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Important Questions *for* GATE 2022

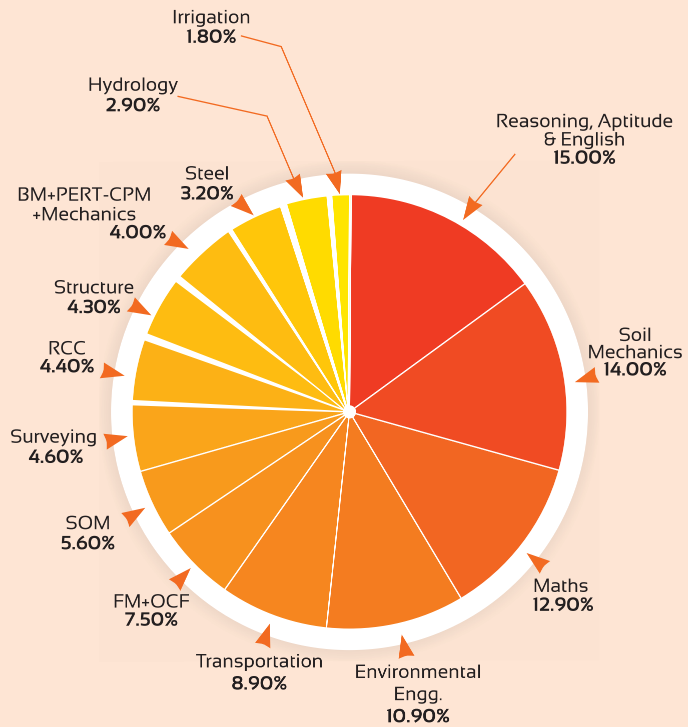
CIVIL ENGINEERING

Day 7 of 8

Q.151 - Q.175 (Out of 200 Questions)

Geotechnical Engineering

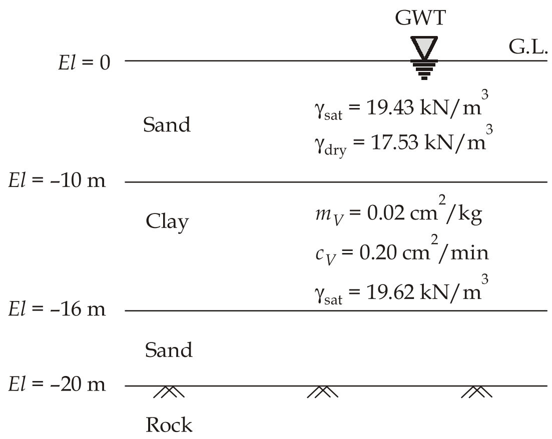
SUBJECT-WISE WEIGHTAGE ANALYSIS OF GATE SYLLABUS



Subject	Average % (last 5 yrs)
Reasoning, Aptitude and English	15.00%
Soil Mechanics	14.00%
Engineering Mathematics	12.90%
Environmental Engineering	10.90%
Transportation Engineering	8.90%
Fluid Mechanics + OCF	7.50%
Strength of Materials	5.60%
Surveying Engineering	4.60%
Reinforced Cement Concrete	4.40%
Structural Analysis	4.30%
Building Materials+PERT-CPM+Mechanics	4.00%
Steel Structures	3.20%
Engineering Hydrology	2.90%
Irrigation Engineering	1.80%
Total	**100%**

Geotechnical Engineering

Q.151 With the soil profile as shown below, if the water table gradually drops to elevation (El) = –6 m, then the settlement is



(a) 5.80 cm
(b) 4.20 cm
(c) 2.40 cm
(d) 6.10 cm

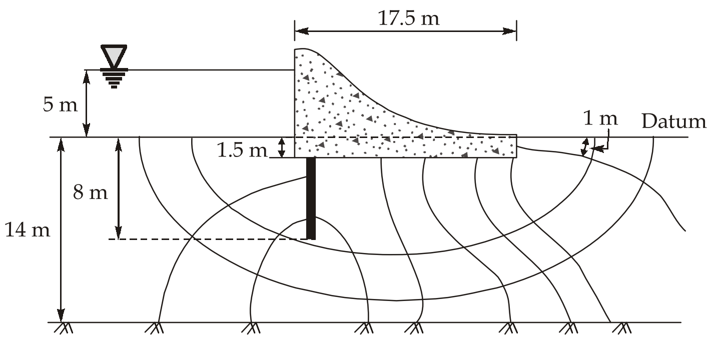
Q.152 The soil at an 18° infinite slope is subjected to full depth seepage. The soil properties are as follows:

$$\gamma_{sat} = 17 \text{ kN/m}^3$$
$$c' = 10 \text{ kN/m}^2$$
$$\phi = 14°$$

The critical height of this slope, measured vertically, is ____________ m.

