CLASS TEST

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REINFORCED CEMENT CONCRETE

CIVIL ENGINEERING

Date of Test : 04/07/2024

ANSWER KEY

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

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. (d)

$$\text{Shrinkage loss} = \in E_C$$

$$\text{Creep loss} = m\,\phi\, f_C$$

$$\Rightarrow \quad \left(\frac{200}{35}\right)\times 1.6\times f_C = 3\times 200\times 10^{-6}\times 200\times 10^{3}$$

$$\Rightarrow \quad f_C = 13.125 \text{ MPa} \simeq 13 \text{ MPa}$$

3. (a)

Concrete if allowed to dry out quickly undergoes considerable early age shrinkage which can cause shrinkage cracks. Besides, curing also ensures the cement hydration reaction to progress steadily producing calcium silicate hydrate gel making the concrete denser thereby decreases the porosity and enhances the physical and the mechanical properties of concrete.

4. (c)

$$\text{Equivalent shear, } V_e = V_U + \frac{1.6\,T_U}{b}$$

Here, $V_U = 0$

$$\therefore \quad V_e = 1.6\times\frac{50\times 10^{3}}{300} = 266.67 \simeq 267\text{ kN}$$

5. (c)

The 3rd assumption is wrong.

The relationship between the compressive stress distribution and the compressive strain in concrete may be assumed to be rectangular, trapezoid, parabola or any other shape.

6. (c)

The zone factor, Z is given in Table – 2 of IS 1893 (Part – 1) : 2002. For zone III, it is moderate and the zone factor, Z is 0.16. For zone II, it is 0.08, for zone IV, it is 0.24 and for zone V, it is 0.36.

7. (a)

Refer **IS 456 : 2000, Clause 26.5.3.2(a)**

8. (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) 1.20 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 i.e. 480 kN/m.

9. (a)

For main reinforcing bars = $3d$ or 300 mm whichever is less

= 3 × 90 or 300 mm = 270 mm

For distribution bars :

Maximum spacing = $5d$ or 450 mm = 5 × 90 or 450 mm whichever is less

= 450 mm

10. (a)

Marcus correction factor = $1-\frac{5}{6}\left(\frac{r^2}{1+r^4}\right)$ where r is the ratio of long span to short span.

11. (b)

Stress in concrete at the level of tendon,

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

$$= \frac{150\times1000}{120\times200}+\frac{150\times20^2\times12\times10^3}{120\times200^3} = 7\text{ MPa}$$

Loss of prestress due to eleastic deformation,

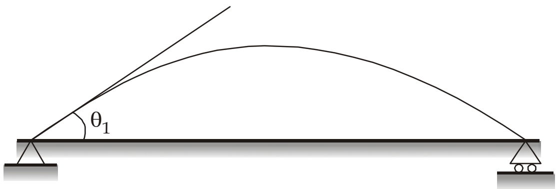
$$\Delta f_s = \frac{E_S}{E_C}\times f_C = \frac{2.1\times10^5}{3\times10^4}\times7 = 49\text{ MPa}$$

Initial stress in steel, $f_S = \frac{P}{A_S} = \frac{150\times10^3}{187.5} = 800\,\text{MPa}$

∴ Percentage loss of prestress = $\frac{\Delta f_S}{f_S}\times100 = \frac{49}{800}\times100 = 6.125\% \simeq 6.13\%$

12. (c)

Slope of beam due to P force



$$\theta_1 = \frac{PeL}{2EI}$$

$$\Rightarrow \quad \theta_1 = \frac{400\times1000\times50\times8000}{2\times2\times10^4\times10^9} = 4\times10^{-3}\text{ rad(upward)}$$

Slope of beam due to UDL

$$\theta_2 = \frac{wL^3}{24EI} = \frac{8\times8000^3}{24\times2\times10^{13}}$$

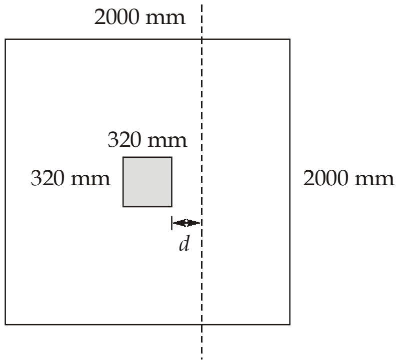
$$= 8.533\times10^{-3}\text{rad (downward)}$$

