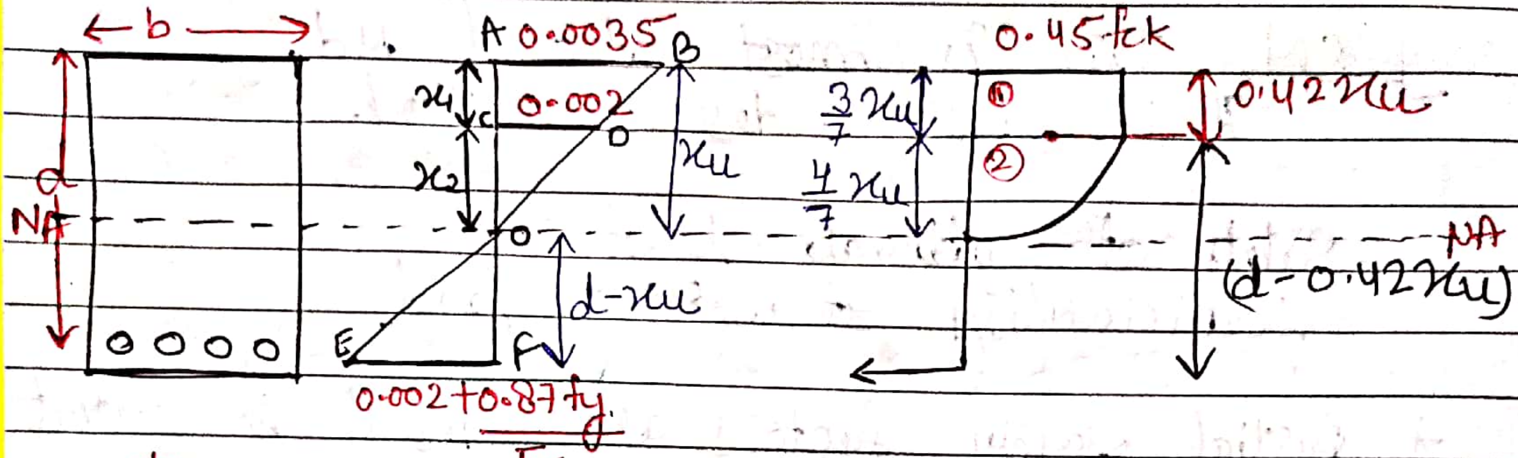
Analysis of SRB by LSM



section

strain Diagram

stress diagram

Analysis of strain Block

In strain diagram, using similar triangle in ΔOAB & ΔOCD

$$\frac{.0035}{x_u} = \frac{0.002}{x_2} \quad \text{so } x_2 = \frac{4}{7}x_u \;\Big|\; x_1 = \frac{3}{7}x_u$$

$$\because x_1 + x_2 = x_u \Rightarrow \frac{4}{7}x_u + \frac{3}{7}x_u$$

now using similar tringle in ΔOAB & ΔOEF

$$\frac{0.0035}{x_u} = \frac{0.002 + \frac{0.87 f_y}{E_s}}{d - x_u}$$

$$\Rightarrow \frac{d - x_u}{x_u} = \frac{0.002 + \frac{0.87 f_y}{E_s}}{0.0035}$$

for different values f_y, x_u will be different.

$$\Rightarrow x_u = 0.53d \quad (Fe250)$$

$$x_u = 0.48d \quad (Fe415)$$

$$x_u = 0.46d \quad (Fe500)$$

Parabolic Area = $\frac{2}{3}$ of Area of Rectangle

Page No.:
Date :

Notes

Analysis of stress block

Comp. in (1) Region

Comp. in (2) Region

Total compressive force = $C_1 + C_2$

(Stress x Area)

Max. comp. ~~stress~~ force in (1) = $\underbrace{0.45 f_{ck}}_{\text{Stress}} \times \underbrace{\frac{3}{7} x_u \times b}_{\text{Area of (1) Rectangle}}$

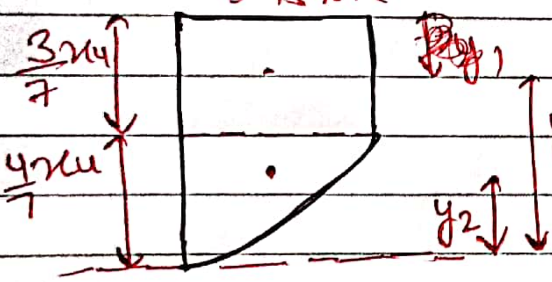
$0.45 f_{ck}$

Max. Comp. force in (2) = $\frac{2}{3} \times 0.45 f_{ck} \times \frac{4}{7} x_u \times b$

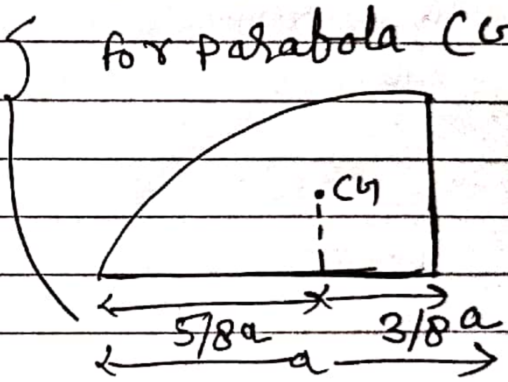
Total comp. = $0.45 f_{ck} \times \frac{3}{7} x_u b + \frac{2}{3} \times 0.45 f_{ck} \times \frac{4}{7} x_u b$

$= 0.36 f_{ck} x_u b$

To find out the C.G of this whole section



for parabola CG



$$\bar{y} = \frac{C_1 y_1 + C_2 y_2}{C_1 + C_2}$$

here $y_1 = \frac{4}{7} + \frac{3}{7} \times \frac{1}{2} = \frac{4}{7} + \frac{3}{14}$

$y_2 = \frac{3}{8} \times \frac{4}{7}$

from ~~bottom fibre~~ Neutral axis $= 0.58 x_u$

Position of C.G $\boxed{\text{from top fibre} = 0.42 x_u}$

Centre of compression force

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comp. force

MOR = C × lever Arm

lever arm is the distance b/w resultant compressive force & resultant tensile force.
here lever arm distance = $d - 0.42x_u$

$$MOR = 0.36 f_{ck} x_u . b \times (d - 0.42x_u)$$

OR

$$MOR = 0.87 f_y \times A_{st} \times (d - 0.42x_u)$$

comp. force.

Notes

MoR = C x lever Arm

leverarm is the distance b/w Resultant Compressive force & Resultant tensile force.
here lever arm distance = $d - 0.42x_u$

$$MoR = 0.36 f_{ck}\, x_u . b \times (d - 0.42 x_u)$$

or

~~MoR = 0.87 fy x Ast x (d - 0.42xu)~~

$$M_u = 0.36 f_{ck} \cdot b \cdot d \cdot x_u \left(1 - \frac{0.42 x_u}{d}\right)$$

$$\Rightarrow \boxed{M_u = 0.36 f_{ck} \cdot \frac{x_u}{d}\left(1 - \frac{0.42 x_u}{d}\right) bd^2}$$

or

$$M_u = T \times \text{leverarm}$$
$$= 0.87 f_y A_{st} \times (d - 0.42 x_u)$$
$$\Rightarrow 0.87 f_y A_{st}\, d\left(1 - \frac{0.42 x_u}{d}\right) \quad \text{—— (1)}$$

for equilibrium of forces

Total tension = Total comp.

$$0.87 f_y A_{st} = 0.36 f_{ck} . x_u . b$$

$$\boxed{x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} . b}}$$

put the value of x_u in Eq (1)

Teacher's Signature

Notes

Page No.:
Date : / /

$$\boxed{M_u = 0.87 f_y A_{st} \cdot d \left(1 - \frac{A_{st} f_y}{bd \cdot f_{ck}}\right)}$$

Limiting value of moment of Resistance

The depth of neutral axis is limited to x_{umax}. The maximum value of neutral axis depth gives the max. or limiting value of moment of resistance.

$$M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42 \frac{x_{umax}}{d}\right) bd^2$$

