CLASS TEST

Sl. : 02_SK_CE_EFGHI_300825

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REINFORCED CEMENT CONCRETE

CIVIL ENGINEERING

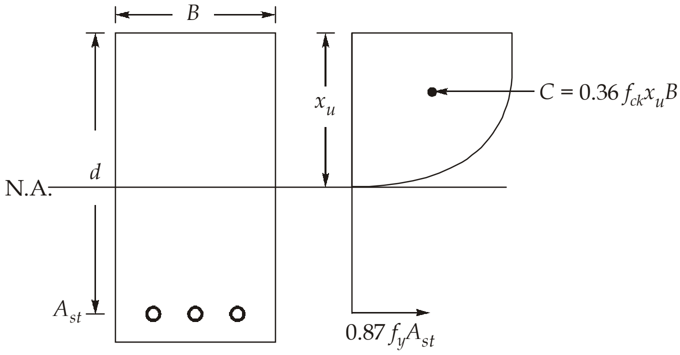
Date of Test : 30/08/2025

ANSWER KEY

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

DETAILED EXPLANATIONS

1. **(b)**



Compressive force in beam section is given as,

$$C = 0.36 f_{ck} B x_u$$

For maximum value of C,

$$x_u = x_{u\lim}$$

Where, $x_{u\lim} = 0.48 \times d = 0.48 \times 500 = 240$ mm

So, $C_{max} = 0.36 \times 30 \times 240 \times 300 \times 10^{-3}$ kN

$= 777.6$ kN

2. **(c)**

For simple supported RCC beam,

$$\frac{L}{d} = 20 \text{ for span} < 10 \text{ m}$$

For span 16 m, $\frac{L}{d}$ ratio will be be given as

$$= 20 \times \frac{10}{16} = 12.5$$

3. **(b)**

Because of non-linearity at higher strains in concrete, larger compressive stress in compression steel is developed, than that indicated by linear elastic theory using normally specified value of modular ratio (m).

4. **(b)**

5. **(b)**

For post tensioned concrete shrinkage strain $= \dfrac{2 \times 10^{-4}}{\log(t+2)} = \dfrac{2 \times 10^{-4}}{\log(8+2)} = 2 \times 10^{-4}$

But when light weight aggregates are used in dry atmospheric condition then the above value is increased by 50%.

So, design shrinkage strain $= 1.5 \times 2 \times 10^{-4}$

$= 3 \times 10^{-4}$

6. **(c)**

Effective length of column, l_e = 1.0 l = 1.0 × 4.8 m = 4800 mm

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

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

7. **(b)**

Maximum stress in steel, $\sigma_s = E_s \cdot \in_s$ ...(i)

From strain compatibility condition

$$\in_s = \in_c$$

$$= \frac{f_c}{E_c} = \frac{0.7\sqrt{f_{ck}}}{5000\sqrt{f_{ck}}}$$

$$= 1.4 \times 10^{-4}$$

Put $\in_s$ in eq. (i)

Max stress in steel, $\sigma_s = E_s \in_s = 2 \times 10^5 \times 1.4 \times 10^{-4}$

$\Rightarrow$ σ_s = 28 MPa

8. **(a)**

Dead load of the beam = 0.25 × 0.375 × 24 = 2.25 kN/m

Imposed load on the beam = 4.25 kN/m

∴ Total load on the beam = 6.50 kN/m

Maximum BM at mid-span = $6.50 \times \frac{8^2}{8} = 52\,\text{kNm}$

Section modulus, $Z = \frac{bd^2}{6} = \frac{250 \times 375^2}{6}$

$Z = 5859.375 \times 10^3\ \text{mm}^3$

When the tendon is concentrically provided,

$$\sigma_t = \frac{P}{A} - \frac{M}{Z} = 0$$

$\Rightarrow$ $\frac{P}{250 \times 375} - \frac{52 \times 10^6}{5859.375 \times 10^3} = 0$

