CLASS TEST

S.No. : 01_JP_CE_16072025

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Design of Steel Structures

CIVIL ENGINEERING

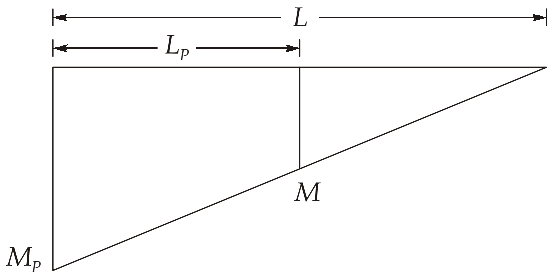
Date of Test : 16/07/2025

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (b)



We know,
$$f = \frac{M_p}{M} = \frac{L}{L - L_p}$$

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

$$\Rightarrow \quad L_p = \left(1 - \frac{1}{f}\right)L = \left(1 - \frac{1}{1.5}\right)L = \left(1 - \frac{2}{3}\right)L = \frac{L}{3}$$

2. (d)

Strength of joint = (Min. of design strength in shear, bearing and net strength of plate)
= 619.89 kN

Strength of solid plate = 727.27 kN

$$\text{So efficiency of joint} = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} = \frac{619.89}{727.27} = 0.8524 \text{ or } 85.24\%$$

3. (d)

Maximum value of slenderness ratio as IS 800 : 2007.

S.No.	Type of Member	(λ)
1.	Tension member prone to reversal of stresses due to the loads other then wind or earthquake.	180
2.	Member carrying compressive loads due to dead and live loads.	180
3.	Member carrying compressive force due to the combination of wind and earthquake only provided deformation of such members does not adversely effect the stress in any part of the structure.	250
4.	Compression flange of a beam restrained against lateral torsional buckling.	300
5.	A member normally acting as a tie in a roof truss or a bracing system not considered when subjected to possible reversal of stresses due to wind or earthquake forces.	350

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4. (b)

Moment of inertia of section,

$$I = \frac{90(120)^3}{12} - \frac{\pi}{64}(60)^4 = 12323827.49 \text{ mm}^4$$

Section modulus of the section,

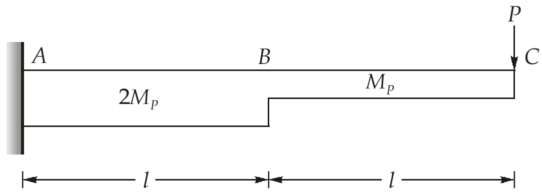
$$Z_e = \frac{I}{y_{\max}} = \frac{12323827.49}{60} = 205397.12 \text{ mm}^3$$

Plastic modulus of section,

$$Z_p = 2\left[90 \times 60 \times 30 - \frac{\pi}{2}(30)^2 \times \frac{4 \times 30}{3\pi}\right] = 288000 \text{ mm}^3$$

$$\text{Shape factor} = \frac{Z_p}{Z_e} = \frac{288000}{205397.12} = 1.4$$

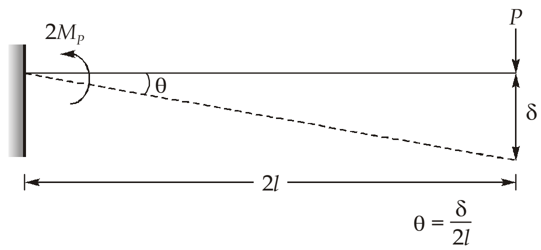
5. (c)



Degree of indeterminacy = 2 – 2 = 0

No. of plastic hinges required to form a mechanism, $n = D_s + 1 = 0 + 1 = 1$

(a) Let the plastic hinge be formed at A, then



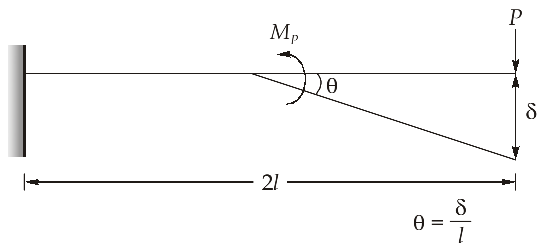
$$\text{Internal work} = \text{External work}$$

