RCC

WSM (assumption) :-

1- concrete, steel behave in linear elastic manner.

2- stress in concrete, steel is within permissible limit.

Permissible stress = $\frac{\text{limiting strength}}{\text{FOS}}$

in WSM FOS < concrete = 3, steel = 1.82

Grade	direct tensile strength	compression direct σ_{cc}	compression bending σ_{cbc}
M15	2	4	5
M20	2.8	5	7
M25	3.2	6	8.5
M30	3.6	8	10
M35	4	9	11.5
M40	4.4	10	13

$\boxed{m = \frac{280}{3\sigma_{cbc}}}$ → for tensile bars

N/mm² σ_{st}	fe250	fe415	fe500
$\phi \le 20$	140	230	275
$\phi > 20$	130		
σ_{sc}	130	190	190

Tension steel	• shrinkage reduces tensile stress ↓ • Creep produces additional tensile stress ↑
comp. steels	• shrinkage & creep causes more stresses

$\boxed{m' = 1.5m \text{ in compression bars}}$

→ due to long term effect of creep & shrinkage of concrete and nonlinearity at higher strains in concrete results in much compressive stress in comp. steel.

WSM

$x_c = \left(\frac{mc}{mc+t}\right)d$

$x_c = kd$

$M_{bal} = Qbd^2 = \left(\frac{1}{2}cjk\right)bd^2$

$j = 1 - k/3$

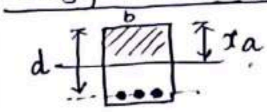
$m = 280/3\sigma_{cbc}$

$c \to \sigma_{cbc}$

$t \to \sigma_{st}$

	$x_c = kd$ (k) WSM	$x_{ulim} = kd$ (k) LSM	M_{ulim} LSM
fe250	0.400	0.53	$0.148 f_{ck} bd^2$
fe415	0.289	0.48	$0.138 f_{ck} bd^2$
fe500	0.254	0.46	$0.133 f_{ck} bd^2$

Singly reinforced section :-

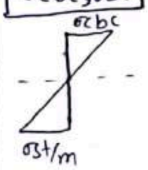


$x_a = ?$ take moment of area about NA

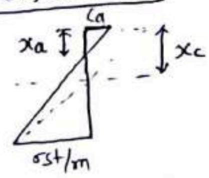
$$\boxed{(b x_a)\frac{x_a}{2} = mA_{st}(d - x_a)}$$

MOR

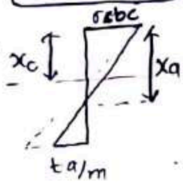
balanced section: $\boxed{x_a = x_c}$ $\boxed{\sigma_{cbc}, \sigma_{st}}$



under reinforced section: $\boxed{x_a < x_c}$ $\boxed{\sigma_{st}, C_a < \sigma_{cbc}}$



over reinforced section: $\boxed{x_a > x_c}$ $\boxed{\sigma_{cbc}, t_a < \sigma_{st}}$



Doubly reinforced section :- when provided ??

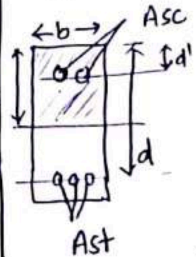
(i) where size (depth) restricted & $M > M_{balance}$

(ii) earthquake zone

use of compression steel : (i) Reduce long term Deflection

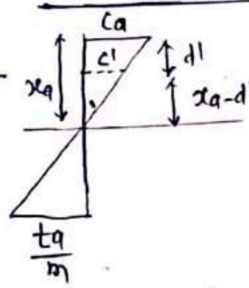
(ii) Increase ductility of beam

(iii) used as anchor bars to hold shear stirrups.



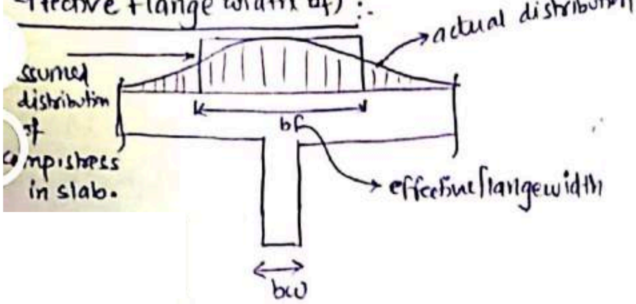
$$\boxed{\frac{b x_a^2}{2} + (1.5m - 1)A_{sc}(x_a - d') = mA_{st}(d - x_a)}$$

General eqn of MOR :-



$$\boxed{MOR = \frac{C_a}{2}(b x_a)\left[d - \frac{x_a}{3}\right] + (1.5m - 1)c' A_{sc}(d - d')}$$

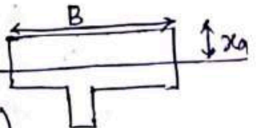
Effective flange width (b_f) :-



Beam casted monolithically with slab	Isolated beam
T: $\min\begin{cases}\frac{l_0}{6}+b_w+6D_f \\ b_w+\frac{l_1}{2}+\frac{l_2}{2}\end{cases}$	$T \Rightarrow \left[\dfrac{l_0}{\frac{l_0}{b}+4}\right]+b_w$
L:- $\min\begin{cases}\frac{l_0}{12}+b_w+3D_f \\ b_w+\frac{l_1}{2}\end{cases}$	$L \Rightarrow \left[\dfrac{0.5 l_0}{\frac{l_0}{b}+4}\right]+b_w$

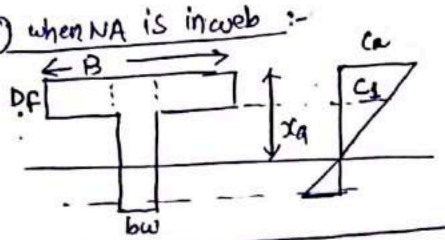
l_0 (point of zero moment dist) :-	l_0
SSB & series of beam supported on masonary wall	l_{eff}
continuous beam	$0.7\, l_{eff}$

T beam :-



(I) when NA in flange :-
(solve as single reinforced beam)

(II) when NA is in web :-

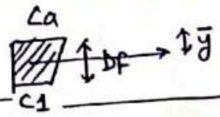


$x_a = ?$

neutral axis calculation

$$\boxed{(b_w x_a)\frac{x_a}{2}+(B-b_w)D_f\left(x_a-\frac{D_f}{2}\right) = mA_{st}(d-x_a)}$$

MOR :-



$$\boxed{(b_w x_a)\frac{c_a}{2}\left(d-\frac{x_a}{3}\right) + \frac{1}{2}(c_a+c_1)(D_f\times(B-b_w))\times[d-\bar{y}]}$$

$$\bar{y}=\left(\frac{c_a+2c_1}{c_a+c_1}\right)\times\frac{D_f}{3}$$

IS 456 :- Admixture

mineral admixture
(I) fly ash
(II) silica fume
(III) rice husk ash

chemical admixture
(1) water reducing / plasticizer [calcium lignosulphate]
(2) accelerating admixture (calcium chloride, fluo silicates, Tri ethanolamine)
(3) Retarding admixture (sodium tartrate, Tartaric acid)
(4) Air entraining Admixture [Aluminium powder, neutralised vinsol resin]

note :-
(I) Admixture → not increase strength
(II) Antibleeder → $Al_2(SO_4)_3$ (Aluminium sulphate)

Grade of concrete :- Amendment no - 4, May 2013.

(3) ordinary concrete	M10, M15, M20
(2) standard concrete	M25, M30, M35, M40, M45, M50, M55, M60
(1) high strength concrete	M65, M70, M75, M80, M85, M90, M95, M100

direct tensile strength < split tensile strength < flexural tensile strength < cylinder comp. strength < cube comp. strength

- direct tensile strength → $0.50\text{-}0.625 f_{cr}$ → $0.7\sqrt{f_{ck}}$
- split tensile strength: $\frac{2P}{\pi d l}$ & $0.66 f_{cr}$ → $0.66\times(0.7\sqrt{f_{ck}}) = 0.462\sqrt{f_{ck}}$
- flexural tensile strength: $f_{cr} = 0.7\sqrt{f_{ck}}$ (modulus of rupture)
- cylinder comp. strength: $0.80 f_{ck}$
- cube comp. strength: f_{ck}

short term modulus of elasticity: $E_c = 5000\sqrt{f_{ck}}$

long term or effective modulus of elasticity using creep coefficient: $E_{ce} = \dfrac{E_c}{1+\theta}$

θ → 2.2 (7 day), 1.6 (28 day), 1.1 (1 year) • Logarithmic variation with time

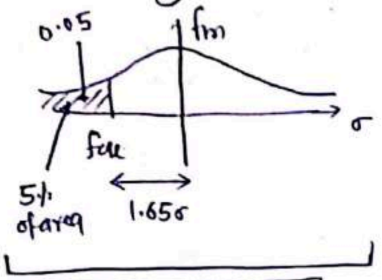
*

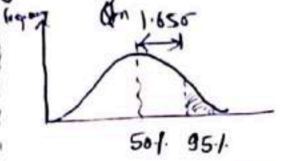
IS 456 :- max. size of coarse agg $\ngtr \frac{1}{4}$th of thickness of member

Target mean strength $\boxed{f_m = f_{cu} + 1.65\sigma}$

Characteristic strength (f_{cu}) :- below which not more than 5% test results are expected to fall.

Characteristic load :- has a 95% probability of not being exceeded during life.





$$\boxed{f_m = f_{cu} + 1.65\sigma}$$

f_{cu} → char. strength

$$\boxed{Q_{cu} = Q_{mean} + 1.65\bar{\sigma}}$$

Q_{cu} → characteristic load; Q_{mean} → $\boxed{\frac{\Sigma x_i}{n}}$; σ → $\sqrt{\frac{\Sigma(x_i-\bar{x})^2}{n-1}}$

M	σ N/mm²
≤15	3.5
20–25	4
30–60	5

Quantity of concrete (m3)	samples
1–5	1
6–15	2
16–30	3

1 sample = 3 specimen

- compliance requirement (Acceptance criteria)

grade (> M15)

(i) mean of group of 4 non-overlapping consecutive test result $\geq$ $f_{cu} + 0.825\sigma$ or $f_{cu} + 3$ } max.

