ESE-2018 PRELIMS TEST SERIES

Date: 15 October, 2017

CE–TEST 04(OBJECTIVE SOLUTION)...

ANSWERS

1. (d)	31. (a)	61. (b)	91. (b)	121. (c)
2. (a)	32. (b)	62. (b)	92. (b)	122. (d)
3. (c)	33. (b)	63. (d)	93. (b)	123. (c)
4. (d)	34. (b)	64. (c)	94. (d)	124. (d)
5. (b)	35. (a)	65. (a)	95. (c)	125. (a)
6. (b)	36. (d)	66. (c)	96. (d)	126. (b)
7. (b)	37. (c)	67. (a)	97. (c)	127. (c)
8. (c)	38. (d)	68. (c)	98. (b)	128. (d)
9. (b)	39. (a)	69. (d)	99. (d)	129. (a)
10. (c)	40. (c)	70. (c)	100. (d)	130. (c)
11. (c)	41. (b)	71. (c)	101. (b)	131. (d)
12. (b)	42. (d)	72. (b)	102. (d)	132. (d)
13. (b)	43. (c)	73. (a)	103. (b)	133. (a)
14. (b)	44. (c)	74. (b)	104. (a)	134. (a)
15. (a)	45. (a)	75. (d)	105. (d)	135. (c)
16. (d)	46. (b)	76. (b)	106. (b)	136. (a)
17. (b)	47. (d)	77. (a)	107. (c)	137. (a)
18. (c)	48. (b)	78. (c)	108. (d)	138. (c)
19. (c)	49. (a)	79. (c)	109. (b)	139. (a)
20. (c)	50. (c)	80. (a)	110. (c)	140. (c)
21. (a)	51. (b)	81. (b)	111. (c)	141. (b)
22. (b)	52. (a)	82. (c)	112. (d)	142. (d)
23. (d)	53. (c)	83. (d)	113. (a)	143. (d)
24. (b)	54. (a)	84. (a)	114. (c)	144. (c)
25. (b)	55. (c)	85. (b)	115. (b)	145. (d)
26. (a)	56. (a)	86. (b)	116. (a)	146. (b)
27. (c)	57. (b)	87. (a)	117. (d)	147. (a)
28. (c)	58. (a)	88. (c)	118. (d)	148. (c)
29. (a)	59. (b)	89. (c)	119. (d)	149. (b)
30. (a)	60. (b)	90. (a)	120. (b)	150. (d)

1. **(d)**

Magnitude has only dimension so it is zero order tensor. Direction has 3 dimension (x, y and z direction) $3^1 = 1^{st}$ order tensor. Stress has 9 dimension ($3^2 = 2^{nd}$ order tensor)

2. **(a)**

An ideal fluid material can't experience any stress on it. An ideal rigid material does not experience strain because there no elongation in this type of material.

3. **(c)**

Point where deflected shape changes curvature is called point of inflection and BM magnitude at this section is zero.

Shear span is this span where SF value remains constant.

4. **(d)**

$$\text{Flexural stiffness} = \frac{\text{Flexural Rigidity}}{\text{Length}}$$

Thus flexural stiffness is directly proportional to flexural rigidity & inversely proportional to length.

5. **(b)**

In load carrying members load can be transmitted through bearing also. hence statement (1) is incorrect. Statement (2) and (3) are correct.

6. **(b)**

Max shear stress theory is also known as Tresca, Guest, Columb theory.

It is not suitable for hydrostatic loading because

$$\sigma_{max} = \sigma_{min} = 'P'$$

$$\text{Thus} \quad \tau_{max} = \frac{\sigma_{max}-\sigma_{min}}{2} = 0$$

$\Rightarrow$ Max. strain energy theory is called as Beltrami haigh theory

7. **(b)**

- In truss, deflection is caused by only internal axial forces
- In frame, major deflection is caused bending

8. **(c)**

9. **(b)**

$$\text{We know } \frac{dv}{dx} = w$$

$$\int w.dx = V$$

$\therefore$ The integration of load diagram gives shear force.

10. **(c)**

$$M_{eq} = \frac{1}{2}\left[M+\sqrt{M^2+T^2}\right]$$

$$= \frac{1}{2}\left[30+\sqrt{(30)^2+(40)^2}\right]\text{kNm}$$

$$= 40 \text{ kNm}$$

11. **(c)**

12. **(b)**

Depth of water applied = S×d×[F.C.–M.C.]

$$\text{Where, } S = \frac{\gamma_d}{\gamma_w} = 1.42$$

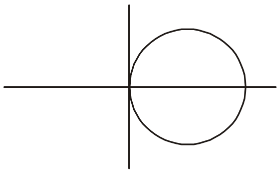
$$\frac{50}{1000} = 1.42\times d\times\left[\frac{12.6}{100}-\frac{8.2}{100}\right]$$

$$d = 800 \text{ mm}$$

13. **(b)**

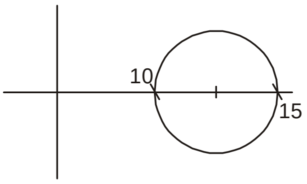
(a) Diameter = (maximum shear stress)

(b)



Here minor principal stress = 0 < τ_{max}

(c)



Diameter $5 \ngtr 10$

(d) In diagram of C at $\tau_{max} = 2.5$ but at that plane

$$\sigma = 12.5 > 2.5$$

14. **(b)**

Assume stiffness of original spring is k.

