DETAILS EXPLANATIONS

CE : Paper-1 (Paper-4) [Full Syllabus]

[PART : A]

1. $E = 2G(1 + \mu)$

Where, E = Modulus of elasticity

G = Shear modulus

2. $f = \frac{M}{Z} = \frac{100 \times 10^6 \times 6}{200 \times 300^2} = 33.33 \text{ N/mm}^2$

3. It is point on beam at which the bending moment diagram changes it sign.

4. Shear force at any section is alzebric sum of all the lateral force calculated either from left or from right upto that section.

5. Weight of soil = 100 gm

Weight of water = 40 gm

So, wight of soil – solids = 100 – 40 = 60 gm

$\therefore$ Water content $= \frac{w_w}{w_s} = \frac{40}{60} \times 100\% = 66.67\%$

6. It the is property of soil, due to which it allows water to flow through it's interconnecting voids.

7. Degree of consolidation

$$U = \frac{\Delta h}{\delta H} \times 100\% = \frac{30}{60} \times 100 = 50\%$$

8. Shear strength of soil

$\sigma_s = \sigma_n \tan\phi + C$

9. It is the unknown number of external support reactions that can not be calculated using equilibrium equation only.

10. A load factor or load for which a distribution of bend movement can be found which satisfy the equilibrum condition is less than or at most equal to the true value of callapse load/factors.

11. Acc to this, the external work done by force is equal to the intternal work done by plastic moment.

12. There are two methods of prestressing :

- Pretensioning
- Post Tensioning

13. For bending moment → Face of column

For One way shear → at 'd' distance from face of column

For two way shear → at $\frac{'d'}{2}$ distance from face of column.

14. Effective width of fiange

$$B_F = \frac{l_0}{(l_0 / B) + 4} + D_F$$

15. For effective depth 200 mm or less Tolerance ±10 mm.

For effective depth more than 200 mm, Tolerance ±15 mm.

16. The critical section for maximum shear in walls is taken at a distance from the base of 0.5 L_w or 0.5 H_w, whichever is less.

17. Design shear strength

$$\Rightarrow \quad V_{dsb} = \frac{f_u(A_{n_b})}{\sqrt{3}\ r_{mb}} = \frac{400}{\sqrt{3} \times 1.25}\left(0.75 \times \frac{\pi}{4} \times 18^2\right) \times 10^{-3}$$

$$= 36.67 \text{ kN}$$

18. Simple connection between members at their junction will net resist any appreciable moment and shall be assumed to be hinged.

19. Plastic section modulus :

$$Z_p = \frac{BD^2}{4} = \frac{200 \times 350^2}{4}$$

$$= 6125000 \text{ mm}^4 = 612.5 \text{ cm}^4$$

20. Block shear strength :
Smaller of

- $\frac{A_{v_g} f_y}{\sqrt{3}\ r_{m_0}} + \frac{0.9 A_{t_n} f_u}{r_{m_1}}$
- $\frac{0.9 A_{v_n} f_u}{\sqrt{3}\ r_{m_1}} + \frac{A_{v_g} f_y}{r_{m_0}}$

[PART : B]

21. Plastic moment capacity

$$M_p = f_g.Z_p = f_y.\frac{BD^2}{4}$$

$$M_p = 150 \times \frac{200 \times 300^2}{4} \times 10^{-6}$$

$$M_p = 675 \text{ kN-m}$$

22. Radius of mohr's circle :

$$r = \sqrt{\left(\frac{\sigma_1 - \sigma_2}{2}\right)^2 + \tau^2} = \sqrt{\left(\frac{100-(-70)}{2}\right)^2 + 0}$$

