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.

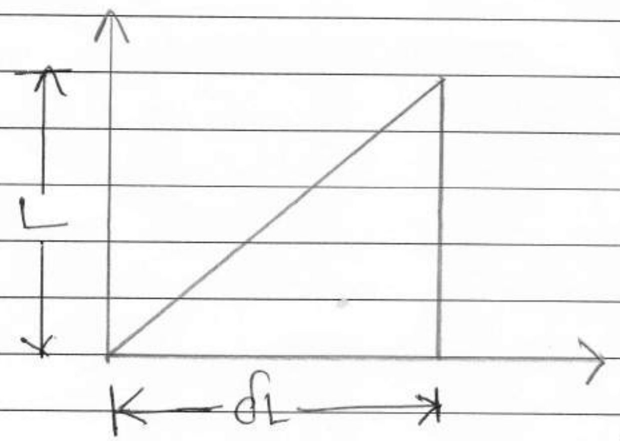
Important notes to examiner

Q .NO	SOLUTION	MARKS
1 A	Attempt any six	
(a)	State moment of inertia of a triangular section about it's base and apex.	
	M.I. of triangular section about base	
	$\boxed{I = \frac{bh^3}{12}}$	01 M
	M.I about apex	
	$\boxed{I = \frac{bh^3}{4}}$	01 M
(b)	Define radius of gyration	
	The radius of gyration of a given area about any axis is that distance from the given axis at which the entire area is assumed to be concentrated without changing the M.I. about the given axis. it is denoted by K or r	01 M
	$\boxed{K = \sqrt{\frac{I_{xx}}{A}}}$	01 M

Q.NO	SOLUTION	MARKS
1 A		
(c)	Define Elastic body giving two example's	
	A body is said to be elastic if it regain's it's original size and shape when an externally applied force causing deformation is entirely removed	1M
	Examples	
	(1) Rubber Band	1 M
	(2) Golf Ball.	write
	(3) Soccer Ball.	any two Ex.
(d)	State Hook's law	
	when a material is loaded within it's elastic limit, the stress produced is directly proportional to the strain.	01 m
	Stress $\propto$ strain	
	$6 \propto e$	01 m
	6 = stress, e = strain	
	$\frac{6}{e}$ = constant.	
(e)	State 4 Assumption's in Euler's column theory.	
	1) intially the column is perfectly straight and load applied is truely axial.	

Q.NO	SOLUTION	MARKS
(2)	the cross-section of the column is uniform throughout it's length	
(3)	the column material is perfectly, elastic, homogeneous. and isotropic, and thus obey's Hook's law.	$\frac{1}{2}$ M for each write any four
(4)	the length of column is very large as compared to it's cross-sectional dimentions.	
(5)	the shortening of column due to direct compression (being very small) is neglected	
(6)	the failure of column occur's due to buckling alone.	
(7)	the weight of the column itself is neglected.	
(f)	Define slenderness ratio and give it's expressions	
	the slenderness ratio is defind as ratio of equivalent length of column to the least radius of gyration of the section.	01 M
	$$\boxed{P_E = \frac{\pi^2 E A}{\left(\frac{L_e}{K}\right)^2}}$$	01 M
	Where $\frac{L_e}{K}$ is known as slenderness ratio	

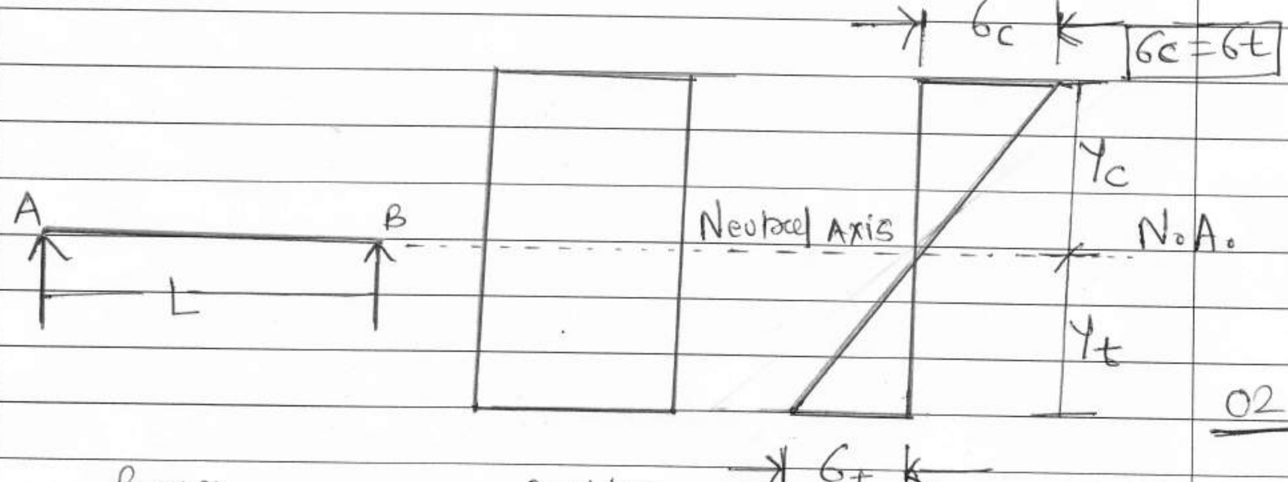
Q .NO	SOLUTION		MARKS
(g)	Differentiate between gradual load & impact load		
	Gradual load	impact load	
i)	this type of loading starting from zero & slowly increasing is called gradual load	i) when load is allowed to fall from a certain height it is called impact load.	1 M
ii)	stress due to gradual load $\sigma = \frac{P}{A}$	ii) stress due to impact load $\sigma_{max} = \frac{P}{A} + \sqrt{\left(\frac{P}{A}\right)^2 + \frac{2PhE}{AL}}$	1 M
(h)	Write the Expression for strain energy due to any type of load.		
	For gradually load work done for load 'P' is given by area of the load deformation diagram.		



Q .NO	SOLUTION	MARKS
	Work done by Ext. load	
	$= \frac{1}{2} P \cdot \delta L$	
	$= \frac{1}{2} P \left(\frac{PL}{AE} \right)$	½ M
	$= \frac{1}{2} \frac{P^2 AL}{A^2 E}$	½ M
	$= \frac{1}{2} \frac{\sigma^2}{E} AL$	½ M
	$= \frac{1}{2} \frac{\sigma^2}{E} (\text{volume})$ $\because$ volume $= A \times L$ & $\sigma = \frac{P}{A}$	
	$= \frac{\sigma^2}{2E} \cdot V$	½ M
	strain energy $\boxed{u = \frac{\sigma^2}{2E} \cdot V}$	

Q .NO	SOLUTION	MARKS
1		
B	Attempt any two	
a	State the flexural formulae giving meaning of the symbol used in it.	
	Flexural formulae	
	$\frac{M}{I} = \frac{\sigma}{y} = \frac{E}{R}$	02 M
	where	
	M = max Bending moment which is equal to moment of Resistance of beam	
	I = M.I of beam section about the neutral axis since neutral axis always lies at the centroid of the section.	
	$I = I_{NA} = I_{xx}$	
	σ = Bending stress in a layer at a dist. y from N.A.	
	y = dist. of the layer from the N.A. of the beam materal.	
	R = Radius of curvature of the bent-up beam.	02 M

