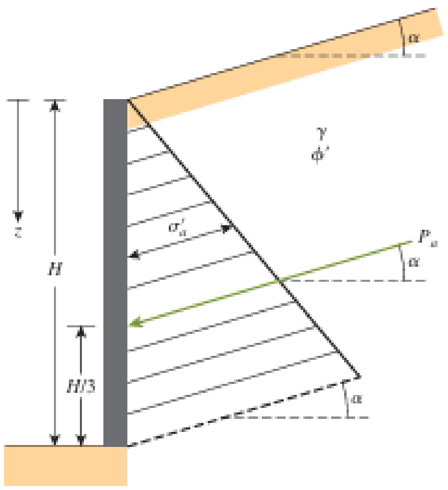
$$z_r = \frac{2c'}{\gamma}\sqrt{\frac{1+\sin\phi'}{1-\sin\phi'}} = \frac{(2)(13.5)}{18}\sqrt{\frac{1+\sin 20}{1-\sin 20}} = 2.14 \text{ m}$$

At $z = 7.5$ m,

$$\frac{c'}{\gamma z} = \frac{13.5}{(18)(7.5)} = 0.1$$

From Table 16.4, for $\phi' = 20°$, $c'/\gamma z = 0.1$, and $\alpha = 10°$, the value of $K_a' = 0.377$, so at $z = 7.5$ m,

$$\sigma_a' = \gamma z K_a' \cos\alpha = (18)(7.5)(0.377)(\cos 10) = 50.1 \text{ kN/m}^2$$



EXAMPLE 16.5

After the occurrence of the tensile crack, the pressure distribution on the wall will be as shown in Figure 16.13, so

$$P_a = \left(\frac{1}{2}\right)(50.1)(7.5 - 2.14) = \mathbf{134.3\ kN/m}$$

and

$$\bar{z} = \frac{7.5 - 2.14}{3} = \mathbf{1.79\ m}$$

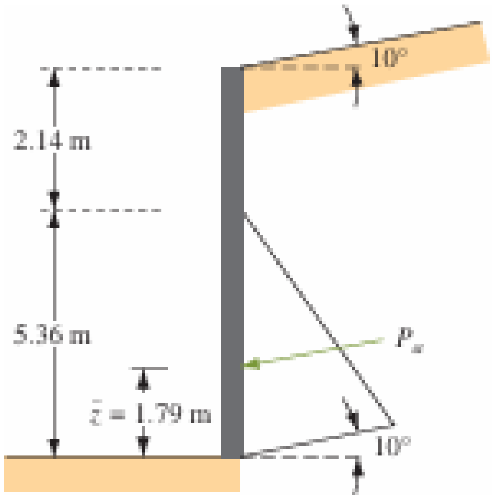
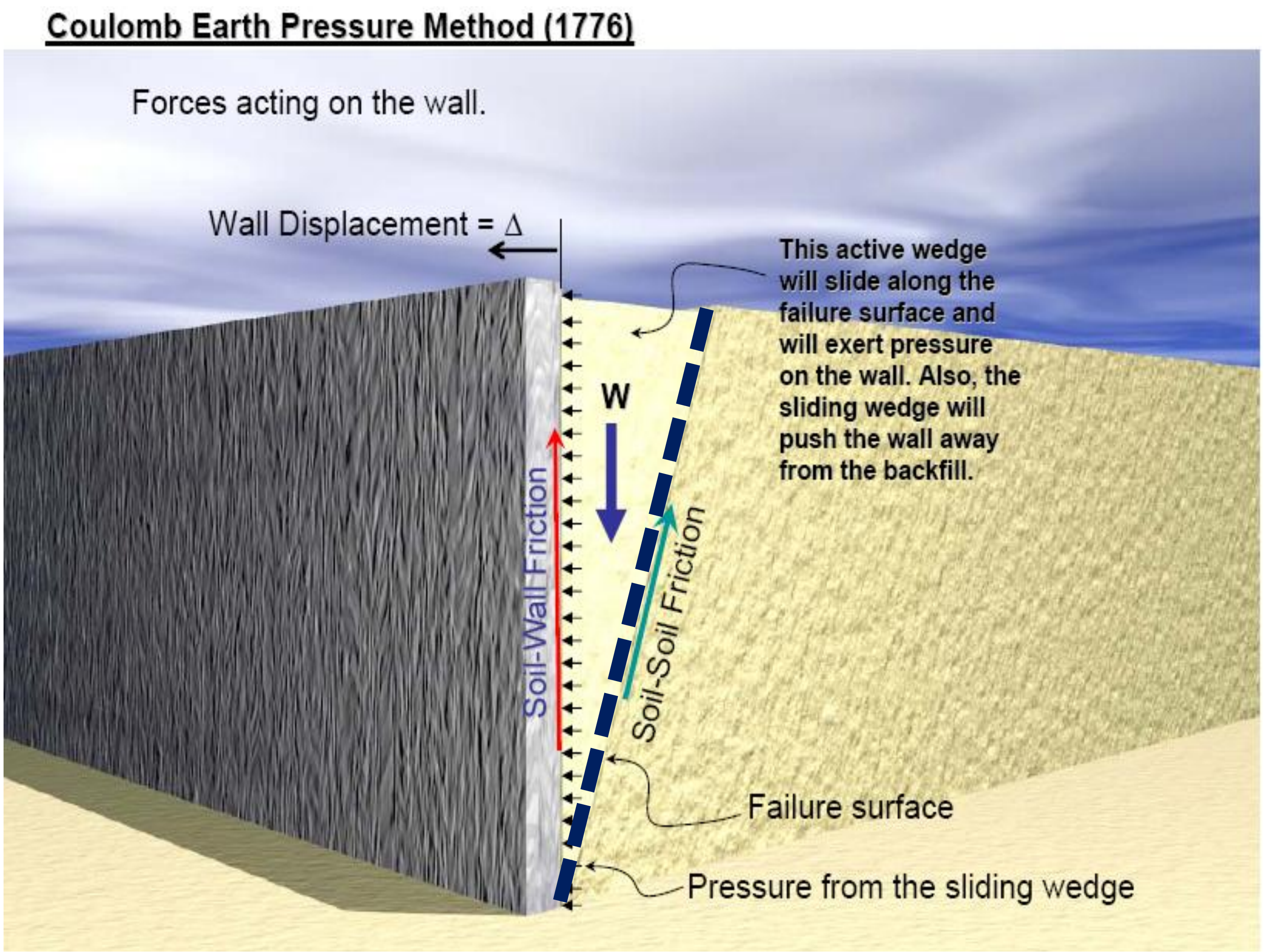
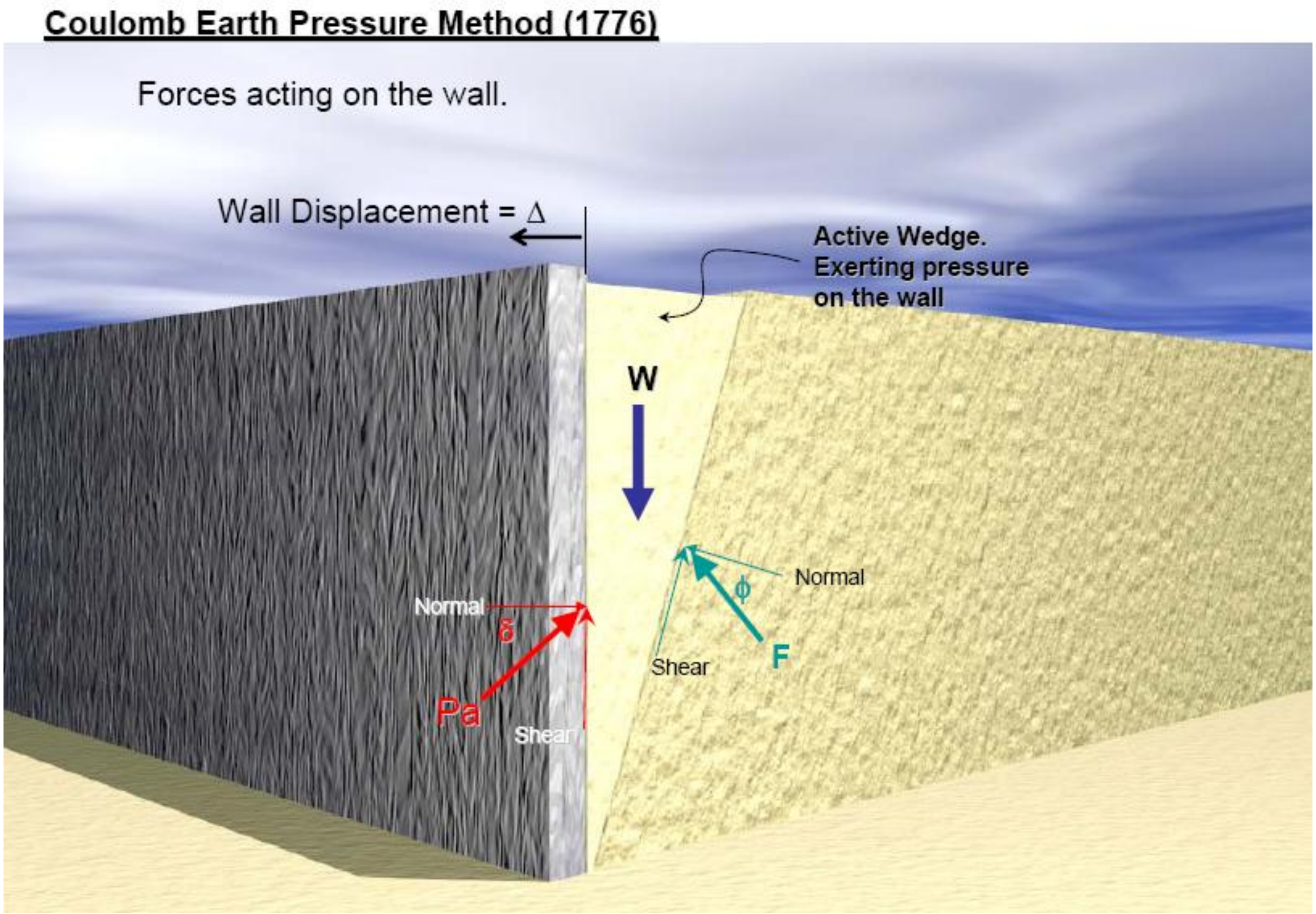


