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.

5) 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.

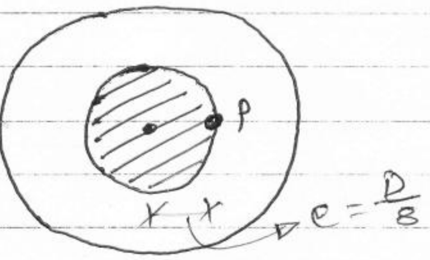
7) For programming language papers, credit may be given to any other programme based on equivalent concept.

Important notes to examiner

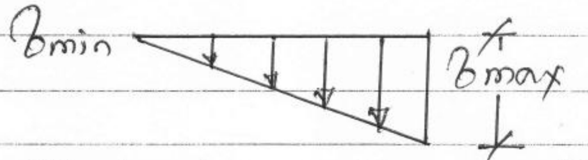
Q1A attempt any six of following

(a) Define "core of section" sketch resultant stress diagram if load acts on the boundary of core of section.

→ Defination:- The Centrally located portion of a section within the load line falls so as to produce only compressive stress is called as core of section. 1m



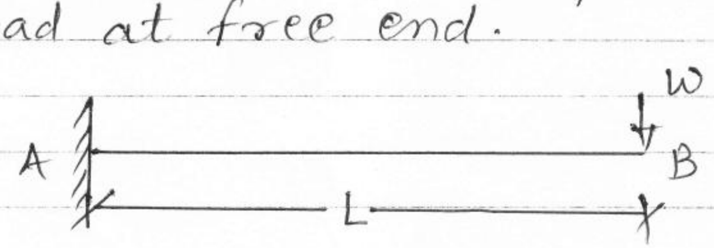
1m



Resultant stress diagram if load acts on the boundary of core of section.

(b) Write values of slope & deflection at free end of Cantilever that Carries point load at free end.

→ Cantilever beam of span 'L' with point load at free end.



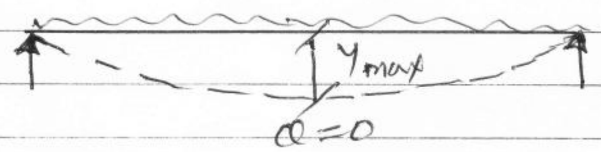
Slope at free end

$\theta_B = \frac{WL^2}{2EI}$ radians 1m

Deflection at free end.

$$y_B = \frac{-WL^3}{3EI}$$ (-ve sign indicates downward deflection) 1M

(c) State boundary conditions for a simply Supported beam using deflect shape.



At support max^m slope developed

i.e. $\frac{dy}{dx} \neq 0$ $(\theta \neq 0)$ 1m

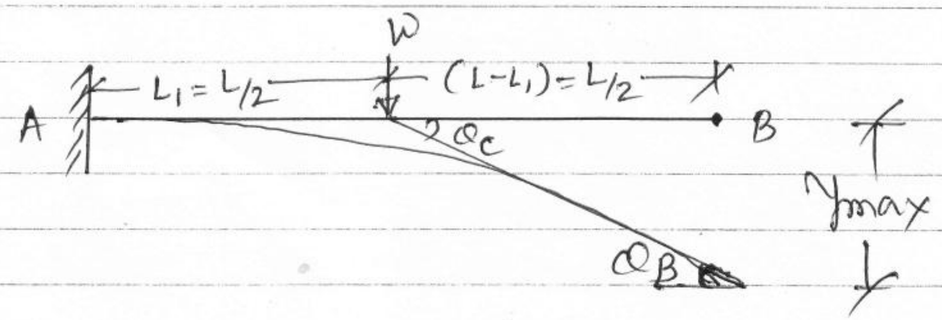
& Deflection is zero i.e. $y=0$

At the point of maximum Deflection

$\theta = \frac{dy}{dx} = 0$ & $y = y_{max}$ 1m

(d) A cantilever of span L carries point load 'W' at L/2 from fixed end. State value of slope & free end.

→ Given:-



$L_1 = L/2$ & $(L - L_1) = L/2$

from the standard case,

θ_B = slope at free end $= \frac{WL_1^2}{2EI}$ rad. 1M

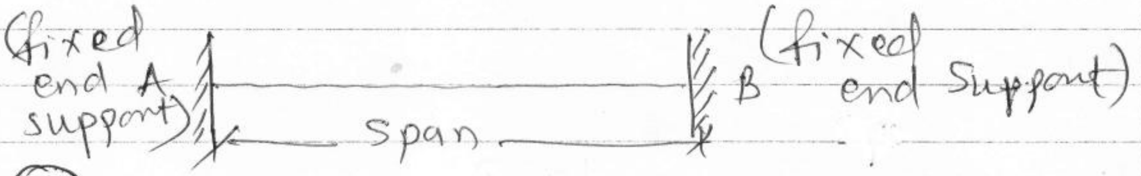
$$= \frac{W\left(\frac{L}{2}\right)^2}{2EI}$$

$$\therefore \boxed{\theta_B = \frac{WL^2}{8EI}}$$ 1M

(e) Define "fixing" & fixed beam.

<u>fixing</u> :- A support is restrained against rotation & vertical movement is know as fixing 1M

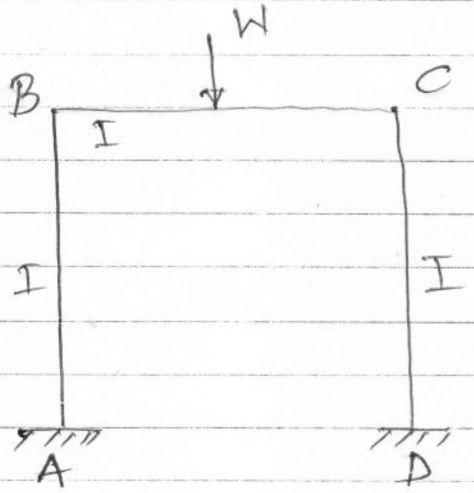
<u>fixed beam</u> :-
A beam whose end is firmly built in the support like wall, pillar or any other structure, then such beam is called fixed beam. 1M



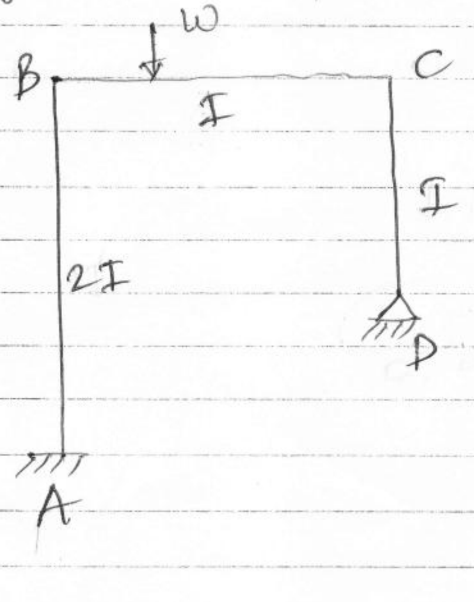
OR A support at which the end slopes are zero is known as fixed beam 1M

(F) Giving diagram, state two types of portal frames,

(1) Symmetrical portal frames. 1m



(2) Unsymmetrical portal frame. 1m



(g) At a continuity, adjoining span have their distribution factors as 0·43 & 0·57. What is the meaning of these values?

→ The values given in question are the distribution factors using these factors the unbalance moment is distributed among the two span by using this distribution factor. 2M

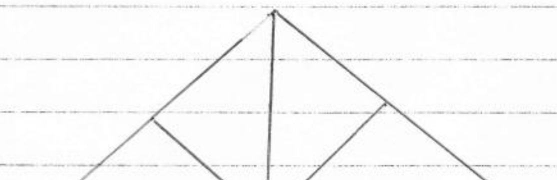
Distributed moment = D·F × unbalance moment.

(h) Define "perfect frame" & draw sketches of any two perfect frames.

→ A frame in which the condition $n = 2j - 3$ is satisfied such frame is called perfect frame. 1M

Where,
n = number of members
j = Number of joints

1M

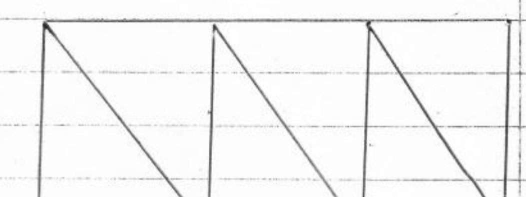


$n = 9$, $j = 6$

$n = 2j - 3$

$9 = 2 \times 6 - 3$

$9 = 9$ (Satisfied)



$n = 13$, $j = 8$

$n = 2j - 3$

$13 = 2 \times 8 - 3$

$13 = 13$ (satisfied)

(OR) A perfect frame is that which is made up of members just sufficient to keep in equilibrium, when loaded, without any change in the shape

Q1(B) Attempt any two of the following.

(a) A pillar is square in section & has side 1m. Values of axial & bending stress are 300 KN/m² & 287 KN/m² respectively. Determine resultant stresses. Draw resultant stress distribution diagram. also state whether the load is within the core or not.

→ Given:-

$$\sigma_b = 287\ KN/m^2$$

$$\sigma_d = 300\ KN/m^2$$

To determine resultant stresses.

$$\therefore \sigma_{max} = \sigma_d + \sigma_b$$ 1M

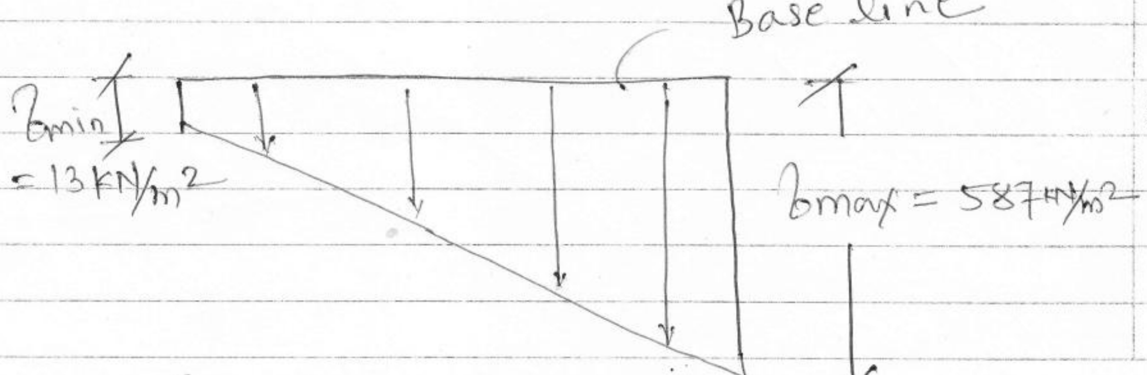
$$= 300 + 287$$

$$= 587\ KN/m^2$$

$$\sigma_{min} = \sigma_d - \sigma_b$$ 1M

$$= 300 - 287$$

$$= 13\ KN/m^2$$



1M

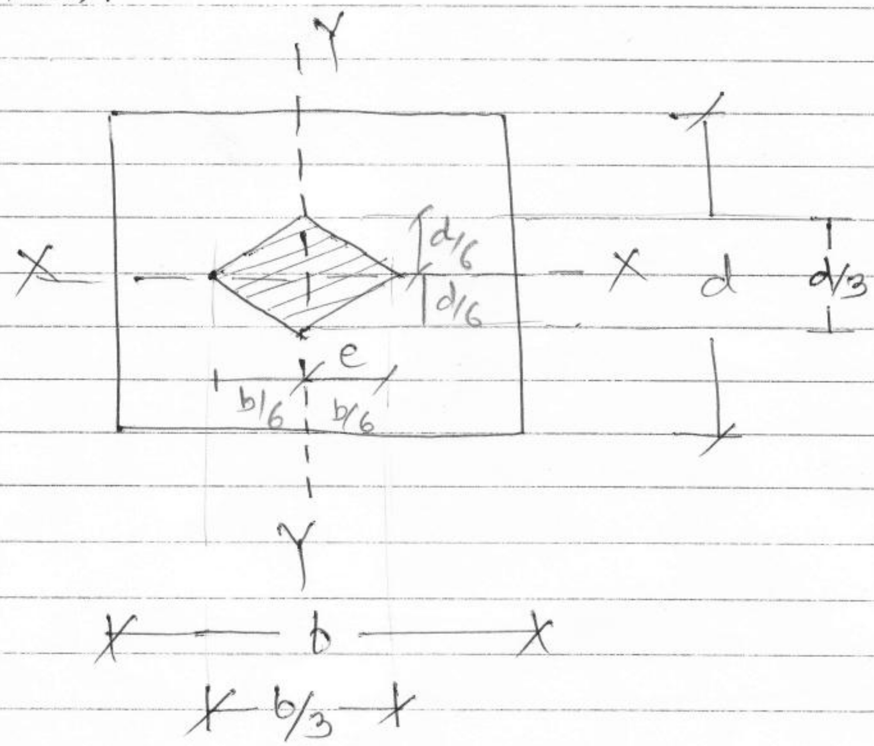
Resultant stress distribution diagram.

