SUMMER – 14 EXAMINATIONS

Subject Code: 17422 **Model Answer** **Total Pages: 41**

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 Examiners:-

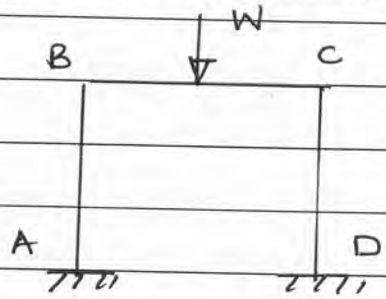
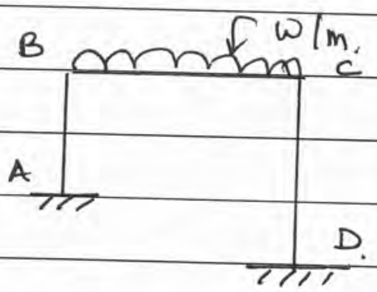
1) In Q. 1 a)-vi) in this question types of port frame is asked but it is not correct, therefore it is consider as portal frame, and examiner may consider the same for giving marks.

2) In Q. 2 c) in this problem the direction of wind pressure acting on which side or face is not mentioned, therefore two solutions are prepared, examiner may consider any one for giving marks.

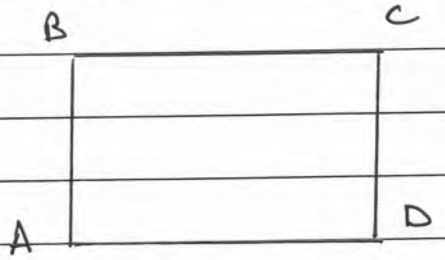
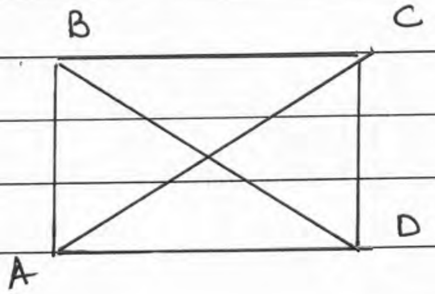
SUMMER – 14 EXAMINATION

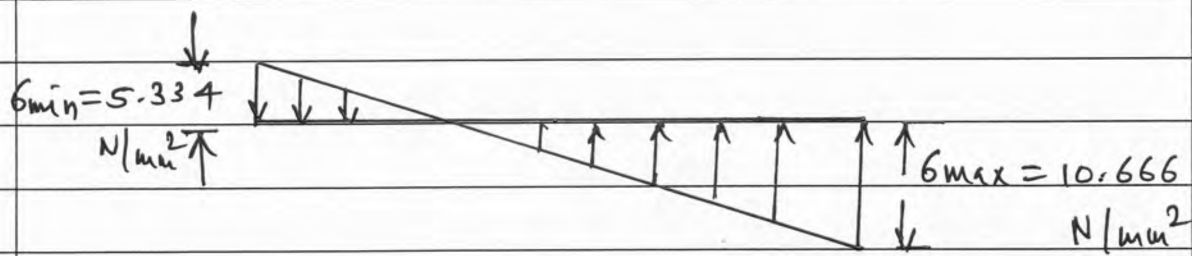
Subject Code: 17422 **Model Answer**

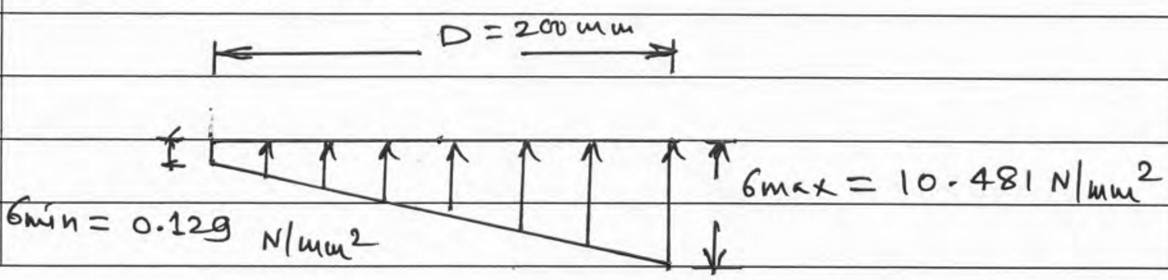
Q .NO	SOLUTION	MARKS
1(a)(i)	**Eccentricity** : The distance between the geometric axis of the body and the line of action	1
	Loading is called an eccentricity.	
	Bending Stress: The stress induced to resist the bending moment due to eccentric load is	1
	called bending stress	
(ii)	**Slope at free end of Cantilever :** $\theta = \frac{WL^2}{6.EI} = \frac{w.l^3}{6EI}$ ∴ where ∴ w = Rate of loading	1
	Deflection at free end of Cantilever : $y = \frac{WL^3}{8.EI} = \frac{w.l^4}{8EI}$ ∴ W = Total load on span.	1
	Where W = Point load acting at free end of cantilever. E = Modulus of Elasticity.	
	L= Span of the cantilever beam. I = Moment of Inertia of beam.	
(iii)	**Defination of Slope of beam:** The slope at any point on the elastic curve of the beam is	1
	defined as the angle in radians that the tangent at that point makes with the original axis	
	of the beam.	
	Defination of deflection of beam: The deflection at any point on the axis of the beam is	1
	the distance between its positions before and after loading.	
(iv)	**Boundary conditions for cantilever beam :**	
	To evaluate C1 : At x = 0, $\frac{\partial y}{\partial x} = 0$. Where x = distance of section from fixed end.	1
	To evaluate C2 : At x = 0, y =0	1
(v)	**Advantages of fixed beam over simply supported beam:**	
	(1) Due to end fixity ,end slope of a fixed beam is zero.	½ for
	(2) A fixed beam is more stronger,stiffer and stable.	any Two
	(3) For same span and loading ,fixed beam has lesser value of Bending moment.	
	(4) Smaller moment permits smaller sections and there is saving in beam material.	
	(5) Fixed beam has lesser deflection for same span and loading as compared to S.S. beam	
	Disadvantages of fixed beam over simply supported beam:	½ for
	(1) A little sinking or settlement of support induces additional moment at each support.	any Two

Q .NO	SOLUTION	MARKS
1 (a)	(2)Secondry stresses are developed due to change in temperature, poor workmanship or	
(v)	Deforrmation of members.	
Cont--	(3)Frequent fluctuations in loading due to moving loads are likely to disturb the end fixity.	
	(4) Extra care has to be taken to achieve correct fixity at supports	
(vi)	**Portal Frame :** A horizontal beam resting on two vertical or two identical columns Whose both or one end may be fixed or hinged known as portal frame.	1
	(1) Symmetrical portal frame . (2)Unsymmetrical portal frame.	
	 	1 for two sketches.
(vii)	**Carry over Moment** :It is defined as 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.	1
	Carry over Factor : It is defined as the ratio of carry over moment to applied moment is called carry over factor.	1
(viii)	**Perfect Frame :** A fame in which the number of members are just sufficient to keep it in Stable equilibrium while carrying applied loads at its joints is called a perfect frame.	1
	Example : A triangular frame is a basic perfect frame. It has three members and three joints	1
	Triangular perfect frame satisfies the relation **n = 2j – 3**	
	In this frame $n = 3, J = 3$ $\therefore n = 2J - 3$ $\therefore 3 = 2 \times 3 - 3$ $\therefore 3 = 3$ OK. $\therefore$ Hence perfect frame.	

Q .NO	SOLUTION	MARKS
1(b) (i)	Direct stress $\sigma o = \frac{P}{A}$	
	$\sigma o = \frac{P}{800*1200}$	1/2
	Bending Stress $\sigma b = \frac{P*e*y}{d*b2/6}$	1/2
	$\sigma b = (6P * e)/(800 * 1200^2)$	
	For no tension at base $\sigma o = \sigma b$	1
	$\frac{P}{800*1200} = (6P * e)/(800 * 1200^2)$	
	e = 1200/6= 200 mm **or**	1
	e = 800/6 = 133.333 mm	1
(ii)	Data given. D = 400mm	
	Thicknes(t) = 25 mm	
	d = 350 mm,P = 200KN	
	e = 50 mm	
	C/s Area A $= \pi/_4 * (400^2 - 350^2) = 29452.431\ mm^2$	1/2
	Direct Stres $\sigma o = \frac{P}{A}$	
	$\sigma o = \frac{200000}{29452.431}$	
	$\sigma o = 6.791\ N/mm^2$	1/2
	$I = \frac{\pi}{64}(D^4 - d^4) = \frac{\pi}{64}(400^4 - 350^4) = 5.2 * 10^8\ N/mm^2$	1/2
	Bending Stress $\sigma b = \frac{P*e*y}{\frac{\pi}{64}(D^4-d^4)} = \frac{200000*50*200}{5.2*10^8}$	
	$\sigma b = 3.846\ N/mm^2$	½
	Maximum Stress σ max. $= \sigma o + \sigma b = 6.791 + 3.846$	
	σ max. $= 10.637\ N/mm^2$	1
	Minimum Stress σ min. $= \sigma o + \sigma b = 6.791 - 3.846$	
	σ min. $= 2.945\ N/mm^2$	1

Q .NO	SOLUTION	MARKS
1(b)(iii)	**Deficient Frame** : A frame in which number of members are less than that required for perfect frame is called deficient frame.	1
	Mathematically, for deficient frame **n** $< 2j - 3$	
	 In this frame $n=4$, $j=4$ $n < 2j-3$. $4 < 2\times4-3$ $4 < 5$ (O.K.) Hence deficient frame	1 for sketch
	Redundent Frame: A frame in which number of members are more than that required for perfect frame is called deficient frame	1
	Mathematically, for deficient frame $\mathbf{n} > \mathbf{2j - 3}$	
	 In this frame $n=6$, $J=4$ $n > 2j-3$ $6 > 2\times4-3$ $6 > 5$ (O.K.) Hence Redundant frame.	1 for sketch
2 (a)	Given Data	
	b= 300 mm, d = 100 mm.	
	Load =P= 80KN=80000N	
	e = 50 mm	
	(1) Direct Stress (σo)	
	$\sigma o = \frac{P}{A}$	

