SOIL MECHANICS

Self confidence | Achieve Daily Target | Multiple revision

To The Point-ByDhyanPal(ESE'17AIR-179,GATE'18AIR-93,GATE'16AIR-145)

① Soil :- unconsolidated material, composed of Solid particle produced by disintegration of rocks.

- void space b/w particles may contain air, water, both.
- Soil particle may contain organic matter.

② Soil mechanics - Term given by Terzaghi in 1925

father of soil mechanics → Terzaghi

As per Terzaghi :- soil mechanics →

- Application of law of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulation of solid particles produced by mechanical & chemical disintegration of rock, regardless of whether or not they contain an admixture of organic constituent.

General Definition of soil mechanics :- Branch of mechanics deals with action of forces on soil and with the flow of water in soil

③ Soil engineering :- applied science dealing with application of principle of soil mechanics to practical Problem.

- It includes site investigation, design, construction of foundation, earth retaining structures.

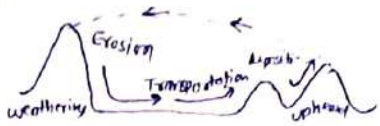
④ Geotechnical engineering :- includes → (soil engineering + rock engineering + Geology)

Sometime Geotech engineering used synonymously with soil engineering.

Geological cycle for formation of soil

Imp

weathering (erosion phase) → transportation → deposition → upheaval



- Exposed rock are eroded & degraded by various physical & chemical process.
- The product of erosion are picked by agencies of transportation (water, wind) and deposited to new location.

Residual Soil (Sedimentary soil)	Transported soil
if the soil stays at the place of its formation just above the parent rock	when the soil has been deposited at a place away from the place of its origin.

note :- Residual soil → has better Engineering property than transported soil

As per transporting Agency (soil classification) :-

① Alluvial deposit	• deposited by river water (running water) • consist of Alternate layer of Sand - silt - clay • low density • liable to liquifaction in earthquake prone areas • found in large part of north India
② lacustrine deposited	• deposited by still water like lakes.
③ marine deposit note :- marine clay → soft & highly plastic	• deposited by sea water (when flowing water carries soil to ocean or sea) • contain large amount of OM • low shear strength, highly compressible • found mainly confined along narrow belt near the coast. (Southwest coast of India)

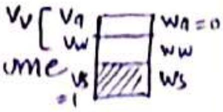
④ Aeolin deposit	• transported by wind. Ex. loess :- • wind blown deposit of silt • formed in arid & semiarid region. • low density • High compressibility. • low bearing capacity • permeability in the vertical direction is large
⑤ Glacial deposit	• Transported by ice. note :- Drift → general Term used for deposits made by glacier directly or indirectly. Ex. Till :- unstratified deposit made by melting of glacier (also known as Boulder-clay)
⑥ gravity deposit	• deposited under action of gravity Ex. colluvial soil (such as Talus)

Some other soils :-

① Black cotton soil	• residual deposits formed from Basalt or trap rocks. • found in large part of central India and a portion of south india. • has High plasticity • must contain montmorillonite minerals • {∴ High shrinkage & High swelling} high compressible. • shear strength → low • Low bearing capacity • in such soil use underream pile.
② lateritic soil	• formed by leaching (removal of Bases & silica and accumulation of iron oxide & Aluminium oxide) note :- due to ferro oxide color of lateritic soil is red or pink color.

③ Desert Soil	Ex. Sand dunes :- (wind transported soil) ↓ uniform in gradation, relatively uniform particles of fine to medium sand.
④ Bentonite soil	• chemically weathered volcanic ash • type of clay having high % of montomorilonite mineral • High plastic clay • High water absorbent
⑤ Calcareous soil	contain large qnt. of calcium carbonate ($CaCO_3$)
⑥ Humus	• dark brown, organic amorphous earth of the top soil • consist of partly decomposed vegetal matter. • not fit for engineering work
⑦ loam	sand + silt + clay
⑧ marl	• Calcareous soil of marine origin. • greenish color
⑨ Peat	• organic soil having fibrous aggregates of macroscopic & microscopic particle • formed from vegetal matter in excess-moisture such as in (swamps) • High compressible • not fit for foundation
⑩ muck	• mixture of fine soil particle and Highly decomposed organic matter • organic matter is in advance stage of decomposition.
⑪ Tuff	fine grain soil ejected from volcanos during its explosion and deposited by wind / water
⑫ varved clay	• deposit consist of Alternate thin layers of silt and clay. • results of deposition in lake during Period of Alternate high & low water.
⑬ Kaolin	white clay pure form
⑭ Indurated clay	Hardening of clay due to heat & pressure.

Soil water relationship :-

unit phase diagram	$\gamma_s = 1$ unit solid volume 
water content	$\omega = \frac{W_w}{W_s}$; $w \geq 0$ (no upper limit) ; $W_s = \frac{W}{1+\omega}$ note:- water content in soil represent → Gravity water + capillary water + hygroscopic water • This water can be removed on oven drying except structural water.
void ratio	$e = \frac{V_v}{V_s}$; $e \neq 0$ ∵ $V_v \neq 0$; $e > 0$ $V_s = \frac{V}{1+e}$ ∵ V_s → constant hence e → more used.
Porosity	$n = \frac{V_v}{V}$; $0 < n < 100\%$; $n = \frac{e}{1+e}$; $e = \frac{n}{1-n}$
degree of saturation	$S = \frac{V_w}{V_v}$; $(0 \leq S < 100\%)$; $Se = wG$
air content	$a_c = 1 - S = \frac{V_a}{V_v}$
Percentage air void	$n_a = n a_c = \frac{V_a}{V}$

S	soil
0-0.25	Humid
0.25-0.50	damp
0.50-0.75	moist
0.75-1	wet

unit weight

$\gamma_s > \gamma_{sat} > \gamma_{bulk} > \gamma_d > \gamma_{sub}$

• unit wt of solids or absolute unit wt: $\gamma_s = \frac{W_s}{V_s}$

• unit wt of water: $\gamma_w = \frac{W_w}{V_w}$

• dry unit wt.: $\gamma_d = \frac{W_{dry}}{V} = \frac{W_s}{V}$

$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{(1-n_a)G\gamma_w}{1+wG} = \frac{\gamma_{bulk}}{1+\omega}$

• $\gamma_{bulk} = \frac{W}{V} = \frac{(G+Se)\gamma_w}{1+e} = \gamma_d + S(\gamma_{sat} - \gamma_d)$

• $\gamma_{sub} = \gamma_{sat} - \gamma_w = \frac{(G-1)\gamma_w}{1+e}$; $\gamma_{sub} \approx \frac{\gamma_{sat}}{2}$

↳ submerged unit weight or boyant unit wt

Specific gravity

Specific gravity of solid or True/absolute specific gravity: $G = \frac{\gamma_s}{\gamma_w}$

mass specific gravity or Apparent ": $G_m = \frac{\gamma_{bulk}}{\gamma_w}$

Inorganic solids $G = 2.6$ to 2.75

organic " $G = 1.2$ to 1.4

note:- more the specific gravity of stone, more heavier & strong ∴ used in construction. range 2.4 – 2.8 stone G

Relative density or density Index

• looseness or denseness of (coarse grain soil) only — Sand, gravel

$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{V_{max} - V}{V_{max} - V_{min}} = \frac{\frac{1}{\gamma_{d\,min}} - \frac{1}{\gamma_d}}{\frac{1}{\gamma_{d\,min}} - \frac{1}{\gamma_{d\,max}}}$

$I_D \propto$ denseness $\propto$ shear strength $\propto \frac{1}{\text{compressibility}}$

note:-

• when particle arrange in Cubical array (loose)

$e_{max} = 91\%$ → ∴ $n_{max} = 47.64$

Imp ↳ calculation experimentally by (dropping coarse grain soil from height)

• when particle arranged in Prismoidal array (Rhomohedral array)

$e_{min} = 35\%$ → ($n_{min} = 25.9\%$)

↳ calculation experimentally by vibration

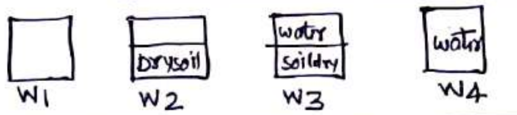
relative compaction

for (cohesive + cohesionless) soil tells denseness

$R_c = \frac{(\gamma_d)_{insitu}}{(\gamma_d)_{max}}$; $R_c\% = 80 + 0.2 I_D\%$

Specific gravity of soil solids (G) determination : Pycnometer method

Imp :- here dry soil is taken **



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

note:- ∵ removal of entrapped air is difficult for cohesive soil hence not suited for cohesive soil.

other methods for determination of specific gravity

1- Density Bottle method
2- measuring flask method
3- Gas Jar method
4- Shrinkage limit method

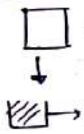
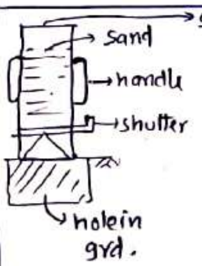
+ (5) Pycnometer method

{ Explained in shrinkage limit determination Topic }

water content Determination :-

① oven-drying method	• Standard laboratory method • takes 24 hrs approx. • gives accurate result Soil — Temp. range: Inorganic — 105–110°c organic — 60°c soil containing calcium component & zypsum — 80°c $w = \frac{W_W}{W_S}$
⑪ Pycnometer method note:- entrapair removal: gentle heating and rigrous shaking; vaccum pump	• used when specific gravity of solid (G) is known. • take wet soil $w = \left[\frac{W_2-W_1}{W_3-W_4}\left(\frac{G-1}{G}\right)-1\right]\times 100\%$ • suited for cohesionless soil $\left[G = \frac{\gamma_s}{\gamma_w} \text{ or } = \gamma_s/\gamma_{kerosine}\right]$
⑪ Calcium carbide method or rapid moisture meter method (lab+field)	• rapid Test / Quick (5–7 min) • water content determined indirectly from pressure of acetylene gas formed • Instrument used → moisture meter $CaC_2 + 2H_2O \rightleftharpoons C_2H_2\uparrow + Ca(OH)_2$
ⓘⓥ sand-bath method (field method)	• not very accurate • used where electric oven not available • soil sample put in container & dried by placing it in a sand bath which is heated on kerosene stove.
ⓥ Alcohol method	• quick Test in field. • alcohol mix → to increase evaporation rate. • not used for organic soil & containg calcium compound.
⑥ Torsion balance method (lab) (quick method)	• Instrument parts: Infrared lamp, torsion balance • Infrared radiation used for drying of soil • drying and weighing done simultaneously Imp note:- suit for soils which quickly reabsorb moisture after drying
⑦ Radiation method	• extremly useful in Insitu water content determination • Radioactive Isotopes used • loss of energy ∝ qnt of water present in soil • proper sheilding Precaution taken.

Determination of Bulk unit weight (γ) :-

(i) water balloon method	• volume of excavated pit is obtained by covering hole with plastic sheet and filled with water.
(ii) Radiation method	• quick
(iii) Submerged mass density method	
(iv) Core-cutter method (for cohesive soil)	 core cutter (W_1) ($V = 1000 cm^3$) → core cutter + soil $= W_2$ $\gamma = \frac{W}{V} = \frac{W_2 - W_1}{1000 cm^3 \text{(volume of soil)}}$ • take sample to get water content (w) • $\gamma_d = \gamma_{bulk} / 1 + w$
(v) sand replacement method (for hard & gravelly soil) basically (cohesionless soil)	 • excavate area • excavated soil sample (w) taken • a calibrated cylinder containing sand is placed over excavated area & pit is filled with sand. $\gamma_{bulk} = \frac{W}{V}$ & $\gamma_d = \frac{\gamma_{bulk}}{1+w}$
(vi) water displacement Method (for cohesive soil only) where it is possible to have a lump sample	• volume of specimen → by water displacement • sample wt $= W_1$ • sample wt + parafin wax $= W_2$ ∴ wt of wax $= W_2 - W_1$ • now put sample in water filled container $V_{water} = V_{wax} + V_{soil}$ $V_{water} = \frac{W_{wax}}{\gamma_{wax}} + V_{soil}$ $V_{water} = \frac{W_2 - W_1}{\gamma_{wax}} + V_{soil}$ → $V_{soil} = ?$ ∴ $\gamma_{bulk} = \frac{W_1}{V_{soil}}$

Index properties of soil :-

- Used for Identification of soil then for - Classification of soil
- Index properties include indices which helps in determining engineering behaviour.
  { strength, load bearing capacity, swelling, shrinkage, settlement

• type of soil	Index property
coarse grain soil	Particle size, grain shape, relative density
fine grain soil	consistency limit

note:- Grain shape → partially used in coarse-grain soil.

in case of sand, gravel	Bulky grain
Submicroscopic crystal of clay mineral	flaky grain
clay mineral (kaolinite)	needle shape grain

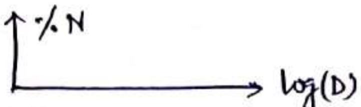
classification of Bulky grain is done on basis of

Sphericity (S) ⇒ $\frac{\text{dia. of equivalent spherical particle} (D_e)}{\text{length of particle} (L)}$

$V = \frac{4}{3}\pi \left(\frac{D_e}{2}\right)^3$

Particle size analysis / grain size analysis or mechanical analysis :-

aim →



(Semilog graph)

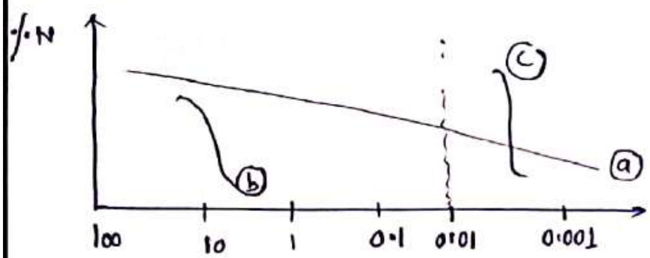
→ to determine gradation & uniformity of soil

- knowledge helps in construction of dam (earth), embankment fillers.

Particle size analysis

- for coarse grain soil → seieve analysis
  - coarse sieve analysis → (for gravels) 4 sieve used (4.75, 10, 20, 80) mm
  - fine sieve analysis (7 sieve used) → (for sand)
    - dry sieving → (when fines are less)
    - wet sieving → { • when fines are more • for cohesive granular soil }
- for fine grain soil → sedimentation analysis
  - Hydrometer method
  - Pipette method

Grain size distribution curve :-



a → well graded soil :- all size of particles present. (gradation curve → smooth)

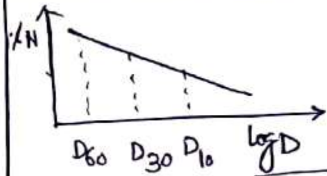
b → poorly/uniformly graded coarse soil } one type of soil particles are more

c → ——————————— fine soil } some particles has deficiency

gap graded soil :- some particles are missing.

grd note

- Position of curve — indicate → type of soil (coarse, fine)
- Shape of curve — indicate → gradation (well, poor)



coff. of uniformity $C_u = \frac{D_{60}}{D_{10}}$

coff. of curvature or coff. of gradation $C_c = \frac{D_{30}^2}{D_{60} \times D_{10}}$

• well graded soil	$1 < C_c < 3$ $C_u > 6$ sand $C_u > 6$ sand
	$C_u = 1$ uniformly graded soil

Sedimentation analysis :-

As per stoke's law

Terminal velocity $V_s = \frac{(G-1)\gamma_w d^2}{18\mu}$ $d = 0.2$ to 0.2μ mm

{ if $d < 0.2\mu$ → brownian motion
$d > 0.2$ mm → turbulence produced }

limitation of stoke's law :-

(I) particle size . assumed spherical* with equivalent diameter.

(II) stokes law Applicable to sphere falling freely without any interference in an infinite liquid medium
{Actual ∄ hindrance & depth of jar limited}

(III) All the soil grain may not have same G. (However avg. value is considered right)

Pretreatment	Post treatment
(to remove organic compound & calcium compound)	• to avoid (flocks)
for organic compound → H_2O_2 (oxidising agent) for calcium compound → $\frac{N}{5}$ HCl	dispersing agent (I) Sodium silicate (II) Sodium oxlate (III) Sodium hexameta phosphate

① Pipette method :-

- standard lab Test for particle size analysis of finegrain soil.
- very accurate
- require sensitive balance

v.imp

> wt. of solids per cc of suspension is determined directly by collecting 10 cc of soil suspension from a depth He (fix)

$$\%N = \frac{\frac{m_d}{V_p} - \frac{m'}{V}}{\frac{M_d}{V}}$$

- m_d → wt. of solids collected from suspension
- m' → wt of dispersing agent
- M_d → original wt of solids added in suspension

$$\frac{H_e}{t} = V_s = \frac{(G-1)\gamma_w d^2}{18\mu} \qquad d = ?$$

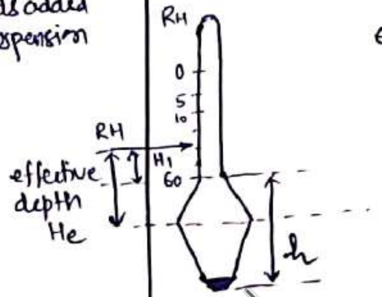
② Hydrometer method :-

v.imp

> wt of solids present at any time is calculated indirectly by reading the density of soil suspension.

v.imp

> Hydrometer → Instrument used for determination of specific gravity of liquid.
>
> - as the specific gravity of soil suspension depends on particle size thus hydrometer can be used for particle size analysis



effective depth

$$H_e = H_1 + \frac{1}{2}\left(h - \frac{V_H}{A_j}\right)$$

(h → length of bulb, V_H → volume of bulb, A_j → Jar c/s area)

$$G_{ss} = 1 + \frac{R_H}{1000}$$

soil suspension specific gravity

$$R_c = R_H \pm C \qquad C = C_m \pm C_t - C_d$$

$$\%N = \left(\frac{G_s}{G_s - 1}\right) \times \frac{R_c}{W_s} \times 100$$

(W_s → wt in gm per 1000 cc)

correction to hydrometer Reading :- *

(I) Hydrometer correction → always (+)ve

(∵ marking on stem increase downward)

(II) Temperature correction

- generally hydrometer calibrated at 27°c

T > 27°c
∵ suspension lighter
∴ actual reading will be less than corrected reading *
hence C_t = (+)ve

T < 27°c
C_t = (–ve) *

(III) Dispersion agent correction / Deflocculating Agent correction

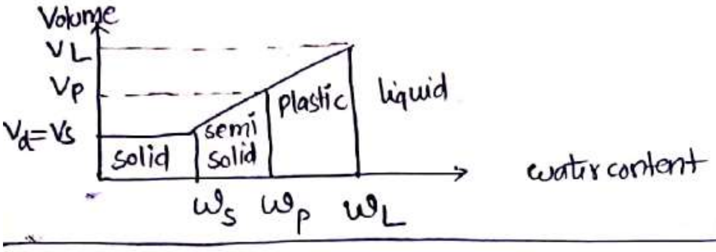
∵ Addition of agent to soil sample cause an increase in specific gravity of suspension.

hence C_d = (–)ve always *

consistency limit :- (mainly for clay)

{related to water content, How with change in water content consistency of soil changes}

consistency :- represent the relative ease with which soil can be deformed.



