MAHARASHTRA STATE BOARD OF TECHNICAL EDUCATION
(Autonomous)

(ISO/IEC -270001 – 2005 certified)

WINTER -2019 EXAMINATION

Subject code: 17422 **Model Answer**

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.

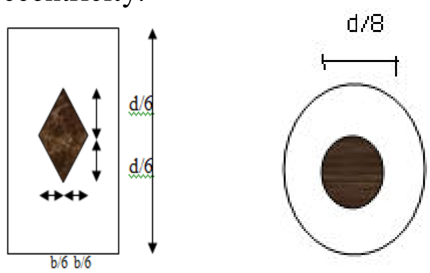
2) The model answer and the answer written by candidate may vary but the examiner may try to assess the understanding level of the candidate.

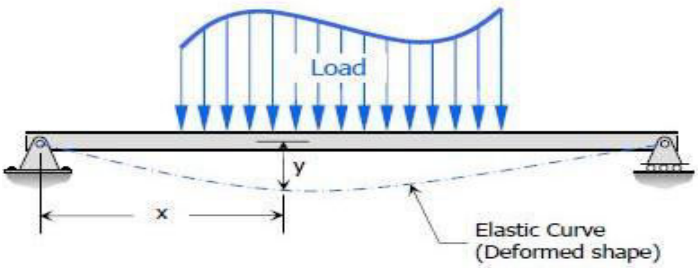
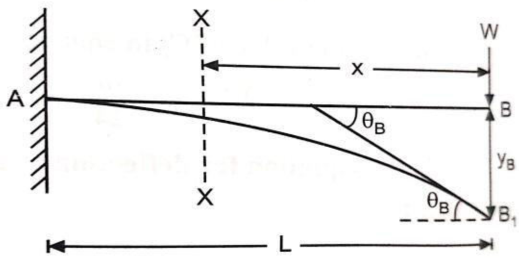
3) The language error such as grammatical, spelling errors should not be given more importance. (Not applicable for subject English and communication skill).

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

Q. No.	Question and Model Answers	Marks
1.A	**Attempt any SIX of the following:**	**12M**
a)	**Define Limit of eccentricity**	
Ans:	Limit of eccentricity: A load whose line of action does not coincide with the axis of a member is called an eccentric load .The distance between the eccentric axis of the body and the point of loading is called an eccentric limit 'e'. The centrally located portion of a section within which the load must act so as to produce only compressive stress is called a core or kernel of section or limit of eccentricity.  $e \le d/6$ or $e \le b/6$ $e \le D/8$ core of section (Limit of eccentricity for a Rectangular and circular section)	**2 M**

b)	**Write the formula for calculation of radius of curvature**	
Ans:	calculation of radius of curvature Bending Equation σ/y = M/I = E/R or R = E *I/M Where, M = Bending moment E = Modulus of elasticity I = Moment of Inertia R = Radius of curvature	**1 M** **1 M**
c)	**Define deflection of beam**	
Ans:	 **Figure: Elastic curve** The vertical Displacement of a point on a beam with respect to its original position before loading is called deflection of beam. It is denoted by “Y”	**2 M**
d)	**A cantilever of span 'L' carries a point load 'w' at 'L' from fixed end.** **State deflection at tree end in terms of El.**	
	 Maximum deflection=Y max = YB = WL3 / 3EI Where W= Point load L= length (span) of beam(m) E= modulus of elasticity(KN/m2) I= moment of inertia of a beam m4	**2 M**
e)	**Define fixed beam**	
Ans:		

2 M

A beam whose end supports are such that the end slopes remain zero is called fixed beam

f) **Define carry over factor of moment distribution method.**

Ans: It is the ratio of moment produced at a joint to the moment applied at the other joint without displacing it . It is $M_A/M_B = ½$ or

$M_A/M_B = 0$ zero

2 M

g) **Define stiffness factor.**

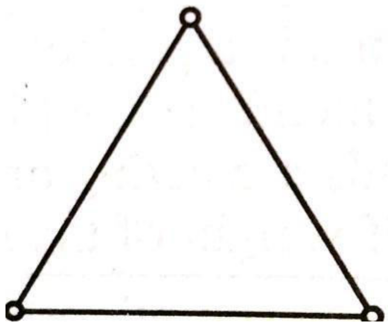
Ans: **Stiffness factor:** It is the moment required to obtain unit rotation at an end, without translating it.

2 M

h) **Explain perfect truss with example.**

Ans: A frame which has members just sufficient to keep in stable equilibrium when loaded at its joints , is called perfect truss its shape remains unchanged .Example For a triangle ,

1 M



Basic perfect frame

1 M

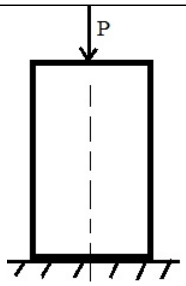
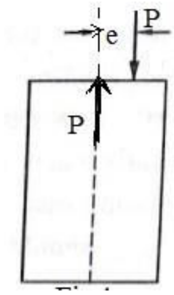
Where, N =3, j =3

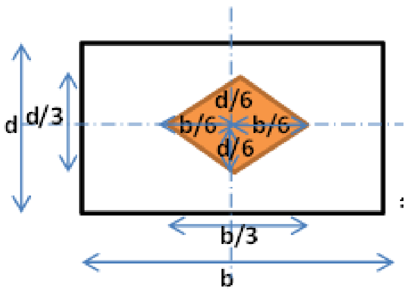
2 j-3 = 2 x 3 =3 therefore the perfect truss .

Q.1 B **Attempt any Two of the following:** **8 M**

a) Differentiate between Direct load and eccentric load

	Direct load	Eccentric load
01	Direct load is that force which acts at centroidal longitudinal axis of the member.	Eccentric load is that force which act away from centroidal longitudinal axis of the member
02	Due to direct axial load causes only direct stress .	Due to effect of eccentricity, eccentric load causes direct as well as bending stresses.
03	Due to direct loading it gives rise to Direct stress either tensile or	Due to eccentric loading it gives rise to bending stresses which are tensile and

	compressive as per the nature of external load.	compressive in nature and they both exist together in a member on either side of neutral axis or centroidal axis.	
04	Direct stress = σo =P/A	Bending stress = σb = M * y / I Resultant stresses reach a higher value. Resultant stresses = σdirect + σbending	**1 M mark each**
05	 Direct Load	 Ecccentric load	

b)	**Define core of section . Sketch it Rectangular section**	
Ans :-	Core of a section: Core of the section is that portion around the centroid in within which the line of action of load must act, so as to produce only compressive stress is called as core of the section. It is also defined as the region or area within which if load is applied, produces only compressive resultant stress. If Compressive load is applied, the there is no tension anywhere in thesection.	**2 M**
	e = Core of section e= d/6 or e = b/6	**1 M**
	 Rectangular section	**1 M**
c)	**Explain steps involved in method of joint for calculation of forces in the member of frame**	
Ans	Calculation of forces in the member of frame by Method of joint , A truss is one of the major types of structures and is especially used in design of bridges and buildings . Step wise 1.Examples of trusses 2. Trusses , joint and forces 3. UsingTrigonometry 4. Draw a free body diagram 5.Solve for reactionary forces of truss 6.Locate a joint with only two members	**½ M for each step**