$$\Rightarrow \quad 2M_p\theta = P \times \delta$$

$$\Rightarrow \quad 2M_p\theta = P\,(2\theta l)$$

$$\Rightarrow \quad P = \frac{M_P}{l} \qquad \text{...(i)}$$

(b) Let the plastic hinge be formed at B, then



$$\text{Internal work} = \text{External work}$$
$$2_p\theta = P \times \delta$$
$$M_p\theta = P \times \theta l$$
$$M_p = Pl$$
$$P = \frac{M_P}{l} \quad \text{...(ii)}$$
$$\text{Collapse load} = \min\left(\frac{M_P}{l}, \frac{M_P}{l}\right) = \frac{M_P}{l}$$

6. (b)

When thin webs are used, it result in buckling due to shear. Hence intermediate transverse stiffeners are provided to improve buckling strength of web.

7. (d)

$$\text{Design shear strength, } V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t_w$$
$$h = 300 \text{ mm}$$
$$t_w = 7.5 \text{ mm}$$
For Fe410, $f_y = 250 \text{ N/mm}^2$

So, $V_d = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 300 \times 7.5 = 295.24 \text{ kN}$

8. (b)

Number of possible location of plastic hinges

$= 6\{A, B, C, D, E, F\}$

Degree of static indeterminacy $= 6 - 3 = 3$

$\therefore$ Number of possible independent mechanisms $= 6 - 3 = 3$

Two beam mechanisms and one sway mechanism.

9. (a)

$$\text{Axial load} = 600 \text{ kN}$$
Maximum shear, $V = \frac{2.5}{100} \times 600 \times 10^3 = 15000 \text{ N}$
$$\text{Transverse shear in each panel} = \frac{V}{N} = \frac{15000}{2} = 7500 \text{ N}$$
$$\text{Compressive force in lacing bars} = \frac{V}{N}\text{cosec}\,\theta$$
$$= 7500 \text{ cosec } 45° = 10606.6 \text{ N}$$
$$= 10.607 \text{ kN} \approx 10.61 \text{ kN}$$

10. (b)

For Fe410 grade steel, f_y=250 MPa

In case of single-V groove weld, incomplete penetration of weld takes place and therefore as per the specifications,

Throat thickness, $te = \frac{5}{8} t_{\min} = \frac{5}{8} \times 16 = 10 \text{ mm}$

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For site welding, $\gamma_{mw} = 1.5$

Factored load $= 1.5 \times 400 = 600$ kN

For butt weld, $P = l_w \times t_e \times \dfrac{f_y}{\gamma_{mw}}$

$\Rightarrow$ $600 \times 10^3 = l_w \times 10 \times \dfrac{250}{1.5}$

$\Rightarrow$ $l_w = 360$ mm

11. (c)

Gross area, $A_g = 2[90 + 60 - 8] \times 8$
$= 2272\ \text{mm}^2$

Gross strength of angle section,

$$T_{dg} = \frac{f_y A_g}{r_{m0}} = \frac{250 \times 2272}{1.1} \times 10^{-3} = 516.36 \text{ kN}$$

Design strength due to net rupture,

$$T_{dn} = \frac{0.9 f_u A_{nc}}{r_{m1}} + \frac{\beta f_y A_{g0}}{r_{m0}}$$

Diameter of bolt hole, $d_h = 16 + 2 = 18$ mm

Net area of connected leg, $A_{nc} = 2\left(90 - 18 - \dfrac{8}{2}\right) \times 8 = 1088\ \text{mm}^2$

Gross area of outstanding leg,

$$A_{go} = 2\left(60 - \frac{8}{2}\right) \times 8 = 896\ \text{mm}^2$$

Shear lag factor, $\beta = 1.4 - 0.076\left(\dfrac{w}{t}\right)\left(\dfrac{f_y}{f_u}\right)\left(\dfrac{b_s}{L_c}\right)$

Here, $w = 60$ mm
$t = 8$ mm
$b_s = 90 + 60 - 8 - 30 = 112$ mm
$L_c = 5 \times 40 = 200$ mm

$\Rightarrow$ $\beta = 1.4 - 0.076\left(\dfrac{60}{8}\right)\left(\dfrac{250}{415}\right)\left(\dfrac{112}{200}\right) = 1.2077 \approx 1.208 > 0.7$