① **Liquid limit (w_L)** :- min water content at which soil has tendency to flow → liq. limit water content

- all soils at liq. limit
  shear strength = 2.7 kN/m² (which is negligible)

liq. limit determination
- Cassagrande's Tool
- Cone penetration method

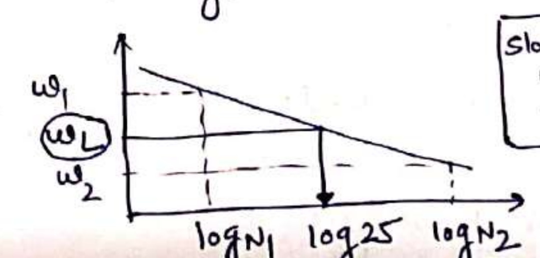
ⓐ **Casagrande Tool** :-

Imp 425μ

{120gm soil → airdried sample}

- mix with water, make uniform paste
- soil put in casagrande apparatus and groove of 2mm size is cut
- The no. of blows which is required to close 2mm grove over rubber pad is noted.

Imp
- The water content at which 25 blows closes the groove → liq. limit



Slope of this curve = Flow Index (I_F)

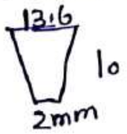
{Flow Curve}

$I_F = \frac{w_1 - w_2}{\log N_2 - \log N_1}$ (N=25...)

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

Imp note:- Flow index represent loss of shear strength of soil with increase in water content.

note:- ASTM tool is used for less plastic soil



ⓑ **cone penetration Test** :- water content at which penetration is 25mm → liq. limit

② **plastic limit w_p** :- → min water content at which soil is in plastic stage → plastic limit water content

- at plastic limit water content, a soil rolled into a thread of 3mm starts to crumble
  {take 425μ ⊖, airdried sample}

③ **Shrinkage limit (w_s)** :-

→ water content at which the soil stop shrinking further and attains a constant volume.

→ max. water content at which further reduction in water content does not cause reduction in the volume of soil sample

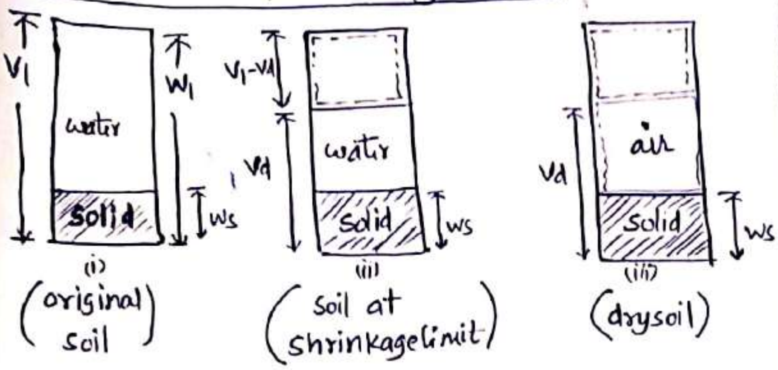
→ lowest (min) water content at which soil is fully saturated {means S=1 at shrinkage limit

$Se = w_s G$

$w_s = e/G$ *}

{Below shrinkage limit the soil does not remain saturated, Air enters the void of soil.}

Determination of shrinkage limit :-



note:- W_1, V_1, W_s, V_s or V_d → known experimentally

M-1 $\quad w_s = W_w / W_s$

at shrinkage limit W_w (in terms of W_1, V_1, W_s, V_s)

$$W_w = (W_1 - W_s) - (V_1 - V_d)\gamma_w$$

(from fig-2) ***

$$\therefore w_s \text{ (Shrinkage limit)} = \frac{(W_1 - W_s) - (V_1 - V_d)\gamma_w}{W_s}$$

imp.

note:-
Shrinkage limit Test → to get specific gravity of soil solids (G)

$$G = \frac{\gamma_s}{\gamma_w} = \frac{W_s}{V_s \gamma_w} = \frac{W_s}{\left[V_1 - \frac{(W_1 - W_s)}{\gamma_w}\right]\gamma_w}$$

fig-2

M-2 if specific gravity is known (G) →

$$w_s = \frac{W_w}{W_s} = \frac{(V_d - V_s)\gamma_w}{W_s} \quad \left\{ \because G = \frac{W_s}{V_s \gamma_w} \right.$$

(fig-2)

$$w_s = \frac{\left[V_d - \frac{W_s}{G\gamma_w}\right]\gamma_w}{W_s}$$

volumetric Shrinkage	$\frac{V_1 - V_d}{V_d} \times 100$ ($V_d \to V_2$)
Degree of shrinkage	$\frac{V_1 - V_2}{V_1} \times 100$

Shrinkage Ratio (R)	• volume change in soil above shrinkage limit expressed as % of dry soil per unit change in water above shrinkage limit $R = \frac{\left(\frac{V_1 - V_d}{V_d}\right)}{w_1 - w_s} = \frac{\gamma_d}{\gamma_w}$ → mass specific gravity at dry state.
Plasticity Index (I_p)	$I_p = w_L - w_p$ (if −ve then report as zero.) • range of consistency within which soil behaves as plastic material. • This property is due to presence of clay mineral

I_p	consistency.
0	nonplastic
<7	lowplastic
7 − 17	mediumplastic
>17	High plastic

note:- low plastic soil → easy to compact hence used in embankment

Shrinkage Index	$w_p - w_s$
consistency Index (I_c) or Relative consistency	$I_c = \frac{w_L - w}{I_p}$ (I_c↑ better foundation material)
liquidity Index (I_L)	$I_c + I_L = 1$ $\therefore I_L = \frac{w - w_p}{I_p}$
Toughness Index (I_t) ($0 \leq I_t \leq 3$)	$I_t = \frac{I_p}{I_f}$ → I_f → flow Index, $I_f = \frac{W_1 - W_2}{\log N_2 - \log N_1}$ I_t → measure of shear strength of soil at plastic limit

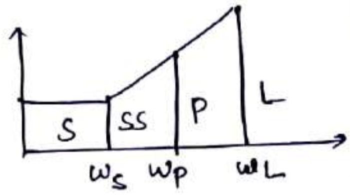
$I_t < 1$	friable soil (easily crushable at plastic limit)
$1 \leq I_t \leq 3$	for clay soil

Thixotropy	Property of soil due to which loss of shear strength after (on) remoulding can be regained if left undisturb for some time.
sensitivity (S)	degree of disturbance acheived on remoulding, is expressed as sensitivity. $S = \frac{q_{undisturbed}}{q_{remoulded}}$ Imp • water content should be same to use this formula note:- $S \leq 1$ ⇒ stiff clay having cracks & fissures
Activity (A_c)	Activity no. $= \frac{I_p}{\text{\% of clay particle } (<2\mu)}$ As per skempton → volume change during swelling & shrinkage ⇒ f^n { I_p, % clay fraction }

Ac	classification of soil
<0.75	Inactive
0.75-1.25	normal
>1.25	Active

(q_u) unconfined compressive strength	$q_u = 2C_u$ ↓ shear strength of clay under undrain condition

v.v. Imp
note:-



	soil A → soil B	soil A → soil B
	$W_L = W_L$ $I_p \downarrow \rightarrow I_p \uparrow$	$I_p = I_p$ $w_L \downarrow \rightarrow w_L \uparrow$
dry strength	increases	decreases
Permeability	decreases	increases
compressibility	same ∵ W_L → same	Increases
toughness I_p/I_f	increases	∵ $W_L \uparrow$ $I_p \uparrow$ ∴ Increases

Dhyan 27/3/2020

Field Identification of soil :-

1- visual examination
2- dilatency Test (Reaction to shaking)
3- toughness (consistency near plastic limit)
4- dry strength (crushing resistance)
5- organic content & color
6- others (acid Test, shine Test)

1- visual Examination :-

- particle > 80 mm removed
- soil (spread in plan)
  - > 50% visible → coarse grain soil
  - < 50% visible → fine grain soil

2- Dilatency Test (reaction to shaking) :-

Reaction → Appearance & disappearance of water with shaking & squeezing.

- Silt → exibit quick reaction
- clay → shows no or slow reaction

3- toughness (consistency near plastic limit) :-

- toughness → resistance to moulding at plastic limit.

- High organic clay (plastic soil)
  - exibit very weak & spongy feel at plastic limit.
- nonplastic soil (Imp.)
  - can not rolled into 3mm dia at any moisture content.

4- dry strength (crushing resistance) :-

if dry pat easily powdered	low dry strength
if fingre required	medium "
not powdered at all	High "

silty fine sand	low dry strength
organic silt	medium "
clay of High plasticity	high dry strength

(5) organic content & color :-

- fresh wet organic soil → (odour of decomposed organic matter) → this odour can be noticed after heating wet sample.
- dark color → ** indication of presence of organic matter of soil

other Test

- Acid Test → * Test for presence of $Ca(CO_3)_2$
  - → soil with high dry strength, a strong reaction indicates that strength may be due calcium carbonate as cementing agent rather than colloidal clay.
- shine Test → * Test for presence of clay
  - performed by cutting a lump of dry / slight moist soil with knife.
  - shine surface → (High plastic clay)
  - dull surface → silt or clay of low plasticity

Soil classification :

1- Unified soil classification system (USCS) (By casagrande)

- mainly for airfield construction.
  (later modified for foundation, dam, etc)

* • major soil group - 4

		Prefix	classification Basis
coarse grain	Gravel, sand	G, S	grain size
fine grain	silt, clay	M, C	Plasticity
organic		O	% of OM & particle of decomposed vegetation
Peat		Pt	–

note :- IS soil classification ⟶ modified version of USCS.

note :- if soil is highly organic & contains a large no. of % OM and particles of decomposed vegetation then it is kept in separate category ⟶ Peat (Pt)

coarse grain soil classification →

$C_u = D_{60}/D_{10}$

$C_c = D_{30}^2 / D_{60} \cdot D_{10}$

① % finer < 5%

(4.75 mm⁻)
- 50⁺ retain → Gravel (G)
- 50⁻ retain → sand (S)

note :- GW (well graded gravel) $1 < C_c < 3$, $C_u > 4$, otherwise GP

SW (well graded sand) $1 < C_c < 3$, $C_u > 6$, otherwise SP

② % finer > 12%

(4.75mm⁻)
- Gravel (G)
- sand (S)

again (2μ⁻)
- 50⁺ silt (M)
- 50⁻ clay (C)

GM :- M > C, $I_P < 4\%$ SM :- M > C, $I_P < 4\%$

GC :- C > M, $I_P > 7\%$ SC :- C > M, $I_P > 7\%$

note :- XYZ मतलब Z > Y > X

③ % finer (5–12%) & if $I_P \Rightarrow 4$ to 7 then dual symbol

Ex. GW–GC
- GW → gradation
- GC → type of fines

Possible case of dual symbol	GW GC	GP GC	SW SM	SP SM
	GW GM	GP GM	SW SC	SP SC

② AASHTO classification system (for Highway construction)

classification Basis
- Particle size
- Plasticity analysis

Group → 8
- A_1 to A_7
- A_8 (for peat & muck)

Soil within group divided into several sub group based on Group Index (GI)

$$GI = 0.2a + \left(\frac{0.2}{40}\right)ac + \left(\frac{0.2}{20}\right)bd$$

$a = P - 35$, $b = P - 15$ } ≯ 40

$c = W_L - 40$, $d = I_P - 10$ } ≯ 20

P ⇒ 75μ⁻ % finer

- GI ∝ 1/Quality of material
- GI → rounded off to nearest whole no.
- GI → if any terms in formula is (–)ve ⇒ make it zero calculation

③ Indian standard soil classification :-

Basic difference than USCS :- in USCS fine grain < low compressibility / High compressibility

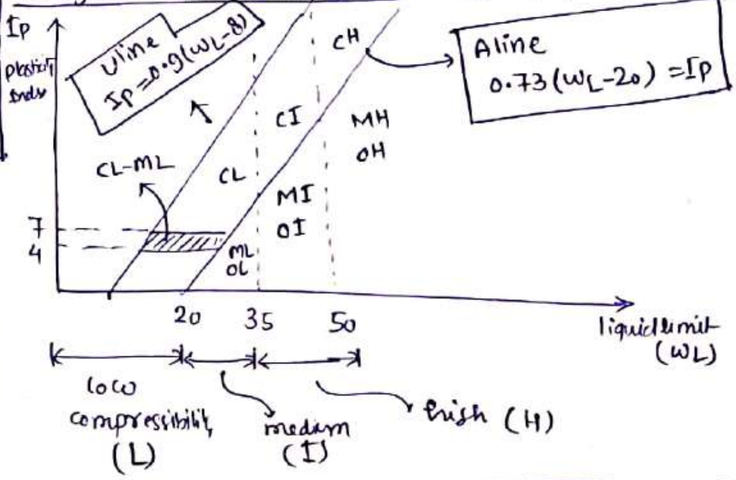
But in IS fine grain
- low
- medium
- High compressibility

Clay	Silt	Sand	Gravel	Cobble	Boulder
< 2μ	2–75 μ	75μ–4.75 mm	4.75–80 mm	80–300 mm	> 300 mm
fine grain soil	Coarse grained soil				

fine sand	75μ – 0.425 mm
medium	0.425 – 2 mm
coarse	2 – 4.75 mm

soil (75μ⁻)
- % finer > 50% (fine grain soil)
- % finer < 50% (coarse grain soil)

fine grain soil classification :- → (Plasticity chart) के आधार पर



- 0–35: low compressibility (L)
- 35–50: medium (I)
- > 50: high (H)

note :- dual symbol if w_L–I_P falls closer to A line.

Imp if liquid limit falls closer to 35%, 50%. use dual symbol.

if soil has equal coarse & fine grain then also use dual symbol

IS soil classification

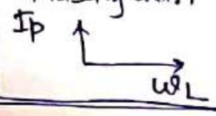
Imp For Coarse grain soil

Based on
- Particle size
- C_u, C_c
- % finer (P)

Well graded $1 < C_c < 3$
- sand $C_u > 6$
- gravel $C_u > 4$

Imp For Fine grain soil

Based on Plasticity chart

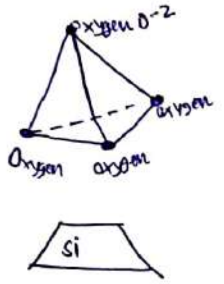


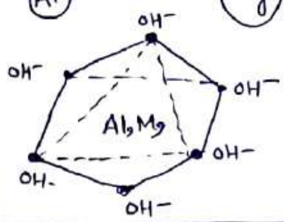
$C_u \simeq 1$ Poorly graded soil or uniformly graded soil

Atomic str. of clay mineral Built of 2 fundamental crystal sheets –

① tetrahedron sheet (silica sheet)
② octahedron sheet (Aluminium sheet)

Silica sheet	Aluminium sheet
• Oxygen atom → 4 (Placed at tip)	• OH^- hydroxyl ion → 6 (Placed at tip) → Gibbsite sheet (Al), Brucite sheet (Mg)
net charge on 1 unit ⇒ (-1)	(-2)





Isomorphous substitution :- in mineral lattice metal ion of one kind substituted by other metallic ion of lower value (but same physical size)

Ex. Si^{+4} is replaced by → Al^{+3} in tetrahedral units

note:- Quartz (sand particle are made of Quartz)
↳ sand is a loose granular material in which most common component of sand is (SiO_2) in the form of Quartz.
• Quartz is hard, insoluble in water & does not decompose easily from weathering Process.

v.v.v.Imp order

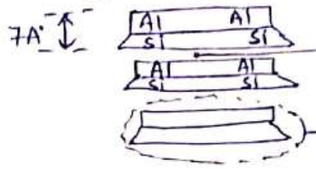
M > I > K	M < I < K
① activity ② surface area ③ dry strength ④ I_p (plasticity index) ⑤ Base exchange capacity	① size (grain size) ② Permeability ③ mineral Thickness,

Various clay minerals :-

① Kaolinite (1:1) Clay mineral → Platy shape

Ex. china clay

• 1:1 ⇒ 1 Silica sheet + 1 Gibbsite sheet (held by very strong hydrogen Bond)



• water cannot easily enter b/w str. unit and cannot cause swelling hence activity of kaolinite mineral is least.

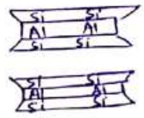
• commonly found in
- sedimentary soil
- residual soil
- old deposits
- High weathered soil with good drainage.

② montmorillonite (2:1) clay mineral (Smectite)

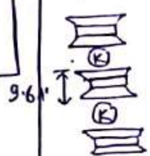
2:1 ⇒ 2 silica sheet + 1 Gibbsite sheet
(By water bond → weakest Bond
Thus max. change in volume due to moisture change.

Ex. Black cotton soil, Bentonite soil (weathered volcanic ash)

• montmorillonite has large specific surface amongst clay mineral.



③ Illite (2:1)



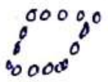
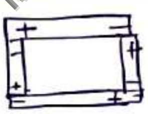
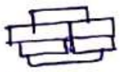
• In this found Ionic bond (K^+ bond) *
• Illite has substantial amount of Isomorphous substitution (Si^{+4} → Al^{+3})
Ex. stiff clay, in Lacustrine soft clay **

note:- Halloysite :- clay mineral of Kaolinite group.
↳ tubular shape
→ successive str. unit are separated by water molecule
→ when air dried they may convert to kaolinite
→ Halloysite & Kaolinite are used for making china ware

✓ • kaolinite clay is also used as an Intestinal absorbent in antidiarrhoeal medicine

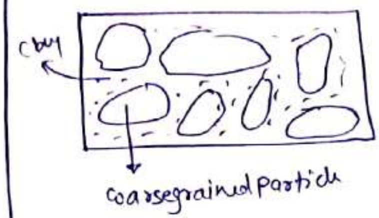
Soil structure :-

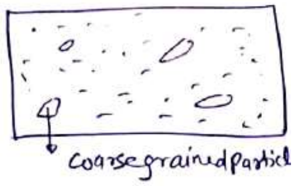
- Geometrical arrangement of soil particles in soil mass.
- generally soil str. is influenced by
  - in coarsegrain soil
    - Particle size distribution
    - Gravity forces
  - in clay soil
    - type of clay mineral
    - Surface force

① Single grained Structure	• in cohesionless soil (sand, gravel) • Particles contact by Gravitational force. • under shock & vibration they show little settlement as these get densified from loose state. Ex. marbles filled in Box.
② Honeycomb Structure 	• in fine sand (75μ–0.425mm) silt (75μ–2μ) • particle held by mutual attraction due to cohesion but they don't possess plasticity. • soils in Honey Comb str. in loose state • under shock & vibration str. collapses and show large deformation.
③ Flocculated Structure	• in clay soil  • attractive force (edge to edge) • str. light in weight • High void ratio • High moisture content • strong (can resist external force because of strong bond due to attraction b/w particles. • insensitive to vibration • low compressibility • High permeability • High shear strength.
④ Dispersed str.	• in clay that have been reworked /remoulded • Repulsive force (Face to face)  • particles more or less parallel orientation. • low shear strength • low permeability. High Compressibility

str. of composit soil

- Coarsegrained skeleton
- clay matrix str.





