CLASS TEST

Sl. : 03DLSK_CE_OMNPQ_131125

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REINFORCED CEMENT CONCRETE

CIVIL ENGINEERING

Date of Test : 13/11/2025

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (a)

In partially prestressed members, tensile stresses are permitted in concrete under service loads with control on the maximum width of crack. The additional reinforcement is required in the cross-section for various reasons such as to resist differential shrinkage, temperature effects and handling stresses.

2. (a)

3. (b)

At the time of initial tensioning, the maximum tensile stress immediately behind the anchorages should not exceed 80% of the ultimate tensile strength of the wire.

4. (c)

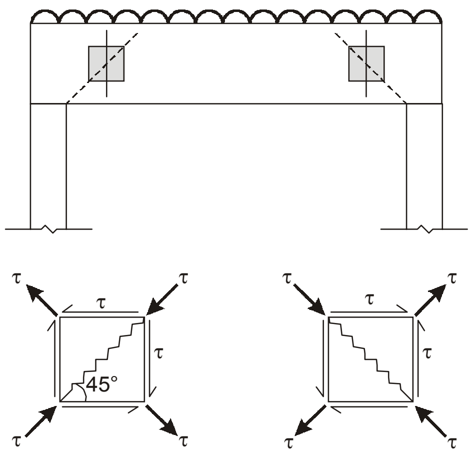
The limiting principal tensile stress in an uncracked prestressed concrete member is given by

$$f_t = 0.24\sqrt{f_{ck}} = 0.24\sqrt{35} = 1.42 \text{ MPa}$$

5. (a)

Diagonal tension failure occurs due to large shear force and lesser bending moment.

It can be seen in the case of pure shear {flexure tensile stress = 0} that maximum tension occurs along the diagonal.



Pure shear case

6. (b)

$$\text{SF along edge } BC = \frac{wl_{BC}}{4} = \frac{12\times4}{4} = 12 \text{ kN}$$

7. (a)

Pitch of helical turns shall not be more than

$$\left.\begin{array}{l}\text{(i) 75 mm}\\ \text{(ii) one-sixth of the core diameter of column } \dfrac{480}{6} = 80 \text{ mm}\end{array}\right\} = 75 \text{ mm}$$

Pitch of helical turns shall not be less than
(i) 25 mm
(ii) 3 times the diameter of steel bar forming the helix
So, maximum pitch is 75 mm.

8. (a)

$$\frac{A_{sv}}{b \cdot S_V} \geq \frac{0.4}{0.87\, f_y}$$

$$\Rightarrow \quad A_{sv} \geq \frac{0.4 \times b \times S_V}{0.87 \times f_y}$$

$$\Rightarrow \quad A_{sv} \geq \frac{0.4 \times 400 \times 100}{0.87 \times 415} = 44.3 \text{ mm}^2 \simeq 45 \text{ mm}^2$$

$\therefore$ Minimum shear reinforcement = 45 mm^2

9. (b)

10. (d)

$$p_{t\,lim} = 41.61\left(\frac{f_{ck}}{f_y}\right)\left(\frac{x_{u\,lim}}{d}\right)$$

$$= 41.61\left(\frac{35}{415}\right)(0.48) = 1.68\%$$

11. (c)

$$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{20 \times (0.87 \times 415)}{4 \times 1.4 \times (1.6 \times 1.25)} = 644.73 \text{ mm}$$

According to **IS 456:2000**, clause 26.2.1.2, the development length should be increased by 33% for four bars in contact.

$$L_d = 1.33 \times 644.73$$
$$= 857.5 \text{ mm}$$

12. (a)

For an isolated T-beam

Effective width of flange, $b_f = \dfrac{l_0}{\dfrac{l_0}{b} + 4} + b_w$

$$b_f = \frac{8000}{\dfrac{8000}{1300} + 4} + 250 = 1037.9 \text{ mm} < 1300 \text{ mm} \quad \text{(OK)}$$

13. (a)

Effective depth = 400 – 40 = 360 mm

Equilibrium of forces, $C = T$

$$\Rightarrow \quad 0.36\, f_{ck} b x_u = 0.87\, f_y A_{st}$$

$$\Rightarrow \quad x_u = \frac{0.87 \times 415 \times 4 \times \frac{\pi}{4} \times 20^2}{0.36 \times 20 \times 200}$$

$$= 315.08 \text{ mm}$$

For Fe415, $x_{u,lim} = 0.48d$

$$= 0.48 \times 360$$
$$= 172.8 \text{ mm} < x_u$$

$\therefore$ Section is over-reinforced.

