Class Test Solution (STRUCTURE) 17-08-2018
Answer key

1.	(b)	**16.**	(–)	**31.**	(d)	**46.**	(b)	**61.**	(b)
2.	(b)	**17.**	(c)	**32.**	(b)	**47.**	(c)	**62.**	(d)
3.	(c)	**18.**	(d)	**33.**	(b)	**48.**	(d)	**63.**	(b)
4.	(a)	**19.**	(b)	**34.**	(c)	**49**	(a)	**64.**	(d)
5.	(b)	**20.**	(c)	**35.**	(b)	**50.**	(c)	**65.**	(a)
6.	(a)	**21.**	(a)	**36.**	(c)	**51.**	(b)	**66.**	(a)
7.	(c)	**22.**	(a)	**37.**	(b)	**52.**	(c)	**67.**	(b)
8.	(c)	**23.**	(b)	**38.**	(c)	**53**	(b)	**68.**	(a)
9.	(b)	**24**	(b)	**39.**	(d)	**54.**	(d)	**69.**	(a)
10.	(d)	**25.**	(b)	**40.**	(b)	**55.**	(c)	**70.**	(c)
11.	(c)	**26.**	(b)	**41.**	(b)	**56.**	(b)	**71.**	(b)
12.	(d)	**27.**	(d)	**42.**	(a)	**57.**	(c)	**72.**	(a)
13.	(d)	**28.**	(c)	**43.**	(c)	**58.**	(c)	**73.**	(d)
14.	(c)	**29.**	(d)	**44.**	(c)	**59.**	(c)	**74.**	(c)
15.	(b, d)	**30.**	(a)	**45.**	(a)	**60.**	(c)	**75.**	(a)

CLASS TEST [STRUCTURE] SOLUTIONS

1. (b)

2. (b)

No. of cuts in (i) required = 4

No. of restraint required = 4 – 1 = 3

$D_s = 3 \times 4 - 3 = 9$

No. of cuts in (ii) required = 4

No. of restraint required = 2 – 1 = 1

$D_s = 3 \times 4 - 1 = 11$

3. (c)

Unknown displacement are

$\theta_{BA}, \theta_{BC}, \Delta_{yB}, \theta_C, \theta_D$ i.e., $D_K = 5$

4. (a)

The unknown displacements are

$\theta_A, \theta_{BA}, \theta_{BC}, \Delta_{yB}, \theta_{CD}, \theta_D, \theta_E$

Since $\theta_{BC} = \theta_{CB}$ and $\Delta_{By} = \Delta_{Cy}$

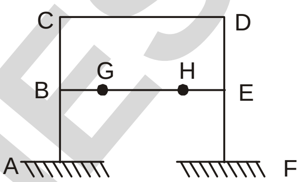
Hence $D_K = 7$

5. (b)

No. of cuts required = 2

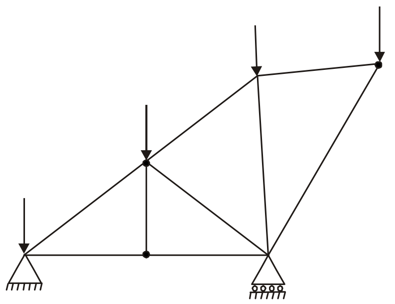
No. of restraint required = 2

$D_S = 3 \times 2 - 2 = 4$



$|\theta_B, \Delta_{BH}| |\theta_c, \Delta_{CH}| |\theta_D| |\theta_{G_1}, \theta_{G_2}, \Delta_{Gy}| |\theta_{H_1}, \theta_{H_2}, \Delta_{Hy}| |\theta_E|$

6. (a)



here $m = 9$

no. of joints $= j = 6$

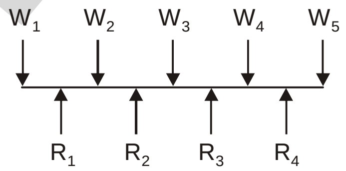
here $\because m = 2j - 3$

$\therefore$ the pin jointed truss is internally determinate and stable hence it is a perfect frame

if $m > 2j - 3$, the truss is internally indeterminate and stable

if $m < 2j - 3$, truss is internally unstable for simple truss

7. (c)



Nos. of reactions (unknown) = 4 (ie. R_1, R_2, R_3, & R_4) but nos. of equilibrium equations available = 2 (ie. one $\Sigma F_y = 0$ & another $\Sigma M = 0$)

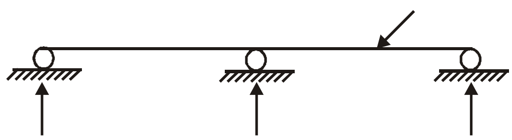
$\therefore$ degree of indeterminacy = 4 – 2 = 2

8. (c)

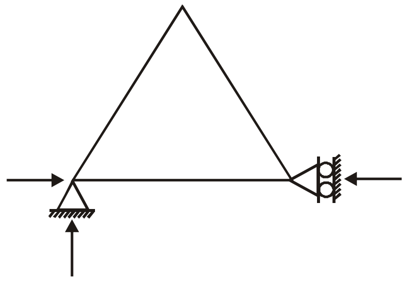
If a body is sufficiently constraint by external reaction such that rigid body movement of structure does not occur, then the structure is said to be stable externally.

Necessary condition for this is that :

- There should be three reactions that are neither concurrent nor parallel (in plane structure).
- Reactions should be non-parallel, non-concurrent and non-coplanar for space structure (concurrent means meeting at a single point).



Parallel reaction : Inclined loading will lead to rigid body movement. Hence unstable externally.



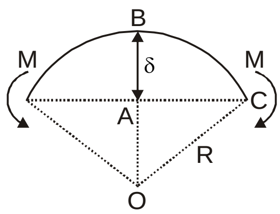
Concurrent Reaction : Hence unstable

9. (b)

OA = $(R-\delta)$ & AC = L/2

In ΔOAC

$OA^2 + AC^2 = OC^2 \Rightarrow (R-\delta)^2 + (L/2)^2 = R^2$



$\Rightarrow -2R\delta + \delta^2 + L^2/4 = 0$

$\Rightarrow R = \frac{L^2}{8\delta} \quad \left[\because \delta^2 \text{ is very small}\right]$

$\because M = \frac{EI}{R} = \frac{8EI\delta}{L^2}$

10. (d)

Fixed end moment at B due to U.D.L

$$= \frac{WL^2}{12}(\text{clockwise})$$

Fixed end moment due to sinking of support

$$B = -\frac{6EI.\Delta}{L^2} = -\frac{6EI}{L^2} \times \frac{WL^4}{96EI}$$

$$= -\frac{3WL^2}{48}$$

Hence, fixing end moment at

$$B = \frac{WL^2}{12} - \frac{3WL^2}{48} = \frac{WL^2}{48}$$

11. (c)