	7. Determine the unknown forces of the joint. $\sum F_x = 0$ $\sum F_y = 0$ 8. Calculation will give you a negative or a positive number designating the real direction of forces. Full analysis of a simple truss by the method of joint	
Q.2	**Attempt any THREE of the following:**	**12 M**
a)	**A rectangular column is 250 mm wide and 100 mm thick. It carries a load of 200 kN at an eccentricity of 100'mm in the plane bisecting thickness. Find the maximum and minimum intensities of stress in section.**	
Ans:	A = 250 x 100 = 25 x 103 mm^2 P= 200 kN Iyy = 100 x 2503 / 12 = 130.208 x 10^6 mm4 Zxx = Iyy / Y = 130.208 x 106 / 125 = 1.041 x 10^6 mm3 M = P x e = 200 x 103 x 100 = 20 x 10^6 N-mm. Direct stress = 6d = P / A = 200 x 10^3 / 25 x 10^3 = 8 N/mm^2 Bending stress 6b = M / Z = 20 x 106 / 1.041 x 10^6 = 19.21 N/mm^2 Maximum stress = 6d + 6b = 8 + 19.21 = 27.21 N/mm^2 (Comp.) Minimum stress = 6d - 6b = 8 - 19.21 = (-) 11.21 N/mm^2 (tensile)	**1 M** **1 M** **1 M** **1 M**
b)	**(b) A hollow C.I. column of external diameter 300 mm and-internal diameter 250 mm carries an axial load 'w' kN and a load of 100 kN at an eccentricity 125 mm. Calculate the maximum value of 'w' so as to avoid the tensile stresses,**	
Ans:	Axial load = W, eccentric load P = 100 kN., eccentricity e = 125 mm Area A = π *($D1^2$- $D2^2$)/4 = π x(300^2- 250^2) /4 = 21.59 x10^3 mm^2 . Moment of Inertia I = π* D^4 /64 = π (300^4- 250^4) /64 = 205.86 x 10^6 mm^4 Z = I / ymax = 205.86 x 106 / 150 = 1.3724 x $10^6 mm^3$ M = P x e = 100 x 10^3 x 125 = 12.5 x 106N-mm. For No tension, 6d = 6b Direct stress = (W / A) +(P / A) 6d = (W / A) +(100 x 10^3 /21.59 x10^3) Bending stress 6b = M / Z = 12.5 x 106 / 1.3724 x 10^6 = 9.108 N/mm^2 For no tension, 6d = 6b (W /21.59 x10^3) + 4.63 = 9.108 W = 4.47 * 21.59 x10^3= =96.64 x 10^3 N	**1 M** **1 M** **1 M** **1 M**
c)	**A chimney having diameter 4 m and 50 m height. It is subjected to a horizontal wind pressure of 1.5 kPa normal to chimney. Find maximum bending stress in chimney.**	
	h = 50 m, p = 1.5 kN/m^2, D= 4m Assum i) Density of masonary Y = 22 kN/m^3 , and Coefficient of Wind pressure C=1 A = π * D^2 /4 = π * 4^2 /4 = 12.56 mm2 ii)WindForce P = C x p x D x h = 1 x 1.5 x 4 x 50 = 300kN iii) Self wtofchimney W = Y x volume = Y x A x h = 22 x 12.56 x50 =13.82 x 10^3 N	**1 M** **1 M**

iv) Direct stress = 6d = W/A = 13.82 x 10^3 / 12.56 = 1.099x 10^3 N /mm^2

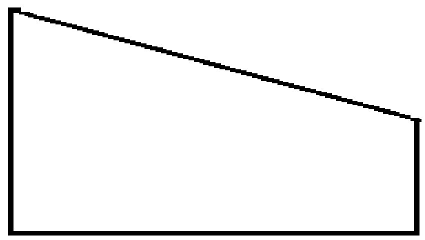
Moment of Inertia I = π (D^4)/64 = π 4^4 /64 =12.566 mm^4

Bending stress 6b = M / Z

= M* Y /I = P e * (d/2) /I = 300 * (50/2) *(4/2) /12.56 **1 M**

=1193. 66 N /mm^2

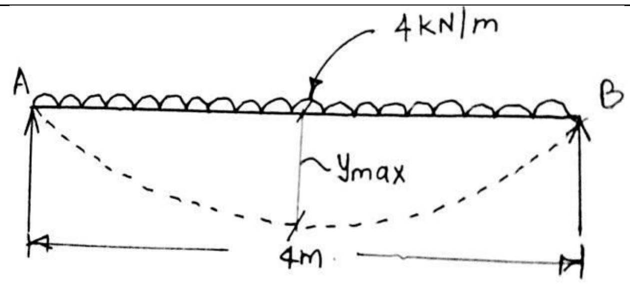
6max =60 + 6b , = 2292N/mm^2 (Comp.) 6min =60 - 6b = 94.66 N/mm2(Comp.) **1 M**



2292N/mm^2 94.66N/mm^2

d) **A simply supported beam carries a u.d.I. of 4 kN/m over entire span 4 m. Find deflection at mid span in terms of EI.**

Ans



1 M

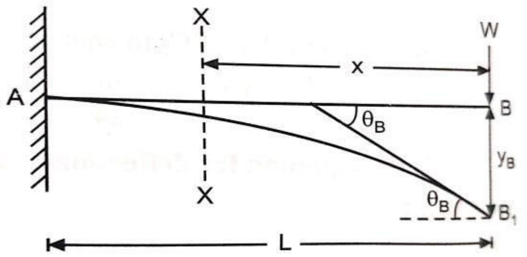
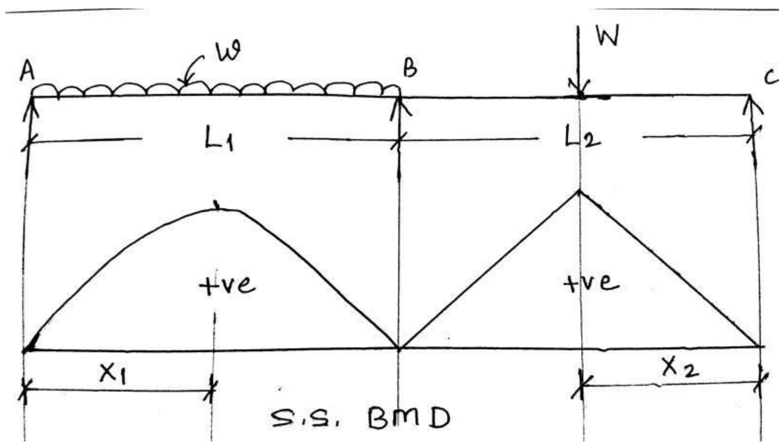
Maximum deflection at mid span in terms **1 M**

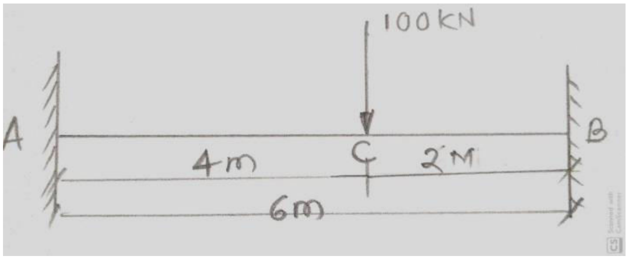
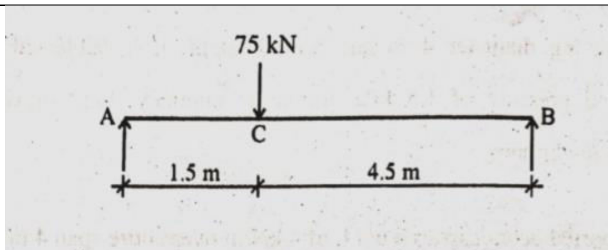
Deflection at Centre Ymax = $\frac{5wL^4}{384EI}$ **1 M**

Ymax = $\frac{5x4x4^4}{384EI}$ **1 M**

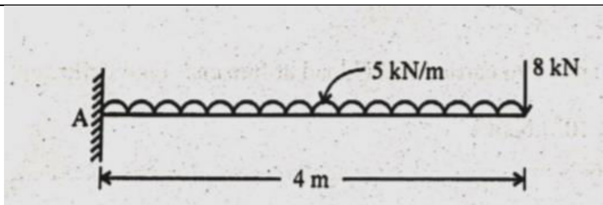
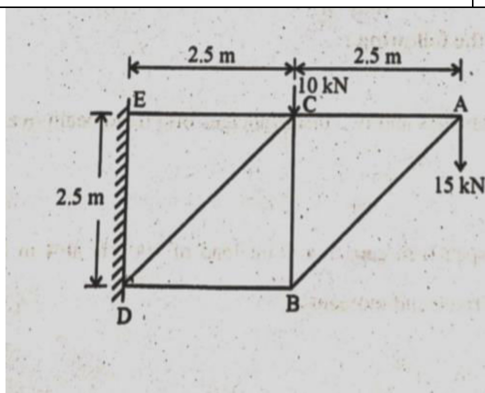
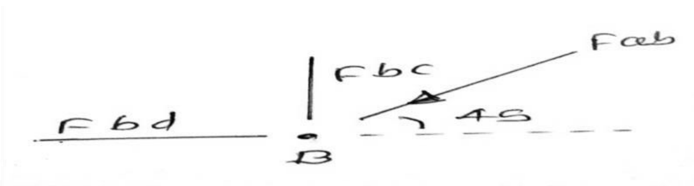
Ymax = $\frac{13.333}{EI}$