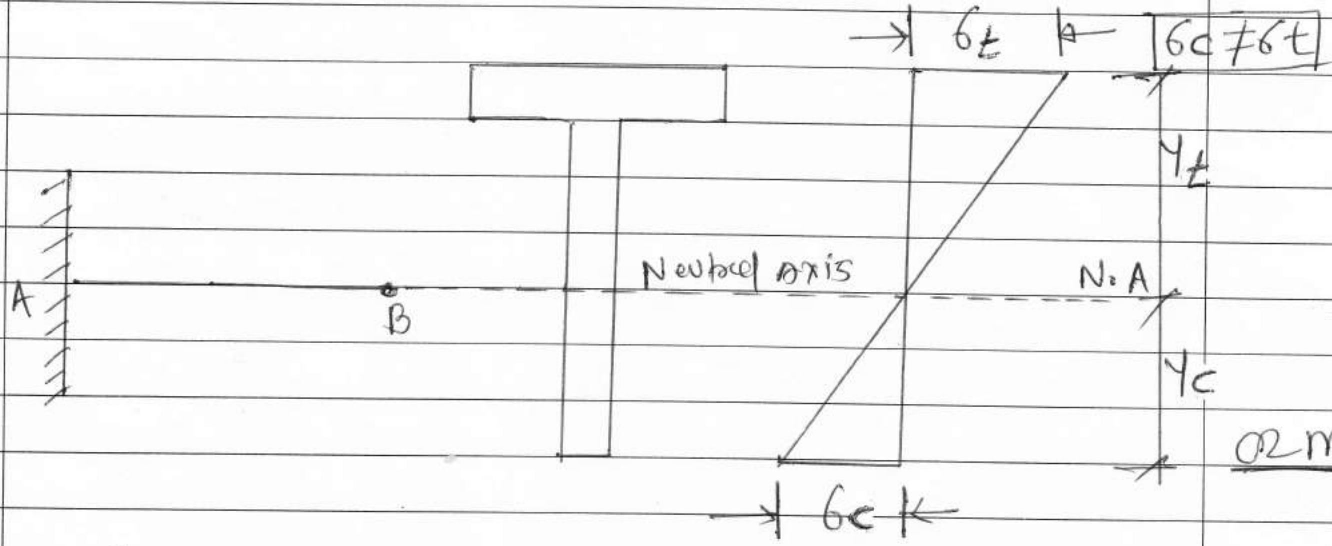
Q.NO	SOLUTION	MARKS
b)	draw Bending stress distribution diag. for the following cases.	
(i)	A beam of Rectangular c/s used as a simply supported beam.	



Beam Section Stress distri. diag.

02 M

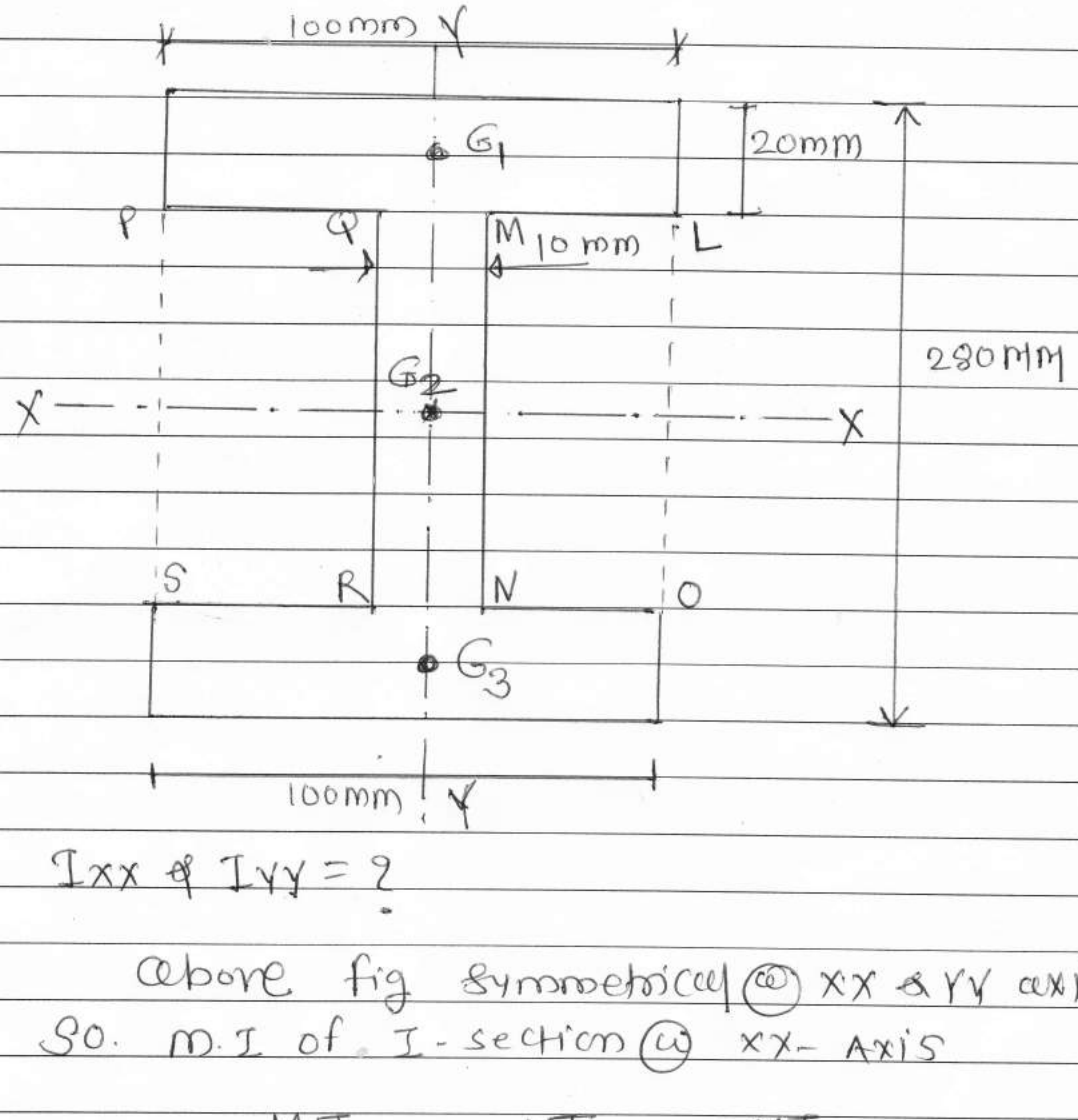
(ii) Beam of 'T' section used as a cantilever.



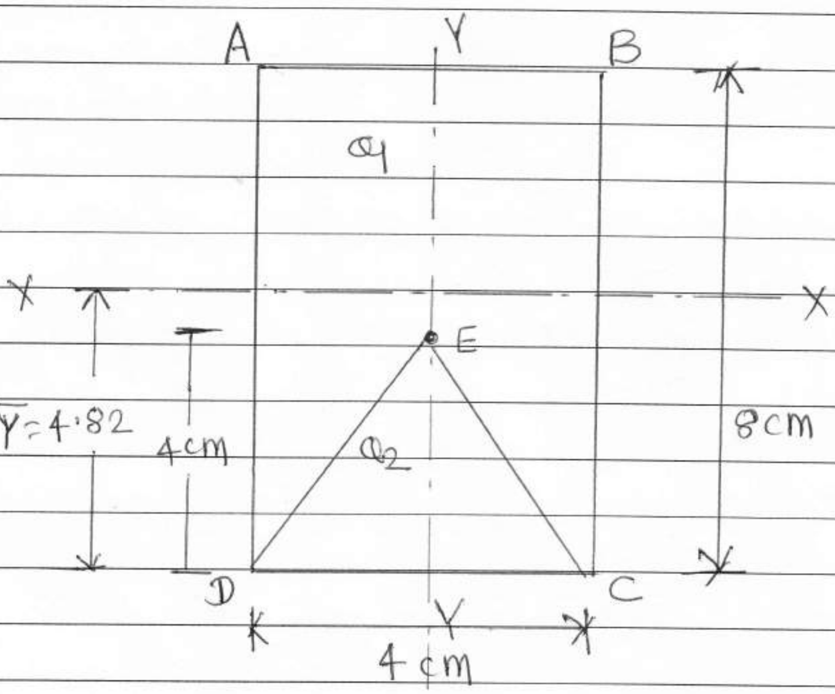
Beam Section Stress distribution diag.

