CLASS TEST

SL.: 03DLSKCE_JKL_17102025

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STRUCTURE ANALYSIS

CIVIL ENGINEERING

Date of Test : 17/10/2025

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (d)

At joint G, two collinear forces and one non-collinear member meet. Therefore, force in non-collinear member i.e. in AG force will be zero. Similarly, force in members BH and DI will be zero. At joint K two non-collinear members meet and there is no external loading. Therefore, force in member KF and KJ will be zero. Now, at joint F, there are 2 collinear forces. So, force in member EF will be zero. Similarly, force in member DE will be zero.

2. (b)

3. (c)

$$H = \frac{W_1}{\pi}\sin^2\theta_1 + \frac{W_2}{\pi}\sin^2\theta_2$$

$$= \frac{80}{\pi}\sin^2(60°) + \frac{120}{\pi}\sin^2 30° = 28.65\text{ kN}$$

4. (c)

Number of members, $m = 16$

Number of unknown reactions,

$r_e = 12$

Number of joints, $j = 15$

Number of reactions released, $r_r = \Sigma(m' - 1)$

where m is number of members meeting at hybrid joint

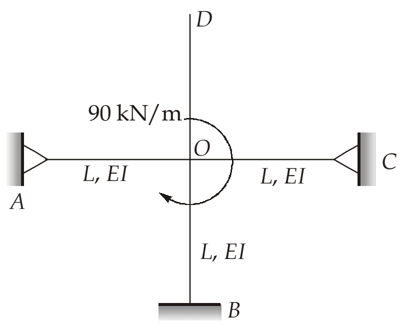
$= 2 + 1 + 1 + 1 = 5$

So, degree of static indeterminacy,

$$D_s = 3m + r_e - 3j - r_r$$
$$= 3 \times 16 + 12 - 3 \times 15 - 5 = 10$$

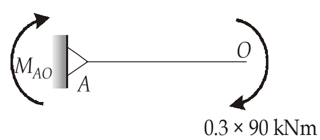
5. (a)

6. (d)



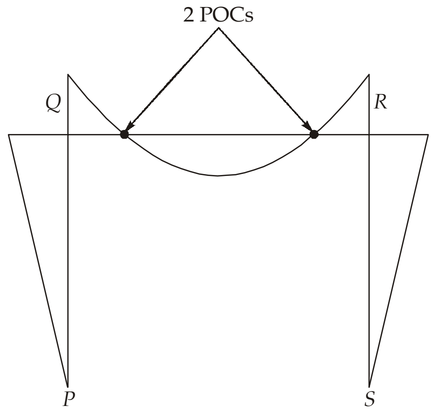
$$(DE)_{OA} = \frac{\frac{3EI}{L}}{\frac{3EI}{L} + \frac{3EI}{L} + \frac{4EI}{L}} = \frac{3}{10} = 0.3$$

$$M_{AO} = 0\text{ kNm}$$



7. (b)

BMD



8. (c)

Slope deflection equations can be written for

M_{AB}, M_{BA}, M_{BC}, M_{CB}, M_{CD}, M_{DC}, M_{CE} and M_{EC}

9. (b)

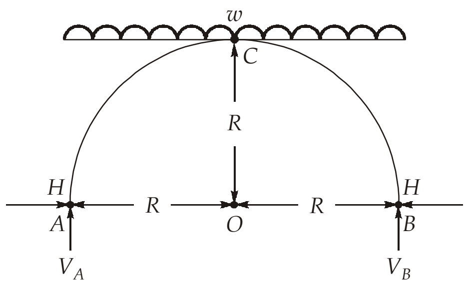
$$V_A = V_B = \frac{w \times 2R}{2} = wR \qquad [\because \text{ Due to symmetry}]$$

$$\Sigma M_C = 0 \qquad \text{(from left)}$$

$$\Rightarrow \quad V_A \times R - H \times R - w \times R \times \frac{R}{2} = 0$$

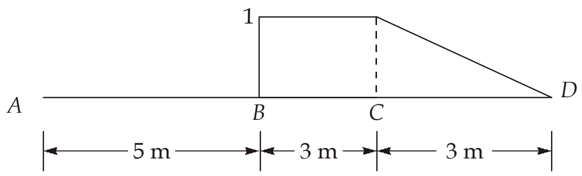
$$\Rightarrow \quad wR \times R - \frac{wR^2}{2} = HR$$

$$\Rightarrow \quad H = \frac{wR}{2}$$



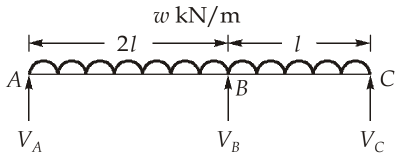
10. (d)

ILD for SF at B:



11. (d)

