→ * Design of Simply Supported Beam:

→ * Step by step procedure:

Step: ① Determination of required Depth:

Step: ② Check for required Depth.

Step: ③ Design for flexure

Step: ④ -Curtullment of Steel.

Step: ⑤ Check for development length.

Step: ⑥ Check for Shear

Step: ⑦ Detailing of reinforcement.

Problem:

A RC rectangular beam of clear span 6M is supported on 300 mm wide masonary walls and carries UDL of intensity 30 KN/m. Design the beam and provide all necessary checks and also sketch the reinforcement details? Use M20 mix & HYSD steel.

Sol: Given data * Clear span = 6M

* Support width = 300 mm

* Working UDL intensity = 30 KN/m

* f_{ck} = 20 mpa & f_y = 415 mpa.

Assume width of the beam 'b' = 300 mm.

Self weight of beam per meter = 5 KN/m.

→ Step: ① Determination of Required Depth:

= [d_{req}]

* Effective span: [Cl: 22.2]

Effective span (L_{eff}) = Clear span + Wall width

= 6000 + 300 = 6300 mm.

Total UDL intensity = $W_D + W_L$

$= 5 + 30 = 35$ KN/m.

Factored UDL intensity $(W_U) = 1.5 \times W$

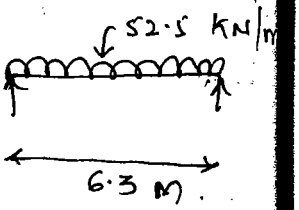
$= 1.5 \times 35 = 52.5$ kN/m.

Ultimate Bending moment $(M_U) = \frac{W_U \times l_{eff}^2}{8}$

$= \frac{52.5 \times 6.3^2}{8} = 260.46$ kN

We have, $M_U = 0.138 f_{ck} b d^2$

$d_{req} = \sqrt{\frac{M_U}{0.138 f_{ck} b}}$



$\Rightarrow d_{req} = \sqrt{\frac{260.46 \times 10^6}{0.138 \times 20 \times 300}}$

$\Rightarrow d_{req} = 560.86$ mm

$\therefore$ Adopt $d = 600$ mm. ; $D = d + d'$

$\Rightarrow D = 600 + 50 = 650$ mm.

Self weight of beam $= 0.3 \times 0.65 \times 25$

$= 4.875$ KN/m. < 5 KN

Hence OK

Step: ② Check for required Depth: [Cl: 23.2]

$L/d \leq k_1 \times k_2 \times k_3 \times k_4 \times k_5$

Where $L = 6000$ mm

$d = 600$ mm

$k_1 = 20$ [for simply supported beam]

$k_2 = 1$ [for span < 10 m]

$k_3 = k_4 = 1$ [Assume]

$k_5 = 1$ [for rectangular section]

$$\Rightarrow \frac{10}{600} \leq 20 \times 1 \times 1 \times 1 \times 1$$

$$\Rightarrow 10 \leq 20 \quad \text{Hence OK.}$$

The beam is safe in deflection.

Step: ③ Design for flexure [Appendix G pg: No-96] IS: 456-2000

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y \times A_{st}}{f_{ck} b d}\right]$$

$$\Rightarrow 260.46 \times 10^6 = 0.87 \times 415 \times A_{st} \times 600 \times \left[1 - \frac{415 \times A_{st}}{20 \times 300 \times 600}\right]$$

$$\Rightarrow A_{st} = 1442.04 \text{ mm}^2$$

No. of bars required $= \frac{A_{st}}{A_\phi} = \frac{1442.04}{314} = 4.59 \simeq 5$ No. of bars

Assume 20 mm dia bars

Provide 5 no – 20 mm dia bars as positive moment reinforcement along the bottom edge of the beam at an effective cover of 50 mm.

