

Strength of Materials (313308)

Model Question Paper — Set C · Model Answers

Units: N, mm, N/mm² (= MPa) unless stated. 1 kN = 10³ N, 1 kN·m = 10⁶ N·mm, 1 GPa = 10³ N/mm².

Q.1

a) Radius of gyration and relation with MI. 2 M

Radius of gyration (k): The distance from a given axis at which the entire area of a lamina may be assumed to be concentrated without changing its moment of inertia about that axis.

Relation: $I = A \cdot k^2 \Rightarrow k = \sqrt{I/A}$

(Definition 1 M, relation 1 M)

b) Modulus of rigidity and bulk modulus. 2 M

Modulus of rigidity (G): Within the elastic limit, the ratio of shear stress to shear strain. $G = \tau/\phi$ (N/mm²).

Bulk modulus (K): When a body is subjected to equal direct stress σ in all three mutually perpendicular directions, the ratio of this stress to the volumetric strain. $K = \sigma/e_v$.

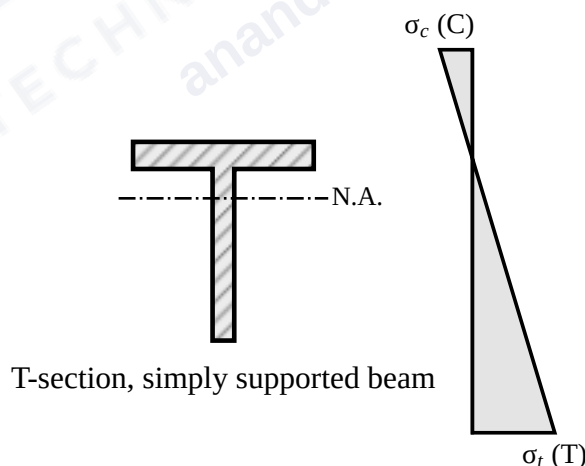
(1 M each)

c) Relation between load, S.F. and B.M. 2 M

- Rate of change of shear force = rate of loading: $dF/dx = -w$ (w downward u.d.l.)
- Rate of change of bending moment = shear force: $dM/dx = F$. Hence B.M. is maximum where S.F. is zero or changes sign.

(1 M each)

d) Bending stress distribution for T-section (SS beam). 2 M



Compression above N.A., tension below; N.A. through centroid (nearer the flange)

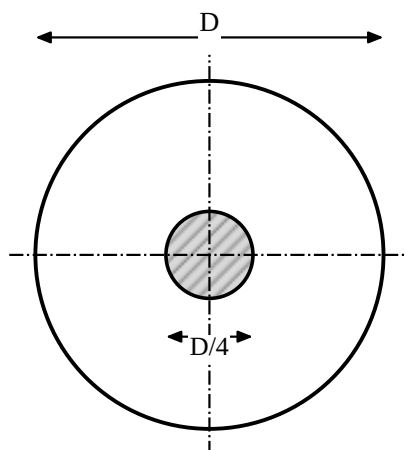
Stress is zero at N.A. and varies linearly; maximum stress occurs at the extreme fibre farther from the N.A. (bottom of web, tension).

(Diagram 2 M)

e) Core of section; core of circular section.

2 M

Core (kernel) of a section: The area around the centroid of a section within which a load must act so that no tensile stress is developed anywhere in the section. For a solid circular section of diameter D it is a concentric circle of diameter $D/4$ ($e \leq D/8$).



Core of a circular section — circle of diameter $D/4$

(Definition 1 M, sketch 1 M)

f) Proof stress and percentage elongation.

2 M

Proof stress: For a material without a definite yield point, the stress at which a specified permanent (non-proportional) strain, usually 0.2 %, is produced. It is used in place of yield stress (e.g. for Tor steel).

Percentage elongation: The increase in gauge length of a tensile test specimen after fracture, expressed as a percentage of the original gauge length $= (L_f - L_0)/L_0 \times 100$. It is a measure of ductility.

(1 M each)

g) MI of triangle and semicircle.

2 M

Section	M.I.
Triangle (base b , height h) about base	$bh^3/12$
Triangle about centroidal axis parallel to base	$bh^3/36$
Semicircle (radius r) about its diameter	$\pi r^4/8 = \pi d^4/128$

(Any two correct, 1 M each)

Q.2

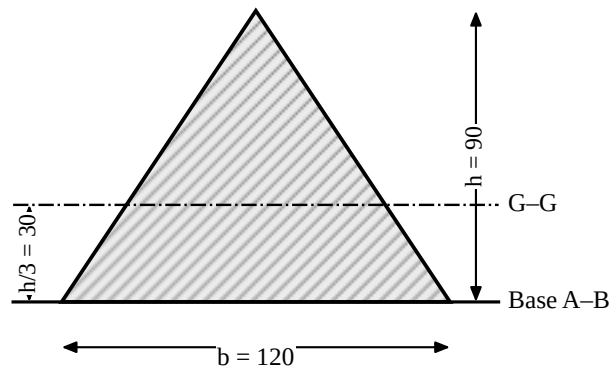
a) Parallel axis theorem; MI of triangle about base.

4 M

Parallel axis theorem: The moment of inertia of a plane area about any axis parallel to its centroidal axis is equal to the moment of inertia about the centroidal axis plus the product of the area and the square of the distance between the two axes.

$$I_{AB} = I_G + A \cdot h^2$$

where I_G = M.I. about centroidal axis, A = area of lamina, h = distance between the two parallel axes.



Triangle with centroidal axis G-G and base

$I_G = bh^3/36 = 120 \times 90^3/36 = 2.43 \times 10^6 \text{ mm}^4$; $A = \frac{1}{2} \times 120 \times 90 = 5400 \text{ mm}^2$; distance of centroid from base $h' = h/3 = 30 \text{ mm}$

$$I_{\text{base}} = I_G + A \cdot (h/3)^2 = 2.43 \times 10^6 + 5400 \times 30^2 = 7.29 \times 10^6 \text{ mm}^4 (= bh^3/12 \checkmark)$$

(Statement 1 M, IG 1 M, I base 2 M)

b) Ultimate, yield, working stress and factor of safety.

4 M

- Ultimate stress:** The maximum stress a material can withstand before failure = maximum load / original cross-sectional area.
- Yield stress:** The stress at which the material starts yielding, i.e. strain increases without appreciable increase in load (yield point).
- Working stress:** The safe (permissible) stress used in design; it is obtained by dividing the ultimate or yield stress by the factor of safety.
- Factor of safety:** The ratio of ultimate stress (or yield stress) to working stress. $F.O.S. = \sigma_u / \sigma_w$ (always > 1).

(1 M each)

c) Minimum diameter of wire and elongation.

