MAHARASHTRA STATE BOARD OF TECHNICAL EDUCATION
(Autonomous)
(ISO/IEC - 27001 - 2005 Certified)

WINTER – 14 EXAMINATIONS

Subject Code: 17311 **Model Answer** **Total Pages: 33**

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.

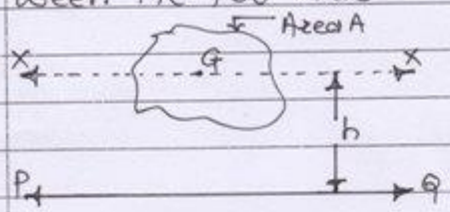
4) While assessing figures, examiners, may give credit for principle components indicated in the figure.

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.

Q.NO	SOLUTION	MARKS
Q.1		
a)	Solve any six of the following:-	12
(i)	State parallel axis theorem along with it's expression.	
	STATEMENT:⇒ "The moment of inertia of a plane section about any axis parallel to the centroidal axis is equal to the moment of inertia of the section about the centroidal axis plus the product of the area of the section and the square of the distance between the two axis"	1M
	 $I_{PQ} = I_G + Ah^2$.	1M
(ii)	Find radius of Gyration of circle of diameter d.	
	∴ Diameter of circle :- d.	
	Moment of Inertia $= \frac{\pi}{64} d^4$	
	Area of circle $= \frac{\pi}{4} d^2$	
	∴ Radius of Gyration $= K = \sqrt{\frac{I}{A}} = \sqrt{\frac{\pi/64\, d^4}{\pi/4\, d^2}}$	1M
	∴ $\boxed{K = d/4}$	1M

Q.NO 1 (a)	SOLUTION	MARKS
(iii)	Define Creep.	
	Creep = "The contineous deformation with time which the material undergoes due to application of external steady loads is called creep.	2 M
	OR	
	At higher stress level strain will increases with time at constant load is called creep.	2 M
(iv)	State the Hook's Law along with Expression.	
	Hook's Law = " When a material is loaded within its elastic limit, the stress produced is directly proportional to the strain	1 M
	$\sigma \propto e$	
	$\frac{\sigma}{e}$ = constant (E)	1 M
(v)	List any two assumptions made in Euler's theory of Long column.	1 M each (Any Two)
	1) The material of the column is homogeneous and isotropic	
	2) The section of the column is uniform throughout	

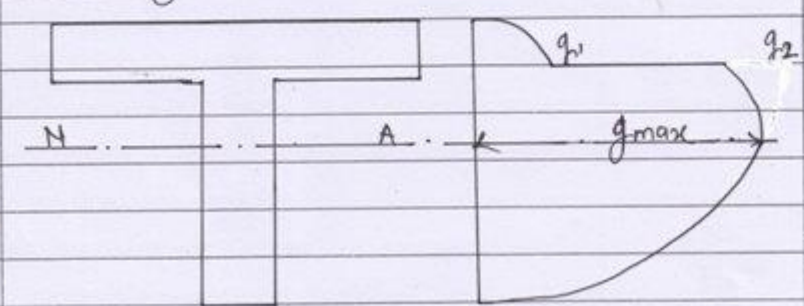
Q.NO	SOLUTION	MARKS
1 (a) (v) cont---	3) The column is initially straight and is loaded axially.	[Any Two 1M Each]
	4) The column fails by buckling alone.	
	5) The self weight of the column is negligible.	
(vi)	What are the limitations of Euler's theory of column.	
	⇒ Euler's formula is based on the assumptions that column fails only by buckling. For long columns with higher values of L/K, since in such cases bending stresses are quite high compared to compressive stresses. But there is range of slenderness ratio for which direct compressive stresses and bending stresses are comparable. In such cases the failure will be neither by crushing only nor by buckling alone. Euler's theory fails to take care of such cases properly.	2 M
	OR	OR
	In case of Mild steel column with both End hinged if slenderness ratio is less than 80, then the Euler's formula will not be valid. $\therefore \frac{le}{K_{min}} \leq 80$ or 78.48.	2M

Q.NO	SOLUTION	MARKS
1 (a)		
(vii)	Define resilience along with it's expression	
	RESILIENCE :⇒ The Total Strain Energy Stored in material, within its elastic limit is known as resilience. It is denoted by U	1M
	$\therefore U = \frac{\sigma^2}{2E} \cdot V$.	1M
	Where.	
	U = Resilience (Strain Energy Stored)	
	σ = Stress within elastic limit.	
	E = modulus of elasticity	
	V = volume = A x h.	
(viii)	Differentiate between gradual load and suddenly applied load with respect to.	

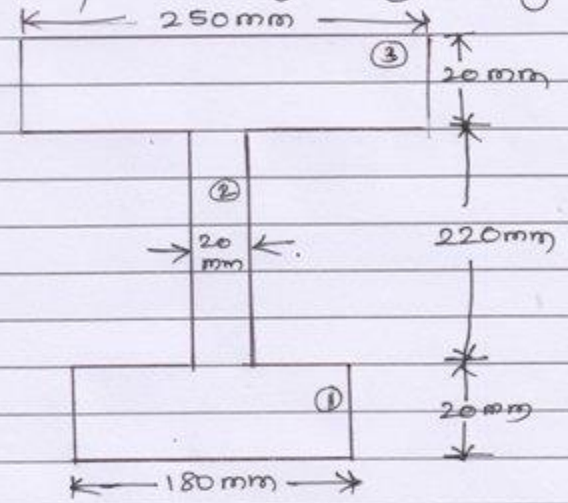
Type of load	Stress produced	Example.	Marks
1) Gradually applied load.	$\sigma = P/A$	Loading on material from zero onwards.	1M
2) Suddenly applied load.	$\sigma = 2P/A$.	The load applied on weighing m/c	1M