FIGURE 16.13 Calculation of Rankine active force, $c' - \phi'$ soil

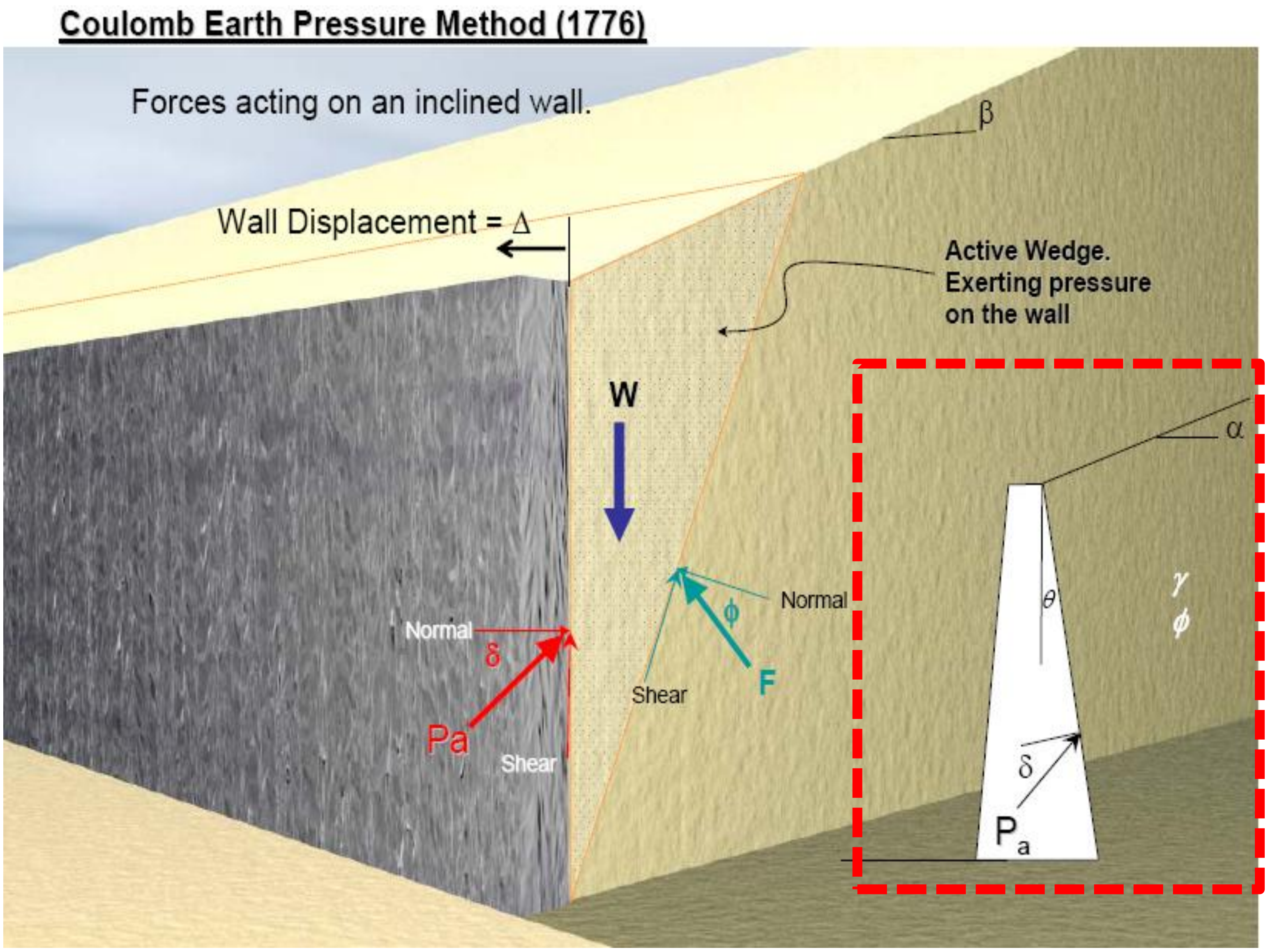
COULOMB'S EARTH PRESSURE



COULOMB'S EARTH PRESSURE



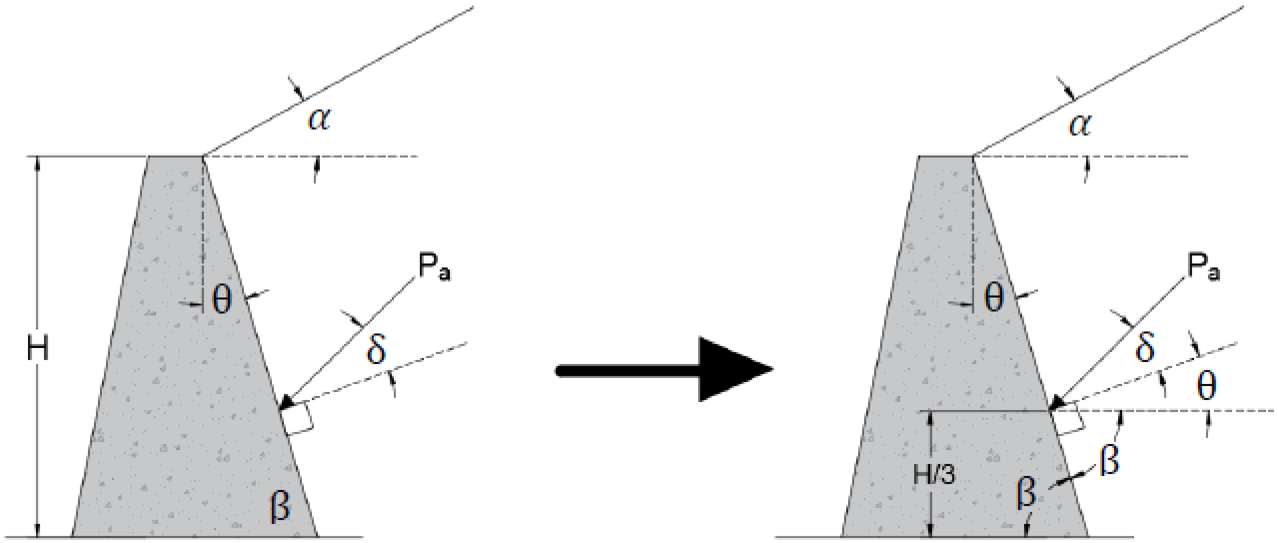
COULOMB'S EARTH PRESSURE



Coulomb's Active Earth Pressure

Granular backfill (c' =0)

General case (inclined wall and inclined backfill):



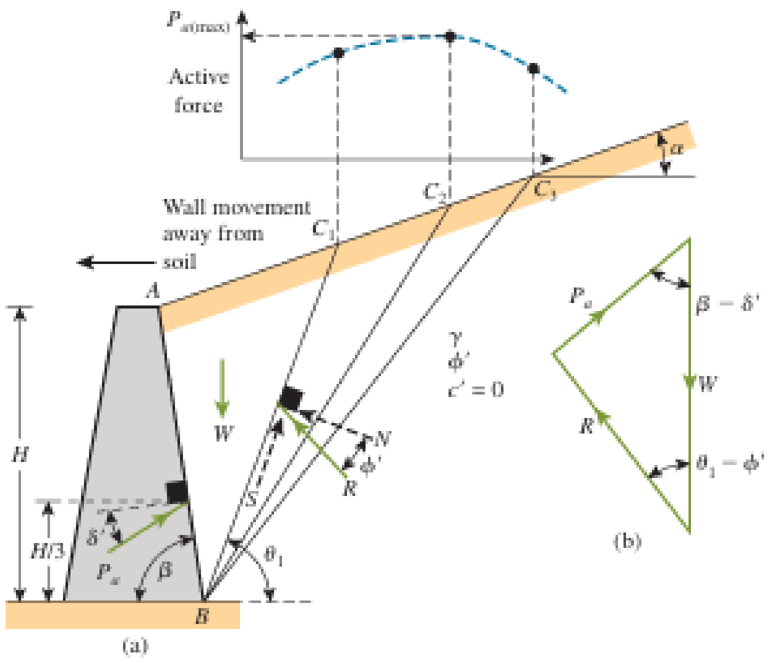
α = inclination of backfill with horizontal

θ = inclination of wall with vertical

β = inclination of wall with the horizontal

δ = friction angle between soil and wall

Coulomb's Active Earth Pressure



The forces acting on this wedge (per unit length at right angles to the cross section shown) are as follows:

1. The weight of the wedge, W.
2. The resultant, R, of the normal and resisting shear forces along the surface, BC_1. The force R will be inclined at an angle ϕ' to the normal drawn to BC_1.
3. The active force per unit length of the wall, P_a, which will be inclined at an angle δ' to the normal drawn to the backface of the wall.

Coulomb's Active Earth Pressure

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

K_a = Coulomb's active earth pressure coefficient

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

When α = 0°, β =90°, δ' = 0°, Coulomb's active earth pressure coefficient becomes equal to (1-sin ϕ')/(1+ sin ϕ'), which is the same as Rankine's active earth pressure coefficient.

Coulomb's Active Earth Pressure

K_a = Coulomb's active earth pressure coefficient

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

The values of the active earth pressure coefficient, K_a for a vertical retaining wall , β =90°
with horizontal backfill α = 0°
are given in Table 16.5

Coulomb's Active Earth Pressure

TABLE 16.5 Values of K_a [Eq. (16.36)] for $\beta = 90°$ and $\alpha = 0°$

	δ' (deg)					
ϕ' (deg)	0	5	10	15	20	25
28	0.3610	0.3448	0.3330	0.3251	0.3203	0.3186
30	0.3333	0.3189	0.3085	0.3014	0.2973	0.2956
32	0.3073	0.2945	0.2853	0.2791	0.2755	0.2745
34	0.2827	0.2714	0.2633	0.2579	0.2549	0.2542
36	0.2596	0.2497	0.2426	0.2379	0.2354	0.2350
38	0.2379	0.2292	0.2230	0.2190	0.2169	0.2167
40	0.2174	0.2098	0.2045	0.2011	0.1994	0.1995
42	0.1982	0.1916	0.1870	0.1841	0.1828	0.1831

Coulomb's Active Earth Pressure

K_a = Coulomb's active earth pressure coefficient

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

The wall friction angle δ' can be determined in the laboratory by means of direct shear test. It is assumed to be between ϕ'/2 and 2ϕ'/3

angle δ' = 2ϕ'/3   Table 16.6

angle δ' = ϕ'/2   Table 16.7

Coulomb's Active Earth Pressure

TABLE 16.6 Values of K_a [from Eq. (16.36)] for $\delta' = \frac{2}{3}\phi'$

α (deg)	ϕ' (deg)	β (deg) 90	85	80	75	70	65
0	28	0.3213	0.3588	0.4007	0.4481	0.5026	0.5662
	29	0.3091	0.3467	0.3886	0.4362	0.4908	0.5547
	30	0.2973	0.3349	0.3769	0.4245	0.4794	0.5435
	31	0.2860	0.3235	0.3655	0.4133	0.4682	0.5326
	32	0.2750	0.3125	0.3545	0.4023	0.4574	0.5220
	33	0.2645	0.3019	0.3439	0.3917	0.4469	0.5117
	34	0.2543	0.2916	0.3335	0.3813	0.4367	0.5017
	35	0.2444	0.2816	0.3235	0.3713	0.4267	0.4919
	36	0.2349	0.2719	0.3137	0.3615	0.4170	0.4824
	37	0.2257	0.2626	0.3042	0.3520	0.4075	0.4732
	38	0.2168	0.2535	0.2950	0.3427	0.3983	0.4641
	39	0.2082	0.2447	0.2861	0.3337	0.3894	0.4553
	40	0.1998	0.2361	0.2774	0.3249	0.3806	0.4468
	41	0.1918	0.2278	0.2689	0.3164	0.3721	0.4384
	42	0.1840	0.2197	0.2606	0.3080	0.3637	0.4302
5	28	0.3431	0.3845	0.4311	0.4843	0.5461	0.6190
	29	0.3295	0.3709	0.4175	0.4707	0.5325	0.6056
	30	0.3165	0.3578	0.4043	0.4575	0.5194	0.5926
	31	0.3039	0.3451	0.3916	0.4447	0.5067	0.5800
	32	0.2919	0.3329	0.3792	0.4324	0.4943	0.5677
	33	0.2803	0.3211	0.3673	0.4204	0.4823	0.5558
	34	0.2691	0.3097	0.3558	0.4088	0.4707	0.5443
	35	0.2583	0.2987	0.3446	0.3975	0.4594	0.5330
	36	0.2479	0.2881	0.3338	0.3866	0.4484	0.5221
	37	0.2379	0.2778	0.3233	0.3759	0.4377	0.5115
	38	0.2282	0.2679	0.3131	0.3656	0.4273	0.5012
	39	0.2188	0.2582	0.3033	0.3556	0.4172	0.4911
	40	0.2098	0.2489	0.2937	0.3458	0.4074	0.4813
	41	0.2011	0.2398	0.2844	0.3363	0.3978	0.4718
	42	0.1927	0.2311	0.2753	0.3271	0.3884	0.4625

Coulomb's Active Earth Pressure

TABLE 16.7 Values of K_a [from Eq. (16.36)] for $\delta' = \frac{1}{2}\phi'$

		β (deg)					
α (deg)	φ′ (deg)	90	85	80	75	70	65
0	28	0.3264	0.3629	0.4034	0.4490	0.5011	0.5616
	29	0.3137	0.3502	0.3907	0.4363	0.4886	0.5492
	30	0.3014	0.3379	0.3784	0.4241	0.4764	0.5371
	31	0.2896	0.3260	0.3665	0.4121	0.4645	0.5253
	32	0.2782	0.3145	0.3549	0.4005	0.4529	0.5137
	33	0.2671	0.3033	0.3436	0.3892	0.4415	0.5025
	34	0.2564	0.2925	0.3327	0.3782	0.4305	0.4915
	35	0.2461	0.2820	0.3221	0.3675	0.4197	0.4807
	36	0.2362	0.2718	0.3118	0.3571	0.4092	0.4702
	37	0.2265	0.2620	0.3017	0.3469	0.3990	0.4599
	38	0.2172	0.2524	0.2920	0.3370	0.3890	0.4498
	39	0.2081	0.2431	0.2825	0.3273	0.3792	0.4400
	40	0.1994	0.2341	0.2732	0.3179	0.3696	0.4304
	41	0.1909	0.2253	0.2642	0.3087	0.3602	0.4209
	42	0.1828	0.2168	0.2554	0.2997	0.3511	0.4177
5	28	0.3477	0.3879	0.4327	0.4837	0.5425	0.6115
	29	0.3337	0.3737	0.4185	0.4694	0.5282	0.5972
	30	0.3202	0.3601	0.4048	0.4556	0.5144	0.5833
	31	0.3072	0.3470	0.3915	0.4422	0.5009	0.5698
	32	0.2946	0.3342	0.3787	0.4292	0.4878	0.5566
	33	0.2825	0.3219	0.3662	0.4166	0.4750	0.5437
	34	0.2709	0.3101	0.3541	0.4043	0.4626	0.5312
	35	0.2596	0.2986	0.3424	0.3924	0.4505	0.5190
	36	0.2488	0.2874	0.3310	0.3808	0.4387	0.5070
	37	0.2383	0.2767	0.3199	0.3695	0.4272	0.4954
	38	0.2282	0.2662	0.3092	0.3585	0.4160	0.4840
	39	0.2185	0.2561	0.2988	0.3478	0.4050	0.4729
	40	0.2090	0.2463	0.2887	0.3374	0.3944	0.4620
	41	0.1999	0.2368	0.2788	0.3273	0.3840	0.4514
	42	0.1911	0.2276	0.2693	0.3174	0.3738	0.4410

Coulomb's Active Earth Pressure

If a uniform surcharge of intensity *q* is located above the backfill

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

$$\gamma_{eq} = \gamma + \left[\frac{\sin\beta}{\sin(\beta + \alpha)}\right]\left(\frac{2q}{H}\right)$$

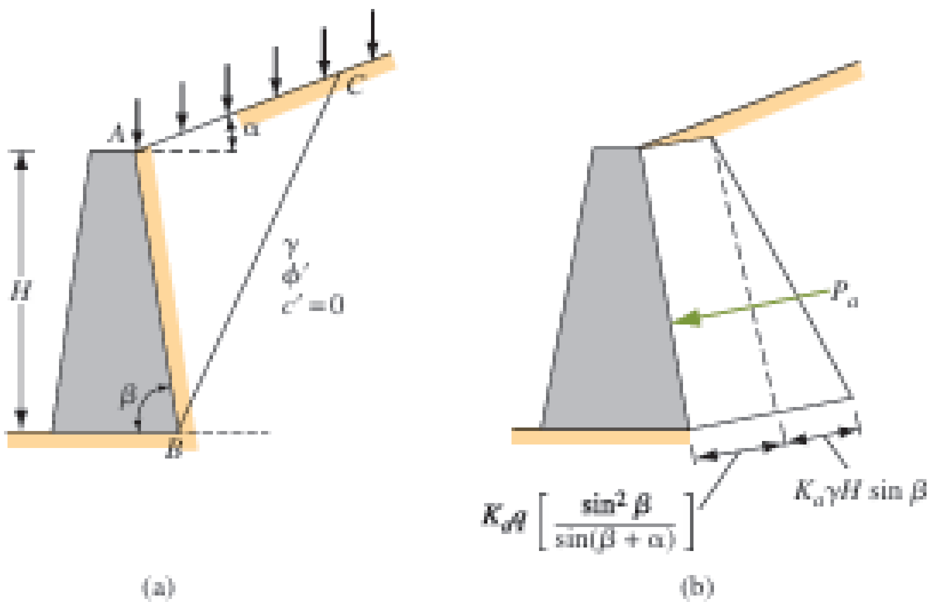


FIGURE 16.15 Coulomb's active pressure with a surcharge on the backfill

EXAMPLE 16.6

EXAMPLE 16.6

Consider the retaining wall shown in Figure 16.14a. Given: $H = 5$ m; unit weight of soil = 17.6 kN/m^3; angle of friction of soil = 35°; wall friction angle, $\delta' = \frac{2}{3}\phi'$; soil cohesion, $c' = 0$; $\alpha = 0$; and $\beta = 90°$. Calculate the Coulomb's active force per unit length of the wall.

