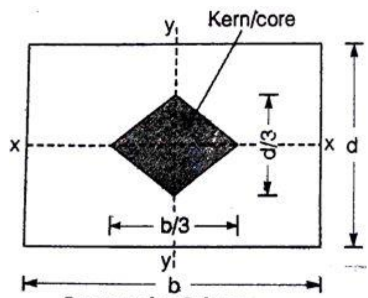
SUMMER– 19 EXAMINATION

Subject Name: Theory of Structures **Model Answer** Subject Code: 17422

Important Instructions to examiners:

1) The answers should be examined by key words 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 errors such as grammatical, spelling errors should not be given more Importance (Not applicable for subject English and Communication Skills.
4) While assessing figures, examiner may give credit for principal components indicated in the figure. The figures 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 some cases, the assumed constant 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 judgement on part of examiner of relevant answer based on candidate's understanding.
7) For programming language papers, credit may be given to any other program based on equivalent concept.

Q. No.	Sub Q. N.	Answer	Marking Scheme
Q.1	(A)	***Attempt any SIX.***	**(12)**
Q.1	A)a) Ans	***Explain the condition for no tension or zero stress at extreme fiber.*** For no tension, magnitude of direct stress must be greater than or equal to the magnitude of bending stress.	**02 M**
Q.1	A)b) Ans	***State the middle third rule.*** Middle third rule: In case of rectangular cross section, if the load is applied at location along the middle third part of both mutually perpendicular axes then the stresses produced are wholly of compressive nature.  **Rectangular Column**	**01 M** **01 M**
Q.1	A)c) Ans	***State the relation between slope and deflection.*** Deflection equation is the integration of slope equation. ∫(Slope) = Deflection	**02 M**
Q.1	A)d) Ans	***State the value of maximum slope and deflection at free end of a cantilever that carries point load at free end.*** Maximum slope at free end = $(\Theta_{max}) = WL^2/2EI$ Maximum deflection at free end= $(y_{max}) = WL^3/3EI$ Where W= Point load L= length (span) of beam(m) E= modulus of elasticity(KN/m^2) I= moment of inertia of a beam m^4	**01 M** **01 M**
Q.1	A)e)	***State advantages 0f fixed beam.***	

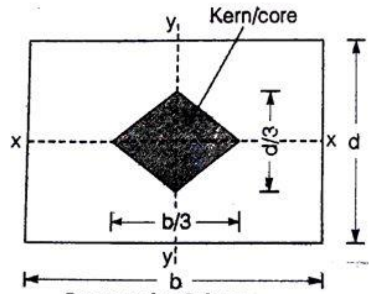
	Ans	**Advantages of fixed beam over simply supported beam:** (1) Due to end fixity, end slope of a fixed beam is zero. (2)A fixed beam is stronger, stiffer and stable. (3) For same span and loading, fixed beam has lesser value of Bending moment. (4) Smaller moment permits smaller sections and there is saving in beam material. (5) Fixed beam has lesser deflection for same span and loading as compared to S.S. beam	**Any Two 01 M for each**
Q.1	A)f) Ans	***Define carry over factor.*** **Carry over factor:-** it is the ratio of moment produce at a joint to the moment applied at the other joint without displacing it.	**02 M**
Q.1	A)g) Ans	***Define distribution factor.*** **Distribution factor: -** it is the ratio of relative stiffness of a member to the total stiffness of all the members meeting at a joint**.**	**02 M**
Q.1	A)h) Ans	***Difference between any two points between Perfect and Imperfect frame.***	**Any two 01 M for each**

Perfect frame	Imperfect frame
It satisfies eqn. m = 2J – 3 M = No. of members. J = No. of joints	It does not satisfy eqn. m = 2J – 3 M = No. of members. J = No. of joints
It is determinant frame.	If m > 2J – 3, then it is redundant frame and If m < 2J – 3, then it is deficient frame.
It can be analyzed using conditions of equilibrium.	It cannot be analyzed only using conditions of equilibrium.

Q.1	A)i) Ans	***Define Redundant frame.*** **Redundant frame:** If the number of members in frame is greater than two times number of members minus 3 [i.e. (n > 2j-3)] then the corresponding frame is called as redundant frame.	**02 M**
Q.1	(B)	***Attempt any TWO.***	**(08)**
Q.1	B)a) Ans	***Calculate limit of eccentricity for rectangular section having width 'b' and depth 'd' and show it on sketch.*** Area A = b x d $Z_{xx} = bd^2 / 6$ $Z_{yy} = db^2 / 6$ Direct stress = 6_d = P / A = P / bd Bending stress $6_{bx} = M_{xx} / Z_{xx}$ $= P \times e_x / (bd^2 / 6)$ $= 6 \times P \times e_x / bd^2$ Bending stress $6_{by} = M_{yy} / Z_{yy}$ $= P \times e_y / (db^2 / 6)$ $= 6 \times P \times e_x / db^2$ For no tension,	**01 M**