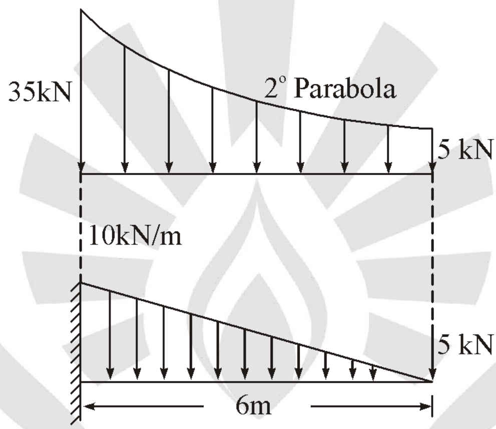
$$r = 85 \text{ N/mm}^2$$

$$\because \quad r_1 = 100 \text{ N/mm}^2$$

$$\sigma_2 = -70 \text{ N/mm}^2$$

$$\tau = 0$$

23. Loading diagram with shear force diagram :



24. ***IS-Code guide lines :***

The allowable load on pile may be taken as 50% of ultimate load at which total settlement of pile is 10% of its diameter. 'or'

Allowable load on pile may be taken as $\frac{2}{3}$ of ultimate load at which total settlement is 12 mm.

Allowable load on pile may be taken as $\frac{2}{3}$ of ultimate load at which net plastic settlement is 6 mm.

25. Saturated density :

$$\rho_{sat} = \left(\frac{G + S.e}{1+e}\right)\rho_w = \left(\frac{G+e}{1+e}\right)\rho_w = \frac{2.65+0.65}{1+0.65} \times 1$$

$$\rho_{sat} = 2 \text{ gm/cc}$$

Active earth pressure

$$p_a = k_a.\sigma_v$$

$$k_a = \frac{1}{3}$$

$$\sigma_v = 14 + 3 \times \gamma_d$$

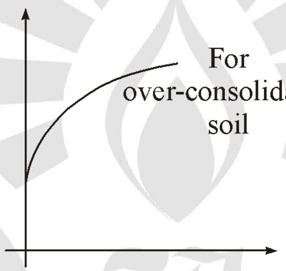
$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.65\times9.81}{1+0.65} = 15.75 \text{ kN/m}^3$$

$$\sigma_v = 14 + (3 \times 15.75) = 61.25 \text{ kN/m}^2$$

$$\therefore \quad p_a = \frac{1}{3}\sigma_v = \frac{1}{3}\times61.25 = 20.2125 \text{ kN/m}^2$$

26. ***The limitations of 'mohr coulomb's theory' :***

- Mohr failure envelope is approximated to a straight line which is found invalid for over-consolidated soils, practical results show that mohr's failure envelope in over consolidated soil is found to be slightly curved.



- The analysis is 2D in which σ_1 and σ_3 is considred whereas actual stress conditions in soil is 3-D in which $\sigma_2 = \sigma_3$.

27. For simply supported beam :

$$\text{Span to depth ratio} = \frac{\text{Span}^2}{10\times A}$$

$$\Rightarrow \frac{\text{Span}}{\text{eff}-\text{depth}} = \frac{25^2}{10\times20} = 3.125$$

$\Rightarrow$ Minimum eff depth required

$$\frac{\text{Span}}{3.125} = \frac{25}{3.125} = 8 \text{ m} = 8000 \text{ mm}$$

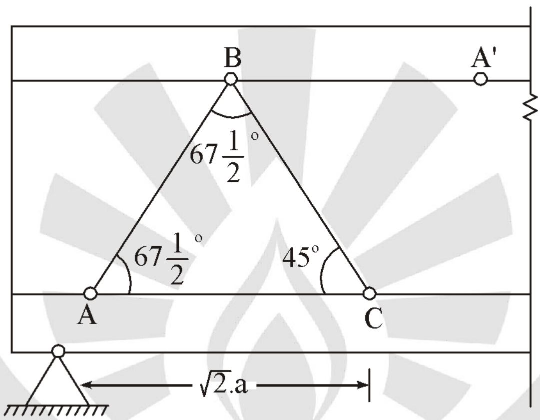
28. When a beam is subjected to torsion, if the depth of the beam is more than 450 mm or for beams not subjected to tersion if the depth of web exceeds 750 mm then side face rainforcement equal to 0.1% of cross sectional area and is equally distributed on both faces of the beam.