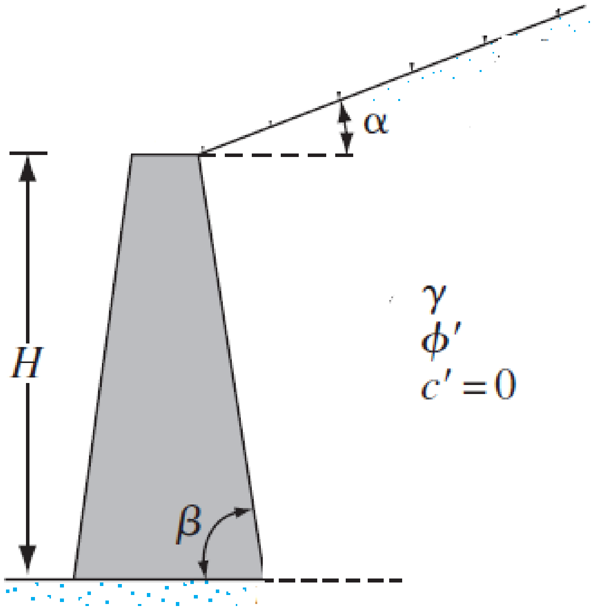
SOLUTION

From Eq. (16.35),

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

From Table 16.6, for $\alpha = 0°$, $\beta = 90°$, $\phi' = 35°$, and $\delta' = \frac{2}{3}\phi' = 23.33°$, $K_a = 0.2444$. Hence,

$$P_a = \frac{1}{2}(17.6)(5)^2(0.2444) = \mathbf{53.77\ kN/m}$$



EXAMPLE 16.7

EXAMPLE 16.7

Refer to Figure 16.15a. Given: $H = 6.1$ m, $\phi' = 30°$, $\delta' = 20°$, $\alpha = 5°$, $\beta = 85°$, $q = 96$ kN/m², and $\gamma = 18$ kN/m³. Determine Coulomb's active force and the location of the line of action of the resultant P_a.

SOLUTION

For $\beta = 85°$, $\alpha = 5°$, $\delta' = 20°$, $\phi' = 30°$, and $K_a = 0.3578$ (Table 16.6). From Eqs. (16.37) and (16.38),

$$P_a = \frac{1}{2}K_a\gamma_{eq}H^2 = \frac{1}{2}K_a\left[\gamma + \frac{2q}{H}\frac{\sin\beta}{\sin(\beta+\alpha)}\right]H^2 = \underbrace{\frac{1}{2}K_a\gamma H^2}_{P_{a(1)}}$$

$$+ \underbrace{K_aHq\left[\frac{\sin\beta}{\sin(\beta+\alpha)}\right]}_{P_{a(2)}}$$

$$= (0.5)(0.3578)(18)(6.1)^2 + (0.3578)(6.1)(96)\left[\frac{\sin 85}{\sin(85+5)}\right]$$

$$= 119.8 + 208.7 = \mathbf{328.5\ kN/m}$$

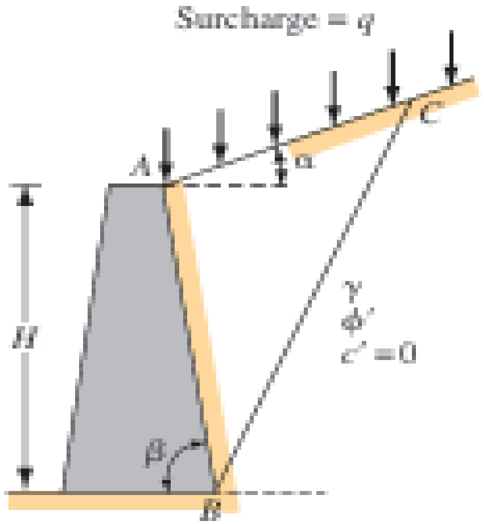
Location of the line of action of the resultant:

$$P_a\bar{z} = P_{a(1)}\frac{H}{3} + P_{a(2)}\frac{H}{2}$$

or

$$\bar{z} = \frac{(119.8)\left(\frac{6.1}{3}\right) + (208.7)\left(\frac{6.1}{2}\right)}{328.5}$$

$$= \mathbf{2.68\ m} \text{ (measured vertically from the bottom of the wall)}$$

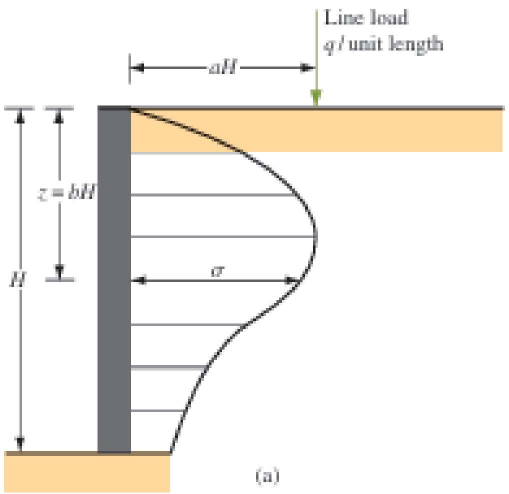


Lateral Earth Pressure Due to Surcharge

Line load of intensity *q*/unit length

$$\sigma = \frac{2q}{\pi H}\frac{a^2 b}{(a^2 + b^2)^2}$$

σ = horizontal stress at depth $z = bH$



Because soil is not a perfectly elastic medium. The modified forms of the equation above are :

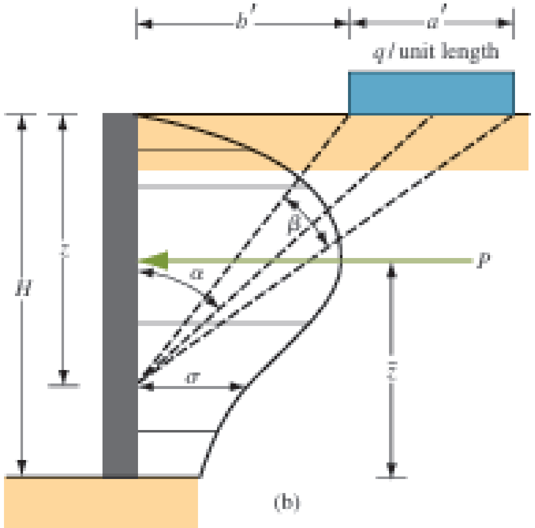
$$\sigma = \frac{4a}{\pi H}\frac{a^2 b}{(a^2 + b^2)} \qquad \text{for } a > 0.4$$

$$\sigma = \frac{q}{H}\frac{0.203b}{(0.16 + b^2)^2} \qquad \text{for } a \le 0.4$$

Lateral Earth Pressure Due to Surcharge

Strip load of intensity *q*/unit area

$$\sigma = \frac{2q}{\pi}(\beta - \sin\beta\cos 2\alpha)$$



The total force per unit length (P) due to the *strip loading only*

$$P = \frac{q}{90}[H(\theta_2 - \theta_1)]$$

$$\bar{z} = H - \left[\frac{H^2(\theta_2 - \theta_1) + (R - Q) - 57.3a'H}{2H(\theta_2 - \theta_1)}\right]$$

$$\theta_1 = \tan^{-1}\left(\frac{b'}{H}\right)(\text{deg})$$

$$\theta_2 = \tan^{-1}\left(\frac{a' + b'}{H}\right)(\text{deg})$$

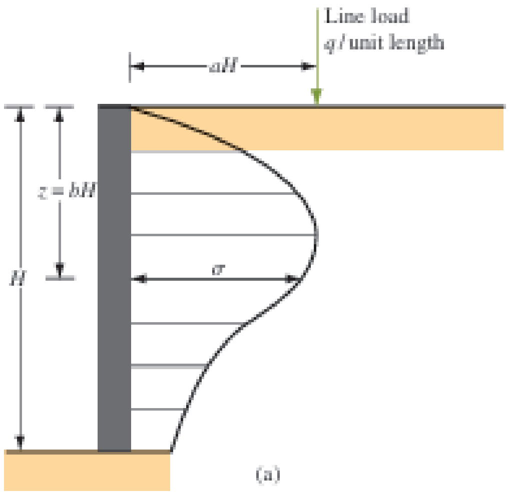
$$R = (a' + b')^2(90 - \theta_2)$$

$$Q = b'^2(90 - \theta_1)$$

EXAMPLE 16.8

EXAMPLE 16.8

Refer to Figure 16.16a, which shows a line load surcharge. Given: $H = 6$ m, $a = 0.25$, and $q = 3$ kN/m. Calculate the variation of the lateral stress σ on the retaining structure at $z = 1, 2, 3, 4, 5,$ and 6 m.



SOLUTION

For $a = 0.25$, which is less than 0.4, we will use Eq. (16.41). Now the following table can be prepared.

z (m)	H (m)	$b = z/H$	a	σ (kN/m^2)
1	6	0.167	0.25	**0.48**
2	6	0.333	0.25	**0.46**
3	6	0.5	0.25	**0.302**
4	6	0.667	0.25	**0.185**
5	6	0.833	0.25	**0.116**
6	6	1	0.25	**0.073**

$$\sigma = \frac{q}{H}\frac{0.203b}{(0.16 + b^2)^2} \qquad \text{for } a \leq 0.4$$

EXAMPLE 16.9

EXAMPLE 16.9

Refer to Figure 16.16b. Here, $a' = 2$ m, $b' = 1$ m, $q = 40$ kN/m^2, and $H = 6$ m. Determine the total force on the wall (kN/m) caused by the strip loading only.

SOLUTION

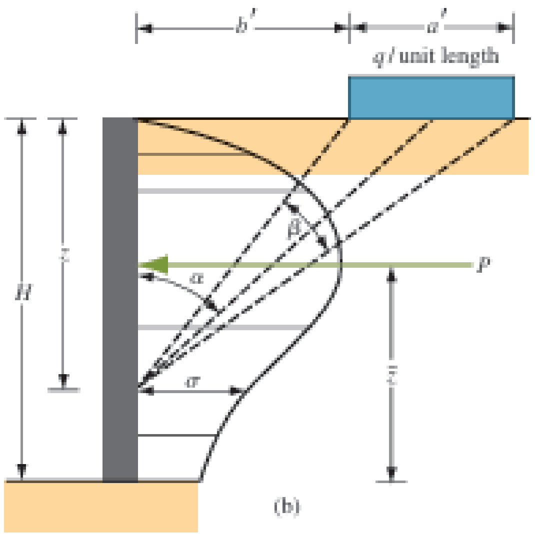
From Eqs. (16.45) and (16.46),

$$\theta_1 = \tan^{-1}\left(\frac{1}{6}\right) = 9.46^\circ$$

$$\theta_2 = \tan^{-1}\left(\frac{2+1}{6}\right) = 26.57^\circ$$

From Eq. (16.44),

$$P = \frac{q}{90}[H(\theta_2 - \theta_1)] = \frac{40}{90}[6(26.57 - 9.46)] = \mathbf{45.63\ kN/m}$$



(b)

EXAMPLE 16.10

EXAMPLE 16.10

Refer to Example 16.9. Determine the location of the resultant $\bar{z}$.

