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REINFORCED CEMENT CONCRETE

CIVIL ENGINEERING

Date of Test : 19/10/2026

ANSWER KEY >

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

DETAILED EXPLANATIONS

1. (b)

Modular ratio, $m = \frac{280}{3\sigma_{cbc}}$

$$\sigma_{cbc} = \frac{f_{ck}}{3} = \frac{25}{3} \text{ MPa} \quad (\text{Approximately})$$

$$\therefore m = \frac{280}{3 \times \frac{25}{3}}$$

$$\Rightarrow m = 11.2$$

2. (a)

For slab

$$(A_{st})_{\min} = 0.0012 \times A_g \quad (\text{if Fe415 is used})$$

$$= 0.0012 \times 150000 = 180 \text{ mm}^2$$

3. (a)

Rotation per unit length, $\phi = \frac{\epsilon_c + \epsilon_{st}}{d}$

$$\Rightarrow \phi = \frac{0.0025 + 0.003}{550}$$

$$\Rightarrow \phi = 1 \times 10^{-5} / \text{mm}$$

4. (a)

$$E_{\text{long}} = \frac{E}{1 + \theta}$$

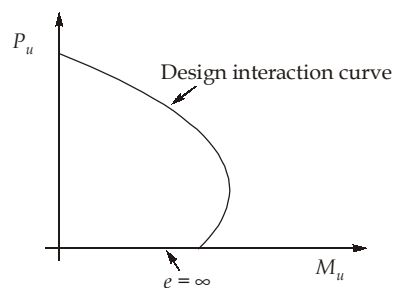
For 28 days, $\theta = 1.6$

$$\therefore E_{\text{long}} = \frac{5000\sqrt{f_{ck}}}{1 + \theta}$$

$$\Rightarrow E_{\text{long}} = \frac{5000 \times \sqrt{30}}{1 + 1.6}$$

$$= 10533.126 \text{ MPa} = 10.5 \text{ GPa}$$

5. (c)



$$P_u \times e = M_u$$

$$\Rightarrow e = \frac{M_u}{P_u}$$

If $e \rightarrow \infty \Rightarrow P_u \rightarrow \text{zero}$

6. (c)

Effective length of column, $l_e = 1.0 l = 1.0 \times 4.8 \text{ m} = 4800 \text{ mm}$

Reduction coefficient, $C_r = 1.25 - \frac{l_{eff}}{48b}$

$$\Rightarrow C_r = 1.25 - \frac{4800}{48 \times 240} = 0.83$$

7. (a)

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

$$\Rightarrow L_d = \frac{0.87 \times 415 \times \phi}{4 \times 1.6 \times 1.2}$$

$$\Rightarrow L_d = 47 \phi \text{ mm}$$

8. (a)

Aspect ratio $= \frac{l_y}{l_x} = \frac{4.4}{4.0} = 1.1$

$$\therefore \alpha_x = 0.074 \quad \alpha_y = 0.061$$

$$\therefore M_x = \alpha_x w_x l_x^2 = 0.074 \times 8 \times 4^2 = 9.472 \text{ kNm} \simeq 9.47 \text{ kNm}$$

$$M_y = \alpha_y w l_x^2 = 0.061 \times 8 \times 4^2 = 7.808 \text{ kNm} \simeq 7.81 \text{ kNm}$$

9. (a)

Area of steel $= 6 \times \frac{\pi}{4} \times 25^2 = 2945.24 \text{ mm}^2$

Ultimate load for the column.

$$P_u = 1.05(0.4 f_{ck} A_c + 0.67 f_y A_s)$$

$$\Rightarrow P_u = 1.05 \left[0.4 \times 20 \times \left(\frac{\pi}{4} \times 400^2 - 2945.24 \right) + 0.67 \times 415 \times 2945.24 \right] \text{ N}$$

$$= 1890.705 \text{ kN}$$

$$\therefore \text{Safe load for the column} = \frac{1890.705}{1.5} = 1260.47 \text{ kN} \simeq 1260.5 \text{ kN}$$

