DETAILED SOLUTIONS

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ESE 2025 : Prelims Exam
CLASSROOM TEST SERIES

CIVIL ENGINEERING

Test 4

Section A : Solid Mechanics [All Topics]
Section B : Geo-technical & Foundation Engineering-I [Part Syllabus]
Section C : Environmental Engineering-I [Part Syllabus]

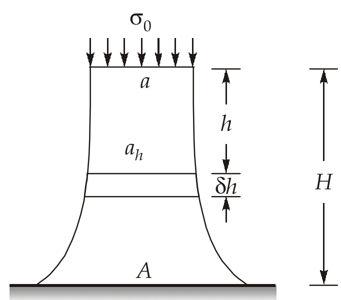
1.	(b)	16.	(d)	31.	(b)	46.	(c)	61.	(c)
2.	(c)	17.	(c)	32.	(d)	47.	(c)	62.	(c)
3.	(c)	18.	(b)	33.	(c)	48.	(c)	63.	(d)
4.	(d)	19.	(c)	34.	(c)	49.	(b)	64.	(c)
5.	(c)	20.	(c)	35.	(c)	50.	(d)	65.	(a)
6.	(d)	21.	(c)	36.	(c)	51.	(d)	66.	(d)
7.	(b)	22.	(c)	37.	(c)	52.	(c)	67.	(d)
8.	(b)	23.	(c)	38.	(b)	53.	(c)	68.	(c)
9.	(a)	24.	(b)	39.	(c)	54.	(d)	69.	(d)
10.	(b)	25.	(*)	40.	(b)	55.	(d)	70.	(c)
11.	(c)	26.	(d)	41.	(a)	56.	(b)	71.	(b)
12.	(b)	27.	(a)	42.	(d)	57.	(d)	72.	(c)
13.	(b)	28.	(c)	43.	(b)	58.	(b)	73.	(d)
14.	(b)	29.	(b)	44.	(c)	59.	(b)	74.	(b)
15.	(c)	30.	(d)	45.	(c)	60.	(a)	75.	(b)

***Q.25 :** *Marks will be awarded to all. [None of the options are correct.]*

DETAILED EXPLANATIONS
Section A : Solid Mechanics

1. (b)

Consider, an element of length 'δh' at distance 'h' from top and having area a_h as shown in figure.



$$\text{Force at depth, } h = \sigma_0 a_h$$

$$\text{Force at depth } h + \delta h = \sigma_0(a_h + \delta a_h)$$

So, $$\text{Weight of element} = \sigma_0(a_h + \delta a_h) - \sigma_0 a_h$$

$$= \sigma_0 \delta a_h \quad \text{...(i)}$$

But $$\text{weight of element} = a_h \delta h \rho \quad \text{...(ii)}$$

Equating (i) and (ii)

$$\sigma_0 \delta a_h = a_h \delta h \rho$$

$$\Rightarrow \quad \frac{\delta a_h}{a_h} = \frac{\rho}{\sigma_0}\delta h \quad \text{...(iii)}$$

Integrating (iii), we get

$$\ln a_h = \frac{\rho}{\sigma_0}h + C$$

At $h = 0$, $a_h = a$

$\therefore$ $C = \ln a$

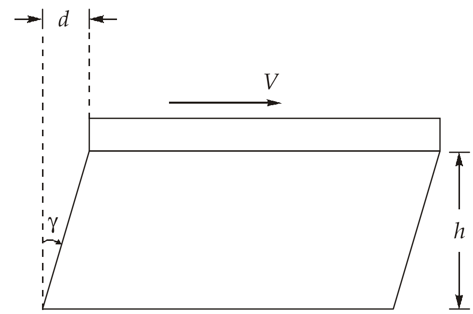
$$\therefore \quad \ln a_h = \frac{\rho}{\sigma_0}h + \ln a$$

$$\Rightarrow \quad a_h = ae^{\rho\frac{h}{\sigma_0}}$$

At $h = H$, $a_h = A$

$$\therefore \quad A = ae^{\rho\frac{H}{\sigma_0}}$$

2. **(c)**



$$\text{Shear stress on plate, } \tau = \frac{V}{ab}$$

$$\text{Shear strain on plate, } \gamma = \frac{\tau}{G}$$

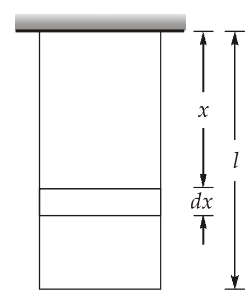
$$= \frac{V}{abG}$$

$$\therefore \quad \text{Horizontal displacement, } d = \gamma h \quad \text{(If } \gamma \text{ is very small)}$$

$$= \frac{Vh}{abG}$$

3. **(c)**

4. **(d)**



Consider an element of length 'dx' at x distance from top.