Para G.1.1 (c) of IS 456

for Fe 250 $x_{umax} = 0.53d$ & $f_y = 250\ N/mm^2$

$M_{ulim} = 0.148 f_{ck} bd^2$

Fe 415, $x_{umax} = 0.48d$

$M_{ulim} = 0.138 f_{ck} bd^2$

Fe 500, $x_{umax} = 0.46d$

$M_{ulim} = 0.133 f_{ck} bd^2$

Balance, under R/f, over R/f sections

The value of $\frac{x_u}{d}$, calculated from the stress block, is equal to

$$\boxed{\frac{x_u}{d} = \frac{0.87 f_y \cdot A_{st}}{0.36 f_{ck} \cdot b}}$$

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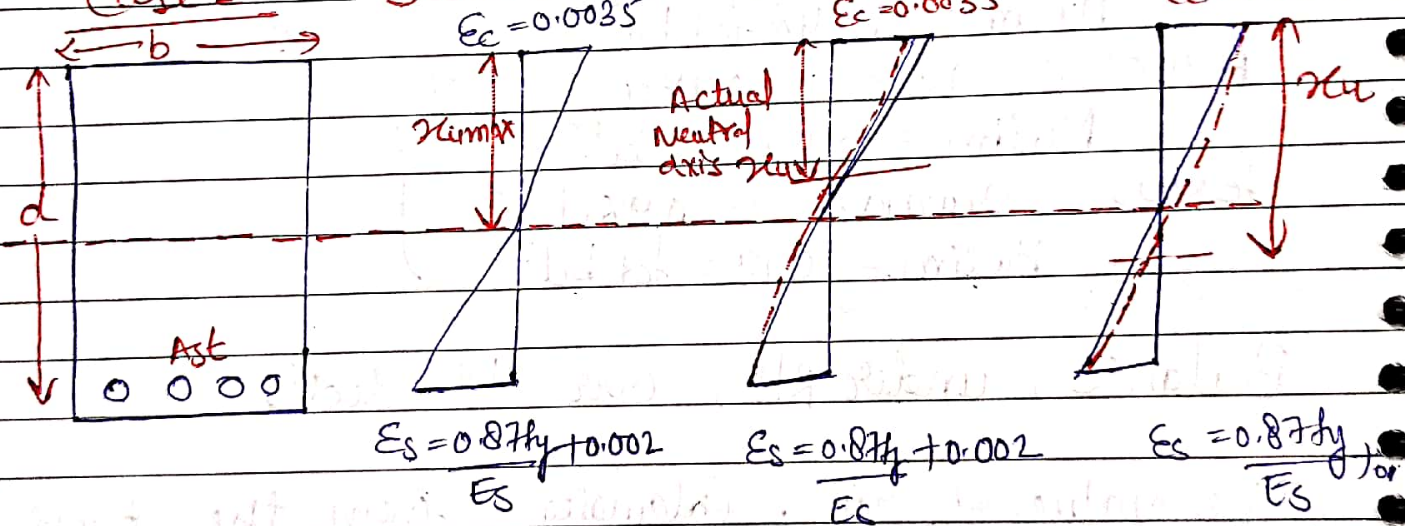
The limiting value of $\frac{x_u}{d}$ i.e. $\frac{x_{umax}}{d}$, as calculated from the strain distribution diag. is as follows.

$$\boxed{\frac{x_{umax}}{d} = \frac{0.0035}{\frac{0.87f_y}{E_s} + 0.0055}}$$

$\frac{x_u}{d}$, & $\frac{x_{umax}}{d}$ are compared and the following 3 cases may arise.

Case 1 Balanced section



(i) $\frac{x_u}{d} = \frac{x_{umax}}{d}$

(ii) $P_t = P_{t\,lim}$

(iii) The yield stress in ~~yield~~ steel &

ultimate strain in concrete reaches their maximum value at the same time

(iv) The moment of Resistance is equal to limiting value of Mulim & can be Calculated as

$$M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right) bd^2$$

or $$M_{ulim} = 0.87 f_y A_{st} d\left(1 - 0.42\frac{x_{umax}}{d}\right)$$

(2) Under Reinforced section $\left(\frac{x_u}{d} < \frac{x_{umax}}{d}\right)$

In this case the steel fails first by reaching its yield ~~val~~ strain value, although in concrete the ultimate ~~strength~~ strain has not achieved. steel is ductile material & it gives sufficient time before failure.

(3) Over R/fd section $\left(\frac{x_u}{d} > \frac{x_{umax}}{d}\right)$

In this strain in concrete reaches its ultimate value earlier than steel. O/R beams fails by crushing failure of concrete. concrete is brittle material it fails suddenly without warning.

IS 456 : 2000 Recommends that over R/F section should be Redesigned.

Page No.:
Date :

Type (I) Finding Moment of Resistance,

1. For the given Grades of concrete & steel. (fck, and fy) find the depth of neutral axis.

$$\frac{x_u}{d} = \frac{0.87 f_y A_{st}}{0.36 f_{ck} B d}.$$

2. Find limiting value of neutral axis

$$\frac{x_{u\,max}}{d} = \frac{0.0035}{\frac{0.87 f_y}{E_s} + 0.0055}$$

or by referring table 4.2 P.N 70 IS 456

f_y	$x_{u\,max}/d$
250	0.53
415	0.48
500	0.46

3. Compare x_u & $x_{u\,max}$

(i) If $\frac{x_u}{d} < \frac{x_{u\,max}}{d}$ the beam is under R/F & MOR is calculated by following eqn.

$$M_u = 0.87 f_y A_{st} \cdot d \left(1 - \frac{0.42 x_u}{d}\right).$$

(ii) If $\frac{x_u}{d} = \frac{x_{u\,max}}{d}$, the beam is assigned as balanced section & MOR

Teacher's Signature ______

Notes

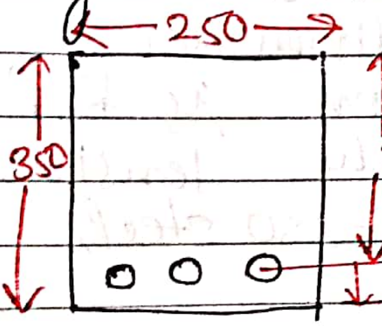
Page No.: Date: / /

is given by following exp.

$$M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right) bd^2$$

(iii) If $\frac{x_u}{d} > \frac{x_{umax}}{d}$, the moment of resistance is equal to Mulim but the code recommend to redesign the section.

Q. Determine the moment of resistance of a beam of dimension 250mm x 350mm.
The area of steel consists 3 bars of 12mm dia placed at a distance of 40 mm from bottom of beam. Use M20 & Fe 415 steel.



D = 350mm
d = 310mm

350 − 40 = 310mm = effective depth

40 = eff. cover

$b = 250mm$, $A_{st} = 3 \times \frac{\pi}{4} \times 12^2 = 339\ mm^2$

$f_{ck} = 20\ N/mm^2$, $f_y = 415\ N/mm^2$

Depth of neutral axis (x_u)

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times 339}{0.36 \times 20 \times 250}$$

$$x_u = 68\ mm$$

Teacher's Signature............

Notes

Page No.:
Date: / /

$x_{umax} = 0.48\,d$ {for Fe415

$\Rightarrow 0.48 \times 310$

$\Rightarrow 148.8\,mm > 68\,mm$

$\therefore x_{umax} > x_u$ hence section is under R/F

Moment of Resistance

$M_u = 0.87 f_y A_{st} (d - 0.42 x_u)$

$\Rightarrow 0.87 \times 415 \times 339 \times (310 - 0.42 \times 68)$

$\Rightarrow$ **34.44 KNm.**

Q. Determine the moment of resistance of the beam having dimension 300x550mm eff. The beam is R/F with 1963 mm^2 of steel in the tension zone use M20 concrete & Fe250 steel.

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Type II

Limit state of Collapse – Shear & Bond

Q. A rectangular beam is 20cm wide & 40cm deep up to the centre of reinforcement. Find the area of r/f required if it has to resist a moment of 25kNm. Use M20 & Fe415 steel.

Soln Given $b = 20cm = 200mm$

$d = 40cm = 400mm$

$M = 25kNm = 25 \times 10^6$ Nmm

$f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$

Factored Bending Moment (γ = load factor = 1.5)

$= \gamma \times BM$

$= 1.5 \times 25 \times 10^6 = 37.5 \times 10^6$ Nmm

Factored BM = Moment of Resistance

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

$$M_u = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{415 A_{st}}{20 \times 200 \times 400}\right]$$

$\Rightarrow A_{st}(400 - 0.10375 A_{st}) = 103863.73$

$\Rightarrow A_{st}^2 - 3855.4 A_{st} + 1001096.192 = 0$

$$A_{st} = \frac{3855.4 \pm \sqrt{(3855.4)^2 - 4 \times 1001096.192}}{2}$$

$A_{st} = 280 mm^2$

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or you can calculate Ast directly by

$$\boxed{Ast = \frac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6 (Mu)}{f_{ck} b d^2}}\right] bd}$$

→ Factored Moment

$$Ast = \frac{0.5 \times 20}{415} \times \left[1-\sqrt{1-\frac{4.6 \times 37.5 \times 10^6}{20 \times 200 \times (400)^2}}\right] 200 \times 400$$

$$= 280.14 \ mm^2$$

Depth of neutral axis

$$x_u = \frac{0.87 f_y . Ast}{0.36 f_{ck} . b}$$

$$= \frac{0.87 \times 415 \times 280}{0.36 \times 20 \times 200} = 70.2 \ mm$$

Limiting depth of neutral axis

$$x_{umax} = 0.48 d \quad (\text{for Fe } 415)$$

$$\Rightarrow 0.48 \times 400 = 192 mm$$

$x_{umax} > x_u$

hence under RIF and design is OK

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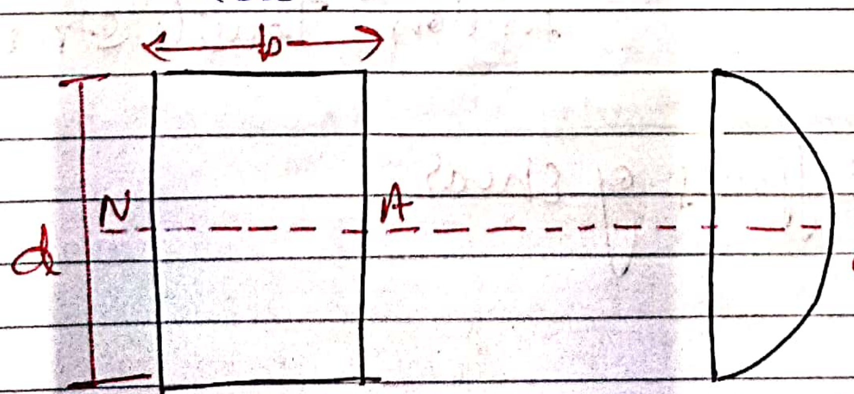
Limit state of collapse: Shear & Bond

A beam loaded with transverse load is subjected to shear force & bending moment. The shear force at any section is equal to the rate of change of bending moment. The shear force results in to shear stresses across cross section which is given by the eqn.

$$q = \frac{V(A\bar{y})}{I \cdot b}$$

where q = shear stress
I = moment of Inertia of the beam section
b = width of section
V = shear force at the section.
$(A\bar{y})$ = First moment of Area above the section about neutral axis.

