Course No CE 341 (Geotechnical Engineering)

Ref: * Foundation Engineering
- Peck and Hanson.

* Soil Mechanics
- Craig

* Soil Mechanics
- BM Das.

Syllabus:

I. Weight volume Ratio.

II. Desity and soil compaction.

III. Fluid Flow through porus medium / Hydraulic Properties of soil.

IV. Consoledation, characteristic of soil.

V. Stress Distritution in soil.

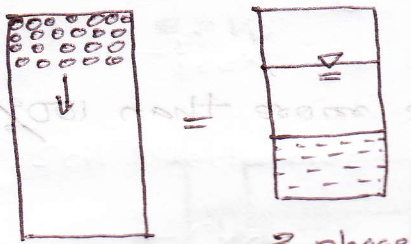
Soil Mechanics.

Soil mechanics is a branch of mechanics which deals by the action of force of soil and flow of water in soil

By Turzaggy:

" Soil mechanics is the application of laws of mechanics and hydraulics to engineering problem dealing with sediment and other unconsoledation accumulation of solid particles produced by mechanical and chemical disintegration of rock regardless~~ness~~ whether or not they contain admixture of organic content.

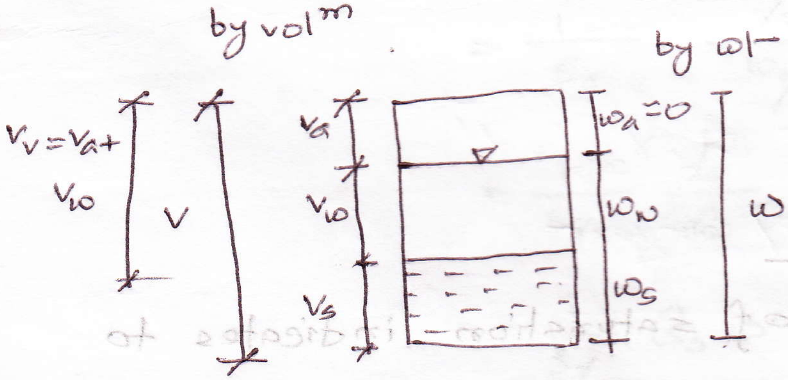
lecture -02
Monday
12-9-2011



3-phase
block diagr

Block Diagram

The diagramic representation of real soil into its elements (soil skeletont, air, water) is called phase diagram which is extremly using useful for studying various terms like void ratio, porosity, void content, degree of saturation, % air content, relative density, moisture content used in soil engineering and their interelationship. infact the phase diagram provides a convenient means of developing wt volume relationship of soil.



Moisture content : (w)

$$\% \quad w = \frac{W_w}{W_s}$$

soil moisture content may be more than 100%

ii) Void Ratio (e)

$$e = \frac{V_v}{V_s} \quad \text{[expressed in fraction]}$$

Value may more than 1.

iii) porosity: (n) [percentage void]

$$\% \quad n = \frac{V_v}{V} \quad \text{[expressed in \%]}$$

can not be more than 100%

$$n = \frac{V_v}{V} = \frac{V_v}{V_v + V_s} = \frac{V_v/V_s}{\frac{V_v + V_s}{V_s}} = \frac{e}{1+e}$$

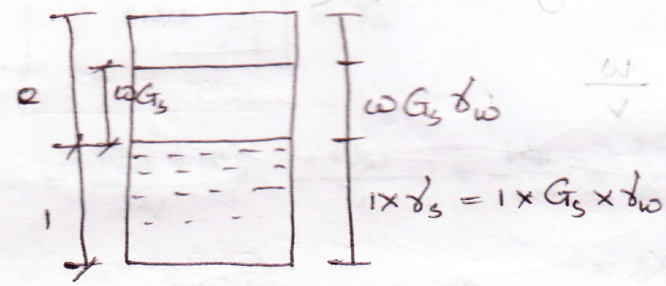
$$\therefore \boxed{n = \frac{e}{1+e}}$$

iv) Degree of saturation

physicall Degree of saturation indicates to what extent the void spaces of a soil sample are filled up with water.

$\% \; S_r = \frac{V_w}{V_v}$ (expressed in percentage)

if $V_s = 1$, $e = V_v$



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

$$\therefore W_w = w \times W_s = w \times G_s \gamma_w$$

$$\text{So,} \quad S_r = \frac{V_w}{V_v} = \frac{w G_s}{e}.$$

other method,

$$e = \frac{V_v}{V_s} = \frac{V_v}{V_w} \cdot \frac{V_w}{V_s}$$

$$= \frac{1}{S_r} \quad \frac{W_w/\gamma_w}{W_s/\gamma_s}$$

$$= \frac{1}{S_r} \frac{\gamma_s}{\gamma_w} \frac{W_w}{W_s}$$

$$= \frac{1}{S_r} G_s w .$$

For fully saturated soil sample $S_r = 1$ $\quad e = w G_s$

$= w_{sat} G_s$

Void ratio of coarser materials are generally smaller than finer materials

Unit weight:

i) Bulk unit weight, γ_{bulk}

$$\gamma_{bulk} = \frac{W}{V}$$

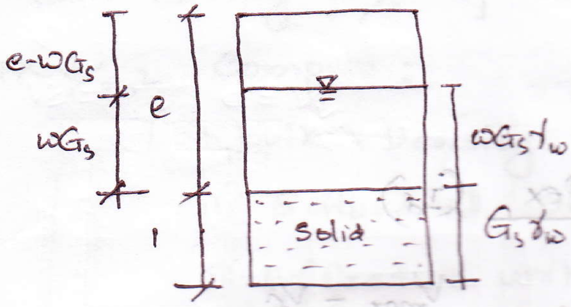
ii) Dry unit wt, γ_D

$$\gamma_d = \frac{\gamma V_s}{V}$$

iii) saturated wt, γ_{sat}

$$\gamma_{sat} = \frac{W_{sat}}{V}$$

iv) Submerged unit wt, $\gamma_{sub} = \gamma'$
(effective unit wt / buoyant unit wt)



$$\gamma_{bulk} = \frac{W}{V} = \frac{W_w + W_s}{V} = \frac{W_w}{V} + \frac{W_s}{V} = \frac{W_w}{W_s} \cdot \frac{W_s}{V} + \frac{W_s}{V}$$

$$= \omega\gamma_d + \gamma_d = \cancel{\omega}(1+\omega)\gamma_d .$$

So, $\boxed{\gamma_{bulk} = (1+\omega)\gamma_d}$

again, $\gamma_{bulk} = \frac{W_w + W_s}{V} = \frac{\omega G_s \gamma_w + G_s\gamma_w}{1+e} = \frac{G_s\gamma_w (1+\omega)}{(1+e)}$

when, $\omega = 0$, $\gamma_d = \frac{G_s\gamma_w}{1+e}$

Now, $\gamma_{sat} = \frac{W_{sat}}{V} = \frac{e\gamma_w + G_s\gamma_w}{1+e}$

From, $\gamma_{bulk} = \frac{G_s\gamma_w (1+\omega)}{(1+e)}$

$$= \frac{\omega G_s \gamma_w + G_s\gamma_w}{(1+e)}$$

$$= \frac{(S_r e + G_s)\gamma_w}{(1+e)}$$

$S_r = \frac{\omega G_s}{e}$

$\omega = \frac{S_r e}{G_s}$

$S_r e = \omega G_s$

$$\gamma_{submerged} = \gamma' = \gamma_b = \gamma_{sub} = \gamma_{sat} - \gamma_w$$

$$= \frac{G_s - 1}{1+e} \gamma_w$$

⊛ Relative Density / Desity Index (I_d)

$$I_d = \frac{e_{max} - e_f}{e_{max} - e_{min}} \times 100\% = \frac{V_{max} - V_f}{V_{max} - V_{min}} \times 100\%$$

Here, $\gamma_d = \frac{G_s \gamma_w}{1+e}$ $\therefore (1+e) = \frac{G_s \gamma_w}{\gamma_d}$

$$\therefore (1+e)_{max} = \frac{G_s \gamma_w}{(\gamma_d)_{min}}.$$

$$\text{So, } I_d = \frac{(e_{max}+1) - (e_f+1)}{(e_{max}+1) - (e_{min}+1)} \times 100\%$$

$$= \frac{\frac{1}{\gamma_{d min}} - \frac{1}{\gamma_{df}}}{\frac{1}{\gamma_{min}} - \frac{1}{(\gamma_d)_{max}}} \times 100\%$$

⊛ state of compaction Vs I_d

state of compaction	I_d
i) Very Lose	0-15
ii) Lose	15-35
iii) Medium	35-65
iv) Dense	65-85
v) Very dense	>85

Mathematical Problem

Soil having 16% of moisture content at dry unit wt of 17KN/m². Compute:

i. bulk Density

ii. Saturated Density

iii. Effective unit wt.

iv. Also compute the additional amount of water required to make soil sample fully saturated

Assume vol^m of soil sample 200m³

Sol^n

$$\gamma_d = \frac{\gamma_{bulk}}{1+w} \therefore 17 = \frac{\gamma_{bulk}}{1+\frac{16}{100}} \Rightarrow \gamma_{bulk} = 19.7$$

$$\gamma_d = \frac{G_s \gamma_w}{1+e} \therefore 17 = \frac{2.7 \times 9.81}{1+e} \therefore e = 0.56$$

$$\gamma_{sat} = \frac{G_s + e}{1+e}\gamma_w = 20.5 \text{ KN/m}^3$$

$$\gamma' = \frac{G_s - 1}{1+e}\gamma_w = 10.7 \text{ KN/m}^3$$

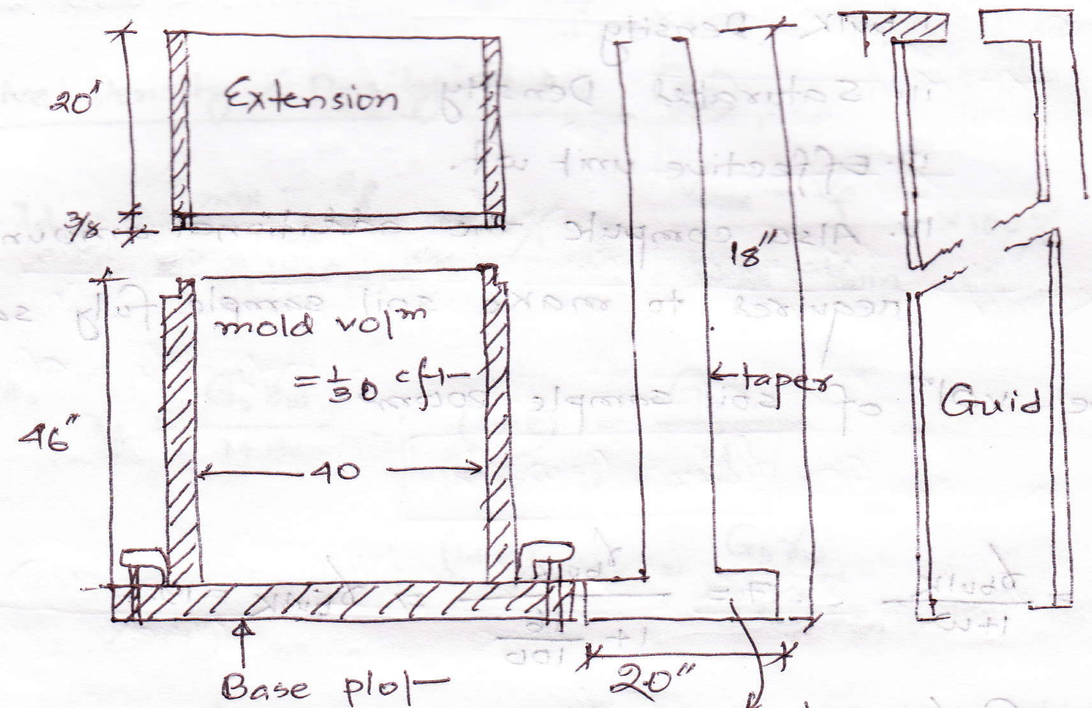
To saturate 1m³ of soil sample water needed $(20.5 - 19.7) = 0.8$

$\therefore$ 200 m³ " " " " " 0.8×200

$= 160$ KN

Lecture - 04

19-09-2011



Purpose of doing compaction

- It reduces compressibility and thereby the settlement of soil.