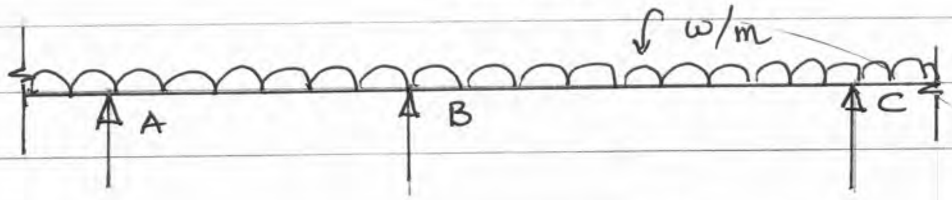
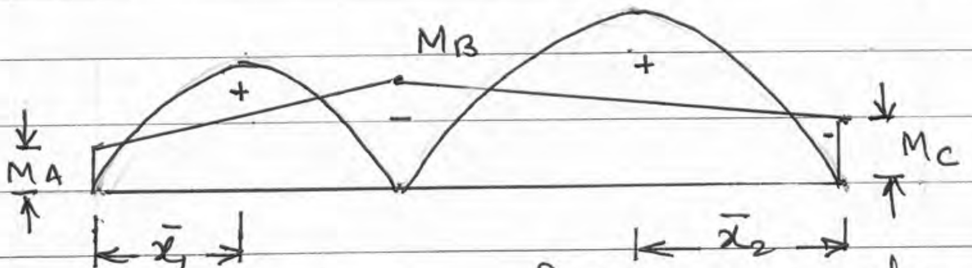
Q .NO	SOLUTION	MARKS
	$\sigma o = \frac{80000}{300*100}$	
	$\sigma o = 2.666\ N/mm^2$	½
	(2) Modulus of Section (Z)	
	$Z = b\ d^2/6$	1/2
	$Z = 300 * 100^2/6$	
	$Z = 5 * 10^5\ mm^3$	1/2
	(3) Bending Stress (σb)	
	$\sigma b = P * e/Z$	½
	$\sigma b = 80000 * 50/5 * 10^5$	
	$\sigma b = 8\ N/mm^2$	1/2
	(4) Maximum Stress σ max. = $\sigma o + \sigma b$ = 2.666 + 8	
	σ max. = 10. 666 N/mm^2	½
	(5)Minimum Stress σ min. = $\sigma o - \sigma b$ = 2.666 − 8	
	σ min. = - 5.334 N/mm^2	1/2
	(6) Stress Distribution Diagram.	
		1/2
		OR
	OR **Resultant Stresses can found by direct Formulae**	
	σ max = $\frac{P}{A}\left[1 + \frac{6*e}{d}\right] = \frac{80000}{300*100}\left[1 + \frac{6*50}{100}\right]$ = 10. 666 N/mm^2	
	σ min = $\frac{P}{A}\left[1 - \frac{6*e}{d}\right] = \frac{80000}{300*100}\left[1 + \frac{6*50}{100}\right]$ = - 5.334 N/mm^2	

Q .NO	SOLUTION	MARKS
2(b)	(1) C/s Area (A)	
	$A = \frac{\pi}{4}(D^2 - d^2) = \frac{\pi}{4}(200^2 - 160^2)$	
	$A = 11309.734\ mm^2$	½
	(1) Direct Stress (σo)	
	$\sigma o = \frac{P}{A}$	
	$\sigma o = \frac{60000}{11309.734}$	
	$\sigma o = 5.305\ N/mm^2$	½
	(2) Modulus of Section (Z)	
	$Z = \frac{\pi}{64}(D^4 - d^4)/100$	½
	$Z = 463699.075\ mm^3$	½
	(3) Bending Stress (σb)	
	$\sigma b = P * e/Z$	½
	$\sigma b = 60000 * 40/463699.075$	
	$\sigma b = 5.176\ N/mm^2$	1/2
	(4) Maximum Stress σ max. $= \sigma o + \sigma b = 5.305 + 5.176$	
	σ max. $= 10.481\ N/mm^2$	½
	(5)Minimum Stress σ min. $= \sigma o - \sigma b = 5.305 - 5.176$	
	σ min. $= 0.129\ N/mm^2$	½
	(6) Stress Distribution Diagram.	
		½

Q .NO	SOLUTION	MARKS
2(c)	**Note : In this problem wind pressure acting on which side or face is not mentioned,**	
	Therefore Two solutions are prepared Examiner should consider any one for giving marks.	
	Solution – (I) Assuming wind pressure acting on 3m side	
	(1) Direct Stress $(\sigma o) = \rho * h$	
	$(\sigma o) = 22 * 10 = 220$ KN/m^2	1/2
	(2) Total wind pressur (P)	
	P = p * projected area	
	P = 1.2 *(3*10)	
	P = 36 KN	½
	(3) Bending Moment of Total wind pressure about base (M)	
	M = P *h / 2	
	M = 36 *10 / 2	
	M = 180 KN-m	½
	(2) Modulus of Section (Z)	
	$Z = bd^2/6 = 3 * 1.5^2/6 = 1.125\ m^3$	1
	(3) Bending Stress (σb)	
	$\sigma b = M/Z = 180/1.125$	
	$\sigma b = 160\ KN/m^2$	½
	(4) Maximum Stress σ max. = $\sigma o + \sigma b = 220 + 160$	
	σ max. = 380 KN/m^2	½
	(5)Minimum Stress σ min. = $\sigma o - \sigma b = 220 - 160$	
	σ min. = 60 KN/m^2	½

Q .NO	SOLUTION	MARKS
2(c)	**Solution – (II)** Assuming wind pressure acting on 1.5m side	
	(1) Direct Stress (σo) = $\rho * h$	
	$(\sigma o) = 22 * 10 = 220$ KN/m^2	½
	(2) Total wind pressur(P)	
	P = p * projected area	
	P = 1.2 *(1.5*10)	
	P = 18 KN	½
	(3) Bending Moment of Total wind pressure about base (M)	
	M = P *h / 2	
	M = 18 *10 / 2	
	M = 90 KN-m	1/2
	(2) Modulus of Section (Z)	
	$Z = db^2/6 = 1.5 * 3^2/6 = 2.25\ m^3$	1
	(3) Bending Stress (σb)	
	$\sigma b = M/Z = 90/2.25$	
	$\sigma b = 40\ KN/m^2$	½
	(4) Maximum Stress σ max. = $\sigma o + \sigma b = 220 + 40$	
	σ max. = 260 KN/m^2	½
	(5)Minimum Stress σ min. = $\sigma o - \sigma b = 220 - 40$	
	σ min. = 180 KN/m^2	½

Q .NO	SOLUTION	MARKS
2(d)	Given Data	
	b = 130 mm. d = 240 mm.	
	L= 2500mm, W = 6KN= 6000 N	
	E = 1 * 10^5 N/mm2	
	(1) Moment of Inertia (I)	
	I = $bd^3/12$	
	I = $130 * 240^3/12$	
	I = 14976 *10^4 mm^4	1
	(2) Deflection at free end(y)	
	$y = \frac{WL^3}{3EI}$	1/2
	$y = \frac{WL^3}{3EI} =$ 6000 * 2500^3 /3*1*10^5 *14976 *10^4	1/2
	y = 2.087 mm	1/2
	(2) Slope at free end(θ)	
	$\theta = \frac{WL^2}{2EI}$	1/2
	$\theta = \frac{6000 * 2500^2}{2 * 1 * 10^5 \ * 14976 \ * 10^4 \ mm^4}$	1/2
	θ = 1.252 *10^{-3} = 0.001252 $radians$	1/2
	.	

Q.NO	SOLUTION	MARKS
Q2(e)		
		$\frac{1}{2}$

Clapeyron's Theorem of three moments.

If AB and BC are any two consecutive spans of a continuous beam having uniform moment of Inertia, supported at A, B and C and subjected to an external loading, the support moments M_A, M_B and M_C at supports A, B and C are given by the relation.

$$M_A \times L_1 + 2M_B(L_1 + L_2) + M_C \times l_2 = -\left(\frac{6a_1\bar{x}_1}{l_1} + \frac{6a_2\bar{x}_2}{l_2}\right)$$ 2

Clapeyron's Theorem when M.I. of beam is varying

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

where

L_1 = Length of the span AB

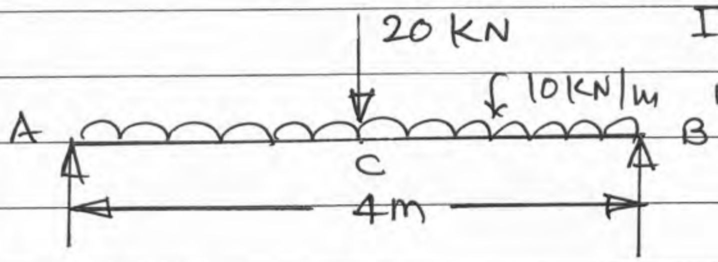
I_1 = Moment of Inertia of beam for span AB.

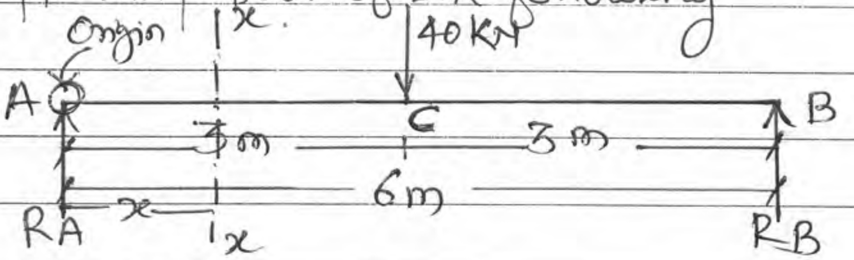
a_1 = Area of free B.M. diagram for the span AB

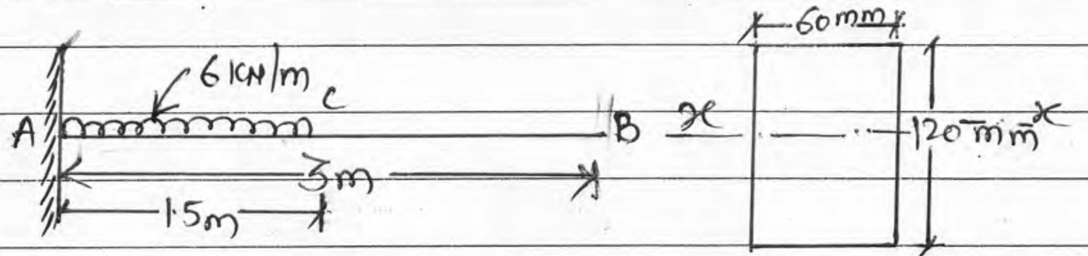
$\bar{x}_1$ = Distance of centroid of free BMD of span AB from 'A'. $\frac{1}{2}$

$\bar{x}_2$ = Distance of centroid of free BMD of span BC from 'C'

∴ Similarly L_2, I_2 and a_2 for the span BC.

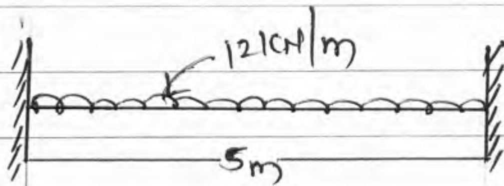
Q .NO	SOLUTION	MARKS
Q 2 (F)	Given Data.	
	$I_{xx} = 2 \times 10^8 \ mm^4$	
	$E = 2 \times 10^5 \ N/mm^2$	
		
	(1) Maximum slope	
	$\theta_{max} = \theta_A = \theta_B = \frac{WL^2}{16EI} + \frac{wL^3}{24EI}$	1
	$= \frac{20000 \times (4000)^2}{16 \times 2 \times 10^5 \times 2 \times 10^8} + \frac{10 \times 4000^3}{24 \times 2 \times 10^5 \times 2 \times 10^8}$	
	$= 5 \times 10^{-4} + 6.666 \times 10^{-4}$	
	$= 11.666 \times 10^{-4} = 1.166 \times 10^{-3}$ radians.	1
	(2) Maximum deflection (Y_{max})	
	$\therefore Y_{max} = Y_C = -\left(\frac{WL^3}{48EI} + \frac{5wL^4}{384EI}\right)$	1
	$\therefore Y_{max} = Y_C = -\left(\frac{20000 \times (4000)^3}{48 \times 2 \times 10^5 \times 2 \times 10^8} + \frac{5 \times 10 \times (4000)^4}{384 \times 2 \times 10^5 \times 2 \times 10^8}\right)$	
	$- (0.666 + 0.833$	
	$\therefore Y_{max} = Y_C = -1.50 \ mm$	1

