DETAILED SOLUTIONS

MADE EASY

India's Best Institute for IES, GATE & PSUs

Test Centres: Delhi, Noida, Hyderabad, Bhopal, Jaipur, Lucknow, Bhubaneswar, Indore, Pune, Kolkata, Patna

ESE 2020 : Prelims Exam
CLASSROOM TEST SERIES

CIVIL ENGINEERING

Test 12

Section A : Design of concrete and Masonry Structures

Section B : Structural Analysis-1

Section C : CPM PERT-2 + Hydrology and Water Resource Engg.-2

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

DETAILED EXPLANATIONS

Section A : Design of Concrete and Masonry Structures

1. (a)

As per **Clause 6.2.2** of **IS 456:2000**, modulus of rupture, f_{cr} is given as

$$f_{cr} = 0.7\sqrt{f_{ck}} = 0.7\sqrt{40}$$

$$= 4.43 \text{ MPa}$$

2. (b)

As per **Clause 26.5.1.6** of **IS 456:2000**, minimum shear reinforcement is given as

$$\frac{A_{SV}}{bS_v} \geq \frac{0.4}{0.87 f_y}$$

$$\Rightarrow \quad A_{SV} \geq \frac{0.4}{0.87 f_y} \times b \times S_v$$

$$\geq \frac{0.4 \times 300 \times 300}{0.87 \times 415}$$

[Characteristic strength of shear stirrups $\ngtr$ 415 N/mm²]

$$\geq 99.71 \text{ mm}^2$$

$$\geq 100 \text{ mm}^2$$

3. (a)

$$\text{Cracking moment, } M_{cr} = \frac{f_{cr} I_g}{y_t} = f_{cr} Z$$

where

$$f_{cr} = 0.7\sqrt{f_{ck}} = 0.7 \times \sqrt{25} = 3.5 \text{ MPa}$$

$$Z = \frac{bD^2}{6}$$

$$= \frac{300 \times 600^2}{6} = 18 \times 10^6 \text{ mm}^3$$

$$\therefore \quad M_{cr} = 3.5 \text{ MPa} \times 18 \times 10^6 \text{ mm}^3$$

$$= 63 \times 10^6 \text{ Nmm}$$

$$= 63 \text{ kNm}$$

4. (c)

When a beam with transverse shear reinforcement is loaded, most of the shear force is initially carried by the concrete. Between flexural and inclined cracking, the external shear is resisted by the concrete V_{cz}, the interface shear transfer V_{az}, and the dowel action V_d. The first branch of shear cracking of the beams with transverse reinforcement is typically the same in nature as that of beams without transverse reinforcement. The shear crack in this case also involves two branches. After the first inclined crack, redistribution of shear stresses occurs, with some parts of the shear being carried by the concrete and the rest by the stirrups, V_s.

5. (b)

As per **Clause 26.3.3 of IS 456:2000,**

- The horizontal distance between parallel main reinforcement shall not be more than three times the effective depth of a solid slab or 300 mm whichever is smaller.
- The horizontal distance between parallel main reinforcement provided against shrinkage and temperature shall not be more than five times the effective depth of a solid slab or 450 mm whichever is smaller.

6. (c)

As per **Cl.41.4.3** of **IS 456:2000**,

$$A_{sv} = \frac{T_u S_v}{b_1 d_1 \left(0.87 f_y\right)} + \frac{V_u S_v}{2.5 d_1 \left(0.87 f_y\right)}$$

For pure torsion, $V_u = 0$

and thus
$$T_u = \frac{A_{sv} b_1 d_1 \left(0.87 f_y\right)}{S_v}$$

$$A_{sv} = 2 \times \frac{\pi}{4} \times 10^2 = 157 \text{ mm}^2$$

$$S_v = 140 \text{ mm}$$

$$b_1 = 250 - 30 \times 2 - 10 \times 2 - 20 = 150 \text{ mm}$$

$$d_1 = 500 - 30 \times 2 - 10 \times 2 - 20 = 400 \text{ mm}$$

$$T_u = \frac{157 \times 150 \times 400 \times (0.87 \times 415)}{140}$$

$$= 24.29 \times 10^6 \text{ Nmm}$$

$$= 24.29 \text{ kNm}$$

7. (c)

Moment coefficients as per **Clause D-1** of **IS 456:2000** are based on inelastic analysis (Yield line theory).