4 M

$$\text{Required area } A = P/\sigma = 10 \times 10^3 / 140 = 71.429 \text{ mm}^2$$

$$\pi d^2/4 = 71.429 \Rightarrow d = \sqrt{(4 \times 71.429/\pi)} = 9.537 \text{ mm (minimum)}$$

$$\text{Using } d = 10 \text{ mm: } A = \pi \times 10^2/4 = 78.54 \text{ mm}^2; \sigma = 10000/78.54 = 127.324 \text{ N/mm}^2 (< 140 \checkmark)$$

$$\text{Elongation } \delta L = \sigma L/E = 127.324 \times 3000 / (2 \times 10^5) = 1.9099 \text{ mm}$$

(Area 1 M, d 1 M, δL 2 M)

d) Cantilever 2 m — SFD and BMD.

4 M

$$\text{Reaction at fixed end A: } R_A = \text{total load} = 3 + 2 + 1 \times 2 = 7 \text{ kN } (\uparrow)$$

Fixing moment at A = 9 kN·m (hogging). For a cantilever, S.F. and B.M. are calculated from the free end.

Shear force calculations (from the free end (right side), upward force on left = +ve):

$$\text{S.F. just right of A} = 1 \times 2 + 3 + 2 = 7 \text{ kN}$$

$$\text{S.F. just left of B} = 1 \times 1 + 3 + 2 = 6 \text{ kN}$$

$$\text{S.F. just right of B} = 1 \times 1 + 2 = 3 \text{ kN}$$

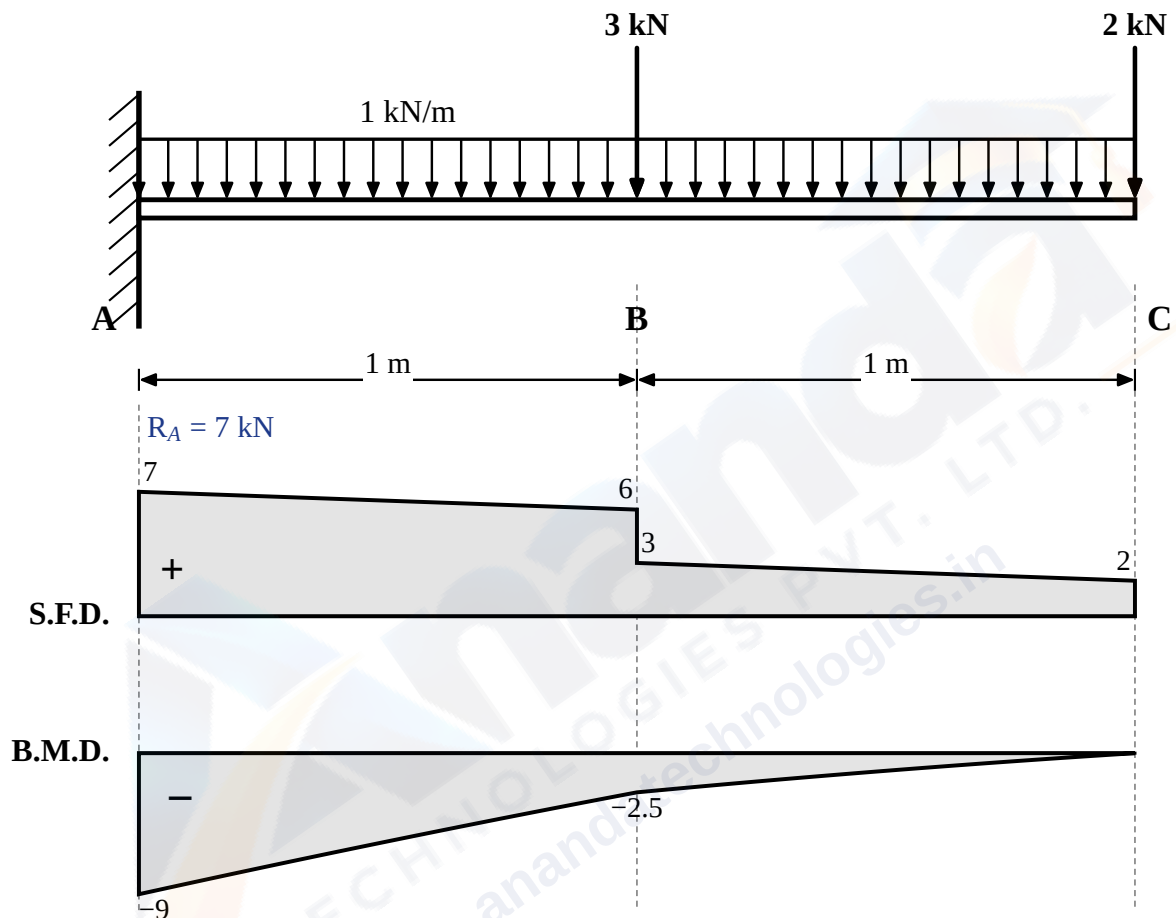
$$\text{S.F. just left of C} = 2 = 2 \text{ kN}$$

Bending moment calculations (sagging = +ve, hogging = -ve):

$$M_A = -1 \times 2 \times 1 - 3 \times 1 - 2 \times 2 = -9 \text{ kN}\cdot\text{m}$$

$$M_B = -1 \times 1 \times 0.5 - 2 \times 1 = -2.5 \text{ kN}\cdot\text{m}$$

$$M_C = 0 \text{ (free end)}$$

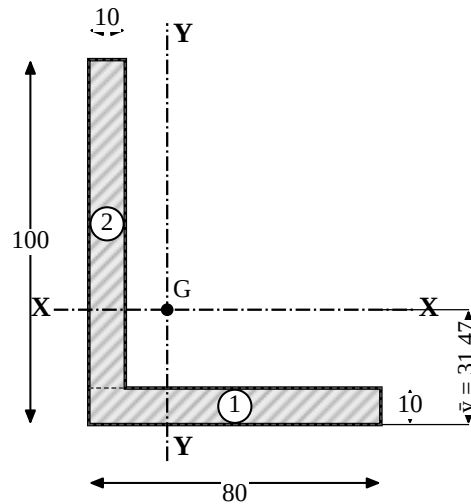


S.F. and B.M. diagrams

Q.3

a) MI of unequal angle section $100 \times 80 \times 10$.

4 M



Angle section, longer leg vertical (mm)

Divide into ① horizontal leg 80×10 and ② vertical leg 10×90 (above it). Reference: bottom-left corner.

Part	a (mm ²)	y (mm)	a·y (mm ³)	I _G own (mm ⁴)	h = y - $\bar{y}$ (mm)	a·h ² (mm ⁴)
Horizontal leg 80×10	800	5	4000	6666.7	-26.47	5.6055×10^5
Vertical leg 10×90	900	55	49500	6.075×10^5	23.53	4.9827×10^5
Σ	1700		53500	6.1417×10^5		1.0588×10^6

$$\bar{y} = \Sigma a \cdot y / \Sigma a = 53500 / 1700 = 31.471 \text{ mm}; \quad \bar{x} = \Sigma a \cdot x / \Sigma a = (800 \times 40 + 900 \times 5) / 1700 = 21.471 \text{ mm}$$

$$I_{XX} = 1.673 \times 10^6 \text{ mm}^4$$

Part	a (mm ²)	x (mm)	a·x (mm ³)	I _G own (mm ⁴)	h = x - $\bar{x}$ (mm)	a·h ² (mm ⁴)
Horizontal leg 80×10	800	40	32000	4.2667×10^5	18.53	2.7467×10^5
Vertical leg 10×90	900	5	4500	7500	-16.47	2.4415×10^5
Σ	1700		36500	4.3417×10^5		5.1882×10^5

$$I_{YY} = 9.5299 \times 10^5 \text{ mm}^4$$

(Centroid 1 M, I_{xx} 2 M, I_{yy} 1 M)

b) Poisson's ratio and modulus of rigidity from E and K.

