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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: DESIGN OF REINFORCED CONCRETE

STRUCTURES

SUBJECT CODE: 20CE5PC02

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SYLLABUS:

UNIT – I

Introduction- Structure - Components of structure - Different types of structures - Equilibrium and compatibility– Safety and Stability - Loads – Different types of Loads – Dead Load, Live Load, Earthquake Load and Wind Load– Forces – What is meant by Design? – Different types of materials – RCC, PSC and Steel – Planning of structural elements- Concepts of RCC Design – Different methods of Design- Working Stress Method and Limit State Method – Load combinations as per Limit state method – Materials – Characteristic Values – Partial safety factors – Behavior and Properties of Concrete and Steel- Stress Block Parameters as per IS 456 -2000.

Limit state Analysis and design of sections in Flexure – Behavior of RC section under flexure - Rectangular, T and L-sections, singly reinforced and doubly reinforced Beams – Detailing of reinforcement.

DESIGN OF REINFORCED CONCRETE STRUCTURES
BTECH III-I
UNIT I

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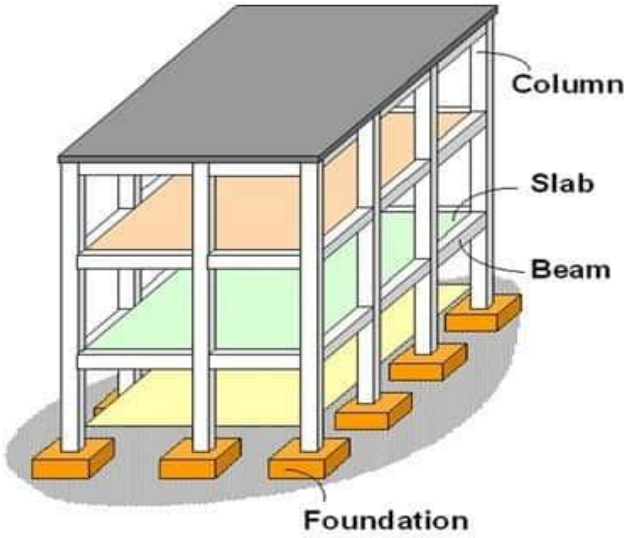
Design of Reinforced Concrete Structure

RCC Reinforced concrete structure :

Concrete : (Mixture of Cement, Sand, and coarse aggregate and water)

Reinforcement : Steel (HYSD Bars)

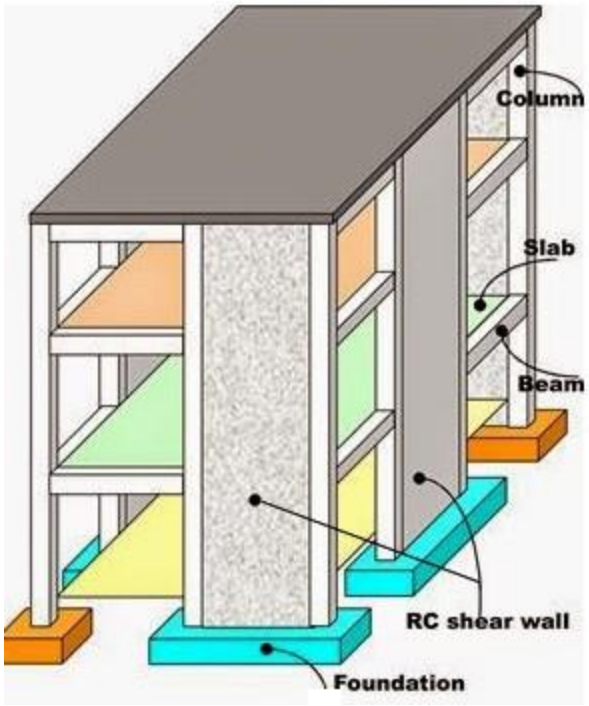




Typical RC Frame Building

Main elements/components of RCC Structure

- SLABS
- BEAMS
- COLUMNS
- FOOTING
- STAIRCASE
- SHEAR WALLS



Slab : A reinforced concrete slab is a crucial structural element and is used to provide flat surfaces(floors and ceilings) in buildings. *Slab transfer the floor load to the beams.*

Beams : RCC beams are structural elements are designed to carry transverse external loads that cause bending moment, shear forces and in some cases torsion across their length.

