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Name. Surendra Sharma Sub. Design of RCC structures

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S. No:	Date	Title	Page No.	Teacher's Sign / Remarks

Codes: → NBC 110: 1994

i. IS 456:2000, Fourth Revision

ii. SP 16 : (456: 1978) → interaction diagram (column design)

iii. IS 13920: 2016 → Ductile design & detailing (for Building design)

⇒ For seismic design:
IS 1893: 2016

[⇒ NBC 105: 2020] → only replaces iii. & IS 1893: 2016 (still need to refer IS 456: 2000)
↳ IS 13920

Books:

i. RCC Design (Limit State) → A. K. Jain

ii. RCC Design (Limit state) → Pillai & Menon.

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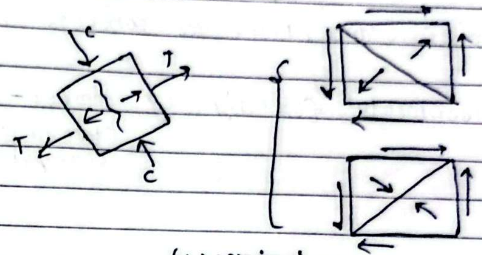
[Chapter 1] Introduction to Reinforced Concrete Structures

RCC: Reinforced cement concrete

only concrete



(weak in tension)

Reinforced by → Bamboo, Fibers, Rebars. ⇒ RCC

Advantages of RCC over Plain cement concrete:

1. Tensile strength is very high compared to RCC.
2. Bending strength is improved.

RCC
- → Concrete → strong in compression but weak in tension (Grade: M20, M25, M30....) IS456
- → Steel rebar
  ↳ equal strong both in tension and compression

M 20
↑ characteristic strength
(Not more than 5% failure)

Steel: Fe 250 → Mild steel
Fe415 → TOR steel
Fe 500 } High strength TMT steel
Fe 550 } (Thermo-Mechanically Treated)

3. Versatility (Min^m rebar kept in various building components to prevent cracks)
4. Durability
5. Fire resistance (More for concrete than ~~rebar~~ others structure timber & steel)
6. Environmental friendly (eg: timber – deforestation)
steel factors – harmful effluents (complete steel str.)

Disadvantages of RCC:
i. High self weight
ii. Low scrap value/salvage value.
iii. High con-struction time for on-site construction.
iv. Problem is caused due to demolition as huge amount of waste is generated.
v. Challenge in quality control (if not constructed properly strength is largely reduced eg. segregation, honey-comb, high w-c ratio)

Why steel is used as reinforcement in RCC structure)
→ Strong Bonding of steel with concrete is good
→ Coefficient of thermal expansion of concrete & steel are comparable, hence there is no problem of differential expansion and thermal stresses.
$\alpha_c \rightarrow 0.000011/°c$
$\alpha_s \rightarrow 0.000012/°c$.
→ high tensile strength
→ readily available.
→ economic

Design methods of RC structures:-
1. Working stress Design Method.
2. Ultimate load design Method.
3. Limit state design method.
4. Performance based design method

1. Working stress Design Method (Elastic theory)
→ Traditional method used for RC design
→ Assume: concrete & steel both as ~~et~~ complete elastic
→ Complete elastic stress-strain diagram used in design (Linear diagram)

[→ ~~FOS provided~~ Designed for permissible or yield stress].

$$\text{Permissible stress} = \frac{\text{Yield stress}}{\text{FOS}}$$

$$\frac{R}{F} \geq L$$

or, $\mu \cdot R \geq L$

R → Material resistance (strength)
L → working load
F → Factor of safety (>1)

$$\mu = \frac{1}{F} \Big] \rightarrow \text{Reciprocal of FOS.}$$

→ Designed for working load.
→ Strength of material is underestimated. Load used is working load or actual Load.

→ FOS → 3 for concrete
2 for steel
check.

2. Ultimate Load Design Method (Load Factor Design Method)

→ Material strength is not varified i.e. not underestimated

→ Load is underestimated i.e. increased by a factor during design.

→ In this method,

$$R \geq \lambda_i L_i$$

L_i – working load

λ_i – Factored load.

R = Material resistance. (strength of material)

λ = Factor

In resistance, can go upto ultimate load.

R → accounts non-linear stress-strain diagram.

→ Only strength is considered but serviceability is not considered.

→ not existed for long time.

3. Limit State design Method.

(In American code: Load Factor & Resistance Design Method (LFRD method)).

→ Limit state of collapse should be achieved for safety.

→ Limit state of serviceability should be achieved for comfort ~~achieved for~~ considered for deflections, cracks, appearance.

Limit state of collapse → considered for Flexure, compression, shear, Torsion

Limit state of serviceability → considered for deflections, cracks, vibration, appearance, durability, etc.

→ $\mu R \geq \sum_{i=1}^{n} \lambda_i L_i$

~~at 3 10~~ $[\mu \leq 1]$

Characteristic strength (f_{ck}): Strength for which chances of failure is less than 5%.

Characteristic Load (P_{ck}): Load ~~for which~~ on the structure, the probability of exceeding above it is more than 5%.

Design process & basis for design:

↓

(i) Idealization of structure for Analysis.

(ii) Estimation of Load [Self load, Earthquake load, wind load, Live load]

For self Load: Preliminary Design is necessary

(iii) Structural Analysis of idealized structure.
[output → internal forces/stresses induced].

(iv) Design of structural members.

(v) Material specification, detailed drawings.

Basis for design:

(i) Designed structure should be Stable, Strong and safe enough to carry all encountered load within its life span.

Safe → in stability & strength.

(ii) Serviceable throughout its life-span.
(no excessive deformation/ vibration/crack)

(iii) Economic

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Working Stress Method of Design

Basic assumption in working stress design:

(1) plane (cross) section before bending remain plane after bending.

(2) Bond between concrete & steel is perfect within elastic limit.

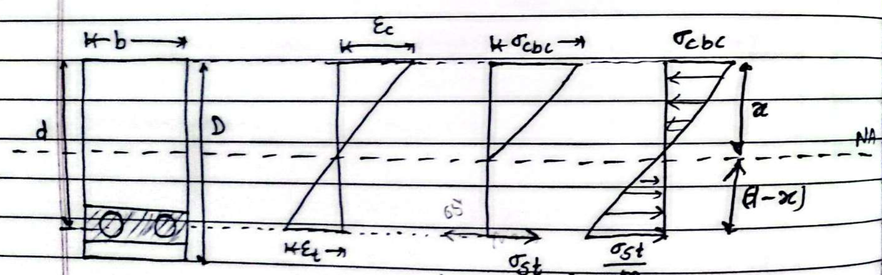
(3) Tensile strength of concrete is ignored.

(4) Concrete is elastic i.e. stress in concrete varies from zero at neutral axis to maximum value at extreme fiber.

(5) The modular ratio; m has value of $m = \frac{280}{3\sigma_{cbc}}$

where, σ_{cbc} = permissible compressive stress in concrete bending

[IS 456: Appendix B]



Strain diagram (Linear) | Stress diagram | Equivalent stress diagram

b → breadth of rectangular section

d → depth

D → Total depth.

x → neutral axis depth.

σ_{st} → permissible tensile stress in concrete steel

→ $m = \frac{E_s}{E_c}$

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

→ m = modular ratio

From Equivalent stress diagram, (similar triangle)

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

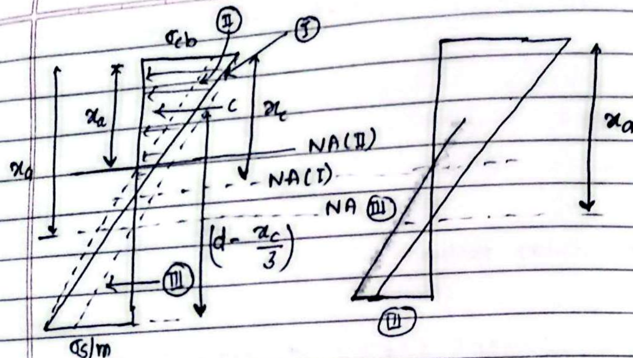
or $\frac{d}{x} = \frac{m\sigma_{cbc} + \sigma_{st}}{m\sigma_{cbc}}$

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

$\frac{x}{d}$ → Neutral axis depth ratio, denoted by (k)

i.e. $[x = kd]$

where, $\left[k = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}\right]$



Moment of resistance

Section	Neutral axis depth	MOR
(I) Balanced section (When permissible stress reaches at same time in steel & concrete)	$x_a = x_c$	$0.5\, b x_a \sigma_{cbc}(d - \frac{x_c}{3})$ or $A_{st} \cdot \sigma_{st}(d - \frac{x_c}{3})$
(II) Under-reinforced section (permissible stress in steel reached before permissible stress in concrete is reached)	$x_c > x_a$	$A_{st} \cdot \sigma_{st}(d - \frac{x_a}{3})$
(III) Over-reinforced section	$x_c < x_a$	$0.5\, b x_a \sigma_{cbc}(d - \frac{x_a}{3})$

pass → Under-reinforced section is best section because failure will be ductile failure (recommended failure) ~~& steel fails before con.~~

→ Brittle failure occurs in over-reinforced section

Actual Neutral axis depth:

For equilibrium:

$C = T$

C → Compression force; T → Tension force.

$$\frac{1}{2} * b x_a * \sigma_{cbc} = \sigma_{st} * A_{st}$$

$$\Rightarrow x_a = \frac{\sigma_{st} * A_{st}}{0.5\, b\, \sigma_{cbc}}$$

$x_a \rightarrow$ Actual NA depth
$x_c \rightarrow$ critical NA depth

(Q) A reinforced concrete beam 200 mm wide has an effective depth of 350mm. The permissible stress in concrete and steel are 5N/mm² and 140 N/mm² respectively. Find the depth of critical neutral axis and area of steel required.

→ Soln.-

$b = 200mm$

$d = 350mm$

$\sigma_{st} = 140 N/mm^2$

$\sigma_{cbc} = 5 N/mm^2$

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

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 * 5}$$

$C = T$

$$\Rightarrow \frac{1}{2} b x_a * \sigma_{cbc} = \sigma_{st} * A_{st} \quad \text{—(1)}$$

$x_c = k_c * d$

$k_c = \frac{m\sigma_c b_c}{m\sigma_c b_c + \sigma_{st}}$

$m = \frac{280}{3\sigma_c b_c} = \frac{280}{3*5} = 18.67$

We have,

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

or, $\frac{x_c}{350} = \frac{18.67*5}{18.67*5+140}$

$[x_c = 140\ mm]$

For area of steel required,

$C = T$

or, $\frac{1}{2} b \cdot x_c \cdot \sigma_{cbc} = A_{st} \cdot \sigma_{st}$ $[x_a = x_c]$

or, $\frac{1}{2} * 200*140*5 = A_{st} * 140$

$\Rightarrow [A_{st} = 500mm^2]$.

i. Balanced section

$x_a = x_c$

M.R. $= C * z = 0.5 b x_c \sigma_{cbc}\left(d - \frac{x_c}{3}\right)$

(Moment of Resistance) $= T * z = \sigma_{st} A_{st}\left(d - \frac{x_c}{3}\right)$

$\because C = T$

$\Rightarrow \frac{1}{2}\sigma_{cbc} * b \cdot x_c = \sigma_{st} * A_{st}$

$\Rightarrow A_{st} = \ldots$

If $A_{st} < (A_{st})_{balanced}$ $\rightarrow$ under reinforced section

If $A_{st} > (A_{st})_{balanced\ section}$ $\rightarrow$ over-reinforced section.

- Neutral axis depth

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

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

where, $m = \frac{E_s}{E_c} = \frac{280}{3\sigma_{cbc}}$

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

$\therefore k_c = \frac{x_c}{d} = \frac{280/3}{\frac{280}{3} + \sigma_{st}} = \frac{280}{280 + 3\sigma_{st}}$

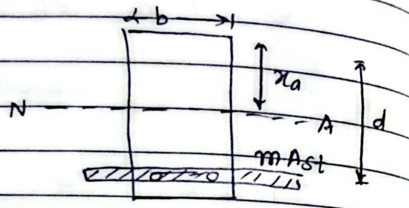
* Under-reinforced and over-reinforced sections:

In any case
Moment of compression area about Neutral axis = Moment of Tension member about neutral axis

i.e. $b x_a \cdot \frac{x_a}{2} = m * A_{st} * (d - x_a)$

m — used to convert steel area to equ. area of concrete

$\Rightarrow x_a = \ldots$



2 cases:

Case (i) $x_a < x_c$

([illegible] (rebar) [illegible] NA [illegible] ⇒ under-reinforced)

$m \rightarrow$ modular ratio.

[$x_a < x_c \Rightarrow$ Under-reinforced section].

$\therefore MR = T * Z$

$= \sigma_{st} \cdot A_{st} * \left(d - \frac{x_a}{3}\right)$

Case (ii) $x_a > x_c$ [Over-reinforced section ⇒ not-recommended].

([illegible] (rebar) [illegible] NA [illegible] i.e. $x_a > x_c$].

Overreinforced
→ Not-recommended
→ Failure will be brittle.
→ Not used in design

$M.R. = C * Z$

$= \frac{1}{2} \sigma_{cbc} * b x \left(d - \frac{x_a}{3}\right)$

Design of singly reinforced Rectangular section

For balanced section:

$M.R. = 0.5 b \cdot x_c \cdot \sigma_{cbc} \cdot \left(d - \frac{x_c}{3}\right)$

$= 0.5 b \sigma_{cbc} * k_c \cdot d \left(d - \frac{k_c \cdot d}{3}\right)$

$= 0.5 \sigma_{cbc} k_c \left(1 - \frac{k_c}{3}\right) bd^2$

$\therefore M.R. = R * bd^2$

where, $R = 0.5 \sigma_{cbc} k_c \left(1 - \frac{k_c}{3}\right)$

↳ depends on grade of steel & grade of concrete

R → Relative bending moment of section

↳ a constant for given grade of steel & concrete

This formula can be used for member section sizing.

Further,

$R = 0.5 \sigma_{cbc} K_c \cdot J_c$

where, $J_c = \left(1 - \frac{k_c}{3}\right)$

↳ Lever arm ratio

$$\boxed{K_c = \frac{280}{280 + 3\sigma_{cbc}}}$$

Design Steps:

1. Calculate maximum service/working moment.
2. Select concrete grade & steel grade and corresponding σ_{cbc} & σ_{st} from Table 21 & 22 of IS 456: 2000.
3. Calculate $k_c = \dfrac{280}{280 + 3\sigma_{st}}$
4. Calculate $R = 0.5\,\sigma_{cbc}\,k_c\left(1 - \dfrac{k_c}{3}\right)$
5. Assuming b, calculate d from $M = R \cdot bd^2$

   ($\frac{d}{b}$ ratio can also be assumed for design)

6. Calculate A_{st} from

$$M = \sigma_{st} A_{st}\left(d - \frac{x_c}{3}\right)$$

$$= \sigma_{st} A_{st} \cdot d\left(1 - \frac{k_c}{3}\right)$$

rebars

7. Choose diameter (ϕ), Nos. & arrangements of rebars and then calculate actual A_{st}.
8. Calculate overall depth (D),

$$D = \underset{\substack{\uparrow \\ \text{effective depth}}}{d} + \underset{\substack{\uparrow \\ \text{clear cover}}}{c} + d_c$$

   $c \rightarrow$ clear cover to steel (rebar)

   $d_c \rightarrow$ distance from C.G. of rebar to extreme fiber of fiber.

   {Step. 7 बाट आएको हुन सक्ने overreinforced section लाई step 8 मा D बढाएर under-reinforced}

9. Round up D suitably to a round value.
10. Calculate actual/effective depth, $d = D - c - d_c$
11. Check for actual x_a from $b \cdot x_a \cdot \frac{x_a}{2} = m A_{st}(d - x_a)$

   {d change $\rightarrow$ x_a change but k_c same},

12. Assume, $x_a < x_c \rightarrow$ underreinforced section

   $[x_c = k_c \cdot d]$

   If not go back to step 9. (increase D)

13. Check actual

   $M.R. = \sigma_{st} A_{st}(d - x_a/3) >$ design M.

Q. Calculate moment of resistance of a RC beam 250mm × 550mm. Reinforcement provided is 3-25mm ϕ rebar with clear cover 25mm. Take M20 concrete and ~~Fe250~~ Fe415 rebar.

(कति भन्ने, तिन overall depth)

→ Solution:-

Rectangular section:

$b = 250\,mm$

$D = 550\,mm$

clear cover, $c = 25\,mm$

3-25mm rebar in single layer.

Materials:

M20 concrete → $\sigma_{cbc} = 7\,N/mm^2$ (Table 21 of IS456)

Fe415 rebar,

↳ $\sigma_{st} = 230\,N/mm^2$ (Table 22 ; IS456)

Effective depth, $d = D - c - \phi/2$

$= 550 - 25 - 25/2$

$= 512.5\,mm$

Now, $k_c = \frac{280}{280 + 3\sigma_{st}} = \frac{280}{280 + 3 \times 230} = 0.289$

Also, $b \cdot x_a \cdot \frac{x_a}{2} = m A_{st} (d - x_a)$

or, $\frac{250}{2} \cdot x_a^2 = \frac{280}{3 \times 7} (512.5 - x_a) \times 3 \times \pi \times \frac{25^2}{4}$

or, $x_a = 215.860\,mm$

Now,

$k_c \cdot$ ... 280 $x_c = k_c \cdot d = 147.94\,mm$

Here,

$x_a > x_c$ → section is over-reinforced.

For over reinforced section,

∴ $M.R. = \frac{1}{2} \sigma_{cbc} \cdot b \cdot x_a \left(d - \frac{x_a}{3}\right)$

$= \frac{1}{2} \times 7 \times 250 \times 215.86 \left(512.5 - \frac{215.86}{3}\right)$

$= 83209353.03\ N\cdot mm$

$= 83.209 \times 10^6\ N\cdot mm$

$\simeq 83.209\ kN\cdot m$

$\left(c + \frac{\phi}{2}\right)$ → effective cover.

Q. A RC beam 300mm × 700mm is reinforced by 3-20ϕ rebars. The center of the bar are 50mm far from under side of beam. The maximum stress in concrete is to be $7\,N/mm^2$ and in steel is to be $190\,N/mm^2$. Find the magnitude of UDL that the beam of span 6m can carry safely. Take $\gamma_{concrete} = 25\,kN/m^3$

(If not mentioned anything ↓ simply supported)

→ Solution:-

$b = 300\,mm$

$D = 700\,mm$

Rebar → 3-20ϕ

Effective cover, $c_e = 50\,mm$

$\sigma_{cbc} = 7\,N/mm^2$

$\sigma_{st} = 190\,N/mm^2$

$\gamma_c = 25\,N/mm^3$

Effective depth, $d = D - C_e$
$= 700 - 50$
$= 650 mm$

$$k_c = \frac{280}{280 + 3\sigma_{st}} = \frac{280}{280 + 3*190} = 0.33$$

$\therefore x_c = k_c \cdot d = 0.33 * 650 = 214.12 mm.$

Also,

$$b \cdot x_a \cdot \frac{x_a}{2} = m\, A_{st}\,(d - x_a)$$

Or $300 * \frac{x_a^2}{2} = \frac{280}{3*7} * 3 * \frac{\pi * 20^3}{4} (650 - x_a)$

$\Rightarrow x_a = 195.196 mm$

$\therefore x_a < x_c$ (under reinforced section)

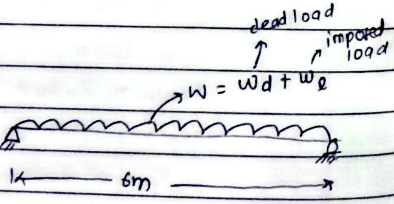
Now, Moment of resistance

$$MR = \sigma_{st} \cdot A_{st} * \left(d - \frac{x_a}{3}\right)$$

$$= 190 * 3 * \frac{\pi * 20^2}{4} \left(650 - \frac{195.196}{3}\right)$$

$= 104744707.7$ Nmm

≈ 104.745 kNm



$104.744 = \frac{w * 6^2}{8}$

$\Rightarrow \boxed{w = 23.28 \text{ kN/m}}$

Also, self-weight of beam,

$w_d = 0.3 * 0.7 * 25 = 5.25$ kN/m

Thus,

Additional imposed load (Live load), $w_l = w - w_d$
$= 23.28 - 5.25$
$= 18.03$ kN/m

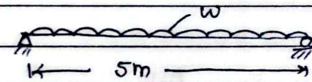
Q. Design a rectangular section of RC beam for a pedestrian crossing of span 5m have to carry a UDL of 25kN/m (excluding self wt.). Take M20 concrete and Fe415 rebar.

(self wt. consider गर्नलाई assume 10-15% excess of load on the beam)

→ Solution:-

Let us increase the UDL by 20% to incorporate the self-weight. So,

$w = 25 * 1.2 = 30$ kN/m



Now,

Working/Service Moment (M) (for UDL) $= \frac{wl^2}{8} = \frac{30 * 5^2}{8} = 93.75$ kNm
$= 93.75 * 10^6$ N-mm

From Table 21 of IS456:2000,
$\sigma_{cbc} = 7 N/mm^2$ for M20 grade concrete

From Table 22 of IS456:2000,
$\sigma_{st} = 230 N/mm^2$ for Fe415 grade steel.

Now,

$$K_c = \frac{280}{280+3\sigma_{st}}$$

$$= \frac{280}{280+3*230}$$

$$= 0.289$$

$$R = 0.5\sigma_{cbc} K_c\left(1 - \frac{K_c}{3}\right)$$

$$= 0.5*7*0.289\left(1 - \frac{0.289}{3}\right)$$

$$= 0.914$$

Let us assume $b = 300mm$

Then, we have

$$M = R.bd^2$$

or, $93.75*10^6 = 0.914*300*d^2$

$\Rightarrow d = 584.725mm$

$\approx 585mm$

Also,

$$M = \sigma_{st} A_{st} d\left(1 - \frac{K_c}{3}\right)$$

or, $93.75*10^6 = 230*A_{st}*585\left(1 - \frac{0.289}{3}\right)$

$\Rightarrow A_{st} = 771.044mm^2$.

Let us provide 3 rebars of dia. ϕ mm

$$\frac{3*\pi*\phi^2}{4} = 771.044$$

$\Rightarrow \phi = 18.09mm$

Let us adopt 3-20ϕ rebars.

Providing clear cover of 40mm.

Overall depth (D) $= d + c + d_c$

$$= 585 + 40 + \frac{20}{2}$$

$$= 635mm$$

Let us adopt $D = 700mm$.

Actual/effective depth; $d = D - c - d_c = 700 - 40 - \frac{20}{2}$

$= 650mm$.

For x_a,

$$b.x_a.\frac{x_a}{2} = m.A_{st}.(d - x_a)$$

or, $300*\frac{x_a^2}{2} = \frac{280}{3*7}*3*\pi*\frac{20^2}{4}*(650 - x_a)$

$\Rightarrow x_a = 195.196mm$.

Now,

$x_c = K_c.d = 0.289*650 = 187.85mm$

$x_a > x_c \rightarrow$ over-reinforced section.

Redesign

Adopt $D = 800mm$

$d = 800 - 40 - \frac{20}{2} = 750mm$

$$300*\frac{x_a^2}{2} = \frac{280}{3*7}*3*\pi*\frac{20^2}{4}*(750 - x_a)$$

$\Rightarrow x_a = 212.25mm$.

