CLASS TEST

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Design of Steel Structures

CIVIL ENGINEERING

Date of Test : 12/11/2025

ANSWER KEY

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

DETAILED EXPLANATIONS

5. (b)

Plastic section can develop plastic moment resistance and plastic hinge, but compact section can reach upto plastic moment of resistance and cannot make plastic hinge.

9. (a)

When $\theta < 60°$

The design force for connection is $V \cot \theta$.

$$V = \frac{2.5}{100} \times 750 = 18.75 \text{ kN}$$

$$\therefore \quad V \cot 45° = 18.75 \text{ kN.}$$

11. (d)

For simultaneous collapse of *AB* and *BC*. Plastic hinge are formed at *A*, *B* and midspan of *AB*.

$$\therefore \quad 2M_p = \frac{wl^2}{8}$$

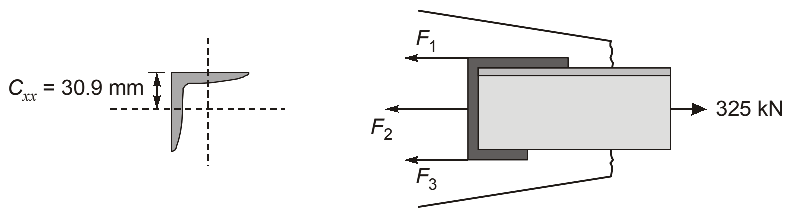
$$\therefore \quad M_p = \frac{wl^2}{16}$$

$$M_p = \frac{wa^2}{2}$$

$$\therefore \quad \frac{a^2}{2} = \frac{l^2}{16}$$

$$\therefore \quad a = \frac{l}{2\sqrt{2}}$$

12. (d)



$$F_1 + F_2 + F_3 = 325 \text{ kN}$$

$$F_2 = \frac{350 \times 110}{1000} = 38.5 \text{ kN}$$

$$\Rightarrow \quad F_1 + F_3 = 286.5 \text{ kN} \quad \text{...(i)}$$

For no torsional moment to develop,

$$F_1(30.9) - F_3(110 - 30.9) - F_2(55 - 30.9) = 0 \quad \text{...(ii)}$$

Solving (i) and (ii)

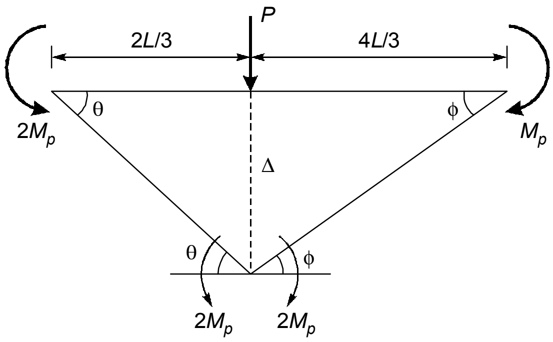
13. (a)

$$D_s = 2$$

∴ No. of plastic hinges required for complete collapse

$$D_s + 1 = 2 + 1 = 3$$

Mechanism-1



$$\Delta = \frac{2L}{3}\theta = \frac{4L}{3}\phi$$

$$\Rightarrow \quad \theta = 2\phi$$

By principle of critical work done,

$$-2M_p\theta - 2M_p\theta - 2M_p\phi - M_p\phi + P\left(\frac{2L}{3}\theta\right) = 0$$

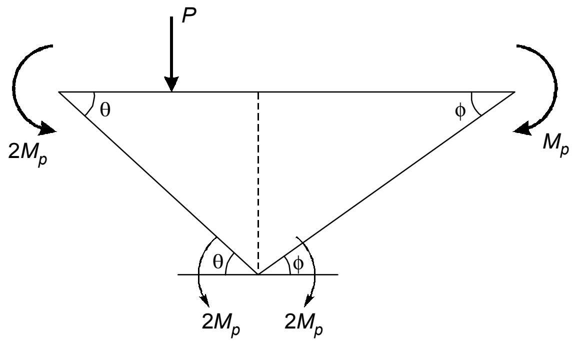
$$\Rightarrow \quad 4\,M_p\theta + 3\,M_p\phi = \frac{2P_L}{3}\theta$$