(ii) individual test result $\geq f_{cu} - 3$

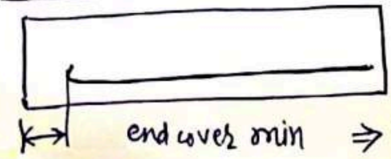
(iii) individual variation ≯ 15% of average

Probability of failure in LSM ⇒ $\boxed{0.0975}$

failure cases	load	×	✓	×
	strength	×	×	✓

$$p \Rightarrow \left(\frac{5}{100}\times\frac{5}{100}\right)+\left(\frac{95}{100}\times\frac{5}{100}\right)+\left(\frac{5}{100}\times\frac{95}{100}\right) = 0.0975$$

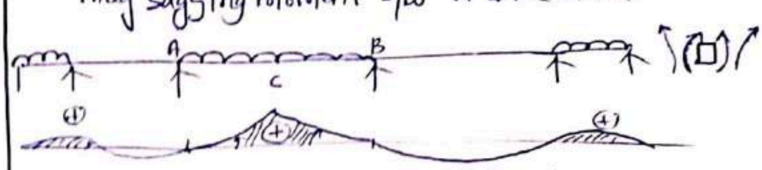
Concrete cover at end of reinforcement bars :-



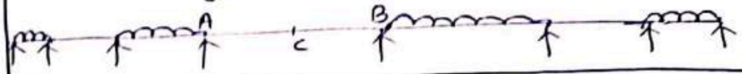
end cover min ⇒ max $\begin{cases} 2\phi \\ 25\text{mm} \end{cases}$

मतलब 25mm तो चाहिए ही चाहिए।

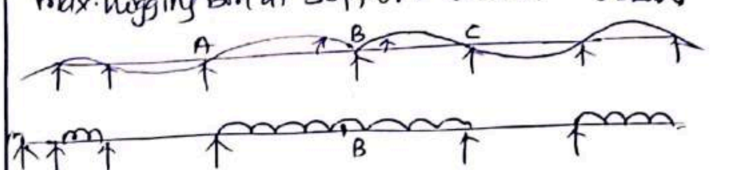
max sagging moment b/w A & B (मतलब C)



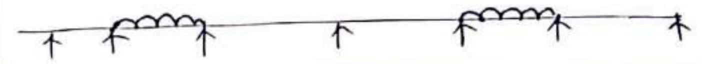
∴ max hogging moment b/w A & B (मतलब C)



max. hogging BM at support (like B)



∴ max. sagging BM at support B



Ultrasonic pulse velocity Test → NDT

- to assess quality of concrete in situ (used to obtain estimate of concrete strength of finished concrete element)
- to determine dynamic modulus of elasticity {Resonant frequency Test ⇒ vibration Technique using ultrasonic wave}
- to locate the presence of cracks in st.

Bond b/w steel & concrete → By Pull out Test acceptable limit for safety & serviceability requirement of str. before failure → limit state

limit state of collapse	limit state of serviceability
(to satisfy this, strength must be adequate to carry load)	• factors →
• factors →	(I) Deflection
(I) Flexural (Bending)	(II) cracking
(II) shear	(III) vibration
(III) Torsion	(IV) corrosion
(IV) compression	(V) fire resistance
	(VI) leakage
	(VII) overturning

MLSM :- Basis of analysis of str. linear elastic theory.

note:-

• characteristic of concrete in actual structure

(1) strength ⇒ $\boxed{0.67 f_{cu}}$ or $\frac{f_{cu}}{1.5}$

[note:- this 1.5 fos → to account loading other than compressive and different shape of RCC member]

(2) design strength of concrete ⇒ $\boxed{0.45 f_{cu}}$ → f_d

मतलब ⇒ $\underbrace{\frac{0.67 f_{cu}}{1.5}}$

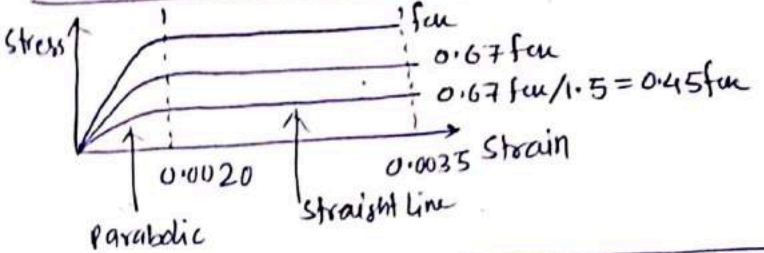
Partial fos of concrete (1.5)
steel (1.15)

(3) design strength of steel (f_d) ⇒ $\frac{f_y}{1.15} = \boxed{0.87 f_y}$

load combination	limit state of collapse			limit state of serviceability		
	DL	LL	WL	DL	LL	WL
DL + LL	1.5	1.5	–	1	1	–
DL + WL	1.5 or 0.9*	–	1.5	1	–	1
DL + LL + WL	1.2	1.2	1.2	1	0.8	0.8

[0.9* ⇒ when stability against overturning & stress reversal is critical where dead load [illegible] gravity dam]

Stress-strain curve for concrete :-



Durability criteria :-

Exposure category	description	M min RCC grade	M min PCC grade	min nominal cover (mm)	RCC min cement (kg/m3)	max W/C ratio
mild	• normal • protect against weather or aggressive condition (except if located in coastal area)	20	–	20	300	0.55
moderate	• Shelter from severe rain • continuously in water • non aggressive soil or ground water • Alternate freezing & Thawing	25	15	30	300	0.50
Severe	• Exposed to coastal environment (Immersed in sea water) • Exposed to severe rain • Alternate drying & wetting • severe condensation	30	20	45	320	0.45
very severe	• sea water spray • contact with or buried under aggressive soil or grd. water	35	20	50	340	0.45
extreme	• tidal zone • members in direct contact with liq./solid aggressive chemical	40	25	75	360	0.40

note (I) mild → if $\boxed{\phi_{main} \le 12mm}$ → then cover reduced by (5mm)

(II) Severe & very severe → if $\boxed{m \ge M35}$ then cover reduced by (5mm)

PH of water should not be less than 6 for making concrete.

msI 24 Permissible limits for solids : (OSCSI)

Organic	sulphates (as SO_3)	chloride (as cl)	suspended solids	Inorganic solids
200	400	500 – RCC 2000 – PCC	2000	3000

type of formwork	min. period Before striking formwork
① vertical formwork (Column wall beam)	16 – 24 hr
② soffit formwork to slab	3 days
③ ——— beam	7 days
④ Props to slab	
span ≤ 4.5 m	7 day
span > 4.5 m	14 day
⑤ Props to beam & arch	
span ≤ 6 m	14 day
span > 6 m	21 day

note:- above specification valid for OPC & where temp > 15°c and adequate curing is done
- for other cements & lower temp. above time can be modified.

Bactarial concrete →
(selfhealing concrete for crack repair

under water concreting methods →
(I) Tremiepipe method (II) Direct placement with pumps (III) Drop bottom bucket
(IV) Grouting

3 constituents adversly affect the concrete :-

① chlorides	· increases rate of corrosion to steel · due to this chloride content of admixture is tested separately
② sulphates (as SO_3)	· cause expansion & disruption of concrete
③ Alkali – Aggregate reaction ($Na_2O + K_2O$)	originating from cement, Producing an expensive rxn which can cause Cracks and disruption of concrete

Degree of workability	placing condition
very low	in highway constrction
low	mass concreting, light reinforced secti
medium	Heavy reinforced section & when pumping of concrete is required
High	Insitu piling
very high	Insitu piling using Tremie concrete.

note:-

	slump
vibrated concrete	12 – 25 mm
Concrete for road work	20 – 30
mass concrete	25 – 50
ordinary RCC work	50 – 100
Column r etaining wall	75 – 150

Assumption in limit state of collapse :- flexure :-

① plain section before bending remains plain after bending (strain diagram → linear)

② max. strain in concrete at outermost compressed fibre = $\boxed{0.0035 \text{ in bending.}}$

③ Relationship b/w comp. stress distribution in concrete & steel can be assumed rectangular, trapezoidal, parabolic or any shape, which results in prediction of strength matching with test result.

④ Tensile strength of concrete is ignored.

⑤ for design purpose partial safety of factor of steel = $\boxed{1.15}$ & stress in steel are derived from stress strain curve.

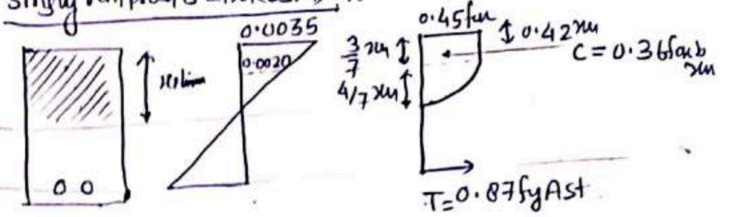
⑥ max. strain in tension reinforcement

$$\not< \boxed{0.0020 + \frac{f_y}{1.15E_s}}$$

note: ⑪ under reinforced section → deeper & undergo larger deflection than Balanced & over rein. section.

⑪ over reinforced section → stiffer than under reinforced section

Singly reinforced section (LSM) :-



c_1 = compressive force of rectangular portion

= area of stress dia × width of section

$= \left[0.45 f_{ck} \times \frac{3}{7} x_u\right] \times b = 0.193 f_{ck} b x_u$

$c_2 = \left[\frac{2}{3} \times 0.45 f_{ck} \times \frac{4}{7} x_u\right] \times b = 0.171 f_{ck} b x_u$ (Parabolic)

$\boxed{C = c_1 + c_2 = 0.36 f_{ck} b x_u}$ $\boxed{T = 0.87 f_y A_{st}}$

$y_1 = \frac{3}{14} x_u$ from top $\quad y_2 = \frac{3}{7} x_u + \frac{3}{8}\left(\frac{4}{7} x_u\right)$ from top

$\bar{y} = \dfrac{c_1 y_1 + c_2 y_2}{c_1 + c_2} = 0.42 x_u$

$\boxed{\therefore \text{ lever arm} = d - 0.42 x_u}$

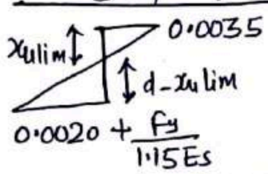
$$\boxed{M_u = 0.36 f_{ck} b x_u (d - 0.42 x_u) = 0.87 f_y A_{st} (d - 0.42 x_u)}$$

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

V. Imp :- if concrete partial fos = 1.5 is not considered then stress in compressed fibre = $0.67 f_{ck}$

hence $C = 1.5 \times (0.36 f_{ck} b x_u) = 0.54 f_{ck} b x_u$

limiting depth of NA : ($x_{u\,lim}$) → corresponds to Balanced section.



→ solve by similar Δ

v.v.v.v.Imp	$x_{u\,lim}$	$M_{u\,lim}$	$P_{t\,lim}$ ($\frac{A_{st\,lim} \times 100}{bd}$)	Strain in tension reinforcement ($0.002 + \frac{0.87 f_y}{E_s}$)
Fe250	0.53d	$0.148 f_{ck} b d^2$	$0.088 f_{ck}$	0.0031
Fe415	0.48d	$0.138 f_{ck} b d^2$	$0.048 f_{ck}$	0.0038
Fe500	0.46d	$0.133 f_{ck} b d^2$	$0.038 f_{ck}$	0.0042

note:- Tensile reinforcement

$$\left[\begin{array}{l} \dfrac{A_{st\,min}}{bd} = \dfrac{0.85}{f_y} \\ A_{st\,max} = 4\% \text{ of gross} \end{array}\right]$$

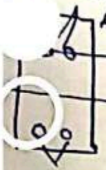
$A_{sc\,max}$ compressive reinforcement = 4% of gross

↓

to avoid compression failure of concrete before tension failure of steel.