$\Rightarrow$ P = 832000 N = 832 kN

9. **(a)**

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

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

∴ $M_x = \alpha_x w_x l_x^2$

$= 0.074 \times 8 \times 4^2 = 9.472\ \text{kNm} \simeq 9.47\ \text{kNm}$

$M_y = \alpha_y w l_x^2$

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

10. (c)

Equivalent shear force, $V_e = V + \frac{1.6T}{b}$

$\Rightarrow \quad V_e = 20 + \frac{1.6 \times 9}{0.3}$

$\Rightarrow \quad V_e = 68$ kN

11. (d)

Area of tension steel $= 4 \times \frac{\pi}{4} \times 16^2 = 804.25\,\text{mm}^2$

Actual depth of NA, $x_u = \frac{0.87 \times 250 \times 1017.88}{0.36 \times 250 \times 20}$

$\Rightarrow \quad x_u = 97.18$ mm

Limiting depth of NA, $x_{u,\text{ limit}} = 0.53d$

$= 0.53 \times 450 = 238.5$ mm

Since, $x_u < x_{u,\text{ limit}},$

Therefore beam section is under-reinforced.

Now, ultimate moment of resistance is

$M_u = 0.36\, bx_u f_{ck}(d - 0.42x_u)$

$\Rightarrow \quad M_u = 0.36 \times 97.18 \times 123 \times 20 \times (450 - 0.42 \times 97.18)$

$\Rightarrow \quad M_u = 71576172 \text{ Nmm} \simeq 71.6 \text{ kNm}$

12. (a)

$$M = \frac{wl^2}{8} = \frac{16 \times 6^2}{8} = 72\,\text{kNm}$$

$e = (300 - 200)$ mm $= 100$ mm

$P = 960$ kN

$$Z = \frac{bd^2}{6} = \frac{400 \times 600^2}{6} = 2.4 \times 10^7 \text{ mm}^3$$

Extreme stresses at the mid span section are:

$$\sigma = \frac{P}{A} \pm \frac{M}{Z} \mp \frac{Pe}{Z}$$

$$\Rightarrow \quad \sigma = \frac{960 \times 10^3}{400 \times 600} \pm \frac{72 \times 10^6}{2.4 \times 10^7} \mp \frac{960 \times 10^3 \times 100}{2.4 \times 10^7}$$

$= (4 \pm 3 \mp 4)$ MPa

$\therefore \quad \sigma_{top} = 4 + 3 - 4 = 3$ MPa

$\sigma_{bottom} = 4 - 3 + 4 = 5$ MPa

13. (c)

Prestressing force, $P = 6 \times \frac{\pi}{4} \times 6^2 \times 1150 \text{ N} = 195.09$ kN

$$I = \frac{120 \times 300^3}{12} = 2.70 \times 10^8 \,\text{mm}^4$$

Stress in concrete at the level of steel is,

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

$$\Rightarrow \quad f_c = \frac{195.09 \times 10^3}{120 \times 300} + \frac{195.09 \times 10^3 \times 55^2}{2.70 \times 10^8}$$

$$\Rightarrow \quad f_c = 7.60 \text{ MPa}$$

∴ Loss of stress due to creep of concrete.

$$= \phi \cdot m \cdot f_c \qquad \text{where } m = \frac{E_s}{E_c}$$

$$= 1.50 \times \frac{20}{3} \times 7.60 = 76 \text{ MPa}$$

14. (c)

We know, for helical reinforcement upto 1 m length as per IS 456 : 2000.

$$\frac{0.36 f_{ck}}{f_y}\left[\frac{A_g}{A_c} - 1\right] = \frac{V_h}{V_c} \qquad \text{...(i)}$$

A_g = Gross column area = $\frac{\pi}{4}D_g^2 = \frac{\pi}{4} \times 550^2$ = 237582.94 mm^2

