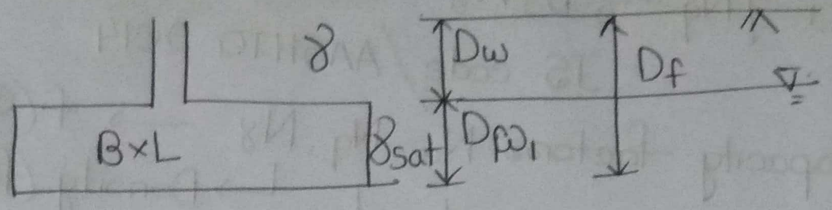
Sat Sun Mon Tues [✓ Wed] Thurs Fri

Eqram Sir

Date: 08 05 '19

Shallow foundation.



Shallow foundation, $\frac{D_f}{B} \le 2.0$

Types: 1. Line/Wall footing
2. Individual footing
3. Combined footing (Superimposed / High load density)
4. Raft footing (>50% superimposed)

3. Double layer reinforcement.

D_f = depth of footing

All formulas assume footing to be on the ground surface. Practically, ~~not~~ given below 1.5m due to:

1. Neighbour/Drain cutting will cause the foundation to be supported.
2. Pests will make their nests

References:

1. Advanced Foundation Engineering
   - VNS Murthy
2. Foundation Engineering
   - Varghese.
3. Peck Hanson, Thornburn

Bearing Capacity

$q_u = CN_c + qN_q + \frac{1}{2}B\gamma N_\gamma$

Terzaghi, ... IS code/AASHTO 2014

Bearing capacity factors: $N_c, N_q, N_\gamma \longrightarrow f(\phi)$

→ Density (for s oil)

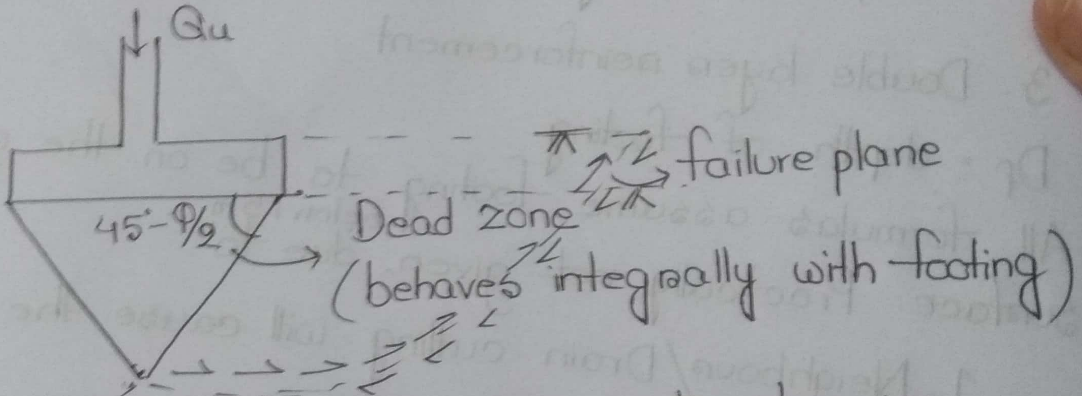
→ Overburden pro

→ Cohesion

$q = \gamma_{soil} D_f$

For clay soil, First two terms.

Sandy soil, Last two terms.



Foundation বায়ে হেলে যাবে। ডানে soil upheaving

Generalized shear failure.

*We want generalized shear failure for greater arc length. ② Ground surface নিচে থাকলে arc length কমে যাবে। ① Df যত বেশি নিব, তত বেশি friction develop করবে।

Q. How does Df contribute to bearing capacity?

Table 5.1. and 5.2 (We use Meyerhof)

Sand φ

Loosest $\varphi = 28°$ (minimum) [Direct shear repor

Dense $> 40°$

$\varphi = \sqrt{20N} + 15°$

$= 0.3N + 27°$

N (SPT) should be measured from $p = 1\,kg/cm^2 = 100\,kPa$.

$p \uparrow N \uparrow$

Confining pressure বেশি হলে N বেশি আসবে।

Correction factor $< 2 \rightarrow p$ বেশি হলে।

$q = \gamma D_f = \gamma D_w + \gamma_{sat} D_{w_1}$

$q_o' = \gamma D_w + (\gamma_{sat} - \gamma_w) D_{w_1}$

Pressure doesnot increase with depth infinitely. At critical depth, the pressure becomes fixed.

$Q_u = q_u \times A_f$ = Ultimate capacity of soil

Net bearing capacity $= q_u - \gamma D_f$

Net allowable bearing capacity, $q_{allow} = \frac{q_u ~~- \gamma D_f~~}{FS (=3)} - \gamma D_f$

Safe allowable bearing capacity $= q_{allow}$, when settlement okay.

(Serviceability consideration)

Settlement $= 1''$

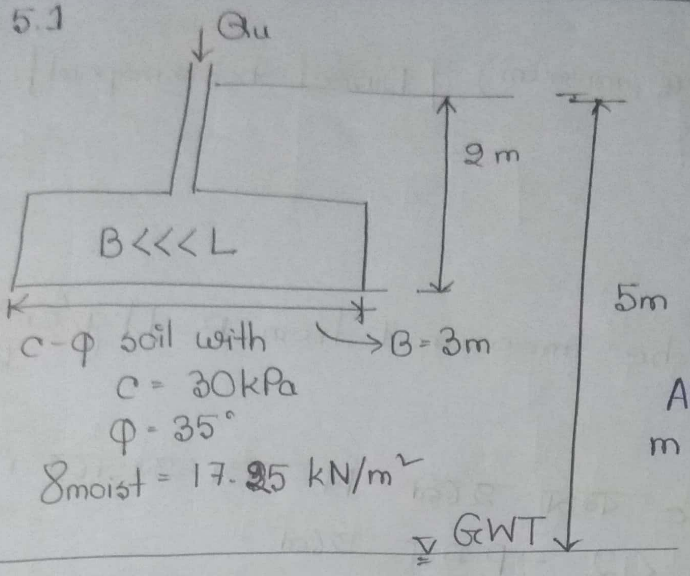
① Increase footing area

$q_u = cN_c S_c D_c i_c + qN_q S_q D_q i_q + \frac{1}{2} B\gamma N_\gamma S_\gamma D_\gamma i_\gamma$

(Generalized bearing capacity)

Formulas are for line footing. For others. Shape factor and Depth factor, Inclination factor (Table 5.3. Meyerhof)

Ex 5.1



c-φ soil with → B = 3m
c = 30kPa
$\varphi = 35°$
$\gamma_{moist} = 17.25\ kN/m^2$

B<<<L → Strip Footing

Find q_u, q_{nu}, q_{na}. Allowable load per m on footing

$$q_u = cN_c S_c d_c + q'N_q S_q d_q + \frac{1}{2} B\gamma N_\gamma S_\gamma d_\gamma$$

Shape factors (Table 5.3 ~ Meyerhof)

$S_c = 1 + 0.2N_\varphi (B/L)$

$N_\varphi = \tan^2(45° + \varphi/2) = 3.69$

$S_q = 1 + 0.1N_\varphi (B/L)$ for $\varphi > 10°$

$S_\gamma = S_q$; for $\varphi > 10$

$S_\gamma = S_q = 1$ $\varphi = 0°$

Depth factors

$d_c = 1 + 0.2\sqrt{N\varphi} \cdot (D_f/B)$

$d_q = 1 + 0.1\sqrt{N_\varphi} \cdot (D_f/B)$

$d_\gamma = d_q$ for $\varphi > 10°$

$d_\gamma = d_q = 1$. $\varphi = 0°$

$S_q = S_c = S_\gamma = 1$

$d_c = 1.257$

$d_q = d_\gamma = 1.128$

[Terzaghi, Table 5.1 for $\varphi = 35^\circ$, $N_c = 57.8$, $N_q = 51.4$, $N_\gamma = 42.4$]

Meyerhof [Table 5.1]

Linear interpolation, [Decimal should be included]

$N_c = 46$, $N_q = 33$, $N_\gamma = 37$ Better practice to use γ'

$q' =$ overburden effective pressure $= \gamma D_f = 17.25 \times 2 = 34.5$ kPa

$q_u = 30 \times 46 \times 1.257 + 34.5 \times 33 \times 1.128 + \frac{1}{2} \times 3 \times 17.25 \times 37 \times 1.128$

$= 1735 + 1284 + 1080$

$= 4099$ kPa

Q. Why γD_f is subtracted from q_u to get q_{nu}?

$q_{nu} = q_u - \gamma D_f = 4099 - 17.25 \times 2 = 4064$ kPa

Allowable, $q_{ne} = \frac{q_u}{FS} - \gamma D_f$ [Q. Why FS is not applied to γD_f?]

$= \frac{4099}{3} - 34.5$

$= 1332$ kPa

Because q_u is uncertain, γD_f is not

$Q_{na} = q_{ne} \times B = 3996$ kN/m

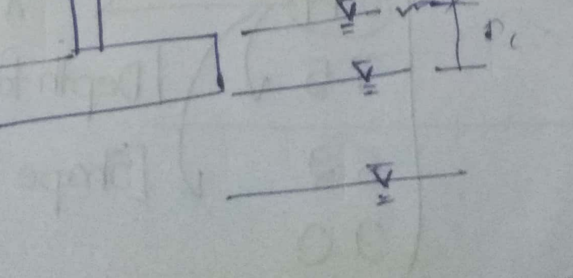
Q. Draw sc

* যদি GWT এর নিচে থাকে তাহলে q এর জন্য γ_{moist} ব্যবহার করতে হবে।

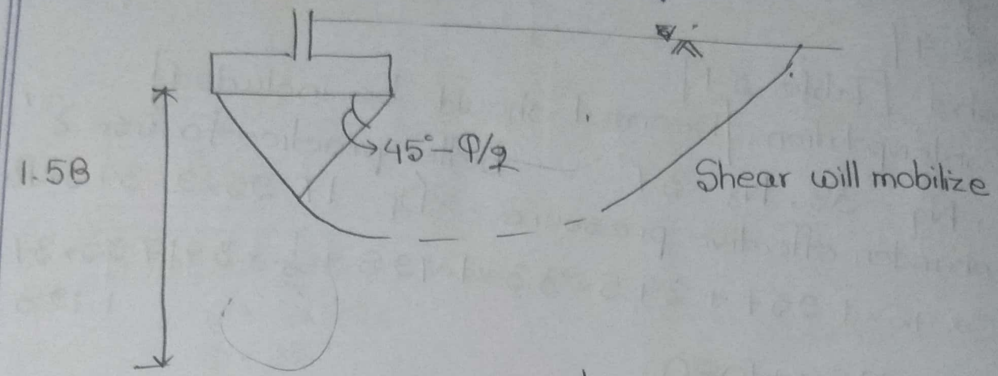
আর $\gamma = \gamma_{sat}\ \gamma_w$

γ_{sat} is maxm value.

