Correction for SPT-N values: P-1

Page 13-18
P.C. Varghese

1T Krawsir
CE-441

Correct the SPT N values for

(a) overburden pressure

(b) submergence in case of very fine silty sand

⇒ (a) All field SPT values after 1999 are corrected as proposed by Peck, Hanson and Thornburn which can be represented as follows:

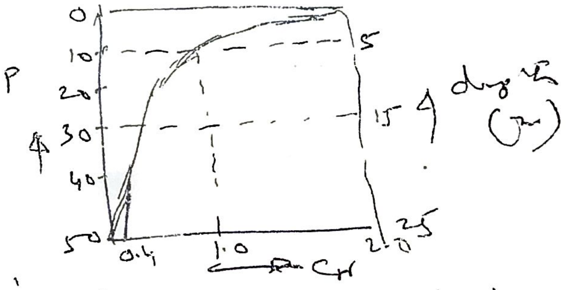
$$C_N = 0.77 \log_{10}(2000/p)$$

N_c (corrected) = $C_N \times N$; N → field SPT

p → overburden pressure in kPa

For p = 100 kN/m² or 1 kg/cm² (for overburden pressure at 5 m depth using $\gamma_{soil} = 20$ kN/m³), the correction factor N_c will be equal to $0.77 \log_{10} 20$ which is equal to unity.

Note that no correction is generally applied to position of water level for sands unless it is fine. (That is, use total overburden pressure to evaluate C_N; see Fig 1.9b, Page 15; Varghese)



(b) ⇒ Correction for silts & fine sands below water level:

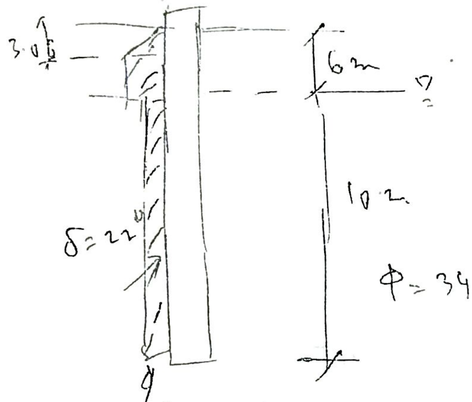
The correction for values of N greater than 15 in fine sands below water level is as follows:

$$N_c = 15 + 0.5(N - 15)$$

This correction is due to the fact that higher values are liable to be recorded due to pore pressure.

16.1
P-963
Bowels

A 460 mm dia. pipe pile is driven closed-end 15 m into a ~~clay~~ cohesionless soil with an estimated ϕ angle of 34°. The soil has a $\gamma_{wt} = 16.5$ kN/m³, $\gamma' = 8.60$ kN/m³. The GWT is 6 m below the ground surface. Estimate the ultimate pile capacity P_u using Terzaghi's method and friction angle $\delta =$



Pressure:

$K = 0.8 \sim 1.0$

~~$K_a = 0.33$~~

$q_f = [\bar{\sigma}_v K_a] \tan\delta \, A_f$

$\gamma_d = 16.5$ kN/m³ (assumed)

$A_p = \frac{\pi}{4}(0.46)^2 = 0.166$ m²

Critical depth:

for $\phi = 34°$; $z_c/d = 0.275 \times 34 - 2.7 = 6.65$

$\therefore z_c = 6.65 \times 0.46 = 3.06$ m

$\sigma_{zc} = \gamma z = 16.5 \times 3.06 = 51$ ~~48.0 kPa~~

$A_{f_1} = \pi(0.46)(3.06) = 4.42$

$A_{f_2} = \pi(0.46)(6 - 3.06) = 4.25$

$A_{f_3} = \pi(0.46)(10.0) =$ ~~57.8~~ $= 14.45$ m²

Critical Depth:

$y = z_c/d, \quad x = \phi°$

$\frac{y-5}{5-7.2} = \frac{x-28}{28-36}$

$\Rightarrow y = 0.275x - 2.7$

$\phi < 36°$

for $\phi > 36°$

$\frac{y-7.2}{7.2-15} = \frac{x-36}{36-40}$

$\Rightarrow y = 1.95x - 63$

End bearing: $q_b = q_o N_q$

for $\phi = 34°$, $N_q = 31$ ~~...~~ 30

$51.0 \times 30 =$ ~~48.0 × 65~~ $= 1530$

$Q_b =$ ~~3120 A_p = 3120 × 0.166~~

$=$ ~~586 kN~~ ~~518 kN~~

$= 1530 \times 0.166 = 254$ kN

Frictional Component:

Part-1: $K = 0.8$ $(= 0.8 \sim$

$Q_{f_1} = \frac{1}{2}(51 \times 0.8 \tan 22°)$ ~~(48 × 0.33 tan 22°)~~ $\times 4.42$

$= 36.4$ kN

~~$= \frac{1.28}{2}$ kN = 14 kN~~

$Q_{f_2} = (51 \times 0.8$ ~~48 × 0.33~~ $\tan 22°) \times \pi \times (0.46)(6-3)$ ~~π(0.46)×4~~

$=$ ~~27.2 kN~~

$= 310$ kN

~~$Q_{f_3} = [48 - 9.806 \times (6 - 3.06)] \times 0.33 \tan 22° \times 14.45 = 37$ kN~~

$\therefore (Q_{ult})_1 = 254 + 36 + 310 = 599$ kN

~~518 + 14 + 27.2 + 37~~ = ~~596~~ kN

$Q_{ult} = (Q_{ult})_1 - W_{s-1} = 599 - 60 = \ldots$ kN

Ex. 16-3
8-940
Braja

Estimate the ultimate pile capacity of a 300-mm round concrete pile that is 30 m long with 24 m driven into a soft clay soil of average $q_u = 24$ kPa. Assume $\gamma' = 8.15$ kN/m³ for the soil. The water surface is 2 m above the ground line.

$A_p = \frac{\pi}{4}(0.3)^2 = 0.071\ m^2$

$A_f = \pi(0.3)(24) = 22.62\ m^2$

P-3

Point Bearing: $Q_b = c N_c A_p = \frac{q_u}{2} N_c A_p$

$= \left(\frac{24}{2}\right)(6.2)(0.071) = 5.3$ kN

since friction pile, better not to use Df/B factor

Friction pile: $Q_f = (\alpha_a c) A_f$

$= \left(0.9 \times \frac{24}{2}\right) \times 22.62 = 244.3$ k

$\alpha_a = 0.9$ (since the soil is soft)

$\therefore Q_{ult} = 5.3 + 244.3 = 250$ kN

Self wt. of the pile: $W_p = [0.071 \times 30] \times 24.0$ kN

$\gamma_{conc} = 24$ kN/m³ $= 51$ kN

$\therefore Q_{net} = Q_{ult} - W_p = 250 - 50 = 200$ kN

Ans.

Tension piles:

- used beneath bldgs to resist uplift from hydrostatic pressure.
- to support structures over expansive soils
- Over-turning caused by wind, ice loads and broken wires may produce large tension forces for a power transmission towers. In this type of structure the piles & piers supporting the tower legs must be designed for both comp. & tension forces.

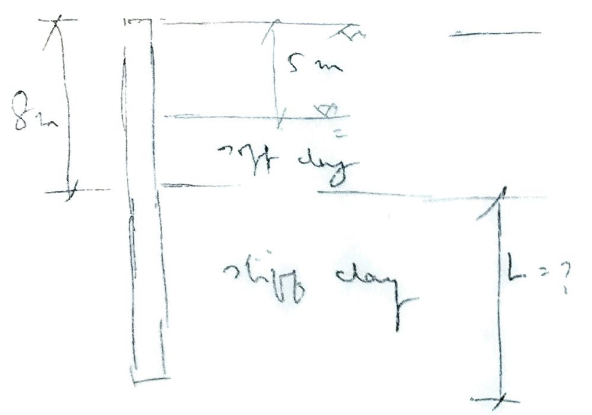
16-3
8-963
Growdo.