Deflection at A in Beam = Compression in column AC

$$\frac{(50-R)L^3}{3EI} = \frac{R.L}{AE}$$

$$\frac{(50-R)L^2}{3I} = \frac{R}{A}$$

$$R = \frac{(50-R) \times 1000^2 \times 75000}{1.5625 \times 10^9 \times 3}$$

$$R = (50 - R) \times 16$$

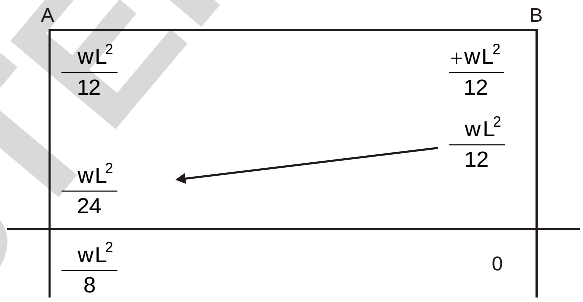
$$17R = 800$$

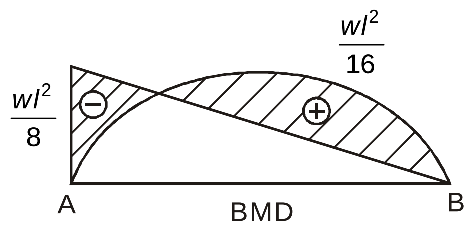
$$R = 47.058 \text{ kN}$$

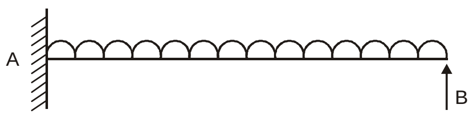
$$\therefore V_B = 50 - 47.058 = 2.942 \text{ kN}$$

12. (d)

13. (d)







$$\Delta_B = 0$$

$$\frac{wL^4}{8EI} = \frac{R_B L^3}{3EI}$$

$$R_B = \frac{3}{8}WL$$

14. (c)

Examples of Force Method

- Castigliano's Theorm (Method of Least Work)
- Strain energy method.
- Claperon's three moment equations (used

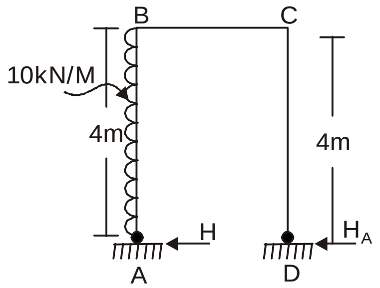
in continous beam analysis)

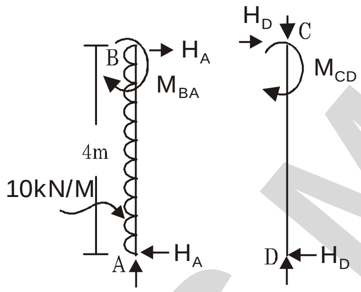
- Column analogy method (used in rigid frames with fixed supports)
- Flexibility matrix method

Examples of displacement Method

- Slope deflection method
- Moment distribution method
- Stiffners matrix method
- Kani's Method

15. (b, d)





$$\Sigma M_B = 0$$

$$M_{BA} + 4\,H_A - \frac{10\times 4^2}{2} = 0$$

or, $M_{BA} + 4H_A - 80 = 0$ ...(i)

Again, $\Sigma M_c = 0$

$$\Rightarrow \quad H_D = -\frac{M_{CD}}{4} \qquad \text{...(ii)}$$

From (i)

$$H_A = \frac{80 - M_{BA}}{4}$$

$$\Sigma H = 10 \times 4$$

$$\Rightarrow \quad H_A + H_D = 40$$

$$\therefore \quad \frac{-M_{CD}}{4} + \frac{80 - M_{BA}}{4} = 40$$

$$\Rightarrow \quad -M_{CD} + 80 - M_{BA} = 160$$

or, $M_{BA} + M_{CD} = -80$

16. (–)

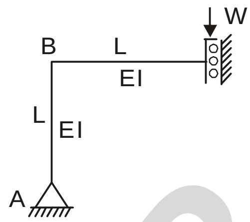


Fig. (i)

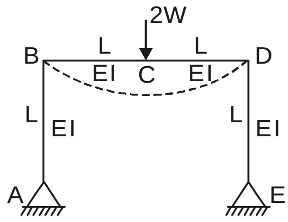


Fig. (ii)

θ_B in fig. (i) = θ_B in fig. (ii)

(Note the symmetricity of fig. (ii).) Fig. (ii) is nothing but <u>Fig. (i) + its mirror image</u>

In fig. (ii)

$$M_{BA} = \frac{3EI}{L}(\theta_B) \qquad \text{...(1)}$$

$$M_{BD} = -\frac{(2W)\times 2L}{8} + \frac{2EI}{2L}(2\theta_B + \theta_D)$$

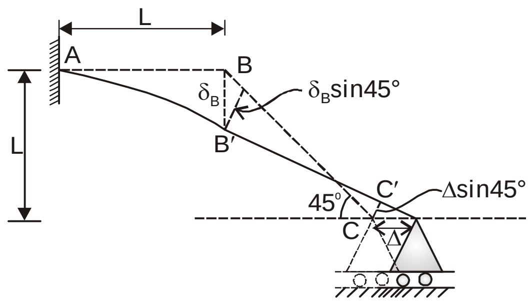
$$= -\frac{WL}{2} + \frac{EI}{L}(\theta_B) \qquad (\text{as } \theta_B = -\theta_D)$$

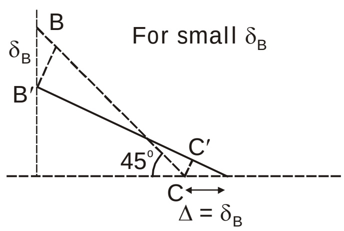
$$M_{BA} + M_{BD} = 0$$

$$\Rightarrow \quad \frac{4EI\theta_B}{L} = \frac{WL}{2}$$

$$\Rightarrow \quad \theta_B = \frac{WL^2}{8EI}$$

17. (c)





$$\delta = \delta_B \sin 45° + \Delta \sin 45°$$

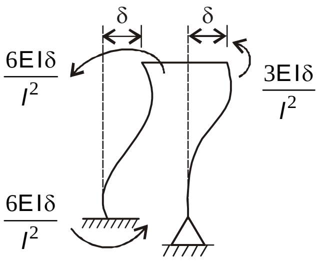
$$= 2\delta_B \sin 45° = \sqrt{2}\,\delta_B$$

$$M_{BC} = 0 + \frac{2EI}{\sqrt{2}\,L} \times \left(2\theta_B + \theta_C - 3\frac{(-\sqrt{2}\,\delta_B)}{\sqrt{2}\,L}\right)$$

$$= \frac{2EI}{\sqrt{2}\,L}\left[2\theta_B + \theta_C + \frac{3\delta_B}{L}\right]$$

$$= \frac{\sqrt{2}\,EI}{L}\left[2\delta_B + \theta_C + \frac{3\delta_B}{L}\right]$$

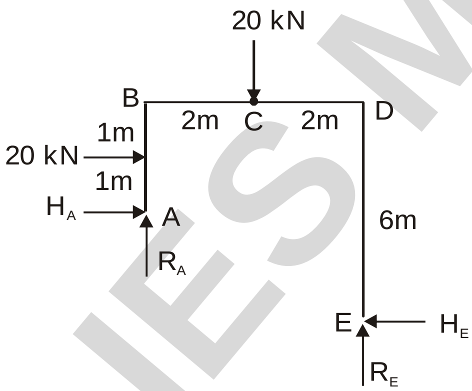
18. (d)



$$\Rightarrow \frac{12EI\delta}{l^3} + \frac{3EI\delta}{l^3} = P$$

$$\Rightarrow \delta = \frac{Pl^3}{15EI}$$

19. (b)



$$\Sigma F_x = 0 \Rightarrow 20 + H_A - H_E = 0 \quad \text{...(i)}$$

$$\Sigma F_y = 0 \Rightarrow R_A + R_E - 20 = 0 \quad \text{...(ii)}$$

$$\Sigma M_A = 0 \Rightarrow 20 \times 1 + 20 \times 2 + 4H_E - 4R_E = 0$$

$$\Rightarrow 60 + 4H_E - 4R_E = 0$$

$$\Rightarrow 15 + H_E - R_E = 0 \quad \text{... (iii)}$$

$$\Sigma M_C = 0$$

$$6H_E = 2R_E \Rightarrow R_E = 3H_E \quad \text{... (iv)}$$

From equation (iv) and equation (iii)

