ENCE 3610 Soil Mechanics

Lecture 14
Retaining Walls
Lateral Earth Pressure Theory

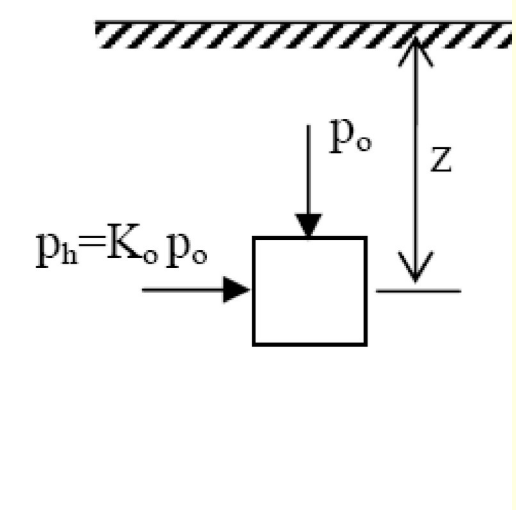
Retaining Walls

- Elevation changes are an unavoidable part of site development
- They can result either from changes in the land elevation or a land/water boundary
- They are dealt with in one of two ways:
  - Earth slopes (unreinforced and reinforced)
  - Retaining Walls
- Retaining Walls
  - Necessary in situations where gradual transitions either take up too much space or are impractical for other reasons
  - Retaining walls are analysed for both resistance to overturning and structural integrity
  - Two categories of retaining walls
    - Gravity Walls (Masonry, Stone, Gabion, etc.)
    - In-Situ Walls (Sheet Piling, cast in-situ, etc.)

Lateral Earth Pressure Coefficient and Conditions of Lateral Earth Pressure

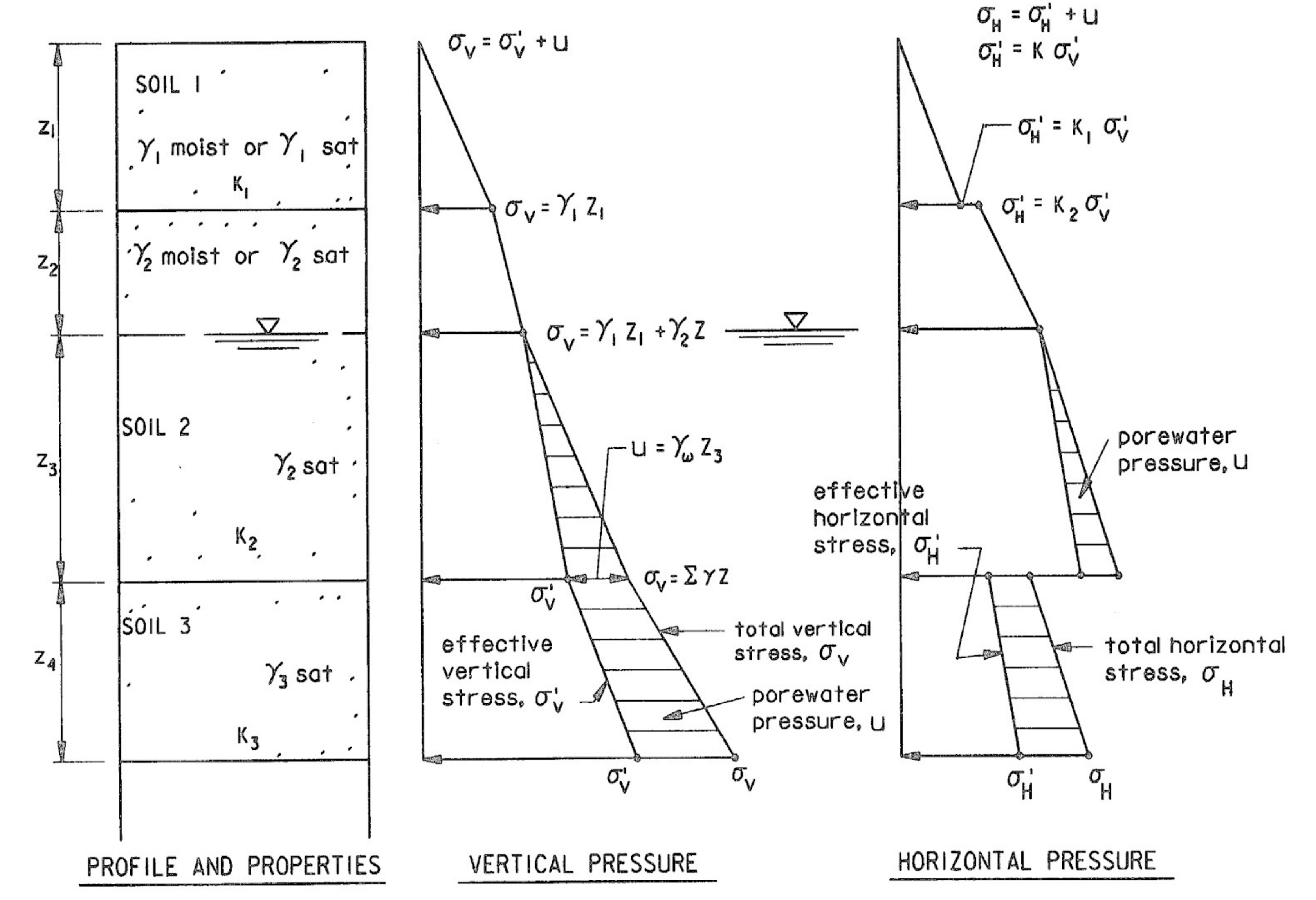
- Lateral Earth Pressure Coefficient K
  - K = lateral earth pressure coefficient
  - σ_x' = horizontal effective stress
  - σ_z' = vertical effective stress
- Ratio of resultant horizontal stress to applied vertical stress
- Similar to Poisson's Ratio for elastic materials

$$K = \frac{\sigma_x}{\sigma_z}$$



Relationship Between Vertical and Horizontal Stresses in Retaining Walls

From EM 1110-2-2502



Conditions of Lateral Earth Pressure Coefficient

- At-Rest Condition
  - Condition where wall movement is zero or “minimal,” or not really a wall
  - Ideal condition of wall, but seldom achieved in reality
- Active Condition
  - Condition where wall moves away from the backfill
  - The lower state of lateral earth pressure
- Passive Condition
  - Condition where wall moves toward the backfill
  - The higher state of lateral earth pressure

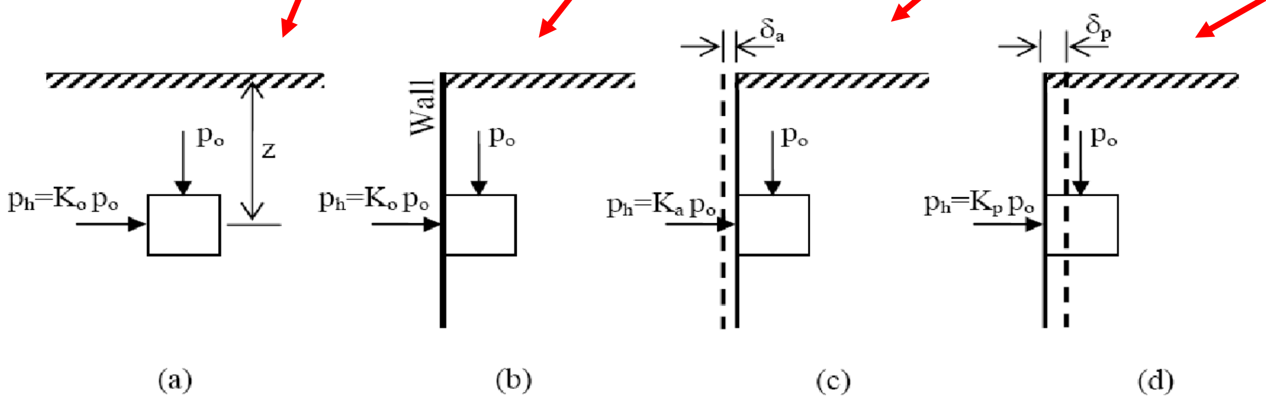
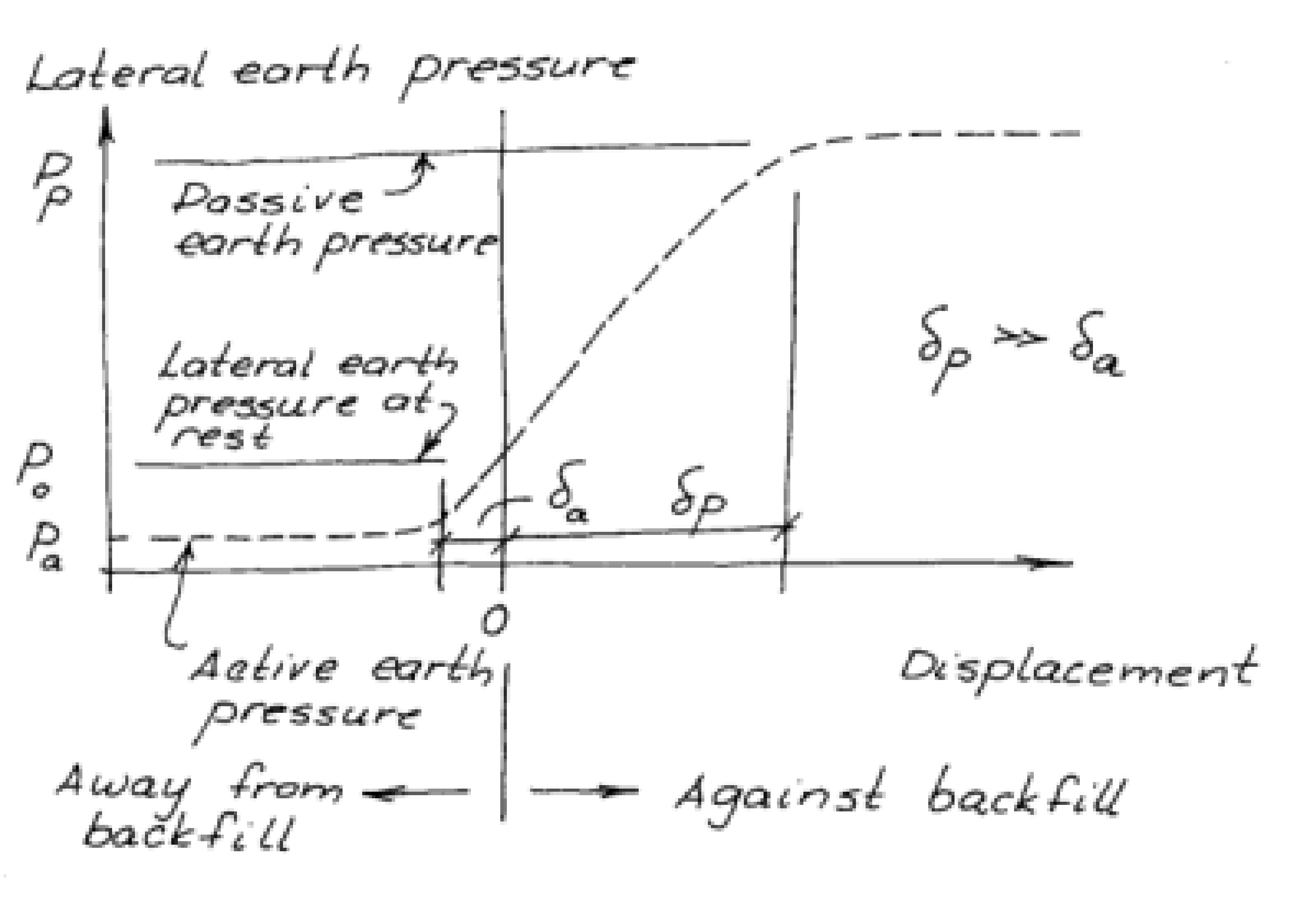


Figure 2-19. Stress states on a soil element subjected only to body stresses: (a) In-situ geostatic effective vertical and horizontal stresses, (b) Insertion of hypothetical infinitely rigid, infinitely thin frictionless wall and removal of soil to left of wall, (c) Active condition of wall movement away from retained soil, (d) Passive condition of wall movement into retained soil.