- It increases the shear strength of soil
- It reduces the permeability of soil

<u>Compaction</u>

Is the the process of densification of soil by giving mechanical energy / by expelling air void from the soil.

Factor affecting degree of compaction

1. moisture content
2. Compacting effort (mechnical Energy)
3. Types of soil.

Purpose of Laboratory moisture test

is to determin the amount of mixing water to use when compacting the soil in the field and the resulting degree of denseness which can be expacted optimum moisture content (to be determined on the lab)

$$\gamma_{dry} = \frac{\gamma_{mold}}{1+w} = \frac{G_s \gamma_w}{1+e} = \frac{G_s \gamma_w}{1+\frac{w G_s}{S_r}}$$

$$S_r = \frac{w G_s}{e} \quad \therefore e = \frac{w G_s}{S_r}$$

% compaction defined by the ratio field dry density and laboratory dry density

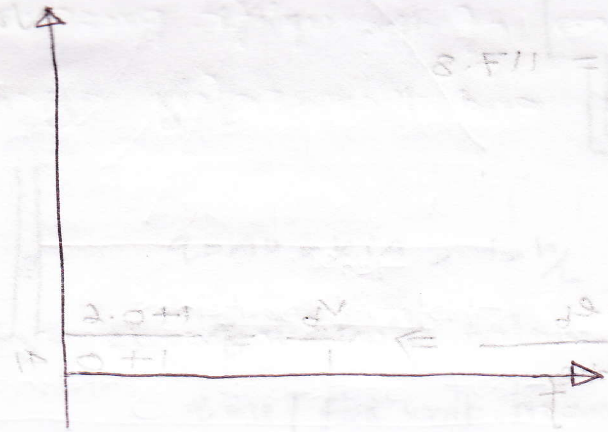
$$\% \text{ compaction} = \frac{\gamma_{d\,field}}{\gamma_{d\,lab}} \times 100\%$$

Removal is a funcn

Lecture – 05
21-09-2011

	Sand	Silty sand	Silt	clay
ω_{opt} —	6-10%	8-12%	12-16%	18-20%

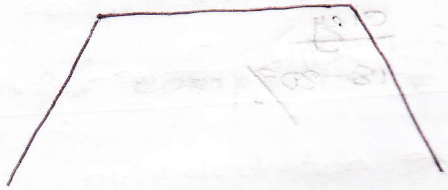
γ_{max} for sand gradually decreasing with going right.



characteristies of compaction curve (P:19 – Peck and Hanson)

01. If I compact the soil dry optimum
02. Permeability:
03. compressibility
04. Stress strain relationship

#10



borrowed area

$\gamma_{max} = 124\#/ft^3$

$v_b \propto (1+e_b)$

95% of $\gamma_d = 124 \times 0.95$

$= 117.8$

$V \propto 1+e$

$v_f \propto (1+e_f)$

So, $\frac{v_b}{v_f} = \frac{1+e_b}{1+e_f} \Rightarrow \frac{v_b}{1} = \frac{1+0.6}{1+0.41}$

So, $v_b = 1.14$ cft

$\gamma_d = \frac{G_s \omega}{1+e_f}$

$\therefore 1+e_f = \frac{2.65 \times 62.4}{117.8}$

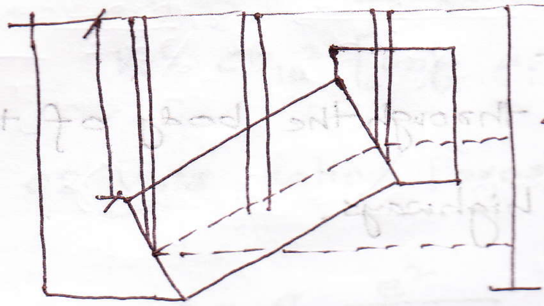
$= 1.41$

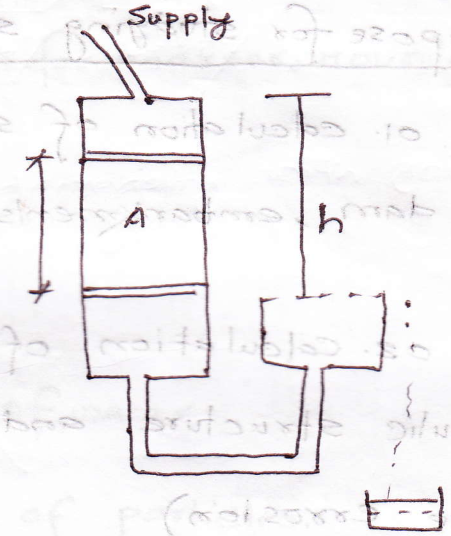
$\therefore e_f = 0.41$

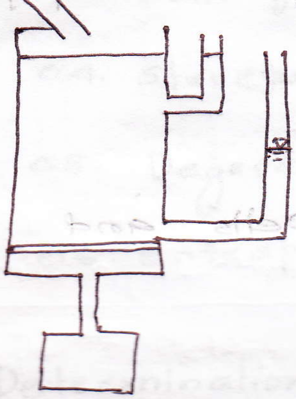
Lecture-06

27-09-2011

CT-Compaction







$q = Av = kiA$, $i = h/L$

$q = Av$

$q = v$ [For unit area of cross section]

Discharge velocity

Apparent velocity

Superficial velocity

$$k = \frac{QL}{Aht}$$

Permeability:

A material is said to permeable if it contains with continuous void.

Permeability is a soil property which indicates the ease with which water will flow through the soil.

Seepage:

Seepage is the slow movement of the water through the continuous void of the soil sample

* purpose for studying seepage

01. calculation of seepage through the body of the earth dam, embankments and highways.

02. Calculation of the uplift pressure under the hydraulic structure and their safety against the piping (surface errosion)

03. Ground water flow towards wells and drainge of the soil.

Discharge velocity:

Discharge velocity is defined as quantity of water percolating in unit time accross an area of unity oriented at right angle to the direction of flow.

$$Q = vA_g = v_s A_v$$

$$v = v_s \frac{A_v \times t}{A_g \times t} = \frac{V_v}{V} v_s = n v_s = \frac{e}{1+e} v_s$$

Relationship exist between seepage and discharge velocity.

Hydraulic conductivity:

01. Particle size / grain size

$K = CD_{10}^2$ [only applicable for coarser, rounded sand]

02. Void ratio / Porosity

$$K = C . \frac{e^2}{1+e}$$

03. Density and viscosity of water

04. structural arrangement of particles.

05. Degrees of saturation.

06. Entrapped air.

⊛ Determination of clay

01. By empirical Method

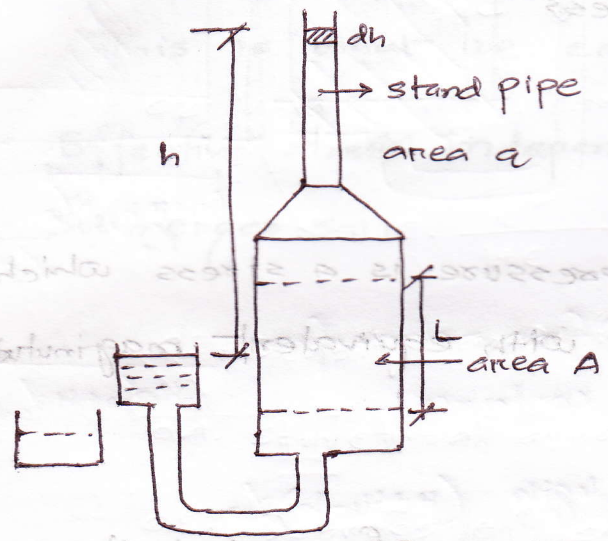
02. By Laboratory method → Constant head / Falling head.

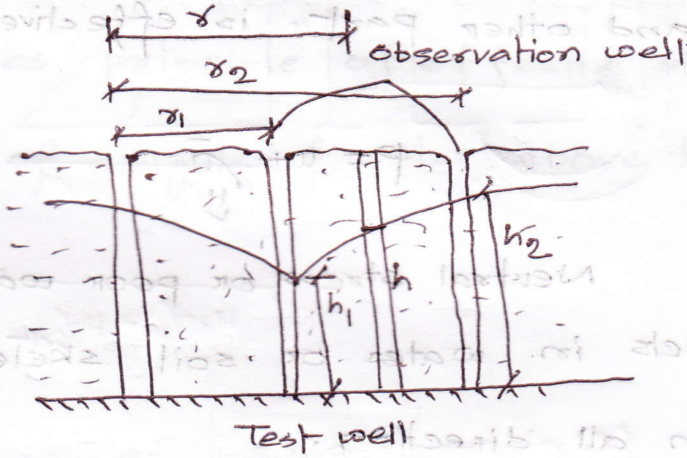
03. Field determination

value of K can be determined indirectly from the consolidation test data.

Lecture

28-09-2011





$$q = \frac{V}{t} = -\frac{a\,dh}{t} = K\frac{h}{L}A.$$

$$\Rightarrow -a\int\frac{dh}{t} = K\int\frac{h}{L}A$$

$$\therefore -\int\frac{dh}{h} = K\int\frac{t}{L}A$$

$$\therefore \boxed{K = 2.3\,\frac{aL}{At}\log_{10}\frac{h_1}{h_2}}$$

$$q = KiA = K\frac{dh}{dr}\times 2\pi r h$$

$$\text{So, } K = \frac{2.3q}{\pi(h_2^2-h_1^2)}\log_{10}\frac{r_2}{r_1}$$

The total stress at a point on a plane through a mass of saturated soil sample is composed of two parts.

One part Neutral stress or poor water pressure and other part is effective stress

$$P = u + \bar{p}$$

Neutral stress or poor water pressure is a stress which acts in water or soil skeleton. with equivalent maginitude in all direction.

