DETAILED SOLUTIONS

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ESE 2025 : Prelims Exam
CLASSROOM TEST SERIES

CIVIL ENGINEERING

Test 12

Section A : Design of Concrete and Masonry Structures [All Topics]

Section B : Structural Analysis-I [Part Syllabus]

Section C : CPM PERT-II + Hydrology and Water Resource Engg-II [Part Syllabus]

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

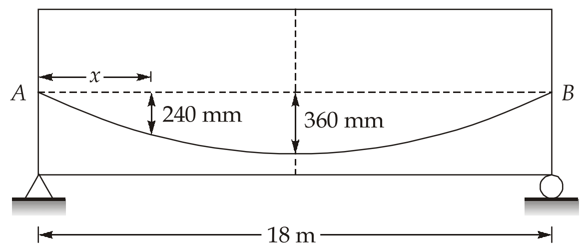
Q.21 :** ***Answer has been Updated.
Q.30 :** ***Answer has been Updated.
Q.64 :** ***Answer has been Updated.

DETAILED EXPLANATIONS

Section A : Design of Concrete and Masonry Structures

1. (a)

Given:



Equation of cable profile considering origin at A,

$$y = \frac{4hx(l-x)}{l^2}$$

$$\Rightarrow \quad 0.24 = \frac{4 \times 0.36x(18-x)}{18^2}$$

$$\Rightarrow \quad 54 = 18x - x^2$$

$$\Rightarrow \quad x^2 - 18x + 54 = 0$$

$$\therefore \quad x = \frac{18 \pm \sqrt{(-18)^2 - 4 \times 1 \times 54}}{2 \times 1}$$

$$\Rightarrow \quad x = \left(9 \pm 3\sqrt{3}\right)\text{m}$$

2. (c)

Let the prestressing force be P kN,

Now, intensity of equivalent load due to prestress acting upwards is

$$w_p = \frac{8Ph}{l^2} = \frac{8P \times 0.25}{10^2} = 0.02\text{P kN/m}$$

At balanced stage,

$$w_p = 20 \text{ kN/m}$$

$$\therefore \quad 0.02P = 20$$

$$\Rightarrow \quad P = 1000 \text{ kN}$$

3. (d)

Creep of concrete increase when.

- Cement content is high.
- Water-cement ratio is high.
- Aggregate content is low.
- Air entrainment is high.
- Relative humidity is low.
- Temperature is high.
- Loading occurs at an early age.

4. (c)

Refer Table 3 of IS 456 : 2000.

5. (b)

Doubly reinforced section is designed when:

- Depth restriction is there.
- High bending moment on the beam.
- Reversal of stress.
- Ductility requirement.
- Reduction in long term deflection.

6. (c)

As per IS 456 : 2000 (Cl 15.2.2)

$$\text{For first } 50 \text{ m}^3 \text{ of concrete} = 4 \text{ samples}$$

$$\text{Remaining volume} = 230 - 50 = 180 \text{ m}^3$$

For each additional 50 m^3 or part thereof, an extra 1 sample is needed.

$$\therefore \quad \frac{180}{50} = 3.6 \simeq 4 \text{ extra samples}$$

$$\therefore \quad \text{Total number samples} = 4 + 4 = 8 \text{ samples}$$

Three test specimens shall be made for each sample for testing of concrete at 28 days.

$$\therefore \quad \text{Total number of specimens} = 3 \times 8 = 24$$

7. (b)

Given: $b = 300$ mm, $d = 500$ mm

As per IS 456 : 2000 (Cl 23.3)

For a simply supported or continuous beam,

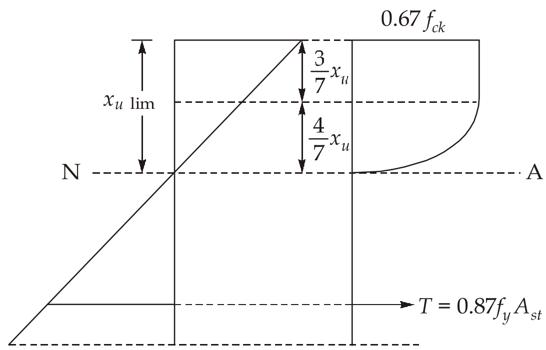
Clear distance between lateral restraints is

$$l \not> \min \begin{cases} \dfrac{250\, b^2}{d} \\ 60\, b \end{cases}$$

$$\Rightarrow \quad l_{max} = \min \begin{cases} \dfrac{250 \times 300^2}{500} \text{mm} = 45 \text{ m} \\ 60 \times 300 \text{ mm} = 18 \text{ m} \end{cases}$$

$$\Rightarrow \quad l_{max} = 18 \text{ m}$$

8. (b)



