TH:1, STRUCTURAL DESIGN-I

→ Working stress Method (WSM)

**Q-1** What is the difference between PCC & RCC ?

page 1

| PCC | RCC |
| --- | --- |
| → PCC stands for plain Cement concrete. | → RCC stands for Reinforced cement concrete. |
| → As by their name it is clear that one which doesnot have steel, it is plain. | → It has steel which is Reinforced. |
| → PCC is weak in Tension loading while strong in compression loading | → RCC is strong in tension as well as compression loading also. |

**Q-2** What are the methods of design of concrete structures ?

**Ans:** There are 3 methods of design:

1. Working stress method.
2. Limit state method.
3. Ultimate load method.

Q-3 Difference between WSM & LSM ?

| WSM | LSM |
|---|---|
| → Working stress method is the old way of designing | → It is used now a days. |
| → The stresses in an element is obtained from the working loads & compared with permissible stresses. | → The stresses obtained from design loads & compared with design strength. |
| → Factor of safety are used in this method. | → partial safety factor are used in this method. |
| → The stress-strain behaviour is linear | → strain behaviour is linear but stress behaviour is not linear |
| → ultimate load carring capacity can't be predicted. | → ultimate stress is allowable stress. |
| → Factor of safety in concrete = 3<br>in steel = 1.8 | → partial safety factor in concred is 1.5 & for steel is 1.15 |

Q-4 Why partial safety factor for concrete is more than the steel in LSM?

Ans: In LSM partial safety factor for concrete is 1.5 & steel is 1.15

→ P.S.F of concrete is more than steel because concrete is cast in working site, proper ~~proof~~ casting may not be made because number of factor are affecting (temperature variation, water quality not good, improper mixing, etc) but steels are casting in factories only. So safety factor for concrete is more than the steel in LSM.

Q-5 What is modular Ratio?

Ans: Modular ratio is the ratio of Modulus of elasticity of stronger material to the modulus of elasticity of weaker material.

→ Denoted as $m = \dfrac{E_1 \leftarrow \text{stronger}}{E_2 \leftarrow \text{weaker}}$

→ calculated as $m = \dfrac{280}{3\sigma_{cbc}}$.

$\sigma_{cbc}$ = characteristic bending stress in concrete. in $N/mm^2$

[CHAPTER : 02]

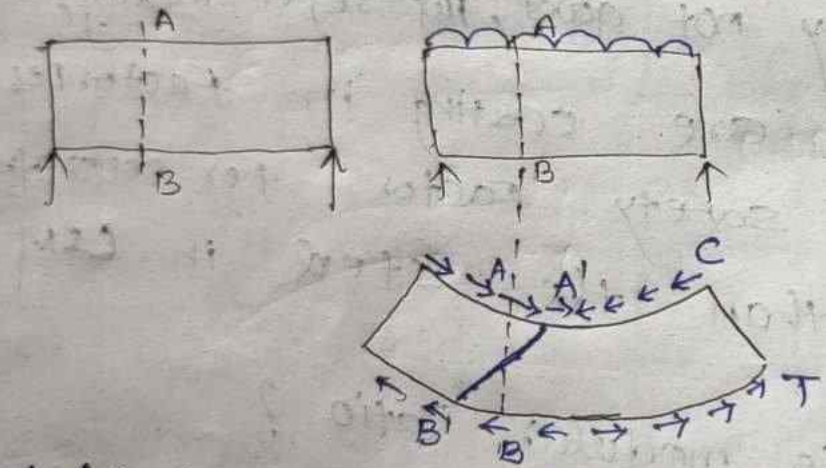
Q-6 What are the assumptions in LSM ?

Ans: Limit state method of collapse:

Flexure

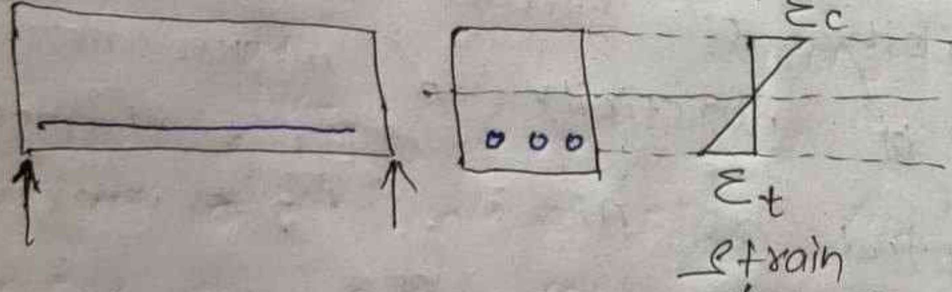
Assumptions:

① The plane Section remains plane before and after the bending.




→ AB is plane secⁿ

→ A'B' is inclined but plane Section.



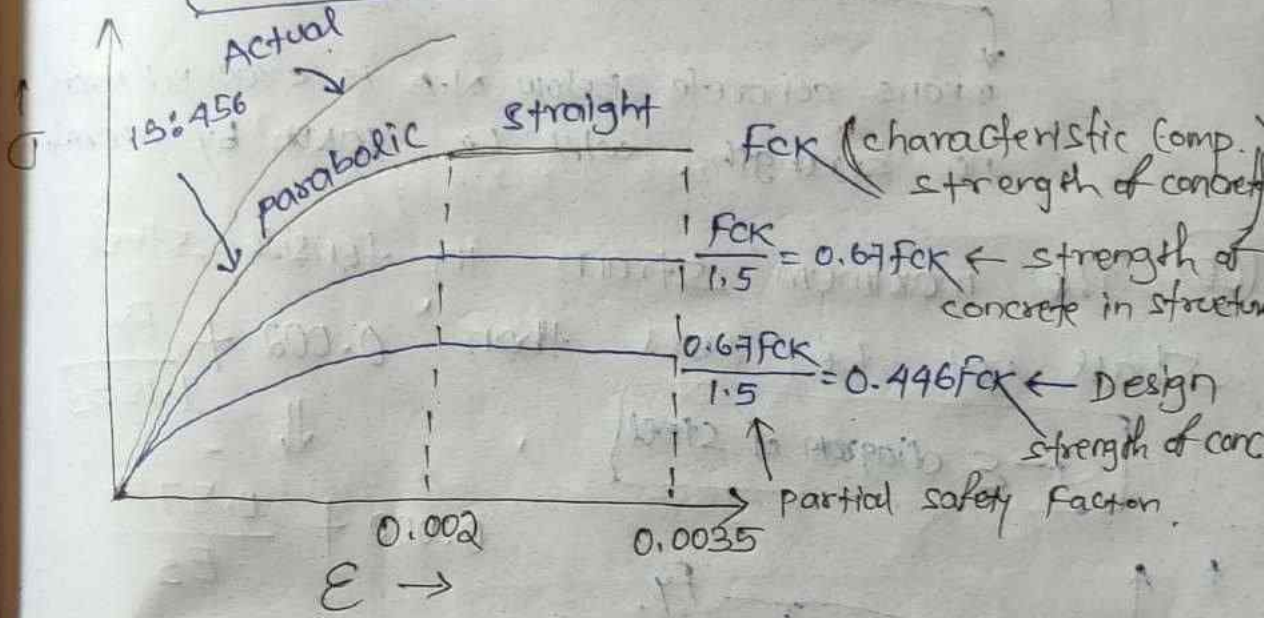

strain diagram.

∴ Assumption-1 means the strain variation is linear.

(2) The maximum compressive strain in the concrete shall be taken as 0.0035 under Flexural condition.

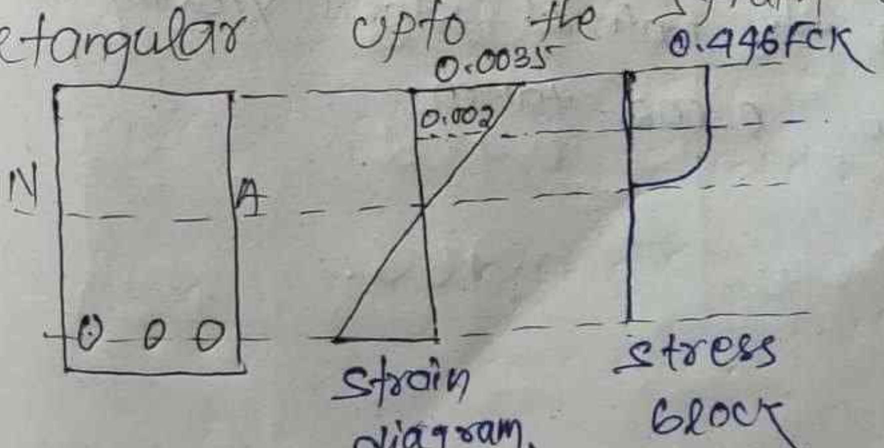
→ compression in the form of strain as 0.0035

$\boxed{\sigma - \varepsilon \text{ diagram of concrete}}$




→ we have taken 0.0035, because after this value permanent deformation occurs means no of cracks appers, so the maximum value for concrete taken.