It state that load is within core because from resultant stresses it is seen that it totally Compressive in nature, so that load is within core of section. 1m

(b) State middle third rule. Draw sketch of core of rectangular & Solid Circular Section. State dimensions of core also

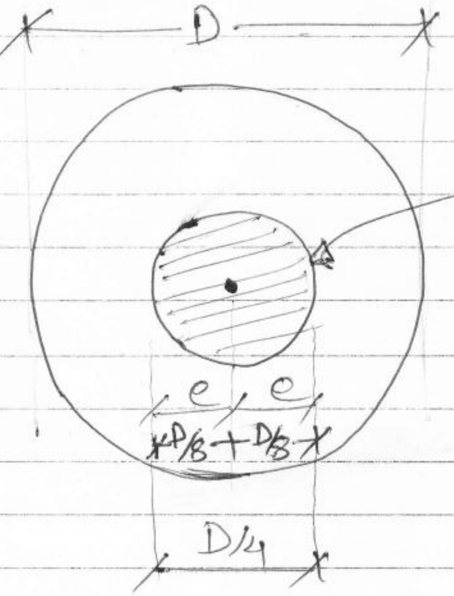
→ Middle third Rule:-

"middle third Rule is used to find the limit of eccentricity for a section so as to produce only compressive stress at the base but no tension at base of section. 2M



1m

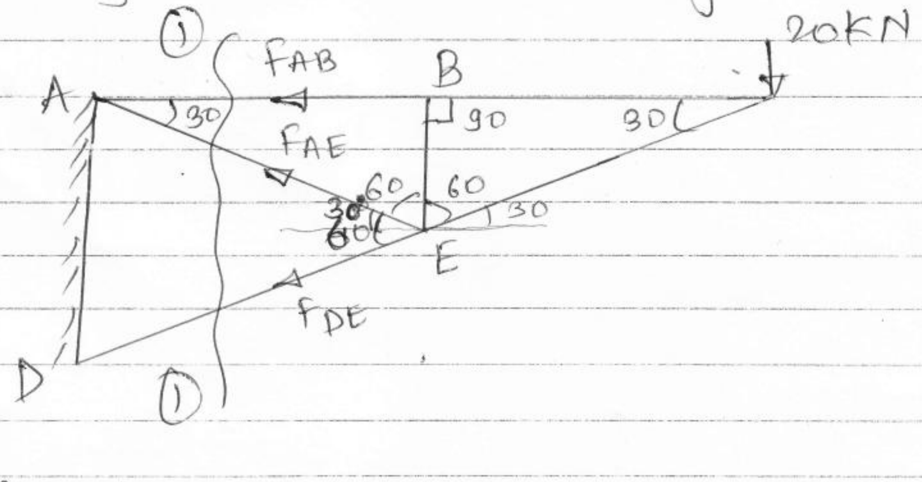
fig:- Core of section for rectangular section.



1m

fig:- core of section for solid circular section.

(C) Using method of section only, determine nature & magnitude of axial forces in member AB & AE only: for the ~~fig~~ truss shown in fig.



Consider section (1) - (1) which cuts AB, AE & DE

Consider the right part of Section (1) - (1). Assume FAB, FAE & FDE to be tensile. Take tensile +ve & Compressive -ve.

Consider Δ ABE

$\therefore \tan 30 = \frac{BE}{AB} = \frac{BE}{3.464}$

$BE = 1.999 \approx 2m$ 1m

Consider the right part of Section (1) - (1) in equilibrium. Taking moment at E we get

(+ ↻ ↺ -) ; $\Sigma M_E = -F_{AB} \times 2 + 20 \times 3.464$

$\boxed{F_{AB} = 34.64\ KN}$ 1m

To find FAE & FDE

$\therefore \Sigma fx = 0$

$-34.64 - F_{AE}\cos 30 - F_{DE}\cos 30 = 0$

$F_{AE}\cos 30 + F_{DE}\cos 30 = -34.64$ —— (1)

$\therefore \Sigma f_y = 0$

$-20 + F_{AE} \sin 30 - F_{DE} \sin 30 = 0$

$F_{AE} \sin 30 - F_{DE} \sin 30 = 20$ —— (2)

eqn (1) & (2) Solving Simultenously we get.

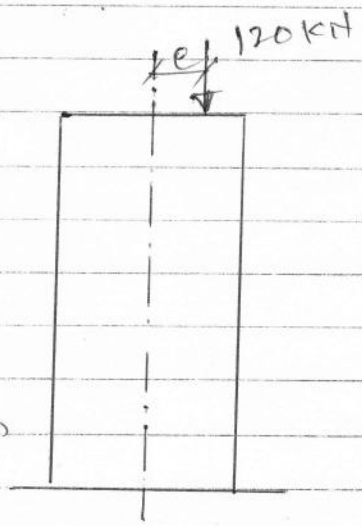
$F_{AE} = 0$ 2m

$F_{DE} = -40$ (Compressive).

Q 2
(a) A rectangular column has size 0.6m × 0.4m. A load of 120 KN acts at an eccentricity of 0.14 m on the axis bisecting shorter side. Determine resultant stresses developed at base. Draw stress distribution diagram also.

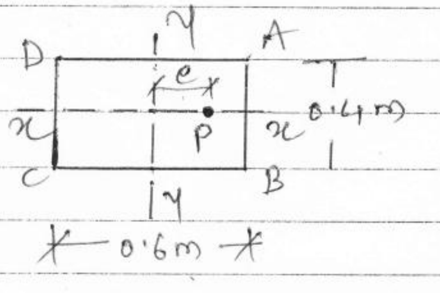
→ given:-

$b = 0.6 m$
$d = 0.4 m$
$P = 120 KN$
$e = 0.14 m$

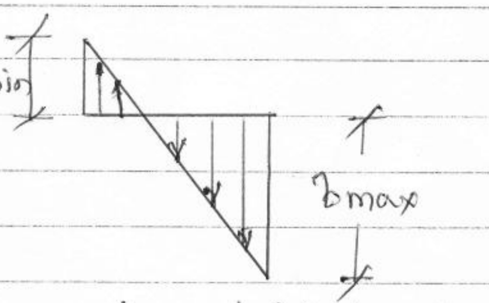


To find maximum & minimum intensities of stress i.e. σ_{max} & σ_{min}

A = Area of section
$= b \times d$
$= 0.6 \times 0.4$
$= 0.24\ m^2$



σ_0 = Direct stress
$= \frac{P}{A} = \frac{120}{0.24}$
$\sigma_0 = 500\ KN/m^2$



stress distribution dia.

1M

(sketch) 1M

σ_b = Bending stress
$= \frac{M}{Z_{yy}} = \frac{P.e}{\frac{db^2}{6}}$

$$= \frac{120 \times 0.14 \times 6}{0.4 \times 0.6^2}$$

$$\sigma_b = 700 \text{ KN/m}^2 \qquad 1M$$

$$\therefore \sigma_{max} = \sigma_0 + \sigma_b$$

$$= 500 + 700$$

$$\therefore \sigma_{max} = 1200 \text{ KN/m}^2 \qquad \tfrac{1}{2}M$$

(Compressive on face AB)

$$\therefore \sigma_{min} = \sigma_0 - \sigma_b \qquad \tfrac{1}{2}M$$

$$= 500 - 700$$

$$= -200 \text{ KN/m}^2$$

(tensile on face DC).

(b) A hallow circular column having external diameter 2m, carries load of 460 KN at an eccentricity of 0.8m Draw resultant stress diagram (for this column Area = 2.356 m^2 & $I_{xx} = I_{yy} = 0.7363 \ m^4$

→ Given :-

$D = 2\ m$
$P = 460\ KN$
$e = 0.8\ m.$
$A = 2.356\ m^2$
$I_{xx} = I_{yy} = 0.7363\ m^4$

To find resultan stress i.e. σ_{max} & σ_{min}.

Direct stress $= \frac{P}{A} = \frac{460}{2.356}$ 1m

$$\sigma_0 = 195.25\ KN/m^2$$

Bending stress $(\sigma_b) = \frac{M}{Z} = \frac{P \cdot e}{\frac{I}{y_{max}}}$

$$= \frac{460 \times 0.8}{\frac{0.7363}{1}}$$

$$\sigma_b = 499.80\ KN/m^2$$ 1m

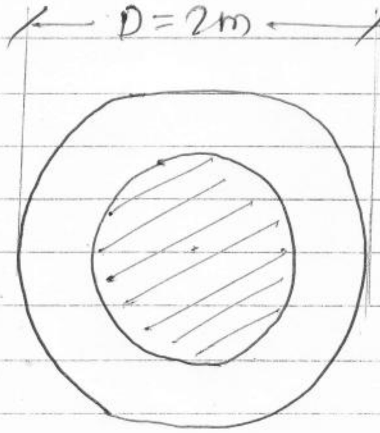
$\sigma_{max} = \sigma_0 + \sigma_b$

$= 195.25 + 499.80$

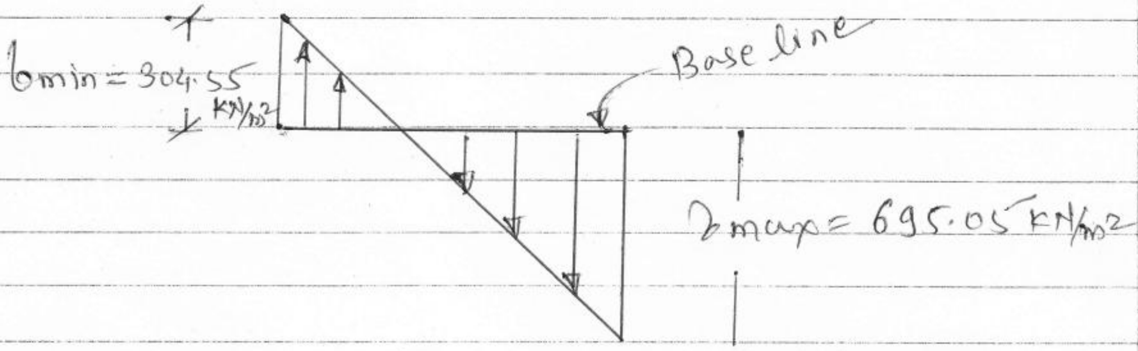
$\sigma_{max} = 695.05\ KN/m^2$ (Compressive) $\frac{1}{2}$ m

$\sigma_{min} = 195.25 - 499.80$

$= -304.55$ (tensile) $\frac{1}{2}$M.



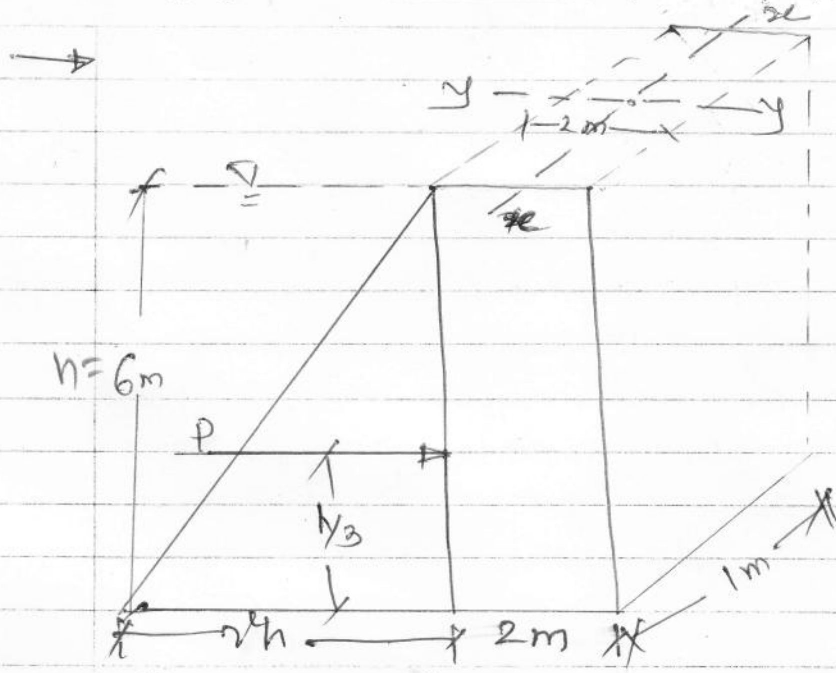
cross section.



1m

Resultant stress distribution dia.

(C) A retaining wall 6m high has uniform thickness 2m. It retain water upto top. Determine total water pressure & net stresses at base. Draw stress diagram. Use unit wt. of water 10 KN/m³ & Unit wt. of wall motored is 18 KN/m³.



Given :- $\gamma = 10\ kN/m^3$

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

Assume unit length for calculation.