$$\text{Stiffness} = \frac{Gd^4}{64R^3n}$$

$$K \propto \frac{1}{n}$$

So, stiffness of 2m part wil be $\frac{5}{2}K = 2.5K$

and stiffness of 3m part will be $\frac{5}{3}K$

For parallel combination

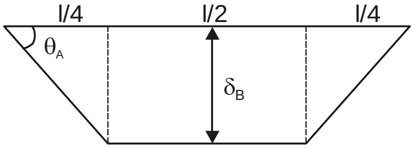
$$K_{eq} = K_1 + K_2$$

$$= \frac{5}{2}K + \frac{5}{3}K = 2.5 + 1.67 = 4.17K$$

$$m = 4.17$$

15. **(a)**

16. **(d)**



$$\boxed{\delta_B = \frac{\theta_A L}{4}}$$

$$U_e = \frac{1}{2}\cdot P\delta_B = \frac{1}{2}P\left(\frac{\theta_A L}{4}\right)$$

$$U_{spring} = 2\times\frac{1}{2}\times M\times\theta_A = K\times\theta_A\times\theta_A$$

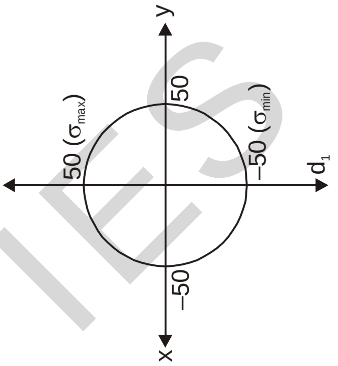
$$\Rightarrow \frac{Pl}{8}\times\theta_A = K.\theta_A.\theta_A$$

$$\Rightarrow \theta_A = \frac{Pl}{8K}$$

$$\delta_B = \frac{Pl^2}{32K}$$

17. **(b)**

Mohr circle of above stress condition.



$\because$ Stress in direction of diagonal d_1 = – 50 (compressive) so reducting takes place

$$e_{min} = \frac{\sigma_{min}}{E} - \mu\frac{\sigma_{max}}{E}$$

$$= -\frac{50}{2\times10^5} - 0.3\times\frac{50}{2\times10^5} = -3.25\times10^{-4}$$

$$e_{min} = \frac{\Delta d_1}{d_1} = -3.25\times10^{-4}$$

% change in length of diagonal

d_1 = – 325 × 10^{-1} × 10^2

% Δd = – 0.0325

18. **(c)**

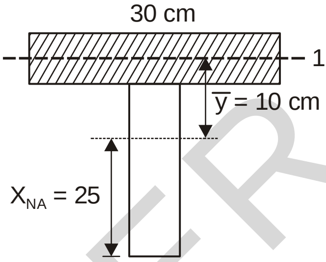
The normal stress in a beam due to bending can be calculated as

$$\sigma_x = -\frac{My}{I}$$

at the neutral axis y = 0

$$\Rightarrow \sigma_x = 0$$

19. **(c)**



$$X_{NA} = \frac{300\times15 + 300\times35}{600} = 25\text{ cm}$$

$$I_{NA} = \frac{1}{12}(10)(30)^3 + (300)(35-25)^2$$

$$+\frac{1}{12}(30)(10)^3 + (300)(25-15)^2$$

$$= 85\times10^3\text{ cm}^4$$

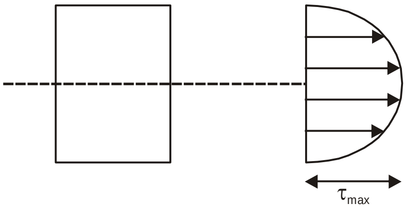
$$\overline{y} = 30 + 5 - 25 = 10\text{ cm}$$

$$Q = A\overline{y} = 10\times30\times10 = 3000\text{ cm}^3$$

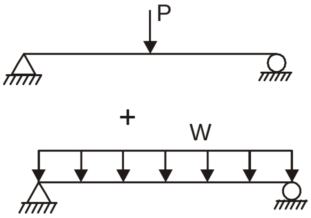
$$q = \frac{(300N)(3000\text{ cm}^3)}{85\times10^3\text{ cm}^4} = 1.058\ \text{kN/m}$$

20. **(c)**

The shear distribution diagram in a cross-section shows that the shear stress is maximum at the neutral axis and zero at edges as shown in figure.



21. **(a)**



$$y_{max} = y_{max(p)} + y_{max(w)}$$

$$y_{max(p)} = -\frac{PL^3}{48EI}$$

$$= \frac{-2000N\times(4m)^3}{48(100\times10^9\ N/m^2)(50\times10^{-8}\ m^4)}$$

$$= -0.0533\,\text{m}$$

$$y_{\max(w)} = -\frac{5wL^4}{384EI}$$

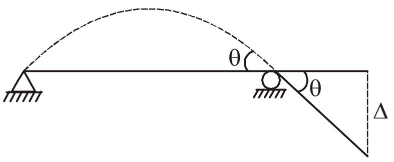
$$= \frac{-5(100)(4)^4}{384(100\times10^9)(50\times10^{-8})}$$

$$= -0.0667\,\text{m}$$

$$y_{\max} = -0.0533 + (-0.0667) = -0.12\ \text{m}$$

$$= -12\ \text{cm}$$

22. **(b)**



$$\theta = \frac{Ml}{3EI} = \frac{(Pl/2)l}{3EI}$$

$$\Delta = \theta.\frac{l}{2} = \frac{Pl^3}{12EI}$$

23. **(d)**

$$\text{Radius of kern} = \frac{D^2+d^2}{8D}$$

$$= \frac{400^2+200^2}{8\times400} = 62.5\,\text{mm}$$

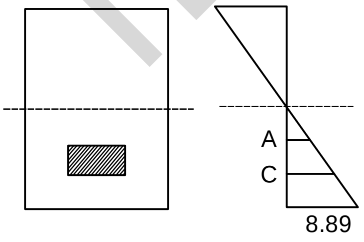
Load is applied at the kern so stress at point B will be zero (from the definition of kern)