(03) The stress block is parabolic from Neutral axis to the strain 0.002 & rectangular upto the strain of 0.0035.



(04) The tensile strength of concrete shall be ignored.

→ Tensile strength of concrete below N.A is ignored and will be taken by steel, by cracked section theory

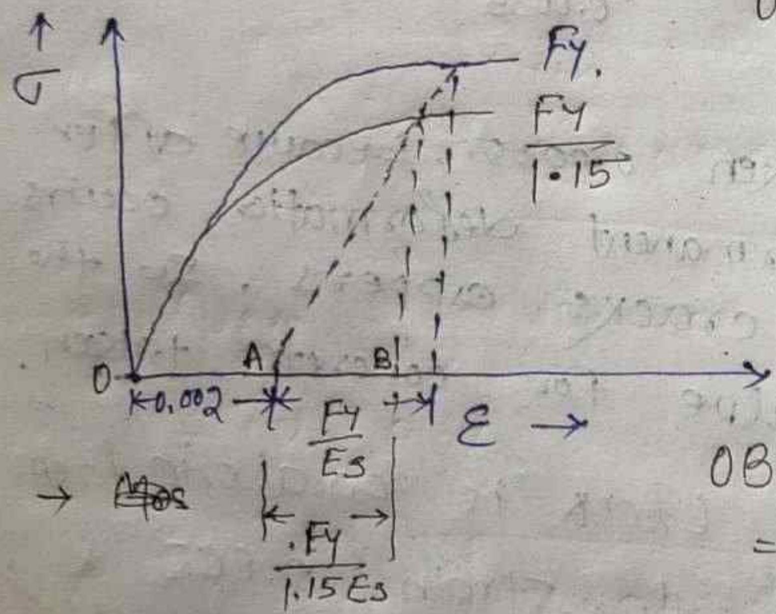
↓ means concrete below N.A is cracked means no strength will be taken by concrete.

(05) The maximum strain in tensile steel shall not be less than $0.002 + \frac{F_y}{1.15 E_s}$

$\Downarrow$

$0.002 + \frac{0.87 F_y}{E_s}$

$\boxed{\sigma - \varepsilon \text{ diagram of steel}}$

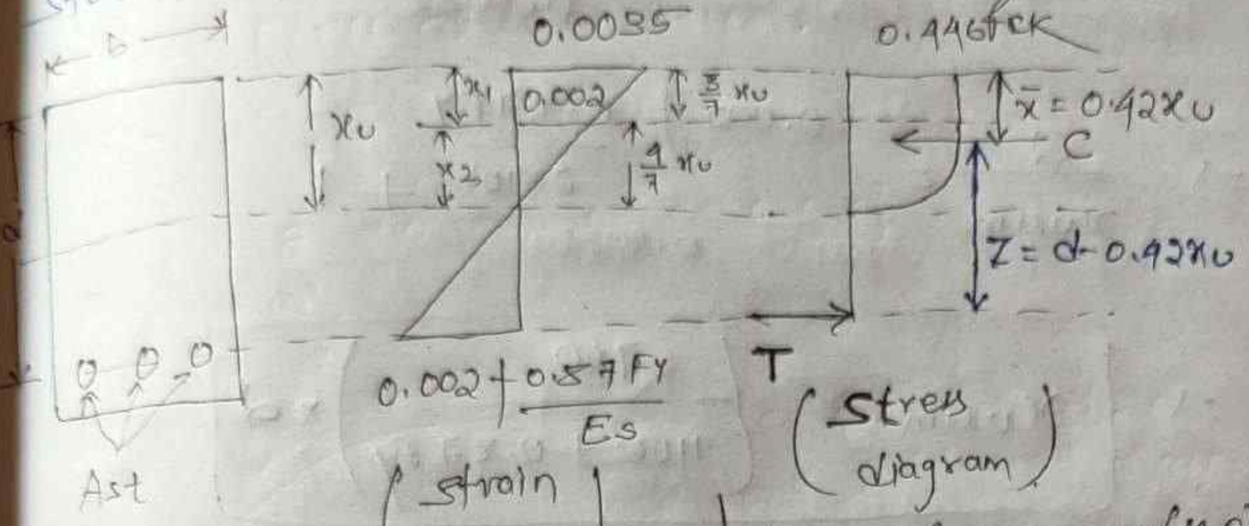



$OB = OA + AB$

$= 0.002 + \frac{0.87 F_y}{E_s}$

→ Most of the material starts yielding after 0.2% so 0.002 taken.

Q-7 Draw stress-strain block for eq. structural section in LSM ?




$T = 0.87 f_y A_{st}$.

$C = 0.36 f_{ck} b . x_u$

Moment of Resistance (M.R)

$= T \times Z$ or $C \times Z$

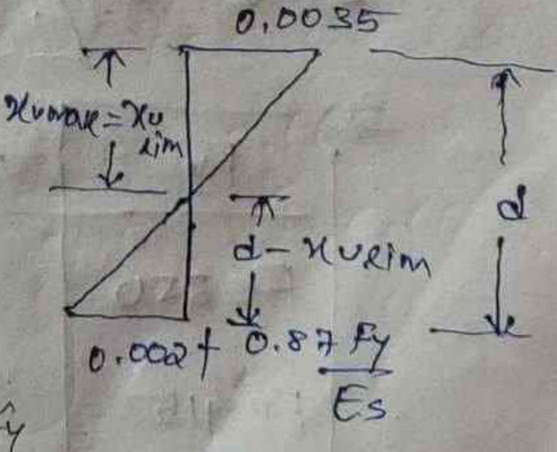
$= 0.87 f_y A_{st} \times (d - 0.42x_u)$

Q-8 What is the limiting depth of Neutral axis?

Ans: → Also called maximum depth of neutral axis.

→ It is the maximum depth of N.A. from top fibre to the N.A. where Bending moment is zero and having no strain or deformation.

→ From fig, By similar triangle,




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

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

$$\Rightarrow \frac{d}{x_{ulim}} - 1 = \frac{0.002 + \frac{0.87 f_y}{2\times10^5}}{0.0035}$$

$$\Rightarrow \frac{d}{x_{ulim}} = \frac{0.002 + \frac{0.87 f_y}{2\times10^5}}{0.0035} + 1$$

$$\Rightarrow \boxed{x_{ulim} = \left(\frac{700}{1100 + 0.87 f_y}\right) \times d}$$

$$\Rightarrow x_{ulim} = K \cdot d.$$

where $K$ = Neutral axis constant.

$$\boxed{K = \frac{700}{1100 + 0.87 f_y}}$$

$\therefore$ $x_{ulim}$ depends upon '$f_y$' or 'grade of steel' only.

So

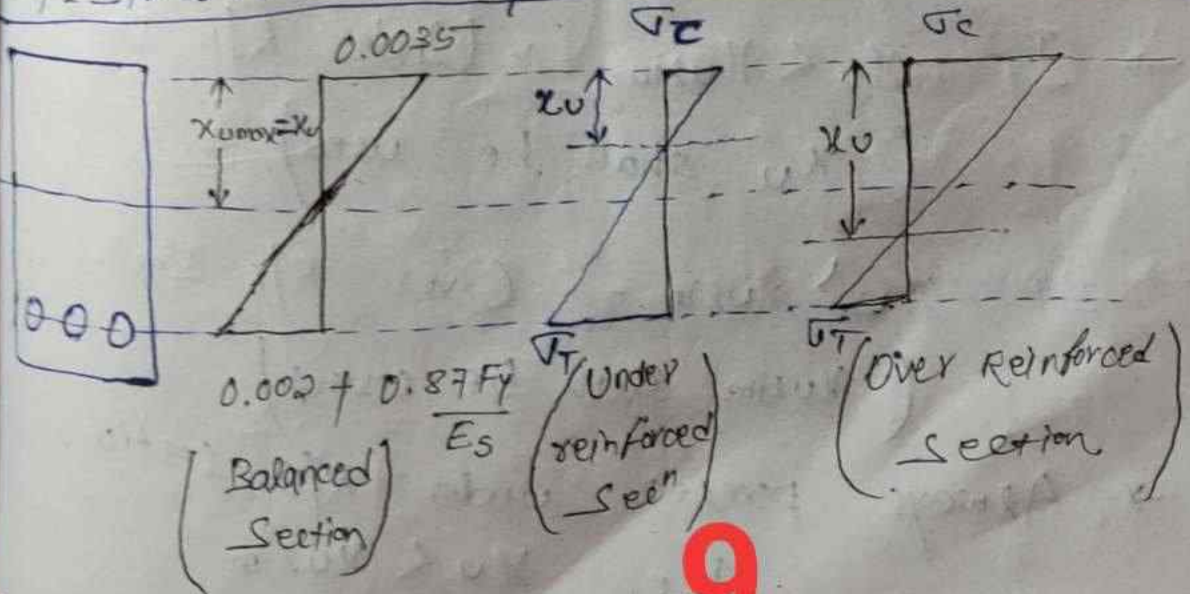
| Fy | $x_{umax} = x_{ulim} = K \cdot d$ |
|---|---|
| Fe 250 | 0.53d |
| Fe 415 | 0.48d |
| Fe 500 | 0.46d. |

Q-9 What are the type of Sections ?