A_{st} provided $= 5 \times 314 = 1570 \text{ mm}^2$ [$> 1442.04 \text{ mm}^2$]

Check for A_{st} : [cl : 26.5.1]

$$A_{st\,min} = \frac{0.85 \times b \times d}{f_y} = \frac{0.85 \times 300 \times 600}{415} = 368.67 \text{ mm}^2$$

$$A_{st\,max} = 0.04 \times b \times D = 0.04 \times 300 \times 650 = 7800 \text{ mm}^2$$

$A_{st\,min} < A_{st} < A_{st\,Max}$ Hence OK

Step: (4) Curtilment of Steel/Reinforcement:

[Cl: 26.2.1]

* Required development length for 20 mm dia ba

$$\Rightarrow L_{d_{req}} = \frac{\phi \sigma_s}{4 \times \tau_{bd}}$$

$$\Rightarrow L_{d_{req}} = \frac{\phi \times 0.87 * 415}{4 \times (1.2 \times 1.6)}$$

$$\Rightarrow L_{d_{req}} = 47\phi$$

At Mid span

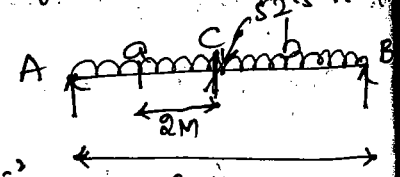
Development length required for positive momen

reinforcement = $47\phi + 12\phi$ or $47\phi + d$ } Which is greater

$\therefore$ Development length req = $47 \times 20 + 600$

= 1540 mm

170.18 $\therefore$ provide the development length of 2 mm o Either side of mid span of beam.



Moment at 1.15 M from support $M_u = V_A \times 1.15 - \frac{52.5 \times 1.15^2}{2}$

$V_A = \frac{wl}{2} = \frac{52.5 \times 6.3}{2}$ $\Rightarrow M_u = 155.46$ kN-m

= 165.4 kN

$$M_U = 0.87 f_y A_{st} \times d \left[1 - \frac{f_y A_{st}}{f_{ck} bd}\right]$$

$$\Rightarrow 155.46 \times 10^6 = 0.87 \times 415 \times A_{st} \times 600 \times \left[1 - \frac{415 \times A_{st}}{20 \times 300 \times 600}\right]$$

$$\Rightarrow A_{st} = 789.47 \text{ mm}^2$$

No. of bars $= 789.47/314$

$= 2.51 \approx 3$ no.

∴ 2 no - 20 mm dia bars are curtailed at distance of 2 m on either side of mid span and the remaining steel is continued into the support on same face.

Step: ⑤ Check for Development length:

Development length provided $= 6.3/2 - 2M$

$= 3.15 - 2 = 1.15 M$

$= 1150$ mm.

$\leq \frac{47}{3}\phi = 313.3$ mm

At support $L_d \leq \frac{M}{B} + L_0$

Where $M = 0.87 f_y A_{st} \times d \left[1 - \frac{f_y A_{st}}{f_{ck} bd}\right]$

Where $A_{st} = 3 \times 314$

$= 942 \text{ mm}^2$

∴ $M_U = 0.87 \times 415 \times 942 \times 600 \left[1 - \frac{415 \times 942}{20 \times 300 \times 600}\right]$

$\Rightarrow M_U = 181.9 \times 10^6$ kN-m

$$V = \frac{WL}{2} = 165.4 \text{ KN}.$$

Initially assume $L_o = 0$

$$L_d \leq M/BV$$

$$\Rightarrow 47\phi \leq \frac{181.9 \times 10^6}{165.4 \times 10^3}$$

$$\Rightarrow 47\phi \leq 1.09 \times 10^3$$

$$\Rightarrow \phi \leq \frac{1.09 \times 10^3}{47}$$

$$\Rightarrow \phi \leq 23.39 \text{ mm}.$$

$[20 < 23.39]$ Hence ok

Provide a 90° bend beyond the center of the support.

Anchorage length $L_o = 8\phi = 8 \times 20 = 160 \text{mm}$.

Step: (6) Check for shear: [Cl: 40]

Nominal shear stress $= \tau_v = V_u / bd = \frac{165.4 \times 10^3}{300 \times 600}$

$$\Rightarrow \tau_v = 0.91 \text{ mpa}.$$

Design shear stress $= \tau_c = 0.49 \text{ mpa}$ [from tab

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 942}{300 \times 600}$$

$$\Rightarrow P_t = 0.52\%.$$

Maximum shear capacity $\tau_{c\,max} = 2.8$ mpa.

