#

CLASS TEST

Sl.: 01_SK_CE_ABCD_24082025

MADE EASY

India's Best Institute for IES, GATE & PSUs

Delhi | Bhopal | Hyderabad | Jaipur | Pune | Kolkata

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

STRUCTURE ANALYSIS

CIVIL ENGINEERING

Date of Test : 24/08/2025

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (b)

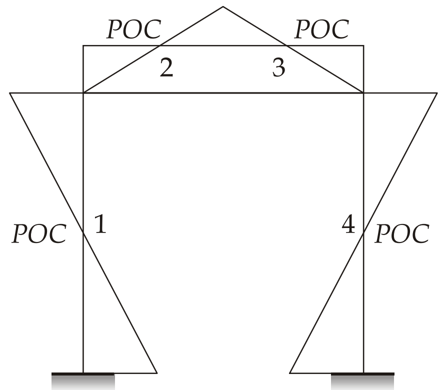
At joint F, $F_{FB} = 0$

At joint C, $F_{BC} = 0$

$F_{CD} = 0$

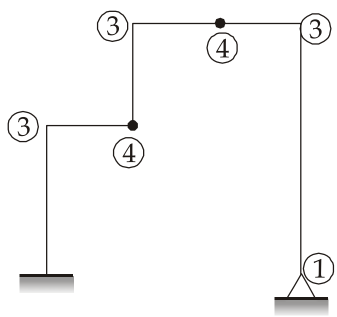
At joint A, $F_{AB} = 0$

2. (c)



4 points of contra-flexure.

3. (b)



Kinematic indeterminacy = 3 + 3 + 3 + 4 + 4 + 1 = 18

4. (b)

$$\Sigma F_y = 0$$

$$\Rightarrow \quad V_A - 10 = 0$$

$$\Rightarrow \quad V_A = 10 \text{ kN } (\uparrow)$$

$$\Sigma M_G = 0$$

$$\Rightarrow \quad H_A \times 3 - 10 \times 9 = 0$$

$$\Rightarrow \quad H_A = \frac{90}{3} = 30 \text{ kN } (\leftarrow)$$

Resultant reaction,

$$R_A = \sqrt{V_A^2 + H_A^2} = \sqrt{10^2 + 30^2}$$

$$= 31.62 \text{ kN}$$

5. **(c)**

Given

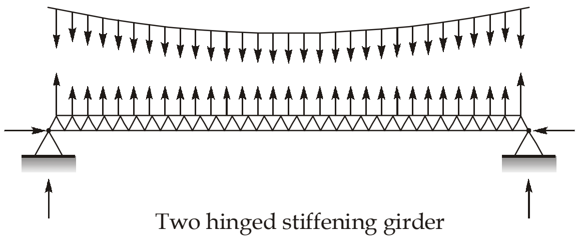
Stiffness matrix $$[k] = \frac{3EI}{L}\begin{bmatrix} 3 & 1 \\ 1 & 3 \end{bmatrix}$$

Flexibility matrix is the inverse of stiffness matrix:

$$[f] = \frac{L}{3EI[3\times3-1\times1]}\begin{bmatrix} 3 & -1 \\ -1 & 3 \end{bmatrix}$$

$$[f] = \frac{1}{24EI}\begin{bmatrix} 3 & -1 \\ -1 & 3 \end{bmatrix}$$

6. **(b)**



Two hinged stiffening girder

$$D_s = r_e - 3$$
$$= 4 - 3 = 1$$

Indeterminate to one degree.

7. **(c)**

Let the parts are x and $(1500 - x)$

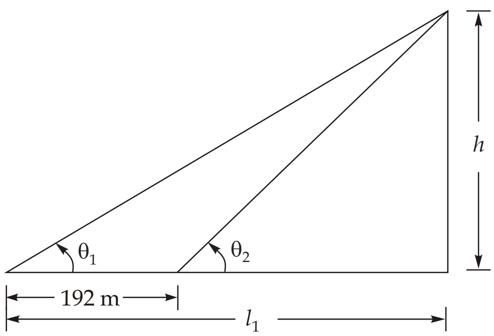
$$\therefore \quad \frac{x\times10\times5}{100} = \frac{(1500-x)\times12.5\times4}{100}$$

$$\Rightarrow \quad x = 1500 - x$$
$$\Rightarrow \quad 2x = 1500$$
$$\Rightarrow \quad x = 750$$

