Important Instructions to examiners:

1) The answer should be examined by keywords and not as word-to-word as given in the model answer scheme.

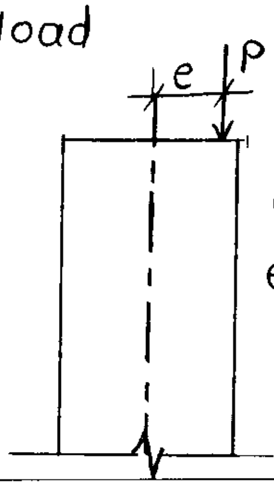
2) The model answer and the answer written by candidate may vary but the examiner may try to assess the understanding level of the candidate.

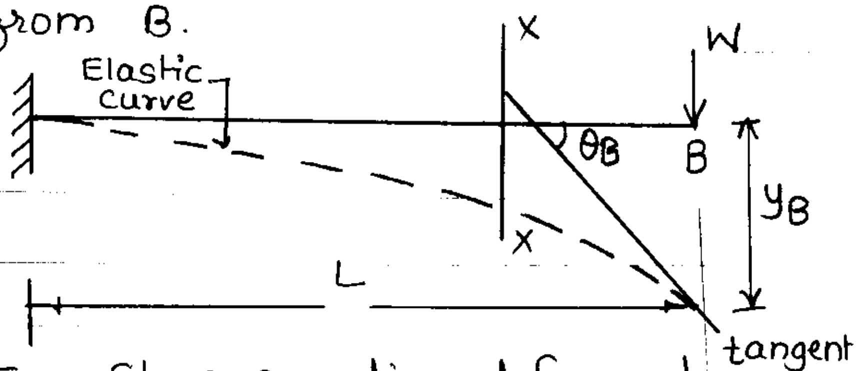
3) The language error such as grammatical, spelling errors should not be given more importance. (Not applicable for subject English and communication skill).

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

5) Credits may be given step wise for numerical problems. In the some cases, the assumed constants values may vary and there may be some difference in the candidates answer 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

	Question and Model Answers	Marks
Q.1 a)		
i)	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 the eccentric load	01 mark
	 P = Applied load e = Eccentricity	01 mark

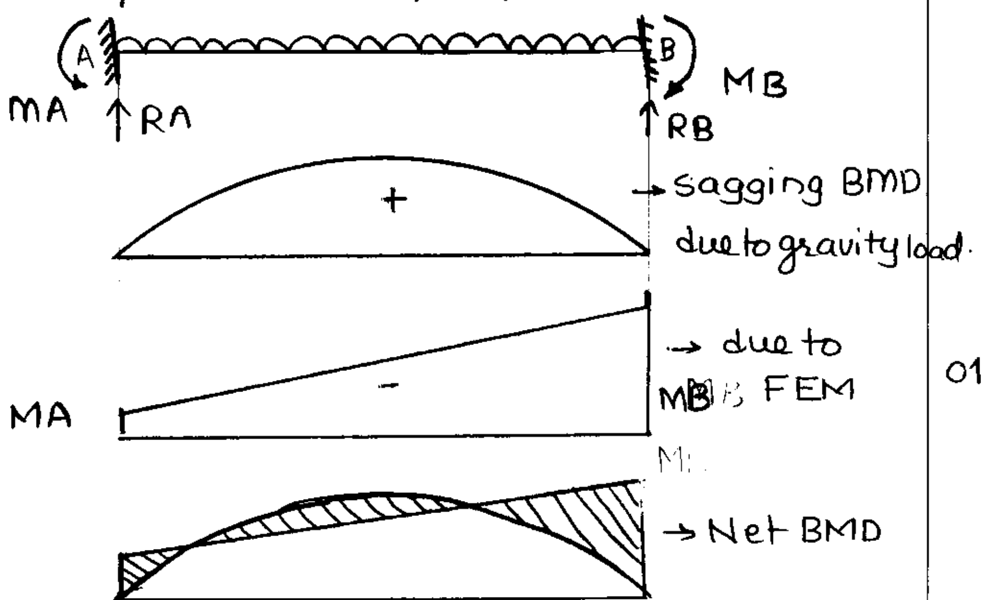
ii) The differential equation for slope -	
$\frac{\delta y}{\delta x} = \int \frac{M_x}{EI}$	
$Y = \int \frac{dy}{\delta x} \cdot \frac{1}{EI}$	
$\frac{dy}{\delta x}$ = Slope	
M_x = loading	
Y = deflection.	01 mark
The differential equation for deflection -	
$y = \iint \frac{M_x}{EI}$	01 mark
iii) Consider a section X-X at a distance x from B.	
	
• For Slope equation at free end.	01 mark
$\frac{dy}{\delta x_B} = -\frac{WL^2}{2EI}$ ----- Slope equation	
• For Deflection equation at free end	01 mark
$y_B = y_{max} = -\frac{WL^3}{3EI}$	

iv) Macaullay's method is used for finding slope and deflection of beam as followes

a. Use of Macaullay's method is very convenint for cases of discontinious and or discrete loading. 01 mark

b. Typically udl & vdl over the span and no. of concentrated loads are conveniently handle using this technique. 01 mark

v) Principle of Superposition →

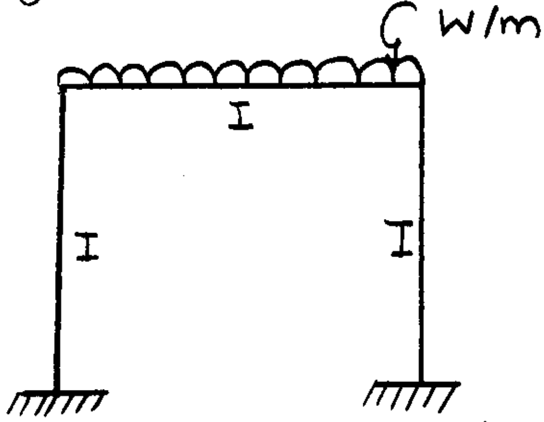


01 m

The Sagging diagram due to gravity load neutralized the hogging diagram due to fixed end moments to some 01 m

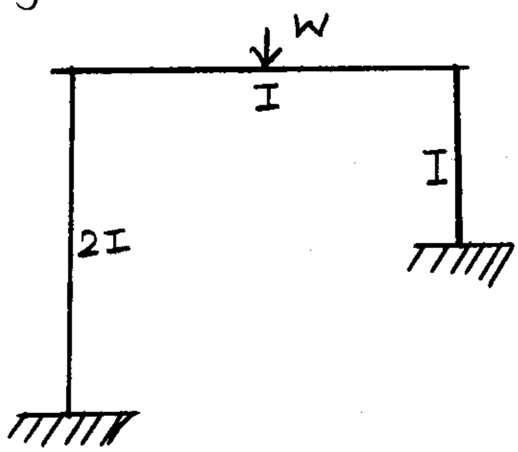
Continued
The remaining diagram indicates the nature and magnitude of net bending moment in the beam, Hence it is named as principle of Superposition

vi) Different types of portal frames can be classified as
a) Symmetrical portal frames
b) Unsymmetrical portal frames. 01 m



Symmetrical portal frame

$\frac{01}{2}$ m



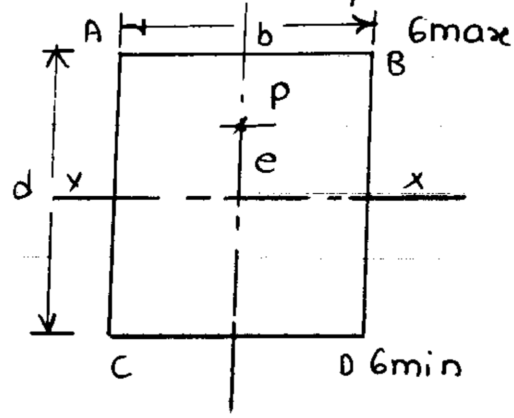
$\frac{01}{2}$ m

Unsymmetrical portal frame

vii) Stiffness factor :→ It is the moment required to produce unit rotation at an end, without translating it.	01m
Distribution factor :→ It is the ratio of relative stiffness of a members to the total stiffness of all the members meeting at a joint.	01m
viii) Perfect frame :→ A frame made up of just sufficient no. of members so that it can remain in stable equilibrium, when loaded at joints. $n = 2j-3$ → perfect.	01m
Imperfect frame :→ A frame made up of either more than or less than, just sufficient number of members, to keep it in static equillibrium is called imperfect frame. $n < 2j-3$ Deficient $n > 2j-3$ Redundant	01m

Q1. b)

i) Limit of eccentricity for rectangular section, d = depth



$\sigma_{max} = \sigma_0 + \sigma_b$ 01 m

$\sigma_{min} = \sigma_0 - \sigma_b$

$= \frac{P}{A} - \frac{M}{I_x} \times y$

Where $M = P \times e$ 01 m

$I_x = \frac{bd^3}{12}$

$y = d/2$

$\therefore \sigma_{min} = \frac{P}{A} - \frac{Pe}{(bd^3/12)} \times \frac{d}{2}$

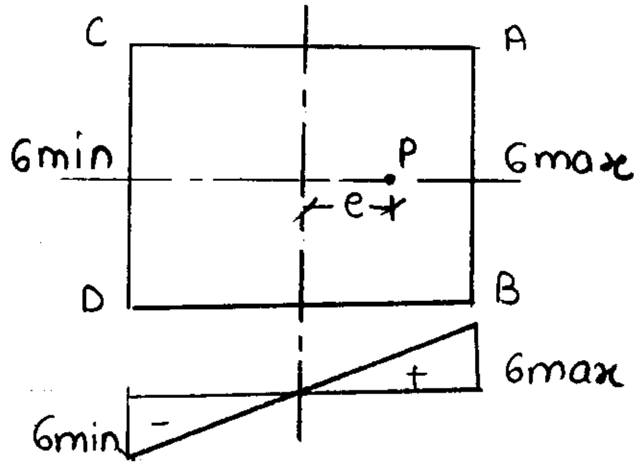
$\therefore$ For No tension $\sigma_{min} = 0$ 01 m

$0 = \frac{P}{A} - \frac{Pe}{bd^2} \times 6$

$\frac{Pe}{bd^2} \times 6 = \frac{P}{A}$

$\therefore \boxed{e \le d/6}$ Limit of eccentricity for rectangular sec. 01 m

ii) Minimum and maximum stresses developed at the base of the section



01m

i) σ_d = direct stress = $\frac{P}{A}$

ii) $\sigma_b = \frac{M}{I} \times y$.