02 M

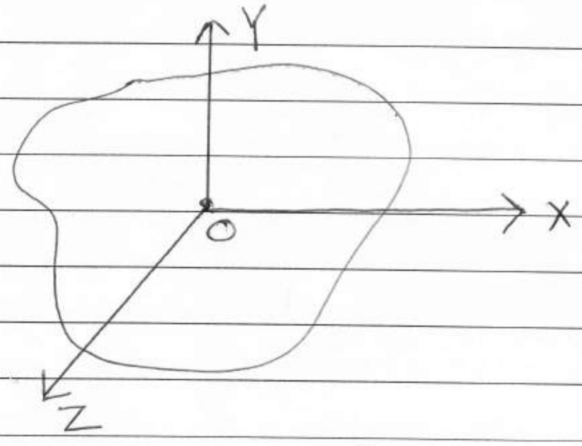
Q .NO	SOLUTION	MARKS
1 (B)		
c)	A column having diameter 200mm is of length 3 meters. Both end of column are hinged find Euler's crippling load. Take $E = 2\times10^5$ mPa.	
	Given $d = 200$mm	
	$L = 3m = 3000$mm	
	$E = 2\times10^5$ mPa	
	P_E = Euler's crippling load = ?	
	∵ Both end of column are hinged	
	$L_e = L = 3000$ mm	
	we have	
	Euler's crippling load	
	$P_E = \frac{\pi^2 E I}{(L_e)^2}$ —— (i)	01 M
	for circular column, M.I is	
	$I = \frac{\pi}{64} d^4$	
	$I = \frac{\pi}{64} \times (200)^4$	
	$I = 78.53\times10^6$ mm^4	01 M
	from eqn (i)	
	$P_E = \frac{\pi^2 \times 2\times10^5 \times 78.53\times10^6}{(3000)^2}$	
	$\boxed{P_E = 17.22\times10^6 \text{ N}}$	02 M

Q .NO	SOLUTION	MARKS
(2)	Attempt any two	
(a)	find the least M.I. of a symmetrical I-section having following details.	
	flanges : 100mm x 20mm	
	overall depth : 280mm	
	thick. of web : 10 mm	
		
	I_{xx} & I_{yy} = ?	
	above fig symmetrical @ XX & YY axis so. M.I of I-section @ XX-Axis	
	$M.I_{xx} = M.I._{ABCD} - MI._{PQRS} - MI._{LMNO}$ ——— (i) eqn	

Q .NO	SOLUTION	MARKS
2)		
a)	$M.I._{ABCD} = \frac{100 \times 280^3}{12} = 182.93 \times 10^6 \ mm^4$	1 M
	$M.I_{PQRS} = \frac{45 \times 240^3}{12} = 51.84 \times 10^6 \ mm^4$	1 M
	$M.I_{LMNO} = \frac{45 \times 240^3}{12} = 51.84 \times 10^6 \ mm^4$	1 M
	from (I)	
	$M.I_{xx} = 182.93 \times 10^6 - [2 \times 51.84 \times 10^6]$	
	$\boxed{M.I_{xx} = 79.25 \times 10^6 \ mm^4}$	1 M
	M.I of I-section @ Y-Y-Axis	
	$I_{yy} = I_{yy_1} + I_{yy_2} + I_{yy_3}$ —— (2)	
	$I_{yy_1} = \frac{db^3}{12} = \frac{20 \times 100^3}{12} = 1.67 \times 10^6 \ mm^4$	1 M
	$I_{yy_2} = \frac{db^3}{12} = \frac{240 \times 10^3}{12} = 20 \times 10^3 \ mm^4$	1 M
	$I_{yy_3} = I_{yy_1} = 1.67 \times 10^6 \ mm^4$	1 M
	from eq^n (2)	
	$I_{yy} = 1.67 \times 10^6 + 20 \times 10^3 + 1.67 \times 10^6$	
	$\boxed{I_{yy} = 3.36 \times 10^6 \ mm^4}$ least. M.I.	1 M

Q .NO	SOLUTION	MARKS
(b)	from a plate 4cm x 8cm a triangular portion as shown in fig 1 is cut. Determine the M.I of the remainder about the horizontal axis. Passing through the top of the lamina.	
		
	$a_1 = 4 \times 8 = 32\ cm$	
	$y_1 = \frac{8}{2} = 4cm$	
	$x_1 = 2\ cm$	
	$a_2 = \frac{1}{2} \times 4 \times 4 = 8cm$	
	$Y_2 = \frac{4}{3} = 1.33\ cm$	
	$x_2 = 2\ cm.$	

Q.NO	SOLUTION	MARKS
	Now,	
	$\bar{y} = \dfrac{a_1 y_1 - a_2 y_2}{a_1 - a_2}$	1m
	$\bar{y} = \dfrac{(32\times4) - (8\times1.33)}{(32-8)}$	
	$\bar{y} = 4.89$ cm	1m
	$\bar{x}$ = due to symmetry = 2cm	1m
	$I_{xx} = [M.I]_1 - [MI_2]$	1m
	$= \left[\dfrac{bd^3}{12} + a_1 h_1^2\right] - \left[\dfrac{bh^3}{36} + a_2 h_2^2\right]$	1m
	$= \left[\dfrac{4\times8^3}{12} + 32(4.89-4)^2\right] - \left[\dfrac{4\times4^3}{36} + 8(4.89-1.33)^2\right]$	
	$= [196.01] - [108.49]$	
	$I_{xy} = 87.51\ cm^4$	1m
	M.I of section at top of lamina	
	$I_{AB} = I_{xx} + Ah^2$	1m
		1m
	$A = A_1 - A_2 = 32 - 8 = 24\ cm^2$	
	$h = 8 - 4.89 = 3.11$	
	$\therefore I_{AB} = 87.51 + 24(3.11)^2 = 319.64\ cm^4$	

Q .NO	SOLUTION	MARKS
(2)		
c)	Explain Perpendicular axis theorem.	
		
	it state that " if I_{xx} & I_{yy} are the M.I. of a Plane section about the two mutually Perpendicular axis meeting at 'O' then the moment of intertia. I_{zz} about the third axis ZZ ⊥er to the Plane and Passing through the intersection of XX & YY axis is given by	01 M
	$$\boxed{I_{zz} = I_{xx} + I_{yy}}$$	01 M
	the third axis ZZ is called as Polar axis the M.I of I_{zz} about the axis ZZ is called Polar moment of intertia. it is denoted by I_p	01 M
	$\therefore I_{zz} = I_{pp}$	
	$$\boxed{\therefore I_p = I_{xx} + I_{yy}}$$	01 M

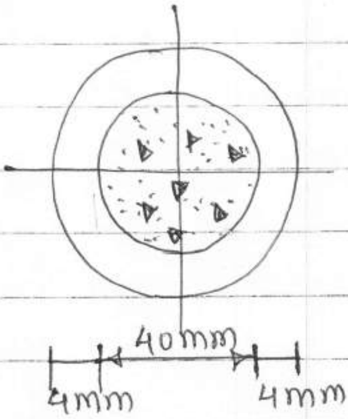
Q .NO	SOLUTION	MARKS
2 (c) ii) a)	Define following terms	
	ultimate stress – the ratio of the max load that the specimen is capable of withstanding and it's original c/s area is called the ultimate stress of the material.	
	$\boxed{\text{ultimate stress} = \dfrac{\text{maximum load}}{\text{original cross-section Area}}}$	01 M
(b)	Yield stress – it is defind as the ratio of the load at yield point and the original cross-section area of the specimen.	
	$\boxed{\text{Yield stress} = \dfrac{\text{Yield load}}{\text{original cross-section Area}}}$	01 M
(c)	Plastic strain – when an elastic body goes into a complete deformation then it can-not retain's it's original shape. in the plastic stage. which result in the cracking or failure of the material. is known as plastic strain.	01 M
(d)	Factor of safety – the ratio of ultimate stress and working stress. for a material is called factor of safety.	
	$\boxed{\text{Factor of safety} = \dfrac{\text{ultimate stress}}{\text{working stress}}}$	01 M

Q.NO	SOLUTION	MARKS
Q3a		
$\longrightarrow$	Given	
	i) $d = 22\ mm$, $t = 150°C$, $t_1 = 100°C$, $t_2 = 30°C$	
	$E_A = 70\ GPa$, $\alpha = 23\times10^{-6}/°C$	
	1) Temperature fall from 150°C to 100°C	
	$\therefore \quad \sigma = \alpha t E$	1M
	$= 23\times10^{-6}\times(150-100)\times70\times10^3$	
	$= 80.5\ N/mm^2$ Compressive	1M
	$\therefore$ Force P	
	$P = \sigma\times A$	1M
	$= 80.5\times\frac{\pi}{4}\times(22)^2$	
	$P = 30.60\ KN$	1M
	2) Temperature fall from 150° to 30°	
	$\therefore$ Stress $\sigma = \alpha t E$	1M
	$= 23\times10^{-6}\times(150-30)\times70\times10^3$	
	$= 193.20\ N/mm^2$	1M
	Force P	
	$P = \sigma\times A$	1M
	$= 193.20\times\frac{\pi}{4}\times(22)^2$	
	$P = 73.44\ N/mm^2$	1M.

