RCC & Prestressed Concrete

Index

Limit state method of design

Limit State Method

IS 456 Standards for Beams and Slabs and Columns

Effective span

A. Simply supported beams and slabs (l_{eff})

Here, l_0 = clear span

w = width of support

d = depth of beam or slab

B. For continuous beam

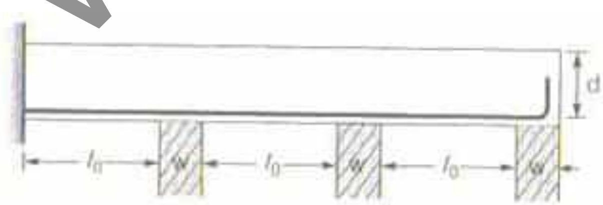
(i) If width of support < 1/12 of clear span

$$l_{eff} = \text{minimum}\begin{cases} l_0 + w \\ l_0 + d \end{cases}$$

(ii) If width of support > 1/12 of clear span

(a) When one end fixed other end continuous or both end continuous.

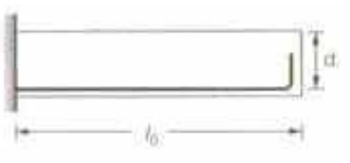
$l_{eff} = l_0$



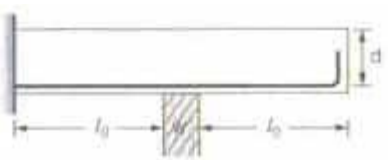
(b) When one end continuous and other end simply supported

$$l_{eff} = \text{minimum}\begin{cases} l_0 + w/2 \\ l_0 + d/2 \end{cases}$$

C. Cantilever

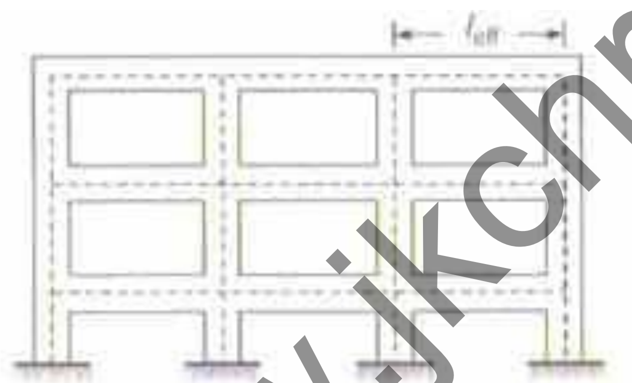
(i) 

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

(ii) 

$$l_{eff} = \left(l_0 + \frac{w}{2}\right)$$

D. Frames



l_{eff} = Centre to centre distance

Control of deflection

(i) This is one of the most important check for limit state of serviceability.

(a) The final deflection due to all loads including the effect of temperature, creep and shrinkage and measured from as cast level of the support of floors, roofs and other horizontal members should not normally exceed span/250

(b) The deflection including the effect of temperature, creeps and shrinkage occurring after erection of partition and application of finishes should not normally exceed span/350 or 20 mm which ever is less.

(ii) The vertical deflection limit may generally be satisfied if

(a) Basic span to effective depth ratio for span upto 10m is

Types of Beams: $\frac{span}{effective\ depth}$

For cantilever → 7

For simply supported → 20

For continuous → 26

(b) For span > 10 m effective depth

$$= \frac{(span)^2}{10 \times 2}$$

Where 'A' is span to effective depth ratio for span upto 10m.

(c) Depending upon the tension reinforcement the value 'A' can be modify by multiplying a factor called modification factor (MF_1)

effective depth $= \frac{span}{A \times MF_1}$

where,

$$f_s = 0.58 f_y \times \frac{Area\,of\ steel\,required}{Area\,of\ steel\ provided}$$

(d) Depending upon area of compression reinforcement, value (A) can be further modified using a modification factor (MF_2)

effective depth = $\frac{span}{A \times MF_1 \times MF_2}$

(e) For flanged beam : A reduction factor is used

(f) Deflection check for two way slab:

Support Condition	Span/overall depth	
	Mild Steel	Fe 415/Fe 500
Simply supported	35	28
Continuous	40	32

Slenderness limit

1. For simply supported or continuous beams

$$l_0 \not> \text{minimum}\begin{cases} 60b \\ 250\dfrac{b^2}{d} \end{cases}$$

2. where, l_0 = Clear span
   b = Width of the section
   and, d = Effective depth

$$l_0 \not> \text{minimum}\begin{cases} 25\,b \\ 100\dfrac{b^2}{d} \end{cases}$$

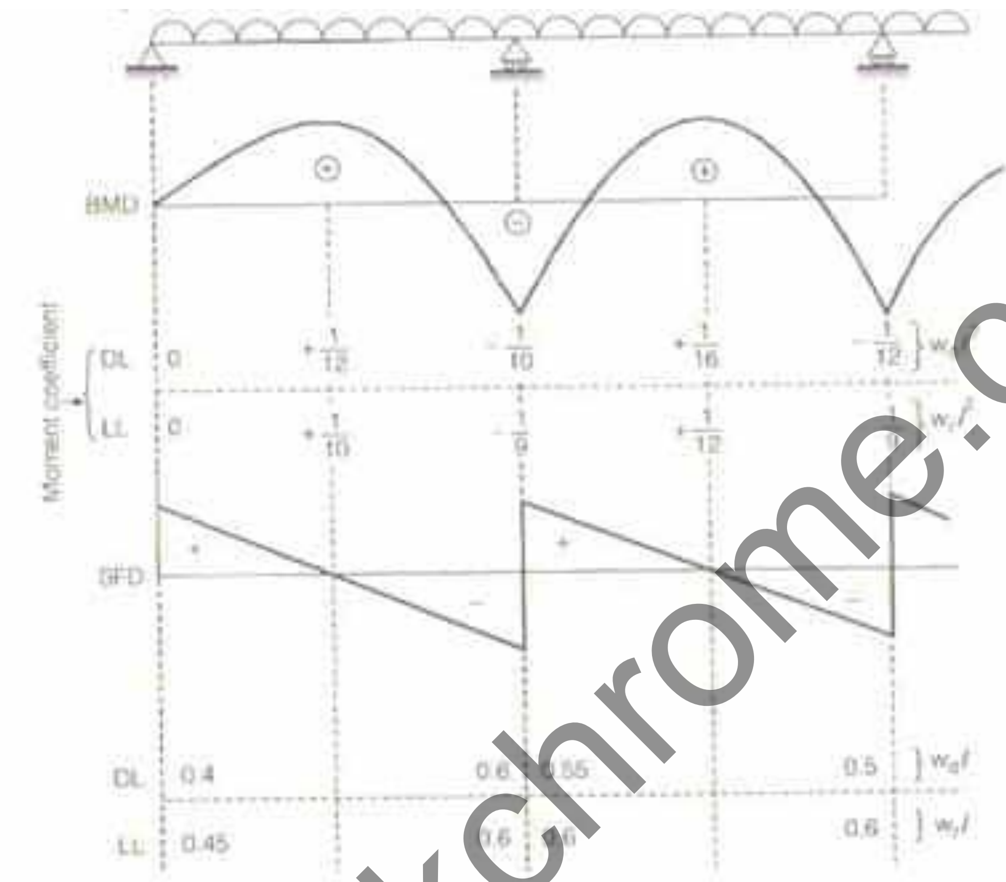
3. For cantilever beam

$$\frac{A_{st}}{bd} = \frac{0.85}{f_y}$$

   1. Minimum tension reinforcement
   2. Maximum tension reinforcement = 0.04 bD
   3. Maximum compression reinforcement = 0.04 bD
      where, D = overall depth of the section
   4. Where, D > 750 mm, side face reinforcement is provided and it is equal to 0.1% of gross cross-section area (b × D). It is provided equally on both face.
   5. Maximum spacing of side face reinforcement is 300 mm.
   6. Maximum size of reinforcement for slab/beam is 1/8 of total thickness of the member
   7. Nominal cover for different members
      Beams → 25 mm
      Slab → 20 to 30 mm

Column → 40 mm
Foundations → 50 mm

8. Moment and shear coefficient for beams/slabs



One way slab

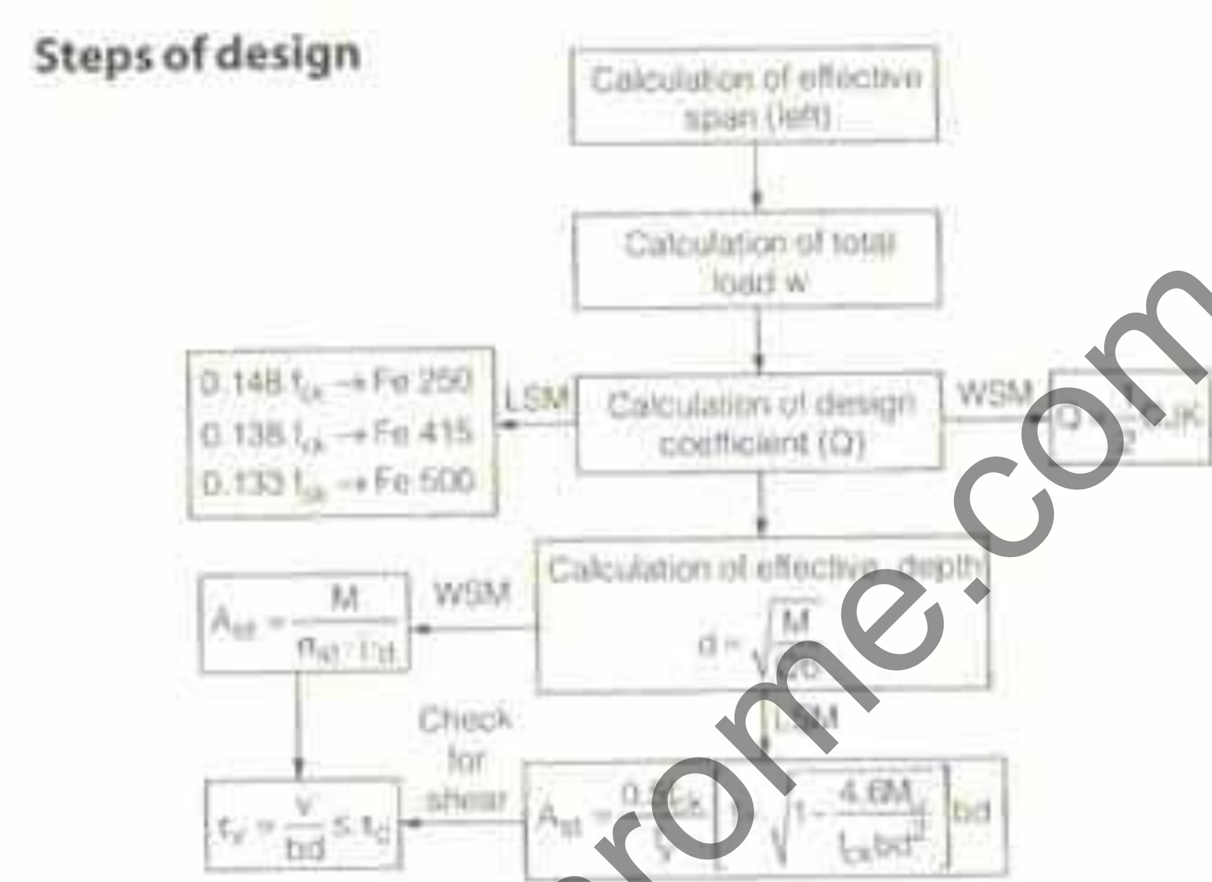
(i) **Ly/Lx ratio** is less than 2

where, l_y = length of longer span

l_x = length of shorter span

(ii) Slab is supported only on two edges.

Design of One way slab



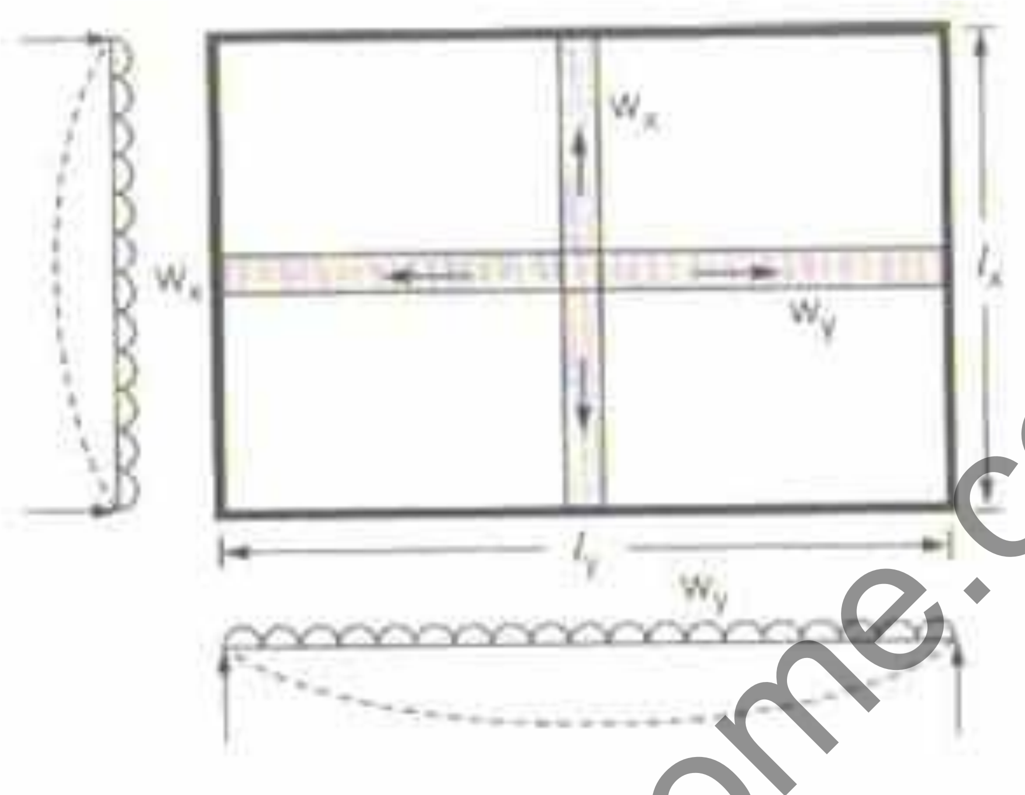
Two way slab

(i) **Ly/Lx ratio** is more than 2

(ii) Slab is supported on all edges.