Q .NO	SOLUTION	MARKS
Q.3.	Attempt any four of the following	16
a)		
	Step No. 1 Determination of Support Reaction.	
	As the pt. load is central, The beam is symmetrical @ C	
	$\therefore$ $RA = RB = \frac{w}{2} = \frac{40}{2} = 20$ kN.	
	Step. No. 2. Determination of ~~Sup~~ Moment @ x-x	
	$\therefore$ $Mx = RA \times x = 20 \times x$. — eqn (1).	01
	Step No. 3. Differential Equation.	
	$EI \frac{d^2y}{dx^2} = Mx$	
	Substitute the value of Mx in above eqn.	
	$EI \frac{d^2y}{dx^2} = 20 \times x$.	
	Integrate above equation w.r.to x.	
	$EI \frac{dy}{dx} = \frac{20x^2}{2} + C_1$ —— eqn (2).	
	Again Integrate above eqn w.r.to x.	
	$EI y = \frac{20x^3}{6} + C_1x + C_2$ —— eqn (3).	01
	Step No. 4 Determination of Integrating constant. For Slope and deflection.	
	Apply Boundary conditions.	
	At origin A deflection $y=0$, i.e. at $x=0$; $y=0$	
	Substituting these values in eqn (3).	
	$\therefore$ $0 = 0 + 0 + C_2$ $\therefore$ $\boxed{C_2 = 0}$	01

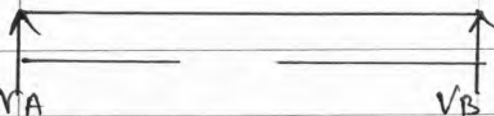
Q .NO	SOLUTION	MARKS
	The maximum slope will occur at A & B and at centre the deflection is maximum.	
	∴ At centre slope is zero.	
	∴ At centre, $x = L/2 = 6/2 = 3m$, $\theta = \frac{dy}{dx} = 0$	
	Substitute these value in eqn ②.	
	∴ $0 = \frac{20(3)^2}{2} + C_1$	
	$\boxed{C_1 = -90}$	01
	∴ $\boxed{C_1 = -90 ;\ C_2 = 0}$	
b)		
	Given data:- $b = 60\,mm$, $d = 120\,mm$, $E = 1\times10^5\,N/mm^2 = 1\times10^8\,KN/m^2$	
	The moment of Inertia @ x-x axis	
	$I = I_{xx} = \frac{bd^3}{12} = \frac{60\times120^3}{12} = 8.64\times10^6\,mm^4 = 8.64\times10^{-6}\,m^4$	01
	$EI = 8.64\times10^6 \times 1\times10^5 = 8.64\times10^{11}\,N\cdot mm^2 = 864\,KN\cdot m^2$.	
	The slope at free End. is.	
	$\theta_B = \frac{WL_1^3}{6EI} = \frac{6\times(1.5)^3}{6\times864}$	01
	$\boxed{\theta_B = 3.906\times10^{-3}\ rad.}$	2
	OR $\boxed{\theta_B = 0.224°.}$	

Q.NO	SOLUTION	MARKS

c)



Step No ① Determination of support Reaction, Due to symmentry & $M_A = M_B$, since the fixed End Moments are equal.



∴ Reaction due fixed End Momet = Reaction of S. supported beam

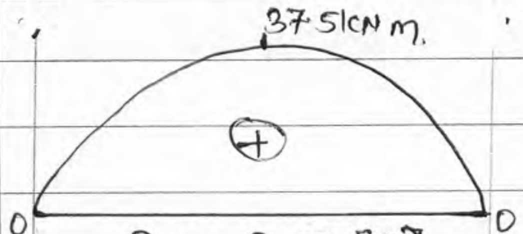
$R_A = V_A = R_B = V_B = \frac{wL}{2} = \frac{12 \times 5}{2} = 30kN$ — 01

∴ $R_A = R_B = 30kN$.

Step No. ② Determination of drawing free B.M.

Free B.M at $C = \frac{wl^2}{8}$

$= \frac{12 \times 5^2}{8} = 37.5\ kN m$. — 01



Free B.M. [a]

The free B.M. diagram is parabola having vertical ordinate 37.5 kN.m & span of 5 m.

Fixed End Moment Dig [a']

Step No ③. Determination of fixed End moment by using first principle.

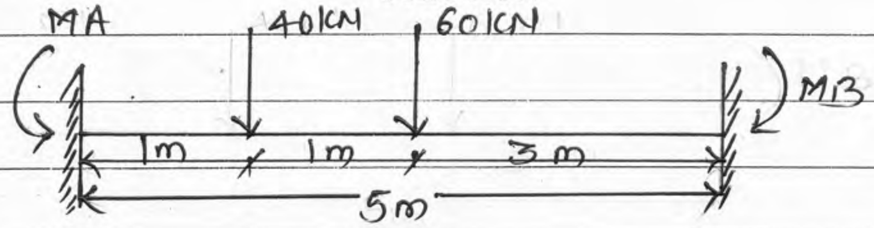
∴ Area of free B.M. dia = Area of fixed B.M. dia.

∴ $a = a'$ — 01

$\frac{2}{3} \times 5 \times 37.5 = -M_A \times 5$

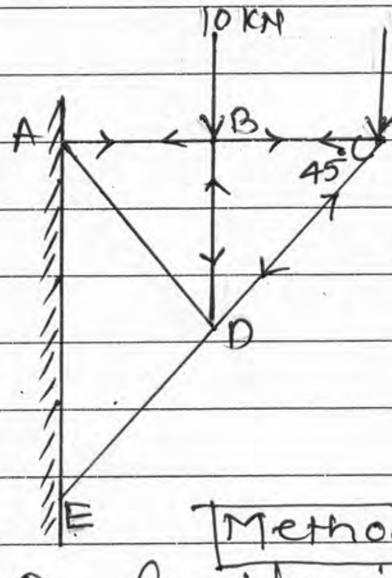
$M_A = -25\ kN.m$

$\boxed{∴ M_A = M_B = 25\ kN.m.\ (Hogging)}$ — 01

Q .NO	SOLUTION	MARKS
d)		
	Let, $W_1 = 40 KN$; $a_1 = 1m$; $b_1 = 4m$. $W_2 = 60 KN$; $a_2 = 2m$; $b_2 = 3m$.	
	∴ The Fixed End Moment @ A.	
	$MA = -\frac{W_1 a_1 b_1^2}{L^2} - \frac{W_2 a_2 b_2^2}{L^2}$	01
	$= -\frac{40 \times 1 \times 4^2}{5^2} - \frac{60 \times 2 \times 3^2}{5^2}$	
	$= -25.6 - 43.2$	
	$= -68.8 \ KN \cdot m.$	01
	$MB = -\frac{W_1 a_1^2 b_1}{L^2} - \frac{W_2 a_2^2 b_2}{L^2}$	01
	$= -\frac{40 \times 1^2 \times 4}{5^2} - \frac{60 \times 2^2 \times 3}{5^2}$	
	$= -6.4 - 28.8$	
	$= -35.2 \ KN \cdot m.$	01
	∴ $MA = 68.8 \ KN \cdot m$ (Hogging) $MB = 35.2 \ KN \cdot m$ (Hogging).	

Q.NO	SOLUTION	MARKS
e)	The following are the assumptions made in the analysis of simple frame. 1) The ends of the members are having perfect pin or Hinged. connections. 2) The self weight of truss or member are negligible. 3) The loads acts at joint only. 4) At any joint, the C.G. of all members coincides at a single point or Joint. 5) The frame is a perfect one i.e. the relation $n = 2j - 3$ is always satisfied.	4 M (for any four)

f)



Let $\angle BCD = \theta$

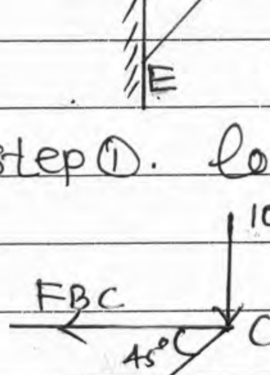
$\tan\theta = \frac{BD}{BC} = \frac{2}{2} = 1$

$\theta = \tan^{-1}(1)$

$= 45°$

$\boxed{\text{Method of JOINT}}$

Step ①. Consider joint C.

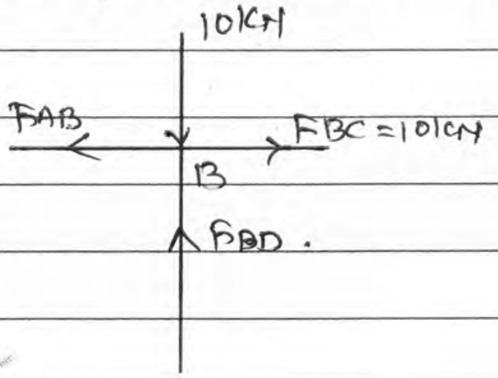
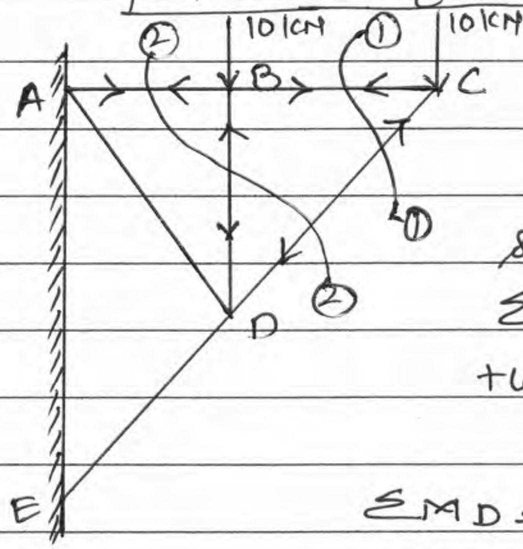


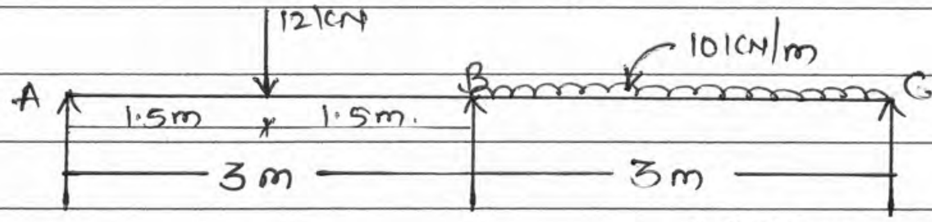
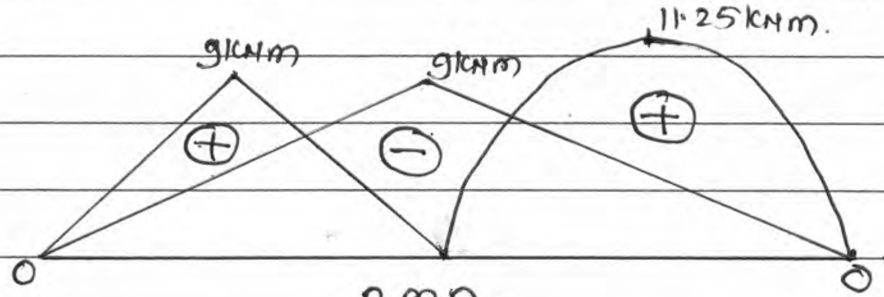
$\Sigma F_y = 0$.

upward = downward

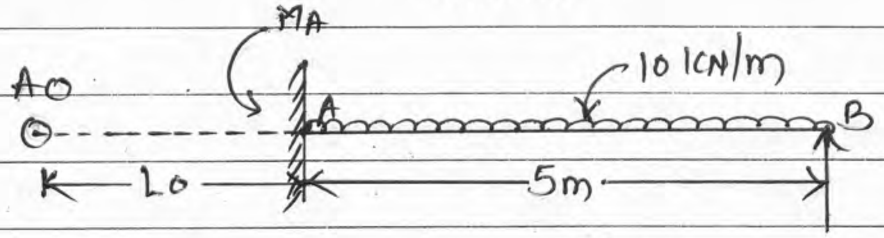
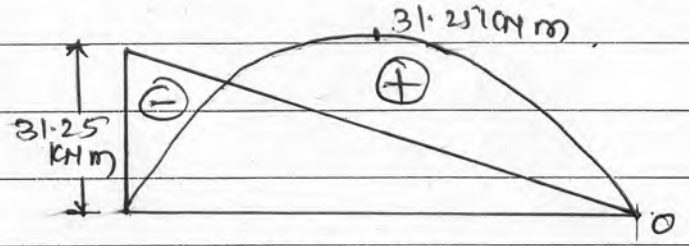
$F_{CD} \sin 45 = 10$

$\boxed{F_{CD} = 14.14 \text{ KN (Comp)}}$ .01

Q .NO	SOLUTION	MARKS
	$\Sigma fx = 0$	
	Rightward = leftward	
	$FBC = FCD \cos 45°$	
	$= 14.14 \cos 45$	
	$\boxed{FBC = 10\,kN \text{ (Tensile)}}$.	01
	Step No. 2. Consider Joint B.	
		
