Conventional Question Practice Programe

Date: 9th April, 2016

CE–Test - 12 (OBJECTIVE SOLUTION)...

ANSWERS

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

1. (c)

The stability of gravity dam is due to the self weight aided by passive resistance developed in front of the toe.

Statement (3) in true for an earthen dam but it is not the reason for the popularity of an earthen dam.

2. (b)

The test to determine the maximum density usually involves some form of vibrations. The test to determine minimum density usually involves pouring oven-dried soil into a container. Unfortunately, the details of these tests have not been entirely standardized and values of the maximum density and minimum density for a given granular soil depend on the procedure used to determine them.

Uniform soil means soil with particle size in a narrow range. A soil with large size particles can also be uniform.

Angular particles arrange themselves in zig-zag orientation leaving large spaces vacant.

3. (a)

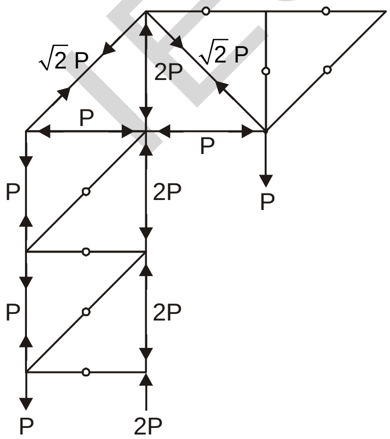
OMC = 15%

At OMC, $$\rho_d = \frac{\rho_w \cdot G}{1+\frac{wG}{S}}$$

$$\Rightarrow \quad 1.82 = \frac{1\times 2.73}{1+\frac{0.15\times 2.73}{S}}$$

$$\Rightarrow \quad S = 0.819$$

4. (c)



5. (d)

Horizontal thrust due to udl on the left half of the span = H_1

$$= \frac{wl^2}{16h} = \frac{32\times 25^2}{16\times 5} = 250\text{ kN}$$

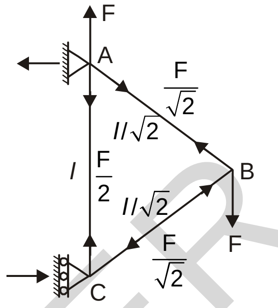
Horizontal thrust due to point load at the crown = H_2

$$= \frac{25}{128}\times\frac{wl}{8} = \frac{25}{128}\times\frac{128\times 25}{5} = 125\text{ kN}$$

Total horizontal thrust = H = $H_1 + H_2$

= 250 + 125 = 375 kN

6. (a)



Let vertical deflection of point B = δ then;

$$1\cdot\delta = \sum\frac{PKl}{AE}$$

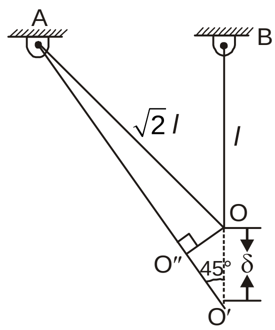
$$= \frac{1}{AE}\left[\left(\frac{F}{\sqrt{2}}\cdot\frac{1}{\sqrt{2}}\cdot\frac{l}{\sqrt{2}}\right)\times 2+\frac{F}{2}\cdot\frac{1}{2}\cdot l\right]$$

$$= \frac{1}{AE}\left[\frac{Fl}{\sqrt{2}}+\frac{Fl}{4}\right]$$

$$\delta = \frac{(2\sqrt{2}+1)Fl}{4AE}$$

7. (c)

By symmetry, there will be no horizontal movement of point O. Let vertical deflection = δ.



Extension of bar AO = $O''O' = \frac{\delta}{\sqrt{2}}$

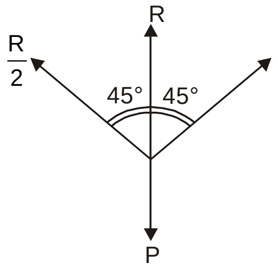
Similarly extension of bar CO = $\frac{\delta}{\sqrt{2}}$

Strain in bar BO = $\frac{\delta}{l}$

Strain in bar AO = $\frac{\delta}{\sqrt{2}\times\sqrt{2}\,l} = \frac{\delta}{2l}$