$x_c = k_c \cdot d = 0.289 * 750 = 216.75 mm$

$x_a < x_c \rightarrow$ under-reinforced section

(Okay)

Check:

Actual M: $= \sigma_{st} \cdot A_{st} \cdot \left(d - \frac{x_a}{3}\right)$

$= 230 * \frac{3 * \pi * 20^2}{4} \left(750 - \frac{212.25}{3}\right)$

$= 147240949.9$ Nmm

$= 147.24$ kNm $>$ Design M ($= 93.75$ kNm)

Okay

Ch-3

Limit state Method of design

(3.1) Safety and serviceability requirement and different limit state of structure.

Safety and serviceability Requirement for structures.

Safety requirement → no collapse

Serviceability → No excessive deflection & deformation, cracks.

* Limit state and different limit state considered design :

In Limit state design, generally two design steps.

(1) Limit state of safety collapse (or strength)
↳ structure are designed for design load

(2) Limit state of serviceability

design load & design strength ⇒ obtained by applying FOS

• Design strength: (S_d)
↳ calculated using characteristic strength & partial safety factor.

$S_d = \dfrac{S_{ck}}{\gamma_m}$

S_{ck} → characteristic strength of material

γ_m → partial safety factor for material

• γ_m : depend on
↳ variation in workmanship
↳ codes (IS, AS, BS, ..)
↳ materials.

→ IS 456, Cl. 36.4.2

$\gamma_m = 1.5$ → concrete
$= 1.15$ → steel

• For design load (w_d)

$w_d = w_{ck} * \gamma_f$

w_{ck} ↓ characteristic load

IS456, Table 18

For different load combination:-

• Limit state of collapse:

(1) (DL + LL) * 1.5 ← partial safety factor

(2) (DL ± WL) * 1.5
or
0.9DL ± 1.5WL

(3) (DL + LL ± WL) * 1.2

WL → wind load (or earthquake load)

DL → Dead load

± → WL can act from either direction

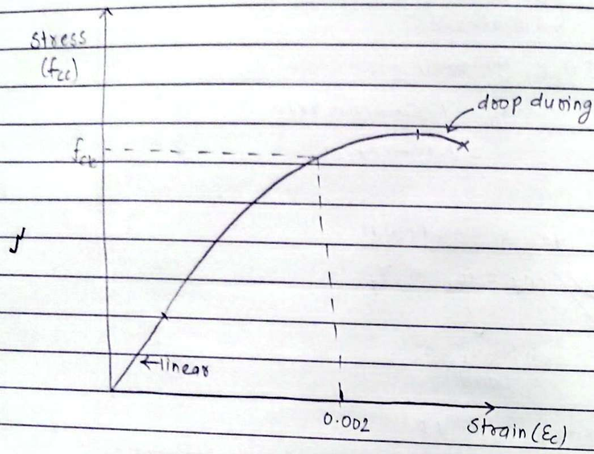
• Limit state of Serviceability

(i) (DL + LL) * 1

(ii) (DL ± WL) * 1

(iii) (DL + 0.8 LL ± 0.8 WL)

Idealized stress-strain curve for concrete & steel:



(Experimental curve)
Fig: stress-strain curve for concrete. (from lab test)

stress corresponding to $\varepsilon_c = 0.002$ is f_{ck} (characteristic stress)

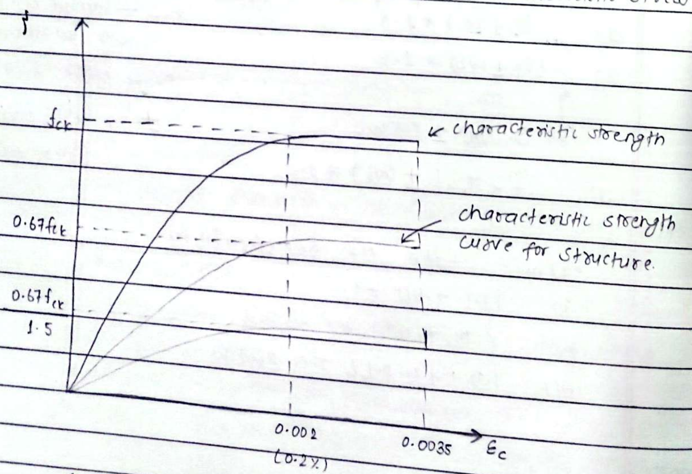


fig: Idealized stress-strain curve (concrete)

IS456

Q. Why $0.67 f_{ck}$ for structure?

→ for safety

→ to consider size factor

(When using cube as a sample we get 100% strength which gets reduced by 20% when sample is cylinder

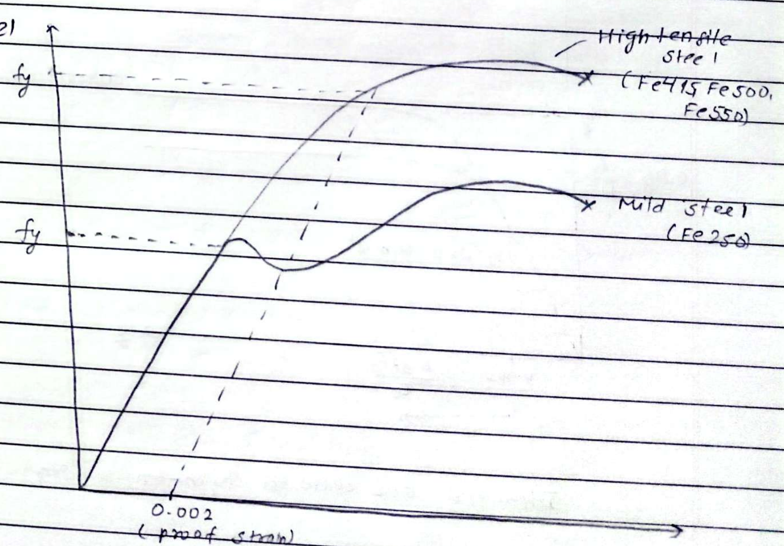
$f_{cu} = 0.8 f_{ck}$ ← cube.
cylinder ↗

The actual structure is longer and to compensate size & shape, factor $0.67 f_{ck}$ is used for structure)

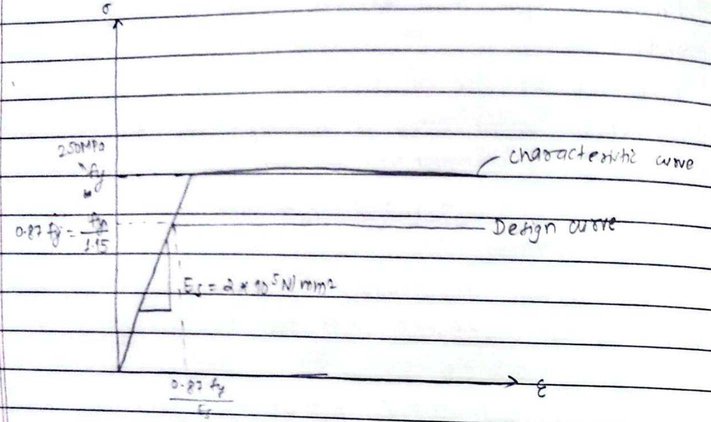
or $0.83 f_{cu}$

$(0.83 \times 0.8 \times f_{ck} = 0.67 f_{ck})$

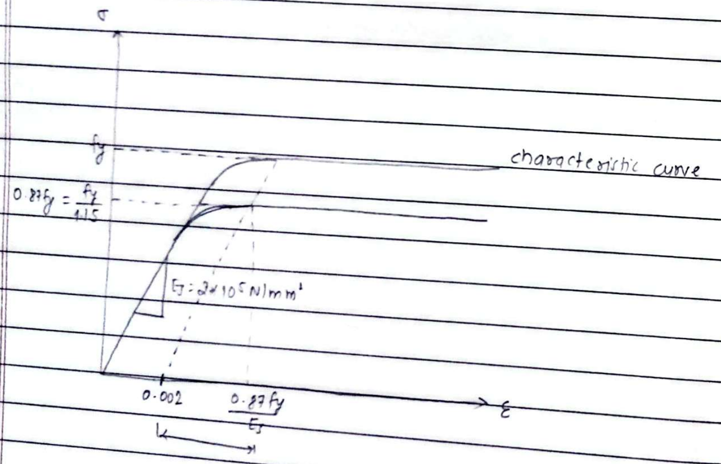
Steel



σ-ε curve for steel
(experimental curve)



Idealized σ-ε Curve for Mild steel



Idealized σ-ε curve for high-tensile steel

Ch. 4 Limit state of collapse in Flexure

Flexure behaviour in reinforced concrete:

→ When beam is subjected to transverse load, it bends causing compression in top & tension at bottom (simply supported)

→ When loads are less, no effect but when load increases it causes cracks at tensile region.

→ Tensile strength of concrete is not considered in design

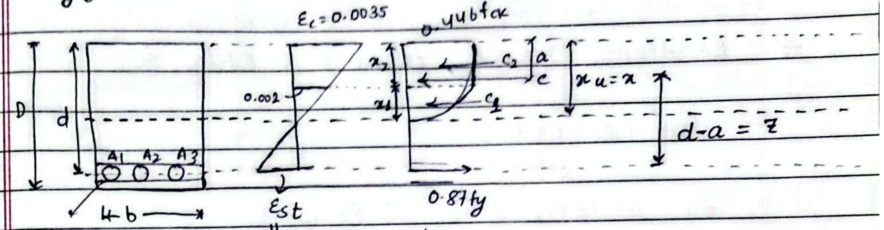
→ Increase in load on beam causes top compressive part to yield ~~(in steel)~~ prior to rebar/steel reaches its yielding stress causing crushing of concrete. (in over-reinforced section)

→ In under-reinforced steel, steel reaches yielding before concrete causing failure in yielding of steel (desired case in RC beam design)

→ concrete elastoplastic → whitney

* Basic assumptions in design of RC beam for limit state of collapse: (Cl. 38.1) → assumptions for limit state of collapse: Flexure
IS 456:2000

* Simply Reinforced RC section



$\varepsilon_{st} = 0.002 + \frac{0.87 f_y}{E_s}$

Strain diagram

Stress-diagram

(x_u = N.A. depth of balanced section i.e. ultimate)

From strain diagram,

$$\frac{x_1}{0.002} = \frac{x}{0.0035}$$

$$\Rightarrow \left[x_1 = \frac{4}{7} x\right]$$

$$\& \quad x - x_1 = x_2 \Rightarrow \left[x_2 = x - \frac{4x}{7} = \frac{3}{7} x\right].$$

$$C_1 = \frac{2}{3}(x_1 * b * 0.446 f_{ck}) = 0.17 f_{ck} \cdot x \cdot b$$

$$C_2 = b x_2 * 0.446 f_{ck} = 0.19 f_{ck} \cdot x \cdot b$$

$$\left[\therefore C = C_1 + C_2 = 0.36 f_{ck} \cdot x b\right]$$

Now,

$$C * a = C_2 * \frac{x_2}{2} + C_1 * \left(x_2 + \frac{3}{8} x_1\right)$$

$$\text{or,} \quad a = \frac{0.19 f_{ck} \cdot x \cdot b * \frac{3}{7} \cdot \frac{x}{2} + 0.17 f_{ck} \cdot x \cdot b * \left(\frac{3}{7} x + \frac{3}{8} \cdot \frac{4}{7} x\right)}{0.36 f_{ck} \cdot x \cdot b}$$

$$\left[a = \frac{5}{12} x\right]$$

→ Resultant compression force = $0.36 f_{ck} \cdot b \cdot x_u$

→ Resultant Tension force = $0.87 f_y \cdot A_{st}$

For equilibrium,

$$C = T$$

i.e. $0.36 f_{ck} \cdot b \cdot x_u = 0.87 f_y \cdot A_{st}$

$$\left[\therefore x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} \cdot b}\right].$$

• From strain diagram,

$$\frac{x_u}{0.0035} = \frac{d - x_u}{\varepsilon_{st}} \left(= \frac{d - x_u}{0.002 + \frac{0.87 f_y}{E_s}} \right)$$

$$\left(\frac{d - x_u}{x_u} = \frac{d}{x_u} - 1\right)$$

$$\Rightarrow \frac{x_u}{d} = \frac{0.0035}{0.0035 + \left(0.002 + \frac{0.87 f_y}{E_s}\right)}$$

$x_{u,l}$

$$\therefore \boxed{\frac{x_u}{d} = \frac{0.0035}{0.0055 + \frac{0.87 f_y}{E_s}}}$$

x_u = Balanced and limiting section
(Limit state will also account the limit for under reinforced section design)
Balanced section depth will be limiting value for it.

• Based on this expression, the limiting depth of neutral axis for different grades of steel are as follows:-

f_y	$x_{u,l}/d$
250 →	0.53
415 →	0.48
500 →	0.46

→ ~~(d)~~ From stress-diagram,

$$[z = d - a = d - 0.42 x_u]$$

Moment resistance capacity:

• From concrete side (i.e. assuming concrete as yielding)

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

(Moment of resistance)$_{concrete}$

• From steel side (i.e. assuming steel as yielding)

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

* Cases of failure of RC section in flexure/Bending:-

⇒ 3 cases arise:-

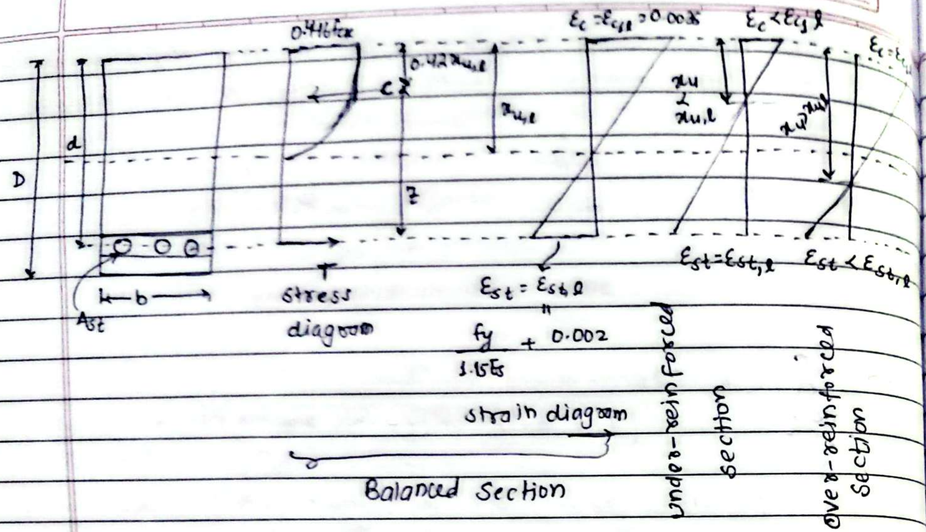
(i) Underreinforced section

→ Reinforcement undergo limit state first

→ Reinforcement will yield first

→ Reinforcement will fail first which will be ductile failure which will be recommended for RC design

→ The name → 'underreinforced section' → because reinforcement for the given section is less than reinforcement for a balanced section for some given section.



(i) contd

Under-reinforced section {steel yield मा पुग्यो but concrete पुगीसकेको छैन}
↓
NA माथी सर्छ

$x_u < x_{u,l}$

$\Rightarrow \frac{x_u}{d} < \frac{x_{u,l}}{d}$

→ x_u can be determined by equating Tension & compression Force.

i.e. $C = T$

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

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

And MOR in this case is given by,

$M_u = T * Z$

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

$M_u = 0.87 f_y A_{st} d \left(1 - 0.42 * \frac{0.87 f_y A_{st}}{0.36 f_{ck} b \cdot d}\right)$

$\frac{0.87 \times 0.42}{0.36} \approx 1.015$

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

or, $\boxed{M_u = R \cdot bd^2}$

where,

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

(ii) Balanced section :-

→ Failure cannot be predicted exactly i.e. brittle or ductile / reinforcement or concrete

→ The name → 'Balanced' → both reinforcement & concrete in balanced state and both will undergo simultaneous failure.

→ Both Reinforcement and concrete in {yield.} attain limit state simultaneously.

i.e. $x_u = x_{u,l}$ {Cl. 38.1 (F) Pg. 70}

$\frac{x_u}{d} = \frac{x_{u,l}}{d} = 0.53$ for $f_y = 250$

$= 0.48$ for $f_y = 415$

$= 0.43$ for $f_y = 500$

and. $M.R. = M_{u,l} = T * Z = C * Z$

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

or, $M_{u,l} = 0.36 f_{ck} x_{u,l} * b \{d - 0.42 x_{u,l}\}$. (C)

or $M_{u,l} = 0.36 f_{ck} * \frac{x_{u,l}}{d} \left\{1 - \frac{0.42 x_{u,l}}{d}\right\} bd^2$

$\therefore M_{u,l} = R_{lim} \cdot bd^2$

where, $R_{lim} = 0.36 f_{ck} \frac{x_{u,l}}{d} \left\{ 1 - 0.42 \frac{x_{u,l}}{d} \right\}$.

For particular grade of steel,

$R_{lim} = 0.149 f_{ck}$, for $f_y = 250$

$R_{lim} = 0.138 f_{ck}$, for $f_y = 415$

$R_{lim} = 0.133 f_{ck}$, for $f_y = 500$

(iii) Over-reinforced section

→ concrete will undergo limit state of failure first i.e. yield first

→ Concrete failure will be brittle hence avoided in RCC design.

→ The name → 'over reinforced' → Reinforced of given section is more than presenting balanced section of same given section.

i.e.

$x_u > x_{u,l}$

→ should be avoided in design

→ section should be redesigned with other dimensions.

N.A. check,

$$M_{u,l} = 0.36 f_{ck} * \frac{x_{u,l}}{d} \left\{ 1 - \frac{0.42 x_u}{d} \right\} bd^2$$

$\frac{\text{Span}}{\text{depth}} = 10-15$ } यदि छ simply supported generally safe

Design of singly reinforced Rectangular sections:

(i) Determining cross-section dimension b & d (or D)

(ii) Select/assume grade of steel & concrete f_y or f_{ck}.

(iii) Calculate design ultimate moment. $\left\{ M = \frac{wl^2}{8};\ w = w_d + w_L \right\}$.

$M_u = 1.5 * \text{service moment}$

(iv) Calculate limiting moment capacity of section.

$M_{u,l} = R_l bd^2$

$R_l = 0.149 f_{ck}$ for $f_y = 250$MPa

$= 0.138 f_{ck}$ for $f_y = 415$MPa

$= 0.133 f_{ck}$ for $f_y = 500$MPa

(v) Check $M_u < M_{u,l}$ → If yes,

Design rebar as under-reinforced sections

(if not increase section to make $M_u < M_{u,l}$)

(vi) Find area of steel required

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

Q. Design a rectangular RC beam section with 8m effective span which is subjected to a dead load of 12 kN/m and live load of 16 kN/m. Use M20 concrete and Fe415 rebar. Sketch detail of reinforcement.

→ Soln:-

$l = 8m$

$w_d = 12\,kN/m$

$w_l = 16\,kN/m$

M20 concrete & Fe415 rebar

total load, $w = 12+16 = 28\,kN/m$

∴ Total factored load, $w_u = 1.5 * 28 = 42\,kN/m$

Maximum ultimate Moment,

$$M_u = \frac{w_u l^2}{8} = \frac{42 * 8^2}{8} = 336\,kN\text{-}m.$$

Using,

$M_{u,l} = 0.138 f_{ck} b d^2$ [For Fe 415]

or, $336 * 10^6 = 0.138 * 20 * b * (2b)^2$ {using, $d = 2b$}

On solving,

$b = 312.22\,mm$

$\approx 350\,mm$ (say)

& $d = 2b = 2 * 312.22 = 624.44\,mm$

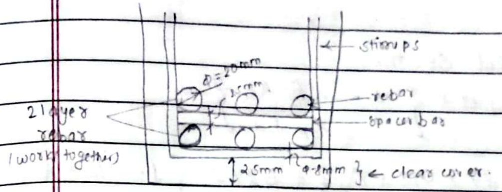
Using/assuming 20mm rebar in 2 layer with 25mm spacer bar, 8mm ϕ rebar stirrups and 25mm clear cover

center of rebar to centroid of spacer bar (mid)

$$\text{total depth, } D = 624.44 + \frac{25}{2} + 20 + 8 + 25$$

$= 689.934\,mm$

$\approx 750\,mm$ (adopt)



$$\therefore \text{actual, } d = 750 - \frac{25}{2} - 20 - 8 - 25 = 684.5\,mm$$

Now,

Self weight of beam (w_s) = $\underset{(b)}{0.35} * \underset{(D)}{0.75} * \underset{\text{per metre}}{1} * 25$

$= 6.5625\,kN/m$

∴ Total design moment including self weight,

$$M_u = 336 + \underbrace{\frac{(1.5 * 6.5625) * 8^2}{8}}_{\text{Factored moment due to self weight}}$$

$= 414.75\,kN\text{-}m.$

Now,

Actual limiting moment of the section

$$M_{u,l} = 0.138 * \underset{f_{ck}}{20} * \underset{b}{350} * \underset{d^2}{(684.5)^2}$$

$= 452.6 * 10^6\,N\text{-}mm$

$= 452.6\,kN\text{-}m$

∴ $M_{u,l} = 452.6\,kN\text{-}m > M_u = 414.75\,kN\text{-}m$

→ Hence design section as singly reinforced section.

Now,

For under-reinforced section,

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

$$\Rightarrow 414.75 \times 10^6 = 0.87 \times 415 \times A_{st} \times 684.5 \times \left\{1 - \frac{415 \times A_{st}}{20 \times 350 \times 684.5}\right\}$$

$$\therefore A_{st} = 2037.92\ mm^2$$

Rebar

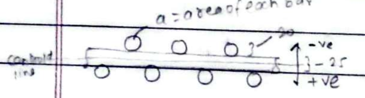
$\therefore$ No of 20mm rebar

$$= \frac{2037.92}{\pi \times (10)^2} = 6.49 \approx 7 \text{ (adopt)}$$

$\therefore$ Provide 7-20ϕ rebar in 2 layers i.e. 4 bars in outer layer and 3 bar in inner layer separated by 25mm spacer bar.

$$\therefore \text{Actual } A_{st} = 7 \times \pi \times 10^2 = 2199.115\ mm^2.$$

$$\text{Actual } d = 684.5 + 3.214 = 687.714\ mm$$



$$\frac{-3a \times \left(\frac{20}{2} + \frac{25}{2}\right) + 4a \times \left(\frac{20}{2} + \frac{25}{2}\right)}{7a}$$

$$= \frac{22.5}{7}$$

$= 3.214$ mm

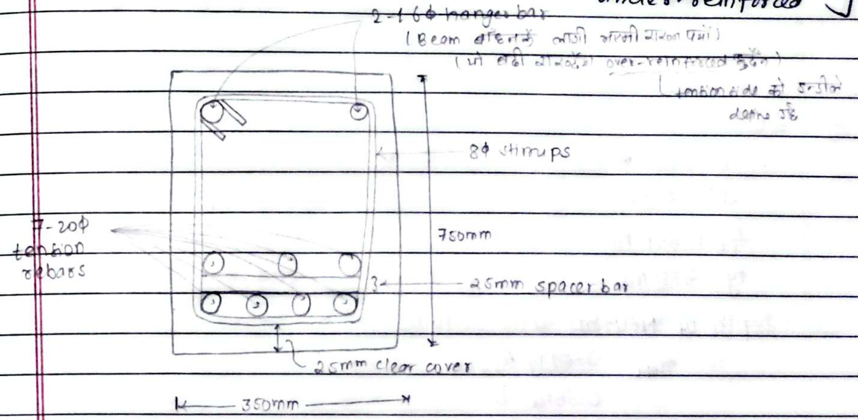
(towards lower rebar from center of spacer bar)

Revised $M_u \rightarrow M_{uR} = 0.87 \times 415 \times 2199.115 \times 687.714 \times \left\{1 - \frac{415 \times 2199.115}{20 \times 350 \times 687.714}\right\}$

$= 442.5 \times 10^6$ N-mm

$= 442.5$ kN-m $> M_u$ {safe for design load}

$= 442.5$ kN-m $< M_{u,l}$ {confirms section is under-reinforced}



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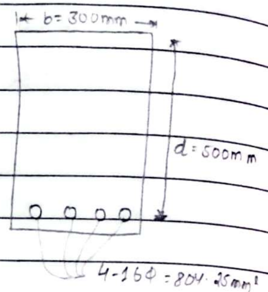
(Q) Find the moment resisting capacity of section shown in figure below.