Effect of Wall Movement



At Rest Lateral Earth Pressure Coefficient K_o

- Although we are discussing wall pressures, there are also lateral pressures in the soil mass as well
- The at-rest condition primarily describes conditions with a semi-infinite (self-confining) soil mass and a level surface

- Jaky's Equation

$$K_o = 1 - \sin\varphi$$

- Modified for Overconsolidated Soils

$$K_o = 1 - \sin\varphi (OCR)^{\sin\varphi}$$

- In spite of theoretical weaknesses, Jaky's equation is as good an estimate of the coefficient of lateral earth pressure as we have

Elastic Theory and At-Rest Earth Pressure Conditions

32.3 Elastic material

A possible approach to the behavior of soils is to consider it as an elastic material. In such a material the stresses and strains satisfy Hooke's law. In a situation in which there can be no lateral deformation, the stresses must satisfy the condition

$$\varepsilon_{xx} = -\frac{1}{E}[\sigma_{xx} - \nu(\sigma_{yy} + \sigma_{zz})] = 0,$$

$$\varepsilon_{yy} = -\frac{1}{E}[\sigma_{yy} - \nu(\sigma_{zz} + \sigma_{xx})] = 0,$$

if the z-direction is vertical. In a medium of large horizontal extent it can be expected that $\sigma_{xx} = \sigma_{yy}$. Then

$$\varepsilon_{xx} = \varepsilon_{yy} = 0 : \quad \sigma_{xx} = \sigma_{yy} = \frac{\nu}{1-\nu}\sigma_{zz}, \tag{32.18}$$

or

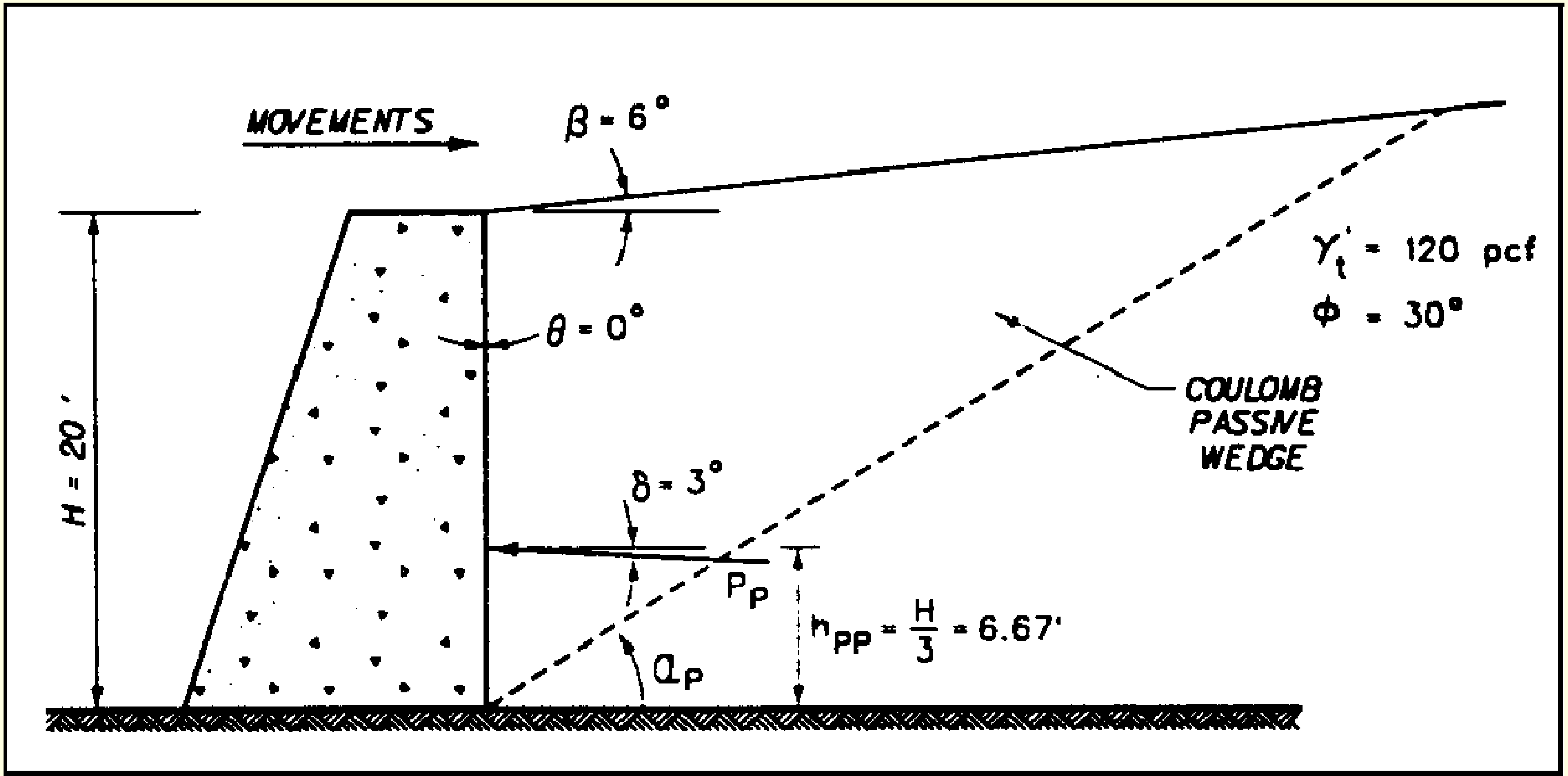
$$K = \frac{\nu}{1-\nu}. \tag{32.19}$$

If Poisson's ratio varies between 0 and 0.5, the value of K varies from 0 to 1.

Valid only if soil is acting elastically.

Example of At Rest Wall Pressure

- Given
  - Retaining Wall as Shown
- Find
  - $P_{o,}$ from At Rest Conditions



At Rest Pressure Example

- Compute at rest earth pressure coefficient

$$K_o = 1 - \sin\varphi$$

$$K_o = 1 - \sin(30°) = 0.5$$

- Compute Effective Wall Force

$$\frac{P_o}{b} = \frac{\gamma_1 z_1^2 K_o}{2} = \frac{120 \times 20^2 \times 0.5}{2} = 12{,}000\, lbs/ft = 12\, kips/ft$$

$$h = \frac{20}{3} = 6.67'$$

(basic concept same for all theories)

Soil Wedge Action on Retaining Walls

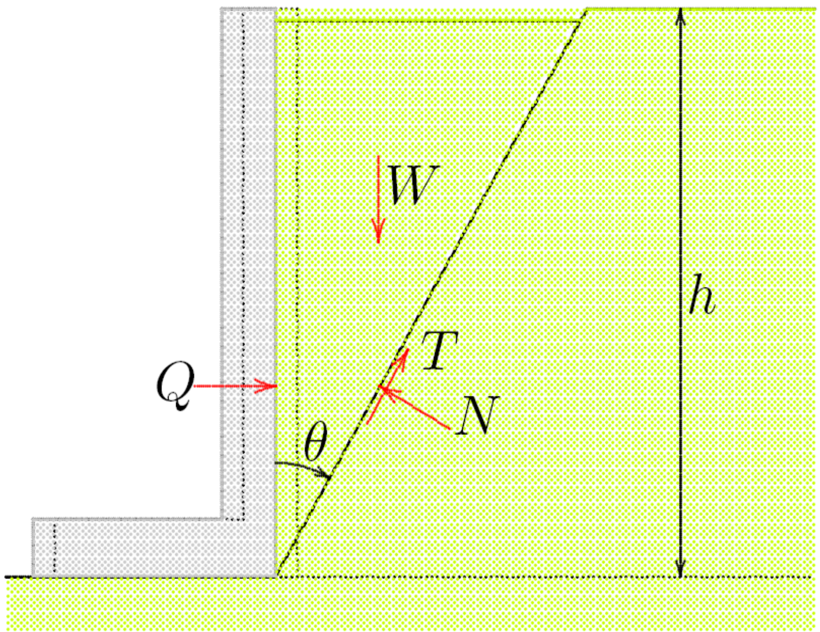


Figure 34.1: Active earth pressure.

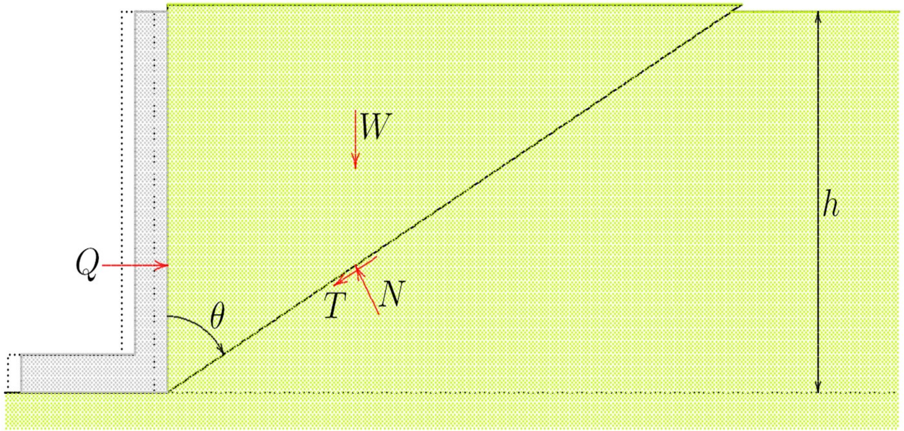
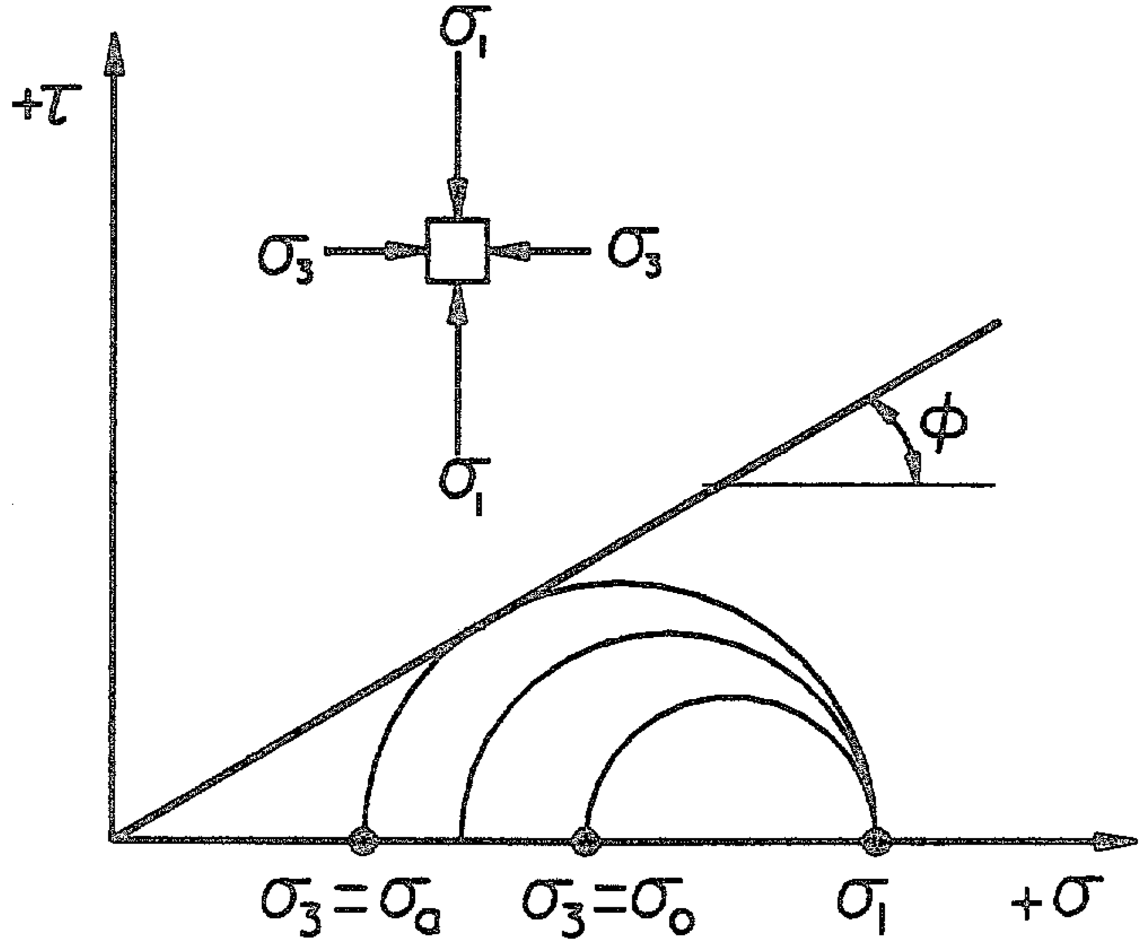
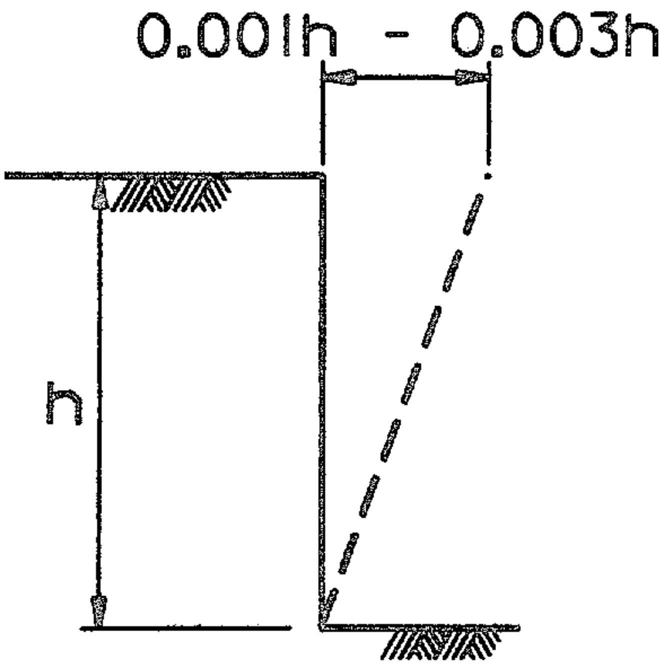


Figure 34.2: Passive earth pressure.