	$\Sigma fy = 0$	
	upward = downward	
	$FBD = 10\,kN$ (Comp).	
	$\Sigma fx = 0$	
	Rightward = leftward	
	$\boxed{FAB = FBC = 10\,kN \text{ (Tensile)}}$.	02
	OR	
*	Method of Section.	
		
	Step No 1 consider section 1-1 & Right Side	
	$\Sigma MB = 0$	
	$+10\times2 - FCD \cos45 \times 2 = 0$	
	$\boxed{FCD = 14.14\,kN \text{ (comp.)}}$	01
	$\Sigma MD = 0$	
	$-FBC \times 2 + 10\times2 = 0$	
	$\boxed{FBC = 10\,kN \text{ (Tensile)}}$.	01
	Step No. 2 Consider Section 2-2.	

Q .NO	SOLUTION	MARKS
	$\Sigma M_D = 0$	
	$-F_{AB} \times 2 + 10 \times 0 + 10 \times 2 = 0$	
	$2F_{AB} = 20$	
	$\boxed{F_{AB} = 10\,kN \text{ (Tensile)}}$.	01
	$\Sigma M_A = 0$	
	$-F_{BD} \times 2 + 10 \times 2 + 10 \times 4 - F_{CD}\cos 45 \times 4 = 0$	
	$-F_{BD} \times 2 + 20 + 40 - 14.14 \cos 45 \times 4 = 0$	
	$-F_{BD} = -20/2$	
	$\boxed{F_{BD} = 10\,kN \text{ (comp.)}}$.	01
Q. 4	Attempt any four of the following.	(16).
(a)		
		
	BMD.	
	Step No. ① Determination of free B.M. for span AB & BC, Assuming individual span as simply supported beam.	

Q .NO	SOLUTION	MARKS
	Free B.M at mid span of AB	
	$= \frac{WL}{4} = \frac{12 \times 3}{4} = 9$ kNm.	
	Free B.M. at mid span of BC.	
	$= \frac{wL^2}{8} = \frac{10 \times 3^2}{8} = 11.25$ kNm.	01
	Step No. 2 Determination the value of $\frac{6a\bar{x}}{L}$	
	∴ For span AB. $a_1 = \frac{1}{2} \times$ base $\times$ ht	
	$= \frac{1}{2} \times 3 \times 9 = 13.5$	
	$\bar{x}_1 = \frac{3}{2} = 1.5$ m.	
	∴ For span BC $a_2 = \frac{2}{3} \times$ base $\times$ ht.	
	$= \frac{2}{3} \times 3 \times 11.25 = 22.5$	
	$\bar{x}_2 = \frac{3}{2} = 1.5$ m.	
	∴ $\frac{6a_1\bar{x}_1}{L_1} = \frac{6 \times 13.5 \times 1.5}{3} = 40.5$	
	∴ $\frac{6a_2\bar{x}_2}{L_2} = \frac{6 \times 22.5 \times 1.5}{3} = 67.5$.	
	Step. No. 3. Determination of Moment @ B.	
	Apply theorem of three moment ~~for~~ span ABC.	
	$M_A L_1 + 2M_B[L_1 + L_2] + M_C L_2 = -\left[\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right]$	01
	As support A & C are simply supported.	
	∴ $M_A = M_C = 0$.	
	$0 + 2M_B(3+3) + 0 = -[40.5 + 67.5]$	01
	$M_B = -9$ kNm.	
	$\boxed{∴ M_B = 9 \text{ kNm (Hogging)}}$	01

Q .NO	SOLUTION	MARKS
b)		
	 BMD.	01
	Step.① Assuming span AB as a simply supported, free B.M at mid span $= \frac{wL^2}{8} = \frac{10\times 5^2}{8} = 31.25\,kNm$	
	& for Imaginary span AoA. $M_0 = 0$.	
	Step. No.② Determination of value of $\frac{6a\bar{x}}{L}$ for Imaginary span. AoA. $a_0 = 0$, $\bar{x}_0 = 0$ $\therefore \frac{6a_0\bar{x}_0}{L_0} = 0$	
	& for span AB. $a_1 = \frac{2}{3}\times base \times ht.$ $= \frac{2}{3}\times 5 \times 31.25 = 104.17$. $\bar{x}_1 = \frac{5}{2} = 2.5\,m$.	
	Step No ③ Determination of fixed End moment A, Apply three moment theorem to span AoAB.	
	$M_0L_0 + 2M_A(L_0+L_1) + M_BL_1 = -\left[\frac{6a_0\bar{x}_0}{L_0} + \frac{6a_1\bar{x}_1}{L_1}\right]$	01

Q .NO	SOLUTION	MARKS

As B support is simply supported propped

$\therefore M_B = 0.$

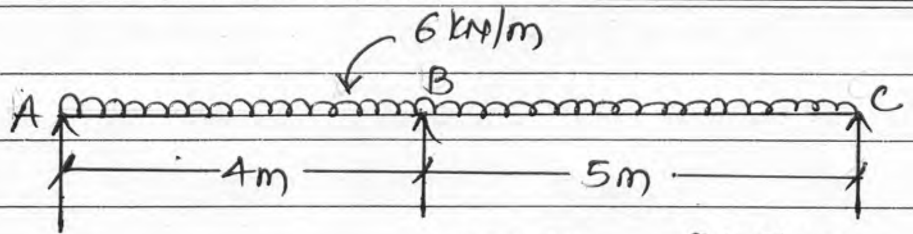
$\therefore 0 + 2M_A(0+5) + 0 = -\left[0 + \frac{6 \times 104.17 \times 2.5}{5}\right]$ 01

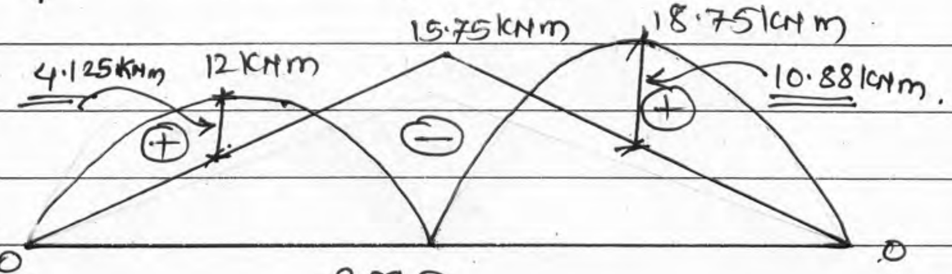
$\therefore 10\,M_A = -312.50$

$M_A = -31.25\,kNm.$ 01

$\boxed{\therefore M_A = 31.25\,kNm\ (\text{Hogging})}$

c)





BMD 01

Step. No. ① Determination of free B.M. for span AB & BC. Assuming individual span as simply supported beam.

Free B.M. at mid span of AB.

$= \frac{wL_1^2}{8} = \frac{6 \times 4^2}{8} = 12\,kNm.$

Similarly free B.M. at mid span BC.

$= \frac{wL_2^2}{8} = \frac{6 \times 5^2}{8} = 18.75\,kNm.$

Q.NO	SOLUTION	MARKS
	Step. No. 2. Determination of value $\frac{6a\bar{x}}{L}$	
	For span AB. $a_1 = \frac{2}{3} \times \text{base} \times \text{ht}.$	
	$= \frac{2}{3} \times 4 \times 12 = 32$	
	$\bar{x}_1 = \frac{L}{2} = \frac{4}{2} = 2m.$	
	For span BC $a_2 = \frac{2}{3} \times \text{base} \times \text{ht}.$	
	$= \frac{2}{3} \times 5 \times 18.75 = 62.5.$	
	$\bar{x}_2 = \frac{L}{2} = \frac{5}{2} = 2.5m.$	
	$\therefore \frac{6a_1\bar{x}_1}{L_1} = \frac{6 \times 32 \times 2}{4} = 96.$	
	$\frac{6a_2\bar{x}_2}{L_2} = \frac{6 \times 62.5 \times 2.5}{5} = 187.5.$	
	Step. No. 3. Determination of moment @ B.	
	Apply three moment theorem for span ABC.	
	$\therefore M_A L_1 + 2M_B(L_1+L_2) + M_C L_2 = -\left[\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right]$	01
	As support A & C are simply supported	
	$\therefore M_A = M_C = 0.$	
	$0 + 2M_B(4+5) + 0 = -[96 + 187.5]$	01
	$M_B = -15.75$ KNm.	
	$\boxed{\therefore M_B = 15.75 \text{ KN.m. Hogging}}$	01

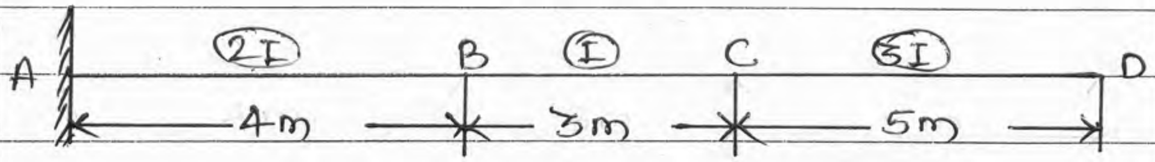
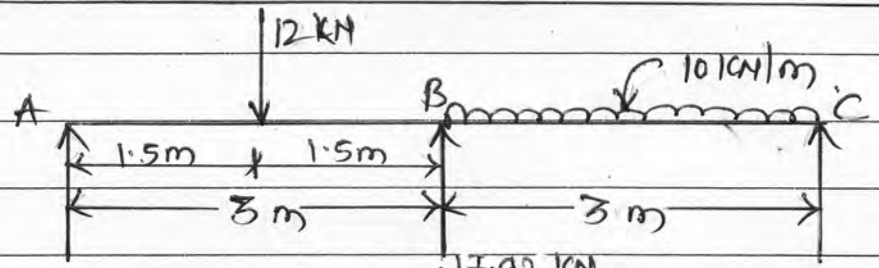
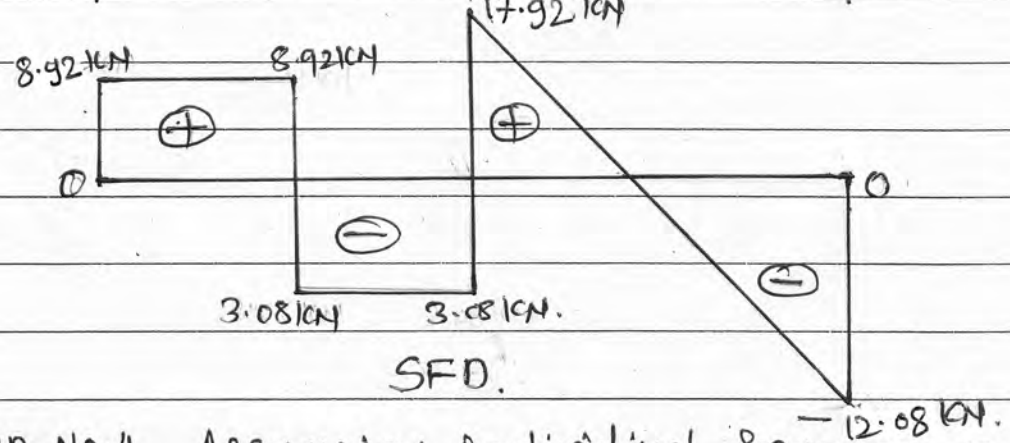
Q .NO	SOLUTION	MARKS
d)		

Table for Distribution factor.