Hence, the sum lent out at 12.5% per annum = 1500 – 750 = Rs. 750

8. **(b)**

Let the height of tower is h and its distance from the point of ground is l_1



$$\therefore \quad \tan\theta_1 = \frac{h}{l_1} = \frac{5}{12} \quad \text{...(i)}$$

$$\tan\theta_2 = \frac{h}{l_1 - 192} = \frac{3}{4} \quad \text{...(ii)}$$

From (i) and (ii)

$$\frac{l_1}{l_1 - 192} = \frac{3}{4} \times \frac{12}{5} = \frac{9}{5}$$

$$\Rightarrow \quad 5l_1 = 9l_1 - 9 \times 192$$

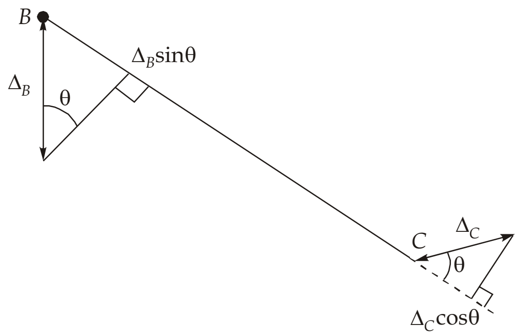
$$4l_1 = 9 \times 192$$

$$l_1 = \frac{9 \times 192}{4}$$

$$l_1 = 432 \text{ m}$$

Hence, height of tower $= \frac{5l_1}{12} = \frac{5}{12} \times 432 = 180$ meters

9. (a)



As length of member will not change because members are inextensible,

$$\Delta_B \sin\theta = \Delta_C \cos\theta$$

$$\Rightarrow \quad \Delta_C = \Delta_B \tan\theta$$

$$\Rightarrow \quad \Delta_C = \frac{4\Delta_B}{3} = \frac{4}{3}\Delta$$

10. (c)

Slope deflection equation for moment of A in the beam AB is given by

$$M_{AB} = (M_F)_{AB} + \frac{2EI}{L}\left(2\theta_A + \theta_B - \frac{3\Delta}{L}\right)$$

11. (b)

$M_A = 0;$ $$V_B \times L = w\frac{L}{4} \times \frac{L}{8}$$

$$\Rightarrow \quad V_B = \frac{wL}{32}$$

$(M_C = 0)_{\text{right}}$ $$V_B \times \frac{L}{2} = H_B \times h$$

$$\Rightarrow \quad \frac{wL}{32} \times \frac{L}{2} = H_B \times h$$

$\Rightarrow \quad H_B = \dfrac{wL^2}{64h}$

So, $\quad k = \dfrac{1}{64}$

12. (b)

Applying Betti's theorem

$$25 \times 0.002 + 15 \times \frac{9}{1000} = 15 \times \theta_A + 22 \times 0.004$$

$\Rightarrow \quad \theta_A = 0.00647$ radian

13. (c)

$$M_{FCB} = -\frac{12 \times 4^2}{12} = -16 \text{ kNm}$$

$$M_{CB} + M_{CA} = 0$$

$$\Rightarrow \quad -16 + \frac{2EI}{4} \times (2\theta_C) + \frac{2EI}{4} \times (2\theta_C) = 0$$

$$\Rightarrow \quad \theta_C = \frac{8}{EI}$$

14. (b)

Using slope deflection method

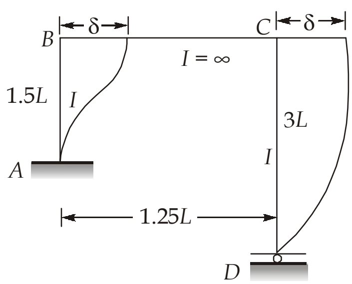
Equilibrium equation at joint B is:

$$M_{BA} + M_{BC} + 10 \times 3 + 12 \times 3 \times 1.5 = 0$$

$$\Rightarrow \quad \frac{2EI}{4}(2\theta_B) + \frac{2EI}{4}(2\theta_B) + 84 = 0$$

$$\Rightarrow \quad \theta_B = \frac{-42}{EI}$$

15. (d)

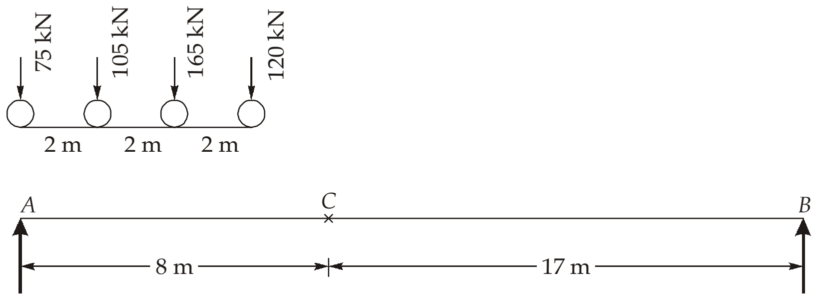


$$M_{BA} = -64 = \frac{6EI\delta}{(1.5L)^2}$$

$$M_{CD} = \frac{3EI\delta}{(3L)^2} = \frac{3EI\delta}{9L^2}$$

$$= -8 \text{ kNm}$$

16. (c)



