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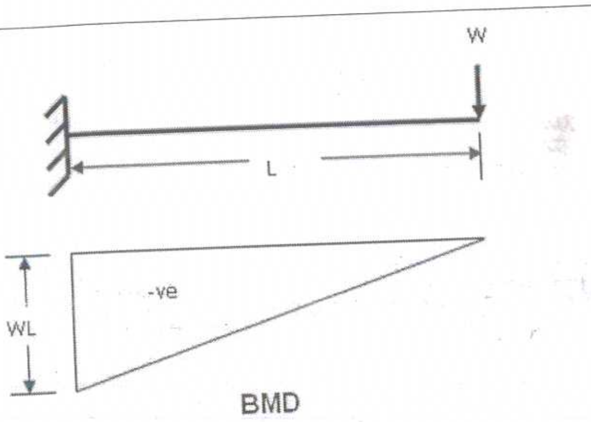
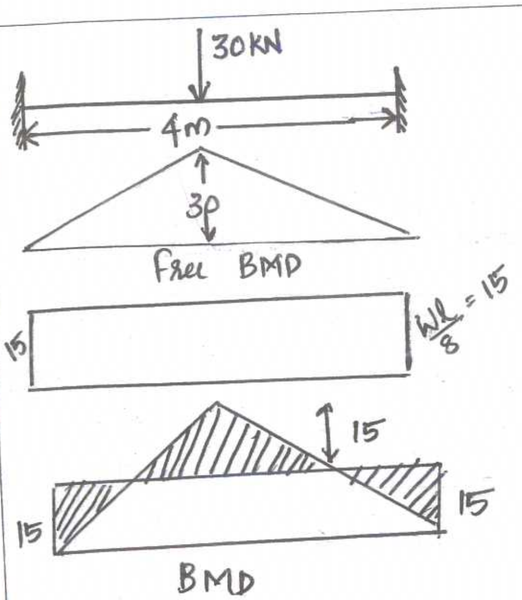
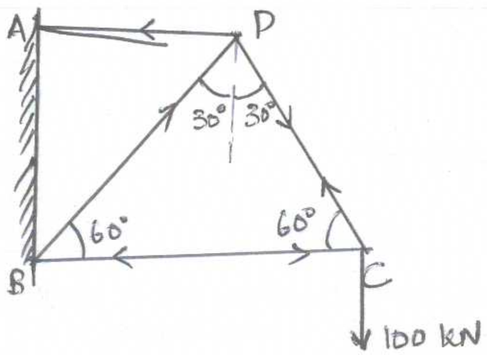
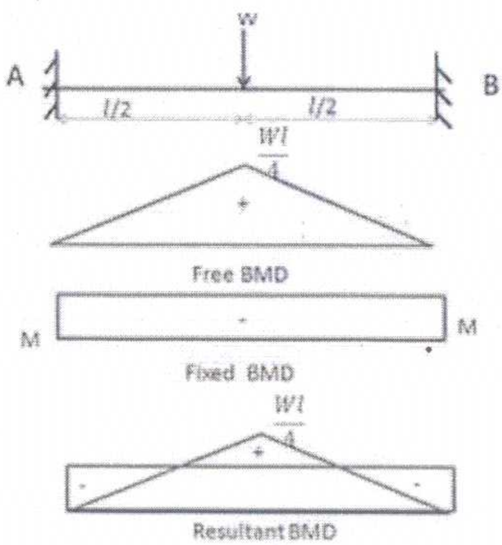
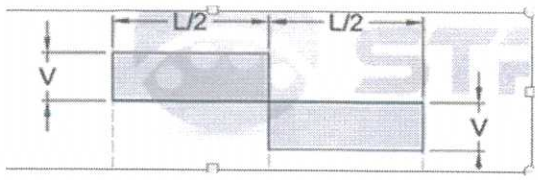
Revision (15) 4014 Course title: **THEORY OF STRUCTURES II**				
Qst. No.	**Scoring indicator**	**Split up score**	**Sub Total**	**Total**
	PART A			
I (1)	Slenderness ratio is the ratio of effective length of a column to least radius of gyration.	2	2	
I (2)	A structural member subjected to axial compressive load ~~Shear force at any section of the beam is the algreaic sum of all vertical forces acting either to the right or left of the section of the beam~~	2	2	
I (3)	A fixed beam is supported between two fixed ends. It is classified as a statically indeterminate beam, which involves more than three unknowns and the equilibrium equations of statics alone are not sufficient to determine the support reactions.	2	2	
I (4)	Slope at any section of a beam is the angle made by the tangent drawn at the corresponding section in the deflected beam with the horizontal.	2	2	
I (5)	The ratio of a moment produced at a joint to the moment applied at the other joint without displacing is called carry over factor.	2	2	
	PART B			
II (1)	The types of end conditions of column are a. Column with both ends hinged-Equivalent length le=l b. One end fixed and other end hinged-Equivalent length le=l/√2 c. Both ends is fixed- Equivalent length le=l/2 d. One end fixed and other end free- Equivalent length le=2l	1.5mark each	6	6
II (2)	b= 20 cm,d=15cm P=1000kN,e=5cm Maximum stress $f_{max}=\frac{P}{A}+\frac{My}{I}=\frac{1000}{20X15}+\frac{1000\,X\,5X10X12}{15\,X20^3}$ =8.33kN/cm^2 Minimum stress $f_{min}=\frac{P}{A}-\frac{My}{I}=\frac{1000}{20X15}-\frac{1000X5X10X12}{15X20^3}$ =1.67 Kn/cm^2	(3) 3	3 3	6
II (3)	When an eccentric load acts in such a way that on region around centre of gravity of section no tensile stress is developed anywhere in the region,then the area of that region is called core of the section. Eccentricity is the distance from the axis of the column or strut to the line of action of load which doesnot coincide with axis of column	(3marks) (3 marks)	3 3	6
II (4)	Strength criteria-In this the beam is designed to resist bending and shear stresses developed in the beam.This is ensured by maximum bending and shear stress are within limits. Stiffness criteria-According to this criteria the beam should be stiff to resist deflection or in other words beam should be stiff to resist deflection	(3marks)		

