Important Instruction to Examiners:-

1) The answers should be examined by key words & not as word to word as given in the model answers scheme.

2) The model answers & answers written by the candidate may vary but the examiner may try to access the understanding level of the candidate.

3) The language errors such as grammatical, spelling errors should not be given more importance.

4) While assessing figures, examiners, may give credit for principle components indicated in the figure. The figures drawn by candidate & model answer may vary. The examiner may give credit for any equivalent figure drawn.

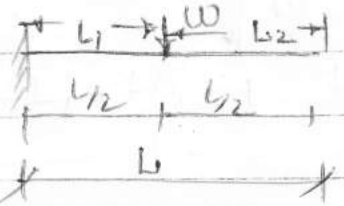
5) Credit may be given step wise for numerical problems. In some cases, the assumed contact values may vary and there may be some difference in the candidate's answers and model answer.

6) In case of some questions credit may be given by judgment on part of examiner of relevant answer based on candidates understanding.

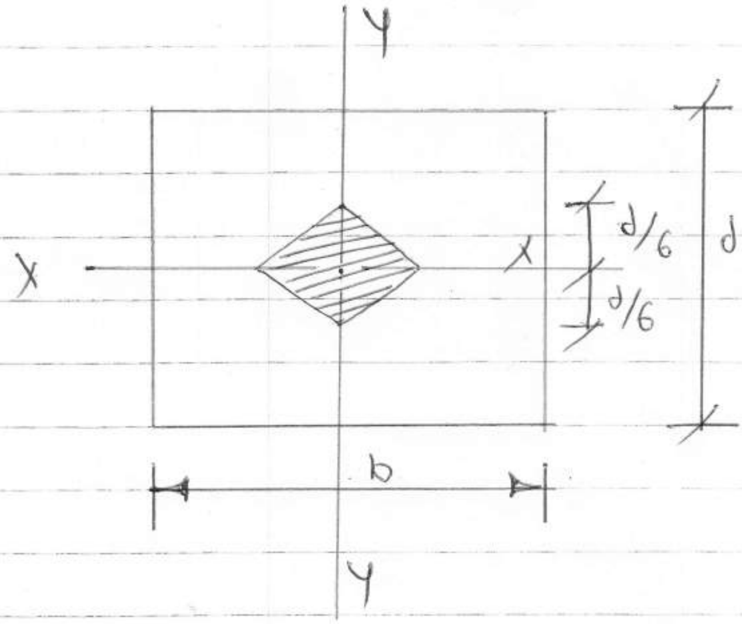
7) For programming language papers, credit may be given to any other programme based on equivalent concept.

Important notes to examiner

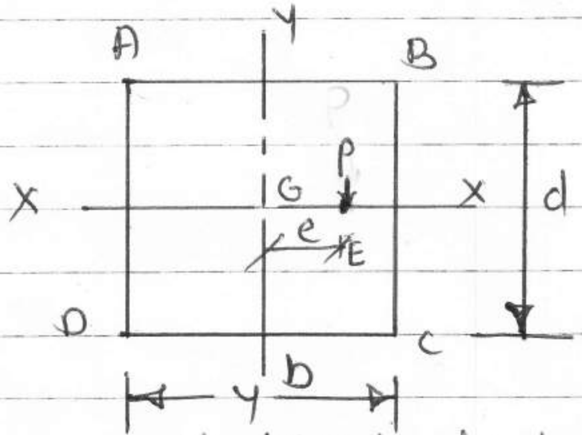
Q.NO	SOLUTION	MARKS
Q-1	a) i) Define axial & eccentric load	
	a) Define axial load →	
	When a load whose line of action coincides with the axis of a member or whose line of action acts at a centroid of a section of member, then it is called as an axial load.	01M
	b) Eccentric load →	
	A load acts away from the centroid of the section or a load whose line of action do not coincide with the axis of member is called as eccentric load.	01M
	ii) a) maximum slope & deflection in case of simply supported beam loaded with central point load.	
	a) maximum slope	
*	$\theta_A = \theta_B = \frac{WL^2}{16EI}$ radians	01M
	b) maximum deflection	
	$y_{max} = y_c = -\frac{WL^3}{48EI}$	01M
	iii) a) slope at free end of cantilever beam with entire udl on span	
	$\theta_B = \frac{wL^3}{6EI}$ (w = udl over entire span)	01M

Q .NO	SOLUTION	MARKS
	b) Deflection at free end of cantilever beam with entire udl on span.	
	$$y_B = \frac{-wL^4}{8EI}$$	01 M
iv)	A cantilever of span L carries a point load 'W' at L/2 from fixed end. State deflection at free end in terms of EI	
		
	$y_B = y_c + (L - L_1)\theta_c$	1 M
	$y_B = \frac{w}{3EI}\left(\frac{L}{2}\right)^3 + \frac{L}{2}\,\frac{w}{2EI}\left(\frac{L}{2}\right)^2$	½ M
	$\boxed{y_B = \frac{5}{48}\frac{WL^3}{EI}}$	½ M
v)	State any two disadvantages of fixed beam	
	a) If any one of the support sinks to a small extent, it induces additional moment at each end.	1 M for each write Any TWO
	b) Since the both the end of beam are fixed, temperature stresses are developed due to variation in temperatures	
	c) The end of fixed end can be disturbed due to frequent fluctuations in loading.	

Q.NO	SOLUTION	MARKS
	vi) Define distribution factor	
	→ The distribution factor for a member at a joint is the ratio of stiffness factor for that member and the total stiffness of all the members meeting at a joint.	02M
	vii) Define carry over moment.	
	The moment induced at the fixed end of a beam by the action of the moment applied at the other simply supported or hinged end is called as carry over moment.	02M
	viii) Enlist four types of simple frames	
	a) simple fink truss	$\frac{1}{2}$M
	b) compound fink or french truss	for
	c) simple fan truss	each
	d) pratt truss	write
	e) Howe truss	any
	f) North light roof truss	four
	g) Queen-post truss	
	h) King-post truss	

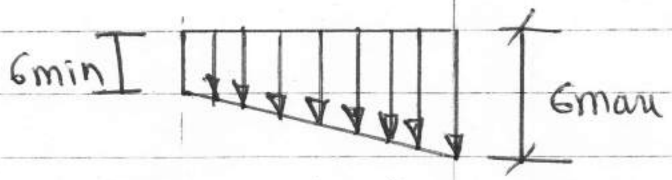
Q.NO	SOLUTION	MARKS
Q-b (i)		1 M
	eccentricity parallel to 'b' $e \leq b/6$	½ M
	eccentricity parallel to d $e \leq d/6$	½ M
	Incase of rectangular sections for No tension is to be produced any where in section, the enternal load should acts with in the middle one third portion of section	1 M

Q .NO	SOLUTION	MARKS
Q-1 b	iii) Draw stress distribution diagram.	



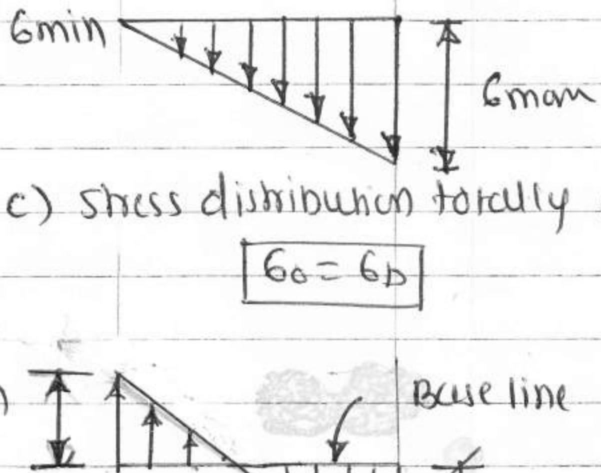
a) Eccentric load at E

1M



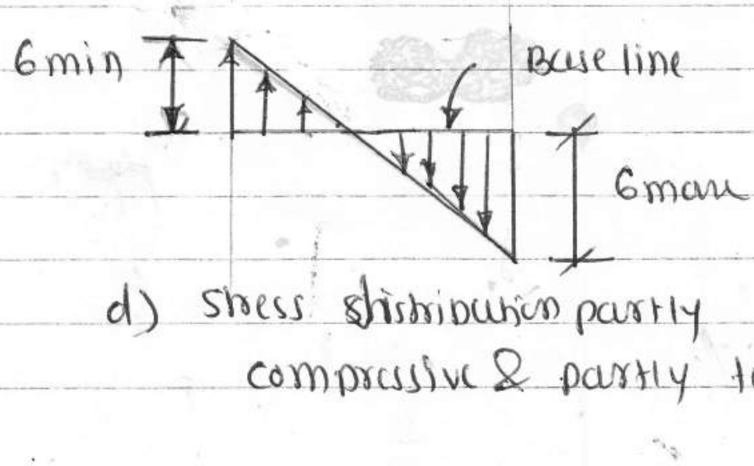
b) stress distribution completly compressive since $\sigma_0 > \sigma_b$

1M



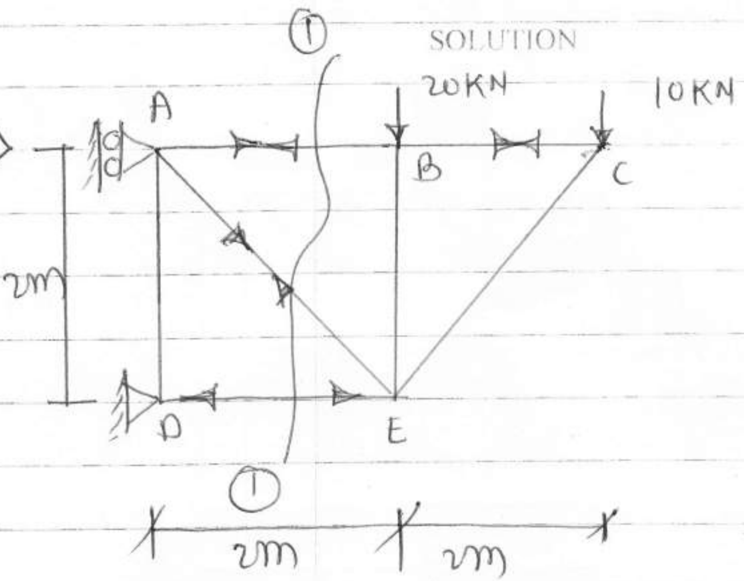
c) stress distribution totally compressive $\boxed{\sigma_0 = \sigma_b}$

1M



d) stress distribution partly compressive & partly tensile $\boxed{\sigma_0 < \sigma_b}$

1M

Q.NO	SOLUTION	MARKS
Q-1 b iii)		1M
	let us consider equilibrium of right part of section.	
	Taking moment @ A $\Sigma M_A = 0$	½M
	$10 \times 4 + 20 \times 2 - f_{DE} \times 2 = 0$	
	$2 \times f_{DE} = 80$	
	$f_{DE} = 40$ KN --- (compressive)	1M
	Taking moment @ E	
	$\Sigma M_E = 0$	½M
	$10 \times 2 + 20 \times 0 - f_{BA} \times 2 = 0$	
	$f_{BA} \times = 20$	
	$f_{BA} = 10$ KN —— (tensile)	1M

Q.NO	SOLUTION	MARKS

Q-2 (a) A square column 150mm side carried a load of 150 KN at an eccentricity of 50mm. find σ_{max} & σ_{min}

→ $b = \text{width} = 150\,mm$ $\quad d = \text{thickness} = 150\,mm$

$P = 150\,KN = 150 \times 10^3\,N$

$e = 50\,mm$

i) $A = b \times d$

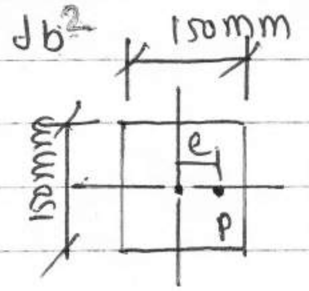
$= 150 \times 150 = 22.5 \times 10^3\,mm^3$ — ½ M

ii) σ_0 = Direct stress

$\sigma_0 = \dfrac{P}{A} = \dfrac{150 \times 10^3}{22.5 \times 10^3} = 6.67\,N/mm^2\ (c)$ — 01 M

iii) σ_b = Bending stress

$\sigma_b = \dfrac{M}{Z_{yy}} = \dfrac{P \times e}{\frac{db^2}{6}} = \dfrac{6Pe}{db^2}$ — 01



