CLASS TEST

S.No. : 02SKCE_B+D_22082023

MADE EASY

India's Best Institute for IES, GATE & PSUs

Delhi | Bhopal | Hyderabad | Jaipur | Pune | Bhubaneswar | Kolkata

Web: www.madeeasy.in | **E-mail:** info@madeeasy.in | **Ph:** 011-45124612

DESIGN OF STEEL STRUCTURES

CIVIL ENGINEERING

Date of Test : 22/08/2023

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (c)

In equilibrium condition, mechanism condition and yield condition are satisfied, a unique value, the lowest plastic limit, is obtained. This is also called kinematic method of plastic analysis.

2. (a)

Web is relatively large in depth and thin. It is poor in compression and hence, the possibility of vertical and diagonal buckling is always there. Either the web is stiffened vertically as well as horizontally or the compressive stress in the web should be low enough to prevent buckling.

3. (a)

As per **IS 800 : 2007, Clause 7.7.2.1**

6. (d)

Rivet value is minimum of 5000 kg, 8000 kg and 6000 kg.

$$\therefore \quad \text{Rivet value} = 5000 \text{ kg}$$

$$\therefore \quad \text{Number of rivets} = \frac{84 \times 1000}{5000} = 16.8 \simeq 17$$

7. (b)

$$\begin{aligned} A_{net} &= (B - nd_{hole}) \times t \\ &= [200 - 1 \times (22 + 2)] \times 12 \\ &= 2112 \text{ mm}^2 \end{aligned}$$

9. (d)

Design steps for tension member:

1. It is an iterative method.
2. Net area, $A_{net} = \dfrac{\gamma_f P}{\dfrac{f_u}{\gamma_{m_1}}}$
3. A_{net} is increased by 20 to 40% and thus trial gross-section is chosen. Also, it is checked for gross-sectional area from the factored load.

$$A_{gross\ chosen} \geq \frac{\gamma_f P}{\dfrac{f_y}{\gamma_{m_0}}}$$

4. Designing of connection is done and safety is checked against net section rupture and against block shear failure.
5. Finally, check for maximum slenderness ratio is also done.

11. (a)

Steel structure have high strength per unit mass. The size of steel structures is small even for large structures. Thus there is saving in space in construction and improving aesthetic view.

There are standard sections of steel available in the market or (OR) can be prefabricated in the workshop/site and structure can be erected as soon as the site is ready. Hence there is saving in construction time.

The disadvantage of steel structure is that it is susceptible to corrosion and hence requires regular painting.

12. (b)

For M20 bolt of grade 4.6, Given: d = 20 mm, f_{ub} = 400 N/mm^2, f_{yb} = 240 N/mm^2

Design strength of bolt in tension,

$$T_{db} = 0.9\frac{f_{ub}}{\gamma_{m1}}A_{nb} \le \frac{f_{yb}}{\gamma_{mo}} \times A_{sb}$$

$$= 0.9\frac{400}{1.25}\times 0.78\times\frac{\pi}{4}\times 20^2\times 10^{-3}\text{ kN} \le \frac{240}{1.1}\times\frac{\pi}{4}\times 20^2\times 10^{-3}\text{ kN}$$

$$= 70.573 \text{ kN} \le 68.54 \text{ kN}$$

$$\therefore \quad T_{db} = 68.54 \text{ kN}$$

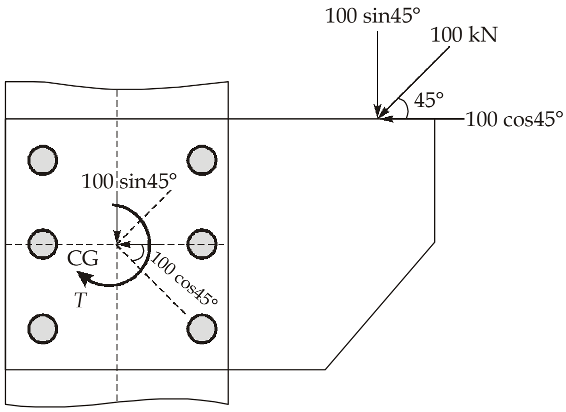
13. (a)

Strength of fillet weld, $P_{dw} = \dfrac{f_u}{\sqrt{3}\gamma_{mw}} \times l_w \times t_t = \dfrac{410}{\sqrt{3}\times 1.5}\times(220+120)\times 0.7\times 3 \text{ N}$