Now,axial force acting on element, $P_x = \gamma A(l - x)$

$$\text{Now, strain energy stored in bar, } U = \int_0^l \frac{P_x^2 dx}{2AE}$$

$$= \int_0^l \frac{[\gamma A(l-x)]^2 dx}{2AE}$$

$$= \frac{\gamma^2 A}{2E}\int_0^l (l-x)^2 dx$$

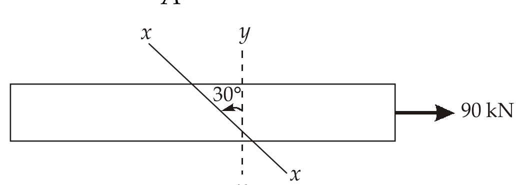
$$= \frac{\gamma^2 A}{2E}\left[l^2 x + \frac{x^3}{3} - lx^2\right]_0^l$$

$$= \frac{\gamma^2 A}{2E}\left[l^3 + \frac{l^3}{3} - l^3\right]$$

$$= \frac{\gamma^2 A l^3}{6E}$$

5. **(c)**

$$\text{Normal stress on plane } y\text{-}y, \ \sigma = \frac{P}{A}$$



$$= \frac{90 \times 10^3}{1200} = 75 \text{ N/mm}^2$$

Now, normal stress on plane x-x $= \sigma\cos^2\theta$

$$= 75 \times \cos^2 30° = 56.25 \text{ MPa}$$

6. **(d)**

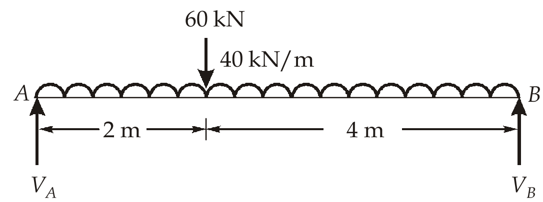
An element in plane stress can never be in state of plane strain except when stresses acting on element are in-plane stress and are equal and opposite i.e. $\sigma_x = -\sigma_y$.

7. **(b)**

8. **(b)**

SFD is one degree higher than loading intensity diagram and BMD is one degree higher than SFD.

9. **(a)**



From the figure $\Sigma F_y = 0$

$\Rightarrow$ $V_A + V_B - 60 - 40 \times 6 = 0$

$\Rightarrow$ $V_A + V_B = 300 \text{ kN}$ ...(i)

$\Sigma M_B = 0$

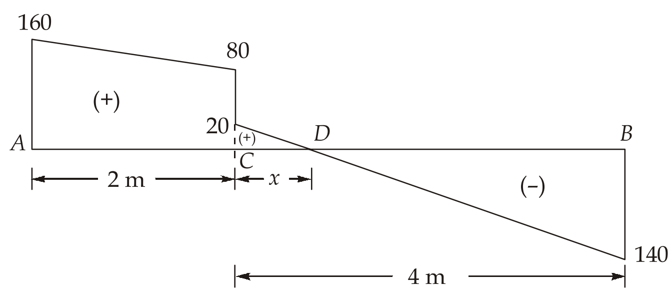
$\Rightarrow$ $V_A \times 6 - 40 \times 6 \times \frac{6}{2} - 60 \times 4 = 0$

$\Rightarrow$ $V_A \times 6 - 720 - 240 = 0$

$\Rightarrow$ $V_A = \frac{960}{6} = 160 \text{ kN}$

$\therefore$ $V_B = 300 - 160 = 140 \text{ kN}$

SFD is shown below.



Let, point D is at x distance from C.

Now, $\frac{x}{4-x} = \frac{20}{140}$

$\Rightarrow$ $x = 0.5 \text{ m}$

So, maximum shear force, V_{max} is at support A.

10. (b)

Maximum bending moment occurs where shear force changes sign i.e. at D.

Now, bending moment at D, BMD = 160 × 2.5 – 40 × 2.5 × 1.25 – 60 × 0.5

$$= 400 - 125 - 30$$

$$= 245 \text{ kN-m}$$

11. (c)

Maximum shear stress for a rectangular section is

$$\tau_{max} = \frac{3}{2} \times \frac{V_{max}}{bd}$$

$$= \frac{3}{2} \times \frac{160 \times 10^3}{250 \times 480} = 2 \text{ N/mm}^2$$

12. (b)

$$\text{Elastic section modulus, } Z = \frac{bh^2}{6}$$

$$= \frac{250 \times 480^2}{6} = 9.6 \times 10^6 \text{ mm}^3$$

$$\therefore \quad \text{Maximum bending stress, } \sigma_{max} = \frac{M_{max}}{Z}$$

$$= \frac{245 \times 10^6}{9.6 \times 10^6} = 25.5 \text{ N/mm}^2$$

13. (b)

Diameter of Mohr's circle represents maximum shear strain.

14. (b)

Column should be axially loaded.