Development of Active Earth Pressure

Range of Values



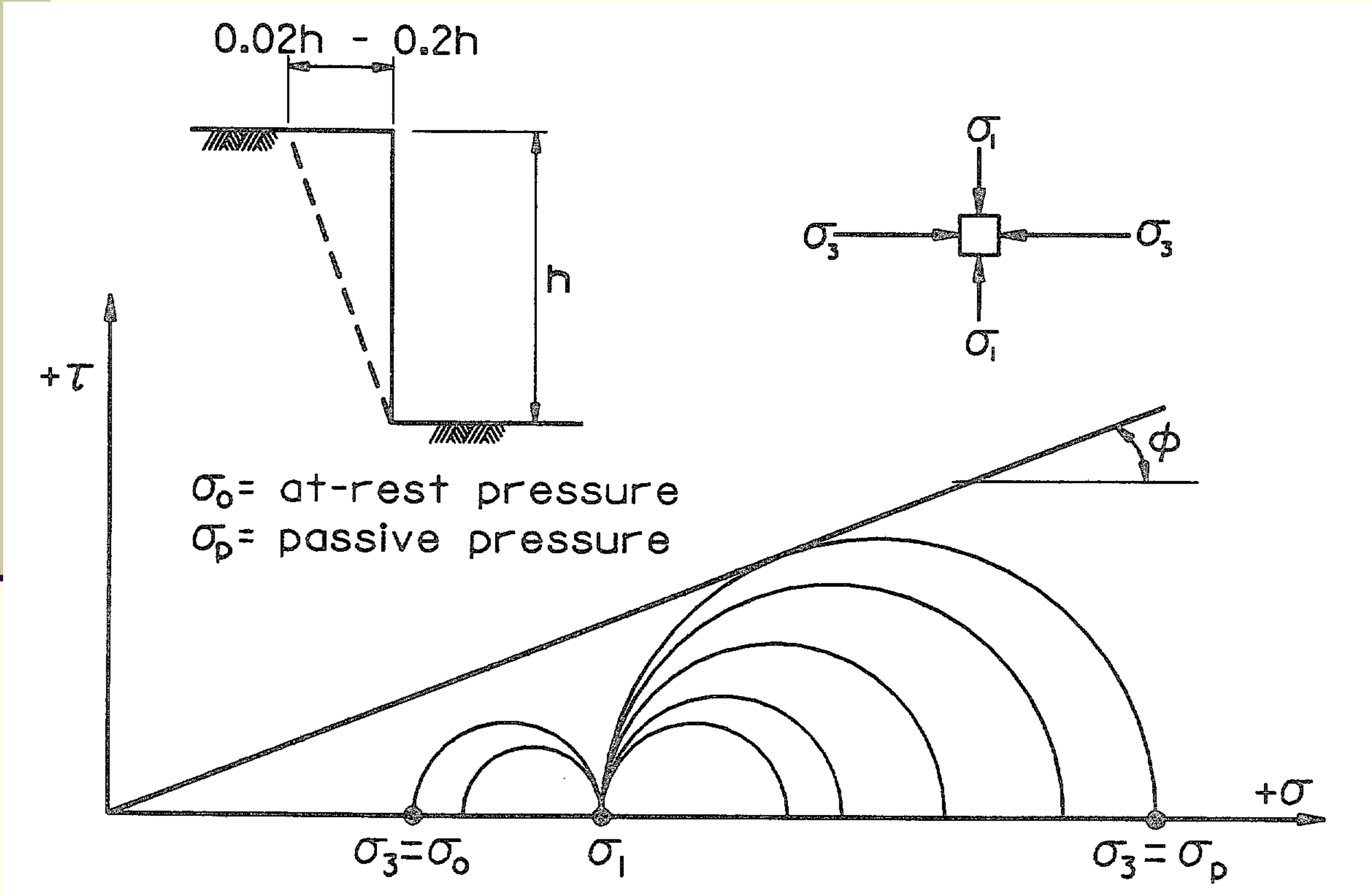


σ_o = at-rest pressure

σ_a = active pressure

Development of Passive Earth Pressure

Range of Values



Earth Pressure Theories (not At-Rest Condition)

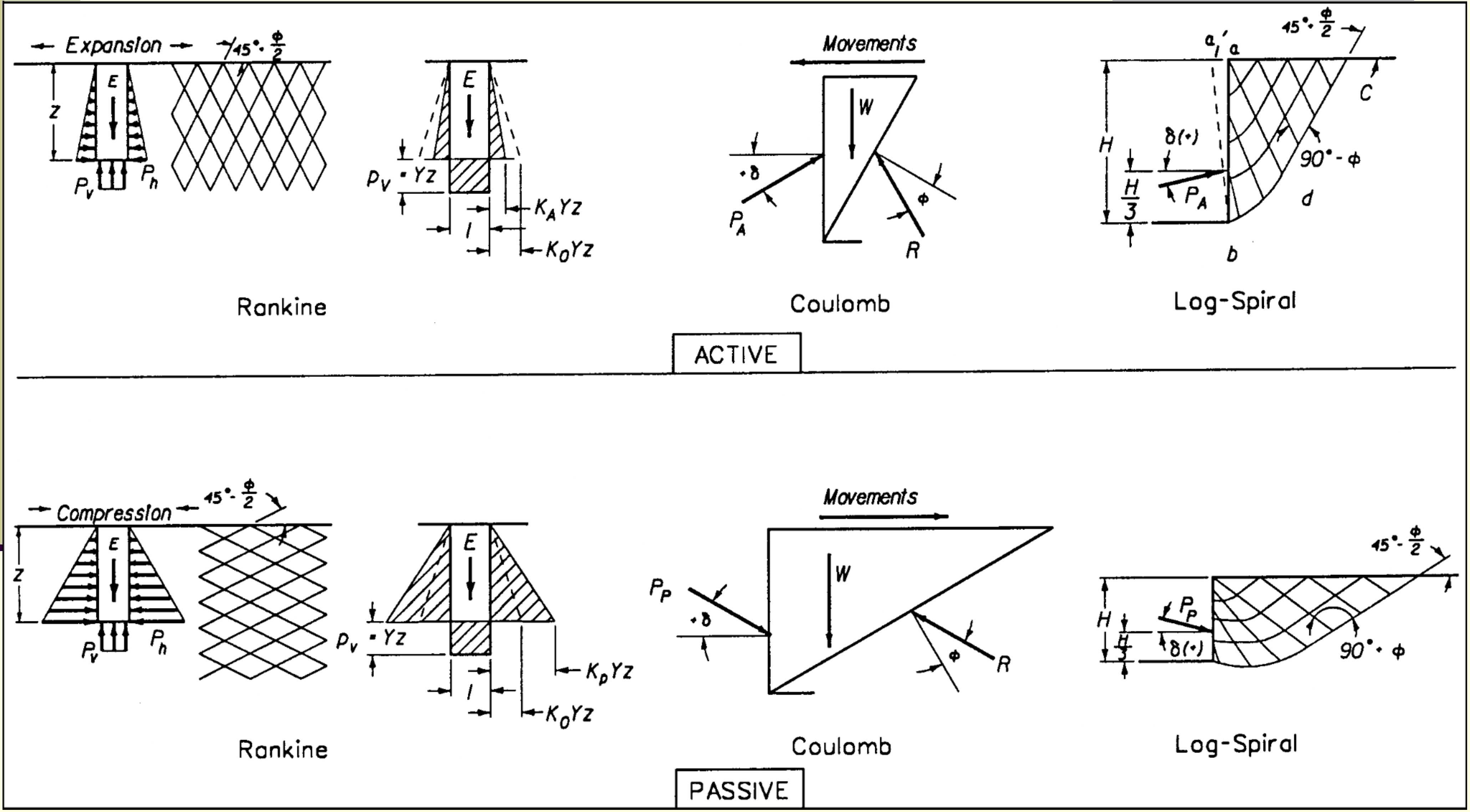
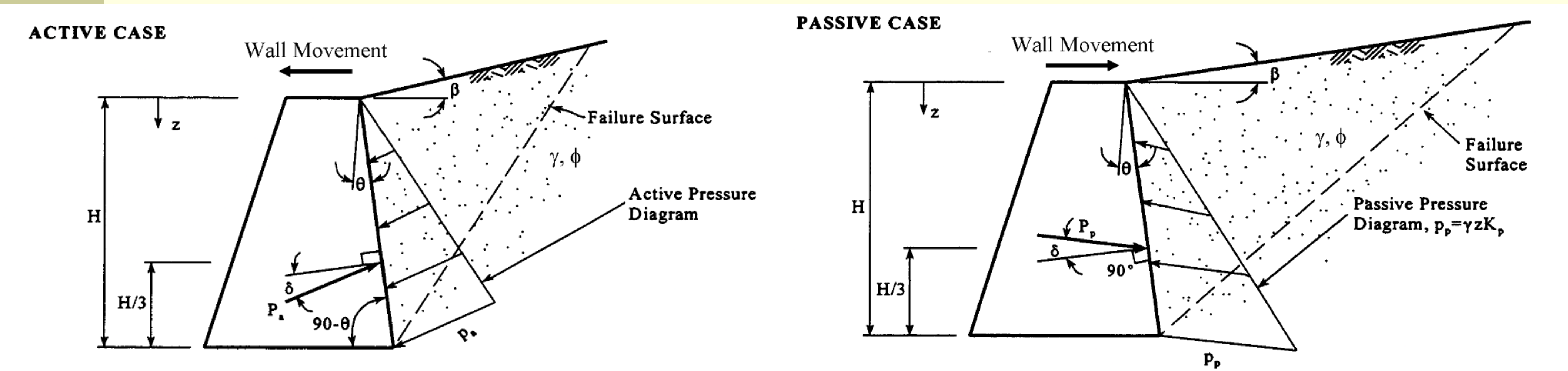
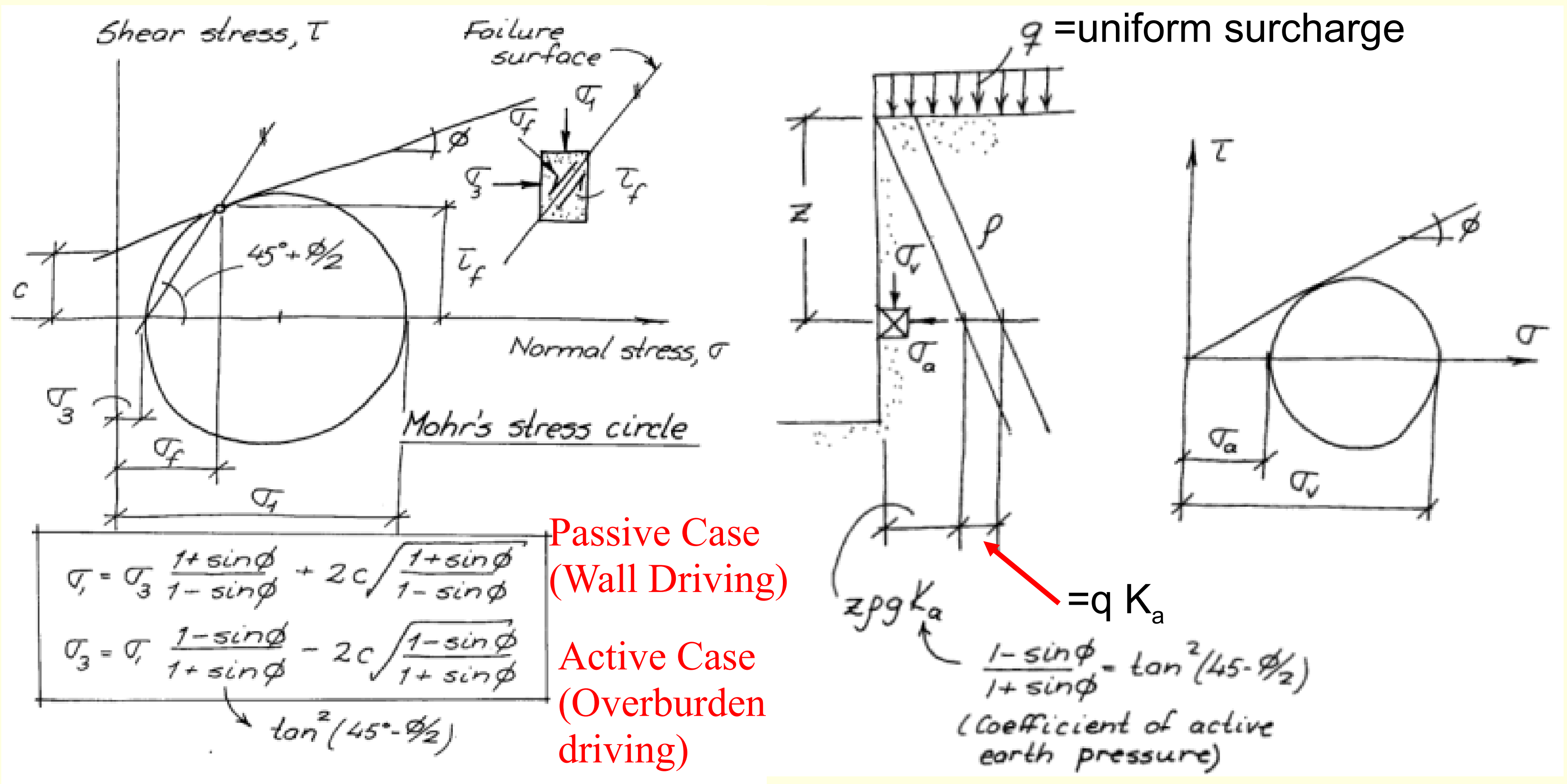