29. ***Bent up Bars :***

Single or a group of bent up bars are provided at distance $\sqrt{2}$ id = $\sqrt{2}$.a from support in such a way that

$$\angle ACB = 45^\circ,\ \angle CAB = \angle CBA' = 67\frac{1}{2}^\circ$$

Generally bear should not be bentup beyond a distance $\frac{l}{4}$ from the support. Where l = length of span.



30. ***Permissible Bending Stress :***

- The maximum compressive stress $\sigma_{bc,\ cal}$ is calculated on gross flange area :

$\sigma_{bc,\ cal} = \frac{ND/2}{I_{gross}} \ngtr$ Permissible beinding stress in compression (σ_{bc})

- The maximum tensile stress $\sigma_{bT,\ cal}$ is calculated on net flage area :

$\sigma_{bT,\ cal} = \frac{ND/2}{I_{gross}} \ngtr$ Permissible beinding stress in tension (σ_{bT})

31. ***General Requirements for Lacing :***

- Radius of gyration about the axis ⊥ to the plane of lacing and radius of gyration about the axis in the plane of lacing.
- The lacing system should not be varied throughout the length of the strut as far as practicable.
- The single laced system on opposite sides of the main components should preferably be in the same direction so, the one be the shadow of the other.

32. $$p_b = \frac{3W.e}{td^2}$$

Throat thickness = t = 0.7 s = 0.7 × 10 = 7 mm

e = 100 mm

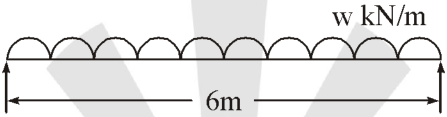
w = 70 kN = 70000 N

d = 350 mm

$$p_b = \frac{3\times70000\times100}{7\times350} = 24.48\ \text{N/mm}^2$$

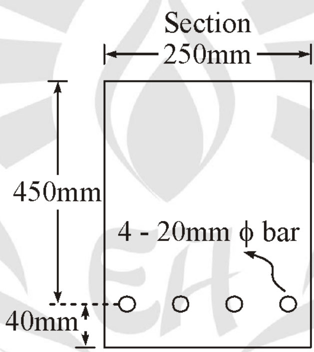
[PART : C]

33. As per the given data in question :



Modular Ratio for M20 = 13.13

$$A_{st} = 4 \times \frac{\pi}{4} = 1256.65\ \text{mm}^2$$



Maximum beinding moment due to total load 'w'

$$BM = \frac{wl^2}{8} = \frac{w(6)^2}{8} = 4.5\ w\ \text{kN-m}$$

Critical dpeth of neutral axis

$$x_c = \left(\frac{m.\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{sT}}\right).d = \frac{(280/3)}{(280/3)+243}\times450$$

$$x_c = 129.89\ \text{mm}$$

Actual depth of neutral axis L

$$\Rightarrow \frac{B.X_a^2}{2} = mA_{st}(d - X_a)$$

$$\Rightarrow \frac{250X_a^2}{2} = 13.33 \times 1256.65(450 - X_a)$$

$\Rightarrow 125X_a^2 = 7538015.02 - 16751.14X_a$

$\Rightarrow X_a^2 + 134.01X_a - 60304.12 = 0$

$$\Rightarrow \quad X_a = \frac{-134.01 \pm \sqrt{(134.01)^2 - 4(1)(-60304.12)}}{2 \times (1)}$$

$\Rightarrow \quad X_a = 187.54$ mm

Since, $X_a > X_c \Rightarrow$ Over Reinforced Section

$\because$ Moment Resistance

$$MR = \frac{1}{2}\ \sigma_{cbc}.B.X_a\left(d - \frac{X_a}{3}\right)$$

$$MR = \frac{1}{2} \times 7.0 \times 250 \times 187.54\left(450 - \frac{187.54}{3}\right)$$