If force in bar BO = R

then force in bar AO = $\frac{R}{2}$ = force in bar CO



$$R + \frac{1}{\sqrt{2}}\cdot\frac{R}{2} + \frac{1}{\sqrt{2}}\cdot\frac{R}{2} = P$$

$$\Rightarrow \quad R = \left(\frac{\sqrt{2}}{\sqrt{2}+1}\right)P$$

Total external work done = Total internal energy stored

$$\Rightarrow \quad \frac{1}{2}\cdot P\cdot\delta = \sum\left(\frac{P_i^2 l_i}{2A_iE_i}\right)$$

$$\frac{P\cdot\delta}{2} = \frac{1}{2AE}\left(R^2 l + \left(\frac{R}{2}\right)^2\cdot\sqrt{2}\,l + \left(\frac{R}{2}\right)^2\cdot\sqrt{2}\,l\right)$$

$$= \left(\frac{2+\sqrt{2}}{4AE}\right)R^2 l$$

$$= \left(\frac{2+\sqrt{2}}{4AE}\right)\times\frac{2P^2 l}{(\sqrt{2}+1)^2}$$

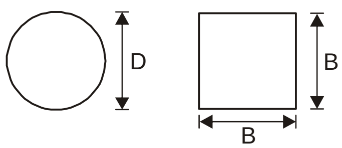
$$\Rightarrow \quad \delta = \frac{(2+\sqrt{2})}{(\sqrt{2}+1)^2}\,\frac{Pl}{AE}$$

8. **(a)**

The capacity of a proposed sewer system is determined by the estimated requirements of the community at the end of the design period.

9. **(d)**

Two sections of different shape are hydraulically equivalent, if they carry same discharge, while running full on the same grade (slope) and are of same material.



$Q_1 = Q_2$

$$\Rightarrow \quad \frac{1}{n}\left(\frac{D}{4}\right)^{2/3}\cdot S^{1/2}\cdot\frac{\pi}{4}D^2$$

$$= \frac{1}{n}\left(\frac{B}{4}\right)^{2/3}\cdot S^{1/2}\cdot B^2$$

(Manning's equation)

$$\Rightarrow \quad D^{8/3}\cdot\frac{\pi}{4} = B^{8/3}$$

$$\left(\frac{B}{D}\right) = \left(\frac{\pi}{4}\right)^{3/8}$$

$$\Rightarrow \quad B = \left(\frac{\pi}{4}\right)^{3/8}\cdot D$$

10. **(a)**

Type of soil	Value of E (kN/m^2)
Soft clay	1500 – 4000
Hard clay	6000 – 15000
Silty sand	6000 – 20000
Dense sand	40000 – 80000

11. **(a)**

$$C_u = \frac{T}{\pi\left(\frac{d^2h}{2}+\frac{d^3}{6}\right)}$$

$$= \frac{2000}{\pi\left[\frac{100\times 20}{2}+\frac{1000}{6}\right]}$$

$$= \frac{12}{7\pi}$$

12. **(a)**

20 × 10 × 0.005 = 1T/m^2

13. **(b)**

Rigid footing on sand $\rightarrow$ soil has lowest modulus of elasticity at edges

Flexible footing on sand $\rightarrow$ soil has high modulus of elasticity at the centre

Flexible footing on saturated clay $\rightarrow$ maximum deflection at the centre

Rigid footing on saturated clay $\rightarrow$ settlement is uniform

14. **(c)**

15. **(a)**

Vertical stress under a circular area is given by

$\sigma_z = I_c\cdot q$ where I_C is the influence coefficient

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

$\because$ For entire semi-infinite soil mass, $R \to \infty$

$$\therefore \lim_{R\to\infty} I_C = \lim_{R\to\infty}\left[1-\left\{\frac{1}{1+\left(\frac{R}{z}\right)^2}\right\}^{3/2}\right] = 1$$

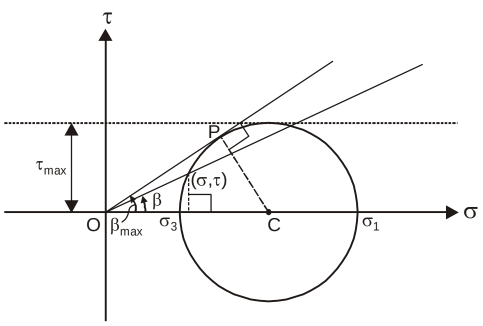
$$\therefore \sigma_z = I_C \cdot q = q$$