Diagram of Variables for Coulomb and Rankine Earth Pressure Coefficients

- The basic variables for the two earth pressure theories are shown below
- Text and reference books vary on the nomenclature, so you should always have this kind of diagram for whatever theory and source of equations you use
- This is for this course (from the SFH book)
- For Rankine theory, δ = 0 always, it is the main difference between Rankine and Coulomb theories

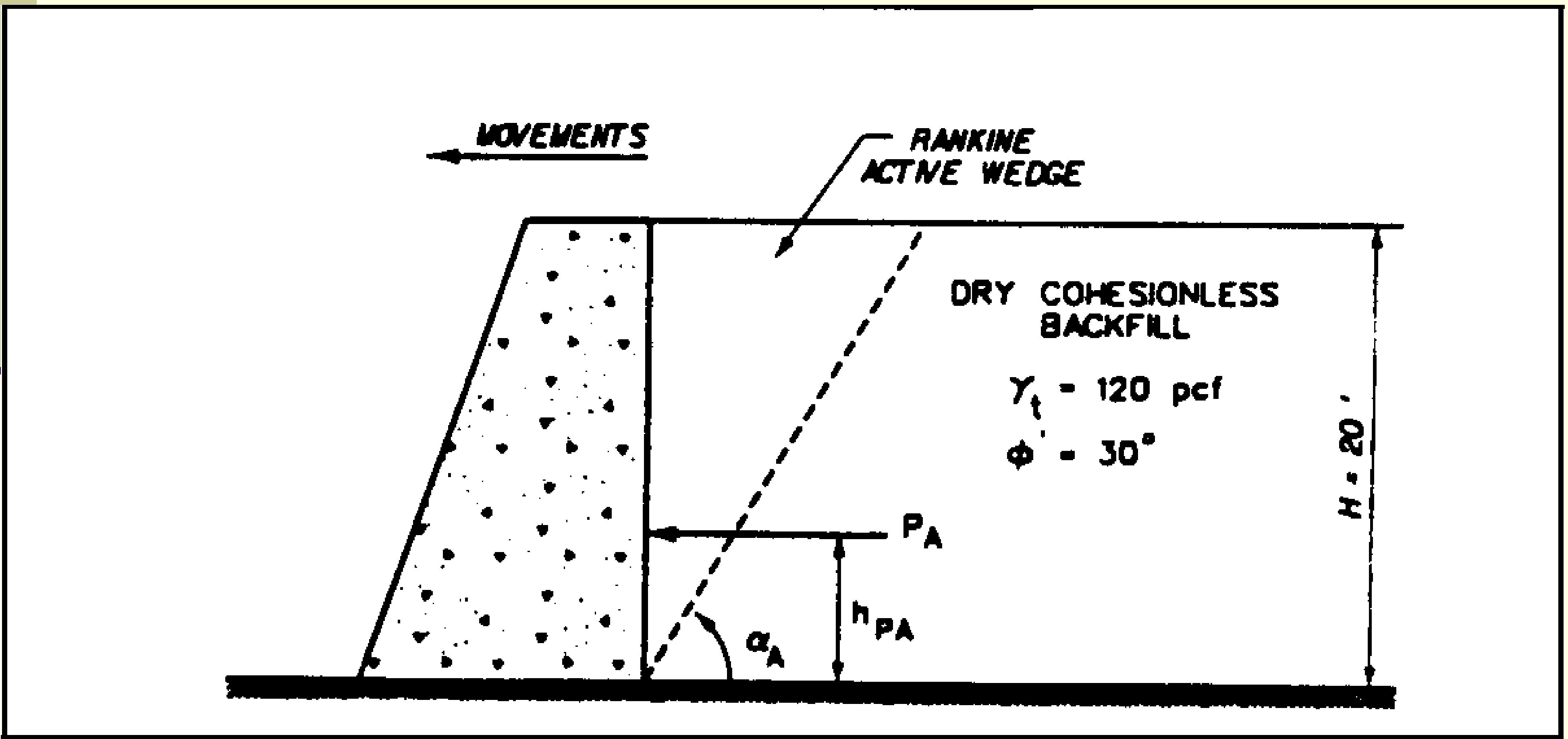


Rankine Earth Pressures from Mohr's Circle (Level Backfill)



Example of Rankine Active Wall Pressure

- Given
  - Retaining Wall as Shown
- Find
  - $P_{A,}$ from Active Conditions



Rankine Active Pressure Example

- Compute active earth pressure coefficient

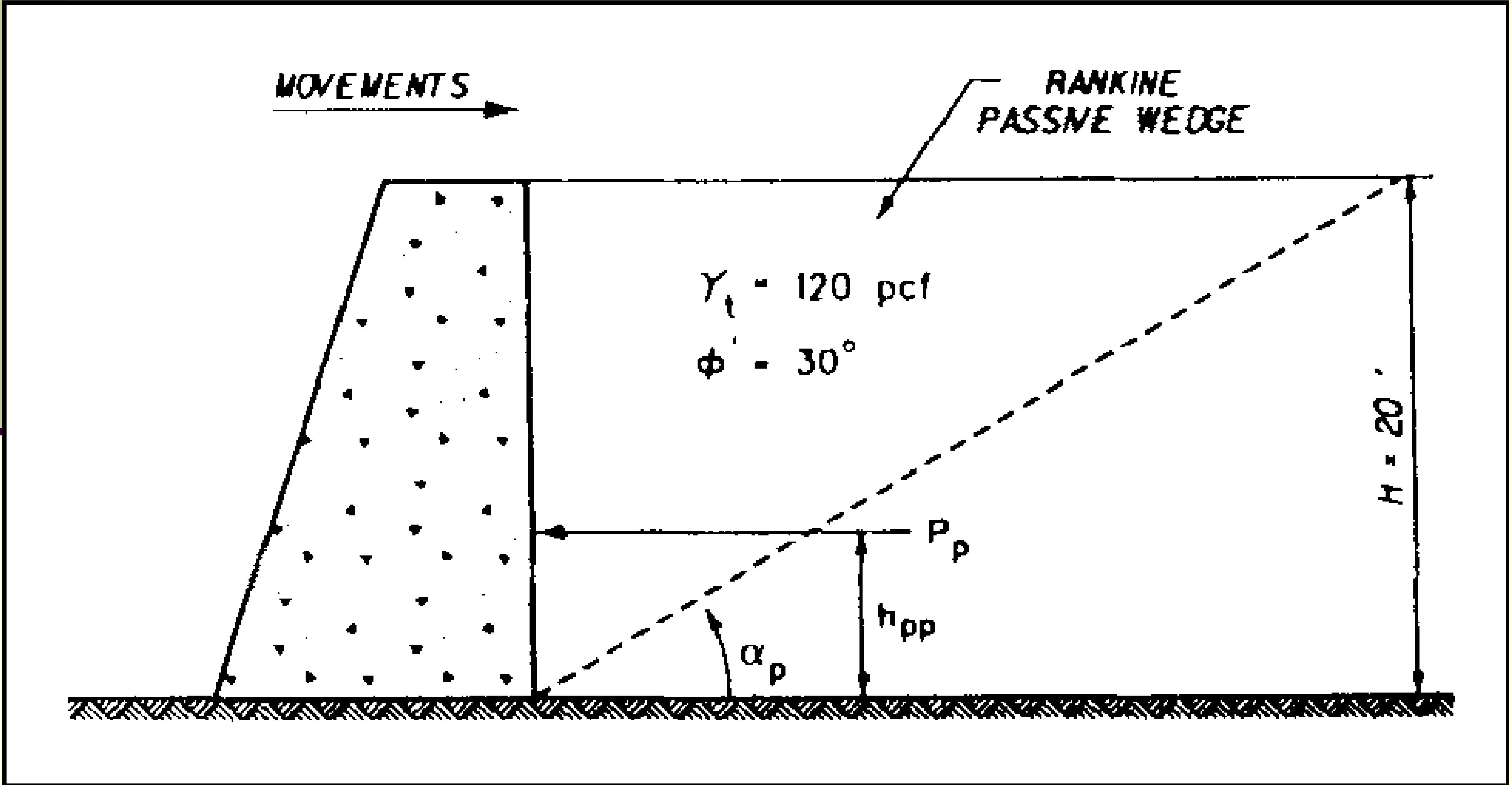
$$K_A = \tan^2\left(45° - \frac{\varphi}{2}\right)$$

- Compute Effective Wall Force

$$K_A = \tan^2\left(45° - \frac{30°}{2}\right) = \frac{1}{3}$$

$$\frac{P_a}{b} = \frac{\gamma_1 z_1^2 K_a}{2} = \frac{120 \times 20^2 \times 1/3}{2} = 8\,kips/ft$$

Rankine Passive Pressure Example



Rankine Passive Pressure Example

- Compute passive earth pressure coefficient

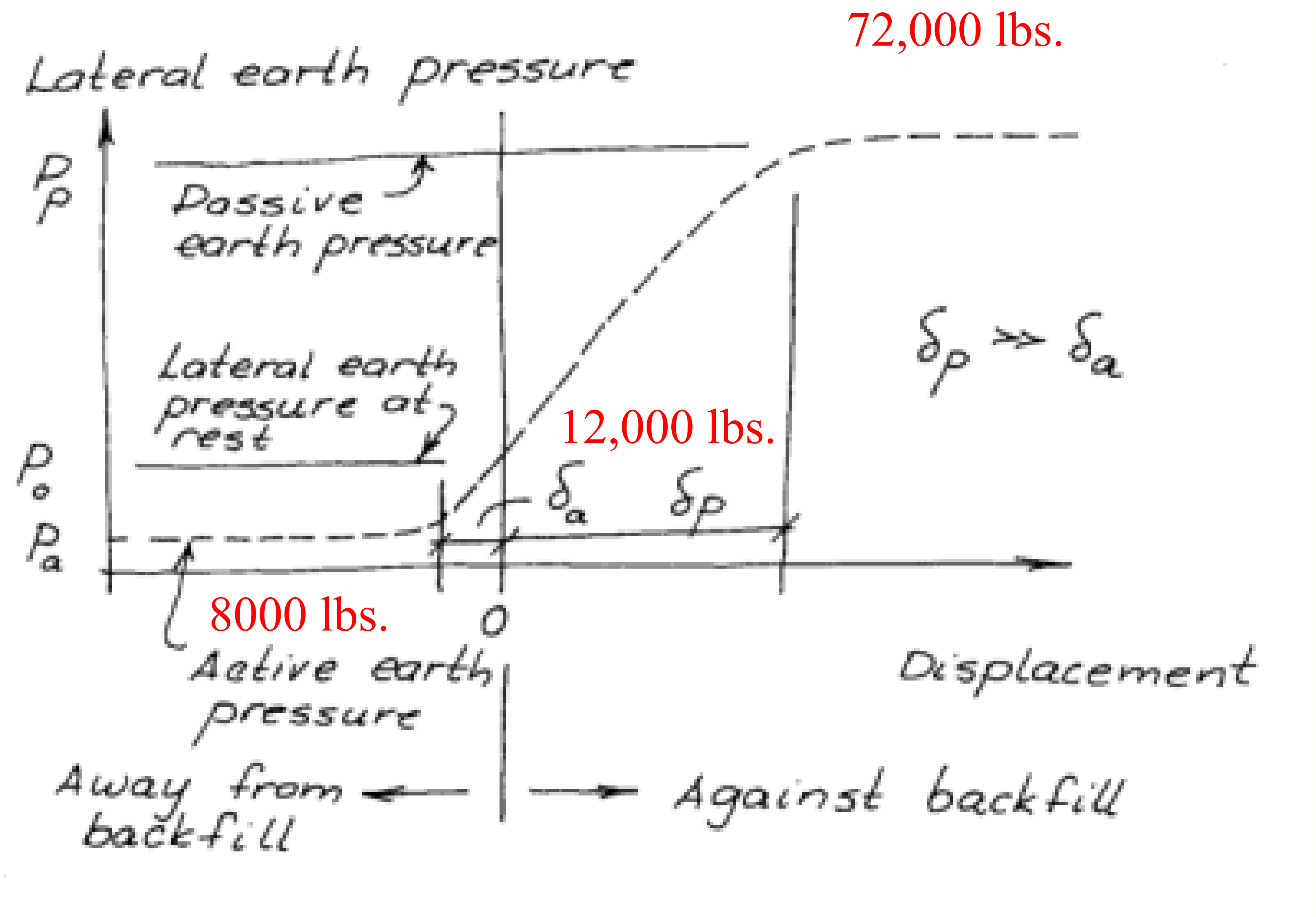
$$K_p = \tan^2\left(45° + \frac{\varphi}{2}\right)$$

$$K_p = \tan^2\left(45° + \frac{30°}{2}\right) = 3$$

- Compute Effective Wall Force

$$\frac{P_o}{b} = \frac{\gamma_1 z_1^2 K_p}{2} = \frac{120 \times 20^2 \times 3}{2} = 72\, kips/ft$$