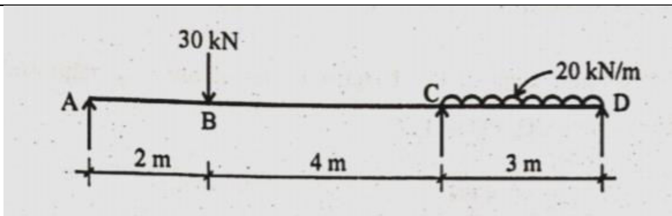
e) **A cantilever of span 2 m carries 10 kN load at free end. Find deflection at free end if El = 15 x 10^3 kN.m2**

Ans:	 El = 15 x 10 kN.m2 L = 2m W = 10 kN Deflection at free end YB = - WL^3/ 3 EI = - 10 x 2^3 / 3 x 15 x 10^3 = - 1.77 mm	**2 M** **2 M**
f)	**Write Clapeyron's moment theorem for a beam with different M.I. giving meaning of each term.**	
Ans:	The clapeyron's theorem of three moment is applicable to two span continuous beams .It state that " For any two consecutive spans of continuous beam subjected to an external loading and having different moment of inertia, the support moments M_A, M_B and M_C at supports A,B and C respectively are given by following equation  If the moment of inertia is not constant then claperon's theorem can be stated in the form of following equation. $M_A \frac{L_1}{I_1} + 2M_B\left(\frac{L_1}{I_1} + \frac{L_2}{I_2}\right) + M_C \frac{L_2}{I_2} = -\left[\frac{6A_1X_1}{L_1I_1} + \frac{6A_2X_2}{L_2I_2}\right]$ Where, L_1 and L_2 are length of span AB and BC respectively. I_1 and I_2 are moment of inertia of span AB and BC respectively. A_1 and A_2 are area of simply supported BMD of span AB and BC respectively.	**1 M** **1 M** **1 M** **1 M**

	X_1 and X_2 are distances of centroid of simply supported BMD from A and C respectively	
Q. 3	**Attempt any <u>FOUR</u> of the following**	**16 M**
a)	**State any two advantages and two disadvantages of fixed beam over simply supported beam**	
Ans.	**Advantages** 1) Fixed beam is more stiff, strong and stable than simply supported beam. 2) For the same span and loading a fixed beam has lesser value of bending moment as compared to simply supported beam. 3) For the same span and loading fixed beam has lesser value of deflection as compared to simply supported beam. **Disadvantages.** 1) A little sinking of one support over ,the other induces additional moment at each end 2) Extra care has to be taken to achieve correct fixity at the ends. 3) Due to end fixity temperature stresses induced due to variation in temperature	**2 M** **2 M**
b)	**Fixed beam of span 6 meter caries a point load of 100 KN at 4 meter from left support calculate fixed end moment**	
Ans.	 MA = wab^2/L^2 $=100*4*(2)^2/6^2$ **<u>= - 44.44 KN/M</u>** MB = wba^2/L^2 $=100*2*(4)^2/6^2$ **<u>= - 88.89 KN/M</u>**	**1M** **1M** **1 M** **1M**
c)	**Calculates Maximum Deflection at A Beam shown in the fig. use Macaulay's Method** **E=2 X108 KN/m2 & I =0.733 X 10-4 m4**	
Ans:	 RA= wb/l	

d)	=75*4.5/6 = **<u>56.25 KN.</u>** RB= wa/l =75*1.5/6 = **<u>18.75 KN.</u>** Consider a section x-x at a distance "x" from 'A' in portion CB. EI d^2y/dx^2 = Mx = 56.25 x -75 (x - 1.5) Integrating w. t .r. x EI dy/dx =56.25 $x^2/2$ + C1-75$(x-1.5)^2/2$ Again integrating w r t x EI y= 56.25^2 $x^3/6$ + C1x+C2 -75 * $(x-1.5)^3/6$ At x=0 & Y= 0 C2=0 At x=6 Y=0 0=56.25 $(6)^3/6$ + c1*6+0-75$(6-1.50)^3/6$ =2025+c1*6-1139.06 C1*6=885.93 C1*6=885.93 **<u>C1=-147.65</u>** EIY= 56.25^2 $(x^3/6)$ -147.65x-75$(x-1.5)^3/6$ E dy/dx = 56.25 $x^2/2$ -147.65 -75 $(x-1.5)^2/2$ dy/dx=0 0=28.125x^2 - 147.5 - 37.5$(x-1.5)^2$ 0=28.125 x^2-147.5 - 37.5 (x^2 3x+2.25). 0= - 28.125x^2-147.5-37.5 x^2 +112.5x -84.37 0= - 9.375x^2 + 112.5x – 231.80 By solving above equation X=112.5$_{+-}$√(112.5^2)-4*9.375*231.78/2*9.375 X=112.5$_{+-}$ √(3964.5)/18.75 X=112.5$_{+-}$62.96/18.75 **<u>X=2.64m X=9.35m</u>** **PUT X=2.64m** EIY=56.25 * 2.64/6 - 147.5*2.64 – 75 $(2.64-1.5)^3$ / 6 EIY=172.49-389.4-18.51 Y=-235.43/EI Y= -235.42/2*10^8*0.733*10^{-4} **<u>Y_{max} = -0.016M</u>** **Y max= - 16 mm. (downward deflection).**	1M 1M 1 M 1 M
d)	**Find the slope at free end of beam as shown in figure.**	

Ans:	 **θ** A- slope at free end of beam due to udl. = $wL^3/6EI$ **θ** B-slope at free end of beam due to point load = $wL^2/2EI$ **θ** max = **θ** A + **θ** B $=wL^3/6EI + wL^2/2EI$ $=5*4^3/6EI + 8*4^2/2EI$ $=53.33/EI + 64/EI$ $=117.33/EI$	**1 M** **1 M** **2 M**
e)	**State Any Four Assumptions Made In Analysis Of Simple Frame**	
Ans:	1) The frame is perfect one i.e. the relation n= 2j-3 always satisfy 2) All the member are hinged or pin jointed at the end 3) The loads are acting only at the joint 4) The self weight of member is neglected.	**1 M EACH**
f)	**Determine the forces in member AB and BC use method of section** **Let us consider the equilibrium of truss to right of section.**	
Ans :	  let us consider the equilibrium of the truss to right of section 1-1 $\sum fy=0$ FAB SIN45=15 FAB=15/ SIN45	

	=21.21KN.(COMP) let us consider the equilibrium of the truss to right of section 2-2 $\sum fy=0$ 0=-FAB Sin 45 + FCB FCB=21.21 Sin45 **FBC=15KN (TENS).**	**2 M** **2 M**
Q. 4	**Attempt any FOUR of the following**	**16 M**
a)	**Write the step wise procedure for analysis of continuous beam**	
Ans :	Step 1 to draw bending moment diagram 1) Assume the continuous beam as a series of simply supported beam and draw the usual μ diagram due to vertical loads 2) Calculate 6ax̄ / L (calculate 6ax̄ / L for varying moment of Inertia) 3) Apply the CLAPEYRON THEOREM , three moment and find the unknown fixed end moment draw the μ diagram. 4) Superimpose the μ diagram over μ diagram and draw the net bending moment diagram Step to draw a **SF** diagram 1) Calculate the reaction of simply supported beam 2) Calculate the reaction due to difference of fixed end moment 3) Superimposed reaction due to above two cases and find the reaction of continuous beam 4) Knowing the support reactions draw SF Diagram as usual.	**2 M** **2 M**
b)	**Find d support moment of a continuous beam as shown in figure use clapeyron's theorem.**	
Ans :	 **GIVEN DATA:-** Span AC=6m & CD=3m Known moments are MA=MD=0 A)Assume The AC & CD As A Simply Supported Beam draw μ diagram $BM_B=Wab_1b_1/l_1$	

$=30*2*4/6$

BM_B =40KN.M — 1 M

$BM_C=wl^2/8$

$=20(3)^2/8$

=22.5KN.M — 1 M

1) for span AC : for simply supp. beam carrying eccentric load.

$a1=(1/2)*40*6=120$

$\bar{x}1=(6+2)/3=2.67$

$6a1\bar{x}1/L_1=(6*120*2.67)/6$

$=320.4$

2) for span CD for simply supp. beam carrying udl.