Load	Average load on AC (y_1) kN	Average load on CB (y_2) kN	Remark
All load on span AC	$\frac{75+105+165+120}{8} = 58.125$	$\frac{0}{17} = 0$	$y_1 > y_2$
120 kN load crosses C	$\frac{75+105+165}{8} = 43.125$	$\frac{120}{17} = 7.06$	$y_1 > y_2$
165 kN load crosses C	$\frac{75+105}{8} = 22.5$	$\frac{165+120}{17} = 16.76$	$y_1 > y_2$
105 kN load crosses C	$\frac{75}{8} = 9.375$	$\frac{105+165+120}{17} = 22.94$	$y_2 > y_1$

So, 105 kN load should be placed at the section.

17. (d)

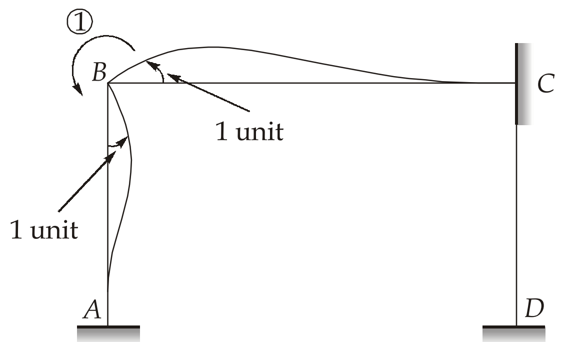
$$\text{Area under free moment diagram} = \text{Area under fixing moment}$$

$$\Rightarrow \quad \frac{1}{2}\times 2\times 370.02 + \frac{1}{2}\times 2\times(370.02+332.18) + \frac{1}{2}\times 2\times 332.18 = \frac{1}{2}\times 6\times(M_A + 228.29)$$

$$\Rightarrow \quad M_A = 239.843 \text{ kNm} \simeq 239.84 \text{ kNm}$$

18. (b)

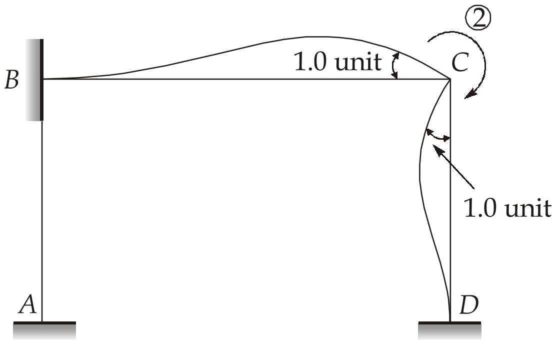
Keeping D_1 = 1.0 and D_2 = 0, the frame will be as shown below:



$$S_{11} = \frac{4\times 3(EI)}{6} + \frac{4EI}{4} = 3EI$$

$$S_{21} = \frac{2\times(3EI)}{6} = -EI$$

Keeping D_1 = 0 and D_2 = 1.0, the frame will be as shown in below:



$$S_{12} = \frac{2 \times 3EI}{6} = -EI$$

and $$S_{22} = \frac{4(3EI)}{6} + \frac{4EI}{4} = 3EI$$

19. (a)

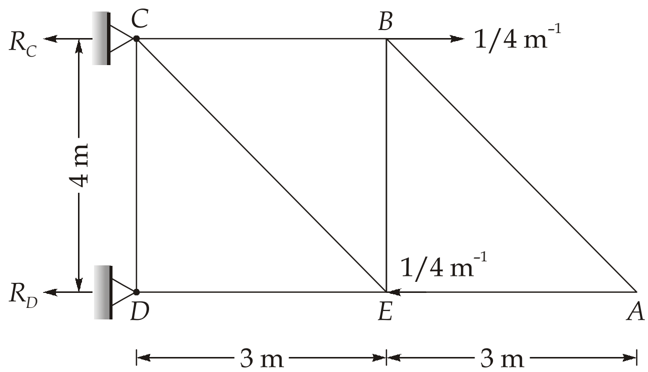
$$\theta_{BE} = \Sigma k\delta$$

δ is given in the problem on the members *AB*, *AE* and *DE*.

Remaining members have no contraction and no elongation.

For the *k* forces:

Apply unit couple as shown below and find k forces in members of the truss.



Since at joint *A*, two non-collinear members are meeting and no external load is applied to it.