$6_d = 6_{bx}$ $P / bd = 6 \times P \times e_x / bd^2$ Hence $e_x = d / 6$	$6_d = 6_{by}$ $P / bd = 6 \times P \times e_x / db^2$ Hence $e_y = b / 6$

02 M

		 **Rectangular Column**	**01 M**
Q.1	B)b) Ans	***Write step by step procedure for determination of minimum and maximum stresses developed at the base of section.*** 1. Calculate area of section 'A' 2. Calculate Moment of Inertia of section about respective axis.(I) 3. Calculate bending moment about respective axis. (P x e) 4. Calculate direct stress = P / A 5. Calculate Bending stress = M x y / I 6. Calculate maximum stress = Direct stress + Bending stress 7. Calculate minimum stress = Direct stress – Bending stress.	**04 M**
Q.1	B)c) Ans	***A solid circular column of diameter 250 mm carries an axial load 'W' kN and a load of 200 kN at an eccentricity of 150 mm. Calculate minimum value of 'W' so as to avoid the tensile stresses at base.*** Axial load = W, eccentric load P = 200 kN., eccentricity e = 150 mm. Area A = $\pi D^2/4$ $= \pi \times 250^2/4$ $= 49087.39\ mm^2$. Moment of Inertia I = $\pi D^4/64$ $= \pi\ 250^4/64$ $= 191747598.5\ mm^4$ $Z = I / y_{max}$ $= 191747598.5 / 125 = 1533980.79\ mm^3$ $M = P \times e = 200 \times 10^3 \times 150 = 3 \times 10^7$ N-mm. Direct stress = 6_d = (W / A) + (P / A) = (W / 49087.39) + (200 x 103 / 49087.39) $= 2.0372 \times 10^{-5}\ W + 4.074$ Bending stress 6_b = M / Z $= 3 \times 10^7 / 1533980.79$ $= 19.56\ N/mm^2$ For no tension, $6_d = 6_b$ $2.0372 \times 10^{-5}\ W + 4.074 = 19.56$ **W = 760161 N.**	 **01 M** **01 M** **01 M** **01 M**
Q.2		**Attempt any FOUR.**	**(16)**
Q.2	a) Ans	***State the slope and deflection at the ends of simply supported beam of span 'L' carrying a udl of w/unit length over entire span.*** Slope at ends = $wL^3 / 24EI$ Deflection at ends = 0	 **02M** **02 M**

Where, E is Modulus of elasticity and I is moment of inertia of the c/s of beam.

Q.2 b) ***Write the equation for slope and deflection at free end for a cantilever beam having u.d.l. over entire span and meaning of terms used in it.***

Ans

Slope at free end = $(\Theta) = wL^3/6EI$ **02 M**

Deflection at free end= $(y) = wL^4/8EI$ **02 M**

Where
w = u.d.l.
L= length (span) of beam(m)
E= modulus of elasticity(KN/m^2)
I= moment of inertia of a beam m^4

Q.2 c) ***Explain step by step procedure of Macaulay's method for finding slope and deflection equation.***

Ans

1. Calculate reactions of beam.
2. Consider any one of the ends of beam as origin and take a section x-x in the last part of the beam from origin.
3. Formulate bending moment equation for section x-x. (It contains macaulay's terms in the form of (x-a), (x-b)….etc.
4. Equating bending moment with $(-)EId^2y/dx^2$
5. Integrating this equation for getting slope equation.
6. Further integrating equation for getting deflection equation.

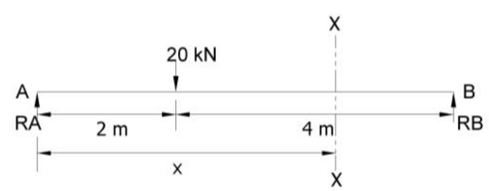
Note: Above both equations contain C1 and C2 (Integration constant)

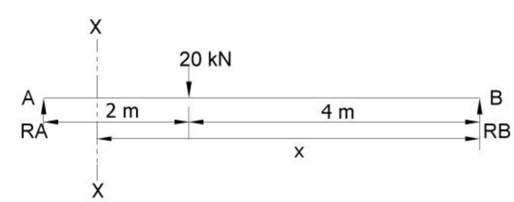
7. Applying end conditions, calculate values of C1 and C2.
8. Putting the values of C1 and C2 in respective equations, get equations for slope and deflection.
9. Put the value of x (distance of section from origin where slope or deflection is required) in respective equation and calculate the value of slope or deflection.

04 M

Q.2 d) ***Find the maximum deflection for a simply supported beam of 6 m span carrying a point load of 20 kN at 2 m from left support as shown in fig. Take E = 2 x 10^6 N/mm^2, I = 2 x 10^7 mm^4.***

Ans





Reactions:

$\Sigma M_A = 0$

$20 \times 2 - R_B \times 6 = 0$

$R_B = 40 / 6$

$= 6.67$ kN. **01 M**

$R_A = 20 - 6.67 = 13.33$ kN.