SOLUTION

From Eqs. (16.48) and (16.49),

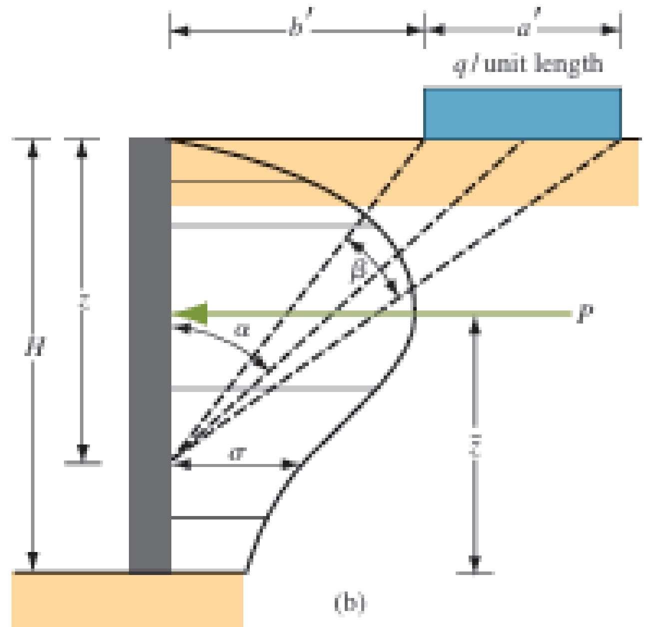
$$R = (a' + b')^2(90 - \theta_2) = (2 + 1)^2(90 - 26.57) = 570.87$$

$$Q = b'^2(90 - \theta_1) = (1)^2(90 - 9.46) = 80.54$$

From Eq. (16.47),

$$\bar{z} = H - \left[\frac{H^2(\theta_2 - \theta_1) + (R - Q) - 57.3a'H}{2H(\theta_2 - \theta_1)}\right]$$

$$= 6 - \left[\frac{(6)^2(26.57 - 9.46) + (570.87 - 80.54) - (57.3)(2)(6)}{(2)(6)(26.57 - 9.46)}\right] = \mathbf{3.96\ m}$$



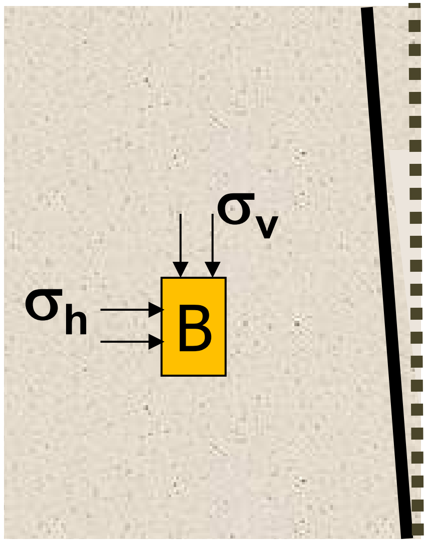
(b)

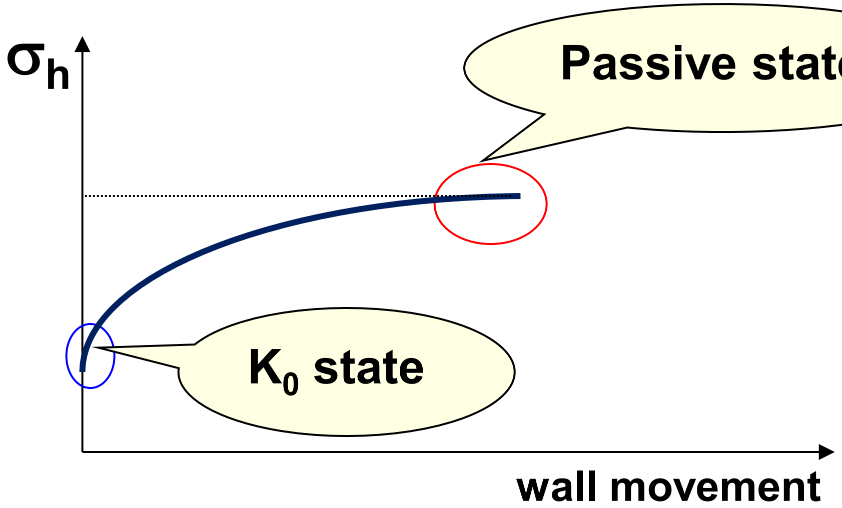
Passive Earth Pressure

Passive earth pressure

- Initially, soil is in K_0 state.
- As the wall moves towards (pushed into) the soil mass,
- σ_v remains the same, and
- σ_h increases till failure occurs. $\sigma_h \rightarrow \sigma_p$

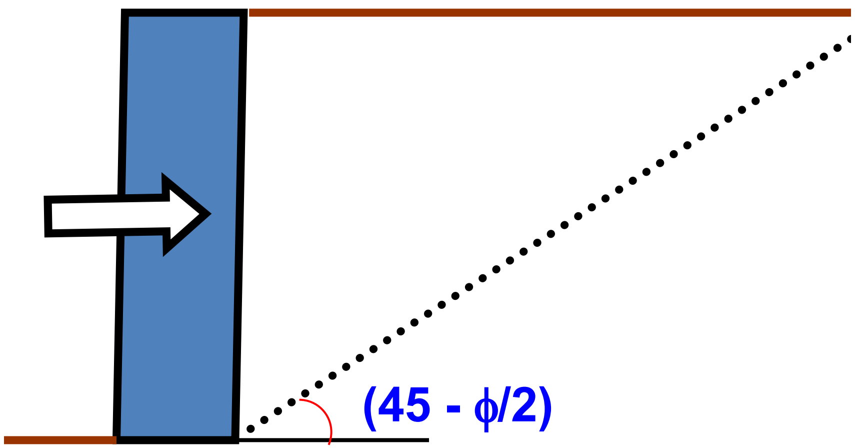
Passive state

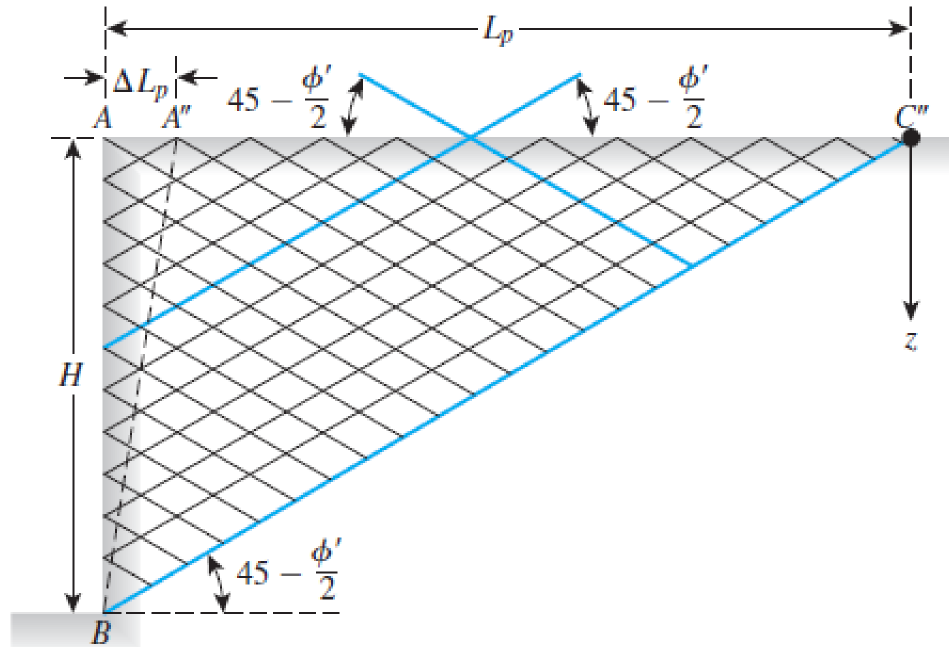




Orientation of Failure Planes

- From Mohr Circle the failure planes in the soil make ± (45 - ϕ/2)-degree angles with the direction of the major principal plane—that is, the horizontal.
- These are called potential *slip planes.*

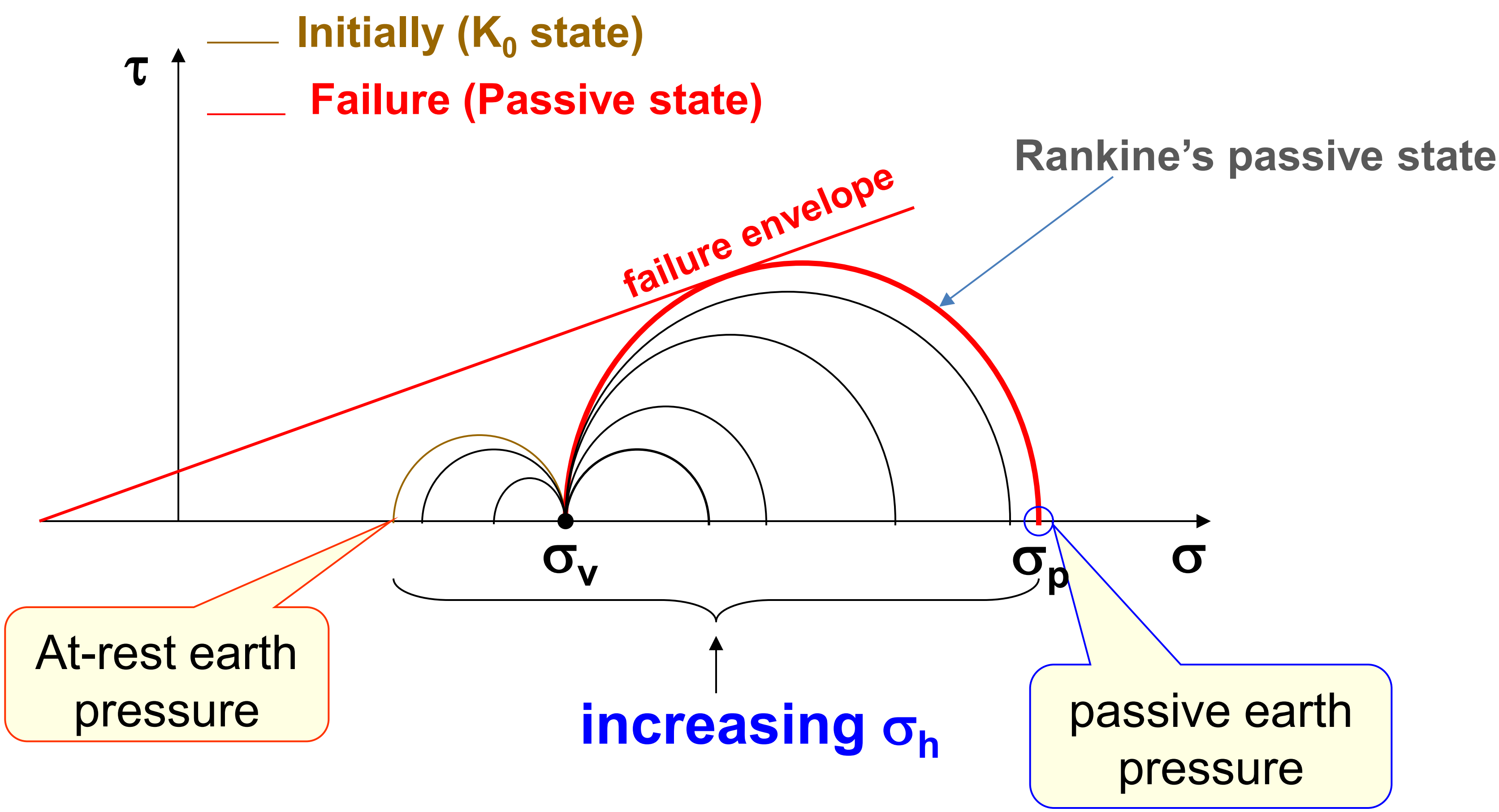




- Because the slip planes make angles of (45 - ϕ/2) degrees with the major principal plane, the soil mass in the state of plastic equilibrium is bounded by the plane *BC'*. The soil inside the zone *ABC"* <u>undergoes the same unit deformation in the horizontal direction everywhere, which is equal to</u> $\Delta L_p / L_p$.

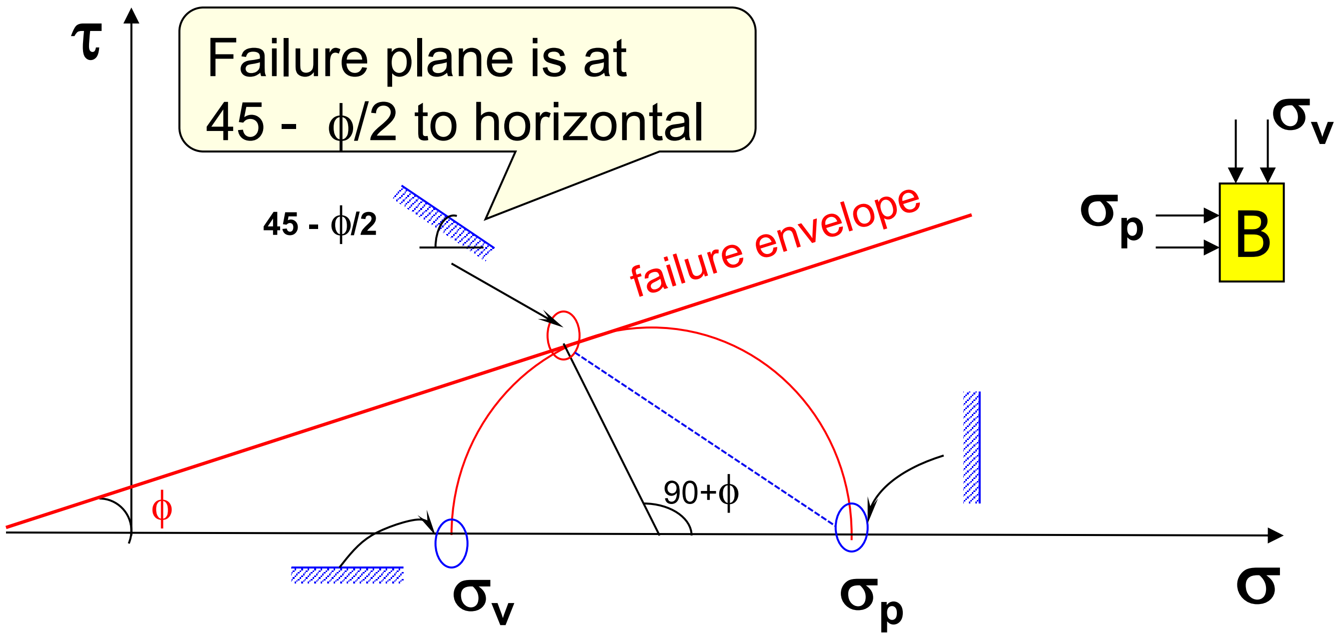
Passive Earth Pressure

- **As the wall moves towards the soil,**

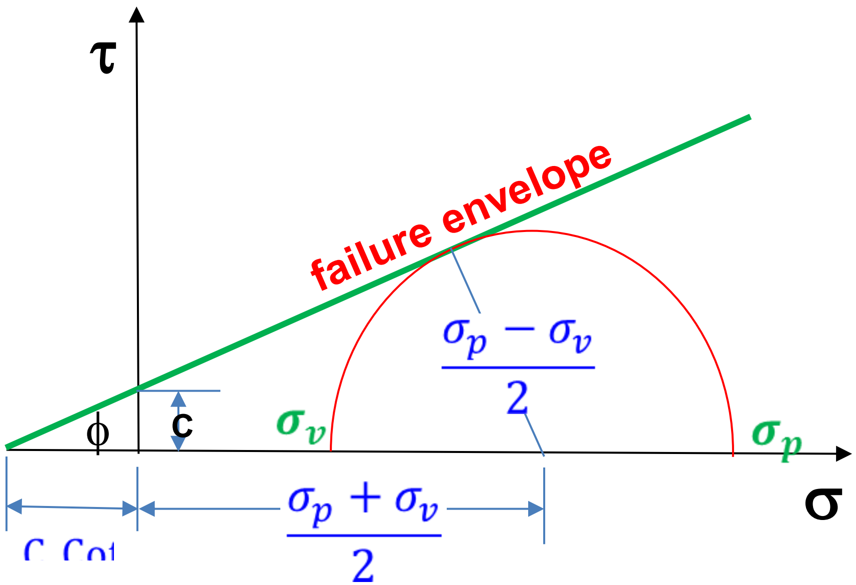


Passive Earth Pressure

Failure plane



Passive Earth Pressure



$$\boldsymbol{Sin\,\emptyset = \frac{\frac{\sigma_p - \sigma_v}{2}}{\mathrm{C.\,Cot}\,\emptyset + \frac{\sigma_p + \sigma_v}{2}}}$$