15. (c)

$$\text{Minimum moment of inertia, } I_{min} = \frac{200 \times 120^3}{12}$$

$$= 28.8 \times 10^6 \text{ mm}^4$$

Now, Critical load, $P_{cr} = \dfrac{\pi^2 EI}{L_e^2}$

Where $L_e = \dfrac{L}{\sqrt{2}} = \dfrac{4000}{\sqrt{2}}$ mm

[$\because$ One end is fixed and other end is hinged]

$$\therefore \quad P_{cr} = \frac{\pi^2 \times 2 \times 10^5 \times 28.8 \times 10^6 \times 2}{(4000)^2}$$

$$= \frac{\pi^2 \times 4 \times 28.8 \times 10^{11}}{16 \times 10^6} \simeq 7106 \text{ kN}$$

16. (d)

Let the spring is subjected to an axial load P,

$\therefore$ Torque, $T = P \times R$

Now, the strain energy stored in a spring is given by,

$$U = \frac{T^2 L}{2GI_P} = \frac{(PR)^2 \times 2\pi Rn}{2G \times \frac{\pi d^4}{32}} = \frac{32P^2R^3n}{Gd^4}$$

Axial deflection of spring under load P, $\Delta = \dfrac{\partial U}{\partial P}$

$$= \frac{64PR^3n}{Gd^4}$$

Now, stiffness of spring, $k = \dfrac{P}{\Delta} = \dfrac{P}{\frac{64PR^3n}{Gd^4}} = \dfrac{Gd^4}{64R^3n}$

17. (c)

18. (b)

Moment of inertia about neutral axis, $I = \dfrac{200 \times 400^3}{12}$

$$= 1066.67 \times 10^6 \text{ mm}^4$$

Now, moment of inertia of shaded portion about N.A,

$$I_0 = \frac{200 \times 100^3}{12} + 200 \times 100 \times 150^2 = 466.67 \times 10^6 \text{ mm}^4$$

Now, moment resisted by shaded portion = $\dfrac{M \times I_0}{I}$

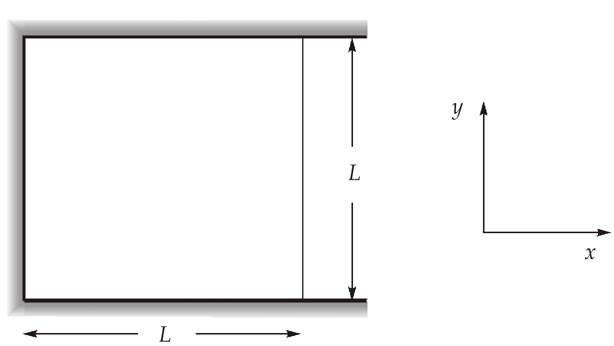
$$= 20 \times \frac{466.67 \times 10^6}{1066.67 \times 10^6} = 8.75 \text{ kN-m}$$

19. (c)

Free expansion in x-direction $= L\alpha T$

Now,compression stress in y-direction due to temperature rise,

$$\sigma_c = E\alpha T$$



So, expansion in x-direction due to σ_c i.e.

$$\delta_x = \frac{\mu\sigma_c}{E}\times L$$

$$= \mu L\alpha T$$

Hence, expansion along x-direciton $= L\alpha T + \mu L\alpha T$

$$= L\alpha T(1 + \mu)$$

20. (c)

Shear force at any section $X - X$, $V_x = \dfrac{dM}{dx}$

$$= \frac{-wx^2}{2L} - wx + \frac{2wL}{3}$$

Now, $V_x = 0$

$$\Rightarrow \quad \frac{-wx^2}{2L} - wx + \frac{2wL}{3} = 0$$

$$\Rightarrow \quad -3x^2 - 6Lx + 4L^2 = 0$$

$$\Rightarrow \quad \frac{3x^2}{L^2} + \frac{6x}{L} - 4 = 0$$

$$\therefore \quad \frac{x}{L} = \frac{-6 \pm \sqrt{6^2 - 4\times 3\times(-4)}}{2\times 3}$$

$$= \frac{-6 \pm \sqrt{36+48}}{6} = \frac{-6 \pm \sqrt{84}}{6}$$

$$= -1 + \frac{\sqrt{84}}{6} \quad \text{(Neglecting – ve sign)}$$

$$= -1 + 1.527 = 0.527$$

$$\therefore \quad x = 0.527\,L$$

21. (c)