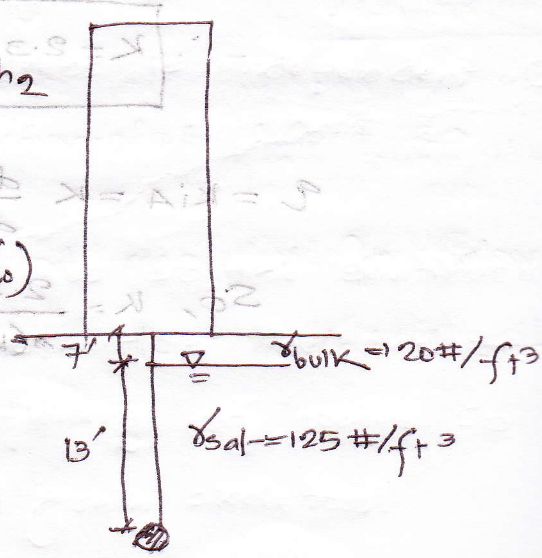
Effective stress is stress in excess of neutral stress which acts excessive btn the contact point of soil and skeleton and these stress solely responsible for including volume change behavior in soil. It also provided frictional resistance.

$$P = 7\times120 + 125\times13 = \gamma_b\times h_1 + \gamma_{sat}\times h_2$$

$$u_w = 13\times62.5 = \gamma_w \times h_2$$

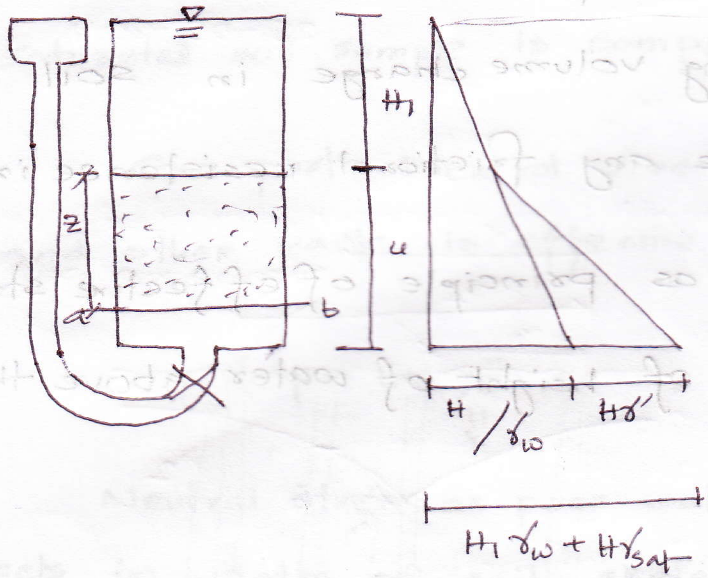
$$\bar{p} = p - u_w = \gamma_{bulk} h_1 + h_2(\gamma_{sat} - \gamma_w)$$

$$= \gamma_{bulk}\cdot h_1 + h_2\gamma'$$

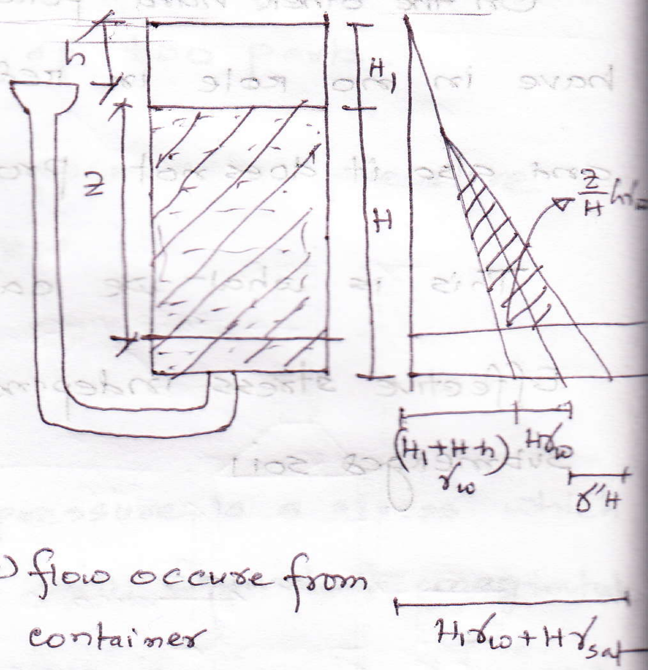


On the other hand pore water pressure or neutral stress have in no role in reducing volume change in soil and also it does not provide any frictional resistence in soil

This is what we called as principle of effective stress. Effective stress independent of height of water above the Submerged soil



(a) at no-flow



(b) flow occure from container

For H legth $(H+H_1-h)\gamma_w$

For H length reduce $h\gamma_w$

Z " " $\frac{h\gamma_w}{H}Z$

$= i\gamma_w z$.

$P_{a-b} = H_1\gamma_w + \gamma_{sat} z$

$u_{a-b} = (H_1+z)\gamma_w$

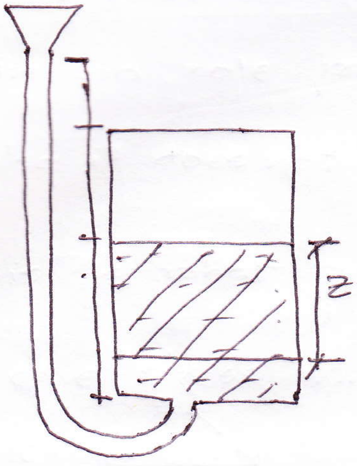
$\bar{P}_{a-b} = P_{a-b} - u_{a-b}$

$= (\gamma_{sat} - \gamma_w)z$

$= \gamma' z$.

The maginitude of effective stress is independent of height of water.

This increasing effective stress so the continuous soil sample seepage pressure it is the result of frictional gap provided by the flowing of water. so the continuous void onto the soil grain skeleton.



$p = (H + H_1 + h)\gamma_w$.

effective stress $\downarrow \frac{h}{H}\gamma_w z$.

Now, $\bar{p} = \gamma' z - i z \gamma_w$.

where, $\bar{p} = 0$. $i_c = \frac{\gamma'}{\gamma_w}$

$= \frac{G_s - 1}{1 + e}$

Quick sand

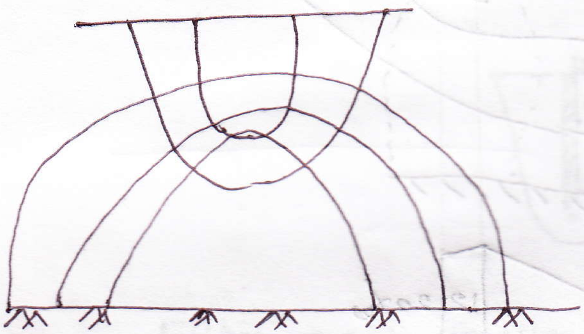
condition $\bar{p} = 0$.

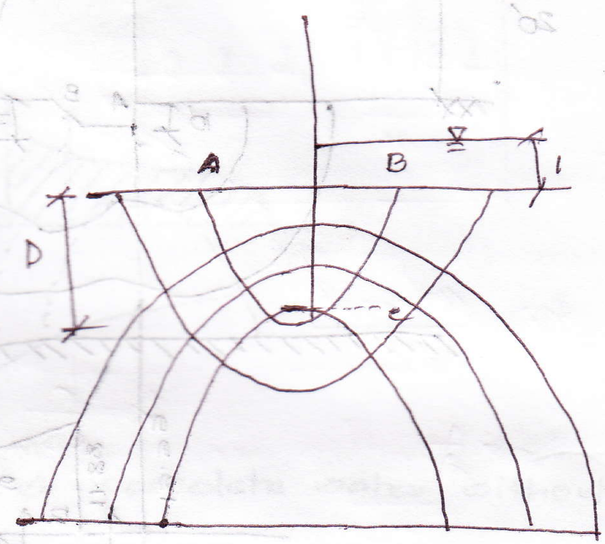
(i) strength of soil $\propto$ effective stress

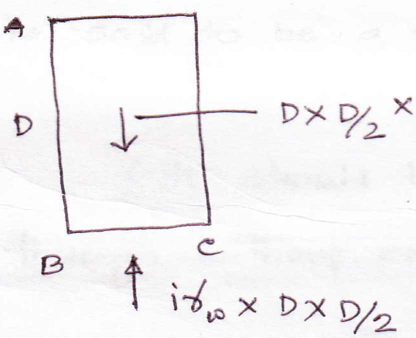
(ii) effective stress has to be 0.

$e = 0.5$ to 1.0 $\quad G_s = 2.65$

$i_c = 0.83$ to 1.13





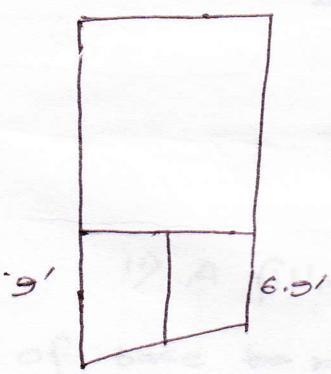


$H = 30' - 6' = 24$

$N_d = 8.$

$N_F = 3$

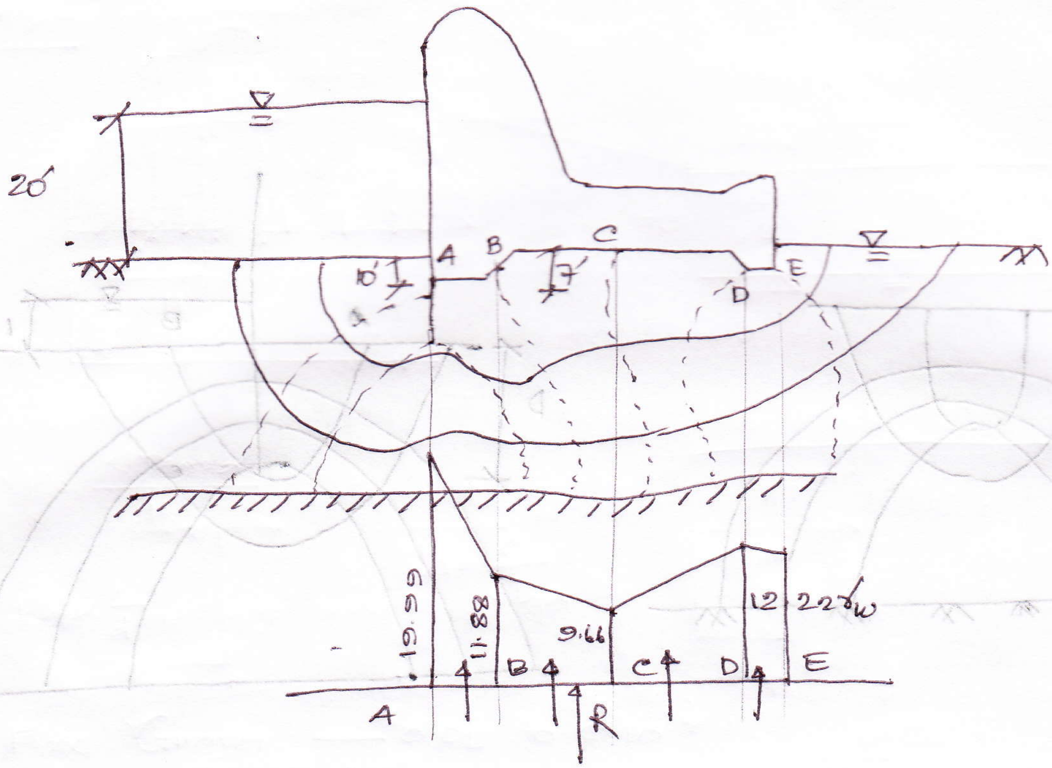
$q = k_H \frac{N_F}{N_d}$

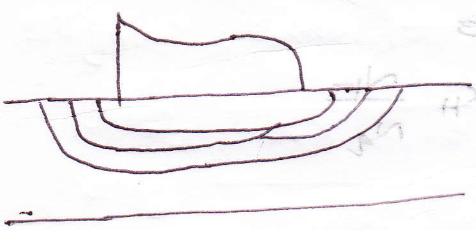


$$F.S. = \frac{D \times D/2 \times 1 \times \gamma'}{D \times D/2 \times 1 \times i_{av}\gamma_w} \qquad i_{av} = \frac{h_{av}}{D}$$