Summary of Rankine and At Rest Wall Pressures



Rankine Coefficients with Inclined Backfills and Vertical Walls

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

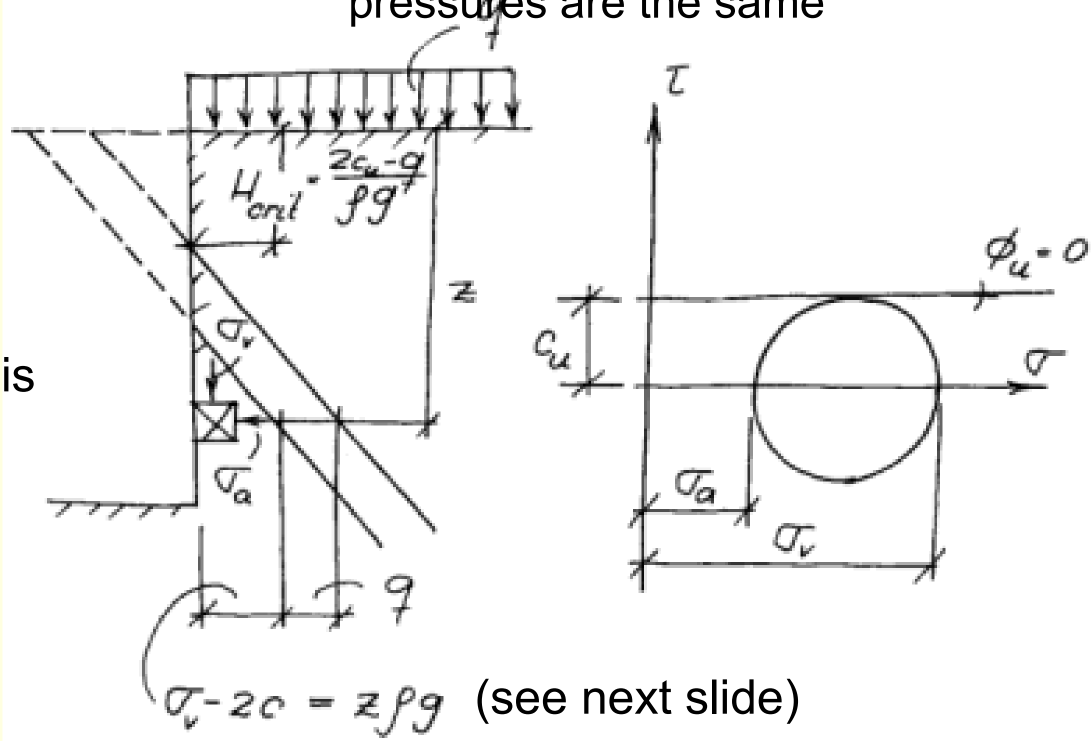
$$K_p = (\cos(\phi))^2 \cos(\beta) \left(\cos(\beta) - 2\sqrt{\frac{\sin(\phi)\sin(\phi+\beta)}{\cos(\beta)}} \cos(\beta) + \sin(\phi)\sin(\phi+\beta) \right)^{-1}$$

These are derived from Coulomb wedge theory; for a more thorough treatment of this topic, click here

Inclined and level backfill equations are identical when $\beta = 0$ for both this and "Rankine extended" theory

Walls With Purely Cohesive Backfill

- Retaining walls should generally have cohesionless backfill, but in some cases cohesive backfill is unavoidable
- Cohesive soils present the following weaknesses as backfill:
  - Poor drainage
  - Creep
  - Expansiveness
- Most lateral earth pressure theory was first developed for purely cohesionless soils (c = 0) and has been extended to cohesive soils afterward
- Mixed soils (as seen earlier) are much more complex in their analysis
- Assumptions behind purely cohesive backfill calculations
  - Lower bound plasticity for behaviour and failure (thus the 2 coefficient shown below)
  - Purely cohesive soils have a lateral earth pressure of unity, thus the vertical and horizontal pressures are the same



(see next slide)

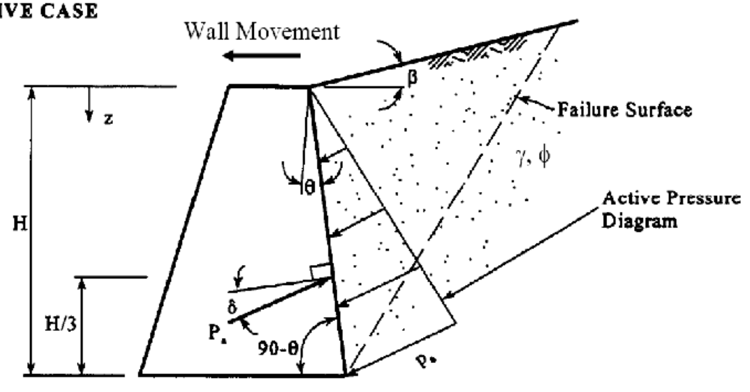
Example of Cohesive Soil, Level Backfill and Uniform Surcharge

- Given
  - Wall similar to those above
  - H = 20’
  - γ = 120 pcf
  - Uniform surcharge q = 100 psf
  - Purely cohesive soil, c = 500 psf
- Find
  - Location of the critical height below the surface
  - Lateral Earth Pressure at Base of Wall
  - Total Resultant Force on Wall
- Solution
  - $H_{crit} = (2c - q)/\gamma$ (from previous slide)
  - $H_{crit} = ((2)(500)-100)/120 =$ 7.5’ (by substitution)
- Solution
  - There are two components to the lateral earth pressure:
    - Part I is due to the self weight of the soil (effective stress)
    - Part II is due to the uniform surcharge
  - Substitution:
    - Part I: $\sigma_{h1}=\gamma H\text{-}2c = (120)(20)\text{-}(2)(500) = 1400$ psf
    - Part II: $\sigma_{h1}=q = 100$ psf
    - Sum = $\sigma_h = 1400 + 100 = 1500$ psf
  - Resultant Force: the earth pressures create a triangle on the wall which is $H - H_{crit}$ high and with a maximum pressure as shown above
    - $P = \sigma_h(H - H_{crit})/2 = (1500)(20\text{-}7.5)/2 = 9375$ lb/ft of wall
    - Resultant is located $(H - H_{crit})/3$ from the base, thus $(20\text{-}7.5)/3 = 4.167$’

Coulomb Earth Pressure Coefficients

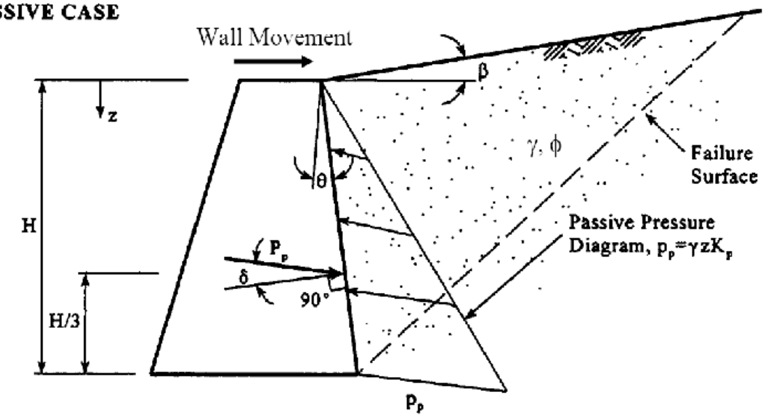
- The following must be noted:
  - Coulomb passive earth pressure coefficients tend to be very unconservative, as discussed in the text. Do not use for high values of δ
  - Verruijt's formulae are the same but use different notations and sign conventions for the geometry and friction angle
  - Values of δ and φ are always positive. Values of β and θ may be either.
  - The height of the wall is the vertical distance from the top of the wall to its base, not the length of the wall (which would be different if $\theta \neq 0$ degrees.

ACTIVE CASE



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

PASSIVE CASE



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

Figure 10-5. Coulomb coefficients K_a and K_p for sloping wall with wall friction and sloping cohesionless backfill (after NAVFAC, 1986b).

Typical Values of Wall Friction

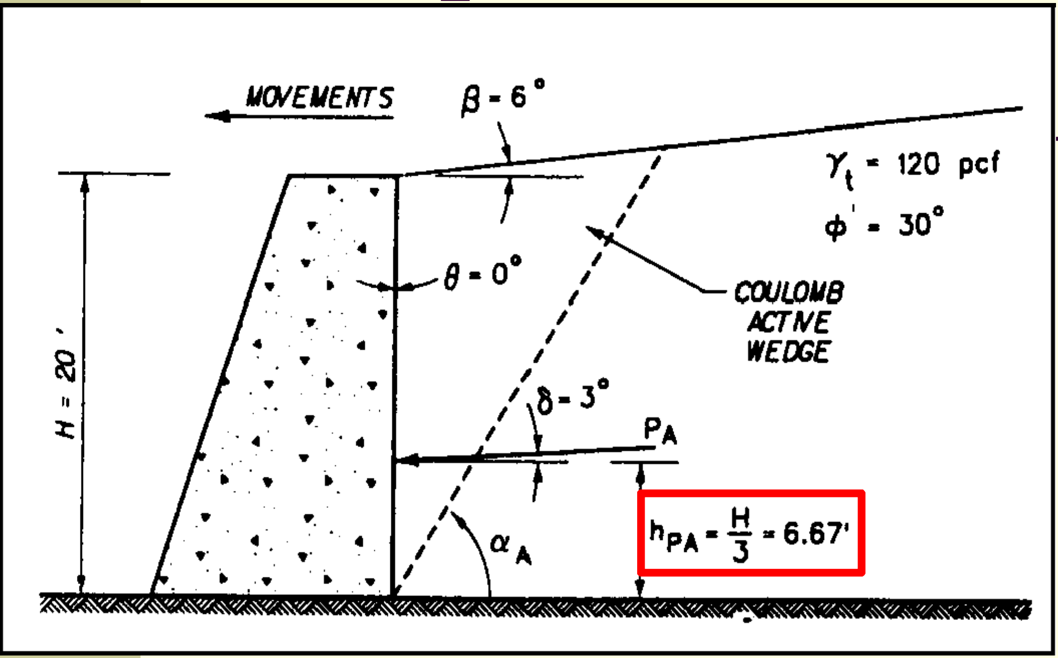
Table 10-1

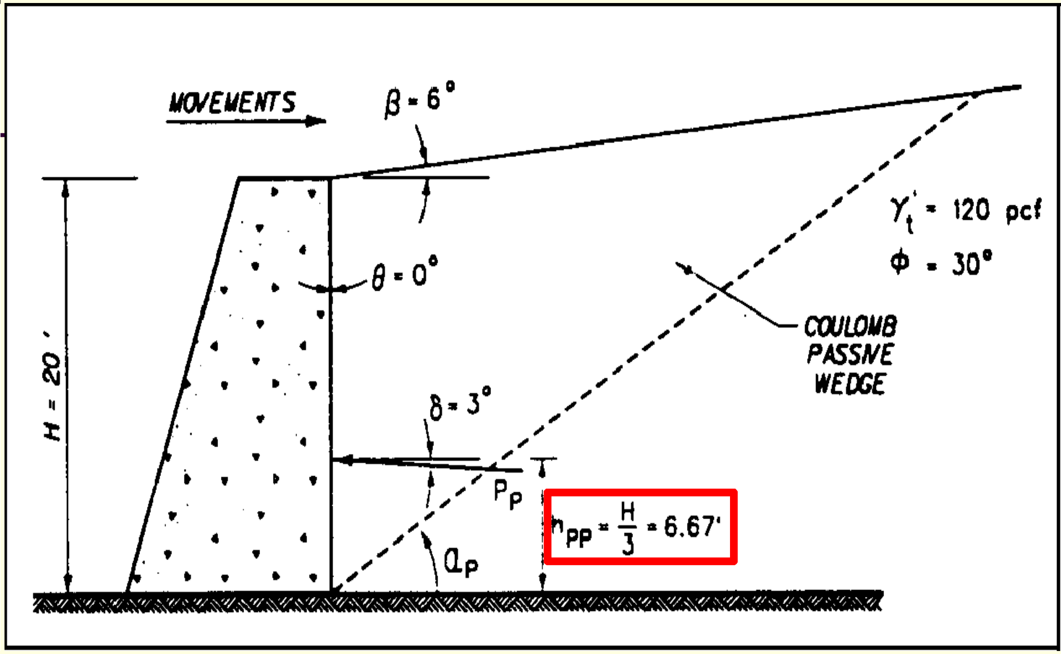
Wall friction and adhesion for dissimilar materials (after NAVFAC, 1986b)