$\sigma_b = \pm \dfrac{6 \times 150 \times 10^3 \times 50}{150 \times 150^2}$

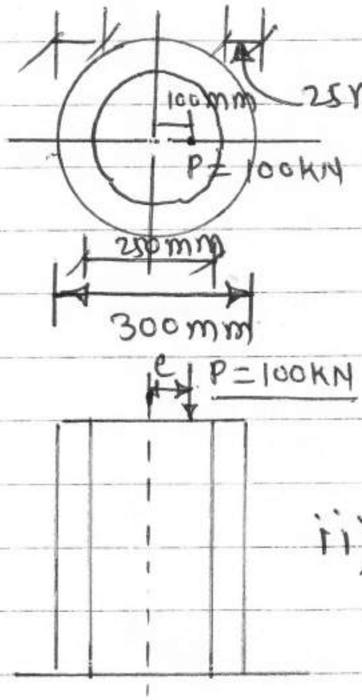
$\sigma_b = \pm \dfrac{45 \times 10^6}{3.375 \times 10^6}$

$\sigma_b = \pm\ 13.33\,N/mm^2$ — ½ M for σ_b

$\sigma_{max} = \sigma_0 + \sigma_b$ $\qquad$ $\sigma_{min} = \sigma_0 - \sigma_b$

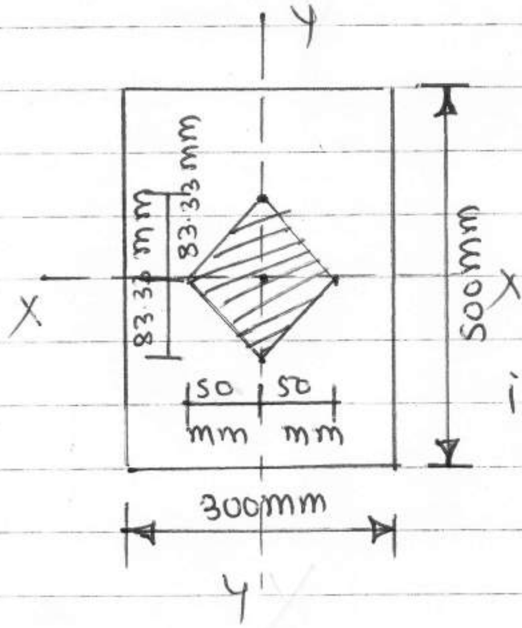
$= 6.67 + 13.33$ $\qquad$ $= 6.67 - 13.33$

$\boxed{\sigma_{max} = 20\ N/mm^2}$ $\qquad$ $\boxed{\sigma_{min} = -6.67\ N/mm^2}$ — ½ for σ_{max} & ½ M for σ_{min}

Q .NO	SOLUTION	MARKS
Q-2 (b)	A hollow circular steel column having external dia 300 mm & thickness 25 mm carries an eccentric load of 100 kN acting at an eccentricity of 100 mm. Calculate the maximum & minimum stresses	
		
	i) given data : $D = 300\ mm$ thickness = 25 mm $d = 250\ mm$ $P = 100\ kN$ $e = 100\ mm$	
	ii) $A = \frac{\pi}{4}(D^2 - d^2) = \frac{\pi}{4} \times (300^2 - 250^2)$ $A = 21.598 \times 10^3\ mm^2$	½M
	iii) $\sigma_0 = \frac{P}{A} = \frac{100 \times 10^3}{21.598 \times 10^3} = 4.629\ N/mm^2\ (C)$	01M
	iv) $\sigma_b = \frac{M}{Z} = \frac{P \times e}{I/y_{max}} = \frac{P \times e}{\frac{\frac{\pi}{64}(D^4 - d^4)}{D/2}}$	½M
	$\sigma_b = \frac{P \times e}{\frac{\pi}{32}\left(\frac{D^4 - d^4}{D}\right)} = \frac{100 \times 10^3 \times 100}{\frac{\pi}{32}\left(\frac{300^4 - 250^4}{300}\right)}$	
	$\sigma_b = 7.286\ N/mm^2$	01M
	$\sigma_{max} = \sigma_0 + \sigma_b = 4.629 + 7.286 = 11.92\ N/mm^2\ (C)$	½M
	$\sigma_{min} = \sigma_0 - \sigma_b = 4.629 - 7.286 = 2.657\ N/mm^2\ (T)$	½M

Q .NO	SOLUTION	MARKS

Q-2 (c) calculate core of section for rectangular c/c 300mm x 500mm in size and draw correct sketch.



i> given
$b = 300\,mm$
$d = 500\,mm$ — 1 M for Fig

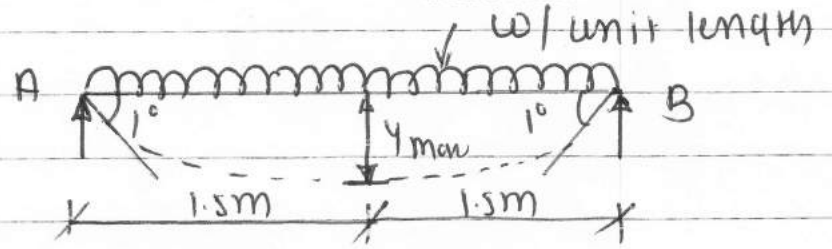
ii> limit of eccentricity
$(e_x) = \frac{d}{6}$ w.r.t axis — ½ M

$e_x = \frac{500}{6} = \underline{83.33\,mm}$ — 1 M

iii> limit of eccentricity
$e_y = b/_6$ w.r.t. y-axis — ½ M
$e_y = 300/_6 = \underline{50\,mm}$ — 1 M

Q-2 (d) A beam of span 3m is simply supported and carries udl 'w' per unit length, if the slope at the end is not to exceed 1° find maximum deflection

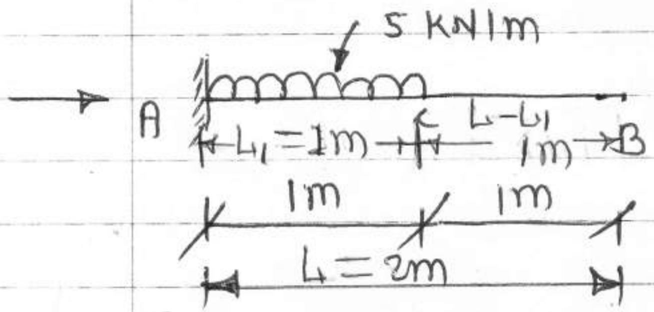
→ given data
i> span = 3m
ii> udl = w/unit length
iii> slope at end not exceed 1°

Q .NO	SOLUTION	MARKS
		
	θ = slope at end = $1^\circ = \left(1 \times \frac{\pi}{180}\right)$ radius $= 0.017$ rad	½M
	θ = Slope at end $= \frac{wL^3}{24EI}$	½M
	$0.017 = \left(\frac{w}{EI}\right) \times \frac{L^3}{24}$	
	$\frac{0.017 \times 24}{L^3} = \frac{w}{EI}$	
	$\frac{0.017 \times 24}{3^3} = \frac{w}{EI}$	
	$0.0151 = \frac{w}{EI}$	
	$\therefore \frac{w}{EI} = 0.0151$ —— (1)	01M
	To find maximum deflection (y_{max}) Manium deflection for simply supported beam entirely loaded by udl is given by following expression	
	$y_{max} = -\frac{5wL^4}{384EI}$ $y_{max} = \frac{-5L^4}{384} \times \left(\frac{w}{EI}\right)$ —— (2)	01M
	put eq^n (1) in eq^n (2)	
	$y_{max} = \frac{-5 \times 3^4}{384} \times 0.0151 = -0.0159m$	
	$y_{max} = -15.9$ mm (−ve sign indicate downward deflection.	01M

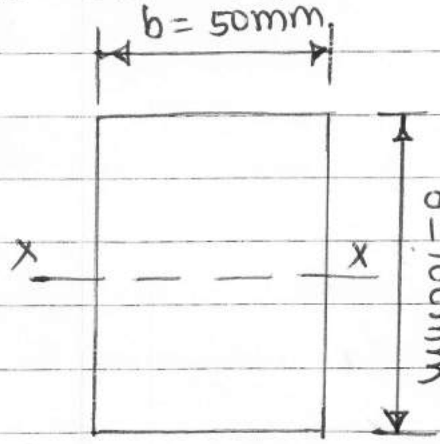
$\boxed{y_{max} = -15.9mm}$

Q .NO	SOLUTION	MARKS

Q-2 (e) A cantilever of length 2m carries a udl of 5 KN/m over half the span from the fixed end. if the section is rectangular 50mm wide & 100mm deep. find the slope & deflection at free end $E = 2\times10^5$ Mpa.



(a) cantilever beam



(b) section

from the standard case of this type of loading on cantilever we know

$$y_B = \frac{wL_1^4}{8EI} + (L - L_1)\frac{wL_1^3}{6EI}$$ 01 M

y_b is Deflection at free end.

$L_1 = 1m = 1000mm$ $L = 2m = 2000mm$

$$I = I_{xx} = \frac{bd^3}{12} = \frac{50\times100^3}{12} = 4.166\times10^6 mm^2$$ 01 M

i) To find deflection at free end B

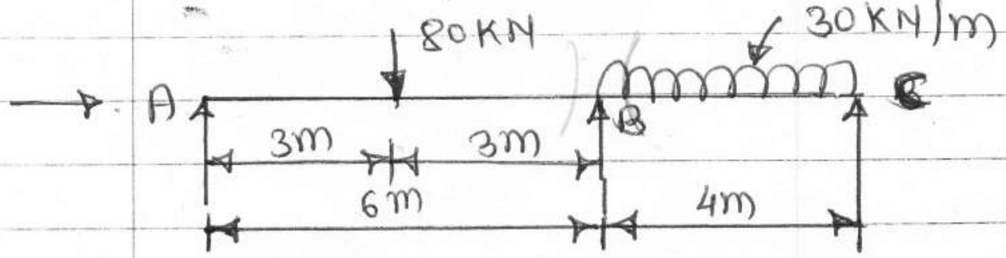
$$y_B = \frac{0.005\times(1000)^4}{8\times200\times4.166\times10^6} + \frac{(2000-1000)\times0.005\times1000^3}{6\times200\times4.166\times10^6}$$

$$\boxed{y_B = 1.750\ mm}$$ 01 M

ii) To find slope at free end B i.e. θ_B

$$\theta_B = \frac{wL_1^3}{6EI} = \frac{0.005\times1000^3}{6\times200\times4.166\times10^6}$$ 01 M

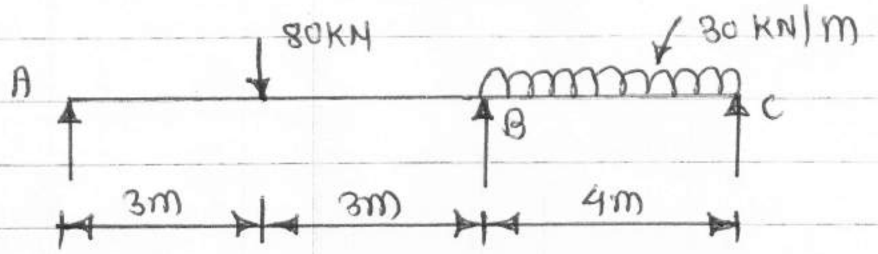
$$\boxed{\theta_B = 1.00\times10^{-3}\ \text{radian}}$$

Q .NO	SOLUTION	MARKS
Q-2-F)	A continuous beam ABC of uniform M.I and carries a central point load 80 KN on span AB. A udl of 30 KN/m is acting over the entire span BC. AB = 6m, BC = 4m. A & C are are simply support. calculate the support moment using three moments theorem only and draw an SFD.	
		
	i> Assume span AB & BC to be simply supported and find free B.M.	
	for span AB	
	$L_1 = 6m \quad W = 80 KN \quad M_A = 0 \quad M_C = 0$	
	$M_{max} = \frac{WL}{4} = \frac{80 \times 6}{4} = 120 KN\text{-}m$	½M
	for span BC	
	$L_2 = 4m \quad w = 30 KN/m$	
	$M_{max} = \frac{wL_2^2}{8} = \frac{30 \times 4^2}{8} = 60 KN\text{-}m$	½M