* Try with different positions of GWT

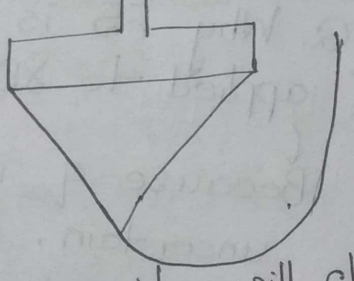


Q. Draw schematic diagram of general shear



1.5B পর্যন্ত compact করা must

ডানের অংশ dense না করা হলে,



Sharp failure, Arc length কমবে. Mobilization shear কমে

Local shear

Bearing capacity will change.

$c = \bar{c}' = 0.67c$

$\tan\bar{\varphi} = 0.67\tan\varphi$

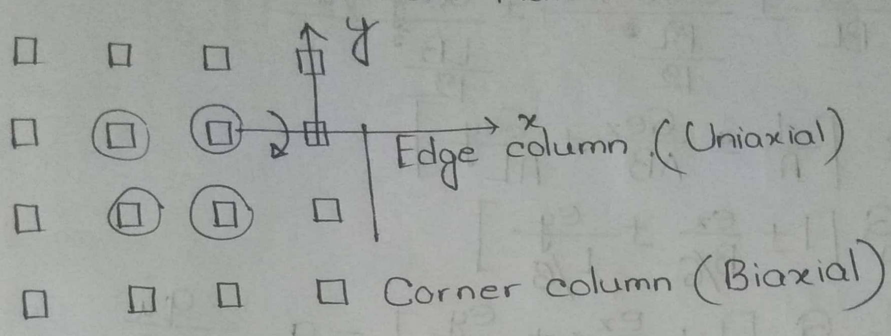
↳ N_c, N_γ, N_q

The lower and upper limiting value of N_c for strip and square footing.

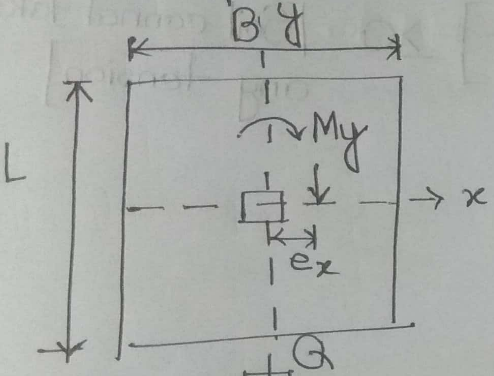
VIVA	Ratio D_f/B	Value of N_c	
Strip	0	5.14	→ $\pi + 2$ ~ (Elongated Circle)
	≥4	7.5	[Depth factor গুণ]
Square	0	6.2	[Shape factor এ গুণ]
	≥4	9.0	

Bearing Capacity of foundation subjected to Eccentric Loading:

Interior column- Unbalanced Moment থাকেনা।



Column pattern.



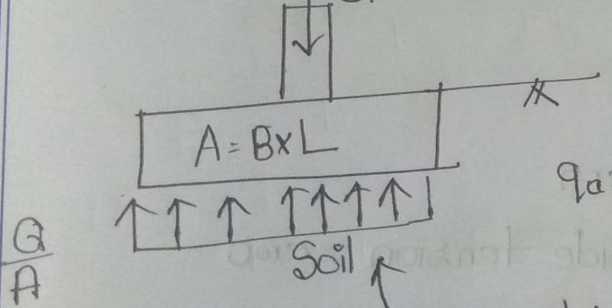
$q_u = CN_c S_c d_c +$

$\phi = f(N)$

Meyerhof, Q

N_c, N_q, N_γ

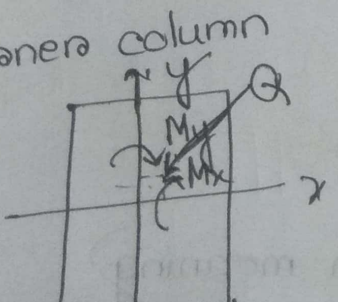
$S_c, S_q, \cancel{S_\gamma}\ d_c, d_q$



$q_a = \frac{q_u}{FS}$

Eccentric হলে uniform থাকবে না।

For corner column



$e_x = e_y$ = eccentricity of vertical loading.

$$q = \frac{Q}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$$

$x = \frac{B}{2}, \ y = \frac{L}{2}$ (Extreme points)

$M_y = Qe_x, \ M_x = Qe_y$

Date:

$$I_y = \frac{LB^2}{12} \quad I_y = \frac{BL^3}{12}$$

$$q = \frac{Q}{BL} \pm \frac{Qe_x \cdot B/2}{\frac{B^3L^2}{12}} \pm \frac{Qe_y \; L/2}{\frac{LB^3}{12}}$$

$$= Q\left[\frac{1}{A} \pm \frac{6e_x}{AB} \pm \frac{6e_y}{AL}\right]$$

$$= \frac{Q}{A}\left[1 \pm \frac{e_x}{B/6} \pm \frac{e_y}{L/8}\right]$$

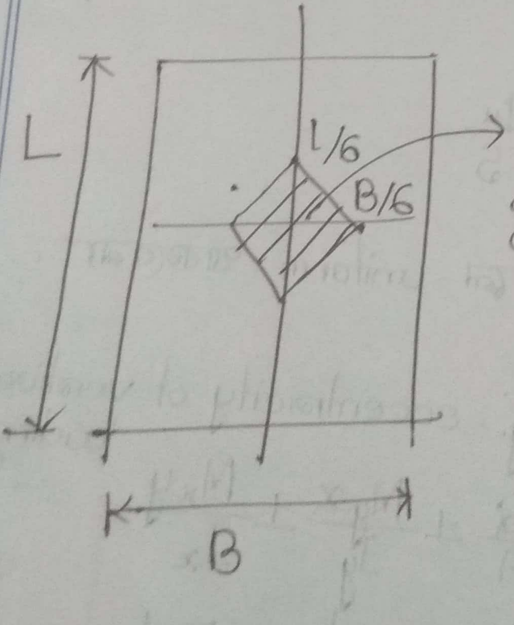
$$q_{max} = \frac{Q}{A}\left[1 + \frac{e_x}{B/6} + \frac{e_y}{L/6}\right] = q_a = \frac{q_u}{FS}$$

$$q_{min} = \frac{Q}{A}\left[1 - \frac{e_x}{B/6} - \frac{e_y}{L/6}\right] \geq 0$$ [Soil cannot take any tension]

$$e_y = 0, \quad 0 = 1 - \frac{e_x}{B/6} - \frac{e_y}{L6}$$

$$e_x = B/6$$

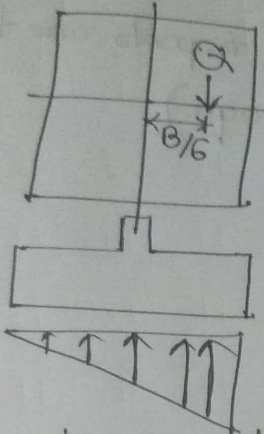
$$e_x = 0, \quad e_y = L/6$$



Kern

Inside tension zero

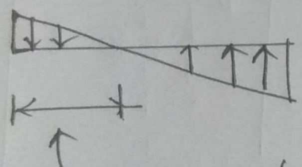
Q Develop Kern and explain meaning.



$q \leq q_a$

Triangular variation.

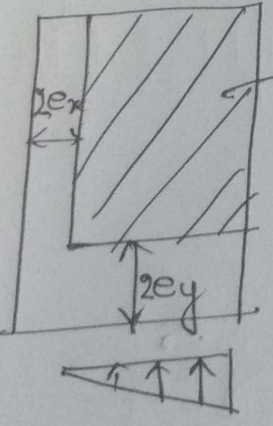
Footing resizing /Strap footing ~~if tension~~. if other cond are not verified.



$q_{max} < q_a$

This distance becomes ineffective.

Effective length of footing =, $B' = B - 2e_x$



→ Effective area of footing).
Capacity অধিক হয়ে যাবে (Triangular Variation)

$Q_u = q_u \times A'$

* Negative হয়ে যাইতে পারে area. e_x, e_y বেশি হলে.