$6a_2\bar{x}_2/L_2=$

$a_2=(2/3)*(3*22.5)$

$a_2=45$

$6a_2\bar{x}_{2=}(6*45*1.5)/3$

$=135$

$MA*L1+2MC(L1+L2)+MD*L2=-\{(6a1\bar{x}1/L_1)+(6a_2\bar{x}_2/L_2)\}$

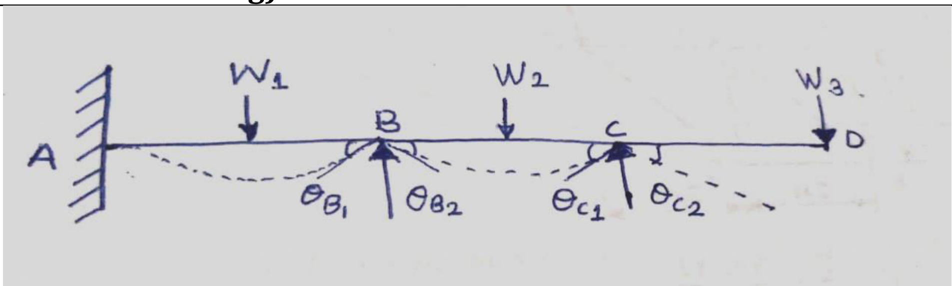
$0+2MC(6+3)+0=-(320.4+135)$

$18MC=455.4$

MC=- 25.3KN-m — 2 M

c) **Draw Typical Deflection Curve For Continuous Beam Of three Spans.(One End Fixed And Other Overhang)**

Ans : — 4 M



θ B1=θ B2

d) **Differentiate between symmetrical and unsymmetrical portal frame**

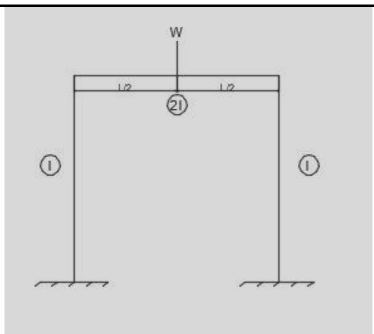
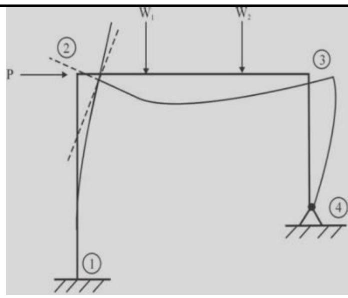
Ans :

SR	SYMMETRICAL	UNSYMMETRICAL
1	A symmetrical portal frame is that in which both columns are identical, having same length, same end condition, same M.I, and same modulus of elasticity and which is subjected to symmetrical loading	A unsymmetrical portal frame is that in which both columns are not same , having different length, different end condition, different M.I, and different modulus of elasticity and which is subjected to unsymmetrical loading
2	Does not sway to any side	Sway to any side

1 M

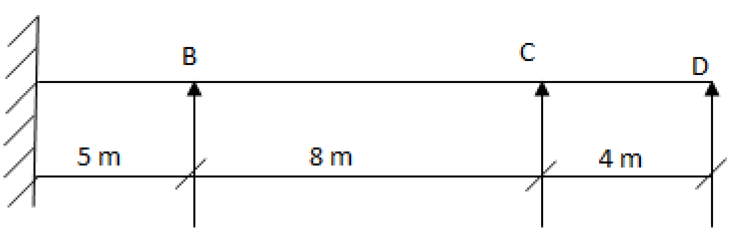
1 M

2 M

3	 	

e) **Continuous beam ABCD is supported at A,B,C& D AB=5 m, BC= 8 m and CD = 4 m. Calculate the distribution factor at joint B and C support a is fixed end**

Ans :



Stiffness factors=

KBA=4EI/L_1 = 4EI/5 = **0.8 EI**

JOINT B

KBC =4EI/L_2 = 4EI/8 = **0.5EI**

∑ KB=1.3 EI

DFBA =0.8EI/1.3EI

=0.62

DFBC = 0.5EI/1.3EI= **0.38**

2 M

JOINT C

= KCB

= 4EI/L_2

=4EI/8

= **0.5EI**

KCD = 3EI/L3 = 3EI/4

=**0.75EI**

∑ KC =1.25EI

2 M

DCB= 0.5EI/1.25EI

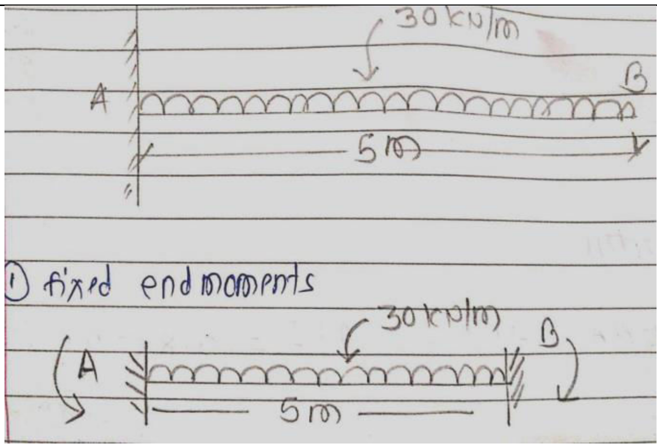
=0.4

DCD= 0.75EI/1.25EI

=0.60

f) **Using moment distribution method determine the moment at fixed end of propped cantilever of span 5 m carrying uniformly distributed load 30KN/m over the entire span**

Ans :



MAB=-WL²/12
=-30*5²/12
=-62.5KN.M

1 M

MBA=WL²/12
=30*5²/12
=62.5KN.M

1 M

<u>Stiffness Factors:</u>

1. As there is no continuation at joint b & joint a. A is fixed then there is no relative stiffness and there will not be any distribution factors**.**
2. distribution factor :- no distribution factor
3. Moment distribution table.

Point	A	B
Member	AB	BA
Distribution factor		
Fixed end moment	-62.5	62.5
Balance Carry Over To B	-31.25	62.25
Final moment	93.75	0

2 M

<u>MAB= 93.75 (HOGG)</u>

Q. 5 **Attempt any <u>TWO</u> of the following:** **16 M**

a)	**A masonry wall 6 m high of solid rectangular section 3 m wide 1 m thick. A horizontal wind pressure 950 N/m^2 acts on a 3 m side. Find the maximum and minimum stress induced at base, if the density of masonry is 19.5 kN/m^3. Draw stress diagram**	

Ans : Given H = 6 m, B = 3 m, T = 1 m, P = 950 N/m^2 act on 3 m side

γ_m = 19.5 kN/m^3

i) Calculate masonry Weight of the wall

W = A x H x γ_m

= 3 x 1 x 6 x 19.5

W = 351 kN **1 M**

ii) Calculate Area A = 3 x 1 = 3 m^2

iii) Calculate Direct stress $\sigma_D = \frac{W}{A} = \frac{351}{3} = \mathbf{117\ kN/m^2}$ **1 M**

iv) Calculate Bending stress

$\sigma_b = \frac{M}{I} x\, Y$

M = P x $\frac{h}{2}$

P = Total wind load

P = wind pressure intensity x projected area

P = ρ x A

P = 0.95 x 3 x 6 = 17.1 kN

M = 17.1 x $\frac{6}{2}$ = 51.3 KN.m **1 M**

v) Calculate Moment of inertia b = 3 m, d = 1 m

$I_{xx} = \frac{bd^3}{12} = \frac{3 \times 1^3}{12} = 0.25$ m^4 **1 M**

y = $\frac{d}{2} = \frac{1}{2} = 0.5$ m.