01m

on face AB $\sigma_{max} = \sigma_d + \sigma_b = +ve$

CD $\sigma_{min} = \sigma_d - \sigma_b = -ve$

For Eccentric loading.

$$\sigma_{max} = \frac{P}{A} + \frac{Pe}{I} \times y$$ 01m

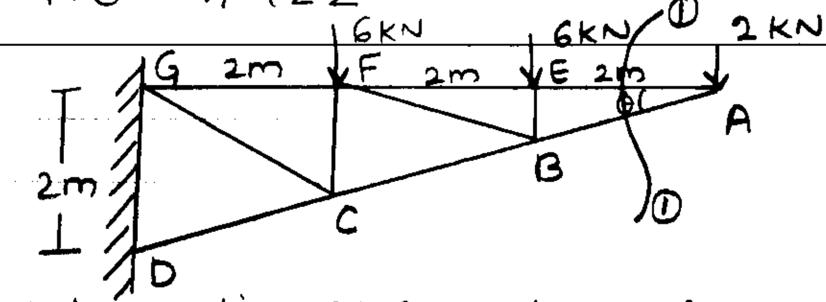
$$\sigma_{max} = \sigma_d + \sigma_b$$

$\therefore$ σ_{max} = direct stress + Bending stress

$$\sigma_{max} = \frac{P}{A} + \frac{Pe}{I} \times y$$ 01m

$$\sigma_{min} = \frac{P}{A} - \frac{Pe}{I} \times y$$

iii)



Take section (1)-(1) such that it passes through members whose force are required

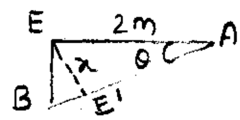
$\tan\theta = \frac{2m}{6m}$

$\theta = 18.43^\circ$

For equillibrium $\Sigma M_B = 0$

$2 \times 2 - F_{AE} \times 0.66$

$\underline{F_{AE} = 6.0 \text{KN Tensile}}$



$\Sigma M_E = 0$

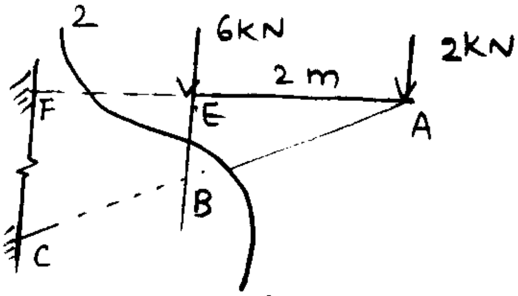
$2 \times 2 + F_{AB} \times 0.632$

$\underline{F_{AB} = -6.32 \text{ KN } \cancel{\text{KN}} (C)}$

$\sin\theta = \frac{EE'}{2}$

$EE' = 0.632$ 01

Section (2)-(2)



Assuming EF & BE tensile

$\Sigma M_A = 0$

$-6 \times 2 - BE \times 2 = 0$

$\therefore \underline{EB = -6 \text{ kN}}$ (Comp)

$\sin\theta = \frac{h}{4}$

$h = \cancel{0.1}$ 1.26

$\Sigma M_B = 0$

$-FE \times 0.66 + 2 \times 2$

$\underline{FE = 6.00 \text{ kN}}$ (Tensile) 01 section (3)-(3)

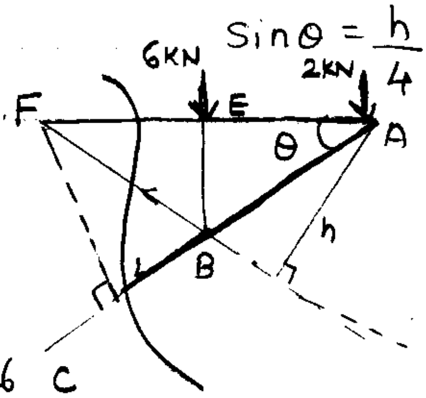
Assuming FB & BC are tensile

$\Sigma M_A = 0$

$h = 1.26$

$FB \times 1.26 - 6 \times 2$

$FB = 9.52\, kN\, (T)$ 01



$\Sigma M_F = 0$

$6 \times 2 + 2 \times 4 + BC \times 1.26$

$BC = -15.87\, kN$ (C) 01

Sr No.	Member	Force	Nature
1	FE	6.00 KN	Tensile
2	FB	9.52 kN	Tensile
3	CB	−15.87kN	Compressive

			mark
Q2.	a)	Given data.	
		dia = 250 mm	
		P = 200 kN	
		e = 150 mm	
		W = ?	
		$\sigma_{min} = \sigma_d - \sigma_b$	01
		For No tension, $\sigma_{min} = 0$	01
		$\therefore 0 = \sigma_d - \sigma_b$	
		$\therefore \sigma_d = \sigma_b$	
		$\frac{P}{A} = \frac{P.e}{I} \times y$	01
		Where,	
		$A = \pi/4 \times d^2 = \pi/4 \times 250^2 = 49087.38$ mm^2	
		$I = \frac{\pi}{64} \times 250^4 = 191.747 \times 10^6$ mm^4	
		$\therefore \frac{(W+200)}{A} = \frac{200 \times 10^3 \times 150}{191.747 \times 10^6} \times \frac{250}{2}$	
		$\boxed{W = 760 \text{ kN}}$	01
		$\therefore$ To avoid the tensile stress at the base, W = 760 kN	

mark

b) Data :→ $P = 250 kN$

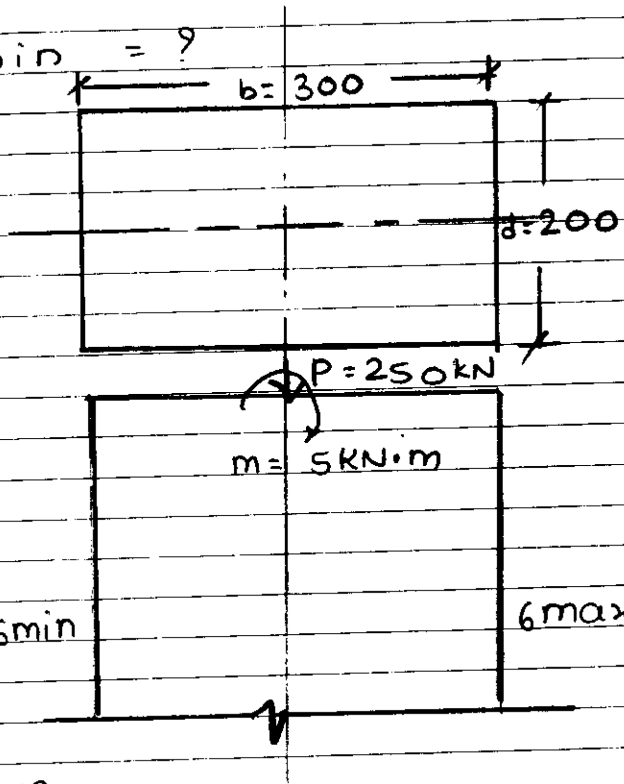
$b = 300 mm$

$d = 200 mm$

$m = 5 kN \cdot m$

$\sigma_{max} = ?$

$\sigma_{min} = ?$



01

Where,

$A = 300 \times 200 = 60000 \ mm^2$ 01

$I_y = \frac{200 \times 300^3}{12} = 450 \times 10^6 mm^2$

$\sigma_{max} = \sigma d + \sigma b$

$= \frac{P}{A} + \frac{M}{I} \times y$

$= \frac{\cancel{180}\ 250 \times 10^3}{60000} + \frac{5 \times 10^6}{450 \times 10^6} \times \frac{300}{2}$

