LECTURE NOTE

ON

STRUCTURAL DESIGN-I

FOR

DIPLOMA IN CIVIL ENGINEERING

(4TH SEMESTER STUDENTS)

AS PER SCTE&VT SYLLABUS

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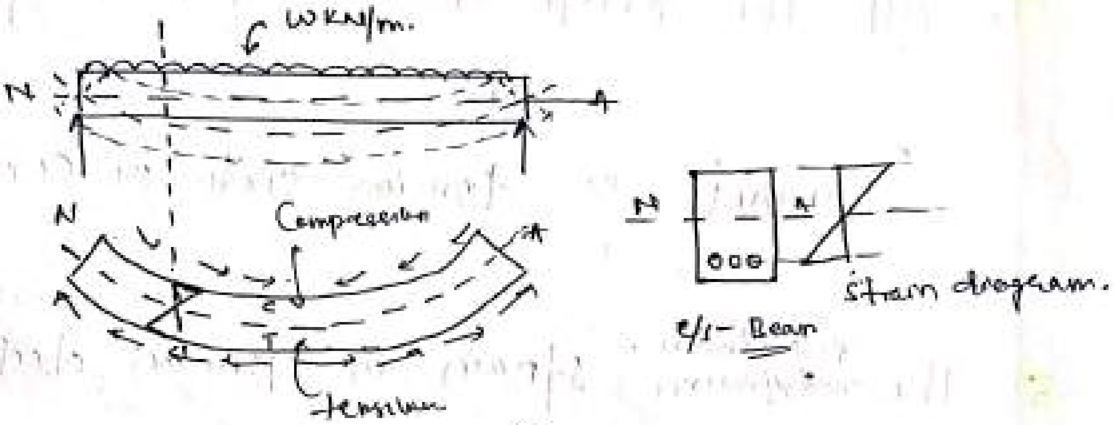
Limit State Of Collapse For Flexure :-

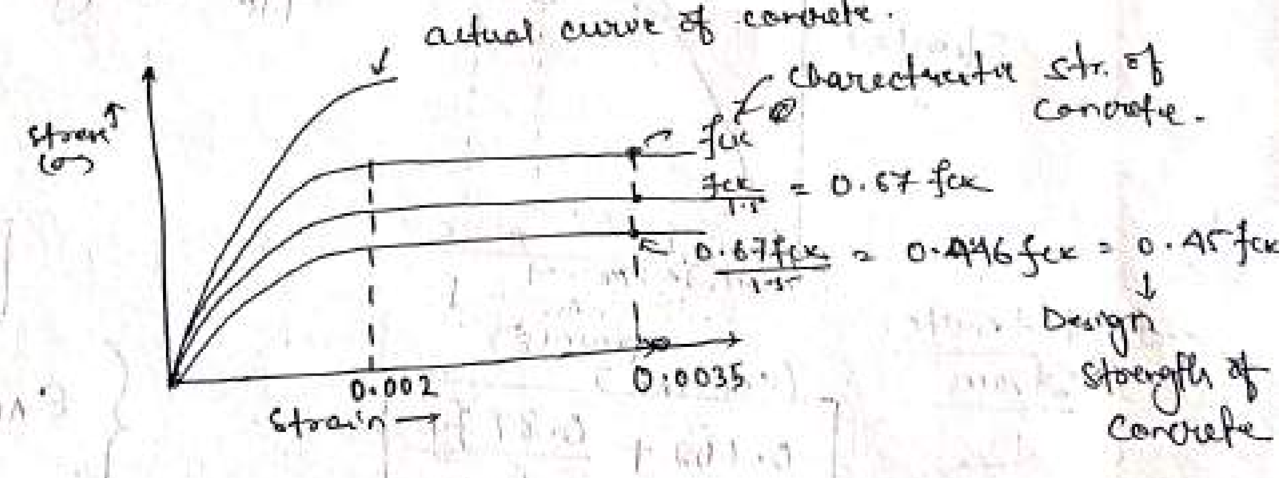
Assumptions | CL-38.1 | Page No - 69

1. Plain section remains plain before and after bending.

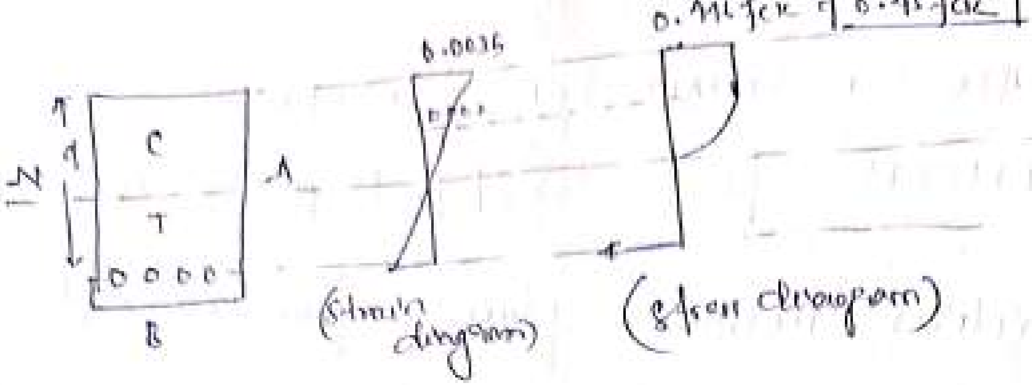


* It mean strain variation is linear

2. The compressive strain in concrete shall not be greater then 0.0035 under flexure condition.



3. The stress block is parabolic upto strain 0.002 but from 0.002 to 0.0035 there will be rectangular.

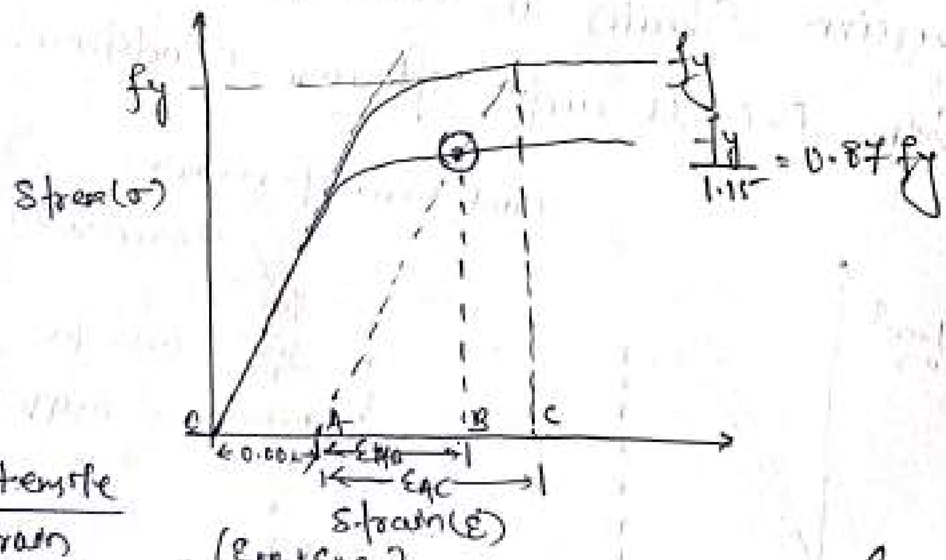


4. All the tensile stress are taken up by steel only

or

"Concrete is tension side is compleatly ignored."

5. The ~~minimum~~ maximum strain in tension steel shouldnot be. less then. $0.002 + \frac{0.87 f_y}{E_s}$.



$$E_s = \frac{\sigma}{\varepsilon}$$

$$\boxed{\varepsilon = \frac{\sigma}{E_s}}$$

~~total~~ tensile Strain ↳

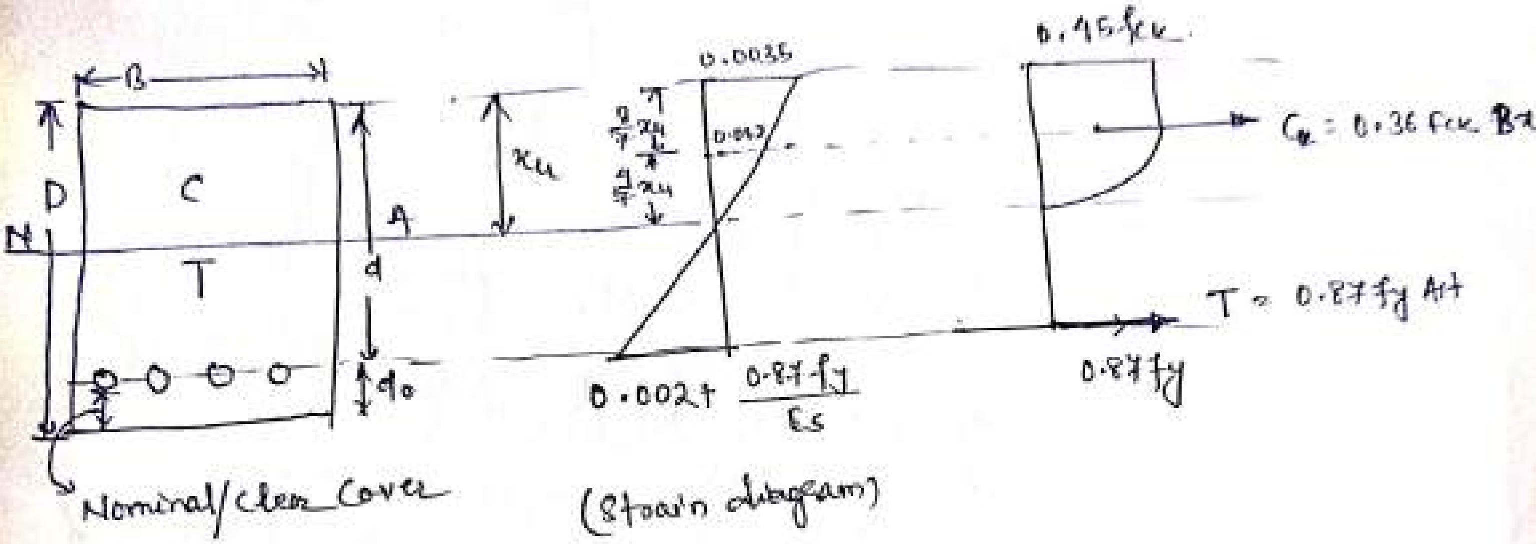
$(\varepsilon_{OA} + \varepsilon_{AB})$

$$\boxed{0.002 + \frac{0.87 f_y}{E_s}}$$

$$\left\{ \varepsilon_{AB} = \frac{0.87 f_y}{E_s} \right.$$

or

$$\boxed{0.002 + \frac{0.f_y}{1.15 E_s}}$$



x_u = Actual depth of neutral axis

~~x_u ... ~~

Actual Depth of Neutral Axis (x_u):-

The axis on which neither comp nor tension force occurs.

Comp. force = Tensile force.

$0.36\, f_{ck}\, B \cdot x_u = 0.87\, f_y\, A_{st}$

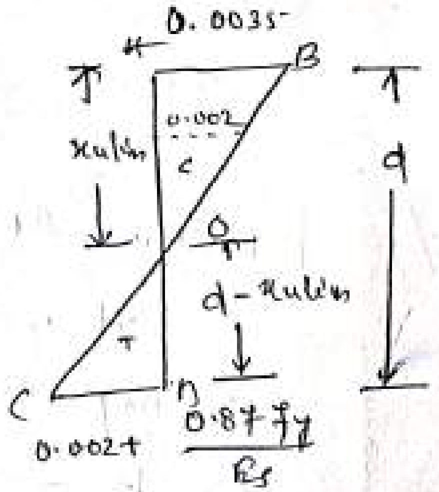
$$\boxed{x_u = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, B}}$$ *** Imp

→ $\boxed{x_u \propto A_{st}}$

As, we increase area of steel the value of actual depth of neutral axis is increase.

Critical depth of neutral Axis (x_{ulim}):-

It is the depth of NA for a balanced section where steel and concrete fail at same time.



from similarity of triangle.

OAB ~ OAC.

$$\Rightarrow \frac{AB}{OA} = \frac{CD}{OD}$$

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

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

where $\frac{700}{1100 + 0.87 f_y} = K \rightarrow$ critical neutral axis co-efficient.

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

the value of x_{ulim} depending on ~~steel~~. Grade of Steel.

Grade of Steel	$x_{ulim} = K \cdot d$
Fe 250	0.53 d
Fe 415	0.48 d
Fe 500	0.46 d.

Note → As we increase the grade of steel, the value of K is decrease.

$$\boxed{K \propto \frac{1}{f_y}}$$

LEVER ARM

It is clear distance between tensile and compressive force.

$$\boxed{Z = d - 0.42 \cdot x_u}$$

Moment of resistance:-

"It is the capacity of beam to resist the bending moment without fail."

Moment = force × Lever arm.

(MOR.) = force × Lever arm.

hence

for U/R section

from tension side

$(MR)_{Tension}$ = Tensile force × L.A.

$$\boxed{(MR)_{Tension} = 0.87 f_y A_{st} (d - 0.42 x_u)}$$

for O/R section

from Comp. side

$(MR)_{comp}$ = Comp. force × LA.

$$\boxed{(MR)_{comp} = 0.36 f_{ck} B . x_u (d - 0.42 x_u)}$$

for Balanced section

$$\boxed{(MR)_{comp} = 0.36 f_{ck} B . x_{ulim} (d - 0.42 x_{ulim})}$$

$$\boxed{(MR)_{tension} = 0.87 f_y A_{st} (d - 0.42 x_{ulim})}$$

Steps to Analyse of S/R Beam. Lec-10

Step 1 ① Calculate Critical depth of neutral axis. ($x_{u,lim}$) :—

$$\boxed{x_{u,lim} = K \cdot d}$$

f_e	$x_{u,lim} = K \cdot d$
Fe 250	0.53 d
Fe 415	0.48 d
Fe 500	0.46 d

Step 2 Calculate actual depth of N.A (x_u)

$$\boxed{x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} B}}$$ { by equating tensile force & compressive force.)

Step-3
(i) If $x_{u\,lim} > x_u$ → U/R section
(ii) If $x_{u\,lim} < x_u$ → O/R section
(iii) If $x_{u\,lim} = x_u$ → Balanced section.

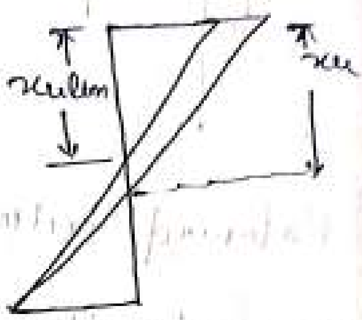
Step 4
Calculate (MR)
(i) U/R section. → from tension side.
(ii) O/R section → from Comp. side.
(iii) Balanced section → ~~Any side.~~ from both tension & comp. side

→ ~~In~~ the section will give proper indication/warning before failure (ductile failure).

3. ~~UR~~

3. Over Reinforced section!-

$\boxed{x_u > x_{ulim}}$



→ Reinforcement is more then balanced section.

→ In this section concrete reaches to its permissible stress before ~~failure~~ ~~of~~ steel.

→ hence, Brittle failure occours.

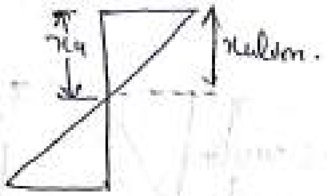
→ This section is ~~strictly~~ strictly prohabited in IS code.

→ If the existing beam is found to be overreinforced section then reduce the moment carrying ~~capaci~~ capacity upto balanced section. By putting ($x_u = x_{ulim}$).

Type Of RCC Section   LSM Method

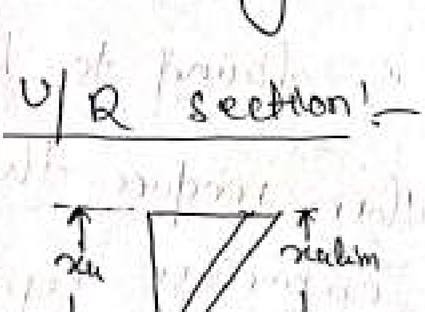
1. Balanced Section :-

$\boxed{x_u = x_{ulim}}$



- Balanced section is used to design the beam only.
- Theoretically it is possible but practically it is not possible.
- In this section steel and concrete reaches to its ~~poo~~ permissible limit ~~same~~ at same time.
- Even in ~~balan~~ balanced section concrete fail suddenly.

2. U/R section :-



$\boxed{x_u < x_{ulim}.}$

- Best section for design.
- In this section steel reache to its permissible limit before ~~of~~ ~~fc~~ concrete.
- steel fail first.

Design of Singly Reinforced Beam:- [Lec - 11]

"always balanced section is used to design of beam."

Step-1 Assume the ~~ecto~~ Given section will be balanced section. $\boxed{\therefore x_u = x_{ulim}}$ hence.

Step-2 Calculate the, ~~depth~~ effective depth provided (d_{prov}) of beam by using ~~depth~~ ~~to~~ span to effective depth ratio.

IS 456-page-no-37 - cl-no- 23.2.1 (a).

Assume a trail section $\rightarrow \left[D = \frac{span}{10} \right]$

Step-3 Calculation of Design Moment (If not given) or factored Moment.

(i) $W = W_L + W_D$

$\boxed{W_u = 1.5\,(W_L + W_D)}$ or $\boxed{W_u = 1.5\,(DL + L.L)}$

(ii) $\boxed{M_u = \frac{W l_{eff}^2}{8}}$

Step-4 Calculate the effective depth ~~provided~~ required (d_{req}) from (MR) comp eq^{n}s.

$\boxed{(MR)_{tension} = (M_R)_{comp} = (MR)_{bal}}$

for balanced section

$(MR)_{comp} = 0.36\, f_{ck}\, B\, x_{ulim} (d - 0.42\, x_{ulim})$

$(MR)_{comp} = 0.36\, f_{ck} \cdot B \cdot K \cdot d\, (d - 0.42\, K \cdot d)$ $\{\because x_{ulim} = K \cdot d$

$(MR)_{comp} = 0.36 f_{ck} B K d^2 (1 - 0.42 K)$

here $(1 - 0.42 K) \rightarrow j \rightarrow$ lever arm co-efficent.

$(MR)_{comp} = 0.36 f_{ck} B K \cdot d^2 (j)$

$(MR)_{comp} = [0.36 \cdot f_{ck} \cdot K j] B \cdot d^2$

here

$0.36 f_{ck} \cdot K \cdot j \rightarrow Q.$

$(MR)_{comp} = Q B \cdot d^2$

**V

The value of Q is depends on grade of steel.

Grade of steel	Q
fe 250	$0.148 f_{ck}$
fe 415	$0.138 f_{ck}$
fe 500	$0.134 f_{ck}$

** Imp

Now, we calulate the value of d_{req} from eqn.

$$\boxed{d_{req} = \sqrt{\frac{(MR)_{comp}}{Q \cdot B}}}$$

*** Imp

Step-5

Compare d_{prov} & d_{req}.

If → $d_{prov} > d_{req}$ → (design is safe) (OK) for calculate A_{st}

If → $d_{prov} < d_{req}$ → (design unsafe)

→ Assume a greater value than d_{req} and redesign the beam.

Step-6

Calculate A_{st} by ~~from~~ tension side.

$$(MR)_{Tension} = 0.87 f_y A_{st} (d - 0.42 x_{ulim})$$

$$(MR)_{Tension} = 0.87 f_y A_{st} (d - 0.42K \cdot d)$$

$$(MR)_{Tension} = 0.87 f_y A_{st} \, d(1 - 0.42K)$$

$$(MR)_{Tension} = 0.87 f_y A_{st} \, d(j)$$

$$\boxed{A_{st} = \frac{(MR)_{Tension}}{0.87 f_y \, d \cdot j}}$$ ***

Step-8

Check for Maximum & Minimum Area of tension Reinforcement.

for minimum area of reinforcement

$$\boxed{\frac{A_s}{Bd} = \frac{0.85}{f_y}}$$

Maximum area of Reinforcement

$$A_{s\,max} = \boxed{0.04\, b \cdot D}$$

Step 7 Calculation of no. of bar (n)

no. of

$$\boxed{n = \frac{Ast}{ast}}$$

ast = area of reinforcement c/s or area of 1 bar.

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

* Calculate the Actual Area of steel.

Step-9

i) Check for strength in flexure.

$$\boxed{Mu_{lim} > Mu} \rightarrow \text{(safe in flexure)}$$

$$Mu_{lim} = \begin{cases} 0.1498\ f_{ck}\ b d^2 \rightarrow Fe\ 250 \\ 0.1389\ f_{ck}\ b d^2 \rightarrow Fe\ 415 \\ 0.1338\ f_{ck}\ b d^2 \rightarrow Fe\ 500 \end{cases}$$

(ii) for deflection control (deflection)

P-38

$$\boxed{f_s = 0.58\ f_y \left(\frac{Ast\ req}{Ast\ prov}\right)}$$

check modification factor → K

$$\left(\frac{L}{d}\right)_{max} > \left(\frac{L}{d}\right)_{prov.} \rightarrow \text{safe in deflection.}$$

f_s → stress on steel in service load

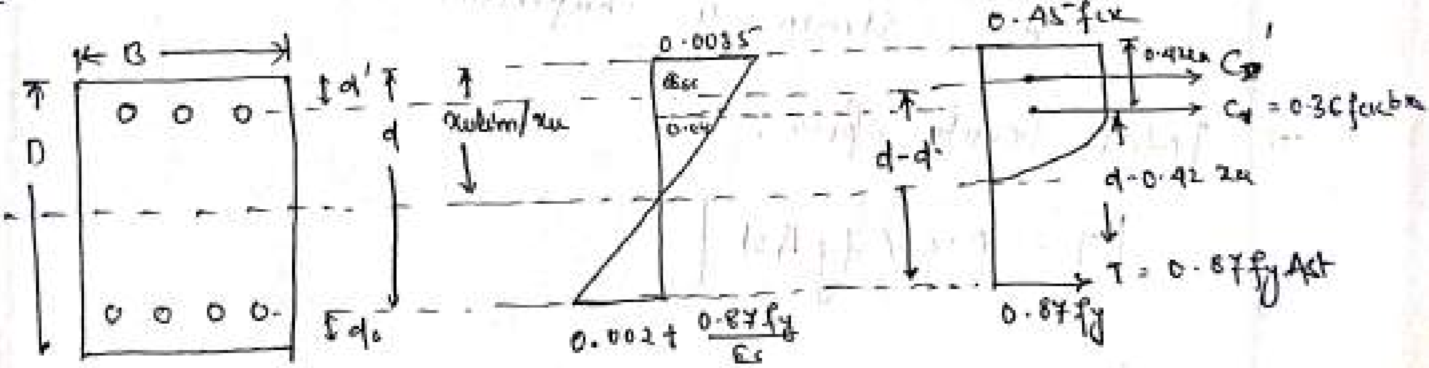
Doubly Rainforced Beam :-

When the beam are subjected to both tension and comp compression side of beam is known as doubly Rainforced beam.

