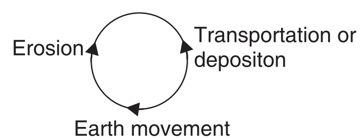
1 Origin of Soil

Soil is an unconsolidated material, composed of soil particles, produced by disintegration of rocks.

Father of Soil Mechanics **Dr. Karl Terzaghi** coined the term "Soil Mechanics" in 1925.



The Geological Cycle

Erosion phase

Physical weathering

- Retains same composition of Parent rock
- Bulky particles
- Shape $\rightarrow$ Angular, Rounded, Flat and Elongated
- Single grain without bonding between them

Chemical weathering

- Change in mineral composition
- Plate like with high surface area
- Crystalline particle of colloidal size known as clay mineral

Transportation phase

Soil	Deposited by
Alluvial Soil	River
Marine Soil	Sea water
Locustrine Soil	Still water like lakes
Aeolian Soil	Wind
Glacial Soil	Ice

Note: Loess is an aelian soil.

Various types of Soil

Alluvial soil: Low density and liable to liquefaction in earthquake prone areas.

Black cotton soil: Residual deposits from basalt or trap rocks, contain clay mineral **Montmorillonite.**

Laterite soil: Iron oxide gives red or pink colour. Residual soil formed from basalt.

Desert soil: Uniform in gradation (eg. dune sand). It's Non-plastic and highly pervious.

Marine soil: Low shearing strength. Highly compressible, soft and Highly plastic.

Bentonite: Formed from volcanic ash, with high percentage of Montmorillonite.

Hard pans: Dense, well graded, cohesive aggregates of mineral particle. They do not disintegrate when submerged in water.

Loam: Mixture of sand, silt and clay.

Peat: Organic soil with fibrous aggregate's formed from vegetable matter in excess moisture (eg. in swamps), Highly compressible not suitable for foundation.

Indurated clay: Hardening of clay due to heat and pressure.

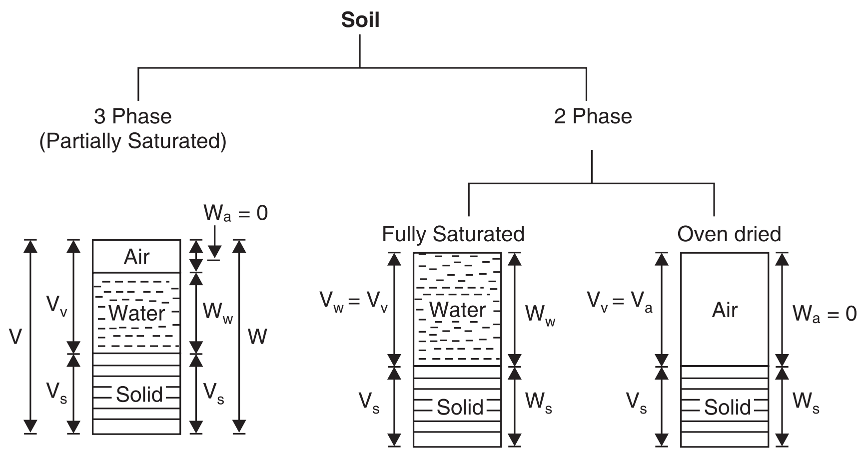
Note: Lithification is a process by which unconsolidated materials are converted into coherent solid rock as by compaction or cementation.

2

Properties of Soil (Soil-Water Relationship)

Phase diagram

Diagrammatic representation of the different phases in a soil mass.



Water Content

$$w = \frac{W_w}{W_s} \times 100$$

$$0 \le w < \text{Infinity}$$

W_w = Gravity water + capillary water + Hygroscopic water

W_w can be removed by oven drying.

Void ratio

$$e = \frac{V_v}{V_s} \quad 0 < e < \text{Infinity}$$

Fine grained soil soil have higher Void ratio than coarse grained soil. While the size of void in coarse grained soil is greater than coarse grained soil.

Porosity

$$\eta = \frac{V_v}{V} \times 100 \quad 0 < \eta < 100$$

Void ratio serves as a useful parameter as compared to porosity void ratio is defined wrt volume of soil solid's which do not change even in compression.

Degree of Saturation

$$S = \frac{V_w}{V_v} \quad 0 \le S \le 100$$

for fully saturated soil S = 100%
for fully dry soil S = 0%
for partially saturated soil $0 < s < 100$

Air content

$$a_c = \frac{V_a}{V_v}$$

Percentage air Void

$$n_a = \frac{V_a}{V} \times 100$$

Percentage air void is always less than air content.

Bulk unit weight

$$\gamma_t = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_w + V_a} \quad \text{units} \rightarrow \frac{KN}{m^3}, \frac{N}{m^3}, \frac{kgt}{cm^3}$$

Unit weight soil

$$\gamma_s = \frac{W_s}{V_s} \qquad \gamma_s = G\,\gamma_w$$

where G = specific gravity of soil solid

Unit weight of water

$$\gamma_w = \frac{W_w}{V_w}$$

Value of γ_w changes with temperature but usually we take

$$\gamma_w = 9.81 \text{ KN/m}^3 \text{ at } 4°C$$

Dry unit weight

$$\gamma_d = \frac{W_s}{V}$$

High value of γ_d indicates more compacted soil.

Saturated unit weight

$$\gamma_{sat} = \frac{\text{Wt. of Saturated Soil}}{\text{Volume of Soil}}$$

It's the unit weight of soil when all the air voids are filled with water

Submerged or Bouyant unit weight

$$\gamma_{sub} = \frac{(W_s)_{sub}}{V}$$

When the soil is below the water table then a Bouyant force acts on the soil solid and its saturated weight is deceased by unit weight of water.

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

Note: Saturated soil may or may not be submerged but a submerged soil will always be saturated.

Specific gravity of solids

$$G = \frac{\gamma_s}{\gamma_w} \qquad \text{No unit}$$

Also known as "Absolute specific gravity" or 'grain specific gravity'.

Mass specific gravity of solid

$$G_m = \frac{\gamma_t}{\gamma_w} \qquad \text{No unit} \quad G_m < G$$

Relative density

$$D_r = \frac{e_{max} - e_{natural}}{e_{max} - e_{min}}$$

$$= \frac{\gamma_{d\,max}}{\gamma_{d\,natural}}\left(\frac{\gamma_{d\,natural} - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}}\right)$$

This parameter (D_r) is generally used for sandy and gravelly soils

Relative density	Classification
0 – 15	Very loose
15 – 35	Loose
35 – 65	Medium dense
65 – 85	Dense
85 – 100	Very dense

Note: For uniformly graded coarse soil having perfectly spherical grain size when particles are arranged in

(*a*) Cubical array

$$e_{max} = 91\%; \quad \eta_{max} = 47.6\%$$

(*b*) Prismoidal array

$$e_{min} = 35\%; \quad \eta_{min} = 25.9\%$$

Some Important Relationships

1. $W_s = \dfrac{W}{1+w}$
2. $V_s = \dfrac{V}{1+e}$
3. $\gamma_d = \dfrac{\gamma_t}{1+w}$
4. $e = \dfrac{\eta}{1-\eta}$
5. $\eta = \dfrac{e}{1+e}$
6. $Se = wG$
7. $\gamma_t = \dfrac{G + Se}{1+e}$
8. $\gamma_{sat} = \dfrac{G+e}{1+e}\gamma_w$
9. $\gamma_d = \dfrac{G}{1+e}\gamma_w$
10. $\gamma_{submerged} = \dfrac{G-1}{1+e}\gamma_w$
11. $\gamma_d = \dfrac{(1-\eta_a)G_s\gamma_w}{1+wG_s}$
12. $S = \dfrac{w}{\dfrac{\gamma_w}{\gamma_t}(1+w) - \dfrac{1}{G_s}}$

Methods of determination of water content

1. **Over drying method:** Soil sample is derived in controlled temperature (105 – 110°C) for 24 hrs in laboratory. Above 110°C, **water of crystallisation** will be lost

$$w = \frac{w_2 - w_3}{w_3 - w_1}$$

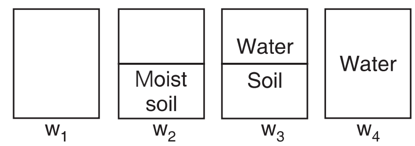
w_1 = wt. of container

w_2 = wt. of container + wt. of moist soil

w_3 = wt. of container + of dry soil.

2. **Torsion balance moisture meter method:** For rapid and accurate determination of water content. Since drying and weighing occur simultaneously, the method is useful for soils which quickly reabsorb moisture after drying.

3. Pycnometer method:



$$w = \left[\left(\frac{w_2 - w_1}{w_3 - w_4}\right)\left(\frac{G-1}{G}\right) - 1\right]$$

- This method is more suitable for cohesionless soil as the removal of entrapped air from cohesive soil is difficult. Hence w_3 cannot be measured accurately.

4. Sand bath method:

➢ Rapid field method, Hence, not accurate.

➢ Sand bath is a large, open vessel containing sand filled to a depth of 3 cm or more.

➢ Same formula of oven drying method.

5. Calcium Carbide method

➢ Quick method but not so accurate.

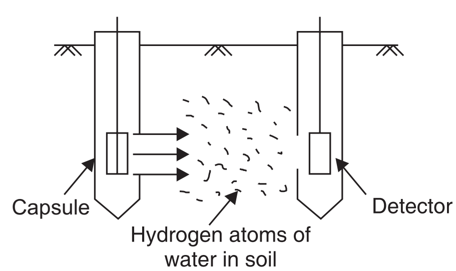
➢ $CaC_2 + 2H_2O \rightarrow C_2H_2 \uparrow + Ca(OH)_2$

➢ Scale is calibrated to give the water content based on total weight (w) of the soil. So, actual water content (based on soil solid) is to be recalculated.

$$\text{Reading given} = w_t = \frac{w_w}{W} = \frac{w_w}{W_s + W_w}$$

$$\text{So} \qquad W_w = \frac{w_t}{1 - w_t}$$

6. Radiation method:

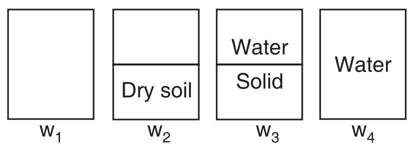


- ➢ Gives water content in in-situ condition
- ➢ Loss of energy by radio-active material (cobalt 60) due to scattering of neutrons by hydrogen atoms of soil while travelling from capsule to detector is proportional to water content.

7. Alcohal method:

- ➢ Quick method
- ➢ Not to be used with soils containing calcium compound or organic content.

Determination of specific gravity of soil solid (G)



- Pycnometer method
- SP. gravity values are reported at 27°C
- G can also be determined indirectly by using shrinkage limit.

$$G_T = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)}$$

To remember, Rewrite as

$$G_T = \frac{W_2 - W_1}{(W_4 - W_3) - (W_1 - W_2)}$$

$$\Rightarrow \qquad G = \frac{(W_1 - W_2)}{(W_1 - W_2) - (W_4 - W_3)}$$

$$G_{27°C} = \frac{G_T \times \text{unit wt. of water at T°C}}{\text{unit wt. of water at 27°C}}$$

Note: Pycnometer method is used for determination of water content as well as specific gravity. In water content determination, w_2 is the moist soil while in determination of specific gravity w_2 is dry soil.

Determination of Unit Weight

1. Core cutter method

- ➢ Field method suitable for, fine grained and clayey soil.
- ➢ Not suitable for stoney, gravelly soil and dry soil.

$$\gamma_t = \frac{W_2 - W_1}{V}$$

W_2 = wt. of core + soil

W_1 = wt. of core

V = volume of core

$$\gamma_d = \frac{\gamma_t}{1+w}$$

w = water content

2. **Water displacement method**
   - ➢ Suitable for cohesive soils only

$$V = V_w - \left(\frac{W_2 - W_1}{\gamma_p}\right)$$

W_1 = wt. of soil sample

W_2 = wt. of soil sample coated with parathin *wax*.

V_w = volume of water displaced by w_2

γ_p = unit wt. of paraffin *wax*

3. **Sand replacement method**
   - ➢ field method
   - ➢ used for gravelly, sandy and dry soil
4. **Water ballon method**
   - ➢ volume of the pit is measured by covering the pit with plastic sheet and then filling it with water.
   - ➢ wt. of water thus calculated is equal to volume of soil excavated.
5. **Radiation method**
   - ➢ Bulk density of in situ soil.
   - ➢ Quick and convenient.

3 Index Properties of Soil

Properties which help to access the **engineering behaviour** of soil and which assist in determining its **classification** accurately are termed as index property:

Index property

→ For coarse soil
Grain shape, grain
size relative density

→ For fine soil
Atterberg's limit
and consistency

Grain Shape

- Sand and gravel have bulky grains of angular or rounded shape.
- Higher the angularity higher will be shearing strength.
- Sphercity $S = \frac{D_e}{L}$

D_e = dia. of equivalent sherical particles

L = Length of particles

where $D_e = \left(\frac{6V}{\pi}\right)^{1/3}$

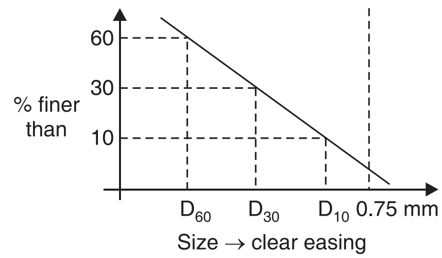
Gain Size

Sieve Analysis

→ Coarse sieve analysis (Dry-Analysis)
- Done for gravels
- For sieves used
  → 80, 20, 10, 4.75 mm

→ Fine sieve analysis (Wet-analysis)
- Done for sand
- 7 sieves used
  → 2mm, 1mm, 600μ, 425μ, 212μ, 150μ, 75μ

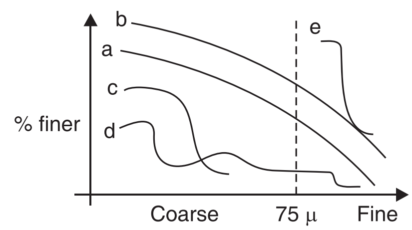
Grain Size Distribution Curves.



D_{10} = effective size of particles i.e. particles which if present alone will cause the same effect as caused by the soil.

D_{60} = Size of the particle such that 60% particle are finer than this size

$D_{60} > D_{30} > D_{10}$



a → well graded

a and b → similarly graded

c → poorly/uniformly graded coarse

d → Gap graded

e → Poorly graded fine soil.

Note: If there is a kink in the graph, then it shows the mixture of soil of two different geological formations.

Coefficient of uniformity $C_u = \dfrac{D_{60}}{D_{10}}$

Coefficient of curvature $C_c = \dfrac{D_{30}{}^2}{D_{60} \times D_{10}}$

Sedimentation Analysis,

Stoke's Law: $V_t = \dfrac{(\gamma_s - \gamma_l)d^2}{18\mu}$

V_t = terminal velocity

d → diameter of the grain

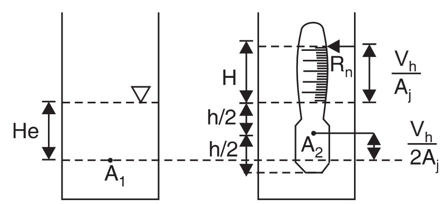
- Range of diameter of particle for stoke's law validity = **0.2 mm to 0.0002 mm**
- It particle size is greater than 0.2 mm, then **turbulent motion** will occur and if particle size is smaller than 0.0002 mm then **Brownian** motion will occur.

$$\frac{h}{t} \propto d^2 \quad \Rightarrow \quad \frac{d_1}{d_2} = \sqrt{\frac{h_1}{t_1} \times \frac{t_2}{h_2}}$$

h = height through which particle falls in t-secs.

Hydrometer Analysis

$$H_e = H + \frac{1}{2}\left(h - \frac{V_h}{A_j}\right)$$



V_h = Volume of hydrometer

A_j = Area of jar

h = length of bulb in hydrometer

Here, H corresponds to reading R_H

Specific gravity of soil suspension at depth H_e

$$G = 1 + \frac{R_h}{1000}$$

$$\frac{(\gamma_s - \gamma_w)d^2}{18\mu} = \frac{H_e}{t} = \frac{H + \frac{1}{2}\left(h - \frac{V_h}{A_j}\right)}{t}$$

Correction's in hydrometer Analysis

Meniscus correction $\rightarrow$ Always positive (C_m)

Defloculating Agent correction $\rightarrow$ Always negative (C_d)

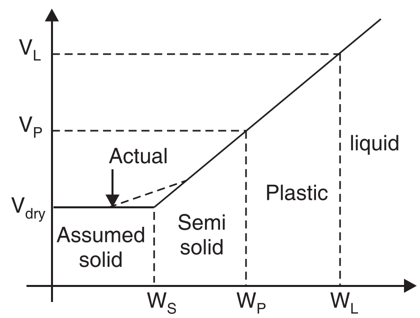
Temperature correction $\rightarrow$ If temp. is more than its positive otherwise negative (C_t)

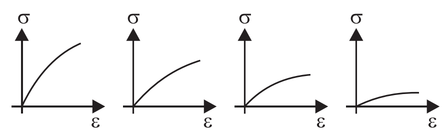
Composite correction

$$C = C_m - C_d \quad C_t$$

Consistency limits

It's the relative ease with which a soil can be deformed. Atterberg classified the consistency in 4-stages → solid, semi-solid, plastic, liquid.





V_L = Volume at liquid limit
V_P = Volume at Plastic limit
V_{dry} = Volume at shrinkage limit
W_L = Water content at liquid limit
W_P = Water content at plastic limit
W_S = Water content at shounkage limit.

$$\frac{V_L - V_P}{W_L - W_P} = \frac{V_P - V_{dry}}{W_P - W_S}$$

Note: Naturally occurring water content is generally between W_L and W_P

- Volume of soil does not deereases when water content is reduced beyond shrinkage limit.

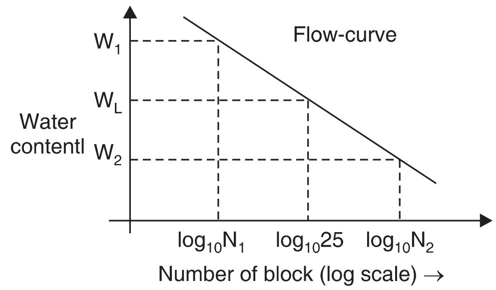
Liquid limit

Minimum water content at which soil has tendency to flow.