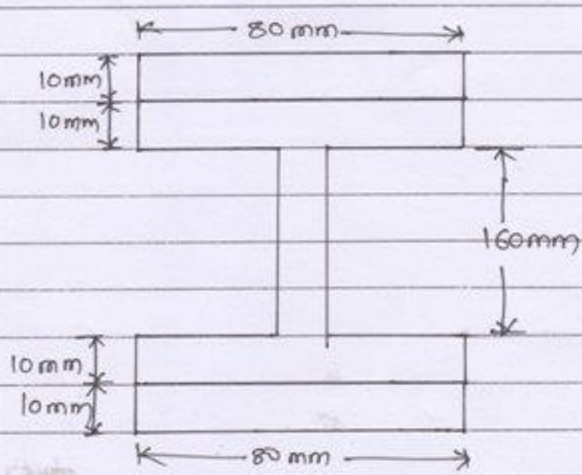
Q.NO	SOLUTION	MARKS
Q. 1		
b)	Solve any TWO of the following.	8
(i)	State four assumptions made in theory of pure bending.	
	1) The beam is initially straight and every layer of it is free to expand or contract.	[Any four 1M each]
	2) The material is homogeneous and isotropic	
	3) Young's Modulus is same in tension and compression.	
	4) Stresses are within elastic limit.	
	5) Plane section remains plane even after bending	
	6) The radius of curvature is large compared to depth of beam.	
(ii)	(1) Give shear stress equation and meaning of each term used in it.	
	τ or $q = \frac{SA\bar{y}}{Ib}$ or $\frac{FA\bar{y}}{Ib}$	1M
	Where	
	τ or q = Intensity of shear stress	
	A = Area of the beam above the layer consideration.	
	$\bar{y}$ = Distance of C.G. of the area considered from N.A.	

S or F = Shear force at a section.

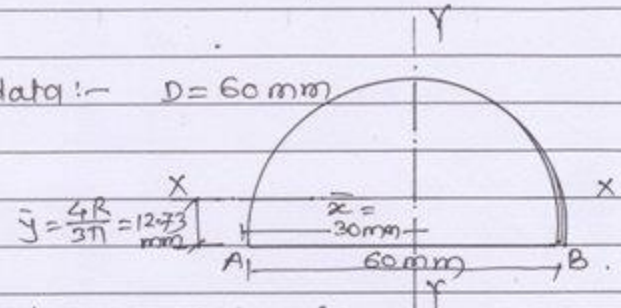
Q.NO	SOLUTION	MARKS
1 (b) (ii) cont...	I = Moment of Inertia about Neutral axis. b = width of the section at a distance y from the N.A.	1M (for meaning)
	* ② Draw shear stress diagram for 'T' section showing important point on it.	
		2M
(iii)	A Column having diameter 200mm and length 3m. Both end of column is hinged. Find Euler's crippling load. Take $E = 2 \times 10^5$ Mpa. Given data:- $D = 200$ mm $L = 3m = 3000$ mm $E = 2 \times 10^5$ Mpa $= 2 \times 10^5$ N/mm^2	

Q .NO	SOLUTION	MARKS
1 (b)	End condition:- both Ends hinged.	
(iii)	$\therefore$ Effective length $Le = L = 3000\,mm$	1M
cnt.--	Soln.	
	Area of circle $= A = \frac{\pi}{4} D^2$	
	$= \frac{\pi}{4} (200)^2$	
	$= 31.416 \times 10^3\, mm^2$	$\frac{1}{2}$ M
	Moment of Inertia.	
	$Ixx = Iyy = Imin = \frac{\pi}{64} D^4.$	
	$= \frac{\pi}{64} (200)^4 = 78.540 \times 10^6\, mm^4.$	$\frac{1}{2}$ M
	Euler's Crippling load	
	$Pe = \frac{\pi^2 E\, Imin}{(le)^2}$	1M
	$= \frac{\pi^2 \times 2 \times 10^5 \times 78.540 \times 10^6}{(3000)^2}$	
	$Pe = 17.225 \times 10^6\, N$	
	$Pe = 17.225 \times 10^3\, KN.$	1M

Q .NO	SOLUTION	MARKS
Q.2.	Solve any Two of the following.	
(a)		
	① Centre of Gravity of section.	
	Due to symmetry @ Y-Y axis.	
	$\bar{x} = \frac{250}{2} = 125\ mm.$	1M
	$\therefore \bar{y} = \frac{a_1y_1 + a_2y_2 + a_3y_3}{a_1 + a_2 + a_3}$	
	$\therefore \bar{y} = \frac{[180 \times 20 \times 10] + [20 \times 220 \times 130] + [250 \times 20 \times 250]}{[(180 \times 20) + (20 \times 220) + (250 \times 20)]}$	
	$\bar{y} = 142.92\ mm.$	1M
	② Moment of Inertia.	
	(a) About x-x. axis.	
	$I_{xx} = I_{xx_1} + I_{xx_2} + I_{xx_3}$	
	$= I_{G1} + A_1h_1^2 + I_{G2} + A_2h_2^2 + I_{G3} + A_3h_3^2$	1M