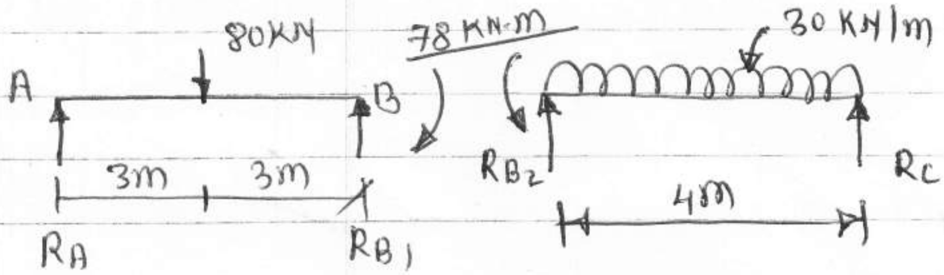
Q.NO	SOLUTION	MARKS
	from above value of free B.M draw the μ-diagram	
	step-2 To find the support moment A, B & C	
	for span AB & BC	
	a_1 = area of μ-diagram of span AB	
	$= \frac{1}{2} \times 6 \times 120 = 360$ KN-m	
	$\bar{x}_1$ = centroidal distn of μ-diagram from A	
	$= 6/2 = 3m.$	
	$\therefore a_1\bar{x}_1 = 360 \times 3 = 1080$	
	a_2 = area of μ-diagram of span BC $= \frac{2}{3} \times 4 \times 60$	
	$\bar{x}_2$ = centroidal distn of μ-dia ~~of B~~ from C	
	$= 4/2 = 2m$	
	$a_2\bar{x}_2 = (\frac{2}{3} \times 4 \times 60) \times 2 = 320$	
	apply clapeyron's theorem of three moments for span AB & BC	
	$M_A L_1 + 2M_B(L_1+L_2) + M_C L_2 = -\left(\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right)$	(1/2M)
	$0 \times 6 + 2M_B(6+4) + 0 \times 4 = -\left(\frac{6 \times 1080}{6} + \frac{6 \times 320}{4}\right)$	
	$2M_B \times 10 = -1080 - 480$	
	$\boxed{M_B = -78 \text{ KN-m}}$ (-ve sign indicate hogging B.M.)	(1/2M)

Q .NO	SOLUTION	MARKS

S.F.D. calculation



Draw the F.B.D for the span AB & BC



$\Sigma M_B = 0$ for span AB

$R_A \times 6 + 78 - 80 \times 3 = 0$

$6R_A + 78 - 240 = 0$

$6R_A = 162$ $\boxed{R_A = 27\ KN}$ (½M)

$\Sigma M_B = 0$ for span BC

$-R_C \times 4 + 30 \times 4 \times 2 - 78 = 0$

$-R_C \times 4 + 240 - 78 = 0$

$-R_C \times 4 + 162 = 0$

$\therefore \boxed{R_C = 40.5\ KN}$ (½M)

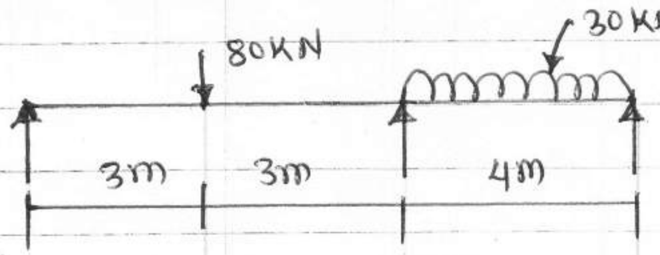
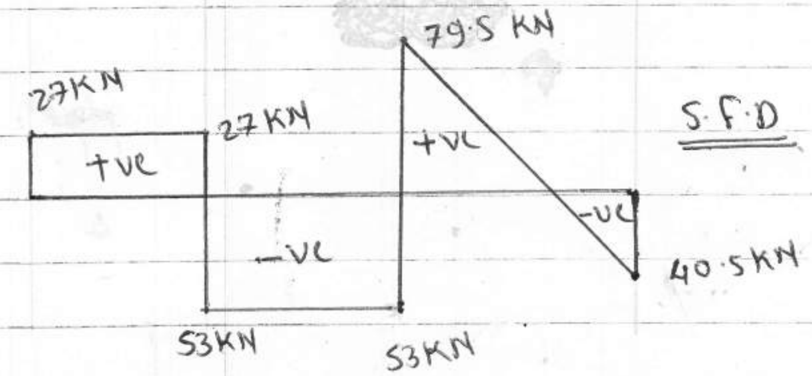
for continuous Beam ABC

$\Sigma F_y = 0$ gives

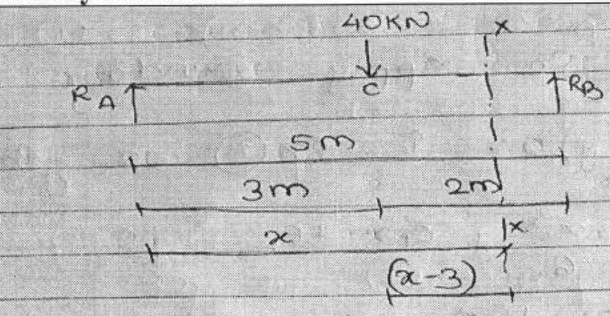
$R_A + R_B + R_C = 200$

$27 + R_B + 40.5 = 200$

$\boxed{R_B = 132.5\ KN}$ (½M)

Q .NO	SOLUTION	MARKS
	or	
	R_B can also be calculated in the following manner	
	$\Sigma M_A = 0$ for span AB gives	
	$80 \times 3 + 78 = R_{B1} \times 6$	
	$\boxed{R_{B1} = 53 KN}$	
	$\therefore \Sigma M_C = 0$ for span BC gives	
	$R_{B2} \times 4 = 78 + (30 \times 4 \times 4/2)$	
	$R_{B2} \times 4 = 78 + 240$	
	$R_{B2} \times 4 = 318$	
	$\boxed{R_{B2} = 79.5 KN}$	
	$\therefore R_B = R_{B1} + R_{B2} = 53 + 79.5 = \underline{132.5 KN}$	
	plot S.F. diagram	
		
	 S.F.D	(1/2 M)

Q .NO	SOLUTION	MARKS
3	Attempt any Four	16
a)	**A simply supported beam of span 5m carries a point load of 40KN at 3m from left support. Calculate the slope and deflection under point load in terms of EI. Use Macalay's Method.**	



$\Sigma F_y = 0$ $\quad R_A + R_B = 40$

$\Sigma M_A = 0$ $\quad R_B \times 5 = 40 \times 3$

$R_B \times 5 = 120$

$\boxed{R_B = 24\,KN}$

$R_A + R_B = 40$

$R_A = 40 - 24$

$\boxed{R_A = 16\,KN}$ — $\frac{1}{2}$M

consider a section x-x at a distance x from support A.

$m_x = R_A \times x - 40(x-3)$

but $m_x = EI\frac{d^2y}{dx^2}$

$\therefore EI\frac{d^2y}{dx^2} = R_A \times x - 40(x-3) \rightarrow (1)$ — $\frac{1}{2}$M

Integrating equation (1) we get

$EI\frac{dy}{dx} = R_A \cdot \frac{x^2}{2} - \frac{40(x-3)^2}{2} + C_1 \rightarrow (2)$ — $\frac{1}{2}$M

$\rightarrow$ slope equation.

Integrating equation (2) we get

$EIy = R_A \cdot \frac{x^3}{6} - \frac{40(x-3)^3}{6} + C_1x + C_2 \rightarrow (3)$ — $\frac{1}{2}$M

$\rightarrow$ Deflection equation.

To calculate C_2 apply boundary condition.
put $x=0$, $y=0$ in eq. (3) we get

$$EIy = R_A \cdot \frac{x^3}{6} + C_1 x + C_2$$

$$0 = 0 + 0 + C_2$$

$$\therefore \boxed{C_2 = 0}$$ $\frac{1}{2}M$

To calculate C_1 put $x=5m$ & $y=0$ in equation (3) we get

$$EIy = R_A \cdot \frac{x^3}{6} + C_1 x + C_2 - \frac{40(x-3)^3}{6}$$

$$0 = \frac{16 \times (5)^3}{6} + C_1 \times 5 - 0 - \frac{40(5-3)^3}{6}$$

$$-C_1 \times 5 = 333.333 - 53.333$$

$$-C_1 \times 5 = 279.999$$

$$-C_1 = 55.999$$

$$\boxed{C_1 = -55.999}$$ $\frac{1}{2}M$

To calculate slope under point load put $x=3m$ & $\frac{dy}{dx} = \theta_c$ in eq. (2)

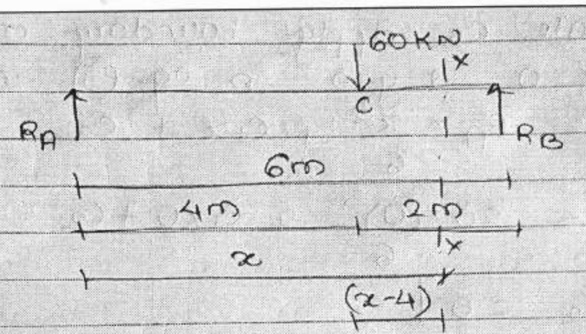
$$EI\frac{dy}{dx} = R_A \cdot \frac{x^2}{2} - \frac{40(x-3)^2}{2} + C_1$$

$$EI\theta_c = \frac{16 \times (3)^2}{2} - \frac{40(3-3)^2}{2} + (-55.999)$$

$$EI\theta_c = 72 - 0 - 55.999$$

$$EI\theta_c = 16$$

$$\boxed{\theta_c = \frac{16}{EI}}$$ $\frac{1}{2}M$

	To calculate deflection under point load put $x = 3m$ $y = y_{max}$ in eq. (3) $EI\,y = R_A \cdot \frac{x^3}{6} - 40\frac{(x-3)^3}{6} + C_1 x + C_2$ $EI\,y_{max} = \frac{16 \times 3^3}{6} - \frac{40(3-3)^3}{6} + (-55.999 \times 3) + 0$ $EI\,y_{max} = 72 - 0 - 56 \times 3 + 0$ $EI\,y_{max} = -$ $\boxed{y_{max} = \frac{-96}{EI}}$	$\frac{1}{2}M$
2	**A Simply supported beam of span 6m carries a point load of 60KN at 4m from left support. Find the value of slope under the load. Take EI=7800KNm^2.**	
	 $\Sigma F_y = 0$ $\quad R_A + R_B = 60KN$ $\Sigma M_A = 0$ $\quad R_B \times 6 = 60 \times 4$ $R_B \times 6 = 240$ $\boxed{R_B = 40KN}$ $R_A + R_B = 60$ $R_A = 60 - 40$ $\boxed{R_A = 20KN}$ To calculate slope and deflection under entire span take section x-x at distance x from support A. $m_x = R_A \times x - 60(x-4)$ but $m_x = EI\frac{d^2y}{dx^2}$	$\frac{1}{2}M$

$$EI\frac{d^2y}{dx^2} = R_A \times x - 60(x-4) \rightarrow (1)$$

Integrate equation (1)

$$EI\frac{dy}{dx} = R_A \times \frac{x^2}{2} - \frac{60(x-4)^2}{2} + C_1 \rightarrow (2)$$ ½ M

→ slope equation

Integrate equation (2) we get

$$EIy = R_A \times \frac{x^3}{6} - \frac{60(x-4)^3}{6} + C_1x + C_2 \rightarrow (3)$$ ½ M

Deflection equation

To calculate C_2 apply boundary condition put $x=0$ & $y=0$ in eq. (3) we get.

$$EIy = R_A \times \frac{x^3}{6} + C_1x + C_2$$

$$0 = \frac{20 \times (0)^3}{6} + C_1 \times 0 + C_2$$

$$\boxed{C_2 = 0}$$ ½ M

To calculate C_1 put $x = 6m$ & $y=0$ in equation (3) we get

$$EIy = R_A \times \frac{x^3}{6} - \frac{60(x-4)^3}{6} + C_1x + C_2$$

$$0 = \frac{20 \times (6)^3}{6} - \frac{60(6-4)^3}{6} + C_1 \times 6 + 0$$

$$-6C_1 = 720 - 80 + 0$$

$$-6C_1 = 640$$

$$\boxed{C_1 = -106.667}$$ ½ M

To calculate slope under point load put $x = 4m$ & $\theta_c = \frac{dy}{dx}$ in eq. (2)

$$EI\frac{dy}{dx} = R_A \times \frac{x^2}{2} - \frac{60(x-4)^2}{2} + C_1$$

$$EI\theta_C = \frac{20\times(4)^2}{2} - \frac{60(4-4)^2}{2} + (-106.667)$$ $\frac{1}{2}$M

$$EI\theta_C = 160 - 0 - 106.667$$

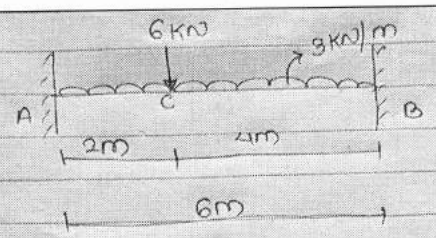
$$EI\theta_C = 53.333$$

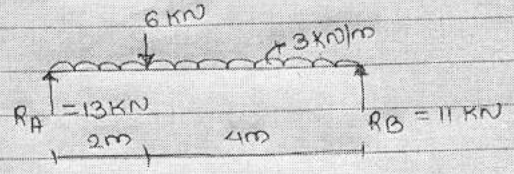
$$\theta_C = \frac{53.333}{EI}$$

$$\theta_C = \frac{53.333}{7800}$$

$$\boxed{\theta_C = 6.837\times10^{-3}\ \text{Radi}}$$ 1M

3 **A uniform beam of 6m span is fixed at A & B. It carries a u.d.l of 3KN/m over entire span in addition to a point load of 6KN at 4m from B. Calculate fixed end moments.**