A pile is driven through a soft cohesive deposit overlying a stiff clay. The GWT is 5-m below the ground ~~water~~ surface and the stiff clay is at the 8-m depth. Other data:

	Soft clay	Stiff clay	
γ_{wet}	17.5	19.3	kN/m³
γ'	9.5	10.6	kN/m³
S_u	50	165	kPa

P-4

Estimate the length of a 550-mm dia. pile to carry an allowable load $P_a = 420$ kN using an $SF = 4.0$ & Terzaghi's method.



Soln:

$A_p = \frac{\pi}{4}(0.55)^2 = 0.238\ m^2$

$(A_f)_1 = \pi(0.55) \times 8 = 13.8\ m^2$

$(A_f)_2 = \pi(0.55) \times L = 1.73L$

Assume: End point is extended into the stiff layer.

End bearing: $q_p = [cN_c A_p]$

$c = S_u$

$= [165][6.2][0.238] = 243$ kN

Skin friction:

Part 1: $[q_f]_1 = [\alpha_a c_1][A_{f_1}]$

$\alpha_a = 0.85$ (soft-to-medium clay)

$= [0.85 \times 50][13.8] = 586.5$ kN

Part 2: $[q_f]_2 = [\alpha_a c_2][A_{f_2}]$

$\alpha_a = 0.5$ (stiff clay)

$= [0.5 \times 165][1.73 \times L]$

$= 142.7L$

$q_{ult} = q_p + [q_f]_1 + [q_f]_2 = 243 + 586.5 + 142.7L$

$\therefore q_{allow} = \frac{q_{ult}}{FS} = \frac{830 + 142.7L}{4} = 207.4 + 35.7$

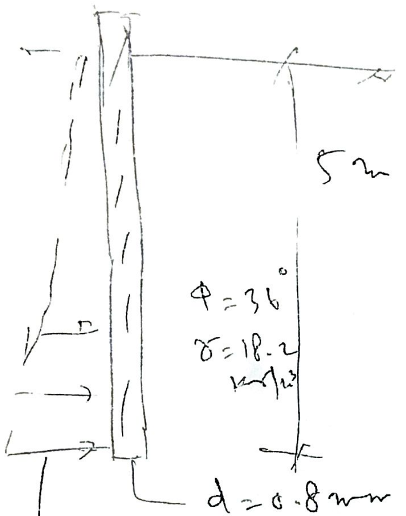
or, $q_{allow} = 420 = 207.4 + 35.7L \Rightarrow L = 6$ m

$\therefore$ Total length of pile $= 8 + L = 8 + 6 = \underline{14\ m}$ Ans

16-8
P-964
Bowels.

What is the approximate ultimate pullout resistance T_u for a tension pile in a medium dense sand with $\phi = 36^\circ$, $\gamma = 18.2$ kN/m³, and using an 800-mm dia. Concrete pile with a length of 5 m (and no bell)?

P-5



$\delta = \frac{2}{3}\phi = \underline{24^\circ}$

for $\phi = 36^\circ$,

$z_c/d = 0.275\phi - 2.7$
$= 0.275 \times 36 - 2.7$
$= 7.2$

$\therefore z_c = 7.2 \times d = 7.2 \times 0.8$
$= 5.76\ m > 5\ m$

so the entire pile will be subjected to lateral earth pressure with hydrostatic variation.

