CLASS TEST

Sl. : 01DLSK_CE_DEFGH_180726

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REINFORCED CEMENT CONCRETE

CIVIL ENGINEERING

Date of Test : 18/07/2026

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (a)

Pitch of helical turns shall not be more than

(i) 75 mm

(ii) one-sixth of the core diameter of column $\frac{480}{6} = 80$ mm

$\left.\right\} = 75$ mm

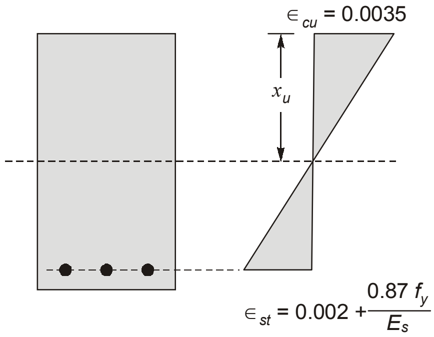
Pitch of helical turns shall not be less than

(i) 25 mm

(ii) 3 times the diameter of steel bar forming the helix

So, maximum pitch is 75 mm.

2. (d)



For Fe415,

$$\in_{st} = 0.002 + \frac{0.87 \times 415}{2 \times 10^5} = 3.805 \times 10^{-3}$$

3. (d)

Design load for collapse:

(i) 1.5 DL + 1.5 LL = 1.5 × 120 + 1.5 × 200 = 480 kN/m

(ii) 1.5 DL + 1.5 WL = 1.5 × 120 + 1.5 × 25 = 217.5 kN/m

(iii) 1.2 DL + 1.2 LL + 1.2 EL = 1.2 × 120 + 1.2 × 200 + 1.2 × 25 = 414 kN/m

So, maximum of these three values will be the design load for collapse condition = 480 kN/m

4. (d)

$$p_{tlim} = 41.61\left(\frac{f_{ck}}{f_y}\right)\left(\frac{x_{ulim}}{d}\right)$$

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

5. (c)

The concept of load balancing is applied for determinate structures. Concordant cable profile is used for indeterminate structures.

6. (b)

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

7. (a)

Maximum pitch or maximum spacing of lateral ties shall be the minimum of:

(i) Least lateral dimension = 400 mm

(ii) 16 times smallest dia. bar = 16 × 12 = 192 mm

(iii) 300 mm

Hence, maximum spacing will be 192 mm.

8. (a)

$$\text{Effective width of flange} = \frac{0.5 l_0}{\frac{l_0}{b} + 4} + b_w$$

$$l_0 = 0.7 \times 15 \text{ (for fixed ends)} = 10.5 \text{ m}$$

$$\text{Effective width of flange} = \frac{0.5 \times 10.5 \times 1000}{\frac{10500}{650} + 4} + 150 = 410.5 \text{ mm}$$

9. (c)

Bars in central, band,

$$\eta_c = \text{Total Bars}\left(\frac{2}{1+\frac{L}{B}}\right) = 20 \times \left(\frac{2}{1+\frac{3}{2}}\right) = 16$$

10. (d)

For under Reinforced, steel yields first.

For ultimate failure, we check for maximum strain level upto fracture that can be sustained by concrete and steel.

Maximum strained upto fracture for concrete is 0.003 – 0.0045

Maximum strained upto fracture for steel is 0.12 – 0.20

For all type of section, strain variation is linear.

So, with increase in load, the extreme fibre of compressive concrete will reach its maximum strain level before steel will.

11. (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.

12. (b)

Stress in concrete at the level of tendon,

$$f_c = \frac{P}{A} + \frac{Pe^2}{I}$$

$$= \frac{150 \times 10^3}{120 \times 200} + \frac{150 \times 20^2 \times 12 \times 10^3}{120 \times 200^3} = 7 \text{ MPa}$$

Loss of prestress due to elastic deformation,

$$\Delta f_s = \frac{E_s}{E_c} \times f_c = \frac{2.1 \times 10^5}{3.0 \times 10^4} \times 7 = 49 \text{ MPa}$$

Total stress in steel, $$f_s = \frac{P}{A_s} = \frac{150 \times 10^3}{187.5} = 800 \text{ MPa}$$