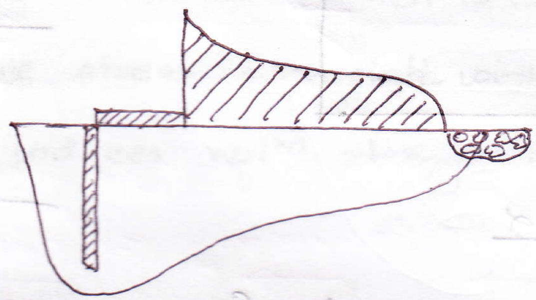
$$\frac{\gamma'}{i_{av}\gamma_w} = \frac{i}{i_{av}} \quad \frac{\gamma}{\gamma_w} = \frac{i_c}{i_{av}}$$

$$p = \gamma z' = i z \gamma_w$$


20'
A
B
C
10'
7'
D
E
19.99
11.88
9.66
12.22γw
A
B
C
D
E
R



Faliure of Hydraulic Structure by piping



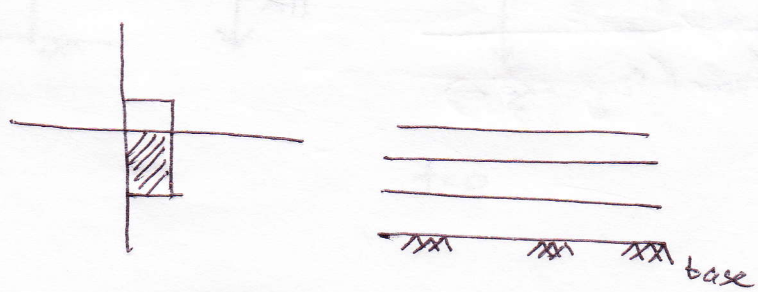
Filter media matter which percolate water without creating seepage force and has to retain the base material. A matter is said to be a filter matter has to

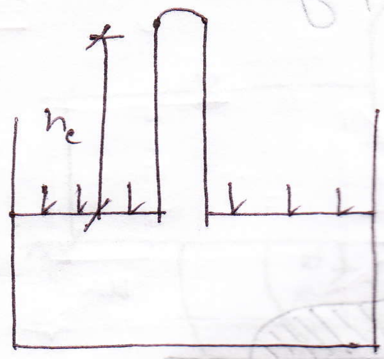
1) It should be coarse enough such that water percolating through without creating seepage force on it.

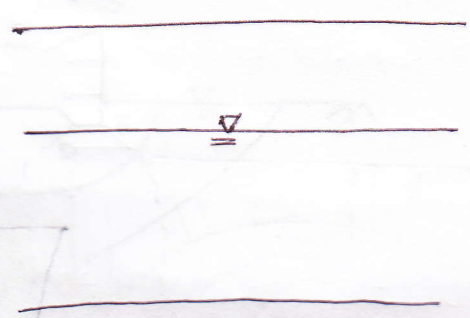
$$\frac{D_{15}(F)}{D_{15}(B)} \geq 4 \text{ to } 5$$

$$\frac{D_{15}(F)}{D_{85}(B)} \leq 4 \text{ to } 5.$$

11) A filter matter is fine enough which prevent the migration of base ~~to~~ material.







$$h_e = \frac{4T\cos\alpha}{d\gamma_w}$$

$$h = \frac{C}{eD_{10}}$$

$$h_e \propto \frac{1}{d}$$

Fine Gravel — 0.02 to 0.10

coarse Gravel — 0.1 to 0.15

medium Sand — 0.15 to .30

Fine Sand — 0.3 to 1.0

Silt ———— 1.0 to 10

clay ———— 10 to 30

Colloid ———— >30

Consoledation

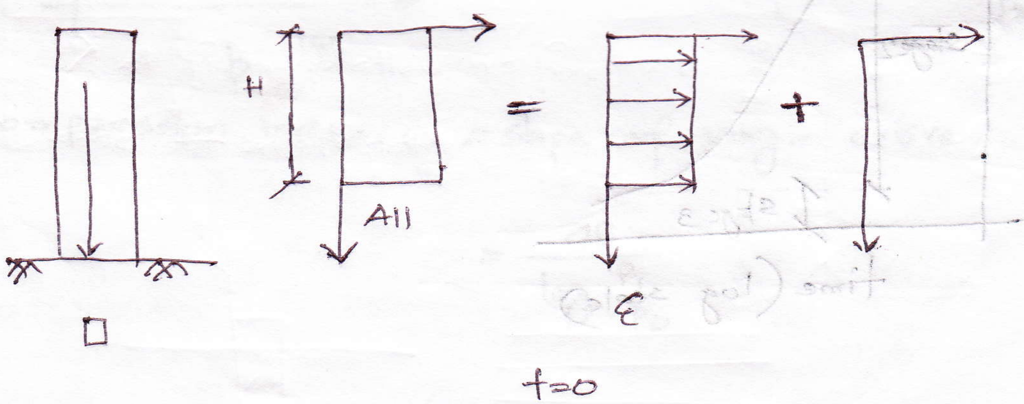
Settlement is a time

effective stress to the soil which is subjected to .
This increasing effective stress associate with the decipiation of excess pore water induces vol^m change in soil which

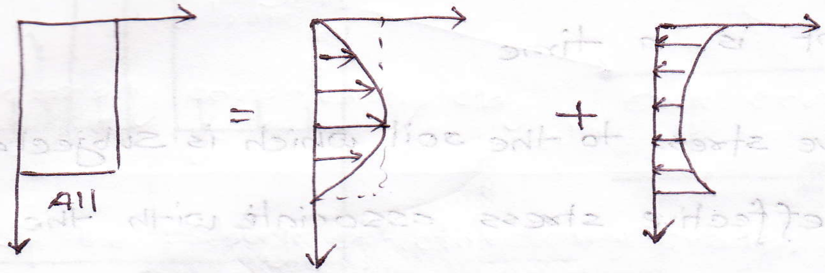
By Trazzaghi

The decrease of water content of sat. soil without the replaceed ment of water by year is called the process of consolation.

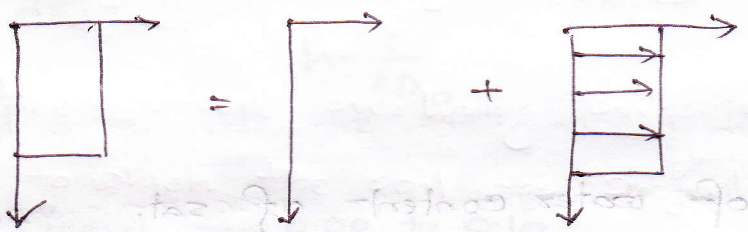
The gradul process of increase of effective stress in the clay layers due to the surger load will results in a sediment which is time dependent phenomena which is reffered to as the process of consoledation.



at $t > 0$



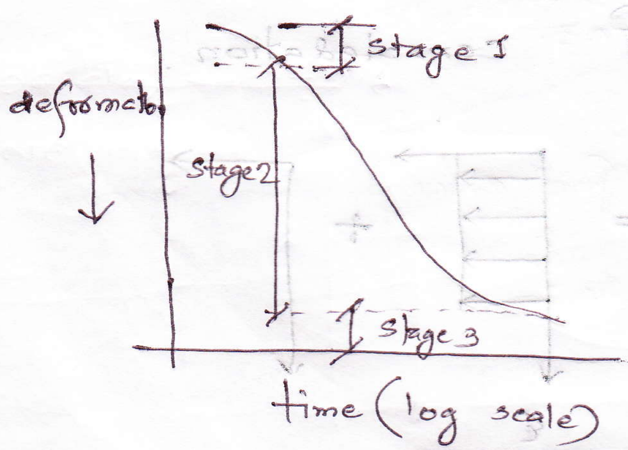
$t \to \infty$



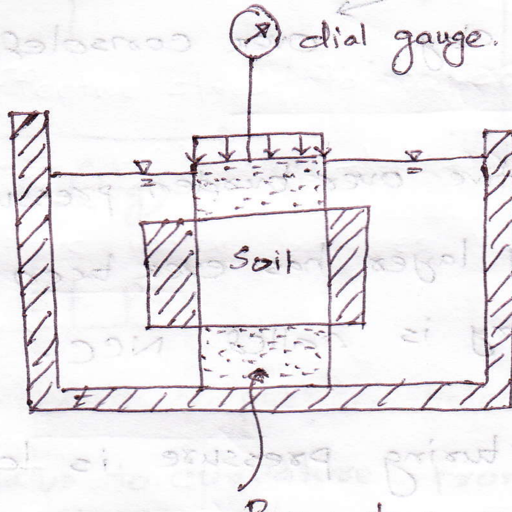
Stage 1 : Sediment due to initial compression

02 : Sediment due to consoledation.

03 : Sediment due to



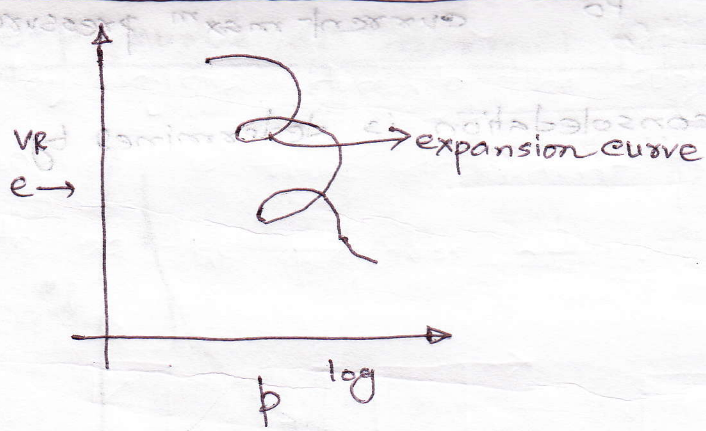
Consoledometer or oddometer:

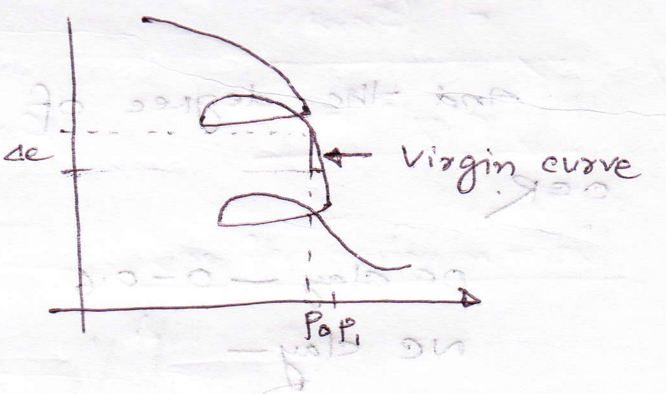


Laboratory reading, 10, 20, 40, 80, 160, 320, 640

Time (sec) → 15, 30, 1 min, 2, 4, 8, 16, ½ hr, 1 hr, 2 hr, 4 hr, 8 hr, 24 h

Void Ratio Effective Pressure





compression index, C_c = Slope of virgin curve.

$$= \frac{\Delta e}{\log \frac{P_1}{P_0}}$$

Depending on the magnitude of pressure

Soil → Normally consoledated clay, over consoledated clay.

NCC If the present effective over burden pressure is the max^m pressure to which the clay layer has ever been subjected to at any time in its history is called NCC.

