## Index and Classification Properties of Soils

Unit weight : $\gamma = \frac{W}{V}$

Dry unit weight : $\gamma_d = \frac{G_s \gamma_w}{1+e}$

Moist unit weight : $\gamma = \gamma_{dry}(1+\omega)$

Saturated unit weight: $\gamma_{sat} = \frac{(Gs+e)\gamma_w}{1+e}$

Zero air void unit weight:

$$\gamma_{zav} = \frac{Gs\gamma_w}{1+e}$$

Moisture content (water content)

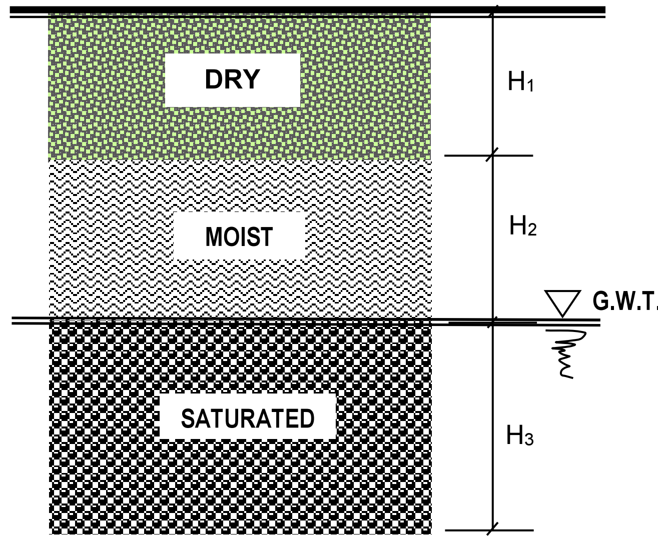
$$\omega = \frac{W_w}{W_s}$$

Degree of saturation : $s = \frac{\omega G_s}{e}$

Porosity : $n = \frac{e}{1+e}$

Air void ratio = n (1 - s)

where : W = total weight
V = volume
$G_s$ = specific gravity
e = void ratio
$\gamma_w$ = unit weight of water
= **9.81 kN/m³** ( 1gram/cm³ )




Phases of Soil

G.W.T. = ground water table

## Relative density ( Density index )

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \quad \text{or}$$

$$D_r = \frac{1/\gamma_{d\,min} - 1/\gamma_d}{1/\gamma_{d\,min} - 1/\gamma_{d\,max}}$$

$e$ = in-situ void ratio
$e_{max}$ = void ratio in loosest condition
$e_{min}$ = void ratio in densest condition
$\gamma_d$ = dry unit weight in the field
$\gamma_{d(max)}$ = dry unit weight in densest condition
$\gamma_{d(min)}$ = dry unit weight in loosest condition

## Shrinkage limit : S.L.

$$S.L. = \frac{(m_1 - m_2)}{m_2} - \frac{(V_1 - V_2)}{m_2}\rho_w \text{ in \%}$$

$m_1$ = initial mass in saturated state
$m_2$ = final mass in dry state
$V_1$ = initial volume in saturated state
$V_2$ = final volume in dry state

## Plasticity Index : P.I.

P.I. = L.L. - P.L.
L.L. = liquid limit
P.L. = plastic limit

## Liquidity Index : L.I.

$$L.I. = \frac{\omega - P.L.}{P.I.}$$

$\omega$ = in-situ moisture content
P.L. = plastic limit
P.I. = plasticity index

## Consistency Index : C.I.

$$C.I. = \frac{L.L. - \omega}{L.L. - P.I.}$$

| *Soil Characteristic* | *Liquidity Index* |
|---|---|
| brittle solid | < 0 |
| plastic | < 1 |
| liquid | >1 |

## Shrinkage Ratio : S.R.

$$S.R. = \frac{m_2}{V_2\rho_w}$$

## Specific Gravity: $G_s$

$$G_s = \frac{1}{\frac{1}{SR} - \frac{SL}{100}}$$

# SOIL CLASSIFICATION

## U.S. Dept of Agriculture (USDA Method)

| Gravel | Sand | Silt | Clay |
|---|---|---|---|
| > 2 mm | 2 to 0.05mm | 0.05 to 0.002mm | < 0.002 mm |

## AASHTO Method

| Gravel | Sand | Silt | Clay |
|---|---|---|---|
| 76.2 to 2mm | 2 to 0.075mm | 0.075 to 0.002mm | < 0.002 mm |

## Unified Soil Classification System

| Gravel | Sand | Fines (Silt & Clay) |
|---|---|---|
| 76.2 to 4.75 mm | 4.75 to 0.075 mm | < 0.075 mm |

## Particle Size Distribution

### Effective Size, $D_{10}$

- is the diameter in the particle size distribution curve corresponding to 10% finer.

### Uniformity coefficient, $C_u$

$$C_u = \frac{D_{60}}{D_{10}}$$

### Coefficient of gradation or curvature, $C_c$

$$C_c = \frac{(D_{30})^2}{D_{60} \cdot D_{10}}$$

$D_{30}$ = particle diameter corresponding to 30% finer

$D_{60}$ = particle diameter corresponding to 60% finer

### Sorting Coefficient, $S_o$

$$S_o = \sqrt{\frac{D_{75}}{D_{25}}}$$

$D_{75}$ = particle diameter corresponding to 75% finer

$D_{25}$ = particle diameter corresponding to 25% finer

### AASHTO Classification System

Group Index, G.I.

$$= (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10)$$

Partial Group Index

$$= 0.01(F - 15)(PI - 10)$$

F = % passing sieve # 200

LL = liquid limit

PI = plasticity index

Group index must be whole and positive, if the computed value is negative use zero. If the group classification is A-2-6 & A-2-7 attached Partial Group Index only.