Permeability :-

ease with which water can flow any medium

Use of Permeability :-

(i) to calculate settlement of structure ($k = C_v m_v \gamma_w$)

(ii) to calculate yield of well $q = \frac{2.303 k \log(r_2/r_1)}{\pi(h_2^2 - h_1^2)}$

(iii) Seepage through & below earth str. calculation ($Q = kiA$)

(iv) k required in design of filter

(used to prevent piping in hydraulic sts. and subgrade drainage, rate of consolidation of compressible soil)

Imp.

Typical values of k → very Important.

Soil	K (cm/sec)	Drainage characteristic
Gravel	>1	Pervious
Sand	$1 - 10^{-3}$	Pervious
Silt	$10^{-3} - 10^{-6}$	Slightly pervious
clay	$<10^{-6}$	Impervious.

Darcy Law :- (for 1D flow)

Discharge through fully saturated soil

$Q = kiA = AV$

V (velocity of flow / discharge velocity / superficial velocity)

actual velocity / seepage velocity $V_s = \frac{V}{n}$

cofficient of percolation $K_p = K/n$

Kozney Karman equation :-

$$K = \frac{1}{C} \times \frac{1}{S^2} \times \frac{\gamma_w}{\mu} \times \frac{e^3}{1+e} \times d^2$$

C → Kozney Karman cofficient which depend on type of soil str. & impurities in soil

S → specific surface area = area / Volume $= \frac{6}{dia} = \frac{6}{\sqrt{lb}}$ a-bt

Allen Hazen equation :- $K = CD_{10}^2$

K → cm/sec; D_{10} → (cm); C → order of hundred (100)

factors affecting permeability :-

(i) particle size $K \propto D_p^2$

(ii) void ratio $K \propto \frac{e^3}{1+e}$

(iii) effect of viscosity & temp $K \propto \frac{\gamma_w}{\mu}$

(K affected more by viscosity than change in unit wt) because unit wt of water does not change much over large range of temp.

$K_{27}\mu_{27} = \mu_T K_T$

(iv) shape of particle $K \propto 1/S^2$ → SSA

for same void ratio $S_{angular} > S_{rounded}$ particle.

(v) Degree of saturation $K_{partially\ saturated} > K_{full\ saturated}$

(vi) effect of impurities in water :-

∵ foreign material choke path hence reduce effective voids hence (K↓)

(vii) effect of effective stress $\bar{\sigma}\uparrow = e\downarrow \Rightarrow K\downarrow$

(viii) effect of absorbed cations of clay mineral surface.

(ix) effect of soil fabric

constant head permeability Test :- ($K > 10^{-3}$ cm/s)

for coarse grain soil, 100% degree of saturation

constant head maintained $Q = kiA = K\left(\frac{h}{L}\right)A$

variable head permeability Test / falling head :-

(for fine soils, $k = 10^{-3} - 10^{-7}$ cm/s)

undisturb specimen tested.

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

h_1 → initial head; a → pipe area; A → sample area; L → sample length; h_2 → final head

Capillarity permeability Test (for partially saturated soil) :- or unsaturated soil

$$\frac{X_2^2 - X_1^2}{t_2 - t_1} = \frac{2k}{Sn}(h_{01} + h_c)$$

$$\frac{X_4^2 - X_3^2}{t_4 - t_3} = \frac{2k}{Sn}(h_{02} + h_c)$$

h_c → capillary height

Imp.

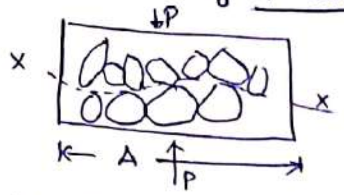
Horizontal flow (i = constant) $K_H = \frac{K_1 H_1 + K_2 H_2}{H_1 + H_2}$

vertical flow (q = constant) $K_V = \frac{H}{\frac{H_1}{K_1} + \frac{H_2}{K_2}}$

always $K_H > K_V$

total stress :- (physical parameter)

Imp → measure by "Pressure cell arrangement"



$$\text{total stress} = \frac{P}{A}$$

P → force on plane x-x for weight above plane xx

A → area of c/s of soil mass.

- total stress on a plane within a soil mass is the force per unit area of soil mass transmitted in normal direction across a plane.

effective stress :- $\bar{\sigma} = \sigma - u$ → concept given by Terzaghi

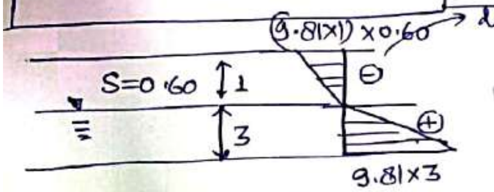
→ not physical parameter here cannot measure.

Porewater Pressure (Neutral pressure) :-

- Pressure of water filling the void space b/w solid particles.
- It acts on all sides of particles but does not cause particles to press against adjacent particles.
- It has no shear component.
- measurement by → piezometer / stand pipe.

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

C → 0.1–0.5 cm^2; D_{10} → cm



note:- Imp.

① if water table is above Grd level (lake, Grd level)

- due to change in depth of water, there will be no change in effective stress of soil as long as position of water table remains above the grd. level.

② if water table is below grd level (grd, GWT)

then → rise in water table ⇒ effective stress decreases {vice versa}

Seepage pressure :- Exerted by water on soil due to friction drag.

Imp.
- drag force / seepage force always acts in direction of flow

Seepage Pressure $P_s = iz\gamma_w = h\gamma_w$

seepage force = seepage pressure × area $= (iz\gamma_w) A = h\gamma_w$

Specific seepage force $= \frac{\text{seepage force}}{\text{volume}} = i\gamma_w$

- note:- seepage pressure, force, specific seepage force always acts in direction of flow / seepage

In downward flow condition :-

effective stress $= (\gamma_{sub}) z \downarrow + iz\gamma_w \downarrow$ (seepage pressure)

'or' eff. stress $= \frac{\text{Buoyant wt of soil} \downarrow + \text{seepage force} \downarrow}{\text{area}}$

In upward flow condition :-

eff. stress $= (\gamma_{sub} z) \downarrow - iz\gamma_w \uparrow$

Quick sand condition / Piping / soil Boiling :-

Hydraulic condition, which exist in cohesionless soil {only in fine sand, coarse silt; not in clay, gravel, coarse sand} when vertical effective stress becomes to zero.

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

∴ to avoid quick sand condition $fos = \frac{i_{cr}}{i}$

→ effective stress = 0 {when Buoyant wt. of soil = upward seepage force}

How to prevent Quick sand condition :-

(I) By lowering the water table at site before excavation.

(II) By increasing the upward flow length by providing sheet pile $\downarrow i = h/L \uparrow$

(III) additional weight on excavation side to increase the net weight of soil mass.

Seepage Through soil :

Seepage :- Process in which liq. leaks through a porous medium from high head to toward low head.

Problem due to Seepage :-

(I) Loss of water from reservoir
(II) Reduction in effective wt. of soil
(III) uplift pressure generated
(IV) piping failure.

Laplase eqn of 2D flow :-

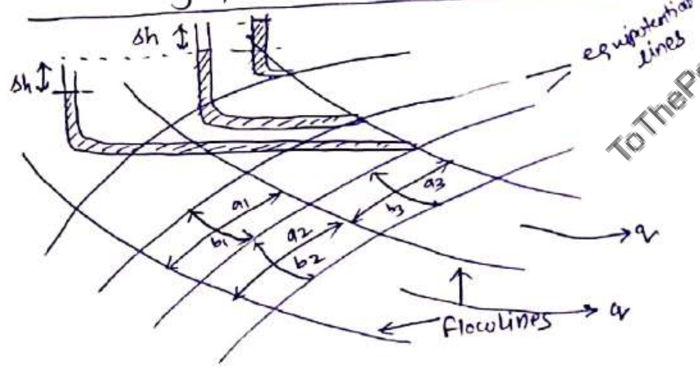
$$k_x \frac{\partial^2 H}{\partial x^2} + k_y \frac{\partial^2 H}{\partial y^2} = 0 \rightarrow \text{anisotropic soil}$$

$$X = X_T\sqrt{\frac{k_x}{k_y}}$$

for Isotropic soil :-

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

flownet :- graphical solution of Laplase eqn :-



- Flow lines ⊥ equipotential line {in Isotropic soil}
- Space b/w 2 adjacent flow lines → Flowpath or Flowchannel
- The figure formed in flownet b/w 2 adjacent flowlines and adjacent equipotential line is called flow field
- all flow fields are elementary squares (linear or curvilinear)

$$\frac{a_1}{b_1} = \frac{a_2}{b_2} = \frac{a_3}{b_3}$$

* (opposite side of square are equal)

- head loss through each successive equipotential line is equal $\Delta h_1 = \Delta h_2 = \Delta h_3 = \Delta h$

- discharge through each flow channel is constant.

$$\Delta q_1 = \Delta q_2 = \Delta q_3 = q$$

- Shape factor $\left(\frac{N_f}{N_d}\right) \Rightarrow f^n\{$Boundary condition$\}$
- flow net will not change if 'k' changes
- ——— if head loss during flow changed.
- Flow net is unique for a given boundary condition. if Boundary condition does not change $\frac{N_f}{N_d}$ will not change
- flow net can be change if extent of flow is changed

Flownet uses :-

(I) determination of seepage discharge

$$Q = KH\frac{N_f}{N_d}$$

H → under total head H discharge passing through flow channel

- H → Hydraulic head
- N_f → no. of flow channel
- N_f = (no of flow lines − 1)
- N_d → no of equipotential drop
- N_d = (no of equipotential lines − 1)
- N_f/N_d = shape factor

(II) uplift pressure calculation. (& seepage pressure also.)

(III) exit gradient calculation → equipotential drop

$$i = \frac{\Delta h}{b} = \left(\frac{h}{N_d}\right) \times \frac{1}{b}$$

b → size of exit flow field (b×b)

(IV) Pore water pressure Measurement.

Phreatic line :-

- Top most flow line which follows the path of base parabola
- It is a streamline.
- The pressure on this line is atmospheric (zero) and below this pressure is hydrostatic.

f/s

to prevent piping → protective filter / Inverted filter at d/s

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

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

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

loaded filter → reduce piping
└→ consist of graded sand & gravel.
fn → to increase the downward force without increasing the upward seepage force.

compaction	consolidation
Instantaneous process	time dependent phenomenon
soil always partially saturated	soil → fully saturated
densification → due to reduction in volume of air voids at a given water content	volume reduction is → due to expulsion of porewater from voids.
specific compaction Technique required	consolidation occurs on account of static load placed on it (soil)

compaction advantage →
- Settlement (ΔH↓) $\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0} = \frac{\Delta V}{V_0}$
- strength↑ ∴ stability↑
- load bearing capacity ↑
- undesirable volume change ↓ (By frost action, swelling, shrinkage)

compaction of cohesionless soil → By vibration

note:- liquifaction may occur in loose sand.

compaction of cohesive soil (clay) :- Application of Static load

Proctor Test :- Before starting compaction in field we must know compaction characteristic of soil.

① This Test gives idea of compaction characteristic of soil.

② It gives the density that must be achieved in field.

③ Provides the moisture range that allows for min. compactive effort to achieve required density.

Standard proctor Test / IS light compaction Test		modified proctor Test / IS heavy compaction Test
volume of mould	942 cc / 1000 cc	942 cc / 1000 cc
no. of layers	3	5
no of blows per layer	25	25
Height of free fall	304.8 mm / 310 mm	457.2 mm / 450 mm
weight of hammer	2.495 kg / 2.6 kg	4.54 kg / 4.9 kg

compactive effort :- measure of mechanical energy applied to soil mass.

∴ compactive energy per unit volume $E = \frac{N \cdot n \cdot W \cdot h}{V}$

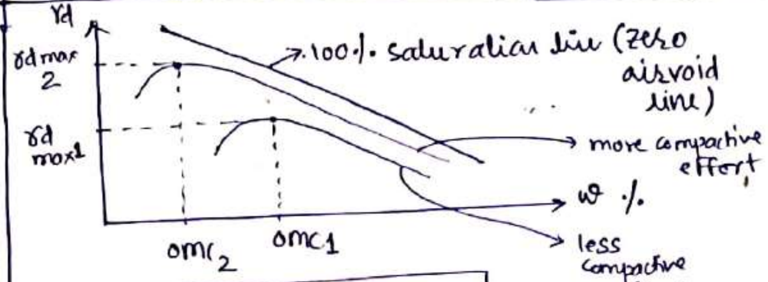
IS light compaction Test $E_1 = \frac{25 \times 3 \times (2.6 \times 9.81) \times 0.310}{1000 \times 10^{-6}}$

$= 593.014\ kJ/m^3$

IS heavy compaction Test $E_2 = 2703.88\ kJ/m^3$

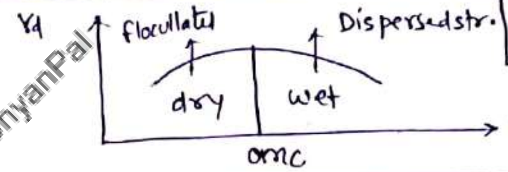
∴ $E_2/E_1 = 4.56$

Trick:- IS: 456 for RCC



$\gamma_d = \frac{(1-n_a) G \gamma_w}{1 + wG} = \frac{G\gamma_w}{1 + \frac{wG}{S}}$

** relative compaction $= \frac{(\gamma_d)_{field}}{(\gamma_d)_{max}}$



	Dry of optimum	Wet of optimum
Structure after compaction	flocculated str. (random)	dispersed str. (oriented)
water Deficiency	more ↑	less ↓
Permeability	more ↑	less ↑
Compressibility	at low stress — less ↓ at high stress — more ↑	more ↑ less ↑
swelling	more ↑ (due to random arrangement of particles)	less ↓
Shrinkage	less ↓	more ↑
Stress strain curve	Brittle (high peak, high elastic modulus)	ductile (no peak, lower elastic modulus)
Porewater pressure	low (∵ water deficiency more)	more (high)
Strength (Undrained) as moulded after saturation	high	low
Sensitivity	more	less

Imp

Project name	compacted at	Reason
Core of earth dam	wet side	permeability ↓ and prevent crack in core
Subgrade of pavement	wet "	∵ volume change less
Homogeneous embankment	dry side	strong soil and to prevent build up of high water pressure

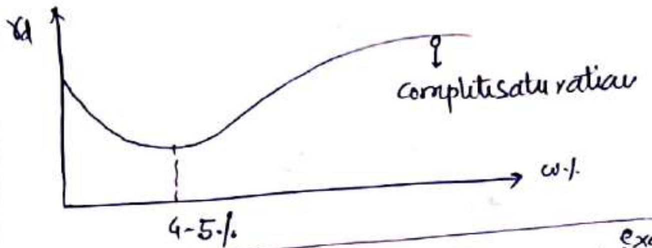
note:-
(I) coarse grain soil → (well graded) → (γ_d ↑)
(II) in clay higher plasticity ⇒ γ_d ↓ (omc ↑)
(III) Poorly graded / uniform sand → lead to lowest dry unit weight γ_d

Compaction Equipments :- v.v imp

equipment	suitable for	nature of project
Rammer or Tampers	all soil	in confined areas (Fills behind retaining wall, Basement walls) Trench fills
Smooth wheeled roller (crushing action)	crushed rocks, Gravel, sand	Road construction
Pneumatic tyre roller [for less plastic clay]	sand, gravel, silt & clay • not suitable for uniformly graded soil	compaction of highway & airfields & earth dam
Sheepfoot roller (kneading action)	clay soil (medium to high plastic clay)	core of earth dam
vibratory roller	sand	embankment for oil storage tank

↳ Best when frequency match with natural frequency of soil

Bulking of sand :-



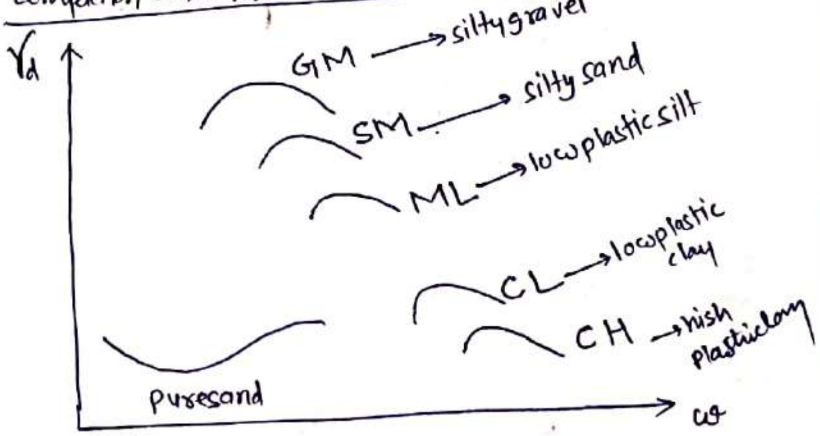
• Bulking effect max. when m/c = 4-5% (4% exact)
increase in volume = 20-30%.

• initially there is decrease in γ_d with increase in w%, this is due to capillary tension in pore water which prevents soil particles to coming closer. → Bulking of sand.

at last meniscus broken (w ↑ γ_d ↑)
↓
Particles are able to move and adopt a closer packing.

note:- Coarse grain soil does not absorb water as fine grain hence Lambe's theory (doubt) not applicable.

Compaction curve for different soils :-



Compressibility

The property of soil due to which decrease in volume occures under compressive force. (stress)

(i) compression & expulsion of air (poreair)

(ii) ———————— of porewater

(iii) Gradual readjustment of clay particles into more stable configuration.

total settlement (S) = $S_{immediate} + S_{1°\,consolidation} + S_{2°\,consolidation}$

(1) **Immediate settlement**: if soil is initially partially-saturated, expulsion of air as well as compression of poreair may take place with the application of external load which is called initial compression. It is Immediate phenomenon.

- Immediate settlement calculation by elastic theory

(2) **Primary settlement** (1° consolidation) :- (time dependent phenomenon)

- It occures due to excess pore water pressure generated due to increase in total stress.
- magnitude of Primary Settlement depends
  - → compressibility of soil
  - → magnitude of stress increased
  - → thickness of soil layer
  - → Permeability of soil.

Ex* when a str. is built over layer of saturated clay.

* when a water table is lowered permanently in a str. overying a clay layer.