Joint	Member	Relative Stiffness	Total Stiffness	Distribution factor	
B	BA	$\frac{4EI}{L} = \frac{4\times E\times 2I}{4} = 2EI.$	$2EI + 1.33EI = 3.33EI$	$\frac{2EI}{3.33EI} = 0.60$	02
	BC	$\frac{4EI}{L} = \frac{4\times E\times I}{3} = 1.33EI$		$\frac{1.33EI}{3.33EI} = 0.40$	
C	CB	$\frac{4EI}{L} = \frac{4\times E\times I}{3} = 1.33EI.$	$1.33EI + 1.80EI = 3.13EI.$	$\frac{1.33EI}{3.13EI} = 0.42$	02
	CD	$\frac{3EI}{L} = \frac{3\times E\times 3I}{5} = 1.80EI$		$\frac{1.80EI}{3.13EI} = 0.58.$	

e).





SFD. 01

Step. No. 1 Assuming individual span as a fixed beam determine fixed End moments by using Hardy Cross sign convention.

Q .NO	SOLUTION	MARKS
	~~For span AB.~~	
	$M_{ab} = -\frac{WL}{8} = -\frac{12 \times 3}{8} = -4.5$ KN.m	
	$M_{ba} = +\frac{WL}{8} = +\frac{12 \times 3}{8} = +4.5$ KN.m	
	For Span BC	
	$M_{bc} = -\frac{WL^2}{12} = -\frac{10 \times 3^2}{12} = -7.5$ KNm	
	$M_{cb} = +\frac{WL^2}{12} = +\frac{10 \times 3^2}{12} = +7.5$ KNm.	
	Step. No. 2 Determination of Distribution factor.	

Joint	Member	Relative Stiffness	Total Stiffness	Distribution factor.
B	BA	$\frac{3EI}{L} = \frac{3EI}{3} = 1EI$	2EI	0.50
	BC	$\frac{3EI}{L} = \frac{3EI}{3} = 1EI$		0.50.

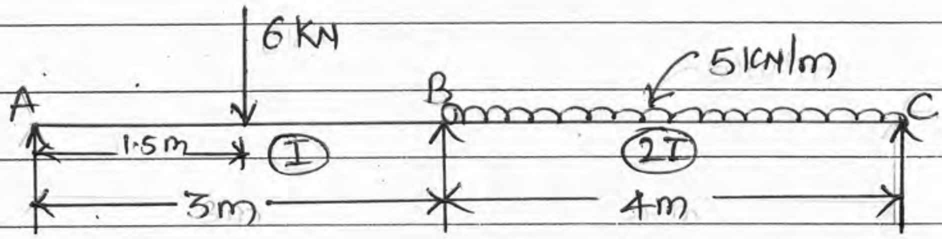
01

Step. No. 3. Determination of support moment.

A B C

A		B		C	Joint
AB	BA	BC	CB		Member
	0.5	0.5			D.F.
-4.5	+4.5	-7.5	+7.5		FEM.
+4.5	+2.25	-3.75	-7.5		Release A & C Carryforward to B.
0	+6.25	-11.25	0		Initial moment
	+2.50	-2.50			Distribute
0	+8.75	-8.75.	0		Final moment.

01

Q .NO	SOLUTION	MARKS
	$\boxed{M_A = M_C = 0,\ M_B = 8.75\ kNm\ (\text{Hogging})}$	
	Step. No. (4) Determination of Support Reaction.	
	Apply conditions of Equilibrium.	
	$\Sigma F_x = 0,\ \Sigma F_y = 0,\ \Sigma M_O = 0.$	
	$\therefore \Sigma F_y = 0$; upward = downward	
	$R_A + R_B + R_C = 12 + 10 \times 3 = 42\ kN$ — eqn (1)	
	Taking moment @ B & consider L.H.S.	
	Anticlockwise = clockwise	
	$12 \times 1.5 + 8.75 = R_A \times 3$	
	$\boxed{R_A = 8.92\ kN.}$	1/2
	Taking moment @ B & consider R.H.S.	
	Anticlockwise = clockwise	
	$R_C \times 3 = 10 \times 3 \times 3/2 \quad 8.75$	
	$\boxed{R_C = 12.08\ kN.}$	
	Substitute R_A & R_C in eqn (1).	
	$8.92 + R_B + 12.08 = 42$	
	$\boxed{R_B = 21\ kN.}$	1/2
(f)		
	Step. No. (1) Assuming individual span as a fixed beam, Determine fixed End moment using Hardy cross sign convention.	

Q.NO	SOLUTION	MARKS

For Span AB.

$$M_{ab} = -\frac{WL}{8} = -\frac{6\times3}{8} = -2.25\ \text{KN.m}$$

$$M_{ba} = +\frac{WL}{8} = +\frac{6\times3}{8} = +2.25\ \text{KN.m}$$

For Span BC

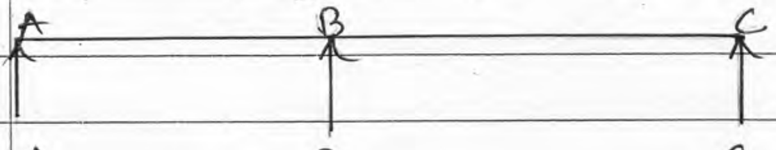
$$M_{bc} = -\frac{wL^2}{12} = -\frac{5\times4^2}{12} = -6.67\ \text{KN.m}$$

$$M_{cb} = +\frac{wL^2}{12} = +\frac{5\times4^2}{12} = +6.67\ \text{KN.m}$$ 01

Step. No. 2: Determination of Distribution factor.

Joint	Member	Relative Stiffness	Total Stiffness	Distribution factor.
B	BA	$\frac{3EI}{L} = \frac{3EI}{3} = 1EI$	2.5EI	0.40
	BC	$\frac{3EI}{L} = \frac{3\times E\times 2I}{4} = 1.5EI$		0.60.

01



A AB	BA	B BC	C CB	Joint Member
	0.40	0.60		Distribution factor
−2.25	+2.25	−6.67	+6.67	Fixed End moment
+2.25			−6.67	Release A & C
	+1.125	−3.335.		carry forward to B.
0	+3.375	−10.005	0	Initial moment
	+2.652	+3.978		Distribute @ B.
0	+6.027	−6.027	0	Final moment.

02

$M_A = M_C = 0$ $M_B = 6.027\ \text{KN.m}$ (Hogging)

Q.NO	SOLUTION	MARKS

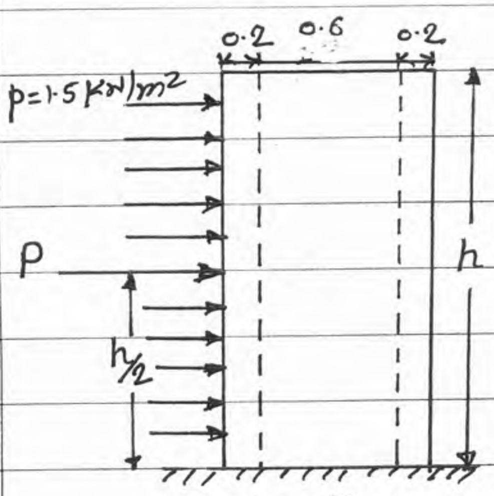
Q5 a) Given data

Outer dimensions $B = 1.6\,m$, $D = 1m$, $t = 200mm = 0.2m$

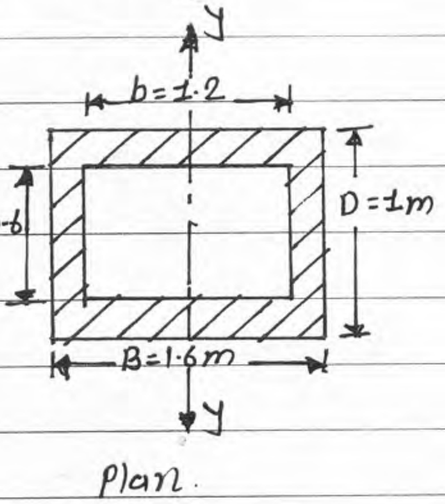
Wind pressure $p = 1.5\ KN/m^2$, $\sigma_{max} = 230\ KN/m^2$

Density of masonry $\rho = 23\ KN/m^3$

Let 'h' be the height of chimney.



Elevation.



Plan.

1) Direct stress $\sigma_o = \dfrac{\text{Weight of chimney }(W)}{\text{C/S Area of chimney }(A)}$ — ½M

Area of chimney $A = BD - bd = 1.6 \times 1 - 1.2 \times 0.6$

$A = 0.88\,m^2$

Weight of chimney $W = A \times h \times \rho$

$= 0.88 \times h \times 23$

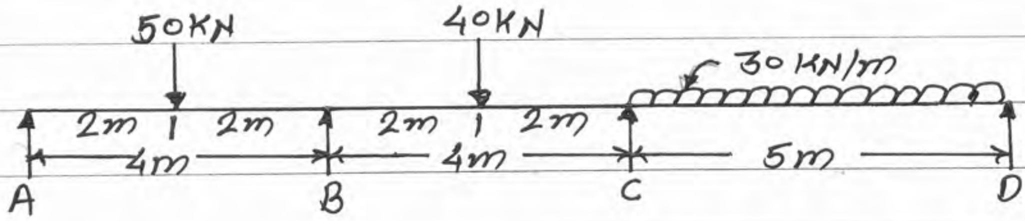
$= 20.24\,h$ KN — 1M

$\therefore \sigma_o = \dfrac{20.24h}{0.88} = 23h$ —(i) KN/m^2 — 1M

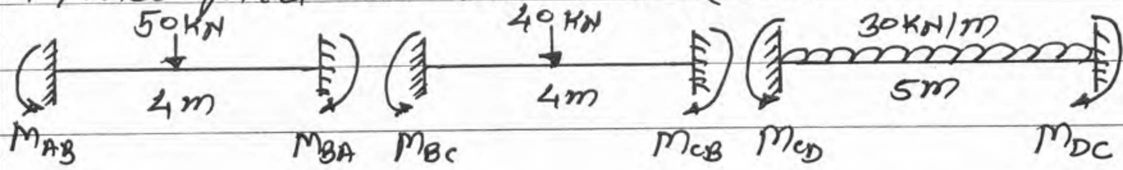
Q .NO	SOLUTION	MARKS
Q5 a) Cont...	2) Bending stress $\sigma_b = \frac{M}{Z} = \frac{P \times h/2}{Z}$	½ M
	Total wind load on chimney $P = p \times$ Projected area	
	$P = 1.5 \times (B \times h) = 1.5 \times (1.6 \times h) = 2.4h$ KN	1M
	Section modulus Z about X-X- axis	
	$Z = \frac{I_{xx}}{y_{max}} = \frac{\frac{BD^3}{12} - \frac{bd^3}{12}}{D/2} = \frac{BD^3 - bd^3}{6D}$	
	$= \frac{1.6 \times 1^3 - 1.2 \times 0.6^3}{6 \times 1} = 0.223\ m^3$	1M
	$\therefore \sigma_b = \frac{2.4h}{0.223} \times \frac{h}{2} = 5.381h^2\ KN/m^2$ ---(ii)	1M
	Now, we know that	
	$\sigma_{max} = \sigma_0 + \sigma_b$ --- put value of σ_0 & σ_b from eq^n(i) & (ii)	
	$230 = 23h + 5.381h^2$	
	$5.381h^2 + 23h - 230 = 0$ ----(A)	1M
	Solving the quadratic eq^n A, we get	
	$h = 4.741\ m$ or $h = -9.015\ m$	
	Now, height of chimney can not be negative	
	$\therefore \boxed{h = 4.741\ m.}$	1M