K = 0.8 ~ 1.0 (should be more than what occurs for at-rest cond^n)

$\sigma_v K_o$
$= \gamma z K_o$
$= 18.2 \times 5 \times 0.8$
$= \underline{72.8\ kPa}$

$(T_u)_{soil} = [\sigma_v K_o \tan\delta]\ A_f$
$= [\cancel{80}\ 73 \tan 24^\circ][\pi(0.8)(5)]$
$= \cancel{168\ kN}\ \underline{408\ kN}$

$\therefore T_u = (T_u)_{soil} + W_{pile}$
$= 408 + [\frac{\pi}{4}(0.8)^2 \times 5 \times 24]$
$= 408 + 60 = \cancel{???\ kN}$
$= \underline{468\ kN}$

$\gamma_{concrete} = 24$ kN/m³

K → lateral pressure coefficient to evaluate f_f for both sand & clay:

Use K = 0.8 (both tension & compr) for low-volume displacement p
= 1.0 for displacement piles

(usually K > K₀ (= 1 − sin φ))

use the same value of K for both sand & clay

P-6

For $\phi = 36°$

$z_c/d = 0.275\phi - 2.7$
$= 7.2$

$z_c = 7.2 \times 0.8 = 5.76 > 5$

So the entire length of the pile will be subjected to lateral pressure of soil of hydrostatic in nature

$\sigma_z = \gamma D_f = \gamma z_c = 18.2 \times 5 = 91$ kPa

$\sigma_h = K\sigma_z = 0.8 \times 91 = \underline{72.8 \text{ kPa}}$

$A_f = \pi(0.8)(5) = 12.57$ m²

Skin friction: $Q_f = \left[(\sigma_z K)\tan\delta\right] A_f$

Since a bell is at the tip, friction will occur between soil-to-soil (not pile-to-soil)

$\therefore \tan\delta = \tan\Phi = \tan 36°$

$Q_f = (72.8)[\tan 36°][12.57] = 665$ kN

Self weight

γ_c – 24 kN/m³ $\quad W_{pile} = \frac{\pi}{4}(0.8)^2(5)(24) = \underline{60 \text{ kN}}$

End Bearing: This component will be derived by the shaded area of the pile & soil

ϕ = of the surrounding of the pile tip (T.B. – given) $= 36°$

$N_q = 38$; for compacted fill

$Q_b = [q\,N_q]\,A_b$
$= 91 \times 38 \times \frac{\pi}{4}(1.25)^2$

$Q_{ult} = Q_b + Q_f + W_{pile}$

Group Capacity of piles

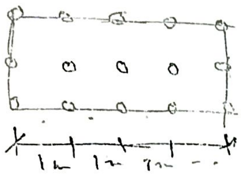
Ex 13.2
Varghese
P-276

A group of friction piles in clay consists of 15 piles of 500 mm dia. (grouped as 5×3) spaced at 1 m apart. If undrained shear strength c of the clay is 0.3 kg/cm² and the piles are 20 m in length estimate the group capacity and its efficiency. (Note: the pile cap will extend 25 cm around the piles and the pile cap will touch the soil but as the soil is clay, the strength of pile cap resting on the ground need not be considered.)

Soln:

step-1: Capacity of each pile and that of the group;

Individual:



End bearing $Q_b = A_b\, c\, N_c = \left[\frac{\pi}{4}(0.5)^2\right] \times (0.3 \times 100)$

$c = 0.3\ kg/cm^2 = 30\ kN/m^2$

$= 0.13 \times 30 \times 9 = 0.13 \times 270 = 35\ kN$

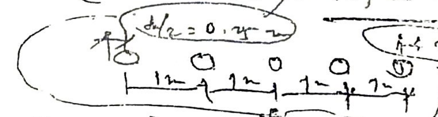
$Q_f = \alpha \times c \times \Sigma A_f$

$\alpha = 0.9$ (soft clay): $= [0.9 \times 30] \times [\pi \times 0.5 \times 20]$

$= 27 \times 25 = 675\ kN$

$\therefore Q_u = n_p (Q_b + Q_f) = 15 \times (35 + 675) = 10,650$

Step-2: Capacity a group:



Group perimeter:

L: 5 piles @ 1 m apart $= (4 \times 1) + \frac{0.5}{2} \times 2 = 4.5$

B: 3 piles @ 1 m apart $= (2 \times 1) + 0.5 = 2.5\ m$

Perimeter $= [4.5 + 2.5] \times 2 = 14\ m$

Projected area by perimeter for end bearing $= 4.5 \times 2.5 = 11.25$

Depth ratio $= D/B = 20\ m / 2.5\ m = 8$ — 2.5 m

N_c from Skempton curve (P-11 B, Fig 6.4, Varghese)

for $D/B > 2.5$, $N_c = 9.0$

$\therefore Q_b = A_b\, c\, N_c = [11.25][30][9] = 3038\ kN$

$Q_f = \alpha\, c\, \Sigma A_f = [1.0] \times [30]\,[14 \times 20] = 8400\ kN$

where $\alpha = 1.0$ [soil-to-soil friction]

$\therefore Q_u = 3038 + 8400 = 11,438\ kN$

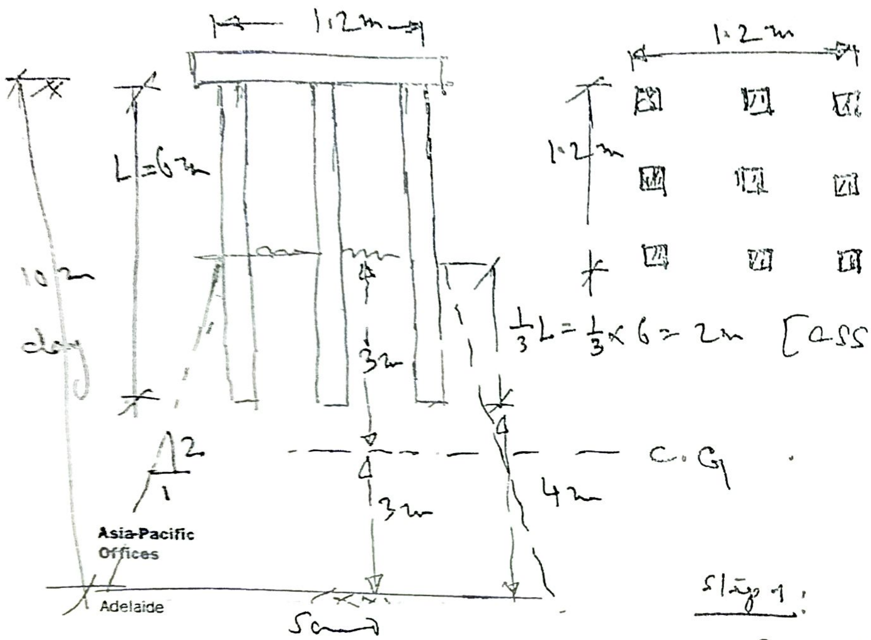
$\therefore$ Lesser of the above two Q_u is the group capacity;

i.e., $Q_u = 10,650\ kN$.

Settlement

P-8

A group of nine piles of 200 mm dia spaced at 0.5 m transfer a load of 400 kN into a 10 m thick clay layer with sand below. It penetrates to a depth of 6 m in the clay layer. If the clay is normally consolidated with a LL = 40%, estimate the probable settlement of the pile group. Assume saturated water content is 39%; $G_s = 2.7$ and the water level is at GL. Let $\gamma = 20$ kN/m³.



$\frac{1}{3}L = \frac{1}{3}\times 6 = 2$m [assume raft level for analysis]

Step 1: Detn characteristics of soil

$C_c = 0.009\,(LL - 10)$
$= 0.009\,(40 - 10) = 0.27$

$e_0 = \frac{w G_s}{S_r}$ $\quad S_r = 100\% = 1$

$= 0.39 \times 2.7 = 1.05$

(b) Calculate 2/3 depth xx, the assume level of raft where Δp is released from the top of pile

$\frac{2}{3}L = \frac{2\times 6}{3} = 4$ m from top of pile

thickness of compressible layer for this pile arrangement, H = 10 − 4 = 6 m

mid depth $H/2 = 6/2 = 3$m ; i.e., 1 m below the pile tip.

(c) Enlarge area at C.G.