All soils at liouid limit have similar shear strength.

Found out by two tools → (*a*) Casagrande's (*b*) Cone penetration. Water content at which 25 blows close the 2mm grove cut in soil sample placed in Casagrande's bowl is called liquid limit

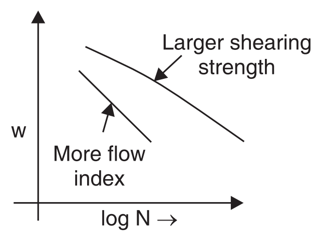
$$W_L = W_N \left(\frac{N}{25}\right)^x$$



Flow Index (I_f): Slope of flow curve is called flow index

$$I_f = \frac{W_1 - W_2}{\log_{10} N_2 - \log_{10} N_1}$$

$$I_f = \frac{W_1 - W_2}{\log \dfrac{N_2}{N_1}}$$



$$\text{Flow Index } \alpha \ \frac{1}{\text{Shear Strength}}$$

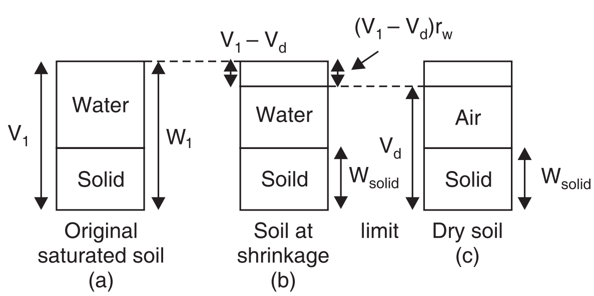
Plastic limit

Minimum water content at which soil is in plastic stage.

At plastic limit, a soil when rolled into a thread of 3 mm starts to crumble shear strength at the plastic limit is about 100 times at the liquid limit.

Shrinking limit

Minimum water content at which soil is completely saturated or the maximum water content at which further reduction in water content does not cause any reduction in the volume of soil sample.



Water content at shrinkage limit = $\dfrac{(W_1 - W_{solid}) - (V_1 - V_d)\gamma_w}{W_{solid}}$

(From figure (*b*))

$$G = \frac{W_{solid}}{V_1\gamma_w - (W_1 - W_{solid})}$$

Shrinkage ratio: Volume change in soil above shrinkage limit expressed as a percentage of dry soil per unit change in water content above shrinkage limit.

$$\text{S.R.} = \frac{\left(\dfrac{V_1 - V_2}{V_d}\right)\times 100}{W_1 - W_2} \qquad \text{S.R.} = \frac{r_d}{r_w}$$

Note: Shrinkage ratio is the mass specific gravity of the soil in dry state.

$$\text{Volumetric Shrinkage} = \frac{V_1 - V_d}{V_d}\times 100$$

Plasticity Index:

$$I_P = W_L - W_P$$

I_P	Consistency
0	Non Plastic
< 7	Low Plastic
7–17	Medium Plastic
> 17	Highly Plastic

Shrinkage Index

$$I_S = W_P - W_S$$

Consistency Index

$$I_C = \frac{W_L - W}{W_L - W_P}$$

Liquidity Index

$$I_L = \frac{W - W_P}{W_L - W_P} \qquad I_C + I_L = 1$$

Solid	Semi solid	Plastic	liquid
$I_L < 0$	$I_L < 0$	$0 < I_L \le 1$	$I_L > 1$
$I_C > 1$	$I_C > 1$	$0 < I_C < 1$	$I_C < 0$

W_S W_P W_L

Consistency	I_C	I_L
Very stiff	> 1	< 0
Stiff	1–0.75	0–0.25
Medium Stiff	0.75–0.5	0.25–0.5
Soft	0.5–0.25	0.5–0.75
Very Soft	0.25–0	0.75–1
liouid State	< 0	> 1

→ Brittle failure (Very stiff)

→ Range of plastic failure (Stiff to liouid State)

Toughness Index

$$I_t = \frac{I_P}{I_f} \qquad I_t = \frac{I_P}{I_f} = \log \frac{S_p}{S_f}$$

S_P = Shear strength at plastic limit

S_e = Shear Strength at liquid limit

Note: For $I_t < 1$, soil is friable i.e., get's crushed at plastic limit.

Thixotropy

It is the increase in strength of soil with **passage of time** due to tendency of clay soil to regain its **chemical equilibrium** with the **reorientation** of water molecules in the **adsorbed layer.**

During Pile driving in clayey soil, frictional resistance by soil increases due to property of thixotropy.

Sensitivity (S_t): Degree of disturbance due to remoulding

$$S_t = \frac{\text{Unconfined Compressive Strength due to undisturbed soil}}{\text{Confined compressive strength due to remoulded soil}}$$

$$S_t = \frac{q_u \text{ (undisturbed)}}{q_u \text{ (remoulded)}}$$

1	Insentive (General and Coarse sand)
2–4	Normal
4–8	Sensitive (Silt and Clay)
8–16	Extra
> 16	Quick

Note: Stiff clay having figures and crack have $S_t \leq 1$.

Activity Number (A_C):

$$A_C = \frac{I_P}{\text{\% of clay size particles}}$$

i.e. size < 2∝

< 0.75	Inactive
0.75 – 1.25	Normal
> 1.25	Active

Note : More activity means more change in volume.

4

Classification of Soil

Soil classification is done on the basis of index properties like grain size distribution and plasticity. Three important systems of classification are

1. **The Unified Soil Classification System** (USCS)**:** Developed by casagrande for the use in airfield construction. Coarse grained soils were classified on the basis of grain size distribution while fine grained soils on the basis of plasticity characteristics.
2. **American Association of Stale Highway and Transportation Official (AASHTO):**

   Soil are classified into 8 groups from A_1 to A_7 with A_8 for muck or peat

   Group index: $GI = 0.2\,a + 0.005\,ac + 0.01\,bd$

where a = % passing through 75 μ serve – 35 $\quad 1 \le a \le 40$

b = % passing through 75 μ serve – 15 $\quad 1 \le b \le 40$

c = liquid limit – 40 $\quad 1 \le c \le 20$

d = plasticity index – 10 $\quad 1 \le d \le 20$

Group index **0 indicates good subgrade material** while group index of **20 indicates very poor subgrade material.**

3. **Indian standard soil classification system:** It's given by A-casagrande

Coarse grained soil
Classified on the basis of
Particle size, fineness, C_c, C_u

Fine grained soil
classified on the basis of
Compressibility, liquid limit, plasticity index.

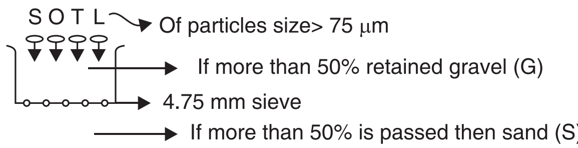
Coarse grained soil classification based on grain size (mm)

Boulder	Cobble	Coarse grained soil					Fine Grained soil	
		Gravel		Sand				
		coarse	fine	coarse	medium	fine	silt	clay
>300	300-80	80-20	20-4.75	4.75-2.0	2-0.425	0.425 0.075	075-0.002	<0.002

Note: Fine sand is a coarse grained soil

Coarse grained soil classification

Case 1: When fines i.e. particles less than 75 μ m) are less then 5%



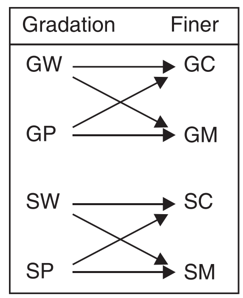
Well graded gravel (GW)	Poorly graded gravel (GP)
$C_4 > 4$, $1 < C_c < 3$	Either or both of these are not satisfied
Well graded sand (SW)	Poorly graded sand (SP)
$C_4 > 6$, $1 < C_c < 3$	Either or both of these are not satisfied

Case 2: When fines are between 5–12 %

- ➢ Dual symbol is used
- ➢ Divided into 8 parts based on gradation and fines
- ➢ For Eq:

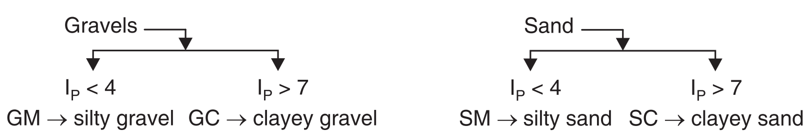
  GW-GM → Well graded gravel with silt as fine

  SP-SC Poorly graded sard with clay as fines.



Case 3: When fines are greater than 12%

Gravels



Note: If I_P between 4 – 7 dual symbols will be used.

Fine grained soil classification

A –line: $I_P = 0.73\,(W_L - 20)$

V – line: $I_P = 0.9\,(W_L - 8)$

$W_L < 35$,

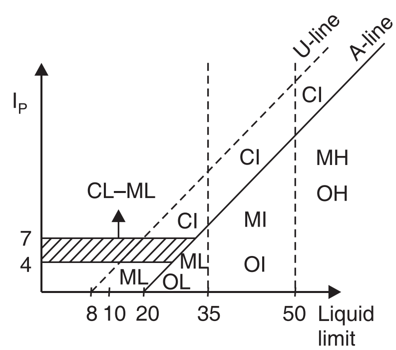
CL → Low plastic **inorganic clay**

ML → Low plastic silt

OL → Low plastic **organic clay**

$35 < W_L < 50$,

CI → Intermediate plastic inorganic clay



MI → Intermediate plastic silt

OI → Intermediate plastic organic clay

$W_L > 50$

CH → Highly plastic inorganic clay

MH → Highly plastic silt

OH → Highly plastic organic clay.

Note: Fine grained soil in indian standard soil classification are sub divided into low, medium, high on the basis of compressibility while in unified soil classification system it has only two categories of low and high compressibility.

5 Clay Mineral and Soil-Structure

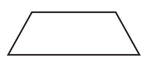
Properties of coarse grained soil are hardly effected by minerological composition because they are formed due to physical forces and retain the minerological composition of parent rock while properties of fine grain soils like clay depend to a large extent on the nature and characteristics of minerals present.

$$\text{Specific Surface} = \frac{\text{Surface Area}}{\text{Mass or Volume}}$$

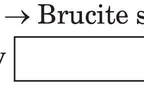
Soil Structure

Atomic sturchure of clay mineral

Tetrahedron Unit

- Four oxygen atoms at the tip of tetrahedron enclosing a silicon atom
- Net charge on one unit = –1
- Represented by 

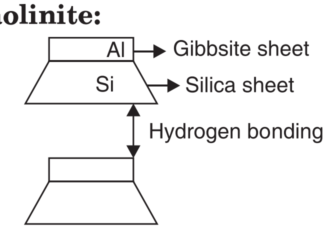
Octahedral Unit

- Six hydroxyl ion at the tip of octahedral enclosing Aluminium or magnesium
- If Al at centre → Gibbsite sheet
- If Mg at centre → Brucite sheet
- Represented by 

Isomorphic Substitution: Substitution of metallic Ion with another metallic Ion of lower valency but same physical size. for eg Si^{+4} replaced by Al^{+3} in a tetrahedral unit.

Various Clay Minerals

1. Kaolinite:



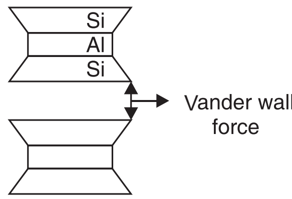
→ 1 : 1 clay mineral

→ No isomorphous substitution

→ Found in old deposits.

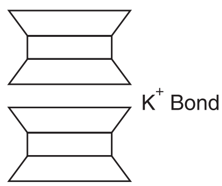
Note: Halloysite when air dried gets converted to Kaolinite and both Halloysite and Kaolinite are used for making chinaware.

2. Montmorillonite:



- ➢ 2 : 1 clay mineral
- ➢ Highly plastic with little internal friction.
- ➢ Common in residual soil derived from volcanic volcanic ash.
- ➢ Bentonite is a montmorillonite clay.
- ➢ Found in Black cotton soil.

3. Illite:



- ➢ 2 : 1 clay mineral
- ➢ Substantial Isomorphous substitution
- ➢ Ionic bonding

Properties of clay minerals

	Grain size	**Base exchange capacity**	**I_P**	**Dry strength**	**Activity**
Kaolinite	Maximum	Min	Min	Min	Min
Illite	↑	↓	↓	↓	↓
Montmorillonite	Minimum	Max	Max	Max	Max

Cation Exchange capacity: Ability at clay particle to absorb ions on its surface or edge is called base or carbon exchange capacity. It is based on size of particles and mineral structure.

Replacement ability of various cations

$$Al^{+3} > Ca^{++} > Mg^{++} > K^{+} > H^{+} > Na^{+} > Li^{+}$$

Note: Sodium clay is stablised by using lime by using cation exchange property only.

Compaction of Soil

6

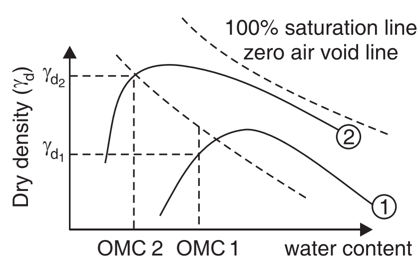
Compaction	Consolidation
Reduction in volume of air voids at a given water content	Volume reduction due to expulsion of pore water from voids
Partially saturated soil	Completely saturated soil
Instantaneous phenomenon	Time dependent phenomenon
Specific compaction Technique	By static load placed on soil

Note: Cohesionless soils are compacted by Vibrations while cohesive soils are compared are compacted by application of static pressure.

Proctor Test

A definite relationship is established between the degree of dry density and soil moishire content. Optimun moisture content (OMC) is the water content at which a particular soil attains maximum dry density (MDD).

- Compactive effort (energy per unit volume) for curve 2 is more than curve 1.



Note: On increasing the compactive effort curve shifts backwards and upwards i.e. OMC decreases and MDD increases.

Compactive effort E = $\dfrac{NnWh}{V}$

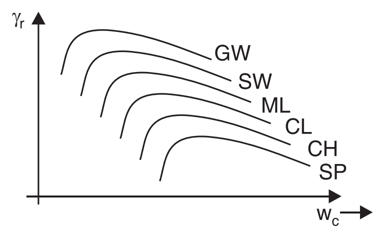
N → No. of blows per layer
n → no. of layers
W → weight of hammer
h → height of fall
V → volume of mould

Note: Zero air void line cannot be practically achieved, as all air voids cannot be ever removed.

Compaction Tests:

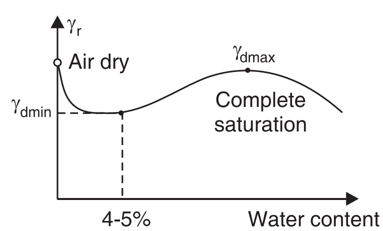
Standard Proctor tests (Light compaction test)		Modified Proctor test (Heavy compaction test)	Indian std. light Compaction test	Indian std. heavy Compaction test
Volume of mould	942 cc	942 cc	1000 cc	1000 cc
No. of layers	3	5	3	5
No. of blows/layer	25	25	25	25
Height of free fall	12 inches	18 inches	310 mm	450 mm
Wt. of hammer	2.495 kg (5.5 lb)	4.54 kg (10 lb)	2.6 kg	4.9 kg

Compaction Curve for different Soils:



- Well graded soil can be compacted more than poorly graded soil
- Order of MDD on basis of grain size only

  Gravel (G) > Sand (S) > SILT (M) > Clay (C)
- Low plastic soil (L) can achieve higher γ_d than highly plastic (H) soils.



For Sands

Note: In compaction of sand, intially γ_d decreases due to bulking of sand at nearly 4-5% water content. Then γ_d increases and reaches to maximum at complete saturation.

Suitability of Compaction Equipment

Type of Equipment	Suitable of Soil Type	Nature of project
Rammers or tempers	All soils	In confined areas such as fills behind retaining walls, basement walls, etc. Trench fills.
Smooth wheeled rollers	Crushed rocks, gravels, sands	Road construction, etc.
Pneumatic tyred rollers	Sands, gravels silts, clayey soils, not suitable for uniformaly graded soils	Base, sub-base and embankment compaction for highways, airfields, etc. Earth dams
Sheepsfoot rollers	Clayey soil, Pure clay	Core of earth dams
Vibratory rollers	Sands	Embankments for oil storage tanks, etc.

Comparison of dry of optimum with wet of optimum conpaction

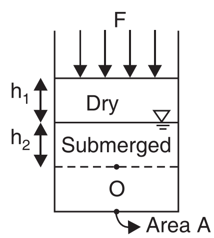
Property	Dry of optimum	Wet of optimum
Structure after compaction	Flocculated (random)	Dispersed (oriented)
Water deficiency	More	Less
Permeability	More, isotropic	Less, anisotropic
Compressibility		
at low stress	Low	Higher
at high stress	High	Lower
Swellability	High	Low
Shrinkage	Low	High
Stress-strain behavious	Brittle: high peak higher elastic modulus	Ductile: no peak, lower elastic modulus
Construction pore water pressure	Low	High
Strength (undrained) as moulded, after saturation	Highsomewhat higher if swelling prevented	Much lower Low
Sensitivity	more	Low

Selection of Compaction water content

Project	Compaction Water Content	Reason
Core of an earth dam	Wet of optimum	To reduce permeability and prevent cracking in core
Homogeneous embankment	Dry of optimum	To have a stronger soil and to prevent build-up of high pore water pressure
Subgrade of pavement	Wet of optimum	To limit volume changes

7 Effective Stress

Effective stress concept is developed by **Terzaghi** and applied to **Fully Saturated** soils only.



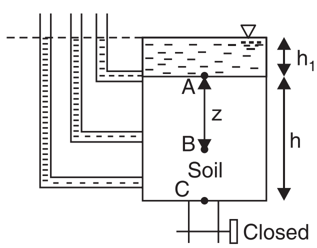
$\bar{\sigma} = \sigma - \mu$
σ = Effective stress
σ = Total stress
μ = Pore water pressure

$$\sigma_0 = \frac{F}{A} + \gamma_d h_1 + \gamma_{sat} h_2 \qquad \bar{\sigma}_0 = \frac{F}{A} + \gamma_d h_1 + \gamma_{sat} h_2$$

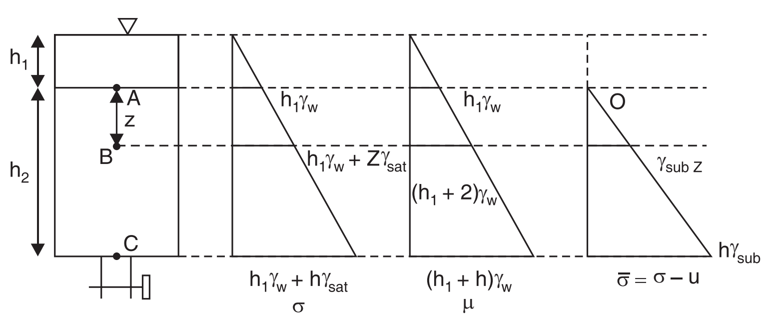
Note: Effective stress is not a physical parameter hence cannot be measured.