For limiting section,

Compressive force (with partial factor of safety = 1.5) = $0.36\, f_{ck}\, b\, x_u$

Compressive force (without partial factor of safety), $C = 1.5 \times 0.36 f_{ck}\, b\, x_u$

$$\Rightarrow \quad C = 1.5 \times 0.36 \times 25 \times 300 \times 0.48 \times 500 \text{ N}$$

$$\Rightarrow \quad C = 972 \text{ kN}$$

9. (a)

Refer IS 456 : 2000. (Cl 23.2)

10. (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**.

11. (c)

Given: $\lambda = 20 > 12$ (Long column)

As per IS 456 : 2000 Cl 39.7.1

The additional moments M_a shall be calculated by

$$M_a = \frac{P_u D}{2000}\left(\frac{l_{\text{eff}}}{D}\right)^2$$

$$\Rightarrow \quad M_a = \frac{1500 \times 0.5}{2000} \times (20)^2$$

$$\Rightarrow \quad M_a = 150 \text{ kN-m}$$

12. (c)

Given, $\phi = 20$ mm, $\tau_{bd} = 1.5$ N/mm^2, $f_y = 415$ N/mm^2

Design bond stress for Fe415 bar will be 1.6 × 1.5 = 2.4 N/mm^2

Since the bar is in compression, design bond stress will be increased by 25%.

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

$$\Rightarrow \qquad L_d = \frac{0.87 \times 415 \times 20}{4 \times 2.4 \times 1.25}$$

$$\Rightarrow \qquad L_d = 601.75 \text{ mm} \simeq 602 \text{ mm}$$

13. (b)

IS 456 : 2000 (Cl 26.5.3.1)

A reinforced concrete column having helical reinforcement shall have at least six bars of longitudinal reinforcement within the helical reinforcement.

14. (a)

Given:

Initial stress in cable, $\sigma_i = 1600 \text{ N/mm}^2$

For post tensioning, total residual shrinkage strain is given as,

$$\epsilon_{cs} = \frac{0.0002}{\log_{10}(t+2)} = \frac{2 \times 10^{-4}}{\log_{10}(8+2)}$$

$$\Rightarrow \qquad \epsilon_{cs} = 2 \times 10^{-4}$$

$$\therefore \qquad \text{Loss of stress, } \Delta\sigma = E_s \times \epsilon_{cs}$$

$$\Rightarrow \qquad \Delta\sigma = 2 \times 10^5 \times 2 \times 10^{-4}$$

$$\Rightarrow \qquad \Delta\sigma = 40 \text{ N/mm}^2$$

$$\therefore \qquad \text{Percentage loss of stress} = \frac{\Delta\sigma}{\sigma_i} = \frac{40}{1600} \times 100 = 2.5\%$$

15. (a)

The critical section for one way shear is at a distance of the effective depth of footing from the face of column.

The critical section for two way shear or punching shear is at a distance of half the effective depth of footing from the face of column.

16. (c)

Limiting percentage of tensile reinforcement for a beam is,

$$p_t = \frac{A_{st,\lim}}{bd} \times 100$$

$$\Rightarrow \qquad p_t = \frac{\left(\dfrac{0.362 f_{ck}\, b\, x_{u,\lim}}{0.87 f_y}\right)}{bd} \times 100$$

$$\Rightarrow \qquad p_t = 41.61 \left(\frac{f_{ck}}{f_y}\right)\left(\frac{x_{u,\lim}}{d}\right)$$

17. (a)

$$\text{Tie diameter, } \phi_t \geq \text{Max}\begin{cases} \dfrac{25}{4} = 6.25\text{ mm} \\ 6\text{ mm} \end{cases}$$

$$\therefore \quad \text{Provide, } \phi_t = 8\text{ mm}$$

$$\text{Tie spacing, } S_t \leq \min\begin{cases} 450\text{ mm} \\ 16 \times 20 = 320\text{ mm} \\ 300\text{ mm} \end{cases}$$

$$\therefore \quad \text{Provide, } S_t = 300\text{ mm}$$

$\therefore$ Provide 8 mmϕ ties @ 300 mm c/c

18. (c)