	within permissible limits.	(3marks)		
II (5)	 Slope of the beam= area of BMD/EI=$WL^2/2EI$	Figure-2 marks Steps-3 marks Final Answer-1 mark	2 3 1	6
II (6)	 $y_c = \frac{Wl^3}{192EI} = \frac{30\times4^3}{192EI} = \frac{10}{EI}$ Fixing End Moment $M_A \cdot l = \frac{1}{2}\times30\times4$ $M_A\times4 = 60$ $M_A = M_B = 15$ KNm	2 2 2	2 2 2	6
II (7)	The distribution factor for a member at a joint is the ratio of the stiffness factor of the member to the total stiffness factor of all members meeting at the joint. Fixed end moments are the moments induced at the ends of the member due to applied loads considering the member to be fixed at both the ends	(3 marks) (3 marks)	3 3	6

	PART C			
III (a)	L=12m=1200cm $A=1m^2=10^4cm^2$ $E= 2 \times 10^4$ kN/cm^2 $I= 100 \times 100^3/12 = 8.33 \times 10^6$ cm^4 a. Both ends are pinned $Pe=\frac{\Pi^2 EI}{L^2} = \frac{\Pi^2\ 2x10^4\ \ x8.33x10^6}{1200^2}$ =1142310.8 kN b. One end fixed and other end free $Pe=\frac{\Pi^2 EI}{L^2} = \frac{\Pi^2\ 2x10^4\ \ x8.33x10^6}{2400^2}$ =285577.7 kN	(Steps-2 Answer-2) (Steps-2 Answer-2)	8	
III (b)	 Joint C $\sum H=0, \sum V=0$ $P_{CD}\sin60=100$ P_{CD}=115.47 kN $P_{CB}=P_{CD}\cos 60=115.47\cos 60=57.73$ kN Joint D $\sum H=0, \sum V=0$ $P_{DC}\cos30=P_{DB}\cos30$ $P_{DC}=P_{DB}$=115.47 kN. $P_{DA}=P_{DC}\sin30+P_{DB}\sin30$ P_{DA}=115.47/2+115.47/2=115.47	(1) (1) (1) (1) (1) (1) (0.5) (0.5)	7	15
IV (a)	Assumptions i. All members are pin joined or hinged ii. Frame is loaded only at joints iii. ~~Frame should be perfect frame~~ iv. The self weight of the members of the structure is neglected. Method of Joints i. In this method, each and every joint in the frame is treated separately as free body in equilibrium. ii. The unknown forces are then determined by equilibrium conditions such as $\sum H=0, \sum V=0$ iii. Sum of all vertical forces and horizontal forces is equated to zero. iv. While taking joints, care should be taken that at any instant joint	(3)	6	15