(Lsm)

Doubly reinforced section :-



to know $x_u = ?$

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

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

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

$$\therefore M_u = 0.36 f_{ck} b x_u (d - 0.42 x_u) + (f_{sc} - 0.45 f_{ck}) A_{sc} \times (d - d')$$

Design :- ($M > M_{bal}$ or M_{lim})	LSM ($M_u > M_{ulim}$)	WSM ($M > M_{bal}$)
A_{st1}	$A_{st1} = \dfrac{M_{ulim}}{0.87 f_y (d - 0.42 x_{ulim})}$	$A_{st1} = \dfrac{M_{bal}}{\sigma_{st}\left(d - \frac{x_c}{3}\right)}$
A_{st2}	$A_{st2} = \dfrac{M_u - M_{ulim}}{0.87 f_y (d - d')}$	$A_{st2} = \dfrac{M - M_{bal}}{\sigma_{st} (d - d')}$
A_{sc}	$A_{sc} = \dfrac{M_u - M_{ulim}}{(f_{sc} - 0.45 f_{ck})(d - d')}$	$A_{sc} = \dfrac{M - M_{bal}}{(1.5m - 1)\, c' (d - d')}$

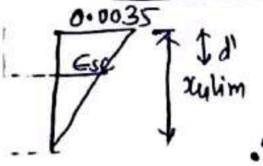
imp note:- if in question it is written →

neglect the reduction of comp. force due to removal of concrete for placing compressive steel (A_{sc})

मतलब eqn में से $0.45 f_{ck} A_{sc}$ वाला term हटा दें

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

note:- How to calculate $\varepsilon_{sc} = ?$



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

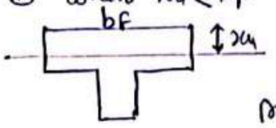
$$\therefore \text{yield strain} = \frac{0.87 f_y}{E_s}$$

$$\therefore f_{sc} = 0.87 \times f_y \rightarrow 250 \checkmark$$

note:- In doubly, strain in compressive reinforcement is higher than the strain in adjoining concrete.

T-beam (Lsm) :-

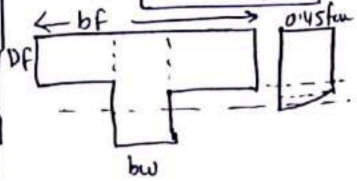
① when $x_u < D_f$



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

$$M_u = 0.36 f_{ck} b_f x_u (d - 0.42 x_u)$$
$$M_u = 0.87 f_y A_{st} (d - 0.42 x_u)$$

② when $\boxed{D_f < 3/7\, x_u}$



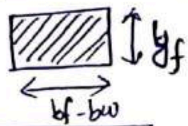
$$0.36 f_{ck} b_w x_u + 0.45 f_{ck} (b_f - b_w) D_f = 0.87 f_y A_{st}$$

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

$$M_u = 0.87 f_y A_{st1} (d - 0.42 f_{ck}) + 0.87 f_y A_{st2} \left(d - \frac{D_f}{2}\right)$$

③ when $\boxed{D_f > 3/7\, x_u}$

convert $(b_f - b_w) D_f \longrightarrow (b_f - b_w) y_f$

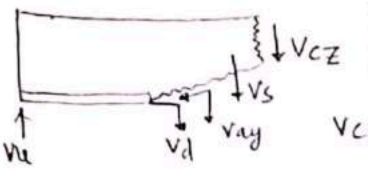


$$\boxed{y_f = 0.15 x_u + 0.65 D_f}$$ then solve.

note:- An Inverted T beam design as rect. beam over intermediate support of a continuous beam when BM is negative.

Shear span ⇒ span where shear force is constant

mechanism of shear resistance :-



$$\boxed{V_u = \underbrace{V_{cz}}_{①} + \underbrace{V_{ay} + V_d}_{②} + \underbrace{V_s}_{③}}$$

V_{cz} ⇒ shear resistance by uncracked portion of concrete

[V_{ay} → vertical component of aggregate interlock force
[V_d → dowel force in tension bar

V_s → shear resisted by stirrups

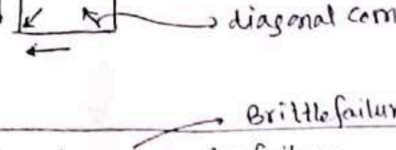
Steps :- ① prior to flexural crack → all shear resisted by V_{cz} (uncracked portion)

② At commencement of crack ⇒ (V_{ay} + V_d) developed

③ at dia. tension crack V_s develops and all 4 mechanism exist.

diagonal Tension failure :- occures due to diagonal tension under large SF and Less BM.

- normally occures at 45° from horizontal.
- for diagonal tension → bent up bars, inclined stirrups & vertical stirrups used
- diagonal tension max at N.A and decreases above & below N.A (N.A → neutral axis)



Brittle failure

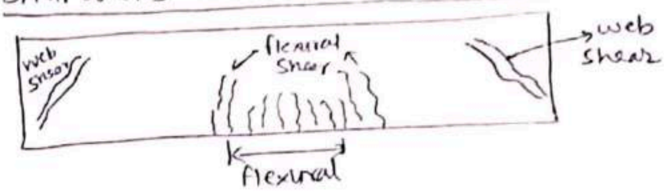
diagonal compression failure :- under large SF

- It is characterised by crushing of concrete, normally it occures in beam which are reinforced against heavy shear.

note: $\tau_{cmax} = 0.62\sqrt{f_{ck}}$
for Ex. M_{20} $\tau_{cmax} = 2.8\ N/mm^2$
note :- for ≥ M_{40} $\tau_{cmax} = 4\ N/mm^2$

Imp.
Imp. By this provision failure of beam by diagonal compression can be prevented.

Shear cracks in structural concrete member :-

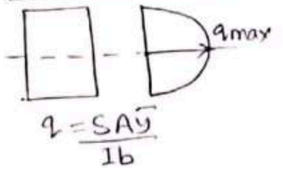


flexural shear crack → forms under large BM & less SF.

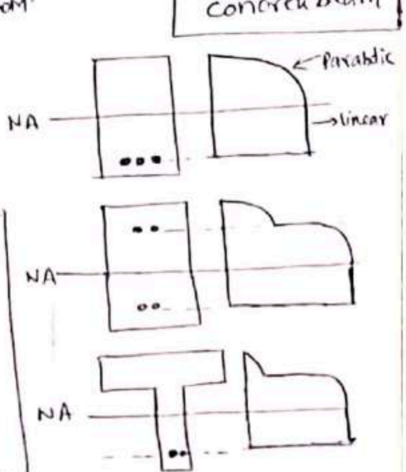
- occures normally at about 90° with horizontal.

shear stress distribution

Homogeneous beam → as in SOM | Reinforced concrete beam



$q = \frac{SA\bar{y}}{Ib}$

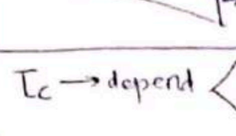


nominal or Avg. Shear stress

$\tau_v = \frac{V_u}{bd} < \frac{V}{b.jd}$ based on elastic Theory

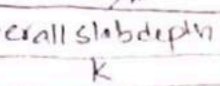
$b = b_w$ in case of flange section

Beam of varying depth :- (But Reinforcement should be inclined)



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

→ (−)ve sign if d↑ ⇒ BM↑) otherwise plus.

V.Imp.
if  → $\tau_v = V_u/bd$ {∵ β = 0}

τ_c → depend < Grade of concrete / P_t (% of tensile reinforcement only $\frac{A_{st}\times 100}{bd}$)

Imp. note :- if P_t ≥ 3 then design shear strength will be constant

(mm) overall slab depth	≥ 300	- - - -	≤150
k	1		1.30

Shear strength of member under axial compression.

WSM - $S = 1 + \frac{5P}{A_g \cdot f_{ck}} \ngtr 1.5$] factors

LSM - $S = 1 + \frac{3P_u}{A_g \cdot f_{ck}} \ngtr 1.5$

min. shear reinforcement :

$$\boxed{\frac{A_{sv}}{b s_v} \geq \frac{0.40}{0.87 f_y}}$$ v.imp

$f_y \not> 415$

even if Fe500 bar used but while calculation min shear reinforcement in this formula put 415 only.

why needed →

① sudden failure of beam Prevented if
- concrete cover burst
- Bond to tension steel is lost.

② Brittle shear failure arrested which would have occured without shear reinforcement therefore improves ductility.

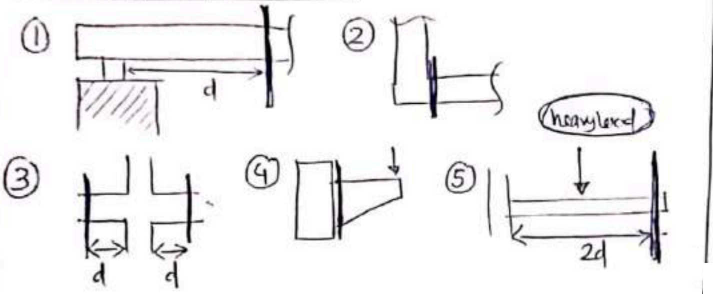
③ Tension failure prevented due to shrinkage, thermal stresses internal cracking in beam.

④ to hold to reinforcement in place when concrete is poured.

⑤ to improve dowel action of longitudinal tension bars.

⑥ Section becomes effective with tie effect of comp. steel

Critical section for shear :-



Shear reinforcement spacing max. min { 0.75d → vertical stirup; d → inclined; 300mm }

note :- stirups will not be able to resist shear unless an inclined crack cross them.

Shear reinforcement can be provided in the form of

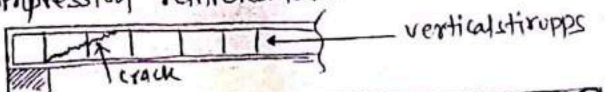
① vertical stirupps :- *

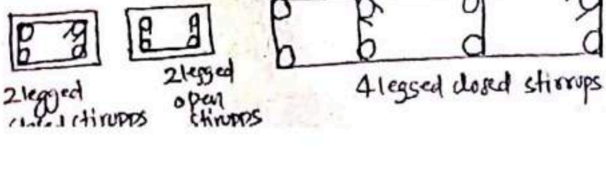
$$\boxed{V_{us} = [(0.87 f_y)(A_{sv})]\left(\frac{d}{s_v}\right)}$$

→ most effective form because →
- Cracks intercepts • improve dowel action of long. reinforcement

imp → Best for load reversal cases.