Q .NO	SOLUTION	MARKS

Q5 b)



1) Find fixed end moments (FEM)



$$M_{AB} = M_{BA} = \pm \frac{WL}{8} = \pm \frac{50 \times 4}{8} = \pm\ 25\ KN \cdot m$$

$$M_{BC} = M_{CB} = \pm \frac{WL}{8} = \pm \frac{40 \times 4}{8} = \pm\ 20\ KN \cdot m$$ 1M

$$M_{CD} = M_{DC} = \pm \frac{wl^2}{12} = \pm \frac{30 \times 5^2}{12} = \pm\ 62.5\ KN \cdot m$$

2) Distribution factors

Joint	member	stiffness of member	Total stiffness of joint	D.F
B	BA	$\frac{3EI}{L} = \frac{3EI}{4}$	1.75 EI	0.43
	BC	$\frac{4EI}{L} = \frac{4EI}{4}$		0.57
C	CB	$\frac{4EI}{L} = \frac{4EI}{4}$	1.6 EI	0.63
	CD	$\frac{3EI}{L} = \frac{3EI}{5}$		0.37

1M

Q.NO	SOLUTION	MARKS

Q5b) Cont...

3) Moment Distribution table

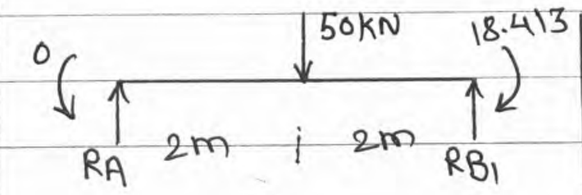
Joint	A	B		C		D	Marks
member	AB	BA	BC	CB	CD	DC	
D.F	–	0.43	0.57	0.63	0.37	–	
F.E.M.	-25	25	-20	20	-62.5	62.5	
Release A&D	+25					-62.5	
carry over to B & C		12.5			-31.25		
Initial moment	0	37.5	-20	20	-93.75	0	
1st Distribution		-7.525	-9.975	46.463	27.288		3M
carry over			23.231	-4.988			
2nd Distribution		-9.989	-13.242	3.142	1.846		
carry over			1.571	-6.621			
3rd Distribution		-0.676	-0.896	4.171	2.450		
carryover			2.086	-0.448			
4th Distribution		-0.897	-1.189	0.282	0.166		
Final Moments		-18.413	18.413	62	-62		

∴ Support moments

$M_A = M_D = 0$ - - - - S.S. end

$M_B = -18.413$ KN.m $\Big\}$ -ve sign indicate
$M_C = -62$ KN.m $\Big\}$ Hogging moments.

Q .NO	SOLUTION	MARKS
Q5b Cont...		



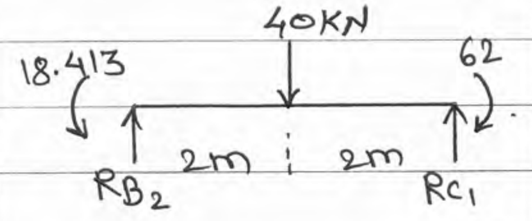
$\Sigma F_y = 0$, $R_A + R_{B_1} = 50$ KN

$\Sigma M = 0$,

$(50 \times 2) + 18.413 - 4R_{B_1} = 0$

$\therefore R_{B_1} = 29.60$ KN

$\therefore R_A = 50 - R_{B_1} = 20.40$ KN



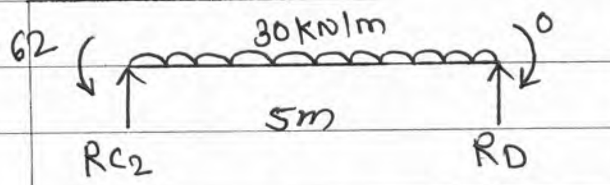
$\Sigma F_y = 0$, $R_{B_2} + R_{C_1} = 40$ KN

$\Sigma M = 0$

$(40 \times 2) - 18.413 + 62 - R_{C_1} \times 4 = 0$

$\therefore R_{C_1} = 30.89$ KN

$\therefore R_{B_2} = 40 - R_{C_1} = 9.11$ KN



$\Sigma F_y = 0$, $R_{C_2} + R_D = 150$ KN

$\Sigma M = 0$

$-62 + (30 \times 5 \times 2.5) - 5R_D = 0$

$\therefore R_D = 63$ KN

$\therefore R_{C_2} = 150 - R_D = 87$ KN

$\therefore$ final Support Reaction

$R_A = 20.40 \qquad = 20.40$

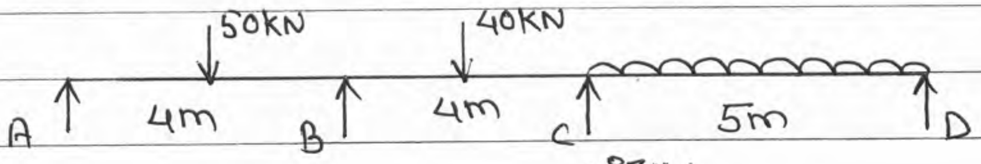
$R_B = R_{B_1} + R_{B_2} = 38.71$

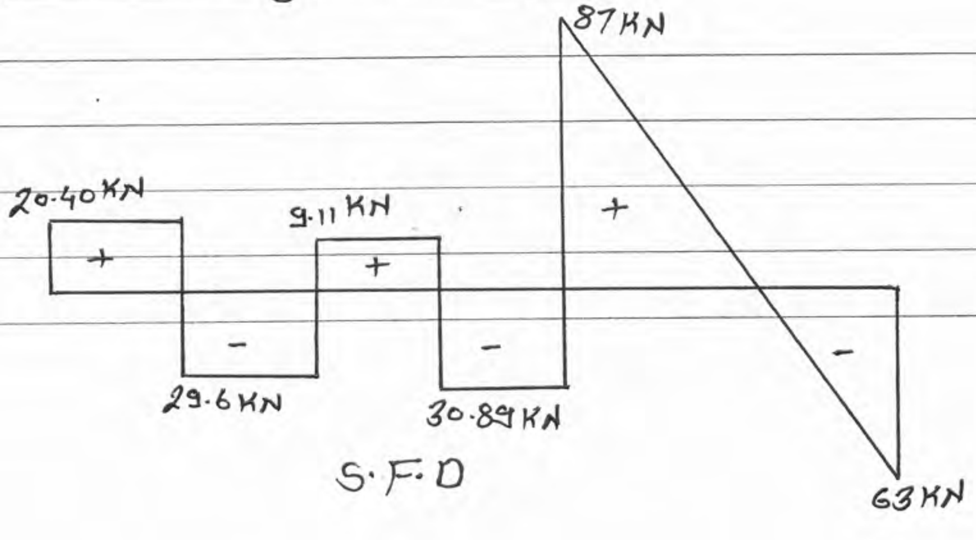
$R_C = R_{C_1} + R_{C_2} = 117.89$ — 02M

$R_D = 63 \qquad = 63$

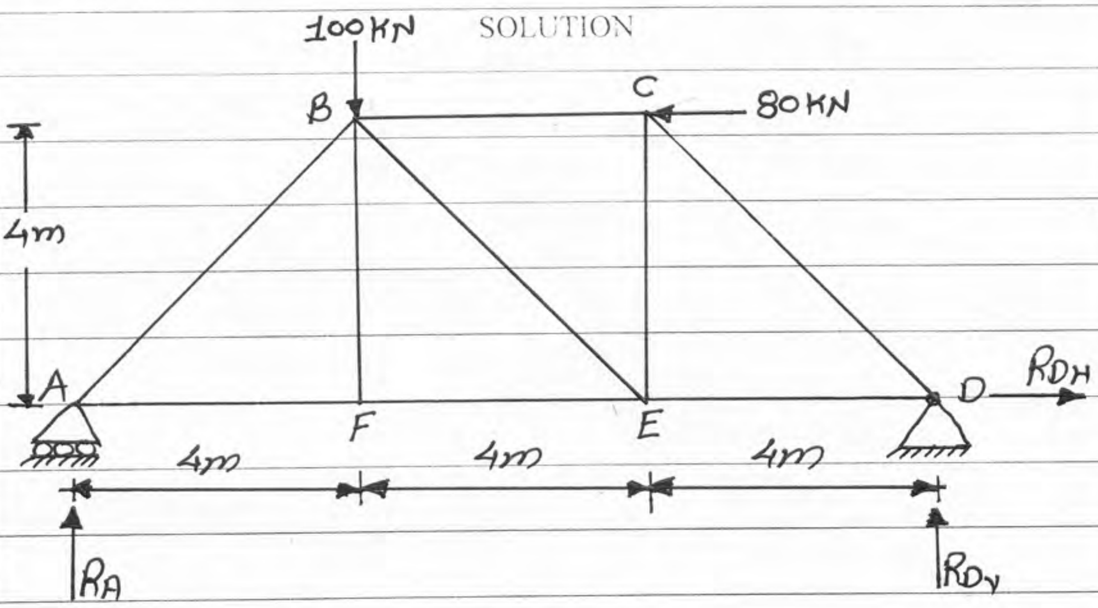
240 KN

which is equal to total load on beam.