$\therefore$ Percentage loss of stress $= \dfrac{\Delta f_s}{f_s} \times 100 = \dfrac{49}{800} \times 100 = 6.125\% \simeq 6.13\%$

13. (a)

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

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

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

$$95 = \frac{P}{A} + \frac{M}{Z} \quad \text{...}(i)$$

$$55 = \frac{P}{A} - \frac{M}{Z} \quad \text{...}(ii)$$

From (i) and (ii), we get

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

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

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

14. (a)

$$e_{min} = \text{Max. of } \frac{l}{500} + \frac{D}{30} \text{ or } 20 \text{ mm}$$

$$= \frac{3000}{500} + \frac{450}{30} = 21 \text{ mm}$$

And, $0.05D = 0.05 \times 450 = 22.5$ mm

As $e_{min} < 0.05D$, It is axially loaded column

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

$$= 0.4 \times 20 \times \left(\frac{\pi}{4} \times 450^2 - A_{sc}\right) + 0.67 \times 415 \times A_{sc}$$

Putting, $$A_{sc} = 6 \times \frac{\pi}{4} \times 20^2 \simeq 1885 \text{ mm}^2$$

$$P_u = 1781.39 \text{ kN} \quad \text{(Factored load)}$$

Maximum service axial load $= \dfrac{1781.39}{1.5} = 1187.59$ kN

15. (b)

$$C_{uc} + C_{us} = T_u$$

$$\Rightarrow \quad 0.36\, f_{ck}\, bx_u + (f_{sc} - 0.45\, f_{ck})A_{sc} = 0.87\, f_y\, A_{st}$$

$$\Rightarrow \quad x_u = \frac{0.87 \times 250 \times A_{st} - (f_{sc} - 0.45 \times 20)A_{sc}}{0.36 \times 20 \times 280}$$

$$A_{st} = 5 \times \frac{\pi}{4} \times 24^2 = 2262\,\text{mm}^2$$

$$A_{sc} = 3 \times \frac{\pi}{4} \times 16^2 = 603.2\,\text{mm}^2$$

Assuming, $f_{sc} = 0.87\, f_y = 0.87 \times 250 = 217.5\ \text{N/mm}^2$

$\Rightarrow \quad x_u = 181.65$ mm

Also, $x_{u,\,max} = 0.53d = 0.53 \times 450 = 238.5$ mm $\quad (\because$ Fe250)

16. (d)

17. (d)

$$\text{Factored shear force} = 1.5 \times 110 = 165\ \text{kN}$$

$$\text{Effective depth} = 500 - 35 = 465\ \text{mm}$$

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

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

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

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

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

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

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

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

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

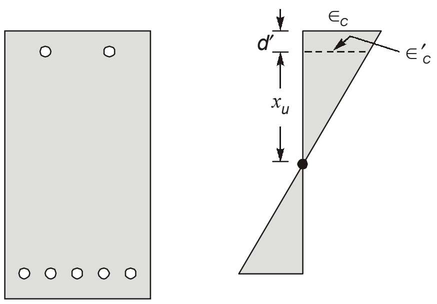
$$L_0 = 16\,\phi = 16 \times 20 = 320\ \text{mm}$$

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

$$\le \frac{1.3 \times 93.48 \times 10^6}{165 \times 1000} + 320$$

$$L_d \le 1056.51\ \text{mm}$$

18. (b)



Maximum strain in concrete = 0.0035

$$\frac{\epsilon'_c}{x_u - d'} = \frac{\epsilon_c}{x_u}$$

$$\epsilon_c' = \epsilon_c\left(\frac{x_u - d'}{x_u}\right) = 0.0035\left(\frac{x_u - d'}{x_u}\right)$$

$$= 0.0035\left(\frac{150-40}{150}\right) = 2.56\times10^{-3}$$

19. (d)

Assuming the neutral axis to lie within the flange and equating the compression and tension.

$$0.36\, f_{ck}\, B\, x_u = 0.87\, f_y\, A_{st}$$

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} B} = \frac{0.87\times250\times1520.53}{0.36\times20\times1000} = 45.93 \text{ mm}$$

$$\left(A_{st} = 4\times\frac{\pi}{4}\times22^2 = 1520.53 \text{ mm}^2\right)$$

But, $D_f = 100$ mm, $x_u < D_f$

Hence our assumption about the position of the neutral axis is correct.

