

Strength of Materials (313308)

Model Question Paper — Set B · 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) Section modulus and neutral axis.

2 M

Section modulus (Z): The ratio of the moment of inertia of a section about its neutral axis to the distance of the extreme fibre from the neutral axis. $Z = I / y_{\max}$ (unit mm³). For a rectangle $Z = bd^2/6$. It indicates the strength of a section in bending ($M = \sigma \cdot Z$).

Neutral axis (N.A.): The line of intersection of the neutral layer with the cross-section of a beam. Bending stress is zero at the neutral axis; it passes through the centroid of the section.

(1 M each)

b) Perpendicular axis theorem.

2 M

Perpendicular axis theorem: The moment of inertia of a plane lamina about an axis perpendicular to its plane (Z–Z) and passing through a point O is equal to the sum of its moments of inertia about two mutually perpendicular axes X–X and Y–Y lying in the plane of the lamina and intersecting at O.

$$I_{ZZ} = I_{XX} + I_{YY}$$

I_{ZZ} is also called the polar moment of inertia (J).

(Statement 1 M, expression 1 M)

c) Linear strain and lateral strain.

2 M

Linear (longitudinal) strain: The ratio of change in length to the original length in the direction of the applied load. $e = \delta L / L$.

Lateral strain: The ratio of change in lateral dimension (diameter, width or thickness) to the original lateral dimension, at right angles to the load. $e_{\text{lat}} = \delta d / d$ or $\delta b / b$. It is opposite in sign to the linear strain.

(1 M each)

d) Shear force and bending moment.

2 M

Shear force (S.F.): The algebraic sum of all the vertical forces (loads and reactions) acting on either side of the section of a beam.

Bending moment (B.M.): The algebraic sum of the moments of all forces acting on either side of the section of a beam, about that section.

(1 M each)

e) Middle third rule.

2 M

Middle third rule: For a rectangular section, no tensile stress is developed anywhere in the section if the load acts within the middle third of the base, i.e. the eccentricity does not exceed $b/6$ on either side of the centroidal axis ($e \leq b/6$).

(2 M)

f) Factor of safety and working stress.

2 M

Working (permissible) stress: The stress to which a member is actually allowed to be stressed in practice; it is kept well below the ultimate (or yield) stress.

Factor of safety (F.O.S.): The ratio of ultimate stress (or yield stress) to the working stress. F.O.S. = ultimate stress / working stress.

(1 M each)

g) MI expressions for rectangle and circle.

2 M

Section	M.I.
Rectangle $b \times d$, about centroidal axis X-X	$I_{XX} = bd^3/12$
Rectangle, about base	$I_{base} = bd^3/3$
Circle of diameter d , about a diameter	$I = \pi d^4/64$

(Any two correct, 1 M each)

Q.2

a) Polar MI of rectangle 80×120 ; radii of gyration.

4 M

Polar moment of inertia (J): The moment of inertia of a plane area about an axis perpendicular to its plane (Z-Z axis). By perpendicular axis theorem $J = I_{XX} + I_{YY}$.

$$I_{XX} = bd^3/12 = 80 \times 120^3/12 = 1.152 \times 10^7 \text{ mm}^4; I_{YY} = db^3/12 = 120 \times 80^3/12 = 5.12 \times 10^6 \text{ mm}^4$$

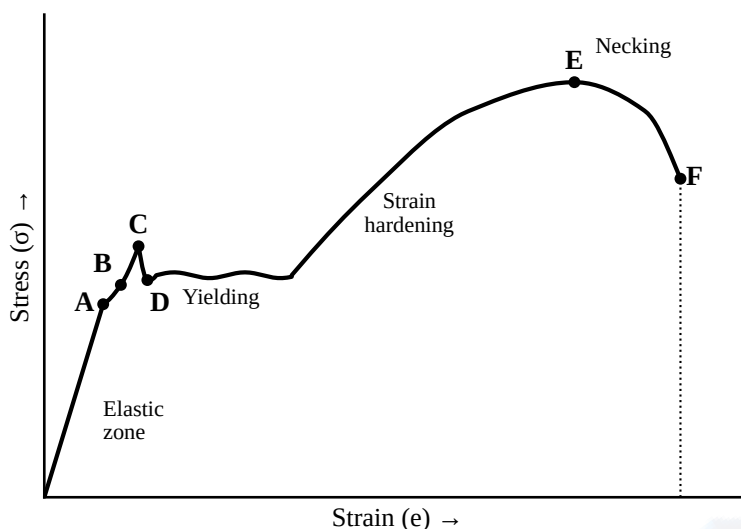
$$J = I_{XX} + I_{YY} = 1.152 \times 10^7 + 5.12 \times 10^6 = 1.664 \times 10^7 \text{ mm}^4$$

$$A = 80 \times 120 = 9600 \text{ mm}^2; k_{xx} = \sqrt{I_{XX}/A} = \sqrt{(1.152 \times 10^7/9600)} = 34.641 \text{ mm}; k_{yy} = \sqrt{(5.12 \times 10^6/9600)} = 23.094 \text{ mm}$$

(Definition 1 M, J 2 M, k 1 M)

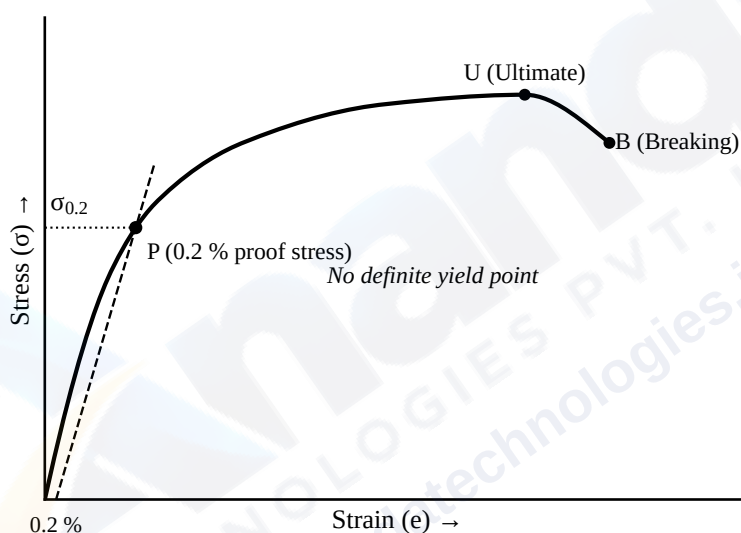
b) Stress–strain curves for mild steel and Tor steel; proof stress.

4 M



A – Limit of proportionality, B – Elastic limit, C – Upper yield point, D – Lower yield point, E – Ultimate stress point, F – Breaking point

Stress–strain curve for mild steel (ductile material) in tension



Stress–strain curve for Tor steel (HYSD bars) — no definite yield point

Tor steel (high yield strength deformed bars) has **no clear yield point**. Its yield strength is taken as the **0.2 % proof stress**: a line drawn parallel to the initial straight portion from 0.2 % strain (0.002) meets the curve at P; the stress at P is the proof stress.

