CLASS TEST

S.No.:02_IGCE_DEF_20092024

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REINFORCED CEMENT CONCRETE

CIVIL ENGINEERING

Date of Test : 20/09/2024

ANSWER KEY

1.	(b)	7.	(d)	13.	(a)	19.	(d)	25.	(c)
2.	(d)	8.	(d)	14.	(b)	20.	(a)	26.	(a)
3.	(c)	9.	(b)	15.	(b)	21.	(a)	27.	(c)
4.	(d)	10.	(a)	16.	(a)	22.	(a)	28.	(b)
5.	(b)	11.	(d)	17.	(c)	23.	(d)	29.	(d)
6.	(b)	12.	(a)	18.	(c)	24.	(a)	30.	(c)

DETAILED EXPLANATIONS

1. (b)

As per IS-456:2000; Table - 3 and table -16, nominal cover $\ngtr$ 30 mm.

and according to clause 26.4.2.1 of IS-456 : 2000, nominal cover $\nless$ 40 mm for column in any case.

So, here, answer will be 40 mm.

2. (d)

- Lap length of *r*/*f* in tension shall not be less than 30ϕ.
- If three bars are bundled together, development length shall be increased by 29%.
- If 2 bars are bundled then 10% if 3 bars are bundled then 20% and if 4 bars are bundled then 33%.

3. (c)

4. (d)

5. (b)

For balancing load, $$w = \frac{8Pe}{L^2} \Rightarrow P = \frac{wL^2}{8e}$$

Here, $$M = \frac{wL^2}{8}$$ [For *udl* on SS beam]

So, $$P = \frac{M}{e}$$

6. (b)

Pitch of helical reinforcement,

1. $p \ngtr 75$ mm
2. $p \ngtr \frac{d_c}{6}$
3. $p \nless 3\phi_h$ mm
4. $p \nless 25$ mm

∴ Pitch lies between 25 to 75 mm.

7. (d)

Side face reinforcement is provided when depth ≥ 750 mm (Without torsion)

or Depth = 450 mm (with torsion)

8. (d)

9. (b)

Maximum spacing for vertical stirrups, $k_1 = 0.75d$ and for inclined stirrups, $k_2 = d$

So, $$\frac{k_1}{k_2} = 0.75$$

10. (a)

Given: B = 200 mm, d = 500 mm, l_{eff} = 6 m, Total load = 20 kN/m

Factored load = 1.5 × 20 = 30 kN/m

$$(BM)_{max} = \frac{wl_{eff}^2}{8} = \frac{30 \times 6^2}{8} = 135 \text{ kNm}$$

Maximum bending moment capacity of balanced beam, for Fe415,

$$M_{cr} = 0.138\, f_{ck} B d^2$$
$$= 0.138 \times 25 \times 200 \times 500^2 \times 10^{-6}$$
$$= 172.5 \text{ kNm}$$
$$(BM)_{max} < M_{cr}$$

∴ URS is provided

11. (d)

- Critical section for one-way shear is at distance 'd' from the face of the column.
- Critical section for maximum bending moment under masonary wall is located at mid-way between the face and middle of wall.

12. (a)

Effective bending moment due to torsion,

$$M_t = \frac{T_u}{1.7}\left(1 + \frac{D}{b}\right) = 259 \text{ kNm}$$

Equivalent bending moment at bottom is,

$$M_e = M_t + M_u$$
$$= 259 + 200 = 459 \text{ kN-m}$$

13. (a)

Width of web, b_w = 250 mm

Effective depth, d = 120 + 380 – 40 = 460 mm

$$\%p_t = \frac{100A_{st}}{bd} = \frac{100 \times 3 \times \frac{\pi}{4} \times 22^2}{250 \times 460} = 0.99\%$$

Nominal shear stress, $\tau_v = \frac{V_u}{b_w d} = \frac{200 \times 1000}{250 \times 460} = 1.74 \text{ N/mm}^2$

Design shear stress, $\tau_{us} = \tau_v - \tau_c = 1.74 - 0.62 = 1.12 \text{ N/mm}^2$

14. (b)

Refer table 3, Clause 8.2.2.1 of IS 456 : 2000.