$\therefore$ $$F_{AB} = F_{AE} = 0$$

$\Sigma M_C = 0$

$$\frac{1}{4} \times 4 - R_D \times 4 = 0$$

$$R_D = \frac{1}{4}$$

At joint *D*, $$\Sigma F_x = 0$$

$$F_{DE} = \frac{1}{4}\text{(Comp.)}$$

Now, $$\theta_{BE} = \Sigma k\delta = k_{DE}\,\delta_{DE} + k_{AB}\delta_{AB} + k_{AE}\delta_{AE}$$

$$= -\frac{1}{4}\left(+\frac{2}{1000}\right) + 0 + 0 = -\frac{1}{2000} \text{ rad}$$

20. (b)

$$k_{11} = \Sigma\left(\frac{AE}{L}\right)\cos^2\theta$$

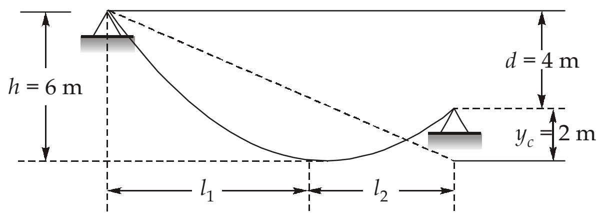
$$= \left[\frac{AE}{L}(\cos 60°)^2\right]_{OD} + \left[\frac{AE}{L}(\cos 90°)^2\right]_{OC}$$

$$+\left[\frac{AE}{L}(\cos 135°)^2\right]_{OB} + \left[\frac{AE}{L}(\cos 150°)^2\right]_{OA}$$

$$= 1.5\frac{AE}{L}$$

Thus $x = 1.5$

21. (c)



$$l_1 + l_2 = l = 100 \text{ m} \quad \text{...(i)}$$

$$d = 4 \text{ m}, h = 6 \text{ m}, y_c = h - d = 2 \text{ m}$$

$$\therefore \quad \frac{l_1}{l_2} = \left(\frac{y_c + d}{y_c}\right)^{1/2} = \left(\frac{2+4}{2}\right)^{1/2} = \sqrt{3} = 1.732$$

$$\Rightarrow \quad l_1 = 1.732\, l_2 \quad \text{...(ii)}$$

From eq. (i) and (ii)

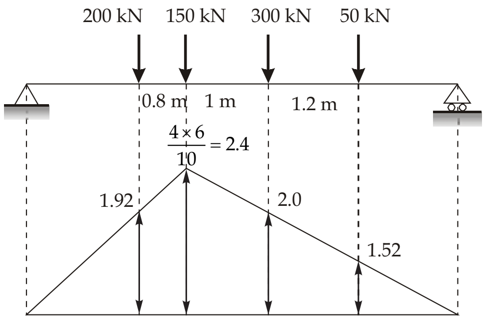
$$l_2 = \frac{100}{2.732} = 36.6 \text{ m}$$

22. (d)

Allow the loads to cross the given section one after another and calculate average loads on *AD* and *BD*

Load crossing section D	Avg. load on AD	Avg. load on BD	Load remarks
50 kN	$\frac{650}{4}$	$\frac{50}{6}$	*AD* > *BD*
300 kN	$\frac{350}{4}$	$\frac{350}{6}$	*AD* > *BD*
150 kN	$\frac{200}{4}$	$\frac{500}{6}$	*BD* > *AD*

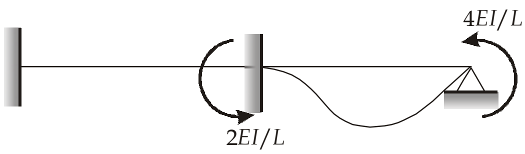
After 150 kN passes the section, average load on *BD* section becomes higher than *AD*.



$$\text{Maximum BM at } D = 200 \times 1.92 + 150 \times 2.4 + 300 \times 2 + 50 \times 1.52$$
$$= 1420 \text{ kNm}$$

23. **(c)**

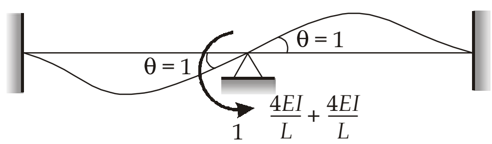
Give unit displacement in the direction (1)



$$k_{11} = \frac{4EI}{L}$$

$$k_{21} = \frac{2EI}{L}$$

Give unit displacement in the direction (2)