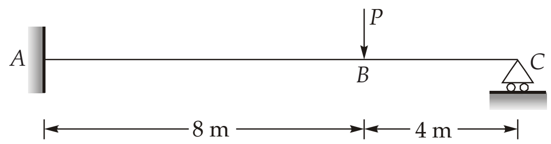
and $< \dfrac{0.9 f_u \gamma_{m0}}{\gamma_{m1} \times f_y} = 1.299$

$\therefore$ $\beta = 1.208$

$\therefore$ $T_{dn} = \left(\dfrac{0.9 \times 410 \times 1088}{1.25} + \dfrac{1.208 \times 250 \times 896}{1.1}\right) \times 10^{-3} = 567.17$ kN

Design strength $= \min\{Td_g, Td_n\}$
$= \min\{516.36, 567.17\}$
$= 516.36$ kN

12. (d)

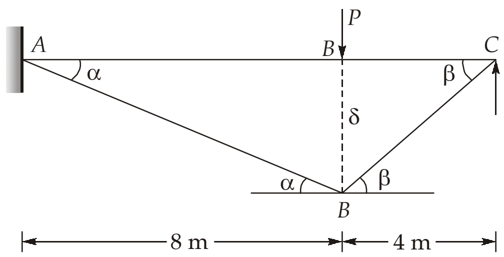


$$D_s = 2 + 1 - 2 = 1$$

No. of hinges required for mechanism formation

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

Mechanism is as shown below (plastic hinges at A and B)



From diagram
$8\alpha = 4\beta$
$2\alpha = \beta$

$$\text{External work} = \text{Internal work}$$

$$\Rightarrow \quad P \times \delta = M_P\alpha + M_P\alpha + M_P\beta$$

$$\Rightarrow \quad P \times \delta = 2M_P\alpha + M_P\beta$$

$$\Rightarrow \quad P \times \delta = 2M_P\beta$$

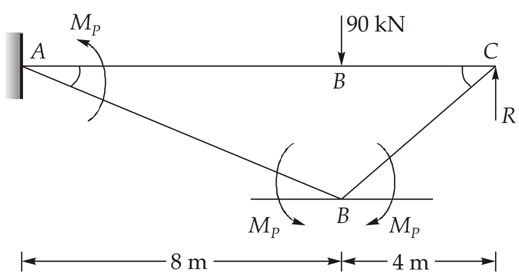
$$\Rightarrow \quad P \times \delta = 2M_P \times \frac{\delta}{4}$$

$$\Rightarrow \quad P = \frac{M_P}{2}$$

$$\Rightarrow \quad P = \frac{180}{2} = 90 \text{ kN}$$

$\therefore$ Collapse load $= 90$ kN

Now, $\Sigma M_A = 0$



$$\therefore \quad M_P - M_P + M_P + R \times 12 = 90 \times 8$$

$$\Rightarrow \quad 180 + R \times 12 = 720$$

$$\Rightarrow \quad R = 45 \text{ kN}$$

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13. (c)

Given, factored load, $P_u = 2000$ kN

Now, bearing pressure on the plate, $w = \dfrac{P_u}{L \times B} = \dfrac{2000 \times 10^3}{560 \times 410} = 8.71 \text{ N/mm}^2$

$\therefore$ Thickness of base plate, $t = \sqrt{\dfrac{2.5w(a^2 - 0.3b^2)}{(f_y/\gamma_{m0})}}$

where, a = larger projection $= \dfrac{560 - 400}{2} = 80$ mm

b = smaller projection $= \dfrac{410 - 250}{2} = 80$ mm

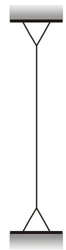
$$\therefore \quad t = \sqrt{\frac{2.5 \times 8.71(80^2 - 0.3 \times 80^2)}{\left(\frac{250}{1.1}\right)}} = 20.718 \text{ mm} \not< t_f (= 12.7 \text{ mm})$$

So, provide, $t = 22$ mm

14. (b)

Non dimensional effective slenderness ratio is given by

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}}$$



$$f_{cc} = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2}$$

$kL = 1 \times 7000 \text{ mm} = 7000$ mm

($\because$ Column is hinged at both the ends)

