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(Revision-2015)

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DIPLOMA EXAMINATION IN ENGINEERING/TECHNOLOGY/
MANAGEMENT/COMMERCIAL PRACTICE, APRIL-2020

THEORY OF STRUCTURE-II

[Maximum marks: 75] (Time: 2.15 Hours)

PART – A

(Answer any *three* questions in one or two sentences. Each question carries 2 marks)

I. (1). When a column is said to be long column.

(2). State the relation between slenderness ratio and least radius of gyration.

(3). Specify the dimensions of the core of circular base of diameter 'd'.

(4). Define stiffness of a structural beam.

(5). Specify the condition and effect of carry over moment. (3 x 2 = 6)

PART – B

(Answer any *four* of the following questions. Each question carries 6 marks)

II. (1). State the different end conditions and corresponding Eulers Crippling load for a long column.

(2). For a rectangular column of width b and depth d, prove the conditions of middle third rule of the base with respect to both axes.

(3). Write the causes by which a dam is liable to fail and the minimum requirement to resist them.

(4). A cantilever of length 2m carries a uniformly distributed load of 2500N/meter length, throughout of the span. If the section is rectangular 120mm wide and 200mm deep find the deflection at free end, E=10000N/mm^2

(5). Write three advantages and disadvantages for a fixed beam.

(6). State the clapeyrons Theorem and specify how it is used for a continuous beam with fixed end supports.

(7). State the following:

(i). Carry over factor. (2). Stiffness factor. (3). Distribution factor. (4 x 6= 24)

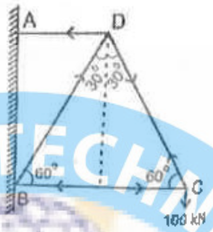
PART – C

(Answer ***any of the three units*** from the following. Each question carries 15 marks)

UNIT –I

III. (a). A solid round bar 3m long and 50mm in dia is used as a strut. Determine crippling load for all the different end conditions taking $E = 2.0 \times 10^5$ N/mm^2 (7)

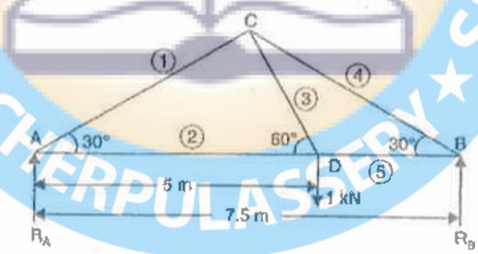
(b). Determine analytically the magnitude and nature of force in the members of truss in the figure.



(8)

OR

IV. A truss of span 7.5 m carries a point load of 1 KN at joint D as shown in fig. find the reaction and forces in the members of the truss.



(15)

UNIT-II

V. (a). A solid circular rod carries a load of 8kN acting at 15mm away from the center of gravity. Determine the diameter of rod if the max. stress developed is not to be exceed 40N/mm^2 (7)

(b). A dam section is 8m height, the maximum depth of water impounded being 7.5m, the top width of section is 1m, the weight of masonry is 22000N/m^3. While the weight of water is 9810 N/cum. Find the minimum base width required. Coefficient of friction between soil and masonry is 0.6, the water face of the dam is vertical. (8)

OR

VI. (a). In a tension specimen 2.6 cm in diameter, the line of pull is parallel to the axis of the specimen but is displaced from it. Determine the distance of the line of pull from the axis when the maximum stress is 20% greater than the means stress on a section normal to the axis. (7)

(b). A masonry trapezoidal dam 4 m high, 1 m wide at its top and 3m width at its bottom retains water on its vertical face. Determine the maximum and minimum stresses at the base (i) when the reservoir is full. Take the weight density of masonry as 19.62kN/m^3 (8)

UNIT-III

VII. (a). A cantilever 2m long is loaded with a concentrated load of 1.0kN at the free end and uniformly distributed load of 2kN/m over a length of 1.2m from the fixed end. If E=11kN/mm^2 and I = 66 x 10^6 mm^4, calculate the deflection at the free end. (8)

(b). A beam 8 meters long fixed at both ends carries a uniformly distributed load over the whole span. Find the load intensity on the beam,

(i). If the maximum bending moment shall not exceed 40kNm

(ii) if the maximum deflection shall not exceed 1/400 of the span.

Take E1 = 9.5 x 10^9 kNmm^2 (7)

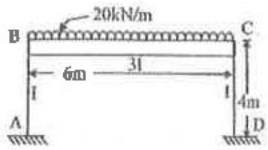
OR

VIII.(a). A simply supported beam of span 6m is carrying two point load. Each of 20kN at 2m and 4m from the left end. Determine the slope at the end and deflection at mid span of the beam. E1 of the beam is 20000kNm^2. Use moment area method. (8)

(b). A fixed beam AB, 4 meters long, is carrying a central point load of 30kN. Determine the fixing moments, deflection under the load and draw the BM diagram. Flexural rigidity of the beam as 5 x 10^4 kN-m^2 (7)

UNIT-IV

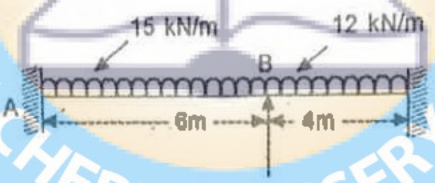
IX. A portal frame ABCD shown in fig, is loaded with a uniformly distributed load of 20kN/m on the horizontal member. $I_{AB} = I_{CD}$ and $I_{BC} = 3I_{AB}$. Determine the end moments and draw the BM diagram.