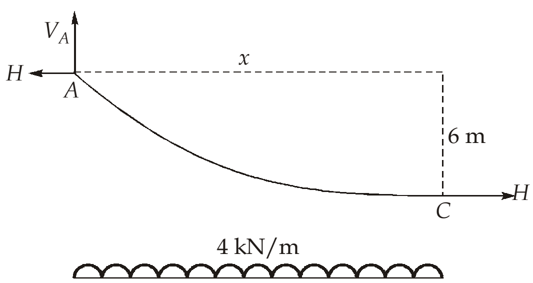


$$k_{12} = \frac{2EI}{L}; \qquad k_{22} = \frac{8EI}{L}$$

$$\therefore \text{ The stiffness matrix } [k] = \begin{bmatrix} \frac{4EI}{L} & \frac{2EI}{L} \\ \frac{2EI}{L} & \frac{8EI}{L} \end{bmatrix}$$

24. **(a)**

Let x be horizontal distance between A and C consider the equilibrium of the part AC of the cable



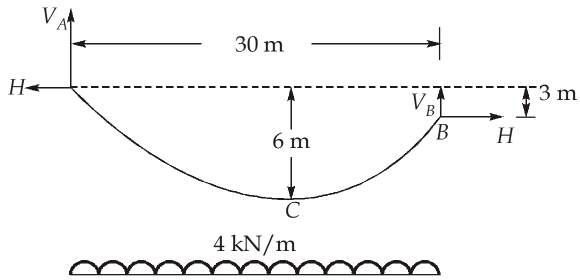
$$\Sigma F_V = 0$$

$$V_A = 4x \quad \text{... (i)}$$

Taking moments about point A, $H \times 6 = 4x \times \frac{x}{2}$

$$\Rightarrow \quad H = \frac{x^2}{3} \quad \text{... (ii)}$$

Now, consider the equilibrium of whole cable,
Taking moments about B,



$$\Sigma M_B = 0$$

$$V_A \times 30 = H \times 3 + (4 \times 30) \times \frac{30}{2}$$

$$\Rightarrow \quad 30V_A = 3H + 1800 \quad \text{...(iii)}$$

From equations (i) and (iii)

$$\Rightarrow \quad 4x \times 30 = \frac{x^2}{3} \times 3 + 1800$$

$$\Rightarrow \quad x^2 - 120x + 1800 = 0$$

After solving we get, $x = 17.574$ m, 102.426 m

Adopt, $x = 17.574$ m

$\therefore \quad V_A = 4 \times 17.574 = 70.296$ kN

$$\therefore \quad H = \frac{(17.574)^2}{3} = 102.95 \text{ kN}$$

$$\therefore \quad \text{Maximum tension, } T_{max} = \sqrt{H^2 + V_A^2} = \sqrt{(70.296)^2 + (102.95)^2}$$

$$= 124.66 \text{ kN}$$

25. (c)

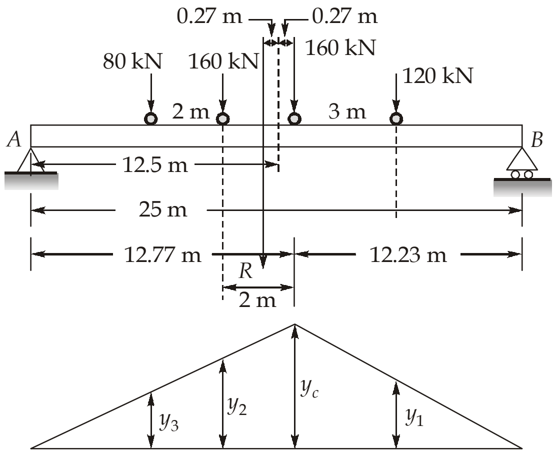
Let CG of loads from leading wheel load (120 kN) be at a distance x,

$$x = \frac{160 \times 3 + 160 \times 5 + 80 \times 7}{120 + 160 + 160 + 80} = 3.54 \text{ m}$$

This is nearer to 160 kN (leading). Hence maximum moment is likely to occur under this load. Its distance from CG,

$$3.54 - 3 = 0.54 \text{ m}$$

Therefore its position from end A $= \frac{25}{2} + \left(\frac{0.54}{2}\right) = 12.77$ m



$$y_c = \frac{12.77 \times 12.23}{25}$$

$$y_1 = \frac{12.77}{25} \times 9.23$$

$$y_2 = \frac{12.23}{25} \times 10.77$$

$$y_3 = \frac{12.23}{25} \times 8.77$$

Therefore, absolute maximum bending moment,

$BM^{max} = 160 \times y_c + 120 \times y_1 + 160 \times y_2 + 80 \times y_3$

$$= 160 \times \left(\frac{12.77 \times 12.23}{25}\right) + 120 \times \left(\frac{12.77 \times 9.23}{25}\right) + 160 \times \left(\frac{12.23}{25} \times 10.77\right) + 80\left(\frac{12.23}{25} \times 8.77\right)$$