A_c = Core area = $\frac{\pi}{4}D_c^2 = \frac{\pi}{4}\left(D_g - 2 \times \text{Cover}\right)^2 = \frac{\pi}{4} \times 450^2$ = 159043.12 mm^2

V_c = Volume of core per meter = 1000 A_c = 159043128.1 mm^3

$V_h = \frac{1000}{P} \times \pi D_H \times \frac{\pi}{4}\phi_h^2$ = Volume of helix (Where $D_H = D_C - \phi_h$ = 450 – 8 = 442 mm, ϕ_h = 8 mm)

$$\Rightarrow V_h = \frac{1000}{P} \times \pi \times 442 \times \frac{\pi}{4} 8^2 = \frac{69797842.32}{P} \text{ mm}^3$$

Substituting in equation (i)

$$\frac{0.36 \times 25}{415}\left[\frac{237582.94}{159043.12} - 1\right] = \frac{69797842.32}{P \times 159043128.1}$$

$$P = 40.97 \text{ mm}$$

$$\text{Also } P = 40.97 \text{ mm} \begin{cases} < 75 \text{ mm} \\ > 25 \text{ mm} \\ > 3\phi_h = 3(8) = 24 \text{ mm} \\ < \frac{D_c}{6} = \frac{450}{6} = 75 \text{ mm} \end{cases}$$

Now volume of helix throughout length of column,

$$= \frac{3500}{P} \times \pi D_h \times \frac{\pi}{4}\phi_h^2$$

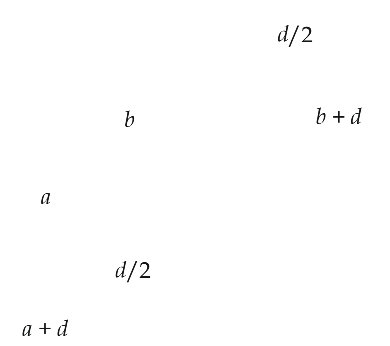
$$= \frac{3500}{40.97} \times \pi \times 442 \times \frac{\pi}{4} \times 8^2 \times 10^{-3}\ \text{cm}^3$$

$= 5962.71\ \text{cm}^3$

15. (a)

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

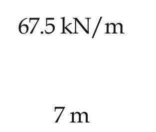
$\because$ Critical section for two way shear will be at $\frac{d}{2}$ distance from column face



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

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

16. (c)



Factored, $\quad \text{BM} = \frac{67.5\times 7^2}{8} = 413.44\ \text{kNm}$

For balanced section of singly reinforced beam,

$$M_{u\ \lim} = 0.36 f_{ck} B (x_u)_{\lim}\left[d - 0.42(x_u)_{\lim}\right]$$

where $\quad (x_u)_{\lim} = 0.48 \times 650 = 312\ \text{mm}$

$\therefore \quad M_{u\ \lim} = 0.36 \times 20 \times 300 \times 312 \times [650 - 0.42 \times 312]$

$= 349.73\ \text{kNm} < \text{BM}$

$(M_u)_{\lim}$ for singly reinforced section of Fe415 is

$$M_{u\ \lim} = 0.138 f_{ck} bd^2$$

$$= 0.138\ (20)\ (300)\ (650)^2 \text{ Nmm}$$
$$= 349.83 \text{ kNm}$$

∴ Beam has to be designed as doubly reinforced beam

∴ $$A_{st} = \frac{MR_1}{0.87 f_y \left[d - 0.42 (x_u)_{\lim}\right]} + \frac{MR_2}{0.87 f_y (d - d_c)}$$

⇒ $$A_{st} = \frac{349.73 \times 10^6}{0.87 \times 415 \times [650 - 0.42 \times 312]} + \frac{(413.44 - 349.73) \times 10^6}{0.87 \times 415 \times [650 - 50]}$$

$$A_{st} = 2160.6 \text{ mm}^2$$

$$A_{sc} = \frac{MR_2}{(f_{sc} - 0.45 f_{ck})(d - d_c)}$$

$$= \frac{(413.44 - 349.73) \times 10^6}{(350 - 0.45 \times 20)(650 - 50)} = 311.4 \text{ mm}^2$$

