ACE Engineering Publications

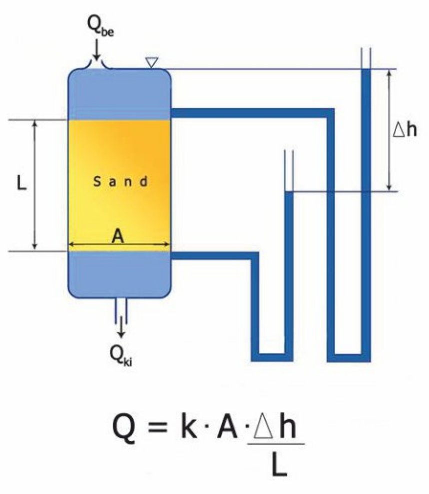
SINCE 2011

GATE | PSUs

CIVIL ENGINEERING

Geotechnical Engineering

(**Text Book** : Theory with worked out Examples and Practice Questions)



Geotechnical Engineering

(Solutions for Text Book Practice Questions)

01. Origin of Soil

09. Ans: (b) & (d)

Sol: **Loam:** It is mixture of sand, silt and clay, sometimes containing some organic matter such as humus.

Loess: It is a loose deposit of wind blown silts that has been weakly cemented with calcium carbonate and Montmorillonite.

Alluvial soil: Are soil that have been deposited from suspension in running water.

Peat: It is highly organic soil.

02. Definitions and Properties of Soil

01. Ans: (c)

Sol: Volume of solids in sample A:

Total volume $V = V_s + V_v$

Void ratio, $e = \dfrac{V_v}{V_s}$

$V_s = V_v$

Total volume $V = 1\ m^3$

$\therefore\ V_s = 1 - V_s$

$$V_s = \frac{1}{2} m^3$$

Volume of solids in sample B:

Total volume, $V = V_s + V_v$

$$1 = V_s + V_v$$

$$V_v = 1 - V_s$$

Void ratio, $e = \dfrac{V_v}{V_s}$

$$1.5\ V_s = V_v$$

$$1.5\ V_s = 1 - V_s$$

$$2.5\ V_s = 1$$

$$V_s = \frac{1}{2.5} = 0.4\,m^3$$

After compaction solids volume cannot change total volume after compaction

$V = 1\ m^3$

$V_s = 0.4 + 0.5 = 0.9\ m^3$

Porosity, $n = \dfrac{V_v}{V} = \dfrac{0.1}{1} = 0.1$

02. Ans: (a)

Sol: Water content of mixed sample $= \dfrac{W_w}{W_{solids}}$

Weight of solids cannot change weight of solids in sample of water content 50%

Water content, $w = \dfrac{W_{w_1}}{W_{solid_1}}$

$$0.5 = \frac{W_w}{W_{solids_1}}$$

$$0.5\,W_{solids_1} = W_w$$

Total weight of sample, W = 1 kg

$W_s + W_W = 1$ kg

$0.5\ W_s = 1 - W_s$

$$W_s = \frac{1}{1.5} = 0.667\,kg$$

Weight of solids in sample of water content 80%

$$w = \frac{W_w}{W_s} \Rightarrow 0.8\,W_s = W_w$$

$$0.8\ W_s = 1 - W_s$$

$$W_s = \frac{1}{1.8} = 0.556\,kg$$

$\therefore$ Total weight of mix = 2 kg

Solids weight of mix = 0.667 + 0.556

= 1.223

$\therefore$ water content of mix $= \frac{W_w}{W_s} = \frac{2-1.223}{1.223}$

= 63.6%

03. Ans: (d)

Sol: $\gamma = \gamma_d (1 + w)$ γ_d is constant

$\gamma \propto 1 + w$

$$\frac{\gamma_2}{\gamma_1} = \frac{1+w_2}{1+w_1} \Rightarrow \frac{\gamma_2}{1.8} = \frac{1.1}{1.05}$$

$\gamma_2 = 1.88$ gm/cc

Common data for Questions 04 & 05

04. Ans: (b)

Sol: In Borrow pit

$\gamma = 1.75$ g/cc

$w_1 = 12\%$

$G = 2.7$; $V_1 = ?$

After compaction

$w_2 = 18\%$

$\gamma_{d_2} = 1.65$ g/cc

$V_2 = 1000$ m^3

$$\frac{V_1}{V_2} = \frac{\gamma_{d_2}}{\gamma_{d_1}}$$

$$\gamma_{d_1} = \frac{\gamma}{1+w} = \frac{1.75}{1+0.12} = 1.56$$

$$\frac{V_1}{1000} = \frac{1.65}{1.56}$$

$\therefore V_1 = 1056$ m^3

05. Ans: (c)

Sol: Amount of water to be added

$= \gamma_{d_2} V[w_2 - w_1]$

$= 1.65 \times 1000\ [0.18 - 0.12]$

$= 99$ tons $\quad (\because 1\text{g/cc} = 1\text{t/m}^3)$

06. Ans: (c)

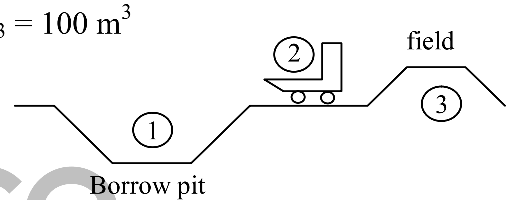
Sol: $\gamma_1 = 1.66$; $w_1 = 8\%$

$\gamma_2 = 1.15$; $w_2 = 6\%$

$\gamma_3 = 1.82$

$w_3 = 14\%$

$v_3 = 100$ m^3



$$\frac{V_1}{V_3} = \frac{1+e_1}{1+e_3} = \frac{\gamma_{d_3}}{\gamma_{d_1}}$$

$$\gamma_{d_1} = \frac{\gamma_1}{1+w_1} = \frac{1.66}{1+0.08} = 1.537$$

$$\gamma_{d_2} = \frac{1.15}{1+0.06} = 1.084$$

$$\gamma_{d_3} = \frac{1.82}{1+0.14} = 1.59$$

$$\frac{V_1}{V_3} = \frac{\gamma_{d_3}}{\gamma_{d_1}}$$

$$\frac{V_1}{100} = \frac{1.59}{1.54}$$

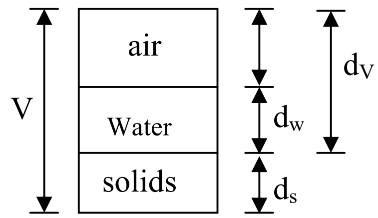
$V_1 = 104.3$ m^3

$$V_2 = \frac{1.596}{1.084} \times 100 = 147.2\,\text{m}^3$$

No. of truck load $= \frac{147.2}{6} = 24.5 = 25$ nos.

07. Ans: (c)

Sol:



$e = 0.51$

$S_r = 80\%$

$d_W = 1$ m

$$S_r = \frac{d_w}{d_v} = \frac{1}{d_v} = 0.80$$

$\therefore d_v = 1.25$

$$e = \frac{d_v}{d_s} = \frac{1.25}{d_s} = 0.5$$

$d_s = 2.5$

$\therefore$ Total $d = d_s + d_v = 2.5 + 1.25$

$= 3.75$ m

Common data for Questions 08 & 09

08. Ans: 2.07

Sol: Volume of cube = $5^3 = 125\ cm^3$

$W_d = 135$ g ; $W = 195$ g

$$\text{water content} = \frac{W - W_d}{W_d} \times 100$$

$e \times S_r = w\,G$

$\therefore e = 0.44\,G$

$$\frac{\gamma_{sat}}{\gamma_w} = \frac{G + e}{1 + e}$$

$$\gamma_{sat} = \frac{\text{Saturated wt of cube}}{\text{volume of cube}}$$

$$= \frac{195}{5^3} = \frac{195}{125}\ g/cc$$

$$\therefore \quad \frac{195}{125} = \frac{G + 0.44\,G}{1 + 0.44\,G}$$

$\therefore G = 2.07$

09. Ans: 0.92

Sol: $e \times S_r = 0.44\,G$

$e = 0.44\,G$

$e = 0.44 \times 2.07$

$\therefore e = 0.92$

10. Ans: 11.87%

Sol: To find initial porosity

$$\gamma_d = \frac{\gamma_w . G}{1 + e}$$

$$16 = \frac{9.80 \times 2.67}{1 + e}$$

$e = 0.637$

$$n_1 = \frac{e}{1+e} = \frac{0.637}{1 + 0.637} = 0.388 \approx 38.8\%$$

$$e = \frac{V_v}{V_s} = 0.637$$

$$n = \frac{V_v}{V} \Rightarrow 0.388 = \frac{V_v}{1} \Rightarrow V_v = 0.388$$

Coarse sand

$V_s = V - V_v = 1 - 0.388 = 0.611\ m^3$

Dry silty soil:

$\gamma_s = G.\ \gamma_W$

$= 2.67 \times 9.80 = 26.16\ kN/m^3$

$$V_s = \frac{W_s}{\gamma_s} = \frac{5.5}{26.16} = 0.21\ m^3$$

Wet Clay:

$$\text{Water content, } w = \frac{W_w}{W_s}$$

$$0.1 = \frac{W - W_s}{W_s}$$

$W_s = 1.3636$ kN

$$V_s = \frac{1.3636}{2.55 \times 9.8} = 0.0545\ m^3$$

After compaction:

Total volume, $V = 1.2\ m^3$

$V_s = 0.611 + 0.21 + 0.0545 = 0.875\ m^3$

$V_s + V_v = V$

$V_v = V - V_s$

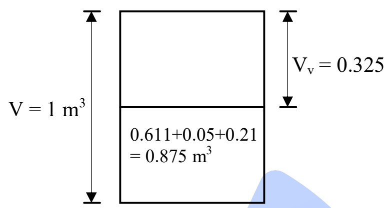
$= 1.2 - 0.875$

$= 0.325$

Final porosity, $n_2 = \frac{0.325}{1.2} \times 100 = 27\%$

Reduction in porosity $= 38.8\% - 27\%$

$= 11.8\ \%$



11. Ans: (b) & (c)

Sol: Void ratio $(e) = \frac{V_v}{V_s}$

If $V_v > V_s$

Then void ratio can be greater than 1 and void ratio can be less than 1 to 0 but not zero.

Porosity $(n) = \frac{V_v}{V}$

$V_v \ngtr V$

Hence n can not be greater than 1

% age air void $(n_a) = \frac{V_a}{V} \times \frac{V_v}{V_v}$

$n_a = a_c \times n$

$V_a \le V_v$

Hence n_a is always less than porosity.

04. Index Properties of Soil

01. Ans: (a)

Sol: At L.L $w_L = 60\%$,

$$e_1 = \frac{w_L.G}{S} = 0.6G$$

$w_s = 25\%$, $e_2 = 0.25\ G$

$$\frac{V_1}{V_2} = \frac{1+e_1}{1+e_2}$$

$$\frac{10}{6.5} = \frac{1+0.6G}{1+0.25G}$$

$G = 2.5$

Common data for Questions 02, 03

02. Ans: (c)

Sol: G_m = Mass specific gravity = 1.88

Water content, $w = 40\%$

On oven drying, mass specific gravity drops to = 1.74

G of clay =?

$$e = \frac{w_s.G}{S_r} = 0.40 \times G$$

$$\gamma_{sat} = \frac{\gamma_w(G+e)}{1+e}$$

$$1.88 = \frac{G+0.40G}{1+.4G}$$

$$1+0.4G = \frac{G(1+0.4)}{1.88}$$

$G = 2.90$

03. Ans: (a)

Sol: w_s =?

$e = 0.4 \times 2.90$

$e = 1.16 \quad \Rightarrow \; e = w_S\, G$

$$w_s = \left(\frac{1}{G_m} - \frac{1}{G}\right)\times 100$$

$$= \left(\frac{1}{1.74} - \frac{1}{2.90}\right)\times 100 = 22.98\%$$

$w_s = 23\%$

Common Data for Questions 04, 05 & 06

04. Ans: (b)

Sol: Initial weight of saturated soil,

$W_1 = 95.6$ gm

Initial volume of saturated soil,

$V_1 = 68.5$cc

Final dry weight = 43.5 gm = W_d

Final dry volume = 24.1 cc = V_d

w_s = ?

$$w_s = \left[\frac{W_1 - W_d}{W_d} - \left(\frac{v_1 - v_d}{W_d}\right)\gamma_w\right]\times 100$$

$$= \left[\frac{95.6 - 43.5}{43.5} - \left(\frac{68.5 - 24.1}{43.3}\right)\right]\times 100$$

$w_s = 17.7\%$

05. Ans: (c)

Sol: $$\gamma_d = \frac{W_d}{V_d} = \frac{43.5}{24.1} = 1.80\,\text{gm/cc}$$

$$G_m = \frac{\gamma_d}{\gamma_w} = \frac{1.80}{1} = 1.80$$

$$W_S = \left(\frac{1}{G_m} - \frac{1}{G}\right)\times 100$$

$$17.7 = \left(\frac{1}{1.80} - \frac{1}{G}\right)\times 100$$

$G = 2.65$

06. Ans: (c)

Sol: To find initial and final void ratio = ?