Angle of twist at free end, $\theta = \dfrac{TL}{GI_P}$

where, $T = 2$ kN-m $= 2 \times 10^6$ N-mm

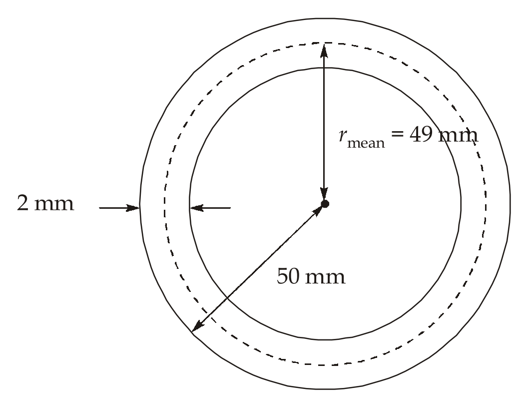
$L = 1000$ mm

$G = 78 \times 10^3$ N/mm^2

$I_P = 2\pi r_{\max}^3 t = 2\pi \times 49^3 \times 2$

$$\therefore \quad \theta = \frac{2\times 10^6 \times 1000}{78\times 10^3 \times 2\pi \times 49^3 \times 2}$$

$= 0.017$ radians



22. (c)

$$\text{Maximum principal stress, } \sigma_1 = \frac{16}{\pi D^3}\left[M + \sqrt{M^2 + T^2}\right]$$

$$\text{Minimum principal stress, } \sigma_2 = \frac{16}{\pi D^3}\left[M - \sqrt{M^2 + T^2}\right]$$

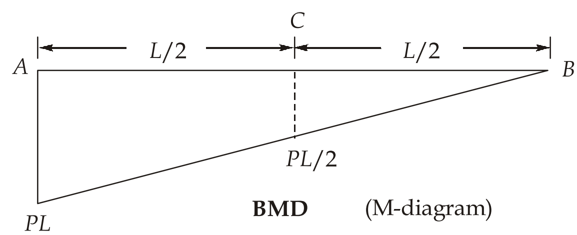
Now, maximum shear stress,
$$\tau_{max} = \frac{\sigma_1 - \sigma_2}{2}$$

$$= \frac{16}{\pi D^3}\left[\frac{\sqrt{M^2+T^2} + \sqrt{M^2+T^2}}{2}\right]$$

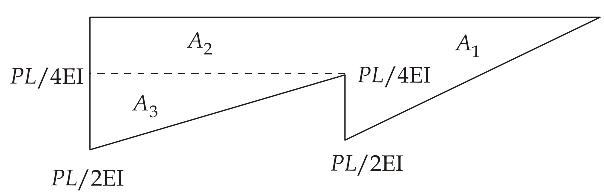
$$= \frac{16}{\pi D^3}\left[\sqrt{M^2+T^2}\right]$$

23. (c)

BMD of beam is shown below



BMD (M-diagram)



***M/EI* diagram**

Using Area-Moment I[st] theorem,

$$\theta_B - \theta_A = A_1 + A_2 + A_3 \qquad (\because \theta_A = 0)$$

$$\Rightarrow \quad \theta_B = \frac{1}{2}\times\frac{PL}{2EI}\times\frac{L}{2} + \frac{PL}{4EI}\times\frac{L}{2} + \frac{1}{2}\times\left(\frac{PL}{2EI} - \frac{PL}{4EI}\right)\times\frac{L}{2}$$

$$\theta_B = \frac{PL^2}{8EI} + \frac{PL^2}{8EI} + \frac{PL^2}{16EI} = \frac{5PL^2}{16EI}$$

24. (b)

$$\because \quad y = \frac{-1}{EI}w_0\left(\frac{L}{\pi}\right)^4 \sin\frac{\pi x}{L}$$

$$\therefore \quad \frac{dy}{dx} = \frac{-1}{EI}w_0\left(\frac{L}{\pi}\right)^4 \times \cos\frac{\pi x}{L}\times\frac{\pi}{L}$$

$$\frac{d^2y}{dx^2} = \frac{-1}{EI}w_0\times\left(\frac{L}{\pi}\right)^4\times\left(-\sin\frac{\pi x}{L}\right)\times\frac{\pi^2}{L^2}$$

$$\frac{d^3y}{dx^3} = \frac{-1}{EI}w_0\times\left(\frac{L}{\pi}\right)^4\times\left(-\cos\frac{\pi x}{L}\right)\times\frac{\pi^3}{L^3}$$

$$\frac{d^4y}{dx^4} = \frac{-1}{EI} w_0 \times \left(\frac{L}{\pi}\right)^4 \times \sin\frac{\pi x}{L} \times \frac{\pi^4}{L^4}$$

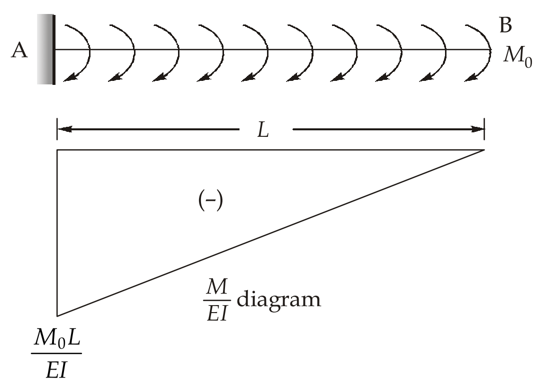
$\Rightarrow$ $$w_x = -EI\frac{d^4y}{dx^4} = w_0 \sin\frac{\pi x}{L}$$