$= 112.676$ kN

$\therefore$ Service load, $\quad P = \dfrac{112.676}{1.5} = 75.12 \simeq 75 \text{ kN}$

14. (a)

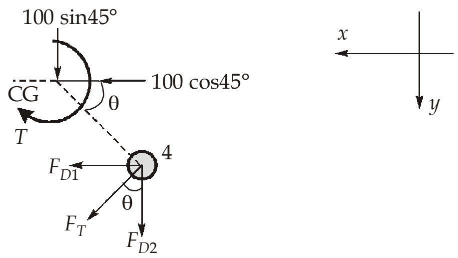


$$\sin\theta = \frac{100}{125},\ \cos\theta = \frac{75}{125}$$

$$r_4 = \sqrt{75^2 + 100^2} = 125 \text{ mm}$$

$$\Sigma r_i^2 = 4(125)^2 + 2(75)^2$$

$$= 73750 \text{ mm}^2$$



$$T = 100 \sin 45° \times 0.37 - 100 \cos 45° \times 0.13$$
$$= 16.97 \text{ kN-m}$$

Direct shear force in bolt '4'

$$F_{D1} = \frac{100 \cos 45°}{6} = 11.785 \text{ kN}$$

$$F_{D2} = \frac{100 \sin 45°}{6} = 11.785 \text{ kN}$$

Torsional shear force in bolt '4'

$$F_T = \frac{Tr_i}{\Sigma r_i^2} = \frac{16.97 \times 10^3 \times 125}{73750} = 28.76 \text{ kN}$$

$\therefore$ $$F_x = F_{D1} + F_T \sin\theta$$

$$= 11.785 + 28.76 \times \frac{100}{125} = 34.793 \text{ kN}$$

$$F_y = F_{D1} + F_T \cos\theta$$

$$= 11.785 + 28.76 \times \frac{75}{125} = 29.041 \text{ kN}$$

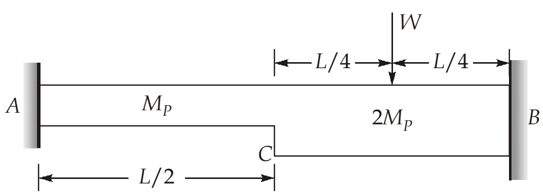
$\therefore$ Resultant force in bolt 4 $= \sqrt{F_x^2 + F_y^2} = \sqrt{(34.793)^2 + (29.041)^2}$

$$= 45.32 \text{ kN}$$

15. (b)

- Double cover butt joint eliminates the eccentricity.
- Slip critical connections are designed on the basis of friction between the plates.

16. (c)



Degree of static indeterminacy, $D_s = 2$

$\therefore$ No. of plastic hinges required for collapse,

$$n = D_s + 1 = 3$$

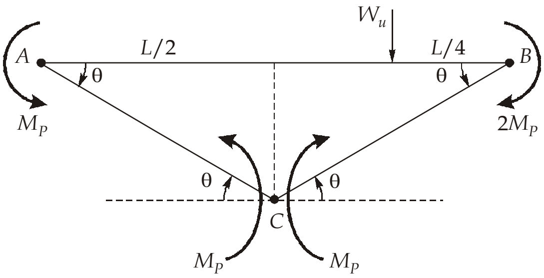
Possible location of plastic hinges are at A, B, C and under the load,

$\therefore$ $$N = 4$$

Number of independent mechanisms $= N - D_s = 4 - 2 = 2$

Mechanism: 1

Formation of plastic hinges at A, B and C



By the principle of virtual work

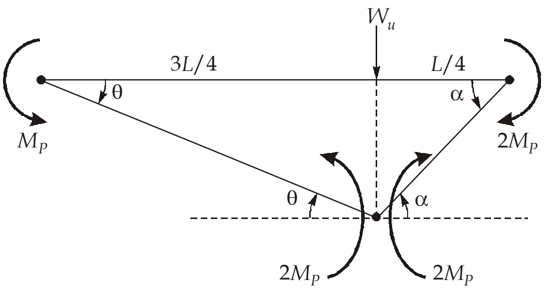
$$W_i = W_E$$

$$\Rightarrow \quad 5M_p\theta = W_u \times \frac{L}{4}\theta$$

$$\Rightarrow \quad W_u = \frac{20M_P}{L}$$

Mechanism: 2

Formation of plastic hinges at A, B and under the point load.