4 M

$$E = 3K(1 - 2\mu) \Rightarrow 2.05 \times 10^5 = 3 \times 1.6 \times 10^5 (1 - 2\mu) \Rightarrow 1 - 2\mu = 2.05/4.8 = 0.4271 \Rightarrow \mu = 0.2865$$

$$E = 2G(1 + \mu) \Rightarrow G = 2.05 \times 10^5 / [2 \times (1 + 0.2865)] = 79676 \text{ N/mm}^2$$

$$(\text{Check: } E = 9KG/(3K + G) = 2.05 \times 10^5 \text{ N/mm}^2 \checkmark)$$

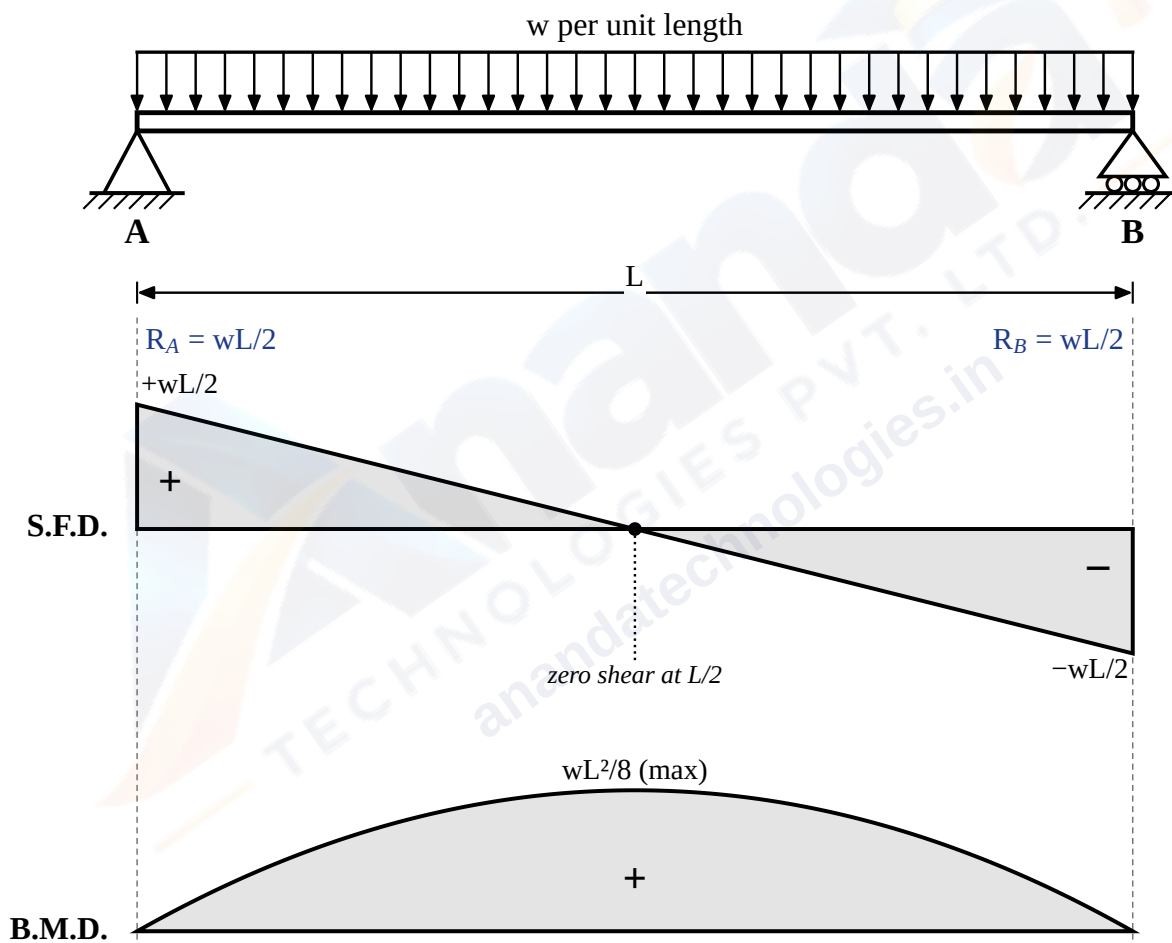
(μ 2 M, G 2 M)

c) SS beam with full-span u.d.l. — standard case; max BM for 8 m span.

4 M

Reactions $R_A = R_B = wL/2$. At a section x from A: S.F. $F_x = wL/2 - wx$ (linear); B.M. $M_x = (wL/2)x - wx^2/2$ (parabolic).

S.F. = $+wL/2$ at A, 0 at $x = L/2$, $-wL/2$ at B. B.M. = 0 at A and B; maximum at $x = L/2$: $M_{\max} = (wL/2)(L/2) - w(L/2)^2/2 = wL^2/8$.



SS beam with u.d.l. w per unit length over span L

For $L = 8 \text{ m}$, $w = 12 \text{ kN/m}$: $M_{\max} = 12 \times 8^2/8 = 96 \text{ kN}\cdot\text{m}$, at mid-span (4 m from either support).

(SFD 1 M, BMD 1 M, max value and position 2 M)

d) Solid circular column — eccentric load.

4 M

Given: $d = 200 \text{ mm}$, $P = 80 \text{ kN}$, $e = 40 \text{ mm}$.

$$A = \pi \times 200^2/4 = 31415.93 \text{ mm}^2$$

$$Z = \pi d^3/32 = \pi \times 200^3/32 = 7.854 \times 10^5 \text{ mm}^3$$

$$\text{Direct stress } \sigma_0 = P/A = 80000 / 31415.93 = \mathbf{2.546 \text{ N/mm}^2 \text{ (C)}}$$