To find e_1:

$$\gamma_{sat} = \frac{w_1}{v_1} = \frac{95.6}{68.5} = 1.39\text{ g/cc}$$

$$1.39 = \frac{2.67 + e_1}{1 + e_1}$$

$e_1 = 3.28 \simeq 3.15$

To find e_2:

$e_2 = w_s G$

$= 0.17 \times 2.65$

$= 0.47$

07. Ans: (c)

Sol: Given:

$V_1 = 100$ cc,

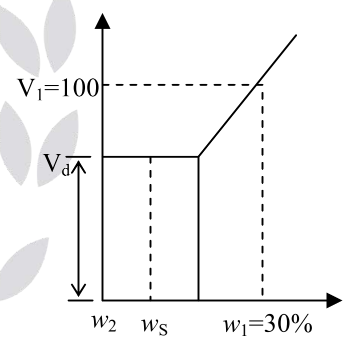
$w_1 = 30\%$

$w_s = 18\%$

$G = 2.72$

V_2 = ?

$w = 15\%$



Let e_1 be void ratio at water content of 30%

$$e_1 = \frac{w_1 G}{S_r} = \frac{0.30\times 2.72}{1} = 0.816$$

Let e_2 be void ratio, at w_s

$$e_2 = \frac{w_s G}{S_r} = \frac{0.18\times 2.72}{1} = 0.489$$

$$\frac{V_1}{V_2} = \frac{1+e_1}{1+e_2}$$

$$V_2 = \frac{100\times(1+0.489)}{1+0.816} = 82\text{ cc}$$

08. Ans: 21.63%. 347 kN and w = 25.24%

Sol: n = 36%

$$e = \frac{n}{1-n} = \frac{0.36}{1-0.36} = 0.563$$

G = 2.6

w_1 = 11%

Let w_2 be w.c @ full saturation

$$e = \frac{w_2.G}{s} = w_2 = 0.216 = 21.6\%$$

$$\gamma_d = \gamma_w\left(\frac{G}{1+e}\right)$$

$$= 9.81\left(\frac{2.6}{1+0.563}\right) = 16.31\ kN/m^3$$

To rise w.c w_1 to w_2

The weight of water to be added additionally

$= w_s(w_2 - w_1)$

$= \gamma_d.v(w_2 - w_1)$

$= 16.31 \times 200\ (0.216 - 0.11)$

$= 346$ kN

$$\frac{V_2}{V_1} = \frac{1+e_2}{1+e_1} \Rightarrow V_2 = 1.06V_1$$

$$\frac{1.06V_1}{V_1} = \frac{1+e_2}{1+e_1}$$

$e_2 = 0.657$

$$e_2 = \frac{w_3G}{s}$$

$$0.657 = \frac{w_3 \times 2.6}{1}$$

$w_3 = 0.254 = 25.4\%$

09. Ans: (a), (b) & (d)

Sol: Given,

$G_m = 1.70$

$G_s = 2.8$

$$G_m = \frac{\gamma_d}{\gamma_w} = 1.70$$

$$\gamma_d = \frac{G\gamma_w}{1+e}$$

$$\frac{\gamma_d}{\gamma_w} = \frac{G}{1+e}$$

$$1.7 = \frac{2.8}{1+e}$$

$$\Rightarrow 1 + e = \frac{2.8}{1.7} = 1.647$$

e = 0.647

$$\text{Porosity (n)} = 1 - n = \frac{1}{1+e}$$

$$1 - n = \frac{1}{1.647}$$

n = 0.39

Shrinkage limit (w_s)

es = wG

$0.647\ (1) = w_s \times 2.8$

$w_s = 0.2310$

$w_s = 23.10\%$

$$\text{Shrinkage ratio } (S_R) = \frac{\gamma_d}{\gamma_w} = 1.7$$

05. Soil Classification

01. Ans: (c)

Sol: $w_L = 60\%$

$w_P = 20\%$

I_P of soil $= w_L - w_P$

$= 60 - 20 = 40\%$

I_P of A line $= 0.73(w_L - 20\%)$

$= 0.73(60 - 20)$

$= 29.2$

As the soil lies above A line chart and its liquid limit is 60%

The given soil is CH.

02. Ans: GW - GM

Sol: $C_u = 18$, $C_c = 2$, $I_p = 6$

From question it is given as gravelly soil.

For GW, $C_u > 4$ and $C_c = 1 - 3$

$18 > 4$ and $C_c = 2$

$\therefore$ Soil is GW

But lines lies 5% and 12%, border line cases require dual symbol

For GM Atterberg limits fall below A line or $I_p < 4$

For GC Atterberg limits above A-line and $I_p > 7$

Here $I_p = 6$ for GC IP must be greater

$\therefore$ Soil is GW-GM

03. Ans: (GM)

Sol: Fine fraction = 45%

Coarse fraction = 100 – 45 = 55%

$\therefore$ Soil is coarse grained

% passing 4.75 mm since = 58% (out of total soil)

% retained or 4.75 mm since = 100 – 58%

= 42% (out of total)

Gravel + sand = 55%

% of Gravel = 42% (out of total soil)

$\therefore$ % retaining on 4.75 mm sieve out of coarse fraction

$$= \frac{42}{55} \times 100 = 76\%$$

(out of coarse fraction)

$\therefore$ it is gravel

$w_L = 40\%$, $w_p = 30\%$

A-line $= 0.73\,(w_L - 20)$

$= 0.73\,(40 - 20) = 14.6\%$

$I_p = 40 - 30 = 10\%$

Point plots below A-line silty gravel (GM)

04. Ans: (a) & (d)

Sol:

Soil	Size
Boulder	> 300 mm
Cobble	80 – 300 mm
Gravel	4.75 mm – 80 mm
Sand	75 μ – 4.75 mm
Silt	75 μ – 2 μ
Clay	< 2 μ

05. Ans: (b) & (d)

Sol: Classification of soil is done on the basis of grain size distribution and plasticity chart.

06. Permeability

01. Ans: (b)

Sol: temperature increases, γ_w decreases to 90% & μ decreases to 90%

$$\gamma_{w_2} = \frac{90}{100}\gamma_{w_1} \; ; \qquad \left[K \propto \frac{\gamma_w}{\mu}\right]$$

$$\mu_2 = \frac{75}{100}\mu_1$$

$$\frac{K_2}{K_1} = \frac{\gamma_{w_2}}{\gamma_{w_1}} \times \frac{\mu_1}{\mu_2}$$

$$\frac{K_2}{K_1} = \frac{90}{100} \times \frac{100}{75}$$

$K_2 = 1.2\,K_1$

$K_2 = 20\%$ (increases by 20%)

02. Ans: 1.35×10^{-4} m³/sec/m

Sol: $H = 7$ m, $H_1 = 2$ m, $h = 3$ m, $L = 40$ m

$$i = \frac{h}{L} = \frac{3}{L/\cos\alpha} = \frac{3}{40.6170} = 0.0738$$

$k = 0.09$ cm/sec $= 0.09 \times 10^{-2}$ m/sec

$$\frac{Q}{1m} = ki\frac{A}{1}$$

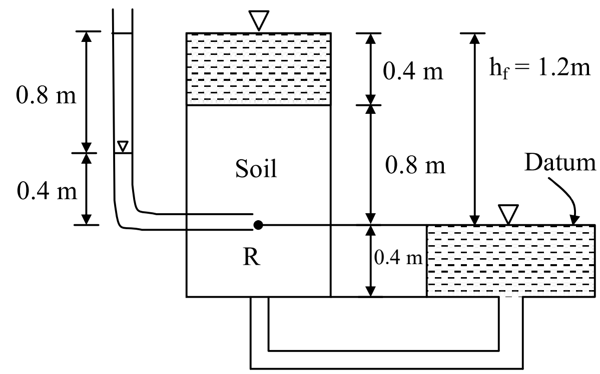
$$= \frac{0.09 \times 10^{-2} \times 0.0738 \times \left[\frac{2}{\cos\alpha} \times 1\right]}{1m}$$

$= 0.09 \times 10^{-2} \times 0.0738 \times 2.0308$

$= 1.35 \times 10^{-4}$ m³/sec/m

Common data for Questions.Q03 & Q04

03. Ans: (a) & (c)



$$i = \frac{h_f}{L} = \frac{1.2}{1.2} = 1$$

Loss of head for a seepage length of 0.8 m is $h_f = i \times L = 1 \times 0.8 = 0.8$ m

Pressure head at R is 0.4 m

Assuming datum at d/s water surface,

Elevation head at R is zero

i.e., Datum head = 0

Total head at R = $\dfrac{P}{\gamma_w} + Z$

$= 0.4 + 0 = 0.4$

If Datum head is chosen at bottom of soil, then Datum (or) Elevation head = 0.4 m

Pressure head = 0.4

Total head at R = 0.4 + 0.4 = 0.8

04. Ans: (a)

Sol: Discharge velocity, $V = k.\,i = k \times 1 = k$

Seepage velocity, $V_s = \dfrac{V}{n} = \dfrac{k}{0.50} = 2k$

05. Ans: 0.183 cm/sec and 0.094 cm/sec

Sol: Weight of water collected in 1 minute

= 6.18 N

Weight = volume × density

6.18 = volume × 9810

$$\text{Volume} = \frac{6.18}{9810} = 6.3\times10^{-4}\ \text{m}^3/\text{min}$$

Q = 1.05 × 10^{-5} m^3/sec

Q = kiA

$$1.05\times10^{-5} = k\times\frac{26}{20}\times\frac{\pi}{4}\times0.075^2$$

$$k_{at\,25°C} = 1.83\times10^{-3}\ \text{m/sec} = 0.183\ \text{cm/sec}$$

$$k \propto \left(\frac{e^3}{1+e}\right)\times\frac{1}{\mu}$$

At 25°C	**At 20°C**
K = 0.183 cm/sec	k_2 =?
n = 40%	n_2 = 35%
$e = \frac{n}{n-1}$	$e_2 = \frac{0.35}{0.65} = 0.5384$
$v_1 = 0.9v_{20°C}$	$v_{20°C}$ ∴

$$\frac{k_2}{k_1} = \left(\frac{e^3}{1+e}\right)_2\left(\frac{1+e}{e^3}\right)_1\times\frac{\mu_1}{\mu_2}$$

$$= \frac{0.5384^3}{1.5384}\times\frac{1.667}{0.667^3}\times0.9$$

k_2 = 0.094 cm/sec

06. Ans: (b) & (c)

Sol: Coarse grained soil is more permeable than fine grained soil.

Type of soil	Gravel	Sand	silt	Clay
Approx K (cm/sec)	10^o	10^{-2}	10^{-4}	10^{-6}

→ $K\alpha\frac{\gamma_w}{\mu}\alpha T$

As temperature increases, viscosity of liquid decreases, permeability increases.

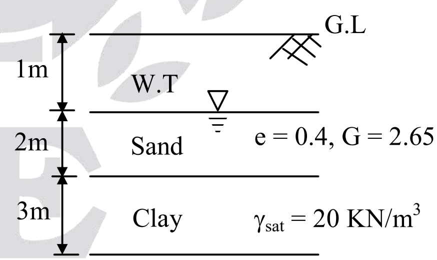
→ Laminar flow prevails in clays, silts and fine sand.

→ Pumping out test is more accurate than the constant head laboratory permeability test.

07. Effective Stress

01. Ans: (d)

Sol:



$$\gamma_{sat}\text{ of sand} = \frac{\gamma_w(G+e)}{1+e}$$

$$= \frac{10(2.65+0.4)}{1+0.4} = 21.785\,\text{kN/m}^3$$

γ_d of soil above water table = ?

$$\gamma_d = \frac{\gamma_w G}{1+e} = \frac{10\times2.65}{1+0.4} = 18.92\,\text{kN/m}^3$$

Effective stress below G.L =?

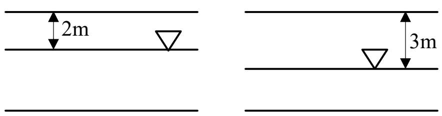
$\sigma' = \sigma - u$

$= (1\times18.92)+(2\times21.785)+(20\times3) - (5\times10)$

$= 72.49\ kN/m^2$

02. Ans: (d)

Sol:



Increase in effective stresses = final effective stress – initial effective stress

= change in effective stresses

$= (\gamma_d - \gamma_w)\ (3 - 2)$

$= (16 - 10)\ 1 = 6\ kPa$

03. Ans: (b)

Sol: σ' at 9m depth below G.L = ?

