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GOVT. OF KERALA
DEPARTMENT OF TECHNICAL EDUCATION
OFFICE OF THE CONTROLLER OF TECHNICAL EXAMINATIONS
THIRUVANANTHAPURAM

DIPLOMA EXAMINATION IN ENGINEERING /TECHNOLOGY/MANAGEMENT

CORRECTION NOTE (Scheme)

Revision & Sub code: 4014C15
Subject: Theory of structures – II
Verified by: PAUL

Quest. No: III a) I_{yy} given 5.069×10^6

but in substitution the value is taken as 5.69×10^6

The result will be changed $P_e = \underline{624}$ kN

VIII a) As per syllabus deflection of beams limited for symmetric loads only. But in given question loading is not symmetric

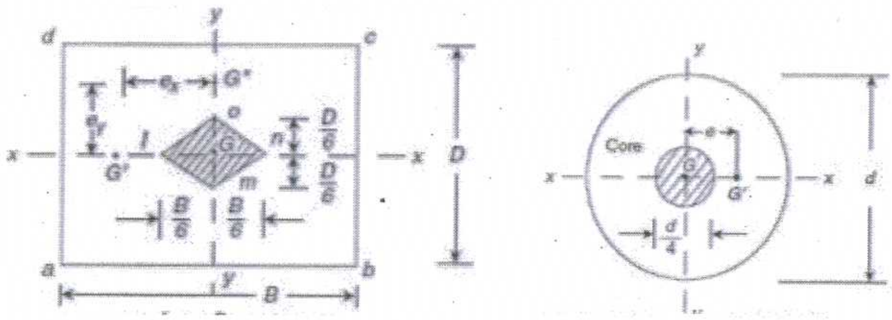
1

SCORING INDICATORS

CODE : (15) 4014 VERSION: A

Qn.No		Split score	Total score

3

Qn.No	PART-A	Split score	Total score
I. 1	Slenderness ratio (λ) is the ratio of the effective length of a column (Le) and the least radius of gyration (r) about the axis under consideration.	2	
2	core area of rectangular and circular sections 	2	
3	A beam that is supported at both ~~free~~ ends and is restrained against rotation and vertical movement. Also known as built-in beam; encastré beam		
4	The deformed axis of the beam from its neutral axis is called its elastic curve.	2	
5	Carry over factor: It is the ratio of moment transferred to the far end and moment applied to the other end of the beam.	2	
		2	10
II) 1	SOLUTION External diam, D = 150mm =0.15m Internal diam, d = 100mm= 0.1m Length of column,l = 10m Factor of safety = 5 $E = 95GN/m^2$		
	$L_e = \frac{l}{\sqrt{2}} = \frac{10}{\sqrt{2}} = 7.07m$	1	
	$P_e = \frac{\pi r^2 EI}{le2}$	2	
	$Pe = \frac{\pi^2 x95x10^9\ x\pi/64(0.15^4-0.1^4)x10^{-3}}{(7.07)^2}$ $= 374kN$	2	
	safe load $=P_{e/F\,O\,S} = 374/5 =$ 74.8kN	1	6

2 SOLUTION

External dia m ,D = 38mm

Thickness,d = 2.5cm

Internal diam = 38-(2x2.5) = 33mm

Yield strength(σ) = $335 N/mm^2$

Rankines constsnt(a) $= \frac{1}{7500}$

$I = \frac{\pi}{64}(D^4 - d^4) = \frac{\pi}{64}(38^4 - 33^4) = 14.05x10^3 \pi\ mm^4$ 1

$A == \frac{\pi}{4}(D^2 - d^2) = 88.75\ \pi\ mm^2$ 1

$K = \sqrt{\frac{I}{A}} = \sqrt{\frac{14.05X10^3}{88.75}} = 12.6mm$ 1

Rankines crippling load $= \frac{\sigma_{cs.A}}{1+a\left(\frac{Le}{k}\right)^2} = \frac{335x88.75\pi}{1+\frac{1}{7500}\{(2.3x10^3)/12.6\}^2}$ 2

$= \underline{17.16kN}$ 1 6

3

1. Against Overturning of dam about the toe
2. Against Sliding – shear failure of gravity dam
3. Against Compression – by crushing of the gravity dam
4. Against Tension – by development of tensile forces which results in the crack in gravity dam.