$\frac{1}{2}$M

$\Sigma F_y = 0 \qquad R_A + R_B = (3\times6)+6$

$R_A + R_B = 24$

$\Sigma M_A = 0 \qquad R_B \times 6 = \left(3\times6\times\frac{6}{2}\right) + (6\times2)$

$$\boxed{R_B = 11\ \text{KN}}$$

$R_A + R_B = 24$

$R_A = 24 - 11$

$$\boxed{R_A = 13\text{KN}}$$ $\frac{1}{2}$M

calculate fixed end moments:-

m_A = Fixed end moment at A

m_B = Fixed end moment at B

$$m_A = -\frac{wab^2}{L^2} - \frac{wl^2}{12}$$ $\frac{1}{2}$M

$$m_A = -\frac{6\times2\times4^2}{6^2} - \frac{3\times6^2}{12}$$

$m_A = -5.333 - 9$

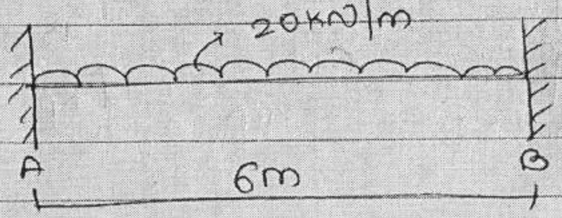
$\boxed{m_A = -14.333 \text{ KNm}}$ — 1M

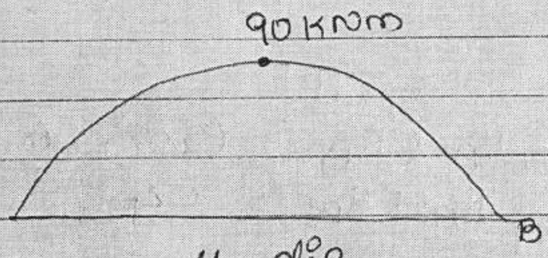
$m_B = -\frac{\omega a^2 b}{L^2} - \frac{\omega l^2}{12}$ — ½M

$= -\frac{6 \times 2^2 \times 4}{6^2} - \frac{3 \times 6^2}{12} = -2.667 - 9$

$\boxed{m_B = -11.667 \text{ KNm}}$ — 1M

4 **A fixed beam of span 6m carries a udl of 20KN/m over entire span. Find fixed end moments from first principle.**





M dig. — ½M

$\Sigma F_y = 0$ $\quad R_A + R_B = 20 \times 6$

$R_A + R_B = 120$

$\Sigma M_A = 0$ $\quad R_B \times 6 = 20 \times 6 \times \frac{6}{2}$

$\boxed{R_B = 60 \text{ KN}}$

$R_A + R_B = 120 \text{ KN}$

$\boxed{R_A = 60 \text{ KN}}$ — ½M

from 1^{st} principle

$$m_A + m_B = -\frac{2a}{L} \rightarrow (1)$$

$$m_A + 2m_B = -\frac{6a\bar{x}}{L^2}$$ $\frac{1}{2}$M

$$a_1 = \frac{2}{3} \times b \times h$$

$$\therefore a_1 = \frac{2}{3} \times 6 \times 90 = 360\, m^2 \qquad x_1 = \frac{b}{2} = \frac{6}{2}$$

$$\boxed{x_1 = 3m}$$ $\frac{1}{2}$M

$$m_A + m_B = -\frac{2 \times 360}{6}$$

$$\boxed{m_A + m_B = -120} \rightarrow (1)$$ $\frac{1}{2}$M

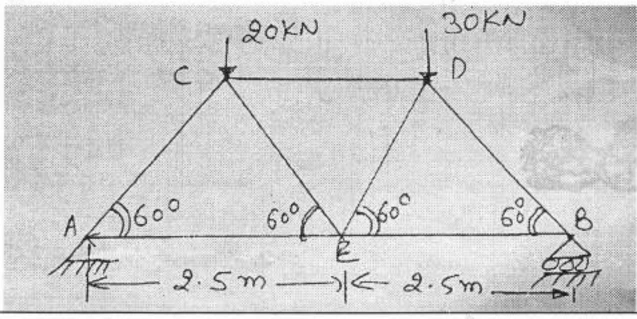
$$m_A + 2m_B = -\frac{6 \times 360 \times 3}{(6)^2}$$ $\frac{1}{2}$M

$$\boxed{m_A + 2m_B = -180} \rightarrow (2)$$

equation equation (1) & (2) we get

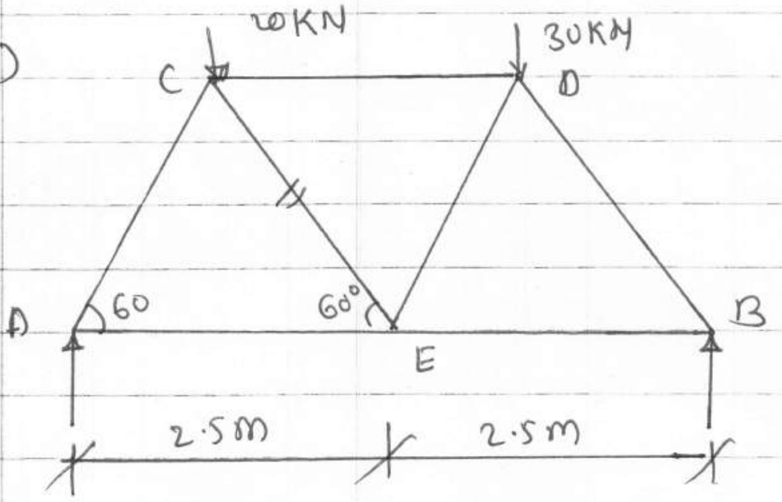
$$\boxed{\begin{aligned} m_A &= -60 \text{ KNm} \\ m_B &= -60 \text{ KNm} \end{aligned}}$$ $\frac{1}{2}$M

5 Using method of joints find forces in the member BE, EC of simple frame shown in fig. 2 .



Q.NO | SOLUTION | MARKS

Q-3 (e)



Step-1 Stability check

No of joint $j = 5$

Number of members $n = 7$

$\therefore 2j-3 = 2(5)-3 = 7$

$n = 2j-3$ so given truss is perfect

Step-2 F.B.D of truss and conditions of eq^n use

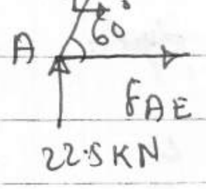
$M_A = R_B \times 5 - (20 \times 1.25) - (30 \times 3.75) = 0$

$0 = R_B \times 5 - 25 - 112.5$

$\boxed{R_B = 27.5\ KN}$

$\boxed{R_A = 22.5\ KN}$ 1M

Joint - A



$\Sigma H = 0$

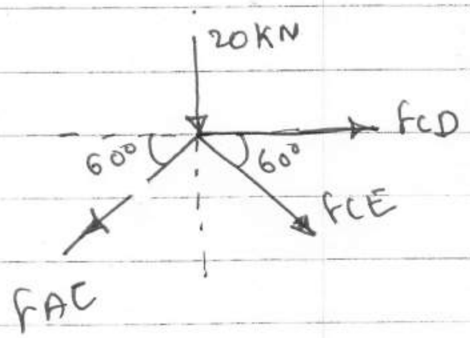
$F_{AE} + F_{AC} \cos 60^\circ = 0$

$F_{AE} = F_{AC} \cos 60^\circ$ —— (1)

$\Sigma V = 0$

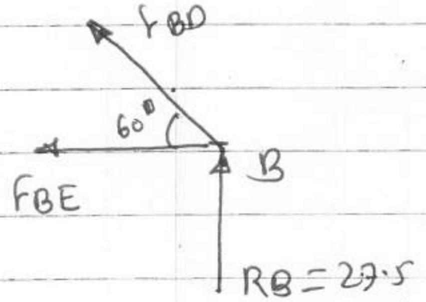
$F_{AC} \sin 60^\circ + 22.5 = 0$

$F_{AC} = \frac{-22.5}{\sin 60^\circ} = 25.98\ KN\ (C)$ ½M

Q.NO	SOLUTION	MARKS
	put the value of F_{AC} in eqn (1) & find F_{AE}	
	$F_{AE} = F_{AC} \cos 60^\circ$	
	$= 25.98 \cos 60^\circ$	
	$\boxed{F_{AE} = 12.99\ KN\ (C)}$	
	consider joint 'c'	
		
	$\Sigma F_x = 0$	1/2 M
	$F_{CE} \cos 60^\circ - F_{AC} \cos 60^\circ + F_{CD} = 0$	
	$F_{CE} \cos 60^\circ - 25.98 \cos 60^\circ + F_{CD} = 0$	
	$F_{CE} \cos 60^\circ - 12.95 + F_{CD} = 0$ — (2)	
	$\Sigma F_y = 0$ $\quad -20 - F_{CE} \sin 60^\circ + F_{AC} \sin 60^\circ = 0$	
	$-20 - F_{CE} \sin 60^\circ + 25.98 \sin 60^\circ = 0$	
	$-20 - F_{CE} \sin 60^\circ + 22.5 = 0$	
	$+2.5 - F_{CE} \sin 60^\circ = 0$	
	$F_{CE} \sin 60^\circ = 2.5$	
	$\boxed{F_{CE} = 2.886\ KN}$ (T)	(1M)

Q .NO	SOLUTION	MARKS

Joint B



$\Sigma fx = 0$

$-F_{BE} - F_{BD}\cos 60^\circ = 0$

$-F_{BE} = F_{BD}\cos 60$

$F_{BE} = -F_{BD}\cos 60$ —— (1)

$\Sigma fy = 0$

$27.5 + F_{BD}\sin 60^\circ = 0$

$-27.5 = F_{BD}\sin 60^\circ$

$\boxed{F_{BD} = -31.75\ KN}$ (comp) (½M)

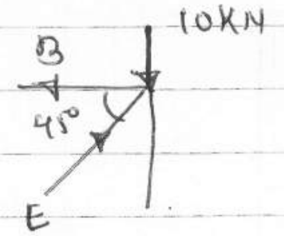
put F_{BD} in eq^n (1)

$F_{BE} = -F_{BD}\cos 60^\circ$

$= 31.75\cos 60^\circ$

$\boxed{F_{BE} = 15.87\ KN}$ (Tensile) (½M)

Q.NO	SOLUTION	MARKS
Q-3 (F)	Joint (C)	



$\Sigma F_y = 0$

$F_{CE} \cos 45^\circ - 10 = 0$

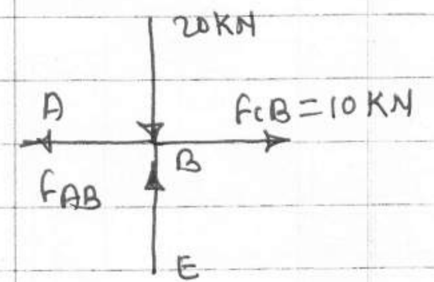
$\boxed{F_{CE} = 14.142\,KN}$ — comp ½M

$\Sigma F_x = 0$

$-F_{CB} + F_{CE} \cos 45^\circ = 0$

$\therefore F_{CB} = 14.142 \cos 45^\circ = 10\,KN$ (Tensile) ½M

Joint B



$\Sigma F_x = 0$

$-F_{AB} + 10 = 0$

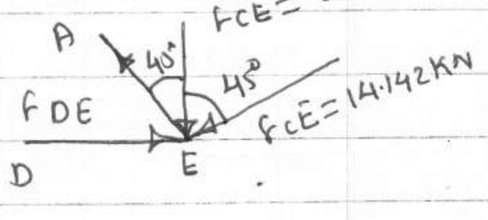
$\boxed{F_{AB} = 10\,KN}$ - - - Tensile ½M