Also $x_{u,max} = 0.53\, d = 0.53 \times 500 = 265$ mm

$\therefore$ $x_u < x_{u,max}$

$\therefore$ The section is under reinforced.

Ultimate moment of resistance,

$$M_u = 0.36 f_{ck} B x_u (d - 0.42 x_u)$$

$$= 0.36\times20\times1000\times45.93\,(500 - 0.42\times45.93) \text{ Nmm}$$

$$= 158.96\times10^6 \text{ Nmm} = 158.96 \text{ kNm}$$

20. (a)

$$\tau_{us} = \tau_v - \tau_c$$

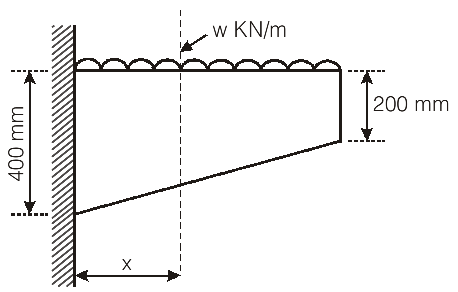
$$\tau_v = 0.6 \text{ N/mm}^2 + 0.74 \text{ N/mm}^2 = 1.34 \text{ N/mm}^2$$

at x = 1m from support

$$\text{depth} = 400 - \left(\frac{400-200}{3}\times1\right) = 333.33 \text{ mm}$$

$$\text{effective depth} = 333.33 - 40 = 293.33 \text{ mm}$$

$$\tau_v = \frac{V_u \pm (M_u/d)\tan\beta}{bd} \quad \text{...(i)}$$



$$\tan\beta = \frac{400-200}{3000} = 0.067$$

$$V_u = 1.5\times[w(L - x)] \qquad \text{[put x = 1 m]}$$

$$= 3w \text{ kN}$$

$$M_u = 1.5\frac{w(l-x)^2}{2} = \frac{1.5w}{2} \times 4 = 3w \text{ kN-m}$$

putting value of M_u, V_u and τ_v in equation (i)

$$1.34 = \frac{3000w - \frac{(3\times10^6)w}{293.33}\times 0.067}{250\times 293.33}$$

[Since BM and depth rae increasing in same direction negative sign will be used]

$$w = 42.45 \text{ N/mm}$$

21. (d)

$$V_{ueq} = 1.5 \times 150 + \frac{1.6\times(1.5\times50)}{0.3} = 625 \text{ kN}$$

$$\Rightarrow \quad \tau_{Veq} = \frac{625\times10^3}{300\times700} = 2.98 \text{ N/ mm}^2$$

$$\tau_{Veq} > \tau_{cmax} \Rightarrow \text{section must be redesigned}$$

22. (a)

Equivalent bending moment (M_{eq}) = M + M_T

$$M_T = \frac{T\left(1+\frac{D}{b}\right)}{1.7} = \frac{20\times10^6\left(1+\frac{400}{400}\right)}{1.7}$$

$$= 23.53 \times 10^6 \text{ N-mm}$$

$$M_{eq} = 60 \times 10^6 + 23.53 \times 10^6$$

$$= 83.53 \times 10^6 \text{ N-mm}$$

Equivalent shear

$$V_{eq} = V + \frac{1.6T}{B}$$

$$= 30000 + \frac{1.6\times20\times10^6}{400} = 110000 \text{ N}$$

$$\frac{M_{eq}}{V_{eq}} = \frac{83.53\times10^6}{0.11\times10^6} = 759.36$$

23. (d)