Mild steel shows A – limit of proportionality, B – elastic limit, C/D – upper/lower yield points, E – ultimate stress, F – breaking point.

Proof stress: For materials having no definite yield point (Tor steel, aluminium), the stress at which a specified permanent strain (usually 0.2 %) remains after unloading is called the proof stress; it is used in place of yield stress.

(Mild steel curve 1½ M, Tor steel curve 1½ M, proof stress 1 M)

c) Stress, strain and E of bar.

4 M

$$A = \pi \times 20^2/4 = 314.16 \text{ mm}^2$$

$$\text{Stress } \sigma = P/A = 40 \times 10^3 / 314.16 = \mathbf{127.324 \text{ N/mm}^2}$$

$$\text{Strain } e = \delta L/L = 0.25/400 = \mathbf{6.25 \times 10^{-4}}$$

$$E = \sigma/e = 127.324 / 0.000625 = \mathbf{2.0372 \times 10^5 \text{ N/mm}^2} = 203.72 \text{ GPa}$$

(σ 1 M, e 1 M, E 2 M)

d) SS beam with two point loads — SFD and BMD.

4 M

Support reactions (R_A and R_D act upward):

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

$$R_D \times 6 = 10 \times 2 + 15 \times 4.5 = 87.5$$

$$\therefore R_D = 87.5 / 6 = \mathbf{14.583 \text{ kN}}$$

$$\Sigma F_y = 0: R_A + R_D = 10 + 15 = 25 \text{ kN}$$

$$\therefore R_A = 25 - 14.583 = \mathbf{10.417 \text{ kN}}$$

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

$$\text{S.F. just right of A} = 10.417 = \mathbf{10.417 \text{ kN}}$$

$$\text{S.F. just left of B} = 10.417 = \mathbf{10.417 \text{ kN}}$$

$$\text{S.F. just right of B} = 10.417 - 10 = \mathbf{0.417 \text{ kN}}$$

$$\text{S.F. just left of C} = 10.417 - 10 = \mathbf{0.417 \text{ kN}}$$

$$\text{S.F. just right of C} = 10.417 - 10 - 15 = \mathbf{-14.583 \text{ kN}}$$

$$\text{S.F. just left of D} = 10.417 - 10 - 15 = \mathbf{-14.583 \text{ kN}}$$

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

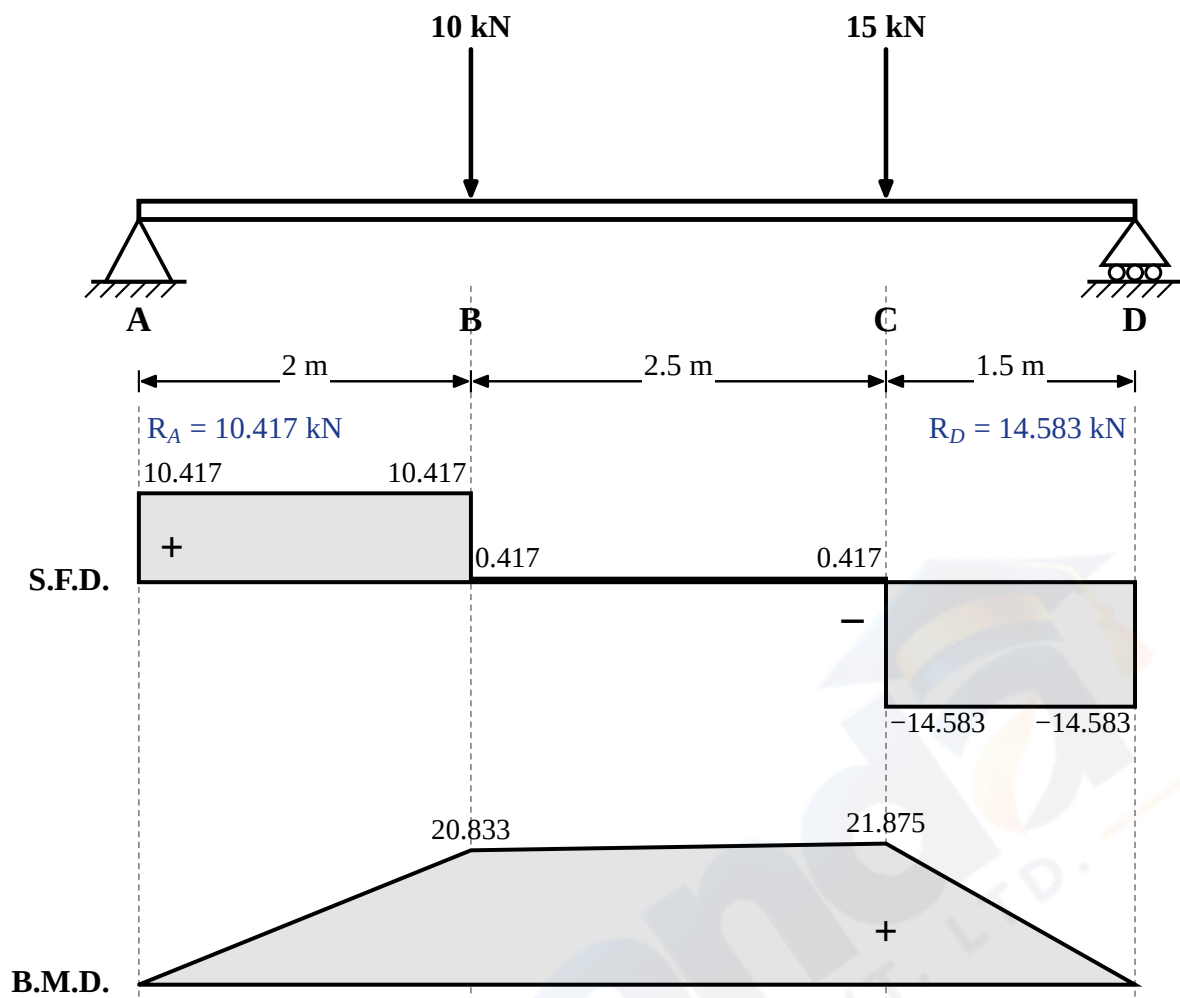
$$M_A = 0 \text{ (simple support)}$$

$$M_B = 10.417 \times 2 = \mathbf{20.833 \text{ kN}\cdot\text{m}}$$

$$M_C = 10.417 \times 4.5 - 10 \times 2.5 = \mathbf{21.875 \text{ kN}\cdot\text{m}}$$

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

S.F. changes sign at C, so B.M. has a maximum (peak) value at C: $M_C = \mathbf{21.875 \text{ kN}\cdot\text{m}}$