15. (b)

For solid slabs, to control deflection, span to overall depth ratios are given as:

	Mild steel	HYSD
SS slab	35	28
Continuous slab	40	32

16. (a)

Over-reinforced section is the one in which the area of tensile steel, is such that, ultimate compressive strain in concrete is reached, however the tensile strain in the reinforcing steel is less than the yield strain (ε_y).

17. (c)

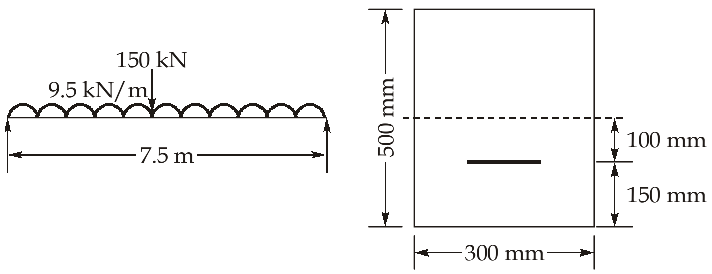
For isolated L-beam

$$(b_{\text{eff}})_f = b_w + \frac{0.5 l_o}{\frac{l_o}{b_f} + 4} \ngtr b_f$$

Given: l_o = 6 m, b_f = 1000 mm, b_w = 300 mm

$$(b_{eff})_f = 300 + \frac{0.5 \times 6000}{\frac{6000}{1000} + 4} = 600 \text{ mm} \ngtr 1000 \text{ mm} \left(= b_f\right) \qquad \text{(OK)}$$

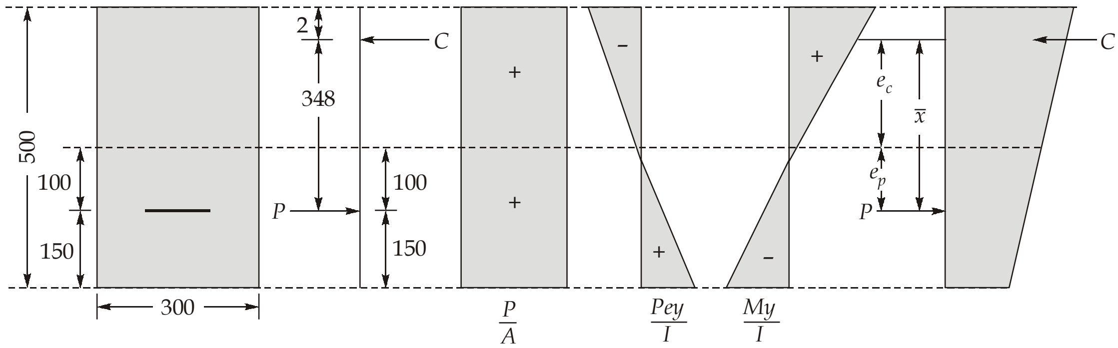
18. (c)



$$M_0 = \frac{wl^2}{8} + \frac{WL}{4} = \frac{9.5 \times 7.5^2}{8} + \frac{150 \times 7.5}{4} = 348.05 \text{ kN-m}$$

$$\bar{x} = \frac{M}{P} = \frac{348.05 \times 10^6}{1000 \times 10^3} = 348 \text{ mm}$$

$\therefore \quad \bar{x}$ from top $= 500 - 150 - 348 = 2$ mm



(All dimensions are in mm)

19. (d)

For axially loaded column,

$$e_{min} = \max \begin{cases} \frac{L}{500} + \frac{B \text{ or } D}{30} < 0.05\,(B \text{ or } D) \\ 20 \text{ mm} \end{cases}$$

$$= \max \begin{cases} \frac{3000}{500} + \frac{400}{30} = 19.33 < 0.05(B \text{ or } D) = 20 \text{ mm} \\ 20 \text{ mm} \end{cases}$$