Q.153 A foundation in a loose sand is 4 m wide, 6 m long and 1.5 m deep. The soil unit weight is 16 kN/m^3 and has an effective angle of internal friction is 22.6°. The safe bearing capacity of soil, adopting a factor of safety of 2, is __________ kN/m^2.
[For ϕ = 22.6°, N_c = 21.55, N_q = 10.16 and N_γ = 7.44]

Q.154 The flow net constructed for the dam is shown in the figure below. The exit gradient is _______.



Impervious Strata

Q.155 Compacted cylindrical specimen 50 mm diameter and 100 mm long is to be prepared from dry soil. If the specimen is required to have a water content of 15% and the percentage of air voids is 20%, calculate the weight of water required (in grams) for the preparation of the soil whose specific gravity is 2.69.

(a) 60.8 (b) 96.4
(c) 45.2 (d) None of these

Q.156 In a falling head permeability test, head causing flow was initially 75 cm and it drops 12 cm in 8 minutes, then the time (in minute) required for the head to fall to 30 cm is

(a) 36 (b) 42
(c) 48 (d) None of these

Q.157 An unsupported excavation is made to the maximum possible depth in a pure clay soil having bulk density of 20 kN/m^3 and cohesion of 80 kN/m^2. The active earth pressure at base level of excavation is ___________ kN/m^2.

Q.158 Water is flowing at a rate of 0.08 cm^3/sec in upward direction through a fine sand sample whose coefficient of permeability is 0.004 cm/sec. The sample thickness is 16 cm and cross-sectional area is 64 cm^2. If the saturated unit weight of sand is 20.8 kN/m^3, then the effective pressure at the bottom section of the sample is

(a) 1.27 kN/m^2 (b) 1.81 kN/m^2
(c) 2.24 kN/m^2 (d) 2.81 kN/m^2

Q.159 A group of nine piles each 12 m long is used as a foundation for a bridge pier. The piles used are 45 cm in diameter with centre to centre spacing of 1.8 m. The sub-soil consists of clay with unconfined compressive strength of 6 t/m^2 and overall adhesion factor of 0.85. If the end bearing action is neglected then the efficiency of pile group is ___________%.

Q.160 An unconfined compression test was conducted on an undisturbed sample of clay. The sample has a diameter of 40 mm and was 80 mm long. The load at failure was 32 N and the axial deformation of the sample at failure was 16 mm. The undrained shear strength of clay is

(a) 5.09 kN/m^2 (b) 8.84 kN/m^2
(c) 10.19 kN/m^2 (d) 15.62 kN/m^2

Q.161 In a laboratory consolidation test, the void ratio of a soil sample reduced from 0.85 to 0.73 as the pressure was increased from 1 to 2 kg/cm^2. If the coefficient of permeability of the soil is 3.3×10^{-4} cm/s, then the coefficient of volume change (in cm^2/kg) and coefficient of consolidation (in cm^2/sec) respectively are

(a) 0.092 and 5.08 (b) 0.065 and 5.08
(c) 0.092 and 3.12 (d) 0.065 and 3.12

Q.162 A 3 m thick soil stratum has coefficient of permeability of 3×10^{-7} m/s. A separate test gave porosity 40% and bulk unit weight 21 kN/m^3 at a moisture content of 31%. The flow (in terms of discharge per unit area) required to maintain the critical condition will be ___________ $\times 10^{-7}$ m^3/s.m^2 [Take γ_w = 10 kN/m^3]

Q.163 While calculating water content of a soil sample, observations were made related to Pycnometer test:

Weight of pycnometer + wet soil sample = 12 N

Weight of pycnometer + wet soil sample + water filling the remaining volume = 20 N

Weight of pycnometer + water = 17 N
Water content of the soil sample is ________ %
[Take specific gravity of soil solids = 2.65 and weight of empty pycnometer = 7 N]

Q.164 The permissible settlement of an isolated foundation on a granular soil is 40 mm. A plate load test is performed with 300 mm square plate undergoing a settlement of 20 mm under the same load intensity. The ultimate bearing capacity of plate is 22t/m^2. The ultimate bearing capacity of foundation will be ________ t/m^2.

Q.165 The sieve analysis and consistency limit tests conducted on a soil sample gave the following results:

Fraction passing through 4.75 mm sieve = 82%
Fraction passing through 75 μm sieve = 9%

D_{10} = 0.11 mm
D_{30} = 0.45 mm
D_{60} = 1.12 mm
Liquid limit = 22%
Plastic limit = 12%

The classification of soil as per IS soil classification will be
(a) SP-SM
(b) SP-SC
(c) SW-SM
(d) SW-SC

Q.166 A consolidated undrained triaxial compression test was performed on a dense saturated sand at a cell pressure of 80 kPa. The ultimate deviator stress was 270 kPa and the pore pressure at peak stress was 50 kPa (negative). The effective angle of internal friction of soil is ___________ degree.