$\Sigma F_y = 0$

$-20 + F_{BE} = 0$

$\boxed{F_{BE} = 20\,KN}$ - - (comp) ½M

Joint C



$\Sigma F_y = 0$

$F_{AE} \cos 45^\circ - 20 - 14.14 \cos 45$

$\boxed{F_{AE} = 42.426\,KN}$ (T) ½M

½M for F.B.D Joint 'c'

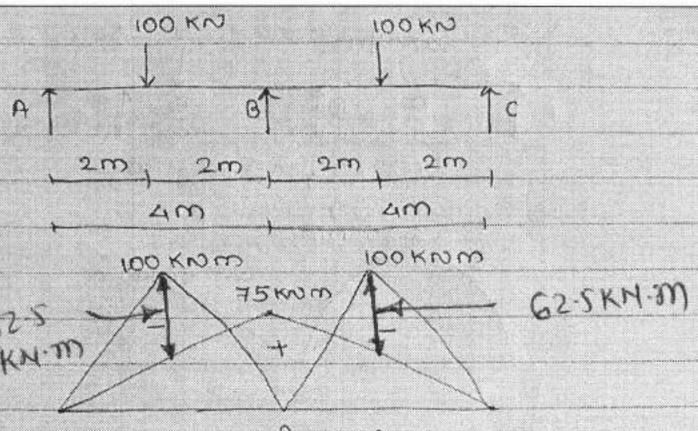
ΣF_x

$F_{DE} = -F_{AE} \cos 45^\circ - F_{CE} \cos 45^\circ = 0$

$F_{DE} = -42.426 \cdot \cos 45^\circ - 14.142 \cos 45^\circ$ 1M

$\boxed{F_{DE} = 40\,KN}$ (comp)

Q.4 a **A continuous beam ABC is supported on three supports at same level AB=BC=4m. Both span carry central point load of 100KN each. Calculate moment at B using theorem of three moment and draw BMD. Giving only net BMD.**



B.M.D in KN-m. 1M

$$M_A L_1 + 2M_B(L_1+L_2) + M_C L_2 = -\left(\frac{6a_1x_1}{L_1} + \frac{6a_2x_2}{L_2}\right)$$ 1M

$a_1 = \frac{1}{2} \times b \times h = \frac{1}{2} \times 4 \times 100 = 200$

$x_1 = \frac{b_1}{2} = \frac{4}{2} = 2m$

$a_2 = \frac{1}{2} \times 4 \times 100 = 200 \qquad x_2 = \frac{b_2}{2} = 2m$ 1M

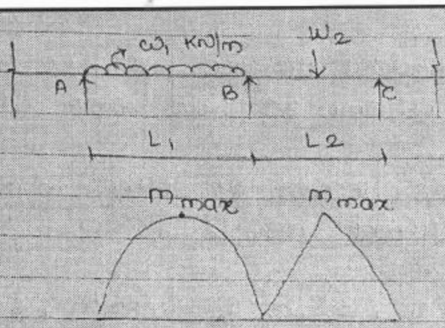
as M_A & M_C are simply supported hence $M_A = M_C = 0$

$\therefore\ 2M_B(4+4) = -\left[\left(\frac{6 \times 200 \times 2}{4}\right) + \left(\frac{6 \times 200 \times 2}{4}\right)\right]$

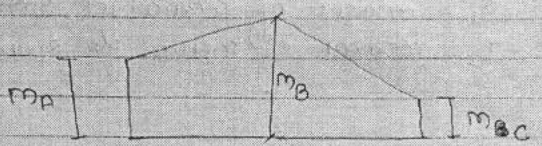
$16M_B = -1200$

$\boxed{M_B = -75\ KNm}$ 1M

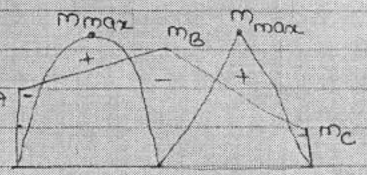
b) **State Clapeyorns theorem of three moment for same EI and different EI and state meaning of each term involved using neat sketch.**



Free BM dig



BMD for support moments.



Net BMD. 1M

1) Clapeyron's theorem of three moments with uniform EI for span AB & BC 1M

$$M_A L_1 + 2M_B(L_1+L_2) + M_C L_2 = -\left(\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right)$$

where M_A = support moment at A
M_B = support moment at B
M_C = support moment at C 1M
L_1 = length of span at AB
L_2 = length of span BC
a_1 = area of free BMD for span AB
a_2 = area of free BMD for span BC
$\bar{x}_1$ = centroidal distance of free BMD over span AB from left end A.

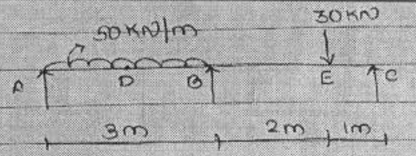
x_2 = centroidal distance of free BMD over span BC from right end C.

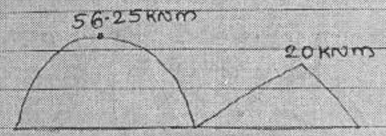
2) Clapeyron's theorem of three moment with different M.I.

$$M_A\frac{L_1}{I_1} + 2M_B\left(\frac{L_1}{I_1} + \frac{L_2}{I_2}\right) + M_C\frac{L_2}{I_2} = -\left(\frac{6a_1\bar{x}_1}{L_1 I_1} + \frac{6a_2\bar{x}_2}{L_2 I_2}\right)$$ 1M

where I_1 = moment of Inertia for span AB
I_2 = moment of Inertia for span BC

c) **A continuous beam ABC is simply supported at A,B,C such that AB=BC=3m. Span AB carries a u.d.l of 50KN/m from A to B. Span BC carries a point load of 30KN at 1m from C. Calculate support moments at B using three moment thorem.**





Since A & C are simply supported

$M_A = M_C = 0$

For free B.M span AB $M_{AB} = \frac{wl^2}{8} = \frac{50\times 3^2}{8}$

$M_{AB} = 56.25$ KNm. — 1M

Free B.M for span BC $M_{BC} = \frac{Wab}{L} = \frac{30\times 2\times 1}{3}$

free B.M $M_{BC} = 20$ KNm.

For span AB

$a_1 = \frac{2}{3}\times b\times h = \frac{2}{3}\times 3\times 56.25 = 112.5\ m^2$

$x_1 = \frac{b}{2} = \frac{3}{2} = 1.5$ m — ½M

For span BC

$a_2 = \frac{1}{2}\times b\times h = \frac{1}{2}\times 3\times 20 = 30\ m^2$

$x_2 = \frac{L+b}{3} = \frac{3+1}{3} = 1.333$ m. — ½M

Apply Clapeyron's theorem of three moment for span AB & BC

$$M_A L_1 + 2M_B(L_1+L_2) + M_C L_2 = -\left(\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right)$$ — 1M

$$0\times 3 + 2M_B(3+3) + 0\times 3 = -\left(\frac{6\times 112.5\times 1.5}{3} + \frac{6\times 30\times 1.333}{3}\right)$$

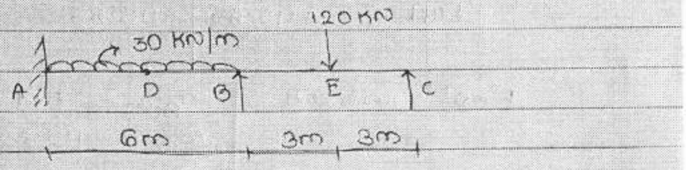
$12M_B = -(337.5 + 79.98)$

$12M_B = -(417.48)$

$\boxed{M_B = -34.79\ \text{KNm}}$

$\boxed{\begin{array}{l} M_A = 0 \\ M_B = -34.79\ \text{KNm} \\ M_C = 0 \end{array}}$ — 1M

d) **Find the moments at A,B,C for a continuous beam as shown in fig. below my moment distribution method.**



For span AB:-

$$m_{AB} = \frac{-wl^2}{12} = \frac{-30\times6^2}{12} = -90\text{ KNm}$$

$$m_{BA} = \frac{+wl^2}{12} = \frac{+30\times6^2}{12} = +90\text{ KNm}$$ ½M

For span BC

$$m_{BC} = \frac{-wl}{8} = \frac{-120\times6}{8} = -90\text{ KNm}$$

$$m_{CB} = \frac{+wl}{8} = \frac{+120\times6}{8} = +90\text{ KNm}$$ ½M

Distribution Factors

Joint	member	Relative Stiffness	Total Stiffness	Distribution Factor.
B	BA	$\frac{4EI}{6} = \frac{I}{6} = 0.1667 I$	$0.1667 I + 0.125 I$	$\frac{0.1667 I}{0.2917 I} = 0.57$
	BC	$\frac{3EI}{6} = \frac{3 I}{4\times6} = 0.125 I$	$= 0.2917 I$	$\frac{0.125 I}{0.2917 I} = 0.43$

1M

A	B		C
AB	BA	BC	CB
	0.57	0.43	
−90	+90	−90	+90
			−90
		−45	
−90	+90	−135	0
12.825 ←	+25.65	+19.35	
−77.175	+115.65	−115.65	0

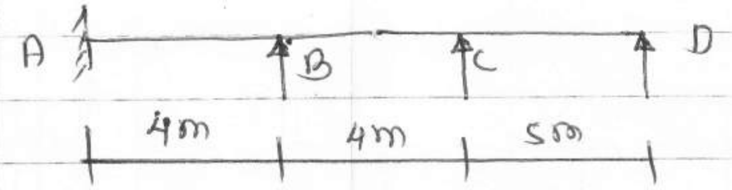
1M

1M

Final moments
$m_A = -77.175\text{ KNm}$
$m_B = 115.65\text{ KNm}$
$m_C = 0$

e)	Determine the distribution factors at continuity for a continuous beam A-B-C-D which is fixed at A and supported over B,C and D. Take AB=4m, BC= 4m and CD=5m. Assume same MI for all span.

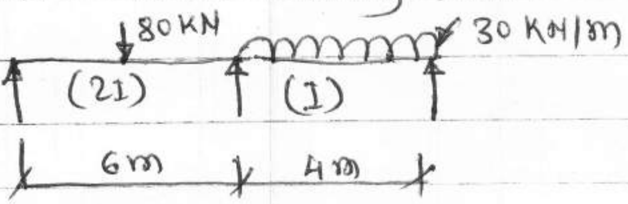
Q.NO	SOLUTION	MARKS



Distribution factor Table

Joint	Member	Relative Stiffness	Total Stiffness	D.F.	
B	BA	$\frac{4EI}{4} = EI$	2 EI	0.5	1M
	BC	$\frac{4EI}{4} = EI$		0.5	1M
C	CB	$\frac{4EI}{4} = EI$		0.625	1M
	CD	$\frac{3EI}{5} = 0.6EI$	1.6 EI	0.375	1M

f) Solve Q-2 F. using moment distribution method.



$$M_{AB} = \frac{-WL}{8} = \frac{-80 \times 6}{8} = -60 \text{ KN}$$

$$M_{BA} = \frac{+WL}{8} = \frac{+80 \times 6}{8} = +60 \text{ KN}$$

$$M_{BC} = \frac{-wL^2}{12} = \frac{-30 \times 4^2}{12} = -40 \text{ KN·m}$$

$$M_{CD} = \frac{+wL^2}{12} = \frac{+30 \times 4^2}{12} = +40 \text{ KN-m}$$ 1M

Q .NO	SOLUTION	MARKS

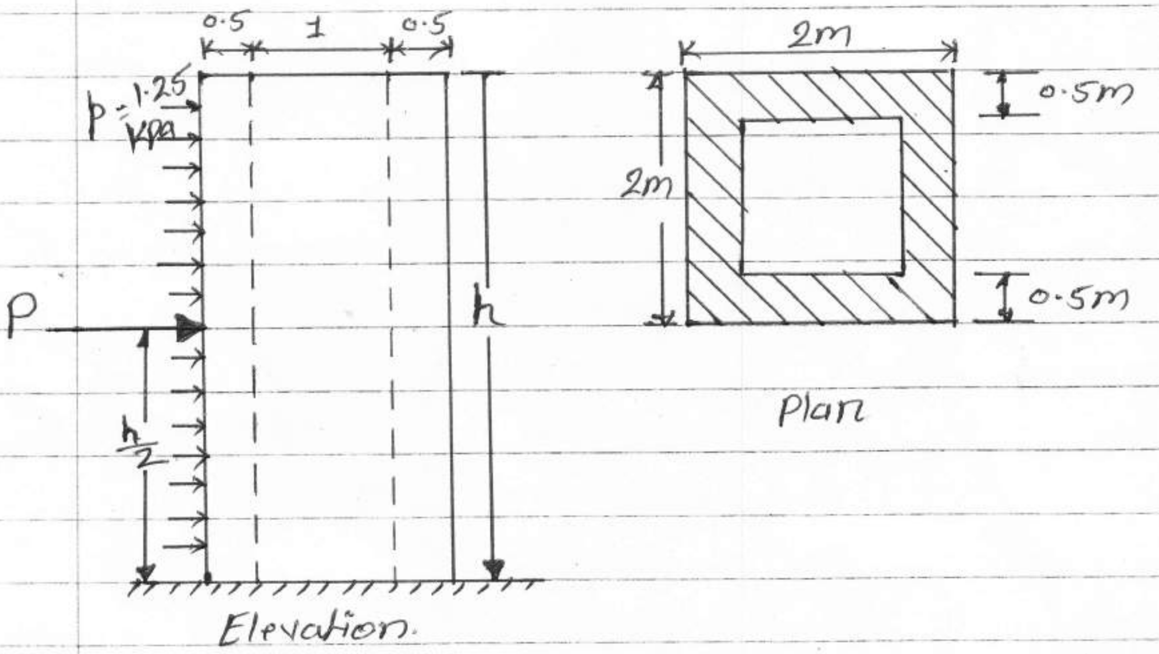
Distribution factor.