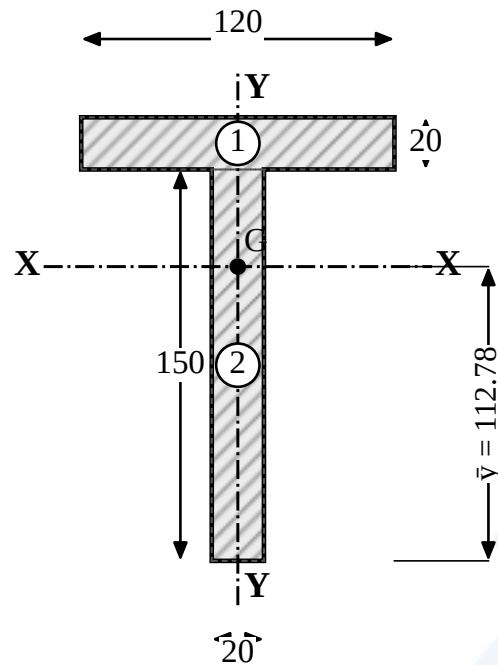


S.F. and B.M. diagrams

Q.3

a) MI of T-section.

4 M



T-section (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)
Flange 120×20	2400	160	3.84×10^5	80000	47.22	5.3519×10^6
Web 20×150	3000	75	2.25×10^5	5.625×10^6	-37.78	4.2815×10^6
Σ	5400		6.09×10^5	5.705×10^6		9.6333×10^6

$$\bar{y} = \Sigma a \cdot y / \Sigma a = 6.09 \times 10^5 / 5400 = \mathbf{112.78 \text{ mm}}$$
 from the bottom of web

$$I_{XX} = \Sigma I_G + \Sigma a \cdot h^2 = \mathbf{1.5338 \times 10^7 \text{ mm}^4}$$

$$I_{YY} = 20 \times 120^3 / 12 + 150 \times 20^3 / 12 = \mathbf{2.98 \times 10^6 \text{ mm}^4}$$
 (Y-Y is the axis of symmetry)

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

b) Cube under all-round pressure — K and change in volume.

4 M

Stress on each face $\sigma = 1500 \times 10^3 / (100 \times 100) = \mathbf{150 \text{ N/mm}^2}$ (compressive, equal in all three directions)

$$\text{Bulk modulus } K = E / [3(1 - 2\mu)] = 2 \times 10^5 / [3(1 - 0.6)] = \mathbf{1.6667 \times 10^5 \text{ N/mm}^2}$$

$$\text{Volumetric strain } e_v = \sigma / K = 150 / 1.6667 \times 10^5 = 9 \times 10^{-4} \text{ (decrease)}$$

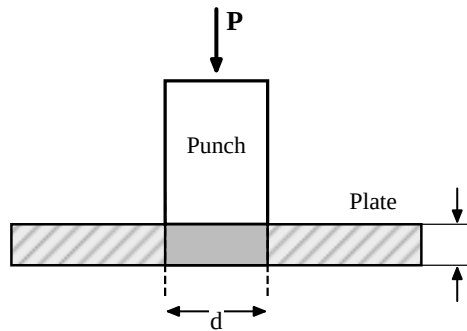
$$\text{(or } e_v = 3\sigma(1 - 2\mu) / E = 3 \times 150 \times 0.4 / (2 \times 10^5) = 9 \times 10^{-4} \checkmark)$$

$$\text{Change in volume } \delta V = e_v \times V = 9 \times 10^{-4} \times 100^3 = \mathbf{900 \text{ mm}^3} \text{ (decrease)}$$

(σ 1 M, K 1 M, δV 2 M)

c) Punching force and compressive stress in punch.

4 M



Punching shear: area resisting shear = $\pi d t$

Area resisting shear = $\pi d t = \pi \times 25 \times 6 = 471.24 \text{ mm}^2$

Force required $P = \tau_u \times \pi d t = 300 \times 471.24 = 141371.7 \text{ N} = \mathbf{141.372 \text{ kN}}$

Compressive stress in punch = $P / (\pi d^2/4) = 141371.7 / 490.87 = \mathbf{288 \text{ N/mm}^2}$

(Shear area 1 M, P 2 M, σ_c 1 M)

d) Cantilever 3 m — SFD and BMD.

4 M

Reaction at fixed end A: $R_A = \text{total load} = 6 + 4 \times 2 = \mathbf{14 \text{ kN}} (\uparrow)$

Fixing moment at A = $34 \text{ kN}\cdot\text{m}$ (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):

S.F. just right of A = $4 \times 2 + 6 = \mathbf{14 \text{ kN}}$

S.F. at B = $4 \times 2 + 6 = \mathbf{14 \text{ kN}}$

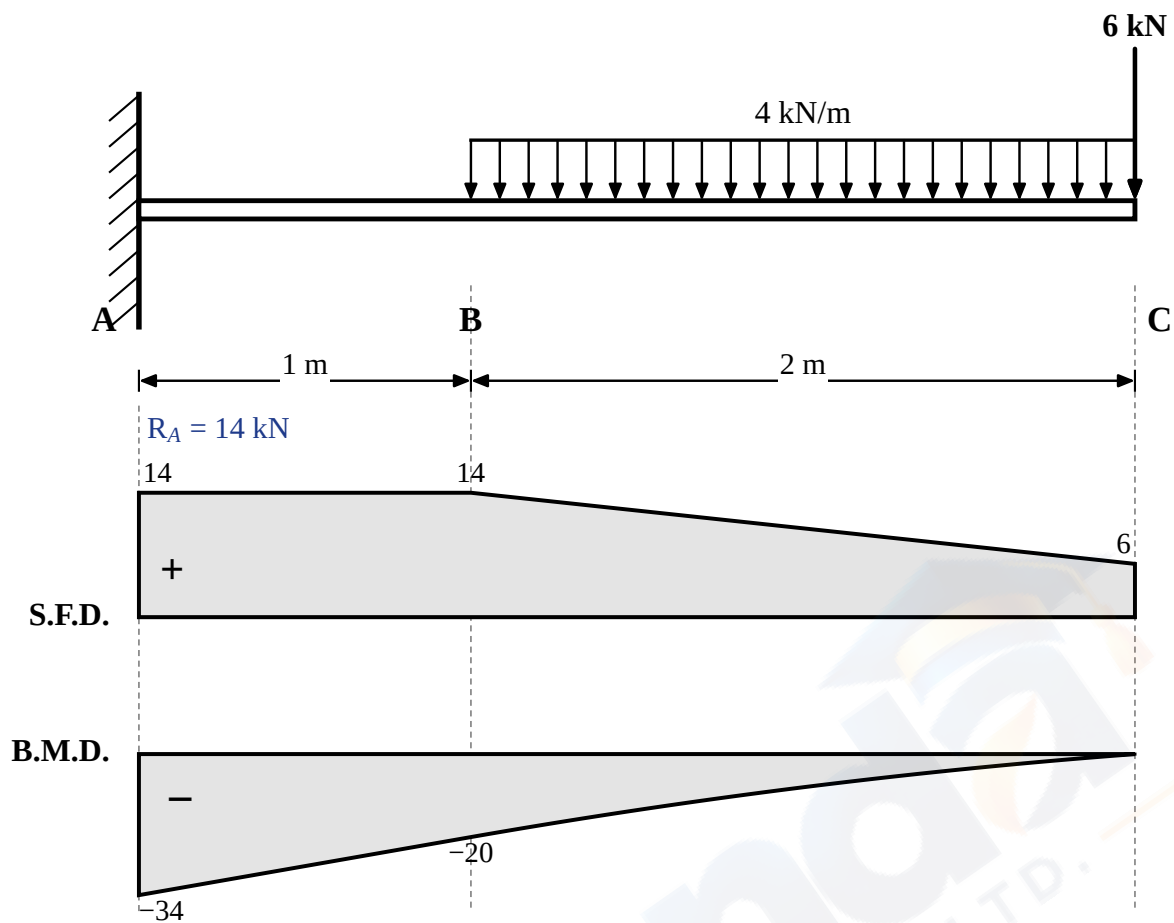
S.F. just left of C = $6 = \mathbf{6 \text{ kN}}$