Q.167 A saturated soil sample has a volume of 23 cm^3 at liquid limit. The shrinkage limit and liquid limit are 18% and 45% respectively. The specific gravity of solids is 2.73. The minimum volume that can be attained by soil is
(a) 12.5 cc
(b) 15.4 cc
(c) 18.7 cc
(d) 20 cc

Q.168 A retaining wall 10 m high retains a cohesionless soil with an angle of internal friction 30°. The surface of the soil is level with the top of the wall. The unit weight of the top 3 m of the soil is 16 kN/m^3 while that of the remaining 7 m of the soil is 20 kN/m^3.
The application of the resultant active thrust (above base) will be at __________ m.

Q.169 An elevated structure with 10,000 kN weight is supported on a tower with four legs. The legs on piers are located the corners of a square of 6 m side. The vertical stress increment due to this loading at the centre of the structure 7 m below ground level will be __________ kN/m^2. Use Boussinesq's equation.

Q.170 A 10 m thick clay layer with single drainage layer undergoes a settlement of 22 mm in 3 years. If the coefficient of consolidation of the layer is 0.2 mm^2/s, then 40 mm consolidation will take an additional time of ________ years. [Expressed to nearest integer value]

Q.171 The in-situ void ratio of a granular soil deposit is 0.4. The maximum and minimum void ratios of the soil were determined to be 0.85 and 0.30 respectively. G_s = 2.67. The ratio of relative density to relative compaction of the deposit is ________.

Q.172 An earth embankment requires 2000 m^3 of soil for construction at a target dry density of 17 kN/m^3. Quotations from three different vendors are shown in the table below along with their distances from site. Assume cost of transportation = ₹10 per km, G = 2.7 and γ_w = 10 kN/m^3.

S.No.	Vendor Name	Void ratio of soil	Rate (₹/m^3)	Distance from site
1.	*A*	0.6	6.0	99.5 km
2.	*B*	0.7	5.5	131 km
3.	*C*	0.8	5.0	175 km

Which vendor would you choose as the most economic solution?
(a) Vendor A
(b) Vendor B
(c) Vendor C
(d) Any of these

Multiple Select Questions (MSQ)

Q.173 While computing the values of limits of consistency and consistency indices, it is found that liquidity index, has a negative value.
Consider the following comments on this value:
1. Liquidity index cannot have a negative value and should be taken as zero.
2. Liquidity index can have a negative value.
3. The soil tested is in semisolid state and stiff.
4. The soil tested is in medium soft state.

Q.174 Consider the following statements:
(a) Secondary consolidation results due to prolonged dissipation of excess hydrostatic pressure.
(b) Primary consolidation happens under expulsion of both air and water from voids in early stages.
(c) Initial consolidation in the case of fully saturated soils is mainly due to compression of solid particles.
(d) Primary consolidation happens more quickly in coarse-grained soils than in fine-grained soils.

Q.175 A soil mass under seepage has a downward flow of water. Which of the following statements are correct with regard to stresses at any point in the soil mass?
(a) Effective stress is decreased by an amount equal to the seepage force
(b) Effective stress is increased by an amount equal to the seepage force
(c) Total stress will change
(d) Total stress will be unaltered

■■■■

Detailed Explanations

151. (a)

Initial effective vertical stress at mid of clay layer,

$$\sigma_0 = \gamma_{sub,\ sand} \times 10 + \gamma_{sub,\ clay} \times 3$$

where,

$$\gamma_{sat,\ clay} = 19.62\ \text{kN/m}^3$$

$$\gamma_{sub,\ clay} = \gamma_{sat,\ clay} - \gamma_w = 19.62 - 9.81 = 9.81\ \text{kN/m}^3$$

$$\gamma_{dry,\ sand} = 17.53\ \text{kN/m}^3$$

$$\gamma_{sat,\ sand} = 19.43\ \text{kN/m}^3$$

$$\gamma_{sub,\ sand} = \gamma_{sat,\ sand} - \gamma_w = 19.43 - 9.81 = 9.62\ \text{kN/m}^3$$

$\therefore$

$$\sigma_0 = 9.62 \times 10 + 9.81 \times 3 = 125.63\ \text{kN/m}^2$$

When water table falls down to elevation = – 6m, then the effective vertical stress (σ_1) at the centre of the clay is

$$\sigma_1 = \gamma_{dry,\ sand} \times 6 + \gamma_{sub,\ sand} \times 4 + \gamma_{sub,\ clay} \times 3$$