16. **(a)**

17. **(c)**

Consolidated drained (CD) test is known as drained test. It is also known as the slow test (S-test)

18. **(b)**



Maximum shear stress = τ_{max} = Radius of Mohr circle

$$= \frac{\sigma_1 - \sigma_3}{2}$$

Resultant stress on a plane = $\sqrt{\sigma^2 + \tau^2}$

Angle of obliquity with the normal of plane

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

Maximum angle of obliquity

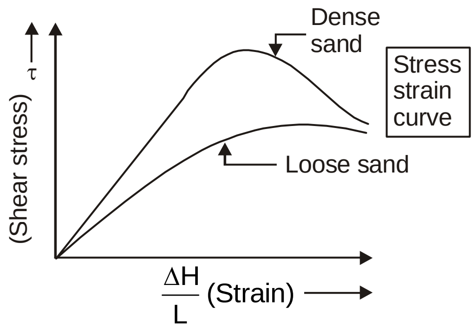
$$= \beta_{max} = \sin^{-1}\left(\frac{CP}{OC}\right)$$

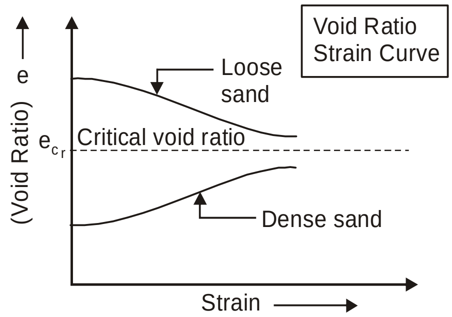
$$= \sin^{-1}\left(\frac{(\sigma_1-\sigma_3)/2}{(\sigma_1+\sigma_3)/2}\right)$$

$$= \sin^{-1}\left(\frac{\sigma_1-\sigma_3}{\sigma_1+\sigma_3}\right)$$

19. **(a)**

Generally, the failure strain is 2 to 4% for dense sand and 12 to 16% for loose sand.

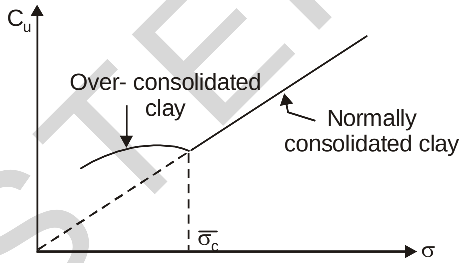




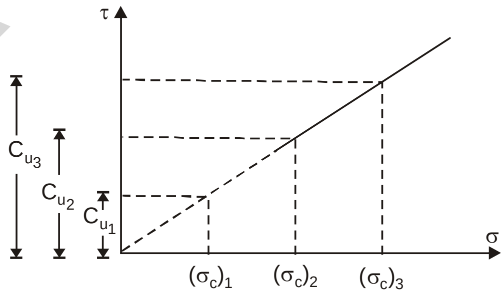
20. **(c)**

The vane shear test does not give accurate results when the failure envelope is not horizontal.

For over-consolidated clays, the plot between C_u and σ is curved until a pressure equal to the preconsolidation pressure $\bar{\sigma}_c$.



C_u–σ Plot for over-consolidated clay



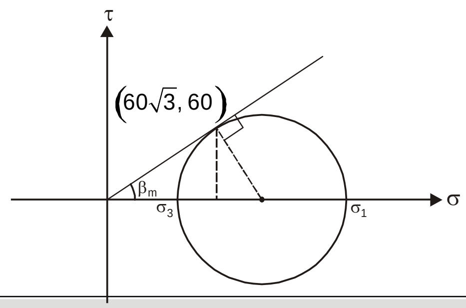
C_u–σ plot for normally consolidated clay

21. **(d)**

In the case of loose sand, the specimen bulges and ultimately fails by sliding simultaneously on numerous planes. The failure is known as the plastic failure. In the case of dense sand, the specimen shows a clear failure plane and the failure is known as the brittle failure.