M20 concrete & Fe 415 rebar



→ Soln:-

$b = 300mm$
$d = 500mm$
$f_{ck} = 20MPa$
$f_y = 415MPa$

Depth of Neutral axis (actual)

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

$$= \frac{0.87 * 415 * 804.25}{0.36 * 20 * 300}$$

$$= 134.43mm$$

Limiting Neutral axis depth, $x_{u,l} = 0.48d$ [for Fe 415].

$$= 0.48 * 500$$

$$= 240mm > x_u$$

Hence, Section is underreinforced.

$$\therefore M_{ul} = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{f_{ck} \cdot b d}\right]$$

$$= 0.87 * 415 * 804.25 * 500 \left[1 - \frac{415 * 804.25}{20 * 300 * 500}\right]$$

$$= 128.95 * 10^6 \text{ N-mm}$$

$$= 128.95 \text{ KNm}$$

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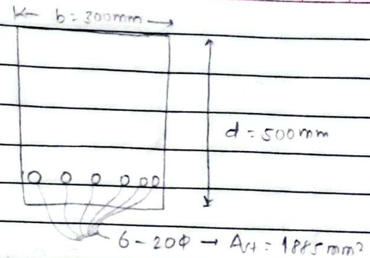
(Q) If,



$b = 300mm$
$d = 500mm$
$A_{st} = 1885mm^2$
$f_y = 415MPa$
$f_{ck} = 20MPa$

Depth of Neutral axis (actual)

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} \cdot b} = \frac{0.87 * 415 * 1885}{0.36 * 20 * 300} = 315.08mm$$

Limiting Neutral axis depth, $x_{u,l} = 0.48d = 0.48 * 2500$
$= 240mm < x_u$

Hence, Section is over-reinforced.

(8) $b = 300\,mm$

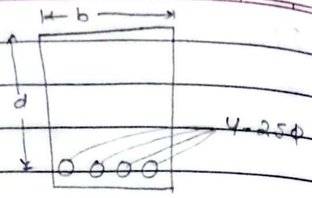
$d = 550\,mm$

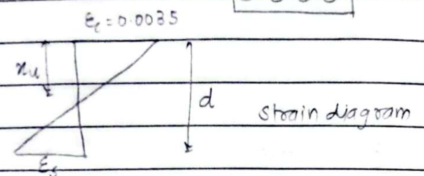
$A_{st} = \frac{4 \times \pi \times 25^2}{4}$

$= 1963.49\,mm^2$

$f_y = 415\,MPa$

$f_{ck} = 20\,MPa$





Calculate Moment Resisting capacity of RC-section shown.

→

$x_{u,l} = 0.48d = 0.48 \times 550 = 264\,mm$

Assuming, $x_u \le x_{u,l}$

$x_u = \frac{0.87 f_y A_{st}}{0.362 f_{ck} b} = \frac{0.87 \times 415 \times 1963.49}{0.362 \times 20 \times 300}$

over reinforced

$= 326.39\,mm$

As, $x_u > x_{u,l}$; steel will not yield;

the actual x_u will be obtained by strain compatibility.

For 1st trial,

assume: $x_u = \frac{264 + 326.4}{2} = 295.2\,mm$

From strain compatibility,

$\varepsilon_s = 0.0035\left(\frac{550}{295.2} - 1\right)$

$= 0.00302$

$\boxed{x_{u_1} \Rightarrow \varepsilon_s \Rightarrow f_s \Rightarrow x_{u_2}}$

compare x_{u_1} & x_{u_2}

Next

$x_{u_3} = \frac{x_{u_1} + x_{u_2}}{2} \Rightarrow \varepsilon_s \Rightarrow f_s \Rightarrow x_{u_4}$

Table 1

By ~~extr~~ interpolation

for, $\varepsilon_s = 0.00303$

$f_s = 351.8 + \left(\frac{360.9 - 351.8}{0.0038 - 0.00276}\right) \times (0.00303 - 0.00276)$

$= 354.2\,MPa$

For, $f_y = 415\,MPa$

Strain, ε_s	Stress (MPa) (f_s)
0	0
0.00144	288.7
0.00163	306.7
0.00192	324.8
0.00241	342.8
0.00276	351.8
≥ 0.0038	360.9

So, for $\varepsilon_s = 0.00303$, $f_s = 354.2\,MPa$

Not 0.87

$x_u = \frac{\cancel{0.87}\, f_s \times A_{st}}{0.362 f_{ck} \times b} = 320\,mm$

(didn't match with 295.2 mm)

Next trial

For 2nd trial,

assume, $x_u = \frac{320 + 295}{2} \simeq 308\,mm$

From strain compatibility,

$\varepsilon_s = 0.0035\left(\frac{550}{308} - 1\right)$

$= 0.00275$

$\therefore f_s = 342.8 + \left(\frac{351.8 - 342.8}{0.00276 - 0.00241}\right) \times (0.00275 - 0.00241)$

$= 351.5\,MPa$

$\therefore x_u = \frac{f_s A_{st}}{0.362 f_{ck} d} = 317.7\,mm$

$f_y = 349 MPa$

For 3rd trial,

$$x_u = \frac{317.7 + 308}{2} = 312.85 \approx 313 mm$$

From strain compatibility,

$$\varepsilon_s = 0.0035 \left(\frac{550}{313} - 1 \right)$$

$$= 2.65 \times 10^{-3}$$

$\therefore f_s = 348.97 MPa$ (by interpolation)

$\approx 349 MPa$

$$\therefore x_u = \frac{f_s A_{st}}{0.362 f_{ck} b} = \frac{348.97 \times 1963.5}{0.362 \times 20 \times 300} = 315.47 mm.$$

For 4th trial,

$$x_u = \frac{313 + 315.47}{2} = 314.23 mm$$

From

So, $x_u = 315 mm$ (approx.)

M.R. (from tension side) $= f_s A_{st} (d - 0.42 x_u)$

$= 349 \times 1963 (550 - 0.42 \times 315)$

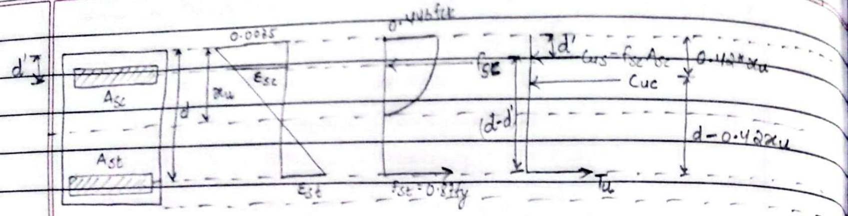
$= 286.16 kNm$

M.R. (from concrete side) $= 0.362 f_{ck} x_u b (d - 0.42 x_u)$

$= 0.362 \times 20 \times 315 \times 300 (550 - 0.42 \times 315)$

$= 285.78 kNm$

Doubly reinforced section:



For Neutral Axis depth; x_u

$$C_{uc} + C_{us} = T_u$$

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

$$\Rightarrow x_u = \frac{0.87 f_y A_{st} - (f_{sc} - 0.446 f_{ck}) A_{sc}}{0.36 f_{ck} \cdot b}$$

M.R. of section,

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

From strain diagram:

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

Values of f_{sc} (Table F, of SP16) (Fig. 3 Amd. J 456 : 2000)

	(d'/d) 0.05	0.10	0.15	0.20
Fe 250 (0.87fy)	217.5	217.5	217.5	217.5
Fe 415	355.1	351.9	342.4	329.2
Fe 500	423.9	411.3	395.1	370.3

Design steps:-

Given b, d, f_{ck}, f_y.

(i) Calculate $x_{u,l}$

(ii) Calculate $M_{u,l} = R_l * bd^2$ (as of single reinforced)

(iii) Check if $M_u \le M_{u,l} \rightarrow$ design as singly reinforced
If $M_u > M_{u,l} \rightarrow$ design as doubly reinforced

(iv) Calculate $\Delta M_u = M_u - M_{u,l}$

(v) Calculate, $A_{st_1} = \dfrac{M_{u,l}}{0.87 f_y (d - 0.42 x_{u,l})}$

(vi) Calculate, $A_{st_2} = \dfrac{\Delta M_u}{0.87 f_y (d - d')}$ (due to extra moment)

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

(vii) $A_{sc} = \dfrac{\Delta M_u}{(f_{sc} - 0.446 f_{ck})(d - d')} = \dfrac{0.87 f_y A_{st_2}}{(f_{sc} - 0.446 f_{ck})}$

(Q) Design a RC beam having cross section 300mm * 600mm with 8m effective span which is subjected to a dead load of 30KN/m and live load of 20KN/m. Use M20 concrete & Fe415 rebar. Sketch detail of reinforcement.

→ Soln:-

Beam section → 300 mm * 600mm

effective span, l = 8m

Self wt. of beam, W_s = 0.3*0.6* 25 = 4.5KN/m

∴ Total load, W = 30 + 20 + 4.5 = 54.5KN/m

∴ Total factored load, W_u = 1.5*54.5 = 81.75KN/m

∴ Max^m factored Moment,

$$M_u = \frac{81.75 \times 8^2}{8} = 654 KN m$$

Now, assuming 25mm rebar in 2 layers with 25mm spacer bar on tension side and 25mm rebar in single layer on compression side, 8mm ϕ stirrups and 25mm clear cover.

∴ effective depth, $d = 600 - 25 - 8 - 25 - \frac{25}{2}$

$= 529.5mm$

effective cover on compression side,

$$d' = 25 + \underbrace{\frac{25}{2}}_{centroid} + \underbrace{8}_{stirrups} = 45.5mm$$

$$\frac{d'}{d} = \frac{45.5}{529.5} = 0.086$$

Now, limiting M.R. for singly reinforced section

$M_{u,l} = 0.138 f_{ck} bd^2$

$= 0.138 * 20 * 300 * 529.5^2$

$= 232.146 * 10^6$ N.mm

$= 232.146$ KN.m

$< M_u$

→ hence, design section as doubly reinforced

Additional moment, $\Delta M_u = M_u - M_{u,l}$

$= 654 - 232.146$

$= 421.854$ KN-m

Now, for Ast

$$M_{u,l} = 0.87 f_y Ast_1 . d \left[1 - \frac{f_y Ast_1}{bdf_{ck}}\right]$$

$Ast_1 = 1513.547 mm^2$

For Ast_2,

$\Delta M_u = 0.87 f_y Ast_2 (d - d')$

⇒ $Ast_2 = 2414.07 mm^2$

Total tension steel,

$Ast = Ast_1 + Ast_2$

$= 3927.617 mm^2$

∴ No. of rebar $= \frac{3927.617}{\pi * (12.5)^2} = 8.0013$

≈ 9 No (adopt)

Now, for Asc

$\Delta M_u = (f_{sc} - 0.446 f_{ck}) A_{sc} (d - d')$

where, $\frac{d'}{d} = 0.086$

$$\therefore f_{sc} = 355 + \frac{(353-355)}{(0.10-0.05)}(0.086-0.05)$$

$$= 353.56\ N/mm^2$$

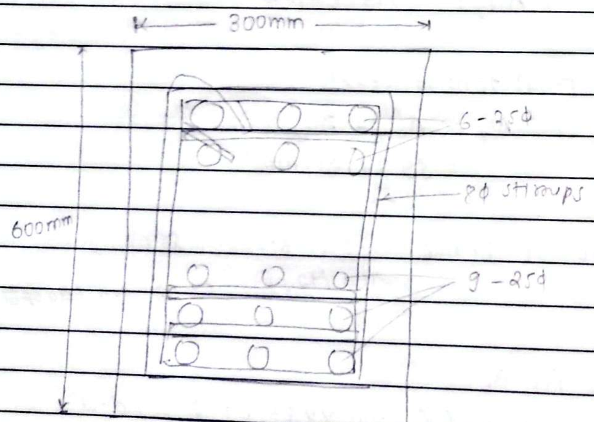
$$421.824\times 10^6 = (353.56 - 0.446\times 20) \times A_{sc} \times (529.5-45.5)$$

$$\Rightarrow A_{sc} = 2533.13\ mm^2$$

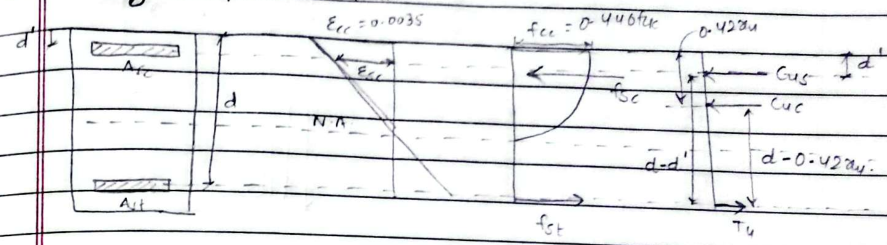
$\therefore$ No. of 25 mm rebar

$$= \frac{2533.13}{\pi \times (12.5)^2} = 5.16 \approx 6\ \text{Nos.}$$

$\therefore$ Provide 9-25mmϕ rebar in 3 layer (3 in each layer) on tension side and 6-25mmϕ rebars in 2 layer (3 in each layer) on compression side.



Doubly reinforced section:-



- For neutral axis depth, x_u

$$C_{uc} + C_{us} = T_u$$

$$\Rightarrow 0.36 f_{ck}\, x_u \cdot b + (f_{sc} - 0.446 f_{ck}) A_{sc} = f_{st} \cdot A_{st}$$

$$\therefore x_u = \frac{f_{st} A_{st} - (f_{sc} - 0.446 f_{ck})}{0.36 f_{ck} \cdot b}$$

[For under-reinforced or balanced section $f_{st} = 0.87 f_y$]

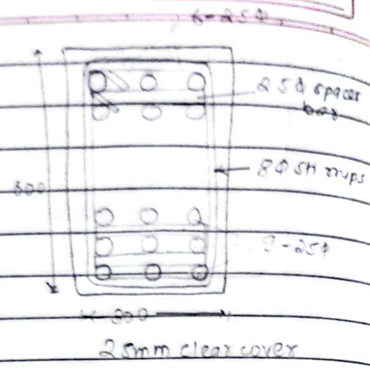
- $$\frac{\varepsilon_{sc}}{0.0035} = \frac{(x_u - d')}{x_u}$$

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

- $$\frac{\varepsilon_{st}}{0.0035} = \frac{d - x_u}{x_u}$$

$$\Rightarrow \varepsilon_{st} = 0.0035\left(\frac{d}{x_u} - 1\right)$$

Q. Prev qsn.



Find Moment of resistance.

(f_{ck} = 20MPa, f_y = 415 MPa)

→ Soln

$d' = 25 + 8 + 25 + \frac{25}{2} = 70.5\,mm$

$d = 600 - (25 + 8 + 25 \times 2 + \frac{25}{2}) = 504.5\,mm$

$b = 300\,mm$

1st trial

~~(Assuming balanced section)~~

Assume $x_u = x_{u,l} = 0.48d = \cancel{0.48 \times 600}\ 0.48 \times 504.5$

$= 242.16\,mm$

$\varepsilon_{sc} = 0.0035\left(1 - \frac{d'}{x_u}\right) = 0.0035\left(1 - \frac{70.5}{242.16}\right)$

$= 0.002481$

$\varepsilon_{st} = 0.0035\left(\frac{d}{x_u} - 1\right) = 0.0035\left(\frac{504.5}{242.16} - 1\right) = 1.08333 \times 0.0035$

$= 0.00379$

$\therefore f_{sc} = 342.8 + \frac{(351.8 - 342.8)}{(0.00276 - 0.00241)}(0.00248 - 0.00241)$

(Interpolating table for ε_s v/s f_s Fe 415)

$= 344.626\,MPa.$

$f_{st} = 351.8 + \frac{(360.9 - 351.8)}{(0.0038 - 0.00276)}(0.00379 - 0.00276)$

$= 360.813\,MPa$

$\therefore x_u = \frac{f_{st} \cdot A_{st} - (f_{sc} - 0.446 f_{ck}) A_{sc}}{0.36 f_{ck} \cdot b}$

$A_{st} = 9\pi \frac{25^2}{4} = 4417.86\,mm^2$

$A_{sc} = 6 \times \frac{\pi \times 25^2}{4} = 2945.24\,mm^2$

$x_u = \frac{360.813 \times 4417.86 - (344.626 - 0.446 \times 20)\,2945.24}{0.36 \times 20 \times 300}$

$x_u = 280.226\,mm > x_{u,l}$

So, the section is over-reinforced.

2nd trial,

$x_u = \frac{242.6 + 280.23}{2} = 261.41\,mm$

$\varepsilon_{sc} = 0.0035\left(1 - \frac{70.5}{261.41}\right) = 0.00256$

$\varepsilon_{st} = 0.0035\left(\frac{d}{x_u} - 1\right) = 0.0035\left(\frac{504.5}{261.41} - 1\right) = 0.00325$

$\therefore f_{sc} = 342.8 + \frac{351.8 - 342.8}{(0.00276 - 0.00241)}(0.00256 - 0.00241)$

$= 346.56\,MPa$

$f_{st} = \cancel{351.8}\ 351.8 + \frac{(360.9 - 351.8)}{(0.0038 - 0.00276)}(0.00325 - 0.00276)$

$= \cancel{359.24\,MPa}\ 356.175\,MPa$

$$\therefore x_u = \frac{\cancel{359.14}\ 356.173 * 4417.865 - (346.4 - 0.446*20)*2945.243}{0.36*20*300}$$

$= \cancel{274.59mm}$

$= 268.32mm > x_{u,l}$

So, the section is over reinforced.

3rd trial-

Assume, $x_u = \frac{261+268}{2} = 264.5mm \approx 265mm$

$\varepsilon_{sc} = 0.0035\left(1 - \frac{70.5}{265}\right) = 0.00257$

$\varepsilon_{st} = 0.0035\left(\frac{504.5}{265} - 1\right) = 0.00316$

$f_{sc} = 342.8 + \frac{(351.8 - 342.8)}{(0.00276-0.00241)}(0.00257-0.00241)$

$= 346.91MPa$

$f_{st} = 351.8 + \frac{(360.9-351.8)}{(0.0038-0.00276)}(0.00316-0.00276)$

$= 355.3 MPa.$

$x_u = \frac{355.3*4417.865 - (346.91 - 0.446*20)*2945.243}{0.36*20*300}$

$= 265.83mm.$

~ Matches ≈

Hence, final $x_u = 265mm$ (adopt)

4th trial ~~Assume, $x_u = \frac{265+267}{2} = 266mm$~~

Moment of Resistance (M.R.)

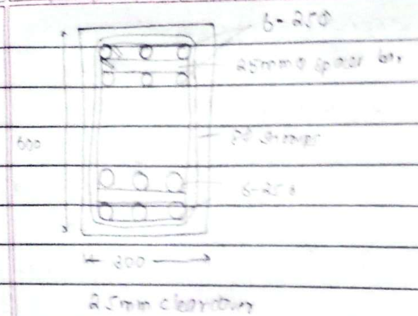
$= 0.36 f_{ck} b x_u (d - 0.42x_u) + (f_{sc} - 0.446 f_{ck}) A_{sc} (d-d')$

$= 0.36*20*300*265(504.5 - 0.42*265) + (346.91 - 0.446*20)*2945.243*(504.5 - 70.5)$

$= 657.098 kNm$

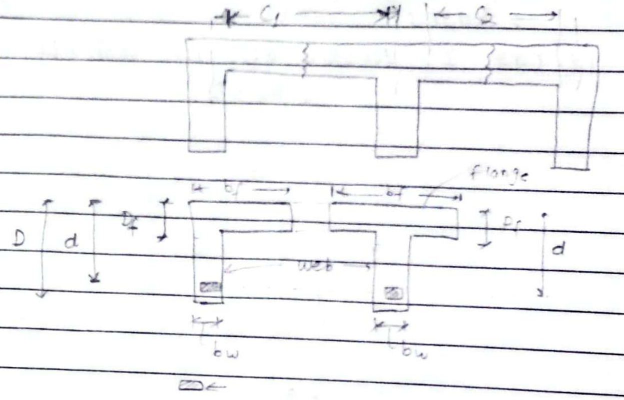
[If previous problem continue, $M_R > M_u$ safe.]

(3) Find Moment of resistance (M.R.)



Flanged Beam/Flanged section:

C_1 & C_2 → clear span



* Effective width of flange (b_f)
  (Cl. 23.1.2, IS456: 2000)

* Combined or series of beam & slab

(i) For T-beams:

$$b_f = \frac{l_0}{6} + b_w + 6D_f \leq b_w + \frac{C_1}{2} + \frac{C_2}{2}$$

l_0 → beam को span मा 2 टा point जसमा moment zero हुन्छ त्यसो ~~बिचको~~ point.

l_0 → distance betn point of contraflexure of beam.

(ii) For L-beams

$$b_f = \frac{l_0}{12} + b_w + 3D_f \leq \left(b_w + \frac{C_1}{2} \right)$$

For isolated beams:

- T-beam:

$$b_f = \frac{l_0}{\left(\frac{l_0}{b} + 4\right)} + b_w < b$$

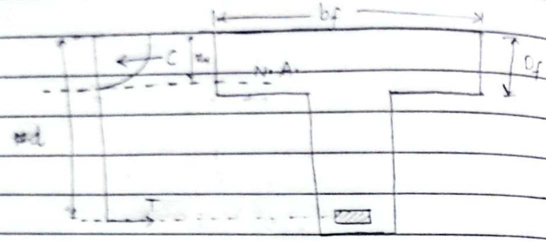
- L-beam:

$$b_f = \frac{0.5 l_0}{\left(\frac{l_0}{b} + 4\right)} + b_w < b.$$

Design Procedure for flanged section

→ 3 cases

(i) Case I: Neutral axis lies ~~with fla~~ within flange ($x_u \le D_f$)



C = T

- $0.36 f_{ck}\, b_f \cdot x_u = f_{st}\, A_{st} = 0.87 f_y A_{st}$ (for under reinforced or balanced section)

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

$$\left.\begin{aligned} M.R. &= 0.87 f_y A_{st}(d - 0.42 x_u) \\ &= 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{b_f d f_{ck}}\right) \end{aligned}\right\} \text{under-reinforced or balanced}$$

$$= 0.36 f_{ck}\, b_f\, x_u (d - 0.42 x_u) \Big\} \rightarrow \text{over-reinforced}$$

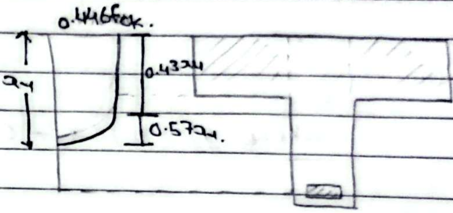
[Refer, G-2, IS 456:2000]

(ii) Case-II : Neutral axis lies in web [C. 3.2.]