24. **(b)**

$$\sigma_{\max} = \frac{My_{\max}}{I} = \frac{20\times10^6\times150}{150\times\dfrac{300^3}{12}}$$

$$= 8.89\ \text{MPa}$$

Figure.



$$\sigma_A = \frac{8.89}{150}\times50 = 2.96\,\text{MPa}$$

$$\sigma_c = \frac{8.89}{150}\times100 = 5.93\,\text{MPa}$$

Tensile force in hatched area

$$= \frac{1}{2}(2.96+5.93)\times50\times50$$

$$= 11.107\ \text{kN}$$

25. **(b)**

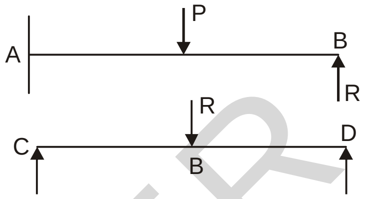
$$\text{Shear flow}\quad q = \frac{T}{2A_m}$$

$$A_m = 110\times110 = 12100\ \text{mm}^2$$

$$z\times t = \frac{T}{2Am}$$

$$z = \frac{2\times10^6}{20\times2\times12100} = 4.13\,\text{MPa}$$

26. **(a)**



Deflection of B will be same for both cases.

$$(\Delta_B)_{\text{cantilever}} = \frac{P(l/2)^3}{3EI} + \frac{P(l/2)^2}{2EI}\times\frac{l}{2} - \frac{Rl^3}{3EI}$$

$$= \frac{Pl^3}{24EI} + \frac{Pl^3}{16EI} - \frac{Rl^3}{3EI}$$

$$(\Delta_B)_{\text{simply supported}} = \frac{Rl^3}{48EI}$$

$$\text{Now,}\quad \frac{Pl^3}{24EI} + \frac{Pl^3}{16EI} - \frac{Rl^3}{3EI} = \frac{Rl^3}{48EI}$$

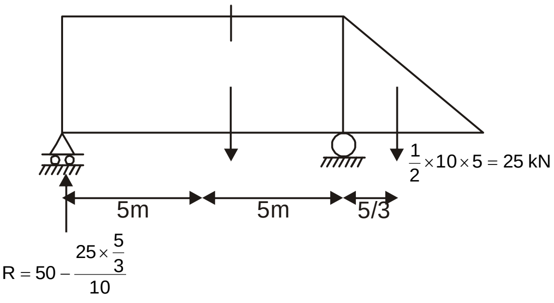
$$\frac{P}{24} + \frac{P}{16} = \frac{R}{3} + \frac{R}{48}$$

$$2P + 3P = 16R + R$$

$$R = \frac{5P}{17}$$

$$R = 10\ \text{kN}$$

27. **(c)**



$$M = R\times5 - \frac{10\times(5)^2}{2}$$

$$= -\frac{625}{30}$$

$$= 125 - \frac{625}{30} = 104.16$$

$$R_A + R_B = 10\times10 + \frac{1}{2}\times5\times10$$

$$= 100 + 25$$

$$R_A + R_B = 125$$

$$\sum MA \uparrow = 0$$

$$R_B \times 10 - \frac{1}{2} \times 5 \times 10 \times \left(10 + \frac{5}{3}\right) \times 10 \times 10 \times 5 = 0$$

$$10R_B - 25(11.67) - 500 = 0$$

$$R_B = 79.167 \text{ kN}$$

$$R_A = 45.83 \text{ kN}$$

$$M_D = R_A \times 5 - 10 \times 5 \times 2.5$$

$$\Rightarrow (45.83 \times 5) - 125 = 104.15 \text{ kNm}$$

28. **(c)**

$$\text{Total load} = \int_0^L W dx = \int_0^L W_0 \sin\left(\frac{x\pi}{L}\right) dx$$

$$= -\frac{W_0 L}{\pi}\left[\cos\left(\frac{\pi x}{L}\right)\right]_0^L$$

$$= -\frac{W_0 L}{\pi}(\cos\pi - \cos 0)$$

$$\text{Total load} = \frac{2W_0 L}{\pi}$$

$$\text{So, Reaction on A} = \frac{\text{Total load}}{2} \Rightarrow \frac{W_0 L}{\pi}$$