* $\tau_v < \tau_{c\,max}$ — ok

* $\tau_v > \tau_c$

Assume 8 mm dia 2 legged vertical stirrups

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.5 \text{ mm}^2$$

$$V_{us} = V_u - \tau_c\, bd$$

$$= 165.4 \times 10^3 - 0.49 \times 300 \times 600$$

$$V_{us} = 77.2 \text{ KN}$$

$$S_{v_1} = 0.87 f_y A_{sv} d / V_{us}$$

$$= \frac{0.87 \times 415 \times 100.5 \times 600}{77.2 \times 10^3}$$

$$\Rightarrow S_{v_1} = 282 \text{ mm}$$

$$S_{v_2} = 0.75\, d = 0.75 \times 600 = 450 \text{ mm}$$

$$S_{v_3} = 300 \text{ mm}$$

[19] ∴ provide 8 mm dia 2 legged vertical stirrups at 250 mm center to center.

→ check for A_{sv} :

$$\frac{A_{sv}}{b s_v} \geq \frac{0.4}{0.87 f_y}$$

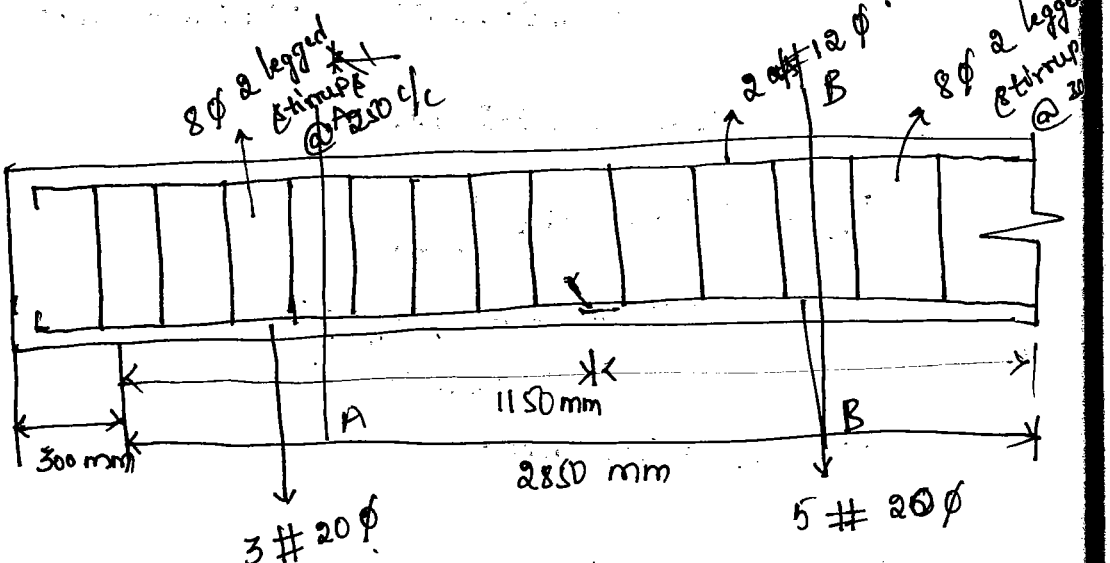
$$\Rightarrow A_{sv} \geq \frac{0.4 \times b \cdot s_v}{0.87 f_y}$$

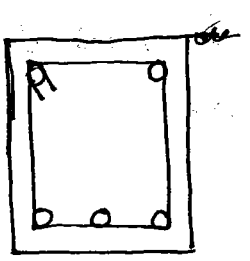
$$\Rightarrow A_{sv} \geq \frac{0.4 \times 300 \times 250}{0.87 \times 415}$$

$$\Rightarrow A_{sv} \geq 83 \text{ mm}^2 \quad [\because < 100.5 \text{ mm}^2]$$

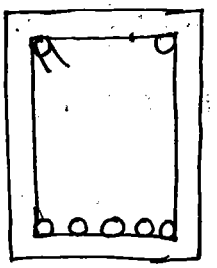
Hence ok

Step: (7) Detailing of reinforcement:





Section A-A



Section B-B

→ * Calculation of Deflection.

Problem: A rectangular cantilever beam of span 4M is 350 mm x 650 mm in cross section. Bending moment at the support due to UDL [Uniformly distributed loads] is 150 KN-m out of which 50% moment is due to permanent loads. Check the beam for deflection. It carries 3-25 mm dia bars in tension at an effective cover of 50 mm. Use M_{20} concrete & HYSD steel.