Related 5.16, 5.15

c-φ soil

Only SPT is believable, almost all othero reporots are fake

* $\varphi = \cancel{\sqrt{Ncot}}\ \sqrt{20corN} + 15$ (Check eqn)

c from unconfined/triaxial

Pile integrity testing

5.40

$2c = q_u\,(kPa) = \bar{K} \times N_{cor}$

$\bar{K} = 12 - 25$

Read yourself: Layered Soil (Not for exam)

C.W

L-4

Date : 03/07/19

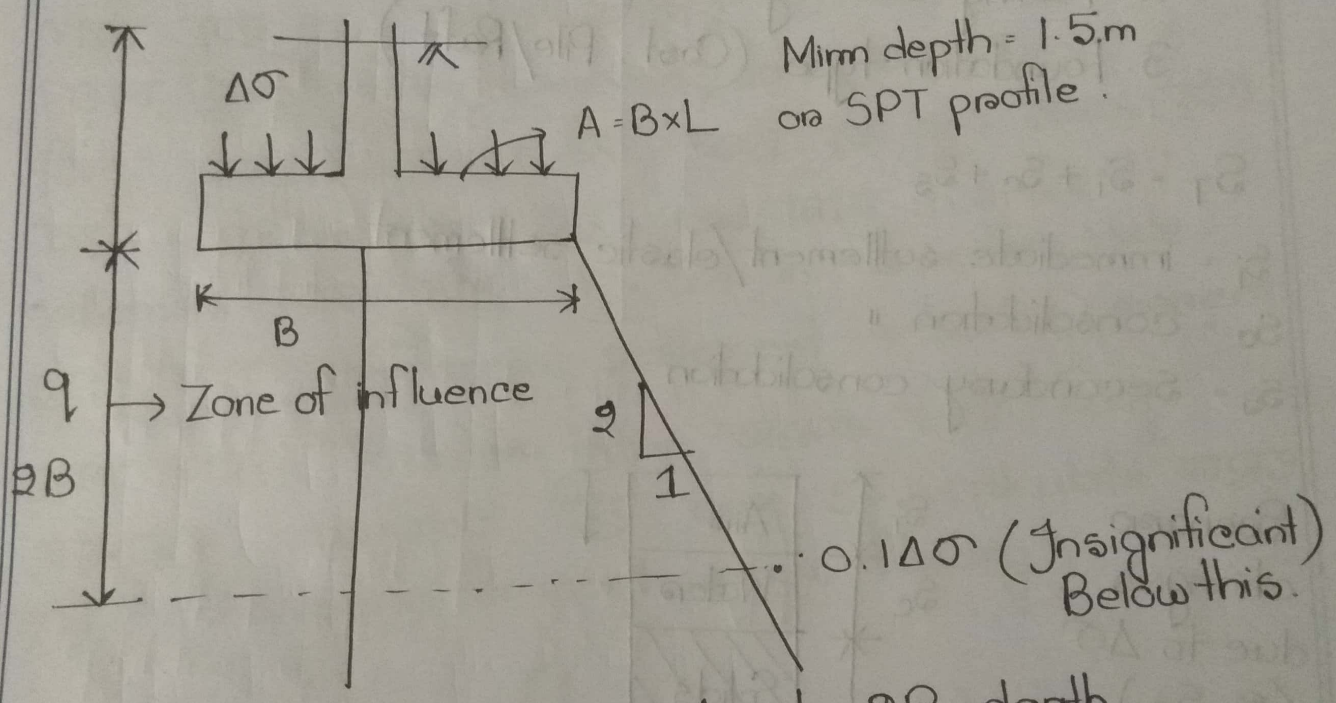
Safe Bearing Pressure and Settlement Calculation

[Chapt-3, Murthy]

$q_{ult} = cN_c S_c d_c + qN_q S_q d_q + \frac{1}{2} is \gamma N_\gamma S_\gamma d_\gamma$

$c \rightarrow \phi \cdot N_c, N_q, N_\gamma \rightarrow f(\phi)$

$N \rightarrow f(\phi)$



Normally, Δσ becomes 10% at 2B depth.

$q_{allow} = \frac{q_{ult}}{FS} \cancel{\pm} \Delta\sigma$

Δσ must be unfactored as FS is applied later.

$q_1' = \gamma D_f = (\gamma_{sat} - \gamma_w) D_f$

Settlement condition:

$S_T < 25.0$ mm (Normal)

$< 2\times25$ mm (Raft)

If settlement is not satisfied, adjust:

1. Footing depth (নিচে নামতে হবে)
2. Footing arrangement (Width)
3. Foundation type (Cost, Pile/Raft)

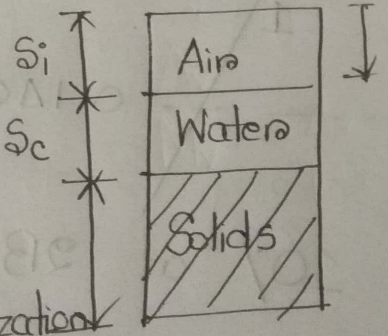
$S_T = S_i + S_c + S_s$

S_i = immediate settlement/elastic settlement

S_c = consolidation "

S_s = Secondary consolidation.

Scenario changes due to $\Delta\sigma$ (S_i, S_c)



Fabric reorganization causes secondary settlement (insignificant)

For, sand, $S_T \approx S_i \longrightarrow f(E, \nu)$

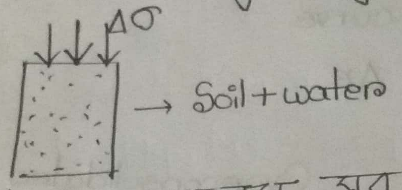
clay $S_T \approx S_c$

E = Elastic modulus

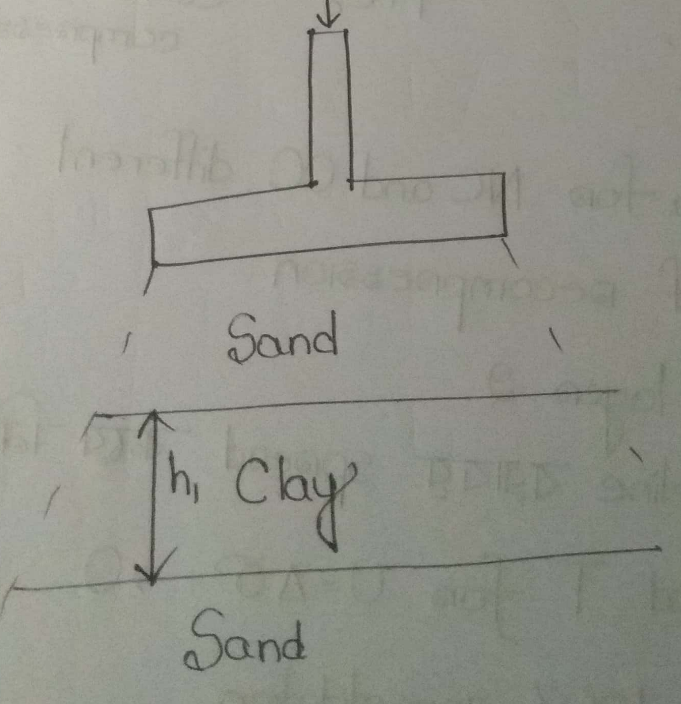
ν = Poisson ratio.

[Math আসবে]

Calculation assuming clay saturated.



যার strain কম সে সব load নিবে। পানি incompressible তাই water load নিবে।
পানি বের হয়ে যাবে। তারপর soil এর উপর pressure পড়বে যদি free drainage / sand থাকবে।
কিন্তু clay র permeability কম তো খুব আস্তে বের হবে। Soil longtime পরে load নিবে। So, final settlement বোঝা সম্ভব না। Soil in transition state.
হঠাৎ করে differential settlement → twisting moment.
ঢাকা শহরে ∇ Water Table অনেক নিচে। কিন্তু তারপরও saturated ধরে max S_T calculate করতে হয়। (Accidental trapped water, flooding)



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

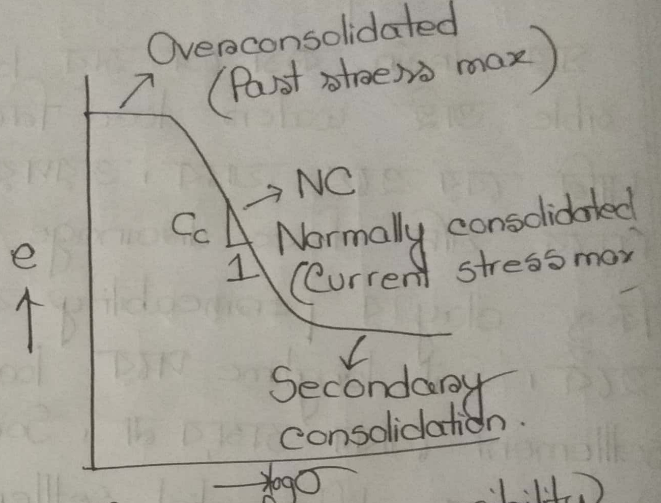
e- logP curve

$q' - 50$ kPa

$\Delta P \rightarrow \Delta e$

$\Delta\sigma - 200$ kPa

σ'	Δe	e_o
10		
20		
40		
80		
200		
400		
800		



C_c = coefficient of compressibility

C_v = coefficient of consolidation

A_v

$M_v = \frac{A_v}{1+e_o}$ | Check

Coefficient of volume compressibility.

$$\frac{C_c}{1+e_o} \log \frac{P+\Delta\sigma'}{\sigma_o}$$

* Settlement formula for NC and OC different

C_r = coefficient of recompression.

Recalculated for layer 2.

$\Delta\sigma$ layer centreline বরাবর spread করে নিতে হবে,

Consolidation period T for $U = \Delta\sigma \rightarrow 0$

* Time reqd for 100% consolidation

$\beta \begin{cases} NC & 0.5-0.7 \\ OC & 1 \end{cases}$

$S = \beta_1 S_{old}$

* Reasons for using factor β
Settlement within design life
বইয়ে আলাদা reason (p222) Fig 6.14.

Example 6.12, P-223
[Exam এ এত layer থাকবেনা]

$$S_{oc} = \beta_1 \frac{C_R}{1+e_0} H \log \frac{\sigma_c'}{\sigma'} \beta_2 \frac{C_c}{1+e_0} H \log \frac{\sigma_0'+\Delta\sigma}{\sigma_c'}$$

Initial/Final

$\beta_1 = 1.0$
$\beta_2 = 0.5-0.7$

$$S_{nc} = \frac{C_c}{1+e_0} H \log \frac{\sigma_0'+\Delta\sigma}{\sigma_0'}$$

C.W.