29. **(a)**

$$\frac{d^4y}{dx^4} = \frac{W}{EI}$$

$$EI\frac{d^4y}{dx^4} = W_0 \sin\left(\frac{x\pi}{L}\right)$$

$$EI\frac{d^3y}{dx^3} = \frac{-W_0 \sin\left(\frac{x\pi}{L}\right) + C_1}{\left(\frac{\pi}{L}\right)^2}$$

$$EI\frac{d^2y}{dx^2} = \frac{-W_0 \cos\left(\frac{\pi x}{L}\right)}{\left(\frac{\pi}{L}\right)} + C_1 x + C_2$$

$$\therefore \quad EI\frac{d^2y}{dx^2} = M$$

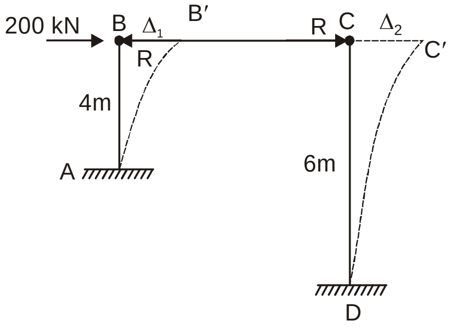
and at $x = 0$, $M = 0$, so $C_2 = 0$

and at $x = L$, $M = 0$, so, $C_1 = 0$

$$\therefore \quad M = \frac{-W_0 \sin\left(\frac{\pi x}{L}\right)}{\left(\frac{\pi}{L}\right)^2}$$

$$\therefore \text{ at } x = \frac{L}{2},\ M = \frac{W_0 L^2}{\pi^2}$$

30. **(a)**



$$\Delta_1 = \Delta_2$$

$$(200 - R)\frac{4^3}{3EI} = \frac{R\,6^3}{3EI}$$

$$\frac{200 - R}{R} = \frac{27}{8}$$

$$27\ R = 1600 - 8R$$

$$35\ R = 1600$$

$$R = 45.71 \text{ kN}$$

$$H_A = 200 - 45.71$$

$$H_A = 154.29 \text{ kN}$$

31. **(a)**

$$\sigma = \frac{4 \times 1000}{\frac{\pi}{4} \times (28^2 - 20^2)} = 13.3 \text{ MPa}$$

32. **(b)**

$$E = 125 \text{ GPa } \mu = 0.25$$

$$G = \frac{E}{2(1+\mu)} = \frac{125}{2(1+0.25)} = 50 \text{ GPa}$$

33. **(b)**

34. **(b)**

35. **(a)**

$$\sigma_c = \frac{Pr}{2t}$$

$$125 \times 10^6 = \frac{P \times 10}{2 \times 10 \times 10^{-3}}$$

$$\Rightarrow \quad P = 0.25 \text{ MPa}$$

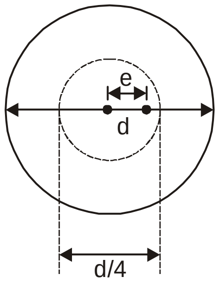
36. **(d)**

37. **(c)**

$$\text{Eccentricity, } e = \frac{1}{2}\left(\frac{d}{8}\right)$$

$$e = \frac{d}{16}$$

Maximum compressive stress, $\sigma_{c_{max}}$



$$\sigma_{c_{max}} = \frac{P}{A} + \frac{P\cdot e\cdot y}{I}$$

$$\sigma_{c_{max}} = \frac{P}{A} + \frac{P\left(\frac{d}{16}\right)\times\left(\frac{d}{2}\right)}{\frac{\pi d^4}{64}}$$

$$\sigma_{c_{max}} = \frac{P}{A} + \frac{P}{\frac{\pi d^2}{4}} \times \frac{1}{2} = \frac{P}{A}\left(1 + \frac{1}{2}\right)$$

$$\sigma_{c_{max}} = \frac{3}{2}\frac{P}{A}$$

$$\sigma_{c\,min} = \frac{P}{A} - \frac{P\cdot e\cdot y}{I}$$

$$\sigma_{c\,min} = \frac{P}{A} - \frac{P\left(\frac{d}{16}\right)\left(\frac{d}{2}\right)}{\frac{\pi d^4}{64}} \Rightarrow \frac{P}{A}\left(1 - \frac{1}{2}\right)$$

$$\sigma_{C_1 min} = \frac{P}{2A}$$

So, difference of maximum and minimum compressive stress

$$= \frac{3P}{2A} - \frac{P}{2A} = \frac{2P}{2A} = \frac{P}{A}$$

$$\therefore \quad \text{in } \% = \frac{\frac{P}{A}}{\frac{P}{2A}} \times 100 = 200\%$$

38. **(d)**

39. **(a)**

$$q = K.H.\frac{N_f}{N_d}$$

$$2.185 = 0.3067 \times 28.5 \times \frac{N_f}{N_d}$$

$$\frac{N_d}{N_f} = \frac{0.3067 \times 28.5}{2.185}$$

$$\boxed{\frac{N_d}{N_f} = 4}$$

40. **(c)**

Since, $e_{max} > e_{natural} > e_{min}$

$$e_{max} = \frac{3}{6} = 0.5$$

$$e_{natural} = \frac{2}{6} = 0.333$$

$$e_{min} = \frac{1}{6} = 0.167$$

$$D_r = \frac{0.5 - 0.33}{0.5 - 0.167} \times 100$$

$$\boxed{D_r = 50\%}$$

41. **(b)**

$$\tau = C + \bar{\sigma}\tan\phi$$

$$\therefore \quad C = 0$$

$$\tau = \bar{\sigma}\tan\phi$$

$$\phi = \tan^{-1}\left(\frac{\tau}{\bar{\sigma}}\right) = \tan^{-1}(1)$$

$$\boxed{\phi = 45^\circ}$$

42. **(d)**

∴ Gw = eS, S = 1 for fully saturate soil.

$$w = \frac{e}{G} = \frac{0.6 \times 1}{2.7}$$

$$w = 0.2222$$

Additional water content req. for full saturation

= 22.22 – 15

= 7.22%

43. **(c)**

44. **(c)**

Group index = 0.2a + 0.005 ac + 0.01 bd

a = % passing 75μ sieve greater than 35 and not exceeding 75

b= % passing 75μ sieve greater than 15 and not not exceeding 55

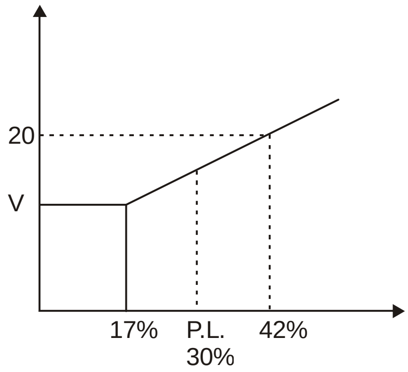
c = part of L.L greater than 40 and not exceeding 60

d = part of plasticity index greater than 10 and not exceeding 30.