22. **(b)**

Assuming that plane of failure is the plane of maximum obliquity.



$$(\sigma_f, \tau_f) = \left(60\sqrt{3}, 60\right)$$

$$\tan\beta_m = \frac{60}{60\sqrt{3}} = \frac{1}{\sqrt{3}}$$

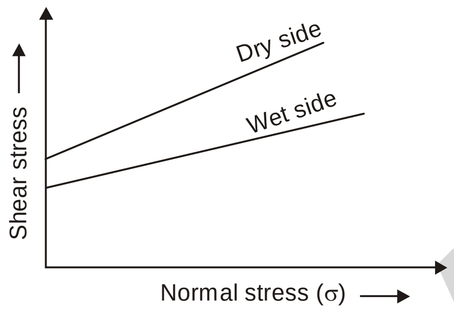
$$\Rightarrow \quad \beta_m = 30^\circ$$

$$\sigma_1 = \frac{\sigma_f}{1-\sin\beta_m} = \frac{\sigma_f}{1-\sin 30^\circ}$$

$$= \frac{60\sqrt{3}}{1-\frac{1}{2}} = 120\sqrt{3}\ \text{N/mm}^2$$

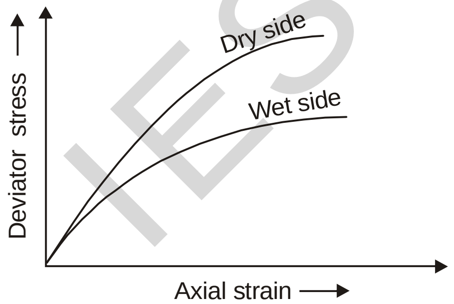
23. **(d)**

Soils compacted at a water content more than the optimum water content usually have a dispersed structure if the compaction induces large shear strains and a flocculated structure if the shear strains are relatively small.



Failure Envelops

The soils compacted dry of the optimum have a steeper stress-strain curve than those on the wet side.



24. **(c)**

Compaction methods cannot remove all the air voids and therefore, the soil never becomes fully saturated. Thus, the theoretical maximum dry density is only hypothetical.

The amount of compaction in the field should be approximately equal to that in the laboratory. The standard proctor test is adequate to represent the compaction of fills behind retaining walls and in highways and earth dams where light rollers are used. In such cases, the optimum water content obtained from the standard proctor test can be used as control criterion.

However, in situations where heavier compaction is required for example in modern highways and runways, the standard proctor test does not represent the equivalent compaction in the laboratory.

Proctor needle is used to determine water content in the field indirectly.

25. **(a)**

$$B = \frac{\Delta u}{\Delta\sigma_3} = \frac{0.15-0.07}{0.26-0.10} = \frac{0.08}{0.16} = 0.5$$

26. **(b)**

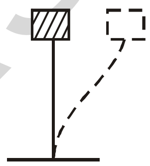
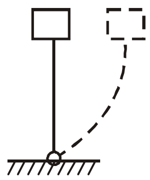
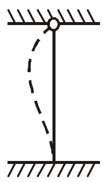
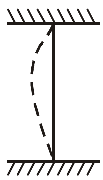
Torsional constant $\rightarrow \Sigma b_i t_i^3/3$

Non-dimensional slenderness ratio $\rightarrow \sqrt{f_y/f_{cr,b}}$

Imperfection factor $\rightarrow 0.21$

$\beta_b \rightarrow Z_e/Z_p$

27. **(b)**

End restrain	Effective length
	1.20 *l*
	2.0 *l*
	0.80 *l*
	0.65 *l*

28. **(c)**

29. **(c)**

Min. no. of intermediate battens should be two.

30. **(d)**

The Euler buckling load of a slender steel column does not depend on the yield strength

(σ_y) of steel.

Lacing spacing depends on the slenderness of column as a whole.

31. **(b)**

32. **(d)**

33. **(d)**

34. **(c)**

35. **(d)**

36. **(a)**

37. **(c)**

Transverse shear resisted by lacing = 2.5% of axial load.

500 × 2.5% = 12.5 kN.

38. **(c)**

Minimum radius of gyration = $\frac{t}{\sqrt{12}}$.

