THEORY OF STRUCTURES-1

THIRD SEMESTER DIPLOMA EXAMINATION IN CIVIL ENGINEERING

13 PAGES

SCHEME OF VALUATION

(Scoring Indicators)

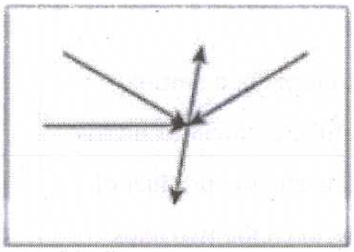
Question Number	Scoring Indicator	Split-up score	Total
I- 1)	Part- A Moment of a force about any point is equal to the algebraic sum of the moments of its components about the same point Or The algebraic sum of the moments of a system of unequal forces about any point in their plane is equal to the moment of their resultant about that point	2	2
I- 2)	The theorem of parallel axis states that the moment of inertia of a lamina about any axis in the plane of lamina equals the sum of the moment of inertia about a parallel centroidal axis in the plane of the lamina and the product of the area of the lamina and the square of the distance between the two axes. $I_{AB} = I_G + Ay^2$ I_{AB} = moment of inertia of a lamina about any axis I_G = moment of inertia about a parallel centroidal axis A= area of the lamina y = distance between the two axes	2	2
I- 3)	Poisson's ratio is defined as the the ratio of the lateral strain to the axial strain (longitudinal strain) is called Poisson's ratio.	2	2
I- 4)	Circumferential stress or otherwise known as hoop stress. In thin cylinders, the pressure due to the fluid inside causes a bursting force on to the cylinder walls due to which the tensile stress are induced in the cylinder. The direction of this tensile stress is along the circumference. The stress so induced is called a hoop or circumferential stress. Given by the equation Pd/2t ; where P is internal pressure intensity, d is the internal diameter, t is the thickness of cylinder wall.	2	2
I- 5)	$\frac{M}{I} = \frac{f}{y} = \frac{E}{R}$ Where M is bending moment I = Moment of inertia f = Bending stress; y is the distance of fiber from neutral axis E = young's modulus; R is the radius of curvature	2	2
II- 1)	Part- B 1) Coplanar concurrent forces : the system of consist of group of forces lying in a same plane and their line of action meet or intersect at a		

point

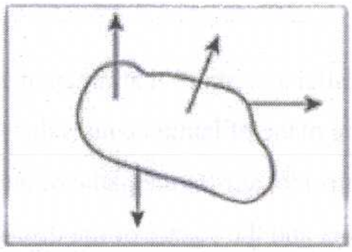
2) <u>Coplanar non concurrent forces</u> : The system of consist of group of forces lying in a same plane and but their line of action do not meet at a point

3) <u>Collinear forces :</u> the system of consist of group of forces lying in a same plane and their line of action lie on same straight line.

4) <u>Coplanar like parallel forces :</u> the system of consist of group of forces lying in a same plane and their line of action are parallel to each other and acting in one direction.

