CLASS TEST

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REINFORCED CEMENT CONCRETE

CIVIL ENGINEERING

Date of Test : 24/07/2023

ANSWER KEY

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

The working stress method assumes that concrete is elastic and the stress in concrete varies linearly from zero at the neutral axis to a maximum at the extreme fibre. The bond between steel and concrete is perfect within the elastic limit of steel.

3. (a)

Concrete i.e. 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)

5. (a)

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

6. (b)

As per IS-456:2000; Table - 3 and table -16, nominal cover $\ngtr$ 30 mm. and according to clause 26.4.2.1 of IS-456 : 2000, nominal cover $\nless$ 40 mm for column any case.

So, here, answer will be 40 mm.

7. (d)

8. (b)

Maximum spacing for vertical stirrups, $k_1 = 0.75d$ and for inclined stirrups, $k_2 = d$

where d is effective depth.

So, $$\frac{k_1}{k_2} = 0.75$$

9. (c)

At failure in URS beam,

$$\text{Strain in steel} >> 0.002 + \frac{0.87 f_y}{E_s}$$

$\therefore$ Failure occurs due to secondary compression failure of concrete.

10. (c)

11. (d)

At initial stage, no losses are considered and only DL is considered.

At mid span, moment due to dead load:

$$M_d = \frac{wl^2}{8} = \frac{10 \times 6^2}{8} = 45 \text{ kNm}$$

Stress at bottom of mid span $= \dfrac{P}{A} + \dfrac{Pe}{Z} - \dfrac{M_d}{Z}$

$$= \frac{500 \times 10^3}{300 \times 500} + \frac{500 \times 10^3 \times 100}{\frac{300 \times 500^2}{6}} - \frac{45 \times 10^6}{\frac{300 \times 500^2}{6}}$$

$= 3.33 + 4 - 3.6 = 3.75 \text{ N/mm}^2$

12. (c)

Here, $L_o = 0.7 \times l_{eff} = 0.7 \times 6 = 4.2$ m

Now, Effective flange width,

$$b_f = \min.\begin{cases} \dfrac{L_o}{6} + b_w + 6D_f \\ \dfrac{L_1}{2} + b_w + \dfrac{L_2}{2} \end{cases} = \min.\begin{cases} 700 + 300 + 1200 \\ 500 + 300 + 1000 \end{cases}$$

$$= \min.\begin{cases} 2200 \text{ mm} \\ 1800 \text{ mm} \end{cases}$$

$$b_f = 1800 \text{ mm}$$

13. (b)

Given: $d = 600$ mm, $N = 300$ mm

$$M_{Tu} = \frac{T_u}{1.7}\left(1 + \frac{D}{B}\right) = \frac{1.5 \times 34}{1.7}\left(1 + \frac{600}{300}\right) = 90 \text{ kNm}$$

Equivalent sagging moment, $M_{ue1} = M_{Tu} + M_u$

$= 0 + 90 = 90$ kNm

Equivalent hogging moment, $M_{ue2} = M_u - M_{Tu}$

[Only applicable when $M_{Tu} > M_u$]

$M_{ue2} = 0 - 90$ kNm

$= -90$ kNm $= 90$ kNm [Hogging Bending Moment]

14. (b)

$$r = \frac{l_y}{l_x} = \frac{4}{3}$$

Maximum shear force along larger edge,

$$F_L = \frac{w_u l_x \times r}{2+r} = \frac{12 \times 3 \times \frac{4}{3}}{2+\frac{4}{3}} = \frac{48 \times 3}{10} = 14.4 \text{ kN/m}$$

Maximum shear force along shorter edge,

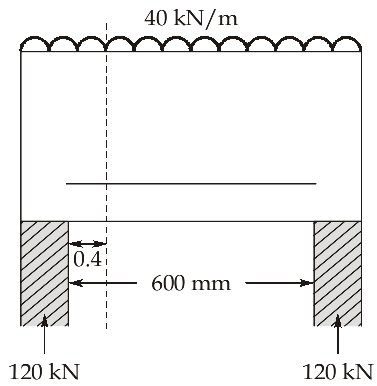
$$F_S = \frac{w_u l_x}{3} = \frac{12 \times 3}{3} = 12 \text{ kN/m}$$

$$\frac{F_L}{F_S} = \frac{14.4}{12} = 1.2$$

15. (d)

Spalling stresses are major concern for post-tensioned PSC members.