	should not contain more than two members in which the forces are unknown.	(3)		
IV (b)	l=4m=4000mm D= 20cm= 200mm $E=2\ x10^5 N/mm^2$ t =1 cm=10mm d= 20-1-1=18cm=180mm $I=\Pi/64(D^4-d^4)=\Pi/64(220^4-180^4)=2.7009\ x10^7 mm^4$ i. When both ends are fixed L=l/2=4000/2=2000mm $P_E=\frac{\Pi^2 EI}{L^2}=\frac{\Pi^2\ 2x10^5\ \ x2.7009x10^7}{2000^2}=13328.8$ kN ii. When one end fixed and other end free L=l/√2=4000/2=2828.42mm $P_E=\frac{\Pi^2 EI}{L^2}=\frac{\Pi^2\ 2x10^5\ \ x2.7009x10^7}{2828.42^2}=6664.4$ kN	1 2 2 2 2	9	
V (a)	Conditions for stability of a dam i. If bending stress is greater than direct stress there will be tensile stress on one side of the dam .So inorder to avoid tension in the base of the dam bending stess should be less than direct stress. $e \leq b/6$,the eccentricity should be less than b/6 ii. For the equilibrium of the dam the restoring moment should be greater than overturning moment. This condition prevent dam from overturning iii. To prevent sliding of dam the maximum available frictional resistance should be greater than horizontal water pressure, hence the dam will not slide $F>\mu W$ iv. To prevent crushing at the base of the dam the maximum stress at the base of the dam should be less than the permissible stress in masonry or concrete	2 2 2 2	8	
V (b)	 			



Mal=Wl/4xlxl/2

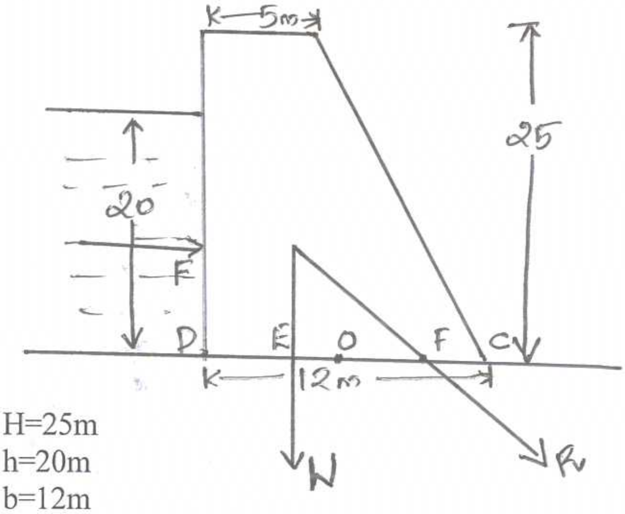
Ma=Wl/8

Mb=Wl/8

$R_A=R_B=W/2$

BM at midspan of a fixed beam=Wl/8

VI (a)

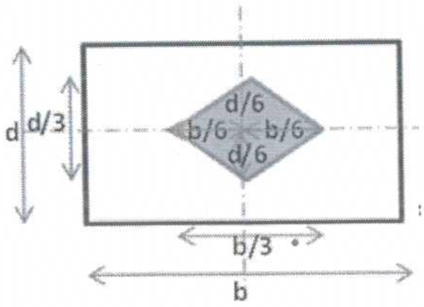


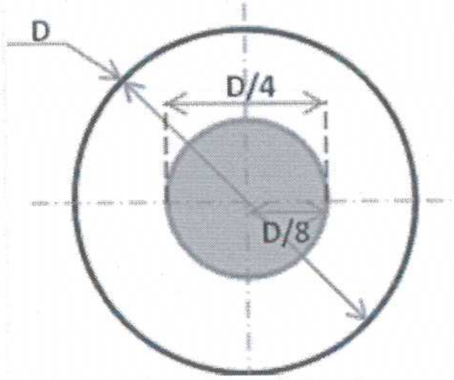
H=25m
h=20m
b=12m
a=5m
P=25 kN/m3
w =10kN/m3
Self weight of the dam,W=25x(12+5)/2x25x1=5312.5kN
Water pressure of the dam $F=wh^2/2 = 10 \times 20^2/2 = 2000$kN
Resultant force $R=\sqrt{(F^2+W^2)} = \sqrt{(2000^2+5312.5^2)}=5676$ kN
Taking moment about F,Fxh/3=WxEF