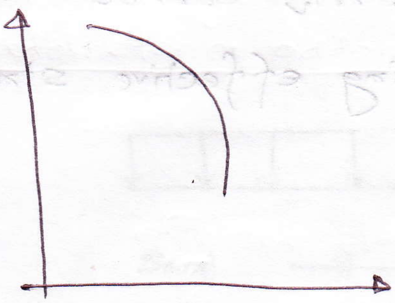
If the current overturing pressure is less than the pressure to which the soil was subjected to in the past over consoledated clay. This past max^m pressure is called pre ~~over~~ consoledated soil

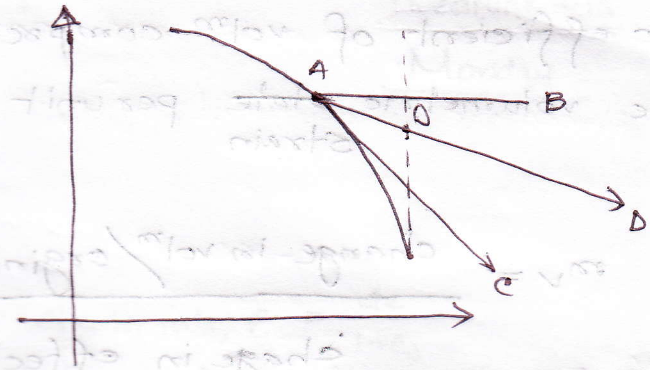
$$\text{Over consoledation ration} = \frac{P_c}{P_o} = \frac{\text{past max}^m \text{ pressure}}{\text{current max}^m \text{ pressure}}$$

And the degree of consoledation is determined by OCR.

OC clay — 0 – 0.6

NC clay —





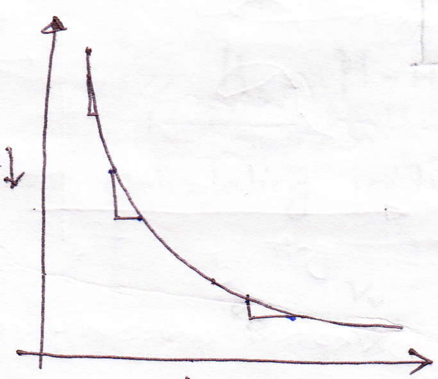
Determination of P_c

Step-1: By visulati inspection select a point which gives you min^m radius to curvature. From A draw a horizontal line. at point A draw a tangent line AC. Bisect the $\angle BAC$ by AD.

Extend the

which intersect the bisector at O. pressure corresponding to o is known as ~~pure~~ consoledation pressure.

e-p curve at normal paper



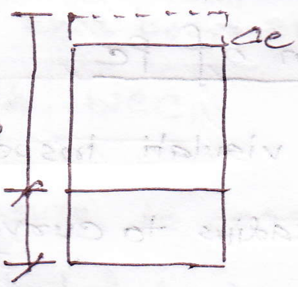
coefficient of compressibility $a_v = \frac{\Delta e}{\Delta p}$

a_v is a function of pressure range under consideration.

co efficient of vol^m compressibility, m_v can be defined as the volumetric ~~state~~ strain per unit increasing effective stress.

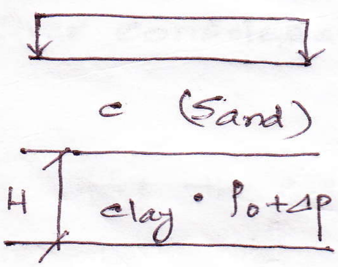
$$m_v = \frac{\text{change in vol}^m/\text{orginial vol}^m}{\text{chage in effective stress}}$$

$$= \frac{\Delta V/V}{\Delta p}$$



Here, $\frac{\Delta V}{V} = \frac{\Delta e}{1+e_0}$

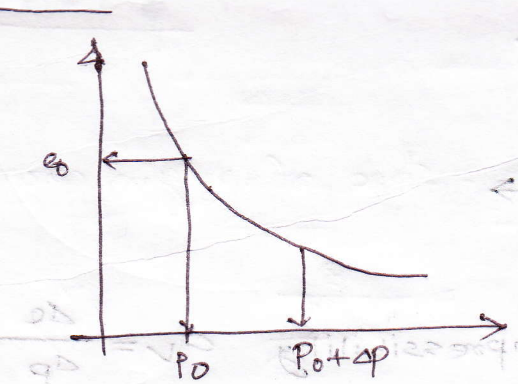
$$\text{So, } m_v = \frac{\Delta e}{(1+e_0)\Delta p} = \frac{\Delta e}{\Delta p} \cdot \frac{1}{1+e_0} = a_v \frac{1}{1+e_0}$$

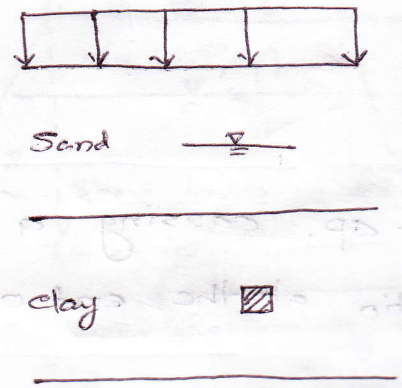


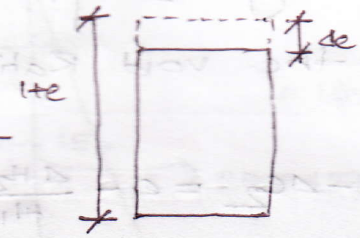
$$\varepsilon = \frac{\Delta e}{1+e_0}$$

$$\therefore S = \boxed{H \Delta p \, m_v}$$

Determination of m_v







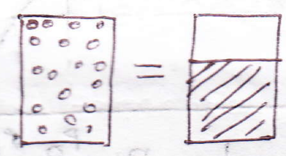
Strain rate, $\varepsilon = \frac{\Delta e}{1+e_0}$

$$S = H\varepsilon$$
$$= H\frac{\Delta e}{1+e_0}$$

Void Ratio - effective pressure Plot

01. Calculate the ht of the solid of the soil.

$$H_S = \frac{W_s}{A\,G_s\,\gamma_w}$$



02. Calculating initial height of void.

$$H_v = H - H_v$$

03. Calculating initial void Ration.

$$e_0 = \frac{V_v}{V_s} = \frac{V_v/A}{V_s/A} = \frac{H_v}{H_s}.$$

04. For the first incremental loading P_1 causing a deformation of Δh_1 calculate the changing void Ratio, Δe_1.

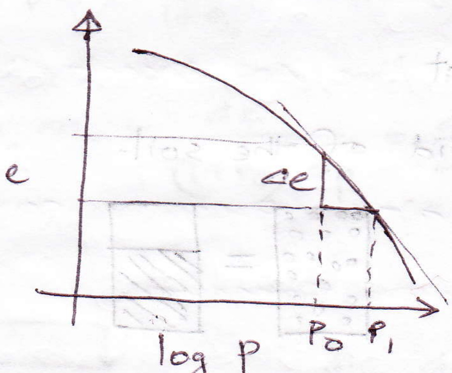
$$\Delta e_1 = \frac{\Delta H_1}{H_s}$$

05 calculate the new void ratio, e_1 after the cosolidation of pressure increment of P_1

$$e_1 = e_0 - \Delta e_1$$

06 For the next loading $P_2 = P_1 + \Delta P$. causing a additional defm of ΔH_2 the void Ratio at the end of consolidation

$$e_2 = e_1 - \Delta e_2 = e_1 - \frac{\Delta H_2}{H_1}$$



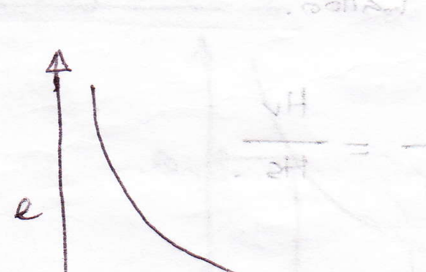
$$\epsilon = \frac{\Delta e}{\log P_1 - \log P_0}$$

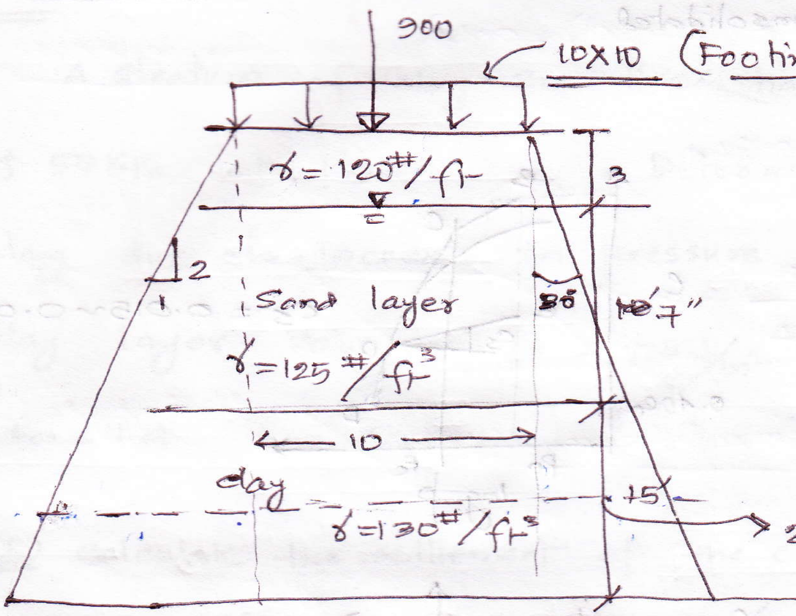
$$= \frac{\Delta e}{\log P_1/P_0}$$

e-logP

$$= \frac{\Delta e}{\log\left(\frac{P_0 + \Delta P}{P_0}\right)} \quad \left[P_1 = P_0 + \Delta P\right]$$

settlement $S = H\epsilon = H \frac{\Delta e}{1+e_0} = \frac{H C_c}{1+e_0} \log\left(\frac{P_0 + \Delta P}{P_0}\right)$





$P_0 = 120 \times 3 + (125 - 62.5)7 + (130 - 62.5) \times 7.5$

$= 1580 \text{ psf}$

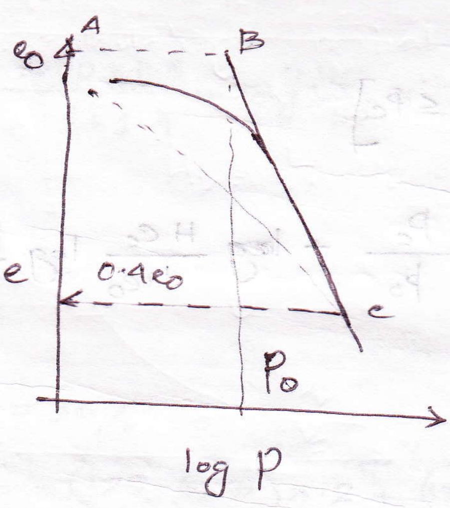
$\Delta p = \dfrac{900}{27.5 \times 27.5} = 1.15$

$P_0 = 1.58 \text{ ksf}$

$e = w G_s$

$= \dfrac{35}{100} \times 2.75 = 0.4025$

$S = \dfrac{15 \times 12 \times 0.1}{1 + .4025} \log \dfrac{1.58 + 1.15}{1.58}$



irrespective of decrease of disturbance induced into a soil the state (virgin) of all e - log p curve intersect at a common point corresponding to 40% of initial void ration

ABC in ideal e-logp cure of undistrubed soil sample

Scharfman

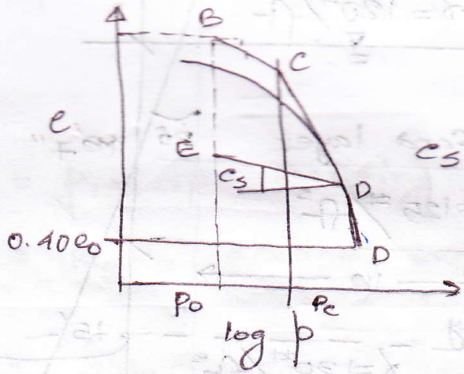
e_0, P_0

e-log p Curve for over consolidated

e_0, p_0, p_c

$$p_c = \frac{q_u/2}{0.11 + 0.0037 I_p}$$

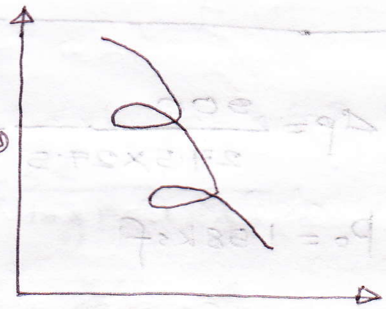
$$= \frac{c}{0.11 + 0.0037 I_p}$$



$C_s = 0.015 \sim 0.035$

Swelling index 10-20% of C_c value

- Require to compute the over consolidated condition.