Design of two way slab

1. Grasoff Ranking method
   It is used for corners not held down position.
   It is purely simply supported case.

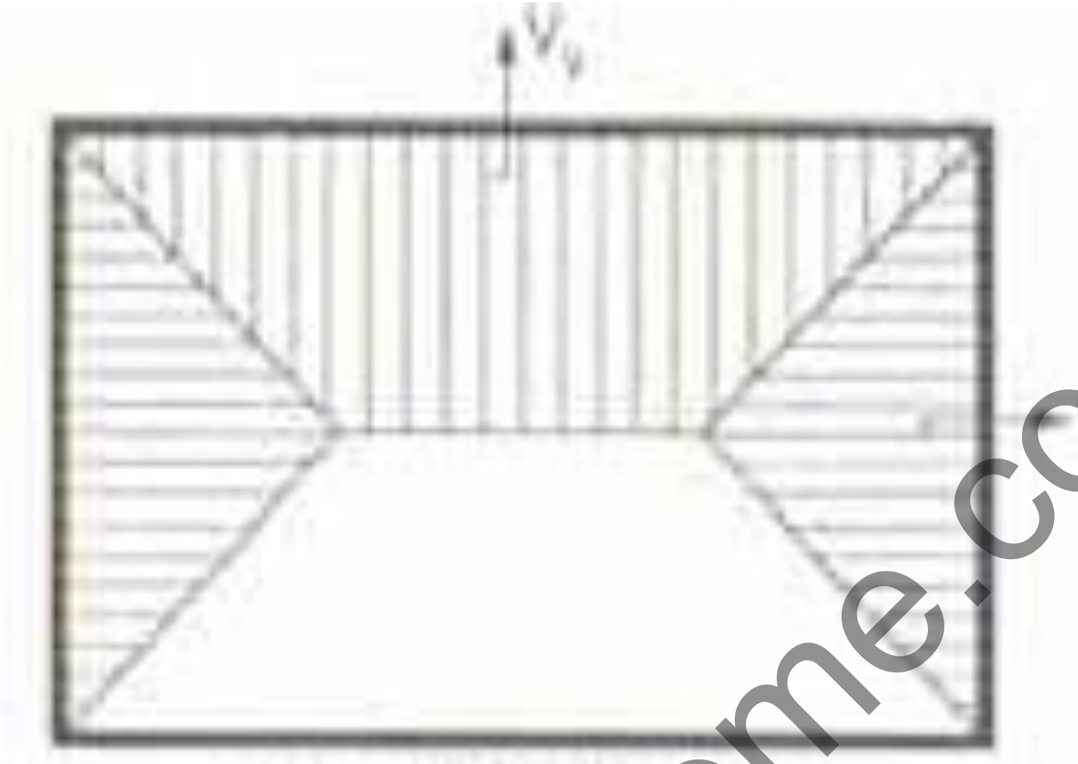


(i) $w_y = \left(\frac{1}{1+r^4}\right)w \quad w_x = \left(\frac{r^4}{1+r^4}\right)w$

(ii) Moment in x-direction $(M_x) M_x = \frac{w_x l_x^2}{8}$

Moment in y-direction $(M_y) M_y = \frac{w_y l_y^2}{8}$

(iii) Shear force



(Load Distribution)

At shorter edge (V_X)

$$V_x = \frac{1}{3} \cdot wl_x$$

At longer edge (V_y)

$$V_y = \left(\frac{r}{2+r}\right) w \cdot l_x$$

1. **Design of slab with corner held down position**

(a) Pigeauds method: $M_x = r_x^{'} \cdot \frac{wl_x^2}{8} \quad M_y = r_y^{'} \cdot \frac{wl_y^2}{8}$

where, the values of $r_x^{'}$ and $r_y^{'}$ are read from table

(b) **I.S. code method**

$M_X = \alpha_x w l_y^2 \quad M_y = \alpha_y w l_x^2$

The values of α_x and α_y read from table (page 91, IS : 456-2000)

Shear, torsion & Bond and Development length

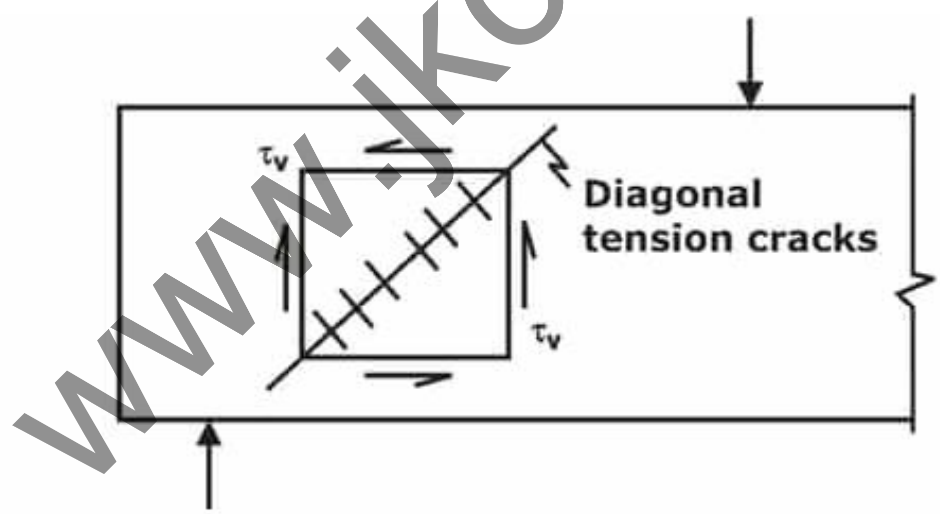
A. LIMIT STATE OF SHEAR

1. INTRODUCTION

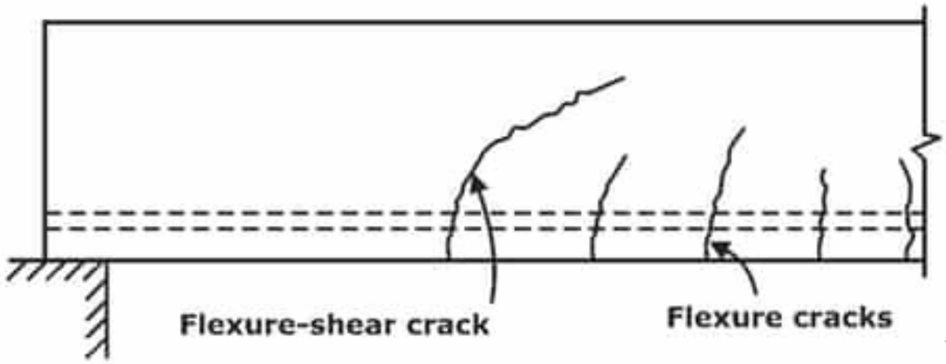
RCC structures are generally subjected three type of shear stresses namely Flexural shear, punching shear and torsional shear. Diagonal tension arises in the section due to the shear forces which results in formation of cracks in concrete as concrete is weak in tension. Hence, limit state of shear must be checked in design of section. Shear reinforcement is provided in the form of vertical stirrup or bent up bars.

Following are the major mechanism in which shear failure takes place:

(i) Diagonal Tension: At the support of simply supported beam, bending stresses are zero and the shear stresses are maximum. Thus, tensile stresses generated along a plane inclined at 45˚ to the horizontal. As concrete is weak in tension, cracks start to develop at an angle of 45˚.



(ii) Flexural Shear: This type of cracks occurs due to large bending moment and small shear force. Cracks are normally inclined at 90˚ to the horizontal.



(iii) Diagonal Compression: This type of failure also takes place along with diagonal tension in heavily reinforced section. It is characterised by crushing of concrete.

NOMINAL SHEAR STRESS

The average shear stress can be calculated using the following formula:

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

Where,

V_u = ultimate shear stress at the section

b = width of the section

d = effective depth of the section

For beams with varying depth

$$\tau_v = \left[\frac{V_2 \pm (M_u/d)\tan\beta}{bd}\right]$$

Where,

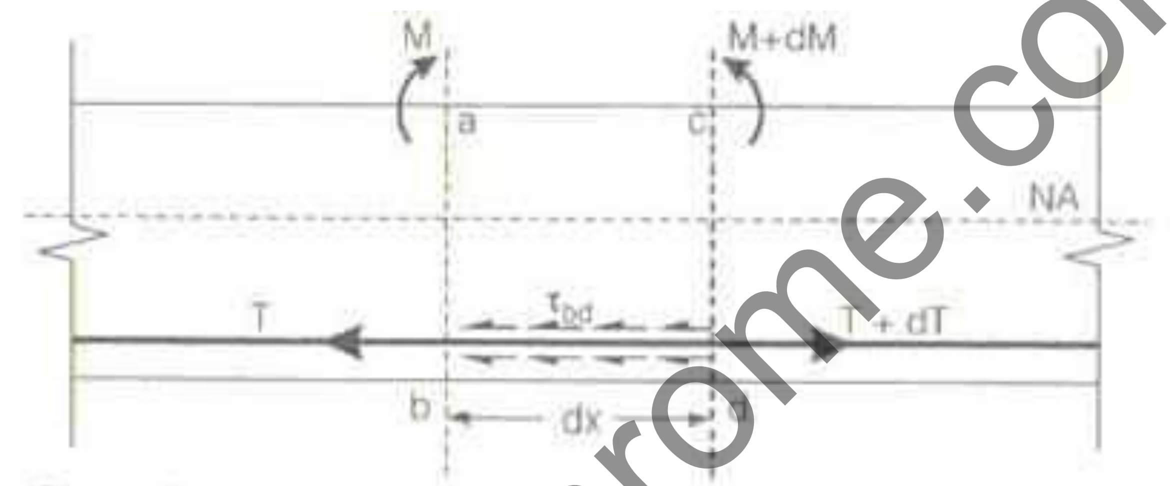
β = inclination of flexural tensile force to the horizontal.

M_u = factored bending moment at the section.

Bond, Anchorage and Development Length

Bond stress (τ_{bd})

$$\tau_{bd} = \frac{V}{\sum pjd}$$



Where V = Shear force at any section

d = Effective depth of the section

∑ p = Sum of all perimeter of reinforcement

$= n \cdot \pi(\phi)$

n = Number of reinforcement

$\phi =$ diameter of reinforcement

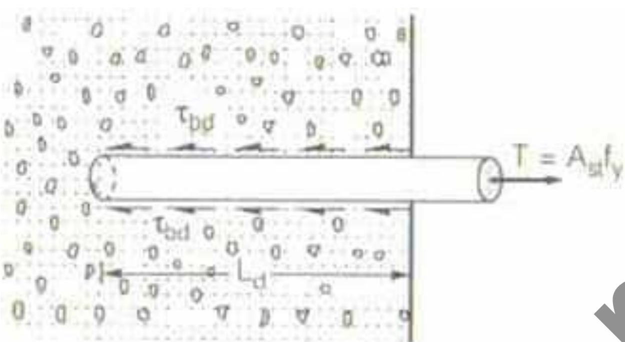
Permissible bond stress

As per IS 456 : 2000

	M15	M20	M25	M30	M35	M40
WSM	0.6	0.8	0.9	1.0	1.1	1.2
LSM	--	1.2	1.4	1.5	1.7	1.9

- These Value of bond stress is for plain bar in tension.
- For deformed bar the above value should be increased by 60%.
- For bar in compression the above value should be increased by 25%.

Development Length (L_d)



$$L_d = \frac{\phi \sigma_{st}}{4 \cdot \tau_{bd}} \; For\ WSM$$

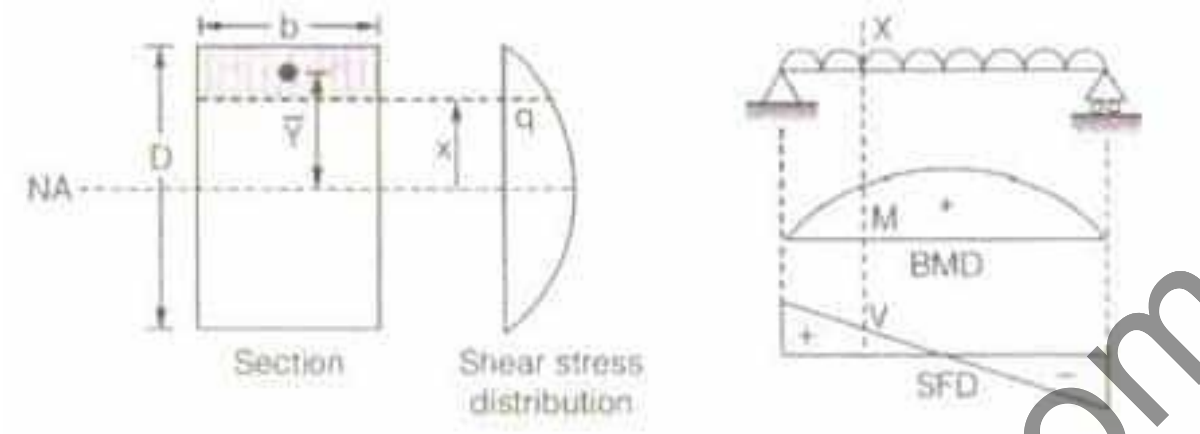
$$L_d = \frac{\phi \cdot 0.87 f_y}{4 \cdot \tau_{bd}} \; For\ LSM$$

Shear stress

Shear stress

(a) For Homogeneous beam

$$q = \frac{V}{Ib} \cdot A\overline{Y}$$



where,

q = shear stress at any section

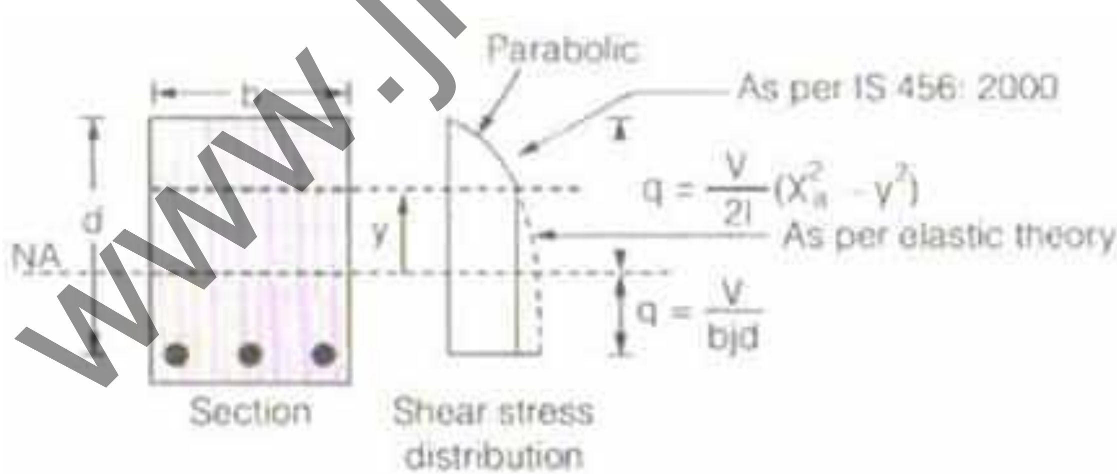
V = shear force at any section

$A\overline{Y} =$ Moment of area of section above the point of consideration

I = Moment of inertia of section

$$= \frac{bD^3}{12}$$

(b) For Reinforced concrete beam



(i) Shear stress *above* NA

$$q = \frac{V}{2l} \cdot (x_a^2 - y^2) \; q_{\max} = \frac{V}{2l} \cdot x_a^2 \; at \; y = 0$$