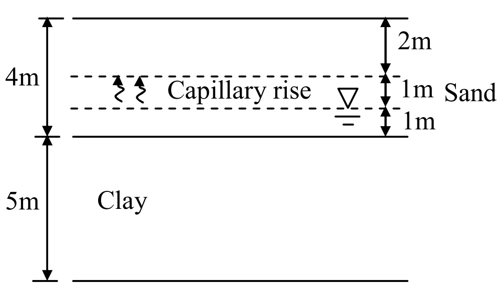
$\sigma' = \sigma - u$

$= (17\times3)+(20\times1)+(18\times5)-(6\times9.81)$

$\sigma' = 102.14\ kN/m^2$

04. Ans: (a)

Sol:



$\Delta\sigma'$ at 9 m depth of soil below G.L = ?

Effective stress after capillary rise at 9m = σ'

$\sigma' = \sigma - u$

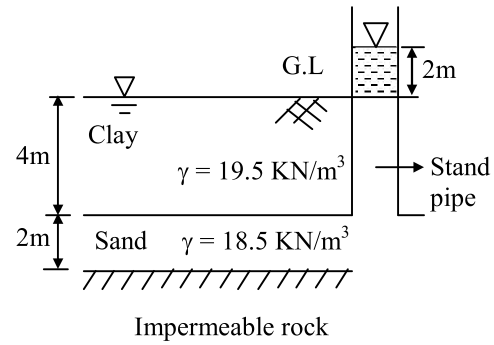
$= (2\times17)+(2\times20)+(18\times5)-6\times9.81$

$\sigma' = 105.14$

Increase in effective stress = 105.14 – 102.14

$\Delta\sigma' = 3\ kN/m^2$

Common Data for Questions Q 05 & Q 06



05. Ans: (d)

Sol: Effective stress at a depth of 6m =?

$\gamma_w = 10\ kN/m^2$

$\sigma' = \sigma - u$

$= (19.5)\times 4 + (18.5\times2) - (8\times10)$

$\sigma' = 35\ kN/m^2$

06. Ans: (a)

Sol: $\Delta\sigma'$ =? when artesian head in the stand is reduced by 1m

Total stress remains same.

Pore water pressure decreases by10 kN/m^2

$\therefore$ Effective stress increases by 10 kN/m^2

$\Delta\sigma' = 10\ kN/m^2$

07. Ans: (a), (b), (c) & (d)

Sol: Pore water pressure $(v) = \sigma - \overline{\sigma}$

Effective stress do not change at sudden, but by desiccation of upper lager by surface drying, change in water table, removal of any building and desiccation due to plant life, total stress decreases, hence pore water pressure decreases, and vice-versa.

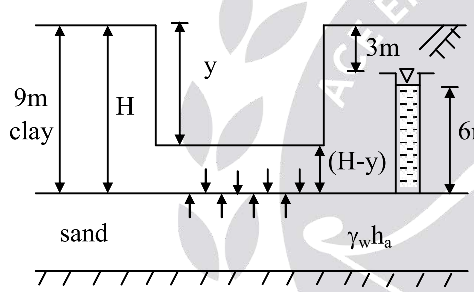
08. Ans: (a) & (c)

Sol: Soil particles get closed and become more compact as effective stress increases. As effective stress increases, compressibility of soil decreases.

08. Seepage Pressure and Critical Hydraulic Gradient

01. Ans: (c)

Sol:



To find depth of safe excavation:

$$\gamma = 20\ \text{kN-m}^3$$

Downward pressure = uplift pressure

$$\gamma\,(H - y) = \gamma_w\, h_a$$

$$20\,(9 - y) = 10 \times 6$$

$$y = 6\ \text{m}$$

$\therefore$ Maximum depth of trench will be excavated without failure is 6 m.

02. Ans: (c)

Sol: $20\,(9 - 7) = \gamma_w\, h_a$

$$h_a = \frac{20 \times 2}{10} = 4\ \text{m}$$

Water table to be lowered = 6 – 4 = 2m

03. Ans: 35

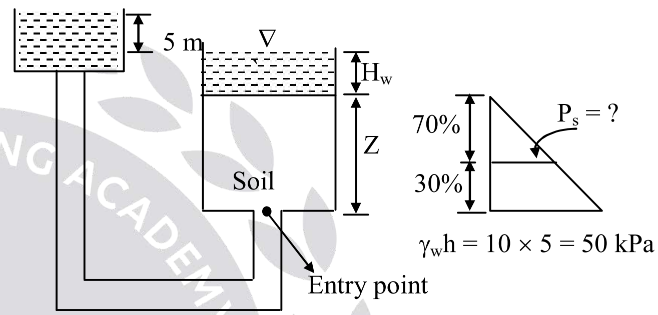
Sol: Given

Net head causing flow h = 5 m

Hydraulic gradient $i = \dfrac{h}{Z} = \dfrac{5}{Z} = 1$

$$\Rightarrow \quad Z = 5\ \text{m}$$

Seepage length Z = 5 m



For 100% $\rightarrow$ 50 kPa

For 70% $\rightarrow \dfrac{50}{100} \times 70 = 35\text{kPa}$

04. Ans: 1.91 m

Sol: Depth of soil layer = 1200 mm

FOS against piping $= \dfrac{i_c}{i}$

$$z = \frac{(G-1)(1-n)}{i}$$

$$i = \frac{1.65 \times 0.7}{2}$$

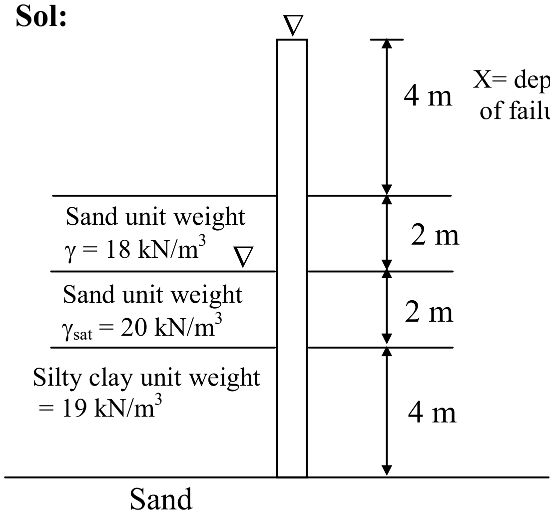
$$\frac{h}{z} = 0.5775\ ; \quad z = 3.1168\ \text{m}$$

Additional depth of coarse sand

= 3.1168 – 1.2 = 1.9168 m

05. Ans: 32 kPa & 7.2m

Sol:



I. At bottom of silty clay

Total stress

$\sigma = 18 \times 2 + 20 \times 2 + 4 \times 19$

$= 152$ kPa

Pore water pressure

$U = (4 + 2 + 2 + 4) \times 10 = 120$ kPa

Effective stress $\sigma' = \sigma - u$

$= 152 - 120 = 32$ kPa

II. At x m of water failure occur. That is effective stress = 0

Total stress $\sigma = 152$ kPa

Pore water pressure $u = 10\,(x + 2 + 2 + 4)$

$\sigma' = 0$

$\therefore \sigma - u = 0$

$\Rightarrow 152 = 10\,[x + 2 + 2 + 4]$

$\therefore x = 7.2$ m

06. Ans: 2.2 & 10.38 kN/m³

Sol: $V_s = \dfrac{V}{n}$

$V = V_s\,.n = 2.4 \times 10^{-4}$ cm/sec

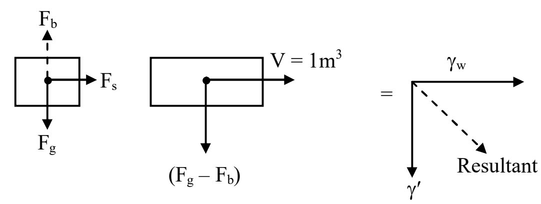
$V = k.i$

$$V = k\left(\frac{h_A - h_B}{L}\right)$$

$$2.4\times10^{-4} = 1.2\times10^{-3}\left(\frac{5-h_B}{14}\right)$$

$h_B = 2.2$ m

$i = 0.2$



$$\text{Resultant} = \sqrt{\gamma'^2 + (i\gamma_w)^2} = 10.377\,\text{kN}$$

07. Ans: (d)

Sol: Total stress at point A

$= 0.7 \times 9.81 + 20.6 \times 1$

$= 27.467$ KPa

Neutral stress at point A = $\gamma_w h$

h = total head at point A

$h = 1 + 0.7 + 0.75 = 2.45$

$\therefore$ Effective stresses at point A, $\sigma' = \sigma - u$

$= 27.467 - 9.81 \times 2.45$

$= 3.4325$ kPa

08. Ans: (c)

Sol: Upward seepage force per unit volume

$$= \frac{\gamma_w h \times A}{\text{volume}} = \frac{\gamma_w h \times A}{A \times z} = \gamma_w i$$

$= 9.81 \times 0.75$

$= 7.3575$ kN/m³

09. Ans: (a) & (d)

Sol: If seepage flow is upward then effective stress will get reduced by seepage force, and vice-versa.

09. Seepage Analysis

01. Ans: 0.0086

Sol: The quantity of flow into the pond per m^2 area

$$Q = ki$$

$$i = \frac{h}{z} = \frac{\text{head loss}}{\text{depth of clay}} = \frac{5m}{5m} = 1$$

$\therefore$ $Q = 10^{-5} \times 10^{-2} \times 1 = 10^{-7}$ m^3/sec

$= 10^{-7} \times 3600 \times 24$ m^3/day

$= 0.0086$ m^3/day

02. Ans: (d)

Sol: Equivalent permeability $k = \sqrt{k_x k_y}$

$= \sqrt{6 \times 1.39 \times 1.39}$

$= 3.404$ m/day

$\therefore$ Seepage per unit width, $q = kH\dfrac{N_f}{N_d}$

$= 3.404 \times 9 \times \dfrac{5}{8} = 19.152$ m^3/day/m

$\therefore$ Total seepage $= q \times b = 19.152 \times 50$

$= 957.6$ m^3/day

03. Ans: 3.933 , 3.367, 1.666, 1.6667,

$\Delta Q = 2.2667 \times 10^{-5}$ m^3/sec/m

Sol:

(A) Total head loss h = (4.5 – 1.1) = 3.4

Head loss per one flow net $= \dfrac{3.4}{6} = 0.566\,m$

$\therefore$ Piezeometric head at point

a = 4.5 – 0.566

= 3.933 m

$\therefore$ Piezeometric head at point

b = 4.5 – 2 × 0.566

= 3.367 m

$\therefore$ Piezeometric head at point

c = 4.5 – 5 × 0.566

= 1. 6667 m

$\therefore$ Piezeometric head at point

d = 4.5 – 5 × 0.566

= 1.6667 m

(B) The rate of seepage through channel II per unit length

$$q = kH\frac{N_f}{N_d}$$

$N_f = 1$, $N_d = 6$

$$q = 4 \times 10^{-3} \times 10^{-2} \times 3.4 \times \frac{1}{6}$$

$= 2.266 \times 10^{-5}$ m^3/sec/m

04. Ans: (a)

Sol: FOS against piping $= \dfrac{i_c}{i_{exit}}$

$$i_c = \frac{G-1}{1+e} = 1.083$$

$$i_{exit} = \frac{\Delta H}{b}$$

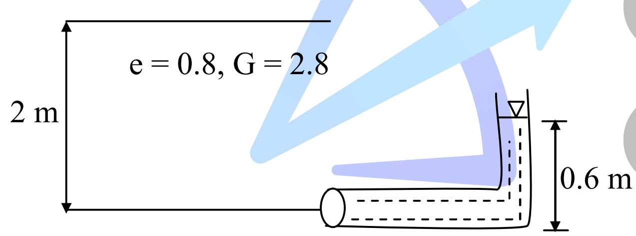
$$\Delta H = \frac{H}{N_d} = \frac{4.2}{8} = 0.525$$

$$i_{exit} = \frac{0.525}{1.65} = 0.3181$$

$$FOS = \frac{1.083}{0.3181} = 3.4$$

05. Ans: (a) & (c)

Sol:



Critical hydraulic gradient

$$I_{cr} = \frac{n-1}{1+e} = \frac{2.8-1}{1+0.8} = 1$$

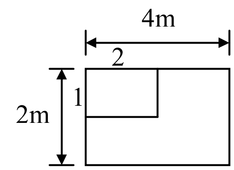
Exit gradient $= \dfrac{h_\ell}{\ell} = \dfrac{0.6}{2} = 0.3\,m$

$$F.O.S = \frac{I_{cr}}{I_\ell} = \frac{1}{0.3} = 3.3$$

10. Stress Distribution

01. Ans: (b)

Sol: At centre:



$\sigma_Z = I\,q$

$Z = 5$ m

To Calculate I:

$$m = \frac{L}{Z} \quad n = \frac{B}{Z}$$

$$m = \frac{2}{5} \quad n = \frac{1}{5}$$

$m = 0.4 n = 0.2$

From Table, $I = 0.0328$

$$\left.\begin{aligned}\sigma_Z &= 0.0328 \times 8\\ &= 0.2624\end{aligned}\right\} \text{At corner of } 1 \times 2 \text{ rectangle}$$

σ_Z at centre $= 0.2624 \times 4 = 1.05$ t/m^2

At corner:

From given table, $I = 0.0931$

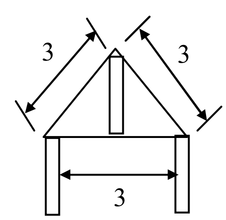
$$m = \frac{4}{5} = 0.8$$

$$n = \frac{2}{5} = 0.4$$

$\sigma_Z = 0.0931 \times 8 = 0.744$ t/m^2

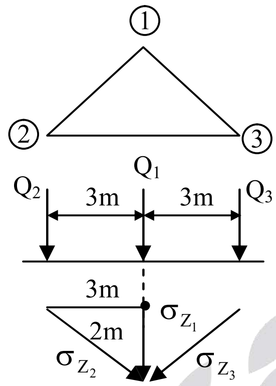
02. Ans: (d)

Sol:



Total load = 200t

Load on each column $= \frac{200}{3} = 66.66\,t$



$$\sigma_Z = \sigma_{Z_1} + \sigma_{Z_2} + \sigma_{Z_3}$$

$$\sigma_{Z_1} = \frac{Q}{Z^2}\frac{3}{2\pi} = \frac{66.66}{2^2}\frac{3}{2\pi} = 7.95\,t/m^2$$

$$\sigma_{Z_2} = \frac{Q}{Z^2}\frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{r}{Z}\right)^2}\right)^{(5/2)}$$

$$= \frac{66.66}{2^2}\frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{3}{2}\right)^2}\right]^{(5/2)}$$

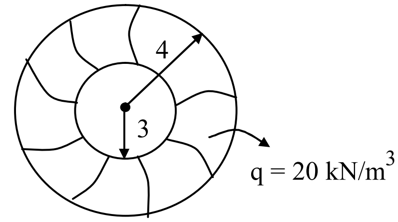
$\sigma_{Z_2} = 0.417\ t/m^2 = \sigma_{Z_3}$

$\sigma_Z = 7.95 + 0.417 + 0.417$

$\sigma_Z = 8.78\ t/m^2$

03. Ans: (c)

Sol:



σ_Z at centre =?

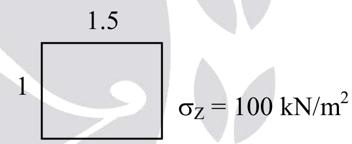
Z = 10 m

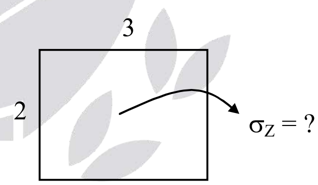
$$\sigma_Z = 20\left[1-\left(\frac{1}{1+\left(\frac{4}{10}\right)^2}\right)^{3/2}\right] - 20\left[1-\left(\frac{1}{1+\left(\frac{3}{10}\right)^2}\right)^{3/2}\right]$$

$\sigma_Z = 1.56$ kPa $= 1.56$ kN/m^2

04. Ans: (d)

Sol:





$\sigma_Z = 4 \times \sigma_Z$ of small rectangle (1×1.5)

$= 4 \times 100 = 400$ kN/m^2

05. Ans: 7.41 m

Sol: Vertical stress due to circular loaded area

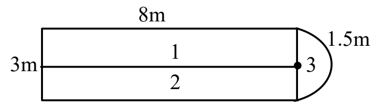
$$\sigma = q\left[1-\left(\frac{1}{1+(r/z)^2}\right)^{3/2}\right]$$

$$0.1q = q\left[1-\left(\frac{1}{1+(2/z)^2}\right)^{3/2}\right]$$

Z = 7.41 m

06. Ans: 83.05 kPa

Sol:



Total vertical stress

= stresses at 1 + stresses at 2 + stresses at section 3

$$= \sigma_{v_1} + \sigma_{v_2} + \sigma_{v_3}$$

$$\sigma_{v_1} = \sigma_{v_2} = qI$$

$$n = \frac{L}{z} = \frac{8}{3} = 2.67$$

$$m = \frac{B}{Z} = \frac{1.5}{3} = 0.5$$

$$\therefore\ m = 0.5,\ n = 2.67 \Rightarrow I = 0.1365$$

$$\sigma_{v_1} = \sigma_{v_2} = 0.1365 \times 200 = 27.3$$

Vertical stress in circular area

$$\sigma_{v_3} = q\left[1 - \left(\frac{1}{1+(r/z)^2}\right)^{3/2}\right]$$

$$= 200\left[1 - \left(\frac{1}{1+\left(\frac{1.5}{3}\right)^2}\right)^{3/2}\right] = 56.89 \text{ KPa}$$

Vertical stress in semi-circular area

$$= \frac{\sigma_{V_3}}{2} = 28.44 \text{ KPa}$$

$\therefore$ Total vertical stresses

$$= 27.3 + 27.3 + 28.44$$

$$= 83.05 \text{ KPa}$$

07. Ans: (a) & (c)

Sol: Assumptions of bossinesq's theory:

i. Soil is homogenous & isotropic

ii. Semi-infinite, elastic medium weight less and load is point load acting on ground surface.

08. Ans: (b), (c) & (d)

Sol: Westergaard, like Boussinesq, also solved the problem of pressure distribution in soils under a point load. For solving this problem, he also made certain assumptions, as were made by Boussinesq.

He assumed the soil mass to be elastic, homogeneous and of semi-infinite extent. He, in fact, assumed that the soil mass contains numerous closely spaced horizontal sheets of negligible thickness of an infinite rigid material, which permit only the downward deformation of the mass as a whole without allowing it to undergo any lateral deformation. In other words, the soil mass would not behave in all directions in a similar fashion, as was the case with Boussinesq. Hence, Westergaard did assume a sort of non-isotropy in the soil mass; whereas, Boussinesq had assumed the soil mass to be fully isotropic.

Westergaard's assumptions are more close to the field reality, especially for over-consolidated and laminated sedimentary soils, which exhibit marked an-isotropy, satisfying Westergaard's assumption of

$$\frac{E_h}{E_v} = \frac{\text{young moudlus of soil in horizontal direction}}{\text{Young modulus soil in vertical direction}} = \infty$$

11. Consolidation

01. Ans: 147.86 mm & 2.86 years

Sol: d = H = 8 m = 800 cm

For a settlement 120 mm in 2 years

$C_v = 6 \times 10^{-3}$ cm^2/s

S_f =?, t_{90} = ?

$$T_v = \frac{C_v t}{d^2}$$

$$= \frac{6\times10^{-3}\times2\times365\times24\times60\times60}{800^2} = 0.5913$$

Since $T_v > 0.282$

$T_v = 1.781 - 0.933 \log_{10}(100 - U\%)$

$0.5913 = 1.781 - 0.933 \log_{10}(100 - U\%)$

U = 81.16%

$$\Rightarrow U = \frac{s}{s_f}$$

$$\Rightarrow 81.16 = \frac{120}{s_f}\times100$$

s_f = 147.86 mm

(b) $T_v = 1.781 - 0.953\log_{10}(100-90\%)$

= 0.848

$$T_v = \frac{C_v t}{d^2} \Rightarrow 0.848 = \frac{6\times10^{-3}\times t}{120^2}$$

$\Rightarrow$ t = 2.86 years

02. Ans: (a)

Sol: ΔH_1 = 1 cm,

$\sigma'_{f_1} = 2$ kg / cm^2, $\sigma'_{0_1} = 1$ kg/cm^2,

ΔH_2 = ?

$\sigma'_{f_2} = 4$ kg/cm^2, $\sigma'_{0_2} = 2$ kg/cm^2

$$\Delta H \propto \log_{10}\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

$$\frac{\Delta H_1}{\Delta H_2} = \frac{\log_{10}\left(\frac{\sigma'_{f_1}}{\sigma'_{0_1}}\right)}{\log_{10}\left(\frac{\sigma'_{f_2}}{\sigma'_{0_2}}\right)}$$

$$\frac{1}{\Delta H} = \frac{\log_{10}\left(\frac{2}{1}\right)}{\log_{10}\left(\frac{4}{2}\right)} \Rightarrow \Delta H = 1\text{cm}$$

03. Ans: (c)

Sol: t_1 = 4 yrs, S_1 = 80 mm

t_2 = 9 yrs, S_2 = ?

For both conditions, soil is same

(Degree of consolidation).

$$U = \frac{S}{S_f}\times100$$

$S_f \rightarrow$ same for both

ΔH = 80 mm ; t_1 = 4 yrs

= 60% (less than)

t_2 = 9 yrs

$$U = \frac{S}{S_f}\times100 \Rightarrow T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$$

$$\Rightarrow C_v\frac{t}{d^2} = \frac{\pi}{4}U^2$$

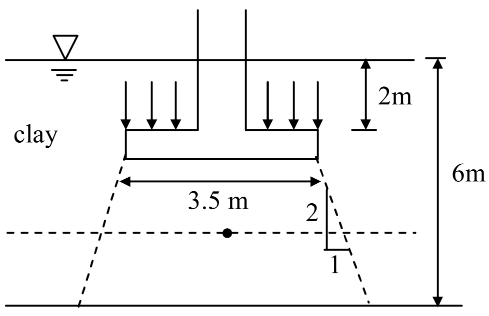
$$\Rightarrow t \propto U^2 \Rightarrow t \propto s^2$$

$$\Rightarrow \frac{t_1}{t_2} = \left[\frac{S_1}{S_2}\right]^2$$

$$\Rightarrow \frac{4}{9} = \left[\frac{80^2}{S_2^2}\right]^2 \Rightarrow S_2 = 120\text{mm}$$

04. Ans: 120 mm

Sol:



$\gamma_w = 10$ kN/m^3, $\gamma_{sat} = 19.3$ kN/m^3,

q = 500 kN

$C_C = 0.36$, $e_0 = 0.92$, $S_f = 120$

$$S_f = H_0 \frac{C_C}{1+e_0} \log_{10}\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

$$S_f = 4 \times \frac{0.36}{1+0.92} \log_{10}\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

$$\Delta\sigma' = \frac{q}{(B+Z)^2} = \frac{500}{(3.5+2)^2}$$

$\Delta\sigma' = 16.528$ kN/m^2

$\sigma_f' = \sigma_o' + \Delta\sigma'$

$= 37.2 + 16.528$

$= 53.728$ kN/m^2

$$S_f' = 4 \times \frac{0.36}{1.92} \times \log_{10}\left[\frac{53.728}{16.528}\right] = 120 \text{ mm}$$

05. Ans: (c)

Sol: $t \propto \dfrac{d^2 m_v}{K}$

$$\frac{t_2}{t_1} = \left(\frac{d_2}{d_1}\right)^2 \left(\frac{m_{v2}}{m_{v1}}\right)\left(\frac{K_1}{K_2}\right)$$

$t_1 = 15$ yrs, $d_2 = 2\,d_1$, $K_2 = 3K_1$,

$m_{v2} = 4\,m_{v1}$

$$t_2 = 15 \times \left(\frac{2}{1}\right)^2 \left(\frac{4}{1}\right)\left(\frac{1}{3}\right)$$

$t_2 = 80$ yrs

Common data for Questions 06 & 07

06. Ans: (a)

Sol: $d_1 = \dfrac{20}{2} = 10\text{mm}$, U_1=50%, t_1= 45min [lab]

$d_2 = 5000$ mm, U_2=50%, t_2 =? [field]