CT. Next Saturday 1 pm. Syllabus: (L 1-2)   L-5

Date: 10/07/19

Pile Capacity for Vertical Loading

- Pile
  - Vertical loading
  - Lateral loading (moment) →

- Material
  - RCC
    - PHC (Prestressed high strength concrete)
    - Solid
  - Steel
    - Hollow
    - I-beam
  - Timber

High moment আসলে hollow pile করে section বাড়ানো হয়।

Coastal/River port.

- Subsoil condition
  - Sand
  - Clay
  - Both

[Different formula]

- Capacity based:
  - Friction → Q_f
  - End bearing → Q_b
  - Both → $Q_f + Q_b = Q_u$

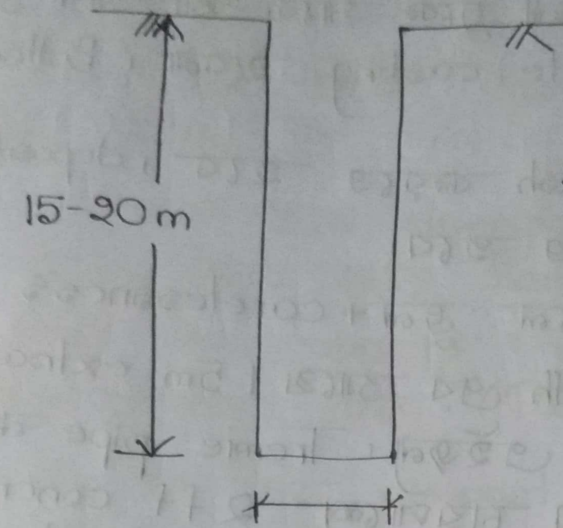
- Driving method -
  - Bored / Cast in situ / Situ
  - Driven / Precast pile

Driven & situ → Different type

- Boring ——
  - Auger Boring
  - Wash boring
  - Rotary drilling / rig

Q_u ও Q_b footing এর মতই বের করতে হবে।

Usually friction and bearing দুইটাই থাকবে।



Red soil = No pile
Keraniganj = 30m

B = D (20"–24") [Building]
= 1.5m [Bridge]

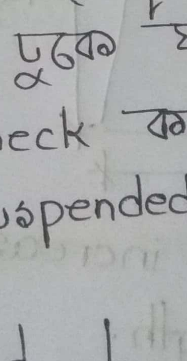
Fig: Bored Piling

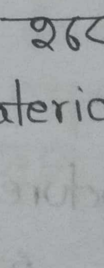
Boring: ছোট dia drill দিয়ে মাটি loose করা যায়।
High pressure water jet
অন্য পাশে water + material.

* For lateral hole stability, bentonite মিশায়ে sp gr বাড়ানো হয়। (Liquid limit: 300-900%)
নাহলে soil ভেঙে ঢুকে যাবে পাশে।

* Outward water check করতে হবে। পানি clean হলে নিচে suspended material থাকবে। তারপর piling.

* Reinforcement
4"-5" cement sand এর roller support দেয়া হয়। Erosion and corrosion.





Bottom এ steel এ soil ঢুকে যাবে। Soil আর Steel আর ঢুকবেনা। Concrete casting হবেনা। Bottom এ corrosion.

তখন steel উঠায়ে wash করতে হবে, deposition.

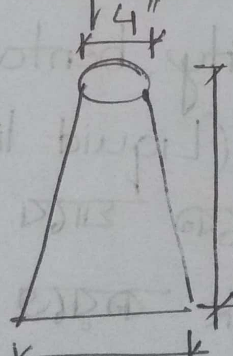
Pile load test করতে হবে।

মাঝরাতে casting হলে ভুল + carelessness

Bored pile normal length এর আগে 1.5m extra concreting করতে হবে এইজন্য। Tremie pipe নামিয়ে concreting দেয়া হয়। Outlet এর চারপাশে 2 ft concreting করতে হবে। তারউপর একটু উঠায়ে fresh concrete এর surface এর নিচেই রাখতে হবে। Surface এর উপরে উঠাইলে mud soil mix ক হয়ে যায়। একজায়গায় রাখলে concrete hardening হয়ে যাবে।

Self compaction required. Flowability থাকা লাগবে।

Slump test:



Add admixture to increase flowability. Adding water decreases strength.

Bottom এর কাদা mixed অবস্থা উপরে উঠতে থাকে Complete হলে top থেকে 2' কেটে ফেলা হয়।

VIP Construction Problems in boring pile:

নিচে loose ও soil থাকে। cast in situ তে friction develop করেনা।

Cast precast হলে strength and quality ensured করা যায়। জোর করে hammering করে ঢুকানোও extra load than capacity ও দেয়া হয় soil. পাক্কের soil তখন compact হবে (Passive fail, $K_p > 1$)

Cast-insitu তে নিচে compact হয়না। End bearing কম থাকেই। আগেই settlement হয়ে যায়।

Rotary drilling: water লাগবেনা। Best method but expensive machine. Also stabilizer লাগে।

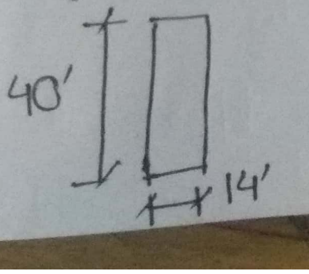
Pile এর সাথে attachment:

Problems: 1. Noise
2. Vibration
3. CO_2 (Diesel only)

Drop hammer (Dynamic)
- Diesel
- Hydraulic (Better but doubly expensive)

Hammer এর wt pile এর 50%.

শহর এলাকায় drop hammer use করা যায়না. শহরের outskirt এ।
এর পরিবর্তে static hydraulic pile driver (380/420ton)



No vibration, pollution or sound but huge transportation cost

Date :

Huge machine transport করার মত road নাই, আর large project location লাগবে (>400 pile) চেপে ধরে hydraulic pressure মাধ্যমে। দেয়া হয়।

C.W

CT Next Wednesday CT, 1.00pm L-6

L-6 upto Math Date: 17/07/19

Vertical Capacity of Pile (Clay)

- Single pile
- Group pile

For footings • Q_u (N_c, N_γ, N_q, D_f, $B \times L$)

Planning, • $Q_a = \frac{Q_u}{FS}$; $Q_{a,net} = Q_a - W_{self\ wt}$

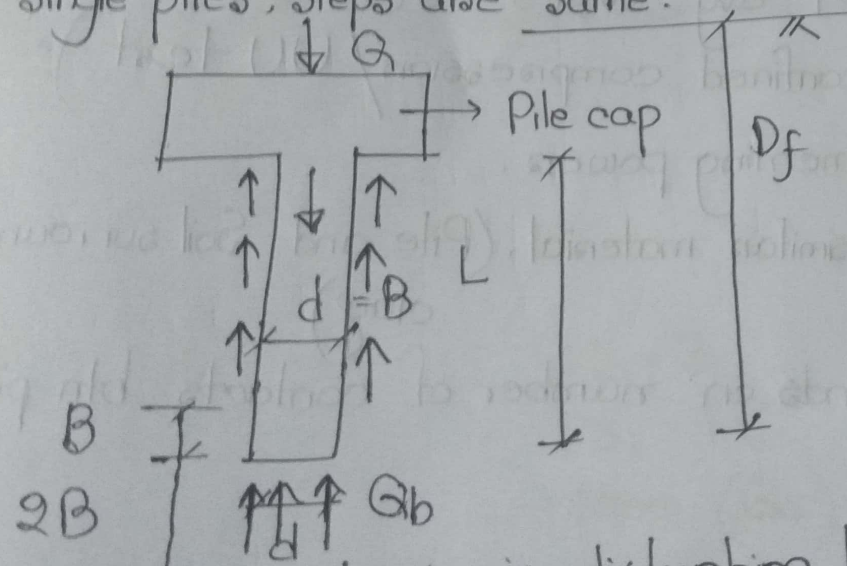
Soil Investigation Check whether, $Q_{a,net}$ > Unfactored Superstructure load.

feasibility Study? If not, change footing area, depth or design

• Check $Q_{a,net}$ or Unfactored superstructural load, with settlement (25mm − footing, 50mm − pile). $Q_{a,net} \simeq Q_{safe}$.

Q. Write down the steps for footing design. from planning

For single piles, steps are same.



Superstructural load is disturbing ~~b~~ load.

⑥ Geotechnical capacity, $Q_u = Q_f + Q_b$

$$Q_b = cN_c S_c D_c + qN_q S_q D_q + \frac{1}{2} B\gamma N_\gamma S_\gamma D_\gamma$$

For clay, $Q_b = cN_c S_c D_c$

D_f/B = Depth factor (Becomes fixed at a certain depth)

$S_c = 1 + 0.2 N_q \frac{B}{L}$

$= 1.2$ (Square or Linear)

N_c = ~~5.16~~ ~~5.18~~ (For linear, surface footing)

$= 5.14$

Pile, $N_c = 5.14 \times 1.2 \times$ Depth factor = 9.0

For pile,

$Q_b = c N_c \frac{\pi}{4} d^2 = \frac{1}{8} q_u N_c \pi d^2$

(Unconfined compression, $c = \frac{q_u}{2}$)

Influence zone 2B পর্যন্ত যে c আসবে দুইটার avg
নিতে হবে। Foundation bottom এর উপরে 1B উপরে আর 2B
ওর মধ্যে SPT avg করতে হবে। তারপর c correlate

c from (Unconfined compression / UU test)

Cohesion = Cementing power.

Adhesion = Dissimilar material. (Pile and Soil surrounding clay)

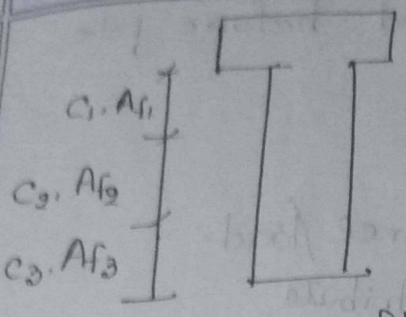
Friction depends on number of contacts btn pile and clay.