$\therefore$ Net slope of beam $= (8.533 - 4) \times 10^{-3}$

$= 4.533 \times 10^{-3}$ rad (downward)

$\therefore$ Total increase in length $= 2e\theta = 2 \times 50 \times 4.533 \times 10^{-3}$ mm $= 0.4533$ mm

13. (a)



One way shear is checked at a distance 'd' from face of column.

$$\text{Upward soil pressure, } w_u = \frac{340}{2 \times 2} = 85 \text{ kN/m}^2$$

$$\text{Overhang} = \left(\frac{2000 - 320}{2} - 250\right) = 590 \text{ mm}$$

So one-way shear force at critical section,

$$V_u = 85 \times 2 \times 0.59 = 100.3 \text{ kN}$$

$$\therefore \quad \text{One-way shear stress}, \tau_v = \frac{V_u}{bd} = \frac{100.3 \times 1000}{2000 \times 250} = 0.2006 \text{ MPa} \simeq 0.20 \text{ MPa}$$

14. (b)

As per **Cl.26.5.1.1** of **IS 456:2000**

$$\frac{A_{st\,min}}{Bd} = \frac{0.85}{f_y}$$

$$\Rightarrow \quad A_{st,min} = \frac{0.85}{500} \times 300 \times 550 = 280.5 \text{ mm}^2$$

$$\text{Number of 12 mm bars} = \frac{280.5}{\frac{\pi}{4} \times 12^2} = 2.48$$

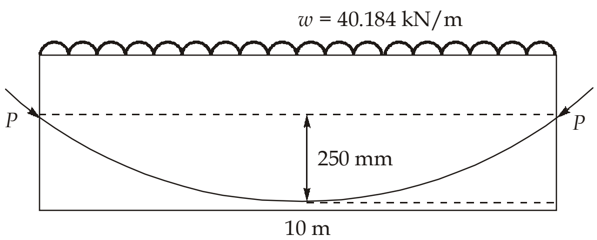
So 3 bars are required.

15. (a)

Dead load $= 24 \times 0.3 \times 0.72 = 5.184$ kN/m

Live load $= 35$ kN/m

Total load $= 35 + 5.184 = 40.184$ kN/m



$$\text{BM at mid span} = \frac{wL^2}{8} = \frac{(40.184)\times(10)^2}{8} = 502.3 \text{ kN.m}$$

Given stress at bottom of mid span = 0

$$\Rightarrow \qquad f_b = 0$$

$$\frac{P}{A} + \frac{Pe}{Z_b} - \frac{M}{Z_b} = 0$$

$$\frac{P\times10^3}{300\times720} + \frac{P\times250\times10^3}{\frac{300\times(720)^2}{6}} - \frac{502.3\times10^6}{\frac{300\times(720)^2}{6}} = 0 \qquad \text{[Assume P (in kN)]}$$

$$P = 1357.56 \text{ kN}$$

16. (a)

17. (b)

Table 26 of **IS 456 : 2000** gives moment coefficients. Support moment is about $\frac{4}{3}$ times the mid-span moment. Load distribution is triangular for short span and trapezoidal for long span.

18. (c)

19. (c)

As per LSM, minimum strain at yield in tension steel, $\in_s \geq 0.002 + \dfrac{f_y}{1.15\times E_S}$

$$\geq 0.002 + \frac{500}{1.15\times2.1\times10^5}$$

$$\geq 0.00407$$

20. (b)

In LSM, concrete is taken to a higher stress level as compared to WSM.

21. (a)

For Fe250

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

So equating tension to compression i.e. $C = T$

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

where, $$A_{sc} = \frac{\pi}{4}\times16^2\times2 = 402.12 \text{ mm}^2$$

$$A_{st} = \frac{\pi}{4}\times25^2\times4 = 1963.5 \text{ mm}^2$$

$0.36 \times 20 \times x_u \times 250 + 217.5 \times 402.12 = 0.87 \times 250 \times 1963.5$

$$\Rightarrow \qquad x_u = 188.67 \text{ mm}$$

Also $\qquad x_{u,\lim} = 0.53 \times d = 238.5 \text{ mm} > x_u$

So it is an UR section

$$M_u = 0.36 f_{ck} x_u B\left[d - 0.42x_u\right] + f_{sc} A_{sc}\left[d - d'\right]$$

$$= 0.36 \times 20 \times 188.67 \times 250\,[450 - 0.42 \times 188.67] + 217.5 \times 402.12\,[450 - 50]$$
$$= 160.896 \times 10^6 \text{ Nmm}$$
$$= 160.896 \text{ kNm}$$

22. (a)

Critical section for shear shall be taken at a distance of $d/2$ from periphery of column/drop panel.