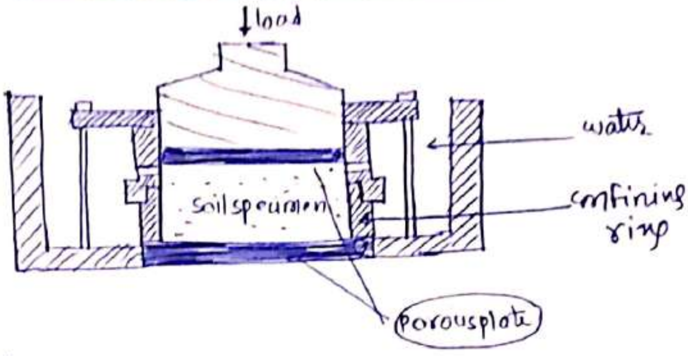
(3) **2° consolidation** :- compression of soil layer don't cease when excess pore water pressure has been completely dissipated to zero, It continues at gradually decreasing rate under constant stress.

- 2° consolidation is due to gradual readjustment of clay particles into a more stable configuration following a structural disturbanance caused by decreased in void ratio.

v.Imp

note:- 2° consolidation Imp. for organic soil (Peat) (not for clay & stiff clay)

Consolidation Test / odeometer Test



Imp

aim → to find out Cv (cofficient of consolidation)
Cv → indicates rate of consolidation

- charactur stic of soil during one dimensional consolidation / swelling or can be determined by this test

Imp
- the soil sample in odeometer test will be in double drainage condition.

- soil is loaded in increament of vertical stress. (usually load in kept for 24 hrs)
- under each increament of loading, soil is allowed to consolidate, note down compression reading in this 24 hr.

- as soil loaded excess porewater pressure developed, if expulsion of pore water is allowed then gradually excess pore pressure will reduce. depending on drainage condition, this will come out either from top or bottom or both.

Imp
- Porewater moves from centre towards top/bottom thus pore water pressure is max. at centre & min. at top & bottom.

- Prepare graph b/w void ratio at each of increament end of Period vs corresponding effective stress.

$$\frac{\Delta H}{H} = \frac{\Delta V}{V} = \frac{\Delta e}{1+e_0}$$ for 1° consolidation

equibrium void ratio at each stress level can be found by

① Height of solid method

$e = \frac{V_v}{V_s} = \frac{V-V_s}{V_s}$

$e = \frac{AH - AH_s}{AH_s} = \frac{H-H_s}{H_s}$

$\because G = \gamma_s/\gamma_w = \frac{W_s/V_s}{\gamma_w}$

$G_s = \frac{W_s/AH_s}{\gamma_w}$

$\therefore H_s = \frac{W_s}{\gamma_w A G_s}$

② change in void ratio method.

∵ soil at the end of this test is assumed to be saturated.

$$e_f = w_f G$$

$$e_f = e_0 - \Delta e$$

$w_f \rightarrow$ water content at end of Test. (noted after overdrying)

Isochrone :- Imp.

(solution of Terzaghi's eqn plotted in form of Isochrone] u, z, t

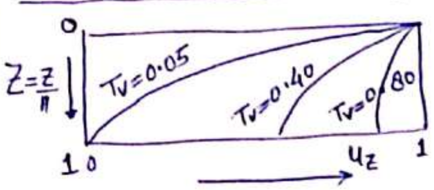
The Progress of consolidation can be shown by — Plotting a series of curve of u (excess pore-water pressure) against z for different value of time (t)

- Isochrone forms depend on
  - Initial distribution of Porewater pressure
  - drainage condition at the boundary of clay layer

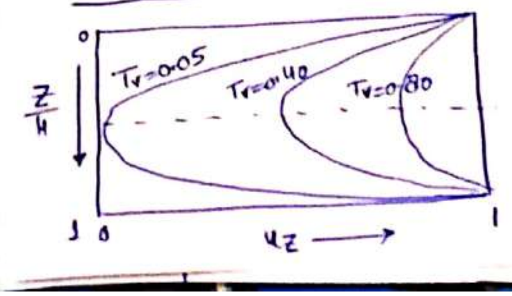
Imp.
note:- slope of Isochrone at any depth 'z' gives hydraulic gradient.

$$i = \frac{\partial h}{\partial z} = \frac{\partial u}{\gamma_w \partial z}$$

Isochrone for single drainage condition →



Isochrone for double drainage condition →



vertical sand drain :- → accelerate consolidation process.

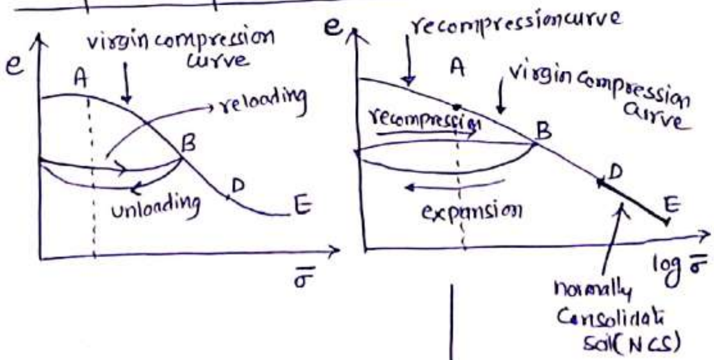
- Based on consolidation theory of radially and vertically drained clay system.

$$(1-U) = (1-U_v)(1-U_r)$$ *

Ex. if $U_v = 20\%$. $U_r = 80\%$

then $U = 84\% = \frac{\Delta h}{\Delta H} \times 100$

compressibility charactristics :-



e-log σ̄ curve
- Straight line (for normally consolidate soil)
- Convex curvature upward — for over consolidated soil

Normally consolidated Soil (NCS) :-

- When existing effective stress is the stress it has ever experienced in its stress history (Past)

- for NCS $OCR = 1$

$$OCR \text{ (over consolidation ratio)} = \frac{\text{max. effective stress in Past 'or' Preconsolidation stress \{PAST\}}}{\text{existing effective stress \{Present\}}}$$

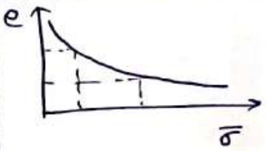
For overconsolidated soil / Preconsolidated soil $OCR > 1$

compressibility NCS > OCS
(amt of deformation Per unit increase in stress)

Imp
over consolidation is due to
- → erosion of overburden soil
- → Permanent rise in water table
- → melting of ice sheet after glaciation.

compressibility of clay can be represented by any of the following parameter.

① cofficient of compressibility (a_v)



$$a_v = \frac{e_1 - e_2}{\bar{\sigma}_2 - \bar{\sigma}_1} = \frac{\Delta e}{\Delta \bar{\sigma}}$$

Plot b/w arithmatic scale Vs arithmatic scale.

② cofficient of compression (C_c)

- significant only for normally consolidated soil (NCS)
- for a given type of soil $C_c \rightarrow$ constant
- $C_c \rightarrow$ not fn of effective stress.
- Higher $C_c \rightarrow$ higher vertical deformation

$$C_c = \frac{e_1 - e_2}{\log \bar{\sigma}_2 - \log \bar{\sigma}_1} = \frac{\Delta e}{\log \frac{\bar{\sigma}_2}{\bar{\sigma}_1}}$$

(Plot b/w arithmetic scale Vs log scale)

undisturb soil of medium sensitivity:
$C_c = 0.009(w_L - 10)$

Remoulded soil of low sensitivity
$C_c = 0.007(w_L - 7)$

} used for Preliminary estimate of settlement

③ cofficient of Volume compressibility (m_v)

$$m_v = \frac{\Delta V / V}{\Delta \bar{\sigma}} = \frac{\text{Vol. change per unit volume}}{\text{increase in effective stress}}$$

$$\rightarrow \frac{\Delta H / H}{\Delta \bar{\sigma}} \quad \left\{ \frac{\Delta H}{H} = \frac{\Delta e}{1+e_0} \right.$$

$$m_v = \frac{a_v}{1+e_0}$$

- for a given type of soil → (m_v) not constant depend on stress range over which it is calculated.

note:- Compression modulus $E_c = \frac{1}{m_v}$

Terzaghi's one dimensional consolidation :-

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

u → excess pore water Pressure at any time 't' and any location 'z'

$\frac{\partial u}{\partial t}$ → rate of change of Porewater pressure

$\frac{\partial^2 u}{\partial z^2}$ → rate of change of Pore pressure with depth.

Terzaghi defined 3 nondimensional Parameters

(i) drainage path ratio $Z = z/H$

(ii) time factor $T_v = \frac{C_v t}{d^2}$ → $T_{50} = 0.196$, $T_{90} = 0.848$

(iii) degree of consolidation U%

$$U\% = \frac{u_i - u}{u_i} \times 100 = \frac{e_0 - e}{e_0 - e_f} \times 100 = \frac{\Delta h}{\Delta H} \times 100$$

$$T_v \begin{cases} \frac{\pi}{4} U^2 & (U \le 60\%) \\ 1.781 - 0.933 \log(100 - U\%) & (U > 60\%) \end{cases}$$

Assumption in Terzaghi 1D consolidation theory :-

① soil → Homogeneous, fully saturated.
(ii) soil particles, water → both are incompressible.
③ compression, Porewater flow → onedimensional (vertical)
④ strain → small ⑤ K, m_v, a_v → constant throughout soil.
⑥ darcy law valid at all hydraulic gradient.
⑦ a_v → constant over stress increment (there is unique relationship independent of time b/w void ratio & effective stress)
⑧ secondary compression → neglected because it exist only at constant effective stress.
Imp ⑨ Hydrodynamic lag considered, Plastic lag Ignored.

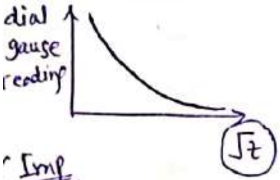
Hydrodynamic lag → time lag due to low permeability of clay, and consequent time require for escape of water.

Plastic lag :- effective stress reach at constant value in secondary consolidation hence ignore in 1D consolidation.

Cv determination

Square root of time fitting method (Taylor's method) (T_{90})

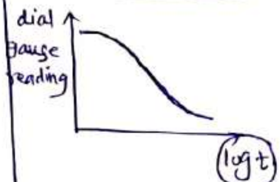
$$C_v = \frac{T_{90} \times d^2}{t_{90}}$$



Imp: Best for higher secondary consolidation soil (High plastic soil)

Logarithm of time fitting method (Casagrande method) (T_{50})

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



note:- $C_v \downarrow \propto \frac{1}{w_L \uparrow} \propto \frac{1}{I_p}$ (Imp)

Compression ratio :- $r_i + r_p + r_s = 1$ *

1- Initial compression ratio $(r_i) = \frac{R_i - R_o}{R_i - R_f}$ → zero

2- Primary consolidation ratio $(r_p) = \frac{R_0 - R_{100}}{R_i - R_f}$

3- Secondary consolidation ratio $(r_s) = \frac{R_{100} - R_f}{R_i - R_f}$

R_i → initial reading of dial gauge
R_f → final —— after secondary consolidation
R_o → 0% consolidation ——
R_{100} → 100% ——

Primary settlement :-

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0} \quad \text{—— (1)}$$

$$\Delta H = H_0 m_v \Delta\bar{\sigma} \quad \text{—— (2)}$$

$$\Delta H = \frac{H_0 C_c}{1+e_0} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right) \quad \text{—— (3)}$$

note: C_c → diff. for different soil layers. ΔH calculate for each soil layer then (sum) them.

Very special case : V. Imp

for over consolidated soil (OCS)

$\left\{ OCR = \frac{\text{Past } \bar{\sigma}_c}{\text{Present } \bar{\sigma}_0} \right.$

case-1

if $\bar{\sigma}_0 + \Delta\bar{\sigma} > \bar{\sigma}_c$ (70, 80, 120)

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

(120/70 ; 150 = 70 + 80 / 120)

→ first term: settlement for overconsolidation stage
→ second term: settlement for normally consolidated stage ($\bar{\sigma}_0 + \Delta\bar{\sigma} > \bar{\sigma}_c$)

Imp: note:- H_0', e_0' → H_0, e_0 put same (will result into very less error)

case-2

$\bar{\sigma}_0 + \Delta\bar{\sigma} < \bar{\sigma}_c$ (70, 80, 180)

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

→ settlement for overconsolidation stage

Secondary settlement :- (in organic soil, High plastic soil)

- Occures at constant effective stress
- not related to disipation of pore water pressure.
- occures due to readjustment of soil skeleton after disturbance of 1° consolidation.
- rate of 2° consolidation is controlled by viscous-absorbed layer surrounding clay particles in soil. (Imp)

$$S_s = \frac{C_s H_{100}}{1+e_{100}} \log\left(\frac{t}{t_{100}}\right)$$

t → avg. time after t_{100} secondary consolidation is calculated.
e_{100} → void ratio at end of primary consolidation

Imp: max. differential settlement: Sand 25 mm; Clay 40 mm

Imp: Settlement – Isolated footing: sand 40 mm; clay 65 mm

Settlement – raft: Sand 40–65 mm; clay 65–100 mm

Stress in soil
- due to applied load
- due to self wt of soil

Bousinesq eq^n (for stress distribution in an elastic medium subjected to concentrated load on its surface.

Assumption:
1. Theory of elasticity → valid
2. soil → Homogeneous, Isotopic, semiinfinite
3. Self weight of soil → 0 (zero)/neglected
4. soil mass → elastic
5. surface free from shear stress (unstressed before Application of load)

Newmark's chart Based on this theory.

$$\sigma_z = \frac{3Q}{2\pi z^2}\left(\frac{1}{1+(r/z)^2}\right)^{5/2} = K_B \times \frac{Q}{z^2} \; ; \; K_B \to \text{no. fn of } (r/z)$$

$r=0$ $\quad \sigma_{z\,max} = 0.4775\,Q/z^2$ $\quad K_{B\,max} = 0.4775$

note: - can be used for upward load (-)ve ∵ vertical stress decrease due to excavation.

note Imp: - ∴ actual stress < Bousinesq stress hence this theory gives conservative result hence generally used.

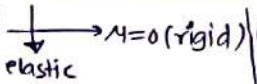
- if $r \to$ constant then for $\sigma_z \to$ max $\frac{d\sigma}{dz} = 0$

$$(\sigma_z)_{max} = \frac{0.0888Q}{r^2} \quad \left\{\frac{r}{z} = \sqrt{2/3} = 0.8164\right\}$$

- shear stress $(\tau) = \sigma_z \times r/z = \frac{3Qr}{2\pi z^3}\left(\frac{1}{1+(r/z)^2}\right)^{5/2}$

∴ $\tau_{max} = 0.0888Q/r^2 \times 0.8164 = 0.0725Q/r^2$

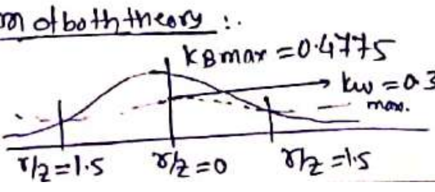
Westergard's eq^n (Fenske's chart Based on this)
1. soil → soil → elastic, semiinfinite
2. non Isotropic soil
3. Stratified soil (layers of silt & clay)
4. $\mu = 0$ (rigid) (soil mass divided into horizontal sheets of negligible thickness closely spaced and infinite rigidity in (horizontal direction. that allows only vertical movement and prevent soil mass as a whole from undergoing any latiral strain.



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

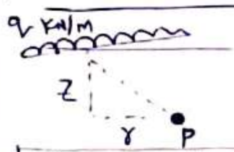
$r=0$ $\quad \sigma_z = 0.3183\,Q/z^2$ $\quad K_{w\,max} = 0.3183$

comparison of both theory:



note: - westergard Theory → gives result close to field test result ∴ Bousinesq theory is more conservative hence used.

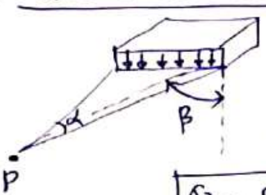
1- vertical stress due to line load (kN/m):



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

when point line below line load $r=0$ ∴ $\sigma_z = \frac{2q}{\pi z}$

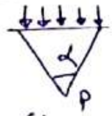
2- vertical stress due to strip load (kN/m²):



$$\sigma_z = \frac{q}{\pi}(\alpha + \sin\alpha \cos 2\beta)$$

when $\beta = 0$

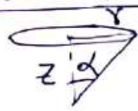
$$\sigma_z = \frac{q}{\pi}(\sin\alpha + \alpha)$$



(below centre of strip)

3- vertical stress below circular area (kN/m²):

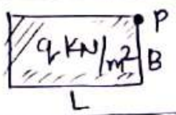
$$\sigma_z = q(1-\cos^3\alpha)$$



$$\left\{\cos\alpha = \frac{z}{\sqrt{r^2+z^2}}\right.$$

Imp note: -
1. for a very large uniformly loaded area: ($\alpha = 90°$) ∴ $\sigma_z = q$ **
2. if entire semiinfinite soil mass loaded with q: $\sigma_z = q$ ($\alpha = 90°$) **

4) vertical stress below corner of rectangular:

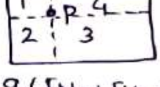


$$\sigma_z = q(I_N) \qquad m = B/z, \; n = L/z$$

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

5) vertical stress at any point under a rectangular area:

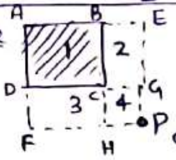
a) point below rectangular area



- divide in 4 rect.
- apply formula of corner
- add all

$\sigma_z = q(I_{N1} + I_{N2} + I_{N3} + I_{N4})$

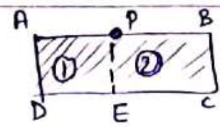
b) Point outside the loaded area



area of rect. ABCD = AEPF − BEPH − DGPF + CGPH

$\sigma_z = q(I_{N1} - I_{N2} - I_{N3} + I_{N4})$

c) Point below edge of loaded area:



$\sigma_z = q(I_{N1} + I_{N2})$

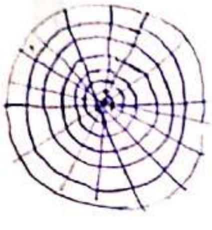
APDE, PBCE

Newmarkchart (not for statrified soil)

∴ this is based on Bousinesq theory.

(i) to know (get) → vertical stress, → Horizontal stress, → shear stress

(ii) due to uniformly loaded area of any shape, below any point (inside, outside of loaded area) we can get the stress by this method.



no. of concentric circle = 10
no. of radial lines = 20

∴

influence of each area = $\frac{1}{\text{total no. of Sectoral area } (10\times 20)}$ = $\frac{1}{mm}$

⇒ 0.005

$$\sigma_z = 0.005 \times q \times N_A$$

- 0.005 → Influence value $\left(\frac{1}{mm}\right)$
- q → load intensity (kN/m^2)
- N_A → total no. of sectorial area of newmarks's chart.

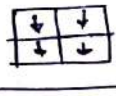
note:- each area unit has same influence whether inside or outside of area.

Approximate methods :-

1- equivalent load method :-

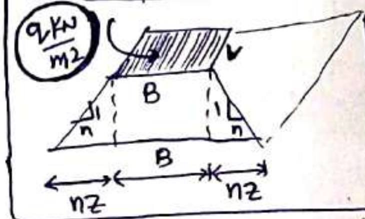
smallest area की largest dimension < $\frac{depth}{3}$ होना चाहिये

↳ otherwise divide in parts.