S.F.D — 01M

Q.NO	SOLUTION	MARKS
Q5C)		

1> Support Reactions i> $\Sigma F_y = 0$

$R_A + R_{DV} - 100 = 0 \quad \therefore R_A + R_{DV} = 100\ KN \quad -(i)$

ii> $\Sigma F_x = 0$

$-80 + R_{DH} = 0 \quad \therefore R_{DH} = 80\ KN\ (\rightarrow)$

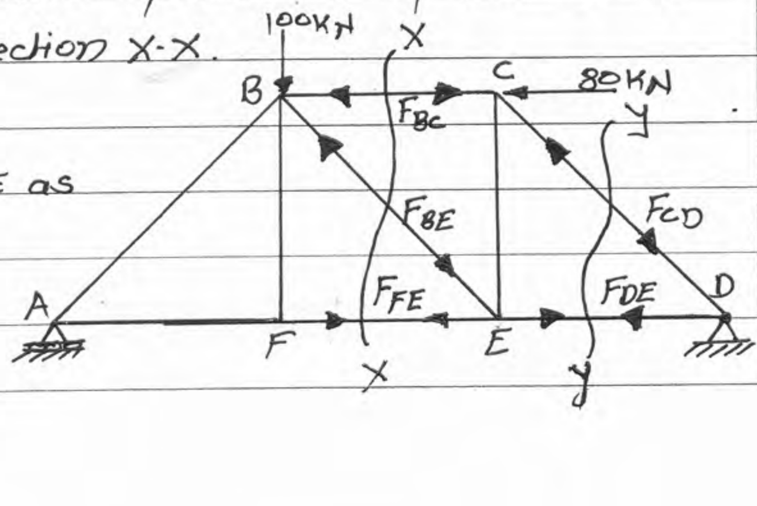
iii> $\Sigma M_A = 0$

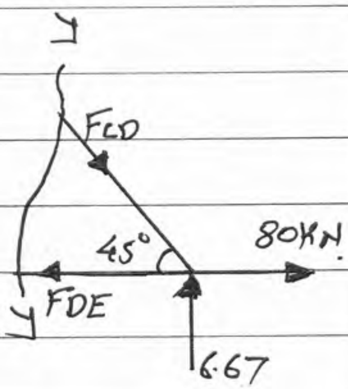
$(100 \times 4) - (80 \times 4) - R_{DV} \times 12 = 0$

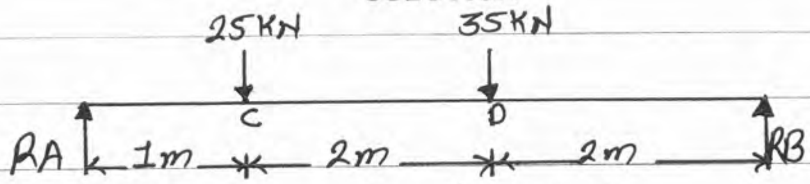
$\therefore R_{DY} = 6.67\ KN \quad \therefore R_A = 100 - 6.67 = 93.33\ KN$

2> Let us consider the equilibrium of the truss to the left of section X-X.

Assume F_{BC} & F_{BE} as Compressive & F_{FE} as Tensile.



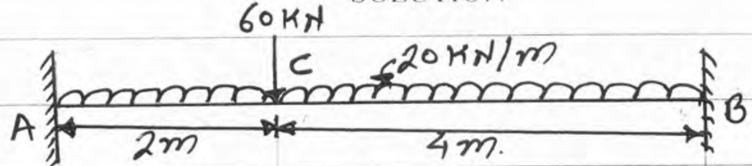
Q.NO	SOLUTION	MARKS
Q5C) Cont...	Taking moment about B	
	$(93.33 \times 4) - F_{FE} \times 4 = 0$	
	$\therefore \boxed{F_{FE} = 93.33 \text{ KN (Tensile)}}$	2M
	Taking moment about E	
	$-(100 \times 4) - F_{BC} \times 4 + (93.33 \times 8) = 0$	
	$4F_{BC} = 346.64$	
	$\therefore \boxed{F_{BC} = 86.66 \text{ KN (Comp.)}}$	2M
	Taking $\Sigma F_x = 0$	
	$-F_{BC} + F_{FE} - F_{BE} \cdot \cos 45^\circ = 0$	
	$-86.66 + 93.33 - F_{BE} \cos 45 = 0$	
	$\boxed{F_{BE} = 9.43 \text{ KN (Comp)}}$	2M
	3) Let us consider the equilibrium of the truss to the right of section Y-Y	
	Assume F_{CD} as Compressive & F_{DE} as Tensile.	
		
	Taking $\Sigma F_y = 0$	
	$6.67 - F_{CD} \sin 45 = 0$	
	$\therefore \boxed{F_{CD} = 9.43 \text{ KN (Comp.)}}$	2M

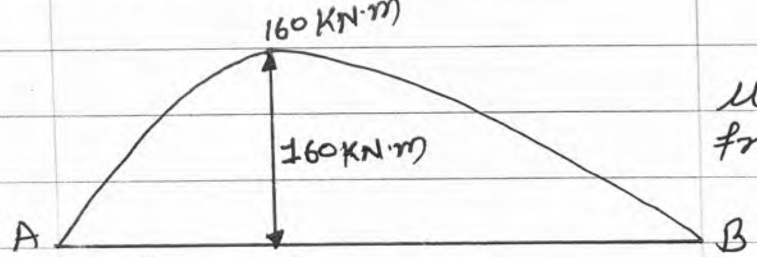
Q.NO	SOLUTION	MARKS
Q6a)		
	$E = 2\times10^5\ N/mm^2 = 2\times10^8\ KN/m^2$	
	$I = 3\times10^8\ mm^4 = 3\times10^{-4}\ m^4$	
	Support Reaction, $\Sigma F_y = 0 \quad R_A + R_B = 60\ KN$.	
	$\Sigma M_A = 0$	
	$(25\times1)+(35\times3) - 5R_B = 0$	
	$\therefore R_B = 26\ KN \qquad \therefore R_A = 34\ KN$	1M
	Consider a section x-x at a distance x from A in Portion DB.	
	$M_x = 34\cdot x - 25(x-1) - 35(x-3)$	1M
	But,	
	$EI\dfrac{d^2y}{dx^2} = M_x = 34x - 25(x-1) - 35(x-3)$ --(A)	½ M
	Integrating eq^n A. w.r.to. x	
	$EI\dfrac{dy}{dx} = \dfrac{34x^2}{2} + C_1 - \dfrac{25(x-1)^2}{2} - \dfrac{35(x-3)^2}{2}$ --(B)	½ M
	Integrating eq^n B w.r.to x.	
	$EI\cdot y = \dfrac{34x^3}{6} + C_1x + C_2 - \dfrac{25(x-1)^3}{6} - \dfrac{35(x-3)^3}{6}$ ----(C)	
	Apply Boundry Conditions to find C_1 & C_2 values	
	At A, $x = 0$, $y = 0$ put in eq^n (C).	
	$0 = 0 + C_1(0) + C_2 \qquad \therefore \boxed{C_2 = 0}$	1M

Q.NO	SOLUTION	MARKS
	At, B, $x = 5m$, $y = 0$, put in eqn C.	
	$0 = \frac{34(5)^3}{6} + C_1 \cdot 5 + 0 - \frac{25(5-1)^3}{6} - \frac{35(5-3)^3}{6}$	
	$0 = 708.33 + 5C_1 - 266.67 - 46.667$	
	$\therefore C_1 = -394.99/5 = -79$	
	$\boxed{C_1 = -79}$	1m
	Substitute the values of C_1 & C_2 in eqn B & C	
	$EI \frac{dy}{dx} = \frac{34x^2}{2} - 79 - \frac{25(x-1)^2}{2} - \frac{35(x-3)^2}{2}$ --- slope eqn.	1m
	$EI \cdot y = \frac{34x^3}{6} - 79x - \frac{25(x-1)^3}{6} - \frac{35(x-3)^3}{6}$ --- Deflection eqn.	1m
	Deflection under 25KN point load,	
	Put $x = 1m$, in Deflection eqn.	
	$EI \cdot y = \frac{34(1)^3}{6} - 79 \times 1 = -73.333$	
	$y_c = -73.333/EI$	
	$y_c = \frac{-73.333}{2\times10^8 \times 3\times10^{-4}} = 1.222\times10^{-3}\ m.$	
	$\boxed{y_c = 1.222\ mm.}$	1m
	Deflection under 25 KN point load is 1.222mm.	

Q.NO	SOLUTION	MARKS

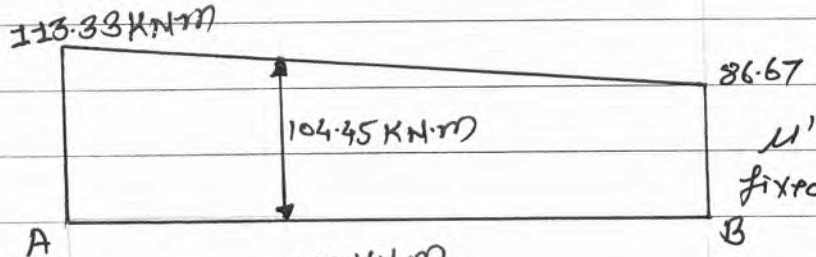
Q6 b)



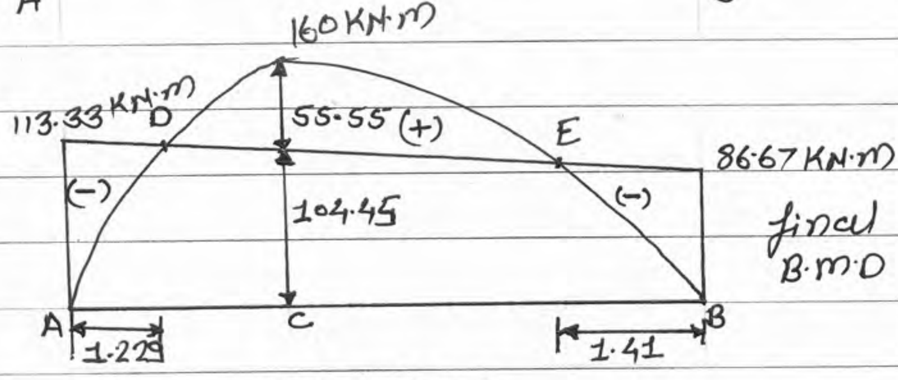


μ dia or free B.M.D

1M



μ' dia. or fixed B.M.D

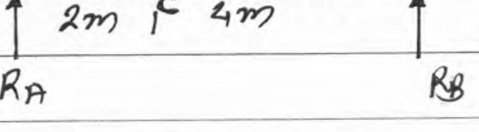


final B.M.D

1M

B.M.D KN·m

1) Assume that the beam is simply supported and calculate the support reaction and draw μ dia.



$\Sigma Fy = 0$; $R_A + R_B = 180$ KN.