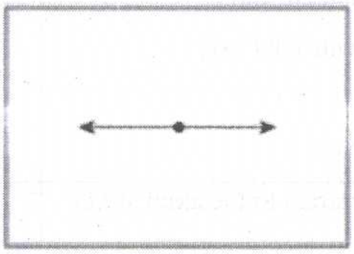


Coplanar concurrent

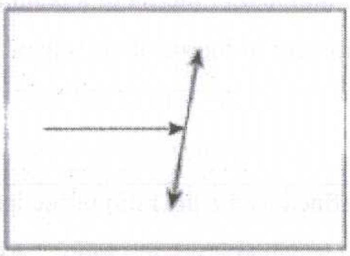


Coplanar non-concurrent

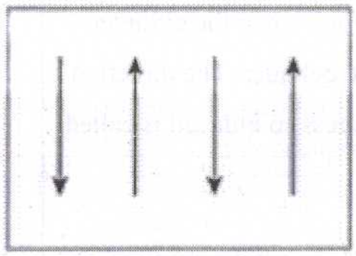
2 marks each (3x2=6) | 6



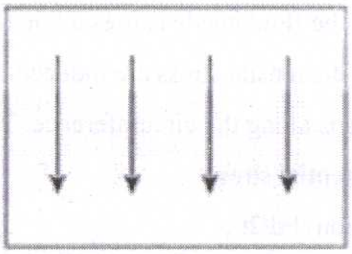
Collinear



Coplanar Non Collinear



Coplanar paraller

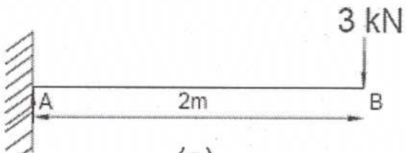
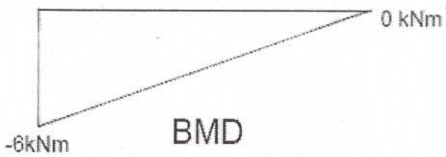


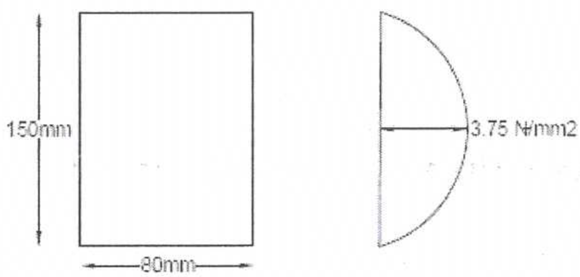
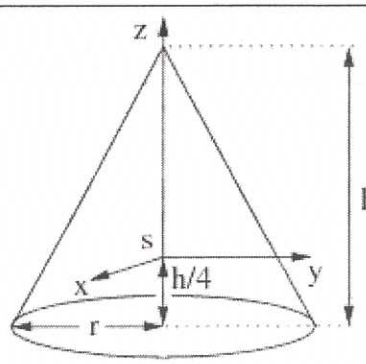
Coplanar like paraller

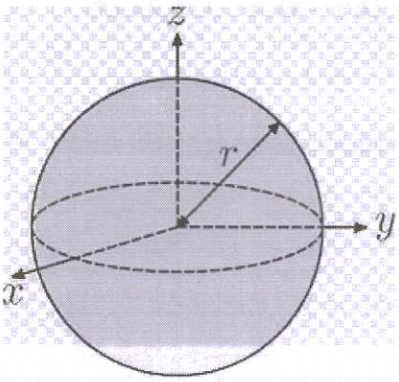
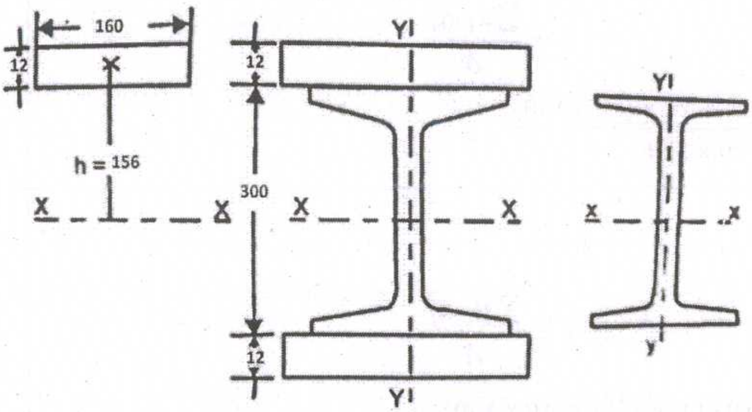
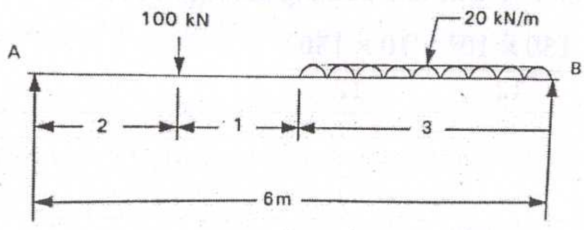
5) <u>Coplanar unlike parallel forces :</u> the system of consist of group of forces lying in a same plane and their line of action are parallel are parallel to each other but acting in different direction.