## SOIL COMPACTION

Compaction – is the densification of soils by the application of mechanical energy. It may also involve a modification of the water content as well as gradation of the soil.

Some methods for determining density of soil in the field:

1. Sand Cone Method
2. Balloon Method
3. Oil Method

To determine the dry unit weight of compaction in the field:

$$\gamma_d = \frac{\text{dry unit weight excavated from the hole}}{\text{volume of the hole}}$$

$$\text{Volume of hole, } V = \frac{W_s - W_c}{\gamma_{d(sand)}}$$

$W_s$ = weight of sand to fill the hole and cone

$W_c$ = weight of sand to fill the cone only

Relative Compaction, R

$$R = \frac{\gamma_{d(field)}}{\gamma_{d(\text{max-lab})}}$$

$\gamma_{d(field)}$ = compacted field dry unit weight

$\gamma_{d(\text{max-lab})}$ = maximum dry unit weight determined in the laboratory by Proctor Test

## PERMEABILITY, SEEPAGE of WATER in SOIL

### Coefficient of Permeability, k
( Laboratory Test)

Constant Head Test : $k = \dfrac{QL}{Aht}$

Q = volume of water collected
A = area of cross section of soil specimen
t = duration of water collection
i = hydraulic gradient = $\dfrac{h}{L}$

Falling Head Test: $k = \dfrac{aL}{At}\ln\dfrac{h_1}{h_2}$

a = cross-sectional area of the stand pipe
A = cross-sectional area of soil specimen
t = duration of water collection
$h_1$ = initial head when $t_1 = 0$
$h_2$ = final head when $t_2 = t$
L = length of soil specimen
i = hydraulic gradient = $\dfrac{h_1 \quad h_2}{L}$

### Rate of water flow in soil, Q

$$Q = k\, i\, A$$

k = coefficient of permeability
i = hydraulic gradient
A = cross-sectinal area of soil sample

### Discharge velocity, V
$$V = k\, i$$

### Seepage velocity, $V_s$
$$V_s = \frac{V}{n}$$

n = porosity

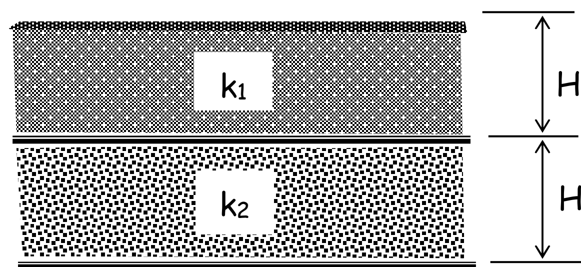
### Absolute Permeability, K
$$K = \frac{k\,\eta}{\gamma_w}$$

k = coefficient of permeability
η = viscosity of water
$\gamma_w$ = unit weight of water

### Transmissibilty of Soil Stratum, T
$$T = K\, b$$

K = average coefficient of permeability
b = thickness of aquifer

### Equivalent Coefficient of Permeability in Layered Soil:



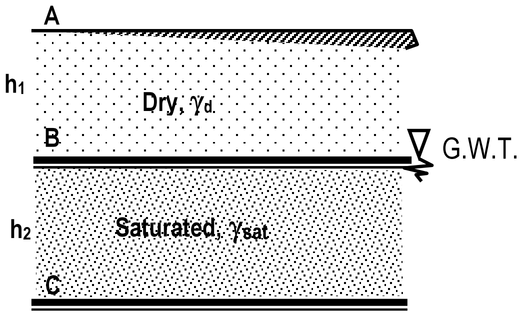

Equivalent Horizontal Coeffient, $K_{H(eq)}$

$$K_{H\,(eq)} = \frac{1}{H}(k_1H_1 + k_2H_2 + \dots + k_nH_n)$$

Equivalent Vertical Coefficient, $K_{V(eq)}$

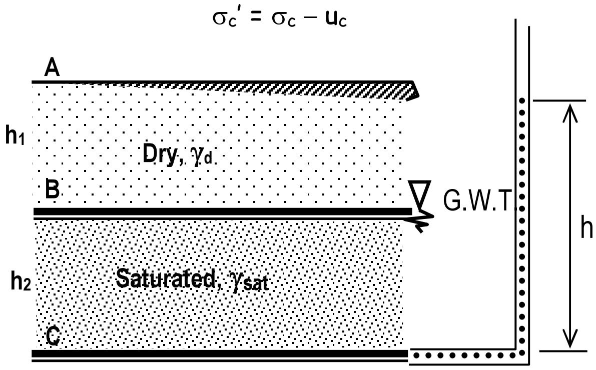
$$K_{V(eq)} = \frac{H}{\dfrac{H_1}{k} + \dfrac{H_2}{k_2} + \dots + \dfrac{H_n}{k_n}}$$