Taking section X-X at distance 'X' from A	Taking section X-X at distance 'X' from B
$M_x = 13.33 \times X - 20 \times (X-2)$	$M_x = 6.67 \times X - 20 \times (X-4)$
$EId^2y/dx^2 = -Mx$	$EId^2y/dx^2 = -Mx$
$= -13.33 \times X + 20 \times (X-2)$	$= -6.67 \times X + 20 \times (X-4)$
Integrating	Integrating

$EIdy/dx = -13.33 \times X^2/2 + 20 \times (X-2)^2/2 + C_1$
Integrating
$EIy = -13.33 \times X^3/6 + 20 \times (X-2)^3/6 + C_1 \times X + C_2$
At X = 0; y = 0 in EIy eq^n.
$0 = 0 + C_2$
$C_2 = 0$
At X = 6; y = 0 in EIy eq^n.
$0 = -13.33 \times 6^3/6 + 20 \times (6-2)^3/6 + C_1 \times 6 + 0$
$C_1 = 44.42$
Hence $C_1 = 44.42$ and $C_2 = 0$
Slope equation-
$dy/dx = (1/EI)[-13.33 \times X^2/2 + 20 \times (X-2)^2/2 + 44.42]$ -----(01)
Deflection equation-
$y = (1/EI)[-13.33 \times X^3/6 + 20 \times (X-2)^3/6 + 44.42 \times X]$ -------(02)
For maximum deflection, slope shall be zero.
$0 = -13.33 \times X^2/2 + 20(X-2)^2/2 + 44.42$
Hence X = 2.73 m. from A.
Put X = 2.73 in eq. 02
$y_{max} = (1/EI)[-13.33 \times 2.73^3/6 + 20 \times (2.73-2)^3/6 + 44.42 \times 2.73]$
= **77.37/EI.**
$EI = 2 \times 10^6 \times 2 \times 10^7 = 4 \times 10^{13}$ N-mm²
$= 4 \times 10^{13} \times 10^{-9}$ kN-m²
$= 4 \times 10^4$ kN-m²
Ymax = $77.37 / 4 \times 10^4$ **= 1.93×10^{-3} m.**
= 1.93 mm.

$EIdy/dx = -6.67 \times X^2/2 + 20 \times (X-4)^2/2 + C_1$
Integrating
$EIy = -6.67 \times X^3/6 + 20 \times (X-4)^3/6 + C_1 \times X + C_2$
At X = 0; y = 0 in EIy eq^n.
$0 = 0 + C_2$
$C_2 = 0$
At X = 6; y = 0 in EIy eq^n.
$0 = -6.67 \times 6^3/6 + 20 \times (6-4)^3/6 + C_1 \times 6 + 0$
$C_1 = 35.56$
Hence $C_1 = 35.56$ and $C_2 = 0$
Slope equation-
$dy/dx = (1/EI)[-6.67 \times X^2/2 + 20 \times (X-4)^2/2 + 35.56]$ -----(01)
Deflection equation-
$y = (1/EI)[-6.67 \times X^3/6 + 20 \times (X-4)^3/6 + 35.56 \times X]$ -------(02)
For maximum deflection, slope shall be zero.
$0 = -6.67 \times X^2/2 + 0 + 35.56$
Hence X = 3.27 m. from B.
Put X = 3.27 in eq. 02
$y_{max} = (1/EI)[-6.67 \times 2.73^3/6 + 0 + 35.56 \times 3.27]$
= **77.37/EI.**
$EI = 2 \times 10^6 \times 2 \times 10^7 = 4 \times 10^{13}$ N-mm²
$= 4 \times 10^{13} \times 10^{-9}$ kN-m²
$= 4 \times 10^4$ kN-m²
Ymax = $77.37 / 4 \times 10^4$ **= 1.93×10^{-3} m.**
= 1.93 mm.

02 M

01 M

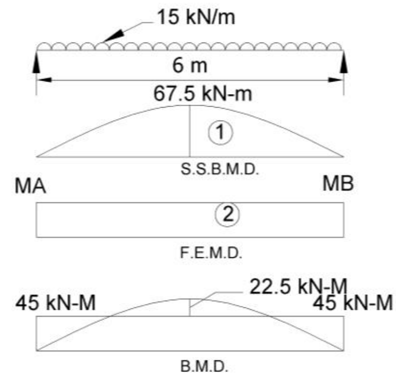
Q.2 e) ***Explain principle of superposition with respect to fixed beam.***

Ans In this, first fixed beam is split in to two simply supported beams. One having loading and other has end moments. Then both the beams are analyzes independently. After that the solutions are combined together for getting the solution of fixed beam.