(ii) Shear stress below NA

$$q = \frac{V}{bjd}$$

Nominal shear stress,

$$\tau_V = \frac{V}{bd}$$

Design shear strength of concrete (τ_C) without shear reinforcement as per IS 456: 2000 τ_C depends on

(i) Grade of concrete

(ii) Percentage of steel,

$$p = \frac{A_{st}}{bd} \times 100$$

Where,

A_{st} = Area of steel

b = Width of the Beam

d = Effective depth of the beam

p	WSM		LSM	
	M 20	M 25	M 20	M 25
$0 \leq 0.15$	0.18	0.19	0.28	0.29
0.25	0.22	0.23	0.36	0.36
0.50	0.30	0.31	0.48	0.49
0.75	0.35	0.36	0.56	0.57
1.00	0.39	0.40	0.62	0.64

Maximum shear stress $\tau_{c,max}$ with shear reinforcement is

	M15	M20	M25	M30	M35	M40 & above
LSM	2.5	2.8	3.1	3.5	3.7	4.0
WSM	1.6	1.8	1.9	2.2	2.3	2.5

$\tau_V \ngtr \tau_{C\max}$

Minimum shear reinforcement (As per IS 456 : 2000)

$$\frac{A_{SV}}{bS_V} \geq \frac{0.4}{0.87\, f_y}$$

This is valid for both WSM and LSM

$$S_V \leq \frac{2.175\, f_y\, A_{SV}}{b}$$

where,

A_{SV} = Area of shear reinforcement

S_V = Spacing for shear reinforcement

Spacing of shear reinforcement

Maximum spacing is minimum of (i), (ii) and (iii)

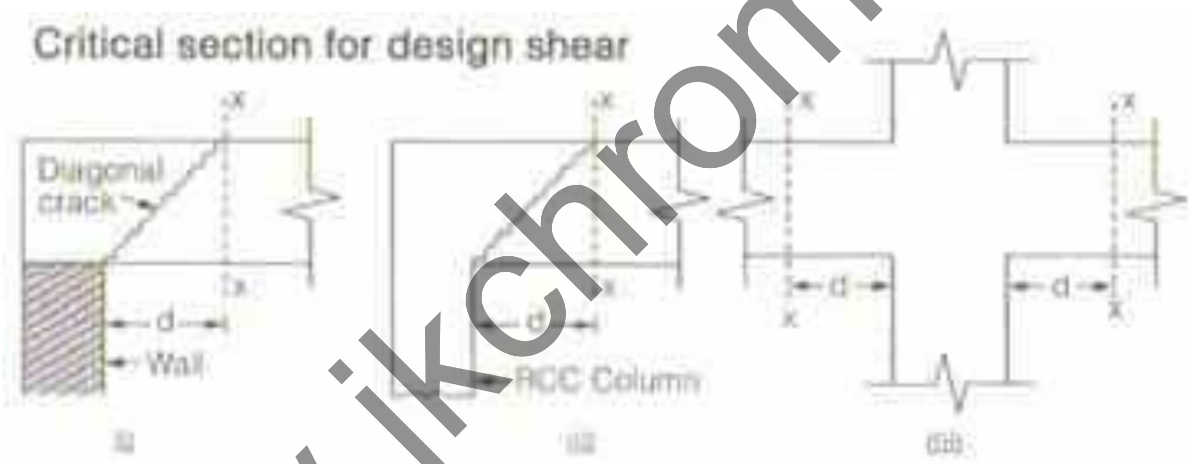
(i) $S_V = \dfrac{2.175\, f_y\, A_{SV}}{b}$

(ii) 300 mm

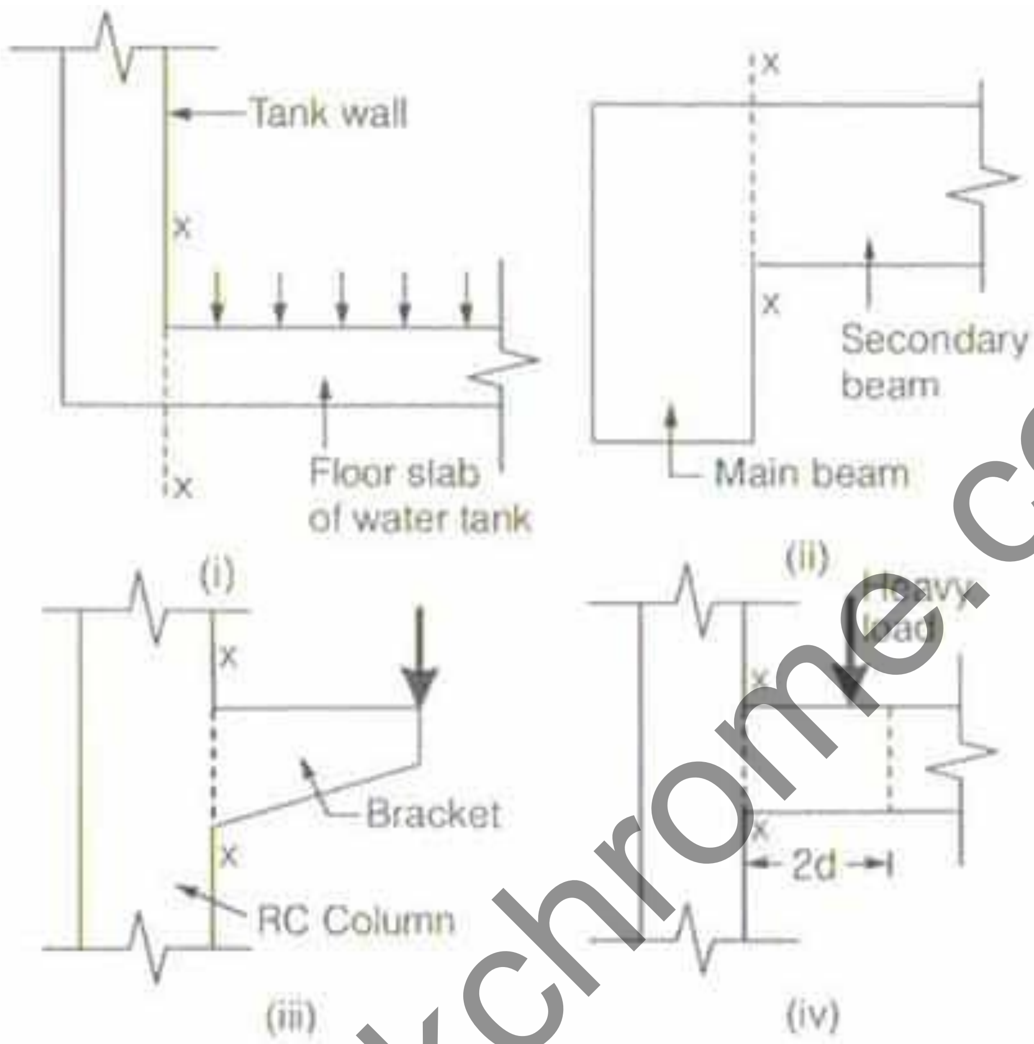
(iii) 0.75 → For vertical stirrups

d → For inclined stirrups

where, d = effective depth of the section



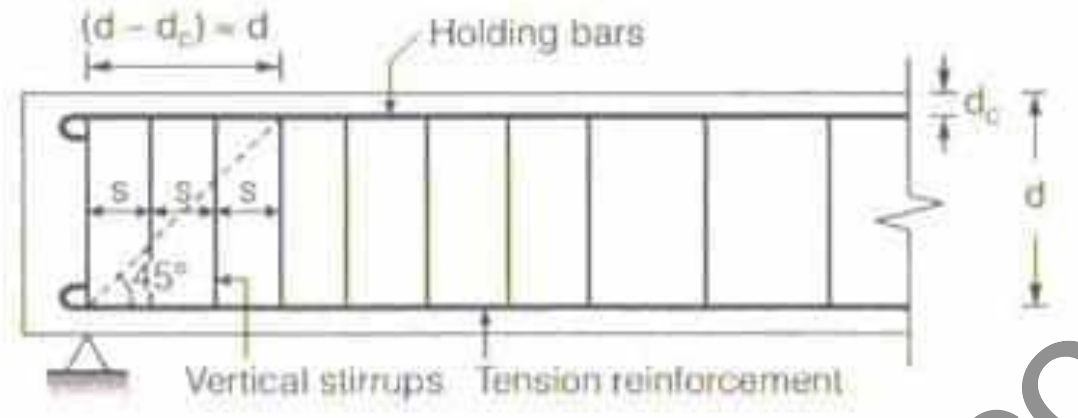
(a) Critical section X-X at d from the face of the support



(b) Critical section X-X at the face of the support

The above provisions are applicable for beams generally carrying uniformly distributed load or where the principal load is located beyond 2d from the face of the support.

Vertical stirrups:



Shear force V_S will be

Resisted by shear Reinforcement provided in 'd' length of the beam,

$$V_S = \left(\frac{d}{S_V}\right) A_{SV} . \sigma_{SV} \ \ for\ WSM$$

where, A_{SV} = Cross-sectional area of stirrups

S_V = Centre to centre spacing of stirrups

$$V_{Su} = \left(\frac{d}{S_V}\right) A_{SV} (0.87\, f_y) \ \ for\ LSM$$

Inclined stirrups: or a series of bars bent-up at different cross-section:

$$V_S = A_{SV} \cdot \sigma_{SV} \cdot (\sin\alpha + \cos\alpha)\left(\frac{d}{S_V}\right) \ for\ WSM$$

$$V_{Su} = A_{SV} \cdot (0.87\ f_y)(\sin\alpha + \cos\alpha)\left(\frac{d}{S_V}\right) \ LSM$$

Bent up Bars:

- Single or a group of bent up bars are provided at distance $\sqrt{2}a = \sqrt{2}\,jd$ from support.

Generally bar should not be bent up beyond a distance l/4 from the support. Where *l* = length of span.

B. LIMIT STATE OF TORSION

1. **INTRODUCTION**

In RCC structures generally torsion is accompanied by flexure and shear. It occurs if the line of action of force does not passes through the shear centre. Torsion is divided into two categories:

(i) Primary Torsion: Developed due to eccentric loading.

(ii) Secondary Torsion: Torsion is induced in the structure due to rotation of a member which depends on torsional stiffness of the member.

1. **DESIGN OF TORSION**

Torsion induces shear stress in the member which causes warping of non-circular section. Diagonal tension develops in the section which causes torsional cracking in the section. Torsional strength is improved by providing longitudinal and transverse reinforcement. The longitudinal reinforcement helps in reducing cracks through dowel action while transverse reinforcement helps in resisting shear stresses.

As per IS 456 the effect of torsional stresses is divided into

(i) Equivalent Shear

(ii) Equivalent moment

2.1. Equivalent Shear

$$V_e = V_u + 1.6\frac{T_u}{B}$$

V_e = Equivalent shear force

V_u = Shear force

T_u = Torsional moment

B = Width of the section

The equivalent nominal shear stress is calculated by dividing the equivalent shear force divided by area of the section.

2.2. Longitudinal reinforcement

The longitudinal tension reinforcement should be designed to carry equivalent bending moment of

$$M_{e1} = M_u + M_t$$

Where,

M_u = Flexural moment

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

T_u = Torsional moment

D = Overall depth of the section

b = width of the section

- If numerical value of M_t exceeds the numerical value of M_u, compression longitudinal reinforcement is provided to resist the bending moment equal to (M_t – M_u).
- The reinforcement should be provided close to the corners.
- If the depth of the section exceeds 450 mm, the area of reinforcement equal to 0.1% of web area should be distributed equally on both faces.

2.3. Transverse Reinforcement

As per Is 456, transverse reinforcement is provided in the form of two legged closed hoops. The area of transverse reinforcement is obtained by the following formula:

$$A_{sv} = \frac{T_u s_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u s_v}{2.5 d_1 (0.87 f_y)}$$

Subjected to a maximum value of $\frac{(\tau_{ve} - \tau_c) b s_v}{0.87 f_y}$.

Where,

T_u = Torsional moment

V_u = Shear force

s_v = Spacing of shear reinforcement

b_1 = centre to centre distance between corner bar in the direction of width

d_1 = centre to centre distance between corner bar in the direction of depth

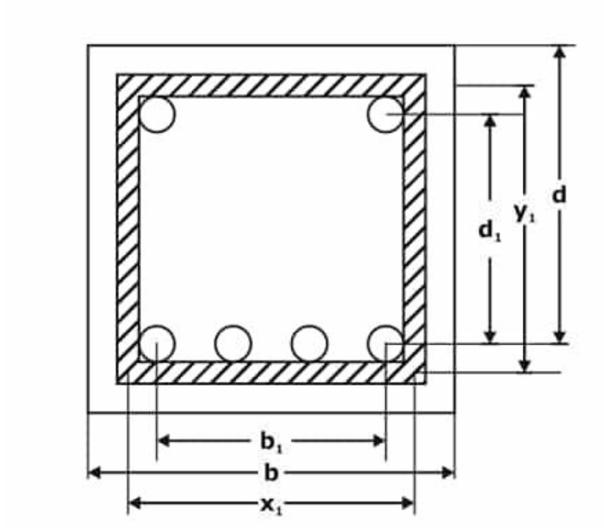
b = width of the member

f_y = Characteristics strength of stirrup reinforcement

τ_{ve} = equivalent nominal shear stress

τ_c = shear strength of concrete

Note: The distribution of transverse reinforcement should be such that the spacing should be a minimum value of x_1, $\frac{x_1 + y_1}{4}$ or 300 mm where x_1 and y_1 are short and long dimension of stirrup.