∴ Total area of steel required $= 2160.6 + 311.4 \text{ mm}^2 = 2472 \text{ mm}^2$

17. (b)

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

18. (c)

Let θ be the inclination of the cable at supports

∴ $$\tan\theta = \frac{1}{24}$$

⇒ $$\theta \simeq \frac{1}{24} \text{radian} \qquad \text{(since } \theta \text{ is small)}$$

Total angle of turn of cable,

$$\alpha = 2\theta = \frac{2}{24} = \frac{1}{12} \text{radian}$$

$$P_x = P_0 \left[1 - (\mu\alpha + kx)\right]$$

⇒ $$P_0 - P_x = (\mu\alpha + kx) P_0$$

⇒ $$\frac{P_0 - P_x}{A_x} = (\mu\alpha + kx) \frac{P_0}{A_x}$$

∴ Loss of prestress due to friction $= \left[0.3 \times \frac{1}{12} + 0.0015 \times 10\right] \times 1000 \text{ N/mm}^2 = 40 \text{ MPa}$

19. (d)

Given, $B = 300$ mm, $D = 500$ mm

$f_y = 415 \text{ N/mm}^2$, $f_{ck} = 15 \text{ N/mm}^2$

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

$$V = 70 \text{ kN}$$

∴ Factored shear force, $V_u = 1.5 \times 70 = 105$ kN

$$A_{sv} = 2 \times \frac{\pi}{4} \times 10^2 = 157.08 \text{ mm}^2$$

$\because$ Clear cover = 25 mm

$\therefore$ Effective cover to reinforcement bars = 25 + Stirrups diameter + $\dfrac{\text{Main reinforcement dia.}}{2}$

$$= 25 + 10 + \frac{20}{2} = 45 \text{ mm}$$

$\therefore$ d = 500 – 45 = 455 mm

Now,percent percentile tensile reinforcement,

$$p_t = \frac{100A_{st}}{Bd} = \frac{100 \times 942.5}{300 \times 455} = 0.69\%$$

From table, for $\dfrac{100A_{st}}{Bd} = 0.69$ and f_{ck} = 15 N/mm^2

$$\tau_c = 0.46 + \frac{0.54 - 0.46}{0.75 - 0.5} \times (0.69 - 0.5) = 0.52 \text{ N/mm}^2$$

Now, $$S_v = \frac{0.87 f_y A_{sv} d}{V_u}$$

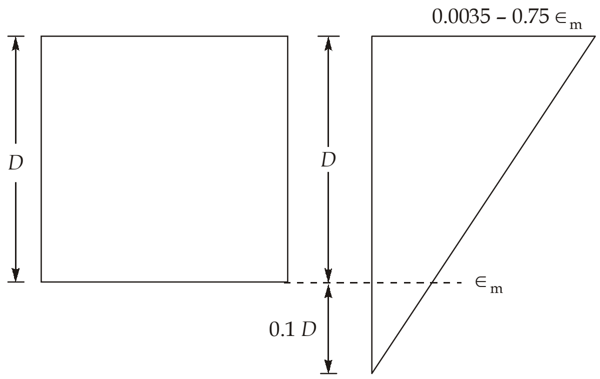
$\Rightarrow$ $$S_v = \frac{0.87 f_y A_{sv} d}{(\tau_v - \tau_c)Bd} \quad \text{where } \tau_v = \frac{V_u}{Bd} = \frac{105 \times 10^3}{300 \times 455} = 0.77 \text{ N/mm}^2$$

$\Rightarrow$ $$S_v = \frac{0.87 \times 415 \times 157.08 \times 455}{(0.77 - 0.52) \times 300 \times 455}$$

$$S_v = 756.18 \text{ mm} > 0.75(d) = 0.75(455) = 341.25 \text{ mm}$$
$$> 300 \text{ mm}$$