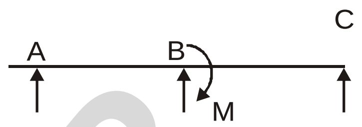
$$15 + H_E - 3H_E = 0$$

$$15 = 2H_E$$

$$\Rightarrow H_E = 7.5 \text{ kN}$$

20. (c)

Initially



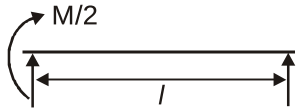
$$M_{BA} = \frac{3EI}{l}(\theta_B) \quad \text{...(1)}$$

$$M_{BC} = \frac{3EI\theta_B}{l} \quad \text{...(2)}$$

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

$$\Rightarrow M_{BC} = \frac{M}{2}$$

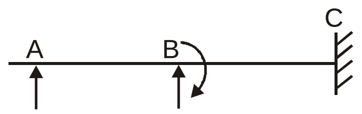
$$\Rightarrow M = \frac{6EI\theta_B}{l}$$



$$M/2l = R_0$$

$$R_0 = \frac{M}{2}l$$

Fix the end C



$$M_{BC} = \frac{3EI}{l}(\theta_B)$$

$$M_{BC} = \frac{4EI\theta_B}{l}$$

$$M_{CB} = \frac{2EI\theta_B}{l}$$

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

$$\frac{3EI\theta_B}{l} + \frac{4EI\theta_B}{l} = M$$

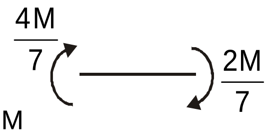
$$\frac{7EI\theta_B}{l} = M$$

$$EI\theta_B = \frac{Ml}{7}$$

$$M_{BA} = \frac{3}{7}M$$

$$M_{BC} = \frac{4M}{7}$$

$$M_{CB} = \frac{2M}{7}$$



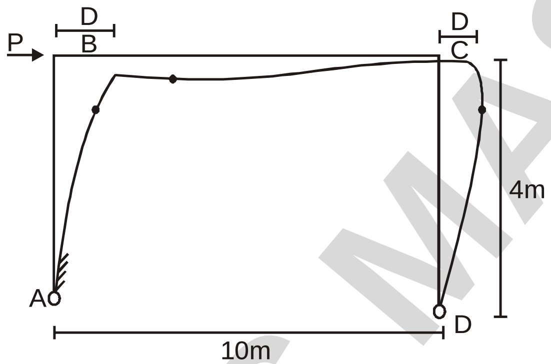
$$\frac{6M}{7l} = R_C > \frac{M}{2l}$$

21. **(a)**

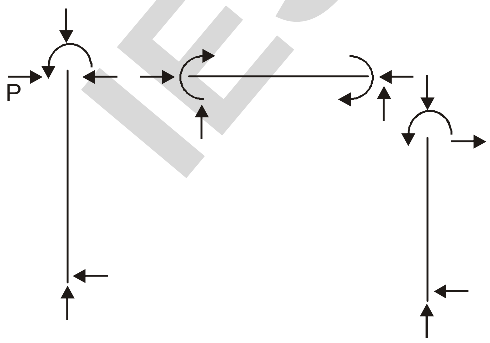
22. **(a)**

23. **(b)**

$$\theta_B = \theta_C = \frac{400}{EI} \text{ radian}$$



By inspection, the sway would be towards right.



Applying slope deflection for beam BC

$$M_{BC} = 0 + \frac{2EI}{10}(2\theta_B + \theta_C)$$

$$= \frac{2EI}{10}\left(\frac{800}{EI} + \frac{400}{EI}\right)$$

$$= \frac{2400}{10} = 240 \text{ kNm}$$

24. **(b)**

Member Stiffness

OA $3 \times \frac{EI}{3} = EI$

OD $4 \times \frac{2EI}{4} = 2EI$

OC $4 \times \frac{EI}{4} = EI$

OB $4 \times \frac{EI}{5} = 0.8EI$

Distribution factor for OC

$$= \frac{EI}{EI + 2EI + EI + 0.8EI} = \frac{1}{4.8}$$

$\therefore$ Moment in member OC at O = $\frac{200}{4.8}$

Carry over factor = $\frac{1}{2}$

Therefore, bending moment at

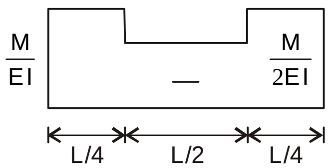
$$C = \frac{200}{4.8 \times 2} = 20.83 \text{ kN-m}$$

25. **(b)**

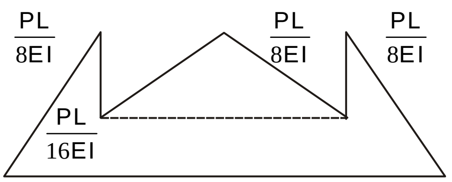
Joint	Member	Relative Stiffness	Total Relative Stiffness	Distribution factor
C	CB	$\frac{4E(2I)}{8}$	$\frac{11EI}{8}$	8/11
	CD	$\frac{3E(I)}{8}$		3/11
	CE	0		0

Since end E is free end, member CE has zero relative stiffness

26. **(b)**



$\frac{M}{EI}$ Diagram due to fixed end moment



$$2\times\frac{M}{EI}\times\frac{L}{4}+\frac{M}{2EI}\times\frac{L}{2}=2\times\frac{1}{2}\times\frac{PL}{8EI}\times\frac{L}{4}+\frac{PL}{16EI}\times\frac{L}{2}$$

$$+\frac{1}{2}\times\frac{PL}{16EI}\times\frac{L}{2}$$

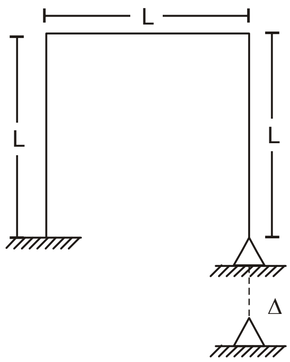
$$\Rightarrow \frac{ML}{2EI}+\frac{ML}{4EI}=\frac{PL^2}{32EI}+\frac{PL^2}{32EI}+\frac{PL^2}{64EI}$$

$$\Rightarrow M=\frac{5PL}{48}$$

27. (d)

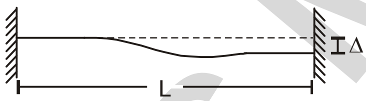
28. (c)

29. (d)



The horizontal member of the frame will have both the ends at relative level difference of Δ.

$\therefore$ It is similar to



$\therefore$ Fixed end moment of the horizontal member $=\frac{6EI\Delta}{L^2}$.