MR = 63586140.275 N-mm = 6358 kN-m

BM = MR

$\Rightarrow 63.58 \times 10^6 = 4.5\ w \times 10^6$

$\Rightarrow \quad w = 14.12$ kN/m

$\because$ Total load = 14.12 kN/m

Self weight = 0.25 × (0.45 + 0.040) × 1 × 25 = 3.06 kN/m

So, Imposed load = 14.12 – 3.06 = 11.06 kN/m

34. Given load transferred from column = 1100 kN = P

So, load/self weight of footing = 10% of P

$$= \frac{10}{100} \times 1100 = 110 \text{ kN}$$

So, totoal load transferred to ground level

$w = 1100 + 110 = 1210$ kN

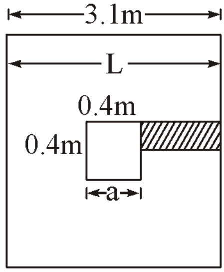
Ultimate load = $w_u = 1.5\ w = 1.5 \times 1210$

$w_u = 1815$ kN

Area required = $A = \dfrac{1815 \times 10^3}{200 \times 10^3} = 9.075\ m^2$

So, Side of footing = $\sqrt{9.075} = 3.0125$ m

Let side of footing = 3.1 m



So, Area provided = 3.1 × 3.1 = 9.61 m^2

Upward pressure on footing = $\frac{1100\times10^3}{3.1\times3.1}$ = 114.46 kN/m^2

Maximum Bending moment $=\frac{w}{2}\left(\frac{L-a}{2}\right)^2=\frac{114.46}{2}\left\{\frac{3.1-0.4}{2}\right\}^2$

B.M. = 104.42 kN-m (For 1 m width)

For depth required :

BM = BR_{lim}

$\Rightarrow$ $104.42 \times 10^6 = 0.36 f_{ck}\, \chi_{u_{lim}} (d - 0.42\, \chi_{u_{lim}})$

$\Rightarrow$ $104.42 \times 10^6 = 0.36 \times 20 \times 1000 \times 0.48d(d - 0.42 \times 0.48 \times d)$

$\Rightarrow$ Eff-depth required

$$d^2 = \frac{104.42\times10^6}{2757.59} = 194.57 \text{ mm}$$

Say d = 200 mm

For steel, $A_{st} = \frac{BM}{0.87 f_y jd} = \frac{104.42\times10^6}{0.87\times415\times(200-0.42\times0.48\times200)}$

A_{st} = 1812.1 mm^2

For 1 m width of footing

Spacing of 25 mm diameter bars :

$\Rightarrow$ $\frac{1000}{1812.1} \times \frac{\pi}{4}(25)^2$ Take 206 spacing c/c

Note: *Since this is square footing so, there is no need of distribution factor* $\left(\frac{2}{1+\beta}\right)$ *for longer and shorter sides of footing.*

35. Given, specific gravity G = 2.70

Weight of the soil = 3200 – 1200 = 2000 gm = 2 kg

Volume of soil = Volume of Core – Cutter = 1000×10^{-6} m^3.

(i) Bulk Density = $\frac{\text{Mass}}{\text{Volume}} = \frac{M}{V} = \frac{2\text{kg}}{1000 \times 10^{-6}}$ = 2000 kg/m^3

(ii) From the given relation : $\Rightarrow$ Se = wG $\Rightarrow$ S.e = 0.15 × 2.7