① Total hydrostatic pressure — 1m

$= \frac{1}{2}\gamma h^2$

$= \frac{1}{2} \times 10 \times 6^2$

$P = 180\ kN$

② Direct stress $\sigma_0 = \frac{\text{Total wt}}{\text{c/s Area}}$ — 1m

$= \frac{\cancel{A} \times h \times \rho}{\cancel{A}}$

$= h \times \rho$

$= 6 \times 18$

$\sigma_0 = 108\ kN/m^2$

$\boxed{OR}$

Total wt $= \rho \times V$

$= \rho \times A \times h$

$= 18 \times 2 \times 1 \times 6$

$= 216\ kN$

$\sigma_0 = \frac{\text{Total wt}}{\text{c/s Area}}$

$= \frac{216}{2 \times 1} = 108$

(3) Bending stress $\sigma_b = \frac{M}{Z}$ 1m

$$= \frac{P \times h/3}{Z}$$

$$Z = \frac{bd^2}{6} = \frac{1 \times 2^2}{6} = 0.667\ m^3$$

$$\sigma_b = \frac{180 \times 6/3}{0.667} = 539.73\ KN/m^2$$

(4) Resultant stresses at the base

$$\sigma_{max} = \sigma_0 + \sigma_b = 108 + 539.73$$

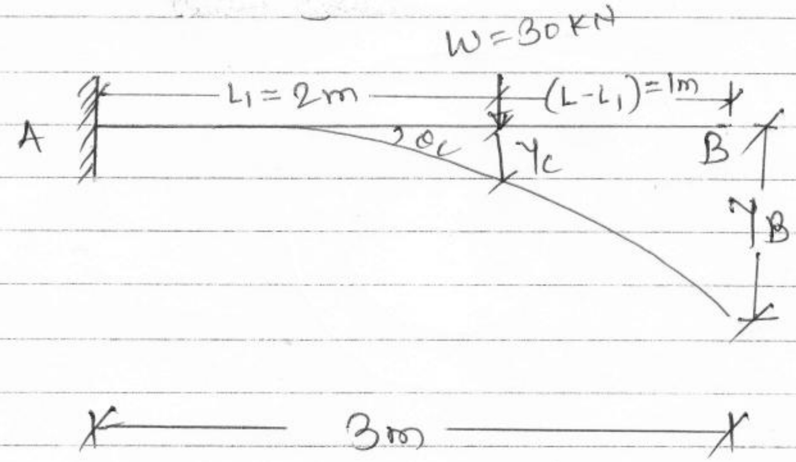
$$\boxed{\sigma_{max} = 647.73\ KN/m^2}\ (Comp)$$ ½ M

$$\sigma_{min} = \sigma_0 - \sigma_b = 108 - 539.73$$

$$= -431.73\ (tensile)$$ ½ M

(d) A Cantilever of span 3m carries point load of 30 kN at 2m from fixed end. Determine slope & deflection at free end. Take $EI = 11000\ kN\cdot m^2$.

→ Given:-



$L = 3m$

$L_1 = 2m$

$W = 30\ kN$

$EI = 11000\ kN\cdot m^2$

To find y_B & θ_B

$$y_B = y_{max} = \frac{WL_1^3}{3EI} + \frac{WL_1^2}{2EI}(L - L_1) \qquad 1M$$

$$= \frac{30\times 2^3}{3\times 11000} + \frac{30\times 2^2}{2\times 11000}\times 1$$

$$= 7.27\times 10^{-3} + 5.45\times 10^{-3}$$

$$\boxed{y_B = 0.0127\ mm} \qquad 1m$$

$$\theta_B = \theta_{max} = \frac{WL_1^2}{2EI}$$ 1m

$$= \frac{30 \times 2^2}{2 \times 11000}$$

$$= 5.45 \times 10^{-3} \text{ rad.}$$

$$\boxed{\theta_B = 0.00545 \text{ rad}}$$ 1m

(e) Determine intensity of uniformly distributed load 'w' for a simply supported beam of span 6m, if maximum deflection allowed is 12.6 mm. also state value of slope at mid span. Take $E = 2 \times 10^5$ MPa, $I = 47.5 \times 10^6$ mm^4.

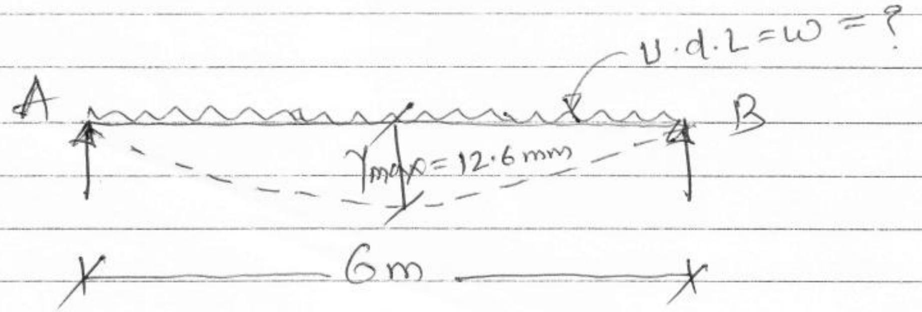
→ Given

$$E = 2 \times 10^5 \text{ MPa} = 2 \times 10^5 \text{ N/mm}^2$$

$$I = 47.5 \times 10^6 \text{ mm}^4$$

$$L = 6 \text{ m} = 6000 \text{ mm}$$

$$y_{max} = 12.6 \text{ mm}$$



To find the value of U.d.L (w) :- for simply supported beam loaded entirely by u.d.l of 'w' KN/m, the maximum deflection is given by the following expression.

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

Since -ve sign indicate downword deflection, neglect -ve sign for UdL,

$$Y_{max} = \frac{5 w L^4}{384 EI}$$ 1M

$$12.6 = \frac{5 \times w \times 6000^4}{384 \times 2 \times 10^5 \times 47.5 \times 10^6}$$

$$w = \frac{12.6 \times 384 \times 2 \times 10^5 \times 47.5 \times 10^6}{5 \times 6000^4}$$

$$\boxed{w = 7.093 \ N/mm}$$ 2M

It states that when simply Supported beam is Subjected to udl over entire span then, deflection is maximum at mid span, so slope is zero at mid span. 1M

$\boxed{\theta \text{ at center} = 0}$

(F) Define "Continuity" & state its effect on span moment & span deflection, as compared to simply supported span. State nature of support moment-

→ Continuity :- When a beam supported over more than two support in such cases the beam are continous over the support which develops additional moment over the support & slopes are same either side of support such effect is known as continuity. 1M

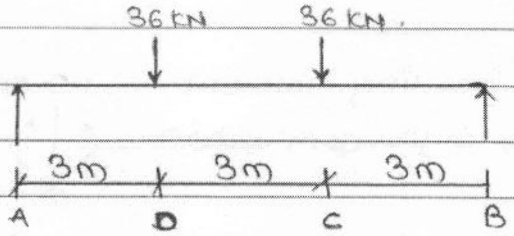
	Effect on moment :- Due to Continuity the Span moment i.e. Sagging moment are reduced & additional Support moment i.e. Hogging moment developed over the Support. The Sagging moment is less as Compared to Simply Supported beam.	1M
	Effect on span deflection:- Span deflection is reduced due to the effect of continuity as Compared to the Simply Supported beam.	1M
	Nature of Support Moment:- for the usual downword loading on the Contineous beam, hogging moment Causing Convexity upword occur at the intermediate Support & Sagging moments Causing Convexity downword occur at mid span.	1M

Q.3

a] A simply supported Beam of span 9m carries two point loads of equal magnitude 36 kN at 3m from both ends. Calculate values of integration constant and write Macaulay's slope and deflection equation.

Sol^n:-

From the given data. a simply supported beam is drawn as shown in fig.



Step-1 Given:-

$W_1 = 36\,kN$ $W_2 = 36\,kN$, $L = 9m$.

Step-2 To Find Support Reaction RA & RB.

$\overset{+}{\uparrow}\ \overset{-}{\downarrow}$ $\Sigma F_Y = R_A - 36 - 36 + R_B$.

$\therefore R_A + R_B = 72$ —— (i).

$\Sigma M_A = 36\times3 + 36\times6 - 9\times R_B$.

$= 108 + 216 - 9R_B$.

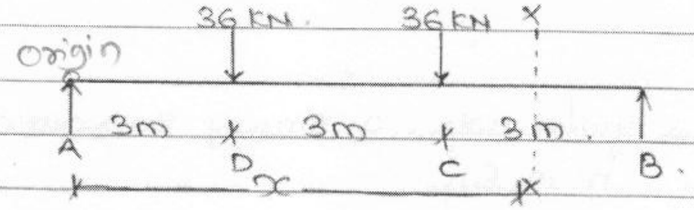
$$\boxed{R_B = 36\,kN}$$

Substituting the value of RB in eqn (i) we get.

$\boxed{R_A = 36\,kN}$ OR Due to symmetrical loading $R_A = R_B = 36\,kN$

Step-3 To construct slope and deflection equation by Macaulay's Method.

Consider Section X-X at a distance x from A in portion BC as shown in fig. below.



∴ The general Bending moment equation at a distance x from A by sign convention

$$EI\frac{d^2y}{dx^2} = M_x = \left|\,36x\,\right| - \left|\,36(x-3)\,\right| - \left|\,36(x-6)\,\right| \quad \text{— eqn (1)}$$ ½M

Integrating eqn (1) w.r.t. x.

$$EI\frac{dy}{dx} = \left|\,\frac{36x^2}{2} + C_1\,\right| - \left|\,\frac{36(x-3)^2}{2}\,\right| - \left|\,\frac{36(x-6)^2}{2}\,\right| \quad \text{— (2)}$$ ½M

Integrating eq (2) w.r.t. x.

$$EIy = \left|\,\frac{36}{2}\frac{(x)^3}{3} + C_1x + C_2\,\right| - \left|\,\frac{36}{2}\frac{(x-3)^3}{3}\,\right| - \left|\,\frac{36}{2}\frac{(x-6)^3}{3}\,\right| \quad \text{— (3)}$$ ½M

step-4 Applying boundary condition to find values of constants C_1 & C_2.

At simple support A $x = 0$, deflection $Y = 0$ ½M
& At simple support B $x = 9$, deflection $Y = 0$.

by condition 1 $x = 0$, $Y = 0$.
Substituting these values in equation (iii) upto first bracket. we get.

$0 = 0 + 0 + C_2$.
$C_2 = 0$

by condition 2. $x = 9$, $Y = 0$ ½M
Substituting these values in equation 3 upto ~~first~~ all bracket.

$$0 = \left| 36 \times \frac{(9)^3}{6} + 9C_1 + 0 \right| - \left| 36\frac{(9-3)^3}{6} \right| - \left| 36\frac{(9-6)^3}{6} \right|$$ 1M

$0 = 4374 + 9C_1 - 1296 - 162$. $\therefore 9C_1 = -2916$

$\therefore C_1 = -\frac{2916}{9}$

$\boxed{C_1 = -324}$ ½M

Hence $C_1 = -324$ & $C_2 = 0$

Q.3

b) As from Q.3 a constant, slope equation, deflection equation are as follows.

$C_1 = -360 \qquad C_2 = 0.$

$$EI\frac{dy}{dx} = \left| \frac{36x^2}{2} + C_1 \right| - \left| \frac{36(x-3)^2}{2} \right| - \left| \frac{36(x-6)^2}{2} \right| \quad \text{— (2)}$$

1M.

$$EIy = \left| \frac{36x^3}{6} + C_1x + C_2 \right| - \left| \frac{36(x-3)^3}{6} \right| - \left| \frac{36(x-6)^3}{6} \right| \quad \text{— (3)}$$

$\therefore$ A D = x = 3m.

$$\therefore EIy_D = \left| \frac{36 \times (3)^3}{6} - 324 \times 3 \right|$$

$$EIy_D = 162 - 972$$

$$\boxed{y_D = \frac{-810}{EI} \text{ mm}}$$

-ve sign indicates downward deflection. 1M

At C, x = 6m.

$$EIy_C = \left| \frac{36}{2} \times \frac{6^3}{3} + (-324 \times 6) \right| - \left| \frac{36}{2} \times \frac{(6-3)^3}{3} \right|$$

$$= 1296 - 1944 - 162$$

$$\boxed{= \frac{-810}{EI} \text{ mm}}$$

-ve sign indicates downward deflection. 1M

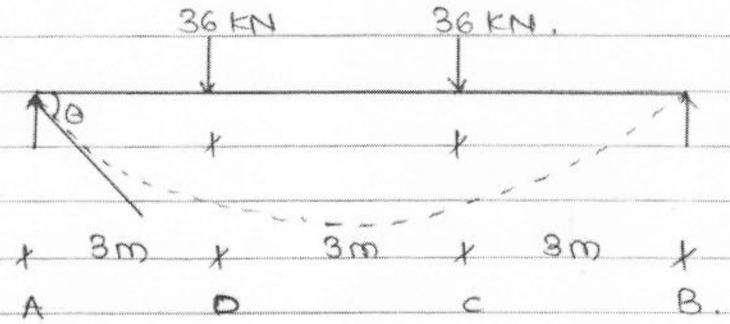
To calculate slope at support A.