$= \left[\frac{3}{2} + 1.2 + \frac{3}{2}\right]^2 = (4.2)^2 = 17.64$ m²

$$\cancel{\Delta P = \frac{400 \times (1.2)^2}{(4+2)^2}}$$

$$\Delta P = \frac{400}{(4.2)^2}$$

$=$

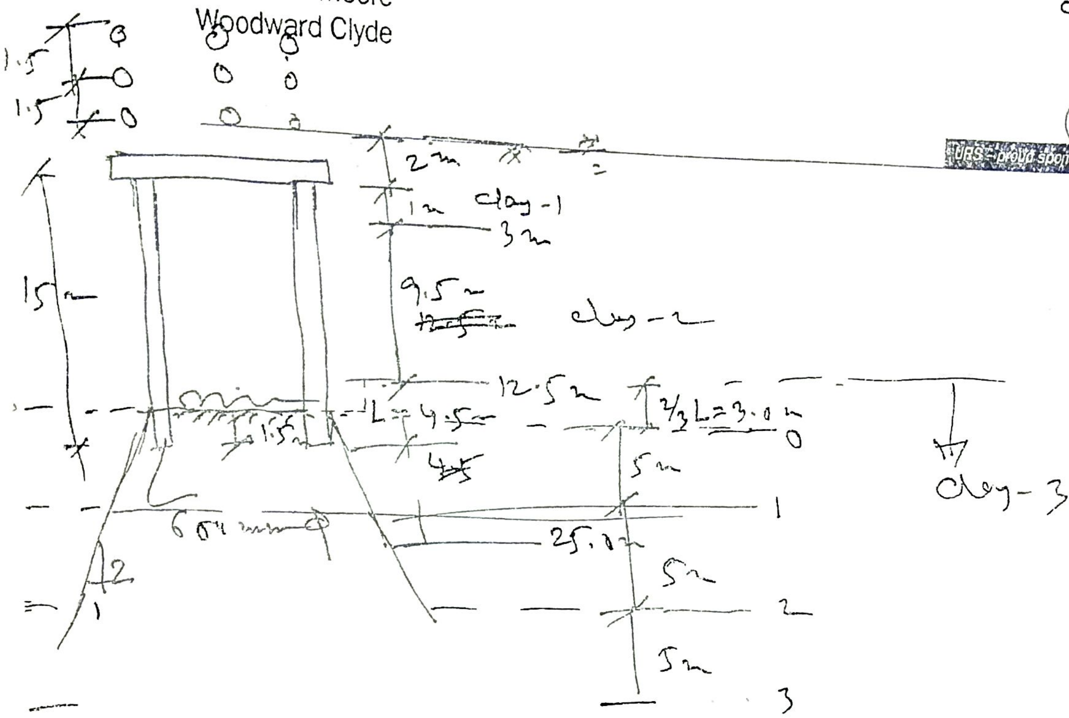
as given in 1st [illegible] kPa, [illegible] convert [illegible] kN by multiplying [illegible] with [illegible] [illegible] with [illegible]

$$P_0 = (20 - 10) \times \left(\underset{\text{(initial}}{4\,m} + \underset{\text{mid depth}}{\tfrac{3}{2}\,m}\right) = 70\ \text{kN/m}^2$$

(assuming submerged)

Class test on CE 343

P-10



load Transfer level at 0-0, $\frac{2}{3}L$ (L = transferrable clay layer = clay-3)

$= \frac{2}{3} \times 4.5 = 3.0$ m

i.e., $4.5 - 3.0 = 1.5$ m up from top of pile

Initial stress at 0-0 (before structural load comes up);

(i) for 3.0 m of clay-1 $= 19.5 \times 3 = 58.5$ kPa

(ii) for 9.5 m of clay-2 $= 19.5 \times 9.5 = 185.25$ kPa

(iii) for 3.0 m of clay-3 $= 20.0 \times 3 = 60.0$ kPa

(iv) Less $3 + 9.5 + 3 = 15.5$ m water $= 9.8 \times 15.5 = 152$ kPa

Effective P_0' at level 0-0 $= 58.5 + 185.5 + 60 - 152 = 152$ kPa

Effective P_0' at level 1-1 (5m below 0-0) $= 152 + [5 \times 20 - 5 \times 9.8] = 203$ kPa