## Vertical Stresses in Soil



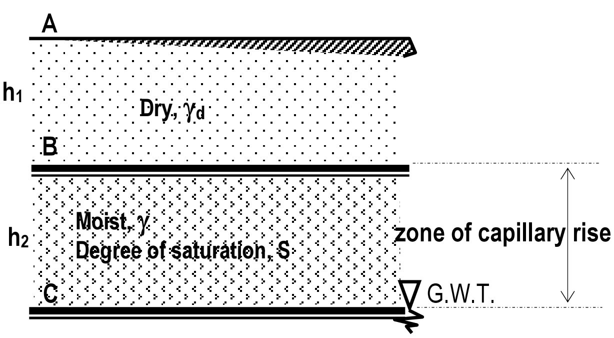

1. Total stress at $C$ : $\sigma_c = \gamma_d h_1 + \gamma_{sat} h_2$
2. Pore water pressure at $C$ : $u_c = \gamma_w h_2$
3. Effective stress at $C$ = Total stress – Pore pressure

$$\sigma_c' = \sigma_c - u_c$$




1. Total stress at $C$ : $\sigma_c = \gamma_d h_1 + \gamma_{sat} h_2$
2. Pore water pressure at $C$ : $u_c = \gamma_w h$
3. Effective stress at $C$ = Total stress – Pore pressure

$$\sigma_c' = \sigma_c - u_c$$




1. Total stress at $C$ : $\sigma_c = \gamma_d h_1 + \gamma h_2$

$$\sigma_B = \gamma_d h_1$$

2. Pore water pressure at $C$ : $u_B = -S \gamma_w h_2$

$$u_c = 0$$

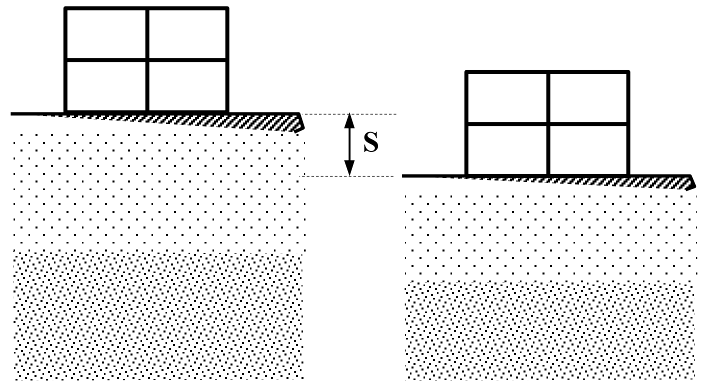
3. Effective stress at $C$ = Total stress – Pore pressure

$$\sigma_c' = \sigma_c - u_c$$

$$\sigma_B' = \sigma_B - (-u_B)$$

## Compressibility of Soil

**Settlement** – the total vertical deformation at the surface resulting from the load. When a soil deposit is loaded (by a structure or a man-made fill) deformation will occur. The movement may be downward with an increase in load or upward (called swelling) with a decrease in load.




Components of Settlement:

$$S = S_1 + S_2 + S_3$$

$S$ = total settlement
$S_1$ = immediate or distortion settlement
$S_2$ = primary consolidation settlement
$S_3$ = secondary consolidation settlement

**Primary Consolidation Settlement, $S_2$**

Normally Consolidated Clays

$$S_2 = \frac{C_c H}{1 + e_0} \log \frac{P_o + \Delta P}{P_o}$$

$C_c$ = compression index
$e_o$ = in-situ void ratio
$P_o$ = effective overburden pressure at the middle of the clay layer
$\Delta P$ = average increase of stress on clay layer
$H$ = thickness of clay layer

Over – Consolidated Clays

1. when $P_o + \Delta P < P_c$

$$S_2 = \frac{C_s H}{1+e_0}\log\frac{P_o+\Delta P}{P_o}$$

$C_s$ = swell index

= ranges from 1/5 to 1/10 of $C_c$

$P_c$ = preconsolidation pressure

2. when $P_o + \Delta P > P_c$

$$S_2 = \frac{C_s H}{1+e_0}\log\frac{P_c}{P_o} + \frac{C_c H}{1+e_0}\log\frac{P_o+\Delta P}{P_c}$$