At $$x = \frac{L}{4}, w = \frac{w_0}{\sqrt{2}}$$

25. (*)

Applying area moment theorem,

$$\theta_B - \theta_A = \frac{1}{2} \times \frac{M_o L}{EI} \times L$$



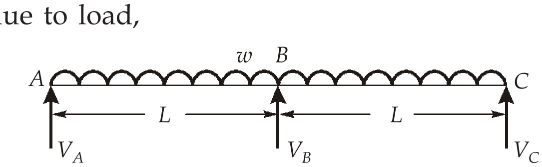
$$\theta_B = \frac{M_o L^2}{2EI} \qquad [\because \theta_A = 0]$$

Also, $$\delta_B - \delta_A = \frac{1}{2} \times \frac{M_o L}{EI} \times \frac{2L}{3} \times L = \frac{M_o L^3}{3EI} \qquad [\because \delta_A = 0]$$

$\Rightarrow$ $$\delta_B = \frac{M_o L^3}{3EI}$$

26. (d)

Downward deflection B due to load,



$$\delta' = \frac{5}{384}\frac{w(2L)^4}{EI} = \frac{5}{24}\frac{wL^4}{EI}$$

Upward deflection at B due to reaction at B,

$$\delta'' = \frac{V_B (2L)^3}{48EI} = \frac{V_B L^3}{6EI}$$

Now, $\delta' = \delta''$ [As deflection is zero at B]

$$\Rightarrow \quad \frac{5}{24}\frac{wL^4}{EI} = \frac{V_B L^3}{6EI}$$

$$\Rightarrow \quad V_B = \frac{5}{4}wL$$

Now, $V_A = V_C = \frac{1}{2}\left[w \times 2L - \frac{5}{4}wL\right]$ [$\because$ Due to symmetry]

$$= \frac{1}{2}\left[2wL - \frac{5}{4}wL\right] = \frac{3}{8}wL$$

27. (a)

By principle of superposition,

$$\text{Slope at } A,\ \theta_A = \frac{M_A L}{3EI} + \frac{M_B L}{6EI}$$

$$\text{Slope at } B,\ \theta_B = \frac{M_A L}{6EI} + \frac{M_B L}{3EI}$$

Now, $$\frac{\theta_A}{\theta_B} = \frac{\frac{M_A L}{3EI} + \frac{M_B L}{6EI}}{\frac{M_A L}{6EI} + \frac{M_B L}{3EI}} = 1.5$$

$$\Rightarrow \quad \frac{2M_A + M_B}{M_A + 2M_B} = 1.5$$

$$\Rightarrow \quad 2M_A + M_B = 1.5\,M_A + 3M_B$$

$$\Rightarrow \quad 0.5M_A = 2\,M_B$$

$$\Rightarrow \quad \frac{M_A}{M_B} = 4$$

28. (c)

High tension steel is more brittle and mild steel is more ductile.

29. (b)

As we know

$$E = 3K(1 - 2\mu)$$

Now, $\mu < 0 < 0.5$

$$\Rightarrow \quad E < 3K$$

$$\Rightarrow \quad \frac{K}{E} > \frac{1}{3}$$

For homogenous and isotropic material, the number of independent elastic constants is 2.

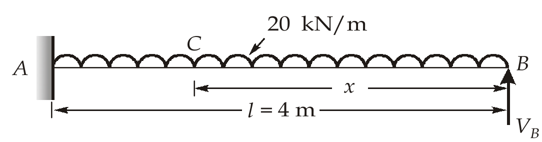
30. (d)

- Under pure normal stress, only volume change may occur.
- Under pure shear stress, only shape change may occur.

31. (b)

Deflection at point B due to UDL and V_B is,

$$\Delta_B = \frac{wl^4}{8EI} - \frac{V_B l^3}{3EI}$$



Now, $\Delta_B = 0$

$\Rightarrow$ $V_B = \frac{3}{8}wl$

$\Rightarrow$ $V_B = \frac{3}{8} \times 20 \times 4 = 30\,\text{kN}$

Let, point of contraflexure is x distance from B at point C.

Now, Bending moment at C (from right) = 0

$\Rightarrow$ $-V_B \times x + \frac{wx^2}{2} = 0$

$\Rightarrow$ $-30x + 20 \times \frac{x^2}{2} = 0$

$\Rightarrow$ $x = 3\text{ m}, 0\text{ m}$

$\therefore$ Point of contraflexure is at 1 m from fixed end.