Ans:

| Balanced Sec$^n$ | Under Reinforced Sec$^n$ | Over Reinforced Sec$^n$ |
|---|---|---|
| → $\sigma_c > \sigma_{c\,per}$<br>→ $\sigma_T > \sigma_{T\,per}$<br>→ $x_u = x_{umax}$<br>→ steel & concrete provided as required.<br>→ so Both will reach to it's permissible limit simultaneously. | → $\sigma_T \geq \sigma_{T\,per.}$<br>→ $\sigma_c < \sigma_{c\,per.}$<br>→ $x_u < x_{umax}$ or $x_{ulim.}$<br>→ steel is provided less than required or limited amount of steel.<br>→ steel will reach to it's permissible limit First so steel will fail First. So failure is ductile<br>→ we get alarm before failure | → $\sigma_c \geq \sigma_{c\,per.}$<br>→ $\sigma_T < \sigma_{T\,per.}$<br>→ $x_u > x_{umax}$ or $x_{ulim.}$<br>→ steel is provided more than required<br>→ so concrete quantity will reduce so concrete will reach to it's permissible limit First. so concrete will Fail First. So failure is brittle failure.<br>→ No alarm. |

| | |
|---|---|
| $\sigma_c$ = stress in concrete.<br>$\sigma_T$ = stress in steel. | $\sigma_{c\,per}$ = permissible stress in concrete |



## Expected problem for your Semester

**Type-1 : Calculate the Moment of resistance (M.O.R)**

Given data : $F_y$, $F_{ck}$, $A_{st}$, b, d.

**Step-1 = Find $x_{ulim}$**

$$\boxed{x_{ulim} = K.d}$$

**Step-2 = find Actual depth of N.A. ($x_u$)**

Compressive force = Tensile force.

$C = T.$

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

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

**Step-3 : Compare $x_u$ with $x_{ulim}$**

→ If $x_u = x_{ulim}$ → Balanced Section
  ($x_{ulim}$ shall be used)

→ If $x_u < x_{ulim}$ → Under R/F Secⁿ
  ($x_u$ shall be used)

→ If $x_u > x_{ulim}$ → Over R/F Secⁿ.
  ($x_{ulim}$ shall be used)

* Always prefer under R/F section.
  I.e. $x_u < x_{ulim}$

Step-4 : Calculate M.O.R.

$$(M.O.R)_{compression} = C \times Z$$

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

$\boxed{or}$

$$(M.O.R)_{Tension} = T . Z$$

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

—o—

Type-2 : Design of beams

Given data : L, Weight or load, Bending Moment, $F_y$ & $f_{ck}$, b

Step-1 : Take $W_u$, $M_u$, $L_{effective}$

or

$$M_u = \frac{W_u L_{eff}^2}{8}$$

Step-2 : Calculate effective depth

$$M_u = M.O.R = 0.36 f_{ck} b . x_u (d - 0.42 x_u)$$

$$= 0.36 f_{ck} . b . (K.d) \{d - 0.42 (K.d)\}$$

$$\Rightarrow \frac{W_u (L_{eff})^2}{8} = 0.36 f_{ck} . b (K.d) \{d - 0.42 K.d\}$$

$d =$ ______ (Ans)

step-3 : calculate area of steel

$C = T$

$\Rightarrow 0.36 f_{ck} \cdot b \cdot x_u = 0.87 f_y A_{st}$

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

or

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

$\Rightarrow \frac{W_u (L_{eff})^2}{8} = 0.87 f_y A_{st} (d - 0.42 \cdot K \cdot d)$

$$\Rightarrow \boxed{A_{st} = \frac{W_u (L_{eff})^2}{8 \times 0.87 f_y d (1 - 0.42K)}}$$

or

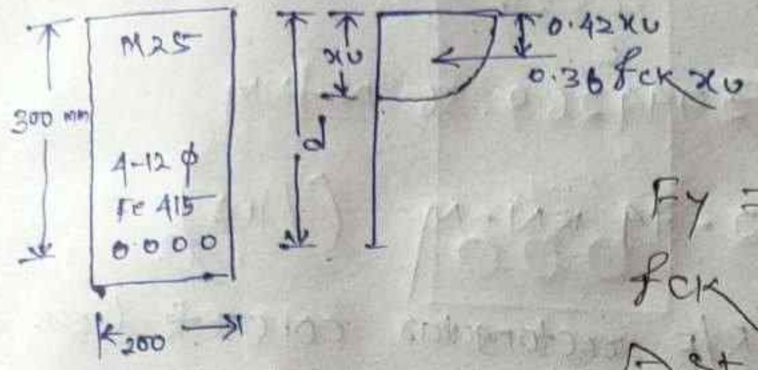
$$\boxed{A_{st} = 0.5 \frac{f_{ck}}{f_y} \left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b \cdot d^2}}\right] b \cdot d}$$

step-4 : Assume diameter of steel & calculate Number of steel.

$$\text{Number of steel} = \frac{A_{st}}{\frac{\pi}{4} d^2}$$

$= \_\_\_\_\_$ (Ans)

Q-10 What is the Moment of resistance by LSM in KN-M ?




$F_y = 415$

$f_{ck} = 25$

$A_{st} = \frac{\pi}{4}(12)^2 \times 4.$

Soln:

Step-1: Calculate $x_{ulim}$

$$x_{ulim} = K \cdot d$$

$$= 0.48 \times 300 \quad (\text{For } Fe415)$$

$$x_{ulim} = 144 \text{ mm}$$

Step-2: Actual depth of N.A

$$x_u = \frac{0.87 F_y A_{st}}{0.36 f_{ck} \cdot b}$$

$$= \frac{0.87 \times 415 \times \left[\frac{\pi}{4}(12)^2 \times 4\right]}{0.36 \times 25 \times 200}$$

$$x_u = 90.664 \text{ mm}$$

Step-3: Compare $x_u$ with $x_{ulim}$

Here $x_u < x_{ulim}$. So the section is under reinforced Section

Step-4: Moment of resistance.

$$M.R = 0.36 f_{ck}\, b \cdot x_u (d - 0.42 x_u).$$

$$M.R = 0.36 \times 25 \times 200 \times 90.664 \times (300 - 0.42 \times 90.664)$$

$$= 42744269.56 \text{ N-MM}$$

$$= 42.74 \text{ KN-M (Ans)}$$

Q-11 A single R/F rectangular concrete beam has a width 150mm & effective depth 330mm. The fck = 20mpa, fy = 415 mpa. The limiting depth of neutral axis is 0.48 times the effective depth of the beam.

(a) what is the limiting value of M.O.R (KN-m)

(b) what is the limiting area of tensile steel ($mm^2$)

Ans:
$b = 150$ mm.

$d = 330$ mm.

$F_{ck} = 20$

$F_y = 415$.

$$x_{u\,lim} = 0.48 \times d = 0.48 \times 330 = 158.4 \text{ mm}$$

$$M.O.R = Q F_{ck} b d^2$$

$$= 0.138 \times 20 \times 150 \times (330)^2$$

$$= 45084600 \text{ N-MM}$$

$$M_{u\,lim} = 45.08 \text{ KN-M (Ans)}$$

$C = T$

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

$$= \frac{0.36 \times 20 \times 150 \times 158.9}{0.87 \times 415}$$

$$= 473.82 \text{ mm}^2 \text{ (Ans)}.$$

## Doubly Reinforced Section:

→ When moment carring capacity of single reinforced section exceeds the $M_{ulim}$, then we provide doubly R/F section.