**Secondary Consolidation Settlement, $S_3$**

$$S_3 = C'_a H \log\frac{t_2}{t_1}$$

$$C'_a = \frac{C_a}{1+e_p}$$

$$C_a = \frac{\Delta e}{\log\frac{t_2}{t_1}}$$

$C_a$ = secondary compression index

$e_p$ = void ratio at the end of primary consolidation

= $e_o - \Delta e$

$$\Delta e = C_c \log\frac{P_o+\Delta P}{P_o}$$

$t_1$ = time for completion of primary consolidation

$t_2$ = time after completion of primary consolidation

**Immediate Settlement, $S_1$**

$$S_1 = C_s\, q\, B\, \frac{(1-\mu^2)}{E_s}$$

$C_s$ = shape and foundation rigidity factor

q = pressure due to load

B = width of foundation or diameter of circular foundation

$\mu$ = Poisson's ratio of soil

$E_s$ = modulus of elasticity of soil

Compression Index, $C_c$

$C_c$ = 0.009 (LL -10) remolded clays

$$C_c = \frac{e_1 - e_2}{\log\frac{P_2}{P_1}}$$

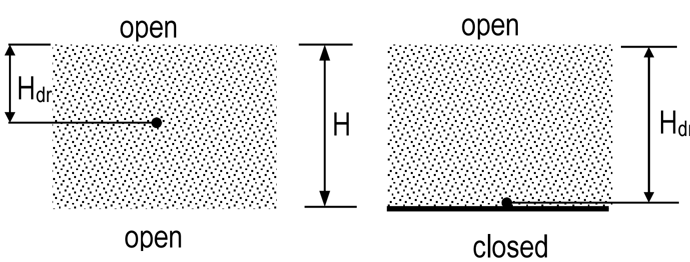
Coefficient of Compressibility, $a_v$

$$a_v = \frac{e_1 - e_2}{P_2 - P_1}$$

Coefficient of Volume Compressibility, $m_v$

$$m_v = \frac{a_v}{1+e_{ave}}$$

**Time Rate of Consolidation, t**




$$T_v = \frac{C_v t}{H^2_{dr}}$$

$C_v$ = coefficient of consolidation

$T_v$ = time factor

$H_{dr}$ = drainage distance of water

**Preconsolidation Pressure, $P_c$**

$$OCR = \frac{P_c}{P_o}$$

OCR = overconsolidation ratio

$P_c$ = preconsolidation pressure

$P_o$ = soil overburden pressure

## Bearing Capacity of Foundations

### Terzaghi's Bearing-Capacity

Ultimate Bearing Capacity, $q_{ult}$

A. General Shear Failure (Dense sand and Stiffclay)

1. Square Footing

   $q_{ult} = 1.3c\,N_c + q\,N_q + 0.4\,\gamma\,B\,N_\gamma$

2. Circular Footing

   $q_{ult} = 1.3c\,N_c + q\,N_q + 0.3\,\gamma\,B\,N_\gamma$

c = cohesion

q = effective pressure at the bottom of the footing

B = width of footing or diameter of footing

$N_c\ N_q\ N_\gamma$ = bearing capacity factors

Bearing Capacity Factors

1. see Tables
2. If no table available

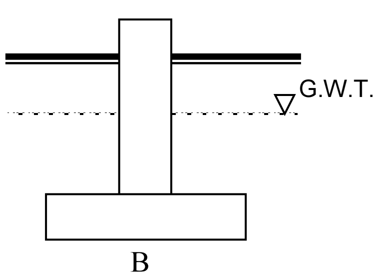
   $N_c = (N_q - 1)\cot\phi$

   $N_q = e^{\pi \tan\phi}\tan^2(45 + \frac{\ }{2})$

   $N_\gamma = (N_q + 1)\tan 1.4\phi$

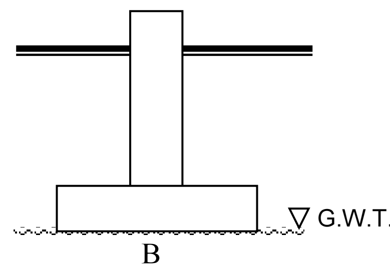
Modification of $\gamma$ value in the 3rd term of the ultimate bearing capacity equation:

Case 1: the water table is located above the bottom of the foundation



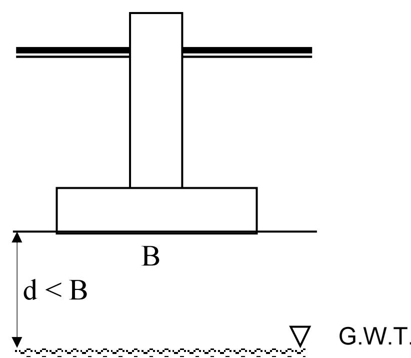

$\gamma = \gamma_{sat} - \gamma_w$

Case 2: the water table is located at the bottom of the foundation




$\gamma = \gamma_{sat} - \gamma_w$

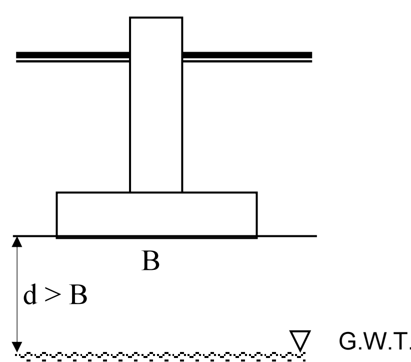
Case 3: the water table is located so that d < B




$$\gamma = \frac{1}{B}\left[\gamma_{dry}(d) + \gamma'(B - d)\right]$$

$\gamma' = \gamma_{sat} - \gamma_w$

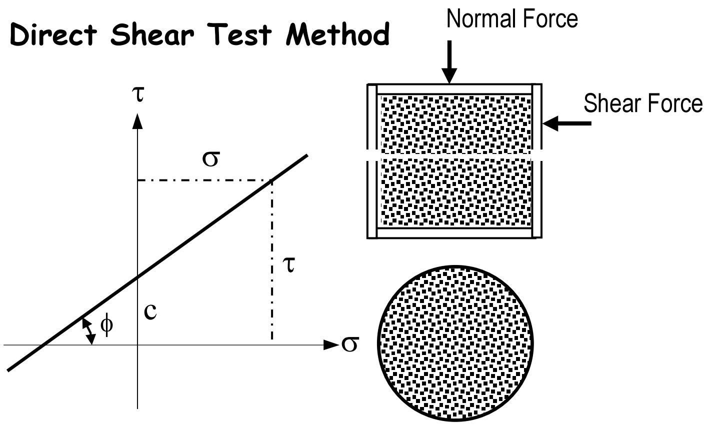
Case 4: the water table is located so that d > B