10. (d)

For the given monolithic T-beam,

Effective width of flange, $B_f = \frac{L_0}{\frac{L_0}{B} + 4} + b_w$

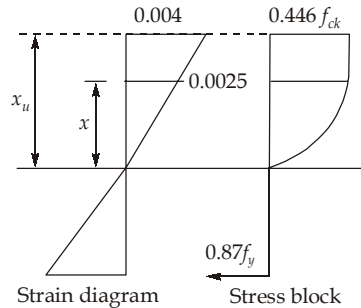
$$L_0 = \text{clear span of beam}$$

For a continuous beam, $L_0 = 0.7 l_{eff} = 0.7 \times 4500 = 3150 \text{ mm}$

$$\therefore B_f = \frac{L_0}{\frac{L_0}{B} + 4} + b_w = \frac{3150}{\frac{3150}{1800} + 4} + 180$$

$$= 727.8 \text{ m} \simeq 728 \text{ mm}$$

11. (d)



From strain diagram,

$$\frac{0.004}{x_u} = \frac{0.0025}{x}$$

$$\Rightarrow x = \frac{0.0025}{0.004} x_u$$

$$\Rightarrow x = \frac{5}{8} x_u$$

 $\therefore$ Total compressive force,

$$C = C_{\text{linear}} + C_{\text{parabolic}}$$

$$\Rightarrow C = 0.446 f_{ck} \times (x_u - x) \times b + \frac{2}{3} \times 0.446 f_{ck} \times x \times b$$

$$\Rightarrow C = 0.446 f_{ck} \left(x_u - \frac{5}{8} x_u \right) \times b + 0.185 f_{ck} b \left(\frac{5}{8} x_u \right)$$

$$\Rightarrow C = 0.167 f_{ck} b x_u + 0.185 f_{ck} b x_u$$

$$\Rightarrow C = 0.352 f_{ck} b x_u = 0.352 \times 25 \times b x_u$$

$$\Rightarrow C = 8.8 b x_u$$

12. (b)

$$b = 250 \text{ mm}, d = 450 \text{ mm}, f_{ck} = 20 \text{ MPa}, f_y = 250 \text{ MPa}$$

$$A_{st} = 4 \times \frac{\pi}{4} \times 18^2 = 1017.876 \text{ mm}^2$$

Depth of neutral axis:

$$0.36 \times 20 \times 250 \times x_u = 0.87 \times 250 \times 1017.876$$

$$\Rightarrow x_u = 122.993 \text{ mm} \simeq 123 \text{ mm}$$

For Fe250, $x_{u, \text{max}} = 0.53 \times 450 = 238.5 \text{ mm}$

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

 $\Rightarrow$ Under-reinforced beam section

$$\therefore \text{MOR} = 0.87 \times 250 \times 1017.876 (450 - 0.42 \times 123) \text{ Nmm}$$

$$= 88.19 \text{ kNm}$$

13. (a)

$$\begin{aligned} b &= 350 \text{ mm} \\ D &= 750 \text{ mm} \\ T_u &= 140 \text{ kNm} \\ M_u &= 200 \text{ kNm} \\ V_u &= 110 \text{ kN} \end{aligned}$$

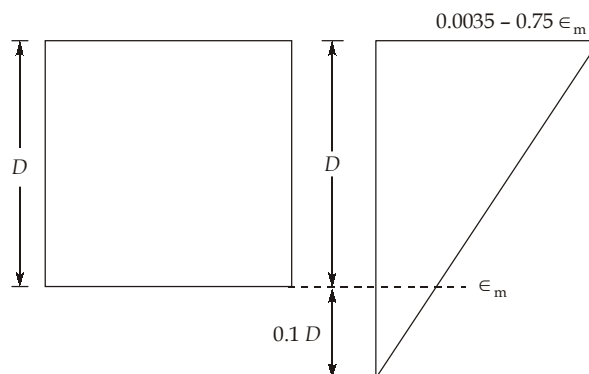
Effective bending moment due to torsion,

$$\begin{aligned} M_t &= \frac{T_u \left(1 + \frac{D}{b} \right)}{1.7} \\ &= \frac{140 \left(1 + \frac{750}{350} \right)}{1.7} = 259 \text{ kNm (tension)} \end{aligned}$$