30. (a)

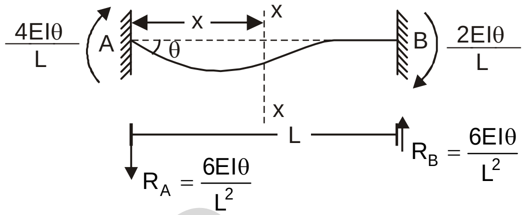
Distribution factor for

$$BA=\frac{\frac{3I}{4I}}{\frac{3I}{4I}+\frac{I}{I}}=\frac{\frac{3}{4}}{\frac{3}{4}+1}=\frac{\frac{3}{4}}{\frac{7}{4}}=\frac{3}{7}$$

Distribution factor for BC $=1-\frac{3}{7}=\frac{4}{7}$

$$\therefore \frac{\text{Distribution factor for BA}}{\text{Distribution factor for BC}}=\frac{\frac{3}{7}}{\frac{4}{7}}=\frac{3}{4}=3:4$$

31. (d)



$$\Sigma M_A=0\Rightarrow R_B.L=\frac{4EI\theta}{L}+\frac{2EI\theta}{L}$$

$$\therefore R_B=\frac{6EI\theta}{L^2}$$

$$M_X=\frac{4EI\theta}{L}-\frac{6EI\theta}{L^2}\cdot x=0$$

(for point contraflexure)

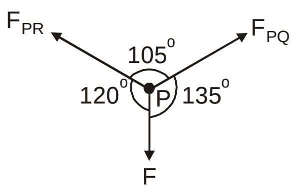
$$\Rightarrow x=\frac{4L}{6}=\frac{2L}{3}$$

32. (b)

$F_{PQ}\longrightarrow$ Tension in member PQ

$F_{PR}\longrightarrow$ Tension in member PR

By Lami's theorem at point P.

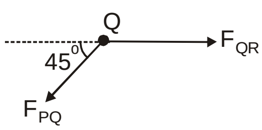


$$\frac{F_{PR}}{\sin 135}=\frac{F_{PQ}}{\sin 120}=\frac{F}{\sin 105}$$

$$\Rightarrow F_{PQ}=\left(\frac{\sin 120}{\sin 105}\right).F$$

$$=0.8966\ F$$

Free body diagram at point Q,



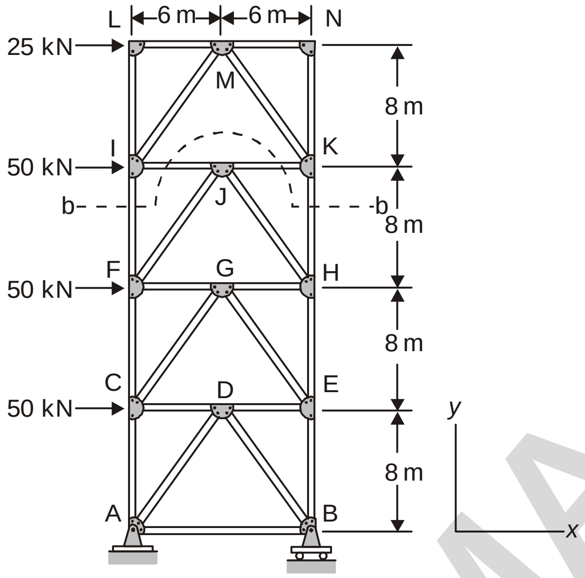
$$\sum F_x = 0$$

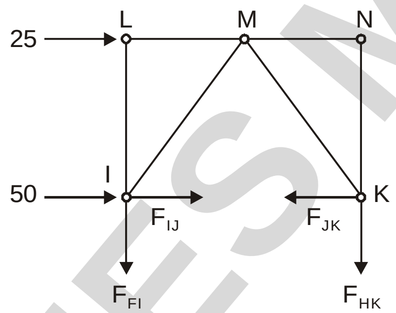
$$F_{QR} = F_{QP} \cos 45^o$$

$$F_{QR} = \frac{0.8966\ F}{\sqrt{2}} = 0.634\ F$$

$$\boxed{F_{QR} = 0.634\ F}$$

33. (b)





(a) (b) Section bb

From Section bb using fig. (b)

$$+\sum M_I = 0 - 25 \times 8 - F_{HK}(12) = 0$$

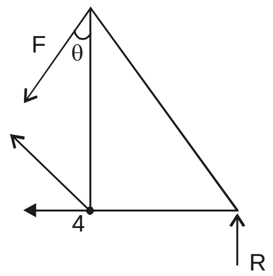
$$F_{HK} = -16.67\ kN$$

$$F_{HK} = 16.67\ kN\ (C)$$

34. (c)

$$20 \times \sqrt{(1.5)^2 + (1.5)^2} = R \times 6$$

$$\Rightarrow \quad R = 7.071\ kN$$



Taking moment of all forces about 4

$$F \sin\theta \times 3 + R \times 3 = 0$$

$$\sin\theta = \frac{1.5}{\sqrt{4.5}} \quad \Rightarrow \quad F = -9.999\ kN$$

35. (b)

$$\sum F_x = 0$$

$$R_H = 12\ kN$$

$$\sum F_y = 0$$

$$R_H + R_I = 0$$

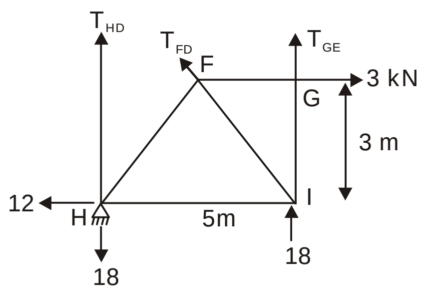
$$\sum M_H = 0$$

$$R_I \times 5 = 3 \times 3 + 3 \times 6 + 3 \times 9 + 3 \times 12$$

$$R_I = 18\ kN$$

$$R_H = -18\ kN$$

Do a cut between HD & GE



take moment about I

$$T_{HD} \times 5 - 18 \times 5 + 3 \times 3 = 0$$

$$T_{HD} \times 5 = 90 - 9$$

$$T_{HD} = 16.2\ (T)$$

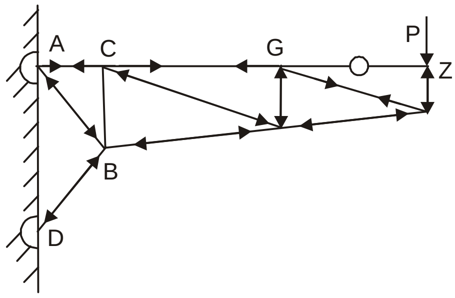
take moment about D

$$\sum M_D = 0$$

$$12 \times 6 - 18 \times 5 - 3 \times - T_{GE} \times 5 = 0$$

$$T_{GE} = -\frac{27}{5} = 5.4\ (Com.)$$

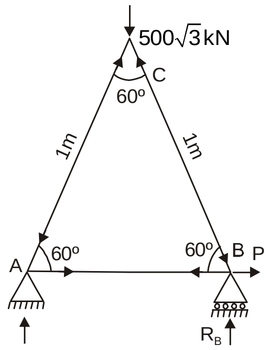
36. (c)



The nature of the forces in members have been depicted in the above figure.