Substitute the values of C_1 & $x = 0$ in slope eqn ② upto First bracket since at A $x = 0$ we get.

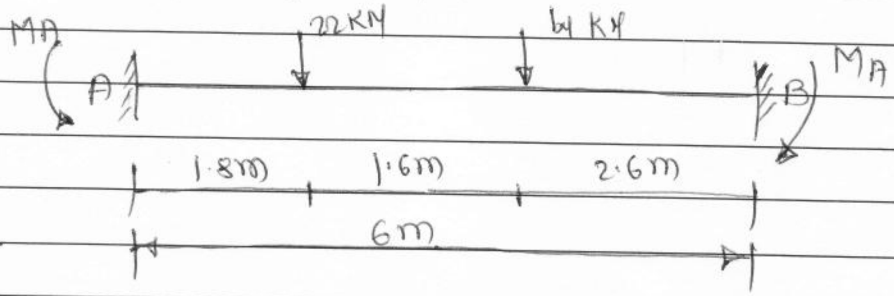
$$EI\left(\frac{dy}{dx}\right)_A = \left| \frac{36(0)^2}{2} - 324 \right|$$

1M.

$$\boxed{\left(\frac{dy}{dx}\right)_A = \frac{-324}{EI} \text{ rad.}}$$



Q-3 (c) from the given data



$M_A = -\frac{W_1 a_1 b_1^2}{L_1^2} - \frac{W_2 a_2 b_2^2}{L^2}$ $a_1 = 1.8m$ $b_1 = 4.2m$ ½M
$a_2 = 3.4m$ $b_2 = 2.6m$
$W_1 = 22 KN$, $W_2 = W$

$= \frac{-22 \times 1.8 \times (4.2)^2}{6^2} - \frac{W \times (3.4) \times (2.6)^2}{6^2}$

$= -19.404 - 0.641W$ ——— (1) ½M

$M_B = \frac{-W_1 b_1 a_1^2}{L^2} - \frac{W_2 b_2 a_2^2}{L^2}$ ½M

$= \frac{-22 \times 4.2 \times 1.8^2}{36} - \frac{W \times 2.6 \times 3.4^2}{36}$

$= -8.32 - 0.835 W$ ——— (2) ½M

as moments at supports are

$\therefore M_A = M_B$

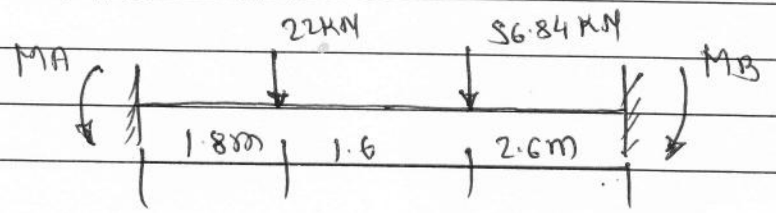
$-19.404 - 0.641W = -8.32 - 0.835 W$

$-19.404 + 8.32 = -0.835W + 0.64W$

$11.084 = 0.195W$

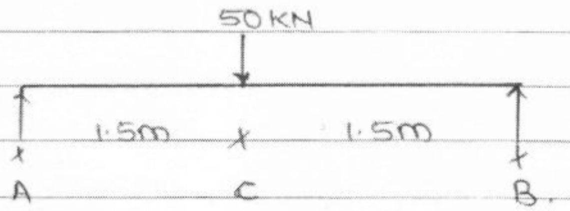
$W = 56.84 KN$ 02M

Hence beam is



Q.3

d] As per given data.



Step-1 :- Support Reaction

$V_A = V_B = W/2 = 50/2 = 25\,KN.$

Step-2 :- Free B.M.

$M_A = M_B = 0,$

$M_C = \frac{Wl}{4} = \frac{50 \times 3}{4} = 37.5\,KN.m$ (sagging). 1M

Step-3 :- Fixed end Moment by first principal :-

M_A = Fixed end Moment at A.

M_B = Fixed end Moment at B.

a = area of M-diagram (Free body diagram of Sagging B.M)
$= \frac{1}{2} \times$ base $\times$ height.

$= \frac{1}{2} \times 3 \times 37.5$

$\boxed{= 56.25}$ ½M

Area of fixed End BMD

$a' = -M_A \times L$ (as $M_A = M_B$) ½M

equating both

$a = a'$

$56.25 = -M_A \times 3$

$M_A = \frac{-56.25}{3}$

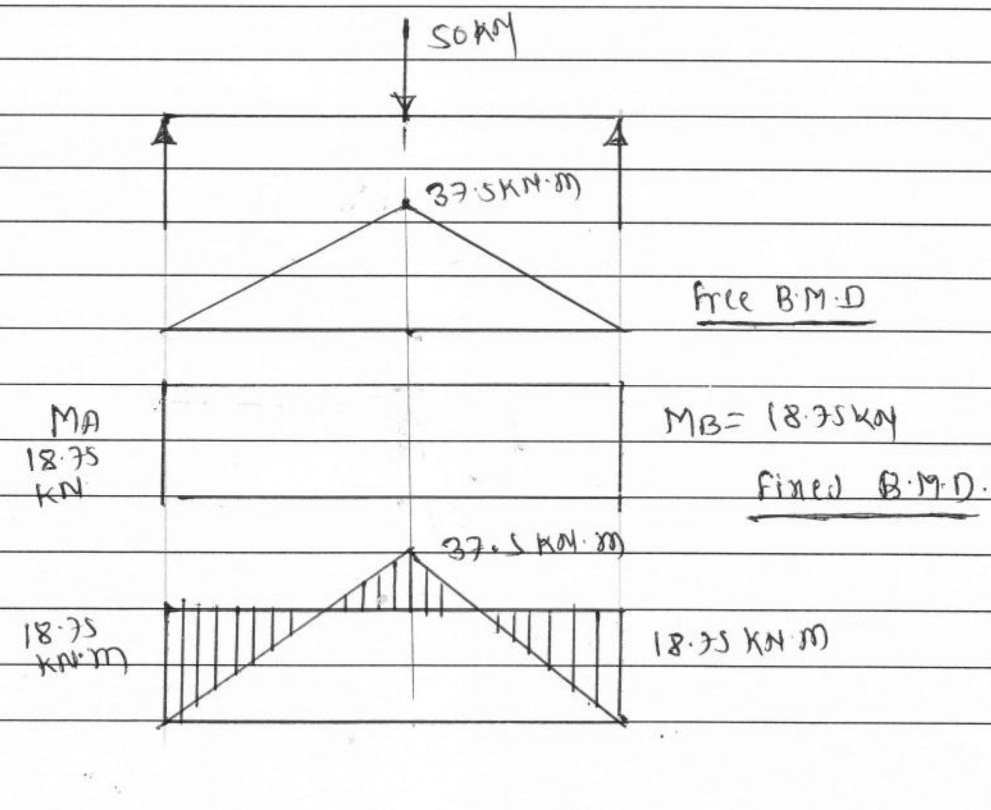
$\therefore M_A = -18.75$ KN-m ½M

Negative sign indicate as Hogging B.M. at support

Due to symmetry

$M_A = M_B$

$\therefore \boxed{M_A = M_B = -18.75 \text{ KN.m}}$ ½M



01M

Q.3

e] A Frame is an assembly of members connected together by rivets or welds. The members of a frame are made up of rolled steel sections such as angles channels, Flats, bars, etc. A Frame is usually loaded and supported externally at the joints. only

Analysis of frames means to calculate the axial force in the members of a frame. A Frame or truss can be analysed analytically by using three Static condition of equilibrium viz; $\Sigma Fx = 0$, $\Sigma Fy = 0$, $\Sigma M = 0$. A Frame can also be analysed graphically by using principles of graphic statics. 1M

Assumptions in the Analysis of Frames:- 1M each

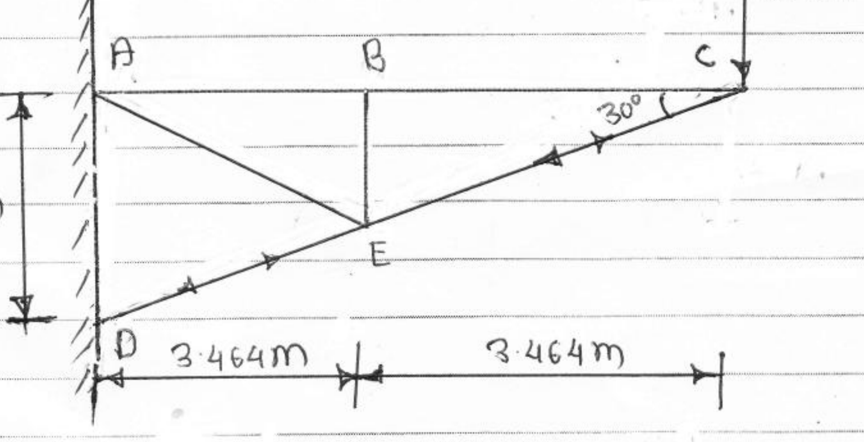
a) The frame is perfect one. i.e., $n = 2j - 3$ is always satisfied. write

b) All the members are hinged or pin-jointed at the end Any

c) Loads are acting only at the joints. Three

d) Self weight of member is neglected. Unless & otherwise mentioned

Q-3 (f)



Q.3

F) To find distance BE

cont.

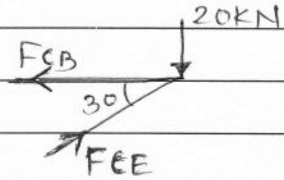
$\tan 30 = \frac{BE}{BC}$

$\therefore BE = BC \tan 30$

$= 3.464 \tan 30 = 1.999 m \simeq 2 m$

Consider Joint C

FBD for joint C



$\therefore \Sigma fy = 0$

$+20 - F_{CE} \sin 30 = 0$

$\therefore - F_{CE} \sin 30 = -20$

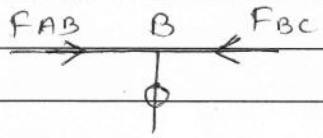
$\therefore F_{CE} = +40 KN$ (compressive) 1M

$\Sigma fx = 0$

$F_{CB} + F_{CE} \cos 30 = 0$

$\therefore F_{CB} = 34.64 KN$ 1M

Consider Joint B



$\Sigma fx = 0$

$F_{AB} - F_{BC} = 0$

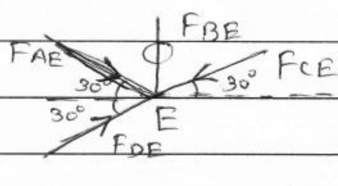
$\therefore F_{AB} = F_{BC} = 34.64 KN$

& $\Sigma fy = 0$ 1M

$\therefore F_{BE} = 0$

Consider Joint E

FBD for joint E



$-34.64 - F_{AE} \cos 30 - F_{DE} \cos 30 = 0$

$\therefore F_{AE} + F_{DE} \cos 30 = -34.64$ —(1)

$\Sigma fy = 0$

$-20 + F_{AE} \sin 30 - F_{DE} \sin 30 = 0$

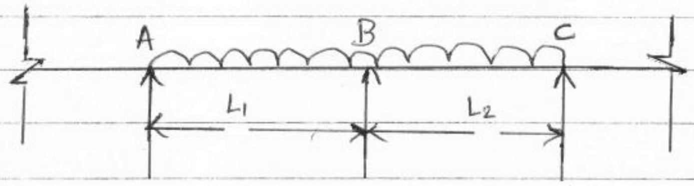
$F_{AE} \sin 30 - F_{DE} \sin 30 = 20$ —(2)

Solving eq^n (1) & (2) simultaneously

$F_{AE} = 0$ and $F_{DE} = -40 KN$ (compressive) 1M

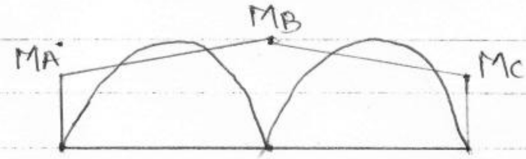
Q.4

a) Clapeyron's Thearem :-



simply supported beam.

01M



Net BMD

Theorem:- for a two spans continous beam having uniform M.I. supported at A, B & C and subjected to any External loading the support moments M_A, M_B & M_C at the support A, B&C Respectively are given by the relation.

$$M_A \times L_1 + 2M_B(L_1+L_2) + M_C \times L_2 = -\left[\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right]$$ 02M

where,

L_1 = Length of span AB.