Equivalent bending moment for design,

$$\begin{aligned} M_e &= M_t + M_u \text{ (Flexure tension at top)} \\ &= 259 + 200 \\ &= 459 \text{ kNm} \end{aligned}$$

14. (c)



$$\begin{aligned} \frac{0.0035 - 0.75 \epsilon_m}{1.1D} &= \frac{\epsilon_m}{0.1D} \\ \Rightarrow 11.75 \epsilon_m &= 0.0035 \end{aligned}$$

$$\Rightarrow \epsilon_m = \frac{0.0035}{11.75}$$

∴ Strain in extreme compression fibre,

$$\begin{aligned} \epsilon &= 0.0035 - 0.75 \epsilon_m \\ \Rightarrow \epsilon &= 0.0035 - 0.75 \times \frac{0.0035}{11.75} \\ \Rightarrow \epsilon &= 0.003276596 \\ \Rightarrow \epsilon &= 3276.596 \mu \end{aligned}$$

15. (d)

$$\begin{aligned} f_{ck} &= 25 \text{ MPa} \\ f_y &= 415 \text{ MPa} \\ D &= 400 \text{ mm}, C = 40 \text{ mm} \end{aligned}$$

Diameter of core,

$$D_c = D - 2 \times C$$

$$= 400 - 2 \times 40 = 320 \text{ mm}$$

$$\phi_{sp} = 6 \text{ mm}$$

As per IS 456 : 2000,

$$\frac{\text{Volume of spiral reinforcement}}{\text{Volume of core}} \geq \left(\frac{A_g}{A_c} - 1 \right) \frac{f_{ck}}{f_y}$$

Volume of spiral reinforcement, $V_s = \frac{\pi(D_c - \phi_{sp}) \frac{\pi}{4} \phi_{sp}^2}{\text{pitch length}} = \frac{\pi(320 - 6) \frac{\pi}{4} \times 6^2}{p_t}$

$$\therefore \frac{\frac{\pi(320 - 6) \frac{\pi}{4} \times 6^2}{p_t}}{\frac{\pi}{4} \times 320^2} \geq 0.36 \left[\frac{\frac{\pi}{4} \times 400^2}{\frac{\pi}{4} \times 320^2} - 1 \right] \frac{25}{415}$$

$\Rightarrow$

$$p_t \leq 28.429 \text{ mm}$$

$$p_t \leq \begin{cases} 75 \text{ mm} \\ \frac{D_c}{6} = \frac{320}{6} = 53.33 \text{ mm} \end{cases}$$

$$p_t \geq \begin{cases} 25 \text{ mm} \\ 3\phi_{sp} = 3 \times 6 = 18 \text{ mm} \end{cases}$$

16. (b)

$$p_o = \frac{600 \times 10^3}{2000 \times 2000} = 0.15 \text{ MPa}$$

$\therefore$ One-way shear stress, $\tau_v = \frac{p_o B \left(\frac{B}{2} - \frac{b}{2} - d \right)}{Bd}$

$$= \frac{0.15 \times 2000 \left(\frac{2000}{2} - \frac{400}{2} - 300 \right)}{2000 \times 300} = 0.25 \text{ MPa}$$

17. (b)