$\sigma_b = \frac{51.3}{0.25} x\ 0.5$ **1 M**

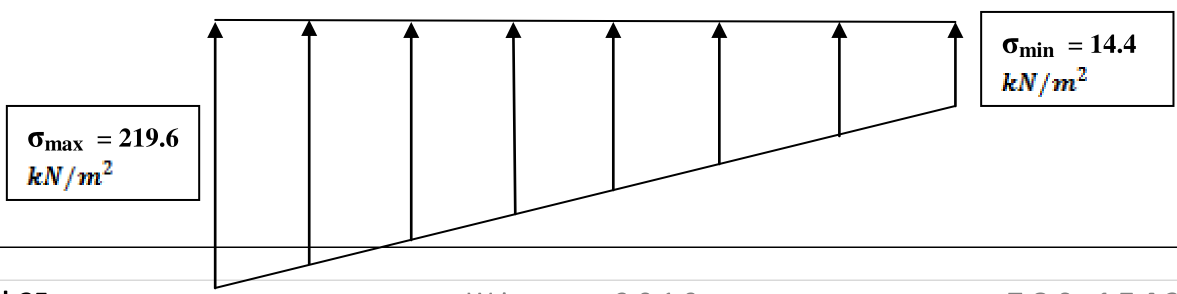
$\boldsymbol{\sigma_b = 102.6\ kN/m^2}$

vi) Calculate Minimum & Maximum stresses **1 M**

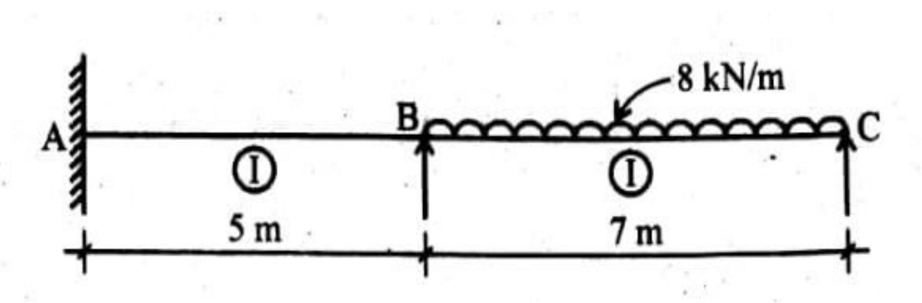
$\sigma_{max} = \sigma_D + \sigma_b$ = 117 + 102.6 = **219.6 kN/m^2**

$\sigma_{min} = \sigma_D - \sigma_b$ = 117 – 102.6 = **14.4 kN/m^2** **1 M**

vii) Draw stress distribution diagram **1 M**



b) **Draw B.M.D. for a continuous beam as shown in fig. Use moment distribution method**



Ans :

1) Calculate fixed end moments

For span AB

$M_{AB} = M_{BA} = 0$ as no load is acting on span AB

For span BC

$M_{BC} = -\frac{WL^2}{12} \; \frac{-8 \times 7^2}{12} = -32.67$ KN.m

$M_{CB} = +\frac{WL^2}{12} = \frac{8 \times 7^2}{12} = +32.67$ KN.m

2 M

2) Calculate Distribution factors

Joint	Member	S. F.	T. S.	D. F.
B	BA	$K_{BA} = \frac{4EI}{L} = \frac{4EI}{5} = 0.8$ EI	1.228 EI	$Df_{BA} = \frac{0.8\,EI}{1.228\,EI} = 0.651$
	BC	$K_{BC} = \frac{3EI}{L} = \frac{3EI}{7} = 0.428$ EI		$Df_{BC} = \frac{0.428\,EI}{1.228\,EI} = 0.349$

2 M

3) Moment distribution Table

A	B	B	C	
	0.651	0.349		D. F.
0	0	-32.67	+ 32.67	FEM
			- 32.67	Release C
		-16.335		Carryover
0	0	- 49	0	Total moments
	+ 31.902	+ 17.102	0	Moment Distribution
+ 15.951				Carryover moment
+ 15.951	+ 31.902	-31.902	0	Final moment.

2 M

1 M

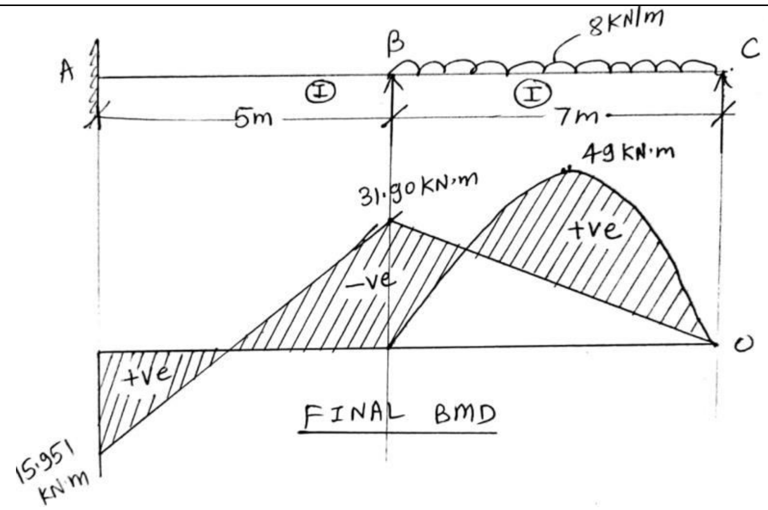
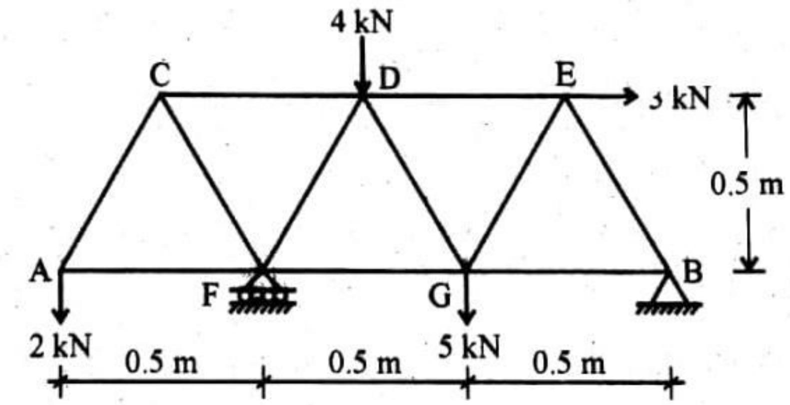
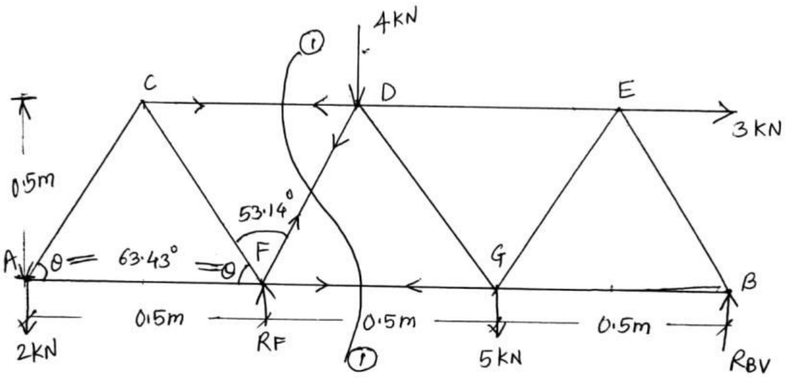
4) Calculate Moment for Simply Supported beam

Span AB = $m_{AB} = 0$

Span BC = $m_{BC} = \frac{WL^2}{8} = \frac{8 \times 7^2}{8} = 49$ kN.m

5) Draw final Bending moment diagram

01 M

		for sketch
c)	**A truss is loaded as shown in fig. Determine the nature and magnitude of forces in the member CD, FD and FG.** 	
Ans :		

1) Calculate Support Reactions

Taking moment at B

$R_F \times 1 + 3 \times 0.5 - 2 \times 1.5 - 4 \times \left[0.5 + \frac{0.5}{2}\right] - 5 \times 0.5$

$R_F = 7$ kN (↑)

Now $R_F + R_{BV} = 4 + 5 + 2$

$R_{BV} = 11 - 7 = 4$ kN. (↑)

Now $R_{BH} = 3$ kN. (←)

1 M sketch

2) Calculate Slope (θ)

angle CAF = angle AFC = θ

$\tan\theta = \left(\frac{0.5}{0.25}\right)$

$[\theta = 63.436]^0$ **1 M**

Now angle CFD = $180 - \theta - \theta$

$= 180 - 63.43 - 63.43$

Angle **CFD = 53.13^0** **1 M**

3) Take the section 1 – 1 passing through members CD, FD, FG
Consider F_{CD}, F_{FD} & F_{FG} as tensile and consider equilibrium of all forces to the left of section 1 -1

a) $\sum M_F = 0$ clockwise +ve and anticlockwise –ve moment

$-\ 2 \times 0.5 + F_{CD} \times 0.5 = 0$

$-\ 1 + F_{CD} \times 0.5 = 0$

$F_{CD} = -\frac{1}{0.5}$

F_{CD} = 2 kN (+ve sign indicate Tension)

b) $\sum M_D = 0$

$-\ 2 \times (0.5 + 0.25) + 7 \times \frac{0.5}{2} - F_{FG} \times 0.5 = 0$

$-\ 1.5 + 1.75 - 0.5\ F_{FG} = 0$ **2 M**

$F_{FG} = \frac{-0.25}{0.5}$

F_{FG} = 0.5 kN (Tensile)