Interface Materials	Friction Factor, tan δ	Friction angle, δ degrees
Mass concrete on the following foundation materials:		
Clean sound rock	0.70	35
Clean gravel, gravel sand mixtures, coarse sand	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay (Masonry on foundation materials has same friction factor)	0.30 to 0.35	17 to 19
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40	22
Clean sand, silty sand-gravel mixtures, single size hard rock fill	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sandy silt, nonplastic silt	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay	0.30	17
Fine sandy silt, nonplastic silt	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock	0.70	35
Dressed hard rock on dressed soft rock	0.65	33
Dressed hard rock on dressed hard rock	0.55	29
Masonry on wood (cross grain)	0.50	26
Steel on steel at sheet pile interlocks	0.30	17

Interface Materials (Cohesion)	Adhesion c_a (kPa)
Very soft cohesive soil (0 - 12 kPa)	0 - 12
Soft cohesive soil (12 - 24 kPa)	12 - 24
Medium stiff cohesive soil (24 - 48 kPa)	24 - 36
Stiff cohesive soil (48 - 96 kPa)	36 - 45
Very stiff cohesive soil (96 - 192 kPa)	45 - 62

Example of Coulomb Theory





- Given
  - Wall as shown above

- Find
  - K_A
  - K_P
  - P_A
  - P_P

Solution for Coulomb Earth Pressures

- Compute Coulomb Active Pressure

$$K_A = \frac{\cos^2(30-0)}{\cos^2(0)\ \cos(0+3)\left[1 + \sqrt{\frac{\sin(30+3)\sin(30-6)}{\cos(3+0)\cos(6-0)}}\right]^2}$$

  - $K_A = 0.3465$

- Compute Total Wall Force

$$P_A = 0.3465 \cdot \frac{1}{2}(120\text{ pcf})(20')^2$$

  - $P_A = 8316$ lb/ft of wall

- Compute Coulomb Passive Pressure

$$K_P = \frac{\cos^2(30+0)}{\cos^2(0)\cos(3-0)\left[1 - \sqrt{\frac{\sin(30+3)\ \sin(30+6)}{\cos(3-0)\ \cos(6-0)}}\right]^2}$$

  - $K_P = 4.0196$

- Compute Total Wall Force

$$P_p = 4.0196 \cdot \frac{1}{2}(120\text{ pcf})(20')^2$$

  - $P_A = 96{,}470$ lb/ft of wall

Log-Spiral Failure Surface

- A more realistic way to model soil failure in many cases
- Requires use of suitable chart for both active and passive coefficients
- Especially useful to address weakness in Coulomb passive pressure (AASHTO)

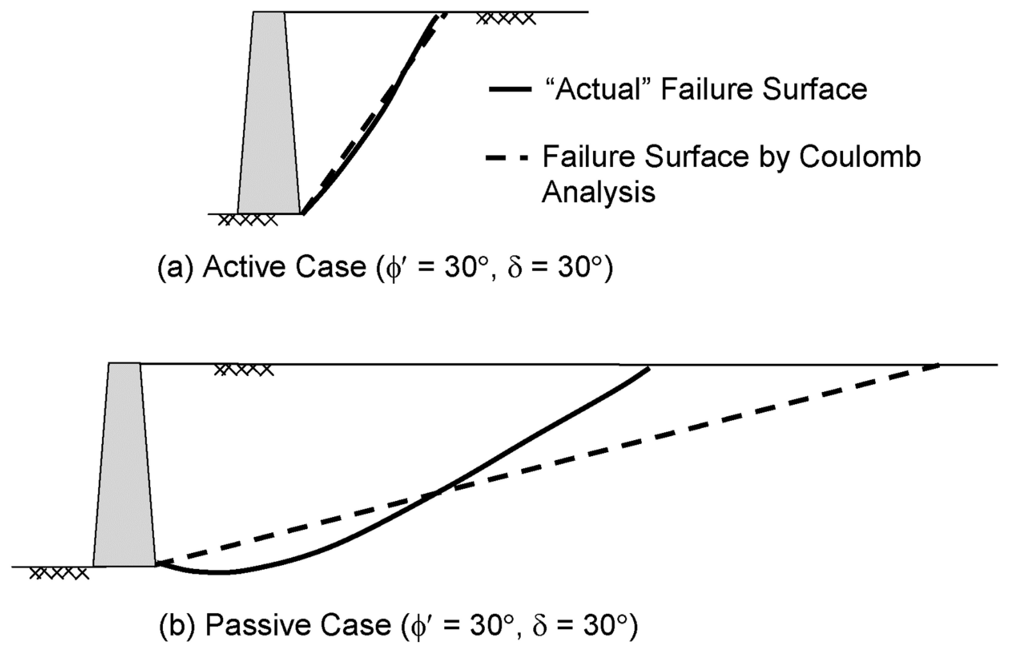


Figure 10-8. Comparison of plane and log-spiral failure surfaces (a) Active case and (b) Passive case (after Sokolovski, 1954) – Note: Depiction of gravity wall is for illustration purpose only.

Differences between Coulomb and Log-Spiral Passive Earth Pressure Coefficients

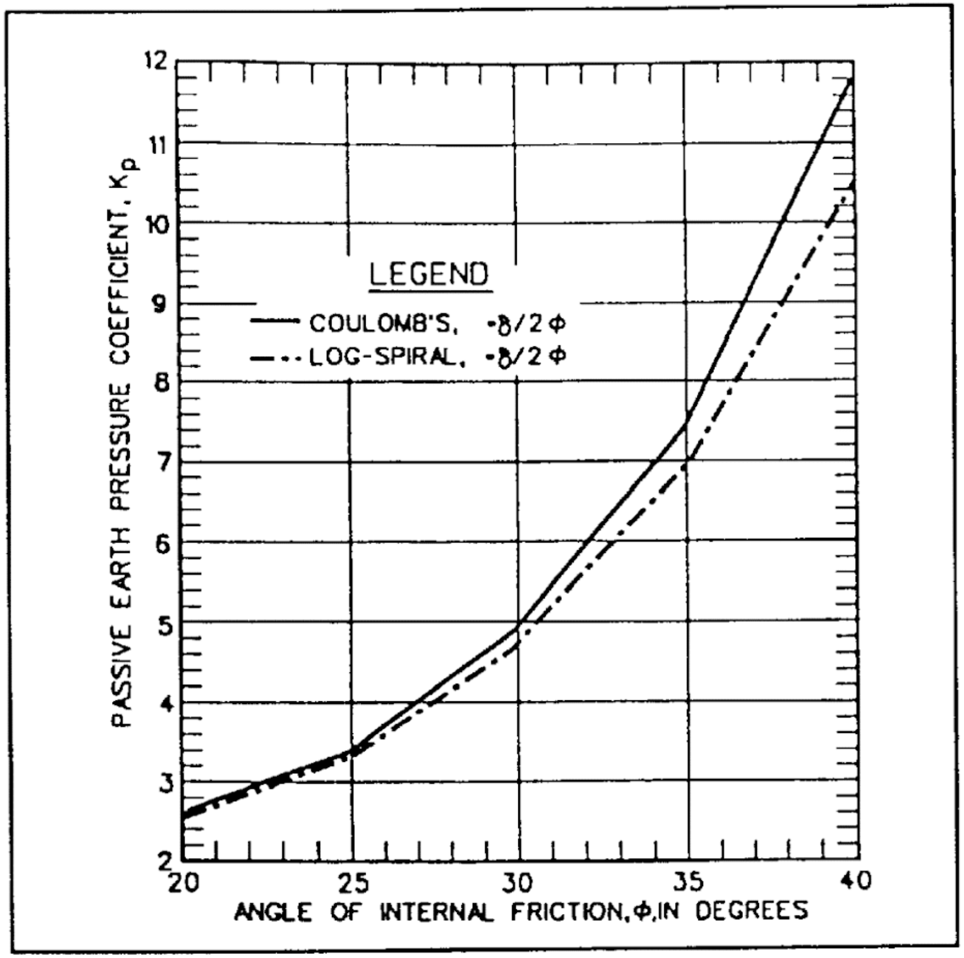


Figure 3.7 Coulomb and log-spiral passive earth pressure coefficients with $\delta = \phi/2$ - vertical wall and level backfill

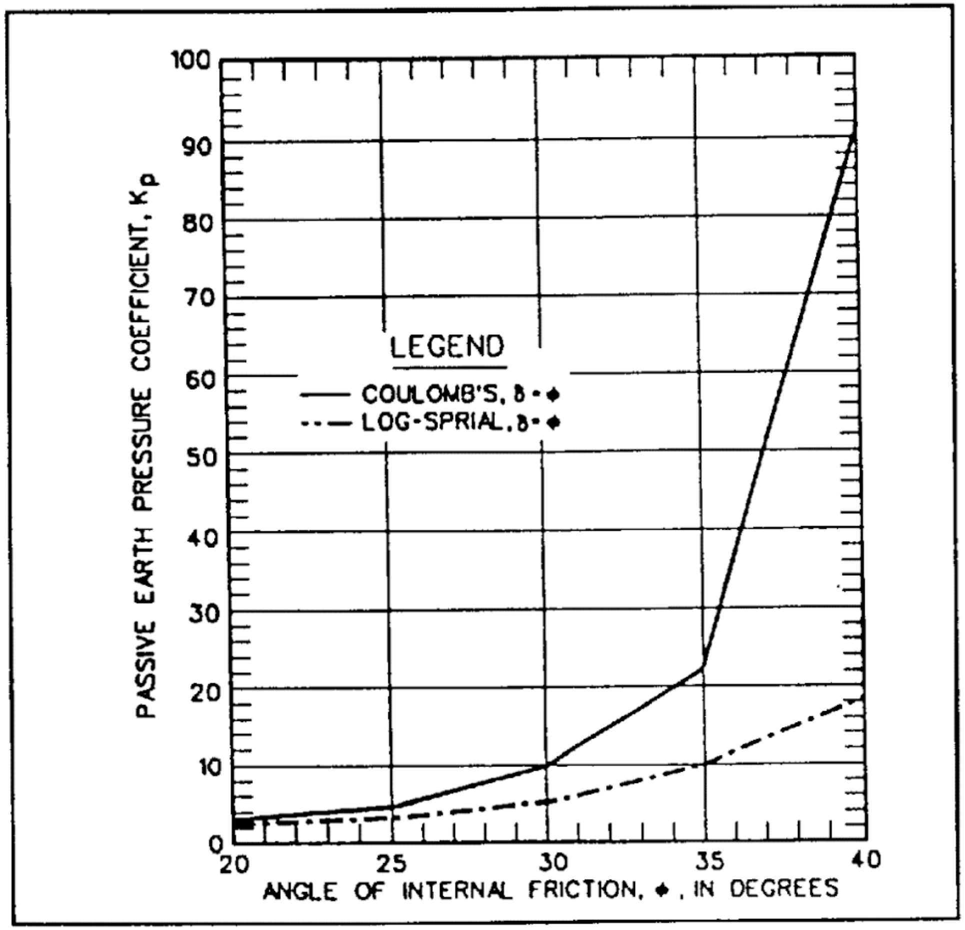
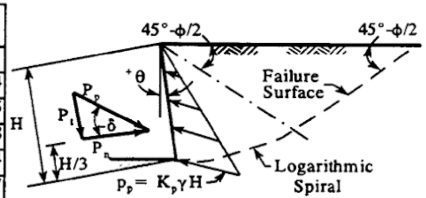


Figure 3.8 Coulomb and log-spiral passive earth pressure coefficients with $\delta = \phi$ - vertical wall and level backfill

Log-Spiral Passive Pressure Charts

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF - δ/ϕ								
ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Note: Curves shown are for $\delta/\phi = -1$



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

$P_n = P_p \cos\delta$

$P_t = P_p \sin\delta$

Example: $\phi = 30°$; $\theta = -10$; $\delta/\phi = -0.6$

$K_p = R(K_p$ for $\delta/\phi = -1) = (0.811)(8.2) = 6.65$

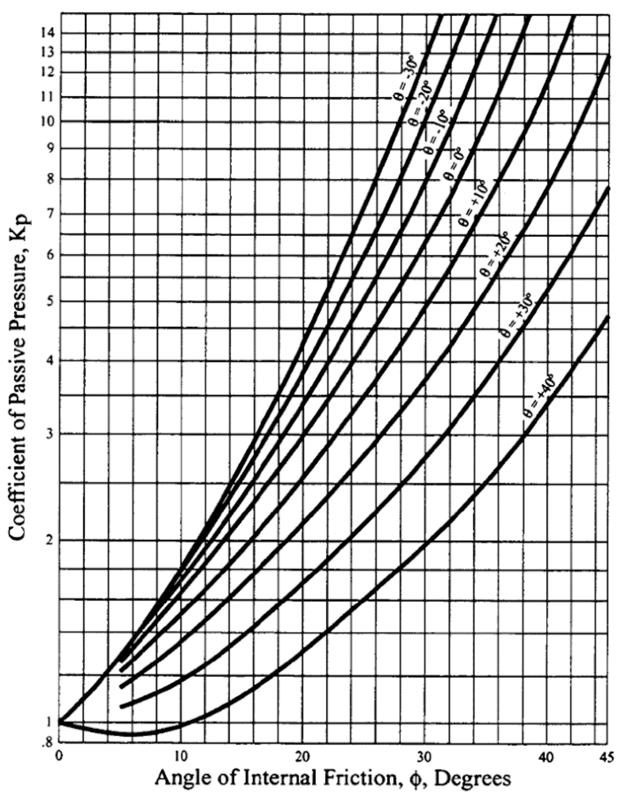
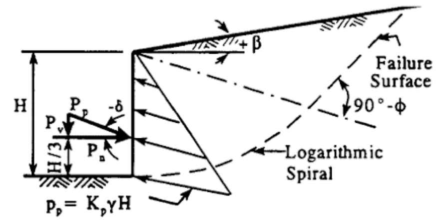


Figure 10-9. Passive coefficients for sloping wall with wall friction and horizontal backfill (Caquot and Kerisel, 1948; NAVFAC, 1986b).