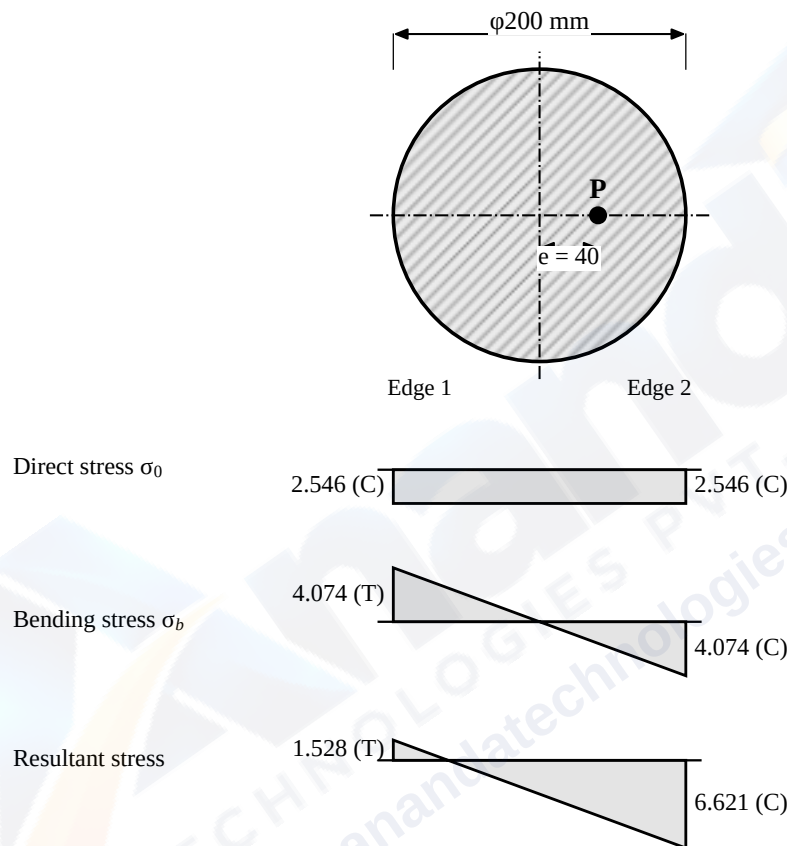
$$\text{Bending moment } M = P \cdot e = 80000 \times 40 = 3.2 \times 10^6 \text{ N}\cdot\text{mm}$$

$$\text{Bending stress } \sigma_b = M/Z = 3.2 \times 10^6 / 7.854 \times 10^5 = \mathbf{4.074 \text{ N/mm}^2}$$

$$\sigma_{\max} = \sigma_0 + \sigma_b = 2.546 + 4.074 = \mathbf{6.621 \text{ N/mm}^2 \text{ (compressive)}}$$

$$\sigma_{\min} = \sigma_0 - \sigma_b = 2.546 - 4.074 = \mathbf{-1.528 \text{ N/mm}^2 \text{ i.e. } 1.528 \text{ N/mm}^2 \text{ tensile}}$$

Limit of eccentricity for no tension: $e_{\max} = Z/A = 7.854 \times 10^5 / 31415.93 = 25 \text{ mm}$. Since $e = 40 \text{ mm} > 25 \text{ mm}$, tension develops at the edge away from the load.



(stresses in N/mm^2 ; C = compression, drawn below the line; T = tension, drawn above)

Fig.: Section with load position and stress distribution

For a solid circle $e_{\max} = d/8 = 200/8 = 25 \text{ mm}$. Since $e = 40 \text{ mm} > 25 \text{ mm}$, **tension is developed** on the side away from the load.

(σ_0 1 M, σ_b 1 M, $\sigma_{\max}/\sigma_{\min}$ 1 M, justification 1 M)

Q.4

a) Safe u.d.l. on timber beam (bending and shear).

4 M

(i) Bending: $Z = bd^2/6 = 120 \times 200^2/6 = 8 \times 10^5 \text{ mm}^3$; $M = \sigma Z = 10 \times 8 \times 10^5 = 8 \times 10^6 \text{ N}\cdot\text{mm} = 8 \text{ kN}\cdot\text{m}$

$$M_{\max} = wL^2/8 \Rightarrow w = 8M/L^2 = 8 \times 8 / 3^2 = 7.111 \text{ kN/m}$$

(ii) Shear: $\tau_{\max} = 1.5 F/(bd) \Rightarrow F = 1 \times 120 \times 200 / 1.5 = 16000 \text{ N} = 16 \text{ kN}$; $F_{\max} = wL/2 \Rightarrow w = 2 \times 16/3 = 10.667 \text{ kN/m}$

Safe u.d.l. = smaller value = 7.111 kN/m (bending governs).

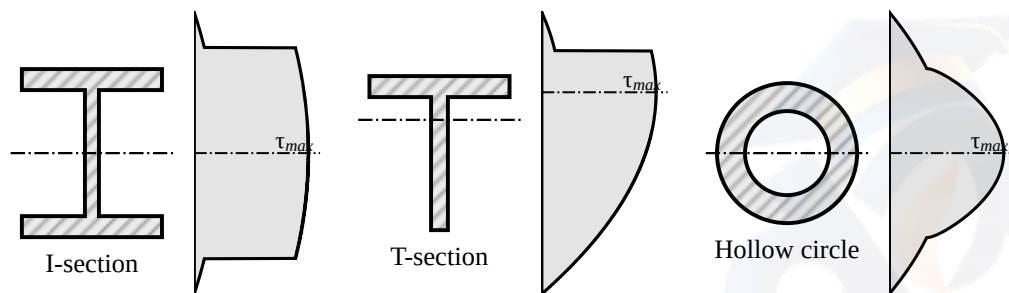
(Bending 2 M, shear 1 M, safe load 1 M)

b) Shear stress equation; distribution for I, T and hollow circular sections.

4 M

$$\tau = F \cdot A \cdot \bar{y} / (I \cdot b)$$

τ = shear stress at the layer considered, F = shear force at the section, A = area of the section above (or below) the layer, $\bar{y}$ = distance of the centroid of area A from the N.A., I = M.I. of the whole section about N.A., b = width of the section at the layer.



Shear stress distribution: I-section (jump at flange–web junction, max at N.A.), T-section, hollow circle

(Equation with terms 1 M, three sketches 3 M)

c) Buckling load, effective length, slenderness ratio; Euler formula.

4 M

Buckling (crippling) load: The maximum axial load which a column can carry just before it starts buckling (bending sideways).

Effective length (L_e): Length of an equivalent column with both ends hinged having the same crippling load; it depends on end conditions (L , $2L$, $L/2$, $L/\sqrt{2}$).

Slenderness ratio: $\lambda = L_e/k_{\min}$, where $k_{\min}$ = least radius of gyration.

Euler's formula (crippling / buckling load of a long column):

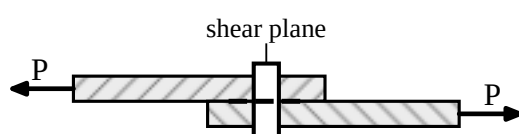
$$P_E = \pi^2 EI_{\min} / L_e^2$$

P_E = Euler's crippling load (N), E = modulus of elasticity (N/mm²), $I_{\min}$ = least moment of inertia of the section (mm⁴), L_e = effective length of column (mm).