$$\sigma'_p = \sigma'_o \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

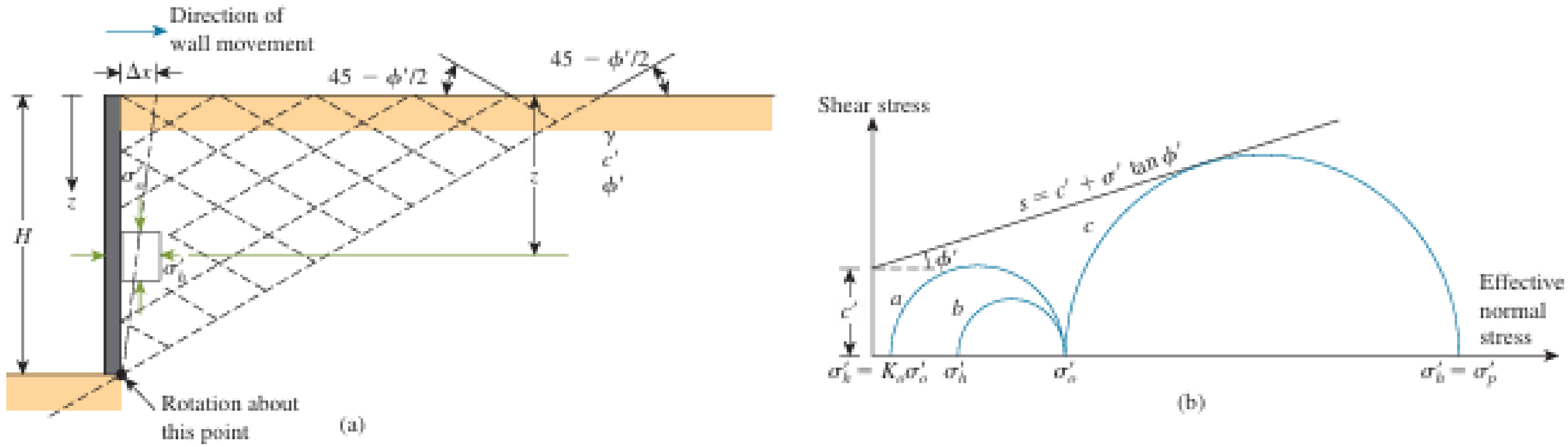
K_p = Rankine passive earth pressure coefficient

$$= \tan^2\left(45 + \frac{\phi'}{2}\right)$$

$$\sigma'_p = \sigma'_o K_p + 2c'\sqrt{K_p}$$

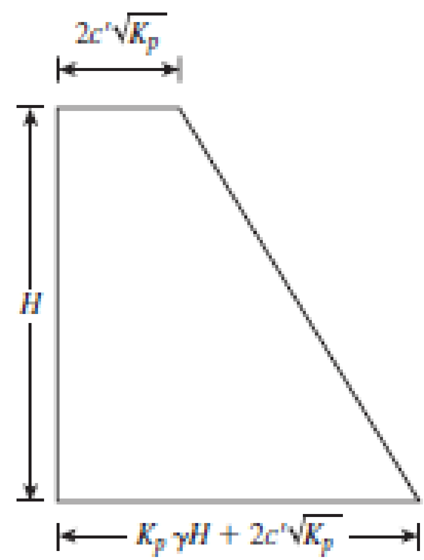
$$P_p = \tfrac{1}{2}\gamma H^2 K_p + 2c' H\sqrt{K_p}$$

Passive Earth Pressure



$$\sigma'_p = \sigma'_o K_p + 2c'\sqrt{K_p}$$

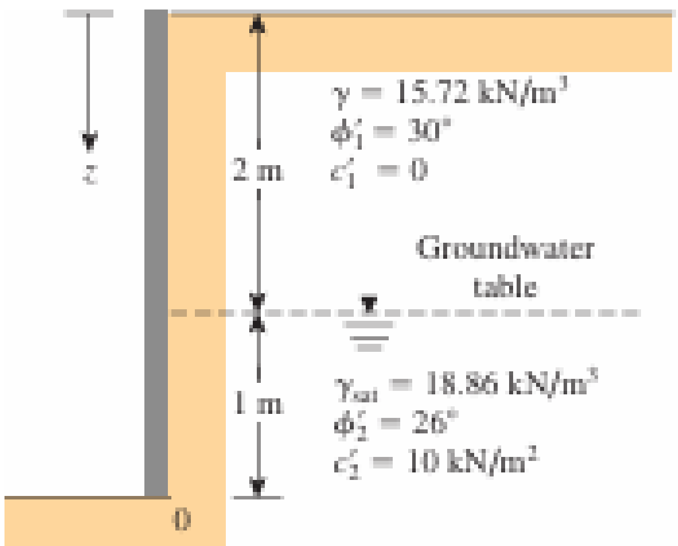
$$P_p = \frac{1}{2}\gamma H^2 K_p + 2c' H\sqrt{K_p}$$



EXAMPLE 16.13

EXAMPLE 16.13

A 3 m high wall is shown in Figure 16.22a. Determine the Rankine passive force per unit length of the wall.



(a)

EXAMPLE 16.13

SOLUTION

For the top layer,

$$K_{p(1)} = \tan^2\left(45 + \frac{\phi_1'}{2}\right) = \tan^2(45 + 15) = 3$$

From the bottom soil layer,

$$K_{p(2)} = \tan^2\left(45 + \frac{\phi_2'}{2}\right) = \tan^2(45 + 13) = 2.56$$

$$\sigma_p' = \sigma_o' K_p + 2c'\sqrt{K_p}$$

where

σ_o' = effective vertical stress

at $z = 0$, $\sigma_o' = 0$, $c_1' = 0$, $\sigma_p' = 0$

at $z = 2$ m, $\sigma_o' = (15.72)(2) = 31.44$ kN/m^2, $c_1' = 0$

So, for the top soil layer,

$$\sigma_p' = 31.44 K_{p(1)} + 2(0)\sqrt{K_{p(1)}} = 31.44(3) = 94.32 \text{ kN/m}^2$$

At this depth (that is, $z = 2$ m), for the bottom soil layer,

$$\sigma_p' = \sigma_o' K_{p(2)} + 2c_2'\sqrt{K_{p(2)}} = 31.44(2.56) + 2(10)\sqrt{2.56}$$
$$= 80.49 + 32 = 112.49 \text{ kN/m}^2$$

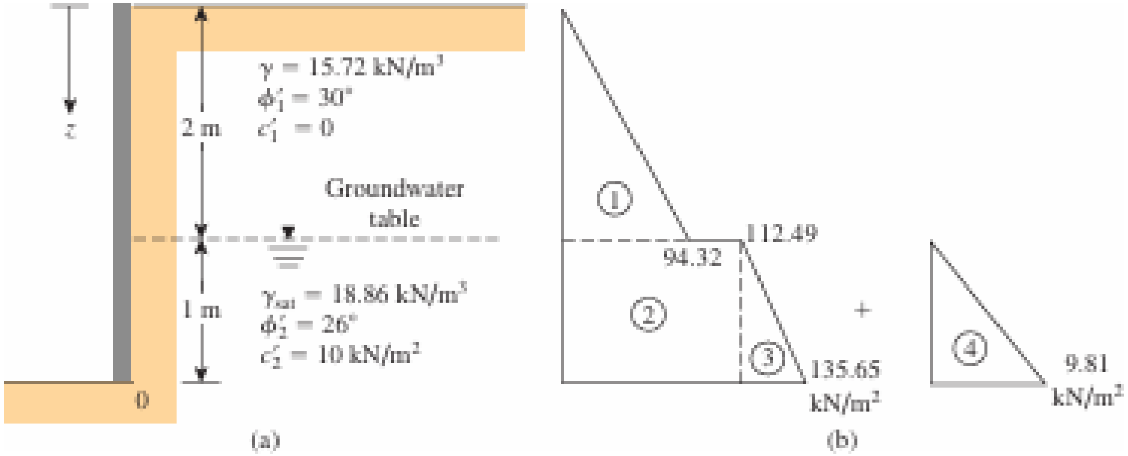
Again, at $z = 3$ m,

$$\sigma_o' = (15.72)(2) + (\gamma_{sat} - \gamma_w)(1)$$
$$= 31.44 + (18.86 - 9.81)(1) = 40.49 \text{ kN/m}^2$$

Hence,

$$\sigma_p' = \sigma_o' K_{p(2)} + 2c_2'\sqrt{K_{p(2)}} = 40.49(2.56) + (2)(10)(1.6)$$
$$= \mathbf{135.65 \text{ kN/m}^2}$$

EXAMPLE 16.13



Area no.	Area	
1	$\left(\frac{1}{2}\right)(2)(94.32)$	$= 94.32$
2	$(112.49)(1)$	$= 112.49$
3	$\left(\frac{1}{2}\right)(1)(135.65 - 112.49)$	$= 11.58$
4	$\left(\frac{1}{2}\right)(9.81)(1)$	$= 4.905$
		$P_p \approx 223.3$ kN/m

Rankine Passive Earth Pressure

Vertical Wall and Inclined Backfill

Granular backfill c' = 0

$$\sigma'_p = \gamma z K_p$$

$$P_p = \frac{1}{2}\gamma H^2 K_p$$

$$K_p = \cos\alpha \frac{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}}$$

Table 16.9

Rankine Passive Earth Pressure

TABLE 16.9 Passive Earth Pressure Coefficient K_p [from Eq. (16.70)]

↓α (deg)	ϕ' (deg) → 28	30	32	34	36	38	40
0	2.770	3.000	3.255	3.537	3.852	4.204	4.599
5	2.715	2.943	3.196	3.476	3.788	4.136	4.527
10	2.551	2.775	3.022	3.295	3.598	3.937	4.316
15	2.284	2.502	2.740	3.003	3.293	3.615	3.977
20	1.918	2.132	2.362	2.612	2.886	3.189	3.526
25	1.434	1.664	1.894	2.135	2.394	2.676	2.987

Rankine Passive Earth Pressure

Vertical Wall and Inclined Backfill

$c' - \phi'$ Soil

$$\sigma'_p = \gamma z K_p = \gamma z K'_p \cos\alpha$$

$$K'_p = \frac{1}{\cos^2\phi'}\left\{\frac{2\cos^2\alpha + 2\left(\frac{c'}{\gamma z}\right)\cos\phi'\sin\phi' + \sqrt{4\cos^2\alpha(\cos^2\alpha - \cos^2\phi') + 4\left(\frac{c'}{\gamma z}\right)^2\cos^2\phi' + 8\left(\frac{c'}{\gamma z}\right)\cos^2\alpha\sin\phi'\cos\phi'}}{}\right\} - 1$$