39. **(a)**

40. **(d)**

41. **(d)**

The design of eccentricity loaded column needs revision when

$$\frac{f'_c}{f_c} + \frac{f'_b}{f_b} > 1$$

where,

f'_c = calculated average axial compressive stress

f_c = allowable axial compressive stress

f'_b = calculated bonding stress in the extreme fibre, and

f_b = allowable bending com-pressive stress on the extreme fibre

42. **(a)**

Maximum permissible slenderness ratio for different types of members as per IS : 800

A member carrying compressive loads resulting from dead and superimposed loads. $\rightarrow$ 180

A member subjected to compressive loads resulting from wind/earthquake. $\rightarrow$ 250

A member normally carrying tension but subjected to reversal of stress due to wind or earthquake forces. $\rightarrow$ 350

Tension member (other than pretensioned member). $\rightarrow$ 400

43. **(c)**

$$M_P = Z_P \cdot \sigma_y$$

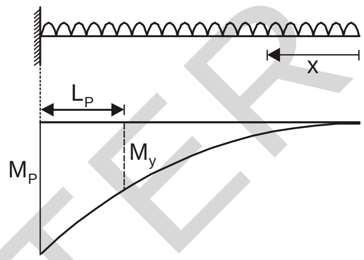
$$10^9 \times 4.8 \times 10^{-4} \sigma_y = 120 \times 10^3 \times 10^3$$

$$\sigma_y = \frac{120 \times 10^3 \times 10^3}{4.8 \times 10^{-4} \times 10^9} = 250 \text{ N/mm}^2$$

$$= 250 \text{ MPa}$$

44. **(d)**

45. **(c)**



$$M_x = \frac{wx^2}{2}$$

$$M_y = \frac{w(L-L_p)^2}{2}$$

$$M_P = fM_y = \frac{wL^2}{2}$$

$$\Rightarrow \frac{1}{f} = \left(\frac{L-L_P}{L}\right)^2$$

$$\Rightarrow \frac{1}{\sqrt{f}} = \frac{L-L_P}{L}$$

$$\Rightarrow L = L\sqrt{f} - L_P\sqrt{f}$$

$$\Rightarrow L_P\sqrt{f} = \left(\sqrt{f}-1\right)L$$

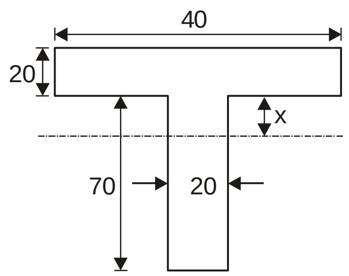
$$\frac{L_P}{L} = \frac{\sqrt{f}-1}{\sqrt{f}} = 1 - \frac{1}{\sqrt{f}}$$

46. **(c)**

47. **(a)**

48. **(a)**

Plastic neutral axis is an equal area axis.



40 × 20 + 20 × x = 20 × (70 – x)

$\Rightarrow$ $800 + 20x = 1400 - 20x$

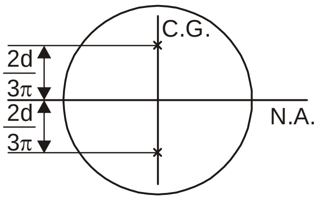
$\Rightarrow$ $40x = 600$

$$x = \frac{600}{40} = 15$$

$\therefore$ Distance from top = 20 + 15 = 35 mm.

49. **(c)**

Plastic section modulus for circular section is



$$Z_p = \frac{\pi d^2}{4\times 2}\left[2\times\frac{2d}{3\pi}\right]$$

$$= \frac{\pi d^2}{8}\times\frac{4d}{3\pi} = \frac{\pi d^3}{6\pi} = \frac{d^3}{6}$$

$$Z = \frac{\pi d^4}{64}\times\frac{2}{d} = \frac{\pi d^3}{32}$$

$$\therefore \quad \frac{Z_p}{Z} = \frac{d^3}{6}\times\frac{32}{\pi d^3} = \frac{16}{3\pi}$$

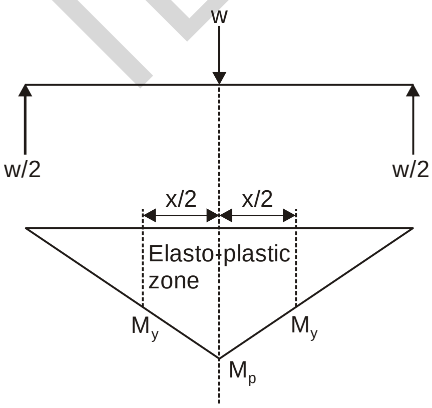
50. **(c)**

Shape factor = $\frac{Z_p}{Z}$. Both Z_p and Z are geometrical parameters.