In figure, at simply supported and A and B, bending moment are zero,



i.e. $\quad M_A = M_C = 0$

Applying three moments's equation for span AB and BC,

$$M_A(2l) + 2M_b(2l + l) + M_C l = \frac{6a_1\bar{x}_1}{2l} + \frac{6a_2\bar{x}_2}{l} \qquad \text{...(i)}$$

where, $$\frac{6a_1\bar{x}_1}{2l} = 6\times\frac{2}{3}\times 2l\times\frac{w(2l)^2}{8}\times\frac{l}{2l} = 2wl^3$$

Similarly, $$\frac{6a_2\bar{x}_2}{l} = 6\times\frac{2}{3}\times l\times\frac{wl^2}{8}\times\frac{l}{2\times l} = \frac{wl^3}{4}$$

Putting values in (i), we get

$$0\times 2l + 6\,M_b l + 0\times l = 2wl^3 + \frac{wl^3}{4}$$

$$\Rightarrow \quad M_b = \frac{3}{8}wl^2$$

Now, $w = 8$ kN/m and $l = 3$ m

So, $M_b = 27$ kN-m

12. **(c)**

Let the span of cable be 'l' m

Then, dip of cable, $h = \frac{l}{12}$ m

Length of cable, $$L = l\left(1+\frac{8}{3}\frac{h^2}{l^2}\right) = l\left(1+\frac{8}{3}\times\frac{l^2}{144\times l^2}\right) = \frac{55}{54}l$$

Let the cross-section area of cable be A mm^2

Then, weight of cable, $$W = \frac{55}{54}l\times\frac{A}{10^6}\times 78\times 10^3 = 0.079\,Al \text{ N}$$

Now, each vertical reaction, $V = \frac{W}{2}$

Horizontal reaction, $$H = \frac{Wl}{8h} = \frac{W}{8}\times\frac{l\times 12}{l} = \frac{3}{2}W$$

So, maximum tension, $$T_{max} = \sqrt{V^2+H^2} = \sqrt{\left(\frac{W}{2}\right)^2+\left(\frac{3}{2}W\right)^2} = 1.58\text{ W} = 1.58\times 0.079\,Al \text{ N}$$

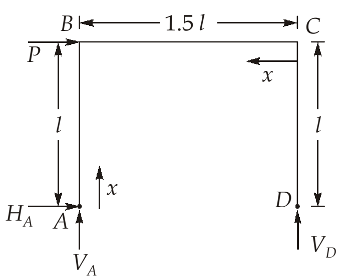
So, maximum stress $$= \frac{T_{max}}{A} = \frac{1.58\times 0.079\,Al\,N}{A\text{ mm}^2} = 1.58\times 0.079l \text{ N/mm}^2$$

But, permissible stress in cable = 175 N/mm^2

So, $1.58\times 0.079l = 175$

$\Rightarrow \quad l = 1402$ m

13. **(b)**



Let, the vertical reactions developed at A and D be V_A and V_D respectively and H_A be the horizontal reaction developed at A.

Now, $\Sigma F_x = 0$

$\Rightarrow \quad P + H_A = 0$

$\Rightarrow \quad H_A = -P$ i.e. $P(\leftarrow)$ ...(i)

$\Sigma M_D = 0$

$\Rightarrow \quad V_A \times 1.5l + P \times l = 0$

$\Rightarrow \quad V_A = \dfrac{-P}{1.5}$ i.e. $P(\downarrow)$ ...(ii)

$\Sigma F_y = 0$

$\Rightarrow \quad V_A + V_D = 0$

$\Rightarrow \quad V_D = -V_A = -\dfrac{(-P)}{1.5} = \dfrac{P}{1.5}\ (\uparrow)$ ...(iii)

Using unit load method,

Member	AB	BC	CD
Origin	A	B	C
M	Px	$\dfrac{P \times x}{1.5}$	0
m	x	$\dfrac{x}{1.5}$	0
Limits	$0-l$	$0-1.5l$	$0-l$

$$\Delta_B = \int \frac{Mmdx}{EI} = \int_0^l \frac{(Px) \times xdx}{EI} + \int_0^{1.5l} \frac{\left(P \times \dfrac{x}{1.5}\right) \times \dfrac{x}{1.5l}dx}{EI}$$

$$= \frac{P}{EI}\left[\frac{x^3}{3}\right]_0^L + \frac{P}{1.5^2 EI}\left[\frac{x^3}{3}\right]_0^{1.5l} = \frac{Pl^3}{3EI} + \frac{Pl^3}{2EI} = \frac{5}{6}\frac{Pl^3}{EI}$$

$\therefore \quad K = 2.5$

14. (d)

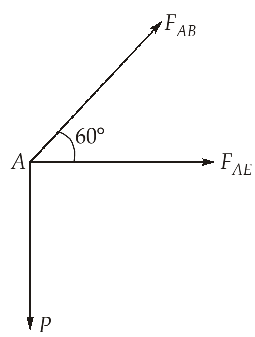
$\Sigma F_y = 0 \Rightarrow \quad R_A + R_E = P$