$$EF=\frac{Fxh}{Wx3}=\frac{2000x20}{5312.5x3}=2.5m$$

DF=DE+EF=4.49+2.5=6.99m
Resultant R cuts the base at 6.99 from D
Eccentricity e=DF-b/2=6.99-12/2=0.99m
Maximum stress occurs at edge C of the base=$f_{max}=\frac{W}{b}\left(1+\frac{6e}{b}\right) = \frac{5312.5}{12}(1+6\frac{0.99}{12})=661.84$ kN/m^2
Maximum stress occurs at edge D of the base= $f_{min}=\frac{W}{b}\left(1-\frac{6e}{b}\right) = \frac{5312.5}{12}(1-6\frac{0.99}{12})=223.54$ kN/m^2

VI (b) For no tension condition
$e \leq Z/A \leq (bd^2/6)/bd \leq d/6$ or b/6
The stress will be compressive .The shaded area is the core of the

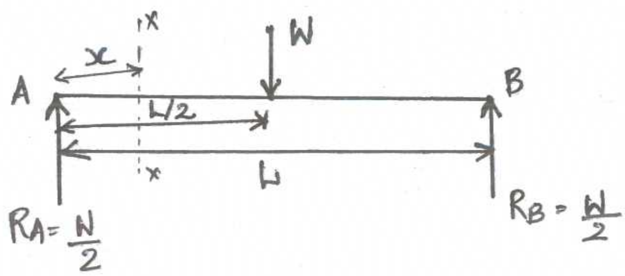
<table>
<tr><td></td><td>section.




Modulus of section of circular area=$\Pi D^3/32$
Area of the circular section=$\Pi D^2/4$
$e \leq Z/A$
$\leq (\pi D^3/32)/ (\pi D^2/4)$
$\leq D/8$</td><td>3.5

3.5</td><td>7</td><td>15</td></tr>
<tr><td>VII
(a)</td><td>R_A = Reaction force at support A = W/2
R_B = Reaction force at support B = W/2
θ_A = Slope at support A
θ_B = Slope at support B
$M = E.I\ \frac{d^2y}{dx^2}$

$E.I\ \frac{d^2y}{dx^2} = \frac{W.X}{2}$

Let us consider the bending moment determined earlier about the section XX and bending moment expression at any section of beam.

</td><td>1

2

2</td><td>9</td><td>15</td></tr>
</table>

	$E.I\frac{dy}{dx}=\frac{w..x^2}{4}+C_1$ Slope will be zero at the centre of the loaded beam dy/dx = 0, at x= L/2 $C_1=-\frac{W.L^2}{16}$ $E.I\frac{dy}{dx}=\frac{w..x^2}{4}-\frac{W.L^2}{16}$ At x=0,dy/dx=ΘA=ΘB $\theta_A=-\frac{W.L^2}{16\,EI}$ $\theta_B=-\frac{W.L^2}{16\,EI}$ $EI\frac{d^2x}{dy^2}=W\,x^3/12+C_1x+C2$ At x=0,C2=0 $EI\frac{d^2x}{dy^2}=W\,x^3/12+C_1x$ $=Wx^3/12-WL^2\,x/16$ At x=L/2 $EI\frac{d^2x}{dy^2}=WL^3/96-WL^3/32$ $=-WL^3/48$ $\frac{d^2x}{dy^2}=WL^3/48EI$	2 2		
VII (b)	W= 50kN $I=15000cm^4$ $E=2x\,104\,cm^4$ l =4m $Yc=Wl_1^3/3EI=50x500^3/(3x2x10^4x15000)=6.994$ cm	6	6	
VIII a.		2	8	15

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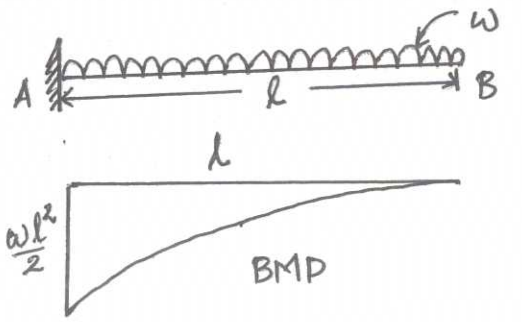