$[x_u > D_f \;\&\; D_f \le 0.43 x_u]$

C = T

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

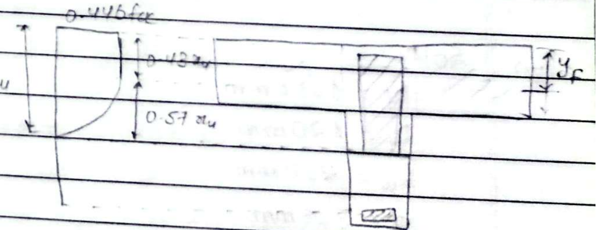


$$M.R. = 0.36 f_{ck} b_w \cdot x_u (d - 0.42 x_u) + 0.446 f_{ck}(b_f - b_w) D_f \left(d - \frac{D_f}{2}\right)$$

(iii) Case III : Neutral Axis lies in web

$[x_u > D_f \;\&\; D_f > 0.43 x_u]$

$\left[\frac{D_f}{d} > 0.2\right]$



C = T

$\Rightarrow 0.36 f_{ck} b_w \cdot x_u + 0.446 f_{ck}(b_f - b_w) * y_f = 0.87 f_y A_{st}$ (underreinforced or balanced section)

where, $y_f = (0.15 x_u + 0.65 D_f)$

$M.R. = (0.36 f_{ck} b_w \cdot x_u (d - 0.42 x_u)) + (0.446 (b_f - b_w) \cdot y_f * (d - y_f/2))$;

$$M_{u,l} = M_{u,l,web} + 0.446 f_{ck} (b_f - b_w) \cdot D_f * \left(d - \frac{D_f}{2}\right)$$

for $\frac{D_f}{d} \le 0.2$

- - - - - - -

and,

$$M_{u,l} = M_{u,l,web} + 0.446 f_{ck} (b_f - b_w) \cdot y_f \cdot \left(d - \frac{y_f}{2}\right)$$

for $\frac{D_f}{d} > 0.2$

where $y_f = 0.15 x_u + 0.65 D_f$

(Q) A RCC T-beam of 1650mm width of flange, 120mm depth of flange, 250mm width of web, and 525mm effective depth has to carry a factored bending moment of 760 KN·m. Determine reinforcement required. Use M25 concrete & Fe 500 rebar.

→ Soln

$b_f = 1650\text{mm}$

$D_f = 120\text{mm}$

$b_w = 250\text{mm}$

$d = 525\text{mm}$

$M_u = 760\text{KNm}. = 760 * 10^6$ N.mm

M25 concrete & Fe 500 rebar

Assuming lever arm z as larger of

$0.9d = 0.9 * 525 = 472.5\text{mm}$

& $d - \frac{D_f}{2} = 525 - \frac{120}{2} = 465\text{mm}$

i.e. $z = 470\text{mm}$ (say)

$$A_{st} \text{ required} = \frac{M_u}{0.87 f_y * z}$$

$$= \frac{760 * 10^6}{0.87 * 500 * 470}$$

$$= 3717.29 \text{ mm}^2$$

Providing 4 rebars

$$\phi_{req} = \sqrt{\frac{3717.29/4}{\pi/4}} = 34.4 \text{ mm}$$

∴ Let us provide 4-36ϕ rebars.

$$\therefore A_{st,prov.} = \frac{4 * \pi * 36^2}{4}$$

$$= 4071.504 \text{ mm}^2$$

Determining actual x_u;

Assume $x_u < D_f$

$C = T$

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

$\Rightarrow x_u = 119.266\text{mm} < 120\text{mm}$

okay.

$\therefore$ M.R. $= 0.87 f_y A_{st} (d - 0.42 x_u)$

$= 841 * 10^6$ N.mm

$= 841 \text{KNm} > 760 \text{KN.m}$

(Hence, safe)

Also, checking: $M_{u,l}$

$$\frac{D_f}{d} = \frac{120}{525} = 0.2285 > 0.2$$

$$M_{u,l} = M_{u,l,web} + 0.446 f_{ck}(b_f - b_w) * y_f * \left(d - \frac{y_f}{2}\right)$$

where, $y_f = 0.15 x_u + 0.65 D_f$

$= 95.8899$

≈ 96 mm

$$M_{u,l} = 0.133 f_{ck} b_w \cdot d^2 + 0.446 f_{ck}(b_f - b_w) * y_f (d - y_f/2)$$

(for 500MPa)

$M_{u,l} = 943 * 10^6$ Nmm

$= 943$ KNm $> M_u$

Section is under-reinforced.

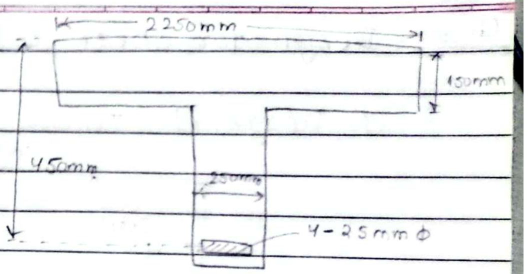
(9) $b_f = 2250$mm

$D_f = 150$mm

$b_w = 250$mm

$d = 450$mm

M20 concrete, Fe415 rebar



→ Sol^n:

Determination of x_u:

Assume $x_u \le D_f$

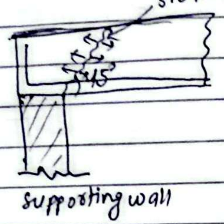
$0.36 f_c b_f \cdot x_u = 0.87 f_y A_{st}$

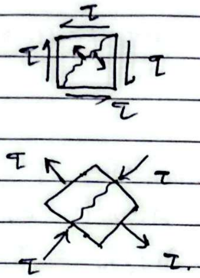
$\Rightarrow x_u = 43.76$ mm

(O.K.)

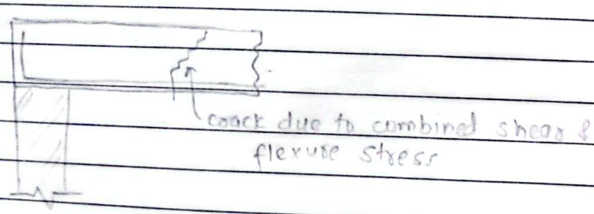
Ch 5.0 Design for Shear & Torsion:-

(1) Diagonal Tension Failure

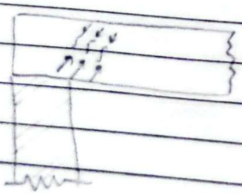




(2) Flexural Shear Failure
(Combined failure under shear & flexure)



(3) Diagonal compression failure:



- $\text{Shear stress} = \dfrac{\text{Shear Force}}{\text{Shear Area}}$

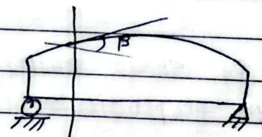
The nominal shear stress at ultimate limit state of shear strength is defined as:-

$$\tau_v = \frac{V_u}{bd} \le \tau_{c,max} \quad (Cl.\ 40.1)\ (Table\ 20)$$

↳ limiting value of design shear stress

Design shear stress, τ_c (Cl. 40.2.1, Table 19)
→ depend grade of concrete, f_{ck}
→ % tension steel in section. $\left\{ p_t = \frac{A_{st}}{bd} \times 100\% \right\}$.

→ For varying depth,



$$\Rightarrow \tau_v = \frac{V_u \pm \frac{M_u}{d} \tan\beta}{bd} \quad [Cl.\ 4.1.1.]$$

$$\Rightarrow \tau_c = \frac{0.85\sqrt{(0.8 f_{ck})}\left(\sqrt{1+5\beta} - 1\right)}{6\beta}$$

where, $\beta = \left.\begin{array}{l} \dfrac{0.8 f_{ck}}{6.89\, p_t} \\ = 1. \end{array}\right\}$ whichever is greater

Shear strength under axial compression [Cl. 40.2.2]

The design shear stress τ_c to be multiplied by a factor δ.

$$\delta = 1 + \frac{3P_u}{A_g f_{ck}} \le 1.5$$

$A_g \rightarrow$ gross cross-sectional area of concrete in mm²

Maximum shear stress $\tau_{c,max}$: [Cl. 40.2.3, Table 20]

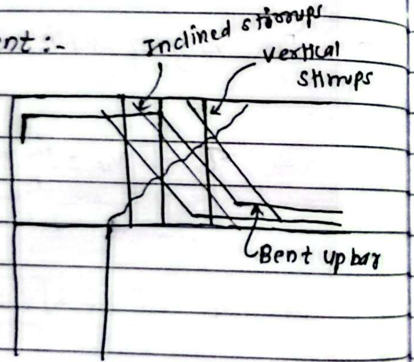
→ Under no circumstances, even with shear reinforcement shall the nominal shear stress in beam τ_v exceeds $\tau_{c,max}$ given in Table 20.

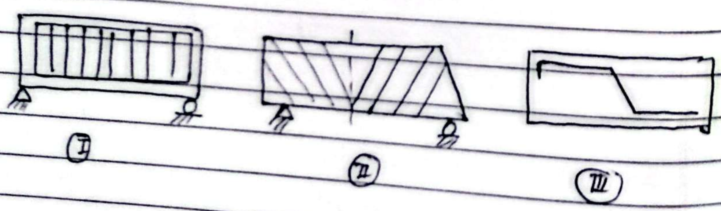
Types of shear reinforcement:-

→ Stirrups perpendicular to beam axis.

→ Stirrups inclined (45° or more) to the beam axis.

→ Longitudinal bars bent up (usually not more than two bars at a time) at 45° to 60° to the beam axis, combined with stirrups.





Shear resisting capacity of RC beam section:

↳ $V_R = V_c + V_s + V_B$

V_c → Shear resisting capacity of beam section without shear reinforcement

V_s → Shear resisting capacity of stirrups

V_B → Shear resisting capacity of bent up bars.

Design of shear reinforcement:
(Design steps)

(i) Calculate $\tau_v = \frac{V_u}{bd}$ ← max^m factored shear force

(ii) Read τ_c from Table 19 of IS456; and $\tau_{c,max}$ from Table 20.

(iii) Check $\tau_v \le \tau_c$ → Provide minimum shear reinforcement given by:- Cl. 26.5.1.6 of IS456.

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

where,

A_{sv} = total cross-sectional area of shear legs

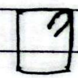


fig: Two legged stirrups

S_v = Spacing of stirrups (c/c)

b = width of beam

f_y = characteristic strength of stirrups. ≤ 415 MPa

(iv) If $\tau_c < \tau_v \le \tau_{c,max}$.

design shear force, $V_{us} = V_u - \tau_c \cdot bd$.

(a) For vertical stirrups,

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

(b) For inclined stirrups, or series of bent up bars

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

where $\alpha \rightarrow$ angle between inclined stirrups or bent up bars with axis of member.

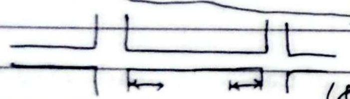
(c) For single bent-up bar,

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

(v) If $\tau_v > \tau_{c,max} \rightarrow$ redesign $\rightarrow$ either increase grade of concrete or size of member.

Criteria for shear stirrups

- Minimum diameter of shear rebar $\geq 8mm$ [except bent up bar] [Cl. 6.3.2. IS 13920]
- Maximum spacing $\leq 0.75d$ or 300mm [Cl. 26.5.1]
- Max. spacing $\leq 0.25d$ or 8ϕ or 100mm [Cl. 6.3.5, IS 13920] {over the length of 2d from face of column}



(ϕ = least dia of longitudinal bar)

& Max^{m} spacing $\leq \frac{d}{2}$ for remaining length.

8d or 75mm

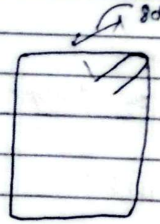


Fig 4 IS 13920 (1st amendment)

(Q) A rectangular beam of size 250mm × 500mm (effective depth) is subjected to a factored shear force of 110KN. The beam is reinforced with 3-22ϕ dia bars in tension. Design shear reinforcement. Use M20 concrete & Fe500 rebar.

→ Sol^{n}:-

$b = 250mm$

$d = 500mm$

$V_u = 110KN = 110 * 10^3 N$

$A_{st} = 3 * \frac{\pi * 22^2}{4} = 1140.398 mm^2$

M20 concrete and Fe500 rebar.

Now,

$$\tau_v = \frac{V_u}{bd} = \frac{110*10^3}{250*500} = 0.88 N/mm^2$$

For, M20 concrete (Table 20; IS456)

$\tau_{c,max} = 2.8 N/mm^2$

$$P_t = \frac{1140.398}{250*500} * 100\% = 0.912\%$$

∴ From 19 of IS456,

$$\tau_c = 0.6 + \frac{(0.62-0.6)}{(1-0.9)} * (0.912-0.9) = 0.6025 N/mm^2$$

(0.598 N/mm^2)

Here,

$\tau_v < \tau_{c,max}$ (Section of O.K.)

Also, $\tau_c < \tau_v$

∴ Design shear force

$\left\{ V_{us} = \frac{0.87 f_y A_{sv} \cdot d}{s_v} \right\}$

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

$$= 110 - \frac{\cancel{0.6025}^{\,0.598} * 250 * 500}{1000}$$

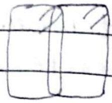
$= \cancel{34.6875KN}$ 35.25KN

Page No.
Date

Providing 2 Legged (2-L) Stirrups @ 100mm c/c

i.e. $A_{sv} = 2 * \pi * \frac{\Phi^2}{4}$

& $S_v = 100mm$.



Then,

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

where, $f_y \leq 415 N/mm^2$

$$\therefore \cancel{34.6875} \overset{35.25}{} * 10^3 = \frac{0.87 * 415 * 2\pi * \frac{\Phi_s^2}{4} * 500}{100}$$

$$\Rightarrow \boxed{\Phi_s = 3.53mm}.$$

Also, checking for Minimum shear reinforcement,

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

or $$\frac{2 * \pi * \Phi_s^2}{4 * 250 * 100} \geq \frac{0.4}{0.87 * 415}$$

$$\boxed{\Phi_s \geq 4.199mm}.$$

(But can't provide stirrups less than 8mm dia.)

Provide 8mm ϕ - 2L - stirrups @ 100 c/c mm

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Date

(9) A simply supported beam 300 * 600 mm (effective) is reinforced with 5 bars of 25mm dia. It carries a UDL of 85kN/m including its self weight over an effective span of 6m. Out of five bars, two bars are bent up ~~section~~ safely near support with 45° angle with axis. Design the shear reinforcement for beam. Use M20 concrete & Fe415 rebar.

→ Soln.,

$b = 300mm$

$d = 600mm$

M20 concrete

Fe415 rebar

Shear force, $V = \frac{wl}{2} = \frac{85 * 6}{2} = 255kN$

$\therefore$ Factored shear force, $V_u = 1.5V = 382.5kN$

$$A_{st} = 3\pi * \frac{25^2}{4} = 1472.621 mm^2$$

$$\therefore P_t (\%) = \frac{1472.621}{300 * 600} * 100 = 0.818\%.$$

Now,

Nominal shear stress,

$$\tau_v = \frac{V_u}{b \cdot d} = \frac{382.5 * 1000}{300 * 600} = 2.125 MPa.$$

$\tau_{c,max} = 2.8 MPa$ for M20 (Table 20, IS456)

$$\tau_c \text{ (from table 19)} = 0.56 + \frac{(0.62 - 0.56)}{(1 - 0.75)} * (0.818 - 0.75)$$

$$= 0.57632 MPa.$$

Here, $\tau_v > \tau_c$ & $\tau_v < \tau_{c,max}$

Hence, total design shear force, $V_{us} = V_u - \tau_c\, b \cdot d$.
$= 278.7624\,kN$

Shear force taken by bent-up bars,
$= 0.87 f_y A_{sv} \sin\alpha$
$= 0.87 * 415 * 2 * \frac{\pi * 25^2}{4} * \sin 45°$

$= 250,641.07\,N$
$\approx 250.641\,kN$.

$\therefore$ Design shear force for stirrups,
$= 278.7624 - 250.641$
$= 28.121\,kN$.

→ (As per Cl. 40.4, IS456; contribution of bent up bar should exceed 50%.)

Let's provide 2L-8Φ stirrups, then
$A_{sv} = 2\pi * \frac{8^2}{4} = 100.53\,mm^2$

Hence for vertical stirrups,
$$V_{us} = \frac{0.87 f_y A_{sv} \cdot d}{S_v}$$
$$\Rightarrow 28.121 * 10^3 = \frac{0.87 * 415 * 100.53 * 600}{S_v}$$
$$\Rightarrow S_v = 774.43\,mm.$$

As, per Cl. 40.4, IS456; contribution of bent up bar should not exceed 50%.
$\therefore$ Design shear force for vertical stirrups.
$= 278.7624 * 0.5$
$= 139.3812\,kN$.

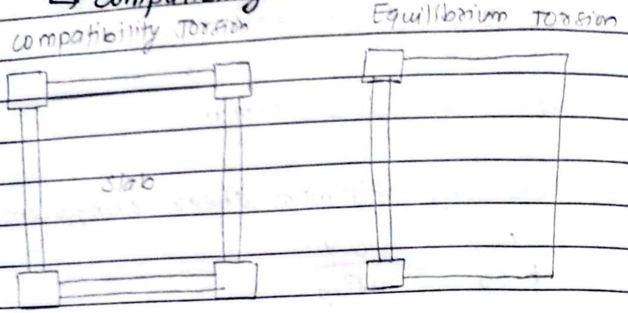
Hence, for vertical stirrups
$$V_{us} = \frac{0.87 f_y A_{sv} \cdot d}{S_v}$$
$$\Rightarrow 139.3812 * 10^3 = \frac{0.87 * 415 * 100.53 * 600}{S_v}$$
$$\Rightarrow S_v = 156.246\,mm$$

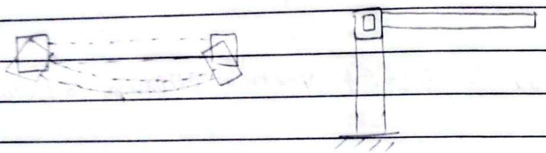
Also, checking minimum shear reinforcement.
$$\frac{A_{sv}}{b \cdot S_v} \geq \frac{0.4}{0.87 f_y}$$
$$\Rightarrow S_v \leq 302.47\,mm$$

$\therefore$ Provide 2L-8Φ vertical stirrups @ 150mm c/c.

Design of RC section of Torsion:

- Torsion
  - → Equilibrium Torsion
  - → Compatibility Torsion





→ At any RC section,

M_u → Factored moment

T_u → factored twisting moment

V_u → Factored shear force

→ Equivalent shear,

$$V_e = V_u + 1.6\frac{T_u}{b} \quad [Cl.\ 41.3.1]$$

→ Equivalent nominal shear stress,

$$\tau_{ve} = \frac{V_e}{bd} \le \tau_{c,max} \quad [\text{Table 20}]$$

if $\tau_{ve} \le \tau_c$ (Table 19) → Neither design for torsional shear nor for ~~longitudinal reinforcement~~

→ ~~Mini Minimum~~

→ Min^m shear reinforcement should be provides as per Cl. 26.5.1.6

$$\left[\frac{A_{sv}}{b s_v} \ge \frac{0.4}{0.87 \ldots}\right]$$

- Torsion Design
  - → Design for longitudinal reinforcement.
  - → Design for shear reinforcement.

If $\tau_c < \tau_{ve} \le \tau_{c,max}$, design for longitudinal reinforcement & shear reinforcement

Design for longitudinal reinforcement

Equivalent moment due to twisting moment

$$M_t = T_u\left[\frac{1 + D/b}{1.7}\right] \quad \{Cl.\ 41.4.2\}$$

→ If $M_u > M_t$ → Beam is designed for $M_{e_1} = M_u + M_t$

↳ reinforcement will be designed for flexural tension face.

→ If $M_u < M_t$ → Beam is designed for

$M_{e_1} = M_u + M_t$ → for flexural tension face.

$M_{e_2} = M_t - M_u$ → for flexural compression face.

(9) A beam of width 300mm and total effective depth ~~500~~ 540 mm has span of 4.5m, tensile $M_u = 50 KNm$, $V_u = 50KN$, $T_u = 20KN.m$ (case I), $T_u = 40KNm$ (case II). For Fe415 & M20 concrete. Design for Torsion.

→ Solution:-

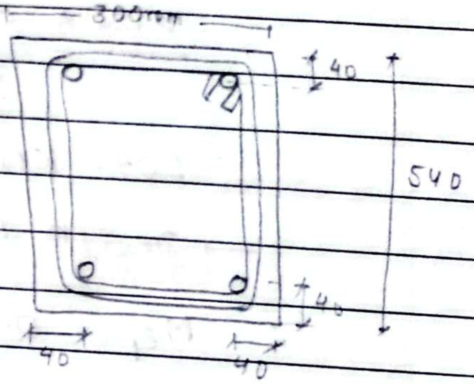
$b = 300mm$

$D = 540mm$

$M_u = 50KNm$

$V_u = 50KN$

$T_u = 20KNm$ (case I)



Case I :.

$M_u = 50KNm$

$T_u = 20KNm$

$V_u = 50KN$

$$V_t = 1.6\left(\frac{T_u}{b}\right) = 106.6KN$$

$$M_t = T_u\left(\frac{1 + D/b}{1.7}\right) = 20\left(\frac{1 + 540/300}{1.7}\right) = 32.941KN.m$$

$$\therefore V_e = V_t + V_u = 106.6 + 50 = 156.6KN$$

$$\therefore \tau_{ve} = \frac{V_e}{bd} = \frac{156.6 * 10^3}{300 * 500} = 1.044 N/mm^2$$

$< \tau_{c,max}$.

Here, $M_t < M_u$

$M_{e_1} = M_t + M_u$

$= 82.94 KNm$

$M_{u,t} = 0.138 f_{ck} bd^2$ (for M20)

$= \frac{0.138 \times 20 \times 300 \times 500^2}{10^6}$ KN·m

$= 207.$ KNm $> M_{e_1}$

→ Design as singly reinforced.

$\therefore M_{e_1} = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} \cdot b \cdot d}\right)$

$\Rightarrow A_{st} = 493.41 mm^2$

Adopt 16Φ bars

No. of bars $= \frac{493.41}{\pi \times 8^2} = 2.45$

≈ 3 (adopt)

Also, provide 2-12Φ hanger bars (on compression side).

Design for shear reinforcement,

$P_t = \frac{A_{st}}{bd} \times 100\% = \frac{3 \times \pi \times 8^2}{300 \times 500} \times 100\% = 0.402\%$

∴ From table 19,

$\tau_c = \left(\frac{0.48 - 0.36}{0.5 - 0.25}\right)(0.402 - 0.25) + 0.36$

$= 0.43 N/mm^2 < \tau_{ve} (= 1.04 N/mm^2)$

Let us assume 8Φ - 2 legged stirrups.