$\Rightarrow$

$$\sigma_1 = 17.53 \times 6 + 9.62 \times 4 + 9.81 \times 3 = 173.09 \simeq 173.1\ \text{kN/m}^2$$

The increase in vertical effective pressure,

$$\Delta\sigma = 173.1 - 125.63 = 47.46\ \text{kN/m}^2$$

Settlement in clay layer (ΔH) $= m_V H_0 \Delta\sigma$

$$= \frac{0.02 \times 6 \times 47.46 \times 10^{-4}}{9.81 \times 10^{-3}} = 5.805\,\text{m}$$

152. 2.96 (2.9 to 3.0)

When the infinite slope is subjected to full depth seepage (submerged condition)

Factor of safety, $F = \dfrac{c' + \gamma' H \cos^2\beta \tan\phi}{\gamma_{sat} H \cos\beta \sin\beta}$

Here, $F = 1$ and $H = H_c$

$\therefore$

$$H_C = \frac{c'}{\cos^2\beta(\gamma_{sat}\tan\beta - \gamma'\tan\phi')}$$

$$= \frac{10}{\cos^2 18°\{17 \times \tan 18° - (17 - 9.81)\tan 14°\}} = 2.96\ \text{m}$$

153. 237.08 (235 to 238)

$$B = 4\ \text{m}, \quad L = 6\text{m}, \quad D_f = 1.5\text{m}$$

Since $D_f < B$, it is a shallow footing. Moreover, it is a rectangular footing with $\dfrac{B}{L} = 0.67$.

Now, for rectangular footing, these factors must be corrected by multiplying with shape factors as:

$$N_{c(r)} = \left(1 + 0.3\frac{B}{L}\right)N_c = \left(1 + 0.3 \times \frac{4}{6}\right)21.55 = 25.86$$

$$N_{\gamma(r)} = \left(1-0.2\frac{B}{L}\right)N_\gamma = \left(1-0.2\times\frac{4}{6}\right)\times 7.44 = 6.448$$

$$N_{q(r)} = N_q = 10.16$$

Now, $$q_u = cN_{c(r)} + \gamma D_f N_{q(r)} + \frac{1}{2}\gamma B N_{\gamma(r)}$$

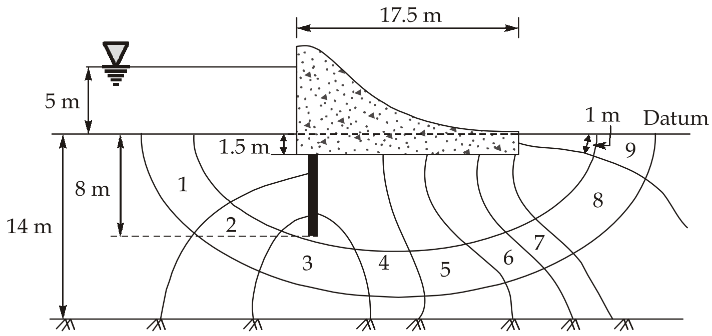
$$= 0 + 16\times 1.5\times 10.16 + \frac{1}{2}\times 16\times 4\times 6.448 \quad (\because c = 0 \text{ for sand})$$

$$= 450.2 \text{ kN/m}^2$$

Safe bearing capacity, $$q_a = \frac{q_u - \gamma D_f}{FOS} + \gamma D_f$$

$$= \frac{450 - 16(1.5)}{2} + 16(1.5) = 237.08 \text{ kN/m}^2$$

154. 0.56 (0.5 to 0.6)



Impervious Strata

The hydraulic gradient at exit, i.e. the exit gradient, can be calculated by considering the flow field nearest to the toe of the dam.

$$\text{Exit gradient} = \frac{\text{Head loss during flow across the last flow field}}{\text{Length of last flow field}}$$

Length of last flow field = 1 m

$$\text{Head loss in last flow field} = \frac{\text{Total head loss}}{\text{Number of potential drop}} = \frac{5}{9} = 0.56\text{ m}$$

$$\therefore \quad \text{Exit gradient} = \frac{0.56}{1} = 0.56$$

155. (c)

Given: $w = 15\%$, $n_a = 20\%$, $G = 2.69$

Volume of specimen, $$V = \frac{\pi}{4}(5)^2 \times 10 = 196.35\text{ cm}^3$$

Mass of water, $M_w = 0.15\,M_d$ and $V_a = 0.20\,V$

Now, $V_w = \dfrac{M_w}{\rho_w} = \dfrac{0.15\,M_d}{\rho_w} = \dfrac{0.15(V_s \cdot G\rho_w)}{\rho_w}$