(15)

OR

X. Find the support moments by the method of moment distribution for the beam shown in Fig. and sketch the B.M diagram. EI is constant for the beam.

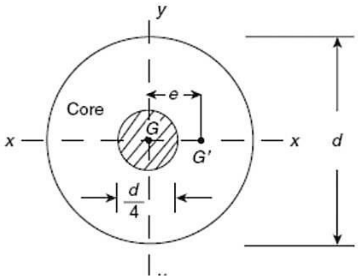


(15)

PART A

1. If the ratio of effective length of the column to it's the least lateral dimension is greater than 12, it is called a long column.
2. The slenderness ratio is the column's length divided by the radius of gyration.

$$s = \frac{L}{r}$$

3. 

4. Beam stiffness is a beam's ability to resist deflection, or bending, when a bending moment is applied.
5. The distributed moments in the ends of members meeting at a joint cause moments in the other ends, which are assumed to be fixed. These induced moments at the other ends are called carry-over moments.

PART B

1.

S. NO	END CONDITION	RELATION BETWEEN EFFECTIVE AND ACTUAL LENGTH	CRIPPLING LOAD(P)
1	Both sides hinged	$L_e = L$	$P_E = (\pi^2 E I) / L_e^2$
2	One fixed and other free	$L_e = 2L$	$P_E = (\pi^2 EI) / 4L_e^2$
3	both fixed	$L_e = L/2$	$P_E = 4(\pi^2 EI) / L_e^2$
4	One fixed and other hinged	$L_e = L/\sqrt{2}$	$P_E = 2(\pi^2 EI) / L_e^2$

2.

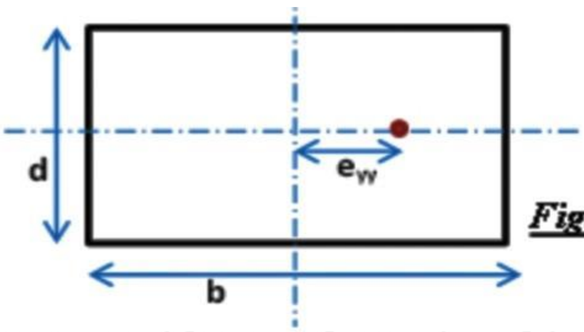


Fig.4.a

Let consider a column is subjected to compressive load 'P' having cross section dimensions breath 'b' and thickness 'd' as shown in fig. 4.a.

Let,

σ_0= Direct stress;

σ_b = Bending stress;

P = Applied load (compressive or tensile);

A = Cross-sectional area

A= b x d;

M = Bending Moment acting on column

M= P.e_{yy};

e_{yy} = Eccentricity w.r.t. Y-axis;

I_{yy} = Moment of inertia of column section along Y-axis

$$I_{yy} = \frac{db^3}{12};$$

y_{max} = Max. centroidal distance;

$$y_{max} = \frac{b}{2}$$

Z = Section modulus;

$$Z = \frac{\left(\frac{db^3}{12}\right)}{\left(\frac{b}{2}\right)} = \frac{db^3}{12} \times \frac{2}{b} = \frac{db^2}{6}$$

Now, for no tension condition;

$$\sigma_0 = \sigma_b$$

$$\frac{P}{A} = \frac{M}{Z} = \frac{P \cdot e_{yy}}{\frac{db^2}{6}}$$

$$e_{yy} = \frac{P}{A} \times \frac{db^2}{6} \times \frac{1}{P}$$

$$e_{yy} = \frac{db^2}{6A} = \frac{db^2}{6 \cdot bd} = \frac{b}{6}$$

This eccentricity when load 'P' on right side of Y-axis. Similarly eccentricity on left side of Y-axis i.e. b/6. Therefore total width of eccentricity at middle portion of section becomes

$$= \frac{b}{6} + \frac{b}{6} = \frac{b}{3}$$

<u>>>Along thickness</u>

As eccentricity calculates in breath same way we can find eccentricity along thickness. It becomes

$$e_{xx} = \frac{d}{6}$$

Total width of eccentricity at middle portion of section

$$= \frac{d}{3}$$

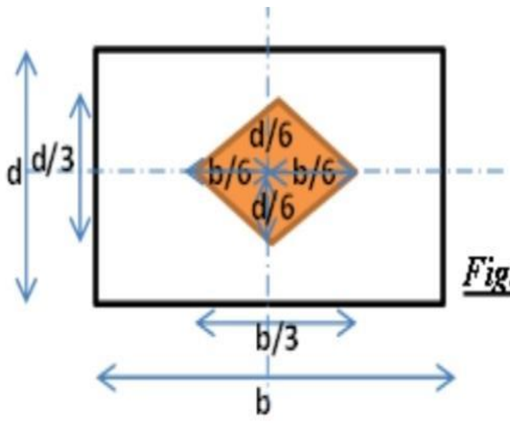


Fig.4.b

In fig.4.b shaded section is called core section. It having dimensions (b/3)X(d/3). This is also called as Middle Third Rule.

