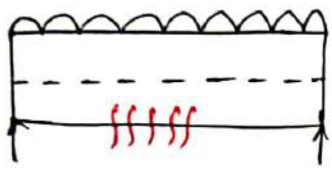
CHAPTER : 01

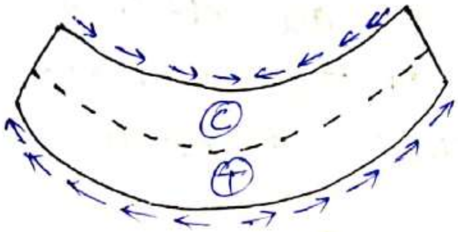
IS CODE RECOMMENDATIONS FOR LIMIT STATE METHOD OF DESIGN

L1:- Introduction to RCC and its design methods.

Reinforced cement concrete (RCC) [IS 456:2000].

Concrete :- * Good in compression
* Weak in Tension.

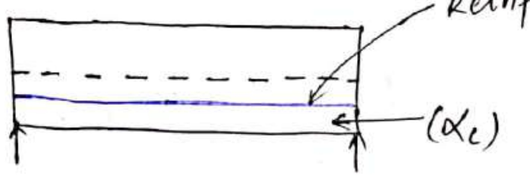




$$\frac{M}{I} = \frac{\sigma}{y}$$

$$\frac{M}{I} \cdot y = \sigma_z$$

Gold
copper
Silver
✓steel
} ductile



(koi b ductile material ko add karna) Hme steel is more imp

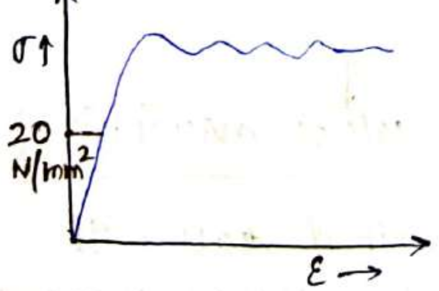
$\alpha_{steel} \simeq \alpha_{concrete}$

RCC

3 Methods to Design RCC

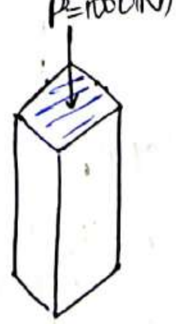
1) Working Stress Method (WSM)

Body me pahle strain aayega fir usko resist karne k liye stress aayega so ε in y-direction & σ in x-dirTn



$$\overset{D}{\overset{\uparrow}{y}} = f(\overset{I}{\overset{\uparrow}{x}})$$

$$\sigma = f(\varepsilon)$$



$$\sigma = P/A$$

$$A = P/\sigma = \frac{1000}{20} = 50mm^2$$

$$A = 50mm^2$$

Note :-

1) uneconomical larger sections are obtained

2) Members are stable & safe.

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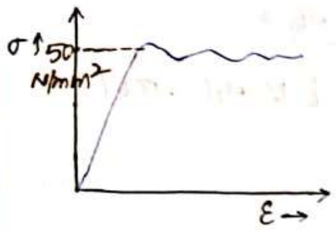
2) Ultimate load Method :-

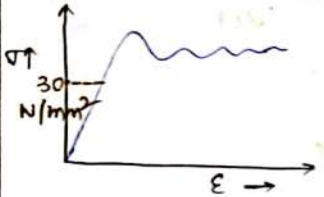


$A = P/\sigma = \frac{1000}{50} = 20mm^2$

$A = 20mm^2$

Note :-
1) cheaper sections are obtained
2) Members are unstable & unsafe
3) Selender members are obtained (means Jisme hum Buckling ki problem face karte hain)

3) Limit State Method [IS 456:2000]



$A = \frac{P}{\sigma} = \frac{1000}{30} = 33.33\ mm^2$

(Random values 1 values For understanding.)

Note :-
1) Economical Method
2) Members are safe & stable.

(permeable concrete can be used in parking area of home eg. car parking). which is used for water harvesting.

L2 :-

* Limit State Method

Limit State of collapse	Limit State of serviceability
① This method deals with SF, BM, TM, AF and all other types of forces which are going to occur on the structure throughout it's life. (AF - compression force method)	① This method deals with control on deflection, cracking, corrosion and abrasion.
② The resistance offered by the structure shall not be less than the load combinations of above forces.	② There should be control on deflection of the members [shall be discussed in detail]

(cracks se kuch nahi girta. cracks k jo effect aane wale rahte hai future me uske girta hai. eg water enters through cracks corrosion will occur means area kam hoga steel ka, due to corrosion.)

* The surface width of the cracks shall not exceed.....

0.3mm :- When cracks are not harmfull to the members. (inside house, building)

0.2 mm :- When the cracks are harmful for the members which are directly exposed to atmosphere, soil, sea shore, Rain etc.

0.1mm :- For extreme conditions eg underwater structures, offshores, chemical attacks on slab etc

③ Load combinations
- $1.5 DL + 1.5 IL$ ← for live loads → eg 180 kNm (B.M.)
- $1.2 (DL + IL \pm WL/EQ)$ → 300 kNm ✓
- $1.5\ DL \pm 1.5\ WL/EQ$ → 275 kNm
- $0.9\ DL \pm 1.5\ WL/EQ$ → 250 kNm

③ Load combinations
- $1.0 DL + 1.0 IL$
- $1.0\ DL \pm 1.0\ WL/ER$
- $1.0\ DL + 0.8\ [IL \pm WL/ER]$

building ready

(First ② Beams, slabs → columns & footings, staircase)

④ Design for limit state of collapse in
(i) Flexure (bending) (ii) Shear (shear) (iii) Torsion (torque hota ho)
(iv) compression.

④ Design for limit state of serviceability
i) Deflection ii) cracks iii) corrosion
iv) Abrasion

⑤ This method is based on the imaginary behaviour of the structure at the time of collapse.

⑤ This method is based on the actual behaviour of the structure during its service.

⑥ This method deals with the safety of the structure.

⑥ This method deals with durability of the structures.

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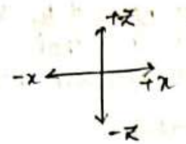
* DL & IL are considered +ve bcz they are Gravity forces so, taken +ve
* ± shows from which dirn force is coming.

<u>understanding load combinations</u>

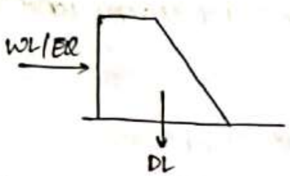


NO vertical (y) bcz very less effect

1.5 DL ± 1.5 WL/ER
↓
1.5 DL ± 1.5 WL ⇒ 2.5 DL + 1.5 WL_x 1.5 DL + 1.5 WL_z
eg 150kNm x
(8)   1.5 DL − 1.5 WL_x 1.5 DL − 1.5 WL_z

1.5 DL ± 1.5 ER ⇒ similar way
eg 300kNm ✓

only 90%
0.9 DL ± 1.5 WL/ER



WL & ER are not used at a time bcz @ it is very costly & from past history it is not been used at a time. life of building may be 50,100 & when WL & ER comes together in 50,100 years so it is to spend so much money.
If eg WD → 150kNm
ER → 300kNm use ER (we cannot under this)

Note:-
1) To consider the effect of overturning 0.9 DL ± 1.5 WL/ER combination has been considered. where only 90% Dead load is being considered.
2) Wind load & Earth quake load ~~should~~ shall not be applied together (Bcz it is costly)

13:- <u>Definitions</u>
1) <u>Characteristics strength</u>:- (mean Actual strength) It is the strength for which not more than 5% results are not expected to fail.

f_{ck} → characteristics compressive strength of concrete
f_y → characteristic/yield strength of steel.

For understanding only
eg For M20 N/mm²
100 cubes   then F_{ck} = 20 N/mm²
> 95 cubes
< 5 cubes

2) <u>Design strength</u>:- The characteristic strength is divided by FOS to get design strength.

$$\boxed{\text{Design strength} = \frac{\text{characteristic strength}}{\text{FOS}}}$$

<u>FOS</u>,
For concrete = 1.5
For Steel = 1.15 } For LSM [IS 456:2000]

eg temp vary, water quality, mixing etc.
FOS Steel ka kam → Bcz factory me banaya jata hai
FOS concrete ka → bcz site p banaya jata (where less quality control)
zyada

3) <u>Characteristic load</u>:- (service load) It is the load which has 95% propability of not exceeding throughout the life of the structure.

Dead load → IS 875 (Part 1)
Live load → IS 875 (Part 2)
Wind load → IS 875 (Part 3)
Snow load → IS 875 (Part 4)
Load combination → IS 875 (Part 5)
(Special loads)
ER loads → IS 1893:2002

4) <u>Design load</u> :- The characteristic load is multiplied by FOS.

$$\boxed{\text{Design load} = \text{characteristic load} \times \text{FOS}}$$

For strength ÷ by FOS
For load × by FOS

FOS For loads = 1.5 } LSM Design

14:- <u>Probability of failure of structure</u> [IS 456:2000]
3rd case is dependent on both. First 2 cases are independent of each other.

A structure can get failures when
(I) When applied loads exceeds characteristic load
(II) When strength of the material is less than its characteristic strength
(III) reason (i) & (ii) both occur together.

The probability of occuring 1st reason = 5% = 0.05
" 2nd " = 5% = 0.05
" 3rd " = 5% × 5% = 0.05 × 0.05
* Bcz both are ind Reasons

Jis material ka σ-ε curve me width (curve) zyada hota hai wo utna hi zyada ductile hota hai. Zyada ductile ka material steel me dalne se steel zyada time payega jaldi toot jayega nahi

The Total probability of failure of structure $= 0.05 \times 0.95 + 0.05 \times 0.95 + 0.05 \times 0.05$

$= 0.0975$. ×100 for %.

L5:-

Q2) $M_{DL} = 50$ kN-m, $M_{LL} = 80$ kN-m, $M_{WL} = 120$ kN-m, $M_{EL} = 180$ kN-m

$1.5 M_{DL} + 1.5 M_{LL} \longrightarrow 1.5 \times 50 + 1.5 \times 80 = 195$ kN-m

$1.2 (M_{DL} + M_{LL} + M_{WL}/M_{EL}) \longrightarrow 1.2(50 + 80 + 180) = 372$ kN-m

$1.5 M_{DL} + 1.5 \cdot M_{WL}/M_{EL} \longrightarrow 1.5 \times 50 + 1.5 \times 180 = 345$ kN-m

$0.9 M_{DL} + 1.5 M_{WL}/M_{EL} \longrightarrow 0.9 \times 50 + 1.5 \times 180 = 315$ kN-m

We have not take … case in question it is not given

L6:-

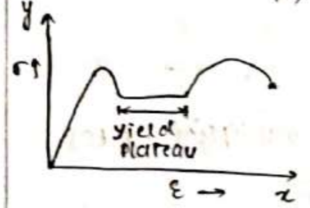
Type of Steel Reinforcements

(i) Mild Steel (MS):- (a) Yield Strength (f_y) = 250 N/mm²

(b) Fe 250 – grade of steel.

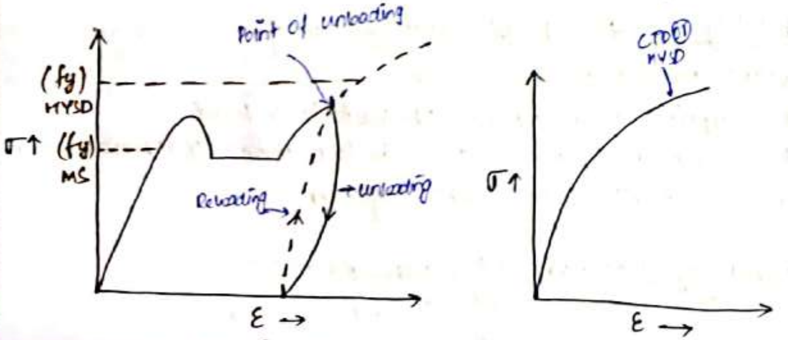
(c) This is the highly ductile grade of steel.

(d) In this method, steel, yield plateau is formed which is undesired.



$y = f(x)$
$\sigma = f(\varepsilon)$

(ii) Cold twisted deformed bars (CTD bars) (or)
High yield strength deformed bars (HYSD bars)



Cold working Process

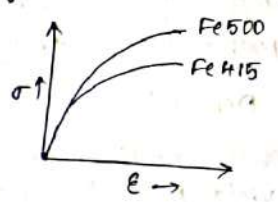
high yield strength hona bhi acchi baat hai.

We dont use square steel reinforcement box at sharp edge pe high stress concentration hota hai jiske karan steel fail hojata hai.

(a) To minimize the problem of yield plateau in the mild steel, Mild steel is stressed by the process of stretching or twisting beyond the yield plateau & then it is unloaded which causes some permanent strain in the material then it is again reloaded. This process is called cold working process.

(b) All the HYSD/CTD bars are essentially Mild steel.

(c) Yield strength ⇒ 415 N/mm² (Fe415)
⇒ 500 N/mm² (Fe500)

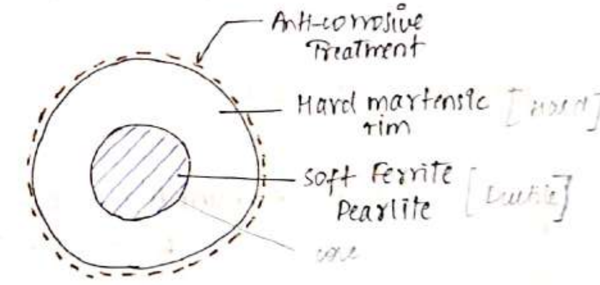


(d) Ductility is comparatively less

(e) Yield point is not well defined.

(iii) Thermo-Mechanically Treated bars (TMT-bars)

* Fe 415
* Fe500 } HYSD/CTD bars



TMT Bars

Ductility of MS > TMT > HYSD (or) CTD

* The cross-section of TMT-bars are provided with core of soft Ferrite pearlite material & outer surface is provided with core of Hard martensite rim.

Most of the metals after 0.002 strain ki yielding done start kardete hain is liye hamne 0.002 liya hai. bad

Merits of Mild Steel (Fe 250)

1) Ductility is higher.
2) Well defined yield point.

Demerits of Mild steel (Fe 250)

1) Lower yield strength.
2) Lower Bond strength.
3) Yield Plateau.

Merits of HYSD/CTD bars [Fe 415 & Fe 500]

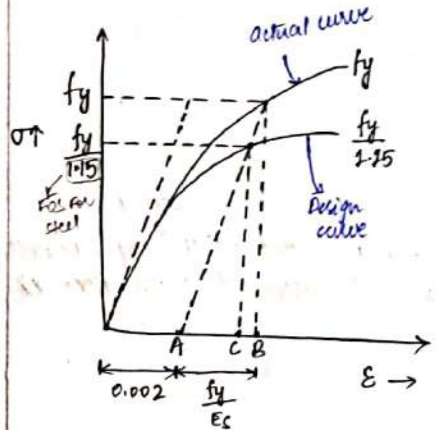
1) Higher yield strength.
2) Higher bond strength.

Demerits

1) Lower ductility as compare to mild steel.
2) Yield point is not well defined.

Note:- The maximum Grade of steel that can be used in Earthquake resistant design shall not be heigher than Fe 415.
(Bcz after that of Fe 500 its ductility reduces)

L7:- Determination of yield point in HYSD bars.



Strain at a point 'A' = 0.002

" " 'B' $= 0.002 + \frac{fy}{Es}$

'C' $= 0.002 + \frac{fy}{1.15\,Es}$

$$\boxed{\text{Design Strain} = 0.002 + \frac{0.87 fy}{E_s}}$$

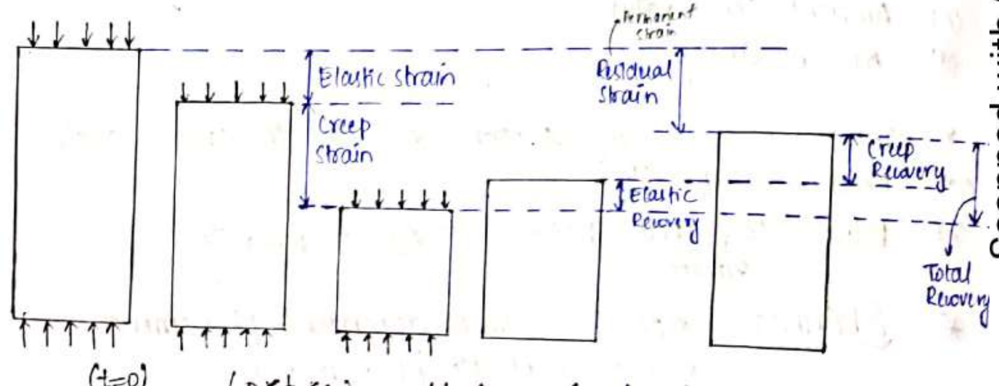
E_s = Young's modulus of steel

* Most of the metals start yielding after the strain of 0.2% (0.002).

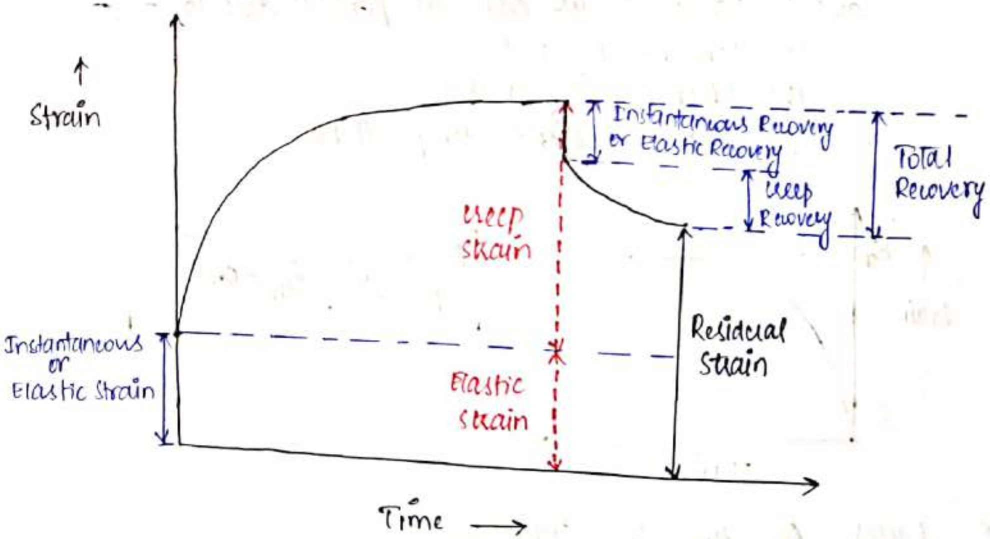
* $0.002 + \frac{0.87fy}{E_s}$ is the minimum strain that the steel reinforcement shall possess, as per [IS 456:2000]

L8:- Creep & Shrinkage in the concrete

Creep:- It is a Time dependent phenomenon which occurs due to the continuous Dead loadings.



t = time elapsed.



$$\text{Creep coefficient} = \frac{\text{Creep Strain}}{\text{Elastic strain}} \Rightarrow \boxed{\phi = \frac{\varepsilon_{cr}}{\varepsilon_e}}$$

IS 456:2000

Time	7 days	28 days	1 year
ϕ	2.2	1.6	1.1

Note:- Increase in creep with decreasing rate with time cause decrease in creep coefficient with the time.

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* Factors Affecting the creep Strain

* Increase in following factors increases the creep strain

(a) cement content [cement paste to aggregate Ratio]

(b) w/c Ratio (water ka ander ka part)

(c) Ambient Temperature (as pas ka tempt)

(d) Air entrainment

* Increase in following factors decreases the creep strain.

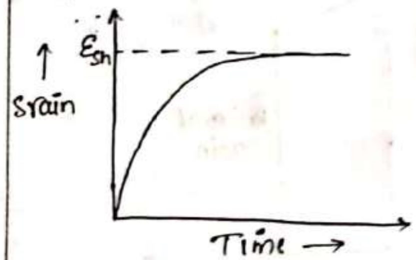
(a) Relative humidity (atmosphere ki humidity)

(b) Volume to surface area ratio (c) Age of concrete.

* Shrinkage:- * It is a time dependent phenomenon.
* It is a plastic phenomenon.

"Shrinkage is the reduction in the volume of the concrete due to loss of water from the pores" due to:-

(a) cement hydration
(b) chemical hydration
(c) high ambient temperature.



For design purpose, $\varepsilon_{sh} = 0.0003$

* Factors Affecting the shrinkage strain (shrink hona)

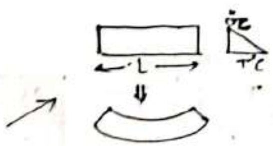
Increase in the following factors increases the shrinkage strain

(a) cement content

(b) w/c Ratio

(c) Ambient temperature.

(d) Temperature gradient in the member.



Increase in the following factors decreases the shrinkage strain.

(a) Relative humidity.

(b) Volume to surface area ratio.

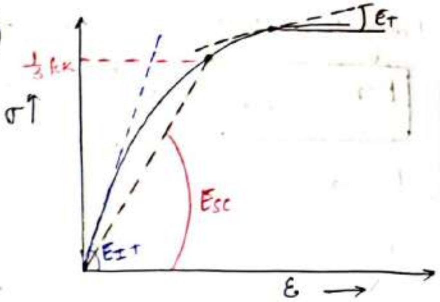
(c) Age of the concrete.

L9:- Modulus of Elasticity of concrete

1) Initial Tangent Modulus (E_{IT})

It is defined as the slope of a tangent drawn at the origin of the stress-strain curve of concrete.

It is also known as dynamic modulus of Elasticity.



2) Tangent Modulus (E_T)

It is defined as the slope of a tangent drawn at any point in the stress-strain curve.

$E_T < E_{IT}$

It is also defined as the ratio of the instantaneous stress to the instantaneous strain.

$$E_T = \frac{\text{Instantaneous stress}}{\text{Instantaneous strain}}$$

3) Secant Modulus of (E_{sc})

It is defined as the slope of a line which connects any chosen point on the stress-strain curve to the origin.

(or)

It is also defined as the ratio of total stress to the total strain at any point in the $\sigma-\varepsilon$ curve.

$$E_{sc} = \frac{\text{Total Stress}}{\text{Total strain}}$$

* As per IS 1343 code, secant modulus is calculated at $\frac{1}{3} f_{ck}$ ($0.3 f_{ck}$)

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* As per IS 456:2000, secant modulus is given in the terms of characteristic compressive strength of concrete as

$$E_c = 5000\sqrt{f_{ck}} \ N/mm^2$$

$E_c \rightarrow$ Short term static Modulus of Elasticity of concrete.
← without considering creep.

Long term static modulus of Elasticity of concrete

$$\boxed{E_{cr} = \frac{E_c}{1+\phi}} \Rightarrow \boxed{E_{cr} = \frac{5000\sqrt{f_{ck}}}{1+\phi}}$$

$E_{cr} \rightarrow$ long term static modulus of elasticity considering the effect of creep.

$E_c \rightarrow$ Short term static modulus of elasticity of concrete ($E_c = 5000\sqrt{f_{ck}}$ Mpa)

$\phi \rightarrow$ creep coefficient

L10:-

Q4) $E_c = 5000\sqrt{f_{ck}} - 5000 \times \sqrt{25} = 25000 \frac{N}{mm^2}$ or Mpa

Q5) $\phi = 1.5$

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

$E_{cr} = \frac{E_c}{1+\phi} = \frac{5000\sqrt{f_{ck}}}{1+\phi} = \frac{5000 \times \sqrt{25}}{1+1.5} = 20{,}000 \frac{N}{mm^2}$ or Mpa

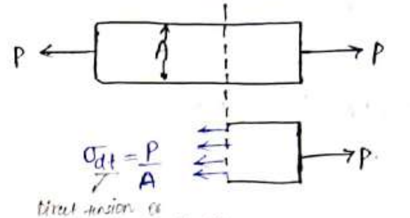
Q6) (See previous graph)

Q7) $f_{ck}\uparrow \rightarrow E_c\uparrow$ ($E_c = 5000\sqrt{f_{ck}}\uparrow$)

$f_{ck}\uparrow \rightarrow E_c\uparrow$

L11:- Tensile Strength of concrete

1) Direct tensile strength (under direct tension)



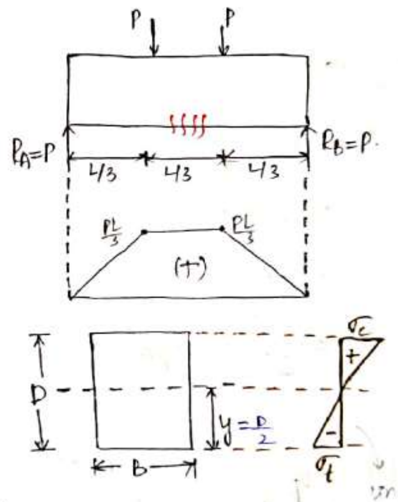
$\sigma_{at} = \frac{P}{A}$

direct tension or direct tensile strength

Under WSM. only

f_{ck} (N/mm^2)	σ_{at} (N/mm^2)
M10	1.2
M15	2.0
M20	2.8
M25	3.2
M30	3.6
M35	4.0
M40	4.4
M45	4.8
M50	5.2

2) Flexural Tensile Strength



Bending equation

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

$$\frac{M}{I} \cdot y = \sigma_t$$

$$\boxed{\frac{M}{\left(\frac{BD^3}{12}\right)} \cdot \left(\frac{D}{2}\right) = \sigma_t}$$

Here Bending k karan tension below the neutral axis.

σ_t = flexural tensile strength of concrete.

As per IS 456:2000

Flexural Tensile strength of concrete / Modulus of Rupture $= 0.7\sqrt{f_{ck}}$

$$\boxed{f_{cr} = 0.7\sqrt{f_{ck}}} \ (N/mm^2)$$

Note:-

$$\boxed{f_{ck} > f_{cr} > f_{at}}$$

f_{ck} ↓ characteristic comp. strength

f_{cr} ↓ flexural tensile strength

f_{at} ↓ (σ_{at}) direct tensile strength.

L12: Q8) $f_{cr} = 0.7\sqrt{f_{ck}}$

↑ Tensile strength of concrete under bending.

↑ $f_{ck} > f_{cr} > f_{ct}$ ↑

Q9) $f_{ck} = 25\ N/mm^2$

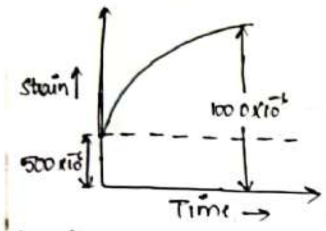
$f_{cr} = 0.7\sqrt{f_{ck}} = 0.7\sqrt{25} = 0.7 \times 5 = 3.5\ N/mm^2$ or Mpa

Q10) $\sigma = 12.5$ Mpa

$f_{ck} = 25$ Mpa

elastic strain — $\varepsilon_e = 500 \times 10^{-6}$

strain at total — $\varepsilon_{\theta} = 1000 \times 10^{-6}$



$\varepsilon_{creep} = (1000 - 500) \times 10^{-6} = 500 \times 10^{-6}$

$\varepsilon_{elastic} = 500 \times 10^{-6}$

$\phi = \dfrac{\varepsilon_{creep}}{\varepsilon_{elastic}} = \dfrac{500 \times 10^{-6}}{500 \times 10^{-6}} = 1$

$E_{cr} = \dfrac{5000\sqrt{f_{ck}}}{1+\phi}$

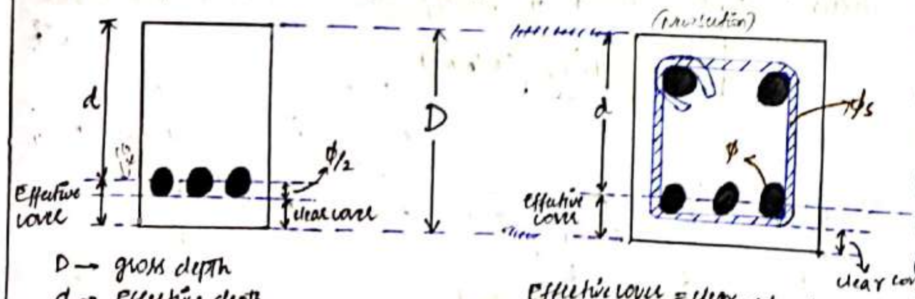
$= \dfrac{5000\sqrt{25}}{1+1}$

$= \dfrac{25{,}000}{2}$

$E_{cr} = 12{,}500$ Mpa

L13: Durability Requirements

Clear cover:- It is the margin/distance of the outer most Reinforcement from the outer surface (Extreme fibre) of the member. This member can be beam, slab, column, footing etc



D → gross depth

d → effective depth

d = D − effective depth cover

$\boxed{d = D - \text{clear cover} - \phi/2}$

Effective cover = clear cover + ϕ_s + $\frac{\phi}{2}$

d = D − Effective cover.

clear cover is imp kyu agar nahi rakhenge to cracks k baad steel expose to atmp hoga corrosion hoga aur area kam hoga. agar zyada rakhenge to tension k karan cracks dikhenge but under limit.

Minimum Nominal clear cover

Members	IS 456:1978	IS 456:2000
Slab	15 mm	20 mm
Beam	25 mm	25 mm
Column	40 mm/25 mm	40 mm/25 mm
Footing	75 mm/50 mm	75 mm/50 mm

Durability Requirements for concrete.

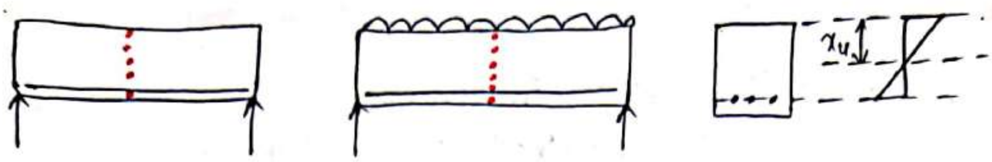
Exposure condition	Durability Table	
	For PCC	For RCC
Mild	M15 can be use here	M20 → max use nowadays
Moderate	M15	M25
Sever	M20	M30
Very Sever	M20	M35
Extreme	M25	M40.

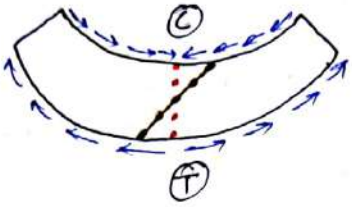
mild → normal condition (no heavy rain, no heavy temp, no heavy humidity).

CHAPTER:- 02 LSM of collapse - Flexure [IS 456 : 2000]

→ Bending.

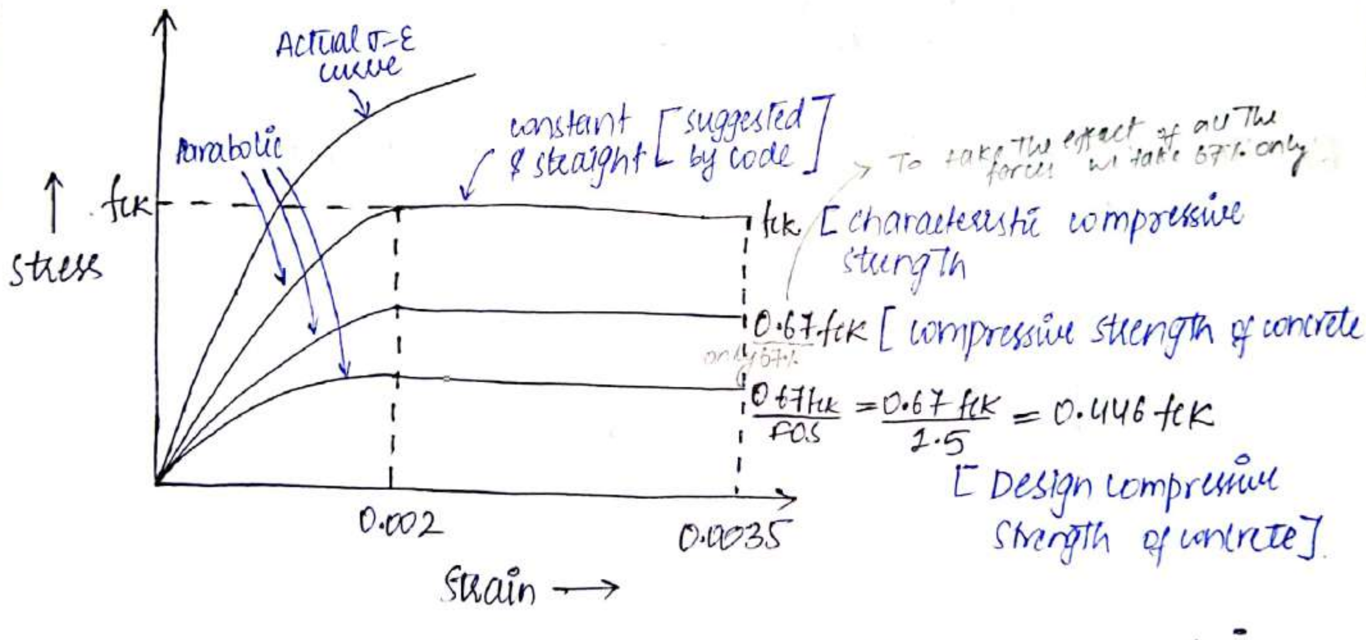
1.1 Assumptions :- ① The plane section remains plane before and after the bending. but sirf tilt hoga plane



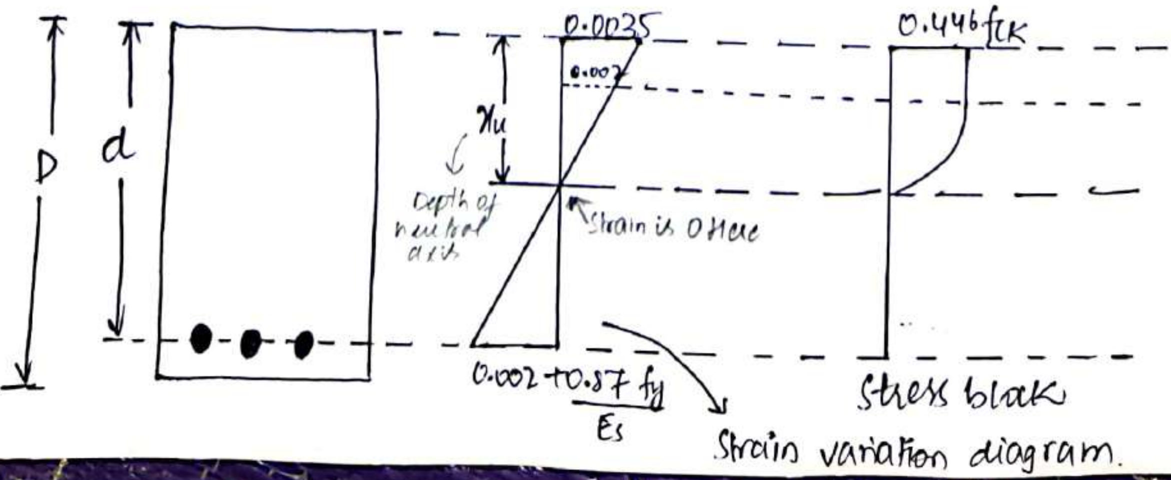


That means strain variation is linear from top to bottom and changes its sign from compression to tension.

2> The maximum compressive strain in the concrete can be taken as 0.0035 in flexure. agar 0.0035 se age jaane to hame bht cracks milenge concrete me.



3> The stress block is parabolic from zero strain to 0.002 strain & rectangular from 0.002 strain to 0.0035 strain.



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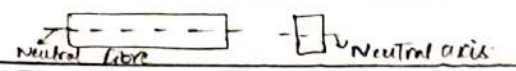


4) The Tensile Strength of concrete is ignored. (Below the neutral axis)

conclusion [If I am ignoring the Tensile strength of concrete (i) concrete is weak in Tension (ii) All the tensile strength is taken by steel only & compressive stress by concrete (iii) Area of concrete below the neutral axis I am ignoring]

That means—(i) concrete is weak in Tension.
(ii) area of concrete below the neutral axis is ignored
(iii) All the Tensile stresses shall be taken by steel reinforcement only.

[discussed above 3 points as per cracked section Theory which say concrete below the Neutral axis can be considered to be cracked].

5) The FOS for concrete & steel can be taken as 1.5 & 1.15 respectively.

$(FOS)_{concrete} > (FOS)_{steel}$ [Bcz steel factory me banayi jati hai]

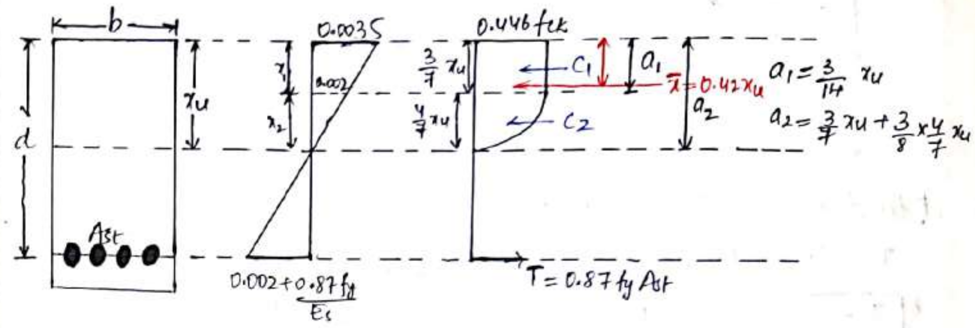
6) The maximum Tensile strain in the steel ε_s shall not be less than $0.002 + \frac{0.87 f_y}{E_s}$

$\boxed{(\varepsilon_{steel})_{min} = 0.002 + \frac{0.87 f_y}{E_s}}$ → f_y: grade of steel; E_s: modulus of elasticity of steel ($2\times10^5 \frac{N}{mm^2}$)

means this is minimum strain in steel

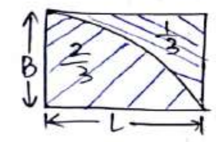
Ast → Area of steel in Tension zone
Asc → Area of steel in compression zone

L2:- Analysis of Singly Reinforced Beam Section



$\frac{0.0035}{x_u} = \frac{0.002}{x_2}$ $\quad$ $\because x_1 + x_2 = x_u$

$\boxed{x_2 = \frac{4}{7}x_u}$ $\quad$ $\boxed{x_1 = \frac{3}{7}x_u}$



Estimation of compressive Forces

$C_1 = (0.446 f_{ck})\, b x_1$ (force = stress × area)

$\boxed{C_1 = (0.446 f_{ck})\, b \cdot \frac{3}{7} x_u}$

$C_2 = (0.446 f_{ck}) \frac{2}{3} b x_u$

$\boxed{C_2 = (0.446 f_{ck}) \frac{2}{3} \cdot b \cdot \frac{4}{7} x_u}$

$\because C = C_1 + C_2$

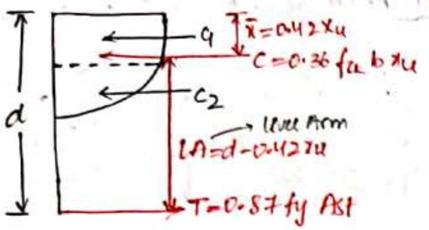
* $\boxed{C = 0.36 f_{ck} b x_u}$ (N) → Total compressive Force

Note:— C_1 is occuring at a distance a_1 from the Top & C_2 is occuring at a distance a_2 from the top. Therefore assuming total compressive force 'C' will be occuring at any distance $\bar{x}$ from the Top.

$\bar{x} = \frac{C_1 a_1 + C_2 a_2}{C_1 + C_2}$

$\boxed{\bar{x} = 0.42 x_u}$

comp force & Tensile force along the length laghe hain [not along the cross section]

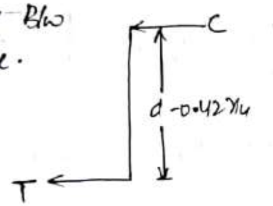


* Estimation of Total Tensile force

$T = (0.87 f_y) A_{st}$ — It is occurring at the C.G of steel reinforcement

* $\boxed{T = 0.87 f_y A_{st}}$ (N)

* Lever Arm:- It is the vertical distance b/w the total compressive force & Total Tensile force.



* $\boxed{LA = d - 0.42x_u}$

$(MOR)_c = C \cdot LA$

$\boxed{(MOR)_c = 0.36 f_{ck} b x_u (d - 0.42x_u)}$ moment of resistance in compressive zone (MOR)c about the neutral axis

$(MOR)_T = T \cdot LA$

$\boxed{(MOR)_T = 0.87 f_y A_{st} (d - 0.42x_u)}$ " " Tension zone below "

MOR → Moment of Resistance against the applied Bending moment.

Todne se bachane wali Taqat hai? ↳ Beam ko Todne wali Taqat

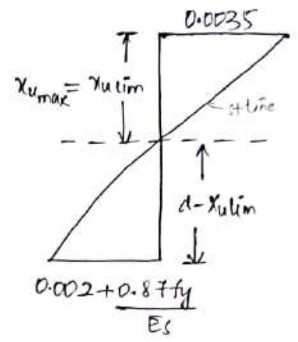
---

also called max depth of Neutral axis

L3:- Limiting depth of Neutral axis

Jahan par bending stress zero hogaye means Total comp force Total Tensile force k barabar hogaye.

Max depth of Neutral axis wo depth of neutral axis hai jis pe concrete apni max permissible strain jo 0.0035 hai ya fir max permissible stress par pahunch chuka ho. & at the same time steel bhi apni max permissible strain ya fir max permissible stress par pahunch chuka ho aise section k andar jo depth of neutral axis hogi unko hum limiting depth of Neutral axis ya max depth of neutral axis kahte hai.



By similar triangle property.

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

$$\frac{d - x_{ulim}}{x_{ulim}} = \frac{0.002 + \frac{0.87 f_y}{2 \times 10^5}}{0.0035}$$

$$\boxed{x_{ulim} = \frac{700}{1100 + 0.87 f_y} \cdot d}$$

$x_{ulim} = k \cdot d$

$k \rightarrow$ Neutral axis constant (depends only on grade of steel (f_y))

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

In the given crosssection of the beam the max depth of Neutral axis or limiting depth of neutral axis depends upon the grade of steel only.

grades of steel	Neutral Axis constant (k)
Fe 250	0.53
Fe 415	0.48
Fe 500	0.46

By putting $250 = f_y$ in k formula

$(MOR)_c = 0.36 f_{ck} b x_u (d - 0.42x_u)$

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

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

$M_{ulim} = 0.36 f_{ck} b \cdot k d^2 (1 - 0.42k)$

$\boxed{M_{ulim} = Q f_{ck} b d^2}$

$\boxed{Q = 0.36k(1 - 0.42k)}$

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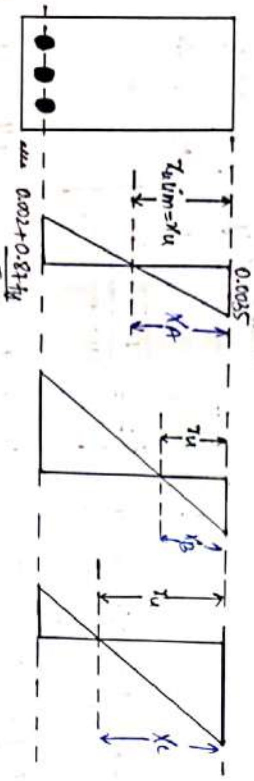
0.446 fck   0.87fy

eg for Fe250; $R = 0.36 \times 0.53 \times (1 - 0.42 \times 0.53)$
$R = 0.148$.
$\therefore M_{ulim} = 0.148\, f_{ck}\, bd^2$

Grade of Steel (F_y)	Maximum depth of Neutral axis ($X_{ulim} = k \cdot d$)	Maximum Moment carrying capacity of the section ($M_{ulim} = R f_{ck} bd^2$)
Fe 250	$X_{ulim} = 0.53d$	$M_{ulim} = 0.148\, f_{ck}\, bd^2$
Fe 415	$X_{ulim} = 0.48d$	$M_{ulim} = 0.138\, f_{ck}\, bd^2$
Fe 500	$X_{ulim} = 0.46d$	$M_{ulim} = 0.133\, f_{ck}\, bd^2$

L4:- Types of RCC sections



Balanced section   under-reinforced section   Over-reinforced Section

In B.S. max depth of neutral axis reaches to lmt of neutral axis.

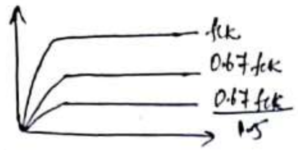
Doubt   Decreasing Order
$X_B > X_A > X_C$

Balanced Section	Under-Reinforced section	Over-Reinforced section
(i) $\sigma_c \geq \sigma_{c\,perm}$	(i) $\sigma_c < \sigma_{c\,perm}$	(i) $\sigma_c \geq \sigma_{c\,perm}$
(ii) $\sigma_t \geq \sigma_{t\,perm}$	(ii) $\sigma_t \geq \sigma_{t\,perm}$	(ii) $\sigma_t < \sigma_{t\,perm}$
(iii) $X_u = X_{ulim}$	(iii) $X_u < X_{ulim}$	(iii) $X_u > X_{ulim}$
(iv) MOR = M_{ulim}	(iv) These type of section can be achieved by providing limited amount of steel reinforcement as compare to balanced section.	(iv) These type of section can be achieved by providing excessive amount of steel reinforcement as compare to Balanced section.
NOTE:- All the section shall be tried to design as a balanced section.	(v) steel will fail first.	(v) concrete will fail first.
	(vi) Type of failure is called ductile failure.	(vi) Type of failure is called brittle failure
	(vii) We get Alarm.	(vii) No Alarm. (Bcz sudden failure)
		Say No to Over-Reinforced section as a good Engg Bcz No alarm

If Balance section → 10 steel bars   If 9 → Under RS
If 11 → Over RS

L5:- Q1✓ $C = (0.36 f_{ck}\, b\, x_u) \times 1.5$

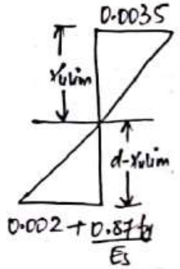
$C = \underline{0.54}\, f_{ck}\, b \cdot x_u$



Q2✓ ✓

Q3✓ ✓

Q4✓



$\varepsilon_{steel} = 0.002 + \dfrac{0.87 \times 415}{200 \times 10^3}$

$\varepsilon_{steel} = 0.0038$

$\varepsilon_{concrete} = 0.0035$

L6:- Expected type of Problems from singly reinforced Beam

Type ① calculate Moment of Resistance [$f_{ck}, f_y, A_{st}, b, d$] Given in Que

Step 1: Calculate X_{ulim}

$X_{ulim} = k \cdot d$

$= 0.53d$ (Fe250)

$= 0.48d$ (Fe 415)

$= 0.46d$ (Fe 500)

Step 2: Calculate actual depth of Neutral axis

$C = T$

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

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

Step 3: compare x_u & X_{ulim}

If $x_u = X_{ulim}$ (Balanced section)

→ use X_{ulim}

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

$$\boxed{M_{ulim} = Q f_{ck} b d^2}$$

$M_{ulim} = 0.148 f_{ck} b d^2$ (Fe250)

$M_{ulim} = 0.138 f_{ck} b d^2$ (Fe415)

$M_{ulim} = 0.133 f_{ck} b d^2$ (Fe500)

M_{ulim} → Maximum moment resisting capacity.

If $x_u < X_{ulim}$ (Under-reinforced section)

→ use Actual depth of Neutral axis (i.e. x_u)

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

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

} we get same ans by using any formula of MOR.

If $x_u > X_{ulim}$ (over reinforced section)

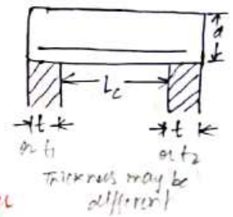
→ use X_{ulim} (Max depth of Neutral axis)

$(MOR)_c = 0.36 f_{ck}\, b\, X_{ulim} (d - 0.42 X_{ulim})$

→ Here sectional dimension is not known

Type ②: Design a singly Reinforced beam [f_{ck} f_y w_u l_{eff} b] Given in Que (Applied load; span of beam)

Step 1: Effective span



$l_{eff} = L_c + d$ (clear span)

or

$L_c + \frac{t}{2} + \frac{t}{2}$

Take whichever is less

Step 2: Calculate Max. Bending moment

$$\boxed{M_{max} = \frac{w_u l_{eff}^2}{8}}$$

Step 3: Calculate Required effective depth

→ Design a Balanced section

$M_{max} = M_{ulim} = Q f_{ck} b d^2$

$$\boxed{d_{req} = \sqrt{\frac{M_{max}}{Q f_{ck} b}}}$$

$Q = 0.148$ (Fe 250)

$Q = 0.138$ (Fe 415)

$Q = 0.133$ (Fe 500)

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$M_u \rightarrow$ applied bending moment
$M_{ulim} \rightarrow$ Limiting moment of resistance or max moment carrying capacity

Step 4: Design of area of reinforcement

$C = T$ (Balanced section)

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

$$\boxed{A_{st} = \frac{0.36 f_{ck}\, b\, x_{ulim}}{0.87 f_y}} \quad \text{— (1)}$$

$M_{max} = MOR = 0.87 f_y A_{st} (d - 0.42 x_{ulim})$

$$\boxed{A_{st} = \frac{M_{max}}{0.87 f_y (d - 0.42 x_{ulim})}} \quad \text{— (2)}$$

unit — N mm

same ans from all 3 will come

$$\boxed{A_{st} = 0.5 \frac{f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_{max}}{f_{ck} b d^2}}\right] b d} \quad \text{— (3)}$$

→ here crosssectional dimension is given (b d)

Type (3): Design a singly Reinforced Beam (fck fy wu leff b d)

Step 1: calculate max bending moment

$$\boxed{M_u = \frac{W_u\, l_{eff}^2}{8}}$$

Step 2: Calculate MOR (M_{ulim}) for Balanced Section

$$\boxed{M_{ulim} = Q\, f_{ck}\, b d^2}$$

Step 3: Compare M_u & M_{ulim}

If $M_u < M_{ulim}$ (Todne wali Taqat kam hai bachane wali Taqat zyada hai) (Tab hum singly reinf Beam design karenge)

→ Design a singly reinforced Beam

$$A_{st} = \frac{0.36 f_{ck}\, b\, x_{ulim}}{0.87 f_y} \text{ — (1)} \quad A_{st} = \frac{M_u}{0.87 f_y (d - 0.42 x_{ulim})} \text{ — (2)}$$

$$A_{st} = 0.5 \frac{f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] b d \quad \text{— (3)}$$

force = Area × Stress

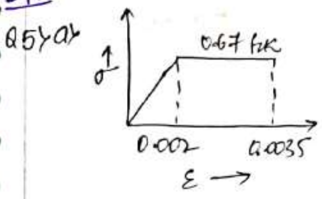
If $M_u > M_{ulim}$

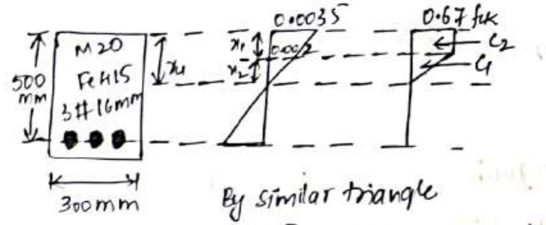
(I) → Increase the cross-sectional dimensions or

(II) → Design doubly reinforced beam without increasing the crosssectional dimensions.