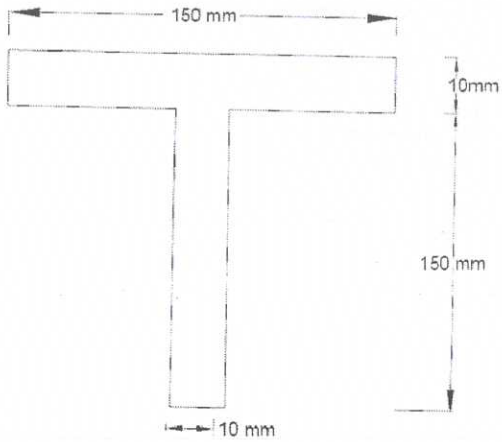
6) <u>Non coplanar concurrent forces</u> : The system of consist of group of forces not lying in a same plane and but their line of action meet at a point

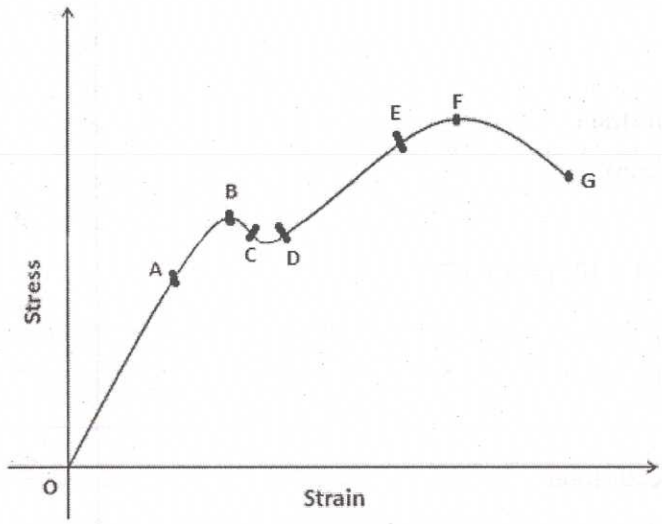
7) <u>Non coplanar non concurrent forces</u> : The system of consist of group of forces not lying in a same plane and their line of action do not meet at a point.

II- 2)	1) <u>Strength :</u> Ability of a material to resist break down 2) <u>Stiffness :</u> Ability of a material to resist deformation 3) <u>Elasticity:</u> Property of material to regain its original shape and size after removal of external load 4) <u>Plasticity:</u> Property of material by which material retains the deformation even after removal of external load. 5) <u>Malleability:</u> Property by which material can be drawn into sheets 6) <u>Ductility:</u> Property by which material can be drawn into wires 7) <u>Hardness:</u> Property by which a material resists penetration 8) <u>Toughness:</u> Property by which material to resist fracture due to impact loads	4 x 1.5=6	6
II- 3)	$E = 2N(1 + \frac{1}{m})$ $E = 3K(1 - \frac{2}{m})$ $E = \frac{9KN}{(3K + N)}$ Where E is modulus of elasticity; N is rigidity modulus; K is bulk modulus and 1/m is poisson's ratio	3x2=6	6
II-4)	 (a)  BMD  SFD	1 3 2	6
II- 5)	Diameter, D= 30mm, radius R= 15mm Torque, T=6kNm = $6x10^6$ Nmm Polalar moment of inertia , J = πD^4/32 = 79521.56 mm^4 Maximum shear stress, q_s= T x R/J = $6x10^6$ x15/79481.25 = 1131.768 N/mm^2	1 2 3	6