G.I. = 0.2 × 20 + 0.005 × 20 × 5 + 0.01 × 40 × 10 = 8.5

45. **(a)**

We know the minimum volume occurs at shrinkage limit.



At L.L. At Shrinkage Limit

$$Se = \omega G \qquad Se = \omega G$$

$$e_{LL} = .42 \times 2.74 \qquad e_{SL} = 0.4658$$

$$e_{LL} = 1.1508 \qquad e = \frac{V_v}{V_s} = 0.4658$$

$$e = \frac{V_V}{V_S} = \frac{V - V_S}{V_S}$$

$$\Rightarrow V = V_S(1 + e)$$

$$\frac{V_{LL}}{V_{SL}} = \frac{(1+e_{LL})V_S}{(1+e_{SL})V_S} = \frac{1+1.1508}{1+0.4658}$$

$$V_{SL} = \frac{V_{LL}}{1.4673} = \frac{20}{1.4673} = 13.63 \text{ cm}^3$$

46. (b)

$$\text{Specific Surface} = \frac{\text{Surface Area}}{\text{Volume}}$$

$$= \frac{4\pi r^2}{\frac{4}{3}\pi r^3} \Rightarrow \frac{6}{d}$$

$$\text{Specific Surface} \propto \frac{1}{d}$$

$$\text{Specific surface of 10 mm particle} = \frac{1}{10} = 0.1$$

$$\text{Specific surface of 0.1 particle} = \frac{1}{0.1} = 10$$

$$\frac{\text{Specific surface of 10 mm particle}}{\text{Specific surface of 0.1 mm particle}} = \frac{0.1}{10} = \frac{1}{100}$$

47. (d)

$$\text{Sensivitity} = \frac{(q_u)\text{undistrubed}}{(q_u)\text{Remoulded}}$$

and as per defination of sensitivity, these strength should be recorded at the same moisture content.

48. (b)

(a) The values of liquid limit and plasticity index for soil having common geological origin in a restricted locality usually define a straight line parallel to A line.

(b) When coarse and fine fraction are in equal % the possible dual symbol in this case are

GM – MI, ML, MH
GC – CI, CH, CL
SM – MI, MH, ML
SC – CI, CH, CL

49. (a)

We know that the degree of saturation in semi-solid stage is (S) = 1

$$Se = WG_S$$

$$W = \frac{0.54}{2.7} = 0.2 \simeq 20\%$$

50. (c)

We know that for soil with I_L (liquidity index) < 0 the soil behave like brittle solid.

51. (b)

Head causing flow = 2m

$$\sigma_{xx} = 2 \times 20 + 1 \times 9.81 = 49.81 \text{ kN/m}^2$$

$$\mu_{xx} = \gamma_w \times 3 + ih\gamma_w$$

$$\mu_{xx} = 3 \times 9.81 + \left(\frac{2}{4}\right) \times 2 \times 9.81$$

$$= 39.24 \text{ kN/m}^2$$

$$\bar{\sigma}_{xx} = 49.81 - 39.24 = 10.57 \text{ kN/m}^2$$

52. (a)

In a flow net all flow fields are elementary square.

$$\frac{a_1}{b_1} = \frac{a_2}{b_2} = \frac{a_3}{b_3} = n$$

For a given set of boundary condition, the $\frac{N_f}{N_d}$ ratio is same or fixed.

53. (c)

54. (a)

$$K = C_v m_v \gamma_w$$

Here, K = K_V (Permeability in vertical direction)

5.7×10^{-9} m/sec = $C_V \times 18.4 \times 10^{-4} \times 9.81$

$C_V = 3.15 \times 10^{-7}$ m^2/sec

55. (c)

- m_V is not constant, it depends on stress range.
- OCR < 1 for soil which is yet to consolidate ex. recent fills.
- Degree of saturation during consolidation is 100%.

56. **(a)**

Overall degree of consolidation "U", due to vertical and radial drainage

$$(1-U) = (1-U_v)(1-U_r)$$

$$(1-U) = (1-0.2)(1-0.8)$$

$$U = 0.84 \simeq 84\%$$

$$U = \frac{\Delta h}{\Delta H}$$

$$0.84 = \frac{\Delta h}{150}$$

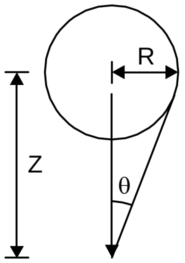
$$\Delta h = 126 \text{ mm}$$

Remaining Settlement = $150 - 126 = 24 \text{mm}$

57. **(b)**

- Influence chart for determination of vertical stress under uniform footing load based on westergaard equation was developed by Fenske.
- Newmark's chart are based on Boussinesq's equation.