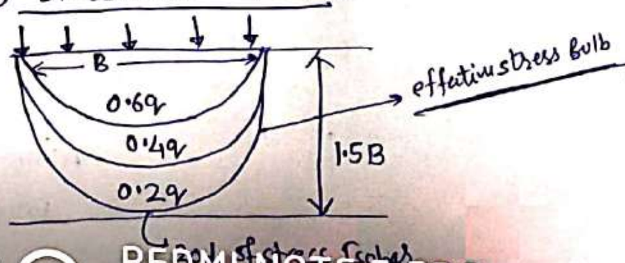
$$\sigma_z = k_{B_1} Q_1/z^2 + k_{B_2} Q_2/z^2 + \cdots$$

2- Trapezoidal method :-



$$\sigma_z = \frac{q(BL)}{(B+2nz)(L+2nz)} \ \frac{kN}{m^2}$$

(3) Stress Isobar method :-



- shear strength is resistance offered against relative motion b/w 2 particles.
- soil may derive it shear strength from
  1. Interlocking between molecules
  2. friction between molecules (Rolling/sliding)
  3. Interaction b/w molecules (cohesive/adhesive)

note:

- coarse grain soil → interlocking + friction
- fine grain soil → friction + cohesion
  - Ex. in pure clay → only from cohesion

coulomb Theory :-

initially $\tau = c + \sigma_n \tan\phi$

later $\tau = c' + \bar{\sigma} \tan\phi'$

c', ϕ' → effective stress shear parameters

note:- $c-\phi$ ⇒ shear strength parameter of soil

$c \& \phi$ → not inherent property of soil these are related to type of test and the condition under which They are measured.

mohr's Theory :- (based on following fact) :-

1. material fails essentially by shear
2. ultimate strength of material is determined by the stress in plane of slip
3. failure criterion is independent of the intermediate principal stress.

note:-

1. under dry stage :-

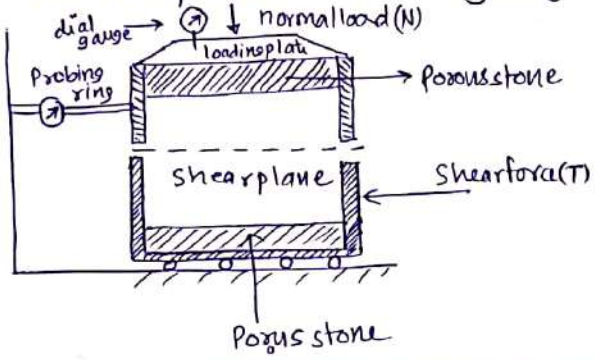
   total stress parameter = effective stress parameter.

2. shear strength
   - under drain condition ⇓ effective stress approach used.
     - Ex. evaluate long term stability post construction stability check.
   - undrain condition
     - when (+) excess pore water pressure developed → total stress analysis ⇓ (short term) Ex. Stability check immediate after construction
     - when (−)ve pore water pressure develop → effective stress analysis → long term stability is chec

Direct shear Test :- (imp.) | in drained condition | ***

{ ∵ drainage can not controll hence rate of loading be such that pore-water pressure does not develop hence it will be a drained condition Test }

- (imp) for free draining soil (sand + gravel)
- (imp) not for clay { ∵ drainage can not controlled) }
- effective stress & total stress ⟶ same
- volume expansion measured using dial gauges.



- Direct shear Test is conducted on a soil-specimen in a shear Box which is split into 2 halves along a horizontal plane at its middle.
- shear Box ⟶ made of Brass or gun metal
- shear Box ⟶ circular or square
  { generally 60×60×50 mm is used }

Disadvantages/ limitation of Direct shear Test :-

(i) Drainage condition can not be controlled & Pore water pressure can not be measured.

$\bar{\sigma} = \sigma - u^{\to 0}$

(ii) failure plane is always horizontal & Predetermined which may not be weakest plane.

(iii) non uniform stress distribution on shear plane.

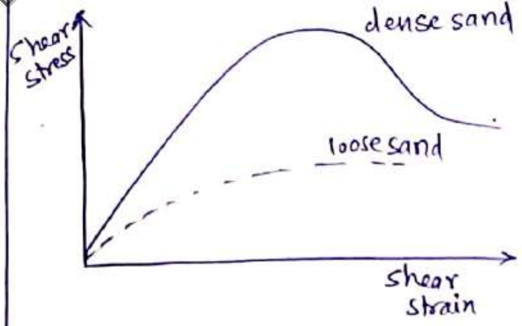
failure starts at edges & progresses towards centre.

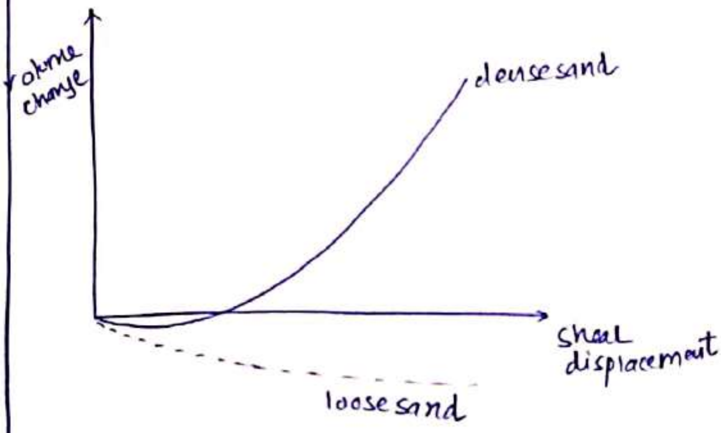
(iv) Area of specimen under normal & Shear does not remain constant during the test. hence, calculation of normal & shear stress are done on the basis of nominal area (original area) which is not correct.

(v) Direction of principal plane are not known at every stage of test.

It is only when mohr failure envelope is known that direction of principal stress will be known.

Result of direct shear Test for (sand) :-



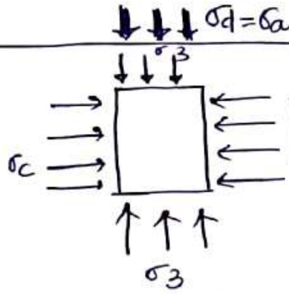


Triaxial Shear Test :

- to determine shear characteristics of all type of soil under different drainage condition.

Imp. • There is complete controll over the drainage condition so test can be easily conducted for all three types of drainage condition (UU, CU, CD, UD Test)
UD Test → not possible

Imp. • There is provision to measure porewater pressure



$\sigma_3 = \sigma_c$ cell pressure (confiningstress)

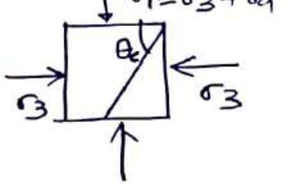
$\sigma_d = \sigma_a$ (additionalaxialstress or deviator stress)

note:- Stage 1 :- $\sigma_3 = \sigma_c$ {equal in all direction} (consolidationstage) applied.

Stage-2 :- additional $\sigma_a = \sigma_d$ applied (shearstage)

note:- Intermediate principal stress (σ_2) in a Triaxial Test is taken as being equal to minor principal stress because of axial symmetry

note:- stress distribution at failure plane is fairly uniform



Imp.

$$\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2)$$

$\theta_c = 45 + \phi/2$ → from major principal plane.

$$A_f = A_0 \left(\frac{1 \pm \epsilon_v}{1 - \epsilon_L} \right)$$

$\epsilon_L = \frac{\Delta l}{l}$

UU Test (Unconsolidated Undrain Test)

$\epsilon_v = 0$

$\therefore A_f = \frac{A_0}{1 - \epsilon_L}$

$\left(\frac{1}{1 - \epsilon_L} \right)$ → area correction factor

CD Test

$\epsilon_v = \frac{\Delta V}{V}$

deviator stress ($\sigma_a = \sigma_d$) ⇒ $\frac{\text{Axial load}}{\text{corrected area } (A_f)}$

Imp. note:- during Triaxial Test

either poreline is open {to get porewater pressure}

or drainage line is open {to get volume change}

Three test condition possible in Triaxial Test :-

① **CD Test** (consolidated drained Test) 'or' {Slow Test / S Test}

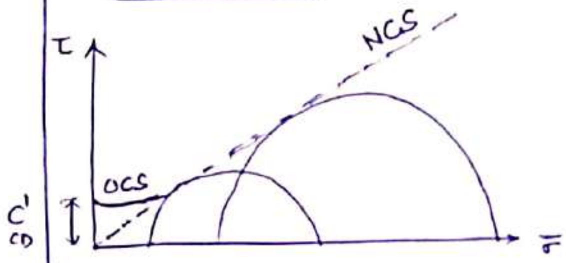
Stage 1	Stage 2
consolidation Stage	shear stage
drainage permitted	drainage permitted

∵ in both stages drainage allowed hence takes time (most time taking Test)

- volume change significant

ϵ_v ✓

- imp Test suitable for saturated sand and also for saturated clay under longterm.
- imp To check longterm stability of embankment.

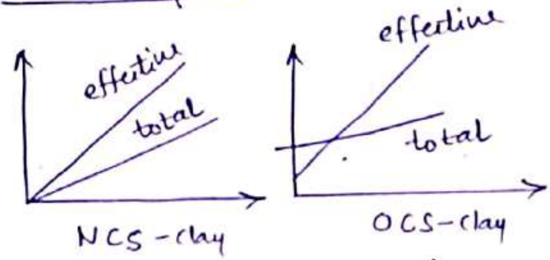


- Stability analysis of retaining wall having sandy fills.

③ **CU Test** (consolidated undrained Test)

(24 hr) Stage 1	(2hr) Stage 2
drainage allowed	drainage not allowed

- imp prefer for soil subjected to sudden change in drainage condition. {soil near hydraulic str canals, wells, dams}
- imp stability analysis of earthen dam against failure caused by sudden drawdown of water

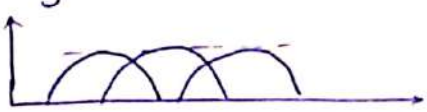


② **UU Test** (unconsolidated undrain Test) or (Quick Test)

$\boxed{\epsilon_v = 0}$

Stage 1	Stage-2
drainage not allow	drainage not allow

- takes only 15 minutes (very quick Test)
- negligible / no volume change observed.
- only 1 (unique) mohr circle obtained



- suitable for saturated clay subjected to fast loading rate
- imp (construction of building over saturated clay)
- sudden loading rate such as rapid construction
- short term stability under construction (during construction only).

④ **UD Test** not possible

Unconfined compression Test (UCS Test): Imp.

→ special form of triaxial Test in which $\sigma_3 = 0$ only 1 mohr circle obtained [only 1 stage (shear stage)]

$\sigma_1 = q_u$, $\sigma_3 = 0$

~~$\sigma_1 = \sigma_3 \tan^2(45+\phi/2) + 2c\tan(45+\phi/2)$~~

$\sigma_1 = q_u$

$\sigma_1 = q_u = 2c\tan(45+\phi/2)$

↳ axial stress at failure

Imp
note: The Test can be conducted only on cohesive soil, the load is rapidly applied hence It is undrain Test. Angle of internal friction is not mobilized ($\phi_u = 0$)

$\therefore q_u = 2c_u$ (unconfined compresive strength)

$\tau_f = c_u \Rightarrow$ undrain shear strength $= \frac{q_u}{2}$

$q_u = \frac{P}{A_f} = \frac{P}{\left(\frac{A_0}{1-\epsilon_L}\right)}$ undrain means $\epsilon_v = 0$

Imp.: This Test is used to calculate sensitivity

$S = \frac{q_u \text{ undisturb}}{q_u \text{ remoulded}}$ @ same water content

Vane shear Test:

Imp
(Best for undrain condition, soft saturated clay, high plastic clay, marine clay, sensitive clay, stiff & fissured clay)

Imp.
- In plastic cohesive soil ($\because$ very sensitive) hence difficult to obtain undisturb sample. shear strength of such soil may be significantly affected during sampling and handling

note:-

- when shearing is done by both top & bottom ends of vane.

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

- when shearing is done such that top end of vane does not shear the soil

$$\tau_f = S = \frac{T}{\pi d^2\left(\frac{h}{2}+\frac{d}{12}\right)}$$

Skempton's pore pressure parameter :-

Use :- to express the response of pore-water pressure to change in total stress under undrain condition and enable the initial value of pore water pressure to be determined.

$B = \frac{(\Delta u)_c}{\Delta \sigma_3}$ → change in pore pressure due to increase in cell-pressure

$0 \le B \le 1$ (0 → dry soil; 1 → saturated soil)

$B = f^n$ (degree of saturation)

$B = \frac{1}{1 + \frac{n C_v}{C_c}}$

$\bar{A} = AB$

$\bar{A} = \frac{(\Delta u)_d}{\Delta \sigma_d} = \frac{(\Delta u)_d}{\Delta\sigma_1 - \Delta\sigma_3}$ → change in pore pressure due to deviator stress; $\Delta\sigma_d$ → change in deviator stress

$A \rightarrow f^n \Rightarrow$ {Strain, Anisotropy, sample disturbance, OCR ratio}

Imp: $A_{max} = 2 \text{ to } 3$ (very loose saturated fine sand)
$A_{min} = -0.5$ (heavily overconsolidated clay)

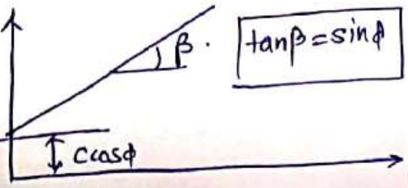
Imp: $\Delta u = \Delta u_c + \Delta u_d = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$

Stress path eqn :-

Ex. $q = 10\sqrt{3} + 0.5P$

Imp (at failure): $\left(\frac{\sigma_1 - \sigma_3}{2}\right) = c\cos\phi + \left(\frac{\sigma_1+\sigma_3}{2}\right)\sin\phi$ — for comparison std. eqn

$q = \left(\frac{\sigma_1 - \sigma_3}{2}\right)$



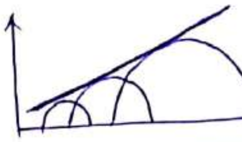
$\tan\beta = \sin\phi$

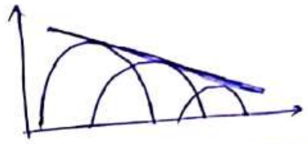
$P = \left(\frac{\sigma_1+\sigma_3}{2}\right)$

Different cases of stress path :-

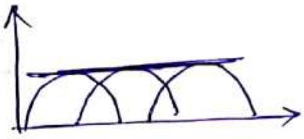
① Embankment construction (opposite → ② pit Excavation)

hint: {σ_1 increases due to weight of soil, $\sigma_1 = K\sigma_3$}

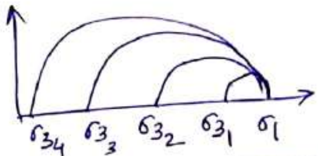




③ Hydrostatic Loading (dia of mohr circle → constant)



④ lateral expansion of backfill.
σ_1 = constant
$\sigma_3 = \downarrow\downarrow\downarrow$ (movement of wall away from backfill)



Soil liquifaction :-

(i) • occures in saturated loose sand ($\bar{\sigma} = 0$)
{ if loading is larger and soil is saturated then (+ve) pore water will develop
$\bar{\sigma} = \sigma - u$ (→ (+ve)) $u\uparrow\uparrow \therefore \bar{\sigma} \rightarrow 0$ }

(ii) occures during
→ pile driving
→ vibration of machine (high frequency)
→ explosive blasting
→ Earthquake shock

note:- (1) there can be cumulative increase in pore water pressure under successive cycle of loading

v.v.imp (2) How =? ⇒ once a complete loss of strength occured in a limited mass of soil, the stress which were carried by affected soil before its liquifaction, all transferred to adjacent parts, this process continues.

2/3/2020

Risk of slope failure

(i) water content ↑

(ii) Steeping of slope ↑

(iii) due to excavation ↑

(iv) increase in wt of soil ↑ ⇒ sliding failure.

(v) surcharge loading ↑

(vi) seepage of water ↑

(vii) seismic forces ↑

Infinite slope :-

$$fos = \frac{resistance}{disturbance} = \frac{c + \bar{\sigma}\tan\phi}{\tau}$$

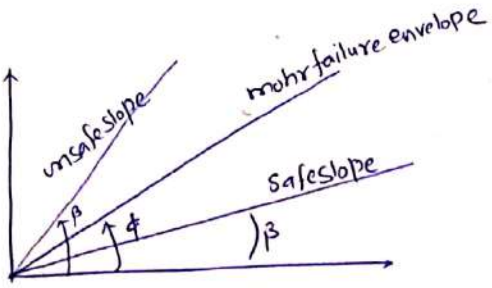
$c + \bar{\sigma}\tan\phi$ = moblised (or developed) shear strength; τ → shear stress

$$fos = \frac{c + \gamma Z \cos^2\beta \tan\phi}{\gamma Z \cos\beta \sin\beta}$$ for c-φ soil

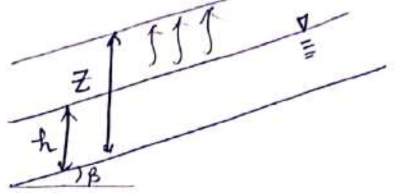
cohesionless soil $c = 0$	cohesive soil $\phi = 0$
$fos = \frac{\tan\phi}{\tan\beta}$	$fos = \frac{c}{\gamma Z \cos\beta \sin\beta}$

$$fos = \frac{H_c}{H} = \frac{C}{C_m} = \frac{\tan\phi}{\tan\phi_m}$$

$$H_c = \frac{4c}{\gamma\sqrt{K_a}}$$



Seepage taking place & water table parallel to the slope in cohesionless soil



$$fos = \left[1 - \frac{\gamma_w h}{\gamma_{avg} Z}\right]\frac{\tan\phi}{\tan\beta}$$

$$\gamma_{avg} = \frac{\gamma_1 h_1 + \gamma_2 h_2}{h_1 + h_2}$$

in this case capillary action

hence $\gamma_{avg} = \gamma_{sat}$

$$\therefore\ fos = \left(1 - \frac{\gamma_w h}{\gamma_{sat} Z}\right)\frac{\tan\phi}{\tan\beta}$$

V. Imp

note:- if water table is at grd level

$$fos = \left(\frac{\gamma_{sub}}{\gamma_{sat}}\right)\frac{\tan\phi}{\tan\beta} \approx \frac{1}{2}\frac{\tan\phi}{\tan\beta}$$

taylor's stability no. (Sn)

$$S_n = \frac{c}{\gamma H_c} = (\tan\beta - \tan\phi)\cos^2\beta$$

- c soil, $\phi = 0$
- cohesionless soil, $c = 0$

S_n (max) → 0.50

S_n min → 0.261

Imp:

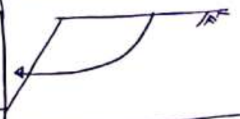
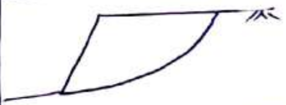
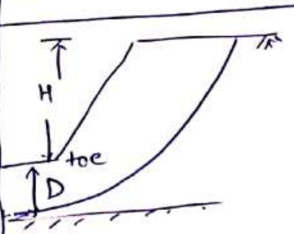
Special case :- sudden drawdown case

weighted friction angle $\phi_\omega = \phi \times \frac{\gamma_{sub}}{\gamma_{sat}}$

↓

इस पर S_n देखें

finite slope failure :-

(i) face failure	
(ii) toe failure	
(iii) Base failure D → Depth of hard stratum below toe H → height of slope above toe	 Depth factor = $\frac{H+D}{H}$

Various methods to analyse finite slope

(I) Swedish circle method (II) friction circle method (radius ⇒ $R\sin\phi$)

(III) Fellineous method (IV) taylor stability no.