Table 16.10

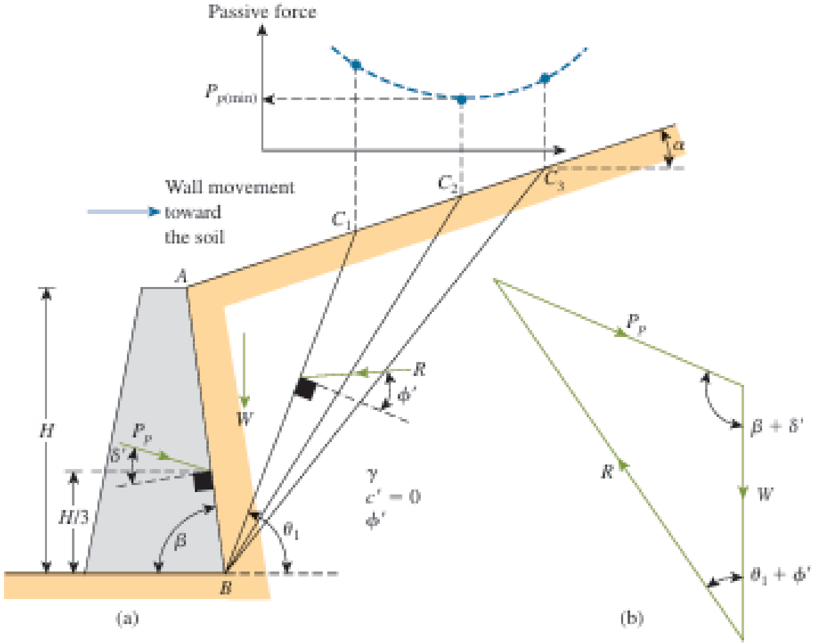
Rankine Passive Earth Pressure

TABLE 16.10 Values of K'_p

		$c'/\gamma z$						
α (deg)	ϕ' (deg)	0	0.025	0.05	0.075	0.1	0.2	0.5
0	15	1.6984	1.7636	1.8287	1.8939	1.9590	2.2197	3.0016
0	20	2.0396	2.1110	2.1824	2.2538	2.3252	2.6109	3.4678
0	25	2.4639	2.5424	2.6209	2.6994	2.7778	3.0918	4.0336
0	30	3.0000	3.0866	3.1732	3.2598	3.3464	3.6928	4.7321
0	35	3.6902	3.7862	3.8823	3.9783	4.0744	4.4586	5.6112
0	40	4.5989	4.7061	4.8134	4.9206	5.0278	5.4567	6.7434
5	15	1.6477	1.7156	1.7830	1.8501	1.9169	2.1823	2.9709
5	20	1.9940	2.0669	2.1396	2.2122	2.2846	2.5734	3.4353
5	25	2.4195	2.4989	2.5782	2.6575	2.7367	3.0529	3.9986
5	30	2.9543	3.0416	3.1288	3.2159	3.3030	3.6511	4.6935
5	35	3.6412	3.7378	3.8342	3.9307	4.0271	4.4126	5.5678
5	40	4.5445	4.6521	4.7597	4.8672	4.9747	5.4046	6.6935
10	15	1.4841	1.5641	1.6408	1.7153	1.7882	2.0700	2.8799
10	20	1.8539	1.9323	2.0097	2.0863	2.1622	2.4615	3.3392
10	25	2.2854	2.3680	2.4502	2.5320	2.6135	2.9370	3.8950

(continued)

Coulomb's Passive Earth Pressure



1. The weight of the wedge, W
2. The resultant, R, of the normal and shear forces on the plane BC_1
3. The passive force, P_p

$$P_p = \frac{1}{2}\gamma H^2 K_p$$

K_p = Coulomb's passive pressure coefficient

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

Coulomb’s Passive Earth Pressure

K_p = Coulomb’s passive pressure coefficient

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

TABLE 16.11 Values of K_p [from Eq. (16.74)] for $\beta = 90°$ and $\alpha = 0°$

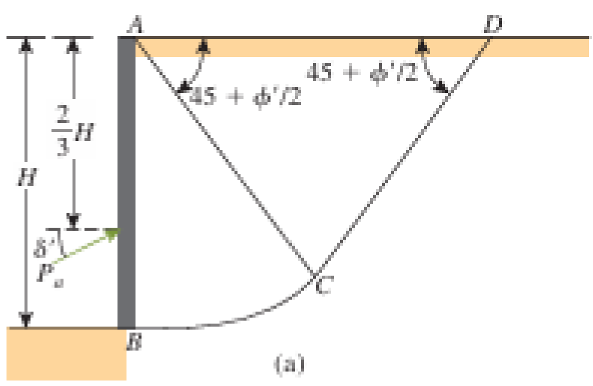
	δ' (deg)				
ϕ' (deg)	0	5	10	15	20
15	1.698	1.900	2.130	2.405	2.735
20	2.040	2.313	2.636	3.030	3.525
25	2.464	2.830	3.286	3.855	4.597
30	3.000	3.506	4.143	4.977	6.105
35	3.690	4.390	5.310	6.854	8.324
40	4.600	5.590	6.946	8.870	11.772

Comments on the Failure Surface Assumption for Coulomb's Pressure Calculations

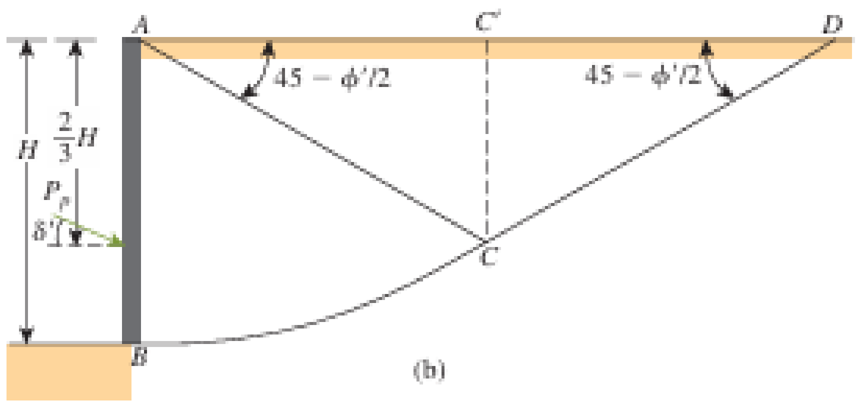
The fundamental assumption in Coulomb's pressure calculation methods for active and passive pressure is the acceptance of *plane failure surface.*

However, for walls with friction, this assumption does not hold in practice. The nature of *actual* failure surface in the soil mass for active and passive pressure is shown in Figure below, respectively (for a vertical wall with a horizontal backfill).

Note that the failure surface *BC* is curved and that the failure surface *CD* is a plane.



(a)



(b)

Comments on the Failure Surface Assumption for Coulomb's Pressure Calculations

Although the actual failure surface in soil for the case of active pressure is somewhat different from that assumed in the calculation of the Coulomb pressure, the results are not greatly different.

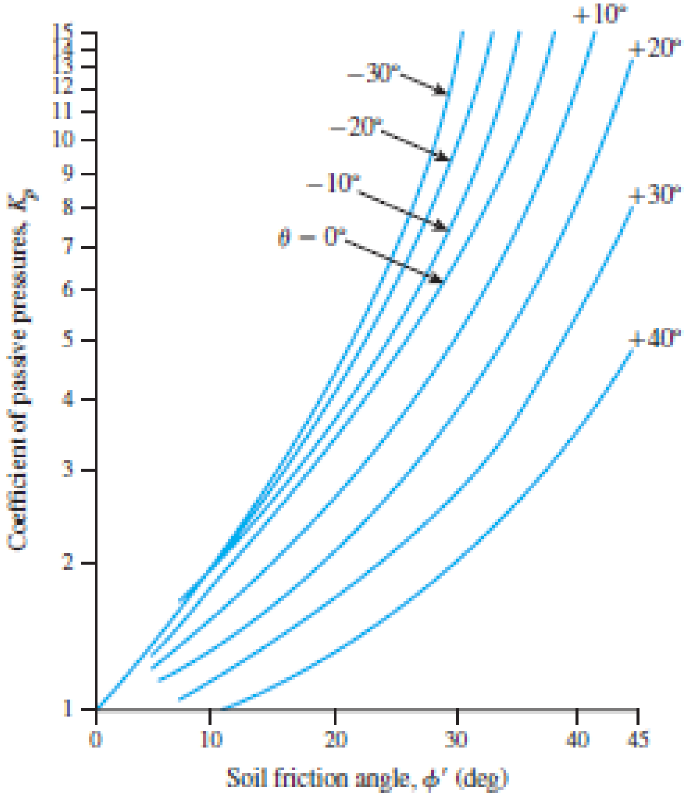
However, in the case of passive pressure, as the value of δ' increases, Coulomb's method of calculation gives increasingly erroneous values of P_p.

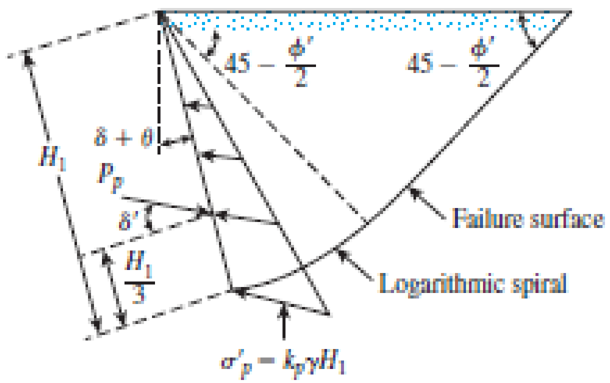
This factor of error could lead to an unsafe condition because the values of P_p would become higher than the soil resistance.

Several studies have been conducted to determine the passive force Pp, assuming that the curved portion *BC* in is an arc of a circle, an ellipse, or a logarithmic spiral (e.g., Caquot and Kerisel, 1948; Terzaghi and Peck, 1967; Shields and Tolunay,1973; Zhu and Qian, 2000).

Caquot and Kerisel Solution for Passive Earth Pressure (Granular Backfill)

Inclined wall with horizontal backfill



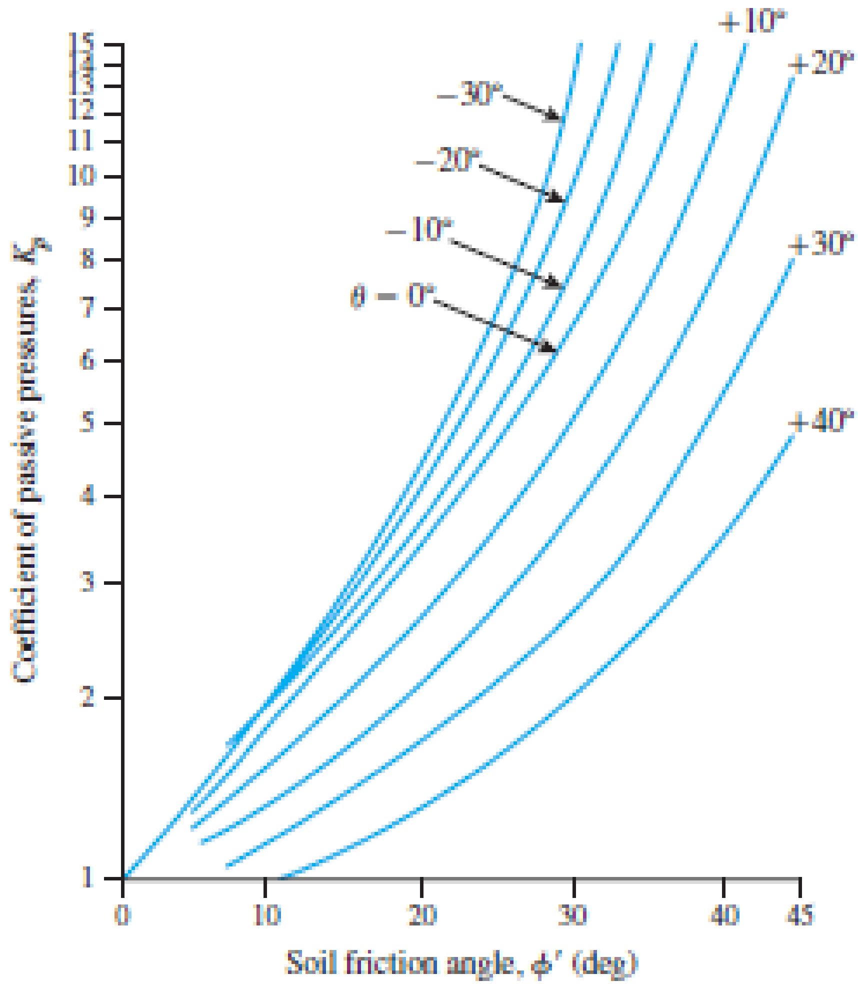


$$P_p = \frac{1}{2}\gamma H_1^2 K_p$$

for $\delta'/\phi' = 1$.

Caquot and Kerisel Solution for Passive Earth Pressure (Granular Backfill)

for $\delta'/\phi' = 1$.



Caquot and Kerisel Solution for Passive Earth Pressure (Granular Backfill)

If $\delta'/\phi' \neq 1$,

1. Assume δ' and ϕ'.
2. Calculate δ'/ϕ'.
3. Using the ratio of δ'/ϕ' (step 2), determine the reduction factor, R', from Table 16.12.
4. Determine K_p from Figure 16.25 for $\delta'/\phi' = 1$.
5. Calculate K_p for the required δ'/ϕ' as

$$K_p = (R')[K_{p(\delta'/\phi'=1)}] \tag{16.76}$$

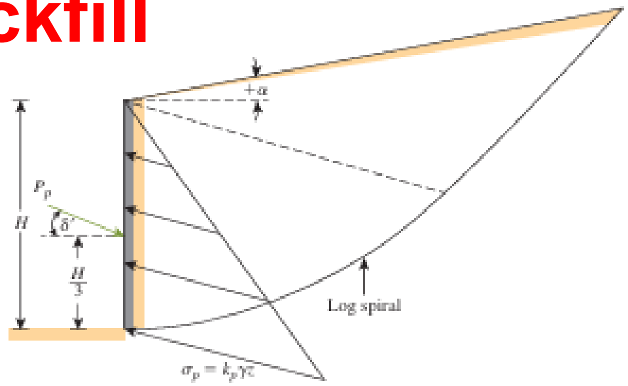
TABLE 16.12 Caquot and Kerisel's Reduction Factor, R', for Passive Pressure Calculation

	δ'/ϕ'							
ϕ'	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
10	0.978	0.962	0.946	0.929	0.912	0.898	0.881	0.864
15	0.961	0.934	0.907	0.881	0.854	0.830	0.803	0.775
20	0.939	0.901	0.862	0.824	0.787	0.752	0.716	0.678
25	0.912	0.860	0.808	0.759	0.711	0.666	0.620	0.574
30	0.878	0.811	0.746	0.686	0.627	0.574	0.520	0.467
35	0.836	0.752	0.674	0.603	0.536	0.475	0.417	0.362
40	0.783	0.682	0.592	0.512	0.439	0.375	0.316	0.262
45	0.718	0.600	0.500	0.414	0.339	0.276	0.221	0.174

Caquot and Kerisel Solution for Passive Earth Pressure (Granular Backfill)

Vertical wall with Inclined backfill

$$P_p = \frac{1}{2}\gamma H^2 K_p$$



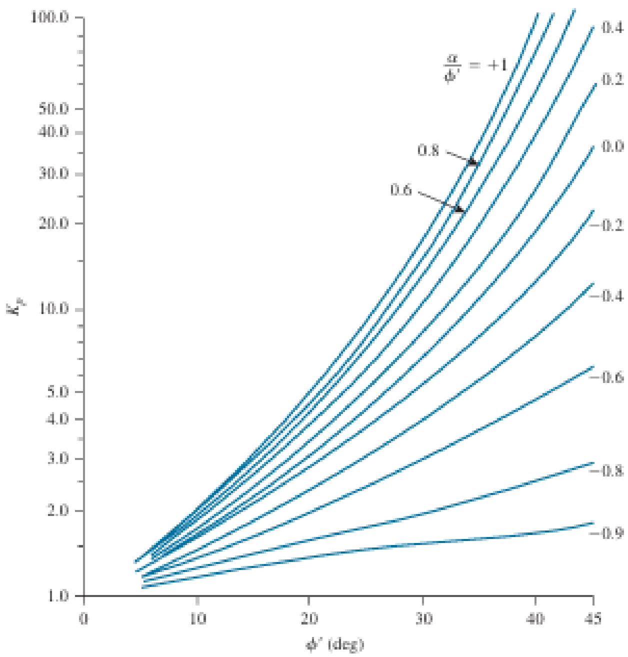
Step 1. Determine α/ϕ' (note the sign of α).