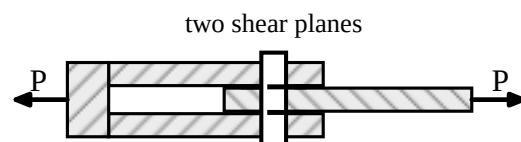
(Each definition 1 M, Euler formula 1 M)

d) Shear stress in pin of knuckle joint (double shear).

4 M



(a) Single shear — lap joint
(one shear plane, area = $\pi d^2/4$)



(b) Double shear — fork and eye (knuckle) joint
(two shear planes, area = $2 \times \pi d^2/4$)

Single shear (lap joint) and double shear (fork and eye / knuckle joint)

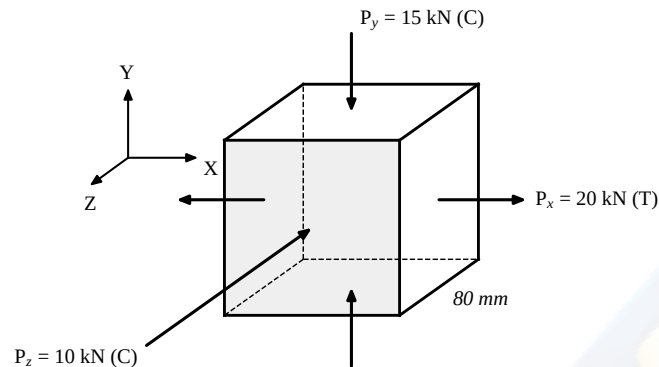
In a knuckle joint the eye (single plate) lies between the two plates of the fork. Under pull, the pin tends to shear at the **two** surfaces between the eye and the fork plates, so it is in **double shear**; area resisting shear = $2 \times \pi d^2/4$.

$$\tau = P / (2 \times \pi d^2/4) = 40 \times 10^3 / (2 \times \pi \times 20^2/4) = 40000 / 628.32 = \mathbf{63.662 \text{ N/mm}^2}$$

(Explanation with sketch 2 M, τ 2 M)

e) Change in volume of cube.

4 M



Cube of 80 mm side

Face area = $80^2 = 6400 \text{ mm}^2$; $\sigma_x = 20000/6400 = 3.125 \text{ (T)}$, $\sigma_y = -2.3438 \text{ (C)}$, $\sigma_z = -1.5625 \text{ N/mm}^2 \text{ (C)}$

$$e_v = (\sigma_x + \sigma_y + \sigma_z)(1 - 2\mu)/E = (3.125 - 2.3438 - 1.5625) \times (1 - 0.5) / (2 \times 10^5) = \mathbf{-1.9531 \times 10^{-6}}$$

$$\delta V = e_v \times V = -1.9531 \times 10^{-6} \times 80^3 = \mathbf{-1 \text{ mm}^3}, \text{ i.e. volume decreases by } 1 \text{ mm}^3.$$

(Stresses 1 M, e_v 2 M, δV 1 M)

Q.5

a) SS beam 8 m — SFD, BMD, zero shear, maximum BM.

6 M

Support reactions (R_A and R_E act upward):

Taking moments about A, $\Sigma M_A = 0$:

$$R_E \times 8 = (4 \times 4) \times 2 + 10 \times 5 + 6 \times 6.5 = 121$$

$$\therefore R_E = 121 / 8 = \mathbf{15.125 \text{ kN}}$$

$$\Sigma F_y = 0: R_A + R_E = 10 + 6 + 4 \times 4 = 32 \text{ kN}$$

$$\therefore R_A = 32 - 15.125 = \mathbf{16.875 \text{ kN}}$$

Shear force calculations (from the left, upward force on left = +ve):

$$\text{S.F. just right of A} = 16.875 = \mathbf{16.875 \text{ kN}}$$

$$\text{S.F. at B} = 16.875 - 4 \times 4 = \mathbf{0.875 \text{ kN}}$$

$$\text{S.F. just left of C} = 16.875 - 4 \times 4 = \mathbf{0.875 \text{ kN}}$$

$$\text{S.F. just right of C} = 16.875 - 4 \times 4 - 10 = \mathbf{-9.125 \text{ kN}}$$

$$\text{S.F. just left of D} = 16.875 - 4 \times 4 - 10 = \mathbf{-9.125 \text{ kN}}$$

$$\text{S.F. just right of D} = 16.875 - 4 \times 4 - 10 - 6 = \mathbf{-15.125 \text{ kN}}$$

$$\text{S.F. just left of E} = 16.875 - 4 \times 4 - 10 - 6 = \mathbf{-15.125 \text{ kN}}$$

Bending moment calculations (sagging = +ve, hogging = -ve):

$$M_A = 0 \text{ (simple support)}$$

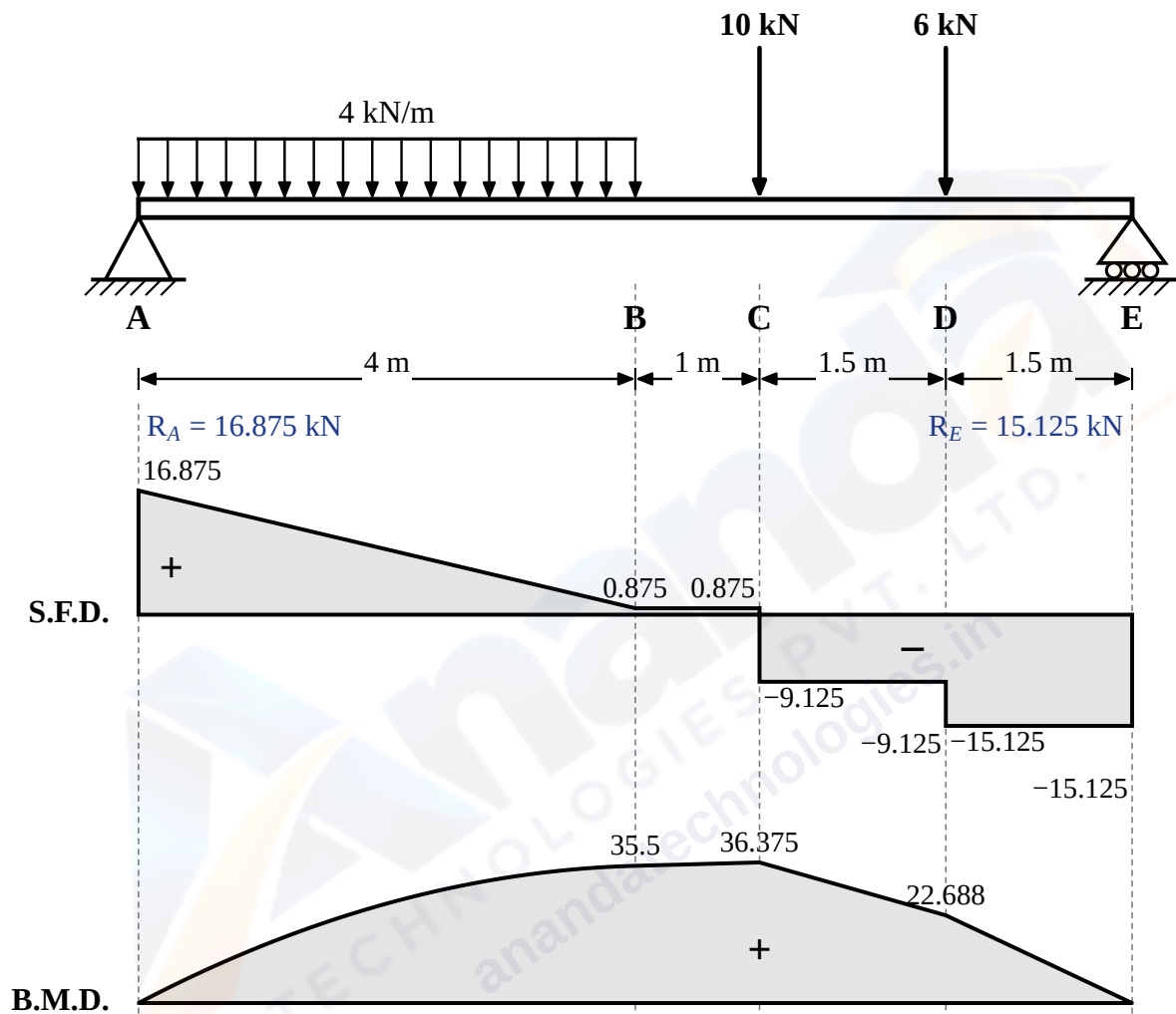
$$M_B = 16.875 \times 4 - 4 \times 4 \times 2 = 35.5 \text{ kN}\cdot\text{m}$$