Assume,

$$x_u < d_f$$

$$\therefore x_u = \frac{0.877 f_y A_{st}}{0.36 f_{ck} \times b_f}$$

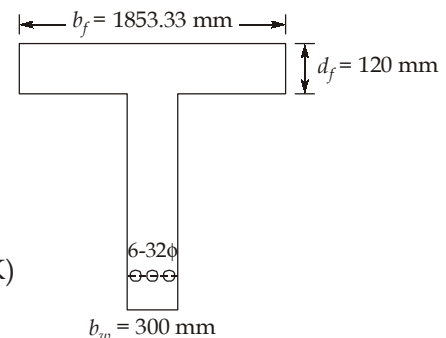
$$A_{st} = 6 \times \frac{\pi}{4} \times 32^2 = 4825.486 \text{ mm}^2$$

$$b_f = \frac{l_0}{6} + b_w + 6d_f = \frac{5000}{6} + 300 + 6 \times 120$$

$$= 1853.33 \text{ mm} < 3500 \text{ mm} \quad (\text{OK})$$

$$\therefore x_u = \frac{0.87 \times 415 \times 4825.486}{0.36 \times 20 \times 1853.33} = 130.56 \text{ mm}$$

$$x_u > d_f (= 120 \text{ mm}) \text{ and hence our assumption is wrong.}$$



If $x_u > d_f$ then,

$$y_f = 0.15 x_u + 0.65 d_f = 0.15 x_u + 0.65 \times 120 \\ = 0.15 x_u + 78$$

$$\Rightarrow 0.36 f_{ck} b_w x_u + 0.45 f_{ck} (b_f - b_w) y_f = 0.87 f_y \times A_{st}$$

$$\Rightarrow 0.36 \times 20 \times x_u \times 300 + 0.45 \times 20 (1853.33 - 300) \times (0.15 x_u + 78) = 0.87 \times 415 \times 4825.486$$

$$\Rightarrow (2160 + 2096.99)x_u = 651804.06$$

$$\Rightarrow x_u = 153.114 \text{ mm} > d_f (= 120 \text{ mm})$$

Also, $\frac{7}{3} \times d_f = \frac{7}{3} \times 120 = 280 \text{ mm} > x_u (= 153.114 \text{ mm})$

$$\therefore \frac{7}{3} d_f > x_u > d_f$$

18. (a)

$$A_{sv} \geq \frac{T_u \cdot S_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u \cdot S_v}{2.5 d_1 (0.87 f_y)}$$

For pure shear case, $V_u = 0$

Given, $T_u = 1.5 \times 30 = 45 \text{ kN-m}$

$$S_v = 160 \text{ mm}$$

$$b_1 = 300 - 2 \times 50 = 200 \text{ mm}$$

$$d_1 = 600 - 2 \times 50 = 500 \text{ mm}$$

$$\therefore A_{sv} = \frac{45 \times 10^6 \times 160}{200 \times 500 (0.87 \times 415)} + 0$$

$$\Rightarrow A_{sv} = 199.42 \text{ mm}^2$$

19. (a)

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

$$\text{Slope} = \tan \theta = \frac{dy}{dx} = \frac{4e}{l^2}(l-2x)$$

$$\text{Slope at support } (x = 0 \text{ and } x = l) = \pm \frac{4e}{l}$$

$$\text{Total eccentricity} = 55 + 65 = 120 \text{ mm}$$

$$\therefore \text{Slope at support } (x = 0) = \frac{4 \times 0.120}{12} = 0.04$$

Net change in slope between supports

$$\alpha = 2 \times 0.04 = 0.08 \text{ radian}$$

Initial prestress, $P_0 = 500 \times 1200 \text{ N} = 600 \text{ kN}$

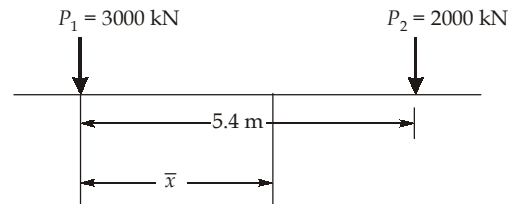
Let, $P_2 = \text{Prestress at other end,}$

$$P_L = (P_0 - P_2) = P_0(Kx + \mu\alpha) \\ = 600(0.0015 \times 12 + 0.55 \times 0.08) \\ = 37.2 \text{ kN}$$

$$\therefore P_2 = P_0 - 37.2 = 600 - 37.2 = 562.8 \text{ kN}$$

$$\therefore \text{Percent loss} = \frac{37.2}{600} \times 100 = 6.2\%$$

20. (b)