L7:-

Q5) a)





By similar triangle

$\frac{0.0035}{x_u} = \frac{0.002}{x_1} \Rightarrow x_1 = \frac{4}{7} x_u$

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

Ist part

$\boxed{C = T}$

$C = C_1 + C_2$ = ½ × base × stress

$C = \frac{1}{2}[0.67 f_{ck}] \frac{4}{7} x_u \cdot b + [0.67 f_{ck}] \times \frac{3}{7} x_u \cdot b$

= ½ × stress × longitudinal

$C = \frac{1}{2} \times 0.67 \times 20 \times \frac{4}{7} x_u \times 300 + [0.67 \times 20] \times \frac{3}{7} \times x_u \cdot (300)$

$C = 1148.57 x_u + 1722.85 x_u$

$C = 2871.43\, x_u$

$T = 0.87 f_y A_{st}$

$T = 0.87 \times 415 \times 3 \times \left[\frac{\pi}{4} \times (16)^2\right]$

$T = 217713.15\, N$

$C = T$

$2871.43\, x_u = 217713.15$

$x_u = 75.82\, mm$

$x_u \simeq \underline{76\, mm}$

Diameter Bar	Area (mm²)
8 mm φ →	50.28
10 mm φ →	78.5
12 mm φ →	113
16 mm φ →	201
20 mm φ →	314
25 mm φ →	490.25

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5 b) 2nd part

$C = 0.36\, f_{ck}\, b\, x_u$

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

$\boxed{C=T}$

$x_u = \dfrac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \dfrac{0.87 \times 415 \times [3 \times 201]}{0.36 \times 20 \times 300} \Rightarrow x_u = 100.79\ mm$

$x_u \simeq 101\ mm$

[IS 456 : 2000]

Difference $= 101 - 76$
$= 25mm$

L8:- Q6)

Step1: x_{ulim}

$x_{ulim} = k \cdot d$
$= 0.48d$
$= 0.48 \times 300$
$x_{ulim} = 144\ mm$

Step 2: x_u

$x_u = \dfrac{0.87 f_y A_{st}}{0.36 f_{ck} b} \overset{C=T}{=} \dfrac{0.87 \times 415 \times [4 \times \frac{\pi}{4}(12)^2]}{0.36 \times 25 \times 200}$

$x_u = 90.664\ mm$

Step3: compare x_u & x_{ulim}

$x_u < x_{ulim}$ (URS)

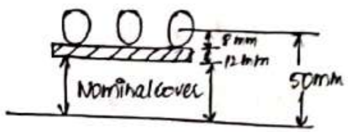
→ Use x_u to calculate MoR

[If you use $(MOR)_T$ ans will be same]

$(MOR)_c = 0.36 f_{ck}\, b\, x_u\, (d - 0.42 x_u)$
$= 0.36 \times 25 \times 200 \times 90.664 \times [300 - 0.42 \times 90.664]$
$= 42744269.56\ N\text{-}mm$

$MOR = \underline{42.74\ kN\text{-}m}$

Q7)

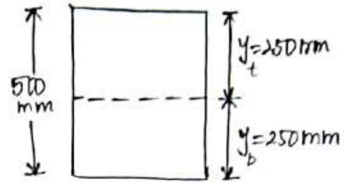


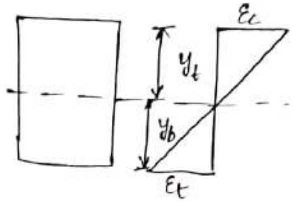
Nominal cover $= 50 - (8+12)$
$= \underline{30mm}$

$y_t \rightarrow y$ top $\quad y_b \rightarrow$ bottom $\quad y$ bottom

$I = \dfrac{Bd^3}{12}$ $\quad \sigma$ = Bending stress

Q8)





$\varepsilon_c = \varepsilon_t = 2.5 \times 10^{-4}$

$\rho = \dfrac{1}{R} \rightarrow$ curvature (m^{-1})

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

$\dfrac{\sigma}{y} = \dfrac{E}{R}$

$\dfrac{1}{R} = \dfrac{\sigma}{E} \cdot \dfrac{1}{y}$

* $\boxed{\dfrac{1}{R} = \dfrac{\varepsilon}{y}}$ $\quad$ curvature $= \dfrac{\text{strain}}{y}$

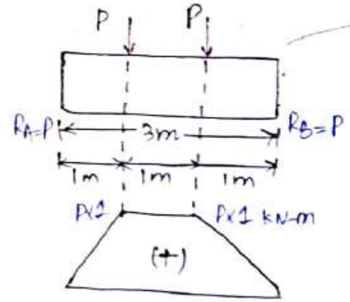
$\rho = \dfrac{1}{R} = \dfrac{\varepsilon}{y} = \dfrac{2.5 \times 10^{-4}}{0.25 \rightarrow \text{in } m}$ $\quad$ 250mm = 0.25m

$\rho = 1 \times 10^{-3}\ m^{-1}$

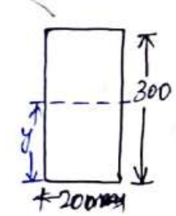
$\rho = \underline{0.001\ m^{-1}}$

L9:- Q9) a)

$b = 200mm$
$D = 300\ mm$
$A_{st} = 2 \times 201$
$d = 300 - 40$
$d = 260mm$
$f_{ck} = 20\ N/mm^2$
$f_t = 2.2\ N/mm^2$



No steel in it



$Y = \dfrac{300}{2} = 150mm$

$M_{max} = P \times 1\ kN\text{-}m$

$M_{max} = P\ kN\text{-}m$ (applied Bending moment)

1st part

$\dfrac{M}{I} = \dfrac{\sigma}{y}$ → This formula is valid when material is Homogeneous.

$\dfrac{P \times 10^6}{\left(\dfrac{200 \times 300^3}{12}\right)} = \dfrac{2.2}{150}$

$\underline{P = 6.6\ KN}$

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2nd part

$X_{ulim} = 0.48d$

$X_{ulim} = 0.48 \times 260$

$X_{ulim} = 124.8\ mm$

$X_u = \dfrac{0.87 f_y A_{st}}{0.36 f_{ck} b}$

$X_u = \dfrac{0.87 \times 415 \times (2 \times 201)}{0.36 \times 20 \times 200} = 100.79\ mm$

$\because X_{ulim} > X_u$

This section is URS

$\Rightarrow$ use X_u

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

$MOR = 0.36 \times 20 \times 200 \times 100.79 \times [260 - 0.42 \times 100.79]$

$MOR = 31.59\ kN\text{-}m$ [ans will be come in mm × by 10^6 in kN-m]

For equilibrium

$M_u = MOR$

$P = 31.59\ kN \approx \underline{31.6\ kN}$

Sol:- Q.20

$b = 250\ mm$

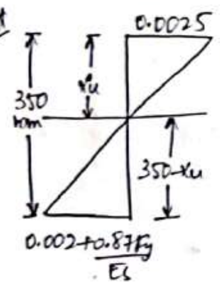
$d = 350\ mm$

$A_{st} = 1500\ mm^2$

M20

Fe250

1st part



$\dfrac{0.0025}{x_u} = \dfrac{0.002 + \frac{0.87 f_y}{E_s}}{350 - x_u}$

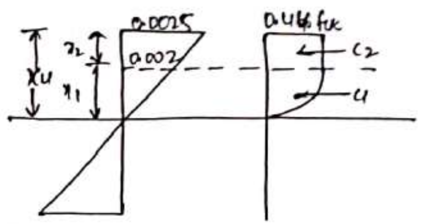
$\dfrac{350 - x_u}{x_u} = \dfrac{0.002 + \frac{0.87 \times 250}{E_s\ 2 \times 10^5}}{0.0025}$

$\dfrac{350}{x_u} - 1 = 1.235$

$x_u = 156.6\ mm \approx 156\ mm$

2nd part

The Formula C = 0.36 fck b xu cannot be use b/z in this we have taken 0.0035 but here in Que 0.0025 is given.



$\dfrac{0.0025}{x_u} = \dfrac{0.002}{x_1}$ $\qquad x_1 + x_2 = x_u$

$x_1 = 0.8 x_u$ $\qquad x_2 = 0.2 x_u$

Total compressive force

$C = C_1 + C_2$

$C_1 = \frac{2}{3} \times (x_1) \times B \times 0.446 f_{ck}$

$C_1 = \frac{2}{3} \times [0.8 \times 156.6] \times 250 \times 0.446 \times 30$

$C_1 = 279.37\ kN$ (by × 10^{-3} we get this)

$C_2 = (0.446 f_{ck}) \times (B x_2)$

$C_2 = 0.446 \times 30 \times 250 \times [0.2 \times 156.6]$

$C_2 = 104.76\ kN$ (")

$C = C_1 + C_2 = 384.135\ kN \approx \underline{389\ kN}$

Singly R.B is design when $M_u < M_{u,lim}$ (Jab factor wali ignt kam ho aur bechane wali target zyada ho)
Doubly R.B is design when $M_u > M_{u,lim}$ (When we donot want to increase section)

L11:-
Q12) b = 300mm
d = 400mm
M25, Fe500

Estimation of No. of bars for Balanced Section

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

$0.36 \times 25 \times 300 \times [0.46 \times 400] = 0.87 \times 500 \times A_{st}$

$A_{st} = 1142.069\ mm^2$

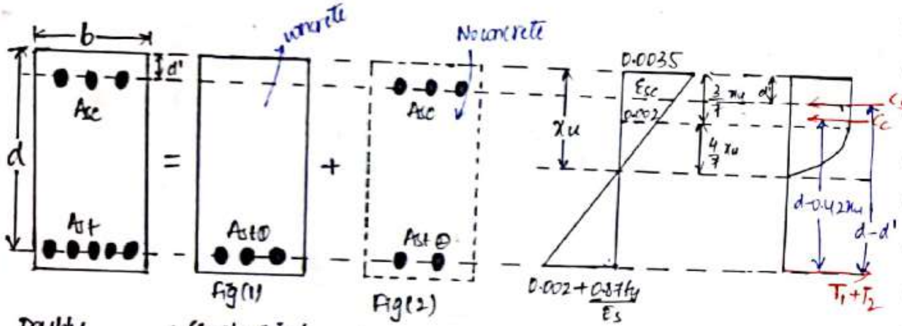
$N = \dfrac{A_{st}}{\frac{\pi}{4}(16)^2} = \dfrac{1142.069}{201} = 5.68$

$N = 5.68$ bars $\Rightarrow$ $\cancel{6}$

5 4 3 2 1 [Bcz 6 is for Balanced Section]

L12:-

Analysis of Doubly Reinforced Beam



Fig(1) Fig(2)

Doubly Reinforced section
→ Singly Reinf Balanced Section
→ $M_{u,lim}$
→ $C_c = T_1$

→ $M_u - M_{u,lim}$
→ $C_s = T_2$

Eg M_u = 450 kN-m
$M_{u,lim}$ = 400 kN-m

C_c → Total compressive force due to concrete
C_s → " steel
ε_{sc} → Strain in steel in comp zone
d' → d' depth pe hum Reinf provide kar rahe hain
T_1 → Tensile force in fig ①
T_2 → Tensile force in fig ②

Lever arm in fig ① $= d - 0.42\,x_u$
Lever arm in fig ② $= d - d'$

f_{sc} → stress in steel in comp zone ⓞ stress in compressive steel.

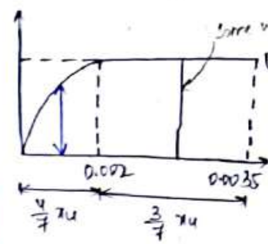
Analysis of compressive forces

In fig(1)

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

In fig (2)

$\boxed{C_s = f_{sc} A_{sc} - (0.446 f_{ck}) A_{sc}}$

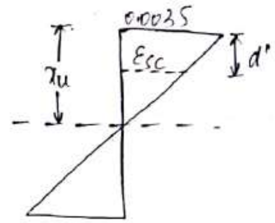


Here $d' < \frac{3}{7} x_u$
[So 0.446fck stress]
If $d' > \frac{3}{7} x_u$

Total compressive force $C = C_c + C_s$

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

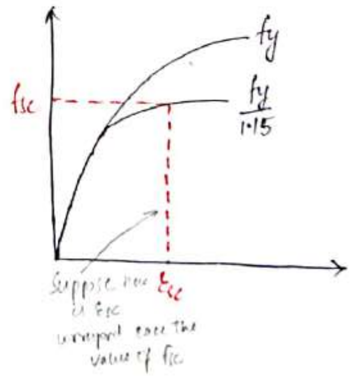
To find f_{sc} :-



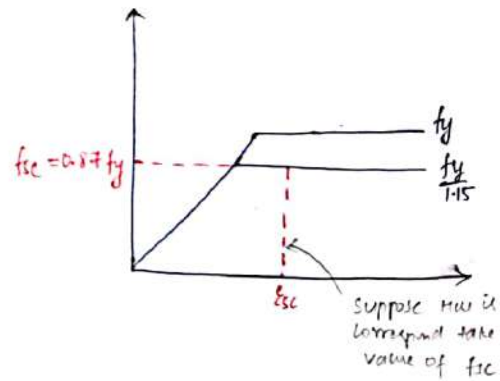
$\dfrac{0.0035}{x_u} = \dfrac{\varepsilon_{sc}}{x_u - d'}$

$\boxed{\varepsilon_{sc} = 0.0035\left(1 - \dfrac{d'}{x_u}\right)}$

For HYSD Bars



For Mild steel [$f_{sc} = 0.87 f_y$]



SRS (singly reinforced) beam → Balance beam, URS, ORS
DRS (Doubly Reinf.) beam →

Analysis of Tensile forces

$$T = T_1 + T_2$$

$$T = 0.87 f_y Ast_{①} + 0.87 f_y Ast_{②}$$

$$T = 0.87 f_y (Ast_{①} + Ast_{②})$$

$$\boxed{T = 0.87 f_y Ast}$$

Moment of Resistance

$$(MOR)_c = C_c (LA)_1 + C_s (LA)_2$$

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

$$(MOR)_T = T_1 (LA)_1 + T_2 (LA)_2$$

$$\boxed{(MOR)_T = 0.87 f_y Ast_{①} (d - 0.42 x_u) + 0.87 f_y Ast_{②} (d - d')}$$

— x —

Unrestricted dimension (b, d) Doubly reinforced beam me freeze kiye jate hain.

L23:- Expected Type of Problems from Doubly Reinforced Beam.

Problem Type① Design of Doubly Reinf. Beam [w_u, l_{eff}, f_{ck}, f_y, b, d]

Step(1) Calculate maximum Bending Moment

$$\boxed{M_u = \frac{w_u l_{eff}^2}{8}}$$

Step(2) Calculate M_{ulim}

$$\boxed{M_{ulim} = Q f_{ck} b d^2}$$

$Q = 0.148$ (Fe250)
$Q = 0.138$ (Fe415)
$Q = 0.133$ (Fe500)

Step(3) Compare M_u & M_{ulim}

if $M_u < M_{ulim}$
→ Design a Singly Reinforced Beam

if $M_u > M_{ulim}$
→ Design a Doubly Reinforced Beam

Step(4) Calculate Area of reinforcements (Ast_1, Ast_2, Asc)

also called limiting area of reinforcement Ast_{lim}

(i) Estimation of $Ast_{①}$

$$(MOR)_T = M_{ulim} = 0.87 f_y Ast_{①} (d - 0.42 x_{ulim})$$

$$\boxed{Ast_{①} = \frac{M_{ulim}}{0.87 f_y (d - 0.42 x_{ulim})}}$$

(ii) Estimation of $Ast_{②}$

$$\boxed{\text{Total } Ast = Ast_{①} + Ast_{②}}$$

$$M_u - M_{ulim} = 0.87 f_y Ast_{②} (d - d')$$

$$\boxed{Ast_{②} = \frac{M_u - M_{ulim}}{0.87 f_y (d - d')}}$$

$Ast_{②}$ hum additional B.M ko resist karne ke liye dalte hain.

$$\boxed{\text{Number of bars} = \frac{A_{st}}{\frac{\pi}{4}(\phi^2)}}$$

valid for both singly & doubly reinf beam also in both tension & comp

(iii) Estimation of A_{sc}

(A) 1st approach

$$M_u - M_{ulim} = C_s (LA)_2 = A_{sc}(f_{sc} - 0.446 f_{ck})(d-d')$$

$$\boxed{A_{sc} = \frac{M_u - M_{ulim}}{(f_{sc} - 0.446 f_{ck})(d-d')}} \quad \text{—(1)}$$

(B) 2nd approach

$$C_s = T_2$$

$$A_{sc}(f_{sc} - 0.446 f_{ck}) = 0.87 f_y A_{st2}$$

$$\boxed{A_{sc} = \frac{0.87 f_y A_{st2}}{(f_{sc} - 0.446 f_{ck})}} \quad \text{—(2)}$$

* Problem Type(2): Calculate Moment of Resistance [f_{ck} f_y b d d' A_{st} A_{sc}]

Step(1): Calculate X_{ulim}

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

$k = 0.53$ (Fe250)
$k = 0.48$ (Fe415)
$k = 0.46$ (Fe500)

Step(2): Calculate Actual depth of Neutral axis

$$C = T$$

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

if Fe 250 then $f_{sc} = 0.87 f_y$
if Fe 415 or Fe 500 then f_{sc} by graph

where $\varepsilon_{sc} = 0.0035\left[1 - \frac{d'}{x_u}\right]$

From the above Eqn x_u can be calculated. [By Number of Iterations]

Step(3): Compare X_u & X_{ulim}

if $x_u = x_{ulim}$ (Balanced section)
→ use x_{ulim} to estimate MOR

if $x_u < x_{ulim}$ (under Reinforced section)
→ use x_u to estimate MOR

if $x_u > x_{ulim}$ (over Reinforced section)
→ use x_{ulim} to estimate MOR

Note:- (i) The design of over-reinforced section is not allowed as per IS 456:2000

(ii) If in any case we find $x_u > x_{ulim}$ then we have to redesign the section.

(iii) If for already constructed existing beam is found to be over-reinforced then to estimate MOR, Maximum depth of neutral axis shall be used. Use MOR of compression zone.

Step(4):- Estimate MOR

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

Now:-

Jab b hum x_u nikalna chahte hain to hum Total CF ko Total TF k equal put kar dete hain.

Fe 250 → mild steel

L14:-

Q23) $b = 300\,mm$

$d = 500\,mm$

$A_{st} = A_{st1} + A_{st2} = 2200\,mm^2$

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

$d' = 50\,mm$

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

$f_y = 250\,N/mm^2$

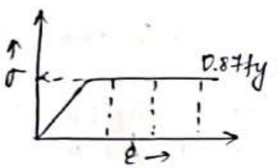
Ist part

$C = T$

$0.36 f_{ck} b x_u + A_{sc}(f_{sc} - 0.446 f_{ck}) = 0.87 f_y (A_{st1} + A_{st2})$

$0.36 \times 20 \times 300 \times x_u + 628(\quad - 0.446 \times 20) = 0.87 \times 250 \times 2200$

$\varepsilon_{sc} = 0.0035\left[1 - \frac{d'}{x_u}\right]$



$0.36 \times 20 \times 300 \times x_u + 628(0.87 f_y \times 250 - 0.446 \times 20) = 0.87 \times 250 \times 2200$

$x_u = \underline{160.88\,mm}$

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

$(MOR)_c = 0.36 \times 20 \times 300 \times$

$x_{u,lim} = k \cdot d$

$= 0.53 \times 500$

$= 265\,mm$

$\because x_u < x_{u,lim}$

The given Section is under reinf Section

→ use actual depth of neutral axis (x_u)

$(MOR)_c = \left[0.36 \times 20 \times 300 \times 160.91\,[500 - 0.42 \times 160.91] + 628(0.87 \times 250 - 0.446 \times 20)[500 - 50]\right] \times 10^{-6}$

To convert N-mm to kN-m

$(MOR)_c = 209.20\,kN\text{-}m$

R/f → Reinforcement

L15:- Minimum & Maximum Area of Steel Reinforcement in Beams

(1) Minimum area of Tension R/f

$$\boxed{\frac{A_{st\,min}}{bd} \geq \frac{0.85}{f_y}}$$

Minimum area of steel reinforcement is provided (in Tension zone) to have minimum ductility in the member.

It is also provided to avoid the problem of sudden collapse/failure.

(2) Maximum area of Tension R/f

$A_{st\,max} = 4\%$ of total cross-sectional area

$$\boxed{A_{st\,max} = 0.04 \times B \times D}$$

B → Gross width
D → Gross depth

The code [IS 456:2000] has given the maximum limit of area of steel R/f to avoid problem of compaction of concrete due to congestion of reinforcement.

(3) Maximum area of compression R/f

$A_{sc\,max} = 4\%$ of Total cross-sectional area

$$\boxed{A_{sc\,max} = 0.04 \times B \times D}$$

* Minimum area of comp R/f is not given kyki zarurat tabhi padti hai jab aap doubly R/f beam design karte hain. otherwise no need

Casting of beam → [illegible] concrete dalna
Intermediate beams [illegible] T-beams banege
[illegible] beams banege [illegible] L-beams

L16:- b=250mm
d = 400mm
Fe415

<u>Minimum area of steel R/f</u>

$$\frac{Ast_{min}}{bd} \geq \frac{0.85}{f_y}$$

$$Ast_{min} \geq \frac{0.85}{415} \times 250 \times 400$$

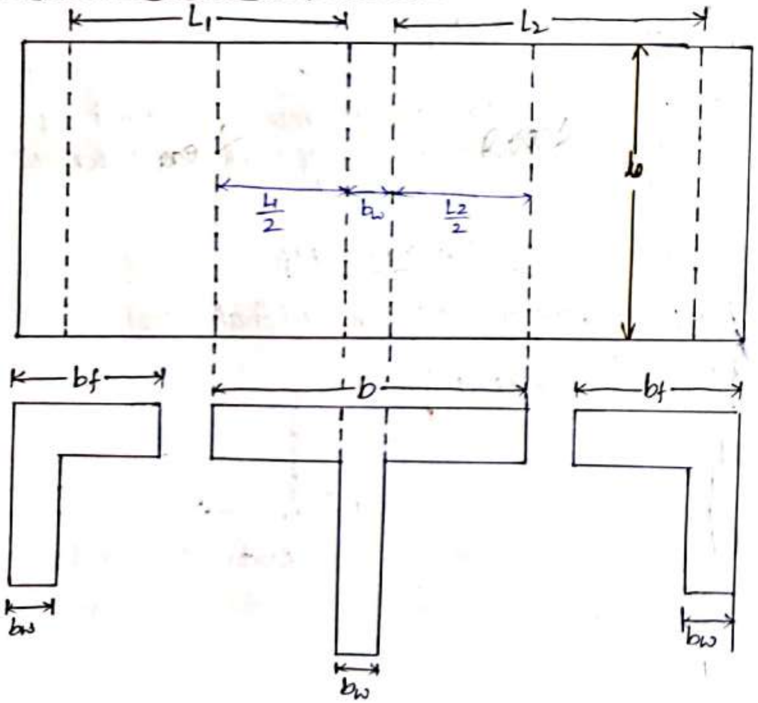
$$Ast_{min} = 204.8 \simeq 205\ mm^2$$

<u>Max area of steel R/f</u>

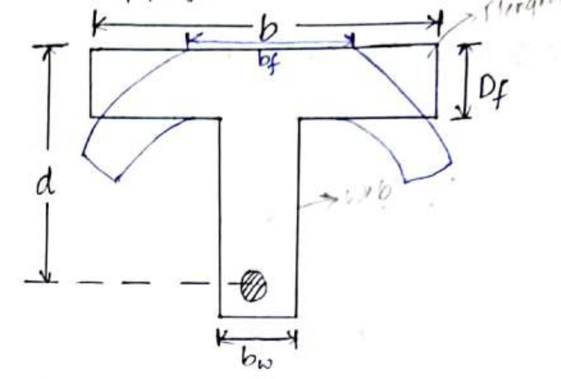
$$Ast_{max} = \frac{4}{100} \times 250 \times 400$$

$$= 4000\ mm^2$$

L17:- <u>Introduction & Effective width of Flanged Beam.</u>
<u>Flanged Beams [T-beams & L-beams].</u>



b_f → effective width of flanged (always greater than b_w & less than b)
d → effective depth
b → actual width of flange
D_f → Depth of flanged
b_w → width of web.
l_o → effective length of the beam span



Case(I) <u>Isolated T-Beams</u>

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

Case(II) <u>Isolated L-Beams</u>

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

Case(III) <u>Continuous T-Beams</u>

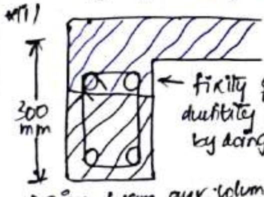
$$\boxed{b_f = \frac{l_o}{6} + b_w + 6D_f}$$

Case(IV) <u>Continuous L-Beams</u>

$$\boxed{b_f = \frac{l_o}{12} + b_w + 3D_f}$$

R/f exchange kar k.

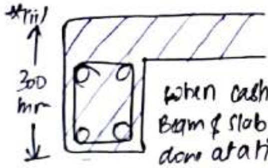
*(i)



← fixity & ductility increases by doing this.

→ aisa beam aur column K beech me b karte hain.

*(ii)



When casting of Beam & slab are done at a time we get flanged action. moment carrying capacity of Beam increases. In monolithic construction.

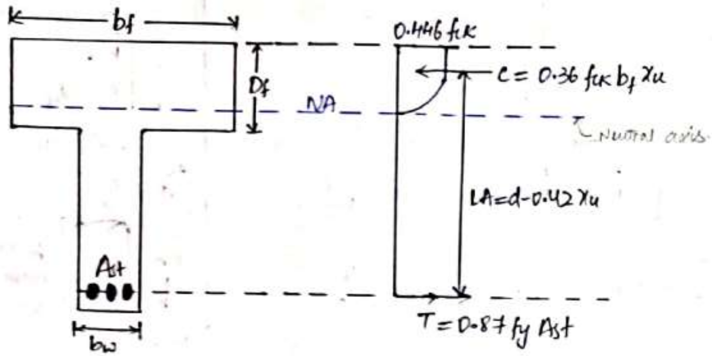
Here slab & Beam becomes a single portion.

Note :-
$(b_f < b)$
If $b_f > b$ then take b only

Effective width of Flange (bf) wo width of flange hai jahan tak apna section load leta hai ye effective rehta hai [so always consider bf]

L18:- Analysis of Flanged Beams [same procedure for L & T beam]

Case(I) When the Neutral Axis is in the flange Portion ($x_u < D_f$)



Step(1): Calculate x_{ulim}

$x_{ulim} = 0.53\,d$ (Fe250)

$= 0.48\,d$ (Fe415)

$= 0.46\,d$ (Fe500)

Step2: Calculate x_u

$C = T$

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

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

Step3: Compare x_u & x_{ulim}

$x_u = x_{ulim}$ (Balanced section) → x_{ulim}

$x_u < x_{ulim}$ (under R/f section) → x_u

$x_u > x_{ulim}$ (over R/f section) → x_{ulim}

Step4: Calculate Moment of Resistance

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

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

for balanced section put $x_u = x_{ulim}$ in these two equations

L19:- Case(II). When the Neutral Axis is in the web Portion ($x_u > D_f$)

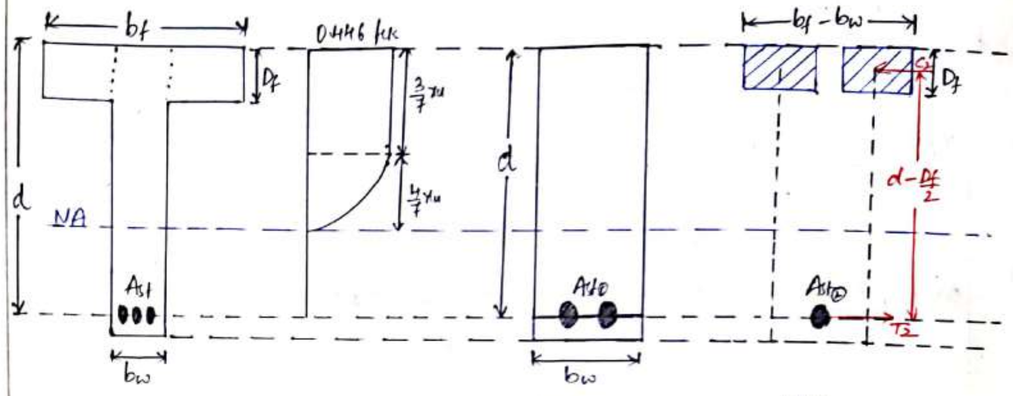


Fig (1) Fig(2)

Case 2.a If Flange is uniformly stressed ($3/7\, x_u \geq D_f$)

Fig(1)

$C_1 = 0.36 f_{ck} b_w x_u$

$T_1 = 0.87 f_y A_{st1}$

$LA_1 = d - 0.42 x_u$

$(MOR_1)_c = C_1 LA_1$

$(MOR_1)_T = T_1 LA_1$

Fig (2)

$C_2 = 0.446 f_{ck} (b_f - b_w) \cdot D_f$

$T_2 = 0.87 f_y A_{st2}$

$LA_2 = d - \frac{D_f}{2}$

$(MOR_2)_c = C_2 LA_2$

$(MOR_2)_T = T_2 LA_2$

$$\boxed{(MOR)_{total} = MOR_1 + MOR_2}$$

$$\boxed{(MOR)_c = 0.36 f_{ck} b_w x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) D_f \left(d - \frac{D_f}{2}\right)} \quad \text{—(1)}$$

$$\boxed{(MOR)_T = 0.87 f_y A_{st1} (d - 0.42 x_u) + 0.87 f_y A_{st2} \left(d - \frac{D_f}{2}\right)} \quad \text{—(2)}$$

Scanned with CamScanner

Q) Calculate Moment of Resistance ? in flange portion

Step① Calculate x_{ulim}

$x_{ulim} = 0.53d$ (Fe 250)
$= 0.48d$ (Fe 415)
$= 0.46d$ (Fe 500)

Step② Calculate x_u

$C = T$

$C_1 + C_2 = T_1 + T_2$

$0.36 f_{ck} b x_u + 0.446 f_{ck} (b_f - b_w) D_f = 0.87 f_y A_{st①} + 0.87 f_y A_{st②}$
$= 0.87 f_y (A_{st①} + A_{st②})$
$= 0.87 f_y A_{st}$

Step③. Compare x_u & x_{ulim}

If $x_u = x_{ulim}$ (Balance section) → use x_{ulim}

If $x_u < x_{ulim}$ (URS) → x_u

If $x_u > x_{ulim}$ (ORS) → x_{ulim}

Step④ Calculate MOR

use eqⁿ ① or eqⁿ ②

[We get the same ans by eqⁿ ① & or eqⁿ ② so use any one
Its better to use $(MOR)_c$ b/z many times A_{st_1} & A_{st_2} doesnot mention separately it will give A_{st} only]

y_f → equivalent depth of flange

L20:- Case b) 2.b) Flange is not uniformly stressed ($\frac{3}{7} x_u < D_f$)

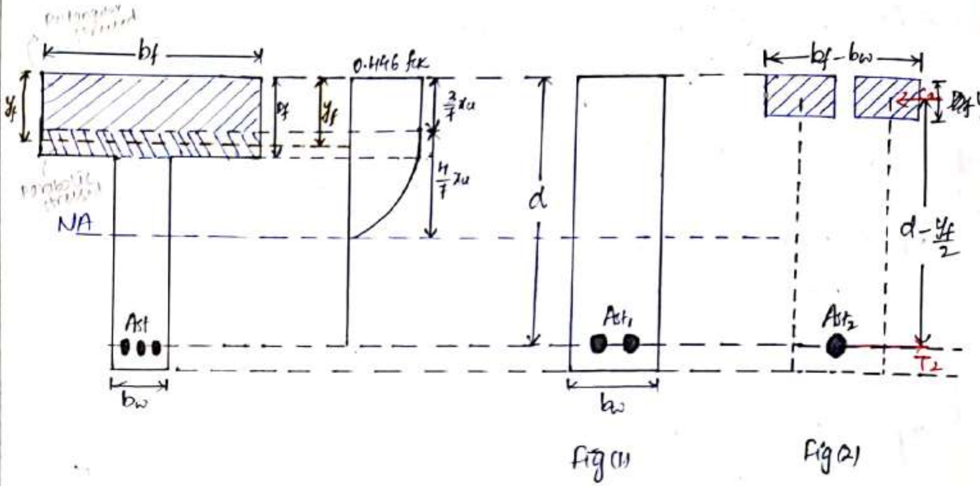


Fig (1)   Fig (2)

Fig (1)

$C_1 = 0.36 f_{ck} b_w x_u$

$T_1 = 0.87 f_y A_{st_1}$

$(LA)_1 = d - 0.42 x_u$

$(MOR_1)_c = C_1 LA_1$

$(MOR_1)_T = T_1 LA_1$

Fig (2)

$C_2 = 0.446 f_{ck} (b_f - b_w) y_f$

$T_2 = 0.87 f_y A_{st_2}$

$(LA)_2 = d - \frac{y_f}{2}$

$(MOR_2)_c = C_2 LA_2$

$(MOR_2)_T = T_2 LA_2$

$$\boxed{(MOR)_{Total} = MOR_1 + MOR_2}$$

$$\boxed{(MOR)_c = 0.36 f_{ck} b_w x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) y_f \left(d - \frac{y_f}{2}\right)} \quad ①$$

$$\boxed{(MOR)_T = 0.87 f_y A_{st①} (d - 0.42 x_u) + 0.87 f_y A_{st②} \left(d - \frac{y_f}{2}\right)} \quad ②$$

Scanned with CamScanner

Q4 Calculate MOR? → (Based on iterations).

Step 1: Calculate y_f

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

y_f → equivalent depth of Neutral axis

Note :-
→ $y_f < D_f$ (Generally)
→ If in any case $y_f > D_f$ from this formula in that case Take $y_f = D_f$

Step 2: Calculate x_{ulim}

$x_{ulim} = 0.53d$ (Fe250)
$= 0.48d$ (Fe415)
$= 0.46d$ (Fe500)

Step 3: Calculate x_u

$C = T$

$C_1 + C_2 = T_1 + T_2$

$0.36 f_{ck} b_w x_u + 0.446 f_{ck} (b_f - b_w) y_f = 0.87 f_y (A_{st_1} + A_{st_2})$
$= 0.87 f_y A_{st}$

Step 4: Compare x_u & x_{ulim}

If $x_u = x_{ulim}$ (Balanced Section) → use x_{ulim}
If $x_u < x_{ulim}$ (URS) → x_u
If $x_u > x_{ulim}$ (ORS) → x_{ulim}

Step 5: Calculate MOR

using eq① & eq② MOR can be estimated
(eq② is recommended).

L21: Q2.4

$A_{st} = 4000\,mm^2$
M25 & Fe415
$x_u > 100\,mm$ (means $x_u > D_f$)

If $x_u > 100\,mm$ is not given in Que
Assuming N.A is in the flange portion
Putting $C = T$
$0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$
$0.36 \times 25 \times 1000 \times x_u = 0.87 \times 415 \times 4000$
$x_u = 160.5\,mm$
$\because x_u = 160.5\,mm > D_f = 100\,mm$
$\therefore$ N.A is in the web portion.
our Assumption is wrong

Analysing the ans is correct or not
When flange is uniform
$\frac{3}{7} x_u \geq D_f$
$\frac{3}{7} \times 236.43 \geq D_f\,100$
$101.33\,mm \geq 100\,mm$
Hence the flange is uniformly covered Therefore my assumption is correct.

1st part: Now it is in case II but here also there are 2 cases 2.a & 2.b
Assuming flange is uniformly stress.
$C = T$
$0.36 f_{ck} b_w x_u + 0.446 f_{ck} (b_f - b_w) D_f = 0.87 f_y A_{st}$
$0.36 \times 25 \times 325 \times x_u + 0.446 \times 25 [1000 - 325] \times 100 = 0.87 \times 415 \times 4000$
$x_u = 236.43\,mm$ ∴ ans matches
If not assume case 2.b.

2nd part

$(MOR)_c = 0.36 f_{ck} b_w x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) D_f \left(d - \frac{D_f}{2}\right)$

[As a good engg first calculate the given section is BS, URS, or ORS]

$x_{ulim} = k \cdot d$
$= 0.48d$
$= 0.48 \times 570$
$= 273.6\,mm$
$\because x_u < x_{ulim}$ ($\because$ URS)
→ use x_u

$(MOR)_c = \left[0.36 \times 25 \times 325 \times 236.43 \times [570 - 0.42 \times 236.43) + 0.446 \times 25 \times (1000 - 325) \times 100 \left(570 - \frac{100}{2}\right)\right] \times 10^{-6}$ ← To convert N-mm to KNm

MOR = 717 KN-m

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M30 mix design — 30 N/mm² comp strength

The cubes should be 15×15×15 cm — These cubes should be cure for 28 days. Then only in comp testing machine (we get comp strength from this we fck)

Characteristic compressive strength of concrete

f_{ck} → characteristic compressive strength is the compressive strength for which not more than 5% results are expected to fall when they are normally distributed.

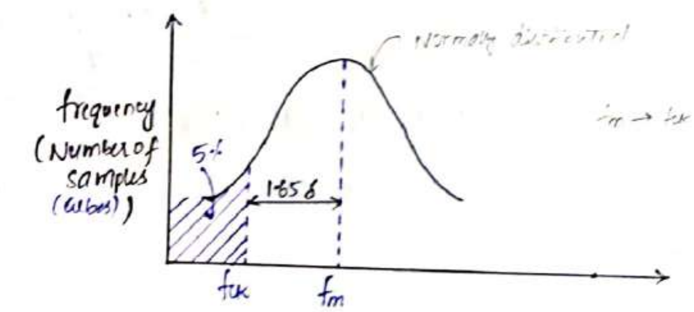
For ex:- 100 cubes are prepared for M30 grade.
If more than 95 cubes (>95) gives comp strength ≥ 30 N/mm²
& less than 5 cubes (<5) give comp strength < 30 N/mm²

$f_{ck} = 30 N/mm^2$

[If out of 100 cubes >80 gives comp strength ≥ 30 N/mm²
& <20 gives comp strength < 30 N/mm²
Then $f_{ck} \neq 30 N/mm^2$]

In short
If it gives more than 95% result than we can't call it f_{ck}.
[eg M30 → $f_{ck} = 30 N/mm^2$]
[eg M20 → $f_{ck} = 20 N/mm^2$].

*



Target mean strength (f_m) or avg strength :- It is defined as the characteristic compressive strength for which not more than 50% results are expected to fall.

$$\boxed{f_m = f_{ck} + 1.65\delta}$$

Note → In concrete mix design, concrete is designed for target mean strength.

Note:- Concrete f_m ke liye design karna hai aur f_{ck} ko achieve karna hai.
(f_m → Target mean strength) (f_{ck} → chart comp strength)

δ → Standard deviation. As per IS 456: 2000 it depends upon the grade of the concrete.
(in N/mm²)

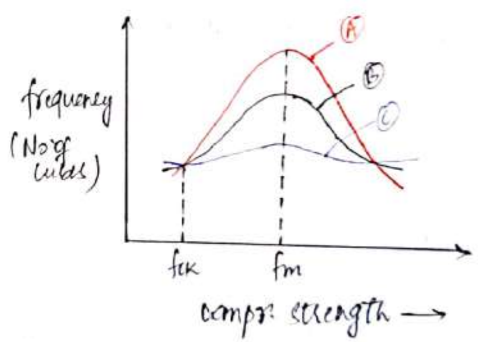
grade of concrete ↑ → value of δ ↑

1.65 → probability factor

Grades	δ (N/mm²)
M-10 M-15	3.5
M-20 M-25	4.0
M-30 upto M-55	5.0

Note:- For poor quality control, values of standard deviation shall be increased by 1 N/mm² [or 1 MPa]

Analysis



(M30) Bangalore
(M30) Ahmedabad
(M30) Raipur

Quality control in increasing order

C < B < A

[Rex in site A less than 5% are expected to fail & other thing there are more cubes in f_m ∴ In B max than 5% are expected to fail & are not more cubes in f_m area. & so on for C]

Non overlapping means 2 cubes agar same comp strength de rahe hain to unme se ek select karna hai

L23:- **Quality control (Acceptance criteria of concrete Mix Design)** otherwise reject

1st criteria :- The average compressive strength of four non-overlapping consecutive samples shall be greater than $f_{ck}+0.825\delta$ or $f_{ck}+4$ which ever is maximum (For $M \geq 20\ N/mm^2$).
and shall be greater than $f_{ck}+0.825\delta$ or $f_{ck}+3$ which ever is maximum (for $M < 20\ N/mm^2$)

$$f_{avg} \geq \left.\begin{matrix} f_{ck}+0.825\delta \\ \text{or} \\ f_{ck}+4 \end{matrix}\right\} \text{whichever is maximum} \quad (\text{for } M \geq 20\ N/mm^2)$$

otherwise rejected

$$f_{avg} \geq \left.\begin{matrix} f_{ck}+0.825\delta \\ \text{or} \\ f_{ck}+3 \end{matrix}\right\} \text{whichever is maximum} \quad (\text{for } M < 20\ N/mm^2)$$

mostly M15 grade

2nd criteria :- The individual (cube test) test results shall not be less than $f_{ck}-4$ ($M \geq 20\ N/mm^2$) or $f_{ck}-3$ ($M < 20\ N/mm^2$)

comp strength

$f_{c\,individual} \nless f_{ck}-4 \quad (M \geq 20\ N/mm^2)$

$f_{c\,individual} \nless f_{ck}-3 \quad (M < 20\ N/mm^2)$

As per New rule $\boxed{f_{c\,ind} \nless f_{ck}-3}$ for all grades of concrete

3rd criteria :- (a) Minimum three cubes shall be tested [from diff region of concrete part]
(b) The individual variation in compressive strength shall not exceed $\pm 15\%$ from the average compressive strength. otherwise reject

i.e. concrete is heterogeneous material.

$$\boxed{\%\ \text{variation in individual comp. strength} = \frac{f_{c\,individual} - f_{avg}}{f_{avg}} \times 100}$$

$\ngtr \pm 15\%$

Q) **Check for Acceptance criteria**

31.1, 32.2, 33.3 N/mm^2 are the compressive strength of the samples prepared for M30 grade of concrete.

$$f_{avg} = \frac{31.1+32.2+33.3}{3} = 32.2\ N/mm^2$$

1st criteria

$$f_{avg} \geq \left.\begin{matrix} f_{ck}+0.825\delta \\ \text{or} \\ f_{ck}+4 \end{matrix}\right\} \max$$

$$f_{avg} \geq \left.\begin{matrix} 30+0.825\times 5 \Rightarrow 34.125 \\ \text{or} \\ 30+4 \Rightarrow 34 \end{matrix}\right\} \max$$

$f_{avg} \geq 34.125$

$f_{avg} = 32.2 \ngtr 34.125$

Fail in 1st criteria.

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2nd criteria

$f_{cind} \geq f_{ck} - 4$

$\geq 30 - 4$

$\geq 26 \cdot N/mm^2$

Sample ①, $31.1 > 26$ ✓

Sample ②, $32.2 > 26$ ✓

Sample ③, $33.3 > 26$ ✓

Pass in 2nd criteria.

3rd criteria

% Individual variation $= \dfrac{f_{cind} - f_{avg}}{f_{avg}} \times 100 \ \not> \pm 15\%$

For 1st sample $= \dfrac{31.1 - 32.2}{32.2} \times 100 \Rightarrow -3.416\%$

For 2nd sample $= \dfrac{32.2 - 32.2}{32.2} \times 100 \Rightarrow 0\%$

For 3rd sample $= \dfrac{33.3 - 32.2}{32.2} \times 100 \Rightarrow 3.416\%$

Pass in 3rd criteria

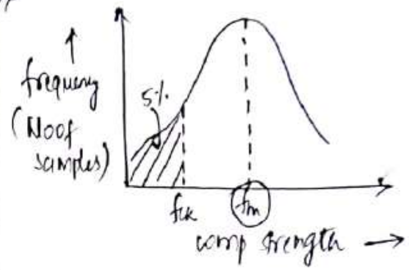
Conclusion
1st Fail
2nd Pass
3rd Pass
} concrete shall not be accepted [if fail in any one criteria].

L 21:

Q15) Minimum ③ $x = 3$

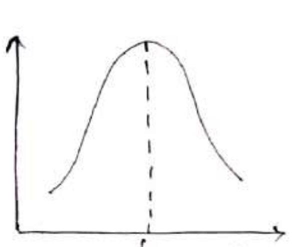
Individual variation $\not> \pm 15\%$ $y = 15$

Q16)



Ans fm

Q17)



fm → characteristic ~~comp~~ strength (25 MPa)

M25 characteristic compressive strength $= \boxed{f_m = 25 mpa}$

Q18) $f_{ck} = 25 MPa$

$\sigma = 4 mpa$

$f_m = f_{ck} + 1.65\sigma$

$f_m = 25 + 1.65 \times 4$

$f_m = 31.6 \ N/mm^2$

W/C Ratio	45	50	55	60
favg	35	25	20	15

x ↑ ... ↓ 31.6

$\dfrac{35-25}{45-50} = \dfrac{35-31.6}{45-x}$

$-2 = \dfrac{3.4}{45-x}$

$45 - x = -\dfrac{3.4}{2} = -1.7$

$45 - x = -1.7$

$x = 45 + 1.7$

$x = 46.7\%$

✓ 46 (Lower Integer) ~~47~~ (Higher Integer).

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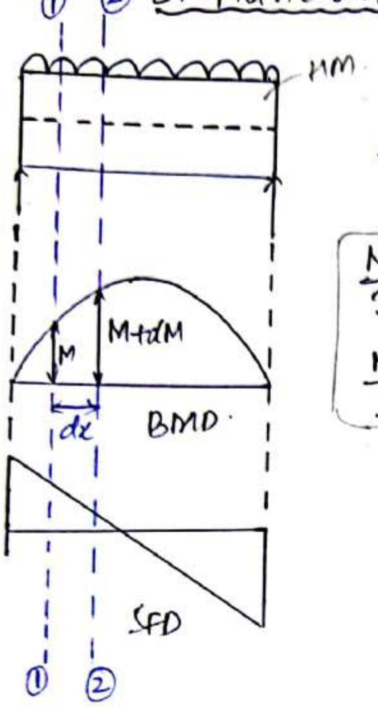
CHAPTER : 03

Shear, Torsion, Bond strength & Development Length

L1:- Limit state of collapse : in shear.

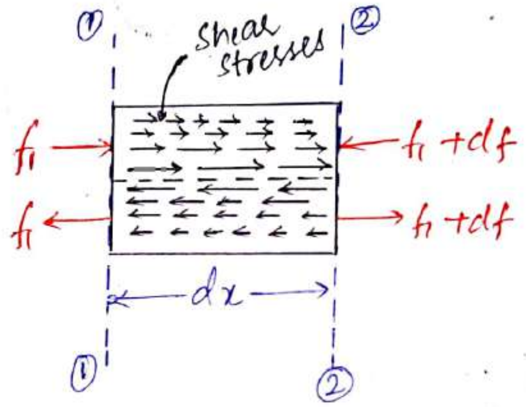
* In plane concrete.



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

$$\frac{M}{I}\cdot y = \sigma_c \qquad \frac{M}{I}\cdot y = \sigma_T \qquad ①$$

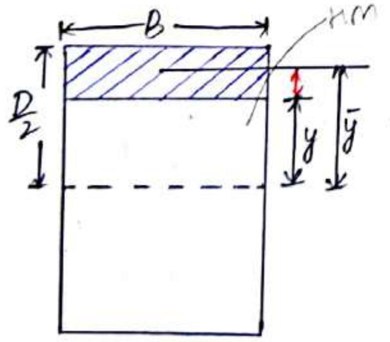
$$\frac{M+dM}{I}\cdot y = \sigma_c + dc \qquad \frac{M+dM}{I}\cdot y = \sigma_T + dt \qquad ②$$



[Neutral fibre k bas stresses zyada hote hain]

* WKT

$$\boxed{\tau = \frac{VA\bar{Y}}{IB}}$$



$$A = \left[\frac{D}{2} - y\right] \times B$$

$$\bar{Y} = \left[\frac{D}{2} - y\right] \times \frac{1}{2} + y$$

$$\bar{y} = \frac{D}{4} - \frac{y}{2} + y$$

$$\bar{y} = \frac{D}{4} + \frac{y}{2} \Rightarrow \frac{1}{2}\left[\frac{D}{2} + y\right]$$

$$\tau = \frac{V}{IB}\left[\left(\frac{D}{2} - y\right)B\left(\frac{D}{2} + y\right)\times\frac{1}{2}\right]$$

$$\tau = \frac{V\cancel{B}}{2I\cancel{B}}\left[\frac{D^2}{4} - y^2\right]$$

$$\boxed{\tau = \frac{V}{2I}\left[\frac{D^2}{4} - y^2\right]}$$

at $y = 0$; at N.A ; $\tau_{max} = \frac{VD^2}{8I}$.

at $y = \frac{D}{2}$; at Top/bottom ; $\tau_{min} = 0$.

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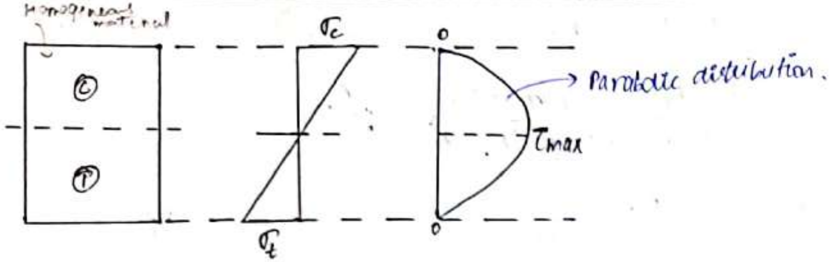
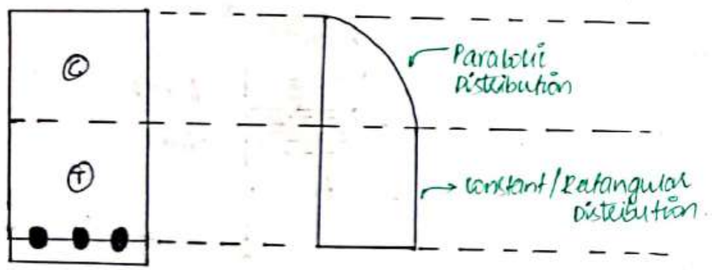
Shear stress Top/bottom p zero hy kez upke upr koi aur layer nhi hai jo slip ho aur shear develop kare.