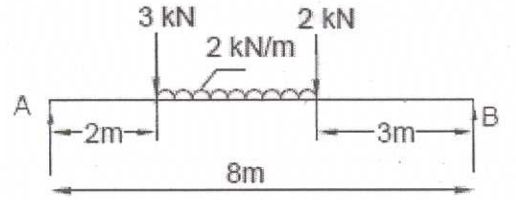
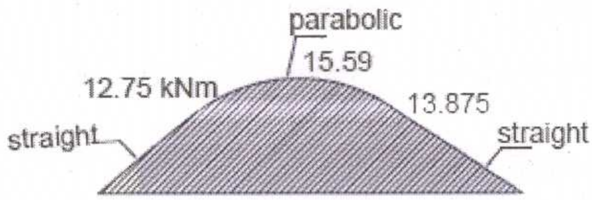
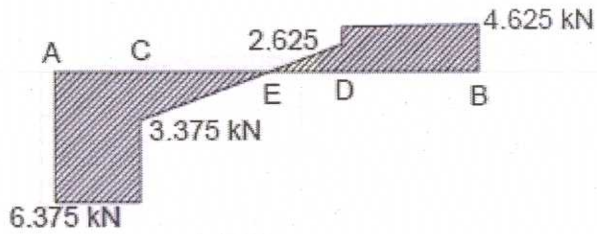
II- 6)	i. The beam is subjected to pure bending and is therefore free from shear force. ii. The material of the beam is homogenous and Isotropic. iii. Each longitudinal fibre is free to expand or contract independently. iv. The value of the Young's modulus is the same for beam material in tension as well as compression. v. A transverse section of the beam which is a plane before bending will remain a plane after bending. vi. The elastic limit is not exceeded. vii. The resultant thrust or pull on a transverse section of the beam is zero. viii. The radius of curvature of the deflected beam is very large compared with the dimensions of the cross section of the beam. ix. The transverse section of the beam is symmetrical about an axis passing through the centroid of the section and parallel to the plane of bending.	6x1=6	6
II- 7)	b= 80mm, d =150mm shearing force,F = 30kN, = 30 x 10^3N Aerage shear stress, τ_{av} = F/A = 30 x 10^3/(80x150) = 2.5 N/mm^2 Maximumshear stress, τ_{max} = 1.5 τ_{av} = 1.5 x 2.5 = 3.75 N/mm^2 	2 2 2	6
	PART C		
III (a)		2	

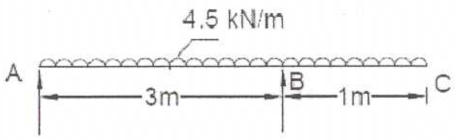
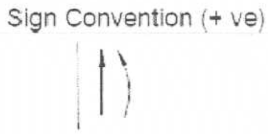
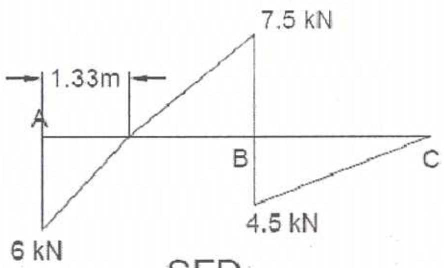
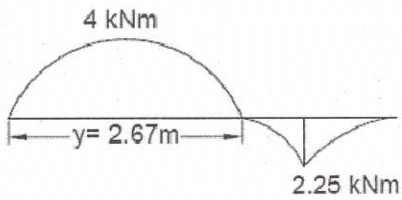
	Center of gravity of cone is (0,0,h/4)  Center of gravity of sphere is (0,0,0)	1 2 1	6
III (b)	 Compound section is symmetrical, The centroid 'G' lies at the point of intersection of two axes of symmetry. M.I about XX is $I_{XX} = I_{XX}$ of I section + I_{XX} of 2 plates $= 73.329 \times 10^6 + [\, I_G + ah^2] \times 2$ $= 73.329 \times 10^6 + [\frac{1}{12}(160 \times 12^3) + 12 \times 160 \times 156^2] \times 2$ $= 1.6683 \times 10^8\ mm^4$	2 3 4	9
IV (a)	**1.**  R_A and R_B be the reactions at supports We have, $\sum F_y = 0$ i.e., $R_A + R_B = 100 + 20 \times 3 = 160kN$ Taking moments about B, $\sum M_B = 0$	1 1	6