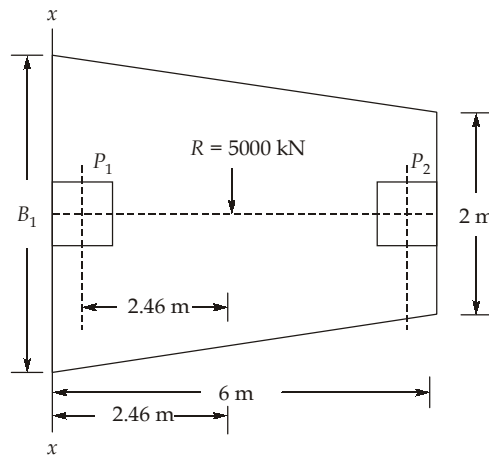
C.G. of column loads from P_1 is given by,

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

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

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

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



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

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

From (1) and (2),

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

$$1.23(B_1 + 2) = B_1 + 4$$

$$\Rightarrow 1.23B_1 + 2.46 = B_1 + 4$$

$$\Rightarrow 0.23B_1 = 1.54$$

$$\Rightarrow B_1 = 6.69 \text{ m} \simeq 6.7 \text{ m}$$

21. (b)

$$B = 400 \text{ mm}, D = 600 \text{ mm}, d = 600 - 50 = 550 \text{ mm}$$

Depth of neutral axis (x_u)

$$\Rightarrow x_u = \frac{0.87 f_y A_{st} - (f_{sc} - 0.45 f_{ck}) A_{sc}}{0.36 f_{ck} B}$$

$$\Rightarrow x_u = \frac{0.87 \times 415 \times 2000 - (350 - 0.45 \times 20) \times 800}{0.36 \times 20 \times 400}$$

$$x_u = 156 \text{ mm}$$

For Fe415, limiting depth of neutral axis is

$$x_{u, \text{lim}} = k_0 \cdot d = 0.48 \times 550 = 264 \text{ mm}$$

$$\therefore x_u < x_{u, \text{lim}}$$

The section is under-reinforced.

Now ultimate moment of resistance,

$$\begin{aligned} \Rightarrow M_{uR} &= 0.36 f_{ck} \cdot B \cdot x_u (d - 0.42 x_u) + (f_{sc} - 0.45 f_{ck}) A_{sc} (d - d_c) \\ M_{uR} &= 0.36 \times 20 \times 400 \times 156 (550 - 0.42 \times 156) + (350 - 0.45 \times 20) \times 800 \Rightarrow \\ &\quad \times (550 - 50) \text{ N.mm} \\ M_{uR} &= 354.067 \text{ kNm} \simeq 354 \text{ kNm} \end{aligned}$$

22. (d)

Minimum eccentricity,

$$\begin{aligned} e_{\min} &= \text{maximum} \left\{ \begin{array}{l} \frac{L}{500} + \frac{B \text{ or } D}{30} \\ 20 \text{ mm} \end{array} \right. \\ &= \text{maximum} \left\{ \begin{array}{l} \frac{4000}{500} + \frac{500}{30} \\ 20 \text{ mm} \end{array} \right. = 24.67 \text{ mm} \end{aligned}$$

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

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

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

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

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

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

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

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

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

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

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

23. (d)

$$\text{Long term modulus of elasticity, } E = \frac{5000 \sqrt{f_{ck}}}{1 + \theta}$$

where, θ is creep coefficient.