Allowable moment resisting capacity,

$$M_{u,\ lim} = 0.36 f_{ck} x_{lim} (d - 0.42 x_{max}) \times b$$
$$= 0.36 \times 20 \times 172.8 \times (360 - 0.42 \times 172.8) \times 200 = 71.52 \text{ kNm}$$

Alternatively,

For Fe415 steel,

$$M_{u,\ lim} = 0.138 f_{ck} bd^2$$
$$= 0.138 \times 20 \times 200 \times 360^2$$
$$= 71.54 \text{ kNm}$$

Another check for over-reinforced section

$$A_{st} = 4 \times \frac{\pi}{4} \times 20^2 = 1256.64 \text{ mm}^2$$

$$\therefore \quad p_t = \frac{A_{st}}{bd} \times 100 = \frac{1256.64}{200 \times 360} \times 100 = 1.75\%$$

$$p_{t\ lim} = 41.61 \left(\frac{f_{ck}}{f_y}\right) \frac{x_{u\,lim}}{d} = 41.61 \left(\frac{20}{415}\right) 0.48$$
$$= 0.96\% < p_t$$

$\therefore$ Over-reinforced section.

14. (b)

15. (b)

Bending moment at the mid span,

$$M = \frac{wl^2}{8} = \frac{15 \times 14^2}{8} = 367.5 \text{ kNm}$$

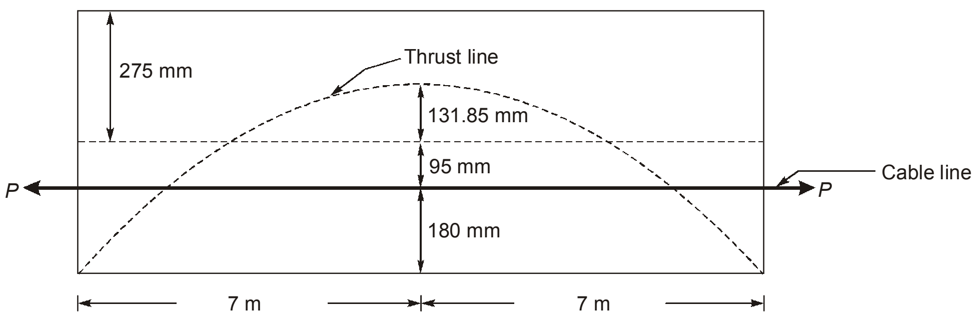
Prestressing force, $P = A_{st} f_s$

$$= \frac{1800 \times 900}{1000} = 1620 \text{ kN}$$

Shift of thrust line, $a = \frac{M}{P} = \frac{367.5 \times 10^6}{1620 \times 10^3} = 226.85$ mm

Eccentricity = 275 – 180 = 95 mm

$\therefore$ Eccentricity of thrust line = 226.85 – 95 = 131.85 mm



16. (a)

$$\text{Area of footing} = 2 \times 3 = 6\,\text{m}^2$$

$$\text{Section modulus, } Z = \frac{b \times d^2}{6} = \frac{2 \times 3^2}{6} = 3\text{m}^3$$

$$\text{Stress, } \sigma = \frac{P}{A} \pm \frac{M}{Z}$$

$$95 = \frac{P}{A} + \frac{M}{Z} \qquad \ldots(i)$$

$$55 = \frac{P}{A} - \frac{M}{Z} \qquad \ldots(ii)$$

From (*i*) and (*ii*), we get

$$\Rightarrow \quad 40 = 2 \times \frac{M}{Z}$$

$$\Rightarrow \quad 40 = 2 \times \frac{M}{3}$$

$$\Rightarrow \quad M = 60 \text{ kNm}$$

17. (c)

$$\text{Area of steel, } A_{st} = 3 \times \frac{\pi}{4} \times 20^2 = 942.5 \text{ mm}^2$$

For Fe 415, $x_{u,\max} = 0.48\,d$

$\Rightarrow$ $x_{u,\max} = 0.48 \times 400 \text{ mm} = 192 \text{ mm}$

Now,Compressive force, $C = 0.36\, f_{ck}\, B x_u$

Tensile force, $T = 0.87\, f_y\, A_{st}$

Since, $C = T$

$$\Rightarrow \quad x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} B} = 151.24 \,\text{mm}$$

As $x_u < x_{u,\max}$, the section is under-reinforced.