Suppose $M_{ulim} = 0.138 f_{ck} b d^2$

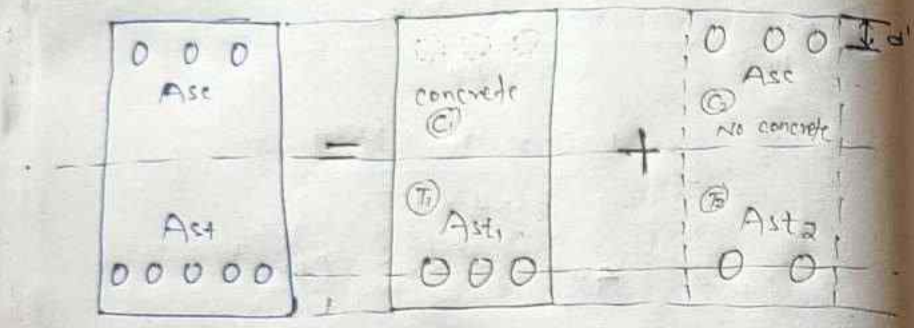
$= 350 \text{ KN-M}$.

But $M_u = \frac{W_u (L_{eff})^2}{8} = 400 \text{ KN-M}$.

$M_{ulim} < M_u$.

∴ we provide extra steel for carring the extra moment. (I.e. $M_u - M_{ulim}$).

→ In this section we provide steel bar in compression zone as well as in tension zone. I.e. in top & bottom of the section.




Doubly R/F Section, = Singly R/F Balanced Section, + 

(400 KN-M) (350 KN-M) (50 KN-M)

$(M_u)$ $\left(M_{ulim} = Q f_{ck} b d^2\right)$ $(M_u - M_{ulim})$

Compressive force — stress in Conc x Area in Conc.

$C_1 = 0.36 f_{ck} b . x_u - (0.446 f_{ck} A_{sc})$

$C_2 = F_{sc} A_{sc}$

$$\boxed{C = C_1 + C_2 = 0.36 f_{ck} b x_u + A_{sc}(F_{sc} - 0.446 f_{ck})}$$

$A_{sc}$ = Area of comp. steel

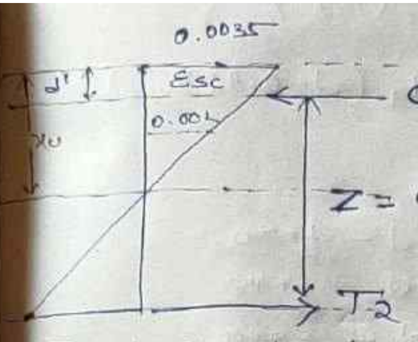
$A_{st}$ = Area of Tension steel.

$F_{sc}$ = force in comp. steel.

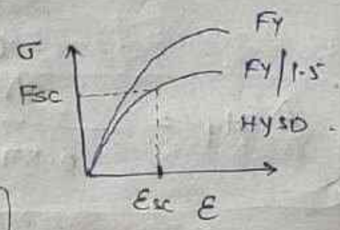
$A_{st} = A_{st1} + A_{st2}$

* $M_u > M_{ulim}$ = Doubly R/F secn

$M_u < M_{ulim}$ = Single R/F secn.

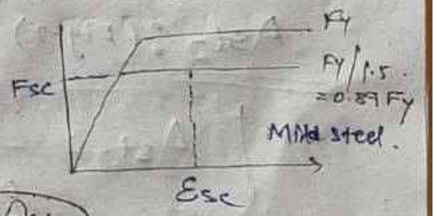



From Similar triangle




$$\frac{0.0035}{x_u} = \frac{E_{sc}}{x_u - d'}$$

$$\Rightarrow \boxed{E_{sc} = 0.0035\left(1 - \frac{d'}{x_u}\right)}$$




Total tensile force :

$T = 0.87 F_y A_{st1} + 0.87 F_y A_{st2}$

$T = 0.87 F_y A_{st}$

For finding $x_u$

$C = T$

$\Rightarrow 0.36 f_{ck} b x_u + A_{sc}(F_{sc} - 0.446 f_{ck}) = 0.87 F_y A_{st}$.

Moment of resistance :

$MOR = 0.36 F_{ck} b x_u (d - 0.42 x_u) + A_{sc}(F_{sc} - 0.446 f_{ck})(d - d')$

$C_1 Z_1 + C_2 . Z_2$

OR

$M.O.R = T_1 Z_1 + T_2 Z_2$

$= 0.87 F_y A_{st1}(d - 0.42 x_u) + 0.87 F_y A_{st2}(d - d')$

To calculate $A_{st_1}$ (Fig-1)

$$M_{ulim} = 0.87 F_y A_{st_1} (d - 0.42 x_{ulim})$$

$$x_{ulim} = K \cdot d$$
$$= 0.53d - Fe250$$
$$= 0.48d - Fe415$$
$$= 0.46d - Fe500$$

$$\boxed{A_{st_1} = \frac{M_{ulim}}{0.87 F_y (d - 0.42 x_{ulim})}}$$

$A_{st_2}$ : (Fig-2)

$$\boxed{A_{st_2} = \frac{M_u - M_{ulim}}{0.87 F_y (d - d')}}$$

Asc :

$$C_2 = T_2.$$

$$\Rightarrow A_{sc}(F_{sc} - 0.446 F_{ck}) = 0.87 F_y A_{st_2}$$

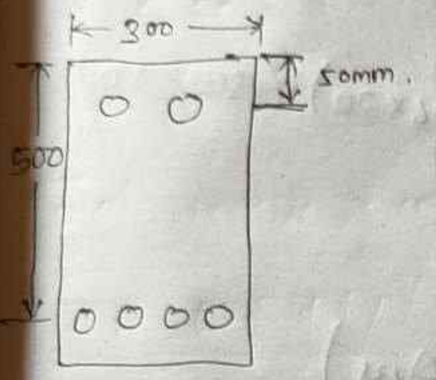
$$\Rightarrow \boxed{A_{sc} = \frac{0.87 F_y A_{st_2}}{F_{sc} - 0.446 f_{ck}}}$$

Q-12 A doubly R/F rectangular concrete beam has a width of 300mm & an effective depth of 500 mm. The beam is reinforced with 2200 mm² of steel in tension & 628 mm² of steel in comp. The effective cover for comp. steel is 50mm. Assume that both tension & comp. steel yield. M20 & Fe250.

(a) The depth of N.A.

(b) M.O.R (KN-M).

Solⁿ: Given data:




$A_{st} = 2200\ mm^2$

$A_{sc} = 628\ mm^2$.

$F_{sc}$ for (Fe250) $= 0.87 f_y$

$= (0.87 \times 250)$

M20 & Fe250 grade.

Step-1: calculate $x_u$

$$0.36 f_{ck} b x_u + A_{sc}(F_{sc} - 0.446 f_{ck}) = 0.87 f_y A_{st}.$$

$$\Rightarrow 0.36 \times 20 \times 300\, x_u + 628(0.87 \times 250 - 0.446 \times 20)$$

$$= 0.87 \times 250 \times 2200.$$

(a.) $x_u = 160.885$ mm (Ans)

Step-2: MOR

$$0.36 f_{ck} b.\, x_u (d - 0.42 x_u) + A_{sc}(F_{sc} - 0.446 f_{ck})(d - d')$$

$x_{ulim} = 0.53\,d$ (For Fe 250)

$= 0.53 \times 500$

$= 265$ mm

So $x_u < x_{ulim}$, So the Secⁿ is under R/r section.

So. M.O.R =

$= 0.36 \times 20 \times 300 \times 160.85\,(500 - 0.42 \times 160.85)$

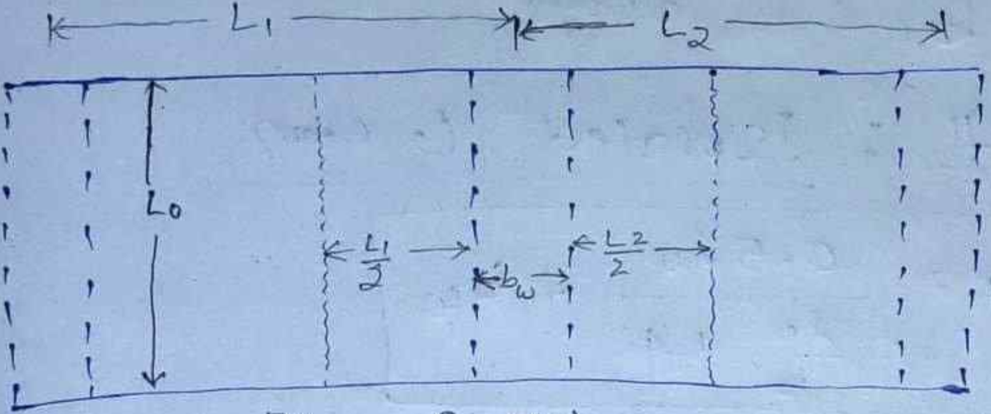
$+ 628\,(0.87 \times 250 - 0.446 \times 20)\,(500 - 50)$.