16. (d)



At critical section, $V_u = 120 - (40 \times 0.4) = 140 \text{ kN}$

Shear strength of concrete, $V_c = \tau_c \times bd = 0.6 \times 300 \times 400$

$= 72 \text{ kN}$

Design shear force, $V_s = V_u - V_c = 104 - 72 = 32 \text{ kN}$

17. (b)

Refer IS 456 : 2000 Cl. 26.2.3.1 and 26.2.3.2

18. (a)

Total vertical load, $W = 300$ kN

Distance of point of application of the resultant force from the heel end = 2 m

hence, Distance of point of application of the resultant force from the toe end = 3.5 – 2 = 1.5 m

$$\text{So, eccentricity, } e = \frac{b}{2} - \bar{x} = \frac{3.5}{2} - 1.5$$

$$= 0.25 \text{ m}$$

$$\frac{b}{6} = \frac{3.5}{6} = 0.583 \text{ m}$$

$$\therefore \quad e < \frac{b}{6}$$

Extreme pressure intensity at the base,

$$= \frac{W}{b}\left[1 \pm \frac{6e}{b}\right]$$

$$= \frac{300}{3.5}\left[1 \pm \frac{6 \times 0.25}{3.5}\right]$$

$= 122.449$ kN/m^2 and 48.98 kN/m^2

or, $\simeq 122$ kN/m^2 and 49 kN/m^2

19. (c)

For simply supported, $\left(\frac{l}{d}\right)_{basic} = 20$

For span > 10 m

$$\left(\frac{l}{d}\right)_{basic} = 20 \times \frac{10}{24}$$

$$\therefore \quad \left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} k_t \, k_c$$

$$\therefore \quad d_{min} = \frac{24000}{\frac{200}{24} \times 1.6 \times 1.2} = 1500 \text{ mm}$$

20. (a)

$$\because \quad E_\theta = \frac{5000\sqrt{f_{ck}}}{1+\theta}$$ (Refer Cl. 4.1 of **IS 456 : 2000**)

$\because \quad f_{ck} = f_m - 1.65\sigma = 36 - 1.65 \times 4 = 29.4$ MPa

$$E_\theta = \frac{5000 \times \sqrt{29.4}}{1+1.5} = 10844.35 \text{ MPa} \approx 10845 \text{ MPa}$$

21. (b)

B.M. at support next to the end support $= \frac{1}{10} w_d L^2 + \frac{1}{9} \times w_l l^2$ (Table 12 of **IS 456 : 2000**)

$$= \frac{1}{10} \times 6 \times 4.5^2 + \frac{1}{9} \times 3 \times 4.5^2$$

$= 18.9$ kN-m

22. (a)

Given $\phi = 18$ mm

$\tau_{bd} = 1.5$ N/mm^2 and Fe415 grade steel

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

$\because$ Bars are in compression and HYSD bars are used, so increase bond stress by 25% and 60% respectively.

$$\because \qquad L_d = \frac{0.87\times415\times18}{4\times1.5\times1.25\times1.6} = 541.575 \text{ mm} \approx 542 \text{ mm}$$