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF - δ/ϕ								
ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Note: Curves shown are for $\delta/\phi = -1$



$P_p = K_p \gamma H^2/2$; $P_n = P_p \cos\delta$; $P_v = P_p \sin\delta$

Example: $\phi = 25°$; $\beta/\phi = -0.2$; $\delta/\phi = -0.3$

$K_p = R(K_p$ for $\delta/\phi = -1) = (0.711)(3.62) = 2.58$

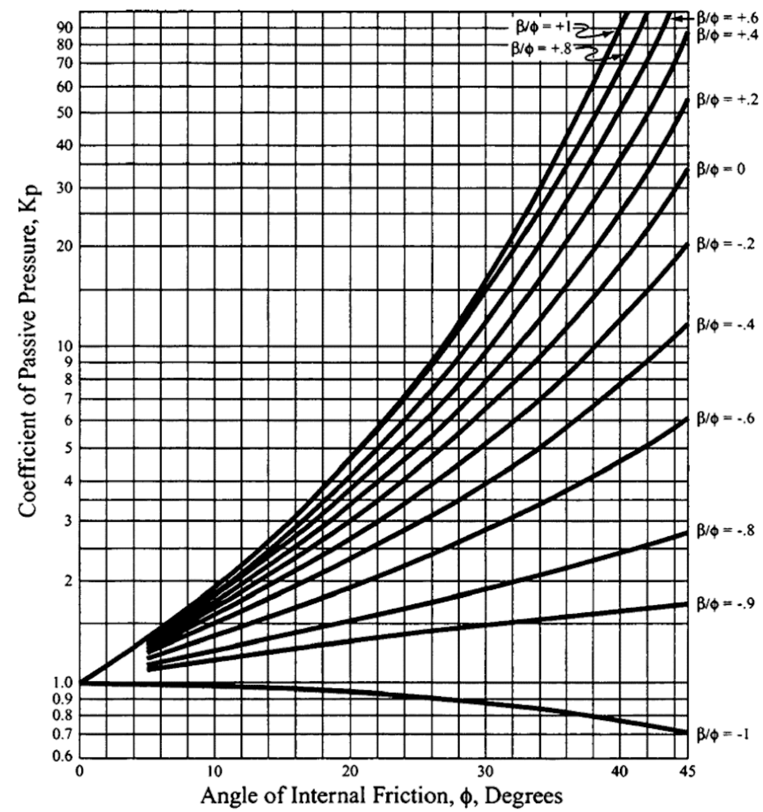


Figure 10-10: Passive coefficients for vertical wall with wall friction and sloping backfill (Caquot and Kerisel, 1948; NAVFAC, 1986b).

Groundwater Effects

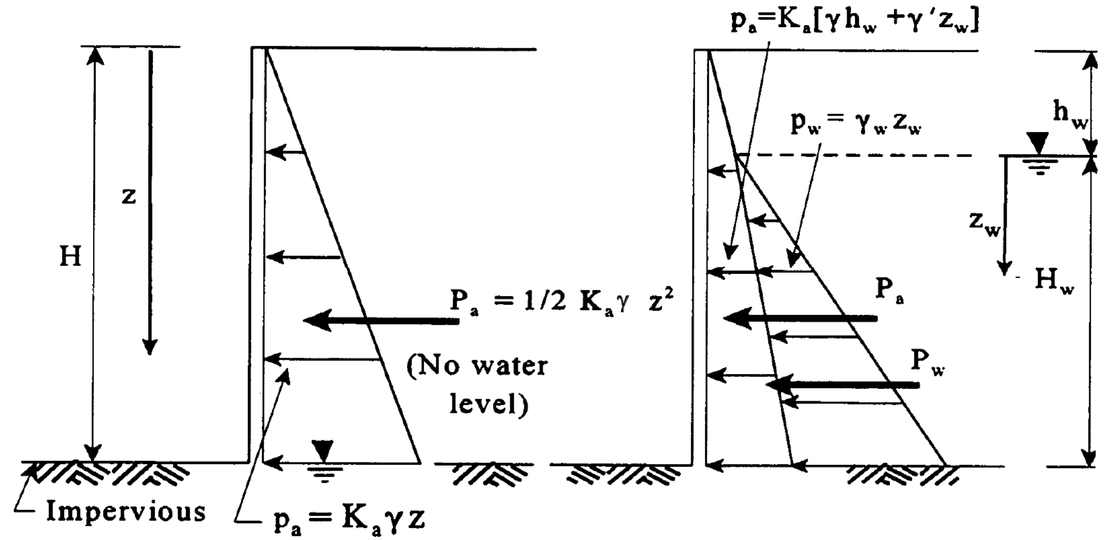
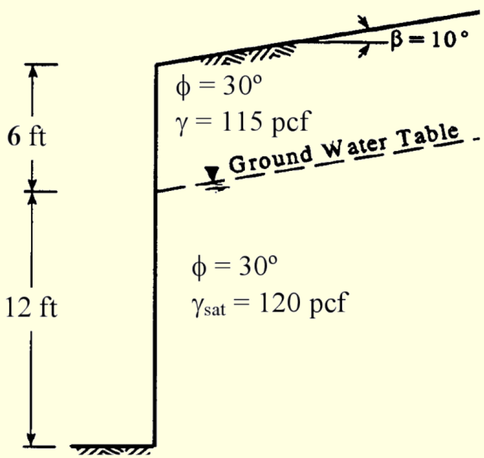


Figure 10-12. Computation of lateral pressures for static groundwater case.

- Steps to properly compute horizontal stresses including groundwater effects:
  - Compute total vertical stress
  - Compute effective vertical stress by removing groundwater effect through submerged unit weight; plot on P_o diagram
  - Compute effective horizontal stress by multiplying effective vertical stress by K
  - Compute total horizontal stress by directly adding effect of groundwater unit weight to effective horizontal stress

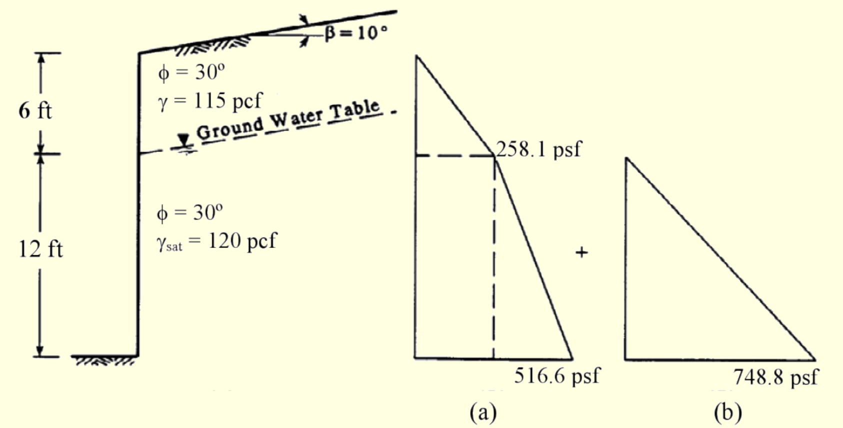
Groundwater Example

Example 10-1: For the wall configuration shown below, construct the lateral pressure diagram. Assume the face of the wall to be smooth ($\delta = 0$, $c_w = 0$).



Hydrostatic Pressure, $u = K_w\, u_w$

z, ft	z_w, ft	$u_w = z_w\, \gamma_w$, psf	Lateral water pressures, u psf
0	0	0	= 0
6	0	0	= 0
18	12	12 ft (62.4 pcf) = 748.8 psf	1.0(748.8 psf) = 748.8 psf



(a) Lateral effective earth pressure diagram and (b) Lateral water pressure diagram.

Solution:

Use the Coulomb method (Figure 10-5) for $\phi = 30^\circ$, $\beta = 10^\circ$, $\theta = 0$, and $\delta = 0$:

$$K_a = 0.374$$

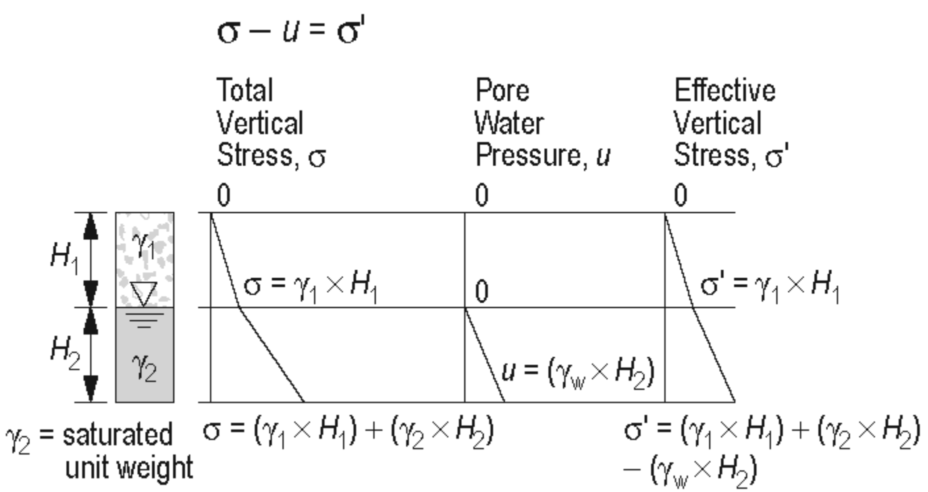
The pressures at various depths can then be calculated as shown in a tabular format as follows. Based on the values in the table, the lateral pressure diagrams due to earth and water can be constructed as shown below. The total lateral pressure diagram is the sum of the two lateral pressure diagrams shown in the figure accompanying this example.

Effective Lateral Earth Pressures, p'_a

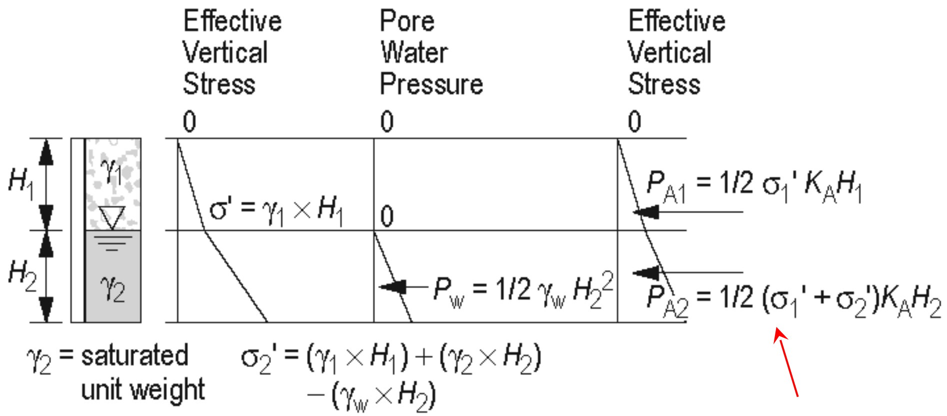
z, ft	p_o, psf		$p_a = K_a\, p_o$, psf
0	0		0
6	(115 pcf) (6 ft)	= 690.0 psf	0.374(690.0 psf) = 258.1 psf
18	690 psf +(120 pcf-62.4 pcf)(12 ft)	= 1381.2 psf	0.374(1381.2 psf) = 516.6 psf

Development of Lateral Earth Pressure and Groundwater Effects

Vertical Stress Profiles



Horizontal Stress Profiles and Forces



Unbalanced!