Middle Third Rule: For rectangular solid section within middle third portion their is no tension condition occurs.

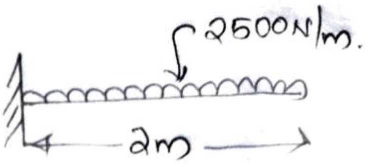
3. Failure of a dam
    - Shear failure(sliding) of gravity dam
    - Crushing of the gravity
    - Development of tensile forces which results in the crack in gravity dam
    - Overturning of dam about the toe

Requirement to resist

- Maximum frictional force at the base is more than the horizontal thrust of the water so that sliding at the base is prevented
- Maximum compressive stress in the masonry is never allowed to be more than its permissible limits to prevent crushing of the dam material.
- Resultant trust cuts the base within the middle third of the base width to avoid tension stress at the base.
- Overturning moment is less than the stabilizing moment for no overturning of the dam. The minimum base width of the dam is worked out equating the above limiting conditions.

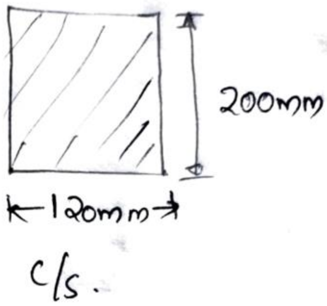
4.

Length = 2m = 2000mm



$w = 2500\ N/m = 2.5\ N/mm.$

$E = 10{,}000\ N/mm^2.$



$$I = \frac{bd^3}{12}$$

$$= \frac{120 \times 200^3}{12}$$

$$= \underline{\underline{80 \times 10^6\ mm^4}}$$

$$\text{Max. deflection} = \frac{wL^4}{8EI}$$

$$= \frac{2.5 \times 2000^4}{8 \times 10000 \times 80 \times 10^6}$$

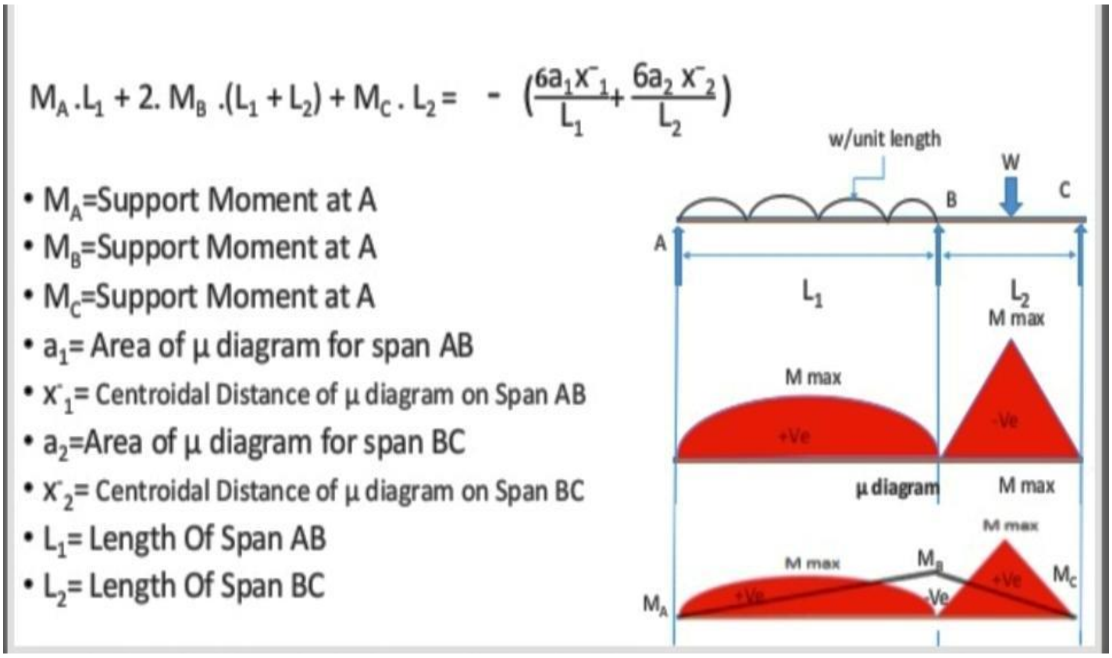
$$= \underline{\underline{6.25\ mm.}}$$

5.

Advantages	Disadvantages
1. It is stiffer stronger and more stable	1. Both ends of the beam must be on the same level or else extra stresses can be developed
2. The slope at the both end are zero	2. Stresses are produced in the beam due to changes in temperature
3. The deflection at the centre is very much reduced	3. The fixity of the beam is reduced due to vibrations

6. CLAPEYRON'S THEOREM

It states that if AB and BC are any two consecutive spans of a continuous beam subjected to external loading the support moments Ma & M_b and M_c at the supports A,B,C respectively can be obtained from the relation



7. Carry over factor:- it is the ratio of the moment applied at one end joint of a member to the moment induced at its other end joint.