$$\frac{3L}{4}\theta = \frac{L}{4}\alpha$$

$$\Rightarrow \quad \alpha = 3\theta$$

By the principle of virtual work

$$W_i = W_E$$

$$\Rightarrow \quad 3M_p\theta + 4M_p\alpha = W_u \times \frac{L}{4}\alpha$$

$$\Rightarrow \quad 3M_p\theta + 4M_p(3\theta) = W_u \times \frac{L}{4} \times 3\theta$$

$$\Rightarrow \quad W_u = \frac{20M_P}{L}$$

Hence, Collapse load = Minimum of above two loads = $\dfrac{20M_P}{L}$

17. (b)

As per IS 800: 2007

$$T_{dn} = \frac{0.9 f_u}{\gamma_{m1}} A_{nc} + \frac{\beta f_y}{\gamma_{m0}} A_{go}$$

A_{nc} = Net area of connected leg

A_{go} = gross area of out standing leg.

18. (b)

Throat thickness, $t_e = \frac{5}{8} t = \frac{5}{8} \times 14$ (As full penetration of weld is not ensured)

$= 8.75$ mm

Effective length weld, $L_w = 200$ mm

$\therefore$ Strength of weld $= \frac{L_w t_e f_y}{\gamma_{mw}}$ $\left[\text{For Fe410, } f_y = 250 \text{ MPa}\right]$

$$= \frac{200 \times 8.75 \times 250}{1.25} \text{N} = 350 \text{ kN}$$

19. (c)

$\because$ Plastic hinge length for simply supported beam subjected to UDL is given as

$$x = L\sqrt{1 - \frac{1}{SF}}$$

$\because$ For diamond section SF = 2

$$\therefore \quad x = L\sqrt{1 - \frac{1}{2}}$$

$$x = 0.707L$$

20. (b)

The minimum size of fillet weld is 5 mm for plate thickness of 10 to 20 mm.

$\therefore$ $S = 5$ mm

Tensile strength of the smaller plate = $130 \times 12 \times 150 \times 10^{-3} = 234$ kN

Strength of the weld $= L \times 0.7 \times 5 \times \sigma_{ws}$

$L \times 0.7 \times 5 \times 110 = 234 \times 1000$

$\Rightarrow$ $L = 607.79$ mm

21. (b)

The section modulus required

$$Z = \frac{M}{\sigma} = \frac{4800 \times 10^6}{150} = 32 \times 10^6 \text{ mm}^3$$

Moment of inertia of plate girder $= Z \cdot \frac{d}{2} = Z \times 500$

$= 32 \times 10^6 \times 500$

$= 16 \times 10^9$ mm^4

$$\text{Moment inertia of web} = \frac{12 \times 1000^3}{12} = 10^9 \text{mm}^4$$

$$\text{Moment of inertia of flange} = 16 \times 10^9 - 10^9$$
$$= 15 \times 10^9 \text{ mm}^4$$

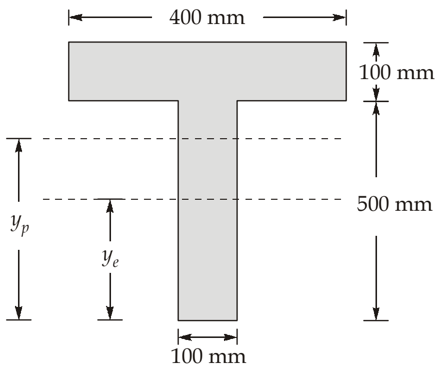
$$\therefore \quad 2A_f\left(\frac{d}{2}\right)^2 = 15 \times 10^9$$

$$\Rightarrow \quad A_f = \frac{15 \times 10^9}{2 \times 500^2} = 30000 \text{ mm}^2$$

$$\therefore \quad \text{Gross area of flange} = \frac{30000}{0.8} = 37500 \text{ mm}^2 = 375 \text{ cm}^2 \text{ on the either side}$$

$$\text{Total gross area} = 2 \times 7 = 750 \text{ cm}^2$$

22. (b)