$S.e = 0.405 \Rightarrow \gamma = \left(\frac{G + Se}{1+e}\right)\gamma_w$ and $\rho = \left(\frac{G+Se}{1+e}\right)\rho_w$

$$\Rightarrow 2000 = \left(\frac{2.7 + 0.405}{1+e}\right) \times 1000 \Rightarrow e = 0.55$$

$$\because Se = 0.405;\ S = \frac{0.405}{0.55} \times 100\% = 73.64\%$$

When soil fully saturated

$$w = \frac{S.e}{G} = \frac{100 \times 0.55}{2.7} = 20.37\%$$

$$\gamma_{sat} = \left(\frac{G + S.e}{1+e}\right)\gamma_w = \left(\frac{2.7+0.55}{1+0.55}\right) \times 1000$$

$$\gamma_{sat} = 2096.77 \text{ kg/m}^3$$

36. The Section is symmetric about y-axis

$$\bar{y} = \frac{A_1 y_1 + A_2 y_2 + A_3 y_3}{A_1 + A_2 + A_3}$$

$$A_1 = \frac{1}{2} \times \frac{\pi}{4} D^2 = \frac{1}{2} \times \frac{\pi}{4}(150)^2 = 8835.75 \text{ mm}^2$$

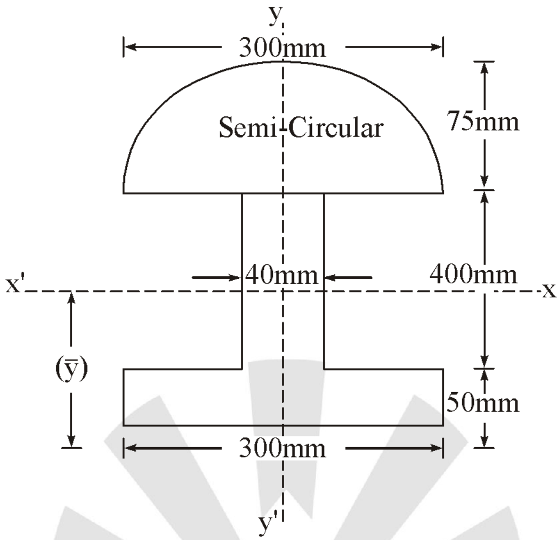
$$A_2 = 400 \times 40 = 16000 \text{ mm}^2$$

$$A_3 = 300 \times 50 = 15000 \text{ mm}^2$$

$$y_1 = \frac{4r}{3\pi} + 400 + 50 = \frac{4 \times 75}{3\pi} + 450 = 481.83 \text{ mm}$$

$$y_2 = 50 + \frac{400}{2} = 250 \text{ mm}$$

$$y_3 = \frac{50}{2} = 25 \text{ mm}$$



$$\bar{y} = \frac{(8835.75 \times 481.83) + (16000 \times 250) + (15000 \times 25)}{(8835.75 + 16000 + 15000)}$$

$= 216.69$ mm

So, Moment of Inertia :

$$I_{yy} = \frac{50 \times 300^3}{12} + \frac{400 \times 40^3}{12} + \frac{1}{2} \cdot \frac{\pi}{64}(150)^4$$

$= 112500000 + 2133333.33 + 12425273.43$

$I_{yy} = 127058606.7675$

$I_{yy} = 1.27 \times 10^8 \text{ mm}^4$

$$I_{xx} = \left(\frac{300 \times 50^3}{12}\right) + \{300 \times 50 \times (216.69 - 25)^2\} + \left(\frac{40 \times 400^3}{12}\right)$$

$$+\{400 \times 40 \times (250 - 216.69)^2\} + \frac{1}{2} \times \frac{\pi}{64}(150)^4$$

$$+\frac{1 \times \pi}{2 \times 4}(150)^2\{308.31 - 59.08\}^2$$

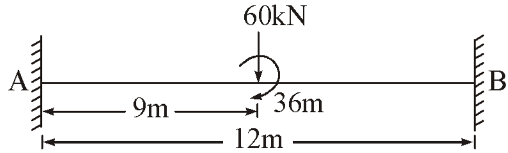
$I_{xx} = 3125000 + 551175841.5 + 213333333.33 + 17752897.6$

$+12425273.4375 + 548837849.966$

$I_{xx} = 1346650195.84 \text{ mm}^4$

$I_{xx} = 13.46 \times 10^8 \text{ mm}^4$

37. The loading diagram given in problem can also be drawn for analysis:



$\because$ Couple = 60 × 0.6 = 36 kN-m

To calculate support reactions and moment reactions, we can use compatibility equations.