= 2751.51 kNm

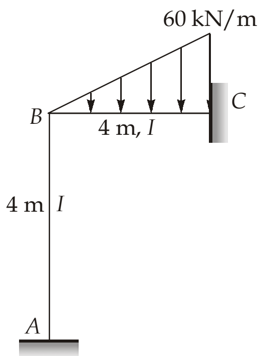
$\therefore$ Maximum bending stress developed:

$$f_{max} = \frac{M}{Z} = \frac{2751.51 \times 10^6}{245.4 \times 10^3} = 11212.35 \text{ N/mm}^2$$

$$= 11.212 \times 10^3 \times 10^6 \text{ N/m}^2 \simeq 11.2 \text{ GPa}$$

26. (b)

Here degree of freedom is 3 i.e. the rotations θ_B θ_C and θ_D. But due to symmetry, we have $\theta_B = -\theta_D$ and $\theta_C = 0$. For analysis of the frame as shown in figure above, only half as of the given structure is considered and $\theta_C = 0$, therefore only θ_B is the only unknown.



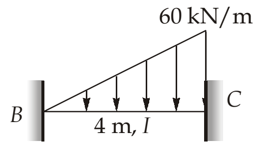
Joint equilibrium equation,

$$M_{BA} + M_{BC} = 0$$

Fixed end moment: Span *AB*

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

Span *BC*:



$$M_{FBC} = -\frac{wL^2}{30} = \frac{60 \times 4^2}{30} = 32 \text{ kNm} \qquad (\theta_A = 0, \Delta = 0)$$

$$\therefore \quad M_{BC} = M_{FBC} + \frac{2EI}{4}\left(2\theta_A + \theta_B - \frac{3\Delta}{4}\right)$$

$$= 32 + \frac{EI\theta_B}{2} \qquad \ldots (\alpha)$$

$$M_{BA} = M_{FBA} + \frac{2EI}{4}\left(2\theta_B + \theta_A - \frac{3\Delta}{4}\right)$$

$$\Rightarrow \quad M_{BA} = EI\theta_B \qquad \ldots (\beta)$$

$$M_{BC} = M_{FBC} + \frac{2EI}{4}\left(2\theta_B + \theta_C - \frac{3\Delta}{4}\right) \qquad (\theta_C = 0, \Delta = 0)$$

$$= -32 + EI\theta_B \qquad \ldots (\gamma)$$

$$M_{CB} = M_{FCB} + \frac{2EI}{4}\left(2\theta_C + \theta_B - \frac{3\Delta}{4}\right)$$

$$= 48 + \frac{EI\theta_B}{2} \qquad \ldots (\delta)$$

Now, from equation (i), $M_{BA} + M_{BC} = 0$

$\Rightarrow$ $EI\theta_B - 32 + EI\theta_B = 0$

$\Rightarrow$ $EI\theta_B = 16$

Now, from equation (δ)

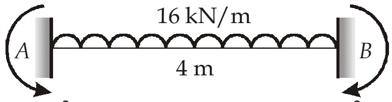
$$M_{CB} = 48 + \frac{16}{2} = 56 \text{ kNm}$$

27. (b)

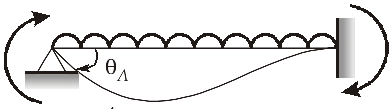
In case of moment distribution method if we have known joint displacements or rotations, the effect of these things are taken in fixed end moment.

Calculation of fixed end moment.

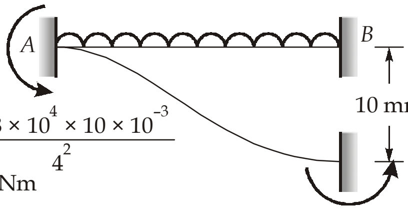
Span AB



$$\frac{16\times4^2}{12} = 21.33\text{ kNm} \qquad \frac{16\times4^2}{12} = 21.33\text{ kNm}$$



$$\frac{4EI\theta_A}{L} = \frac{4\times8\times10^4\times0.002}{4} = 160\text{ kNm} \qquad \frac{2EI\theta_A}{L} = \frac{2\times8\times10^4\times0.002}{2} = 80\text{ kNm}$$



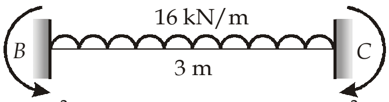
$$\frac{6EI\Delta_B}{L^2} = \frac{6\times8\times10^4\times10\times10^{-3}}{4^2} = 300\text{ kNm}$$

$$\frac{6EI\Delta_B}{L^2} = \frac{6\times8\times10^4\times10\times10^{-3}}{4^2} = 300\text{ kNm}$$