Overturning of Gravity Dam:

The ratio of the resisting moments about toe to the overturning moments about toe is called the factor of safety against overturning. Its value generally varies between 2 and 3. 2

Factor of safety against overturning is given by

FOS = sum of overturning moments/ sum of resisting moments

Against Sliding of Gravity Dam:

Factor of safety against sliding can be given based on

- Frictional resistance
- Frictional resistance and shear strength of the dam

factor of safety based on frictional resistance:

$$\text{FOS against sliding} = \text{FOS} = \frac{\mu \Sigma V}{\Sigma H}$$

2

gravity Dam Failure due to Tension Cracks

The dam loses contact with the bottom foundation due to this crack and becomes ineffective and fails. Hence, the effective width B of the dam base will be reduced. This will increase pmax at the toe.

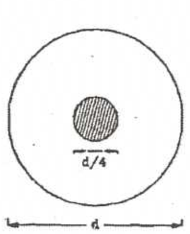
Hence, a tension crack by itself does not fail the structure, but it leads to the failure of the structure by producing excessive compressive stresses.

For high gravity dams, certain amount of tension is permitted under severest loading conditions in order to achieve economy in design. 1

Gravity Dam Failure due to Compression

A gravity dam may fail by the failure of its material, i.e. the compressive stresses produced may exceed the allowable stresses, and the dam material may get crushed. 1 6

4



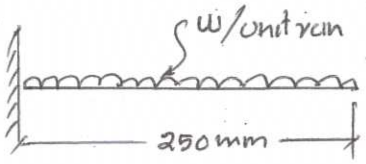
$$A = \frac{\pi d^2}{4} \quad ,I= \frac{\pi d^4}{64}$$ 2

$$Z= \frac{I}{Y} = \frac{\pi d^4 X2}{64Xd} = \frac{\pi d^3}{32}$$ 2

For no tension in the base

$$e = \frac{Z}{A} = \frac{\pi d^3 X4}{32X\pi X d^2} = \frac{d}{8}$$ 2 6

5 Solution



Length, l = 250 mm = 0.25 m.

Width b = 40 mm = 0.04 m

Depth d = 30 mm = 0.03 m

M I , $I = \frac{bd^3}{12} = \frac{0.04 \times 0.03^3}{12} = 9 \times 10^{-8}\ mm^4$ 1

$E = 70\ GN/m^2$

Maximum deflection, y_{max} = 0.5 mm = 0.0005 m.

Maximum deflection, $y_{max} = \frac{wl^4}{8EI}$ 1

$$0.0005 = \frac{w \times (0.25)^4}{8 \times 70 \times 10^9 \times 9 \times 10^{-8}} = 0.0005$$ 1

$$w = \frac{0.0005 \times 8 \times 70 \times 10^9 \times 9 \times 10^{-8}}{0.25^4} = 6451\ N/m$$ 2

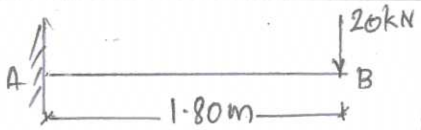
w = 6.451 kN/m 1 6

5

6

Solution

$b = 120\,mm$, $d = 150\,mm$, $l = 1.80\,m$.



$E = 200\,GPa = 200 \times 10^3\ N/mm^2$

$I = \frac{bd^3}{12} = \frac{120 \times 150^3}{12} = 33.75 \times 10^6\ mm^4$ 1

slope at the free end $= \frac{Wl^2}{2EI} = \frac{(20 \times 10^3) \times (1.8 \times 10^3)^2}{2 \times (200 \times 10^3) \times (33.75 \times 10^6)}$ 2

$i_B = \underline{0.0048\ rad}$ 1

Deflection $= y_B = \frac{Wl^3}{3EI} = \frac{20 \times 10^3 \times (1.8 \times 10^3)^3}{3 \times (200 \times 10^3) \times (33.75 \times 10^6)} = \underline{5.76\ mm}$ 2

6

7

The moment distribution method for beams may be summarized as follows:

1. Determine the stiffness for each member.

   For a member that is fixed at both ends, use equation
   $K_{AB} = 4EI/L$

   For a member that has a pin at one end, use equation
   $K_{AB} = 3EI/L$ 1

2. Determine the distribution factors for each member at each node based on relative stiffness of the members using equation 1

   $$DF_{AB} = k_{AB} / \sum ki$$

   Use a distribution factor of zero for a fixed support and 1.0 for a pinned support with only one connected member.