L_2 = length of span BC

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

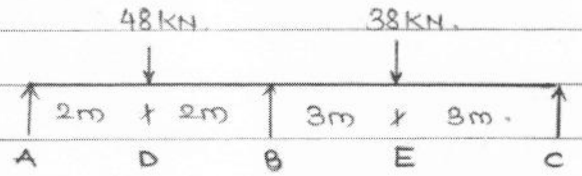
a_2 = Area of free B.M.D for the span BC 01M

$\bar{x}_1$ = Distance of C.G of free BMD over the span AB from the left end A

$\bar{x}_2$ = Distance of C.G. of free BMD over the span BC from the Right support end C

Q.4

b] From given data



Step 1 Free Bending Moment span AB

$L_1 = 4m$ $W = 48KN$ $M_A = M_B = 0.$

$M_D = M_{Dmax} = \frac{WL}{4} = \frac{48 \times 4}{4} = 48 \, KN \cdot m$ ½ M

Free Bending Moment span BC

$L_2 = 6m$ $W = 38KN$ $M_B = M_C = 0.$

$M_E = M_{Emax} = \frac{WL}{4} = \frac{38 \times 6}{4} = 57 \, KN \cdot m$ ½ M

Step-2 To Find Support moment at A, B & C.

M_A = due to simply Support $M_A = 0$.

M_B = due to simply & contineous Support M_B.

M_C = due to simply Support $M_C = 0$.

For span AB

a_1 = area of M-diagram.

= ½ × base × height.

= $\frac{1}{2} \times 4 \times 48$

= $96 \, m^2$.

For span BC

a_2 = area of M-diagram

= ½ × base × height.

= $\frac{1}{2} \times 6 \times 57$.

= $171 \, m^2$.

½ M for AB span

½ M for BC span

$\bar{x}_1 = 4/2 = 2m$.

$\bar{x}_2 = 6/2 = 3m$.

Applying clapeyron's theorem,

$M_A L_1 + 2M_B(L_1 + L_2) + M_C L_2 = -\left(\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right)$ ½ M

$$0 + 2M_B \times (4+6) + 0 = -\left(\frac{6\times 96\times 2}{4} + \frac{6\times 171\times 3}{6}\right)$$ ½M

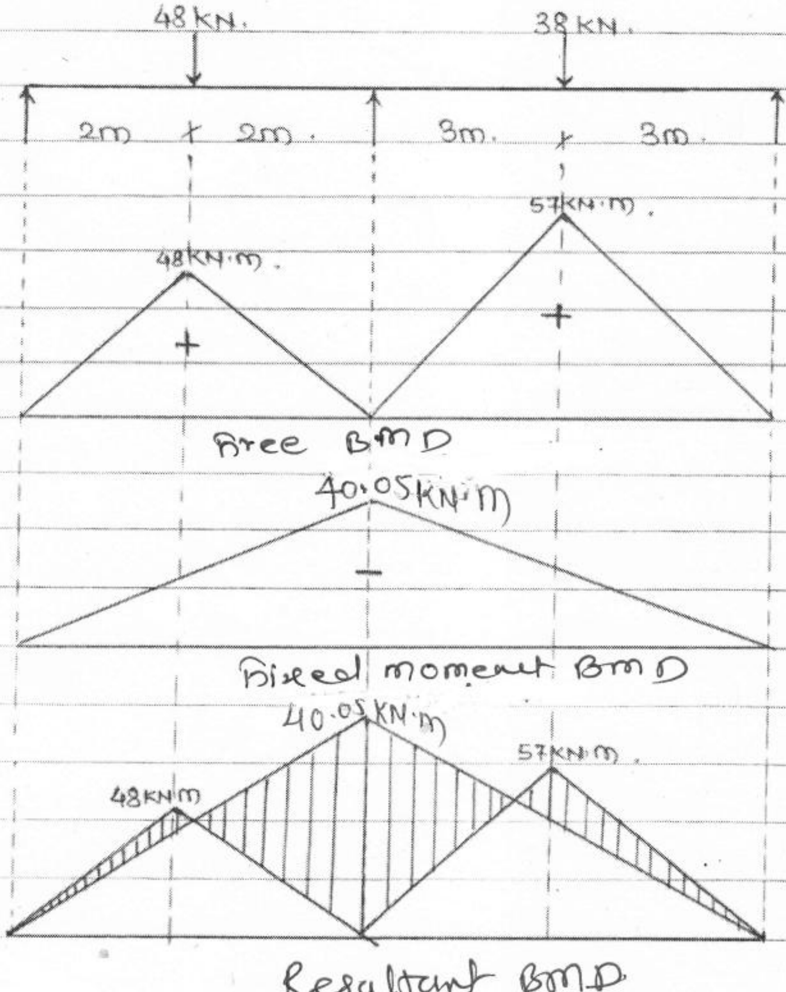
$$20M_B = -(288 + 573)$$

$$\boxed{M_B = -40.05 \text{ kN·m.}}$$ ½M

(∵ -ve sign indicates hogging bending moment).

$$M_A = M_C = 0.$$

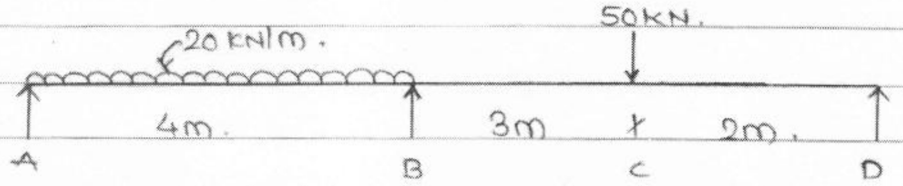
From the above value of support moment draw the M-diagram M'-diagram and superimposing both diagrams.



½M

Q.4

c] From the given data :-



Step-1 Assume span AB and BC to simply support and find free BM.

For span AB :-

$L_1 = 4m \quad W = 20\ KN/m$

$M_{max}\ AB = \frac{Wl^2}{8} = -\frac{20 \times 4^2}{8} = -40\ KN.m.$ ½M

For span BC :-

$a = 3m \quad b = 2m \quad W = 50\ KN.$

$M_B = M_C = 0.$

$\therefore M_D = \frac{Wab}{l} = \frac{50 \times 3 \times 2}{5} = 60\ KN.m.$ ½M

Step-2 To find support moment.

Span AB	Span BC
$a_1 = 2/3 \times base \times height.$	$a_2 = ½ \times base \times height.$
$= 2/3 \times 4 \times 40.$	$= ½ \times 5 \times 60$
$= \cancel{213.33}\ m^2.$	$= 150\ m^2.$
$= 106.67\ m^2$	$x_2 = \frac{L+b}{3} = \frac{5+2}{3} = 2.33\ m.$
$x_1 = 4/2 = 2m$	
$a_1x_1 = 213.33\ m^2.$	$a_2x_2 = 349.95.$

½M for span AB

½M for BC span

Applying clapeyron's theorem, for span AB & BC.

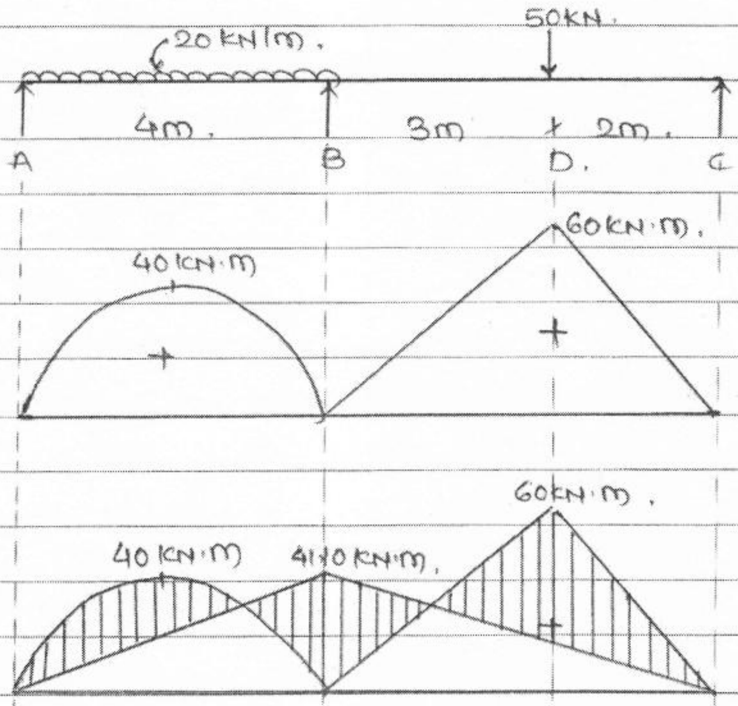
$$M_A \cdot L_1 + 2M_B(L_1+L_2) + M_C L_2 = -\left(\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right)$$ ½M

$$0 + 2M_B(4+5) + 0 = -\left(\frac{6\times 213.33}{4} + \frac{6\times 349.95}{5}\right)$$

$$18\,M_B = -(319.99 + 419.95)$$

$$M_B = -\frac{739.93}{18}$$

$$\boxed{M_B = -41.107\,\text{kN·m}}$$ ½M

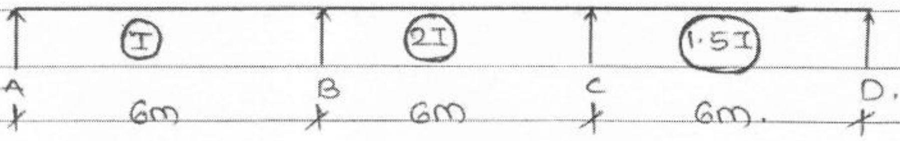


½M

½M

Q.4

d] From the given data:-



soln:-

Step 1 To calculate stiffness factor. (K)

Joint B:-

$$K_{BA} = \frac{3EI}{L_1} = \frac{3EI}{6} = 0.5EI.$$

$$K_{BC} = \frac{4EI}{L_2} = \frac{4E(2I)}{6} = 1.33EI.$$

$$\Sigma K = K_{BA} + K_{BC}.$$

$$= 0.5EI + 1.33EI.$$

$$= 1.83EI.$$

Joint C:-

$$K_{CB} = \frac{4EI}{L_2} = \frac{4E(2I)}{6} = 1.33EI.$$

$$K_{CD} = \frac{3EI}{L_3} = \frac{3E(1.5I)}{6} = 0.75EI.$$

$$\Sigma K = K_{CB} + K_{CD}.$$

$$= 1.33EI + 0.75EI.$$

$$= 2.08EI.$$

Step-2 Distribution factor calculation

Joint B

$(Df)_{BA} = \frac{K_{BA}}{\Sigma K} = \frac{0.5EI}{1.83EI} = 0.27$ 1M

$(D.f)_{BC} = \frac{K_{BC}}{\Sigma K} = \frac{1.33EI}{1.83EI} = 0.73$ 1M

Check $(D.f)_{BA} + (Df)_{BC}$
$= 0.28 + 0.72$
$= 1$ ---- Ok

Joint- C

$(Df)_{CB} = \frac{K_{CB}}{\Sigma K} = \frac{1.33EI}{2.08EI} = 0.64$ 1M

$(D.f)_{CD} = \frac{K_{CD}}{\Sigma K} = \frac{0.75EI}{2.08EI} = 0.36$ 1M

check $(Df)_{CB} + (Df)_{CD} = 0.64 + 0.36 = 1$ --ok

OR

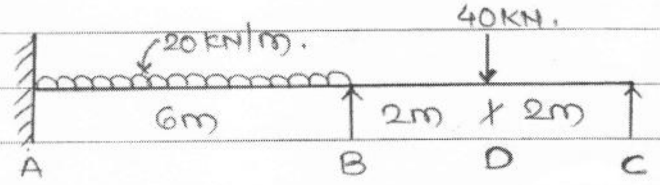
Joint	member	stiffness factor	Total stiffness	D.f.	
B	BA	$\frac{3EI}{L} = \frac{3EI}{6} = 0.5EI$	1.83EI	0.27	1M
	BC	$\frac{4EI}{L} = \frac{8EI}{6} = 1.33EI$		0.73	1M
C	CB	$\frac{4EI}{L} = \frac{8EI}{6} = 1.33EI$	2.083EI	0.64	1M
	CD	$\frac{3EI}{L} = \frac{3E \times 1.5I}{6} = 0.75EI$		0.36	1M

Q.4

e] Given:-

$L_1 = 6m$ $L_2 = 4m$ $w = 20\ kN/m$ $W = 40\ kN$

From given data.



Step 1 :- Fixed End Moment Calculation

For span AB

$$M_{AB} = -\frac{wL_1^2}{12} = \frac{-20 \times (6)^2}{12} = -60\ kN.m$$

$$M_{BA} = +\frac{w.L_1^2}{12} = +\frac{20 \times (6)^2}{12} = +60\ kN.m$$

01M

For span BC

$$M_{BC} = -\frac{WL_2}{8} = -\frac{40 \times 4}{8} = -20\ kN.m$$

$$M_{CB} = +\frac{WL_2}{8} = \frac{+40 \times 4}{8} = +20\ kN.m.$$

01M

Step 2 Distribution Factor. (Given)

$(DF_{BA}) = 0.57$ $(DF)_{BC} = 0.43$

Step 3 Moment Distribution Table.

	A		B		C
D.F.		0.57	0.43		
FEM.	-60	+60	-20	+20	
				-20	
Release C and carry over to B.			-10		
Initial moments	-60	+60	-30	0	
1st distribution	-8.55	-17.1	-12.9		
Final moment	-68.55	+42.9	-42.9	0	

02M

Step-4 Free Bending Moment.