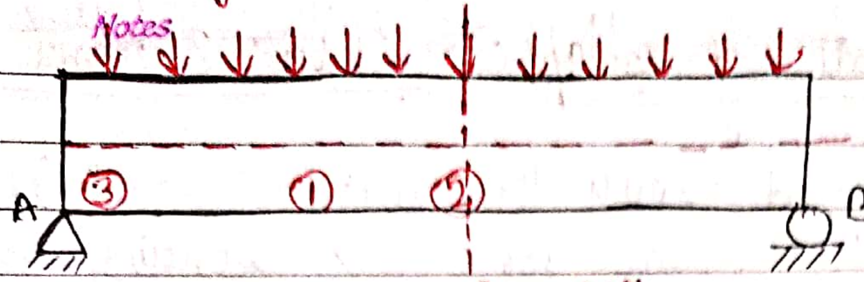
On the basis of above eqn the shear stress distribution across a rectangular X-section is shown in fig. It is parabolic with zero at top & bottom and the max. shear stress occurs at neutral axis is equal to $\frac{3V}{2bd}$



$q_{max} = \frac{3V}{2bd}$

Shear stress dis. in rect section.

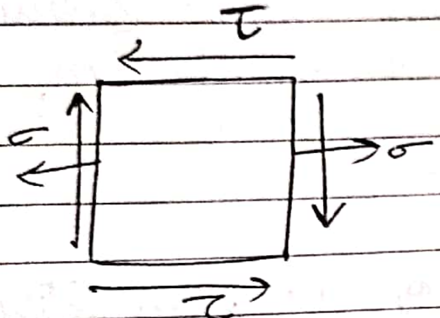
Effects of Shear: Diagonal tension



B.M = 0
SF max

BM = max
SF = 0

fig (a)

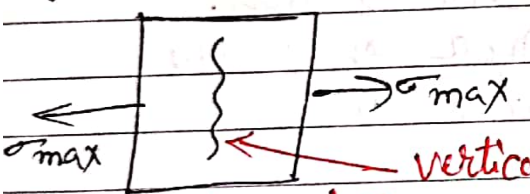


σ - bending stress at section

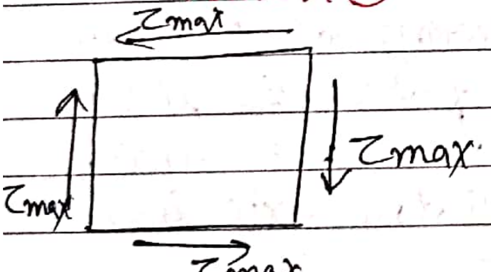
τ = shear stress at section

fig (b) element 1

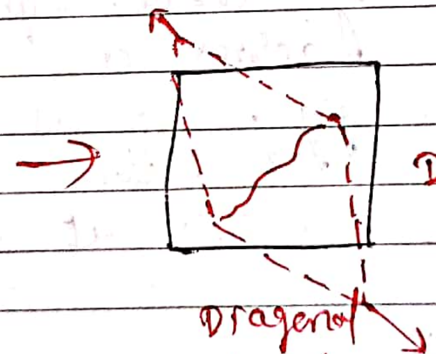
σ_{max} - Max. Bending stress



element 2



element 3



Diagonal crack at (45°)

Diagonal tension

Tensile stress

Effects of shear

Teacher's Signature ____________

Notes

Page No.:
Date : / /

This tension which is caused in the tensile zone of the beam due to shear, at or near the support is called as diagonal tension. The diagonal tension results in cracks at 45° degrees. Concrete is quite strong in shear but the diagonal tension which is caused by shear cannot be resisted by concrete alone. So shear R/f is provided in the RCC beam to take up the diagonal tension and prevent cracking of beam.

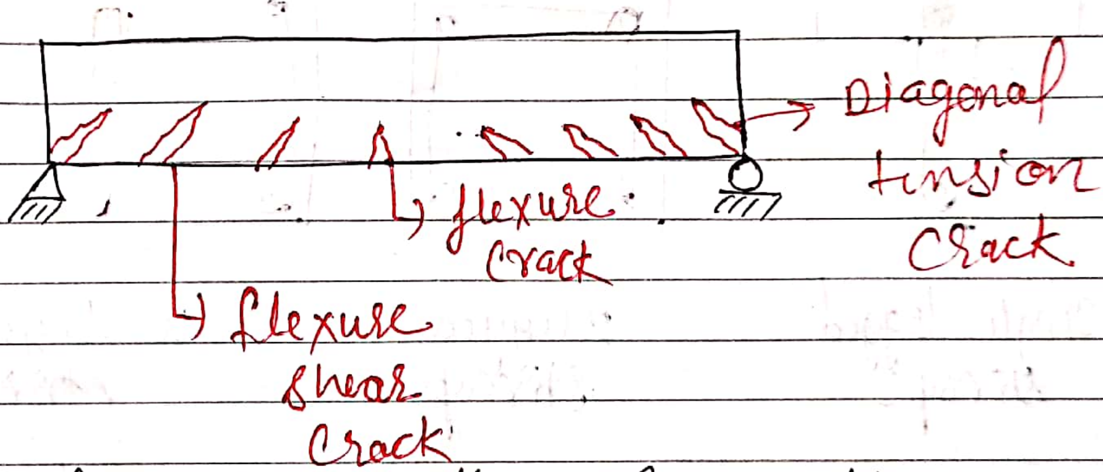


fig. Crack pattern in Simply supp. Beam

1. At or near the midspan, the crack will be vertical (flexure cracks due to bending alone)
2. At or near the supports the crack are inclined at 45° (shear or diagonal tension crack.
3. In b/w the support and mid span the cracks inclination vary from 45° to 90° Gradually (flexure shear crack)

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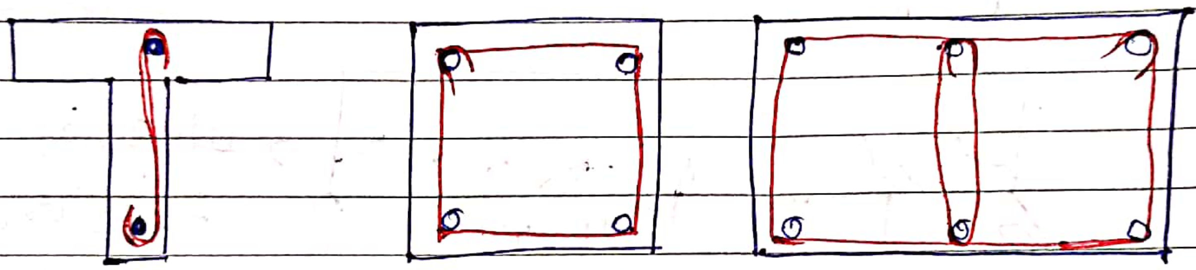
Types of Shear Reinforcement

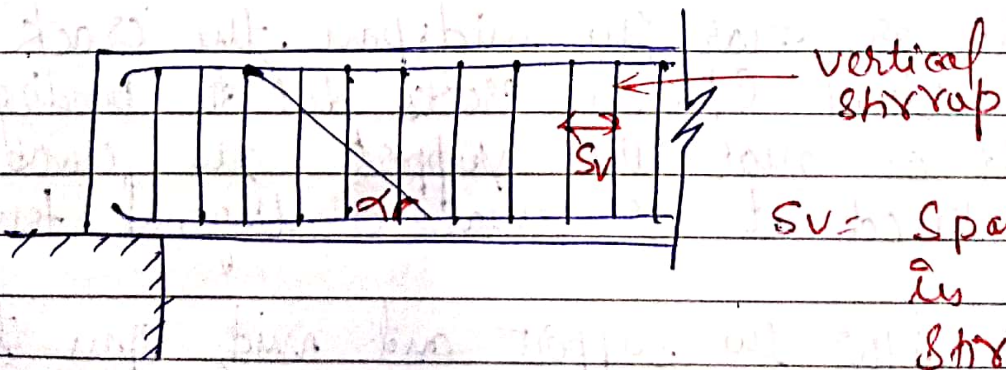
1. Vertical stirrups
2. Bent up bars along with stirrups
3. Inclined bars.

1. **Vertical stirrups** – along the length of Beam.
   Dia – 6 to 16 mm.



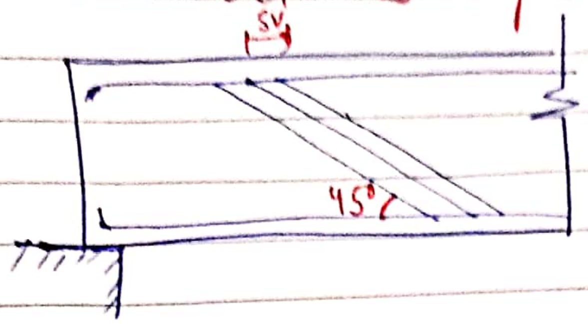
Single legged stirrups | 2 legged stirrups | 4 legged stirrups

2. **Bent up bars along with vertical stirrups**



Sv = Spacing in Stirrups

③ Inclined stirrups



Shear strength of R/f Concrete Beam

1. Grade of concrete - The shear resistance of concrete increases with the increase in the grade of concrete. Higher the Grade Higher is the shear strength.