$$M_C = 16.875 \times 5 - 4 \times 4 \times 3 = 36.375 \text{ kN}\cdot\text{m}$$

$$M_D = 16.875 \times 6.5 - 4 \times 4 \times 4.5 - 10 \times 1.5 = 22.688 \text{ kN}\cdot\text{m}$$

$$M_E = 0 \text{ (simple support)}$$

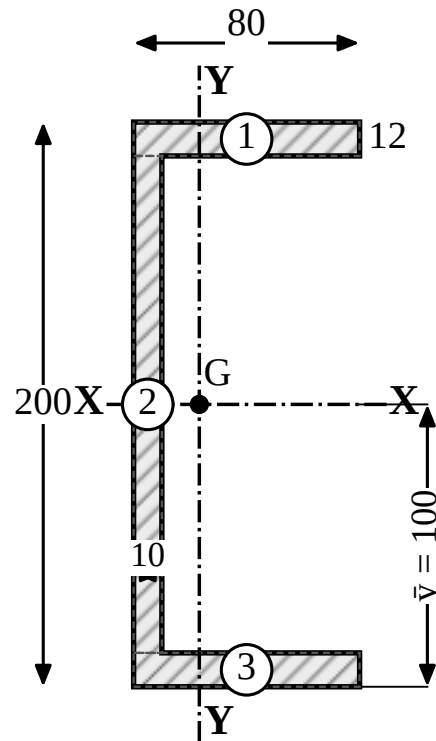
S.F. changes sign at C, so B.M. has a maximum (peak) value at C: $M_C = 36.375 \text{ kN}\cdot\text{m}$



S.F. and B.M. diagrams

b) MI of channel section.

6 M



Channel section (mm)

Parts: ① top flange 80×12 , ② web 10×176 , ③ bottom flange 80×12 . The section is symmetric about the horizontal axis, so $\bar{y} = 100$ mm.

Part	a (mm ²)	y (mm)	a·y (mm ³)	I _{G own} (mm ⁴)	h = y - $\bar{y}$ (mm)	a·h ² (mm ⁴)
Top flange 80×12	960	194	1.8624×10^5	11520	94	8.4826×10^6
Web 10×176	1760	100	1.76×10^5	4.5431×10^6	0	0
Bottom flange 80×12	960	6	5760	11520	-94	8.4826×10^6
Σ	3680		3.68×10^5	4.5662×10^6		1.6965×10^7

$$I_{XX} = 2.1531 \times 10^7 \text{ mm}^4$$

Centroid from the back (left) of web:

Part	a (mm ²)	x (mm)	a·x (mm ³)	I _{G own} (mm ⁴)	h = x - $\bar{x}$ (mm)	a·h ² (mm ⁴)
Top flange 80×12	960	40	38400	5.12×10^5	16.74	2.6899×10^5
Web 10×176	1760	5	8800	14667	-18.26	5.8689×10^5

Bottom flange 80×12	960	40	38400	5.12×10^5	16.74	2.6899×10^5
Σ	3680		85600	1.0387×10^6		1.1249×10^6

$$\bar{x} = \Sigma a \cdot x / \Sigma a = 23.261 \text{ mm}; I_{YY} = 2.1635 \times 10^6 \text{ mm}^4$$

(I_{xx} 3 M, $\bar{x}$ 1 M, I_{yy} 2 M)

c) Bar with forces at intermediate sections — net change in length.

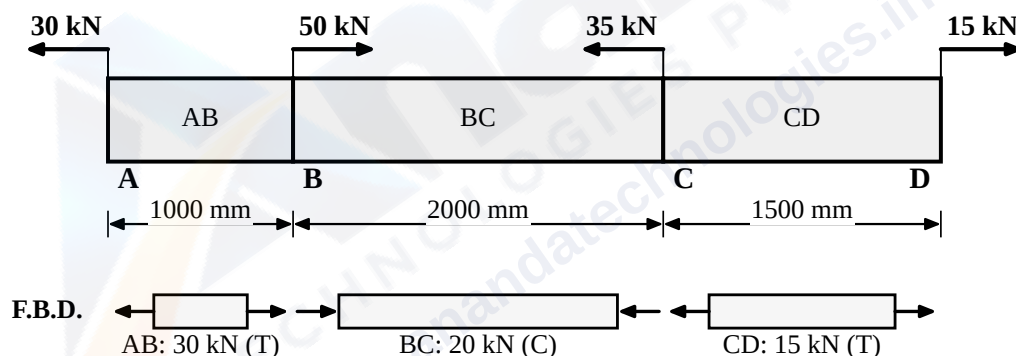
6 M

Equilibrium check: $-30 + 50 - 35 + 15 = 0 \checkmark$. $A = 600 \text{ mm}^2$, $E = 2 \times 10^5 \text{ N/mm}^2$.

Internal forces (free body diagram of each part, considering forces to the left of a section; tension +ve):

Part	Axial force P	L (mm)	A (mm ²)	Stress $\sigma = P/A$ (N/mm ²)	$\delta L = PL/AE$ (mm)
AB	30 kN (T)	1000	600	50	$+30000 \times 1000 / (600 \times 2 \times 10^5) = 0.25$
BC	20 kN (C)	2000	600	-33.333	$-20000 \times 2000 / (600 \times 2 \times 10^5) = -0.33333$
CD	15 kN (T)	1500	600	25	$+15000 \times 1500 / (600 \times 2 \times 10^5) = 0.1875$

Total change in length $\delta L = \Sigma \delta L = 0.10417 \text{ mm}$ (increase)



Free body diagrams of AB, BC and CD

(FBD / internal forces 3 M, δL 3 M)

Q.6

a) Overhanging beam — SFD, BMD and contraflexure.