Shear stress distribution in homogeneous material or in plain concrete parabolic hota hai

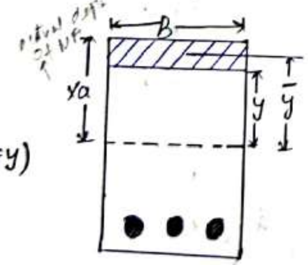
Plain cement concrete me (homogeneous hai) iso parabolic hota hai

L2:- Shear stress distribution in RCC sections :- [only Diag is imp don't practice derivations]



Case I:- Above the neutral axis

WKT $\boxed{\tau = \frac{VA\bar{y}}{I_{eq}B}}$



$A = (x_a - y) \times B$

$\bar{y} = (x_a - y) \times \frac{1}{2} + y \Rightarrow \frac{x_a}{2} - \frac{y}{2} + y \Rightarrow \frac{1}{2}(x_a + y)$

$\tau = \frac{V}{I_{eq}B}\left[(x_a - y)B \times \frac{1}{2}(x_a + y)\right]$

$\tau = \frac{V}{2I_{eq}}\left[x_a^2 - y^2\right]$

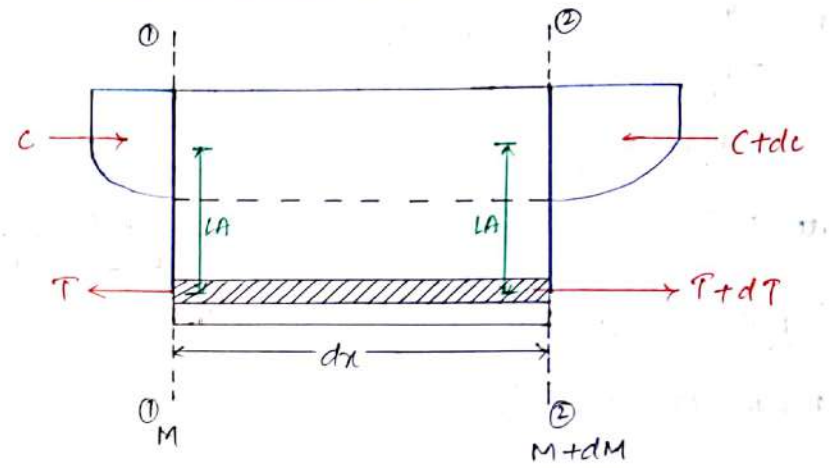
at $y = 0$; $\tau_{max} = \frac{V}{2I_{eq}} \cdot x_a^2$

at $y = x_a$; $\tau_{min} = 0$

For Plain cement concrete / homogeneous material Shear stress dist is upto below is parabolic

For composite material like RCC Shear stress dist → First parabolic, than constant till C.G of steel upto NA

Case II) Below the Neutral axis



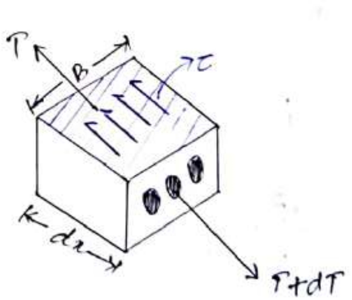
$M = T \cdot LA$ —①

$M + dM = (T + dT) \cdot LA$ —②

② - ①.

$dM = dT(LA)$

$dT = \frac{dM}{LA}$ —③



Force due to shear stress (τ)

$= \tau \cdot B \cdot dx$ —④

③ = ④

$dT = \tau B dx$

$\frac{dM}{LA} = \tau B dx$

$\frac{dM}{dx} = \tau B \cdot LA$

$\boxed{\tau = \frac{V}{B \cdot LA}}$ (Below the N.A)

Iequivalent

C.G of concrete we have take only above the N.A bcz concrete is neglected below the N.A

- $I_{eq} = I_{concrete} + I_{steel}$

$I_{concrete} = I_G + Ah^2$

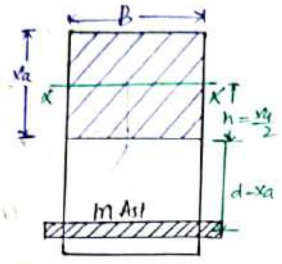
$= \frac{Bx_a^3}{12} + (Bx_a)\cdot\frac{x_a^2}{4}$

$= \frac{Bx_a^3}{12} + \frac{Bx_a^3}{4}$

$\boxed{I_{concrete} = \frac{Bx_a^3}{3}}$

$I_{steel} = \cancel{I_G} + Ah^2$ (bcz value will be very small)

$\boxed{I_{steel} = mA_{st}\cdot(d-x_a)^2}$

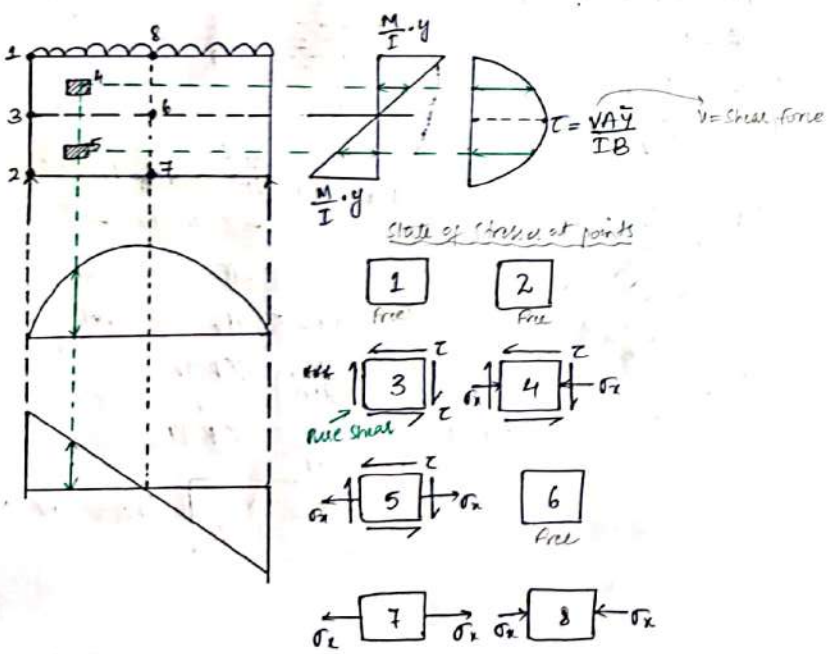


$I_{eq} = \frac{Bx_a^3}{3} + mA_{st}\cdot(d-x_a)^2$

where m = modular ratio $= \frac{E_s}{E_c}$

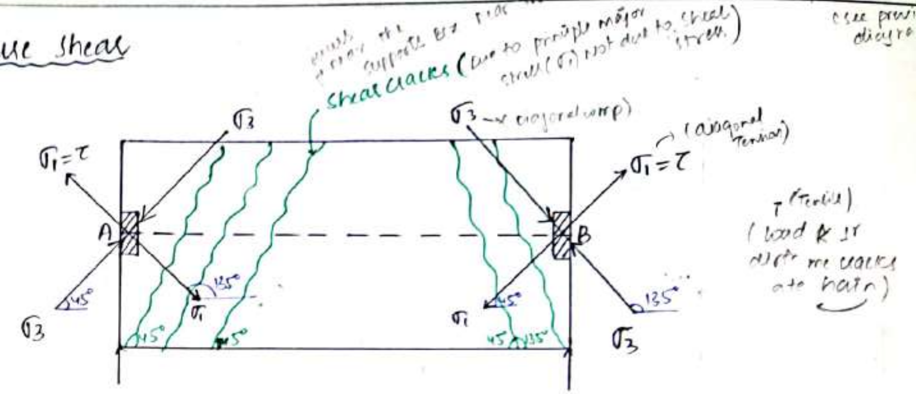
L30-

- Types of Cracks :-



Pure shear → Jitne sif shear stresses lagte hain.
Ductile mate (or steel) are weak in shear.

Pure Shear



Negative shear stress $(-\tau)$ — Positive shear stress $(+\tau)$

$$\sigma_{1,3} = \frac{\sigma_x+\sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x-\sigma_y}{2}\right)^2+\tau^2}$$

Element (A)

major principle stress $\sigma_1 = 0 + \sqrt{0+(-\tau)^2} = +\tau$ (Tensile) → 135°

minor principle stress $\sigma_3 = 0 - \sqrt{0+(-\tau)^2} = -\tau$ (compressive) → 45°

$\tan 2\theta = \frac{2\tau}{\sigma_x-\sigma_y} \Rightarrow \frac{-(2\tau)}{0-0} \Rightarrow -\infty \quad \therefore \theta = 135°, 45°$

Element (B)

$\sigma_1 = +\tau$ (Tensile) → 45°

$\sigma_3 = -\tau$ (compressive) → 135°

$\tan 2\theta = \frac{2\tau}{\sigma_x-\sigma_y} = \frac{+2\tau}{0-0} = +\infty \quad \therefore \theta = 45°, 135°$

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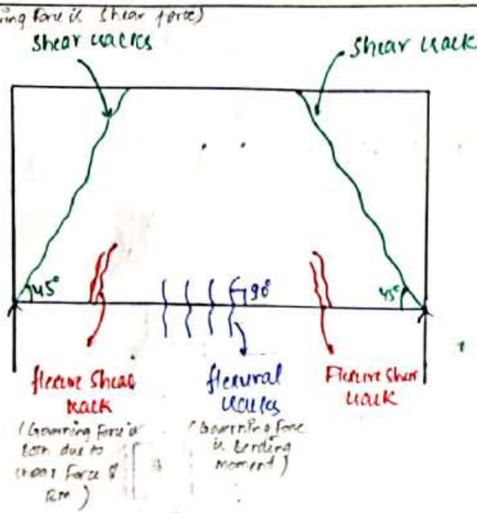
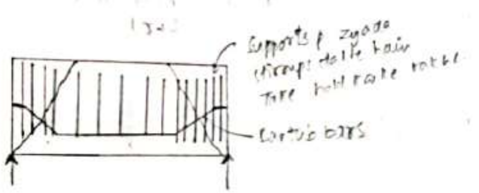
3 Types of cracks





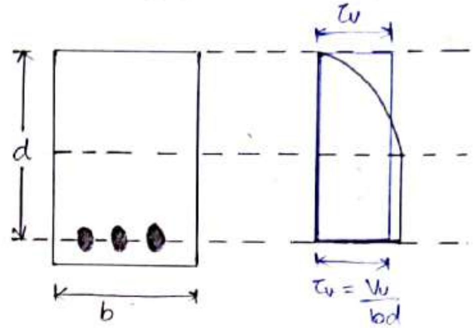
NOTE :- Stirrups hum shear stress ure ke liye nahi provide karte hain balke axial force use k liye karte hai (σ_1 & σ_3) jiski value shear stress (τ) k barabar hai.

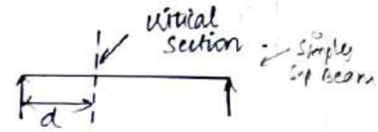
Principal stresses

for gate start from this lecture :-

14:- ~~Nominal~~ Nominal Shear stress, shear strength & maximum shear strength of concrete. (τ_v, τ_c & $\tau_{c\,max}$)

1) Nominal Shear stress (τ_v) :-

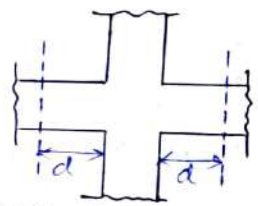




For Prismatic beam $\boxed{\tau_v = \frac{V_u}{bd}}$

Throughout the length of beam c/s is same

$V_u \rightarrow$ factored shear force



2) Shear Strength of concrete (τ_c) :-

Shear Strength of concrete (RCC) depends upon the grade of concrete & % of steel reinforcement.

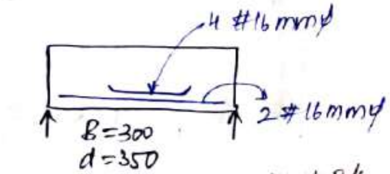
If, $\%A_{st} \uparrow \rightarrow \tau_c \uparrow$

$f_{ck} \uparrow \rightarrow \tau_c \uparrow$

τ_c akele concrete ki taqat nahi hai usme jo % of steel hai (% Ast) uski bhi taqat hai

$$\%A_{st} = \frac{(A_{st})_{support}}{bd} \times 100$$

Use Table 19 [IS 456:2000].



$$\%A_{st} = \frac{2 \times \pi/4 (16)^2}{300 \times 350} \times 100$$

support R/f

(max batkane wali Taqat hai) value b nahi ye payegi).
(concrete, R/f aur stirrups ki Taqat Jo k max hai use aage increase na ki)

3) Maximum Shear Strength of concrete ($\tau_c max$):-

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

If $f_{ck} \uparrow \rightarrow \tau_{cmax} \uparrow$

No need to remember Table (you can get it by below formula)

f_{ck}	M15	M20	M25	M30	M35	M40 & above
τ_{cmax} (N/mm^2)	2.5	2.8	3.1	3.5	3.7	4.0

$$\boxed{\tau_{cmax} = 0.62\sqrt{f_{ck}}}$$

L5:- Analysis & Design of shear Reinforcement:- (stirrups)

Case(I) $\tau_v < \tau_c$:- (mene Todne wali Taqat kam hai bachane wali zyada) (fir b min stirrups to provide karna hi hoga)

Minimum Shear reinforcement Shall be provided.

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

ye formula sabhi cases k liye hai.

[mainly my yaha se spacing nikalna chahta hoon]

→ Isme ko dalna hi hai chahe kuch b case ho (by code)

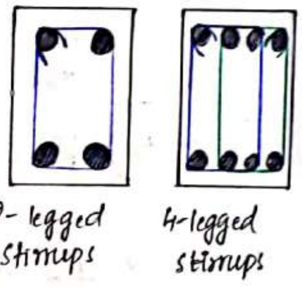
where $f_y \not> 415\ N/mm^2$ → ye stirrups ki grade hai (R/f ki nahi)

S_v → centre to centre Spacing of Stirrups (in mm)

A_{sv} → Area of Shear R/f

$$\boxed{A_{sv} = n \cdot \frac{\pi}{4}(\phi^2)}$$ n → No of legs

stirrups ko dB long R/f k bahar rakhi hona chahiye.



2-legged stirrups

4-legged stirrups

Two legged stirrups

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

Four legged Stirrups

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

Case(II) If $\tau_v > \tau_c$ [But $\tau_v < \tau_{cmax}$]

then Design Shear Reinforcement.

V_u → applied Shear force

$\boxed{V_u = \tau_v bd}$ → Todne wali Taqat

V_c → Shear force taken by concrete

$\boxed{V_c = \tau_c bd}$ → Bachane wali Taqat

V_{us} → Design shear force taken by stirrups

$V_{us} = V_u - V_c$

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

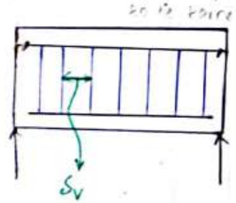
let $V_u = 100$ kN
& $V_c = 70$ kN (shear force concrete + Tension R/f lenge)
$V_{us} = 100 - 70 = 30$ kN
Additional force (30 k stirrups lenge Design karne ko)
→ Thread se design karte hain.

(a) For Vertical Stirrups

[In Singly R/f, Sirf Bottom me R/f dalte hain par stirrup k liye hum minimum 2 bar upar b dalenge. Top me]

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

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



Agar Doubly R/f beam hai to rahi chalne padega

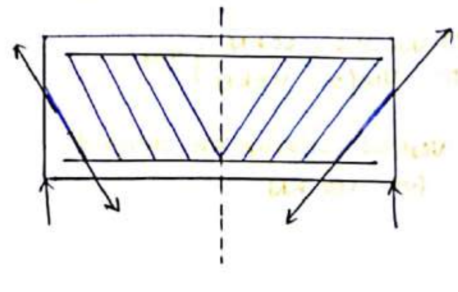
$f_y \not> 415\ N/mm^2$ → ye stirrups ki grade hai

$A_{sv} = n \cdot \frac{\pi}{4}(\phi^2)$ n → No of legs (vertical stirrups ka) → Beam k longitudinal R/f se bada hoi hona chahiye

S_v → centre to centre spacing of vertical stirrups (in mm)

Note:- Fe 500 grade of steel shall not be used in Shear R/f.

(b) Inclined Stirrups



dost-door spacing → kam shear R/f (stirrups)
pas-pas spacing → zyada shear R/f (stirrups)

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

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

$\alpha \to$ angle of stirrups
$\alpha \not< 45°$
$f_y \not> 415$ N/mm² or Mpa

(c) Bent up Bars (with vertical stirrups)

additional force or load

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

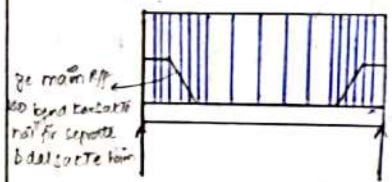
- Some portion (of V_{us}) is taken by bent up bars (V_{sb})
- Some portion (of V_{us}) is taken by vertical stirrups ($V_{us} - V_{sb}$)

$\boxed{V_{sb} = 0.87 f_y A_{sb} (\sin\alpha)}$ → shear force taken by bent up bars.

Bent up bars hamesha vertical stirrups k sath provide kiya jata hai bcz
Bent up bars alone cannot resist V_{us}.

Vertical stirrups are designed to carry the shear force equal to

max of $\begin{cases} \text{(i) } V_{us} - V_{sb} \\ \text{(ii) } \frac{V_{us}}{2} \end{cases}$ (means whichever's max take that one)



Supports me zyada stirrups dalte hain bcz support p pure shear develop hota hai (jise vertical hote hain (diagonal tension k karan)

ex:- $V_u = 400$ kN, $V_c = 100$ kN, $V_{us} = V_u - V_c = 300$ kN

Case (I)
$V_{sb} = 100$ kN

(i) $V_{us} - V_{sb} = 200$ kN
(ii) $V_{us}/2 = 150$ kN } max

vertical stirrups are designed for 200 kN

Case (II)
$V_{sb} = 250$ kN

(i) $V_{us} - V_{sb} = 50$ kN
(ii) $V_{us}/2 = 150$ kN } max

Vertical stirrups are designed for 150 kN.

yahan par τ_v concrete ko crush karne wali taqat ban sakti hai

Case (III) $\tau_v > \tau_{c\,max}$

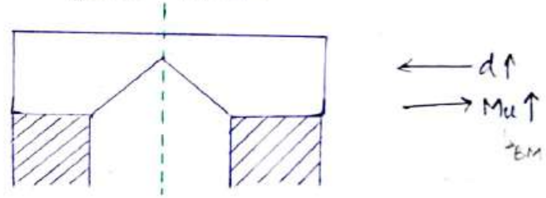
- This Test/check is required for diagonal compression failure.
- if $\tau_v > \tau_{c\,max}$ then "Redesign the section".
  (that means % R/f badhao, ya dimension badhao, grade of concrete badhao fir wapas se shuru se calculation start karo)

Q6:- Nominal shear stress in varying depth sections :-

$$\boxed{\tau_v = \frac{V_u \pm \frac{M_u}{d}\tan\beta}{bd}}$$ (IS 456:2000)

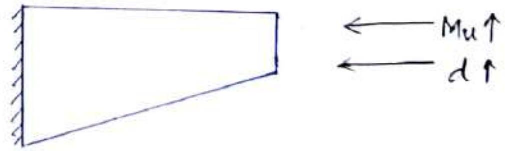
+ve sign is taken, when depth & BM both are increasing in two opposite directions.

eg.



-ve sign is taken when depth & BM both are increasing in the same direction.

eg



Rest is same procedure

only τ_v is diff rest is same procedure.

Case (I) $\tau_v < \tau_c$ → Minimum shear R/f

Case (II) $\tau_v > \tau_c$ (But $\tau_v < \tau_{c\,max}$) → Design shear R/f

Case (III) $\tau_v > \tau_{c\,max}$ → Redesign the section

C/C → centre to centre

(or shear R/f)

L7:- IS code Recommendation for Spacing of stirrups:-

The maximum C/C Spacing b/w stirrups shall be the minimum of (these 3)

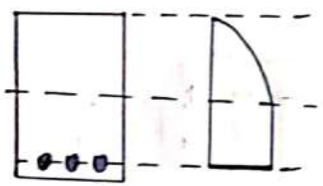
(i) $0.75d$ [Vertical stirrups]
(ii) d [inclined stirrups]
(iii) 300 mm [For all types of stirrups]

Spacing should be fixible (casting undestandable)

Note → 100 mm → 4"
150 mm → 6"
175 mm → 7"
200 mm → 8"
→ For knowledge only

L8:-

Q1)



Q2) Ans) $E_c = 5000\sqrt{f_{ck}}$ ($E_c \uparrow$ $f_{ck} \uparrow$)

$R = \dfrac{\text{Tensile strength}}{\text{compressive strength}}$ (Ratio)

$R\uparrow \rightarrow$ brittleness $\downarrow$

$R\downarrow \rightarrow$ brittleness $\uparrow$

(not a brittleness Formula but idea to get brittleness)

Q3)

Beam P

$B = 400$ mm
$d = 750$ mm
$\tau_{cmax} = 2.1$ N/mm^2
$\tau_c = 0.75$ N/mm^2
$V_u = 400$ kN

Beam Q

$B = 400$ mm
$d = 750$ mm
$\tau_{cmax} = 2.1$ N/mm^2
$\tau_c = 0.75$ N/mm^2
$V_u = 750$ kN

Sol) $\tau_v = \dfrac{V_u}{bd} = \dfrac{400\times10^3}{400\times750}$

$\tau_v = 1.333$ N/mm^2

[$\because \tau_v > \tau_c \rightarrow$ Design shear R/f.]

$V_{us} = (\tau_v - \tau_c)\,bd$
$= (1.333-0.75)\,400\times750$
$V_{us} = 175$ kN

$\tau_v = \dfrac{V_u}{bd} = \dfrac{750\times10^3}{400\times750}$
$= 2.5$ N/mm^2

$\because \tau_v > \tau_{c\,max}$

Redesign the Section.

L9:-

Q1)

$b = 230$ mm
$d = 450$ mm
$A_{st} = 4\times113$
M20 & Fe500

$V_u = 45$ kN
$S_v = 375$ mm
$\phi = ?$

% Ast	τ_c
0.25	0.36 N/mm^2
0.50	0.48 N/mm^2

Sol) $\tau_v = \dfrac{V_u}{bd} = \dfrac{45\times10^3}{230\times450} = 0.435$ N/mm^2

$\% A_{st} = \dfrac{A_{st}}{bd}\times100 = \dfrac{4\times113}{230\times450}\times100 = 0.437\%$

So, Now we have to do Interpolation

$\dfrac{0.50-0.25}{0.48-0.36} = \dfrac{0.50-0.437}{0.48-\tau_c}$

$2.0833 = \dfrac{0.063}{0.48-\tau_c}$

$\tau_c = 0.45$ N/mm^2

& $\tau_v = 0.435$ N/mm^2

$\because \tau_c > \tau_v$

minimum shear R/f shall be provided.

$\dfrac{A_{sv\,min}}{b\cdot S_v} \ngtr \dfrac{0.4}{0.87 f_y} \Rightarrow \dfrac{2\times\frac{\pi}{4}(\phi^2)}{230\times375} \ngtr \dfrac{0.4}{0.87\times500}$

$\phi = 7.1 \approx 8$ mm

L10:-
Q5b

$$\tau_v = \frac{V_u \pm \frac{M_u}{d}\tan\beta}{bd}$$

$\therefore V_{ud} = \tau_v \cdot bd$ ↳ Design shear force.

$$V_{ud} = V_u \pm \frac{M_u}{d}\tan\beta$$

tve :- when BM & depth both are increasing in two opp direction

$$V_{ud_{x-x}} = V_{u_{x-x}} + \frac{M_{u_{x-x}}}{d_{x-x}}\tan\beta$$

From fig.

$R_A + R_B = 20 \times 10 = 200$ kN

$\Sigma M_B = 0$

$R_A \times 20 - 10 \times 20 \times 10 = 0$

$R_A = 100$ kN

$R_B = 100$ kN

Shear force at section x-x

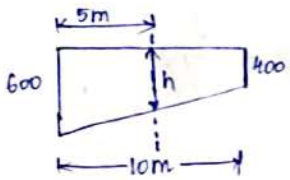
$V_{u_{x-x}} = R_A - 10 \times 5 = 50$ kN

BM at section x-x

$M_{u_{x-x}} = R_A \times 5 - 10 \times 5 \times \frac{5}{2}$

$= 500 - 10 \times 5 \times 5/2 = 375$ kNm

calculate now dear matlab is section (x-x) par kitni depth available hai.



$d_{x-x} = 600 - \frac{600-400}{10 \times 10^3} \times [5 \times 10^3]$

$d_{x-x} = 500$ mm

[No need for the calcul. you can get ans by the diagr itself] (if you notice carefully)

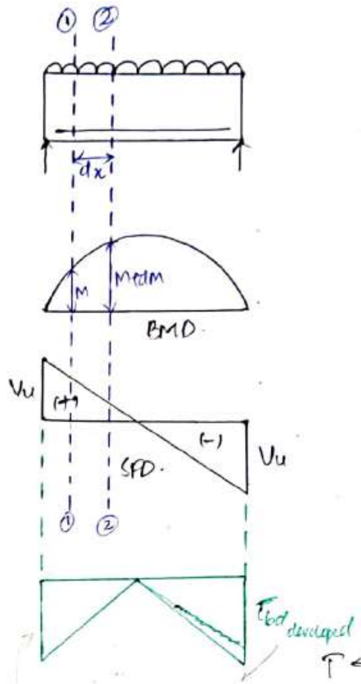
$\tan\beta = \frac{0.2}{10} = 0.02$

$V_{ud_{x-x}} = 50 + \frac{375}{0.5} \times 0.02$

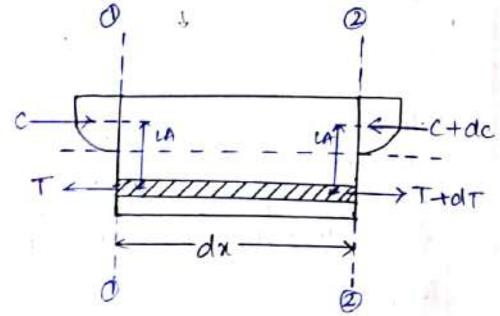
$V_{ud_{x-x}} = 50 + 15 = 65$ kN

L11:-

Bond Stress & its Estimation



The section is in balance section



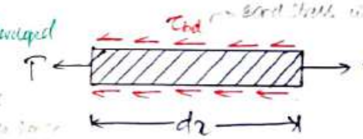
$M = T.LA$ —①

$M + dM = (T + dT)LA$ —②

eq ② - ①

$dM = dT.LA$

$$\boxed{dT = \frac{dM}{LA}} \text{—③}$$



$dT = \tau_{bd}[(\pi\phi)\,dx]n$

$$\boxed{dT = \tau_{bd}[(n\pi\phi)\,dx]} \text{—④}$$

eq ③ = ④

$$\frac{dM}{LA} = \tau_{bd}[(n\pi\phi)dx]$$

$$\tau_{bd} = \frac{dM}{dx} \times \frac{1}{n\pi\phi} \times \frac{1}{LA}$$

$$\boxed{\tau_{bd} = \frac{V}{(n\pi\phi)(LA)}}$$

where n → no of bars

LA → lever arm

Vu → shear force

$$\boxed{\tau_{bd\ developed} = \frac{V_u}{(n\pi\phi)\,LA}}$$

↳ Bond stress

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Bond check

$\boxed{\tau_{bd\,developed} \le \tau_{bd\,permissible}}$ → means beam is safe in bond.

bond stress. Bond strength of concrete

IS 456:2000 has given $\tau_{bd(perm)}$ (Bond strength) for plain R/f (Fe250) in tension zone.

This table is for Fe 250 (mild steel)

f_{ck}	M20	M25	M30	M35	M40 & above
$\tau_{bd(perm)}$ N/mm²	1.2	1.4	1.5	1.7	1.9

*Note :- For HYSD bars (Fe 415) the above values of $\tau_{bd(perm)}$ shall be increased by 60%. (× by 1.6)

* • For compression Rf the above value of $\tau_{bd(perm)}$ shall be increased by 25%. (× by 1.25).

Example:- ① Fe 415 used as Tensile Rf, (M20)

$\tau_{bd(perm)} = 1.2 \times 1.6 = 1.92\ N/mm^2$

② Fe 415 used as compression Rf, (M20)

$\tau_{bd(perm)} = 1.2 \times 1.6 \times 1.25 = 2.4\ N/mm^2$

③ Fe 250 (mild steel) used as compr Rf (M20)

$\tau_{bd(perm)} = 1.2 \times 1.25 = 1.5\ N/mm^2$.

L12:- Assignment Q1)

factored shear force · factored load

First step shear force V_u

$V_u = \frac{W_u l_{eff}}{2} = \frac{(1.5 \times 30) \times 7.4}{2} = 166.5 KN$

$l_{eff} = l_c + d \Rightarrow 7 + 0.4625 \Rightarrow 7.4625\ m$
or
$l_c + \frac{w}{2} + \frac{w}{2} \Rightarrow 7 + 0.4 \Rightarrow 7.4m$ } min

$d = D - 25 - \frac{25}{2}$

$d = 500 - 25 - \frac{25}{2}$

$d = 462.5mm$

$\boxed{LA = d - 0.42x_u}$

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

$x_u = \frac{0.87 \times 415 \times [2 \times 490.25]}{0.36 \times 25 \times 300}$

$x_u = 131.115\ mm$

$x_{ulim} = 0.48d = 0.48 \times 462.5 = 222\ mm$

$\because x_{ulim} > x_u$

$\therefore$ URS

$\therefore$ we x_u

$LA = d - 0.42x_u$
$= 462.5 - 0.42 \times 131.115$
$= 407.432\ mm$

$\tau_{bd(dev)} = \frac{V_u}{(n\pi\phi) LA} = \frac{166.5 \times 10^3}{(2 \times \pi \times 25) \times (407.432)}$

$\tau_{bd(dev)} = 2.602\ N/mm^2$

Now

$\tau_{bd(perm)} = 1.4 \times 1.6 = 2.24\ N/mm^2$ (M25 for HYSD)

Fail in bond criteria because $\tau_{bd(dev)} > \tau_{bd(perm)}$ in the given design of beam.

Now

$A_{st} = 2 \times 490.25 = 980.5\ mm^2$

$N = \frac{980.5}{\frac{\pi}{4}(16)^2} = 4.89 \simeq 5$ bars

Replacing 2#25 mm ϕ by 5#16 mm ϕ.

$d \ne D$ - by this d will change & hence

$d = D - \text{clear cover} - \frac{\phi}{2}$

$d = 500 - 25 - \frac{16}{2}$

$d = 467\ mm$

$l_{eff} = l_c + d \Rightarrow 7 + 0.467 \Rightarrow 7.467$
or
$l_c + \frac{w}{2} + \frac{w}{2} \Rightarrow 7 + 0.4 \Rightarrow 7.4$ } min

$l_{eff} = 7.4m$ ∴ nothing for leff

$\therefore V_u = 166.5\ kN$

$LA = d - 0.42x_u$

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Note: You should provide more no of bars of smaller diameter to satisfy the bond requirements (Hui bonding is good)

For $x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times [5 \times 201]}{0.36 \times 25 \times 300} = 134.4$ mm

$x_{ulim} = 0.48 d = 0.48 \times 467 = 224.16$ mm $\quad \because x_u < x_{ulim}$

$\therefore$ URS, use x_u for LA calculation

$LA = d - 0.42 x_u$

$= 467 - 0.42 \times 134.4 = 410.552$ mm

$\tau_{bd(act)} = \frac{V_u}{(n\pi\phi) LA} = \frac{165 \times 10^3}{(5 \times \pi \times 16)\, 410.552} = 1.614\ N/mm^2$

$\tau_{bd(per)} = 2.24\ N/mm^2$

* $\boxed{\tau_{bd(act)} < \tau_{bd(perm)}}$ Safe

Zyada surface area rakhna chahiye bars rakhenge by providing more no of bars of smaller dia

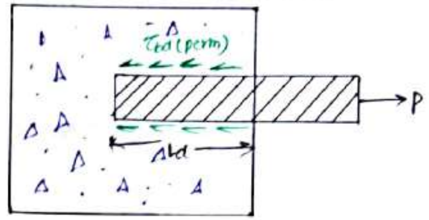
Note:- To control the bond stresses smaller dia bars should be provided more in numbers.
(Dont change the area of Rlf).

minimum length itna to rakhna hoga ok steel concrete se bahar na aa jaye
connection bana rahe junction with me
ex:- column & footing, slabs & beams etc.

L13:-

Development length & Lap Splices

Development length (L_d) :-



Pulling force = stress in steel × Area

$P = \sigma_{st} \cdot \left[\frac{\pi}{4}\phi^2\right]$ —① → applied force

Force due to Bond Stress

$= (\tau_{bd})_{perm} [\pi \phi L_d]$ —② → resistance force

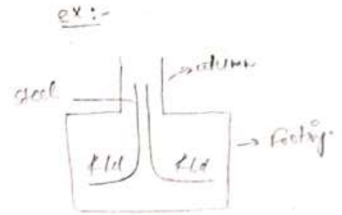
$(\tau_{bd})_{perm} \pi \phi L_d = \sigma_{st} \frac{\pi}{4} \phi^2$

$\boxed{L_d = \frac{\sigma_{st}\phi}{4\tau_{bd\,perm}}}$ (WSM) working s.m

$\boxed{L_d = \frac{0.87 f_y \phi}{4\tau_{bd\,perm}}}$ (LSM). limit s.m.

ϕ - dia of steel

ex:-



(i) Fe415 (Fe415), M20, Tension zone

$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87 \times 415 \times \phi}{4 \times (1.2 \times 1.6)} = 47\phi = L_d$

If dia is 10 mm
$L_d = 470$

(ii) Fe415, M25, compression zone

$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87 \times 415 \times \phi}{4 \times (1.4 \times 1.6 \times 1.25)} \Rightarrow L_d = 32.24\phi$

(iii) Fe250, M20, Tension zone.

$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87 \times 250 \times \phi}{4 \times 1.2} \Rightarrow L_d = 45.31\phi$

(iv) Fe250, M25, compression zone

$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87 \times 250 \times \phi}{4 \times 1.4 \times 1.25} \Rightarrow L_d = 31.07\phi$

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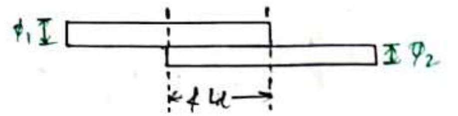
overlapping

Lap splices



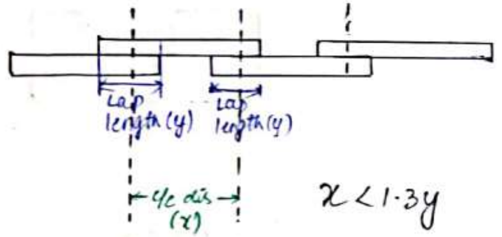
$\phi_1 < \phi_2$

$L_d = \frac{0.87 f_y \phi_1}{4\tau_{bd}(\text{perm})}$ → (vi) case.

(i) Lap splices shall not be done for $\phi > 36$ mm. ~~Bars~~.
Bars ($\phi > 36$ mm) are welded for lapping.

(ii) lap splices for bars $\phi > 36$mm can be permitted if additional spirals are provided

(iii) Lap splices can be staggered if the centre to centre distance of the splices $< 1.3 \times$ lap length.



$x < 1.3y$

Lap length depends on condition,

Tension zone

(iv) Lap length including anchorage value of hooks for bars in flexure $= \left.\begin{matrix} L_d \\ \text{or} \\ 30\phi \end{matrix}\right\} max$

(v) lap length in direct tension $= \left.\begin{matrix} 2L_d \\ \text{or} \\ 30\phi \end{matrix}\right\} max$

(vi) lap length of straight bars $= \left.\begin{matrix} 15\phi \\ \text{or} \\ 200\,mm \end{matrix}\right\} max$

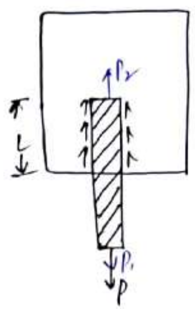
use this to maintain eff depth same

(v) lap length in compression $\geq \left.\begin{matrix} L_d \\ \text{or} \\ 24\phi \end{matrix}\right\} max$

(vi) when two bars of different diameter are overlapped then development length (Lap length) will be based on smaller ϕ.

L14:-

Q6.



$\left.\begin{matrix} P_1 = \sigma_{st}\left(\frac{\pi}{4}D^2\right) \\ P_2 = \sigma_b(\pi DL) \end{matrix}\right\} min$

$P_{max} = \text{minimum}\left\{\begin{matrix} \sigma_{st}\left(\pi/4\, D^2\right) \\ \& \\ \sigma_b(\pi DL) \end{matrix}\right.$

Q7. $\tau_{bd} = 1.2$ MPa for M20 (→ Bond strength)

$\sigma_{st} = 360$ mpa

$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{\sigma_{st}\phi}{4\tau_{bd}} = \frac{360 \times \phi}{4 \times 1.2 \times 1.6} \Rightarrow L_d = 46.875\phi.$

[If ask for nearest integer ans will be 47ϕ].

Q8. $L_d = \frac{1}{K}\left(\frac{\phi\sigma_{st}}{\tau_{bd}}\right)$ —(A)

WKT $L_d = \frac{\sigma_{st}\phi}{4\tau_{bd}}$

For (HYSD) $L_d = \frac{\sigma_{st}\phi}{4 \times 1.6\,\tau_{bd}}$

$L_d = \frac{\sigma_{st}\phi}{6.4\,\tau_{bd}}$ —(B)

$K = 6.4$

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compulsory for slab, beam, footing.

L-15:- **Development length check.**

$$(\tau_{bd})_{developed} = \frac{V_u}{(n\pi\phi)\,LA}$$

$$(\tau_{bd})_{perm} = \frac{0.87 f_y \phi}{4 L_d} \qquad \because \left(L_d = \frac{0.87 f_y \phi}{4\,\tau_{bd\,perm}} \right)$$

$$(\tau_{bd})_{dev} \le (\tau_{bd})_{perm}$$

$$\frac{V_u}{(n\pi\phi)LA} \le \frac{0.87 f_y \phi}{4 L_d}$$

$$L_d \le \frac{0.87 f_y \left[n \cdot \frac{\pi}{4}\phi^2\right] LA}{V_u}$$

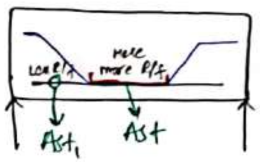
$$L_d \le \frac{0.87 f_y\, A_{st}\,(LA)}{V_u}$$

$$L_d \le \frac{M_1}{V_u}$$

As per IS 456:2000

At the Support, in Simply supported beam, for tension R/f

$$\boxed{L_d \le \frac{M_1}{V_u} + L_o}$$ → Additional Factor of safety.



(x_u) Depth of NA will differ so first cal x_u

$$x_u = \frac{0.87 f_y A_{st_1}}{0.36 f_{ck} b} \qquad M_1 = 0.87 f_y A_{st_1} (d - 0.42 x_u)$$

V_u → Shear Force at a given section

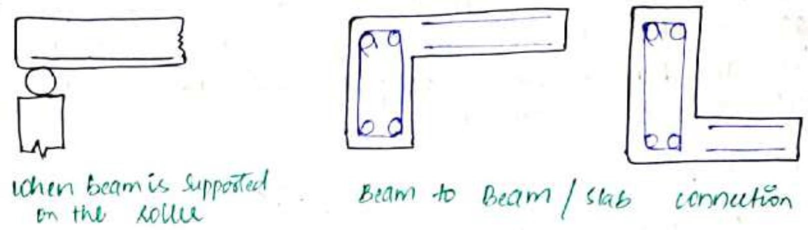
L_o → Sum of anchorage

$L_o = 12\phi$ or d } whichever is greater

Support p Bm kam hota hai isliye hum support p kam R/f dalte hain.

* $L_d < \frac{M_1}{V_u} + L_o$ shall be used when tension R/f is not confined by compressive reaction.

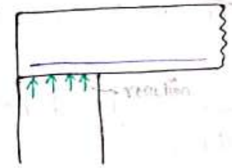
ex:-



* $L_d \le 1.3 \frac{M_1}{V_u} + L_o$, shall be used when tension R/f is confined by compressive reaction.

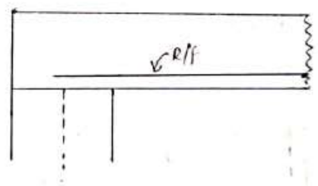
{when connected to beam it is occur confined by comp reaction}

ex:-

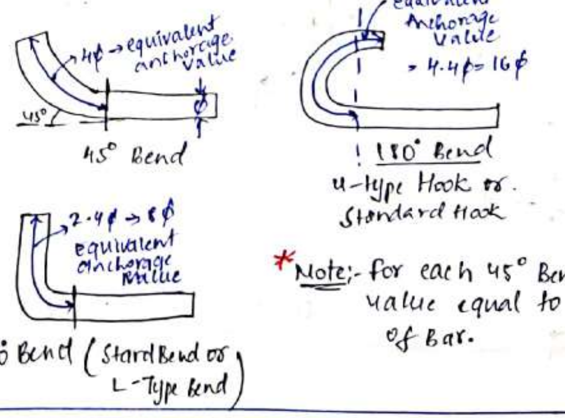


Note when confined by comp reaction in simply supp beam first term is increased by 30%. i.e. 1.3

L-16:- **Explanation :- In short**



Note:- L_o.



***Note:-** for each 45° Bend, consider the anchorage value equal to 4 times the diameter of Bar.

Anchorge - Bend - Hook :-

Some times it is observed that, due to insufficient space in the beam at the support, the straight development length of Rlf is not satisfied. In that case, the bars are bent or hooks are provided to find the full development length.

1) Anchoring bars in Tension :-

Mild steel me ← Bends & Hooks dalna hi dekha hai.

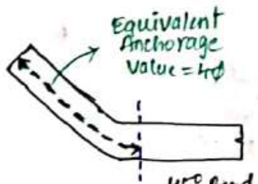
(a) Deformed bars (HYSD) may be used without end anchorage if development length requirement is satisfied. Generally Hooks & bends are normally provided in plain reinforcement (mild steel).

(b) Anchorage value can be taken as 4ϕ for each 45° bend.

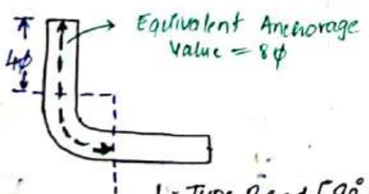
(c) 8ϕ for L-type bend & 16ϕ for U-type hook can be taken as anchorage value.

(d) The most common type of anchorages provided are U-Type hook & L-Type bend.

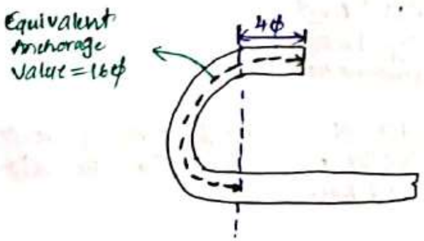
Standard Bend - L Type Bend
Standard Hook - U Type Hook



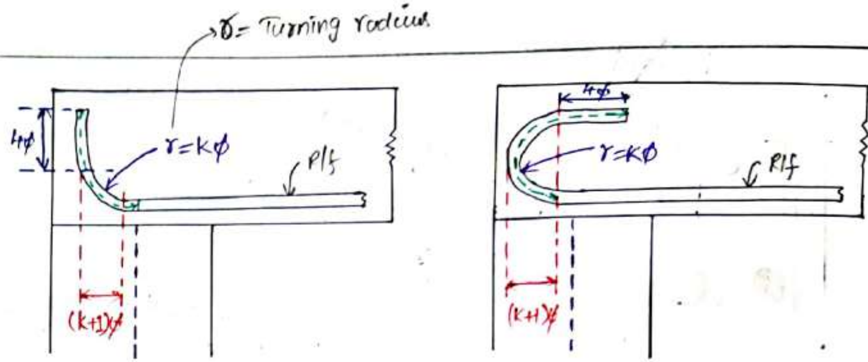
45° Bend



L-Type Bend [90° Bend]



U-Type Hook [180° Bend]



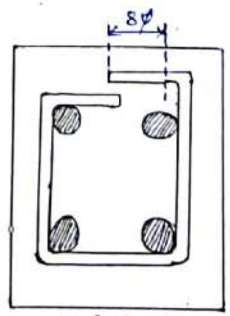
Note:-
* For mild steel, $k=2$, ∴ Turning radius $r = 2\phi$
* For HYSD/CTD, $k=4$, ∴ Turning radius $r = 4\phi$

(2) Anchoring Bars in compression :-

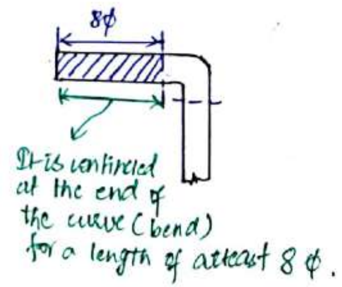
The anchorage length of straight bar in compression shall be equal to the development length of bars in compression. The projected length of hooks, bends are straight lengths beyond bends if provided for a bar in compression, shall only be considered for development length.

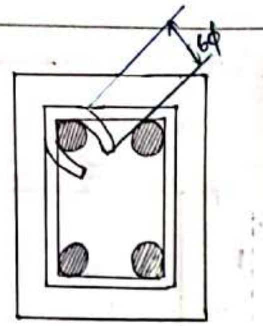
[In simple language :- we don't generally provide hooks & bends in compression Rlf, if suppose provide karte b hain in any case then what we do we consider that anchorage is the part of development length.]

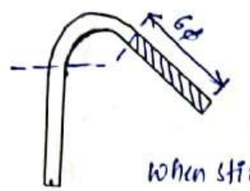
(3) Anchorage bars in Shear :-



90° Bend

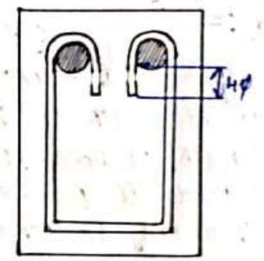


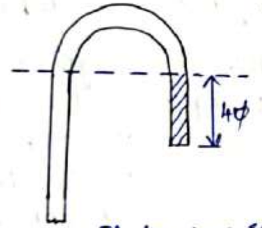




When stirrups is bend at 135° then it is continued beyond the end of the cure (bend) atleast for a length of 6ϕ.

135° Bend
(It is v.good Bend)
we use more this type of Bend





180 Bend

If the bar is bend through an angle of 180° then it shall be continued beyond the end of the curve atleast for a length of 4ϕ.

L17:- Assgs-Q6 When confined by compressive reaction in simply supported beam first term is incred. by 30% i.e (1.3).

$$L_d \le \frac{1.3 M_1}{V_u} + L_0$$

* $M_1 = ?$

$X_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b}$ [Depth of N.A hum support k liye nikal rahe hain]

$X_u = \frac{0.87 \times 415 \times (2 \times \pi/4 \times 20^2)}{0.36 \times 20 \times 250} \Rightarrow 125.96$ mm

[Now we have to check the section is under R/f, Bal., over R/f for that we have to calculate X_{ulim} & for X_{ulim} we have to calculate effective depth(d)]

Effective depth(d) = D − clear cover $- \frac{\phi}{2}$

$= 500 - \underset{\text{Assuming}}{(25)} - 20/2$ → you can assume 30, 35 also

$= 465$ mm

Now $X_{ulim} = k \cdot d$

$= 0.48 \times 465$

$X_{ulim} = 223.2$ mm

Compare X_u & X_{ulim}

∴ $X_u < X_{ulim}$

→ It is under R/f section

→ Use X_u

M_1 (moment of Resistance near the support) ye hum tension me ya comp zone me nikal sakte hain, ans same ayega. (but prefer in Tension zone). bcz it is easy

$M_1 = 0.87 f_y A_{st} (d - 0.42 X_u)$

$= 0.87 \times 415 \times (2 \times \pi/4 \times 20^2)[465 - 0.42 \times 125.96]$

$= 93438581$ Nmm × by 10^{-6} to convert into kN-m

$M_1 = 93.44$ kN-m.

Note:- We have studied earlier that for HYSD bars (or Deformed Bars) generally no need to provide Bend or Hooks, provide straight R/f If condition is satisfied. But if condition is not satisfied provide Hooks & Bends.
In mild steel, me dalne hi dalna hai Bends or hook.

* Since Deformed bar is being used therefore straight steel R/f is provided, that means No anchorage ($l_o = 0$).

$$L_d \le \frac{1.3 M_1}{V_u} + \cancel{l_o}^{\,0}$$

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 20}{4 \times 1.2 \times 1.6} = 940.23\text{mm}$$

$$\frac{1.3 M_1}{V_u} = \frac{1.3 \times 93.44 \times 10^6}{165 \times 10^3}$$

10^6 → To convert kN-m to N mm

10^3 → To convert kN to N

$$= 736.2 \text{ mm}$$

$$\therefore L_d \le \frac{1.3 M_1}{V_u}$$

$940.23 > 736.2$

$\therefore$ $\boxed{\text{Not Safe in bond}}$ [If safe in bond ans is over here]

Note * Since we have fail in development length check condition we have to provide Bends or Hooks. First provide L-Type Bend than U-Type Hook (Bcz U-Type hook is costly).

Providing L-Type Bend or standard bend (90° Bend)

So $l_o = 8\phi = 8 \times 20 \Rightarrow 160\text{mm}$

$$L_d \le \frac{1.3 M_1}{V_u} + l_o$$

$940.23 \quad 736.2 + 160$

$940.23 > 896.2$

$\boxed{\text{Not Safe in Bond}}$ [If safe in this Type Bend ans is over here]

Providing U-Type Hook (180° Bend) or Standard Hook

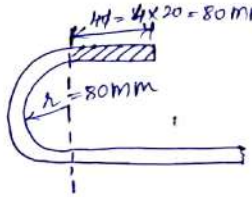
$l_o = 4.4\phi = 16\phi \Rightarrow 16 \times 20 = 320$ mm

$$L_d \le \frac{1.3 M_1}{V_u} + l_o$$

$940.23 \quad 736.2 + 320$

$940.23 < 1056.2$

Safe in Bond ⟹ That means provide U-Type Hook in Beam.