$\Sigma M_A = 0 \Rightarrow \quad R_E \times 4 = P(4 + 4)$

$\Rightarrow \quad R_E = 2P(\uparrow)$

$\therefore \quad R_A = P(\downarrow)$

Joint A:



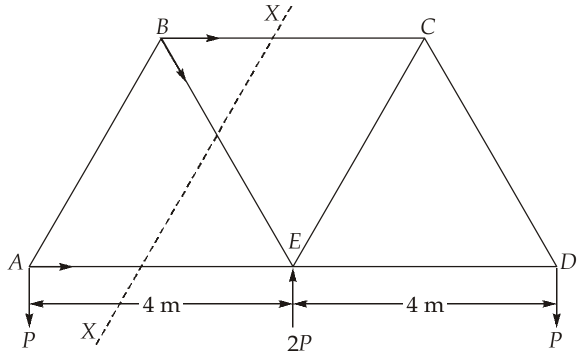
$\Sigma F_y = 0 \Rightarrow \quad F_{AB} \sin 60° = P$

$\Rightarrow \quad F_{AB} = \dfrac{2P}{\sqrt{3}}$(Tension)

$\Sigma F_x = 0 \Rightarrow \quad F_{AB} \cos 60° + F_{AE} = 0$

$\Rightarrow \quad F_{AE} = -\dfrac{F_{AB}}{2} = -\dfrac{P}{\sqrt{3}}$ or $\dfrac{P}{\sqrt{3}}$(Compression)

Applying method of section (X-X),



$$\Sigma M_E = 0 \text{(Left portion)}$$

$$\Rightarrow \quad F_{BC} \times 4 \sin 60^\circ = P \times 4$$

$$\Rightarrow \quad F_{BC} = \frac{2P}{\sqrt{3}} = \frac{2P}{\sqrt{3}} \text{(Tension)}$$

In vertical direction (left portion), $\Sigma F_y = 0$

$$\Rightarrow \quad F_{BE} \times \sin 60^\circ + P = 0$$

$$\Rightarrow \quad F_{BE} = -\frac{2P}{\sqrt{3}} = \frac{2P}{\sqrt{3}} \text{(Compression)}$$

Since the structure and loading constitute a symmetric system,

$$F_{AB} = F_{CD}$$
$$F_{AE} = F_{ED}$$
$$F_{BE} = F_{CE}$$
$$F_{BC} = F_{BC}$$

Maximum tensile force obtained = $\frac{2P}{\sqrt{3}}$

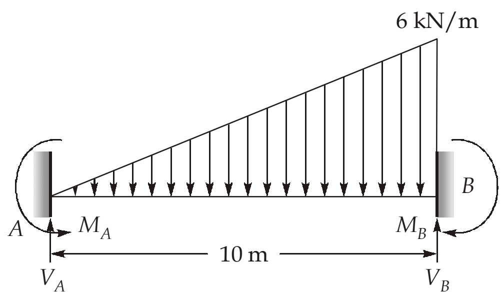
Maximum compressive force obtained = $\frac{2P}{\sqrt{3}}$

$$\therefore \quad \text{minimum } \{6 \text{ kN}, 8 \text{ kN}\} = \frac{2P}{\sqrt{3}}$$

$$\Rightarrow \quad \frac{2P}{\sqrt{3}} = 6 \text{ kN}$$

$$\Rightarrow \quad P = \frac{6\sqrt{3}}{2} = 3\sqrt{3} \text{ kN}$$

15. (b)



Let V_A and V_B be the vertical reactions developed at A and B and M_A and M_B be the fixed end moments at A and B.

Now,
$$M_A = \frac{-wl^2}{30} = \frac{-6 \times 10^2}{30}$$
$$= -20 \text{ kN-m} \quad \text{...(i)}$$
$$= 20 \text{ kN-m} \; (\circlearrowleft)$$
$$M_B = \frac{wl^2}{20} = 30 \text{ kN-m} \; (\circlearrowright) \quad \text{...(ii)}$$

Now,
$$\Sigma M_B = 0$$
$$\Rightarrow V_A \times 10 - M_A - \frac{1}{2} \times 10 \times 6 \times \frac{1}{3} \times 10 + M_B = 0$$
$$\Rightarrow V_A \times 10 - 20 - 100 + 30 = 0$$
$$\Rightarrow V_A = 9 \text{ kN}$$

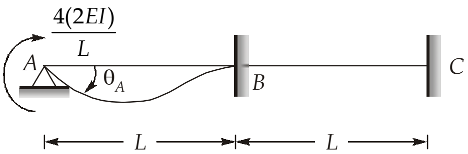
16. (c)