- Closed stirrups are better because it resist torsion & helps in confining the compression reinforcement in doubly-reinforced section and reduces the chances of buckling of compression reinforcement.

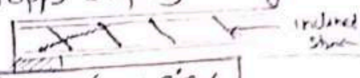




② inclined stirupps : *

$$\boxed{V_{us} = [(0.87 f_y)(A_{sv})]\left[\frac{d}{s_v}(\alpha + \sin\alpha)\right]}$$

- Ineffective in load reversal • most effective in reducing width of inclined cracks.
- can be provided in member having axial tension resulting in full depth of inclined cracks
- $\boxed{\alpha \not< 45^\circ}$ because if $\alpha < 45^\circ$ ⇒ there is a Possibility of inclined stirupps sliping along the longitudinal bars.

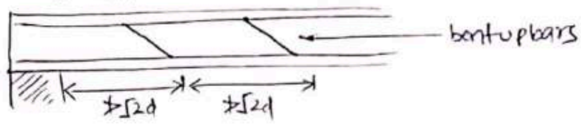


③ Bentup bars :- $\boxed{V_{us} = (0.87 f_y)(A_{sv}) \sin\alpha}$ → जितने बार (bars) Bent किये उनका area

- These are main tensile bars bend at appropriate locations.
- don't enhance dowel action.

Imp
- $\boxed{\text{It is always provided with vertical stirupps}}$

imp
- when Bentup bars are provided there shear resistance contribution $\not>$ 50% of total shear to be resisted thr. shear reinforcement.



Example :- $V_{concrete} = 63$ $V_{stirups} = 46$ $V_{bentup} = 102$

here Bentup % $= \frac{102}{102+46} \times 100 = 68\%$ ($\not> 50\%$) As per IS

∴ $V_{u\,permitted} = \underset{c}{63} + \underset{s}{46} + \underset{Bentup}{46}$

v.v.imp

$\tau_v > \tau_c$	shear reinforcement for $\boxed{(\tau_v - \tau_c)\, bd}$
$\tau_v < \tau_c$	→ $\tau_v < 0.5\tau_c$ → * $\boxed{\text{no shear Reinforcement}}$ → $0.5\tau_c \leq \tau_v \leq \tau_c$ → $\boxed{\text{Provide min shear reinforcement}}$ *
$\tau_v > \tau_{c\,max}$ ↓ $0.62\sqrt{f_{cu}}$	• concrete failure in diagonal compression. (Brittle failure) • in such case either adopt concrete higher grade or dimension of section is need to be increased.

Bond stress $\tau_{bd} = \dfrac{V_u}{(n\pi\phi)(LA)}$

true

most economical method to increase τ_{bd}
⇒ more no. of thinner bars (less dia bars)

others method to improved bond strength →
(I) increase grade of concrete (II) Use deformed bars inplace of plain bars
(III) increase cover provided around the bar
(IV) provide bends, hooks, mechanical anchorage
(V) any method which increases confinement — around bar would increase the bondstrength of bars.

$L_d = \dfrac{\phi\,\sigma_{st}}{4\tau_{bd}}$

τ_{bd} → bond stress in Plainbar in tension

τ_{bd}	M15	...	M40
wsm	0.6		1.2
Lsm	—	1.2	1.9

$\{\tau_{bd}(\pi\phi)L_d = (0.87f_y)\frac{\pi}{4}\phi^2\}$

note:-
- Deformed bar / HYSD τ_{bd} ⇒ +60% ↑
- Compression τ_{bd} = +25% ↑

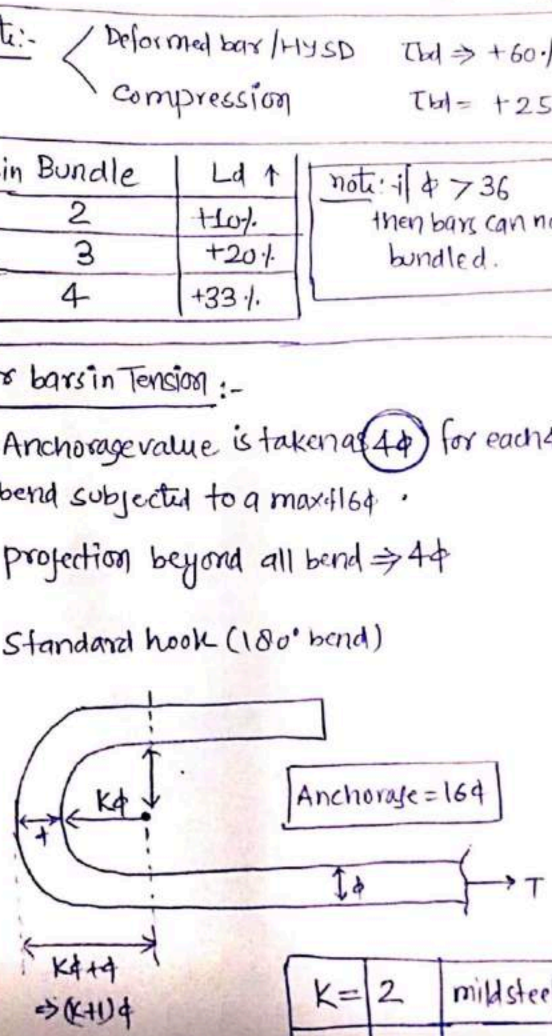
bars in Bundle	L_d ↑
2	+10%
3	+20%
4	+33%

note: if $\phi > 36$ then bars can not be bundled.

for bars in Tension :-

- Anchorage value is taken as 4ϕ for each 45 degree bend subjected to a max of 16ϕ.
- projection beyond all bend ⇒ 4ϕ

Standard hook (180° bend)



K=	2	mildsteel
K=	4	HYSD

min. turning radius (internal) for hook
$r \geq 0.456\phi\left(\dfrac{f_y}{f_{cu}}\right)\left[1 + \dfrac{2\phi}{a}\right]$

mild steel	$r = 2\phi$	(K=2)
Coldwork Deform bar (HYSD)	$r = 4\phi$	(K=4)

true

note:- no lap splice $\phi \geq 36$ mm
Use welded splice for larger dia.

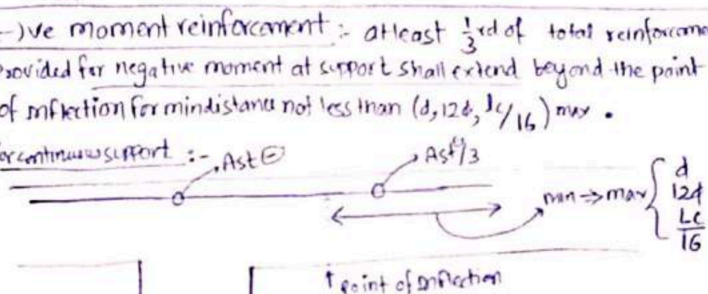
min Lap Length
- in compression: $\max[L_d, 24\phi]$
- in tension
  - Flexural Tension: $\max(L_d, 30\phi)$
  - direct tension: $\max(2L_d, 30\phi)$

true

note: If the anchorage length beyond zero moment ⇒ L_0
$\boxed{L_d \leq \frac{M}{V} + L_0}$ → not to check (−)ve reinforcement
- for simple support with compression confinement
$\boxed{L_d \leq 1.3\frac{M}{V} + L_0}$

Positive (+)ve moment tension reinforcement :-
atleast $\frac{1}{3}$rd of tension reinforcement in SSB & $\frac{1}{4}$th of tensile reinforcement in continuous member shall extend along the same phase of member into support length equal to $L_d/3$. [min distance ⇒ max $(d, 12\phi)$ from centre of support]

$\boxed{SSB @ Ast/3 * \frac{L_d}{3}}$ + $\boxed{continuous @ \frac{Ast}{4} * \frac{L_d}{3}}$



(−)ve moment reinforcement :- atleast $\frac{1}{3}$rd of total reinforcement provided for negative moment at support shall extend beyond the point of inflection for min distance not less than $(d, 12\phi, l_c/16)$ max.

for continuous support :-

equivalent shear $\boxed{V_{ue} = V_u + 1.6\dfrac{T_u}{b}}$

$$\boxed{M_{ue} = \frac{T_u}{1.7}\left(1 + \frac{D}{b}\right)}$$

$\boxed{M_{ue1} = M_{ue} + M_u}$ $\boxed{M_{ue2} = M_{ue} - M_u}$

(equivalentmoment)

A_{st1} A_{st2} + A_{sc}

$$\boxed{A_{st3} = \frac{M_{ue2}}{(0.87f_y)(d-d')}}$$

Provide on compression side

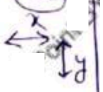
note:- if $\tau_{ve} = \tau_c$ then noadditional reinforcement for torsion

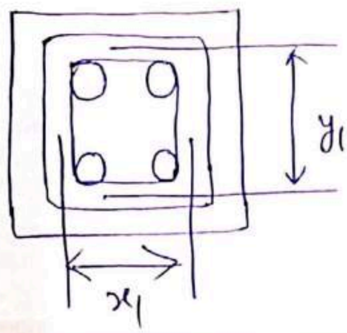
Spacing of shear reinforcement :- (in torsioncase)

$$\boxed{\frac{A_{sv}}{b s_v} \geq \frac{(\tau_{ve} - \tau_c)}{0.87 f_y}}$$

Spacing (max) ⇒ min $\begin{cases} x_1 \\ \dfrac{x_1 + y_1}{4} \\ 300\,mm \end{cases}$

(4c)





sideface reinforcement :- $\boxed{0.1\%\text{ of webarea}}$

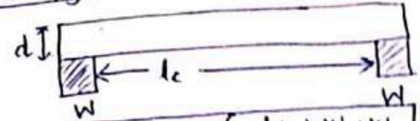
(provideequally onbothside)

when $\begin{cases} \text{(I) } D_{\text{(webdepth)}} > 750\text{ mm in simple design} \\ \text{(II) } D > 450 \text{ in Torsion design} \end{cases}$

• spacing max ⟶ min $\begin{cases} \text{web thickness (beam width)} \\ 300\text{ mm} \end{cases}$

effective span :-

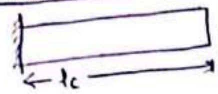
① simply supported beam/slab



d → effective depth

$$l_{eff} \Rightarrow \min \begin{cases} l_c + \frac{W}{2} + \frac{W}{2} \\ l_c + d \end{cases}$$

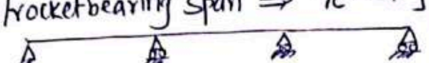
② cantilever beam :-



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

③ frame :- in analysis of continuous frame, c/c dist

[roller/rocket bearing span ⇒ c/c dist]



$\boxed{l_{eff} = l_c}$

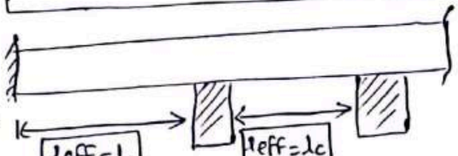
④ continuous beam or slab :-

case-1 :- $\boxed{\text{width of support} < \frac{\text{clear span } (l_c)}{12}}$

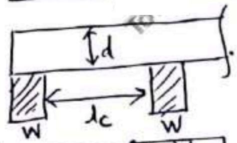
└ same as SSB $l_{eff} \Rightarrow \min \begin{cases} l_c + d \\ l_c + W/2 + W/2 \end{cases}$

case-2 :- $\boxed{\text{if } W > \frac{l_c}{12}}$

for one end fixed other continuous 'or' both end continuous (intermediate span)	one end simply supported, other continuous.