$\sum F_x = 0$ gives

$+\ 3 + F_{CD} + F_{FG} + F_{FD} \cos\theta = 0$

$+\ 3 - 1 + 0.5 + F_{FD} \cos 63.43^0 = 0$

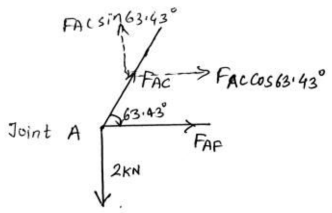
$F_{FD} = \frac{-2.5}{\cos 63.43}$

F_{FD} = - 5.59 kN (- ve sign indicate compression) **2 M**

OR

The problem can be solved using Method of joints

Consider joint A



1 M

OR

$\sum F_y = 0$

$-2 + F_{AC} \sin 63.43^0 = 0$

F_{AC} = 2.236 kN (Tension)

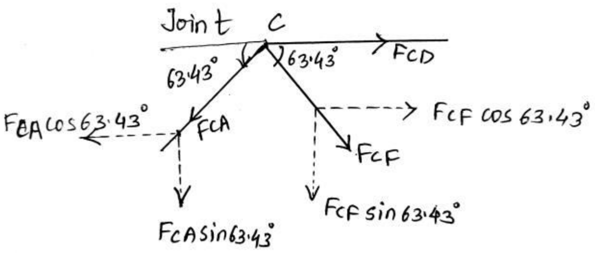
$\sum F_x = 0$

$F_{AF} + F_{AC} \cos 63.43^0 = 0$ **1 M**

$F_{AF} + 2.236 \cos 63.43^0 = 0$

F_{AF} = - 1kN (-ve sign indicate Compression)

Consider joint C



1 M

$\sum F_y = 0$ **1 M**

- $F_{CA} \sin 63.43^0 - F_{CF} \sin 63.43^0 = 0$

$-2.236 \times \sin 63.43^0 - F_{CF} \sin 63.43^0 = 0$

$\mathbf{F_{CF} = - 2.236\ kN}$

$\sum F_x = 0$

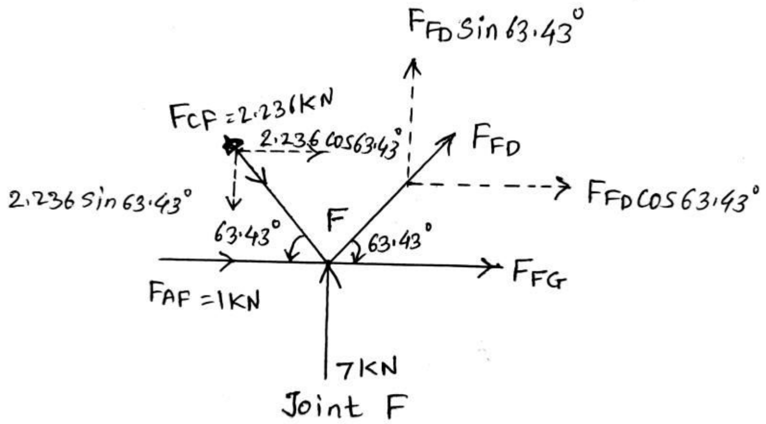
$F_{CD} + F_{CF} \cos$

$F_{CD} – 2.236 \cos 63.43^0 – 2.236 \cos 63.43^0 = 0$ **1 M**

$F_{CD} = 1 + 1$

$\mathbf{F_{CD} = 2\ kN}$ **(Tension)**

Consider Joint F



1 M

1 M

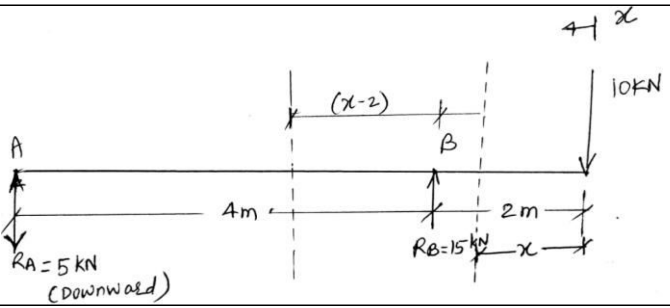
$\sum F_y = 0$

$7 + F_{FD} \sin 63.43^0 – 2.236 \sin 63.43^0 = 0$ **1 M**

$\mathbf{F_{FD} = - 5.59\ kN}$ (-ve sign indicates Compression)

Member	Force	Nature
F_{CD}	2 KN	Tension
F_{FG}	0.5 KN	Tension
F_{FD}	5.59 KN	Compression

1 M

Q. 6.	**Attempt any TWO of the following:**	**16 M**
a)	**A beam ABC, AB=4 m, BC = 2 m. Simply supported at A and B carrying point load at tree end 'C' 10 kN. Compute maximum deflection in a beam, if I = 8 X 10^7 mm4; E = 2 X 10^5 N/mm^2.**	
Ans :		**01 M for skech**

Calculate maximum deflection of beam if $I = 8 \times 10^7\ mm^4$
$E = 2 \times 10^5\ mm^2$
Using macullays method
Calculate support reactions
$\sum M_A = 0$ clockwise +ve and anticlockwise –ve moment
$-R_B \times 4 + 10 \times 6 = 0$
$R_B = 15$ kN

$\sum f_y = 0$
$R_A + R_B = 10$
$R_A + 15 = 10$
$R_A = -5$ kN (- ve sign indicate downward reaction) **1 M**

$EI\ d^2y/dx^2 = M$ --------Differential equation
Consider x from free end of overhang and Considering right side of section (Anti clock wise +ve and Clockwise -ve sign convension)

$EI\ d^2y/dx^2 = 10x \Big| + 15(x-2) \Big|$
X = 2 m x = 6 m

$EI\ d^2y/dx^2 = -10x \Big| + 15\frac{(x-2)^2}{2}$ --------- Slope equation

$EI\ y = -5\frac{x^3}{3} + C_1 x + C_2 \Big|\ 15\frac{(x-2)^3}{6}$ - --------Deflection equation

Calculate the constants of integration using boundry condition i) At x = 2 m y = 0 putting in Deflection equation

$EI(0) = -\frac{5}{3}(2)^3 + C_1 \times 2 + C_2 + \frac{15(2-2)^3}{6}$

$0 = -13.333 + 2C_1 + C_2$
$2C_1 + C_2 = +13.333$ -------------------- (I)

ii) At x = 6 m, y = 0 putting in Deflection equation
$EI(0) = -\frac{5}{3}(6)^3 + 6C_1 + C_2 + \frac{15(6-2)^3}{6}$

$= -360 + 6C_1 + C_2 + 160$

$6C_1 + C_2 = +200$ ------------------------(II)

Solving two simultaneous equation
$C_1 = 46.67$
$C_2 = -80$ **2 M**

Putting values if C_1 & C_2 & rewriting slope & deflection equation

$EI\frac{dy}{dx} = -5x^2 + 46.67 \Big| + 15\frac{(x-2)^2}{2}$ ------- Final Slope equation

$EI\,y = -5\,\frac{x^3}{3} + 46.67\,x - 80 \;\Big|\; + \frac{15(x-2)^3}{6}$ --------------- Final Deflection equation

1 M

Maximum deflection will be their where slope is zero.

Putting $\frac{dy}{dx} = 0$ in slope equation to get distance x where deflection level is maximum.

$0 = -5\,x^2 + 46.67 + 7.5\,(x-2)^2$

$0 = -5\,x^2 + 46.67 + 7.5\,(x^2 - 4x + 4)$

$0 = -5\,x^2 + 46.67 + 7.5\,x^2 - 30x + 30$

$2.5\,x^2 - 30x + 76.67 = 0$

Solving quadratic equation

x = 8.3 m, x = 3.69 m deflection is maximum putting in deflection equation.