$$\Rightarrow \quad 8\,M_p\phi + 3\,M_p\phi = \frac{4P_L}{3}\phi$$

$$\Rightarrow \quad P_u = \frac{33M_p}{4L}$$

$$\Rightarrow \quad P_u = 8.25\,M_p/L$$

Mechanism-2



$$2\,M_p\theta + M_p\theta + M_p\theta + M_p\theta = P\left(\frac{2L}{3}\right)$$

$$\Rightarrow \quad P_u = \frac{15}{2L}M_p$$

$$\Rightarrow \quad P_u = \frac{7.5}{L}M_p$$

14. (a)

$$\text{Vertical force on rivet 1} = \frac{P}{4} = \frac{54.8}{4} = 13.7 \text{ kN}$$

$$\text{Moment to which the river is subjected} = 54.8 \times 0.25 = 13.7 \text{ kNm}$$

$$\text{Force acting on rivet 1 due to moment, } r = \sqrt{80^2 + 80^2} \simeq 13.7 \text{ mm}$$

$$\frac{M \times r}{4r^2} = \frac{13.7 \times 0.113}{4 \times 0.113^2} = 30.27 \text{ kN}$$

$$\text{Net force acting} = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

$$= \sqrt{13.7^2 + 30.27^2 + 2 \times 13.7 \times 30.27 \times \frac{1}{\sqrt{2}}} = 41.11 \text{ kN}$$

$$F_1 = 214.45 \text{ kN}$$

15. (c)

$$\text{Design strength of weld}, P_{dw} = l_w \times t_t \times \frac{f_u}{\sqrt{3}\gamma_{mw}} \quad \text{...(i)}$$

Minimum permissible weld size upto 20 mm thick plate as per **IS 800:2007** is

$$s = 5 \text{ mm}$$

$$t_t = 0.7 \times 5 = 3.5 \text{ mm} \qquad (> 3 \text{ mm})$$

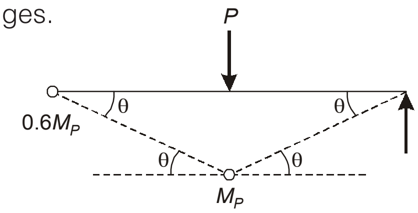
Substituting in equation (i)

$$\Rightarrow \quad 200 \times 10^3 = l_w \times 3.5 \times \frac{410}{\sqrt{3} \times 1.5}$$

$$\Rightarrow \quad l_w = 362.1 \text{ mm}$$

16. (a)

Failure in the left/right span can be caused by formation of two hinges.



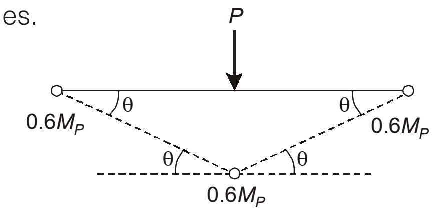
(Right Span)

Using virtual work method

$$\Rightarrow \quad P \cdot \frac{L}{2}\theta = M_P \cdot (2\theta) + 0.6M_P \cdot \theta$$

$$\Rightarrow \quad P = \frac{5.2M_P}{L}$$

The failure in the middle span will be caused by formation of 3 hinges.



(Middle Span)

$$\Rightarrow \quad P \cdot \frac{L}{2} \cdot \theta = 0.6M_P(\theta + 2\theta + \theta)$$

$$\Rightarrow \quad P = \frac{4.8M_P}{L}$$

Hence, collapse load is the minimum of the above two values, i.e. $\frac{4.8M_P}{L}$.