Same U, T_v

$t \propto d^2$

$$\frac{t_2}{t_1} = \frac{d_2^2}{d_1^2}$$

$$t_2 = 45\left(\frac{5000}{10}\right)^2 = 11250000 \text{ min}$$

$= 21.4$ years

07. Ans: (b)

Sol: $t_2 = 4 \times 21.4 = 85.6$ yrs

Common data for Questions 08 & 09

$U = \frac{80}{300} = 26.6\%$

$T_v = \frac{C_v t}{d^2}$

$\frac{\pi}{4}(26.6)^2 = \frac{C_v t}{d^2}$

$\frac{C_v}{d^2} = \frac{\pi}{225}$

08. Ans: (b)

Sol: $T_v = \frac{C_v}{d^2} \times t = \frac{\pi}{225} \times 25 = 0.35$

$U = 65\%$

$\frac{S}{300} \times 100 = 0.65$

$S = 195$ mm

09. Ans: (d)

Sol: At U% = 70%, $T_v = 0.403$

$T_v = \frac{C_v}{d^2} \times t$

$0.403 = \frac{\pi}{225} \times t \Rightarrow t = 28.8\text{yrs}$

10. Ans: (c)

Sol: **NOTE:** The time is measured from middle of construction period

$t = 5$ yrs,

$S = 90$ mm,

$S_f = 360$

$T_v = \frac{C_v t}{d^2}$

$\frac{t_1}{t_2} = \frac{T_{v_1}}{T_{v_2}} = \frac{(S/360)^2}{(90/360)^2}$

$S^2 = 90^2 \times \frac{9}{4}$

$S = 135$ mm

Common data for Questions 11 & 12

11. Ans: (b)

Sol: $\gamma_{sat} = 18$ kN/m^3

$\gamma_{sat} = 20$ kN/m^3

$\gamma_w = 10$ kN/m^3

Depth = 4 m ; $\gamma = 19$ kN/m^3

a) Immediately after load

$\sigma = q + (18 \times 5) + (20 \times 3)$

$= 226$ kPa

$U = U_{static} + U_{dynamic}$

$= 8\gamma_w + q = 156$

$\sigma' = 70$ kPa $= 70$ kN/m^2

12. Ans: (c)

Sol: Many years after (At the end of consolidation)

$\sigma = 226$ kPa

$U = 80 \quad (\because \overline{U} = 0)$

$\sigma' = 146$ kPa

13. Ans: 422.7 mm

Sol:

Preliminary analysis	Detailed investigation
H_1	$H_2 = 1.2H_1$
$\Delta\sigma'_1 = 24$KPa	$\Delta\sigma'_2 = \Delta\sigma' + 1 \times \gamma_w$ $= 24 + 9.81$ $= 33.81$ KPa
$Sf_1 = 250$ mm	$Sf_2 =?$

$S_f = m_v H\Delta\sigma'$

$S_f \propto H.\ \Delta\sigma'$ assuming 'm_v' remains same

$$\frac{s_{f_1}}{s_{f_2}} = \frac{H_1\Delta\sigma'}{H_2\Delta\sigma'}$$

$$\frac{250}{S_{f_2}} = \frac{H_1 24}{1.2H33.81} = S_{f_2} = 422\,\text{mm}$$

14. Ans: (a), (b) & (c)

Sol: Rate of settlement is directly related to rate of dissipation of excess pore water pressure.

$K = c_v\, m_v\, \gamma_w$

Hence rate of consolidation (i.e. flow of water) is controlled by, permeability, compressibility and excess pore water pressure.

$$U = \frac{u_i - u_z}{u_i}$$

15. Ans: (a), (b) & (c)

Sol: → Correction is applied for the effect of 3 – dimensional consolidation.

→ Settlement of rigid footing = 0.8 times the settlement at center of flexible footing.

→ For foundation located at certain depth, a depth factor correction has been suggested by IS 8009 – part I – 1976.

16. Ans: (a) & (c)

Sol: Immediate settlement occurs by expulsion of pore air, so volume only change by decrease of air not by decrease of water, so volume change does not occur in idealized manner. Secondary settlement is done by plastic theory.

17. Ans: (a), (b) & (d)

Sol: Compression index (c_c)

For remoulded soil (given by skempton):

$C_c = 0.007\ (w_l - 10\%)$

For undisturbed sample and filed condition (given by terzagi & peck)

$C_c = 0.009\ (w_l - 10\%)$

For all clays:

$C_c = 1.15\ (e_0 - 0.35)$

18. Ans: (a), (c) & (d)

Sol: Assumption in Terzagi theory:

i. Compression and flow are one-dimensional

ii. Darcy law is valid

iii. Soil is homogenous

iv. Soil is completely saturated.

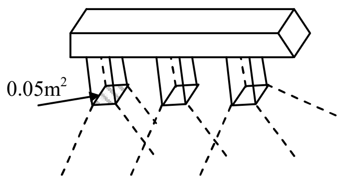
v. Soil grains and water are both incompressible

vi. Strains are small

12. Compaction

01. Ans: (c)

Sol:



Energy given by rammer per m^3 of soil in the field

$$= \frac{40 \times 1.5}{0.05 \times 0.3} = 4000 \text{ kgm/m}^3$$

Energy given in IS light compaction test in kg-m/m^3 of volume of soil

Standard value 595 kJ/m^3

= 60673.11 kg-m/m^3

$\therefore$ No. of purses required

$$= \frac{60673.11}{4000} = 15.16 \approx 16 \text{ No's}$$

02. Ans: (a), (b) & (d)

Sol: Compaction reduces the compressibility of soil.

03. Ans: (a), (b) & (d)

Sol: Coarse grained soil exhibit immediately settlement due to expulsion of pore air.

Fine grained soil exhibit consolidation due to expulsion of water it is time dependent phenomena.

Settlement occurs in fine grained soil is more than coarse grained soil.

13. Shear Strength

01. Ans: (a)

Sol: Direct stress, $\sigma_1 = 5$ Kg/cm^2

All round stress, $\sigma_3 = 3.2$ Kg/cm^2

Shear on failure plane, $\tau_f = 0.9$

$\phi = ?$

$$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2} = 0.9$$

$$\because \tau_f = \tau_{max} \Rightarrow \phi = 0$$

Another method:

$$\tau_f = \frac{\sigma_1 - \sigma_3}{2} \sin 2\alpha_f$$

$$0.9 = \frac{5 - 3.2}{2} \sin 2\left(45 + \frac{\phi}{2}\right)$$

$$1 = \sin 2\left[45 + \frac{\phi}{2}\right]$$

$$1 = \cos\phi$$

$$\phi = 0$$

Common data for Questions 02 & 03

02 & 03 Ans: (c) & (b)

Sol: Given:

Unconfined compressive test ($\phi = 0$)

$q_u = 1.2$ kg/cm^2

$\alpha_f = 50$

Cohesion of soil =?

$$\alpha_f = \left(45 + \frac{\phi}{2}\right)$$

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

$\because \alpha_f = 50$

$$50 = 45 + \frac{\phi}{2}$$

$$5 \times 2 = \phi$$

$$\phi = 10°$$

$$q_u = 2\, C_u \tan\left(45 + \frac{\phi}{2}\right) \text{ if } \phi > 0$$

$$q_u = 2\, C_u \tan\left(45 + \frac{\phi}{2}\right) \text{ if } \phi = 0$$

$$1.20 = 2\, C_u \tan\left(45 + \frac{10}{2}\right)$$

$$C_u = 0.5 \text{ kg/cm}^2$$

04. Ans: $C_u = 0$, $\phi_u = 15.8°$, $C' = 0$, $\phi' = 22°$

Sol: $\sigma_3 = 200$ kN/m^2

$\sigma_d = 150$ kN/m^2

$u_f = 75$ kN/m^2

NCC in C_u test $C_u = 0$ & $C' = 0$

To find, ϕ_u^1 & ϕ_u^{11} =?

$$\sigma_1 = \sigma_3 + \sigma_d = 200 + 150$$
$$= 350 \text{ kN/m}^2$$

$$350 = 200 \tan^2\left(45 + \frac{\phi_u}{2}\right) + 2(0)$$

$$\frac{350}{200} = \tan^2\left(45 + \frac{\phi_u}{2}\right)$$

$$\phi_u = 15.8°$$

To find ϕ'

$$\sigma_1^1 = \sigma_3^1\left(45 + \frac{\phi}{2}\right)$$

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

$$275 = (125)\tan^2\left(45 + \frac{\phi}{2}\right)$$

$$\phi = 22°$$

05. Ans: B = 0.70 & A = –0.228

Sol: Change = final value – Initial value

In consolidation stage:

$\Delta u_3 = 10 - (-60) = 70$ kN/m^2

$\Delta \sigma_3 = 100 - 0 = 100$ kN/m^2

$\Delta u_3 = B \times \Delta\sigma_3 \Rightarrow B = 0.7$

In shearing stage (or) failure stage

$\Delta u_d = -70 - 10 = -80$ kN/m^2

$\because$ u = Pore water pressure

$\Delta \sigma_d = 500$ kN/m^2

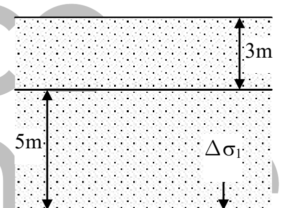
$\Delta u_d = AB\, \Delta\sigma_d$

$-80 = A \times 0.7 \times 500$

$\Rightarrow A = -0.228$

06. Ans: 78.20 kN/m^2

Sol:



$\Delta\sigma_3 = ½\, \Delta\sigma_1$

$\Delta\sigma_1 = 3\gamma = 48.6$ KPa

$\Delta\sigma_3 = ½\, \Delta\sigma_1 = 24.3$

$\Delta\sigma_d = (\Delta\sigma_1 - \Delta\sigma_3)$

$= 48.6 - 24.3 = 24.3$

$\Delta u = B(\Delta\sigma_3 + A\, \Delta\sigma_d)$

$= 31.29$ KPa

To find $\sigma' = \sigma - u$

$= 8 \times 16.2 - 31.29 = 98.31$ KPa

$S = C' + \sigma' \tan$

$= 50 + 98.31 \times \tan(16°)$

$= 78.18$ KPa

07. Ans: (a), (b), (c) & (d)

Sol: An empirical formula has been suggested by Brinch Hansen and Lundgren (1960), which takes into account the influence of the major factors influencing ϕ' in sands and gravels.

This formula is expressed as:

$\phi' = 36^o + \phi_1^o + \phi_2^o + \phi_3^o + \phi_4^o$

Where

ϕ_1^o = Grain shape correction factor, with the following values;

For angular grains = + 1^o

For sub-angular grains = 0^o

For grounded grains = – 3^o

For well rounded grains = –6^o

ϕ_2^o = Grains size correction factor with the following values:

For sand = 0^o

For fine gravel = + 1^o

For medium and coarse gravel = + 2^o

ϕ_3^o = Correction factor for Gradation, with the following values:

For poorly graded soil = – 3^o

For medium uniformity = 0^o

For well graded soil = +3^o

ϕ_4^o = Correction factor for relative density, with the following values:

For loose packing = – 6^o

For medium density = 0^o

For densest packing = + 6^o

Note:

1. The influence of relative density (i.e., density index) is the most important as can be seen above.
2. The value of 36^o is for average conditions.
3. Typical values of ϕ' for different types of sands and gravels may range between 20^o to 48^o.

08. Ans: (a), (b), (c) & (d)

Sol: **Liquefaction property in sands:** Loose saturated sands, as explained above, may undergo large scale reduction in their shear strength. When subjected to a sudden shock or other dynamic loads, since such a soil mass behaves like an un drained system, where there will occur a tendency for reduction in volume of the soil, which cannot actually occur in the available short period of time. This induces sudden increases pore pressures, reducing the effective stress. If this decrease is such that the effective stress almost reduces to zero then the soil in that localized zone will be transformed into a fluid like mass with hardly any shear strength.

The soil is then said to have liquefied, and this property is known liquefaction. Once a complete loss of strength has occurred in a limited mass of soil, the stresses which were carried by the affected soil before its liquefaction, gets transferred to the adjacent soil pats, against throwing that part of soil mass into a state of liquefaction, and the process may continue, causing large scale failure of earthen sloped, etc.