3. Determine the fixed end moments for all members that have external loads applied between the end nodes 1

4. For each node in turn:

   A. Determine the unbalanced moment on the node 1

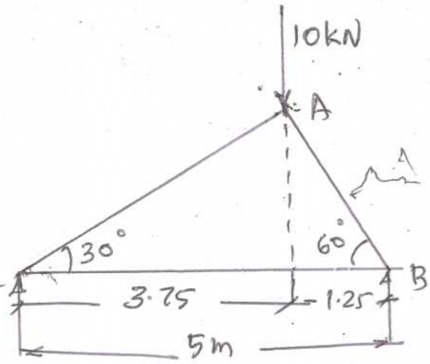
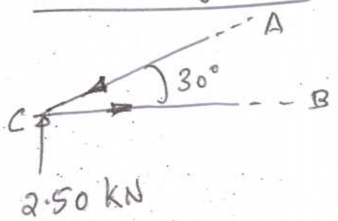
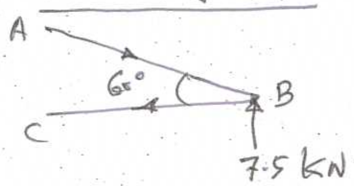
   B. Distribute the unbalanced moment to each member connected to the node in proportion to the distribution factors in the reverse direction of the unbalanced moment.

   C. For each member that the moment has been distributed to, carry over some of the moment to the opposite end of the member according to equations. For a member with a fixed end opposite (a regular locked node), carry over half of the moment that was applied by the distribution. For a member with a pinned end opposite (where there are no other members connected to that pin) do not carry over any moment.

5. Repeat the previous step for each node, multiple times as necessary until the carry over moments are a small fraction of the total moments at each member end 1

6. Sum all of the moments in each member end from all previous steps (including the original fixed end moments). This sum gives the total moment at each member end in the real system. 1

6

	PART-C		
III (a)	Solution		
	Given $I_{XX} = 6.09 \times 10^{6}\ mm^{4}$ and $I_{YY} = 5.069 \times 10^{6}\ mm^{4}$		
	Since I_{YY} is less than I_{XX}, the column will tend to buckle in YY-direction.	1	
	$\therefore I = I_{YY}$		
	$L_e = l = 4 \times 10^{3}\ mm$ (since both ends hinged)	1	
	$P_e = \dfrac{\pi^2 EI}{L_e^2} = \dfrac{\pi^2 \times (200 \times 10^3)(\times 5.69 \times 10^6)}{(4 \times 10^3)^2}$	4	
	$P_e = 702 \times 10^{3}\ N$	2	8
	$= \underline{\underline{702\ kN}}$		
(III) (b)	Solution 		
	From geometry of the truss		
	Taking moments about C		
	$R_B \times 5 = 10 \times 3.75$		
	$R_B = 7.50\ kN$	1	
	$R_C = 10 - 7.5$		
	$= 2.50\ kN$	2	
	Consider joint C 		
	P_{AC} and P_{BC}		
	$P_{AC} \sin 30° = 2.50$		
	$\therefore P_{AC} = \dfrac{2.50}{\sin 30} = \underline{\underline{5.0\ kN\ (C)}}$	1	
	Consider joint B 		
	$P_{AC} \cos 30° = P_{BC} = 5 \times 0.866$		
	$P_{BC} = \underline{4.33\ kN\ (T)}$	1	
	$P_{AB} \sin 60° = 7.5$		
	$P_{AB} = \dfrac{7.5}{\sin 60°} = \underline{\underline{8.66\ kN\ (C)}}$	1	
	$P_{BC} = P_{AB} \cos 60°$		
	$P_{BC} = 8.66 \times 0.50 = \underline{\underline{4.33\ kN\ (T)}}$	1	7

7

IV (a)	Solution Since I_{YY} is less than I_{XX}, there for the column will tend to buckle in Y-Y direction. $I = I_{YY} = 4.404 \times 10^6 mm^4$ Total area, $A = 5047 mm^2$ Crashing strength, $\sigma = 315$ MPa $= 315 N/mm^2$, $a = \frac{1}{7500}$ Column is fixed at one end and hinged at other end, $Le = \frac{l}{\sqrt{2}}$ $Le = \frac{l}{\sqrt{2}} = \frac{4 \times 10^3}{\sqrt{2}} = 2.83 \times 10^3 mm$ $\therefore k = \sqrt{I/A} = \sqrt{\frac{4.404 \times 10^6}{5047}} = 29.5 mm$ Rankines Crippling load $= \frac{\sigma_c \cdot A}{1+a\left[\frac{Le}{k}\right]^2}$ $= \frac{315 \times 5047}{1+\frac{1}{7500}\left[\frac{2.83 \times 10^3}{29.50}\right]^2} = 714 kN$ Safe load on the column $= \frac{\text{Crippling load}}{FOS} = \frac{714}{3.5}$ $= \underline{\underline{204 kN}}$	1 1 1 1 1 2 1 1	9
(b)	The equivalent column length can be defined as the length of an equivalent pin-ended column having the same load-carrying capacity as the member under consideration.	2	
	End Condition / **Relationship between the equivalent length and the actual length** 1. Both Ends Hinged — $Le=L$ 2. One End Fixed, Other End Free — $Le=2L$ 3. Both Ends Fixed — $Le = \frac{L}{2}$ 4. One End Fixed, Other End Hinged — $Le = \frac{L}{\sqrt{2}}$	6	8