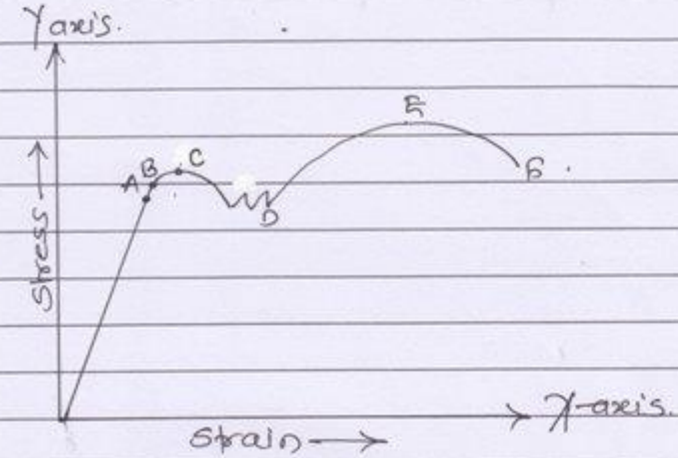
Q.NO	SOLUTION	MARKS
2 (a)	$h_1 = 142.92 - 10 = 132.92$ mm	
2 (a)	$h_2 = 142.92 - 130 = 12.92$ mm	
cont--	$h_3 = 250 - 142.92 = 107.08$ mm.	1M
	$I_{xx} = [(120\times10^3)+63.60\times10^6] + [(7.75\times10^6)+734.47\times10^3)$	
	$[(166.67\times10^3+57.33\times10^6)]$	
	$\therefore I_{xx} = 139.40\times10^6$ mm^4.	2M
	Due to Symmentry	
	$I_{yy} = I_{G1} + I_{G2} + I_{G3}$	
	$= [9.72\times10^6 + 146.67\times10^3 + 26.04\times10^6]$	
	$I_{yy} = 35.908\times10^6$ mm^4.	2M
(b)		
	Due to Symmentry about both the axis.	
	centre of Gravity.	
	$\bar{x} = \frac{80}{2} = 40$mm, $\bar{y} = \frac{20+160+20}{2} = 100$mm	1M

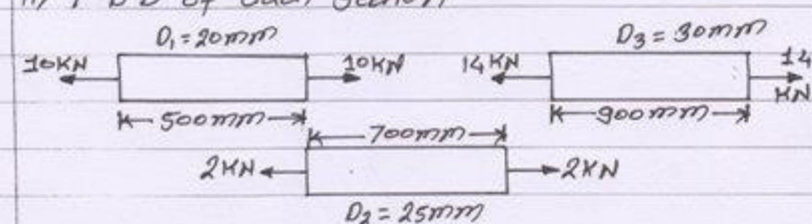
Q.NO	SOLUTION	MARKS
2(b) conf..	Moment of Inertia about x-x axis.	
	$I_{xx} = \frac{BD^3}{12} - \frac{bd^3}{12}$	1M
	$\therefore B = 80\,mm,\ D = 200\,mm$	
	& $b = 80 - 10 = 70\,mm$	
	$d = 200 - 40 = 160\,mm$.	1M
	$\therefore I_{xx} = \frac{80\times200^3}{12} - \frac{70\times160^3}{12}$	
	$= 29.44\times10^6\,mm^4$.	1M
	Moment of Inertia about y-y axis.	
	$I_{yy} = 2\times$ M.I. of flanges + M.I. of web.	1M
	$= \left[2\times\frac{20\times80^3}{12}\right] + \frac{160\times10^3}{12}$	
	$= 1.72\times10^6\,mm^4$.	1M
	$\therefore I_{min} = I_{yy} = 1.72\times10^6\,mm^4$.	
	Total Area of section $= A = 4\times10\times80 + 160\times10$	
	$A = 4800\,mm^2$	1M
	Radius of Gyration.	
	$K_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{1.72\times10^6}{4800}}$	
	$K_{min} = 18.929\,mm$.	1M

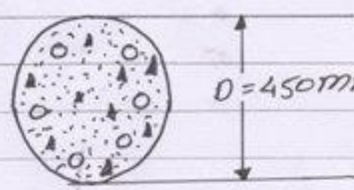
Q.NO	SOLUTION	MARKS
Q-2 (c)		
(i)	Given data :- $D = 60\ mm$	
		
	Radius of semi-circle $= \frac{60}{2} = 30\ mm$	
	M.I. about x-x- axis.	
	$I_{xx} = 0.11\ R^4 = 0.11 \times 30^4$	
	$= 89.10 \times 10^3\ mm^4$	1M
	M.I. about y-y axis.	
	$I_{yy} = I_{AB} = \frac{1}{2}\left[\frac{\pi}{64} D^4\right] = \frac{\pi}{128} D^4$	
	$= \frac{\pi}{128} \times (60)^4$ $\boxed{OR}$ $0.393\ R^4 = 0.393 \times 30^4$	1M
	$= 318.086 \times 10^3\ mm^4$.	
	$I_{min} = I_{xx} = 89.10 \times 10^3\ mm^4$.	
	Minimum Radius of Gyration.	
	$K_{min} = \sqrt{\frac{I_{min}}{A}}$	
	$= \sqrt{\frac{89.10 \times 10^3}{\pi R^2 / 2}} = \sqrt{\frac{89.10 \times 10^3}{\frac{\pi \times 30^2}{2}}}$	
	$K_{min} = 7.938\ mm$.	1M
	Polar M.I $= I_p = I_{xx} + I_{yy}$	

$= 89.10 \times 10^3 + 318.086 \times 10^3$

$= 407.18 \times 10^3\ mm^4$ 1M

Q.NO	SOLUTION	MARKS
Q2 (c) (ii)		1M
	A = Limit of porportionality, B = Elastic limit C = Upper yield point, D = Lower yield point E = Maximum load point, F = Breaking point.	1M
	LIMIT OF PROPORTIONALITY :⇒ The limit upto which stress is directly proportional to strain is called as limit of proportionality	1M
	Elastic limit :– Upto certain limit the material regain its original position on the removal of External load is known as Elastic limit. [OR] The value of intensity of stress Corresponding to the limiting load is known as elastic limit.	1M