$\boxed{l_{eff} = l_c}$ $\boxed{l_{eff} = l_c}$



$$l_{eff} = \min \begin{cases} l_c + \frac{d}{2} \\ l_c + \frac{W}{2} \end{cases}$$

Deflection check :-

① final deflection → due to All load, temp., creep, shrinkage

$$\boxed{\not> \frac{span}{250}}$$

⑪ deflection (including effect of temp, creep, shrinkage) after errection of partition, finish

$$\not> \boxed{\min \left\{ \frac{span}{350}, 20\,mm \right\}}$$

Beam/slab :- Vertical deflection limit

	$\frac{\text{span}}{\text{eff. depth}(d)}$
cantilever	7
SSB	20
continuous (1 end)	23
continuous (both end)	26

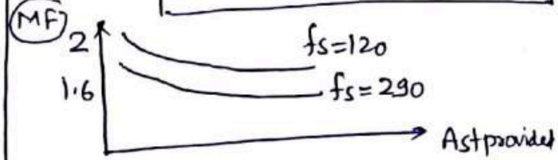
} for span upto 10 meter

note :- ① for span like 15 (>10) SSB ⇒ $20 \times \frac{10}{15}$ → $\frac{10}{span}$

⑪ Separate deflection calculation for cantilever beam (if span > 10m)

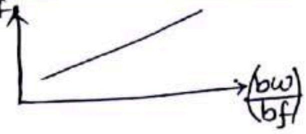
$MF_1 \uparrow$ → Based on area of tensile reinforcement ↑

$$\boxed{MF_1 \uparrow} \Rightarrow \boxed{f_s \downarrow \Rightarrow 0.58 f_y \frac{Ast\ req.}{Ast\ provided \uparrow}}$$



$MF_2 \uparrow$ → Based on area of compression reinforcement ↑

MF_3 → for flange beam

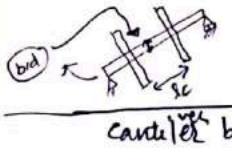
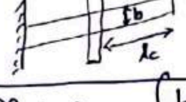


for 2 way slab :- deflection check :- $\left(\frac{\text{Short span}}{\text{overall depth } D}\right)$

upto 3.5m & LL ≤ 3 KN/m2 then only shortspan

	mild steel	HYSD 415/500
SSB	35	35×0.80 = 28
continuous	40	40×0.8 = 32

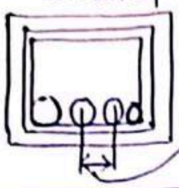
Lateral stability Beam → slenderness limit

SSB or continuous beam 	clear span (l_c) $\not>$ min $\begin{cases} 60b \\ \frac{250b^2}{d} \end{cases}$
cantilever beam 	clear span (l_c) $\not>$ min $\begin{cases} 25b \\ \frac{100b^2}{d} \end{cases}$

d → eff. depth { b → breadth of compression face midway b/w the lateral restraint }

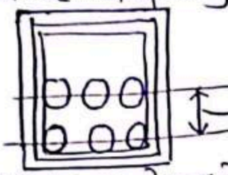
min distance b/w indivisual bar
so that concrete can place easily

(I) min horizontal spacing



max { max agg size + 5, max bar dia }

(II) min vertical spacing



max { 2/3 x max agg. size, max bar dia, 15 mm }

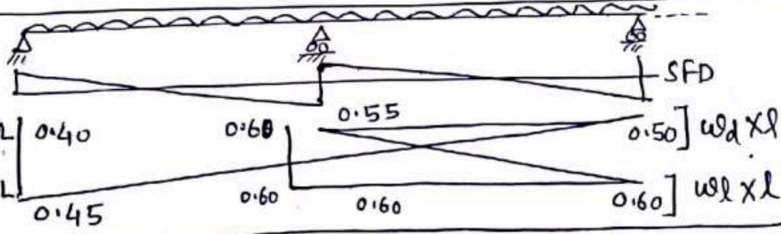
मतलब 15 mm तो कम से कम चाहिये

continuous beam (each supporting a monolith reinforced slab)

- for span moment → design as T beam
- for support moment → design as rect. beam

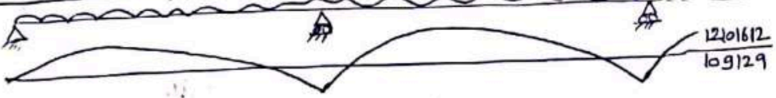
moment & shear cofficient uses →

- if min span = 3 & udl loaded & span will not differ by 15% of longest span.
- if span of 2 side of support or loading are different then support moment will be calculated from both side & avg. of both will be taken.



$w_d \times l$

$w_l \times l$



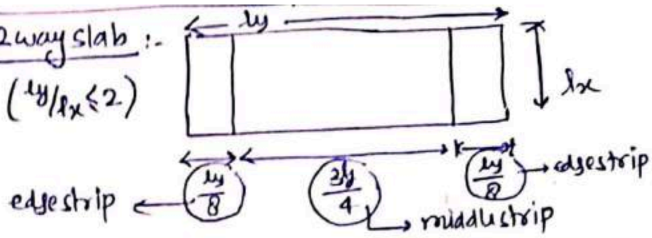
DL	$+\frac{w_d l^2}{12}$	$-\frac{w_d l^2}{10}$	$+\frac{w_d l^2}{16}$	$-\frac{w_d l^2}{12}$
LL	$+\frac{w_l l^2}{10}$	$-\frac{w_l l^2}{9}$	$+\frac{w_l l^2}{12}$	$-\frac{w_l l^2}{9}$

for interior span
- BM at support = (−)ve moment = $\frac{w_d l^2}{12} + \frac{w_l l^2}{9}$
- BM at mid span = (+)ve moment = $\frac{w_d l^2}{16} + \frac{w_l l^2}{12}$
- SF = $\underbrace{0.55 w_d l + 0.60 w_l l \;\&\; 0.50 w_d l + 0.60 w_l l}_{\text{Avg.}}$

for end span
- BM at support (−)ve ⇒ $\frac{w_d l^2}{10} + \frac{w_l l^2}{9}$
- BM at mid span (+)ve ⇒ $\frac{w_d l^2}{12} + \frac{w_l l^2}{10}$
- SF = $0.40 w_d l + 0.45 w_l l$ & $0.60 w_d l + 0.60 w_l l$ Avg.

REDMI NOTE 5 PRO

2 way slab :- ($l_y/l_x \le 2$)



min tension reinforcement ⇒ $\boxed{\frac{A_{st}}{bd} = \frac{0.85}{f_y}}$

- (to avoid sudden failure of steel due to transfer of full tension to steel on cracking of concrete when beam is loaded
- to avoid rupture of steel in a flexural failure occurs

max tension or max. compression reinforcement = 4% of gross (0.04 bD)

in slab
- min fe250 ⇒ 0.15% of gross c/s area
- min fe415/500 ⇒ 0.12%

in slab max
- main bars spacing ⇒ min (3d, 300 mm)
- secondary bars spacing ⇒ min (5d, 450 mm)

max size of bar in beam/slab ⇒ D/8 mm

Distribution bars (Temp. & shrinkage steel) :-
- hold the slab on either side (way)
- resist load, temp stress, shrinkage stress
- It occupies larger area.

if slab main bar ≤ 10 mm
then clear cover ⇒ 20 − 5 ⇒ 15 mm
reduced by 5mm
(in case of mild)

[one way slab $l_y/l_x > 2$, supports only on 2 edges]

$r = \frac{l_y}{l_x}$ = Aspect ratio

2way slab ($\frac{l_y}{l_x} < 2$) → never be cantilever

- Simply supported 2way slab
  - corner's not held down (use → Grasshoff rankine method)
  - corner's held down
    - Pigeaud's method
    - marcus method
    - IS method → $m_x = \alpha_x w . l_x^2$, $m_y = \alpha_y w l_x^2$
- slab edges fixed or continuous (IS method) use

Circular slab (fix at ends) — when loaded → Saucer/Paraboloid shape

$w \rightarrow \frac{kN}{m}$

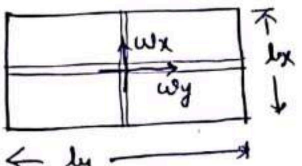
- Radial moment: $\frac{w}{16}(R^2 - 3r^2)$
  - max(+)ve radial moment: $\frac{wR^2}{16}$ at centre
  - max(-)ve radial moment: $\frac{2wR^2}{16}$ at edges
- Circumferential moment: $\frac{w}{16}(R^2 - r^2)$

note :- max. Radial moment at centre $= \frac{wR^2}{16} + \frac{2wR^2}{16}$
$= \frac{3wR^2}{16}$

measurement	tolerance
linear (like beam, slab thickness)	0.01 m
area	0.01 m2
volume	0.01 m3

Deep beam
- $\frac{l}{D} < 2$ (SSB)
- $\frac{l}{D} < 2.5$ (continuous beam)

Grasshoff Rankine method →
(corners not held down ⇒ corners can be lifted)



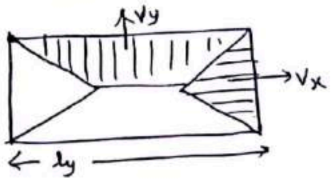
$w = w_x + w_y$

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

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

$\therefore \boxed{w_x = w\left(\frac{r^4}{1+r^4}\right) \quad w_y = \frac{w}{1+r^4} \quad w_x > w_y}$

$\therefore \boxed{M_x = \frac{w_x l_x^2}{8} \ \& \ M_y = \frac{w_y l_y^2}{8}}$ → moment in x & y direction



$\boxed{V_x = \frac{w l_x}{3}}$ at shorter edge

$\boxed{V_y = \left(\frac{r}{r+2}\right) w l_x}$ at longer edge

marcus method (corner's held down) *
↳ use reduction factor ⇒ $\boxed{\frac{5}{6}\frac{r^2}{1+r^4}}$

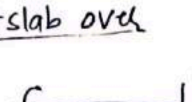
$\therefore M_x = \underset{\downarrow \ \frac{wr^4}{1+r^4}}{w_x}\frac{l_x^2}{8}\left[1 - \frac{5}{6}\frac{r^2}{1+r^4}\right]$

$M_y = \underset{\downarrow \ \frac{w}{1+r^4}}{w_y}\frac{l_y^2}{8}\left[1 - \frac{5}{6}\frac{r^2}{1+r^4}\right]$

Flatslab :- • Slab built monolithically with the supporting column without beam (सीधे पुटदिरेक्ट्ली)
- min flat slab thickness ⇒ 125 mm
-
  - L/32 for end panels with drop
  - L/36 for end panels without drop.