If $Mu \leq Mulim$ → underRainforced section
↓
Design as S/R Beam.

If $Mu > Mulim$ → OverRainforced section
↓
Not advised to design by IS Code
↓
hence it will be designed as doubly Rainforced beam

#



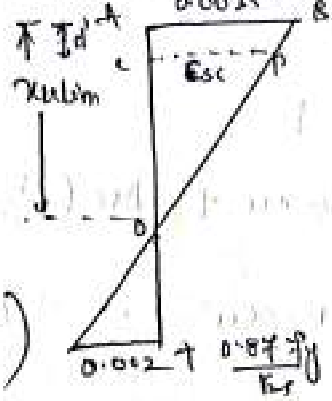
Strain in comp. steel.

from similarity of tringle.

ε_{sc} $\frac{AB}{AO} = \frac{CD}{CO}$

$$\frac{0.0035}{x_{ulim}} = \frac{\varepsilon_{sc}}{(x_{ulim} - d')}$$

$$\varepsilon_{sc} = 0.0035\left(\frac{x_{ulim} - d'}{x_{ulim}}\right)$$



Calculation of f_{sc}.

$\boxed{\text{Page no- 96 \& 70}}$

Total Compressive force.

$C_1 = 0.36 f_{ck} B \cdot x_u - f_{cc} \cdot A_{sc}$

$C_2' = f_{sc} \cdot A_{sc}$

$C = C + C'$

$C = 0.36 f_{ck} B \cdot x_u - f_{cc} \cdot A_{sc} + f_{sc} A_{sc}$

$= 0.36 f_{ck} B \cdot x_u + f_{sc} \cdot A_{sc} - f_{cc} \cdot A_{sc}$

$= 0.36 f_{ck} B \cdot x_u + A_{sc} (f_{sc} - f_{cc})$

$$\boxed{C = 0.36 f_{ck} B \cdot x_u + A_{sc} (f_{sc} - 0.45 f_{ck})}$$

Where,

$f_{cc} \Rightarrow$ stress in compression concrete at level of steel. $= 0.45 f_{ck}$

$f_{sc} \Rightarrow$ stress in compression steel.

A_{sc} = Area of steel in compression.

ε_{sc} = Strain in compression steel.

Total tensile force.

$$\boxed{T = 0.87 f_y A_{st}}$$

Depth of neutral Axis

by equating the tension force to comp force:

$C = T$

$0.36 f_{ck} B \cdot x_u + A_{sc} (f_{sc} - 0.45 f_{ck}) = 0.87 f_y A_{st}$

$0.36 f_{ck} B \cdot x_u = 0.87 f_y A_{st} - A_{sc} (f_{sc} - 0.45 f_{ck})$

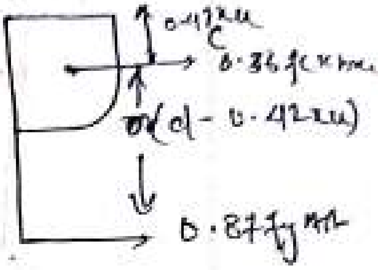
$$\boxed{x_u = \frac{0.87 f_y A_{st} - A_{sc}(f_{sc} - f_{cc})}{0.87 f_y A_{st}}}$$

Moment of Resistance :-

Compression side

MR_1 : C x Lever arm.

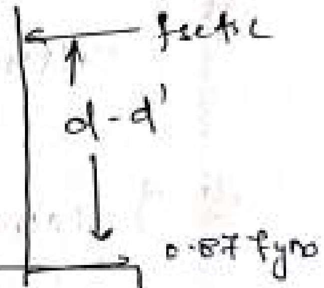
$= 0.36 f_{ck} b \, x_u (d - 0.42 x_u)$



MR_2 = C' x Lever arm.

$= \cancel{f_{sc} A_{sc}}$

$A_{sc}(f_{sc} - f_{cc}) \times (d - d')$



Comp side

$$\boxed{MR = 0.36 f_{ck} B x_u (d - 0.42 x_u) + A_{sc}(f_{sc} - f_{cc})(d - d')}$$

Tension side

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

Analysis of D/R Beam

step-1
Calculation of fsc (from IS 456 page no-96 (70))
(& SP-16 ~~table no~~ page no-14)

step-2
Determine the actual depth of NA (x_u)

$$\boxed{x_u = \frac{0.87 f_y A_{st} - f_{sc} A_{sc}}{0.36 f_{ck} B}}$$

step-3
Determine value of x_{ulim}. (Same as S/R Beam)

step-4 Compare the ~~section~~ x_u to x_{ulim}.

$x_u > x_{ulim} \rightarrow$ O/R section.
$x_u = x_{ulim} \rightarrow$ Balanced section.
$x_u < x_{ulim} \rightarrow$ U/R section.

step-5 Calculate Moment of Resistance (MR)

$$\left[\begin{aligned} (MR)_{comp} &= 0.36 f_{ck} B \cdot x_u (d - 0.42 x_u) + A_{sc}(f_{sc} - f_{cc})(d - d') \\ (MR)_{tension} &= 0.87 f_y A_{st} (d - 0.42 x_u) \end{aligned}\right]$$

Fe 415   M

$$\frac{M}{I} = \frac{E}{R} = \frac{\sigma}{y}$$

$$\boxed{\frac{M}{I} = \frac{\sigma}{y}}$$

$$M_{cr} = \sigma_{cr} \frac{I_T}{y_T}$$

section modulus

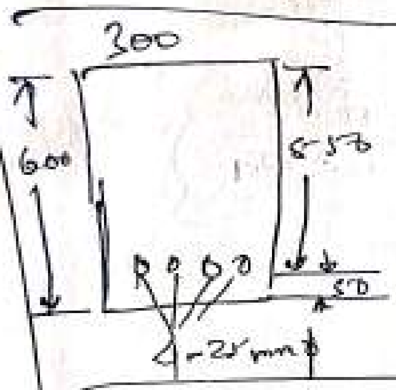
y_T : distance between top in extreme tension fiber

I_T = second moment of area of section (MI)

$m = 13.33$, $\sigma_{st} = 230$

$\sigma_{cbc} = 7$.

$\boxed{M_{max} = 116 \text{ kNm}}$



Example - 4.1

~~M20 Fe415~~

M20
Fe 415

$$^{14}C_2 = \frac{14!}{(14-2)!\ 2!}$$

$$= \frac{14!}{12! \times 2!}$$

$$= \frac{14 \times 13 \times \cancel{12!}}{\cancel{12!} \times 2!}$$

$$= \frac{182}{2} = 91 \text{ Ans}$$

SD-1 Lect-5

Design of Singly Reinforced beam (WSM) :-

* Assume the section will be balanced section.

* Assume : $\boxed{B = \frac{D}{2}}$

step-1. Calculate ~~the~~ depth from Assumption.

$$B = \frac{D}{2}$$

$$\boxed{D = 2B}$$

step-2 Calculate total ~~live load~~ ~~load~~ Moment due to a given Load.

$$\boxed{\text{total Load} = \text{live load} + \text{dead Load}}$$

calculate total Moment. (If not given)

Step-3 Calculate effective depth of beam (d) from compression side, & ~~Area of steel (Ast) from~~

Step-4 Check.

Case-1 If (d) assumed $\geq$ (d) designed ~~where it~~

then beam is (safe)

then calculate the Area of steel from tension side.

Case-2 (d) assumed < (d) design

Section is not safe.

~~Construct the section~~

Redesign the ~~sel~~ beam by taking depth is ~~more~~ (d design)

follow step-2 & 3

again then calculate Ast from tension side. then ~~follow step-2 & 3~~ to calculate Ast from tension from comp side

$$\boxed{d = \sqrt{\frac{(MR)_{comp}}{Q \cdot B}}}$$ **

$$\boxed{x = k \cdot d}$$

$$Q = \frac{1}{2}\sigma_{cbc}\, j\, k$$

$j \rightarrow \left(1 - \frac{k}{3}\right)$ lever arm constant.

$k \leftarrow \frac{280}{280 + 3\sigma_{st}}$ or $\frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$

from tension side

$(MR)_t$

$$\boxed{A_{st} = \frac{(MR)_{Tension}}{\sigma_{st} \cdot j \cdot d}}$$ **

Calⁿ of d from the comp side

$$(MR)_{comp} = \frac{1}{2} \cdot \sigma_{cbc} \cdot B x_c \left(d - \frac{x_c}{3}\right)$$

$$(MR)_{comp} = \frac{1}{2} \sigma_{cbc} \cdot B \cdot K \cdot d \left(d - \frac{Kd}{3}\right) \quad (\because x_c = K \cdot d)$$

$$(MR)_{comp} = \frac{1}{2} \sigma_{cbc} \cdot B \cdot K \cdot d^2 \left(1 - \frac{K}{3}\right)$$

$$(MR)_{comp} = \frac{1}{2} \cdot \sigma_{cbc} \cdot B \cdot K \cdot j \cdot d^2 \quad \because \underbrace{1 - \frac{K}{3} = j}_{\text{lever arm constant}}$$

$$\Rightarrow (MR)_{comp} = Q \cdot B \cdot d^2 \qquad \boxed{\frac{1}{2} \sigma_{cbc} \cdot K \cdot j = Q}$$

$$\boxed{d_{min} = \sqrt{\frac{(MR)_{comp}}{Q \cdot B}}}$$

$$\left\{Q = \frac{1}{2} \sigma_{cbc} K \cdot j\right\}, \left\{j = 1 - \frac{K}{3}\right\} \left\{K = \frac{280}{280 + 3\sigma_{st}}\right\}$$

Calⁿ of "Ast" from tension side.

$$(MR)_{tension} = \sigma_{st} \, A_{st} \left(d - \frac{x_c}{3}\right)$$

$$\Rightarrow (MR)_{tension} = \sigma_{st} \cdot A_{st} \left(d - \frac{Kd}{3}\right)$$

$$\Rightarrow (MR)_{Tension} = \sigma_{st} \cdot A_{st} \cdot d \left(1 - \frac{K}{3}\right) \rightarrow j$$

$$\Rightarrow (MR)_{Tension} = \sigma_{st} \cdot A_{st} \cdot d \, j$$

$$\cancel{A_{st} = \frac{(MR)_{Tension}}{\sigma_{st} \cdot d \cdot j}}$$

$$\boxed{A_{st} = \frac{(MR)_{tension}}{\sigma_{st} \, d \, j}} \qquad \boxed{j = 1 - \frac{K}{3}}$$

Singly Reinforced Beam (WSM Method) (Lec-4)

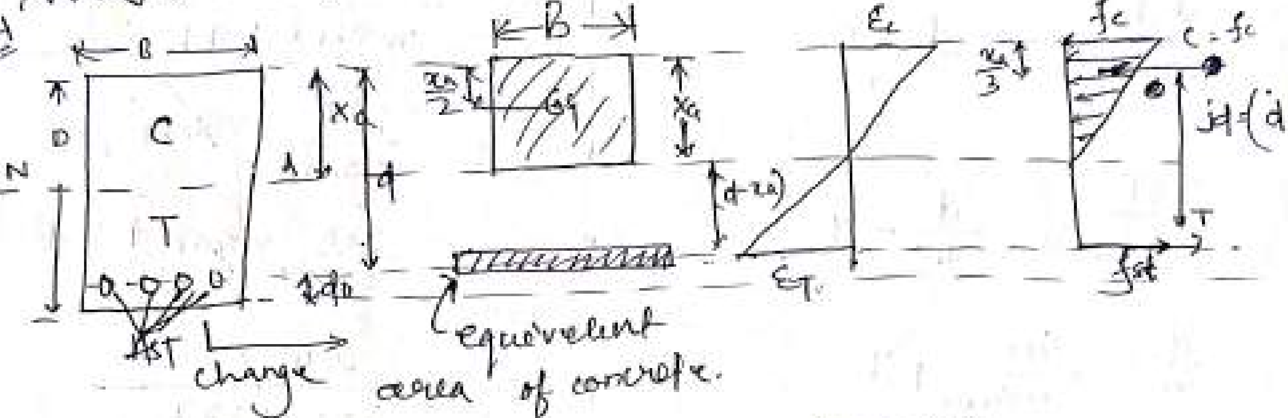
(W.S.M Method)

* In singly reinforced beam the reinforcement is provided in single side of N.A (In tension side).

* NA → Neutral Axis → It is the axis which divides the beam into 2 section "tension & compression".

* Analysis of Single Reinforced beam :-

Step 1



Step 2: Calculation the depth of neutral axis (x_a)

Method 1: depth of Neutral axis is calculated by equating the moment of area of both side of NA.

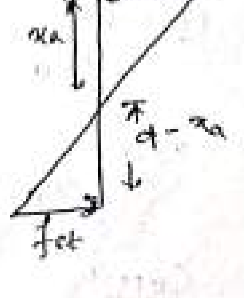
$$\boxed{\text{Moment of area of concrete} = \text{Moment of Area of steel}}$$

$$\therefore \boxed{[B \cdot x_a]\left(\frac{x_a}{2}\right) = mA_{st}\,(d - x_a)}$$

Area of concrete. ⊥r dist. Area of equivalent steel concrete. ⊥r distance.

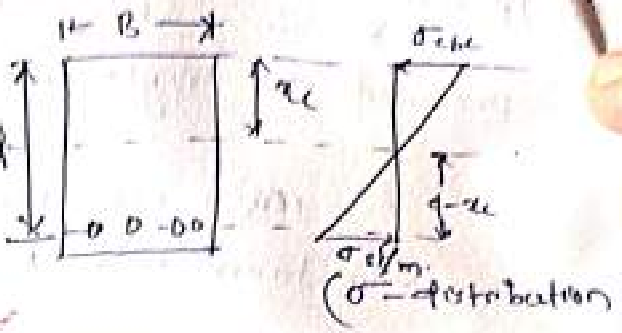
Method 2: It also calculated as application of similar triangle on stress diagram.

$$\boxed{\frac{f_c}{x_a} = \frac{f_{st}}{d - x_a}}$$



Step-2
Calculate the critical depth of neutral axis (x_c)

$$\boxed{\frac{\sigma_{cbc}}{x_c} = \frac{(\sigma_{st}/m)}{d - x_c}}$$



$$\frac{x_c}{m\sigma_{cbc}} = \frac{d - x_c}{\sigma_{st}}$$

$$\frac{\sigma_{st}}{m\sigma_{cbc}} = \frac{d - x_c}{x_c}$$

$$\frac{\sigma_{st}}{m\sigma_{cbc}} = \frac{d}{x_c} - 1$$

$$\frac{d}{x_c} = \frac{\sigma_{st}}{m\sigma_{cbc}} + 1$$

$$\frac{d}{x_c} = \frac{\sigma_{st} + m\sigma_{cbc}}{m\sigma_{cbc}}$$

$$\frac{x_c}{d} = \frac{m\sigma_{cbc}}{\sigma_{st} + m\sigma_{cbc}}$$

$$\boxed{x_c = \underbrace{\left(\frac{m\sigma_{cbc}}{\sigma_{st} + m\sigma_{cbc}}\right)}_{k} \times d}$$ Imp

$$\boxed{x_c = K d}$$

$$\boxed{K = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}}$$ Imp

$$= \frac{\frac{280}{3\sigma_{cbc}} \times \cancel{\sigma_{cbc}}}{\frac{280}{3\cancel{\sigma_{cbc}}} \times \cancel{\sigma_{cbc}} + \sigma_{st}}$$

$$\Rightarrow \frac{280/\cancel{3}}{\frac{280 + 3\sigma_{st}}{\cancel{3}}}$$

$$\boxed{K = \frac{280}{280 + 3\sigma_{st}}}$$

Imp — Critical depth of neutral axis is the depth of neutral axis at balanced section.

Step-3: $\boxed{\text{Compare } x_a \text{ \& } x_c}$

Case-1: $x_a = x_c$ → balance section. $\boxed{f_{st} = \sigma_{st}}$, $\boxed{f_c = \sigma_{cbc}}$

$\boxed{f_{st} = T_a, \; f_{sc} = C_a}$

Case-2: $x_a > x_c$ → over reinforced section. $\boxed{f_{st} < \sigma_{st}}$, $\boxed{f_c = \sigma_{cbc}}$

Case-3: $x_a < x_c$ → Under reinforced section. $\boxed{f_{st} = \sigma_{st}}$, $\boxed{f_c < \sigma_{cbc}}$

Step-9 : Calculation of Moment of resistance (MR) :-

(MR) → Maximum bending Moment that a beam can take without fail is known as "Moment of Resistance".

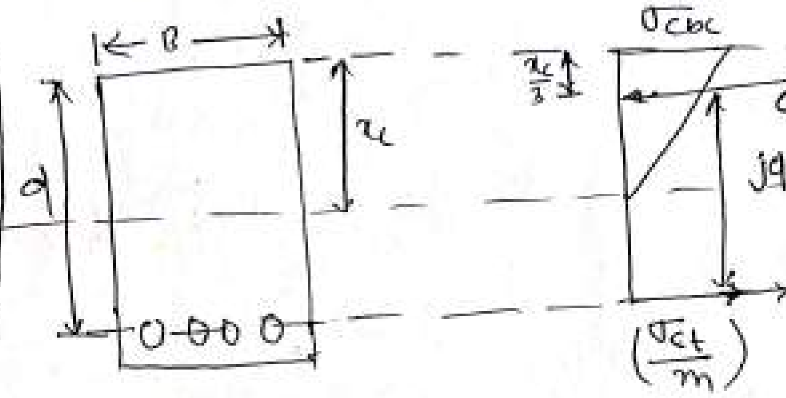
"BM" → Moment coming on beam due to external Load.

Case-1

for balanced section !—

$\boxed{x_a = x_c}$

The MR should be calculated from both tension & compression side.



$C = \frac{1}{2} \times \sigma_{cbc} \times B \times x_c$

$jd = \left(d - \frac{x_c}{3}\right)$

$T = \frac{\sigma_{st}}{m} \times m A_{st}$

$= \sigma_{st} A_{st}$

MR cal^n from Compression Side !—

$(MR)_{comp} =$ Comp. force × lever arm

$$\boxed{(MR)_{comp} = \frac{1}{2}\sigma_{cbc} B \cdot x_c \left(d - \frac{x_c}{3}\right)} \;**$$

comp. force

= Avg. comp stress × Area

$= \left(\frac{\sigma_{cbc} + 0}{2}\right) \times B \times x_c$

$\boxed{C = \frac{1}{2}\sigma_{cbc} B x_c}$

MR cal^n from Tension Side !—

$(MR)_{tension} =$ tensile force × lever arm.

$$\boxed{(MR)_{tension} = \sigma_{st} A_{st} \left(d - \frac{x_c}{3}\right)} \;**$$

tensile force

= tensile stress × Area.

$= \frac{\sigma_{st}}{m} \times m A_{st}$

$\boxed{T = \sigma_{st} \cdot A_{st}}$

Case-2

for under reinforced section !—

$\boxed{x_a < x_c}$ → MR should be calculated from tension side.

$$\boxed{(MR)_{tension} = \sigma_{st} A_{st} \left(d - \frac{x_a}{3}\right)}$$

Numerical-1

find the Moment of resistance of ~~the~~ a beam of size 400 × 600 mm, effective cover = 50 mm, steel provided = 4-16mm ϕ, use M_{25} & Fe 415 grade of concrete and steel.

Soln

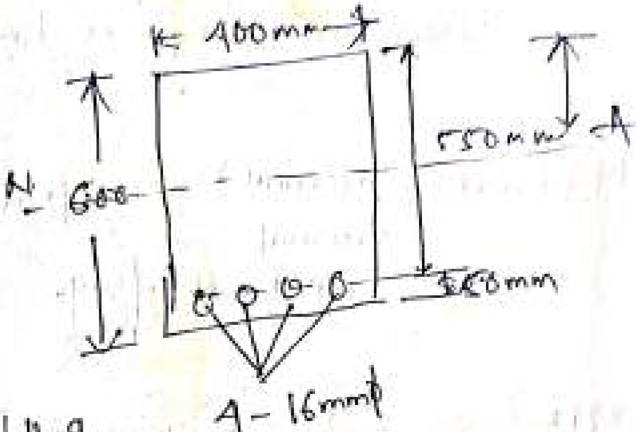
Data given

$f_{ck} = 25$ mpa

$\sigma_{cbc} = 8.5$ mpa

$f_y = 415$ mpa

$\sigma_{st} = 230$ mpa



$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 8.5} = 10.9$

$= 11$

$B = 400$ mm

$d = 550$ mm

step 1 x_a caln

$$B x_a \times \frac{x_a}{2} = m A_{st} (d - x_a)$$

$$\Rightarrow \frac{400}{2} \times x^2 = 11 \times \frac{\pi}{4} \times (16)^2 \times 4 \left(550 - x_a\right)$$

44.23

$$\Rightarrow 200 x_a^2 = 304106 - 553 x_a$$

$$\Rightarrow \cancel{x_a^2 + 2.765 x_a - 1520.53 = 0}$$

$$x_a^2 + 44.23 x_a - 24328.5 = 0$$

$$\Rightarrow x_a = 135.42 \text{ mm}$$

step-2 Caln x_c

$$x_c = k \cdot d \qquad \because k = \frac{280}{280 + 3\sigma_{st}}$$

$$= \frac{280}{280 + 3 \times 230} = 0.288$$

$= 0.288 \times 550$

$= 158.76$ mm

BOOK

(1st) # Assumption in WSM Method:—

1. At any cross section, plane before bending, remain plane after bending.

2. All tensile stresses are taken by the Reinforcement and non by concrete. except otherwise partially permeated.

3. ~~Stress & strain variation is~~ Stress in concrete varies linearly from zero at neutral axis ~~and~~ to a maximum at extream fiber.

4. Stress are linearly proportional to Strain for both. concrete and steel. (Hooks law is valid)

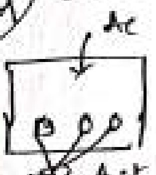
5. Bond between ~~the~~ steel and concrete is perfect. with in elastic limit.