Q. NO.	SOLUTION	MARKS
Q3 a)	Given, $E = 210\ GPa$	
	i) Magnetude of P	
	Consider equilibrium of the entire bar	
	$\Sigma F_x = 0 \; (\rightarrow +ve \; ; \leftarrow -ve)$	
	$\therefore -10 + 8 - P + 14 = 0$	
	$P = 12\ kN\ (\leftarrow)$	1M
	ii) F.B.D of each section	
		1M
	$P_1 = 10\ KN$ | $P_2 = 2KN$ | $P_3 = 14\ KN$	
	$L_1 = 500mm$ | $L_2 = 700\ mm$ | $L_3 = 900\ mm$	
	$A_1 = \frac{\pi}{4}(20)^2$ | $A_2 = \frac{\pi}{4}(25)^2$ | $A_3 = \frac{\pi}{4}(30)^2$	
	$A_1 = 314.5\ mm^2$ | $A_2 = 490.87\ mm^2$ | $A_3 = 706.85\ mm^2$	1M
	iii) Total elongation (δL)	
	$\delta L = \delta L_1 + \delta L_2 + \delta L_3$	
	$\delta L_1 = \frac{P_1 L_1}{A_1 E} = \frac{10 \times 10^3 \times 500}{314.15 \times 210 \times 10^3} = 0.07579\ mm$	1M
	$\delta L_2 = \frac{P_2 L_2}{A_2 E} = \frac{2 \times 10^3 \times 700}{490.87 \times 210 \times 10^3} = 0.01358\ mm$	1M

Q.NO	SOLUTION	MARKS
Q3 a) Cont...	$\delta L_3 = \frac{P_3 L_3}{A_3 E} = \frac{14\times10^3 \times 900}{706.85\times210\times10^3} \doteq 0.08488\ mm$	1M
	$\therefore\ \delta L = 0.07579 + 0.01358 + 0.08488$	
	$\delta L = 0.174\ mm$ increase.	1M
	iv) Minimum stress	
	$\sigma_1 = \frac{P_1}{A_1} = \frac{10\times10^3}{314.15} = 31.83\ N/mm^2$	
	$\sigma_2 = \frac{P_2}{A_2} = \frac{2\times10^3}{490.87} = 4.07\ N/mm^2$	
	$\sigma_3 = \frac{P_3}{A_3} = \frac{14\times10^3}{706.85} = 19.80\ N/mm^2$	
	$\therefore\ \sigma_{min} = \sigma_2 = 4.07\ N/mm^2$	1M
Q3 b)	Given, $d = 450\ mm$; $A_{st} = 6 - 16\ mm\ \phi$	
	$\sigma_c = 5\ N/mm^2$; $\sigma_s = 125\ N/mm^2$	
	$E_c = 0.14\times10^5\ N/mm^2$; $E_s = 2.1\times10^5\ N/mm^2$	
		

Q.NO.	SOLUTION	MARKS
Q4b Cont...	i) Gross area $A_g = \frac{\pi}{4}(450)^2 = 159.04 \times 10^3 \ mm^2$	1M
	ii) Area of steel $A_s = 6 \times \frac{\pi}{4}(16)^2 = 1206.37 mm^2$	1M
	iii) Area of Concrete $A_c = A_g - A_s$	
	$A_c = 159.04 \times 10^3 - 1206.37$ $A_c = 157.83 \times 10^3 \ mm^2$	2M
	iv) Safe load the column can carry P = Load taken by steel (P_s) + Load taken by Concrete (P_c)	
	$P = P_s + P_c$	1M
	$P = \sigma_s A_s + \sigma_c A_c$ $P = (125 \times 1206.37) + (5 \times 157.83 \times 10^3)$ $P = 150.79 \times 10^3 + 789.15 \times 10^3$ $P = 939.94 \times 10^3 \ N.$	1M
	$P = 939.94 \ KN$	2M
	The safe load the column can carry is 939.94 KN	

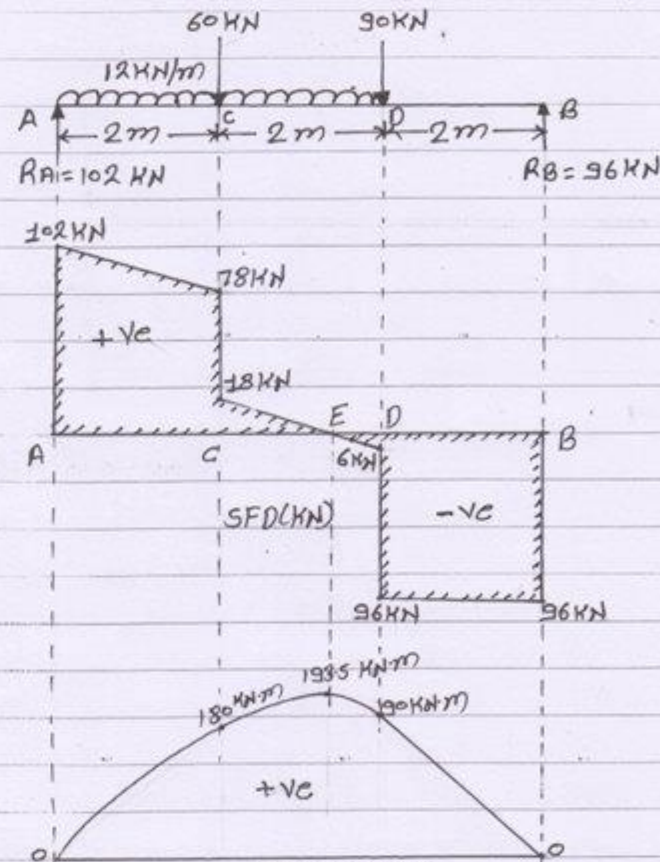
Q.NO	SOLUTION	MARKS
	student can write in two Possible relations. Examiner can consider any one.	
Q3 c) i)	$\boxed{\begin{array}{l} E = 2G(1+\mu) \cdots\cdots (i) \\ E = 3K(1-2\mu) \cdots\cdots (ii) \end{array}}$ $\longrightarrow$	4 M
	From eq^n (i)	
	$1+\mu = E/2G$	
	$\therefore \mu = \frac{E}{2G} - 1 \cdots\cdots (iii)$	
	Substitute the value of μ in eq^n (ii)	
	$E = 3K\left[1 - 2\left[\frac{E}{2G} - 1\right]\right] = 3K\left[1 - \frac{E}{G} + 2\right]$	OR
	$E = 3K\left[3 - \frac{E}{G}\right] = 3K\left[\frac{3G-E}{G}\right]$	
	$\therefore EG = 3K(3G-E)$	
	$EG = 9KG - 3KE$	
	$EG + 3KE = 9KG$	
	$E(G+3K) = 9KG$	
OR	$\boxed{E = \frac{9KG}{G+3K}}$	4M