$A_{sv} = \frac{\pi \times 8^2}{4} = 50.265 mm^2$

(code commentry ... ??)

$\therefore S_v = \frac{0.87 f_y A_{sv} \cdot b_1 d_1}{T_u + 0.4 V_u b_1}$

$b_1 = 300 - 2 \times 40 = 220mm$

$d_1 = 540 - 2 \times 40 = 460mm$

$x_1 = 300 - (2 \times 40) + (2 \times \frac{16}{2} + 2 \times 8) = 252mm$

$y_1 = 460 + 2 \times \frac{16}{2} + 2 \times 8 = 492mm.$

$\therefore S_v = 82.75mm.$

Also,

$S_v \le \frac{0.87 f_y A_{sv}}{(\tau_{ve} - \tau_c) b}$

$\therefore S_v \le 98.5mm$

Also,

cl. 26.5.1.7 (codal provision)

$S_v \le x_1 (= 252mm)$

$S_v \le \frac{y_1 + x_1}{4} (= 186mm)$

$S_v \le 300mm$

Thus, Adopt $S_v = 80mm$.

Chapter-6 :-

Design for Bond & Development Length

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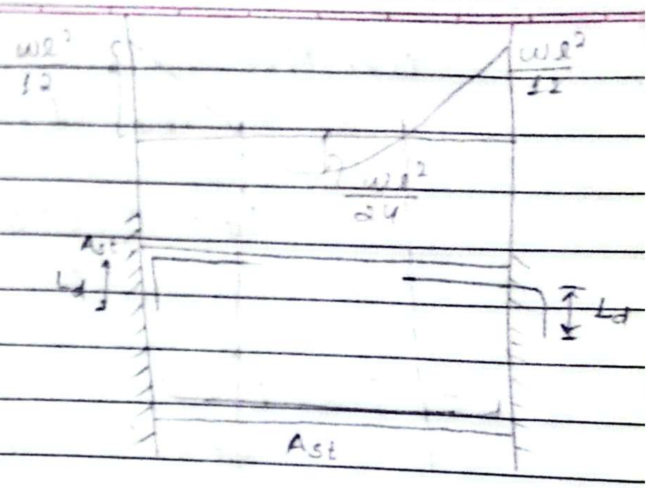
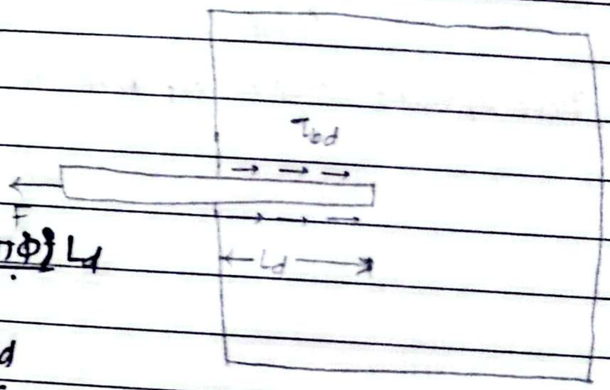
→ **Development Length:**
[Cl. 26.2.1.]



L_d → development length.



$F = 0.87 f_y A_{st}$

$F \le f_s A_{st} = \tau_{bd} * (\pi\Phi) L_d$
↓
design bond stress

$\Rightarrow \cancel{\tau_{bd}}\ f_s * \frac{\pi\Phi^2}{4} = \tau_{bd} * \pi\Phi L_d$

or. $\tau_{bd} = \frac{f_s \Phi}{4 L_d}$

or $L_d = \frac{f_s \Phi}{4\tau_{bd}}$

For full strength transfer.
$[f_s = 0.87 f_y]$

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

→ For M20 concrete & Fe415 rebar,

$\tau_{bd} = 1.2 * 1.6$ (Tension case)

$$L_d = \frac{0.87 * 415 * \phi}{4 * 1.2 * 1.6} = 47.01\phi$$

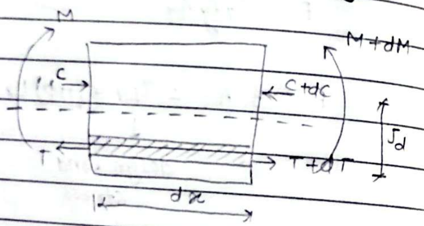
→ For M25,

$L_d = 40.20\phi$

→ We have,

$$\tau_{bd} = \frac{\phi f_s}{4 L_d} \quad \text{---- (i)}$$

• Development length of bar in flexure [Cl. 26.2.3.3].



We have,

$M + dM = (T + dT) * J_d$

or, $M + dM = T \cdot J_d + dT \cdot J_d$

or, $M + dM = M + dT \cdot J_d$

$$\left[\therefore dT = \frac{dM}{J_d}\right]$$

→ $$\tau_{bd} * \pi * \phi . dx = \frac{dM}{J_d}$$

$$\therefore \tau_{bd} = \frac{dM}{dx} \cdot \frac{1}{J_d \cdot \pi\phi} = \left[\frac{V}{J_d \cdot \pi \cdot \phi}\right] \quad \text{--- (ii)}$$

Equating equation (i) and (ii)

$$\frac{V}{J_d \cdot \pi \cdot \phi} = \frac{\phi f_s}{4 L_d}$$

$$L_d = \frac{\pi \phi^2 f_s J_d}{4} * \frac{1}{V}$$

$$= T * J_d * \frac{1}{V}$$

$$= \frac{M}{V} \quad [M \rightarrow \text{Moment capacity of the section}]$$

(Some time denoted by M_0 (M_1))

$$\therefore \left[L_d = \frac{M}{V}\right] \quad V \rightarrow \text{factored shear at section}$$

→ $$L_d = \frac{M_1}{V}$$

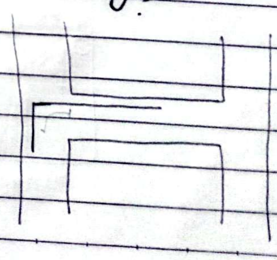
$$(L_d)_{max} \leq \frac{M_1}{V} + L_0$$

$$\text{i.e. } (L_d)_{max} \leq \left.\frac{1.3 M_1}{V} + L_0\right] \text{ie.}$$

Achorage

deformed bar → Not compulsory

Plain bar → must provide anchorage)

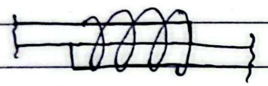


- Equivalent ~~ach~~ anchorage value [26.2.2.1].
  - 45° bend → 4ϕ
  - 90° bend → 8ϕ
  - 135° bend → 12ϕ
  - 180° " → 16ϕ

Lap splices [26.2.5.1.]

For ϕ > 36mm] → Lap splices not allowed
(welding must be done)

{If welding not practicable (remote areas)

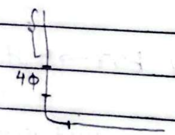


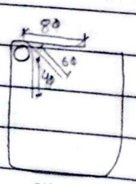
(binding wire spiral is provided)

Development Length

- ↳ Anchorage
- ↳ Bend
- ↳ Hook
- ↳ Mechanical Anchor

[Cl. 26.2]





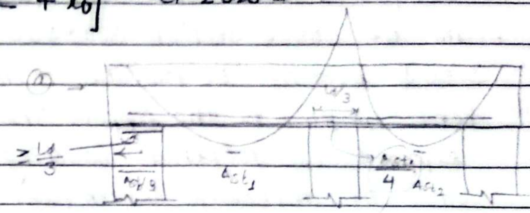
Stirrups

90° bend → 8ϕ
135° bend → 6ϕ
180° bend → 4ϕ

Positive Moment reinforcement

© $\left[L_d \leq \frac{1.3M_1}{V} + l_0\right]$ ~~Cl-26.2.3.2~~

Cl. 26.2.3.3 ⓐ →

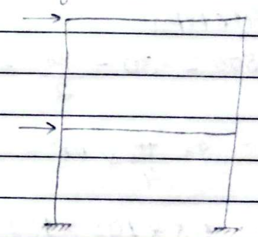


support का $\frac{L_d}{3}$ [illegible]
- → simply supported → $A_{st}/3$
- → continuous → $\frac{A_{st}}{4}$

$\frac{A_{st_1}}{4}$ or $\frac{A_{st_2}}{4}$

Box wall

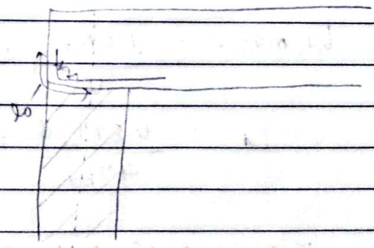
26.2.3.3 (b) → eg. Moment resisting Frame (Portal frame)



$L_d \leq \frac{1.3M_1}{V} + l_0$

i.e. $L_d \leq L_{d,max}$.

$l_0 \leq d$
$l_0 \leq 12\phi$



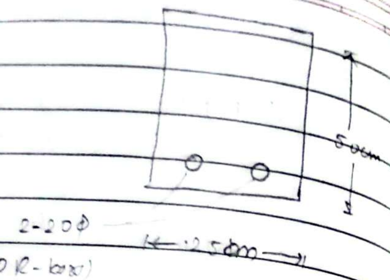
→ section is under-reinforced.

$\therefore M_1 = 0.87 f_y A_{st} (d - 0.42 x_u)$
$= 138.79 \text{ kN.m.}$

$\therefore L_{d,max} = \frac{1.3 * 138.79 * 10^6}{157.5 * 10^3} + 160$
$= 1305.5 \text{mm} > L_d$
<u>satisfied</u>.

Hence, L_0 provided by $90°$ bend at support is sufficient.

Q.



$V_u = 110 * 1.5 = 165 kN$

$l_0 = ?$

M_{20} Concrete.

Assume 25mm clear cover.

$\therefore d = 500 - 25 - \frac{20}{2} = 465 mm$

$x_{u,l} = 0.48d = 0.48 * 465 = 223.2 mm$

$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} \cdot b} = 126mm < x_{u,l} \rightarrow$ under-reinforced.

$\therefore MOR = M_1 = 0.87 f_y A_{st} (d - 0.42 x_u)$

$= 0.87 \cdot$

$= 93.45 * 10^6 N.mm.$

Now,

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

If the bar is provided with 90°-bend at support, then

$l_0 = 8\phi = 160 mm.$

For $L_d \leq \frac{1.3 M_1}{V_u} + l_0$

or, $47\phi \leq \frac{1.3 * 93.45 * 10^6}{165 * 10^3} + 8\phi$

or, $39\phi \leq \frac{1.3 * 93.45 * 10^6}{165 * 10^3}$

or, $\phi \leq 18.88 mm$

Since, actual bar dia. is 20mm which is greater than 18.88mm; there is need to increase the anchorage length, let us assume the anchorage length, $l_0 = 12\phi$

$= 240 mm.$

i.e. $47\phi \leq \frac{1.3 * 93.45 * 10^3}{165} + 12\phi$

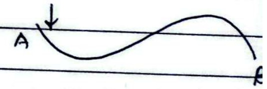
i.e. $\phi \leq 21 mm.$

Chapter-7

Nominal cover:

Table 16/Table 16A

	IS456	IS 13920 : 2016
• Min^{m} dia:-	6mm	8mm
• Max. spacing: (Stirrups बीचको)	0.75d or 300mm (whichever lesser) d→effective depth of beam	*Beam: d/4 or 8ϕ or 100mm (whichever is lesser) ϕ→ मोटो डडी को diameter Cl. 6.3.5 →upto 2d distance from column length. ~~(special confining shear rebar)~~ →For remaining portion, d/2 spacing * Column: (6.3.) sway to right $1.4\left[\frac{M_u^{A_s} + M_u^{B_h}}{L_{AB}}\right]$ (sagging, hogging)



sway to left

$$1.4\left[\frac{M_u^{A_h} + M_u^{B_s}}{L_{AB}}\right]$$

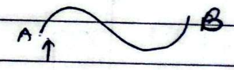


Fig. 12 (IS 13920:2016) Cl 8.1

(a) l_0 = larger dimension of cross-section of column
or, 1/6 of clear span
or, 450mm

whichever is greater

(b) have spacing : 1/4 of min^m dimension of column.
'or' 6 times smallest longitudinal bar dia
'or' 100mm

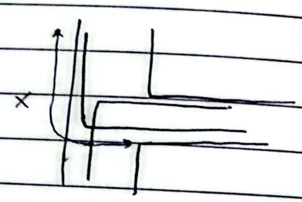
whichever is lesser

&

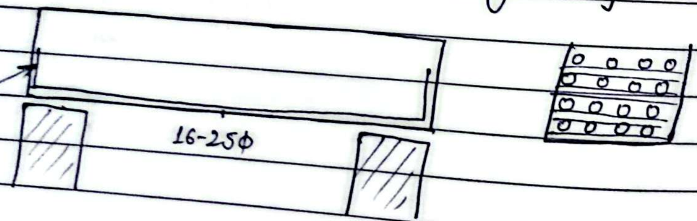
d/2 for remaining portion

Cl. 6.2.5

$x = L_d + 10\phi - 8\phi = L_d + 2\phi$



* Curtailment of flexural reinforcement
[Building SP34]



Design of Slab

Slab Transverse load लिने

Types:

- Solid (normally used)
- Hollow slab
- Grid slab / waffle / ribbed slab

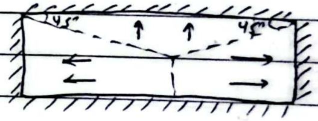
Based on support condition

- Simply supported (Brick wall मा राखेको)
- Continuous (B. Multi-wall को दुवै तिर)
- Fixed (support सँग Fixed भएको)

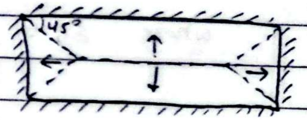
Based on load transfer Mechanism

(i) One way slab → Load transfer in one side

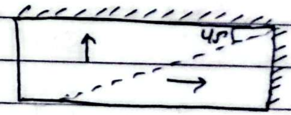
(ii) Two way slab



Support on all three sides by beam



Support on all Four side by beam.



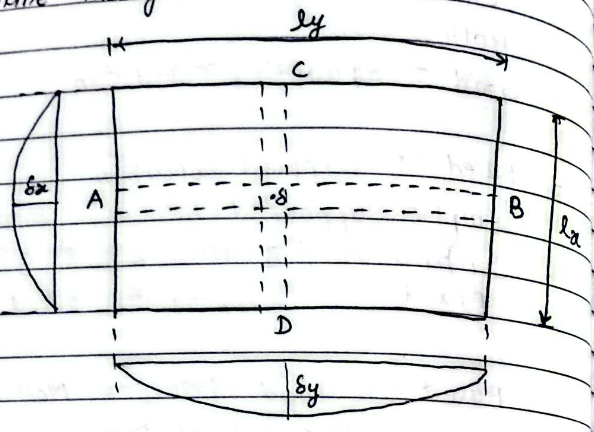
Support on two side by beam

$$\frac{l_y}{l_x} > 2 \rightarrow \text{One way slab}$$

$$\frac{l_y}{l_x} \leq 2 \rightarrow \text{Two-way slab}$$

* Load distribution theory in slab:

• Grasshoff - Rankine Theory:



case: simply supported.

$\delta_x = \delta_y = \delta$ = deflection at centre of slab

where, $\delta_x = \frac{5}{384} \frac{w_x l_x^4}{EI}$

$\delta_y = \frac{5}{384} * \frac{w_y l_y^4}{EI}$

$\therefore \frac{5}{384} * \frac{w_x l_x^4}{EI} = \frac{5}{384} * \frac{w_y l_y^4}{EI}$

$\Rightarrow w_x l_x^4 = w_y l_y^4$

$\Rightarrow w_x = w_y \left(\frac{l_y}{l_x}\right)^4$

Also, $w = w_x + w_y$

(total distributed load on slab)

$\therefore w = w_y \left(\frac{l_y}{l_x}\right)^4 + w_y$

let, $\frac{l_y}{l_x} = r$

$\therefore w = w_y (r^4 + 1)$

$\therefore w_y = w * \frac{1}{(r^4+1)}$

& $w_x = w \frac{r^4}{(r^4+1)}$ $\{\because w_x = w - w_y\}$

case (i) $l_x = l_y \Rightarrow r = 1$

$w_x = 0.5w$

$w_y = 0.5w$

case(ii) $l_y = 1.5 l_x \Rightarrow r = 1.5$

$\therefore w_x = 0.835w$

$w_y = 0.165w$

case(iii) $l_y = 2l_x \Rightarrow r = 2$

$w_x = 0.942w$

$w_y = 0.058w$

case(iv) $l_y = 3l_x \Rightarrow r = 3$

$w_x = 0.988w$

$w_y = 0.012w$

* Design of one-way slab:-

1. Find the depth of slab using deflection criteria.

Take: $\frac{span}{d} = 25$ for simply supported slab

$\frac{span}{d} = 30$ for continuous slab

$D \geq 100mm$ in normal case and 125mm in earthquake resistant design.

2. Determine the design load on its unit meter strip and find design BM & SF at critical section.

For continuous one-way slab, BM & SF at critical sections can be found using BM & SF coefficient as given in Cl. 22.5 of IS456 (Table 12 & Table 13)

(3) One-way slab is designed for BM & SF as rectangular beam of one meter width, and effective depth 'd'.

* Also, verify the depth of section with respect to depth of balanced section for singly reinforced Under-Reinforced section (SRURS).

$$d > d_{bal} = \sqrt{\frac{Mu}{R \cdot b}}$$

* At critical section of slab find A_{st}, Number of bars and their spacing

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

$A_{st \cdot min} > 0.0012 bD$

For Fe415 & Fe500 (High grade)

$> 0.0015 bD$

For Fe250

* Check for shear

For slab section without shear reinforcement

$\tau_v \leq K \cdot \tau_c$

Nominal shear

Table 19

K → Solid slab (Cl. 40.2.1.1.)

where, K is depth factor

(4) Check for deflection control: (Cl. 23.2)

(Check for slab at limit state of serviceability i.e. deflection and cracking)

* For deflection generally $\frac{span}{d}$ of slab is controlled

if $\frac{l_x}{d} \leq \alpha \beta \gamma \lambda \delta$

* For crack control generally reinforcement detailing rules are employed.

$$\frac{l_x}{d} \leq \alpha\beta\gamma\lambda\delta \quad (Cl.\ 23.2.1)$$

where,

- α is as per (Cl. 23.2.1.(a))
  - ↳ cantilever beam/slab $\alpha = 7$
  - ↳ $\alpha = 20$ for simply supported
  - ↳ $\alpha = 26$ for continuous

- β is as per (Cl. 23.2.1.(b))
  - ↳ $\beta = 1$ for span $\leq 10m$
  - ↳ $\beta = \frac{10}{span}$ for span $> 10m$

- γ is as per (Cl. 23.2.1.(c)) (Fig. 4)

$$f_s = 0.58 f_y \times \frac{\text{Area of X-section of steel required}}{\text{Area of X-section of steel provided}}$$

$$\left(P_t = \frac{A_{st}}{bd}\right)$$

$\gamma \rightarrow$ Modification factor for ~~st~~ actual stress in steel

- λ is as per (Cl. 23.2.1.(d)) (Fig. 5) [$A_{sc} \rightarrow$ compression reinforcement]

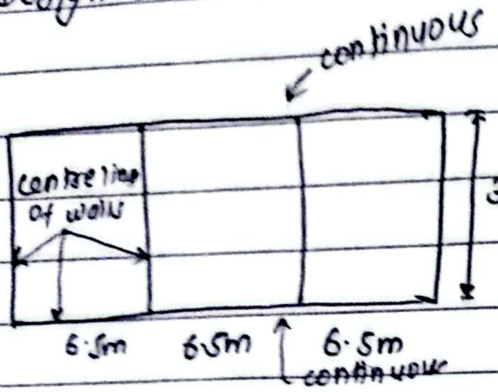
- δ is as per Cl. 23.2.1.(e) / (Fig. 6)
  - ↳ For flanged section

[λ & $\delta = 1$ for singly reinforced rectangular solid slab].

Design of one-way slab

Example-1

Design a slab of a floor system as shown in figure.



Take $LL = 4\,kN/m^2$

$FF = 1\,kN/m^2$

M20 concrete & Fe415 rebar.

→ Solⁿ:-

$$\left.\begin{array}{l} l_x = 3m \\ l_y = 6.5m \end{array}\right\} \frac{l_y}{l_x} = \frac{6.5}{3} > 2 \text{ (One way slab}$$

Assign the preliminary depth of slab

Take $\frac{span}{d} = \cancel{28}\ \cancel{25}\ 30$

[one-way → span → shorter span].

or, $\frac{3000}{d} = 30$

or, $[d = 100mm]$

Total depth, $D = d +$ Effective cover (clear cover + $\frac{सरिया}{2}$) ($\phi/2$)

$= 100 + 15 + \frac{10}{2}$

$= 120mm$

$\approx 125mm$ (whole no. inches)

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$\therefore$ Required depth $= \sqrt{\dfrac{M_{u,s}}{0.138 f_{ck} b}}$

$= \sqrt{\dfrac{30.44 * 10^6}{0.138 * 20 *}}$

→ Hence, section is Singly-Reinforced Under-reinforced section (SRURS).

• For span moment:

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

$\Rightarrow 10.04 * 10^6 = 0.87 * 415 * A_{st} * 105 * \left(1 - \dfrac{415 * A_{st}}{20 * 1000 * 105}\right)$

$\Rightarrow A_{st} = 280 mm^2.$

• For support moment,

$11.57 * 10^6 = 0.87 * 415 * A_{st} * 105 * \left(1 - \dfrac{415 * A_{st}}{20 * 1000 * 105}\right)$

$\Rightarrow A_{st} = 326.225 mm^2.$

Check

$(A_{st})_{min} = 0.0012 b D$

$= 0.0012 * 1000 * \cancel{105}\ 125$

$= 150 mm^2.$

Okay

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For 8 mm ϕ bar spacing of rebar at span

$= \dfrac{\text{Total width}}{\dfrac{A_{st}}{\left(\dfrac{\pi * \phi^2}{4}\right)}}$

$= \dfrac{1000}{\dfrac{280}{\dfrac{\pi * 8^2}{4}}}$

$= 179.52 mm$

Spacing for support moment

$= \dfrac{1000}{\dfrac{326.225}{\dfrac{\pi * 8^2}{4}}} = 154.08 mm.$

[Spacing → Round down]

$\therefore$ Provide 8-ϕ rebars @ 150mm c/c spacing at both top and bottom reinforcement.

$\therefore (A_{st})$ provided $= \left(\dfrac{1000}{150}\right) * \dfrac{\pi * 8^2}{4}$

↑ No

$= 335.10 mm^2.$

* Check for shear

$V_u = 21.94 KN$

$\therefore \tau_v = \dfrac{21.94}{1000 * 105} = 0.21 N/mm^2$

$P_t = \dfrac{A_{st,provided}}{1000 * 105} * 100\% = 0.32\%$

↑ Table 19.

$\therefore \tau_c = 0.394 \, N/mm^2$ (Table 19)

(shear capacity w/o shear reinforcement)

Now,

$\tau_v \leq k \cdot \tau_c$

where, $k = 1.3$ (Cl.40.2.1.1.)

or $0.21 \leq 1.3 \times 0.394$

OK

=) safe in shear.