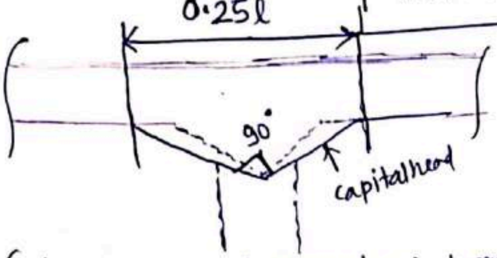
• Drop :- thickened part of flatslab over its supporting column.

• drops mainly resist shear.



• drop panel increases shearstrength of slab, increases (-)ve moment capacity of slab and stiffen the slab, hence reduce Deflection.

capital or column head : enlarge head of supporting column of flat slab



(Slab without drop panel and column with column head)

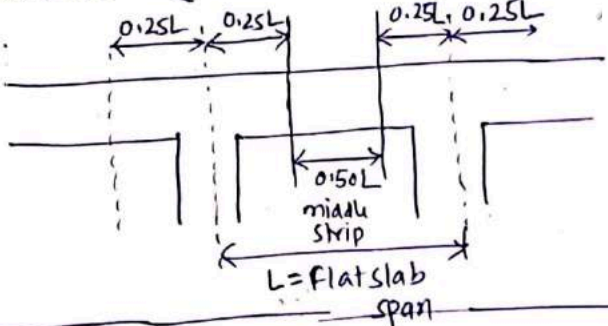
- columnhead fn :- Used to reduce punching shear in slab.
- dia. of column head ⇒ 0.25l

lengthof largerspan

critical section for shear in flat slab :-

$\frac{d}{2}$ from perifery of column / capital / drop panel, perpendicular to the plane of slab.

flat slab → (column & middle strip)



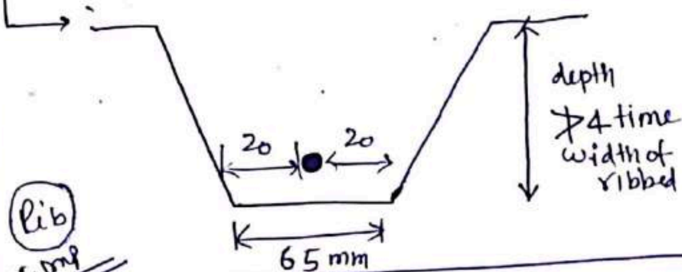
(1) column strip ⇒ 0.25 L on both side of column

∴ total column strip = 0.25L + 0.25L
= 0.50L

(2) middle strip ⇒ 0.50L

Ribbed slab or waffle slab :-

- consist of series of parallel reinforced concrete girder.
- The slab is the flange of beam also called concrete topping and the extended part of web is rib.
- rib are tapered in c/s in its lower part
- uses → A plain ceiling
→ Thermal insulation
→ Acoustic Insulation.



Imp

(I) min thickness of Rib = 65 mm

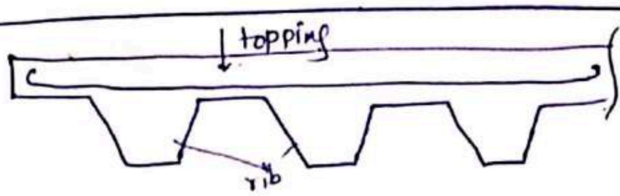
(II) max dia in rib slab = 22 mm

(III) min. nominal cover to bars ⇒ 20mm.

(IV) Insitu ribs max. spacing = 1.5m
(spacing ⊁ 1.5m)

(V) depth (excluding any topping) ⊁ 4 width of rib

(VI) Thickness of topping of ribbed slab ⇒ 5cm to 10 cm.
(min 54mm) for single reinforcement
& min 80-100mm for double reinforcement



walls

1. min Thickness = 100 mm

note:- if thickness > 200 mm then vertical & horizontal reinforcement given in 2 grids, one near each face of wall.

2. $\dfrac{H_{eff}}{t} \not> 30$

3. design $e_{min} = 0.05t$

4. vertical reinforcement

max. spacing = Min(3t, 450mm)

(V.R)

- $\geq$ fe415 + welded wire fabric, $\phi \not> 16$mm → (0.12 % gross)
- fe 250 & other → (0.15 % of gross)

(Trick → Same as slab criteria)

5. horizontal reinforcement

max spacing = min(3t, 450mm)

H.R

- $\geq$ fe415 + welded wire fabric, $\phi \not> 16$mm → (0.20 % of gross)
- fe250, other → (0.25 % of gross)

$\sum^{12\ 15}_{20\ 25}$ V H

Limit state of collapse : compression (column)

Assumption :- (i) plain section remains plain

(ii) stress dia of concrete → any shape
(result should match with test results)

(iii) concrete tensile strength → ignore

(iv) stress in bar taken from the stress-strain diagram of particular steel.

imp. (v) $\boxed{\text{max. compressive strain in concrete in axial compression} = 0.0020}$

(vi) the max. compressive strain at highly-compressed extreme fibre subjected to

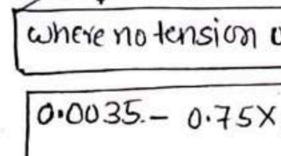
axial compression & bending

- imp — where no tension on section: $0.0035 - 0.75 \times$ strain at least compressed fibre
- imp — where part of section in tension: 0.0035 least layer

column		
Pedestal	$\lambda < 3$	$\lambda = l_{eff}/l_{min}$ slenderness ratio
Short column	$3 < \lambda < 12$	
Long column	$\lambda > 12$	

$$e_{min} = \frac{l_{unsupported}}{500} + \frac{B \text{ or } D}{30}, \text{ min } 20\text{ mm}$$

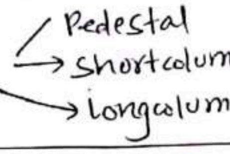
note:- when Biaxial bending is considered, It is sufficient to ensure that eccentricity exceeds the min. about one axis at a time

Ex. 

major e (xx) $\Rightarrow \frac{3000}{500} + \frac{600}{30}$

minor e (yy) $\Rightarrow \frac{3000}{500} + \frac{450}{30}$

$l_{un} = 3000$

Column : 
→ max. metal area = 20% of gross

note:- imp. Pedestal for plain concrete column
min longitudinal bar = (0.15%) of gross

---

longitudinal reinforcement :-> column

min ⇒ 0.8% of gross

(i) ensure flexural resistance under unseen resistance in loading.

(ii) to prevent yielding of bar due to creep and shrinkage which results in a transfer of load from concrete to steel.

max ⇒ 6% of gross

but 4% take for better concrete placing, at lapped splice location

- rectangular column min longitudinal bars = 4
- Circular = 6
- max. spacing along periphery of column = 300 mm
- column clear cover minimum = 40 mm

[Special :- if $D \le 200$ mm, $\phi_{main} \le 12$ mm then clear cover = 25 mm]

Transverse reinforcement in column :-

(i) Prevent premature buckling of individual bar.

(ii) to confine the concrete in core thus improving strength & ductility.

(iii) to hold the bars in position during construction

(iv) to provide resistance against shear & torsion

lateral ties

min diameter

$$\max \begin{cases} \frac{\phi_{max}}{4} \\ 6\text{mm} \end{cases}$$

max. pitch

$$\min \begin{cases} \text{least lateral dim.} \\ 16\phi_{min} \\ 300\text{ mm} \end{cases}$$

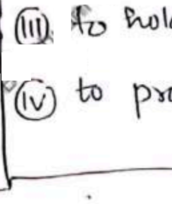
Helical reinforcement → increase strength by 5%

min diameter $< \left[\frac{\phi_{max}}{4}, 6\text{mm}\right]$ max

$$\frac{V_h}{V_c} \nless \frac{0.36 f_{ck}}{f_y}\left[\frac{A_g}{A_c} - 1\right]$$

Pitch

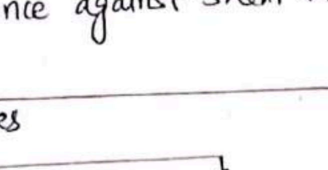
- min: $\max \begin{cases} 3\phi_h \\ 25\text{mm} \end{cases}$
- max: $\min \begin{cases} d_c/6 \\ 75\text{mm} \end{cases}$



$d_c = D - 2 \times$ clear cover to helical bar

d_h = c/c dist b/w helix $= d_c - \phi_h$

$V_c = A_c \times 1 = \left(\frac{\pi}{4} d_c^2\right) \times 1$



$\sqrt{(\pi d_h)^2 + (p)^2} \approx \pi d_h$

V_h = (c/s area of spiral) × (no. of turn) × 1 turn length

$$= \left(\frac{\pi}{4}\phi_h^2\right) \times \left(\frac{1000}{p}\right) \times \pi d_h$$

Slenderness limit column

unsupported length b/w end restraint $\not>$ 60 least lateral (b)	If in any given plane, one end of column is unrestraint: unsupported length $\not> \frac{100b^2}{D}$

Short axially loaded column

$$P_u = 0.40 f_{ck} A_c + 0.67 f_y A_{sc} \quad \{e_{min} \leq 0.05D\}$$

Concentric load column → $(e=0)$ (11% कम है)

$$P_u = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$$

valid (i) $e > 0.05D$ (ii) axial comp. + Biaxial bending

Column :- axial compression + uniaxial bending :-

→ Use Interaction curve (SP-16)

$\frac{P_u}{f_{ck} bD}$ → $\frac{M_u}{f_{ck} bD^2}$

for a particular grade of f_{ck}: $\frac{d'}{D} = \frac{40}{100}$ ⇒ solve

$\boxed{p/f_{ck} = 0.09}$ $\quad e = \frac{M_u}{P_u}$

$\boxed{p = \frac{A_{sc}}{bD} \times 100}$

$\boxed{p_{min} = 0.8\%}$ ⇒ 0.008

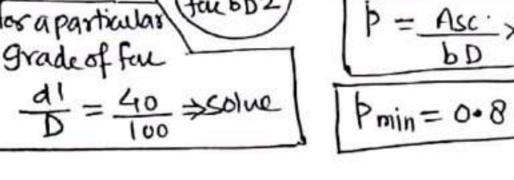
Column :- axial compression + biaxial bending :-

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$$

M_{ux}, M_{uy} → due to design load

M_{ux1}, M_{uy1} → max. uniaxial moment capacity for axial load of P_u

α_n → value based on $\left(\frac{P_u}{P_{uz}}\right)$ $\quad \left[P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}\right]$

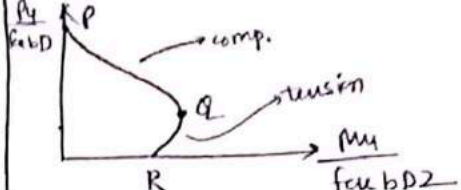


long column :-

$$M_{ax} = \frac{P_b}{2000}\left(\frac{l_{ex}}{b}\right)^2$$

$$M_{ay} = \frac{P_D}{2000}\left(\frac{l_{ey}}{D}\right)^2$$

$\{M_{ux} = M_{ux1} + M_{ax}\}$ $\{M_{uy} = M_{uy1} + M_{ay}\}$



(i) Balanced failure :- when $\boxed{e = e_b}$ → balanced eccentricity (e = loading eccentricity)

• both crushing of concrete (in highly compressed edge) & yielding of steel (in outermost tension steel) occure simultaneously.