$= 5.82 \ N/mm^2$ (Compressive)   01

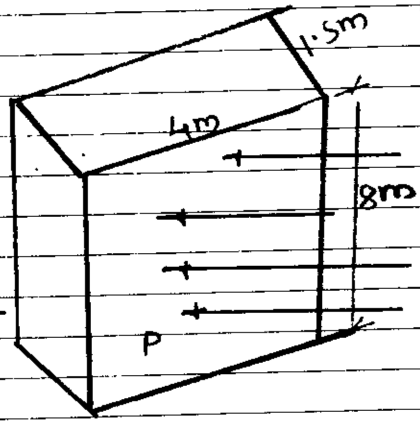
$\sigma_{min} = \sigma_d - \sigma_b$

$= \frac{P}{A} - \frac{M}{I} \times y$

$= 4.16 - 1.66$

$= 2.49\ N/mm^2$ (compressive) 01

Q) Data,



$h = 8m$

$b = 4m$

$t = 1.5m$

$p = 2.5 kN/m^2$

$\rho = 24\ kN/m^3$

Wind Pressure acting on 4m side,

→ To find maximum & minimum stress,

1) Self wt of wall $W = \rho \times \text{Volume}$ 01

$= 24 [4 \times 1.5 \times 8]$

$= 1152 KN$

2) Wind Force $P = Cp. \text{Area of Wall}$

$= 2.5 \times [4 \times 8]$

$= 80\ KN.$

3) Area at base $= 4 \times 1.5 = 6\ m^2$ 01

4) $I_y = \frac{4 \times 1.5^3}{12} = 1.125\ m^4$

5) Stresses,

$\sigma_{max} = \sigma_d + \sigma_b$

$$\sigma_{max} = \frac{W}{A} + \frac{P \cdot H/2}{I} \times y$$

$$= \frac{1152}{6} + \frac{80 \times 8/2}{1.125} \times \frac{1.5}{2}$$

$$= 192 + 213.33$$

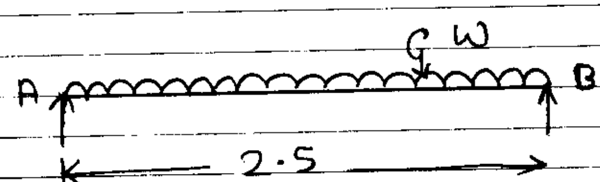
$$\boxed{\sigma_{max} = 405.33 \text{ N/mm}^2 \text{ (comp)}}$$ 01

$$\sigma_{min} = \frac{W}{A} - \frac{P \cdot H/2}{I} \times y$$

$$= 192 - 213.33$$

$$\boxed{\sigma_{min} = 21.33 \text{ N/mm}^2 \text{ (Tensile)}}$$ 01

d)



Data :→

Slope = 1.5°

maximum deflection = y = ?

where,

$\frac{dy}{dx}$ = Slope = 1.5°

$$1.5^\circ = \frac{WL^3}{24EI}$$

$$\frac{1.5 \times \pi}{180} = \frac{W \times 2.5^3}{24EI}$$

$$W = 0.040 EI$$ 02

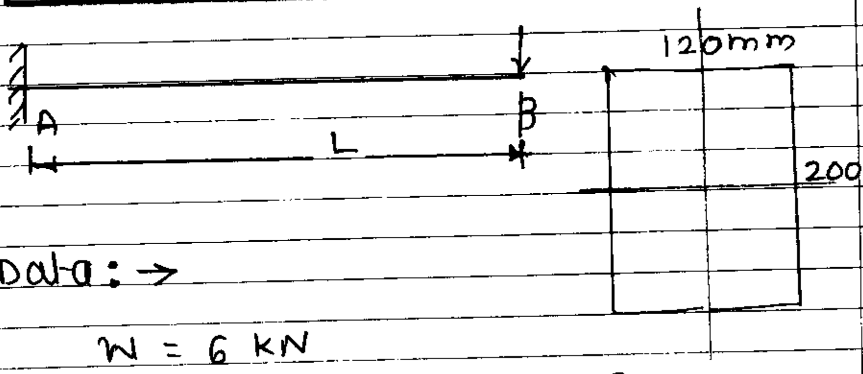
but $y_{max} = \frac{-5WL^4}{384EI}$

$$y_{max} = \frac{-5 \times 0.040 EI \times 2.5^4}{384 EI}$$

$$= 0.02045 \text{ m}$$

$$\boxed{\therefore y_{max} = 20.45 \text{ mm downward}}$$ 02

e)



Data: →

$W = 6$ kN

$\frac{dy}{dx_B}$ = Slope = 1.5×10^{-3} radians

$E = 100$ kN/mm^2

$= 1 \times 10^8$ kN/m^2 01

Where,

$$I = \frac{120 \times 200^3}{12} = 80 \times 10^6 \text{ mm}^4$$ 01

$$= 80 \times 10^{-6} \text{ m}^4$$

$$\therefore \text{Slope} = \frac{dy}{dx_B} = \frac{WL^2}{2EI}$$ 01

$$1.5 \times 10^{-3} = \frac{6 \times L^2}{2 \times 1 \times 10^8 \times 80 \times 10^{-6}}$$

$$L^2 = 4$$

$$\boxed{\therefore L = 2 \text{m}}$$ 01

mark

f) Clapeyron's Theorem :→

Let ABC be a continious beam, selected for analysis and loaded as shown below.

a_1 & a_2 — are the area of BMD

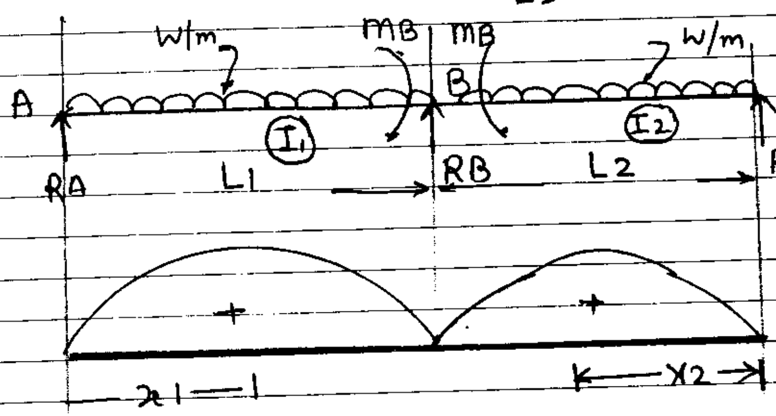
$M_A = M_C = 0$

∴ The supports are simple support

To obtain M_B Clapeyron's thm. 01 m

$$M_A \frac{L_1}{I_1} + 2M_B \left[\frac{L_1}{I_1} + \frac{L_2}{I_2}\right] + M_C \frac{L_2}{I_2}$$

$$= -6\left[\frac{a_1 x_1}{L_1 I_1} + \frac{a_2 x_2}{L_2 I_2}\right]$$ 01 M



01 M

Sagging BMD

Where,

M_A & $M_C = 0$

I_1 & I_2 are the moment of Inertia of beam

M_B can be obtained from the above equation.

L_1 & L_2 are span

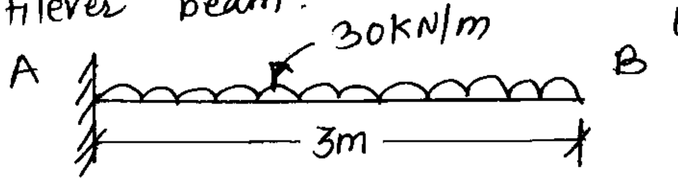
x_1 & x_2 are distance of CG of BMD 01 M

Question & Sub Question	Answer	Marks

Q. 3 Attempt any FOUR of the following.

3(a) Calculate Deflection and slope at free end for Cantilever beam.

$EI = 40,000 \text{ kN.m}^2$



1) Deflection at free end

$$y_B = y_{max} = -\frac{WL^4}{8EI}$$ — 1 (Mark)

$$= -\frac{30 \times (3)^4}{8 \times 40,000}$$

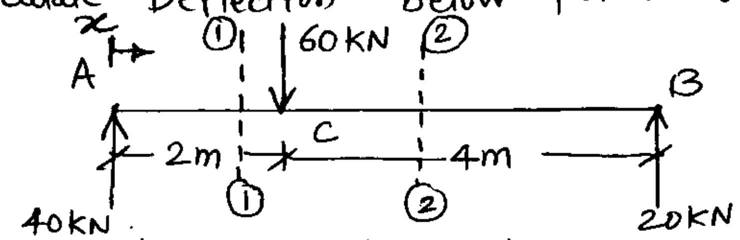
$$\boxed{y_{max} = -7.593 \text{ mm}}$$ -ve sign indicate Downward deflection — 1 Mark

2) Slope at free end –

$$\theta_B = \frac{WL^3}{6EI}$$ — 1 Mark

$$\theta_B = \frac{30 \times 3^3}{6 \times 40,000} = 0.003375 \text{ radians}$$ — 1 Mark

3(b) Calculate Deflection below point load. Using Macaulay's Method.



1) To Calculate Support Reaction.

$\Sigma M_A = 0$ (↶ +) $R_A + R_B = 60 \text{ kN}$

$$-R_B \times 6 + 60 \times 2 = 0$$

$$R_B = \frac{-120}{-6}$$

$$R_B = 20 \text{ kN}$$

$$\therefore R_A = 60 - 20 = 40 \text{ kN}$$

2) $EI \frac{d^2y}{dx^2} = M$ – Differential equation

$$EI \frac{d^2y}{dx^2} = 40x \;\Big|\; -60(x-2) \;\Big|$$

$x = 2m$ $x = 6m$

Question & Sub Question	Answer	Marks

Integrating With respect to x

$$EI\frac{dy}{dx} = \frac{40x^2}{2} + C_1 \;\Big|_{x=2m}\; -\frac{60(x-2)^2}{2} \;\Big|_{x=6m}$$ — slope eq^n — 1 Mark

Again Integrating w. r. to . x .

$$EI \cdot y = \frac{20x^3}{3} + C_1 x + C_2 \;\Big|\; -\frac{60(x-2)^3}{6}$$ — Deflection eq^n — 1 Mark

3) Calculation of Constants of Integration

Boundary Condition .