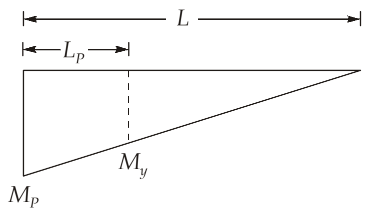
Let distance of elastic neutral axis is y_e from bottom

$$\therefore \quad y_e = \frac{\left(100 \times 500 \times \frac{500}{2}\right) + (400 \times 100 \times 550)}{400 \times 100 + 100 \times 500}$$
$$= 383.33 \text{ mm}$$

Let y_p is the distance of plastic neutral axis from bottom

$$\therefore \quad 100 \times y_p = 400 \times 100 + (500 - y_p) \times 100$$
$$\Rightarrow \quad 200\, y_p = 400 \times 100 + 500 \times 100$$
$$\Rightarrow \quad y_p = 450 \text{ mm}$$
$$\therefore \quad |y_p - y_e| = 450 - 383.33 = 66.67 \text{ mm}$$

23. (b)



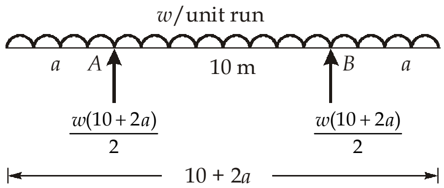
$$\frac{M_P}{M_y} = f = \frac{L}{L-L_P} \qquad \text{where } f \text{ is the shape factor}$$

$$\Rightarrow \quad \frac{1}{f} = \frac{L-L_P}{L} = 1-\frac{L_P}{L}$$

$$\Rightarrow \quad \frac{L_P}{L} = 1-\frac{1}{f} = 1-\frac{1}{2} \quad [\because \text{ Shape factor for diamond section} = 2]$$

$$\Rightarrow \quad L_P = 10\times\frac{1}{2} = 5 \text{ m}$$

24. (a)



For collapse of the overhang, BM at $B = M_P = \dfrac{w(a)^2}{2}$

For the collapse of the span AB at the stage of collapse of the overhangs,

$$M_P = \frac{w(10+2a)}{2}\times 5-\frac{w(5+a)^2}{2}$$

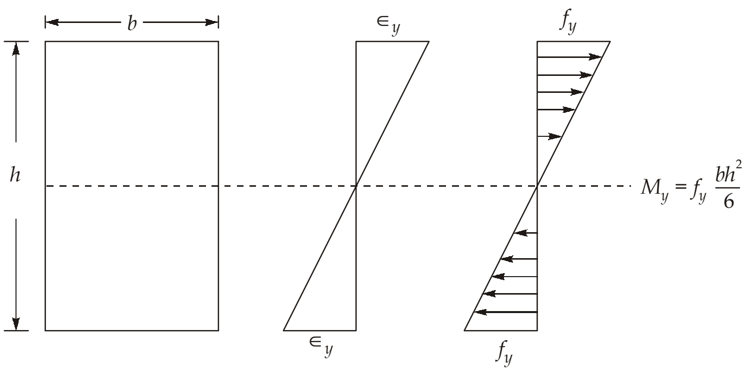
$$\therefore \quad \frac{wa^2}{2} = \frac{w(10+2a)}{2}\times 5-\frac{w(5+a)^2}{2}$$

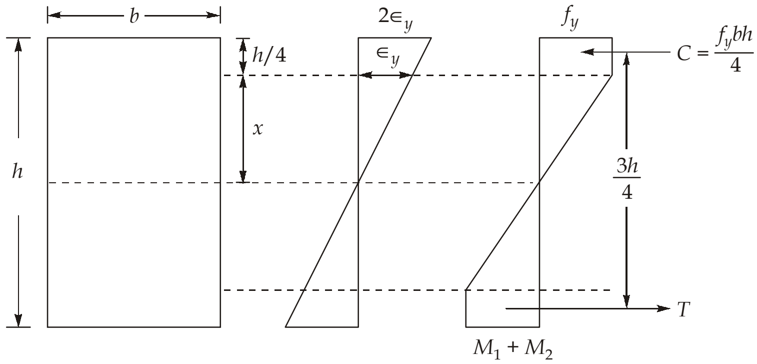
$$\Rightarrow \quad a^2 = 50 + 10a - 25 - a^2 - 10a$$

$$\Rightarrow \quad 2a^2 = 25$$

$$\Rightarrow \quad a = 3.54 \text{ m}$$

25. (b)





$$\frac{x}{\in_y} = \frac{h/2}{2\in_y}$$

$\Rightarrow$ $$x = \frac{h}{4}$$

$$M_y = f_y \cdot \frac{bh^2}{6}$$

$$M = M_1 + M_2 = \left[f_y \frac{b(h/2)^2}{6}\right] + \left[f_y \cdot \frac{bh}{4}\left(\frac{3h}{4}\right)\right]$$