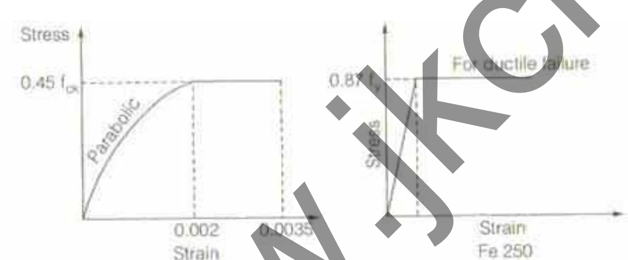
$x_1 = b_1$ + Diameter of longitudinal bar + Diameter of stirrup

$y_1 = d_1$ + Diameter of longitudinal bar + Diameter of stirrup

Design of Beam & Design of slabs

Design of Beam

Design stress-strain curve at ultimate state



Design value of strength

For concrete

$$f_d = \frac{f}{\gamma_{mc}} = \frac{0.67\, f_{ck}}{1.5} = 0.45\, f_{ck}$$

where,

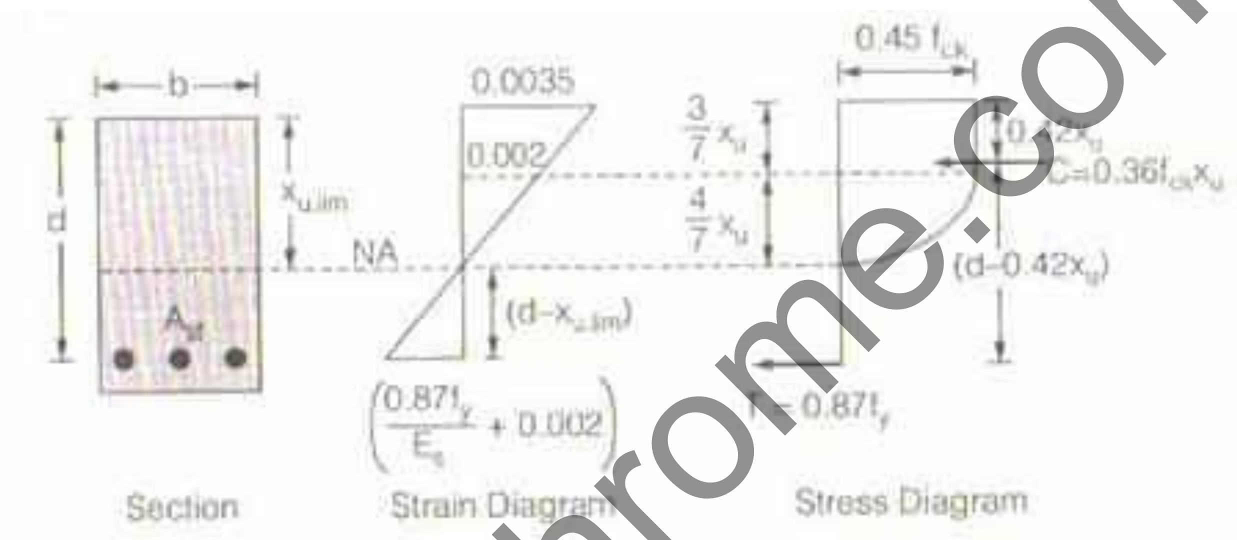
$\gamma_{mc} =$ Partial factor of safety for concrete = 1.5

f_d = design value of strength

For steel

$$f_d = \frac{f_y}{1.15} = 0.87\ f_y$$

Singly Reinforced Beam



1. Limiting depth of the neutral axis ($x_{u,\,lim}$)

$$x_{u,\lim} = \frac{700}{0.87\, f_y + 1100} \times d$$

Steel	Xu, lim
Fe 250	0.53 d
Fe 415	0.48 d
Fe 500	0.46 d

Here d = effective depth of beam

1. Actual depth of neutral axis (X_u)

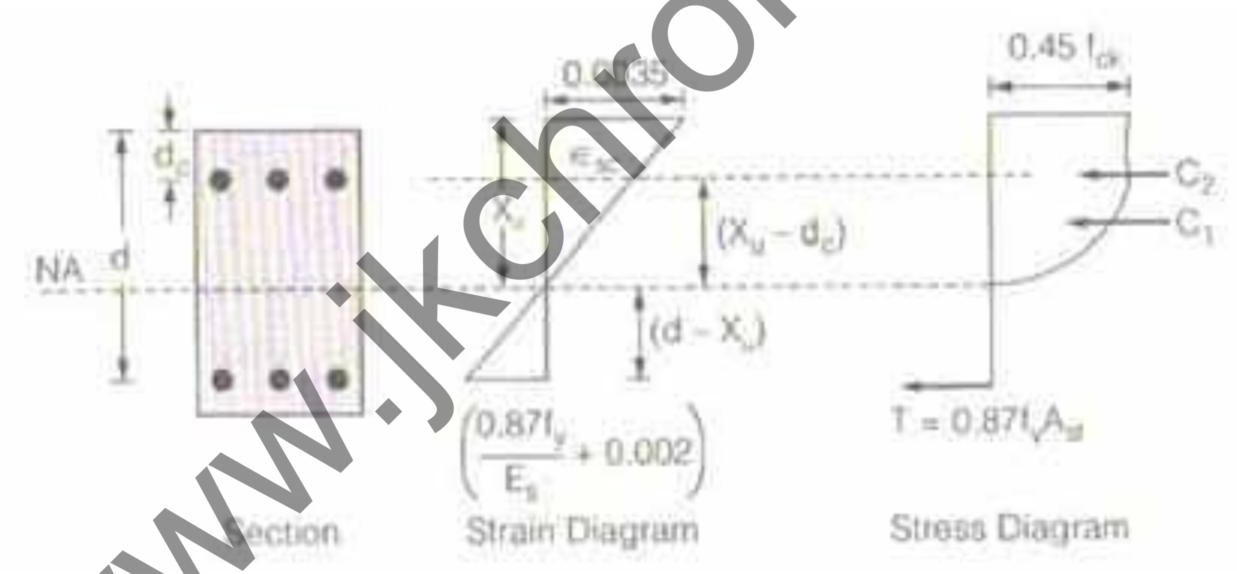
$$C = T \Rightarrow X_u = \frac{0.87\, f_y A_{st}}{0.36\, f_{ck} b}$$

2. Lever arm = d – 0.42 X_u
3. Ultimate moment of resistance

Some special cases

1. When $X_u < X_{u,lim}$
   It is an under-reinforced section
   $$M_u = 0.36 f_{ck} b X_u (d - 0.42 X_u)$$
   $$or\ M_u = 0.87 f_y A_{st} (d - 0.42 X_u)$$
2. When $X_u = X_{u,lim}$
   It is balanced section
   $$M_u = 0.36 f_{ck} b X_{u,lim} (d - 0.42 X_{u,lim})$$
   $$or\ M_u = 0.87 f_y A_{st} (d - 0.42 X_{u,lim})$$
3. When $X_u > X_{u,lim}$
   It is over reinforced section. In this case, keep X_u limited to $X_{u,lim}$ and moment of resistance of the section shall be limited to limiting moment of resistance, ($M_{u,lim}$)

Doubly Reinforced Section



1. Limiting depth of neutral axis

1. For actual depth of neutral axis (X_u)

$$C = T \Rightarrow C_1 + C_2 = T$$
$$\downarrow$$
$$0.36 f_{ck} b X_u + (f_{sc} - 0.45 f_{ck}) A_{sc} = 0.87 f_y A_{st}$$

1. Ultimate moment of resistance

$$M_u = 0.36 f_{ck} b X_u (d - 0.42 X_u) + (f_{sc} - 0.45 f_{ck}) A_{sc} (d - d_C)$$

where f_{SC} = stress in compression steel and it is calculated by strain at the location of compression steel (f_{SC})

T-Beam

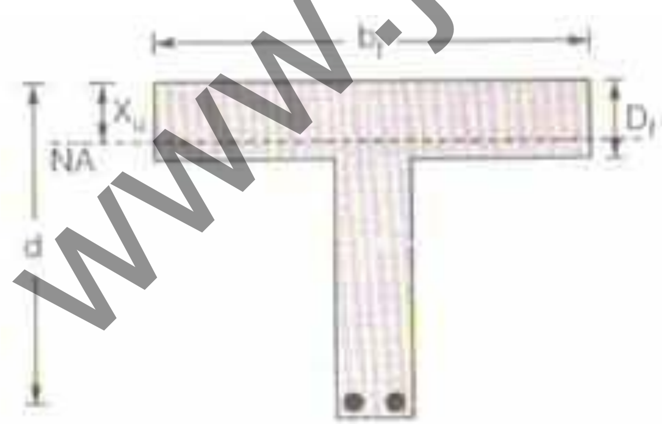
1. Effective width of flange
   Discussed in WSM
2. Limiting depth of neutral axis

$$X_{u,\lim} = \frac{700}{0.87 f_y + 1100} \times d$$

Singly reinforced T-Beam

Case-1: When NA is in flange area

i.e., $X_u < D_f$



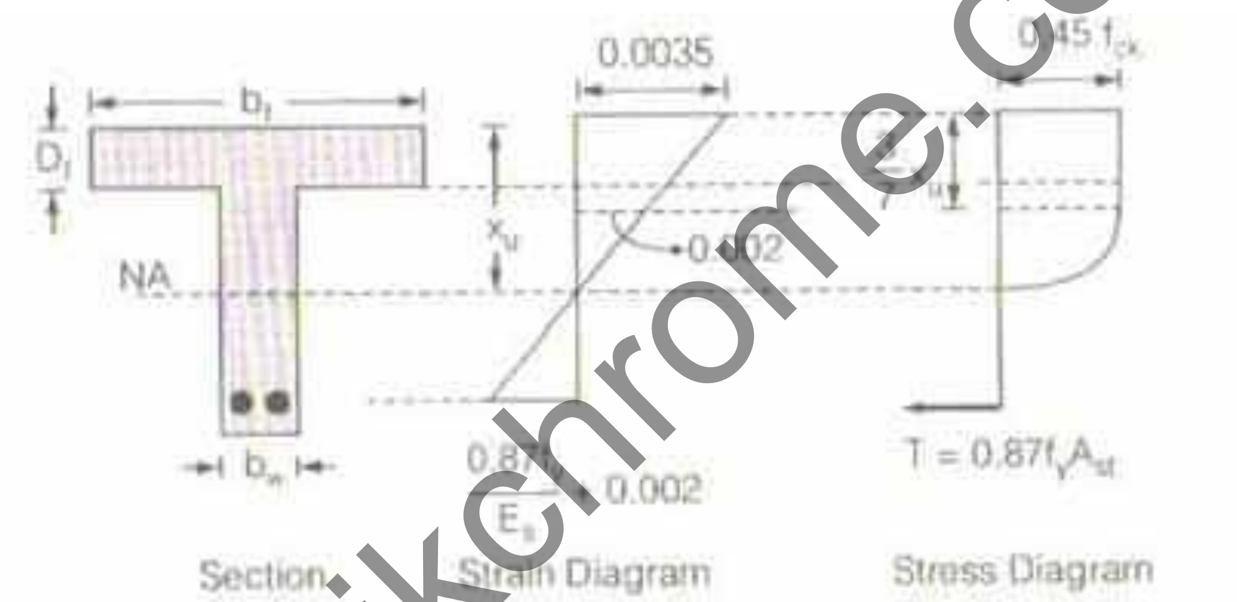
(a) for Xu

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

(b) Ultimate moment of resistance

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

Case-2: When NA is in web area ($X_u > D_f$)



Case (a) when $X_u > D_f$ and

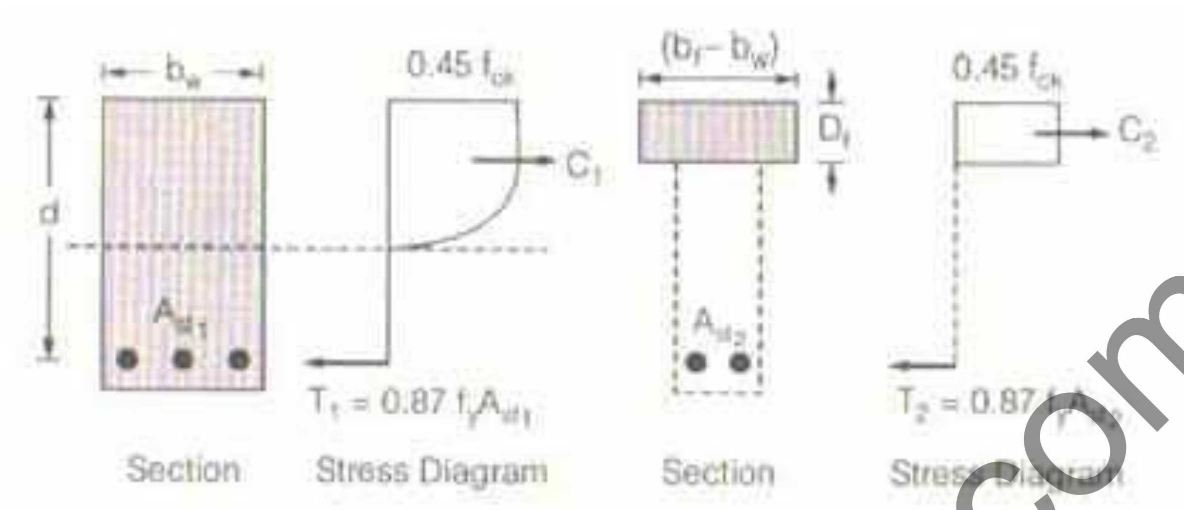
$$D_f < \frac{3}{7} X_u$$

i.e., depth of flange in less than the depth of the rectangular portion of stress diagram.

1. For actual depth of neutral ais

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

1. Ultimate moment of resistance



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

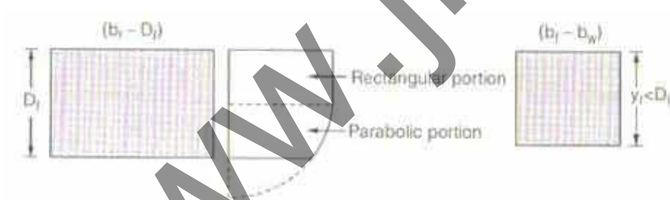
$$M_u = 0.87 f_y A_{st_1}(d - 0.42 x_u) + 0.87 f_y A_{st_2} \left(d - \frac{D_f}{2} \right)$$

$$A_{st_1} = \frac{0.36 f_{ck} b_w x_u}{0.87 f_y}, A_{st_2} = \frac{0.45 f_{ck}(b_f - b_w) D_f}{0.87 f_y}$$

Special Case (2): When Xu > Df and

$$D_f > \frac{3}{7} X_u$$

i.e., depth of flange is more than depth of rectangular portion of stress diagram.



