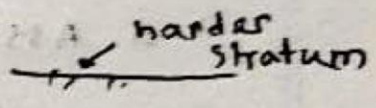
EWCE 343 — SB

1. Defn of geotech engg.
   - Branch of CE
   - Deals w/ soils, rocks and underground water
   - their relation to design, construct and operation of CE structure

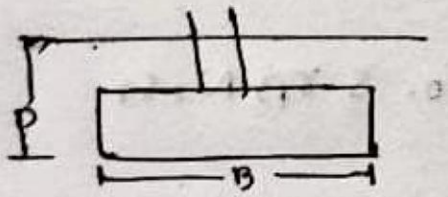
2. Bedrocks - harder stratum
   - by drilling, boring, excavation



Shallow foundation

$D/B \leq 1$

normally upto 3m



3. Importance

4. Issues to be address
   - Subsurface soil condition
   - Safely support
   - GW condition
   - Impact of plan, excavation, grading and filling
   - Slope and stability

foundation

Retaining wall

- Tunnelling / underground opening

  Eg

  contamination

5. Geotech eng. in design and execution process

   a. Assessment of subsurface soil condition

   b. performance of geotech analysis

   c. dev of geotech inputs

$$q_{ult} = cN_c s_c + \bar{q} N_q + 0.5 \gamma B N_\gamma s_\gamma$$

bearing capacity

requirements of geotech eng

SB/L02

01/12

Foundation Engg.

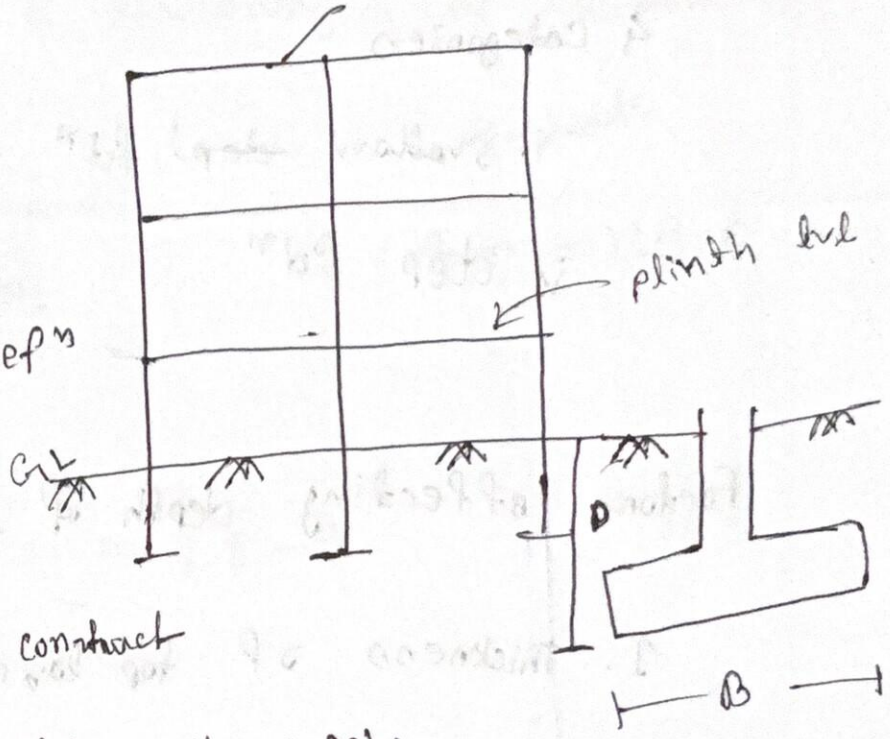
1. Fdn defn

2. Fdn Engg defn

- Branch of Geotech

- Design & construct

- Considering soil mechs structural engg & project serviceability reqs



3 Loads: LL
DL
wind ld
Seismic ld
Dynamic ld

Built up
= carpet area + wall area
→ usable space

4. Categories

i. Shallow ~~dep~~ fdn → 1–3 m, $D/B < 1$

ii. Deep fdn

Factors affecting depth of shallow fdn

1. Thickness of top layer
2. Ground freezing – ice lens
3. Depth of volm change (block cotn, swallowing)
4. " of scour
5. GW level
6. Underground utilities & defects
7. Property line
8. Footing on diff lvl
9. Sloping gnd

5. Advantages & disadvantages of shallow fdn

$q_{ult} = \frac{Q_{ult}}{L \times B}$; $L = 1m \rightarrow LB = B$

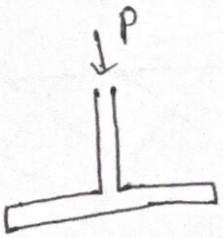
$Q_{ultimate}$

$q = \frac{q_{ult}}{SF}$ ← 2.5 – 3

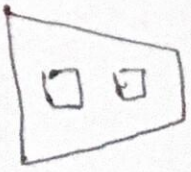
$S = c + \sigma' \tan\phi'$

$\tau > S \rightarrow$ failure

Types of shallow fdn



1. Isolated footing
2. Strip / Wall
3. combined footing → rectangular, trapezoidal



a. Slab type
b. " beam "
c. Slab strap "

4. Raft fdn



Use of combine footing

Raft fdn

Types ~~Mat~~ raft

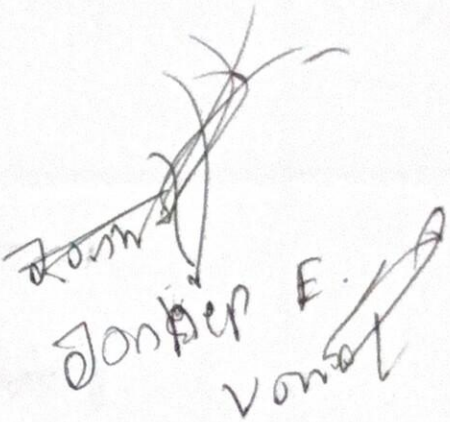
Flat plate

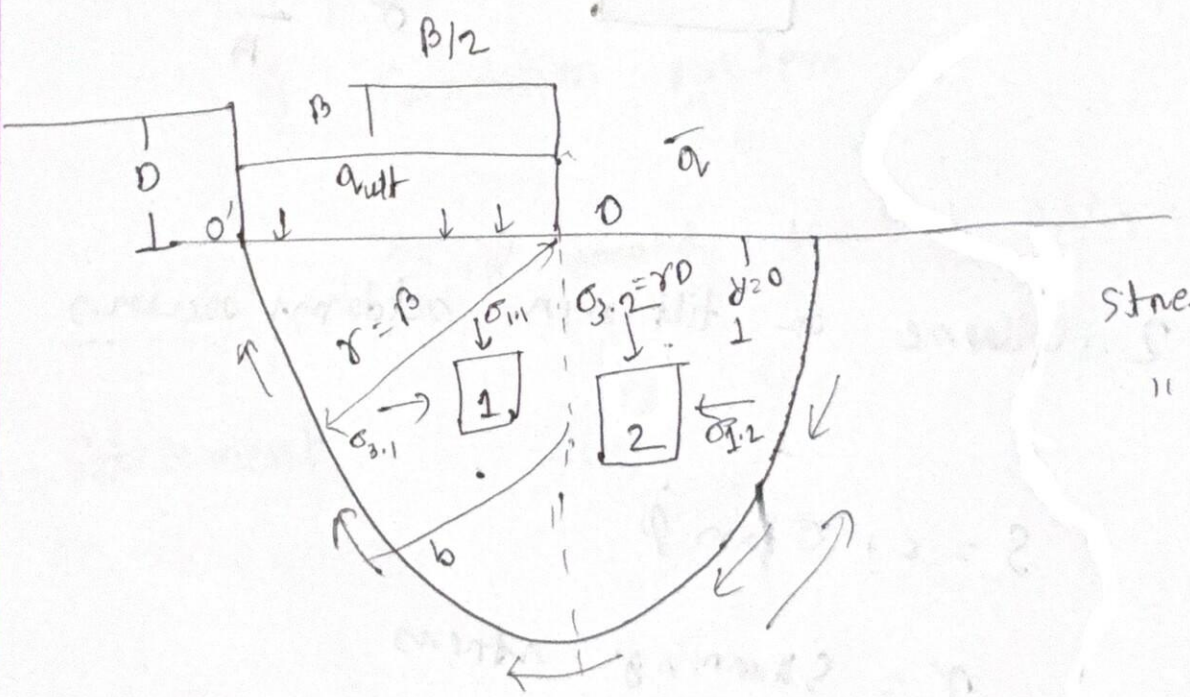
Plate



Fundamental Eqn for q_{ult} of soil

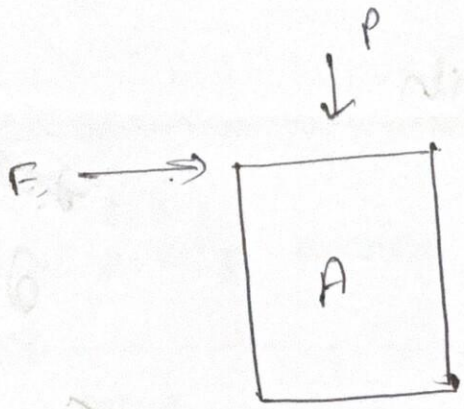
Footing on $\phi = 0$ soils





Failure pattern: Punch & simultaneous rotation
— Slip surface failure

1 Structural "collapse" or "tilt over" occur in foundation due to a. base shear failure
b. settlement



$$\tau = \frac{F}{A}$$

$$\sigma = \frac{P}{A}$$

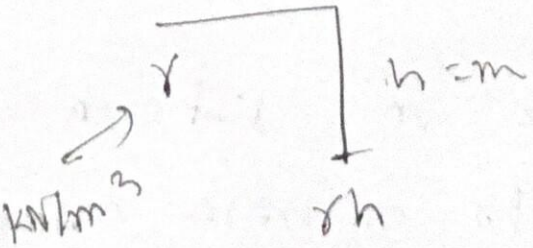
2. Collapse or tilt over seldoms occurs

$$S = c + \sigma \tan \phi$$

τ = Shearing stress

S = shear strength

$\tau > S$



Due to structural distress (base shear failure)

1. cracks
2. tilt
3. Doors & windows jamming
4. Foundation problem

← if ignored, leads to collapse

---

b. Settlement

---

* Need to investigate both → to calc

a. Settlement control q_a

b. base shear dictates ~~solution~~ swelling ~~suctions~~ q_a

q_c → allowable bearing capacity

$q_d \simeq q_a$ but not allow

$$q_a = \frac{q_{ult}}{S.F}$$

Safety Factor → A digit used to limit q_{ult}

→ 2 – 3 normally

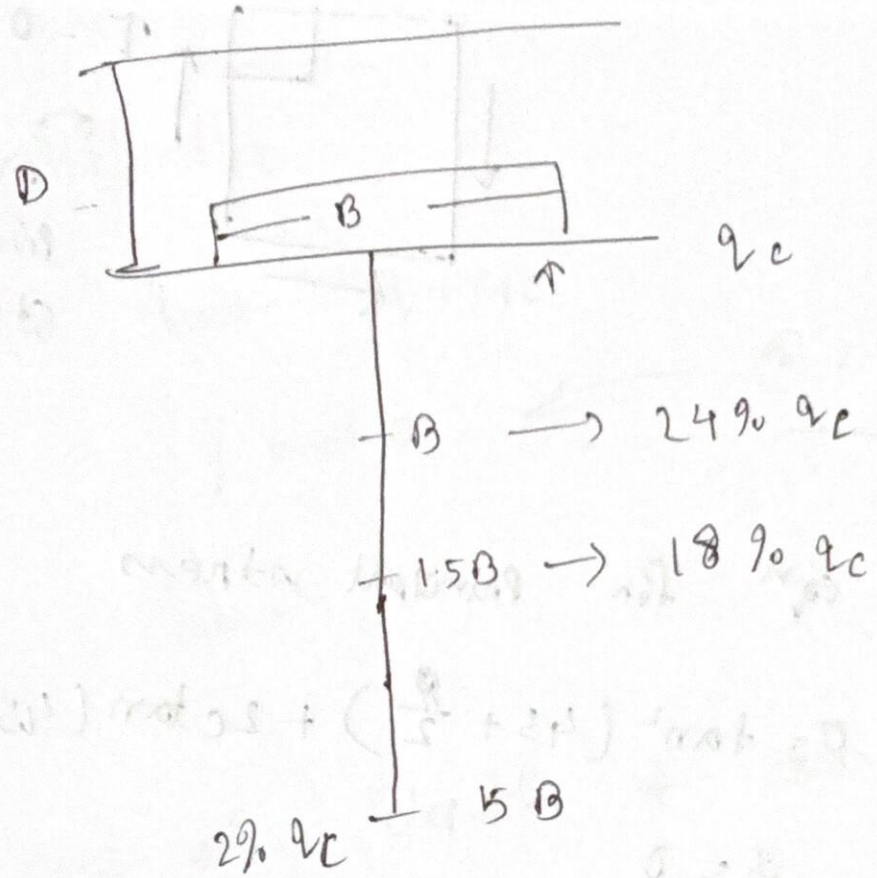
1. Types of soil
2. Reliability of soil param
3. Structural info

Recommended q_a for design based on:

a. Limiting settlement within allowable limit

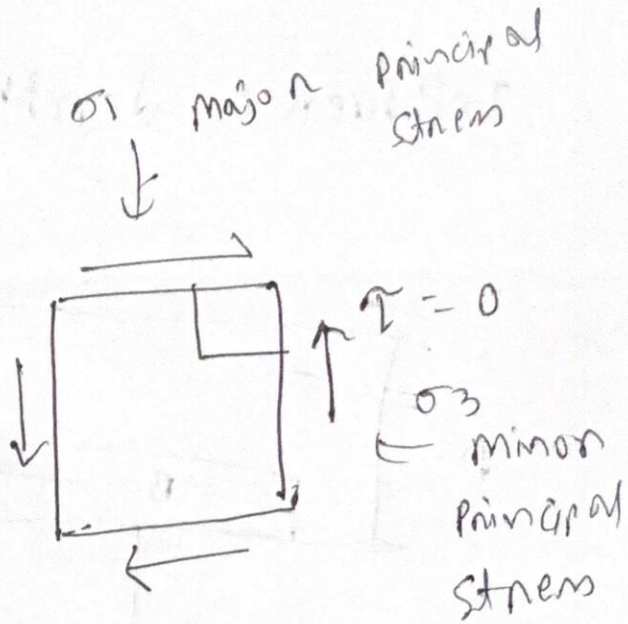
b. q_{ult} considering soil strength

Influence depth of bore contact pressure



Principal planes

Principal stress



Bell's eqn for Principal stress

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \phi/2\right)$$

$$\phi = 0$$

$$= \sigma_3 + 2c$$

$$\sigma_{1.1} = \sigma_{3.1} + 2c$$

If two soils are very close to each other

$$\sigma_{3.1} = \sigma_{1.2}$$

$$\sigma_{1.1} = \sigma_{1.2} + 2c = \gamma D + 2c + 2c = \bar{q} + 4c$$

$\sigma_{1.2} = \sigma_{3.2} + 2c$

$\therefore = \gamma D + 2c$

$q_{1.1} = q + 4c$

Ultimate

$q_{ult} = \bar{q} + 4c$

$\bar{q} \Longrightarrow 0$ ← @ surface but

$q_{ult} = 4c$

Critical depth of excavation

$q_{ult} = \bar{q} + 4c$

$= \gamma D + 4c$

if $q_{ult} = 0$ $\quad \gamma D c = -4c$

$D c = -\frac{4c}{\gamma}$

can exist since under ground

Footing on $\phi = 0$ Soils

Failure: Slip Surface failure

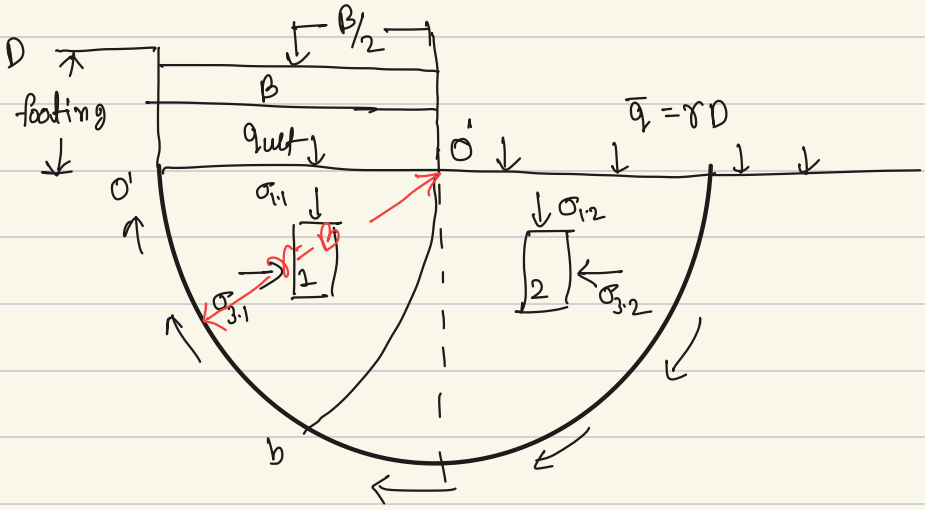
a. Foundation pushes downward

b. Simultaneous rotation

$q_{ult} = 4c$

$= \bar{q} + 4c$

$D_c = \frac{-4c}{\gamma}$



Moment at O':

$\Sigma M = 0$

$q_{ult} \times B \times \frac{B}{2} - \frac{2\pi B}{2} c \times B - \bar{q} \times B \times \frac{B}{2} = 0$

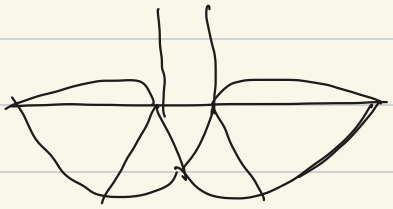
$\Rightarrow q_{ult} = 2\pi c + \bar{q}$ $\qquad \bar{q} \longrightarrow 0$

$= 2\pi c$

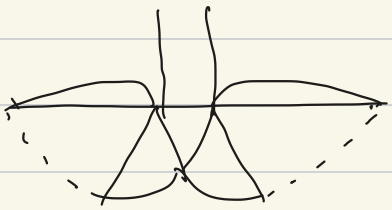
$= 6.28c$

Footing on C-Ø Soils

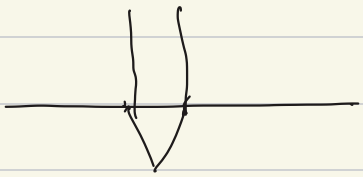
Failure: Wedge failure



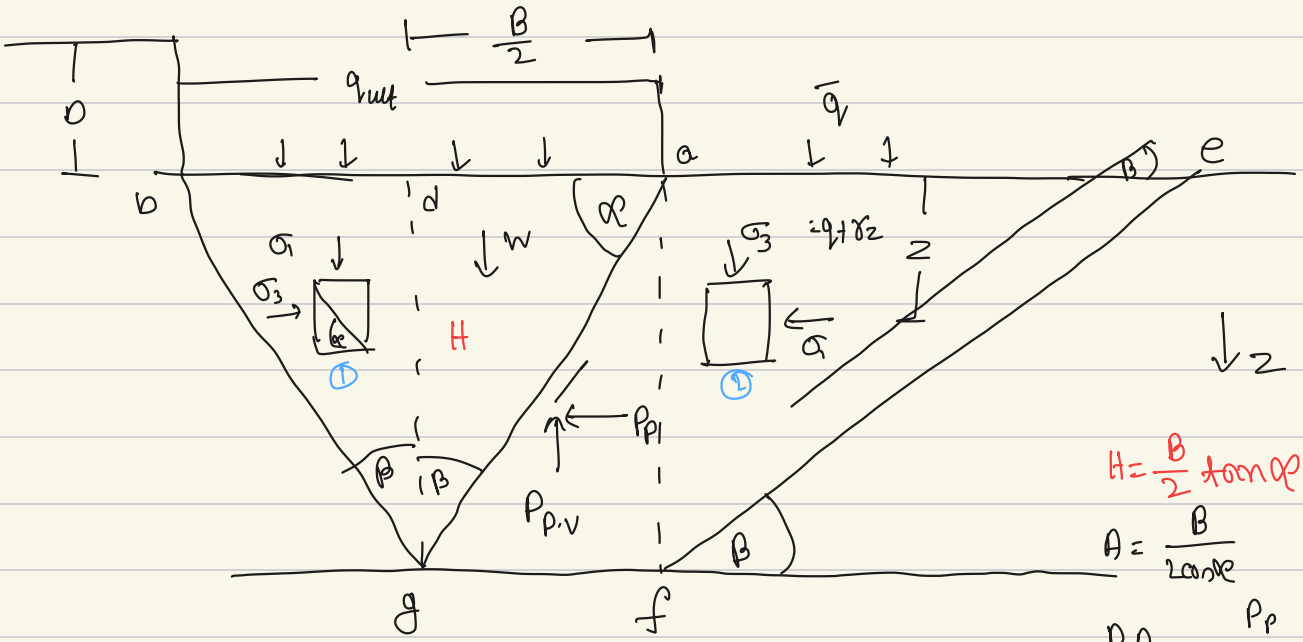
General failure



failure



Punch failure



$H = \frac{B}{2} \tan\phi$

$A = \frac{B}{2\cos\phi}$

$P.P = \frac{P_p}{\cos\phi}$

$W = \frac{1}{2} B/2 \times H \times \gamma$

$$\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2) = \sigma_3 \cdot K_p + 2c\sqrt{K_p}$$

$$= (\gamma_2 + \bar{q}) K_p + 2c\sqrt{K_p}$$

$$P_p = \int_0^H \sigma_1 \, dz = \int_0^H (\gamma_2 + \bar{q}) K_p \, dz + \int_0^H 2c\sqrt{K_p} \, dz = \frac{\gamma H^2}{2} K_p + \bar{q} H K_p + 2cH\sqrt{K_p}$$