Q.NO	SOLUTION	MARKS

Q-3(b) A steel tube 40 mm inside diameter & 4 mm metal thickness is filled with concrete. Determine stress in each material due to an axial thrust of 100 KN.

Take $E_s = 2.1 \times 10^5 \ N/mm^2$ & $E_c = 0.14 \times 10^5 \ N/mm^2$



$d = 40 mm$

$D = 40 + 4 + 4 = 48 mm$

$E_s = 2.1 \times 10^5 \ N/mm^2$

$E_c = 0.14 \times 10^5 \ N/mm^2$

i) $A_{steel} = \frac{\pi}{4}(D^2 - d^2) = \frac{\pi}{4} \times (48^2 - 40^2)$ 1M

$A_{steel} = 552.92 \ mm^2$

ii) A_c = Area of concrete $= \frac{\pi}{4} \times 40^2$

$A_c = 1256.63 \ mm^2$ 1M

iii) To find stress in each material

$$\frac{\sigma_s}{E_s} = \frac{\sigma_c}{E_c}$$ 1M

$$\sigma_s = \frac{E_s}{E_c}\sigma_c$$

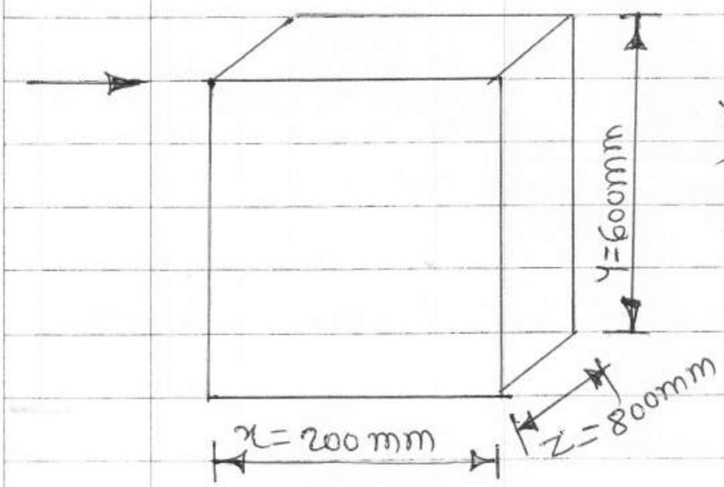
$$\sigma_s = \frac{2.1 \times \cancel{10^5}}{0.14 \times \cancel{10^5}}\sigma_c$$

$$\boxed{\sigma_s = 15\,\sigma_c}$$ 1M

Q.NO	SOLUTION	MARKS
	$P = P_s + P_c$	
	$= \sigma_s A_s + \sigma_c A_c$	1M
	$100 \times 10^3 = \sigma_s \times 552.92 + \sigma_c \times 1256.03$	
	$100 \times 10^3 = 15 \sigma_c \times 552.92 + 1256.03 \sigma_c$	
	$100 \times 10^3 = 8.2938 \times 10^3 \sigma_c + 1256.03 \sigma_c$	
	$100 \times 10^3 = 9.55 \times 10^3 \sigma_c$	1M
	$\sigma_c = \dfrac{100 \times 10^3}{9.55 \times 10^3}$	
	$\boxed{\sigma_c = 10.47 \ N/mm^2}$	1M
	$\sigma_s = 15 \times \sigma_c$	
	$\sigma_s = 15 \times 10.47$	
	$\boxed{\sigma_s = 157.07 \ N/mm^2}$	1M

Q.NO	SOLUTION	MARKS

Q-3 (c) In a biaxial stress system, the stresses along the two directions are $\sigma_x = 50\ N/mm^2$ (T) $\sigma_y = 60\ N/mm^2$ (C). Find the changes in dimensions and volume if $x = 200$ mm $y = 600$ mm & $z = 800$ mm Take $E = 200\ kN/mm^2$ & $m = 4$



i) $V = 200 \times 800 \times 600$

$V = 96 \times 10^6\ mm^3$

$x = 200$ mm

$y = 600$ mm

$z = 800$ mm

i) strain in x-direction $e_x = \frac{\sigma_x}{E} - \frac{\mu \sigma_y}{E}$

$e_x = \frac{50}{2 \times 10^5} - \frac{0.25(-60)}{2 \times 10^5} = \boxed{3.25 \times 10^{-4} = e_x}$ 1M

ii) $e_y = \frac{\sigma_y}{E} - \frac{\mu \sigma_x}{E}$

$e_y = \frac{-60 - 0.25 \times 50}{2 \times 10^5}$ $\boxed{e_y = -3.625 \times 10^{-4}}$ 1M

iii) $e_z = -\frac{\mu \sigma_x}{E} - \frac{\mu \sigma_y}{E}$

$e_z = \frac{-0.25 \times 50}{2 \times 10^5} - \frac{0.25 \times (-60)}{2 \times 10^5}$

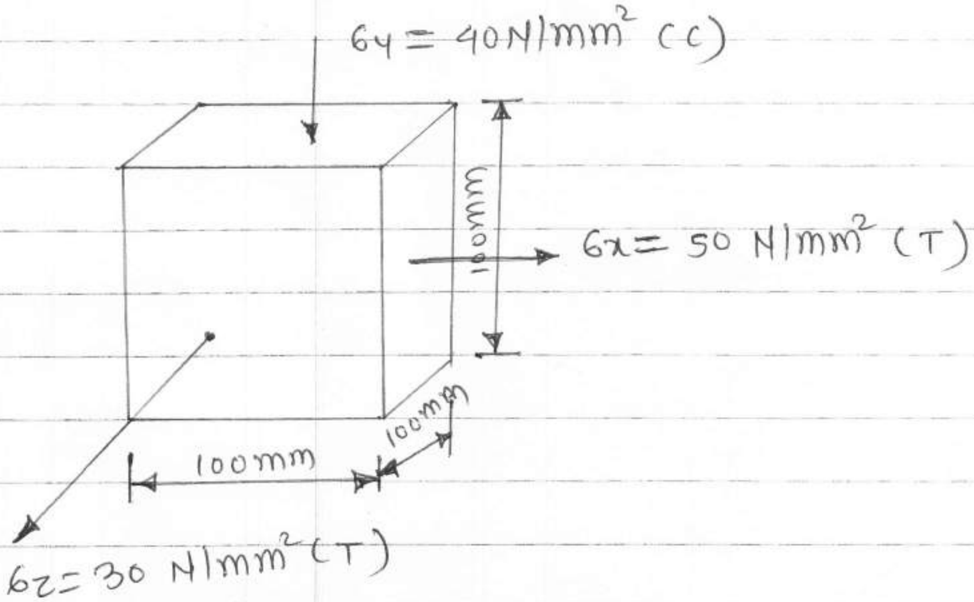
$e_z = -6.25 \times 10^{-5} + 7.5 \times 10^{-5}$