Step 2. Knowing ϕ' and α/ϕ', use Figure 16.26a to determine K_p for $\delta'/\phi' = 1$.

Step 3. Calculate δ'/ϕ'.

Step 4. Go to Table 16.12 to determine the reduction factor, R'.

Step 5. $K_p = (R')\,[K_{p(\delta'/\phi'=1)}]$. (16.78)



EXAMPLE 16.14

EXAMPLE 16.14

Consider a 3 m high (H) retaining wall with a vertical back ($\theta = 0°$) and a horizontal granular backfill. Given: $\gamma = 15.7\ \text{kN/m}^3$, $\delta' = 15°$, and $\phi' = 30°$. Estimate the passive force, P_p, by using

a. Coulomb's theory
b. Caquot and Kerisel's theory

SOLUTION

Part a

From Eq. (16.73),

$$P_p = \frac{1}{2} K_p \gamma H^2$$

From Table 16.11, for $\phi' = 30°$ and $\delta' = 15°$, the value of K_p is 4.977. Thus,

$$P_p = \left(\frac{1}{2}\right)(4.977)(15.7)(3)^2 = \mathbf{351.6\ kN/m}$$

Part b

From Eq. (16.75), with $\theta = 0, H_1 = H$,

$$P_p = \frac{1}{2} \gamma H^2 K_p$$

From Figure 16.25a, for $\phi' = 30°$ and $\delta'/\phi' = 1$, the value of $K_{p(\delta'/\phi'=1)}$ is about 5.9. Also, from Table 16.12, with $\phi' = 30°$ and $\delta'/\phi' = 0.5$, the value of R' is 0.746.

Hence,

$$P_p = \frac{1}{2}\gamma H^2 K_p = \frac{1}{2}(15.7)(3)^2(0.746 \times 5.9) \approx \mathbf{311\ kN/m}$$

EXAM

The soil conditions adjacent to a sheet pile wall are given in the Figure below. A surcharge pressure of 50 kN/m^2 being carried on the surface behind the wall.
For soil 1, a sand above the water table, c' = 0 kN/m^2 and ϕ' = 38^o and γ = 18 KN/m^3.
For soil 2, a saturated clay, c' = 10 kN/m^2 and ϕ' = 28^o and γ_{sat} = 20 KN/m^3.

- Calculate K_a and K_p for each of soils (1) and (2).
- Complete the given table for the Rankine active pressure at 6 and 9 m depth behind the wall shown in the Figure.
- Complete the given table for the Rankine passive pressure at 1.5 and 4.5 m depth in front of the wall shown in the Figure.

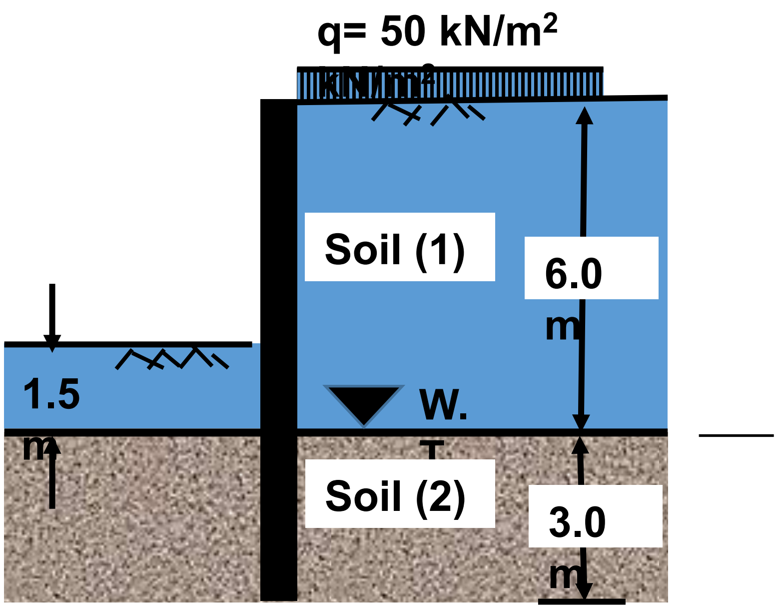


Table . Active and passive earth pressures on sheet pile wall

Depth (meter)	Soil		
		Active Pressure (kN/m^2)	
0	1		
6	1		
6	2		
9	2		
		Passive Pressure (kN/m^2)	
0	1		
1.5	1		
1.5	2		
4.5	2		

SOLUTION

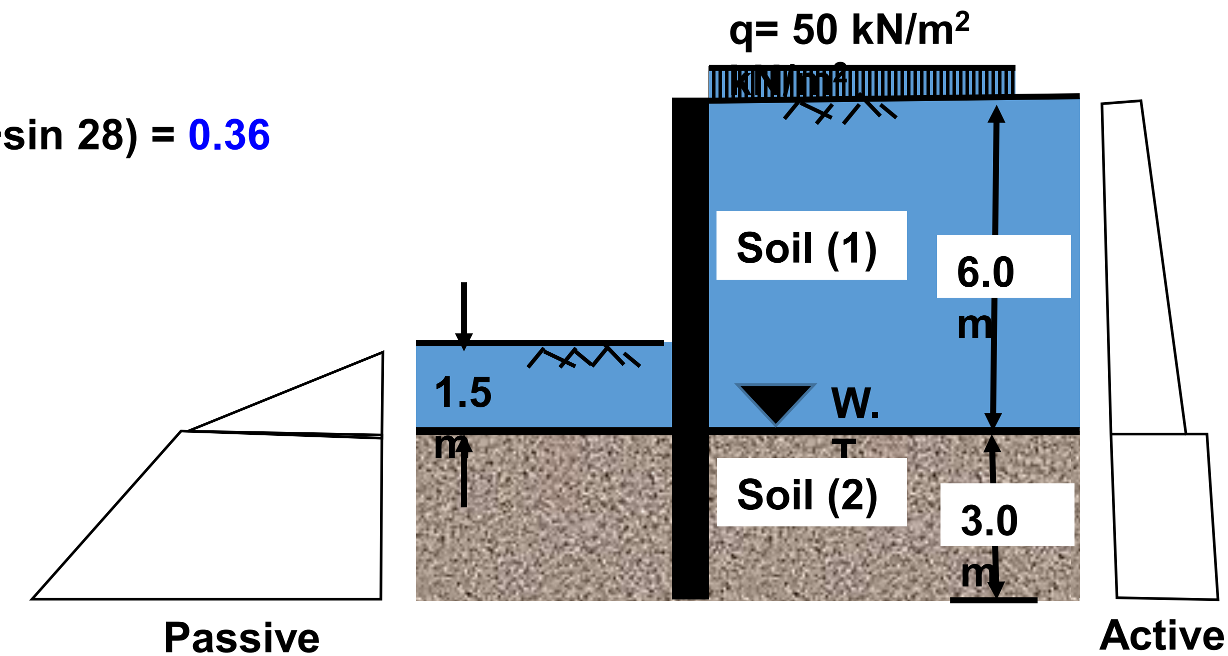
SOLUTION

Soil 1: $K_a = (1 - \sin 38)/(1 + \sin 38) = 0.24$

$K_p = 1/K_a = 4.17$

Soil 2: $K_a = (1 - \sin 28)/1 + \sin 28) = 0.36$

$K_p = 1/K_a = 2.78$



SOLUTION

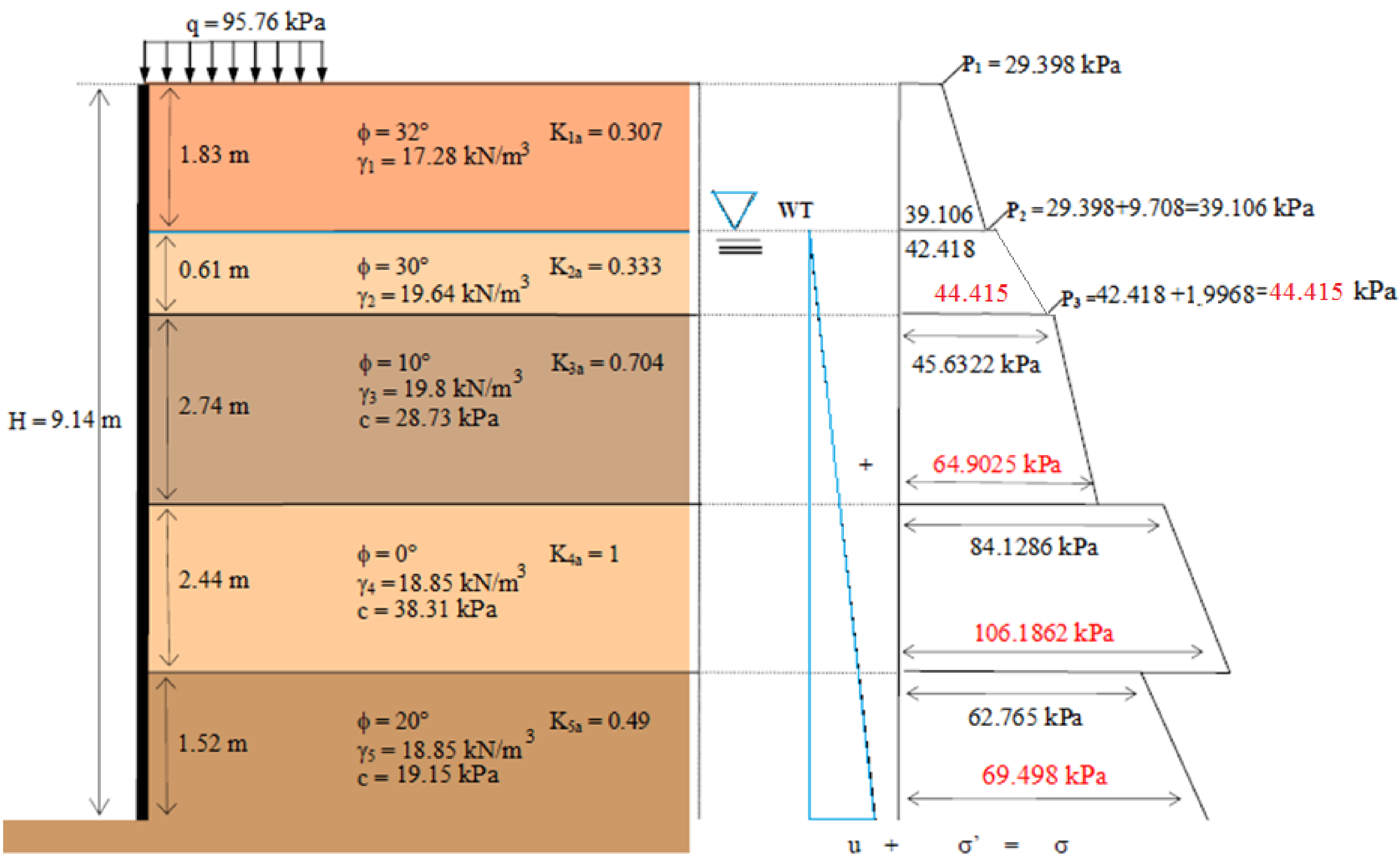
Depth (meter)	Soil		
		Active Pressure (kN/m²)	
0	1	0.24 x 50	= 12
6	1	0.24 x (50 + 18 x 6)	= 37.9
6	2	0.36 x (50 + 18 x 6) - 2 x$\sqrt{0.36}$ x 10	= 44.9
9	2	0.36 x (50 + 18 x 6 + 10.2 x 3) - 2 x$\sqrt{0.36}$ x 10 + 9.81 x 3	= 85.33
		Passive Pressure (kN/m²)	
0	1		= 0
1.5	1	4.17 x 18 x 1.5	= 112.6
1.5	2	2.78 x 18 x 1.5 + 2 x $\sqrt{2.78}$ x 10	= 108.4
4.5	2	2.78 x (18 x 1.5 + 10.2 x 3)+ 2 x $\sqrt{2.78}$ x 10 + 9.81 x 3	= 222.93

RECOMMENDED PROCEDURE

1. Calculate the appropriate k for each soil
2. Calculate σ_V at a specified depth
3. Add q if any
4. Multiply the sum of (σ_{V} + q) by the appropriate k (for upper and lower soil) and subtract (or add for passive) cohesion part if exists.
5. Calculate water pressure
6. Divide each trapezoidal area into a rectangle and a triangle
7. Calculate areas and that give the lateral forces
8. Locate point of application for each force
9. Find the resultant force
10. Take moments about the base of the wall and find location of the resultant

EXAMPLE

Plot the Rankine pressure diagram and find the resultant force F and its location under an active pressure condition.