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{13533 \times 10^4}{28000}}$$

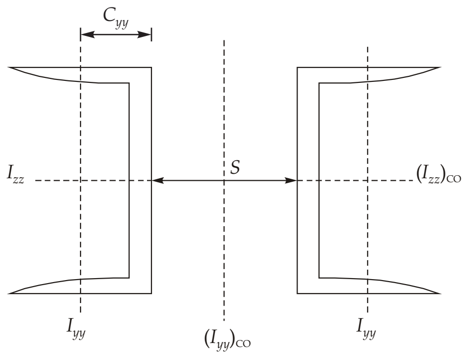
$\Rightarrow \quad r = 69.521$ mm

$$f_{cc} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{7000}{69.521}\right)^2} = 194.699 \simeq 194.7 \text{ N/mm}^2$$

$$\therefore \quad \lambda = \sqrt{\frac{250}{194.7}} = 1.133 \simeq 1.13$$

15. (c)

Channels are placed back to back such that $(I_{zz})_{combined} = (I_{yy})_{combined}$.

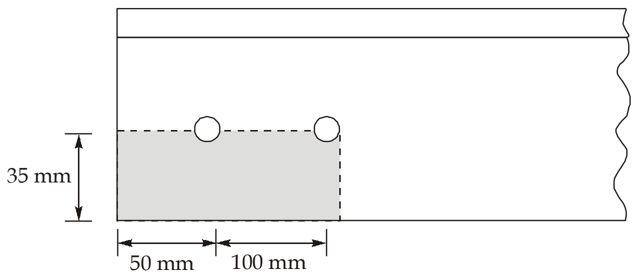


$$\therefore \qquad 2I_{zz} = 2\left[I_{yy} + A\left(\frac{S}{2} + C_{yy}\right)^2\right]$$

$$\Rightarrow \qquad 2 \times 6321 \times 10^4 = 2\left[310 \times 10^4 + 4564 \times \left(\frac{S}{2} + 23.6\right)^2\right]$$

$$\Rightarrow \qquad S = 182.325 \text{ mm} \simeq 182.3 \text{ mm}$$

16. (a)



The shaded area will shear out.

For 20 mm dia. bolt, of bolt hole = 22 mm

$$A_{vg} = (100 + 50) \times 8 = 1200 \text{ mm}^2$$

$$A_{vn} = \left[(100 + 50) - \left(2 - \frac{1}{2}\right) \times 22\right] \times 8 = 936 \text{ mm}^2$$

$$A_{tg} = 35 \times 8 = 280 \text{ mm}^2$$

$$A_{tn} = \left(35 - \frac{22}{2}\right) \times 8 = 192 \text{ mm}^2$$

$$T_{db_1} = \frac{A_{vg}.f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9A_{tn}f_u}{\gamma_{m1}}$$

$$\Rightarrow \qquad T_{db_1} = \left[\frac{1200 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 192 \times 410}{1.25}\right] \times 10^{-3} \text{ kN}$$

$$T_{db_1} = 214.14 \text{ kN}$$

$$T_{db_2} = \frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg}f_y}{\sqrt{3}\gamma_{m0}}$$

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$$\Rightarrow \quad T_{db_2} = \left[\frac{0.9 \times 936 \times 410}{\sqrt{3} \times 1.25} + \frac{280 \times 250}{1.1}\right] \times 10^{-3}\ \text{kN}$$

$$T_{db_2} = 223.16\ \text{kN}$$

So block shear strength is minimum of T_{bd_1} and T_{bd_2} = 214.14 kN

17. (c)

$\because$ Angle is jointed with gusset plate

$\therefore$ Bolt is in single shear (threaded portion)