Average initial stress in concrete

$$f_c = \left(\frac{300\times10^3}{250\times250}\right)$$

$$= 4.8 \text{ N/mm}^2$$

Modular ratio = m = $\frac{E_s}{E_c}$ = 6.56

Loss due to elastic deformation

$$= \frac{E_s}{E_c}\times f_c = 6.58 \times 4.8$$

$$= 31.58 \text{ N/mm}^2$$

Loss due to creep of concrete

$$\theta m f_c = 1.6 \times 6.56 \times 4.8 = 50.381 \text{ N/mm}^2$$

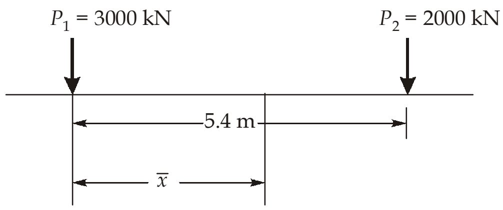
Loss due to shrinkage of concrete

$$= (200 \times 10^{-6}) \times E_s$$
$$= 42 \text{ N/mm}^2$$

Therefore, all options (i), (ii) and (iii) are correct.

24. (b)

C.G. of column loads from P_1 is given by,

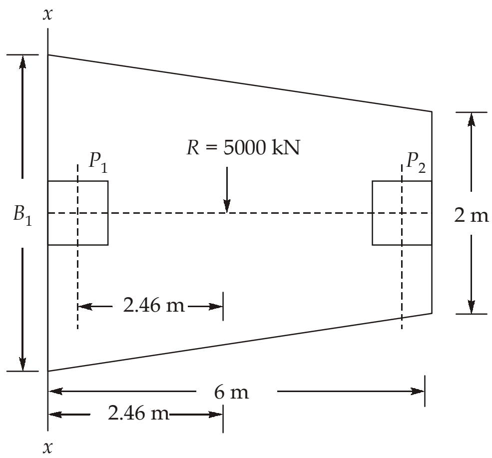


$$\bar{x} = \frac{P_1 \times 0 + P_2 \cdot (6 - 0.3 - 0.3)}{P_1 + P_2} = \frac{3000 \times 0 + 2000 \times (5.4)}{3000 + 2000} = 2.16 \text{m}$$

Distance of C.G. of footing from left edge i.e. x-x is

$$\bar{y} = \bar{x} + \left(\frac{0.6}{2}\right)$$

$$\Rightarrow \quad \bar{y} = 2.16 + 0.3 = 2.46 \text{ m} \quad \text{...(1)}$$



Now,
$$\bar{y} = \left[\frac{B_1 + 2 \times (2)}{B_1 + 2}\right] \times \left(\frac{6}{3}\right)$$

$$\Rightarrow \quad \bar{y} = \left[\frac{B_1 + 4}{B_1 + 2}\right](2) \quad \text{...(2)}$$

From (1) and (2),

$$2.46 = \left(\frac{B_1 + 4}{B_1 + 2}\right)(2)$$

$$1.23(B_1 + 2) = B_1 + 4$$
$$\Rightarrow \quad 1.23B_1 + 2.46 = B_1 + 4$$
$$\Rightarrow \quad 0.23B_1 = 1.54$$
$$\Rightarrow \quad B_1 = 6.69 \text{ m} \simeq 6.7 \text{ m}$$

25. (d)
Minimum eccentricity,

$$e_{min} = \text{maximum}\begin{cases}\dfrac{L}{500}+\dfrac{B \text{ or } D}{30} \\ 20 \text{ mm}\end{cases} = \text{maximum}\begin{cases}\dfrac{4000}{500}+\dfrac{500}{30} = 24.67\text{mm} \\ 20 \text{ mm}\end{cases}$$

$\therefore \quad e_{min} = 24.67$

For axially loaded short column, e_{min} should be less than 0.05B (or D) i.e. 0.05 × 500 = 25 mm

Now, ultimate load, $P_u = 1.5(P) = 1.5 \times 1600 = 2400$ kN

But, $P_u = 0.4 f_{ck} \cdot A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}$

$\Rightarrow \quad 2400 \times 10^3 = 0.4 \times 20 \times 500 \times 500 + [0.67 \times 415 - 0.4 \times 20] A_{sc}$