$\Rightarrow \quad V_w = 0.15 \times 2.69\,V_s = 0.4035\,V_s$

and, $V = V_a + V_w + V_s$

$= 0.20\ V + 0.4035\ V_s + V_s$

$\Rightarrow \quad 0.8\ V = 1.4035\ V_s$

$\Rightarrow \quad V_s = 0.57\ V = 0.57 \times 196.35 = 111.92\ \text{cm}^3$

$\therefore \quad M_d = V_s G \rho_w = 111.92 \times 2.69 \times 1 = 301.1\ \text{g}$

Hence, $M_w = 0.15\ M_d = 0.15 \times 301.1\ \text{g} = 45.2\ \text{g}$

156. (b)

Falling head permeability test:

We know, $$K = 2.303\frac{aL}{At}\log_{10}\left(\frac{h_1}{h_2}\right)$$

Take, $$2.303\frac{aL}{A} = C$$

$$\therefore \quad K = \frac{C}{t}\log_{10}\left(\frac{h_1}{h_2}\right)$$

when, $h_1 = 75$ cm, $h_2 = 75 - 12 = 63$ cm, $t = 8$ min $= 480$ sec

$$\Rightarrow \quad \frac{K}{C} = \frac{1}{480}\log_{10}\left(\frac{75}{63}\right)$$

when, $h_1 = 75$ cm, $h_2 = 30$ cm, $t = ?$

$$\therefore \quad \frac{1}{480}\log_{10}\left(\frac{75}{63}\right) = \frac{1}{t}\log_{10}\left(\frac{75}{30}\right)$$

$\Rightarrow \quad t = 2522.58$ sec

$\Rightarrow \quad t \simeq 42$ min

157. (160)

Maximum depth of unsupported excavation i.e., critical depth

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

For pure clay, $\phi = 0$ and thus $k_a = 1$

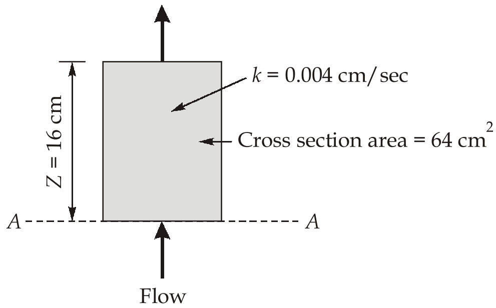
$$\therefore \quad H_C = \frac{4c}{\gamma} = \frac{4 \times 80}{20} = 16\ \text{m}$$

$\therefore$ Active earth pressure at base level of excavation is

$$P_a = k_a\gamma H_c - 2c\sqrt{k_a}$$

$$= 20 \times 16 - 2 \times 80$$
$$= 160 \text{ kN/m}^2$$

158. (a)



In upward seepage flow, effective stress at A-A is

$$\bar{\sigma} = \gamma_{sub} \times Z - P_s \qquad \text{(where } P_s \text{ is seepage pressure which is } iz\gamma_w\text{)}$$

$\because \quad Q = Aki$

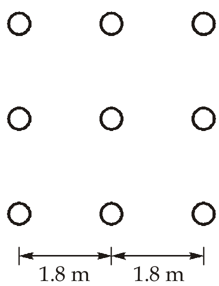
$$\therefore \quad i = \frac{Q}{kA} = \frac{0.08}{0.004 \times 64} = 0.3125$$

$$\therefore \quad \bar{\sigma} = [(20.8 - 9.81) \times 0.16] - [0.3125 \times 9.81 \times 0.16]$$
$$\simeq 1.27 \text{ kN/m}^2$$

159. 149.7 (148 to 150)

When piles act individually,

$$Q_i = \alpha C_u (\pi dl), \; C_u = \frac{q_u}{2} = \frac{6}{2} = 3 \text{ t/m}^2$$
$$= 0.85 \times 3 \times (\pi \times 0.45 \times 12)$$
$$= 43.2 \; t$$



When piles act in a group,

$$Q_g = C_u \times 4 \text{ BL}$$
$$= 3 \times 4 \, (2 \times 1.8 + 0.45) \times 12$$
$$= 583.2 \; t$$

$$\therefore \text{ Efficiency,} \quad \eta = \frac{Q_g}{nQ_i} = \frac{583.2}{9 \times 43.26} \times 100 = 149.7\%$$

160. (c)

Axial strain, $$\in = \frac{16}{80} = 0.2$$

Actual area of the specimen at failure is

$$A_f = \frac{A_o}{1-\in} = \frac{\frac{\pi}{4}\times 40^2}{1-0.2} = 1570.80 \text{ mm}^2$$