$= 150.29 + 58.945$

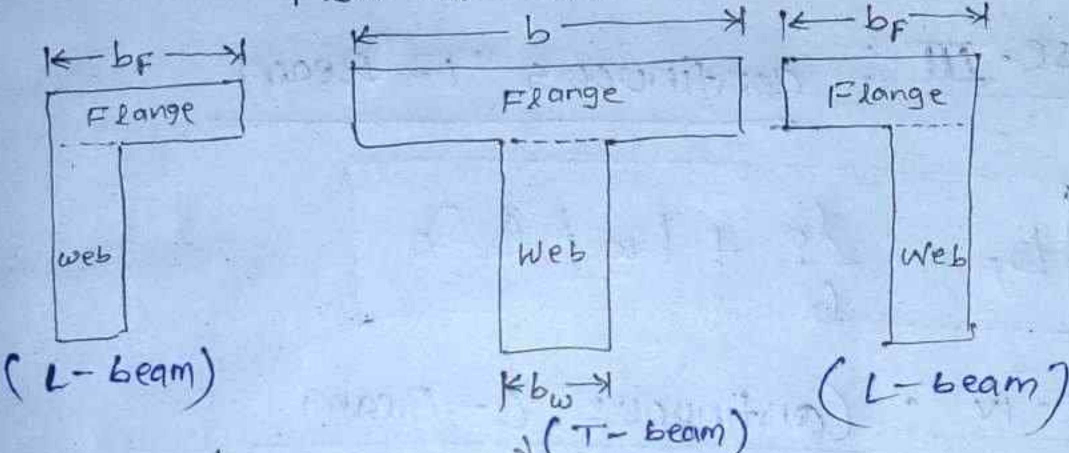
$= 209.234$ KN-M (Ans).

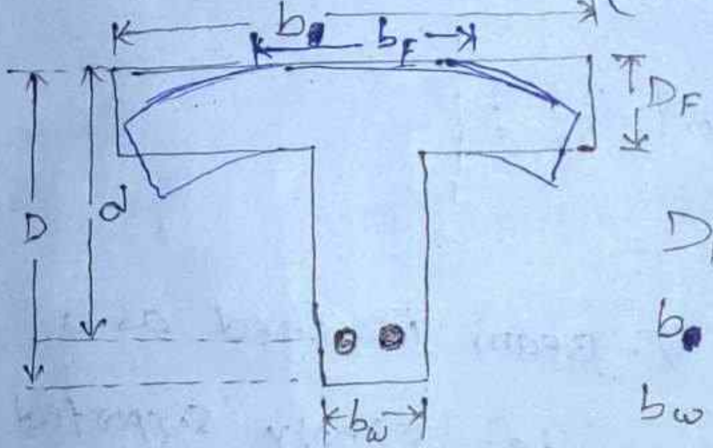
1

# Analysis & Design of Flanged Beams :




Plan of slab




where

$D_F$ = Depth of flange.

$b$ = Breadth of flange.

$b_w$ = Breadth of web.

$D$ = overall depth.

$d$ = effective depth.

## Case-1 : Isolated T- Beam

$$\boxed{b_F = \frac{l_0}{\frac{l_0}{b} + 4} + b_w}$$

$b_F$ = effective width of flange.

$l_0$ = effective length of beam.

2

→ $b = \frac{L_1}{2} + b_w + \frac{L_2}{2}$

→ $b_F < b$ always.

case - II : Isolated L - beam

$$\boxed{b_F = \frac{0.5\, l_0}{\frac{l_0}{b} + 4} + b_w}$$

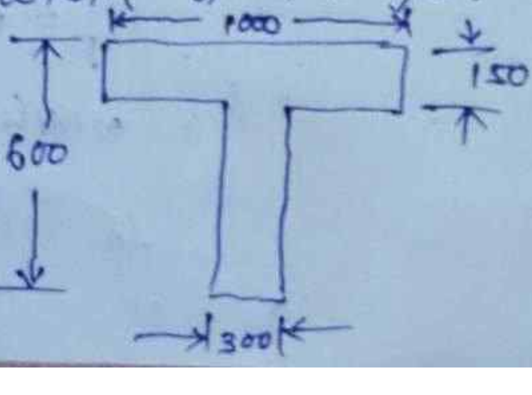
case - III : continuous T - Beam

$$\boxed{b_F = \frac{l_0}{6} + b_w + 6 D_F}$$

case - IV : Continuous L - Beam

$$\boxed{b_F = \frac{l_0}{12} + b_w + 3 D_F}$$

Q-13 An isolated T-Beam is used as a walkway. The beam is simply supported with an effective span of 6 m. The effective width of Flange. For the c/s



3

Ans Given data :

(b)

$l_0$ = effective span = 6m.

$D_F$ = 150 mm.

$b_w$ = 300 mm.

$b$ = 1000 mm.

$D$ = 600mm.

Isolated T - Beam

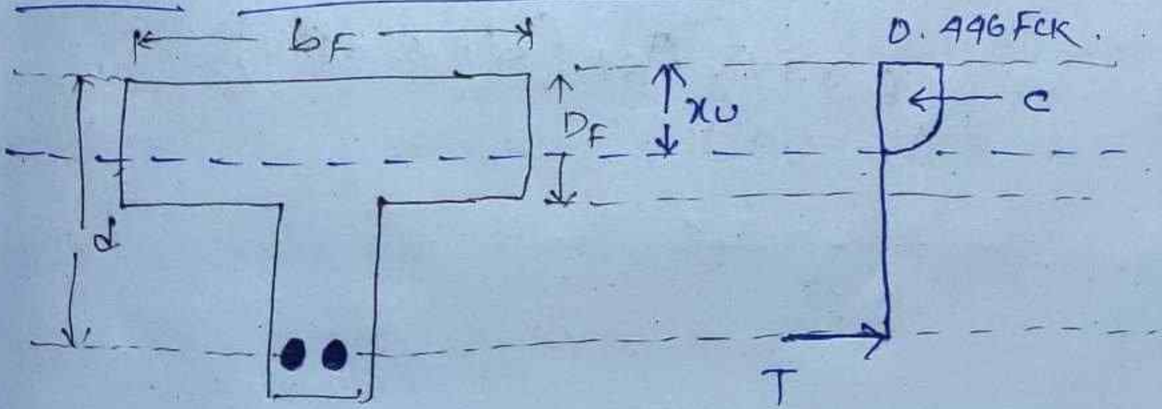
$$b_F = \frac{l_0}{\frac{l_0}{b} + 4} + b_w$$

$$= \frac{6 \times 1000}{\frac{6000}{1000} + 4} + 300$$

$$= 600 + 300 = 900 \text{ mm (Ans)}.$$

Analysis of Flanged beam:

Case-I : When N.A. is in Flange portion :



$C = 0.36 f_{ck} b_f . x_u$.

$T = 0.87 f_y A_{st}$.

→ $x_{ulim} = K . d$.

→ $x_u =$

$C = T$

$\Rightarrow x_u = \dfrac{0.87 f_y A_{st}}{0.36 f_{ck} b_f}$

→ compare $x_u$ with $x_{ulim}$.

$x_u = x_{ulim}$ – Balanced secn – $x_{ulim}$

$x_u < x_{ulim}$ – URS – $x_u$

$x_u > x_{ulim}$ – ORS – $x_{ulim}$.