Refer IS 456:2000 (Cl. 26.2.5.1)

19. (a)

Tensile stress in concrete,

$$\sigma_{st} = \frac{T}{bD + (m-1)A_{st}}$$

$$= \frac{150 \times 10^3}{1000 \times 120 + (10-1) \times 4 \times \frac{\pi}{4}(20)^2}$$

$$= 1.14\text{ N/mm}^2$$

20. (d)

21. (b)

Refer IS456 : 2000 (Cl. 26.3.3 (b))

22. (b)

Given,

$$G = 2.8\text{ m}$$
$$x = 1.1\text{ m}$$
$$y = 0.8\text{ m}$$
$$\text{Effective span } (l) = G + x + y$$

where;

$$x = \min\begin{cases} x = 1.1\text{ m} \\ 1\text{m} \end{cases} = 1\text{ m}$$

$$y = \min\begin{cases} y = 0.8\text{ m} \\ 1\text{m} \end{cases} = 0.8\text{ m}$$

$$\therefore \quad l = 2.8 + 1 + 0.8$$

$$\Rightarrow \quad l = 4.6\text{ m}$$

23. (c)

Given $P_{working} = 1200$ kN

Factored load $P_u = 1.5 \times 1200 = 1800$ kN

Longitudinal steel reinforcement, $A_{sc} = 1\%\ A_g = 0.01\ A_g$ (where A_g is gross area of column)

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

$$\Rightarrow \quad 1800 \times 10^3 = 0.4 \times 25 (A_g - 0.01 A_g) + 0.67 \times 415 \times 0.01 A_g$$

$$\Rightarrow \quad 1800 \times 10^3 = (0.4 \times 25 \times 0.99 + 0.67 \times 415 \times 0.01)\frac{\pi}{4}D^2$$

$$\Rightarrow \quad D = 425.13 \text{ mm} \simeq 426 \text{ mm}$$

Round off to higher value. If $D = 425$ mm

then $P_u = 1798.89$ kN $\neq 1800$ kN

So, round-up the result obtained.

24. (b)

As per IS 456 : 2000 (Cl. 24.1), the following span to overall depth ratio values are as below:

	Fe250	Fe415
Simply supported two-way slabs	35	28
Continuous two way slabs	40	32

25. (d)

IS 1343 : Code of practice for prestress concrete.

IS 1893 : Criteria for earthquake resistance design of structures.

IS 3343 : Natural moulding sand for use in foundries.

IS 13920 : Ductile detailing of reinforced concrete structure.

26. (c)

Given: $q = 180$ kN/m^2, $\gamma = 18$ kN/m^3, $\phi = 30°$

Minimum depth of foundation is determined by the Rankine formula as follows.

$$\text{Depth of footing} = \frac{q}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2 = \frac{180}{18}\left(\frac{1-\sin 30°}{1+\sin 30°}\right)^2$$

$$= 1.11 \text{ m}$$

27. (b)

Given: Length of footing $(L) = 4$ m

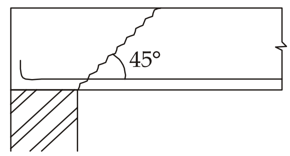
Width of footing $(B) = 3$ m

$$\text{Reinforcement provided in central band} = A_{st}\left(\frac{2}{\beta+1}\right) \qquad \text{where } \beta = \frac{L}{B} = \frac{4}{3}$$

$$= 4200 \times \left(\frac{2}{\frac{4}{3}+1}\right) = 4200 \times \frac{6}{7} = 3600 \text{ mm}^2$$

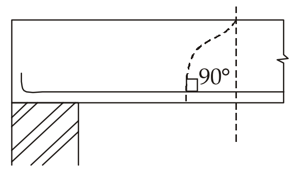
28. (a)

Diagonal tension failure occurs under large shear force and less bending moment.



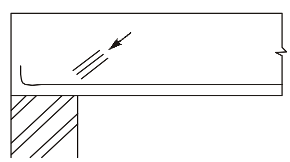
Diagonal tension failure

Flexural shear failure occurs under large bending moment and less shear force.



Flexure shear failure

Diagonal compression failure, which occurs under large shear force, is characteristics by crushing of concrete.