So, from logarithmic interpolation,

$$\frac{1.6 - 2.2}{\log_{10} 28 - \log_{10} 7} = \frac{\theta - 2.2}{\log_{10} 14 - \log_{10} 7}$$

$$\Rightarrow \theta = 1.9$$

Now,

$$\text{Long term modulus of elasticity} = \frac{5000\sqrt{25}}{1 + 1.9} = 8620.69 \text{ N/mm}^2$$

24. (a)

$$\text{Shear force at section X-X} = \frac{24 \times 15}{2} - 15 \times 6 = 90 \text{ kN}$$

Depth of section X-X

$$D = 400 + \frac{200}{12} \times 6 = 500 \text{ mm}$$

$$\therefore d = 500 - 50 = 450 \text{ mm}$$

Moment at section X-X

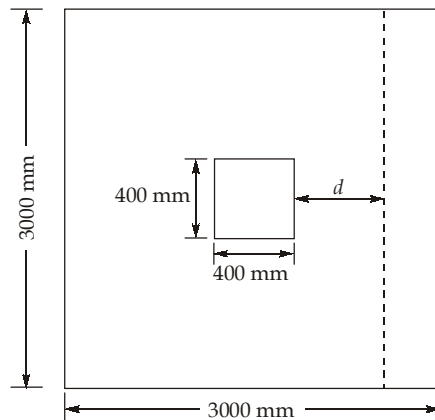
$$M_u = 180 \times 6 - \frac{15 \times 6 \times 6}{2} = 810 \text{ kNm}$$

$\therefore$ Design shear force at section X-X

$$\begin{aligned} V_{u, \text{design}} &= V_u + \frac{M_u}{d} \tan \beta \\ &= 90 + \frac{810}{0.45} \times \frac{200}{12000} = 120 \text{ kN} \end{aligned}$$

$$\therefore V_{u, \text{design}} = 120 \text{ kN}$$

25. (a)



$$\text{Uniform pressure, } p_o = \frac{P}{A} = \frac{400}{3 \times 3} = 44.44 \text{ kN/m}^2$$

The critical section for one way shear is at a distance of $d = 300 \text{ mm}$ from the face of the column.

$$\begin{aligned} V_u &= P_o B \left[\frac{1}{2} (B - b) - d \right] \\ &= 44.44 \times 3 \left[\frac{1}{2} (3 - 0.4) - 0.3 \right] \end{aligned}$$

$$= 133.32 \text{ kN}$$

$$\begin{aligned} \text{Shear stress, } \tau_v &= \frac{V_u}{B.d} = \frac{133.32 \times 10^3}{3 \times 10^3 \times 300} \\ \Rightarrow \tau_v &= 0.148 \text{ N/mm}^2 \end{aligned}$$

26. (c)

$$\therefore \frac{A_{st,min}}{b.d} = \frac{0.85}{f_y} \quad [\text{Cl. 26.5.1.1 (a)}]$$

$$\begin{aligned} \Rightarrow A_{st,min} &= \frac{0.85 \times 300 \times 450}{415} \text{ mm}^2 \\ &= 276.506 \text{ mm}^2 \simeq 277 \text{ mm}^2 \end{aligned}$$

$$\text{Maximum reinforcement } A_{st,max} = 0.04bD \quad [\text{Cl. 26.5.1.1 (b)}]$$

$$D = \text{Total depth} = d + \text{effective cover}$$

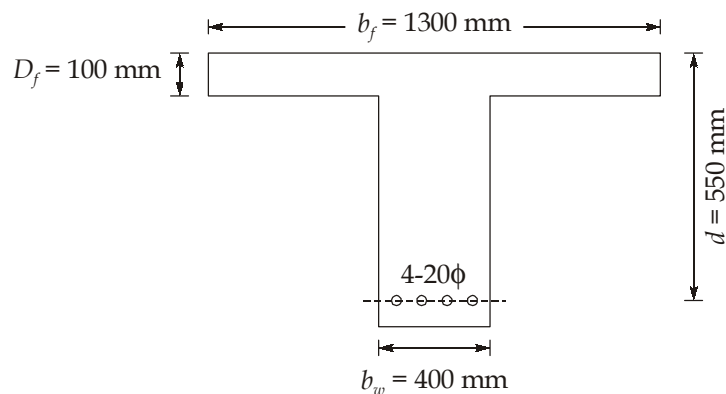
$$= 450 + \frac{16}{2} + 25 + 8 = 491 \text{ mm}$$

$$\therefore A_{st,max} = 0.04 \times 300 \times 491 \text{ mm}^2 = 5892 \text{ mm}^2$$