V (a)

solution

$D = 350\ mm$

$d = 350 - (2 \times 25) = 300\ mm$

$W = 80\ kN$

$e = \frac{350}{2} = 175\ mm$ ($\because$ load is at outer edge) 1

$A = \frac{\pi}{4}(0.35^2 - 0.30^2) = 0.0255\ m^2$ 1

$Z = \frac{\pi}{32}\left(\frac{D^4 - d^4}{D}\right) = \frac{\pi}{32}\frac{(0.35^4 - 0.30^4)}{0.35} = 0.001937\ m^3$ 2

Direct stress, $f_a = W/A = \frac{80}{0.0255} = 3137.255\ kN/m^2$ 1

Bending Stress, $f_b = \frac{We}{Z} = \frac{80 \times 0.175}{0.001937} = 7227.672\ kN/m^2$ 1

Maximum Stress $= f_a + f_b = 3137.255 + 7227.672$

$= \underline{10404.927}\ kN/m^2$ 1

Minimum stress $= f_a - f_b = 3137.255 - 7227.672$ 1

$= \underline{-4050.417}\ kN/m^2$

V 8

(b)

For simply supported conditions

Maximum BM $= \frac{wl^2}{8} = \frac{3 \times 6^2}{8}$ 1

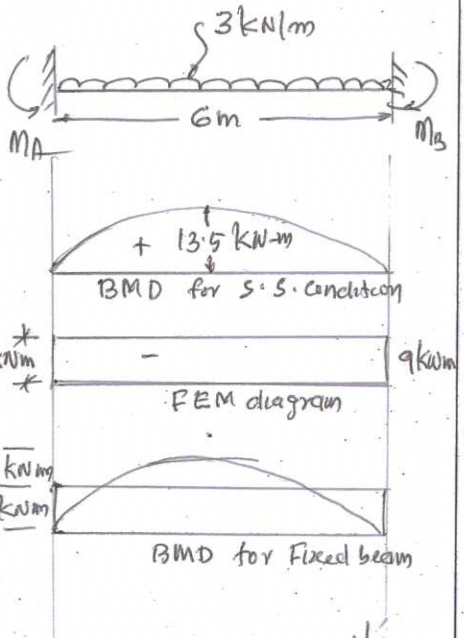
$= 13.5\ kN.m$ 1

By symmetry in loading

F.E.M at A = F.E.M at B

ie $M_A = M_B = \frac{wl^2}{12}$ 1

$= \frac{3 \times 6^2}{12} = 9\ kN.m$



i) maximum positive BM

$= 13.5 - 9 = \underline{4.5\ kN}$ 2

$R_A = R_B = \frac{3 \times 6}{2} = 9\ kN.$

ii) Maximum Deflection occures at the mid span $= \frac{wl}{384\ EI}$

$y_{max} = \frac{3 \times 6^4 \times 1000^3}{384 \times 20 \times 10^8} = \underline{5.063\ mm}$ 2 7

VI (a)

Solution

Given, $W = 80 kN$

$e = 40 mm = 0.04 m$

$b = d = 100mm = 0.1 m$

Area, $A = b \times d = 0.1 \times 0.1 = 0.01 m^2$ — 1

$Z = \frac{1}{6} bd^2 = \frac{1}{6} \times 0.1 \times 0.1^2$ — 1

$Z = 0.1667 \times 10^{-3} m^3$

Direct stress, $f_a = \frac{W}{A} = \frac{80}{0.01} = 8000 kN/m^2$ — 1

Bending stress, $f_b = \frac{W.e}{Z} = \frac{80 \times 0.04}{0.1667 \times 10^{-3}} = 19196.16 kN/m^2$ — 1

$f_a = 8 N/mm^2$, $f_b = 19.196 N/mm^2$

$\therefore f_{max.} = 8 + 19.196 = 27.196 N/mm^2$ (Compression) — 2

$f_{min.} = 8 - 19.196 = -11.196 N/mm^2$ (Tensile) — 2

Total: 8

VI (b)

1.Weep Holes

provided in earth retaining structures like retaining walls, underpasses, wing walls and other below ground drainage structures.

Weep Hole is provided in these structures to relieve hydrostatic pressure or water pressure on the walls.