Q NO	SOLUTION	MARKS
Q3 c ii)	Given, $d = 100\,mm$; $L = 600\,mm$; $P = 900\,KN$	
	$\mu = 0.30$; $E = 210\,KN/mm^2$	
	i) Linear strain (e)	
	$e = \frac{\delta L}{L} = \frac{PL}{AE} \times \frac{1}{L} = \frac{P}{AE}$	1M
	$e = \frac{900 \times 10^3}{\frac{\pi}{4}(100)^2 \times 210 \times 10^3}$	
	$e = 5.45 \times 10^{-4}$	1M
	ii) Modulus of Rigidity (G)	
	$E = 2G(1+\mu)$	
	$210 \times 10^3 = 2G(1+0.3)$	
	$210 \times 10^3 = 2.6\,G$	
	$\therefore G = 80.769 \times 10^3\,N/mm^2$	1M
	iii) Bulk modulus (K)	
	$E = 3K(1-2\mu)$	
	$210 \times 10^3 = 3K(1 - 2 \times 0.3)$	
	$210 \times 10^3 = 1.2\,K$.	
	$\therefore K = 175 \times 10^3\,N/mm^2$	1M

Q NO	SOLUTION	MARKS
Q4 a)	Given, $b = 20mm$; $t = 15mm$; $L = 3m$	
	$P = 30KN$; $E = 2\times10^5\ N/mm^2$; $\mu = 0.30$	
	i) change in length (δL)	
	$\delta L = \frac{PL}{AE} = \frac{30\times10^3 \times 3000}{20\times15\times2\times10^5} = 1.5\ mm$ Increase	1M
	ii) Linear strain (e)	
	$e = \frac{\delta L}{L} = \frac{1.5}{3000} = 5\times10^{-4}$	1M
	iii) Lateral strain (e_L)	
	$e_L = \mu \cdot e = 0.3\times5\times10^{-4} = 1.5\times10^{-4}$	1M
	iv) Change in width (δb)	
	$e_L = \frac{\delta b}{b}$;	
	$1.5\times10^{-4} = \frac{\delta b}{20}$	
	$\therefore \delta b = 3\times10^{-3}\ mm$ decrease	1M
	v) change in thickness (δt)	
	$e_L = \frac{\delta t}{t}$	
	$\delta t = e_L \times t = 1.5\times10^{-4}\times15$	1M

Q NO	SOLUTION	MARKS
Q4 a) Cont...	$\delta t = 2.25\times10^{-3}$ mm decrease	
	vi) Volumetric strain (e_v)	
	$e_v = e(1-2\mu)$	1M
	$= 5\times10^{-4}(1-2\times0.3)$	
	$e_v = 2\times10^{-4}$	1M
	vii) change in volume (δV)	
	$e_v = \frac{\delta V}{V}$	
	$\delta V = e_v \times V$	
	$= e_v \times b\times t\times L$	
	$= 2\times10^{-4}\times20\times15\times3000$	
	$\delta V = 180\ mm^3$ Increase	1M.
Q4 b)	Given $d = 30mm$; $P = 60KN$; $\delta L = 0.09mm$	
	$L = 200mm$; $\delta d = 0.039mm$.	
	i) stress (σ)	
	$\sigma = \frac{P}{A} = \frac{60\times10^3}{\frac{\pi}{4}(30)^2} = 84.88\ N/mm^2$	2M

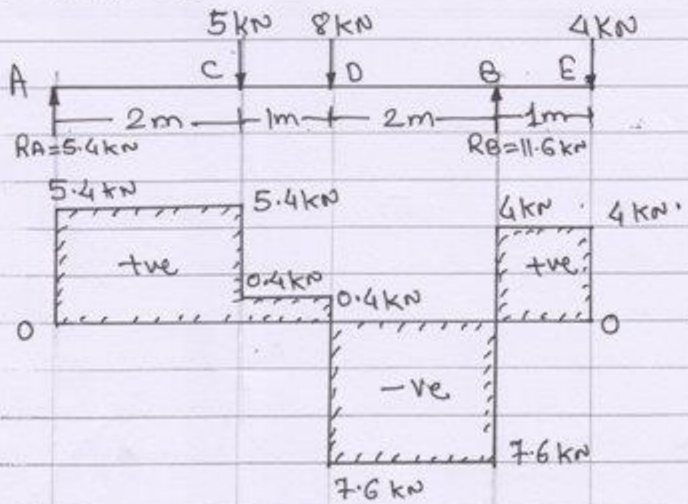
Q NO.	SOLUTION	MARKS
Q4 b) Cont...	ii) Strain (e)	
	$e = \frac{\delta L}{L} = \frac{0.09}{200} \doteq 4.5\times10^{-4}$	1M
	iii) Modulus of Elasticity (E)	
	$E = \frac{\sigma}{e} = \frac{84.88}{4.5\times10^{-4}} = 188.62\times10^{3}\ N/mm^2$	1M
	iv) Lateral strain (e_L)	
	$e_L = \frac{\delta d}{d} = \frac{0.039}{30} = 1.3\times10^{-3}$	1M
	v) Poissons ratio (μ)	
	$\mu = \frac{e_L}{e} = \frac{1.3\times10^{-3}}{4.5\times10^{-4}} = 2.89$	1M
	vi) Modulus of Rigidity (G)	
	$E = 2G(1+\mu)$	
	$E = 2G(1+2.89)$	
	$188.62\times10^{3} = 7.78\ G$	
	$\therefore\ G = 24.24\times10^{3}\ N/mm^2$	1M
	vii) Bulk Modulus (K)	
	$E = 3K(1-2\mu)$	
	$188.62\times10^{3} = 3K(1-2\times2.89)$	
	$188.62\times10^{3} = -14.34\ K.$	
	$\therefore K = -13.15\times10^{3}\ N/mm^2$	1M