$A_f = \pi D L$

$Q_f = c \alpha_c A_f$ α_c = adhesion factor

= 1.0 for clay soil to soil

< 1.0, otherwise



If $c_1 \approx c_2 \approx c_3$, avg c
If not, calculate separately.

Varagese
Table 9.2
Figure 9.6

α_c = Very soft = 1.0
Very stiff = 0.3 – 0.2

For soft soil, high frictional capacity.

$Q_b = 90\% \, Q_u \underline{>>} Q_f$ [End bearing pile., Usual]

$Q_f = 90\% \, Q_u >> Q_b$ [Friction pile, Unusual]

↑ If soft layer is very deep.

Comilla has highly compressible soil, so deep friction pile reqd.

[Math from sheet]

$Q_u = Q_b + Q_f$

FS; $Q_{a_1} = \frac{Q_b}{FS_1} + \frac{Q_f}{FS_2}$

or. $Q_{a_2} = \frac{Q_b + Q_f}{2.5}$

$FS_1 = 3$
$FS_2 = 1.5$
Minm (Q_a)

Uncertainty in load and soil material.

Certainty, $Q_b < Q_f$

Pile settlement = Elastic + E + Consolidation + Secondary

Part of Q_f already mobilized before pile.

Q_b will be anc carrying.

Certainty বেশি Q_f এ।

Structural Capacity?

$Q_s = 0.25 f_c' A_c$ >> Design force/load.

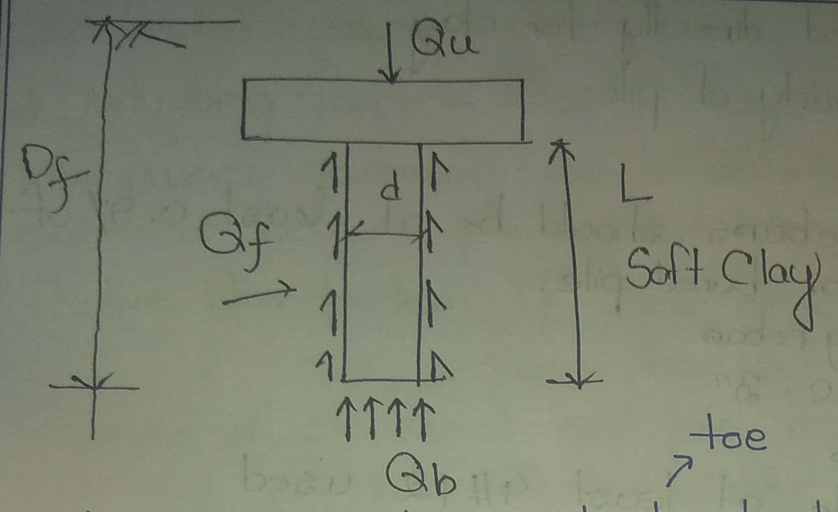
Reinforcement doesnot contribute.

C.W

Sat Sun Mon Tue [✓ Wed] Thurs Fri

L-7

Date: 24/07/19



Q Why do we choose pile tip to be rested on the granular deposition rather than stiff clay?

$Q_b = [\underset{150}{c}N_c] * A_b$ [End bearing if pile tip rested on stiff clay]

$= [\cancel{100}\times 9] A_b$

$Q_b = [qN_q] A_b$, [Rested on dense sand]

Berenzeshaw's $N_q = \cancel{2}75$

$= 150\times 75\times A_b$ $\gamma_{sat} = 20-9.8$

$D_f = 15m$

$\frac{Q_{b,sand}}{Q_{b,clay}} > 8$ [Design for ten times greater strength on clay]

$Q_u = Q_b + Q_f$

↓

Q_a

| Settlement

↓

Q_{safe}.

① Must also check structural capacity.

② Qb, Qf has limiting values. (For sand, Qb

Ref. Varghese P 184-187

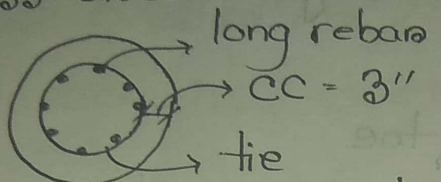
For clay: $Q_{f,limit} \leq 7$ ton/m² to be applied on ultimate capacity.

Q_b can be used directly for clay.

Structural capacity of pile:

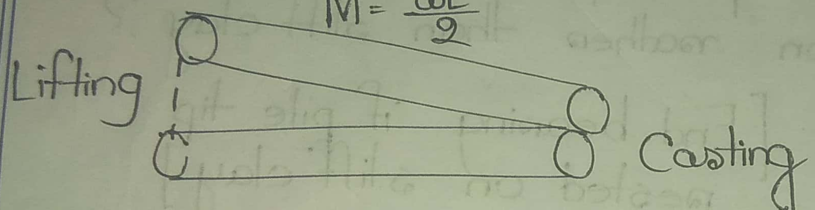
Similar to pile.

. Longitudinal rebars should be at least 0.8% of gross area for bored pile.



For precast pile, at least 4#12 used

$$M = \frac{wL^2}{2}$$



Lifting করার সময় moment আসবে।

Contribution of longitudinal rebar must not be considered in evaluation of structural capacity of pile.

$$(Q_{str})_{allowable} \le 0.25 f_c' * A_c \le 5\ N/mm^2$$

(~~B~~. BS Code)

Cube strength.

$Q_{str,a}$, Q_a এর মধ্যে যেটা ছোট ওইটা govern করবে।

$Q_u = Q_b + Q_f$

Skin friction: $c\alpha_c \longrightarrow Q_f$

↓ Disturbing force = Reaction force ↑

Pile pushes downward due to disturbing load, relative movement between pile and soil material.

Positive skin friction: If piles move/shorten faster than soil.

Because of structural load, pile moves primarily elastically and then elastoplastically. S_1

At the same time, surrounding soil will undergo deformation due to – consolidation acted on by overlying fill. (surcharge)
S_2 – seasonal variation of water table

$S_1 - S_2 \Rightarrow$ Positive friction (↑)

$S_2 - S_1 \Rightarrow$ Negative skin friction (↓) – Soil will ~~be~~ dragg~~ed~~ down pile.

in that case, effective friction of the pile is downward and acting along the direction of working load.

$$Q_u + \underset{\downarrow}{Q_f} = Q_b$$

আগে FS = 1.5 apply করতাম, এখন করবনা.

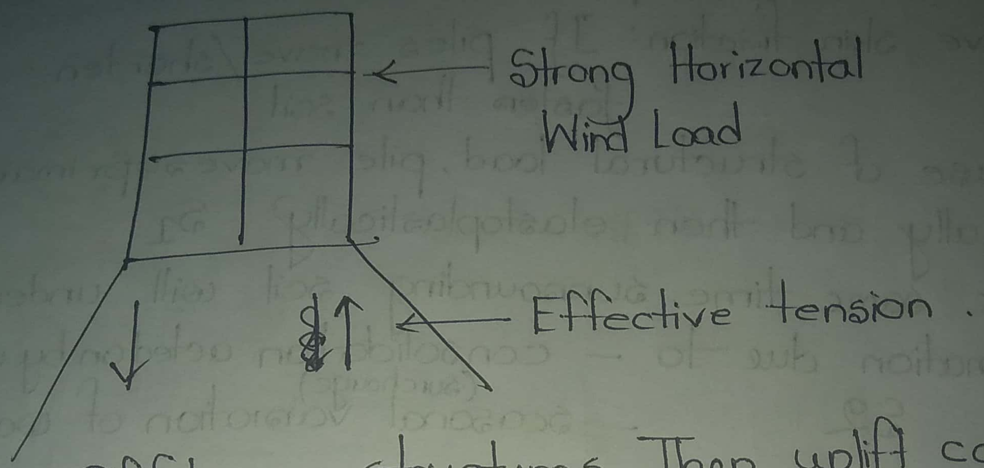
Ref: P-191 Varghese.

- reason/area negative skin friction appears
- remedial measures to avoid –skin friction.

(1. Driven pile এ bitumen layer দেয়া হয় so no friction)

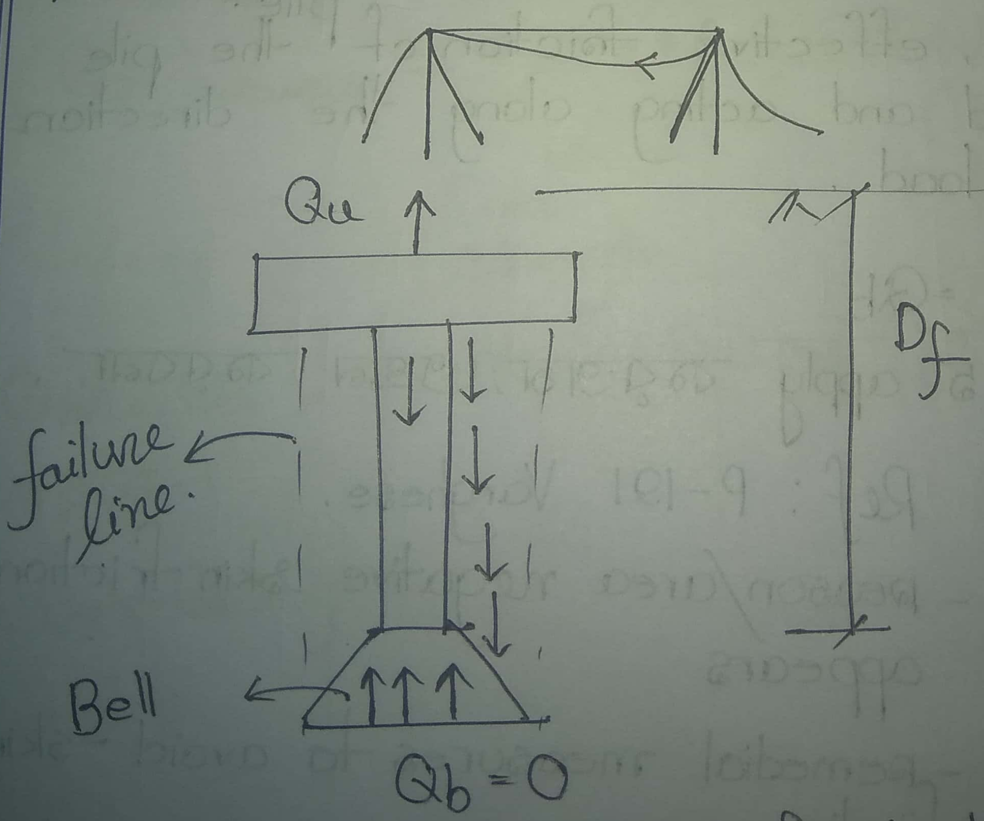
2. First casing, then viscous material দিয়ে casing উঠায় নেয়া হয়।

Uplift Capacity of Pile



Coastal, Offshore structures. Then uplift capacity governs.