$\Rightarrow$ $$M = \frac{11}{48} bh^2 f_y$$

$\therefore$ $$\frac{M}{M_y} = \frac{\frac{11}{48} bh^2 f_y}{f_y \cdot \frac{bh^2}{6}}$$

$\Rightarrow$ $$M = \frac{11}{8} M_y = 1.375 M_y$$

$\because$ $M_y = 50$ kNm

$\therefore$ $M = 1.375 \times 50 = 68.75$ kNm

27. (a)

The maximum bending moment,

$$M = \frac{wl^2}{2} = \frac{20 \times 3^2}{2} = 90 \text{ kN-m}$$

Section modulus of beam,

$$Z = \frac{I_{xx}}{(200/2)} = \frac{1696.6 \times 10^4}{100}$$

$$= 1696.6 \times 10^2 \text{ mm}^3$$

Bending stress, $$\sigma = \frac{M}{Z} = \frac{90 \times 10^6}{1696.6 \times 10^2}$$

$$= 530.47 \text{ N/mm}^2$$

Maximum shear force, $V = wl$

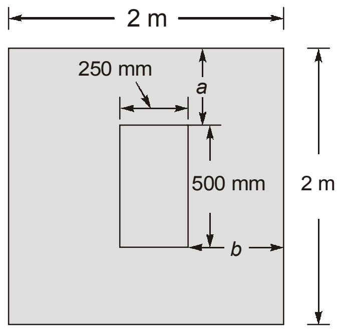
$$= 20 \times 3 = 60 \text{ kN}$$

$$\text{Shear stress, } \tau = \frac{V}{t_w \times d}$$

Thickness of web, $t_w = 5.4$ mm

$$\therefore \quad \tau = \frac{60 \times 10^3}{5.4 \times 200} = 55.56 \text{ N/mm}^2$$

28. (c)



500 mm × 250 mm column leaves projections as

$$a = \frac{2000 - 250}{2} = 875 \text{ mm}$$

$$b = \frac{2000 - 500}{2} = 750 \text{ mm}$$

Pressure on the underside of base slab

$$= \frac{2500}{4} = 625 \text{ kN/m}^2 = 0.625 \text{ N/mm}^2$$

$$\text{Thick of slab base, } t = \sqrt{2.5w\left(a^2 - 0.3b^2\right)\left(\frac{\gamma_{m0}}{f_y}\right)}$$

$$= \sqrt{2.5 \times 0.625\left(875^2 - 0.3 \times 750^2\right) \times \frac{1.1}{250}}$$

$$\simeq 64 \text{ mm}$$

29. (c)

A tension member in which reversal of direct stress occurs due to load other than wind or seismic loading

$$\frac{l_{max}}{r_{min}} \leq 180$$

Limiting slenderness ratio $= \dfrac{l_{max}}{r_{min}} \leq 180$

l_{max} = Limiting length of flat tie

r_{min} = Minimum radius of gyration of flat

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{bt^3/12}{bt}} = \frac{t}{\sqrt{12}} = \frac{20}{\sqrt{12}} = 5.7735 \text{ mm}$$

Limiting length of flat tie, l_{max} $= 180 \times 5.7735 = 1039.23$ mm

≈ 1040 mm

30. (b)

$$\text{Strength of bolt per pitch length} = \min\begin{cases} 2 \times 45.26 = 90.52 \text{ kN} \\ 2 \times 80 = 160 \text{ kN} \end{cases}$$

$= 90.52$ kN (∵ two bolts fall in one pitch)

Net strength of plate per pitch length = 230.25 kN (given)

$$\therefore \quad \text{The strength of joint per pitch length} = \min\begin{cases} 90.52 \\ 230.25 \end{cases} = 90.52 \text{ kN}$$

Now, efficiencyof the joint,

$$\eta = \frac{\text{Strength of joint per pitch length}}{\text{Strength of solid plate per pitch length}} \times 100$$

$$\therefore \quad \eta = \frac{90.52}{295.5} \times 100 = 30.63\%$$

■■■■