$\gamma = \gamma_{dry}$

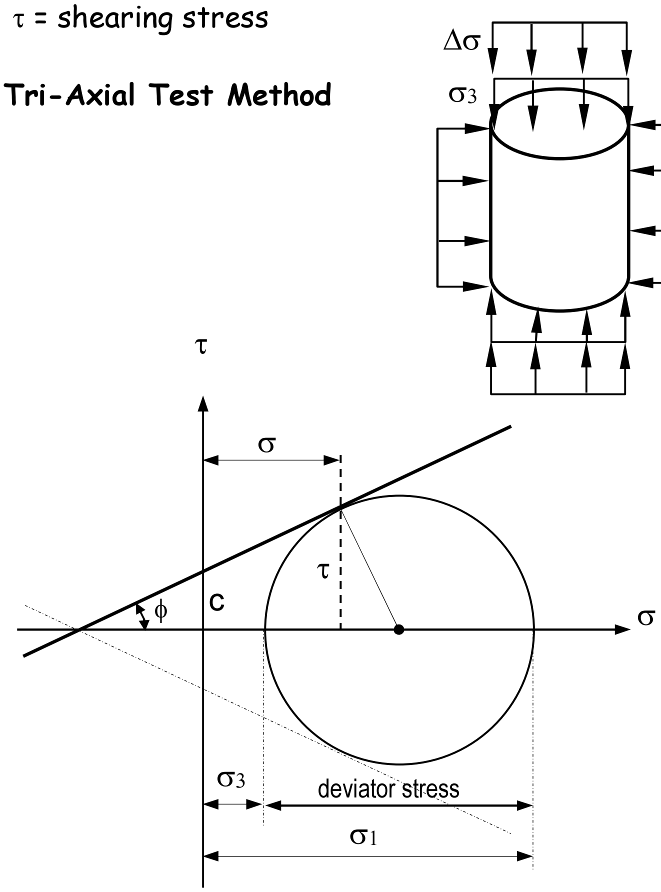
# Shear Strength of Soil

## Direct Shear Test Method



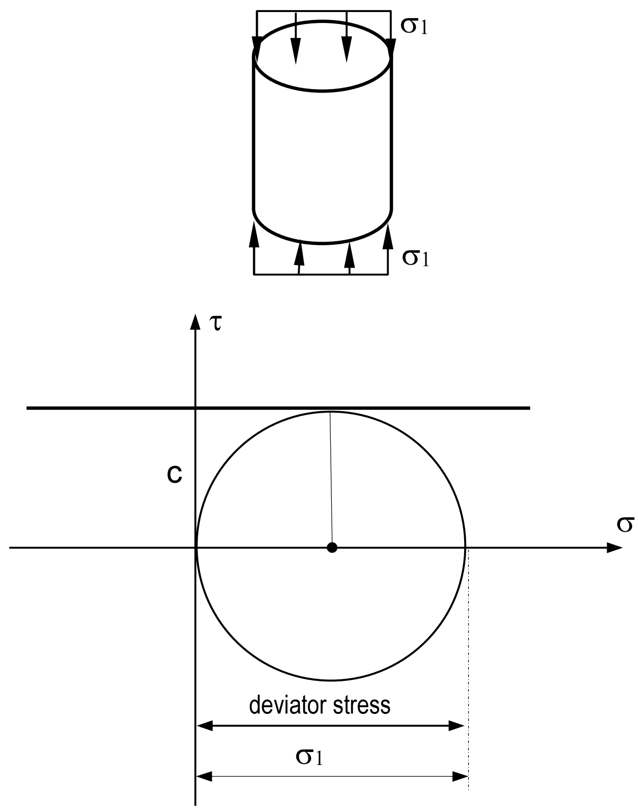

$\phi$ = angle of internal friction
c = cohesion
= 0 if normally consolidated clay
$\sigma$ = normal stress
$\tau$ = shearing stress

## Tri-Axial Test Method

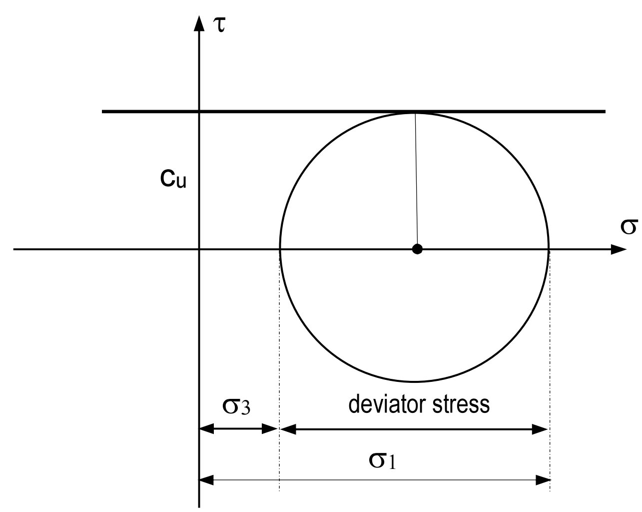



$\sigma_3$ = chamber confining pressure, cell pressure
= minor principal stress
$\sigma_1$ = major principal stress
$\Delta\sigma$ = deviator stress

## Unconfined Compression Test Method

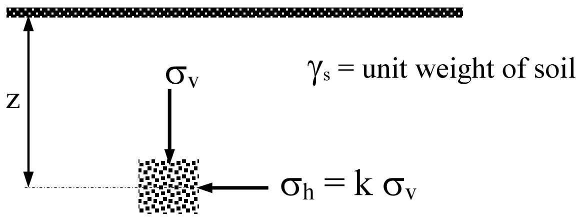



## Unconsolidated - Undrained Test Method




$c_u$ = undrained shear strength

## Lateral Earth Pressure




$\sigma_v$ = vertical pressure at depth z
$\sigma_h$ = lateral pressure at depth z
k = coefficient of lateral earth pressure

Lateral Earth Pressure on Retaining Wall with Horizontal Backfill

At rest earth pressure coefficient, $k_o$
(normally consolidated soil)