As per IS 456 : 2000

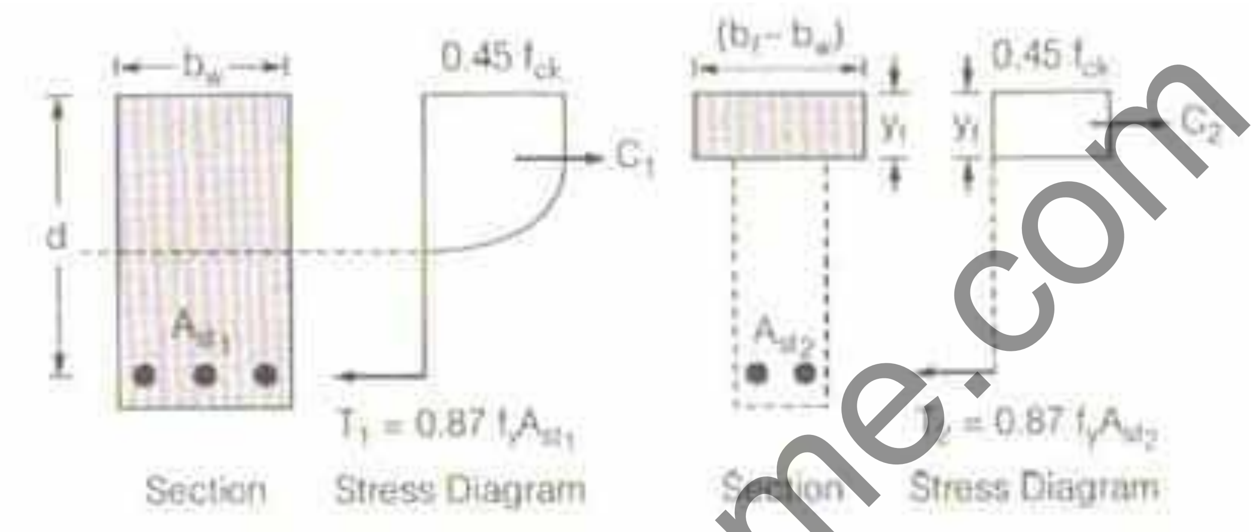
$(b_f - b_w)$ D_f portion of flange is converted into $(b_f - b_w)y_f$ section for which stress is taken constant throughout the section is 0.45 f_{ck}.

As per IS 456 : 2000

$$y_f = 0.15 X_u + 0.65 D_f < D_f$$

1. For actual depth of neutral axis

$$0.36 f_{ck} b_w X_u + 0.45 f_{ck}(b_f - b_w) y_f = 0.87 f_y A_{st_1} + 0.87 f_y A_{st_2}$$
$$or\ 0.36 f_{ck} b_w X_u + 0.45 f_{ck}(b_f - b_w) y_f = 0.87 f_y A_{st}$$



$$\boxed{M_u = 0.36 f_{ck} b_w X_u (d - 0.42 X_u) + 0.45 f_{ck} (b_f - b_w) y_f \left(d - \frac{y_f}{2}\right)}$$

$$\boxed{M_u = 0.87 f_y A_{st_1} (d - 0.42 X_u) + 0.87 f_y A_{st_2} \left(d - \frac{y_f}{2}\right)}$$

$$\boxed{A_{st_1} = \frac{0.36 f_{ck} b_w X_u}{0.87 f_y}} \quad \text{and} \quad \boxed{A_{st_2} = \frac{0.45 f_{ck} (b_f - b_w) y_f}{0.87 f_y}}$$

DESIGN OF SLAB

One way slab

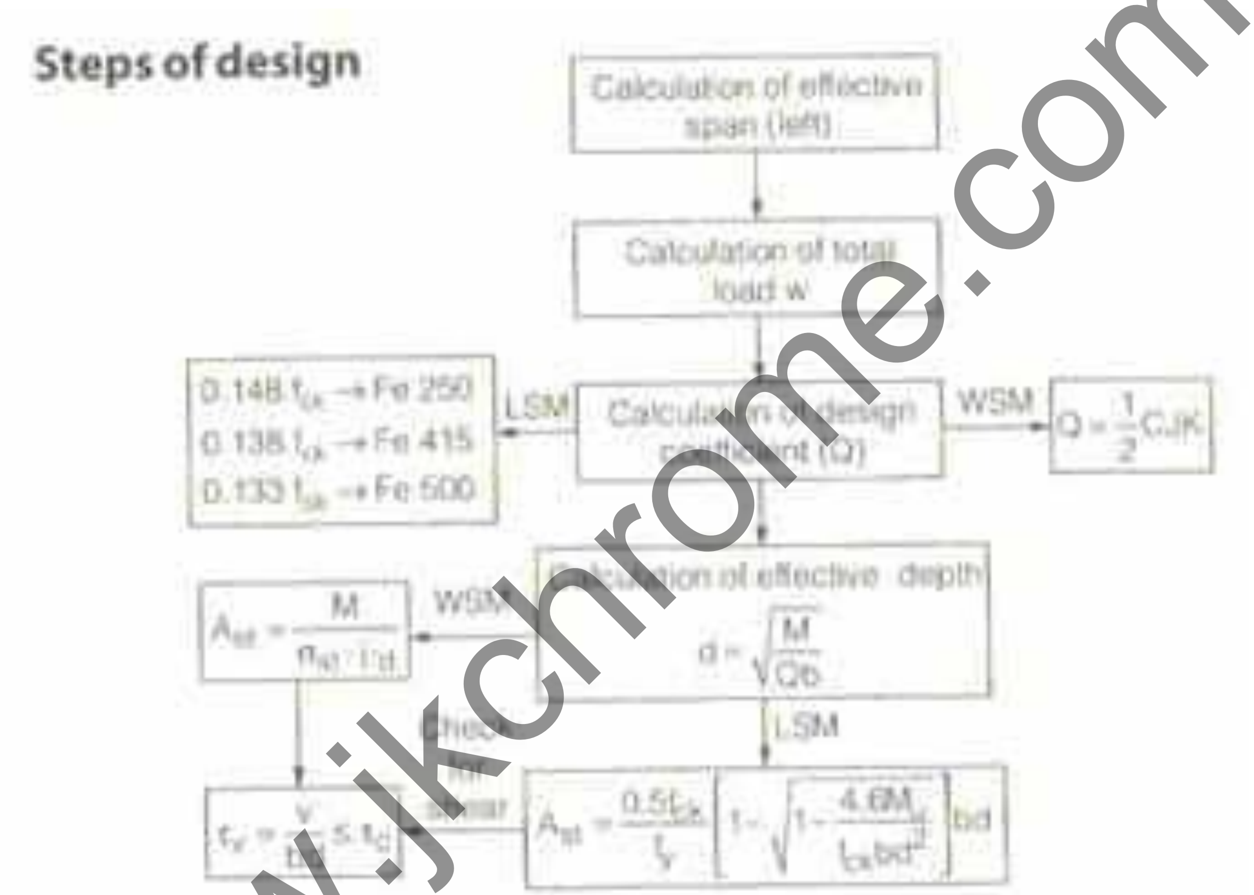
(i) **Ly/Lx ratio** is less than 2

where, l_y = length of longer span

l_x = length of shorter span

(ii) Slab is supported only on two edges.

Design of One way slab



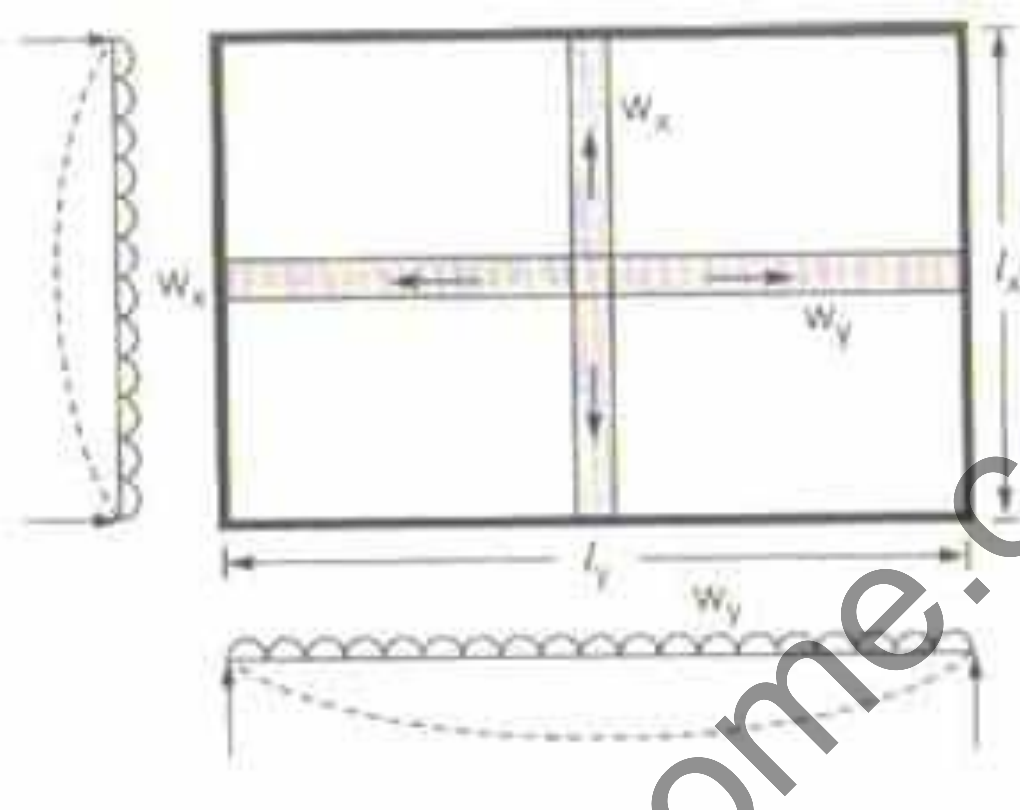
Two Way slab

(i) **Ly/Lx ratio** is more than 2

(ii) Slab is supported on all edges.

Design of two way slab

1. Grasoff Ranking method
   It is used for corners not held down position.
   It is purely simply supported case.



(i)

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

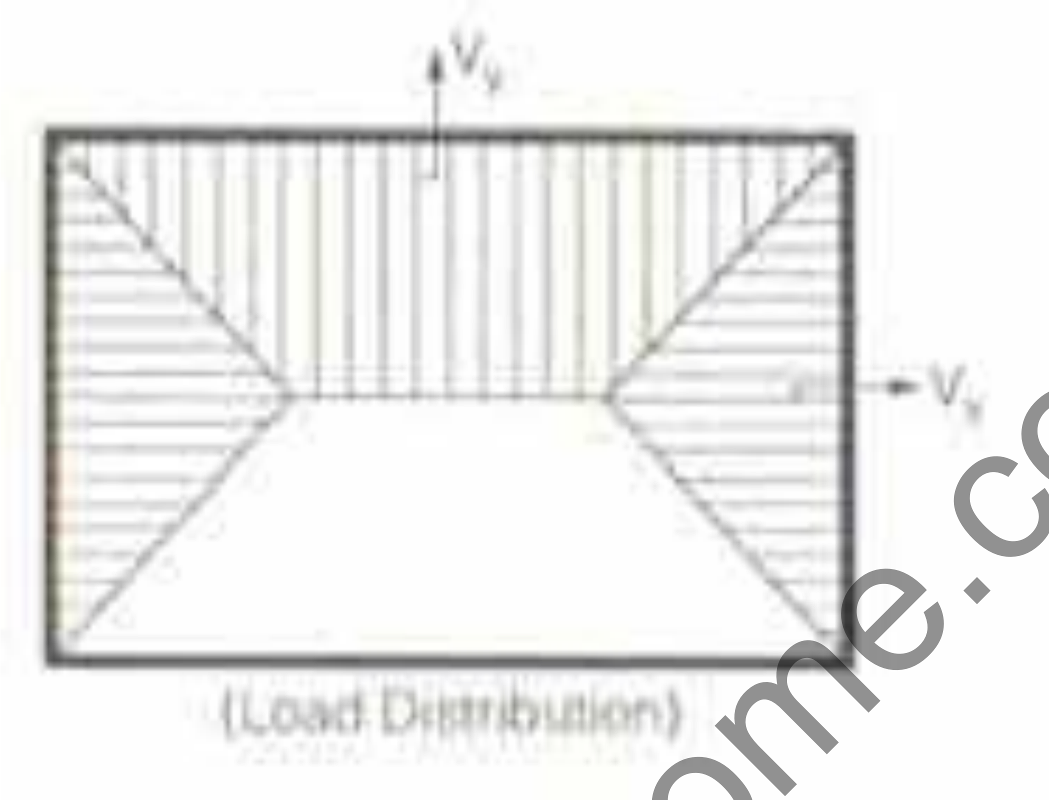
(ii) Moment in x-direction

$$(M_x)M_x = \frac{w_x l_x^2}{8}$$

Moment in y-direction

$$(M_y)M_y = \frac{w_y l_y^2}{8}$$

(iii) Shear force



At shorter edge (V_X)

$$V_x = \frac{1}{3} \cdot wl_x$$

At longer edge (V_y)

$$V_y = \left(\frac{r}{2+r}\right) w \cdot l_x$$

1. **Design of slab with corner held down position**

(a) Pigeauds method: $M_x = r_x^{'} \cdot \frac{wl_x^2}{8} \quad M_y = r_y^{'} \cdot \frac{wl_y^2}{8}$

where, the values of $r_x^{'} \ and \ r_y^{'}$ are read from table

(b) **I.S. code method**

$M_X = \alpha_x w l_y^2 \quad M_y = \alpha_y w l_x^2$

The values of α_x and α_y read from table (page 91, IS : 456-2000)

Design of Column & footings

Design of Column

Working Stress Method

Slenderness ratio (λ)

$$\lambda = \frac{effective\ length}{least\ lateral\ dimension}$$

If $\lambda > 12$ then the column is long.