$\boxed{e_z = 1.25 \times 10^{-5}}$ 1M

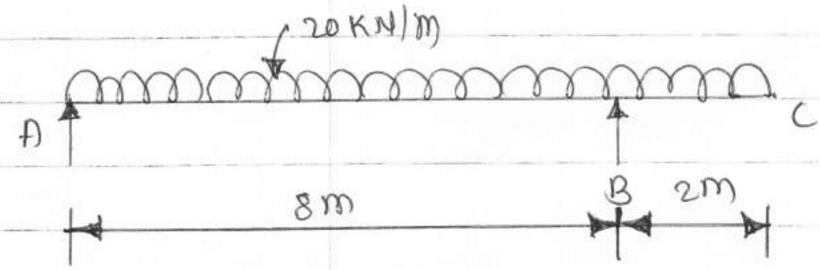
Q.NO	SOLUTION	MARKS
	iv) change of dimensions	
	$e_x = \frac{dx}{x}$ $\therefore dx = e_x \times x$	
	$dx = 3.25 \times 10^{-4} \times 200$	
	$\boxed{dx = 0.065 \text{ mm}}$	1M
	$e_y = \frac{dy}{y}$ $\therefore dy = e_y \times y$	
	$dy = -3.625 \times 10^{-4} \times 600$	
	$\boxed{dy = -0.2175 \text{ mm}}$	1M
	$e_z = \frac{dz}{z}$ $\therefore dz = e_z \times z$	
	$dz = 1.25 \times 10^{-5} \times 800$	
	$\boxed{dz = 0.01 \text{ mm}}$	1M
	v) change of volume	
	$\frac{\delta V}{V} = e_x + e_y + e_z$	
	$\delta v = (e_x + e_y + e_z) \times V$	1M
	$\delta v = [(3.25 \times 10^{-4}) + (-3.625 \times 10^{-4}) + (1.25 \times 10^{-5})] \times 96 \times 10^{6}$	
	$\boxed{\delta V = 2400 \text{ mm}^3 (c)}$	1M
	or	or
	$\frac{\delta V}{V} = \frac{\sigma_x + \sigma_y}{E}(1 - 2\mu)$	1M
	$\frac{\delta V}{V} = \frac{50 - 60}{2 \times 10^5}(1 - 2 \times 0.25)$	
	$\delta v = \left[\frac{-10 \times (0.5)}{2 \times 10^5}\right] 96 \times 10^6$ $\boxed{\delta v = 2400 \text{ mm}^2 (c)}$	1M

Q.NO	SOLUTION	MARKS
Q-4 (a)	A metal rod of 20 mm diameter and 2.5 m long when subjected to a tensile force 70 KN. showed an elongation 2.5 mm & reduction in diameter -0.006 mm. calculate modulus of elasticity and modulus of rigidity.	
→	given :	
	$d = 20$ mm	
	$L = 2.5$ m $= 2500$ mm	
	$P = 70$ KN	
	$dL = 2.5$ mm $\quad dd = -0.006$ mm	
	i) $A = \frac{\pi}{4} \times 20^2 = 314.15$ mm^2	
	ii) $\sigma = \frac{P}{A} = \frac{70 \times 10^3}{314.15} = 222.82$ N/mm^2	1M
	iii) $e = \frac{dL}{L} = \frac{2.5}{2500} = 1 \times 10^{-3}$	1M
	iv) E = modulus of elasticity $= \frac{\sigma}{e}$	
	$\therefore E = \frac{222.82}{1 \times 10^{-3}} = 222.82 \times 10^3$ N/mm^2	1M
	v) $\mu = \frac{\text{lateral strain } (e_{La})}{\text{linear strain } (e)}$	
	$\therefore$ But lateral strain $(e_{La}) = \frac{dd}{d}$	
	$e_{La} = \frac{-0.006}{20}$	

Q.NO	SOLUTION	MARKS
	$e_{la} = -3 \times 10^{-4}$	
	$\therefore e_{la} = 3 \times 10^{-4}$ (decrement)	1M
	$\mu = \frac{e_{la}}{e} = 0.3$	
	$\boxed{\mu = 0.3}$	1M
	vi) $E = 2G(1+\mu)$	1M
	$\frac{E}{2(1+\mu)} = G \quad \therefore G = \frac{222.82 \times 10^3}{2(1+0.3)}$	1M
	$G = \frac{222.82 \times 10^3}{2.6}$	
	$G = 85.7 \times 10^3 \ N/mm^2$	1M
b)	A cube of 100 mm side is acted upon by stresses along the three directions such that $\sigma_x = 50 \ N/mm^2$ (T), $\sigma_y = 40 \ N/mm^2$ (C) $\sigma_z = 30 \ N/mm^2$ (T) Find : i) strains in each direction ii) Change in the volume of a cube iii) if $\sigma_z = 0$ what will be the strain along z-directions Take $E = 2 \times 10^5 \ N/mm^2$ & $\mu = 0.25$	

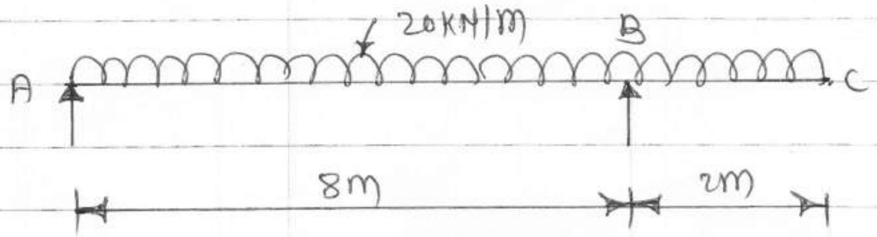
Q .NO	SOLUTION	MARKS
		
	i) strain in x-direction (e_x)	
	$e_x = \frac{\sigma_x}{E} - \frac{\mu \sigma_y}{E} - \frac{\mu \sigma_z}{E}$	1M
	$e_x = \frac{1}{E}[\sigma_x - \mu\sigma_y - \mu\sigma_z)$	
	$e_x = \frac{1}{2\times10^5}[50 - 0.25(-40) - 0.25\times30]$	
	$e_x = \frac{1}{2\times10^5}[50+10-7.5]$	
	$\boxed{e_x = 2.625\times10^{-4}}$	1M
	ii) $e_y = \frac{1}{E}[\sigma_y - \mu\sigma_x - \mu\sigma_z)$	1M
	$= \frac{1}{2\times10^5}[-40 - 0.25(50) - 0.25(30)]$	
	$= \frac{1}{2\times10^5}[-40 - 12.5 - 7.5]$	

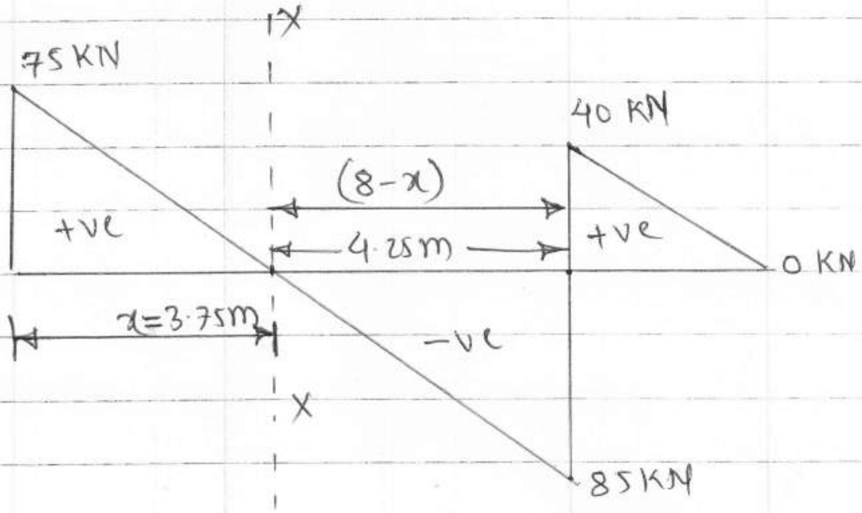
Q .NO	SOLUTION	MARKS
	$e_y = \frac{-60}{2\times10^5}$	
	$\boxed{e_y = -3\times10^{-4}}$	1M
	iii) Strain in z-direction (e_z)	
	$e_z = \frac{1}{E}\left[\sigma_z - \mu\sigma_x - \mu\sigma_y\right]$	1M
	$e_z = \frac{1}{2\times10^5}\left[30 - 0.25(50) - 0.25(-40)\right]$	
	$= \frac{1}{2\times10^5}\left[30 - 12.5 + 10\right]$	
	$= \frac{1}{2\times10^5}\left[40 - 12.5\right]$	
	$\boxed{e_z = 1.375\times10^{-4}}$	1M
	iv) To find change in volume of the cube δv	
	$\frac{\delta v}{v} = e_x + e_y + e_z$	
	$\delta v = (e_x + e_y + e_z)v$	
	$\delta v = \left[(2.625\times10^{-4}) + (-3\times10^{-4}) + (1.375\times10^{-4})\right]\times100^3$	
	$\delta v = (1\times10^{-4})\times100^3$	
	$\boxed{\delta v = 100\ mm^3}$	1M