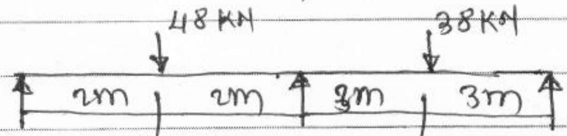
$M_A = 0,\ M_B = 0$ Max. Bending Moment.

$$= \frac{WL^2}{8} = \frac{20 \times 6^2}{8} = 90 \text{ KN.m}$$

$M_B = 0,\ M_C = 0$ Max. Bending Moment.

$$= \frac{W.L}{4} = \frac{40 \times 4}{4} = 40 \text{ KN.m}$$

Q-4(f)



Given:

$L_1 = 4m$ $L_2 = 6m$ $W_1 = 48 KN$ $W_2 = 38 KN$

Step-1 Fixed End Moment Calculation.

For span AB

$$M_{AB} = -\frac{WL}{8} - \frac{48 \times 4}{8} = -24 \text{ KN·m}$$

$$M_{BA} = \frac{WL}{8} = \frac{48 \times 4}{8} = +24 \text{ KN·m}$$

For span BC

$$M_{BC} = -\frac{WL}{8} = \frac{-38 \times 6}{8} = -28.5 \text{ KN·m}$$

$$M_{CB} = +\frac{WL}{8} = \frac{+38 \times 6}{8} = +28.5 \text{ KN·m}$$ ½M

Step-2 Stiffness Factor Calculation:

Span AB

$$K_{BA} = \frac{3EI}{L} = \frac{3EI}{4} = 0.75 EI.$$

$$K_{BC} = \frac{3EI}{L} = \frac{3EI}{6} = 0.5 EI$$

$$\Sigma K = K_{BA} + K_{BC}$$
$$= 0.75EI + 0.5EI$$
$$=$$

½M

$$\boxed{\Sigma K = 1.25 EI}$$

Step -3 Distribution factor calculation

$(DF)_{BA} = \frac{K_{BA}}{\Sigma K} = \frac{0.75\ EI}{1.25\ EI} = 0.6$

$(DF)_{BC} = \frac{K_{BC}}{\Sigma K} = \frac{0.5\ EI}{1.25\ EI} = 0.4$ ½M

Step - 4 Moment Distribution table

Joint	A	B		C
Member	AB	BA	BC	CB
D.F		0.6	0.4	
F.E.M	-24	+24	-28.5	+28.5
Release A	+24			-28.5
& C & C.O		12	-14.25	
Initial moment	0	36	-42.75	0
1st Distribution		+ 4.05	+ 2.70	
Final moment		40.05	- 40.05	

02M

Bending moment at support B = 40.05 kN.m

Step-5 span moment

span AB = $\frac{WL}{4} = \frac{48 \times 4}{4} = 48$ kN.m

span BC = $\frac{WL}{4} = \frac{38 \times 6}{4} = 57$ kN.m ½M

Q5 a)

→ Given

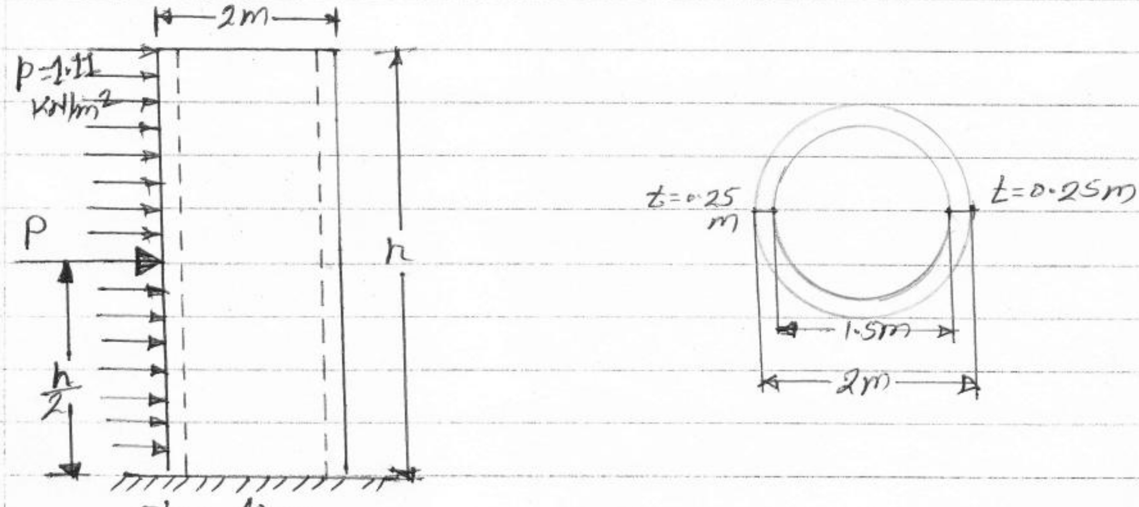
$D = 2m$

$t = 0.25m$ ∴ $d = D - 2t = 1.5m$

$p = 1.11\ KPa = 1.11\ KN/m^2$

$C_p = 0.60$

$\rho = 21\ KN/m^3$



Elevation

Let h be the height of chimney.

1) Direct stress $\sigma_0 = \dfrac{\text{Weight of chimney } W}{\text{c/s Area of chimney } A}$

$\sigma_0 = \dfrac{\cancel{A} \times h \times \rho}{\cancel{A}} = \rho h$

∴ $\sigma_0 = 21 \times h$ ------ (i) 1m

2) Bending stress $\sigma_b = \dfrac{M}{Z} = \dfrac{P \times h/2}{Z}$ 1m

P = Total wind force on chimney = $C_p \times p \times$ Projected Area

$P = 0.6 \times 1.11 \times 2 \times h$

$P = 1.332h$ 1m

Section modulus of section.

$$Z = \frac{\pi}{32}\left(\frac{D^4-d^4}{D}\right) = \frac{\pi}{32}\left(\frac{2^4-1.5^4}{2}\right)$$

$$Z = 0.5369 \ m^3$$ 1m

$$\therefore \ \sigma_b = \frac{1.332h \times \frac{h}{2}}{0.5369} = 1.24h^2 \quad KN/m^2$$ 1m

For No tension Condition

$$\sigma_0 = \sigma_b$$ 1m

$$21h = 1.24h^2$$

$$\therefore \ h = 16.935 \ m.$$ 1m

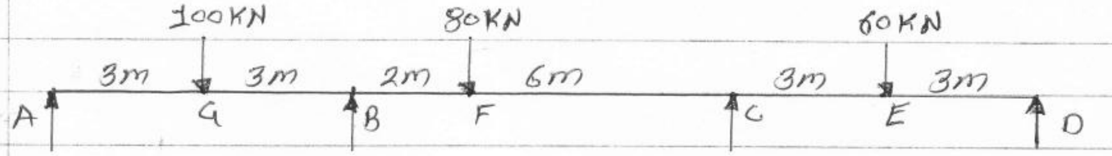
Maximum stress $\sigma_{max} = \sigma_0 + \sigma_b$

$$= 21 \times 16.935 + 1.24 \times (16.935)^2$$

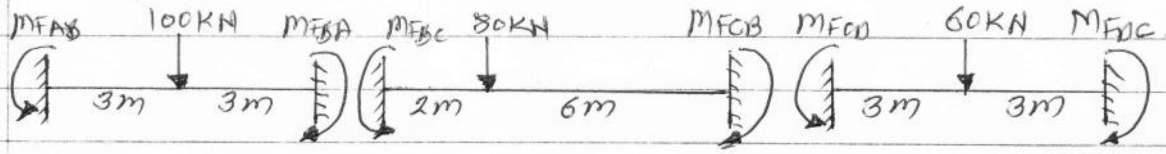
$$\sigma_{max} = 711.25 \quad KN/m^2$$ 1m.

Q5b

→



1) Find fixed end moments (FEM)



$M_{FAB} = M_{FBA} = \pm \frac{WL}{8} = \pm \frac{100 \times 6}{8} = \pm 75$ KN·m

$M_{FBC} = - \frac{Wab^2}{L} = - \frac{80 \times 2 \times 6^2}{8^2} = - 90$ KN·m

$M_{FCB} = + \frac{Wa^2b}{L^2} = + \frac{80 \times 2^2 \times 6}{8^2} = +30$ KN·m. 1M

$M_{FCD} = M_{FDC} = \pm \frac{WL}{8} = \pm \frac{60 \times 6}{8} = \pm 45$ KN·m.

2) Distribution factor

Joint	member	Stiffness of member	Total Stiffness of joint	D.F
B	BA	$\frac{3EI}{L} = \frac{3EI}{6} = 0.5EI$	1EI.	0.50
	BC	$\frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$		0.50
C	CB	$\frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$	1EI.	0.50
	CD	$\frac{3EI}{L} = \frac{3EI}{6} = 0.5EI$		0.50.

2M

37 Moment Distribution table.

Joint	A	B		C		D
member	AB	BA	BC	CB	CD	DC
D.F.	-	0.50	0.50	0.50	0.50	-
F.E.M.	-75	75	-90	30	-45	45
Released A & D	+75					-45
Carry over to B & C		37.5			-22.5	
Initial moment	0	112.5	-90	30	-67.5	0
1st Distribution		-11.25	-11.25	18.75	18.75	
Carry over			9.37	-5.62		
2nd distribution		-4.68	-4.68	2.81	2.81	
Carry over			1.40	-2.34		
3rd distribution		-0.70	-0.70	1.17	1.17	
Carry over			0.58	-0.35		
4th distribution		-0.29	-0.29	0.17	0.17	
final moments		95.58	-95.58	44.60	-44.60	

3M

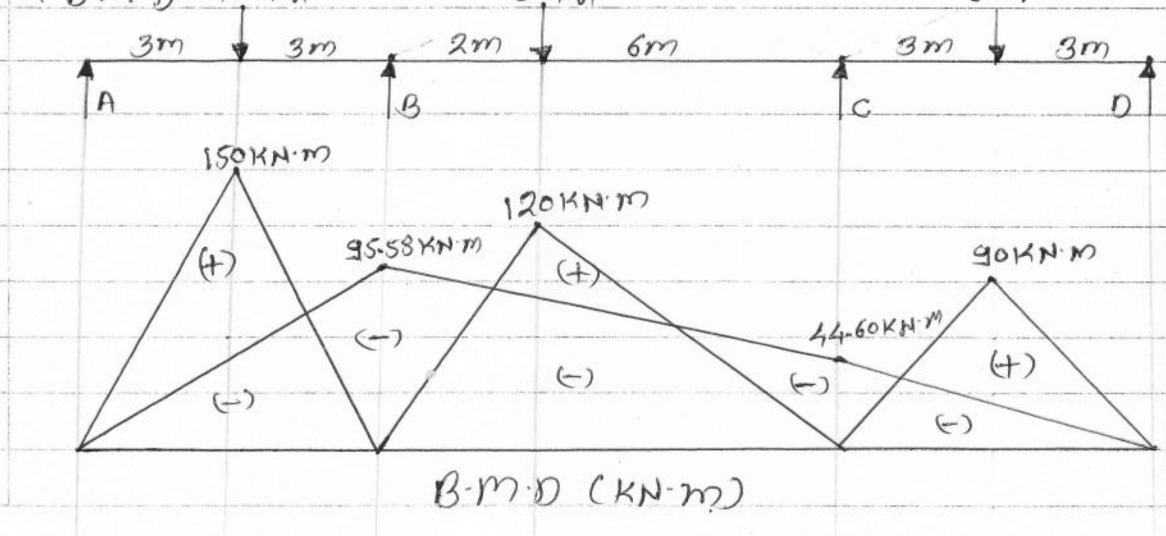
∴ Support moments

$M_A = M_D = 0$ .... S.S. ends

$M_B = 95.58$ KN.m Hogging moment

$M_C = 44.60$ KN.m Hogging moment

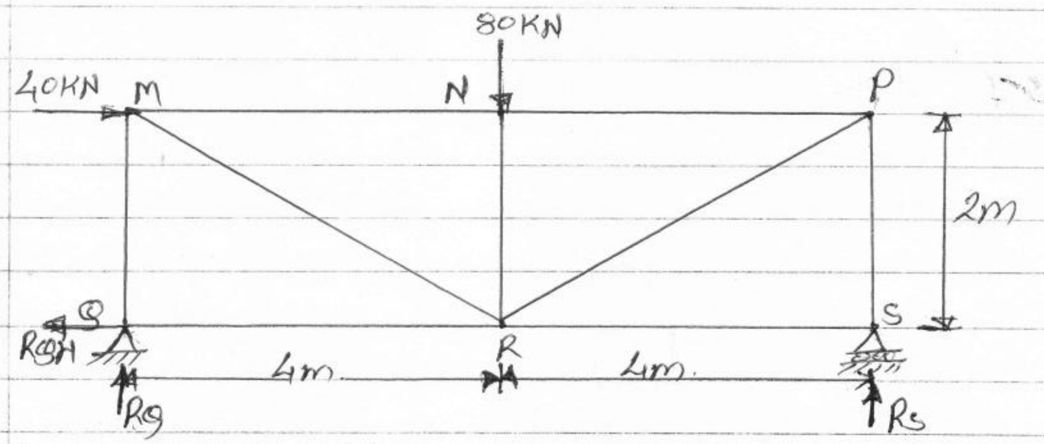
4) B.M.D.