Diagonal compression failure

29. (d)

As per IS 456 : 2000

$$k = \begin{cases} 1.3; & \text{for } D \le 150 \text{ mm} \\ 1.6 - 0.002D; & \text{for } 150 \text{ mm} < D < 300 \text{ mm} \\ 1.0; & \text{for } D \ge 300 \text{ mm} \end{cases}$$

Where D is the overall depth of the slab

Given, $D = 200$ mm

$\therefore$ $k = 1.6 - 0.002 \times 200 = 1.2$

Alternative solution:

k value may be interpolated between 1.0 and 1.30 as:

$$k = 1.30 - \frac{(1.30 - 1.00)}{(300 - 150)} \times (200 - 150) = 1.20$$

30. (d)

Given: $L = 3$ m

$t_f = 160$ mm, $b_w = 300$ mm; $B = 1$ m

Effective width of flange

$$B_f = \frac{l_o}{\left(\frac{l_o}{B}+4\right)} + b_w$$

where $l_o = 0.7\,L$ (for continuous beam)

$= L$ (for isolated beam)

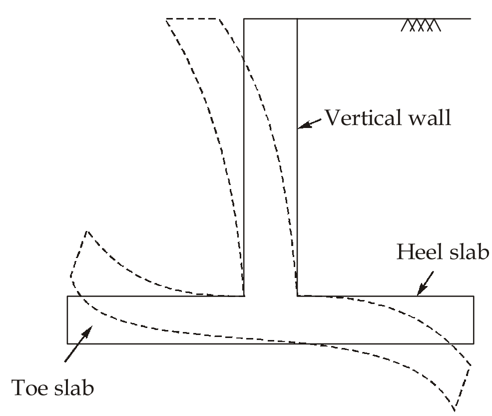
$$\therefore \quad B_f = \frac{0.7\times 3000}{\left(\frac{0.7\times 3000}{1000}+4\right)} + 300$$

$\Rightarrow \quad B_f = 644.26 \text{ mm} < 1000 \text{ mm}$

$\therefore$ Effective width of flange, $B_f = 644.26 \text{ mm} \simeq 650 \text{ mm}$

31. (a)

T-shaped cantilever retaining wall:



From the above diagram:

- The moment induced in the vertical wall causes tension on the face of the earth retained.
- The heel slab is subjected to net downward pressure that causes tension at the top of the slab.
- The toe slab is subjected to net upward pressure that causes tension at the bottom of the slab

32. (a)

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

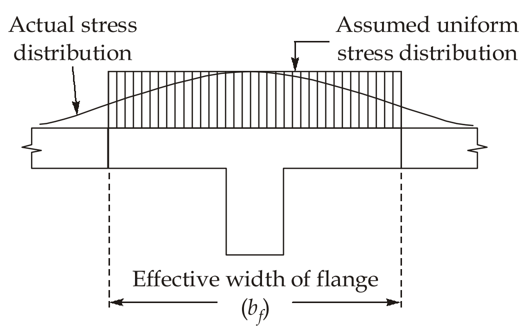
$$f_{cr} = 0.7\sqrt{f_{ck}} = 0.7\sqrt{40}$$

$$= 4.43 \text{ MPa}$$

33. (b)

34. (a)

The flexural compressive stress is not uniform over its width. The stress varies from a maximum value in the web region to progressively lower values at points farther away from the web on either side of the web. Thus uniform stress distribution is assumed across the width of the section called as effective width of flange.



35. (d)

Bends and hooks are ineffective in anchoring bars in compression. Hence, **Clause 26.2.2.2** of IS 456:2000 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.

36. (b)

Factor $\sqrt{\dfrac{A_1}{A_2}}$ is limited to 2, because very high axial compressive stresses give rise to transverse tensile strains which may lead to spalling, lateral splitting or bursting of concrete.