Ist Column:

Give unit displacement in direction of coordinate-1,

$\therefore$ $\theta_A = 1$

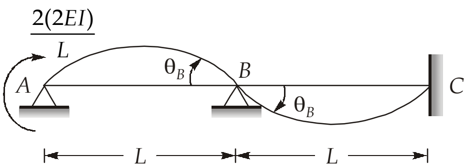
and $\theta_B = 0$



$$K_{11} = \frac{8EI}{L}$$
$$K_{21} = \frac{4EI}{L} \quad [\because \text{ carry over factor is 0.5}]$$

IInd Column:

Give unit displacement in direction of coordinate-2,



$\therefore$ $\theta_B = 1$

and $\theta_A = 0$

$$K_{12} = \frac{4EI}{L}$$
$$K_{22} = \frac{4(2EI)}{L} + \frac{4EI}{L} = \frac{12EI}{L}$$

Stiffness matrix, $[s] = \frac{EI}{L}\begin{bmatrix} 8 & 4 \\ 4 & 12 \end{bmatrix}$

17. (a)

Stiffness of beam, $K_b = \frac{192EI}{L^3}$

Now, $EI = 2\times10^4\times10^6\times\frac{N}{m^2}\times10^{-4}\text{m}^4 = 2\times10^6$ N-m^2

$\therefore$ $K_b = \frac{192\times2\times10^6}{5^3} = 3072\times10^3$ N/m

Now, as the spring and beam system is in series,

So, $\frac{1}{k_{eq}} = \frac{1}{k_b}+\frac{1}{k_s}$ where k_s is spring constant

$= \frac{1}{3072\times10^3}+\frac{1}{20\times10^3}$

$\Rightarrow$ $k_{\text{eq}} = 19.87\times10^3$ N/m

Now, Time period, $T = \frac{2\pi}{\omega}$

where, $\omega = \sqrt{\frac{k_{eq}}{m}} = \sqrt{\frac{19.87\times10^3\times9.81}{25\times10^3}} = 2.79$ rad/s

So, $T = \frac{2\pi}{2.79} = 2.25$ sec

18. (a)

Fixed end moments,

$$M_{FAB} = \frac{-WL}{8} = \frac{-40\times8}{8} = -40 \text{ kN-m}$$

$$M_{FBA} = \frac{WL}{8} = \frac{40\times8}{8} = 40 \text{ kN-m}$$

$$M_{FBC} = \frac{M_o}{4} = \frac{80}{4} = 20 \text{ kN-m}$$

$$M_{FCB} = \frac{M_o}{4} = \frac{80}{4} = 20 \text{ kN-m}$$

Now, $M_{AB} = M_{FAB}+\frac{2EI}{L}(2\theta_A+\theta_B)$

$= -40+\frac{2EI}{8}(2\theta_A+\theta_B)$

$= -40 + 0.5EI\,\theta_A + 0.25\,EI\,\theta_B$ ...(i)

Similarly, $M_{BA} = 40+\frac{2EI}{8}(2\theta_B+\theta_A)$

$= 40 + 0.25EI\,\theta_A + 0.5\,EI\,\theta_B$ ...(ii)

$$M_{BC} = 20 + \frac{2EI}{4}(2\theta_B) \qquad [\because C \text{ is fixed}]$$

$$= 20 + EI\,\theta_B \qquad \text{...(iii)}$$

Now, $M_{AB} = 0$ [$\because$ end A is hinged]

Also, $\Sigma M_B = 0 \Rightarrow M_{BA} + M_{BC} = 0$

Equating (i) to zero,

$$0.5EI\,\theta_A + 0.25\,EI\,\theta_B = 40 \qquad \text{...(iv)}$$

Adding (ii) and (iii) and equating to zero,

$$0.25EI\,\theta_A + 1.5\,EI\,\theta_B = -60 \qquad \text{...(v)}$$

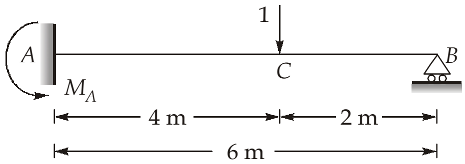
Solving (iv) and (v), we get

$$EI\,\theta_A = 109.09$$

$$EI\,\theta_B = -58.18$$

So, $$\left|\frac{\theta_A}{\theta_B}\right| = \left|\frac{109.09}{-58.18}\right| = 1.875$$

19. (a)



$$M_{FAB} = \frac{-Wab^2}{l^2} = \frac{-1\times4\times2^2}{6^2} = \frac{-4}{9}\text{m}$$

$$M_{FBA} = \frac{-Wba^2}{l^2} = \frac{1\times2\times16}{6^2} = \frac{8}{9}\text{m}$$