↓ Imp:

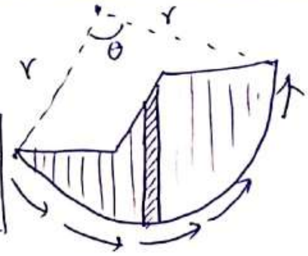
angle used for shearing resistance = mobilised angle

Swedish circle method :-

$$fos = \frac{M_R}{M_0} = \frac{CR\theta + \Sigma N \tan\phi}{\Sigma T}$$

$\Sigma N = \Sigma W\cos\alpha$

$\Sigma T = \Sigma W\sin\alpha$

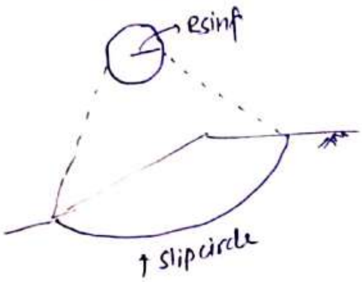


friction circle method :- radius = $R\sin\phi$

(i) wt. of sliding wedge of slope

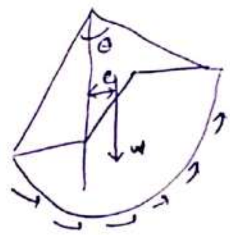
(ii) Resultant reaction R of slip

(iii) total cohesive resistance developed along the slip circle



fellinious method :-

$$fos = \frac{(CR)R\theta}{We}$$



magnitude of lateral earth pressure dependents on
(i) mode of movement of wall
(ii) flexibility of wall
(iii) soil property
(iv) drainage condition.

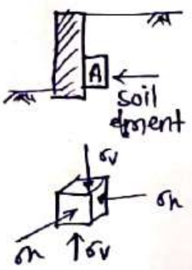
Retaining structures →
- Retaining wall
- Bracings in cut
- Bridge abutment
- Sheet pile / anchored sheet pile

note:- retaining wall design for active earth pressure

Types of lateral earth pressure :-

① Earth pressure at rest :-

- wall doesnot move at all
- soil element not subjected to any strain
- analysis by theory of elasticity.
- if the wall is rigid & unyielding, the soil mass is retained by it is in the state of rest and there is no deformation & Deflection.



at rest condition :-

Strain in horizontal direction = 0 (ϵ_h)

$$-\frac{\sigma_h}{E}+\frac{\mu\sigma_h}{E}+\frac{\mu\sigma_v}{E}=0$$

$$\sigma_h=\frac{\mu}{1-\mu}\sigma_v=K_0\sigma_v$$

$$K_0=\frac{\mu}{1-\mu}\qquad K_0=\frac{\sigma_h}{\sigma_v}$$

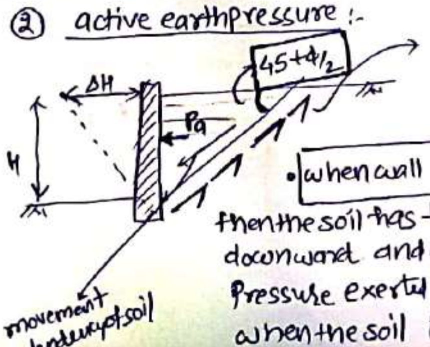
Imp.

$$\therefore \mu=\frac{\sigma_h}{\sigma_h+\sigma_v}=\frac{\sigma_3}{\sigma_1+\sigma_3}$$

Sand	$K_0 = 1-\sin\phi$
OCS	$K_{0\,OCS} = K_{0\,NCS} \times \sqrt{OCR}$

OCS → overconsolidated soil
NCS → normally consolidated soil.

② active earth pressure :-

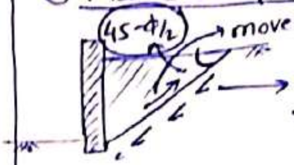


shear stress on soil block on the verge of failure

- when wall moves away from soil then the soil has tendency to move downward and outward. The pressure exerted by soil on the wall when the soil is on the verge of failure is called active earth pressure.

$$K_a=\frac{1-\sin\phi}{1+\sin\phi}=\tan^2(45-\phi/2)=\frac{1}{\tan^2(45+\phi/2)}$$

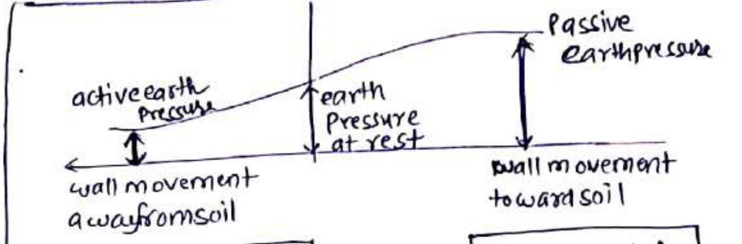
③ Passive earth pressure :-



→ shear stress on soil block on verge of failure.

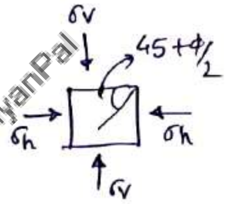
- Pressure developed on wall when wall moves towards soil

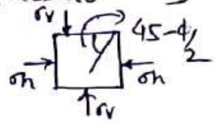
$$K_p=\frac{1+\sin\phi}{1-\sin\phi}=\tan^2(45+\phi/2)\qquad K_a\cdot K_p=1$$



ΔH = 0.2 − 0.5% of H	ΔH = 5 to 15% of H
dense sand, loose sand	dense sand, lose sand

movement of wall required for generation of active & passive earth Pressure.





v.v Imp :-

After compaction, due to wall friction, due to cohesion

- Active earth pressure ↓ decreases
- Passive earth Pressure ↑ increases.

Rankine Theory :-

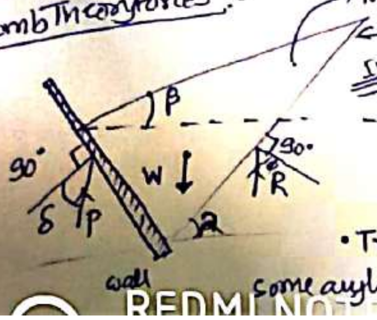
Imp: ① overestimate active earth pressure. **
② underestimate passive "

- Rankine Theory considered stress in soil-mass when it attains plastic equilibrium.
  { Plastic eqb → every point in soil mass experience shear failure under the effect of shear stress developed. }

assumption / points

Rankine Theory	Coulamb Theory
elemental failure	wedge failure
Soil → semi infinite, Homogeneous, Isotropic, dry, cohesionless (later extend for cohesive soil, submerge soil)	✓ ✓ ✓ ✓ ✓
Back of wall → vertical	→ can Inclined
Back fill → Horizontal	→ Can Inclined
wall → smooth (frictionless)	→ friction (wall + soil)
rupture surface → planar surface which is obtained by considering Plastic equilibrium of soil	→ failure plane is assumed to be a plane surface (actualy curved.
• soil is in state of plastic condition at time of active & passive pressure generation.	• sliding wedge assumed to be a rigid Body. • Position & line of action of earth pressure will be known in advance

Columb Theory forces :-



Imp trial wedge
P, R, W → direction of these are known. (magnitude unknown)

• Trail wedge assumed at some angle (a) from horizontal.

note:- **Imp.** effect of live load considered in retaining wall. → Culman's method (rupture occure in plane)

Imp.

Rankine Theory valid for	coulamb Theory valid for
• Cantilever retaining wall	• gravity retaining wall
• counterfort retaining wall	• semigravity "

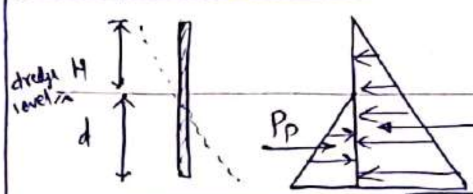
(RCC)

Imp note:- concrete retaining wall → Rankine Theory not valid (∵ friction exist)

Anchor sheet pile :-

advantages :- ① depth of penetration reduce
② Height to support increases ↑
③ we can use light section.
{ Less BM and Deflection }

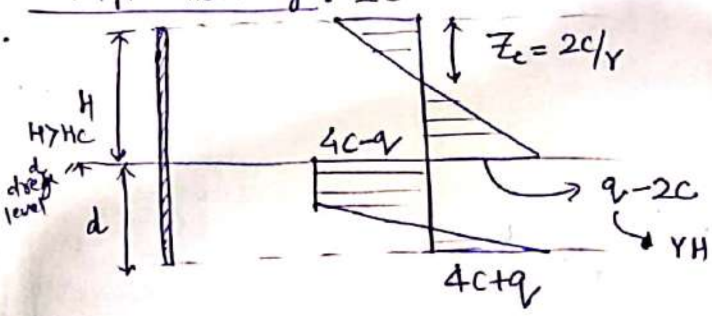
Sheet pile walls in sand :-



$P_p \times \frac{d}{3} = P_a \left(\frac{H+d}{3}\right)$

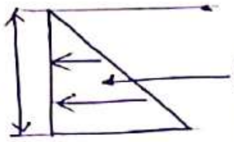
$FOS = \frac{P_p}{P_a} = \frac{\frac{1}{2} k_p \gamma d^2}{\frac{1}{2} k_a \gamma (H+d)^2}$

Sheet pile walls in clay :- 2C



Cases : → very important : {cohesionless soil}

①

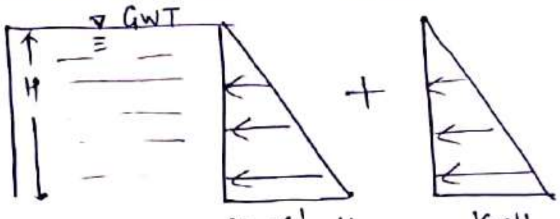


earth pressure / Pressure force

$P_a = \left(\frac{1}{2} \times K_a \gamma H \times H\right) \times 1$ (KN/m) (per unit of wall length)

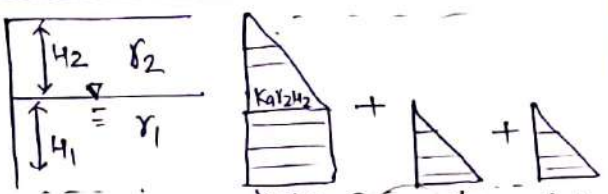
$p_a = K_a \gamma H$ → KN/m2 → earth pressure intensity

②



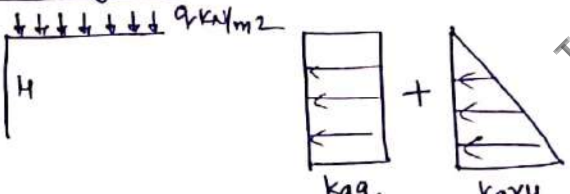
$K_a \gamma'_{sub} H$ $\gamma_w H$

③



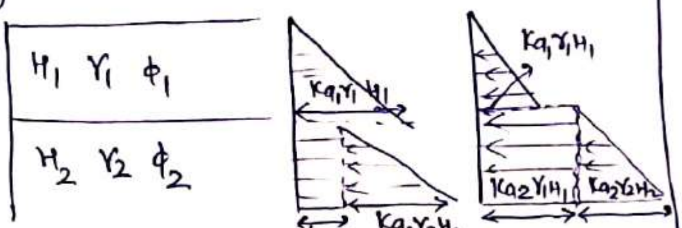
$K_a \gamma_2 H_2$ $K_a \gamma'_1 H_1$ $\gamma_w H_1$

④ surcharge case :-



$K_a q$ $K_a \gamma H$

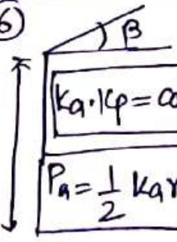
⑤



$K_{a2}\gamma_1 H_1$ $K_{a2}\gamma_2 H_2$

$\phi_1 < \phi_2$, $K_{a1} > K_{a2}$	$\phi_1 > \phi_2$, $K_{a1} < K_{a2}$

⑥



$K_a \cdot K_p = \cos^2\beta$

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

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

$K_p = \frac{+}{-}$

⑦

$\tan\alpha = W / P_{av}$

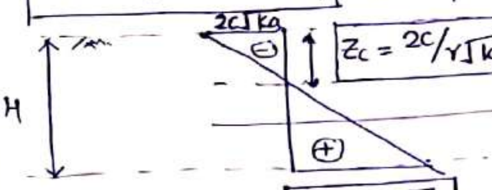
$P_{av} = \frac{1}{2} K_a \gamma H^2$

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

Active Pressure for clays :- {cohesive soil}

① when no Tension crack developed.

$p_a = K_a \gamma Z - 2c\sqrt{K_a}$ at any depth 'Z' active earth pressure intensity



$Z_c = 2c / \gamma\sqrt{K_a}$

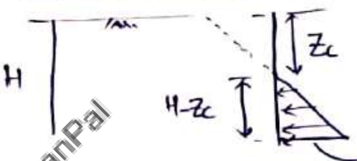
$H_c = 2Z_c = \frac{4c}{\gamma\sqrt{K_a}}$ → critical height

$H - H_c$

$K_a \gamma H - 2c\sqrt{K_a}$

$P_{a1} = \left[\frac{1}{2} K_a \gamma H^2 - 2c\sqrt{K_a} H\right]$ → case 1

② when Tension crack developed :-

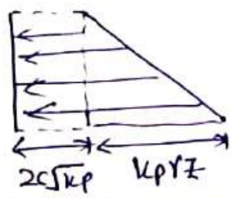


$K_a \gamma H - 2c\sqrt{K_p}$

$P_{a2} = \left[\frac{1}{2} K_a \gamma H^2 - 2c\sqrt{K_a} H\right] + \frac{2c^2}{\gamma}$ ** → more pressure than case 1.

Passive earth pressure for clay :- {cohesive soil} :-

$P_p = K_p \gamma Z + 2c\sqrt{K_p}$ at any depth 'Z' passive earth pressure intensity



$2c\sqrt{K_p}$ $K_p \gamma Z$

Shallow foundation :

As per Terzaghi

$\frac{D_f}{B} \leq 1$ ⟶ shallow foundation

transfer load at smaller depth

$\frac{D_f}{B} > 1$ ⟶ deep foundation

foundation type

- Shallow foundation
  - Raft
  - footing
    - Isolated
      - Rectangular
      - square
      - circular
    - combined
      - Trapezoidal
      - Rectangular
    - strip
    - Strap
    - spread
- Deep foundation
  - Pile foundation
  - well/caisson

General requirement for foundation

(i) shear failure criteria 'or' Bearing Capacity

(ii) Settlement criteria

(iii) location & Depth criteria

note :- generally for sand ⟶ settlement criteria

clay ⟶ shear strength or Bearing Capacity

General Definition :

(i) Gross pressure Intensity (q_g)

– total pressure at the base of footing (due to weight of super structure + self weight of footing + weight of earth fill)

(ii) Net pressure Intensity $q_{net} = q_g - \gamma D_f$ (or q_n)

(γD_f = overburden pressure, $\bar{\sigma}$ ↓ effective stress)

Imp :

note if raft is constructed to such a depth that $\frac{P}{B^2} - \gamma D_f = 0$ then soil is called upon not to resist any load, such raft is called fully compensated or floating raft

(3) Ultimate bearing capacity (q_u) :- max. gross intensity of loading that soil can support before it fails in shear.

(4) Net ultimate bearing capacity $(q_{nu}) = q_u - \gamma D_f$

min net pressure causing shear failure.

(5) net safe bearing capacity $q_{ns} = \frac{q_{nu}}{FOS}$

Imp [used to find net external safe load]

(6) safe bearing capacity or gross safe bearing capacity

$q_s = q_{ns} + \gamma D_f$ ($\gamma D_f \to \bar{\sigma}$)

note :- safe bearing pressure : (q_{ps}) max. net-intensity of loading that can be allowed on soil without the excess settlement than permissible

• no FOS ⟶ used when deal with settlement

note :- As per IS code :-

allowable bearing pressure ⟶ min { (i) net safe bearing capacity (q_{ns}), (ii) Safe bearing Pressure (q_{ps}) }

Imp.

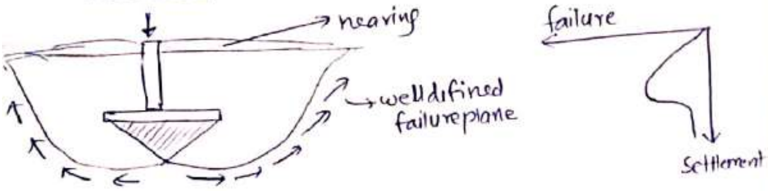
Determination of Bearing capacity of soil

- Analytical method (based on Shear failure criteria)
- Building code
- Field Test method
  - SPT
  - Plate load Test
  - SCPT

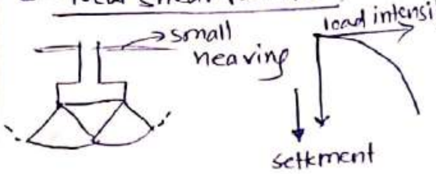
Analytical method :
3 modes of shear failure :

1- General shear failure : → sudden shear failure

- in soil having Brittle type shear stress curve.
- Dense sand, silt, overconsolidated clay
  { (soil of Low compressebility) (ϕ>36°) • soil relative density I_D > 70% }
- failure pattern → well defined
- Heaving (Bulging) of ground surface adjacent to foundation at both side,
- tilting of foundation (due to slip moment)
- plastic equilibrium reached



2- local shear failure :

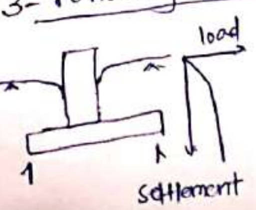


- not sudden
- only slight heaving
- partial development of state of plastic equilibrium
- no tilting

Imp
- Ex. loose sand with relative density (I_D = 30-70%)
- considerable compression of soil under footing.

Imp note: if ϕ ≤ 29° → local shear failure is assumed

3- Punching shear failure :-



- shearing in vertical direction around the edge of footing
- no heaving
- no tilting

Imp
Ex. in very loose sand (I_D < 30%), in deep footing or deep foundation.