09. Ans: (b), (c) & (d)

Sol: Shear strength $\tau = C' + \overline{\sigma}\ \tan\phi'$

Where $\overline{\sigma}$ is effective stress and C' & ϕ' are effective stress shear strength parameter. Shear strength also depends upon drainage condition wheather it is drained or untrained, by knowing the drainage condition, effective and total shear strength parameter are taken into account

14. Earth Pressure

01. Ans: 316.7 kN

Sol: Given:

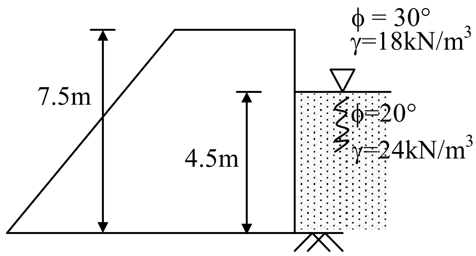


Fig. (1)

$k_a = \frac{1}{3}$, $\quad k_{a_1} = \frac{1-\sin 20}{1+\sin 20} = 0.49$

$\therefore$ at top $\sigma_v = 0, P_a = 0$

$\therefore$ at 3 m below, $\sigma_v = 18 \times 3 = 54$ kN/m^2

$$P_a = 54 \times \frac{1}{3} = 18 \text{ kN/m}^2$$

$\therefore$ at 3m just below $P_{a_1} = 54 \times 0.49 = 26.46$

At 7.5 m, $\sigma_v = 18 \times 3 + 4.5 \times 14 = 117$ kN/m^2

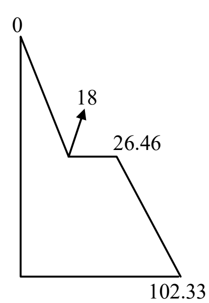
$$P_{a_2} = 0.49(117) + 10 \times 4.5$$

$$= 102.33 \text{ kN/m}^2$$

Total active thrust

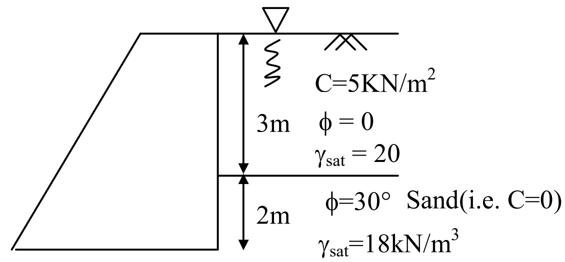
$$= \frac{1}{2} \times 18 \times 3 + \left(\frac{26.46 + 102.33}{2}\right) 4.5$$

$$= 316.77 \text{ kN/m}$$



02. Ans: (167 kN/m)

Sol: Given:



To find: Total active thrust on the back

a) At top:

As there is water $K_{a_1} = 1, K_{a_2} = 0.33$

So $\sigma'_v = 0$

$\sigma_h = K_a (0)$

$\sigma_h = 0$

$P_a = K_{a_1} \sigma_v - 2C_1 \sqrt{K_{a_1}} = -10 \text{ kPa}$

b) At 3 m depth:

a) Just above:

$\sigma'_v = 3\gamma' = 3(20-10) = 30 \text{ kPa}$

$P_a = K_{a_1} \sigma'_v - 2C_1 \sqrt{K_{a_1}} + \gamma_w h$

$= 1\,(30) - 2\,(5)\,(1) + 10 \times 3$

$= 50$ kPa

b) Just below:

$P_a = K_{a_2} \sigma'_v - 2C_2 \sqrt{K_{a_2}} + \gamma_w h$

$= 0.33\,(30) - 2\,(0)\sqrt{0.33} + 10 \times 3$

$= 39.9 \cong 40$ kPa

c) At bottom:

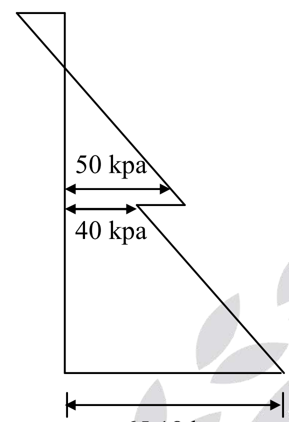
$\sigma'_v = 3\gamma' + 2\gamma'$

$\sigma'_v = 3(20-10) + 2(18-10) = 46 \text{ kpa}$

$P_a = K_{a_2} \sigma'_v - 2C_2 \sqrt{K_{a_2}} + \gamma_w h$

$= 0.33\ (46) - 2\ (0)\ \sqrt{K_{a_2}} + 10 \times 5$

$= 65.18$ kPa



To compute force:

F = $P_a \times A$

$Z_C = \frac{2C}{\gamma}\sqrt{K_a}$ } where there is no water

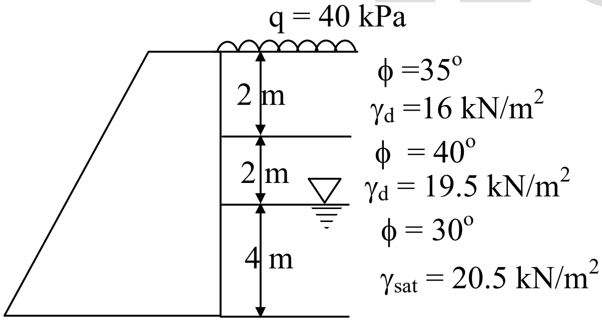
F = P_a = area of +ve portion of diagram

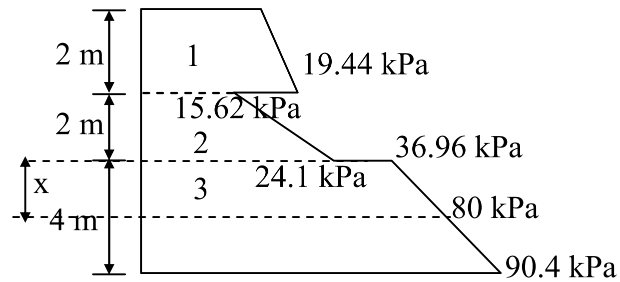
$= \left(\frac{1}{2}\times 50 \times 2.5\right) + \frac{1}{2}(40 + 65.4)\times 2$

F = 167 kN/m

03. Ans. 7.23 m

Sol:





Active Pressure distribution

$$K_{a_1} = \frac{1-\sin 35^o}{1+\sin 35^o} = 0.27$$

$$K_{a_2} = \frac{1-\sin 40^o}{1+\sin 40^o} = 0.217$$

$$K_{a_3} = \frac{1-\sin 30^o}{1+\sin 30^o} = \frac{1}{3}$$

Pressure at top:

$\sigma_v = q$

$P_a = \sigma_v \times K_{a_1} = 10.8$ kPa

Pressure at a depth 2 m

Case (a): Just above the 2 m depth

$\sigma_v = q + \gamma_a \times 2$

$= 40 + 16 \times 2 = 72$ kPa

$P_a = \sigma_v\ K_{a_2} = 72 \times 0.27$

$= 19.44$ kPa

Case (b): Just below 2 m depth

$\sigma_v = 72$ kPa

$P_a = \sigma_v\ ka_2 = 72 \times 0.217$

$= 15.62$ kPa

Pressure at a depth 4 m depth

Case (a) : Just above the 4 m depth

$\sigma_v = q + \gamma_{d_1} \times 2 + \gamma_{d_2} \times 2$

$= 40 + 16 \times 2 + 19.5 \times 2 = 111$ kPa

$P_a = \sigma_v\ K_{a_2} = 111 \times 0.217 = 24.1$ kPa

Case(b) : Just below the 4 m depth

$\sigma_v = 111$ kPa

$P_a = \sigma_v\ K_{a_3} = 111 \times \frac{1}{3} = 36.96$ kPa

Pressure at base:

$\sigma_v = q + \gamma_{d_1} \times 2 + \gamma_{d_2} \times 2 + \gamma' \times 4$

$= 40 + 16 \times 2 + 19.5 \times 2 + (20.5 - 9.81) \times 4$

$= 153.76$ kPa

$P_a = \sigma_v\ K_{a_3} + \gamma_w \times 4 = 90.4$ kPa

In the third layer : At $P_a = 80$ kPa

$\sigma'_v = 40 + 2[16 + 19.5] + [20.5 - 9.81]x$

$= 111 + 10.69\ x$ (x = depth in the third layer at which $p_a = 80$ kPa)

$80 = \sigma'_v \times K_{a_3} + \gamma_w \times x$

$80 = \frac{1}{3}[111 + 10.69x] + 9.81x$

$\Rightarrow x = 3.23$ m

From top $= 2 + 2 + x = 7.23$ m

04. Ans: (c) & (d)

Sol: Assumption

i. Soil is semi-infinite

ii. Wall back is smooth

iii. Soil is homogeneous, dry and cohesionless

iv. Plastic equilibrium developed.

14. Stability of Slopes

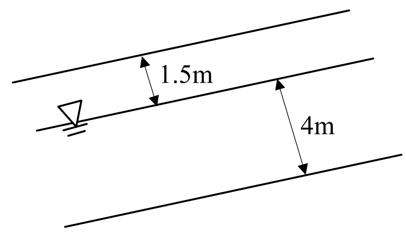
01. Ans: (d)

Sol: $\phi' = 35°$,

$\gamma_{sat} = 19$ kN/m^3

$i = 28°$,

$\gamma_w = 9.8$ kN/m^3



Against translational failure,

$$FOS = \frac{C + rz\tan\phi\cos i}{rz\cos i \sin i}$$

$$= \frac{0 + (r1.5 + (4 - 1.5)r')\tan\phi\cos^2 i}{(r1.5 + (4.15)r')\cos i \sin i}$$

$$= \frac{(19 \times 1.5 + 2.5 \times (19 - 9.8))\tan 35 \cos^2 28}{(19 \times 1.5 + 2.5 \times 19)\cos 28 \sin 28}$$

$= 0.89$

02. Ans: 4.77

Sol: Infinite slope, seepage parallel to slope

$$F = \frac{C' + \gamma z \cos^2 i \tan\phi'}{\gamma_{sat} z \cos i \sin i}$$

$Z = H_c$

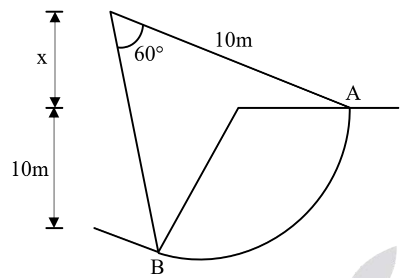
$$1 = \frac{25 + 8 \times H_c \cos^2(35°)\tan(28°)}{18 \times H_c \cos(35°)\sin(35°)}$$

$H_c = 4.77$

03. Ans: 1.184, 2.66

Sol: $C = 50\ kN/m^2$

$\phi = 10^o$ $\quad \sigma' = 255\ kN/m^2$, t = 840 kN



$N = \sigma' I.1$

$= 255 \times 1.04 \times 1 = 265.2$

$$\hat{L} = \frac{(2\pi r)\times 60}{360} = 10.47$$

$$F = \frac{C.\hat{L} + \Sigma N \tan\phi}{\Sigma T} = \frac{C.\hat{L} + N\tan\phi}{T}$$

$$F = \frac{50\times 10.47 + 255(10.47)\tan 10}{840}$$

$= 1.1836$

w.r. to height

$$H_C = \frac{4C}{\gamma\sqrt{k_a}}$$

$$= \frac{4C}{\gamma}\tan\left[45+\frac{\phi}{2}\right] = \frac{4\times 50}{18}\tan 50 = 13.24\,m$$

$H = R - x$

$= 10 - R\cos 60^o = 5$

$$F = \frac{H_C}{H} = \frac{13.24}{5} = 2.67$$

04. Ans: $F_c = 1.16$ & $F_\phi = 1.2$

Sol: Given:

Cutting is to be made in soil

Slope of soil = 25°

Depth of soil = 25 m

Cohesion soil $C = 0.35\ kg/cm^2$

Angle $\phi = 15°$

Bulk density $\gamma = 2$ gm/cc

a) FOS w.r.t cohesion, if FOS desired with respect to friction = 1.5

As we know

$$F_S = \frac{\tan\phi}{\tan\phi_m} \Rightarrow 1.5 = \frac{\tan 15^\circ}{\tan\phi_m}$$

$$\tan\phi_m = \frac{\tan 15^\circ}{1.5}$$

$\phi_m = 10°$, $\quad S_n = 0.06$

$$S_n = \frac{C}{F_C\,\gamma H}$$

$$0.06 = \frac{3500}{F_C \times 2000 \times 25}\ F_C = 1.16$$

b) If FOS with respect to cohesion is 1.5, then what is FOS with respect to friction = ? (F_ϕ = ?)

$$S_n = \frac{C}{F_C\,\gamma H}$$

$$S_n = \frac{3500}{1.5\times 2000\times 25}$$

$S_n = 0.049 \cong 0.05$

$\because \phi_m = 12.5°$

$$F_\phi = \frac{\tan\phi}{\tan\phi_m} = \frac{\tan 15^\circ}{\tan 12^\circ 5'} = 1.2$$

05. Ans: 3.56 & 1.18

Sol: Given:

A new canal is excavated with

Depth of canal h = 5 m

$C = 1.4\ t/m^2$; $\phi = 15°$

$\gamma_{sat} = 1.945\ t/m^3$

Slope of bank = 1 : 1

To find:

a) FOS w.r.t cohesion when canal runs full = ?

b) If it is suddenly emptied, FOS = ?

a) $$S_n = \frac{C}{F_c \gamma^1 H}$$

$$0.083 = \frac{1.4}{F_C\,(1.945-1)\,5} \Rightarrow F_C = 3.56$$

For $\phi = 15^0$; $S_n = 0.083$

For $\phi = 7.5^0$; $S_n = 0.122$

b) $$\phi_m = \frac{\gamma}{\gamma_{sat}} \times 15 = 7.5$$

$$S_n = \frac{C}{F_C\,\gamma_{sat}\,H}$$

$$F_C = \frac{1.4}{0.122 \times 1.945 \times 5} = 1.179$$

06. Ans: $F_c = 2.4$ & $F_\phi = 2.89$

Sol: Given:

Embankment is to be made of a soil

Shear parameters of soil:

$C' = 30\ KN/m^2$; $\phi' = 15°$

To find F_ϕ if $F_C = 1$

$$S_m = \frac{C'}{F_C} + \frac{\sigma' \tan\phi}{F_\phi} \quad ; \quad (\phi' = 15')$$

$$39.25 = \frac{30}{1} + \frac{100 \times \tan 15}{F_\phi}$$

$F_\phi = 2.89$

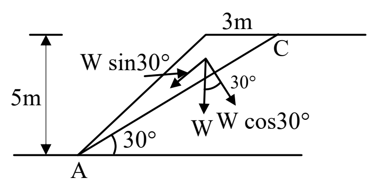
To find F_C if $F_\phi = 1$

$$S_m = \frac{C'}{F_C} + \frac{\sigma' \tan\phi'}{F_\phi}$$

$$39.25 = \frac{30}{F_C} + \frac{26.79}{1} \Rightarrow F_C = 2.40$$

07. Ans: 4.63

Sol:



Area of wedge:

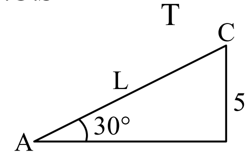
$$a = \frac{1}{2} \times b \times H = \frac{1}{2} \times 3 \times 5 = 7.5\,m^2$$

$W = a \times 1 \times \gamma$

$N = w \cos 30°$

$T = w \sin 30°$

$$F.O.S = \frac{C.L + N\tan\phi}{T}$$



$$\sin 30° = \frac{5}{L}; \; L = \frac{5}{\sin 30°} = 10$$

$\therefore$ F = 4.63

08. Ans: (a) & (c)

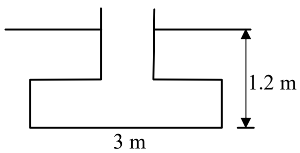
Sol: Taylor stability method is suitable for C - ϕ soil and ϕ = 0 i.e., pure clay soil, and it is valid in finite slope only. The max value is 0.261 and it occurs in soft clay having angle of internal friction is zero.