Now, $$M_{AB} = M_{FAB} + \frac{2EI}{l}(\theta_B) \qquad [\because \theta_A = 0]$$

$$= \frac{-4}{9} + \frac{2EI\theta_B}{6} = \frac{-4}{9} + \frac{EI\theta_B}{3} \qquad \text{...(i)}$$

$$M_{BA} = M_{FBA} + \frac{2EI}{l}(2\theta_B) = \frac{8}{9} + \frac{2EI}{6}\times 2\theta_B$$

$$= \frac{8}{9} + \frac{2}{3}EI\theta_B \qquad \text{...(ii)}$$

Now, $M_{BA} = 0$ [$\because$ 'B' end is hinged]

$$\Rightarrow \quad \frac{2}{3}EI\theta_B + \frac{8}{9} = 0$$

$$\Rightarrow \quad EI\,\theta_B = \frac{-8\times3}{9\times2} = \frac{-4}{3}$$

Putting value of $EI\,\theta_B$ in (i), we get

$$M_{AB} = \frac{-4}{9} - \frac{4}{3\times3} = \frac{-8}{9}\ \text{m}$$

20. **(d)**

Joint	Member	Stiffness	Total stiffness	D.F.
B	BA	$\frac{4EI}{4}$	$\frac{8EI}{4}$	0.5
	BC	$\frac{4EI}{4}$		0.5

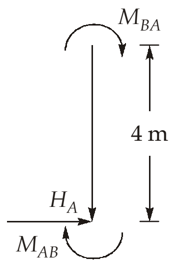
$$M_{FAB} = M_{FBA} = 0$$

$$M_{FBC} = \frac{-24\times 4^2}{12} = -32 \text{ kN-m}$$

$$M_{FCB} = \frac{24\times 4^2}{12} = 32 \text{ kN-m}$$

	A	B		C
		0.5	0.5	
F.E.M.	0	0	–32	32
Bal. Mom.		16	16	
C.O.M.	8			8
	8	+16	–16	8

Consider free body diagram of AB



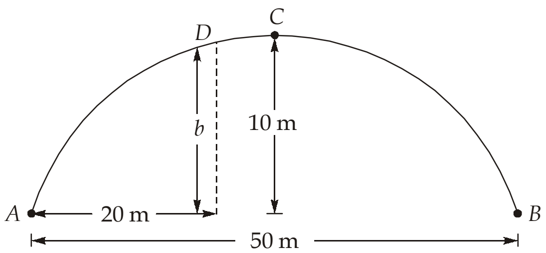
$$\Sigma M_B = 0$$

$$\Rightarrow \quad M_{AB} + M_{BA} = H_A \times 4$$

$$\Rightarrow \quad H_A = 6 \text{ kN}$$

Since, this unbalance H_A is causing the sway of frame, and so 6 kN force will be required at C to prevent sway of frame.

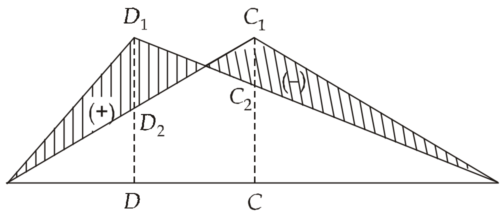
21. **(c)**



Now, $$b = \frac{4hx}{l^2}(l-x) \qquad \text{(where } x = 20 \text{ m)}$$

$$\Rightarrow \quad b = \frac{4\times 10\times 20\times 30}{50^2} = 9.6 \text{ m}$$

I.L.D. for bending moment at D.



Now, $\quad D_1D = \dfrac{a(l-a)}{l}$ (where a = 20 m)

$$= \frac{20\times 30}{50} = 12 \text{ units}$$

$$C_1C = \frac{lb}{4h} = \frac{50\times 9.6}{4\times 10} = 12 \text{ units}$$

$$DD_2 = \frac{12}{25}\times 20 = 9.6$$

So, $\quad D_1D_2 = (12 - 9.6) = 2.4$

For maximum positive bending moment, place one of 100 kN load at D and other at 5 m left of D.

Net height of ILD at 5 m left from D

$$= \left(\frac{20-5}{20}\right)\times 2.4 = 1.8$$

Therefore,

Maximum positive moment $= 100 \times 1.8 + 100 \times 2.4$

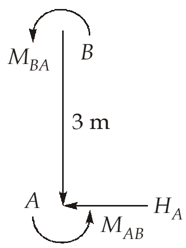
$= 420$ kN-m

22. (c)

Considering free body diagram of AB

$$H_A = \frac{M_{AB}+M_{BA}}{3} = \frac{\dfrac{6EI\Delta}{3^2}+\dfrac{6EI\Delta}{3^2}}{3}$$

$$= \frac{12EI\Delta}{27}(\leftarrow)$$



Considering free body diagram of CD,

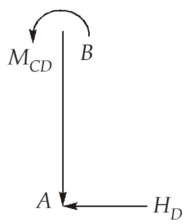
$$H_D = \frac{M_{CD}}{3} = \frac{3EI\Delta}{3^2\times 3} = \frac{3EI\Delta}{27}$$