$x = 0$, $y = 0$ Putting in Deflection equation

$$0 = 0 + C_2 - 10(\cancel{0}-3)^3$$ Neglecting -ve terms .

$$\boxed{C_2 = 0}$$

At $x = 6\,m$, $y = 0$ putting in deflection equation

$$EI(0) = \frac{20}{3}(6)^3 + 6C_1 + 0 - 10(6-2)^3$$

$$0 = 1440 + 6C_1 - 640$$

$$\boxed{C_1 = -133.333}$$ 1 Mark

4) To Calculate Deflection below point load

At $x = 2m$, $y = y_c$ putting in Deflection eq^n

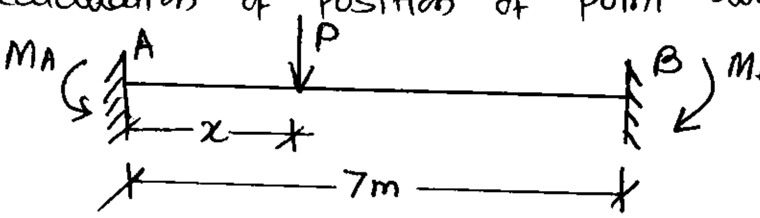
with Constants of Integration .

$$EI \cdot y_c = \frac{20}{3}(2)^3 - 133.333(2) + 0 \;\vdots\; 10(\cancel{2}-2)^3$$

$$y_c = \frac{53.34 - 266.66}{EI}$$

$$\boxed{y_c = -\frac{213.32}{EI}}$$ 1 Mark .

Q3 (c) Calculation of position of point load for fixed Beam



$$MA = 2MB .$$ 1 Mark

Question & Sub Question	Answer	Marks

Fixed End moments.

$$M_A = -\frac{Wab^2}{L^2} = \frac{-P \times x \times (7-x)^2}{(7)^2}$$ 1 Mark

$$M_B = -\frac{Wa^2b}{L^2} = \frac{-P \times x^2 \times (7-x)}{(7)^2}$$ 1 Mark

Putting values of M_A & M_B in eq^n (I)

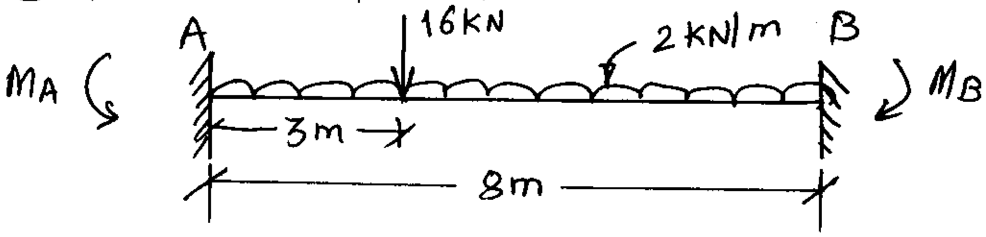
$$\frac{-\cancel{P} \times \cancel{x} \times (7-x)^{\cancel{2}}}{\cancel{7^2}} = 2\left[\frac{-\cancel{P} \times x^{\cancel{2}} \times (\cancel{7-x})}{\cancel{7^2}}\right]$$

$$(7-x) = 2x$$

$$3x = 7$$

$$\boxed{x = 2.333m}$$ 1 Mark

Q.3(d) Calculation of fixed end moments.



Fixed End moments

$$M_A = M_{A1} + M_{A2} = -\frac{wL^2}{12} - \frac{Wab^2}{L^2}$$ 1 Mark

$$M_A = \frac{-2 \times 8^2}{12} - \frac{16 \times 3 \times 5^2}{8^2}$$

$$M_A = -10.67 - 18.75$$

$$\boxed{M_A = -29.42 \text{ kN.m}}$$ 1 Mark

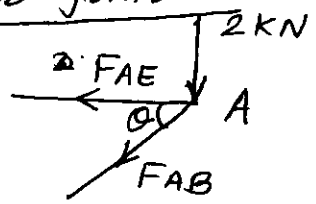
$$M_B = M_{B1} + M_{B2} = -\frac{wL^2}{12} - \frac{Wa^2b}{L^2}$$

$$M_B = \frac{-2 \times 8^2}{12} - \frac{16 \times 3^2 \times 5}{8^2} = -10.67 - 11.25$$

$$\boxed{M_B = -21.92 \text{ kN.m}}$$ 1 Mark

Question & Sub. Question	Answer	Marks.
Q.3(e)	Assumptions made in Analysis of simple frames i) The members are pin jointed at ends. ii) The frame is perfect frame i.e. it satisfies $n = 2j - 3$ Condition. iii) The loads are acting at joints only. iv) Self wt. of members is neglected. v) Only Axial forces (tensile and Compressive) are induced in the member.	Any 4 1×4 = 4 marks.
Q.3f.	Find forces in member AB, AE, EB & EF using method of joint	

Consider joint A



Consider Δ ADG in fig. 1.

$$\tan\theta_A = \frac{2}{6}$$

$$\theta_A = \tan^{-1}(0.333)$$

$$\left[\theta_A = 18.433^\circ\right]$$

Considering equations of equillibrium

$$\Sigma F_x = 0 \qquad \theta = 18.433^\circ$$

$$-F_{AE} + F_{AB}\cos\theta = 0$$

$$-F_{AE} - 0.948\,F_{AB} = 0 \quad \text{— (I)}$$

$$\Sigma F_y = 0$$

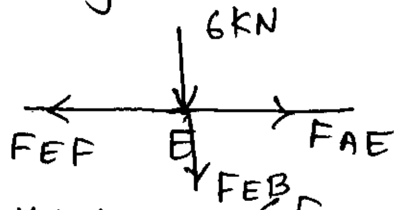
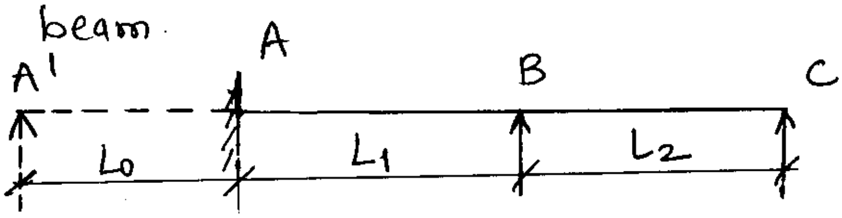
$$-2 - F_{AB}\sin(18.433) = 0$$

$$\left[F_{AB} = -6.325\,kN\right]$$ -ve sign indicates Compression — 1 Mark

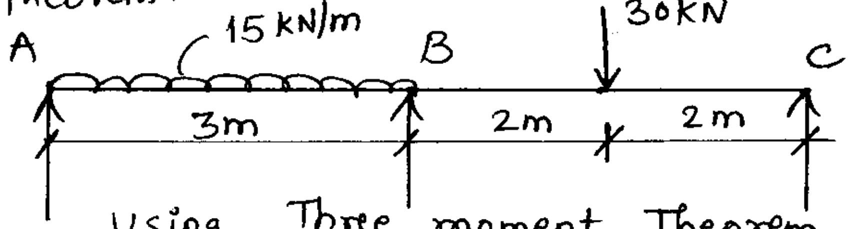
Putting F_{AB} in eq^n (I)

$$-F_{AE} - 0.948 \times -6.325 = 0$$

$$\left[F_{AE} = +6\,kN\right]$$ +ve sign indicates Tension. — 1 Mark

Question & Sub Question	Answer	Marks
	Consider joint E	
		
	For equillibrium $\Sigma F_x = 0$	
	$-F_{EF} + 6 = 0$	
	$\boxed{+F_{EF} = +6\ kN}$ +ve sign indicate Tension	1 Mark
	$\Sigma F_y = 0$	
	$-6 - F_{EB} = 0$	
	$\boxed{F_{EB} = -6\ kN}$ -ve sign indicate compression.	1 Mark
Q.4	Attempt Any FOUR of the following.	
4(a)	Concept of imaginary zero span :- This concept is used when one end of continuous beam is fixed and solve Problem using Clapeyron's Theorem. Consider following continuous beam.	
		2 Mark
	When the one of the end of continuous beam is fixed as shown in fig. an imaginary span AA' is considered whose length is zero. and the load acting on span is also zero such type of span is called zero span.	1 Mark
	First clapeyron's theorem is applied to span A'A and AB and then to span AB & BC to get two equations for M_A & M_B, These two equations can be solved simultaneously to obtain support moments M_A & M_B	1 Mark

Question & sub Question	Answer	Marks
4(b)	Calculate support moment using Three moment Theorem.	



Using Three moment Theorem.

$$M_A \cdot L_1 + 2M_B(L_1+L_2) + M_C(L_2) = -\left[\frac{6A_1\bar{x}_1}{L_1} + \frac{6A_2\bar{x}_2}{L_2}\right]$$ 1 Mark

To Calculate S.S. B.M.