Load carrying capacity for short column

$$P = \sigma_{sc} A_{sc} + \sigma_{cc} A_c$$

where, A_C = Area of concrete, $A_C = A_g - A_{SC}$

σ_{SC} Stress in compression steel

σ_{CC} Stress in concrete

A_g Total gross cross-sectional area

A_{SC} Area of compression steel

Load carrying capacity for long column

$$P = C_r(\sigma_{SC} A_{SC} + \sigma_{CC} A_C)$$

where,

C_r = Reduction factor

$$C_r = 1.25 - \frac{l_{eff}}{48B}$$

$$or\ \ C_r = 1.25 - \frac{l_{eff}}{160\, i_{min}}$$

where, l_{eff} = Effective length of column

B = Least lateral dimension

i_{min} = Least radius of gyration and

$$i_{min} = \sqrt{\frac{I}{A}}$$

where, I = Moment of inertia and A = Cross-sectional area

Effective length of column

Effective length of Compression Members

Degree of End Restraint of compression members	Symbol	Theoretical value of Effective Length	Recommended value of Effective Length
(i)	(ii)	(iii)	(iv)
Effectively held in position and restrained against rotation in both ends		0.50 *l*	0.65 *l*
Effectively held in position at both ends, restrained against rotation at one end		0.70 *l*	0.80 *l*
Effectively held in position at both ends, but not restrained against rotation		1.00 *l*	1.00 *l*

Degree of End Restraint of compression members	Symbol	Theoretical value of Effective Length	Recommended value of Effective Length
(i)	(ii)	(iii)	(iv)
Effectively held in position and restrained against rotation at one end, and at the other restrained against rotation but not held in position		1.00 *l*	1.20 *l*
Effective held in position and restrained against rotation in one end, and at the other partially restrained against rotation but not held in position		—	1.50 *l*

Degree of End Restraint of compression members	Symbol	Theoretical value of Effective Length	Recommended value of Effective Length
(i)	(ii)	(iii)	(iv)
Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position		2.00 *l*	2.00 *l*
Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end.		2.00 *l*	2.00 *l*

Column with helical reinforcement

Strength of the column is increased by 5%

$P = 1.05(\sigma_{SC} A_{SC} + \sigma_{CC} A_C)$ for short column

$P = 1.05\, C_r (\sigma_{SC} A_{SC} + \sigma_{CC} A_C)$ for long column

Longitudinal reinforcement

(a) Minimum area of steel = 0.8% of the gross area of column

(b) Maximum area of steel

(i) When bars are not lapped A_{max} = 6% of the gross area of column

(ii) When bars are lapped A_{max} = 4% of the gross area of column

Minimum number of bars for reinforcement

For rectangular column 4

For circular column 6

Minimum diameter of bar = 12 mm

Maximum distance between longitudinal bar = 300 mm

Pedestal: It is a short length whose effective length is not more than 3 times of lest lateral dimension.

Transverse reinforcement (Ties)

$$\phi = \max imum \begin{cases} \frac{1}{4} \cdot \phi_{main} \\ 6\,mm \end{cases}$$

where $\phi_{main} =$ dia of main logitudnal bar

φ = dia of bar for transverse reinforcement

Pitch (p)

$$\phi = \text{minimum} \begin{cases} \text{least lateral dimension} \\ 16\,\phi_{\min} \\ 300\,mm \end{cases}$$

where, φ_{min} = minimum dia of main longitudinal bar

Helical reinforcement

(i) Diameters of helical reinforcement is selected such that

$$0.36\left[\frac{A_g}{A_C}-1\right]\frac{f_{ck}}{f_y}\le\frac{V_h}{V_C}$$

(ii) Pitch of helical reinforcement: (p)

$(a)\ p \ngtr 75\,mm$

$(b)\ p \ngtr \frac{1}{6}dc$

$(c)\ p \nless 3\phi_h$

$(d)\ p \nless 25\,mm$

where,

d_C = Core diameter = d_g – 2 × clear cover to helical reinforcement

A_G = Gross area

$$=\frac{\pi}{4}(d_g)^2$$

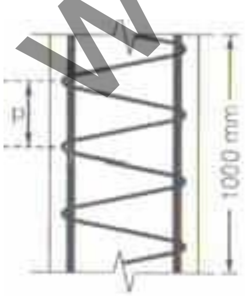
d_g = Gross diameter

V_h = Volume of helical reinforcement in unit length of column

φ_h = Diameter of steel bar forming the helix

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

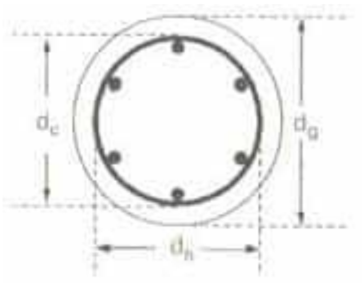
$$A_C=\frac{\pi}{4}(d_C)^2\quad V_C=A_C\times 1$$



d_h = centre to centre dia of helix

= d_g – 2 clear cover - φ_h

φ_h = diameter of the steel bar forming the helix



Some others IS recommendations

(a) Slenderness limit

(i) Unsupported length between end restrains 60 times least lateral dimension.

(ii) If in any given plane one end of column is unrestrained than its unsupported length

$$\not> \frac{100B^2}{D}$$

(b) All column should be designed for a minimum eccentricity of

$$e_{\min} = \text{maximum}\begin{cases} \dfrac{l}{500} + \dfrac{'B'\, or\, 'D'}{30} \\ 20\ mm \end{cases}$$

Limit state method

1. Slenderness ratio (λ)

$$\lambda = \frac{\text{effective length}}{\text{least lateral dimension}}$$

if

λ<12 Short column

1. Eccentricity

$$e_{min} = \text{maximum}\begin{cases} \dfrac{l}{500} + \dfrac{B\,or\,D}{20} \\ 30\,mm \end{cases}$$

If $e_{min} \leq 0.05D$ then it is a short axially loaded column.
where, P_u = axial load on the column
2. Short axially loaded column with helical reinforcement
$P_u = 0.4 f_{ck} A_C + 0.67 f_y A_{SC}$
3. Some others IS code Recommendations
$P_u = 1.05(0.4 f_{ck} A_C + 0.67 f_y A_{SC})$

(a) Slenderness limit

(i) Unsupported length between end restrains $\not>$ 60 times least lateral dimension.

(ii) If in any given plane one end of column is unrestrained than its unsupported length $\not> \dfrac{100B^2}{D}$

(b) All column should be designed for a minimum eccentricity of

$$e_{min} = \text{maximum}\begin{cases} \dfrac{1}{500} + \dfrac{B}{30} \\ 20\text{ mm} \end{cases}$$

Concentrically Loaded Columns

Where e = 0, i.e., the column is truly axially loaded.

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

B. Footing/Foundation

INTRODUCTION

Foundation is important part of any superstructure. It transfers load from superstructure to soil.

Bearing capacity of soil

Bearing capacity of soil governs the dimensions and depth of foundation. Under no case the loading on foundation can be greater than bearing capacity of foundation

(A) Gross Bearing capacity: Total bearing capacity at based on foundation which includes weight of foundation, super structure load, earth lying over footing.

(B) Net Bearing capacity: It can be defined as follows

Net bearing capacity= Gross Bearing capacity - W

W= weight of soil at level of footing before trench was made for footing

Depth of foundation

following formula must be use to find the depth of foundation

$$H = \frac{p}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2$$

p = Bearing capacity of foundation

γ = Density of soil

ϕ = angle of repose

H= Depth of foundation

Types of Foundation

Based on depth it can be into two parts

(i) **Shallow foundation**- if total depth (D) of footing is less than width (B) of foundation then foundation is called shallow foundation.

(ii) **Deep foundation-** If total depth (D) is less than width (B) of foundation than foundation is called deep foundation.

Nominal cover as per IS 456:200

Minimum Nominal cover as per exposure condition

Member	**Mild (mm)**	**Moderate (mm)**	**Severe (mm)**	**Very severe (mm)**	**Extreme (mm)**
Foundation	40	50	50	50	75

DESIGN OF FOOTING

Let's take an example to understand important aspect of footing

Example: Design a square footing using LSM for a column load of 1000 kN. If bearing capacity, density of soil is 150 KN/m^2 and 20 kN/m^3. Use M30 /Fe 415.

Dimension of column= 500 ×500 mm

Sol.

Axial load P_1 = 1000 kN

Weight of footing $P_2 = 0.20 \times P_1$ = 200 kN

Note = P_2 can be assumed to 10 to 20% of

Total load = P_T = 1200 kN

(i) Area of footing required

Area = P_T/q_0

q_0 = bearing capacity of soil

Area = 8 m^2

Assume 4 m × 4 m square footing is provided

Area provided = 16 m^2 > Area required

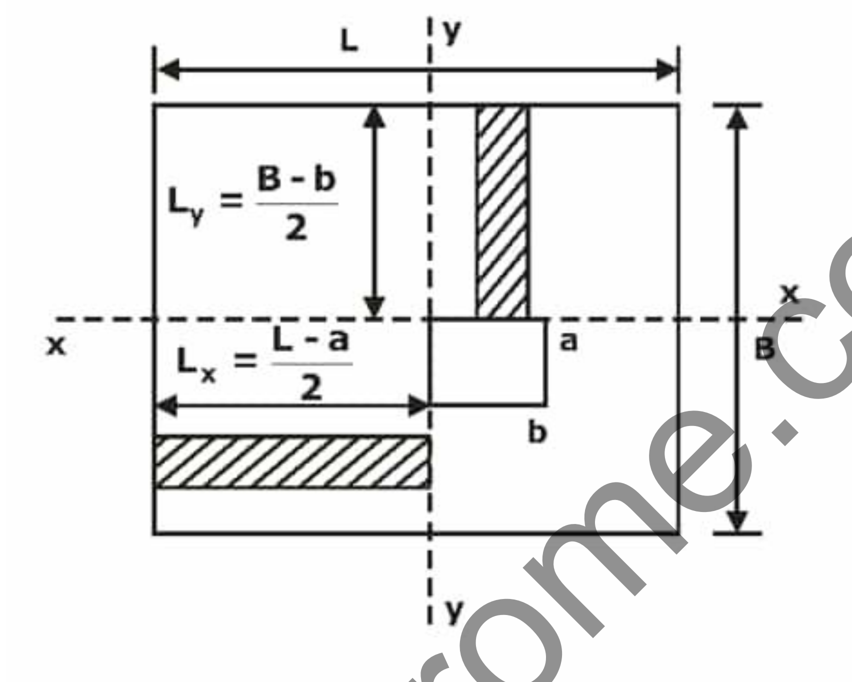
(ii) Net soil pressure

$w_0 = P_1 / A$ = 62.5 kN/m^2

For LSM design $w_{40} = 1.5 \times w_0$ = 93.75 kN/m^2

1.5 → Partial factor of safety

(iii) Check for bending moment



As per our design

a = b = 500 mm

L = B = 4 m

{In case of rectangular footing a, b, L, B}

will not be same

for 1 m width → about xx

$$M_{u_x} = W_{u_c} \times 1 \times \left(\frac{B-b}{L}\right)\left(\frac{B-b}{4}\right)$$

$$= W_{u_0} \times \frac{1}{8} \times (B-b)^2$$

$$M_{ux} = \frac{93.75 \times 1}{8}(4-0.5)^2$$

= 143.55 kNm

Moment about y-y

For 1 m width

$$M_{uy} = W_{40} \times 1 \times \left(\frac{L-d}{2}\right)\left(\frac{L-a}{4}\right)$$

$$M_{uy} = W_{40} \times 1 \, \frac{(L-a)^2}{8}$$

$$M_{uy} = 93.75 \times \frac{(4-0.5)^2}{8} = 143.55 \text{ kNm}$$

$$\cong 143.6 \text{ kNm}$$

In case M_{ux} and M_{uy} comes differently then take maximum value

(iv) depth of footing

$$\text{depth}(d) = \sqrt{\frac{M_{u\max}}{QB}}$$

For 1 m width

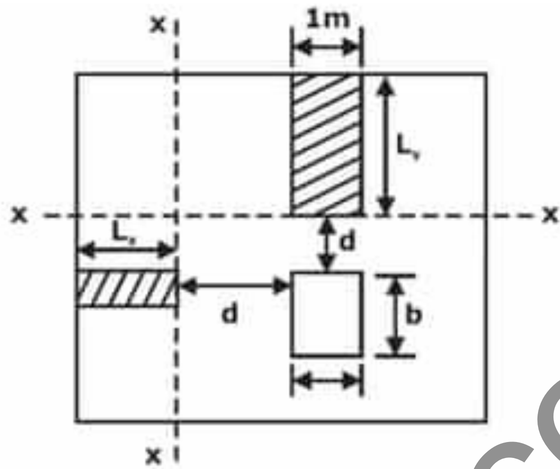
$$d = \sqrt{\frac{1436 \times 10^6}{0.138 \times 30 \times 100}} = 186 \text{ mm}$$

Assume effective depth 250 mm and overall depth = 300 m

Note – Critical section for bending moment is at face of column

(v) Check for shear

Critical section for one way shear is at Q from face of column



$$L_y = \left(\frac{B-b}{2}\right) - d$$

$$L_x = \left(\frac{L-u}{2}\right) - d$$

$V_{ux} = W_{40} \times L_y$ and $V_{uy} = W_{40} \times L_x$

Nominal shear $\tau_V = \frac{V_{max}}{Bb} < \tau_c$

τ_c = permissible shear stress

$\tau_c = 0.29 \rightarrow$ for M30

$V_u = 93.75 \times 10^3 \times 1\times[(4\text{-}0.5)/2\text{-}0.3]$=

= 13594 kN

For B = 1m

$\tau_V = V_u/Bd = 0.54$

$\tau_V > \tau_c \rightarrow$ Not safe in shear increasing depth (G) of footing to 600 mm

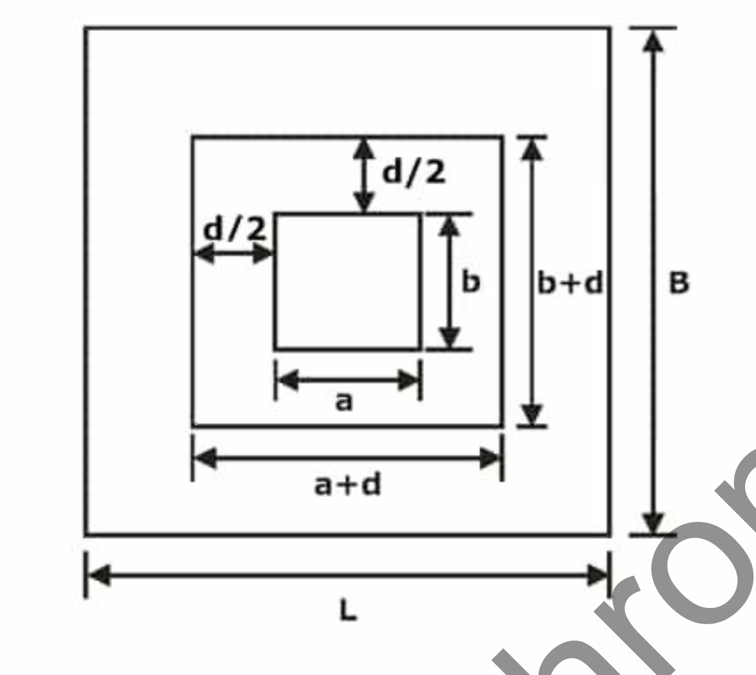
$V_u = 93.75 \times 10^3 \times 1\times[(4\text{-}0.5)/2\text{-}0.6]$=