18. (a)

$$\text{Strain due to shrinkage} = \frac{2 \times 10^{-4}}{\log_{10}(t+2)} = 2 \times 10^{-4}$$

$\therefore$ Loss of prestress, $\Delta\sigma = 2 \times 10^{-4} \times E_s$

$= 2 \times 10^{-4} \times 2 \times 10^5 = 40 \text{ N/mm}^2$

$$\therefore \quad \text{Percentage of prestress loss} = \frac{40 \times 100}{500} = 8\%$$

19. (d)

Factored shear force $= 1.5 \times 110 = 165$ kN

Effective depth $= 500 - 35 = 465$ mm

$$A_{st} = 2 \times \frac{\pi}{4} \times (20)^2 = 628.32 \text{ mm}^2$$

Characteristic strength of steel, $f_y = 415 \text{ N/mm}^2$

Moment of Resistance, $M_u = 0.87\, f_y\, A_{st}\, (d - 0.42 x_u)$

Depth of neutral axis, $x_u = \dfrac{0.87\ f_y A_{st}}{0.36\ f_{ck} b} = \dfrac{0.87 \times 415 \times 628.32}{0.36 \times 20 \times 250}$

$= 126.03 \text{ mm} < x_{u,\lim}\ (x_{u,\lim} = 0.48\text{d} = 0.48 \times 465 = 223.2 \text{ mm})$

$M_u = 0.87 \times 415 \times 2 \times \dfrac{\pi}{4} \times (20)^2 \times (465 - 0.42 \times 126.03)$

$= 93.48 \times 10^6 \text{ N-mm} = 93.48 \text{ kNm}$

The anchorage value of a standard *U*-type hook is equal to 16 ϕ.

($\because$ For every 45° bend, anchorage value is 4ϕ)

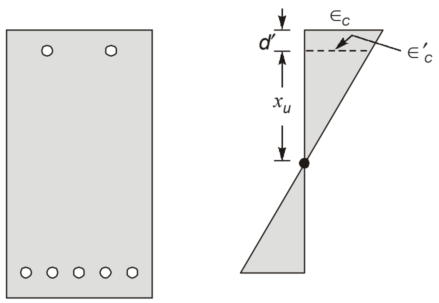
$L_0 = 16\ \phi = 16 \times 20 = 320 \text{ mm}$

According to IS 456, $L_d \leq \dfrac{1.3 M_1}{V} + L_0$

$\leq \dfrac{1.3 \times 93.48 \times 10^6}{165 \times 1000} + 320$

$L_d \leq 1056.51 \text{ mm}$

20. (b)



Maximum strain in concrete= 0.0035

$$\frac{\in'_c}{x_u - d'} = \frac{\in_c}{x_u}$$

$$\in'_c = \in_c\left(\frac{x_u - d'}{x_u}\right) = 0.0035\left(\frac{x_u - d'}{x_u}\right)$$

$$= 0.0035\left(\frac{150 - 40}{150}\right) = 2.56 \times 10^{-3}$$

21. (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

22. (b)

$\because$ D > 400 mm and column is helically reinforced, so

As per IS 456 : 2000,

$\therefore$ $P_u = [0.4\, f_{ck}\, A_c + 0.67\, f_y\, A_{sc}] \times 1.05$

where A_c = Area of concrete

$= A_g - A_{sc}$

A_{sc} = Area of steel in compression

A_g = gross cross-section area of compression member

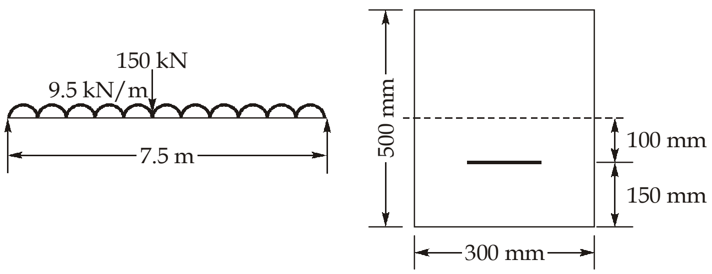
5% increment in load carrying capacity as it is helically reinforced.