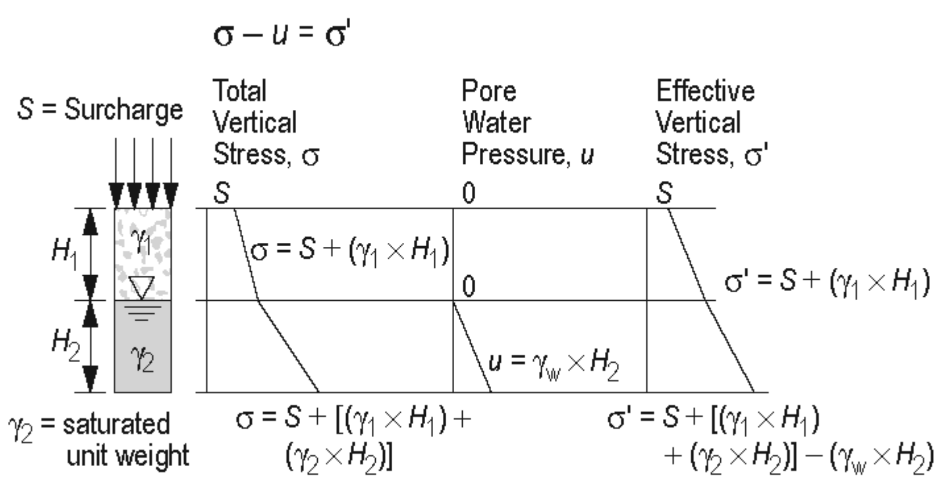
Active forces on retaining wall per unit wall length (as shown):

K_A = Rankine active earth pressure coefficient (smooth wall, $c = 0$, level backfill) $= \tan^2(45° - \phi/2)$

Passive forces on retaining wall per unit wall length (similar to the active forces shown):

K_P = Rankine passive earth pressure coefficient (smooth wall, $c = 0$, level backfill) $= \tan^2(45° + \phi/2)$

Vertical Stress Profiles with Surcharge



Effects of Earthquake Loading

- In seismically active areas, earthquake loading can be a major cause of catastrophic retaining wall failure
- Most common method of analyzing retaining walls for earthquake loads is the Mononobe-Okabe Procedure
  - Developed in Japan in the 1920’s
  - Tends to be conservative
  - A “pseudo-static” method, i.e., turns a dynamic problem into an equivalent static one
  - Although newer methods are coming into use, still the “standard” method for analyzing earthquake loads on retaining walls

Assumptions of M-O Method

(a) The wall is free to yield sufficiently to enable full soil strength or active pressure conditions to be mobilized.

(b) The backfill is completely above or completely below the water table, unless the top surface is horizontal, in which case the backfill can be partially saturated.

(c) The backfill is cohesionless.

(d) The top surface is planar (not irregular or broken).

(e) Any surcharge is uniform and covers the entire surface of the soil wedge.

(f) Liquefaction is not a problem.

M-O Governing Equations (Active and Passive)

Active Earth Pressure Coefficient:

$$K_{AE} = \left(\cos(-\phi+\psi+\theta)\right)^2 \left(\cos(\psi)\right)^{-1} \left(\cos(\theta)\right)^{-2} \left(\cos(\delta+\theta+\psi)\right)^{-1} \left(1+\sqrt{-\frac{\sin(\phi+\delta)\sin(-\phi+\psi+\beta)}{\cos(\delta+\theta+\psi)\cos(-\beta+\theta)}}\right)^{-2}$$

Active Earth Pressure Force (Uniform Case, No Water):

$$P_{AE} = \frac{1}{2} K_{AE} \gamma \left(1-k_v\right) h^2$$

Passive Earth Pressure Coefficient:

$$K_{PE} = \left(\cos(-\phi+\psi-\theta)\right)^2 \left(\cos(\psi)\right)^{-1} \left(\cos(\theta)\right)^{-2} \left(\cos(\delta-\theta+\psi)\right)^{-1} \left(1+\sqrt{-\frac{\sin(\phi+\delta)\sin(-\phi+\psi-\beta)}{\cos(\delta-\theta+\psi)\cos(-\beta+\theta)}}\right)^{-2}$$

Passive Earth Pressure Force (Uniform Case, No Water)

$$P_{PE} = \frac{1}{2} K_{PE} \gamma \left(1-k_v\right) h^2$$

The equations are subject to the same limitations as apply to Coulomb's Equations. Definitions (use the diagrams on the next slide):

- γ =unit weight of soil
- k_v =vertical acceleration, g's
- k_h =horizontal acceleration, g's
- h = height of wall
- ϕ =internal friction angle of soil
- $\psi = \arctan(\frac{k_h}{1-k_v})$ =seismic inertia angle
- θ =inclination of wall angle from the vertical
- δ=wall friction angle
- β =inclination of backfill behind wall

Nomenclature for M-O Equations

Note: M-O passive equations assume planar surfaces, thus are subject to same limitations as Coulomb method. Book charts for passive pressures based on log-spiral failure surface, should use these instead of equations

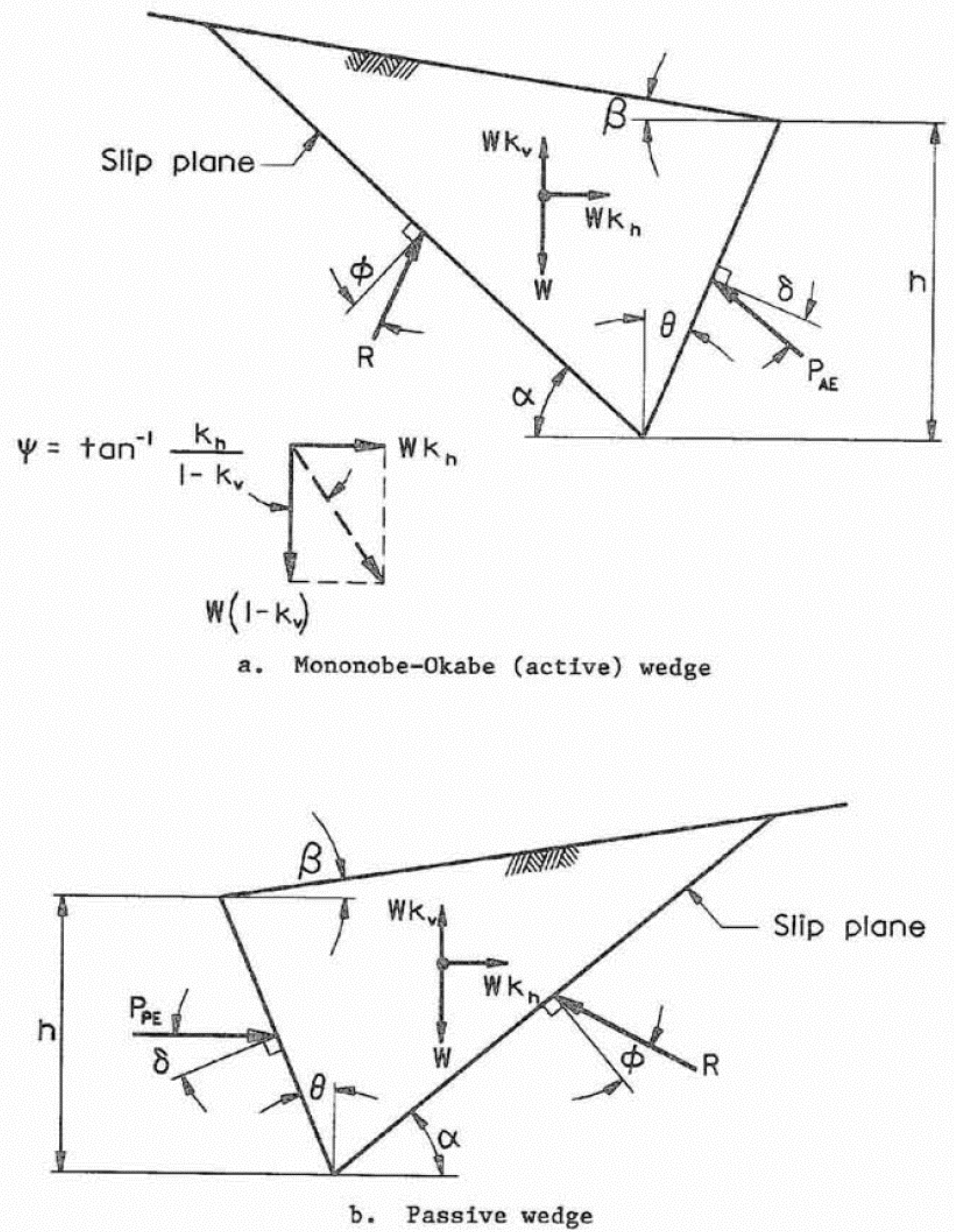


Figure 3-34. Driving and resisting seismic wedges, no saturation

Simplified M-O Equations

(3) Simplifying Conditions. For the usual case where k_v, δ, and θ are taken to be zero, the equations reduce to:

$$K_{AE} = \frac{\cos^2(\phi - \Psi)}{\cos^2\Psi\left[1 + \sqrt{\dfrac{\sin\phi\,\sin(\phi - \Psi - \beta)}{\cos\beta\,\cos\Psi}}\right]^2} \qquad [3\text{-}48]$$

$$K_{PE} = \frac{\cos^2(\phi - \Psi)}{\cos^2\Psi\left[1 - \sqrt{\dfrac{\sin\phi\,\sin(\phi - \Psi + \beta)}{\cos\beta\,\cos\Psi}}\right]^2} \qquad [3\text{-}49]$$

where

$$\Psi = \tan^{-1}(k_h)$$

and

$$P_{AE} = 1/2\ K_{AE}\gamma h^2$$

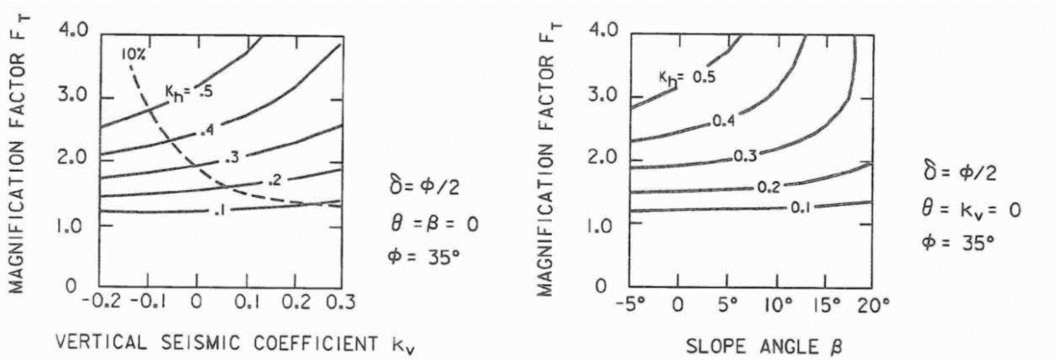
$$P_{PE} = 1/2\ K_{PE}\gamma h^2$$

Observations on the M-O Equations

(4) Observations. General observations from using Mononobe-Okabe analysis are as follows:

(a) As the seismic inertia angle Ψ increases, the values of K_{AE} and K_{PE} approach each other and, for a vertical backfill face ($\theta = 0$), become equal when $\Psi = \phi$.

(b) The locations of P_{AE} and P_{PE} are not given by the Mononobe-Okabe analysis. Seed and Whitman (1970) suggest that the dynamic component ΔP_{AE} be placed at the upper one-third point, ΔP_{AE} being the difference between P_{AE} and the total active force from Coulomb's active wedge without the earthquake. The general wedge earthquake analysis described in paragraph 3-26c places the dynamic component ΔP_{AE} at the upper one-third point also, but computes ΔP_{AE} as being the difference between P_{AE} and the total active



a. Influence of vertical seismic coefficient on magnification factor

b. Influence of backfill slope angle on magnification factor

Figure 3-35. Influence of k_v and β on magnification factor (after Applied Technology Council 1981)

force from the Mononobe-Okabe wedge. The latter method for computing ΔP_{AE} , which uses the same wedge for computing the static and dynamic components of P_{AE} , is preferred.

(c) Another limitation of the Mononobe-Okabe equation is that the contents of the radical in the equation must be positive for a real solution to be possible, and for this it is necessary that $\phi \geq \Psi + \beta$ for the driving wedges and $\phi \geq \Psi - \beta$ for the resisting wedges. This condition could also be thought of as specifying a limit to the horizontal acceleration coefficient that could be sustained by any structure in a given soil. The limiting condition for the driving wedge is:

$$k_h \leq (1 - k_v) \tan(\phi - \beta) \qquad [3\text{-}50]$$

and for the resisting wedge:

$$k_h \leq (1 - k_v) \tan(\phi + \beta) \qquad [3\text{-}51]$$

(d) Figure 3-35a (Applied Technology Council 1981) shows the effect on the magnification factor F_T (equal to K_{AE}/K_A) on changes in the vertical acceleration coefficient k_v. Positive values of k_v have a significant effect for values of k_v greater than 0.2. The effect is greater than 10 percent above and to the right of the dashed line. For values of k_h of 0.2 or less, k_v can be neglected for all practical purposes.

(e) K_{AE} and F_T are also sensitive to variations in backfill slope, particularly for higher values of horizontal acceleration. This effect is shown in Figure 3-35b.

Questions