Section B : Structural Analysis-I

37. (c)

Degree of static indeterminacy

$$D_s = (3m + R) - (3j + r)$$

where, r = extra equations due to internal hinge

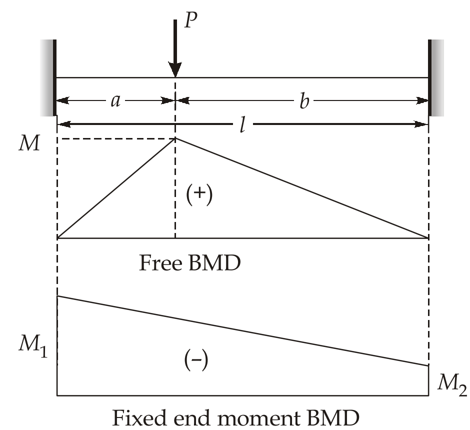
$$\Rightarrow \quad D_s = (3 \times 22 + 9) - (3 \times 17 + 4)$$

$$\Rightarrow \quad D_s = 20$$

External indeterminacy, $D_{se} = R - 3 = 9 - 3 = 6$

Internal indeterminacy, $D_{si} = D_s - D_{se} = 20 - 6 = 14$

38. (a)



$\because$ Slope at fixed end is zero.

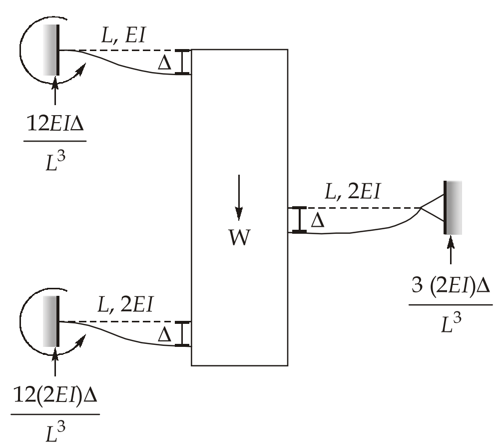
$\therefore$ Area of free BMD = Area of fixed end moment BMD

$$\Rightarrow \quad \frac{1}{2}Ml = \frac{1}{2}(M_1 + M_2)l$$

$$\Rightarrow \quad M_1 + M_2 = M$$

39. (d)

Deflected shape of the system is shown below.



For equilibrium, $\Sigma F_y = 0$

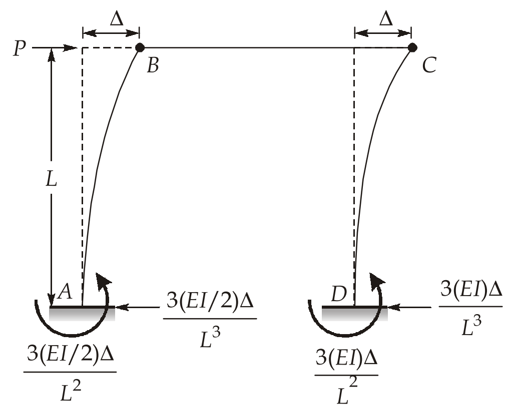
$$\Rightarrow \quad \frac{12EI\Delta}{L^3} + \frac{24EI\Delta}{L^3} + \frac{6EI\Delta}{L^3} = W$$

$$\Rightarrow \quad \frac{42EI\Delta}{L^3} = W$$

$$\Rightarrow \quad \Delta = \frac{WL^3}{42EI}$$

40. (b)

Deflected shape of the frame can be drawn as below.



For equilibrium, $\Sigma F_x = 0$

$$\Rightarrow \quad \frac{3(EI/2)\Delta}{L^3} + \frac{3EI\Delta}{L^3} = P$$

$$\Rightarrow \quad \frac{9EI\Delta}{2L^3} = P$$

$$\Rightarrow \quad \Delta = \frac{2PL^3}{9EI}$$

Now, Moment of A, $M_A = \frac{3EI\Delta}{2L^2} = \frac{3EI}{2L^2} \times \frac{2PL^3}{9EI} = \frac{PL}{3}$

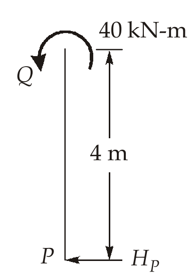
$$\Rightarrow \quad M_A = \frac{500 \times 3}{3}$$

$$\Rightarrow \quad M_A = 500 \text{ kN-m}$$

41. (b)

Let H_s and H_P be the horizontal reactions at S and P respectively.