Unconfined compressive strength of clay sample is

$$q_u = \frac{P_f}{A_f} = \frac{32\times 10^{-3}}{1570.80\times 10^{-6}} = 20.37 \text{ kN/m}^2$$

∴ Undrained shear strength is given by

$$C_u = \frac{q_u}{2} = \frac{20.37}{2} \simeq 10.19 \text{ kN/m}^2$$

161. (b)

Given: $e_0 = 0.85$, $e_f = 0.73$, $\Delta\sigma_0 = (2-1)\text{ kg/cm}^2 = 1\text{ kg/cm}^2$

$\Delta e = 0.85 - 0.73 = 0.12$, $k = 3.3\times 10^{-4}$ cm/sec

∴ Coefficient of volume change,

$$m_v = \frac{\Delta e}{(1+e_0)}\times\frac{1}{(\Delta\sigma_0)} = \frac{0.12}{(1+0.85)}\times\frac{1}{1}$$

$$m_v = 0.065 \text{ cm}^2/\text{kg}$$

∴ Coefficient of consolidation,

$$C_v = \frac{k}{(m_v\gamma_w)} = \frac{3.3\times 10^{-4}}{(0.065\times 10^{-3}\times 1)} = 5.08 \text{ cm}^2/\text{sec}$$

162. 3.00 (2.9 to 3.1)

Given,

Coefficient of permeability, $k = 3\times 10^{-7}$ m/s

Thickness of soil stratum, $L = 3$ m

Porosity, $n = 40\% = 0.40$

Bulk unit weight, $\gamma_b = 21 \text{ kN/m}^3$

Moisture content, $w = 0.31$

∴ Void ratio, $$e = \frac{n}{1-n} = \frac{0.40}{1-0.40} = \frac{2}{3}$$

$$\gamma = \frac{G\gamma_w}{1+e}(1+w)$$

$$\Rightarrow \quad 21 = \frac{G\times 10}{1+\frac{2}{3}}(1+0.31)$$

$$\Rightarrow \quad G = 2.6717$$

Critical hydraulic gradient, $i_c = \dfrac{G-1}{1+e} = \dfrac{2.67-1}{1+0.67} = 1$

Critical discharge, $Q_c = k\,i_c\,A$

$\therefore$ Flow, $V_c = \dfrac{Q_c}{A} = \dfrac{k i_c A}{A} = k i_c$

$= (3 \times 10^{-7})\ (1) = 3 \times 10^{-7}\ \text{m}^3/\text{s}/\text{m}^2$

So, flow required to maintain critical condition will be $3.00 \times 10^{-7}\ \text{m}^3/\text{s}/\text{m}^2$ area of soil stratum.

163. 3.77 (3.7 to 3.9)

$$w = \left[\frac{(w_2 - w_1)}{(w_3 - w_4)}\left(\frac{G-1}{G}\right) - 1\right] \times 100$$

$$= \left[\frac{(12-7)}{(20-17)}\left(\frac{2.65-1}{2.65}\right) - 1\right] \times 100$$

$$= 3.77\%$$

164. 53.17 (52 to 54)

$$\frac{S_f}{S_p} = \left(\frac{B_f}{B_p} \times \frac{B_p + 0.3}{B_f + 0.3}\right)^2$$

$$\Rightarrow \quad \frac{40}{20} = \left(\frac{B_f}{0.3} \times \frac{0.3 + 0.3}{B_f + 0.3}\right)^2$$

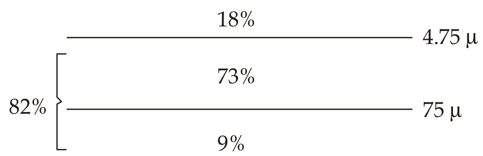
$$\Rightarrow \quad B_f = 725 \text{ mm}$$

Now, $q_{nf} = q_{up}\left(\dfrac{B_f}{B_p}\right) = 22 \times \dfrac{725}{300}$

$= 53.17\ \text{t/m}^2$

165. (d)

Give results shows



Therefore, the soil is predominantly sand. As fines lie between 5% and 12%, this soil will be classified by dual symbol representation.

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.12}{0.11} = 10.18 \qquad (\therefore\ C_u > 6)$$

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} = \frac{0.45^2}{1.12 \times 0.11} = 1.64 \qquad (\therefore\ 1 < C_c < 3)$$

Therefore sand is well graded.

Plasticity index, $I_P = w_L - w_P$
$= 22 - 12 = 10\% > 7\%$

Equation of A-line, $I_P = 0.73\,(w_L - 20)$
$= 0.73\,(22 - 20) = 1.46$

$\therefore$ Soil lies above A-line i.e. it contains clay.

Soil is SW – SC : Well graded sand containing clay in sand.