B.M.D (KN.m)

2M

Q5c →



1) Support Reactions

a) $\Sigma F_y = 0$

$R_Q + R_S = 80$ KN.

b) $\Sigma F_x = 0$

$R_{QH} = 40$ KN.

c) $\Sigma M@Q = 0$

$(80\times4)+(40\times2) - R_S\times8 = 0$

$\therefore R_S = 50$ KN 1M

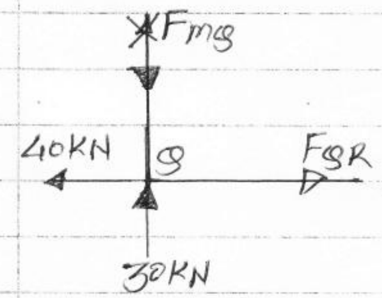
$\therefore R_Q = 30$ KN.

NOTE

In this problem they have not mension the method of analysis, i.e method of joint or method of section Student may assume any method for analysis.

2) Member forces by method of joint

i) Joint Q.



a) $\Sigma F_x = 0.$

$-40 + F_{QR} = 0$

$\therefore F_{QR} = 40$ KN.

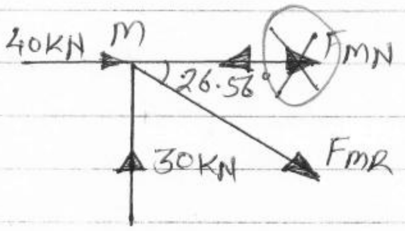
b) $\Sigma F_y = 0.$

$30 + F_{mQ} = 0$

$F_{mQ} = -30$ KN

$\therefore F_{mQ} = 30$ KN Comp.

ii> Joint M.



a> $\Sigma F_y = 0$.

$30 - F_{MR} \sin 26.56 = 0$

$\therefore F_{MR} = 67.09\ KN$. 1M

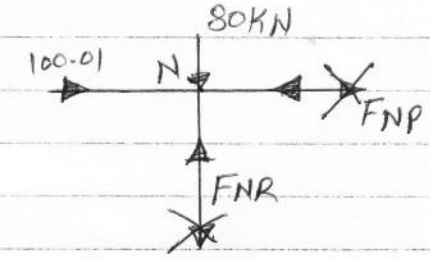
b> $\Sigma F_x = 0$.

$40 + F_{MN} + 67.09 \cos 26.56 = 0$.

$F_{MN} = -100.01\ KN$.

$\therefore F_{MN} = 100.01\ KN$ (Comp) 1M

iii> Joint N.



a> $\Sigma F_x = 0$.

$100.01 + F_{NP} = 0$

$\therefore F_{NP} = -100.01\ KN$

$F_{NP} = 100.01\ KN$ (Comp.) 1M

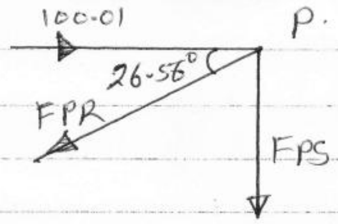
b> $\Sigma F_y = 0$.

$-80 - F_{NR} = 0$

$F_{NR} = -80\ KN$

$\therefore F_{NR} = 80\ KN$ (Comp) 1M

iv> Joint P



a> $\Sigma F_x = 0$

$100.01 - F_{PR} \cos 26.56 = 0$

$\therefore F_{PR} = 111.81\ KN$. 1M

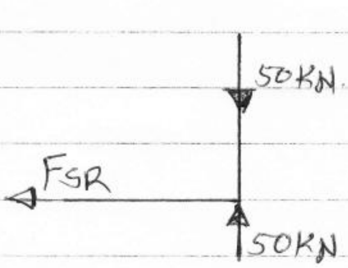
b> $\Sigma F_y = 0$.

$-111.81 \sin 26.56 - F_{PS} = 0$

$F_{PS} = -50\ KN$.

$\therefore F_{PS} = 50\ KN$ Comp. 1M

v> Joint S.



a> $\Sigma F_x = 0$

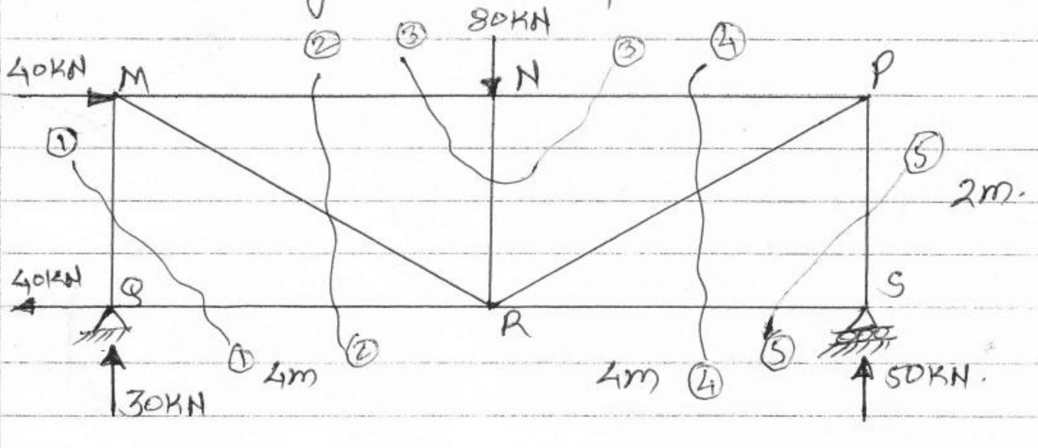
$F_{SR} = \underline{0}$

3) Final member forces.

Member	Magnetute	Nature
QR	40 KN	Tensile.
MQ	30 KN	Comp.
MN	100.01 KN	Comp.
MR	67.09 KN	Tensile
NP	100.01 KN	Comp.
NR	80 KN	Comp
PR	111.81 KN	Tensile
PS	50 KN	Comp.
SR	0 KN	—

1M

* Solve this by method of section.

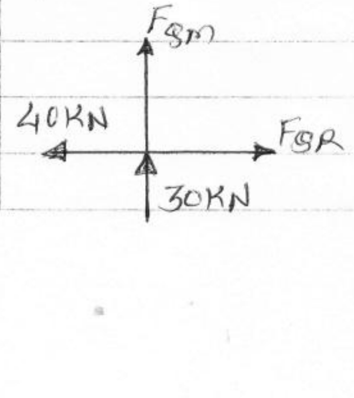


i) Support reactions

$R_Q = 30$ KN $R_{QH} = 40$ KN $R_S = 50$ KN 1M

2) Member forces

i) Consider LHS of section ①-①



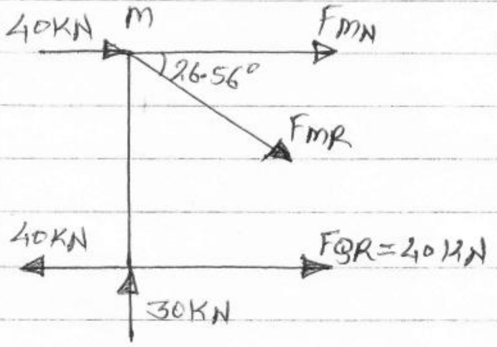
a) $\Sigma Fx = 0;\ -40 + F_{QR} = 0$

$\therefore F_{QR} = 40$ KN.

b) $\Sigma Fy = 0;\ 30 + F_{QM} = 0$

$\therefore F_{QM} = 30$ KN ...(Comp)

ii> Consider LHS of Section ②-②



a> $\Sigma F_y = 0$; $30 - F_{MR} \sin 26.56 = 0$

$\therefore F_{MR} = 67.09$ KN. 1m

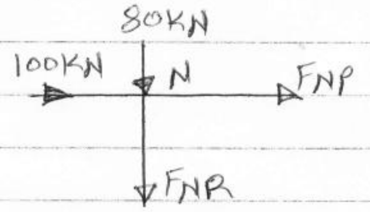
b> $\Sigma F_x = 0$

$-40 + 40 + F_{QR} + F_{MR} \cos 26.56 + F_{MN} = 0.$

$40 + 67.09 \cos 26.56 = - F_{MN}$

$F_{MN} = 100$ KN (Comp) 1m

iii> Consider section ③-③



a> $\Sigma F_x = 0$; $100 + F_{NP} = 0$

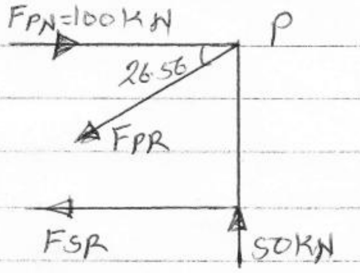
$\therefore F_{NP} = 100$ KN (Comp) 1m

b> $\Sigma F_y = 0.$

$-80 - F_{NR} = 0$

$\therefore F_{NR} = 80$ KN ....(Comp.) 1m

iv> Consider section ④-④



a> $\Sigma F_y = 0$

$50 - F_{PR} \sin 26.56 = 0$

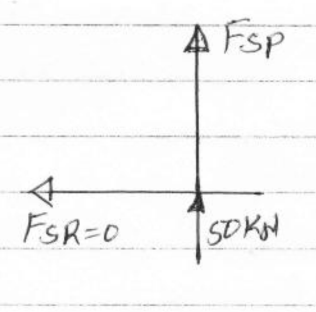
$\therefore F_{PR} = 111.82$ KN. 1m

b> $\Sigma F_x = 0$

$100 - F_{PR} \cos 26.56 - F_{SR} = 0$

$\therefore F_{SR} = 0$ KN.

v> Consider section ⑤-⑤



a> $\Sigma F_y = 0.$

$50 + F_{SP} = 0$

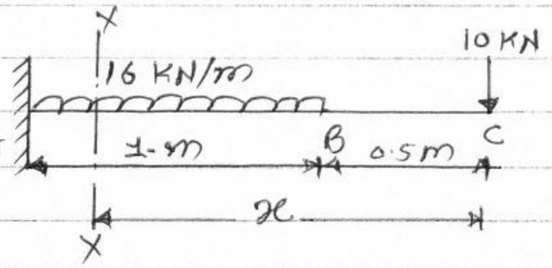
$\therefore F_{SP} = 50$ KN ....(Comp) 1m

37 Final member forces.

member	Magnetute	Nature.
QR	40 KN	Tensile
MQ	30 KN	Comp.
MN	100 KN	Comp
MR	67.09 KN	Tensile
NP	100 KN	Comp
NR	80 KN	Comp.
PR	111.82 KN	Tensile
PS	50 KN	Comp.
SR	0 KN	—

1M

Q6a)



Take the free end C as a origine & Conside a Section X-X at a distance x from C in portion AB as shown in fig.

$$M_x = -10x - \frac{16(x-0.5)^2}{2}$$

But, $EI\frac{d^2y}{dx^2} = M_x$. ½M

$$\therefore EI\frac{d^2y}{dx^2} = -10x - 16\frac{(x-0.5)^2}{2} \text{------(A)}$$ ½M

Integrate eqn A w.r.to x.

$$EI\frac{dy}{dx} = -\frac{10x^2}{2} - \frac{16(x-0.5)^3}{6} + C_1 \text{ ----(B)}$$ ½M

Again integrate eq^n B w.r.to x.

$$EI\ y = -10\frac{x^3}{6} - 16\frac{(x-0.5)^4}{24} + C_1x + C_2 \quad \text{....} (C)$$ $\frac{1}{2}$m

To find integrating Constants C_1 & C_2
Apply Boundary Conditions
i> At A ; when $x = 1.5$ m, $\frac{dy}{dx} = 0$ ... put in eq^n (B)

$$0 = -10\frac{(1.5)^2}{2} - 16\frac{(1.5-0.5)^3}{6} + C_1$$

$$0 = -11.25 - 2.67 + C_1$$

$$\therefore C_1 = 13.92$$ 1m

ii> At A ; when $x = 1.5$ m $y = 0$ ... put in eq^n (C)

$$0 = -10\frac{(1.5)^3}{6} - 16\frac{(1.5-0.5)^4}{24} + 13.92 \times 1.5 + C_2$$

$$0 = -5.625 - 0.67 + 20.88 + C_2$$

$$\therefore C_2 = -14.58$$ 1m

Substitute values of C_1 & C_2 in eq^n B & C respectively.

$$EI\frac{dy}{dx} = -10\frac{x^2}{2} - 16\frac{(x-0.5)^3}{6} + 13.92 \quad \text{..... Slope eq}^n$$ $\frac{1}{2}$m

$$EI\ y = -10\frac{x^3}{6} - 16\frac{(x-0.5)^4}{24} + 13.92x - 14.58 \quad \text{... Def}^n \text{ eq}^n$$ $\frac{1}{2}$m