$\therefore$ S_v = 300 mm

20. (c)



$$\frac{0.0035 - 0.75 \in_m}{1.1D} = \frac{\in_m}{0.1D}$$

$\Rightarrow$ $11.75 \in_m = 0.0035$

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

$\therefore$ Strain in extreme compression fibre,

$$\in = 0.0035 - 0.75 \in_m$$

$$\Rightarrow \qquad \in = 0.0035 - 0.75 \times \frac{0.0035}{11.75}$$

$$\Rightarrow \qquad \in = 0.003276596$$

$$\Rightarrow \qquad \in = 3276.596\ \mu$$

21. (c)

Refer Clause 17.4 of IS 456:2000 [Page no. 30]

22. (b)

Refer clause 8.2.4.2, clause 8.2.5.3 and Table 4 of IS-456 : 2000.

23. (a)

This can be proved as follows for the case of a simply supported rectangular beam, subjected to a uniformly distributed load w per unit length.

$$\Delta = \frac{5}{384}\frac{wl^4}{EI}$$

$$M_{\max} = \frac{wl^2}{8}$$

$$w = (M_{\max})\frac{8}{l^2} = (\sigma Z)\frac{8}{l^2}$$

$$= \left(\sigma\frac{bD^2}{6}\right)\frac{8}{l^2}$$

where σ is the bending stress at service loads and $z = \frac{bD^2}{6}$ is the section modulus.

$$\frac{\delta}{L} = \text{constant}\left(\frac{L}{d}\right)$$

where, in the present case of simply supported beam with uniformly distributed loading, the 'constant' works out to $\frac{5\sigma}{24E}$.

Concrete is not linearly elastic material.

24. (a)

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

Ultimate load for the column.

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

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

$$= 1890.705\ \text{kN}$$

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

25. (d)

Refer Clause 40 of IS 456:2000.

26. (c)

In direct tension,

$$\text{Lap length} = \text{Maximum of} \begin{Bmatrix} 2L_d \\ 30\,\phi \end{Bmatrix}$$

$$L_d = \frac{\phi(0.87 f_y)}{4\tau_{bd}} = \frac{16 \times 0.87 \times 415}{4 \times 1.2 \times 1.6} = 752.1875 \text{ mm}$$

$$2L_d = 2 \times 752.1875 = 1504.375 \text{ mm}$$

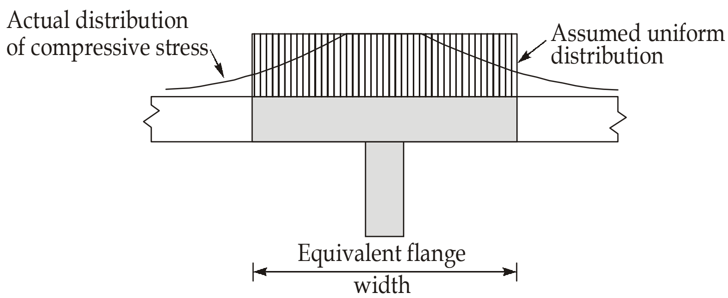
$$30\phi = 30 \times 16 = 480 \text{ mm}$$

Note: For bars of unequal diameters, lap length is calculated corresponding to smaller diameter bar.

$$\therefore \quad \text{Lap length in direct tension} = \text{Maximum of} \begin{cases} 1504.375 \text{ mm} \\ 480 \text{ mm} \end{cases}$$

$$= 1504.375 \text{ mm} \simeq 1504.4 \text{ mm}$$

27. (a)



28. (a)

Refer Fig. 4 of IS 456 : 2000

29. (c)

In pre-tensioned member, total shrinkage strain must be considered, while in post-tensioned member, shrinkage of concrete after transfer of prestressing force is only taken into account.

30. (a)

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