37. (b)

To calculate, horizontal diflection of the joint B, applying a force 'p' at B.



$$\Sigma M_A = 0$$

$$\Rightarrow 500\sqrt{3} \times \frac{1}{2} + 0 = 1 \times R_B$$

$\therefore$ $R_B = 250\sqrt{3}$ kN

$\therefore$ $R_A = 250\sqrt{3}$ kN

At joint c,

$F_{CA} \cos 30^o + F_{CB} \cos 30^o = 500\sqrt{3}$

and F_{CA} Sin $30^o = F_{CB}$ sin 30^o

$\Rightarrow$ $F_{CA} = F_{CB}$

$\therefore$ $2F_{CA} \frac{\sqrt{3}}{2} = 500\sqrt{3}$

$\Rightarrow$ $F_{CA} = 500$ kN

$\therefore$ $F_{CB} = 500$ kN

At joint B,

F_{CB} Sin $60^o = 250\sqrt{3}$ and

$F_{BC} \cos 60^o + P = F_{BA}$

or, $F_{BA} = P + 500 \times \frac{1}{2} = P + 250$

$\therefore$ $U = \sum \frac{F^2L}{2AE}$

Using castigliano's theorem (2nd theorem)

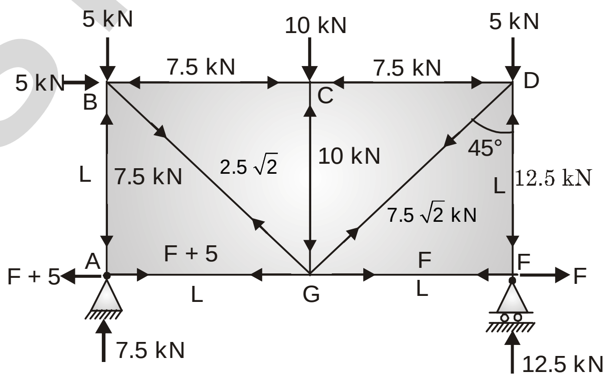
$$\Delta_B = \frac{\partial U}{\partial P} = \sum \frac{F\frac{\partial F}{\partial P}}{AE} . L$$

$$= \frac{(P+250) \times 1 \times 10^3 \times 10^3}{50 \times 2 \times 10^5}$$

$$= \frac{250 \times 1 \times 10^3 \times 10^3}{50 \times 2 \times 10^5} \quad (\because P = 0)$$

$$= 2.5 \text{ mm}$$

38. (c)



For getting horizontal deflection at F, and load, F in the horizontal direction is applied.

$\Sigma M_A = 0$

$\Rightarrow$ 5L + 10 L + 5 × 2L = 2L R_F

$\Rightarrow$ $R_F = \frac{25}{2} = 12.5$ kN

$\therefore$ $R_A = 20 - 12.5 = 7.5$ kN

Using castigliano's method,

$$\Delta F = \sum_{i=1}^{n} p_i \frac{\partial p_i}{\partial F} \cdot \frac{l_i}{A_i E_i} \quad \text{... (i)}$$

Where p_i in member forces in the i^{th} member due to combined actrion of external forces and F. Afterward, setting F = 0 in equation

(i) $\Delta_F = \sum\left(p_i \dfrac{\partial p_i}{\partial F} \cdot \dfrac{l_i}{A_i E_i}\right)_{at\ F=0}$

Sign convention

Tensile force is taken as +ve and compressive force is taken as –ve. If deflection calculated in equation (i) is the , it is in the direction of applied load at the point of desired deflection. Otherwise opposite to the direction of applied bad.

Considering joint at D,

$F_{DG} \cos 45° + 5 = 12.5$

$\Rightarrow \quad F_{DG} = 7.5\sqrt{2}$ kN

$\therefore \quad F_{DC} = F_{DG} \sin 45° = 7.5$ kN

at joint B,

$F_{BG} \cos 45° + 5 = 7.5$

$\Rightarrow \quad F_{BG} = 2.5\sqrt{2}$

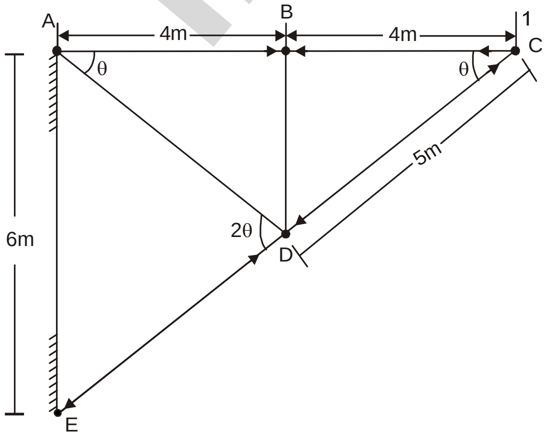
Thus, members having 'F' term involved in their internal forces are member AG and member GF. Thus these two members only will be involved in finding horizontal deflection at point F. Other remaining members have no. 'F' term involved in their internal forces and on differention w.r.t force F, will give zero term.

39. (d)

$\cos\theta = \dfrac{4}{5}$

$\sin\theta = 0.6$

$\theta = 36.87^o$



Fabrication error case (Lack of fit case)

Member lengths are generally made longer or shorter than actual required length in order to introduce member forces which will compensate for deflection due to the dead load.

Due to this lack of fit, joint deflection from original level will take place. This joint deflection in this case is calculated using virtual work principle.

$1.\Delta = \sum u_i dL_i$

Where dL_i = fabrication error in i^{th} member

Sign connection

dL_i = +ve if member is longer than normally expected

dL_i = –ve if member is shorter than normally expected

Tension = the

Compression = –ve

Δ = +ve if deflection in the direction of applied unit load

Δ = –ve if deflection opposite to the direction of applied unit load.

Here in question, vertical deflection is found outr at point c, a unit load at C in vertical direction is applied and member forces one found out.

At joint C,

$F_{CD} \sin\theta = 1 \Rightarrow F_{CD} = \dfrac{1}{0.6} = 1.667$

and FCD $\cos\theta = F_{CD} \Rightarrow F_{CB} = 1.33$

At joint B,

$F_{AB} = F_{BC} = 1.33$

At Joint D,

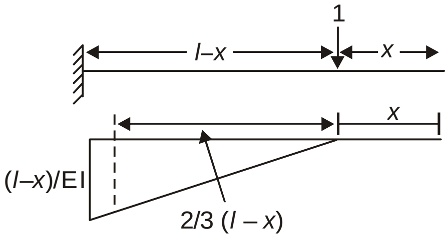
$F_{ED} = F_{CD} = 1.667$

Member	u	DL (mm)	UΔL(mm)
CD	–1.667	–10	+16.67
CB	+1.33	0	0
BD	0	0	0
AB	+1.33	0	0
AD	0	0	0
DE	–1.667	0	0

Σ = + 16.67 mm (down)

40. (b)

Let unit load be at a distance x from free end



$\frac{M}{EI}$ diagram

Influence line ordinate for deflection will

be $= \frac{A\bar{x}}{EI}$

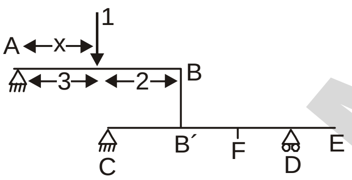
(moment about the free end)

$$y_B = \frac{1}{2}\times\frac{l-x}{EI}\times l-x\times\left(x+\frac{2}{3}(l-x)\right)$$

$$= \frac{(l-x)^2}{2EI}\times\frac{(2l+x)}{3}$$

$$\Rightarrow \quad \frac{(l-x)^2(2l+x)}{6EI}.$$

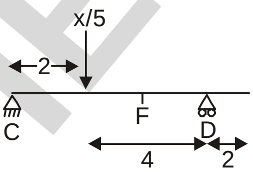
41. (b)