6 M

Support reactions (R_A and R_C act upward):

Taking moments about A, $\Sigma M_A = 0$:

$$R_C \times 6 = 20 \times 2 + (5 \times 2) \times 7 = 110$$

$$\therefore R_C = 110 / 6 = 18.333 \text{ kN}$$

$$\Sigma F_y = 0: R_A + R_C = 20 + 5 \times 2 = 30 \text{ kN}$$

$$\therefore R_A = 30 - 18.333 = \mathbf{11.667 \text{ kN}}$$

Shear force calculations (from the left, upward force on left = +ve):

$$\text{S.F. just right of A} = 11.667 = \mathbf{11.667 \text{ kN}}$$

$$\text{S.F. just left of B} = 11.667 = \mathbf{11.667 \text{ kN}}$$

$$\text{S.F. just right of B} = 11.667 - 20 = \mathbf{-8.333 \text{ kN}}$$

$$\text{S.F. just left of C} = 11.667 - 20 = \mathbf{-8.333 \text{ kN}}$$

$$\text{S.F. just right of C} = 11.667 + 18.333 - 20 = \mathbf{10 \text{ kN}}$$

$$\text{S.F. just left of D} = 11.667 + 18.333 - 20 - 5 \times 2 = \mathbf{0 \text{ kN}}$$

Bending moment calculations (sagging = +ve, hogging = -ve):

$$M_A = 0 \text{ (simple support)}$$

$$M_B = 11.667 \times 2 = \mathbf{23.333 \text{ kN}\cdot\text{m}}$$

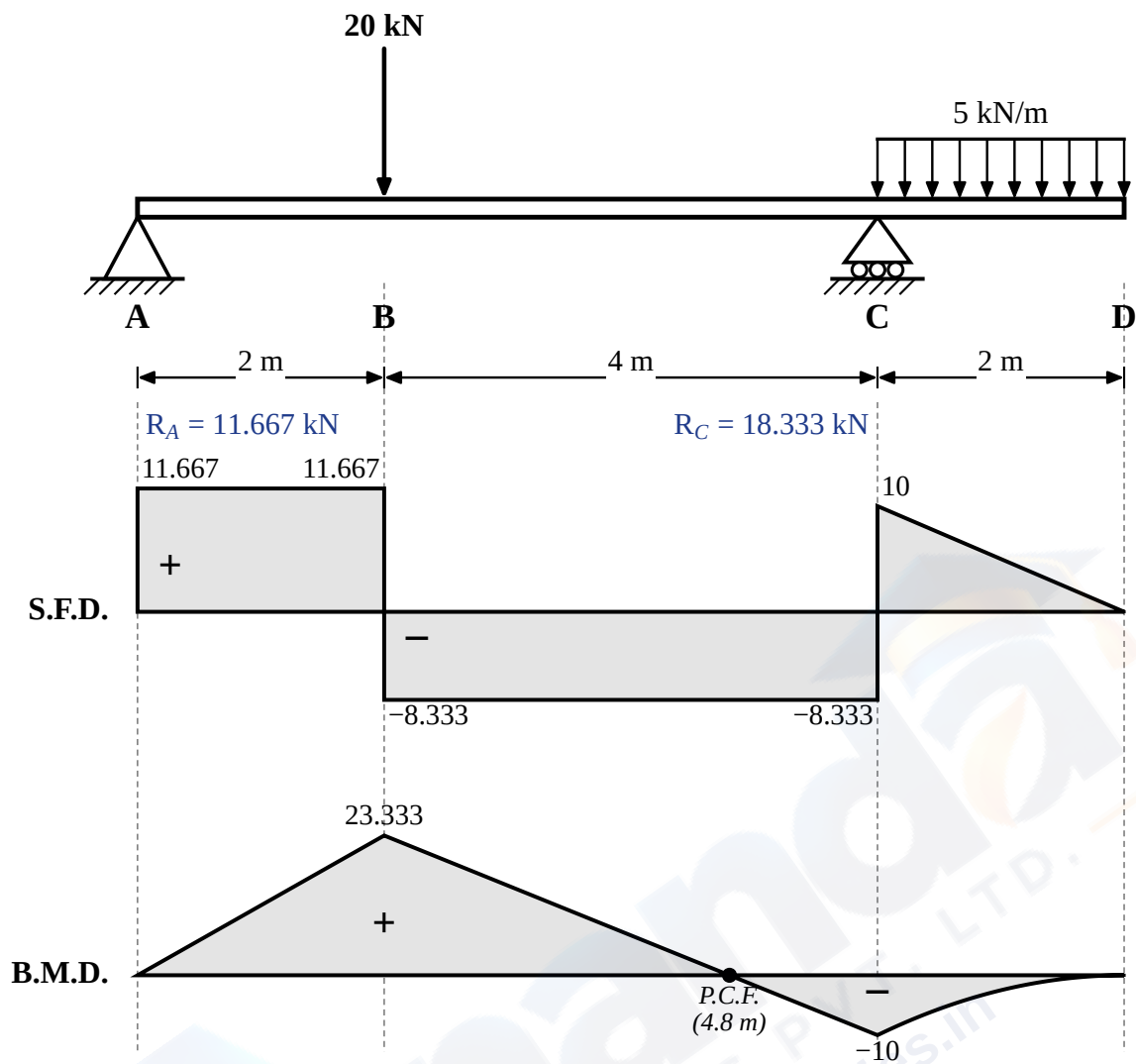
$$M_C = 11.667 \times 6 - 20 \times 4 = \mathbf{-10 \text{ kN}\cdot\text{m}}$$

$$M_D = 0 \text{ (free end)}$$

S.F. changes sign at B, so B.M. has a maximum (peak) value at B: $M_B = \mathbf{23.333 \text{ kN}\cdot\text{m}}$

S.F. changes sign at C, so B.M. has a maximum (peak) value at C: $M_C = \mathbf{-10 \text{ kN}\cdot\text{m}}$

Point of contraflexure (B.M. changes sign between B and C): taking a section at x m from A, $M_x = -8.333x + 40 = 0 \Rightarrow \mathbf{x = 4.8 \text{ m from A (2.8 m from B)}}$.