17. (d)

$$\text{Total area of cross-section} = 40 \times 100 + 100 \times 20 = 6000 \text{ mm}^2$$

For equal area axis

$$40 \times y = \frac{6000}{2}$$

$$\therefore \quad y = 75 \text{ mm}$$

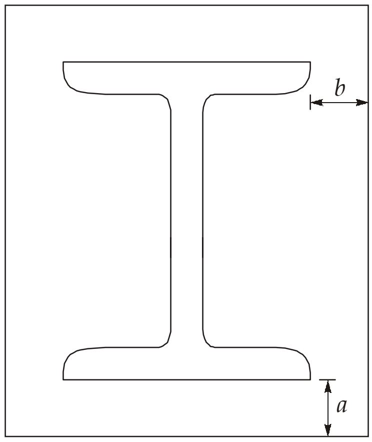
18. (b)

Bearing strength of concrete

$$= 0.60\, f_{ck} = 0.60 \times 20 = 12 \text{ N/mm}^2$$

For factored load, $P_u = 1000$ kN

Bearing pressure, $$w = \frac{1000 \times 10^3}{400 \times 300} = 8.33 \text{ N/mm}^2 < 12 \text{ N/mm}^2$$



Now, longer projection, $$a = \frac{400 - 300}{2} = 50 \text{ mm}$$

Smaller projection, $$b = \frac{300 - 250}{2} = 25 \text{ mm}$$

So, minimum thickness of base plate required is

$$t = \sqrt{\frac{2.5w\left(a^2 - 0.3b^2\right)\gamma_{mo}}{f_y}}$$

$$= \sqrt{\frac{2.5 \times 8.33 \times \left\{50^2 - 0.3(25)^2\right\} \times 1.1}{250}} = 14.56 \text{ mm}$$

19. (c)

$$d = 20 \text{ mm}, \; f_{ub} = 400 \text{ N/mm}^2$$

In this connection, packing plate of 8 mm thickness is to be used. Hence there shall be reduction in the shear strength of bolt.

This reduction factor is given by

$$\beta_{pk} = \left(1 - 0.0125 t_{pk}\right)$$
$$= (1 - 0.0125 \times 8)$$
$$= 0.9$$

$$\text{Net area of one bolt} = \frac{\pi}{4}(20)^2 \times 0.78$$

So design shear strength of one bolt in double shear

$$= \beta_{pk} \times A_{net} \times 2 \times \frac{f_{ub}}{\sqrt{3} \times 1.25}$$

$$= 0.9 \times 2 \times \frac{\pi}{4} \times (20)^2 \times 0.78 \times \frac{400}{\sqrt{3} \times 1.25} \text{N}$$

$$= 81.49 \text{ kN}$$

20. (c)

For efficient minimum spacing 'S' between the channels,

$$I_{yy} \geq I_{xx}$$

Now $$I_{xx} = 2 \times 10000 \times 10^4 = 20000 \times 10^4 \text{ mm}^4$$

$$I_{yy} = 2\left[434 \times 10^4 + \left(24.4 + \frac{S}{2}\right)^2 \times 5366\right]$$

Now, $$I_{yy} \geq I_{xx}$$

$$\Rightarrow \quad 2\left[434 \times 10^4 + \left(24.4 + \frac{S}{2}\right)^2 \times 5366\right] \geq 20000 \times 10^4$$

$$S \geq 218.20 \text{ mm}$$

21. (c)

Throat thickness of weld, $$t_t = 0.7 \text{ s}$$
$$= 0.7 \times 8 = 5.6 \text{ mm}$$

Design stress in weld, $$f_{wd} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.4 \text{ N/mm}^2$$

Design strength of weld per mm length of cylinder

$$= 2 \times 189.4 \times 1 \times 5.6$$
$$= 2121.28 \text{ N/mm}$$

Let P_d = Design fluid pressure inside the cylinder

Design hoop tension/pressure per mm length of cylinder

$$\Rightarrow \quad P_d \frac{D}{2} = \frac{P_d \times 500}{2} = 2121.28$$

$$\Rightarrow \quad P_d = 8.48 \text{ N/mm}^2$$

22. (c)

Design wind speed, $V_z = k_1\, k_2\, k_3\, k_4 V_b$

where V_b = Basic wind speed = 47 m/s

So, $$V_z = 1 \times 0.92 \times 1 \times 1.15 \times 47$$
$$= 49.726 \text{ m/s}$$

Wind pressure at height 'z' is

$$P_z = 0.6\, V_z^2$$
$$= 0.6 \times (49.726)^2$$
$$= 1483.6 \text{ N/m}^2 \text{or } 1.48 \text{ kN/m}^2$$

23. (b)

∵ Section is plastic (given)

And $V < 0.6\, V_d$

So, it is a case of low shear and thus design bending

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} \leq \frac{1.2 Z_e f_y}{\gamma_{m0}}$$

β_b = 1 for plastic section

So, $$M_d = \frac{1 \times 651.74 \times 10^3 \times 250}{1.1 \times 10^6} \le \frac{1.2 \times 573.6 \times 10^3 \times 250}{1.1 \times 10^6}$$

$$= 148.12 \text{ kNm} \le 156.44 \text{ kNm (OK)}$$

So design bending strength = 148.12 kNm.