6. The modular Ratio, $m = \frac{E_s}{E_c}$, has the value

$$\boxed{m = \frac{280}{3\sigma_{cbc}}}$$ ***

$\boxed{\sigma_{cbc} \rightarrow \text{permissiable stress of concrete at Bending}}$

(2nd) Note Total Load = concrete + steel



$P = P_c + P_s$

$$\boxed{P = \sigma_c A_c + \sigma_s A_s} \quad -(1)$$

Any elongation $\Delta L_c = \Delta L_s$ [for composite. section]

hence $\frac{P_c L_c}{A_c E_c} = \frac{P_s L_s}{A_s E_s}$

$\frac{\sigma_c}{E_c} = \frac{\sigma_s}{E_s}$

$\frac{E_s}{E_c}\sigma_c = \sigma_s \Rightarrow \boxed{m\sigma_c = \sigma_s}$

$\Rightarrow \boxed{\sigma_c = \frac{\sigma_s}{m}} \quad -(2) \quad \because \frac{E_s}{E_c} = (m)$

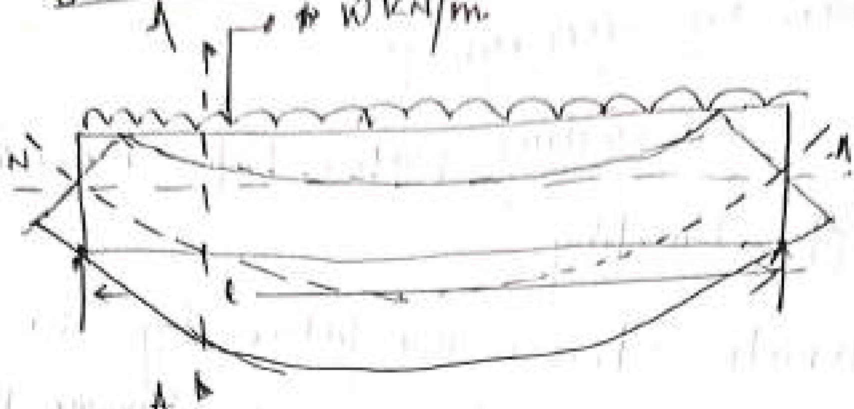
Put $\boxed{\sigma_s = m\sigma_c}$

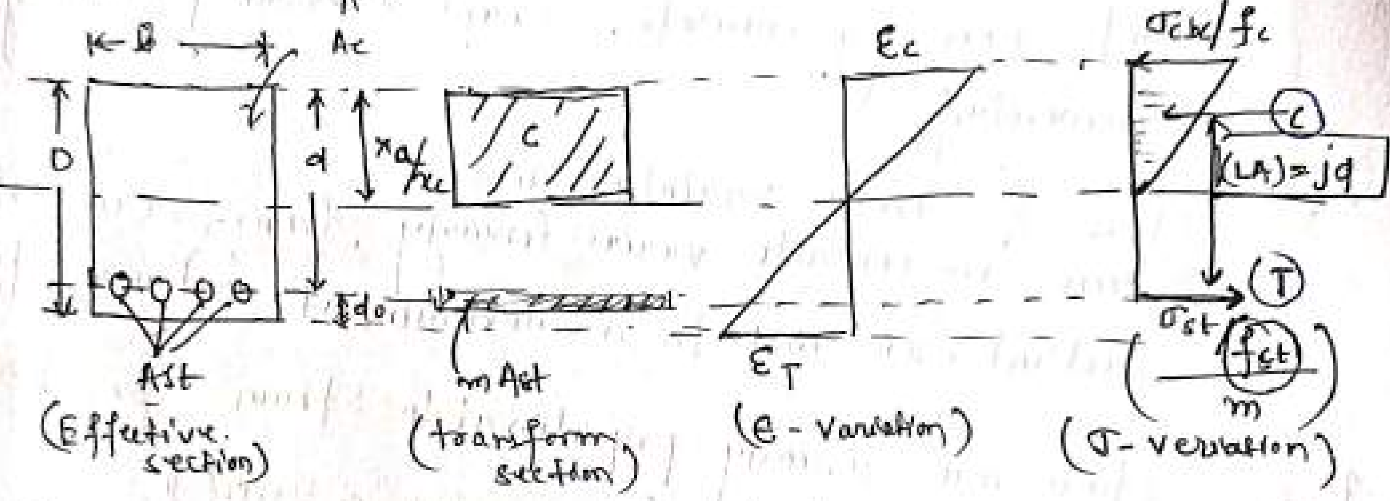
$P = \sigma_c A_c + m\sigma_c A_s$

$$\boxed{P = \sigma_c (A_c + mA_s)} \quad -(3)$$

Bending * flexural Member ... Beam

(2)





where

B = width / breadth of beam.

D = depth of ~~section~~ beam / overall depth of ~~ck~~ Beam.

d = ~~depth of~~ effective depth	
d_0 = Clear Cover.	jd = Lever arm.
Ast = Area of steel.	C = Total compression force.
Ac = Area of concrete.	T = Total tensile force.
x_a = actual depth of neutral axis	L = length of beam.
x_c = critical depth of neutral axis.	
m = Modular ratio.	
ε_c = strain in concrete.	
ε_T = strain in steel.	

σ_{cbc} = Permissible stress of concrete due to bending compression.

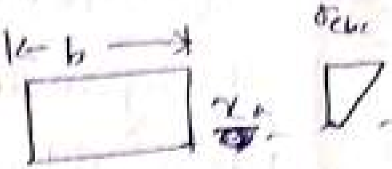
σ_{st} = Permissible stress in steel in tension.

f_c = stress in concrete.

f_{st} = stress in steel ~~subjected~~ subjected to combine axial ~~bending~~ Load and banding ?

(Cap^n)

* total compressive force (C)

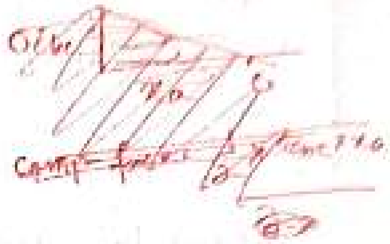


Comp. ~~force~~: comp. stress = $\frac{\text{Comp force}}{\text{Area}}$

$$\boxed{\text{comp force} = \text{comp. stress} \times \text{Area}}$$

$C = \sigma_{cbc} \times b \times x_a$

$$\boxed{C = b\, x_a \times \sigma_{cbc}}$$



* total tensile force (T)

tensile stress = $\frac{\text{tensile force}}{\text{Area}}$

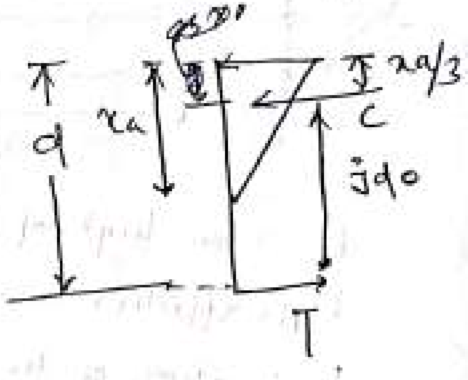
$$\boxed{\text{tensile force} = \text{tensile stress} \times \text{Area}}$$

$$T = \left(\frac{\sigma_{st}}{m}\right) \times (m \cdot A_{st})$$

$$\boxed{T = \sigma_{st} \cdot A_{st}}$$

* Lever arm (jd):—

$$\boxed{jd = \left(d - \frac{x_a}{3}\right)}$$



Definitions

* Actual depth of neutral Axis (x_a):—

It is the depth of neutral Axis which divide the overall section in compression and tension.

* Critical depth of neutral Axis (x_c):—

It is the depth of neutral axis when the steel and concrete fails simultaneously at that time overall structure fails.

$\sigma_{cbc} \rightarrow$ Permissible stress of concrete.

$\sigma_{st} \rightarrow$ permissible stress of steel.

FOR WSM

Permissible stress of concrete (σ_{cbc}) :-

$\boxed{\text{I.S } 456 : 2000}$ Page-81

$\boxed{\text{table no- 21}}$

Permissible stress of steel (σ_{st}) :-

$\boxed{\text{IS } 456 : 2000}$ Page-82

$\boxed{\text{table no. 22}}$

Note table

$\boxed{\sigma_{cbc} = \frac{f_{ck}}{\text{safety (FOS) concrete}}}$ WSM

cal^n $\therefore m = \frac{280}{3\sigma_{cbc}}$

$= \frac{280}{3 \times 5}$

$= 19$

① (σ_{cbc} 4m)

Grade of Concrete	σ_{cbc} $\left(\frac{f_{ck}}{3}\right)$	m
M15	5	19
M20	7	13.33
M25	8.5	11
M30	10	9
M35	11.5	8
M40	13	7

② (σ_{st})

	Mild Steel (N/mm²)	Medium Steel	HYSD Bar (N/mm²)
In tension (a) for ϕ upto 20 mm	140	190	230
(b) ϕ > 20mm.	130		
In Compression	130	130	190

Lec-2 D1- 07/01/21

WSM

Modular Ratio (m) :-

In case of the working stress analysis of reinforced RCC section, it is usually to transform the composite section into an equivalent concrete section.

Accordingly, for the RCC, the modular ratio 'm' is defined as the ratio of the elastic modulus of steel (Es) to Elastic Modulus of concrete (Ec)

$$\boxed{m = \frac{E_s}{E_c}}$$

$$\boxed{E_s = 2 \times 10^5 \ N/mm^2}$$

As per : Cl. 6.2.3.1 (IS 456:2000)* (Page-16)

$$\boxed{E_c = 5000\sqrt{f_{ck}}}$$

← short term Modular ratio (without consideration of creep)

• According to WSM :-

~~$\frac{280}{3\sigma_{cbc}}$~~

$$\boxed{m = \frac{280}{3\sigma_{cbc}}}$$

where $\sigma_{cbc} \rightarrow$ permissible stress in concrete in compression.

PERMISSIBLE STRESS

Permissible stress is the maximum stress a material can withstand without failing under a given Loading condition. It is also ~~considered~~ known as Working stress.

used the horizontal distance between bars of a group may be reduced to two-thirds the nominal maximum size of the coarse aggregate, provided that sufficient space is left between groups of bars to enable the vibrator to be immersed.

c) Where there are two or more rows of bars, the bars shall be vertically in line and the minimum vertical distance between the bars shall be 15 mm, two-thirds the nominal maximum size of aggregate or the maximum size of bars, whichever is greater.

26.3.3 *Maximum Distance Between Bars in Tension*

Unless the calculation of crack widths shows that a greater spacing is acceptable, the following rules shall be applied to flexural members in normal internal or external conditions of exposure.

a) *Beams* — The horizontal distance between parallel reinforcement bars, or groups, near the tension face of a beam shall not be greater than the value given in Table 15 depending on the amount of redistribution carried out in analysis and the characteristic strength of the reinforcement.

b) *Slabs*

1) The horizontal distance between parallel main reinforcement bars shall not be more than three times the effective depth of solid slab or 300 mm whichever is smaller.

2) The horizontal distance between parallel reinforcement bars provided against shrinkage and temperature shall not be more than five times the effective depth of a solid slab or 450 mm whichever is smaller.

26.4 Nominal Cover to Reinforcement

26.4.1 *Nominal Cover*

Nominal cover is the design depth of concrete cover to all steel reinforcements, including links. It is the dimension used in design and indicated in the drawings. It shall be not less than the diameter of the bar.

26.4.2 *Nominal Cover to Meet Durability Requirement*

Minimum values for the nominal cover of normal-weight aggregate concrete which should be provided to all reinforcement, including links depending on the condition of exposure described in **8.2.3** shall be as given in Table 16.

26.4.2.1 However for a longitudinal reinforcing bar in a column nominal cover shall in any case not be less than 40 mm, or less than the diameter of such bar. In the case of columns of minimum dimension of 200 mm or under, whose reinforcing bars do not exceed 12 mm, a nominal cover of 25 mm may be used. 200 or less

26.4.2.2 For footings minimum cover shall be 50 mm.

26.4.3 *Nominal Cover to Meet Specified Period of Fire Resistance*

Minimum values of nominal cover of normal-weight aggregate concrete to be provided to all reinforcement including links to meet specified period of fire resistance shall be given in Table 16A.

26.5 Requirements of Reinforcement for Structural Members

26.5.1 *Beams*

26.5.1.1 *Tension reinforcement*

a) *Minimum reinforcement*—The minimum area of tension reinforcement shall be not less than that

Table 15 Clear Distance Between Bars

(*Clause* 26.3.3)

f_y	Percentage Redistribution to or from Section Considered				
	- 30	- 15	0	+ 15	+ 30
	Clear Distance Between Bars				
N/mm²	mm	mm	mm	mm	mm
250	215	260	300	300	300
415	125	155	180	210	235
500	105	130	150	175	195

NOTE — The spacings given in the table are not applicable to members subjected to particularly aggressive environments unless in the calculation of the moment of resistance, f_y has been limited to 300 N/mm² in limit state design and σ_{st} limited to 165 N/mm² in working stress design.

such as increasing the length of lap and/or using spirals or closely-spaced stirrups around the length of the splice.

26.2.5.1 *Lap splices*

a) Lap splices shall not be used for bars larger than 36 mm; for larger diameters, bars may be welded (*see* **12.4**); in cases where welding is not practicable, lapping of bars larger than 36 mm may be permitted, in which case additional spirals should be provided around the lapped bars.

b) Lap splices shall be considered as staggered if the centre to centre distance of the splices is not less than 1.3 times the lap length calculated as described in (c).

c) Lap length including anchorage value of hooks for bars in flexural tension shall be L_d (*see* **26.2.1**) or 30ϕ whichever is greater and for direct tension shall be $2L_d$ or 30ϕ whichever is greater. The straight length of the lap shall not be less than 15ϕ or 200 mm. The following provisions shall also apply:

Where lap occurs for a tension bar located at:

1) top of a section as cast and the minimum cover is less than twice the diameter of the lapped bar, the lap length shall be increased by a factor of 1.4.

2) corner of a section and the minimum cover to either face is less than twice the diameter of the lapped bar or where the clear distance between adjacent laps is less than 75 mm or 6 times the diameter of lapped bar, whichever is greater, the lap length should be increased by a factor of 1.4.

Where both condition (1) and (2) apply, the lap length should be increased by a factor of 2.0.

NOTE—Splices in tension members shall be enclosed in spirals made of bars not less than 6 mm diameter with pitch not more than 100 mm.

d) The lap length in compression shall be equal to the development length in compression, calculated as described in **26.2.1**, but not less than 24ϕ.

e) When bars of two different diameters are to be spliced, the lap length shall be calculated on the basis of diameter of the smaller bar.

f) When splicing of welded wire fabric is to be carried out, lap splices of wires shall be made so that overlap measured between the extreme cross wires shall be not less than the spacing of cross wires plus 100 mm.

g) In case of bundled bars, lapped splices of bundled bars shall be made by splicing one bar at a time; such individual splices within a bundle shall be staggered.

26.2.5.2 *Strength of welds*

The following values may be used where the strength of the weld has been proved by tests to be at least as great as that of the parent bar.

a) *Splices in compression* — For welded splices and mechanical connection, 100 percent of the design strength of joined bars.

b) *Splices in tension*

1) 80 percent of the design strength of welded bars (100 percent if welding is strictly supervised and if at any cross-section of the member not more than 20 percent of the tensile reinforcement is welded).

2) 100 percent of design strength of mechanical connection.

26.2.5.3 *End-bearing splices*

End-bearing splices shall be used only for bars in compression. The ends of the bars shall be square cut and concentric bearing ensured by suitable devices.

26.3 Spacing of Reinforcement

26.3.1 For the purpose of this clause, the diameter of a round bar shall be its nominal diameter, and in the case of bars which are not round or in the case of deformed bars or crimped bars, the diameter shall be taken as the diameter of a circle giving an equivalent effective area. Where spacing limitations and minimum concrete cover (*see* **26.4**) are based on bar diameter, a group of bars bundled in contact shall be treated as a single bar of diameter derived from the total equivalent area.

26.3.2 *Minimum Distance Between Individual Bars*

The following shall apply for spacing of bars:

a) The horizontal distance between two parallel main reinforcing bars shall usually be not less than the greatest of the following:

1) The diameter of the bar if the diameters are equal,

2) The diameter of the larger bar if the diameters are unequal, and

3) 5 mm more than the nominal maximum size of coarse aggregate.

not less than (Max value)

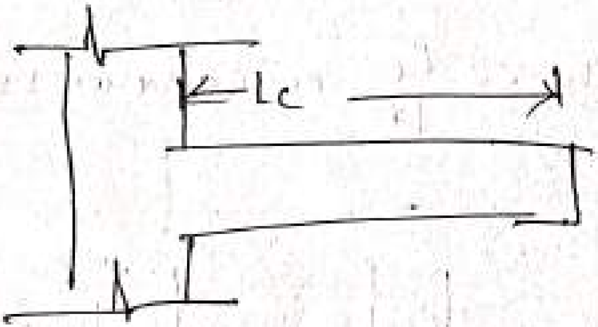
NOTE—This does not preclude the use of larger size of aggregates beyond the congested reinforcement in the same member; the size of aggregates may be reduced around congested reinforcement to comply with this provision.

b) Greater horizontal distance than the minimum specified in (a) should be provided wherever possible. However when needle vibrators are

Case-3

If beam is continuous over roller or rocker bearing, the effective span Leff is to center to center dist between bearing.

Cantilever

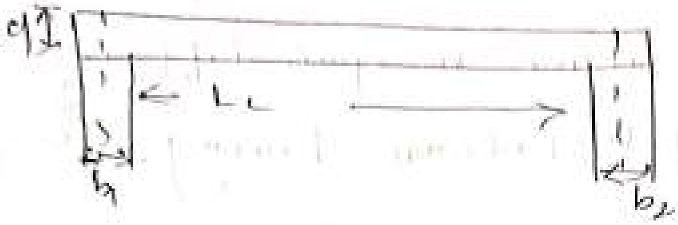


$$\boxed{l_{eff} = l_c + \frac{d}{2}}$$

Effective span of Beam / Slab :

→ portion that effectively participate in bending

SSB



b = width of support

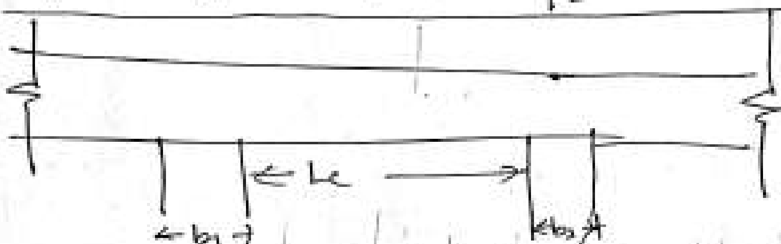
d : deep effective depth.

Lc : clear span.

$$L_{eff} = \min \begin{cases} L_c + d \\ \frac{b_1}{2} + L_c + \frac{b_2}{2} \end{cases}$$

Contineous

Case-1 If support width $< \frac{L_c}{12}$ or same as SSB.

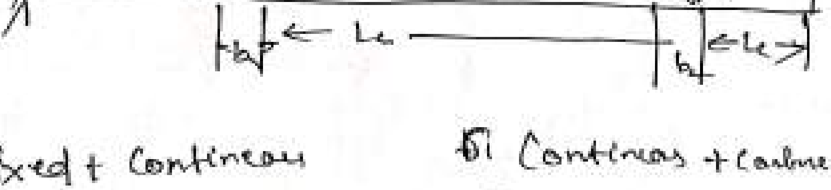


$$\boxed{b_1 + b_2 < \frac{L_c}{12}}$$

Leff = min

Case-2 If support width $> \frac{L_c}{12}$ or 600 mm.

$\{ b_1 + b_2 > \frac{L_c}{12}$ or 600 mm



fixed + Contineous

$$\boxed{L_{eff} = L_c}$$

Contineous + Contineous

$$\boxed{L_{eff} = L_c}$$

Cont. + free.

$$L_{eff} = \min \begin{cases} L_c + d/2 \\ L_c + \frac{b}{2} \end{cases}$$

* horizontal distance between parallel reinforcement bar against shrinkage and temperature.

$5 \times$ effective depth.
450 mm } not more than (min value)

c) 26.4 # Nominal Cover to Reinforcement :-

Table-no-16

26.4.1

* for all steel reinforcement shell not be less than dia of bar.

26.4.2.1

Longitudinal reinforcement bar in column. clear cover

(i) not be less then

40 mm.
less then the dia of bar }

(ii) If column minimum dimension $\leq$ 200 mm., dia of bar $\leq$ 12.

nominal cover $\rightarrow$ 25 mm (My wed?)

(iii) footing minimum cover $\rightarrow$ 50 mm.

for fire resistance clear cover $\Rightarrow$ Table - 16A

Codal provision   Lec- 37

Spacing of Reinforcement

26.3.2 ~~Minimum distance between~~

* Minimum horizontal between individual bar. Main Reinforcing bar :-

(a) dia. of bar (If dia are equal)
dia of Larger bar (If dia. are unequal)
nominal maximum size of aggregate + 5mm
} not less then. (max value)

(b) for needle vibrator is used.

[$\frac{2}{3}$ x nominal max. size of course aggregate.]

(c) for 2 or more raw of vertical bar:

$\boxed{\text{min. vertical distance}}$

15mm.
$\frac{2}{3}$ x max nominal size of agg.
} not less then (max value)

26.3.3
* Maximum distance between Bar in tension:-

(i) Beam → table - 15

(ii) Slab:-

* Main bars (horizontal distance)

3x effective depth of slab
300 mm
} not More then. (min. value)

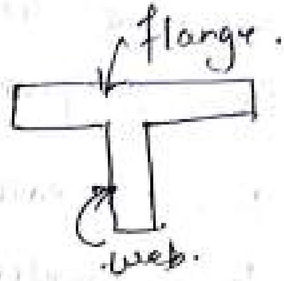
Compression Reinforcement :-

Cl-26.5.1.2 Maximum area of comp. reinforcement shall

$\boxed{\not> 0.04\, bD}$ or $\boxed{4\% \text{ of Area of beam}}$

T-Beam

Cl-1.3 web in beam exceed 750 mm.



$\boxed{0.1\% \text{ of web area}}$
↳ should not Less than.

Spacing → not exceeded → 300 mm
web thickness } whichever is (Less)

SHEAR Reinforcement

26.5.1.5 Maximum spacing of shear reinforcement :-

~~Along~~ ~~Along axis~~ Along Axis

not exceeded

0.75 d → vertical stirrups.	d → effective depth of section
d → Inclined stirrups. (45°)	

should not exceed 300 mm.

→ In no case spacing should not exceed 300 mm.

Minimum shear Reinforcement

$$\boxed{\frac{A_v}{b\,s_v} \geqslant \frac{0.4}{0.87\,f_y}}$$

$A_{sv} \rightarrow$ total area of stirrup.

$s_v \rightarrow$ spacing between stirrups.

$b \rightarrow$ breath of section.