10. (d)

Factored load, $P_u = 1.5P$

$= 1.5 \times 1500 \text{ kN} = 2250 \text{ kN}$

Given steel percentage $= 1\%$ of A_g

$\therefore \quad P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

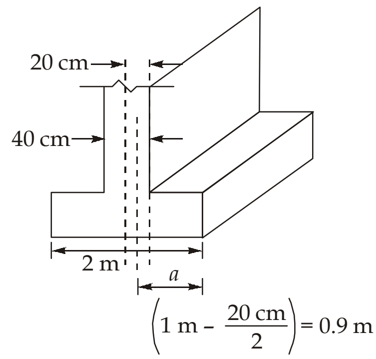
$$\Rightarrow \quad 2250 \times 10^3 = 0.4 \times 25 \times \left(A_g - 0.01 A_g\right) + 0.67 \times 415 \times 0.01 A_g$$

$$\Rightarrow \quad A_g = 177437.79 \text{ mm}^2 = \frac{\pi}{4} D^2$$

$$\Rightarrow \quad D = 475.31 \text{ mm} \simeq 480 \text{ mm} \qquad \text{(say)}$$

11. (b)

The critical section for bending moment in footing under masonry wall occurs half way between the middle and the edge of the wall.



$$a = 0.9\text{ m}$$

12. (b)

- Contraction joint is a movement joint with a deliberate discontinuity but no initial gap between the concrete on either side of the joint. The purpose of this joint is to accommodate contraction of concrete.
- Expansion joint is a movement joint with complete discontinuity in both reinforcement and concrete and intended to accommodate either expansion or contraction of the structure.

13. (c)

$$M_d = \frac{w_d l^2}{16} + \frac{w_l l^2}{12}$$

$$= \frac{4 \times 5^2}{16} + \frac{18 \times 5^2}{12}$$

$$= 43.75\text{ kNm}$$

14. (c)

For simply supported circular slabs subjected to uniformly distributed loads, the bending moments are given by

$$M_r = \frac{w}{16}(3+\mu)\left(r^2 - a^2\right)$$

and $$M_t = \frac{w}{16}\left[r^2(3+\mu) - a^2(1+3\mu)\right]$$

where M_r and M_t are radial and tangential moments respectively, a is the radius where the bending moment is determined $(0 \le a \le r)$ and μ is the Poisson's ratio which may be taken as 0 or 0.15 for reinforced concrete. Assuming μ = 0, we get maximum moment at centre of slab as

$$M_{r,\max} = M_{t,\max} = \frac{3wr^2}{16}$$

15. (c)

$$A_{st} = 3 \times \frac{\pi}{3} \times 20^2 = 942\text{ mm}^2$$

$$0.36\, f_{ck}\, A_c = 0.87\, f_y\, A_{st}$$
$$0.36 \times 25 \times A_c = 0.87 \times 415 \times 942$$
$$A_c = 37789.9 \text{ mm}^2 = bx_u = 200\, x_u$$
$$x_u = 189 \text{ mm} < 0.48 \times 550 = 264 \text{ mm}$$

Hence section is under reinforced

$$M = 0.87\, A_{st}\, f_y(d - 0.42x_u)$$
$$= 0.87 \times 942 \times 415 \times (550 - 0.42 \times 189) = 160 \text{ kNm}$$

16. (c)

In statement 2, the smaller of 1 m or half of the width of landing is selected.

17. (d)

- As per point (d) of **Clause 38.1 of IS 456:2000** tensile strength of concrete is ignored in limit state of collapse in flexure as well as in compression.
- Statement 2 - Point number (a) of **Clause 39.1 of IS 456:2000**
- Statement 3 is an assumption of limit state of collapse in flexure.

18. (b)

Concrete surfaces exposed to sea water spray is said to be in very severe exposure condition. Minimum grade of concrete exposed to extreme environment for RCC should be M40.