Reducing the water pressure on the walls will reduce the structural design demand of the water or earth resisting wall by reducing its thickness as well as reinforcement requirements. — 2

2.Active earth pressure

The minimum value of lateral earth pressure exerted by soil on a structure and the wall moves away from the backfill , occurring when the soil is allowed to yield sufficiently to cause its internal shearing resistance along a potential failure surface to be completely mobilized. — 2½

3.Passive earth pressure

When the wall moves towards the back fill, there is an increase in the pressure on the wall and this increase continues until a maximum value has reached after which there is no increase in the pressure and the value will become constant. This kind of pressure is known as passive earth pressure. — 2½

Total: 7

VII (a)

Solution Let the central point load be 'W'

$I = \frac{bd^3}{12} = \frac{1}{12} \times 120 \times 240^3 = 138.24 \times 10^6\ mm^4$ — 1

$E = 110\ kN/mm^2$, $y_{max} = 4\ mm$.

$y_{max} = \frac{WL^3}{48EI}$ — 2

$4 = \frac{W \times 4^3 \times 10^9}{48 \times 110 \times 138.24 \times 10^6} =$

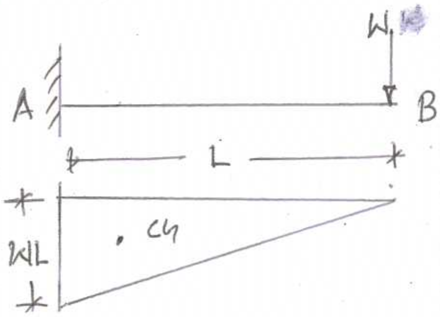
$W = \underline{\underline{45.6\ kN}}$ — 1

(b) Slope at the ends $\theta_A = -\frac{WL^2}{16EI} = -\frac{45.6 \times 4^2 \times 1000^2}{16 \times 110 \times 138.24 \times 10^6}$ — 2

$\theta_A = \theta_B = \underline{-0.003\ radians}$.

$\theta_B = \underline{+0.003\ radians}$. — 2 — 8

VII (b)



Consider a cantilever beam AB of Length 'L' fixed at A carrying a point load 'W' at the Free end.

1. Mohr's Theorem I

$\theta_B = \frac{\text{Area of BM diagram}}{EI}$ — 1

Area, $A = WL \times \frac{L}{2} = \frac{WL^2}{2}$

$\theta_B = \underline{\underline{\frac{WL^2}{2EI}}}$ — 2

2. Mohr's Theorem - II

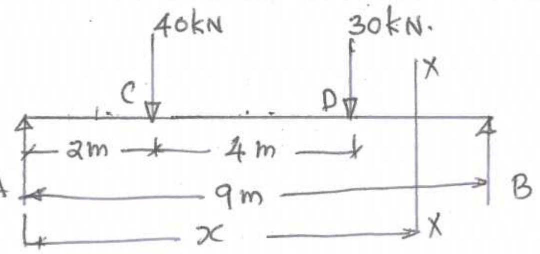
Deflection at B with respect to A

$y_B = \frac{A \cdot \bar{x}}{EI}$ — 1

$\bar{x} = \frac{2}{3}L$ — 1

$\therefore y_B = \frac{WL^2}{2} \times \frac{2}{3}L \times \frac{1}{EI} = \underline{\underline{\frac{WL^3}{3EI}}}$ — 2 — 7

VIII (a) <u>solution</u>



Taking moments about 'B'

$$R_A \times 9 = 40 \times 7 + 30 \times 3$$

$$R_A = \frac{370}{9} = 41.11 \text{ kN}, \quad R_B = 70 - 41.11 = 28.89 \text{ kN} \quad 1$$

Consider a section XX at a distance x from 'A'. 1

$$M_x = 41.11x \mid - 40(x-2) \mid - 30(x-6)$$

$$\therefore EI\frac{d^2y}{dx^2} = 41.11x \mid - 40(x-2) \mid - 30(x-6) \quad 1$$

$$EI\frac{dy}{dx} = \frac{41.11x^2}{2} + C_1 \mid -\frac{40}{2}(x-2)^2 \mid -\frac{30}{2}(x-6)^2 \quad 1$$

$$EIy = \frac{41.11x^3}{6} + C_1x + C_2 \mid - \frac{40}{6}(x-2)^3 \mid - \frac{30}{6}(x-6)^3 \quad 1$$

At $x = 0$, $y = 0$, Hence $C_2 = 0$

At $x = L$, $y = 0$, substitute again in deflection eqn.

$$0 = \frac{41.11}{6}(9)^3 + C_1 \times 9 - 9 - \frac{40}{6}(9-2)^3 - \frac{30}{6}(9-6)^3$$

$$C_1 = -285.9 \quad 1$$

Substitute the value of C_1 in slope eqn.