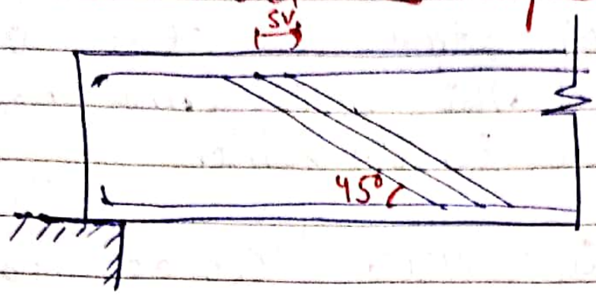
2. Percentage of tensile R/f - The shear strength of Rc beam ↑ with the % of tensile R/f.

Design shear strength of concrete

Depending upon the Grade of concrete and the percentage of tensile steel, the design shear strength in concrete (τ_c) in beams without shear R/f is given in Table 19 of (IS 456).

Teacher's Signature............

(3) Inclined stirrups



Shear strength of R/f Concrete Beam

1. Grade of concrete - The shear resistance of concrete increases with the increase in the grade of concrete. Higher the Grade Higher is the shear strength.

2. Percentage of tensile R/f - The shear strength of Rc beam ↑ with the % of tensile R/f.

Design shear strength of concrete

Depending upon the Grade of concrete and the percentage of tensile steel, the design shear strength in concrete (τ_c) in beams without shear R/f is given in Table 19 of (IS 456).

Max. Shear stress ($\tau_{c\,max}$) If the shear strength of the concrete beam is less than the nominal shear ~~strength~~ stress (τ_v) due to the loads coming on the beam then shear R/f is to be provided.

Teacher's Signature............

Notes

Page No.:
Date: / /

The nominal shear stress in the beam with shear R/f shall not exceed max. shear stress (τ_{cmax}) given in the table.

IS456

* Max. shear stress in concrete (τ_{cmax}) Table 20

Grade of concrete	M15	M20	M25	M30	M35	M40 & above
τ_{cmax} (N/mm^2)	2.5	2.8	3.1	3.5	3.7	4

* Min. shear R/f (Cl. 26.5.1.6 of IS456)

When the nominal shear stress τ_v is less than design shear strength (τ_c) then no shear R/f is to be designed. But min. shear R/f is provided in the form of stirrups

$$\boxed{\frac{A_{sv}}{b \cdot S_v} \geq \frac{0.4}{0.87 f_y}}$$

A_{sv} = total X area of stirrup leggs effective in shear

S_v = spacing of stirrup along the length

b = width of beam

f_y = yield strength of steel.

Maximum spacing of stirrups (Cl. 26.5.1.5)

The max. spacing of vertical stirrup shall not exceed 0.75d or 300mm which ever is less. In the case of inclined stirrups at

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45°, the max spacing should not be greater than d or 300mm whichever is less.

Design of Shear Reinforcement

when τ_v exceeds τ_c shear reinforcement is to be designed and can be provided in the following forms.

1. **Vertical stirrups** The spacing of the vertical stirrups is given by

$$\boxed{V_{us} = \frac{0.87 f_y \cdot A_{sv} \cdot d}{S_v}}$$

2. **Bent up bars** when bent up bars are provided their contribution towards shear resistance should not be more than half of the total shear reinforcement. It means that their contribution should not be more than $\frac{V_{us}}{2}$.

Shear force taken by bent up bars

$$V_{us}' = 0.87 f_y A_{sv} \sin\alpha$$

$$V_{us}' \ngtr \frac{V_{us}}{2}$$

α = angle b/w bent up bars and the member axis, $\alpha \geq 45^0$

Teacher's Signature............

3. <u>Combined system</u> First the contribution of bent up bars (V'_{us}) is ~~Compared~~ Computed. then vertical Stirrups are designed for remaining shear force ie. $V_{us} - V'_{us}$

Steps for design of shear R/F.

<u>Given</u>

1. loads
2. Span of beam
3. Material Concrete Grade & type of steel.
4. Area of tensile steel. (A_{st})

<u>Procedure.</u> –

1. Calculate shear force (V_u) at the critical section of the beam due to given load.

2. Determine nominal shear stress. (τ_v)

$$\tau_v = \frac{V_u}{bd} \quad \left(\text{stress} = \frac{\text{force}}{\text{Area}} \right)$$

3. Depending on the grade of concrete & % of steel $\left(\frac{100 \cdot A_{st}}{bd}\right)$ find out shear strength of concrete (τ_c) from table 19.

4. Compare τ_v & τ_{cmax} if $\tau_v > \tau_{cmax}$ then Redesign the section, τ_{cmax} — Table 20

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Notes

Page No.:
Date: / /

5. Compare τ_v & τ_c

(a) If $\tau_v < \tau_c$: Nominal shear R/f is to be provided in the form of vertical stirrup.

The spacing of vertical stirrups is given by

$$\boxed{\frac{A_{sv}}{b \cdot S_v} \geq \frac{0.4}{0.87 f_y}}$$

(b) If $\tau_v > \tau_c$ then shear R/f is to be designed as follows -

* Calculate $\boxed{V_{us} = V_u - \tau_c bd}$

* If vertical stirrups are provided then their spacing is governed by following equation.

$$\boxed{V_{us} = \frac{0.87 f_y \cdot A_{sv} \cdot d}{S_v}}$$

* If bent up bars are also used then first of all compute the shear force taken by bent up bars V'_{us} and $V'_{us} \not> \frac{V_{us}}{2}$

$$\boxed{V'_{us} = 0.87 f_y A_{sv} \cdot \sin\alpha}$$

* For the balance shear force $(V_{us} - V'_{us})$ design the vertical stirrups.

Teacher's Signature............

Notes

Page No.:
Date: / /

6. The spacing of stirrups should not exceed 0.75d or 300 mm, whichever is less.

7. The spacing of stirrups can be varied along the length of the beam by calculating the distance from the supports up to which shear reinforcement is to be designed and for rest of the length of the beam min. shear R/f may be provided.

Q. An RCC beam 300 X 600mm effective, is reinforced with 5-25mm ϕ bars. If it is subjected to a design shear force of 200KN. Comment on its shear design. Use M20 concrete & Fe 415 steel.

$b = 300mm$, $d = 600mm$

$V_u = 200KN, = 200 \times 10^3 N$

$A_{st} = 5 \times \frac{\pi}{4} \times 25^2 = 2454.3 mm^2$

$f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$

Nominal shear stress (τ_v)

$$\tau_v = \frac{V_u}{bd} = \frac{200 \times 10^3}{300 \times 600}$$

$$= 1.11 N/mm^2$$

Teacher's Signature ________

$\tau_{cmax} = 2.8 \ N/mm^2$ for M20 table 20

$\tau_v < \tau_{cmax}$ hence OK

Design shear strength of Concrete – (τ_c)

Pt. of steel = $\frac{100 A_{st}}{bd} = \frac{100 \times 2454.3}{300 \times 600} = 1.36\%$

for M20 & pt = 1.36%

$$\tau_c = 0.67 + \frac{0.72 - 0.67}{1.5 - 1.25} \times (1.36 - 1.25)$$

$\tau_c = 0.69 \ N/mm^2$

$\tau_c < \tau_v$ hence shear design is reqd.

Q An RCC beam 200 x 400mm eff. carries a UDL of 70kN/m over a clear span of 6m. The beam is reinforced with 1% steel on tension side. Comment on the shear design of beam. Use M20 & load factor 1.5.

Q A simply supported RCC beam 250 x 450mm eff. is reinforced with 4–18mm ϕ. Design the shear R/F use M20 & Fe415 Shear force = 150 kN at service state.

Given b = 250mm d = 450mm

$A_{st} = 4 \times \frac{\pi}{4} \times 18^2 = 1018 mm^2$

V = 150 kN

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Factored shear force (V_u)

$$V_u = 1.5 \times 150 = 225\ kN$$
$$= 225000\ N$$

Nominal shear stress (τ_v)

$$\tau_v = \frac{V_u}{bd} = \frac{225000}{250 \times 450} = 2\ N/mm^2$$

$$\tau_{cmax} = 2.8\ N/mm^2$$

$\tau_v < \tau_{cmax}$ hence ok

Design shear strength of concrete (τ_c)

$$P_t = \frac{100 \cdot A_{st}}{bd} = \frac{100 \times 1018}{250 \times 450} = 0.9\%$$

For $P_t = 0.97\%$ and M20 concrete from table 19.

$$\tau_c = 0.56 + \frac{0.62 - 0.56}{1 - 0.75} \times (0.9 - 0.75)$$

$$\tau_c = 0.596\ N/mm^2$$

$\tau_v > \tau_c$ hence shear R/F is to be provided.

Design of shear R/F

Notes

Page No.:
Date: / /

Shear taken by stirrups = V_{us}

$V_{us} = V_u - \tau_c b d$

$= 225000 - 0.596 \times 250 \times 450$

$\Rightarrow V_{us} = 157950\ N$

using 8mm ϕ 2 legged stirrups

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

Spacing of stirrups $S_v = \frac{0.87 f_y \cdot A_{sv} \cdot d}{V_{us}}$

$S_v = \frac{0.87 \times 415 \times 100.5 \times 450}{157950}$

$S_v = 103\ mm$

Spacing of nominal shear R/f

$S_v = \frac{0.87 f_y A_{sv}}{0.4 b} = \frac{0.87 \times 415 \times 100.5}{0.4 \times 250} = 362\ mm$

Check for spacing.