$\Rightarrow \quad A_{sc} = 1481.21 \text{ mm}^2$

$A_{sc} \simeq 1482 \text{ mm}^2$

But as per IS 456 – 2000, $(A_{sc})_{min}$ $= 0.8\%$ of A_g

$$\Rightarrow \quad (A_{sc})_{min} = \frac{0.8}{100} \times 500 \times 500$$

$(A_{sc})_{min} = 2000 \text{ mm}^2$

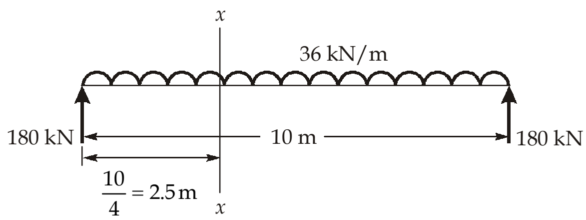
Hence, provide $A_{sc} = 2000 \text{ mm}^2$

26. (b)

Dead load $= B \times D \times 1 \times \gamma_c = 0.4 \times 0.6 \times 1 \times 25 = 6$ kN/m

Live load $= 30$ kN/m

Total load, $w = 30 + 6 = 36$ kN/m



$$BM_x = 180 \times 2.5 - 36 \times 2.5 \times \frac{2.5}{2}$$

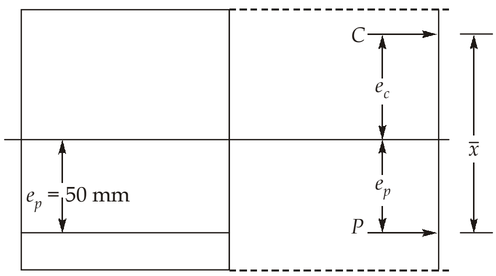
$= 337.5$ kNm

Prestressing force in wire, $P = 3000$ kN

$$\text{Now, shift of C line from } P_{line}, \ \bar{x} = \frac{BM_x}{P}$$

$$\Rightarrow \quad \bar{x} = \frac{337.5 \times 10^3 \text{ kN-mm}}{3000 \text{kN}}$$

$\Rightarrow \quad \bar{x} = 112.5$ mm



$$\bar{x} = e_c + e_p$$

$$\Rightarrow \quad e_c = \bar{x} - e_p = 112.5 - 50$$

$$\Rightarrow \quad e_c = 62.5 \text{ mm}$$

Location of thrust line from top $= \dfrac{D}{2} - e_c = 300 - 62.5 = 237.5$ mm

27. (d)

$$\frac{P_u}{f_{ck}.BD} = \frac{500 \times 10^3}{20 \times 500 \times 500} = 0.1$$

Therefore from the interaction diagram,

$$\frac{M_u}{f_{ck} \cdot BD^2} = 0.3$$

$$\Rightarrow \quad M_u = \frac{0.3 \times 20 \times 500 \times 500^2}{10^6} \text{kN-m}$$

$$\Rightarrow \quad M_u = 750 \text{ kN-m}$$

Maximum uniaxial eccentricity (e)

$$e = \frac{M_u}{P_u} = \frac{750 \text{kN-m}}{500 \text{kN}} = 1.5 \text{ m} = 1500 \text{ mm}$$

28. (a)

Factored shear force, $V_u = 1.5 \times 200 = 300$ kN

Nominal shear stress, $\tau_v = \dfrac{V_u}{bd} = \dfrac{300 \times 10^3}{300 \times 500} = 2 \text{ N/mm}^2 < 2.8 \text{ N/mm}^2$

$\therefore \quad \tau_v < \tau_{c,\,max}$ (OK)

Shear force taken by stirrups,

$$V_{us} = (\tau_v - \tau_c)bd$$

$$\Rightarrow \quad V_{us} = (2 - 0.62) \times 300 \times 500 \times 10^3$$

$$V_{us} = 207 \text{ kN}$$

Now, $\quad V_{us} = \dfrac{0.87 f_y \cdot A_{sv} \cdot d}{S_v}$

Since design shear stress in stirrups is limited to f_y to 415 N/mm².

$$\therefore \quad 207 \times 10^3 = \frac{0.87 \times 415 \times \left[2 \times \frac{\pi}{4} \times 8^2\right] \times 500}{S_v}$$

$$\Rightarrow \quad S_v = 87.67 \text{ mm} = 8.767 \text{ cm} \simeq 8.8 \text{ cm}$$

Hence provide spacing 8.8 cm

29. (a)

30. (a)

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