Analysis of Seepage force on effective stress

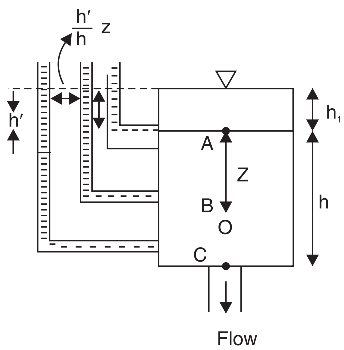
1. Hydrostatic condition (no flow condition)



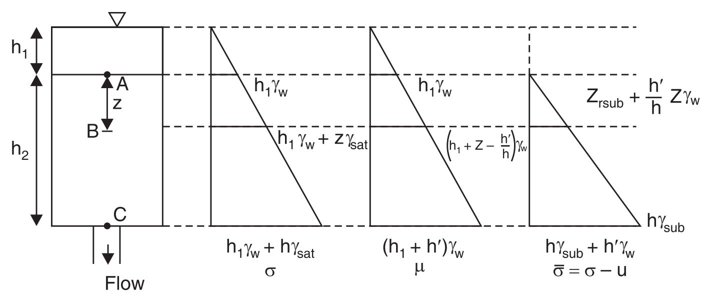
Points	Pressure head	Elevation head	Total head
A	h_1	h	$h_1 + h$
B	$h_1 + z$	$h - z$	$h_1 + h$
C	$h_1 + h$	0	$h_1 + h$



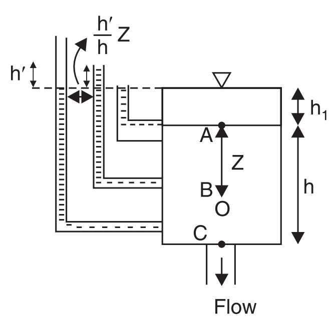
2. Downward flow condition



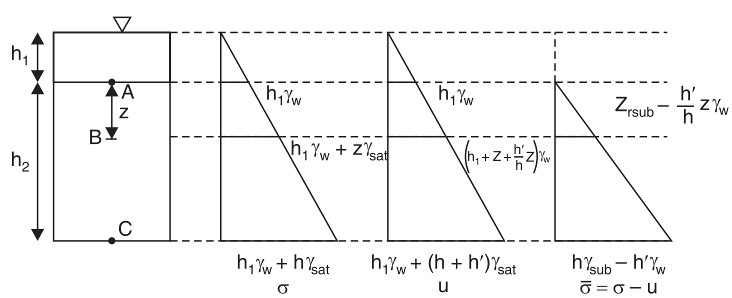
Points	Pressure head	Elevation head	Total head
A	h_1	h	$h_1 + h$
B	$h_1 + z - \frac{h'}{h} z$	$h - z$	$h_1 + h - \frac{h'}{h} z$
C	$h_1 + h - h'$	0	$h_1 + h - h'$



3. Upward flow condition

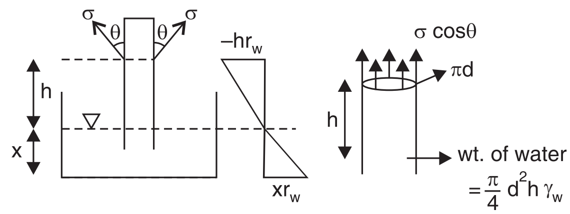


Points	Pressure head	Elevation head	Total head
A	h_1	h	$h_1 + h$
B	$h_1 + z + \frac{h'}{h} z$	$h - z$	$h_1 + h + \frac{h'}{h} z$
C	$h_1 + h + h'$	0	$h_1 + h + h'$



8 Capillarity and Permeability

Water which is held above the water table by phenomenon of surface tension is called capillary water. Due to effect of capillary rise there is increase in unit weight of the soil up to the height of capillary rise



$$\pi d\sigma\cos\theta = \frac{\pi}{4}d^2h\gamma_w$$

$$h = \frac{4\sigma\cos\theta}{\gamma_w d}$$

For $\theta = 0^0$ $$h = \frac{4\sigma}{\gamma_w d}$$

For pure glass,

$\sigma = 73$ N/m,

$r_w = 9.8$ kN/m^3 $h = \dfrac{0.03}{d_{mm}}$

d = dia of tube in mm

h = height of capillary rise in m

$$h = \frac{0.03}{0.2\,D_{10}}$$

D_{10} = Effective size of particle in mm

Other emperical formula:

$$h_{cm} = \frac{C}{eD_{10\,cm}}$$

c = emperical constant

$= 0.1 - 0.5$ cm^2

Note: Capillary moistuer in fine sand and silt allows unsupported excavation to be made because of stability it provide by virtue of **induced shear** strength.

- Bulking of sand also occurs due to capillary. It produces **apparent cohesion** which holds the particles in clusters, enclosing honey combs.

Quick sand condition

In case of **upward seepage flow**, if the upward seepage force becomes equal to the buoyant weight of soil the effective stress in the soil becomes zero.

Critical hydraulic gradient

$$i_{cr} = \frac{\gamma_{sub}}{\gamma_w} = \frac{G-1}{1+e}$$

Note: Quick sand condition occurs in sand but not in clay because in clay cohesion exists.

$$\text{Factor of safety} = \frac{i_{cr}}{i_e}$$

i_e = exit hydraulic gradient.

Permeability: It is the ease with which water can flow through any medium.

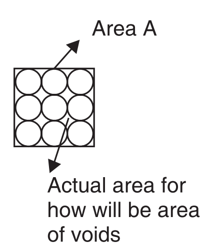
Darcy's law: In one dimensional flow, discharge through fully saturated soil is given by

$$q = K\,i\,A \text{ or } V = Ki \qquad i - \frac{\Delta h}{l}$$

q = discharge
A = cross sectional area of the soil corresponding to flow 'q'
i = hydraulic gradient
Δh = loss of head in length 'L'
K = coefficient of permeability
V = Discharge velocity or superficial velocity

(*a*) $V_S = \dfrac{V}{\eta} \qquad V_S > V$

η = porosity (< 1)
V_S = Achial velocity or seepage velocity corresponding to area of void's in x-sectional area A



(*b*) $K_P = \dfrac{K}{\eta}$

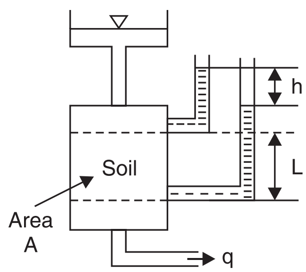
where K_P = coefficient of percolation.

Determination of coefficient of Permeability

- Laboratory methods
  - Constant head
  - Variable/Falling head
- Field methods
  - Pumping out
  - Pumping in
- Indirect method
  - From consolidation data
  - From particle size and specific surface
- Capillarity permeability test
  - For unsaturated soil sample

[C]onstant head permeab-ility test

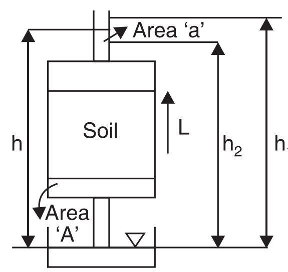
- [C]oarse grained soil



$$K = \frac{qL}{Ah}$$

[F]alling head permeability method

[F]ine grained soil.

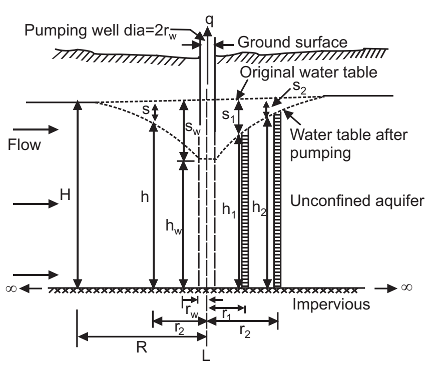


$$K = \frac{2.303\, a\, L}{A\, t} \log_{10}\left(\frac{h_1}{h_2}\right)$$

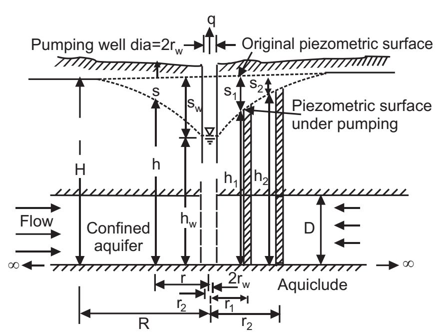
Note: Reliability of laboratory method depends on the extent to which the test specimen represents the original ground conditions.

Pumping out test: Method useful for homogenous coarse grained soil.

Unconfined Aquifer



Confined Aquifer



$$K = \frac{2.303\,q}{\pi\,(h_2^2 - h_1^2)}\log_{10}\left(\frac{r_2}{r_1}\right) \qquad K_2 = \frac{2.303\,q}{2\pi D(h_2 - h_1)}\log_{10}\left(\frac{r_2}{r_1}\right)$$

Note: Radius of influence, is the circle over which the effect of pumping is observed. It gradually merges asymptotically to the water table

$R = 3000\,d\,\sqrt{K}$ **Sichardt's formula**

R = radius of influence in m

d = draw down in the well in m

K = coefficient of permeability (m/sec)

Pumping in test: More economical but less reliable than pumping out test as it gives coefficient of permeability of stratum which is close to the hole. It is also of two types.

- Open end tests
- Packer tests

From consolidation equation:

$K = c_v\, m_v\, \gamma_w$

c_v = coefficient of consolidation

m_v = coefficient of volume compressibility

γ_w = unit wt. of water.

From particle size and specific surface

- Kozeny-canman eouation
- Allen Hazen's
- Loudon's formula

Kozeny-canman eouation

$$K = \frac{1}{CS^2}\frac{r_w}{\mu}\frac{e^3}{1+e}$$

C = Shape factor coefficient

S = Surface area per unit volume

$$S = \frac{6}{D} = \frac{6}{\sqrt{ab}}$$

D → dia of particle or size b/w a_{mm} and b_{mm}.

Allen Hazen's

$K = CD_{10}^2$

$K \rightarrow$ cm/sec

$D_{10} \rightarrow$ mm

or

$K = 100\,D_{10}^2$

$K \rightarrow$ cm/sec

$D_{10} \rightarrow$ cm

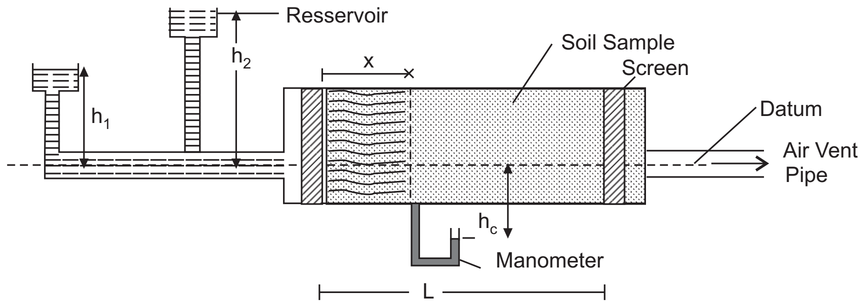
Loudon's formula

$\log_{10}(KS^2) = a + b\eta$

$\eta \rightarrow$ porosity

$a, b \rightarrow$ constants

Capillarity permeability test:



$$\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2K}{S\eta}(h_1 + h_c)$$

where, S = degree of saturation

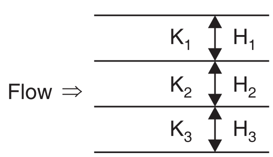
η = porosity

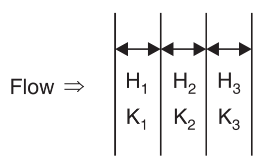
h_c = capillary head

water move from x_1 to x_2 in time t_1 to t_2

Permeability of Stratified Soil

- Horizontal flow (Parallel to bedding plane)
- Vertical flow Normal to bedding plane)





$$K_H = \frac{K_1 H_1 + ..., + K_n H_n}{H_1 + H_2 + H_3}$$

$$K_V = \frac{H_1 + H_2 + ... + H_n}{\frac{H_1}{K_1} + \frac{H_2}{K_2} ... + \frac{H_n}{K_n}}$$

9 Seepage Though Soil

Laplace Equation in two dimensional Flow

$$\boxed{\frac{\partial^2\phi}{\partial x^2}+\frac{\partial^2\phi}{\partial y^2}=0}$$

Potential function $\phi = KH$

where, H = Total head

K = Permeability coefficient

$$\boxed{\frac{\partial^2 H}{\partial x^2}+\frac{\partial^2 H}{\partial y^2}=0}$$

Laplace equation for homogenous **isotropic** soil (in 2D)

where $\phi = K_x H$ and $\phi = K_y H$ and $K_x = K_y$

$$\boxed{K_x\frac{\partial^2 H}{\partial x^2}+K_y\frac{\partial^2 H}{\partial y^2}=0}$$

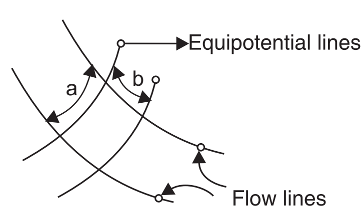
Laplace equation for **Anisotropic** soil (in 2D) where $\phi_x = K_x H$ and $\phi = K_y H$ and $K_x \neq K_y$

Note: For Anisotropic soil, the section is transformed with x-distance changed to $x\sqrt{\frac{K_y}{K_x}}$ while keeping the vertical dimension constant.

Coefficient of permeability of transformed section $K' = \sqrt{K_x K_y}$ (in 2D).

Calculation of discharge through a flow net

$$q = KH\frac{N_f}{N_d}\left(\frac{a}{b}\right)$$



q = flow per unit width perpendicular to the pressure plane of section

H = Total head loss

N_f = No. of flow channels

N_d = No. of equipotential drops

$N_f = N_\psi - 1$ $\quad$ N_ψ = No. of flow lines

$N_D = N_\phi - 1$ $\quad$ N_ϕ = No. of equipotential lines

$$\frac{N_f}{N_D} = \text{Shape factor}$$

a, b → dimensions of an elementary square hence ratio of a/b is generally 1

Note: Shape factor (N_f/N_D) is the function of boundary conditions **only**, and will change only when extent of flow is changed. It will not change even if U/S and D/S water levels are interchanged and direction of flow is reversed.

Keeping the boundary conditions same, if the value of N_f is changed then the value of N_D will also be automatically changed in such a way to keep the value of shape factor constant.

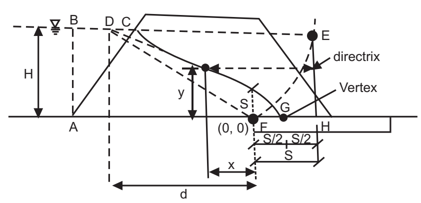
Methods of obtaining a flow net

- Analytical method
- Electrical flow analogy
- Capillary flow analogy
- Sand model
- Graphical method

Phreatic Line: It is the top flow line which follows the path of base parabola. It is a stream line. The pressure on this line is atmospheric and below this line is hydrostatic.

(a) **Phreatic line with filter**

$$x + S = \sqrt{x^2 + y^2}$$



At $x = 0$ $y = S$

$$\frac{dy}{dx} = \frac{S}{\sqrt{S^2 + 2xS}}$$

At $\quad x = 0 \quad \frac{dy}{dx} = 1$

At F $\quad \frac{dy}{dx} = 1$ and $y = S$ then $q = KS$

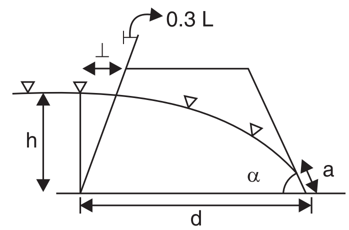
Hence, here q is found out without knowing the complete flownet.

(*b*) **Phreatic line with out filter**

(*i*) For $\alpha < 30°$

$$q = Ka \sin^2 \alpha$$

$$a = \frac{d}{\cos \alpha} - \sqrt{\frac{d^2}{\cos^2 \alpha} - \frac{h^2}{\sin^2 \alpha}}$$



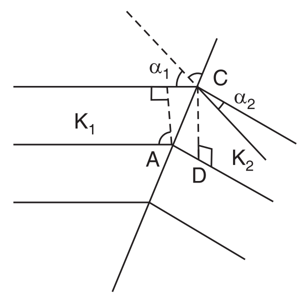
(*ii*) For $\alpha > 30°$

$$q = Ka \sin \alpha \tan \alpha$$

$$a = \sqrt{d^2 + h^2} - \sqrt{d^2 - h^2 \cot^2 \alpha}$$

Flow through Non-homogenous section

$$\frac{K_1}{K_2} = \frac{\tan \alpha_1}{\tan \alpha_2}$$



If $K_1 > K_2$ then $\alpha_1 > \alpha_2$, the flow gets deflected towards the normal otherwise vice a versa.

Filter specifications by Terzaghi

1. **Upper limit** of grain size to ensure **no significant invasion** of particles

$$\frac{D_{15}\text{ (filter)}}{D_{85}\text{(protected material)}} < 5$$

2. **Lower limit** to ensure **sufficient head loss** without excessive seepage pressure

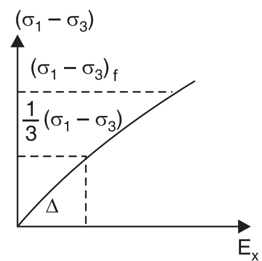
$$4 < \frac{D_{15}\text{(filter)}}{D_{15}\text{(protected)}} < 20$$

3. Additional guideline

$$\frac{D_{50}\text{(filter)}}{D_{50}\text{(protected material)}} < 25$$

10 Vertical Stressess

Modulus of Elasticity of Soil: Determined by **Triaxial** test. It is taken as the **secant** modulus (1/2 to 1/3) of the peak stress. Sometimes initial **tangent modulus** or tangent modulus is also used.