$k_o = 1 - \sin\phi$ $\phi$ = angle of internal friction

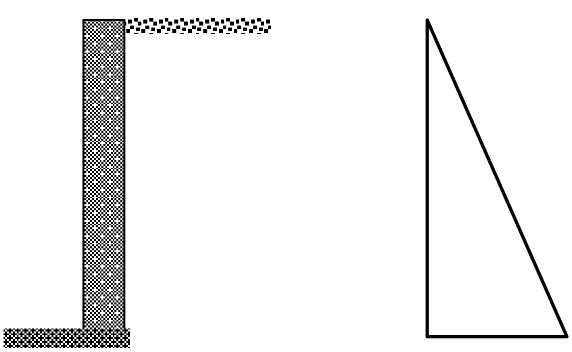
Rankine active earth pressure, $k_a$

$$k_a = \frac{1-\sin\phi}{1+\sin\phi}$$

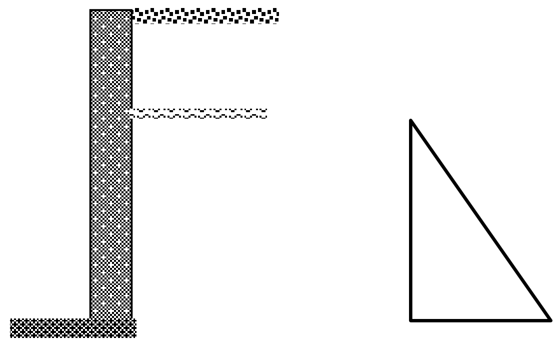
Rankine passive earth pressure, $k_p$

$$k_p = \frac{1+\sin\phi}{1-\sin\phi}$$

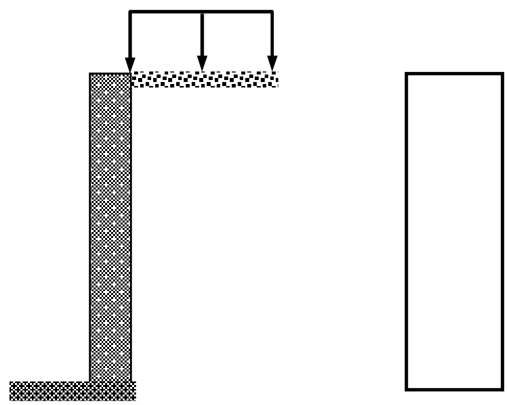
Pressure diagram due to effective unit weight of soil:



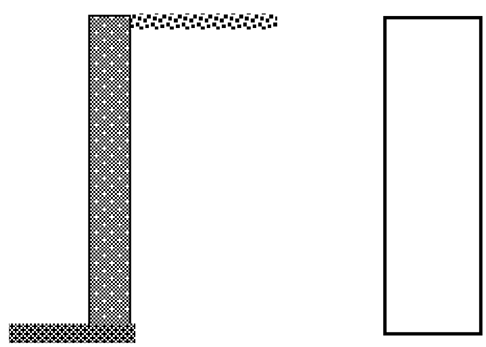
Pressure diagram due to water:



Pressure diagram due to surcharge:

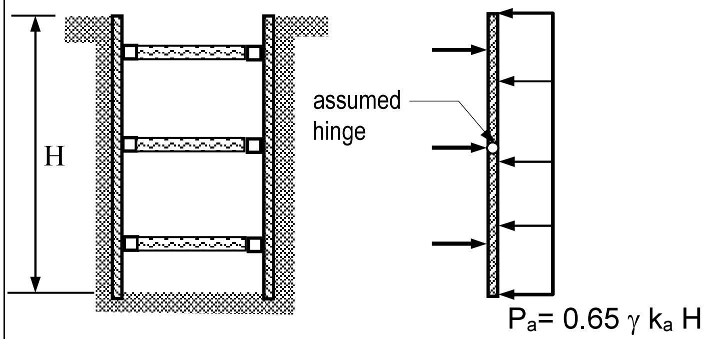


Pressure diagram due to cohesion of soil:

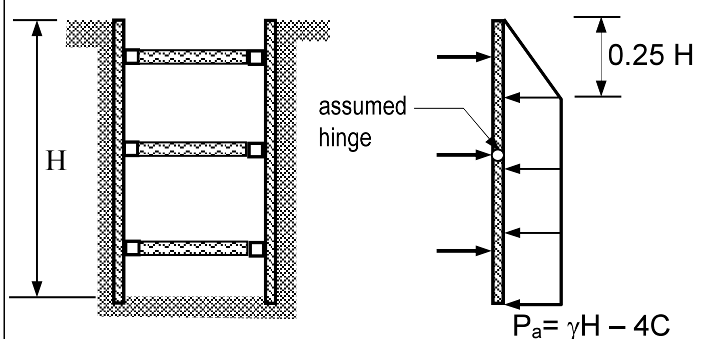


## Braced Sheetings

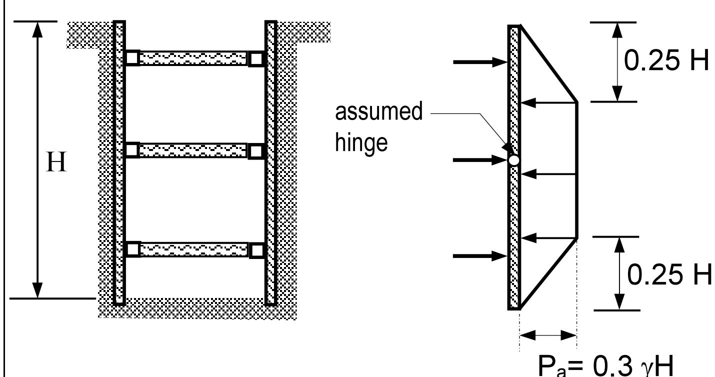
### Cuts in Sand




### Cuts in Clay when $\frac{\gamma H}{C} > 4$



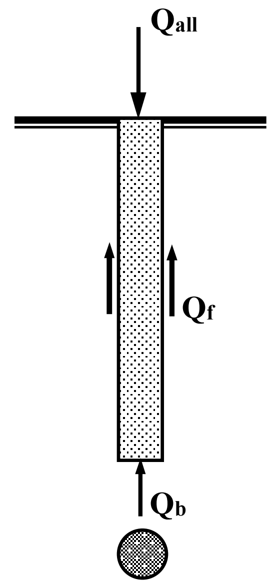

### Cuts in Clay when $\frac{\gamma H}{C} < 4$




$k_a$ = Rankine active pressure coefficient
C = cohesion of clay
$\gamma$ = unit weight of soil
H = depth of cut