Considering joint equilibrium at joint Q,



$$M_{QR} + M_{QP} = 0$$

$\Rightarrow \quad M_{QP} = 40$ kN-m (Anticlockwise)

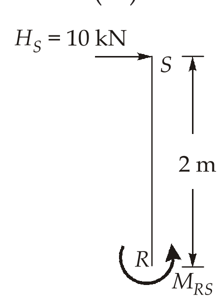
$$\Sigma M_Q = 0$$

$$\Rightarrow \quad H_P \times 4 = 40$$

$\Rightarrow \quad H_P = 10$ kN ($\leftarrow$)

Also, $\Sigma F_x = 0$

$\Rightarrow \quad H_S = 10$ kN ($\rightarrow$)



Now, $\Sigma M_R = 0$

$\Rightarrow$ $M_{RS} = 10 \times 2$

$\Rightarrow$ $M_{RS} = 20$ kNm (Anticlockwise)

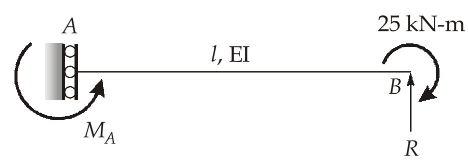
42. (b)

Distribution factor:

Joint	Member	Stiffness	Total stiffness	D.F.
B	BA	$\frac{EI}{l}$	$\frac{8EI}{l}$	$\frac{1}{8}$
	BD	$\frac{4EI}{l}$		$\frac{4}{8}$
	BE	$\frac{3EI}{l}$		$\frac{3}{8}$
	BC	0		0

$$\therefore \quad (DF)_{BA} = \frac{1}{8}$$

$$\therefore \quad M_{BA} = 200 \times \frac{1}{8} = 25 \text{ kN-m}$$



Now by considering equilibrium of member *BA*.

$$-M_A + 25 = 0$$

$$\Rightarrow \quad M_A = 25 \text{ kN-m} \quad \text{(Anticlockwise)}$$

43. (a)

$$\frac{M_B}{M_Q} = \frac{\frac{4(2EI)}{4} + \frac{3(EI)}{3}}{\frac{4(2EI)}{2} + \frac{3(2EI)}{3} + \frac{4(EI)}{4}}$$

$$\Rightarrow \quad \frac{M_B}{M_Q} = \frac{3}{7}$$

44. (b)

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

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

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

45. (a)

Determinate structures:

- The number of unknown support reactions or internal forces can be determined using equilibrium equations only.
- They do not experience significant secondary stresses.
- They are more flexible (less stiffer) but less efficient in resisting loads.

Indeterminate structures:

- They have more unknowns than that can be determined using equilibrium equations and thus require compatibility equations for analysis.
- These structures are stiffer and experience additional internal stresses due to temperature variations, settlement of supports etc.
- They require more material thereby increasing the construction cost.

46. (c)

- Column analogy method is not limited to frames only but it can also be used for beams and fixed arches.
- If degree of indeterminacy is higher than 3, then this method becomes complex and less practical.

47. (b)

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

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

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

So, the frame will sway towards right.

48. (d)

49. (a)

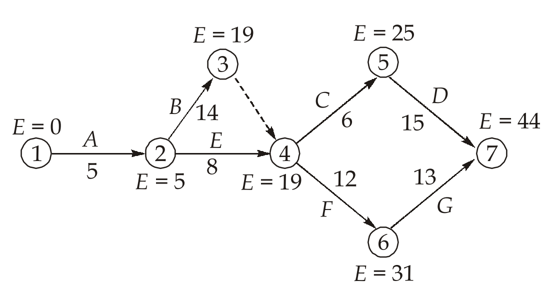
50. (a)

A cable is a flexible structure that can not resist bending and shear. It can only carry pure tension under a uniformly distributed load such as self weight or external loading and thus the shape of the cable is a parabola.

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

51. (c)

Network diagram can be drawn as shown below.



The longest path is A → B → F → G and its duration is 44 days.

52. (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.

53. (b)

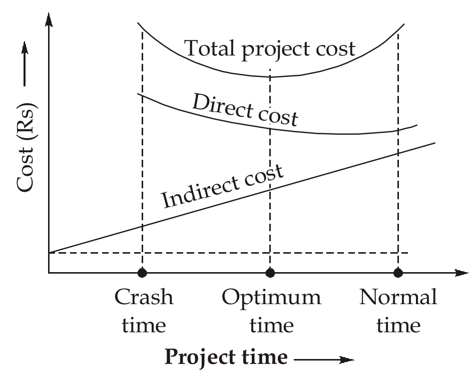
All the expenses related to the contractor's office and establishment are termed as general overhead costs which is a recurring expenditure and does not depend upon the volume of the work under execution.

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

54. (d)

A-O-N system (precedence diagram) of the network completely eliminates the use of dummy activities.