Q .NO	SOLUTION	MARKS
	v) if $\sigma_z = 0$ what will be strain along z-direction.	
	$e_z = \frac{1}{E}(\sigma_z - \mu\sigma_x - \mu\sigma_y)$	1M
	$e_z = \frac{1}{2\times10^5}(0 - 0.25\times(50) - 0.25(-40))$	
	$e_z = \frac{1}{2\times10^5}(-12.5+10)$	
	$\boxed{e_z = -1.125\times10^{-5}}$	1M
c)	Draw S.F.D & BMD of Beam as shown in fig. also find the point of contra flexure.	
		
	i) To find reactions RA & RB	
	$R_A + R_B = 20\times10$	
	$R_A + R_B = 200$ ——— i)	

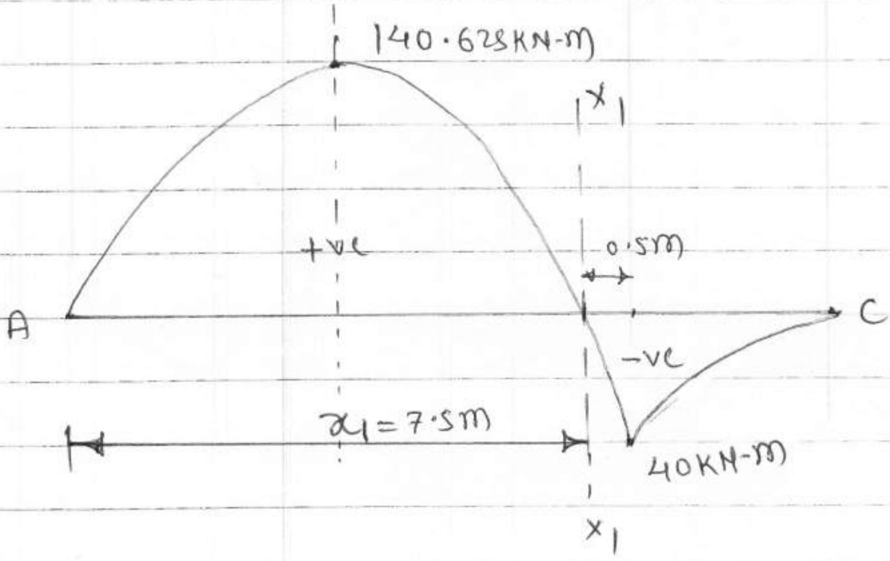
Q.NO	SOLUTION	MARKS
	$\Sigma M_A = 20 \times 10 \times \frac{10}{2} - R_B \times 8$	
	$\Sigma M_A = 1000 - 8R_B$	
	$8R_B = 1000$	
	$\boxed{R_B = 125\ KN}$	
	$\therefore R_A = 200 - R_B$	
	$R_A = 200 - 125$	
	$\boxed{R_A = 75\ KN}$	
	ii) step-2 shear force calculation	
	$F_C = 0$	
	$F_{BR} = 20 \times 2 = 40\ KN$	
	$F_{BL} = 40 - 125$	
	$F_{BL} = -85\ KN$	
	$F_A = -85 + 20 \times 8$	
	$F_A = -85 + 160$	
	$F_A = 75\ KN$	02M
	iii) B.M. calculation	
	$M_C = 0\ KN\text{-}m$	
	$M_B = -(20 \times 2) \times \frac{2}{2} = -20 \times 2 \times 1 = -40\ KN\text{-}m$	
	$M_A = 0\ KN\text{-}m$	02M
	iv) To locate the position of point of contrashear	
	$\frac{85}{8-x} = \frac{75}{x}$	
	$85x = 75(8-x)$ $\therefore 160x = 600$	01M
	$85x = 600 - 75x$ $\boxed{x = 3.75m}$ from left.	

Q.NO	SOLUTION	MARKS





01M



01M

$M_{xx} = M_{pres} = 75 \times x - 20 \times x \times x/2$

$M_{xx} = 75 \times 3.75 - 20 \times 3.75 \times 3.75/2$

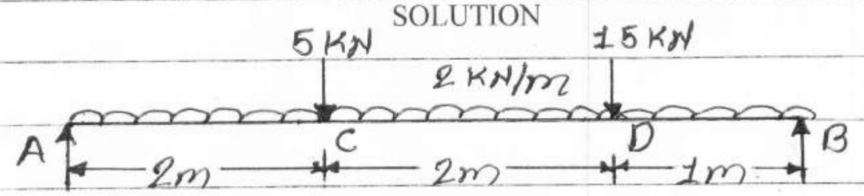
$M_{xx} = 281.25 - 140.625$

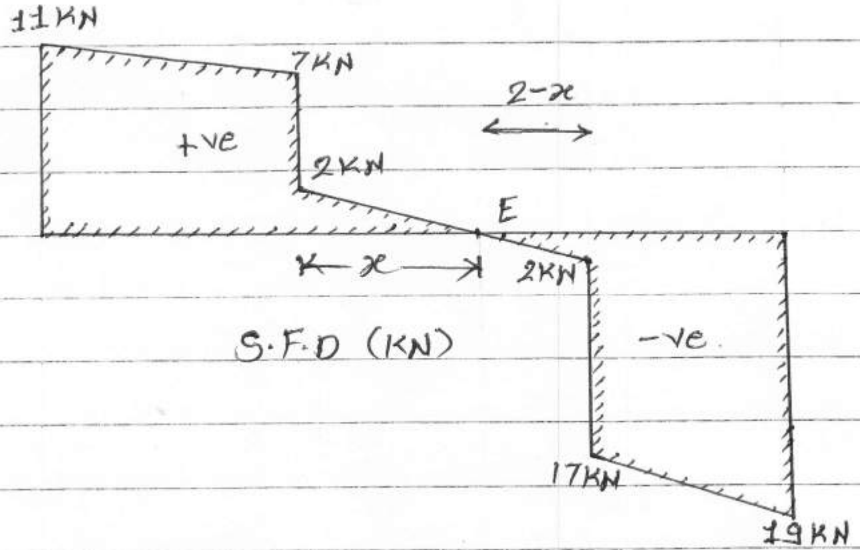
$\boxed{M_{xx} = 140.625 \text{ KN-m}}$ (sagging ½M

Q.NO	SOLUTION	MARKS
	To locate the point of contraflexure (P_{cf})	
	$M_{x_1} = 75x_1 - 20 \times x_1 \times \frac{x_1}{2}$	
	$0 = 75x_1 - 10x_1^2$	
	$0 = x_1(75 - 10x_1)$	
	$\therefore 75 - 10x_1 = 0$	
	$75 = 10x_1$	
	$x_1 = 7.5$ m from A	
	point of contraflexure (P_{cf}) = 7.5 m from 'A'	½ M

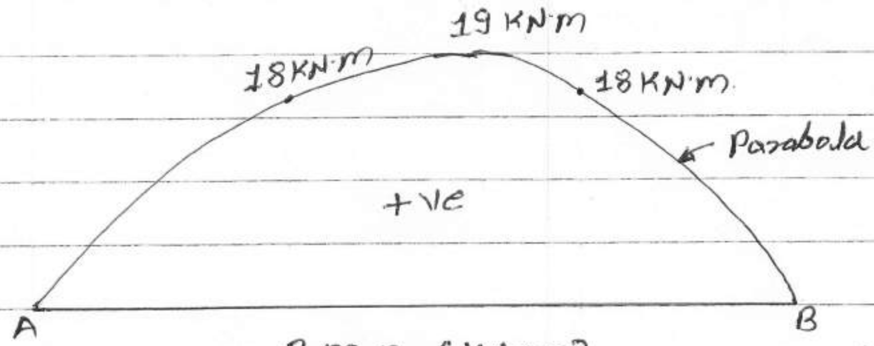
Q .NO	SOLUTION	MARKS

Q5 a)