Stiffness factor:- it is the moment required at one end of a member to produce a unit angle of rotation at that end.

Distribution factor:- The ratio of the moment induced in a certain member to the moment applied at the joint is called distribution factor for that member.

III (a).

III (a).

$L = 3m = 3000mm.$

$d = 50mm.$

$P = ?$

$E = 2\times10^5\ N/mm^2.$

$I = \frac{\pi d^4}{64} = \frac{\pi\times50^4}{64}$

$= 30.664\times10^4\ mm^4$

(i) Both ends are fixed.

$L_e = \frac{L}{2} = 1500mm.$

$P = \frac{\pi^2 E I_{min}}{L_e^2} = \frac{\pi^2\times2\times10^5\times30.664\times10^4}{1500^2} = 268.742\ kN$

(2) Both ends are hinged.

$$P = \frac{\pi^2 E I_{min}}{L_e^2}.$$

$L_e = L = 3000mm.$

$$P = \frac{\pi^2 \times 2\times10^5 \times 30.664\times10^4}{3000^2} = \underline{\underline{67.185\ kN}}$$

(3) One end fixed other end hinged

$$L_e = \frac{L}{\sqrt{2}} = \frac{3000}{\sqrt{2}} = \underline{\underline{2121.32\ mm.}}$$

$$P = \frac{\pi^2 \times 2\times10^5 \times 30.664\times10^4}{2121.32^2} = \underline{\underline{134.371\ kN}}$$

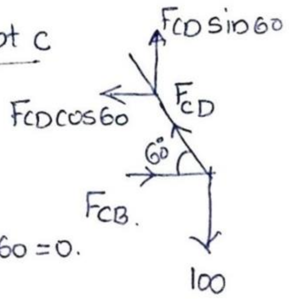
(4) One end fixed other end free.

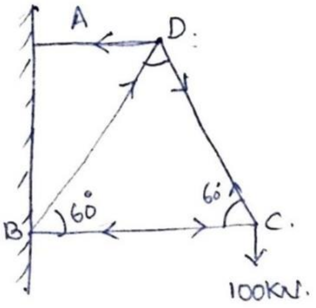
$L_e = 2L = \underline{\underline{6000mm}}$

$$P = \frac{\pi^2 \times 2\times10^5 \times 30.664\times10^4}{6000^2} = \underline{\underline{16.796\ kN}}.$$

III(b)

Consider Joint C





$\Sigma V = 0.$

$-100 + F_{CD}\sin 60 = 0.$

$F_{CD}\sin 60 = 100.$

$F_{CD} = 100/\sin 60 = 115.47$ (Tension) kN.

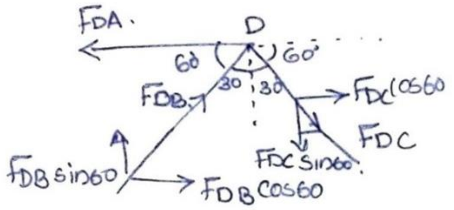
$\Sigma H = 0.$

$F_{CB} - F_{CD}\cos 60 = 0.$

$F_{CB} = F_{CD}\cos 60$

$= \underline{57.73\,kN}$ (compressive).

Consider Joint D.



$\Sigma H = 0.$

$F_{DC}\cos 60 + F_{DB}\cos 60 = F_{DA}.$

$115.47 \times \cos 60 + F_{DB}\cos 60 = F_{DA}.$

$57.73 = F_{DA} - F_{DB}\cos 60. \quad (1).$

$\Sigma V = 0.$

$F_{DB}\sin 60 = F_{DC}\sin 60.$

$F_{DB} = \underline{115.47\,kN}$ (compressive)

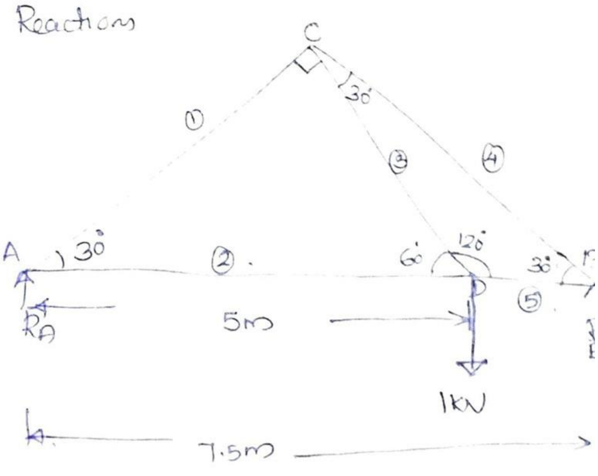
$\therefore F_{DA} = 57.73 + F_{DB}\cos 60 = 57.73 + 115.47 \times \cos 60.$

$= \underline{115.47\,kN}$ (Tensile)

IV (a)