Columns : A vertical member carrying compressive load is called as column. Column is a very important structural element as it transfer load from slab/beam to footing.

Footing: The bottom most part of a vertical structure (column, wall) which ultimately transfer the weight from walls and columns to the soil or bedrock. Footing is basically part of foundation of any modern structure.

A **stairway**, **staircase**, **stairwell**, **flight of stairs**, or simply **stairs,** is a construction designed to bridge a large **vertical** distance by dividing it into smaller vertical distances, called **steps**. Stairs may be straight, round, or may consist of two or more straight pieces connected at angles.

Concept of Reinforced Concrete Design

The design should ensure :

- Adequate safety in terms of strength and stability.
- Adequate serviceability in terms of stiffness and durability, and
- Reasonable economy.

Types of Structures

1) RCC : Reinforced Concrete Structures
2) PSC : Prestressed concrete structures
3) PTS : Pre/Post Tensioning Structures
4) Steel Structures

Important IS Codes for RCC Structures

IS 456 : Code of practice for plain and reinforced concrete.

IS 875 :

Part – I : For Dead Loads
Part – II : For Live Loads
Part – III : For Wind Loads
Part - IV : For Snow Loads
Part – V : Special Loads and Loads combinations

IS 1893 : Criteria for Earth quake resistance design of structures.

IS 10262 : Guidelines for concrete mix proportioning.

IS 13920 : Code of practice for ductile detailing of RCC structure subjected to seismic forces.

SP 16 : Design aid for reinforced concrete to IS 456.

Types of Loads

1. Dead Loads:

Dead loads consist of self-weight of the structure (weight of walls, floors, roofs etc). The weight of the foundation and footings and all other permanent loads acting on the structure. These can be computed by finding the weights of cubical contents of the different materials used for constructing the structure.

2. Live Loads:

Live loads consist of moving or variable loads like people, furniture, temporary stores etc. It is also called super-imposed load.

The Wind acts horizontally on the surfaces of the walls, roofs and inclined roof of the structure. That means it exerts uniform pressure on the structural components on which it acts and tends to disturb the stability of the structure.

5. Seismic Load:

These loads are internal forces which act on the structure due to earthquake developed ground movements.

Different Methods of Design

Methods of Design :

1) Working Stress Method (or Modular Ratio Method)
2) Ultimate load method (Load factor method)
3) The limit state method.
4) The experimental approach.

Difference Between WSM, ULM & LSM

WSM	ULM	LSM
This method evolved around 1900	This method evolved around 1950	This method evolved around 1973
This method is based on linear elastic theory	This method is based on Ultimate strength at Ultimate load.	This method takes into account ultimate strength, serviceability and durability.
This method restricts stresses in materials from working loads.	In this method, service loads are enhanced by load factor.	In this method, service loads are enhanced by factor of safety, and checked for serviceability and durability.
The WSM uses factor of safety 3 with respect to concrete and FOS of 1.8 for steel.	In ULM method different load factors are used under combined loading conditions.	In LSM partial safety factor of 1.5 is used for loads and concrete and 1.15 for steel.

Merits and Demerits of WSM, ULM & LSM

WSM	ULM	LSM
Merits		
Simple in concept and application uses linear part of elastic curve.	ULM uses non-linear part of elastic curve.	LSM uses both linear and non-linear part of elastic curve.
Large sections give better serviceability.	This method gives slender section.	This method gives section which are good in strength and durability
WSM forms part of limit state design.	ULM forms part of limit state design.	This method utilizes good features of WSM and ULM.
De Merits		
WSM gives heavier sections.	ULM gives thinner sections	
Uneconomical sections	Economical sections but fail in durability.	