Note: In undrained condition, value of poisson's ratio is 0.5 while in drained condition it's less than 0.5.

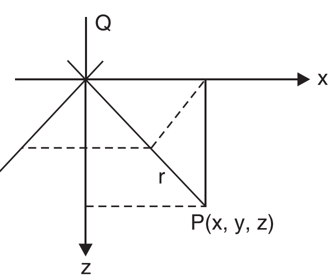
Vertical Stress due to concentrated load

Boussineq's
(For isotropic soil)

$$\sigma_{z_b} = \frac{3}{2\pi}\frac{Q}{z^2}\frac{1}{\left(1+\left(\frac{r}{z}\right)^2\right)^{5/2}}$$

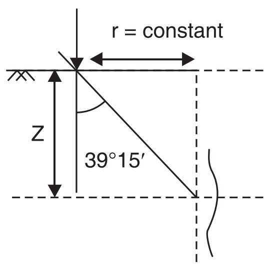
Westergaard's Eouation
(For An-Isotropic soil)
Poison's ratio assumed zero.

$$\sigma_{z_W} = \frac{1}{\pi}\frac{Q}{z^2}\frac{1}{\left(1+2\left(\frac{r}{2}\right)^2\right)^{3/2}}$$



Note: $\frac{r}{z} < 1.5 \quad \sigma_{z_B} > \sigma_{z_W}$; $\frac{r}{z} > 1.5 \quad \sigma_{z_B} < \sigma_{z_W}$

Bossinesq's result



when z is constant

$$\sigma_{z\,\max} = 0.0888\,\frac{Q}{r^2}$$

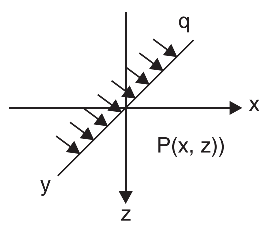
when r is constant

$$\sigma_{z\,\max} = 0.1332\,\frac{Q}{z^2}$$

Note: Westergaard's result are more close to the field conditions but Boussineq results are used for calculation because they provide conservative results.

Vertical stress due to line load

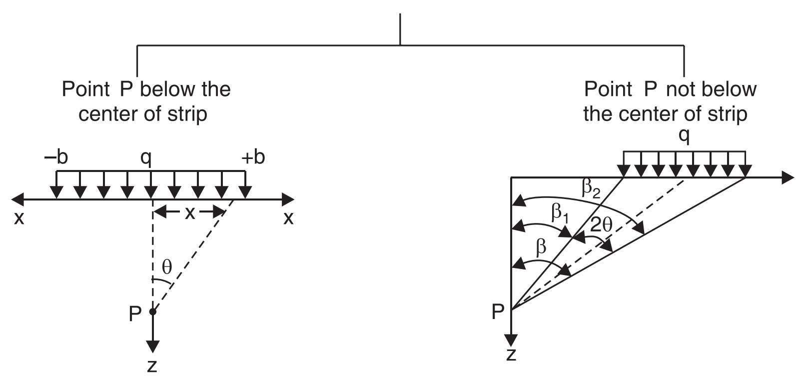
$$\sigma_z = \frac{2q}{\pi z}\left(\frac{1}{1+\left(\frac{x}{z}\right)^2}\right)^2$$



$\Rightarrow$ At $x = 0$ at depth z

$$\sigma_z = \frac{2}{\pi}\frac{q}{z}$$

Vertical stress due to strip load

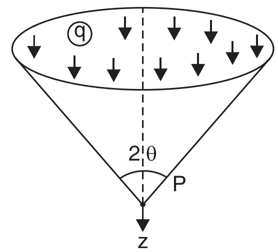


$$\sigma_z = \frac{q}{\pi}(2\theta + \sin 2\theta)$$

$$\sigma_z = \frac{q}{\pi}(2\theta + \sin 2\theta \cos 2\phi)$$

Vertical stress due to a circular area

$$\sigma_z = q\,(1 - \cos^3 \theta)$$



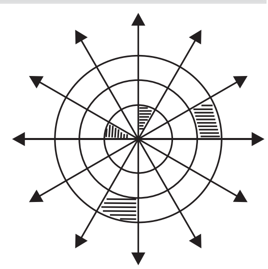
Vertical stress below the corner of a rectangle

$$\sigma_z = \frac{q}{2\pi}\left[\frac{mn}{\sqrt{m^2+n^2+1}} \times \frac{m^2+n^2+2}{m^2+n^2+m^2n^2+1} + \sin^{-1}\left(\frac{mn}{\sqrt{m^2+n^2+m^2n^2+1}}\right)\right]$$

where $m = \dfrac{B}{z}$ $n = \dfrac{L}{z}$

Note: The value of m and n can be interchaged in equation.

Newmark's Influence chart: Newmark developed the influence chart based on **Boussinesq's** equation to compute **vertical** stress, **horizontal** and **shear** stress due to uniformly loaded area of any shape (regular or irregular) **below** any point, **inside** or **outside** the loaded area



$$\sigma_z = \frac{1}{m \times n} \times q \times N$$

m = No. of concentric circles (normally 10)

n = No. of radial lines (normally 20)

q = Intensity of load

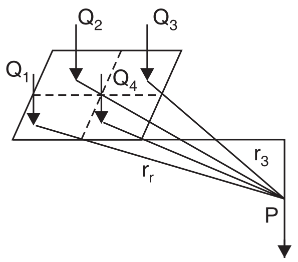
N = Equivalent no. of areas

All the area units wall have equal influence at the center whether the area is inside or outside the loaded area, it will have the same influence at the center of the chart.

Approximate method's

- Equivalent load method
- Trapezoidal method
- Stress isobar method

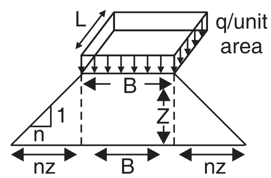
Equivalent load method



$$\sigma_z = \sigma_{z_1} + \sigma_{z_2} + \sigma_{z_3} + \ldots\ldots$$

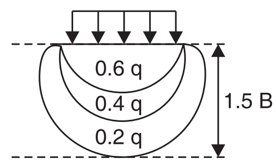
$$\sigma_{z_1} = K_{B_1}\frac{Q_1}{z^2} \quad \sigma_{z_2} = K_{B_2}\frac{Q_2}{z^2}$$

Trapezoidal method



$$\sigma_z = \frac{q(B \times L)}{(B + 2nz)\,(L + 2nz)}$$

Stress isobar method



$0.2\ q$ = 20% Stress isobar

Area bounded by $0.2\ q$ Stress isobar is considered to be stressed by vertical stress on loading.

11 Compressibility and Consolidation

Settlement

Gradual sinking of the structure due to compression of the soil below

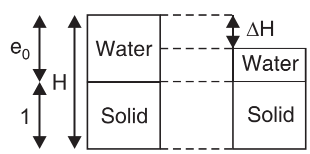
$$\boxed{S_t = S_{\text{immediate}} + S_{1^\circ-\text{consolidation}} + S_{2^\circ-\text{consolidation}}}$$

Due to **expulsion of air** or compression of pore air

Due to expulsion of **excess** pore water pressure generated due to increases in **Total stress.**

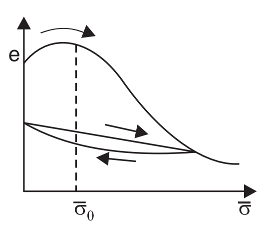
Due to gradual re-adjustment of clay particles into more stable configuration under **constant effective stress.**

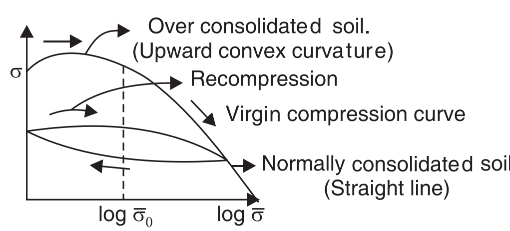
Note: one dimensional consolidation (ie zero lateral strain) is measured by oedometer test/consolidation test.



$$\frac{\Delta H}{H} = \frac{\Delta V}{V} = \frac{\Delta e}{1+e_o}$$

Compressibility characterstics





$\frac{\bar{\sigma}_0}{\sigma}$ = Over consolidation ratio (OCR)

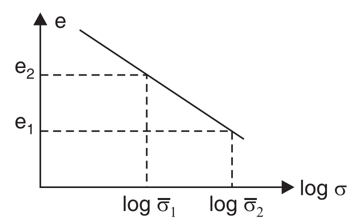
$\bar{\sigma}_0$ = Pre consolidation stress

If existing effective stress $\bar{\sigma} > \bar{\sigma}_0$, then soil is normally consolidated

ie OCR = 1

If $\bar{\sigma} < \bar{\sigma}_0$, then over consolidated soil an i.e. OCR > 1

Coefficients in Compressibility of clay



1. Compression index c_c

$$c_c = \frac{e_1 - e_2}{\log_{10}\bar{\sigma}_2 - \log_{10}\bar{\sigma}_1} = \frac{\Delta e}{\log_{10}\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)}$$

(*a*) $c_c = 0.009\,(w_L - 10)$ undisturbed soil of medium sensitivity, w_L = liquid limit %

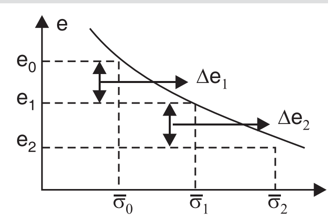
(*b*) $c_c = 0.007\,(w_L - 7)$ Remoulded soil of low sensitivity

(*c*) $c_c = 0.4\,(e_o - 0.25)$ undisturbed soil of medium sensitivity

(*d*) $c_c = 1.15\,(e_o - 0.35)$ Remoulded soil of low sensitivity.

Note: c_c has a **constant value** for a given type of soil and is not a function of effective stress.

2. Coefficient of Compressibility (a_v)



$$a_v = \frac{\Delta e}{\Delta\bar{\sigma}}$$

⇒ a_v decreases with the increase in effective stress.

3. Coefficient of volume compressibility (m_v)

$$m_v = \frac{\text{Volume change per unit volume}}{\text{Increase in effective stress}}$$

$$m_v = \frac{a_v}{1 + e_0} \qquad e_0 = \text{Initial void ratio}$$

⇒ value of m_v depends on stress range and is not constant for a particular soil.

$\Rightarrow$ compression modulus $E_c = \dfrac{1}{m_v}$

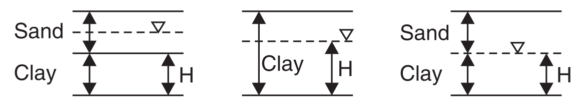
Computation of Primary Settlement

H_0 = Depth in clay below water table only

ΔH = Change in depth (settlement)

e_0 = Initral void ratio.

1. $\dfrac{\Delta H}{H} = \dfrac{\Delta e}{1 + e_0}$



2. $\Delta H = m_v \, \Delta \sigma \, H_0$
3. $\Delta H = \dfrac{c_c H_0}{1 + e_0} \log \left(\dfrac{\bar{\sigma}_0 + \Delta\bar{\sigma}_0}{\bar{\sigma}_0} \right)$
4. Over consolidated soil $\qquad \bar{\sigma}_0 + \Delta\sigma < \bar{\sigma}_c$

$$\Delta H = \frac{c_r \, H_0}{1 + e_0} \log \left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}} \right)$$

Normally consolidated soil $\bar{\sigma}_0 + \Delta\bar{\sigma} > \bar{\sigma}_c$

$$\Delta H = \frac{c_r H_0}{1 + e_0} \log \left(\frac{\bar{\sigma}_c}{\sigma_0} \right) + \frac{c_c H_0'}{1 + e_0'} \log \left(\frac{\bar{\sigma}_0 + \Delta\sigma}{\bar{\sigma}_c} \right)$$

Computation of Secondary Settlement:

$$\Delta H = \frac{c_\alpha \, H_0}{1 + e_0} \log_{10} \frac{t_2}{t_1}$$

e_0 = void ratio at the end of 1° settlement

H_0 = height at the end of 1° settlement.

$$c_\alpha = \frac{\Delta e}{\log (t_2 / t_1)} = \frac{\Delta e}{\Delta \log t}$$

c_α is 4-6% of the value of $\dfrac{c_c}{1 + e_o}$

Terzaghi's one dimensional consolidation equation:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2}$$

u = excess pore pressure

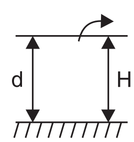
$\frac{\partial u}{\partial t}$ = Rate of change of pore pressure with time

where $C_V = \frac{K}{m_v \gamma_w}$ $\qquad$ C_V = coefficient of consolidation

$\frac{\partial u}{\partial z}$ = Rate of change of pore pressure with depth

Time factor (T_V):

$$T_V = \frac{C_V\, t}{d^2}$$



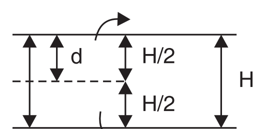
One way drainage

C_V = coefficient of consolidation in cm^2/sec

d = length of drainage path

d = H in one way drainage

$d = \frac{H}{2}$ in two way drainage



Two way drainage

(*a*) $u \leq 60\%$ $T_V = \frac{\pi}{4}(u)^2$ $\qquad$ u = Avg. degree of consolidation

(*b*) $u > 60\%$ $T_V = 1.781 - 0.933 \log(100 - u)$

$T_{50} = 0.196$

$T_{90} = 0.848$

Degree of consolidation

(*a*) $\qquad U = \frac{\Delta h}{\Delta H} \times 100$

Δh = settlement at any stage

ΔH = settlement at end of consolidation.

(*b*) $\qquad U = \frac{e_o - e}{e_o - e_f}$

e_o = initial void ratio

e = void ratio of any stage

e_f = final void ratio

(*c*) $$U = \frac{U_i - U}{U_i - U_f}$$

U_i = Initial pore water pressure

U = Pore water pressure at any stage

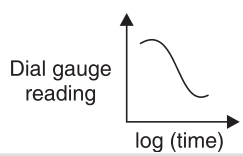
(*d*) $$U = \frac{U_i - U_z}{U_i}$$

U_f = Pore water pressure at the end of stress increment

U_z = excess pore pressure at any depth Z.

Determination of coefficient of consolidation (C_v)

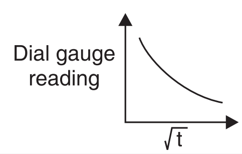
Casagrande's method (Logarithm of time fitting method)



$$C_v = \frac{T_{50}H^2}{t_{50}}$$

$T_{50} = 0.196$

Taylor's method (Square root time fitting method)



$$C_v = \frac{T_{90}H^2}{t_{90}}$$

$T_{90} = 0.848$

Note: C_V is inversely proportional to liquid limit (w_L) where as c_c is directly proportional to liquid limit.

Value of C_V decreases with increases in plasticity.

Compression Ratio's

(*a*) Initial compression ratio $$r_0 = \frac{R_i - R_0}{R_i - R_f}$$

(*b*) 1° – consolidation ratio $$r_p = \frac{R_o - R_{100}}{R_i - R_f}$$

(*c*) 2° – consolidation ratio $$r_s = 1 - (r_0 + r_p)$$

$$r_s = \frac{R_{100} - R_f}{R_i - R_f}$$

R_i = initial dial gauge reading

R_0 = Dial gauge reading for beginning of 1° – consolidation

R_{100} = Dial gauge reading for completion of 1° – consolidation

R_f = final dial gauge reading

12 Shear Strength of Soil

It is the capacity of the soil to resist shearing stress. It is defined as the maximum value of shear stress that can be moblised with in a soil mass.

It depends on two factors

- Frictional strength
  - Particle to particle friction
  - Interlocking
- Cohesive strength
  - True cohesion electrostatic attraction or cementation due to addicted of Ca to clay.
  - Apparent cohesion capillary rise or suction due to negative pressure

Note: Whatever be the nature of loading on soil, failure on soil occurs by shearing, it never occurs by crushing of particles.

Mohr's hypothesis: Shear stress on failure plane at failure reaches a value which is a unique function of normal stress on that plane.

$$\tau_{ff} = f(\sigma_{ff})$$

τ_{ff} = Shearstress on failure plane at failure

σ_{ff} = Normal stress on failure plane at failure.

Coulomb's hypothesis:

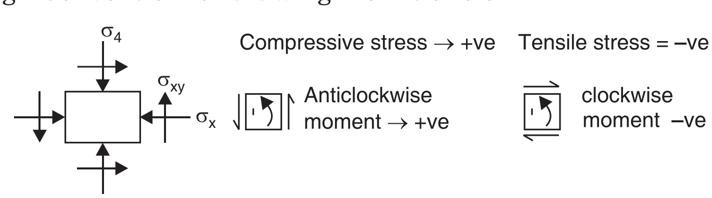
$$\tau_f = C + \sigma \tan \phi$$

$$\tau_f = C' + \bar{\sigma} \tan \phi'$$

where C, ϕ are total stress parameters and C′, ϕ' are effective stress parameters.

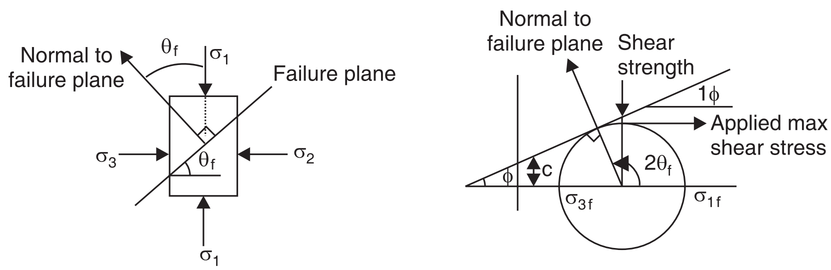
Note: Shear strength parameter's C, ϕ , C′, ϕ' are not the inherent properties of soil. There are related to the type of test and the condition under which these are measured.