Q NO	SOLUTION	MARKS
Q4 c)		2M
	BMD (KN.m)	2M
	1> Support reactions	
	i> $\Sigma F_x = 0$	
	$R_A - (12\times4) - 60 - 90 + R_B = 0$	
	$R_A + R_B = 198$ KN	

Q.NO	SOLUTION	MARKS
Q4 c) Cont...	ii) $\Sigma M@A = 0$	
	$(12\times4\times2)+(60\times2)+(90\times4)-R_B\times6=0$	
	$96+120+360=6R_B$	
	$R_B = 96\ KN \quad \therefore R_A = 198-96 = 102\ KN$	1M
	2) SF Calculation	
	a) S.F at just left of A = 0 KN	
	b) S.F at just right of A = R_A = 102 KN	
	c) S.F at just left of C = $102-(12\times2)=78$ KN	
	d) S.F at just right of C = $78-60 = 18$ KN	
	e) S.F at just left of D = $18-(12\times2)=-6$ KN	
	f) S.F at just right of D = $-6-90 = -96$ KN	1½ M
	g) S.F at just left of B = -96 KN	
	h) S.F at just right of B = $-96+R_B = 0$ KN	
	3) Point of Contrashear	
	from similler tringle	
	$\frac{18}{x} = \frac{6}{2-x}$	
	$x = \frac{18\times2}{18+6} = 1.5\ m$	
	4) B.M. Calculation	
	a) $BM_A = BM@B = 0$ ..... S.S. ends	1½ M
	b) $BM@C = (R_A\times2)-(12\times2\times\frac{2}{2}) = 180$ KN.m	
	c) $BM@D = (R_A\times4)-(12\times4\times4/2)-(60\times2) = 190$ KN.m	
	d) $BM@E = (R_A\times3.5)-(12\times3.5\times3.5/2)-(60\times1.5) = 193.5$ KN.m	

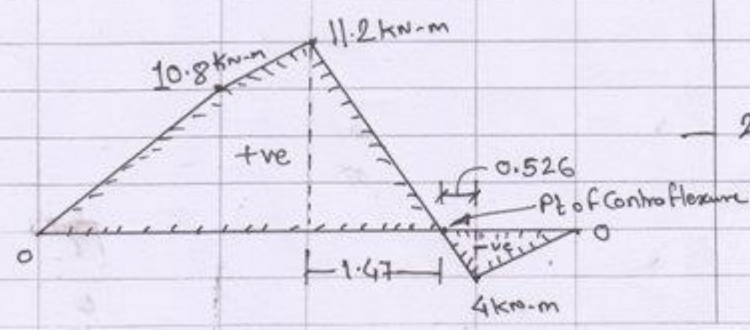
Q.NO	SOLUTION	MARKS
Q.5)	Attempt any Two of the following:	16
→ a)	(A) Support Reaction calculation:	
	i) $\Sigma fy = 0$	
	$\therefore R_A + R_B = 4 + 8 + 5 = 17$	
	ii) $\Sigma M @ A = 0$	
	$\therefore -(4\times6) + (R_B \times 5) - (8\times3) - (5\times2) = 0$	
	$\therefore R_B = 11.6$ kN.	1m
	& $\therefore R_A = 17 - 11.6 = 5.4$ kN.	
	B) Shear force calculation : (↑ -ve, ↓ +ve)	
	i) SF @ E (R) = 0	
	E (L) = 4 kN.	
	ii) SF @ B (R) = 4 kN	
	B (L) = 4 − 11.6 = −7.6 kN.	
	iii) SF @ D (R) = 4 − 11.6 = −7.6 kN.	1m
	D (L) = 4 − 11.6 + 8 = 0.4 kN.	
	iv) SF @ C (R) = 0.4 kN.	
	C (L) = 0.4 + 5 = 5.4 kN.	
	v) SF @ A (R) = 5.4 kN.	
	A (L) = 5.4 − 5.4 = 0 kN.	
	C) Bending Moment Calculation:	
	(↻ -ve, ↺ +ve)	1m
	i) BM @ E = 0	

Q NO	SOLUTION	MARKS
Q 5(a) cont..	ii) $BM@B = -(4\times1) = -4$ kN-m.	
	iii) $BM@D = -(4\times3) + (11.6\times2) = 11.2$ kN-m	
	iv) $BM@C = -(4\times4) + (11.6\times3) - (8\times1) = 10.8$ kN-m	
	v) $BM@A = 0$	



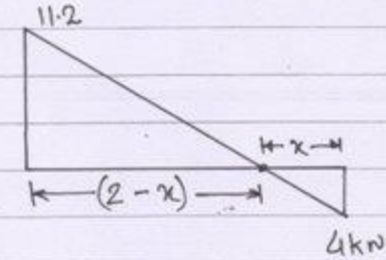
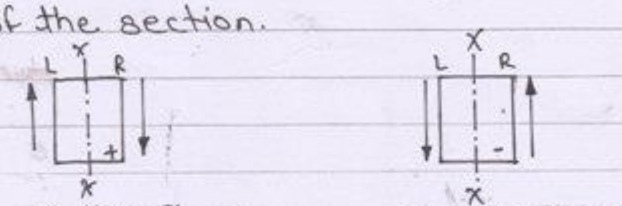
SFD