$$EI\frac{dy}{dx} = \frac{41.11x^2}{2} - 285.90 \mid - 20(x-2)^2 \mid - 15(x-6)^2$$

and
$$EIy = \frac{41.11x^3}{6} - 285.90x \mid - \frac{40}{6}(x-2)^3 \mid - \frac{30}{6}(x-6)^3$$

<u>Deflection Under load</u>

Deflection under 40 kN load, substitute $x = 2$

$$y_C = \frac{1}{EI}\left[\frac{41.11 \times 2^3}{6} - 285.90 \times 2\right] = -1.615 \text{ mm} \quad 1$$

Deflection under 30kN load, substitute $x = 6$.

$$y_D = \frac{1}{EI}\left[\frac{41.11 \times 6^3}{6} - 285.90 \times 6 - \frac{40}{6}(6-3)^3\right] \quad 1$$

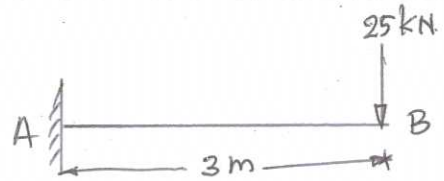
$$y_D = 2.06 \text{ mm} \quad 8$$

VIII (b)

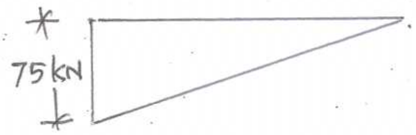
Solution

Maximum $BM = 25 \times 3$

$= 75\ kNm$ — 1



Area of BM diagram



$A = \frac{1}{3} \times 3 \times 75$

$= 112.5\ kNm^2$ — 1

C.G of BM diagram from $B = \frac{2}{3} \times 3 = 2m$. — 1

slope at $B = \theta_B = \frac{\text{Area of BM diagram}}{EI}$

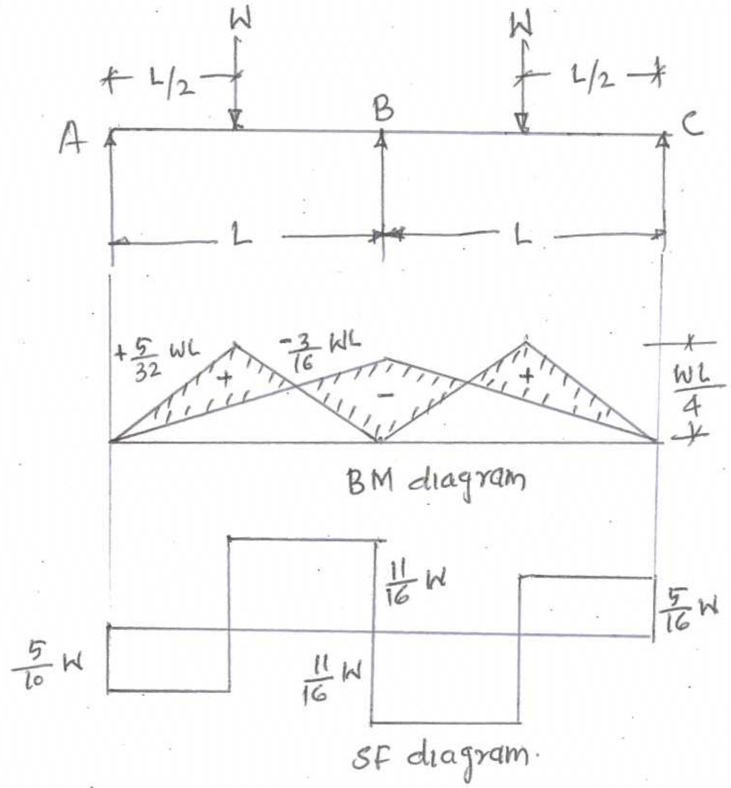
$= \frac{112.5 \times 10^6}{200 \times 360 \times 10^6} = \underline{\underline{0.00156}}$ radians — 2

Deflection at 'B' with respect to 'A'

$= \frac{\text{Moment of Area of BM diagram about A}}{EI}$

$= \frac{112.5 \times 2 \times 10^9}{200 \times 360 \times 10^6} = \underline{\underline{3.125\ mm}}$ — 2 — 7

IX (a)



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1

Free BM diagram

BM under load W $= \frac{W}{2} \times \frac{L}{2} = \frac{WL}{4}$ 1

$\therefore$ Area, $A_1 = A_2 = \frac{1}{2} \times L \times \frac{WL}{4} = \frac{WL^2}{8}$ 1