166. 30.63 (30.5 to 31)

For saturated sand, $C = 0, \quad C' = 0$

$$\sigma_3 = 80\ \text{kN/m}^2, \qquad \sigma_d = 270\ \text{kN/m}^2$$
$$u = -50\ \text{kN/m}^2$$
$$\sigma_1 = \sigma_3 + \sigma_d = 80 + 270 = 350\ \text{kN/m}^2$$
$$\sigma'_1 = \sigma_1 - u = 350 - (-50) = 400\ \text{kN/m}^2$$
$$\sigma'_3 = \sigma_3 - u = 80 - (-50) = 130\ \text{kN/m}^2$$
$$\sin\phi' = \frac{\sigma'_1 - \sigma'_3}{\sigma'_1 + \sigma'_3}$$

where ϕ' = effective internal friction of soil

$$\therefore \quad \sin\phi' = \frac{400 - 130}{400 + 130} = \frac{270}{530}$$
$$\phi' = 30.63°$$

167. (b)

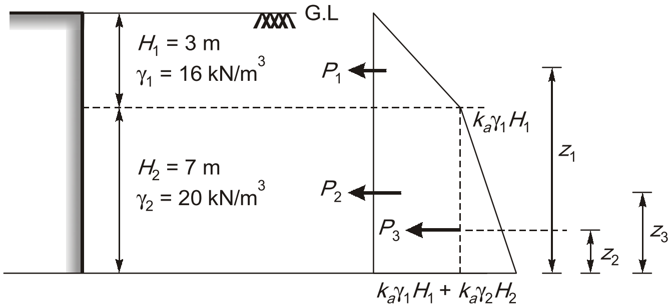
At liquid limit,
$$w_S = \frac{1}{G_D} - \frac{1}{G}$$
$$w_S = \frac{1}{R} - \frac{1}{G}$$
$$w_S = \frac{1}{\left(\dfrac{\dfrac{V_L - V_S}{V_S} \times 100}{w_L - w_S}\right)} - \frac{1}{G}$$
$$0.18 = \frac{1}{\left(\dfrac{\dfrac{23 - V_S}{V_S} \times 100}{45 - 18}\right)} - \frac{1}{2.73}$$
$$V_S = 15.4\ \text{cc}$$

168. 3.22 (3.1 to 3.3)

Given, $\phi = 30°, \quad H_1 = 3$ m,
$H_2 = 7$ m, $\quad \gamma_1 = 16\ \text{kN/m}^3$
$\gamma_2 = 20\ \text{kN/m}^3$

Coefficient of active earth pressure,

$$k_a = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = \frac{1-\frac{1}{2}}{1+\frac{1}{2}} = \frac{1}{3}$$



Retaining wall profile   Earth pressure diagram

Pressure per unit length of wall,

$$P_1 = \frac{1}{2}k_a\gamma_1 H_1^2 = \frac{1}{2}\times\left(\frac{1}{3}\right)\times 16\times 3^2 = 24\,\text{kN/m}$$

$$P_2 = (k_a\gamma_1 H_1)\times H_2 = \left(\frac{1}{3}\times 16\times 3\right)\times 7 = 112\text{ kN/m}$$

$$P_3 = \frac{1}{2}\times k_a\gamma_2 H_2^2 = \frac{1}{2}\times\left(\frac{1}{3}\right)\times(20)\times 7^2 = 163.33\text{ kN/m}$$

Lever arms above base, $z_1 = 7+\frac{1}{3}\times 3 = 8\,\text{m}$

$$z_2 = \frac{7}{2} = 3.5\text{ m}$$

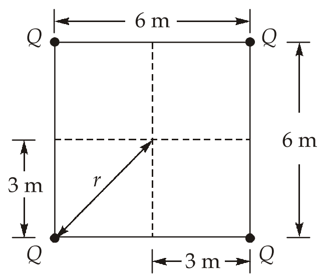
$$z_3 = \frac{7}{3} = 2.33\text{ m}$$

Distance of application of resultant active thrust,

$$z = \frac{P_1 z_1 + P_2 z_2 + P_3 z_3}{P_1 + P_2 + P_3}$$

$$= \frac{24\times 8 + 112\times 3.5 + 163.33\times 2.33}{24+112+163.33} = 3.22\text{ m above base}$$

169. 44.57 (42 to 47)



Load on each pier $\quad Q = \dfrac{10000}{4} = 2500 \text{ kN}$

Horizontal radial distance from each pier to the centre of the structure is

$$r = \sqrt{3^2+3^2} = 3\sqrt{2} = 4.243 \text{ m}$$

At z = 7 m, vertical stress due to load from all piers.