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

$M_A = -4 \times 2 \times 2 - 6 \times 3 = \mathbf{-34 \text{ kN}\cdot\text{m}}$

$M_B = -4 \times 2 \times 1 - 6 \times 2 = \mathbf{-20 \text{ kN}\cdot\text{m}}$

$M_C = 0$ (free end)



S.F. and B.M. diagrams

Q.4

a) Diameter of circular cantilever.

4 M

Maximum B.M. at the fixed end $M = W \cdot L = 20 \times 2 = 40 \text{ kN}\cdot\text{m} = 4 \times 10^7 \text{ N}\cdot\text{mm}$

For a circular section $Z = \pi d^3/32$. Flexure formula $M = \sigma \cdot Z$

$$4 \times 10^7 = 120 \times \pi d^3/32 \Rightarrow d^3 = 32 \times 4 \times 10^7 / (\pi \times 120) = 3.3953 \times 10^6 \text{ mm}^3$$

d = 150.3 mm, say **d = 152 mm** (next convenient size).

(M 1 M, Z 1 M, d 2 M)

b) Shear stress distribution in circular section.

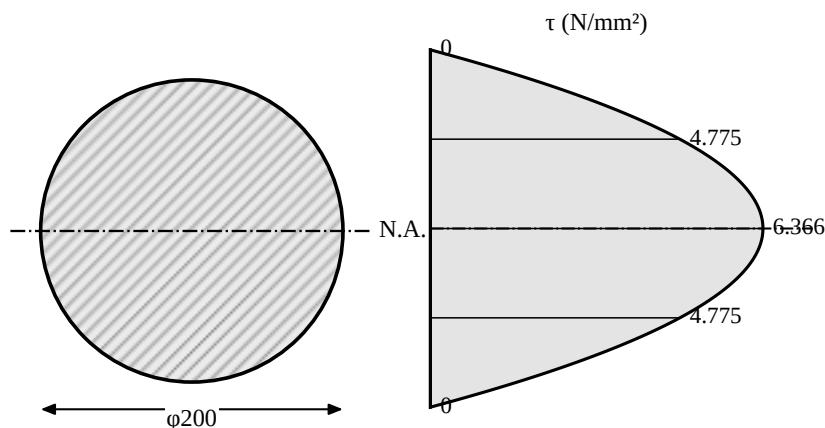
4 M

$$A = \pi \times 200^2/4 = 31415.93 \text{ mm}^2; \tau_{\text{avg}} = F/A = 150 \times 10^3 / 31415.93 = 4.775 \text{ N/mm}^2$$

For a circular section $\tau_{\text{max}} = (4/3) \tau_{\text{avg}} = (4/3) \times 4.775 = 6.366 \text{ N/mm}^2$ (at N.A.)

At 50 mm from N.A. ($\tau = F \cdot A \bar{y} / (I \cdot b)$, $b = 2\sqrt{(100^2 - 50^2)} = 173.21 \text{ mm}$): $\tau = 4.775 \text{ N/mm}^2$

$$\text{Ratio } \tau_{\text{max}}/\tau_{\text{avg}} = 4/3 = 1.333$$



Shear stress distribution for circular section (N/mm²)

(τ_{avg} 1 M, τ_{max} 1 M, diagram 1 M, ratio 1 M)

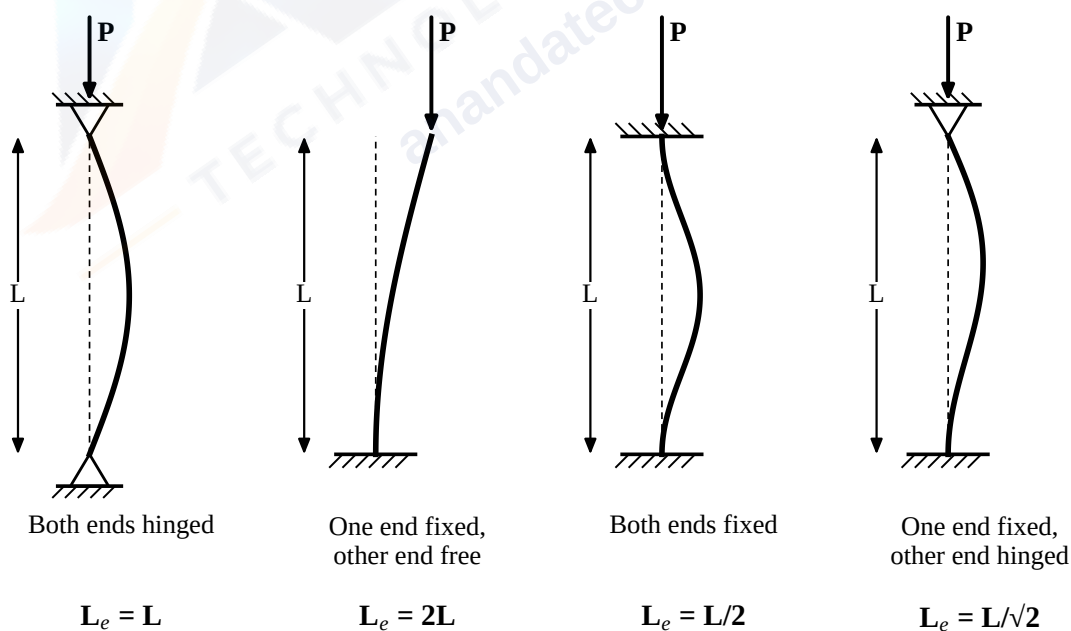
c) Effective length, slenderness ratio; end conditions.