Strength of 16 mm bolt in shear

$$= \frac{1 \times 400}{\sqrt{3} \times 1.25}\left[0.78 \times \frac{\pi}{4} \times 16^2\right]\ \text{N}$$

$$\Rightarrow \quad V_{dsb} = 28.959\ \text{kN}$$

Strength of 16 mm bolt in bearing

$$= \frac{2.5 k_b \times d \times t \times f_{ub}}{1.25}$$

Given, $\quad k_b = 0.65$

$$\therefore \quad V_{dpb} = \frac{2.5 k_b \times d \times t \times f_{ub}}{1.25}$$

$$= \frac{2.5 \times 0.65 \times 16 \times 8 \times 40}{1.25}\ \text{N} = 6650\ \text{N}$$

$$\Rightarrow \quad V_{dpb} = 66.56\ \text{kN}$$

$$\therefore \quad \text{Bolt strength} = \min(V_{dsb}, V_{dpb})$$

$$= 28.959\ \text{kN}$$

No. of bolts required for the connection not to fail

$$= \frac{\text{Factored load}}{\text{Bolt strength}} = \frac{1.5 \times 60}{28.959} = 3.108 \simeq 4\ \text{bolts}$$

18. (c)

1. Shear force applied = 250 kN

Design shear force = 1.5 × 250 kN = 375 kN

Design shear strength of section $V_d = \dfrac{f_y}{\sqrt{3}\gamma_{mo}} h t_w$

$$= \frac{250}{\sqrt{3} \times 1.1} \times 350 \times 8\ \text{N} = 367.405\ \text{kN} < 375\ \text{kN}$$

$\therefore$ The beam is unsafe in shear.

2. Bearing strength, $F_w = A_e \dfrac{f_y}{\gamma_{mw}}$

$$= (b + n_1) t_w \frac{f_y}{\gamma_{m0}} = \left\{75 + 2.5\left(t_f + R\right)\right\} \times 8 \times \frac{250}{1.1}\ \text{N}$$

$$= (75 + 2.5(11.4 + 12)) \times 8 \times \frac{250}{1.1}\ \text{N} = 242.727\ \text{kN}$$

$\therefore$ The beam will cripple.

19. (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.

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

$$f_b = 80 \times 10^3 \text{ kN/m}^2 = 80 \text{ MPa}$$

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

$$q = 65.66 \text{ MPa}$$

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

$$\therefore \quad f_e \leq \frac{f_u}{\sqrt{3}\gamma_{mw}} \qquad \{\text{for site conditions } \gamma_{mw} = 1.5\}$$

$$\Rightarrow \quad f_u \geq 483.5 \text{ MPa}$$

20. (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,

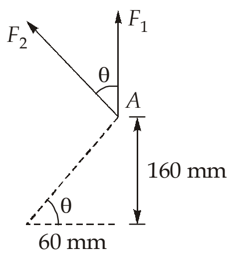
$$F_2 = \frac{Per_A}{\Sigma r_i^2}$$

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

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

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

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, $P_1 = 145.96 \text{ kN}$

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

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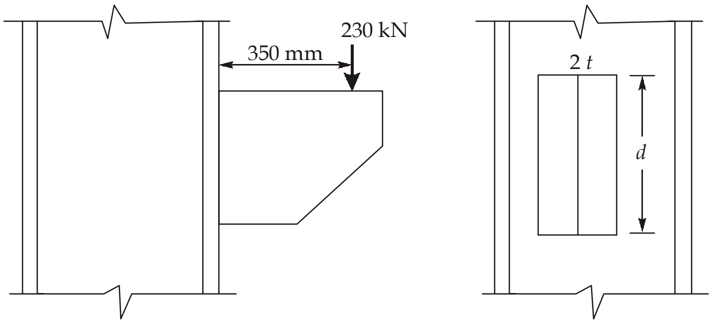
21. (d)



Direct shear stress, $q = \dfrac{P}{2t \times d}$

where t is throat thickness = 0.7 × size of weld

$$= \frac{230 \times 10^3}{2 \times 0.7 \times 10 \times 400} = 41.07 \text{ N/mm}^2$$

Bending stress, $f_b = \dfrac{M}{I} \times y$

$$= \frac{P.e}{\dfrac{2t \cdot d^3}{12}} \times \frac{d}{2}$$

$$= \frac{230 \times 10^3 \times 350 \times 12 \times 400}{2 \times 0.7 \times 10 \times 400^3 \times 2}$$

$$= 215.63 \text{ N/mm}^2$$

Resultant stress, $f_r = \sqrt{f_b^2 + 3q^2}$

$$= \sqrt{(215.63)^2 + 3 \times (41.07)^2} = 227.06 \text{ N/mm}^2 \simeq 227 \text{ MPa}$$