$f_y \rightarrow$ characteristic strength of reinforcement.
(not greater then 415 Mpa)

After

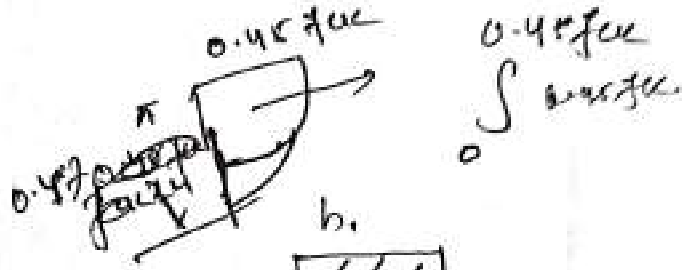
$x_u = x_1 + x_2$

$x_1 = x_u - x_2$

$\boxed{x_1 = \frac{3}{7} x_u}$ $\Rightarrow$ $\boxed{x_1 = 0.42\, x_u}$

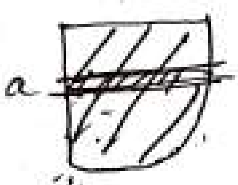
$\boxed{x_2 = \frac{4}{7} x_u}$

Tensile force (T):-



$f = \cancel{sp}$ stress $\times$ area.

$\boxed{T = 0.87 f_y A_{st}}$



area $= \frac{2}{3} \times a \times b$

Compressive force (C):-

Comp. force $\rightarrow$ stress $\times$ Area.

$C_1 = 0.45 f_{ck} \times b \times \frac{3}{7} x_u$ ——— eq^n(1)

$C_2 = 0.45 f_{ck} \times \frac{2}{3} \times \boxed{\frac{4}{7} x_u \cdot b}$ ——— eq^n (2)

Area

Now, we can calculate the combined Compressive force.

$C_{combined} = C_1 + C_2$

$C_c = 0.45 f_{ck} \times \frac{3}{7} x_u \times b + 0.45 f_{ck} \times \frac{2}{3} \times \frac{4}{7} x_u \cdot b$

$\boxed{C_c = 0.36 f_{ck}\, B \cdot x_u}$

Singly Reinforcement Beam :- [Lec-7]

"When R/f is provided only in tension side. It is called singly R/f beam."

Beam

↓ Analysis

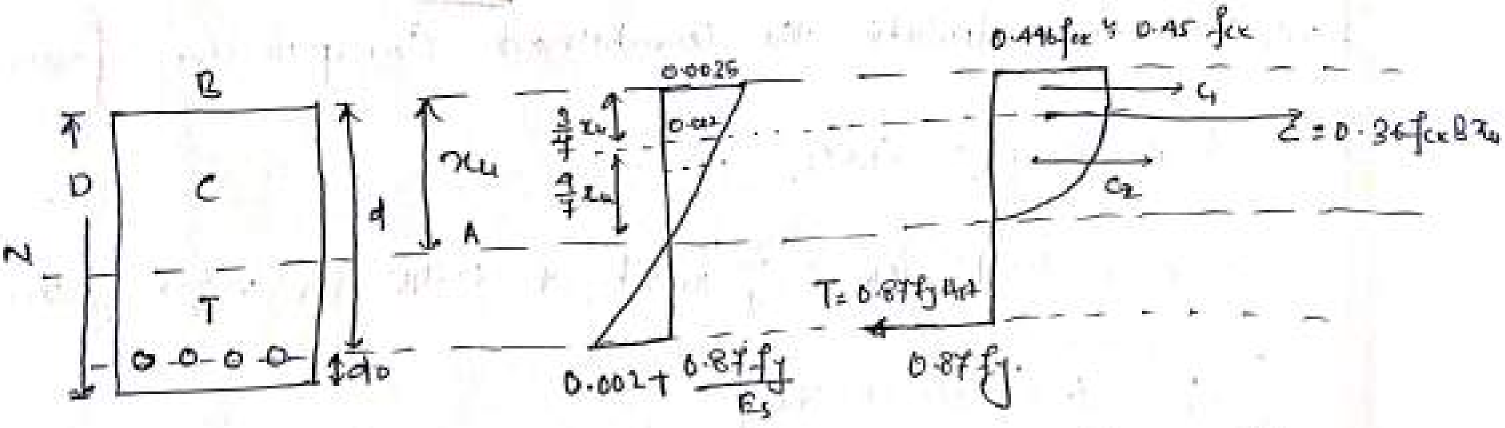
x- C/s dimension, M-, fc-, Ast.

Aim → Calculat. MOR.

↓ Design.

M-, fc-, Loading

Aim → Dimension, Area of steel.



(strain diagram)

(stress diagram)

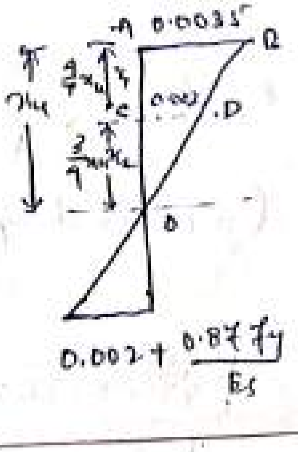
$\boxed{\text{Effective depth } (d) = D - 40}$

Analysis:-

$$\frac{AB}{OA} = \frac{CD}{OC}$$

$$\frac{0.0035}{x_u} = \frac{0.002}{x_u\,(x_2)}$$

$x_2 :$



$x_1 = 0.428\, x_u$

$x_2 = 0.571\, x_u$

(~~longitudinal~~ tension reinforcement) Clause no - ①. $\boxed{26 \cdot 5 \cdot 1 \cdot 1}$

Minimum of area of reinforcement. IS 456.

$$\frac{(A_{st})_{min}}{bd} = \frac{0.85}{f_y}$$

$(A_{st})_{min}$ = min area of steel in tension.

f_y = yield st. of steel.

b = width of steel.

d = effective depth of steel.

$$\left\{ (A_{st})_{min} = \frac{0.85}{f_y} \times bd \right.$$

$$= \frac{0.85}{250} \times bd$$

$$= 0.34\% \, bd$$

Grade of steel	$(A_{st})_{min}$
fe 250	0.34%. bd
fe 415	0.20% bd
fe 500	0.17 bd.

Maximum Area of reinforcement :-

$\boxed{A_{max} \not> 4\% \text{ of Beam Area}}$

$\boxed{A_{max} \not> 0.04 \, bD}$

Location of reinforcement

SD-1 Lec-6 Dt-13/02/21

CH-2

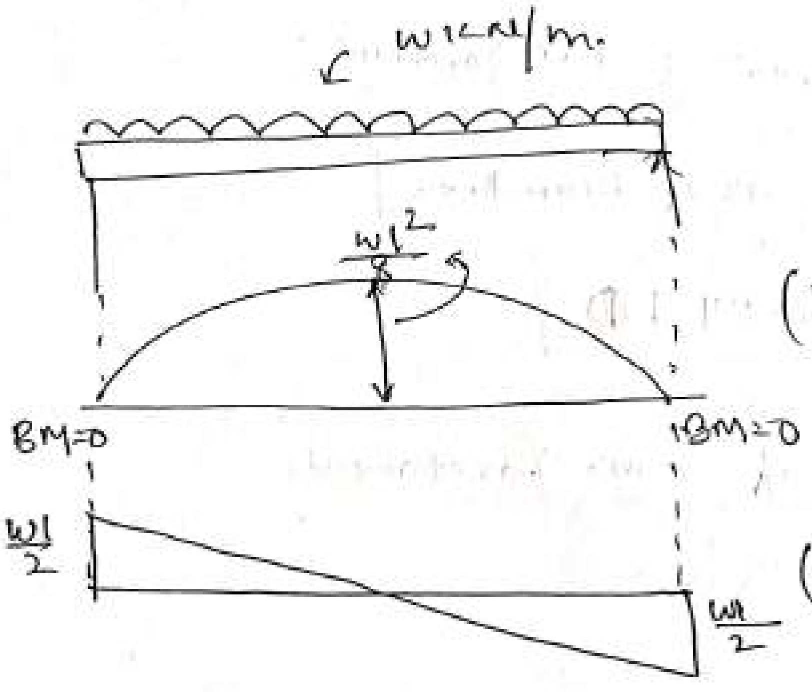
Limit State Method

It is a condition just before collapse upto the limit that member is safety to resist external load and also gives proper service throughout the life.

LSM

- Limit state of collapse (safe design)
  - → flexure
  - → shear
  - → compression
  - → shear
- Limit state of serviceability (long life)
  - → deflection
  - → cracking
  - → shrinking
  - → vibration.
  - → Durability
  - → creep
  - → fire resistance.

oad



(Bending Moment provide longitudinal reinforcement)

(due to shear force provide transverse reinforcement in the form of ~~stirru~~ stirrup)

~~For ...~~

In Limit State Method.

Safety factor we for both Material & Load.
so it is called partial safety factor.

In Working Stress Method:-
It only consider partial safety factor for Material only.

LSM	WSM
i) Concrete → 1.5	$(FOS)_{concrete} \to 3$
ii) ~~steel~~ Steel → 1.15	$(FOS)_{steel} \to 1.78 - 1.80$

$$(FOS)_{LSM} < (FOS)_{WSM}$$

[FOS → factor of safety]

RCC   Lec-1

CH-1 : Intro

① Plane Concrete / Plain Cement Concrete :—

PCC is produced by mixing approximate proportion of building materials such as cement (binding agent), aggregat gravel (Coarse Agg.), Sand (fine Agg), water, and admixture.

Advantages :

- high compressive strength.
- Durability
- Can be casted in various shape and size.
- Relatively economical and easy avilability of material.

Dis Advantages :

Remarkably weak in tension (tensile strength ≈ $\frac{1}{10}$th time of comp. strength.)

Binding Material — Cement; Bulkness of concrete — gravel; Filler — sand; Hydration — water; Enhance Extra property — Admixture

Concrete = Cement + graval + sand + water + Admixture.

Concrete: $\boxed{\gamma = 24\ kg/m^3}$

Cement: $\rho = 1440\ kg/m^3$, $V = 35\ Lt / 0.003478\ m^3$

Admixture → Chemical (1. Accelerator, 2. Retrader); Mineral (Surkhi, Pozzolana)

RCC → (Francois ~~Coignet~~ Coignet).

- It is a composite material with steel bars embaded in concrete, It has the ability to resist ~~sufficient~~ significant tensile stress in addition to compressive stress.
- The invention of RCC has made possible the construction of flexure mambers such as beam and slabs.

$\boxed{\rho = 2500\ kg/m^3}$

~~Tyrge~~ Type of load and beam! -

(2)

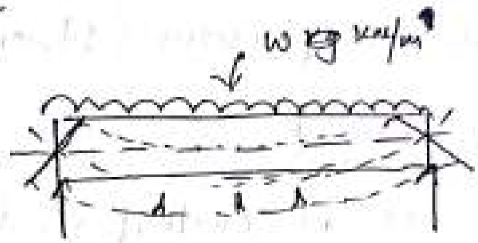
1. point/concentrated
2. UDL
3. UVL

1. cantilever beam.
2. Simply ~~of~~ supported.
3. continuous.
4. fixed
5. over hanging

$\boxed{PCC}$ ⇨ $\boxed{RCC}$

PCC:
* Strong in Compression
* Weak in tension
* Brittle ~~ep~~ failure

RCC:
* Strong in both tension and compression.
* Ductile failure



Any material that can take tension may be used Reinforcement. e.g. → steel, Aluminium, coppor, ~~fiber~~, fiber, Bamboo etc.

steel is most preferable because of following Reson

→ Economical as compared to other material.

→ High tensile strength.

→ Co-efficient of thermal Expension.

~~Strain~~ = ~~Change in length~~

Strain $\boxed{\varepsilon = \frac{\delta L}{L}}$

thermal stress $\boxed{\Delta L = L\alpha\Delta T}$

α of diff. material

$\alpha_{concrete} = 11 \times 10^{-6}/°C$

$\alpha_{steel} = 12 \times 10^{-6}/°C$

$\alpha_{copper} = 17 \times 10^{-6}/°C$

$\alpha_{aluminium} = 20 \times 10^{-6}/$

Concrete.

Brittle material.

$\boxed{\sigma_{cu} > \sigma_{tu} > \sigma_{su}}$

grade — M 20 (f_{ck})

M → Mix

20 → characteristic comp. strength @ 28 days curing. (N/mm²)

Steel.

Ductile material.

$\boxed{\sigma_{st} > \sigma_{cu} > \sigma_{u}}$

→ Mild steel (fe 250)

→ HYSD (High yield strength deformable bar)

fe 415 — fe 500

fe → (Iron)

415 → yield strength of steel. (f_y)

Grade of concrete

$\boxed{\text{IS 456 : 2000}}$

$\boxed{\text{Table-2}}$

1. Ordinary concrete → $M_{10} - M_{20}$
2. Standard concrete → M_{20} – M_{55}
3. High strength concrete → $M_{65} - M_{80}$.

IS preference

IS - 10262 → concrete mix design (Guide line)

IS - 456 - 2000 → Plain & Reinforced concrete.

IS - 875 (Part-1-5) : 1987 → Code of practice of design loads

part 1 → Dead load

part 2 → Imposed (Live load)

part 3 → Wind load.

part 4 → Snow load

part 5 → Special load and load combination.

Objectives of Structural Design:-

The design of a structure must satisfy following req:-

stability: It includes overturning, sliding, or ~~buckling~~, buckling of structure as a whole or part of it under the action of load.

strength:- It includes resisting the stresses included by various loads on the structural member.

~~stability:-~~

Serviceability:- It is ~~so~~ related to satisfy performance of structure under service load condition, ~~these include~~ ~~which will ham~~ these include hamper to economy.

Economy:- In order to fulfil above 3 criterial, a structural designer can design as over safe structure which will hamper the economy.

~~Asthet~~ Aesthetics:- Huge cost ~~is~~ are included in making Civil Engineering structure, so efforts should be made of make ~~as~~ aesthetics.

Design Methods of Concrete Structure:-

BIS → Bureau of Indian Standard.

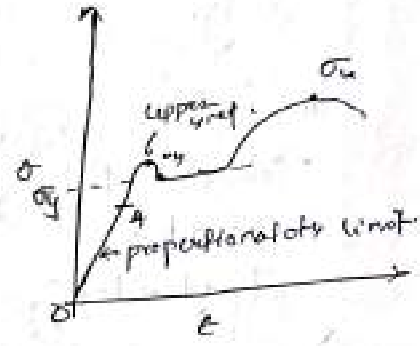
1. Working stress Method :- (IS : 1957)
2. Ultimate load Method :- (IS : 1964)
3. Limit state Method : [IS 456 : 1978]
   ↓ onward.

1. WSM Method :- (It does not considered reversal of stresses) → (fatigue)

On Elastic Limit.

" Stress within the elastic limit".

It mean $\boxed{\sigma_{design} < \sigma_y}$



→ stability (↑)
→ cost (↑) / uneconomical.

$\boxed{\text{used for → Dam, water tank, Bridge.}}$

2. ULM Method :-

"It considered full strength of material"

$\boxed{\sigma_{design} = \sigma_u}$

→ stability (↓)
→ Economical section.

3. LSM Method :- ~~design and analysis~~ of

→ Limit state of collaps → flexure, shear, compression, torsion
→ Limit state of serviceability → deformation, deflection, Cracking, shrinkage, Creep.

DESIGN OF DOUBLY REINFORCED BEAM :- 				Dt- 09/03/21.

(1)

In D/R Beam

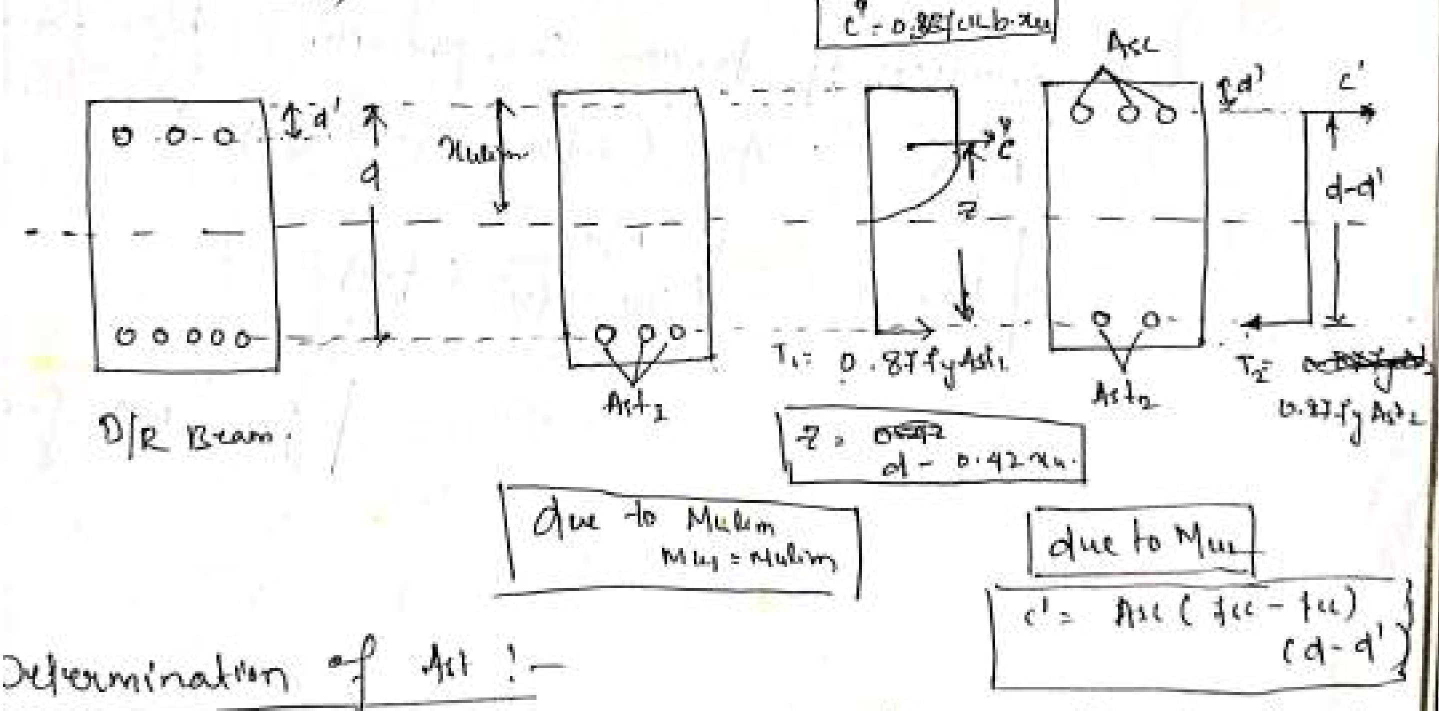
1. dimension of beam are already fixed or Restricted.
(B, D is fixed)

2. $\boxed{M_u > M_{ulim}}$

$\boxed{M_u = M_{ulim} + M_{u\,extra}}$

For designing always balanced section is used.
It means $\boxed{x_u = x_{ulim}}$

Aim → Calculation of area of steel in tension side (Ast) and area of steel in comp. side (Asc).



Determination of Ast :-
Area of steel corresponding to singly Reinforced.
balanced section : A_{st_1} 		$\boxed{A_{st} = A_{st_1} + A_{st_2}}$

Area of steel section-1

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

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

or

$$\boxed{A_{st_1} = \frac{M_{ulim}}{0.87 f_y . d . j}}$$

Moment of resistance of section 2.

$$\boxed{M_{u_2} = M_u - M_{ulim}}$$

$\underline{MR = Area \times LA}$

$$M_{u_2} = 0.87 f_y \cdot A_{st_2} (d - d')$$

$$\boxed{A_{st_2} = \frac{M_{u_2}}{0.87 f_y (d - d')}}$$ **

$$\boxed{A_{st} = A_{st_1} + A_{st_2}}$$

* Determination of ~~tension~~ Compression Reinforcement.

$$M_{u_2} = A_{sc} (f_{sc} - f_{cc}) (d - d').$$

$$\boxed{A_{sc} = \frac{M_{u_2}}{(f_{sc} - f_{cc})(d - d')}}$$ **

$$\boxed{f_{cc} = 0.45 f_{ck}}$$

Steps Involved in the design of doubly reinforced Beam

Given

Dimension of beam i.e. b×D or b×d and d'

Grade of concrete and type of steel.

Factored bending Moment (M_u).

Step-1

Determine the value of f_{sc} for $\frac{d'}{d}$ ratio from SP-34 table.

Step-2

Determine x_{ulim} : limiting depth of neutral axis and M_{ulim}

$M_{ulim} = 0.148 f_{ck} b d^2 \rightarrow Fe\ 250$

$0.138 f_{ck} b d^2 \rightarrow Fe\ 415$

$0.134 f_{ck} b d^2 \rightarrow Fe\ 500$

Step-3 Calculate M_u extra/ M_{u_2}

Determine A_{st_1}

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

Step-4

Determine M_{u_2} and A_{st_2}

$$\boxed{M_{u_2} = M_u - M_{ulim}}$$

$$\boxed{A_{st_2} = \frac{M_{u_2}}{0.87 f_y (d - d')}}$$

Step-5

$$A_{st} = A_{st_1} + A_{st_2}$$

(then calculate the no. of bar)

A_{st} provided.

Step-6

Determine ~~or~~ area of compression steel. (A_{sc})

$$\boxed{A_{sc} = \frac{M_{u_2}}{(f_{sc} - f_{cc})(d - d')}}$$

provide A_{sc} by choosing suitable diameter of the bar.

Step-7 : Check for deflection control.

for

As per IS 456

$P_t = \frac{A_{st}}{bd} \times 100$ and $f_s = 0.58 f_y \left[\frac{A_{st}\ prov}{A_{st}\ req}\right]$

$\boxed{\text{Fig-4}}$ Fig K_t ~~(fig-6)~~

$P_c = \frac{A_{sc}}{bd} \times 100$ from K_c

$\boxed{\text{Fig-5}}$ ~~(fig-5c)~~

$$\boxed{\left(\frac{L}{d}\right)_{max} = 20 \times K_t \times K_c.}$$

SSB

(C)

$$\boxed{\left(\frac{l}{d}\right)_{max} = 7 \times K_t \times K_c}$$ (Cantilever beam)

If $\left(\frac{l}{d}\right)_{max} > \left(\frac{l}{d}\right)_{provided}$ then (OK)

$\left(\frac{l}{d}\right)_{max} < \left(\frac{l}{d}\right)_{provided}$
(then redesign the section)

$K_t \rightarrow$ Modification factor for tension Reinforcement

$K_c \rightarrow$ Modification factor for compression Reinforcement.

$\boxed{\text{IS 456 - Page no. - 38 \& 39}}$

$M_u \rightarrow$ factored Moment

$M_{ulim} \rightarrow$ Moment of resistance of balancing Section.