58. **(a)**



Vertical stress under a circular area

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

$$\sigma_Z = q\, I_C$$

Influence coefficient for Circular Area

$$= I_C = \left[1-\left[\frac{1}{1+\left(\frac{R}{Z}\right)^2}\right]^{3/2}\right]$$

Let $\tan\theta = \frac{R}{Z}$ [R = Radius of circle]

$$I_C = 1-\left[\frac{1}{1+\tan^2\theta}\right]^{3/2}$$

$$I_C = 1-(\cos^2\theta)^{3/2}$$

$$I_C = 1-\cos^3\theta \qquad \text{....(i)}$$

Hence, equation (i) indicates that as θ tends to 90° the valve I_C approaches unity.

Hence, $\sigma_Z = q$

59. **(b)**

Split spoon sampler yields representative samples.

60. **(b)**

$$I_0 = \frac{3}{2\pi\left(1+\frac{25}{36}\right)^{5/2}} = 0.128$$

$$\sigma_z = 0.128 \times \left(\frac{1000}{36}\right) = 3.548 \text{ kN/m}^2$$

$$\tau = \sigma_z \times \left(\frac{r}{z}\right) = 3.548 \times \frac{5}{6}$$

$$= 2.96 \text{ kN/m}^2$$

61. **(b)**

$$\sigma = 2.04 \times 9.81 + 3 \times 19.69$$

$$= 79.08 \text{ kN/m}^2$$

$$U = 5.04 \times 9.81 = 49.44 \text{ kN/m}^2$$

$$\bar{\sigma} = 79.08 - 49.44 = 29.64 \text{ kN/m}^2$$

Note: Increase or decrease in water level above ground level does not effect effective stress.

62. **(b)**

Advantages of achored sheet pile

1. Reduction in depth of penetration
2. Increase in the height to be supported
3. Allowing use of lighter section, due to the reduction of bending moments and deflections.

63. **(d)**

64. **(c)**

SPT test may be applied to both cohesive and cohesionless soils correlating number of blows with.

(a) Angle of shearing resistance ϕ

(b) Bearing capacity factors N_γ and N_q

(c) Allowable bearing pressure of foundation in sand

(d) Relative density of sand

(e) Consistency/uncontinued compressive strength of clay.

65. **(a)**

Radius of friction circle = $R \sin\phi$

$$= 22 \sin(15°)$$

$$(\because \sin 15 = \sin(45-30) = \frac{1}{\sqrt{2}} \times \frac{\sqrt{3}}{2} - \frac{1}{2\sqrt{2}})$$

$$= 5.7 \text{ m}$$

66. (c)

$$S_n = \frac{C}{\gamma H F_C} = \frac{20}{17\times 12\times 1.25}$$

$$S_n = 0.0784$$

$$\frac{0.098-0.0784}{0.098-0.063} = \frac{45^\circ-\theta}{45^\circ-30^\circ},$$

where θ is the slope $\theta = 36.61^\circ$

67. (a)

$$\eta = 1-\frac{\theta}{90}\left(\frac{m(n-1)+(m-1)n}{mn}\right)$$

$$\theta = 20$$

$$m = n = 4$$

$$\eta = 1-\frac{20}{90}\left(\frac{4\,(3)+(3)4}{4\times 4}\right)$$

$$= 1-\frac{20}{90}\left(\frac{24}{16}\right)$$

$$\eta = \frac{2}{3} = 66.7\%$$

68. (c)

$$Q_a = \frac{Wh}{6\,(S+C)}$$

$$W = 30 \text{ kN},\ h = 100 \text{ cm},$$

$$C = 0.25 \text{ cm},\ Q_a = 250 \text{ kN}$$

$$250 = \frac{30\times 100}{6(S+0.25)}$$

$$S = 17.5 \text{ mm}$$

69. (d)

70. (c)

71. (c)

Direction of principal planes are not known at every stage of the test and that's why it is not possible to draw Mohr circle at intermediate loading.

72. (b)

Hydrodynamic lag – The time lag due to low permeability of clay and consequent time required for the escape of pore water.

Plastic lag– The plastic action in absorbed water near grain to grain contacts, which does not allow quick transmission of the applied stress to the grains and the effective stress to reach a constant value.

The Terzaghi theory does not recognise the existence of plastic lag.

73. (a)

$$(h_c)_{max} = \frac{C}{eD_{10}} = \frac{20}{0.5\times 0.02}$$

$$h_c = 2 \text{ m}$$

74. (b)

Seismic refraction method can be used only if the soft layer overlies a hard layer.

75. (d)

Correction for overburden = $N\times\dfrac{350}{\bar{\sigma}_0+70}$

For 1.5 m depth, N = 10

$\bar{\sigma}$ = 1.5 × 18 [No water table at this depth]

$$N' = \frac{10\times 350}{27+70} = 36.08$$

and there will be no correction for dilatancy because soil is above W.T.

76. (b)

As clay will be remoulded.

77. (a)

78. (c)

79. (c)

80. (a)

$$G_E = \frac{H}{d}\times\frac{1}{\pi\sqrt{\lambda}}$$

and $$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

and $$\alpha = \frac{b}{d} = \frac{30}{6} = 5$$

$\therefore$ $$\lambda = \frac{1+\sqrt{1+5^2}}{2} = 3.05$$

$$G_E = \frac{6}{6}\times\frac{1}{3.14\times\sqrt{3.05}} = \frac{1}{5.48} \cong \frac{1}{5.5}$$

81. (b)

82. (c)

83. (d)

84. (a)

85. (b)

$$SAR = \frac{Na^+}{\sqrt{\dfrac{Ca^{2+}+Mg^{2+}}{2}}} = \frac{22}{\sqrt{\dfrac{3+1.5}{2}}} \cong 14.67$$

$\Rightarrow$ SAR is between 10 and 18

$\therefore$ Medium sodium water

86. **(b)**

This is so because in furrow method since water is not applied to the entire land surface, the water losses are considerably reduced.