$r = K\phi$ $\quad$ ϕ = Dia of Bar.
$= 4 \times 20$ $\quad$ r = Turning radius..
$= 80$mm

L18:- Positive & Negative moment Reinforcement :-

IS code Recommendation

For Positive Moment R/f ⇒ means the moment ko resist karne k liye hum +ve moment R/f dalte hain.

① At least $\frac{1}{3}^{rd}$ of positive moment R/f in simple members (means simply supported Beam) & $\frac{1}{4}^{th}$ of positive moment R/f in continuous members (continuous Beam) shall extend along the same face of the member into (near) the support, to a length equal to $\frac{L_1}{3}$.

② At a simple support & a point of inflection, positive moment tension R/f shall be limited so that

$$L_d \le \frac{M_1}{V_u} + l_o \quad \text{or} \quad L_d \le \frac{1.3 M_1}{V_u} + l_o$$

(when not confined By CR) $\quad$ (when confined By CR)

Positive moment ⇒ sagging	Negative moment ⇒ hogging
	

Provided in Tension ex -ve moment

for Negative Moment Reinforcement :-

Atleast $\frac{1}{3}^{rd}$ of the Total R/f provided for Negative moment at the support shall extend beyond the point of inflection for a distance

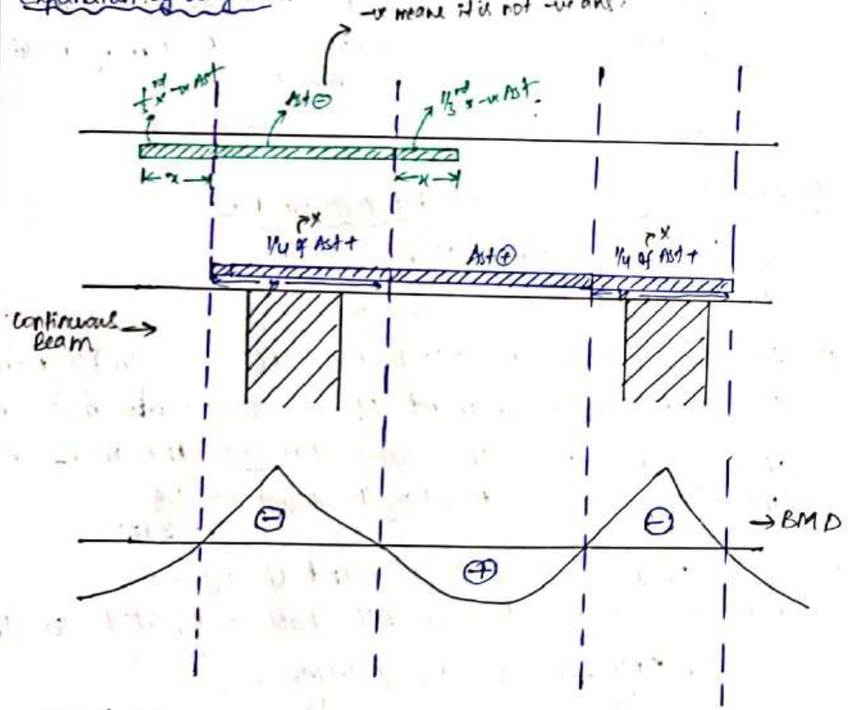
(i) d → effective depth of Beam.

(ii) 12ϕ → dia of bar

(iii) $\frac{1}{16}\times$ clear span

} Whichever is greater

Explanation by diagram:-



$x = d$ or $= 12\phi$ or $= \frac{1}{16}\times l_c$ } max

$y = \frac{l_1}{3}$

Ex:- Ast ⊖ = 9 bars

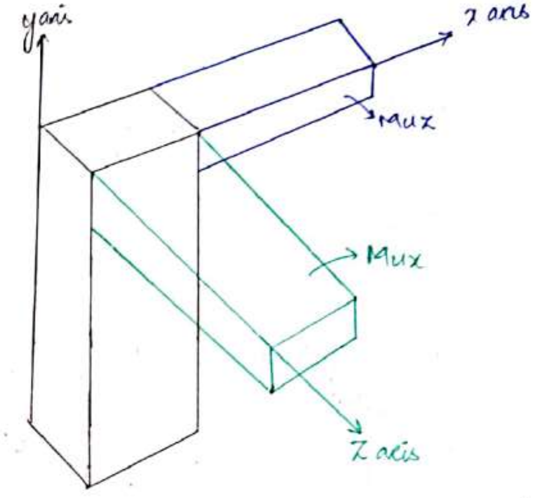
then $x = \frac{1}{3}\times 9 = \underline{3 \text{ bars}}$

ex: Ast ⊕ = 12 bars

then $y = \frac{1}{4}\times 12 = 3$ bars

Torque → Force ko show karta hai
Torsion → Deformation
} Torque lagaoge To Torsion milega.

2.9:- Limit State of collapse under Torsion condition :-



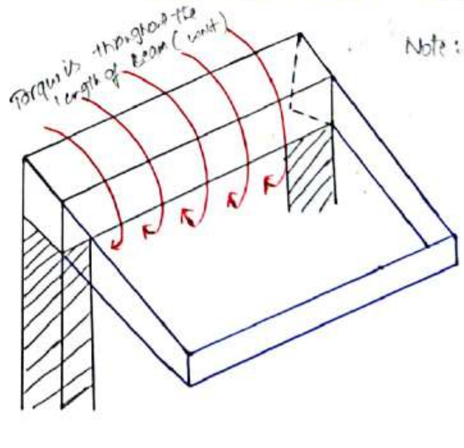
Mux → B.m about z axis

Mux → B.m about x axis

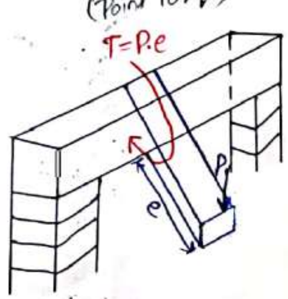
Jab bhi durse k axis k about mere me yadi moment aaraha hai to usko B.M ka nam dunga aur jab khud k axis a about mere me moment aarahai hai to usko mai torque kahunga.

Torque k karan jo angular Deformation hoga usko hum torsion kahte hain.

Note:- Generally overhang portion p Torque ata hai.



(Point Torque)



Equivalent Shear Force

$$V_{ueq} = V_u + V_T$$

$$\boxed{V_{ueq} = V_u + 1.6\,\frac{T_u}{b}}$$

$V_u \rightarrow$ shear force due to live load or dead load etc
$V_T \rightarrow$ S.F due to torque

Equivalent Bending moment

$$M_{ueq} = M_u + M_T$$

$$\boxed{M_{ueq} = M_u + T_u\,\frac{[1+(D/b)]}{1.7}}$$

$M_u \rightarrow$ B.M due to live load or dead load etc
$M_T \rightarrow$ B.M due to torque

Case I :- Design a beam [when c/s dimensions are not given]
[Given data $\rightarrow f_{ck}, f_y, M_{ueq}, V_{ueq}, b$]

If suppose M_{ueq} & V_{ueq} is not given then cal by above formulas.

Step 1 :- calculate Effective depth

$$d_{required} = \sqrt{\frac{M_{ueq}}{R f_{ck} b}}$$

$\rightarrow$ Same as Single R/f beam formula (only Replace M_{ueq})

$R \Rightarrow 0.148$ (Fe250)
$\Rightarrow 0.138$ (Fe 415)
$\Rightarrow 0.133$ (Fe 500)

Step 2 :- calculate area of steel R/f

$$A_{st} = 0.5\,\frac{f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6\,M_{ueq}}{f_{ck}\,b d^2}}\right] bd$$

Case II :- Design a beam [When c/s dimensions are known]
[Given data :- $B, d, f_{ck}, f_y, M_u, V_u$] $\xrightarrow{}$ S.F
B.M $\quad T_u \rightarrow$ Torque

Step 1 :- Estimate M_{ulim}

$$\boxed{M_{ulim} = R f_{ck} b d^2}$$

bachane wali Taqat.

$R = 0.148$ (Fe 250)
$= 0.138$ (Fe 415)
$= 0.133$ (Fe 500)

Step 2 :- Calculate Equivalent B.M

$$\boxed{M_{ueq} = M_u + T_u\,\frac{(1+\frac{D}{b})}{1.7}}$$

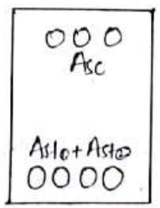
Todne wali Taqat.

if $M_{ueq} < M_{ulim}$
Design singly R/f Beam

$$A_{st} = 0.5\,\frac{f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6\,M_{ueq}}{f_{ck}\,b d^2}}\right] bd$$

← Design finish hear for singly R/f Beam

If $M_{ueq} > M_{ulim}$
Design a doubly R/f Beam



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

$$A_{st_2} = \frac{M_{ueq}-M_{ulim}}{0.87\,f_y\,(d-d')}$$

$$A_{sc} = \frac{0.87\,f_y\,A_{st_2}}{f_{sc}-0.446 f_{ck}}$$

Special case :- (valid for case I & case II only when $M_T > M_u$) otherwise no need for this case.

WKT $M_{ueq} = M_u + M_T$

if $M_T > M_u$ → That means Torque is dominating. Therefore additional compression R/f is provided for the equivalent B.m M_{e_2}

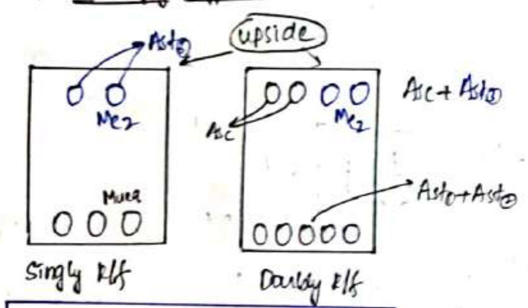
Where $\boxed{M_{e_2} = M_T - M_u}$

M_{e_2} works in opposite sense of M_u

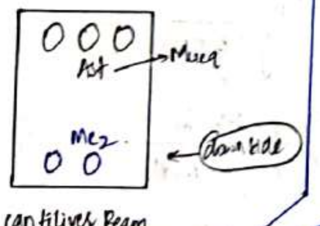
if $M_u \rightarrow$ Sagging
$M_{e_2} \rightarrow$ Hogging & vice versa

Ex:-

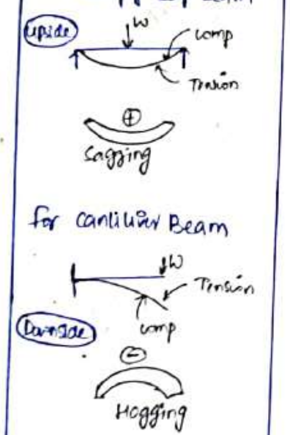
a) For simply supp. beam



b) For cantiliver Beam



Explanation



Ast_2 hum M_{e_2} ko resist / lene k liye dalte hain aur use comp zone me dalte hain.

$$\boxed{Ast_{(2)} = \frac{M_{e_2}}{0.87\, f_y\, (d-d')}}$$

Here comes the Design of stirrups R/f in beam. Before this we have (Design only for main/longitudinal R/f.

120:- Design of Shear R/f considering the Effect of Torque :-

* Equivalent Nominal Shear Stress (τ_{veq})

$$\boxed{\tau_{v_{eq}} = \frac{V_{ueq}}{bd}}$$

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

where $V_{u_{eq}} = V_u + 1.6\,\frac{T_u}{b}$

Case I: When $\tau_{v_{eq}} < \tau_c$

$\tau_{v_{eq}}$ → Todne wali Taqat. τ_c → Bachane wali Taqat.

Provide minimum Shear R/f

$$\boxed{\frac{Asv_{min}}{b \cdot S_v} \geq \frac{0.4}{0.87\, f_y}}$$

$Asv \rightarrow n \times \frac{\pi}{4} (\phi^2)$.

$n \rightarrow$ no. of legs.

$f_y \not> 415$ N/mm²

$\tau_c \rightarrow$ shear strength of concrete [Table 19].

$$\% Ast = \frac{Ast}{bd} \times 100$$

$Ast \rightarrow$ area of Tensile steel R/f in tension zone near the support designed for M_{ueq}.

%Ast	M20	M25	M30
0.10			
0.15			
0.20			
0.25			
0.50			

Case II: When $\tau_{v_{eq}} > \tau_c$

Design the Stirrups

$V_{us} = V_{ueq} - V_c$ | WKT $V_{us} = V_u - V_c$

OR

$V_{us} = (\tau_{veq} - \tau_c)\, bd$

Spacing of Stirrups.

$$\boxed{S_v = \frac{0.87\, f_y\, Asv\, d_1}{\frac{V_u}{2.5} + \frac{T_u}{b_1}}} \quad (1)$$

→ 250mm

Scanned with CamScanner

Stirrups / vertical stirrups { In torque bentup bars are not provided bcx they are not capable of taking the torque value of shear.

In this case when $\tau_{veq} > \tau_c$ minimum shear R/f shall be provided i,e

$$\boxed{\frac{A_{sv\,min}}{b \cdot S_v} \geq \frac{(\tau_v - \tau_c)}{0.87\, f_y}} \quad -②$$

200mm

$$S_v = \min \begin{cases} eq① \\ eq② \end{cases}$$

Spacing provided must be min of eq① & eq②

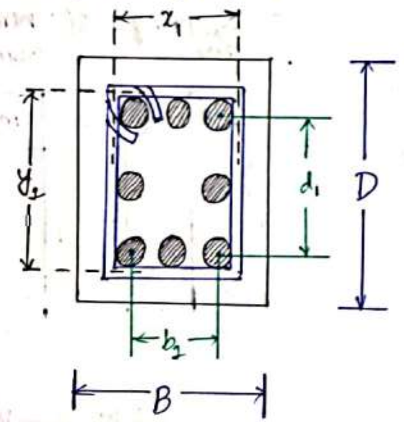
case III When $\tau_{veq} > \tau_{c\,max}$

Redesign the Section

$\tau_{c\,max}$ → max shear strength of concrete after providing the shear R/f

[Sabkuch provide kardiya longi R/f, Shear R/f, Jo max shear strength nikalti hai concrete ki wo hai $\tau_{c\,max}$]

* Explanation of d_1 & b_1 :-



d_1 → c/c Distance of R/f along the depth

b_1 → c/c Distance of R/f along the width

by considering any case

The maximum Spacing of Stirrups shall not Exceed

(i) x_1 → 200 mm
(ii) $\frac{x_1 + y_1}{4}$ → 180 mm
(iii) 300mm

whichever is minimum

iv) $\frac{A_{sv\,min}}{b S_v} \geq \frac{0.4}{0.87\, f_y}$

120mm provide that

L21:- Q9) (M) $M_u = 200$ kN-m   nothing will happen ans will be same here
(V) $V_u = 20$ kN
(T) $T_u = 9$ kN-m
(b) $b = 300$ mm
(D) $D = 425$ mm

a) $V_{ueq} = V_u + 1.6\frac{T_u}{b} = 20 + 1.6 \times \frac{9}{0.3} = 20 + 48 \Rightarrow 68$ kN

b) $M_{ueq} = M_u + \frac{T_u(1+D/b)}{1.7} = 200 + \frac{9 \times \left[1 + \frac{425}{300}\right]}{1.7}$

$= 200 + 12.79$

$M_{ueq} = 212.8$ kN-m

$\simeq 213$ kN-m

L22:-
Q10)
$b = 230$ mm
$d = 400$ mm
$V_u = 120$ kN
$f_{ck} = 20$ N/mm²
main R/f ← $f_y = 415$ N/mm²
stirrups ← $f_y = 250$ N/mm²
(Shear R/f) $\tau_c = 0.48$ N/mm² (Bachane wali Taqat)

First cal Nominal Shear stress

a) $\tau_v = \frac{V_u}{bd} = \frac{120 \times 10^3}{230 \times 400} = 1.304$ N/mm²
(τ_v: Todne wali Taqat)

$\because \tau_v > \tau_c$
Design shear stirrups

[we have 3 option ① bentup bars provide kardo ② yo inclined stirrups ③ ya vertical stirrups. most of the case hum vertical stirrups hi provide karte hain jab tak aur me specify na kare Bentup bars karo ya inclined stirrups. By default hum vertical stirrups ki hi baat karenge]

ye additional S.F hai jisko hum resist nahi kar pa rahe hain. Hamara concrete resist nahi kar pa raha hai so itni value ko resist karne ke liye hume shear R/f (stirrups) dalna padega. (Design karna padega)

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

$V_{us} = (1.304 - 0.48) \cdot 230 \times 400$

$V_{us} = 75808$ N

Desiging stirrups.

$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v} = \frac{0.87 \times 250 \times [2 \times \pi/4 \times 8^2] \times 400}{75808}$

$S_v = 115.41$ mm —①

① Conditions satisfying

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

$\frac{2 \times \pi/4 \times 8^2}{230 \times S_v} \geq \frac{0.4}{0.87 \times 250}$

$S_v \leq 237.7$ mm —②

ye value se kam hame rakhna hi hoga so eq① is ans.

[If we get 250mm in eq① then the ans is eq 2].

② Condition $S_v \leq 0.75d$ or 300 mm } min → 300mm

$\therefore S_{v\,max} \leq 300$ mm —③

ye max spacing hai isse zyada spacing hume kabhi nahi rakhna chahiye chahe wo inclined stirrups ho ya vertical stirrups.

S_v = 115.41 mm ✓ or 237.7 mm or 300 mm } min

b) $T_u = 10.090$ kN-m

$V_{ueq} = V_u + 1.6 \frac{T_u}{b} = 120 + \frac{1.6 \times 10.090}{0.23} = 195.826$ kN

Todne wali Taqat

Wkt. $V_c = \tau_c bd$

Bachane wali Taqat

$= 0.48 \times 230 \times 400$

$= 44.16$ kN

$V_{us} = V_{ueq} - V_c$

$= 195.826 - 44.16$

$V_{us} = 151.67$ kN

Since $V_{ueq} > V_c$

Total 195.826 SF aaraha hai aur humari concrete ne 44.16 KN SF resist karliya hai. Jo bache value k liye hame shear stirrups provide karna padega. i.e V_{us}

A_{ssf} → side face Area of steel.

23: IS code Recommendation for side face R/f :-

Minimum side face R/f is provided when

(i) $D > 750$ mm & Beam is subjected to or may not subjected to torsion

(ii) $D > 450$ mm & Beam is subjected to torsion.

$A_{ssf_{min}} = 0.1\%$ of Total c/s area (when Beam is rectangular)

$= 0.1\%$ of total web area in the case of flanged beam.

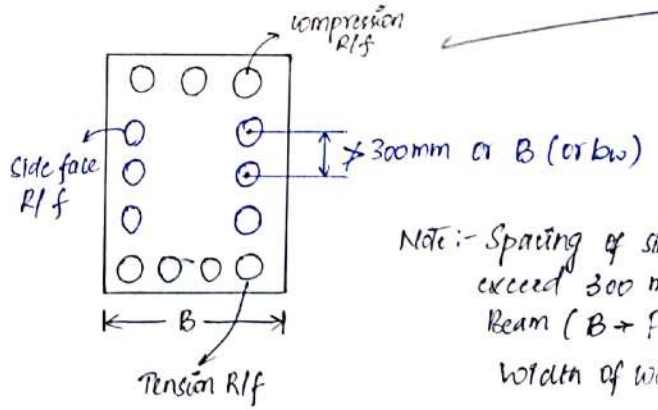
(agar Flanged beam hai, uska jo web portion hota hai uska 0.1% area dalna padega)

Ex:- (For Rectangular Section)

$$A_{ssf_{min}} = \frac{0.1}{100} \times B \times D.$$

& No of bars $= n = \dfrac{A_{ssf\,min}}{\frac{\pi}{4}(\phi^2)}$ ⇒ so if we get 6 no of bars then equally distribute 6 bars

This side face R/f shall be equally distributed on both the side faces.



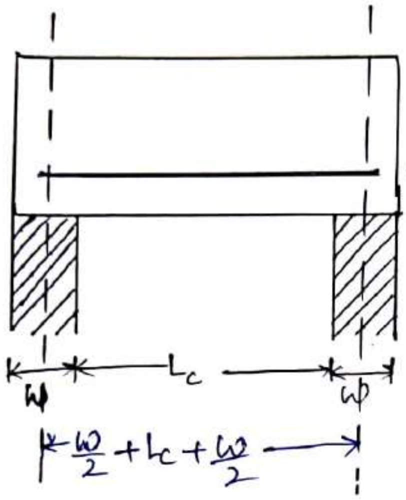
Note:- Spacing of side face R/f shall not exceed 300 mm or width of Beam (B → For Rect beams) or width of web (b_w → For when flanged Beams)

CHAPTER: 04 SLAB AND STAIR CASE

L9:- Effective Span in Different Support conditions :-

(a) Effective Span

(a) Simply Supported Beam/Slab



$l_{eff} = L_c + \frac{w}{2} + \frac{w}{2}$ OR $L_c + d$ } whichever is less

(you may take diff width of sup also).

(b) continuous Beam/slab

1^{st} condition : if $w < \frac{l_c}{12}$

$w \rightarrow$ width of support
$l_c \rightarrow$ clear span of continous Beam/slab

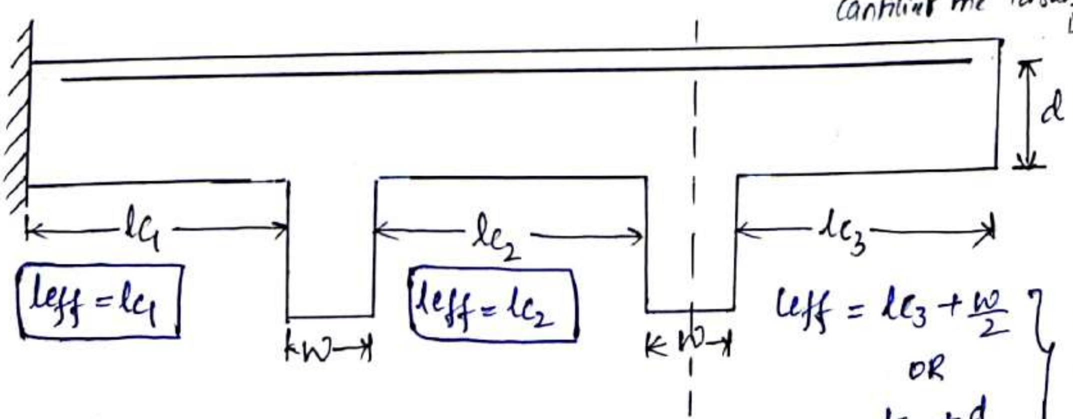
$l_{eff} = l_c + \frac{w}{2} + \frac{w}{2}$ or $l_c + d$ } whichever is less.

It is same as previous case of simply supported beam/slab.

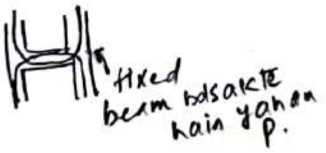
eg, 450mm   > 600 mm   w > 450 mm

2^{nd} condition : if $w > \frac{l_c}{12}$ or 600mm } whichever is less.

Cantiliver me Tension Top me ata hai isliye main R/f top me dalte hain



$l_{eff} = l_{c_3} + \frac{w}{2}$ OR $l_{c_3} + \frac{d}{2}$ } whichever is less



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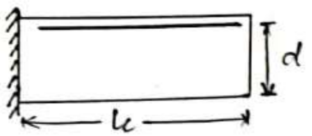
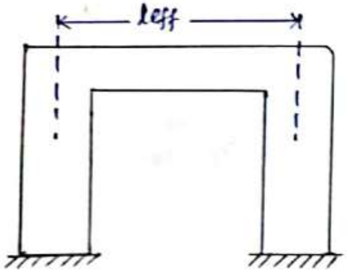
(C) Cantilever Beam/slab

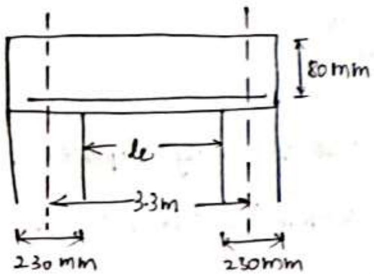


$$\boxed{l_{eff} = l_c + \frac{d}{2}}$$

(d) Frames.



L2:- Q1)

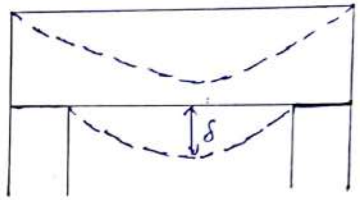


$l_{eff} = l_c + d \Rightarrow 3.07 + 0.08 = 3.15m$
or
$l_c + \frac{w}{2} + \frac{w}{2} \Rightarrow 3.3m$ } less

$l_c = 3.3m - 0.23$
$l_c = 307$ m

L3:- Check for Deflection in Beam & slab :-

(i) Final deflection occuring including all the loads & the effect of temperature, creep & shrinkage etc. shall not exceed $\frac{span}{250}$ and it is measured from cast level of the support.



$\delta \not> \frac{span}{250}$.

(ii) The deflection due to all the loads, Tempt, Shrinkage & creep, after the erection of partition wall and application finishes shall not exceed $\frac{span}{350}$ or 20mm, whichever is less.

L4:- Control on Deflection in Beam & one way slab :

Basic ratio of span to effective depth → A

For cantilever Beam/one way slab ; A = 7
For simply supported Beam/one way slab ; A = 20
For continous Beam/one way slab ; A = 26

To beam ya slab apki ready hai uski value hai ye

$$\boxed{\frac{Span}{eff \cdot depth(d)} \leq A \times f_1 \times f_2 \times f_3 \times f_4}$$

A → Basic span to eff. depth Ratio.

$f_1 f_2 f_3 f_4$ → These all are known as correction/modification/Reduction factors.

$$\boxed{d \geq \frac{Span}{A\, f_1 f_2 f_3 f_4}}$$

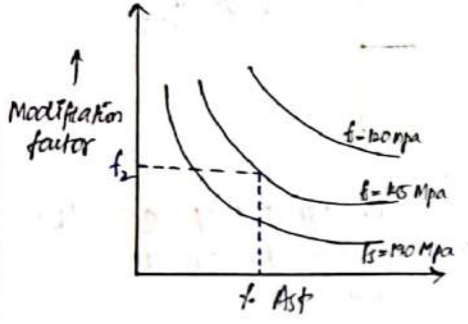
By this eff. depth of Beam or slab can be calculated. (Dimension)

$f_1 \rightarrow$ It is applied when span exceeds 10m.

$$\boxed{f_1 = \frac{10}{\text{Span in meters}}}$$

$f_1 = 1 \rightarrow$ for spans less than 10.m.

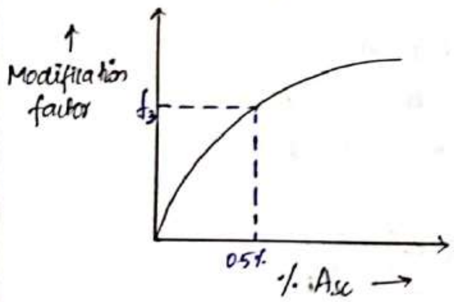
$f_2 \rightarrow$ Modification factor which depends upon the % of tension R/f.



$$f_s = 0.58\, f_y \frac{Ast_{required}}{Ast_{provided}}$$

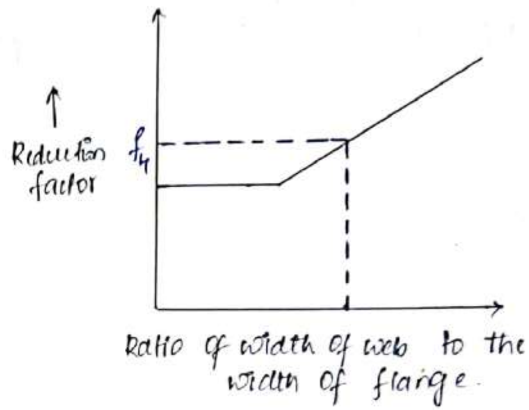
(fig. 38) IS 456:2000

$f_3 \rightarrow$ Modification factor which depends upon % of compression R/f.



f_4:– It is a Reduction factor which depends upon the Ratio of width of web to the width of flange. in The case of flanged. Beam.

For Rectangular Beams, $f_4 = 1.0$



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depth matlab yahan deflection se related hai.

L5:- **Question Based on control on Deflection**

Q4 calculate minimum depth of a Rectangular cantiliver Beam of span 6m.

Sol WKT $d \geq \frac{span}{A \times f_1 f_2 f_3 f_4}$

$d \geq \frac{6000}{7 \times 1 \times 1 \times 1 \times 1}$ (bcz $f_1 f_2 f_3 f_4$ are not given)

$d \geq 857.14 mm \approx 860 mm$

eff. cover → 40 mm

$\boxed{D = 900 mm}$ → Itne to Depth rakhna hi hoga tabhi apki beam safe rahegi. (only concrete beam ke liye (no R/f))

provided area of tension R/f = 0.8% } Fe 415 (But code ne mujhe min R/f rakhne ke liye kaha hai)
required area of tension R/f = 0.4% from the design

calculate minimum depth.

$d \geq \frac{Span}{A \times f_1 \times f_2}$

$d \geq \frac{6000}{7 \times 1 \times 1.8}$ (only $f_1 f_2$ are given)

$d \geq 476.2 mm \approx 480 mm$

eff cover = 40 mm

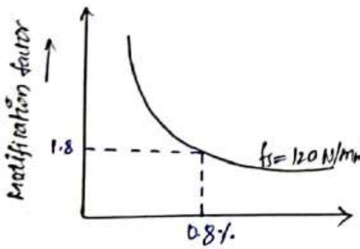
$\boxed{D = 520 mm}$

(After providing R/f of 0.8% depth of beam reduced from 900mm to 520mm)

∵ Span < 10m, $f_1 = 1$

$f_s = 0.58 \times f_y \times \frac{Ast_{required}}{Ast_{provided}}$

$= 0.58 \times 415 \times \frac{0.4\%}{0.8\%} = 120.35 N/mm^2$



$f_2 = 1.8$

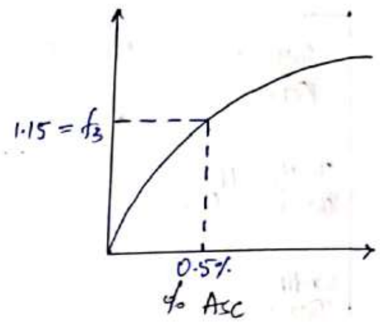
provide compression R/f of 0.5%. Find the minimum depth

$d \geq \frac{Span}{A f_1 f_2 f_3}$

$d \geq \frac{6000}{7 \times 1 \times 1.8 \times 1.15}$

$d \geq 414 mm \approx 420 mm$

$f_1 = 1$
$f_2 = 1.8$
$f_3 = ?$



Effcover = 40 mm

$\boxed{D = 460 mm}$

(comp R/f dalne par value aur kam hogai depth of Beam ki)

f_4 → Reduction factor, which is equal to 1.0 for Rectangular beam section.

[It is only for flanged Beam].

So final minimum Depth of Rectangular Beam is 460 mm (you can increase also 500 mm... 600mm)

100 mm → 4"
200 mm → 8"
300 mm → 12"
400 mm → 16"
500 mm → 20"

L6:- Control on Deflection in Two way Slab

or check.

Support Condition	Basic Ratio of Span to gross Depth ($\frac{L}{D}$)	
	Mild steel	HYSD/CTD/TMT
Simply supported Two way slab	$\frac{L}{D} = 35$	$0.8 \times 35 = 28 = \frac{L}{D}$ (80% x 35)
continuous Two way slab	$\frac{L}{D} = 40$	$0.8 \times 40 = 32 = \frac{L}{D}$ (80% x 40)

condition :- The above values of $\frac{L}{D}$ can be used upto the loading class of $3 KN/m^2$ & span not exceeding 3.5m.

means slab p $3 KN/m^2$ ka load lagraha hai to ye useful hai

Load $\not> 3 KN/m^2$
Span $\not> 3.5 m$ [Tabhi ye uper k values use kar sakte hain]

Q. calculate expected/probable depth of two way slab for the given span of 2.5 m. [use Fe415] [Simply Supported]

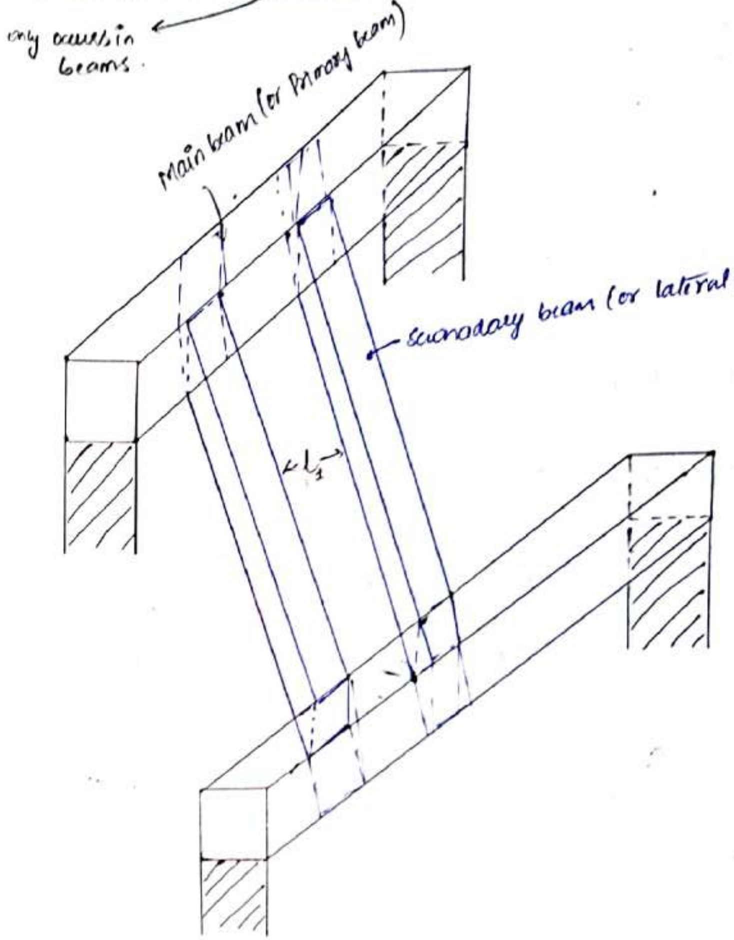
Sol. $\frac{L}{D} = 28$

$D \geq \frac{2.5 \times 1000}{28}$ } min. itna to hai hi chahiye

Depth deflection ko govern karte hai. (Depth is responsible for deflecn)
(Depth zyada hoga to deflection kam hoga)

L7:- Check for Lateral Stability Of Beams

only occurs in beams.

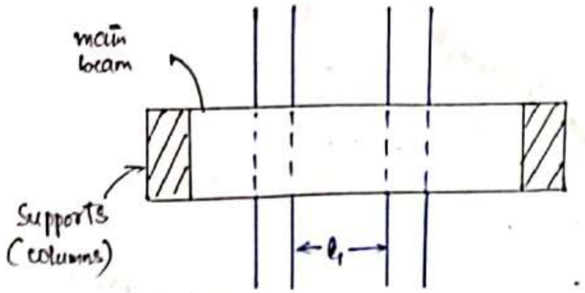


(a) For Simply Supported / Continuous Beam.

Clear distance b/w two lateral restraints shall not exceed $60b$ or $\frac{250b^2}{d}$ whichever is less

$l_1 \leq 60b$
$\leq \frac{250b^2}{d}$ } whichever is less

(Take beam bukle na ho)



(b) Cantilever Beam :-

Clear distance b/w two lateral restraints shall not exceed $25b$ or $\frac{100b^2}{d}$ whichever is less

$$\left.\begin{aligned} l_1 &\le 25b, \\ &\le \frac{100b^2}{d} \end{aligned}\right\} \text{whichever is less.}$$

$b \rightarrow$ width of main beam

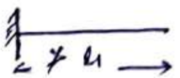
$d \rightarrow$ eff depth of main beam

Explanation

- In this case def & buckling both will occur.
- we have studied def criteria in previous lectures

For Buckling criteria
length of cantilever beam should not be more than l_1

eg



→ here buckling will not occur.

Q.4 A Beam of span 5.6m is used as cantilever of size 250 mm x 700 mm. check for the beam for Deflection & lateral stability.

(eff depth)

Sol:

Explanation :- Deflection check means ⇒ depth ko check karna hai k wo sahi provided hai ya nahi.

Lateral stability check means ⇒ width of section ko check karna hai wo sahi provided hai ya nahi.

Sol: # Deflection criteria

$$d \ge \frac{span}{A \times f_1 f_2 f_3 f_4}$$

$$d \ge \frac{5600}{7}$$

$$\boxed{d \ge 800mm}$$

∵ required depth (800mm) > provided depth (700mm)

∴ Beam fails in deflection.

Lateral stability criteria

Ist method $l_1 \le 25b \rightarrow 25 \times 250 \Rightarrow 6250$ mm

or

$\frac{100b^2}{d} \rightarrow \frac{100 \times (250)^2}{700} \Rightarrow 8928.6$ mm

} whichever is less.

$$l_1 \le 6250 \text{ mm}$$

$$\boxed{l_1 \le 6.25 \text{ m}}$$

⇒ means span 6.25 m se chota hona chahiye tabhi safe hoga

Span 5.6m < 6.25m ∴ Hence Safe

∴ No lateral unstability (means no need of lateral restraints).

II Method:-

* Minimum width of the section required is :-

$$b \ge \frac{l_1}{25}$$

$$b \ge \frac{5600}{25}$$

$$\boxed{b \ge 224 \text{ mm}}$$

required width = 224mm

provided width = 250mm

∴ Hence Safe

Conclusion:- × Fail in deflection
× Pass in lateral stability

So, depth should be increased i.e $\boxed{d \geq 800mm}$

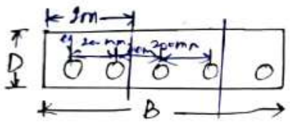
L8:-

IS code Recommendation for Design of Slabs:-

1) Minimum R/f (Distribution R/f) in the slab.
= 0.15% of gross area [Mild Steel]
= 0.12% of gross area [HYSD Steel].

$\boxed{\text{Gross area} = B \times D = 1000 \times D}$
meter

Slab k case me total 6 nahi lete hain per meter p baat karte hain

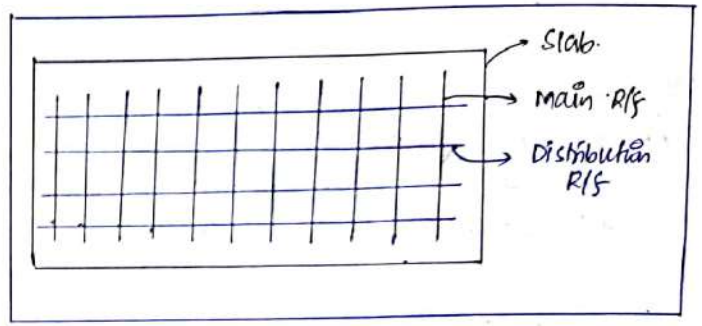


2) Maximum diameter of bar in slab.
$\phi_{max} \not> \frac{1}{8} \times$ thickness of slab.

Eg:- The maximum diameter of bar that can be provided in 75mm thick RCC slab.
a) 8mm b) 10mm c) 12mm d) 16mm

Sol:
$\phi_{max} \not> \frac{1}{8} \times 75$
$\phi_{max} \not> 9.375$
(a)

3) Maximum Spacing of Bars.



(a) Main bars(R/f).
(i) 3d
(ii) 300mm } whichever is less

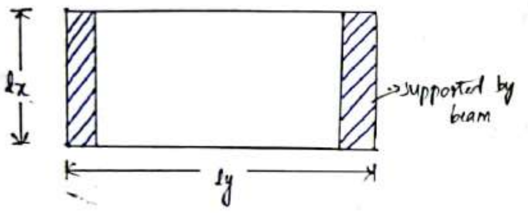
(b) Distribution R/f (Minimum R/f).
(i) 5d
(ii) 450mm } whichever is less

19:-

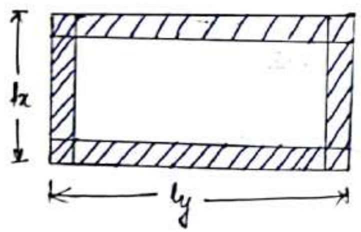
Types of Slabs :- → One way slab. → Two way slab.

One way slab :-

Conditions:- (a) If the slab is supported on the two opposite edges then it is always an one way slab then whatever will be the l_y & l_x.



(b) If the slab is supported on all the four edges and $\frac{l_y}{l_x} > 2.0$ then it is called one way slab.



$\frac{l_y}{l_x} > 2.0$

where $l_y \rightarrow$ longer span.
$l_x \rightarrow$ shorter span.

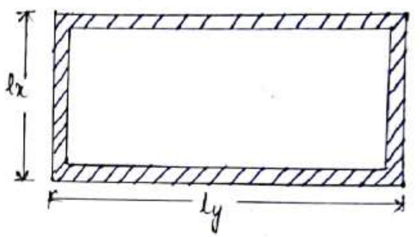
If the slab is supported on two supports than no need to check the ratio of $\frac{l_y}{l_x}$ (Dimension). It is one way slab.

If slab is supported on all the sides then check the ratio. If $\frac{l_y}{l_x} > 2$ then one way slab.

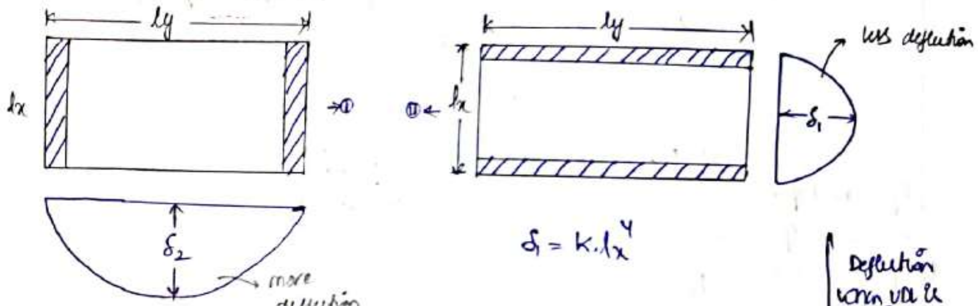
Q:- Which span is governing in Bending and deflection in One way slab.

Ans:- Shorter span. (means shorter span me bending & deflection zyada hota hai).

*



$\Rightarrow \frac{l_y}{l_x} > 2$

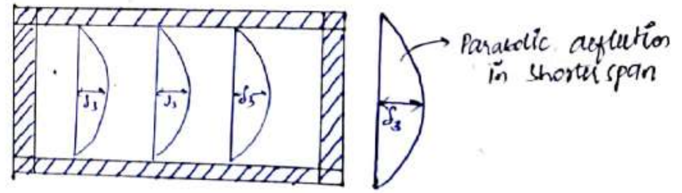


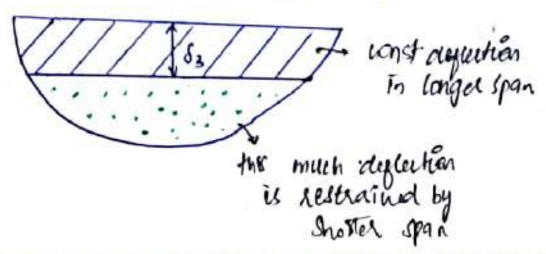
$\delta_2 = K \cdot l_y^4$ $\quad \delta_2 > \delta_1$

$\delta_1 = K \cdot l_x^4$

Deflection when UDL is applied $\left(\frac{5}{384} \frac{wl^4}{EI}\right)$
$\delta = Kl^4$

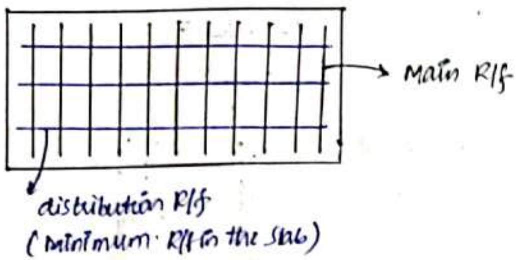
Adding both ① & ②





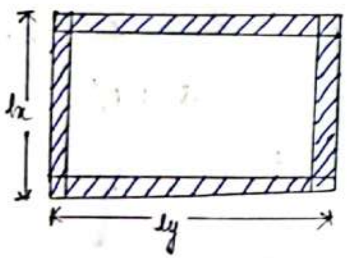
∴ Shorter span is critical.
Isliye main R/f along the shorter span dalte hain one way slab me.

Design of one way slab.



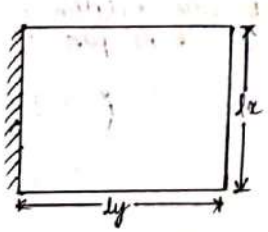
Two way Slab:- If the slab is supported on all the four edges and $\frac{l_y}{l_x} \leq 2.0$ then it is called two way slab.

l_y → larger span
l_x → shorter span
respective of x & y direction



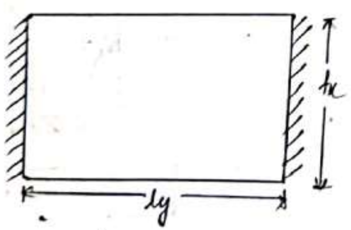
$\frac{l_y}{l_x} \leq 2$

Followings are not the two way slabs.



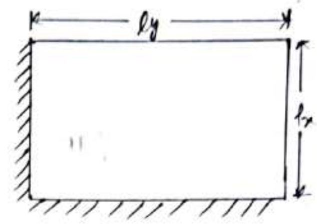
$\frac{l_y}{l_x} < 2.0$

Cantilever slab



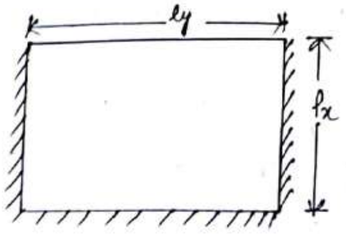
$\frac{l_y}{l_x} < 2.0$

One way slab.



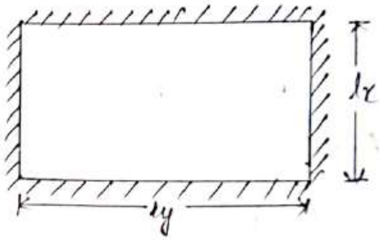
$\frac{l_y}{l_x} < 2.0$

Two end supported slab
or Two adjacent end supt slab



$\frac{l_y}{l_x} < 2.0$

Three end supported slab.



$\frac{l_y}{l_x} > 2.0$

One way slab.

L20Y Q.24

(1) $m = \frac{280}{3\sigma_{cbc}}$ ∴ it depends upon the grade of concrete only.

(2) $\tau_v < \tau_{cmax}$

τ_v → Shear strength of concrete after providing the longitudinal R/f

τ_{cmax} → max. shear strength of concrete after providing the longitudinal R/f as well as shear R/f.

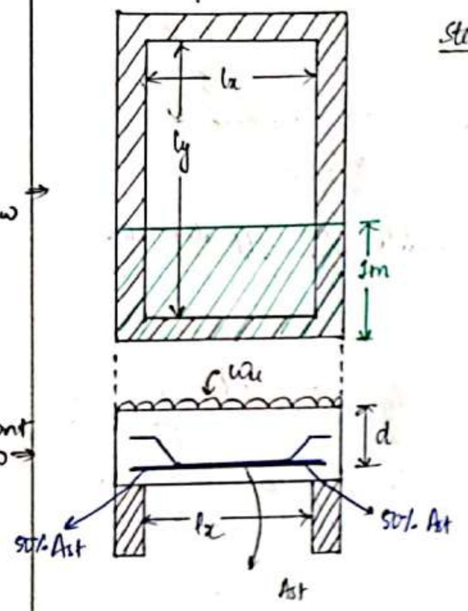
slabs ko hum one one meter ko design karte hain

L-11

Analysis and Design of one way slab

Top view

Front view



Step 1 Estimate Effective Span

$l_{eff} = l_c + \frac{w}{2} + \frac{w}{2}$ or $l_c + d$ } whichever is less.

we don't no the value of d so take

Initially ; $d \geq \frac{span}{A}$ — assumed value (it can be modified further)

$$\boxed{d \geq \frac{l_x}{A}}$$

A → Basic span to Eff depth Ratio

A → 7 (cantilever)
A → 20 (simply supported)
A → 26 (continuous).

Step 2:- Calculation of Loads

Dead load $(W_d) = 25 \times D \times 1$ kN/m
Live load $= 3\ kN/m^2 \times 1 = W_L$ kN/m
Floor finishing $= 24 \times t_f \times 1 = W_f$ kN/m

(kN/m³) 25 → unit wt of RCC
D → gross depth
1 → 1m strip of the slab.
24 kN/m³ → unit wt of concrete.
t_f → floor thickness

Total load ⇒ $W_T = W_d + W_L + W_f$

Factored UDL ; $\boxed{W_u = 1.5 W_T}$

Step 3:- Calculate Bending moment

$M_u = \frac{W_u l_{eff}^2}{8}$ (simply supported) M_u → max B.M.

$M_u = \frac{W_u l_{eff}}{2}$ (cantilever slab)

$M_u = ???$ (continuous slab) → next lecture me

Step 4:- Estimate required depth against above B.M.

$d_{req} = \sqrt{\frac{M_u}{R f_{ck} b}}$

if $d_{ass} < d_{req}$ chng dass & repeat the steps

R → 0.148 [Fe250]
→ 0.138 [Fe415]
→ 0.133 [Fe500].

∵ $d_{assumed\ or\ provided} > d_{required}$ it is safe and OK.

D = d + Effective cover

$$\boxed{D = d + \text{clear cover} + \frac{\phi}{2}}$$

Step 5:- Design of main steel R/f.

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

From the above formula the area of tension steel R/f is being calculated for 1m strip of the slab therefore put b = 1000 mm in the above formula.

Beams me hum no of R/f calcu karte haim.
Slabs me hum spacing of R/f calcu karte haim.

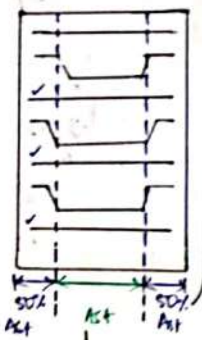
Step 6: Spacing of main steel R/f :-

$$\boxed{\text{Spacing} = \frac{\frac{\pi}{4}\phi^2}{A_{st}} \times 1000}$$ → Per meter me nikal rahe hai isliye x by 1000

Step 7:- Check Maximum Spacing criteria.

Spacing $\leq$ 3d or 300 mm } whichever is less.

Step 8:- Provide main R/f along the shorter span.



Bcz supports p zyada R/f ka requirement nahi hota hai.

Bcz depth is max here.