Effective P_0' at level 2-2 (5m below 1-1) $= 203 + [5 \times 20 - 5 \times 9.8] = 203 + 51 = 254$ kPa

Effective P_0' at level 3-3 (5m below 2-2) $= 254 + 51 = 305$ kPa

Step-2: ~~Consolidation~~ Increase of pressure assuming 2:1 distribution at various levels due to dead load component of structural load:

dead load $=$ ~~15000~~ 8000 kN

Δp at 0-0 level $= 8000 / \frac{\pi}{4}\left(1.5 + 1.5 + \frac{0.6}{2} \times 2\right)^2$ $= \frac{\pi}{4}(3.6)^2$

$= 786$ kPa

2:1 distribution; Δp at 1-1 level $= \dfrac{786 \times \frac{\pi}{4}(3.6)^2}{\frac{\pi}{4}\left(3.6 + \frac{5}{2} \times 2\right)^2} = 786 \times \left(\dfrac{3.6}{8.6}\right)^2 = 138$ kPa

Δp at 2-2 : $= \dfrac{786 \times (3.6)^2}{\left(3.6 + \frac{10}{2} \times 2\right)^2} = 55.0$ kPa $<$ 10% of Δp at 0-0

Δp at 3-3 : $= \dfrac{786 \times (3.6)^2}{\left(3.6 + \frac{15}{2} \times 2\right)^2} = 29.4$ kPa

③ p_0' and Δp at mid heights :

(i) Mid height of levels 0-0 & 1-1

$p_0' = \dfrac{152 + 203}{2} = 177.5$ kPa

$\Delta p = \dfrac{786 + 138}{2} = 462$ kPa

$\therefore p_0' + \Delta p = 177.5 + 462 = 640$ kPa

(ii) Mid height of levels 1-1 & 2-2

$p_0' = \dfrac{203 + 254}{2} = 229$ kPa

$\Delta p = \dfrac{138 + 55}{2} = 96.5$ kPa

$\therefore p_0' + \Delta p = 325.5$ kPa

(iii) Mid height of levels 2-2 & 3-3

$p_0' = \dfrac{254 + 305}{2} = 280$ kPa

$\Delta p = \dfrac{55.0 + 29.4}{2} = 42$ kPa

$\therefore p_0' + \Delta p = 280 + 42 = 322$ kPa

④ Find e_0, C_c etc :

Soil will compress in clay-3, the bearing layer :

$C_c = 0.009\,(LL - 10) = 0.009 \times (40 - 10) = 0.27$

$S_r = 1.0$ for 100% saturation

$e_0 = \dfrac{w\,G_s}{S_r} = \dfrac{\frac{25}{100} \times 2.71}{1.0} = 0.68$

⑤ Settlement Calculations :

$S_1 = \left\{ \dfrac{C_c}{1 + e_0} \log_{10} \dfrac{p_0' + \Delta p}{p_0'} \right\} \times H_1$

$\therefore S_1 = \boxed{\dfrac{0.27}{1 + 0.68} \log_{10} \dfrac{640}{177.5}} \times 5000 = 450$ mm

$= 0.16 \times 0.56 \times 5000$

Similarly, $S_2 = 0.16 \times \left\{ \log_{10} \dfrac{325}{229} \right\} \times 5000 = 0.16 \times 0.15 \times 5000 = 120$ mm

$S_3 = 0.16 \times \left\{ \log_{10} \dfrac{322}{280} \right\} \times 5000 = 0.16 \times 0.06 \times 5000 = 48$ mm

$S_t = \Sigma S_i = 450 + 120 + 48 = 618$ mm $= 61$ cm $=$ Ans