Sign Convention for drawing Mohr's circle



Relation between of ϕ_f, ϕ, σ_{1f} and ϕ_{3f}

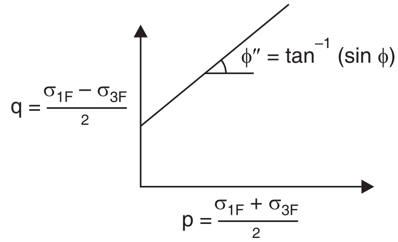
Based on Mohr's Colomb's failure envelope



$$\theta_f = 45 + \frac{\phi}{2}$$

i.e. failure plane makes an angle of (45 + ϕ/2) degree with major principal plane

$$\frac{\sigma_{1f} - \sigma_{3f}}{2} = \frac{\sigma_{1f} + \sigma_{3f}}{2} \sin\phi + C\cos\phi$$



p-q plot

$$\sigma_{1f} = \sigma_{3f}\tan^2\left(45 + \frac{\phi}{2}\right) + 2C\tan\left(45 + \frac{\phi}{2}\right)$$

$$\sigma_{3f} = \sigma_{1f}\tan^2\left(45 - \frac{\phi}{2}\right) - 2C\tan\left(45 - \frac{\phi}{2}\right)$$

Stability analysis based on drainage conditions

Drained Condition: If the rate of loading is such that water in the pores of soil gets sufficient time to drain out, the condition of loading is called drained condition.

Undrained Condition: The rate of loading is rapid such that the water in the pores does not gets sufficient time to drain out, that condition of loading is called undrained condition.

Note: Both drained and undrained conditions depends on rate of loading and type of soil. Hence these conditions are actually relative.

(*a*) Shear strength under drained condition

- ➢ Effective stress approach is used
- ➢ Drained analysis is used to evaluate long term stability.

(*b*) Shear strength under undrained condition

(*i*) +ve excess pore water pressure develops

- ➢ Total stress approach is used
- ➢ Shear strength evaluated at the end of construction period

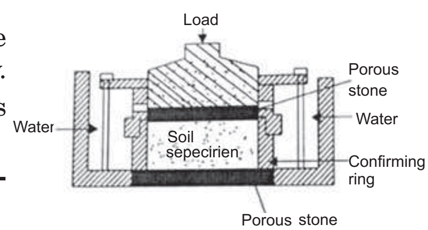
(*ii*) –ve excess pore water pressure develops

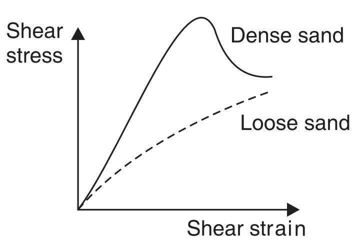
- ➢ Effective stress approach is used
- ➢ Long term stability is analysed

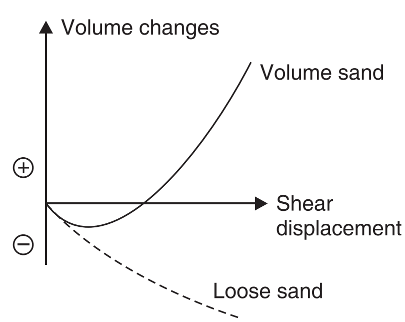
- Drained condition
  - Long term stability
  - Effective stress approach
- Undrained condition
  - +ve pore water pressure
    - Immediate analysis
    - Total stress approach
  - –ve pore water pressure
    - Long term analysis
    - Effective stress approach

Direct shear test

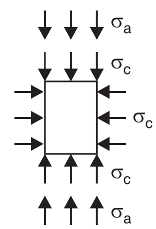
- Good for free draining soil like sand and gravel not used for clay.
- Effective stress and total stress are same
- Shear normally applied at **constant rate of strain**
- Shear and vertical deformations are measured during test using dial gauge







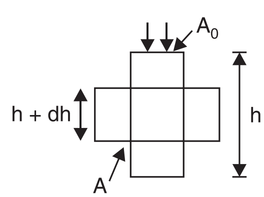
Triaxial test

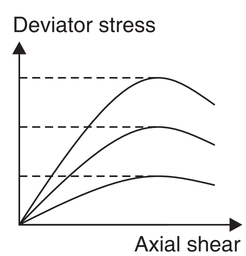


- Suitable for all types of soil
- Pore water pressure can be measured under undrained condition.
- Volume change can also be measured under drained condition.

$$\sigma_3 = \sigma_C = \text{confining pressure}$$
$$\sigma_1 = \sigma_C + \sigma_a \quad \sigma_a = \text{deviator stress.}$$
$$\sigma_C = \bar{\sigma}_{V_0}\left(\frac{1 + 2K_0}{3}\right)$$
$$K_0 = \text{Coefficient of earth pressure at rest} = 1 - \sin\phi$$
$$\bar{\sigma}_{V_0} = \text{Vertical effective stress in the field.}$$

$$\sigma_a = \sigma_1 - \sigma_3 = \frac{\text{Axial load}}{\text{Corrected area}}$$





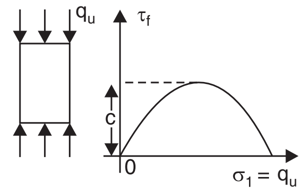
$$A(h + dh) = V = V_0 + dV$$
$$A = \frac{A_0(1 - \varepsilon_v)}{(1 - \varepsilon_a)}$$

For undrained test $\varepsilon_v = 0$

So $$A = \frac{A_0}{1 - \varepsilon_a}, \quad \varepsilon_a = \frac{dh}{h}$$

Note: At any time in triaxial test either pore water line is open and drainage line is closed or vice-versa, i.e. either pore water pressure measurement will be made under undrained condition or volume change is measured under drained condition.

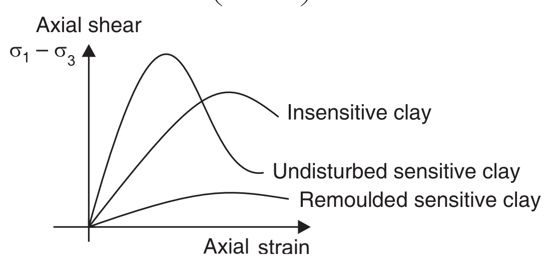
Unconfined Compression Test (UC Test)



- For fully saturated soil. (otherwise $\phi \neq 0$)
- No confining pressure is applied $(\sigma_C = 0)$
- Used to test cohesive soil, in non-cohesive soil sample cannot be prepared without confining pressure.

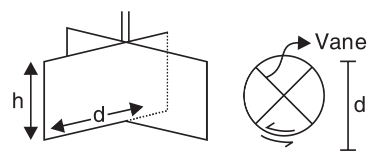
$$q_u = \text{unconfined compressive strength}$$

$$q_u = \frac{P}{A_f} = \frac{P}{\left(\dfrac{A_0}{1-\varepsilon_a}\right)}$$

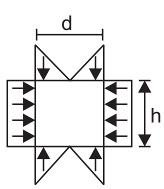


- Used for rapid assesment of **consistency.**
- Also used for finding **sensitivity** of clay soil, other than fissured clay.

Vane Shear test



- For plastic cohesive soil which is very sensitive
- Maximum torque applied is the total shear



$$\tau_f = \frac{T}{\pi d^2\left(\dfrac{h}{2}+\dfrac{d}{6}\right)}$$

[when both top and bottom end shear the soil]

$$\tau_f = \frac{T}{\pi d^2\left(\dfrac{h'}{2}+\dfrac{d}{12}\right)}$$

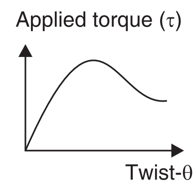
[when top end does not shear's the soil n']

h′ → height of vane used in shearing.

Here, τ_f = Shear strength = C_u (undrained cohesion)

$$C_u = \frac{q_u}{2}$$

$$\text{Sensitivity} = \frac{C_u \text{ undisturbed}}{C_u \text{ remoulded}}$$



	Lab size	Field size
Height of Vane (H)	20 mm	10–20 cm
Dia of Vane (D)	17 mm	10 cm
Thickness of Vane (t)	0.5–0.1 mm	2–3 cm

Type of Triaxial Tests

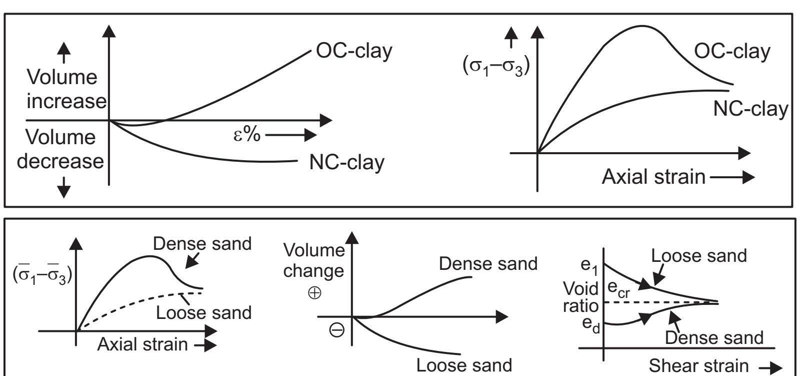
Consolidated drained (CD)
- Drainage allowed in both stages
- Takes long time

Consolidated Undrained test (CU)
- Drainage allowed in 1st stage only
- 24 hr's in 1st stage 2 hr's in 2nd stage

Unconsolidated undrained (UU)
- Drainage not allowed in any stage
- 15 min test

Consolidated Drained Test

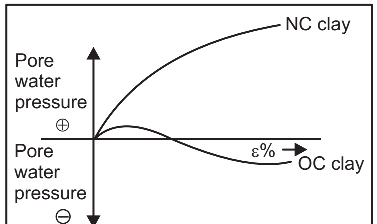
- Used in Analysis of gradual loading condition
- To check long term stability of embankment which has been in existence since long



Results of CD Test

Consolidated Undrained Test

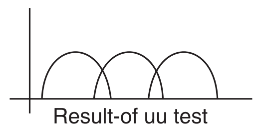
- Sudden unloading such as draw down or dewatering
- Unloading of an embankment that has lived some of its life (i.e. already consolidated)
- Gives both total stress and effective stress parameters



Results of CD Test

Unconsolidated Undrained test

- Test is suitable for soil of low permeability or when loading is very fast.
- Total stress analysis is performed as it is an undrained test.
- Used in sudden loading such as rapid construction
- Short term stability under pore water pressure.


Result-of uu test

Note: $\dfrac{C_u}{\bar{\sigma}_\tau} = 0.11 + 0.0037\, I_P\%$

For NC clays, shear strength depends on plasticity Index.

Soil Liquefaction: The phenomenon in which **Saturated loose sand** in **undrained** condition under rapid loading developes **positive pore water water pressure** which ultimately reduces the **effective stress to zero**, is called liquefaction. It occurs at high frequency of Vibration e.g. during pile driving, vibration of machine, explosive blasting and earthquake shock.

Pore Pressure Coefficients: It is used to express the response of pore water pressure to change in total stress under undrained conditions and enable the initial value of pore water pressure to be determined.

$$\Delta u = B[\Delta\sigma_3 + \Delta(\Delta\sigma_1 - \Delta\sigma_3)]$$

$$\Delta U_1 = B\Delta\sigma_3$$

$$\Delta U_2 = AB\,(\Delta\sigma_1 - \Delta\sigma_3)$$

$$B = \frac{\Delta U_1}{\Delta\sigma_3}$$

ΔU_1 = Change in pore pressure due to increase in cell pressure

ΔU_2 = Change in pore pressure due to increase in deviator stress.

- For saturated soil   B = 1
- For dry soil   B = 0
- B varies with the stress range
- A can be found from CD test
- B can be found from first stage in UU test and AB from its second stage.
- A is also not a constant, depends on type of soil, stress condition etc.
- For a given soil A depends on strain, sample distribution, anisotropy and OCR.

Value of A	Type of Soil
– 0.5 to 0	Heavily over consolidated Soil
< 0	Over consolidated soil
0.5–1	Normally consolidated soil
2–3	Loose saturated time sand

Stability of Slopes 13

Assumption's in analysis of slope stability

(*a*) Slope–stability problem is a two dimensional problem.

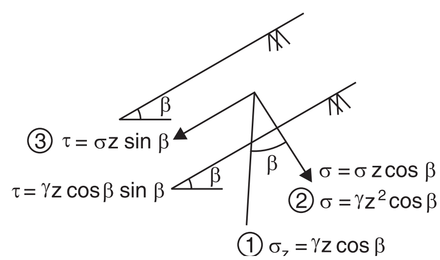
(*b*) Shear parameters of the soil are constant along any possible slip surface.

(*c*) In problems involving seepage of water, flow net can be constructed and the seepage force can be determined.

Stability of Infinite Slope

$$\text{F.O.S} = \frac{\tau_f}{\tau}$$

$$\text{F.O.S} = \frac{C + \sigma \tan \phi}{\gamma z \cos \beta \sin \beta}$$

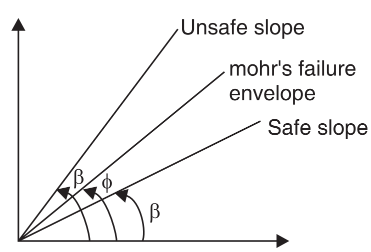


(*a*) $F = \dfrac{\tan \phi}{\tan \beta}$

when $C = 0$

$\sigma = \gamma z \cos^2 \beta$

(*b*) $F = \dfrac{C + \gamma z \cos^2 \beta \tan \phi}{\gamma z \cos \beta \sin \beta}$



$\beta < \phi$, then $F > 1$

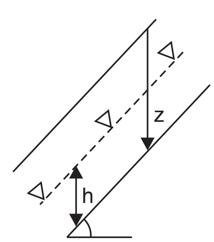
(*c*) $F = 1$, and $Z = H_C$ then

$$H_C = \frac{C}{\gamma \cos^2 \beta (\tan \beta - \sin \phi)}$$

(*d*) $S_n = \dfrac{C}{\gamma H_C} = \cos^2 \beta (\tan \beta - \sin \phi)$

Hence, S_n is a dimensionless Quantity

(*e*) Seepage taking place and W.T. is parallel to the slope in cohensionless soil.



$$F_S = \left[1 - \left(\frac{\gamma_w}{\gamma}\right)\left(\frac{h}{z}\right)\right] \frac{\tan \phi'}{\tan \beta}$$

ϕ' = effective friction angle

γ = avg. total unit weight of the soil above the slip surface upto the ground level.

$$\gamma = \frac{\gamma_1 h_1 + \gamma_2 h_2}{h_1 + h_2}$$

(*f*) If water table is at ground level

$$F_S = \frac{\gamma'}{\gamma_{sat}} \frac{\tan\phi}{\tan\beta} \approx F_S = \frac{1}{2}\frac{\tan\phi}{\tan\beta}$$

(*g*) Infinite slope of purely cohesive soil.

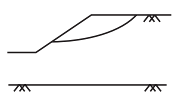
$$F_C = \frac{C}{\gamma z \sin\beta \cos\beta}$$

(*h*) C – ϕ soil in infinite slope

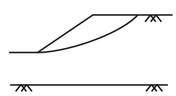
$$F_S = \frac{C}{\gamma z \sin\beta\cos\beta} + \frac{\tan\phi}{\tan\beta}$$

Stability of Finite Slope

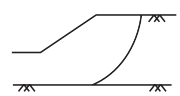
(*a*) **Swedish Circle Method:** Surface of sliding is assumed as "arc of circle"



Face/slope failure soil close to the toe is quite strong

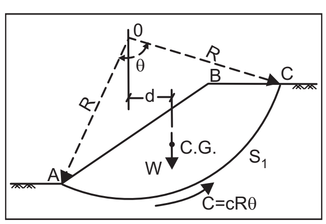


Toe failure most common mode of failure $\beta > 53° \ \phi > 3°$



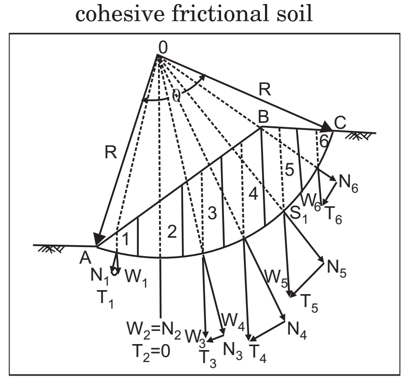
Base failure soil below the toe is soft and slope is flat $\beta < 53° \ \phi < 3°$

Purely cohesive soil



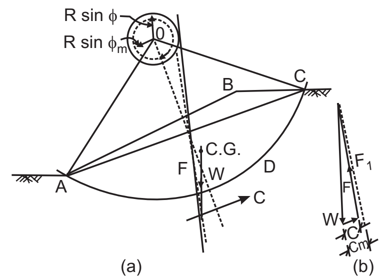
$$F = \frac{CR^2\theta}{Wd}$$

cohesive frictional soil



$$F = \frac{CR\theta + \Sigma N \tan\phi}{\Sigma T}$$

(*b*) **Friction Circle Method:** This method is based on the assumption that the resultant force R on the rupture surface is **tangential** to a circle of radius **r=R sin** ϕ which is concentric with the trial slip circle.



- ➢ friction is assumed to be fully moblised
- ➢ Total stress approach is used

$$F_C = \frac{C}{C_m}$$

$$F_\phi = \frac{\tan\phi}{\tan\beta} = \frac{\tan\phi}{\tan\phi_m}$$

(*c*) **Taylor's stability number:** It's a **dimensionless** parameter. It is obtained for factor of safety wrt cohesion while the factor of safety wrt friction $\mathbf{F}_\phi$ is assumed to be unity.

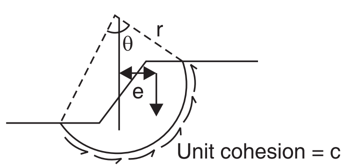
$$S_n = \frac{C}{\gamma H_C} = \frac{C}{\gamma F_C H}$$

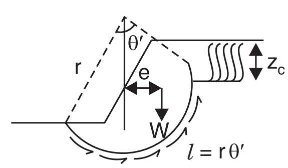
In case of saturated slope replace γ by γ°_{sat} while in submerged slope replace it by γ'.

$$\phi_w = \frac{\gamma'}{\gamma_{sat}}\,\phi$$

ϕ_w = weight friction angle.

(*d*) **Fellinious Methods:** For purely cohesive soil





$$F = \frac{Cr^2\theta}{We}$$

r = radius of rupture curve

$$F = \frac{Cr^2\theta'}{We}$$

$$Z_C = \frac{2C}{\gamma}$$

14 Earth pressure and Retaining walls

Earth pressure is the lateral force exerted by the soil on any structure retaining that soil. There are several types of retaining structure's like retaining walls (gravity type, cantilever type, counter fort type), bracings in cuts, abutment of a bridge, sheet pile/anchored sheet pile.