24. **(a)**

For built-up section

Web crippling strength, $$F_w = (b_1 + 2.5t_f)\frac{f_y}{\gamma_{mo}}t_w$$

where, $$b_1 = 100 \text{ mm}$$
$$t_f = 14 \text{ mm}$$
$$t_w = 16 \text{ mm}$$

So, $$F_w = (100 + 2.5 \times 14) \times \frac{250}{1.1} \times 16 \text{ N}$$

$$= 490.91 \text{ kN}$$

25. **(d)**

Clause 10.5.10.2.2 refers to combined bearing, bending and shear stresses in butt weld.

The equivalent stress, f_e as obtained from the following formula, shall not exceed the values allowed for the parent metal.

$$f_e = \sqrt{f_b^2 + f_{br}^2 + f_b \cdot f_{br} + 3q^2} \le \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

$$f_b = 80 \times 10^3 \text{ kN/m}^2 = 80 \text{ MPa}$$
$$f_{br} = 90 \times 10^3 \text{ kN/m}^2 = 90 \text{ MPa}$$
$$q = 65.66 \text{ MPa}$$

∴ $$f_e = \sqrt{80^2 + 90^2 + 80 \times 90 + 3 \times 65.66^2} = 186.1 \text{ MPa}$$

∴ $$f_e \le \frac{f_u}{\sqrt{3}\gamma_{mw}}$$ {for site conditions γ_{mw} = 1.5}

⇒ $$f_u \ge 483.5 \text{ MPa}$$

26. **(b)**

Section is plastic (given)

and $$V < 0.6 V_d$$

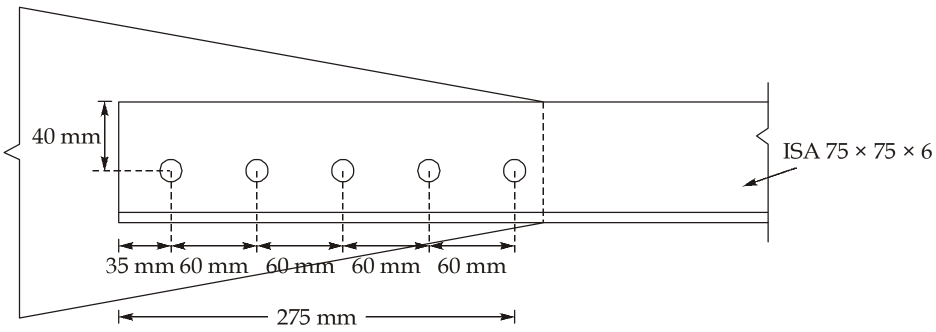
So, $$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} \le \frac{1.2 Z_e f_y}{\gamma_{m0}}$$

β_b = 1 for plastic section

So, $$M_d = \frac{1 \times 651.74 \times 10^3 \times 250}{1.1 \times 10^6} \le \frac{1.2 \times 573.6 \times 10^3 \times 250}{1.1 \times 10^6}$$

$$= 148.1 \text{ kNm} \le 156.44 \text{ kNm (OK)}$$

27. (b)