(ii) compression failure : when $\boxed{e < e_b}$

• yielding of long steel in tension does not take place & failure occures at the ultimate limit state by crushing of concrete at the highly compressed edge.

(iii) Tension failure :- when $\boxed{e > e_b}$ failure will be initiated by yielding of tension steel.

imp
note :- The effective lateral support to Reinforced compression member given by transverse reinforcement either in circular ring {capable of taking up circumferential torsion} or by polygon link (lateral ties) with $\boxed{\text{internal angle} \not> 135^\circ}$

WSM :- $\boxed{P = \sigma_{sc} A_{sc} + \sigma_{cc} A_c}$ → short column.

σ_{sc}: Fe250 → 130 N/mm²; Fe415/500 → 190; $\sigma_{cc} = f_{ck}/4$

M	15	20	25	30
σ_{cc}	4	5	6	8

$\boxed{\text{long column } P = C_r [\sigma_{sc} A_{sc} + \sigma_{cc} A_c]}$ → C_r reduction factor

$\boxed{C_r = 1.25 - \frac{l_{eff}}{48b}}$ → b = core or least lateral dimension

$\boxed{C_r = 1.25 - \frac{l_{eff}}{160 r_{min}}}$ → $\boxed{r_{min} = \sqrt{\frac{I_{min}}{A}}}$

Thickness of reinforced concrete footing

- on piles: ≮ 300mm
- on soil: ≮ 150mm

• minimum clear cover footing = 50mm

① area of footing = $\dfrac{\overset{(load)}{P} + \overbrace{0.10P}^{selfwt}}{q_o}$

② net soil pressure = $\dfrac{P}{A_{provided}}$ → WSM = ω; LSM → ω_u = 1.5ω

check for BM @ critical section :-

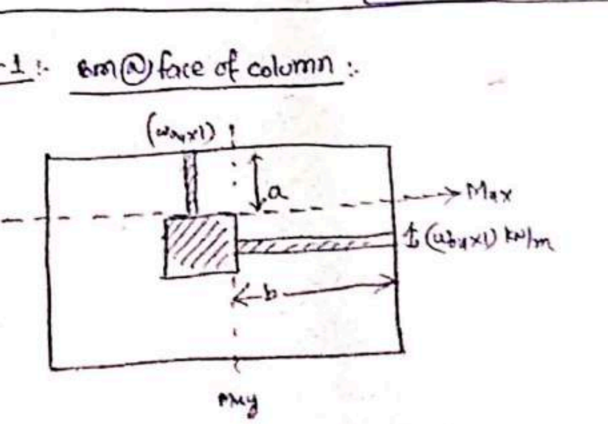
① at face of the column — or pedestal or wall → for footing supporting monolithic constructed column, wall

{ here wall → RCC wall }

② Half way b/w the centre line & edge of wall, for **footing under masonry wall**

③ Half way b/w the face of column or pedestal and edge of gusseted base, **for footing under gusseted Base.**

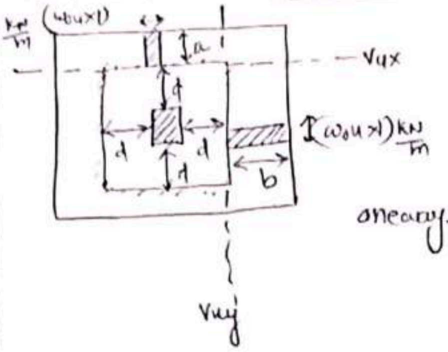
Point-1 :- BM @ face of column :-



$M_{ux} = [\omega_{u} \times 1] \times \dfrac{a^2}{2}$ $M_{uy} = [\omega_{u} \times 1] \times \dfrac{b^2}{2}$

check for 1 way shear :- → effective depth of footing

(onesill) Critical section → 'd' from face of column or wall



$V_{ux} = [\omega_u \times 1] \times a$ kN

$V_{uy} = [\omega_u \times 1] \times b$ kN

oneway shear stress

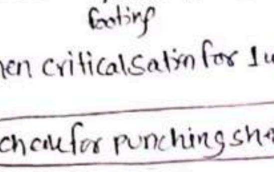
$\tau_v = \dfrac{V_{ux} \text{ or } V_{uy}}{(1000)\, d}$ (b)

$\tau_v < k\tau_c$; $D > 300$, $k = 1$

note :- if slab is rested on pile footing then critical section for 1 way shear → $\frac{d}{2}$ dist.

check for punching shear :- $\frac{d}{2}$ away from face of column or pedestal

Critical section :-



$\tau = \dfrac{P_u - \omega_u (a+d)(b+d)}{\underbrace{2(a+d+b+d)}_{Perimeter} \times d}$

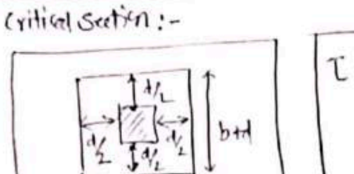
$\tau < k_\beta \times 0.16\sqrt{f_{ck}}$ → WSM

$\tau < k_\beta \times 0.25\sqrt{f_{ck}}$ → LSM

$k_\beta = 0.5 + \dfrac{\text{short column}}{\text{long } ,,}$

$k_\beta ≯ 1$

note :-

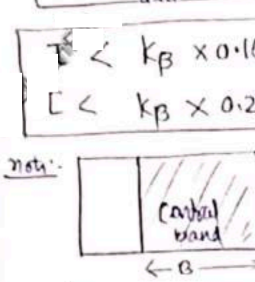


$n_c = \dfrac{2 n_T}{1+\beta}$ → total of bars

bars in central band

$\beta = \dfrac{\text{long side of footing}}{\text{short side of footing}}$

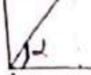
Plain concrete Pedestal footing :-



$\tan\alpha ≮ 0.9\sqrt{\dfrac{100 q_o}{f_{ck}} + 1}$

Trapezoidal footing for 2 axially loaded column :-
① when length of footing is restricted ② Projection of footing beyond heavier column is restricted

note :- Depth of footing for an Isolated column is govern generally by shear criteria (ie oneway + 2 way shear)



side 6m along plane of BM.
for no tension $\frac{P}{A} - \frac{My}{I} = 0$

$\dfrac{450}{4\times 6} - \dfrac{M\times 3}{\frac{4\times 6^3}{12}} = 0$

WSM → Permissible bearing stress → $0.25 f_{ck}$

LSM → $0.45 f_{ck}$

Pretensioning → bond action → min $\boxed{M40}$
Post → bearing " → min $\boxed{M30}$
min strength of concrete at the transfer of prestress = $\boxed{0.50 f_{ck}}$

Pretensioning stages :- Ex. Hoyer line method (Railsleeper)

Anchoring (tendons against end abutment) → Placing of Jacks → Apply tension to tendon → concrete casting → cut the tendon

Post tensioning stages :-

concrete casting → tendon placed → anchorage block and jack placed → apply tension → seating of wedges → cutting of tendons

note:: जैक (Jack) लगाने के बाद ही tension लगाना है।

Posttensioning methods :-

method	arrangement of tendon in duct	Type of anchorage arrangement
fressinet	annular, spaced by helical wirecore • no of large wire can be tensioned at a time	• conical serrated concrete wedge driven by Jack into female cone embedded at the end of beam
Gifford-Udall	evenly spaced by perforated spacers	split conical wedge
Lee-mccall	single wire	High strength nut, spacing washer bearing on steel plate on end of beam
magnel-Blaton	Horizontal rows of 4 wires, spaced by metal grills at interval	Pairs of wires held by Flat steel wedges.

Analysis of Prestress (assumption) :-

① concrete → homogeneous & elastic material
② within range of working stress, both steel and concrete behave elastically.
③ Plain section remain plain after bending (strain diagram → linear)
④ stress in reinforcement → does not change along the length of tendon.
⑤ variation of stress in steel due to external load is negligible.

Imp

at time of transfer prestress → DL + Prestress force (without loss)
at service load condition → DL + LL + P(with loss) after

load factor (against cracking) = $\dfrac{\text{LL req. for cracking}}{\text{LL actually acting}}$

load factor (wrt total load) = $\dfrac{\text{total Load causes cracking}}{\text{total load actual acting}}$

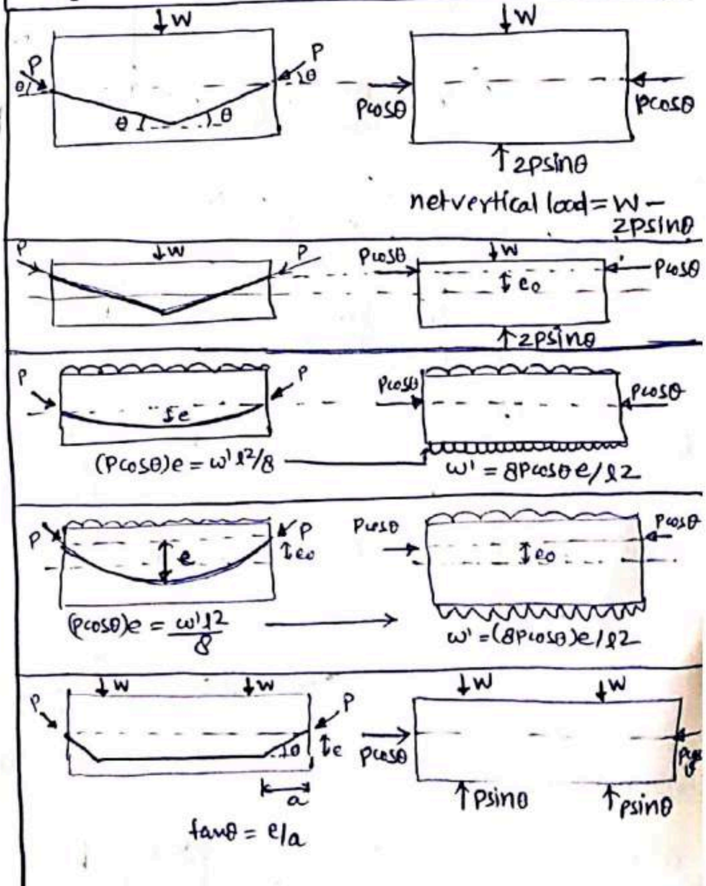
Analysis of member under Flexture → stress concept, load Balance concept, C line - P line concept

Pline → tendon line
Cline → Pressure line (Thrust line) → line of action of total compressive force

distance b/w C & pline
$\Rightarrow \text{shift} (\bar{x}) = \dfrac{M}{P} = \dfrac{M}{C}$

extreme fibre stress $\Rightarrow \dfrac{C}{A} \pm \dfrac{C \times \text{eccentricity of 'C'}}{Z}$

load Balancing concept :- Beam under self load, C/s is subjected to axial stress only if the load is fully Balanced.