→ M.O.R $= C Z$

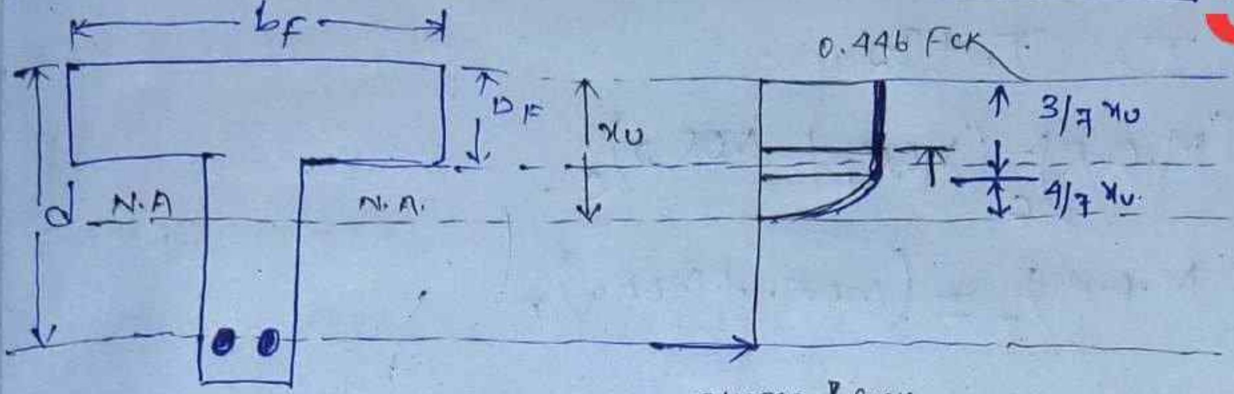
$= 0.36 f_{ck} b_f x_u (d - 0.42 x_u)$.

or,

$T.Z$

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

## Case-II : When N.A. is in the web portion




stress block.

**Sub case-I : When the Flange is uniformly stressed** $\left(\frac{3}{7}x_u \geq D_f\right)$

From above diagram:

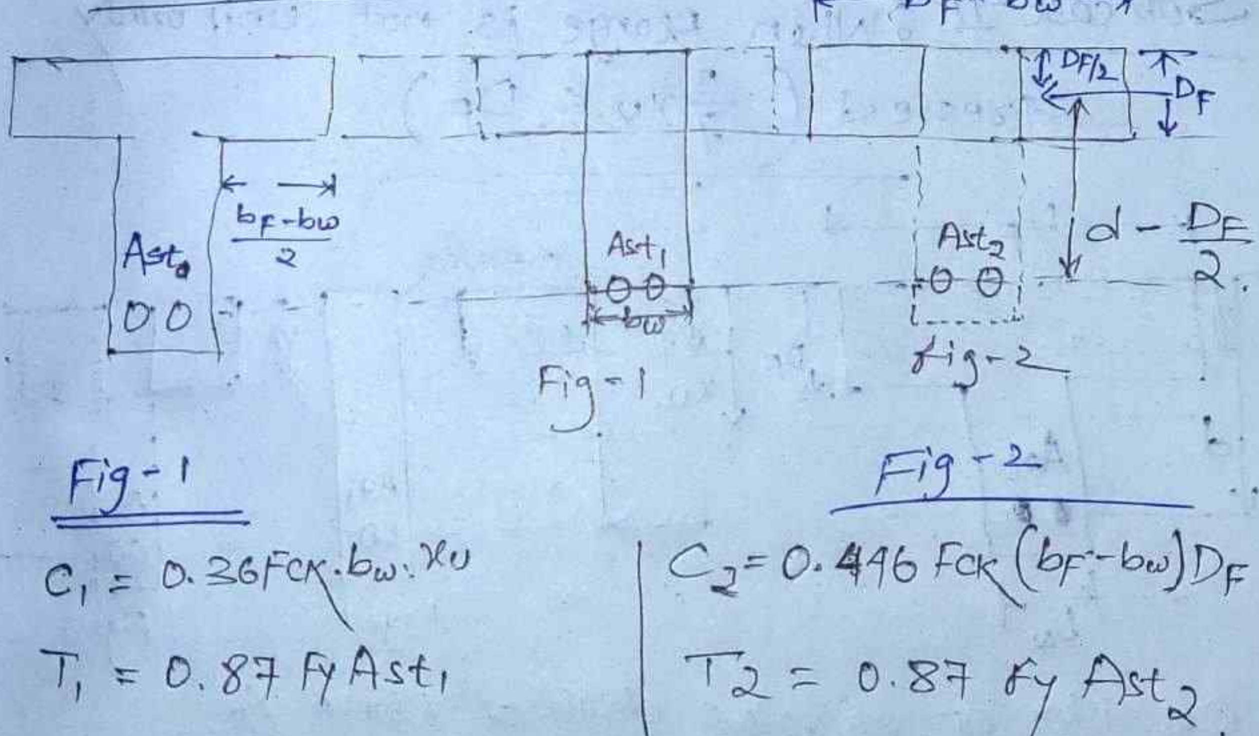



**Fig-1**

$C_1 = 0.36 F_{CK} \cdot b_w \cdot x_u$

$T_1 = 0.87 F_y Ast_1$

Lever arm $(L.A_1) = d - 0.42 x_u$

$M.O.R_1 = 0.36 f_{ck} b_w x_u (d - 0.42 x_u)$

$MOR_1 = 0.87 f_y Ast_1 (d - 0.42 x_u)$

**Fig-2**

$C_2 = 0.446 F_{CK} (b_F - b_w) D_F.$

$T_2 = 0.87 f_y Ast_2.$

$LA_2 = d - \frac{D_F}{2}.$

$M.O.R_2 = 0.446 F_{CK} (b_F - b_w) D_F \left(d - \frac{D_F}{2}\right)$

$M.O.R_2 = 0.87 f_y Ast_2 \left(d - \frac{D_F}{2}\right)$

6

$\rightarrow C = C_1 + C_2$

$T = T_1 + T_2$

$(M.O.R)_C = (MOR_1 + MOR_2)_C$

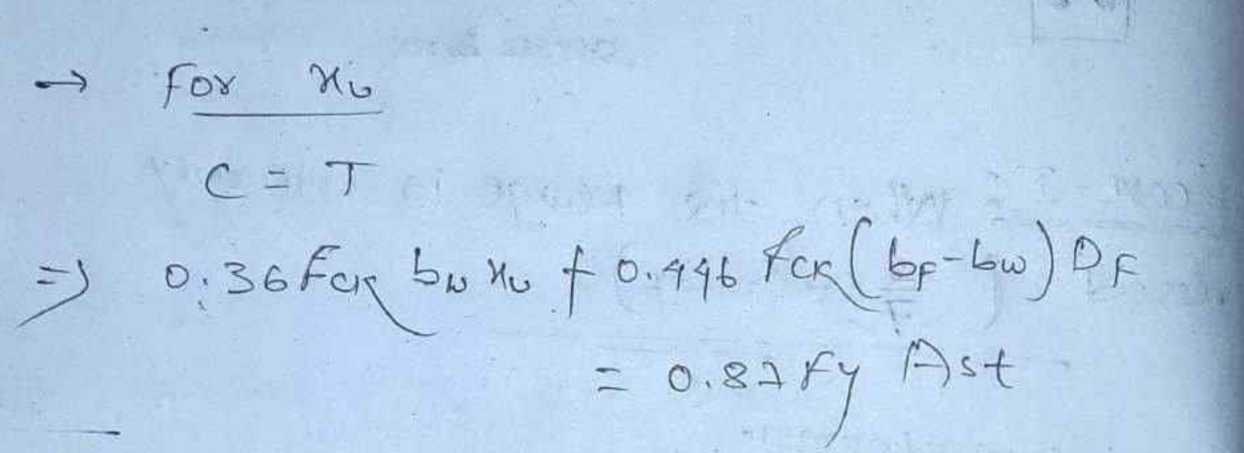
$(M.O.R.)_T = (MOR_1 + MOR_2)_T$

$\rightarrow$ For $x_u$

$C = T$

$\Rightarrow 0.36 f_{ck} b_w x_u + 0.446 f_{ck} (b_f - b_w) D_f$
$= 0.87 f_y A_{st}$

**Sub case-II : When flange is not uniformly stressed $\left(\frac{3}{7} x_u < D_f\right)$**




Consider an equivalent depth of flange '$y_f$'.