1 M

Hence deflection is maximum at x = 3.69m

$EI\,y_{max} = -\frac{5}{3}(3.69)^3 + 46.67 \times (3.69) - 80 + \frac{15}{6}(3.69-2)^3$

$EI\,y_{max} = -83.739 + 172.212 - 80 + 12.067$

$y_{max} = \frac{20.54}{EI}$

Putting values of $E = 2 \times 10^5\ N/mm^2$, $I = 8 \times 10^7\ mm^4$

1 M

$EI = 16 \times 10^3\ kN.m^2$

$y_{max} = \frac{20.54}{16 \times 10^3}$

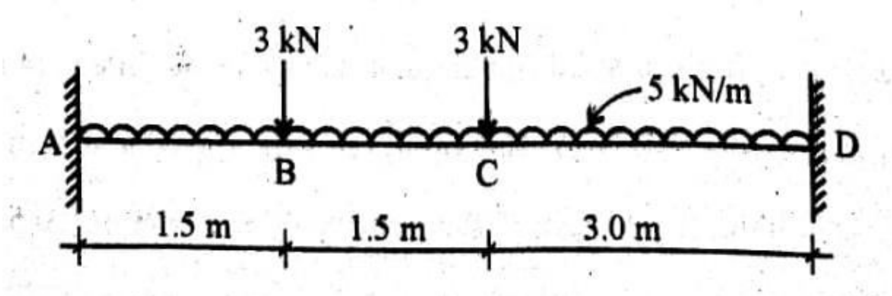
$y_{max} = 1.2837 \times 10^{-3}$ m

ymax = 1.2837 mm

(distance x can be considered from left support and problem can be solved if student solves problem by considering x from left support appropriate marks shall be given to the students accordingly.)

1 M

b) Draw SFD and BMD for beam show in fig.



Ans :

1) Calculate Reactions of a beam by considering beam as a simply supported beam

$\sum M_A = 0$

$-R_D \times 6 + 3 \times 1.5 + 3 \times 3 + 5 \times 6 \times \frac{6}{2} = 0$

$R_D = 17.25$ kN

$R_A + R_D = 3 + 3 + 5 \times 6$

$R_A = 36 - 17.25$

$R_A = 18.75$ kN

1 M

2) Calculate Simply Supported BM

$m_A = M_D = 0$

$m_B = 18.75 \times 1.5 - 5 \times 1.5 \times \frac{1.5}{2}$

$m_B = 28.125 - 5.625$

m_B = 22.5 kN.m (Sagging)

$m_C = 18.75 \times 3 - 3 \times 1.5 \times - 5 \times 3 \times \frac{3}{2}$

$m_C = 56.25 - 4.5 - 22.5$

m_C = 29.25 kN.m (Sagging)

1 M

3) Calculate Fixed end moments

$M_A = \frac{W1a1b1^2}{L^2} - \frac{W2a2b2^2}{L^2} - \frac{WL^2}{12}$

$M_A = \frac{-3 \times 1.5 \times (4.5)^2}{6^2} - \frac{3 \times 3 \times 3^2}{6^2} - \frac{5 \times 6^2}{12}$

$M_A = -2.531 - 2.25 - 15$

M_A = - 19.781 kN.m

1 M

$M_D = \frac{W1a1^2b1}{L^2} - \frac{W2a2^2b2}{L^2} - \frac{WL^2}{12}$

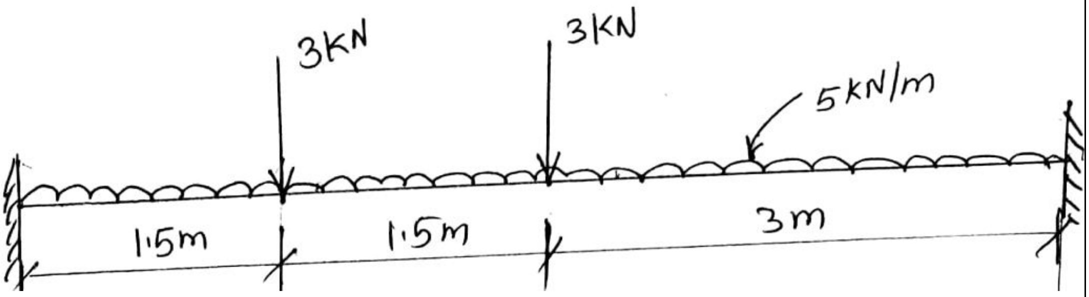
$M_D = \frac{-3 \times (1.5)^2 \times 4.5}{6^2} - \frac{3 \times 3^2 \times 3}{6^2} - \frac{5 \times 6^2}{12}$

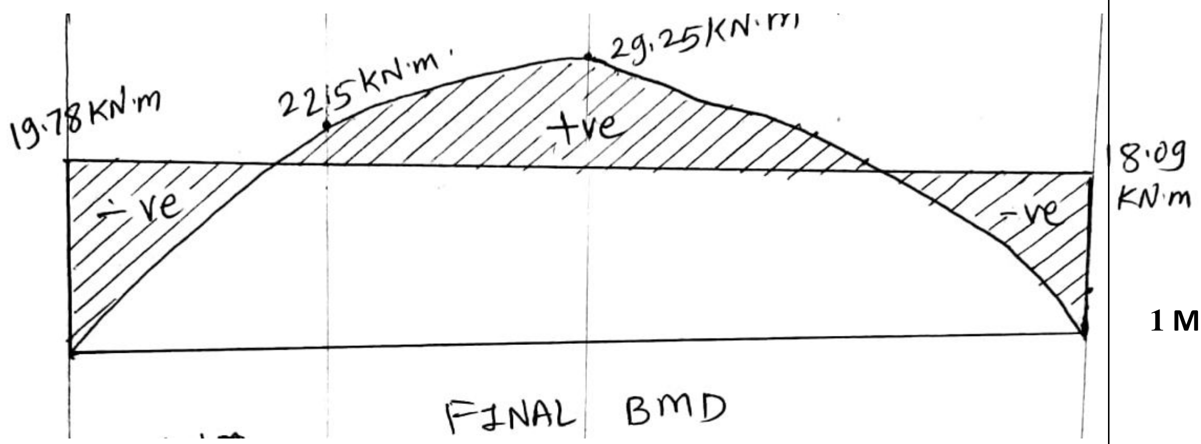
$M_D = -0.844 - 2.25 - 15$

M_D = - 18.094 kN.m

4) Draw final BMD by drawing S. S. BMD & Fixed diagram & Super imposing each other.

1 M





1 M

5) Calculate Support Reactions for fixed beam

$\sum M_A = 0$

$3 \times 1.5 + 3 \times 3 + 5 \times 6 \times \frac{6}{2} - 19.78 + 18.09 - R_D \times 6 = 0$