4 M

Effective length (L_e): The length of an equivalent column of the same material and section with both ends hinged, which has the same crippling load as the given column. It is the distance between adjacent points of zero B.M. (inflexion) of the buckled column.

Slenderness ratio: The ratio of the effective length of a column to its least radius of gyration, $\lambda = L_e/k_{min}$.

End condition	L_e
Both ends hinged	L
One end fixed, other free	$2L$
Both ends fixed	$L/2$
One end fixed, other hinged	$L/\sqrt{2}$



Effective length of columns for different end conditions

(Definitions 2 M, sketches with L_e 2 M)

d) Changes in length, width and thickness of flat.

4 M

$$A = 100 \times 10 = 1000 \text{ mm}^2; \sigma = 120 \times 10^3 / 1000 = \mathbf{120 \text{ N/mm}^2}$$

$$\text{Linear strain } e = \sigma/E = 120 / (2 \times 10^5) = 6 \times 10^{-4}; \delta L = e \times L = 6 \times 10^{-4} \times 3000 = \mathbf{1.8 \text{ mm}}$$

(increase)

$$\text{Lateral strain} = \mu e = 0.3 \times 6 \times 10^{-4} = 1.8 \times 10^{-4}$$

$$\delta b = 1.8 \times 10^{-4} \times 100 = \mathbf{0.018 \text{ mm}}$$
 (decrease); $\delta t = 1.8 \times 10^{-4} \times 10 = \mathbf{0.0018 \text{ mm}}$ (decrease)

(σ and e 1 M, δL 1 M, δb 1 M, δt 1 M)

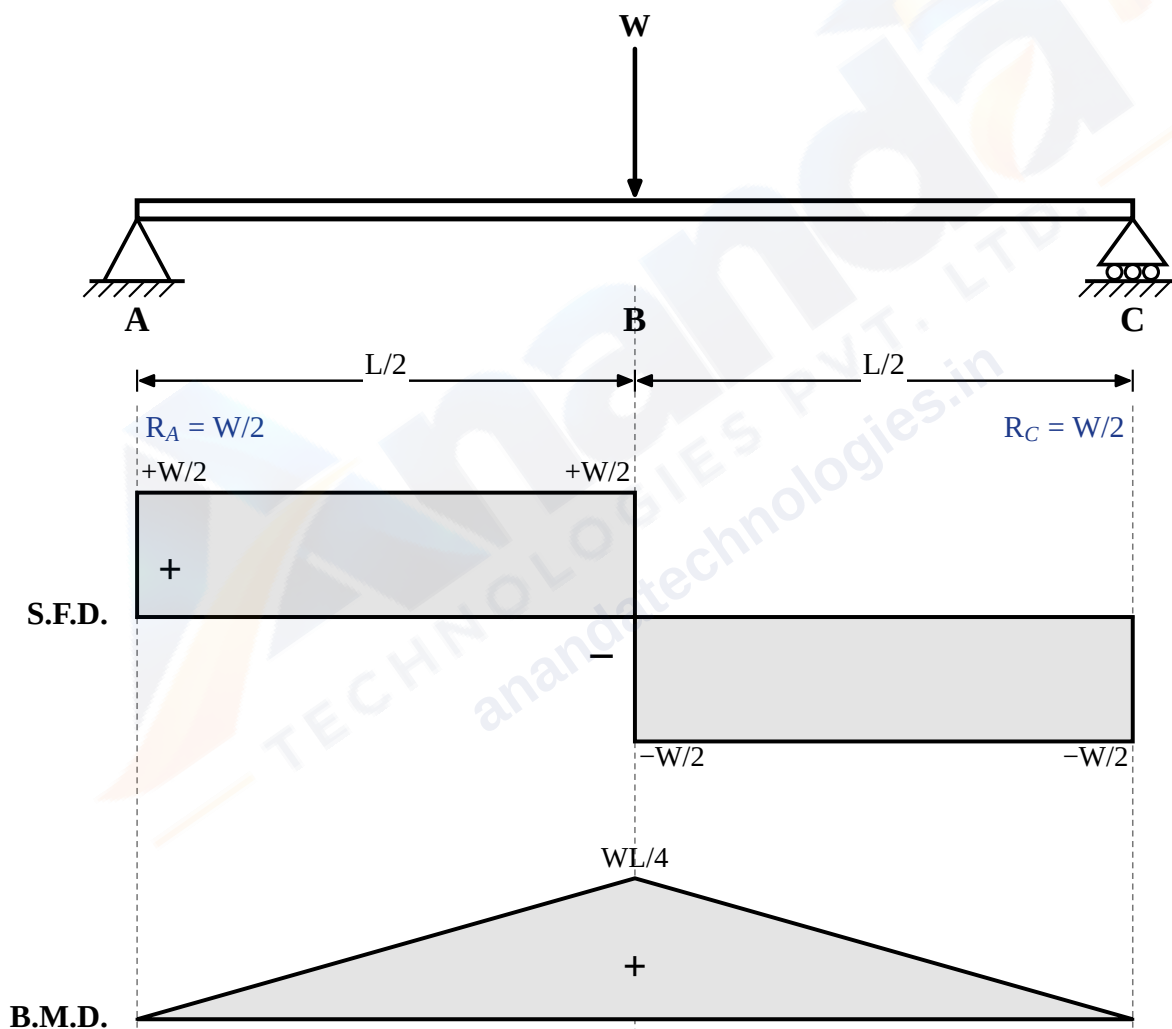
e) SS beam with central point load — standard case.

4 M

Let A and C be the supports and B the mid-span point where W acts. Reactions: $R_A = R_C = W/2$ (symmetry).

S.F. from A to B = $+W/2$; from B to C = $W/2 - W = -W/2$ (S.F. changes sign at B).

B.M. at A and C = 0; B.M. at B = $(W/2)(L/2) = \mathbf{WL/4}$. B.M. varies linearly (triangular BMD).



SS beam with central point load W (span L)

Maximum S.F. = $W/2$ (at supports) and **maximum B.M. = $WL/4$** (at mid-span).

(Reactions 1 M, SFD 1 M, BMD 1 M, values 1 M)

Q.5

a) Beam with overhangs on both sides — SFD, BMD and points of contraflexure.