Step 9:- Distribution R/f along the longer span

Distribution → $(A_{st})_D = \frac{0.15}{100} \times BD \Rightarrow \boxed{(A_{st})_D = \frac{0.15}{100} \times 1000 \times D}$ (Fe 250)

$(A_{st})_D = \frac{0.12}{100} \times BD \Rightarrow \boxed{(A_{st})_D = \frac{0.12}{100} \times 1000 \times D}$ (HYSD/CTD/TMT)

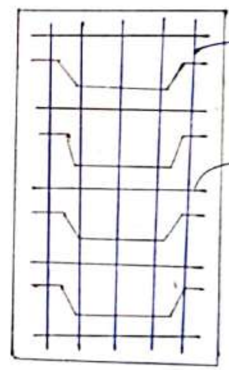
Step 10:- Spacing of distribution R/f

$$\boxed{\text{Spacing} = \frac{\frac{\pi}{4}\phi^2}{(A_{st})_D} \times 1000}$$

Step 11:- Maximum spacing criteria for distribution R/f

Spacing $\leq$ 5 d or 450 mm } which ever is less.

Step 12:- Provide Distribution R/f along the longer span



(shrinkage aur temp. stresses ko lene k liye b dalte haim).

Step 13:- Check for deflection

$$\frac{\text{Span}}{\text{Eff. depth}} \leq A\, f_1 f_2 f_3 f_4$$

$$\boxed{\underset{(d)\ \text{required}}{\text{Eff. depth}} \geq \frac{\text{Span}}{A\, f_1 f_2 f_3 f_4}}$$ → ye required eff depth hai.

If $(d)_{required} < (d)_{provided}$

$\therefore$ Safe & OK.

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Step 14:- Check for Shear

$V_u = \frac{W_u \cdot l_{eff}}{2}$ (Simply Supported slab).

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

Calculate τ_c from Table (19)

near the support bar

% Ast	M20	M25	M30
0.25	✓	✓	✓
0.50	✓	✓	✓
0.60	✓	✓	✓

Total Ast provided in 1st step.

$A_{st} = \left(\frac{1000}{spacing} + 1\right)\frac{\pi}{4}\phi^2$

$\% A_{st} = \frac{A_{st}}{bd} \times 100$

$\boxed{\% A_{st} = \frac{A_{st}}{1000 \times d} \times 100}$

provided at mid span.

∴ calculate τ_c for $\frac{\% A_{st}}{2}$ } 50% of Ast.

if $\tau_v < K\tau_c$ safe

Depth (D) in mm	≥300	275	250	225	200	175	≤150
K	1.0	1.05	1.10	1.15	1.20	1.25	1.3

Step 15:- Check for Bond [OR Check for development length]

$L_d \le \frac{M_1}{V_u} + l_o$ when tension Rlf is not confined by compression Reaction.

$L_d \le \frac{1.3 M_1}{V_u} + l_o$ when tension Rlf is confined by compression reaction.

M_1 & V_u → support p nkalunge...

l_o → Anchorage Value

$= 12\phi$ or d } whichever is greater.

To calculate M_1

$M_1 = 0.36 f_{ck}\, b\, x_u (d - 0.42 x_u)$

or

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

A_{st_1} → Area of tension Rlf provided at the support.

$x_u = \frac{0.87 f_y A_{st_1}}{0.36 f_{ck} (1000)}$

V_u → shear force at a given section where M_1 has been calculated.

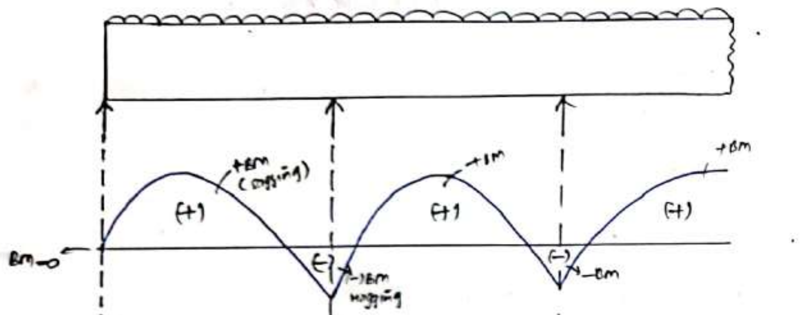
where

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

L12:-

IS code Recommendation :-

Bending Moment and shear force coefficients for a continuous beam or slab.



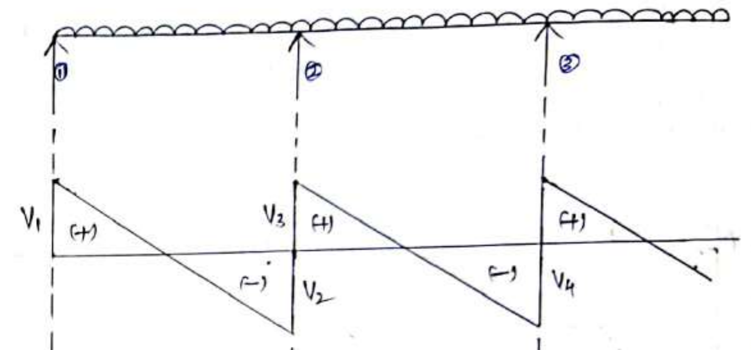
DL	$+\frac{1}{12}$	$-\frac{1}{10}$	$+\frac{1}{16}$	$-\frac{1}{12}$	$\times W_d l_{eff}^2$
LL	$+\frac{1}{10}$	$-\frac{1}{9}$	$+\frac{1}{12}$	$-\frac{1}{9}$	$\times W_L l_{eff}^2$
	more +BM		less +BM		

Maximum positive Bending Moment (Maximum sagging B.M)

$$M_u = \frac{W_d l_{eff}^2}{12} + \frac{W_L l_{eff}^2}{10}$$

Maximum negative B.M (Maximum hogging B.M)

$$M_u = -\left[\frac{W_d l_{eff}^2}{10} + \frac{W_L l_{eff}^2}{9}\right]$$



DL	0.40	0.6	0.55	0.5	$\times W_d l_{eff}$
LL	0.45	0.6	0.60	0.6	$\times W_L l_{eff}$

$$\boxed{V_1 = 0.40\, W_d l_{eff} + 0.45\, W_L l_{eff}}$$

Just left of the support 2

$$\boxed{V_2 = -0.6\, W_d l_{eff} - 0.6\, W_L l_{eff}}$$

Just right of the support 2

$$\boxed{V_3 = 0.55\, W_d l_{eff} + 0.6\, W_L l_{eff}}$$

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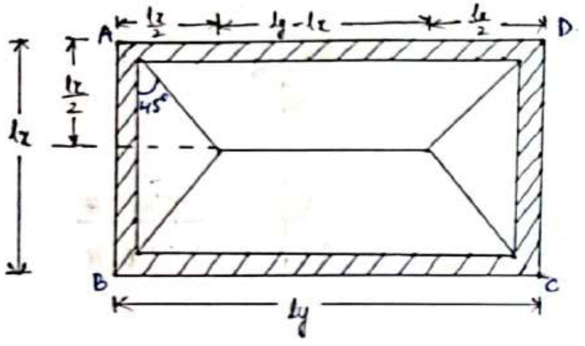
Shorter span par triangular Distribution load ka (Triangle k utna load lega)
Longer span par trapezium Distribution load ka (Trapezium k utna load lega)

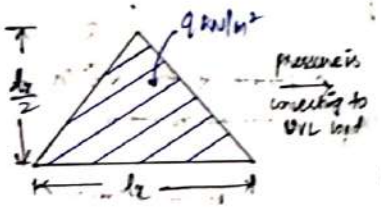
L13:-

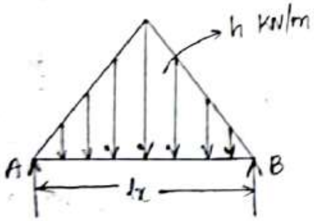
Q Pressure / Load Distribution in Two way Slab

(AB beam k liye ya fir BC beam k liye mujhe kitna UDL k liye ye beams design karna hai mujhe yahi pata karna hai)
(Slab se kitna load beam par ayega uski value calculate karna hai) (UDL load lagega).



Slab is subjected to the uniform load intensity of q kN/m²





UVL is converting to UDL

Total force / load $= \left(\frac{1}{2} \times l_x \times \frac{l_x}{2}\right) q$

$\boxed{\text{Total load} = \frac{q l_x^2}{4}}$ — (i)

Total force / load $= \frac{1}{2} l_x h$

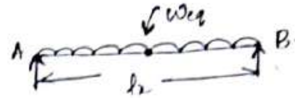
$\boxed{\text{Total load} = \frac{1}{2} h l_x}$ — (ii)

eq (i) = eq (ii)

$\frac{q l_x^2}{4} = \frac{1}{2} h l_x$

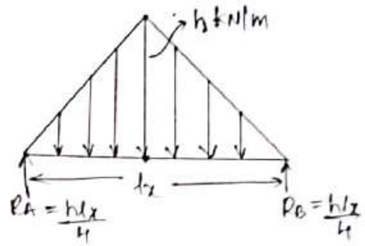
$\boxed{h = \frac{q l_x}{2}}$

→



$\boxed{M = \frac{W_{eq} \cdot l_x^2}{8}}$ — (iii)

Bending moment at mid span under Triangular loading.



$M = R_A \times \frac{l_x}{2} - \frac{1}{2} \times \frac{l_x}{2} \times h \times \left[\frac{1}{3} \times \left(\frac{l_x}{2}\right)\right]$

$M = \frac{h l_x}{4} \times \frac{l_x}{2} - \frac{h l_x^2}{24}$

$M = \frac{h l_x^2}{8} - \frac{h l_x^2}{24}$

$M = \frac{2 h l_x^2}{24}$

$\boxed{M = \frac{h l_x^2}{12}}$ — (iv)

eq (iii) = eq (iv)

$\frac{W_{eq} l_x^2}{8} = \frac{h l_x^2}{12}$

$W_{eq} = \frac{8}{12} h$

$W_{eq} = \frac{8}{12} \times \frac{q l_x}{2} \rightarrow \frac{q l_x}{3}$

For shorter span
Derivation is not imp only
This formula is imp

$\boxed{W_{eq} = \frac{q l_x}{3}}$ → iske sath self wait of the beam b add karna hota

q → pressure on the slab (kN/m²)
due to live load, floor finishing, self wt etc.

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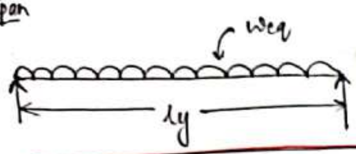
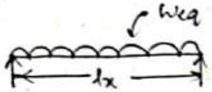
* Longer span



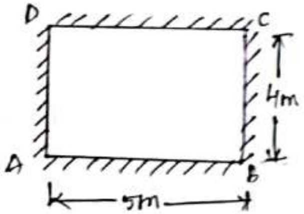
$$\boxed{W_{eq} = \frac{q l_x}{2}\left[1 - \frac{1}{3\left(\frac{l_y}{l_x}\right)^2}\right]}$$

* Shorter span.



$$\boxed{W_{eq} = \frac{q l_x}{3}}$$

Q6 The RC slab, simply supported on all edges as shown in figure below, is subjected to a UDL of $12\ kN/m^2$. The maximum shear force/unit length along the edge 'BC' is (ESE : 2020)



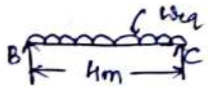
(A) 16 KN
(B) 12 KN
(C) 8 KN
(D) 30 KN

Sol $q = 12\ kN/m^2$

$\frac{l_y}{l_x} < 2$

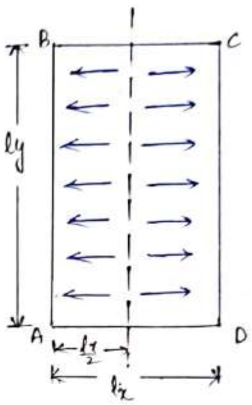
$\frac{5}{4} = 1.25 < 2$

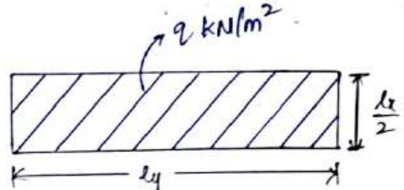
$\therefore$ Two way slab



$W_{eq} = \frac{q l_x}{3} = \frac{12 \times 4}{3} = 16\ KN/m$

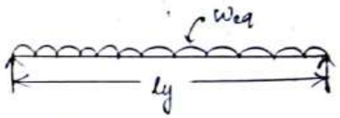
L4:- Load Distribution in one way slab





Total load $= l_y \cdot \frac{l_x}{2} \cdot q$

$\boxed{\text{Total load} = \frac{q\, l_x l_y}{2}}$ — ①



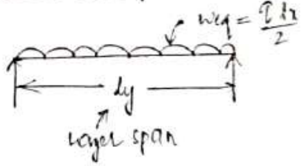
Total load $= W_{eq} \cdot l_y$ — ②

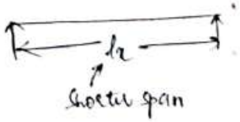
eq ① = eq ②

$\frac{q l_x l_y}{2} = W_{eq} \cdot l_y$

$$\boxed{W_{eq} = \frac{q l_x}{2}}$$

* In one way slab





Note:- In one way slab longer span Beams shall be designed for $\frac{q l_x}{2}$ plus self wt.
But shorter span Beams are designed only for self wt.

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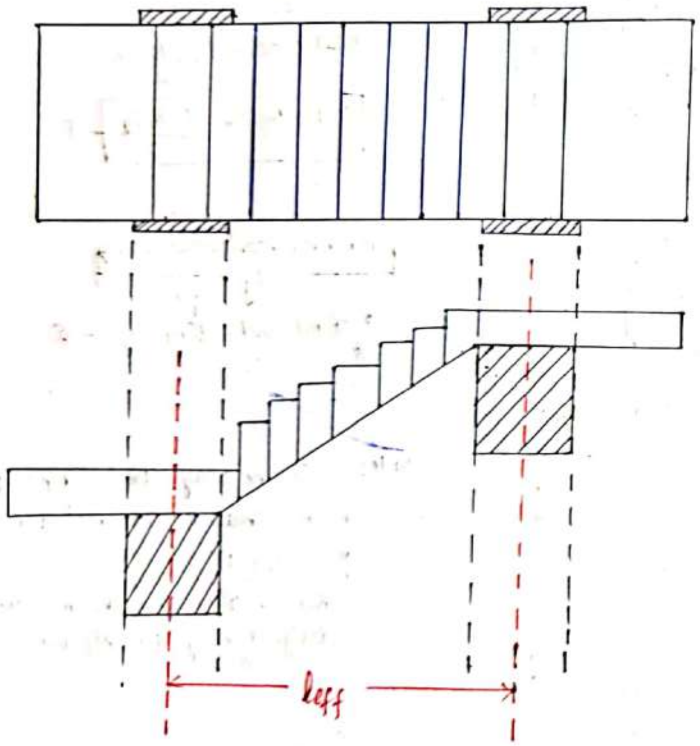
L15:- **Introduction to staircase & it's Effective Span.**

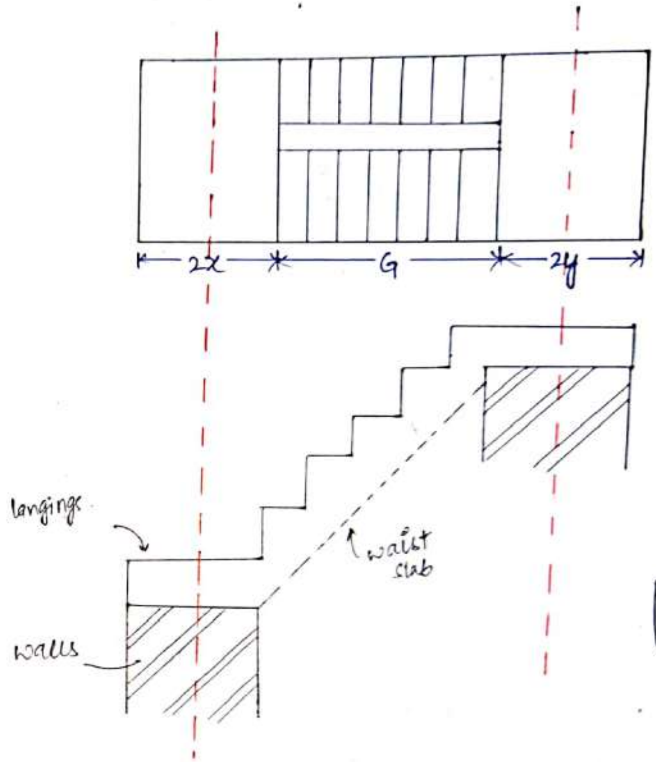
Design of staircase

Effective span :- (a) stair case supported on Beams parallel to risers.



Effective span is taken c/c horizontal distance b/w Beams.

(b) landings are supported on side walls and waist slabs are supported on landings.

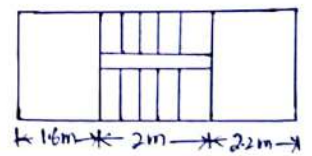


$\boxed{l_{eff} = G + x + y}$

$x \not> 1m$ $\{x_{max} = 1m\}$

$y \not> 1m$ $\{y_{max} = 1m\}$

Eg:-



$l_{eff} = 2 + 0.8 + 1$
$= 3.8\,m$

(c) staircase supported on beam, landing is spanning in same direction of waist slab.

⇒ l_{eff} is taken c/c distance b/w the supports.

L16:-
Q4.

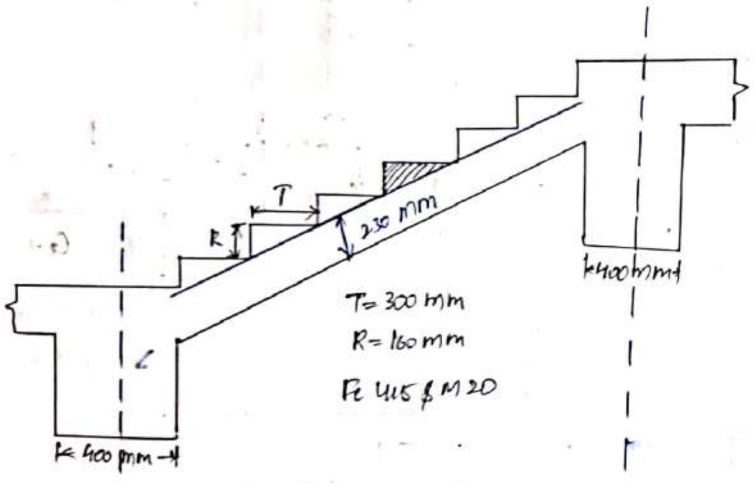
$2x = 1200\ mm$ $\quad 2y = 2200\ mm$

$x = 600\ mm$ $\quad y = 1100\ mm$ WKT $\boxed{y \not> 1m}$

$\therefore y = 1000\ mm$ (taken)

$$l_{eff} = G + x + y$$
$$= 3000 + 600 + 1000$$
$$= 4600\ mm$$

L17:-
Ass(a1)



$l_{eff} = 200 + 300 \times 12 + 200$

$\boxed{l_{eff} = 4000\ mm}$

Step① Calculate the Eff span of the waist slab.

$\boxed{l_{eff} = 4000\ mm}$

Step② Calculate effective depth (Assume)

$$d \geq \frac{span}{A} \Rightarrow \boxed{d \geq \frac{span}{20}}$$

$$d \geq \frac{4000}{20} = 200\ mm$$

Consider effective cover = 30 mm

$D = 200 + 30$

$\boxed{D = 230\ mm}$

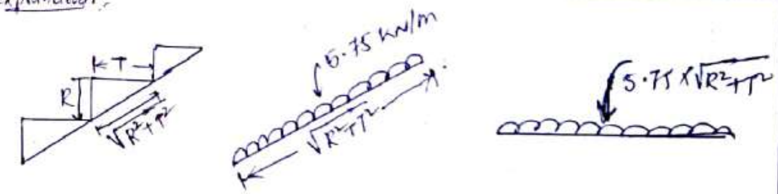
Step③ Load calculation

unit wt of RCC 25 kN/m³ → D → 1 meter k liye

Self wt of the waist slab per meter width of the slab for inclined slab $= 25 \times 0.23 \times 1$

$= 5.75\ kN/m$

Explanation:



Self wt of the waist slab per meter width on the horizontal span $=$ Self wt of slab on inclined span $\times \frac{\sqrt{R^2+T^2}}{T}$

$$= 5.75 \times \frac{\sqrt{160^2+300^2}}{300}$$

$$= 6.52\ kN/m$$

Self wt of waist slab on horizontal span $= 6.52\ kN/m$ → per meter width me nikla hai

Self weight = Area × unit weight

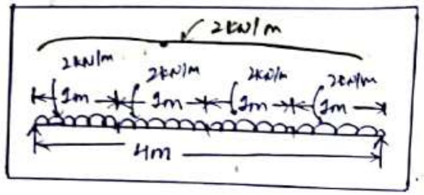
Dead load of steps

Self wt of a single step = $\left[\frac{1}{2} \times T \times R \times 25\right]$

Self wt of the steps in 1m length of the waist slab $= \left[\left(\frac{1000}{T}\right) \times \frac{1}{2} \times T \times R \times 25\right]$

No of steps in 1m

$= 1 \times \frac{1}{2} \times 0.16 \times 25 = 2\,kN/m$



Self wt of floor finishing = 1 kN/m
Live load (in school building) = 5 kN/m

Total load = 6.52 + 2 + 1 + 5
$W = 14.52\,kN/m$

$W_u = 1.5 \times 14.52$

$\boxed{W_u = 21.78\,kN/m}$

Step(4): Calculate Bending Moment

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

$M_u = \frac{21.78 \times 4^2}{8}$

$\boxed{M_u = 43.56\,kN\text{-}m}$

Note: Staircase & Footing me → main Rlf along the longer span dalte hain.
Slab k case me → main Rlf along the shorter span dalte hain.

Step(5): Calculate required Eff. depth

$d_{req} = \sqrt{\frac{M_u}{Q f_{ck} b}}$

$= \sqrt{\frac{43.56 \times 10^6}{0.138 \times 20 \times 1000}}$

$\boxed{d_{req} = 125.63\,mm}$ $\boxed{d_{provided} = 200\,mm}$

$d_{pro} > d_{req}$
Safe & OK.

If $d_{pro} < d_{req}$
Restart from starting. by increasing the value of d

Step(6): Design of main Rlf (along the longer span)

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

$A_{st} = 0.5 \times \frac{20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 43.56 \times 10^6}{20 \times 1000 \times (200)^2}}\right] \times 1000 \times 200$

$\boxed{A_{st} = 646.97\,mm^2}$

Spacing $= \frac{\frac{\pi}{4}(12)^2}{646.97} \times 1000 = 174.6\,mm$

Provide 12mmϕ bars @ 170 mm c/c spacing along the longer span.

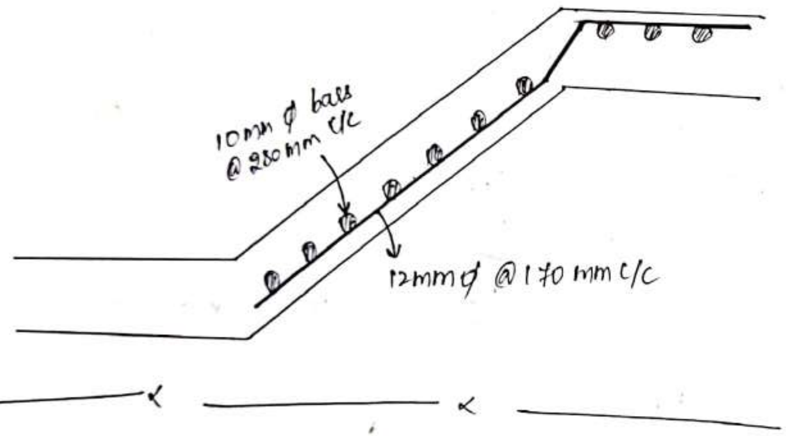
Step(7): Design Distribution Rlf (along the shorter span)

$(A_{st})_D = \frac{0.12}{100} \times 1000 \times 230$

$\boxed{(A_{st})_D = 276\,mm^2}$

$$\text{Spacing} = \frac{\frac{\pi}{4}(10)^2}{276} \times 1000 = 284.4 \text{ mm}$$

provide 10 mmϕ bars @ 280 mm c/c spacing along the shorter span.



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Vertical members are compression members.

CHAPTER : 5

LIMIT STATE METHOD OF COLLAPSE — COMPRESSION

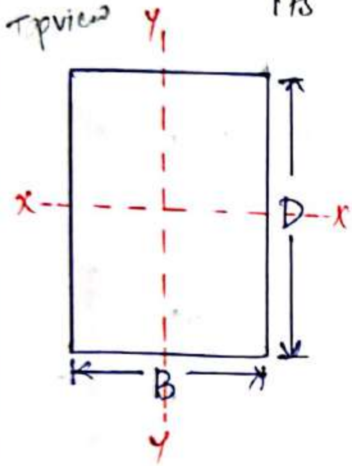
V.V.I

(aise members jinke ander compressive force ate hain aur unko kaise design karen).

Important Definitions & IS code Recommendations.

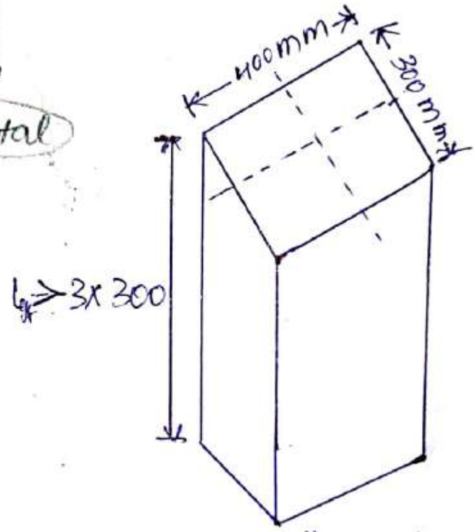
(on design basis)

① **column** :- It is the compression member the effective length of which is greater than three times of its least lateral dimensions.



$\boxed{l_{eff} > 3B}$

otherwise pedestal



column

② **Slenderness Ratio** (λ) :- It is defined as the Ratio of effective length of the least lateral dimension.

$$\boxed{\lambda = \frac{l_{eff}}{B}} \qquad \lambda \propto \frac{1}{B}$$

$$\lambda_x = \frac{l_{ex}}{D}$$

$$\lambda_y = \frac{l_{ey}}{B}$$

if $\lambda \uparrow$ → chances of Buckling $\uparrow$

if $\lambda \downarrow$ → chances of Buckling $\downarrow$

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Point of contraflexure or Inflection → where B.M = 0

③ Short column & long column :- If the Selenderness Ratio is less than 12 then It is called Short column or else long column.

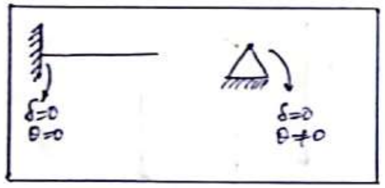
$\left.\begin{array}{l}\lambda_x < 12 \\ \lambda_y < 12\end{array}\right\}$ short column

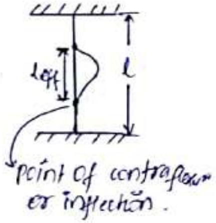
where $\lambda_x = \frac{l_{ex}}{D}$

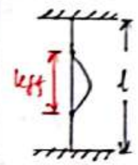
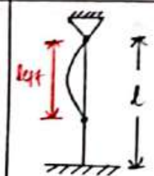
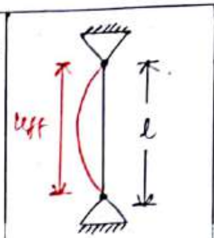
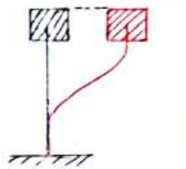
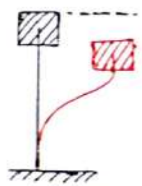
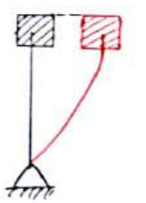
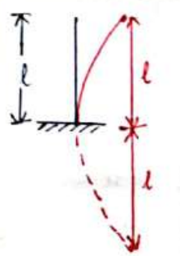
$\lambda_y = \frac{l_{ey}}{B}$

$\left.\begin{array}{l}\lambda_x > 12 \\ \lambda_y > 12\end{array}\right\}$ long column

④ Effective length :- It is the distance /height b/w two points of Inflection in the column.

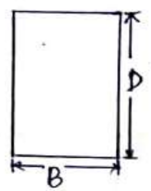




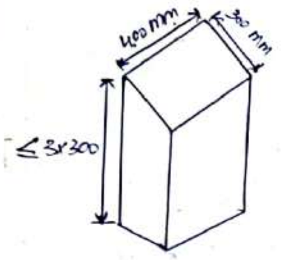
Degree of Restraints	Symbol	Theoretical value of Eff. length	[IS 456 : 2000] Design value of Eff length.
Effectively held in position & restraint against rotation		0.50l $\left(\frac{1}{2}\right)$	0.65 l
Effectively held in position at both ends restrained against rotation at one end		0.70l $\left(\frac{1}{\sqrt{2}}\right)$	0.80 l
Effectively held in position at both ends, but not restrained against rotation.		1.00 l	1.00 l
Effectively held in position & restrained against rotation at one end, and at the other end restrained against rotation but not held in position.		1.00 l	1.20 l
Effectively held in position and restrained against rotation in one end, and at the other partially restrained against rotation but not held in position		-	1.50 l
Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position.		2.00 l	2.00 l
Effectively held in position & restrained against rotation at one end but not held in position nor restrained against rotation at the other end.		2.00 l	2.00 l

leff here is → Jo load carry karta hai actually uski length hai.
(This is the length available to carry actual load).

⑤ **Pedestal** :- It is the compression member, the eff length Of which is less than three times of its least lateral dimension.



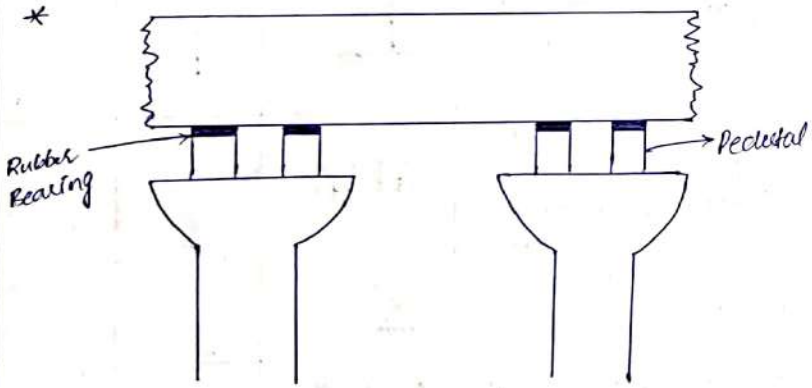
$\boxed{l_{eff} \leq 3B}$



⇓
Pedestal.

Extra :- Flyover

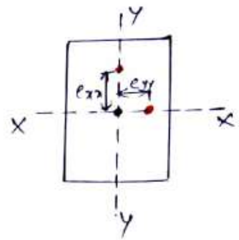
*



* Footing

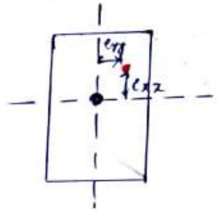
L2:- **Minimum Eccentricity** (e_{min}) :- [IS 456: 2000]

Explanation:- Additional BM will developed in the column bcz of eccentricity. (shifting of load) (if load is shifted).



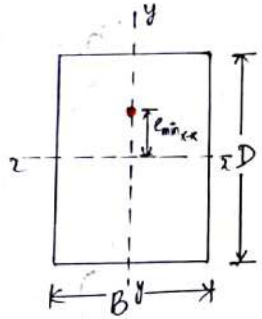
$M_{yy} = Pe_{yy}$

$M_{xx} = Pe_{xx}$



$$e_{min} = \left.\begin{array}{l} \dfrac{\text{unsupported length}}{500} + \dfrac{\text{lateral Dimension}}{30} \\ \text{or} \\ 20\ mm \end{array}\right\} \text{whichever is maximum.}$$

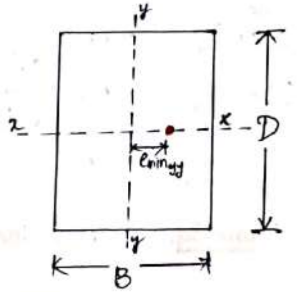
$$e_{min_{xx}} = \left.\begin{array}{l} \dfrac{l_0}{500} + \dfrac{D}{30} \\ \text{or} \\ 20\ mm \end{array}\right\} \text{whichever is maximum.}$$



$\boxed{(M_{uxx})_{min} = P_u \times e_{min_{xx}}}$

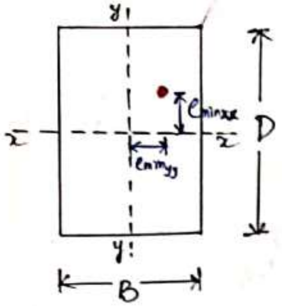
the moment at value column me hona hi chahiye. (Resist karenk liye)

$$e_{min_{yy}} = \left.\begin{array}{l} \dfrac{l_0}{500} + \dfrac{B}{30} \\ \text{or} \\ 20\ mm \end{array}\right\} \text{whichever is maximum}$$



$\boxed{(M_{uyy})_{min} = P_u \times e_{min_{yy}}}$

*

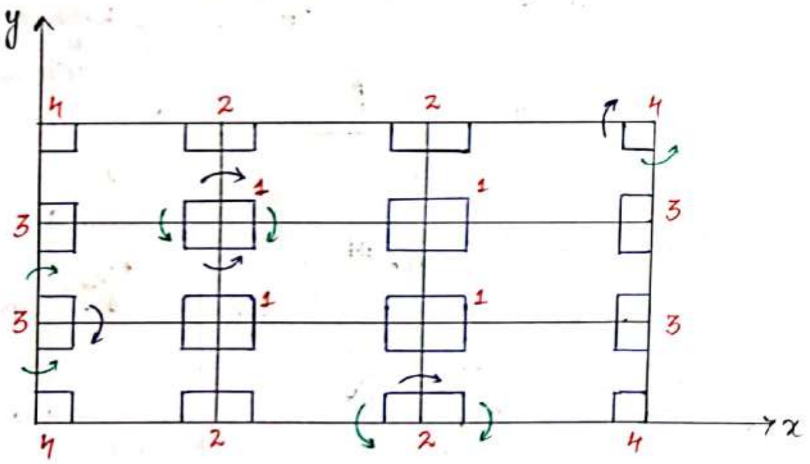


$$e_{minxx} = \left.\begin{matrix}\frac{l_o}{500} + \frac{D}{30} \\ \text{or} \\ 20\text{ mm}\end{matrix}\right\}_{max} \qquad \boxed{M_{uxx\,min} = P_u \cdot e_{minxx}}$$

$$e_{minyy} = \left.\begin{matrix}\frac{l_o}{500} + \frac{B}{30} \\ \text{or} \\ 20\text{ min}\end{matrix}\right\}_{max} \qquad \boxed{M_{uyy\,min} = P_u \cdot e_{minyy}}$$

$P_u \rightarrow$ axial load carrying capacity of column.

L5:- Different Types of column Design :-



Column Type 1 :- Axially loaded columns (P_u) ⇒ (sirf axial load k liye design karte hain)

Column Type 2 :- Axial load & uniaxial Bending (P_u, M_{ux})

Column Type 3 :- Axial load & Uniaxial Bending (P_u, M_{uy})

Column Type 4 :- Axial load & Biaxial Bending (P_u, M_{ux} & M_{uy}) .

(These columns may be short or long)

Note:- In columns R/f are provided to take compression (not tension).
In columns NA will be out of the column.

L6:- Assumptions in limit state of collapse under the compression condition.

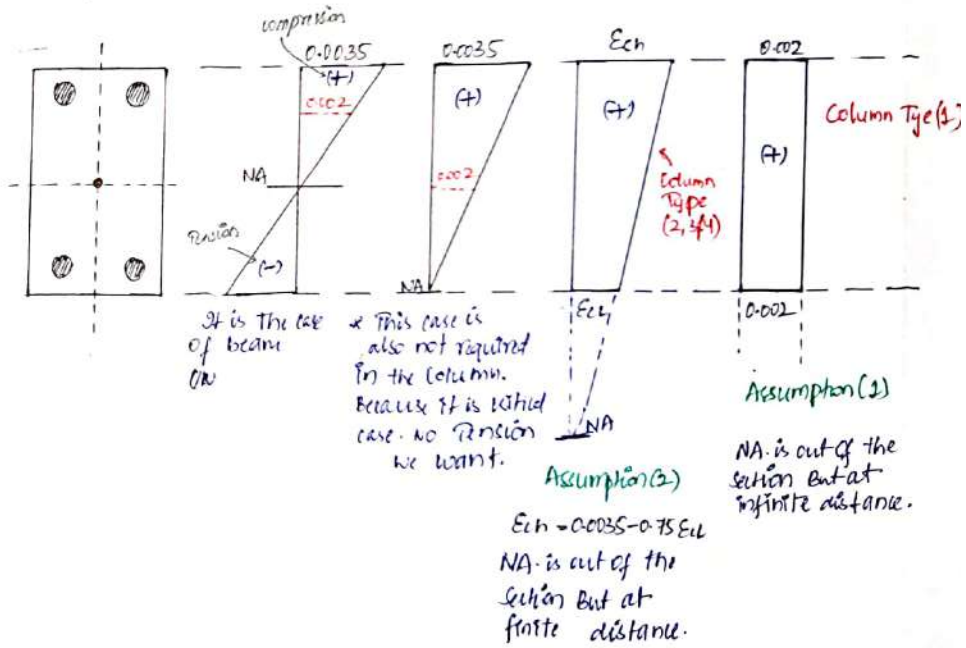
Assumptions :-

1) The maximum compressive strain in axial compression can be taken as 0.002.

2) The maximum compressive strain in axial compression & bending but No tension, can be taken as 0.0035 minus 0.75 times of the strain at the least compressed fibre.

$$\boxed{\varepsilon_{ch} = 0.0035 - 0.75\,\varepsilon_{cL}}$$

Where $\varepsilon_{ch} \rightarrow$ compressive strain at the highly compressed fibre.
$\varepsilon_{cL} \rightarrow$ compressive strain at the least compressed fibre.



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Note:- * The maximum compression strain in concrete in axial compression is 0.002

~~0.0035~~ ~~0.002~~

* The maximum compression strain in concrete in flexure/Bending is 0.0035

L5:-

Q1) (B)

Q2) (A)

Q3) (C)

Q4) Effective length is independent of loads (Any type of loads it may be any member).

(B)

C.G → centre of gravity.

L6:- Analysis & Design of Axially loaded columns [Column Type 1]

[Short column]

$\boxed{P_u = \text{load taken by concrete} + \text{load taken by steel}}$

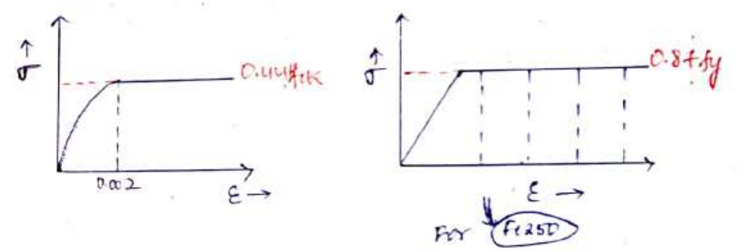
The maximum compressive strain in axial compression can be taken as 0.002

$\boxed{P_u = \sigma_c A_c + \sigma_{st} A_{sc}}$

$\sigma = \frac{P}{A} \Rightarrow P = \sigma \cdot A$

σ_c → stress in concrete.
A_c → Area of concrete.
σ_{st} → stress in steel.
A_{sc} → Area of compression steel.

Explanation:-



* The stress in concrete at 0.002 strain is 0.446 fck
* The stress in steel at 0.002 strain can be given as,

at 0.002 strain, $\sigma_{st} = 0.87 f_y$ [Fe 250]
at 0.002 strain, $\sigma_{st} = 0.79 f_y$ [Fe 415] } HYSD
at 0.002 strain, $\sigma_{st} = 0.75 f_y$ [Fe 500] }

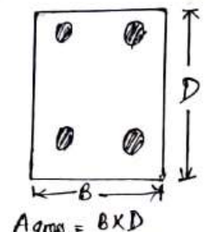
ye dono k liye σ_{st}

$\boxed{\sigma_{st} = 0.75 f_y}$

$P_u = 0.446 f_{ck} [A_g - A_{sc}] + 0.75 f_y A_{sc}$

$\boxed{P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}}$ — (1)

This formula is used when pure axial compression load is acting axially at the C.G of the column. [Or column me resist karne ki capacity tabhi hoti hai jab load exactly C.G ke paas hota hai].



$A_{gross} = B \times D$
$A_c = A_g - A_{sc}$

Note: Jab b apne column me moment carrying capacity badhana hai to load carrying capacity kam karna hota hai.

* But code ne [IS 456 2000] ne mujhe kaha hai ki sabhi columns ko minimum eccentricity ka effect karte hue design karna hai.

* site par during the casting improper mixing k karan, improper compaction k karan load shift ho bi jata hai, to is shifting of load k karan hamare member (column) me additional moment develop hojate hain. Jiske karan load carrying capacity column ki kam hojata hai.

* Considering effect of minimum eccentricity, 10% load is reduced from the eq①, That means 90% load carrying capacity is being considered.

$$P_u = 0.9\,(P_{u*}) = 0.9\,(0.45 f_{ck} A_c + 0.75 f_y A_{sc})$$

$$\boxed{P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}} \quad \text{—②}$$

If you design your column by eq① Then it is capable to allow carry only pure axial compression capacity
[if the loads shift by e (minimum eccentricity the column will fail]

But if you design your column by eq② Then it is capable to carry not only pure axial compression capacity but also minimum moment carrying capacity.

By eq① :- Pure axial compression capacity.

By eq② :- Pure axial compression capacity + Minimum moment carrying capacity.

*** if $e_{min} \le 0.05\,(D \text{ or } B)$. use eq②

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \quad \text{—②}$$

eq② considers the effect of minimum eccentricity. If column is designed using eq② that means column has minimum moment carrying capacity.

$$M_{uxx_{min}} = P_u\, e_{xx\,min}$$

$$M_{uyy_{min}} = P_u\, e_{yy\,min}$$

$$e_{min} = \left.\begin{array}{l} \dfrac{l_0}{500} + \dfrac{D \text{ or } B}{30} \\ \text{or} \\ 20\,mm \end{array}\right\} \max$$

*** if $e_{min} \nleq 0.05\,(D \text{ or } B)$ use eq①

$$P_u = 0.45 f_{ck} A_c + 0.75 f_y A_{sc} \quad \text{—①}$$

eq① doesnot consider the effect of minimum eccentricity. It shows only the pure axial load carrying capacity of the column.

Extra :- For short column

$\frac{l_{eff}}{D} < 12$ — $e_{min} \le \frac{12D}{500} + \frac{D}{30}$

$l_{eff} < 12D$ — $e_{min} \le 0.0573\,D$

$l < 12D$ — $\boxed{e_{min} \le 0.05D}$ ← [yahan ye ayi e_{min} ki condition]

(By seeing the Dimensions of column I can find out which formula to use)

$e_{min} \le 0.05\,D$

$20 \le 0.05D$

$\boxed{D \ge 400\,mm}$

If B or D ≥ 400 mm then eq② can be used.
(otherwise eq①)

L7:- **IS code Recommendations :- [IS 456:2000]**

1) The minimum 0.8% area of steel R/f shall be provided

$$\boxed{Asc_{min} = \frac{0.8}{100} \times B \times D.}$$ Provided to have minimum ductility.

2) The maximum area of steel R/f shall be provided as

= 6% of gross area [When bars are not overlapped]

$$\boxed{Asc_{max} = \frac{6}{100} \times B \times D.}$$

= 4% of gross area [When bars are overlapped]

$$\boxed{Asc_{max} = \frac{4}{100} \times B \times D}$$

3) The minimum 4 bars in Rectangular/square and 6 bars in circular column shall be provided.

4) The maximum spacing b/w the longitudinal R/f shall not exceed 300 mm along the periphery.

5) Minimum 12mm ϕ of bars shall be used.

6) Minimum 40mm nominal clear cover shall be provided.

7) Minimum 25 mm nominal clear cover can be used when diameter of bar is restricted to 12mm in the columns of sizes upto 200 mm.

8) The Area of steel R/f is not governed in strength calculation of the pedestal ($l_{eff} \le 3B$) then also minimum 0.15% area of steel of total gross area shall be provided.

9) The diameter of Transverse R/f shall not be less than

(i) 6 mm
(ii) $\frac{1}{4}^{th}$ of the ~~least~~ largest diameter of the bar

} max.

Transverse R/f are provided to keep longitudinal R/f straight (not to take load.) (0.1% ka b load uspe nahi lage ga)

10) The maximum spacing of Transverse R/f shall be

(i) 16ϕ [$\phi \rightarrow$ Diameter of longitudinal R/f].
(ii) Least lateral dimension of the column.
(iii) 300 mm.

11) The minimum spacing b/w two longitudinal R/fs (in beams, columns, footings, slabs) shall be maximum of the followings.

(i) ϕ [when equal diameter bars are provided]
(ii) ϕ_{max} [when unequal diameter bars are provided]
(iii) 5mm + Nominal size of Aggregate.

[Where $\phi \rightarrow$ diameter of main/longitudinal R/fs].

18:-

Q5) 300 mm x 300 mm

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

$A_{sc} = 4 \times 314 \quad (\frac{\pi}{4} \times 20^2 = 314)$

$f_y = 415\ N/mm^2$

$P_u = 0.45 f_{ck} (A_g - \xrightarrow{\text{Ignoring}} \cancel{A_{sc}}) + 0.75 f_y A_{sc}$ —①

$P_u = 0.4 f_{ck} (A_g - A_{sc}) + 0.67 f_y A_{sc}$ —②

If $e_{min} \le 0.05D$ (use eq② otherwise eq①)

WKT $e_{min} = \left.\begin{matrix} \frac{l_0}{500} + \frac{B}{30} \\ \text{or} \\ 20\ mm \end{matrix}\right\} max$

(In Que the value of l_0 is not given so take 20 mm)

$e_{min} = 20\ mm$

$20 \le 0.05 \times 300$

$20\ mm \nleq 15\ mm$

$P_u = 0.45 f_{ck} A_g + 0.75 f_y A_{sc}$

$P_u = 0.45 \times 20 \times [300 \times 300] + 0.75 \times 415 \times [4 \times 314]$

$P_u = 1200.93\ kN \approx 1198\ kN$

Q6) $B = 250\ mm$

$D = 400\ mm$

$A_{sc} = 5 \times \frac{\pi}{4} \times 20^2 = 5 \times 314$

· M30 & Fe500

WKT for minimum eccentricity

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$ where $(A_c = A_g - A_{sc})$

(Here in this Que it is not mention 'Ignoring the reduction in the area of concrete due to steel R/f.')

∴ you should take $A_g - A_{sc}$

$P_u = 0.4 f_{ck} (A_g - A_{sc}) + 0.67 f_y A_{sc}$

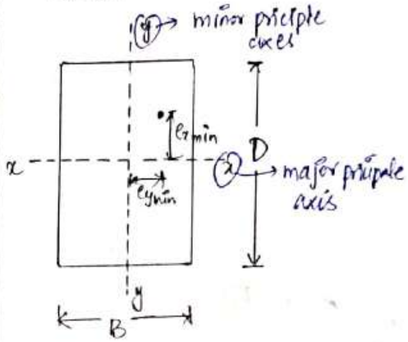
$P_u = 0.4 \times 30 \times [250 \times 400 - 5 \times 314] + 0.67 \times 500 \times (5 \times 314)$

$P_u = \underline{1707.11}$.

Q7) $B = 450\ mm$

$D = 600\ mm$

$l_0 = 3.0\ m$



$e_{x min} = \left.\begin{matrix} \frac{l_0}{500} + \frac{D}{30} \Rightarrow \frac{3000}{500} + \frac{600}{30} \Rightarrow 26 \\ \text{or} \\ 20\ mm \end{matrix}\right\} max$

$\boxed{e_{x min} = 26\ mm}$

$e_{y min} = \left.\begin{matrix} \frac{l_0}{500} + \frac{B}{30} \Rightarrow \frac{3000}{500} + \frac{450}{30} \Rightarrow 21\ mm \\ \text{or} \\ 20\ mm \end{matrix}\right\} max$

$\boxed{e_{y min} = 21\ mm}$

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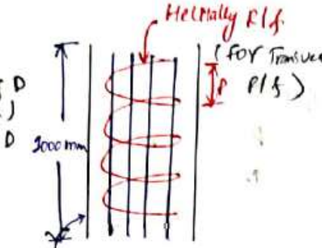
A ko 5% kam karna hai to → $0.05 \times A$
A ko 5% badhana hai to → $1.05 \times A$.

19:- ## Analysis & Design of Helically Reinforced column :-

Due to helical R/f axial load carrying capacity of column is increased by 5%.

** If $e_{min} < 0.05$ (B or D) → circular column me sirf D hota hai (B nahi hota) so take D

$$\boxed{P_u = 1.05\,[0.4 f_{ck} A_c + 0.67 f_y A_{sc}]}$$



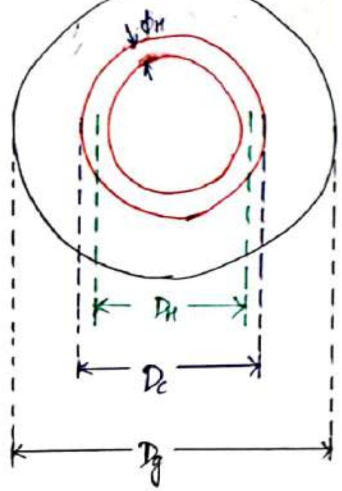
P → Pitch → c/c spacing of helical R/f

In helically Reinforced column

$$0.36\,\frac{f_{ck}}{f_y}\left[\frac{A_g}{A_c}-1\right] \le \frac{V_H}{V_c}$$

→ If you satisfies this condition the design of helically R/f column is correct.

Top view



ϕ_H → diameter of helical ring/R/f

D_g → gross diameter

$A_g \to \frac{\pi}{4}(D_g)^2$ → gross area of column

D_c → Diameter of core

$A_c = \frac{\pi}{4}(D_c)^2$ → Area of core

V_c → Volume of core

V_c = Area of core × unit length of column

$V_c = A_c \times (1000)\ mm^3$

V_H → Volume of helical R/f in the same height/length of Volume of core.

V_H = No of Turns × perimeter of helical Ring × Area of helical R/f.

$$\boxed{V_H = \frac{1000}{P} \times (\pi D_H) \times \frac{\pi}{4}\phi_H^2}$$ where $D_H = D_c - \frac{\phi_H}{2} - \frac{\phi_H}{2}$.