$M_{u\,extra} \rightarrow$ Extra moment of that are use for compression reinforcement design

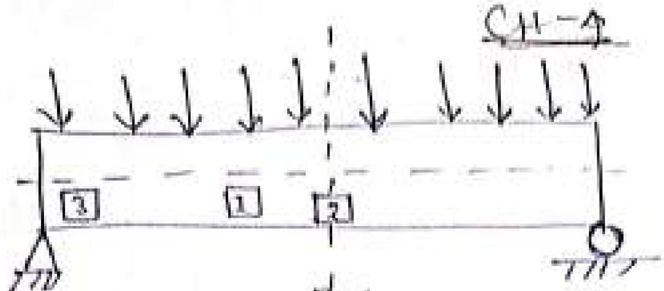
$$\boxed{M_{u\,extra} = M_u - M_{ulim}}$$

Effect of shear stress

(Diagonal tension)

CH-4

DT - 14/05/21



BM = 0
SF = M

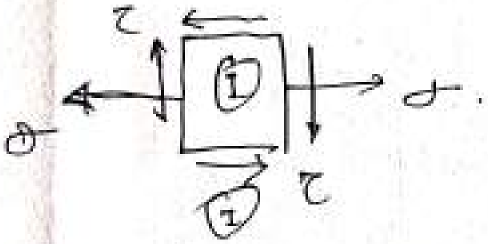
BM = max
SF = 0

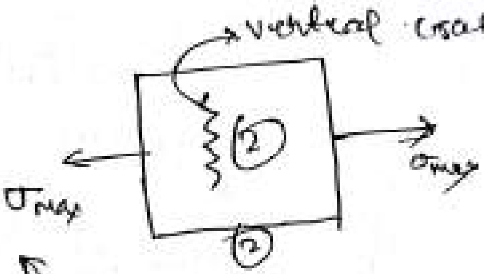
BM = 0
SF = max

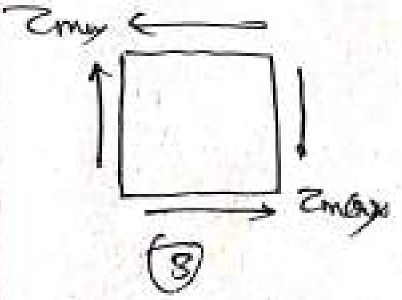
Element - 1

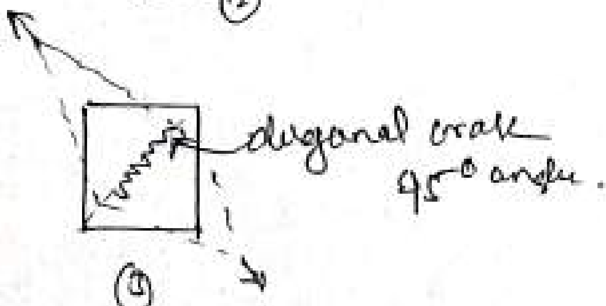
Diagonal

Bending & Shear

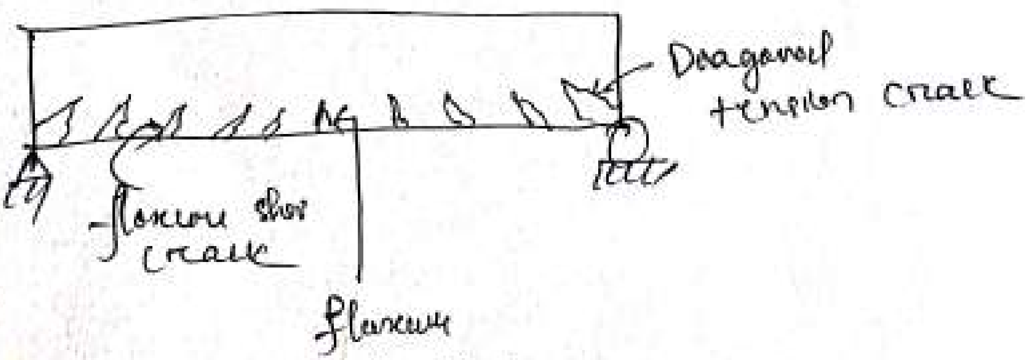








diagonal tension



Maximum shear force for UDL

$V_u = \frac{wl}{2}$ (SSB) →

$V_u = wl$ (Cantilever) →

Shear stress

$$\boxed{\tau = \frac{V}{Ib} A\bar{y}}$$

$V \rightarrow$ shear force at section x-x.

I : Moment of inertia about N.A.

b : Width of the section.

$A\bar{y}$: Moment of Area.

* location of Maximum & minimum shear stress

min τ

$$\left(y = \pm \frac{d}{2}\right) \longrightarrow \text{minimum shear stress}$$

$$\boxed{\tau_{min} = 0}$$

τ_{max}

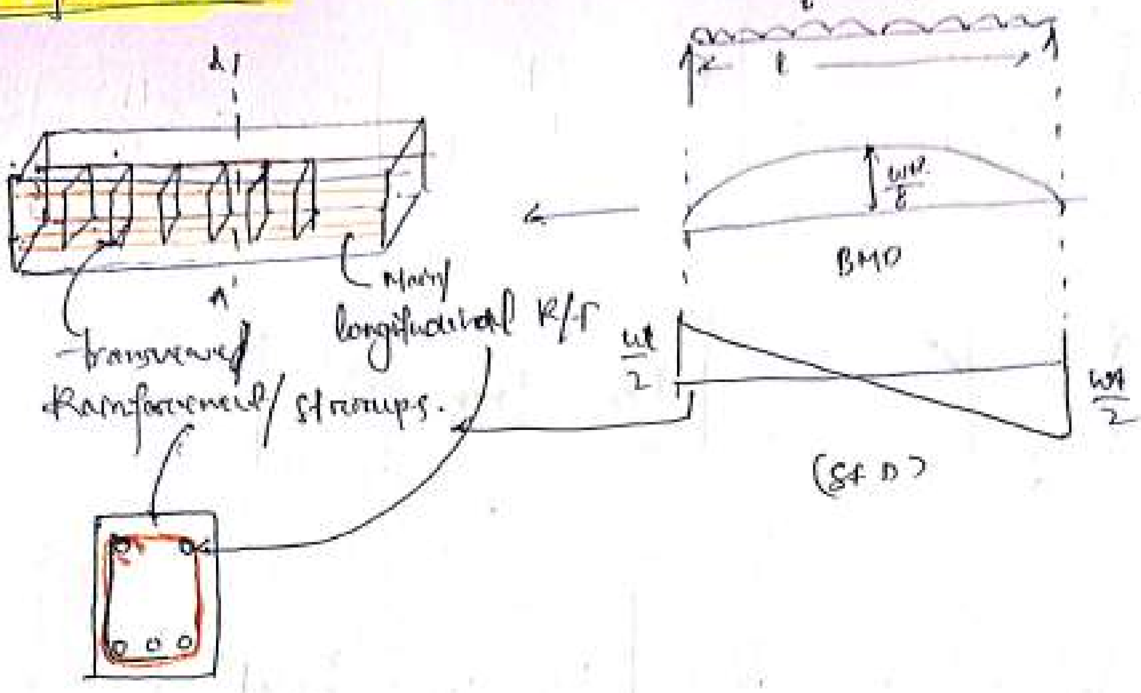
$$(y = 0) \longrightarrow \text{maximum shear stress}$$

$$\boxed{\tau_{max} = \frac{3}{2} \frac{V}{bd}}$$

CH-1 Design for shear

Design for shear.

D1-15/03/?



1) For slab and foundation, shear designing is not required. There can be save from shear by increasing the depth of section.

2) In case of beam we can be design shear R/f in the form of stirrups.

Shear Stress :-

$$\boxed{\tau = \frac{V}{A}}$$

$$\boxed{\tau = \frac{V}{b \cdot d}}$$

V : shear force

A : area

Shear stress distribution :-

$$\boxed{\tau = \frac{VA\bar{y}}{Ib}}$$

V → shear force

b → width of the beam/section

I → Moment of inertia.

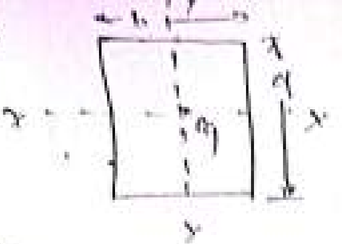
$\bar{y}$: distance between N.A. and C.G of a section.

A = Area of section.

$A\bar{y}$: Moment of area.

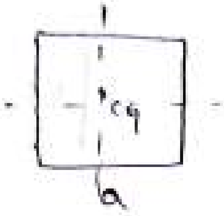
Moment of Inertia:-

Rectangular section



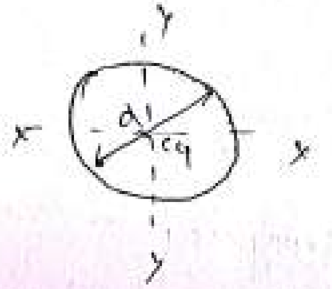
$$\boxed{I_{xx} = \frac{bd^3}{12}} \qquad \boxed{I_{yy} = \frac{db^3}{12}}$$

Square section



$$\boxed{I_{xx} = I_{yy} = \frac{a^4}{12}}$$

Circular section



$$\boxed{I_{xx} = I_{yy} = \frac{\pi}{64} d^4}$$

Type of shear:-

1. Flexure shear → Beam design.
2. Torsional shear → torsion design.
3. Punching shear → footing

$$\boxed{\frac{\tau}{J} = \frac{T}{R} = \frac{G\theta}{L}}$$

T: torsional moment

$$\boxed{T = \frac{\tau J}{R}}$$

J → Polar moment of inertia

R → Radius

⇒ Beam is subjected to flexure shear.

~~Rate of change of bending moment is called shear force~~

$$\boxed{\cancel{M = \frac{dM}{dx}}}$$

Empirical Relationship between shear force, Bending Moment, load intensity:-

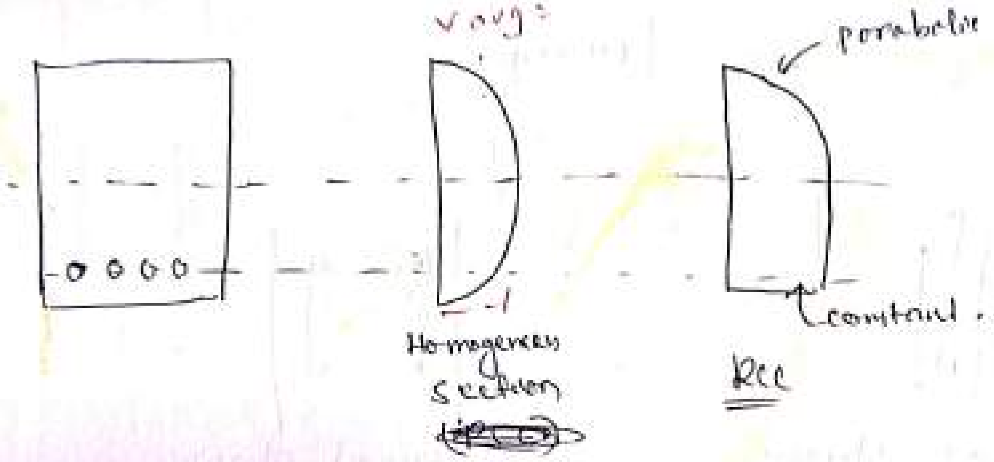
⇒ Shape of Bending Moment diagram ~~represent the~~ at a point represent ~~load intensity~~ shear force

$$\boxed{V = \frac{dM}{dx}}$$

Slope of shear force diagram represent at a point represent the load intensity.

$$\boxed{w = -\frac{dv}{dx}}$$

IS 456 Aproach



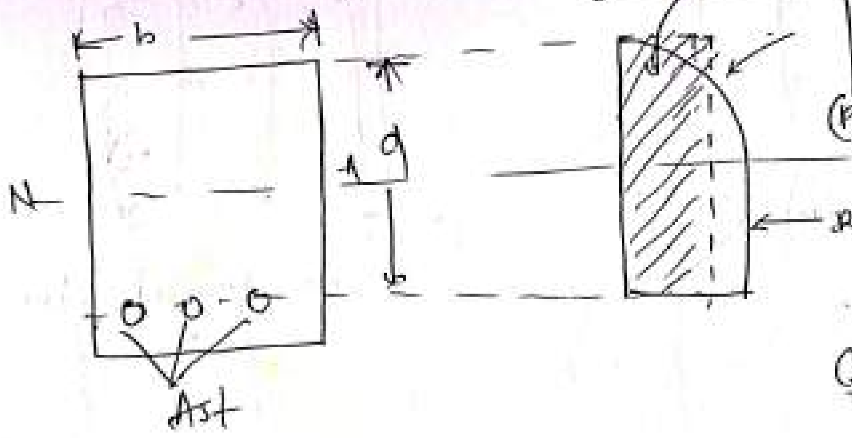
Shear stress distribution

Nominal shear stress (IS 456 : 2000)

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

$$\boxed{\tau = \frac{V}{2I_{eq}}\left(x_u^2 - y^2\right)}$$

(Parabolic) eq^n



$$\left[\tau = \frac{V}{(L.A)\cdot B} = \frac{V}{bjd}\right]$$

(Linear eq^n)

Book

$j = (1 - 0.42k)$ → Lever arm ~~constant~~ Co-efficient.

$\boxed{I_{eq} = I_{concrete} + I_{steel}}$

V_u = design shear force / shear force for design load.

b = breadth of the member.

d = depth of the member.

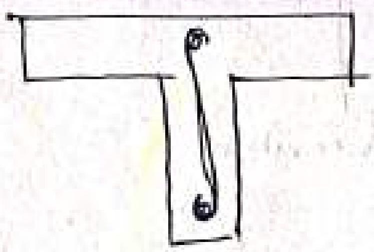
As per IS 456:2000 – we are assume nominal shear stress for designing.

~~Shear stress~~

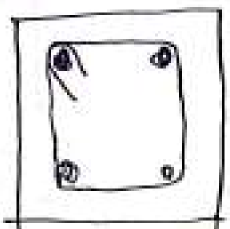
→ In shear designing we only calculate c/c dist of stirrups.

Generally shear steel are provided 8mm, 10mm, etc.

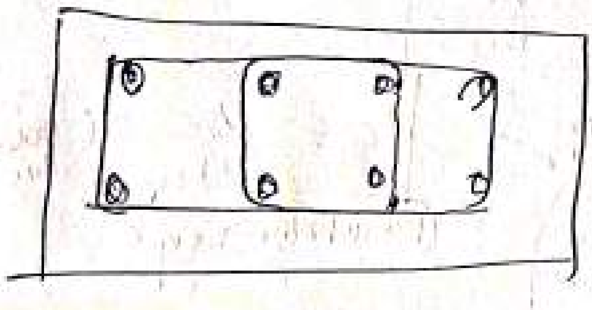
Type of shear stirrup!-



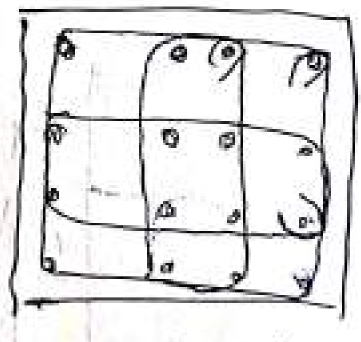
One-legged stirrups



2-legged stirrups



4-legged stirrups.



Six-legged stirrups

Design due to Shear Stress

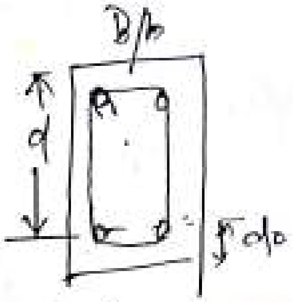
(Shear Designing) (51)

Given → B, D, Ast, V, Grade of concrete & Ast.,

Aim → ~~Nominal shear stress~~
Shear Reinforcement (Spacing)

Step-1 : Nominal shear stress (τ_u)

$$\boxed{\tau_u = \frac{V_u}{bd}}$$



IS 456 : Page-No-72

where → V_u = factored shear force.

Step-2 : Maximum shear stress (τ_{cmax})

| from IS 456 : 2000 / Table no-20 / Page - 73 |

↳ τ_{cmax} depend on grade of concrete.

Case-1 If $\boxed{\tau_{cmax} > \tau_v}$, then there is no diagonal compression / ~~failure~~. Failure tension (No shear stress required)

Case-2 If $\boxed{\tau_{cmax} < \tau_v}$, the diagonal comp. failure would occure. But it can be rectified @ designing stage & can be prevented by $\boxed{\text{redesigning}}$ the section.

↓

by Increasing ~~the steel~~ the depth

Step-3 : design shear strength (τ_c) / permi (6)

I.S 456 : 2000 / Page no - 19

→ the value of τ_c depend upon grade of concrete

p: %age of steel

$$\boxed{P_t = 100 \times \frac{A_t}{bd}}$$

x	a
y	?
z	b

Inter mediate value of τ_c max will be get by Interpolation

$$\tau_c = \boxed{a + \left(\frac{b-a}{z-x}\right)(y-x)}$$

Case-1 If $\tau_c > \tau_v$ then there is no diagonal tension failure.

In the case beam is safe in shear, here no need of shear R/F provided..

But min^m shear R/F will be provided.

because:
→ To improve ductility.
→ To resist diagonal tension
→ To resist the cracks development due to creep & shrinkage.

As per IS 456 : 2000

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

A_{sv} = area of stirrups

S_v = spacing b/w stirrups

Case-2 If $\tau_c < \tau_v$, here diagonal tension failure will now occur.

Hence, In this case we have to design the beam for shear R/F.

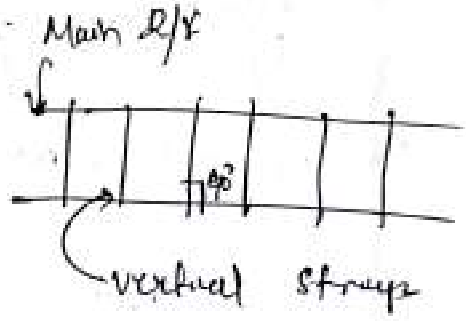
** finally, If $\boxed{\tau_{c\,max} > \tau_v > \tau_c}$, in this case we have use to provide shear R/F in the form of ~~stirrup~~ stirrup in the beam.

Step-4

As per IS 456 : 2000 (Page - no - 73)

a) Vertical stirrups :-

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



where

V_{us} = design shear force.

$\boxed{V_{us} = V_u - \tau_c b d}$ → IS 456 : Page - 73

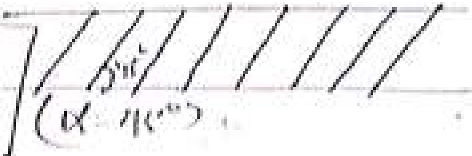
or $[(\tau_v - \tau_c) b d]$

If support is under comp., so critical section is at 'd' distance away from the face of column / support (45°)

If support is under tension, so critical section is at face of column. (90°)

b. Inclined stirrups:-

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



(c) Bentup bars:-

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

Note

→ for resistance of diagonal failure. we are to provide bentup bar.

⇒ we can use stirrups as ~~combined~~ individual & we can use it in combined.

Step-5

~~Minimum~~ Maximum spacing for of shear R/f :-

IS 456 page - 47

(i) Vertical stirrups

0.75 d
300 mm } (min value)

(ii) ~~Vertical~~ Inclined stirrups:-

d
300 } (min value

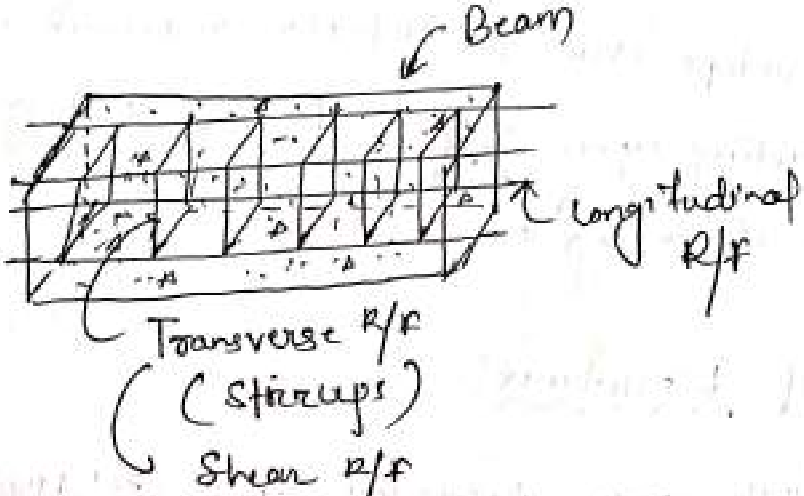
Step-6
detailing of shear R/f

$\boxed{\text{Sv should not exceed 300 mm in no case}}$

DA - 28/03/21 ①

BOND STREANGTH AND DEVELOPEMENT LENGTH

RCC Beam → Concrete + steel
↳ Ductile.



Bond :— Adhesion between steel and concrete which resist slipping of steel bars from concrete is known as bond.

→ Bond is responsible for transfer the stress from steel to concrete.

→ The bond develops due to setting of concrete on drying which result in gripping of steel bars.

Bond b/w steel and concrete develope due to :—

1. Chemical Adhesion.
2. friction b/w concrete and steel.
3. Mechanical Resistance.

1. Chemical Adhesion (1)

It is the grip developed due to Gum like property of Hydration product of cement in concrete (CSH type)!

2. friction b/w concrete and Steel:-

It develops due to relative movement b/w concrete and steel depending upon surface of bar and grip developed due to shrinkage of concrete.

3. Mechanical Resistance:-

→ The actual grip provided in HYSD bar for batter bond.

→ In case of Fe 250 (Mild steel) ⇒ Bond developed due to adhesion and friction.

→ In case of HYSD bar Bond develops due to ⇒ Adhesion. friction and mechanical resistance and due to the bond stress is increased by 60%.

How to increase Bond strength:-

→ Use HYSD bars (Bond ↑ ⇒ 60%)
→ Closed spacing b/w stirrups to avoid cracks.
→ Higher grade of Concrete is used ⇒ (M20-40)
→ Higher grade of stirrups.
→ Increase cover.
→ Mechanical anchorage (Use of ribs).

Bond Stress (τ_{bd})

- It is the shear developed at the interface of steel and concrete.
- Expressed as, tangential force per unit nominal surface area of reinforcement bar.

τ_{bd} Calculated

$$\boxed{\tau_{bd} = \frac{V}{\Sigma O \cdot LA}} \rightarrow \boxed{\tau_{bd}\ \text{Calculated}}$$

$$\boxed{\tau_{bd} = \frac{V}{n\pi\phi(d - 0.42x_{ulim})}}$$

(τ_{bd}) permissible :-

IS 456 : 2000 $\boxed{\text{table no-21}}$

- for mild steel tension :-

M-	M_{20}	M_{25}	M_{30}	M_{35}	M_{40}	M_{45}	M_{50}
τ_{bd}	1.2	1.4	1.5	1.7	1.9	1.9	1.9

It is depend upon grade of steel.