6 M

Support reactions (R_B and R_C act upward):

Taking moments about B, $\Sigma M_B = 0$:

$$R_C \times 6 = -10 \times 2 + (6 \times 6) \times 3 + 12 \times 8 = 184$$

$$\therefore R_C = 184 / 6 = \mathbf{30.667 \text{ kN}}$$

$$\Sigma F_y = 0: R_B + R_C = 10 + 12 + 6 \times 6 = 58 \text{ kN}$$

$$\therefore R_B = 58 - 30.667 = \mathbf{27.333 \text{ kN}}$$

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

$$\text{S.F. just right of A} = -10 = \mathbf{-10 \text{ kN}}$$

$$\text{S.F. just left of B} = -10 = \mathbf{-10 \text{ kN}}$$

$$\text{S.F. just right of B} = 27.333 - 10 = \mathbf{17.333 \text{ kN}}$$

$$\text{S.F. just left of C} = 27.333 - 10 - 6 \times 6 = \mathbf{-18.667 \text{ kN}}$$

$$\text{S.F. just right of C} = 27.333 + 30.667 - 10 - 6 \times 6 = \mathbf{12 \text{ kN}}$$

$$\text{S.F. just left of D} = 27.333 + 30.667 - 10 - 6 \times 6 = \mathbf{12 \text{ kN}}$$

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

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

$$M_B = -10 \times 2 = \mathbf{-20 \text{ kN}\cdot\text{m}}$$

$$M_C = 27.333 \times 6 - 10 \times 8 - 6 \times 6 \times 3 = \mathbf{-24 \text{ kN}\cdot\text{m}}$$

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

Point of zero shear (contra-shear): let it be at x m from A. Between B and the next point S.F. = $17.333 - 6(x - 2) = 0 \Rightarrow x = \mathbf{4.889 \text{ m}}$ from A.

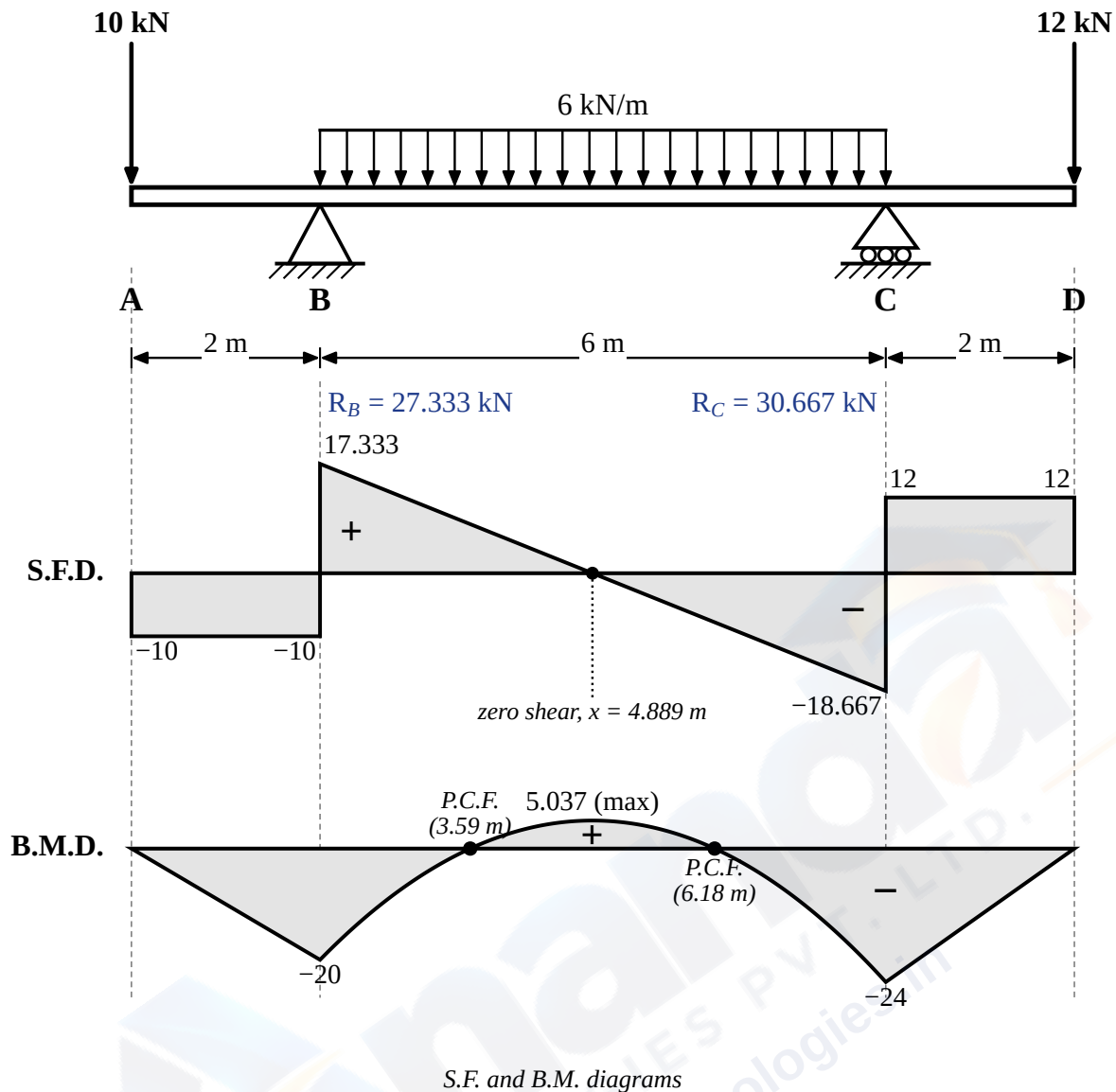
Maximum B.M. at this section: $M_{\max} = 27.333 \times 2.889 - 10 \times 4.889 - 6 \times 2.889 \times 1.444 = \mathbf{5.037 \text{ kN}\cdot\text{m}}$

S.F. changes sign at B, so B.M. has a maximum (peak) value at B: $M_B = \mathbf{-20 \text{ kN}\cdot\text{m}}$

S.F. changes sign at C, so B.M. has a maximum (peak) value at C: $M_C = \mathbf{-24 \text{ kN}\cdot\text{m}}$

Point of contraflexure (B.M. changes sign between B and C): taking a section at x m from A, $M_x = -3x^2 + 29.333x - 66.667 = 0 \Rightarrow x = \mathbf{3.593 \text{ m from A}}$ (1.593 m from B).

Point of contraflexure (B.M. changes sign between B and C): taking a section at x m from A, $M_x = -3x^2 + 29.333x - 66.667 = 0 \Rightarrow x = \mathbf{6.185 \text{ m from A}}$ (4.185 m from B).



b) Bending stress in symmetrical I-section beam.