22. (c)

Refer clause 7.5.1.2 of IS 800 : 2007,

Equivalent slenderness ratio,

$$\lambda_e = \sqrt{k_1 + k_2 \times \lambda^2{}_{vv} + k_3 \times \lambda_\phi^2}$$

$$\lambda_{vv} = \frac{l/r_{vv}}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}}; \qquad \lambda_\phi = \frac{\dfrac{b_1 + b_2}{2t}}{\varepsilon\sqrt{\dfrac{\pi^2 E}{250}}} \qquad \varepsilon = \sqrt{\frac{250}{f_y}}$$

For Fe 410 grade, $f_y = 250 \text{ N/mm}^2$;

$E = 2 \times 10^5 \text{ N/mm}^2$

$\varepsilon = 1$

$$\lambda_{vv} = \frac{(2750/17.5)}{\sqrt{\dfrac{\pi^2 \times 2 \times 10^5}{250}}} = 1.77$$

$$\lambda_\phi = \frac{(90+90)/2\times 8}{\sqrt{\frac{\pi^2\times 2\times 10^5}{250}}} = 0.127$$

∴ When the ends of the member are fixed, and two bolts are provided for the connection at each end,

$$k_1 = 0.2;\ k_2 = 0.35;\ k_3 = 20$$

∴ $$\lambda_e = \sqrt{0.2 + 0.35\times(1.77)^2 + 20\times(0.126)^2} = 1.27$$

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

$= 148.12$ kNm ≤ 156.44 kNm (OK)

So design bending strength = 148.12 kNm.

24. (a)

Weight of galvanised iron sheets = 140 × 2 = 280 N/m

Dead load of purlins = 100 N/m

So, Total dead load = 280 + 100 = 380 N/m

This dead load acts vertically downward.

The component of dead load normal to roof = 380 cos30°

= 329.1 N/m

Wind pressure = $0.6\ V_z^2 = 0.6 \times 45^2$

= 1215 N/m^2

Wind load acting normal to the roof= 1215 × 2 = 2430 N/m

Total load on purlin normal to roof = 329.1 + 2430 = 2759.1 N/m

∴ Total factored load = 1.5 × 2759.1 = 4138.65 N/m

= 4.14 kN/m

25. (c)

Depth of web: $d = h - 2\,(t_f + R_1)$

$= 350 - 2\,(11.2 + 16)$

$= 295.2$ mm

Slenderness ratio, $$\left(\frac{kL}{r}\right) = 2.5\frac{d}{t_w} = 2.5\times\frac{29.52}{7.4} = 99.73$$

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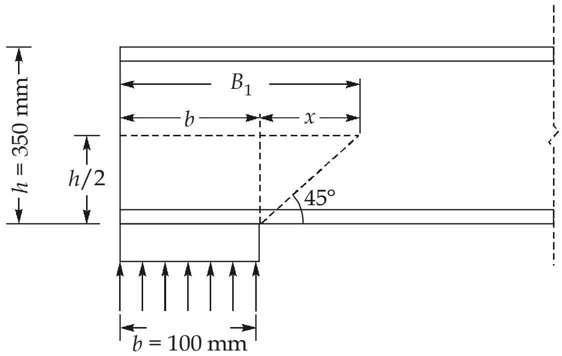
From table given: $$f_{cd} = 121 + \frac{107-121}{100-90}(99.73-90) = 107.38\ \text{N/mm}^2$$



$$B_1 = b + x = b + \frac{h}{2} = 100 + \frac{350}{2} = 275\ \text{mm}$$

∴ Web buckling strength, $F_{wb} = B_1 \times t_w \times f_{cd}$
$= 275 \times 7.4 \times 107.38 \times 10^{-3}$ kN
$= 218.52$ kN

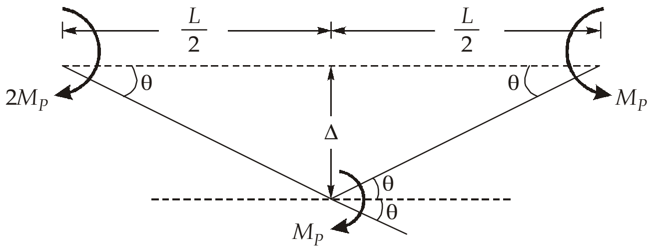
26. (b)

No. of independent mechanisms = Possible number of plastic hinges – Degree of redundancy
$= 4 - 2 = 2$

∵ Degree of redundancy = 2

∴ Number of plastic hinges required for collapse = 2 + 1 = 3

<u>**Case - I :**</u> Two plastic hinge at supports, and one plastic hinge at the point where cross – section changes. The plastic hinge will form at B in the limb BC and its value will be M_p.