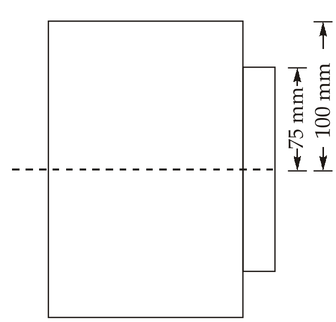
32. (d)

33. (c)

Maximum stress in timber, $\sigma_{max,\,w} = 8\ \text{N/mm}^2$ at 100 mm from NA

Now, stress in timber at 75 mm from NA, $\sigma_w = \frac{8 \times 75}{100} = 6\ \text{N/mm}^2$

So, stress in steel at 75 mm from NA, $\sigma_s = 6 \times 20 = 120\ \text{N/mm}^2$



Now, Moment of resistance, MOR $= \sigma_{max,w} \times \frac{150 \times 200^2}{6} + \sigma_s \times \frac{25 \times 150^2}{6}$

$= 8 \times 1 \times 10^6 + 120 \times 0.09375 \times 10^6$

$= (8 + 11.25) \times 10^6 = 19.25$ kNm

34. (c)

35. (c)

Strain gauge is in direction of major principal strain.

Major principal strain, $\varepsilon_1 = 400 \times 10^{-6}$

Minor principal strain, $\varepsilon_2 = -400 \times 10^{-6}$

$\therefore$ Maximum shear strain, $\phi_{max} = \varepsilon_1 - \varepsilon_2$

$= (400 - (-400)) \times 10^{-6}$

$= 800 \times 10^{-6}$

$\therefore$ Shear modulus, $G = \frac{\tau_{max}}{\phi_{max}} = \frac{20}{800 \times 10^{-6}}$

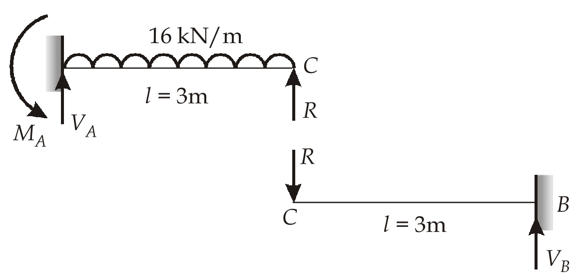
$= 25000$ N/mm^2 = 25 GPa

36. (c)

Let, the reaction at C is R.

Now, deflection at C in beam AC is,

$$\Delta' = \frac{wl^4}{8EI} - \frac{Rl^3}{3EI}$$



$$= \frac{16 \times 3^4}{8EI} - \frac{R \times 3^3}{3EI} = \frac{162}{EI} - \frac{9R}{EI}$$

Deflection at C in beam CB, $\Delta'' = \frac{Rl^3}{3EI} = \frac{R \times 3^3}{3EI} = \frac{9R}{EI}$

Now, $\Delta' = \Delta''$

$\Rightarrow$ $\frac{162}{EI} - \frac{9R}{EI} = \frac{9R}{EI}$

$\Rightarrow$ $R = 9$ kN

Now, $V_A + R = 16 \times 3$

$\Rightarrow$ $V_A = 48 - 9 = 39$ kN

37. (c)

$$\text{Effective length of column, } l_e = \frac{L}{2} \qquad (\because \text{ Both ends are fixed})$$

Axial compression force in column due to rise is temperature by ΔT, $P = E\alpha\Delta T \times A$

Also, Euler's load, $P_{cr} = \frac{\pi^2 EI}{(L_e)^2} = \frac{4\pi^2 EI}{L^2}$

Now, $P = P_{cr}$

$$\Rightarrow \quad E\alpha\Delta TA = \frac{4\pi^2 EI}{L^2}$$

$$\Rightarrow \quad \Delta T = \frac{4\pi^2\left(\frac{\pi}{64}d^4\right)}{L^2 \times \alpha \times \frac{\pi}{4}d^2} = \frac{\pi^2 d^2}{4\alpha L^2}$$

Section B : Geo-technical & Foundation Engineering-I

38. (b)

$$\text{Bulk unit weight of soi, } \gamma_t = \frac{\text{Weight of soil}}{\text{Volume of soil}}$$

where weight of soil $= 2770 - 946$

$= 1824$ gm

So, $\gamma_t = \frac{1824}{1000} = 1.824\,\text{gm/cc}$

Now, $\gamma_t = \frac{G(1+w)\rho_w}{1+e}$

$$\Rightarrow \quad 1.824 = \frac{2.65(1+0.1)\times 1}{1+e}$$

$$\Rightarrow \quad e = \frac{2.65 \times 1.1}{1.824} - 1 = 0.598$$

39. (c)

As we know,

$$w_s = w_L - \frac{(V_L - V_d)\times\rho_w}{M_d}$$

where $w_L = 60\%$, $w_s = 20\%$, $V_L = 40$ cc, $V_d = 23.5$ cc

M_d = Mass of dry soil

$$\therefore \quad 0.2 = 0.6 - \frac{(40 - 23.5)}{M_d}$$

$$\Rightarrow \quad M_d = \frac{16.5}{0.4} = 41.25\,\text{gm}$$

Hence, dry density of soil, $\rho_d = \frac{M_d}{V_d} = \frac{41.25}{23.5}$

Now,specific gravity of soil solids, $G = \frac{1}{\frac{1}{\rho_d} - \frac{w_s}{100}}$

$$\Rightarrow \quad G = \frac{1}{\frac{23.5}{41.25} - \frac{20}{100}}$$

$$= \frac{1}{0.57 - 0.2} = 2.7$$

40. (b)

41. (a)

42. (d)

43. (b)

When a soil is stressed to a level greater than maximum stress to which it was ever subjected in past, a kind of breakdown occurs in soil structure resulting in higher compressibility.