Limit State Method

LSM:

The acceptable limit of safety and serviceability requirements, before failure occurs, is called limit state.

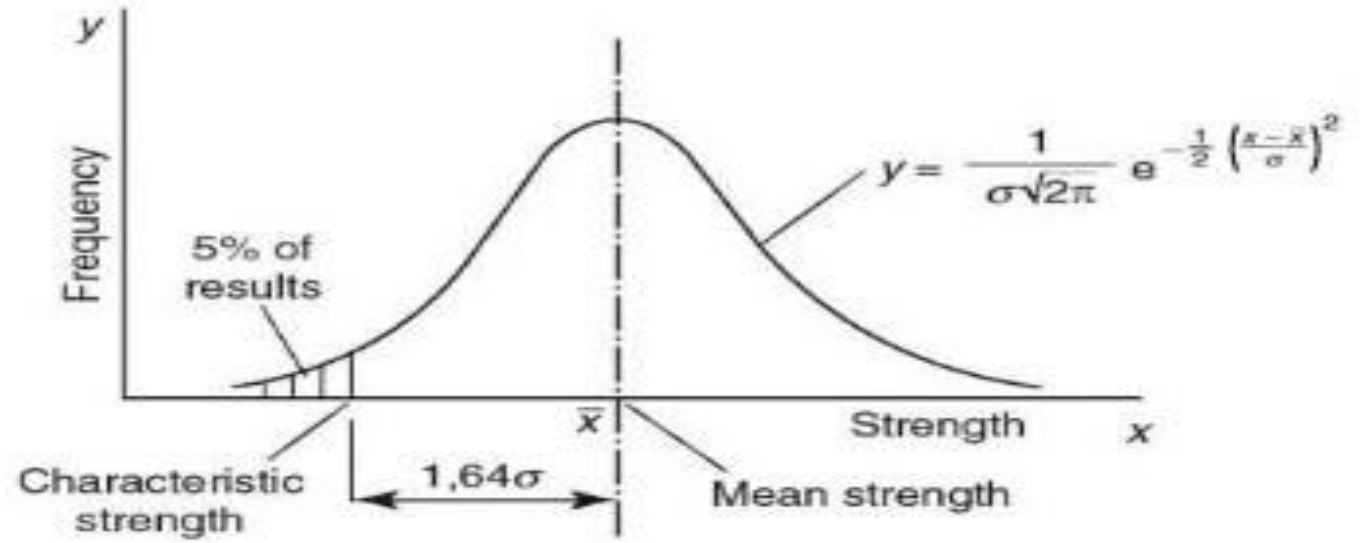
LSM is an ideal method which takes into account not only ultimate strength, but also, the serviceability and durability requirements.

Two categories of limit states are considered in design :

1) Limit state of collapse : Flexure, compression, shear, bond, torsion and tension.
2) Limit state of serviceability : Deflection, cracking, vibration, fire resistance and durability.

Characteristic Strength of Material

The term **characteristic strength** means that value of strength of material below which not more than 5% of test results are expected to fall.



Characteristic Strength of Concrete

The **characteristic strength of concrete** is denoted by $f_{ck}(N/mm^2)$ and its values for different grades is given as per IS Code 456. In table M stands for mix.