$$\sigma_z = 4\frac{Q}{z^2}I_B$$

$$I_B = \frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{\frac{5}{2}} = \frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{4.243}{7}\right)^2}\right]^{\frac{5}{2}} = 0.2184$$

$$\therefore \quad \sigma_z = \frac{4\times2500\times0.2184}{7^2} = 44.57 \text{ kN/m}^2$$

170. (10)

Let at 22 mm consolidation, degree of consolidation is $U \le 60\%$

$$T_v = \frac{\pi}{4}U^2$$

Also, $\quad T_v = \dfrac{C_v t}{d^2}$

$$\Rightarrow \quad T_v = \frac{C_v t}{d^2} = \frac{0.2\times(3\times365\times24\times60\times60)}{(10\times1000)^2}$$

$$= 0.189216 < 0.283 \qquad (\therefore U < 60\%)$$

$$\because \quad T_v = \frac{\pi}{4}U^2$$

$$\Rightarrow \quad U = 49.1\%$$

We know, degree of consolidation is given by

$$U = \frac{\text{Settlement at time 't'}(S_t)}{\text{Final settlement }(S_f)} = \frac{22}{S_f}$$

$$\Rightarrow \quad S_f = \frac{22}{0.491} = 44.82 \text{ mm}$$

Now for 40 mm consolidation,

$$U = \frac{40}{44.82}\times100 = 89.25\% \ (> 60\%)$$

$$\therefore \quad T_v = 1.781 - 0.9332\log_{10}(100-U)$$

$$= 1.781 - 0.9332\log_{10}(100-89.25) = 0.8186$$

Time required, $\quad t = \dfrac{T_v d^2}{C_v} = \dfrac{0.8186\times(10\times1000)^2}{0.2}$

$$= 12.979 \text{ years} \simeq 13 \text{ years}$$

Additional time $= 13 - 3 = 10$ years

171. 0.88 (0.8 to 0.9)

$$e_{nat} = 0.4$$
$$e_{max} = 0.85$$
$$e_{min} = 0.3$$

Relative density, $I_D = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}} \times 100 = \frac{0.85 - 0.4}{0.85 - 0.3} \times 100 = 81.82\%$

$\because$ Relative compaction $= \frac{\gamma_{d(\text{in-situ})}}{\gamma_{d(\max)}}$

$$\gamma_{d\,(\text{in-situ})} = \frac{G_s \gamma_w}{1 + e_{in-situ}} = \frac{2.67 \times 9.81}{1 + 0.4} = 18.709 \text{ kN/m}^3$$

$$\gamma_{d(\max)} = \frac{G_s \gamma_w}{1 + e_{\min}} = \frac{2.67 \times 9.81}{1 + 0.3} = 20.148 \text{ kN/m}^3$$

$\Rightarrow$ Relative compaction $= \frac{18.709}{20.148} \times 100 = 92.858\%$

$\therefore$ Required ratio $= \frac{81.82}{92.858} = 0.88$

172. (d)

Dry density for embankment,

$$\gamma_d = 17 \text{ kN/m}^3$$

Weight of soil solids required,

$$W_s = \gamma_d \times \text{Volume} = 17 \times 2000 = 34000 \text{ kN}$$

Void ratio is given by,

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

$$\Rightarrow \quad e + 1 = \frac{V_v}{V_s} + 1 = \frac{V_v + V_s}{V_s} = \frac{V}{V_s}$$

$\Rightarrow$ Total volume required, $V = V_s(e+1) = \frac{W_s}{G\gamma_w}(e+1)$

Vendor A :

Volume required, $V_A = \frac{34000(0.6+1)}{2.7 \times 10} = 2014.81 \text{ m}^3$

Cost of soil = 2014.81 × 6 = ₹12088.86

Total cost = 12088.86 + 99.5 × 10

= ₹13083.86

Vendor B :

Volume required, $V_B = \frac{34000(0.7+1)}{2.7 \times 10} = 2140.74 \text{ m}^3$

Cost of soil = 2140.74 × 5.5 = ₹ 11774.07

Total cost = 11774.07 + 131 × 10 = ₹ 13084.07

Vendor C :

Volume required, $V_C = \dfrac{34000(0.8+1)}{2.7 \times 10} = 2266.67\ m^3$

Cost of soil $= 2266.67 \times 5.0 =$ ₹11333.33

Total cost $= 11333.33 + 175 \times 10$

$=$ ₹13083.33

As final cost of soil from all the vendors is almost the same, we can choose any vendor.

173 **(b, c)**

174. **(c, d)**

Secondary consolidation is due to readjustment of soil particles.

Primary consolidation happens due to expulsion of excess pore water.

175. **(b, d)**

In the case of downward flow of water the pore pressure is decreased and effective stress increased. Total stress remains unaltered in the flow.

■■■■