44. (c)

$$\text{Settlement, } \Delta H = \frac{C_r H_0}{1+e_0} \log \frac{\bar{\sigma}_c}{\bar{\sigma}_0} + \frac{C_c H_0}{1+e_0} \log \frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_c}$$

$$= \frac{0.03 \times 3.8}{1+0.9} \log \frac{120}{60} + \frac{0.27 \times 3.8}{1+0.9} \log \frac{150}{120}$$

$$= 0.06 \log 2 + 0.54 \log 1.25$$

45. (c)

Undrained shear strength of clay,

$$C_u = \frac{T}{\pi d^2 \left(\frac{h}{2} + \frac{d}{6}\right)}$$

$$\Rightarrow \quad C_u = \frac{40 \times 1000}{\pi \times 50^2 \left(\frac{100}{2} + \frac{50}{6}\right)} = 0.087 \text{ N/mm}^2$$

$$= 87 \text{ kN/m}^2$$

46. (c)

Since clay is normally consolidated, $C = 0$

$$\bar{\sigma}_{1f} = \sigma_1 - u = 200 + 150 - 75 = 275 \text{ kN/m}^2$$

$$\bar{\sigma}_{3f} = \sigma_3 - u = 200 - 75 = 125 \text{ kN/m}^2$$

Now, $\phi' = \sin^{-1}\left(\frac{\bar{\sigma}_{1f} - \bar{\sigma}_{3f}}{\bar{\sigma}_{1f} + \bar{\sigma}_{3f}}\right) = \sin^{-1}\left(\frac{275 - 125}{275 + 125}\right) = \sin^{-1}\left(\frac{150}{400}\right)$

47. (c)

48. (c)

Failure envelope in terms of effective stress parameters is a straight inclined line. While in terms of total stress parameters, it is a horizontal line.

49. (b)

50. (d)

51. (d)

- Quicksand condition occurs when flow is upward through the soil.
- In clays, the shear stress does not get reduced to zero even when the effective stress becomes zero, owing to its cohesion.
- High artesian pressure in a coarse sand is major reason for development of quicksand condition.

52. (c)

As we know
$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{G\gamma_w}{1+\frac{Gw}{S}}$$

where $G = 2.7$, $\gamma_w = 9.81$ kN/m^3, $S = 0.8$

$\therefore$
$$\gamma_d = \frac{2.7\times 9.81}{1+\frac{2.7\times w}{0.8}} = \frac{26.49}{1+3.38w}$$

53. (c)

Mass specific gravity in dry state, $G_{\text{md}} = \frac{\rho_d}{\rho_w} = 1.8$

$\Rightarrow$ $\rho_d = 1.8$ g/cm^3

Now, void ratio of soil, $e = \frac{G\rho_w}{\rho_d} - 1$

$$= \frac{2.7}{1.8} - 1 = 0.5$$

Now, mass specific gravity in saturated state,

$$G_{ms} = \frac{\rho_{\text{sat}}}{\rho_w}$$

$\Rightarrow$
$$G_{ms} = \frac{G+e}{1+e}$$
$$= \frac{2.7+0.5}{1+0.5} = \frac{3.2}{1.5} = 2.13$$

54. (d)

55. (d)

$$S_S = \frac{H_0 C_\alpha \log(t_2/t_1)}{1+e_f} = \frac{H_0 C_\alpha \Delta \log t}{1+e_f}$$

56. (b)

$$\text{Initial area of soil specimen, } A_0 = 1100 \text{ mm}^2$$

$$\text{Axial strain, } \varepsilon = \frac{15}{100} = 0.15$$

$$\text{Final area of soil specimen, } A_f = \frac{A_0}{1-\varepsilon} = \frac{1100}{1-0.15} = 1294.12 \text{ mm}^2$$

$$\therefore \quad \text{Unconfined compressive strength, } q_u = \frac{30\times10^{-3}}{1294.12\times10^{-6}} = 23.18 \text{ kN/m}^2$$

Section C : Environmental Engineering-I

57. (d)

Schistomiasis also known as sleeping sickness is caused by snails carrying the intermediate vector (parasitic flat worms called schistosomes) for disease. These snails (crustaceans) thrive in clean water canals in many tropical countries and can create a major health hazard.