To find slope at C put $x = 0$ in slope eq^n.

$$EI\left(\frac{dy}{dx}\right)_c = 0 - 0 + 13.92$$

$$\therefore \left(\frac{dy}{dx}\right)_c = \theta c = \frac{13.92}{EI}$$ 1m

To find deflection at C put $x = 0$ in deflection eq^n.

$$EI \cdot y_C = 0 - 0 + 0 - 14.58$$

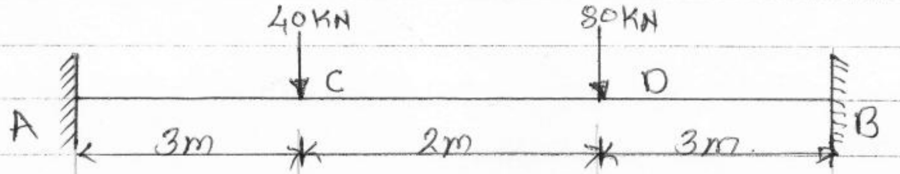
$$\therefore y_C = -\frac{14.58}{EI}$$ -ve sign indicate downward deflection. 1M

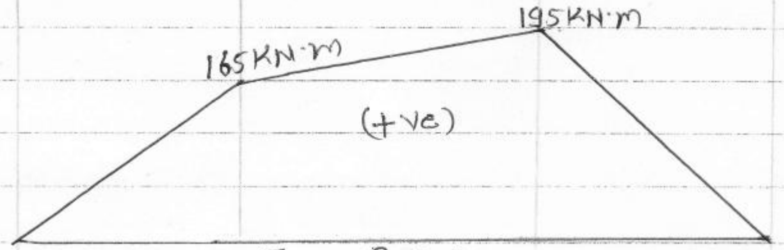
To find deflection at B, put $x = 0.50m$ in deflection eq^n

$$EI \; y_B = -\frac{10(0.5)^3}{6} + (13.92 \times 0.5) - 14.58$$

$$y_B = -\frac{7.828}{EI}$$ -ve sign indicate downward deflection. 1M

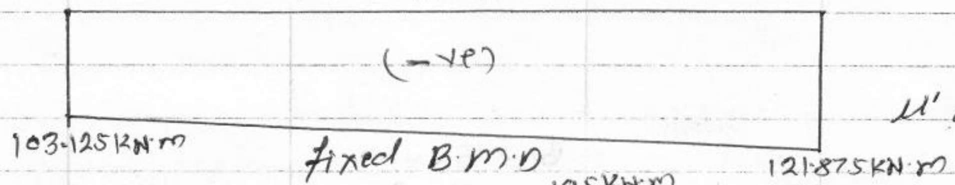
Q6 b)





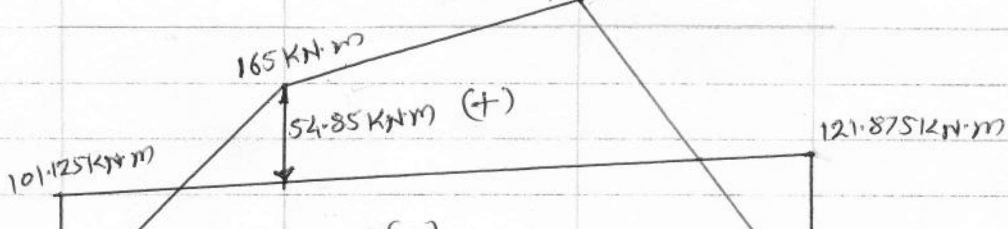
μ dig.

Free B.M.D



μ' dig.

fixed B.M.D



2M

Resultant BMD

1) Assume that the beam is simply supported & Calculate support reaction & draw μ diagram.

i) $\Sigma F_y = 0$; $R_A + R_B = 40+80 = 120$ KN.

ii) $\Sigma M @ A = 0$

$(40\times3)+(80\times5) = R_B \times 8$

$\therefore R_B = 65$ KN $\qquad \therefore R_A = 55$ KN — 1M

$M_A = M_B = 0$

$M_C = R_A \times 3 = 55\times3 = 165$ KN·m — 1M

$M_D = R_B \times 3 = 65\times 3 = 195$ KN·m

2) Calculate the fixed end moments & draw μ' dia.

$W_1 = 40$ KN $\qquad W_2 = 80$ KN

$a_1 = 3m$ $\qquad a_2 = 5m$

$b_1 = 5m$ $\qquad b_2 = 3m$

$$M_A = -\left[\frac{W_1 a_1 b_1^2}{L^2} + \frac{W_2 a_2 b_2^2}{L^2}\right] = -\left[\frac{40\times3\times5^2}{8^2} + \frac{80\times5\times3^2}{8^2}\right]$$

$\therefore M_A = -103.125$ KN·m — 1M

$$M_B = -\left[\frac{W_1 a_1^2 b_1}{L^2} + \frac{W_2 a_2^2 b_2}{L^2}\right] = -\left[\frac{40\times3^2\times5}{8^2} + \frac{80\times5^2\times3}{8^2}\right]$$

$\therefore M_B = 121.875$ KN·m. — 1M

3) Superimposed μ' diagram over μ diagram & draw final B.M. diagram.

4) Calculate reaction of a fixed beam.

$\Sigma F_y = 0 \quad R_A + R_B = 120$ KN.

$\Sigma M @ A = 0$

$(40\times3)+(80\times5) - 103.125 + 121.875 = 8R_B$

$\therefore R_B = 67.34$ KN $\qquad \therefore R_A = 52.66$ KN — 1M

5) Net B.M under 40 KN. Load.

$M_C = -M_A + R_A \times 3 = -103.125 + 52.66 \times 3$

$M_C = 54.85$ KN.m. 1M

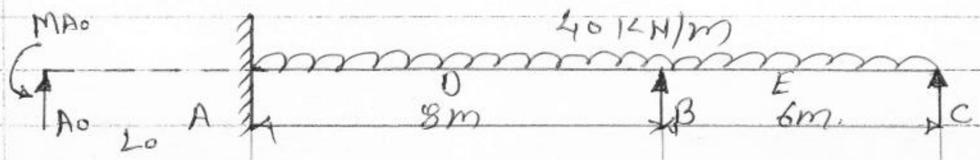
OR

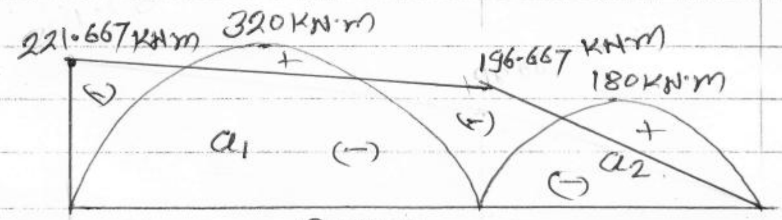
Net B.m under 40 KN Load can also be calculated as follows. OR

$M_C = 165 - \left[103.125 + \left(\frac{121.875 - 103.125}{8}\right) \times 3\right]$

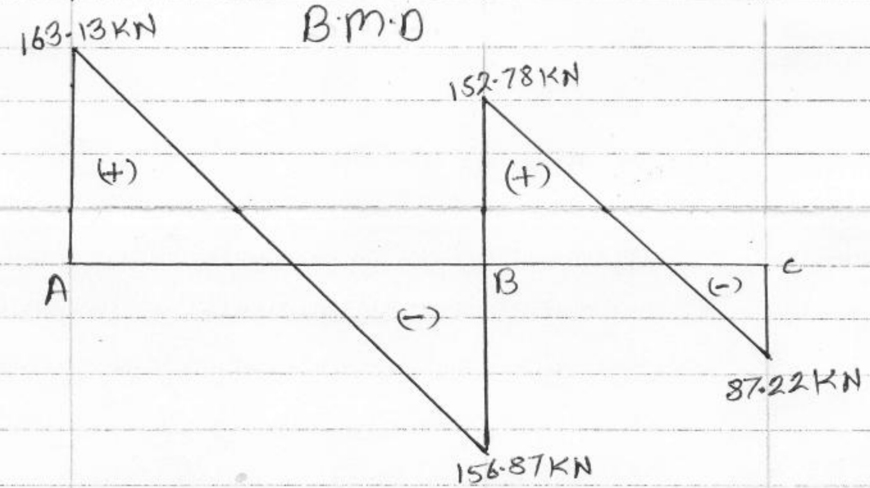
$M_C = 54.85$ KN. 1M

Q6C





B.M.D



SFD (KN)

2M for only SFD

1) Assume the span AB & BC as simply supported & draw ll diagram.

Free B.M at mid span of AB

$M_D = \frac{wl^2}{8} = \frac{40 \times 8^2}{8} = 320$ KN.m. — ½ m

Free B.M at mid span of BC

$M_E = \frac{wl^2}{8} = \frac{40 \times 6^2}{8} = 180$ KN.m. — ½ m

2) Since support A is fixed assume an imaginary span AA_0 to the left of A of length L_0.

Apply Clapeyrons theorem to span A_0AB.

$$M_{A0}(L_0) + 2M_A(L_0+L_1) + M_B(L_1) = -\left[\frac{6a_0\bar{x}_0}{L_0} + \frac{6a_1\bar{x}_1}{L_1}\right]$$ — ½ m

$M_{A0} = 0$	$a_1 = \frac{2}{3} \times 8 \times 320 = 1706.67$
$a_0 = 0$	$\bar{x}_1 = \frac{L_1}{2} = \frac{8}{2} = 4m$
$L_0 = 0$	$L_1 = 8m.$
$\bar{x}_0 = 0$	

$$0 + 2M_A(0+8) + M_B(8) = -\left[0 + \frac{6 \times 1706.67 \times 4}{8}\right]$$

$$16M_A + 8M_B = -5120.01 \quad \text{——— (i)}$$ — ½ m

3) Apply theorem of three moment to span ABC.

$$M_A(L_1) + 2M_B(L_1+L_2) + M_C(L_2) = -\left[\frac{6a_1\bar{x}_1}{L_1} + \frac{6a_2\bar{x}_2}{L_2}\right]$$ — ½ m

Known moment $M_C = 0$ ---- S.S end.

$a_1 = \frac{2}{3} \times 8 \times 320 = 1706.67$	$a_2 = \frac{2}{3} \times 6 \times 180 = 720$
$\bar{x}_1 = \frac{L_1}{2} = \frac{8}{2} = 4m$	$\bar{x}_2 = \frac{L_2}{2} = \frac{6}{2} = 3m$
$L_1 = 8m$	$L_2 = 6m$

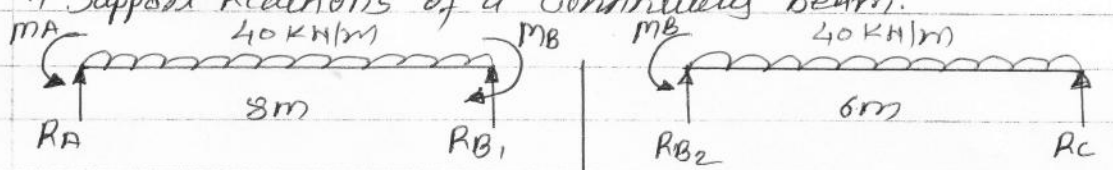
$$M_A(8)+2M_B(8+6)+0(6) = -\left[\frac{6\times1706.67\times4}{8}+\frac{6\times720\times3}{6}\right]$$

$8M_A + 28M_B = -7280.01$ ——— (ii) $\frac{1}{2}$M

Solving eqn (i) & (ii) we get

$M_A = -221.667$ KN·m } Hogging moments 1M
$M_B = -196.667$ KN·m

4) Support Reactions of a Continuous beam.



i) $\Sigma F_y = 0$; $R_A + R_{B1} = 320$ KN	i) $\Sigma F_y = 0$; $R_{B2} + R_C = 240$ KN
ii) $\Sigma M@A = 0$	ii) $\Sigma M@B = 0$.
$-221.66 + (40\times8\times4) + 196.66 = 8R_{B1}$	$-196.66 + (40\times6\times3) = R_C \times 6$
$\therefore R_{B1} = 156.87$ KN	$\therefore R_C = 87.22$ KN
$\therefore R_A = 163.13$ KN	$\therefore R_{B2} = 152.78$ KN

$\therefore R_A = 163.13 \quad = 163.13$ KN 1M
$R_B = R_{B1} + R_{B2} \quad = 309.65$ KN
$R_C = 87.22 \quad = 87.22$ KN.

5) S.F. Calculations

S.F at just left of A = 0 KN
S.F at just right of A = R_A = 163.13 KN
S.F at just left of B = 163.13 − (40×8) = −156.87 KN 1M
S.F at just right of B = −156.87 + 309.65 = 152.78 KN
S.F at just left of C = 152.78 − (40×6) = −87.22 KN
S.F at just right of C = −87.22 + 87.22 = 0 KN.