19. (b)

According to **Clause 25.4** of **IS 456 : 2000**, all columns shall be designed for a minimum eccentricity equal to the unsupported length of column/500 plus lateral dimensions/30, subject to a minimum of 20 mm.

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

But, $e_{\min}$ = 20 mm

Hence, minimum eccentricity = 20 mm

20. (c)

As per **Clause 32.2.5** of **IS 456:2000**,

$$P_{uw} = 0.3\left(t - 1.2e - 2e_a\right) f_{ck}$$

21. (d)

Torsion, shear and flexure come under the category of limit state of collapse.

22. (c)

Hoyer's method is used for pretensioned concrete.

23. (d)

Stress in concrete at level of steel,

$$\sigma_c = \frac{P}{A} = \frac{250 \times 1000}{150 \times 300}$$
$$= 5.56 \text{ N/mm}^2$$

$$\text{Loss} = \theta \times m \times \sigma_c \qquad \left\{\text{where, } m = \frac{E_s}{E_c}\right\}$$
$$= 2 \times 7 \times 5.56 \text{ N/mm}^2$$
$$= 77.84 \text{ N/mm}^2$$

24. (c)

$\because$ Span = 12 m > 10 m

$\therefore$ As per **Cl.23.2.1 (b)** of **IS 456:2000,**

$$\text{Effective depth} > \frac{\text{Span}}{20 \times (10/\text{Span})}$$
$$> \frac{12 \times 12}{20 \times 10} = 0.72 \text{ m} = 720 \text{ mm}$$

25. (d)

$$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}}$$

Where, $\tau_{bd} = 1.6 \times 1.25 \times 1.5 = 3 \text{ N/mm}^2$ (for deformed bar in compression)

$$\therefore \quad L_d = \frac{0.87 \times 500 \times 20}{4 \times 3} = 725 \text{ mm}$$

26. (d)

Refer **Clause D-1.4 to D-1.6** of **IS 456:2000**.

27. (d)

- Response reduction factor (R) : It is the factor by which the base shear induced in a structure, if it were to remain elastic, is reduced to obtain the design base shear.
- Peak ground acceleration is the maximum acceleration of the ground in a given direction.
- Special moment resisting frame also meets the detailing of **IS:13920**. While OMRF doesn't meet the requirements of **IS:13920**.

28. (c)

Vertical geometric irregularity shall be considered to exist when the horizontal dimension of the lateral force resisting system in any storey is more than 125% of the storey below.

Refer **IS 1893:2016** for additional guidelines on EQ resistant design.

29. (a)

- Formwork shall not be removed until concrete has achieved a strength of at least twice the stress to which the concrete may be subjected to, at the time of removal.
- For 55 m^3 of concrete, there must be 5 samples 4 + 1 additional sample for 0.50 m^3 after 50 m^3 concrete work.
- Concrete in the member represented by a core shall be considered acceptable if the average equivalent cube strength of the cores is equal to atleast 85% of cube strength of grade of concrete.

30. (b)

According to **Clause 26.2.5.1** of **IS 456,** lap splices shall not be used for bars larger than 32 mm.

31. (a)

Design parameters:

For seismic zone II, zone factor, $Z = 0.1$

Importance factor, $I = 1.5$

Response reduction factor, $R = 5$

Seismic weight:

Floor area = 8 × 8 = 64 m^2

For live load upto (including) 3 kN/m^2, percentage of live load to be considered = 25%

For live load > 3 kN/m^2, percentage of live load to be considered = 50%

As per **Clause 7.3.2 of IS 1893** imposed load on top roof slab is neglected.