= 107.81 kN

$\tau_V = 0.17 < \tau_c \rightarrow$ safe now so new depth of footing = 600 mm

overall depth of footing = 660 mm

(vi) Check for punching shear / two way shear



Note →

Critical section for punching shear is at d/2 for five of column all-around

$P_{net} = P_u - W_u\ (a + d)\ (b + d)$

$P_{net} = 1.5 \times P - W_u\ (a + d)\ (b + d)$

$$\tau_v = \frac{P - (W_u (a+d)(b+d))}{2[(a+d)+(b+d)] \times d}$$

$$\text{punching shear stress} = \frac{\text{Net punching force}}{\text{Cross - sec tion area of resisting section}}$$

d = 0.6

$$\tau_{vp} = \frac{1.5 \times 1000 \times 10^3 - 93.75(0.5+0.6)(0.5+0.6) \times 10^3}{2\left[(0.5+0.6)+(0.5+0.6)\right] \times d}$$

$= 0.49$ N/mm^2

$$\tau_{cp} = k_s \times 0.25\sqrt{f_{ck}}$$

$k_s = 0.5 + \beta_c$

$= 0.5 + (b/a)$

$= 0.5 + 1 = 1.5$

maximum possible value of k_s =1 so

$k_s = 1$

$\tau_{cp} = 1.36$ N/mm^2 > τ_{vp} → safe

so d = 600 mm

D = 660 mm

(v) Area of steel

$$A_{st} = \frac{0.5\, f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\, M_u}{f_{ck}\, Bd^2}}\right] \times Bd$$

For 1000 mm width

$A_{st} = 560$ mm^2

$$A_{st\,min} = \frac{0.12}{100} \times BD$$

$= \frac{0.12}{100} \times 1000 \times 660 = 72$ mm^2 for 1m width

(3) Total area of steel

$= L \times A_{st} = 4 \times 560 = 2240$ mm^2

Total no of bars for 16 mm ϕ

n= 2240/[(π/4)×16^2]=12

(4) number of central bend

$$n_c = n_T \frac{2}{\left(1+\frac{L}{B}\right)}$$

$$= \frac{12 \times 2}{(1+1)} = 12$$

$\therefore \ \ell = B$ for width all value remains same

$\therefore$ for width all value remains same

(5) check for bearing

$$F_b = \frac{P_u}{a \times b} = \frac{1.5 \times 1000 \times 10^3}{500 \times 500}$$

$$= 6 \text{ N/mm}^2$$

$$F_b < 0.45\, f_{ck} \sqrt{\frac{A_1}{A_2}} \rightarrow \text{safe}$$

Prestressed Concrete

Introduction

- Prestress is defined as a method of applying pre-compression to control the stresses resulting due to external loads below the neutral axis of the beam tension developed due to an external load which is more than the permissible limits of the plain concrete.
- Prestressed concrete is basically concrete in which internal stresses of a suitable magnitude and distribution are introduced so that the stresses resulting from the external loads are counteracted to a desired degree.

Terminology

1. **Tendon**: A stretched element used in a concrete member of structure to impart prestress to the concrete.
2. **Anchorage**: A device generally used to enable the tendon to impart and maintain prestress in concrete.
3. **Pre tensioning**: A method of prestressing concrete in which the tendons are tensioned before the concrete is placed. In this method, the concrete is introduced by the bond between
steel & concrete.
4. **Post-tensioning**: A method of prestressing concrete by tensioning the tendons against hardened concrete. In this method, the prestress is imparted to concrete by bearing

Materials for prestressing concrete members

1. **Cement**: The cement used should be any of the following

(a) Ordinary Portland cement conforming to IS269
(b) Portland slag cement conforming to IS455. But the slag content should not be more than 50%.
(c) Rapid hardening Portland cement conforming to IS8041.
(d) High strength ordinary Portland cement conforming to IS8112.

2. **Concrete**: Prestress concrete requires concrete, which has a high compressive strength reasonably early age with comparatively higher tensile strength than ordinary concrete. The concrete for the members shall be air-entrained concrete composed of Portland cement, fine and coarse aggregates, admixtures and water. The air-entraining feature may be obtained by the use of either air-entraining Portland cement or an approved air-entraining admixture. The entrained air content shall be not less than 4 per cent or more than 6 per cent.

- Minimum cement content of 300 to 360 kg/m3 is prescribed for the durability requirement.
- The water content should be as low as possible.

3. **Steel**:- High tensile steel, tendons, strands or cables

High strength steel should contain:

- 0.7 to 0.8% carbons,
- 0.6% manganese,
- 0.1% silica

Why high grade of concrete & steel?

- Higher the grade of concrete higher the bond strength which is vital in pre-tensioned concrete
- Also, higher bearing strength which is essential in post-tensioned concrete as well as in pre-tensioned concrete
- Creep & shrinkage losses are minimum with high-grade concrete.
- Generally, minimum M30 grade concrete is used for post-tensioned & M40 grade concrete is used for pre-tensioned members
- The losses in pre-stress members due to various reasons are generally in the range of 250 N/mm^2 to 400 N/mm^2
- If mild steel or deformed steel is used the residual stresses after losses is either zero or negligible

Advantage of Prestressed Concrete

- The use of high strength concrete and steel in prestressed members results in lighter and slender members than is possible with RC members.
- In fully prestressed members the member is free from tensile stresses under working loads, thus the whole of the section is effective.
- In prestressed members, dead loads may be counter-balanced by eccentric prestressing.
- Prestressed concrete member posses better resistance to shear forces due to effectof compressive stresses presence or eccentric cable profile.
- Use of high strength concrete and freedom from cracks, contribute to improving durability under aggressive environmental conditions.
- Long span structures are possible so that saving in weight is significant & thus it will be economic.
- Factory products are possible.
- Prestressed members are tested before use.
- Prestressed concrete structure deflects appreciably before ultimate failure, thus giving ample warning before the collapse.
- Fatigue strength is better due to small variations in prestressing steel, recommended to dynamically loaded structures.

Disadvantages of Prestressed Concrete

- The availability of experienced builders is scanty.
- Initial equipment cost is very high.
- Availability of experienced engineers is scanty.
- Prestressed sections are brittle
- Prestressed concrete sections are less fire resistant.

Classifications and Types

Prestressed concrete structures can be classified in a number of ways depending upon the feature of designs and constructions.

1. **Pre-tensioning**: In which the tendons are tensioned before the concrete is placed, tendons are temporarily anchored and tensioned and the prestress is transferred to the concrete after it is hardened.
2. **Post-tensioning**: In which the tendon is tensioned after the concrete has hardened. Tendons are placed in sheathing at suitable places in the member before casting and later after hardening of concrete.

Tensioning Devices

- **Mechanical devices**: The mechanical devices generally used include weights with or without lever transmission, geared transmission in conjunction with pulley blocks, screw jacks with or without gear devices and wire-winding machines.
- **Hydraulic devices**: These are simplest means for producing large prestressing force, extensively used as tensioning devices
- **Electrical devices**: The wires are electrically heated and anchored before placing concrete in the mould. This method is often referred to as thermo-prestressing and used for tensioning of steel wires and deformed bars.
- **Chemical devices**: Expanding cement are used and the degree of expansion is controlled by varying the curing condition. Since the expansive action of cement, while the setting is restrained, it induces tensile forces in tendons and compressive stresses in concrete.

Prestressing System

1. **Pretensioning system:** In the pre-tensioning systems, the tendons are first tensioned between rigid anchor-blocks cast on the ground or in a column or unit –mould types pre-tensioning bed, prior to the casting of concrete in the mould. The tendons comprising individual wires or strands are stretched with constant eccentricity or a variable eccentricity with tendon anchorage at one end and jacks at the other.

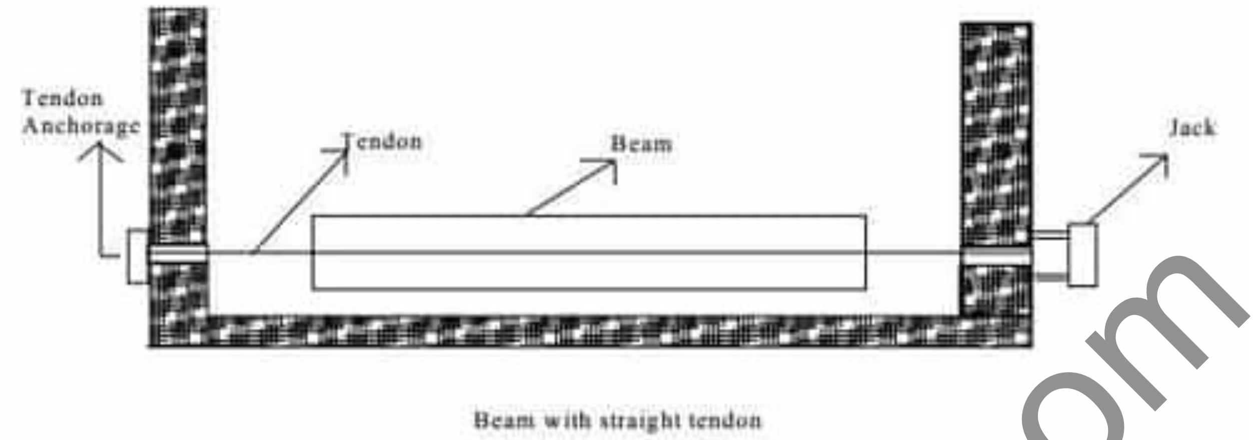


Fig. 1 Prestressing system

2. Post-tensioned system: In post-tensioning, the concrete unit is first cast by incorporating ducts or grooves to house the tendons. When the concrete attains sufficient strength, the high-tensile wires are tensioned by means of jack bearing on the end of the face of the member and anchored by wedge or nuts.

Most of the commercially patented prestressing systems are based on the following principle of anchoring the tendons:

- Wedge action producing a frictional grip on the wire.
- Direct bearing from the rivet or bolt heads formed at the end of the wire.
- Looping the wire around the concrete.

Methods:

1. Freyssinet system
2. Gifford-Udall system
3. Magnel blaton system
4. Lee-McCall system

Differences of Prestressed Concrete Over Reinforced Concrete:

- In prestressed concrete member steel plays an active role. The stress in steel prevails whether the external load is there or not. But in R.C.C., steel plays a passive role. The stress in steel in R.C.C members depends upon the external loads. i.e., no external load, no stress in steel.

- In prestress concrete the stresses in steel is almost constant where as in R.C.C the stress in steel is variable with the lever arm.
- Prestress concrete has more shear resistance, where as the shear resistance of R.C.C is less.
- In prestress concrete members, deflections are less because the eccentric pre stressing force will induce couple which will cause upward deflections, where as in R.C.C., deflections are more.
- In prestress concrete fatigue resistance is more compared to R.C.C. because in R.C.C. stress in steel is external load dependent where as in P.S.C member it is load independent.
- Prestress concrete is more durable as high grade of concrete is used which is denser in nature. R.C.C. is less durable.
- In prestress, concrete dimensions are less because external stresses are counterbalanced by the internal stress induced by prestressing. Therefore reactions on column & footing are less
  as a whole the quantity of concrete is reduced by 30% and steel reduced by about 60 t 70%. R.C.C. is uneconomical for long span because in R.C.C. dimension of sections are large requiring more concrete & steel.

Comparative Study: Pretension Vs Post-tensioned Member

Pretension member	**Post-tensioned member**
In pre-tensioned prestress concrete, steel is tensioned prior to that of concrete. It is released once the concrete is placed and hardened. The stresses are transferred all along the wire by means of the bond.	Concreting is done first then wires are tensioned and anchored at ends. The stress transfer is by end bearing not by the bond.
Suitable for short span and precast products like sleepers, electric poles on mass production.	Suitable for long span bridges
In pre-tensioning, the cables are basically straight and horizontal. Placing them in curved or inclined position is difficult. However, the wire's can be kept with eccentrically. Since cables can not be aligned similarly to B.M.D. structural advantages are less compared to that of post-tensioned.	The post-tensioning cables can be aligned in any manner to suit the B.M.D due to external load system. Therefore it is more economical, particularly for long span bridges. The curved or inclined cables can have a vertical component at ends. These components will reduce the design shear force. Hence post-tensioned beams are superior to pre-tensioned beams both from flexural and shear resistances point
Prestress losses are more compared to that of post-tensioned concrete.	Losses are less compare to pre-tensioned concrete

Analysis of Prestress Member

Basic Assumptions

1. Concrete is a homogeneous elastic material.
2. Within the range of working stress, both concrete & steel behave elastically, notwithstanding the small amount of creep, which occurs in both the materials under the sustained loading.
3. A plane section before bending is assumed to remain plane even after bending, which implies a linear strain distribution across the depth of the member.

- Prestress Concrete is one in which there have been introduced internal stresses of such magnitude and distribution that stresses resulting from given external loading is counter balanced to a desired degree.

Analysis of prestress and Bending stress

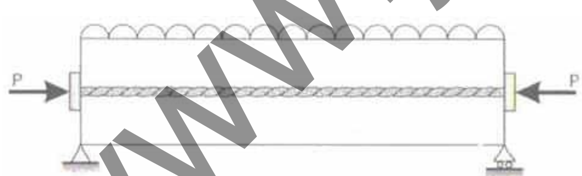
Following are the three concepts of analysis

1. Stress concept analysis
2. Strength concept analysis
3. Load balancing method

Stress concept Method-

Following are the two cases for analysis

Case-(i) Beam provided with a concentric tendon:

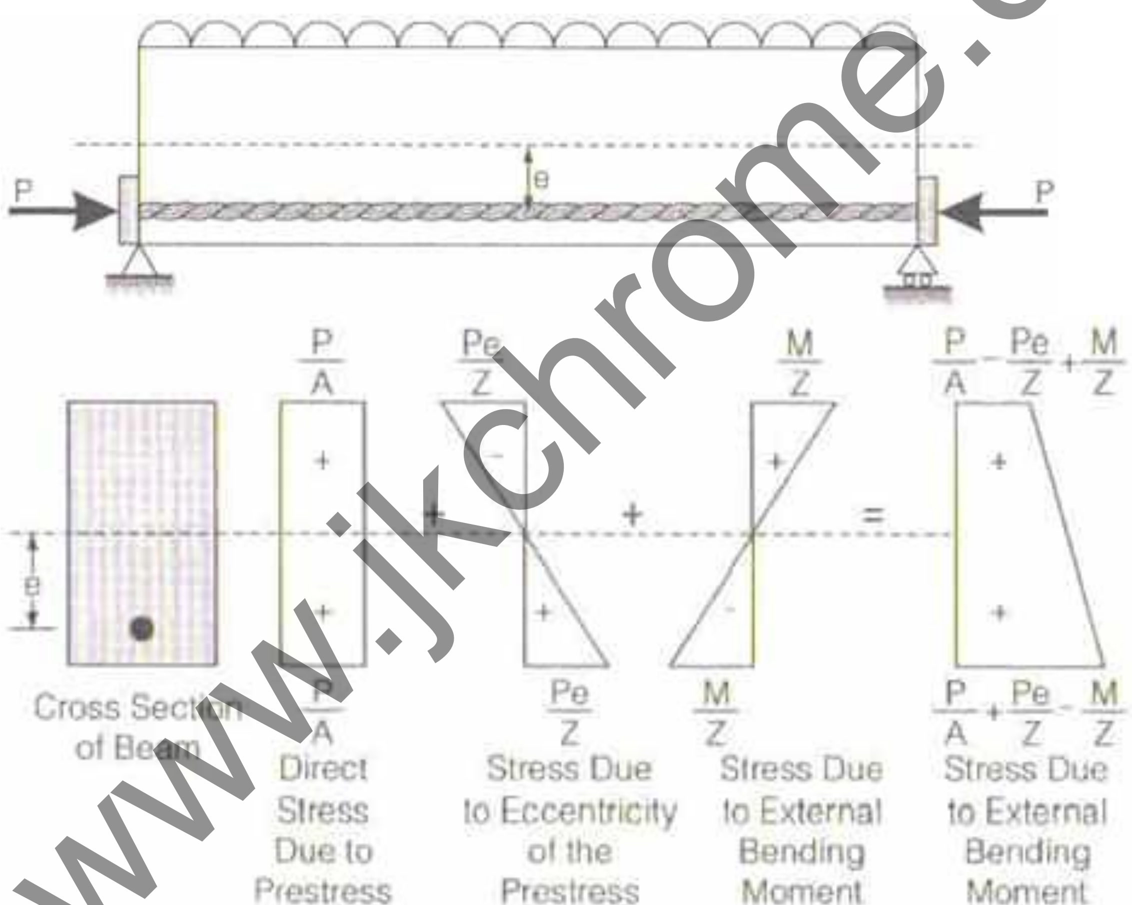


Let, P prestressing force applied by the tendon. Due to this prestressing force, the direct compressive force induced is given by, $f_a = \frac{P}{A}$.