The spacing of stirrups should be min of

(i) $0.75d = 0.75 \times 450 = 337\ mm$

(ii) 300 mm

(iii) 103 mm

(iv) 362 mm

So provide 2 legged 8mm ϕ stirrups @ 100mm c/c throughout the length of the beam.

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Q1 A simply supported beam 300 X 600 mm (eff.) is reinforced with 5 bars of 25 mm dia. It carries a UDL of 80 KN/m (including its own weight) over an eff. span of 6 m. Out of the 5 main bars, two bars can be bent up safely near the support. Design the shear Reinforcement for the beam. Use M20 grade of concrete and Fe415 steel.

Soln Given $b = 300mm$ $d = 600mm$

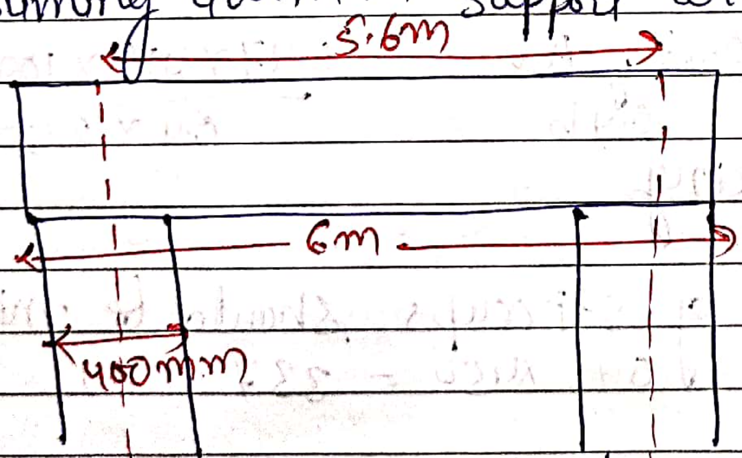
$w = 80 \; KN/m$ $f_{ck} = 20 \; N/mm^2$

$f_y = 415 \; N/mm^2$ $l_{eff} = 6m = 6000mm$

Factored shear force (V_u)

Clear span $L = 6000 - 400 = 5600mm = 5.6 m$

{Assuming 400mm support width}



factored or design loads

$W_u = W \times$ load factor

$= 120 KN/m = 120000 \; N/m$

Teacher's Signature ________

$$V_u = \frac{w_u \cdot L}{2} = \frac{120000 \times 5.6}{2} = 336000 N$$

Nominal shear stress (τ_c)

$$\tau_c = \frac{V_u}{bd} = \frac{336000}{300 \times 600} = 1.86 \ N/mm^2$$

$\tau_{cmax} = 2.8 N/mm^2$ from table M20

$\because \tau_c < \tau_{cmax}$ hence ok

Design shear strength of concrete

Area of steel available near support is 8 bars of 25 mm ϕ

$$A_{st} = 3 \times \frac{\pi}{4} \times (25)^2 = 1472.6 \ mm^2$$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 1472.6}{300 \times 600} = 0.82\%$$

for M20 concrete & $P_t = 0.82\%$

$$\tau_c = 0.56 + \frac{0.62 - 0.56}{1.00 - 0.75} \times (0.82 - 0.75)$$

$$\tau_c = 0.58 \ N/mm^2$$

$\tau_v > \tau_c$ hence shear R/F is to be desi

Shear resistance of concrete $= \tau_c bd$

Shear taken by shear R/f

$$V_{us} = V_u - \tau_c bd$$
$$= 336000 - 0.58 \times 300 \times 600$$
$$= 231600 N$$

$$\frac{V_{us}}{2} = 115800 N$$

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* Bentup bars

Shear resistance of 2 bent up bars = V_{us}'

$= 2(0.87 f_y \cdot A_{sv} \cdot \sin\alpha)$

$\Rightarrow 2\left(0.87 \times 415 \times \frac{\pi}{4} \times (25)^2 \times \sin 45°\right)$

$\Rightarrow V_{us}' = 250641 \text{ N} > \frac{V_{us}}{2}$

But $V_{us}' \ngtr \frac{V_{us}}{2}$

$\therefore V_{us}' = \frac{V_{us}}{2} = 115800\text{N}$

Balance shear force to be carried by vertical stirrup is

Assuming 8mm ϕ 2 legged stirrups

$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.54 \text{ mm}^2$

Spacing of stirrups, $S_v = \frac{0.87 f_y \cdot A_{sv} \cdot d}{V_{us}}$

$S_v = \frac{0.87 \times 415 \times 100.54 \times 600}{115800} = 188 \text{ mm}$

$S_v \simeq 180 \text{ mm}$

Page No.:
Date:

Notes

Spacing corresponding to nominal (minimum) shear R/F

$$S_v = \frac{0.87 f_y A_{sv}}{0.4 b}$$

$$\Rightarrow \frac{0.87 \times 415 \times 100.5}{0.4 \times 300} = 302 \text{ mm}$$

Maximum Spacing

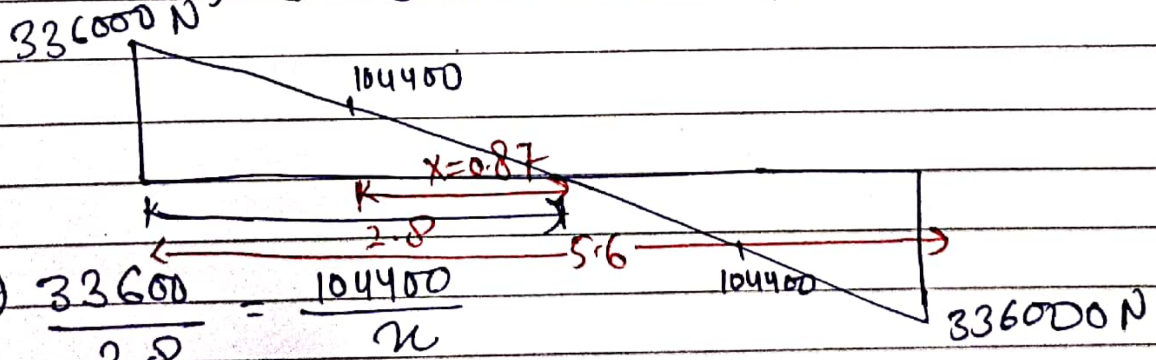
→ The max. spacing of stirrups shall be least of the following

(i) $0.75d = 0.75 \times 600 = 50$ mm
(ii) 300 mm
(iii) 180 mm
(iv) 302 mm

∴ provide 8mm 2 legged stirrups @ 180mm c/c

Zone for nominal shear R/F

Shear resistance of concrete $= \tau_c \cdot bd =$ ~~0.58~~ ×
$\Rightarrow 0.58 \times 300 \times 600 = 104400$ N



$$\Rightarrow \frac{33600}{2.8} = \frac{104400}{x}$$

$x = 0.87$ mm

provide 8mm ϕ 2 legged stirrups @ 180mm c/c upto a distance of $(2.8 - 0.87) = 1.93$ m from support and 8mm ϕ 2 legged stirrup @ 300mm c/c in the middle portion.

Teacher's Signature.......

Scanned with CamScanner

Notes

Page No.:
Date: / /

* Bond & Development length –

The term bond refers to the adhesion b/w concrete and steel which resist the slipping of steel bar from the concrete in R.C.C. The bond develops due to setting of concrete on drying which results in gripping of the steel bars.

The bond resistance in RC beam is obtained by

1) Chemical Adhesion – Due to setting and hardening of concrete.
2) Frictional Resistance &
3) Gripping action
4) Mechanical Interlock

Bond can be ↑ by

→ using Deformed or twisted bars
→ Using rich mix
→ Adequate Compaction and proper curing
→ Providing Hooks at ends of bars.

Anchorage Bond (Development length)

To avoid Slipping

$$\boxed{L_d \geq \frac{0.87 f_y \phi}{4 \tau_{bd}}}$$

Teacher's Signature

Page No.:
Date: / /

Notes

L_d = Development length
τ_{bd} = Design bond strength of bar
ϕ = dia of bar

τ_{bd} depends upon grade of concrete & type of steel (Cl. 26.2.1.1)

As per IS Code 456 : 2000

$$\boxed{L_d \leq \frac{M_1}{V} + l_o}$$

It shows requirement

availability

l_o = additional length provided for Safety (Anchorage Length)

M_1 = Moment of Resistance
V = Factored shear force

* Anchorage value of standard U-type hook is equal 16 times the dia. of the bar.

* The anchorage value of a standard 90° bend is 8 times the dia.

(Cl 26.2.2 of IS 456)

45° — 4ϕ

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Development length - length of steel bar needed to be embedded in to the column to establish the desired bond strength b/w concrete & steel.

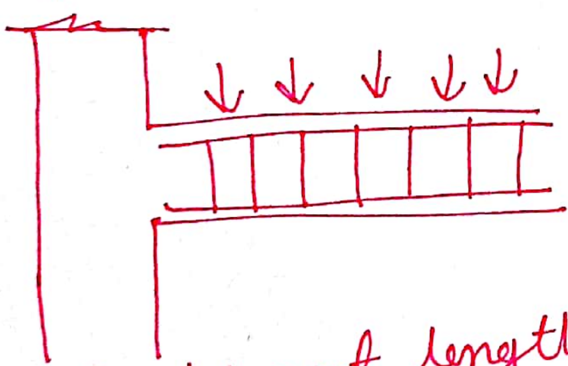
→ It holds 2 concrete member together - Beam, Column, footing etc.