23. (c)

$$\frac{P_u}{f_{ck}bD} = \frac{810\times10^3}{20\times450\times450} = 0.2$$

From the interaction diagram

For $\quad \dfrac{P_u}{f_{ck}bD} = 0.2$

$$\frac{M_u}{f_{ck}bD^2} = 0.4$$

$$\Rightarrow \qquad M_u = 0.4 \times 20 \times 450 \times 450^2 \text{ N-mm}$$

$$= 729 \times 10^6 \text{ N-mm}$$

$$\because \qquad e = \frac{M_u}{P_u} = \frac{729\times10^6}{810\times10^3} \text{ mm}$$

$$\Rightarrow \qquad e = 900 \text{ mm}$$

$$L_d = \frac{0.87\times415\times18}{4\times1.6\times1.25\times1.5} = 541.575 \text{ mm} \simeq 541.58 \text{ mm}$$

24. (d)

$$\because \qquad \frac{\text{Reinforcement in central band width}}{\text{Total reinforcement in short direction}} = \frac{2}{\beta+1}$$

where $\quad \beta = \dfrac{\text{Long side}}{\text{Short side}} = \dfrac{4.5}{3} = 1.5$

$$\therefore \quad \text{Reinforcement in central band width} = \frac{2}{1.5+1}\times825 = 660 \text{ mm}^2$$

25. (b)

$$\text{Factored load, } P_u = 1.05\left(0.4 f_{ck}A_c + 0.67 f_y A_{sc}\right)$$

$$= 1.05\left[0.4\times20\times\left(\frac{\pi}{4}\times420^2 - 2100\right) + 0.67\times415\times2100\right] \text{ N}$$

$$= 1759.23 \text{ kN} \approx 1760 \text{ kN}$$

26. (c)

In post-tensioned prestressed beam

Shrinkage strain in concrete,

$$\in = \frac{2\times10^{-4}}{\log_{10}(t+2)} = \frac{2\times10^{-4}}{\log_{10}(28+2)} = \frac{2\times10^{-4}}{\log_{10}30} = 1.354\times10^{-4}$$

$$\therefore \quad \text{Loss of stress in steel} = E_s \times \in$$

$$= 1.354 \times 10^{-4} \times 2.1 \times 10^5 = 28.43 \text{ N/mm}^2$$

27. (a)

$$\text{Target mean strength} = f_{ck} + 1.65\,\sigma$$
$$= 20 + 1.65 \times 4$$
$$= 26.6 \text{ MPa}$$

$$\text{Water content required} = 50 - \frac{50-45}{35-25} \times (26.6 - 25) = 49.2$$

28. (d)

$$\because \quad E_s = \frac{E}{1+\theta}$$

$$\Rightarrow \quad 11904.76 = \frac{5000 \times \sqrt{25}}{1+\theta}$$

$$\Rightarrow \quad \theta = \text{Creep coefficient} = 1.1$$

From table, $t = 365$ days

29. (c)

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

$$\max \begin{cases} (i) & \dfrac{d_{\max}}{4} = \dfrac{32}{4} = 8 \text{ mm} \\ (ii) & 6 \text{ mm} \end{cases}$$

So, diameter of tie = 8 mm

$$\text{Spacing,} \quad S_v = \min \begin{cases} \text{(i)} & \text{Least lateral dimeniosn} = 450 \text{ mm} \\ \text{(ii)} & 16 \times \text{smallest dia. of longitudinal reinforcement} = 16 \times 16 = 256 \text{ mm} \\ \text{(iii)} & 300 \text{ mm} \end{cases}$$

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

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

30. (c)

Given: $A_{st} = 1000 \text{ mm}^2$

$A_g = B \times D = 300 \times 450 = 135000 \text{ mm}^2$

As per Cl. 40.2.2 of IS 456 : 2000, for a beam subjected to axial compression, the design shear strength is multiplied by a factor

$$\delta = 1 + \frac{3P_u}{A_g f_{ck}} \qquad \text{but} \not> 1.5$$

$$\therefore \quad \delta = 1 + \frac{3 \times 1.5 \times 120 \times 10^3}{135000 \times 20}$$

$$= 1.2 < 1.5 \qquad \text{(OK)}$$

$\therefore$ Design shear strength concrete = $1.2 \times 0.512 = 0.614 \text{ N/mm}^2$