$\bar{x}_1 = \bar{x}_2 = \frac{L}{2}$

Applying clapeyron's theoram

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

Since the support A and C are simply supported and

$L_1 = L_2$; $M_A = M_C = 0$

$\therefore 0 + 2M_B(2L) + 0 + \frac{6}{L} \times \frac{WL^2}{8} \times \frac{L}{2} + \frac{6}{L} \times \frac{WL^2}{8} \times \frac{L}{2} = 0$

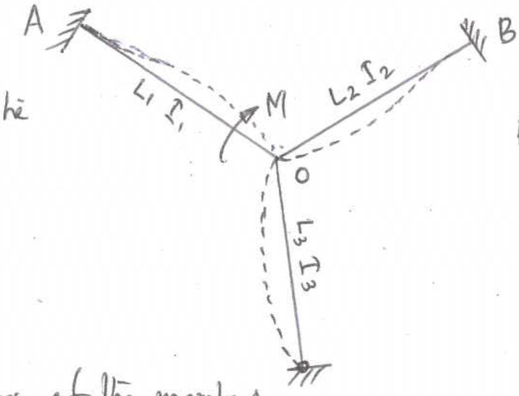
$\therefore 4M_B L = -\frac{3}{4}\frac{WL^3}{L}$

$\therefore M_B = \frac{3}{16} WL$ (Hogging) 1

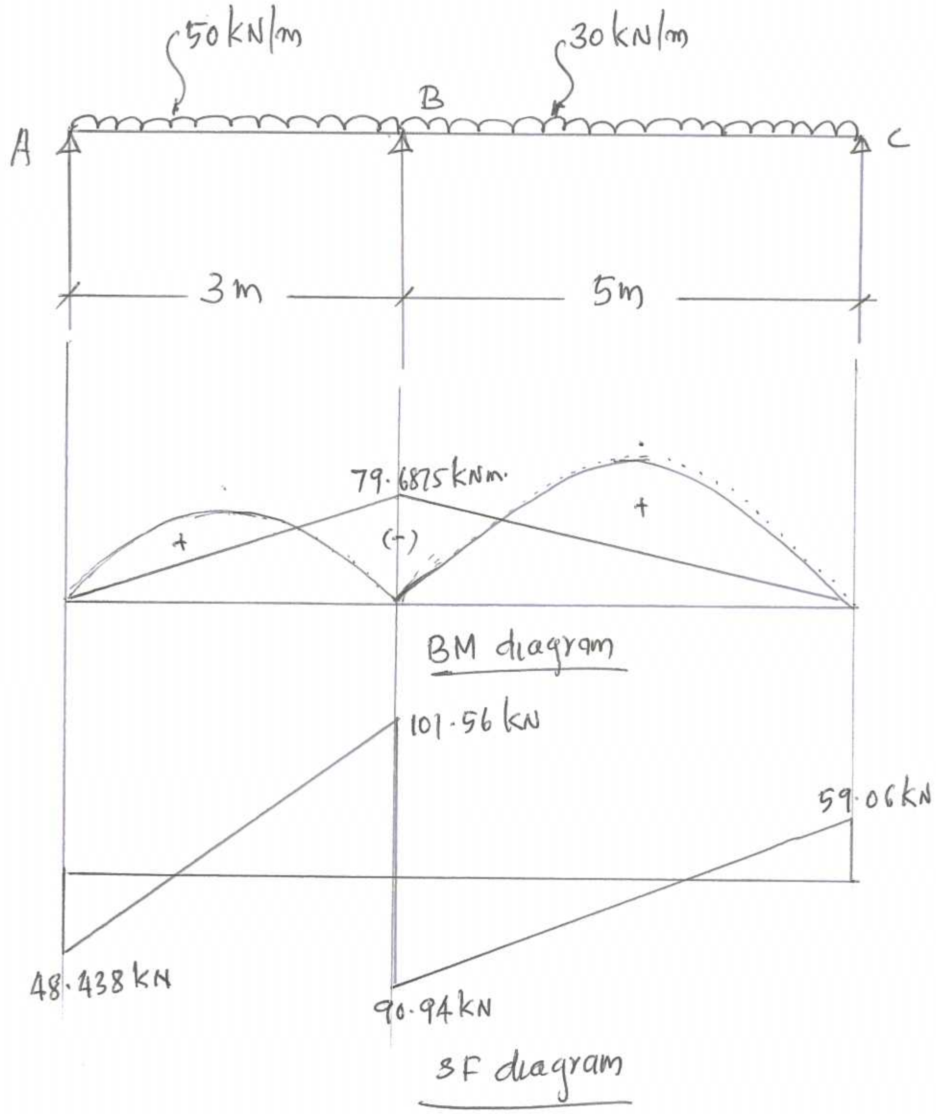
By super imposing

$M_D = M_E = \frac{WL}{4} - \frac{1}{2}\left[\frac{3}{16}WL\right] = +\frac{5}{32}WL$ 1