04 M

Q.2 f) ***A fixed beam of span 6 m carries an udl of 15 kN/m over entire span. Find fixed end moment from first principle and draw B.M.D.***

Ans SSBM = $15 \times 6^2 / 8 = 67.5$ kN-m.
Area of SSBMD = $a_1 = 2 \times 6 \times 67.5 / 3 = 270$
Due to symmetry, $M_A = M_B$
Area of FEMD = $a_2 = M_A \times 6 = 6M_A$
Condition 1, Area of SSBMD = area of FEMD.
$270 = 6M_A$
MA = 270 / 6 = 45 kN-m.
And MB = 45 kN-m.
Net B.M at mid-span = 67.5 – 45 = 22.5 kN-m.

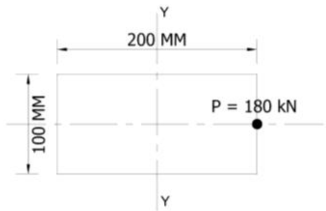


02 M for calculations.
02 M for dia.

Q.3 Attempt any FOUR. (16)

Q.3 a) *A rectangular column is 200 mm wide and 100 mm thick. It carries a load of 180 kN at an eccentricity of 100 mm in the plane bisecting thickness. Find the maximum and minimum intensities of stress in section.*

Ans



$A = 200 \times 100 = 2 \times 10^4$ mm^2

$I_{yy} = 100 \times 200^3 / 12 = 6.67 \times 10^7$ mm^4 — **01 M**

$Z_{xx} = 6.67 \times 10^7 / 100 = 6.67 \times 10^5$ mm^3

$M = P \times e = 180 \times 10^3 \times 100 = 18 \times 10^6$ N-mm.

Direct stress = 6_d = P / A

= $180 \times 10^3 / 2 \times 10^4$ — **01 M**

= 9 N/mm^2

Bending stress 6_b = M / Z

= $18 \times 10^6 / 6.67 \times 10^5$ — **01 M**

= 27 N/mm^2

Maximum stress = $6_d + 6_b$

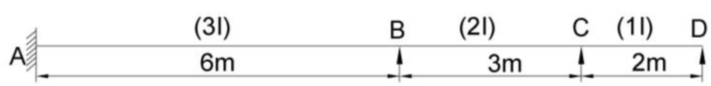
= 9 + 27 = 36 N/mm^2 (Comp.) — **01 M**

Minimum stress = $6_d - 6_b$

= 9 - 27 = (-)18 N/mm^2 i.e. 18 N/mm^2 (tensile)

Q.3 b) *Determine distribution factor at continuity for a continuous beam ABCD which is fixed at A and simply supported at B, C and D. Take AB = 6 m, BC = 3 m and CD = 2 m. If M.I. for span is I_{AB} = 3I, I_{BC} = 2I and I_{CD} = 1I.*

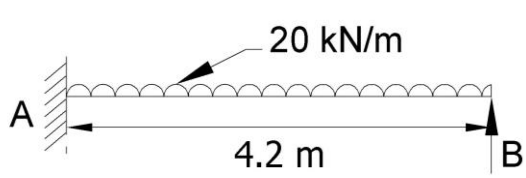
Ans



Joint	Member	K	ΣK	Dist. Factor	Marks
	BA	4x3EI/6 = 2EI	4.67EI	2EI/4.67EI = 0.43	01 M
	BC	4x2EI/3 = 2.67EI		2.67EI/4.67EI = 0.57	01 M
C	CB	4x2EI/3 = 2.67EI	4.167EI	2.67EI/4.167EI = 0.64	01 M
	CD	3x1EI/2 = 1.5EI		1.5EI/4.167EI = 0.36	01 M

Q.3 c) *A propped cantilever AB of span 4.2 m is fixed at A and propped at B, carrying UDL of 20 kN/m. Using clapeyron's theorem, calculate support moment and draw BMD.*

Ans



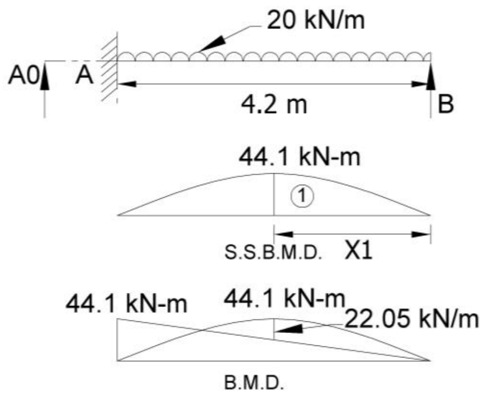
Consider zero span at A.

S.S.B.M. = $20 \times 4.2^2 / 8 = 44.1$ kN-m. — **01 M**

$a_1 = 2 \times 44.1 \times 4.2 / 3 = 123.48$

$x_1 = 2.1$ m.