When load is on AB at a distance *x* from A

$R_B = x/5$

Now, $\sum M_c \Rightarrow R_D \times 6 = 2 \times R_B$



$$R_D = \frac{x}{15}$$

$$M_F = R_D \times 2$$

$$= \frac{x}{15}\times 2$$

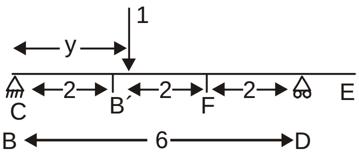
At $x = 0 \quad M_F = 0$

$x = 5 \quad M_F = + 2/3.$

Now, when load is on B´F at a distance y from C

$R_D \times 6 = 1 \times y$

$R_D = y/6$



$M_F = R_D \times 2 \Rightarrow y/3$

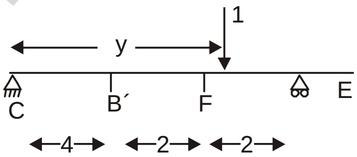
At $y = 2 \quad M_F = 2/3$

$y = 4 \quad M_F = 4/3.$

When load is in FE at 'y' distance from C

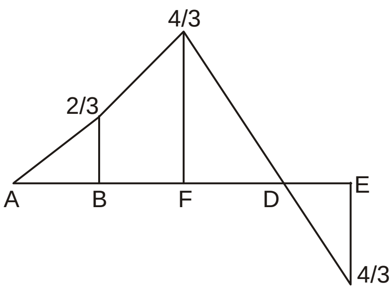
$M_F = R_D \times 2 - 1\ (y - 4)$

$$\Rightarrow \quad \frac{y}{6}\times 2-(y-4)$$



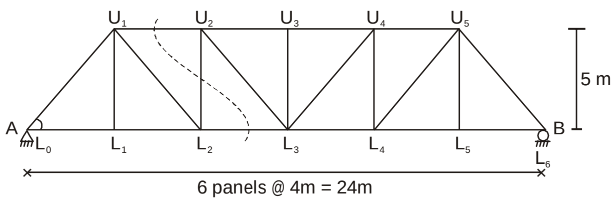
At $y = 6 \quad M_F = 0$

$$y = 8 M_F = \frac{8}{3}-(4)=-\frac{4}{3}.$$



42. (a)

Cut section as shown :



When unit load is at L_2 –

$$R_B = \frac{1}{3}, \quad R_A = \frac{2}{3}$$

$$\therefore \quad F_{L_2U_2} = \frac{1}{3}\ \text{(Tension)}$$

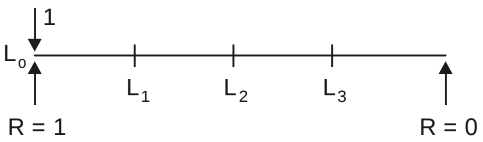
When unit load is at L_3 –

$$R_A = R_B = \frac{1}{2}$$

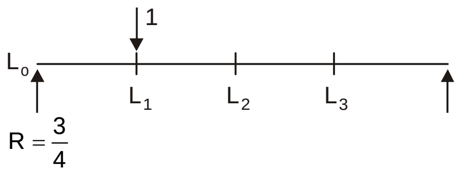
$$\therefore \quad F_{L_2U_2} = \frac{1}{2} \text{ (Compression).}$$

43. (c)

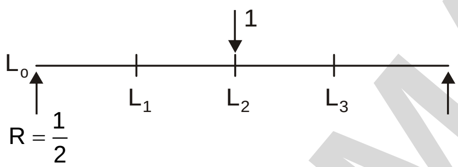
Shear in Panel L_oL_1



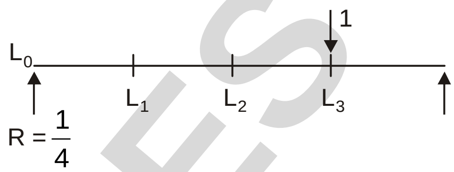
SF in Panel $L_oL_1 = 0$



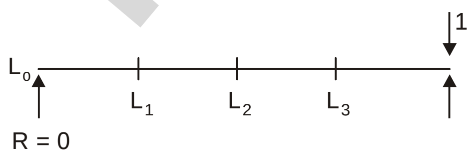
SF in Panel $L_oL_1 = \frac{3}{4}$



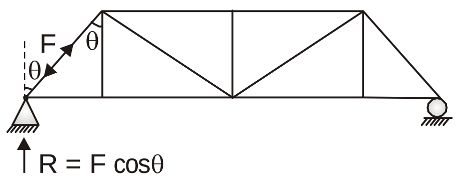
SF in Panel $L_oL_1 = \frac{1}{2}$



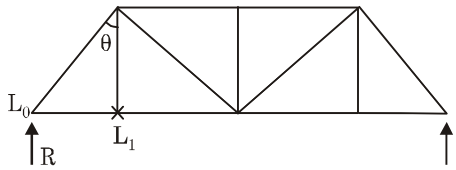
SF in Panel $L_oL_1 = \frac{1}{4}$



SF in Panel $L_oL_1 = 0$



$\Rightarrow$ $F = R\sec\theta$



Moment at $L_1 = R\ L_oL_1$

$= F\cos\theta\,.\ L_oL_1$

$$\boxed{\frac{M_{L_1}}{\cos\theta\, L_oL_1} = F}$$

$\Rightarrow$ ILD for force in member L_oU_1 is obtained bymultiplying the ordinate of ILD for shear in panel L_oL_1 by $\sec\theta$ also by dividing the ordinate of ILD for moment at L_1 by $\cos\theta \times L_oL_1$

44. (c)

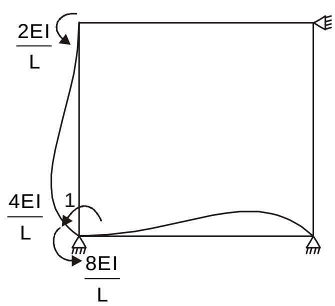
Muller Breslau theorem is valid for both statically determinate and indeterminate structures.

45. (a)

- An influence line represents the variation of either the reaction, shear, moment or deflection at a specified point in a member as a concentrated unit force moves over the member.
- Influence lines represent the effect of a moving load only at a specified point on a member whereas shear and moment diagram represents the effect of fixed loads at all points along the member.
- Thus influence line helps in deciding at a glance where should the moving loads be placed on the structure so that it creates greatest influence at the specified point.

46. (b)

for $\therefore$ $K_{11} = \frac{4EI}{L} + \frac{4EI}{L} = \frac{8EI}{L}$ & $K_{21} = \frac{2EI}{L}$



Hence option (b) is correct.

47. **(c)**

48. **(d)**

$$K_{11} = \frac{4E(2I)}{4} + \frac{4EI}{4} + \frac{4E(1.5I)}{3} + \frac{4EI}{4} = 6EI$$

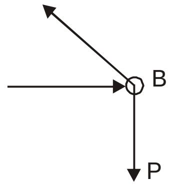
$$K_{21} = \frac{2E(2I)}{4} = EI$$

$$K_{12} = K_{21}$$

49. **(a)**

50. **(c)**

For equilibrium of point B, the forces must be balanced. Drawing reactions at point B



For balanced forces, member CB should be under tension while member AB should be under compression as shown in above figure.