$y_f = 0.15 x_u + 0.65 D_f$

7

## Fig-1

$C_1 = 0.36 f_{ck} b_w x_u$

$T_1 = 0.87 F_y A_{st_1}$

$L.A_1 = d - 0.42 x_u$

$MOR_1 = 0.36 F_{ck} b_w x_u (d - 0.42 x_u)$

$MOR_1 = 0.87 f_y A_{st_1} (d - 0.42 x_u)$

## Fig-2

$C_2 = 0.446 F_{ck} (b_F - b_w) D_F$

$T_2 = 0.87 F_y A_{st_2} \cancel{\left(d - \frac{y_F}{2}\right)}$

$L.A_2 = d - \frac{y_F}{2}$

$MOR_2 = 0.446 F_{ck} (b_F - b_w) y_F \left(d - \frac{y_F}{2}\right)$

$MOR_2 = 0.87 F_y A_{st_2} \left(d - \frac{y_F}{2}\right)$

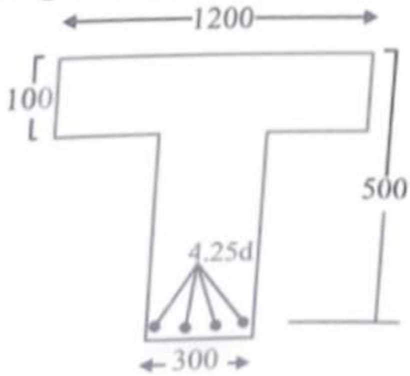
$C = C_1 + C_2$

$T = T_1 + T_2$

$(MOR)_c = (MOR_1 + MOR_2)_c$

$(MOR)_T = (MOR_1 + MOR_2)_T$

**(c) Calculate the moment of resistance of a T-beam as shown in figure. Assuming M20 mix and Fe415 grade steel.**




**Ans.** Grade of steel = fe 415

Grade of concrete =M20

B =1200 MM

t = 100 MM

$B_r$ = 300 MM

D = 560MM

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

$$= 3 \times \frac{\pi}{4} \times (25)^2 = 1963.49\text{mm}^2$$

Assume that the Neutral Axis is whit in the flange

so.

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

$$X_u = \frac{0.87\, fy\, Ast}{0.36\, fck\, b}$$

$$= \frac{0.87 \times 415 \times 1963.49}{0.36 \times 20 \times 1200} = 82.05 \text{ mm}$$

T = 100mm $X_u < t$

Hence our assumption is correct

$X_{u\ max}$ = 0.48 d = 268.8 mm

$X_u < X_{u\ max}$ hence it is under reinforced beam

M.O.R = 0.87 fy $A_{st}$(d–0.42.$X_u$)

= 0.87 × 415 × 1963.49(560 – (0.42 × 82.05))

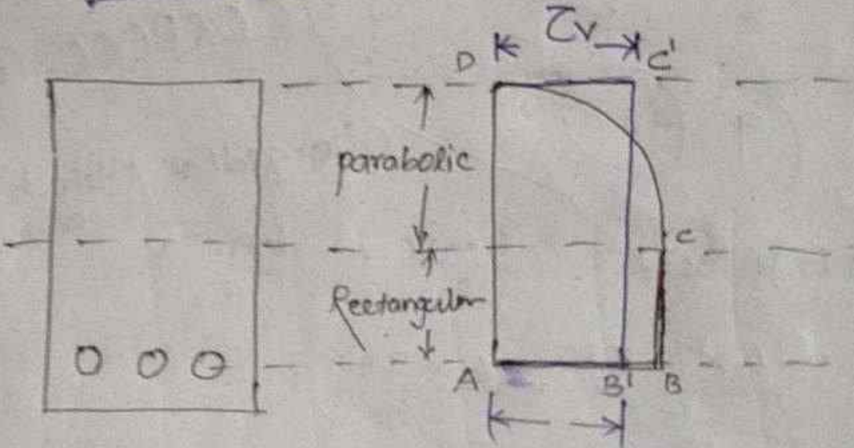
= 708918.06 × (525.539)

= 372564088.3 = 372.56 × $10^6$N/mm²

# SHEAR, BOND & DEVELOPMENT LENGTH

## Limit state of collapse : shear

**1** Nominal shear stress ($\tau_v$) :




$$\tau_v = \frac{V_u}{bd}$$

ABCD = Shear stress distribution (Actual)

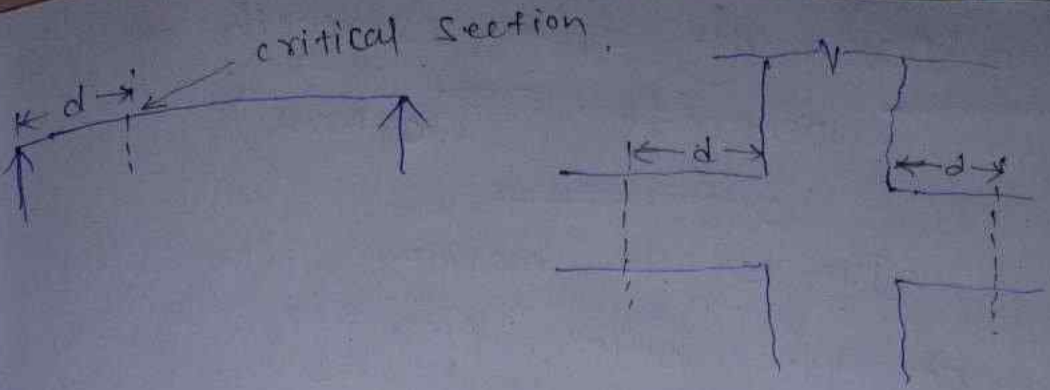
AB'C'D = considered shear stress

$V_u$ = Factored shear forced

For prismatic beam $\boxed{\tau_v = \frac{V_u}{bd}}$

→ $V_u$ should calculate at a section 'd' distance from support.

Because it is critical section.




## 2 Shear strength of concrete ($\tau_c$):

Shear strength of concrete (RCC) depends upon grade of concrete & percentage of steel reinforcement.

→ If. % $A_{st}$ ↑ → $\tau_c$ ↑.

$F_{ck}$ ↑ → $\tau_c$ ↑.

$$\% A_{st} = \frac{(A_{st})_{support}}{b \cdot d} \times 100$$

→ Use Table No-19 [IS 456 : 2000]

| % Ast | Fck | | | |
|---|---|---|---|---|
| | M20 | M25 | M30 | M35 |
| 0.25 | | | | |
| 0.50 | | | | |
| 1.00 | | | | |
| 1.20 | | | | |

$\tau_c$ value

## 3 Maximum shear strength of concrete ($\tau_{c\,max}$)

It is the maximum shear strength of concrete after providing the shear reinforcement & % Ast.

If $f_{ck} \uparrow \rightarrow \tau_{c\,max} \uparrow$

| Fck | M15 | M20 | M25 | M30 | M35 | M40 | & above |
|---|---|---|---|---|---|---|---|
| $\tau_{c\,max}$ N/mm² | 2.5 | 2.8 | 3.1 | 3.5 | 3.7 | 4.0 | |

$$\rightarrow \boxed{\tau_{c\,max} = 0.62\sqrt{F_{ck}}}$$

# * Design of shear R/F :

## Case-I : $\tau_v < \tau_c$

→ Resistance is more.

$\tau_v$ = Produced

$\tau_c$ = Resistance.

No require to provide R/f.

→ But Minimum shear reinforcement shall be provided.

$$\boxed{\frac{A_{sv\,min}}{b.S_v} \geq \frac{0.4}{0.87 f_y}}$$

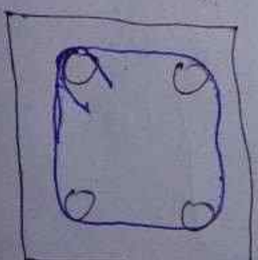
$A_{sv}$ = Area of shear reinforcement.

$S_v$ = Spacing in steel bars (stirrups)

→ $A_{sv} = n.\frac{\pi}{4}(\phi^2)$ , $\phi$ = stirrup's dia

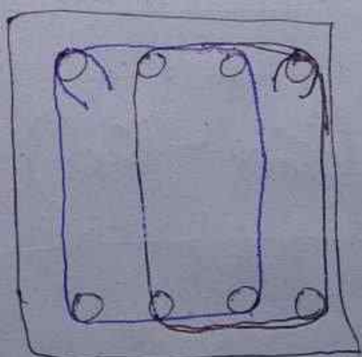
$n$ = No. of legs.

→ $F_y \not> 415\ N/mm^2$



2-legged stirrups

$A_{sv} = 2 \times \frac{\pi}{4}.\phi^2$



4-legged stirrups

$A_{sv} = 4 \times \frac{\pi}{4}.\phi^2$

## Case-II : $\tau_v > \tau_c$ ($\tau_v < \tau_{c\,max}$)

→ Here applied shear stress is more than the resistance. So provide extra shear r/F.

→ **Design of shear R/F :**

$$\boxed{V_u = \tau_v . bd}$$

↳ Applied shear force.

$$\boxed{V_c = \tau_c . bd}$$

↳ shear force taken by concrete.