16. Bearing Capacity of Soil

01. Ans: 2.54, 2.03

Sol:

(a)



Net ultimate bearing capacity

$q_{nu} = CN_c + (N_q - 1)\,\gamma_D + 0.4\,\gamma BN_\gamma$

$C = 0,\ N_q = 22,\ N_\gamma = 20$

$q_{nu} = 21 \times 17 \times 1.2 + 0.4 \times 17 \times 3 \times 20$

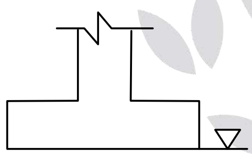
$= 836.4$ KPa

Safe bearing capacity $q_s = \frac{q_{nu}}{F} + \gamma_D$

$$350 = \frac{836.4}{F} + 17 \times 1.2$$

$F = 2.54$

(b)



$q_{nu} = (N_q - 1)\gamma D + 0.4\,\gamma' BN_\gamma$

$= 21 \times 17 \times 1.2 + 0.4\,(20 - 9.81)\,3 \times 20$

$= 672.96$

Safe bearing capacity

$$q_s = \frac{q_{nu}}{F} + \gamma D$$

$$350 = \frac{672.96}{F} + 17 \times 1.2$$

$F = 2.04$

02. Ans: (b)

Sol: Given:

Depth = 1m

Square plate = 30 cm^2

Load = 7.2 tones

S_p settlement = 25 mm

To find:

If settlement is limited for 10 mm

Allowable bearing pressure=?

$$q_1 = \frac{7.2}{(0.3)^2} = 80\,t/m^2$$

$S_2 = 10$ mm

$q = ?$

($S \propto q$ in case of granular soils)

$$\frac{S_2}{S_1} = \frac{q_2}{q_1}$$

$$\frac{10}{25} = \frac{q_2}{80}$$

$q_2 = 32\ t/m^2$

03. Ans: 439.55

Sol: B = 2 m

L = 2 m

$e = 1 - 0.85$

$= 0.15$ m

$B' = B - 2e$

$= 2 - 2 \times 0.15 = 1.70$ m

There is no effect of water table as it is located well below the base of footing.

$Q_{nu} = [\gamma DN_q S_q d_q i_q + 0.5_\gamma\, B' N_\gamma S_\gamma i_\gamma d_\gamma]$

Given:

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

D = 1 m $N_q = 33.3$

$N_\gamma = 37.16$ $B' = 1.70$ m

Shape factors, S_q or $F_{qs} = 1.314$

S_γ or $F_{\gamma s} = 1.314$

Depth factors d_q or $F_{qd} = 1.113$

d_γ or $F_{\gamma d} = 1.113$

Inclination factors

i_q or $F_{qi} = 0.444$

i_γ or $F_{\gamma i} = 0.02$

Substituting in the equation,

$$q_{ns} = \begin{bmatrix} (18\times1\times33.3\times1.314\times1.113 \\ \times0.444)+(0.5\times18\times1.7\times37.16 \\ \times1.314\times113\times0.02 \end{bmatrix}$$

$= 405.85\ kN/m^2$

$q_{nu} = q_u - \gamma D = 405.85 - 18 \times 1$

$= 387.85\ kN/m^2$

$$q_{ns} = \frac{q_{nu}}{F} = \frac{387.85}{3} = 129.28\ kN/m^2$$

Net safe load $= A' \times q_{ns}$

$= B'L \times q_{ns}$

$= 1.7 \times 2 \times 129.28 = 439.55$ kN

04. Ans: 5.01 m

Sol: For design safety, $q_n \le q_{na}$

(smaller of q_{ns} and q_{np})

If q_{np} is not given, then $q_{na} = q_{ns}$

$q_n \le q_{ns}$

or $q_g \le q_s$

Gross load = co. load + 5% col.load

$$= 1962 + \frac{5}{100}1962$$

$= 2060.1$ kN

$$q_g = \frac{Q_s}{A} = \frac{2060.1}{1.5^2}\ kN/m^2$$

$= 915.6$ kPa

$$q_s = \frac{q_n - \gamma D_f}{F} + \gamma D_f$$

$$q_s = \frac{1.3N_c + \gamma D_f(N_q - 1) + 0.4\gamma BN_r}{F} + \gamma D_f$$

$915.6 = q_s$

$$= \frac{0 + 20.6\times D_f \times(22-1) + 0.4\times20.60\times2.5\times20}{3} + 20.6\times17$$

equate $q_g = q_s$

$D_f = 5.01$ m

05. Ans: 6.55 m

Sol: Given:

Size of foundation = 14m×21m

Unconfined compressive strength = 15 kN/m^2

$$C_u = \frac{15}{2} = 7.5\ kN/m^2$$

Gross pressure intensity $q_u = 140\ kN/m^2$

FOS = 3; $\gamma_{clay} = 19\ kN/m^3$

For safety $q_n \le q_{na}$

Where, $q_{na} \to$ net allowable bearing capacity of soil which is smaller of q_{ns} & q_{np}

According to skemptons;

$q_{nu} = CN_c$

For Rectangular footing;

$$N_c = 5\left[1 + 0.2\frac{D}{B}\right]\left[1 + 0.2\frac{B}{L}\right]$$

$$q_{ns} = \frac{q_{nu}}{F.O.S}$$

$$N_c = 5\left[1 + 0.2\frac{D}{14}\right]\left[1 + 0.2\times\frac{14}{21}\right]$$

$$N_c = \frac{17}{3}\left(1 + 0.2\frac{D}{14}\right)$$

$$q_{nu} = 7.5\times\frac{17}{3}\left(1 + 0.2\frac{D}{14}\right)$$

$$= 42.5\left(1 + 0.2\frac{D}{14}\right)$$

$$q_{ns} = \frac{q_{nu}}{FOS} = \frac{42.5}{3}\left(1 + 0.2\frac{D}{14}\right)$$

$$= 14.17\left(1 + 0.2\frac{D}{14}\right)$$

Since there is a provision for basement floor, the footing is not back filled. Hence,

$q_n = q_u - \gamma D$

$= 140 - 19\times D$

$140 - 19\times D = 14.17\left(1 + 0.2\dfrac{D}{14}\right)$

$140 - 19\times D = 14.17 + 0.202D$

$125.83 = 19.202D$

$D = 6.55$ m

06. Ans: (a), (c) & (d)

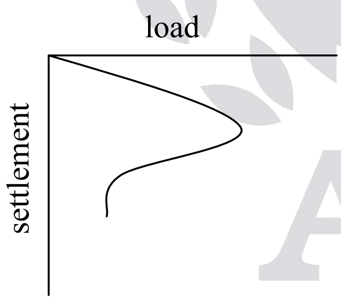
Sol: General Shear Failure:

1. Brittle type stress strain behavior (dense sand is characterized by

(a) well defined failure pattern.

(b) a sudden catastrophic failure accompanied by tilting of foundation and

(c) Bulging of ground surface adjacent to the foundation.

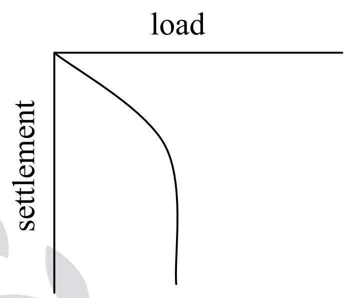


2. Punching shear failure: Occurs in soil possessing the stress – strain characteristics of a very plastic soil.

i. Poorly defined shear plane

ii. Soil zone beyond the loaded area being little affected.

iii. Significant penetration of a wedge-shaped soil zone beneath foundation accompanied by vertical shear beneath the edge of foundation.

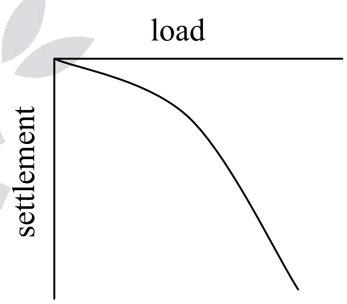


3. **Local shear failure:** It has some characteristics of both GSF and PSF. Main feature of local shear failure are.

i. Well defined wedge and slip surface only beneath the foundation.

ii. Slip surface not visible beyond edge of foundation.

iii. Slight bulging of ground surface adjacent to foundation.



17. Pile Foundation

01. Ans: Q_u = 134.3 kN

Sol: Given:

Diameter of bored concrete pile = 30cm

Length passes through stiff fissures = 6.50m

Depth of shrinkage & swelling=1.50m

Average undrained stress of clay = 50 kPa

below pile = 100 kPa

$\alpha = 0.3$

To find:

Ultimate load capacity = ?

$Q_u = A_b\, C\, N_c + A_s\, \alpha\, C$

$= 0.070\times100\times9+4.71\times0.3\times50$

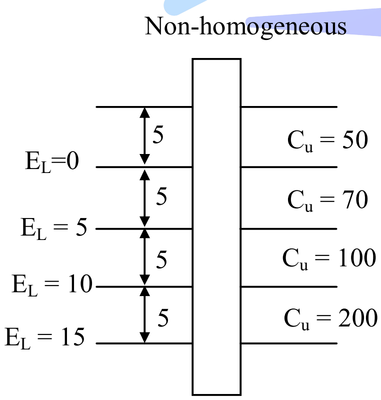
$= 134.3$ kN

$\because A_S = \pi\, d\, l$

$= 3.14\times5\times0.3 = 4.71\ m^2$

02. Ans: 669 kN

Sol:



Given:

L = 20 m

ϕ = 500 mm = 0.05 m

$\alpha = 0.4$

F = 2.5

$N_c = 9$; $\phi_u = 0$

To find:

Q_{safe} = ?

$$Q_{safe} = \frac{1}{F}[A_b C N_C + A_s \alpha C]$$

At base:

$$Q_{safe} = \frac{1}{2.5}\left(\begin{array}{l}\frac{\pi}{4}\times0.5^2\times200\times9+(\pi\times0.5)\times5\times0.4\times50\\ \quad+\pi\times0.5\times5\times0.4\times70\\ \quad+\pi\times0.5\times5\times0.4\times100\\ \quad+\pi\times0.5\times5\times0.4\times200]\end{array}\right)$$

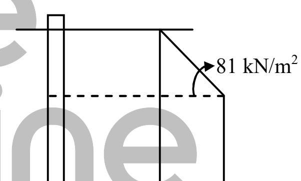
$= (353.25+1318)$

$q_u = 1672.26$

$$q_s = \frac{q_u}{F} = \frac{1672.05}{2.5} = 669\ \text{kN}$$

03. Ans: 813.41 kN

Sol:



Critical depth = 15 × diameter

= 15 × 0.3 = 4.5 m

Effective vertical pressure $\sigma'_v = 4.5\times18$

= 81 kN/m^2

$\therefore\ Q_u = A_b f_b + A_s f_s$

$$= \frac{\pi}{4}\times d^2\times\sigma'_v\times N_q + A_s.\sigma_v k\tan\delta$$

$$= \frac{\pi}{4}\times0.3^2\times81\times137+2\times\tan40\left(\frac{1}{2}\times81\times4.5+81\times7.5\right)$$