$M_B = 0$ (End simple support)



Using three moment theorem;

$M_{A0} \times L_0 + 2M_A(L_0 + L_1) + M_B \times L_1 = -[(6 \times a_0 \times \bar{x}_0/L_0) + (6 \times a_1 \times \bar{x}_1/L_1)]$

$0 + 2M_A(0 + 4.2) + 0 = -[(0) + (6 \times 123.48 \times 2.1/4.2)]$

$8.4M_A = -(0 + 370.44)$

$M_A = -370.44 / 8.4$

$= -44.1$ kN-m

02 M

01 M for BMD

Q.3 d) *State the method of analysis of frame.*

Ans

There are two types of frames.

1. Portal frames
2. Pin jointed frames (Trusses)

1. Methods for analysis of portal frames.
   - i. Moment distribution method
   - ii. Clapeyron's theorem of three moments.
   - iii. Strain energy method.
   - iv. Column analogy method.
   - v. Rotation contribution method.
   - vi. Matrix method.
2. Methods for analysis of pin jointed frames.
   - i. Section method
   - ii. Joint method.
   - iii. Matrix method.

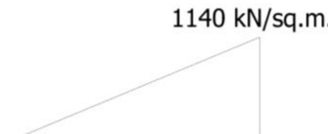
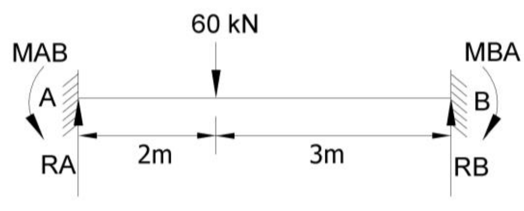
04 M

Q.3 e) *Differentiate between symmetrical and unsymmetrical portal frame.*

Ans

Symmetrical portal frame	Unsymmetrical portal frame
It has same pattern of loading on both left and right part of the axis of symmetry.	It has different pattern of loading on both left and right part of the vertical axis..
It has same pattern of geometry of the frame on both left and right part of the axis of symmetry.	It has different pattern of geometry of the frame on both left and right part of the vertical axis.
It has no sway.	It has sway.
It has same parameters like moment, slope, deflection, reaction etc. on both sides of axis of symmetry.	It has different parameters like moment, slope, deflection, reaction etc. on both sides of vertical axis.

04 M

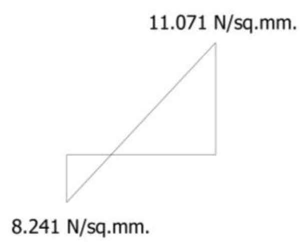
Q.3	f) Ans	***State Clapeyron's theorem of three moments and give meaning of each terms involved.*** For a two span continuous beam having uniform moment of inertia, supported at ends A, B and C subjected to any external loading , the support moments MA, MB and MC at the supports A,B and C respectively are given by the relation $M_AL_1+ 2M_B(L_1+L_2)+M_CL_2 = -(6a_1x_1/L_1+6a_2x_2/L_2)$ Where L_1= length of span AB L_2= length of span BC a_1 = area of free BMD for the span AB a_2 = area of free BMD for the span BC x_1= distance of C.G. of free BMD over the span AB from Left end A x_2= distance of C.G. of free BMD over the span BC from right end C	**02M** **02M**
Q.4		***Attempt any TWO.***	**(16)**
Q.4	a) Ans.	***A circular chimney has external diameter 60% more than internal diameter. The height of chimney is 30 m and is subjected to a horizontal wind pressure of 1.70 kN/m². Find out the diameter of chimney so as to avoid tension at the base of chimney and also draw stress distribution diagram. Unit weight of chimney material is 19 kN/m³ and C = 0.60.*** h = 30 m, p = 1.7 kN/m2, D = 1.6d OR d = 0.625D, Y = 19 kN/m³, C = 0.6 Direct stress = 6_d = Yh = 19 x 30 = 570 kN/m² **(01 M)** M = C x p x D x h² / 2 = 0.6 x 1.7 x D x 30² / 2 = 459D **(01 M)** Moment of Inertia I = π (D⁴ – d⁴)/64 = π [D⁴ – (0.625D)⁴]/64 **(01 M)** = 0.0416D⁴ mm⁴ Bending stress 6_b = (M x D) /2 x I = (459D x D) / 2 x 0.0416D⁴ **(01 M)** = 5516.83 / D² For no tension, 6_d = 6_b 570 = 5516.83 / D² D = 3.111 m **(02 M)** And d = 0.625D = 0.625 x 3.111 = 1.944 m. 6_{max} = 2 x 6_d = 2 x 570 = 1140 kN/m² (Comp.) 6_{min} = 0 	**01 M** **01 M** **01 M** **01 M** **02 M** **02 M for stress dia**
Q.4	b) Ans	***A fixed beam 5 m long carries a load of 60 kN at 2 m from left end. Calculate the fixed end moments, net B. M. under the load and end reactions.***  M_{AB} = Wab² / l² = 60 x 2 x 3² / 5² = 43.2 kN-m. M_{BA} = Wa²b / l² = 60 x 2² x 3 / 5² = 28.8 kN-m. Calculation of reactions. ΣM_A = 0 = 60 x 2 + 28.8 – 43.2 – R_B x 5	**02 M** **02 M**