Value of C_s gets lower for lower plasticity and lower over consolidation ratio

$$S = \frac{H \Delta e}{1+e} = \frac{H C_s}{1+e} \log \frac{p_0 + \Delta p}{p_0} \quad [p_0 + \Delta p \le p_c]$$

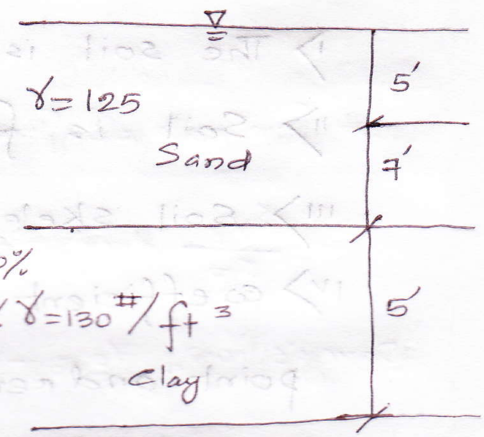
when, $p_0 + \Delta p > p_c$, $S = \frac{H C_s}{1+e_0} \log \frac{p_c}{p_0} + \frac{H C_c}{1+e_0} \log \frac{p_0 + \Delta p}{p_c}$

Prob:

A stratum of clay 2m thick has an initial over burden pressure of 50 KPa at its . Determine the final settlement of clay due to increase in pressure 40kPa in the middle of the clay layer. Value of $P_c = 75 \text{kN/m}^2$. Given $C_c = 0.25$, $C_s = 0.05$

$e_0 = 1.4$

(2) Calculate the settlement of the clay layer when WT drops by 12', The γ sand is reduced by $120^{\#}/ft^3$ at the time.



Solve

1. $S = \frac{H C_s}{1+e_0} \log \frac{P_c}{P_0} + \frac{H C_c}{1+e_0} \log \frac{P_0 + \Delta P}{P_0}$

$= \frac{2000 \times 0.5}{1+1.4} \log \frac{75}{50} + \frac{2000 \times 0.25}{1.14} \log \frac{90}{75}$

2

$w = 30\%$

$P_0 = (125 - 62.5) \times 12 + (130 - 62.5) \times 2.5 = 1.0875 \text{ Ksf}$

$P = 12 \times 120 + 2.5(130 - 62.5) = 1.608$

$\Delta P = (-1.0875 + 1.608)$

$= 0.52125$

$C_c = 0.007\,(LL-10) = 0.007\,(30-10) = 0.14$

$e_0 = wG_s = 0.32 \times 2.7 = 0.864$

$$S = \frac{5 \times 12 \times 0.14}{1+0.86} \log \frac{1.608}{1.087} = 0.8''$$

Terzaghy 1D consolidation theorey (1955)

Assumption

i> The soil is homogeneous and isotrophic

ii> Soil is fully saturated

iii> Soil skeleton and water in void are imcom

iv> coefficient of permeability have same value at all point and remains constant during the entire time for consolidation

v> Darcy law is valid during the entire time of consolidation

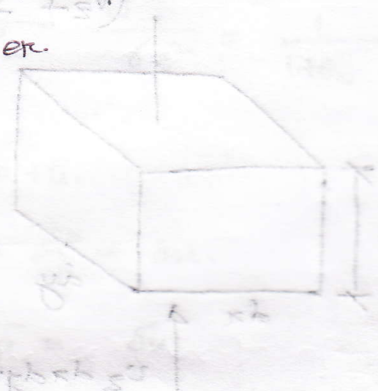
i.e $v = Ki$

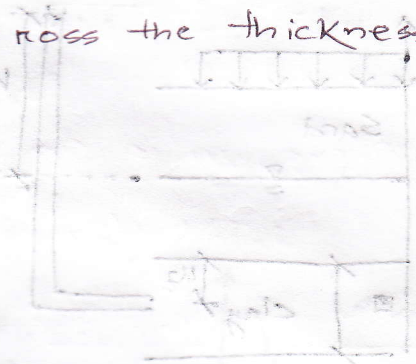
vi> Soil is laterally confined so that the consolidation takes place only in axial direct

vii> There is unique relationship btn e and μ' and the

in the other word, mv (Co-efficient volm compressibility remains constant. during that particular load increment

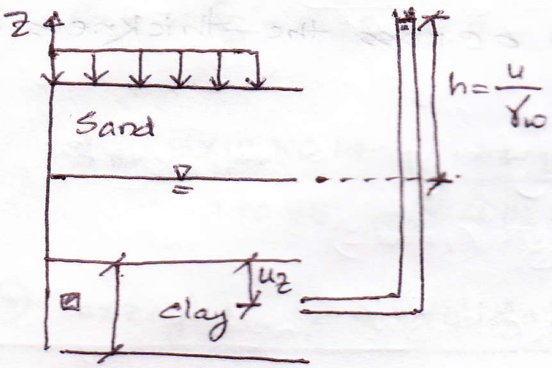
viii) By applying apressure uniform along a horizontal plane and the initial excess pore water pressure due to applied pressure also uniform across the thickness of clay layer.

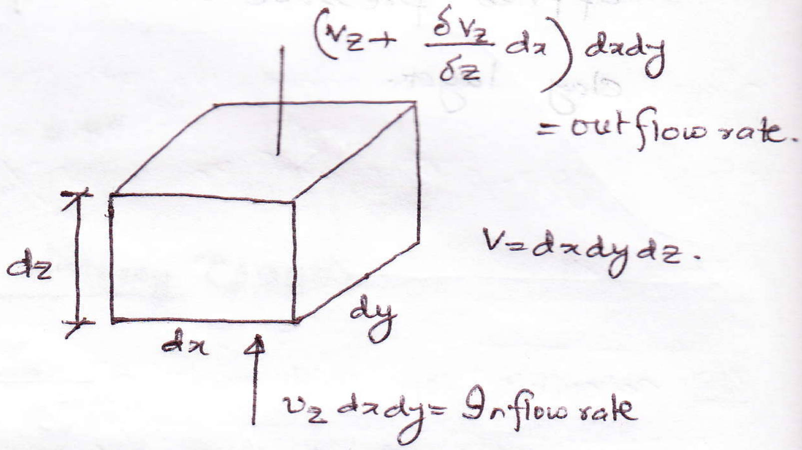




Lecture-18

07-12-2011





out flow rate - Inflow rate = rate of vol^m change

$$\frac{\delta v_z}{\delta z} dx dy dz = \frac{dV}{dt}$$

Darcy's law, $v_z = -k i_z = -k \frac{\delta h}{\delta z} = -\frac{k}{\gamma_w} \frac{\delta u}{\delta z}$

So, $$\frac{\delta}{\delta z}(v_z) = \frac{1}{dx dy dz} \frac{dV}{dt}$$

Or, $$-\frac{k}{\gamma_w} \frac{\delta^2 u}{\delta z^2} = \frac{1}{dx dy dz} \frac{dV}{dt}$$

Here, $V = V_s + V_v = V_s + eV_s$. and, $\frac{V_s}{V} = \frac{1}{1+e_0}$

$$\therefore \frac{dV}{dt} = V_s \frac{\delta e}{\delta t} \qquad \therefore V_s = \frac{V}{1+e_0} = \frac{dx dy dz}{1+e_0}$$

$$= \frac{dx dy dz}{1+e_0} \frac{\delta e}{\delta t}$$

$$\text{So, } -\frac{K}{\gamma_w}\frac{\delta^2 u}{\delta z^2} = \frac{1}{dx\,dy\,dz}\,\frac{dx\,dy\,dz}{1+e_0}\,\frac{\delta e}{\delta t}$$

$$\therefore -\frac{K}{\gamma_w}\frac{\delta^2 u}{\delta z^2} = \frac{1}{1+e_0}\,\frac{\delta e}{\delta t}$$

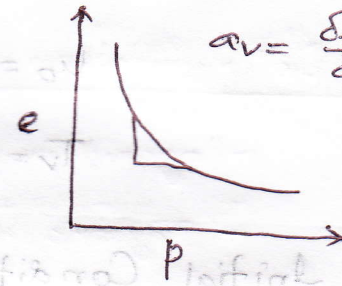
Again, $p = \bar{p} + u$.

$\therefore \delta p = \delta \bar{p} + \delta u.$

$\therefore \delta \bar{p} = \delta u$

$\delta e = -a_v \delta p.$

$= -a_v \delta u.$



$$a_v = \frac{\delta e}{\delta p}$$

$$\text{So, } -\frac{K}{\gamma_w}\frac{\delta^2 u}{\delta z^2} = -\frac{a_v}{1+e_0}\,\frac{\delta u}{\delta t}.$$

$$\text{Or, } \frac{K}{\gamma_w}\frac{\delta^2 u}{\delta z^2} = m_v\frac{\delta u}{\delta t}.$$

$$\therefore \frac{\delta^2 u}{\delta z^2} = \frac{m_v \gamma_w}{K}\,\frac{\delta u}{\delta t}$$

$$\text{So, } \frac{\delta u}{\delta t} = \frac{K}{m_v \gamma_w}\,\frac{\delta^2 u}{\delta z^2}$$

$$= c_v \frac{\delta^2 u}{\delta z^2}$$ $\left[c_v = \frac{K}{m_v \gamma_w} = \text{co-efficient of consistency} \right]$

Soln

$$u = \sum_{m=0}^{m=\infty} \frac{2u_0}{M} \sin\left(\frac{Mz}{H}\right) e^{-M^2 T_v}$$

where, m is an integer

$M = (2m+1)\pi/2$

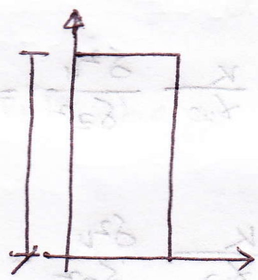
$u_0 = $ I.E.P.W.P

$T_v = $ time factor $= \frac{C_v t}{H_{dr}^2}$

Initial Condition $t = 0$, $u = u_0$

and, $u = 0$ at $z = 0$

$u = 0$ $z = 2H$



For both way drainage the length of the drainage path h_{dr} is $= \frac{1}{2}$ of thickness of clay layer. But for one way drainage the length of drainge path = total length of clay

at a depth z,

$$U_z = \frac{u_0 - u_z}{u_0} = \text{Degree of Consolidation}$$

$$= 1 - \frac{u_z}{u_0} = \frac{\text{Excess pore water pressure dissipated}}{\text{Initial Excess p.w. pressure}}$$