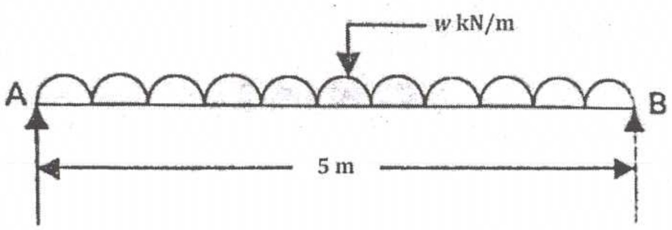
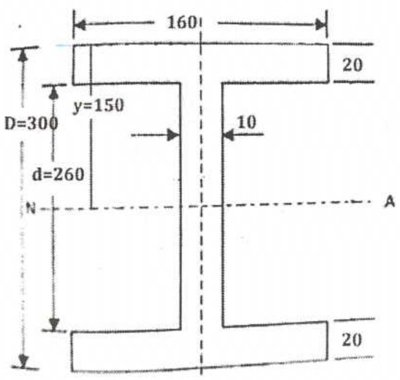
	$R_A \times 6 = 100{\times}4{+}20{\times}3{\times}\frac{3}{2} \quad = 400{+}90 = 490$ $R_A \quad = \frac{460}{9} = 81.67\ \text{kN}$ $R_B \quad = 160 - 81.67 = 78.33\ \text{kN}$	2 2	
IV (b)	 Calculation of Centriod; $(\bar{x}, \bar{y})$ $$\bar{x} = \frac{\sum x_i\ a_i}{A}$$ $= \frac{75 \text{ x } (10 \text{ x } 150) + 75 \text{ x } (10 \text{ x } 150)}{2 \text{ x } 10 \text{ x } 150}$ = 75 mm $$\bar{y} = \frac{\sum y_i\ a_i}{A}$$ $= \frac{75 \text{ x } (10 \text{ x } 150) + (150 + \frac{10}{2}) \text{ x } (10 \text{ x } 150)}{2 \text{ x } 10 \text{ x } 150}$ = 115 mm Moment of inertia parallel to X-X axis and passing through centroid $I_{xx} = \frac{10 \times 150^3}{12} + 10\ \times 150 \times (115 - 75)^2 + \frac{150 \times 10^3}{12} + \ 10 \times 150 \times (155 - 115)^2$ $= \ 7.625 \text{ x } 10^6 \text{ mm}^4$ Moment of inertia parallel to Y-Y axis and passing through centroid $$I_{yy} = \frac{150 \times 10^3}{12} + \frac{10 \times 150^3}{12}$$ $= \ 2.825 \text{ x } 10^6 \text{ mm}^4$	1 1 3.5 3.5	9
V (a)	Area of Column, $A = \frac{\pi d^2}{4} = \frac{\pi (350)^2}{4} = 96162.5\ \text{mm}^2$	1	