51. **(d)**

Plastic hinge will be formed at centre of span.

Elastoplastic zone is the region where BM experienced by beam during plastic collapse is greater than plastic yield moment M_y.



Using similar Δ,

$$\frac{M_p}{\frac{L}{2}} = \frac{M_y}{\frac{L}{2}-\frac{x}{2}}$$

$$\Rightarrow \frac{M_p}{M_y} = \frac{L/2}{L/2 - x/2}$$

But $\frac{M_p}{M_y}$ = shape factor = 1.5 for rectangular sections.

$$\Rightarrow \frac{L}{L-x} = 1.5$$

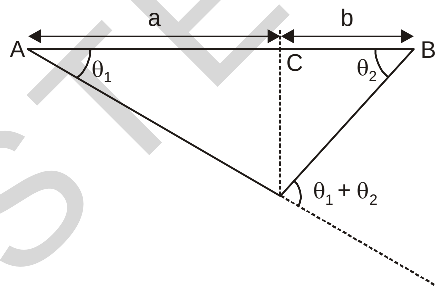
$\Rightarrow$ 1.5L – 1.5x = L

$$\Rightarrow x = \frac{0.5L}{1.5} = \frac{L}{3}$$

52. **(c)**

53. **(b)**

At collapse three plastic hinges will form at A, B and C.



–[Internal work done] = [External work done]

$$M_p\theta_1 + M_p\theta_2 + M_p(\theta_1+\theta_2) = W_u a\theta_1$$

$$\therefore \quad M_p[2\theta_1 + 2\theta_2] = W_u a\theta_1$$

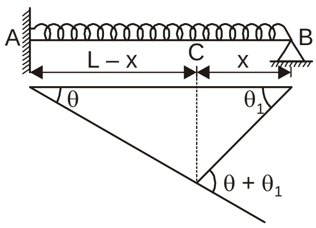
$$\left[\text{where } a\theta_1 = b\theta_2 \Rightarrow \theta_2 = \frac{a}{b}\theta_1\right]$$

$$\therefore \quad M_p 2\left[1+\frac{a}{b}\right]\theta_1 = W_u a\theta_1$$

$$\therefore \quad W_u = \frac{2M_p(a+b)}{ab} = \frac{2M_pL}{ab}$$

54. **(c)**

Let plastic hinge forms at distance x from B



$$(L-x)\theta = x\theta_1$$

$$\Rightarrow \theta_1 = \frac{L-x}{x}\theta$$

– [Internal work done] = [External work done]

$$\Rightarrow M_p\theta + M_p(\theta+\theta_1) = \frac{1}{2}W_uL(L-x)\theta$$

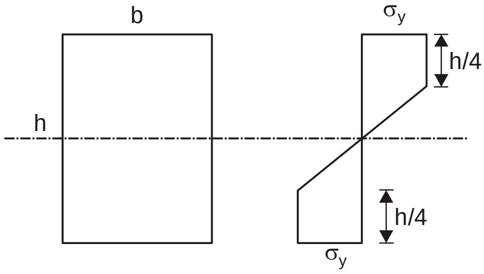
$$\Rightarrow M_p = \frac{W_uL}{2}\,\frac{x(L-x)}{(L+x)}$$

Point C is a limiting case which experiences maximum BM in beam AB before becoming plastic.

$$\Rightarrow \frac{dM_p}{dx} = 0$$

$$\Rightarrow x = 0.414\ L$$

55. (c)



$$M = \left[\frac{bh}{4}\cdot\frac{3h}{8}\sigma_y + \frac{1}{2}\cdot\frac{bh}{4}\cdot\frac{2}{3}\cdot\frac{h}{4}\cdot\sigma_y\right]\times 2$$

$$= \frac{bh^2}{32}\sigma_y\left[3+\frac{2}{3}\right]\times 2 = \frac{11}{48}bh^2\sigma_y$$