Joint	Member	Relative Stiffness	Total Stiffness	D.F.	
B	BA	$\frac{3E\times 2I}{6} = EI$	1.75EI	0.57	½M
	BC	$\frac{3EI}{4} = 0.75EI$		0.43	½M

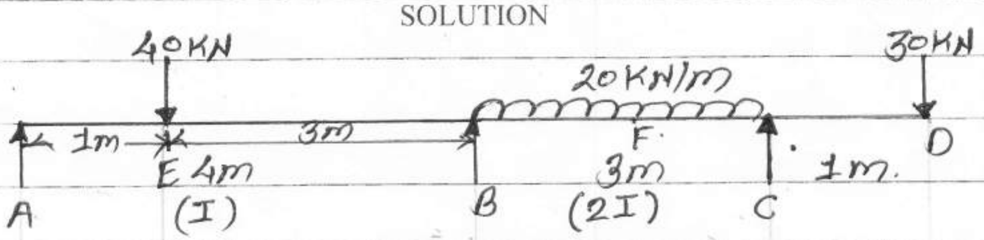
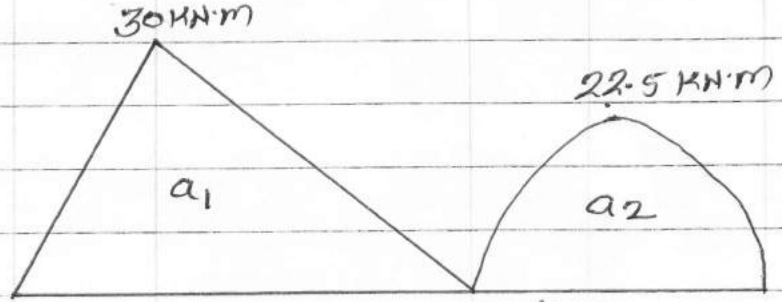
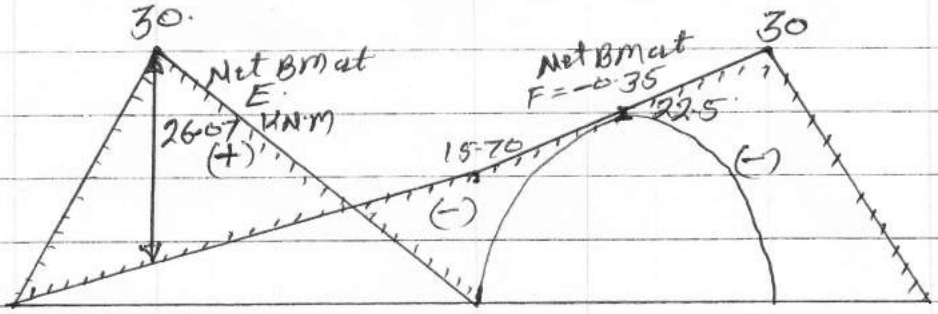
Final moment

A		B		C	
AB	BA	BC	CB		
	0.57	0.43			
-60	+60	-40	+40		1M
+60			-40		
	+30	-20			
0	90	-60	0		
	-17.1	-12.9			1M
0	+72.9	-72.9	0		

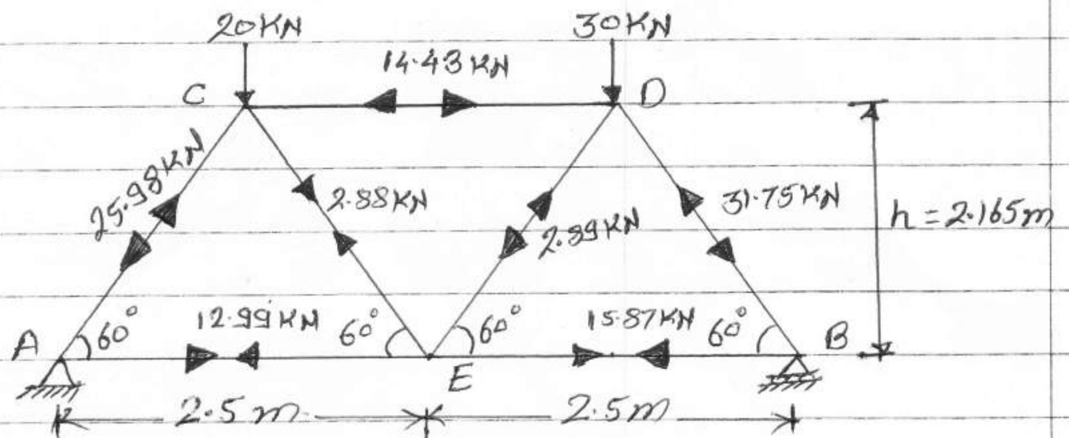
$M_A = 0$ $M_B = 72.9$ kN·m $M_c = 0$

Q.NO	SOLUTION	MARKS
Q5 a)	Given data	
	outer diamension $= 2\times 2\ m$, $t = 0.5\ m$	
	wind pressure $p = 1.25\ KPa$,	
	unit wt of masonary $= 20\ KN/m^3$	
		
	Let h be the height of chimney.	
	1) Direct stress $\sigma_0 = \frac{\text{Weight of chimney}(W)}{\text{c/s Area of chimney}(A)}$	1M
	Area of chimney $A = (2\times2) - (1\times1)$ $= 3\ m^2$	
	Weight of chimney $W = A\times h\times \rho$ $= 3\times h\times 20$ $W = 60h$	
	$\therefore\ \sigma_0 = \frac{60h}{3} = 20h \cdots (i)\ KN/m^2$	1M

Q .NO	SOLUTION	MARKS
Q5 a) Cont...	2) Bending stress $\sigma_b = \frac{M}{Z} = \frac{P \times \frac{h}{2}}{Z}$	1m
	Total wind load on chimney $P = p \times$ Projected Area.	
	$P = 1.25(2 \times h) = 2.5h$ KN	1m
	Section modulus of square section.	
	$Z = \frac{\frac{B^4 - b^4}{12}}{\frac{B}{2}} = \frac{B^3 - b^4}{6} = \frac{2^3}{}$	
	$Z = \frac{\frac{2^4 - 1^4}{12}}{\frac{2}{2}} = 1.25\ m^3$	1m
	$\therefore \sigma_b = \frac{2.5h \times \frac{h}{2}}{1.25} = h^2$ KN/m²	1m
	for No tension condition.	
	$\sigma_0 = \sigma_b$	1m
	$20h = h^2$	
	$\therefore \boxed{h = 20\ m}$	1m

Q.NO	SOLUTION	MARKS
Q5 b) →		
		
	Free B.M.D or M diagram.	
		2M
	1) Assume the spans AB & BC as simply supported & draw M diagram.	
	Free moments, B.M at 40 KN point load of AB	
	$M_E = \frac{Wab}{L} = \frac{40 \times 1 \times 3}{4} = 30\ KN.m$	½M
	B.M at midspan of BC	
	$M_F = \frac{wl^2}{8} = \frac{20 \times 3^2}{8} = 22.5\ KN.m$	½M

Q.NO	SOLUTION	MARKS
Q5 b) Cont...	2) Apply clapeyron's theorem of three moments to spans AB & BC	
	$M_A\left(\frac{L_1}{I_1}\right) + 2M_B\left(\frac{L_1}{I_1} + \frac{L_2}{I_2}\right) + M_C\left(\frac{L_2}{I_2}\right) = -\left[\frac{6a_1\bar{x}_1}{L_1 I_1} + \frac{6a_2\bar{x}_2}{L_2 I_2}\right]$	1m
	Known moments, $M_A = 0$ ---- s.s. end	
	$M_C = -30\times1 = -30\,kN{\cdot}m$	
	$a_1 = \frac{1}{2}\times4\times30 = 60$; $\bar{x}_1 = \frac{L+a}{3} = \frac{4+1}{3} = 1.67$; $L_1 = 4$, $I_1 = 1$ | $a_2 = \frac{2}{3}\times3\times22.5 = 45$; $\bar{x}_2 = \frac{L_2}{2} = \frac{3}{2} = 1.5$; $L_2 = 3m$, $I_2 = 2$	1m
	$0\left(\frac{4}{1}\right) + 2M_B\left(\frac{4}{1} + \frac{3}{2}\right) - 630\left(\frac{3}{2}\right) = -\left[\frac{6\times60\times1.67}{4\times1} + \frac{6\times45\times1.5}{3\times2}\right]$	
	$11M_B - 45 = -[150.3 + 67.5]$	
	$11M_B = -217.8 + 45$	
	$\therefore M_B = -15.70\ kN{\cdot}m$	1m
	-ve sign indicate hogging moment	
	Net B.M at $E = -\left(\frac{15.70}{4}\times1\right) + 30$	
	$= 26.07\ kN{\cdot}m.$	1m
	Net B.M at $F = -\left[15.70 + \left(\frac{30-15.7}{2}\right)\right] + 22.5$	
	$= -0.35\ kN{\cdot}m.$	1m

Q.NO	SOLUTION	MARKS
Q5c) →		

This problem can be solve by method of joint or method of section.

A] Method of Joints

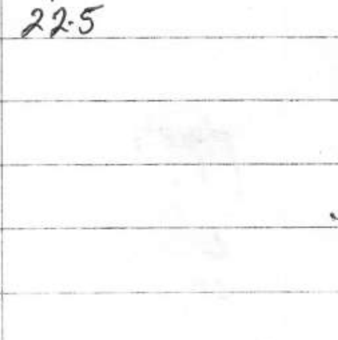
1> Support reactions

$\Sigma F_y = 0$; $R_A + R_B = 20 + 30 = 50$

$\Sigma M@A = 0$; $(20 \times 1.25) + (30 \times 3.75) - R_B \times 5 = 0$

$\therefore R_B = 27.5$ KN $\quad \therefore R_A = 22.5$ KN — 1m

2> Joint A



$\Sigma F_y = 0$

$22.5 + F_{AC} \sin 60 = 0$

$\therefore F_{AC} = -\frac{22.5}{\sin 60} = -25.98$ KN

$\therefore F_{AC} = 25.98$ KN (Comp) — 1m

$\Sigma F_x = 0$

$F_{AE} - F_{AC} \cos 60 = 0$

$\therefore F_{AE} = 25.98 \cos 60$

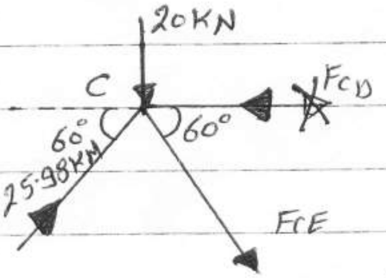
$F_{AE} = 12.99$ KN (Tensile) — 1m

Q.NO	SOLUTION	MARKS

Alternative Solution

Q5 c) Cont...