$SF_B=wl^2/2xlxl/EIx1/3=wl^3/6EI$
$\Theta B= wl^3/6EI$
$BM_B=wl^3/6EIx\frac{3}{4}l=wl^4/8EI$
$y_B=wl^4/8EI$
</td>
<td>6</td>
<td></td>
<td></td>
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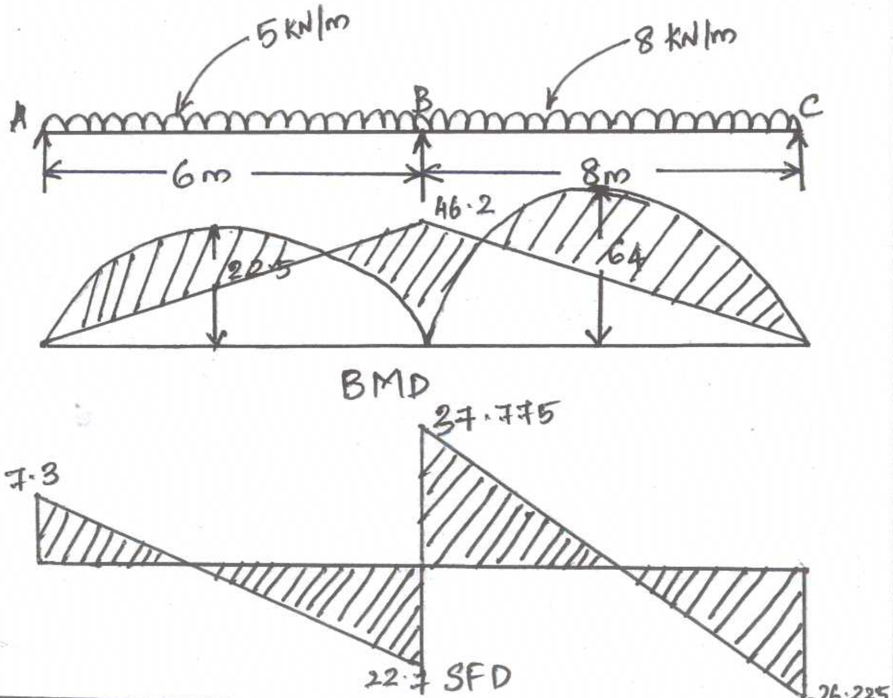
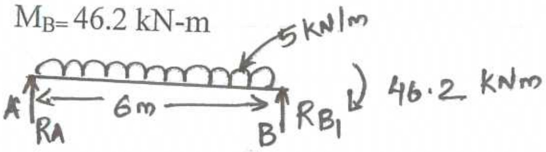
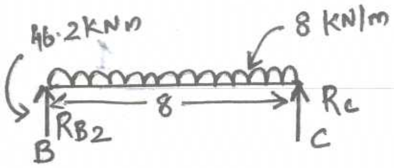
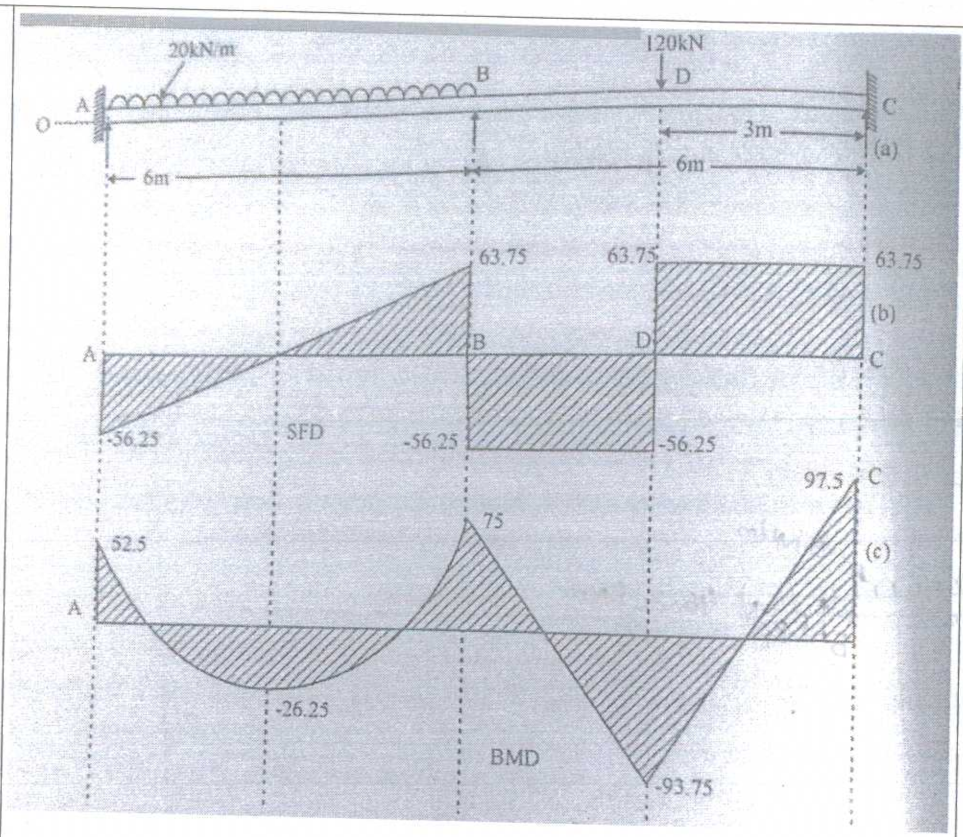
<tr>
<td>VIII
b.</td>
<td>
l=6m=6000mm
$I=300 \times 10^4\ mm^4$
Yc=4 mm
$E=200 \times 10^3\ N/mm^2$
$Yc=\frac{5wl^4}{384EI}=\frac{5w6000^4}{384x200x10^3x300x10^4}$,
4=0.281w
w=14.2 kN/m
</td>
<td>2
2
2
1</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>IX
a</td>
<td>