* Check for deflection control :-

$\frac{L}{d} < \alpha \cdot \beta \cdot \gamma \cdot \lambda \cdot \delta$

where,

- $\alpha = \frac{20+26}{2} = 23$ (simply supported, continuous)
- $\beta = 1$ for span < 10m
- $\gamma =$ (Fig. 4)

$f_s = 0.58 f_y \times \frac{(A_{st})_{required}}{(A_{st})_{provided}}$ < span

$= 0.58 \times 415 \times \frac{280}{335.1}$

$= 201.12 \, MPa$

For $P_t = 0.32\%$, $f_s = 201.12 MPa$] $\gamma = 1.7$

- $\lambda = 1$ (No compression rebar)
- $\delta = 1$ (for solid slab)

So,

$\frac{3000}{105} < 23 \times 1 \times 1.7 \times 1 \times 1$

or $28.5 < 39$ (O.K.)

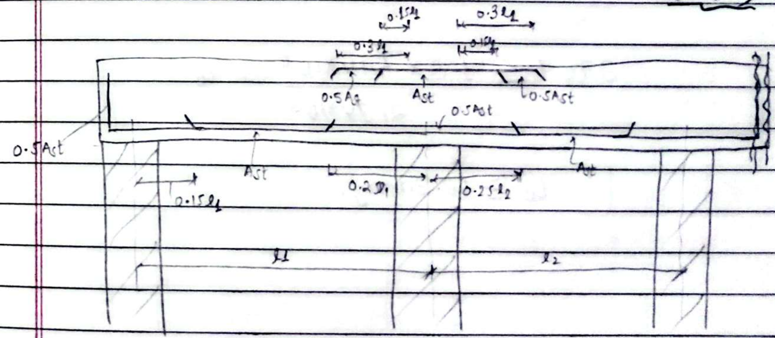
=) Hence, slab is safe in deflection

* Reinforcement detailing:-

→ $A_{st} = 335.10 \, mm^2$.

→ $0.5 A_{st} = 167.55 mm^2$

$A_{st.min} = 150 mm^2 < 0.5 A_{st}$ (SP34)



* Check for L_d at support

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

$$M_1 = 0.87 f_y (0.5A_{st}) * d\left(1 - \frac{f_y (0.5A_{st})}{f_{ck}\ bd}\right)$$

$$= 0.87*415*167.55*105*\left(1 - \frac{415*167.55}{20*1000*105}\right)$$

$= 6.14$ kN-m.

$V = 21.94$ kN.

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

$(\tau_{bd} = 1.2 * 1.6 \rightarrow \text{Table})$

$$= \frac{0.87*415*8}{4*1.2*1.6} = 376.1\text{ mm.}$$

$$\therefore 376.1 \le 1.3*\frac{6.14*10^6}{21.94*10^3} + L_0$$

$[\therefore L_0 \ge 12.3\text{mm}]$.

* Design steps of two-way slabs:

(1) Take $\frac{\text{span}}{d} = 28$ for simply supported slab

$= 32$ for continuous slab

$[D \ge 125\text{mm}]$

(2) Design moment (Refer Annex D_1 & D_2)

$M_x = \alpha_x w \cdot l_x^2$

$M_y = \alpha_y \cdot w \cdot l_x^2$

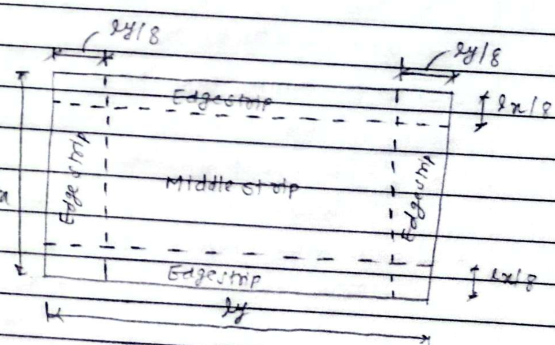
{ l_x = shorter span, l_y = longer span }

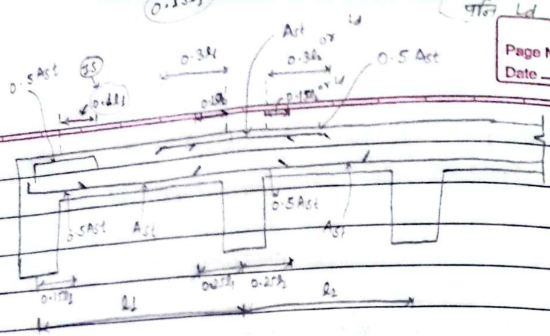
(3) Design shear

($w \rightarrow$ Load per unit area of the slab)

$$V_x = \frac{w l_x}{3}$$

$$V_y = w l_x \left(2 - \frac{l_x}{l_y}\right)$$

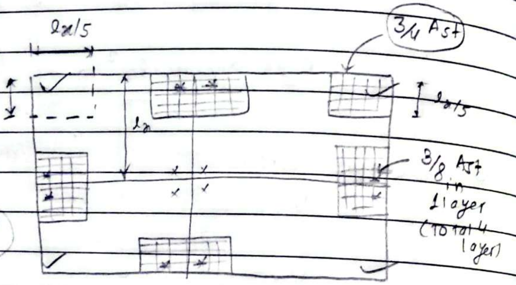




* Edge strip में minimum reinforcement 0.0012bD रखते हैं
(But less practice in field but can economize with this)

D-1.8

A_{st} → Midspan



जहाँ simply supported

✓ → दोनों तरफ continuous
✓ → दोनों तरफ simply supported वाले corner पर
× → दोनों तरफ not simply supported
* → एक तरफ simply supported, एक तरफ continuous

Simply supported (supported on brick wall)

✓ → 3/4 Ast
* → 3/8 Ast

→ यदि masonic wall से slab को छोड़ते हैं तो moment नहीं आता, slab उठ जाता है so no torsion reinforcement required (i.e. acts as fixed)

→ But brick wall से roof को lift होने से torsion reinforcement required

* But monolithic construction of Beam, slab, column done like in RC structure no torsion reinforcement required anywhere (not even roof)

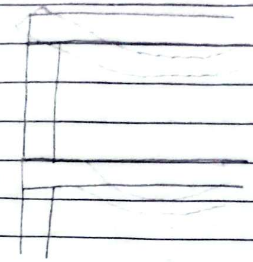
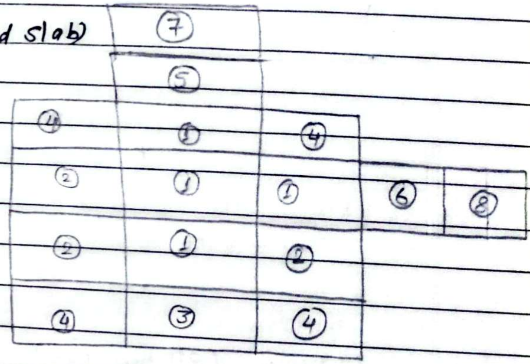


Table 26 (Restrained slab)

1. All continuous
2. Misprint: one short edge discontinuous
3. One long edge discontinuous
4.
5. Two short edge discontinuous
6.
7.
8.
9. Four edge discontinuous



α_x for (l_y/l_x)

		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	l_y/l_x
1	Negative moment									
	Positive									

Design (i) $d \to D$

(ii) w_d

w_l

$w_u = 1.5(w_d + w_l)$

(iii) Moments at mid span & support:

$\alpha_x^+ \to M_x^+ \to A_{st,x}^+$

$\alpha_x^- \to M_x^- \to A_{st,x}^-$

$\alpha_y^+ \to M_y^+ \to A_{st,y}^+$

$\alpha_y^- \to M_y^- \to A_{st,y}^-$

$\geq A_{st,min} = 0.0012bD$ (For Fe415 & Fe500, Fe550)

$A_{st,min} = 0.0015bD$ (for Fe250)

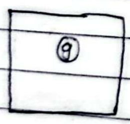
* सकुचन पट्टे d/o of उस्तो adopt (real practical)
(Both way same उस्तो)

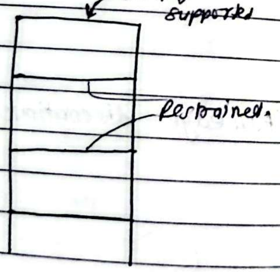
iv. Check

* Short span - मा आपको M_x^+

⇒ ~~9th type slab~~ (9th type slab)
(4-edges discontinuous)

restrained ↓ wall ले रोक्ने





(2) 4-edges simply supported] ⟶ Table 27

$\frac{l_y}{l_x} \to 2.5, 3.0$??
└ one way or two way ??

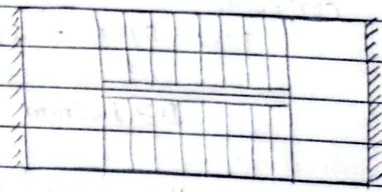
→ Single panel ⇒ Two-way * design गर्ने (Table 27)ले
But > 3 → one-way

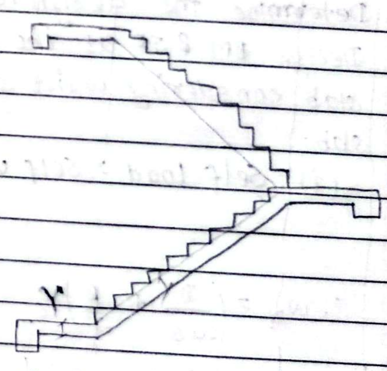
Staircase.

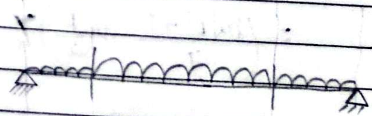
Types of stair:

i. Straight stair.
ii. Dog-legged stair.
iii. Open well stair
iv. Quarter turn stair

* Design of Dog-legged stair:

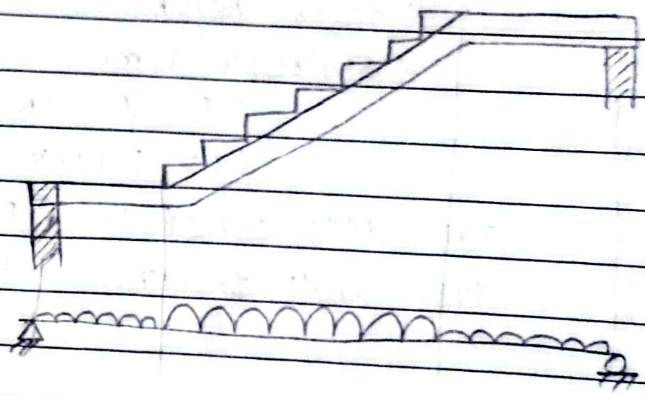






$\left[\frac{\text{span}}{d} = 30\right]$

→ Analyse waist slab for critical BM & SF.



→ Find Max. SF = (Max^{m} support reaction)

→ Find zero shear location & corresponding BM which will be max. BM.

→ Load calculation:-

On going of staircase

$$w_{DL} = \left(0.2 * \frac{\sqrt{0.16^2 + 0.25^2}}{0.25} + \frac{0.16}{2}\right) * 25 + 1.5$$

(FF)

$w_{DL} = \cancel{14}.\ 9.44\ KN/m^2$

$w_{LL} = 3\ KN/m^2$

$\therefore\ w_u = 1.5 * (9.44 + 3) = 18.66\ KN/m^2$

Let us analyse staircase for 1m width.

On floor & landing,

$W_{DL} = 0.2 * 25 + 1.5 = 6.5\ KN/m^2$

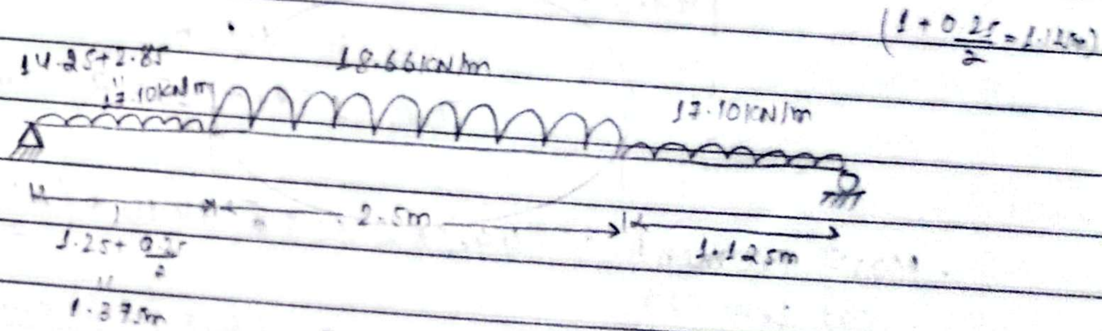
(FF)

$W_{LL} = 3\ KN/m^2$

$W_u = 1.5(6.5 + 3) = 14.25\ KN/m^2$.

Load from mid portion of slab of both flight

$= 14.25 * \frac{0.4}{2} = 2.85/m$



$\therefore$ Max^m shear, V_u (at right support) $= \dfrac{1.125 * 17.10 * \left(5.0 - \frac{1.125}{2}\right)}{5.0}$

$+ \dfrac{18.66 * 2.5 * \left(1.375 + \frac{2.5}{2}\right)}{5.0}$

$+ \dfrac{17.10 * 1.375 * \left(\frac{1.375}{2}\right)}{5.0}$

$= 44.8\,KN$

$\therefore$ Max^m shear, V_u (at left support) $=$

$= 44.6\,KN$

Thus, $V_{u,max} = 44.8\,KN$.

• Location of zero shear $=$ 2.505m from left support

$44.6 - 17.10 * 1.375 - 18.66 * (x) = 0$

$\Rightarrow x = 1.13m$

Thus, $1.375 + 1.13 = 2.505m$ from left support

• Max^m Moment (at zero SF) $= 44.6 * 2.505 - 17.10 * \left(\frac{1.375}{2}\right) + \ldots$

$- 18.66 * \left(\frac{2.505 - 1.375}{2}\right)$

$= 57.076\,KNm$.

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

or, $57.076 * 10^6 = 0.87 * 500 * A_{st} * 172 \left(1 - \dfrac{500 A_{st}}{1000 * 172 * 20}\right)$

$\left[d = 200 - 20 - \frac{16}{2} = 172mm\right]$

Solving,

$A_{st} = 874mm^2 \leftarrow$ required.

• $A_{st,min} = 0.0012 * 1000 * 20 = 240mm^2 < 874mm^2$

ok

• spacing of 16ϕ rebar $= \dfrac{1000}{\frac{874}{\left(\frac{\pi * 16^2}{4}\right)}} = $ ~~175.78mm~~ 230.05mm.

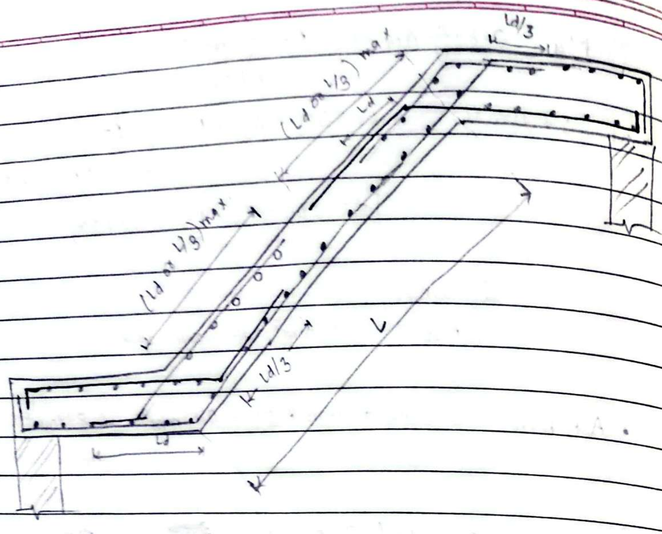
$\therefore$ ~~Provide 16ϕ rebar at 175mm c/c main bar.~~

• Spacing of 12ϕ rebar $= \dfrac{1000}{\frac{874}{\frac{\pi * 12^2}{4}}} = 129.2m$

$\therefore$ Provide 12ϕ rebar @ 125mm c/c main bar.

• Also, spacing of distribution bar (8mm ϕ) $= \dfrac{1000}{\left(\frac{240}{\pi * 4^2}\right)} = 20.9mm$

Provide 8mm ϕ @ 150c/c distribution bar.



• Check for shear:-

$V_u = 44.8 kN$

$\tau_v = \frac{V_u}{bd} = \frac{44.8 \times 10^3}{1000 \times 172} = 0.26 N/mm^2$

$P_t(\%) = \frac{\frac{(\pi \times 6^2) \times 1000}{125}}{1000 \times 172} \times 100\% = 0.526\%$

$\therefore \tau_c = 0.48 + \frac{(0.56-0.48)}{(0.75-0.5)}(0.526-0.5)$

(Table 19)

$= 0.488 N/mm^2$

Also, as per Cl. 40.2.1.1,

~~from~~ $K = 1.2$

For safe in shear,

$\tau_v \le K \cdot \tau_c$

i.e. $0.26 \le 1.2 \times 0.49$

ok

(Hence safe in shear)

($\frac{l}{d} = 25$) ↳ simply supported.

• Check for deflection control :

$l = 5.00m = 5000m$

$d = 172mm$

$\frac{l}{d} = \frac{5000}{172}$

Now,

$\frac{l}{d} \le \alpha \cdot \beta \cdot \gamma \cdot \delta \cdot \lambda$

where, $\alpha = 20$

$\beta = 1$ for span < 10m

$P_t\% = 0.526\%$

$A_{st,req} = 874 mm^2$

$A_{st} prov = \frac{1000}{125} \times \pi \times 6^2 = 904.78 mm^2$

$\therefore f_s = 0.58 \times \frac{874}{904.78} = 232.48 N/mm^2$

$\therefore$ From fig. 4,

$\gamma = 1.32$

$\delta = 1$

$\lambda = 1$

Here,

$29.07 \le 20 \times 1 \times 1.32 \times 1 \times 1$

$29.07 \le 26.4$

Not ok

Increased

Design of compression Members

- Column:- height $\geq$ 3 * Lateral dimension
- Pedestal: height < 3 * Lateral dimension
- Short column & Long column:-

  As per IS456, Cl. 25.1.2;

  $\frac{\text{Effective height } (l_e)}{\text{least lateral dimension}} < 12 \longrightarrow$ short column

  $\geq 12 \longrightarrow$ Long/Slender column

Annex E, Fig 26, Table 28

- Effective length of compression Member:

  (Column/strut)

  $\beta_1 = \frac{\Sigma K_c}{\Sigma K_c + \Sigma K_b}$ at top

  $\beta_2 = \frac{\Sigma K_c}{\Sigma K_c + \Sigma K_b}$ at bottom

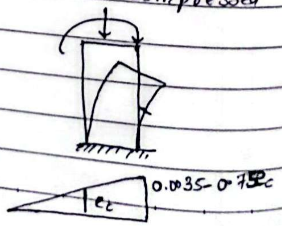
- Limit state of collapse in compression:

  Assumptions:

  → All assumptions for flexure are valid except:

  → Max^m compressive strain in concrete in axial compression is 0.002.

  → The max^m compressive strain at highly compressed extreme fibers in concrete subjected to axial compression and bending is 0.0035 minus 0.75 times strain at the least compressed extreme fibres.



- Minimum Eccentricity:

  All compression members shall be designed for the minimum eccentricity given below:

  $$e_{min} = \left(\frac{l}{500} + \frac{b}{30}\right) \text{ or } 20\,mm$$

  whichever is greater

  l = unsupported length of column

  b = lateral dimension along which eccentricity is calculated.

- Slenderness Limits for column:

  The code (Cl. 25.3.1, IS456) specifies that the ratio of unsupported length to the least lateral dimension (d) of a column should not exceed a value of 60.

  i.e. $\frac{l}{d} \leq 60$

  If one end of column is free in any plane, then

  $$l \leq \frac{100 b^2}{D}$$

  where, b = width of cross section

  D = depth of cross section

  (measured in the plane under consideration)

lateral load carrying column
lateral design

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Date

• Code Requirements for reinforcement & detailing:

→ Minimum Reinforcement → 0.8% of gross section

→ Maximum " → should not be greater than 4% normally but in extreme case can be taken as 6%.

→ Minimum diameter of rebar should not be less than 12mm and should not be spaced more than 300mm c/c

→ Minimum number of rebars in rectangular column should ~~not~~ be ≥ 4.
and in that of circular column is 6.

→ Minimum clear cover of 40 mm to lateral ties. (stirrups की बाहिर) is recommended. And a reduced clear cover of 25mm is permitted for small-sized column with lateral dimension less or equal to (≤) 200mm.

→ Lateral ties :

• tie dia., $\phi_t \geq \begin{cases} \dfrac{\phi_{long.,max}}{4} \\ 6mm \end{cases}$ (≥ 8mm as per IS 13920) (overseed)

↑ IS456

• tie spacing, $S_t \leq \begin{cases} D \text{ (Min}^{m} \text{ lateral dimension of column)} \\ 16 \times \phi_{long.,min} \\ 300mm \end{cases}$ ← IS 456

But,

$S_t \leq \begin{cases} D/4 \\ 6 \times \phi_{long.,min} \\ 100mm \end{cases}$ IS 13920 (Cl. 8.1.(b))

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→ Spiral/Helical reinforcement :

• Pitch, $S_t \leq \begin{cases} 75mm \\ \dfrac{\text{Core dia}}{6} \end{cases}$ (Maximum)

Pitch, $S_t \geq \begin{cases} 25mm \\ 3\phi_t \end{cases}$ (Minimum)

→ As per IS 13920; Cl 8.1.(c)

$A_{sh} = \text{Maximum of} \begin{cases} 0.09\, S_v D_k \left(\dfrac{A_g}{A_k} - 1\right) \\ 0.024\, S_v D_k \left(\dfrac{f_{ck}}{f_y}\right) \end{cases}$

A_{sh} = cross section of bar forming link/spiral
S_v = Pitch/spacing
D_k = Core diameter

(IS456)
→ As per 39.4 → strength of column with helical/spiral reinforcement satisfying the condition 39.4.1 shall be taken as 1.05 times the strength of similar column with lateral ties.

IS456 Cl. 39.4.1

↳ The ratio of volume of helical reinforcement to the volume of core shall not be less than

$$0.36\left(\frac{A_g}{A_c} - 1\right)\left(\frac{f_{ck}}{f_y}\right)$$

Design of Axially Loaded Short Column
(Cl. 39.3, IS 456)

→ When $\frac{l_e}{b} < 12$ (or D)

& eccentricity, $e < 0.05D$

Min^m eccentricity $e_{min} < 0.05D$

Then,

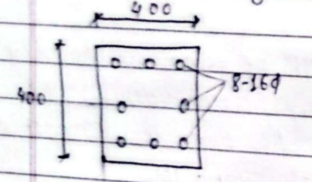
$P_u = f_c A_c + f_s A_{sc}$

$= 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

Example:-

Calculate P_u for Fe250, M20

unsupported length of column = 2.25m



→ Sol^n:-

$\frac{L}{b} = \frac{2.25*1000}{400} = 5.625 < 12$

→ Short column

$e_{min} = \left(\frac{l}{500} + \frac{b}{30}\right)$ or 20mm

$= \left(\frac{2.25*1000}{500} + \frac{400}{30}\right)$ or 20mm

= 17.833 mm or 20mm

= 20mm

Also, Check,

$0.05*b = 0.05*400 = 20mm \geq e_{min}$

$\therefore P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$= 0.4*20(A_g - A_{sc}) + 0.67*250*A_{sc}$

$= 0.4*20(400*400 - 8\pi*8^2) + 0.67*250*8\pi*8^2$

= 1536,476N

≃ 1536.5kN

Eg. Design RC Rectangular column for following data.