Joint C



$\Sigma F_y = 0;$

$-20 + 25.98 \sin 60 - F_{CE} \sin 60 = 0$

$F_{CE} = \dfrac{-20 + 25.98 \sin 60}{\sin 60}$

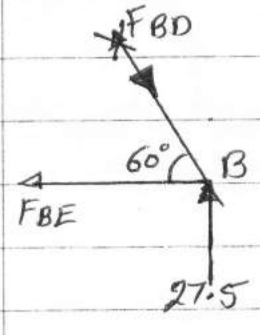
$F_{CE} = 2.88$ KN (Tensile) — 1m

$\Sigma F_x = 0;$

$25.98 \cos 60 + F_{CE} \cos 60 + F_{CD} = 0$

$F_{CD} = -14.43$ KN $= 14.43$ KN (comp) — 1m

Joint B



$\Sigma F_y = 0; \; 27.5 + F_{BD} \sin 60 = 0$

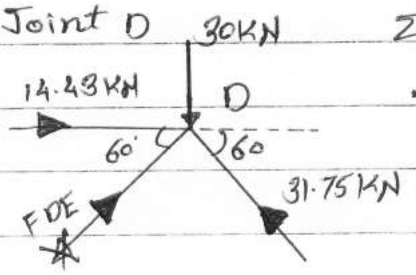
$F_{BD} = -\dfrac{27.5}{\sin 60} = -31.75$ KN

$\therefore F_{BD} = 31.75$ KN (comp) — 1m

$\Sigma F_x = 0; \; -F_{BE} + F_{BD} \cos 60 = 0$

$\therefore F_{BE} = 31.75 \cos 60 = 15.87$ KN (Tensile) — 1m

Joint D



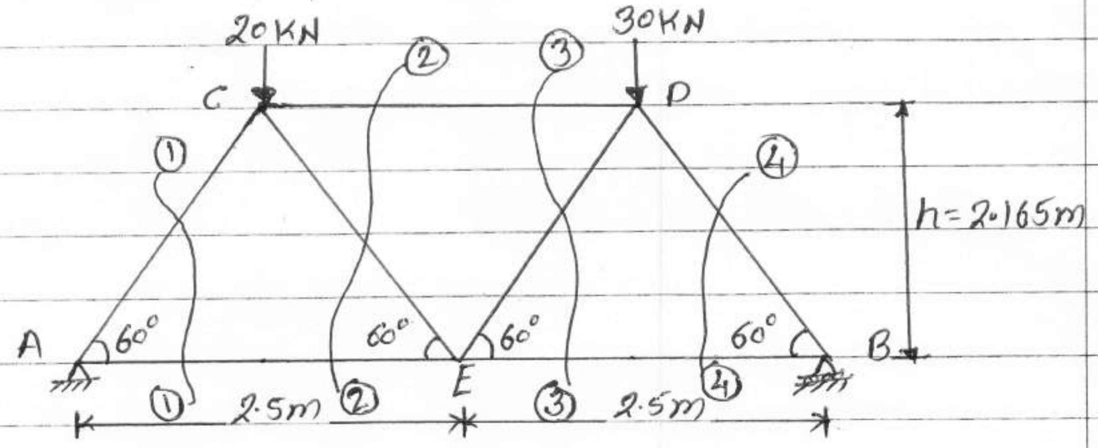
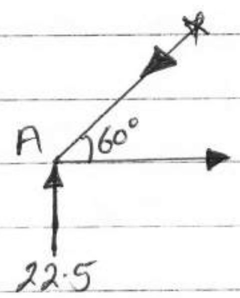
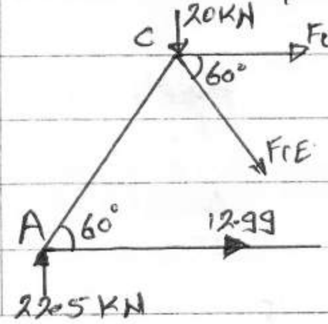
$\Sigma F_y = 0; \; -30 + 31.75 \sin 60 - F_{DE} \sin 60 = 0$

$\therefore F_{DE} = \dfrac{-30 + 31.75 \sin 60}{\sin 60} = -2.89$ KN

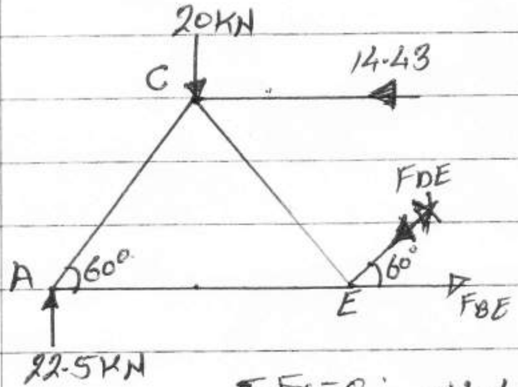
$\therefore F_{DE} = 2.89$ KN (comp) — 1m

Force Table.

Sr No.	member	Force	Nature
1	AC	25.98 KN	Compressive
2	AE	12.99 KN	Tensile
3	CE	2.88 KN	Tensile
4	CD	14.43 KN	Compressive
5	DE	2.89 KN	Compressive
6	BD	31.75 KN	Compressive
7.	BE	15.87 KN	Tensile

Q .NO	SOLUTION	MARKS
Q5C	Method of Section	
		
	1) Support reaction	
	$\Sigma F_y = 0$; $R_A + R_B = 20 + 30 = 50$	
	$\Sigma M@A = 0$; $(20 \times 1.25) + (30 \times 3.75) - (R_B \times 5) = 0$	
	$\therefore R_B = 27.5\ KN \quad \therefore R_A = 22.5\ KN.$	1m
	$\tan 60 = \frac{h}{1.25} \quad \therefore h = 1.25 \tan 60 = 2.165\ m$	
	2) Consider L.H.S of section (1)-(1)	
		
	$\Sigma F_y = 0$; $22.5 + F_{AC} \sin 60 = 0.$	
	$\therefore F_{AC} = \frac{-22.5}{\sin 60} = -25.98\ KN$	
	$\therefore F_{AC} = 25.98\ KN$ (comp)	1m
	$\Sigma F_x = 0$; $-25.98 \cos 60 + F_{AE} = 0.$	
	$\therefore F_{AE} = 12.99\ KN$ (Tensile)	1m
	3) Conside equilibrium of LHS of section (2)-(2)	
		
	$\Sigma F_y = 0$; $22.5 - 20 - F_{CE} \sin 60 = 0.$	
	$\therefore F_{CE} = \frac{22.5 - 20}{\sin 60} = 2.88\ KN$ (Tensile)	1m
	$\Sigma F_x = 0$; $12.99 + 2.88 \cos 60 + F_{CD} = 0$	
	$\therefore F_{CD} = -14.43\ KN = 14.43$ (Comp)	1m

Q .NO	SOLUTION	MARKS
Q5C Cont...	4) Consider equilibrium of L.H.S. of section ③-③	



$\Sigma F_y = 0;\ 22.5 - 20 + F_{DE}\sin 60 = 0$

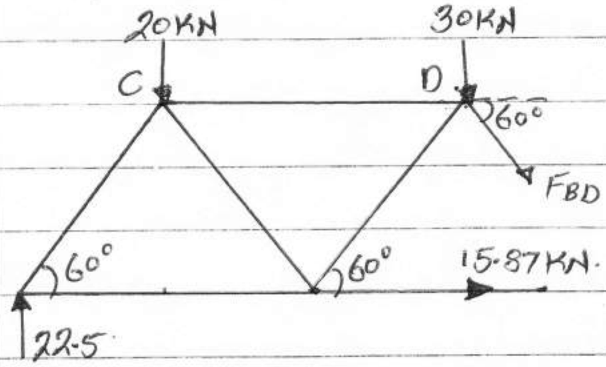
$\therefore F_{DE} = \frac{-22.5+20}{\sin 60} = -2.88\,KN$

$\therefore F_{DE} = 2.88\,KN$ (Comp) — 1m

$\Sigma F_x = 0;\ -14.43 - 2.88\cos 60 + F_{BE} = 0$

$\therefore F_{BE} = 15.87\ KN$ (Tensile) — 1m

5) Consider equilibrium of L.H.S of section ④-④.



$\Sigma F_y = 0.$

$22.5 - 20 - 30 - F_{BD}\sin 60 = 0$

$\therefore F_{BD} = \frac{-27.5}{\sin 60} = -31.75\,KN$

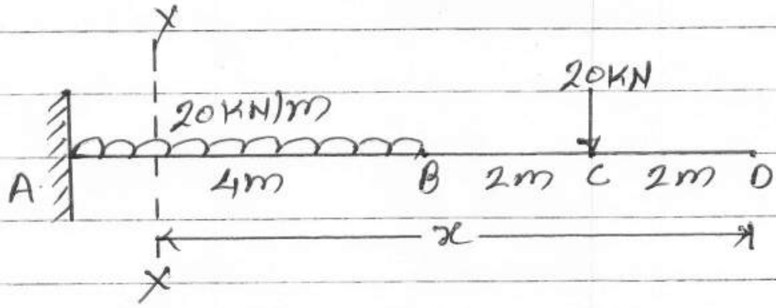
$F_{BD} = 31.75\ KN$ (comp) — 1m

Force Table

Sr. No	member	Force	Nature
1	AC	25.98 KN	Compressive
2	AE	12.99 KN	Tensile
3	CE	2.88 KN	Tensile
4	CD	14.43 KN	Compressive
5	DE	2.88 KN	Compressive
6	BD	31.75 KN	Compressive
7	BE	15.87 KN	Tensile

Q.NO | SOLUTION | MARKS

Q6a)



Take the free end D as a origine & Consider a section x-x at a distance x from D in the portion AB as shown in fig.

$$EI\frac{d^2y}{dx^2} = M_x = -20(x-2) - \frac{20(x-4)^2}{2}$$

$$EI\frac{d^2y}{dx^2} = -20(x-2) - 10(x-4)^2 \quad \ldots (i)$$ 1M

Integrating eqⁿ(i) w.r. to. x.

$$EI\frac{dy}{dx} = -\frac{20(x-2)^2}{2} - \frac{10(x-4)^3}{3} + C_1 \quad \ldots (ii)$$ 1M

Again integrating eqⁿ (ii) w.r. to. x

$$EI(y) = -\frac{20(x-2)^3}{6} - \frac{10(x-4)^4}{12} + C_1 x + C_2 \quad \ldots (iii)$$ 1M

To find integration Constants C_1 & C_2

Apply Boundary Condition

At. A, i.e, $x = 8$, $\frac{dy}{dx} = 0$ put in eqⁿ(ii)

$$0 = -\frac{20(8-2)^2}{2} - \frac{10(8-4)^3}{3} + C_1$$

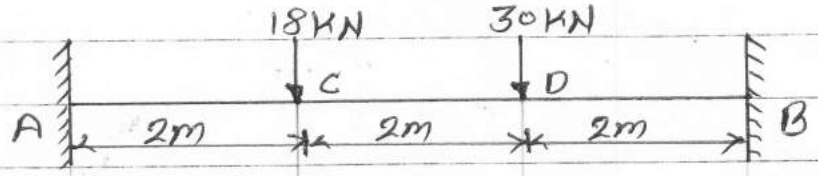
$$0 = -360 - 213.33 + C_1$$

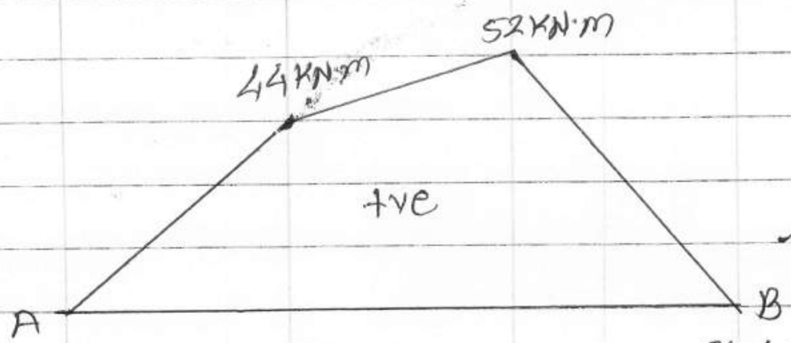
$$\therefore C_1 = 573.33$$ 1M

Q .NO	SOLUTION	MARKS
Q6a) Cont...	At A, i.e. $x = 8$, $y = 0$ put in eq^n(iii)	
	$EI(0) = -\frac{20(8-2)^3}{6} - \frac{10(8-4)^4}{12} + 573.33\times 8 + C_2$.	
	$0 = -720 - 213.33 + 4586.64 + C_2$	
	$\therefore C_2 = -3653.31$.	1M
	Substitute values of C_1 & C_2 in eq^n (ii) & (iii) respectively.	
	$EI\frac{dy}{dx} = -\frac{20(x-2)^2}{2} - \frac{10(x-4)^3}{3} + 573.33$ ... slope eqn	$\frac{1}{2}$M
	$EI(y) = -\frac{20(x-2)^3}{6} - \frac{10(x-4)^4}{12} + 573.33x - 3653.31$ ... Deflection eq^n.	$\frac{1}{2}$M
	To find deflection at B, put $x = 4m$ in Deflection eq^n.	
	$EI(y)_B = -\frac{20(4-2)^3}{6} + 573.33\times 4 - 3653.31$	
	$= -26.67 + 2293.32 - 3653.31$	
	$y_B = -\frac{1386.66}{EI}$ -ve sign indicate downward deflection	1M
	To find slope at C, put $x = 2$ in slope eq^n.	
	$EI\left(\frac{dy}{dx}\right)_c = 573.33$	
	$\left(\frac{dy}{dx}\right)_c = \theta_c = \frac{573.33}{EI}$ rad.	1M