55. (c)



- Decrease/Increase in project duration from optimum duration increases the total project cost.

56. (c)

57. (a)

Cost of material is the part of direct cost.

58. (b)

Statement 3 is incorrect because profitability depends on broader factors (e.g. Revenue, market demand) not directly on rate analysis, which focuses on cost estimation.

59. (b)

$$\sigma_{max.} = \gamma_w H(G_c - c + 1)$$

$$\Rightarrow \quad H_{max.} = \frac{\sigma_{max.}}{\gamma_w (G_c - c + 1)}$$

$$\Rightarrow \quad H_{max.} = \frac{300\,\text{kN/m}^2}{(9.81\,\text{kN/m}^3)(2.5 - 0.5 + 1)}$$

$$\Rightarrow \quad H_{max.} = 10.19 \text{ m}$$

$$\text{Height of dam } (H = 12 \text{ m}) > H_{max.}$$

i.e. unsafe condition.

Hence, high gravity dam.

60. (d)

61. (a)

The outlet should be such as to avoid interference by cultivators, thus preventing wrongful tapping of water by cultivator.

62. (c)

Given:Culturable command area, CCA = 1200 ha

$$\text{Area irrigated under sugarcane} = 1200 \times \frac{20}{100} = 240 \text{ ha}$$

$$\text{Area irrigated under wheat} = 1200 \times \frac{40}{100} = 480 \text{ ha}$$

$$\text{Duty of sugarcane} = 730 \text{ ha/cumec}$$

$$\text{Discharge required for sugarcane} = \frac{240}{730} = 0.329 \text{ cumec}$$

$$\text{Duty of wheat} = 1800 \text{ ha/cumec}$$

$$\text{Discharge required for wheat} = \frac{480}{1800} = 0.267 \text{ cumec}$$

$$\therefore \quad \text{Total discharge required} = 0.329 + 0.267 = 0.596 \simeq 0.6 \text{ cumec}$$

63. (d)

For time factor 0.8, the channel runs for fewer days than the crop days.

$$\therefore \quad \text{Design discharge} = \frac{0.6}{0.8} = 0.75 \text{ cumec}$$

64. (a)

Eddy viscosity is defined as the rate of mass exchange per unit area between adjacent layers.

65. (a)

Soil cement lining is a type of earth type linings

66. (b)

Attracting groyne points downstream and repelling groyne points upstream.

67. (a)

This type of escapes are preferred these days, as they give better control and can be used for completely emptying the canal.

68. (b)

Allowable compressive stress in dam if it is of concrete is generally taken as 3000 kN/m^2.

69. (d)

Both (a) and (b) are adopted for both reservoir full and reservoir empty condition of stability analysis by gravity method.

70. (c)

$$\text{For no scouring in channel, } d = 11\, RS$$

$$\Rightarrow \quad R = d/11S$$

$$\text{By Strickler's formula, } n = \frac{d^{1/6}}{24}$$

$$\text{Using Manning's equation,} \quad V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$\Rightarrow \quad V = \left(\frac{24}{d^{1/6}}\right) \times \left(\frac{d}{11S}\right)^{2/3} \times S^{1/2}$$

$$\Rightarrow \quad V = 4.85\, d^{1/2}\, S^{-1/6}$$

71. (d)

$$\text{Exit gradient, } G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

$$\text{where,} \quad \lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\alpha = \frac{b}{d} = \frac{10}{5} = 2$$

$$\therefore \quad \lambda = \frac{1+\sqrt{1+2^2}}{2} = \frac{1}{2} + \frac{\sqrt{5}}{2} \simeq 1.62$$

$$\Rightarrow \quad G_E = \frac{8}{5} \times \frac{1}{\pi\sqrt{\lambda}} \simeq 0.4 = \frac{2}{5}$$

72. (b)

Compared to an aqueduct, a super passage is inferior and should be avoided wherever possible.

73. (c)

Sinking fund factor

$$SFF = \frac{A}{F} = \frac{i}{(1+i)^n + 1}$$

Where; A = A single payment in a series of 'n' equal payments

F = Future sum of money.

i = Interest rate per period

n = number of periods

74. (a)

75. (b)

A weir or a barrage may fail not only due to seepage (i.e., sub-surface flow) as stated by Bligh, but may also fail due to the surface flow. The surface flow may cause scour, dynamic action and in addition will cause uplift pressure.

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