23. (a)

Refer **clause B. 5.4** of **IS 456 : 2000**

24. (d)

$$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{20 \times 0.87 \times 415}{4 \times 1.4 \times 1.6 \times 1.25} \simeq 645 \text{ mm}$$

25. (a)

Cross-sectional area, $A = (250 \times 600) = 15 \times 10^4 \text{ mm}^2$

Section modulus, $Z = \left(\frac{250 \times 600^2}{6}\right) = 15 \times 10^6 \text{ mm}^3$

Area of prestressing bar,

$$A_s = \left(\frac{\pi \times 4 \times 14^2}{4}\right) = 616 \text{ mm}^2$$

$$e = 100 \text{ mm}$$

Prestressing force, $P = (616 \times 700) = 431200$ N

$$\left(\frac{P}{A}\right) = 2.87 \text{ N/mm}^2,$$

$$\left(\frac{Pe}{Z}\right) = 2.87 \text{ N/mm}^2$$

Prestress at the soffit of the beam = (2.87 + 2.87) = 5.74 N/mm^2

If M = Maximum moment on the section for zero tension at the bottom face, then

$$\left(\frac{M}{Z}\right) = 5.74$$

$\therefore$ $M = (5.74 \times 15 \times 10^6) = 86.1 \times 10^6 \text{ Nmm} = 86.1 \text{ kNm}$

26. (d)

$$\text{Minimum eccentricity} = \frac{L}{500} + \frac{B}{30} = \frac{3000}{500} + \frac{450}{30}$$
$$= 21 \text{ mm} > 20 \text{ mm}$$

So, $e_{min} = 21$ mm

27. (b)

Given : l = 3200 mm, D = 400

$\Rightarrow$ Slenderness ratio $= \frac{l}{D} \leq \frac{3200}{400} = 8.0$ (as column is braced)

As $\frac{l}{D} \leq 12$, the column may be designed as a short column

Minimum eccentricity

$$e_{min} = \frac{l}{500} + \frac{D}{30}$$

$$e_{min} = \frac{3200}{500} + \frac{400}{30} = 19.73 \text{ mm } (< 20.0 \text{ mm})$$

Factored load,

$$P_u = 1500 \text{ kN (given)}$$

For spiral column, $P_u = 1.05\left[0.4 f_{ck} A_g + \left(0.67 f_y - 0.4 f_{ck}\right) A_{sc}\right]$

Design of longitudinal reinforcement

$$\frac{1500 \times 10^3}{1.05} = 0.4 \times 25 \times \frac{\pi \times 400^2}{4} + (0.67 \times 415 - 0.4 \times 25) A_{sc}$$

$$\Rightarrow \quad 1428.6 \times 10^3 = 1256.6 \times 10^3 + 268.05 A_{sc}$$

$$\Rightarrow \quad A_{sc} = (1428.6 - 1256.6) \times \frac{10^3}{268.05}$$

$$= 642 \text{ mm}^2$$

Percentage reinforcement $= \dfrac{642}{\frac{\pi}{4} \times (400)^2} \times 100 = 0.51\% < 0.8\%$

So, $A_{sc,min}$ @ 0.8% of $A_g = \dfrac{0.8}{100} \times \dfrac{\pi \times 400^2}{4} = 1005 \text{ mm}^2$

28. (d)

For slab resting on top flange of beam, only friction force resists lateral buckling. But in design, friction is not considered and shear connectors are designed for distributing full lateral force.

(a) Both Statement (I) and Statement (II) are individually true and Statement (II) is the correct explanation of Statement (I).

(b) Both Statement (I) and Statement (II) are individually true but Statement (II) is NOT the correct explanation of Statement (I).

(c) Statement (I) is true but Statement (II) is false.

(d) Statement (I) is false but Statement (II) is true.

29. (b)

30. (c)

The shear resistance reinforcement can be provided by bending the longitudinal tensile reinforcement. Generally two bars are bent at a section where they are no more required for resisting bending moment.

Now a days, the practice of bending up of reinforcement bars for shear consideration has been discontinued.

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