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

$P_u = 0.4 f_{ck} [A_g - A_{sc}] + 0.67 f_y A_{sc}$

where A_c = Area of concrete

A_g = Gross area of column

A_{sc} = Area of compression steel

$1650 \times 10^3 = 0.4 \times 20\,[400^2 - A_{sc}] + 0.67\,(500)\,A_{sc}$

$1650 \times 10^3 = 1280000 - 8\,A_{sc} + 335\,A_{sc}$

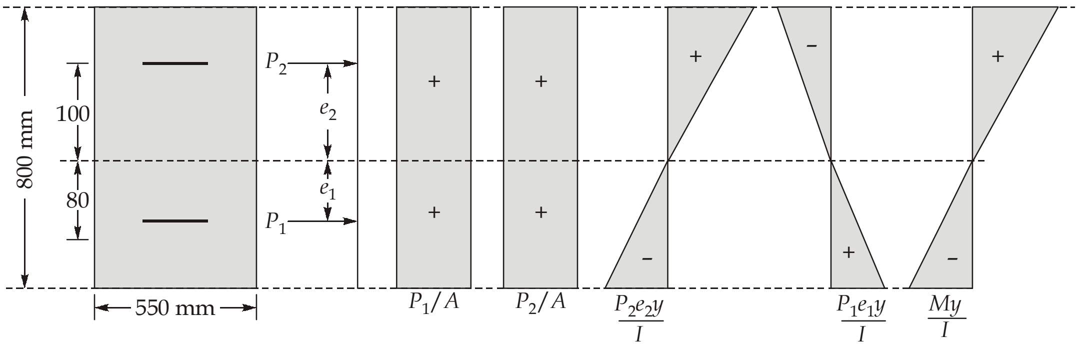
$A_{sc} = 1131.498 \text{ mm}^2 \simeq 1131.50 \text{ mm}^2$

But as per IS 456, $(A_{sc})_{min}$ = 0.8% of cross-sectional area

$$= \frac{0.8}{100} \times 400^2 = 1280 \text{ mm}^2$$

$\therefore \quad A_{sc} = 1280 \text{ mm}^2$

20. (a)



$$P_1 = P_2 = 2500 \times 10^3 \text{ N}$$
$$A = 550 \times 800 = 44 \times 10^4 \text{ mm}^2$$
$$y = \frac{800}{2} = 400 \text{ mm}$$
$$e_1 = 80 \text{ mm}$$
$$e_2 = 100 \text{ mm}$$
$$I = \frac{BD^3}{12} = \frac{550 \times 800^3}{12} = 23.47 \times 10^9 \text{ mm}^4$$
$$M = 800 \times 10^6 \text{ N-mm}$$

Stress at top,
$$\sigma_t = \frac{P_1}{A} + \frac{P_2}{A} + \frac{P_2 e_2 y}{I} - \frac{P_1 e_1 y}{I} + \frac{My}{I}$$

$$\Rightarrow \sigma_t = \frac{2500 \times 10^3}{44 \times 10^4} \times 2 + \frac{2500 \times 10^3 \times 100 \times 400}{23.47 \times 10^9} - \frac{2500 \times 10^3 \times 80 \times 400}{23.47 \times 10^9} + \frac{800 \times 10^6 \times 400}{23.47 \times 10^9}$$

$$\Rightarrow \sigma_t = 25.86 \text{ N/mm}^2$$

21. (a)

τ_{bd} for M25 = 1.4 N/mm²

It has to increased by 60% as HYSD bar is used.

$$\therefore \quad L_d = \frac{0.87 f_y}{4\tau_{bd}} \phi = \frac{0.87 \times 500}{4 \times (1.4 \times 1.6)} \phi = 48.55\phi$$

22. (a)

$$\text{Required footing area} = \frac{1.1P}{q_u}$$

where P = Column load

Self-weight of footing and backfill soil considered is 10% of column load.

$$\therefore \quad A = \frac{1.1 \times 2000}{210} = 10.48\ \text{m}^2$$

$$\frac{L}{B} = \frac{600}{400} = \frac{3}{2}$$

$$\therefore \quad L \times B = 10.48$$

$$\Rightarrow \quad \frac{3}{2}B \times B = 10.48$$

$$\therefore \quad B = 2.64\ \text{m}$$

and L = 3.96 m

Hence, nearest answer is option (a).