** If HYSD Bar are used, τ_{bd} is increased by 60%.

** If HYSD Bars are used in compression by increase by 25% more.

// for safe in bond stress

$$\boxed{(\tau_{bd})_{permissible} \geq (\tau_{bd})_{calculated}}$$

Note # We know that

$$(\tau_{bd})_{\text{permi calculated}} = \frac{V}{n \times p (1 \cdot 4)}$$

$$\boxed{(\tau_{bd})_{cal} \propto V}$$

\# if shear force is increase then the τ_{bd} value also increased.

or

The maximum bond stress is developed at where SF. will be maximum.

As for the

for SSB

max SF @ support

As for Continuous

max SF @ point of contraflexure.

\# To Improve Bond Stress Use thinner bar in more numbers

Q.3.c PYQ - 2018

fy 250, fck = 20

b = 400 mm. l = 6 m

d = 560 mm.

W_u = 175 KN/m.

$V_u = \frac{W_u L}{2} = \frac{175 \times 6}{2}$

= 525

$A_{st} = \left[\frac{\pi}{4} \times (28)^2\right] \times 4$

= 2464

$P_t = \frac{A_{st}}{b \cdot d} \times 100$

$= \frac{2464}{400 \times 560} \times 100 = 1.1$

step-1

$\tau_v = \frac{V_u}{b \cdot d} = \frac{525 \times 10^3}{560 \times 400} = 2.34$ N/mm²

step-2

$\tau_{cmax} = 2.8$

$\boxed{\tau_c < \tau_{cmax}}$ → ok

$\tau_c \rightarrow P_t = 1.1$ → 0.62
1.25 → 0.67

$\tau_c = 0.62 + \left(\frac{0.67 - 0.62}{1.25 - 1}\right)(1.1 - 1)$

= 0.64 N/mm²

Shear Resistance of concrete $\tau_c b \cdot d$

$V_c = 0.64 \times 560 \times 400$

= 143360.

$V_{us} = V_u - V_c$

$= 525 \times 10^3 - 143360$

= 381640

Use 8 mm

10 mm 4 legged stirrup.

$\frac{\pi}{4} \times (10)^2 \times 4$

= 314.15

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

$= \frac{0.87 \times 250 \times 314.15 \times 560}{381640}$

= 100 mm c/c.

300 mm.

$0.75 d = 0.75 \times 560$

= 420 mm

Necessary ... bentup bar

$$\boxed{V_{us}' \not> \frac{V_{us}}{2}}$$

$$\boxed{V_{us} = V_u - \underbrace{\tau_c\, b\, d}_{\text{shear resistance of concrete}}}$$

Step-1

Calculate the shear resistance of bentup bar V_{us}'

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

$$\boxed{V_{us}' \not> \frac{V_{us}}{2}}$$

Calculate shear force carried by vertical stirrups.

$$\boxed{V_{us_2} = V_{us} - V_{us}'}$$

then design for vertical stirrups.

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

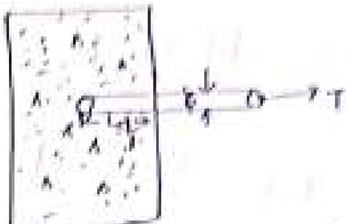
$$\boxed{S_v = \frac{0.87\, f_y\, A_{sv} \cdot d}{S_v}}$$

---

hence provide.

4 legged 10mm ϕ bar vertical stirrup @ 100 mm c/c.

** DEVELOPEMENT LENGTH (L_d)



Defination

It is the minimum length of Reinforcement Bar which is required to be embbeded in concrete such that the strength in bond is equal to tensile force applied on R/F Bar.

[Stress × Area = force]

$$\boxed{\tau_{bd} \times \pi \phi \cdot L_d = T}$$

$$\boxed{\begin{matrix} 2\pi r = \phi \\ L = L_d \end{matrix}}$$

Surface area of concrete = $2\pi r.l$

$$\boxed{SA = \pi \phi L_d}$$

L_d = developement length

τ_{bd} = permeasible bond stress.

Calⁿ of L_d

$\because \quad \tau_{bd} \times \pi \phi L_d = T$

$\tau_{bd} \times \cancel{\pi} \cancel{\phi} L_d = \sigma_{st} \frac{\cancel{\pi}}{4} \phi^2$

$$\boxed{L_d = \frac{\sigma_{st} \phi}{4 \tau_{bd}}} \text{ General}$$

LSM

$$\boxed{L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}}} \; *** \quad \boxed{\text{Imp}}$$

We know that → (τ_{bd}) permissible ≥ (τ_{bd}) calculated.

Now $L_d = \frac{\sigma_{st} \phi}{4 \tau_{bd}}$ hence, $\boxed{L_d \propto \frac{1}{\tau_{bd}}}$

$$\boxed{(L_d)_{\text{permeasible}} \leq (L_d)_{\text{provided}}}$$

Rout up Rout

$(\tau_{bd})_{perm}$ ~~acord~~ according to IS 456:2000

Table ~~[illegible]~~ (2)

Which shows the value of τ_{bd} permissible

M	M20	M25	M30
τ_{bd}	1.2	1.4	1.5

1.4 — 1.6

~~For mild steel~~

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

~~HYSD~~

$$\cancel{L_d = \frac{0.87 f_y \phi}{4(1.6 \tau_{bd})}}$$

for Mild steel

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

for HYSD

$$\boxed{L_d = \frac{0.87 f_y \phi}{4(1.6 \tau_{bd})}}$$

for HYSD In comp

$$\boxed{L_d = \frac{0.87 f_y \phi}{4(1.6 \times 1.25 \tau_{bd})}}$$

Imporstent Note

We Know that :-

* $(\tau_{bd})_{perm} \geq (\tau_{bd})_{cal.}$

$$(\tau_{bd})_{cal.} = \frac{V_u}{\Sigma O \cdot L \cdot A}$$

$$(\tau_{bd})_{cal} = \frac{V_u}{n \pi \phi \cdot (d - 0.42 x_u)} \quad \text{— (1)}$$

* Develoment length $L_d = \dfrac{0.87 \cdot f_y \phi}{4 \tau_{bd}}$

$$(\tau_{bd})_{cal} = \frac{0.87 f_y \phi}{4 L_d} \quad \text{— (2)}$$

equating eqn (1) & (2)

$$\frac{V_u}{n \pi \phi (d - 0.42 x_u)} = \frac{0.87 \cdot f_y \phi}{4 \cdot L_d}$$

$$L_d = \frac{0.87 \cdot f_y \cdot n \pi \phi^2 \cdot (d - 0.42 x_u)}{4 \cdot V_u}$$

$$L_d = \frac{0.87 \cdot f_y \cdot A_{st} \cdot (d - 0.42 x_u)}{V_u}$$ ③

$$\boxed{L_d = \frac{M_u}{V_u}}$$

When are is a case of confined compression then $\frac{M_u}{V_u}$ is increase by 30%.

$$\boxed{L_d = 1.3 \frac{M_u}{V_u}}$$

from the previous eqn.

$$\boxed{L_d = \frac{M_u}{V_u}}$$

Now for safety purpose (we providing Anchorage)

$$\boxed{(L_d)_{(prov)} = \frac{M_u}{V_u} + L_0}$$

where l_0 = anchorage value.

* for each 45° bond anchorag value is taken 4ϕ

WAY TO DESIGN DEVELOPMENT LANGTH

As per IS 456 : 2000 developement length is check by the following eqn.

$$\boxed{\frac{M_u}{V_u} + L_0 \geq L_d}$$ or

If check not satisfied following steps measure will be taken.

1. Reduce dia ϕ for same value of A_{st}.

2. Increase the value of L_0 by providing anchorage.

3. Reduce the no. of bentup bars. (7)

~~Type of No~~ **Anchorage Value for different bends**

	Anchorage Value (L_0)
45°	4ϕ
90°	8ϕ
135°	12ϕ
180°	16ϕ

Type of probleme

Check for development length

Calculate the value of development length (L_d)

Step-1

$$M_u/M_1 = 0.87 f_y A_{st} (d - 0.42 x_u)$$

or

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

M_u, V_u & L_d calculate.

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

For both tension & comp

Step-2 Apply check for developement length.

$$\frac{M_u}{V_u} + L_0 \geq L_d.$$

(by using different anchrage. bar)

REINFORCEMENT SPLICING :— ①

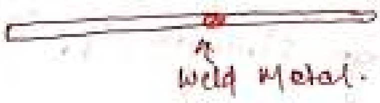
→ Joining the bars of standard length to get the design length is known as splicing. (12 m) * standard length.

→ The splicing may be one of 3 types.

(i) Lap Splices / joints

(ii) Mechanical Joints.

(iii) Welded Joints.



→ It ~~is~~ Recomended that splices for flexure should not be provided at section where the bending moment is / more then 50% ~~then~~ of the moment of resistance.

* IS codal Recommendation for lap Splices :—

(i) ~~[illegible]~~ Lap splice should not be used for bars larger then 36 mm. Larger dia bar may welded together.

(ii) Value of Lap Length.

Flexure tension – $\begin{cases} L_d \\ 30\phi \end{cases}$ (max)

Direct tension $\begin{cases} 2L_d \\ 30\phi \end{cases}$ (max)

Should not be less then 200 mm or 15 ϕ.

Increase the value of ...

(iii) C/c ~~d~~ distance of the splices ~~m~~ is not less than 1.3 times the lap length. (1.3 ld).

(iv) Lap length value in Compression.

$$\begin{cases} L_d \\ 24\phi \ (\text{max}) \end{cases}$$

(v) Lap length is ~~caluted~~ calculated on the basis of dia. of smaller bar when bars of two different diameter are to be spliced.

(vi) In case of bundle individual bar splices are suggested

IS Recommendation for ~~@~~ Welded & Mechanical Splice :-

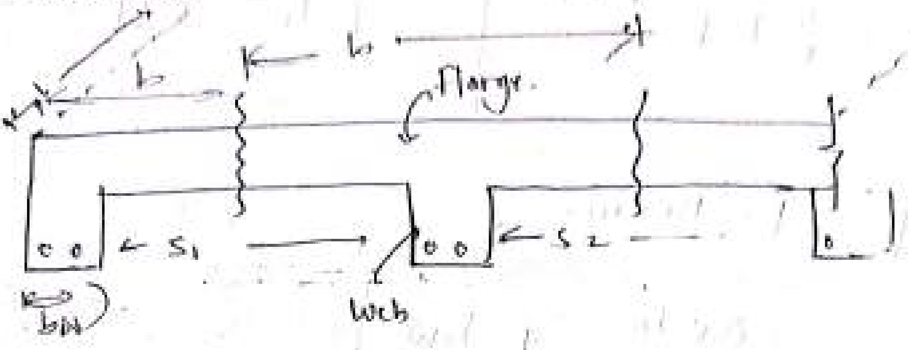
(i) Considered 80% design strength for tension splices.
100% design strength of compression splices.

(ii) ~~Considered~~ If the ~~weded~~ welded ~~area~~ area not more than 20% of Reinforcement bar then adopt the design strength as 100%,

(iii) Mechanical splice then also ~~tey~~ take the design strength as 100%.

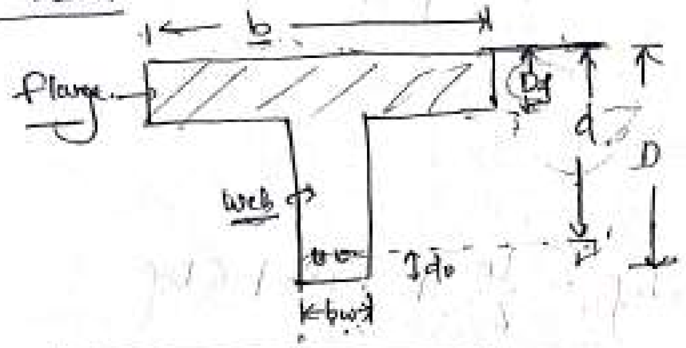
CH-5 // **FLANGED BEAM** — IS 456 Annex - G — Dt-16/09/24

↓ Monolithic Beam.



L - Beam
T - Beam

T-beam



$$\boxed{b = \frac{S_1}{2} + bw + \frac{S_2}{2}}$$

b = Actual width of flange.

Df = Depth of flange

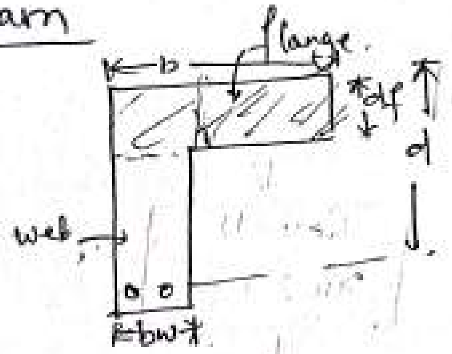
bw = width of web.

d = effective depth.

L = clear span

L_0 = effective span. (distance between 2 point of contraflexure)

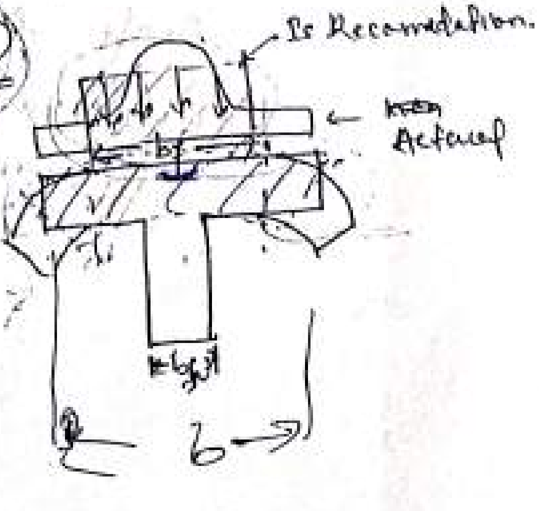
L-beam



$$\boxed{b = \frac{S_1}{2} + bw}$$

Effective width of flange :- (b_f)

$$\boxed{b_f < b}$$



1. Isolated T- beam.

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

$l_0 = l \rightarrow$ SSB

$l_0 = 0.7\,l \rightarrow$ cantilever

2. Isolated L- Beam.

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

3. Continuous beam :—

T- Beam $\rightarrow$ $b_f = \frac{l_0}{6} + b_w + 6D_f$

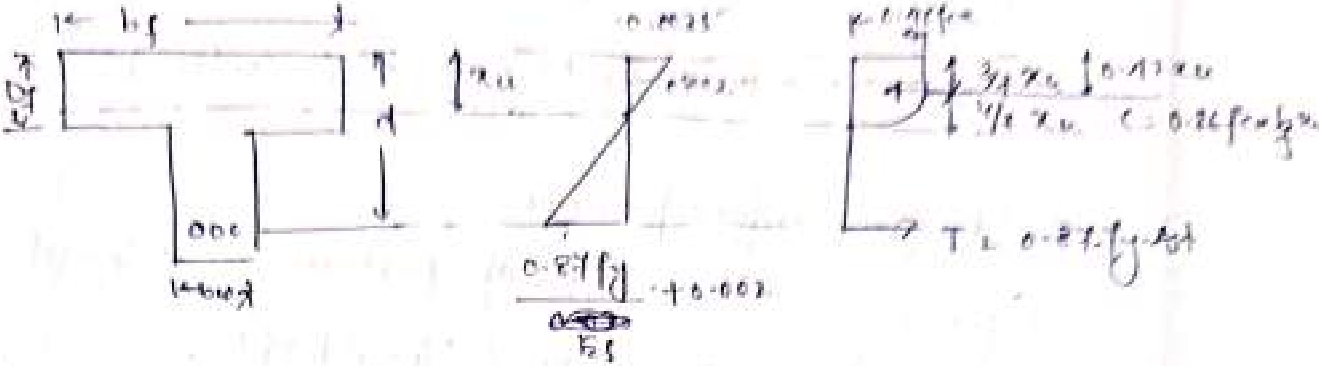
L- Beam $\rightarrow$ $b_f = \frac{l_0}{12} + b_w + 3D_f$

ANALYSIS OF T-BEAM :—

Case

- T-Beam
  - Case-I ($x_u < D_f$) → Analyse As Singly reinforced beam.
  - Case-II $x_u > D_f$
    - Case-II.a $D_f \leq \frac{3}{7} x_u$
    - Case-II.b $D_f > \frac{3}{7} x_u$

Case - I ($D_f > x_u$)



Also
* Depth of neutral axis (x_u) :-

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

* Critical depth of neutral axis :-

$x_{u\,lim}$ = 0.53 d (fe 250)

0.48 d (fe 415)

0.46 d. (fe 500)

* Type of section

(i) $x_u < x_{u\,lim}$ (under Reinforced section)

(ii) $x_u = x_{u\,lim}$ (Balanced section)

(iii) $x_u > x_{u\,lim}$. (Over Reinforced section)

* Moment of Resistance :-

(Case-1) Under Reinforced section.

$$\boxed{M_u = 0.87 f_y \cdot A_{st}\,(d - 0.42\,x_u)}$$

Case-2 Balanced section.

$$M_{ulim} = 0.36 f_{ck} b\, x_{ulim} (d - 0.42 x_{ulim})$$

$$0.87 f_y A_{st} (d - 0.42 x_{ulim})$$

Case-3. Over Reinforced section

Generally this type of section should be restricted as per IS code 456:2000 (G-2.1).

Moment of Resistance of this type of section will be calculated as $x_u = x_{ulim}$.

$$\boxed{M_{ulim} = 0.36 f_{ck} b\, x_{ulim} (d - 0.42 x_{ulim})}$$

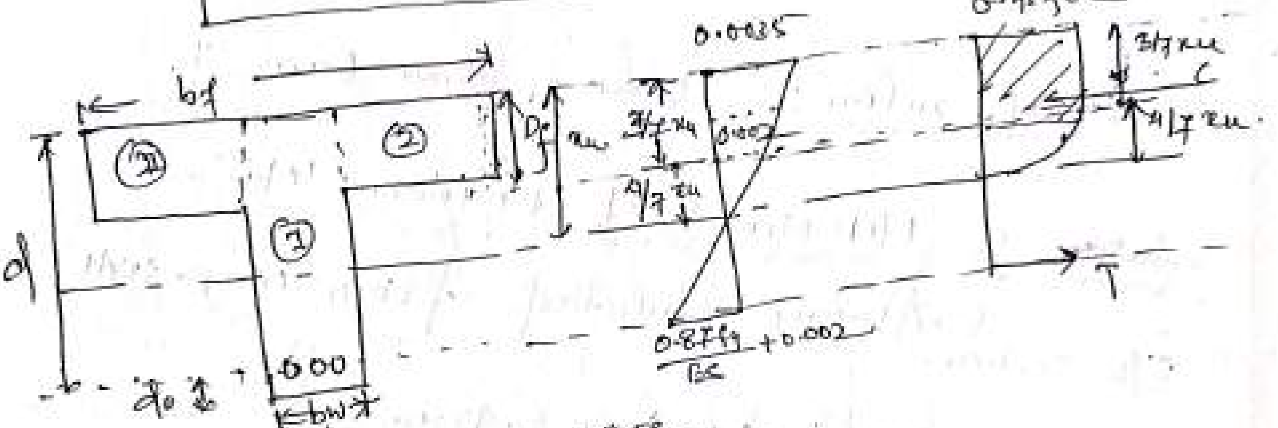
FLANGED BEAM

Analysis of T-Beam

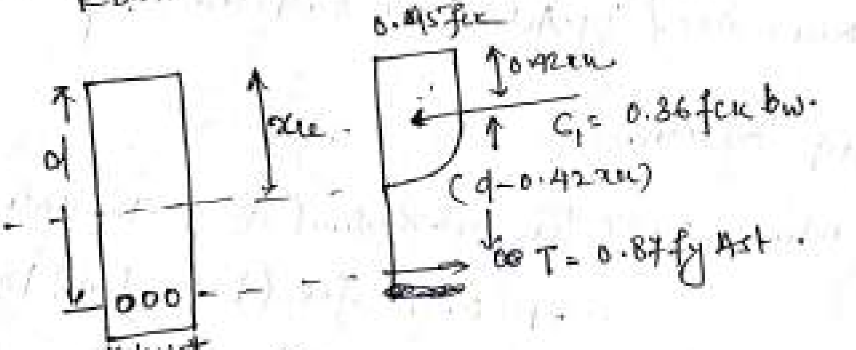
Case-II.(a) :-

Depth of N.A. lies in web and flange lies within the rectangular stress block portion.

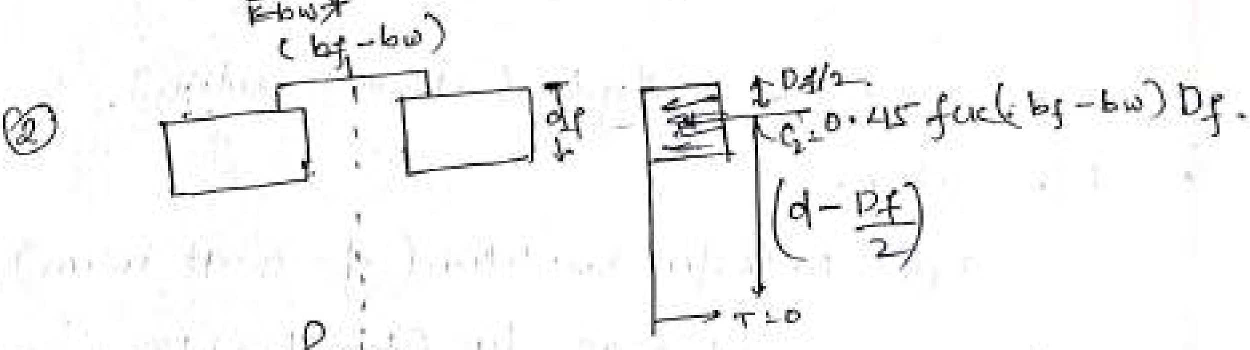
$$\boxed{x_u > D_f \quad , \quad D_f < \frac{3}{7} x_u}$$



(1)



(2)



Total compressive force.

$$C = C_1 + C_2$$

$$C_1 = 0.36 f_{ck} b_w \cdot x_u.$$

$$C_2 = 0.45 f_{ck} (b_f - b_w) \cdot D_f$$

Tensile force.

$$T = 0.87 f_y \cdot A_{st}.$$

Calculation of Depth of ~~Max~~ Neutral Axis :- (x_u). ②

Tensile force = Compressive force.

$$T = C$$

$$0.87 f_y \cdot A_{st} = 0.36 f_{ck}\, b_w\, x_u + 0.45 \cdot f_{ck} (b_f - b_w) D_f$$

$$x_u = \frac{0.87 f_y \cdot A_{st} - 0.45 f_{ck} (b_f - b_w) D_f}{0.36 f_{ck}\, b_w}$$

Calculation of x_{ulim} :- Same as ~~above~~ Case- I.