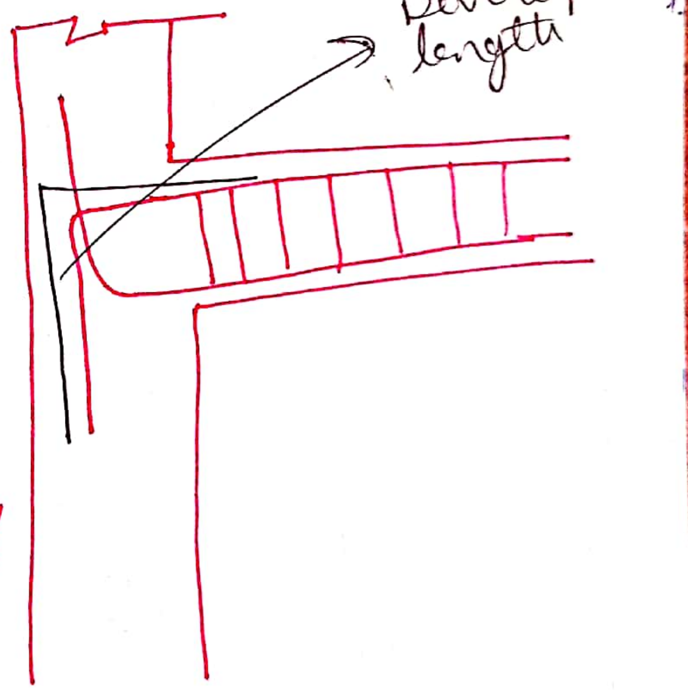
→ To safely transfer stresses from beam to column

→ Development length create a safe bond b/w bar surface & concrete.

→ It also ensures that in ultimate load condition R/f should not slip out through the concrete.

→ If we do not provide adequate development length then the structure will fail due to slippage of joints





* Development length is the min. length of R/f is the min. length required to be embedded inside concrete so that bond strength of concrete is not less than tensile strength of steel.

To ensure Strength in Bond $\geq$ Strength in tension

Q. An RCC beam 250mm x 500mm has a clear span of 5.5m. The beam has 2-20mm ϕ bars going into the support. Factored shear force is 140kN. Check for development length if Fe415 and M20 Grade of concrete is used.

Sol^n

$b = 250$ mm

$D = 500$ mm

$d = 500 - 20 - 10 = 470$ mm {assuming 20 mm clear cover

$l = 5.5$ m.

A_{st} at support $= 2 \times \frac{\pi}{4} \times 20 = 628$ mm

$V = 140$ kN $= 140 \times 10^3$ N

M20 concrete, Fe415 steel.

Moment of Resistance (M_1) at the section

$$M_1 = 0.87 f_y A_{st} . d \left[1 - \frac{A_{st} . f_y}{b d f_{ck}}\right]$$

$$= 0.87 \times 415 \times 628 \times 470 \left[1 - \frac{628 \times 415}{250 \times 470 \times 20}\right]$$

$$M_1 = 94.748 \times 10^6 \text{ Nmm}$$

Page No.:
Date: / /

Notes

Check for development length

$$\frac{M_1}{V} + l_0 \geq L_d$$

$$\Rightarrow L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}}$$

$\tau_{bd} = 1.6 \times 1.2 = 1.92\ N/mm^2$ for M20 Concrete Cl.26.2.1.1

$$L_d = \frac{0.87 \times 415 \times 20}{4 \times 1.92} = 940.2\ mm$$

* Providing a 90° bend at the centre of support

Anchorage length $l_0 = 8\phi = 8 \times 20 = 160 mm$

$$\frac{M_1}{V} + l_0 = \frac{94.748 \times 10^6}{140 \times 10^3} + 160$$

$$= 836.77 < L_d$$

Hence codal Requirements are not satisfied so there is a need to increase anchorage length.

* Providing a U bend at the end of the bar

Anchorage length $= 16\phi = 16 \times 20 = 320 mm$

$$\frac{M_1}{V} + l_0 = \frac{94.748 \times 10^6}{140 \times 10^3} + 320 =$$

$$= 997 mm > L_d$$

hence codal Requirements are satisfied.

Teacher's Signature............

Design of Singly R/f Beam

I. Determining area of tensile steel if b, d & Mu are given

1.) Determine $\frac{x_{umax}}{d}$ for given steel from (Cl 38.1)

2) Determine the limiting moment of resistance

$$M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - \frac{0.42 x_{umax}}{d}\right) bd^2$$

3. Compare Mu & Mulim

(i) $M_u = M_{ulim}$ the section is balanced and the area of steel can be found as

$$A_{st} = \frac{M_u}{0.87 f_y (d - 0.42 x_{umax})}$$

$$A_{st} = \frac{0.36 f_{ck}\, b\, x_{umax}}{0.87 f_y}$$

(ii) If $M_u < M_{ulim}$ the section is under R/f and the area of steel is calculated from following eqn-

$$M_u = 0.87 f_y A_{st} \cdot d\left(1 - \frac{A_{st} f_y}{bd f_{ck}}\right)$$

(iii) $M_u > M_{ulim}$ the section is over Reinforced and hence should be redesigned,

Teacher's Signature ________

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Notes

Page No.:
Date : / /

or desigen a doubly reinforced section.

II Determining Dimensions of the beam and area of tensile Reinforcement

1. Determine design bending moment (If Mu is not given)

(i) assume suitable value of depth and width
Depth of the beam can be assumed as $\frac{l}{12}$ to $\frac{l}{15}$ and width may be taken as $\frac{1}{2}$ to $\frac{1}{3}$ of de

(ii) Calculate Self wt of the beam and determine total load (W) by adding imposed load to self wt.

(iii) Determine design or factored load. (Wu)
$\Rightarrow$ $W_u = W \times 1.5$ { 1.5 is partial safety factor for load and is taken as 1.5

iv) Calculate the factored moment (M_u)

for Simply supported beam $M_u = \frac{W_u l^2}{8}$

for Cantilever beam $M_u = \frac{W_u l^2}{2}$

2. Determine $\frac{x_{umax}}{d}$ from IS code & R_u.

$$R_u = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right)$$

Teacher's Signature............

Notes

Page No.:
Date: / /

3. Determine the min. depth reqd.

$$d_{reqd} = \sqrt{\frac{M_u}{R_u b}}$$

4. Compare dreqd. with assumed value of effective depth.

(i) If assumed $d > d_{reqd}$, then our assumption is correct and provide the assumed depth and calculate overall depth.

ii) If assumed $d < d_{reqd}$, then redesign the section.

5. Determine the area of steel required.

The effective depth provided is greater than the depth required for a balanced section, so we are designing an under reinforced section. The area of steel corresponding to under reinforced section is calculated from the following equation.

$$M_u = 0.87 f_y \cdot A_{st} \cdot d \left(1 - \frac{A_{st} \cdot f_y}{b d \cdot f_{ck}}\right)$$

Teacher's Signature ____________

Scanned with CamScanner

Page No.:
Date: / /

Notes

This area of steel (Ast) should be more than the min. area of steel (As) specified by code.

$$A_s = \frac{0.85\,bd}{f_y}$$

Choose suitable dia (ϕ) of steel bar,

$$A\phi = \frac{\pi}{4} \times \phi^2$$

No. of bars required $= \frac{Ast}{A\phi}$

If instead of an under Rfd section, a balanced section is designed i.e. d= dreqd. then the area of steel can be estimated directly from the following eqn.

$$Ast = \frac{M_u}{0.87\,f_y\,(d - 0.42\,x_{u\,max})}$$

6. Check for deflection

(i) Calculate service stress (f_s) and P_t (% of steel)

$$f_s = 0.58\,f_y \left[\frac{Ast\ reqd}{Ast\ provided}\right] \quad P_t = \frac{100\,Ast}{bd}$$

(ii) Find out modification factor (k_t) & f_s and P_t values from Code.

Teacher's Signature............

Notes

Page No.:
Date: / /

(iii) $\left(\frac{l}{d}\right)_{max} = 20 \times k_t$ {for Simply supported beam.

$\left(\frac{l}{d}\right)_{max} = 7 \times k_t$ {for Contilever beam

if $\left(\frac{l}{d}\right)_{max} > \left(\frac{l}{d}\right)_{provided}$, hence ok

if $\left(\frac{l}{d}\right)_{max} < \left(\frac{l}{d}\right)_{provided}$ they redesign.

7. Design for shear.

8. Check for development length

9. Give design summary and sketch for showing RF.

Teacher's Signature ____________

Notes

Page No.:
Date : / /

Qu. A Reinforced concrete beam is 300 X 700 mm is subjected to a bending moment of 150 kNm. Determine the area of Reinforcement if M20 concrete and Fe415 steel is used. Take effective cover as 40 mm.

soln

Given $b = 300mm$ $\quad d = 700 - 40 = 660mm$

$M = 150 kNm$ $\quad f_y = 415 N/mm^2$

$f_{ck} = 20 N/mm^2$

Design bending moment :

$$\frac{x_{umax}}{d} = 0.48 \quad \text{from IS 456}$$

Limiting moment of resistance (Mulim)

$$M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right) bd^2$$

$$\Rightarrow 0.36 \times 20 \times 0.48 \left(1 - 0.42 \times 0.48\right) 300 \times 660^2$$

$$\Rightarrow M_{ulim} = 360.58 \times 10^6 \ Nmm$$

$$M_{ulim} = 360.58 \ kNm$$

Design moment

$$M_u = M \times V_F$$

$$150 \times 1.5$$

$$M_u = 225 \ kNm = 225 \times 10^6 \ Nmm$$

Teacher's Signature.............