Avg. degree of consolidation, $U = \frac{\frac{1}{2H}\int_0^{2H} u_z \, dz}{u_0}$

$$\therefore U = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} e^{-M^2 T_v}$$

$$u = 1 - \frac{2}{M^2} e^{-M^2 T_v}$$

Or, $$0.1 = 1 - \frac{2}{\left(\frac{\pi}{2}\right)^2} e^{-(\pi/2)^2 T_v}$$

Or, $$0.9 = 1 - \frac{8}{\pi^2} e^{-\frac{\pi^2}{4} T_v}.$$

Or, $$0.1 = \frac{8}{\pi^2} e^{-\pi^2/4\, T_v}$$

Or, $$e^{-\pi^2/4\, T_v} = \frac{\pi^2 \times 0.1}{8}$$

$$\therefore T_v \left(-\frac{\pi^2}{4}\right) = \ln \frac{\pi^2 \times 0.1}{8}$$

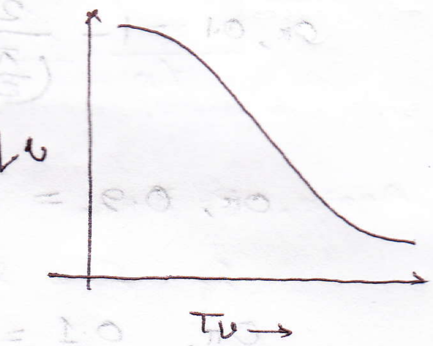
$$\therefore T_v = \cancel{\frac{0.1}{2}} - \frac{4}{\pi^2} \ln \frac{\pi^2 \times 0.1}{8}$$

So, $0.5 =$

$$T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 \quad 0 \le U \le 60.$$

$$T_v = 1.781 - 0.933 \log_{10}(100-U) \quad U \ge 60.$$

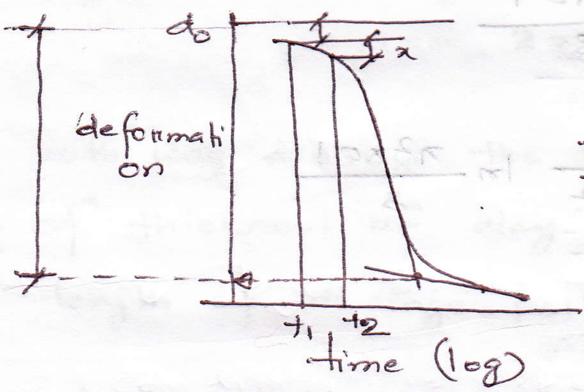
$$T_v = \frac{c_v \times t}{H_{dr}^2}.$$



Determination c_v

- logarithm of time method.
- Taylor's method or square root of time method.

logarithm of time method

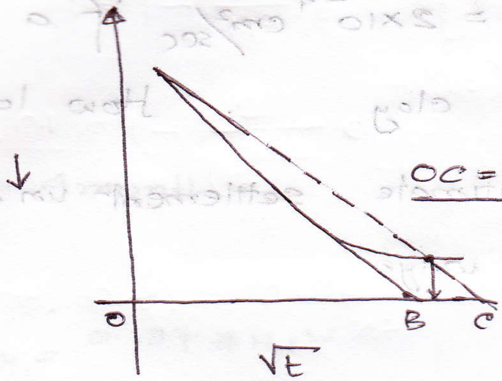


$t_2 = 4t_1$

time required to attain d_{50} 50% Consolidation, $d_{50} = \frac{d_0 + d_{100}}{2}$

$$0.197 = T_v = \frac{c_v \times t_{50}}{H_{dr}^2}$$

Square Root of time Method



$t_{90} = 0.848$, $T_v = \frac{c_v \times t_{90}}{H_{dr}^2}$

OC = 1.15 OB

$\therefore c_v = \frac{T_v \, H_{dr}^2}{t_{90}}$

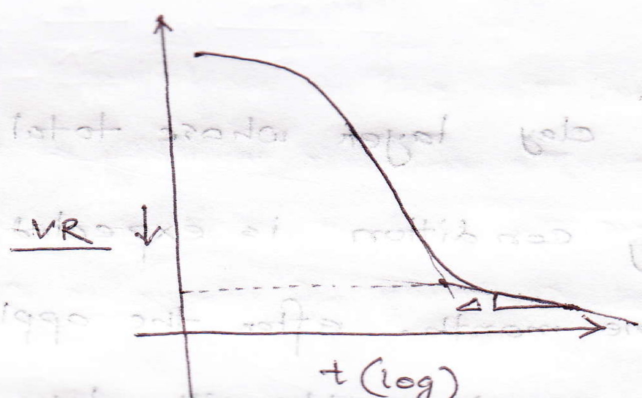
Coefficient of vol^m compressibility, $c_v = \frac{K}{\gamma_w m_v}$

$\therefore K = c_v \gamma_w m_v$

Secondary Consolidation

- organic soil.
- Highly compressible in organic soil

$C_r = \frac{\Delta e}{\log t_2/t_1}$



$S = H \frac{\Delta e}{1+e_0}$

$= \frac{C_r}{1+e_0}$

An undisturbed soil sample of clay stratum 2m thick was tested in laboratory and $c_v = 2\times10^{-4}\ cm^2/sec$ if a structure is built on the top of the clay. How long it will take to half of its ultimate settlement under the load of structure. Assume both ways

$C_v = 2\times10^{-4}$

$U = 50\%$

$$T_v = 0.197 = \frac{c_v \times t_{50}}{H_{dr}^2} = \frac{2\times10^{-4}\times t_{50}}{(100cm)^2} =$$

$$\therefore t_{50} = \frac{1970}{2\times10^{-4}} = 144 \text{ days}$$

A clay layer whose total settlement under a given loading condition is expected to be 12 cm. Settle 3 cm in one month after the application of load increment. How much time it will take to reach 6cm settlement and how much the structure will settle in 10 month. Assume both way drainage.

$S_f = 12cm$

$u_1 = 25\%$

$u_2 = 50\%$

$$\frac{T_{v_1} * H_{dr}^2}{t_{25}} = \frac{T_{v_2} * H_{dr}^2}{t_{50}}$$

$$\therefore t_{50} = \frac{T_{v_2}}{T_{v_1}} \times t_{25}$$

$$T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$$

$$=$$

$\therefore t_{50} = 4$ month.

So, $c_v = 0.197 * H_d^2$

$$\frac{0.0492}{1} = \frac{T_{v_2}}{10}$$

$\therefore T_{v_2} = 0.492$

Now, $0.492 = 1.781 - 0.933 (100 - U)$

So, $U = 76\%$

Stress Distribution in Soil

Load under consideration:

1. Point load
2. Line load with finite width
3. Circular load.
4. Rectangular/square
5. Any arbitrary shape.

Basic Assumption:

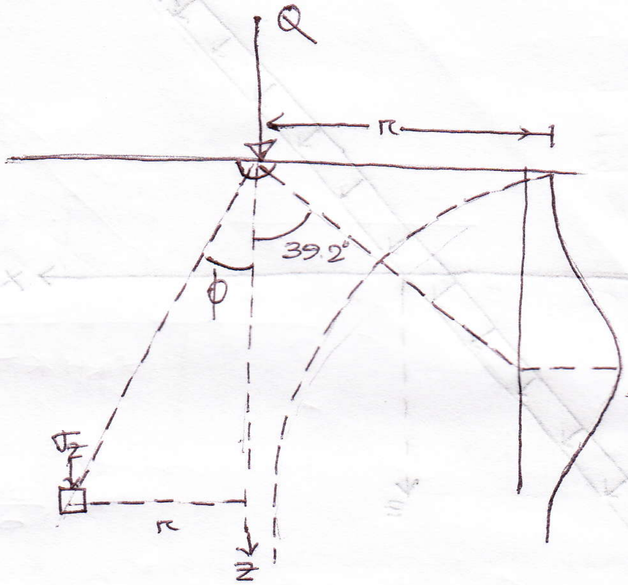
01. Soil is elastic, homogeneous, isotrophic and semifinite which extends infinitely in all direction, form a level surface and which obey hook's law.

02. The soil is weightless

03. The soil acts vertically at a point on the horizontal

04. The soil is initially unstressed

05. Continuity of stress is considered in the medium



Here,

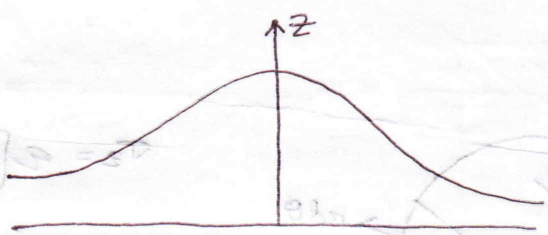
$$r = \sqrt{x^2+y^2}$$

$$R = \sqrt{x^2+y^2+z^2}$$

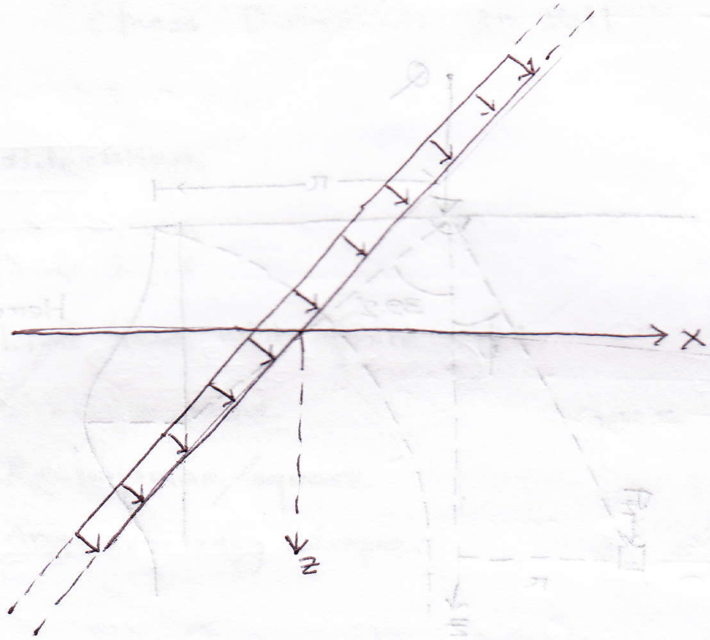
$$\sigma_z = \frac{3Q}{2\pi} \frac{z^3}{(r^2+z^2)^{5/2}}$$

$$= \frac{3Q}{2\pi} \left(\frac{z}{R}\right)^5$$

$$= \frac{3Q}{2\pi z^2} \cos^5 \psi$$



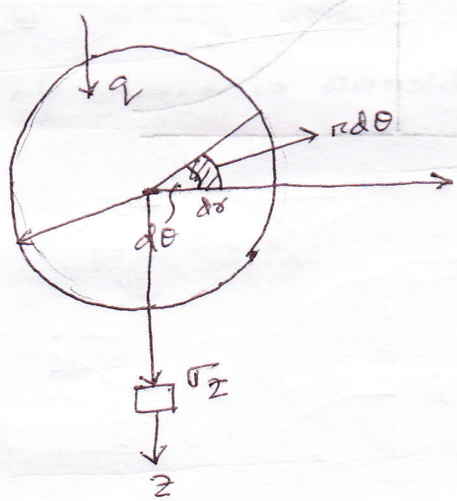
Line load:



The vertical stress due to a line load (wt/unit length) on a surface located at a point located of at depth z and distance x laterally away is given by,

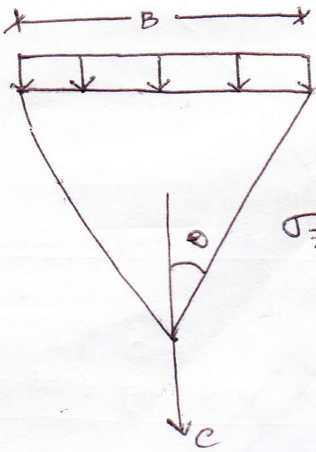
$$\sigma_z = \frac{2q_1}{\pi} \frac{z^3}{(x^2+z^2)^2}$$