Footing on $\phi = 0$ soil

not used for design → $q_{ult} = 4c$ ← pushing the ground

$= 6.28c$

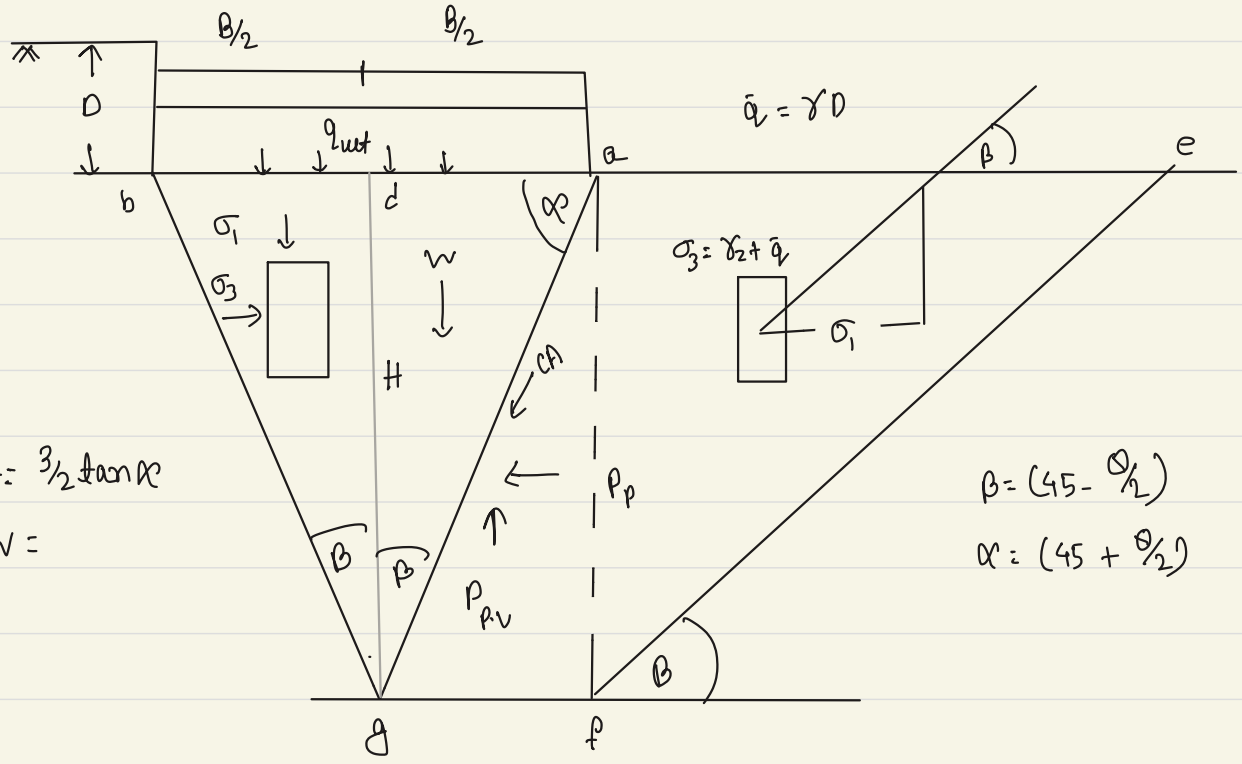
$D_e = -4c/\gamma$ ← critical depth; safe to excavate w/o collapsing wall

$q_a = \frac{q_{ult}}{SF}$ ← 2 – 3 for soil

$= q_d$ (may be designed)

Footing on $c-\phi$ soil

#wedge failure;



$H = 3/2 \tan\alpha$

$W =$

$\beta = (45 - \phi/2)$

$\alpha = (45 + \phi/2)$

$k_p = \tan^2(45 + \phi/2)$

$k_a = \tan^2(45 - \phi/2)$

$\Sigma F_v = 0$

$\Rightarrow B/2 \times q_{ult} + \frac{1}{2}(B/2 * H)\gamma + CA\cos\beta - \frac{P_p}{\cos\phi \sin\phi} = 0$

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

$= \sigma_3 k_p + \ldots$

$$P_p = \int_0^H \sigma_1 = \text{- - -}$$

$$q_{ult} = c\left[\frac{2K_p}{\cos\alpha} + V_{Kp}\right] + \bar{q}\left(\frac{K_p V_{Kp}}{\cos\phi}\right) + \frac{1}{4}\gamma_B\left(\frac{K_p^2}{\cos\theta} - V_{Kp}\right)$$

$$q_{ult} = cN_c + \bar{q}N_q + \gamma B N_\gamma$$

[for all unit width]

Very conservative

Reasons for conservative result of q_{ult}

1. Block agf not well defined
2. Block cgfe poorly defined
3. Soln based on unit width, specially for circular ftgn, & finite strip footing (need shape factor)
4. Shear resistance along ae neglected (need depth factor)
5. For inclined ld, need inclination

Used with modification using various factors

- Shape factor = s
- Depth factor = d
- inclination factor = i
- ground factor = g
- Base factor = b

} all added to upgrade q_{ult}

$N_c \rightarrow$ N factor for cohesion

$$N_c = \frac{2K_p}{\cos c_p} + V_{Kp}$$

$$K_p = \tan^2(45 + \phi/2)$$

$$N_q = \frac{K_p V_{Kp}}{\cos\phi}$$

$$N_\gamma = \frac{K^2_p}{\cos\phi} - \sqrt{K_p}$$

BC Equation

1. Terzaghi Eq^n

$$q_{ult} = \underbrace{cN_c S_c}_{0.8} + \underbrace{\bar{q}N_q}_{} + \underbrace{0.5\gamma B N_\gamma S_\gamma}_{1.3 \rightarrow \text{Square footing}}$$

Shape factor $\rightarrow S_x$

	Strip	Round	Sqr
S_c	1.0	1.3	1.3
S_γ	1.0	0.6	0.8

N-factor — from table (ϕ value)

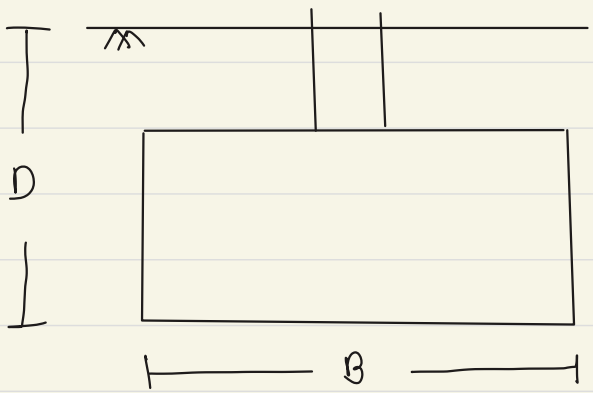
$\frac{q_{ult}}{S.F} \longrightarrow q_a$

$q_a \simeq q_d$

[q_a may not be used as q_d based on engg. experience & other factor

E.g. $\rightarrow q_a \rightarrow 600$ kpa, $q_d \rightarrow 400$ kPa]

$D/B \leq 1$



Eq^n not applicable in 3 cases:

2. Meyerhof BC Equation: ← factors from table

For vertical load; $c N_c S_c d_c + \bar{q} N_q S_q d_q + 0.5 \gamma B N_\gamma S_\gamma d_\gamma$

Inclined load: $c N_c d_c i_c + \bar{q} N_q d_q i_q + 0.5 \gamma B N_\gamma d_\gamma i_\gamma$

$D/B > 1$

if, $D \simeq B$, result almost same

3. Hansen

$c = \frac{1}{2} S_u$

$q_{ult} = c N_c S_c d_c i_c g_c b_c + \bar{q} N_q \ldots$

$\phi = 0$

$q_{ur} = 5.14 S_u (1 + s_c' + d_c' - i_c' - b_c' - g_c') + \bar{q}$

4. Vesic Equation

Rough / CT-2 23/12

Lec 01

Geotecnical engg. — Branch of civil engineering that deals with soils, rocks. and underground water and their relation with design, construction, and operation of engg. projects.

Bedrock — comparative harder stratum beneath the surface materials of loose deposits (e.g, soils) that carry the structural load.

important bcz structures must be supported by them.

Issues to be addressed by Geotech engg. (SSG-ISFR-TEC)

- Soil & rock layer cond.
- Can they support the proj.
- Ground water level
- Impact of planned excavation
- Stability of slopes
- Type of fdn necessary
- Retaining wall req? and design
- Tunnel
- EQ
- Chemical composition

Geotechnical engineering design process

1. Assess the underground condition of various strata at site
2. Engineering analysis of collected data
3. Result analysis to generate geotechnical input
4. Work as part of a team
5. Service during construction phase
6. Occasionally continue service beyond the end of construction

Requirements of Geotechnical enggn.

1. Geotechnical engineering knowledge
2. Avoidance of failure
3. Assess sub surface condition
4. Involvement in construction

Lec 02

Foundation — Structural element that connects structure to the ground & transfer loads safely & economically.

Foundation engineering — branch of geotechnical engineering that applies the knowledge of soil mechanics, structural engineering, and project serviceability requirements to design & construct foundation.

Loads → DL → building wt
LL → occupants
Dynamic ld → ld by traffic, breaking, acc.
Wind ld
EQ ld

Bearing capacity failure — Shear stress in soil exceeds shear strength of the soil.

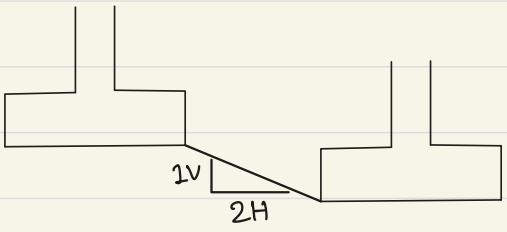
Categorized into — General, local, punching shear failures

Categories → Shallow fdn — 1–3 m, $D/B \leq 1$, ← $D \leq B$
Deep fdn

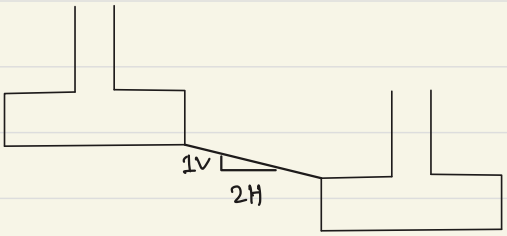
Shallow foundation:

Factors affecting depth of shallow fdn:

1. Top layer thickness — below organic mats
2. Ground freezing — below frost penetration
3. Vol change — below zone of vol chng
4. Depth of scour
5. Ground water lvl — above
6. Underground utilities & defects
7. Property line
8. Footing at diff lvls:

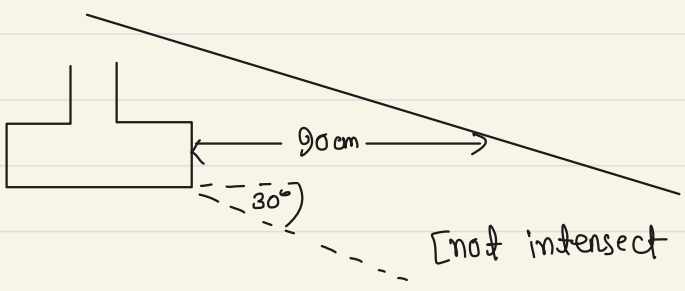


Granular soil



Clay soil

9. Sloping ground



[not intersect w/ ground with 90 cm

Advantages of shallow fdn

1. Construction works are simple
2. Less costly — equipments and in general
3. Less time
4. Less uncertainty
5. Less disturbance to soil

Disadvantages of ---

1. Chances of scouring
2. Uneconomical if GW is high
3. Can't be used if soil's bearing capacity not adequate

Types of shallow fdn:

1. Spread footing → col of a larger area
2. Strip footing → cont. on wall
3. Combined footing → two or more isolated, combined
   - i) Slab type → two cols are close together
   - ii) Slab beam type → bearing cap low
   - iii) Strap beam type → dimension restricted one side
4. Raft fdn → thick concrete slab, resting over the entire area of the struct. on more, reinforced with steel, supporting col, or walls.

Use → poor soil condition, storage tanks, silos, (tall tower to store grains) heavy industrial equ, basement required

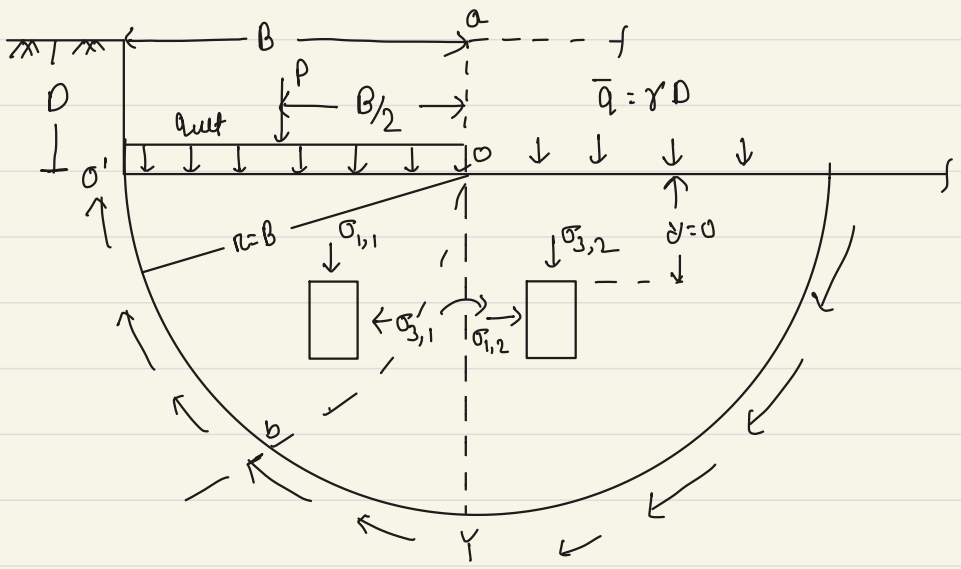
Bearing capacity

Allowable bearing cap, q_a, is min of:

i) Limiting settlement to a tolerable amount
ii) Ultimate bearing capacity.

$$q_a = \frac{q_{ult}}{S.F}$$ → types of soil, reliability of param, structural info

Footing on $\phi = 0$ soil:



Foundation pushes into the ground:

Bell's equation, $\sigma_1 = \sigma_3 \tan^2(45° + \frac{\phi}{2}) + 2c \tan(45° + \frac{\phi}{2})$

For $\phi = 0$ → $\sigma_1 = \sigma_3 + 2c$

Now, $\sigma_{3,2} = \bar{q}$

$\therefore \sigma_{1,2} = \sigma_{3,2} + 2c = \bar{q} + 2c$

Again, $\sigma_{1,2} = \sigma_{3,1} = \bar{q} + 2c$ [if two blocks are very close to each other]

$\sigma_{1,1} = \sigma_{3,1} + 2c$

$= \bar{q} + 2c + 2c$

$= \bar{q} + 4c = q_{ult}$

For bearing capacity, $D = 0$,

$q_{ult} = 4c + \bar{q}$

$\Rightarrow q_{ult} = 4c$

maximum depth to dig a hole without wall collapsing

Critical excavation depth,

$q_{ult} = 0 = 4c + \bar{q}$

$\Rightarrow 0 = 4c + \gamma D_c$

$\Rightarrow D_c = -4c/\gamma \longrightarrow D_c = \frac{4c}{\gamma}$

Footing rotation about point 'O':

Taking moment at O,

$$q_{ult} \times B \times \frac{B}{2} - c\pi B \times B - \bar{q} \times B \times \frac{B}{2} = 0$$

$$\Rightarrow q_{ult} \times B^2 - 2c\pi B^2 - \bar{q} B^2 = 0$$

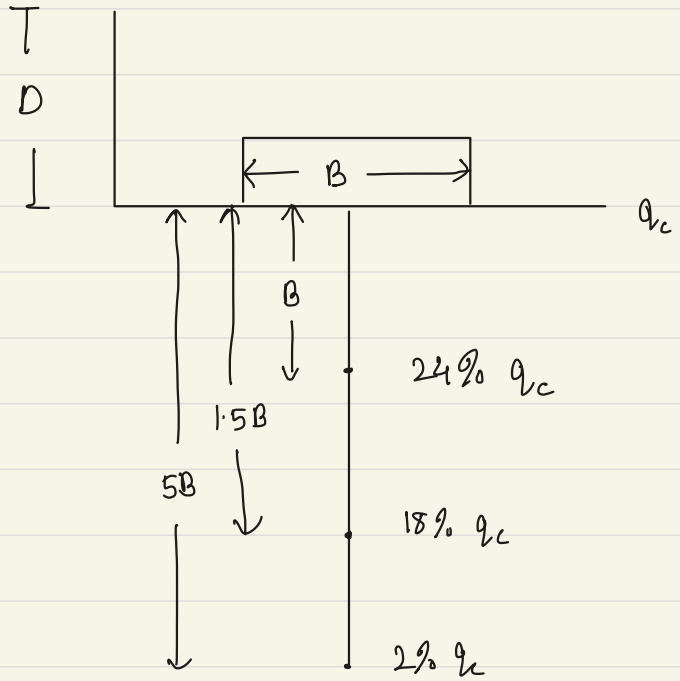
$$\Rightarrow q_{ult} = 2c\pi + \bar{q}$$

$$\therefore q_{ult} = 6.28c + \bar{q}$$

$$= 6.28c \quad [\bar{q} = 0]$$

q_{ult} = maximum vertical pressure that can be developed at ground surface before shear failure mechanisms start developing in supporting soil.

Influence depth of base contact pressure:



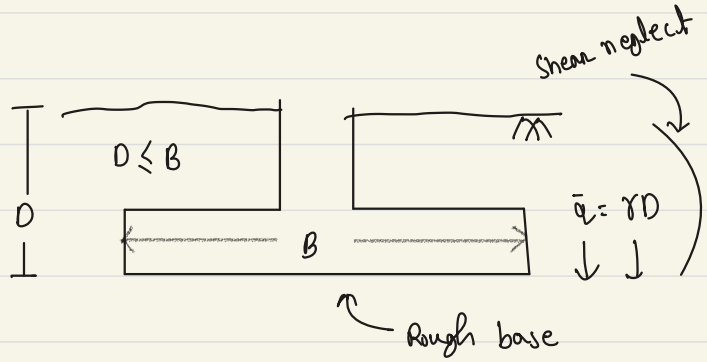
L06

29/12

Problem 1 — q_a using Terzaghi eqn

S.F = 3

$$q_{ult} = C N_c S_c + \bar{q} N_q + 0.5 \gamma B N_\gamma S_\gamma$$



Assume B

S_i = Shape factor

Assume square footing

$S_c = 1.3$

$S_\gamma = 0.8$

$\bar{q} = \gamma D$ → 17.3 kN/m³, 1.2 m

$C = 20$ kPa

N-factors — Table 4-2 for ϕ value

$B = ?$

General eqn for c-ϕ soils:

$$q_{ult} = C N_c + \bar{q} N_q + \gamma B N_\gamma$$

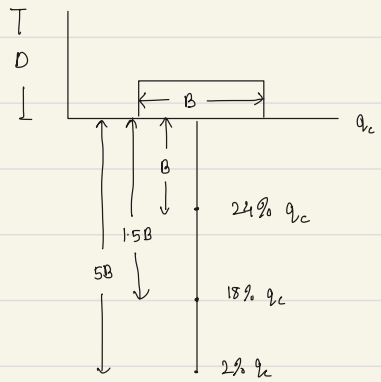
$$= C\left(\frac{2K_p}{\cos\phi} + \sqrt{K_p}\right) + \bar{q}\,\frac{\sqrt{K_p}K_p}{\cos\phi} + \gamma B \frac{1}{4}\left[\frac{K_p^2}{\cos\phi} - \sqrt{K_p}\right]$$

$K_p = \tan^2(45 + \phi/2) = \tan^2 55^\circ \rightarrow \sqrt{K_p}$

$\therefore q_{ult} = C N_c S_c + \bar{q} N_q + 0.5 \gamma_B N_\gamma S_c$

$= 20 \times 17.7 \times 1.3 + (17.3 \times 1.2) \times 7.4 + 0.5 \times 17.3 \times B \times 5 \times 0.8$

$= 613.8 + 34.6\,B$



$$q_a = \frac{q_{ult}}{3}$$

$= 204.6 + 11.5\,B$; $B = 1.5 - 3$ m

At $B = 1.5$ m, $q_a = 221.8$ kPa

$B = 3$ m, $q_a = 239$ kPa

If $B > 2m$, Terzaghi recommended a reduction factor γ_r

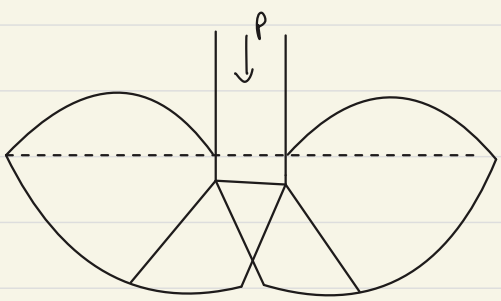
$$\gamma_r = 1 - 0.25 \log B/K \qquad B > 2m$$
$$= 0.95 \qquad K = 2$$

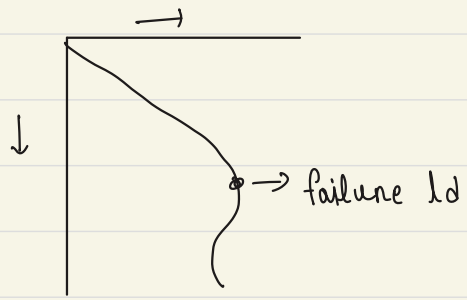
$$q_{ult} = CN_c S_c + \bar{q} N_q + 0.5 \gamma B N_\gamma S_\gamma \times \gamma_r$$

when Terzaghi developed this, he assumed

General Shear Failure with Dense Soil

$D_r = 65 - 85\%$ if D_r given on specified dense soil, then above formula





$C' = 0.67 \times C \quad = 13 kPa$

$\phi' = \tan^{-1}(0.67 \tan\phi)$

$= 13$

Local shear failure

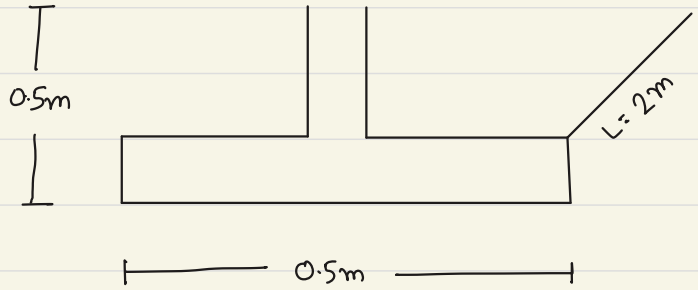
Using basic eqn

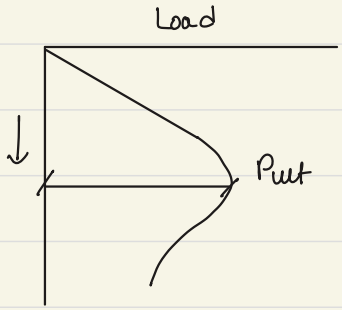
Assignment → Rest

Problem 2

$C = 0$

$\phi_{triaxial} = 42.7°$





BC Eqn: 1. Terzaghi
2. Meyerhof
3. Hansen

Hansen:

$g = 1$ (hon. ground)

$C = 0$

$$q_{ult} = cN_c S_c d_c i_c g_c b_c + \bar{q} N_q S_q d_q g_q b_q + 0.5 \gamma B' N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

$$= \gamma D N_q S_q d_q + 0.5 \gamma B' N_\gamma S_\gamma d_\gamma$$

$$= 9.31 \times 0.5 \times 287 \times 1.18 \times 1.15 + 0.5 \times 9.31 \times 0.5 \times 347 \times 0.9 \times 1$$

$q_{ult(computed)} = 2042 \ kN/m^3$

$P_{ult} = measured$

$= 1863 \ kN$

$$q_{ult(measured)} = \frac{1863}{B \times L} = 1863 \ kN/m^2$$

$L/B = 2/0.5 = 4 > 2$

$\phi_{ps} = 1.5 \times 42.7 - 17 = 47°$

$\phi = \phi_{ps}$

ϕ_{ps} = if $L/B \leq 2 \rightarrow \phi_{tn}$

$L/B > 2 \rightarrow 1.5\phi_{tn} - 17°$

$\phi_{tn} < 34° \rightarrow \phi_{tn}$

N- Factor (N_q, N_γ) Table 4-4

$N_q = 208$

$N_\gamma = 347$



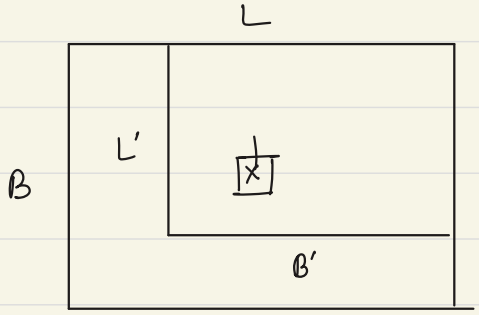
Shape Factor (S_q, S_γ) Table 4-5a

$S_{qH} = 1 + \frac{B'}{L'} \sin\phi$ (eff)

$= 1 + \frac{0.5}{2} \sin 47° = 1.18$

$S_{\gamma H} = 1 - 0.4 \frac{B'}{L'} \geq 0.6$

$= 0.9$



Depth Factor (d_q, d_γ)

$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2 K$ $K = - \cdot -$

$= 1.25$

$d_\gamma = 1$

Problem 03 :

$N_c = 5.14$

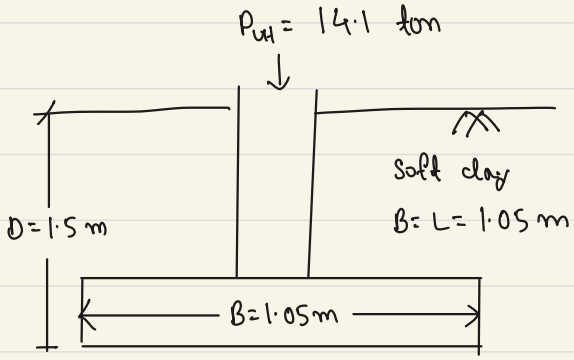
Hansen, [See Table 4-1, 4-4]

$q_{ult} = 5.14 S_u (1 + s_c' + d_c' + i_c' + b_c' + g_c') + \bar{q}$

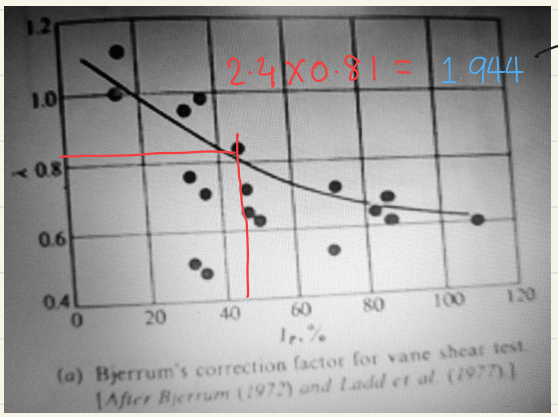
$\Rightarrow q_{ult} = 5.14 S_u (1 + s_c' + d_c)$

1.92 ton/m^2

$I_p = w_L - w_p = 80 - 35 = 45\ \%$



Settlement = 1" = 0.25 mm



(a) Bjerrum's correction factor for vane shear test. [After Bjerrum (1972) and Ladd et al. (1977).]