Notes

Page No.:
Date : / /

Mulim > Mu, the Section is under R/f

Area of steel (Ast)

$$M_u = 0.87 f_y . A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b d}\right)$$

$$225 \times 10^6 = 0.87 \times 415 \times A_{st} \times 660 \left(1 - \frac{415 \times A_{st}}{20 \times 300 \times 660}\right)$$

~~$A_{st}^2 +$~~

$$A_{st}^2 - 9542 A_{st} + 9009865.8 = 0$$

$$A_{st} = 1062.45 \ mm^2$$

Q.2 A Singly reinforced R.C.C beam is subjected to a moment of 80 kNm. The width of the beam is 200mm. Calculate the depth of beam and area of steel R/f required for balance design. Use M20 Concrete and Fe 415 Steel.

Soln Given $b = 200mm$ $M = 80kNm$

$f_{ck} = 20 N/mm^2$ $f_y = 415 N/mm^2$

$$\frac{x_{umax}}{d} = 0.48 \quad \text{from IS code}$$

Design moment – $M_u = M \times V_f = 80 \times 1.5$

$M_u = 120 \ kNm = 120 \times 10^6 \ Nmm$

For Balanced Design – $M_u = M_{ulim}$

$$M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d} \left(1 - \frac{0.42 x_{umax}}{d}\right) b d^2$$

Teacher's Signature

Page No :
Date : / /

Notes

$$120 \times 10^6 = 0.36 \times 20 \times 0.48 \left(1 - 0.42 \times 0.48\right) 200 d^2$$

$d = 466.3 mm$ say $470 mm$

$\therefore$ Depth required for balanced section $= 470 mm$

$x_{umax} = 0.48 d = 0.48 \times 470 = 225.6 mm$

Area of steel (Ast)

Designing it as balanced section and calculating area of steel required for balanced design

$$A_{st} = \frac{M_u}{0.87 f_y (d - 0.42 x_{umax})} = \frac{120 \times 10^6}{0.87 \times 415 (470 - 0.42 \times 225}$$

$A_{st} = 885.7 mm^2$

Ques An RCC Beam is required to resist a bending moment of 70 kNm. Design the beam for flexure, taking $\frac{b}{d} = 0.5$. Use M20 & Fe415 bars.

Given
$M = 70 kNm$
$f_{ck} = 20 N/mm^2$
$f_y = 415 N/mm^2$
$\frac{b}{d} = 0.5$

Design or factored moment

$M_u = M \times \gamma_f = 70 \times 1.5$
$= 105 kNm = 105 \times 10^6 Nmm$

Teacher's Signature............

Effective depth required

$$R_u = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42 \frac{x_{umax}}{d}\right)$$

$$\frac{x_{umax}}{d} = 0.48 \quad \text{from IS 456}$$

$$R_u = 0.36 \times 20 \times 0.48 \,(1 - 0.42 \times 0.48)$$

$$R_u = 2.76$$

$$M_u = R_u\, b d^2$$

$$\Rightarrow 105 \times 10^6 = 2.76\,(0.5)\, d^3$$

$$d^3 = 76086956.52$$

$$d = 423.7 \text{ mm} \quad \text{say } 425 \text{ mm}$$

$$b = \frac{425}{2} = 212.5 \quad \text{say } 215 \text{ mm}$$

Area of steel =

The balanced steel required for this section is calculated as

$$A_{st} = \frac{M_u}{0.87 f_y (d - 0.42 x_{umax})} \qquad \left(x_{umax} = 0.48 d \Rightarrow 0.48 \times 425 = 204 \text{ mm}\right)$$

$$A_{st} = \frac{105 \times 10^6}{0.87 \times 415\,(425 - 0.42 \times 204)}$$

Notes

Page No.: Date: / /

$A_{st} = 857.1 \, mm^2$

Area of one 20 mm ϕ bar $A\phi = \frac{\pi}{4} \times (20)^2 = 314 \, mm^2$

Number of bars required $= \frac{857.1}{314} = 2.72$ say 3

∴ Provide ~~8Ф~~ 3-20mm ϕ bars.

Assuming eff. cover = 45 mm

Overall depth = 425+45 = 470 mm

Beam dimension = 215 mm × 470 mm.

Q. An RCC beam is required to carry a UDL of 25 kN/m inclusive of its self wt. The effective span of the beam is 8 m. Design the beam for flexure only. Use M30 & Fe 415 steel.

Q. A rectangular RCC beam is simply supported on 2 masonary walls of 230 mm thick and 6 m apart c/c. The beam is carrying an imposed load of 15 kN/m. Design the beam with all necessary check for flexure & shear. Use M25 & Fe415.

Soln Given $l = 6m$

Imposed load = 15 kN/m

$f_{ck} = 25 \, N/mm^2$

$f_y = 415 \, N/mm^2$

Assuming a trial depth of 500mm $\left(\frac{l}{12} = \frac{6000}{12} = 500mm \right.$

and $b = 250$

Teacher's Signature............

Notes

Page No.:
Date: / /

effective depth $d = 500 - 50 = 450$ mm
($\because$ assuming effective cover = 50mm)

* effective span (l) The effective span is least of following.

(i) Centre to Centre of supports = 6m

(ii) Clear span + d = 5.77 + 0.45 (Clear span = 6 − 0.23 = 5.77m)
= 6.22m

$l = 6$ m

* Design load (W_u) and factored moment (M_u)

(i) Self wt of beam = $0.5 \times 0.25 \times 25$ (RCC unit wt = 25 kN/m³)
= 3.125 kN/m

Imposed load = 15 kN/m

Total load, w = 18.125 kN/m

Design load = $w \times 1.5$
= $18.125 \times 1.5 = 27.2$ kN/m

$$M_u = \frac{w_u l^2}{8} = \frac{27.2 \times 6^2}{8} = 122.3 \text{ kNm}$$

$\Rightarrow 122.3 \times 10^6$ Nmm

* Min effective depth required −

$$\frac{x_{umax}}{d} = 0.48$$

$$R_u = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right)$$

$$\Rightarrow 0.36 \times 25 \times 0.48 (1 - 0.42 \times 0.48)$$

$R_u = 3.45$

Teacher's Signature

Notes

Page No :
Date : / /

$$d_{reqd} = \sqrt{\frac{Mu}{Ru \cdot b}}$$

$$d_{reqd} = \sqrt{\frac{172.3 \times 10^6}{3.45 \times 250}} = 376\,mm < 450\,mm, \text{hence ok}$$

since the depth of section is more than that required for balanced section. The section is designed as an under reinforced section.

adopt $D = 500\,mm$ and $b = 250\,mm$

$$d = 500 - 20 - 8 - \frac{20}{2} = 462\ mm$$

(assuming clear cover as 20mm, 8mm as dia of stirrup and 20 mm dia main bars.)

* Area of steel required.

For an under reinforced section the area of steel required is calculated as follows.

$$Mu = 0.87\, fy \cdot Ast \cdot d \left(1 - \frac{fy \cdot Ast}{bd\, fck}\right)$$

$$122.3 \times 10^6 = 0.87 \times 415\, Ast \times 462 \left(1 - \frac{415\, Ast}{250 \times 462 \times 25}\right)$$

$$122.3 \times 10^6 = 166805.1\, Ast - 23.97\, Ast$$

$$Ast^2 - 6958.9\, Ast + 5102211.1 = 0$$

Teacher's Signature............

Notes

Page No.: Date: / /

$$A_{st} = \frac{6958 \pm \sqrt{(6958.9)^2 - 4 \times 5102211.1}}{2}$$

A_{st} reqd = $833\ mm^2$

Minimum area of steel (A_{st})

$$A_{s} = \frac{0.85bd}{f_y} = \frac{0.85 \times 250 \times 462}{415}$$

$A_{s} = 236\ mm^2 < 833\ mm^2$, hence ok

Area of one 20mm dia bars

$$A\phi = \frac{\pi}{4} \times (20)^2 = 314\ mm^2$$

Number of bars reqd $= \frac{A_{st}}{A\phi} = \frac{833}{314} = 2.65$ say $= 3$

∴ Provide 3-20 mm dia bars

A_{st} provided $= 3 \times 314 = 942\ mm^2$

Check for deflection

$$P_t = \frac{100 \times 942}{250 \times 462} = 0.8\%$$

$$f_s = 0.58 f_y \left[\frac{A_{st}\ reqd}{A_{st}p.}\right] = 0.58 \times 415 \left[\frac{833}{942}\right]$$

$= 212\ N/mm^2$

Teacher's Signature

Page No :
Date : / /

Notes

Interpolating for $f_s = 212\, N/mm^2$ & $P_t = 0.8\%$

$$K_t = 1.35 - \frac{(1.35 - 1.2)}{240 - 190} \times (212 - 190)$$

$\because$ (for $f_s = 190$, $K_t = 1.35$, $f_s = 240$, $K_t = 1.2$)

$$K_t = 1.29$$

$$\left(\frac{l}{d}\right)_{max} = 20 \times 1.29 = 25.8$$

$$\left(\frac{l}{d}\right)_{provided} = \frac{6000}{462} = 12.9$$

$$\left(\frac{l}{d}\right)_{max} > \left(\frac{l}{d}\right)_{provided}$$, hence ok

* Design for shear

$$V_u = \frac{W_u \cdot L}{2} = \frac{27.2 \times 5.77}{2}$$ { L = Clear span = 5.77m }

$\Rightarrow 78.5\, kN = 78500\, N$

* Nominal shear stress

$$\tau_v = \frac{V_u}{bd} = \frac{78500}{250 \times 462}$$

$$\tau_v = 0.68\, N/mm^2$$

Design shear strength of Concrete (τ_c)

Teacher's Signature............