$W = \sum W_i$ = Sum of loads from all the floors which includes DLs and appropriate percentage of live loads.

$$\therefore \quad W = \underbrace{64 \times (16 + 0.5 \times 12)\,\text{kN}}_{1^{st}\text{ floor}} + \underbrace{64 \times 16\text{kN}}_{\text{Roof}}$$

$$= 2432 \text{ kN}$$

$$A_h = \left(\frac{Z}{2}\right)\left(\frac{I}{R}\right)\left(\frac{Sa}{g}\right) = \frac{0.1 \times 1.5 \times 2.5}{2 \times 5} = 0.0375$$

$$\therefore \quad \text{Design base shear, } V_B = A_h \times W$$

$$= 0.0375 \times 2432 = 91.2 \text{ kN}$$

33. (a)

$$\sigma_{top}/\sigma_{bottom} = \frac{P}{A} \mp \frac{Pe}{Z}$$

$$= \frac{150 \times 10^3}{120 \times 200} \mp \frac{150 \times 10^3 \times 20 \times 6}{120 \times (200)^2}$$

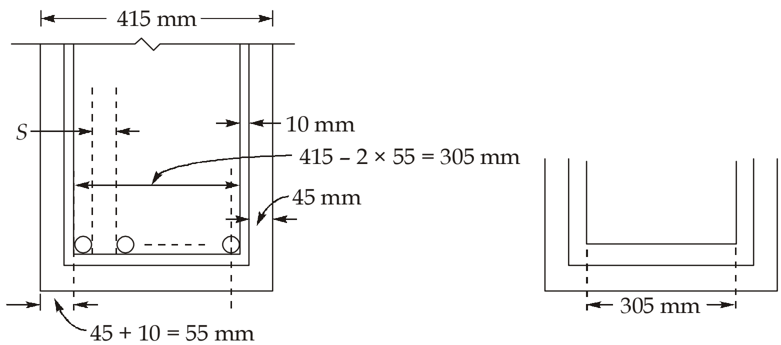
$$= 6.25 \mp 3.75$$

$$\sigma_{top} = 6.25 - 3.75 = 2.5 \text{ N/mm}^2 \text{ (C)}$$

$$\sigma_{Bottom} = 6.25 + 3.75 = 10 \text{ N/mm}^2 \text{ (C)}$$

34. (b)

As per table 16 of IS 456:2000 (Clause 26.4.2), the minimum clear cover for severe exposure condition is 45 mm.



$$S \nless \begin{bmatrix} \text{Maxumum diameter of} \\ \text{longitudinal bar} = 32 \text{ mm} \\ \text{Maximum size of aggregate} + 5 \text{ mm} \\ = 20 + 5 = 25 \text{ mm} \end{bmatrix} \text{Clause 26.3.2}$$

Take

$$\begin{aligned} S &= 32 \text{ mm} \\ 305 &= n\phi + (n-1)S \\ 305 &= n(32) + (n-1)32 \\ 305 &= (2n-1)\,32 \\ n &= 5.26 \end{aligned}$$

Maximum number of bars that can be provided is 5 since if we provide more than 5 bars spacing will become less than 32 mm.

$$\begin{aligned} 305 &= 5 \times 32 + 4S \\ S &= 36.25 && \text{(OK)} \\ 305 &= 6 \times 32 + 5S \\ S &= 22.6 \text{ mm} && \text{(Not OK)} \end{aligned}$$

35. (a)

Both the statements are correct and statement (II) is the correct explanation of statement (I).

36. (c)

- For concrete mix design, apart from meeting the criteria for characteristic strength, concrete must be workable in fresh state and impermeable and durable in hardened state.
- Nominal mix concrete is permitted only in ordinary concrete (i.e. upto M20 concrete).

37. (d)

Bends and hooks are ineffective in anchoring bars in compression. Hence, code (**Clause 26.2.2.2**) specifies that for bars in compression, only the projected length of hooks, bends and straight lengths beyond bends shall be considered for development length. However, for bars in compression, it is doubtful whether extensions beyond bends can meaningfully provide anchorage.

38. (d)

The design forces specified in IS 1893:2016 are much less than actual forces that may act on a structure.