6 M

Maximum B.M. $M = wL^2/8 = 20 \times 5^2/8 = 62.5 \text{ kN}\cdot\text{m} = 6.25 \times 10^7 \text{ N}\cdot\text{mm}$

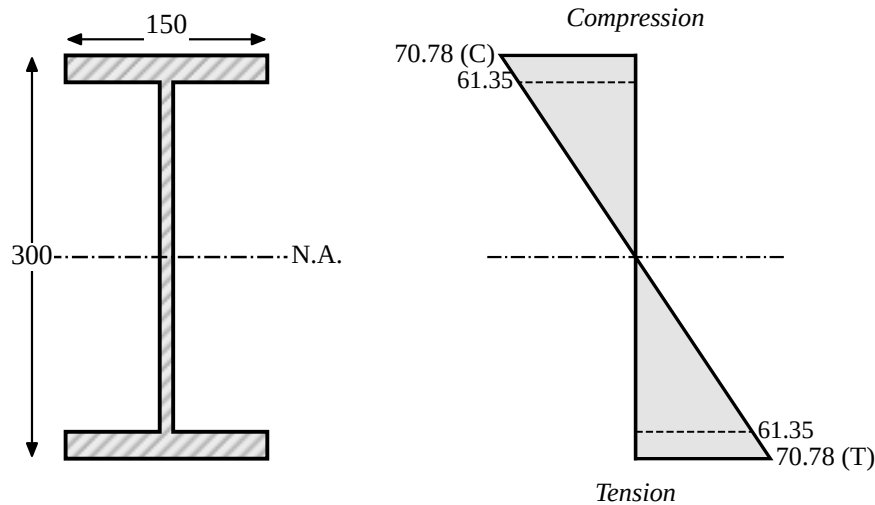
$I_{XX} = (BD^3 - bd^3)/12 = (150 \times 300^3 - 140 \times 260^3)/12 = 1.3245 \times 10^8 \text{ mm}^4$ ($b = 150 - 10 = 140 \text{ mm}$, $d = 260 \text{ mm}$)

(check by parts: $1.3245 \times 10^8 \text{ mm}^4 \checkmark$)

$y_{\max} = 150 \text{ mm}$; $\sigma_{\max} = M \cdot y_{\max} / I = 6.25 \times 10^7 \times 150 / 1.3245 \times 10^8 = 70.783 \text{ N/mm}^2$

(compression at top, tension at bottom)

At junction of flange and web $y = 150 - 20 = 130 \text{ mm}$: $\sigma = 6.25 \times 10^7 \times 130 / 1.3245 \times 10^8 = 61.345 \text{ N/mm}^2$



Bending stress distribution (stresses in N/mm²)

(M 1 M, I 2 M, σ_{max} 1 M, $\sigma_{junction}$ 1 M, diagram 1 M)

c) Stepped bar — P for equilibrium, elongation, minimum stress.

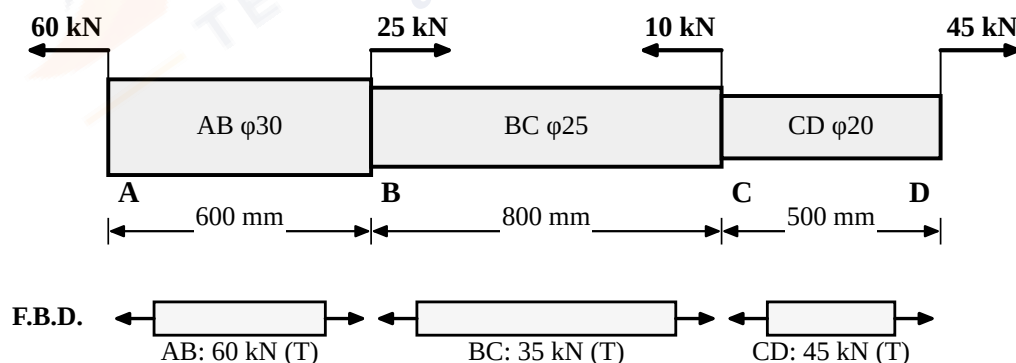
6 M

Equilibrium: $\Sigma F_x = 0$. Taking forces to the right as positive: $-60 + 25 + 45 + P = 0 \Rightarrow P = 10 \text{ kN}$ acting to the left ($\leftarrow$).

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	60 kN (T)	600	$\pi \times 30^2 / 4 = 706.86$	84.883	$+60000 \times 600 / (706.86 \times 2 \times 10^5) = 0.25465$
BC	35 kN (T)	800	$\pi \times 25^2 / 4 = 490.87$	71.301	$+35000 \times 800 / (490.87 \times 2 \times 10^5) = 0.28521$
CD	45 kN (T)	500	$\pi \times 20^2 / 4 = 314.16$	143.239	$+45000 \times 500 / (314.16 \times 2 \times 10^5) = 0.3581$

Total change in length $\delta L = \Sigma \delta L = 0.89795 \text{ mm}$ (increase)



Free body diagrams of the parts

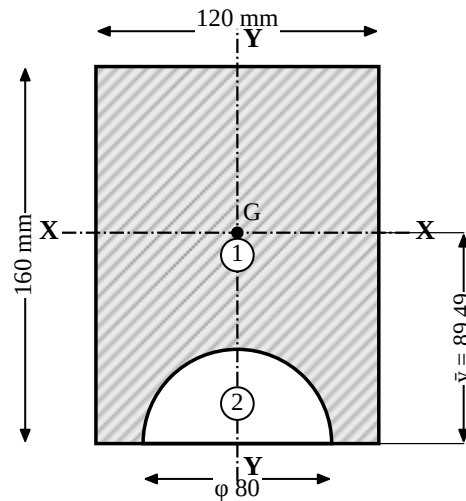
Minimum stress = 71.301 N/mm² in part BC; maximum = 143.239 N/mm².

(P 1 M, internal forces 2 M, δL 2 M, min stress 1 M)

Q.6

a) MI of rectangle with semicircular cut-out.

6 M



Section (rectangle minus semicircle)

(i) About the base (the diameter of the semicircle lies on the base):

$$I_{\text{base}} (\text{rectangle}) = bd^3/3 = 120 \times 160^3/3 = 1.6384 \times 10^8 \text{ mm}^4$$

$$I_{\text{base}} (\text{semicircle about its diameter}) = \pi r^4/8 = \pi \times 40^4/8 = 1.0053 \times 10^6 \text{ mm}^4$$

$$I_{\text{base}} = 1.6384 \times 10^8 - 1.0053 \times 10^6 = \mathbf{1.6283 \times 10^8 \text{ mm}^4}$$

(ii) About the centroidal axis — centroid from the base:

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)
Rectangle 120 × 160	19200	80	1.536 × 10 ⁶	4.096 × 10 ⁷	-9.49	1.73 × 10 ⁶
Semicircle φ80 (removed)	-2513.27	16.98	-42667	-2.8098 × 10 ⁵	-72.52	-1.3216 × 10 ⁷
Σ	16686.73		1.4933 × 10⁶	4.0679 × 10⁷		-1.1486 × 10⁷

(semicircle: area $\pi r^2/2$, centroid $4r/3\pi = 16.977$ mm above the base, own $I_G = 0.1098 r^4$)

$$\bar{y} = \Sigma a \cdot y / \Sigma a = \mathbf{89.492 \text{ mm}}$$
 from the base

$$I_{XX} = I_{\text{base}} - A \cdot \bar{y}^2 = 1.6283 \times 10^8 - 16686.73 \times 89.492^2 = \mathbf{2.9193 \times 10^7 \text{ mm}^4}$$
 (table gives $2.9193 \times 10^7 \text{ mm}^4$ ✓)

(I base 2 M, centroid 2 M, Ixx 2 M)

b) Shear stress distribution in hollow rectangular section.