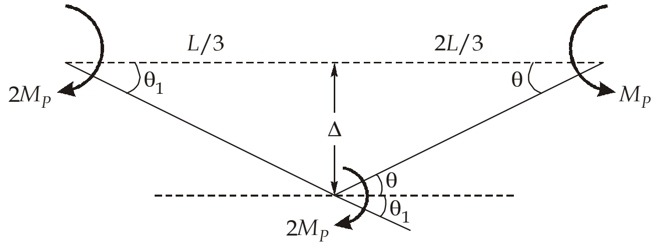
External work done = Internal work done

$$\Rightarrow \quad W \times \frac{L}{3}\theta = 2\,M_p\theta + M_p(\theta + \theta) + M_p\theta$$

$$\Rightarrow \quad W \times \frac{L}{3}\theta = 5\,M_p\theta$$

$$\Rightarrow \quad W = 15\frac{M_P}{L}$$

Case - II : Two plastic hinges at the supports and one below the concentrated load.



$$\Delta = \frac{L}{3}\theta_1 = \frac{2}{3}L\theta \Rightarrow \theta_1 = 2\theta$$

External work done = Internal work done

$$\Rightarrow \quad W \times \frac{L}{3}\theta_1 = 2M_P\theta_1 + 2M_P(\theta + \theta_1) + M_P\theta$$

$$\Rightarrow \quad \frac{2}{3}WL\theta = 2M_P \times (2\theta) + 2\,M_P\,(\theta + 2\theta) + M_P\theta$$

$$\Rightarrow \quad \frac{2}{3}WL\theta = 11\,M_P\theta$$

$$\Rightarrow \quad W = 16.5\frac{M_P}{L}$$

$$\text{So, collapse load} = 15\frac{M_P}{L}$$

27. (d)

Span, L = 24 m

Total factored load, w = 52 kN/m

$$\text{So, maximum moment} = \frac{wL^2}{8} = \frac{52 \times (24)^2}{8} = 3744 \text{ kNm}$$

Now, if stiffeners are to be avoided then

$$k = \frac{d}{t_w} \le 67$$

So economic depth of web,

$$d = \sqrt[3]{\frac{Mk}{f_y}} = \left(\frac{3744 \times 10^6 \times 67}{250}\right)^{1/3}$$

$$= 1001.13 \text{ mm} \simeq 1000 \text{ mm (say)}$$

So, provide d = 1000 mm.

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28. (d)

The principal moment of inertia may be calculated by

$$I_{uu}/I_{vv} = \frac{I_{xx}+I_{yy}}{2} \pm \sqrt{\left(\frac{I_{xx}-I_{yy}}{2}\right)^2 + I_{xy}^2}$$

$$= \frac{2\times10^7 + 0.97\times10^7}{2} \pm \sqrt{\left(\frac{2\times10^7 - 0.97\times10^7}{2}\right)^2 + \left(-0.83\times10^7\right)^2}$$

$$= 1.485\times10^7 \pm \sqrt{0.265225\times10^{14} + 0.6889\times10^{14}}$$

$$= 1.485 \times 10^7 \pm 0.977 \times 10^7$$

$$I_{uu} = (1.485 + 0.977) \times 10^7 = 2.462 \times 10^7 \text{ mm}^4$$

$$I_{vv} = (1.485 - 0.977) \times 10^7 = 0.508 \times 10^7 \text{ mm}^4$$

$$\therefore \quad I_{min} = I_{vv} = 0.508 \times 10^7 \text{ mm}^4$$

$$\therefore \quad r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{0.508\times10^7}{5025}} = 31.8 \text{ mm}$$