$P_u = 3000kN$

$l_e = 3m$ [Take, D = 1.5b]

L = 3.25m (unsupported)

M20 concrete & Fe415

Find Preliminary size of column (Rectangular)

Sol^n:-

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

Assume, $P = \frac{A_{sc}}{bD} * 100\% = 2.0\%$

$\Rightarrow A_{sc} = \frac{2}{100} * A_g = 0.02A_g$

$\therefore 3000*10^3 = 0.4*20*(A_g - 0.02A_g) + 0.67*415*0.02A_g$

$\Rightarrow A_g = 223863.89 \, mm^2$

$\therefore D*b = 223863.89$

$\Rightarrow 1.5b*b = 223863.89$

$\Rightarrow b = 386.32mm$

$\& D = 1.5b = 579.48mm$

Adopt, D = 600 mm
b : 400 mm { uniaxial column design को / Min^m 400 mm लिएको }

* Check for minimum (accidental) eccentricity.

$e_{D,min} = \frac{L}{500} + \frac{D}{30} \nless 20mm$

$= \frac{3250}{500} + \frac{600}{30}$

$= 26.5mm \nless 0.05D$

$(0.05D = 0.05*600 = 30 mm)$

OK.

$e_{b,min} = \frac{L}{500} + \frac{b}{30} \nless 20mm$

$= \frac{3250}{500} + \frac{400}{30}$

$= 19.83mm \nless 20mm$

$= 20mm$

$(0.05b = 0.05*400 = 20mm)$

O.K.

* Check for slenderness ratio

$\frac{L_e}{D} = \frac{3000}{600} = 5 < 12$

$\frac{L_e}{b} = \frac{3000}{400} = 7.5 < 12$

→ (8, 10, 12, 16, 20, 25, 32)

* Design for longitudinal rebar.

For Axially loaded short column,

$P_u = 0.4 f_{ck} (A_g - A_{sc}) + 0.67 f_y A_{sc}$

or $300*10^3 = 0.4*20*(400*600 - A_{sc}) + 0.67*415*A_{sc}$

$\Rightarrow A_{sc} = 3999.26 mm^2$

∴ Provide, 4-32ϕ & 4-25ϕ steel.

$\therefore A_{st,prov.} = 4\pi \left[\left(\frac{32}{2}\right)^2 + \left(\frac{25}{2}\right)^2 \right]$

$= 5180.49 mm^2$

* Design of lateral ties / transverse reinforcement:

$\phi_t \nless 6mm$ (IS456)

$\nless 8mm$ (IS13920)

$\nless \frac{D_{long,max}}{4} = \frac{32}{4} = 7.5$ (max^m long dia लिने)

IS456

∴ Adopt, $\phi_t = 8mm$

$S_v \leq$ lateral dimension (= 400 mm)

$< 300 m$

$< 16 \phi_{long,min} = 16*25 = 400mm$

IS13920

$S_v < 300 mm$ } other zone

$S_v < D/2$

$S_v \leq D/4$ } special confining zone

$S_v \leq 6\phi_{long} = 150$ (min^m long dia लिने)

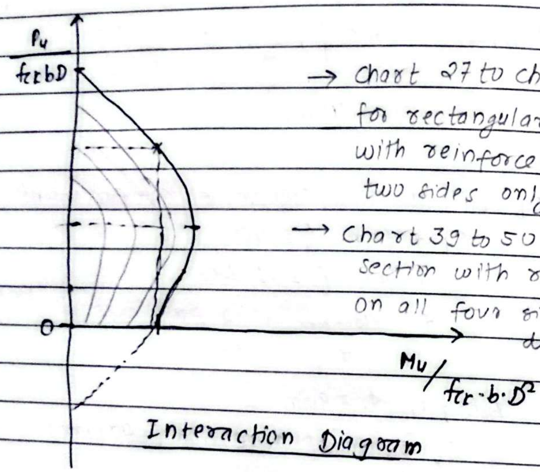
$S_v \leq 100$

Provide 8ϕ lateral ties @ 100mm c/c in special confining zone and @ 20 mm c/c in other zone.

Compression member subjected to combined axial and uniaxial bending:-

→ For given cross section dimension and given area of reinforcement in column.

└ $p = \frac{A_{sc}}{bD} \times 100\%$



Interaction Diagram

→ Chart 27 to chart 38 are for rectangular section with reinforcement on two sides only.

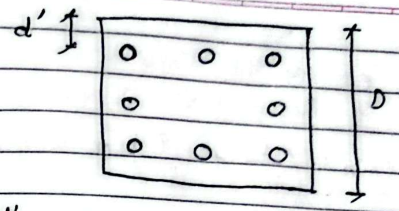
→ Chart 39 to 50 for rectangular section with reinforcement on all four sides equally distributed

→ Chart 27 to chart 38 for rectangular section with reinforcement on two sides only.

→ Chart 39 to 50 for rectangular section with reinforcement on all four sides equally distributed.

→ Chart 51 to 62 for circular section.

(chart → diff. grade steel को लागी छिमेको हुन्छ for Fe250, Fe415, Fe500)



$\frac{d'}{D} = 0.05, 0.10, 0.15, 0.20$

(different $\frac{d'}{D}$ ratio को लागी for each grade को steel chart छिमेको हुन्छ)

Design Steps:- (P_u, M_u) uniaxial moment

(1) Determine axial load and bending moment acting on column, based on suitable analysis method. Calculate factored load & moment.

(2) Select trial cross section dimension b & D based on experience, grade of steel, and grade of concrete.
(Gen. M20 concrete & Fe500 ⇒ common in market)

④ Minimum size of column should not be less than 200mm in normal case and 300mm for lateral load resisting column (earthquake/wind load resisting column)

eg. Dimensioning P_u, M_u → uniaxial case.

⇒ $(1.20 \text{ to } 1.25) * P_u$ pure गर्दै Area of section निकाल्दा

⇒ $(1.20 \text{ to } 1.25) * P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

And, Assume. % of steel $p = \frac{A_{sc}}{bD} \times 100\%$ (सके सम्म थोरै value assume) ⇓ ≈1%

(3) Check

eccentricity, $e = \frac{M_u}{P_u}$

& $e_{min} = \left(\frac{L}{500} + \frac{D}{30}\right)$ or 20mm

if $e < e_{min}$

& $e < 0.05b$ & $0.05D$

And, $\frac{L_e}{D} < 12$ & $\frac{L_e}{b} < 12$

If O.K

fail in both ⇓ biaxial design

↳ जुनको लागि eccentricity मा fail हुन्छ त्यसको लागि doing

If OK design as axially loaded column.

Otherwise, design column as axially loaded along with uniaxial moment

(4) Design of reinforcement for column with axial load & uniaxial moment

Calculate $d'/D = n$, $\frac{P_u}{f_{ck} \cdot bD} = n$, $\frac{M_u}{f_{ck} \cdot bD^2} = n$

read percentage of reinforcement (p) from chart (27 to 38) in case of reinforcement distributed on two ~~sides~~ ~~equally~~ ~~nearly~~ opposite sides equally only ~~and~~

and

from chart (39 to 50) in case of reinforcement distributed equally on four sides of column.

'OR'

from chart (51 to 62) in case of circular column

Example:-

Design a RC column with both ends fixed and effectively held in position is subjected to $P_u = 3375$ kN and $M_{ux} = 500$ kNm. Design column of square section. Take M20, Fe415. (Take effective length as 4m)

→ Soln:-

→ Find preliminary size of column,

(Cl. 39.3) $1.20 P_u = 0.4 f_{ck}\left(A_g - \frac{pA_g}{100}\right) + 0.67 f_y \cdot \frac{p}{100} * A_g$

Take, $p = 2\%$.

or, $1.20 * 3375 * 10^3 = 0.4 * 20[A_g - 0.02 * A_g] + 0.67 * 415 * 0.02 * A_g$

$\Rightarrow A_g = 302216$ mm²

∴ $b = D = 549.74$mm

Adopt 600 * 600 size of column (though can adopt 550 as well)

∴ $b = D = 600$mm

→ Check for eccentricity:-

$e_{b,min} = e_{D,min} = \frac{L}{500} + \frac{b}{30} \nless 20$mm

$= \frac{4000}{500} + \frac{600}{30}$

$= 28$ mm

$e_{D,actual} = \frac{M_{ux}}{P_u} = \frac{500*1000}{3375} = 148.15$ mm.

$0.05D = 0.05 * 600 = 30$mm

$e_{D,actual} > 0.05D$

↳ Design for uniaxial column

→ Check for slenderness ratio:

$$\frac{l_e}{D} = \frac{l_e}{b} = \frac{0.65*4000}{600} = 4.33 < 12$$

→ short column

⇒ Design of column as column is loaded with axial load & uniaxial Moment.

$$\frac{P_u}{f_{ck} \cdot b \cdot D} = \frac{3375*10^3}{20*600*600} = 0.469$$

$$\frac{M_{ux}}{f_{ck} \cdot b \cdot D^2} = \frac{500*10^6}{20*600*600^2} = 0.116$$

$$\frac{d'}{D} = \frac{56}{600} = 0.093$$

$$[d' = 40 + 8 + \frac{16}{2} = 56mm$$

For $f_y = 415MPa$ & $\frac{d'}{D} = 0.05$ (chart 43)

$$\frac{p}{f_{ck}} = \cancel{0.098}\ 0.1$$

And for $f_y = 415MPa$ & $\frac{d'}{D} = 0.10$ (chart 44)

$$\frac{p}{f_{ck}} = \cancel{0.13}\ 0.11$$

$$\text{Reqd } \frac{p}{f_{ck}} = 0.10 + \frac{0.11-0.10}{(0.10-0.05)}(0.093-0.05)$$

$$= 0.1086$$

$$\therefore \frac{A_{sc}}{f_{ck}\, bD} \cancel{100\%} = 0.1086 = \frac{p}{f_{ck}}$$

$$\text{or, } A_{sc} = \frac{0.1086*600*600*20}{100}$$

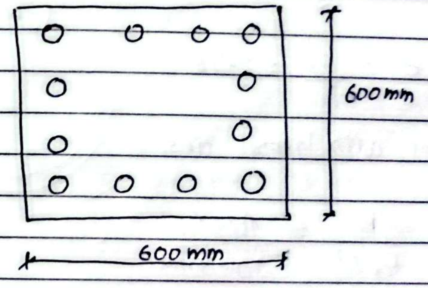
$$= 7819.2\ mm^2$$

Using 32φ rebar

$$\text{No. of rebars} = \frac{7819.2}{\pi * 16^2} \simeq 9.72 = 12 \text{(adopt)}$$

(equally distributed on ~~four~~ side adopt [illegible])

∴ ~~Desig~~ provide 12 - 32φ reinforcement equally distributed on four sides and provide lateral ties as minimum required by code.



Example

Determine the reinforcement to be provided in a circular column with following data.

Diameter of column = 50cm

Grade of column = M20

" " steel = Fe500

P_u = 1600KN

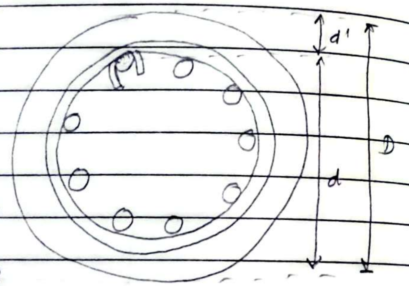
M_u = 125KNm

→ Soln; Here,

Assuming 20mm dia longitudinal rebar & 8mm ϕ lateral ties are used with 40mm clear cover.

Now,

$d' = 40 + 8 + \frac{20}{2} = 58mm$



Now, $\frac{d'}{D} = \frac{58}{500} = 0.116$

For column with lateral ties,

$\frac{P_u}{f_{ck} b D} = \frac{P_u}{f_{ck} D^2} = \frac{1600 \times 10^3}{20 \times 500^2}$

(b = D) circular

$= 0.32$

$\frac{M_u}{f_{ck} \cdot b D^2} = \frac{125 \times 10^6}{20 \times 500^3} = 0.05$

For, f_y = 500 MPa, referring to,

Chart 60, $\frac{d'}{D} = 0.1 \rightarrow \frac{p}{f_{ck}} = 0.05$

chart 61, $\frac{d'}{D} = 0.15 \rightarrow \frac{p}{f_{ck}} = 0.055$

∴ Required $\frac{p}{f_{ck}}$ by linear interpolation,

$\frac{p}{f_{ck}} = 0.05 + \frac{0.055 - 0.05}{(0.15 - 0.10)} \times (0.116 - 0.1)$

$= 0.0516$

∴ $p = 0.0516 \times 20$

$= 1.032\% > 0.8\%$

(ok).

??

∴ $\frac{A_{st}}{\pi D^2} \times 4 \times 100 = 1.032$

?

or, $A_{st} = 2026.327 mm^2$

∴ No of 20mm dia rebar,

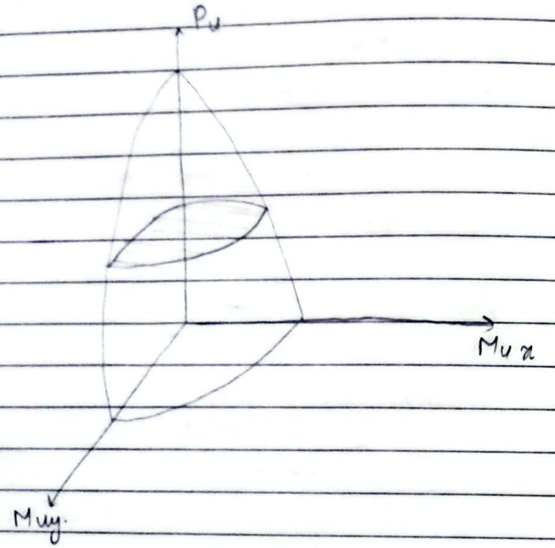
$= \frac{2026.327mm^2}{\pi \times 10^2} = 6.45 \approx \underline{\underline{7}}$.

∴ Provide 7 Nos. of 20ϕ steel.

Provide Lateral ties as minimum required by IS 456/ IS 13920.

Compression member subjected to combined axial load & biaxial loading

$\frac{l_e}{b}$ and $\frac{l_e}{D} < 12$; e_x & $e_y > 0.05b$ & $0.05D$



$$\left(\frac{M_{ux}}{M_{ux,1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy,1}}\right)^{\alpha_n} \le 1$$

M_{ux}, M_{uy} → moments about X & Y-axis due to design loads respectively.

$M_{ux,1}$, M_{uy1} → Maximum, uniaxial moment capacity For an axial load P_u, bending about x- & y-axes respectively.

Value of α_n depends upon $\frac{P_u}{P_{uz}}$, where

Cl. 39.6

$$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$$

P_u/P_{uz}		α_n
≤ 0.2	⟶	1.0
≥ 0.8	⟶	2.0

For intermediate values of $\frac{P_u}{P_{uz}}$, interpolation can be used.

Cl. 39.7

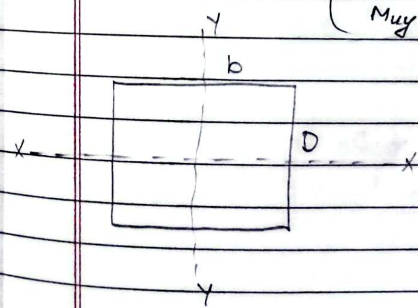
$$\left.\begin{array}{l} \frac{L_e}{b} > 12 \\ \frac{L_e}{D} > 12 \end{array}\right\} \text{Slender}$$

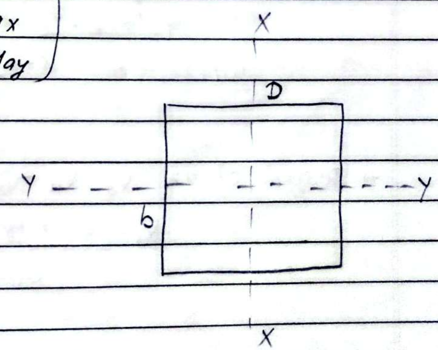
Then,

$$M_{ax} = \frac{P_u \cdot D}{2000}\left(\frac{L_{ex}}{D}\right)^2$$

$$M_{ay} = \frac{P_u \cdot b}{2000}\left(\frac{l_{ey}}{b}\right)^2$$

Then, $\begin{pmatrix} P_u \\ M_{ux} + M_{ax} \\ M_{uy} + M_{ay} \end{pmatrix}$





Cl. 39.7.1.1

k multiply to Max & May

$$\left\{K = \frac{P_{uz} - P_u}{P_{uz} - P_b} \le 1\right\}$$

$$K = \frac{1 - P_u/P_{uz}}{1 - P_b/P_{uz}}$$

SP-16 Table 60 Pg 131

Table 60

- Rectangular section:-

$$\frac{P_b}{f_{ck}\, bD} = k_1 + k_2 \cdot \frac{p}{f_{ck}}$$

- Circular section:

$$\frac{P_b}{f_{ck}\, D^2} = k_1 + k_2 \cdot \frac{p}{f_{ck}}$$

* Design of column in shear

→ $\tau_v > \tau_c'$

$\tau_c' = \delta \cdot \tau_c$

where, $\delta = 1 + \frac{3P_u}{A_g f_{ck}} \le 1.5$

→ Design shear, $V_s = V_u - \tau_c' \cdot b \cdot D$

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

Q Example:-

Design a RC column for the following data:

$P_u = 1000\,kN$

$M_{ux,bottom} = 110\,kN\cdot m$

$M_{uy,bottom} = 40\,kN\cdot m$

$M_{ux,top} = 80\,kNm$

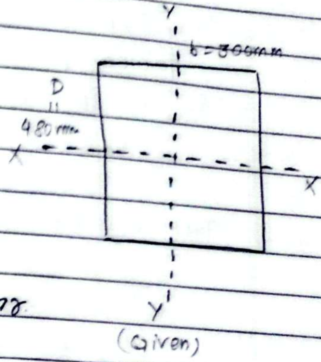
$M_{uy,top} = 30\,kNm$

Unsupported length, $L = 5.8\,m$

effective length, $l_{ex} = 5.4\,m$

$l_{ey} = 4.2\,m$

M20 concrete & Fe415 rebar



→ Sol^n:- (sizing not given)

→ Check for slenderness ~~given~~ ratio:

$$\frac{l_{ex}}{D} = \frac{5.4 * 1000}{480} = 11.25 < 12$$

$$\frac{l_{ey}}{b} = \frac{4.2 * 1000}{300} = 14 > 12$$

Column is short about x-x axis and slender about Y-Y axis.

→ Find the additional moment about y-y axis & design BM:-

$$M_{ay} = \frac{P_u \cdot b}{2000}\left(\frac{l_{ey}}{b}\right)^2 * K$$

where, $K = \frac{P_{uz} - P_u}{P_{uz} - P_b}$

$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$

(Take 2.5% steel)

$P_{uz} = 0.45 * 20 * (300*480 - 0.025*300*480)$
$\quad + 0.75*415*0.025*300*480$

$= 2384100N$

Table 60,

$$\frac{P_b}{f_{ck}\, b \cdot D} = k_1 + k_2 \cdot \frac{P}{f_{ck}}$$

Take effective cover, $d' = 60mm$.

$\cancel{\frac{d'}{D} = \frac{60}{480} = 0.125}$

$$\frac{d'}{b} = \frac{60}{300} = 0.2$$

किनकि यदि additional moment निकाल्न खोजेको हो → My

Thus,

$\therefore k_1 = 0.184$

$k_2 = 0.028$

$$\therefore \frac{P_b}{f_{ck}\, b\, D} = 0.184 + 0.028 * \frac{2.5}{20}$$

$= 0.1875$

$P_b = 0.1875 * 20 * 300 * 480$

$= 540000N$

$$\therefore k = \frac{\overset{P_{uz}}{2384100} - \overset{P_u}{1000000}}{\underset{P_{uz}}{2384100} - \underset{P_b}{540000}}$$

$= 0.75$

$$\therefore M_{ay} = \frac{P_u \cdot b}{2000}\left(\frac{l_{ey}}{b}\right)^2 * k$$

$$= \frac{1000*300*10^{-3}}{2000}\left(\frac{4.2*1000}{300}\right)^2 * 0.75$$

$= 22.05\, kNm$

$M_{ax} = 0$

$\therefore$ Design loads:-

$P_u = 1000kN$

$M_{ux} = 110kN$ (max^m of top & bottom)

$M_{uy} = 40 + 22.05 = 62.05kNm$ //

→ Check for eccentricity:

$$e_{D,min} = \frac{L}{500} + \frac{D}{30} = 27.6mm > 0.05D$$
$= 24mm$

$$e_{b,min} = \frac{L}{500} + \frac{b}{30} = 21.6mm > 0.05b$$
$= 15mm$

* Actual eccentricity

$$e_D = \frac{M_{ux}}{P_u} = \frac{110*10^6}{1000*10^3} = 110mm$$
$> 0.05D.$

$$e_b = \frac{M_{uy}}{P_u} = \frac{62.05*10^6}{1000*10^3} = 62.05mm$$
$> 0.05b$

→ Now,

$$\left(\frac{M_{ux}}{M_{ux,l}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy,l}}\right)^{\alpha_n} \leq 1$$

→ Design/check for longitudinal reinforcement :-

* Uniaxial moment capacity of section about x-x axis:-

$d' = 60mm$

x-axis ←

$$\therefore \frac{d'}{D} = \frac{60}{480} = 0.125$$

$$\frac{P}{f_{ck}} = \frac{2.5}{20} = 0.125$$

$$\frac{P_u}{f_{ck} \cdot bD} = \frac{1000 \times 10^3}{20 \times 300 \times 480} = 0.347$$

For $f_y = 415$, referring chart chart 44, & chart 45,

chart 44 (for $d'/D = 0.1$) $\rightarrow \frac{M_u}{f_{ck} \cdot bD^2} = 0.135 \rightarrow 0.16$

Chart 45

for $d'/D = 0.15 \rightarrow \frac{M_u}{f_{ck} \cdot bD^2} = \cancel{0.135}\ 0.14 \rightarrow$

Required, $\frac{M_u}{f_{ck} \cdot bD^2} = 0.1375$

$\therefore M_{ux,1} = 0.1375 \times 20 \times 300 \times 480^2$

$= 190080000\ N \cdot mm$

$= 190.08\ KNm.$

* Uniaxial moment capacity of section about, y-axis.

$d' = 60mm$

$\therefore \frac{d'}{b} = \frac{60}{300} = 0.2$

$$\frac{P}{f_{ck}} = \frac{2.5}{20} = 0.125$$

$$\frac{P_u}{f_{ck}\,bD} = \frac{1000 \times 10^3}{20 \times 300 \times 480} = 0.347$$

For $f_y = 415$,

Referring chart 46,

for $\frac{d'}{D} = 0.2 \rightarrow \frac{M_u}{f_{ck} \cdot Db^2} = 0.12$

$\therefore M_{uy,1} = 0.12 \times 20 \times 480 \times 300^2$

$= 103.68\ KNm.$

Now,

$$\frac{P_u}{P_{uz}} = \frac{1000}{2384.1} = 0.4195$$

$$\therefore \alpha_n = 1 + \frac{(2-1)}{0.8-0.2}(0.4195 - 0.2) = 1.367$$

Check for interaction relation:-

$$\left(\frac{M_{ux}}{M_{ux,1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy,1}}\right)^{\alpha_n}.$$

$$= \left(\frac{110}{190.08}\right)^{1.37} + \left(\frac{62.05}{103.68}\right)^{1.37}$$

$= 0.967 < 1$

$\rightarrow$ Hence, section is safe for $P = 2.5\%$.