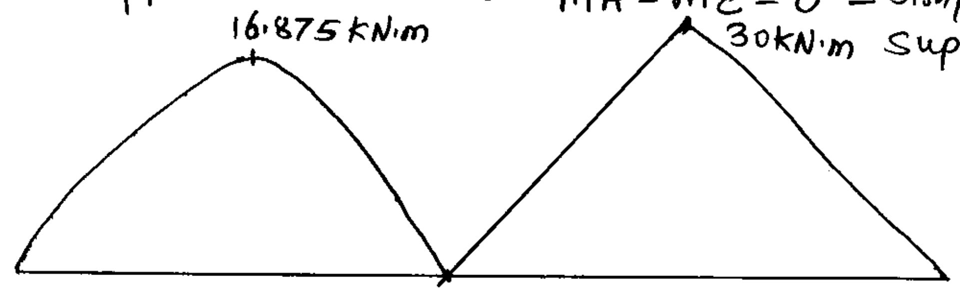
Consider span AB

$$M_{max} = \frac{wl^2}{8} = \frac{15 \times 3^2}{8} = 16.875 \text{ kN.m}$$

Span BC

$$M_{max} = \frac{WL}{4} = \frac{30 \times 4}{4} = 30 \text{ kN.m}$$ 1 Mark

Support moments $M_A = M_C = 0$ – Simple supports.



A_1 = Area of B.M. diag for span AB

$$A_1 = \frac{2}{3} \times b \times h = \frac{2}{3} \times 16.875 \times 3 = 33.75 \text{ kN.m}^2$$

A_2 = Area of B.M. diag for span BC

$$A_2 = \frac{1}{2} \times b \times h = \frac{1}{2} \times 4 \times 30 = 60 \text{ kN.m}^2$$

$$\bar{x}_1 = \frac{3}{2} = 1.5 \text{ m} \qquad \bar{x}_2 = \frac{4}{2} = 2\text{m}$$ 1 mark

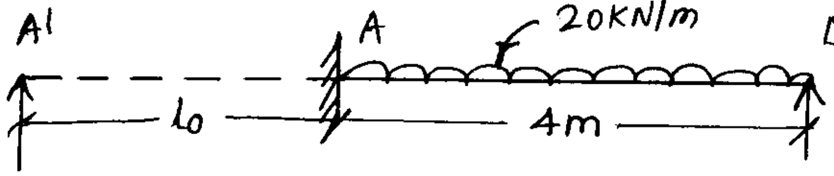
$$0 + 2M_B(3+4) + 0 = -\left[\frac{6 \times 33.75 \times 1.5}{3} + \frac{6 \times 60 \times 2}{4}\right]$$

$$2M_B(7) = -[101.25 + 180]$$

$$\boxed{M_B = -20.089 \text{ kN.m}}$$ 1 Mark

Question & sub Question	Answer	Marks

Q.4(c) Propped Cantilever Beam.



Consider the imaginary span AA' at fixed support having zero span and loading. Apply claperon's Theorem for span $A'A$ and AB

$$M_{A'}(L_0) + 2M_A(L_0+L_1) + M_B(L_1) = -\left[\frac{6A_0\bar{X}_0}{L_0} + \frac{6A_1\bar{X}_1}{L_1}\right]$$ — 1 mark

Simply Supported BMD.

for span $A'A = 0$

span AB $\quad M_{max} = \frac{20\times4^2}{8} = 40\ kN{\cdot}m.$

$A_0 = 0$, $\bar{X}_0 = 0$, $A_1 = \frac{2}{3}bh = \frac{2}{3}\times40\times4$

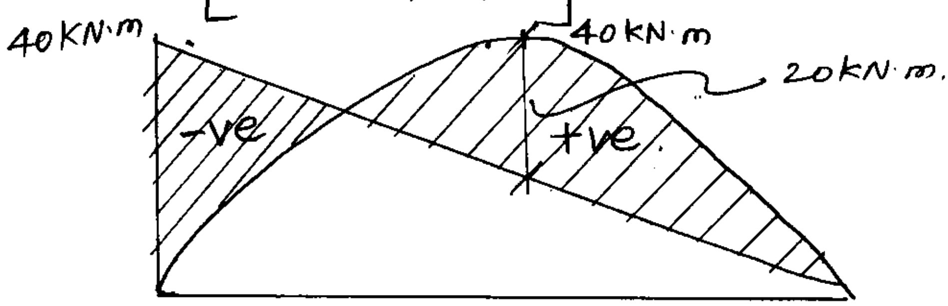
$A_1 = 106.67\ kN{\cdot}m^2$

$\bar{X}_1 = \frac{4}{2} = 2m.$ — 1 Mark

$M_B = 0$ – For simple support

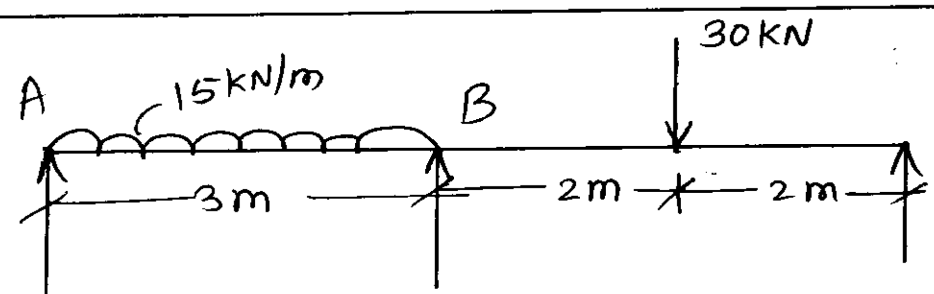
$$2M_A(0+4) = -\left[\frac{6\times0\times0}{0} + \frac{6\times106.67\times2}{4}\right]$$

$$\boxed{M_A = -40kN{\cdot}m}$$ — 1 Mark



FINAL BMD — 1 mark.

Question & sub Question	Answer	Marks
Q.4(d)		



1) To calculate fixed end moments.

Span AB
$$M_{FAB} = -\frac{wl^2}{12} = -\frac{15\times 3^2}{12} = -11.25\text{ kN·m}$$
$$M_{FBA} = +\frac{WL^2}{12} = +11.25\text{ KN·m}$$

Span BC
$$M_{FBC} = -\frac{WL}{8} = \frac{-30\times 4}{8} = -15\text{ kN·m}$$
$$M_{FCB} = +\frac{WL}{8} = \frac{30\times 4}{8} = +15\text{ kN·m}$$

1 Mark

2) Calculate Distribution Factor

Joint	Member	Stiffness Factor	Total Stiffness	Distribution factor
B	BA	$K_{BA} = \frac{3EI}{L}$ $K_{BA} = \frac{3}{3} = 1EI$	T.S. = 1+0.75 = 1.75 EI 1.75 EI	D.F. = $\frac{1}{1.75}$ $DF_{BA} = 0.571$
	BC	$K_{BC} = \frac{3EI}{L}$ $K_{BC} = \frac{3EI}{4} = 0.75EI$		$DF_{BC} = \frac{0.75\,EI}{1.75\,EI}$ $DF_{BC} = 0.429$

1 Mark

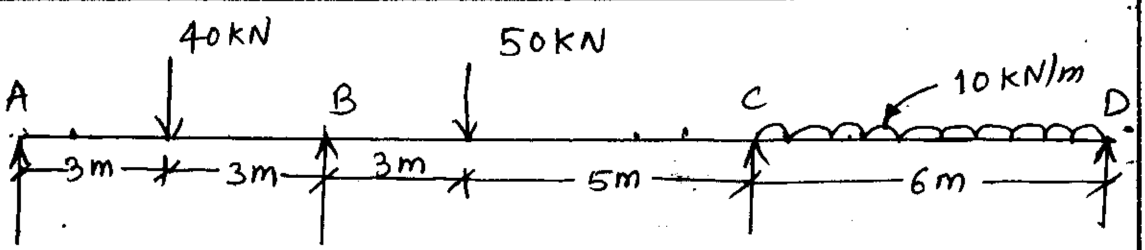
A	B		C	Remark
	0.571	0.429		
-11.25	+11.25	-15	+15	
+11.25 → 50%			50% ← -15	S.S. span mmt zero & Carry over
	+5.625	-7.5		
0	+16.875	-22.5	0	Balancing mmt.
	+3.211	+2.413		
0	+20.086	-20.086	0	

$\boxed{M_B = 20.086\text{ kN·m}}$ Hogging moment

2 Marks

Question & Sub Question	Answer	Marks

Q.4 (e)



1) To Calculate fixed end moments.

$$M_{FAB} = -\frac{WL}{8} = \frac{-40\times 6}{8} = -30\,kN\cdot m$$

$$M_{FBA} = +\frac{WL}{8} = +30\,kN\cdot m$$

$$M_{FBC} = -\frac{Wab^2}{L^2} = \frac{-50\times 3\times 5^2}{8^2} = -58.59\,kN\cdot m$$

$$M_{FCB} = +\frac{Wa^2b}{L^2} = \frac{50\times 3^2\times 5}{8^2} = 35.156\,kN\cdot m$$

$$M_{FCD} = \frac{-10\times 6^2}{12} = -30\,kN\cdot m$$

$$M_{FDC} = \frac{10\times 6^2}{12} = +30\,kN\cdot m$$

1 Mark

2) To Calculate Distribution factor

Joint	Member	Stiffness factor	Total stiffness	Distribution Factor (D.F.)
B	BA	$K_{BA} = \frac{3EI}{L} = \frac{3EI}{6}$ $K_{BA} = 0.5EI$		$DF_{AB} = \frac{0.5EI}{1EI}$ $DF_{AB} = 0.5$
	BC	$K_{BC} = \frac{4EI}{L} = \frac{4EI}{8}$ $K_{BC} = 0.5EI$	1EI	$DF_{BC} = \frac{0.5EI}{1EI}$ $DF_{BC} = 0.5$
C	CB	$K_{CB} = \frac{4EI}{L} = \frac{4EI}{8}$ $K_{CB} = 0.5EI$		$DF_{CB} = \frac{0.5EI}{1EI}$ $DF_{CB} = 0.5$
	CD	$K_{CD} = \frac{3EI}{6}$ $K_{CD} = 0.5EI$	1EI	$DF_{CD} = \frac{0.5}{1}$ $DF_{CD} = 0.5$

1 Mark

Question & Sub Question	Answer	Marks
	3) Calculation of Moment Distribution Method.	