(1). Determination of Support Reactions

$\Sigma V = 0, \ \Sigma H = 0, \ \Sigma M = 0.$



$\Sigma V = 0.$

$R_A + R_B = 1.$ —(1)

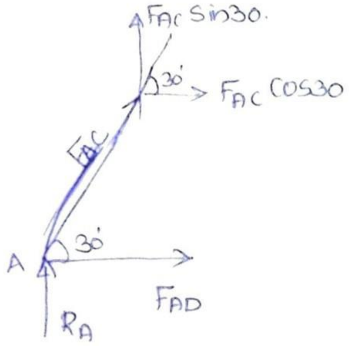
$\Sigma M_A = 0.$

$1 \times 5 = R_B \times 7.5$

$R_B = 5/7.5 = 0.667 \text{ kN}.$

$\therefore R_A = \underline{0.333 \text{ kN}}.$

(2). Consider Joint A.



$\Sigma V = 0.$

$R_A + F_{AC} \sin 30 = 0.$

$F_{AC} = -R_A / \sin 30 = \underline{-0.666 \text{ kN}}.$

$F_{AC} = 0.666 \text{ kN}$ (compressive).

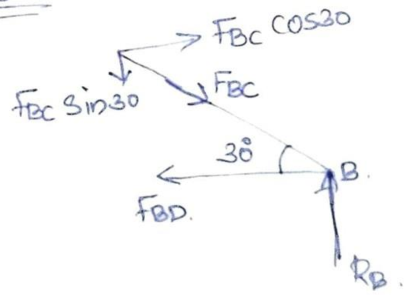
$\Sigma H = 0.$

$F_{AD} + F_{AC} \cos 30 = 0.$

$F_{AD} = -F_{AC} \cos 30 = - \times -0.666 \times \cos 30.$

$= \underline{0.5767} \text{ kN}.$ (Tensile).

(3) Consider Joint B.



$\Sigma V = 0.$

$R_B - F_{BC} \sin 30 = 0.$

$F_{BC} = R_B / \sin 30 = \frac{0.667}{\sin 30}$

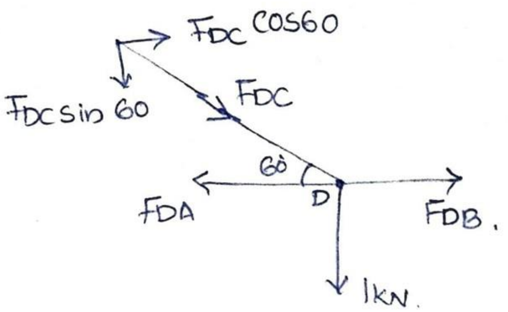
$F_{BC} = \underline{1.334 \text{ kN}}$ (Compressive)

$\Sigma H = 0.$

$F_{BC} \cos 30 = F_{BD}.$

$F_{BD} = 1.334 \times \cos 30 = \underline{1.155 \text{kN}}.$ (Tensile)

(4) Consider Joint D.



$\Sigma V = 0.$

$-1 - F_{DC} \sin 60 = 0.$

$F_{DC} \sin 60 = -1$

$F_{DC} = -1/\sin 60 = \underline{-1.154}$

$\underline{F_{DC} = 1.154}$ (Tensile).

$\Sigma H = 0.$

$F_{DB} - F_{DA} + F_{DC} \cos 60 = 0.$

$1.155 + -1.154 \times \cos 60 = F_{DA}.$

$\underline{F_{DA} = 0.577}$ (Check)

Member.	Force (KN)	Tensile / compressive.
1	0.666.	Compressive
2.	0.577	Tensile.
3.	1.155.	Tensile.
4.	1. 334	Compressive
5.	1.155.	Tensile.

V(a)

$P = 8 kN. = 8000 N.$

$e = 15 mm.$

$d = ?$

$\sigma_{max} = f_{max} = 40 N/mm^2.$

$$\sigma_{max} = \frac{P}{A}\left[1 + \frac{6e}{d}\right).$$

$$40 = \frac{8000}{\frac{\pi}{4} \times d^2}\left[1 + \frac{6 \times 15}{d}\right]$$

$$40 = \frac{4 \times 8000}{\pi \times d^2}\left[1 + \frac{90}{d}\right]$$

$$(0.003925) = \frac{1}{d^2} + \frac{90}{d^3}.$$

$$0.003925 d^3 = d + 90.$$

$$d^3 - 254.77d = 22929.936.$$

V(b)