Now,

$$\frac{A_{sc}}{300 \times 480} \times 100 = 2.5$$

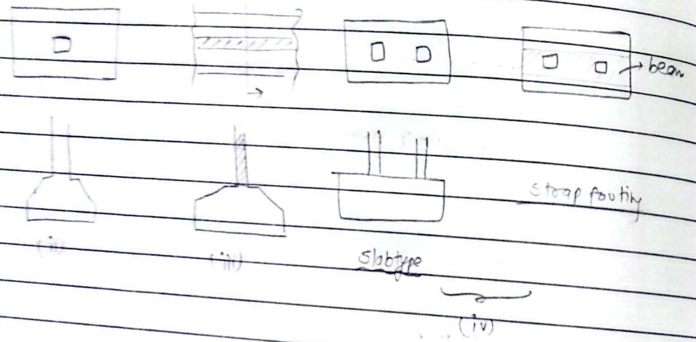
$\Rightarrow A_{sc} = 3600\ mm^2$

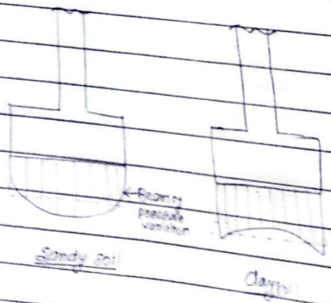
Provide $12 - 20\phi$ reinforcement with actual

$A_{sc} = 12 \times \pi \times 10^2 = 3770\ mm^2$

Design of Reinforced Concrete Foundation

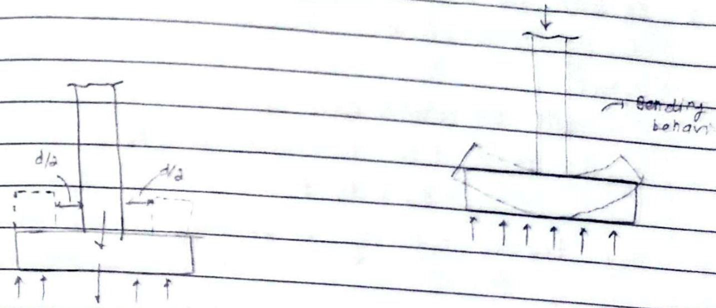
Foundation
- Shallow Foundation
  - (i) Spread Footing
  - (ii) Isolated Footing (column)
  - (iii) Strip Footing (wall)
  - (iv) Combined Footing
    - Slab type (Flat type)
    - Strap Footing
- Deep Foundation
  - (i) Pile Footing
  - (ii) Well / cassion foundation

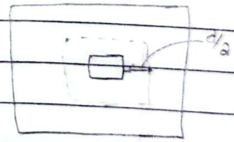




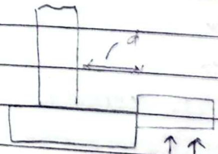
tension not allowed in bearing pressure

* Foundation behaviour under load:





→ Two-way shear (punching shear)



One-way shear

* Design steps of concentrically loaded isolated Footing/Foundation.

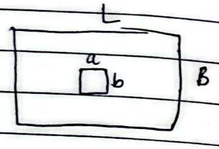
→ Given,
  size of column ~~axis~~ $(a \times b)$

→ Service load, P or factored load, P_u
  $P_u = 1.5P$

→ Safe bearing capacity of soil, q_a (Apply factor of safety from 2 to 3)
  (allowable bearing capacity)

→ Grade of concrete

→ Grade of steel.



(Step 1) Calculate area of footing required based on service load P

i.e. $A_{req} = \frac{1.15 \times P}{q_a} = \frac{1.15}{q_a} \times \frac{P_u}{1.5}$

where, factor is 1.15 is used to consider load due to self wt. of foundation & overburden soil as 15% of applied.

(Step 2) Calculate the size of footing such that

$L \times B = A_{req}$ $\quad [(L-a) = (B-b)]$

$\Rightarrow L \times (L - a + b) = A_{req}$

$\Rightarrow$ gives $\underline{\underline{L \,\&\, B}}$

Step 3: Calculate factored bearing pressure on footing by soil.

$$q_u = \frac{1.15 \times P_u}{L \times B} = \frac{1.15 \times 1.5 \times P}{L \times B}$$

(Step 4) Thickness of footing based on shear

(a) One way shear

→ The critical section lies at a distance 'd' from face of column

shear strength ≥ actual shear force

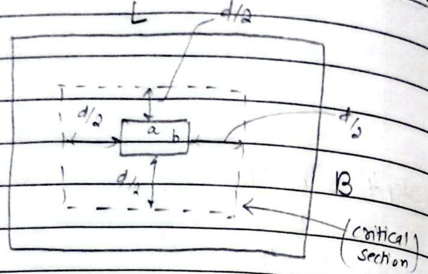
$$B \times d \times \tau_c \geq B \left\{ \frac{(L-a-2d)}{2} \right\} \times q_u$$

or $$L \times d \times \tau_c \geq L \left\{ \frac{(B-b-2d)}{2} \right\} \times q_u.$$

i.e. $$d \times \tau_c \geq \frac{(L-a-2d)}{2} \times q_u$$

where τ_c is as per Table 19; IS456
(For this purpose, P_t can be assumed as $(0.15 - 0.25\%)$)

(b) Two-way / Punching shear [τ_c as per Cl. 31.6.3; IS456]



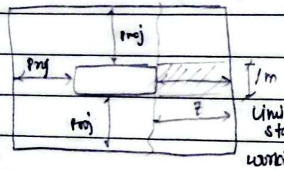
→ the critical section lies at $d/2$ from the periphery of the column.

Shear strength ≥ actual punching force.

i.e. $2[(a+d)+(b+d)]*d*k_s*\tau_c \ge q_u[L*B-(a+d)(b+d)]$

where,

$k_s = 0.5+\beta_c \le 1$



$\beta_c = b/a$;

limit state ⇒ $\tau_c = 0.25\sqrt{f_{ck}}$ (Cl. 31.6.3)

working stress ⇒ $\tau_c = 0.16\sqrt{f_{ck}}$

Greater value of d from (a) & (b) adopted.

(step 5) Design for flexural reinforcement:

* Factored moment at column face per m width

$$M_u = q_u * z * \frac{z}{2} \quad \left(\frac{l-a}{2} \text{ or } \frac{B-b}{2}\right)$$

Now,

$$M_u = 0.87 f_y A_{st} d'\left(1 - \frac{f_y A_{st}}{f_{ck} b d}\right) \quad \text{— (i)}$$

gives Ast.

Check for $A_{st,min}$: $\frac{0.12*1000*D}{100}$ — (ii)

Check for Ast assumed for one way shear

$$A_{st} = \frac{P_t * D * 1000}{100} \quad \text{— (iii)}$$

$$D = d + \frac{\phi}{2} + \phi + 50$$

Step 6: Check Bearing stress on Footing

Actual Bearing stress = P_u

Design of footing:

Example: Column size 400mm x 400mm

$DL = 100KN$

$LL = 400KN$

$q_{allowable} = 200KN/m^2$

M20 & Fe415

(i) Determine size of footing

$$A = \frac{1.15(DL+LL)}{q_{allow}} = \frac{1.15(100+400)}{200} = 8.05\ m^2$$

(ii) Size of foundation:

$L = B = \sqrt{A} = 2.84$

$\approx 2.9m$ (adopt)

(iii) Factored Bearing pressure

$$q_u = \frac{1.15(100+400)*1.5}{2.9*2.9}$$

$= 287.16\ KN/m^2$

(iv) Thickness of footing based on shear force

a. One way shear

Shear strength $\geq$ actual shear force

i.e. $B*d*\tau_c \geq B\left\{\frac{L-a-2d}{2}\right\}*q_u$

where, $\tau_c = 0.28 N/mm^2$

for $P_t = 0.13\%$ (say)

i.e. $d * 0.28 \geq \frac{2900-400-2d}{2} * 287.16*10^{-3}$

$[d \geq 632.89mm]$

b) Two way shear

Shear strength $\geq$ actual punching force

$(a+d+b+d)*2*d*K_s*\tau_c \geq q_u\{L*B-(a+d)(b+d)\}$

i.e. $(800+2d)*2*d*1*0.25\sqrt{20} \geq 287.16*10^{-3}[2900^2-(400+d)^2]$

where, $\tau_c = 0.25\sqrt{f_{ck}}$

& $K_s = 0.5+\beta_c \leq 1$

$\beta_c = \frac{b}{a} = \frac{400}{400} = 1$

$\Rightarrow d \geq 524.648mm$

$\therefore$ Finally $d \geq 632.89mm$

$\therefore D = 632.89 + 16 + \frac{16}{2} + 50$ (Assume 16ϕ rebar both ways)

$= 706.89mm$

$\approx 725mm$ (say)

$\therefore$ Actual $d = 725 - 16 - \frac{16}{2} - 50 = 651mm$

(v) Design for flexural reinforcement:

lever arm, $Z = \frac{L-Z_L}{2} = \frac{2900-400}{2} = 1250mm$

($\because 1m = 1000mm$)

$\therefore M_u = \frac{q_u*Z^2}{2} = \frac{287.16*10^{-3}*1250^2}{2}*1000$

$= 224343.750\ Nmm$

Now,

$M_u = 0.87 f_y A_{st} d\left\{1-\frac{f_y A_{st}}{bdf_{ck}}\right\}$

$\Rightarrow 224343750 = 0.87*A_{st}*415*651*\left\{1-\frac{415*A}{1000*651*20}\right\}$

$\Rightarrow (A_{st} = 989.428mm^2)$

Check for

$A_{st,min} = 0.0012 * 1000 * 725$

$= 870 \text{ mm}^2$

A_{st} for $(P_t = 0.15\%) = \frac{0.15}{100} * 1000 * 725$

$= 1087.5 \text{ mm}^2$

Let us provide 12Φ @ 100 mm c/c both ways.

$\therefore A_{st,prov} = \frac{1000}{100} * \pi * 6^2 = 1130.97 \text{ mm}^2$

(OK)

(vi) Check for bearing stress (at column base)

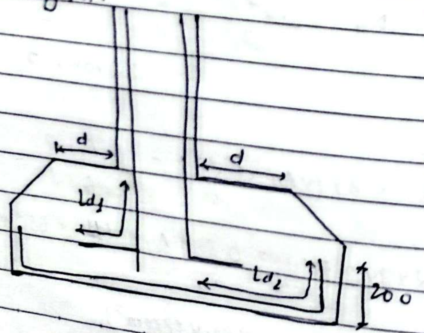
Bearing stress $= \frac{1.5 * 1400 * 1000}{400 * 400} = 13.125 \text{ N/mm}^2$

$\therefore$ Permissible bearing stress $= 0.45 f_{ck} * \sqrt{\frac{A_g}{A_2}}$

where, $\sqrt{\frac{A_g}{A_2}} = \sqrt{\frac{(2900)^2}{(400)^2}} \not> 2$

$= 0.45 * 20 * 2 = 18 \text{ N/mm}^2 > 13.125$. OK

Development length:



(Cl. 26.2.1)

$\sigma_s = \frac{f_y}{1.15} = 0.87 f_y$

(Assuming column rebar of 25 mm.)

$L_{d1} = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 * 415 * 25}{4 * 1.2 * 1.6} = 1175 \text{ mm}$

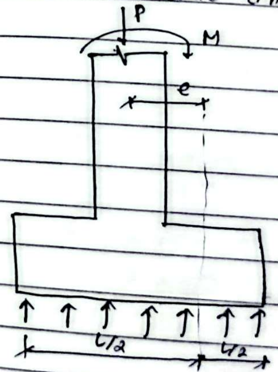
$1175 - 84 - 651 = 325 \approx 350 \text{ mm}$

$L_{d2} = \frac{0.87 * 12 * 415}{4 * 1.2 * 1.6} = 564 < 1250 \text{ mm}$

ISOLATED FOOTING

[Look in book easily explained, Pillai & Menon]

(with axial load and uniaxial moment)

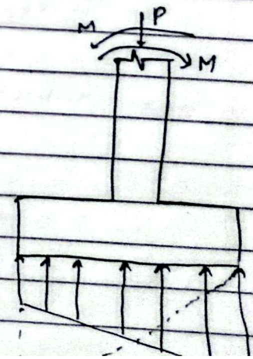


Area $= \frac{1.1 * P_u}{q_a}$

Reversal moment (load reversal)

$q_{min} > 0$

$q_{max} < q_a$.



$$e = \frac{M}{P}$$

$$q_{max} = \frac{P}{A} + \frac{M}{I} \cdot \frac{L}{2}$$

$$\left[I = \frac{BL^3}{12} \right]$$

$$= \frac{P}{A} + \frac{P * e}{BL^3} * 12 * \frac{L}{2}$$

$$= \frac{P}{A} + \frac{6Pe}{BL^2}$$

$$q_{min} = \frac{P}{A} - \frac{6Pe}{BL^2} \geq 0$$

$$\frac{P}{BL} - \frac{6Pe}{BL^2} \geq 0$$

$$1 - \frac{6e}{L} \geq 0$$

$$1 \geq \frac{6e}{L}$$

$$[L \geq 6e]$$

If

Depth of foundation = D

allowable bearing capacity = q_a

service load from column = P

unit length of soil = γ_s

$$P + D \cdot \gamma_s A = q_a * A$$

$$\left[\text{or } A = \frac{P}{(q_a - D\gamma_s)} \right]$$

P → Let 2000 kN

M → 500 kNm

$$e = \frac{M}{1.15P} = \frac{500}{1.15 * 2000} \Rightarrow 217.39\,mm$$

$$L \geq 6e \approx 1305\,mm$$

$$q_{min} = \frac{1.15P}{L * B} - \frac{6M}{BL^2} \geq 0 \quad (\text{Assume } L = B)$$

$$\Rightarrow \frac{1.15 * 2000 * 10^3}{L * L} - \frac{6 * 500 * 10^6}{L^3} \geq 0$$

$$\Rightarrow L = 1304.347\,mm$$

∴ Adopt $L = B = 1350\,mm = 1.35\,m$

$$q_{a,min} = \frac{1.15 * 2000 * 10^3}{(1350)^2} - \frac{6 * 500 * 10^6}{(1350)^3} = 0.043\,N/mm^2$$

$$= 42.676\,kN/mm^2.$$

$$q_{a,max} = \frac{P * 1.15}{A} + \frac{6M}{BL^2}$$

$$= 2.481\,N/mm^2.$$

$$= 2481.33\,kN/m^2.$$

If $P = 2000\,kN$ $q_a = 200\,kN/m^2$
$M = 50\,kNm$

$$e = \frac{M}{1.15P} = 21.74\,mm$$

$$L \geq 6e = 130.44\,mm$$

$$q_{min} = \frac{1.15P}{L*B} - \frac{6M}{BL^2} > 0$$

$$\Rightarrow \frac{1.15*2000*10^3}{L*L} - \frac{6*50*10^6}{L^3} = 0$$

$$\Rightarrow L = 130.43\,mm.$$

$$q_{max} = \frac{1.15*2000*10^3}{L^2} + \frac{6*50*10^6}{L^3} = 200*10^{-3}$$

$$\Rightarrow L = 3454.60\,mm$$

Adopt, $L = B = 3500\,mm = 3.5\,m$

$$q_{a,min} = \frac{1.15*2000*10^3}{(3500)^2} - \frac{6*50*10^6}{(3500)^3}$$

$= 0.18\,N/mm^2$

$= 180.76\,kN/m^2$

$q_{a,max} = 1.5 * q_{a,max} = 0.1947\,N/mm^2$

$= 194.7\,kN/m^2$

factored Bearing pressure

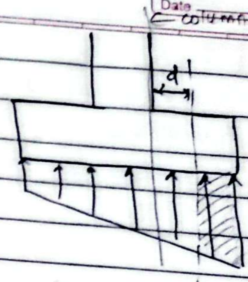
$q_{u,min} = 1.5 * q_{a,min} = 271.1\,kN/m^2$

$q_{u,max} = 1.5 * q_{a,max} = 292.05\,kN/m^2$

Shear strength $\geq$ Actual shear

$$B*d*\tau_c \geq \left(\frac{q_u + q_{max}}{2}\right)*\left(\frac{L-a}{2}-d\right)*B \quad \text{①}$$

solve for d,



$$q_{u,d} = \left[271.1 + \underbrace{\frac{292.05 - 271.1}{3.5}}_{k_1} * \left\{\underbrace{\frac{3.5-0.4}{2} + 0.4}_{k_2} + d\right\}\right]$$

$$= 271.1 + k_1(k_2 + d)$$
$\quad\quad\Downarrow\quad\Downarrow$
$\quad\quad 5.98 \quad 1.95$

One way shear

$$q_{u,d} = 271.1 + 5.98(1.95 + d)$$

Assume, $\tau_c = 0.28\,N/mm^2$ for $P_t = 0.15\%$
$= 280\,kN/m^2$.

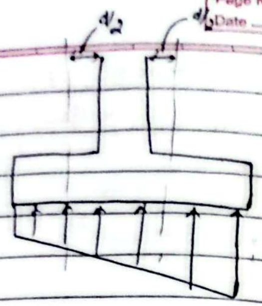
Now, from ①,

$$d*280 \geq \left(\frac{271.1 + 5.98(1.95+d) + 292.05}{2}\right)*\left(\frac{L-a}{2} - d\right)$$

($L = 3.5$, $a = 0.4$)

$\Rightarrow d \geq 0.789\,m$

$d \geq 789\,mm$



$\left(\frac{271.1 + 292.05}{2}\right) \times (0.4 + d)$

$k_s \tau_c \times (0.4 + d) \times 4 \times d \geq (\overset{3.5m}{l} - \overset{0.4m}{a} - d)^2 \times \left(\frac{271.1 + 292.05}{2}\right)$

$\downarrow \quad \downarrow$
$1 \quad 1120$

$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.12 \ N/mm^2$
$= 1120 \ kN/m^2$

$k_s = 0.5 + \beta_c = 0.5 + 1 = 1.5 < 1$

Solving,
$d \geq 0.485m$
$[d \geq 485mm]$

One-way slab governs design.
Thus, $d \geq 789mm$
$\therefore D = 789 + 50 + 8$
$= 847mm$
$\approx 850mm$ Adopt, 850mm

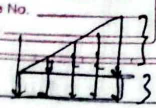
$\therefore$ Actual $[d = 792mm]$

Now,

$q_{u,column\ face,max} = 271.1 + \left(\frac{292.05 - 271.1}{3.5}\right)\left(\frac{3.5 - 0.4}{2} + 0.4\right)$

$= 282.772 kN/m^2$

$z = \frac{3.5 - 0.4}{2} = \frac{3.1}{2} = 1.55m.$



$M_{face} = \frac{282.772 \times 1.55^2}{2} + \frac{1}{2} \times 1.55 \times (292.05 - 282.772) \times \frac{2}{3} \times 1.55$

$= 347.109 kNm.$

Combined Footing
(column size : $x \times y$)

$\rightarrow$ $A_{req} = \frac{1.1(P_1 + P_2)}{q_a} = w$

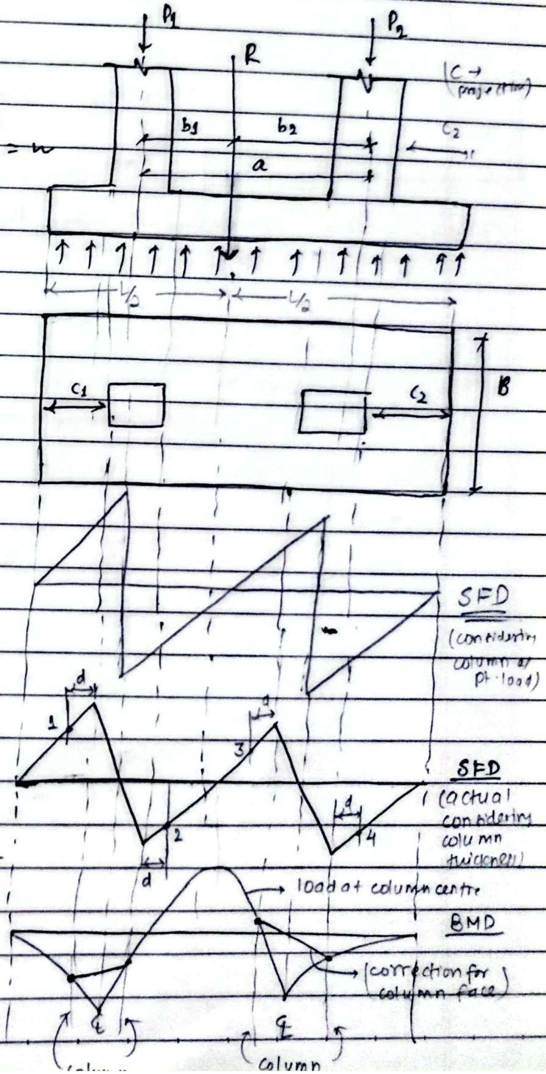
$\rightarrow$ Assume B, find $L = w$) trial

$\Rightarrow$ (Max^m of 1, 2, 3, 4 SF)

One way shear,

$\tau_c \times B \times d \geq$ actual Max^m shear force.

~ If R lies L/2 from each face bearing pressure rectangular/uniform else, bearing pressure trapezoidal)



Two way shear

Check for max. column force

$(P_1 > P_2)$ say.

$$K_s \cdot \tau_c \, (x+y+2d) * 2 * d \;\geq\; P_1 - (x+d)(y+d) * q_u$$

find d?

जो area का bending moment minimum

(Pillai)

Bending को Max के अनुसार चुनें

(AK Jain → 0.75d को place करना d)

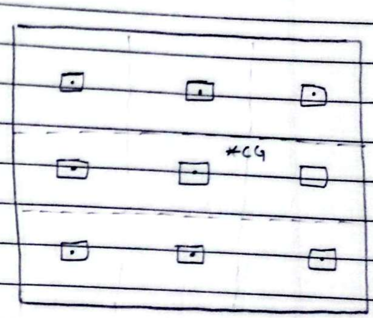
Mat raft foundation

(1) ΣF, ΣM_x, ΣM_y } at C.G. of foundation (Assume ΣF i.e. Resultant in 1st quadrant)

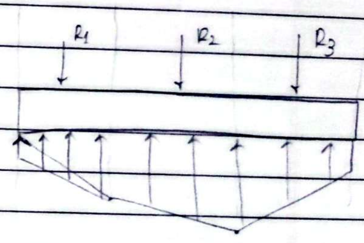
(2) Bearing pressure below each column,

$$q = \frac{\Sigma F}{A} + \frac{\Sigma M_x}{I_x} \cdot y + \frac{\Sigma M_y}{I_y} \times x$$

A = total area of mat



[IS 456 → table 12] Moment coefficient



(Strip जैसा मानें)