Effective length of the strut or l_{eff} = 4.5 m

$$\therefore \quad \lambda = \frac{l_{eff}}{r_{min}} = \frac{4.5\times10^3}{31.8} = 141.51$$

$$\Rightarrow \quad \sigma_{ac} = 51 + \frac{45-51}{150-140}\times(141.51-140)$$

$$= 50.094 \text{ MPa}$$

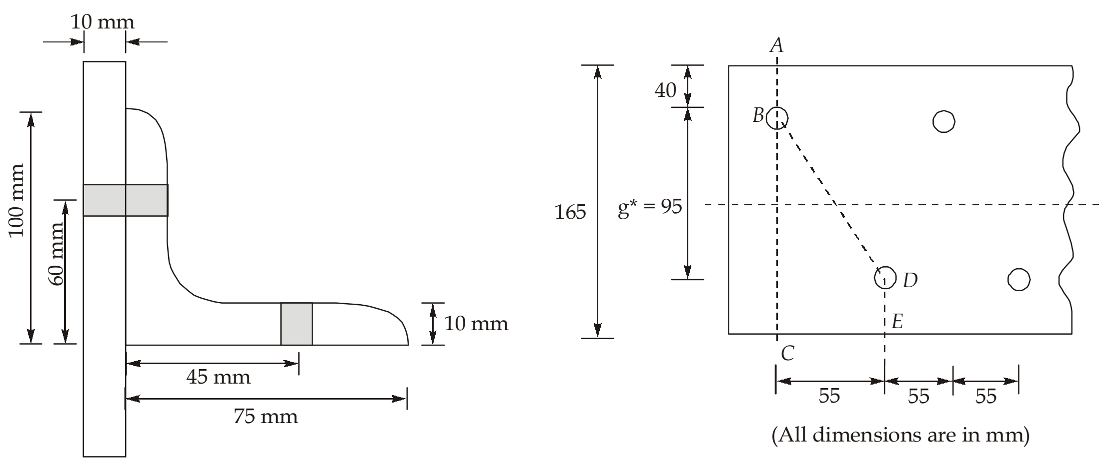
∴ Load on the member

$$= \sigma_{ac} A$$

$$= 50.094 \times 5025 \times 10^{-3}$$

$$= 251.72 \text{ kN}$$

29. (a)



(All dimensions are in mm)

Since both legs are connected and so outstanding leg is rotated and resulting section is shown in above figure.

$$g^* = g_1 + g_2 - t = 60 + 45 - 10 = 95 \text{ mm}$$

Width of equivalent plate $= 100 + 75 - 10 = 165$ mm

Net area along ABC, $A_1 = (B - nd_o)t$

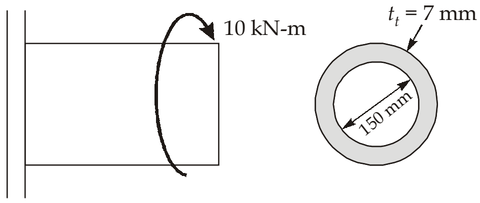
$$\Rightarrow \quad A_1 = (165 - 1 \times 20) \times 10 = 1450 \text{ mm}^2$$

$$\text{Net area along ABDE, } A_2 = \left[B - nd_0 + \frac{\Sigma p^2}{4g}\right]t$$

$$\Rightarrow \quad A_2 = \left[165 - 2 \times 20 + \frac{1 \times 55^2}{4 \times 95}\right] \times 10 = 1329.60 \text{ mm}^2$$

Minimum of A_1 and A_2 is net area $= 1329.60$ mm^2.

30. (b)



Throat thickness, $t = 0.7 \cdot s$ where s is size of weld.

$= 0.7 \times 10 = 7$ mm

Now, maximum shear stress in weld, $f_s = \dfrac{T}{Z_p}$

here Z_p = Polar modulus of weld area

$$= A \times r$$

$$= \pi d \cdot t \cdot \times \frac{d}{2}$$

$$= \pi d^2 \times \frac{t}{2}$$

So,

$$f_s = \frac{10 \times 10^6}{\pi \times 150^2 \times \frac{7}{2}} = 40.4 \text{ N/mm}^2$$

■■■■

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