V(b). $H = 8m.$

$h = 7.5m.$

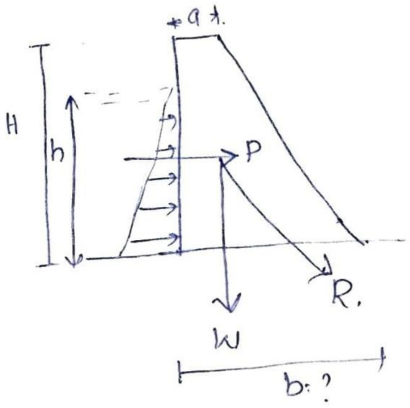
$a = 1m.$

$\rho = 22000 N/m^3 = 22 KN/m^3.$

$w = 9.81 KN/m^3.$

$b = ?$

$\mu = 0.6$



$$P = \frac{w \times h^2}{2} = \frac{9.81 \times 7.5^2}{2} = \underline{275.9 KN}.$$

$$W = \frac{(a+b)}{2} \times H \times \rho$$

For preventing Sliding

$$\frac{\mu W}{P} > 1$$

for desin purpose

$$\frac{\mu W}{P} = 1.5$$

$$\mu W = 1.5 \times P = 1.5 \times 275.9 = \underline{413.86}$$

$$0.6 \times \left(\frac{a+b}{2}\right) \times H \times \rho = 413.86.$$

$$0.6 \left(\frac{1+b}{2}\right) \times 8 \times 22 = 413.86.$$

$$(1+b) \times 52.8 = 413.86.$$

$$1+b = \frac{413.86}{52.8} = 7.83.$$

$$b = \underline{6.83 m}$$

VI(a)

VI(b)

VI (b)

$H = 4m.$

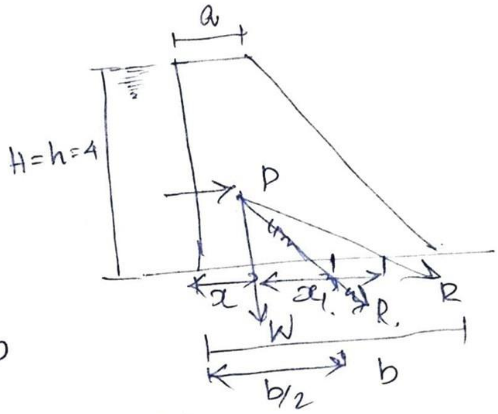
$a = 1m$

$b = 3m$

$\rho = 19.62 kN/m^3$

$w = 9.81 kN/m^3.$

Reservoir is full $h = 4m$



(1) $P = \frac{w \times h^2}{2} = \frac{9.81 \times 4^2}{2} = \underline{78.48} \, kN/m$ length.

(2) $W = \frac{(a+b)}{2} \times H \times \rho = \frac{(1+3) \times 4 \times 19.62}{2} = 156.96$. per m length.

(3) $R = \sqrt{P^2 + W^2} = \sqrt{78.48^2 + 156.96^2} = 175.49 \, kN/m$ length.

(4) $x = \frac{a^2 + b^2 + ab}{3(a+b)} = \frac{1^2 + 3^2 + 1 \times 3}{3(1+3)} = \underline{1.083m}$

(5) $x_1 = \frac{P}{W} \times \frac{h}{3} = \frac{78.48}{156.96} \times \frac{4}{3.} = \underline{0.667 \, m.}$

(6) $d = x + x_1 = \underline{1.7496 \, m}$

(7) $e = d - b/2 = 1.7496 - 1.5 = \underline{0.249}$

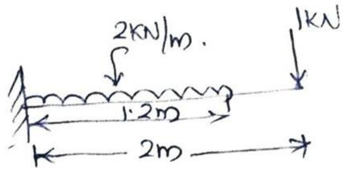
(8). $\sigma_{max} = \frac{W}{b}\left[1 + \frac{6e}{b}\right] = \frac{156.96}{3}\left[1 + \frac{6 \times 0.249}{3}\right]$

$= \underline{78.445 \, kN/m^2}$

(9) $\sigma_{min} = \frac{W}{b}\left[1 - \frac{6e}{b}\right] = \frac{156.96}{3}\left[1 - \frac{6 \times 0.249}{3}\right]$

$= \underline{26.26 \, kN/m^2}.$

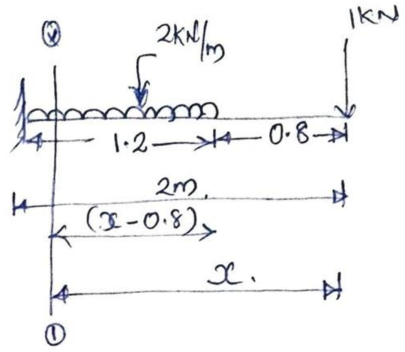
VII(a)



$E = 11 \, kN/mm^2$

$I = 66 \times 10^6 \, mm^4$.

Double integration Method.



$$EI \cdot \frac{d^2y}{dx^2} = M_x$$

$$EI \cdot \frac{d^2y}{dx^2} = -1 \times x - 2(x-0.8)\frac{(x-0.8)}{2}$$

$$EI \cdot \frac{d^2y}{dx^2} = -x - (x-0.8)^2$$

integrating.

$$EI \cdot \frac{dy}{dx} = \frac{-x^2}{2} - \frac{(x-0.8)^3}{3} + C_1$$

Applying Boundary Cond^n.