Section B : Structural Analysis-1

40. (a)

$$\theta_A = 0,\ \theta_B \neq 0,\ \theta_C = 0$$

$$M_{fba} = \frac{30 \times 4^2}{12} = 40 \text{ kNm},\ M_{fbc} = -40 \text{ kNm}$$

By slope deflection equation,

$$M_{BA} = M_{fba} + \frac{2EI}{l} \times \left(2\theta_B + \theta_A - \frac{3\delta}{l}\right)$$

$$\Rightarrow \quad M_{BA} = 40 + \frac{2 \times 10000}{4} \times \left(2\theta_B - \frac{3 \times 10}{4000}\right)$$

$$= 2.5 + 10000\ \theta_B$$

$$M_{BC} = M_{fbc} + \frac{2EI}{l}\left(2\theta_B + \theta_C - \frac{3\delta'}{l}\right)$$

$$= -40 + \frac{2 \times 10000}{4}\left(2\theta_B - \frac{3 \times 20}{4000}\right)$$

$$= -115 + 10000\,\theta_B$$

Joint equilibrium equation at B,

$$M_{BA} + M_{BC} = 0$$

$$\Rightarrow \quad 2.5 + 10000\theta_B - 115 + 10000\theta_B = 0$$

$$\Rightarrow \quad \theta_B = 5.625 \times 10^{-3} \text{ radians}$$

41. (a)

The induced moment at the fixed end is in the same direction as the applied moment.

42. (c)

Downward deflection at point B due to UDL if there was no support at B,

$$\delta_B = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384} \times \frac{w \times (2l)^4}{EI}$$

$$\Rightarrow \quad \delta_B = \frac{5}{24}\frac{wL^4}{EI}$$

Upward deflection of B due to R_B,

$$\delta'_B = \frac{R_B \times (2l)^3}{48EI} = \frac{R_B l^3}{6EI}$$

As there is no sinking of support B and thus,

$$\delta_B = \delta'_B$$

$$\Rightarrow \quad \frac{5wl^4}{24EI} = \frac{R_B l^3}{6EI}$$

$$\Rightarrow \quad R_B = \frac{5}{4}wl$$

Also, $R_A + R_B + R_C = 2wl$

And $R_A = R_C$ due to symmetry

$$\therefore \quad R_A = R_C = \frac{3}{8}wl$$

43. (a)

$$M_{AB} = 0 \qquad \text{(Hinged)}$$

$$M_{BA} = \frac{-3EI_1\Delta}{H_1^2}$$

$$M_{DC} = M_{CD} = \frac{-6EI_2\Delta}{H_2^2}$$

$$\therefore \quad \frac{M_{BA}}{M_{CD}} = \frac{I_1 H_2^2}{2I_2 H_1^2}$$

$$= \frac{2I \times 8^2}{2 \times I \times 4^2} = \frac{8}{2} = 4$$

44. (c)

Both statements are correct.

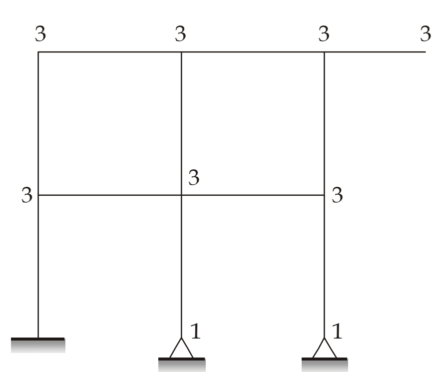
Statement 1 corresponds to moment area theorem-I.

Statement 2 corresponds to moment area theorem-II.

$$\text{Also curvature} = \frac{1}{R} = \frac{M}{EI}$$

Thus curvature diagram is same as $\frac{M}{EI}$ diagram

45. (b)



$$D_K = 23$$

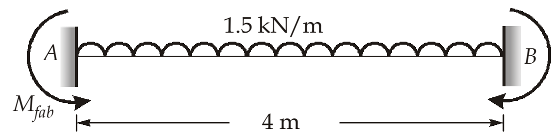
$$D_K \text{ (If member are inextensible)} = 23 - 11 \text{ (No. of members)}$$
$$= 12$$

46. (a)

$$\text{Fixed end moment at } A = \frac{wl^2}{30} = \frac{15 \times 10 \times 10}{30} = 50 \text{ kNm}$$

$$\text{Fixed end moment at } B = \frac{wl^2}{20} = \frac{15 \times 10 \times 10}{20} = 75 \text{ kNm}$$