$$\therefore \quad P_u = 1.05\left[0.4 \times 25 \times \left(\frac{\pi}{4} \times 500^2 - 8 \times \frac{\pi}{4} \times 20^2\right) + 0.67 \times 500 \times \frac{\pi}{4} \times 8 \times 20^2\right] \times 10^{-3}$$

$$\Rightarrow \quad P_u = 1.05\,[1938362.667 + 841946.83] \times 10^{-3}$$

$$\Rightarrow \quad P_u = 2919.32 \text{ kN} \simeq 2920 \text{ kN (say)}$$

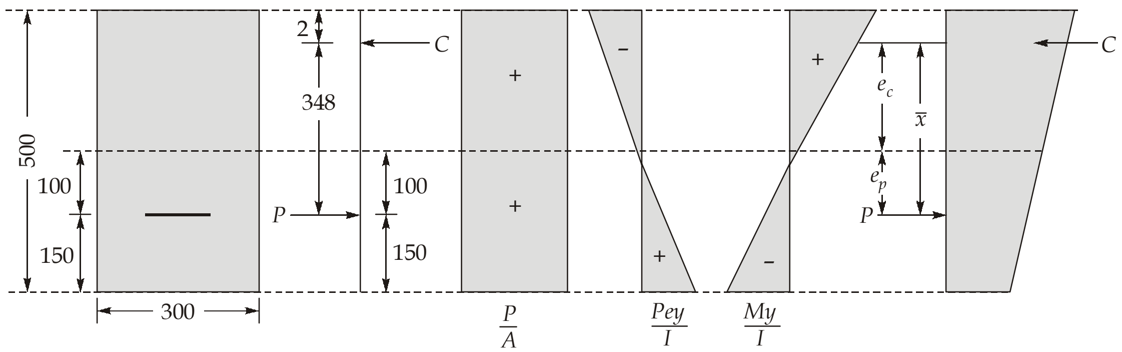
23. (c)



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

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

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



(All dimensions are in mm)

24. (d)

For axially loaded column,

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

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

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

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

where A_c = Area of concrete

A_g = Gross area of column

A_{sc} = Area of compression steel

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

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

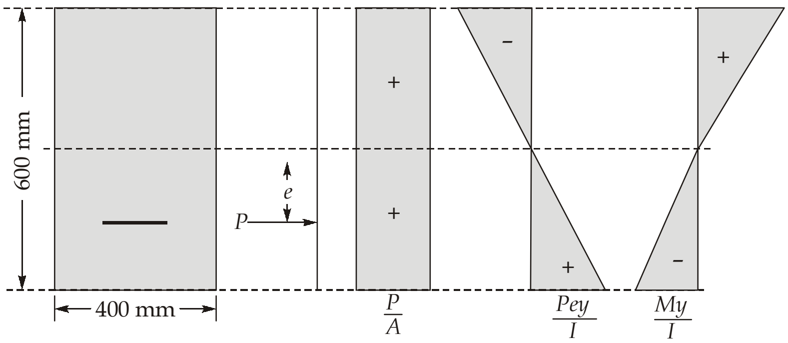
$$A_{sc} = 1131.498 \text{ mm}^2 \simeq 1131.50 \text{ mm}^2$$

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

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

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

25. **(c)**



$$\sigma_t = \frac{P}{A} - \frac{Pey}{I} + \frac{My}{I} = 10 \text{ N/mm}^2 \quad \text{... (i)}$$

$$\sigma_b = \frac{P}{A} + \frac{Pey}{I} - \frac{My}{I} = 13 \text{ N/mm}^2 \quad \text{... (ii)}$$

From equation (i) and (ii),

$$\sigma_t + \sigma_b = \frac{2P}{A}$$

$$\Rightarrow \quad 10 + 13 = \frac{2 \times P}{400 \times 600}$$

$$\Rightarrow \quad P = 2760 \text{ kN}$$

Substituting 'P' in equation (i)

$$\frac{2760 \times 10^3}{400 \times 600} - \frac{2760 \times 10^3 \times e \times 300}{400 \times \frac{600^3}{12}} + \frac{230 \times 10^6 \times 300}{400 \times \frac{600^3}{12}} = 10$$

$$\Rightarrow \quad e = 96.38 \text{ mm} \simeq 96.4 \text{ mm}$$

26. **(a)**

$$M_x = \frac{Cw_x l_x^2}{8} \text{ (As per Marcus method)}$$

where $C = 1-\frac{5}{6}\left[\frac{r^2}{1+r^4}\right] \left(r=\frac{l_y}{l_x}=\frac{7}{4}=1.75\right)$