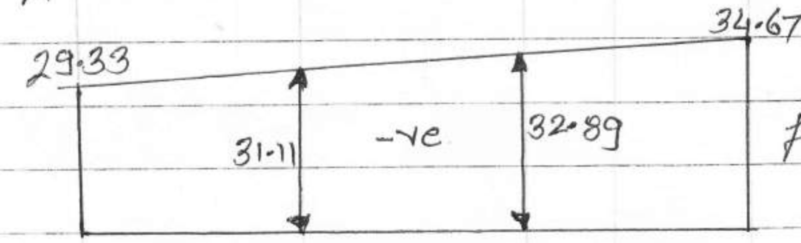
Q.NO	SOLUTION	MARKS

Q6 b) →

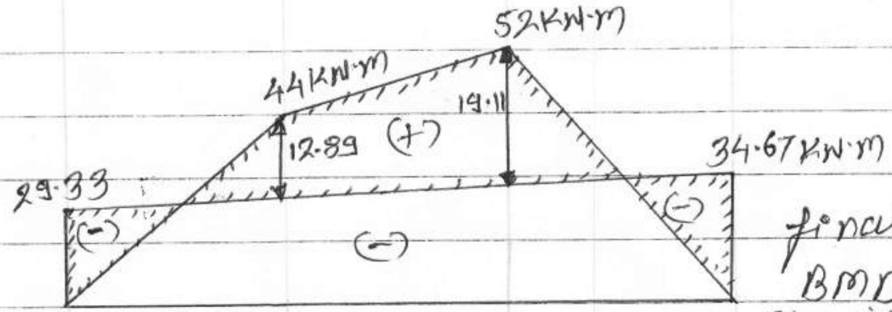




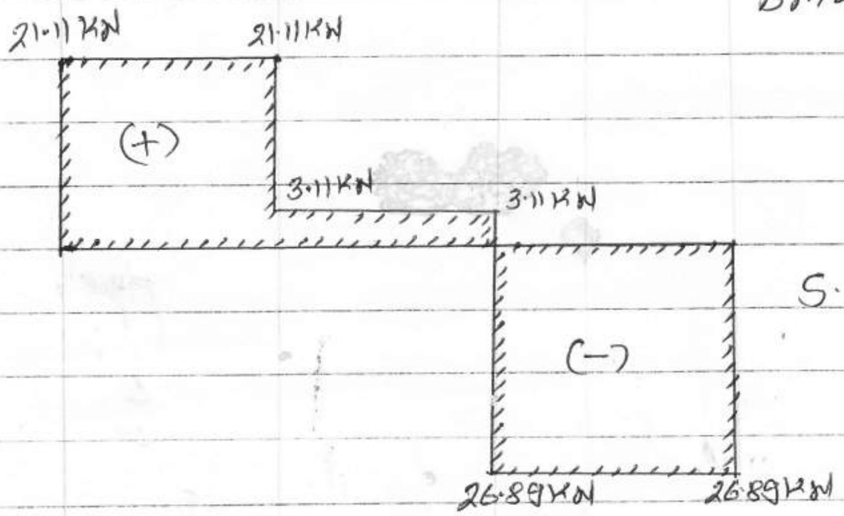
M dia or free BMD.



M' dia. or fixed BMD

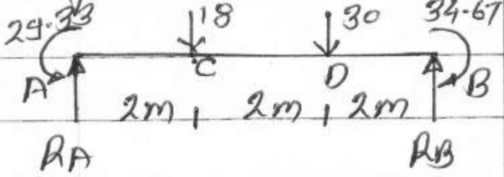
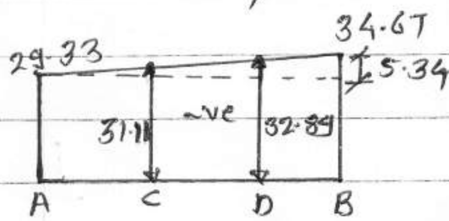


final BMD. Showing Net BM at C & D. 1M

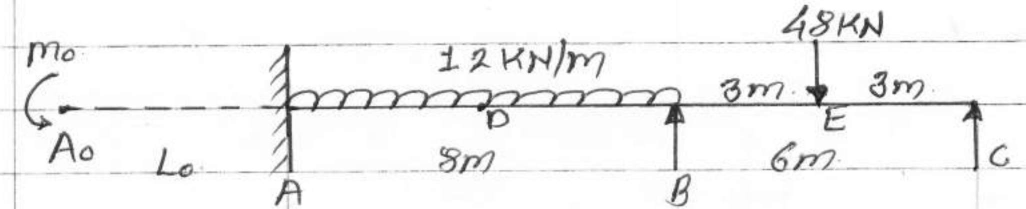


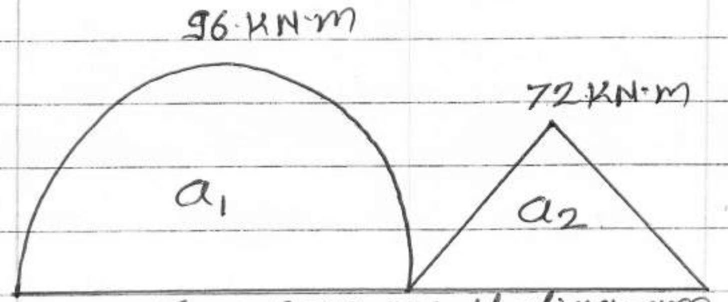
S.F.D. 1M

Q.NO	SOLUTION	MARKS
Q6b Cont...	1) Assume that the beam is simply supported and calculate the support reaction & draw M diagram.	
	i) $\Sigma F_y = 0$; $R_A + R_B = 18 + 30 = 48$ KN.	
	$\Sigma M@A = 0$; $(18\times2) + (30\times4) - R_B\times6 = 0$.	
	$\therefore R_B = 26$ KN $\quad \therefore R_A = 22$ KN.	
	$M_A = M_B = 0$	
	$\therefore M_C = 22\times2 = 44$ KN·m.	
	$M_D = 26\times2 = 52$ KN·m.	
	2) Calculate the fixed end moments & draw M' dia.	
	$W_1 = 18$ KN $\quad W_2 = 30$ KN	
	$a_1 = 2m \quad a_2 = 4m$	
	$b_1 = 4m \quad b_2 = 2m$.	
	$M_A = -\frac{W_1 a_1 b_1^2}{L^2} - \frac{W_2 a_2 b_2^2}{L^2} = -\frac{18\times2\times4^2}{6^2} - \frac{30\times4\times2^2}{6^2}$	
	$M_A = -29.33$ KN·m	1m
	$M_B = -\frac{W_1 a_1^2 b_1}{L^2} - \frac{W_2 a_2^2 b_2}{L^2} = -\frac{18\times2^2\times4}{6^2} - \frac{30\times4^2\times2}{6^2}$	
	$M_B = -34.67$ KN·m.	1m
	3) Superimpose M' diagram over M diagram & draw final B.M. diagram.	

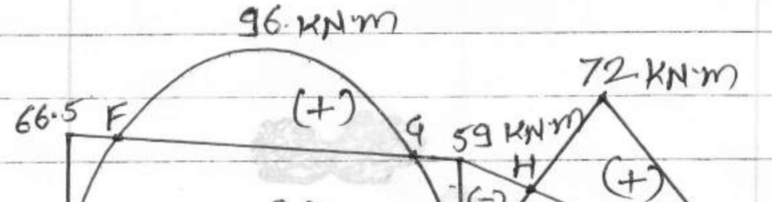
Q.NO	SOLUTION	MARKS
Q6b Contd…	4) Calculate reaction of a fixed beam.	
	$\Sigma F_y = 0;\ R_A + R_B = 48\ KN$	
	$\Sigma M @ A = 0.$	
		
	$-29.33 + (18\times2) + (30\times4) + 34.67 - R_B\times6 = 0$	
	$\therefore R_B = 26.89\ KN \quad \therefore R_A = 21.11\ KN.$	1M
	5) Net BM at C	
	$M_C = -29.33 + R_A\times2 = -29.33 + 21.11\times2$	
	$M_C = 12.89\ KN\cdot m.$	1M
	OR.	
	Net BM at C can also be calculated as follows	OR
		
	Net BM at C	
	$M_C = 44 - \left[29.33 + \frac{5.34}{6}\times2\right]$	
	$M_C = 12.89\ KN\cdot m.$	1M
	6) Net B.M at D	
	$M_D = -34.99 + R_B\times2 = -34.99 + 26.89\times2$	
	$M_D = 19.11\ KN\cdot m.$	1M
	OR.	OR
	Net BM at D can also be calculated as follows	
	Net B.M at D.	
	$M_D = 52 - \left[29.33 + \frac{5.34}{6}\times4\right]$	
	$M_D = 19.11\ KN\cdot m.$	1M

Q.NO	SOLUTION	MARKS
Q6b Cont...	7) S.F. Calculation	
	S.F. at Left of A = 0 KN.m	
	S.F at just right of A = R_A = 21.11 KN.	
	S.F at just left of C = 21.11 KN.	
	S.F at just right of C = 21.11 − 18 = 3.11 KN	1M
	SF at just left of D = 3.11 KN.	
	S.F at just right of D = 3.11 − 30 = −26.89 KN	
	SF at left of B = −26.89 KN.	
	S.F at just right of B = −26.89 + R_B = 0 KN.	
Q6C		





Free BMD or M diagram.



Final BMD. 2M

points F, G & H are the points of Contraflexure.

Q.NO	SOLUTION	MARKS
Q6C) Cont...	1) Assume the spans AB & BC as simply Supported & draw μ diagram	
	Free B.M at mid span of AB	
	$M_D = \frac{wl^2}{8} = \frac{12\times8^2}{8} = 96$ KN·m.	$\frac{1}{2}$M
	Free B.M at mid span of BC	
	$M_E = \frac{WL}{4} = \frac{48\times6}{4} = 72$ KNm.	$\frac{1}{2}$M
	2) Since Support A is fixed assume imaginary span AA_0 of Length L_0 to the left of A. Apply clapeyron's theorem of three moment to spans AA_0 & AB.	
	$M_{A_0}(L_0) + 2M_A(L_0+L_1) + M_B(L_1) = -\left[\frac{6a_0\bar{x}_0}{L_0} + \frac{6a_1\bar{x}_1}{L_1}\right]$	1M
	$M_{A_0} = 0$, $a_0 = 0$, $\bar{x}_0 = 0$, $L_0 = 0$ | $a_1 = \frac{2}{3}\times8\times96 = 512$, $\bar{x}_1 = \frac{8}{2} = 4m$, $L_1 = 8m$.	
	$0 + 2M_A(0+8) + M_B(8) = -\left[0 + \frac{6\times512\times4}{8}\right]$	
	$16M_A + 8M_B = -1536 \cdots\cdots (i)$	1M
	3) Apply theorem of three moment to spans AB & BC.	
	$M_A(L_1) + 2M_B(L_1+L_2) + M_C(L_2) = -\left[\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right]$	1M

Q.NO	SOLUTION		MARKS
Q6 c) Cont....	$a_1 = \frac{2}{3} \times 8 \times 96$	$a_2 = \frac{1}{2} \times 6 \times 72$	
	$= 512$	$= 216$	
	$\bar{x}_1 = \frac{L_1}{2} = \frac{8}{2} = 4m$	$\bar{x}_2 = \frac{L_2}{2} = \frac{6}{2} = 3m$	
	$L_1 = 8m$	$L_2 = 6m$	
	$M_A(8) + 2M_B(8+6) + 0(6) = -\left[\frac{6 \times 512 \times 4}{8} + \frac{6 \times 216 \times 3}{6}\right]$		
	$8M_A + 28M_B = -2184$ ----- (ii)		1m
	Solving eq^n (i) & (ii) we get		
	$M_A = -66.5$ KN·m -ve sign indicate hogging moment $M_B = -59$ KN·m		1m