Now, $\quad \Sigma F_x = 0$

So, $\quad P - H_A - H_D = 0$

$\Rightarrow \quad P = \dfrac{12EI\Delta}{27}+\dfrac{3EI\Delta}{27} = \dfrac{15EI\Delta}{27}$

$\therefore \quad k = \dfrac{15}{27} = 0.56$



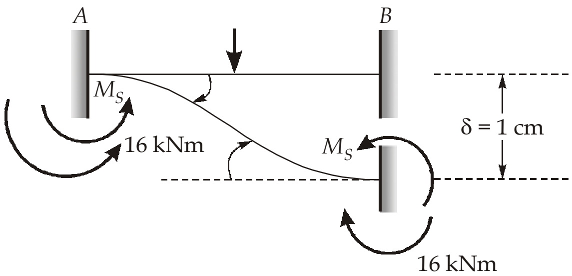
23. (c)

Sinking moment induced, $M_s = \dfrac{6EI\delta}{l^2} = \dfrac{6 \times 2.8 \times 10^6 \times 10^3 \times 1}{(800)^2} = 26250\,\text{Ncm}$

$$= 0.2625 \text{ kNm}$$

$$M_{FAB} = -\frac{16 \times 8}{8} = -16 \text{ kNm}$$

$$M_{FBA} = \frac{16 \times 8}{8} = 16 \text{ kNm}$$



	A	1 B 0
FEM	–16.2625	15.7375
Balance		–15.7375
	–7.86875	
Final moments	–24.13125	0

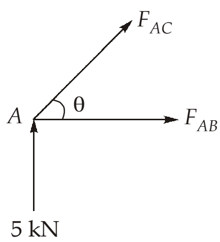
$$M_A = -24.13125 \text{ kNm} \simeq -24.13 \text{ kNm}$$

24. (b)

Since truss and loading are symmetric,

$\therefore$ $R_A = R_B = 5$ kN

At joint A



$$\tan\theta = \frac{4.5}{6} = \frac{3}{4} \Rightarrow \quad \sin\theta = \frac{3}{5} = 0.6, \ \cos\theta = \frac{4}{5} = 0.8$$

$\Sigma F_y = 0 \Rightarrow$ $F_{AC}\sin\theta + 5 = 0$

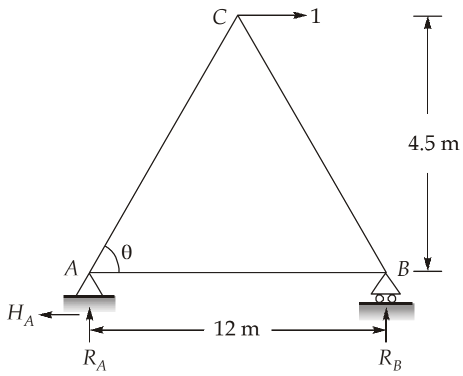
$\Rightarrow$ $F_{AC} = -\dfrac{25}{3}$ kN

$\Sigma F_x = 0 \Rightarrow$ $F_{AC}\cos\theta + F_{AB} = 0$

$\Rightarrow$ $F_{AB} = \dfrac{25}{3} \times \dfrac{4}{5} = \dfrac{20}{3}$ kN

By symmetry, $F_{AC} = F_{BC} = -\dfrac{25}{3}$ kN

For horizontal deflection of joint C, apply unit load in horizontal direction.



$$\Sigma F_y = 0 \Rightarrow \quad R_A + R_B = 0$$

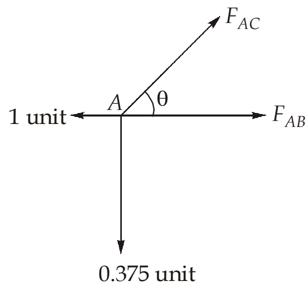
$$\Sigma F_x = 0 \Rightarrow \quad H_A = 1 \text{ unit}$$

$$\Sigma M_A = 0 \Rightarrow \quad 1 \times 4.5 = R_B \times 12$$

$$\Rightarrow \quad R_B = \frac{4.5}{12} = 0.375 \text{ unit} (\uparrow)$$

$$\therefore \quad R_A = 0.375 \text{ unit} (\downarrow)$$

At joint A



$$\Sigma F_y = 0 \Rightarrow \quad F_{AC}\sin\theta = 0.375 \text{ unit}$$

$$\Rightarrow \quad F_{AC} = \frac{0.375}{0.6} = 0.625 \text{ unit}$$

$$\Sigma F_x = 0 \Rightarrow \quad F_{AC}\cos\theta + F_{AB} = 1$$

$$\Rightarrow \quad 0.625 \times 0.8 + F_{AB} = 1$$

$$\Rightarrow \quad F_{AB} = 0.5 \text{ unit}$$

Member	P(kN)	k(unit)	L(mm)	$A(mm^2)$	$\frac{PkL}{AE}$
AC	$-\frac{25}{3}$	0.625	7500	200	−0.977
BC	$-\frac{25}{3}$	−0.625	7500	200	0.977
AB	$\frac{20}{3}$	0.5	12000	150	1.333

$$\Rightarrow \quad \delta_C = \Sigma\frac{PkL}{AE} = 1.33 \text{ mm}$$

∴ Magnitude of horizontal displacement of C is 1.33 mm.