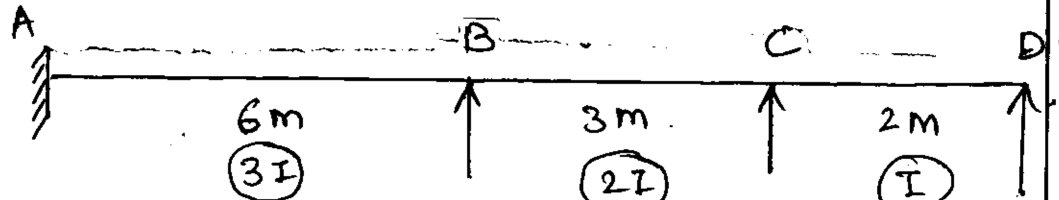
						Remark
	0.5	0.5	0.5	0.5		
−30	+30	−58.59	+35.156	−30	+30	
+30 50% →	+15			−15	← 50% −30	S.S. mmt zero & Carryover
0	+45	−58.59	+35.156	−45	0	
	+6.795	+6.795	+4.922	+4.922		Balancing moment
0	+51.795	−51.795	+40.078	−40.078	0	

Support Moments.

$M_A = 51.795$ kN.m

$M_B = 40.078$ kN.m — 2 Marks.

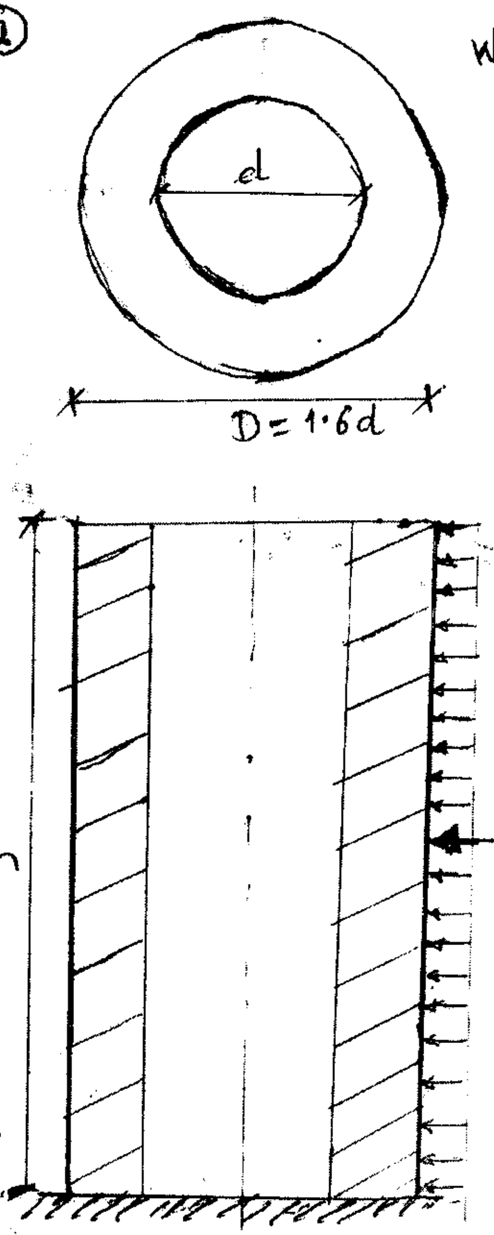
Q.4(f) Calculation of Distribution Factor



Joint	Member	Stiffness Factor	Total Stiffness	Distribution Factor	Marks
B	BA	$K_{BA} = \frac{4EI}{L} = \frac{4(3EI)}{6}$ $K_{BA} = 2EI$	ΣK_B $= 2 + 2.667$ $= 4.667EI$	$DF_{BA} = \frac{2EI}{4.667EI}$ $DF_{BA} = 0.428$	1 Mark
	BC	$K_{BC} = \frac{4EI}{L} = \frac{4(2I)}{3}$ $K_{BC} = 2.667EI$		$DF_{BC} = \frac{2.667EI}{4.667EI}$ $DF_{BC} = 0.57D$	1 Mark
C	CB	$K_{CB} = \frac{4EI}{L} = \frac{4E(2I)}{3}$ $K_{CB} = 2.667EI$	ΣK_C $= 2.667EI + 1.5EI$ $= 4.167EI$	$DF_{CB} = \frac{2.667EI}{4.167EI}$ $DF_{CB} = 0.64$	1 Mark
	CD	$K_{CD} = \frac{3EI}{L} = \frac{3E(I)}{2}$ $K_{CD} = 1.5EI$		$DF_{CD} = \frac{1.5EI}{4.167EI}$ $DF_{CD} = 0.36$	1 Mark.

Question and Model Answers	Marks
Q.5 Attempt Any TWO —————— 2x8	(16)

(a)



Where $h = 32\ m$

$\gamma_m = 18\ kN/m^3$

$C = 0.6$

$p_d = 1.75\ kN/m^2$

$f_{max} = ?$

$f_{min} = ?$

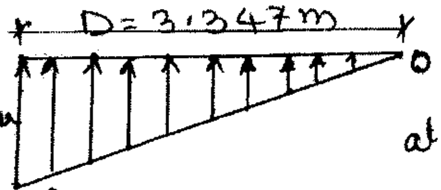
No tension at base

Direct stress due to self weight (f_d):-

$$f_d = \frac{\gamma_m A h}{A} = 18 \times 32 = 576\ kN/m^2$$ (1M)

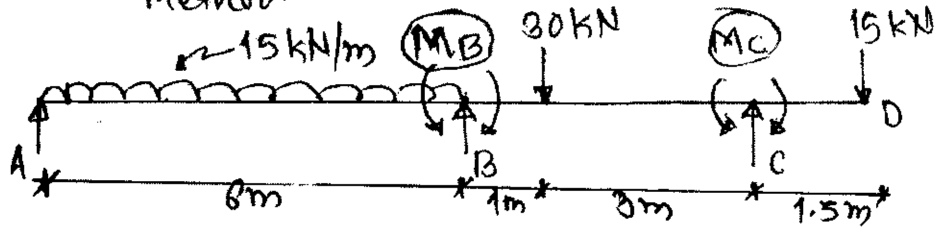
Bending Stress due to wind load (f_b):-

Bending Moment due to wind force (M)

Question and Model Answers	Marks
$M = P \times h/2$	
$M = (0.6 \times 1.75)(D \times 32)(32/2)$	(2M)
$\underline{M = 860.16\,d \text{ kN.m}}$ $\quad \because D = 1.6d$	
Moment of Inertia (I) about bending Axis	
$I = \dfrac{\pi(D^4 - d^4)}{64} = \dfrac{\pi((1.6d)^4 - d^4)}{64}$	(1M)
$\underline{I = 0.273\,d^4 \text{ m}^4}$	
& Distance of extreme fibre ~~from~~ N.A.	
$y = \dfrac{D}{2} = \dfrac{1.6d}{2} = 0.8D.$	(1M)
$\because f_b = \dfrac{M}{I} \cdot y = \dfrac{860.16\,d \times 0.8d}{0.273 d^4}$	
$\therefore \underline{f_b = 2520.62/d^2}$ kN/m²	(1M)
To avoid tension at base of chimney,	
$f_d = f_b$	
$\therefore 576 = \dfrac{2520.62}{d^2}$	
$\therefore$ Internal Diameter $= \underline{d = 2.092\text{m}}$ & External Diameter $\underline{= D = 3.347\text{m}}$	(1M)
Extreme fibre stress at the base of chimeny 	(1M)
$f_d + f_b = f_{max}$ $2(576) = 1152$ kN/m² @ the leeward side.	
$f_{min} = f_d - f_b$ $f_{min} = 0$ kN/m² at the windward side	

Question and Model Answers	Marks

Q5 (b) Draw BMD from Moment Distribution Method.