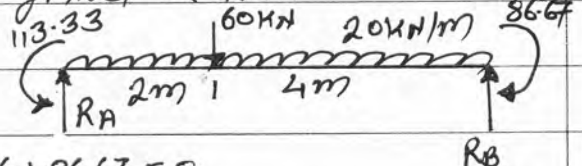
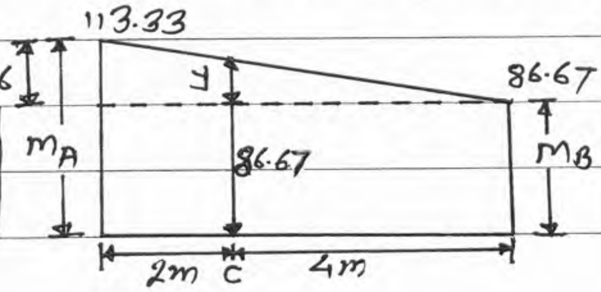
$\Sigma M = 0$; $(20\times6\times3) + 60\times2 - 6R_B = 0$

1M

$\therefore R_B = 80$ KN $\quad \therefore R_A = 100$ KN

$M_C = 100\times2 - (20\times2\times1) = 160$ KN·m

$M_A = M_B = 0$

Q.NO	SOLUTION	MARKS
	2) Calculate the fixed end moments & draw μ' diagram	
	$M_A = -\frac{wL^2}{12} - \frac{Wab^2}{L^2} = -\frac{20\times6^2}{12} - \frac{60\times2\times4^2}{6^2} = -113.33$ KN.m	1M
	$M_B = -\frac{wL^2}{12} - \frac{Wa^2b}{L^2} = -\frac{20\times6^2}{12} - \frac{60\times2^2\times4}{6^2} = -86.67$ KN.m	1M
	3) Superimpose μ' diagram over μ diagram and draw final B.M. diagram.	
	4) Calculate reaction of a fixed beam.	
	$\Sigma F_y = 0$ $R_A + R_B = 180$ KN. 	
	$\Sigma M = 0.$	1M
	$-113.33 + (20\times6\times3) + (60\times2) - R_B\times6 + 86.67 = 0$	
	$\therefore R_B = 75.55$ KN $\therefore R_A = 104.45$ KN	
	5) Net B.M. at C.	
	$M_C = -113.33 + (104.45\times2) - (20\times2\times1) = 55.57$ KN.m	1M
	OR	
	Net B.M at C can also be calculated as follows.	OR
	Net B.M at C $M_C = 160 - \left[86.67 + \frac{26.66}{6}\times4\right]$ 	1M

Q .NO	SOLUTION	MARKS

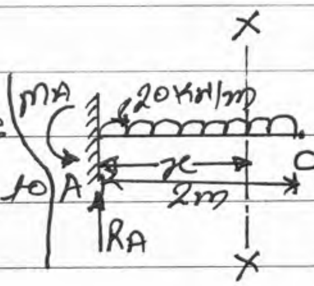
$\therefore$ Net B.M at C $M_C = 55.57$ KN.m

6> Point of Contraflexure

Let points of Contraflexure D&E in the portion AC & CB respectively

i> To locate D.

Take a section xx at a distance x from A, find M_x & equate it to Zero



$M_x = R_A \cdot x - w \cdot x \cdot \frac{x}{2} - M_A$

$0 = 104.45x - \frac{20x^2}{2} - 113.33$

$0 = 104.45x - 10x^2 - 113.33$

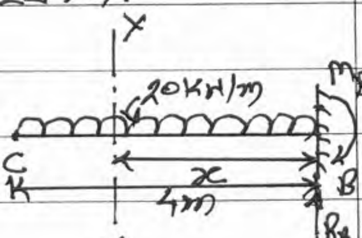
$\therefore x = 1.229\ m$ & $x = 9.215\ m$ is not Possible — 01

$\therefore$ Distance of D from A = 1.229 m.

OR

ii> To locate E

Take a section xx at a distance x from B, find M_x & equate it to Zero



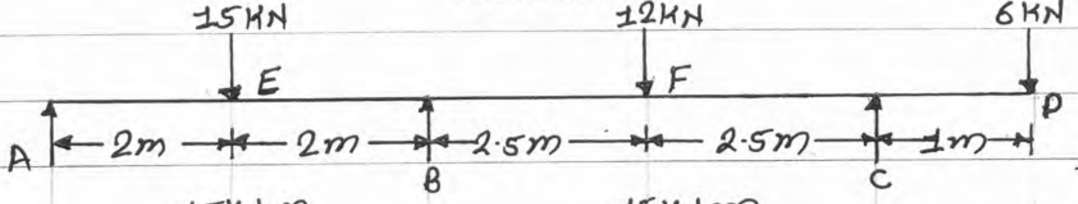
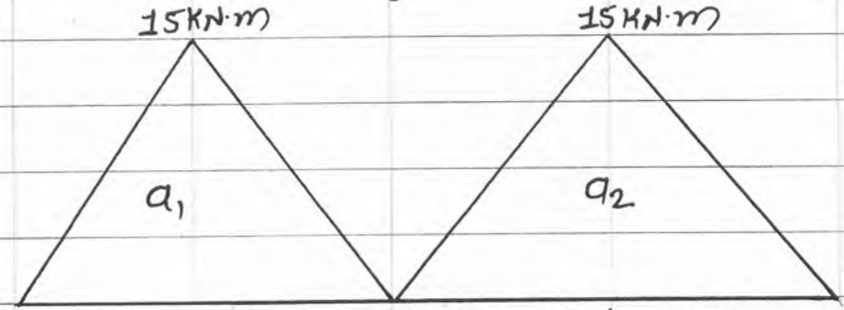
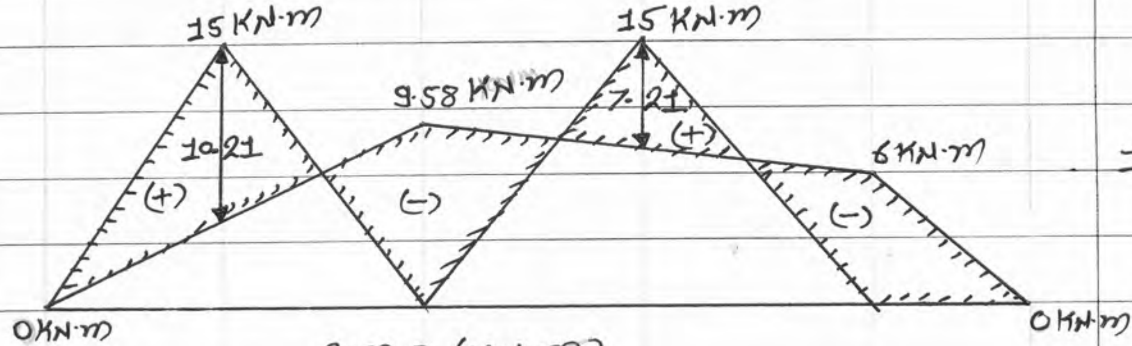
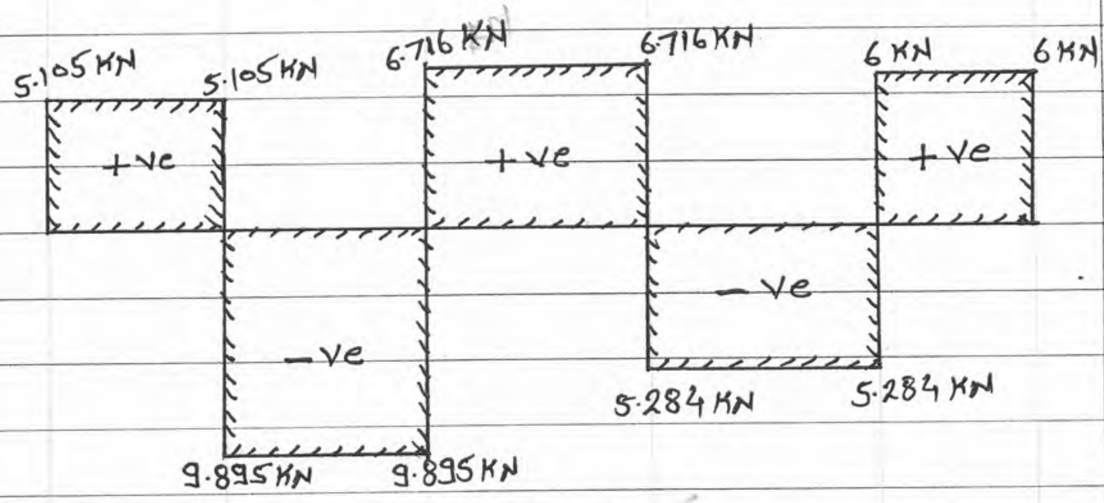
$M_x = R_B \cdot x - M_B - w \cdot x \cdot \frac{x}{2}$

$0 = 75.55x - 86.67 - \frac{20x^2}{2}$

$0 = -10x^2 + 75.55x - 86.67$

$\therefore x = 1.410\ m$ or $x = 6.144\ m$ is not possible

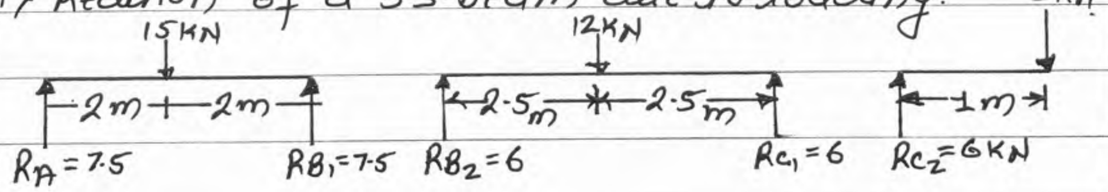
$\therefore$ Distance of E from B = 1.41 m. — 1M

Q .NO	SOLUTION	MARKS
Q6 c)		1M
		
	Free B.M.D or M diagram	
		1M
	B.M.D (KN.m)	
		1M
	S.F.D.(KN)	

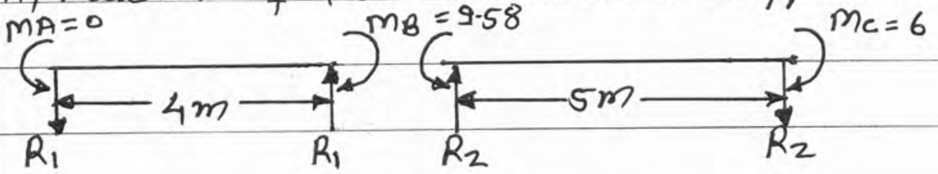
Q .NO	SOLUTION	MARKS
Q6c) cont...	1) Assume the spans AB & BC as simply supported and draw μ diagram.	
	Free moments, B.M. at mid span of AB	
	$M_E = \frac{WL}{4} = \frac{15\times4}{4} = 15\ KN\cdot m.$	
	B.M. at mid span of BC,	
	$M_F = \frac{WL}{4} = \frac{12\times5}{4} = 15\ KN\cdot m$	1M
	2) Apply clapeyron's theorem of three moments 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
	Known moments, $M_A = 0$ ... s.s. end.	
	$M_C = -6\times1 = -6\ KN\cdot m$	
	$a_1 = \frac{1}{2}\times4\times15 = 30$, $\bar{x}_1 = \frac{L_1}{2} = \frac{4}{2} = 2m$, $L_1 = 4m$	
	$a_2 = \frac{1}{2}\times5\times15 = 37.5$, $\bar{x}_2 = \frac{L_2}{2} = \frac{5}{2} = 2.5m$, $L_2 = 5m$	
	$0(4)+2M_B(4+5)-6(5) = -\left[\frac{6\times30\times2}{4} + \frac{6\times37.5\times2.5}{5}\right]$	
	$18M_B - 30 = -[90+112.5]$	
	$18M_B = -202.5+30$	
	$\therefore M_B = -9.58\ KN\cdot m$ -ve sign indicate hogging moment	1M

Q.NO	SOLUTION	MARKS
Q6c) Cont...	3) Support Reactions	

i) Reaction of a S.S. beam due to loading.



ii) Reaction of a S.S. beam due to support moments.



$$R_1 = \frac{M_A - M_B}{4} = \frac{0 - 9.58}{4}$$

$$R_1 = 2.395\ KN$$

$$R_2 = \frac{M_B - M_C}{5} = \frac{9.58 - 6}{5}$$

$$R_2 = 0.716\ KN.$$

Reaction of a Continuous beam.

$R_A = R_A - R_1 = 7.5 - 2.395 = 5.105\ KN$

$R_B = R_{B_1} + R_{B_2} + R_1 + R_2 = 7.5 + 6 + 2.395 + 0.716 = 16.611\ KN$

$R_C = R_{C_1} + R_{C_2} - R_2 = 6 + 6 - 0.716 = \underline{11.284\ KN}$ — 2M

33KN