R_B = 21.12 kN. **02 M**
R_A = 60 – 21.12 = 38.88 kN.
B. M. under point load = 21.12 x 3 – 28.8
= 34.56 kN-m. **02 M**

Q.4 c) ***A C.I. hollow circular column section has external diameter 250 mm and internal diameter 200 mm. It is subjected to a vertical load of 25 kN at a distance of 350 mm from the vertical axis of column. Calculate the maximum and minimum stresses at the base of the column and draw stress diagram.***

Ans

P = 25 kN, e = 350 mm.
Area A = π $(D^2 – d^2)/4$ **01 M**
= π x $(250^2 – 200^2)/4$
= 17671.46 mm^2.
Moment of Inertia I = π $(D^4 – d^4)/64$
= π $(250^4 – 200^4)/64$ **01 M**
= 113267782.1 mm^4
Z = I / y_{max}
= 113267782.1/ 125 = 906142.26 mm^3
M = P x e = 25 x 10^3 x 350 = 875 x 10^4 N-mm.
Direct stress = 6_d = P / A
= 25 x 10^3 / 17671.46 **01 M**
= 1.415 N/mm^2
Bending stress 6_b = M / Z
= 875 x 10^4 / 906142.26
= 9.656 N/mm^2 **01 M**
Maximum stress = 6_d + 6_b
= 1.415 + 9.656 = 11.071 N/mm^2 (Comp.) **01 M**
Minimum stress = 6_d - 6_b
= 1.415 - 9.656 = (-)8.241 N/mm^2 i.e. 8.241 N/mm^2 (tensile) **01 M**

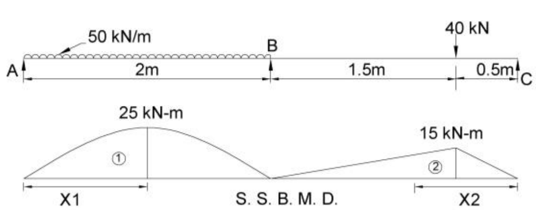


02 M

Q.5 ***Attempt any TWO.*** **(16)**

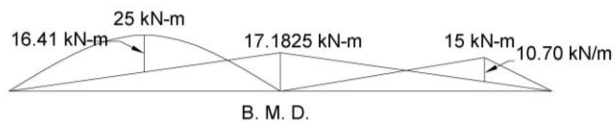
Q.5 a) ***A continuous beam ABC is simply supported at A, B and C such that AB = BC = 2 m. Span AB carries a u.d.l. of 50 kN/m. from A to B, span BC carries a point load of 40 kN at 0.5 m from C. Draw bending moment diagram and calculate support reactions. Use Clapyeron's theorem of moment.***

Ans



S.S.B.M. for AB = 50 x 2^2 / 8 = 25 kN-m.
S.S.B.M. under load 40 kN = 40 x 1.5 x 0.5 / 4 **01 M**
= 15 kN-m.

a_1 = 2 x 2 25 / 3 = 33.33 x_1 = 1 m
a_2 = 0.5 x 2 x 15 = 15.0 x_2 = (2 + 0.5) / 3 = 0.833 **01 M**
$M_A = M_B = 0$ (End simple supports)
Applying Clapeyron's theorem for span AB and BC
$M_AL_1 + 2M_B(L_1+L_2)+M_CL_2 = -6(a_1x_1/L_1+a_2x_2/L_2)$
$0 + 2M_B(2 + 2) + 0 = -6[(33.33 \times 1 /2) + (15 \times 0.833 / 2)]$ **02 M**
$8M_B$ = (-) 137.5
M_B = (-) 17.1875 kN-m.

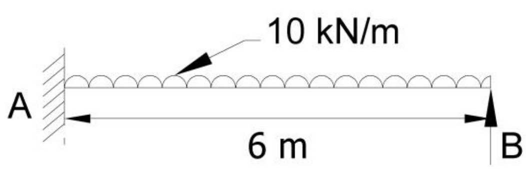


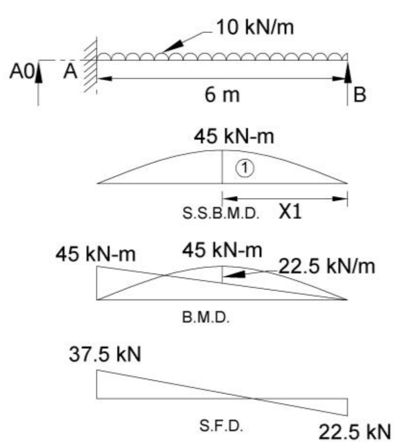
02 M

Calculation of reactions.
$\Sigma M_B = 0$ (Consider left side)
$R_A \times 2 - 50 \times 2 \times 1 + 17.1875 = 0$
R_A = 41.41 kN. **02 M**
$\Sigma M_B = 0$ (Consider right side)
$-R_c \times 2 + 40 \times 1.5 - 17.1875 = 0$
R_C = 21.41 kN.
RB = 50 x 2 + 40 – 41.41 – 21.41 = 77.18 kN.