Type's of lateral earth pressure

- Earth pressure at rest (wall does not moves at all)
- Active earth pressure (wall moves away from backfill)
- Postive earth pressure (wall moves towards the backfill)

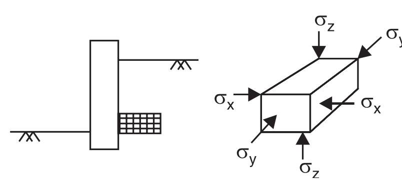
1. Earth pressure at rest

$\Rightarrow$ $\sigma_x = \sigma_y$ and $\varepsilon_x = 0$

So $$\sigma_x = \frac{\mu}{1-\mu}\,\sigma_z$$

$$\sigma_x = K_0\,\sigma_z$$

Hence, $$K_0 = \frac{\mu}{1-\mu}$$



$\Rightarrow$ For cohesion less soil $(C = 0)$ $K_0 = 1 - \sin\phi$

$\Rightarrow$ For normally consolidated soil (N. C. Soil)

$$K_0 = 0.19 + 0.233 \log (I_P)$$

$\Rightarrow$ For over consolidated soil (OC soil) $K_{o(oc)} = K_{o(NC)}\sqrt{O.C.R}$

where OCR = over consolidation ratio.

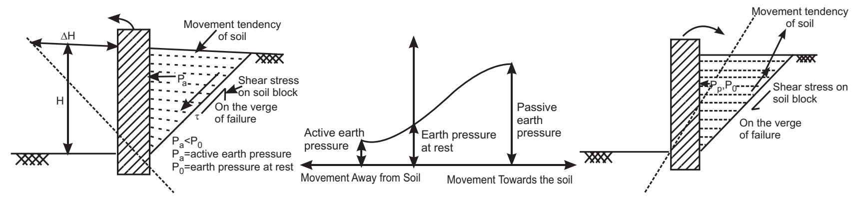
Type of soil	K_0
Dense sand	0.4 – 0.5
Loose sand	0.45 – 0.5
Mechanically compacted	0.8 – 1.0
N.C clays	0.5 – 0.6
O.C clays	1.0 – 4.0

Note: K_0 for dens sand is lesser than that of loose sand.

For NC clays $K_0 < 1$

For OC clays $K_0 > 1$

Active earth pressure and passive earth pressure



Active earth pressure	Passive earth pressure
⇒ Failure plane is inclined at 45 + $\phi/2$ with the horizontal	⇒ Failure plane is inclined at 45 – $\phi/2$ with the horizontal
⇒ Very little movement is required to mobilise the active pressure	⇒ much higher movement is required to mobilise the pressure to mobilise the pressure
ΔH = 0.2 % of H Dense sands ΔH = 0.5 % of H loose sands ΔH = 0.4 % of H clay's	ΔH = 2% of H Dense sands ΔH = 15% of H loose sands
⇒ length of failure block $= H \cot\left(45+\frac{\phi}{2}\right)$ $\Rightarrow K_q = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2\left(45+\frac{\phi}{2}\right)$	⇒ length of failure block = H cot $\left(45-\frac{\phi}{2}\right)$ $\Rightarrow k_P = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left(45+\frac{\phi}{2}\right)$

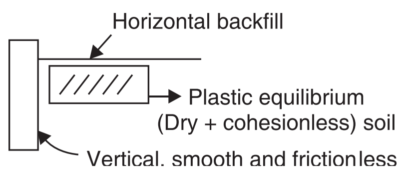
Earth pressure theories

1. Rankine theory (1857)
2. Coulomb's theory (1776)

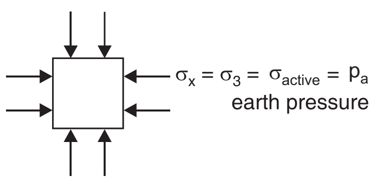
Note: Coulomb's theory came before Rankine's theory and was even more versatile as it considered friction between wall and the soil but it is still not used because this theory does not satisfies the static equilibrium condition occuring in nature. The three forces do not meet at a common point when sliding surface is assumed to be planer.

Rankine's theory **over estimates the active earth pressure** and **under estimates the passive earth pressure** and as retaining walls are designed for active earth pressure hence, Rankine theory gives more conservative results.

1. **Rankine's theory.** Originally Rankine's theory was only for cohesionless soil but later it was extended to cohesive as well as submerged soil.



(*a*) Active earth pressure



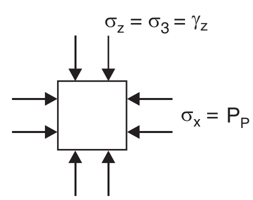
$$p_a = k_a \gamma z - 2c\sqrt{k_a}$$

$$\text{where } k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2\left(45 - \frac{\phi}{2}\right)$$

(*b*) Passive earth pressure

$$P_P = k_P \gamma_z + 2c\sqrt{k_P}$$

$$\text{where } k_P = \frac{1 + \sin\phi}{1 - \sin\phi} = \tan^2\left(45 + \frac{\phi}{2}\right)$$

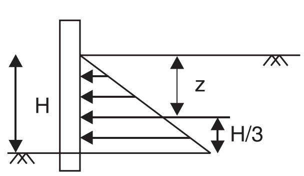


Note: By compacting soil active earth pressure decreases where as passive earth pressure increases

$$k_a = \frac{1}{k_p}$$

Various cases of earth pressures

1. **Cohesionless soil on a vertical smooth wall**



$$p_{az} = k_a \gamma z$$

$$p_{ap} = k_p \gamma z$$

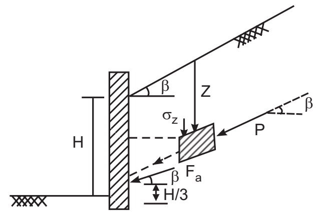
$$F_a = \frac{k_a \gamma H^2}{2} \qquad F_p = \frac{k_p \gamma H^2}{2}$$

(6) **Soil with Inclined back fill**

$$\sigma_z = \gamma z \cos\beta$$

$$p_a = k_a \gamma z \cos\beta$$

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



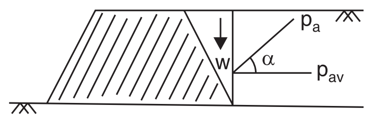
$$K_p = \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$F_a = \frac{k_a \gamma H^2}{2} \cos\beta$$

7. **Horizontal back fill with inclined wall**

$$P_a = \sqrt{W^2 + p_{av}^2}$$

$$p_{av} = \frac{1}{2} k_a \gamma H^2$$

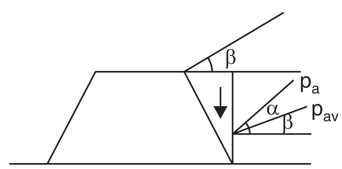


$$\tan\alpha = \frac{W}{p_{av}}$$

8. **Inclined back fill with inclined wall**

$$P_a = \sqrt{P_{av^2} + W^2 + 2P_{av} W \sin\beta}$$

$$\tan\alpha = \frac{W + P_{av}\sin\beta}{P_{av}\cos\beta}$$

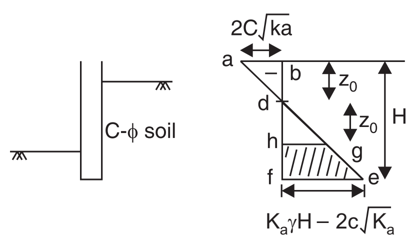


9. **Active earth pressure on cohesive soil.**

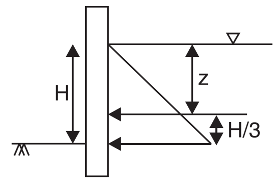
$$H_C = 2 z_0 = \frac{4C}{\gamma\sqrt{K_a}}$$

No-contact loss = Active earth pressure corresponds to area efgh

After contact loss = Active earth pressure corresponds to area fde

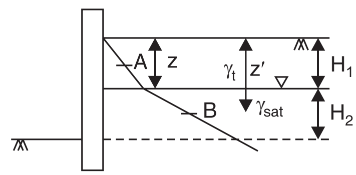


2. **Submerged cohesionless soil on vertical smooth wall**



$$p_{az} = k_a\,\gamma_{sub}\,Z + \gamma_w\,Z$$
$$p_{pz} = K_P\,\gamma_{sub}\,Z + \gamma_w\,Z$$
$$F_a = \frac{K_a \gamma_{sub} H^2}{2} + \frac{\gamma_w H^2}{2}$$

3. **Partially submerged cohesionless soil on vertical smooth wall**



For point A

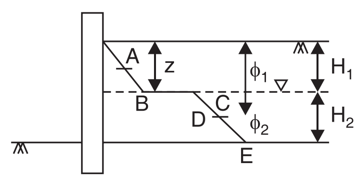
$$p_a = k_a\,\gamma_t\,z$$

For point B

$$p_a = K_a \underbrace{(\gamma_t H_1 - \gamma_{sub}\,(Z' - H_1))}_{\text{Effective stress}} + \gamma_w\,(Z' - H_1)$$

Note: water pressure due hydrostatic condition is same in all directions hence hydrostatic pressure is not multiplied by k_a

4. **Back fill with two different soils (ϕ_1 and ϕ_2)**



A: $p_a = K_{a1}\,\gamma_2\,Z$

B: $p_a = K_{a1}\,\gamma_1\,H_1$

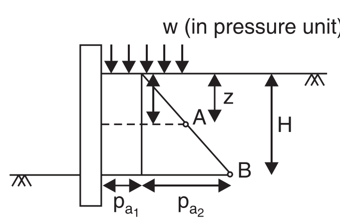
C: $p_a = K_{a2}\,\gamma_1\,H_1$

D: $p_a = K_{a2}\,(\gamma_1\,H_1 + \gamma_2\,(Z' - H_1))$

E: $p_a = K_{a2}\,(\gamma_1\,H_1 + \gamma_2\,H_2)$

5. **Soil with surcharge load**

$$p_a = p_{a_1} + p_{a_2}$$



A: $p_{a_1} = k_a w$

$p_{a_2} = k_a\,\gamma z$

B: $p_{a_1} = k_a w \quad pa_2 = k_a\,\gamma\,H$

$$F = K_{a1}\,w\,H + \frac{k_a \gamma\,H^2}{2}$$

For No contact loss

$$F_a = \left[\frac{K_a \gamma Z^2}{2} - 2C\sqrt{K_a}\,Z + \frac{2C^2}{\gamma}\right] - \left[\frac{1}{2} \times \frac{2C}{\gamma\sqrt{K_a}} \times 2C\sqrt{K_a}\right]$$

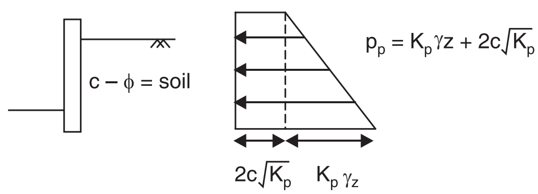
$$F_a = \frac{K_a \gamma Z^2}{2} - 2C\sqrt{K_a}Z$$

After contact loss

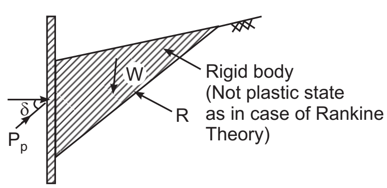
$$F_a = \frac{K_a \gamma Z^2}{2} - 2C\sqrt{K_a}Z + \frac{2C^2}{\gamma}$$

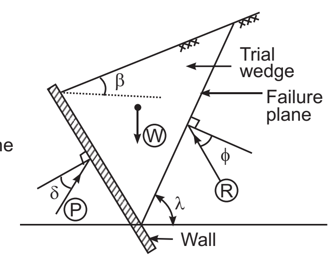
Note: As there is no net earth pressure upto a depth of $2Z_0$ we can make unbraced cut in clayey soil upto depth of $2Z_0$. i.e. $\dfrac{4C}{\gamma\sqrt{K_a}}$

10. **Passive earth pressure on cohesive soil:**



Coulomb's theory of earth pressure:





Here sliding wedge is assumed as a at an angle λ from horizontal rigid body.

Forces acting on trial wedge will be W, R, P whose directions will be known. The position and line also of action of earth pressure will also be known in advance. By assuming various trial wedges at different trial angle λ the value of P will be calculated.

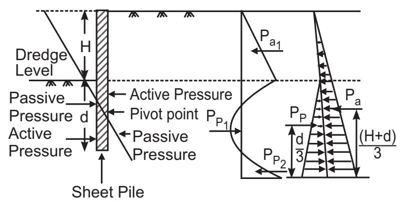
Highest value of P is for active earth pressure while lowers value of P is for passive earth pressure.

15 Cantilever Sheet Pile and Anchored Bulkhead

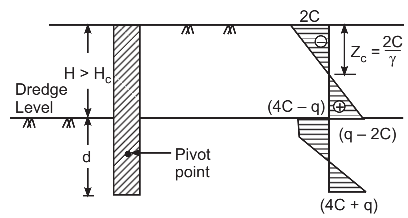
A sheet pile wall is an earth and water retaining structure which behaves as a fixed vertical cantilevers in resisting lateral earth pressure

Cantilever sheet pile

Granular Soil



Cohesive soil



$$P_P \cdot \frac{d}{3} = P_a \frac{(H+d)}{3} \quad \text{without FOS}$$

$$\frac{P_P}{FOS} \times \frac{d}{3} = P_a \frac{(H+d)}{3} \quad \text{with FOS}$$

$$P_P = \frac{1}{2} K_a \gamma d^2$$

$$P_P = \frac{1}{2} K_a \gamma (H+d)^2$$

$q = \gamma H$

At depth H,

$P_a = q - 2C$

$P_P = 2C$

Resultant at depth H,

$P_P - P_a$

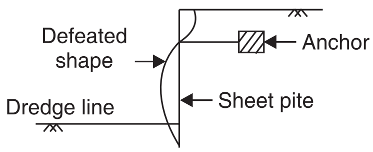
$P_P - P_a = 4C - q$

Resultant earth pressure at H + d

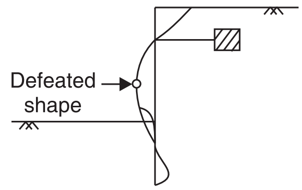
$P_P - P_a = 4C \pm q$

Anchored Bulkhead

If the wall height is large, support against lateral pressure is provided by embedment in the ground as well as by tie rods near the top. This type of earth retaining structure is called an anchored bulkhead.



Anchored bulkhead at shallow depth



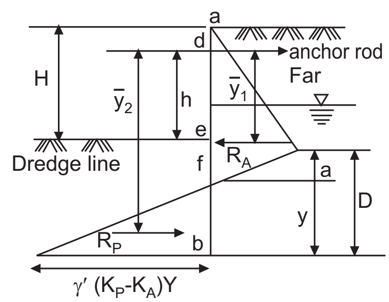
Anchored bulkhead down to deeper depth

(a) Anchored bulkhead in granular soil.

F_{ar} = Force in anchor rod

R_A = Resultant active earth pressure acting at $\bar{y}_1$ below the anchor rod level.

R_P = Resultant passive earth pressure acting at $\bar{y}_2$ below the anchor rod.



(a) Granular Soil

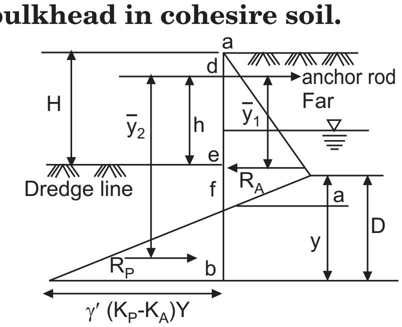
For equillibriun

$F_{ar} + R_P - R_a = 0$

Depth 'a' to the print of zero pressure

$$a = \frac{p_{AC}}{\gamma' (K_P - K_a)}$$

(b) **Anchored bulkhead in cohesire soil.**



(a) Granular Soil

Coffer Dam: It is a temporary structure constructed usually in river lake etc to keep the working area dry for construction of other structure.

After the constriction of coffer dam area is dewatered by pumping.

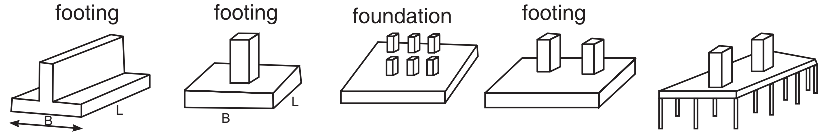
Types of coffer dam: Earth embankments, cantilever sheet pile, double wall coffer dam, braced coffer dans.

16 Shallow Foundation

Footings are the lower most supporting part of the structure known as sub-structure and are last structural elements through which load is transferred to foundation comprising soil/rock.

Types of footings

- Strip footing
- Isolated footing
- Roft/mat foundation
- Combined footing
- Pile foundation



General requirement for foundation

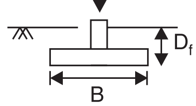
(*a*) Foundation must be safe against shear failure.

(*b*) Settlement of the foundation should be with in permissible limits.

(*c*) Foundation should be located at such depth, such that its performance is not affected by seasonal volume changes.

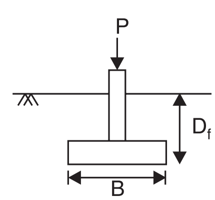
Types of footings (As per Terzaghi)

- Shallow foundation: $\frac{D_F}{B} \le 1$. Eg:- Isolated footings combined footing, raft foundation
- Deep foundation: $\frac{D_F}{B} > 15$. Eg;- Pile foundation



Basic Definitions

1. **Bearing Capacity:** The load carrying capacity of foundation soil or rock which enables it to bear and transmit loads from the structure.
2. **Gross Pressure or Gross Loading Intensity** (q_g)**:** It is the total pressure at the base of the footing due to weight of the super-structure,



Square footing

self weight of the footing and weight of the earth fill.

$$q_g = \frac{P}{B^2} + \gamma D_f$$

3. **Net Pressure Intensity:** Its the excess of gross pressure to over burden pressure.

$$q_{\text{net}} = q_g - \gamma D_f$$

For safe design

$$\frac{P}{B^2} \leq q_{\text{net-allowable}}$$

Note: It excavation is not backfilled (i.e. in case of basements)

$$q_{\text{net}} = \frac{P}{B^2} - \gamma D_f \quad \text{and} \quad \frac{P}{B^2} - \gamma D_f \leq q_{\text{net-allowable}}$$

In this case load carrying capacity of the soil is increased. If the raft is constructed at the depth such that $\frac{P}{B^2} - \gamma D_f = 0$, then soil is called upon not to resist any load. Such as raft is called fully **compensated raft** or floating raft.