$q_u = 3.0 \text{ ton/m}^2$

$S_{u\,vane} = 2.4 \text{ ton/m}^2$

$w_L = 80\%, \quad w_p = 35\ \%$

$q_{ult} = ?$ by Hansen method

$q_{ult(measured)} = \frac{P_{ult}}{A}$

$k = \tan^{-1}\left(\frac{D}{B}\right) * \frac{\pi}{180} = 0.96$

$d_c = 0.4k = 0.38$

$i_c = b_c' = g_c' = 0$

$\bar{q}$ = Neglect

$= \gamma h$

$S_u = S_{u\,design}$

$q_{ult} = 5.14 S_u (1 + s_c' + d_c)$

Calculated $= 5.14 \times 1.92 (1 + 0.2 + 0.38)$

$= 15.6 \text{ ton/m}^2$

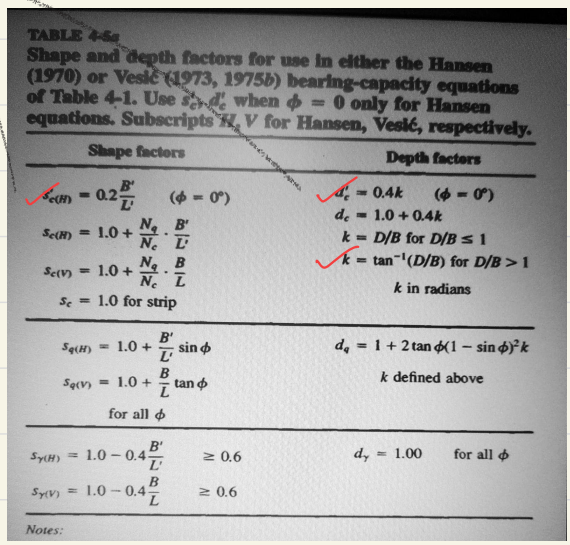


TABLE 4-6a
Shape and depth factors for use in either the Hansen (1970) or Vesić (1973, 1975b) bearing-capacity equations of Table 4-1. Use s_c', d_c' when $\phi = 0$ only for Hansen equations. Subscripts H, V for Hansen, Vesić, respectively.

Shape factors	Depth factors
$s_{c(H)} = 0.2\frac{B'}{L'}$ $(\phi = 0°)$	$d_c' = 0.4k$ $(\phi = 0°)$
$s_{c(H)} = 1.0 + \frac{N_q}{N_c}\cdot\frac{B'}{L'}$	$d_c = 1.0 + 0.4k$
$s_{c(V)} = 1.0 + \frac{N_q}{N_c}\cdot\frac{B}{L}$	$k = D/B$ for $D/B \le 1$
$s_c = 1.0$ for strip	$k = \tan^{-1}(D/B)$ for $D/B > 1$
	k in radians
$s_{q(H)} = 1.0 + \frac{B'}{L'}\sin\phi$	$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2 k$
$s_{q(V)} = 1.0 + \frac{B}{L}\tan\phi$	k defined above
for all ϕ	
$s_{\gamma(H)} = 1.0 - 0.4\frac{B'}{L'} \ge 0.6$	$d_\gamma = 1.00$ for all ϕ
$s_{\gamma(V)} = 1.0 - 0.4\frac{B}{L} \ge 0.6$	

Notes:

$q_{ult(measured)} = \frac{P_{ult}}{A}$

$= \frac{14.1}{1.05 \times 1.05} = 12.8 \text{ ton/m}^2$

$q_u = 3.0 \text{ ton/m}^2$;

$c = q_u/2 = 1.5 \text{ ton/m}^2$

$q_{ult} \text{ for } q_u = \frac{15.6}{1.92} \times 1.5 = 12.2 \text{ ton/m}^2$

L09 19/01

SPT - N values

SPT sampler, Shell by Tube Sampler
Blow count
most disturbed
Range — 2 (Soft soil) to 30 (Hard soil)

Problem 4-4 Meyerhof's Equation Bearing Capacity

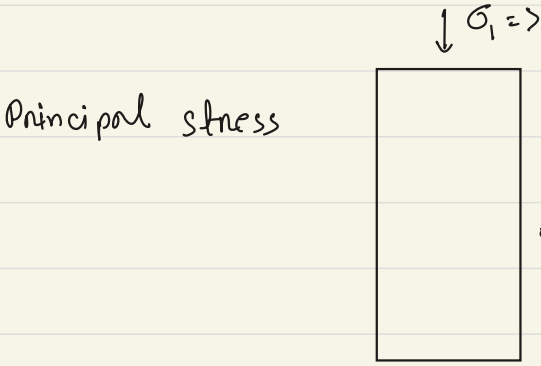
Cohesive soil $\phi = 0$

$q_{ult} = c N_c S_c d_c + \bar{q} N_q S_q d_q + 0.5 \gamma B' N_\gamma S_\gamma d_\gamma$

$= c N_c S_c + \bar{q} N_q$

$= c \times 5.14 \times S_c + \bar{q}$

Table 4-1 — Formula
4-4 — N factor
4-3 — Other factors



Principal stress

Meyerhof (1963).* See Table 4-3 for shape, depth, and inclination factors.

Vertical load: $q_{ult} = cN_c s_c d_c + \bar{q} N_q s_q d_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma$

Inclined load: $q_{ult} = cN_c d_c i_c + \bar{q} N_q d_q i_q + 0.5\gamma B' N_\gamma d_\gamma i_\gamma$

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

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

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

N factors → s (shape) d (depth) i (inclination) g (ground) b (base)

TABLE 4-4
Bearing-capacity factors for the Meyerhof, Hansen, and Vesić bearing-capacity equations
Note that N_c and N_q are the same for all three methods; subscripts identify author for N_γ

ϕ	N_c	N_q	$N_{\gamma(H)}$	$N_{\gamma(M)}$	$N_{\gamma(V)}$	N_q/N_c	$2\tan\phi(1-\sin\phi)^2$
0	5.14*	1.0	0.0	0.0	0.0	0.195	0.000
5	6.49	1.6	0.1	0.1	0.4	0.242	0.146
10	8.34	2.5	0.4	0.4	1.2	0.296	0.241
15	10.97	3.9	1.2	1.1	2.6	0.359	0.294
20	14.83	6.4	2.9	2.9	5.4	0.431	0.315
25	20.71	10.7	6.8	6.8	10.9	0.514	0.311
26	22.25	11.8	7.9	8.0	12.5	0.533	0.308
28	25.79	14.7	10.9	11.2	16.7	0.570	0.299
30	30.13	18.4	15.1	15.7	22.4	0.610	0.289
32	35.47	23.2	20.8	22.0	30.2	0.653	0.276

$\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2)$ ($\sigma_3 = 0$)

$q_u = 0 + 2c \Rightarrow c = \frac{q_u}{2}$

$q_a = \frac{q_{ult}}{3}$

$= 1.03 q_u + 0.3\bar{q}$

Solution:

TABLE 4-3
Shape, depth, and inclination factors for the Meyerhof bearing-capacity equations of Table 4-1

Factors	Value	For
Shape:	$s_c = 1 + 0.2K_p\frac{B}{L}$	Any ϕ
	$s_q = s_\gamma = 1 + 0.1K_p\frac{B}{L}$	$\phi > 10°$
	$s_q = s_\gamma = 1$	$\phi = 0$
Depth:	$d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$	Any ϕ
	$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D}{B}$	$\phi > 10$
	$d_q = d_\gamma = 1$	$\phi = 0$
Inclination:	$i_c = i_q = \left(1 - \frac{\theta°}{90°}\right)^2$	Any ϕ
	$i_\gamma = \left(1 - \frac{\theta°}{\phi°}\right)^2$	$\phi > 0$
	$i_\gamma = 0$ for $\theta > 0$	$\phi = 0$

Where $K_p = \tan^2(45 + \phi/2)$ as in Fig. 4-2
θ = angle of resultant R measured from vertical without a sign; if $\theta = 0$ all $i_c = 1.0$.

1.2

Terzaghi Eqn:

$$q_{ult} = c N_c S_c + \bar{q} N_q + 0.5 \gamma B N_\gamma S_\gamma$$

$$= \frac{q_u}{2} \times 5.7 \times 1.3 + q$$

$$q_c = \frac{q_{ult}}{3} = 1.4 q_u + 0.3 \bar{q}$$

$$1.24 \sim 1.03 = 1.0$$

$$q_a = q_u$$

$$= 200 \text{ kPa} > 75 \text{ kPa}$$

TABLE 4-1
Bearing-capacity equations by the several authors indicated

Terzaghi (1943). See Table 4-2 for typical values and for $K_{p\gamma}$ values.

$$q_{ult} = cN_c s_c + \bar{q}N_q + 0.5\gamma B N_\gamma s_\gamma$$

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

$$a = e^{(0.75\pi - \phi/2)\tan\phi}$$

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

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)$$

For:	strip	round	square
s_c =	1.0	1.3	1.3
s_γ =	1.0	0.6	0.8

TABLE 4-2
Bearing-capacity factors for the Terzaghi equations

Values of N_γ for ϕ of 0, 34, and 48° are original Terzaghi values and used to back-compute $K_{p\gamma}$

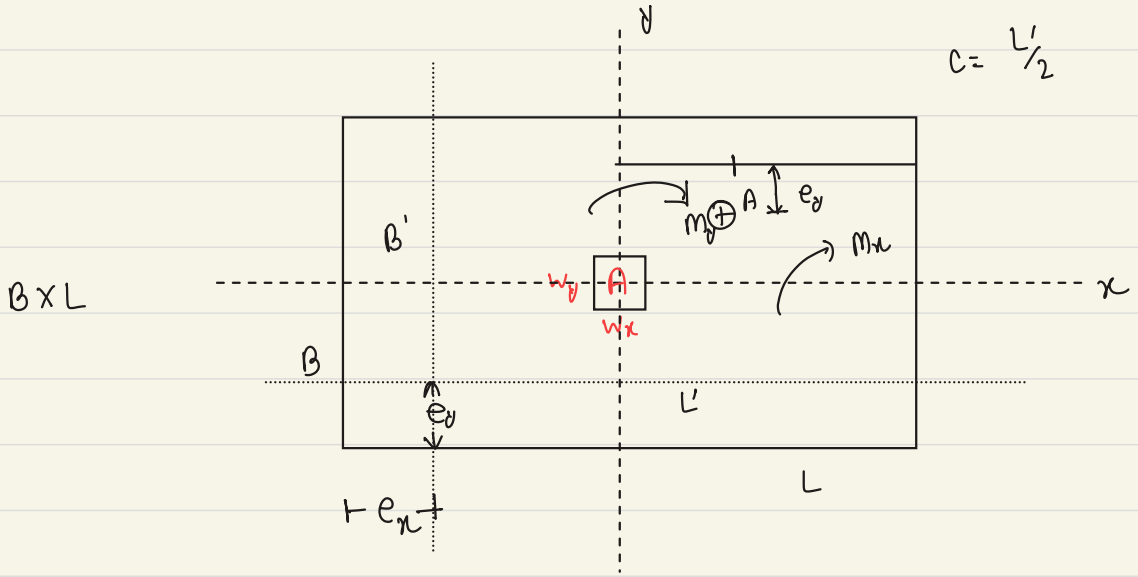
ϕ, deg	N_c	N_q	N_γ	$K_{p\gamma}$
0	5.7*	1.0	0.0	10.8
5	7.3	1.6	0.5	12.2
10	9.6	2.7	1.2	14.7
15	12.9	4.4	2.5	18.6
20	17.7	7.4	5.0	25.0
25	25.1	12.7	9.7	35.0
30	37.2	22.5	19.7	52.0
34	52.6	36.5	36.0	
35	57.8	41.4	42.4	82.0
40	95.7	81.3	100.4	141.0
45	172.3	173.3	297.5	298.0
48	258.3	287.9	780.1	
50	347.5	415.1	1153.2	800.0

*$N_c = 1.5\pi + 1$. [See Terzaghi (1943), p. 127.]

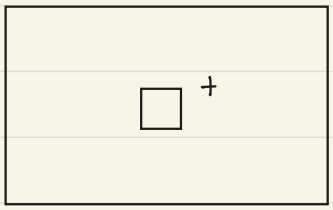
L10

26/01

Footing with Eccentricity



Footings may be loaded eccentrically with axial load and moment at one or both directions



May result from column:

a) Initially not centrally located

b) Footing may be off center due to redesign / design change

Effective dimension

$L' = L - 2e_x \Rightarrow 2e_x = L - L' \Rightarrow e_x = L/2 - L'/2 = L/2 - c = L/2 - L'/2$

$B' = B - 2e_y$ $\Rightarrow c = L'/2$

$e_x + c = L/2 \Rightarrow e_x = L/2 - c$

ACI – 318

Minimum dimension

$$B_{min} = 4e_y + w_y$$

$$L_{min} = 4e_x + w_x$$

---

q_{ult} → obtained by two methods, either following Hansen / Vesic equation on Meyerhof's equation.

Hansen (1970).* See Table 4-5 for shape, depth, and other factors.

General:† $q_{ult} = cN_c s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q i_q g_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$

when $\phi = 0$

use $q_{ult} = 5.14 s_u (1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$

vertical load

N_q = same as Meyerhof above

N_c = same as Meyerhof above

$N_\gamma = 1.5(N_q - 1)\tan\phi$

Method 1: Either Hansen or Vesic

Following adjustments to be made:

B' = in term $\gamma B' N\gamma$

Shape factor (S_i) — B' s' L'

Depth factor (d_i) — B s' L

} Table 4-5 a

Method 2: Meyerhof Equation with Reduction factor Re

$$q_{ult,\,design} = q_{ult,\,computed} \times Re$$

$$Re_B = 1 - \frac{2e}{B} \quad \text{(cohesive soil)}$$

$$= 1 - \sqrt{\frac{e}{B}} \quad \text{(cohesionless soil)} \longrightarrow 0 < \frac{e}{B} < 0.3$$

$$Re_L = \cdots$$

$$Re = Re_B \times Re_L$$

if $e/B = 0.5$

$\Rightarrow e = 0.5B$

then center of gravity A' will be at edge of bas

Problem 5

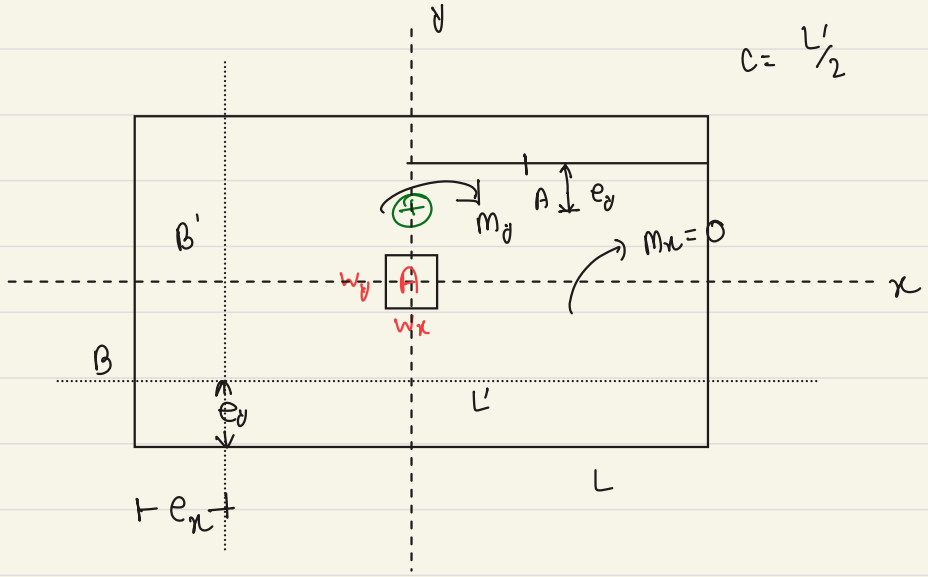
A sqr footing is 1.8m x 1.8m with 0.4m x 0.4m sqr column. It is loaded with axial load of 1800 KN. and m_x = 450 KN-m and m_y = 360 KN-m. Undrained triaxial test give $\phi = 30°$ & $c = 20$ KPa. Footing depth = 1.8 m and soil's unit wt 18 KN/m³. GW table is 6.1 m below ground surface.

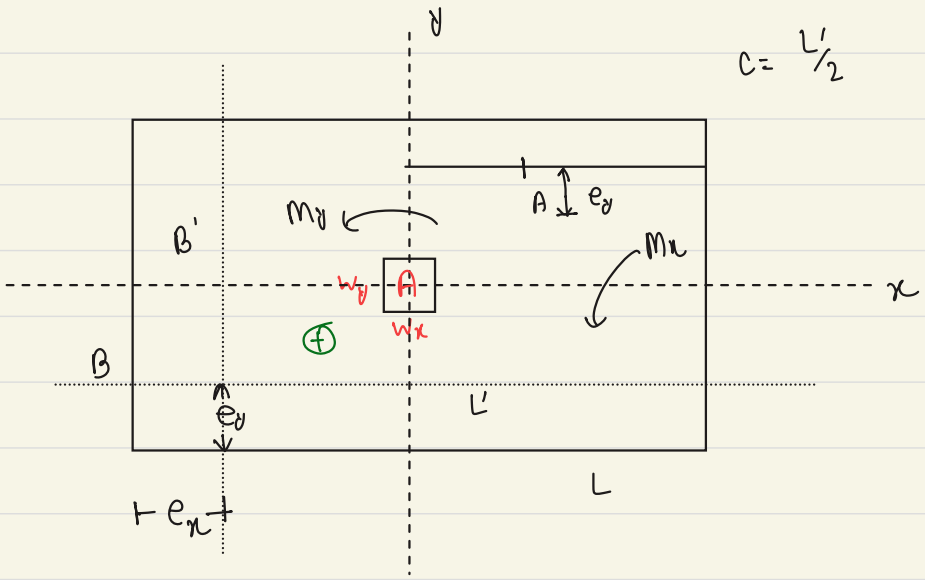
V_{ult}

Check

no eff.