25. (a)

Applying, Work done by load = Strain energy stored in beam

Strain energy stored in beam= $U_{RQ} + U_{QP}$

$$= \int_o^2 \frac{(3x)^2\,dx}{2EI} + \int_o^4 \frac{(6)^2\,dx}{2EI}$$

$$= \frac{9}{2EI}\frac{x^3}{3}\Bigg|_0^2 + \frac{36}{2EI}x\Bigg|_0^4 = \frac{9}{2}\times\frac{8}{3EI}+\frac{36}{2EI}\times 4 = \frac{84}{EI}$$

$$\Rightarrow \quad \frac{1}{2}\times P\times\Delta = \frac{84}{EI}$$

$$\Rightarrow \quad \frac{1}{2}\times 3\times\Delta = \frac{84}{EI}$$

$$\Rightarrow \quad \Delta = \frac{56}{10^4}\times 10^3 = 5.6\text{ mm}$$

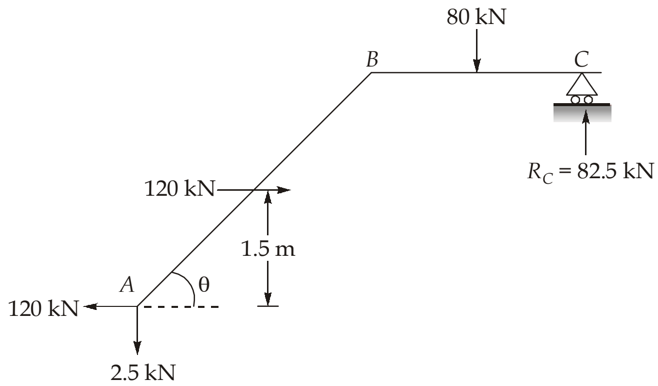
26. (c)

$\Sigma F_y = 0 \quad \Rightarrow \quad R_A + R_C = 80$ kN

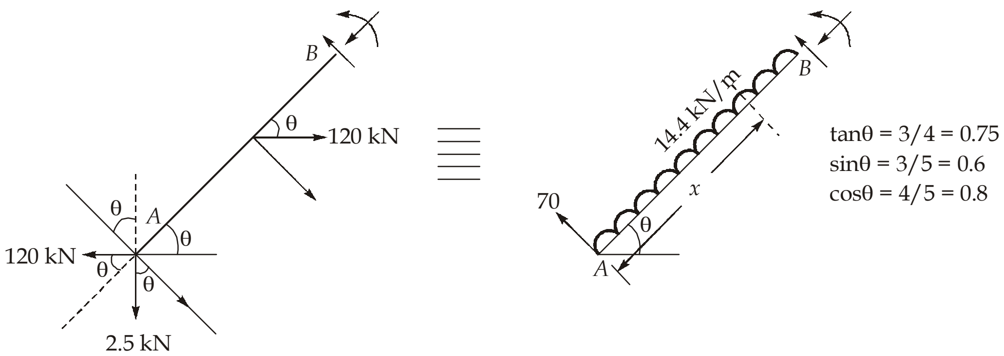
$\Sigma M_A = 0 \quad \Rightarrow \quad R_C \times 8 = 80 \times 6 + 40 \times 3 \times 1.5$

$\Rightarrow \quad R_C = 82.5$ kN (↑)

$\therefore \quad R_A = -2.5$ kN $= 2.5$ kN(↓)



Now, resolving forces perpendicular to AB



$\therefore$ Force perpendicular to AB $= 120 \sin\theta - 2.5 \cos\theta$

$= 120 \times 0.6 - 2.5 \times 0.8$

$= 70$ kN

Also, transformed UDL $= \dfrac{120\times\sin\theta}{5} = \dfrac{120\times 0.6}{5} = 14.4$ kN/m