Transmission towers:



$$Qu = \frac{Q_f}{FS} + \underbrace{\text{Overburden wt of soil} + W_{pile}}_{\text{Known}}.$$

$Q_f = c\alpha_c A_f$

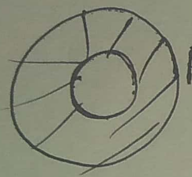
$= (c\alpha_c)(\pi DL)$

- increase b. overburden pressure (limited)
- increase dia (costly)
- provide bell $\downarrow$

$D = D_{Bell}$

~~D~~ =

$\alpha_c = 1$ (Sail to soil action along failure line)



Pile tip / toe.

$Q_b \neq 0$.

C.W

Sat Sun Mon Tues [✓ Wed] Thurs Fri

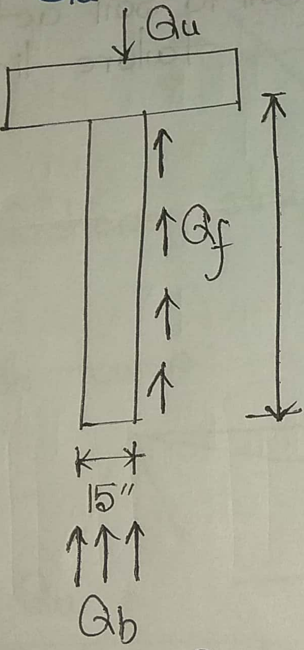
Ref: Varghese
Murthy

L-8

Date: 31/07/19

Problems:

A concrete pile of 15" dia, 40' long is driven into a homogenous stratum of of clay with the WT at the GL. The clay is of medium stiff consistency. with the undrained shear strength $C_u = 600 \#/ft^2$ Compute Q_b by Skemptons method and ~~d~~ Q_f by α-method. Also determine the pullout capacity and the allowable pullout load with FS = 3.0 Determine Q_a for ~~fo~~ overall ~~p~~ FS = 2.5.



Skempton method, $Q_b = cN_cA_b$

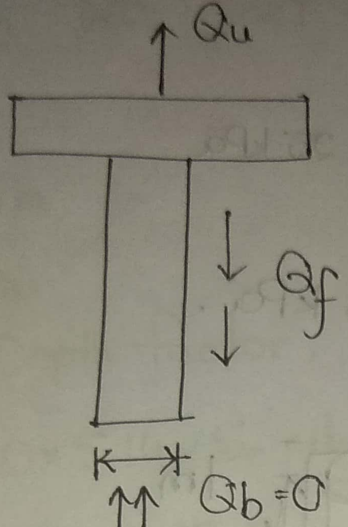
α-method, $Q_f = \alpha_c\, c\, A_f$

$Q = Q_b + Q_f$ [Careful about upper limit of Q_f]

[FS maybe applied by parts or overall, whichever gives minm Q]

Q. Why the FS values are different?

Pullout



$Q_u = Q_f + W_{pile}$

$= \frac{Q_f}{FS} + W_{pile}$ [As $Q_b = 0$, there is no secondary resistance besides Q_f. So high uncertainty, $FS = 3$]

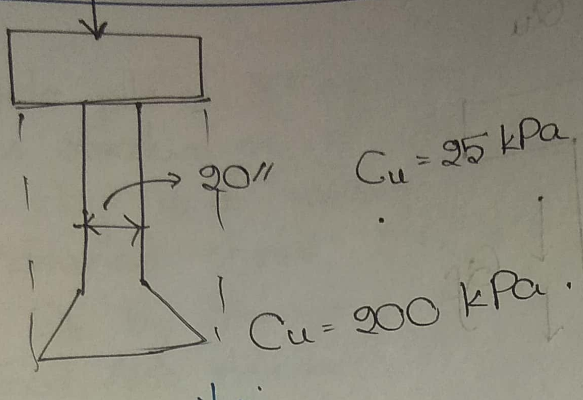
$=$

Tips for uplift :

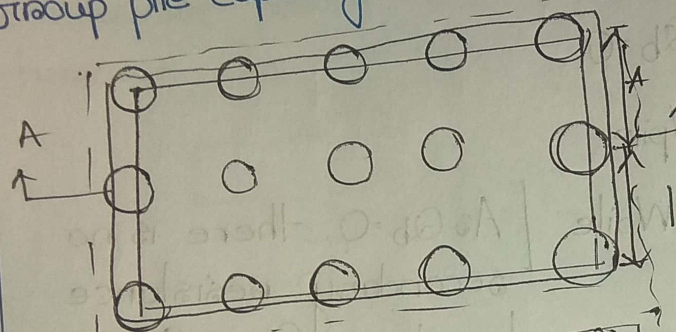
- Which loads are downward and which are upward.
- Uplift capacity ≈ < $\frac{2}{3}$ Downward capacity
- Brow and Devis (1980)
- Then apply $FS = 3.0$ to determine Q_{allow} and Q_{safe}.

No settlement criteria.

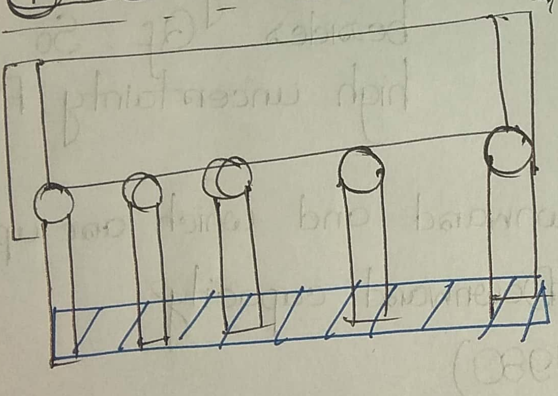
$Q_{safe} = 50^k$



Group pile capacity:



N = 5×3 = 15 piles.



S_c at 1-1

Single pile কখনো দেয়না।

S = (Spacing) = 2.5 − 3D

Considering single pile option:

Apply single pile action. $(Q_s)_u = \cancel{(Q_L)^{*} \; Q}$

$= (Q_b)_s + Q_f$

$Q_u = 1.5 Q_u$

Offset বাদ দিয়ে একটা footing ~~deep~~ depth of

$(Q_b)_g$ =

D_f = 20 m

B = 2.5m

$D_f/B \rightarrow$ Depth factor, = $\left(1 + 0.2\frac{AB}{L}\right)$ = upper limit

$(Q_b)_g$ = $30 \times 5.14 \times S_c \times d_c$ = ~~t~~. > 9.0

(Figure 16.14)

Deep foundation আর normal same হবে।

~~$A_f = (4.5 + 2.5) \times 2 \times L$~~ (Perimeter)

~~A_f = Perimeter = $(4.5 + 2.5) \times 2 \times L$~~

$A_f = \underbrace{(4.5+2.5) \times 2}_{\text{Perimeter}} \times L$

Around 90 prerimeter, 99% is clay to clay cohesion. Reasonable to assume $\alpha_c = 1$.

$(Q_f)_g = c\alpha_c A_f$

Group action ও single pile এর মধ্যে minimum

Efficiency = $\frac{\text{~~Sin~~ Group action}}{\text{Single pile}} \times 100$

Group action ছোট হবে due to small spacing of pile.

Excersizes

5.21

$Q_{ult} = A'\left[qN_qS_qd_q + \frac{1}{2}B'N_\gamma S_\gamma d_\gamma\right]$

$A' = B' \times L$

$= (B - 2e_x) \times L$

$= (10 - 2) \times 20$

$= 8 \times 20$

$= 160\ ft^2$

$q = \gamma D_f = 110 \times 8 = 880\ lb/ft^2$

$N_q = 33.55$ (M)

$N_\gamma = 37.75$ (M)

$S_q = 1 + 0.1 N_\phi \frac{B'}{L} = 1.1476$

$S_\gamma = S_q = 1.1476$

$d_q = 1 + 0.1\sqrt{N_\phi}\frac{D_f}{B'} = 1.192$

$d_\gamma = d_q = 1.192$

$N_\phi = \tan^2\left(45^\circ + \frac{35}{2}\right) = 3.69$

$Q_{ult} = \underline{6497 \cdot kip}$

5.22

$A' = B' \times L'$

$= (25 - 2\times4)\ (25 - 2\times3)$

$= 17 \times 19$

$= 323\ ft^2$

$q = \gamma D_f = 120 \times 8 = 960\ lb/ft^2$

$N_q = 64.1$ (M)

$N_\gamma = 93.6$ (M)

~~$s_q = 1 + 0.1 N_q \frac{B}{L}$~~

~~$s_\gamma = s_q$~~

~~$d_q = 1 + 0.1 \sqrt{N_q} \frac{D_f}{B}$~~

$s_q = 1 + \frac{B'}{L'} \tan\varphi = 1.75$ (H)