Scenario :

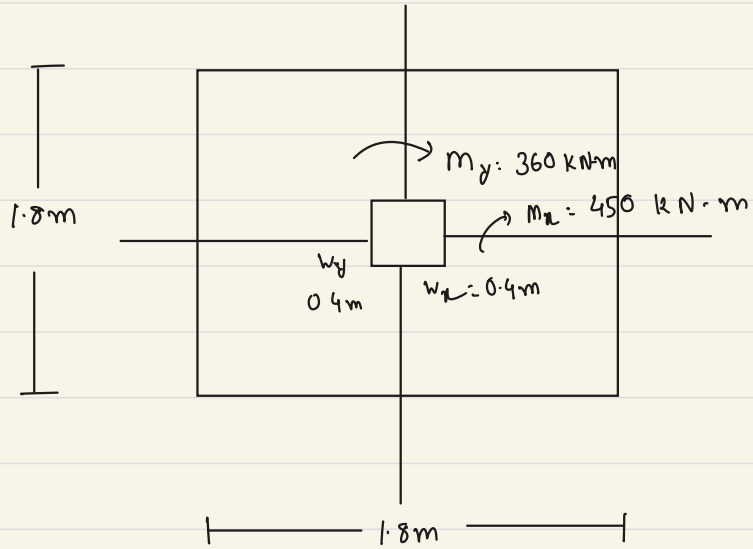


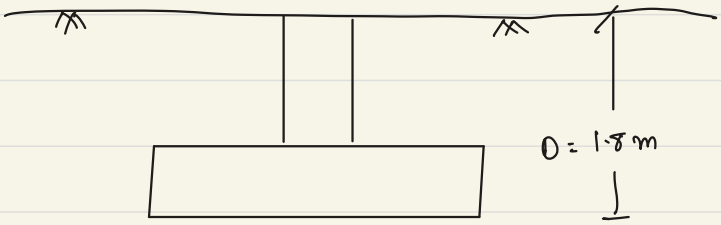


L11

15/02

Footing with Eccentricity





ϕ_{tn}: 36°

c = 20 kPa

$\gamma = \gamma' = 18\ kN/m^3$

ϕ_{tn}, for 3 reasons:

↓

ϕ_{ps} - Drainage condition (strictly controlled)

- u → can be measured

$= \gamma_w h_w$

- Simulates in situ - stress more accurately

$L/B \leq 1$ use ϕ_{tn}

$L/B > 2$ =

} See table

$e_x = \frac{M_y}{V} = 0.20$

$e_y = \frac{M_x}{V} = 0.25$

Check $e \leq \frac{B}{6}$

$c = \frac{L'}{2}$

$B_{min} = 4e_x + w_x = 1.2\,m < 1.8\,m$

$L_{min} = 4e_y + w_y = 1.4\,m < 1.8\,m$

$B' = B - 2e_y = 1.4\,m > B'$

$L' = L - 2e_x = 1.3\,m < L'$

Ex 4 – Method 1 : Hansen on vesic Eqn $e = \frac{M}{V}$

Adjustments : $B' \longrightarrow 0.5\gamma' B + e_x$

$S_i \rightarrow B'L'$

$d_i \rightarrow B, L$

Hansen eqn :

$$q_{ult} = cN_c S_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q i_q g_q b_q + 0.5\gamma' B N_\gamma S_\gamma d_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

$i_i = g_i + b_i = 1.0$

$$q_{ult} = cN_c S_c d_c + \bar{q} N_q S_q d_q + 0.5\gamma' B' N_\gamma S_\gamma d_\gamma$$

$B' = B - 2e_y$

$= 1.8 - 2 \times 0.25 = 1.3\,m$

$L' = L - 2e_x$

$= 1.8 - 2 \times 0.2 = 1.4\,m$

$\phi_{tn} = 36°$; ϕ_{ps}

$L/B \leq 1$ use ϕ_{tn}

$= \frac{1.8}{1.8} \leq 1$ $\therefore \phi_{ps} = 36°$

Shape factor: Table 4.5 b

$$S_c = 1.0 + \left(\frac{N_q}{N_c}\right) * \frac{B'}{L'} = 1.69$$

($\frac{N_q}{N_c} \to 0.746$, $B' \to 1.3$, $L' \to 1.4$)

$$S_q = 1 + \frac{B'}{L'} \sin\phi = 1.55$$

$$S_\gamma = 1 - 0.4 \frac{B'}{L} \geq 0.6$$

→ take 0.6 if less

$= 0.62 \geq 0.6$ (OK)

N-factor — Table 4.4

For $\phi = 36°$

$N_c = 50.55 \simeq 51$

$N_q = 37.7 \simeq 38$

$N_\gamma (Ht) = 40 = 40$

$\frac{N_q}{N_c} = 0.746$

$2 \tan\phi (1 - \sin\phi)^2 = 0.242$

$D/B = 1.8/1.8 = 1$

Depth factor

$d_c = 1 + 0.4 K$

$= 1.4$

surchange →

$d_q = 1 + 2 \tan\phi (1 - \sin\phi)^2 K$

$= 1 + 0.247 \times 1 = 1.25$

$d_\gamma = 1.0$ for all ϕ

$K = \frac{D}{B}$ for $\frac{D}{B} \leq 1.0$

$= \frac{1.8}{1.8} = 1$

$$q_{ult} = c N_c s_c d_c + \bar{q} N_q s_q d_q + 0.5 \gamma' B' N_\gamma s_\gamma d_\gamma$$

$$= 20(51) * 1.69 * 1.4 + 18 * 1.8(38)(1.55) + 0.5 * (18)(1.3)(40)(0.62)(1)$$

$$= 2413.32 + 2385.45 + 290.16 = 5088 \text{ KPa}$$

$$q_a = \frac{q_{ult}}{SF}$$

$$= \frac{5088}{3}$$

$$= 1696 \simeq 1700 \text{ KPa}$$

SF → depends:

1. Uncertainty of soil prop
2. Unexpected loading
3. Failure consequences - mitigation
4. Variability of construction

From loading test

$$q_a = \frac{V}{B'L'} = \frac{1800}{1.3 * 1.4} = 989 \text{ KPa}$$

$$q_a = q_d = 500 \text{ KPa}$$

L12 16/02

Footing with Eccentricity

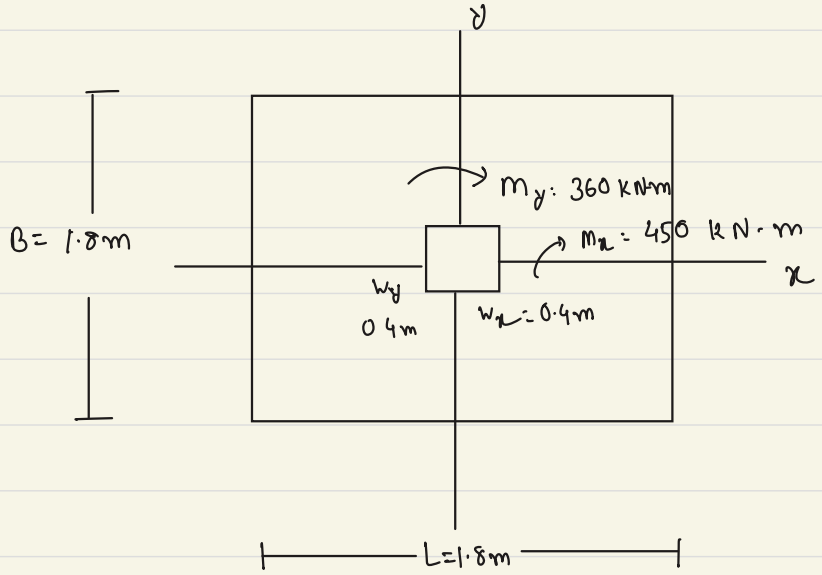
Method: Using Meyerhof's equation with Re and BL

$$q_{ult} = cN_c S_c d_c + \bar{q} N_q S_q d_q + 0.5 \gamma' B N_\gamma S_\gamma d_\gamma$$

Re = Reduction for eccentricity

$q_{ult,design} = q_{ult,comp} \times Re$; $Re = 1 - \frac{2e}{B}$ [cohesive soil]

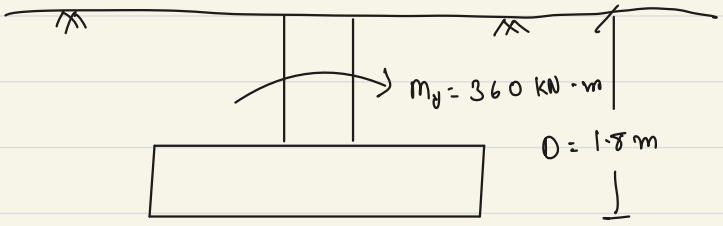
$1 - \sqrt{\frac{e}{B}}$ [cohesionless soil]



$Re_B = 1 - \sqrt{\frac{e_x}{B}}$

$Re_L = 1 - \sqrt{\frac{e_y}{L}}$

V = 1800 KN



C = 20 kPa

$\gamma = \gamma' = 18$ KN/m³

$e \leq \frac{B}{6} \rightarrow \frac{1.8}{6} = 0.3$

$e = 0.5B$

$\frac{e}{B} = 0.5$

$e = 0.5B \rightarrow$ unstable footing

e value seldom greater than 0.2

$e_x = \frac{m_y}{V}$

$= \frac{360}{1800} = 0.2$

$e_y = 0.25$

check → $B_{min} = ?$

$L_{min} = ?$

$e = ?$

$\phi_{tn} =$

$L/B \leq 2$ use ϕ_{tn}

$L/B > 2$ $\phi_{ps} = 1.5\phi_{tn} - 17$

$\phi_{tn} \leq 34$, use $\phi_{tn} =$

$\phi = 36°$

$L/B \leq 1 = \frac{1.8}{1.8} = 1$

Table 4-4:

$N_c = 51$

$N_q = 38$

$N_\gamma(m) = 44.4 \simeq 44$

Shape factor s'

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

$K_p = \tan(45 + \phi/2) = 3.86$

$\sqrt{K_p} = 1.96$

$S_q = S_\gamma = 1 + 0.2\sqrt{K_p}\ \frac{B}{L} = 1.39$

Depth factor

$d_c = 1 + 0.2\sqrt{K_p}\ \frac{D}{B} = 1.39$

$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\ D/B = 1.20$

$$q_{ult} = cN_c S_c d_c + \bar{q} N_q S_q d_q + 0.5\gamma' B N_\gamma S_\gamma d_\gamma$$

$$= \dots$$

$$= 5752 \text{ kPa}$$

$$R_{eB} = 1 - \sqrt{\frac{0.2}{1.8}} = 0.63$$

$$R_{eL} = 1 - \sqrt{\frac{0.25}{1.8}} = 0.62$$

$$R_e = R_{eB} * R_{eL} = 0.422$$

$$q_{ult,design} = q_{ult,comp} \times R_e$$

$$= 5752 \times 0.422 = 2427 \text{ kPa}$$

$$q_a = \frac{q_{ult,design}}{SF} = \frac{2427}{3} = 809 \text{ kPa}$$

Assume $q_d = 500$ kPa

Hansen: $q_d = 500$ kPa

$$\frac{B'}{L'} = \frac{1.3}{1.4} = 0.928$$

$$B' = 0.928 L'$$

$$B' \times L' = \frac{1800 \text{ kN}}{500 \text{ kN/m}^2}$$

$$\Rightarrow 0.928 L' \times L' = 3.6 \text{ m}^2$$

$$\Rightarrow L'^2 = \frac{3.6}{0.928} = 3.875$$

$$\Rightarrow L' = 1.97 \text{ m}$$

$$B' = 0.928 \times 1.97$$

$$= 1.828 \text{ m}$$

$$B = B' + 2e_y = 1.828 + 2 \times 0.25 = 2.328 \text{ m}$$

$$L = L' + 2e_x = 1.97 + 2 \times 02 = 2.37 \text{ m}$$

$B \times L \rightarrow 2.32 \times 2.37 \rightarrow 2.4\text{m} \times 2.4\text{m}$

Meyerhof: $q_d = 500\ kPa$

$$B \times L = \frac{1800}{500} = 3.6$$

$$\Rightarrow L = 1.89$$

$$B = 1.89$$

$$B \times L = 1.89 \times 1.89$$

$$= 2.0\ m \times 2.0\ m$$

Allowable ld, $P_a = q_d \times A$

Hansen $P_a = 500 \times 2.4 \times 2.4 = 2880\ KN$

Meyerhof $P_a = 500 \times 2 \times 2 = 2000\ KN$

---

q_{ult}

$q_{ult,design}$

q_a

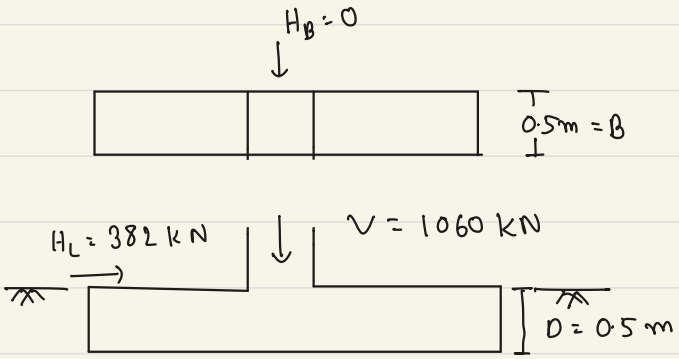
q_d

footing dim

Footing with inclined load Hansen & Vesic

Produced when footing is loaded with both

→ vertical ld and → gravity ld
→ Horizontal components of loading → wind ld
↓
Equipment foundation



$\gamma' = 9.43\ kN/m^3$

$C = 0$

$\phi = 43°$

Hansen eqn,

$$q_{ult} = cN_c S_c d_c i_c + \bar{q} N_q S_q d_q i_q + 0.5\gamma B' N_\gamma S_\gamma d_\gamma i_\gamma$$
$$= \bar{q} N_q S_q d_q i_q + 0.5\gamma B' N_\gamma S_\gamma d_\gamma i_\gamma$$

Along B direction

$$q_{ult} = \bar{q} N_q S_{qB} d_{qB} i_{qB} + 0.5\gamma B' N_\gamma S_{\gamma B} d_{\gamma B} i_{\gamma B}$$

Along L direction

$$q_{ult} = \bar{q} N_q S_{qL} d_{qL} i_{qL} + 0.5\gamma B' N_\gamma S_{\gamma L} d_{\gamma L} i_{\gamma L}$$

} Smaller

$\phi_{tn} = 43°$ $\quad L/B \leq 1$ Same value

$L/B > 2$ $\quad 1.1\,\phi_{tn} = 47.3$

$\phi_{ps} = 47°$

$2/0.5 = 4 > 2 \rightarrow 1.5\,\phi_{tn} - 17° = 47.5$

[4-5a]

Table 4-4

$N_q = 208$

$N_\gamma = 347$

$N_q/N_c = 1.22$

$2\tan\phi\,(1-\sin\phi)^2 = 0.155$

Shape factor : Table 4-5a

$d_q = 1 + 2\tan\phi\,(1-\sin\phi)^2 K$

$d_{qB} = 1 + 0.155\,(1) = 1.15$

$d_{qL} = 1 + 0.155\,(0.25) = 1.04$

$K = D/B$ when $D/B < 1$

$D/B > 1 \quad K = \tan^{-1}(D/B)$ in radians

$K = D/L = \frac{0.5}{2} = 0.25$

$d_{\gamma L} = d_{\gamma B} = 1.0$ for all ϕ

Inclination factors : Table 4-5b

$$i_q = \left(1 - \frac{0.5\,H_i}{V + A_f C_a \cot\phi}\right)^{\alpha}$$

$2 \leq \alpha_1 \leq 5 \quad \alpha_1 = 2.5$

can be any

$$i_{qB} = \left(1 - \frac{0.5\,H_B}{V + A_f C_a \cot\phi}\right)^{2.5} = 1.0$$

($H_B \leftarrow 0$)

$C_a = 0.6C$

$$i_{qL} = \left(1 - \frac{0.5\,H_L}{V + A_f C_a \cot\phi}\right)^{2.5}$$

$$= \left(1 - \frac{0.5 \times 382}{1060 + 0}\right)^{2.5} = 0.608$$

$$i_\gamma = \left(1 - \frac{0.7 H_1}{V + A_1 C_a \cot\phi}\right)^{\alpha_2 = 3.5} \qquad 2 \le \alpha_2 \le 5$$

$i_{\gamma B} = 2.0$

$i_{\gamma L} = 0.361$

Shape factor 4-5a

$$S_q = 1 + \sin\phi \frac{B'}{L'}$$

$$S_{qB} = 1 + \sin\phi \frac{B' i_{qB}}{L'}$$

$$= 1 + \sin 47^\circ \frac{0.5 \times 1}{2} = 1.18$$

$$S_{qL} = 1 + \sin\phi \frac{L' i_{qL}}{B} = 2.78$$

$$S_\gamma = 1 - 0.4 \frac{B'}{L'}$$

$$S_{\gamma B} = 1 - 0.4 \frac{B' i_{\gamma B}}{L' i_{\gamma L}} > 0.6$$

$$= 0.723 > 0.6 \quad [OK]$$

$S_{\gamma L} = 0.422 < 0.6$

$=$ use 0.6

Along B,

$$q_{ult} = \bar{q} N_q S_{qB} d_{qB} i_{qB} + 0.5 \gamma B' N_\gamma S_{\gamma B} d_{\gamma B} i_{\gamma B}$$

$$= 0.5(9.43)(208)(1.18)(1.16)(1.0) + 0.5(9.43)(0.5)(347)(0.723)1$$

$$= 1900 \text{ kPa}$$

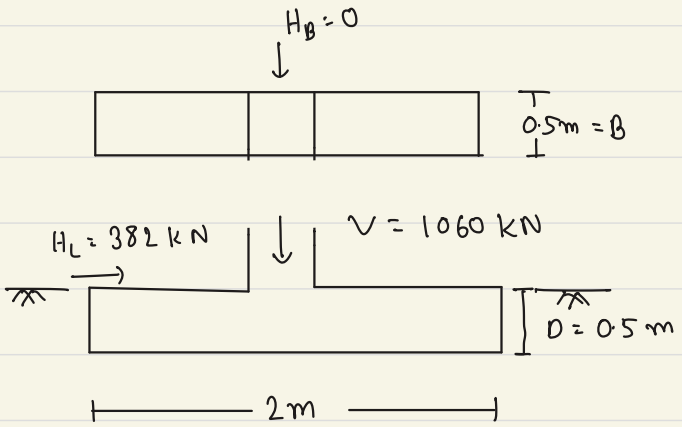
Along L,

$$q_{ult} = 2100 \text{ kPa}$$

Use $q_{ult} = 1900$ kPa

$$q_{ult} = V/A = \frac{1060}{2 \times 0.5} = 1060 \text{ kPa}$$

Footing with inclined load — Vesic's method



Hansen (1970).* See Table 4-5 for shape, depth, and other factors.

General:†	$q_{ult} = cN_c s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q i_q g_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$
when	$\phi = 0$
use	$q_{ult} = 5.14 s_u (1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$
	N_q = same as Meyerhof above
	N_c = same as Meyerhof above
	$N_\gamma = 1.5(N_q - 1)\tan\phi$

$\gamma' = 9.43 \text{ kN/m}^3$

$c = 0$

$\phi = 43°$

$g_i = b_i = 1.0$

1. $q_{ult} = \bar{q} N_q s_q d_q b_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma i_\gamma$

2. $L/B = \frac{2}{0.5} = 4 > 2$

$\phi_{ps} = 1.5\phi_{tn} - 17°$

$= 47.5° \simeq 47°$

for $L/B \leq 2 \rightarrow \phi_{tn}$

$L/B > 2 \rightarrow 1.5\phi_{tn} - 17°$

$\phi_{tn} < 34° \rightarrow \phi_{ps}$

3 N-factors Table 4–4;

$N_q = 187$

$N_\gamma(V) = 404$

$2 \tan(1 - \sin\phi)^2 = 0.155$

4. Shape & depth factor (s, d) Table 4-5a

$$S_q = 1 + \frac{B}{L}\tan\phi$$

$$= 1 + \frac{0.5}{2.0}\tan 47 = 1.27$$

$$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 k$$

$$= 1 + 0.155(1) = 1.26$$

$k = D/B$ for $D/B \le 1$

$\tan^{-1}(D/B)$

$D/B = 1 \le 1$

$= 1$

$$S_\gamma = 1 - 0.4\frac{B}{L} = 1 - 0.4\frac{0.5}{2.0} =$$

$$= 0.9 > 0.6 \quad [OK]$$

$$d_\gamma = 1.0$$

5. Inclination factor

$$i_q = \left[1 - \frac{H_i}{V + A_f C_a \cot\phi}\right]^m$$

H_L

1060 kN

base adhesion

$C_a = 0.6c = 0$

$$m_L = m = \frac{2 + \frac{L}{B}}{1 + L/B}$$

$$= \frac{6}{5} = 1.2$$

$$= \left[1 - \frac{382}{1000}\right]^{1.2} = 0.585$$

$$i_\gamma = [\ldots]^{m+1} = 0.374$$

$$q_{ult} = \bar{q} N_q s_q d_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma$$