Q.5 b) ***A propped cantilever of span 6 m carries a u.d.l. of 10 kN/m over the entire span. Prop is free end. Calculate the fixed end moments using Clapeyron's theorem of three moments. Also draw SFD and BMD.***

Ans





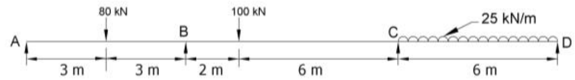
Consider zero span at A.
S.S.B.M. = 10 x 6^2 / 8 = 45 kN-m.
a_1 = 2 x 45 x 6 / 3 = 180 **02 M**
x_1 = 3 m.
$M_B = 0$ (End simple support)

Using three moment theorem;
$M_{A0} \times L_0 + 2M_A(L_0 + L_1) + M_B \times L_1 = -[(6 \times a_0 \times x_0/L_0) + (6 \times a_1 \times x_1/L_1)]$

		$0 + 2M_A(0 + 6) + 0 = -[(0) + (6 \times 180 \times 3/6)]$ $12M_A = -(0 + 540)$ $M_A = -540 / 12$ $= -45$ kN-m Calculation of reactions. $\Sigma M_A = 0$ $-R_B \times 6 + 10 \times 6 \times 3 - 45 = 0$ $R_B = 22.5$kN. and $R_A = 10 \times 6 - 22.5 = 37.5$ kN. Shear force calculations. At B just right = 0 At B just left = – 22.5 kN At A just right = –22.5 + 10 x 6 = 37.5 kN. At A just left = 0	**02 M** **02 M** **01 M for BMD & 01 M for SFD**

Q.5 c) ***A continuous beam ABCD is loaded as shown in fig No. 2. Using moment distribution method, find the support moments and draw B.M.D.***

Ans



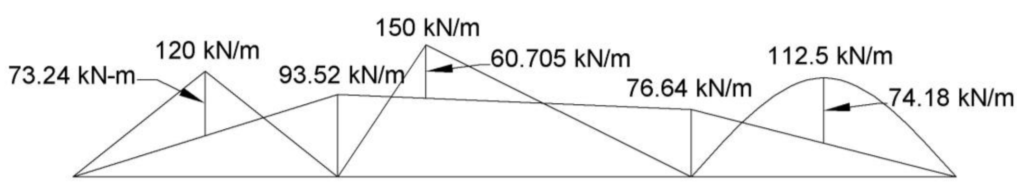
M_{AB} = – 80 x 6 / 8 = – 60.0 kN-m M_{BA} = 80 x 6 / 8 = 60.0 kN-m
M_{BC} = – 100 x 2 x $6^2/8^2$ = – 112.5 kN-m M_{BC} = 100 x 2^2 x $6/8^2$ = 37.5 kN-m   **02 M**
M_{CD} = – 25 x 6^2/12 = – 75 kN-m M_{DC} = 25 x 6^2/12 = 75 kN-m

Joint	Member	Stiffness (k)	Σk	D.F. = k/Σk
B	BA	3 x EI/6 = 0.5EI	1.0EI	0.5EI/1.0EI = 0.5
	BC	4 x EI/8 =0.5EI		0.5EI/1.0EI = 0.5
C	CB	4 x EI/8 =0.5EI	1.0EI	0.5EI/1.0EI = 0.5
	CD	3 x EI/6 = 0.5EI		0.5EI/1.0EI = 0.5

02 M

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
Dist. Fact.	1.0	0.5	0.5	0.5	0.5	1.0
FEM	– 60	60	– 112.5	37.5	– 75	75
Bal.	60	26.25	26.25	18.75	18.75	– 75
C.O.		30	9.375	13.125	- 37.5	
Bal.		– 19.6875	– 19.6875	12.1875	12.1875	
C.O.			6.09	– 9.84		
Bal.		– 3.045	– 3.045	4.92	4.92	
Final Moments	0	93.5175	– 93.5175	76.6425	– 76.6425	

02 M



B.M.D.