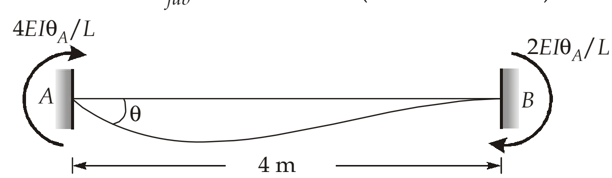
47. (a)



Fixed end moment at left support due to loading

$$M_{fab} = \frac{-wl^2}{12} = \frac{-1.5 \times 4^2}{12}$$

$$\Rightarrow \quad M_{fab} = -2 \text{ kNm} \quad \text{(Anticlockwise)}$$



Moment due to rotation at A

$$M'_{fab} = \frac{4EI\theta_A}{l}$$

$$= \frac{4 \times 10^6 \times 0.001}{4} \qquad (10^{10} \text{ N-cm}^2 = 10^6 \text{ Nm}^2)$$

$$= 1000 \text{ Nm}$$

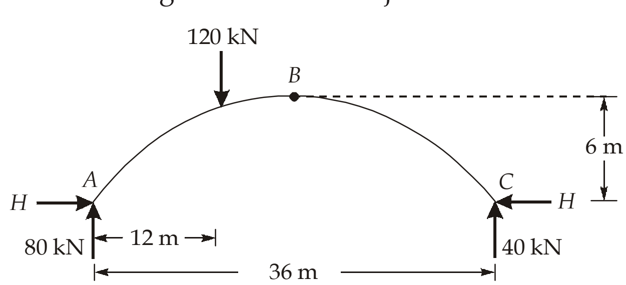
$$= 1 \text{ kNm} \quad \text{(Clockwise)}$$

So, fixed end moment developed at A

$$= -2 + 1 = -1 \text{ kNm i.e. anticlockwise}$$

48. (a)

For point load, maximum bending moment occurs just below the load in a three-hinged arch.



$$M_C = 0$$

$\Rightarrow$ $$V_A \times 36 = 120 \times 24$$

$\Rightarrow$ $$V_A = 80 \text{ kN}$$

$\therefore$ $$V_C = 120 - 80 = 40 \text{ kN}$$

$$M_B = 0 \quad \text{(from right)}$$

$\Rightarrow$ $$V_C \times 18 = H \times 6$$

$\Rightarrow$ $$H = \frac{40 \times 18}{6} = 120 \text{ kN}$$

Equation of parabolic arch is, $$y = \frac{4hx(l-x)}{l^2}$$

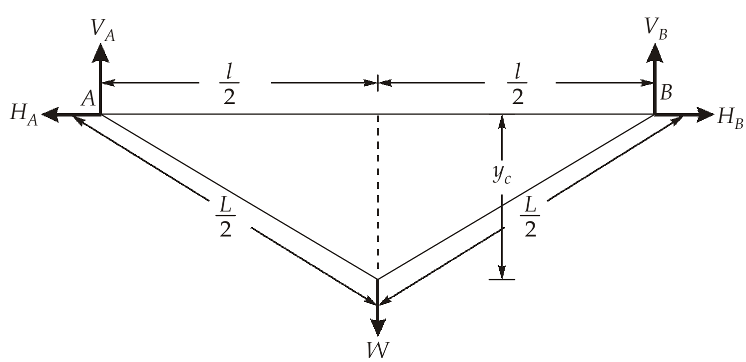
$$y_{(x = 12\text{ m})} = \frac{4 \times 6 \times 12(36-12)}{36^2}$$

$\Rightarrow$ $$y_{(x = 12\text{ m})} = \frac{16}{3} \text{ m}$$

So bending moment under load is,

$$M_{max} = 80 \times 12 - 120 \times \frac{16}{3} = 320 \text{ kNm}$$

49. (b)



$$\frac{L}{2} = \sqrt{\left(\frac{l}{2}\right)^2 + y_c^2} \qquad \text{where, } L = 18 \text{ m}, l = 16 \text{ m}$$