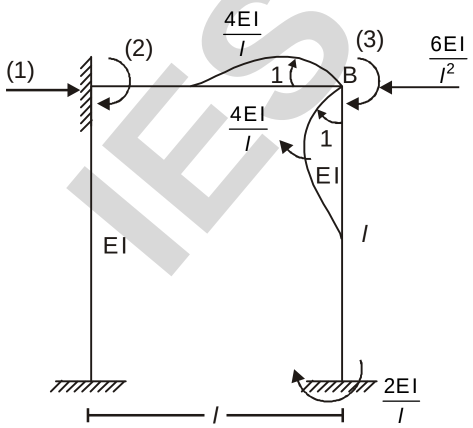
Hence, neither bar AB nor BC is subjected to bending.

51. **(b)**

52. **(c)**

53. **(b)**

54. **(d)**



$$K_{23} = \frac{2EI}{l}$$

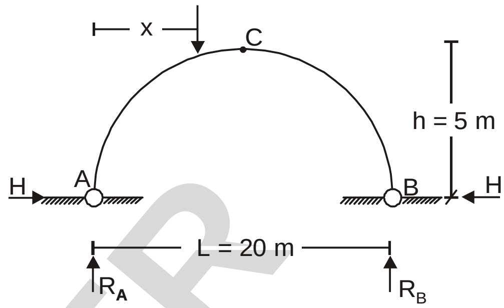
$$K_{33} = \frac{4EI}{l} + \frac{4EI}{l} = \frac{8EI}{l}$$

$$K_{13} = -\frac{6EI}{l^2}$$

55. **(c)**

56. **(b)**

57. **(c)**



$$\sum M_C\big|_{right} = 0 \Rightarrow 10R_B = 5H$$

$$\Rightarrow \quad H = 2R_B \quad \text{... (1)}$$

$$\sum M_C\big|_{left} = 0$$

$$\Rightarrow \quad 10R_A = 5H + P(10 - x) \quad \text{...(2)}$$

From (1) and (2)

$$10R_A = 10R_B + P(10 - x) \quad \text{...(3)}$$

$$R_A + R_B = P \quad \text{... (4)}$$

From (3) and (4)

$$10R_A = 10(P - R_A) + P(10 - x)$$

$$\text{or} \quad 20R_A = 10P + 10P - Px$$

$$= 20P - Px$$

$$= P(20 - x)$$

$$\therefore \quad R_A = \frac{(20-x)}{20}P$$

Again, $\because$ $10R_A = 5H + P(10 - x)$

$$\therefore \quad \frac{(20-x)P - 2P(10-x)}{2} = 5H$$

$$\text{or} \quad \frac{(20-x)P - P(20-2x)}{2} = 5H$$

$$\text{or,} \quad 5H = \frac{xP}{2}$$

$$\Rightarrow \quad H = \frac{xP}{10}$$

By question,

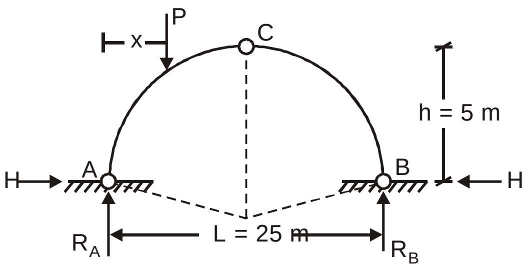
$$\because \quad \frac{R_A}{H} = \frac{2}{1}$$

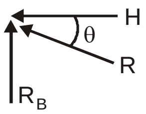
$$\Rightarrow \quad \frac{(20-x)P}{20} = \frac{2xP}{10}$$

$$\Rightarrow \quad 20 - x = 4x$$

$$\Rightarrow \quad 5x = 20 \Rightarrow x = 4\text{m}$$

58. (c)





$$\sum M_C\big|_{right} = 0 \Rightarrow 12.5\, R_B = 5H$$

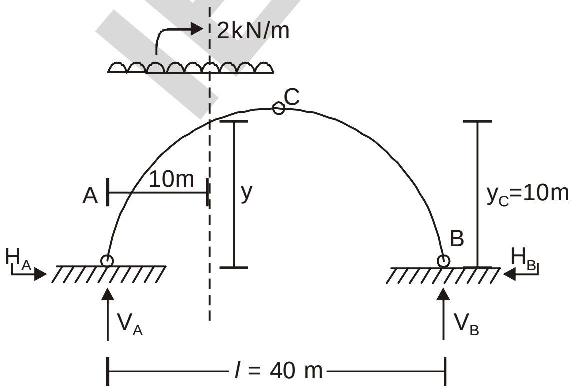
$$\Rightarrow \quad H = 2.5\, R_B \quad \text{... (i)}$$

By question;

$$\tan\theta = \frac{R_B}{H} = \frac{R_B}{2.5R_B} = \frac{1}{2.5}$$

$$\Rightarrow \quad \tan\theta = 0.4$$

59. (c)



$$\Sigma M_A = 0$$

$$\Rightarrow \quad \frac{2\times 20^2}{2} - 40V_B = 0$$

$$\Rightarrow \quad V_B = 10 \text{ kN}$$

$$\therefore \quad V_A = 20 \times 2 - 10 = 30 \text{ kN}$$

$$\Sigma M_C = 0$$

$$\Rightarrow V_A \times 20 - H_A \times 10 - 2 \times \frac{20^2}{2} = 0$$

or $30 \times 20 - H_A \times 10 - 20^2 = 0$

$$\Rightarrow \quad H_A = 20 \text{ kN}$$

$$\therefore \quad H_A = H_B = 20 \text{ kN}$$

<u>Bending moment at x = 10 m from A</u>

$$y = \frac{4y_c x(l-x)}{l^2}$$

$$y = \frac{4\times 10}{40^2} \times 10\,(40-10)$$

$$= 7.5\text{m at 10 from A.}$$

$$\therefore \quad BM = +V_A \times 10 - H_A \cdot y - 2\times\frac{10^2}{2}$$

$$= 30\times10 - 20\times7.5 - \frac{2\times10^2}{2}$$

$$= 50 \text{ kNM}$$

If θ be the inclination of any point x on the centre line of an arch, then the vertical shear V_x and horizontal thrust H would combine to produce a radial shear R_x and a normal thrust N_x at the section.

Then F_x (or) $R_x = V_x\cos\theta - H\sin\theta$

and P_x (or) $N_x = V_x\sin\theta + H\cos\theta$

Radial shear force at x = 10m

R = Radial shear force = $V\cos\theta - H\sin\theta$

where V = Net vertical shear force at x = 10 m from A

H = Horizontal thrust

$$\tan\theta = \frac{dy}{dx} = \left(\frac{4y_c}{\ell^2}(\ell - 2x)\right)$$

$$\therefore \quad \theta = \tan^{-1}\left(\frac{4\times10}{40^2}(40-2)\times10\right)$$

$$= \tan^{-1}(0.5) = 26.57°$$

$$V = V_A - \omega\ell/4 = 30 - \frac{2\times 40}{4} = 10\,kN$$

$\therefore$ Radial shear force,

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

$$= 10 \times \cos 26.57° - 20$$

$$\sin 26.57° = 0$$

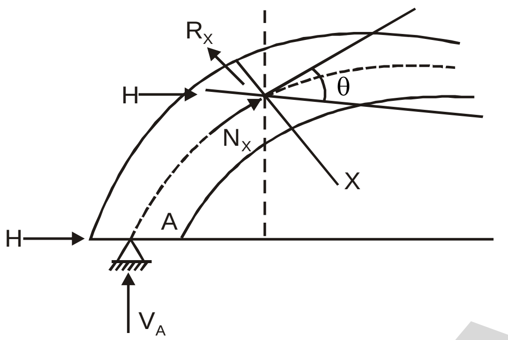
Normal thrust at x = 10 m from A

Normal thrust,

$$N = V\sin\theta + H\cos\theta$$

$$= 10 \sin 26.57° + 20 \cos 26.57°$$

$$= 22.36\ kN$$



If θ be the inclination of any point x on the centre line of an arch, then the vertical shear V_x and horizontal thrust H would combine to produce a radial shear R_x and a normal thrust N_x at the section.