R_D = 16.960 kN

$R_A + R_D - 3 - 3 - 5 \times 6 = 0$

$R_A = 36 - 16.96$

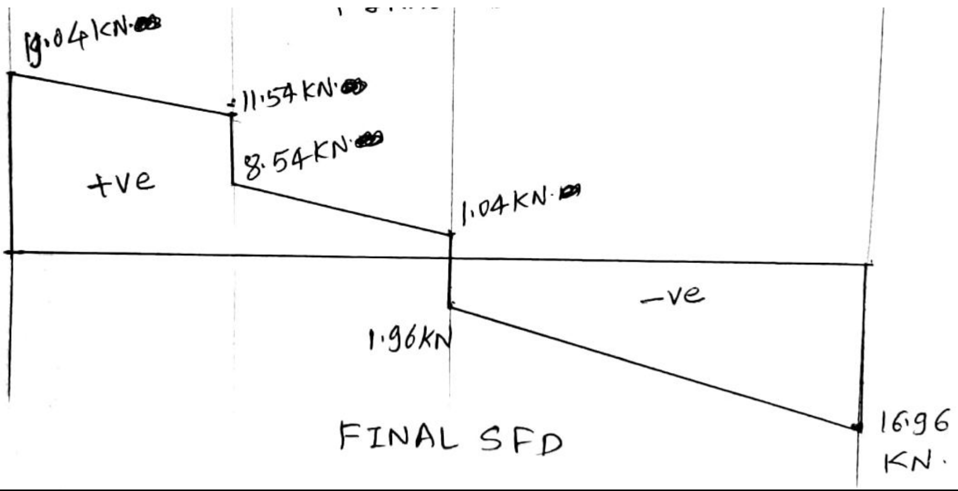
R_A = 19.04 kN

1 M

6) Shear Force Calculations

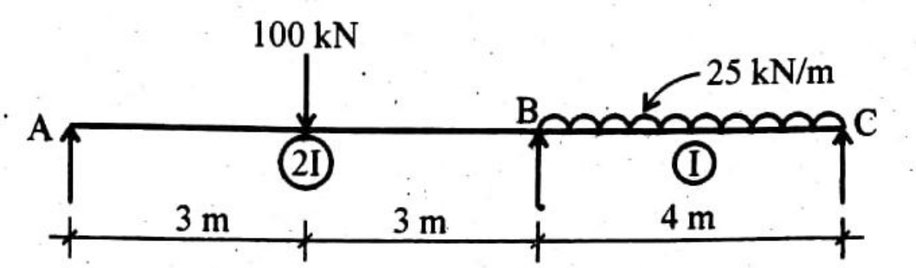
Shear Force at A just Left = 0
Just Right = + 19.04 kN
Shear Force at B just Left = + 19.04 – 3 x 1.5
= + 11.54 kN
Just Right = + 11.54 – 3
= + 8.54 kN
Shear Force at C just Left = + 19.04 – 3 – 5 x 3
= + 1.04 kN
Just Right = + 1.04 – 3
= - 1.96 kN
Shear Force at D just Left = + 19.04 – 3 –3 - 5 x 6
= - 16.96 kN
Just Right = - 16.96 + 16.96 = 0

1 M



1 M

c) **Draw SFD and BMD for beam show in fig. by claperon's thermo of three moments.**



Ans :

1) Using Claperon's Three moment thermo for span AB & BC

$$M_A\frac{L_1}{I_1}+2M_B\left(\frac{L_1}{I_1}+\frac{L_2}{I_2}\right)+M_C\frac{L_2}{I_2}+M_C\frac{L_2}{I_2}=-\left[\frac{6A_1X_1}{L_1I_1}+\frac{6A_2X_2}{L_2I_2}\right]$$

1 M

$M_A = M_c = 0$ --------- as it is simply supported

Putting values of L_1 & L_2, I_1 & I_2 in above equation

$$M_A\frac{6}{2I}+2\,M_B\frac{6}{2I}+\frac{4}{I}+M_C\frac{4}{I}=-\frac{6A_1X_1}{2Ix6}+\frac{6A_2X_2}{Ix4}$$ -------------------------------- (I)

2) Calculate Simply Supported BM & Draw S. S. BMD

$m_{AB} = \frac{PL}{4}$

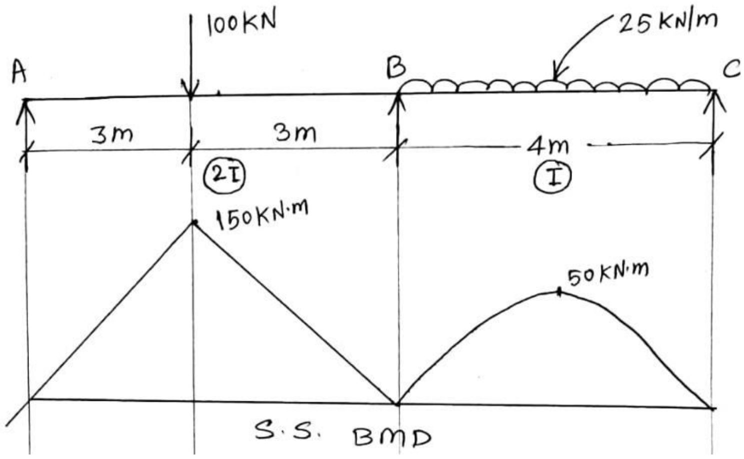
$m_{AB} = \frac{100\,x\,6}{4}$

m_{AB} = 150 kN.m

$m_{BC} = \frac{wL^2}{8}$

$m_{BC} = \frac{25 \times 4^2}{8}$ **1 M**

$m_{BC} = 50$ kN.m



1 M

$A_1 = \frac{1}{2} \times 6 \times 150$ $A_1 = 450$ kN.m_2

$A_2 = \frac{2}{3} \times 4 \times 50$ $A_2 = 133.33$ kN.m_2

$X_1 = \frac{6}{2}$ $X_1 = 3$ m

$X_2 = \frac{4}{2}$ $X_2 = 2$ m

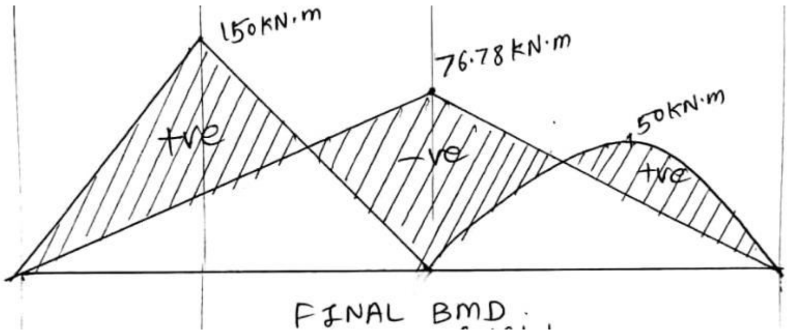
Putting all values in equation (I)

$0 + 0 + 2\,M_B\,[\,7\,] = -\left[\frac{6 \times 450 \times 3}{12} + \frac{6 \times 133.33 \times 2}{4}\right]$ **1 M**

$14\,M_B = -\,[\,675 + 400\,]$

$\mathbf{M_B = -76.785}$ **kN.m**

3) Draw final BMD



1 M

4) Calculate Support Reactions

Consider Span AB

Taking moment at B

$\sum M_A = 0$

$R_A \times 6 - 100 \times 3 + 76.78 = 0$

$\mathbf{R_A = 37.203}$ **kN.m**

Consider Span BC

Taking moment at B

$\sum M_B = 0$ **1 M**

$- R_C \times 4 + 25 \times 4 \times \frac{4}{2} - 76.78 = 0$

$\mathbf{R_C = 30.805}$ **kN.m**

$\sum M_B = 0$

$R_A + R_B + R_C - 100 - 25 \times 4 = 0$

$37.203 + R_B + 30.805 = 200$

R_B = 131.99 kN.m

5) **Shear Force Calculations**

Shear Force at A just Left = 0

Just Right = 37.203 kN

Shear Force at D just Left = + 37.203

Just Right = + 37.203 – 100

= - 62.797 kN

Shear Force at B just Left = + 37.203 – 100

= - 62.797 kN

Just Right = - 62.797 + 131.99

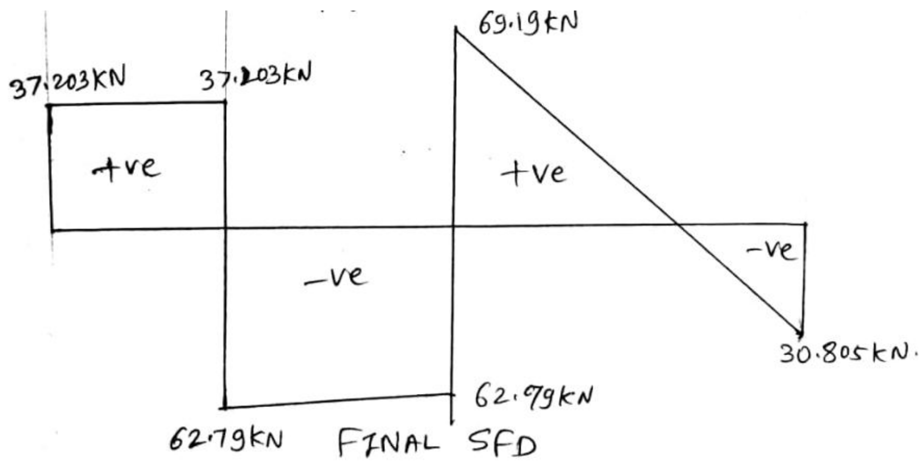
= 69.193 kN

Shear Force at C just Left = + 37.203 – 100 + 131.99 - 25 x 4

= - 30.805 kN

Just Right = - 30.805 + 30.805 = 0

1 M



1 M