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

$$= \text{---} + \text{---}$$

$$= 759.87 + 320.6$$

$$= 1080.5 \simeq 1080 \text{ kPa}$$

$P = 1060\ kN$

$A = B \times L$

$= 0.5 \times 2.0$

$q_{ult} = \frac{P}{A}$

$= \frac{1060}{1}$

$= 1060\ kPa$

~~~~ ø value from Triaxial Test

a. Allows precise control of drainage condition and porewater pressure measurement

b. Application of varied containing pressure stress condition

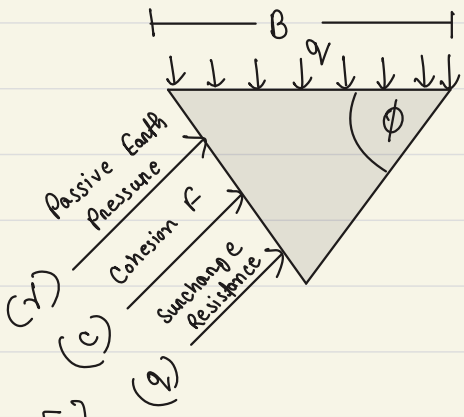
c. Most reliable measurement of soil strength parameters

$$\delta = c + \sigma \tan \phi$$

Rough / midterm 28/02

Footing on ø-c soil

- Wedge failure



Common format

$$q_{ult} = cN_c + qN_q + \gamma B N_\gamma$$

$$q_{ult} = c\left[\frac{2K_p}{\cos\phi} + \sqrt{K_p}\right] + q\frac{K_p\sqrt{K_p}}{\cos\phi} + \gamma\frac{B}{4}\left[\frac{K_p^2}{\cos\phi} - \sqrt{K_p}\right]$$

$$K_p = \tan^2(45° + \phi)$$

Terzaghi Bearing Capacity Equation

- Used shape factor (strip, round, sqr → S_c, S_γ)
- Intended for shallow fdn ($D \leq B$)

Meyerhof Eq^n

- Similar to Terzaghi
- Added shape factor S_c, S_q, S_γ
  Depth factor D_c, D_q, D_γ
  Inclination factor i_c, i_q, i_γ
- Produces similar value to Terzaghi upto $D/B \approx 1$

Hansen's Eq^n

- Proposed general eq^n
- Meyerhof's, in addition of b_i → footing inclination
  g_i → soil surface inclination
- Implicitely allow any D/B ratio

Vesic's

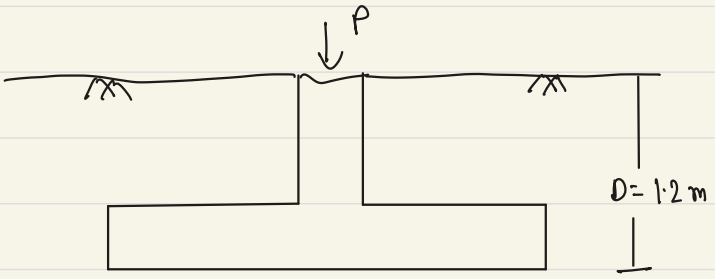
- Same as Hansen's
- Except: N_γ, i_i, b_i, g_i
- Easier to use than Hansen's

When to use which:

1. Terzaghi → Best for cohesive soil where $D/B \leq 1$ or for a quick test of q_{ult}.
   Not to be used for footings w/ moments or hor. force present on sloping ground.
2. Meyerhof, Hansen, Vesić → Any situation that applies, depending on user preference on familiarity
3. Hansen, Vesić → when base is tilted, footing is on a slope, or $D/B > 1$.

Good to use two methods and compare the result and go for a third if varies. Use weighted avg for q_a.

Problem 4-1



$\gamma = 17.3$ kN/m³

$\phi = 20°$

$c = 20$ kPa

N-factors → $N_c = 17.7$; $N_q = 7.4$; $N_\gamma = 5.0$

S-factor → $S_c = 1.3$; $S_\gamma = 0.8$

$\bar{q} = \gamma H = 17.3 \times 1.2 = 20.76$ kN

Terzaghi eqn:

$$q_{ult} = cN_c S_c + \bar{q} N_q + 0.5 \gamma B N_\gamma S_\gamma$$

$$= 20 \times 17.7 \times 1.3 + 20.76 \times 7.4 + 0.5 \times 17.3 \times B \times 5 \times 0.8$$

$$= 613.8 + 34.6\,B$$

$$q_a = \frac{q_{ult}}{3} = 204.6 + 11.53\,B$$

For $B \leq 1.5$ m, $n_\gamma = 1.0$

$B = 3$ m, $n_\gamma = 0.95$

$\therefore q_a = 204.6 + 11.53 \times 1.5 = 222$ kPa

$= 204.6 + 11.53 \times 3 \times 0.95 = 238$ kPa

$\rightarrow 220 \approx 240$ kPa | −tan +5

General eqn:

$$q_{ult} = cN_c + qN_q + \gamma B N_\gamma$$

$$= c\left[\frac{2K_p}{\cos\phi} + \sqrt{K_p}\right] + q\left[\frac{K_p\sqrt{K_p}}{\cos\phi}\right] + \gamma\frac{B}{4}\left[\frac{K_p^2}{\cos\phi} - \sqrt{K_p}\right]$$

$$= 20\left[\frac{2 \times 2.04}{\cos 20°} + \sqrt{2.04}\right] + 17.3 \times 1.2\left[\frac{2.04\sqrt{2.04}}{\cos 20}\right] + 17.3 \times \frac{B}{4}\left[\frac{2.04^2}{\cos 20°} - \sqrt{2.04}\right]$$

$$= 179.77 + 12.98\,B$$

$$q_a = \frac{q_{ult}}{3}$$

$$= 59.92 + 4.33\,B$$

$$= 60 \sim 70 \text{ kPa}$$

$K_p = \tan^2(45° + \frac{\phi}{2})$

$= \tan^2(55°)$

$= 2.04$

$q = \gamma D$

Problem 4-2:

$D = 0.5m$; $B = 0.5m$; $L = 2m$

$\gamma' = 9.31\ KN/m^3$; $\phi_{tx} = 42.7°$; $c = 0$

$q_{ult} = 1863\ KPa$ (measured)

ϕ_{ps}: $\frac{L}{B} = \frac{2}{0.5} = 4 > 2$

$\phi_{ps} = 1.5\,\phi_{tx} - 17°$

$= 47°$

Hansen eqn: $c = 0$

$$q_{ult} = \bar{q} N_q S_q d_q i_q g_q b_q + 0.5 \gamma B' N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

$$= \bar{q} N_q S_q d_q + 0.5 \gamma B' N_\gamma S_\gamma d_\gamma$$

N-factors $N_q = 208.22$ $N_\gamma = 347.26$

Shape factors $S_q = 1 + \frac{B'}{L'} \sin\phi$

$= 1 + \frac{0.5}{2} \sin 47°$

$= 1.18$

$S_\gamma = 1 - 0.4 \frac{B'}{L'} \geq 0.6$

$= 1 - 0.4 \frac{0.5}{2} \geq 6$

$= 0.9$

Depth factors $d_q = 1 + 2\tan\phi(1-\sin\phi)^2 K$

$= 1 + 2\tan 47°(1-\sin 47°)^2 1$

$= 1.15$

$d_\gamma = 1.0$

$D/B = \frac{0.5}{0.5} = 1$

$\therefore K = 1$

$\therefore q_{ult} = 9.31 \times 0.5 \times \ldots$

$= 2040\ kPa$

Meyerhof eqn: $c = 0$

$$q_{net} = \bar{q} N_q d_q s_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma$$

$N_q = 208.22$ $\quad N_\gamma = 506.06$

$$d_q = d_\gamma = 1 + 0.1 \sqrt{K_p} \frac{D}{B}$$

$$= 1 + 0.1 \sqrt{6.44} \frac{0.5}{0.5}$$

$$= 1.25$$

$$K_p = \tan^2 \left(45 + \frac{\phi}{2}\right) \quad \leftarrow 47^\circ$$

$$= 6.44$$

$$s_q = 1 + 0.1 K_p \frac{B}{L} = s_\gamma$$

$$= 1.16$$

$\therefore q_{net} = 3113$ kPa

Ex 4-3 :

1.05 m sqr footing $D = 1.5$ m

$q_u = 3$ ton/m² $S_u = 3$ ton/m² $S_{u,vane} = 2.4$ ton/m²

$w_L = 80$ $w_p = 35$ $\therefore I_p = w_L - w_p = 45$

Hansen eqn : $\phi = 0$

$$q_{ult} = 5.14\, S_u (1 + s_c' + d_c') + \bar{q}$$ ← neglected

$S_{u,design} = \gamma S_{u,vane}$; $\gamma = 0.8$ from γ vs I_p graph

$= 1.92$ ton/m²

$s_c' = 0.2 \frac{B'}{L'} = 0.2$ $d_c' = 0.4K = 0.38$ $\frac{D}{B} = \frac{1.5}{1.05} = 1.43 > 1$

$K = \tan^{-1}\left(\frac{D}{B}\right) = 0.96$

$\therefore q_{ult} = 15.6$ ton/m²

$q_{actual} = \frac{P}{A} = \frac{14.1}{1.05^2} = 12.8$ ton/m²

For unconfined compression, $S_u = \frac{q_u}{2} = 1.5$ ton/m²

$$q_{ult} = \frac{15.6}{1.92} \times 1.5 = 12.2 \text{ ton/m}^2$$

Ex. 4-4

$q_u = 200$ kPa SF = 3

Meyerhof eqn: $\phi = 0$

$$q_{ult} = cN_c s_c d_c + \bar{q} N_q s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma$$

$N_c = 5.14$ $N_q = 1.0$ $N_\gamma = 0.0$

$S_c = 1.2$ $S_q = 1$

$d_c = ? \rightarrow 1$ $d_q = 1$

$$\therefore q_{ult} = 5.14 c \times 1.02 + \bar{q}$$

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

$$\therefore q_{ult} = \frac{q_u}{2} \times 5.14 \times 1.2 + \bar{q} = 3.084 q_u + \bar{q}$$

$$q_{all} = \frac{q_{ult}}{3} = 1.03 q_u + 0.33 \bar{q}$$ ← neglect

Terzaghi eqn:

$$q_{ult} = cN_c s_c + \bar{q} N_q + 0$$ (5.7, 1.3, 1)

$$= 7.41 c + \bar{q}$$

$$= 3.71 q_u + \bar{q}$$

$$q_{all} = 1.2 q_u + 0.33 \bar{q}$$

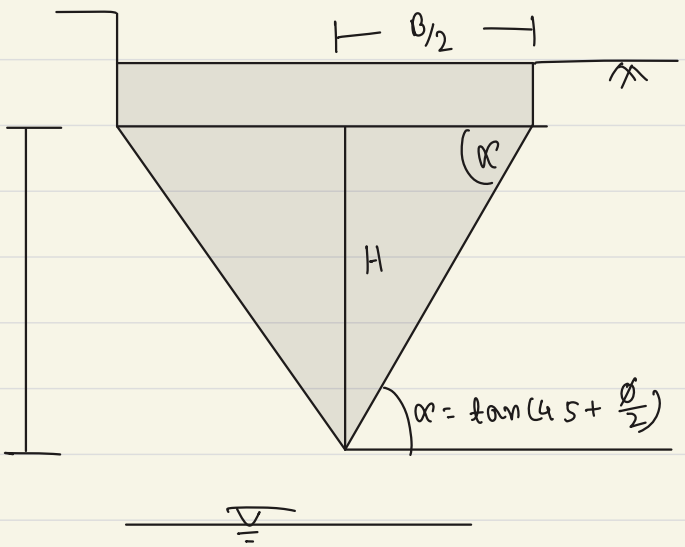
$q_a = q_u = 200$ kPa

Effects of WT on BC of Soils

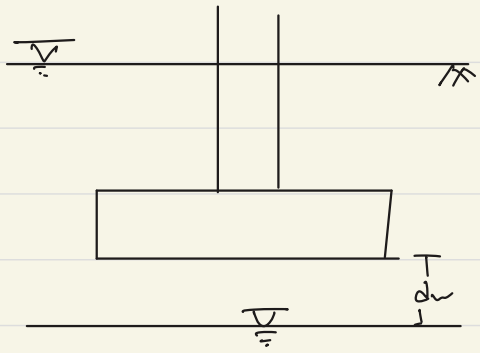
γ' = effective unit wt
$= \gamma_{sat} - \gamma_w$

For $\bar{q} = \gamma_D$
$0.5\ \gamma B'$

1. wt is seldom above base → $\bar{q}$ is adjusted
$\gamma = d_w \gamma_{we} + d_w \gamma$
2. WT is at surface
Total press = ½(WT on on below footing level)
$\gamma' = \frac{1}{2}\gamma_{sat}$



$H = 0.5 AB \tan(45 + \frac{\phi}{2})$



WT below wedge

Neglected

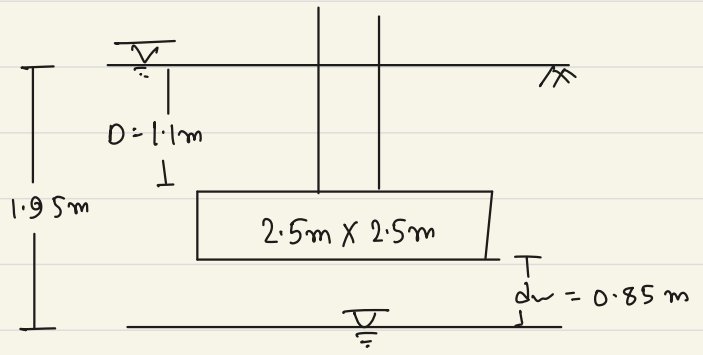
WT is within wedge zone

eqn won't be provided

$$\gamma_c = (2H - d_w)\frac{\gamma_{wet}}{HV} d_w + (H - d_w)^2 \times \frac{\gamma'}{H^2}$$

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

#



$\gamma_{wet} = 18.10$ KN/m^3

$c = 0$

$\phi_{tn} = 35°$

$G_s = 2.68$

$w_N = w = 10\%$

Check ϕ_{tn}

$$L/B = \frac{2.5}{2.5} = 1 \leq 1 \text{ ; use } \phi_{tn} = \phi_{ps} = \phi$$

Find q_a by Hansen SF = 2.0

$i_i = g_i = b_i = 1.0$

$$q_{ult} = q N_q S_q d_q + 0.5 \gamma_B N_\gamma S_\gamma d_\gamma$$

$$= \gamma_{wet} D N_q S_q d_q + 0.5 \gamma_B B' N_\gamma S_\gamma d_\gamma$$

Table 4-4 $\phi = 35$ by interpolation

$N_q = 33$ $2 \tan\phi (1 - \sin\phi)^2 = 0.255$

$N_\gamma = 34$

Table 4-5 a $\quad s_q = 1 + \frac{B'}{L} \sin\phi = 1.57$ $\qquad L/B \leq 1 \quad K = \frac{D}{B}$

$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 K$ $\qquad = \frac{1.1}{2.5}$

$= 1 + 0.255 \times 0.44 = 0.11$ $\qquad = 0.44$

$$\gamma_{dry} = \frac{\gamma_{wet}}{1+w} = \frac{18.1}{1+0.1} = 16.45$$

$$V_s = \frac{\gamma_{dry}}{G_s \gamma_w} = \frac{16.45}{2.68 \times 9.81} = 0.626 \ m^3$$

$$V_v = 1 - 0.626 = 0.374 \ m^3$$

memo

γ_{sat} = γ_{dry} + wt of water in voids

$= 16.45 + 0.374 * 9.807$

$= 20.12 \quad KN/m^3$

$$\gamma' = \gamma_{sat} - \gamma_w = 20.12 - 9.807$$

$$= 10.31 \ KN/m^3$$

$$H = 0.5 * 2.5 * \tan\left(45 + \frac{35}{2}\right)$$

$= 2.4 \ m$

$$\gamma_e = (2H - d_w)\frac{\gamma_{wet}}{H^2} d_w + (H - d_w)^2 \frac{\gamma}{H^2}$$

$$= (2 \times 2.4 - 0.85)\frac{18.10}{2.4^2} * 0.85 + (2.4 - 0.85)^2 * \frac{10.31}{2.4^2}$$

$= 14.85 \ 12N/m^3$

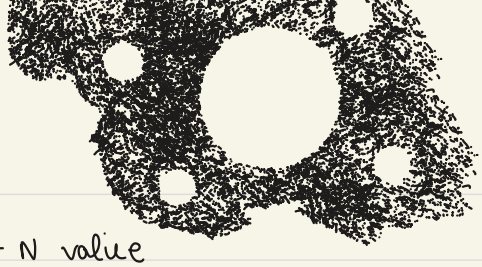
$$q_{ult} = \gamma_{wet} D N_q S_q d_q + 0.5 \gamma_B B' N_\gamma S_\gamma d_\gamma$$

$$= 18.10 * 1.1 * 33 (1.57) (1.11) + 0.5 * 14.85 * 2.5 * 34 * 0.6 \ (1)$$

$$= 1145 + 379 = 1524 \text{ KPa}$$

$$q_a = \frac{q_{ult}}{SF} = \frac{1524}{2} = 762 \text{ KPa} \qquad q_d = 500 \text{KPa}$$

$$= \frac{1145}{2} = 572 \text{ KPa}$$



Estimating of BC from SPT-N value

Estimate q_a where $N_{70} = 24$

$D = 1.0$

$B = 3.0\ m$

$$q_a = \frac{N_{55}}{F_2}\left[\frac{B+F_3}{B}\right]^2 K_d$$

$F_1 = 0.05$

$F_2 = 0.08$

$F_3 = 0.3$

$F_4 = 1.2$

Conditions:

1. N_{55} (convent)
2. $B > F_4$ (1.2)
3. $K_d = 1 + 0.33\frac{D}{B} \lesssim 1.33$

Soln:

$$N_{55} = \frac{70}{55} \times 24 \simeq 30$$

$B = 3.0 > F_4$

$$K_d = 1 + 0.33\ \frac{1.0}{3.0}$$

$$= 1.11 \lesssim 1.33 \quad [OK]$$

$$q_a = \frac{30}{0.08}\left[\frac{3+0.3}{3}\right]^2 \times 1.11$$

$$= 500.35$$

$$= 500\ kPa$$

Average $N_{70} = 6$ in effective zone for a footing located at $D = 1.6\,m$ in range of $1m - 21m$ depth

Estimate q_a for $\Delta H = 40\,mm$.

$q_u = 500\,kPa$ for $\Delta H = 25\,mm$

$\Delta H = 40\,mm$, $q_a = {}^{40}/_{25} * {}^{20}/_{500}$

$= 800\,kPa$

$$q_a = \frac{N_{55}}{F_2}\left[\frac{B+F_3}{B}\right]^2 k_d$$

$$N_{55} = \frac{70}{55} * 6 = 9$$

$$k_d = 1 + 0.33\,\frac{1.6}{1.5} = 1.35 \geqslant 1.33$$

$= 1.33$

N_{55}
$B > F_4$
$\Delta H = 25\,mm$
$k_d \leqslant 1.33$

Assume $B = 1.5\,m \sim 3.0\,m$

q_a vs B

$B = 1.5\,m$

$$q_a = \frac{9}{0.08}\left[\frac{1.5+0.3}{1.5}\right]^2 \times 1.33 = 216\,kPa$$

$B = 3\,m$

$$q_a = \frac{9}{0.08}\left[\frac{3+0.3}{3}\right]^2 \times 1.33 = 160\,kPa$$

$$k_d = \left[1 + 0.33\,\frac{1.6}{3}\right]$$

$= 1.18$ [OK]

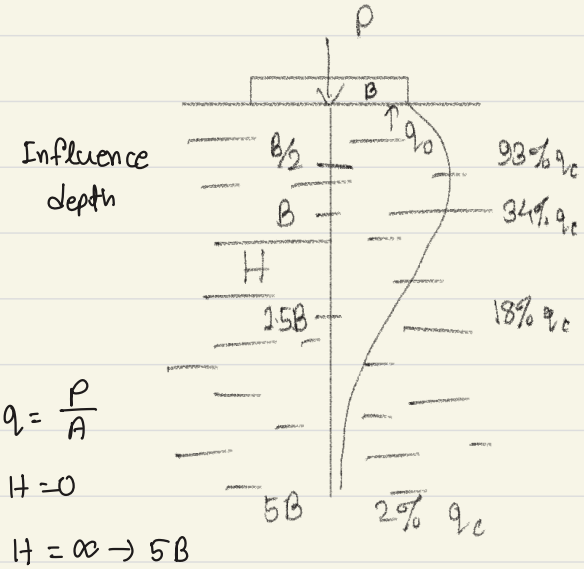
DH → statistical accumulation of vertical deformation due to application of axial load in the direction of load.

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Assignment

1. Problem (Question)
2. Figure
3. Data given
4. Detail calc
5. Refenence – table & chart

Soil settlement



1. What happens when load is applied

   a. Stress transition

   b. Change in stress Δq

   -> due to time dependent accumulation of soil particles

   -> Rolling

   -> Sliding

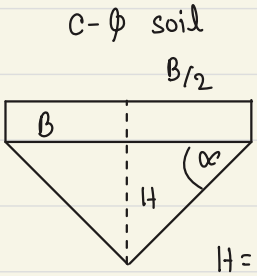
   -> Crushing

   with limited influence zone

$q = \frac{P}{A}$

$H = 0$

$H = \infty \rightarrow 5B$



$H = \frac{B}{2} \tan(45 + \phi/2)$

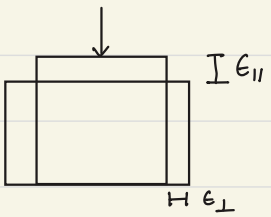
ΔH = vertical settlement

2. Principle components of vertical settlement

   a. Particles rolling, sliding

   b. " crushing

   c. Elastic deformation due to elastic parametens of soil



Elastic params:

$E = \frac{\sigma}{\epsilon}$; $G = \frac{\tau}{\gamma}$; $\mu = \frac{\epsilon_{\perp}}{\epsilon_{11}}$

3. Major problems with soil settlement analysis

a. Obtaining reliable values of elastic parameter of soils (E, G, μ)
   i. Problems of recovering undisturbed soil sample
   ii. Lab test $\geq$ 50%
   iii. Used in-situ tests results
   iv. Anisotrophy is a common problem