56. (d)

$$\text{Load factor} = \frac{1.12\times1.50}{1.20} = 1.4$$

57. (a)

58. (a)

59. (c)

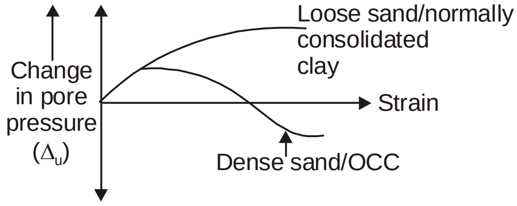
Spandrel Beam - Supporting load from exterior wall and slab and spanning from column to column.

60. (b)

61. (b)

The soils most susceptible to liquefaction are the saturated, fine and medium sands of uniform particle size. When such deposits have a void ratio greater than the critical void ratio and subjected to a sudden shearing stresses there is decrease in volume and the pore pressure increases. The soil momentarily liquefies and behaves as a dense fluid.

In a consolidated undrained test, there is an increase in the pore water pressure throughout for loose sand (and normally consolidated clay). However, in case of dense sands (and over-consolidated clay), the pore water pressure increases at low strains but at large strains it becomes negative (below atmospheric pressure).



Pore pressure diagram

On a failure plane, the angle of obliquity is maximum.

62. (b)

63. (d)

64. (a)

65. (c)

$$e_{min} = \frac{l}{500}+\frac{D}{30} = \frac{3000}{500}+\frac{300}{30} = 16\text{ mm}$$

But $e_{min} \nless 20$ mm

Hence e_{min} = 20 mm.

66. (b)

67. (c)

$$\text{Bending stress} = \frac{M}{I_{xx}}\times y$$

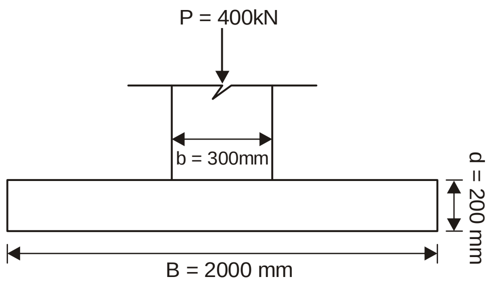
$$= \frac{\left(\frac{20\times9}{8}\right)\times10^3\times10^3}{1696.6\times10^4}\times100$$

$$= 132.62\text{ N/mm}^2$$

$$\text{Shear stress} = \frac{V}{t_w\cdot D} = \frac{\left(\frac{20\times3}{2}\right)\times10^3}{5.4\times100}$$

$$= 27.78\text{ N/mm}^2$$

68. (c)



Net effective shear force

$$= F = q\left[\left(\frac{B-b}{2}\right)-d\right]B$$

where q = Net upward pressure

$$= \frac{\text{Load on the column}}{\text{Footing area}}$$

$$= \frac{400\times10^3}{(200)^2} = 0.1\,\text{N/mm}^2$$

$$F = 0.1\left[\left(\frac{2000-300}{2}\right)-200\right]2000$$

= 130000 N

Nominal transverse (one way) shear = $\frac{F}{Bd}$

$$= \frac{130000}{2000\times200} = 0.325\ \text{N/mm}^2$$

69. **(d)**

70. **(a)**

71. **(d)**

Compared to the flat plate system, the flat slab system is suitable for higher loads and larger spans, because of its enhanced capacity is resisting shear and hogging moments near the supports.

72. **(d)**

73. **(c)**

74. **(c)**

Shear walls are also frequently placed along the transverse direction of a building, either as exterior walls or as interior walls.