27. (d)

- High strength concrete is necessary in prestressed concrete, as the material offers high resistance in tension, shear, bond and bearing. High strength concrete is less liable to shrinkage cracks, and has a higher modulus of elasticity and smaller ultimate creep strain, resulting in a smaller loss of prestress in steel.
- For mass production (line Railway Sleepers, poles etc.) of pretensioned elements, the long-lone process developed by Hoyer is generally used in the factory.
- The concordant cable profile does not produce reactions at the supports or secondary moment in the spans.

28. (a)



Actual depth of Neutral axial (Assuming $x_u \leq D_f$)

$$\begin{aligned} x_u &= \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f} \\ &= \frac{0.87 \times 415 \times \left(4 \times \frac{\pi}{4} \times 20^2 \right)}{0.36 \times 30 \times 1300} \end{aligned}$$

$$= 32.32 \text{ mm} < D_f (= 100 \text{ mm}) \quad (\text{OK})$$

For Fe415, $x_{u \text{ lim}} = 0.48 \times d = 0.48 \times 550 = 264 \text{ mm} > x_u$

Thus section is under-reinforced and hence ultimate moment of resistance is given by

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

$$\Rightarrow M_u = 0.36 \times 30 \times 1300 \times 32.32 (550 - 0.42 \times 32.32) \times 10^{-6} \text{ kNm}$$

$$\Rightarrow M_u = 243.32 \text{ kNm} \simeq 243 \text{ kNm}$$

29. (d)

$$\text{Area of footing required} = \frac{1.1 \times 1800}{180} = 11 \text{ m}^2$$

$$\text{Assume } \frac{L}{B} = \frac{450}{300} = \frac{3}{2}$$

$$\therefore L \times B = 11$$

$$\Rightarrow \frac{3}{2} B^2 = 11$$

$$\therefore B = 2.71 \text{ m} \simeq 2.7 \text{ m}$$

$$\therefore L = \frac{3}{2} \times 2.7 = 4.05 \simeq 4.1 \text{ m}$$

$$q_u = \frac{P_u}{L \times B} = \frac{1.5 \times 1800}{2.7 \times 4.1} = 243.9 \text{ kN/m}^2$$

$$M_{\max} = \frac{q_u B (L - a)^2}{8} = \frac{243.9 \times 2.7 (4.1 - 0.45)^2}{8} = 1096.66 \text{ kNm}$$

$$\therefore d = \sqrt{\frac{M_{\max}}{0.138 f_{ck} \times B}} = \sqrt{\frac{1096.66 \times 10^6}{0.138 \times 25 \times 2700}} = 343.12 \text{ mm}$$

Provide effective depth of 350 mm.

30. (c)

As per **Clause 26.5.3.2** of IS 456 : 2000, diameter of transverse/lateral ties is

$$\max \left\{ \begin{array}{l} \text{(i) } \frac{d_{\max}}{4} = \frac{32}{4} = 8 \text{ mm} \\ \text{(ii) } 6 \text{ mm} \end{array} \right.$$

So, diameter of tie = 8 mm

$$\text{Spacing, } S_v = \min \left\{ \begin{array}{l} \text{(i) Least lateral dimension} = 450 \text{ mm} \\ \text{(ii) } 16 \times \text{smallest dia. of longitudinal reinforcement} = 16 \times 16 = 256 \text{ mm} \\ \text{(iii) } 300 \text{ mm} \end{array} \right.$$

$\therefore$ Provide spacing of 250 mm c/c (< 256 mm)

Hence provide 8 mm ϕ ties @ 250 mm c/c as transverse reinforcement.