Calculation of ~~Max~~ Moment of Resistance (M_u) :-

(Always calculated from comp side)

~~(i) O/R Section.~~

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

(ii) Balanced section.

$$M_u = 0.36 f_{ck}\, b_w \cdot x_{ulim} (d - 0.42 x_{ulim}) + 0.45 f_{ck} (b_f - b_w) D_f \left(d - \frac{D_f}{2}\right)$$

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

(iii) O/R section.

$$M_u = 0.36 f_{ck}\, b_w \cdot x_{ulim} (d - 0.42 x_{ulim}) + 0.45 f_{ck} (b_f - b_w) D_f \left(d - \frac{D_f}{2}\right).$$

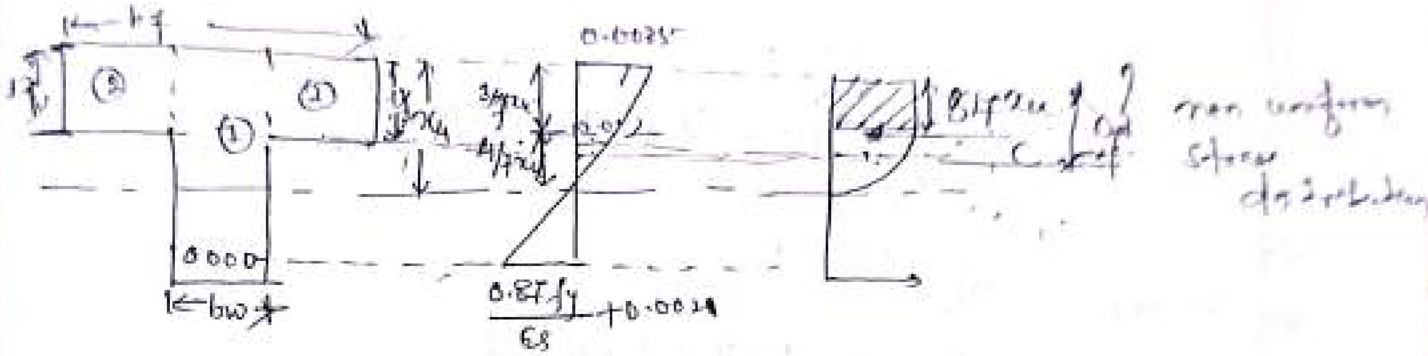
(iii) U/R Section

$$M_u = 0.36 f_{ck} \cdot b_w\, x_u (d - 0.42 x_u) + 0.45 f_{ck} (b_f - b_w) D_f \left(d - \frac{D_f}{2}\right)$$

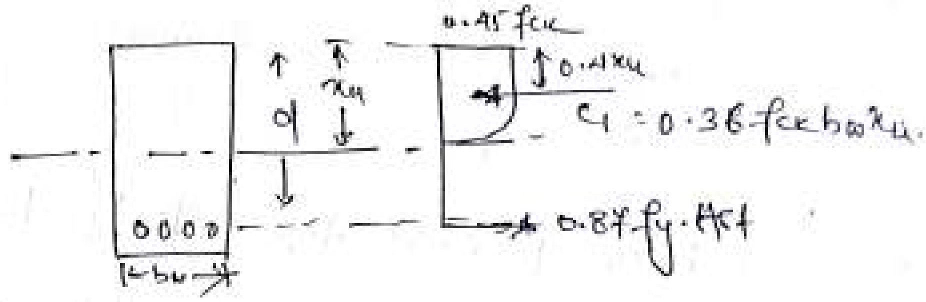
Case – II(b) :– (3)

N.A. lies on web & Df Depth of flange is greater than $\frac{3}{7}x_u$.

$$(x_u > D_f\ ,\ D_f > \frac{3}{7}x_u)$$



(1)



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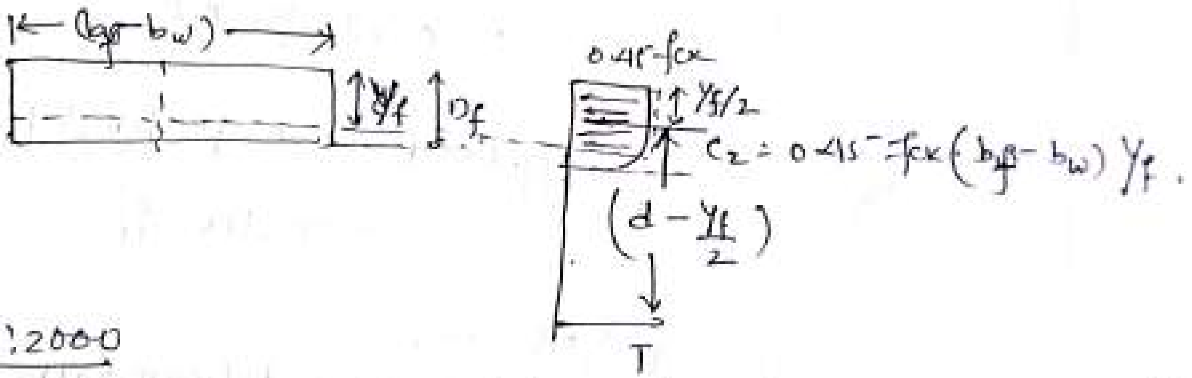


As for IS 456:2000

** For the depth Df, the stress distribution is non-uniform. Hence, we considered uniform stress distribution diagram for an equivalent depth y_f.

$$\boxed{y_f = 0.15\,x_u + 0.65\,D_f}$$

where y_f = equivalent depth of flange for which we assumed the stress block is linear.

Tensile force:

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

Comp. force

$$C = C_1 + C_2$$

$$C_1 = 0.36 f_{ck} b_w \cdot x_u$$

$$C_2 = 0.45 f_{ck} \cdot (b_f - b_w) \cdot y_f$$

~~Step: # Moment of Resistance.~~

Actual depth of N.A. (x_u).

Comp. force = Tensile force

$$C = T$$

$$0.36 f_{ck} b_w x_u + 0.45 f_{ck} (b_f - b_w) y_f = 0.87 f_y \cdot A_{st}$$

$$x_u = \frac{0.87 f_y A_{st} - 0.45 f_{ck} (b_f - b_w) y_f}{0.36 f_{ck} b_w}$$

Calⁿ of Moment of Resistance :- (Always calculated from Comp. side)

~~(i) U/R Section.~~

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

(ii) Balanced section & O/R section.

$$M_{ulim} = 0.36 f_{ck} b_w x_{ulim} (d - 0.42 x_{ulim}) - 0.45 f_{ck} (b_f - b_w) y_f (d - \frac{y_f}{2})$$

(iii) U/R section

$$M_u = 0.36 f_{ck} b_w \cdot x_u (d - 0.42 x_u) - 0.45 f_{ck} (b_f - b_w) y_f (d - \frac{y_f}{2})$$

CH-6 : ANALYSIS AND DESIGN OF SLAB AND STAIRCASE.

Codal provision (IS 456 : 2000)

I. Effective length or Span (Beam & Slab)

(i) SSB

$$L_{eff} = \left.\begin{array}{l} l_0 + \frac{w}{2} + \frac{w}{2} \\ l_0 + d \end{array}\right\} \text{min.}$$

(ii) fixed or Continuous :-

(a) when $w \leq \frac{l_0}{12}$

L_{eff} = Same as SSB.

(b) If $w > \frac{l_0}{12}$ or 600 mm.

Case-b.1 (One end fixed one end continuous)

$L_{eff} = L_0$

Case b.2 (One end simply supported and other end continuous)

$$L_{eff} = \left.\begin{array}{l} l_0 + \frac{w}{2} \\ \text{or} \\ l_0 + \frac{d}{2} \end{array}\right\} \text{min}$$

(iii) Cantilever beam :-

$L_{eff} = l_0 + \frac{d}{2}$

. (a

2# Permissible Deflection

(limit state of serviceability)

(i) The final deflection after all load. (DL + LL + WL) &
considering the creep, shrinkage & temperature from the
cast level of support $\ngtr \frac{span}{250}$.

(ii) Total deflection after considering creep, shrinkage.
and temp. after the erection of partition walls and
the application of finishes.

$$\left.\begin{array}{c} \ngtr \frac{span}{350} \\ \text{or} \\ 200\text{ mm} \end{array}\right\} \text{min.}$$

3# Deflection Control Criteria:-

It deals with the ratio of span to effective depth
the ratio should not be α-value.

$$\boxed{\frac{\text{Span}}{\text{effective ratio}} = \frac{L}{d} \ngtr \alpha\text{-value.}}$$

$$\left\{\begin{array}{lcl} \text{Cantilever} & \rightarrow & 7 \\ \text{SSB} & \rightarrow & 20 \\ \text{Continuous} & - & 26 \end{array}\right\}$$

Note If the span is greater than 10m the α-value
should be multiplied by $\frac{10}{span}$.

Note

Note:– the 4-value will be multiplied further by modification factor k_c & k_t.

$k_t \rightarrow$ depends ~~upo~~ on % of tension R/F.

$k_c \rightarrow$ depend on % comp R/F.

$K_t \rightarrow$ depends in %age of tension R/F.

IS 456:2000 – page-No-38 → Given Graph. on k_t upon % of Tension R/F.

f_s = Service stress.

$$f_s = 0.58 f_y \left(\frac{A_{st\ req}}{A_{st\ prov}} \right)$$

$K_c \rightarrow$ depends ~~up~~ on %age of Compression R/F.

IS 456:2000 → page no – 39. → Graph Given on k_c upon %age comp. R/F

<u>Final deflection Criteria.</u>

$$\boxed{\left(\frac{l_0}{d} \right)_{prov.} \ngtr \underbrace{\left(\frac{l}{d} \right)}_{4\text{ value}} \times k_t \times k_c.}$$

If. span > 10

$$\boxed{\left(\frac{l_0}{d} \right)_{prov.} \ngtr \underbrace{\left(\frac{l}{d} \right)}_{4\text{-value.}} \times \frac{l_0}{10} \times k_t \times k_c.}$$

A. Slenderness limit to ensure lateral stability.

(i) SSB

$$l_0 \ngtr \left.\begin{matrix} 60B \\ \text{or} \\ \dfrac{250B^2}{d} \end{matrix}\right\} \text{min.}$$

(ii) Cantilever

$$l_0 \ngtr \left.\begin{matrix} 25B \\ \text{or} \\ \dfrac{100B^2}{d} \end{matrix}\right\} \text{min}$$

5.1 Minimum Reinforcement :-

0.15% of gross area → mild steel
0.12% of gross area → HYSD bar.

Minimum reinforcement in slab is provided to prevent ~~[illegible]~~ the shrinkage and temp cracks.

Note
Minimum R/F in slab is less than min R/F in beam.

5.2. Diameter of Main Bar :-

(i) Max dia of bar should not be greater than 1/8 th of bar

$$\boxed{\phi_{main} \ngtr \frac{1}{8} \text{ thickness of slab.}}$$

i) Min dia of main bar should not be less than 10mm ϕ for mild steel.

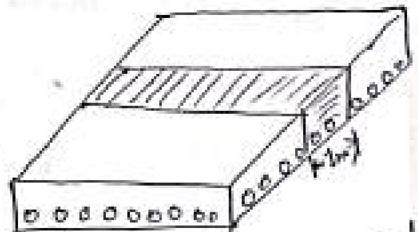
(ii) Min dia. of main bar should not be less than 8mm for non HYSD bar.

(iii) Minimum dia. of distribution bar should not be less than 6mm.

5.3. Spacing b/w R/F :-

Maximum spacing in main bar ≯ 3d, 300mm } min.

Maximum spacing in Distribution bar ≯ 5d, 450mm } min.



FEATURES OF SLAB :-

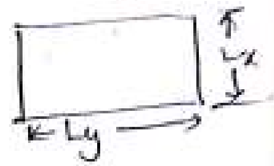
* for convinence, we design a slab for a strip of 1m
* for uniaxial flexure the slabs are designed as beam.
* No- compression Reinforcement are provided in the slab
* No - shear Reinforcement are provided in slabs.
* Main bars are provided along the shorter span (lx) in one way slabs. & It is provided to resist flexure.

→ Distribution bars are provided along ... span (Ly) in one way slab. It is provide to prevent temp & shrinkage cracks.

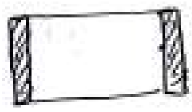
Types of Slabs :-

1(i) One way slab:- (i) The ratio of longer span to shorter span is greater than 2, then it is a one way slab.

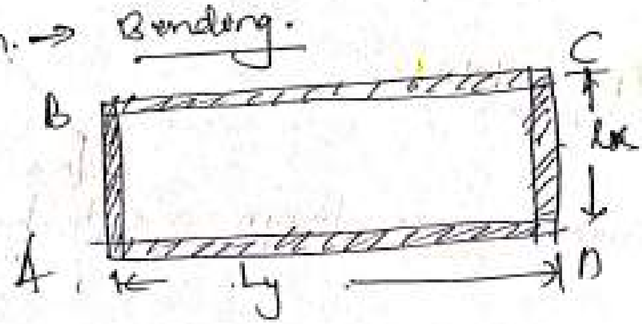
$$\boxed{\frac{L_y}{L_x} > 2}$$



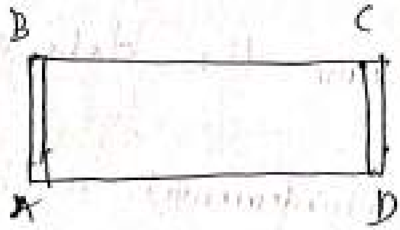
(ii) If the two opposite edge of slab are supported, then it is always a one-way slab.



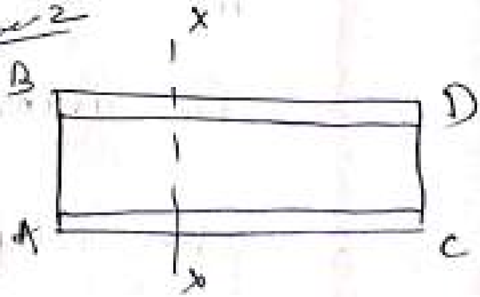
(iii) Main R/f are provided in shorter span. Because of main bar resist flexure which is come in shorter span. → Bending.



Case 1



Case 2



$$\delta_1 = \frac{5}{348}\frac{wL_y^4}{EI} \qquad \delta_2 = \frac{5}{348}\frac{wL_x^4}{EI}$$

$$\delta_1 \propto L_y^4 \qquad \delta \propto L_x^4$$

vent? Bending is dominat in shorter span of slab.

B. Two way slab:-

~~If the longer span to short~~

(i) If the ratio of longer span to shorter span is less then or equals to 2, if is a two way slab.

$$\boxed{\frac{ly}{lx} \le 2}$$

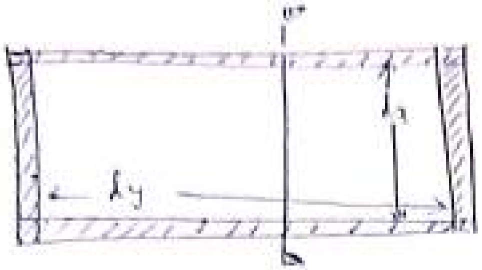
(ii) All edges are supported in case of two way slab.

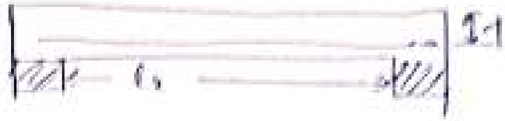
(iii) Deflection control craterea:-

$$\frac{\text{Span}}{\text{depth}} \not> \text{B-value.}$$

Type of support	B-value	
	mild steel	HYSD bar
1. Simply supported	35	28
2. Continuous	40	30

Analysis And Design Of One way Slab





Step-1 : Calculation of "Leff" and "d".

(1)

$$\frac{\text{Span}}{\text{effective depth}} \not> \text{A-value}$$

$$\boxed{\text{Effective depth} = \frac{\text{Span}}{\text{A-value}}}$$

A-value
- Cantilever → 7
- S.S.B → 20
- Continuous → 27.

Note : take some extra ~~value~~ increased value from this formula

(2) Effective length (L_0) :-

(S.S.B)

$$L_{eff} = \left.\begin{array}{l} l_x + \frac{w}{2} + \frac{w}{2} \\ l_x + d \end{array}\right\} \text{min.}$$

Step-2

Load Calculation

25 = Unit weight of concrete

$DL = 25 \times B \times D$ kN/m $= W_D$ (B = 1m = 1000mm)

LL = ? kN/m. (IS 875) → DL, LL, WL

Floor finishing : $24 \times b_{floor} \times t_{floor} = W_f$ (24 = density)

$$\boxed{W = W_{DL} + W_{LL} + W_{FL}}$$

factored load

$$\boxed{W_u = 1.5 \times W}$$

Step-3

Bending Moment Calculation

SSB : $M_u = \frac{W_u l_{eff}^2}{8}$

Cantilever $M_u = \frac{W_u \cdot l_{eff}^2}{2}$

Continuous slab M_u = Co-efficient requirement.

Step-4

$$\boxed{d_{req.} = \sqrt{\frac{M_u}{Q \cdot B}}}$$

Q value
- 0.148 fck (Fe 250)
- 0.138 fck (Fe 415)
- 0.134 fck (Fe 500)

Check for the condition $\boxed{d_{pro} > d_{req}}$

↳ (design is safe)

Step-5 : Area of Main Reinforcement Bar :-

Method-1 Direct formula

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

Method-2

Calculate x_{ulim}.

then A_{st}

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

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

Step-6: Spacing of Main bar:-

$$\text{Spacing} = \frac{\text{Area of } \phi \text{ one R/F}}{A_{st}} \times 100$$

$$= \frac{\frac{\pi}{4}\phi^2}{A_{st}} \times 100 \ldots$$

Max spacing $\ngtr$ 3·d or 300 mm } min

} min of all are provided.

Step-7: Design of Distribution bar:-

(these Bar are provided in the longer span)

A_{st} min = 0.15 % of Gross Area → mild steel

0.12 % of Gross Area → HYSD Bar.

$$\boxed{A_{st}\,\text{min} = \frac{0.12}{100} \times B \cdot D \quad \text{(HYSD Bar)} \qquad \frac{0.15}{100} \times B \cdot D \quad \text{(mild steel)}}$$

Step-8:

$$\text{Spacing of distribution bar} = \frac{\text{Area of one R/F}}{A_{st}\,\text{min}} \times 100$$

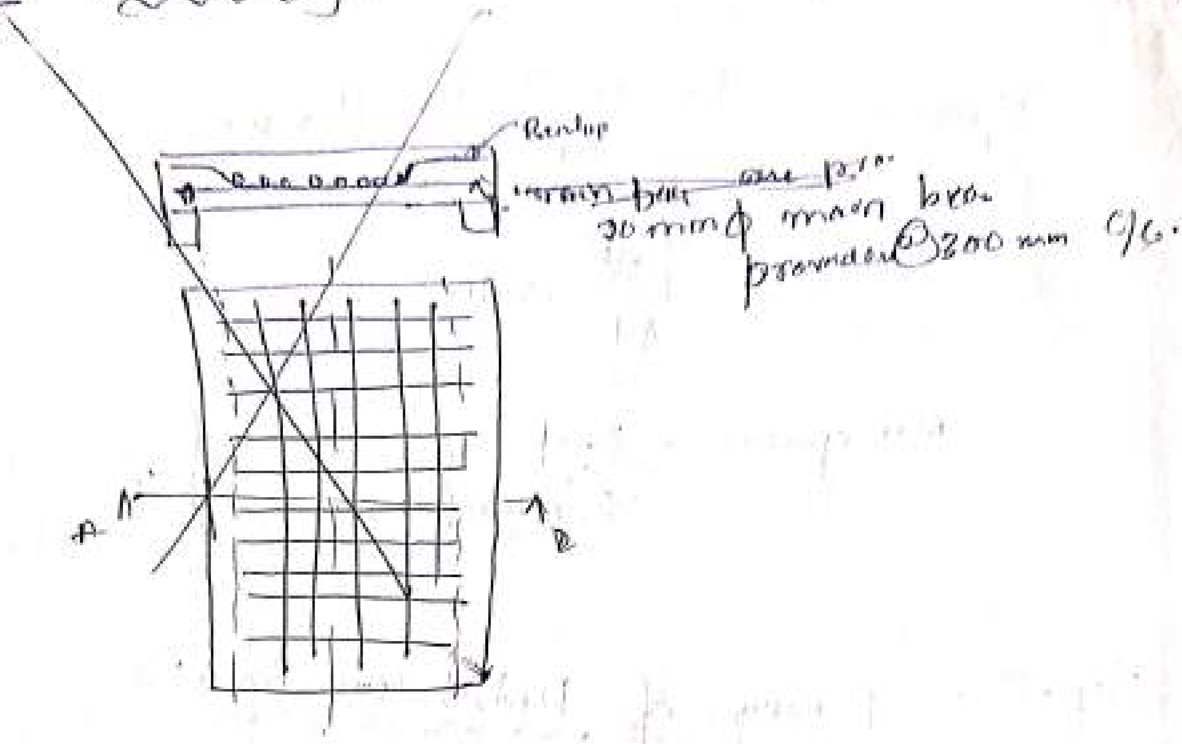
$$= \frac{\frac{\pi}{4} \times \phi^2}{A_{st}\,\text{min}} \times 100$$

Max spacing of Distribution $\ngtr$ 5d or 45 mm } min.

} min of all will be provided.

Step-9 * Detailing :-

Last Step



Step 9

Also do the check for deflection and shear. & development length.

* Check for deflection.

$$\left(\frac{l}{d}\right)_{prov} < \left[4\text{-value} \times k_t \times k_c = \left(\frac{l}{d}\right)_{max} \right]$$

* Check for shear.

$$\boxed{\tau_c > \tau_v} \qquad \boxed{\tau_v = \frac{V_u}{bd}}$$

* IS 456 table value

$$\boxed{\tau_{c_{slab}} = k \cdot \tau_c}$$

* Check for development length.

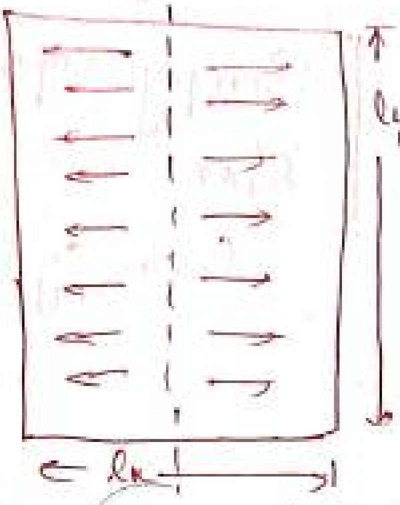
Step 10 Detailing of Slab

~~Step-12~~

~~Detailing of~~ slab

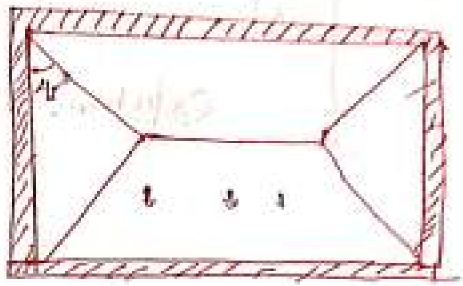
Load Distribution on slab

One way slab



So On one way slab the load ~~di~~ transformation will be on shorter span only. Hanane.
the main ~~bar~~ reinforcement are provided on ~~the~~ shorter span ~~only~~. only and. ~~longer spa span provided~~ ~~only~~ distribution bar are provided on longer span.