Group	Grade Description	Mix Ratio	Characteristic Compressive strength N/mm2
Ordinary Concrete	M10	1:3:6	10
	M15	1:2:4	15
	M20	1:1.5:3	20
Standard Concrete	M25	1:1:2	25
	M30	Design Mix	30
	M35	Design Mix	35
	M40	Design Mix	40
	M45	Design Mix	45
	M50	Design Mix	50
	M55	Design Mix	55
High Strength Concrete	M60	Design Mix	60
	M65	Design Mix	65
	M70	Design Mix	70
	M75	Design Mix	75

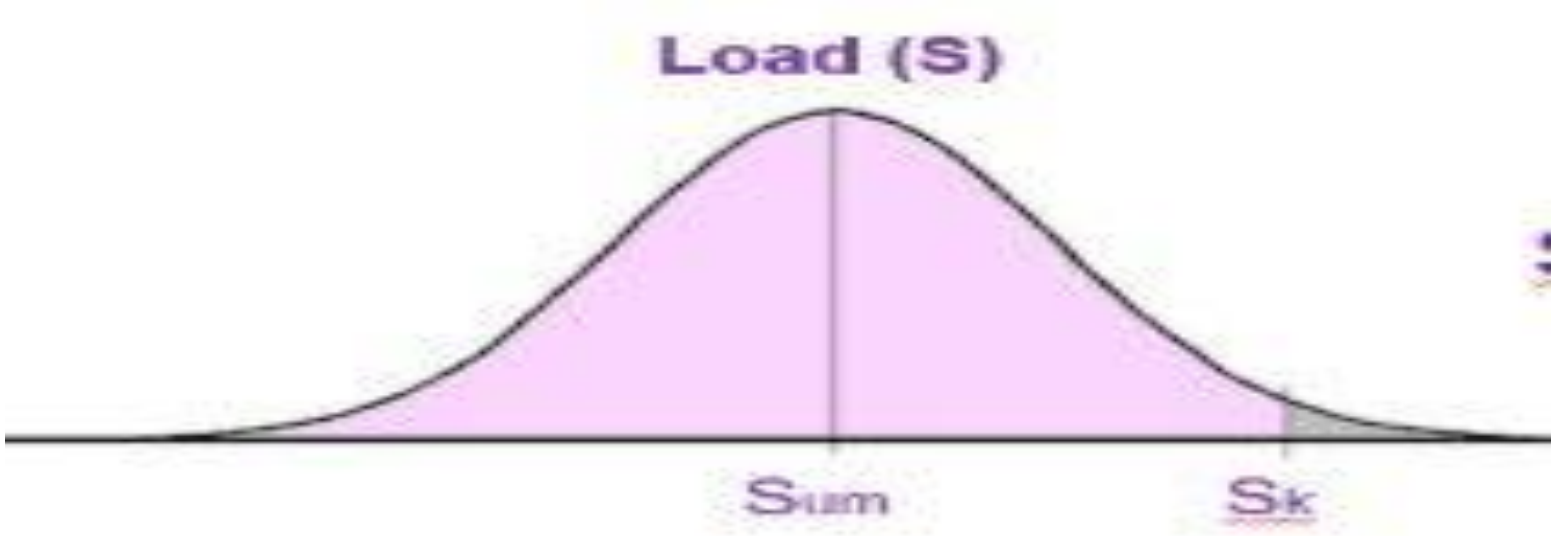
Characteristic Strength of Steel

The **characteristic strength of steel** is denoted by $f_y(N/mm^2)$ and its values for different grades is given as per IS Code 456.

Example : Fe 250, Fe 415, Fe 500, Fe 550 etc.

Characteristic loads

The **characteristic load** is defined as that value of load which has a 95 percent probability of not being exceeded during the life of the structure.