$\Rightarrow$ $$\left(\frac{18}{2}\right)^2 = \left(\frac{16}{2}\right)^2 + y_c^2$$

$\Rightarrow$ $$9^2 - 8^2 = y_c^2$$

$\Rightarrow$ $$y_c = 4.12 \text{ m}$$

50. (b)

$$\delta h = \frac{3l^2 \alpha t}{16h}$$

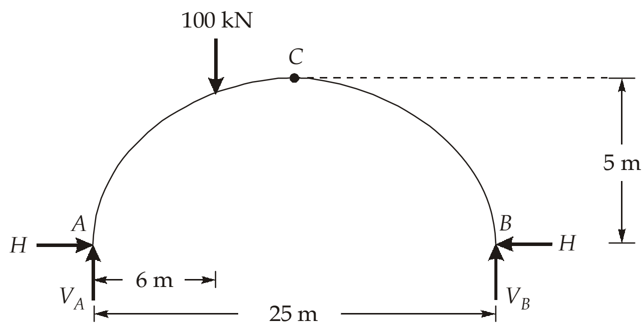
$\Rightarrow$ $$\delta h = \frac{3 \times 80^2 \times 12 \times 10^{-6} \times 20}{16 \times 6}$$

$\Rightarrow$ $$\delta h = 0.048 \text{ m}$$

51. (a)

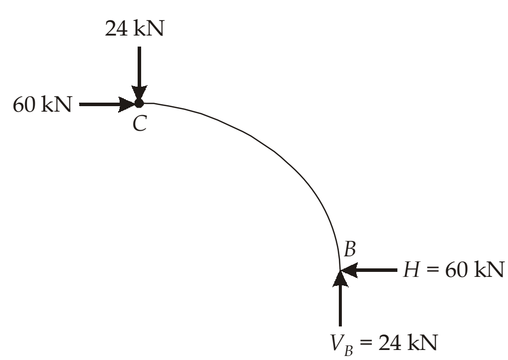
$$\text{Number of cuts} = 14$$
$$\text{Static indeterminacy of 3-D frame} = \text{Number of cuts} \times 6 = 14 \times 6 = 84$$

52. (d)



$$M_B = 0$$
$$\Rightarrow \quad V_A \times 25 = 100 \times 19$$
$$\Rightarrow \quad V_A = 76 \text{ kN}$$
$$\therefore \quad V_B = 100 - 76 = 24 \text{ kN}$$
$$M_C = 0 \quad \text{(from right)}$$
$$\Rightarrow \quad V_B \times 12.5 = H \times 5$$
$$\Rightarrow \quad H = \frac{24 \times 12.5}{5} = 60 \text{ kN}$$

Drawing *FBD* of part *CB*,



$$\text{Resultant reaction at } C = \sqrt{60^2 + 24^2} = 64.62 \text{ kN}$$

53. (d)

Fixed end moments,

$$M_{fab} = \frac{-2 \times 6^2}{12} = -6 \text{ kNm}$$
$$M_{fba} = 6 \text{ kNm}$$
$$M_{fbc} = \frac{-9 \times 2 \times 4^2}{6^2} = -8 \text{ kNm}$$

$$M_{fcb} = \frac{9 \times 2^2 \times 4}{6^2} = 4 \text{ kNm}$$

Since length of *BA* and *BC* is same, support at end *A* and *C*, also same and *EI* is constant, so, distribution factor for *BA* = 0.5 and *BC* = 0.5

	A	*B*	*B*	*C*
D.F.		0.5	0.5	
FEM	–6	6	–8	4
Bal. moment	+6			–4
carryover moment		+3	–2	
	0	9	–10	0
		0.5	0.5	
Final moment	0	9.5	–9.5	0

55. (b)

$$\text{Stiffness of } AB = \frac{I}{4}$$

$$\text{Stiffness of } CD = \frac{I}{6}$$

$$\text{Stiffness of } CD < \text{Stiffness of } AB$$

⇒ So, the frame will sway towards right.

57. (d)

Column analogy method is force method of analysis.