87. **(a)**

Discharge = 1 = cumec

Loss = 25%

Discharge at field = 12 × 0.75 = 9 cumec

Duty at field = 1250 hec/cumec

⇒ Area irrigated = 1250 × 9

= 1.125×10^4 hectares

88. **(c)**

GCA = 20,000 hectares

CCA = 0.75 × 20,000 = 15,000 hectares

⇒ Area to be irrigated with 40% intensity

= 0.4 × 15,000 = 6,000 hectares

$$\therefore \quad \text{Outlet discharge} = \frac{6{,}000}{1{,}800} = 3.33 \text{ cumec}$$

89. **(c)**

$$\text{Design capacity} = \frac{0.27+0.36}{0.9} = 0.7 \text{ m}^3/\text{s}$$

90. **(a)**

$$\text{Depth water lost} = 700 \times \frac{1200}{1000} \times 0.75 \times (0.3-0.1)$$

= 126 mm

$$\therefore \quad \text{Frequency} = \frac{126}{12.6} = 10 \text{ days}$$

91. **(b)**

92. **(b)**

$$S = \frac{4K}{q}(b^2 - a^2)$$

$$= \frac{4 \times 10^{-6}}{1.8 \times 10^{-6}}(5^2 - 4^2) = 20 \text{ m}$$

93. **(b)**

94. **(d)**

95. **(c)**

Discharge should also be constant.

96. **(d)**

97. **(c)**

$$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}$$

$$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{2}{4}}$$

$$\frac{\tau_c'}{\tau_c} = \sqrt{\frac{1}{2}}$$

$$\therefore \quad \tau_c' = \frac{2.83 \times 10^{-3}}{\sqrt{2}} \text{ KN/m}^2 \ (\because 2\sqrt{2} \simeq 2.83)$$

$$\therefore \quad \tau_c' = 2 \times 10^{-3} \text{ KN/m}^2$$

98. **(b)**

$$n = \frac{1}{24} \times d^{1/6} \quad \text{as per strickler}$$

where d = size of particle in meter

$$\therefore \quad n = \frac{1}{24} \times (0.05)^{1/6} = 0.0253$$

99. **(d)**

Minimum size of particle, d = 10.8 RS

⇒ d = 10.8 × 0.8 × 0.0041

⇒ d = 0.0354 m ⇒ 35.42 mm ≃ 36 mm

100. **(d)**

101. **(b)**

$$H_{max} = \frac{f}{\gamma_w(S_c+1)} = \frac{4000}{10 \times (2.5+1)}$$

= 114.28 ≅ 115 m

102. **(d)**

Fetch > 32 km

$$h_w = 0.032\sqrt{V.F.}$$

$$\therefore \quad h_w = 0.0325\sqrt{100 \times 36}$$

$$\therefore \quad h_w = 1.92 \text{ m}$$

103. **(b)**

60,000 kN × 0.95 = 57000 kN

104. **(a)**

Bouyant wt of concrete = seepage force on unit area

(G – 1) × t × 1 = 4.5 × 1

$$\therefore \quad t = \frac{4.5}{2.5-1} = 3 \text{ m}$$

105. **(d)**

The discharging capacity of the undersluices is provided as maximum of the following :

1. Two times the maximum discharge of the offtaking canal
2. Maximum-winter discharge
3. 20% of the maximum flood discharge

106. **(b)**

$$\eta_{dist} = \left[1-\frac{y}{d}\right]\times 100$$

$$= \left[1-\frac{0.168}{1.76}\right]\times 100$$

$$= 0.905 \times 100$$

$$= 90.5\%$$

107. **(c)**

$$V_0 = 0.55\ my^{0.64}$$

$$V_0 = 1\ m/s$$

$$m = 1$$

$$0.55 = 0.55 \times 1 \times y^{0.64}$$

$$y^{0.64} = \frac{0.55}{0.55}$$

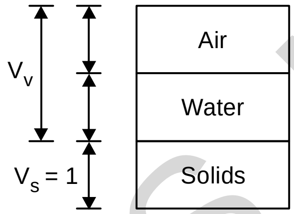
$$y = \left(\frac{0.55}{0.55}\right)^{1/0.64}$$

$$y = 1\ m$$

108. **(d)**

When a soil-phase diagram is shown with the volume of solids as unity, then it is referred as

Unit-phase diagram



109. **(b)**

$$L.R. = \frac{D_i - C_u}{D_i}$$

$$0.8 = \frac{D_i - 6}{D_i}$$

$$\Rightarrow\ 6 = 0.2D_i$$

$$\therefore\ D_i = 30\ cm$$

110. **(c)**

111. **(c)**

$$f = 1.76 \times r0.36 = 1.056$$

112. **(d)**

113. **(a)**

114. **(c)**

115. **(b)**

$$\text{Outside clearance } C_o = \frac{D_2 - D_4}{D_4}\times 100$$

D_2 = Outside diameter of outing edge

D_4 = Outside dia of sampling tube

$$C_o = \frac{74-72}{72}\times 100 = 2.78\%$$

116. **(a)**

Principal strains

$$\varepsilon_1/\varepsilon_2 = \frac{\varepsilon_x+\varepsilon_y}{2} \pm \sqrt{\left(\frac{\varepsilon_x-\varepsilon_y}{2}\right)^2 + \left(\frac{\phi_{xy}}{2}\right)^2}$$

$$= \frac{(600+100)\times 10^{-6}}{2} \pm \sqrt{(250\times 10^{-6})^2 + (200\times 10^{-6})^2}$$

$$= \left(350\times 10^{-6}\right) \pm 320.15\times 10^{-6}$$

$$\boxed{\begin{aligned}\varepsilon_1 &= 670\times 10^{-6}\\ \varepsilon_2 &= 29.85\times 10^{-6}\end{aligned}}$$