For maximum BM, SF $= 0$

$\Rightarrow \quad 70 - 14.4x = 0$

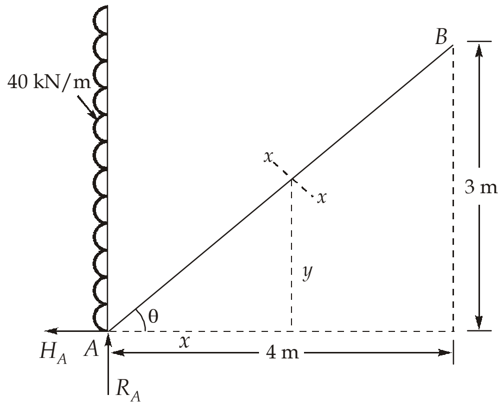
$$x = \frac{70}{14.4} = 4.86 \text{ m} \quad \text{(4.86 m is along AB)}$$

Distance vertically above A, $d = 4.86 \sin\theta = 4.86 \times 0.6 = 2.916 \text{ m} \simeq 2.92 \text{ m}$

Alternatively:

$$\tan\theta = \frac{y}{x} = \frac{3}{4}$$

$$\Rightarrow \quad x = \frac{4}{3}y$$



Now, bending moment at section X-X,

$$M_{xx} = R_A x - 40\frac{y^2}{2} + H_A y$$

$$\Rightarrow \quad M_{xx} = -2.5\left(\frac{4}{3}y\right) - 40\frac{y^2}{2} + 120y$$

$$\Rightarrow \quad M_{xx} = \left(120 - \frac{2.5 \times 4}{3}\right)y - 20y^2 = 116.67y - 20y^2$$

For maximum bending moment, $\frac{dM_x}{dy} = 0$

$$\Rightarrow \quad 116.67 - 20 \times 2y = 0$$

$$\Rightarrow \quad y = 2.917 \text{ m} \simeq 2.92 \text{ m}$$

27. **(a)**

$$\frac{dh}{h} = \frac{3}{16}\left(\frac{l}{h}\right)^2 \alpha t$$

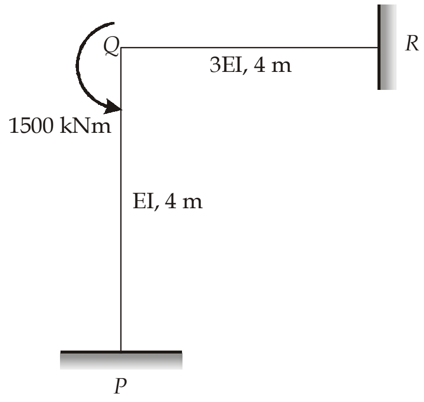
$$\Rightarrow \quad dh = \frac{3}{16}\frac{l^2}{h}\alpha t$$

$$\Rightarrow \quad dh = \frac{3}{16} \times \frac{(200)^2}{(12.5)} \times 11 \times 10^{-6} \times 20$$

$$\Rightarrow \quad dh = 0.132 \text{ m} = 13.2 \text{ cm}$$

28. (d)

$$\text{Moment at } Q = 1000 + 500 \times 1 = 1500 \text{ kNm}$$

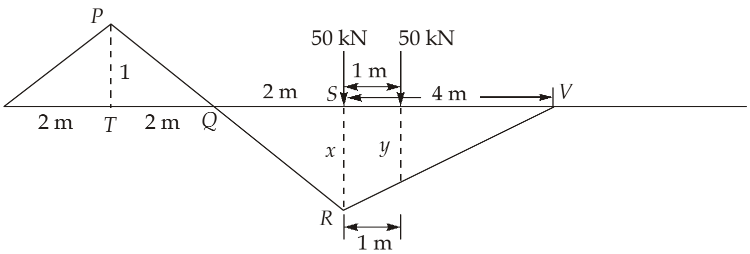


$$\theta_Q = \frac{M}{k_Q} = \frac{1500 \times 10^6}{\frac{4EI}{4 \times 10^3} + \frac{4(3EI)}{4 \times 10^3}} = \frac{1500 \times 10^6}{\frac{16}{4 \times 10^3} EI}$$

$$= \frac{1500 \times 10^6 \times 10^3}{4 \times 10^{10}} = 37.5 \text{ radian}$$

29. (b)

As per Muller Breslau principle,



$$BM_{max} = 50 \times x + 50 \times y$$

From Δs PTQ and QSR

$$\frac{x}{2} = \frac{1}{2} \Rightarrow x = 1$$

Using ΔSVR

$$\frac{x}{4} = \frac{y}{4-1} \Rightarrow \frac{1}{4} = \frac{y}{3} \Rightarrow y = \frac{3}{4}$$

$$BM_{max} = 50 \times 1 + 50 \times \frac{3}{4} = 87.5 \text{ kNm}$$

30. (b)

Applying Betti's theorem

$$25 \times 0.003 + 15 \times \frac{8}{1000} = 15 \times \theta_A + 20 \times 0.008$$

$$\Rightarrow \quad \theta_A = 2.33 \times 10^{-3} \text{ radians}$$

■■■■