S.F.D (KN) — 1M



B.M.D (KN·m) — 1M

→ i) Support reactions

$\Sigma F_y = 0$

$R_A + R_B = 5 + 15 + (2\times5) = 30$ KN

$\Sigma M@A = 0$ Taking moment @ A

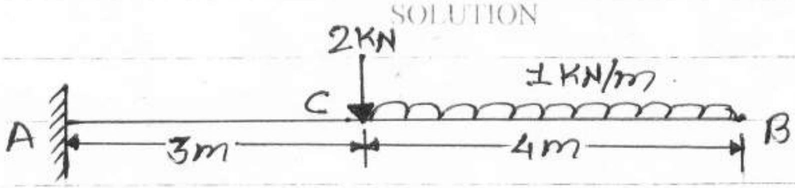
$(2\times5\times2.5) + (5\times2) + (15\times4) - R_B\times5 = 0$

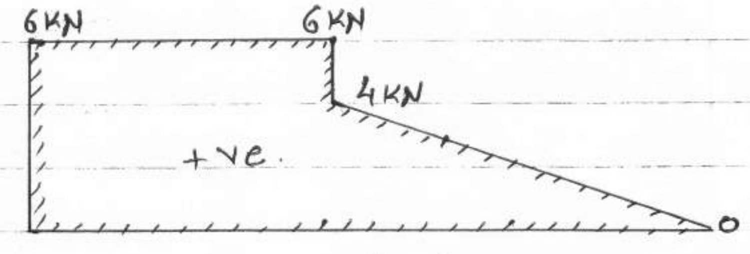
$\therefore R_B = 19$ KN — 1M

$\therefore R_A = 30 - 19 = 11$ KN

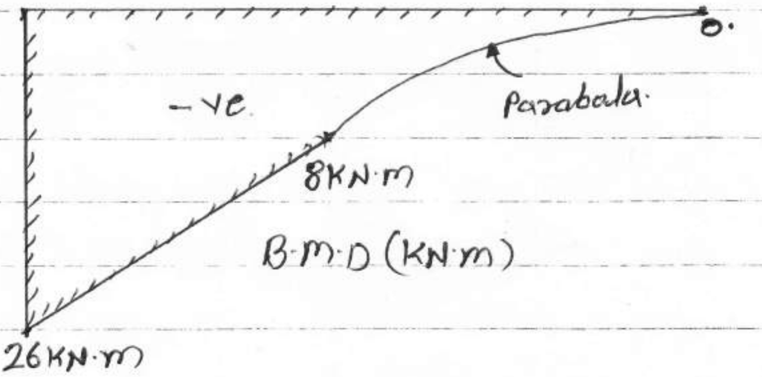
Q.NO	SOLUTION	MARKS
Q5a) Cont...	ii) Shear force Calculation [↑+ve ↓-ve]	
	S.F at just left of A = 0	
	S.F at just right of A = R_A = 11 KN	
	S.F at just left of C = 11 − (2×2) = 7 KN	
	S.F at just right of C = 7 − 5 = 2 KN	
	S.F at just left of D = 2 − (2×2) = −2 KN	2m
	S.F at just right of D = −2 − 15 = −17 KN	
	S.F at just left of B = −17 − (2×1) = −19 KN	
	S.F at just right of B = −19 + R_B = 0 KN	
	Point of Contrashear (E)	
	from similar triangle.	
	$\frac{2}{x} = \frac{2}{2-x}$	
	$(2-x)2 = 2x$	
	$4 = 4x \quad \therefore x = 1m.$	1m
	iii) Bending moment Calculation [↻+ve, ↺-ve]	
	B.M@A = B.M@B = 0 ...... S.S. ends	
	B.M@C = (11×2) − (2×2×1) = 18 KN.m	
	B.M@D = (11×4) − (2×4×2) − (5×2) = 18 KN.m	2m
	B.M@E = Max.B.M = (11×3) − (2×3×1.5) − (5×1)	
	= 19 KN.m.	

Q. NO	SOLUTION	MARKS
Q5 b)		





S.F.D (KN) — 2m



B.M.D (KN.m) — 2m

i) Support reaction

$\Sigma F_y = 0$

$R_A = 2 + (1 \times 4) = 6\ KN$

ii) Shear force Calculation [↑+ve ↓-ve]

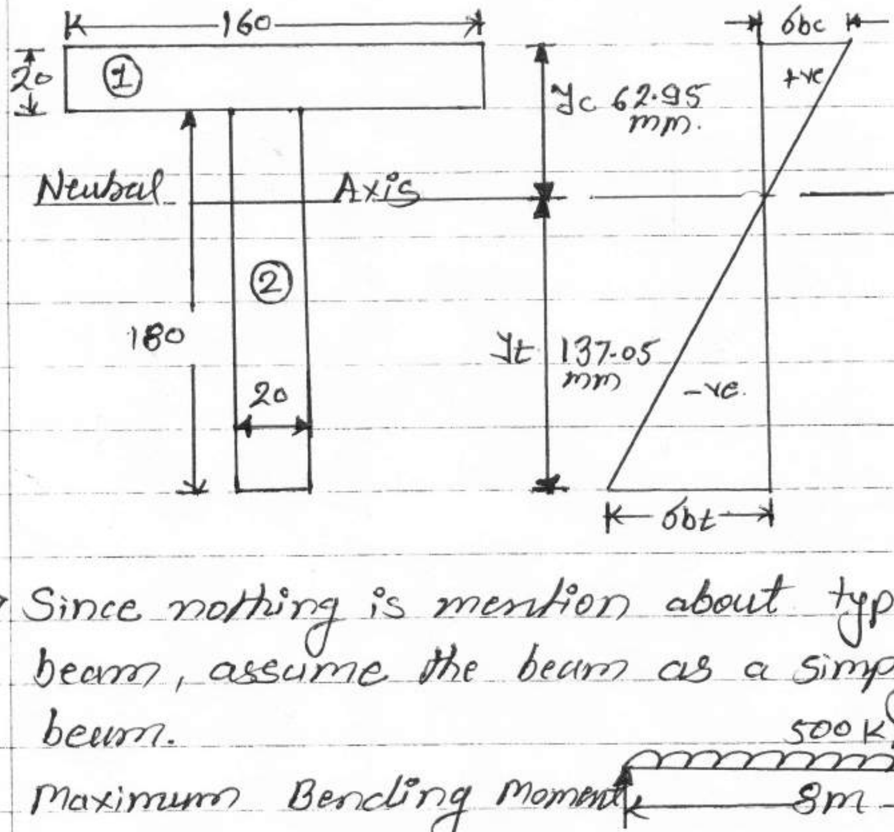
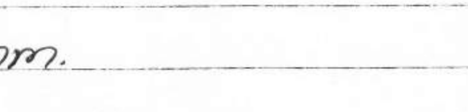
S.F at just left of A = 0 KN