Recommendations for Pitch (P).

(i) $P \not> 75$ mm
(ii) $P \not< 25$ mm
(iii) $P \not< 3\phi_H$
(iv) $P \not> \frac{1}{6} D_c$

Recommendations for ϕ_H.

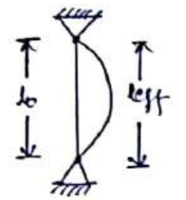
(i) 6 mm
(ii) $\frac{1}{4}$ × largest diameter of longitudinal bars

} max.

L10: (Ass Q2)

Design a helically Rlf circular column
$P = 1500\,kN$ (working load)
$D_g = 450$ mm
Unsupported length = 3.5 mm
Both ends hinged. use M25, Fe 415

Sol:



$l_{eff} = 1.0\,l_o$

$\boxed{l_{eff} = 3.5\,m}$

Step 1: Estimate λ

$$\lambda = \frac{l_{eff}}{450} = \frac{3500}{450} = 7.78$$

$\lambda < 12$ $\therefore$ it is a short column.

Step 2: Estimate e_{min}

$$e_{min} = \frac{l_o}{500} + \frac{D}{30} \Rightarrow \frac{3500}{500} + \frac{450}{30} \Rightarrow 22\,mm \text{ or } 20\,mm \Big\} \max$$

$\boxed{e_{min} = 22\,mm}$

Step 3: Estimate Axial load carrying capacity

$e_{min} = 22$ mm

$0.05\,D = 0.05 \times 450 = 22.5$ mm

$e_{min} \leq 0.05\,D$

$P_u = 1.05\,[0.4 f_{ck} A_c + 0.67 f_y A_{sc}]$

$\underbrace{(1.5 \times 1500)}$ $= 1.05\,[0.4 \times 25 \times (A_g - A_{sc}) + 0.67 \times 415 \times A_{sc}]$

↳ working load is converted into ultimate load by × FOS (1.5)

Gross diameter (D_g) matches diameter of column.

$$2250 \times 10^3 = 1.05\,[0.4 \times 25 \times (\tfrac{\pi}{4} \times 450^2 - A_{sc}) + 0.67 \times 415 \times A_{sc}].$$

$\boxed{A_{sc} = 2063.9\,mm^2}$

Number of bars of 16 mmϕ

$$N = \frac{A_{sc}}{\frac{\pi}{4}(16)^2} = \frac{2063.9}{201} = 10.3$$

$\boxed{N = 11}$

$$0.36\,\frac{f_{ck}}{f_y}\left[\frac{A_g}{A_c} - 1\right] \leq \frac{V_H}{V_c} \quad \text{—①}$$

$A_g = \frac{\pi}{4}(D_g^2) = \frac{\pi}{4}(450^2) = 159043.1\,mm^2$

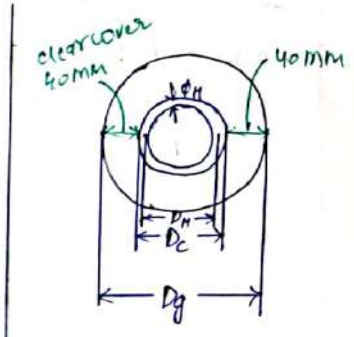
$A_c = \frac{\pi}{4}(D_c^2) = \frac{\pi}{4}(370)^2 = 107521\,mm^2$

$V_c = A_c \times \overset{\text{a 1m}}{1000} = 107.521 \times 10^6\,mm^3$

$V_H = \frac{1000}{P} \times \pi D_H \times \frac{\pi}{4}(\phi_H)^2$

$V_H = \frac{1000}{P} \times (\pi \times 362) \times \frac{\pi}{4} \times 8^2$

$V_H = \frac{57.165 \times 10^6}{P}$



$D_c = D_g - (2 \times 40)$

$D_c = 450 - (80)$

$\boxed{D_c = 370\,mm}$

Let $\phi_h = 8\,mm$

$D_H = D_c - \frac{\phi_H}{2} - \frac{\phi_H}{2}$

$D_H = 370 - 8$

$\boxed{D_H = 362\,mm}$

Putting all values in eq①

$$0.36 \times \frac{25}{415} \times \left[\frac{159043.1}{107521} - 1\right] \leq \frac{57.165 \times 10^6}{P \times 107.521 \times 10^6}$$

$\boxed{P \leq 51.16\,mm}$

Providing 8mmϕ bar helical Rlf at 50 mm c/c.

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Check: $P \not> 75$ mm ✓

$P \not< 25$ mm ✓

$P \not< 3\phi_H = 3 \times 8 = 24$ mm ✓

$P \not> \frac{1}{6} D_c = \frac{1}{6} \times 370 = 61.6$ mm ✓

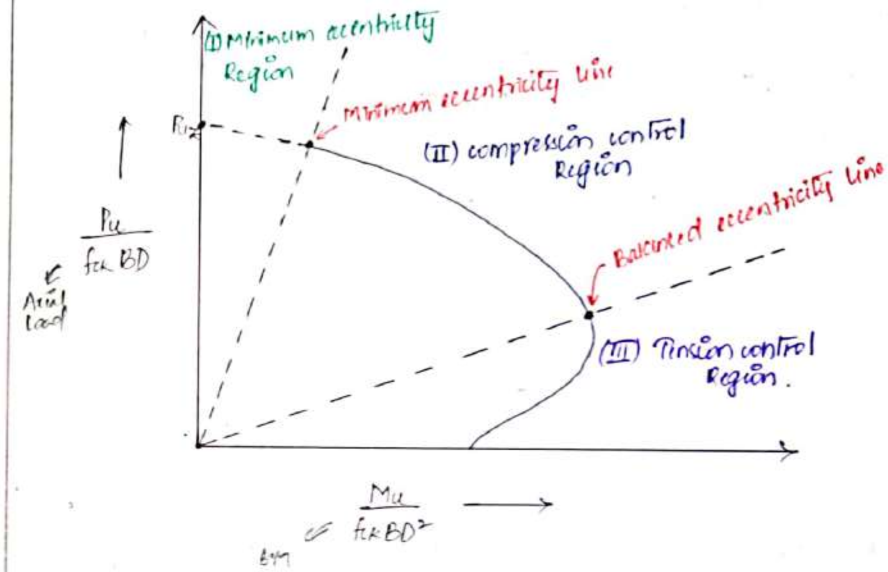
Result:

→ 11 or 12 # Bars 16 mm ϕ → Longitudinal R/f

→ 8 mm ϕ helical R/f with 50 mm pitch c/c.

L11:- **Column Interaction Diagram :- [SP 16].**

(aise diagrams jo apke axial load or BM k behaviour ko show karte hai) (Jab dono load ek sath lage ho column p).



(I) Minimum eccentricity Region :-

(i) In this region those columns are designed in which dominating force is axial load.

[That means column Type 1]

(ii) If columns are designed in this region then columns will have minimum moment carrying capacity.

(iii) $$e_{min} = \left.\begin{array}{l} \frac{l_0}{500} + \frac{B \text{ or } D}{30} \\ \text{or} \\ 20 \text{ mm} \end{array}\right\} \max$$

(iv) The maximum compressive strain in this region is 0.002.

[Assumption. 1]

Bending k karan Tension aata hai
(agar limit me BM ayega to tension nahi hoga)

In this region; If $e_{min} \le 0.05$ D or B

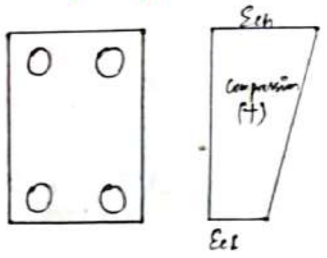
$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

(II) Compression control Region:-

(i) In this region, those columns are designed in which Axial load & Bending moment both are dominating. [column Type 2, 3 & 4]

[Bending may be uniaxial or Biaxial]

(ii)



$$\varepsilon_{ch} = 0.0035 - 0.75\, \varepsilon_{cl}$$

(iii) No Tension occurs in this Region due to bending therefore it is called compression control Region.

(iv) On decreasing the axial load on the column moment carrying capacity increases in this region.

(III) Tension control Region:-

(i) The dominating force is bending moment which causes tension in the section, Therefore columns are not designed in this Region.

(ii) In this region, if axial load is decreased then moment carrying capacity also starts decreasing.

L-2:- Design of columns subjected to Axial load & Uniaxial Bending:-

[column Type 2]

[Given $\underbrace{B, D, B', D', f_{ck}, f_y}_{\text{you can assume this if not given}}$, P_u, M_{ux}, M_{uy}]

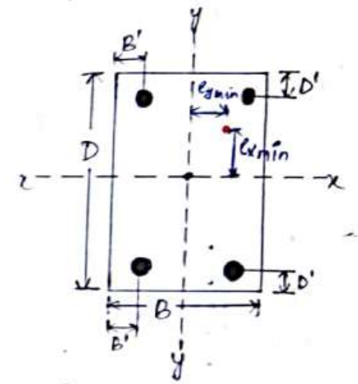
Step 1:→ Estimate Slenderness Ratio

$$\boxed{\lambda = \frac{l_{eff}}{B}}$$ If $\lambda < 12$ [Short column]

Step 2:→ Estimate Minimum eccentricity

$$e_{x\,min} = \left.\begin{matrix} \frac{l_o}{500} + \frac{D}{30} \\ \text{or} \\ 20\text{ mm} \end{matrix}\right\} \max$$

$$e_{y\,min} = \left.\begin{matrix} \frac{l_o}{500} + \frac{B}{30} \\ \text{or} \\ 20\text{ mm} \end{matrix}\right\} \max.$$



Step 3:→ Minimum moment carrying capacity

$$\boxed{M_{ux\,min} = P_u \cdot e_{x\,min}}$$

$$\boxed{M_{uy\,min} = P_u \cdot e_{y\,min}}$$

Step 4:→ Compare M_{ux}, M_{uy} & $M_{ux\,min}$, $M_{uy\,min}$

If $\left.\begin{matrix} M_{ux} < M_{ux\,min} \\ M_{uy} < M_{uy\,min} \end{matrix}\right\}$ Design for axial load only.

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$ } Design using this eqn.

If $M_{ux} > M_{ux\,min}$
$M_{uy} < M_{uy\,min}$ } Design for axial load & uniaxial moment M_{uz}.

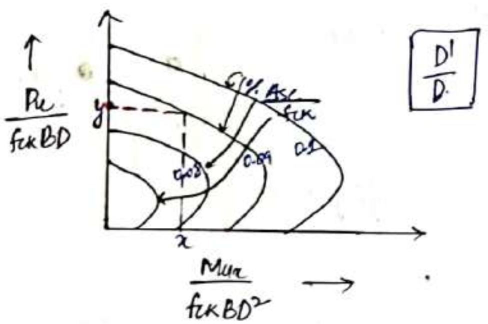
Step 5:→ Estimate the followings:

(i) $\frac{D'}{D}$

(ii) $\frac{P_u}{f_{ck} BD} = y$

(iii) $\frac{M_{ux}}{f_{ck} BD^2} = x$

Step 6 :→ Choose a column interaction diagram from the SP 16, Based on the Ratio of $\frac{D'}{D}$.



$\boxed{\frac{D'}{D}}$

from the above curve Estimate $\frac{\% Asc}{f_{ck}} = K$

$\% Asc = K f_{ck}$

$\% Asc = \frac{Asc}{BD} \times 100$

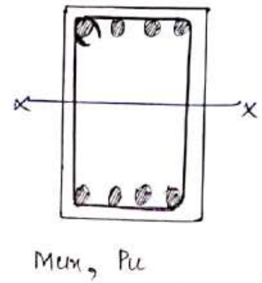
$$\boxed{Asc = \frac{K \cdot f_{ck} \times BD}{100}}$$

Minimum Diameter of bar provided in column is 12 mm.
Minimum clearcover for column is 40 mm

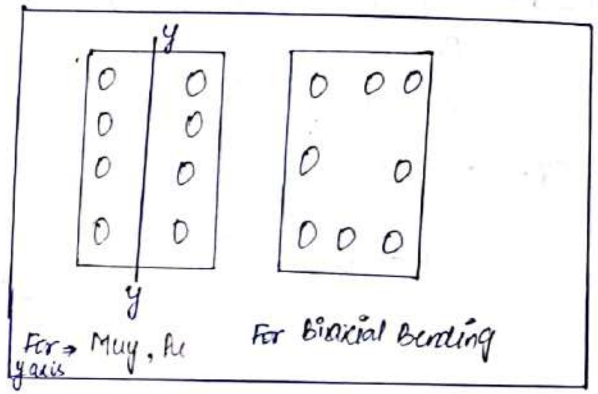
Step 7:→ Calculate Total Number of bars.

$$\boxed{N = \frac{Asc}{\frac{\pi}{4}(\phi)^2}}$$ Let 8 bars

Step 8:→ Detailing of Reinforcement



M_{ux}, P_u



Step 9:→ Provide stirrups.

Spacing $\ngtr 16\phi$
$\ngtr B$
$\ngtr 300mm$ } min.

Stirrups/Rings → $\frac{1}{4} \times \phi_{max}$ or 6 mm } max.

(y axis ka apne aap padhlo se nahi bataye hain)
ie column Type 3

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L13:- Design of columns subjected to Axial load and Biaxial Bending:-

[Column Type 4]

[Given $\underbrace{B, D, B', D', f_{ck}, f_y}_{\text{you can also assume this if not given}}$, $\underbrace{P_u, M_{ux}, M_{uy}}_{\text{by structural analysis (ex. Stad pro etc)}}$]

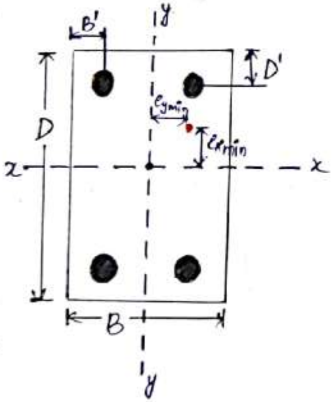
Step(1):→ Estimate Slenderness Ratio.

$$\boxed{\lambda = \frac{l_{eff}}{B}} \; ; \quad \lambda < 12 \text{ [Short column]}$$

Step 2:→ Estimate Minimum eccentricity

$$e_{x\,min} = \left.\begin{array}{l} \dfrac{l_o}{500} + \dfrac{D}{30} \\ \text{or} \\ 20 \text{ mm} \end{array}\right\} \max.$$

$$e_{y\,min} = \left.\begin{array}{l} \dfrac{l_o}{500} + \dfrac{B}{30} \\ \text{or} \\ 20 \text{ mm} \end{array}\right\} \max.$$



Step3:→ Estimate minimum moment carrying capacity

$$\boxed{M_{ux\,min} = P_u \cdot e_{x\,min}}$$

$$\boxed{M_{uy\,min} = P_u \cdot e_{y\,min}}$$

Step4:→ Compare M_{ux}, M_{uy} & $M_{ux\,min}$, $M_{uy\,min}$

if $M_{ux} > M_{ux\,min}$

$M_{uy} > M_{uy\,min}$

Design the column for axial load & biaxial Bending.

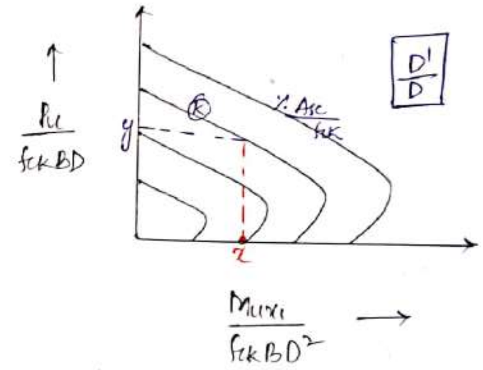
Step5:- Estimate the followings:-

(i) $\frac{D'}{D}$

(ii) $\frac{P_u}{f_{ck}BD} = y$

(iii) $\frac{\% A_{sc}}{f_{ck}} = k$ Note:- Assume 1% to 1.4% area of steel R/f.

Step6:→ Based on $\frac{D'}{D}$ ratio select a proper column Interaction Diagram



$$\frac{M_{ux1}}{f_{ck}BD^2} = x$$

$$\boxed{M_{ux1} = x \cdot f_{ck} BD^2}$$

M_{ux1} → moment carrying capacity about x axis (↑ bachane wali taqat)

M_{ux} → applied moment about x axis (↳ todne wali taqat)

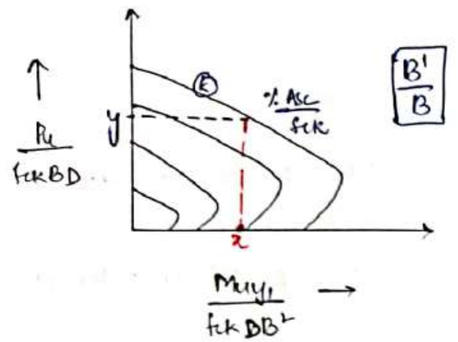
Step7:→ Estimate the followings:

(i) $\frac{B'}{B}$

same (already taken) (ii) $\frac{P_u}{f_{ck}BD} = y$

(iii) $\frac{\% A_{sc}}{f_{ck}} = k$

Step 8: → Choose a curve based on the $\frac{B'}{B}$ ratio from SP 16.



$$\frac{M_{uy_1}}{f_{ck}DB^2} = x \Rightarrow \boxed{M_{uy_1} = x \cdot f_{ck} DB^2}$$

Step 9: → Satisfy the following check.

$$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy_1}}\right)^{\alpha_n} \le 1.0$$

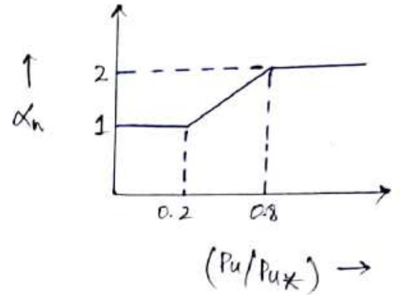
Governs Depth(D), Governs width(B).

If 1st term is more than increase Depth.
If 2nd term is more than increase B.
If addition of Both terms is $0.3 \le 1$ then the section is more safe causes economic cost so decrease dimensions, % of steel etc.

⇒ where α_n depends upon the ratio of $\frac{P_u}{P_{uz}}$

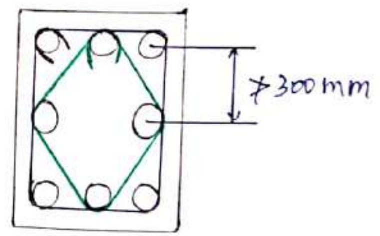
P_u/P_{uz}	α_n
≤ 0.2	1
0.2 to 0.8	1 to 2
≥ 0.8	2

- P_u → Applied axial load
- P_{uz} → $0.45 f_{ck} A_c + 0.75 f_y A_{sc}$



*** If the above check is satisfied then assumed values of D, B', D, D', f_{ck}, f_y, A_{sc}, are correct.

Step 10: → Detailing



Step 11: → Provide stirrups.

Spacing ≯ 16 ϕ
≯ B
≯ 300 mm
} min

ye max condition hai use zyada nahi hona chahiye (kam hosakta hai)

Stirrups/Rings → $\frac{1}{4} \times \phi_{max}$ or 6 mm } max.

ye min condition hai use kam nahi hona chahiye zyada hosakta hai.

L14:-

Q8Y

$B = 400\ mm$

$D = 400\ mm$

$P_u = 640\ kN$

$f_{ck} = 20\ MPa$

$$\frac{P_u}{f_{ck} B D} = \frac{640 \times 10^3}{20 \times 400 \times 400} = 0.2$$

For $\frac{P_u}{f_{ck} B D} = 0.2$, $\frac{M_u}{f_{ck} B D^2} = 0.3$ (from fig)

$$M_u = 0.3 \times f_{ck}\, B D^2$$
$$= 0.3 \times 20 \times 400 \times 400^2$$
$$= 384 \times 10^6\ N\,mm$$
$$= 384 \times 10^6 \times 10^{-6}\ kNm$$
$$\boxed{M_u = 384\ kNm}$$

$M_u = P_u \cdot e$

$$e = \frac{M_u}{P_u} = \frac{384 \times 10^6}{640 \times 10^3}$$

$e = 600\ mm$

L15:- **Design of long columns**

if $\frac{l_{eff}}{B} > 12$ [long column]

In the long columns selenderness is higher therefore columns are designed for additional bending moments.

M_{ax} → Additional Bending Moment about x-x axis

M_{ay} → Additional Bending Moment about y-y axis

longs columns should be designed considering these 2 moments.

That means (For long columns, columns should be designed for the following moments)

uniaxially loaded column → P_u, M_{ax}, M_{ay} [Type 1]

Axial load & uniaxial moment → P_u, $M_{ux} + M_{ax}$, M_{ay} [Type 2]

(or)

P_u, M_{ax}, $M_{uy} + M_{ay}$ [Type 3]

Axial load & Biaxial Moment → P_u, $M_{ux} + M_{ax}$, $M_{uy} + M_{ay}$ [Type 4]

As per I.S 456:2000

where $M_{ax} = \frac{P_u D}{2000}\left(\frac{l_{eff\,x}}{D}\right)^2$, $M_{ay} = \frac{P_u B}{2000}\left(\frac{l_{eff\,y}}{B}\right)^2$

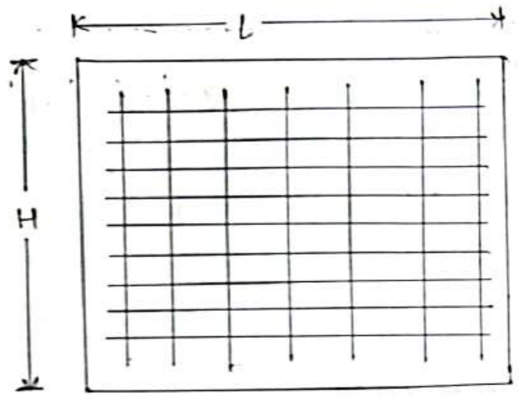
Check:-

$$\boxed{\left(\frac{M_{ux} + M_{ax}}{M_{ux_1}}\right)^{\alpha n} + \left(\frac{M_{uy} + M_{ay}}{M_{uy_1}}\right)^{\alpha n} \leq 1}$$

Procedure is same for Biaxial Bending

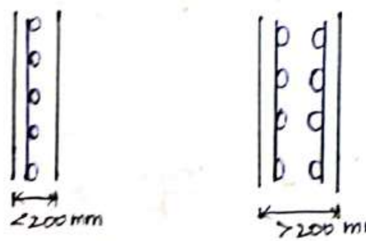
Shear walls ki thickness (D) → 150 - 200 mm hoti hai.

L16:- <u>IS code Recommendations for Design of RCC walls :-</u>



Types of steel	Minimum vertical R/f	Minimum Horizontal R/f
HYSD Bars.	= 0.12% of gross area $= \frac{0.12}{100} \times 1000 \times D$ (1m, Thickness of wall)	= 0.20% of gross area $= \frac{0.20}{100} \times 1000 \times D$
Mild steel	= 0.15% of gross area $= \frac{0.15}{100} \times 1000 \times D$	= 0.25% of gross area $= \frac{0.25}{100} \times 1000 \times D$

<u>Top view of RC wall</u>



If thickness of wall is <200 mm one layer is provided

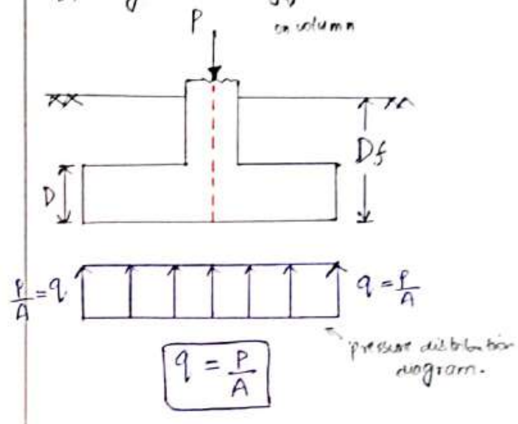
otherwise 2 layer is provided.

Footing or Foundation

We donot want tension in footing & column. (Bcz concrete is weak in Tension)

L17:- <u>Analysis & Design of footing & Estimation of upward soil pressure.</u>

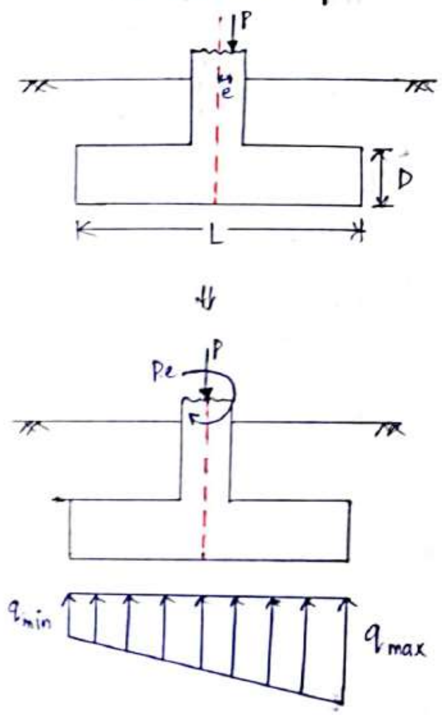
(i) only load acting, no moments



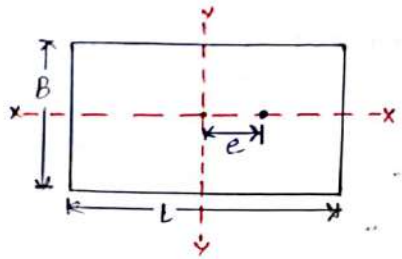
$$\boxed{q = \frac{P}{A}}$$

where A → area of footing
P → load acting on column
D_f → Depth of footing
D → Thickness of footing
q → uniform pressure
L → length of footing.
B → width of footing

(ii) uniaxially or biaxially with moments occuring.



<u>Top View</u>



These stresses may be + or - depending upon tension or compression

$$\sigma = \frac{P}{A} \pm \frac{M}{I} \cdot y$$

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

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

$$q_{max} = \frac{P}{A} + \frac{M}{I} \cdot y$$

$$q_{min} = \frac{P}{A} - \frac{M}{I} \cdot y$$

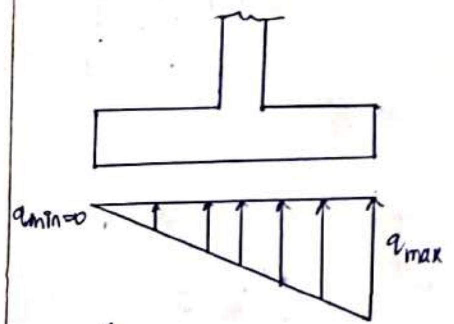
where $I = \frac{BL^3}{12}$, $y = \frac{L}{2}$, $M = P.e$

Explanation:-

We donot want Tension in footing also. if tension develops footing will fail.

In q_{min} if $\frac{M}{I}.y$ value is more than q_{min} will become (-), therefore Tension develops.

So, we have to calculate minimum eccentricity for no tension in footing



Estimation of max eccentricity for not to develop Tension

$$q_{min} = \frac{P}{A} - \frac{M}{I} \cdot y$$

putting $q_{min} = 0$ (for no tension, no compression)

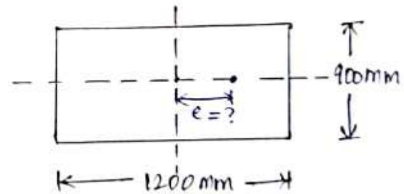
$$0 \geq \frac{P}{BL} - \frac{Pe}{\frac{BL^3}{12}} \cdot \left(\frac{L}{2}\right)$$

$$\frac{6Pe}{BL^2} \leq \frac{P}{BL}$$

$$\boxed{e_{max} \leq \frac{L}{6}}$$

→ Itna minimum eccentricity hona chalega isse zyada hoga to tension develop hoga aur footing fail hoga. bcz concrete is weak in Tension.

Q Ex:- What is the maximum value of eccentricity (e) which will not cause tension any where in Section?

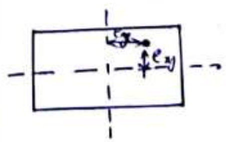


A) 200 mm
B) 300 mm
C) 600 mm
D) 400 mm

Sol

$$e_{max} \leq \frac{1200}{6} = \underline{\underline{200}}$$

For Biaxial Bending :



$M_{ux} = P_u . e_x$

$M_{uy} = P_u . e_y$

$$\frac{P}{A} \pm \frac{M_{ux}}{I} \cdot y \pm \frac{M_{uy}}{I} \cdot y$$

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L18:- Is code Recommendations for design of footings:-

(1) Minimum Nominal clear cover

Members	IS456:1978	IS 456:2000
Slab	15 mm	20 mm
Beam	25 mm	25 mm
Column	40mm or 25 mm	40mm or 25 mm
footing	50 mm or 75 mm	50 mm or 75 mm

Why two values of column:-

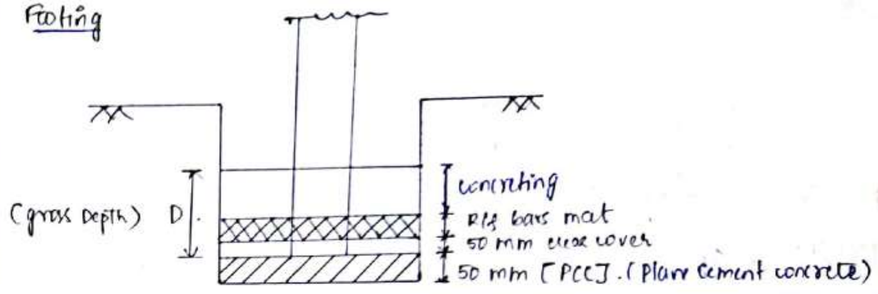
* In the column, minimum diameter of bar is 12 mm. In that case minimum 40 mm clear cover shall be provided.
* If Diameter of bar in the column is restricted (less than 12mm) to 12mm Then minimum 25 mm clear cover can be provided.
[diameter of bar less than 12 mm can be used when sides of column are not exceeding 200 mm]

Why Two values of Footing:-

* When concrete is laid over the pcc layer of 50mm then minimum 50 mm nominal clear cover shall be provided in the footing.
* If concrete is directly laid over The ground/soil surface then minimum 75 mm clear cover shall be provided in The footing. [ex. Raft footing].

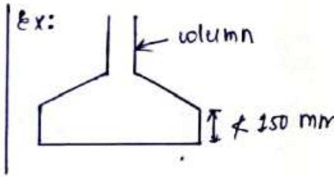
Exploration:-

Footing



(2) Minimum 150 mm thickness shall be provided at the edge of footing.

Ex:



(3) Minimum area of R/f in footing

= 0.15% of the gross area [Fe 250/MS] Mild steel

= 0.12% of the gross area [Fe 415, Fe 500/HYSD] footing is always design for working load. (i,e actual loads not Factored loads).

19:- Area (Squ, circle, rect) of footing is calculated for working load.
but Thickness of footing is calculated & design for factored loads.

Design of footing :-

working load applied from column = P

weight of footing is considered as = 10% of axial load

$= \frac{10}{100} \times P$

$= 0.1P$

$\boxed{\text{Total axial working load} = P + 0.1P = 1.1P}$

Design of Area of footing :-

$$\text{Area of footing} = \frac{\text{Total working load}}{\text{safe Bearing capacity of soil}}$$

$$A_{req} = \frac{1.1P}{SBC}$$

Ex:
If $A_{req} = 5.8\ m^2$
then provide $A_{prov} = 2m \times 3m$

$$\boxed{A_{provided} = L \times B}$$

L → length of footing
B → width of footing

$$\text{Upward soil pressure} = \boxed{q = \frac{P}{A_{provided}}}$$ → for uniform pressure

$$\boxed{q = \frac{P}{A_{provided}} \pm \frac{M}{I} \cdot y}$$ → for non uniform pressure (for Biaxial moment one more term must be added.)

To Design the thickness of footing factored upward soil pressure shall be used.

$$\boxed{q_u = 1.5 \times q}$$

*** As per the clause 34.1 [IS 456: 2000]
footings shall be designed to sustain the service/working loads only.
[Area of footing shall be calculated for working loads only But thickness of footing shall be calculated for factored load.]

There are 3 criteria for designing thickness of footing :-

1) Bending moment criteria. d_1
2) One way Shear criteria. d_2
3) Two way Shear criteria. d_3

d_1, d_2, d_3 — depths — whichever is Maximum

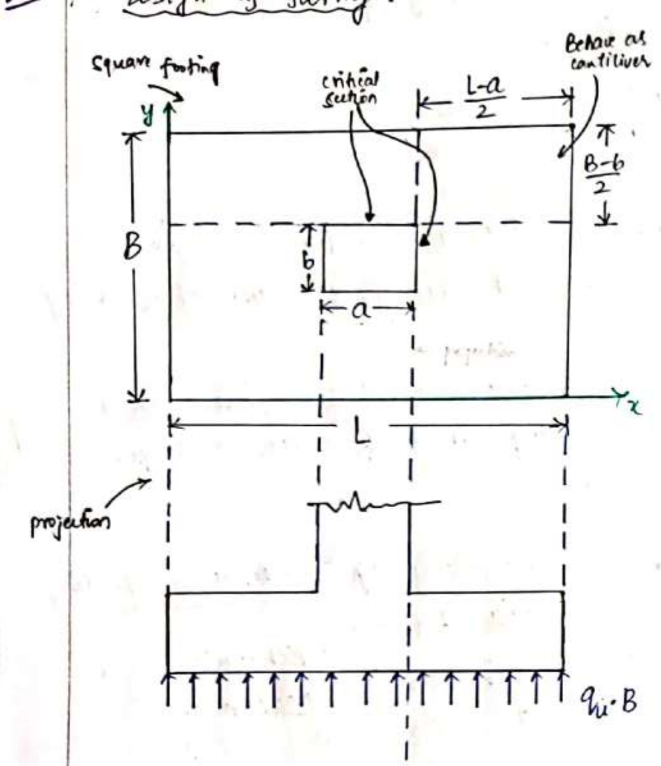
$q_u \rightarrow$ ultimate soil pressure

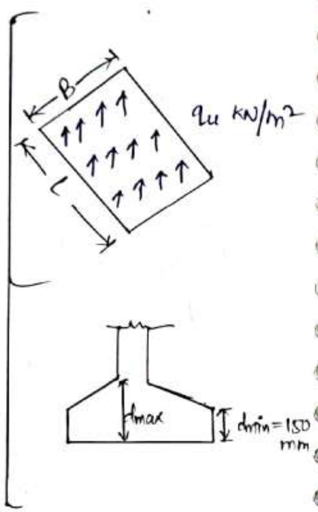
$M_u \rightarrow$ maximum B·M

more span, more value of B·M

less span, less value of B·M.

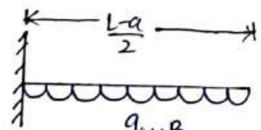
L20:- **Design of footing :-**





1st criteria :- Bending Moment criteria

* Critical section occurs at the face of the column. Because maximum cantilever Bending occurs at the support [face of the column].

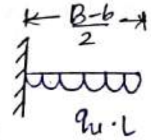


$$M_u = \frac{wl^2}{2}$$

$$M_u = \frac{q_u \cdot B \cdot \left[\frac{L-a}{2}\right]^2}{2}$$

$$\boxed{M_u = q_u \cdot B \cdot \frac{[L-a]^2}{8}} \Rightarrow \boxed{M_{uy} = q_u B \frac{(L-a)^2}{8}} \quad (1)$$

about y-y axis



$$\boxed{M_{ux} = q_u L \frac{(B-b)^2}{8}} \quad (2)$$

about x-x axis

* M_{max} = maximum of $\begin{cases} M_{ux} \\ \& \\ M_{uy} \end{cases}$. (here for this diag M_{uy} will be max)

WKT $M_{ulim} = Q f_{ck} b d^2$

$$d = \sqrt{\frac{M_{ulim}}{Q f_{ck} b}}$$

$$\Downarrow$$

$$\boxed{d_1 = \sqrt{\frac{M_{umax}}{Q f_{ck} b}}}$$

$Q = 0.148$ [Fe 250]

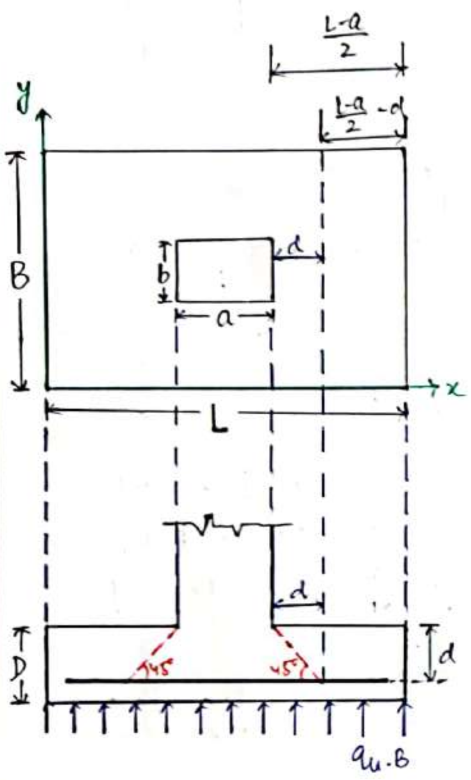
$= 0.138$ [Fe 415]

$= 0.133$ [Fe 500]

** $d_1 \rightarrow$ Effective depth / thickness of the footing calculated from the Bending Moment criteria.

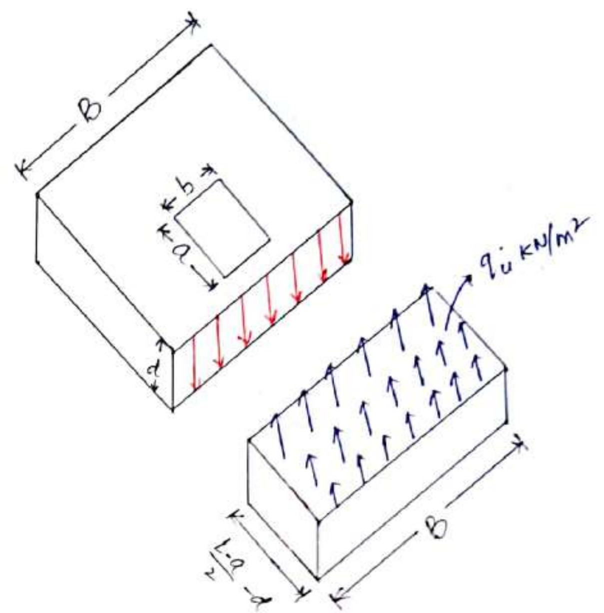
** Minimum 150 mm thickness shall be provided at the edge of the footing.

21:- Design of footing :-



2nd criteria :→ One way shear criteria

* Also known as Transverse shear criteria
* The critical section occurs at a distance equal to effective depth of the footing from the face of the column.
* It is calculated along the larger span only.



in upward dirth — Pressure × Area

Applied Shear force (↑) $= q_u B\left[\frac{L-a}{2}-d\right]$ — ① → Todne wali taqat.

Shear force Resistance (↓) $= (K\tau_c)\,B\cdot d$ — ② Bachane wali Taqat.

in downward dirth

if ② ≥ ① [Safe condition]

$$\boxed{K\tau_c B d \geq q_u B\left[\frac{L-a}{2}-d\right]} \quad \text{— ③}$$

where τ_c → Shear strength of concrete [Table 19] → For Assuming value of % Ast.

D(mm)	≥300	275	250	225	200	175	≤150
K	1	1.05	1.10	1.15	1.20	1.25	1.30

* From the eq③, Effective depth d_2 can be calculated.

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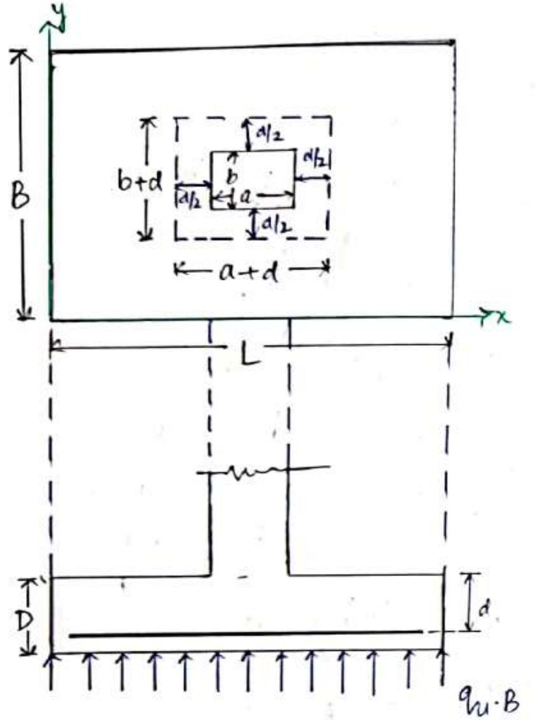
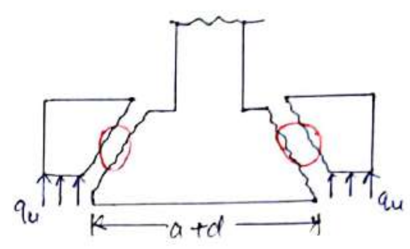
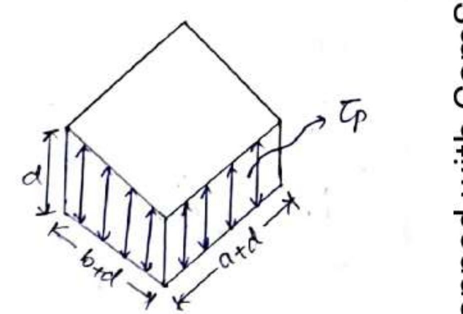
L22:- Design of footing :-



3^{rd} criteria :- Two way shear criteria :-

* Also known as punching shear criteria.
* critical section occurs at a distance equal to $d/2$ from the face of the column.
* It is calculated in both the direction in the plan area of the footing ∴ It is called as two way shear criteria.





$k_s = 0.5 + \beta_c$

$\beta_c = \frac{\text{shorter side of column}}{\text{longer side of column}}$

$\beta_c = \frac{b}{a}$

β_c has maximum value as 1.

$\tau_p = k_s \cdot \tau_{pc}$

τ_{pc} → punching shear strength of concrete

$\tau_{pc} = 0.25\sqrt{f_{ck}}$ [LSM]

$= 0.16\sqrt{f_{ck}}$ [WSM]

Applied punching shear force (↑) $= q_u \left[B \times L - (a+d)(b+d) \right]$ — ① (→ Todne wali Taqat)

Punching shear Resistance force (↓) $= \tau_p \cdot \left[2\{(a+d)+(b+d)\} \right] \cdot d$ — ② (↳ Bachane wali Taqat)

② ≥ ① [Safe condition]

$$\boxed{\tau_p \cdot \left[2\{(a+d)+(b+d)\} \right] \cdot d \geq q_u \left[B \times L - (a+d)(b+d) \right] \quad -③}$$

From the eqn ③ ; effective depth d_3 can be calculated.

$d_1 \rightarrow 1^{st}$ criteria
$d_2 \rightarrow 2^{nd}$ criteria
$d_3 \rightarrow 3^{rd}$ criteria
} whichever is max.

eff depths.

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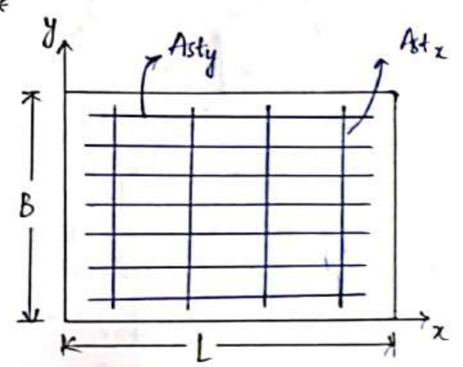
$N_T \rightarrow$ Total No of bars.

L23:

Design of Depth & steel R/f for footing :-

WKT $d = \max \begin{Bmatrix} d_1 \\ d_2 \\ d_3 \end{Bmatrix}$

*



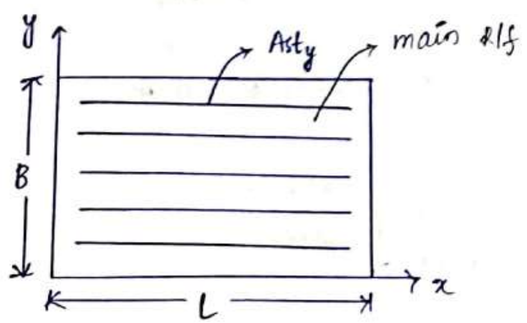
$M_{uy} \rightarrow Ast_y$

$M_{ux} \rightarrow Ast_x$

$$Ast_y = 0.5 \frac{f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_{uy}}{f_{ck} B d^2}}\right] Bd.$$

$$N_T = \frac{Ast_y}{\frac{\pi}{4}(\phi^2)}$$ [along the longer span]

Ex:- If $N_T = 5$ bars



$N_c \rightarrow$ No of bars to be provided in central portion.

$$Ast_x = 0.5 \frac{f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_{ux}}{f_{ck} B d^2}}\right] Bd.$$

$$N_T = \frac{Ast_x}{\frac{\pi}{4}(\phi^2)}$$ [along the shorter span]

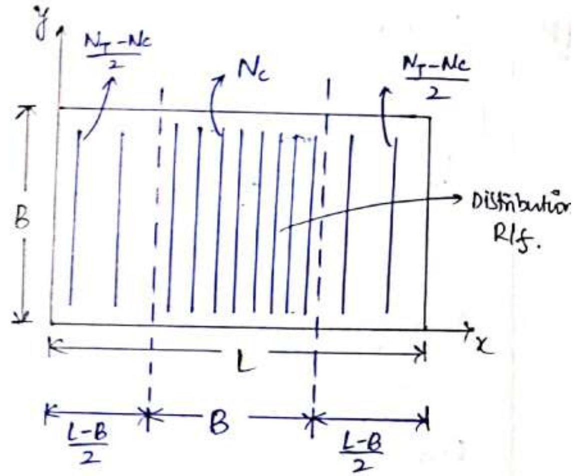
$$N_c = N_T\left[\frac{2}{1+\frac{L}{B}}\right]$$

Ex:- $L = 4m$, $B = 2m$, $N_T = 12$

$N_c = 12 \times \left(\frac{2}{1+2}\right)$

$= \cancel{12}^{4} \times \frac{2}{\cancel{3}}$

$N_c = 8$



Minimum area of R/f in footing

$Ast_{min} = 0.15\%$ of gross area
$= \frac{0.15}{100} \times B \times D.$ } Mild steel (Fe 250).

$Ast_{min} = 0.12\%$ of gross area
$= \frac{0.12}{100} \times B \times D$ } HYSD (Fe 415, Fe 500)

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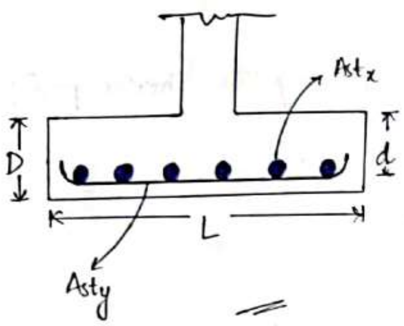
Note :- main reinf niche rakhte hai
Distribution reinf uske upar rakhte hain .

c/s of footing :-



L24:- Transfer of loads :-

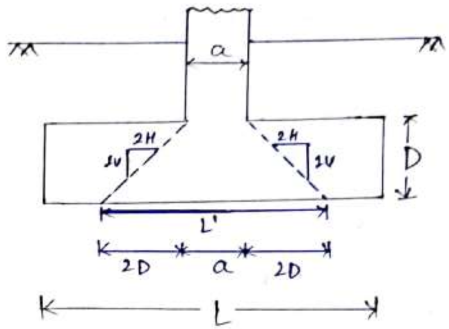
Pressure Applied by column = Pressure taken by concrete by its Bearing strength + Pressure taken by dowel bars. —①

Bearing strength of concrete = $0.45\, f_{ck}$ [LSM]
$= 0.25\, f_{ck}$ [WSM]

Bearing strength of concrete in supporting system $= 0.45\, f_{ck} \sqrt{\frac{A_1}{A_2}}$

[Since we are discussing about LSM we have taken 0.45]

Dowel bars → Reinf provided at a junction of column & footing.



$L' = 2D + a + 2D$
$B' = 2D + b + 2D$
$A_1 = L' \times B'$
$A_2 = a \times b$ (area of column)

$\sqrt{\frac{A_1}{A_2}} \not> 2$

∴ max value can be 2

eq① can be written as

Applied load = load taken by bearing strength of concrete + load taken by dowel bars

$$P_u = \left[0.45\, f_{ck} \sqrt{\frac{A_1}{A_2}} \right] (a \times b) + P_{dowel}$$

$$\boxed{P_{dowel} = P_u - 0.45\, f_{ck} \sqrt{\frac{A_1}{A_2}} \times a \times b}$$

$$\boxed{\text{Area of dowel Bars} = \frac{P_{dowel}}{0.67\, f_y}}$$

$$\boxed{\text{No. of dowel Bars} = \frac{\text{Area of dowels}}{\frac{\pi}{4}(\phi^2)}}$$

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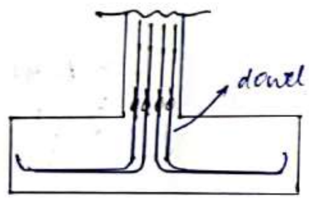
* Minimum 0.5% of total gross area of supporting System [column] shall be provided as dowel bars area. (ya sir)

* Minimum 4 number of longitudinal Rft bars of column can be continued into the footing.

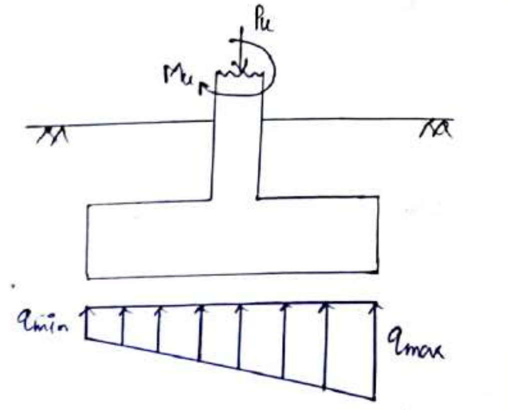
* The diameter of dowel bars shall not exceed the diameter of the longitudinal Rft of the column by more than 3 mm.



Here completes the design of footing

q_u - upward soil pressure.

125:- Q9) $P_u = 450\ kN$,
$M_u = 60\ kN\text{-}m$.
$B = 2m$,
$L = 3m$.