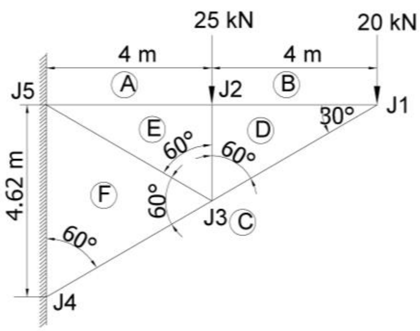
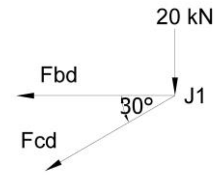
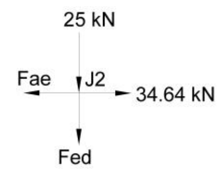
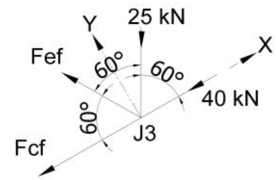
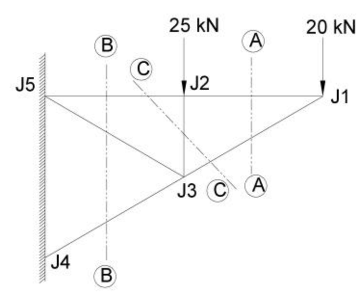
02 M

Q.6 ***Attempt any TWO.*** **(16)**

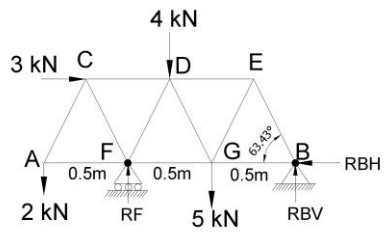
a) ***Find out the forces in the member by method of section.***

Ans

NOTE : If students attempted to solve considering appropriate figure/ data, give marks accordingly.

Q.6	b) Ans	***Find out the forces in the members of a cantilever truss as shown in fig. No. 3.***	
		 **Joint Method** Joint J1 $\Sigma V = 0 = -20 - Fcd \sin 30$ $Fcd = -40$ i.e. 40 kN (C) $\Sigma H = 0 = - Fbd - Fcd \cos 30$ $Fbd = - (-40 \cos 30)$ $Fbd = 34.64$ kN (T) 	03 M
		Joint J2 $\Sigma V = 0 = -25 - Fed$ $Fed = -25$ i.e. 25 kN (C) $\Sigma H = 0 = - Fae + Fbd$ $Fae = 34.64$ kN (T) 	02 M
		Joint J3 $\Sigma Y = 0 = Fef \sin 60 - Fed \sin 60$ $Fef = Fed$ $Fef = 25$ kN (T) $\Sigma X = 0 = - Fcd - Fcf - Fef \cos 60 - Fed \cos 60$ $= -40 - Fcf - 25 \cos 60 - 25 \cos 60$ $Fcf = -65$ i.e. 65 kN (C) 	03 M
		Section Method	**OR**
		 Assuming all forces tensile in nature. Section A-A $\Sigma MJ1 = 0 = - Fbd \times 2.31 + 20 \times 4$ $Fbd = 34.64$ kN (T) $\Sigma H = 0 = - Fbd - Fcd \cos 30$ $Fcd = - 34.64/\cos 30$ $= -40$ i.e 40 kN (C)	03 M
		Section B-B $\Sigma MJ3 = 0 = 20 \times 4 - Fae \times 2.31$ $Fae = 34.64$ kN (T) $\Sigma MJ5 = 0 = 25 \times 4 + 20 \times 8 + Fcf \sin 60 \times 4.62$ $Fcf = - 65$ i.e. 65 kN (C) $\Sigma MJ1 = 0 = -25 \times 4 + Fef \cos 60 \times 4 + Fef \sin 60 \times 2.31$ $Fef = 25$ kN (T)	03 M
		Section C-C $\Sigma V = - 25 - Fed$ $Fed = - 25$ i.e. 25 kN (C)	02 M
Q.6	c)	***A truss is loaded as shown in fig. 4 Determine the nature and magnitude of truss forces in the members BE, BG and ED. Use method of joint only.***	

Ans



Tan(θ) = 0.5 / 0.25 **02 M**

θ = 63.43^{0}

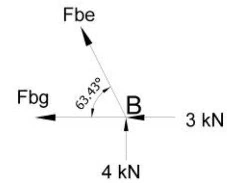
Reactions.

ΣM_B = -2 x 1.5 + 3 x 0.5 – 4 x 0.75 – 5 x 0.5 + R_F x 1 = 0

R_F = 7 kN. **02 M**

R_{BV} = 2 + 4 + 5 – 7 = 4 kN.

R_{BH} = 3 kN.



Joint B

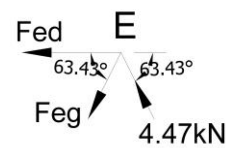
ΣV = Fbe Sin63.43 + 4 = 0

Fbe = - 4.47 i.e 4.47 kN (C) **02 M**

ΣH = - Fbg - Fbe Cos63.43 - 3 = 0

Fbg = - 1.0 i.e 1.0 kN (C)

Joint E



ΣV = - Feg Sin63.43 + 4.47 Sin63.43 = 0

Feg = 4.47 kN (T)

ΣH = - Fed - Feg Cos63.43 – 4.47 Cos63.43 = 0 **02 M**

= - Fed – 4.47 Cos63.43 – 4.47 Cos63.43

Fed = - 4 i.e 4.0 kN (C)