$V_{us}$ → Design shear force taken by stirrups.

$$\boxed{V_{us} = V_u - V_c}$$

$$\boxed{V_{us} = (\tau_v - \tau_c) b.d}$$

## a For vertical Stirrups:

$$V_{us} = \frac{0.87\, F_y . A_{sv} . d}{S_v}$$

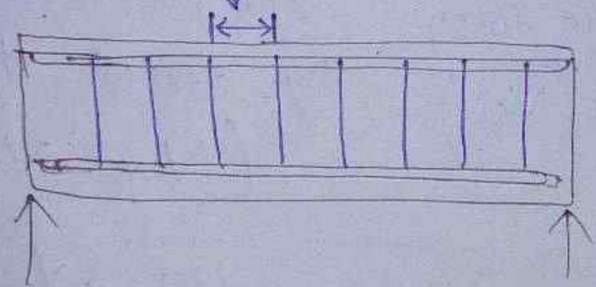
$$\boxed{S_v = \frac{0.87\, F_y . A_{sv} . d}{V_{us}}}$$

$F_y \ngtr 415\ N/mm^2$

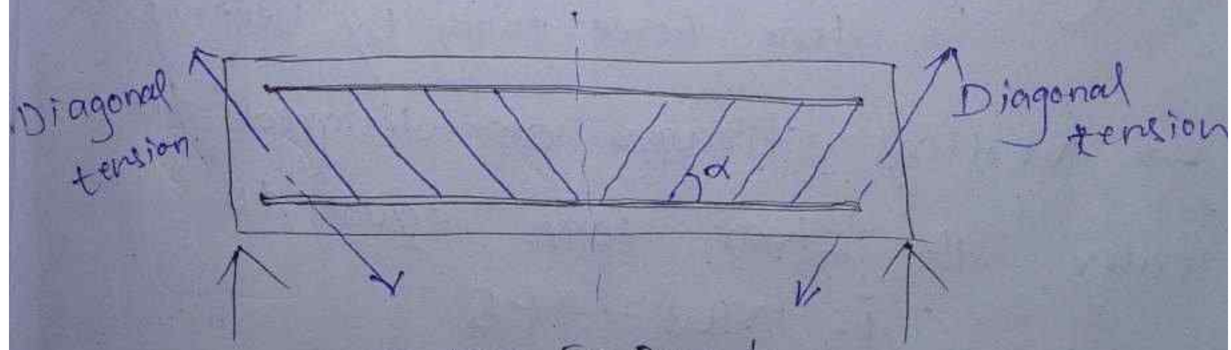
$A_{sv} = n \cdot \frac{\pi}{4}(\phi)^2$

$n$ = No. of legs.

$S_v$ = centre to centre spacing of vertical stirrups.



## b Inclined Stirrups:




$$V_{us} = \frac{0.87\, F_y . A_{sv} . d}{S_v}(\sin\alpha + \cos\alpha)$$

$$\boxed{S_v = \frac{0.87\, F_y . A_{sv} . d}{V_{us}}(\sin\alpha + \cos\alpha)}$$

Limitation

→ $\alpha \not< 45^\circ$

→ $F_y \not> 415\ N/mm^2$

## C Bent-up bars:

Here provide some stirrups &

some bent-up bars.

$$\boxed{V_{us} = (\tau_v - \tau_c) bd}$$

Some portion is taken by bent-up bars ($V_{sb}$)

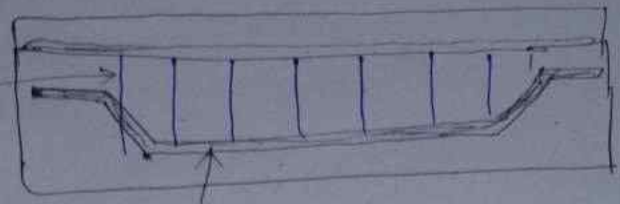
Some portion is taken by vertical stirrups ($V_{us} - V_{sb}$)

$$\boxed{V_{sb} = 0.87 F_y A_{sb} (\sin\alpha)}$$

↳ shear force taken by bent-up bars

→ Vertical stirrups are designed to carry the shear force equal to

Maximum $\begin{cases} \text{i. } V_{us} - V_{sb} \\ \text{ii. } \dfrac{V_{us}}{2} \end{cases}$




**Ex**

$V_u = 400$ KN. Design the vertical stirrups.

$V_c = 100$ KN.

**case - I**

$V_{sb} = 100$ KN

$V_{us} = 400 - 100 = 300$ KN

i $V_{us} - V_{sb} = 300 - 100 = 200$ KN

ii. $\frac{V_{us}}{2} = \frac{300}{2} = 150$ KN

$\Big\}$ Max^m

→ Vertical stirrups designed For 200 KN.

**case - II**

$V_{sb} = 250$ KN.

i. $V_{us} - V_{sb} = 50$ KN

ii. $\frac{V_{us}}{2} = 150$ KN

$\Big\}$ max^m

→ vertical stirrups are Designed for 150 KN.

— o —

**case - III :** $\tau_v > \tau_{c\,max}$

*** This test / check is required for diagonal compression Failure.

→ $\tau_v > \tau_{c\,max}$ = Then Redesign the Section.

**ARC beam has an effective depth of 500 mm and a breadth of 350 mm. It contains 4-25 mm bars calculate the shear reinforcement needed for a factored shear force of 350 kN for M20 mix and Fe250 grade steel.**

**Ans.** RC. Beam have effective depth = 500mm

Breadth =350mm

Tension bars = 4.25mm

Factored shear force = 350mm

Grade of concrete = M20

Grade of steel = fe250

$$\text{Percentage of steel} = \frac{Ast}{bd} \times 100$$

$$Ast = 4 \times \frac{\pi}{4} \times (25)^2 = 1963.50mm^2$$

$$= \frac{Ast \times 940}{bd} = \frac{1963.50}{350 \times 500} = 1.12$$

$\sigma_c$ = design shear strength

| | |
|---|---|
| X = 1.0 | f(x) = 0.62 |
| X = 1.25 | f(x) = 0.67 |
| X = 1.12 | f(x) = ? |

$$F(x) \text{at } x = 1.12 = 0.62 + \frac{0.67 - 0.62}{0.25}(1.12 - 1)$$

$$= 0.62 + \frac{0.05}{0.25}(0.12)$$

$$= 0.62 + \frac{0.006}{0.25} = 0.62 + 0.024 = 0.644$$

So design strength $\sigma_c = 0.644N/mm^2$

$\Rightarrow$ Sc = 112.700kn

Here $\sigma_v > \sigma_c$

So net shear to be resisted by shear reinforcement

=350 – 112.700 = 237.3kn

Provide 2legged 6mm dia stirrup

$$\text{Spacing} = \frac{0.87fyAw \times d}{Vus}$$

$$= \frac{0.87 \times 250 \times (2 \times 28) \times 500}{237.3 \times 1000} = 25.66mm$$

Spacing will be list of

(i) 0.75d = 375mm

(ii) $\frac{0.87Awfy}{0.4b} = \frac{0.87 \times (2 \times 28) \times 250}{0.4 * 350} = 87mm$

(iii) 300mm

(iv) 25.66 mm

provide a spacing 25mm.

# Design of Bond:

→ Bond in reinforced concrete refers to the adhesion between the reinforcing steel and the surrounding concrete.

→ Bond, which is responsible for the transfer of axial force from a reinforcing bar to the surrounding concrete.

→ Due to bond, strain compatibility and 'composite action' of concrete and steel.

→ Inadequate bond refers to 'slipping'.

## * Mechanisms of Bond Resistance:

Bond resistance in RCC is achieved through

1. Chemical adhesion:

Due to a gum-like property in the products of hydration.

2. Frictional Resistance:

Due to the surface roughness of the reinforcement & the grip exerted by the concrete shrinkage.

3. Mechanical interlock :

Due to the surface ribs provided in deformed bars.

Bond stress :

It is achieved by the development of tangential stress component along the interface between reinforcing bar & the surrounding concrete. The stress so developed at the interface is called bond stress.

$$\text{Bond stress} = \frac{\text{Tangential stress}}{\text{Nominal Area}}.$$

→ Two types of bond stress.

1. Flexural bond.
ii. Anchorage bond.

## Development length ( Ld ) :

The calculated tension or comp. in any bar at any section shall be developed on each side of the section by an appropriate development length or end anchorage by a combination there of.

→ 'Development length' is that a certain minimum length of the bar is required on either side of a point of maximum steel stress, to prevent the bar from pulling out the under ~~tension~~ tension.

$$L_d = \frac{\phi F_s}{4 \tau_{bd}}.$$