$s_\gamma = 1 - 0.4 \frac{B'}{L'} = 0.642$ (H)

$d_q = 1 + 2\tan\varphi (1 - \sin\varphi)^2 \frac{D_f}{B'} = 1.02$ (H)

$d_\gamma = 1$ (H)

$Q_{ult} = 55276$ kip

$Q_{allow} = \frac{55276}{3} = 18425$ kip

5.23

$Q = 800 \text{ ton} = 1760 \text{ kip}.$

$$q_{max} = \frac{Q}{A}\left(1 + \frac{6e_x}{L} + \frac{6e_y}{B}\right) = \frac{\cancel{176}\,800}{25^2}\left(1 + \frac{6\times3}{25} + \frac{6\times4}{25}\right) = 3.4304 \text{ tsf}$$

$$q_{min} = \frac{Q}{A}\left(1 - \frac{6e_x}{L} - \frac{6e_y}{B}\right) = \frac{800}{25^2}\left(1 - \frac{6\times3}{25} - \frac{6\times4}{25}\right) =$$

$$= -0.8704 \text{ ton/ft}^2$$

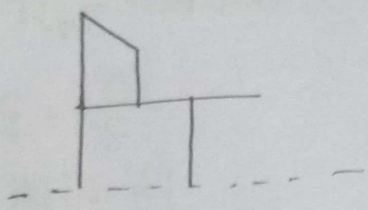
CW

Sat Sun Mon [✓ Tue] Wed Thu Fri

L-19

Date : 27/06/19

Example 9.2



β-method (For cohesionless soil)

$$Q_f = \int_0^L P\bar{q}_0 \bar{K}_s \tan\delta \, dz$$

For a given soil and pile material, β constant.

$$\beta = \bar{K}_s \tan\delta$$

Poulus's curve will give greater friction than meyerhof

So, meyerhof should be used when conservative.

Boring friction < Driven pile.

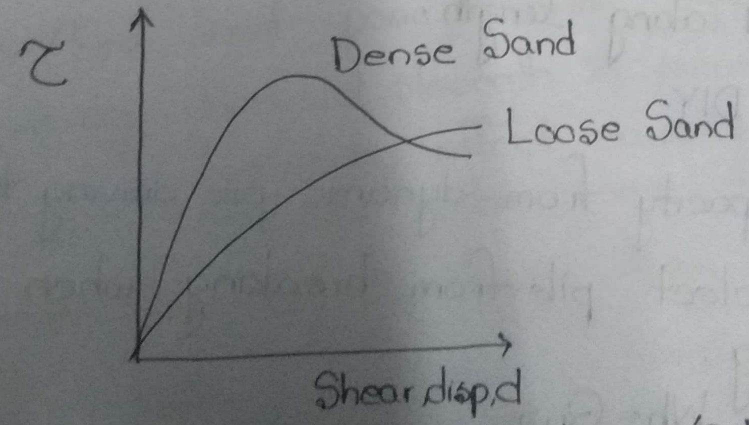
↓

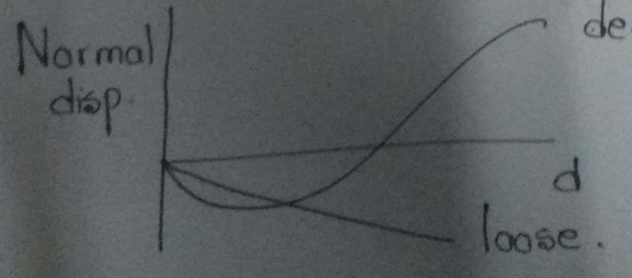
When holes are made, soil becomes loose.

$\varphi = 0.75\varphi_1 + 10°$ [driven] ~ becomes compact while driving

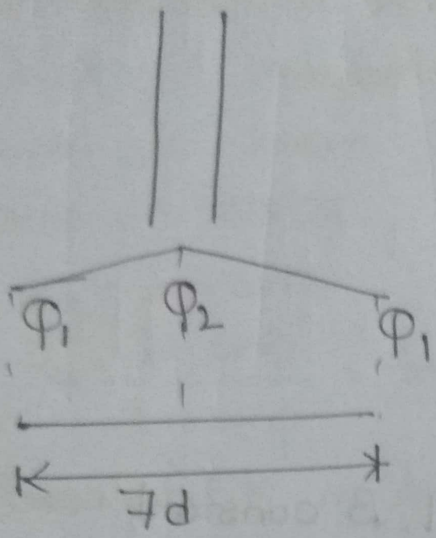
$\varphi = \varphi_1 - 3°$ [bored] ~ becomes loose while boring

যত খুঁড়ে যাব φ কমবে।





(Dilatancy – Volm বাড়ে কারণ একটা particle আরেকটার উপরে উঠে যায়)



$\varphi_2 = \frac{\varphi_1 + 40}{2}$ φ_1 from test

Insitu $\varphi = \sqrt{20N_{cor}}+15^\circ$ (For sand)

* Example 9.1, 2 এ φ modify করা হয়নাই।

Bearing capacity of piles in granular soils based on SPT-N

Unit sensitive.

N = SPT near end bearing zone

$\overline{N}$ = avg SPT along length.

Example 9.3 DIY

Pile bearing capacity from dynamic pile driving formula

Pile cap : to protect pile from breaking when driving.

Ideal condition, $Wh = Q_{us}$

Energy loss.

Hiley formula

η_h = hammer efficiency.

$$\varepsilon = \frac{\sigma}{E}$$

$$\Delta L = \varepsilon L$$

$$= \frac{\sigma}{E} L$$

$$= \frac{P}{AE} L$$

C_r = coefficient of restitution

$$= \frac{\text{velocity after collision}}{\text{" before "}}$$

Engineering News Formula

$$Q_a = \frac{Wh}{6(s+c)} \qquad \underline{F.S = 6}$$

Applicability is important

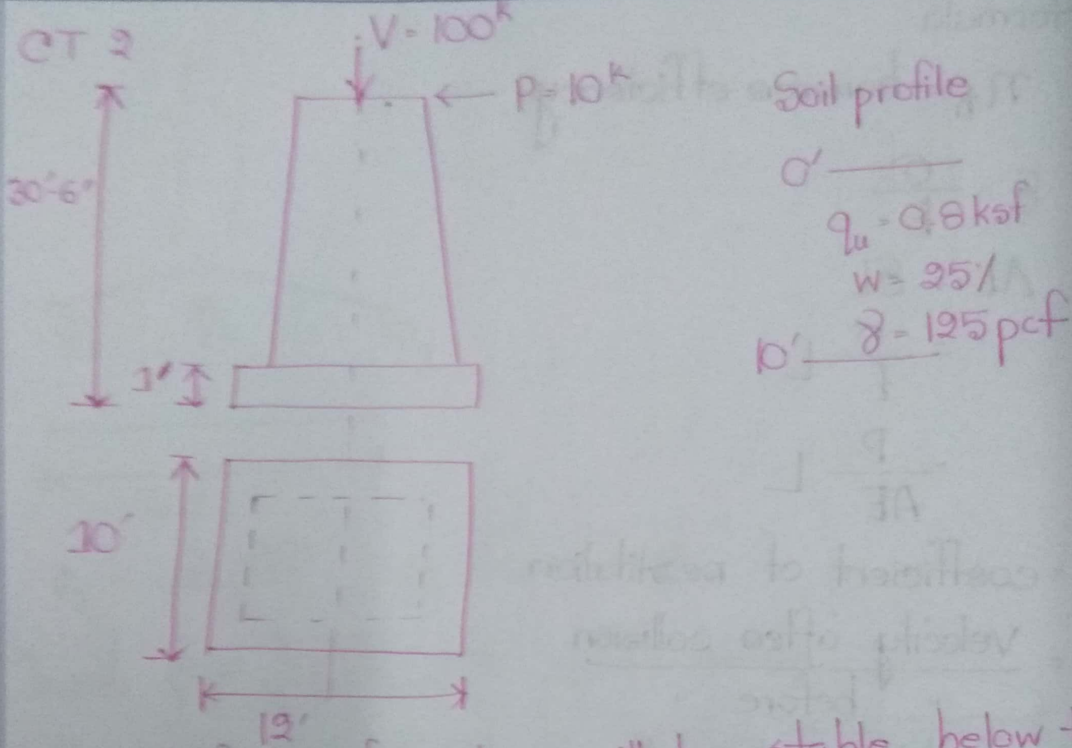
VIP Comments on dynamic formulae

Silty / Fine sand – pore pressure – liquefaction.

Sensitivity : Reduced strength in remoulded clay.
Thixotrophy effect causes to regain strength within some time.

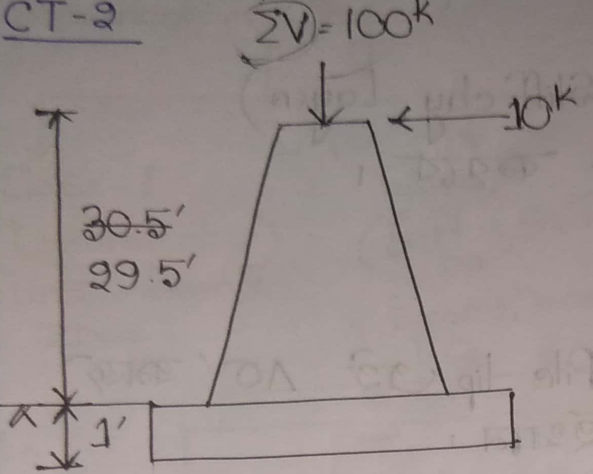
Dynamic load test is faster than static load test.

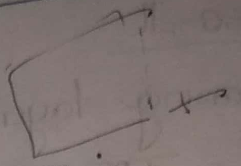
CT 2



Comment if the foundation will be stable below the structure subjected to 100k vertical and 10k horizontal load. Also write the remedial measures.