<u>Circular load:</u>

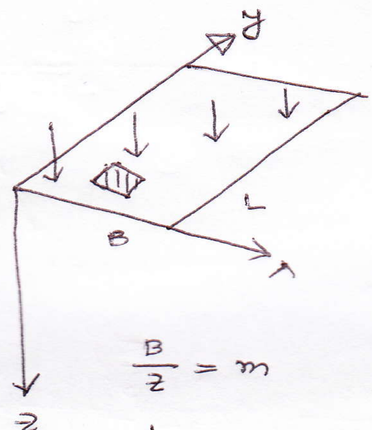


$$\sigma_z = q\left[1 - \frac{1}{\{1+(a/z)^2\}^{3/2}}\right]$$

Lecture
19-12-2011



$$\sigma_z = \frac{B}{A}(2\theta + \sin 2\theta)$$



$$\frac{B}{z} = m$$

$$\frac{L}{z} = n$$

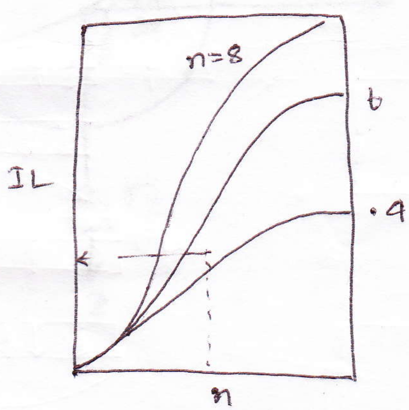
$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2} \quad \frac{m^2+n^2+2}{m^2+n^2+1} + \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}\right]$$

applicable except $m^2+n^2+1 < m^2n^2$

For $m^2+n^2+1 < m^2n^2$

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2} \quad \frac{m^2+n^2+2}{m^2+n^2+1} + \pi - \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}\right.$$

<u>Fadum's chart:</u>

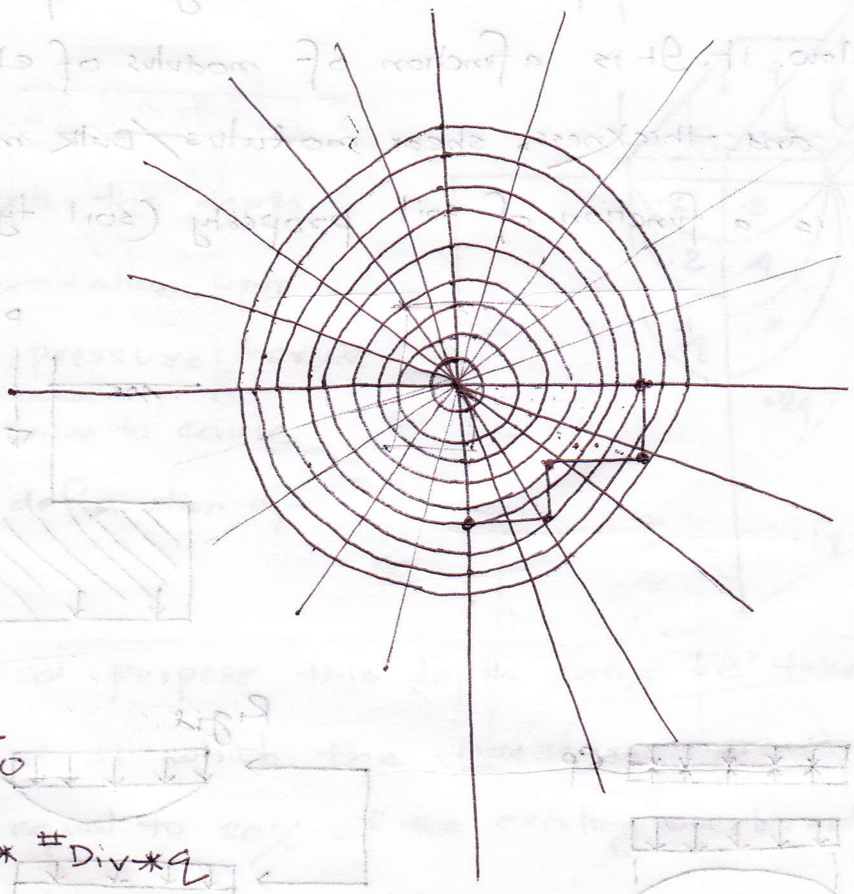


$$\frac{a}{z} = \left\{ \left(1 - \frac{q}{\sigma_z}\right)^{-2/3} - 1 \right\}^{1/2}$$

$\frac{q}{\sigma_z}$	$\frac{a}{z}$	$\frac{a}{\sigma_z}$	$\frac{a}{z}$
0	0	0.6	.92
0.1	0.27	0.7	1.11
0.2	0.4	0.8	1.39
0.3	.52	0.9	1.91
0.4	.64	1.0	α.
0.5	.72		

$\frac{a}{z} = 1.91$,

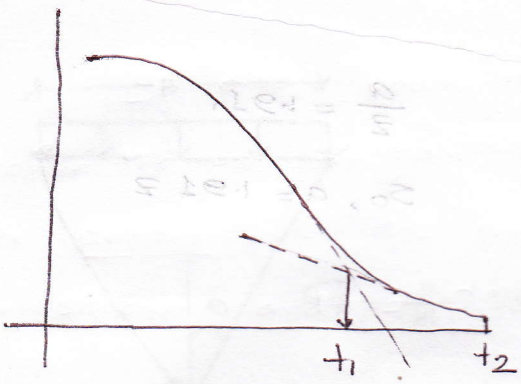
So, $a = 1.91\,z$



$I_F = \frac{1}{200}$

$IF = \frac{1}{200}$

$\sigma_z = I.F * \#Div * q$



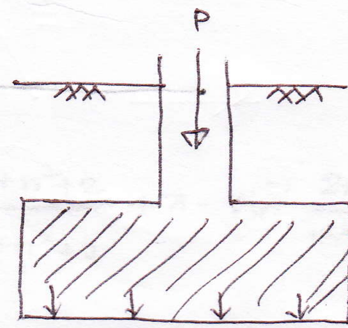
$$S = H C_\alpha \log \frac{t_2}{t_1}$$

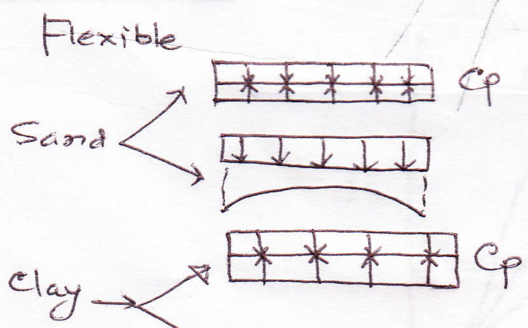
Contact pressure distribution

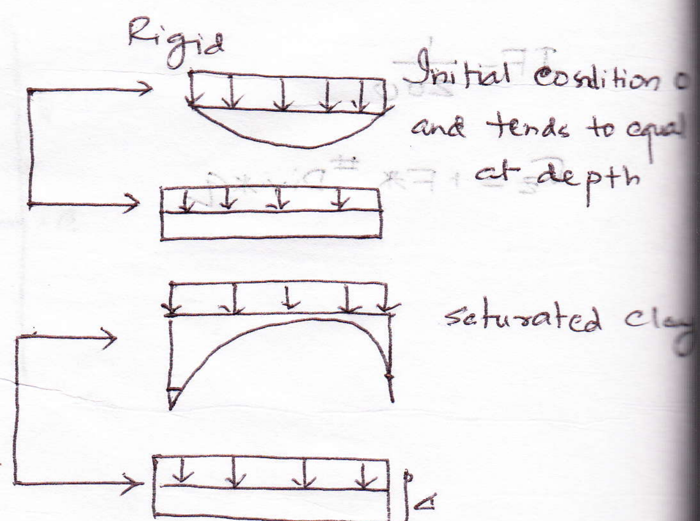
Contact pressure is a pressure exerted by the foundation on the soil right below it. It is a function of modulus of elasticity E_E, Poison's ratio and thickness, shear modulus / Bulk modulus. and also it is a function of soil property (soil type, nature) i.e c and ϕ.

rigid –

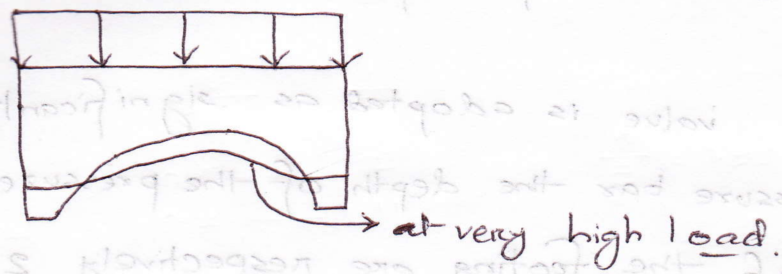
flexible –





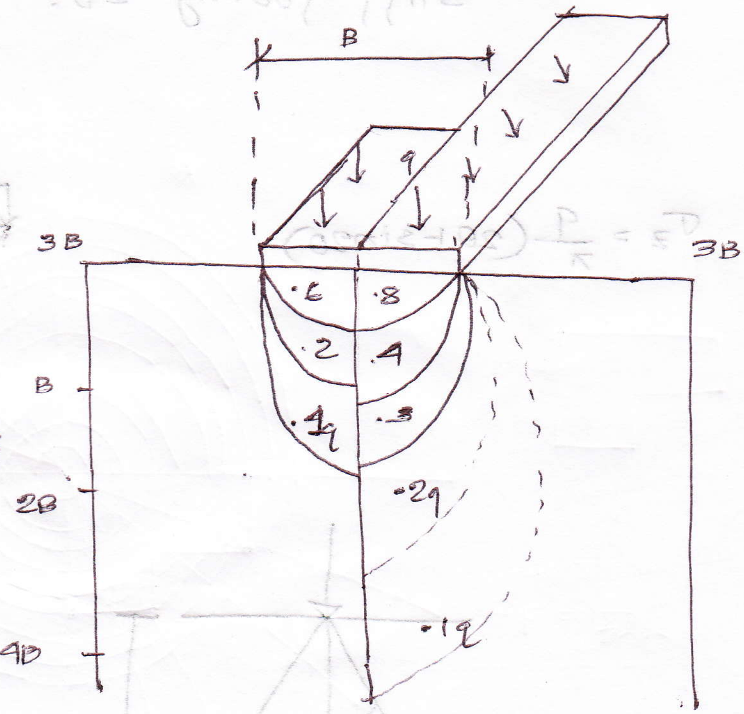


Clay



Stress distribution of soil under continuous footing and square footing

line of equal magnitude of stress is called isober.



Signifi

It represents the depth below a foundation upto which the pressure increase may be assumed to cause significant deformation of the soil.

For practical purpose this depth may be taken to the level at which the pressure increase due to foundation loading is equal to 20% of the existing overburden pressure

Sometimes the significant depth value is taken to correspond to an isobar of 0.1q or 0.2q intensity and these define the depth of pressure bar.

If 0.1q value is adopted as significant stress to define the pressure bar the depth of the pressure bar below the <u>center</u> of the footing are respectively 2B and 6B for square and stiff footing

if 0.2q square footing 1.5B
stiff footing 3B.

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