Solution: Given data

* Span of the beam 'L' = 4M = 4000 mm.
* Width 'b' = 350 mm
* Overall depth 'D' = 650 mm.
* Effective cover 'd' = 50 mm.
* Working Bending moment 'M' = 150 KN-m.
* Area of reinforcement 'A_{st}' = $3 \times \frac{\pi}{4} \times 25^2$
  = 3×491
  = $1473\ mm^2$.
* percentage of moment due to permanent loads = 50%.
* f_{ck} = 20 mpa.
* f_y = 415 mpa.

→ Step: ① Calculation of short term deflection: [clause - C.5.2]

~~$\alpha_c = \frac{W \times L^4}{8EI}$~~ = $\frac{W \times L^2}{2}\left[\frac{L^2}{4EI}\right]$ ∵ $M = \frac{W \times L}{2}$

$\alpha_c = \frac{W \times L^4}{8EI} = \frac{W \times L^2}{2}\left[\frac{L^2}{4EI}\right]$ ∵ $M = \frac{W \times L}{2}$

$= M \cdot \frac{L^2}{4EI}$.



Where $M = 150 \times 10^6$ N-mm.

$L = 4000$ mm.

$E_c = 5000\sqrt{f_{ck}}$. [clause: 6.2.3 in IS:456-2000]

$$I = I_{eff} = \frac{I_{cr}}{1.2 - \frac{M_{cr}}{M}\frac{z}{d}\left[1 - \frac{x}{d}\right]\frac{b_w}{b_f}}$$

$$I_{cr} = \frac{bx^3}{3} + m \cdot A_{st}[d-x]^2$$

m = Modular ratio $= E_s / E_c = \frac{2 \times 10^5}{22360} = 8.94$

To find neutral axis depth 'x'

$$\frac{b(x^2)}{2} = m \cdot A_{st}[d-x]$$

$$\Rightarrow \frac{350(x^2)}{2} = 8.94 \times 1473\,[600 - x]$$

$$\Rightarrow x = 178.16 \text{ mm}$$

Lever arm $z = d - \frac{x}{3} = 540.61$ mm.

$$\therefore I_{cr} = \frac{bx^3}{3} + m \cdot A_{st}[d-x]^2$$

$$= \frac{350 \times (178.16)^3}{3} + 8.94 \times 1473\,[600 - 178.16]^2$$

$$= 659.74 \times 10^6 + 2.34 \times 10^9$$

$$= 2.99 \times 10^9 \text{ mm}^4$$

$$I_{gr} = \frac{bD^3}{3} = \frac{350 \times 650^3}{12} = 8 \times 10^9 \text{ mm}^4$$

$f_{cr} = 0.7\sqrt{f_{ck}} = 3.13$ mpa. [cl: 6.2.2].

$y_t = \frac{D}{2} = \frac{650}{2} = 325$ mm.

$M_{cr} = \frac{f_{cr} \times I_{gr}}{y_t} = \frac{3.13 \times 8 \times 10^9}{325} = 77.04$ KN-m.

$\frac{b_w}{b_f} = 1$ [for rectangular section]

$$\therefore I = I_{eff} = \frac{2.99 \times 10^9}{1.2 - \frac{77.04 \times 10^6}{8.3 \times 150 \times 10^6} \times \frac{540.61}{600}\left[1 - \frac{178.16}{600}\right] \times 1}$$

$$= \frac{2.99 \times 10^9}{874.5 \times 10^{-3}}$$

$I = I_{eff.} = 3.41 \times 10^9$ mm^4

$\therefore$ Deflection $\alpha_e = \frac{M \cdot L^2}{4 E_c I_{eff.}}$

$$= \frac{150 \times 10^6 \times 4000^2}{4 \times 22360 \times 3.41 \times 10^9}$$

$\alpha_e = 7.86$ mm.

→ Step: ② Calculation of shrinkage deflection: [cl: C-3]

$\alpha_{cs} = k_3 \psi_{cs} L^2$

Where $k_3 = 0.5$ for cantilever beam.

$L = 4000$ mm.

ψ_{cs} = Shrinkage Curvature = $K_4 \frac{\varepsilon_{cs}}{D}$

where $\varepsilon_{cs} = 0.0003$

=

[pg: NO - 16]
IS: 456 - 2000

$D = 650$ mm.