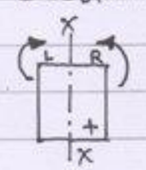
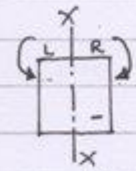
2m



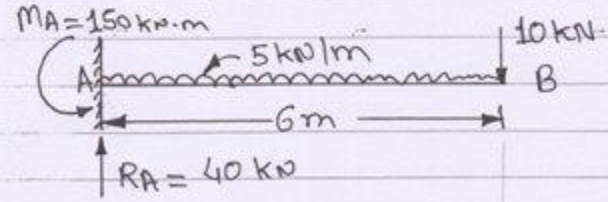
2m

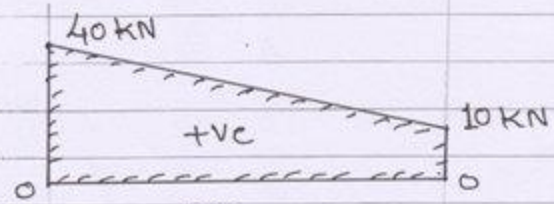
BMD

Q.NO	SOLUTION	MARKS
5 a) cont..	Location of point of Contraflexure :	
		
	$\frac{x}{4} = \frac{(2-x)}{11.2}$	
	$\therefore x = 0.526\,m$ from B.	1m
	[or]	
	BM @ pt of Contra = 0	
	$0 = -(4 \times (1+x)) + (11.6 \times x)$	
	$\therefore \quad x = 0.526\,m$ from B.	
Q.5)b	(i) Shear force : Shear force at any cross-section of the beam is the algebric sum of all vertical forces on the beam acting on the right or left side of the section.	1m
		1m
	Positive Shear Negative Shear	

Q. NO.	SOLUTION	MARKS
5 (b) (i) cont--	<u>Bending Moment</u> : Bending Moment at any cross-section of the beam is the algebraic sum of the moments of all the forces acting on the right or left of the section.	1m
	 Positive B.M. (Sagging)  Negative B.M. (Hogging)	1m
Q.5(b)	ii) <u>Cantilever Beam:</u>	
	A) Support Reaction Calculation:	
	i) $\Sigma fy = 0$	
	$RA = 10 + (5 \times 6) = 40$ kN.	
	ii) $\Sigma M @ A = 0$	
	$-(10 \times 6) - (5 \times 6 \times \frac{6}{2}) + M_A = 0$	
	$\therefore M_A = 150$ kN-m.	
	B) Shear force Calculation: ($\uparrow$-ve, $\downarrow$+ve)	
	i) SF@ B(R) = 0	
	B(L) = 10 kN.	1m
	ii) SF@ A(R) = 10 + (5X6) = 40 kN	
	A(L) = 40-40 = 0	

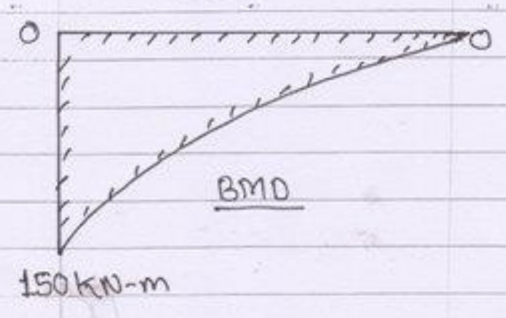
Q.NO	SOLUTION	MARKS
5 (b) (ii) cont...	c) Bending Moment Calculation: (↺ -ve, ↻ +ve)	
	i) BM @ B = 0	
	ii) BM @ A(R) $= -(10\times6)-(5\times6\times\frac{6}{2})$	1M
	$= -150$ kN-m.	
	A(L) $= -150+150 = 0$	





SFD

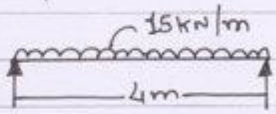
1M



1M

Q. NO.	SOLUTION	MARKS

Q.5) (C) (i) Bending Stress:



Given: $b = 150\ mm$

$d = 300\ mm$

$L = 5\ m,$

$\omega = 15\ kN/m.$

Find: $\sigma_b = ?$

Soln:

i) $BM_{max} = \frac{\omega L^2}{8} = \frac{15 \times 4^2}{8}$

$= 30\ kN\text{-}m$

$= 30 \times 10^6\ N\text{-}mm$ — 1 m

ii) $I = \frac{bd^3}{12} = \frac{150 \times 300^3}{12}$

$= 337.5 \times 10^6\ mm^4$ — 1 m

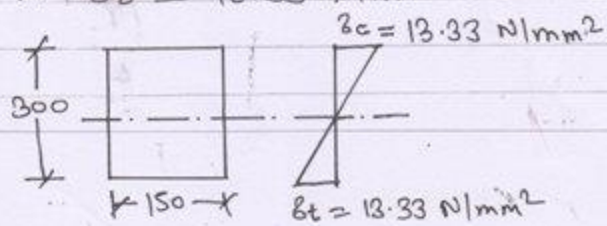
iii) $y = \frac{d}{2} = \frac{300}{2} = 150\ mm$

Using Flexural formula:

$\frac{M}{I} = \frac{\sigma_b}{y}$ — ½ m

$\therefore \frac{30 \times 10^6}{337.5 \times 10^6} = \frac{\sigma_b}{150}$

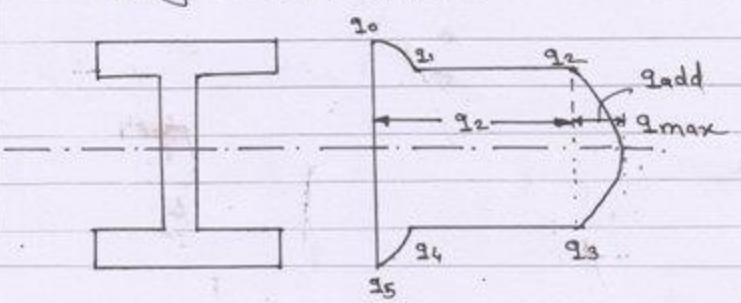
$\therefore \sigma_b = 13.33\ N/mm^2 = \sigma_c = \sigma_t$ — ½ m



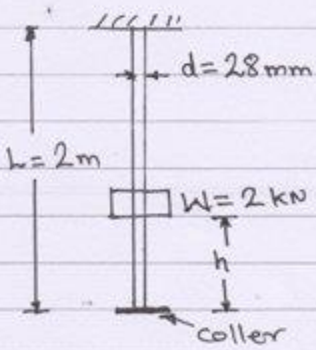
— 1 m

Q NO	SOLUTION	MARKS
Q.5.(c)(ii)	Shear Stress :	
	Given: $D = 100\ mm$	
	$q_{max} = 100\ N/mm^2$	
	Find: $q_{avg} = ?$	
	$q_{at\ face} = ?$	
	Sol^n:	
	for Circular Section	
	$q_{max} = \frac{4}{3} q_{avg}$ —	1M
	$\therefore 100 = \frac{4}{3} q_{avg}$ —	1M
	$\therefore q_{avg} = \frac{100 \times 3}{4} = 75\ N/mm^2$ —	1M
	Shear stress at the face of beam $= 0\ N/mm^2$ —	1M
	$\therefore q_{at\ face} = 0\ N/mm^2$.	
Q.6	Attempt any TWO of the following:	
Q.6)(a)	Given: $I_{xx} = 2 \times 10^8\ mm^4$	
	$S = 100\ kN = 100 \times 10^3\ N$	
	find: $q_{max} = ?$	
	$q_{avg} = ?$	
	Sol^n:	
	Maximum shear stress for symmetrical I-section is at N.A. of section	

Q NO	SOLUTION	MARKS
6 (a) cont..	$q_{max} = q_2 + q_{add}$	1m
	$\because q_2 = \left(\frac{S A \bar{y}}{I b}\right)_2$	
	$= \frac{100 \times 10^3 \times 4500 \times 165}{2 \times 10^8 \times 20}$	
	$q_2 = 18.5625 \ N/mm^2$	2m
	& $q_{add} = \left(\frac{S A \bar{y}}{I b}\right)_{add}$	
	$= \frac{100 \times 10^3 \times (150 \times 20) \times 75}{2 \times 10^8 \times 20}$	2m
	$= 5.625 \ N/mm^2$	
	$\therefore q_{max} = q_{NA} = 18.5625 + 5.625$	
	$= 24.1875 \ N/mm^2$	1m
	$q_{avg} = \frac{S}{A_T} = \frac{100 \times 10^3}{2(150 \times 30) + (300 \times 20)}$	1m
	$q_{avg} = 6.67 \ N/mm^2$	1m



Q NO	SOLUTION	MARKS
Q.6(b)	Given: $E = 2\times10^5$ N/mm² $\sigma_c = 320$ N/mm² Find: limiting value of slenderness ratio, λ = ? by Euler's theory.	
	Soln: By Euler's theory:	
	$P = \frac{\pi^2 EI}{Le^2}$	1m
	$\because I = AK^2$ $P = \frac{\pi^2 E A K^2}{Le^2}$	1m
	$\frac{P}{A} = \pi^2 E \left(\frac{K}{Le}\right)^2$ $\quad \because \lambda = \frac{Le}{K}$	1m
	$\therefore \sigma_c = \pi^2 E \left(\frac{1}{\lambda}\right)^2$	1m
	$\left(\frac{1}{\lambda}\right)^2 = \frac{\sigma_c}{\pi^2 E}$ $\therefore \lambda^2 = \frac{\pi^2 E}{\sigma_c}$	
	$\therefore \lambda = \sqrt{\frac{\pi^2 E}{\sigma_c}}$	2m
	$\lambda = \sqrt{\frac{\pi^2 \times 2\times10^5}{320}}$	
	$\boxed{\lambda = 78.54}$	2m

Q. NO.	SOLUTION	MARKS
Q.6 (C)	Given:	
	$W = 2\times10^3$ N	
	$L = 2000$ mm	
	$d = 28$ mm	
	$\sigma = 120$ N/mm^2	
	$E = 2\times10^5$ N/mm^2	
		
	Find: i) $h = ?$	
	ii) $\delta L = ?$	
	Sol^n:	
	Elongation $\delta L = \frac{\sigma L}{E}$	1m
	$\therefore \delta L = \frac{120\times2000}{2\times10^5}$	
	$\boxed{\delta L = 1.2\text{ mm}}$	1m
	$\sigma = \frac{P}{A} + \sqrt{\left(\frac{P}{A}\right)^2 + \frac{2Eh}{L}\left(\frac{P}{A}\right)}$	2m
	$120 = x + \sqrt{x^2 + \frac{2Eh}{L}x}$	
	$\because x = \frac{P}{A} = \frac{2\times10^3}{\frac{\pi}{4}28^2} = 3.248$	1m
	$120 = 3.248 + \sqrt{3.248^2 + \frac{2\times2\times10^5\times3.248\times h}{2000}}$	

Q NO	SOLUTION	MARKS
6 (c)	$120 - 3.248 = \sqrt{10.549 + 649.6\,h}$	
cont..	Squaring on both side	
	$116.75^2 = 10.549 + 649.6\,h$	1m
	$\therefore \dfrac{116.75^2 - 10.549}{649.6} = h$	
	$\therefore \boxed{h = 20.96\ mm}$	2m