117. **(d)**

$$\therefore\ \frac{\Delta_A}{\Delta_B} = \frac{\left(\frac{PL}{AE}\right)_A}{\left(\frac{PL}{AE}\right)_B}$$

$$\frac{\Delta_A}{\Delta_B} = \frac{\left(\frac{PL}{AE}\right)}{\left(\frac{2PL}{AE}\right)}$$

$$\boxed{\Delta_A = \frac{1}{2}\Delta_B}$$

118. **(d)**

119. **(d)**

120. **(b)**

$$H_{max} = \frac{f}{\gamma_w(S_c+1)} = \frac{4000}{10\times(2.5+1)}$$

$$= 114.28 \cong 115\ m$$

121. **(c)**

122. **(d)**

Mild steel is tougher than cast iron, because failure strain is more in ductile material.

123. **(c)**

The following assumptions are made during analysis of the behaviour of shafts subjected to torque.

(1) The material is homogenous.

(2) The material is elastic

(3) The stress does not exceed the elastic limit

(4) The circular section remains circular

(5) Cross-section remains plane

(6) Cross-section rotate as if rigid i.e., every diameter rotates through the same angle.

Shearing strains is the angle between final and original position of the generators and this definition of shearing strain holds at the interior point of the bar.

124. (d)

Amount of elastic deformation is same irrespective of the stresses being tensile or compressive, because E and μ are assumed same in tension as well as compression.

125. (a)

In presence of shear, plane section before bending does not remain plane after bending because warping of section occurs in this case. We know that shear stress leads to distortion of the section.

126. (b)

127. (c)

128. (d)

If the plane of loading/couple contains one of the principle axis of the cross section then NA will be parallel to the direction of moment.

129. (a)

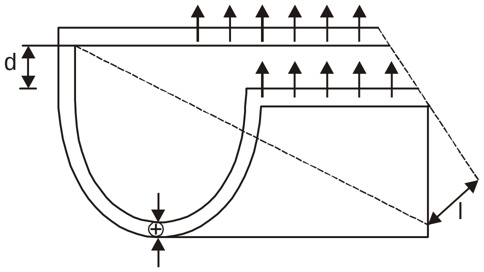
130. (c)

If the section is having 2 axis of symmetry the shear centre will coincide with the CG of section however if the section is having only one axis of symmetry the shear centre will be on the line of symmetry.

131. (d)

Shear stress does not exists on the section on which hoop stress acts as $\left(\sigma_n = \text{hoop stress} = \frac{Pd}{2t}\right)$ is a principal stress.

σ_n = hoop stress = tensile.



132. (d)

It is necesary to subtract negative skin friction force from the total load that the pile can support.

$$\text{FOS} = \frac{\text{Ultimate load capacity of single pile or a group of pile}}{\text{Working load} + \text{negative skin friction}}$$

133. (a)

134. (a)

135. (c)

When particle size decreases both liquid limit and plastic limit increases but liquid limit increase at greater rate so, plasticity index increases.

136. (a)

137. (a)

$$n + \frac{1}{e+1} = 1$$

$$\therefore \quad \gamma_d = \frac{(1-\eta_a)G\gamma_w}{1+\frac{wG}{S}}$$

For zero air void $\eta_a = 0$

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

$\therefore$ Incase of full saturation S = 1

$$\gamma_d = \frac{G\gamma_w}{1+\frac{wG}{S}} = \frac{G\gamma_w}{1+wG}$$

138. (c)

A and B both varies with the stress range.

139. (a)

140. (c)

141. (b)

142. (d)

A divide wall is a long masonry or concrete wall or groyne (an embankment protected on all sides by store or concrete blocks) which is constructed at right angles to the axis of the weir to separate the undersluices from the rest of the weir.

143. (d)

The axis of the regulator may be kept at an angle varying from 90° to 110°.

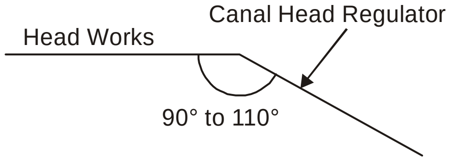


Fig. Relative indication of head regulator and barrage axes.

144. (c)

The drain will be effective in reducing the uplift pressure only if there are properly maintained and are not allowed to get clogged.

145. (d)

The tenacity with which water is retained in the soil is defined as soil moisture tension.

Soil moisture stress is defined as the sum of soil moisture tension and osmotic pressure of soil solution.

146. (b)

147. (a)

148. (c)

Seepage is responsible for the instability of dam by increasing the actuating force and decreasing the resisting force.

149. (b)

By adding the intensities of irrigation for all the crop seasons the yearly intensity of irrigation may be obtained.

150. (d)

Kennedy never mentioned anything about shape of the section of a stable channel.