</td>
<td>2
2</td>
<td>15</td>
<td>15</td>
</tr>
</table>

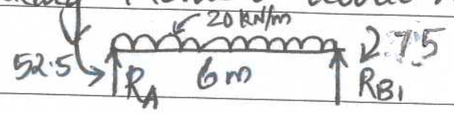
	For span ABC			
	$M_A(l_1)+2M_B(l_1+l_2)+Mcl2=6a_1x_1/l_1+6a_2x_2/l_2$	1	1	
	$l_1=3m$			
	$l_2=3.6m$			
	$M_A=0, Mc=0$			
	$6a_1x_1/l1=5x\ 6^3/4=270$			
	$6a_2x_2/l2=8x8^3/4=1024$	2	2	
	$2M_B(6+8)+0=270+1024$			
	$M_B=46.2$ kN-m			
				
	Taking moments about A			
	RAx6-5x6x6/2+46.2=0			
	RA=7.3kN			
	RB1=30-7.3=22.7kN	4	4	
				
	Taking moment about B	4	4	
	Rcx8-8x8x8/2+46.2=0			
	RC=26.225kN			
	RB2=37.775kN			

X a		3	15	15
	M_{FAB}=-wl²/12=-20x6²/12=-60 kNm M_{FBA}=-wl²/12 =20x6²/12= 60 kNm M_{FBC}=-Wl/8 =120x6/8=- 90 kNm M_{FBA}=Wl/8 =120x6/8= 90 kNm Stiffness factor at joint B K_{BA}=4EI/6=2EI/3 K_{BC}=4EI/6=2EI/3 At joint B distribution factors are k_{BA} =k_{BA}/∑k_B =1/2 k_{BC}=k_{BC}/∑k_B =1/2	1 1		

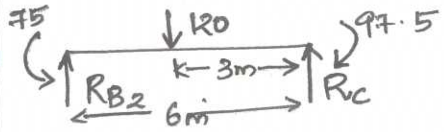
JOINT	A	B	B	C
		1/2	1/2	
Fixed end moments	-60	60	-90	90
Distribute		15	15	
Carryover	7.5			7.5
Final end moments	-52.5	75	-75	97.5

4

Taking Moment about A.



$R_A + R_{B1} = 120\,kN$

<table>
<tr><td></td><td>R_A-20X6X6/2+75-52.5=0
R_A=56.25 kN
R_{B1}=120-56.25=63.75 kN


Taking moment about B
R_{B2}+R_C=120
R_CX6-120X3-97.5+75=0</td><td>3</td><td></td><td></td></tr>
<tr><td></td><td>R_C=63.75 kN
R_{B2}=56.25 kN</td><td>3</td><td></td><td></td></tr>
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