Partial Safety Factor

The characteristic strength and loads are modified by using a factor known as **"partial safety factor"**

Partial safety factor for loads : $\Upsilon_l = 1.5$

Partial safety factor for concrete : $\Upsilon c = 1.5$

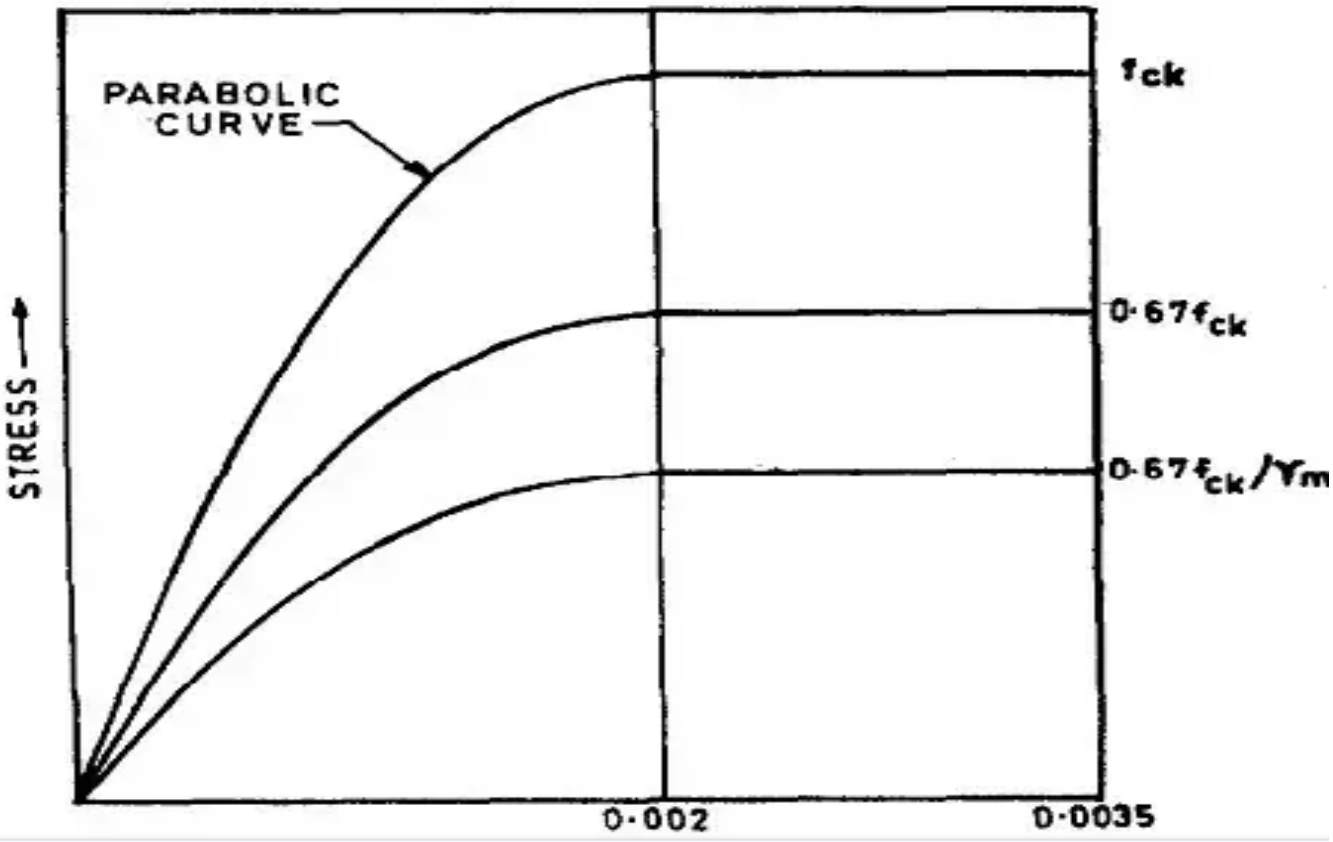
Partial safety factor for steel : $\Upsilon s = 1.15$