S.F at just right of A = R_A = 6 KN

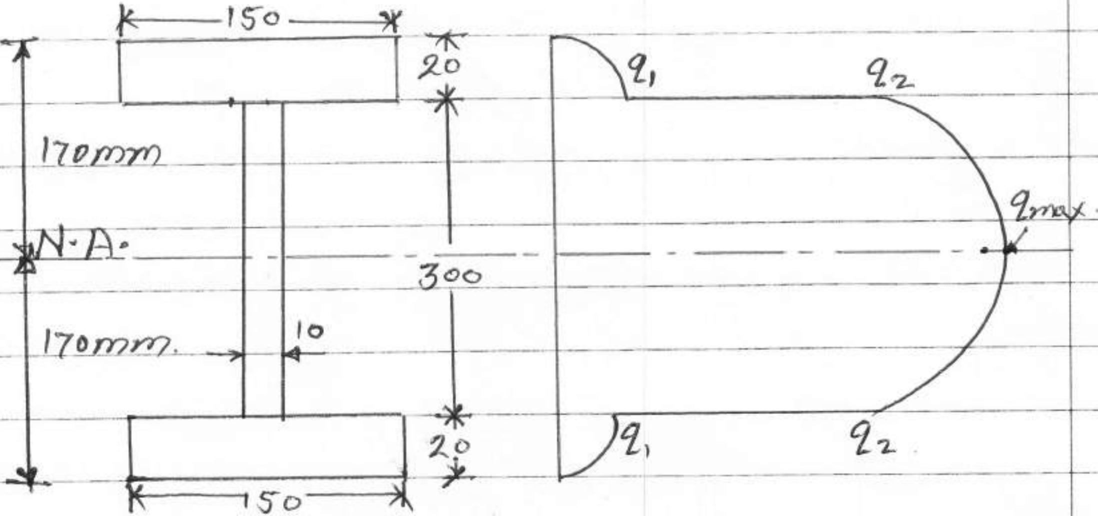
S.F at just left of C = 6 KN — 2m

S.F at just right of C = 6 − 2 = 4 KN

S.F at B = 4 − (1×4) = 0 KN

Q.NO	SOLUTION	MARKS
Q5b Cont...	iii) Bending moment Calculation [-ve , +ve]	
	B.M @ B = 0 ..... free end.	
	B.M @ C = $-(1\times4\times2) = -8$ KN.m.	2m
	B.M @ A = $-(1\times4\times5)-(2\times3) = -26$ KN.m.	
Q5c)		1M
	i) Since nothing is mention about type of beam, assume the beam as a simply supported beam.	
	Maximum Bending Moment 	
	$M_{max} = \frac{wl^2}{8} = \frac{500\times8^2}{8} = 4000$ KN.m.	1m.
	$M_{max} = 4000\times10^6$ N.mm.	
	ii) Depth of N.A from base.	
	$y_t = \frac{a_1y_1+a_2y_2}{a_1+a_2}$.	

Q. NO	SOLUTION	MARKS
Q5c Cont...	$y_t = \dfrac{(160\times20\times190)+(180\times20\times90)}{(160\times20)+(180\times20)} = 137.05\ mm$	1m
	$\therefore\ y_c = 200 - y_t = 200-137.05 = 62.95\ mm$	1m.
	iii) Moment of Inertia @ X-X axis.	
	$I_{xx} = I_{N.A} = \left[\dfrac{bd^3}{12}+Ah^2\right]_1 + \left[\dfrac{bd^3}{12}+Ah^2\right]_2$	
	$= \left[\dfrac{160\times20^3}{12}+160\times20(137.05-190)^2\right] + \left[\dfrac{20\times180^3}{12}+180\times20(137.05-90)^2\right]$	
	$I_{xx} = 9.078\times10^6 + 17.68\times10^6 = 26.75\times10^6\ mm^4$	1m
	iv) Using flexural formula. $\dfrac{M}{I} = \dfrac{\sigma_b}{y}$	1m
	$\therefore\ \sigma_{bc} = \dfrac{M}{I}\cdot y_c = \dfrac{4000\times10^6}{26.75\times10^6}\times62.95$	
	$\sigma_{bc} = 9.413\times10^3\ N/mm^2$	1m
	$\therefore\ \sigma_{bt} = \dfrac{M}{I}\times y_t = \dfrac{4000\times10^6}{26.75\times10^6}\times137.05$	
	$\sigma_{bt} = 20.493\times10^3\ N/mm^2$	1m

Q.NO	SOLUTION	MARKS
Q6a)		
	→ Since section is symmectrical the N.A. will be at a distance of 170 mm from the base	1M
	1) M.I. of section $I_{xx} = \frac{BD^3}{12} - \frac{bd^3}{12}$	2M
	$I_{xx} = \frac{150 \times 340^3}{12} - \frac{140 \times 300^3}{12} = 176.3 \times 10^6 \, mm^4$	
	Let us Consider portion of beam above N.A.	
	2) Shear stress at the junction of flange & web by Considering width of flange. ($b = 150mm$)	
	$q_1 = \frac{SA\bar{y}}{bI} = \frac{100 \times 10^3 \times 150 \times 20 \times 160}{150 \times 176.3 \times 10^6}$	
	$q_1 = 1.81 \; N/mm^2$	1M
	3) Shear stress at the junction of flange & web by Considering width of web ($b = 10 \, mm$)	

Q.NO	SOLUTION	MARKS
Q6a) Cont...	$q_2 = \frac{SA\bar{Y}}{bI} = \frac{100\times10^3 \times 150\times20\times160}{10\times176.3\times10^6}$	
	$q_2 = 27.22\ N/mm^2$	1M
	4) Additional shear stress due to web area above N.A.	
	$q_{additional} = \frac{SA\bar{Y}}{bI} = \frac{100\times10^3\times(150\times10)\times75}{10\times176.3\times10^6}$	
	$q_{additional} = 6.38\ N/mm^2$	1M
	5) The maximum shear stress is at N.A & is given by.	
	$q_{max} = q_{N.A} = q_2 + q_{additional}$.	
	$q_{max} = 27.22 + 6.38 = 33.60\ N/mm^2$	2M
Q6b →	Given, $L = 4m$, $P_b = 2KN$, Column hinged at both the ends.	
	Case - I,	
	$Le = L = 4m$ .... both ends are hinge	1M
	buckling load $P_b = \frac{\pi^2 EI}{Le^2}$	
	$\therefore\ 2 = \frac{\pi^2 EI}{(4)^2}$	
	$\therefore\ EI = \frac{2\times(4)^2}{\pi^2} = 3.242\ KN\cdot m^2$	1M

Q .NO	SOLUTION	MARKS
Q6 b Cont...	Case- II	
	Same tube of length 4.5m used as a Column if.	
	i> Both ends are fixed.	
	$\therefore Le = \frac{L}{2} = \frac{4.5}{2} = 2.25m.$	1m
	$\therefore$ Buckling load $P = \frac{\pi^2 EI}{(Le)^2} = \frac{\pi^2 \times 3.242}{(2.25)^2}$	
	$\therefore P = 6.32$ KN.	1m
	ii> One end is fixed & the other is hinged.	
	$\therefore Le = \frac{L}{\sqrt{2}} = \frac{4.5}{\sqrt{2}} = 3.18m.$	1m
	$\therefore$ Buckling load $P = \frac{\pi^2 EI}{(Le)^2} = \frac{\pi^2 \times 3.242}{(3.18)^2}$	
	$\therefore P = 3.164$ KN.	1m
	iii> One end is fixed & the other free	
	$\therefore Le = 2L = 2\times4.5 = 9m.$	1m
	$\therefore$ Buckling load $P = \frac{\pi^2 EI}{(Le)^2} = \frac{\pi^2 \times 3.242}{(9)^2}$	
	$\therefore P = 0.395$ KN	1m

Q.NO	SOLUTION	MARKS
Q6 c	Given,	
→	$d = 20mm$	
	$L = 1000mm$	
	$P = 1000 N$	
	$h = 250mm$	
	$E = 2\times10^5 N/mm^2$	
	∴ Area of bar $= \frac{\pi}{4}(20)^2 = 314.15 mm^2$	1m
	Volume of bar $V = A\times L = 314.15\times1000$	
	$V = 314.15\times10^3 mm^3$	
	i) Maximum instantaneous stress (σ)	
	$\sigma = \frac{P}{A} + \sqrt{\left(\frac{P}{A}\right)^2 + \frac{2PhE}{AL}}$	1m
	$= \frac{1000}{314.15} + \sqrt{\left(\frac{1000}{314.15}\right)^2 + \frac{2\times1000\times250\times2\times10^5}{314.15\times1000}}$	
	$= 3.18 + \sqrt{10.11 + 318.319\times10^3}$	
	$\sigma = 567.38 N/mm^2$	2m
	ii) Elongation of bar (δL)	
	$\delta L = \frac{\sigma L}{E} = \frac{567.38\times1000}{2\times10^5} = 2.83mm$	2m
	iii) Strain energy stored (U)	
	$U = \frac{\sigma^2}{2E}\times V = \frac{\sigma^2}{2E}\times AL$	1m
	$U = \frac{(567.38)^2}{2\times2\times10^5}\times314.15\times1000 = 252.82\times10^3 N\cdot mm$	1m
	∴ $U = 252.82$ N·m or Joule	