## Piles on Clay

### $\alpha$ method



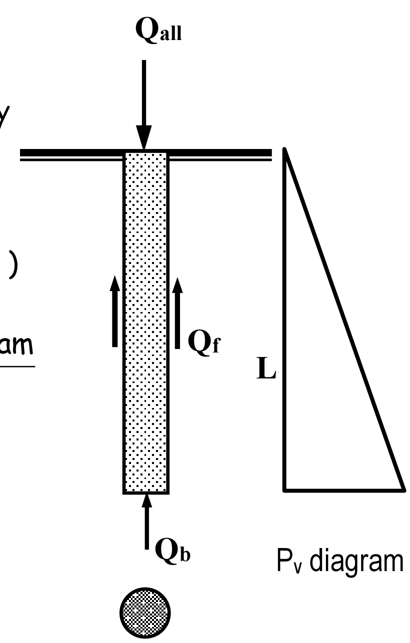

End bearing capacity
$Q_b = C\ N_c\ A_{tip}$

Frictional capcity
$Q_f = \Sigma\ \alpha CPL$

C = cohesion of soil
$N_c$ = bearing capacity factor
$A_{tip}$ = area of plie at the tip
$\alpha$ = adhesion factor
P = perimeter of pile
L = length of pile

Design Load : $Q_{all} = \frac{Q_b + Q_f}{F.S.}$

### $\lambda$ method



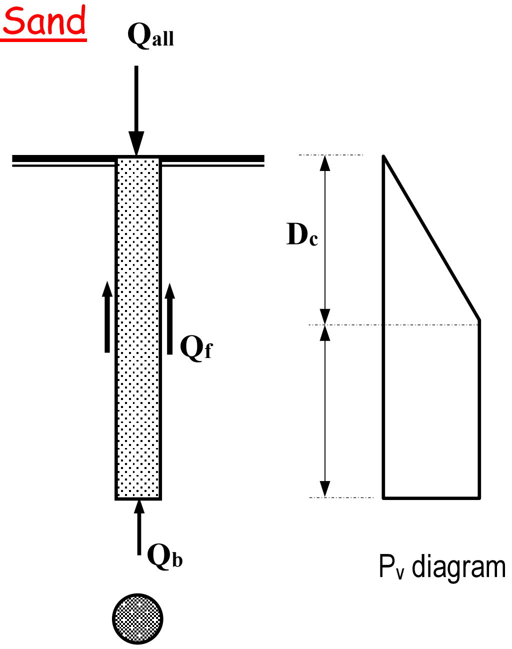

End bearing capacity
$Q_b = C\ N_c\ A_{tip}$

Frictional capcity
$Q_f = PL\lambda\ (Q_v + 2C)$

$Q_v = \frac{\text{area of } P_v \text{ diagram}}{L}$

Design Load : $Q_{all} = \frac{Q_b + Q_f}{F.S.}$

## Piles on Sand




End bearing capacity

$Q_b = P_v N_q A_{tip}$

Frictional capcity

$Q_f = P$ ( area of $P_v$ diagram ) $K \mu$

Design Load : $Q_{all} = \dfrac{Q_b + Q_f}{F.S.}$

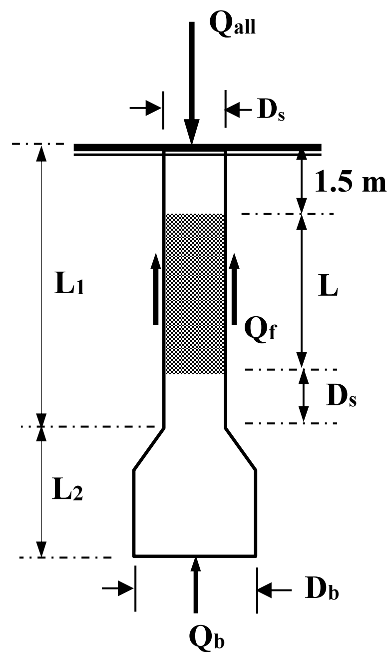
$P_v$ = vertical soil pressure at the tip
$N_q$ = bearing capacity factor
K = coef of lateral bet pile and sand earth pressure factor
$\mu = \tan \theta$
$\theta$ = angle of friction bet pile and sand

Critical Depth, $D_c$

i. $D_c$ = 20 x diameter of pile for dense sand
ii. $D_c$ = 10 x diameter of pile for loose sand

Note : The pressure below the critical depth, $D_c$ is assumed to be uniform.

## Drilled Piles on Clay




**End bearing capacity**

$Q_b = q_p A_{tip}$

$q_p = 6C [ 1 + 0.2 \dfrac{L_1 + L_2}{D_b} ]$ but not greater than $CN_c$

$D_b$ = bell diameter
$D_s$ = shaft diameter

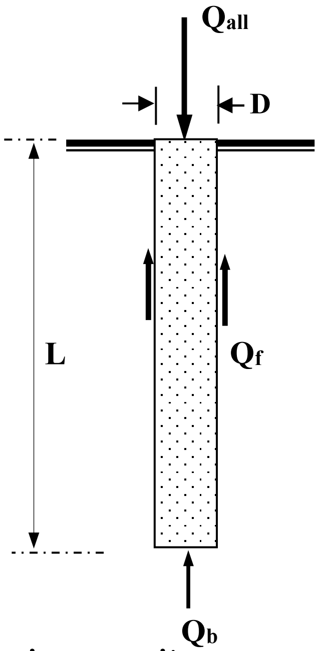
**Frictional capcity**

$Q_f = \Sigma \alpha C P L$

$\alpha$ = adhesion factor
= 0 for the top 1.5 m and diameter $D_s$ above the bottom of the drilled shaft or above the top of the bell.
P = perimeter of pile
L = effective length of pile that resist friction