CT-2





$M = 10 \times 30.5$
$= 305$ k-ft

Soil profile এ q_u = unconfined compression strength দিল।

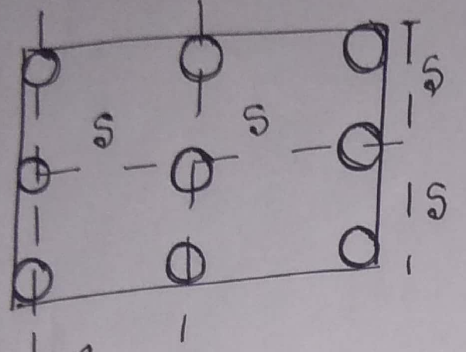
$$\sigma_x = \frac{P}{A} \pm \frac{Mc}{I}$$
$$= \frac{100}{10\times12} - \frac{305\times6'}{10\times\frac{12^3}{12}}$$

Remedy,

① Increasing depth wont affect much

② Good practice – change to pile.

P482/ Uplift and Group pile
Murthy



9 pile, 30 cm dia

Prob 10.1

A group pile of 9 friction piles arranged in a square pattern is to be proportioned in a deposit of medium stiff clay. Assume the piles are 30cm diameter and 10 cm long. Assume $\alpha = 0.8$ and $C_u = 50$ kpa. The bottom of the pile cap embedding all the piles rest at a depth of 1.5m below the ground surface. Find the optimum spacing for the piles.

⟹ Optimum = 100% efficiency.

Individual pile action:

Friction pile, $Q_b = 0$ [Neglecting]

$Q_u = Q_f = \alpha c A_f = 0.8 \times 50 \times (\pi \times \frac{30}{100} \times 10)$

= ~~379~~ KN = 377 kN

$Q_{ui} = Q_u \times n = 377 \times 9 = 3391$ kN

Group action, $Q_b \approx 0$; perimeter, $p' = 4 \times (s + s + 2 \times \frac{d}{2})$

$Q_{ug} = (\alpha c) A_{fg}$ = [Soil to soil, $\alpha = 1$]

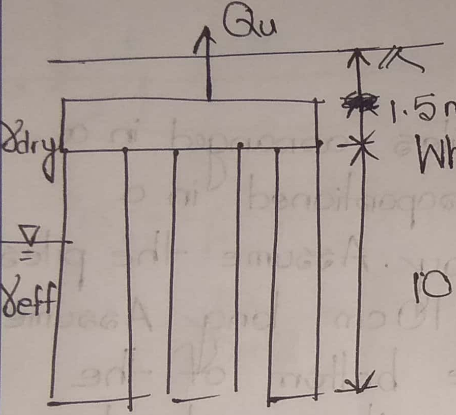
$= 1 \times 50 \times 4 \times (2s + 0.3) \times 10$

Optimum spacing

$Q_{ui} = Q_{ug}$ $[\eta = 100\%]$

$2000(25+0.3) = 3391$

$S = 0.7\ cm = 2.33d$



What happens then?

For downward loading, Qui

Uplift capacity $= \left(\frac{2}{3} Q_{ui,f} = 3391 \times \frac{2}{3} + W_{pile}\right.$

$= \frac{2}{3} Q_{ug,f} + W_{soil}$

$W_{pile} = \frac{\pi}{4} \times (0.3)^2 \times 10 \times 24 \times 9$ (Individual pile)

$= \gamma_{concrete} = 24\ kN/m^3$

$Q_{allowable} = \frac{2/3\, Q_u}{FS} + W$ (Pile weight known, but loading uncertain) so FS = 2

W_{soil} = Volume of block × γ_{soil}

$= (25+0.3)^2 \times 10 \times 18\ kN/m^3$

GWT থাকলে $\gamma - \gamma_w$ calculate করে use করতে হবে।

UATION OF INFILTRATION RATE FROM SOIL PROPERTIES USING

C.W

Sat Sun Mon Tue Wed Thu Fri

Collect A section note.

L-11

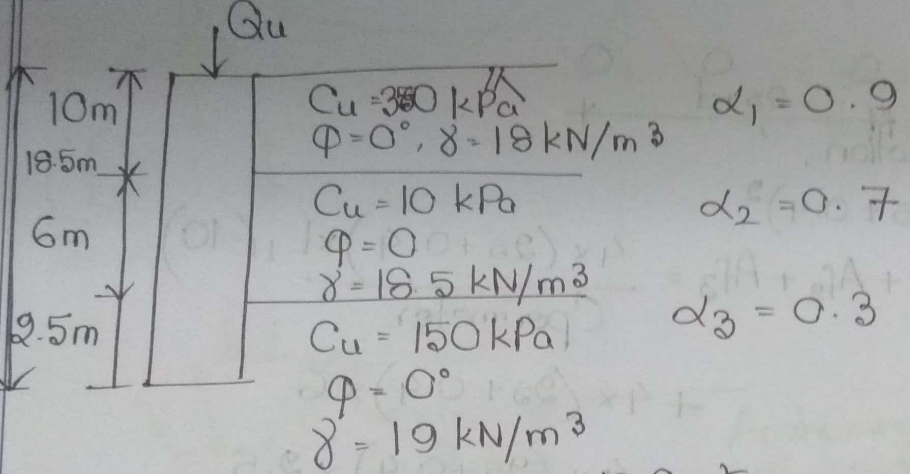
Date: 11/09/19

Prob 8.11 (P 317, Murthy)

A pile of 40cm dia and 18.5m long passes through two layers of clay and is embedded in a third layer (details given in the figure)

Compute Q_f by α-method and Q_b by skemptons

Determine Q_a for FS = 2.5



$A_{f_1} = \pi D L_1 = \pi \times 0.4 \times 10 = 12.6\ m^2$

$A_{f_2} = \pi \times 0.4 \times 6 = 7.54\ m^2$

$A_{f_3} = \pi \times 0.4 \times 2.5 = 3.14\ m^2$

$Q_b = \left[\frac{\pi}{4}(0.4)^2 \times\right] \times 150 \times 9$ $\left[\begin{matrix} C_u = 150 \\ N_c = 9 \end{matrix}\right]$

$= 170\ kN$

$Q_f = \sum(\alpha C_u) A_f$

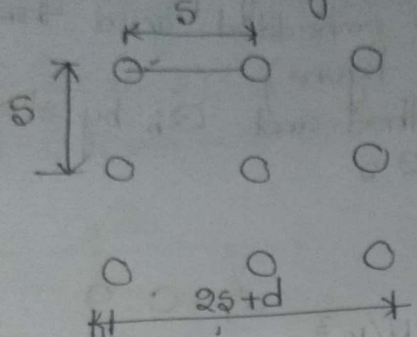
$= 0.9 \times 30 \times 12.6 + 0.7 \times 10 \times 7.54 + 0.3 \times 150 \times 3.14$

$= 744\ kN$

$Q_u = 744 + 170 = 914\ kN$

$Q_a = \frac{914}{2.5} = 366\ kN$

If there 9-pile group in a square grid in the same direction. Find s to get optimum capacity [না বলা থাকলে Qu]



Group action,

$A_b = (2s+0.4)^2$

$A_f = A_{f_1} + A_{f_2} + A_{f_3} = \underbrace{4 \times (2s+0.4)}_{\text{perimeter}} \times L, (=10)$

$+ 4 \times (2s+0.4) \times 6$

$+ 4 \times (2s+0.4) \times 2.5$

$= \cancel{(2s+0.4) 4 \times 18.5}$

$Q_{uf} = (\sum \alpha C_u) A_f$

$= 30 \times 1 \times 40 \times (2s+0.4) + 10 \times 1 \times 24 \times (2s+0.4)$

$+ 150 \times 1 \times 10 \times (2s+0.4)$

$Q_b = A_b c N_c = 150 \times 9 \times (2s+0.4)^2$

$\frac{D_f}{B} > 2.5 \quad Q_{ug} = Q_f + Q_b$

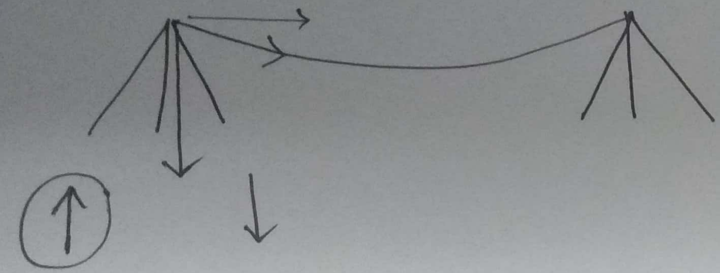
$Q_{ui} \times 9 = Q_{ug}$

$914 \times 9 = 2940 (2s+0.4) + 1350 \times (2s+0.4)^2$

$s = 0.6m = 1.5d$ [But code specifies 2.5-3.0d so minm = 1 m]

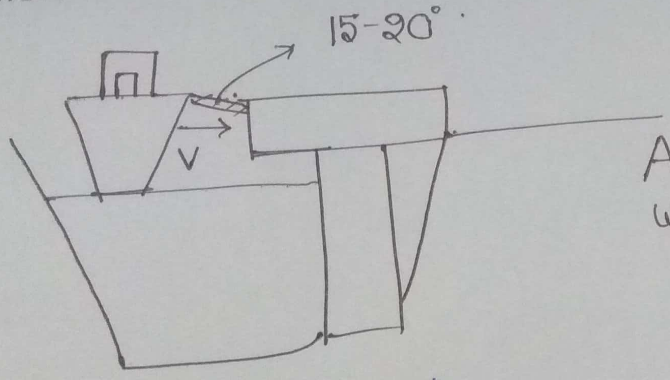
If the loading was upward, $Q_{ug} = \frac{2/3 \, Q_{fg}}{FS} + \text{Wt of } \cancel{\text{pile}}$

$Q_{ui} = \frac{2/3 \, Q_f}{FS} + \text{Wt of soil}$ pile.



Cable Transmission

Tension



Anchoring will cause huge tension.

Q: Find the optimum angle.