At Support A $x = 2m$, $\frac{dy}{dx} = 0$.

$$EI \frac{dy}{dx} = \frac{-x^2}{2} - \frac{(x-0.8)^3}{3} + C_1$$

$$0 = \frac{-2^2}{2} - \frac{(2-0.8)^3}{3} + C_1$$

$$C_1 = \underline{\underline{2.576}}$$

$$\therefore EI \frac{dy}{dx} = \frac{-x^2}{2} - \frac{(x-0.8)^3}{3} + 2.576. \quad \text{———} (1)$$

Again integrating eqn (1)

$$EIy = \frac{-x^3}{6} - \frac{(x-0.8)^4}{12} + 2.576x + C_2$$

Applying boundary condn.

At $x = 2m$. $y = 0$.

$$0 = \frac{-2^3}{6} - \frac{(2-0.8)^4}{12} + 2.576 \times 2 + C_2$$

$$0 = -1.333 - 0.17028 + 5.152 + C_2.$$

$$C_2 = \underline{\underline{-3.6462}}$$

$$\therefore EIy = \frac{-x^3}{6} - \frac{(x-0.8)^4}{12} + 2.576x - 3.642.$$

∴ Max. deflection occurs at free end.

$x = 0$.

$EIy = -3.607$

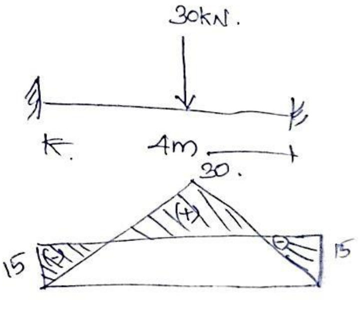
$$y = \frac{-3.607}{E \times I} = \frac{-3.607}{726} = \underline{\underline{0.0049m}}.$$

$y_{max} = \underline{\underline{4.96mm}}$

VII(b)

VII (b)

$EI = 5 \times 10^4 \, kN \cdot m^2$.



Bending Moment $= \frac{WL}{4} = \frac{30 \times 4}{4}$

$= 30kN \cdot m$.

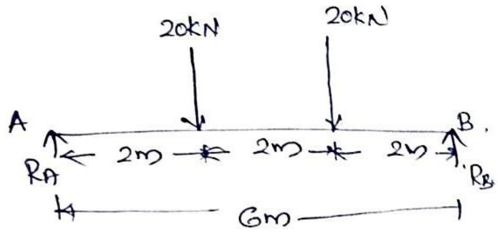
fixing end moments.

$$M_A = \frac{WL}{8} = \frac{30 \times 4}{8} = 15 \, kN \cdot m$$

$$M_B = \frac{WL}{8} = 15kN \cdot m.$$

$$\text{Deflection} = WL^3/192EI = \frac{30 \times 4^3}{192 \times 5 \times 10^4} = \underline{\underline{0.0002m}}$$

VIII(a)



$EI = 20000 kN \cdot m^2$.

By moment Area method.

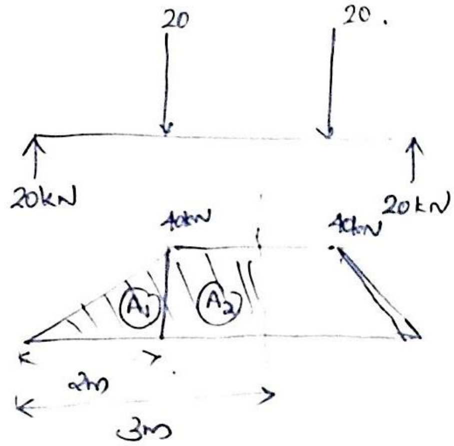
$\Sigma V = 0$.

$R_A + R_B = 40kN$

$\Sigma M_A = 0 \quad 20 \times 2 + 20 \times 4 = R_B \times 6$

$\therefore R_B = 20kN$ & $R_A = 20kN$

Bending moment diagram.



Slope @ midspan of the beam.

$$\theta = \frac{A}{EI}$$

$A = A_1 + A_2$

$A_1 = \frac{1}{2} \times 2 \times 40 = 40$.

$A_2 = 40 \times 1 = 40$.

$A = 80 kN.m$.

$$\theta_{max} = \frac{80}{20000} = \underline{\underline{0.004}} \text{ rad.}$$

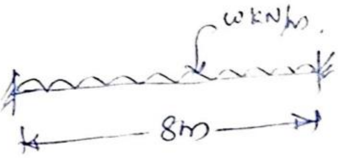
$$y_{max} = \frac{A}{EI} \bar{x}$$

$$\bar{x} = \frac{40 \times 2 \times \frac{2}{3} + 40 \times 2.5}{80}$$

$= \underline{\underline{1.91}}$

$y_{max} = 0.004 \times 1.91 = \underline{\underline{3.66}}$.