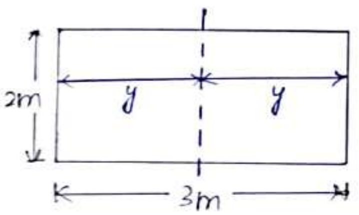


WKT $q_{max} = \frac{P}{BL} + \frac{M}{I} \cdot y$

$$= \frac{450}{2\times3} + \frac{60}{\left(\frac{2\times3^3}{12}\right)} \cdot \left(\frac{3}{2}\right)$$

$$= 75 + 20$$

$q_{max} = 95\ kN/m^2$



WKT $q_{min} = \frac{P}{BL} - \frac{M}{I} \cdot y$

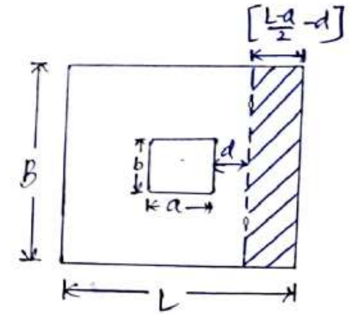
$q_{min} = 75 - 20$

$q_{min} = 55\ kN/m^2$

Agar jis b side ko cut karegi usko cube karo I calculate karte waqt

$I = \frac{2\times3^3}{12}$

Q10) $L = 2m$
$B = 2m$
$a = 300\ mm$
$b = 300\ mm$
Let $P_u = 320\ kN$
$d = 200\ mm$



Note:- One way shear is always calculated along the longer span.

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$$q_u = \frac{P_u}{B \times L} = \frac{320 \times 10^3}{(2 \times 2) \times (10^3)^2}$$

m² is converted into mm²

$$q_u = 0.08 \ N/mm^2$$

WKT $q_u B \left[\frac{L-a}{2} - d \right] = \tau_c B d$

$$0.08 \times (2000) \left[\frac{2000-300}{2} - 200 \right] = \tau_c (2000 \times 200)$$

$$\tau_c = 0.26 \ N/mm^2 \text{ or } Mpa$$

WST ka weightage bhi kam hai nahi k barabar (sir said)

L26, L27 dekh loge to bo chalega (sir said)

L26:- Introduction to Working Stress Method & Modular Ratio.

Jahan par cost se xyada safety imp consider kiya jata hai wahan working stress method use karte hain.

ex:- Dams, bridges, big constructions etc.

(m) Modular Ratio :→ It is defined as the ratio of Modulus of elasticity of steel to the modulus of elasticity of concrete.

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

WKT, $E_s > E_c$ (nearly 10 to 12 times greater)

$\therefore m > 1$

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

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

> Apart from RCC, in general modular ratio is defined as "Ratio of two ~~diff~~ modulus of elasticity of two different materials"

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

$$m = \frac{2 \times 10^5}{5000\sqrt{f_{ck}}}$$

short term static modulus of elasticity

⇓

Short term modular Ratio

Bcz of without considering the effect of creep.

E_{cr} → modulus of elasticity of concrete considering the effect of creep.

* $m = \frac{E_s}{E_{cr}}$

$m = \frac{2\times10^5}{\left(\frac{5000\sqrt{f_{ck}}}{1+\phi}\right)}$ ⇒ long term static modulus of elasticity

⇓

long term modular ratio,
[Bcz of full effect of creep has been considered]

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

This modular ratio considers the partial effect of creep.

This value of modular ratio is used in working stress method of design.

σ_{cbc} → Permissible compressive strength of concrete in bending/flexure ⇒ (agar compression bending k karan aa raha hai)

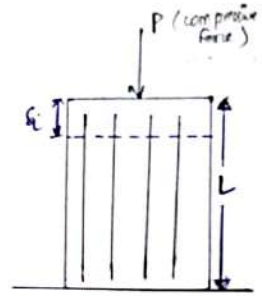
(Design stress @ strength in concrete in flexure in WSM)

(σ_{cbc} is generally considered as $\frac{1}{3}$rd of f_{ck})

{ in LSM design stress @ strength in concrete is $0.446 f_{ck}$ }

f_{ck}	M15	M20	M25	M30	M35	
$\sigma_{cbc} \approx \frac{1}{3} f_{ck}$	5	7	8.5	10	11.5	(N/mm^2)
$m = \frac{280}{3\sigma_{cbc}}$	19	13	11	9	8	(No unit)

12:: Design philosophy of working stress method



P = load taken by concrete + load taken by steel.

$\boxed{P = \sigma_{cc} A_c + \sigma_{sc} A_{sc}}$

{ σ_{cc} → stress in concrete in compression.
σ_{sc} → stress in steel in compression.
A_c → Area of concrete.
A_{sc} → Area of steel in compression. }

$(\delta_L)_{concrete} = (\delta_L)_{steel}$

$\frac{P_c L}{A_c E_c} = \frac{P_s L}{A_{sc} E_s}$

$\frac{\sigma_{cc}}{E_c} = \frac{\sigma_{sc}}{E_s}$

$\sigma_{sc} = \frac{E_s}{E_c}\sigma_{cc}$

$\boxed{\sigma_{sc} = m.\sigma_{cc}}$

{ σ_{cc} → direct compressive stress in concrete (in column)
σ_{sc} → stress direct compressive stress in steel }

$P = \sigma_{cc} A_c + (m\,\sigma_{cc}) A_{sc}$

$\boxed{P = \sigma_{cc}\left[A_c + m A_{sc}\right]}$ ⇒ in compression

A_c → area of concrete

$m A_{sc}$ → equivalent area of concrete. ⇒ column me se steel nikal k wo jagah (area) itna concrete cover karega. utni jagah (area) ko bolte hain equi. area of concrete.

In Tension

$T = \sigma_{dt} [A_c + m A_{st}]$

Tensile force

$$\boxed{\sigma_{dt} = \frac{T}{A_c + m A_{st}}}$$

$\sigma_{dt} \rightarrow$ direct tension in concrete
(or)
direct tensile strength of concrete.

WKT where $A_c = A_g - A_{st}$

$$\sigma_{dt} = \frac{T}{A_g - A_{st} + m A_{st}}$$

$$\boxed{\sigma_{dt} = \frac{T}{A_g + (m-1) A_{st}}}$$

* 3 strength of concrete

$\sigma_{cbc} \rightarrow$ bending compressive strength of concrete ($\simeq \frac{1}{3} f_{ck}$)
(used in Beams, slabs, staircase, footings design)

$\sigma_{cc} \rightarrow$ Direct compressive strength of concrete in compression
(used in column design)

$\sigma_{dt} \rightarrow$ Direct tensile strength of concrete.
(when members are subjected to the direct tensile force).

(Page-80 & 81)
Pdf file

f_{ck}	σ_{cbc} (N/mm²)	σ_{cc} (N/mm²)	σ_{dt} (N/mm²)
M10	3	2.5	1.2
M15	5	4.0	2
M20	7	5	2.8
M25	8.5	6	3.2
M30	10	8	3.6
M35	11.5	9	4
M40	13.0	10	4.4
M45	14.5	11	4.8
M50	16	12	5.2

Ye values exceed nahi karni chahiye

No need to remember this table

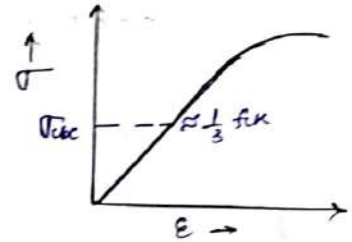
L28:- Assumptions & Recommendations in working stress Method for design of Beams under the flexure condition.

1) The plane section remains plane before & after the bending.
That means strain variation is linear. (straight line)

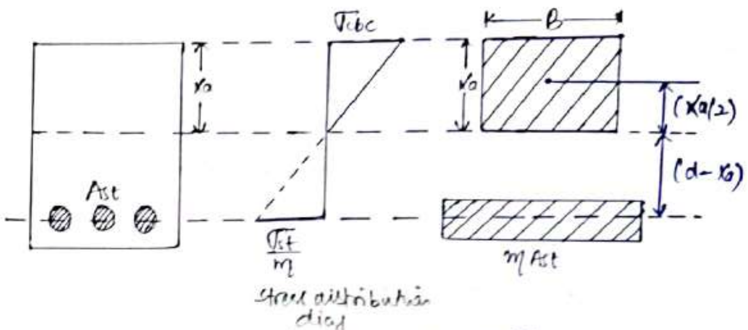
2) All the tensile stresses shall be taken by steel only. That means concrete below the Neutral axis is considered to be cracked/Ineffective.

3) Modular ratio can be considered as $\frac{280}{3\sigma_{cbc}}$ considering partial effect of creep.
σ_{cbc} → It is the permissible compressive stress in concrete in bending.

4) Linear stress-strain curve of concrete can be consider. Linear elasticity has been considered.



x_a → actual depth of neutral axis. | in LSM → 0.87fy we take
in WSM → σ_{st} ≈ 0.55fy we take



$\sigma_{st} = m\sigma_{cc}$

$\sigma_{cc} = \frac{\sigma_{st}}{m}$

$m A_{st}$ → Equivalent area of concrete

$\frac{\sigma_{st}}{m}$ → stress in surrounding concrete

Total tensile force ⇒ stress x Area
$\Rightarrow \frac{\sigma_{st}}{m} \times m A_{st}$
$\Rightarrow \sigma_{st} A_{st}$
$(\approx 0.55 f_y) A_{st}$

Estimation of depth of Neutral axis (method is diff from LSM)

Take moment of area about Neutral axis.

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

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

x_a → Actual depth of Neutral axis.

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Recalling

In LSM

Flexure → $0.87 f_y$ (Tensile stress)

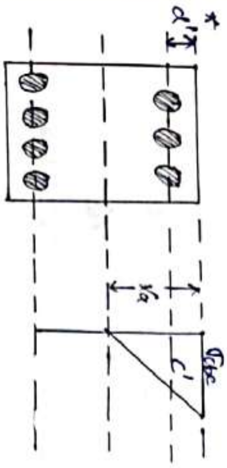
→ f_{sc} (stress in steel in compression (compressive stress) zone) (doubly R/f beam)

compression (column) → $0.75 f_y$ (Pure compression)

→ $0.67 f_y$ (considering the effect of eccentricity)

ye sab values WSM me kitne hote hain dekhte hain.

* m → modular Ratio → $\frac{280}{3\sigma_{cbc}}$

* 

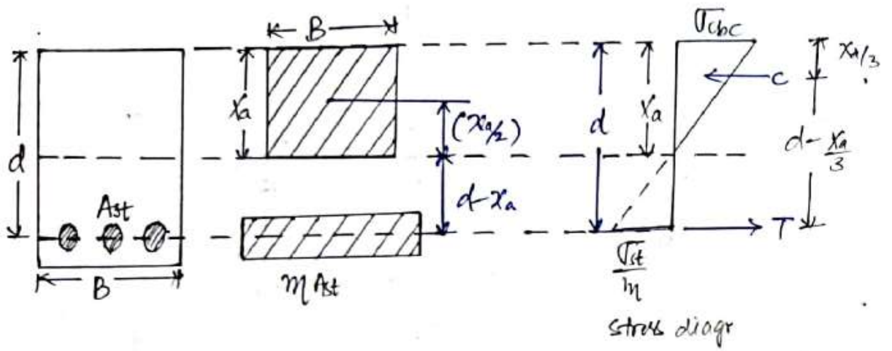
{By apply similar Δle concept c' can be calculated}

* $\boxed{P = \sigma_{cc} A_c + \sigma_{sc} A_{sc}}$ {Formula for design of column in WSM}

	Types of stress in steel R/f	Fe250 ≈ $0.55 f_y$ (N/mm²)	Fe415 ≈ $0.55 f_y$ (N/mm²)	Fe500 = $0.55 f_y$ (N/mm²)
1)	Tensile stress in steel in bending (σ_{st} → used in design of Beam, slab, stair case, footing)	140 ($\phi \le 20$ mm) 130 ($\phi > 20$ mm)	230 N/mm²	275 N/mm²
2)	compressive stress in steel in Bending (stress taken in compression R/f in Bending in doubly R/f Beam)	$1.5 mc'$	$1.5 mc'$	$1.5 mc'$
		c' → stress in concrete at the level of compression R/f in the compression zone.		
3)	Axial compressive strength / Stress in steel R/f (σ_{sc} → It is compressive stress in steel used in column design.)	130 N/mm²	190 N/mm²	190 N/mm²

ye sab values yaad honi chahiye (sir said)

Q29:- Analysis & Design of a singly Rlf beam section using WSM



$\sigma_{st} \rightarrow 140\ N/mm^2$ or $130\ N/mm^2$ } Fe 250

$\rightarrow 230\ N/mm^2$ (Fe 415)

$\rightarrow 275\ N/mm^2$ (Fe 500)

Estimation of depth of Neutral Axis

Taking moment of area about Neutral axis.

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

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

Total compressive force

$$C = (\sigma_{cbc}) \times \frac{1}{2} B x_a$$

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

Total Tensile force

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

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

$$(MOR)_c = C \times LA = \frac{1}{2} \sigma_{cbc} B x_a \left(d - \frac{x_a}{3}\right)$$

$$\boxed{(MOR)_c = \frac{1}{2} \sigma_{cbc} B x_a \left(d - \frac{x_a}{3}\right)}$$

$$(MOR)_T = T \times LA = \sigma_{st} A_{st} \left(d - \frac{x_a}{3}\right)$$

$$\boxed{(MOR)_T = \sigma_{st} A_{st} \left(d - \frac{x_a}{3}\right)}$$

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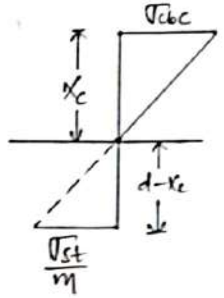
130-

Critical Depth of Neutral axis & Type of sections in WSM

* Critical depth of Neutral axis (x_c)



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

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

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

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

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

$$\boxed{x_c = \frac{(280/3)}{(280/3) + \sigma_{st}} \cdot d}$$

$$x_c = k \cdot d$$

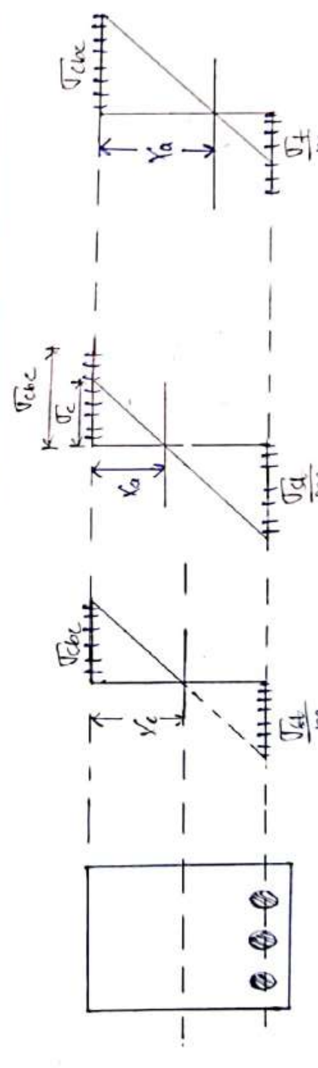
k → critical Neutral axis constant

$$\boxed{k = \frac{m\,\sigma_{cbc}}{m\,\sigma_{cbc} + \sigma_{st}}}$$

Note :- critical depth of Neutral axis in the given Section depends upon the grade of Steel only.

In LSM — WSM
x_{ulim} ↔ x_c
R_{ulim} → (MOR)

* Types of Sections



Balanced Secⁿ
- $x_a = x_c$
- $\sigma_c \geqslant \sigma_{cbc}$
- $\sigma_t \geqslant \sigma_{st}$
- $(MOR)_{Bal}$

Under R/f secⁿ
- $x_a < x_c$
- $\sigma_c < \sigma_{cbc}$
- $\sigma_t \geqslant \sigma_{st}$
- Steel will fail first
- Ductile failure
- less amount of steel R/f is provided as compare to Balanced Secⁿ
- Alarm

Over R/f Secⁿ
- $x_a > x_c$
- $\sigma_c \geqslant \sigma_{cbc}$
- $\sigma_t < \sigma_{st}$
- concrete will fail first
- Brittle failure
- Excessive amount of steel R/f is provided as compare to Balanced secⁿ
- No Alarm.

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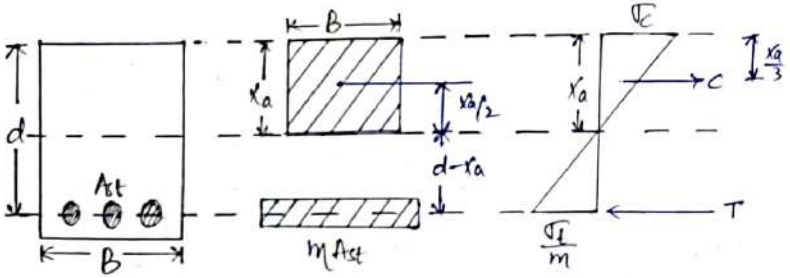
We have taken σ_c & $\frac{\sigma_t}{m}$ but we don't know which type of section is this (Bal, URS, ORS)

L39 **Expected Type of Problem from singly Rf beam using WSM**

Problem Type 1:- calculate MOR [for f_y B d Ast]

We don't know which type of section is this



Step 1:- calculate x_c

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

Step 2:- calculate x_a

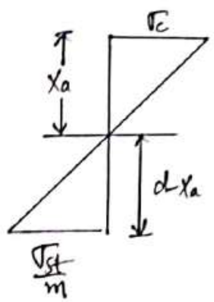
$$\boxed{\frac{B x_a^2}{2} = m\,A_{st}\,(d - x_a)}$$

Step 3:- compare x_a & x_c

(a) if $x_a < x_c$ (URS)

$\sigma_c < \sigma_{cbc}$ ⇒ stress of concrete apni permissible value par nahi pahunch paye hain.

$\sigma_t = \sigma_{st}$ ⇒ stress of steel apni perm. " " pahunch k khate hain.



similarly

$$\frac{\sigma_c}{x_a} = \frac{\sigma_{st}}{m(d-x_a)}$$

$$\boxed{\sigma_c = \frac{\sigma_{st}\,(x_a)}{m\,(d-x_a)}}$$

$$(MOR)_c = \frac{1}{2}(\sigma_c)\,B \cdot x_a\left(d - \frac{x_a}{3}\right)$$

$$(MOR)_T = \sigma_{st}\,A_{st}\left(d - \frac{x_a}{3}\right)$$

b) if $x_a = x_c$ (Balanced section)

$\sigma_c = \sigma_{cbc}$

$\sigma_t = \sigma_{st}$

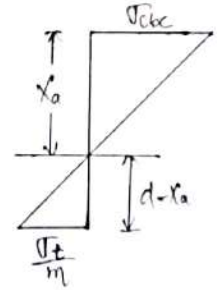
$$(MOR)_c = \frac{1}{2}\sigma_{cbc}\,B\,x_c\left(d - \frac{x_c}{3}\right)$$

$$(MOR)_T = \sigma_{st}\,A_{st}\left(d - \frac{x_c}{3}\right)$$

c) if $x_a > x_c$ (ORS)

$\sigma_c = \sigma_{cbc}$

$\sigma_t < \sigma_{st}$



$$\frac{\sigma_{cbc}}{x_a} = \frac{\sigma_t}{m(d-x_a)}$$

$$\boxed{\sigma_t = \frac{\sigma_{cbc}\,m\,(d-x_a)}{x_a}}$$

$$(MOR)_c = \frac{1}{2}\sigma_{cbc}\,B\,x_a\left(d - \frac{x_a}{3}\right)$$

$$(MOR)_T = \sigma_t\,A_{st}\left(d - \frac{x_a}{3}\right)$$

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L32:-

Problem Type :-2 :- Design Singly R/f Beam

(When cross-sectional dimensions are not given)

$$(MOR)_c = \frac{1}{2}\sigma_{cbc} B x_c \left(d-\frac{x_c}{3}\right)$$

[We always design Balanced section]

$$(MOR)_c = \frac{1}{2}\sigma_{cbc} B (k \cdot d)\left(d-\frac{k \cdot d}{3}\right)$$

$$MOR = \frac{1}{2}\sigma_{cbc} B k d^2\left(1-\frac{k}{3}\right)$$

* $\boxed{(MOR) = Q B d^2}$ (for Balanced section)

(in LSM we take $M_{ulim} = Q f_{ck} B d^2$)

$$Q = \frac{1}{2}\sigma_{cbc} k\left(1-\frac{k}{3}\right)$$

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

$$d_{req} = \sqrt{\frac{MOR}{QB}}$$

* $\boxed{d_{req} = \sqrt{\frac{B \cdot M_o}{QB}}}$ → unfactored B.M

Calculation of Ast

$$(MOR)_T = \sigma_{st} A_{st}\left(d-\frac{x_c}{3}\right)$$

$$B \cdot M_o = (MOR)_T = \sigma_{st} A_{st}\left(d-\frac{x_c}{3}\right)$$

* $\boxed{A_{st} = \frac{B \cdot M_o}{\sigma_{st}\left(d-\frac{x_c}{3}\right)}}$

Problem Type 3:- Design a beam

(when cross-sectional dimensions are known)

(Given data → $B, d, f_{ck}, f_y, B \cdot M_o$)

$$(MOR)_{Bal} = Q B d^2 \quad \text{(for Balanced section)}$$

$$Q = \frac{1}{2}\sigma_{cbc} k\left(1-\frac{k}{3}\right)$$

$$k = \frac{m\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{st}}$$

compare $(MOR)_{Bal}$ & Applied $B \cdot M_o$

If $(MOR)_{Bal} \geq B \cdot M_o$

Design singly R/f Beam.

If $(MOR)_{Bal} < B \cdot M_o$

Design Doubly R/f Beam.

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L33:-

Question based on MOR using WSM

Q) Calculate MOR

$b = 400$ mm

$d = 550$ mm

$m = 11$

$A_{st} = 4$ # 16 mmϕ

use M25 & Fe 415.

Sol

Step(1):- Calculate x_c

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

$$x_c = \frac{11 \times 8.5}{11 \times 8.5 + 230} \times (550)$$

$$\boxed{x_c = 158.96 \text{ mm}}$$

Step(2):- Calculate x_a

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

$$400\frac{x_a^2}{2} = 11 \times (4 \times 201) \times (550 - x_a)$$

$$200 x_a^2 + 8844 x_a - 4864200 = 0$$

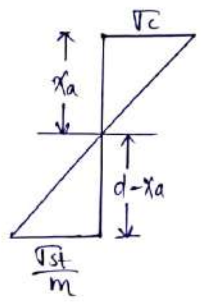
$$\boxed{x_a = 135.40 \text{ mm}}$$

Step 3:- compare x_a & x_c

$\because x_a < x_c$ [URS]

$\therefore$ $\sigma_c < \sigma_{cbc}$

$\sigma_t = \sigma_{st}$



$$\frac{\sigma_c}{x_a} = \frac{\sigma_{st}}{m(d - x_a)}$$

$$\sigma_c = \frac{\sigma_{st} \cdot x_a}{m(d - x_a)}$$

$$\sigma_c = \frac{230 \times 135.4}{11 \times (550 - 135.4)}$$

$$\boxed{\sigma_c = 6.828 \text{ N/mm}^2}$$

$$(MOR)_c = \frac{1}{2}\sigma_c\, B\, x_a \left(d - \frac{x_a}{3}\right)$$

$$(MOR)_c = \frac{1}{2} \times 6.828 \times 400 \times 135.40 \left(550 - \frac{135.40}{3}\right)$$ $\times 10^6$ to convert in kN-m

$$\boxed{(MOR)_c = 93.35 \text{ kN-m}}$$

$$(MOR)_T = \sigma_{st} A_{st} \left(d - \frac{x_a}{3}\right)$$

$$(MOR)_T = 230 \times (4 \times 201) \times \left(550 - \frac{135.40}{3}\right)$$

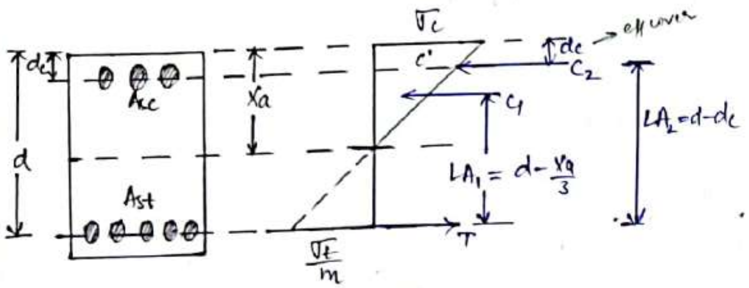
$$\boxed{(MOR)_T = 93.36 \text{ kN-m}}$$

sir has done full ans for analysis but in exam cal only σ_t & get the ans in 1 min. (for URS only)

(for ORS cal $(MOR)_c$)

We have taken σ_c & $\frac{\sigma_t}{m}$ but we don't know which type of section is this (Bal or URS or ORS)

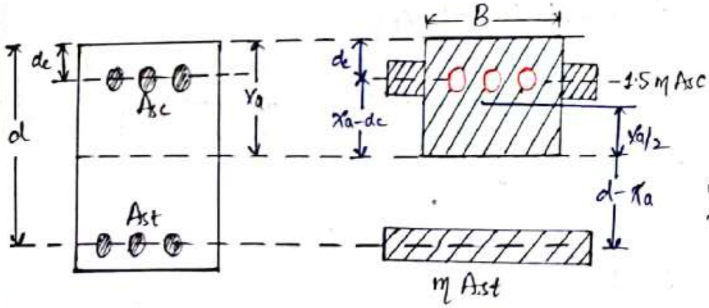
L34:- Analysis of Doubly R/f Beam using WSM



Step 1:- calculate x_c

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

Step 2:- calculate x_a



modular ratio
comp R/f me 1.5m lete hain
Tension R/f me m lete hain.

$$B x_a \cdot \frac{x_a}{2} - A_{sc}(x_a - d_c) + 1.5\,m\,A_{sc}(x_a - d_c) = m\,A_{st}(d - x_a)$$

$$\boxed{B\frac{x_a^2}{2} + (1.5m - 1)A_{sc}(x_a - d_c) = m\,A_{st}(d - x_a)}$$

c' → stress in concrete at the level of steel R/f.
C_2 → compressive force taken by steel R/f above Neutral axis.

Step 3:- compare x_a & x_c

if $x_a = x_c$	if $x_a < x_c$	if $x_a > x_c$
Balanced.	URS	ORS
$\sigma_c = \sigma_{cbc}$	$\sigma_c < \sigma_{cbc}$	$\sigma_c = \sigma_{cbc}$
$\sigma_t = \sigma_{st}$	$\sigma_t = \sigma_{st}$	$\sigma_t < \sigma_{st}$

Step 4:- calculate MOR

$$(MOR)_c = C_1 LA_1 + C_2 LA_2$$

$C_1 = \frac{1}{2}\sigma_c B x_a$

$LA_1 = d - \frac{x_a}{3}$

$C_2 = f_{sc} A_{sc} - c' A_{sc}$

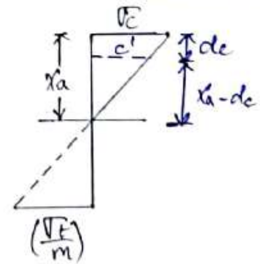
$= 1.5m\,c' A_{sc} - c' A_{sc}$

$C_2 = (1.5m - 1)c' A_{sc}$

$LA_2 = d - d_c$

$$\boxed{(MOR)_c = \frac{1}{2}\sigma_c B x_a \left(d - \frac{x_a}{3}\right) + (1.5m - 1)c' A_{sc}(d - d_c)}$$

$$\boxed{(MOR)_T = \sigma_t A_{st_1}\left(d - \frac{x_a}{3}\right) + \sigma_t \cdot A_{st_2}(d - d_c)}$$



$$\frac{\sigma_c}{x_a} = \frac{c'}{x_a - d_c}$$

$$\boxed{c' = \frac{\sigma_c(x_a - d_c)}{x_a}}$$

L35:-

Design of Doubly R/f Beam using WSM

from previous diag

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

$A_{st_1} \rightarrow (MOR)_{bal}$ ko lene k liye daale hai

$A_{st_2} \rightarrow BM-(MOR)_{Bal}$ ko lene k liye daale hain

$$(MOR)_c = \frac{1}{2} \sigma_c B x_a \left(d-\frac{x_a}{3}\right) + (1.5m-1)\sigma_c' A_{sc}(d-d_c)$$

$$(MOR)_T = \sigma_t A_{st_1}\left(d-\frac{x_a}{3}\right) + \sigma_t \cdot A_{st_2}(d-d_c)$$

$$\boxed{A_{st_1} = \frac{(MOR)_{Bal}}{\sigma_{st}\left(d-\frac{x_c}{3}\right)}}$$

$$(MOR)_{Bal} = QBd^2$$

$$Q = \frac{1}{2}\sigma_{cbc}\, k\left(1-\frac{k}{3}\right)$$

$$k = \frac{m\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{st}}$$

$$\boxed{A_{st_2} = \frac{BM-(MOR)_{Bal}}{\sigma_{st}(d-d_c)}}$$

$$\boxed{A_{sc} = \frac{BM-(MOR)_{Bal}}{(1.5m-1)\sigma_c'(d-d_c)}}$$

Tension / Tensile force

L36:- Q11) WKT $P = \sigma_{cc} A_c + \sigma_{sc} A_{sc}$ $\quad \boxed{\sigma_{sc} = m\sigma_{cc}}$

$$= \sigma_{cc} A_c + m\sigma_{cc} A_{sc}$$

$$= \sigma_{cc}(A_g - A_{sc}) + m\sigma_{cc} A_{sc}$$

$$= \sigma_{cc} A_g - \sigma_{cc} A_{sc} + m\sigma_{cc} A_{sc}$$

$$\boxed{P = \sigma_{cc} A_g + (m-1)\sigma_{cc} A_{sc}}$$

Que's notations $\rightarrow$ σ_c A_c $\quad$ σ_c A_s

$$P = \sigma_c A_c + (m-1)\sigma_c A_s$$

Q12)

Q13) $T = 120\,kN$
$b = 250\,mm$
$D = 400\,mm$
$A_{st} = 4\#20mm$
$Fe415$
$m = 10$

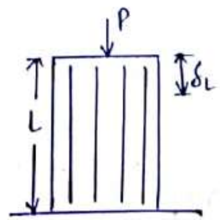
WKT $\sigma_t = \dfrac{T}{A_g + (m-1)A_{st}}$

$$\sigma_t = \frac{120\times10^3}{(250\times400)+(10-1)(4\pi/4)}$$

$$\sigma_t = 1.078\ N/mm^2$$

Q14) Net area $\xrightarrow{\text{means}}$ area of concrete $\xrightarrow{\text{matlab}}$ excluding the area of steel R/f from gross area

$A_{sc} = 0.01 A_c$
$m = 10$



$\varepsilon_{concrete} = \varepsilon_{steel}$

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

$$\frac{P_c}{A_c}\cdot\frac{E_s}{E_c} = \frac{P_s}{A_{sc}}$$

$\sigma_c = \dfrac{P_c}{A_c}$

$\sigma_s = \dfrac{P_s}{A_{sc}}$

$$m\frac{A_{sc}}{A_c} = \frac{P_s}{P_c}$$

$$10 \times 0.01 = \frac{P_s}{P_c}$$

$$\frac{P_s}{P_c} = 0.1$$

in %

$$\frac{P_s}{P_c} = 0.1 \times 100 = \underline{10\%}$$

Q15) $m = 13$

M20

Fe415

$\frac{x_c}{x_{ulim}} = ?$

WSM

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

$$= \frac{13 \times 7}{13 \times 7 + 230} \cdot d$$

$$\boxed{x_c = 0.2835d}$$

LSM

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

$$\boxed{x_{ulim} = 0.48 \cdot d}$$

$$\frac{x_c}{x_{ulim}} = \frac{0.2835d}{0.48d} = 0.59 \text{ or } \frac{7}{12}$$

Q16) M25

$\sigma_{cbc} = 8.5\ N/mm^2$

Short-terms ⇒ (without considering the effect of creep)

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

$$m = \frac{2 \times 10^5}{5000\sqrt{25}}$$

$$\underline{\underline{m = 8}}$$

long term → considering partial effect of creep (WSM)

→ considering full effect of creep

for this we need φ. but in Que φ value is not given so take ~~partial~~ above method.

∴ long Term

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

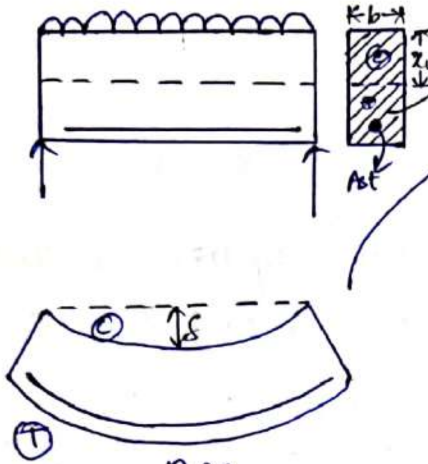
$$m = \frac{280}{3 \times 8.5} = 10.98 \approx 11$$

portion

CH:-06 PRESTRESSED CONCRETE

L1:- Introduction to prestressed concrete.

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

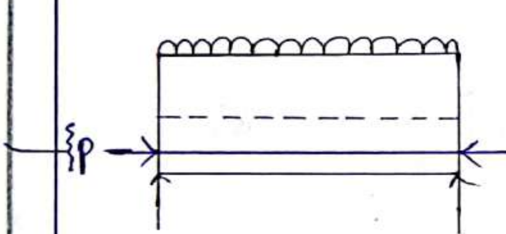


RCC

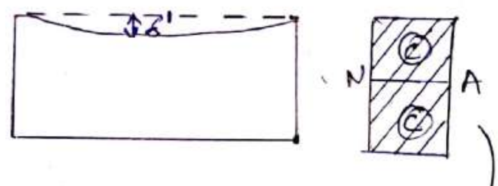
This area of concrete is useless.

Deflection is more in case of RCC beams ∴ It (RCC) cannot be used in large span girders. Bcz large depth of crss-sec is requiured & cost is more.

Bcz of this problem we study PRESTRESSED CONCRETE to overcome this problem.



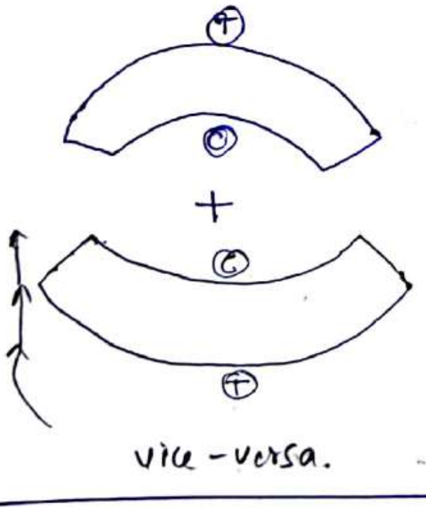
P = prestressing force (compressive force)



$\delta' < \delta$

↓ PSC ↓ RCC

Total area of concrete is effective in PSC. Bcz comm. force below NA & ∴ Dimensions can be reduced.



This is ≠ 0 bcz member has some wt.

vice-versa.

According to ACI committee of prestressed concrete

"The prestressed concrete is one in which there have been introduced internal stress of such magnitude & distribution that the stresses of resulting from the external loading can be counter balanced upto desigared degree."

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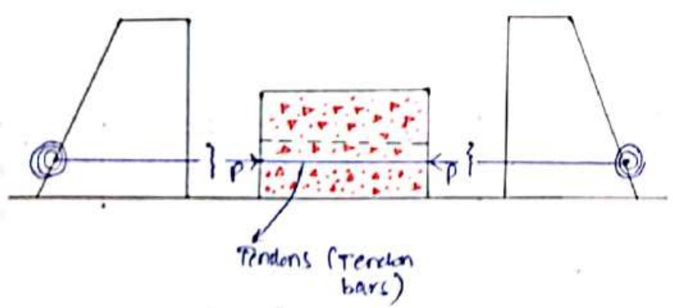
In RCC we were ignoring the concrete area & have not taken any values of concrete below N.A

L21:- Types of prestressed concrete or Type of prestressing method :-

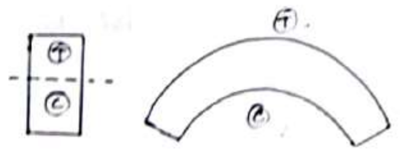
1) Pretensioned prestressed concrete. } same results but
2) post-tensioned prestressed concrete. } diff methods

1) Pre-tensioned prestressed concrete :-

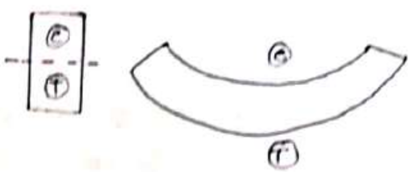
steps {
a) Tensioning. ⇒ 1st steel R/f is stretched by method of Tensioning
b) concreting. → After the tensioning concreting is done
c) Transfer of load. ⇒ After 28 days of concreting the stretched steel R/f is cut & due to the direct contact b/w steel & concrete loads are transfered.



Due to prestressing →



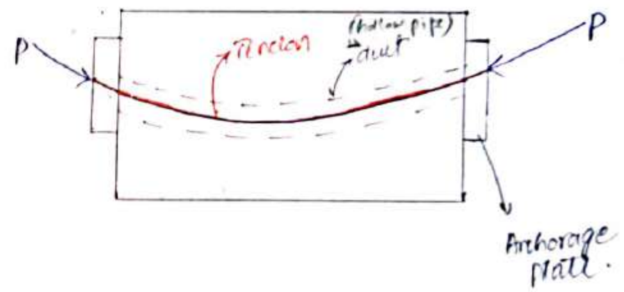
Due To loading →

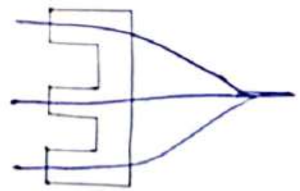


Form work - ? (length) Tendons hum bundles me rakhte hain
1 bundle me 25-50 R/f horakte hain.

2) Post-tensioned prestressed concrete :-

a) concreting ⇒ 1st concreting is done.
After 28 days of concreting, Tensioning process is done.
b) Tensioning ⇒ (During Tensioning prestressing force will develop.
(By the help of Anchorage plates))
c) Anchoring ⇒ loads are transferring by & through the anchorage plate & during the tensioning





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L3:- Merits & Demerits of prestressed concrete :-

Merits :→

① Can be used in larger span girders of bridges to carry higher loads.

② The complete area of concrete is effective therefore required cross-sectional dimensions of the member get reduced.

③ Due to the use of higher grade concrete & steel (Tendon bars), the chances of occurring cracks get reduced.

④ These members can be precasted in the factories.

⑤ Before the use, they can be tested. (Bcz they are precasted).

Demerits :-

① Initial cost is very high. (machines are requ for tensioning process)

② Well experienced labours & Engineers are required.

③ Prestressed sections are generally brittle in nature.

④ These are less fire resistant. (Bcz of use of high grade steel & concrete)

In cantilever Beam → Tendon bars provided above the NA.

In SSB → " " below the NA.

L4:- Methods to Analyse prestressed concrete sections

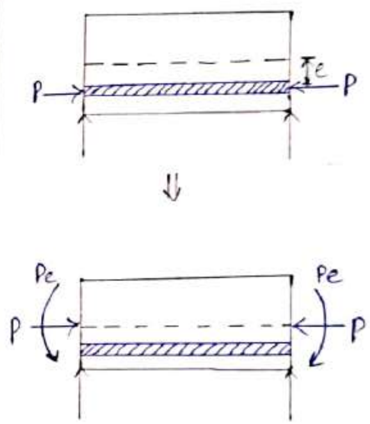
(1) Stress concept Method.

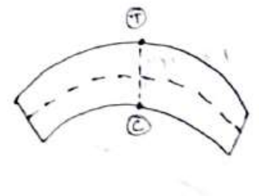
(2) Load Balancing Method.

(3) C-Line/P-Line Method.

} same results but diff methods (1) is easy for (GATE) point of view

(1) Stress concept Method :-





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

$$-\frac{Pe}{I} \cdot y_t = \sigma_{top}$$ (− tensile stress)

$$+\frac{Pe}{I} \cdot y_b = \sigma_{bottom}$$ (+ compressive stress)

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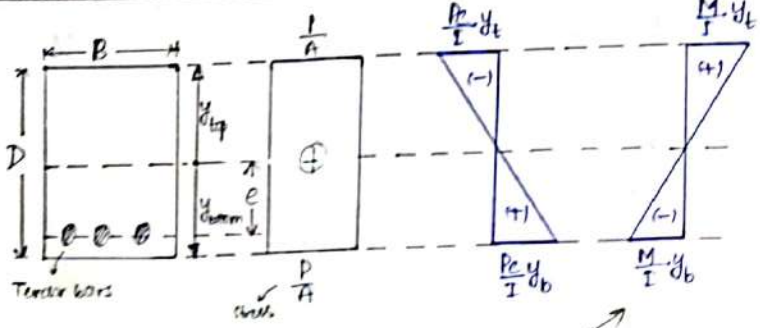
$M_{D,L}$ → dead load & live load ka comb. of bm



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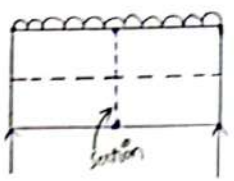
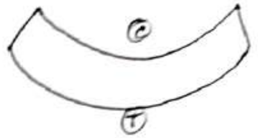


$\sigma_{top} = +\frac{M}{I} \cdot y_{top}$

$\sigma_{bottom} = -\frac{M}{I} \cdot y_{bottom}$



Imp

Stresses in simply supported Beams

** $\sigma_{top} = \frac{P}{A} + \frac{M_{DL}}{I} \cdot y_t - \frac{Pe}{I} \cdot y_t$

* $\sigma_{bottom} = \frac{P}{A} - \frac{M_{DL}}{I} \cdot y_b + \frac{Pe}{I} \cdot y_b$

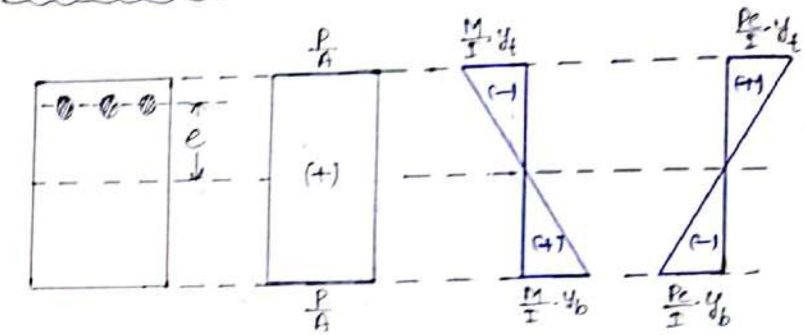
} At any section

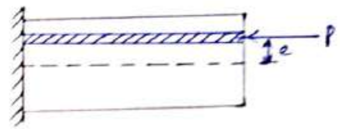
* $\sigma_{top} = \frac{P}{A} - \frac{Pe}{I} \cdot y_t$

** $\sigma_{bottom} = \frac{P}{A} + \frac{Pe}{I} \cdot y_b$

} At the support.

In cantilever Beam





$\sigma_{top} = \frac{P}{A} - \frac{M_{DL}}{I} \cdot y_t + \frac{Pe}{I} \cdot y_t$

$\sigma_{bottom} = \frac{P}{A} + \frac{M_{DL}}{I} \cdot y_b - \frac{Pe}{I} \cdot y_b$

} At any section (including supported)

$\sigma_{top} = \frac{P}{A} + \frac{Pe}{I} \cdot y_t$

$\sigma_{bottom} = \frac{P}{A} - \frac{Pe}{I} \cdot y_b$

} At the free ends.

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L5:-

Q1)

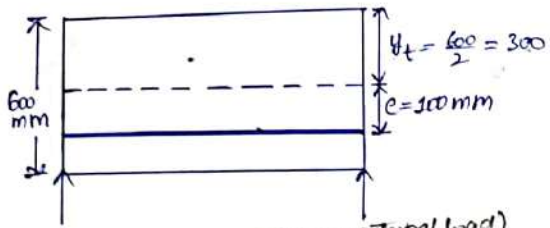
$L = 6m$

$B = 300\ mm$

$D = 600\ mm$

$e = 100\ mm$

$P = 1000\ kN$



$$\sigma_{top} = \frac{P}{A} + \cancel{\frac{M}{I}} \cdot y - \frac{Pe}{I} y_t$$ 0 (Bcz no external load)

$$= \frac{1000 \times 10^3}{300 \times 600} - \frac{1000 \times 10^3 \times 100 \times 300}{\left(\frac{300 \times 600^3}{12}\right)}$$

$$= 5.55 - 5.55$$

$$= 0$$

$$\sigma_{bottom} = \frac{P}{A} - \cancel{\frac{M}{I}} \cdot y_b + \frac{Pe}{I} y_b$$ 0 (no external load)

$$= 5.55 + 5.55$$

$$= 11.11\ N/mm^2$$

$$\text{max. comp. stress} = \max \begin{cases} 0 \\ 11.11\ N/mm^2 \end{cases}$$

$$= 11.11\ N/mm^2$$

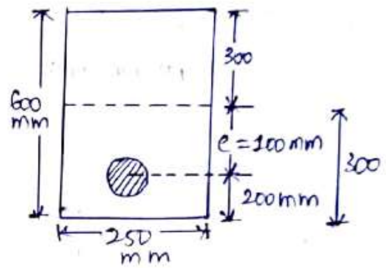
Q2)

$$\boxed{\begin{array}{l} B = 250\ mm \\ D = 600\ mm \end{array}}$$

$$A_s = \frac{\pi}{4}(14^2) \times 16 = 615.75\ mm^2$$

$$\sigma = 700\ mpa$$

$$\boxed{e = 100 mm}$$



$$P = \sigma A_s$$

$$= 700 \times 615.75$$

$$= 431025\ N$$

$$\boxed{P = 431.025\ KN}$$

$$I = \frac{BD^3}{12} = \frac{250 \times 600^3}{12}$$

$$\boxed{I = 4500 \times 10^6\ mm^4}$$

$$\sigma_{bottom} = \frac{P}{A} - \frac{M}{I} y_b + \frac{Pe}{I} y_b$$

$$0 = \frac{431.025 \times 10^3}{250 \times 600} - \frac{M \times 300}{4500 \times 10^6} + \frac{431.025 \times 10^3 \times 100 \times 300}{4500 \times 10^6}$$

$$0 = 2.874 - \frac{M \times 300}{4500 \times 10^6} + 2.874$$

$$\frac{M \times 300}{4500 \times 10^6} = 5.748$$

$$M = 86220000\ N\text{-}mm$$

$$M = 86220000 \times 10^{-6}\ KN\text{-}m.$$

$$M = 86.22\ KN\text{-}m$$

$$M = 86.2\ KN\text{-}m$$

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Vertical components → $P\sin\theta$
Horiz com → $P\cos\theta$

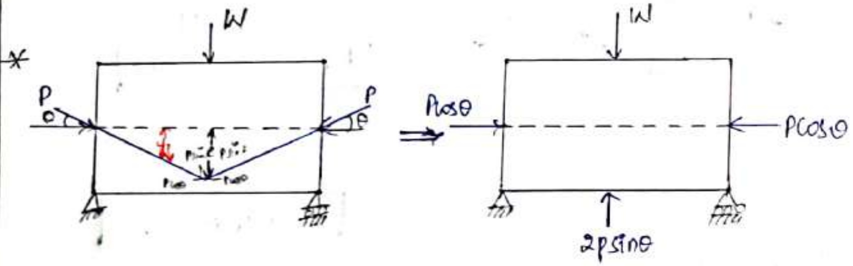
L6:-

2) Load Balancing Method

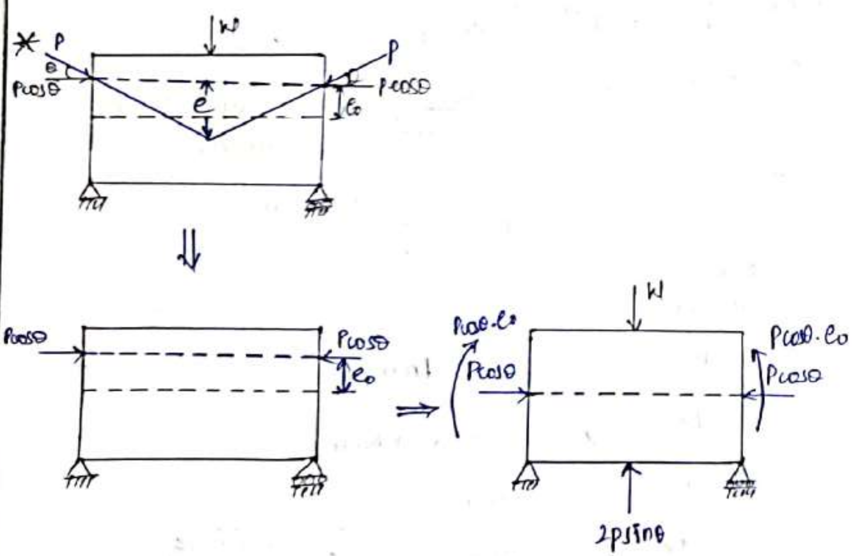
This Method is generally suitable for curved shape cable profile (Tendon bars)

[cable ka profile, BM Diag p depend karta hai
matlab jaisa BM Diag banega waisa cable provide karenge]
(BM ko line k liye)

* Triangular profile

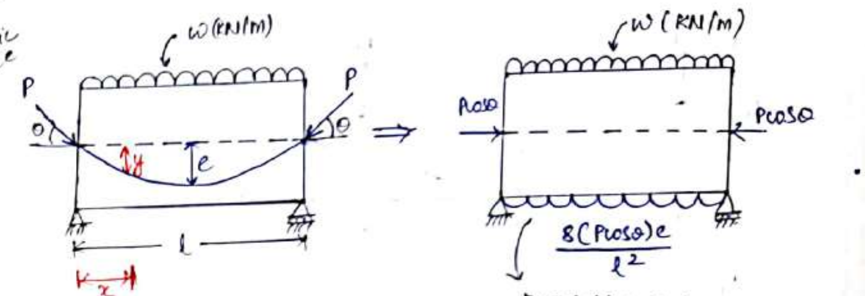


* Linear profile



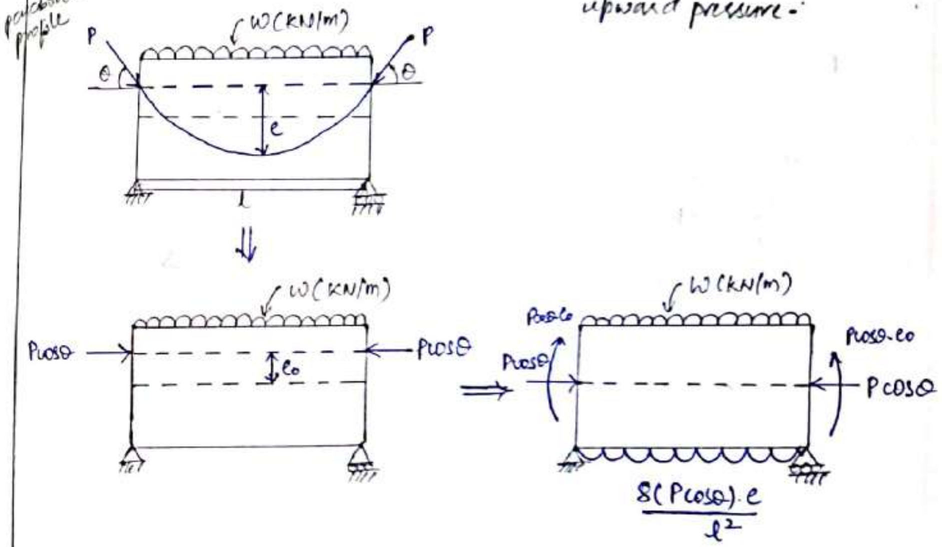
Note:- In triangular profile, eccentricity at any section can be calculated by similar triangle properties.