Distribution factors At jt. B. - DF

Jt. B - BA Relative stiffness $= 3EI/6$ — 0.4

BC $= \frac{3EI}{4}$ — 0.6 (1M)

Total stiffness $= 1.25\,EI$. $\Sigma = 1.0$

Fixed End moments (FEM)

$M_{AB} = \frac{15\times(6)^2}{12} = -45$ kN.m (anticlockwise) (½M)

$M_{BA} = \frac{15\times(6)^2}{12} = +45$ kN.m (clockwise) (½M)

$M_{BC} = \frac{30\times1\times(3)^2}{(4)^2} = -16.875$ kN.m (anticlockwise) (½M)

$M_{CB} = \frac{30\times(1)^2\times3}{(4)^2} = +7.5$ kN.m (clockwise) (½M)

$M_{CD} = 15\times1.5 = -22.5$ kNm (anticlockwise) (½M)

Sagging BM. for (s/s for every span)

$M^+_{AB} = \frac{15\times(6)^2}{8} = +67.5$ kN.m (½M)

$M^+_{BC} = \frac{30\times1\times9}{4} = +22.5$ kN.m

Question and Model Answers	Marks

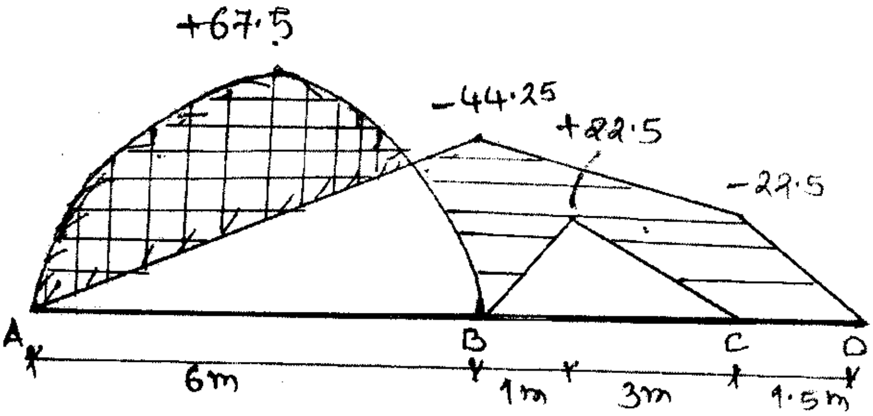
Moment Distribution Table

Joint	A	B	B	C	C	D
D.F.	-	0.4	0.6	1	0	-
FEM	-45	+45	-16.875	+7.5	-22.5	-
Release A & Balance C	+45	+22.5	+7.50	+15.0		
Initial Moment	0	+67.50	-9.375	+22.5	-22.5	-
Distribute 'B'		-23.25	-34.875			
Final Moment	0	+44.25	-44.25	+22.5	-22.5	-

(2M)

Release 'A' : Because of S/S end.

Balance 'C' : Because of overhang

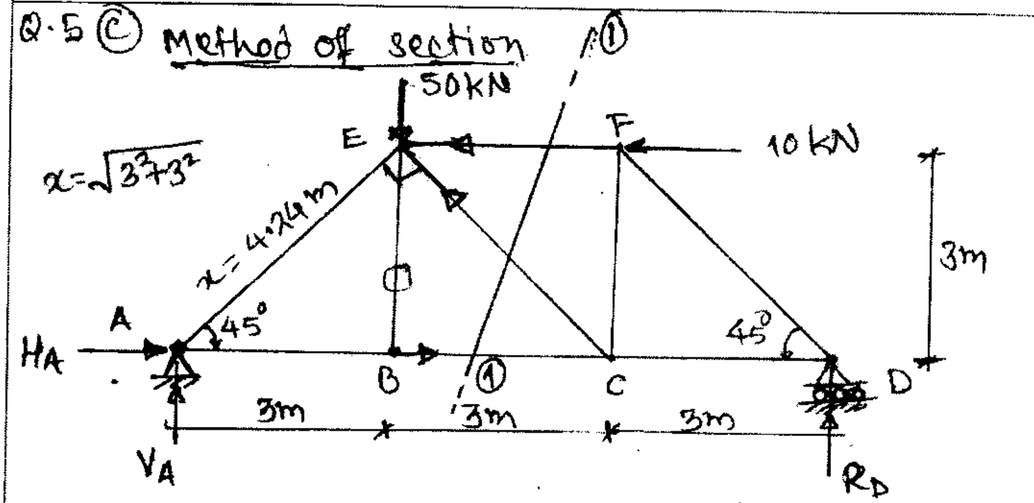


(2M)

BMD (kN.m)

Question and Model Answers	Marks

Q.5 (C) Method of section



Calculate Support Reaction Using condition of Equilibrium for whole truss.

$\Sigma F_x = 0 \quad \therefore \quad \underline{H_A = 10 \text{ kN}}$ (½ M)

$\Sigma M@A = 0 = +(50 \times 3) - (10 \times 3) - (R_D \times 9)$

$\therefore \quad \underline{R_D = 13.33 \text{ kN}}$ (½ M)

$\Sigma F_y = 0 \quad \therefore \underline{V_A} = 50 - 13.33 = \underline{36.67 \text{ kN}}$ (½ M)

Taking section @ (1)(1) as shown in fig, - & using condition of Equilibrium as follows for sections, -

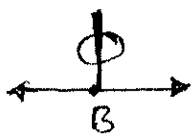
Taking moment @ E

$\Sigma M_E = 0 = +(36.67 \times 3) - (10 \times 3) - (BC \times 3)$

$\therefore \quad \boxed{F_{BC} = +26.67 \text{ kN}}$ Assumed direction is ok $\therefore$ Tension (½ M)

Taking moment @ C

$\Sigma M_C = 0 = +(36.67 \times 6) - (50 \times 3) - (F_{EF} \times 3)$

Question and Model Answers	Marks
∴ $\boxed{F_{EF} = +23.34\ kN}$ ∴ Assumed direction is ok. ∴ compressive	($\frac{1}{2}$M)
<u>Taking moment @ A</u>	
$\Sigma M_A = +(50 \times 3) - (F_{EC} \times 4.24) - (23.34 \times 3) = 0$	
∴ $\boxed{F_{EC} = +18.86\ kN}$ ∴ Assumed direction is ok. ∴ Compressive	(1M)
∴ At joint B for equilibrium, $\boxed{F_{BE} = 0}$ 	($\frac{1}{2}$M)
<u>Result in Tabulate form</u>	

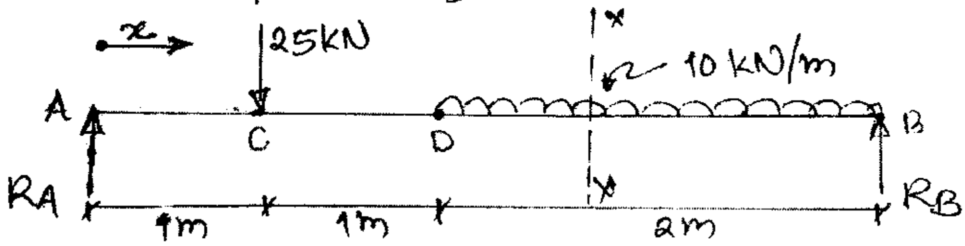
SR No.	MEMBER	MAGNITUDE (KN)	NATURE
1.	BC	26.67	Tension
2.	BE	0	—
3.	EF	23.34	Compressive
4.	EC	18.86	Compressive

(2M)

Question and Model Answers	Marks
Q. 6 Attempt Any TWO — 2×8	16

Q. 6. (a) Slope & deflection by Macaulay's Method.

$\theta_A = ?$ $\quad Y_B = ?$ $\quad EI = 40{,}000\ kN \cdot m^2$



Using conditions of Equilibrium,-

$\Sigma M@A = 0 = +(25\times1) + (10\times2\times(\frac{2}{2}+2)) - (R_B\times4)$ (1M)

$\therefore \boxed{R_B = 21.25\ kN}$

$\Sigma F_y = 0 = -25 - (10\times2) + (21.25) + R_A$ (1M)

$\therefore \boxed{R_A = 23.75\ kN}$

For Macaulay's Method, section x-x as shown in fig with origin @ A.

$M_{xx} = 23.75x \,|\, -(25)(x-1) \,|\, -10\frac{(x-2)^2}{2}$ (1M)

Using Macaulay's Method,

$EI\frac{d^2y}{dx^2} = M_{xx} = 23.75x \,|\, -25(x-1) \,|\, -5(x-2)^2$ (1M)

Integrating on both side,

$EI\frac{dy}{dx} = \frac{23.75x^2}{2} + C_1 \,|\, -\frac{25(x-1)^2}{2} \,|\, -\frac{5(x-2)^3}{3}$ (1M)

Integrating on both side,-

$EIy = \frac{23.75}{6}x^3 + C_1x + C_2 \,|\, -\frac{25(x-1)^3}{6} \,|\, -\frac{5(x-2)^4}{12}$ (1M)

Question and Model Answers	Marks

Using boundary Condition,-

i) at $x=0$, $y=0$

$$EIy = 0 = C_2 = 0$$

ii) @ $x=4$, $y=0$

$$0 = EIy = \frac{23.75(4)^3}{6} + C_1(4) + 0 \Big| -\frac{25(4-1)^3}{6} \Big| -\frac{5(4-2)^4}{12}$$

$$\therefore C_1 = -33.54$$

for slope at end span @ $x=0$

$$EI\frac{dy}{dx} = \frac{23.75x^2}{2} - 33.54 \Big| -\frac{25(x-1)^2}{2} \Big| -\frac{5(x-2)^3}{3}$$

$$EI\theta_A = -33.54$$

$$\boxed{\theta_A = \frac{-33.54}{40,000} = 0.8385 \times 10^{-3}\ rad} \qquad (1)$$