Section C : CPM PERT-2 + Hydrology and Water Resource Engg.-2

58. (d)

All the above mentioned statements are the objectives of time study.

Time study can be defined as an art of observing and recording the time required to perform an activity by a normal worker under normal conditions.

59. (c)

Muster roll is used to record the attendance and out turn of workers, for the purpose of payment of wages.

60. (a)

When the contractor fails to fulfill his contractual obligation in respect of progress or quality of work even after being given due notice by the owner, a debitable agency is appointed.

Liquidated damage is the fixed stipulated sum payable by the contractor on account of penalty for delays and does not bear any relationship with the real damage to the owner.

Unliquidated damage is known as ordinary damage having relation with the actual damage done resulting by breach of the contract.

61. (c)

The procedures adopted before execution of civil engineering works are

1. Administrative approval
2. Technical sanction
3. Expenditure sanction
4. Land acquisition
5. Preparation of tender documents etc.

The procedures adopted during execution of the works/project are

1. Supervision
2. Site order book
3. Quality control during construction
4. Progress report
5. Payment etc.

62. (b)

All the expenses related to the contractor's office and establishment are termed as general overhead costs which is a recurring expenditure and does not depend upon the volume of the work under execution.

Temporary sheds for materials and godown rents are covered under job overhead charges.

63. (a)

$$\text{Injury index} = \frac{\text{Frequency rate} \times \text{Severity rate}}{1000}$$

$$= \frac{12.5 \times 0.125}{1000} = 1.5625 \times 10^{-3}$$

64. (d)

Direct cost of an accident consists of compensation, medical and legal payments directly by the organisation or through their insurance schemes.

Indirect cost are not covered by the insurance. The cost of lost time of the injured employee and other employees, and the damage to the equipment, plant or property comes under indirect cost of an accident.

65. (a)

Cost of material is the part of direct cost.

66. (c)

Bligh's method holds good so long as the horizontal distance between the pile lines is greater than twice their depth.

67. (b)

We have, $$\alpha = \frac{b}{d} = \frac{20}{4} = 5$$

Hence, $$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

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

$$G_E = \frac{5}{4}\times\frac{1}{\pi\sqrt{3.05}}$$
$$= 0.228 \simeq 0.23$$

68. (a)

Kennedy did not give any slope equation.

69. (d)

$$Q_D = \frac{4k\left(b^2-a^2\right)}{L}$$

$$\frac{(Q_D)_A}{(Q_D)_B} = \frac{4k_A\left(b^2-a^2\right)L_B}{4k_B\left(b^2-a^2\right)L_A}$$
$$= \frac{2}{1}\times\frac{5}{6}\times 1.5 = 2.5$$

70. (b)

The difference in bed level of the canal and drainage is 4 m while the flow depth in drainage is 10 m. Thus HFL of drain at 125 m (115 m + 10 m) is higher than the canal bed at 120 m. Therefore syphon aqueduct is most suitable.

71. (b)

Total depth of water required by the crop = 108 cm

Contribution of rainfall during the base period = 22 cm

∴ Delta required, Δ = 108 – 22 = 86 cm = 0.86 m

∴ Duty of crop $= \frac{8.64\,B}{\Delta} = \frac{8.64\times 146}{0.86} = 1466.8$ ha/cumec

72. (c)

Rives on alluvial plains may be broadly classified into three types:

1. The aggrading type
2. The degrading type
3. The meandering type

73. (a)

Distribution efficiency, $\eta_d\% = \left(1-\frac{d}{D}\right)\times 100$

$$D = \frac{1.5+1.1}{2} = 1.3 \text{ m}$$

$$d = \frac{|1.5-1.3|+|1.1-1.3|}{2} = 0.2$$

$$\eta_d = \left(1 - \frac{0.2}{1.3}\right) \times 100$$
$$= 84.6\%$$

74. (d)

Once the offer of a bidder is accepted by the client, the bidder is required to deposit about 10% of the tender amount with the client. This amount is called security deposit.

While submitting the tender, the bidder will have to deposit an amount which is about 2% of the estimated contract value, of the project. This amount is called earnest money deposit.

❍❍❍❍