4. **Ultimate bearing capacity** (q_u): The maximum grass intensity of loading that the soil can support before it fails in **shear** is called ultimate bearing capacity.
5. **Net ultimate bearing capacity:** It is the minimum net pressure causing shear failure of soil.

$$q_{\text{nu}} = q_u - \gamma D_f = \frac{P}{B^2}$$

6. **Net Safe bearing Capacity:**

$$q_{\text{ns}} = \frac{q_{\text{nu}}}{\text{FOS}}$$

where FOS of 2-3 is adopted.

7. **Gross safe bearing Capacity:**

$$q_s = \frac{q_{nu}}{\text{FOS}} + \gamma D_f$$

8. **Safe bearing Pressure:** Maximum net intensity of loading that can be allowed on soil without the **settlement** exceeding the permissible value.

Note: No factor of Safety is used when dealing with settlement.

9. **Allowable bearing pressure:** Maximum net intensity of loading that can be imposed on the soil with no possibility of **shear** failure or the possibility of **excessive** settlement.

Note: It is smaller of Net safe bearing capacity and safe bearing pressure.

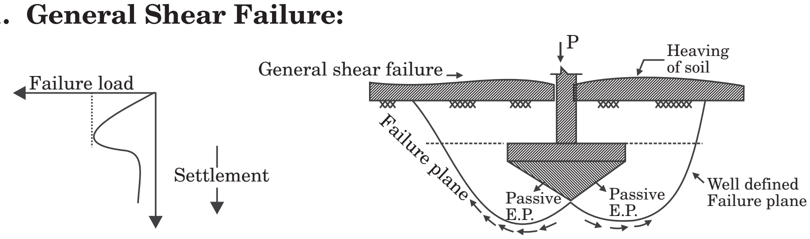
Determination of bearing capacity

- Analytical methods
  - General shear failure
  - Punching shear failure
  - Local shear failure
- Bulding codes
  - Rankine's method
  - Bell's theory
  - Fellinious method
  - Prandtl method
  - Terzaghi method
  - Skempton's method
  - Meyor's method
  - Hensen's Recommendations
- Field test methods
  - Standard penetration test (SPT)
  - Plate load test
  - Static cone penetration test

Analytical Method’s

Based on shear failure criteria

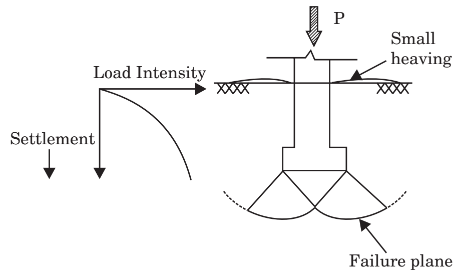
1. **General Shear Failure:**



Brittle type stress
strain curve

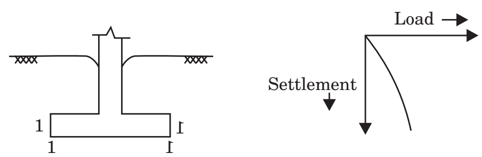
- ➢ Brittle type shear-stress curve
- ➢ Over consolidated clay with low compressibility
- ➢ Well defined failure pattern
- ➢ Failure due to tilting of foundation
- ➢ Occurs in soil with relative density > 70%
- ➢ Occurs after plastic equilibrium state is reached.

2. **Local Shear Failure:**



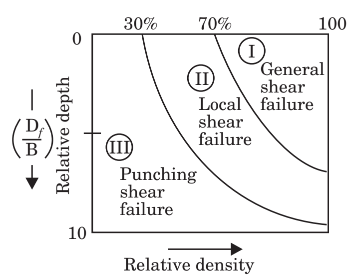
- ➢ Partial development of state of plastic equillibrium
- ➢ Slight heaving of soil adjacent to foundation
- ➢ Foundation doesn't tilts
- ➢ Failure is not sudden
- ➢ Plastic stress-strain curve
- ➢ Occurs in loose sand with relative density 30–70%

3. **Punching Shear Failure:**



- ➢ Shearing in vertical direction around the edge
- ➢ No heaving of adjacent soil
- ➢ No tilting of foundation
- ➢ Very loose sand with relative density less than 30%
- ➢ Deep foundations generally fail by punching only

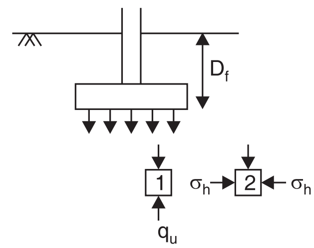
Parameter	General shear failure	Local Shear failure
1. Friction angle(ϕ)	> 36°	< 28°
2. Strain at failure	≤ 5°	≥ 15°
3. SPT number	> 30	< 5
4. Relative density	> 17%	< 20%
5. Void ratio	< 0.55	> 0.75
6. Unconfined Compressive Strength	> 100 KN/m^2	< 80 KN/m^2



Vesic's Curve for Sand

Building Codes

1. **Rankine's Method** (ϕ-soil):



$$q_u - \gamma D_f \tan^4\left(45 + \frac{\phi}{2}\right)$$

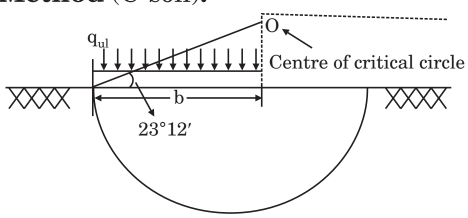
$$q_u = \gamma D_f \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$$

2. **Bell Theory** (C – ϕ soil):

$$q_u = CN_C + \gamma D_f N_q$$

for pure clays C = 4, $N_q = 1$

3. **Fellinious Method** (C-soil):



Location of critical circle for surface footing in Fellenius' method

Failure due to slip and consequent heaving of a mass of soil on one side

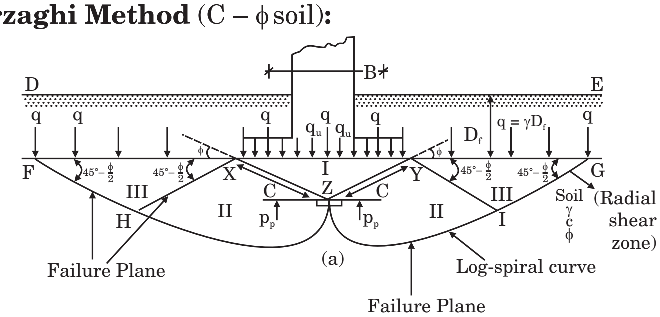
$$q_{ult} = \frac{Wlr + CR}{b \,.\, lo} \qquad q_{ult} = 5.5\ C$$

4. **Prandtl Method** (C – ϕ soil):

$$q_u = CN_C + rD_f N_q + 0.5\ Br\ N_r \quad \text{for strip footing}$$

For C-soil $N_C = 5.14$ Nq = 1 Nγ = 0

5. **Terzaghi Method** (C – ϕ soil):



(a)

- General shear failure governed by Mohr's criteria
- Strip footing at shallow depth ($D_f/B \leq 1$)
- Shear resistance of soil between ground surface and footing base is neglected

Zone I → Zone of elastic Equillibrium

Zone II → Radial Shear Zone

For C-soil → circular shape

C – ϕ soil → logarithmic shape

Zone III → Passive plastic Equillibrium with $\left(45 - \frac{\phi}{2}\right)$ angle with the Horizontal.

For strip footing,

$$q_u = CN_C + \gamma D_f N_q + 0.5\, B\gamma N_r$$

$$q_{nu} = \underbrace{CN_C}_{\text{Cohesion}} + \underbrace{(\gamma D_f - 1)N_q}_{\text{Over b urden}} + \underbrace{0.5\, B\gamma N_r}_{\text{Soil in Shearing Zone}}$$

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

$$N_q = \frac{a^2}{2\cos^2(45 + \phi/2)}$$

$$a = e^{(3\pi/4 - \phi/2)\tan\phi}$$

$$N_r = \frac{1}{2}\tan\phi\left[\frac{K_{p\gamma}}{\cos^2\phi} - 1\right]$$

Note: N_c, N_q, N_γ are the function's of ϕ only. Even K_p (passive earth pressure coefficient) is also function of ϕ. Hence the bearing capacity increases as the value of ϕ increases.

For clayey soil ($\phi = 0$) $N_C = 5.7$ $N_q = 1$ $N_\gamma = 0$

Modification in Terzaghi's Equation

1. For Source footing

$$q_{nu} = 1.3\, CN_C + q\,(N_q - 1) + 0.4\, B\gamma N_r$$

2. For Circular footing

$$q_{nu} = 1.3\, CN_C + q\,(N_q - 1) + 0.3\, B\gamma Nr$$

3. For rectangular footing

$$q_{nu} = \left(1 + \frac{0.3\,B}{L}\right)CN_C + q(N_q - 1) + \left(1 - \frac{0.2\,B}{L}\right)(0.5\, B\gamma N_\gamma)$$

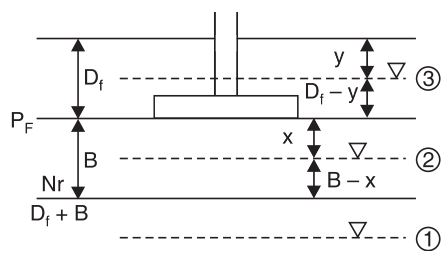
4. For local shear failure ($\phi < 29°$)

$$C_m \rightarrow \frac{2}{3}C \qquad (\tan\phi)_m = \frac{2}{3}\tan\phi$$

From C_m and ϕ_m find N_C', N_q', N_r'

$$q_{nu} = \frac{2}{3}CN_C' + q(N_q' - 1) + 0.5\,B\gamma\,N\gamma'$$

5. For water table, in strip footing



(*a*) For (1) → water table below $D_f + B$

$$q_{nu} = CN_C + \gamma_t D_f (N_q - 1) + 0.5\,B\,\gamma_t\,N_r$$

(*b*) For (2) → water table between D_f and $D_f + B$

$$q_{nu} = CN_C + \gamma_t D_f (N_q - 1) + 0.5\,(x\gamma_t + (B - x)\,\gamma_{sub})N_r\;D_f$$

(*c*) For (3) water table between 0 and D_f

$$q_{nu} = CN_C + [\gamma_t y + (D_f - y)\,\gamma_{sub}]\,(N_q - 1) + 0.5\,B\,\gamma_{sub}]\,N_r$$

Another way of modification due to water table

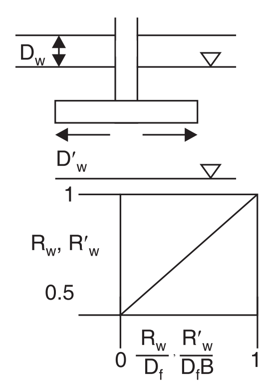
$$q_{nu} = CN_C + D_f r_t\,(N_q - 1)\,R_w + 0.5\,Br_t\,N_r\,R_w'$$

$$R_w = 0.5\left(1 + \frac{D_w}{D_f}\right)$$

when $\qquad 0 < \dfrac{D_w}{B} \leq 1$

$$R_w' = 0.5\left(1 + \frac{D_w'}{B}\right)$$

when $\qquad 0 < \dfrac{D_w'}{B} < 1$



6. **Skempten's Method** (C-soil)

⇒ Applicable only for saturated clay

⇒ Applicable to deep foundations also.

$$q_{nu} = CN_C$$

'C' can be found out from

(*i*) U–U test

(*ii*) Uncombined compression test

(*iii*) Vane shear test

For value of NC

(*a*) Strip footing

$$N_C = 5\left(1 + 0.2\frac{D_f}{B}\right), \frac{D_f}{B} < 2.5 \qquad \text{for } \frac{D_f}{B} \geq 2.5 \quad N_C = 7.5$$

(*b*) Source/circular footing

$$N_C = 6\left(1 + 0.2\frac{D_f}{B}\right), \frac{D_f}{B} < 2.5 \qquad \text{For } \frac{D_f}{B} \geq 2.5 \quad N_C = 9$$

(*c*) For rectangular footing

$$N_C = 5\left(1 + 0.2\frac{D_f}{B}\right)\left(1 + 0.2\frac{B}{L}\right), \frac{D_f}{B} < 2.5$$

For $\frac{D_f}{B} \geq 2.5 \quad NC = 7.5\left(1 + \frac{0.2\,B}{L}\right)$

7. **Meyerhoff's Method** (C-ϕ soil)

   Applicable for both shallow as well as deep foundation.

Note: Failure surface is assumed to go above the foundation level.

$$q_u = CN_C\,S_C\,d_C\,i_C + qN_q\,S_q\,d_q\,i_q + 0.5\,B\gamma\,N_\gamma\,S_\gamma\,d_\gamma\,i_\gamma$$

where S, *d*, *i* are emperical correlation factors for shape, depth, inclination. For ϕ-soil $N_C = 5.14$ $N_q = 1$ $N_\gamma = 0$

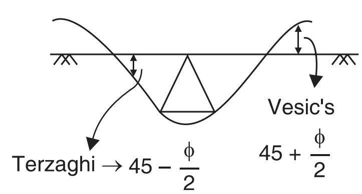
8. **Hensen's Recommenda-tions** (ϕ-soil)

$$q_u = CN_C S_C\,d_C + qN_q\,S_q\,d_q\,i_q + 0.5\,B_\gamma\,N_\gamma\,S_\gamma\,d_\gamma\,i_\gamma$$

For $\phi = 0$; $\quad q_{nu} = CN_C\,(1 + S_C + d_C - i_C)$

9. **Vesic's Bearing Capacity:**

   Failure surface assumed by Vesic is similar to Terzaghi but the angle of inclination of failure surface with the horizontal is $45 + \frac{\phi}{2}$ rather than $45 - \frac{\phi}{2}$ as given by Terzaghi.



10. **I.S. Code Method** (IS 6403–1981):

$q_{nu} = CN_C\,(S_C d_C i_C) + q(N_q - 1)\,(S_q d_q i_q) + 0.5B\gamma N\gamma\,(S_\gamma d_\gamma i_\gamma)W'$

$N_C, N_q, N_\gamma \rightarrow$ From Vedic's equation

$W' \rightarrow$ water table correction factor

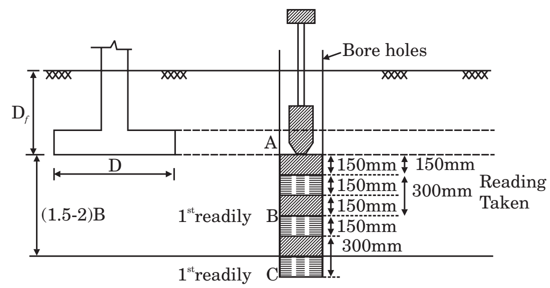
⇒ $W' = 1$ if water table below $D_f + B$

⇒ $W' = 0.5$ if water table at D_f

⇒ Interpolation if water table between D_f and $D_f + B$

Field Tests

1. **Standard Penetration Test**



⇒ For Granular soils only

⇒ N-value is determined at selected number of bore holes and avg. value of corrected N is calculated for the depth from $D_f + (1.5 - 2)B$.

⇒ Any value greater than 50% of the avg. value is discarded and new avg. value is found out.

⇒

N	ϕ	Relative Density
< 4	25-30	0
4-10	27-32	15

⇒ Over burden Correction

$$N_1 = N_0\left(\frac{350}{\bar{\sigma} + 70}\right)$$

N_0 = observed SPT value

N_1 = Corrected N value of overburden

$\bar{\sigma} \le 280$ KN/m² So $N_1 > N_0$

⇒ Dilatancy correction required only in saturated fine sand or silt

⇒ More significant in case of fine dense sand ($N_1 > 15$) as it has tendency to dilate under rapid loading and –ve pore pressure will develop.

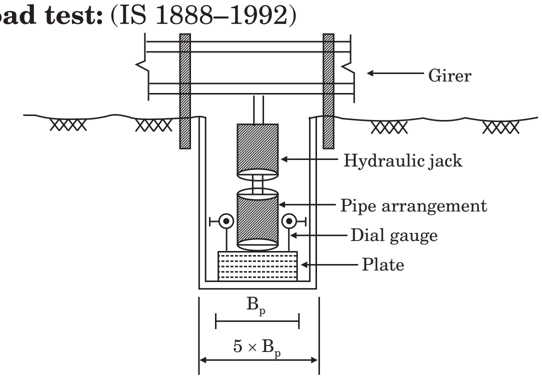
$$N_2 = 15 + \frac{1}{2}(N_1 - 15)$$

⇒ **Teng's Formula** for q_{nu} in granular soil.

$$q_{nu} = \frac{1}{6}(3N^2 BR_w' + 5(100 + N^2)\, D_f R_w). \text{ Continuous or strip footing}$$

$$q_{nu} = \frac{1}{3}\,(N^2 BR_w' + 3(100 + N^2)\, D_f R_w) \text{ Square or circular footing}$$

2. **Plate load test:** (IS 1888–1992)



It is used to calculate

(*a*) Ultimate bearing capacity
(*b*) Allowable bearing capacity
(*c*) Safe settlement of foundation

⇒ Significant only for cohesionless soil

For Clayey soil $\quad q_{uf} = q_{up}$

$$\frac{S_f}{S_P} = \frac{B_f}{B_P}$$

For Granular Soil

$$\frac{q_{uf}}{q_{up}} = \frac{B_f}{B_P}$$

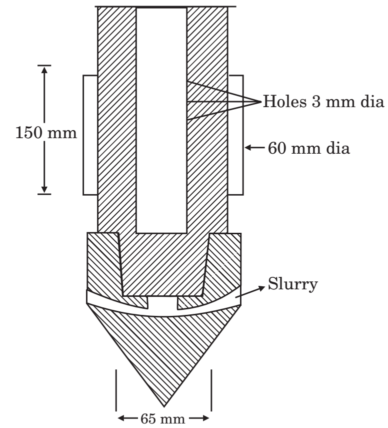
$$\frac{S_f}{S_P} = \left[\frac{B_f}{B_P}\left(\frac{B_P + 0.3}{B_f + 0.3}\right)\right]^2 \quad B_f, B_p \text{ in metres}$$

Depth Correction

$$S_f\,(\text{correction}) = S_f\left(\frac{1}{1 + \dfrac{D_2}{B_f}}\right)^{0.5}$$

D_2 = depth of foundation from the level at which plate load test is carried out.