   v. Use connelation
   vi. For layered soils, more one set of values

b. Obtaining reliable stress profile from applied load

$$\Delta H = \int_0^{H=5B} \epsilon \, dH \qquad \epsilon = \frac{\Delta q}{E_s} \qquad \Delta q = f(H, \text{load})$$

$$E_s = f(H, \text{soil variation})$$

$$\Delta H = H, E_s, \Delta q$$

$$\frac{\Delta H_{measured}}{\Delta H_{computed}} = 0.5 \text{ to } 0.8 - 1.2$$

Immediate soil settlement As soon as load applied on within 7 days

$$\Delta H = q_0 B' \frac{1-\mu^2}{E_s} m I_s I_f$$

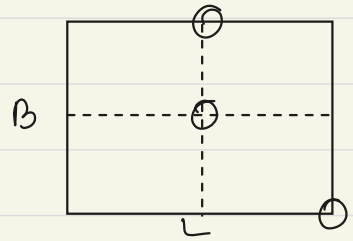
$$q_0 = \frac{P}{A} \leftarrow \gamma_c \times D$$

$B' = B/2;\ B;\ B/2$

m = number of corners contributed to settlement
→ 4, 2, ...

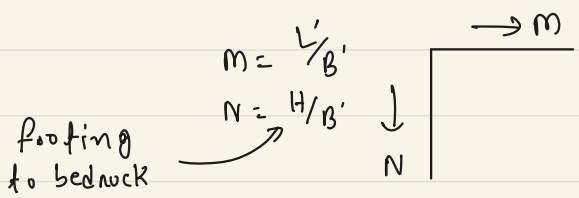
μ = chart → 0.3 to 0.5 normally

G_s = 2.65 – 2.80 Table 5–2



I_s = Influence factor from stren...

$$= I_1 + \frac{1-2\mu}{1-\mu} I_2$$



I_f = Influence factor from Fox eqn Figure 5-7; L/B, μ, D/B

Consolidation settlement Due to expulsion of water

a. Time dependent
b. 3—10 years
c. Pisa Tower > 800 year

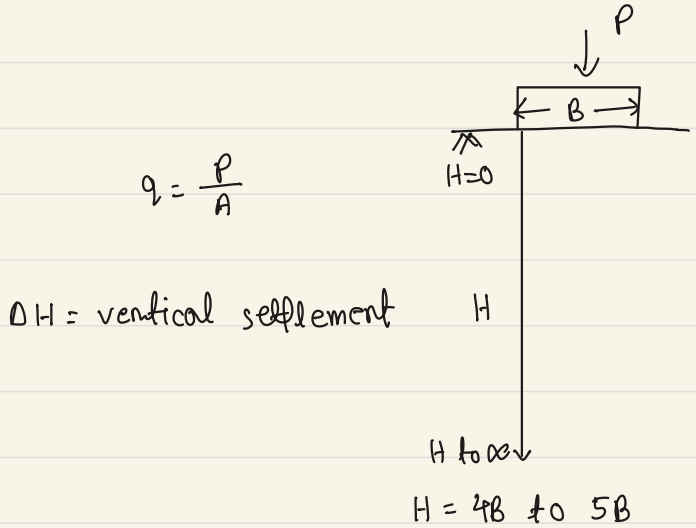
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Settlement

1. Def^n:
   Statistical accumulation of deformation of soils due to applied load in the direction of interest

2. Settlement estimation:
   a. Expected deformation
   b. Causes change in stress
   c. Produces time dependent accumulation of particles:
      i. Rolling & sliding
      ii. Grains crushing
      iii Elastic deformation

   within limited zone of interest due to applied load



$q = \frac{P}{A}$

ΔH = vertical settlement

$$\Delta H = \int_H^0 \epsilon \, dH$$

Strain, $\epsilon = \frac{\Delta q}{E_s}$ → chng in stress

$\Delta q = \int (H, \text{load})$

$E_s = \int (H, \text{soil v} \cdots)$

3. Principal components of soil settlements

a. Particles rolling & sliding

Produce void change, $e = \frac{V_v}{V_s} > 0$

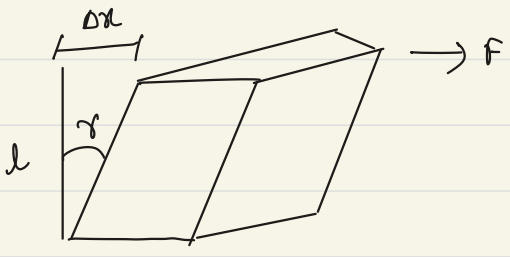
b. Grain crushing

c. Elastic deformation

Elastic params: E_s, G, μ, K_s

$$E_s = \frac{\sigma}{\epsilon}$$

$$G = \frac{\tau}{\gamma} = \frac{\text{Shear stress}}{\text{Shear strain}} = \frac{F/A}{\Delta x/l}$$



μ = Poision ratio

$$= \frac{\text{Lateral strain}}{\text{Longitudinal strain}}$$

$= \cdots$

K_s = Coefficient of subgrade constant

$$= \frac{q}{\delta} = \frac{\text{Contact pressure}}{\text{settlement}}$$

4. Problems with soil settlement analysis

a. Obtaining reliable values of soil parameters

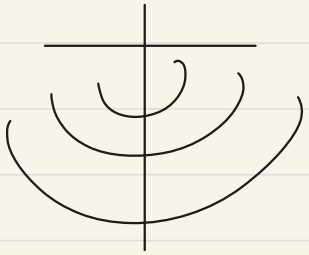
Lab tests: error 50% or more

In-situ tests: obtain horizontal value

Use of correlation

b. Obtaining reliable stress profile

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



Types of soil settlement

1. Immediate soil settlement (within handover period or 7 days)
2. Consolidation settlement (expulsion of water → vol ↓)

   Clay + water

   Time dependent — 3 to 10 years

   800 years

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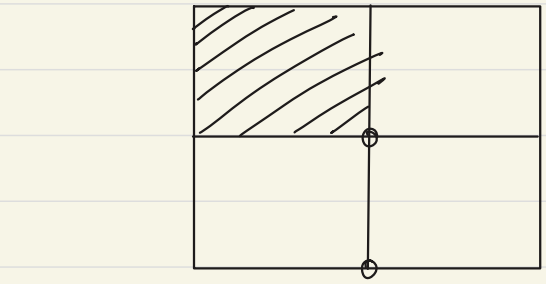
Estimation of soil settlement (immediate)

Equation from theory of elasticity (Timoseska and Goodier)

$$\Delta H = q_0 B' \frac{1-\mu^2}{E_s} m I_s I_F$$

q_0 = Intensity of pressure (in E_s unit)

$= \frac{P}{A} = \frac{P}{L \times B}$; convert non-rect to equiv rect (B, L)



← For corner settlement (in ΔH unit)

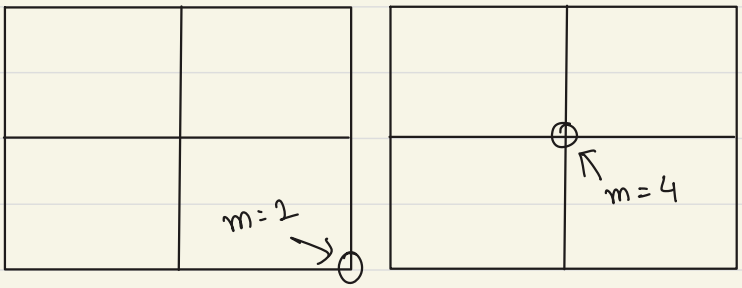
B' = Least lateral dimension

$= B/2$ for center

$= B$ for corner

$L' = L/2$ for center

$= L$ for corner



μ = From Table 2-7

E_s = From Table 2-8 (x1000 → kPa)

← m = Number of corners contributing settlement

I = Influence factor from Steinbrenner eqn

$= I_1 + \frac{1-2\mu}{1-\mu} I_2$

→ I_1, I_2 from Table 5-2

$m = L/B$; $N = H/B'$ ← influence depth

Do interpolation if reqd

I_F = Influence factor from Fox eqn

From graph → μ, L/B → round up

→ 0.35 → 0.5

$q_o = 134$ kPa

$B \times L = 33.5 \times 39.5$ m

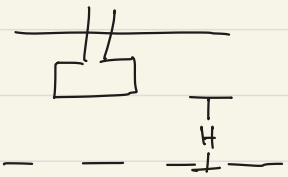
Sand is layered with one sand seam from GS to sandstone bedrock at -14 m and mat at -3 m

E_s = 3 to 6 m = 42.5 mPa

6 to 14 m = 60 mPa

E_s = Sand stone > 500 mPa

Required: Est ΔH at center of footing



$$\Delta H = q_o B' \frac{1-\mu^2}{E_s} m I_s I_f$$

m → mm

$$= 134(16.75)\frac{1-0.35^2}{55\times1000} \times 4 \times 0.121 \times 0.95 \times 1000$$

MPa → kPa

$$= 16.47 \text{ mm}$$

$H = 14\text{m} - 3\text{m} = 11\text{m}$

$B' = B/2 = 33.5/2 = 16.75$ m

$N = H/B' = 11/16.75 = 0.66 \simeq 0.7$

$m = \frac{L}{B} = \frac{39.5}{33.5} = 1.18 \simeq 1.2$

$I_1 = \frac{0.063 + 1.1}{2} = 0.0815$

$I_2 = \frac{0.083 + 0.090}{2} = 0.0865$

$\mu = 0.35$

$I_s = I_1 + \frac{1-2\mu}{1-\mu} I_2$

$= 0.0815 + \frac{1-2\times0.35}{1-0.35} \times 0.0865$

$= 0.121$

$E_{s,avg} = \frac{42.5\times3 + 60\times8}{3+8} = 55$ mPa

$m = 4$

$I_F = 0.95$

$L/B = 1.2$
$D/B = 3/33.5 = 0.089 \simeq 0.1$

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Immediate settlement

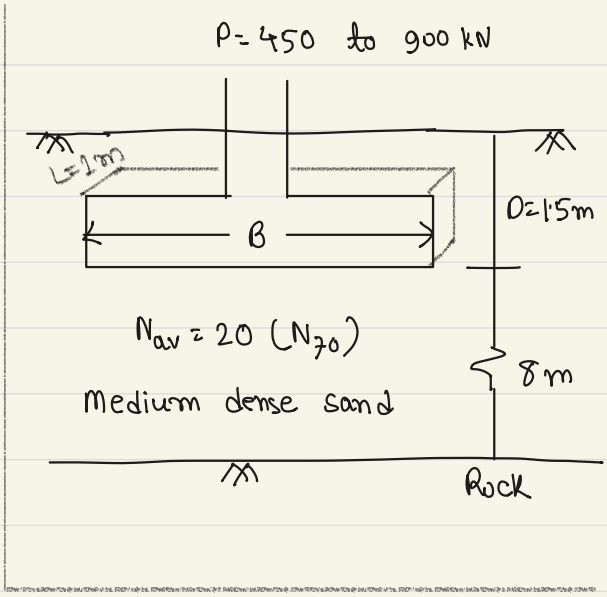
1. Joseph E Bowles — Fdn analysis and Design
2. Donald P Coduto — Geotech Engg. Principles & Practices
3. Hansen, Peck, & Thornburn — Fdn Engineering

Total settlement of a structure

$$\Delta H = \delta_i + \delta_c + \delta_s$$ (δ_i → immediate; δ_c → consolidation; δ_s → secondary compression)

$$= \Delta H_i + \Delta H_c + \Delta H_s$$

Required: Recommend q_a for this project so that $\Delta H \not> 25mm$



$$q_a = \frac{N_{55}}{F_2}\left(\frac{B+F_3}{B}\right)^2 K_d ; \quad B > F_4$$

$$= \frac{25}{0.08}\left[\frac{B+0.3}{B}\right]^2 \times \left(1+0.33\frac{1.5}{B}\right)$$

$F_2 = 0.08$

$F_3 = 0.3$

$F_4 = 1.2$

$$K_d = 1 + 0.33\frac{D}{B} \leq 1.33$$

$$N_{55} = \frac{70}{55} \times 20 = 25.45 \simeq 25$$

B (m)	$1+0.33^{1.5}/B \leq 1.33$	q_a (KPa)
1.2	1.33	$649.4 \simeq 650$
2.0	$1.247 \simeq 1.25$	$512.4 \simeq 515$
3.0	$1.165 \simeq 1.17$	440

$q_a = 535$ KPa $\simeq 500$ KPa

Actual q

$$q = \frac{P}{A} = \frac{450}{1.5 \times 1.5} = 200 \text{ KPa}$$

$$q = \frac{P}{A} = \frac{900}{1.5 \times 1.5} = 400 \text{ kPa}$$

<< KPa in Table

Tentative value Recommended $q_a = 250$ KPa

Note: The max^m $q_a = 500$ KPa with max^m $\Delta H = 25$ mm

12mm < 25 mm

250 KPa

table → 0.3 — 0.4 → 0.3

$$\Delta H = q_0 B' \frac{1-\mu^2}{E_s (\times 1000)} m I_s I_F$$

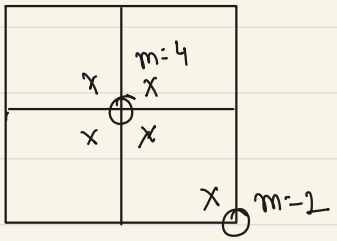
I_s → 0.507

I_F → 0.65

E_s → 20,000 KPa

$0.8 = \frac{1.6}{2}$

From theory of elasticity | See prev ds — — —



$E_s = 500(N+15) = 20000$ KPa (25)

$= 7000\sqrt{N} = 35000$ KPa

$= 2600N = 65000$ KPa

one may be used

TABLE 5-6
Equations for stress-strain modulus E_s by several test methods
E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$	$E_s = (2 \text{ to } 4)q_c$ $= 8000\sqrt{q_c}$
	‡E_s = (15 000 to 22 000) · ln N	$E_s = 1.2(3D_r^2 + 2)q_c$ *$E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0$ $F = 3.5$ $e = 0.6$ $F = 7.0$
Sands, all (norm. consol.)	¶E_s = (2600 to 2900)N	
Sand (overconsolidated)	†$E_s = 40\,000 + 1050N$ $E_{s(OCR)} \approx E_{s,NC}\sqrt{OCR}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6)$ $N \leq 15$ $= 600(N + 6) + 2000$ $N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use $^{\S}E'_s = 2.5q_c$ $2500 < q_c < 5000$ use $E'_s = 4q_c + 5000$ where E'_s = constrained modulus = $\frac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \frac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

to determine if a cohesionless deposit is overconsolidated or what the OCR

B — check $(B \times L) q_a = P_{av}$

$B^2 \times 250 = 675$

$B^2 = 2.7$

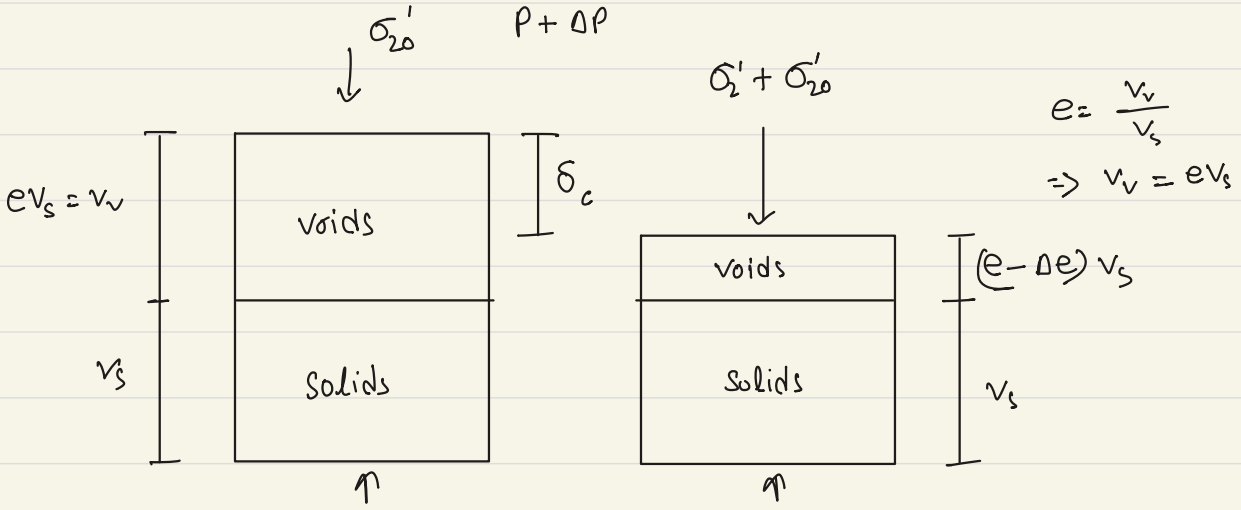
$B \cdot 1.6$ m

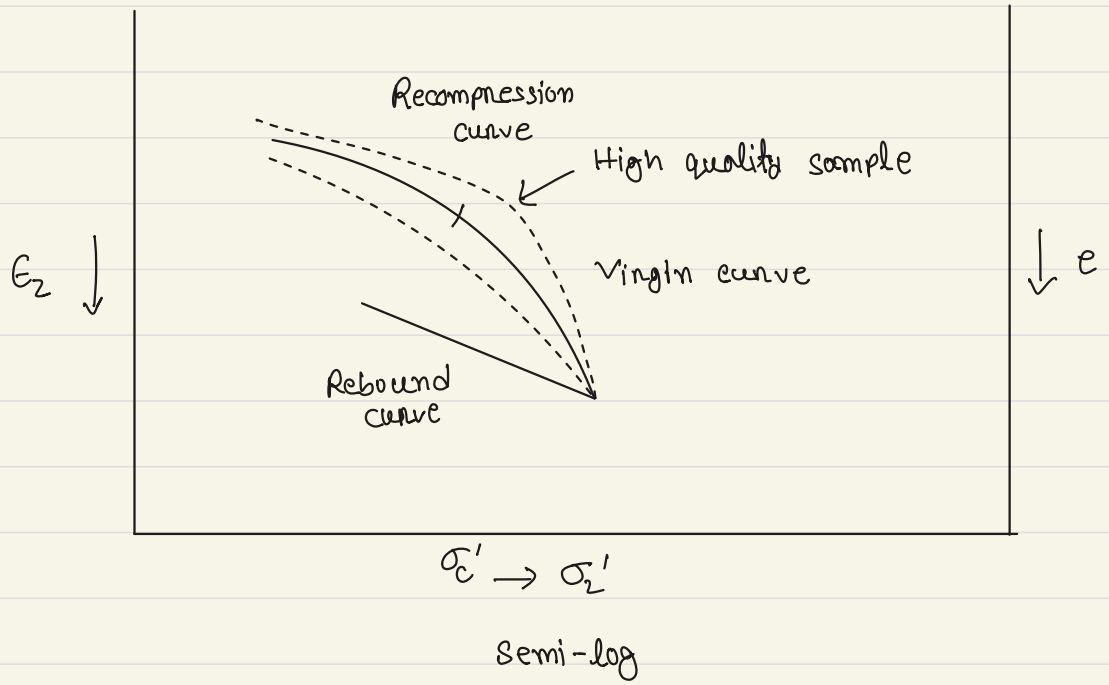
OH for $E_s = 20000\ kPa = 12\ mm$

$= 65000\ kPa = 3.69\ mm$

Consolidation settlement

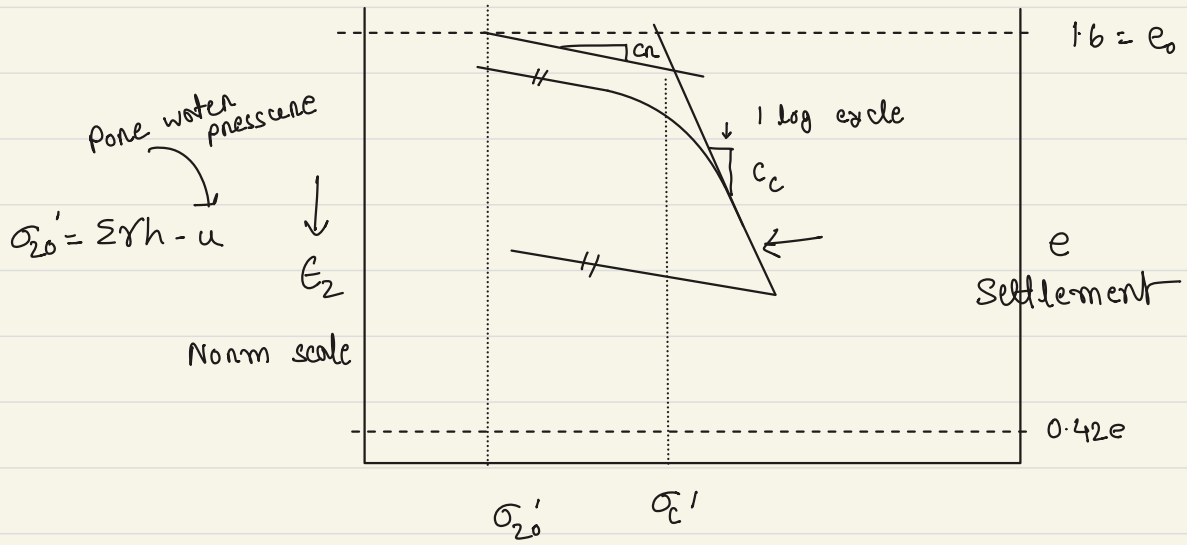
- Estimated with great care: Building, bridges, on high cost struct
- Embankments, dams, leeves, etc ← room for loose calculation
  ↖ consolidation settlement





Casagandne procedure to determine σ'_c
↓
Preconsolidation stress

Schmermann procedure to reconstruct consolidation curve



C_c = Compression index

C_r = Recompression index

$\frac{C_c}{1+e_0}$ → compression ratio

$\frac{C_r}{1+e_0}$ → Recompression ratio

Normally consolidated soils (NC):

$$\sigma_c' \simeq \sigma_{20}'$$

$$\delta_{c,ult} = \varepsilon \frac{C_c}{1+e_0} H \log\left(\frac{\sigma_{2f}'}{\sigma_{20}'}\right)$$

$\frac{C_c}{1+e_0}$ ↓ compression ratio

H = thickness of soil layers

σ_{20}' = Initial eff. stress

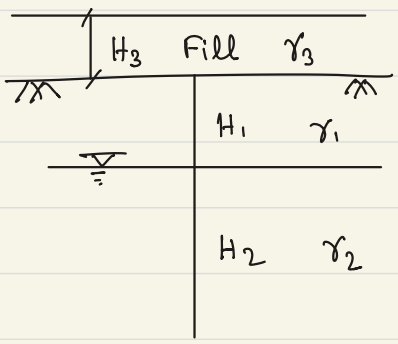
σ_{2f}' = Final " "

$$\sigma_{20}' = \Sigma\gamma h - u$$

$$= \gamma_1 H_1 + \gamma_2 H_2 - \gamma_w H_2$$

$$\sigma_{2f}' = \sigma_{20}' + \gamma_3 H_3$$

σ_c'



Consolidation settlement

$$\delta_{c,ult} = \sum \frac{C_c}{1+e_0} H \log\left(\frac{\sigma_{zf}'}{\sigma_{z0}'}\right)$$

$\sigma_c' \simeq \sigma_{z0}'$ approx 20%

↓

Soil is normally consolidated NC soil

Ex 11.5

$C_c = 0.40$

$e_0 = 1.10$

$C_r = 0.08$

$\sigma_c' = 72$ kPa

σ_c' = Preconsolidation stress

σ_{z0}' = Init eff. stress

$= \Sigma \gamma h - u (= \gamma_w h_w)$

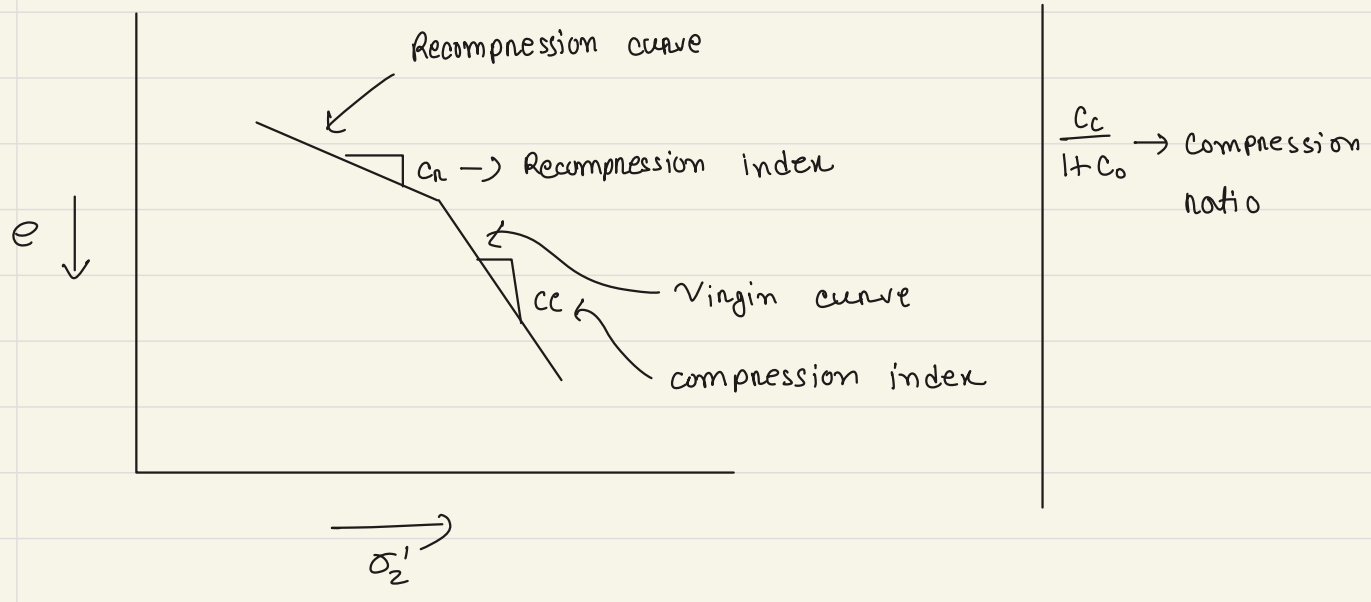
OC → Over consolidated

$$\sigma_{z0}' = 18.5(1.5) + 19.2(2) + 16.0(4) - 9.81(2+4)$$

$$= 72 \text{ kPa} \quad [\simeq \sigma_c'] \quad [\because \text{Soil is NC}]$$

$$\sigma_{zf}' = \sigma_{z0}' + \gamma_{fill} h_{fill} = 72 + 19.2(3) =$$

$$= \sigma_{z0}' + 57.6 \text{ kPa}$$



$\frac{C_c}{1+e_0} = \frac{0.4}{1+1.0} = 0.19$

$\frac{C_c}{1+e_0} = 0.008$

divide arbitrarily

1 – 4 m

$C_c = 0.40$

$C_r = 0.08$

$e_0 = 1.10$

$\sigma_c' = 70$ kPa

Soil type : NC — Normally consolidated soil

OC —

σ_{z0}' = Initial - eff. stress

= 72 kPa

$$\delta_{c,ult} = \sum \frac{C_c}{1+e_0} H \log \frac{\sigma_{zf}'}{\sigma_{z0}'}$$

$$\sigma_{zf}' = \sigma_{z0}' + \gamma_{fill} h_{fill}$$

$$= \sigma_{z0}' + 19.2 \times 3$$

$$= \sigma_{z0}' + 57.6$$

} for each layer

Layer 1 : $H = 1.5$ m

midpoint = $\frac{1.5}{2} = 0.75$ m

$\frac{C_c}{1+e_0} = 0.008$

→ Dr % from table

$\sigma_{z0}' = \Sigma \gamma h - u$

$= 18.5 \times 0.75 - 0 = 13.875$

$\sigma_{zf}' = \sigma_{z0}' + \gamma_{fill} h_{fill}$

$= 13.87 + 57.6 = 71.5$

$$\delta_{c1,ult} = \sum \frac{C_c}{1+e_0} H \log \frac{\sigma_{zf}'}{\sigma_{z0}'}$$

$$= 0.008(1.5) \log \frac{71.5}{13.875} = 0.008 \text{ m} = 8 \text{ mm}$$

Layer 1 : $H = 2.0\ m$

$$\text{midpoint} = \frac{2}{2} = 1.0\ m$$

$$\sigma_{z0}' = \Sigma \gamma h - u$$

$$= 18.5 \times 1.5 - 19.5(2) - 9.81(1) = 37.4\ kPa$$

$$\sigma_{zf}' = \sigma_{z0}' + \gamma_{fill} h_{fill}$$

$$= 37.4 + 57.6 = 95\ kPa$$

$$\delta_{c,ult} = \Sigma \frac{C_c}{1+e_0} H \log \frac{\sigma_{zf}'}{\sigma_{z0}'}$$

$$= 0.008(2) \log \frac{95}{37.4} = 0.006\ m = 6\ mm$$

Layer 3 : $H = 3\ m$

$$\text{midpoint} = \frac{3}{2} = 1.5\ m$$

$$\frac{C_c}{1+e_0} = \frac{0.4}{1+1.10} = 0.19$$

$$\sigma_{z0}' = \Sigma \gamma h - u$$

$$= 18.5(1.5) + 19.5(2) + 16.0(1.5) - 9.81(2+1.5) = 56.4\ kPa$$

$$\sigma_{zf}' = \sigma_{z0}' + \gamma_{fill} h_{fill}$$

$$= 56.4 + 57.6 = 114\ kPa$$

$$\delta_{c,ult} = \Sigma \frac{C_c}{1+e_0} H \log \frac{\sigma_{zf}'}{\sigma_{z0}'}$$

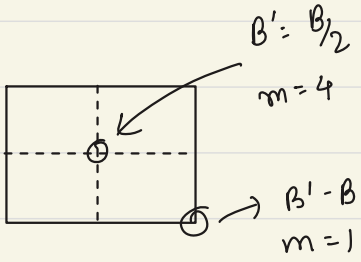
$$= 0.19\ (3) \log \frac{114}{56.4} = 0.1746\ m = 175\ mm$$

Consolidation settlement :

1. Normally consolidated soils (NC soils)
2. Overconsolidated soils (OC soils)