If due to dead load & external loads, the bending moment at the section is M, then the extreme stresses at the section due to bending moment alone is $f_0 = \pm \frac{M}{Z}$

Hence final stress at the extreme top edge $= \frac{P}{A} + \frac{M}{Z}$ and stress at the extreme bottom edge $= \frac{P}{A} - \frac{M}{Z}$

Case–(ii): Beams with eccentrics tendon:



(i) Direct stresses due to prestressing force $= +\frac{P}{A}$

(ii) Extreme stresses due to an eccentricity of the prestressing force $= \mp \dfrac{P.e}{Z}$

(iii) Extreme stresses due to bending moment $= \pm \dfrac{M}{Z}$

Final stresses

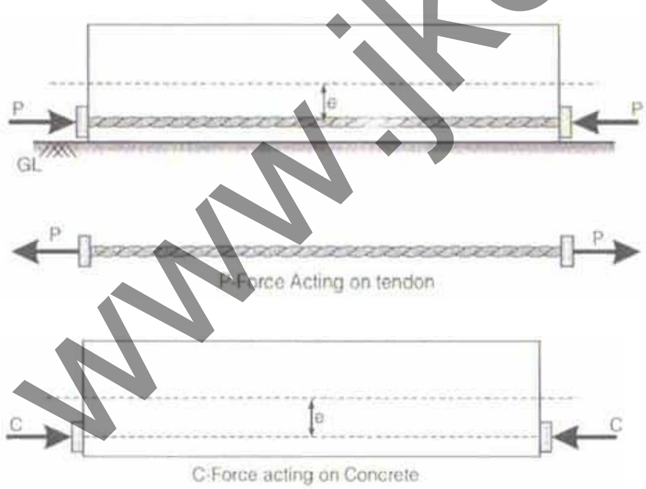
Stress at top fibre $= \dfrac{P}{A} - \dfrac{P.e}{Z} + \dfrac{M}{Z}$

Stress at bottom fibre $= \dfrac{P}{A} + \dfrac{P.e}{Z} - \dfrac{M}{Z}$

By providing an eccentricity to the tendon, a hogging moment (P.e.) is developed which will produce stresses, which will counteract the stresses due to external bending moment.

Strength Concept method-

Consider a beam of length l provided with a tendon at an eccentricity e. Suppose the beam is lying on the ground i.e. the beam is not subjected to any external load. Hence there is no external bending moment on the beam.



The following equal forces are existing

(i) The P-force which is the tension in the tendon.

(ii) The C-force which is the compressive force acting on the concrete.

Stresses in concrete are produced entirely due to C-force.

In the absence of any external bending moment the C-force and P-force act at the same level. Line of action of P-force is called the P-line. The P-line is nothing but the tendon line itself. The line of action of the C-force is called the C-line or Pressure line. Hence in the absence of any external bending moment the P-line and the C-line coincide.

Suppose the beam is subjected to a bending moment M, then the C-line will be shifted from the P-line by a distance 'a' called lever arm.

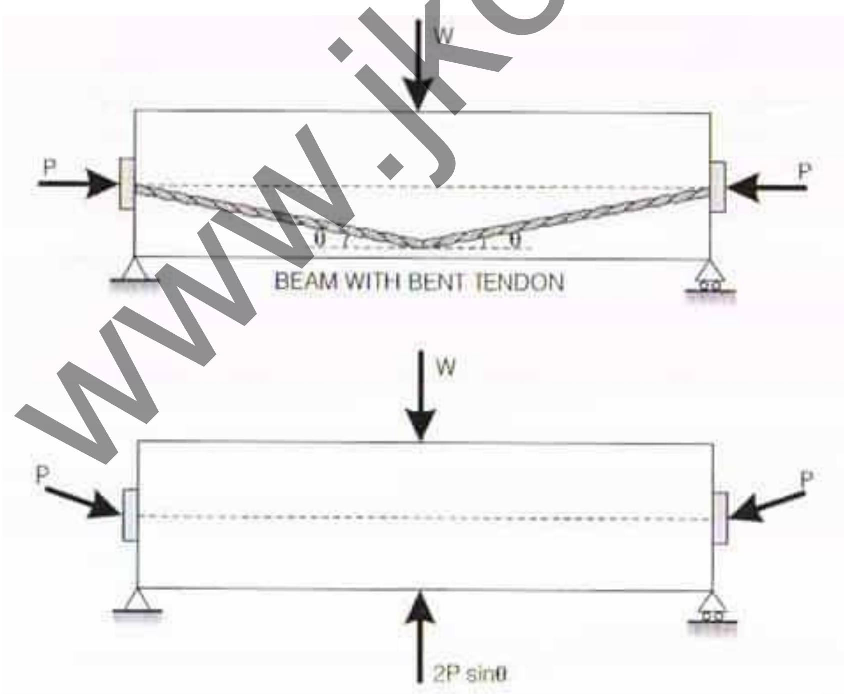
$$a = \frac{M}{P} = \frac{M}{C}$$

Extreme stresses in concrete are given by

$$= \frac{C}{A} \pm \frac{C \times eccentricity\, of\, C}{Z}$$

Load Balancing Concept-

Prestressed Beam with Bent Tendon



By providing bent fendons, the tendons will exert an upward pressure on the concrete beam and will therefore counter act a part of the external downward loading.

Considering the concrete as a free body. We find an upward force 2P sin θ.

The net downward load at the centre will be (W-2P sinθ)

The axial longitudinal force provided by the tendon = $P\cos\theta$ = P {since θ is small}

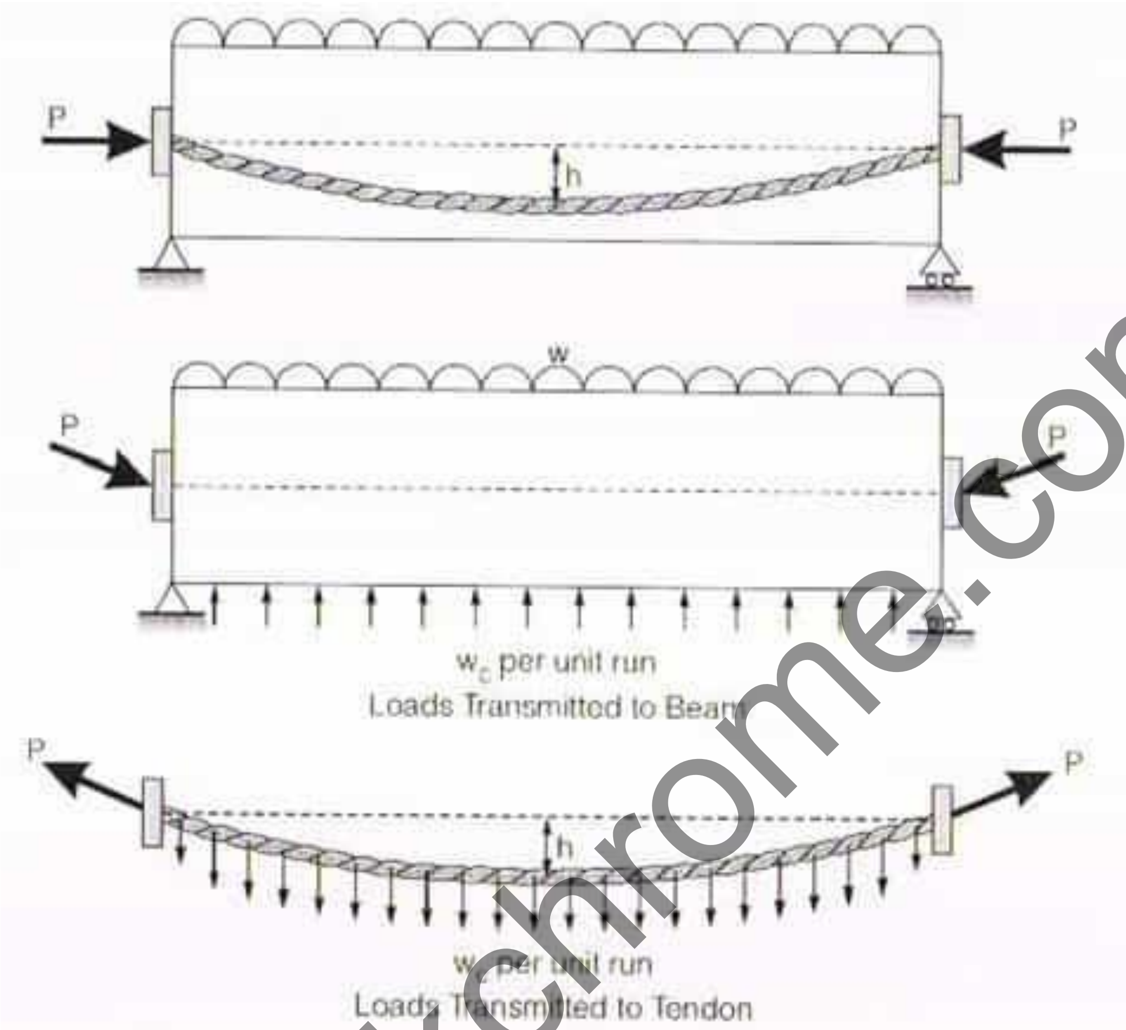
Direct stress on the section $= \frac{P\cos\theta}{A} = \frac{P}{A}$

Net BM, $M = \frac{(W - 2P\sin\theta)l}{4} + \frac{wl^2}{8}$

Where, w = dead load per unit length of the beam. Extreme fibre stress $= \frac{P}{A} \pm \frac{M}{Z}$

It may be realized that the profile of the tendon should follow the shape of the bending moment diagram for the given external loads in order it may offer considerable and effective upward forces. For e.g., if the loading on the beam is a uniformly distributed load, the tendon may be provided along a parabolic profile.

Tendon with Parabolic Profile



Let *l* be the span of the beam and h be the dip of the cable.

The cable will exert an upward udl = w_C/m on the beam, but the cable will be subjected to downward udl of w_C per unit run.

Let V and H are vertical and horizontal components of P.

$$V = \frac{w_C\, l}{2}$$

The cable is an absolutely flexible member, therefore BM at every section of cable is zero. Hence BM at the centre of the cable is

$$\frac{w_C.l}{2} \times \frac{l}{2} - w_C.\frac{l}{2} \cdot \frac{l}{4} - H \cdot h = 0$$

$$H = \frac{w_C l^2}{8h}$$

Since dip of the cable is very small, we can make approximation

cos*α* = 1 and *P*cos*α = P*

Now consider the beam, it is subjected to

(i) External load w per unit length

(ii) Upward udl transmitted by the cable = w_C per unit length.

Net UDL = w – w_C

Net BM at the centre $= \frac{(w - w_C)}{8} l^2$

Extreme stresses $= \frac{P}{A} \pm \frac{Net\ BM}{Z}$

Losses of Prestress

The steel wires of a prestressed concrete member do not retain all the preliminary prestress. A certain amount of loss of prestress always takes place.

Losses may be classified as follows:

Types of Losses	Losses in pretensioned member	Losses in posttensioned member
1. Loss of prestress during tensioning process due to friction. (a) Loss due to length effect (b) Loss due to curvature effect (c) Loss due to both length and curvature effect	No Loss No Loss No Loss	P_0kx $P_0\mu\alpha$ $P_0(kx+\mu\alpha)$ Here, P_0 = Prestressing force at the jacking end K = Wobble friction factor 15×10^{-4} per meter < K < 50×10^{-4} per meter. α = Cumulative angle in radians through which tangent to the cable profile has turned between any two points under consideration. μ = Coefficient of friction in curves = 0.25 to 0.55

Types of Losses	Losses in pretensioned member	Losses in posttensioned member
2. Loss of prestress at the anchoring stage	No Loss	$\frac{\Delta l}{l} \cdot E_S$ Here, Δl = effective slip of the wire. l = Length of the tendon. E_S = Young's modulus for tendon wires

Types of Losses	Losses in pretensioned member	Losses in posttensioned member
3. Loss of prestress occurring Sub-sequently		
(a) Loss of stress due to shrinkage of concrete	$(3\times10^{-4})E_s$ Here, E_S = Youngs modulus for tendon wire	$\frac{2\times10^4}{\log_{10}(T+2)} \cdot E_s$ Here T = Age of concrete at the time of transfer of stress (in days)
(b) Loss of stress due to creep to concrete	$\phi \cdot m \cdot f_c$ Here, m = Modular ratio $= E_S/E_C$ f_C = Original prestress in concrete at the level of steel	$\phi \cdot m \cdot f_c$ zero it all the bars are tensioned at same time

(c) Loss of stress due to elastic shortening of concrete	$m \cdot f_c$ Here f_c = Initial stress in concrete at the level of steel.	$\frac{\Delta l}{l} \cdot E_s$ for subsequent tensioning
(d) Loss of stress due to creep of steel or loss due to stress relaxations.	1 to 5% of initial prestress.	1 to 5% of initial prestress