	Area of Steel, $A_S = 4 \times \frac{\pi(25)^2}{4} = 1962.5\ mm^2$ Area of concrete, $A_C = A - A_S = 94200\ mm^2$ Stress in the concrete, $\sigma_C = 4.5\ N/mm^2$ Stress in the steel, $\sigma_S = E_S/E_C \times \sigma_C = 18 \times 4.5 = 81\ N/mm^2$ Safe load, $P = \sigma_S A_S + \sigma_C A_C$ $= 582.86$ kN.	1 1 1 2	6
V (b)	 a : Limit of proportionality b : Elastic limit c : Upper yield point d : Lower yield point d-e : Ductile stage e-f : Plastic stage f : Max, or ultimate load g :Breaking point or fracture load	4.5 4.5	9
VI (a)	l = 2000mm temperature, t = 30K i) Stress in the bar do not yield $\sigma_1 = \alpha\, t\, E$ $= 24 \times 10^{-6} \times 30 \times 120 \times 10^3 = 86.4\ N/mm^2$ ii) Stress in the bar when the ends yield by 1mm $\sigma_2 = [\alpha\, t - \frac{\Delta}{l}]\, E$	3	

	$= \{24 \times 10^{-6} \times 30 - \frac{1}{2000}\}\ 120 \times 10^3 = 26.4\ N/mm^2$	3	6
VI (b)	l = 4000mm b=30mm t- 20mm axial pull, P= 120kN = $120 \times 10^3 N$ E = $200 \times 10^3\ N/mm^2$ Poisson's ratio = 0.3 = 1/m Change in length, $\delta l = \frac{Pl}{AE}$, $\delta l = \frac{120 \times 10^3 \times 4 \times 10^3}{30 \times 20 \times 200 \times 10^3}$ = 4mm Change in width, δb = b x lateral strain Lateral strain = 1/m x (linear strain) = 1/m x (δl/l) = 0.3 x $(4/4 \times 10^3)$ = 0.3x $(1 \times 10^{-3}) = 3 \times 10^{-4}$ δb = $30 \times 3 \times 10^{-4} = 9 \times 10^{-3}$ mm change in thickness, δt = t x lateral strain = $20 \times 3 \times 10^{-4} = 6 \times 10^{-3}$ mm	3.5 3 2.5	9
VII (a)	Torque , T = $50 \times 10^6\ N/mm^2$ Maximum shear stress, $\tau = 80\ N/mm^2$ T = (π/16) x τ x D^3 $D^3 = 50 \times 10^6 / 7.839$ D= 147.1 mm	2 2 2	6

VII (b)			
	Sign Convention (+ ve) 		
	 BMD	2	
	 SFD		
	Taking moment about A R_B x 8 = 3x2 + (3x2) 3.5 + 2x5 R_B = 4.625kN R_A= 3 + 2 x 3 +2 = 11kN ∴ R_A= 6.375kN	1	
	Shear Force S.F at B , F_B (just right) = 0 F_B (just left) = 4.625kN F_D (just right) = 4.625kN F_D (just left) = 2.625kN F_C (just right) = -3.375 kN F_C (just left) = 6.375kN F_A = 6.375kN	2	9
	Bending moment, M_B = 0 M_D = 4.625 x 3 = 13.875 kNm M_C = 6.375 x 2 = 12.75 kNm Maximum bending moment	2	
	Maximum bending will occur at point E Let CE be x From the geometry of the two triangles between CD $$\frac{x}{3.375} = \frac{3-x}{2.625}$$	1	

	$x = 1.69m$ Max BM at E = $6.375(2 + 1.69) - 3\text{x}\ 1.69 - (2 \text{ x } 1.69)\frac{1.69}{2}$ = 15.59 kNm	1	
VIII (a)	Circumferential or hoop stress = $\frac{pd}{2t}$ P= 1 N/mm^2 ; d = 4000mm and t = 10mm Circumferential stress = $\frac{1 \times 4000}{2 \times 10}$ = 200 N/mm^2 Longitudinal stress = $\frac{pd}{4t}$ = 400 N/mm^2	3 3	6
VIII (b)	   SFD  BMD $R_A + R_B$ = 4.5 x 4 = 18kN R_B X 3 = 4.5 x 4 x 2 = 36 R_B = 12 kN R_A= 6 kN SF at C = 0 kN B (right) = -4.5 x 1 = -4.5 kN B (left) = -4.5 x 1 + 12 = 7.5 kN A (right) = -6kN A (left) = 0 kN Bending moment BM at A= C = 0	2 1 2	9