$K_4 = 0.72 \times \frac{P_t - P_c}{\sqrt{P_t}} \leq 1.0$

for $0.25 \leq P_t - P_c < 1.0$

$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 1473}{350 \times 600} = 0.7\%$

$P_c = 0$

$\therefore K_4 = 0.72 \times \frac{0.7 - 0}{\sqrt{0.7}} = 0.60$

$\psi_{cs} = K_4 \frac{\varepsilon_{cs}}{D}$

$= 0.6 \times \frac{0.0003}{650} = 2.78 \times 10^{-7}$

$\therefore$ Shrinkage deflection

$\alpha_{cs} = K_3 \psi_{cs} L^2$

$= 0.5 \times 2.78 \times 10^{-7} \times 4000^2$

$= 2.22$ mm

→ Step: (3) Calculation of Creep Deflection [C1: C-4]

$$\alpha_{cc\,(perm)} = \alpha_{i,\,cc\,(perm)} - \alpha_{i\,(Perm)}$$

Where $\alpha_{i\,cc\,(perm)} = \dfrac{M \cdot L^2}{4 E_{ce} I_{eff}}$

Where $E_{ce} = \dfrac{E_c}{1+\theta} = \dfrac{22360}{1+1.6}$ [C1: 6.2.5.1] $\theta = 1.6$ for 28 days age of Concrete

$\Rightarrow E_{ce} = 8600$ mpa.

Then, Modular ratio $m = \dfrac{E_s}{E_{ce}} = \dfrac{2 \times 10^5}{8600}$

$\Rightarrow m = 23.25$

To find Neutral axis depth 'x'

$$\frac{bx^2}{2} = m \cdot A_{st} \cdot [d-x]$$

$$\Rightarrow \frac{350 \times x^2}{2} = 23.25 \times 1473 [600 - x]$$

$\Rightarrow x = 258.51$ mm

Lever arm $Z = d - \dfrac{x}{3} = 600 - \dfrac{258.51}{3}$

$\Rightarrow Z = 513.83$ mm

$$I_{cr} = \frac{bx^3}{3} + m \cdot A_{st} [d-x]^2$$

$$= \frac{350 \times 258.51^3}{3} + 23.25 \times 1473 [600 - 258.51]^2$$

$$= 2.01 \times 10^9 + 3.99 \times 10^9$$

$I_{cr} = 6 \times 10^9$ mm^4

$$I = I_{eff} = \frac{I_{cr}}{1.2 - \frac{M_{cr}}{M_a}\frac{Z}{d}\left[1-\frac{x}{d}\right]\frac{b_w}{b_f}}$$

$$= \frac{6 \times 10^9}{1.2 - \frac{77.04}{150} \times \frac{513.83}{600}\left[1-\frac{258.51}{600}\right]}$$

$$I = I_{eff} = 6.3 \times 10^9 \text{ mm}^4$$

$$\therefore \alpha_{i(CC)(Perm)} = \frac{M_{(Perm)} L^2}{4 E_{ce} \times I_{eff}}$$

$$= \frac{0.5 \times 150 \times 10^6 \times 4000^2}{4 \times 8600 \times 6.3 \times 10^9}$$

$$\Rightarrow \alpha_{i(CC)(Perm)} = 5.5 \text{ mm}$$

$$\alpha_{i(Perm)} = \frac{M_{(Perm)} \cdot L^2}{4 E_{ce} \times I_{eff}}$$

$$= \frac{0.5 \times 150 \times 10^6 \times 4000^2}{4 \times 22360 \times 3.41 \times 10^9}$$

$$\Rightarrow \alpha_{i(Perm)} = 3.93 \text{ mm}$$

$$\therefore \alpha_{(CC)(Perm)} = 5.5 - 3.93 = 1.57 \text{ mm}$$

→ Step : ④ Calculation of Total Deflection [Cl: C-1]

Total Deflection 'α' = Shortterm deflection
+ long term deflection

$= \alpha_{ie} + [\alpha_{cE} + \alpha_{cc}]$

$= 7.86 + [2.22 + 1.57]$

$\alpha = 11.65$ mm.

—* Check for total deflection [Cl: 23.2.1]

* Permissiable vertical deflection of a structural member of the age of 28 days = span/250

$= 4000/250 = 16$ mm,

$> \alpha = 11.65$ mm,

Hence beam is safe in deflection