Design Load : $Q_{all} = \dfrac{Q_b + Q_f}{F.S.}$

## Drilled Piles on Sand




**End bearing capacity**

$Q_b = q_p A_{tip}$

a. $N_{value} < 50$

$q_p = 57.5 N_{value} < 2900$ kPa

b. $N_{value} > 50$

$$q_p = 0.59[N_{value}(\frac{P_a}{P_{vb}})]^{0.8}$$

$P_a$ = 100 kPa

$P_{vb}$ = effective vertical pressure at base elevation

**Frictional capcity** : $Q_f = \Sigma \beta P_v P L$

a. $N_{value} \geq 15$

$\beta = 1.5 - n(z)^{0.5}$ (drilled on sand)

$\beta = 2 - 0.15(z)^{0.5}$ (drilled on gravel)

b. $N_{value} < 15$

$$\beta = \frac{N_{value}}{15}[1.5 - n(z)^{0.5}]$$

$P_v$ = effective vertical overburden pressure at depth z.

z = height from ground surface to mid-height of a given layer

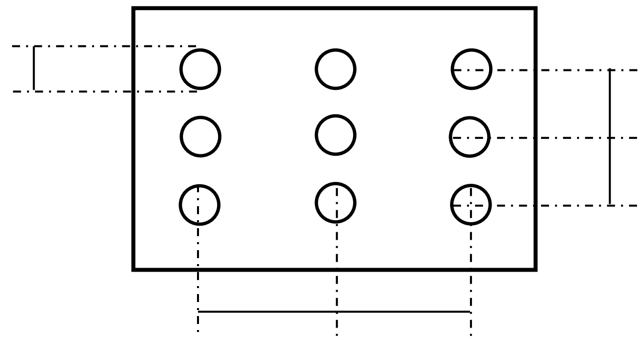
n = 0.245

P = perimeter of pile

L = length of pile

## Group of Piles

Efficiency of Group of Piles, $E_g$



**Converse – Labarre Equation:**

$$E_g = 1 - \frac{\theta[(n-1)m + (m-1)n]}{90\ m\ n}$$

**Bowles :**

$$E_g = \frac{2(m+n-2)S + 4D}{\pi D\ m\ n}$$

m = number of rows of piles

n = number of piles in a row

$$\tan\theta = \frac{D}{S}$$

D = diameter of pile

S = spacing of piles center to center

## Settlement of Piles

$$S_e = S_{e1} + S_{e2} + S_{e3}$$

Elastic Settlement of Pile, $S_{e1}$

$$S_{e1} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$$

$Q_{wp}$ = load carried at the pile point under working load condition

$Q_{ws}$ = load carried by frictional resistance under working load condition

$A_p$ = cross sectional area of pile

L = length of pile

Settlement of Pile caused by the Load at the Pile Tip , $S_{e2}$

$$S_{e2} = \frac{q_{wp}\,D}{E_s}(1 - \mu_s^2)\,I_{wp}$$

$$q_{wp} = \frac{Q_{wp}}{A_p}$$

$E_s$ = modulus of elasticity of soil
$\mu_s$ = Poisson's ratio of soil
$I_{wp}$ = influence factor

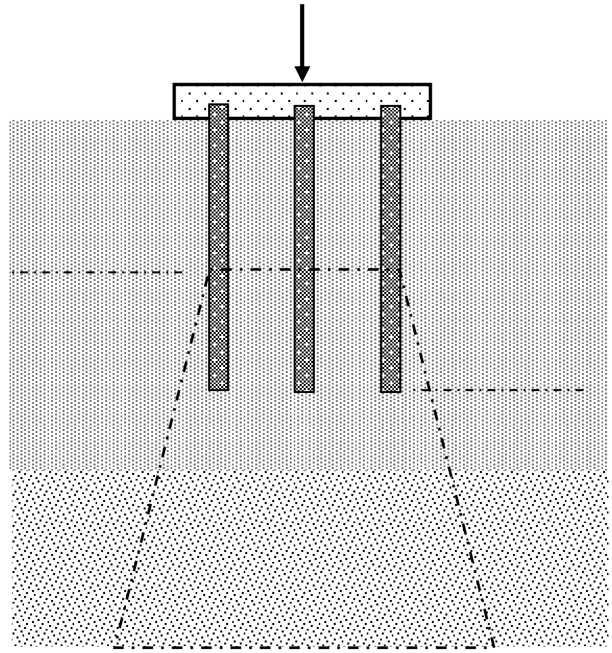
Settlement of Pile caused by the Load transmitted along of the pile shaft , $S_{e3}$

$$S_{e3} = \frac{Q_{ws}\,D}{PLE_s}(1 - \mu_s^2)\,I_{ws}$$

$$I_{ws} = 2 + 0.35\sqrt{\frac{L}{D}}$$

P = perimeter of pile
L = length of pile
$I_{ws}$ = influence factor
D = diameter of pile

## Settlement of Group of Piles



$$S = \Sigma\frac{C_cH}{1+e_o}\log\frac{P_o+\Delta P}{P_o}$$

$C_c$ = compression index
H = thickness of clay layer
$e_o$ = initial void ratio
$\Delta P$ = average increase in pressure on clay
$P_o$ = effective overburden pressure at the mid-height of the clay layer