For displacement at mid span @ $x = 2m$

$$EIy = \frac{23.75x^3}{6} - 33.54(x) \Big| -\frac{25(x-1)^3}{6} \Big| -\frac{5(x-2)^4}{12}$$

$$EIy_D = \frac{23.75(2)^3}{6} - 33.54(2) \Big| -\frac{25(2-1)^3}{6} \Big| -\frac{5(2-2)^4}{12}$$

$$\therefore EIy_D = -39.58$$

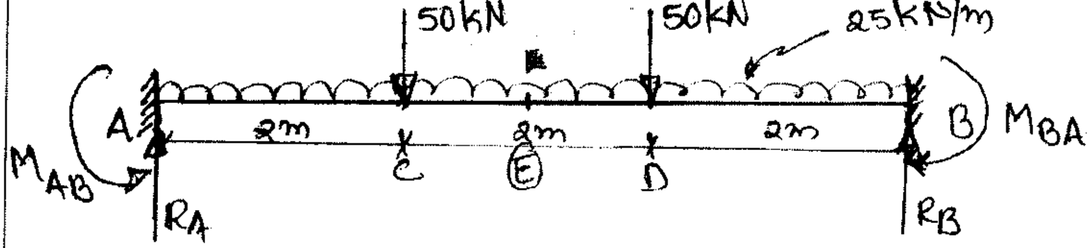
$$y_D = \frac{-39.58}{40,000} = -0.9895 \times 10^{-3}\ m$$

$$\therefore \boxed{y_D = -0.9895\ mm}$$

-ve sign indicates vertically downward displacement (1)

Question and Model Answers	Marks

Q.6 (b) SFD & BMD of fixed Beam



As symmetrical loading,

$$R_A = R_B = \frac{50 + 50 + (25 \times 6)}{2} = 125\text{ kN}$$ (1M)

$$\& \; M_{AB} = M_{BA} = \frac{50(2)(4)^2}{(6)^2} + \frac{50(4)(2)^2}{(6)^2} + \frac{25(6)^2}{8}$$ (1M)

(1M)

$\therefore M_{AB} = -141.66$ kN.m (anticlockwise)

$M_{BA} = +141.66$ kN.m (clockwise)

S.F. calculation

SF. @ A = 125 kN

SF. @ C = $125 - 25 \times 2 \times \frac{2}{2} = 75$ kN (1M)

SF @ C with 50 kN = +75 − 50 = +25 kN

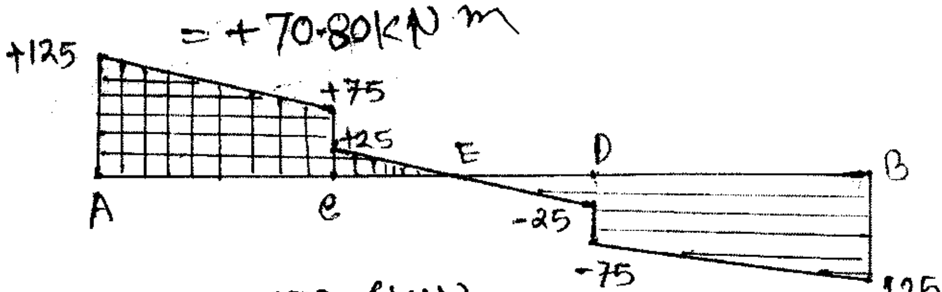
BM. calculation

BM @ A & B = −141.66 kN.m

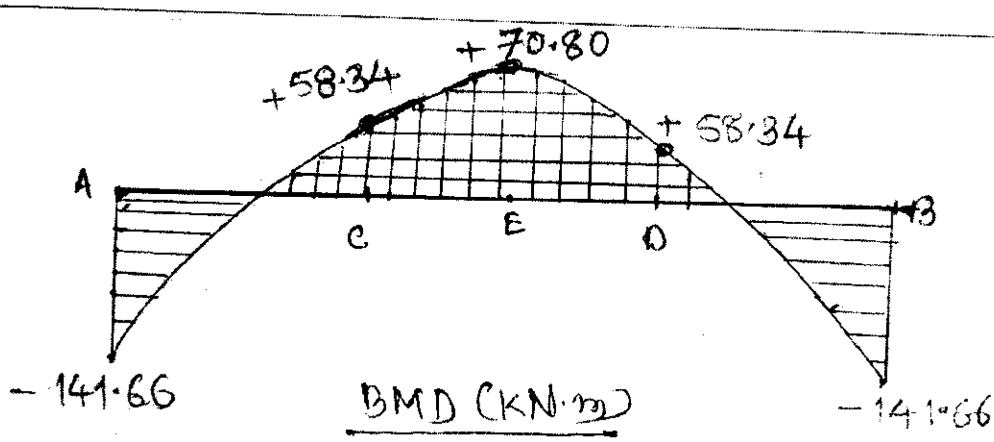
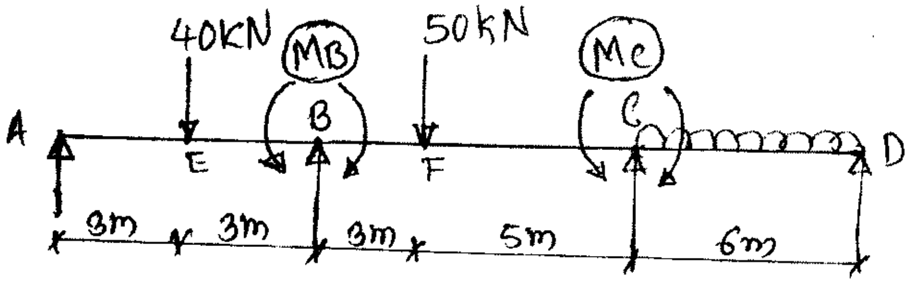
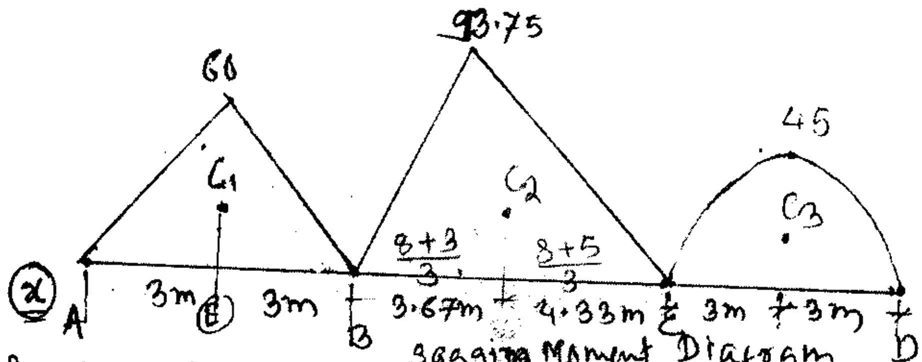
BM @ C & D = $-141.66 + (125 \times 2) - (25 \times 2 \times \frac{2}{2}) = +58.34$ kN.m (1M)

BM @ E = $-141.66 + (125 \times 3) - (25 \times 3 \times \frac{3}{2}) - (50 \times 1)$

= +70.80 kN m



SFD (KN) (1M)

Question and Model Answers	Marks
 BMD (kN.m)	(2M)
Q.6 (c) Support Reactions & Moments by Three moment theorem.   Sagging Moment Diagram	(1M)
Sagging Moments	
Span AB $= \frac{40 \times (6)}{4} = 60$ kN.m	(½M)
Span BC $= \frac{50 \times 3 \times 5}{8} = 93.75$ kN.m	(½M)

Question and Model Answers	Marks
span CD $= \frac{10 \times 6^2}{8} = 45$ kN·m	(½M)
Area of Moment diagram :-	(½M)
for span AB $= a_1 = \frac{1}{2} \times 6 \times 60 = 180$	(½M)
for span BC $= a_2 = \frac{1}{2} \times 8 \times 93.75 = 375$	(½M)
for span CD $= a_3 = \frac{2}{3} \times 6 \times 45 = 180$	
Using Three Moment Theorem for	
$\boxed{\text{Pair-ABC}}$	
$(\cancel{M_A \times 6}) + 2M_B(6+8) + (M_C \times 8) = -\frac{6(180 \times 3)}{6} - \frac{6(375)(4.33)}{8}$	
As $M_A = 0$, (s/s at end)	
$\therefore \quad 28M_B + 8M_C = -1757.82 \quad \cdots (1)$	(1M)
$\boxed{\text{Pair B-C-D}}$	
$(M_B \times 8) + 2M_C(8+6) + (\cancel{M_D \times 6}) = -\frac{6(375)(3.67)}{8} - \frac{6(180)(3)}{6}$	
As $M_D = 0$, (s/s at end)	
$8M_B + 28M_C = -1572.19 \quad \cdots (2)$	(1M)
solving eqn (1) & (2)	
$\boxed{M_B = -50.89 \text{ kN.m}}$ (Hogging)	(½M)
& $\boxed{M_C = -41.61 \text{ kN.m}}$ (Hogging)	(½M)

Question and Model Answers	Marks

<u>for Support Reaction</u>

① Take section at B & moment at Left side of B

$(R_A \times 6) - (40 \times 3) = -50.89$

$\therefore \boxed{R_A = +11.52\,kN}$

② Take section @ C & moment at Right side C

$(R_D \times 6) - (10 \times 6 \times \frac{6}{2}) = -41.61$

$\therefore \boxed{R_D = +23.07\,kN}$ (½M)

③ Take section @ C & moment at Left side of C

$(11.52 \times 14) - (40 \times 11) + (R_B \times 8) - (50 \times 5) = -41.61$

$\boxed{R_B = +60.89\,kN}$

④ Using $\Sigma F_y = 0$ for overall beam,

$R_A + R_B + R_C + R_D = 40 + 50 + (10 \times 6)$ (½M)

$\therefore \boxed{R_C = 54.52\,kN}$

———X———

24/11/16