58. (b)

59. (b)

60. (a)

This process is not suitable for highly turbid waters, because the suspended impurities get deposited around the zeolite particles, and thus cause obstruction to working of zeolite.

61. (c)

$$\text{Daily water demand} = 3\text{MLD}$$

$$\text{Daily chlorine requirement} = 0.3 \times 10^{-6} \times 3 \times 10^{6} = 0.9 \text{ kg}$$

Now,daily requirement of bleaching powder

$$= \frac{0.9}{0.3} = 3 \text{ kg}$$

62. (c)

$$\text{Head loss through bed, } h_{Le} = D(1-n)(G-1)$$
$$= 0.6(1-0.6)(2.65-1)$$
$$= 0.396 \text{ m}$$

63. (d)

- Iron salts produce heavy floc and can therefore remove much more suspended matter than alum.
- Handling and storage of iron salts require more skill and control as they are corrosive and deliquescent.

64. (c)

$$\text{Amount of alum required daily} = 12 \times 10^{-6} \times 15 \times 10^{6} = 180 \text{ kg/day}$$

Now, chemical reaction of alum is given below.

$$Al_2(SO_4)_3 \cdot 18H_2O + 3Ca(HCO_3)_2 \rightarrow 2Al(OH)_3 + 3CaSO_4 + 18H_2O + 6CO_2$$

$$\text{Molecular mass of alum} = 666 \text{ gm}$$

$$\text{Molecular mass of } CO_2 = 44 \text{ gm}$$

Hence, 666 gm of alum releases = 6×44 gm of CO_2

$$\text{180 kg/day of alum will release} = \frac{6 \times 44}{666} \times 180 = 71.35 \text{ kg/day of } CO_2$$

65. (a)

$$\text{Requirement of cloth area} = \frac{20 \times 60}{4} = 300 \text{ m}^2$$

$$\text{Now, area of one bag} = \pi DH = \pi \times 0.5 \times 6 \text{ m}^2$$

So,

$$\text{Number of bags} = \frac{300}{\pi \times 0.5 \times 6} = \frac{100}{\pi}$$

66. (d)

In a settling chamber

$$\frac{V_t}{H} = \frac{V_h}{L}$$

where V_t is terminal settling velocity.

$$\Rightarrow \quad V_t = \frac{V_h H}{L} \quad \text{...(i)}$$

Also,

$$V_t = \frac{g(\rho_P - \rho_a) d_P^2}{18\mu} \quad \text{...(ii)}$$

Equating (i) and (ii), we get

$$\frac{g(\rho_P - \rho_a) d_P^2}{18\mu} = \frac{V_h H}{L}$$

$$\Rightarrow \quad d_p = \left(\frac{18\mu V_h H}{g \rho_P L}\right)^{1/2} \quad [\because \rho_P >>> \rho_a]$$

67. (d)

68. (c)

$$\text{Drawdown at 100 m, } S_{100} = \frac{Q}{4\pi T}W(u)$$

where

$$Q = 1800 \text{ lpm} = 1.8 \text{ m}^3/\text{min}$$

$$T = kB = 30 \times 30 = 900 \text{ m}^2/\text{day} = \frac{900}{24 \times 60} = 0.625 \text{ m}^2/\text{min.}$$

$$W(u) = 3.355$$

$$\therefore \quad S_{100} = \frac{1.8}{4\pi \times 0.625} \times 3.355 = \frac{2.4156}{\pi}\text{m} \simeq \frac{2.42}{\pi}\text{m}$$

69. (d)

70. (c)

71. (b)

Year	Population in thousands	Increase in population	% increase in population	Decrease in % increase
1990	80	40	50	16.67
2000	120	40	33.33	13.33
2010	160	32	20	
2020	192			
Total				30%
Average per decade				$\frac{30}{2} = 15\%$

$$\therefore \text{ Expected population at end of 2030} = 192000 + \left(\frac{20-15}{100}\right) \times 1920000$$

$$= 201600$$

72. (c)

$$\text{Maximum daily draft} = 1.8 \times \text{Average daily draft}$$
$$= 1.8 \times 25 = 45 \text{ MLD}$$
$$\text{Maximum hourly draft} = 2.7 \times \text{Average daily draft}$$
$$= 2.7 \times 25 = 67.5 \text{ MLD}$$

Now,

$$\text{Coincident draft} = \text{Max.}\begin{bmatrix} \text{Maximum daily draft + Fire demand} \\ \text{or} \\ \text{Maximum hourly draft} \end{bmatrix}$$

$$= \text{Max.}\begin{bmatrix} 45+60 \\ \text{or} \\ 67.5 \end{bmatrix} = 105 \text{ MLD}$$

73. (d)

74. (b)

As per Sichardt equation,

$$\text{Radius of influence (in m)} = 3000s\sqrt{k}$$

where s is drawdown in m

k is permeability in m/s

75. (b)

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