$M_B = -\frac{3}{16} WL$		
$M_D = +\frac{5}{32} WL$		
Taking moments about B for span BC		
$R_C \times L = W\frac{L}{2} - \frac{3}{16} WL$		
$\therefore R_C = \frac{5}{16} W$	1	
Taking moments about B for span AB		
$R_A \times L = W\frac{L}{2} - \frac{3}{16} WL$		
$\therefore R_A = \frac{5}{16} W$	1	
$R_A + R_B + R_C = 2W$		
$\therefore R_B = 2W - \frac{5}{16} W - \frac{5}{16} W$		
$R_B = \frac{11}{8} W$	1	10
$R_A = R_C = \frac{5}{16} W$		
$R_B = \frac{11}{8} W$		

IX (b)	Solution Let the length and Moment of inertia of the members be OA = L_1 and I_1 OB = L_2 and I_2 OC = L_3 and I_3  Fig. Let E = youngs Modulus of the members Let a clockwise moment M be applied at 'O', therefor the joint 'O' will undergo deformation as shown in the fig. The applied moment 'M' will be distributed among all the connecting members at the joint in proportion to their stiffness. Due to the rigidity of the joint all the members will be rotated through the same angle. Let M_1, M_2 and M_3 be the BM carried by each member $M_1 = \frac{4EI_1\theta_1}{L_1} = k_1\theta \quad \because \theta_1 = \theta_2 = \theta_3$ $M_2 = \frac{4EI_2\theta_2}{L_2} = k_2\theta$ $M_3 = \frac{4EI_3\theta_3}{L_3} = k_3\theta$ Total stiffness of the members = $\Sigma k = k_1 + k_2 + k_3$ Total moment applied at the joint = $M = \Sigma k\theta$ $\therefore \frac{M_1}{M} = \frac{k_1}{\Sigma k}$ or $M_1 = M \cdot \frac{k_1}{\Sigma k}$ $\frac{k_1}{\Sigma k}$ is called Distribution factor for the member 'OA' Similarly distribution factors for OB and OC are $\frac{k_2}{\Sigma k}, \frac{k_3}{\Sigma k}$	1 1 1 2	5

X	Solution $L_1 = 3m$, $L_2 = 5m$ $W_1 = 50 kN/m$ $W_2 = 30 kN/m$		
	For AB Max free BM $= \frac{W_1 L_1^2}{8} = \frac{50 \times 3^2}{8} = 56.25 kN.m$	1	
	Area $A_1 = \frac{2}{3} \times 56.25 \times 3 = 112.5 kN m^2$	1	
	$\bar{x}_1 = \frac{3}{2} = 1.5 m$	1	
	For BC Max. free BM $= \frac{W_2 L_2^2}{8} = \frac{30 \times 5^2}{8} = 93.75 kN.m$	1	
	$A_2 = \frac{2}{3} \times 93.75 \times 5 = 312.5 kN.m^2$	1	
	$\bar{x}_2 = \frac{5}{2} = 2.5 m$	1	
	Applying theoram of three moments to spans AB and BC. $M_A L_1 + 2M_B(L_1+L_2) + M_C L_2 + \frac{6A_1\bar{x}_1}{L_1} + \frac{6A_2\bar{x}_2}{L_2} = 0$	1	
	$M_A = M_B = 0$ $\therefore 0 + 2M_B(3+5) + 0 + \frac{6}{3} \times 112.5 \times 1.5 + \frac{6}{5} \times 312.5 \times 2.5 = 0$ $16 M_B = -337.5 - 937.5 = -1275$		
	$\therefore M_B = \frac{-1275}{16} = -79.6875 kN.m$ (Hogging moment)	1	
	For Support Reactions Taking moments about B for ABC and BC $R_A \times 3 - 50 \times 3 \times \frac{3}{2} + 79.6875 = 0$	1	
	$\therefore R_A = 48.438 kN.$		
	$R_C \times 5 - 30 \times 5 \times 2.5 + 79.6875 = 0$ $R_C = 59.06 kN$	1	
	$\therefore R_B = 300 - (48.438 + 59.063)$ $= 192.5 kN.$	1	



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$M_B = 79.6875$ kN m

$R_A = 48.438$ kN

$R_B = 192.50$ kN

$R_c = 59.06$ kN