According to this then net slope and deflection at any end is zero. (Due to external loading and support reactions)

Deflection at end B_1 due to given loading

$$= \frac{60\times 9^3}{3EI}+\left(\frac{60\times 9^2\times 3}{2EI}\right)+\frac{36\times 9^2}{2EI}+\left(\frac{36\times 9}{EI}\times 3\right)$$

$$= \frac{24300}{EI}(\downarrow)$$

Due to support reaction

$$= \frac{60\times (9)^2}{2EI}+\frac{36\times 9}{EI} = \frac{2754}{EI}$$

Due to support reactions

$$= \frac{R_B(12)^3}{3EI}+\frac{M_B(12)^2}{2EI} = \frac{576R_B+72M_B}{EI}$$

For compatibility, net $\Delta_B = 0$

$\Rightarrow \quad 576R_B + 72M_B = 24300 \quad ...(1)$

Net $\theta_B = 0$

$\Rightarrow \quad 72R_B + 12M_B = 2754 \quad ...(2)$

From equation (1) and (2) we get

$R_B = 54$ kN(↑)

$M_B = -94.5$ kN-m (clockwise)

$\Rightarrow \quad R_A + R_B = 60$

$\Rightarrow \quad R_A + 54 = 60$

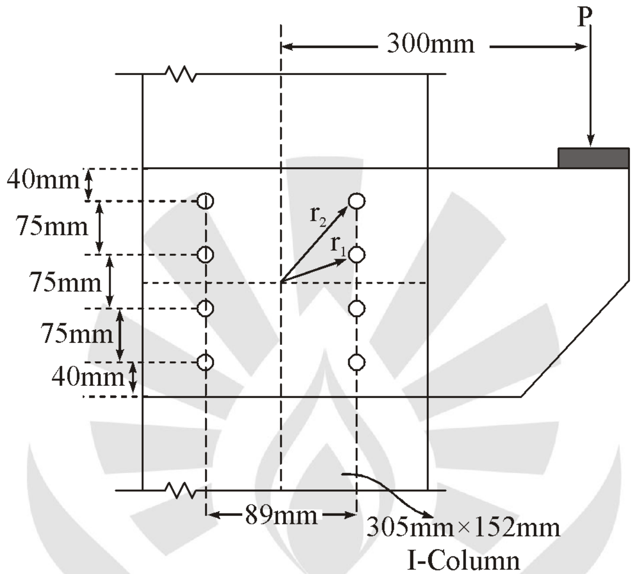
$\Rightarrow \quad R_A = 6$ kN

$\Rightarrow \quad M_A = 94.5 + 36 + 60 \times 9 - 54 \times 12$

$= 22.5$ (Anticlockwise)

38. $r_1 = \sqrt{(12.5)^2 + \left(\frac{89}{2}\right)^2} = 120.98 \text{ mm}$

$r_2 = \sqrt{(37.5)^2 + \left(\frac{89}{2}\right)^2} = 58.19 \text{ mm}$



$\Sigma r_i^2 = 4\, 4r_1^2 + 4r_2^2 = 4 \times (120.98)^2 + 4 \times (58.19)^2$

$\Sigma r_i^2 = 72092 \text{ mm}^2$

Direct force, $P_1 = \frac{P}{8} = 0.125 \text{ P}$

Force due to moment,

$$P_2 = \frac{P \times 300}{72092} \times 120.98 = 0.503 \text{ P}$$

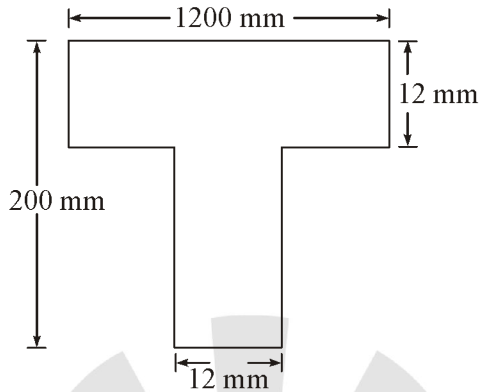
$$\theta = \tan^{-1}\left(\frac{112.5}{44.5}\right) = 68.41^\circ$$