75. **(d)**

76. **(b)**

77. **(d)**

78. **(d)**

79. **(a)**

80. **(d)**

81. **(d)**

82. **(b)**

83. **(c)**

84. **(a)**

85. **(c)**

86. **(d)**

$$\frac{\text{Minimum ratio of vertical reinforcement to gross concrete area}}{\text{Minimum ratio of non vertical reinforced to gross concrete area}}$$

$$= \frac{0.0012}{0.0020} = \frac{12}{20} = \frac{3}{5}$$

87. **(a)**

88. **(a)**

89. **(d)**

90. **(c)**

91. **(c)**

92. **(c)**

$$E = \frac{\sigma}{\varepsilon}$$

Loss of prestress = σ_{loss} = 200GPa × 0.0008

= 160 MPa

$\Rightarrow\ \sigma_{left} = 200 - 160 = 40$ MPa

93. **(b)**

94. **(c)**

95. **(c)**

96. **(b)**

97. **(c)**

$$\sigma_{max} = \frac{P}{A}+\frac{M}{Z}$$

$$= \frac{360}{2\times3}+\frac{75}{\left(\frac{2\times3^2}{6}\right)} = 60+25 = 85\ \text{kN/m}^2$$

Similarly,

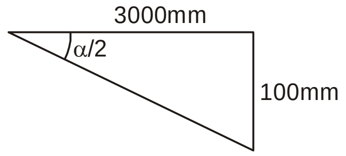
$$\sigma_{min} = 60-25 = 35\ \text{kN/m}^2$$

98. **(a)**

Loss in prestress = $P_l = P_0(kx+\mu\alpha)$

$= 1200(0.0015\times12+0.45\times\alpha)$

where $\alpha = 2\times\frac{100}{3000} = \frac{1}{15}$



$$P_l = 1200\left(0.018+\frac{0.45}{15}\right) = 57.6\ \text{kN}$$

99. **(c)**

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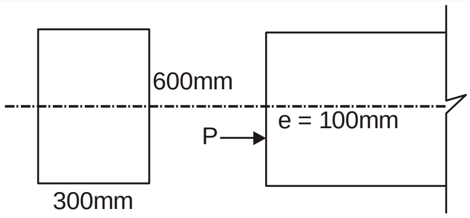

$$\sigma_{max}/\sigma_{min} = \frac{1000 \times 10^3}{300 \times 600} \pm \frac{1000 \times 100 \times 10^3}{\left(\frac{300 \times 600^2}{6}\right)}$$

$$= \frac{1000}{18} \pm \frac{6 \times 10^9}{108 \times 10^6}$$

$$= \frac{1000}{18} \pm \frac{1000}{18}$$

Hence σ_{max} = 11.11 N/mm^2.

100. (d)

101. (b)

In Pratt truss longer member carries tension, while in Howe truss longer member carries compression. Longer truss members have high chance of buckling under compression.

102. (c)

Clay pipe is a good sewer conduit for sanitary sewers because it is inert and sensistive to acid attack.

103. (d)

Sewer lines and pumping stations are difficult to enlarge, thus they have design periods from 25 to 50 years. Treatment plants are easier to expand and have design periods from 10 to 25 years.

104. (a)

105. (b)

In deep sand deposits, the modulus of elasticity increases with an increase in depth and therefore Boussinesq solution will not give satisfactory results. In this case, the assumption of proportionality between stress and strain can not be justified. For such a case, non-linear elastic solutions or elastic-plastic solutions are used.

106. (c)

Most of the shear tests are conducted as strain-controlled. The stress-strain characteristic are easily obtained in these tests, as the shape of the stress-strain curve beyond peak point, can be observed only in a strain-controlled test.

Stress-controlled tests are preferred for conducting shear tests at a very low rate, because an applied load can easily be kept constant for any given period of time. Further, the loads can be conveniently applied and removed. The stress-controlled test represents the field conditions more closely.

107. (d)

In a direct shear test, the stresses on planes other than the horizontal plane are not known. It is, therefore, not possible to drawn Mohr stress circle at different shear loads. However, the Mohr circle can be drawn at the failure condition assuming that the failure plane is horizontal.

108. (a)

109. (b)

110. (b)

111. (c)

112: (a)

113. (b)

114. (a)

115. (a)

116. (a)

117. (b)

118. (d)

IS456 : 2000 limits the maximum diameter of reinforcing bars in slabs to one-eighth of the total thickness of the slab.

119. (c)

When the slab is supported on all four sides, and the plans dimensions of length and breadth are comparable to each other, the slab bends in two directions (along the length and along the breadth); hence, it is called a two way slab. However, if the shape is a long rectangle (Length greater than about twice the width), the bending along the longitudinal direction is negligible in comparison with that along the transverse (short-span) direction and the resulting slab action is effectively one-way.

120. (a)