Terzaghi's Bearing capacity Theory :- for (c-ϕ) soil

assumption : (I) Strip footing (L>>B) (2D plane strain condition prevails)

(II) Shallow foundation ($D_f/B \le 1$)

(III) Base of footing → rough (consider only base resistance, ignore side resistance)

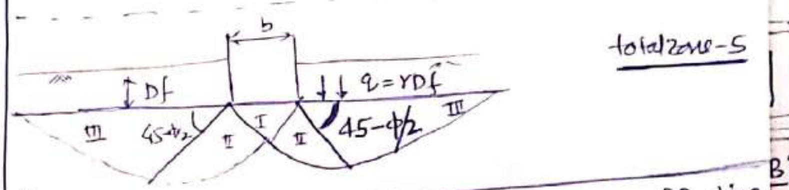
(IV) General shear failure occures

(V) Ground horizontal

(VI) loading is vertical & symmetrical (moment = 0)

(VII) shear resistance b/w ground surface and Base of footing is neglected. (thus footing considered as surface footing with uniform surcharge (γDf) at base of footing)

(VIII) shear strength govern by Mohr coulmb criteria



Imp
- failure plane → not extending above base of footing

Zone
- ① elastic equilibrium
- ② Radial shear zone
  - circular (c-soil)
  - log-spiral (c-ϕ soil)
- ③ Rankine's passive zone → (45-ϕ/2 from Horizontal)

for strip footing	$q_u = cN_c + qN_q + 0.5\gamma BN_\gamma$
Square	$q_u = 1.3cN_c + qN_q + 0.4\gamma BN_\gamma$
circular	$q_u = 1.3cN_c + qN_q + 0.3\gamma DN_\gamma$
Rectangular	$q_u = \left(1+0.3\frac{B}{L}\right)cN_c + qN_q + \left(1-0.2\frac{B}{L}\right)0.5\gamma BN_\gamma$

cN_c ⟶ due to constant component of shear strength of soil

qN_q ⟶ due to surcharge above footing (overburden)

$0.5\gamma BN_\gamma$ ⟶ due to bearing capacity due to self wt. of soil (effect of soil in shearing zone)

Bearing capacity factors (depends on ϕ)

$N_q = N_\phi e^{\pi \tan\phi}$ $\quad N_\phi = \tan^2(45+\phi/2)$ ⟶ Influence factor.

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

$N_\gamma = 1.8\tan\phi\,(N_q - 1)$

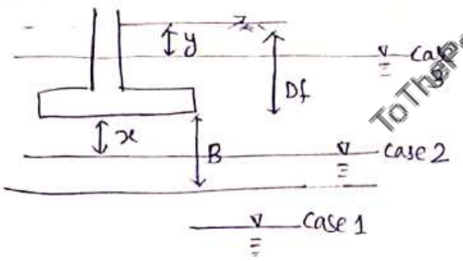
for c-soil (clay)	$N_c = 5.7$	$N_q = 1$	$N_\gamma = 0$

if local shear failure ⟶

$c' = \frac{2}{3}c$ $\qquad \tan\phi' = \frac{2}{3}\tan\phi$

Effect of water table :-

method-1 :-



case 1 :- when WT below depth $D \geq (D_f + B)$ (total) (मतलब B से ज्यादा नीचे)

no correction

case-2 :- $D_f \leq WT \leq D_f + B$

$q_u = cN_c + qN_q + 0.5\gamma_e BN_\gamma$ $\qquad \gamma_e = \frac{x\gamma_{bulk} + (B-x)\gamma'_{sub}}{B}$

case-3 :- $WT \leq D_f$

$q_u = cN_c + (\gamma_e D_f)N_q + 0.5\gamma_{sub} BN_\gamma$

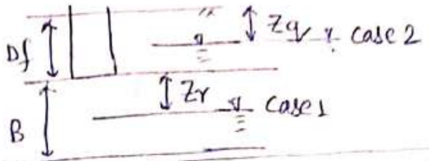
$\gamma_e = \frac{y\gamma_{bulk} + (D_f - y)\gamma_{sub}}{D_f}$

V. Imp note:- Bearing capacity of footing increases with increasing depth of ground water Table.

मतलब case 1 > case 2 > case 3

method-2 :- (water table correction factors)

$q_u = cN_c + qN_q R_q^* + 0.5\gamma BN_\gamma R_\gamma^*$



case-1 :- $R_\gamma^* = \frac{1}{2}\left[1 + \frac{Z_\gamma}{B}\right]$ $\qquad R_q^* = 1$

case 2 :- $R_q^* = \frac{1}{2}\left[1 + \frac{Z_q}{D_f}\right]$ $\qquad R_\gamma^* = 0.5$

note:- if water rise to ground level then $R_q^* = R_\gamma^* = \frac{1}{2} = 0.5$

Skempton's method (only for c-soil) :-

$q_{nu} = cN_c$ $\qquad c_u = \frac{q_u}{2}$

Strip footing $0 \leq \frac{D_f}{B} \leq 2.5$	$N_c = 5\left[1 + 0.2\frac{D_f}{B}\right]$
rectangular footing $0 \leq \frac{D_f}{B} \leq 2.5$	$N_c = 5\left(1+0.2\frac{D_f}{B}\right)\left(1+0.2\frac{B}{L}\right)$

{Put B = L for Square & circular footing}

for strip footing N_c	5 to 7.5
rect, square, circular	6 - 9

note:- failure surface assumed to go above foundation level

Some other method's to get bearing capacity

(i) Rankine methods (ϕ-soil)	$q = \gamma D_f K_p^2$ → $\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$ v.imp ∴ min depth as per rankine $= \frac{q}{\gamma K_p^2}$
(ii) Prandtl method (c-ϕ)	for c-soil $N_c = 5.14$, $N_q = 1$, $N_\gamma = 0$
(iii) meyerhoff method	meyerhoff investigated importance of shear resistance of soil lying above Base of footing. $q_u = cN_c S_c d_c i_c + qN_q S_q d_q i_q + 0.5\gamma B N_\gamma S_\gamma d_\gamma i_\gamma$ S → shape factor (for strip footing = 1) d → depth factor i → inclination correction factor
(iv) IS code method	
(v) Bells Theory	
(vi) Fellinius Theory	
(vii) Vesic "	
(ii) Hensen "	
(viii) Teng "	

Housel's approach :-

$$Q_f = mA_f + nP_f$$

$$Q_p = mA_p + nP_p$$

m, n constant

Q_p → allowable load on plate

P → perimeter

A_p → area of plate

A_f → area of foundation

Plate load Test :- (IS:- 1888–1992)

- only for cohesionless soil (sand)
- Short duration Test hence only result in immediate settlement.
- Used to calculate → q_u; → Settlement of foundation itself; → allowable bearing pressure corresponding to particular permissible settlement of foundation (Imp)
- circular/square bearing plate of mild steel used
  - Size = 30 – 75 cm. (dia or dimension)
  - thickness = 25 mm (min)
- Smaller size plate → for dense/stiff soil
- larger " → for Loose/soft soil

computation

- q_u
  - (i) clay: $(q_u)_f = (q_u)_p$
  - (ii) sand: $\frac{(q_u)_f}{(q_u)_p} = \frac{B_f}{B_p}$
- Settlement
  - clay: $\frac{S_f}{S_p} = \frac{B_f}{B_p}$
  - sand: $\frac{S_f}{S_p} = \left[\frac{B_f(B_p+0.3)}{B_p(B_f+0.3)}\right]^2$

Standard Penetration Test (SPT):

- Insitu Test, significant for granular (cohesionless soil) {which can not be easily sampled}
- Borehole must
- Obtained disturb sample by splitspoon sampler
- used to find
  - → ID (relative density) of cohesionless soil
  - → ϕ (depend on 'N' value)
  - → q_{ucs} of cohesive soil
- 65kg (Impact Load/hammer) having 75cm free fall
- N value

15cm	ignore
15cm	N1
15cm	N2

} $N_1 + N_2 = N$ ⇒ no. of blows required to penetrate the sampler to this 30cm is ⟶ SPT 'N' value.

Correction in SPT 'N' value :-

① Overburden correction :

$$N_1 = N_0\left(\frac{350}{\bar{\sigma}+70}\right) \quad \bar{\sigma} \not> 280\ kN/m^2$$

or

$$N_1 = N \times 0.77 \log\left(\frac{2000}{\bar{\sigma}}\right)$$

→ effective overburden pressure.

note:- 2 granular soil possessing the same relative density but having different confining pressure are Tested ⟶ The one with higher confining pressure will give higher N value.

note:- ∵ Confining pressure increases with depth, The N values at shallow depths are under estimated & N values at larger depths are over estimated.

② Dilatency correction :

- applied After 'N' values is corrected for overburden
- Imp• Correction required when $N_1 > 15$ in saturated fine sand & silt (ie when water table is above Test level)
- This correction becomes more significant for fine dense sand.
- note:- $N_2 = 15 + \frac{(N_1 - 15)}{2}$ $(N_1 > 15)$

↳ $N_1 > 15$ represent the dense sand which will have the tendency to dilate under rapid loading (undrain condition) and (-ve) pore water pressure will develop. hence observe N value will be more because shear resistance will increase.

Reason (assumption) :

fine sand & silt below water table offer higher resistance to driving due to the development of excess pore pressure which could not be dissipated immediately leading to apparent soil resistance gives higher 'N' value.

Imp note:- sometime Energy correction also applied (because of hammer efficiency) in SPT

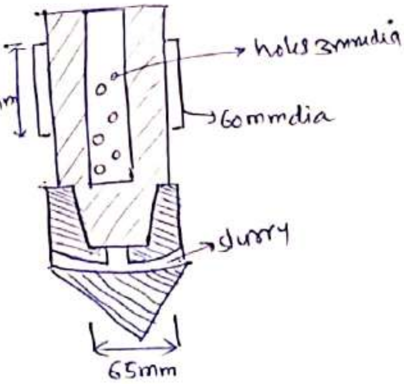
Static cone Penetration Test or Cone penetration Test (CPT) :-

Imp
- Simple Test widly used inplace of SPT

particularly for soft clay, silt, fine to medium sand deposit.

Imp.
Test performed to obtained a continuous record of soil resistance by penetrating a cone.

- area of cone = $10 cm^2$ apex angle → 60°
- The cone & sleeve are pushed into the soil at rate of 20 mm/sec upto a 100 mm. The resistance of soil offered to the penetration is recorded as cone penetration resistance.



Imp
note :- SCPT → not suitable for dense sand

Settlement of foundation :

$$S = S_{immediate\,(S_i)} + S_{1^{\circ}consolidation} + S_{2^{\circ}consolidation}$$

1- Immediate Settlement calculation :- (by Theory of elasticity.)

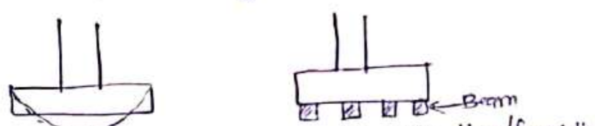
2- net elastic settlement for flexible surface foundation Based on theory of elasticity.

$$S_i = \frac{q_n B (1-\mu^2)}{E_s} \times I_f$$

S_i → immediate elastic settlement < sand, clay

q_n → net foundation pressure
B → width of foundation
μ → Poisson ratio
E_s → modulus of elasticity — By < Triaxial Test, field Test

imp. I_f → Influence factor which depend on shape & rigidity of structure.

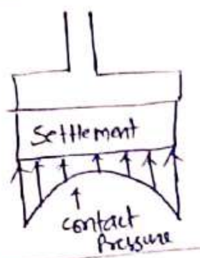
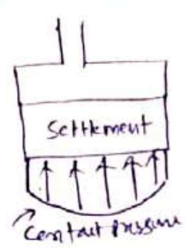
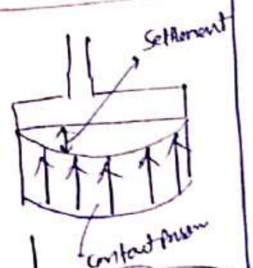
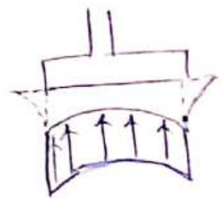


flexible footing — rigid footing/foundation

V.Vimp. Note**

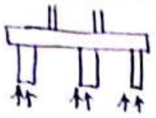
$$S_{rigid\,Immediate} = S_{flexible\,immediate} \times 0.80$$

V. imp.

foundation	clay	sand
rigid ↓ (settlement constant / uniform)		
flexible ↓ (Contact pressure ↓ uniform)	 ↓ dish shape	 • deflection more at edges (∵ E is less at edges)

Pile types based on mode of transfer of load

1- **end bearing pile** :- transmit the load through Bottom tip (through bearing action)



- Ultimate capacity of pile depend on bearing Capacity of rock.

2- **Friction pile** .. (in stiff clay **) load transfer by skin friction b/w embedded surface of pile & surrounding soil

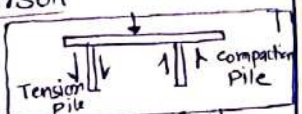
3- combined end bearing & friction pile

Pile classification Based on action/function :-

1- **load bearing pile** → transfer load of structure by end bearing or friction or both.

2- **Tension/uplift pile** :- (to resist uplift load)
- used to anchor str. subjected to uplift force due to hydrostatic pressure or overturning moment due to horizontal forces.

3- **Compaction pile (short pile)** :→ (not to carry load)
- used to compact loose granular soil to increase bearing capacity.



4- **Anchor pile** :. provide anchorage against horizontal pull from water or sheet piling.

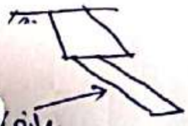
5- **fender pile** — Protect water front str against impact of ships & other floating object.

6- **sheet pile** :- used as bulkhead or cut off to reduce seepage & uplift in the hydraulic str.

7. **Batter pile** — resist lateral loads (horizontal + inclined load)
- especially in waterfront str.

↓

(Raking pile → inclined piles)



Pile classification Based on Installation :

1- **Driven piles** :- driven into position either vertical or inclined. (upto 500 mm) (batter / raking pile)

Common methods of pile driving :

① Hammer driving	ⓘ Drop hammer ⓘⓘ single acting hammer ⓘⓘⓘ disel hammer ⓘⓥ double acting hammer
② vibratory pile driver	• Useful only for sandy, gravely soil
③ Jetting Technique	• water under pressure is discharged at pile bottom
④ Partial Augering method	• Batter pile (Inclined piles) are usually advanced by this method.

Imp note :- Drop on pile < min = 120cm, max = 240cm

2- **Bored & cast insitu** :- (Bore pile dia ⇒ 2 to 3 m)
- only concrete piles can be cast insitu, as holes are drilled & filled with concrete

3- **Driven & cast insitu** :. (Ex. Fraki piles)
- a closed end casing / shell is driven into ground, later casing filled by concrete.

4- **Jack piles** :. driven in soil by Hydraulic Jack

5- **Screw pile** :- Screwed in soil by means of hydraulic Jack.

Pile classification Based on displacement of soil :

1- displacement pile : during installation if a large volume of soil is displaced laterally/upward.

Ex. in loose sand :- such pile densify the sand upto distance of (3.5xpiledia).

This compaction leads to increase in shear-resistance within the zone of Influence.

Ex. in clay :- large displacement of pile-remoulds the soil to a distance = (2xpiledia)

2- nondisplacement pile :- during Installation no heaving, no noise, no vibration.

- In such pile voids are formed in the soil by boring / Excavation and then these voids are filled with concrete.

Special Topic :-

free vibration with viscous damping

$$\lambda_{1,2} = -\frac{c}{2m} \pm \sqrt{\left(\frac{c}{2m}\right)^2 - k/m}$$

$\lambda \rightarrow$ arbitrary constant

$c \rightarrow$ damping cofficient (kN-s/m)

$m \rightarrow$ mass of rigid body

$k \rightarrow$ spring constant

Critical damping cofficient (C_c) ⇒

when $\left(\frac{c}{2m}\right)^2 = \frac{k}{m}$ {to make squareroot term = 0}

$\therefore c \longrightarrow C_c$

$$C_c = 2\sqrt{km}$$ **

$$\text{damping ratio } \xi = \frac{c}{C_c} = \frac{c}{2\sqrt{km}}$$

Pile load Test

- **Initial Test**
  - to assess the allowable load or check settlement at working load.
- **Routine Test** (load = 1.5 x design load)
  - carried on working piles for assessment of settlement under working load.

Imp.
- Pile load Test → useful in (sand, clay) but in clay result affected by disturbance caused by pile driving, increase in pore-water pressure and sufficient time is not given for consolidation.

Imp.
- Test pile → especially bored for purpose of conducting the Test and will not be part of foundation in future.

- IS code :- > 200 piles
  - Initial Test: min piles = 2
  - Routine Test: min = 0.5 – 2% piles

V.V.Imp :- As per IS :-

Safe load on single pile for Initial Test

min of:
- 50% of load
  - $\frac{1}{10}$th of pile dia (10% of pile dia)
  - 7.5% of Bulb dia (underream pile)
- $\frac{2}{3}$rd of load → total settlement = 12mm
- $\frac{2}{3}$rd of load → total plastic settlement = 6mm

V.Imp

Safe load on Group of piles for initial Test

min of:
- Final load → total settlement = 25mm
- $\frac{2}{3}$rd of load → Settlement = 40mm

Types of pile load Test :-

① Vertical load Test 'or' Compression load Test	• used to do intial & routine Test • to establish load settlement - relationship and to determine allowable load on pile.
② latral load Test	• to determine safe lateral load on pile
③ pull out Test	• to determine safe tension on pile
④ constant rate of Penetration Test (0.25 – 5 mm/min)	• to determine Ultimate load capacity of pile. (Ultimate load determine by load – settlement curve drawn) • In this Test the load on Test is continuously increased to maintain a constant rate of penetration 0.25-5 mm/min
⑤ Cyclic load Test (Initial Test) Final load = 2 x design load (1.5-)	• to determine skin friction & end bearing separately on single pile. • load applied in the increaments 20% of estimated safe load. • measurement of settlement by dial gauge • each Increament of load is maintained till the rate of settlement is 0.25mm per hr. and final load is maintained for 24 hrs.

Requirement of pile foundation
- Shear failure
- settlement

in both → ensure safe.

Pile load capacity methods
- Static pile load formula
- Pile load Test
- pile driving formula
- co-relation with penetration data.

Analytical method :-

$$Q_{up} = Q_{eb} + Q_{sf} = q_b A_b + q_s A_s$$

Q_{eb} → due to end bearing; Q_{sf} → due to skin friction

{ although this eqn is not correct when max bearing is developed friction reduced from its max. value }

for clay :-

$$Q_{up} = 9 C A_b + \alpha \bar{C} A_s$$

C → unit cohesion at base of pile; A_b → bearing area; $\bar{C}$ → avg. cohesion over depth of pile; α → adhesion factor; A_s → surface area

($N_c = 9$) as per skempton for deep foundation

Sand :-

$$Q_{up} = q_s A_s = \left[\frac{1}{2} \bar{\sigma} K \tan \delta\right] A_s$$

$\bar{\sigma}$ → γ'ℓ

REDMI NOTE 5 PRO
MI DUAL CAMERA

Dynamic approach :-

↳ Based on Penetration resistance imparted to Pile driving.