3. **State cone penetration test** (CPT):
   ⇒ Particularly for soft clays, silts and fine to medium sand deposits.
   ⇒ Continuous record of soil resistance
   ⇒ Cone area 10 cm^2, Apex angle 60°
   ⇒ Rate of pushing cone 20 mm/sec upto depth of 100 mm.



Gravels

Allowable Bearing Capacity ($Q_{A\ net}$)

It is the **minimum** of bearing strengths obtained from shear failure criteria and settlement criteria. Emperical relations for Q_A are:

1. **Peak Henson's Formula**

$$q_{a(\text{net})} = 0.44 \text{ NS } C_w$$

$$C_w = 0.5\left(1 + \frac{D_w}{D_f + B}\right)$$

N = corrected SPT N-value
S = Permissible settlement (mm)
C_w = Water table correction factor
D_w = depth of WT below ground

2. Teng's formula:

$$q_{a(\text{net})} = 1.4(\text{N-3})\left(\frac{B + 0.3}{2B}\right)^2 . \text{S}.C_w . C_D$$

C_D = depth correction factor

$$C_w = 0.5\left(1 + \frac{D_w}{B}\right)$$

$$C_D = \left(1 + \frac{D_f}{B}\right) \le 2$$

3. **I.S. Method for Raft:**

$$q_{\text{net}}(\text{safe}) = 0.88 \text{ N S } C_w$$

C_w → from Peek henson's

Settlement of foundation:

$$S = S_{immidiate} + S_{1^\circ} + S_{2^\circ}$$

Where $\quad S_{immediate} = \dfrac{q_n B(1-\mu^2)}{E_S} \times I_f$

$S_{immediate}$ $\Rightarrow$ Elastic settlement for both sandy and clayey soil

μ = Poisson's ratio

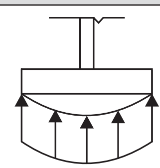
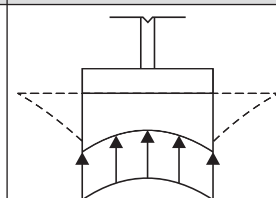
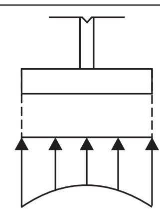
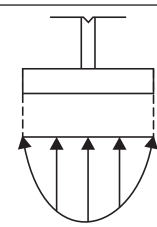
E_s = modular of elasticity

I_f = Influence factor = f(shape, rigidity of structure)

E_S can be calculated from Triaxial tests or field tests.

$$\underset{\text{immediate}}{S_{\text{rigid}}} = \underset{\text{immediate}}{S_{\text{flexible}}} \times 0.8$$

Deflected Shape of Footings

	Clayey Soil	Granular soil
Fexible Footing Settlement varies Pressure constant		
Rigid Footing Settlement constant Pressure varies		

Permissible Settlements in Shallow Foundation

	Total settlement (mm)	Differential Settlement (mm)	Angular distortion (mm)
Isolated footing on clay	65	0.0015L	1/666
Isolated footing on sand	40	0.0015L	1/666
Raft on clay	65-100	0.0021L	1/500
Raft on sand	40-65	0.0021L	1/500

17 Deep Foundation

When the soil at shallow depth is poor and the load to be transmitted is heavy then the depth of the foundation has to be increased till the suitable soil strata is met, such foundation are called deep foundations.

Pile: Small dia shaft which is driven or bored into ground.

Piers and wells: large diameter shafts constructed by excavation and sunk to required depth.

Classification of pile

- Material
  - Timber
  - Steel
  - Concrete
  - Composite
- Mode of load transfer
  - End bearing
  - Friction
  - Combined
- Function or action
  - Load bearing
  - Tension or uplift
  - Compaction
  - Fender piles
  - Sheet piles
  - Batter piles
- Installation method
  - Driven
  - Bored, cast in site
  - Jack piles
  - Screw piles
- Displacement of soil
  - Displacement pile
  - Non-displacement pile

End bearing piles: Used in stiff clay. Dense sand.

Friction pile: Used in soft soil, clay.

Tension or uplift piles: anchor structures subjected to **hydrostatic pressure or overturning moment.**

Compaction pile: compact loose granular soil

Anchor pile: Anchor against **horizontal pull** from water or sheet piling.

Fender pile: protect water-front structures against **impact from ships** and other floating objects.

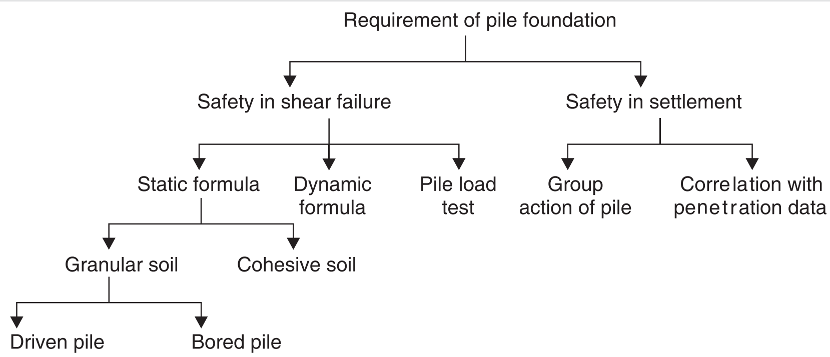
Sheet piles: Used as cut-offs to reduce seepage and uplift in hydraulic structure.

Batter piles: Resist horizontal and inclined forces, especially in water front structure.

Displacement piles: It displaces the pile laterally or upward. In loose sand, a pile densifies the sand upto a distance of 3.5 times the diameter of pile measured from the centre of pile. In case of clays, large displacement piles remould the soil to a distance of 2 times the diameter of pile.

Non-displacement piles: Are bored piles. Such piles are formed in the soil by boring or excavation and then these voids are filled with concrete sides are either supported by casing or by Bentonite slurry.

Note: Driven concrete piles are generally of diameter upto 500 mm. But bored piles may be even 2-3 m.



1. Static formula

$$Q_u = Q_{pu} + Q_f$$

Q_u = Ultimate load

Q_{pu} = Ultimate point load

Q_f = Ultimate skin friction.

$$Q_{pu} = q_{pu} \,.\, A_b$$

$$Q_f = Fs \,.\, A_s$$

q_{pu} = Unit point bearing resistance

A_b = Area of base

F_s = Unit skin friction resistance

A_s = Surface area of pile in contact with soil.

So $Q_u = q_{pu} A_b + F_s A_s$

For $C - \phi$ soil $q_{pu} = CN_c + \gamma D_F N_q + 0.5 B\gamma N\gamma$

Neglect $0.5 B\gamma N\gamma$ wrt. γD_f as $B <<< Df$

Hence, $q_{pu} = CN_c + \gamma D_f N_q$

for C – soil $q_{pu} = CN_c$

for ϕ – soil $q_{pu} = \gamma D_f N_q$

(*a*) static formula in driven granular soil.

$$Q_u = (\gamma D_f N_q) A_b + F_s A_s$$

where $F_s = K \sigma_{aug} \tan \delta$

For loose sand to from loose to medium sand D_f = 15 (dia of pile)

For dense sand D_f = 20 (dia of pile)

For values of K and S

Pile mateual	δ	Value of K	
		Loose	Dense
Steel	20°	0.5	1
Concrete	3/4 ϕ	1	2
Timber	2/3 ϕ	1.5	4.0

Note: As per IS code δ = ϕ and K = 1 – 3 for driven piles in loose to medium send

When there are several layers of soil then

$$Q_p = K \tan \delta (\sigma_{avg1}. A_1 + \sigma_{avg2} A_2 + --- + \sigma_{avgn} A_n)$$

(*b*) Static formula in bored granular soil.

Point bearing of bored cast in situ piles = $\frac{1}{2}$ × point bearing resistance of driven piles.

Due to boring, the value of K becomes very small and taken as 0.5. Rest all calculations as above.

Note: Maximum upper limit of frictional stress (f_s) is = 110 KN/m²

Maximum upper limit of point bearing stress.

(*a*) 11000 KN/m² → silica sand

(*b*) 5000 KN/m² → calcarious sand

(*c*) Static formula for piles in clay.

$$Q_u = (C_{ub} N_c) A_b + (\alpha C_u) A_s$$

C_{ub} = Undrained cohesion at the base of the pile

C_u = Undrained cohesion in the embedded length of pile.

N_c = 9 (By **skempton's**).

α = Depends on adhesion between soil and pile called adhesion factor

α = 0.1 → for very loose clays

α = 0.3 → for very stiff clays

Note: Smaller the undrained strength, softer is the consistency and greater is the tendency to adhere to the pile.

2. **Dynamic formula:** It is based on resistance to penetration hence used in driven piles only

Energy Imparted = Work done in pile driving

$$Q_u \times S = W \times H$$

(*a*) Engineering News Formula

$$Q_{allowable} = \frac{WH}{FOS\,(S + C)}$$

W = load in Kg

H = Height of fall in cm

S = Settlement/blow in cm

= It is corresponding to last 5-blows of drop hammer also called, as real set per blow.

= last 20 blows of steam hammer.

C = Emperical factor

= 2.5 cm for drop hammer

= 0.25 cm for single acting steam hammer.

FOS = 6

Another form: $Q_{KN} = \dfrac{166.64\, E_{KJ}}{S + 2.54}$

S = Settlement/Avg penetration for last 100 mm of driving per blow. Minimum permissible value of S = 1.25 mm.

(*b*) Modified Hilly Formula:

Ultimate Driving Resistance (R) = $\dfrac{Wh\eta}{S + \dfrac{C}{2}}$

W = Weight of hammer (Tonnes)

h = Ht of fall (cm)

S = Final set per blow (last– one – cm)

C = Total elastic compression per blow ie of soil + pile + Dolly

η = Efficiency of blow.

3. Pile load test: (IS 2911) It is the only direct method for determining the allowable loads on the piles and, is considered to be most reliable as it is an insitu test.

Initial
To asses the allowable load or to cheek the settement at working load

Routine test
On working piles for the assesment of settement under working load

Note: As per IS code, for more than 200 piles there should be a minimum of two initial test where as routine test is done on 0.5% to 2% of total number of piles.

Type of pile load test

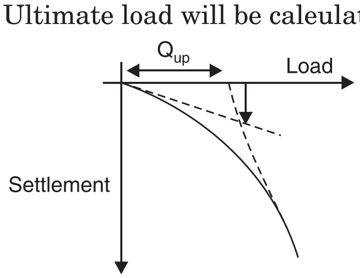
- Vertical load/ compression test
- Lateral load test
- Pull out test
- Constant rate of penetration test
- Cyclic load test

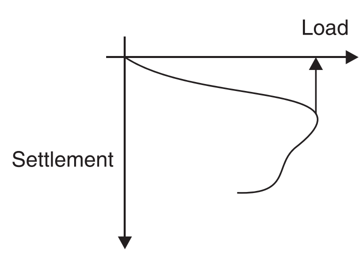
Cyclic load: test is carried out to determine **skin friction and end bearing seperately** for a pile load on a single pile. It is generally an initial test.

Note: Test pile is a pile which is especially bored for the purpose of conducting test and will not be the part of foundation in future.

While working pile is a pile which is a part of foundation and is being used for the purpose of testing at present.

Ultimate load will be caleulated from the load settlement curve





Allowable load on single pile: Will be the minimum of the following cases.

(*i*) 50% of the ultimate load at which total settlement is equal to the 1/10 of the pile diameter.

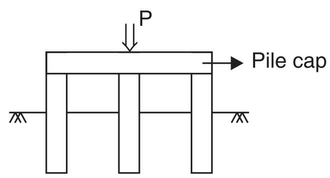
(*ii*) 2/3rd of the load at which total settlement is 12 mm.

(*iii*) 2/3rd of the load at which net settlement is 6 mm (total settlement – elastic settlement)

Group Action of Piles

When piles are driven there is uncertainty regarding vertical installation of piles.

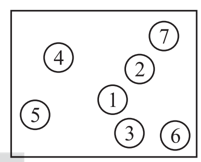
Hence minimum number of piles driven are 3. where in case of bored piles single pile can also be used.



Free standing pile used in expansive soil

Note: To void tightening of ground, pile in sand should begin at centre and then moved out-ward. (numbering in while piles to be driven)

$$\text{Group efficiency } \eta = \frac{Q_{ug}}{n\,Q_u}$$



Q_{ug} = ultimate load capacity of pile group
Q_u = ultimate load capacity of single pile
n = No. of piles.

Note: Minimum spacing between piles according to I.S code

For loose sand or back filled soil = 2 × Diameter

For point bearing piles = 2.5 × Diameter

For friction piles = 3 × Diameter.

In case of non-circular piles, diameter of the circumscribed circle is taken as diameter

Ultimate bearing capacity of pile group.

Clay

Block failure occurs when spacing < (2 to 3)D

$Q_{ug} = C_{ub} N_c A_b + \alpha C_u A_s$

A_s = Area of base of block

A_s = Perimeter of block × pile length

Individual pile failure occurs when spacing S > 8 D

$Q_{ug} = n\,Q_u$

n = no, of piles

Q_u = ultimate load capacity of single pile.

Sand

Group efficiency of driven piles in loose or medium sand is greater than 1 due to compaction of soil from vibration. Where as in dense sand efficiency can become less than 1. Hence for design purpose we take $\eta = 1$

So $Q_{uq} = n\,Q_u$ In sands.

Safe load capacity

$$Q_s = \left[\frac{\text{minimum of } [Q_{ug}, n\,Q_u]}{\text{F.O.S}}\right]$$

Converse labarre pile group efficiency

$$\eta_g = 1 - \frac{\theta}{90}\left[\frac{(n-1)\,m + (m-1)\,n}{mn}\right]$$

$$\tan\theta = \frac{d}{s}$$

d = diameter of pile

s = centre to centre spacing.

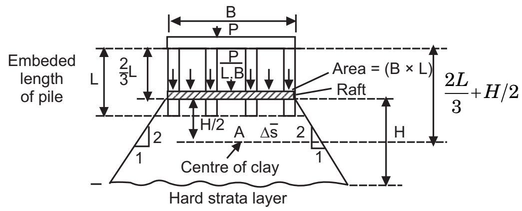
m = no. of rows

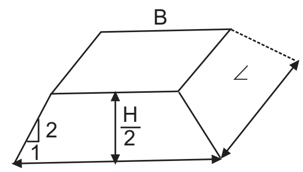
n = no. of piles in a row.

Settlement of pile groups

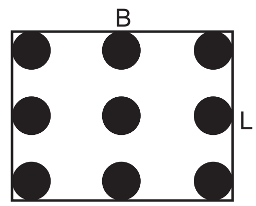
As the zone of influence of pile group is generally more than the individual pile so settlement of pile group is generally greater than the settlement of individual pile to same loading. (ie same load per pile)

1. Settlement of pile group in clays





(*a*) When pile is in uniform clay deposit



$$\Delta H = \frac{C_C}{1+e_o} H \log\left(\frac{\bar{\sigma}_o + \Delta\bar{\sigma}}{\bar{\sigma}_o}\right)$$

$$\bar{\sigma}_o = \left(\frac{2}{3}L + \frac{H}{2}\right)\gamma_t$$

$$\Delta\bar{\sigma}_o = \frac{P}{\left(B + \frac{H}{2}\right)\left(L + \frac{H}{2}\right)}$$

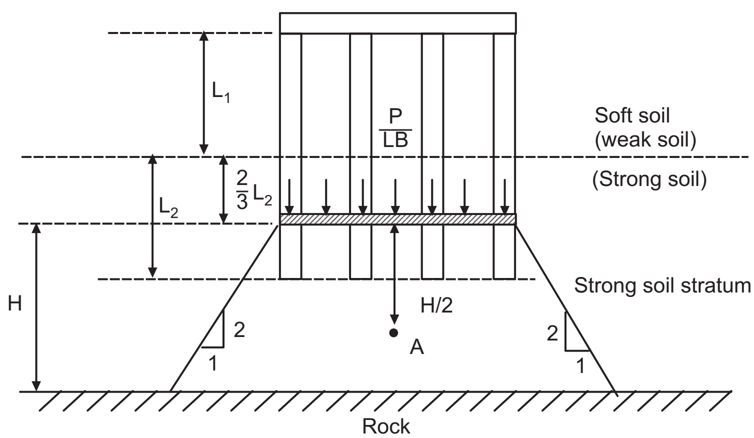
(*b*) When piles are driven into strong stratum through an overlaying weak stratum

Here depth H is measured from $\frac{2}{3}$ L_2 to the bottom solid surface

where L_2 is the depth of embedment in strong soil.

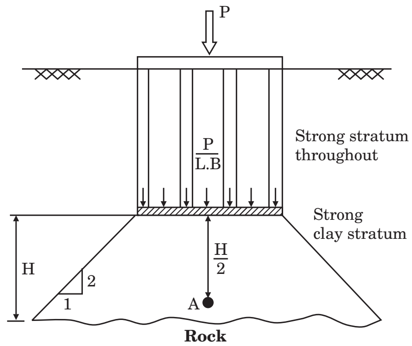
$$\Delta H = \frac{C_C}{1+e_o} H \log\left(\frac{\bar{\sigma}_o + \Delta\bar{\sigma}}{\bar{\sigma}_o}\right)$$

where $\bar{\sigma}_o$ is measured at point A (mid depth of H)



(*c*) In case of bored piles or end bearing piles resting on firm stratum

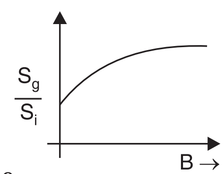
Here H is measured from the bottom of piles to the bottom hard strata



$$\Delta H = \frac{C_C}{1+e_o} H \log\left(\frac{\bar{\sigma}_o + \Delta\bar{\sigma}}{\bar{\sigma}_o}\right)$$

$\bar{\sigma}_o$ = effective stres at point A

2. Settlement of pile groups in sand



$$\text{Group settlement ratio} = \frac{S_g}{S_i} = \left(\frac{4B + 2.7}{B + 3.6}\right)^2$$

S_g = Group settlement at the same load of pile group.

B = Size of pile group in meter.

Si = Settlement of individual pile calculated from the pile load test