Then F_x (or) $R_x = V_x\cos\theta - H\sin\theta$

and P_x (or) $N_x = V_x\sin\theta + H\cos\theta$

60. (c)

The behaviour of a structure subjected to horizontal forces depends on its height to width ratio. The deformation in low-rise structures where the height is smaller than its width, is characterised predominantly by shear deformation. In high rise building where height is several times greater than its lateral dimensions, is dominated by bending action. There are two methods-namely portal method and canti lever method to analyse the structures subjected to horizontal loading.

Portal Method :

The portal method is an approximate analysis for analysing building frames subjected to lateral loads such as wind loads/ seismic forces.

Since, shear deformations are dominant in low rise structures, the method makes simplyfying assumptions regarding horizontal shear in columns. Each bay of a structure is treated as a portal frame and horizontal force is distributed equally among them.

Assumptions in Portal Method :

1. The points of inflection are located at the mid-height of each column above the first floor. If the base of the column is fixed, the point of inflection is assumed at mid heght of the ground floor columns as well otherwise, it is assumed at the hinged column base.
2. Points of inflection occur at mid span of beams.
3. Total horizontal shear at any floor is distributed among the columns of that floor such that the exterior columns carry half the force carried by the inner columns.

   The basis of this third assumption in the frame is composed of individual portals having one bay only.

Cantilever Method :

This method is applicable to high rise structures. This is based on the simplyfing assumptions regarding the axial force in columns.

Assumptions in Cantilever Method

1. The points of inflection are located at the mid-height of each column above the first floor. If the base of the column is fixed, the point of inflection is assumed at mid height of the groun d floor columns as well. Otherwise it is assumed at the hinged column base.
2. Points of inflection occur at mid span of beams.
3. The basic assumption of the method can be stated as the axial force in the column at any floor is linearly proportional to its

distance from the centroid of all the columns at that level.

61. (b)

62. (d)

63. (b)

64. (d)

- Slope deflection method is a stiffness method in which unknown joint displacement are found out by applying the equilibrium condition at end .
- In slope deflection equation, we use the principle of superposition by considering separately the moments developed at each support due to each of the displacement θ_A, θ_B, Δ and then the loads, so displacement at joint are independent.

65. (a)

Kani's method

Gaspar Kani's method of structural analysis is similar to cross moment distribution in that both these methods use Gauss-Seidell iteration procedure to solve the slope deflection equation without explicitly writting them down. However, whereas the moment distribution method obtains the unknowns (i.e., the end moments of the structural members) by iterating their increments, Kani's method iterates these unknowns themselves. This method essentially consists of a single, simple numerical operation performed repeatedly by the joints of a structure in a chosen sequence. Results of any desired accuracy may be obtained by peforming this operation a sufficient number of times using the required number of significant digits. Kani's method is specially useful for the analysis of multistorey frames. It has the advantages of simplicity, speed, economy of time, labour and space and of accuracy. However, perhaps, the two most attractive features of this method are as follows:

- It has a built-in error elimination so that computational errors automatically disappear in subsequent operations. This also makes possible the introduction of any changes in loads or member lengths that may become necessary during calculations without necessitating a new analysis. Such changes are inserted in the computational scheme wherever required and the analysis simply continued.
- It requires only one table of calculations even for highly irregular frames with multiple side sway. Compared to most other methods, Kani's method involves substantially less labour and time in the analysis of such frames.

The above advantages make Kani's method one of the most powerful techniques applicable to all types of continuous beams and frames.

Moment distribution method.

This method consists of solving slope deflection equations by successive approximation that may be carried out to any desired degree of accuracy. Essentially, the method begins by assuming each joint of a structure is fixed. Then by unlocking and locking each joint in succession, the internal moments at the joints are distributed and balanced until the joints have rotated to their final or nearly final positions. This method of analysis is both repetitive and easy to apply. Manual analysis of gable frames mostly uses the moment distribution or slope deflection methods. These methods are usually lengthy and have no built-in-error elimination capability.

66. (a)

67. (b)

68. (a)

69. (a)

70. (c)

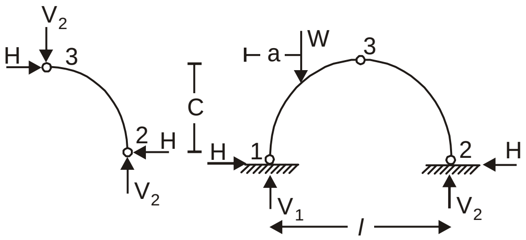
71. (b)

72. (a)

73. (d)

74. (c)

For a three-hinged arch subjected to vertical loads only, the horizontal support reactions at the arch springings are equal and oppoiste and act inward. The vertical support reactions at the arch springings are equal to those of a simply supported beam of identical length with identical loads.



For the above three hinged arch, the vertical reaction at support 2 is obtained by considering moment eqilibrium about support I.

Hence $M_1 = 0 = lV_2 - Wa$ and $V_2 = wa/l$

$\Rightarrow V_1 = W - V_2 = w(1 - a/l)$.

These values for V_1 and V_2 are identical to the reactions of a simply supported beam of the same span as the arch with the same applied load W.

The horizontal thrust at the springings is determined from a free body diagram of the right half of the arch. Considering moment equilibrium about the crown hinge at 3;

$$M_3 = 0 = lV_2/2 - HC \Rightarrow H = \frac{l}{2C} \times V_2$$

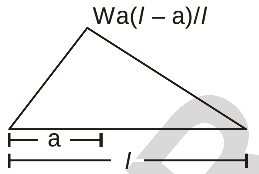
This value for H is identical to the bending moment at the centre of a simply supported beam of the same length with the same applied load W multplied by 1/C. The bending moment in the arch at any point a distance x from the left support is given by

$M_x = V_1 x - H.y$ for $x \le a$

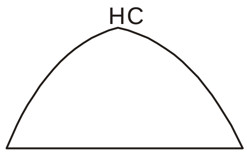
$= V_2(l - x) - Hy$ for $a < x \le l$

where y is the height of the arch at a distance x from the left support.

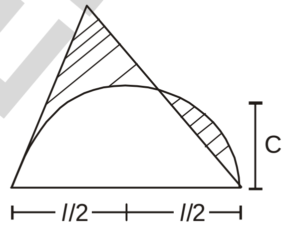
The expressions for bending moment may be considered as the superposition of the bending moment of a simply supported beam of the same span with the same applied load W plus the bending moment due to the horizontal thrust.



BM due to applied load on a S.S. beam



BM due to horizontal thrust



BM of the three hinged arch

75. (a)

Method of joint

In a planar-truss, at every point there are two conditon of equilibrium

$\Sigma F_x = 0$ and ● = 0

Since all the members at a joint are assumed to pass through a single point, moment about the joint will always be zero. Hence, $\Sigma M = 0$ will not be of any consequence. Thus two unknowns can be found out from two equilibrium equations. Likewise, we can proceed to other joints and find out member forces by using equilibrium equation if no. of unknown forces at the joint are at most two in number.