SOLUTION

At h=0' $p_1 = q\,K_{1a} = (95.76)(0.307) = 29.398$ kPa

At h = -1.83 $\Delta p_2 = \gamma_1 h\,K_{1a} = (17.28)(1.83)(0.307) = 9.708$ kPa

At h = -(1.83+dh) = $[q + (\gamma_1)\,1.83]\,K_{2a} = [95.76 + (17.28)(1.83)]\,(0.333) = 42.418$ kPa

At h = -2.44 $\Delta p_3 = (\gamma_2 - \gamma_w)\,h\,K_{2a} = (19.64 - 9.81)\,(0.61)(0.333) = 1.9968$ kPa $\rightarrow$ $\boxed{42.418 + 1.9968 = 44.415 \text{ kPa}}$

At h = -(2.44+dh) = $[q + (\gamma_1)\,1.83 + (\gamma_2 - \gamma_w)\,0.61]\,K_{3a} - 2c\,\sqrt{(K_{3a}}$ from $p = \gamma h\,K_a - 2c\sqrt{K_a}$

$= [95.76 + (17.28)1.83 + (19.64 - 9.81)0.61]\,(0.704) - 2(28.73)\,(0.84) = 45.6322$ kPa

At h= -5.18 $\Delta p_4 = (\gamma_3 - \gamma_w)\,h\,K_{3a} = (19.8\text{-}9.81)\,(2.74)\,(0.704) = 19.27$ kPa $\therefore 45.632 + 19.27 = 64.9025$ kPa

At h = - (5.18 + dh) = $[95.76 + 31.62 + 5.996 + 27.3726]\,(1) - 2(38.31)(1) = 84.1286$ kPa

At h = -7.62 $\Delta p_5 = (\gamma_4 - \gamma_w)\,h\,K_{4a} = (18.85 - 9.81)\,(2.44)\,(1) = 22.0576$ kPa

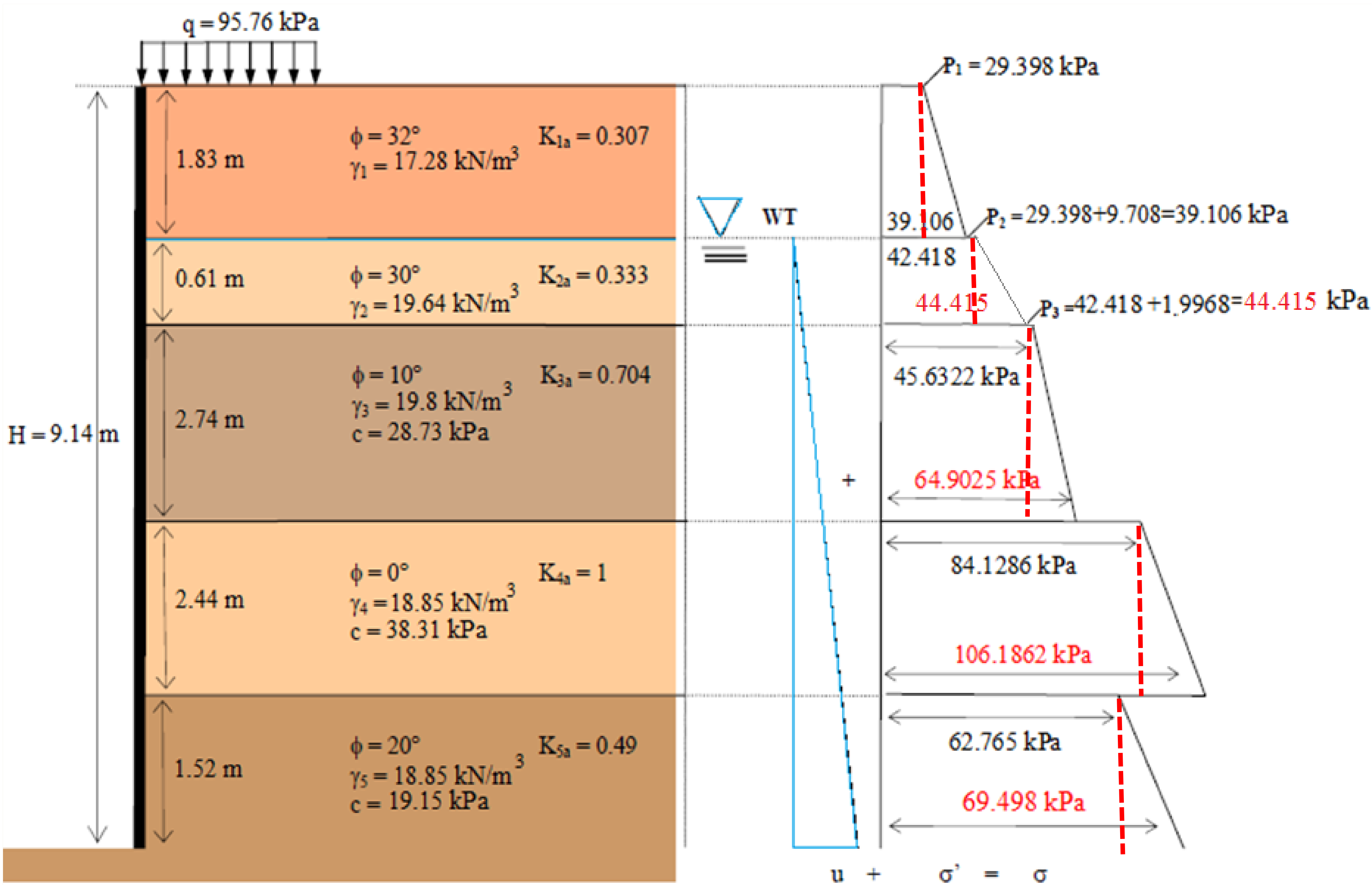
$\therefore 84.1268 + 22.0576 = 106.1862$ kPa

At h = - (7.62 + dh) = $[95.76 + 31.62 + 5.996 + 27.3726 + 22.0576](0.49) - 2(19.15)(0.7) = 62.765$ kPa

At h = -9.14 $\Delta p_6 = (\gamma_5 - \gamma_w)\,h\,K_{5a} = (18.85\text{-}9.81)(1.52)\,(0.49) = 6.733$ kPa

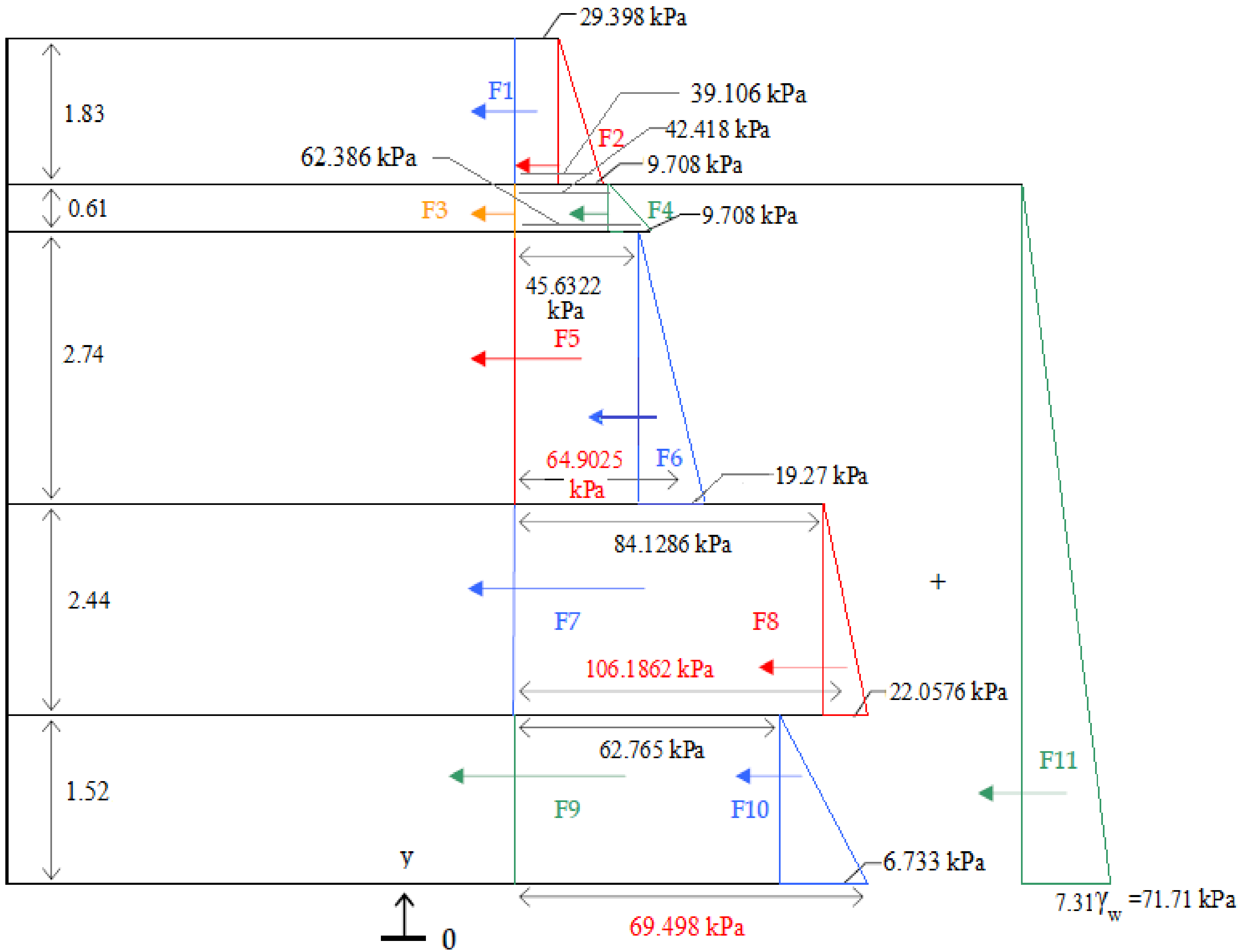
$\therefore 62.765 + 6.33 = 69.498$ kPa

SOLUTION



q = 95.76 kPa
H = 9.14 m
1.83 m
ϕ = 32°
γ1 = 17.28 kN/m3
K1a = 0.307
0.61 m
ϕ = 30°
γ2 = 19.64 kN/m3
K2a = 0.333
2.74 m
ϕ = 10°
γ3 = 19.8 kN/m3
c = 28.73 kPa
K3a = 0.704
2.44 m
ϕ = 0°
γ4 = 18.85 kN/m3
c = 38.31 kPa
K4a = 1
1.52 m
ϕ = 20°
γ5 = 18.85 kN/m3
c = 19.15 kPa
K5a = 0.49
WT
P1 = 29.398 kPa
39.106
P2 = 29.398+9.708=39.106 kPa
42.418
44.415
P3 = 42.418 + 1.9968 = 44.415 kPa
45.6322 kPa
64.9025 kPa
84.1286 kPa
106.1862 kPa
62.765 kPa
69.498 kPa
+
u + σ' = σ

SOLUTION


29.398 kPa
1.83
F1
39.106 kPa
42.418 kPa
F2
62.386 kPa
9.708 kPa
0.61
F3
F4
9.708 kPa
45.6322
kPa
F5
2.74
64.9025
kPa
F6
19.27 kPa
84.1286 kPa
2.44
+
F7
F8
106.1862 kPa
22.0576 kPa
62.765 kPa
1.52
F9
F10
F11
y
6.733 kPa
0
69.498 kPa
7.31γw =71.71 kPa

SOLUTION

$F1 = (29.398\ kPa)(1.83) = 53.798\ kN$

$F2 = 0.5(9.708\ kPa)(1.83) = 8.8828\ kN$

$F3 = (42.418)(0.61) = 25.875\ kN$

$F4 = 0.5(19.968\ kPa)(0.61) = 7.8919\ kN$

$F5 = (45.6322\ kPa)(2.74) = 125.032\ kN$

$F6 = 0.5(19.27\ kPa)(2.74) = 26.3999\ kN$

$F7 = (84.1286\ kPa)(2.44) = 205.2738\ kN$

$F8 = 0.5(22.0576\ kPa)(2.44) = 26.91\ kN$

$F9 = (62.765\ kPa)(1.52) = 95.4\ kN$

$F10 = 0.5(6.733\ kPa)(1.52) = 5.117\ kN$

$F11 = 0.5(71.71)(7.31) = 262.104\ kN$

The resultant R is, $\mathbf{R = \Sigma\ F}\boldsymbol{i} = 842.68448\ kN$

The location of R is……$\Sigma M_0 = 0$ (about 0)

$842.68448(y) = (53.798)(8.225) + (8.8828)(7.92) + (25.875)(7.005) + (7.8919)(6.903) + (125.032)(5.33) + (26.3999)(4.873) + (205.273)(2.74) + (26.91)(2.33) + (95.4)(0.76) + (5.117)(0.506) + (262.104)(2.4366)$

$\therefore 57.1\ y = 2882.53945 \rightarrow\rightarrow\rightarrow\rightarrow\rightarrow y = 3.4206$ m <u>**above "0"**</u>

The end