* Parabolic profile



Parabolic cable k karan upward pressure.

Parabolic profile



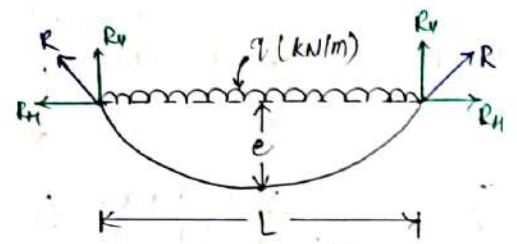
eqⁿ for parabolic cable

$$\boxed{y = \frac{4ex(L-x)}{L^2}}$$

$$\frac{dy}{dx} = \frac{4e(L-2x)}{L^2}$$

$$\tan\theta = \frac{4e(L-2x)}{L^2}$$

By doing $\theta = \tan^{-1}\left(\frac{4e(L-2x)}{L^2}\right)$ you can cal θ



$$\boxed{R_V = \frac{qL}{2}}$$

$$R_V \cdot \frac{L}{2} - q\frac{l}{2}\cdot\frac{l}{4} - R_H e = 0$$

$$\frac{ql^2}{4} - \frac{ql^2}{8} - (P\cos\theta)e = 0$$

$$\frac{ql^2}{8} = (P\cos\theta)e$$

$$q = \frac{8(P\cos\theta)e}{l^2}$$

Ans Q1)

a) Stress concept method @ at mid span

$$y = \frac{4ex[L-x]}{L^2}$$

$$\tan\theta = \frac{dy}{dx} = \frac{4e[L-2x]}{L^2}$$

$$\tan\theta = \frac{4\times 0.2\times[8-2\times\frac{8}{2}]}{8^2}$$ → at mid span

$\tan\theta = 0$ $[\theta = 0°]$ that means slope is 0 at mid span.

$$\boxed{\cos\theta = 1}$$

at mid span.

$$\sigma_{top} = \frac{P\cos\theta}{A} + \frac{M}{I}\cdot y_t - \frac{(P\cos\theta)\cdot e}{I}\, y_t$$

$$\sigma_{top} = \frac{P}{A} + \frac{M}{I}\cdot y_t - \frac{Pe}{I}\cdot y_t \quad [\cos\theta = 1 \text{ at mid span}]$$

$$M_{mid\ span} = \frac{wl^2}{8} = \frac{22.2\times 8^2}{8} = 177.6 \text{ kN-m} = 177.6\times 10^6 \text{ N-mm}$$

$$I = \frac{BD^3}{12} = \frac{400\times 750^3}{12} = 1.40625\times 10^{10} \text{ mm}^4$$

$$\sigma_{top} = \frac{1500\times 10^3}{400\times 750} + \frac{177.6\times 10^6\times 375}{1.40625\times 10^{10}} - \frac{1500\times 10^3\times 200}{1.40625\times 10^{10}}\times(375)$$

$$\sigma_{top} = 5 + 4.736 - 8$$

$$\boxed{\sigma_{top} = 1.736 \text{ N/mm}^2}$$

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(ii) At quarter span

$$y = \frac{4ex[L-x]}{L^2}$$

$$\tan\theta = \frac{dy}{dx} = \frac{4e[L-2x]}{L^2}$$

$$\tan\theta = \frac{4\times0.2\times[8-2\times2]}{(8)^2}$$

$\tan\theta = 0.05$

$\theta = 2.862$

$\cos\theta = 0.999 \approx 1$

$\boxed{\cos\theta = 1}$

$$\sigma_{top} = \frac{P\cos\theta}{A} + \frac{M}{I}\cdot y_t - \frac{(P\cos\theta)y}{I}\cdot y_t$$

$$\sigma_{top} = \frac{P}{A} + \frac{M}{I}\cdot y_t - \frac{Py}{I}\cdot y_t$$

$$M_{(x=2)} = \frac{wl}{2}x - w\cdot x\cdot\frac{x}{2}$$

$$M_{(x=2)} = \frac{22.2\times8\times2}{2} - \frac{22.2\times(2)^2}{2} = 133.2\ kN\text{-}m = 133.2\times10^6\ N\text{-}mm$$

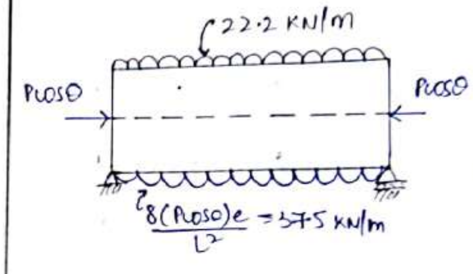
$$y = \frac{4ex[L-x]}{L^2} = \frac{4\times0.2\times2[8-2]}{8^2} = 0.15m = 150\ mm$$

$$\sigma_{top} = \frac{1500\times10^3}{400\times750} + \frac{133.2\times10^6\times375}{1.40625\times10^{10}} - \frac{1500\times10^3\times150\times375}{1.40625\times10^{10}}$$

$$\sigma_{top} = 5 + 3.552 - 6$$

$\boxed{\sigma_{top} = 2.552\ N/mm^2 \text{ or } Mpa}$

(b) Load balancing Method



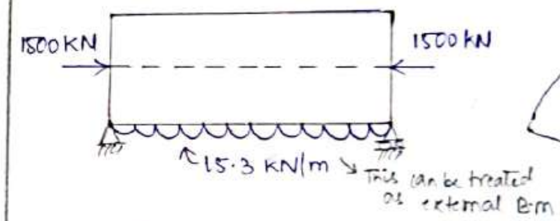
$\cos\theta = 1$

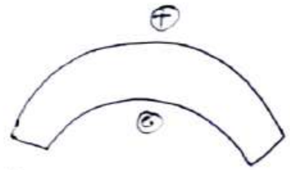
$$\frac{8(P\cos\theta)e}{L^2} = \frac{8\times1500\times0.2}{8^2} = 37.5\ KN/m$$

Net load intensity $= (22.2)\downarrow - (37.5)\uparrow$

$= -15.3\ KN/m \uparrow$

$\Downarrow$





Mid span ($\cos\theta = 1$)

$$\sigma_{top} = \frac{P\cos\theta}{A} - \frac{M}{I}\cdot y_t$$

$$M_{midspan} = \frac{wl^2}{8} = \frac{15.3\times8^2}{8} = 122.4\ KN\text{-}m = 122.4\times10^6\ N\text{-}mm$$

$$\sigma_{top} = \frac{1500\times10^3}{400\times750} - \frac{122.4\times10^6\times375}{1.40625\times10^{10}}$$

$$\sigma_{top} = 5 - 3.264$$

$\boxed{\sigma_{top} = 1.736\ N/mm^2}$

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Quarter Span

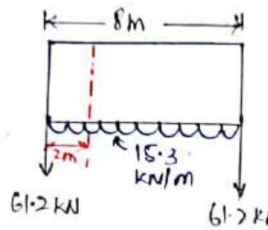
$$\sigma_{top} = \frac{P\cos\theta}{A} - \frac{M}{I} \cdot y_t$$

$$\sigma_{top} = \frac{1500 \times 10^3}{400 \times 750} - \frac{91.8 \times 10^6 \times 375}{1.40625 \times 10^{10}}$$

$$\sigma_{top} = 5 - 2.448$$

$$\boxed{\sigma_{top} = 2.552 \ N/mm^2}$$

Reactions loading k opposite lagte hain.



$$M_{quarter} = 61.2 \times 2 - 15.3 \times 2 \times 1$$
$$= 91.8 \ kN\text{-}m$$
$$= 91.8 \times 10^6 \ N\text{-}mm$$

Note:- If you are cal on mid span consider, slope at midspan
B.m at midspan
eccentricity at midspan

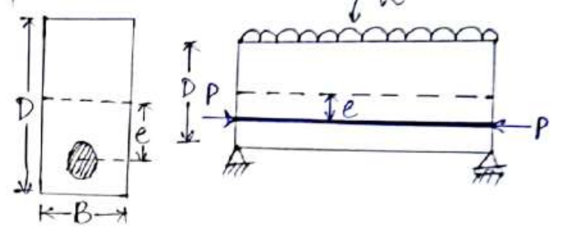
(Jahan ka section cal karte ho wahan ka consider karo)

Use 1st method don't use this method (sir said)

you can skip this method (sir said)

LS:-

3) C-Line/P-Line Method



Take $\boxed{e = \frac{D}{6}}$

& $\boxed{\sigma_{bottom} = zero}$ (at mid span) only

$$\sigma_{bottom} = \frac{P}{A} - \frac{M}{Z} + \frac{Pe}{Z}$$

$$\because \frac{I}{y} = Z$$

$$\frac{BD^3}{12 \times \frac{D}{2}} = Z$$

$$\boxed{Z = \frac{BD^2}{6}}$$

At the mid span

$$\sigma_b = \frac{P}{BD} - \frac{(wl^2/8)}{(BD^2/6)} + \frac{P.(D/6)}{(BD^2/6)} \quad ; \quad \sigma_t = \frac{P}{A} + \frac{M}{Z} - \frac{Pe}{Z}$$

$$0 = \frac{P}{BD} - \left(\frac{wl^2/8}{BD^2/6}\right) + \frac{P}{BD} \qquad \sigma_t = \cancel{\frac{P}{BD}} + \frac{wl^2/8}{BD^2/6} - \cancel{\frac{P(D/6)}{(BD^2/6)}}$$

$$\frac{wl^2/8}{BD^2/6} = \frac{2P}{BD} \qquad \sigma_t = \frac{wl^2/8}{BD^2/6}$$

$$\boxed{\sigma_{top} = \frac{2P}{BD}}$$

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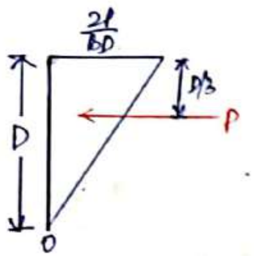


(Stress dist-diag).

Resultant force $\Rightarrow \frac{1}{2} \times \frac{2P}{BD} \times D \times B \Rightarrow P$

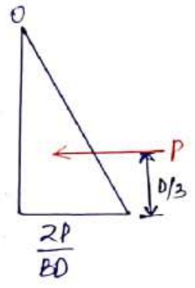
<u>At the support</u> (M=0)

$\sigma_{top} = \frac{P}{A} - \frac{Pe}{Z}$; $\sigma_{bottom} = \frac{P}{A} + \frac{Pe}{Z}$

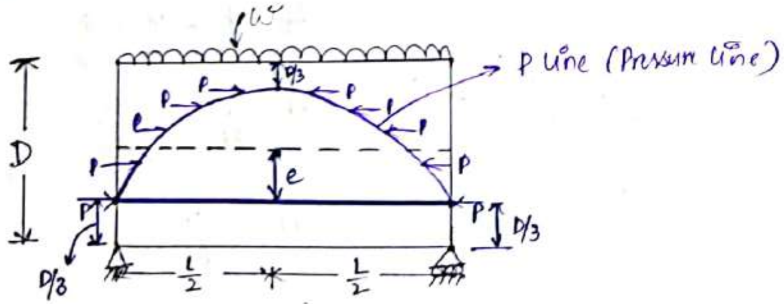
$\sigma_{top} = \frac{P}{BD} - \frac{P \times (D/6)}{(BD^2/6)}$ $\quad \sigma_b = \frac{P}{BD} + \frac{P(D/6)}{(BD^2/6)}$

$\boxed{\sigma_{top} = 0}$ $\quad \boxed{\sigma_b = \frac{2P}{BD}}$

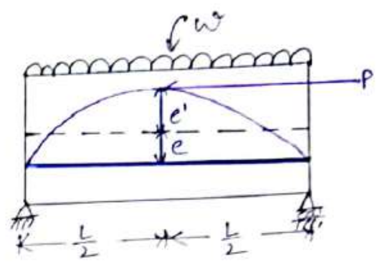
// same cal



Diag Becomes



P line ek wo line hy jiske har ek point par prestressing force (P) ki value same aarahi hai jiska ka starting point at the support from the base is D/3 & mid span from the top D/3 hai.
& is parabolic form. (Bcz B.M diag we get is parabolic form)



$M = P(e+e')$

$e+e' = \frac{M}{P}$

$\boxed{e' = \frac{M}{P} - e}$

If $e' \to +ve$, P-line is above N.A
$e' \to -ve$, P-line is below N.A

$\sigma_{top} = \frac{P}{A} - \frac{Pe'}{I} \cdot y_t$.

$\sigma_{bottom} = \frac{P}{A} + \frac{Pe'}{I} \cdot y_b$.

Ans Q1) By cline/P-line method

σ_{top} → mid span, quarter span.

σ_{bottom} → mid span, quarter span.

At mid span

$$M = \frac{wl^2}{8} = \frac{22.2 \times 8^2}{8}$$

$\boxed{M = 177.6 \text{ kN-m}}$

$\boxed{P = 1500 \text{ kN}}$ (case-1)

$\boxed{e = 200 \text{ mm}}$

$$e' = \frac{M}{P} - e$$

$$= \frac{177.6}{1500} - 0.2$$

$\boxed{e' = -81.6 \text{ mm}}$

Since e' is -ve therefore P-line is below the N.A.

$$\sigma_{top} = \frac{P}{A} - \frac{Pe'}{I} \cdot y_t$$

Take (+ve)

$$\sigma_{top} = \frac{1500 \times 10^3}{400 \times 450} - \frac{1500 \times 10^3 \times 81.6}{1.40625 \times 10^{10}} \times (375)$$

$$\sigma_{top} = 5 - 3.264$$

$\boxed{\sigma_{top} = 1.736 \text{ N/mm}^2}$

$$\sigma_{bottom} = \frac{P}{A} + \frac{Pe'}{I} \cdot y_t$$

$$\sigma_{bottom} = 5 + 3.264$$

$\boxed{\sigma_{bottom} = 8.264 \text{ N/mm}^2}$

At quarter span

$$M = \frac{22.2 \times 8}{2} \times 2 - (22.2 \times 2 \times 1)$$

$\boxed{M = 133.2 \text{ kN-m}}$

$\boxed{P = 1500 \text{ kN}}$

$\boxed{Y = 150 \text{ mm}}$

WKT $y = \frac{4e x (L-x)}{L^2}$

$$y = \frac{4 \times 0.2 \times 2 \times [8-2]}{8^2}$$

$$y = 0.15 \text{ m}$$

Now $e' = \frac{M}{P} - y$

$$e' = \frac{133.2}{1500} - 0.15$$

$\boxed{e' = -61.2 \text{ mm}}$

Since e' is -ve P-line is below N.A.

$$\sigma_{top} = \frac{P}{A} - \frac{Pe'}{I} \cdot y_t$$

$$\sigma_{top} = \frac{1500 \times 10^3}{400 \times 450} - \frac{1500 \times 10^3 \times 61.2 \times 375}{1.40625 \times 10^{10}}$$

$$\sigma_{top} = 5 - 2.448$$

$\boxed{\sigma_{top} = 2.552 \text{ N/mm}^2}$

$$\sigma_{bottom} = \frac{P}{A} + \frac{Pe'}{I} \cdot y_t$$

$$\sigma_{bottom} = 5 + 2.448$$

$\boxed{\sigma_{bottom} = 7.448 \text{ N/mm}^2}$

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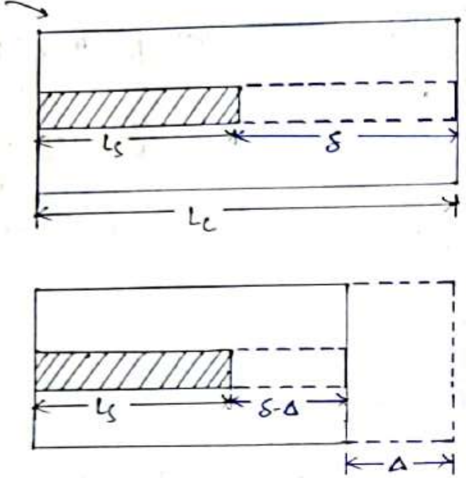
19:-

Loss of Prestress (steel me hota hai, concrete k shrink hone k karan & kabhi steel b responsible hota hai)

concrete block



$L_c \rightarrow$ length of concrete
$L_s \rightarrow$ length of steel
$E_c \rightarrow$ modulus of elasticity of steel

Initial stress in steel $= \dfrac{\delta}{L_s} \times E_s$ (strain)

Loss of stress in steel $= \dfrac{\Delta}{L_s} \times E_s$

Net available stress in steel $= \dfrac{\delta-\Delta}{L_s} \times E_c$

conclusion:- loss of elongation of steel is responsible for loss of prestress.

LOSS OF PRESTRESS

- Immediate losses (occurs during transfer of loads) (is during tensioning)
  - Elastic shortening
  - Friction
  - Anchorage slip
- Time dependent losses
  - Creep
  - Shrinkage
  - Relaxation of steel tendons.

10:-

steel me hota hai — concrete me hota hai

1) Loss of prestress due to creep [Time dependent loss]

$\Delta\sigma = \varepsilon_{cr} E_s$

WKT $\phi = \dfrac{\varepsilon_{cr}}{\varepsilon_e}$

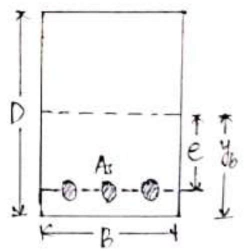
$\Delta\sigma = \phi\, \varepsilon_e E_s$

$\Delta\sigma = \phi \dfrac{f_c}{E_c} E_s$

$\Delta\sigma = \dfrac{E_s}{E_c} \times \phi \times f_c$

$$\boxed{\Delta\sigma = m \phi f_c}$$

$\Delta\sigma$ = loss of prestress.
ε_{cr} = creep strain in concrete.
E_s = Modulus of elasticity of steel.
ε_e = elastic strain in concrete.
f_c = stress in concrete at the level of steel tendons.
E_c = modulus of elasticity of concrete.
$\dfrac{E_s}{E_c}$ = modular ratio (m).
ϕ = creep coefficient.



$\sigma_{bottom} = \dfrac{P}{A} - \dfrac{M}{I} \cdot y_b + \dfrac{Pe}{I} y_b$

$$\boxed{f_c = \frac{P}{A} - \frac{M}{I} \cdot e + \frac{Pe^2}{I}}$$

$\sigma \rightarrow$ initial prestress.
$\Delta\sigma \rightarrow$ loss of prestress.

$$\boxed{\%\ \text{loss of prestress} = \frac{\Delta\sigma}{\sigma} \times 100}$$ (in terms of stress)

$P_1 \rightarrow$ initial prestressing force
$P_2 \rightarrow$ final prestressing force

$$\boxed{\%\ \text{loss of prestressing force} = \frac{P_1 - P_2}{P_1} \times 100}$$ (in terms of force).

If $P \rightarrow$ Initial prestressing force (N)
$A_s \rightarrow$ cross-sectional area of steel tendons.

then $\sigma \rightarrow$ Initial prestress $= \frac{P}{A_s}$ (N/mm²).

> Extra :- (Interview).
>
> High grade steel ⇒ 1200 N/mm² to 2000 N/mm² (yield strength)
>
> Net contraction in terms of strain ⇒ $0.0008 \times 2 \times 10^5$ (E_s)
>
> → 160 N/mm² (Loss) → loss of prestress
>
> This is why we don't use Fe415 or Fe500 or Fe250
> (415 N/mm²) (500 N/mm²) (250 N/mm²)

L11 2) Loss of Prestress due to Elastic shortening of concrete.
[Immediate loss].

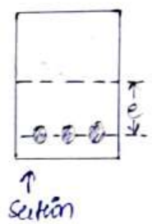
Case (I) Pre tensioned prestressed concrete

Due to elastic shortening of concrete at the time of transfer of loads, loss of elongation of steel occurs which causes loss of prestress.

$\Delta\sigma = \varepsilon_c E_s$

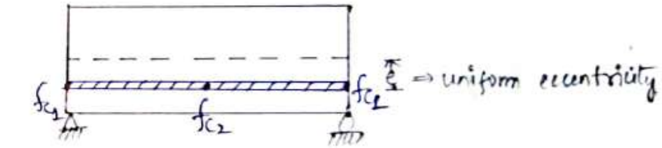
$\Delta\sigma = \frac{f_c}{E_c} \cdot E_s$

$\boxed{\Delta\sigma = m \cdot f_c}$



$\boxed{f_c = \frac{P}{A} - \frac{M}{I} \cdot e + \frac{Pe^2}{I}}$

$f_c \rightarrow$ stress in concrete at the tendon level.



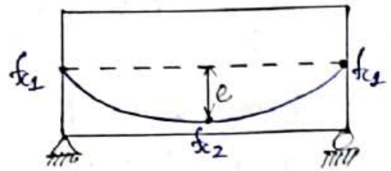
$\boxed{f_{c\,avg} = \frac{f_{c_1} + f_{c_2}}{2}}$ → when eccentricity is uniform

$\boxed{\Delta\sigma = m \cdot f_{c\,avg}}$

Note :- Agar Que me sirf section diya ho then calcu only f_c & put in $\Delta\sigma$ formula.

Agar beam diya ho que me (length, c/s diya ho) to woo apko $f_{c\,avg}$ cal karwana chahte hain. & put that $f_{c\,avg}$ in $\Delta\sigma$ formula.

(loss of elongation → length me kami hona



→ ∵ eccentricity is not uniform

Condition (1) If $f_{c_1} < f_{c_2}$

$$f_{c\,avg} = f_{c_1} + \frac{2}{3}\left(f_{c_2} - f_{c_1}\right)$$

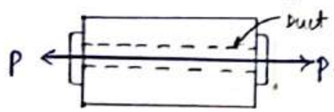
Condition (2) If $f_{c_2} < f_{c_1}$

$$f_{c\,avg} = f_{c_2} + \frac{2}{3}\left(f_{c_1} - f_{c_2}\right)$$

$$\boxed{\Delta\sigma = m\, f_{c\,avg}}$$

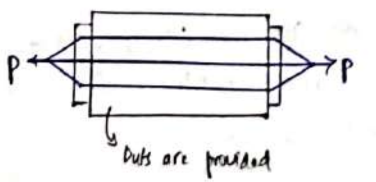
Case (II) Post-tensioned prestressed concrete

Cond^n (A): Single tendon stretching (No loss will occur)

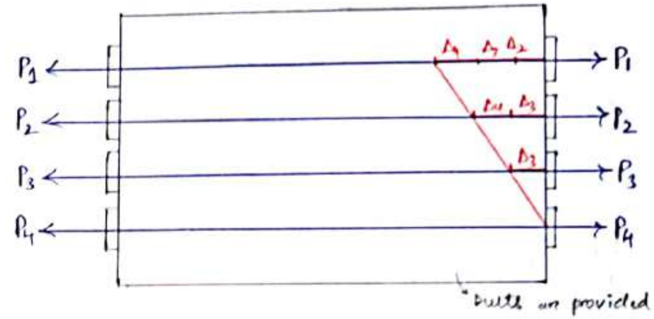


Jitni Force se pull karte hai utni hi force se plate push b karte hain.

Cond^n (B): All the Bars are Stretched together (No loss will occur)



Cond^n (C) When Bars are Stretched one by one



Major loss will occur at 1st tendon & zero loss will occur at last tendon.

Note (1) No loss of prestress occurs in single wire (Tendon bars) stretching.

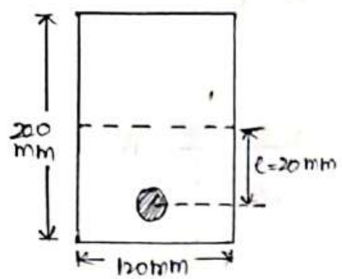
(2) No loss of prestress occurs when all the wires are stretched together.

(3) If wires are stretched one by one max. loss of prestress occurs in the 1st tendon and zero loss of prestress occurs in the last tendon.

(Same to same all formulas Here, also as in case (I)).
(Case II)

L12:-

Q3)



$P = 150\ kN$
$e = 20\ mm$
$A_s = 187.5\ mm^2$
$E_s = 2.1 \times 10^5$
$E_c = 3 \times 10^4$

WKT $\boxed{\Delta\sigma = m f_c}$

$m = \frac{E_s}{E_c} = \frac{2.1\times10^5}{3\times10^4}$; $f_c = \frac{P}{A} - \cancel{\frac{M}{I}(e)}^{0} + \frac{Pe^2}{I}$

$m = 7$

$= \frac{150\times10^3}{120\times200} + \frac{150\times10^3 \times (20)^2}{\left(\frac{120\times200^3}{12}\right)}$

$= 6.25 + 0.75$

$f_c = 7\ N/mm^2$

$\Delta\sigma = 7\times7 = 49\ N/mm^2$

$\Delta\sigma = 49\ N/mm^2$

% loss of Prestress $= \frac{\Delta\sigma}{\sigma} \times 100$

$= \frac{49}{800} \times 100$

$= 6.125\%$

$\sigma = \frac{P}{A_s} = \frac{150\times10^3}{187.5}$

$\sigma = 800$ mpa @ $\frac{N}{mm^2}$

$f_{c_{12}} \rightarrow$ stretching of 2nd bar k karan 1st bar k level par kitna stress in concrete present hai.

L13:- Ans Q2):-

Succession $\Rightarrow$ one by one @ sequencial manner.

dip @ $e = 50\ mm$

$P = 240\ kN$

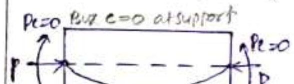
$I = \frac{BD^3}{12} = 225\times10^6\ mm^4$

* When 1st tendon is Stretched then No loss occurs in any Tendon.
* When 2nd tendon is stretched then loss occurs in 1st Tendon.

Loss in 1st Tendon due to stretching of 2nd Tendon.

At the support; $f_{c_{12}} = \frac{P}{A} + \cancel{\frac{Pe}{I}e}^{0}$ B.M=0 (Bcz no external loading)

bcz e=0

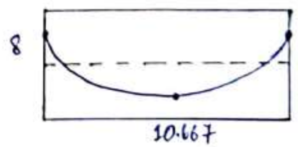


$f_{c_{12}} = \frac{240\times10^3}{300\times100} = 8\ N/mm^2$

At the mid span; $f_{c12} = \frac{P}{A} + \frac{Pe}{I}(50) = \frac{240\times10^3}{300\times100} + \frac{240\times10^3\times(50)\times(50)}{225\times10^6}$

$= 8 + 2.667$

$= 10.667\ N/mm^2$



$f_{cavg} = 8 + \frac{2}{3}[10.667 - 8]$

$f_{cavg} = 9.78\ N/mm^2$

$\Delta\sigma_{12} = m.f_{cavg}$

$\Delta\sigma_{12} = 6\times9.78$

$\Delta\sigma_{12} = 58.68\ N/mm^2$

$\Delta\sigma_{12} \rightarrow$ loss of prestress in 1st tendon due to stretching of second tendon.

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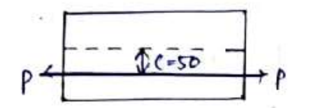
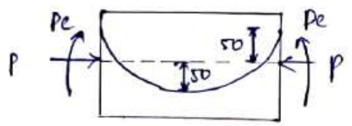
Loss in 1st tendon due to stretching of 3rd tendon:

At the support; $f_{c13} = \frac{P}{A} - \frac{Pe}{I}.(50)$

$\Rightarrow \frac{240\times10^3}{300\times100} - \frac{24\times10^3(50)\times(50)}{225\times10^6}$

$\Rightarrow 5.33\ N/mm^2$





At the mid span; $f_{c13} = \frac{P}{A} + \frac{Pe}{I}(50)$

$\Rightarrow 8 + 2.667$

$\rightarrow 10.667$

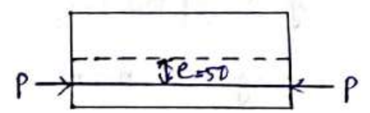
$f_{cavg} = 5.33 + \frac{2}{3}[10.667 - 5.33] = 8.89\ N/mm^2$

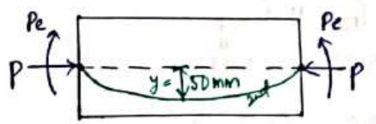
$\Delta\sigma_{13} = m f_{cavg} \Rightarrow 6\times8.89 \Rightarrow 53.33\ N/mm^2.$

$(\Delta\sigma)_1 = \Delta\sigma_{12} + \Delta\sigma_{13}$

$\boxed{(\Delta\sigma)_1 = 112\ N/mm^2}$ ⇒ Total loss in Tendon 1

Loss in 2nd Tendon due to stretching of 3rd tendon





at the support; $f_{c23} = \frac{P}{A} + \cancel{\frac{Pe}{I}}.(0) = \frac{240\times10^3}{100\times300} = 8\ N/mm^2$

at mid span, $f_{c23} = \frac{P}{A} + \frac{Pe}{I}.(50) = \frac{240\times10^3}{100\times200} + \frac{240\times10^3\times50\times50}{225\times10^6}$

$= 8 + 2.667$

$= 10.667$

$f_{cavg} = 8 + \frac{2}{3}(10.667 - 8)$

$f_{cavg} = 9.778\ N/mm^2$

$\Delta\sigma_{23} = m f_{cavg}$

$= 6\times9.778$

$\boxed{\Delta\sigma_{23} = 58.67\ N/mm^2}$ ⇒ Total loss in 2nd tendon.

~~(Δσ)₂ = 58.67 N/mm²~~

Ans

* $(\Delta\sigma)_1 = 112\ N/mm^2$.

$(\Delta\sigma)_2 = 58.67\ N/mm^2$.

$(\Delta\sigma)_3 = zero.$

[∴ max ~~tension~~ loss of prestress will occur at 1st tendon & zero ~~to~~ loss of prestress at last tendon].

L4:- 3) Loss of prestress due to shrinkage in concrete [Time dependent loss]

Case(I) Pretensioned prestressed concrete

$\Delta\sigma = \varepsilon_s E_s$ { $\varepsilon_s \rightarrow$ shrinkage strain

$$\boxed{\Delta\sigma = (0.0003) E_s}$$

Case(II) Post-tensioned prestressed concrete

$\Delta\sigma = \varepsilon_s E_s$

$$\boxed{\Delta\sigma = \frac{0.0002}{\log_{10}(t+2)} \times E_s}$$

{ $t \rightarrow$ time/age of concrete at transfer level (in days) which can be 28 days, 21 days, 14 days etc.

4) Loss of prestress due to relaxation of steel tendons

* It is time dependent loss.
* This loss of prestress is considered when stress in steel exceeds 50% of it's characteristic strength.

Stress in steel	Loss of prestress (N/mm²)
50% = $0.5 f_p$	0
60% $0.6 f_p$	35 N/mm²
70% $0.7 f_p$	70 N/mm²
80% $0.8 f_p$	80 N/mm²

$f_p \rightarrow$ characteristic strength of steel.

5) Loss of prestress due to Anchorage slip :-

* Immediate loss of prestress.
* This loss occurs in post tensioned prestressed concrete only.

$$\boxed{\Delta\sigma = \overbrace{\frac{\Delta L}{L}}^{\text{strain}} \times E_s}$$

$\Delta L \rightarrow$ Anchorage slip
$L \rightarrow$ length of member/cable.

* In shorter span beams/girders the loss of prestress due to anchorage slip is higher than the longer span beams/girders.

$\therefore \quad \Delta\sigma \propto \frac{1}{L} \quad ; \quad L\uparrow, \Delta\sigma\downarrow$

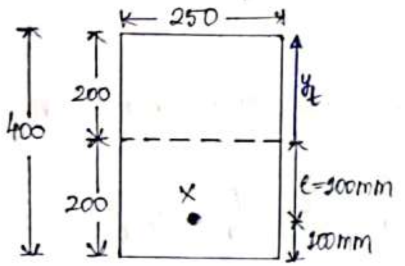
Q4) $\Delta L = 3$ mm.
$L = 30$ m
$\sigma = 1200$ N/mm²
$E_s = 2.1 \times 10^5$ N/mm².

WKT $\Delta\sigma = \frac{\Delta L}{L} \times E_s$
$= \frac{3}{30 \times 10^3} \times 2.1 \times 10^5$
$\Delta\sigma = 21$ N/mm²

% loss of prestress $= \frac{\Delta\sigma}{\sigma} \times 100$
$= \frac{21}{1200} \times 100$
$= 1.75\%$.

10% ko loss initial prestressing force me hua hai.

Q5)
L15:-



Let $P \to$ Initial prestressing force

$P - 0.1P = 750$ (10%)

$0.9P = 750$

$\boxed{P = 833.33 \text{ KN}}$

$$\sigma = \frac{P}{A} - \cancel{\frac{M}{I}}^{\,0}\cdot y_b + \frac{Pe}{I}\cdot y_b$$

No external loading

$$= \frac{833.33\times10^3}{250\times400} + \frac{833.33\times10^3\times100\times200}{\left(\frac{250\times400^3}{12}\right)}$$

$$= 8.33 + 12.5$$

$$\boxed{\sigma = 20.83 \text{ N/mm}^2} \Rightarrow$$ +ve value matlab compressive stresses ley rahe hain (compression).

$$\sigma_{top} = \frac{P}{A} + \cancel{\frac{M}{I}}^{\,0} y_t - \frac{Pe}{I}\cdot y_t$$

$$= \frac{833.33\times10^3}{250\times400} - \frac{83.33\times10^3\times100\times200}{\left(\frac{250\times400^3}{12}\right)}$$

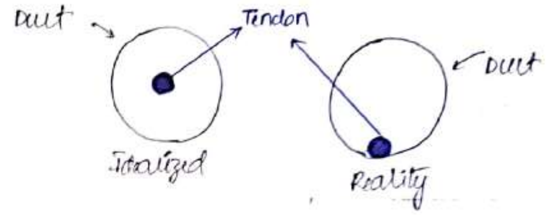
$$= 8.33 - 12.5$$

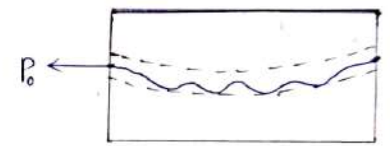
$$\boxed{\sigma_{top} = -4.167 \text{ N/mm}^2}$$ (Tension)

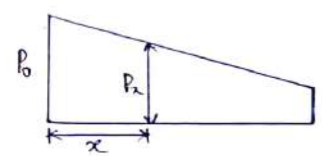
Tendon bar/wire $\to$ 2m x 3m ke hole hai bundles me generally

L16:- 6) Loss of prestress due to friction [Immediate loss]

* It occurs in post-tensioned prestressed concrete only.







If there is no loss $P_x = P_0$

$$\boxed{P_x = P_0 e^{-[kx + \mu\alpha]}}$$ $\Rightarrow$ ye form thoda km me 6 likh sakte hain.

$P_0 \to$ Initial prestressing force

$P_x \to$ Available prestressing force at any section at a distance of x from the jacking end

$k \to$ Wobble correction factor. [coefficient @ wave correction factor]

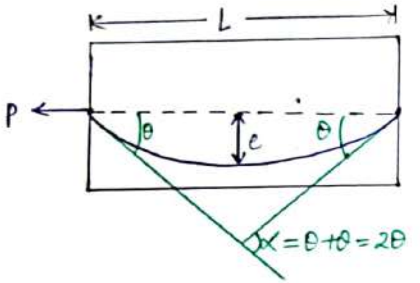
$\mu \to$ Friction coefficient due to curvature. (jitna zyada curvature hoga utna zyada loss of prestress hoga)

$\alpha \to$ cumulative angle (in radians) B/w two ends.

Types of Jacking :-

1) One end Jacking → sirf side se pull
2) Two End Jacking → Both side se pull

1) One End Jacking :- (to find α) $(x = L)$



WKT $y = \frac{4ex(L-x)}{L^2}$

$\frac{dy}{dx} = \frac{4e(L-2x)}{L^2}$

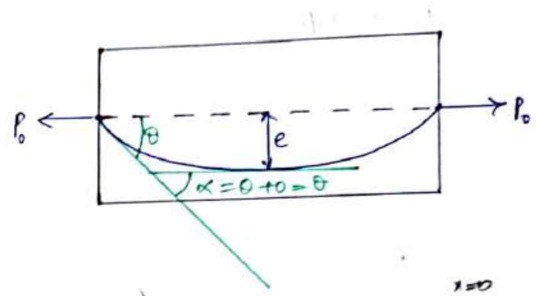
$\frac{dy}{dx} = \frac{4e(L-2L)}{L^2}$

$\tan\theta \approx \theta = -\frac{4e}{L}$

$\alpha = \frac{4e}{L} + \frac{4e}{L}$ ($x=0$, $x=L$)

$$\boxed{\alpha = \frac{8e}{L}}$$

2) Two end jacking (to find α) $[x = \frac{L}{2}]$



WKT $y = \frac{4ex(L-x)}{L^2}$

$\frac{dy}{dx} = \frac{4e(L-2x)}{L^2}$

$\tan\theta \approx \theta = \frac{4e(L-L)}{L^2}$

$\theta = 0^\circ$

$\alpha = \theta + \theta$ ($x=0$, $x=\frac{L}{2}$)

$\alpha = \frac{4e}{L} + 0$

$$\boxed{\alpha = \frac{4e}{L}}$$

$$\boxed{\begin{array}{l} P_x = P_0 e^{-[Kx + \mu\alpha]} \text{ can be written as} \\ P_x = P_0[1 - (Kx + \mu\alpha)] \end{array}}$$

given stress k form me bhi calculate hai
$\sigma_x = \sigma_0 e^{-[Kx + \mu\alpha]}$

$$\boxed{\% \text{ loss of prestress} = \frac{P_0 - P_x}{P_0} \times 100}$$

Total loss of stress ($P_0 - P_x$); Initial stress (P_0)

Note:- First 4 loss of prestress occur in both pretensioned prestressed concrete & as well as post tensioned prestressed concrete.
But 5 & 6 loss of prestress occurs only in post tensioned prestressed concrete.
i.e (anchorage slip & friction loss).

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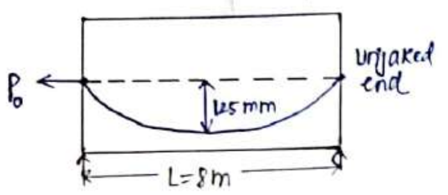
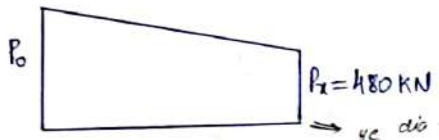
117:- Ass(Q3) $B = 280\ mm$ $\quad e = 125\ mm$ $\quad \sigma_x = 800\ Mpa$

$D = 450\ mm$ $\quad L = 8\ m$

$A = 600\ mm^2$ $\quad \mu = 0.5$

$k = 0.0025/m$.





$\left\{ \begin{array}{l} P_x = 800 \times 600 \times 10^{-3} \\ P_x = 480\ kN \end{array} \right\}$ $\left\{ \boxed{P = \sigma \times A} \right\}$

→ ye do stress k term me 6 bona sakte hain abhi ye force k term me hai.

$P_x = P_o e^{-[kx + \mu\alpha]}$

$480 = P_o e^{-[0.003 \times 8 + 0.5 \times 0.125]}$

$\boxed{P_o = 523.37\ KN}$

$\left\{ \alpha = \frac{8e}{L} = \frac{8 \times 0.125}{8} = 0.125 \right.$

% Loss of prestress $= \frac{\text{loss of prestressing force}}{\text{Initial prestressing force}} \times 100 = \frac{P_o - P_x}{P_o} \times 100$

$= \frac{43.37}{523.37} \times 100 = 8.29\%$

OR

$\sigma_x = \sigma_o e^{-[kx + \mu\alpha]}$

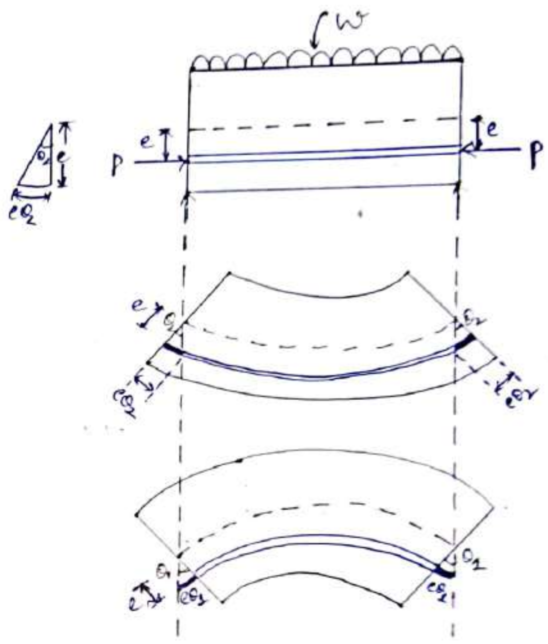
$800 = \sigma_o e^{-[0.003 \times 8 + 0.5 \times 0.125]}$

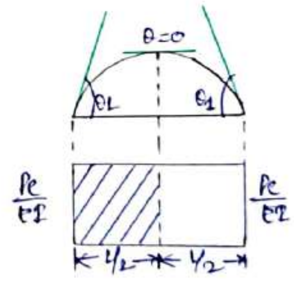
$\sigma_o = 872.28\ mpa$

% loss of prestress $= \frac{\text{loss of stress}}{\text{initial stress}} \times 100 = \frac{\Delta P}{\sigma_o} \times 100 = \frac{\sigma_o - \sigma_x}{\sigma_o} \times 100$

$= \frac{872.28 - 800}{872.28} \times 100 = 8.29\%$

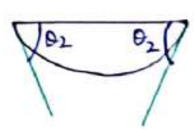
118:- Gain & loss of prestress due to External loading:-





$\theta_1 - 0 = \frac{Pe}{E_c I} \times \frac{L}{2}$

$\boxed{\theta_1 = \frac{PeL}{2 E_c I}}$



$\boxed{\theta_2 = \frac{Wl^3}{24 E_c I}}$

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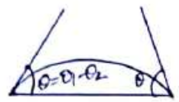
Case(I) If $\theta_1 > \theta_2$

$\theta_1 - \theta_2$

Net slope$(\theta) = \frac{PeL}{2EcI} - \frac{Wl^3}{24EcI}$

merging both fig



Net loss of elongation $= e\theta + e\theta = 2e\theta$

$$\boxed{\text{Net loss of prestress} = \frac{\overbrace{2e\theta}^{\Delta L}}{L} \times E_s}$$

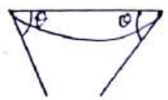
strain

Case(II) If $\theta_2 > \theta_1$ (This is good)

On merging we get

Net slope$(\theta) = \theta_2 - \theta_1$

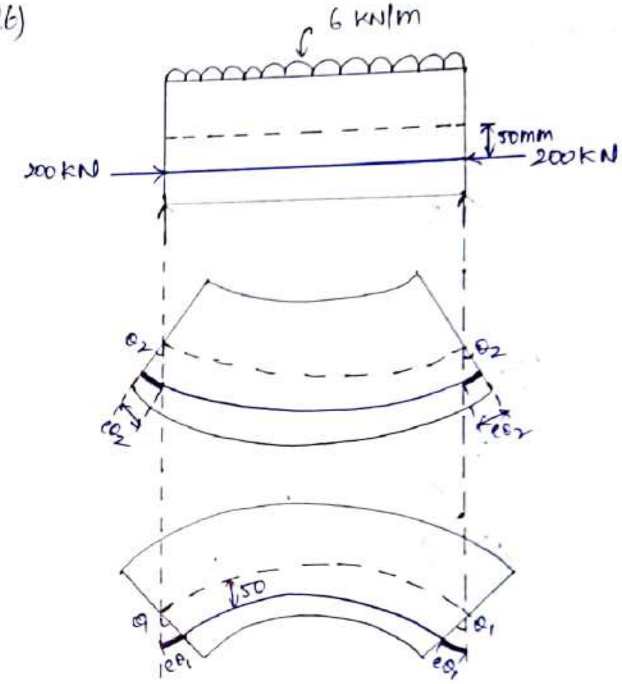
$$\theta = \frac{Wl^3}{24EcI} - \frac{PeL}{2EI}$$

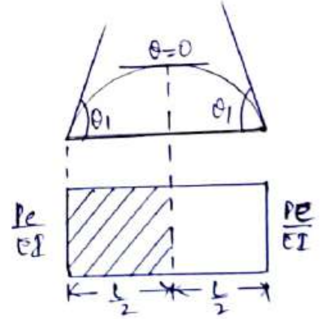


Net elongation of steel tendon $= e\theta + e\theta = 2e\theta$

$$\boxed{\text{Net gain/increase in prestress} = \frac{2e\theta}{L} \times E_s}$$

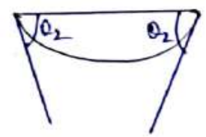
L19:- Q(ℓ)





$$\theta_1 - 0 = \frac{Pe}{EI} \times \frac{L}{2}$$

$$\boxed{\theta_1 = \frac{PeL}{2EI}}$$



$$\boxed{\theta_2 = \frac{Wl^3}{24EI}}$$

$\theta_1 = \frac{200 \times (50 \times 10^{-3}) \times 6}{2 \times 2 \times 10^5} = 1.5 \times 10^{-3}$

$\theta_2 = \frac{6 \times (6)^3}{24 \times [2 \times 10^4]} = 2.7 \times 10^{-3}$

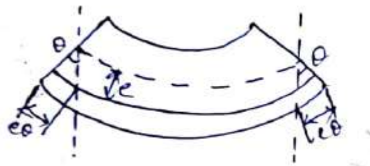
$\therefore \theta_2 > \theta_1$ means elongation

Net $\theta = \theta_2 - \theta_1$

$= 2.7 \times 10^{-3} - 1.5 \times 10^{-3}$

$\boxed{\theta = 1.2 \times 10^{-3}}$ radian

Net elongation :-



$\boxed{\text{Total elongation} = 2e\theta}$

Total elongation $= 2e\theta$

$= 2 \times 50 \times 1.2 \times 10^{-3}$

$= 0.12\,mm$

Additional

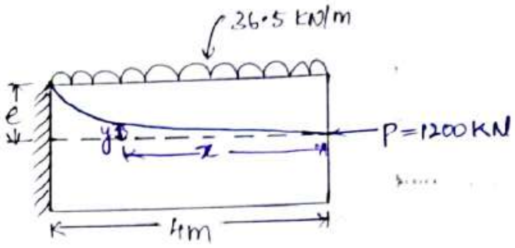
Q) what is the increase in stress. ?

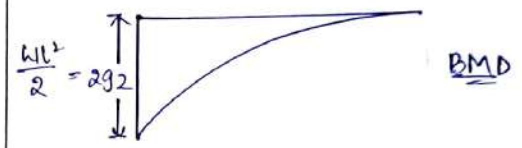
Sol When, $\Delta\sigma = \frac{\Delta L}{L} \times E_s$

$= \frac{0.12}{6000} \times 2 \times 10^5$

$= 4\ N/mm^2$

Lec: Ass (Q4)





BMD

For complete balancing

$Pe = \frac{wl^2}{2}$

$1200 \times e = \frac{36.5 \times (4)^2}{2}$

$e = 0.2433\,m \Rightarrow \boxed{e = 243.33\ mm}$

cable profile

$Py = \frac{wx^2}{2}$

$1200 \times y = \frac{36.5 \times x^2}{2}$

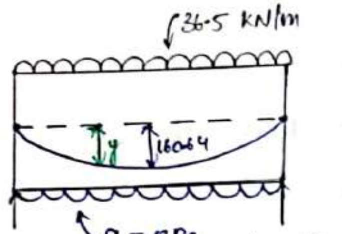
$\boxed{y = 0.0152 x^2}$ ⇒ x ki kisi b value par cable ki height nikal na hai to ye hoga use. For this problem

If $x = 4m$

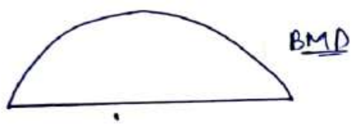
$y = 0.243\,m$

Ans Q5)

$L = 6.5m$



$q = \frac{8Pe}{L^2}$ (udl-1)



For complete Balancing condⁿ

$q = w$

$\frac{8Pe}{L^2} = 36.5$

$\frac{8 \times 1200 \times e}{(6.5)^2} = 36.5 \Rightarrow e = 0.1606m \Rightarrow \boxed{e = 160.64\ mm}$

Parabolic cable Profile

$y = \frac{4ex(L-x)}{L^2} = \frac{4 \times (0.1606) x [6.5-x]}{(6.5)^2}$

$\boxed{y = 0.0152\, x\,(6.5-x)}$

Q1:- Difference b/w pretensioned & posttensioned concrete :-

	Pretensioned Prestressed concrete	Post tensioned prestressed concrete
1)	20 to 25% loss of prestress occurs (total loss)	1) 10 to 15% loss of prestress occurs. (total loss)
2)	Minimum M40 grade of concrete is used (loss is more)	2) minimum M30 grade of concrete is used
3)	loss of prestress occurs due to (a) Elastic shortening of concrete (b) creep in concrete (c) shrinkage in concrete (d) Relaxation of steel tendons.	3) loss of prestress occurs due to (a) Elastic shortening [conditional]. (i) It is zero, in single wire stretching (ii) it is zero, All the tendons are stretched together. (iii) it is maximum in the tendon which is stretched first & it is zero in the tendon which is stretched at the last. when stretching is done in succession (one by one). (b) Creep in concrete (c) shrinkage in concrete (d) Relaxation of steel tendon (e) Anchorage slip (f) friction loss
4)	Total No of losses are less as compare to post tensioned prestressed concrete but net amount/ total amount of loss is higher, i.e 20 to 25% in pretensioned prestressed concrete.	4) Total No of losses are more as compare to pretensioned concrete but total/net amount of loss is less i.e 10 to 15%.

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