Page No :
Date : / /

Notes

for $P_t = 0.8\%$ & M25 Concrete

$$\tau_c = 0.57 + \left(\frac{0.64 - 0.57}{1 - 0.75}\right) \times (0.8 - 0.75)$$

$$\Rightarrow 0.58 \, N/mm^2 < \tau_{c\,max} \; (\tau_{c\,max} = 3.1 \, N/mm^2)$$

$\therefore$ $\tau_v > \tau_c$ hence shear R/f is required.

shear to be carried by reinf. = V_{us}

$$V_{us} = V_u - \tau_c b d = 78500 - 0.58 \times 250 \times 462$$

$$V_{us} = 11510 \, N$$

using 8mm ϕ 2legged stirrups

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.53 \, mm^2$$

spacing of stirrups $S_v = \dfrac{0.87 f_y A_{sv} \cdot d}{V_{us}}$

$$S_v = \frac{0.87 \times 415 \times 100.53 \times 462}{11510}$$

$$S_v = 1456 \, mm.$$

Maximum spacing as per minimum R/f

$$S_v = \frac{0.87 A_{sv} \cdot f_y}{0.4 b}$$

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Notes

Page No.:
Date : / /

$\Rightarrow \dfrac{0.87 \times 100.53 \times 415}{0.4 \times 250} = 362 \text{ mm}$

The spacing should be least of the following also.

(i) $0.75d = 0.75 \times 462 = 346$ mm

(ii) 300 mm

$\therefore$ Spacing $S_v = 300$ mm

provide 8 mm ϕ 2 legged @ 300 mm c/c throughout the length of the beam

Provide 2-10 mm ϕ anchor bars in the compressio zone.

Check for development length

$$M_1 = 0.87 f_y \cdot A_{st} \cdot d \left(1 - \frac{A_{st} \cdot f_y}{b d f_{ck}}\right)$$

$$\Rightarrow 0.87 \times 415 \times 942 \times 462 \left(1 - \frac{942 \times 415}{250 \times 462 \times 25}\right)$$

$M_1 = 135856988$ N

$V_u = 78500$ N

Using no bond, $l_0 = 0$

$$\frac{M_1}{V_u} + l_0 = \frac{135856988}{78500} + 0 = 1730 \text{ mm}$$

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$\Rightarrow L_d = \frac{\phi\, 0.87 f_y}{4 \tau_{bd}} = \frac{20 \times 0.87 \times 415}{4 \times 2.24} = 805.9 mm$

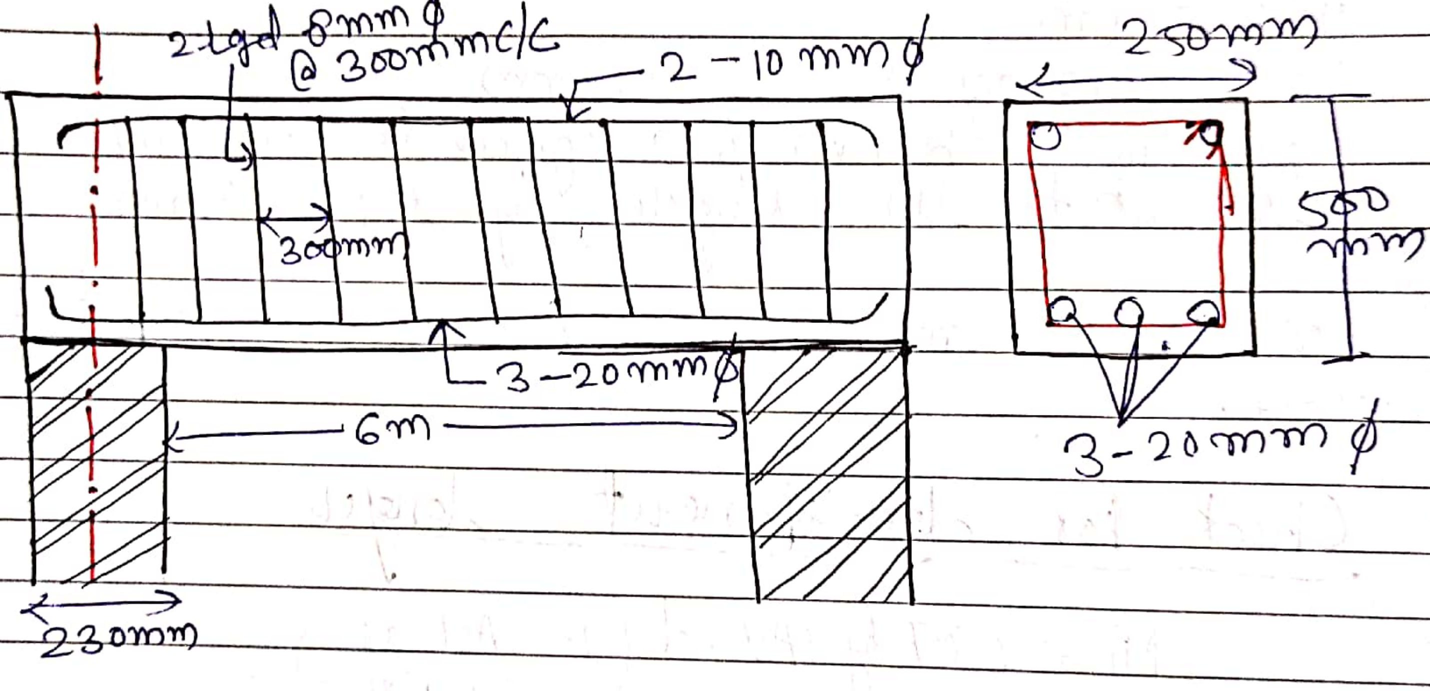
$\therefore \frac{M_1}{V} + L_0 > L_d$ hence codal requirements are satisfied.

Design Summary Beam size = 250mm X 500mm

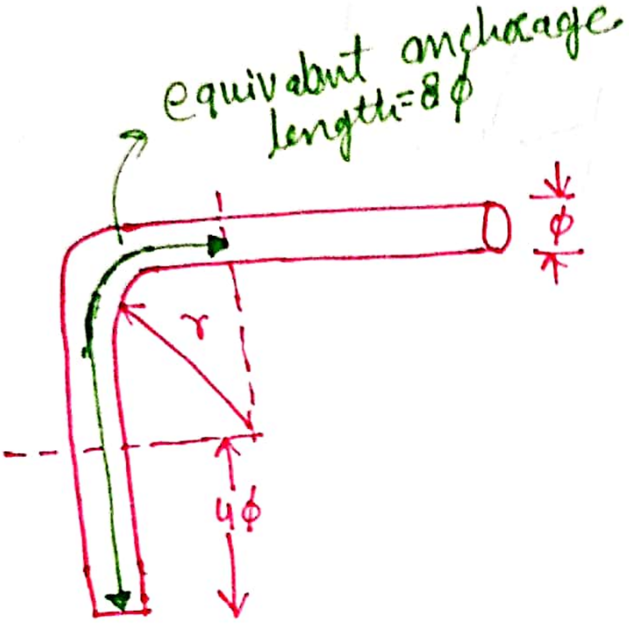
Main tensile bars = 3-20mm ϕ bars of Fe 415 steel

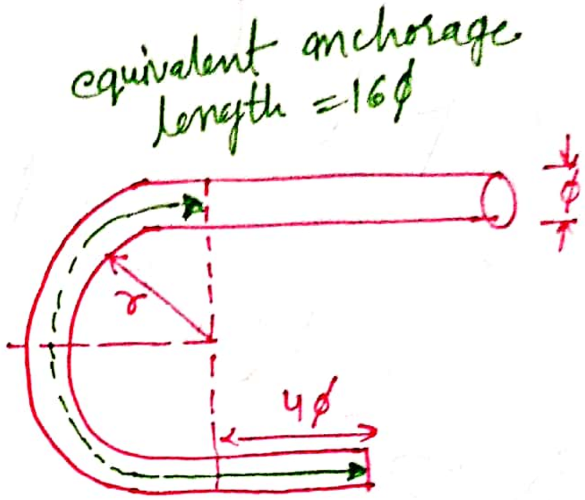
Shear stirrups = 8mm ϕ 2 lgd @ 300mm C/C

Anchor bars = 2 - 10 mm ϕ



Bends & hooks IS 456 (cl 26.2.2.1)





Q M20, Fe415 calculate development length in terms of diameter

Case (i) Bars in tension

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}}$$

$f_y = 415$

$\tau_{bd} = 1.2 \times 1.6$

$\Rightarrow L_d = 47.01\phi$

Case (ii) Bars in compression

$$L_d = \frac{0.87 f_y \phi}{\tau_{bd}}$$

$f_y = 415$

$\tau_{bd} = 1.2 \times 1.6 \times 1.25$

$L_d = 37.60\phi$