$$\Delta H = q_0 B' \frac{1-\mu^2}{E_s} m I_s I_F$$

(q_0: P/A; B': eff. breadth; μ: 0.3 to 0.4)



$$I_s = I_1 + \frac{1-2\mu}{1-\mu} I_2$$

Table 5.2 m vs N

m ↓ L/B N ↓ H/B'

$I_F = \cdots$

SPT N

$$q_a = \frac{N_{55}}{F_2} \left[\frac{B + F_3}{B} \right]^2 K_d$$

$$K_d = 1 + 0.33 \frac{D}{B} \leq 1.33$$

$F_2 = 0.08$

$F_3 = 0.3$

$F_4 = 1.2$

consolidated settlement

(σ_{z0}': init eff. stress; σ_c': Pre consolidation stress)

$$\sigma_{z0}' \simeq \sigma_c'$$

$$\varepsilon \frac{C_c}{1+e_0} H \log \frac{\sigma_{zf}'}{\sigma_{z0}'}$$

$\sigma_{z0}' < \sigma_c'$: Overconsolidated soils

Case 1: $\sigma_{z0}' < \sigma_{zf}' \leq \sigma_c'$

$$\delta_{c,ult} = \sum \frac{C_r}{1+e_0} H \log \frac{\sigma_{zf}}{\sigma_{z0}'}$$

Case 2: $\sigma_{z0}' < \sigma_c' < \sigma_{zf}'$

$$\delta_{c,ult} = \sum \left[\frac{C_r}{1+e_0} H \log \frac{\sigma_c'}{\sigma_{z0}'} + \frac{C_c}{1+e_0} H \log \frac{\sigma_{zf}'}{\sigma_c'} \right]$$

$$\delta = \delta_i + \delta_c + \delta_s$$

Rough / Final 29/04

Lec 01 Intro

Geotechnical engg: Branch of CE that deals with soil, rock, and underground water and their relation to design, const., and op.

Bedrock: Comparatively harder stratum beneath surface deposits of loose soil, it can carry structural load.

Importance of studying Geotech engg.:
Almost all CE structures are supported by soil

Issues to be addressed: SSG-ISFR-TEC

1. Subsurface soil & rock condition
2. Can they safely support...
3. GWT location
4. Impact of excavation, drilling, filling
5. Stability of proposed/natural earth slopes stable
6. Type of fdn req.
7. Retaining wall req
8. Tunnel req
9. Earthquake respond
10. Is ground contaminated

Geotechnical engineering process:

1. Assess of underground condition at site
2. Engineering analysis of collected data
3. Result analysis to generate geotech input
4. Work as part of team – struct. engg., architect
5. Service during construction phase
6. Service beyond construction phase

Req. of geotechnical engineer:

1. Geotechnical knowledge
2. Avoidance of failure
3. Assessment of subsurface condition
4. Involvement in construction

Lec 02 Foundation engineering

Fdn: Element of structure that connects structure to ground and transfers load safely & economically.

Fdn engineering: Branch of Geotech engg. that applies soil mechanics and structural engineering for the purpose of design and construction of foundations.

Purpose of fdn:

1. Distribute ld
2. Prevent bearing cap. failure and excess settlement
3. Find hard strata
4. Improve stability of struct

Bearing capacity failure: when ss > stren of soil (shear)

→ 3 cat: General, local, punching

Soil's shear stren:

(i) Interlocking
(ii) Frictional resistance btwn invidual soil particles
(iii) Adhesion: attachment btwn water & soil
(iv) Cohesion: like particles to hold on to each other

$$\tau = c + \sigma \tan\phi$$

Type of fdn: Shallow, deep fdn

$D \leq B$

Shallow fdn:

Advantage:

(i) Construction simple, less costly
(ii) Equipments are less costly & simple
(iii) Less time req
(iv) Less disturbance of soil
(v) Lower uncertainty of behavior

Disadv:

(i) Chances of scouring if near river
(ii) Uneconomical if GWT is high
(iii) Can't be used if soil's bearing cap is lower

Type of shallow fdn:

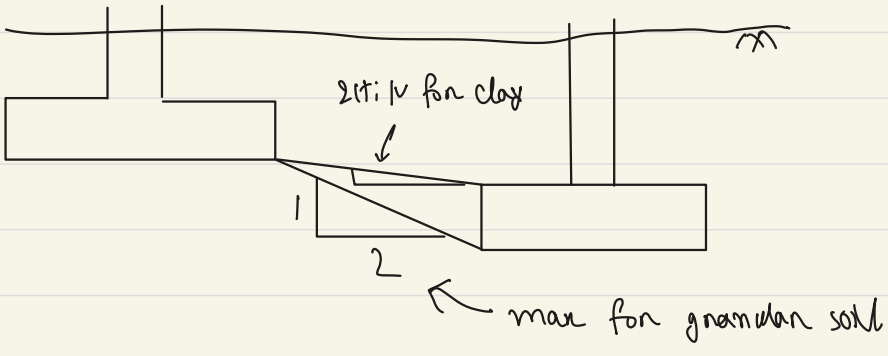
(i) Isolated footing
(ii) Continuous footing
(iii) Combined footing
(iv) Raft fdn

Use of combined footing:

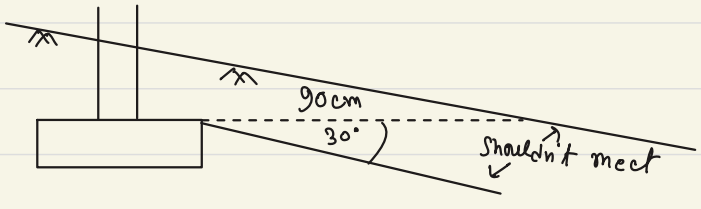
(i) Strap footing would overlap
(ii) Footings are closely together (due to low q_net)
(iii) Dimension of one footing is restricted

Factors affecting depth of footing

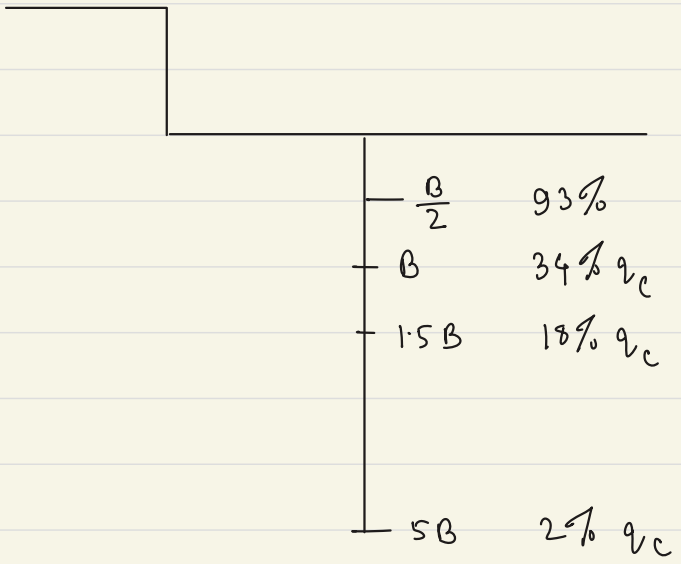
(i) Top Layer: below thickness on organic mats
(ii) Ground freeze: below frost penetration
(iii) Vol change: below zone of vol chng
(iv) GWT: above
(v) Depth of scour
(vi) Underground utilities
(vii) Property line
(viii) Footing at diff level



(ix) sloping ground:



Influence depth of base contact pressure:



Bearing capacity

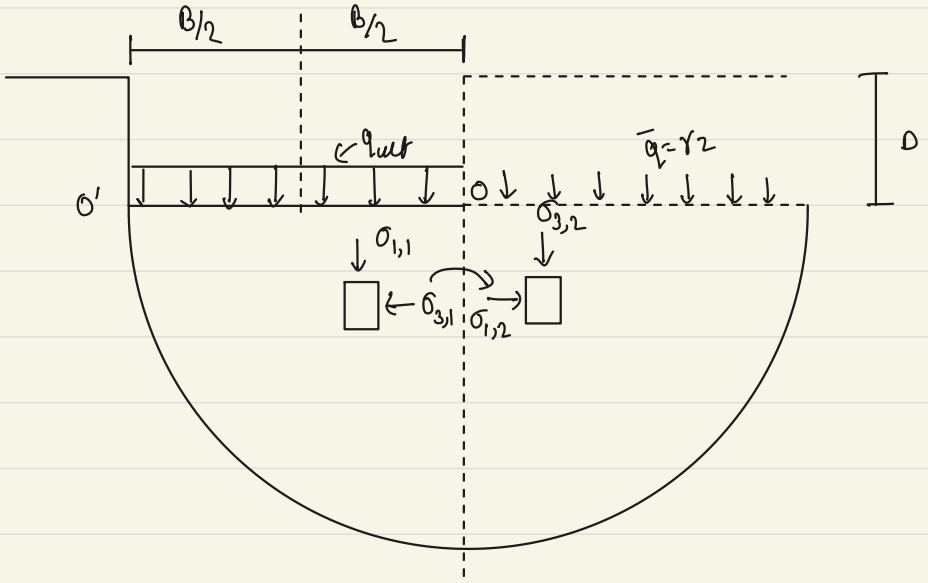
Allowable bearing cap is minimum of

i) Limiting settlement to a tolerable amount
ii) Ult. bearing cap

$$q_a = \frac{q_{ult}}{S.F}$$

S.F. depends on types of soil
reliability of param
structural info

Bearing capacity of $\phi = 0$ soil:



From Bells eqn: $\sigma_1 = \sigma_3 \tan^2(45° + \frac{\phi}{2}) + 2c \tan(45° + \frac{\phi}{2})$

when $\phi = 0$: $\sigma_1 = \sigma_3 + 2c$

From diagram: $\sigma_{3,2} = \bar{q}$

$\therefore \sigma_{1,2} = \bar{q} + 2c$

Again, $\sigma_{3,1} = \sigma_{1,2} = \bar{q} + 2c$

$\therefore \sigma_{1,1} = \bar{q} + 2c + 2c$

$= \bar{q} + 4c$

$\therefore q_{ult} = \bar{q} + 4c$

Bearing capacity, $D = 0$; $q_{ult} = 0 + 4c = 4c$

Critical excavation depth, $0 = \bar{q} + 4c \Rightarrow \gamma D_c = -4c$

$\therefore D_c = -\frac{4c}{\gamma} \rightarrow D_c = \frac{4c}{\gamma}$

Considering footing rotation about point O:

footing pressure q_{ult} resisted by perimeter shear + overburden pressure

$$q_{ult} \times B \times \frac{B}{2} = \pi B \times C \times B + \bar{q} \times B \times \frac{B}{2}$$

$$\Rightarrow q_{ult} B^2 = 2\pi C B^2 + \bar{q} B^2$$

$$\Rightarrow q_{ult} = 2\pi C + \bar{q}$$

$$= 6.28\, C + \bar{q}$$

At $D = 0$: $q_{ult} = 6.28\, C$

Avg: $\frac{6.28 + 4}{2} C = 5.14\, C$

Footing on c–φ soil:

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

$$q_{ult} = cN_c + qN_q + \gamma B N_\gamma$$

$$= c\left[\frac{2K_p}{\cos\phi} + \sqrt{K_p}\right] + q\left[\frac{K_p\sqrt{K_p}}{\cos\phi}\right] + \gamma B \frac{1}{4}\left[\frac{K_p^2}{\cos\phi} - \sqrt{K_p}\right]$$

q_{ult} by above eqn is underestimated, because:

i) Block agf is neglected
ii) Block agfe is poorly defined
iii) considers unit width strip footing → need shape factor
iv) Neglects friction from ground surface and ae → needs depth fact
v) For inclination, need inclination factor

BC equations:

1. Terzaghi eqn: for shallow fdn
used shape factors

2. Meyerhof's eqn: included shape factor, depth factor, inclination fact
upto $D \approx B$, similar to Terzaghi

3. Hansen eqn: Proposed general eqn
In addition to Meyerhof's: b_i → footing inclination
g_i → ground surface incl.
Allows any D/B implicitely

4. Vesic eqn: Same as Hansen except N_γ, i_i, b_i, g_i
Easier to use than Hansen

When to use which:

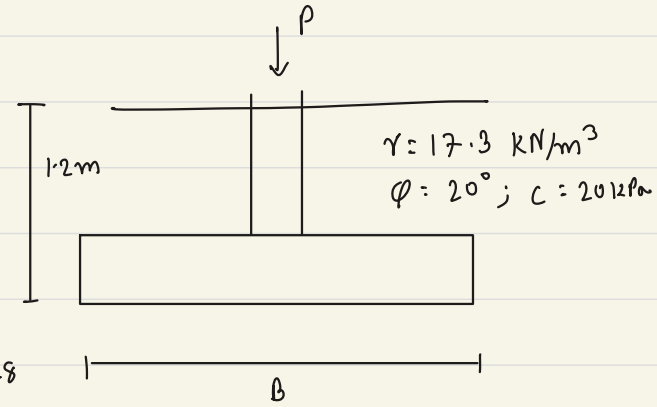
Terzaghi: when soil is cohesive, $D/B \leq 1$
For quick test of q_{net}
No to be used when moment is present
on hor. force, sloping ground

Meyerhof Hansen, Vesic: Any situation that applies, based on user preference

Hansen, Vesic: when base is tilted, ground is sloped, or $D/B > 1$

Use to then compare and perform 3

Problem 1



Using Terzaghi eqn:

$q_{ult} = cN_c s_c + \bar{q} N_q + 0.5 \gamma B N_\gamma s_\gamma$

$= 20 \times 17.7 \times 1.3 + 17.3 \times 1.2 \times 7.4 + 0.5 \times 17.3 \times B \times 5 \times 0.8$

$= 613.8 + 34.6B$

$q_a = \frac{q_{ult}}{3} = 204.6 + 11.5B$

B likely to range from 1.5m to 3m

Factors:

$N_c = 17.7$, $N_q = 7.4$, $N_\gamma = 5$

$s_c = 1.3$, $s_\gamma = 0.8$

$B = 1.5$, $R_\gamma = 1$, $q_a = 204.6 + 11.5 \times 1.5 = 222$ kPa

$B = 3$ m, $R_\gamma = 0.95$, $q_a = 0.95 \times 204.6 + 11.5 \times 3 = 229$ kPa

$\therefore q_a = 200 - 220$ kPa

General eqn:

$$q_{ult} = c\left[\frac{2K_p}{\cos\varphi} + \sqrt{K_p}\right] + q\left[\frac{K_p\sqrt{K_p}}{\cos\varphi}\right] + \frac{\gamma B}{4}\left[\frac{K_p^2}{\cos\varphi} - \sqrt{K_p}\right]$$

$= 179.8 + 12.8B$

$K_p = \tan^2\left(45° + \frac{\varphi}{2}\right)$

$= 2.04$

$q_a = 59.9 + 4.3B$

$B = 1.5 \Rightarrow 66$ kPa

$3.0 \Rightarrow 73$ kPa

Problem 2:

$D = 0.5\ m, \quad B = 0.5\ m, \quad L = 2.0\ m$

$\gamma' = 9.31\ kN/m^3 \qquad \phi_{tn} = 42.7°$

$c = 0$

$\gamma_B = \frac{2}{0.5} = 4 > 2; \quad \therefore \phi_{ps} = 1.5\phi_{tn} - 17° = 47°$

Using Hansen eqn: $c = 0$

$$q_{ult} = \cancel{cN_c}^{0} s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q i_q g_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

$$i_c = g_c = b_c = i_q = g_q = b_q = i_\gamma = g_\gamma = b_\gamma = 1$$

N-factors: $N_q = 208.22; \quad N_\gamma = 347.3$

S-factors: $s_q = 1 + \frac{B'}{L'}\sin\phi \qquad s_\gamma = 1.0 - 0.4\frac{B'}{L'} \geq 0.6$

$= 1 + \frac{0.5}{2}\sin 47° \qquad \therefore 0.9$

$= 1.18$

Depth-factors: $d_q = 1 + 2\tan\phi(1 - \sin\phi)^2 k \qquad D/B = 1 \leq 1$

$= 1.15 \qquad \therefore k = 1$

$d_\gamma = 2.00$

$\therefore q_{ult} = 9.31 \times 0.5 \times 208.22 \times 1.18 \times 1.15 + 0.5 \times 9.31 \times 0.5 \times 347.3 \times 0.9 \times 1$

$= 2043\ kPa$

Meyerhof eqn : $c = 0$

$$q_{ult} = \bar{q} N_q s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma$$

N-factors : $N_q = 208.22$; $N_\gamma = 506.1$

Shape factors : $s_q = s_\gamma = 1 + 0.1 K_p \frac{B}{L}$; $K_p = \tan^2(45° + \frac{\phi}{2})$

$= 1.16$ $= 6.44$

Depth factors : $d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D}{B}$

$= 1.25$

$\therefore q_{ult} = 9.3 \times 0.5 \times 208.22 \times 1.16 \times 1.25 + 0.5 \times 9.31 \times 0.5 \times 506.1 \times 1.16 \times 1.25$

$= 3113$ kPa

$$q_{ult(measured)} = \frac{P_{ult(mes)}}{B \times L} = 1863 \text{ kN/m}^2$$

Ex 4-3 $B = L = 1.05\ m$, $D = 1.5\ m$,

At 1" settlement, load = 14.1 tons

$q_u = 3.0\ ton/m^2$; $S_u = 3.0\ ton/m^2$, $S_{u,vane} = 2.4\ ton/m^2$

$w_L = 80\%$, $w_p = 35\%$

Hansen eqn: $\phi = 0°$; hor.

$$q_{ult} = 5.14 s_u (1 + s_c' + d_c') + \bar{q}$$

$S_{u,design} = \gamma S_{u,vane} = 0.8 \times 2.4\ ton/m^2 = 1.92\ ton/m^2$

$I_p = 80 - 35 = 45 \rightarrow \gamma = 0.8$

Shape factor: $s_c' = 0.2 \frac{B'}{L'} = 0.2 \times 1 = 0.2$

Depth factor: d_c' $0.4K = 0.384$ | $D/B = \frac{1.5}{1.05} > 1.0$

neglect ↙

$\therefore q_{ult} = 5.14 \times 1.92 \times (\ldots) + 0$ | $\therefore K = \tan^{-1}(D/B) = 0.96^R$

$= 15.63\ ton/m^2$

$$q_{ult(mes)} = \frac{P}{A} = \frac{14.1}{1.05^2} = 12.79\ ton/m^2$$

From q_u: $S_u = 1/2\, q_u = 1.5\ ton/m^2$

$$\therefore q_{ult} = \frac{15.63}{1.92} \times 1.5 = 12.21\ ton/m^2$$

Ex. 4-4: unconfined compression tests → clay ($\phi = 0$)

$q_u = 200$ kPa ; SF = 3.0

Meyerhof eqn:

$$q_{ult} = cN_c s_c d_c + \bar{q} N_q s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma$$

$c = \frac{q_u}{2} = 100$ kPa

N factors: $N_c = 5.14$, $N_q = 2.0$, $N_\gamma = 0$

Shape factors: $s_c = 1 + 0.2 K_p \frac{B}{L}$ $\quad K_p = \tan^2(45 + \frac{\phi}{2})$

$= 1 + 0.2 \times 1 = 1.2$ $\quad = 1$

$s_q = 1$

Depth factors: $d_c = 1 + 0.2 \sqrt{K_p} \frac{D}{B} \rightarrow 1$

$d_q = 1$

$\therefore q_{ult} = c \times 5.14 \times 1.2 + \bar{q} = 6.17\, c + \bar{q}$

$q_a = 2.06\, c + 0.33 \bar{q} = 1.03\, q_u + 0.33 \bar{q} \sim 1.0\, q_u$

Terzaghi eqn:

$$q_{ult} = cN_c s_c + \bar{q} N_q$$

$N_c = 5.7$, $N_q = 1.0$

$s_c = 1.3$

$= 5.7 \times 1.3 \times c + \bar{q}$

$= 7.41\, c + \bar{q}$

$q_a = 2.47 c + 0.33 \bar{q}$ ← neglect

$= 1.24\, q_u + 0.33 \bar{q}$

$\therefore q_a \approx 200$ kPa

when SPT sample is used, $q_a \approx q_w$

Not recommended if $q_u < 75$ kPa

Footing with eccentricity:

$L' = L - 2e_x$

$B' = B - 2e_y$

$e_x = \frac{M_y}{V}$; $e_y = \frac{M_x}{V}$

For design, ACI 318 - :

$B_{min} = 4e_y + w_y$

$L_{min} = 4e_x + w_x$

cal $w_x \times w_y$ dim