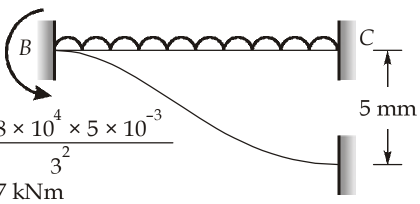
$\therefore$

$$M_{FAB} = -21.33 + 160 - 300 = -161.33\text{ kNm}$$
$$M_{FBA} = 21.33 + 80 - 300 = -198.67\text{ kNm}$$

Span BC



$$\frac{16\times3^2}{12} = 12 \qquad \frac{16\times3^2}{12} = 12$$



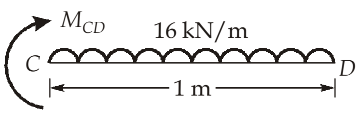
$$\frac{6EI\Delta_C}{L^2} = \frac{6\times8\times10^4\times5\times10^{-3}}{3^2} = 266.67\text{ kNm}$$

$$\frac{6EI\Delta_C}{L^2} = \frac{6\times8\times10^4\times5\times10^{-3}}{3^2} = 266.67\text{ kNm}$$

$\therefore$

$$M_{FBC} = -12 - 266.67 = -278.67\text{ kNm}$$
$$M_{FCB} = 12 - 266.67 = -254.67\text{ kNm}$$

Span *CD*:



$$M_{CD} + \frac{16\times(1)^2}{2} = 0$$

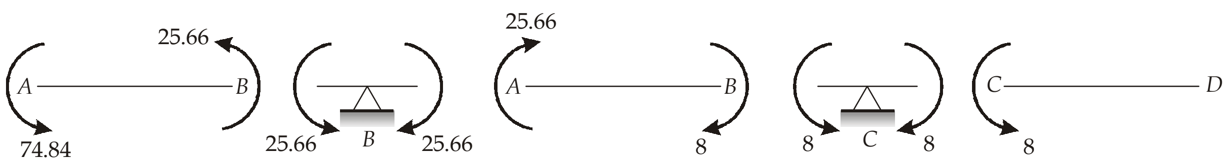
$\Rightarrow$

$$M_{CD} = -8\text{ kNm}$$

Distribution factor (DF):

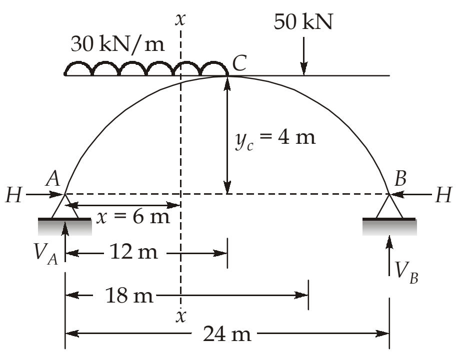
Joint	Member	Member Stiffness(MS)	Member Stiffness(JS)	$D_F = \frac{MS}{JS}$
B	BA	$\frac{4EI}{4} = EI$	$2EI$	0.5
	BC	$\frac{3EI}{3} = EI$		0.5

A	B		C	D
	1/2	1/2		
		4 ←	8	−8
−161.33	−198.67	−278.67	−254.67	
		127.335 ←	+254.67	
−161.34	−198.66	−147.335	8	−8
	+173	+173		
86.5			0	
−74.84	−25.66	25.66	8	−8



∴ Support moment at B = 25.66 kNm (Hogging)

28. (a)



The arch is shown in figure above,

Taking moments about B,

$\Sigma M_B = 0$

$$V_A \times 24 - 30 \times 12 \times 18 - 50 \times 6 = 0$$

$\Rightarrow$ $V_A = 282.5$ kN

∴ $V_B = 30 \times 12 + 50 - 282.5 = 127.5$ kN

$\Sigma M_C = 0$ (From right)

$$V_B \times 12 - H \times 4 - 50 \times 6 = 0$$

$$\Rightarrow \quad H = \frac{127.5 \times 12 - 50 \times 6}{4} = 307.5 \text{ kN}$$

Now, at 6 m from left support, $y = \frac{4y_c}{L^2} x(L-x) = \frac{4 \times 4}{24^2} \times 6(24-6) = 3 \text{ m}$

Also, $\frac{dy}{dx} = \frac{4y_c}{L^2}(L-2x)$

At $x = 6$ m, $\frac{dy}{dx} = \tan\theta = \frac{4 \times 4}{24^2}(24 - 2 \times 6) = \frac{1}{3}$

$$\therefore \quad \cos\theta = \frac{3}{\sqrt{10}}; \quad \sin\theta = \frac{1}{\sqrt{10}}$$