Net area in shear, $A_{vn} = (275 - 4.5 \times 20) \times 6 = 1110\ \text{mm}^2$

Gross area in shear, $A_{vg} = 275 \times 6 = 1650\ \text{mm}^2$

Net area in tension, $A_{Tn} = (40 - 0.5 \times 20) \times 6 = 180\ \text{mm}^2$

Gross area in tension, $A_{Tg} = 40 \times 6 = 240\ \text{mm}^2$

$$T_{db} = \min\begin{cases} \dfrac{A_{vg}f_y}{\sqrt{3}\gamma_{m0}} + \dfrac{0.9A_{Tn}f_u}{\gamma_{m1}} \\ \dfrac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{m1}} + \dfrac{A_{Tg}f_y}{\gamma_{m0}} \end{cases} = \min\begin{cases} \left(\dfrac{1650\times250}{\sqrt{3}\times1.1} + \dfrac{0.9\times180\times410}{1.25}\right)\times10^{-3} = 269.64\ \text{kN} \\ \left(\dfrac{0.9\times1110\times410}{\sqrt{3}\times1.25} + \dfrac{240\times250}{1.1}\right)\times10^{-3} = 243.73\ \text{kN} \end{cases}$$

Hence, block shear strength = min{269.64, 243.73}
= 243.73 kN

28. (a)

$I_{x\text{-}x}$ for compound column = $I_{x\text{-}x}$ of ISHB 250 + $I_{x\text{-}x}$ of cover plates

$$= 7983.9\times10^4 + 2\left[\frac{200\times20^3}{12} + 200\times20\times(125+10)^2\right]$$

$= 22590.57 \times 10^4\ \text{mm}^4$

$I_{y\text{-}y}$ for compound column = I_{yy} of ISHB 250 + I_{yy} of cover plates

$$= 2011.7\times10^4 + 2\times\left(\frac{20\times200^3}{12}\right) = 4678.37\times10^4\ \text{mm}^4$$

Area of compound column, A_c = Area of ISHB 250 + Area of cover plates
= 6971 + 2 × 200 × 20
= 14971 mm²

Radius of gyration along x-x,

$$r_{xx} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{22590.57\times10^4}{14971}} = 122.81\ \text{mm}$$

Radius of gyration along y-y,

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{4678.37\times10^4}{14971}} = 55.9\ \text{mm}$$

∴ Least radius of gyration = 55.9 mm

29. (b)

Let, P_1 be the factored load,

So, service load, $P = \dfrac{P_1}{1.5}$

The bolt which is stressed maximum is bolt A,

Direct force, $F_1 = \dfrac{P_1}{n} = \dfrac{P_1}{10}$

Force in the bolt due A to torque,

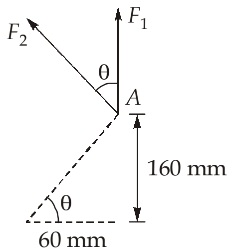
$$F_2 = \frac{Per_A}{\Sigma r_i^2}$$

Now, $r_A = \sqrt{160^2 + 60^2} = 170.88$ mm

$$\Sigma r_i^2 = 4[160^2 + 60^2] + 4[80^2 + 60^2] + 2 \times 60^2 = 164000 \text{ mm}^2$$

$$\therefore \quad F_2 = \frac{P_1 \times 250 \times 170.88}{164000} = 0.2605 P_1$$

Also, $\cos\theta = \dfrac{60}{\sqrt{160^2 + 60^2}} = 0.3511$



The resultant force on the bolt A should be less than or equal to the strength of bolt

$$45.26 \geq \sqrt{\left(\frac{P_1}{10}\right)^2 + (0.2605P_1)^2 + 2 \times \frac{P_1}{10} \times 0.2605P_1 \times 0.3511}$$

On solving we get, $P_1 = 145.96$ kN

$\therefore$ Maximum service load, $P = \dfrac{145.96}{1.5} = 97.307 \text{ kN} \approx 97.31$ kN

30. (a)

Throat thickness of weld, t_t = k.S
= 0.7×8
= 5.6 mm

Strength of fillet weld, $P_{dw} = \dfrac{f_u}{\sqrt{3}_{mw}} \times L_w \times t_t$

For connection to be safe, $P_{dw} = P_u$

$$\therefore \quad \frac{410}{\sqrt{3} \times 1.5} \times L_w \times 5.6 = 300 \times 10^3$$

L_w = 339.469 mm

$2 \times 2w$ = 339.469

$$n = \frac{139.469}{2} = 69.73 \text{ mm} \simeq 70 \text{ mm}$$

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