$\pi\times0.3$

$= 784.40 + 1249.12$

$Q_u = 2033.52$

$$\therefore\ \text{safe load capacity} = \frac{Q_u}{F} = \frac{2033.52}{2.5}$$

= 813.40 kN

04. Ans: (Q_g = 27390.6 kN)

Sol: Given:

n = 25

L = 12 – 2 = 10 m

Dia = 0.5 m

S = 1 m c/c

C = 180 kPa

C_{avg} = 110 kPa

α = 0.45

$B_0 = L_0 = 4S + d$
$= 4.5$ m

$$Q_{gi} = n[\frac{\pi}{4}(0.5)^2 \times 180 \times 9 + \pi \times 0.5 \times 10 \times 0.45 \times 100]$$
$$= 27390.76 \text{ kN}$$

$Q_{gb} = (4.5)^2 \times 9 \times 180 + 4 \times 4.5 \times 10 \times 110$
$= 52605$ kN

Q_g = 27390.6 kN

$\left(\begin{array}{l}\text{take minimum of two}\\ \text{i.e., } Q_{gi} \;\&\; Q_{gb}\end{array}\right)$

05. Ans: S = 2.18d

Sol: Given:

n = 16 pile group

α = 0.6

$$Q_{gi} = n\left[\frac{\pi}{4}d^2 \times C \times 9 + \pi d \times L \times 0.6C\right]$$

(neglect end bearing)

$= n\,[\pi dL \times 0.6\,C]$

$Q_{gb} = 4(3S + d) \times L \times C$

For optimum spacing

$Q_{gi} = Q_{gb}$ $\qquad(\eta_g = 100\%)$

$16[\pi dl \times 0.6C] = 4(3S + d) \times L \times C$

$4\pi d \times 0.6 = 3S + d$

6.54 d = 3 S

S = 2.18 d

06. Ans. 635 kN

Sol: λ Method:

$$Q_u = A_b \times C \times N_c + A_s\lambda \times [\sigma'_{va} + 2c]$$

λ = constant = 0.15,

Dia = 0.4 m, γ = 18 kN/m^3, F.O.S = 3

Depth (H) = 25 m, N_c = 9 for pile in clay

σ'_{va} = Average effective vertical pressure along the pile length

$$\sigma'_{va} = \frac{0 + \gamma H}{2} = \frac{0 + 18 \times 25}{2}$$
$= 225$ kPa

$$Q_u = \frac{\pi}{4}(0.4^2) \times 80 \times 9 + \pi \times 0.4 \times 25 \times 0.15 \times (225 + 2 \times 80)$$
$= 1904.74$ kN

Safe load (or) Allowable load

$$Q_{safe} = \frac{Q_u}{F.O.S} = \frac{1904.74}{3}$$
$= 635$ kN

07. Ans: 68.25%; 6825 kN

Sol: Engineering News formula for Drop Hammer:

It is based on the assumption that kinetic energy delivered by the hammer during driving operation is equal to work done on the pile.

According to Engineering New's formula,

$$Q_s = \frac{W.h.\eta_h}{F(S + C)}$$

Where,

Q_S =Safe Pile capacity

W = Weight of hammer

h = height of drop

η_h = Efficiency of pile hammer

S = penetration of pile per hammer blow

C = constant

For drop hammer, C = 2.54 cm, for steam hammer C = 0.254 cm

Factor of safety = 6

Applications:

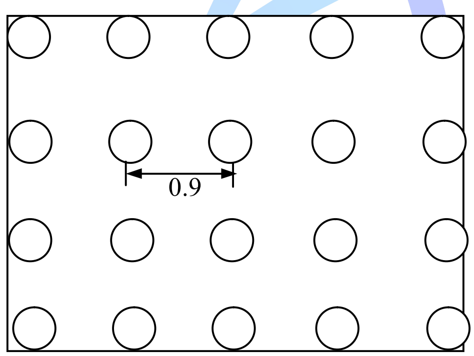
- This formula is more applicable to piles driven into cohesionless soil.
- If the pile is driven into saturated loose sand and silt, liquefaction might result, reducing the pile capacity. So it is not applicable to saturated loose sand.

4 × 5 pile group

Diameter of each pile = 0.3 m

C/C spacing = 0.9 m

capacity of a single pile = 500 kN



According to converse Labarre formula:

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

m → no. of rows of piles = 4

n → no.of piles in each row= 5

$$\theta = \tan^{-1}\left(\frac{d}{s}\right) = 18.43$$

$$\eta_g = 1 - \frac{18.43}{90}\left[\frac{(5-1)4 + 5(4-1)}{4 \times 5}\right]$$

$$= 1 - \frac{18.43}{90}\left[\frac{16+15}{20}\right]$$

$$\eta_g = 68.25\ \%$$

Capacity of free standing pile group

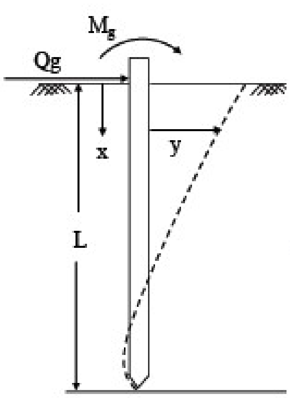
$$= \eta_g \times Q_{gi} \times n = 0.6825 \times 500 \times 20 = 6825\ \text{kN}$$

08. Ans: (a), (b) & (c)

Sol: A pile subjected to lateral loading is one of the classes of problem that involve interaction of soil and structure the main aim of project is to calculate structural parameters like slope, deflection, moment, slope and soil reaction at each joint.

09. Ans: (b)

Sol:



Q_g = horizontal load

M_g = moment at ground level

If pile is fixed to the pile cap, the pile cap provides fixity to pile head. The fixity provided is equivalent to moment M_g = – 0.93 Q_g T at the ground level. The negative sign indicates that moment caused by M_g and Q_g are of different sign in case of free head pile, moment M_g due to fixity of pile head is zero while for a completely fixed-head pile. Moment m_g is –0.93 Q_gT. Thus for piles with intermediate fixity, the value of moment can be interpolated.

10. Ans: (a), (b) & (c)

Sol: Advantage of bored and cast in situ piles i.e. under reamed pile

i. Very little displacement and no risk of ground heave.

ii. Length can be readily varies

iii. Soil can be inspected and checked with soil investigation data.

iv. Piles can be installed in very great length and very large diameter and end enlargement of upto 2 or 3 shaft diameter are possible in clays and soft rock.

v. Pile can be installed without much noise and vibration and with limited had room.

Disadvantage

i. Boring method may loosen sandy or gravelly soils.

ii. Concreting under water will pore problem.

iii. Concrete cannot be subsequently inspected.

iv. Susceptible to waisting or necking in squeezing

v. Enlarged bases can not be formed in granular soil.

11. Ans: (b), (c) & (d)

Sol: Precast concrete piles are prismatic or circular in section. These piles may be of uniform diameter or tapered. They are usually cast a central casting yard, cured and transported to the construction site. If arrangements for casting and curing are available, they may be cast at the site also.

As a precast concrete pile is usually in a state of compression, no reinforcement is required from load bearing considerations. However, piles are reinforced to take care of handling stresses. Solid sections of 0.2 m to 0.3 m side are usually used. Precast, hollow cylindrical sections are also used where large stiffness and higher bearing capacity is required, though in India, hollow sections are not popular. Precast concrete piles in lengths upto 20 m and precast hollow pipe piles upto 60 m length have been used. Shorter piles can carry loads upto 600 kN, whereas the capacity of longer piles can, in some cases, be as large as 2000 kN.

Concrete piles are considered more or less permanent. However, in exceptional Circumstances, the soil may contain deleterious substances which may affect the pile. Splicing of precast concrete piles is difficult. Further, if the pile at the site proves to be too long, chopping off the extra length is difficult and is likely to damage the pile. Handling and driving of precast concrete piles also requires heavy equipment. Heave and disturbance of surrounding soil may also cause difficulties.

12. Ans: (a) & (c)

Sol: A test pile is a pile which is used only in a load test and does not carry load of super structure. The minimum test load on such a piles should be twice the safe load or load at which the total settlement attains a value of 10% of pile diameter in case of single and 40 mm in case of pile group.

18. Soil Exploration

02. Ans: (c)

Sol: $N = 6 + 6 + 8 + 7 = 27$

$N' = C_N N = 1 \times 27 = 27$

$$N'' = 15 + \left(\frac{N' - 15}{2}\right) = 21$$

03. Ans: 14

Sol: Corrected value $N' = C_N N$

C_N=correction factor for over burden pressure

$$C_N = 0.77 \log_{10}\left(\frac{1905}{\sigma_o'}\right)$$

$$\sigma_o' = 2 \times 18 + (18 - 9.81) \times 3$$

$$= 60.57 \text{ kN/m}^2$$

$$C_N = 0.77 \log_{10}\left(\frac{1905}{60.57}\right) = 1.153$$

$N' = 1.153 \times 12 = 13.8 \simeq 14$

04. Ans: (b) & (c)

Sol: Standard penetration test (SPT) is done for granular soil not for cohesive soil. SPT test represents the relative density of soil and its value increases with increase in denseness of soil. It is performed only in field.

19. Sheet Piles

01. Ans: 98.7 kN

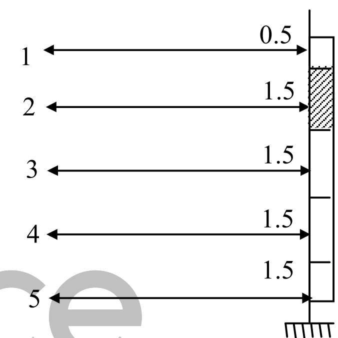
Sol: $k_a = \dfrac{1 - \sin\phi}{1 + \sin\phi} = 0.259$

$\gamma = 20 \text{ kN/m}^3$

$H = 6.5$ m

$\phi = 36°$

$C = 0$



$P' = 0.65\, K_a H\gamma$

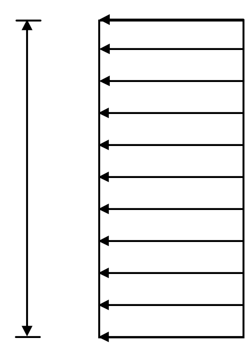
$P' = 21.93 \text{ kN/m}^2$

Total pressure acting

$P = 21.93 \text{ kN/m}^2 \times \text{Height} \times \text{Width}$

$P = 21.93 \times 6.5 \times 3$

$P = 427.7$ kN



$P' = 0.65\, K_a H\gamma$

The average load taken by the strut

$$= \frac{427.7}{5} = 85.55 \text{ kN}$$

But in the problem they asked maximum load taken by the strut

struts (1) (2) (3) (4) are taken maximum loads, (5) struts are taken minimum load.

Strut (2) taken load = 1.5×3×P′

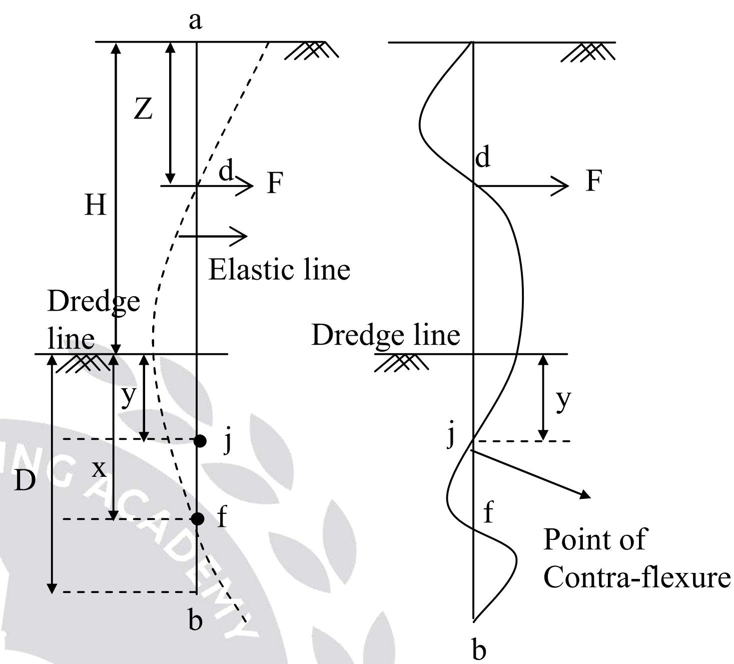
= 1.5×3×21.93 kN

= 98.68 kN

02. Ans: (c) & (d)

Sol: A number of methods are used for design of anchored bulk head. However the method commonly used are free earth support method and fixed earth support.

Fixed earth support method



When depth is small, the anchored sheet pile would be called as free-earth support.