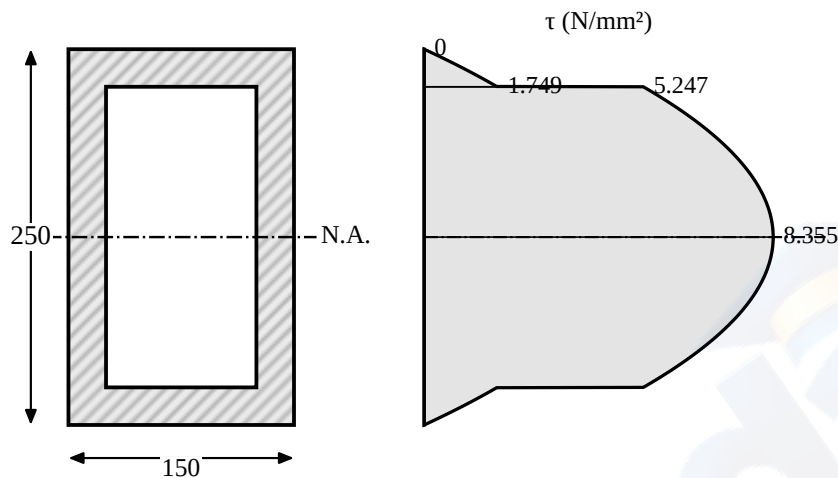
6 M

$$I = (150 \times 250^3 - 100 \times 200^3)/12 = \mathbf{1.2865 \times 10^8 \text{ mm}^4}; \tau = F \cdot A \bar{y} / (I \cdot b), F = 80 \times 10^3 \text{ N}$$

- At top fibre: $A \bar{y} = 0 \Rightarrow \tau = 0$

- At junction (bottom of top flange), $A\bar{y} = 150 \times 25 \times 112.5 = 4.2188 \times 10^5 \text{ mm}^3$
in flange ($b = 150 \text{ mm}$): $\tau = 80000 \times 4.2188 \times 10^5 / (1.2865 \times 10^8 \times 150) = \mathbf{1.749 \text{ N/mm}^2}$
in webs ($b = 2 \times 25 = 50 \text{ mm}$): $\tau = 80000 \times 4.2188 \times 10^5 / (1.2865 \times 10^8 \times 50) = \mathbf{5.247 \text{ N/mm}^2}$
- At N.A. ($b = 50 \text{ mm}$): $A\bar{y} = 4.2188 \times 10^5 + 2 \times (25 \times 100 \times 50) = 6.7188 \times 10^5 \text{ mm}^3$; $\tau_{\max} = 80000 \times 6.7188 \times 10^5 / (1.2865 \times 10^8 \times 50) = \mathbf{8.355 \text{ N/mm}^2}$

$$\tau_{\text{avg}} = F/A = 80000 / 17500 = 4.571 \text{ N/mm}^2; \text{ratio } \tau_{\max}/\tau_{\text{avg}} = \mathbf{1.828}$$



Shear stress distribution (N/mm^2) — jump at the junction of flange and webs

(I 1 M, τ values 3 M, diagram 1 M, ratio 1 M)

c) Rectangular pier — eccentric load.

6 M

The load lies on the axis bisecting the 800 mm side, so the eccentricity $e = 250 \text{ mm}$ acts along the 1200 mm dimension ($b = 1200 \text{ mm}$, $d = 800 \text{ mm}$).

$$\text{Area } A = 1200 \times 800 = 960000 \text{ mm}^2$$

$$\text{Section modulus (about the axis } \perp \text{ to plane of eccentricity) } Z = d \cdot b^2 / 6 = 800 \times 1200^2 / 6 = 1.92 \times 10^8 \text{ mm}^3$$

$$\text{Direct stress } \sigma_0 = P/A = 6 \times 10^5 / 960000 = \mathbf{0.625 \text{ N/mm}^2 \text{ (C)}}$$

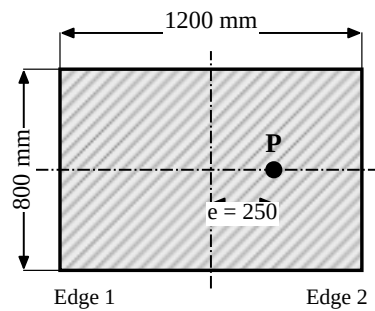
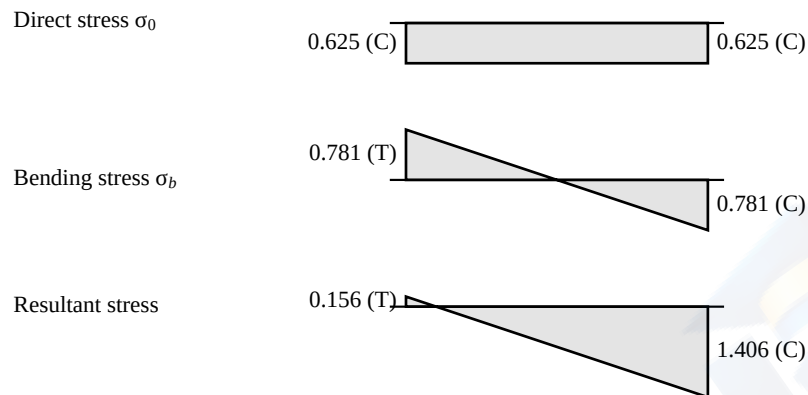
$$\text{Bending moment } M = P \cdot e = 6 \times 10^5 \times 250 = 1.5 \times 10^8 \text{ N} \cdot \text{mm}$$

$$\text{Bending stress } \sigma_b = M/Z = 1.5 \times 10^8 / 1.92 \times 10^8 = \mathbf{0.781 \text{ N/mm}^2}$$

$$\sigma_{\max} = \sigma_0 + \sigma_b = 0.625 + 0.781 = \mathbf{1.406 \text{ N/mm}^2 \text{ (compressive)}}$$

$$\sigma_{\min} = \sigma_0 - \sigma_b = 0.625 - 0.781 = \mathbf{-0.156 \text{ N/mm}^2 \text{ i.e. } 0.156 \text{ N/mm}^2 \text{ tensile}}$$

Limit of eccentricity for no tension: $e_{\max} = Z/A = 1.92 \times 10^8 / 960000 = \mathbf{200 \text{ mm}}$. Since $e = 250 \text{ mm} > 200 \text{ mm}$, 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 rectangle $e_{\max} = b/6 = 1200/6 = \mathbf{200 \text{ mm}}$ (middle third rule).

(σ_0 1 M, σ_b 1 M, $\sigma_{\max}/\sigma_{\min}$ 2 M, diagram 1 M, $e_{\max}$ 1 M)

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