	B=4.5 x 1 x 0.5 = 2.25kNm Max BM will occur at x $$\frac{x}{6}=\frac{3-x}{7.5}$$ x=1.33m Max BM = 6 x 1.33 – 4.5x 1.33x1.33/2 = 4kNm Point of contraflexure 6 x y – 4.5 x y x y/2 = 0 $2.25y^2 - 6y = 0$ y=2.67m	2 1 1	
IX (a)	 Span = l = 5000mm Width = b = 100mm Depth = d = 200mm $\tau_{max} = 0.6\ N/mm^2$ This beam is of rectangular sector having cross section 100 mm × 200 mm. For a rectangular beam maximum shear force is given by, $F_{max} = \frac{\tau_{max} \times bd}{1.5}$ $\frac{wl}{2} = \frac{0.7 \times 100 \times 200}{1.5} = 9333.33$ $w = \frac{9333.33 \times 2}{5} = 3733.33\ N/m$ $w = 3.733\ kN/m$	3 3	6
IX (b)	 Given maximum bending moment, M = 30 kNm	1	

	$= 30 \times 10^6$ Nmm Moment of inertia about XX axis, $I_{XX} = \frac{1}{12}[BD^3 - bd^3]$ $= \frac{1}{12}[160 \times 300^3 - 150 \times 260^3]$ $= 1.403 \times 10^8\ mm^4$ $\frac{M}{I} = \frac{f}{y}$ $\Rightarrow f = \frac{M \cdot y}{I}$ y=300/2 =150 section modulus = I/y = $9.3533 \times 10^8\ mm^3$ $f = \frac{30 \times 10^6}{9.3533 \times 10^8} = 32.074\ N/mm^2$ Maximum bending stress = $32.074\ N/mm^2$	2 2 2	9
X (a)	Here, $l = 8m = 8000mm$ $d = 300mm$, so that $y = \frac{300}{2} = 150mm$ $f = 120\ N/mm^2$ we have to find Moment of Inertia, $I = \frac{bd^3}{12}$ $I = \frac{300 \times 200^3}{12}$ $= 200 \times 10^6\ mm^4$ To find w i.e., rate of udl For a simply supported beam carrying with w N/m of udl over entire length, maximum BM = $\frac{wl^2}{8}$ ∴moment of resistance, $M = \frac{wl^2}{8}$ $M = w \times \frac{8000^2}{8}$ N-mm But $M = f \times Z = f \times \frac{I}{y}$ $w \times \frac{8000^2}{8}$ N-mm $= 120 \times \frac{200 \times 10^6}{100}$ $w = 30\ kN/m$	2 2 2	6
X (b)	Maximum shear force = $F_{max} = 60\ kN = 60 \times 10^3 N$		

	The beam is of rectangular sector with b = 100mm and d = 300mm. i) $\tau_{avg} = \frac{F_{max}}{bd} = \frac{60 \times 10^3}{100 \times 300} = 2\ N/mm^2$ ii) Maximum shear stress is 1.5 times the average shear stress. i.e., $\tau_{max} = 1.5\ \tau_{avg}$ $= 1.5 \times 2 = 3\ N/mm^2$ iii) Shear stress at a distance 30mm above neutral axis y=30mm $I = \frac{bd^3}{12} = 100 \times 300^3/12 = 225 \times 10^6 mm^4$ $\tau = \frac{F}{2I}\left[\frac{d^2}{4} - y^2\right]$ $\tau = \frac{60 \times 10^3}{2 \times 225 \times 10^6}\left[\frac{300^2}{4} - 30^2\right]$ $\tau = 2.88 N/mm^2$	2 2.5 4.5	9