1- Engineering News Record formula :-

$$Q_{allowable}\ (Q_{ap}) = \frac{Q_{up}}{(FOS=6)} = \frac{WH}{6(S+C)}$$

W → load kg
H → cm (fall)

C
- 2.5cm → drop hammer
- 0.25cm → steam hammer (single acting or double acting)

Imp: S → Settlement per blow (final set per blow)
- last 5 blows of drop hammer
- last 20 blows of steam hammer

Steam hammer
- Single acting: $Q_{ap} = \frac{WH}{6(S+0.25cm)}$
- double acting: $Q_{ap} = \frac{(W+ap)H}{6(S+0.25)}$

a → area of hammer on which pressure act
p → steam pressure

2- modified Hilley formulae :-

$$Q_{ap} = \frac{\eta_h \eta_b WH}{3(S+C/2)}$$

(numerator over $(S+C/2)$ → Q_{up}); FOS=3

η_h → efficiency of hammer
- drop ⇒ 1
- single/double ⇒ 0.75–0.8

η_b → efficiency of blow
- W > ep: $\eta_b = \left(\frac{W+e^2p}{W+p}\right)$
- W < ep: $\eta_b = \left(\frac{W+e^2p}{W+p}\right) - \left(\frac{W-ep}{W+p}\right)^2$

W → hammer weight (kg)
P → pile gross weight (wt of pile + pile cap)
e → coff. of restitution (0.25–0.55)
S → final set per blow
C → total elastic compression of pile + pile cap + soil
H → height of fall of hammer.

Requirement of Pile foundation — Shear failure, settlement — in both → ensure safe.

Pile load capacity methods:
- Static pile load formula
- Pile load Test
- Pile driving formula
- Co-relation with penetration data.

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($\bar{\sigma}$ → $\gamma' l$)

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2- modified Hilley formulae :-

$$Q_{ap} = \frac{\eta_h \eta_b W H}{3(S+C/2)}$$

FOS = 3 (numerator over $(S+C/2)$ → Q_{up})

η_h → efficiency of hammer: drop ⇒ 1; single/double ⇒ 0.75–0.80

η_b → efficiency of blow:
- $W > ep$: $\eta_b = \left(\frac{W+e^2p}{W+P}\right)$
- $W < ep$: $\eta_b = \left(\frac{W+e^2p}{W+p}\right) - \left(\frac{W-ep}{W+p}\right)^2$

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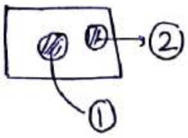
C → total elastic compression of pile + pile cap + soil
H → height of fall of hammer.

Group action of pile :-

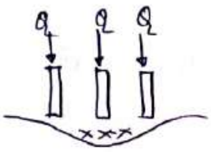
① min piles req. for group

- Bored pile: min = 1 {∵ verticality can be ensured}
- driven pile: min – 3 [∵ uncertainty regarding vertical installation]

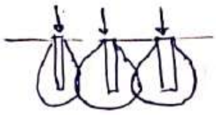
② to avoid ground tightening pile in sand begin at centre & moved outer.



③ point bearing piles



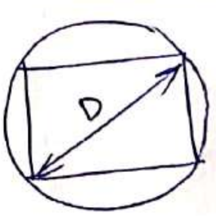
④ friction piles



Imp

min pile spacing As per IS	2 x dia	In loose sand & fill deposit
	2.5 x dia	In end bearing pile
	3 x dia	friction pile

note :- noncircular pile :-



D(dia) → dia. of circumscribed circle

Ultimate capacity of pile group → min { (I) Based on block failure; (II) Individual pile failure }

efficiency of pile group :-

① $\eta_g = \frac{Q_{ug}}{n Q_{up}}$

- cohesionless soil
  - loose / medium dense sand: $\eta_g > 1$ {∵ soil around and b/w the piles gets compacted due to vibration caused during the driving operation}
  - dense sand: $\eta_g < 1$
- cohesive soil: $\eta_g < 1$

② group efficiency by Converse–Labarre formula :-

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

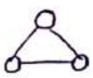
$$\phi = \tan^{-1}(d/s)$$

m → no. of rows, n → no of column
d → dia pile, s → c/c pile spacing

③ Feld's rule (group efficiency) :-

- reduces the capacity of each pile by 0.0625 for each adjacent pile. (spacing of pile not considered)

Ex.



↓ 3 piles in a group

$\eta_g \Rightarrow 1 - (0.0625 \times 2) = 0.875$
$= 87.5\%$

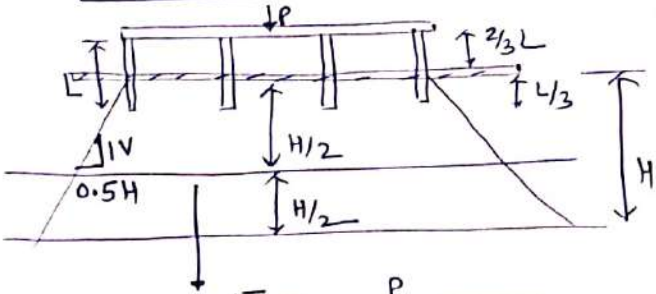
Settlement of pilegroup :-

- Generally settlement of Indivisual pile is more than group . { sameloading per pile }

1- Settlement of pile group in clay :-

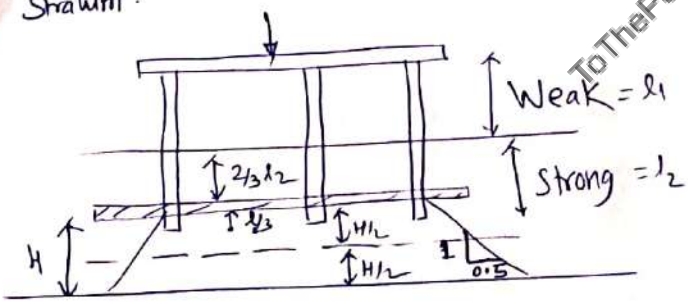
(by equivalent raft method)

$$\Delta H = H_0 \frac{C_c}{1+e_0} \log \left(\frac{\bar{\sigma_0} + \Delta\bar{\sigma}}{\Delta\bar{\sigma_0}} \right)$$

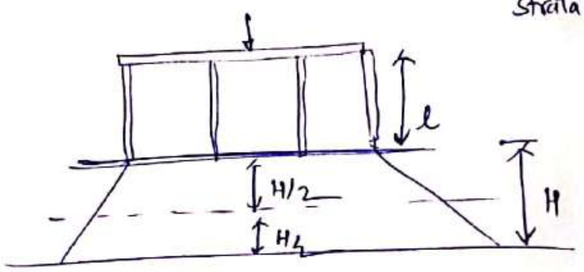


$$\Delta\bar{\sigma} = \frac{P}{(B+2nz)(L+2nz)}$$

Special case-1 :- when piles are driven into strong stratum through an overlying weak stratum.



Specialcase-2 :- Bored pile /end Bearing pile / resting on firm Strata.



2- Settlement of pile group in (sand) :-

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

$B \rightarrow$ size of pile group in meter.

23/3/2020

Expansive / Swelling soil → Black cotton soil
{ soil → montmorillonite mineral }

Identification of Expansive soil (Based on swelling potential)

- Free Swell Test
- Differential Free Swell Test
- Swelling Test
- Swelling Pressure Test

- Plasticity Index
- Shrinkage limit
- Colloidal content

Lab Test
1. Differential Thermal analysis (DTA)
2. 2° X-ray diffraction method
3. 3° electron microscopy.

$$DFS(\%) = \frac{\text{Soil volume in (water)} - \text{Soil volume in (kerosine)}}{\text{Soil volume in (kerosine)}} \times 100$$

DFS %	degree of expansiveness (swelling potential)	(Ip) PI
0 – 20	low	0-15
20 – 35	moderate	to 35
35 – 50	high	
> 50	very high	

Imp.

note :- Shallow foundation → not advised in Soil having High & very high (DFS)

1- Free swell Test :-

425μ (sample) 10 cm³ dry soil → 100 cm³ graduated cylinder (soil + water)

- measure volume of swelled soil after 24 hrs.

$$\text{free swell } \% = \frac{V_f - V_i}{V_i} \times 100$$

note :- free swell Test not adequate to predict accurate swelling characteristics.

2- Differential free Swell Test (DFS Test) :-

(2 sample)

- 10 cm³ dry soil (425μ–) → 50 cm³ → nonpolar solvent (Kerosine)
- 10 cm³ dry soil (425μ–) → 50 cm³ → distill water

- measure volume after 24 ...

③ Swelling Test :-

{ volume change measure in different situations }

at time of construction choose 2 identical undisturb sample

- undisturb Sample-1 → dry → moisture content < shrinkage limit → measure volume by mercury displacement method.
- undisturb Sample-2 → loaded at Pressure = Structure load → allow free access to water → at eq^b, measure volume

with help of these 2 data, estimate can be made for volume change of soil in – different field situation.

④ Swelling Pressure Test :-

Imp. (done in Oedometer)

↓

{ and requires continuous adjustment of Soil pressure on specimen such that volume of Specimen remains same throughout the Test }

Swelling pressure :- Pressure required to be applied over swelling soil specimen to prevent its expansion when it comes in contact with water.

- Swelling pressure does not have unique value (It varies)
- Swelling pressure depends →
  - initial moisture content
  - initial γ_d
  - method of compaction
  - confining surcharge
  - height of soil specimen
- if swelling pressure < 20 kN/m2 indicate low degree of expansiveness hence shallow foundation can be used.
- Some soil of Bentonite swelling pressure ≈ 200 kN/m2 order

⑤ Plasticity Index (PI) (Ip) / shrinkage limit & colloidal content :-

Swelling potential ∝ Ip ($\omega_L - \omega_p$) ∵ Ip↑ ⇒ more water absorbed by soil hence Swelling↑
low shrinkage limit ⇒ swelling starts at low water content.
Higher colloidal content ∝ High swelling

note:- Imp: Expansive soil compacted at wet side of optimum.

note:- ① overconsolidated or Highly compacted soil has high tendency of swelling when it comes with water.

② Imp. Expansive soil → (PL) → plastic limit; (SL) → shrinkage limit; (LL) → Liq. limit; Volumetric shrinkage

Design of foundation on swelling soil :-

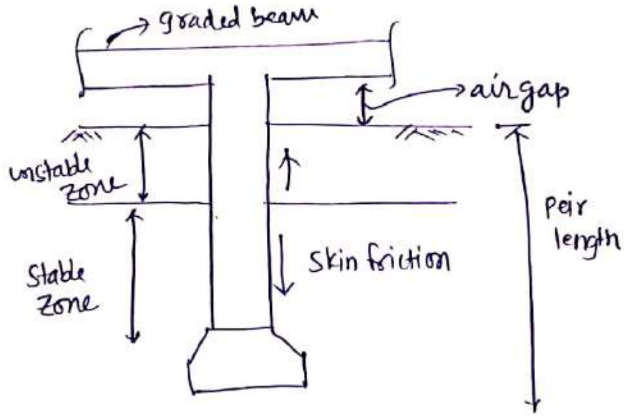
1- Strong & rigid structure.

2- flexible str. → so that they change their shape as per swelling of soil (means effect of differential swelling is not felt)

3- Isolating foundation :.

{ Deep foundation < Belled peirs, underreampiles; construct to isolate the foundation from swelling effect of soil }

- Sometime a graded beam is provided at the top of Belled peir & under reampile
- an air gap is provided b/w graded beam and ground surface to permit swelling of soil.



4 - Preventing swelling :- By providing an Impervious Apron. The moisture gradient b/w the centre of str. and its edges is minimized, hence differential swelling is controlled.

Elimination of possibility of swelling :-

1- Prewetting the soil mass to moisture content = equilibrium moisture content.

2- provide large enough external load which exceeds swelling pressure.

3- chemical stablization with lime :

∴ LL↓ → swelling potential ↓↓

PL↑

lime req ⇒ 3-8% for black cotton soil

Soil stablization :-

by which strength & stability of soil mass in improved & increased.

Imp :-

(I) Soil stablization with lime :-

- for high plastic soil like black cotton soil (expansive soil)
- lime required for stablization = 3 to 8 % of expansive soil

Imp. fine clay particles reacts with lime and get floculated or aggregated into larger particle group which are fairly stable under subsequent soaking.

- lime stablization leads to

(I) LL↓ PL↑ SL↑

(II) Reduction in swelling

(III) Reduction in Ip (Plasticity index)

(IV) reduction in max. dry density

(V) floculation of clay particles.

(II) Soil stablization with cement : (Soil cement) :-

Gravel	5-10 %
sand	7-12 %
silt	12-15 %
clay	12-20 %

(III) Soil stablization with Bitumen :

- when Bituminous material added to soil, it imparts both cohesion & reduced water absorption.

(IV) chemical stablization of soil :

(V) electrical stablization of clay soil :

(VI) soil stablization by grouting :

(VII) Soil stablization by geo textile & fabrics.

Soil Exploration

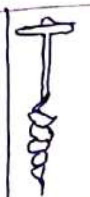
Stage-1 :- Preliminary stage :-
(Reconnisance, geological study)

Stage-2 :- Detailed stage.
(Boring and detailed sampling done)
(• nature, thickness, sequence of subsoil layers, their lateral variations, position of water table)

Boring of hole → making & advancing of bore hole is known as boring
It is 1st step of collection of sample.

method of Boring :-

① Auger Boring



• for small depth of exploration
Ex. shallow foundation
highway
Borrow pits

Imp • done in partially saturated → sand, silt, medium to stiff clay

• Highly disturb sample got
∴ used for only Identification purpose only

② wash Boring
• for All type of soil except hard & cemented soil rock
• slurry (soil water mix) → will tell soil type
• change in soil strata → determine by rate of progress and slurry coming out.
• sample → Highly disturb ∵ soil water mix hence no value

③ Percussion boring
• Best for Bouldery & Gravelly stratum.

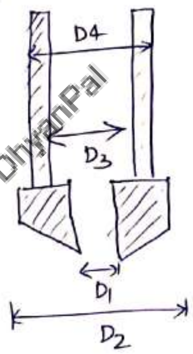
④ Rotary Boring
↓
large dia bore hole ..mm
• All type of soil & rock except in stoney or porous soil and fissured rock.
• usefull in soil highly resistance to auger & wash boring { dense sand & stiff clay }

Soil samples

① Disturb sample :- natural soil str. get modified but with precaution we can preserve
✓ natural moisture content, ✓ proportion of mineral constituent.
known as Representative sample.
↓
used for Identification purpose.

representative samples required → Trick (PCS)
① Particle size distribution ② consistency limit
③ specific gravity

② undisturb sample :- original soil structure, mineral properties → not changed
usefull in (PCS)
① Permeability ② consolidation parameter (C_v, m_v)
③ shear strength parameter (c-ϕ)
④ water content ⑤ density



D_1 → Inside dia of cutting edge
D_2 → outside
D_3 → Inside dia of sample tube
D_4 → outside

inside clearance (soil enters in tube) $C_i = \frac{D_3 - D_1}{D_1} \times 100$

$1 < C_i < 3$

• to reduce friction b/w soil sample & sampler

outside clearance
$C_o = \frac{D_2 - D_4}{D_4} \times 100$

$0 < C_o < 2$

↓ when sample driven

recovery ratio (L_r)

$L_r = \frac{\text{recovered length of sample}}{\text{penetration length of sampler}}$

Imp
$L_r > 1$ → good recovery
$L_r < 1$ Sample compressed
$L_r > 1$ Swelled

area ratio :-
$A_r = \frac{D_2^2 - D_1^2}{D_1^2} \times 100$

undisturb $A_r < 10\%$
Split spoon $A_{r\,max} = 30\%$
$A_r < 10\%$ for soft clay
$A_r < 20\%$ for stiff clay.

Imp note:- thin wall sampler
⇒ thickness < 2.5% of dia.

Types of sampler :

note:- Thinner the sampler wall → lower the degree of disturbances of collected soil-sample

Imp: order of disturbance (PCS-R) :- Piston < Chunk < Split spoon < remoulded sampler

Piston → min disturbance; remoulded sampler → max. disturbance.

Sampler Type :

① open drive sampler

open drive sampler

- thin wall sampler ($A_r < 10\%$) → to get undisturb sample.
- thick wall sampler ($A_r = 6-25\%$) → to get representative samples. (disturb)
  - such sampler can not penetrate in gravelly soil
  - whereas in too soft or too wet soil cut soil sample can not retained.

② Piston Sampler

- usefull in sampling of saturated sands and soil too soft too wet, which can not be sampled by open drive sampler.

③ Rotary Sampler

- double walled tube sampler
- in hard cohesive soil & rock.

note:- rock quality can be estimated by using the core-recovery ratio term as Rock Quality designation (RQD).

Subsurface Investigation (Field Test)

- Geophysical methods
  - Seismic refraction method
  - electrical resistivity method
- Penetration Test
  - SPT
  - SCPT
  - DCPT
- Pressuremeter Test
- Vane shear Test
- plate load Test

1- Seismic Refraction method :-

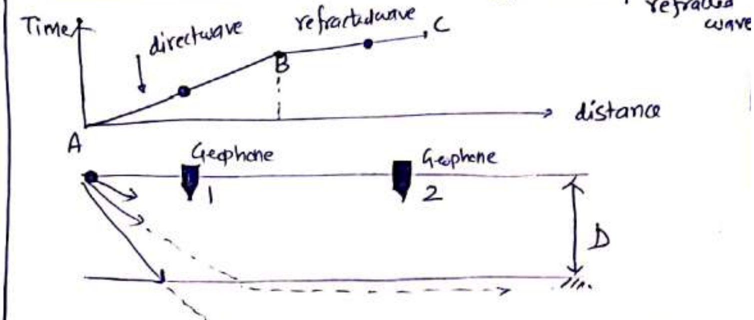
$$D = \frac{d}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$

D = depth of strata

d → distance b/w 2 station

V_1 → velocity of direct wave

V_2 → velocity of refracted wave



Principle :- elastic shock wave has different velocity in different material.

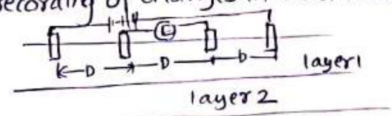
Limitation :- ① can not use for frozen surface layer

② can not use for areas covered by concrete, ashphalt, pavement having high seismic velocity.

③ used where soft layer lies over hard layer.

② electrical resistivity method :

Based on measurement and recording of changes in mean resistivity of various soil.

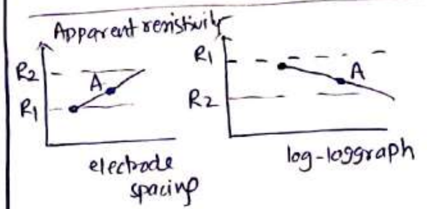


apparent resistivity ρ (ohm-m) $= \frac{2\pi D V}{I}$

D → dist b/w spikes

V → potential diff (volt)

I → current (amp)



resistivity = resistance b/w opposite face of unit cube of material.