23. (d)

Given: h = 300 mm, L = 15 m, y = 130 mm,

Equation of parabolic cable is

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

$$0.13 = \frac{4(0.3)(x)(15-x)}{15^2}$$

$$\Rightarrow \quad 29.25 = 18x - 1.2x^2$$

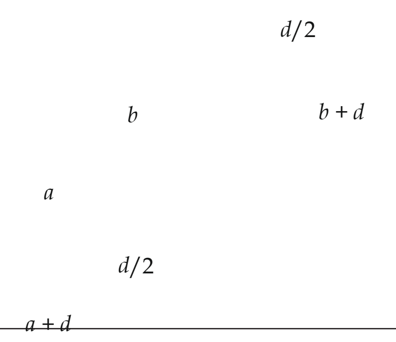
$$\Rightarrow \quad 1.2x^2 - 18x + 29.25 = 0$$

$$\therefore \quad x = 1.85\ \text{m}$$

24. (a)

$$(\tau_{ve})_{developed} = \frac{P_o - w_o[(a+b)(b+d)]}{2(a+d+b+d)\times d}$$

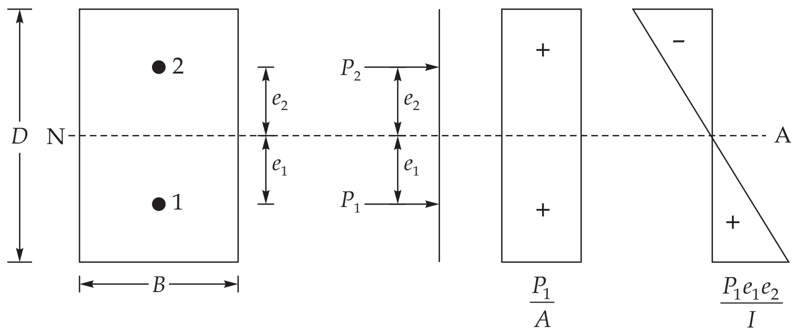
$\because$ Critical section for two way shear will be at $\frac{d}{2}$ distance from column face



$$(\tau_{ve})_{developed} = \frac{1300 - 205[(0.4+0.75)(0.5+0.75)]}{2[0.75\times(0.4+0.75+0.5+0.75)]}\ \text{kN/m}^2$$

$$= 0.279\ \text{N/mm}^2 \simeq 0.28\ \text{N/mm}^2$$

25. **(c)**



$$\therefore \quad f_c = \frac{P_1}{A} - \frac{P_1 e_1 e_2}{I} \quad \text{(Stress in wire-2 due to tensioning in wire-1)}$$

$$\text{Loss due to elastic shortening} = mf_c = m\left(\frac{P_1}{A} - \frac{Pe_1 e_2}{I}\right)$$

where m = Modular ratio

26. **(a)**

$$\delta = \left(1 + \frac{3P_u}{f_{ck}BD}\right) \le 1.5 = \left(1 + \frac{3\times15\times500\times10^3}{25\times300\times500}\right) \le 1.5$$

$$\delta = 1.6 \not> 1.5$$

So, $\delta = 1.5$

27. **(c)**

Value of partial safety factor (γ_f) for loads under various load combinations:

Load combination	Limit state of collapse			Limit state of sericieability		
	DL	IL	WL/EL	DL	IL	WL/EL
DL + IL	1.5	1.5		1	1	
DL + WL/EL	1.5 or 0.90	-	1.50	1	-	1
DL + IL + WL/FL	1.2	1.2	1.2	1.0	0.8	0.8

28. **(b)**

Loss of shrinkage of concrete in post-tensioned PSC beam

$$= \frac{(2\times10^{-4})E_s}{\log(T+2)} = \frac{(2\times10^{-4})\times(2\times10^5)}{\log(8)} = 44.45\ \text{N/mm}^2$$

$$\%\text{loss} = \frac{44.45}{1000}\times100 = 4.45\%$$

29. (d)

30. (c)

At failure in URS beam,

$$\text{Strain in steel} >> 0.002 + \frac{0.87 f_y}{E_s}$$

∴ Failure occurs due to secondary compression failure of concrete.

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