Radial shear at a section 6 m from left support,

$$Q = V\cos\theta - H\sin\theta$$

$$V = \text{Vertical shear at x-x}$$

$$= V_A - wx = 282.5 - 30 \times 6$$

$$= 102.5 \text{ kN}$$

$$\therefore \quad Q = 102.5 \times \frac{3}{\sqrt{10}} - 307.5 \times \frac{1}{\sqrt{10}} = 0$$

29. (b)

$$M_x = \frac{wR^2}{2}(\sin\theta - \sin^2\theta)$$

For maximum bending moment,

$$\frac{dM_x}{d\alpha} = 0$$

$$\frac{wR^2}{2}(\cos\theta - 2\sin\theta\cos\theta) = 0$$

$$\Rightarrow \quad \cos\theta(1 - 2\sin\theta) = 0$$

$$\Rightarrow \quad \cos\theta = 0 \Rightarrow \theta = 90° \text{ and } \sin\theta = \frac{1}{2} \Rightarrow \theta = 30°$$

$$\theta = 90° \text{ (Rejected as BM = 0)}$$

$$\therefore \quad \theta = 30°$$

$$\therefore \quad M_x = \frac{-wR^2}{2}(\sin 30° - \sin^2 30°)$$

$$= \frac{-wR^2}{8} \text{ or } \frac{wR^2}{8} \text{ (Hogging)}$$

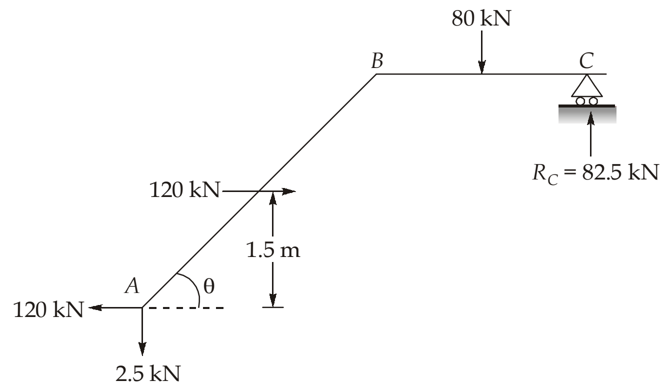
30. (c)

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

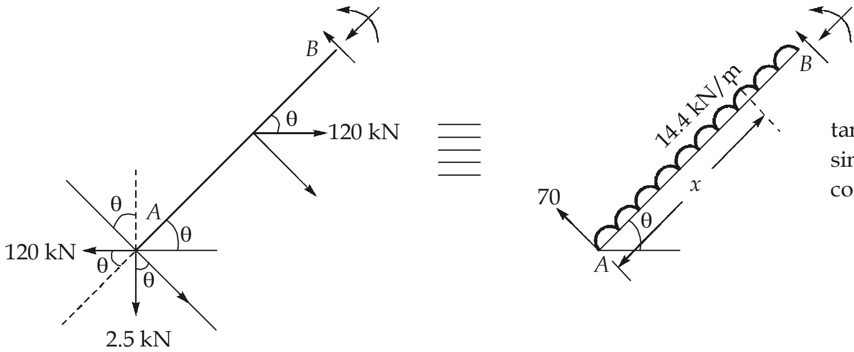
$\Sigma M_A = 0 \quad \Rightarrow \quad R_C \times 8 = 80 \times 6 + 40 \times 3 \times 1.5$

$\Rightarrow \quad R_C = 82.5$ kN $(\uparrow)$

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



Now, resolving forces perpendicular to AB



tanθ = 3/4 = 0.75
sinθ = 3/5 = 0.6
cosθ = 4/5 = 0.8

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

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

$$= 70 \text{ kN}$$

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

For maximum BM, SF = 0

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

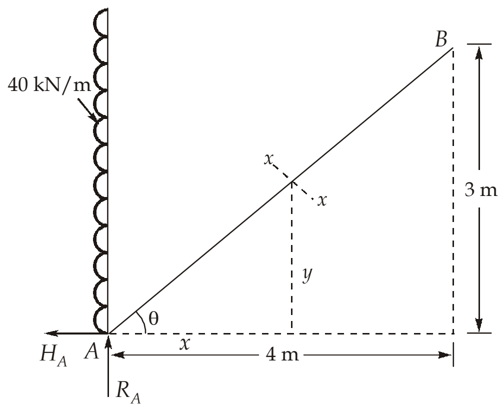
$$x = \frac{70}{14.4} = 4.86 \text{ m} \quad (4.86 \text{ m is along AB})$$

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

Alternatively:

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

$\Rightarrow$ $x = \dfrac{4}{3}y$



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, $\frac{dM_x}{dy} = 0$

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

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

■■■■