$P_{resultant} \le P_{permissible}$

$\Rightarrow 45 \times 10^{-3} = \sqrt{(0.125P)^2 + (0.503P)^2 + (2 \times 125P \times 0.503P \cos 68.42^\circ)}$

$\Rightarrow 45 \times 10^{-3} = 0.561 \text{ P} = 80.193 \text{ kN}$

39. Given,



Shear-force, V = 200 kN

Shear-Stress $\tau = \dfrac{VA\bar{y}}{IB}$

For Beam-centroid ($\bar{y}$) = $\dfrac{A_1y_1 + A_2y_2}{A_1 + A_2}$

$$\bar{y} = \frac{(120\times12\times6)+(188\times12\times106)}{(120\times12)+(188\times12)}$$

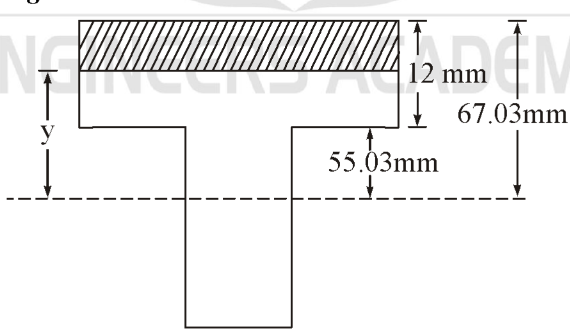
$$\bar{y} = 67.03 \text{ mm (from top)}$$

Moment of Inertia about Neutral axis :

$$= \frac{120\times12^3}{12} + \left\{120\times12\times(67.03-6)^2\right\} + \frac{188^3\times12}{12}$$

$$+\left\{188\times12\times(106-67.03)^2\right\} = 1545 \times 10^6 \text{ mm}$$

In Flange :



$$\tau = \frac{200\times10^3\times(67.03-y)\times120\times\left(\dfrac{67.03+y}{2}\right)}{120\times15.45\times10^6}$$

$$\tau = 0.645(4493 - y^2) \times 10^{-2} \text{ MPa}$$

$$\tau = 29 - (6.47 \times 10^{-3})$$

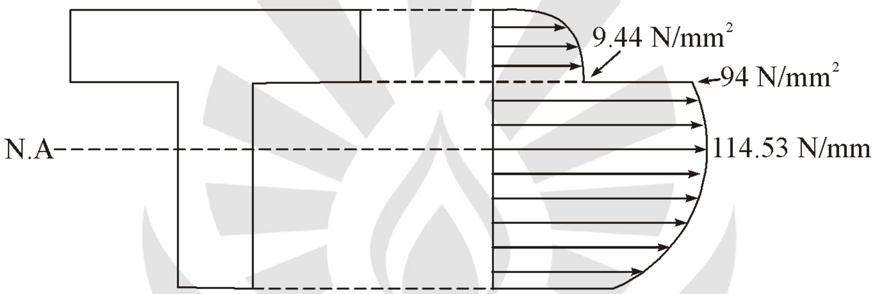
$$\tau = 200 \times 10^3 \times (120\times12\times61.03)+\left(\frac{(55.03^2 - y^2/2)}{12\times15.45\times10^6}\right)$$

$$= \frac{200\times10^3(87883.2+18169.80-6y^2)}{12\times15.45\times10^6}$$

$$= 0.108 \times (106053 - 6y^2) \times 10^{-2}$$

$$= 114.53 - 6.46\times 10^{-3}\ y^2$$

So, Plotting of shear force diagram is as following :



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