Det^n q_{ult} : 2 methods

2. Use Hansen or vesic BC eq w/ adjust:

Use B' & L' in $\gamma B N_\gamma$ term

B' & L' in shape factors

B & L in depth "

2. Use Meyerhof eq w/ reduction factor:

$0 < \frac{e}{B} < 0.3$

$q_{ult,des} = q_{ult,comp} \times R_e$

$R_{eB} = \left(1 - \frac{2e_x}{B}\right)$ for cohesive or $\left(1 - \sqrt{\frac{e_x}{B}}\right)$ for cohesionless

$R_{eL} = 1 - \frac{2e_x}{L}$ $\left(1 - \sqrt{\frac{e_x}{L}}\right)$

$R_e = R_{eB} \times R_{eL}$

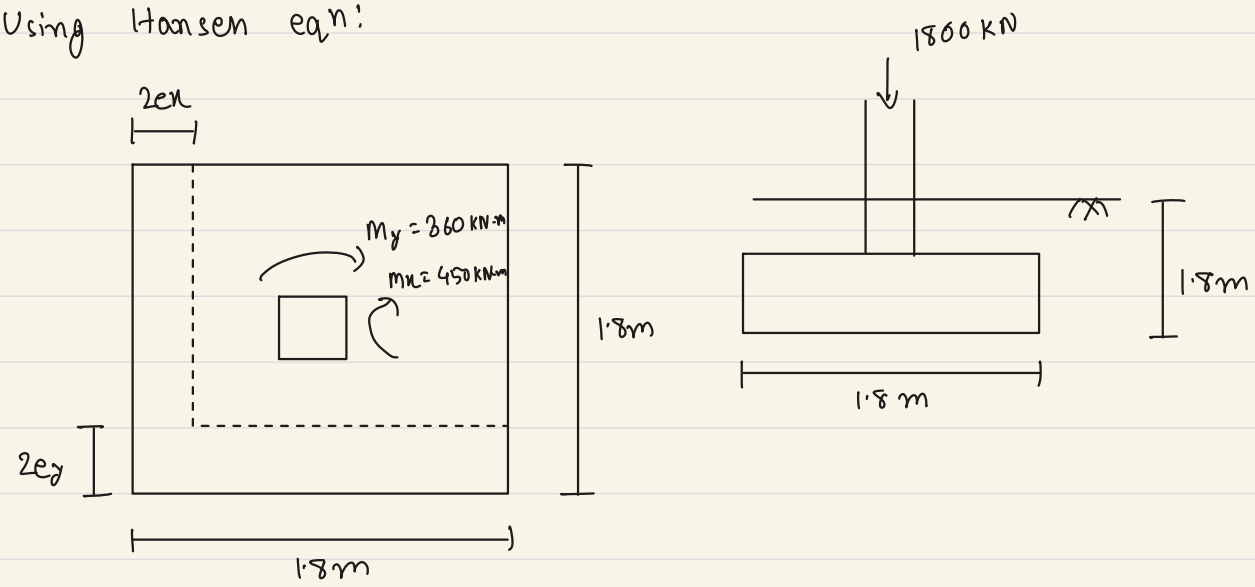
Ex. 4.5: Sqr footing, 1.8m x 1.8m, $P = 1800$ KN, $m_x = 450$ KN-m

$m_y = 360$ KN-m,

$\phi_{tr} = 36°$, $C = 20$ KPa, $\gamma = 18$ KN/m^3

$D = 1.8$ m, $D_{WT} = 6.1$ m

Using Hansen eqn:



$$e_x = \frac{m_y}{V} = \frac{360}{1800} = 0.20 \qquad e_y = \frac{450}{1800} = 0.25$$

$$B' = B - 2e_y = 1.3\text{ m}; \quad L' = L - 2e_x = 1.4\text{ m}$$

$$q_{net} = cN_c s_c d_c + \bar{q} N_q s_q d_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma$$

$$\phi: \frac{L}{B} = 1 \leq 2 \quad \therefore \phi = \phi_{tr} = 36°$$

N-factors: $N_c = 50.55$, $N_q = 37.7$, $N_\gamma = 40$

Shape: $s_c = 1 + \frac{N_q}{N_c}\frac{B'}{L'} = 1.69$

$$s_q = 1.0 + \frac{B'}{L'}\sin\phi = 1.55$$

$$s_\gamma = 1 - 0.4\frac{B'}{L'} = 0.63$$

Depth factors: $d_c = 1 + 0.4k = 1.4$ $\quad \frac{D}{B} = 1 \leq 1$

$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 k \quad \therefore k = 1$

$= 1.25$

$d_\gamma = 1$

$\therefore q_{ult} = 2392 + 2366.62 + 294.84 = 5053 \text{ kPa}$

$\therefore q_a = \frac{q_{ult}}{3} = 1684 \text{ kPa} \simeq 1700 \text{ kPa}$

[alt: $\frac{e_x}{L} = 0.11 < 0.3$]

Using Meyerhof method w/ Re: (Considering cohesionless as C is only 20kPa)

$Re = Re_L \times Re_B$

$= 0.418$

$Re_L = 1 - \sqrt{\frac{e_x}{L}} = 0.667$

$Re_B = 1 - \sqrt{\frac{e_y}{B}} = 0.627$

$q_{ult} = cN_c s_c d_c + \bar{q} N_q s_q d_q + 0.5\gamma B^{1.8m} N_\gamma s_\gamma d_\gamma$

N-factors: $N_c = 50.55, \quad N_q = 37.7, \quad N_\gamma = 44.4$

Shape fac.: $s_c = 1 + 0.2 K_p \frac{B}{L}$

$= 1.77$

$K_p = \tan^2(45° + \frac{\phi}{2})$

$= 3.85$

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

$= 1.39 = s_\gamma$

Depth fac.: $d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$

$= 1.39$

$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\frac{D}{B}$

$= 1.20$

$\therefore q_{ult} = 2487 + 2037 + 1200 = 5724 \text{ kPa}$

$q_{ult,ne} = 2393 \text{ kPa} \quad q_{all} = 797 \simeq 800 \text{ kPa}$

Suggestion of footing : Limit q_a = 500 KPa

$\therefore A' = \frac{1800}{500} = 3.6\ m^2$

$\frac{B'}{L'} = \frac{1.3}{1.4} = 0.929$; $3.6 = L' \times 0.929 L' \Rightarrow L' = 1.97\ m$

$\therefore B' = 1.83\ m$

$L = L' + 2e_x = 1.97 + 0.4 = 2.37\ m$

$B = B' + 2e_y = 1.83 + 0.5 = 2.33\ m$

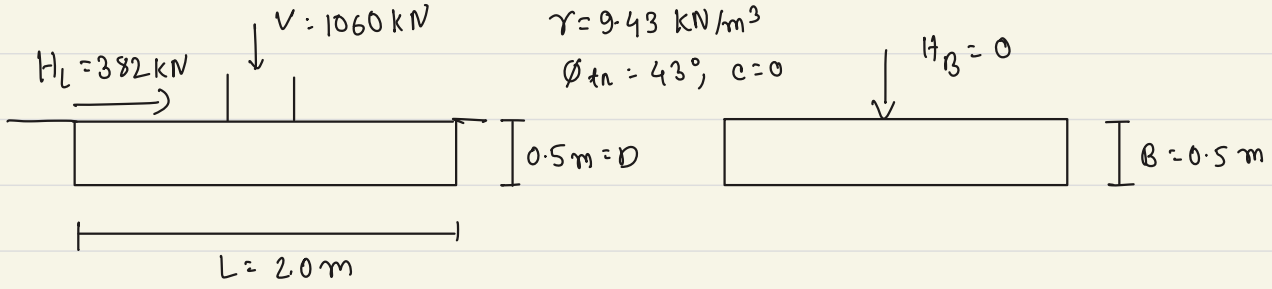
$\therefore$ Footing of 2.4m X 2.4m

Using Meyerhof :

$L \times B = L^2 = 3.6 \Rightarrow L = 1.9\ m = B$

$\therefore$ 2m x 2m footing

Footing w/ inclined load: Hansen & Vesic



$L/B = 4 > 2$; $\phi_{ps} = 1.5 \times 43 - 17^\circ$
$= 47.5^\circ$

Hansen method: $C = 0$

$$q_{ult} = \bar{q} N_q S_{qL} d_{qL} i_{qL} + 0.5 \gamma L' N_\gamma S_{\gamma L} d_{\gamma L} i_{\gamma L}$$

$$\text{and} \quad = \bar{q} N_q S_{qB} d_{qB} i_{qB} + 0.5 \gamma B' N_\gamma S_{\gamma B} d_{\gamma B} i_{\gamma B}$$

—

N: $N_q = 20.8$, $N_\gamma = 34.7$

i: $i_q = \left[1 - \frac{0.5 H_i}{V + A_f C_a \cot\phi}\right]^{2.5}$ ($A_f C_a \cot\phi \to 0$)

$\therefore i_{qB} = [1 - 0) = 1$

$i_{qL} = 0.608$

$i_\gamma = \left[1 - \frac{0.7 H_i}{V + A_f C_a \cot\phi}\right]^{3.5}$

$\therefore i_{\gamma B} = 1$

$i_{\gamma L} = 0.362$

Shape:

$$S_{qB} = 1 + \frac{B' i_B}{L'} \sin\phi = 1.18$$

$$S_{qL} = 1 + \frac{L' i_B}{B'} \sin\phi = 2.79$$

$$S_{\gamma B} = 2.0 - 0.4 \frac{B' i_B}{L' i_B} = 0.724$$

$$S_{\gamma L} = 1 - 0.4 \frac{L' i_L}{B' i_B} = 0.421$$

Depth:

$$d_{qB} = 1 + 2\tan\phi (1 - \sin\phi)^2 \times K_B$$

$$= 1.15$$

$$d_{qL} = 1 + 2\tan\phi (1 - \sin\phi)^2 \times K_L$$

$$= 1.04$$

$$d_\gamma = 1$$

$\frac{D}{B} = 2$; $K = 1$

$\frac{D}{L} = 0.25$; $K = 0.25$

$\therefore q_{ult,B} = 1923$ kPa

$q_{ult,L} = 2248$ kPa

$q_{ult} = 1900$ kPa

Vesic method:

$$q_{ult} = \bar{q} N_q s_q d_q i_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma i_\gamma$$

$N_q = 147$, $N_\gamma = 404$ [from eqn]

Shape: $s_q = 1 + \frac{B}{L} \tan\phi = 1.27$

$s_\gamma = 1 - 0.4 \frac{B}{L} = 0.9$

Depth: $d_q = 1.15$ $K = 1$

$d_\gamma = 1$

Inclination: $i_q = \left[1 - \frac{Hi}{V + \cdots}\right)^m$; $m = m_L = \frac{2 + L/B}{1 + L/B}$

$\therefore 0.585$ $= 1.2$

$i_\gamma = 0.374$

$\therefore q_{ult} = 913$ kPa

ϕ value from triaxial test:

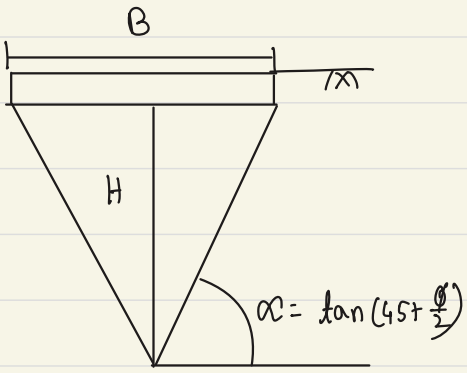
a) Allows precise control of drainage condition & pore water pressure
b) varied containing pressure stress cond.
c) most reliable soil stren param

Effect of WT on BC of soil

water seldom above base ($\bar{q}$ is adjusted)	For terms $\bar{q}$
$\gamma = d_w \gamma_{wet} + d_w \gamma'$	$0.5 \gamma' B N_\gamma$
water is at surface	$\gamma' = \gamma_{sat} - \gamma_w$
$\gamma' = 1/2 \gamma_{sat}$	
water below wedge → neglect	

WT within wedge zone:

$$\gamma_e = (2H - d_w) \frac{d_w \gamma_{wet}}{H^2} + \frac{(H - d_w)^2}{H^2} \gamma'$$



$$H = 0.5 B \tan(45 + \frac{\phi}{2})$$

$$\gamma_{dry} = \frac{\gamma_{wet}}{1 + w}$$

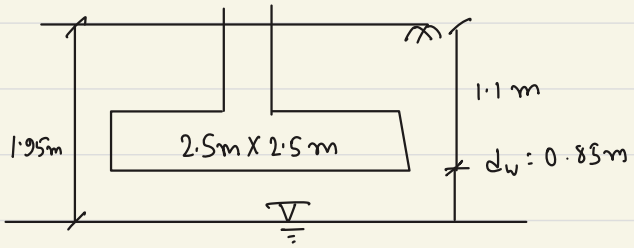
$$V_s = \frac{\gamma_{dry}}{G_s \gamma_w}$$

$$V_v = 1 - V_s$$

$$\gamma_{sat} = \gamma_{dry} + V_v (\gamma_w)$$

Hansen eqn:

$$q_{ult} = \bar{q} N_q s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma$$



ϕ: $L/B = 1$; $\phi = \phi_{tri} = 35°$

N: $N_q = 33.6$; $N_\gamma = 34.4$

S: $s_q = 1.57$; $s_\gamma = 0.6$

D: $d_q = 1.11$; $d_\gamma = 1.10$

$\gamma_{wet} = 18.10$ kN/m^3

$c = 0$

$\phi_{tri} = 35°$

$G_s = 2.68$

$w_N = w = 10\%$

$H = 0.5 B \tan(45° + \phi/2) = 2.4$ m

$$\gamma_e = (2H - d_w)\frac{d_w \gamma_{wet}}{H^2} + \frac{(H - d_w)^2}{H^2}\gamma'$$

$= 14.85$ kN/m^3

$\therefore q_{ult} = 1587$ ...

$\gamma' = \gamma_{sat} - \gamma_w = 10.3$

$\gamma_{sat} = \gamma_{dry} + V_v \times \gamma_w = 20.11$

$\gamma_{dry} = \frac{\gamma_{wet}}{1+w} = 16.45$ kN/m^3

$V_v = 1 - V_s = 0.374$

$V_s = \frac{\gamma_{wet}}{G_s(\gamma_w)} = 0.626$

Estimating BC from SPT

25 mm settlement

$$q_a = \frac{N_{55}}{F_2}\left(\frac{B+F_3}{B}\right)^2 \times k_d$$

$B > F_4$; $k_d = 1 + 0.33\frac{D}{B} \leq 1.33$

F_1	0.05
F_2	0.08
F_3	0.3
F_4	1.2

$N_{70} = 24$; $D = 1m$; $B = 3\ m$

$N_{55} = \frac{70}{55} \times 24 \approx 30$; $B > (F_4 = 1.2)$

$k_d = 1 + 0.33\frac{1}{3} = 1.11$

$\therefore q_a = \frac{N_{55}}{F_2}\left(\frac{B+F_3}{B}\right)^2 k_d = \frac{30}{0.08}\left(\frac{3+0.3}{3}\right)^2 \times 1.11$

$= 503\ kPa \longrightarrow 500\ kPa$

4-8

assume

$N_{70} = 6$; $N_{55} = \frac{70}{55} \times 6 \approx 8$ ← 9 for matching

For 25 mm: $q_a = \frac{N_{55}}{F_2}\left[\frac{B+F_3}{B}\right]^2 K_d$; $K_d = 1 + 0.33\frac{D}{B}$

$B - 1.5\,m \rightarrow K_d = 1.33 \rightarrow q_a = 215$ kPa

$B - 3.5\,m \rightarrow K_d = 1.18 \rightarrow q_a = 160$ kPa

$q_a \rightarrow 200$ kPa

For 90 mm: $q_a = \frac{200}{25} \times 40 = 320$ kPa

Settlement:

Defⁿ: Accumulation of deformation of soil due to load application in the dir of interest.

Principal components of soil settlement:

a. Particles rolling & sliding
b. Crushing
c. Elastic deformation

Problems with settlement analysis:

a. Obtaining reliable values of elastic param

→ Probs w/ recovering undisturbed sample
→ Lab test err: > 50%
→ Anisotropy
→ Correlation use

b. Obtaining reliable stress profile of applied ld

Types of settlement

1. Immediate (within 7 days)
2. Consolidation (expulsion of water, takes years)

Estimation of soil settlement (immediate):

$$\Delta H = q_0 B' \frac{1-\mu^2}{E_s} m I_s I_f$$

$q_0 = P/A = P/L \times B$

B' = least dim (corner → B, center → B/2)

m = num of corners

$I_s = I_1 + \frac{1-2\mu}{1-\mu} I_2$

P1 $q_0 = 134$ kPa

$B \times L = 33.5 \times 39.5$ m

Soil is layered clay; with one sand seam from GS to sandstone bedrock at -14 m; mat at -3 m

E_s from 3 to 6 m = 42.5 MPa

6 to 14 m = 60 MPa

sandstone > 500 MPa

$$\Delta H = q_0 B' \frac{1-\mu^2}{E_s} m I_s I_f$$

Here: $q_0 = 134$ kPa

$B' = B/2 = \frac{33.5}{2} = 16.75$

$\mu = 0.35$ (estimated from table)

$$E_s = \frac{3 \times 42.5 + (14-6) \times 60}{3+14-6} = 55 \text{ MPa}$$

$m = 4$

$$I_s = I_1 + \frac{1-2\mu}{1-\mu} I_2 \quad ; \quad I_1 = 0.0815, \; I_2 = 0.0865$$

$= 0.121$

$I_f = 0.95$

$D/B = 0.09 \approx 0.1$

$m = \frac{L}{B} = 1.2$

$N = \frac{H}{B'} = \frac{14-3}{\frac{33.5}{2}} = 0.7$

$\therefore \Delta H = 0.0165 \text{ m} = 16.5 \text{ mm}$

Prob. 2 $N'_{70} = 20$ | $D = 1.5\,m$, $H = 8\,m$

P: 450 to 900 N

Rec. q_a for not over 25 mm

$$q_a = \frac{N_{55}}{F_2}\left[\frac{B+F_3}{B}\right]^2 k_p \; ; \quad k_p = 1 + 0.33\frac{D}{B}$$

B	k_p	q_a (kPa)
1.5	1.33	599
2	1.25	516
3	1.17	442

$$N_{55} = \frac{70}{55} \times 20 = 25$$

$\therefore q_a \approx 500$ kPa

$$q_{act.} = \frac{P}{A} = \frac{450}{1.5^2} = 200\ kPa \qquad q_{act} = \frac{900}{1.5^2} = 400\ kPa$$

$q_{act} < q_a$

Tentative $q_a = 250$ kPa

$$\therefore q_a = \frac{P}{A} \Rightarrow 250 = \frac{450+900}{B^2} \Rightarrow B = 1.64\,m$$

$$L = 1.64\,m$$

$$\Delta H = q_0 B' \frac{1-\mu^2}{E_s} m I_s I_F \qquad m = 4$$

$B' = \frac{B}{2} = 0.82\,m$; $q_0 = 250$ kPa

$\mu = 0.3$ (picked); $E_s = 500(N+15) = 20000$ kPa ✓ (25)

$= \ldots = 35000$

$= \ldots = 150000$

$$I_s = I_1 + \frac{1-2\mu}{1-\mu} I_2 \; ; \quad I_1 = 0.437$$

$$I_2 = 0.021$$

$$= 0.455$$

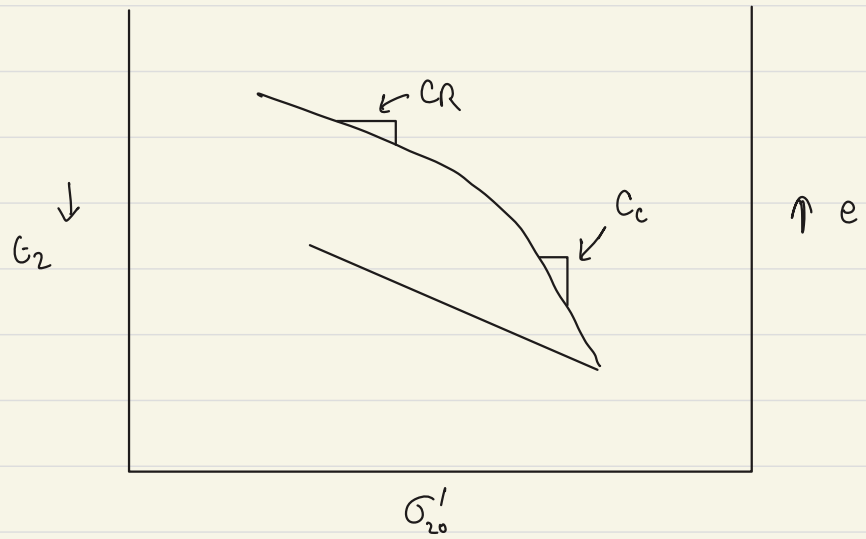
$$m = L/B = 1$$

$$N = H/B = 4.9 \rightarrow 5$$

$$I_f = 0.65$$

$$\therefore \Delta H = 0.011\ m = 11\ mm$$

Consolidation settlement:



Compute settlement

σ_{20}' = eff. init stress $\approx \sigma_c' \longrightarrow$ NC soil

$= \Sigma \gamma h - \Sigma \gamma_w h_w$

Formulate eqn: $\sigma_{2f}' = \sigma_{20}' + \sigma_{fill}$

$$\delta_c = \Sigma \frac{C_c}{1+e_0} H \log \frac{\sigma_{2f}'}{\sigma_{20}'} \leftarrow \text{for each layer at midpoint}$$

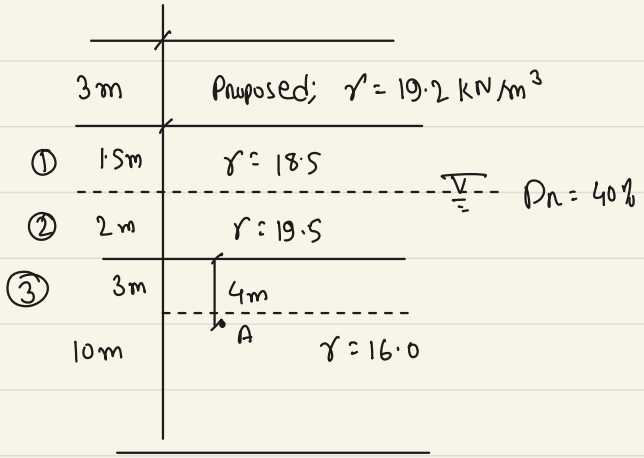
P2 Sample A:

$C_c = 0.4$

$C_r = 0.08$

$e_0 = 1.10$

$\sigma_c' = 70$ kPa



$\sigma_{20}' = \Sigma \gamma h - \Sigma \gamma_w h_w$

$= 18.5 \times 1.5 + (19.5 - 9.81) \times 2 + (16 - 9.81) \times 4$

$= 71.89$ kPa $\approx \sigma_c'$; NC soil

$\sigma_{2f}' = \sigma_{20}' + h_{fill} \gamma_{fill} = \sigma_{20}' + 57.6$

$\delta_c = \Sigma \frac{C_c}{1+e_0} H \log \frac{\sigma_{2f}'}{\sigma_{20}'}$

Layer 1: $H = 1.5$; mid = 0.75

$\sigma_{20}' = 18.5 \times 0.75 = 13.88$ kPa

$\sigma_{2f}' = 71.48$ kPa

$Dr = 40\%$

$\therefore \frac{C_c}{1+e_0} = 0.008$

$\therefore \delta_{21} = 0.008 \times 1.5 \log \frac{71.48}{13.88} = 0.0085$ m

$= 8.5$ mm

Layer 2: $H = 2$m, mid = 1m

$\sigma_{20}' = 18.5 \times 1.5 + (19.5 - 9.81) \times 1 = 37.44$

$\sigma_{2f}' = 95.04$

$\delta_{21} = 0.008 \times 2 \log \frac{95.04}{37.44} = 6.47$ mm

layer 3: $H = 3m$; mid = 1.5 m

$$\sigma_{20}' = 18.5 \times 1.5 + 19.5 \times 2 + 16.0 \times 1.5 - 9.81 \times 3.5$$
$$= 56.41$$
$$\sigma_{2f}' = 114$$

$$\delta = \frac{0.4}{1+1.1} \times 3 \times \log \frac{114}{56.41} = 175 \text{ mm}$$

layer 4: . . .

$$\Sigma \delta = \ldots$$