Two way slab



On two way slab the load distribution/ transformation are both the longer and shorter span so here we required to provide the main reinforcement on ~~th~~ both longer and shorter span.

Due to the ~~dir~~ triangular load distribution ~~there~~ in two way slab ~~at~~ the edge it will be generate torsion. which causes the lift.

Type of two way slab

Restrained slab

[Monolithic construction of both slab and ~~~~ beam.]

Unrestrain slab

$\boxed{\text{Simply supported slab.}}$

C on syllabus.

Design of Two way slab:-

↳ Main bars are provided on both the span

Step-1 : Calculation of effective depth & span.

Step-2: Load calculation.

Step-3: Design Bending Moment and Shear force calculation

$$\boxed{\begin{aligned} M_x &= \alpha_x \cdot w \cdot l_x^2 \\ M_y &= \alpha_y \cdot w \cdot l_y^2 \end{aligned}} \quad \boxed{V_u = \frac{wl}{2}}$$

α_x & α_y - Given in IS 456 in Table no. 27 (Annex - G. Page no. 91)

Step-4: Calculation of Min. depth required.

$$\boxed{d_{req} = \sqrt{\frac{M_{max}}{Q \cdot B}}} \quad \boxed{d_{provided} > d_{req}}$$

↳ Hence - OK

Step-5 : Calculation of Area of Steel:-

• Longer span

$$A_{st_y} = \frac{M_{uy}}{0.87 f_y (d - 0.42 x_{ulim})}$$

Spacing

• Shorter span

$$A_{st_x} = \frac{M_{ux}}{0.87 f_y (d - 0.42 x_{ulim})}$$

Check for. ~~Larger of~~ A_{st_x} & A_{st_y} should be greater than $A_{st\,min}$

~~Step-~~

--

Step-6 : Calculation of Spacing

Spacing @ l_x direction : $\frac{a_{st}}{A_{st_x}} \times 1000$

Spacing @ l_y direction : $\frac{a_{st_y}}{A_{st_y}} \times 1000$

or 3.d

or 300

which ever is minimum.

Step 7 Apply all checks

Check for shear

Check for Deflection

Check for development length.

Step-8:- Provide Torsion reinforcement.

$A_{st\,Torsion} = \frac{3}{4}(A_{st\,x})$

=

length of torsion bar = $\frac{1}{5} \times$ shorter span

Calculate spacing by assuming diameter of the steel.

load). It is reinforced with 4 bars of 28 ϕ mm diameter tension steel (A_{st} = 2464 mm^2) which continue right into the support. Take f_{ck} = 20N/mm^2, f_y = 250 N/mm^2. Design shear reinforcement using LSM. 7

4. (a) Give two reasons why doubly reinforced sections are adopted. 2

(b) Calculate development length required to be provided for M25 concrete and Fe 415 steel of diameter ϕ for bars (i) In tension (ii) In compression. 5

4

(c) Determine the moment of resistance of the T-beam. Given data : b_f = 1000 mm, D_f = 100 mm, b_w = 300 mm, cover = 50 mm, d = 450 mm and A_{st} = 1963 mm^2 (4-25#). Use M20 and Fe415. 7

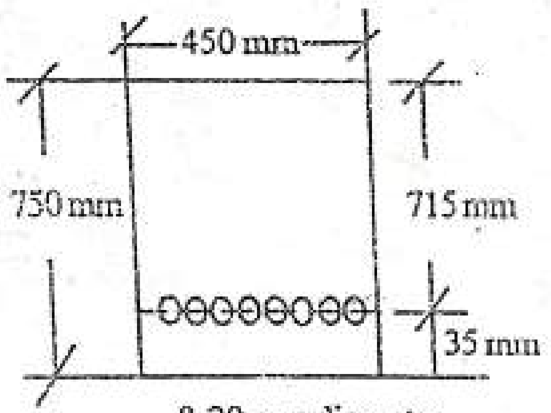
5. (a) Write the expression for effective width of flange of an isolated T-beam. 2

(b) A double reinforced beam section is 250 mm wide and 500 mm deep to the centre of the tensile reinforcement. It is reinforced with 2 bars of 16 mm dia. as compression reinforcement at an effective cover of 50 mm and 4 bars of 25 mm dia. as tensile steel. Using M20 concrete and Fe 250 steel. Calculate the ultimate moment of resistance of the beam. 5

(c) A simply supported one-way slab for an office building of a clear span 3m is supported over beams of 300 mm width. Design the slab for a live load of 3 kN/m^2. The materials used are M20 and Fe 415 steel. (Use LSM). 7

(a) What is the minimum percentage of distribution steel for a slab for mild steel and HYSD steel ? 2

(b) Derive the stress block parameters for limit state analysis for flexure. 5



8-20 mm diameter

WSM Not in Syllabus

(c) A reinforced concrete beam is simply supported over a span of 5m and it carries a uniformly distributed load of 25000 N/m including its own weight. If the size of the beam is restricted to 300 × 500 mm, determine the areas of tension and compression steel if required. Given M20 concrete and HYSD bars of Fe415. ~~Use WSM~~. LSM 7

2. (a) Define characteristic strength as per IS 456-2000. 2

(b) Write down the codal provision for minimum reinforcement in (i) beams (ii) slab (iii) columns. 5

(c) Calculate the area of steel of grade Fe415 required for section of width 250 mm and overall depth 500mm (effective depth 460 mm) in M20 if the limit state moment to be carried by the beam section is (i) 146 kNm (ii) 100 kNm. 7

3.

(a) What is the effective span of a simply supported beam or slab which is not built integrally with its support? 2

(b) State the assumptions made in limit state of collapse : Flexure. 5

(c) A simply supported beam with clear span 6m, width(b) = 400 mm, effective depth (d) = 560 mm, carries a limit state load of 175 kN/m (including self weight, dead load and live

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Total Pages—6 V—Sem/CIVIL/2018 (W)(New)

STRUCTURAL DESIGN-I

(Code—CET-501)

Full Marks : 70

Time : 3 hours

Answer any five questions

Figures in the right-hand margin indicate marks

Use of IS 456-2000 is allowed in examination

1. (*a*) Which type of section is known as under-reinforced section ? 2

(*b*) Find the moment of resistance of the RCC beam section shown in *figure*, if the stresses in steel and concrete are not to exceed $230\ N/mm^2$ and $7\ N/mm^2$ respectively. $m = 13{\cdot}33$. Use WSM. 5

(Turn Over)

2

DESIGN OF A DOG-LEGGED STAIR CASE

Step-1: Proportioning the dimension of Stair Case.

Assume 2 flight, ~~spacing between two flight = 0.15m~~

width of one flight = 1.35m.

Spacing between two flight = (b - 2× dimension of one flight)

Height of one flight = $\frac{\text{Height of floor}}{2}$.

Assuming height of riser → 150mm.

$$\text{no. of riser} = \frac{\text{flight height}}{\text{riser height (150 mm)}}$$

No. of tread required = no. of riser - 1

Assume width of tread = 300 mm.

Total going = 300 × no. of tread.

$$\text{Width of landing} = \frac{l - \text{total going}}{2}.$$

Step-2: Effective span & thickness of slab

$$\text{Effective span} = \boxed{L + \frac{w}{2} + \frac{w}{2}}$$

w → width of support wall

effective thickness of ~~slab~~ waist slab $\boxed{(d) = \frac{l}{20}}$ (SSB case)

thickness of slab $\boxed{D = d + 40}$

Step 3: Calculation of load

for steps & going

Weight of waist slab $= D\gamma\sqrt{1+\frac{R^2}{T^2}}$ (kN/m)

R → Riser
T → tread

Weight of steps

$= \frac{25RT}{2T}$ (kN/m)

Total load = wt. of slab + weight of steps

$\boxed{W = DL + LL}$

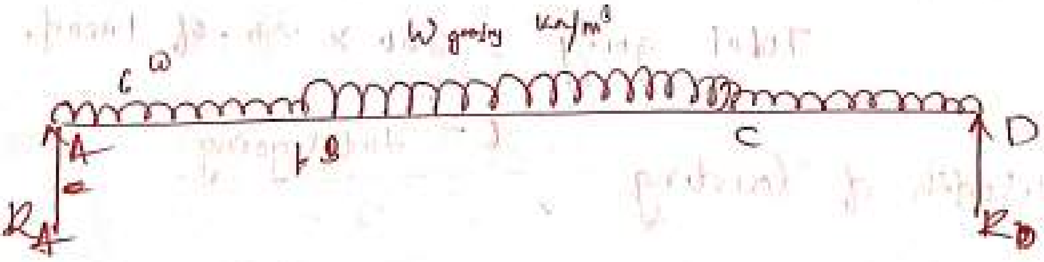
$\boxed{W_u = 1.5\,W}$

for landing

wt of slab = $d \times b \times 25$ ($b = 1000$ mm) DL
1 m.

$\boxed{W = DL + LL}$

$\boxed{\text{factored load } w_u = 1.5\,(W_{d.\,landing})}$



Step 4: Calculation of design bending moment.

by using SOM – SFD & BMD

then calculate M_{ulim}.
check for over or u/r section.

If $M_u < M_{ulim}$ → Design as singly Reinforced beam

$M_u = M_{ulim}$ → singly Reinforced beam

$M_u > M_{ulim}$ → Doubly Reinforced beam.

Step-6

Calculation of area of steel.

Main steel

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

Assume ~~ct~~ ϕ of bar and calculate spacing.

- Clear

Distribution steel → ~~15~~% 0.12% of bD → HYSD $\boxed{b, 1000\,mm}$.
~~0.~~ 0.15% of bD → mild steel

Assume ϕ of bar spacing.

Step-7: Developement Length. Calⁿ

Steps - Summary

1. Proportioning the dimensions. of ~~sta~~ stair case
2. Calculate effective depth & ~~Ef~~ effective span of stair case
3. Calculation of Load.
4. Calculation of B.M (Design) & check for type of section.
6. Calculation of main and distribution steel & spacing
7. developmet length (l_d).
8. Detailing of ~~sta~~ staircase.

CH-7 DESIGN OF COLUMN AND FOOTING

COLUMN

Types of Compression Member :-

(i) Column :- A column is defined as a vertical compression member which is mainly subjected to axial load and effective length of which ~~exceeds~~ exceeds three times of its least lateral dimension.

(ii) Pedestal :- A compression member whose effective length is less than three times of its Least Lateral dimensions. (LLD).

(iii) Struts :- A compression member which is inclined or horizontal and it is subjected to axial loading.

CLASSIFICATION OF COLUMNS :-

Column are classified on the basis of different criteria.

1. Shape of cross-section.
2. Material of construction.
3. Type of loading.
4. Slenderness ratio.
5. Type of lateral ties.

7. Shape of c/s - sections:—

(i) Square.

(ii) Rectangular.

(iii) Circular.

(iv) Pentagonal

(v) Hexagonal

(vi) Octagonal

(vii) T-shape & L-shape.

8. Material of construction:—

(i) Timber Column.

(ii) Masonary Column.

(iii) RCC Column.

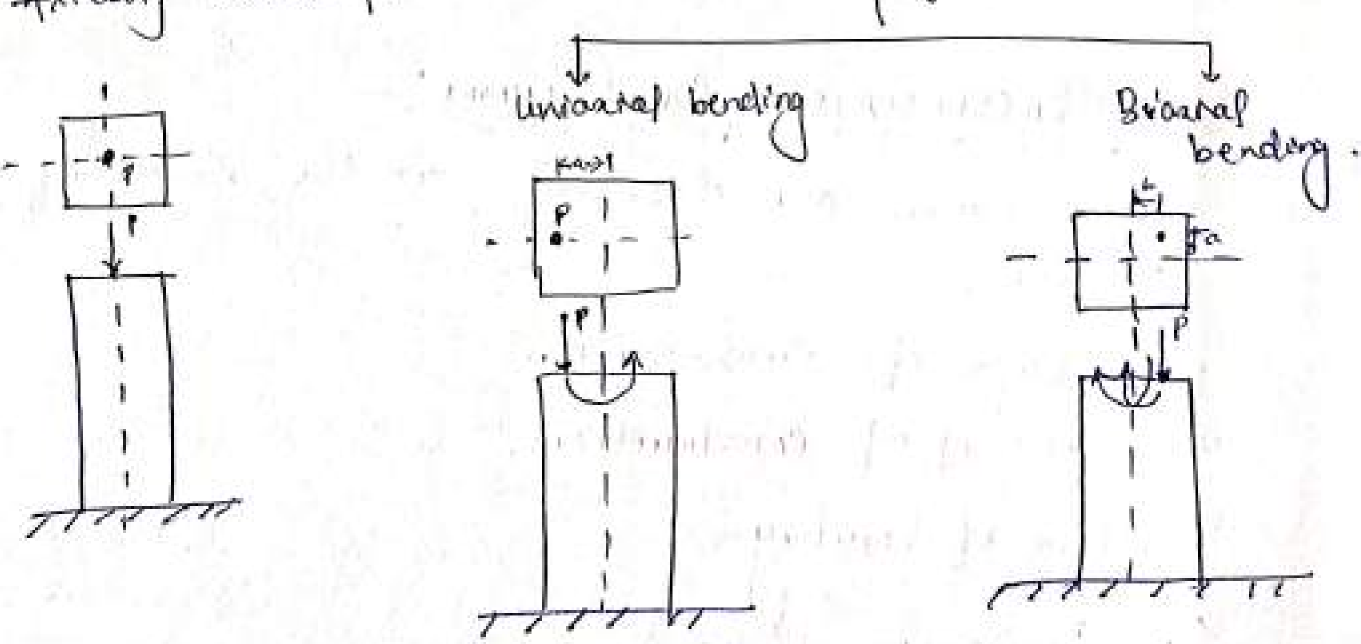
(iv) Steel Column.

(v) Composite Column.

8. Type of loading:—

(i) Axially Loaded.

(ii) Eccentrically Loaded.



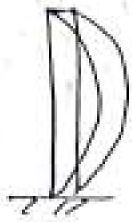
4. Slenderness Ratio :-

Columns

- long Column

$$\boxed{\frac{L_{eff}}{LLD} > 12}$$

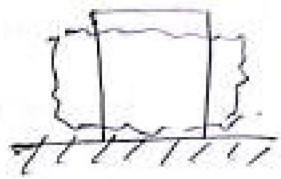
(failure in buckling)



- Short column.

$$\boxed{\frac{L_{eff}}{LLD} \leq 12}$$

(failure by crushing)



Slenderness ratio $\boxed{\lambda = \frac{L_{eff}}{LLD}}$ (RCC)

$\lambda = \frac{l_{eff}}{\gamma_{min}}$ (SOM)

γ_{min} → minimum Radius of Gyration.

$$\boxed{\gamma_{min} = \sqrt{\frac{I_{min}}{A}}}$$

$\lambda \leq 40$ → short column.

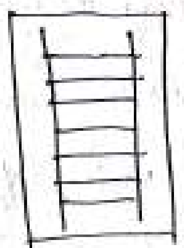
$\lambda > 40$ → long column.

5. Type of Lateral Reinforcement :-

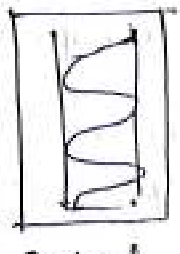
(i) Column with lateral ties

(ii) Column with spiral ties.

(iii) Composite. steel. Column → T, I, C section used.



(lateral ties)



(Spiral ties)

Assumption for
LIMIT STATE OF COLLAPSE : COMPRESSION :—

(i) Plane section remain before bending remain plain after bending.

(ii) All tensile stress taken by steel only.

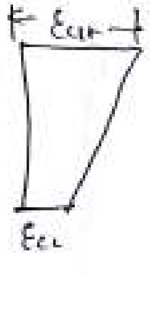
(iii) The stress strain relation is assumed to be parabolic.

(iv) The maximum compressive strain in the ~~~~ ~~compression zone~~ column subjected to axial compression only will be 0.002.

(v) Maximum comp. strain subjected to axial compression and bending when part of the column section is in tension 0.0035.

(vi) Maximum comp. strain in the column subjected to axial compression and bending when "no tension" is considered any where in the column section is :

$$\boxed{\epsilon_{ch} = 0.0035 - 0.75\,\epsilon_{cL}}$$



ϵ_{ch} → ~~least~~ height compressive strain.

ϵ_{cL} → Least compressive strain

(vii) The maximum compressive stress of ~~steel~~ concrete assumed to be 0.67 fck. the partial safety factor $\gamma_m = 1.5$. is ~~~~ applied to 1.5 then a Design strength of concrete.

$= \dfrac{0.67\,fck}{1.5} = 0.45\,fck$.

Step-5

Provida ... of

IS CODE PROVISION FOR COLUMN:-

1. # Effective length of column:-

Table-No-28 (IS 456:2000) (Annex-E)

(i) ⊥ → held in position & Restrain agenst Rotation (Fixed support)

(ii) △ → ~~Restrain agenst~~ held in position but not Re straned agenst Rotation → ~~Simply~~ (hing)

(iii) neither held in position nor Restrain agenst rotation i.e. free end.

(iv) Restrain agest Rotation but not held in position.

| Theoritical | | Recomended value |

2. Rainforcement:-

Longitudinal R/F

(i) minimum Rainforcement = 0.8 % of Gross c/s area

$$\boxed{Pt_{min} = \frac{0.8}{100} \times B \times D}$$

* It is provided to aviod the ~~B~~ brittle failure.

(ii) Maximum Rainforcement = 6% of Gross c/s area. (when bars are not overlaped.)

= 4% of Gross c/s area. (bars are overlaped)

* Maximum rainformet is limited to have adquete compaction or to avoid congection.

fy

fy

fy

(iii) Min Dia of steel Reinforcement is 12 mm provided for longitudinal R/f.

(iv) Minimum no. of bar.

4 bars → Rectangular/ square

6 bar → Circular

(v) Spacing between R/f should not be exceed 300 mm.

(vi) for pedestal min steel R/f = 0.15% of gross Area.

(vii) Minimum nominal cover is 40 mm. but it will be reduced to 25 mm. for column c/s less than 200 mm.

Transverse R/F

Ties bars

ties bars are used to prevent buckling of main R/f. also to keep the longitudinal bar on position.

(i) Dia of tie bars :- (for both lateral & helical ties)

$$\left.\begin{matrix} \frac{\phi_{min}}{4} \\ 6mm \end{matrix}\right\}$$ max value.

(ii) Pitch/ Spacing of tie bars :- P

(a) Lateral ties →

$$\left.\begin{matrix} LLD \\ 16\,\phi_{min} \\ 300\ mm \end{matrix}\right\}$$ min. value

(b) helical tie →

Provide area Ø ..

4. Minimum Eccentricity :-

All column are designed for minimum eccentricity.

$$e_{min} = \frac{\text{Unsupported length}}{500} + \frac{\text{lateral dimension}}{30}$$

or

20 mm

} greater

$$\boxed{e_{min} = \frac{l_0}{500} + \frac{D}{30} \quad \text{or} \quad 20\text{ mm} \;\Big\}\text{ greater}}$$

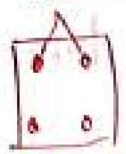
ANALYSIS AND DESIGN OF SHORT COLUMN

Analysis - Calculate the load carrying capacity of Column.

(i) Axially loaded column

Pu = load carrying capacity of concrete + load carrying capacity of steel

Asc.



$$\boxed{P_u = f_c \cdot A_c + f_s A_s}$$

$A_g = B \times D$

$f_c \rightarrow$ stress in concrete

$A_c \rightarrow$ Area of concrete

$f_s \rightarrow$ stress in steel.

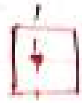
A_s : Area of steel.

$f_c = 0.45 f_{ck}$, $f_s = 0.75 f_y$.

Fe 250 , $f_s = 0.87 f_y$
Fe 415 , $f_s = 0.79 f_y$
Fe 500 , $f_s = 0.79 f_y$

$A_c = (A_g - A_s)$,

$P_u = f_c \cdot A_c + f_s \cdot A_s$



for purely axial compression

$$\boxed{P_u = 0.45 f_{ck} A_c + \cancel{0.} \; 0.75 \cdot f_y \cdot A_{sc}}$$

$$\boxed{A_c = A_g - A_{sc}}$$

\# In practice, we consider min eccentricity as per IS Code provision

* This value of P_u is decreased by 10%, only 90% of P_u is considered, for considering minimum eccentricity.

Considering e_{min}

$M_{uxx} = P_u \cdot e_{min\,xx}$

$M_{uyy} = P_u \cdot e_{min\,yy}$

$$P_u = 0.9 \left[0.45 f_{ck} A_c + 0.75 f_y \cdot A_s \right]$$

$$\boxed{P_u = 0.4 f_{ck} A_c + 0.67 f_y A_s}$$ Imp

Condition

(1) $\boxed{e_{min} \leq 0.05 \, (B \text{ or } D)}$

then considering the effect of eccentricity.

(2) $\boxed{e_{min} \not\leq 0.05 \, (B \text{ or } D)}$

then calculate P_u without considering the eccentricity effect.

Provide area of ...

<u>Note</u>

(i) $\boxed{e_{min} \leq 0.05 D}$ [$D \geq 400$]

$$\boxed{P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}}$$

(ii) $\boxed{e_{min} \geq 0.05 D}$ [$D < 400$]

$$\boxed{P_u = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}}$$

) <u>for the case of helical Reinforcement</u>

$\boxed{P_{u_{helix}} = 1.05 P_u}$

~~$P_u = 1.05$ (f_{ck}~~

$$\boxed{P_u = 1.05 \left(0.4 f_{ck} A_c + 0.67 f_y A_{sc} \right)}$$

Step-5

Provide area of ~~stee~~ Reinforcement (A_{sc})

Calculate no. of bar.

Step-6 : Provide lateral ties as per specification.

For lateral ties

ϕ. of ~~bar~~ ties

$\left.\begin{array}{l} \frac{1}{4} \times \phi_{min} \\ \text{or } 6mm \end{array}\right\}$ max value. or grater

Pitch of ties

$\left.\begin{array}{l} LLD \\ 16 \times \phi_{min} \\ 300mm \end{array}\right\}$ min value or less

For Helical ties

ϕ . of ties

$\left.\begin{array}{l} \frac{1}{4} \times \phi_{min} \\ 6 mm \end{array}\right\}$ max or grater.

Pitch of ties

$\left.\begin{array}{l} D \\ 16 \phi_{min} \\ 300 mm. \end{array}\right\}$ min or less