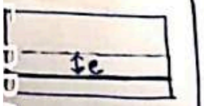


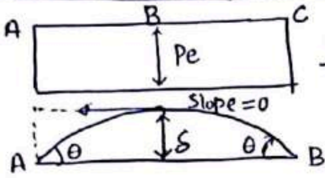
note:- Imp

choose cable Profile → Like BMD shape due to external loading
→ Point load → triangular cable profile   → udl → Parabolic "   → 2 point load (separated) → Trapezoidal "

effect of loading on stress in tendon :-

{ $\delta_{A/C}$ ⇒ put tangent at C, take moment about A }, As per moment area Theorem.



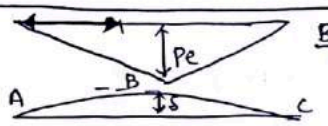


$\frac{M}{EI} = \frac{Pe}{EI}$

$\theta_B - \theta_A$ = area b/w A & B

$\boxed{\theta_A = \frac{ML}{2EI}}$ V.V.Imp

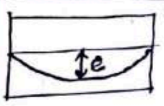
$\delta_{A/B} = \left(\frac{M}{EI} \times \frac{L}{2}\right) \times \frac{L}{4}$ $\boxed{\delta = \frac{ML^2}{8EI}}$ ↑ V.V.Imp

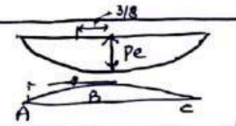


$\frac{BMD}{EI}$ (Mope)

$\theta_B - \theta_A = \frac{1}{2} \times \frac{L}{2} \times \frac{Pe}{EI}$ $\boxed{\theta_A = \frac{(Pe)L}{4EI}}$ $\frac{ML}{4EI}$

$\delta_{A/B} = \left(\frac{PeL}{4EI}\right) \times \left(\frac{2}{3} \times \frac{l}{2}\right) = \boxed{\frac{ML^2}{12EI}}$ ↑



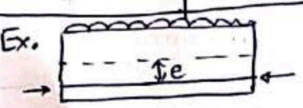


$\theta_A = \frac{2}{3} \times \frac{Pe}{EI} \times \frac{l}{2}$

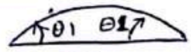
$\delta = \left(\frac{2}{3} \times \frac{Pe}{EI} \times \frac{l}{2}\right) \times \left(\frac{5}{8} \times \frac{l}{2}\right)$

$\boxed{\delta = \frac{5ML^2}{48EI}}$

Ex.



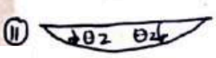
(I)



$\theta_1 = \frac{ML}{2EI}$

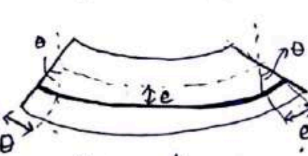
due to Prestress (P) deflected shape

(II)



due to ext. load $\theta_2 = \frac{wl^3}{24EI}$

(III) $\theta_2 > \theta_1$ net slope (↑)



(IV) net elongation in steel reinforcement $= e\theta + e\theta = 2e\theta$ ($\because \theta_2 > \theta_1$)

(V) ∴ strain $= \frac{2e\theta}{l}$ (VI) stress change $= \left(\frac{2e\theta}{l}\right) \times E_s$

(VII) change in force $= \left[\left(\frac{2e\theta}{l}\right) \times E_s\right] \times A_s$

losses in prestressing :-

Tensioning stage → anchoring stage → subsequent loss

(friction loss) (anchorage slip)

due to
E → elastic shortening
C → creep of concrete
S → shrinkage
R → Relaxation of steel

types

Immediate losses	long term losses
• friction loss • anchorage slip • elastic shortening	due to • creep of concrete • shrinkage " • relaxation of steel

	Pretensioning	Post tensioning
elastic Shortening	3 %	1 %
Creep	6 %	5 %
shrinkage	7 %	6 %
Relaxation	2 %	3 %
total	18 %	15 %
∴ min grade	M40	M30

∴ loss more in pretensioning

loss which are only in post tensioning < friction loss, anchorage slip

Imp.

(1) friction loss :- (only for post tensioning) $(P_0 - P_x)$

length effect

$\boxed{P_x = P_0 e^{-kx}}$

Curvature effect

$\boxed{P_x = P_0 e^{-\mu\alpha}}$

$P_x = P_0 e^{-(kx + \mu\alpha)}$

$P_x = P_0 (1 - (kx + \mu\alpha) + \cdots)$

$\boxed{\therefore \text{loss} = P_0 (kx + \mu\alpha)}$

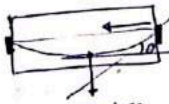
& $\boxed{\text{stress at } x = P_0 - \text{loss}}$

Jacking from 1 end

$\boxed{x = l \\ \alpha = 2\theta}$ $\theta = \tan^{-1}\left[\frac{4h}{l}\right]$

$y = \frac{4hx(l - x)}{l^2}$

Jacking from both end



$\boxed{x = l/2 \\ \alpha = \theta}$

x → length of reinforcement considered from end point to where loss is max.

α → total change in gradient b/w 2 considered point.

P_0 → Prestressing force at tensioning end.

K → friction coff. / wobble correction factor (0.0015 – 0.0050 per meter)

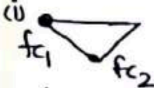
μ → coff. of friction.

(2) Anchorage slip :- (Only in post tensioned) $\boxed{\text{loss} = \frac{\Delta}{L} \times E_s}$

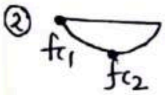
(3) elastic shortening loss :- $\boxed{\text{due to change in strain in tendon}}$

Pretensioning

$\boxed{\text{loss} = m f_c}$ → stress in concrete at same level of steel

(i)  $\boxed{f_c = \frac{f_{c1} + f_{c2}}{2}}$

(Straight cable profile)

(2)  $\boxed{f_c = \frac{f_{c1} + 2f_{c2}}{3}}$

(Parabolic cable profile)

Post tensioning

(i) if there is only 1 bar $\boxed{\text{loss} = 0}$

(ii) if all bars stressed simultaneously and anchored. $\boxed{\text{loss} = 0}$

{ ∵ intial prestressing force measure only after tensioning and hence - elastic shortening would have already occured before bar is anchored.

Imp.

(iii) if 3 bars (All bars) stresses → successively

(a) when (1st) bar is tensioned & anchored $\boxed{\text{no loss in bar 1}}$

(b) 2nd ⇒ $\boxed{\text{no loss in bar 2}}$ + $\boxed{\text{loss in bar 1 due to tensioning of 2 bar} \; \underbrace{m f_{c12}}_{\text{avg. of stress in concrete at bar 1 level}}}$

(c) 3rd ⇒ $\boxed{\text{no loss in bar 3}}$ + $\boxed{\text{loss in bar 2} \; (m f_{c23})}$ at level of 2. + $\boxed{\text{loss in bar 1} \; m f_{c13}}$ at level of 1.

(4) loss of Prestress due to creep of concrete :-

$\boxed{\text{loss} = m \theta f_c}$

age at loading	θ
7 day	2.2
28	1.6
1 year	1.1

note:- Imp

• creep cofficient (θ) vary logarithm with time

• creep decreased with log of time in days.

Ex. for $\theta_{15} = \underset{(2.2)}{\theta_7} + \frac{(\overset{1.6}{\theta_{28}} - \overset{2.2}{\theta_7})}{(\log 28 - \log 7)} \left[\log 15 - \log 7\right]$ } interpolation

(5) Shrinkage of concrete के कारण loss :.

$\boxed{\text{Pre}}$ $(3 \times 10^{-4}) E_s$ → total residual shrinkage strain

$\boxed{\text{Post}}$ $\frac{(2 \times 10^{-4}) E_s}{\log(T+2)}$ → T: age of concrete at transfer in days.

(6) loss due to relaxation of steel (creep of steel) :-

↳ "decrease in stress with time" under constant strain

$\boxed{\text{loss} = 1 - 1.5\% \text{ of Initial stress}}$

note:- $\theta = \frac{\text{creep strain}}{\text{elastic strain}} = \frac{\text{long term strain} - \text{short term}}{\text{short term strain}}$

Imp

Concordant tendon profile :- (in continuous beam)

• eccentricity ∝ BM caused by loading at all c/s

Imp

• Pressure line (due to prestress) coincide with cable profile.

• Shift of Pressure line due to external load can be measure from profile directly.

• no reaction at support

• no any secondary moment in span.

IS:1343: Prestress Concrete		
	type 1	no cracking under service load
	type-2	no visible crack $\sigma_t < 3$ MPa
	type-3	Crack width within allowable limit

↓

Pre compressive force is induced to eliminate or reduce tensile stress and hence crack - development is not permitted.

• The limiting principle tensile stress in an uncracked prestress concrete ⇒ $\boxed{0.24\sqrt{f_{ck}}}$

IS: 1343 :- (i) Allowable upward deflection in prestress concrete member under serviceability in Limit state ⇒ $\boxed{\frac{\text{span}}{300}}$

(ii) at time of initial tensioning.

max. tensile stress immediately behind anchorage ≯ 0.80 × ultimate tensile strength of wire

(iii) single wire pretensioning :-

min spacing ⇒ max $\begin{cases} 3\phi \\ \frac{4}{3} \text{ of max. agg. size} \end{cases}$

• In prestress str :-

Better deflection controll + crack controll

∴ use high performance concrete

Bursting Tension in anchorage Zone $\Rightarrow$ fn $\left[\dfrac{\text{depth of anchorage}}{\text{depth of equivalent prism}}\right]$

Transmission length at the end of Pretension member depend on $\longrightarrow$ dia of high-tensile wire

Very near end face of Pretension beam $\longrightarrow$ Tensile stress in concrete developed