$\Rightarrow$ $C = 1-\frac{5}{6}\left[\frac{1.75^2}{1+1.75^4}\right]$

$\Rightarrow$ $C = 0.7541$

$$w_x = w\left(\frac{r^4}{1+r^4}\right)$$

$$= 25\left(\frac{1.75^4}{1+1.75^4}\right) = 22.60 \text{ kN/m}^2$$

$\therefore$ $M_x = \frac{0.7541\times 22.60\times 4^2}{8} = 34.09 \text{ kNm} \simeq 34 \text{ kNm}$

27. (d)

$V = 180$ kN

$\therefore$ $\tau_v = \frac{V}{Bd} = \frac{180\times 1000}{350\times 480} = 1.07 \text{ N/mm}^2$

$\tau_c = 0.65 \text{ N/mm}^2$ (given)

$\therefore$ Net shear force for which stirrups to be designed,

$V_s = (\tau_v - \tau_c)\, Bd$

$= (1.07 - 0.65) \times 350 \times 480$

$= 70560$ N

$\therefore$ $S_v = \frac{A_{sv}\left(0.87 f_y\right)d}{V_s}$

$\Rightarrow$ $S_v = \frac{2\times\frac{\pi}{4}\times 10^2\times 0.87\times 415\times 480}{70560} = 385.80 \text{ mm c/c}$

But, $S_v \leq 300$ mm

$\leq 0.75\, d = 0.75 \times 480 = 360$ mm

$\therefore$ $S_v = 300$ mm c/c

28. (b)

$$V_h = \pi D_H \times \frac{\pi}{4}\phi_h^2 \times \frac{1000}{P}$$

where ϕ_h = Diameter of helical reinforcement bar = 10 mm

D_H = Diameter of helix

$= D_c - \phi_h = 400 - 10 = 390$ mm

Core diameter, $D_c = D_g - (2 \times \text{Cover})$

$= 500 - 2 \times 50 = 400$ mm

P = Pitch = 45 mm

$$V_h = \pi(390)\times\frac{\pi}{4}\times 10^2\times\frac{1000}{45}$$

$\Rightarrow$ $V_h = 2138414.287 \text{ m}^3$

$$V_c = \frac{\pi}{4} D_c^2 \times 1000$$
$$= \frac{\pi}{4} \times 400^2 \times 1000$$
$$= 125663706.1 \text{ m}^3$$
$$\therefore \quad \frac{V_h}{V_c} = \frac{1}{58.76}$$

29. (a)

$$P_L = P_0 [kx + \mu\alpha]$$
$$\alpha = \frac{4h}{l}$$
$$\tan\alpha = \frac{4 \times 250}{12000} = 0.0833 \; (\therefore \alpha = 4.76° = 0.0831 \text{ radian})$$

Given,
$$k = 0.15 \text{ per } 1000 \text{ m}$$
$$= \frac{0.15}{1000} \text{ per m}$$
$$x = 6.0 \text{ m } \{\text{as loss is asked at mid-span of beam}\}$$
$$\therefore \quad P_L = 1200\left[\frac{0.15}{1000} \times 6.0 + 0.35 \times 0.0831\right]$$
$$\Rightarrow \quad P_L = 35.982 \text{ kN} \simeq 36 \text{ kN}$$

30. (b)

One-way shear stress, $\tau_v = \frac{V_u}{Bd}$

V_u = Factored vertical shear force at critical section

Net factored pressure on the footing $= \frac{400 \times 1000}{2000 \times 2000} = 0.1 \text{ N/mm}^2$

As the critical section is at a distance d (250 mm) from the face of the column,

$$V_u = 0.1 \times \left(\frac{2000 - 300}{2} - 250\right) \times 2000$$
$$= 0.1 \times 600 \times 2000 = 120 \times 10^3 \text{ N}$$

Now,
$$\tau_v = \frac{120 \times 10^3}{2000 \times 250} = 0.24 \text{ N/mm}^2$$

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