VIII(b)



(1) If the maximum bending Moment Shall not exceed 40kNm

$$M_{max} = \frac{WL^2}{8}$$

$$40 = \frac{W \times 8^2}{8}$$

$$W = 40/8 = 5\ kN/m.$$

(2) If the max. deflection shall not exceed 1/400 of the span,

$$y_{max} = \frac{1}{400} \times 8 = 0.02m$$

$$EI = 9.5 \times 10^9\ kN.mm^2$$

$$EI = 9.5 \times 10^9 \times 10^{-6}\ kN.m^2$$

$$= 9.5 \times 10^3\ kN.m^2$$

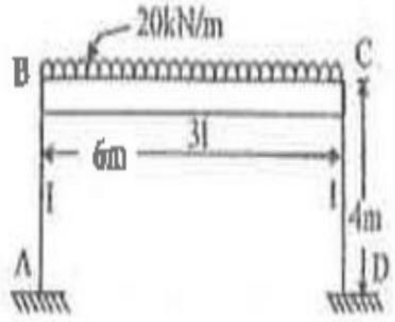
$$y_{max} = \frac{WL^4}{384EI}$$

$$0.02 = \frac{W \times 8^4}{384 \times 9.5 \times 10^3}$$

$$\therefore W = \frac{0.02 \times 384 \times 9.5 \times 10^3}{8^4}$$

$$W = 17.8125\ kN/m.$$

IX



1 FIXED END MOMENTS

Fixed end moments @ AB	0	0
Fixed end moments @ BA	0	0
Fixed end moments @ BC	(wxLxL)/12	60
Fixed end moments @ CB	(wxLxL)/12	-60
Fixed end moments @ CD	0	0
Fixed end moments @ DC	0	0

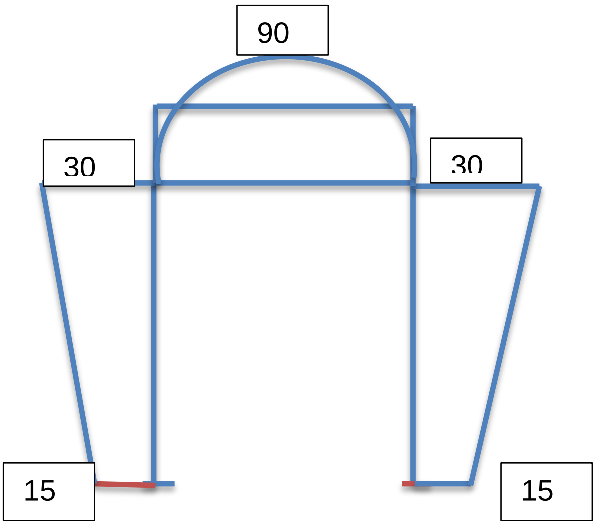
2 FIND DISTRIBUTION FACTOR

JOINT	MEMBER	K		SUM K	DF=K/SUM K
B	BA	I/L	0.25		0.333333333
		3I/L	0.5	0.75	0.666666667
C	CB	3I/L	0.5		0.666666667
	CD	I/L	0.25	0.75	0.333333333

3 MOMENT DISTRIBUTION

JOINT	A	B		C		D
MEMBER	AB	BA	BC	CB	CD	DC
DF		0.3333333	0.66666667	0.6666667	0.33333333	
FEM	0	0	-60	60	0	0
SUM M	0	60		-60		0
SUM M* DF	0	20	40	-40	-20	0
CO = (SUM M* DF)/2	10	0	-20	20	0	-10
SUM M		20		-20		
SUM M* DF	0	6.6666667	13.3333333	-13.333333	-6.6666667	0
CO = (SUM M* DF)/2	3.333333	0	-6.6666667	6.6666667	0	-3.3333333
SUM M		6.666666667		-6.666666667		
SUM M* DF		2.2222222	4.44444444	-4.4444444	-2.2222222	
CO = (SUM M* DF)/2	1.111111		-2.2222222	2.2222222		-1.1111111
SUM M		2.222222222		-2.222222222		
SUM M* DF		0.7407407	1.48148148	-1.4814815	-0.7407407	
CO = (SUM M* DF)/2	0.37037		-0.7407407	0.7407407		-0.3703704
SUM M		0.740740741		-0.740740741		
SUM M* DF		0.2469136	0.49382716	-0.4938272	-0.2469136	
CO = (SUM M* DF)/2	0.123457		-0.2469136	0.2469136		-0.1234568
SUM M		0.24691358		-0.24691358		
SUM M* DF		0.0823045	0.16460905	-0.1646091	-0.0823045	
CO = (SUM M* DF)/2	0.041152		-0.0823045	0.0823045		-0.0411523
FEM	15.0	30.0	-30.0	30.0	-30.0	-15.0

BMD



X.

1 FIXED END MOMENTS

Fixed end moments @ AB	(wxLxL)/12	-45
Fixed end moments @ BA	(wxLxL)/12	45
Fixed end moments @ BC	(wxLxL)/12	-16
Fixed end moments @ CB	(wxLxL)/12	16

2 FIND DISTRIBUTION FACTOR

JOINT	MEMBER	K		SUM K	DF=K/SUM K
B	BA	I/L	0.166666667		0.4
	BC	I/L	0.25	0.416666667	0.6

3 MOMENT DISTRIBUTION

JOINT	A	B		C
MEMBER	AB	BA	BC	CB
DF		0.4	0.6	0
FEM	-45	45	-16	16
SUM M	-45	-29		16
SUM M* DF		-11.6	-17.4	
CO = (SUM M* DF)/2	-5.8	0	0	-8.7
FEM	-50.8	33.4	-33.4	7.3