S.F. and B.M. diagrams

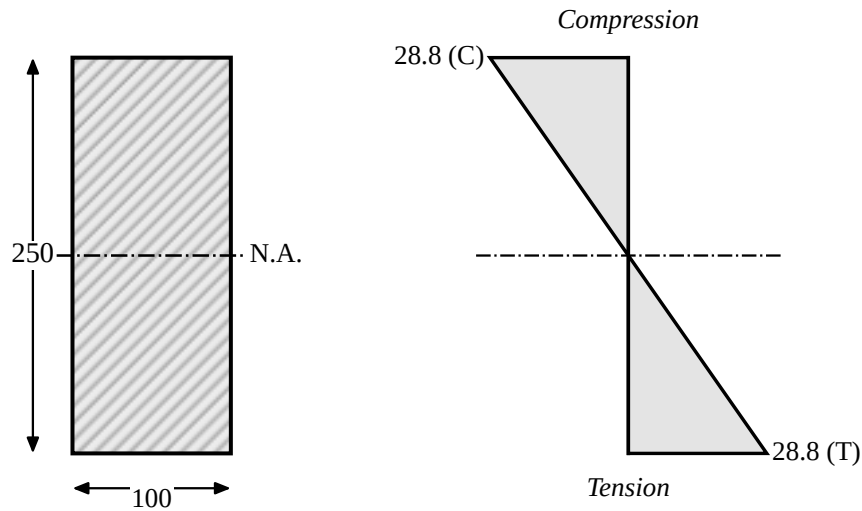
b) Bending and shear stresses in rectangular SS beam.

6 M

$$M_{\max} = WL/4 = 30 \times 4/4 = 30 \text{ kN}\cdot\text{m}; F_{\max} = W/2 = 15 \text{ kN}$$

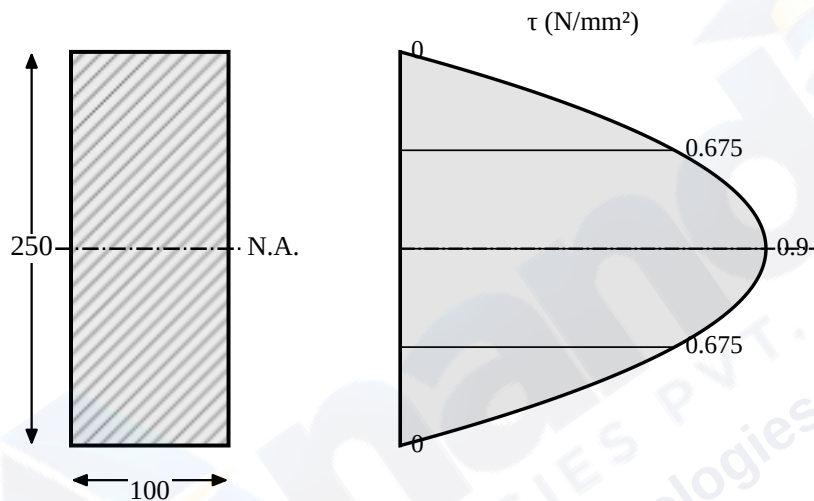
$$Z = bd^2/6 = 100 \times 250^2/6 = 1.0417 \times 10^6 \text{ mm}^3; \sigma_{\max} = M/Z = 3 \times 10^7/1.0417 \times 10^6 = 28.8 \text{ N/mm}^2$$

(C at top, T at bottom)



Bending stress distribution (stresses in N/mm²)

$$\tau_{avg} = F/(bd) = 15000/(100 \times 250) = 0.6 \text{ N/mm}^2; \tau_{max} = 1.5 \tau_{avg} = \mathbf{0.9 \text{ N/mm}^2} \text{ (at N.A.)}$$



Shear stress distribution (N/mm²)

(M and F 1 M, σ_{max} 1 M, bending diagram 1 M, τ_{max} 2 M, shear diagram 1 M)

c) Hollow rectangular column — eccentric load.

6 M

The load is on the axis bisecting the 300 mm side, so $e = 60$ mm acts along the 400 mm dimension (B = 400, D = 300; inner 300 × 200).

Outer 400 × 300 mm, inner 300 × 200 mm.

$$A = 400 \times 300 - 300 \times 200 = 60000 \text{ mm}^2$$

$$I = (300 \times 400^3 - 200 \times 300^3)/12 = 1.15 \times 10^9 \text{ mm}^4$$

$$Z = I / y_{max} = 1.15 \times 10^9 / 200 = 5.75 \times 10^6 \text{ mm}^3$$

$$\text{Direct stress } \sigma_0 = P/A = 4 \times 10^5 / 60000 = \mathbf{6.667 \text{ N/mm}^2 \text{ (C)}}$$

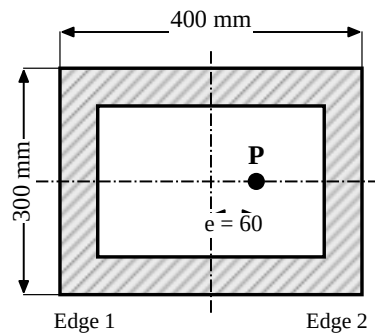
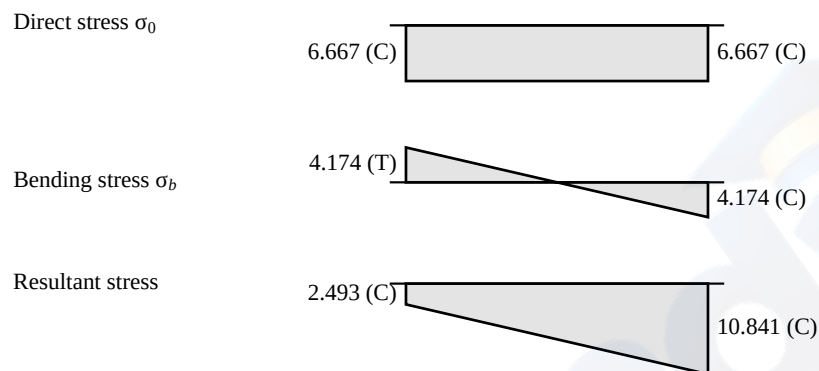
$$\text{Bending moment } M = P \cdot e = 4 \times 10^5 \times 60 = 2.4 \times 10^7 \text{ N}\cdot\text{mm}$$

$$\text{Bending stress } \sigma_b = M/Z = 2.4 \times 10^7 / 5.75 \times 10^6 = \mathbf{4.174 \text{ N/mm}^2}$$

$$\sigma_{max} = \sigma_0 + \sigma_b = 6.667 + 4.174 = \mathbf{10.841 \text{ N/mm}^2 \text{ (compressive)}}$$

$$\sigma_{min} = \sigma_0 - \sigma_b = 6.667 - 4.174 = \mathbf{2.493 \text{ N/mm}^2 \text{ (compressive)}}$$

Limit of eccentricity for no tension: $e_{\max} = Z/A = 5.75 \times 10^6 / 60000 = 95.83 \text{ mm}$. Since $e = 60 \text{ mm} < 95.83 \text{ mm}$, no tension develops.



(stresses in N/mm^2 ; C = compression, drawn below the line; T = tension, drawn above)

Fig.: Section with load position and stress distribution

(A and Z 1 M, σ_0 1 M, σ_b 1 M, $\sigma_{\max}/\sigma_{\min}$ 1 M, limit 1 M, diagram 1 M)

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