Stress – Strain curve for Concrete

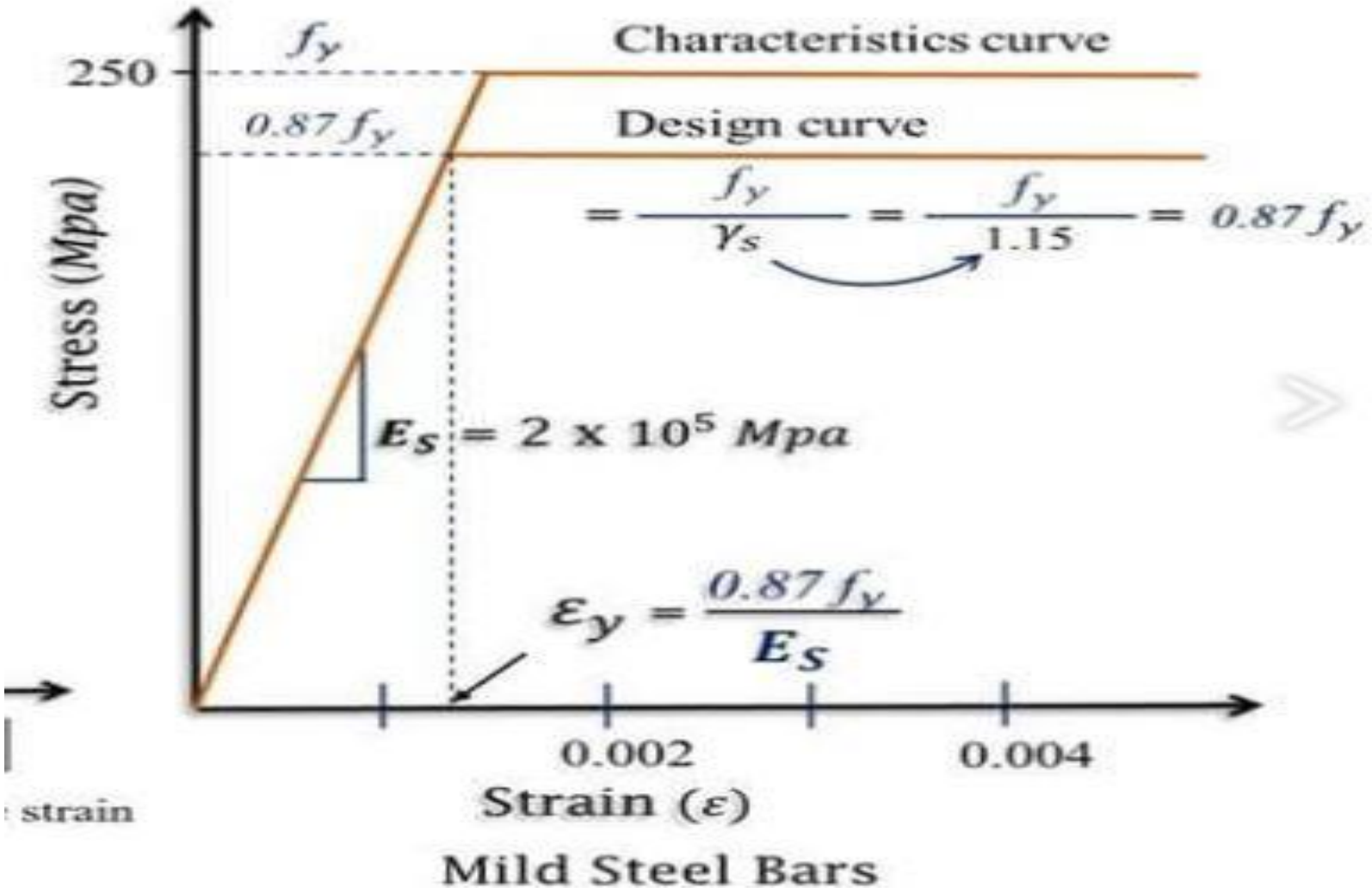
Curve 1 : Represents actual curve

Curve 2 : Based on assumption that the concrete in structure develops a strength of only 0.67 times the strength of cube.

Curve 3 : It is design curve obtained by using partial safety factor of 1.5

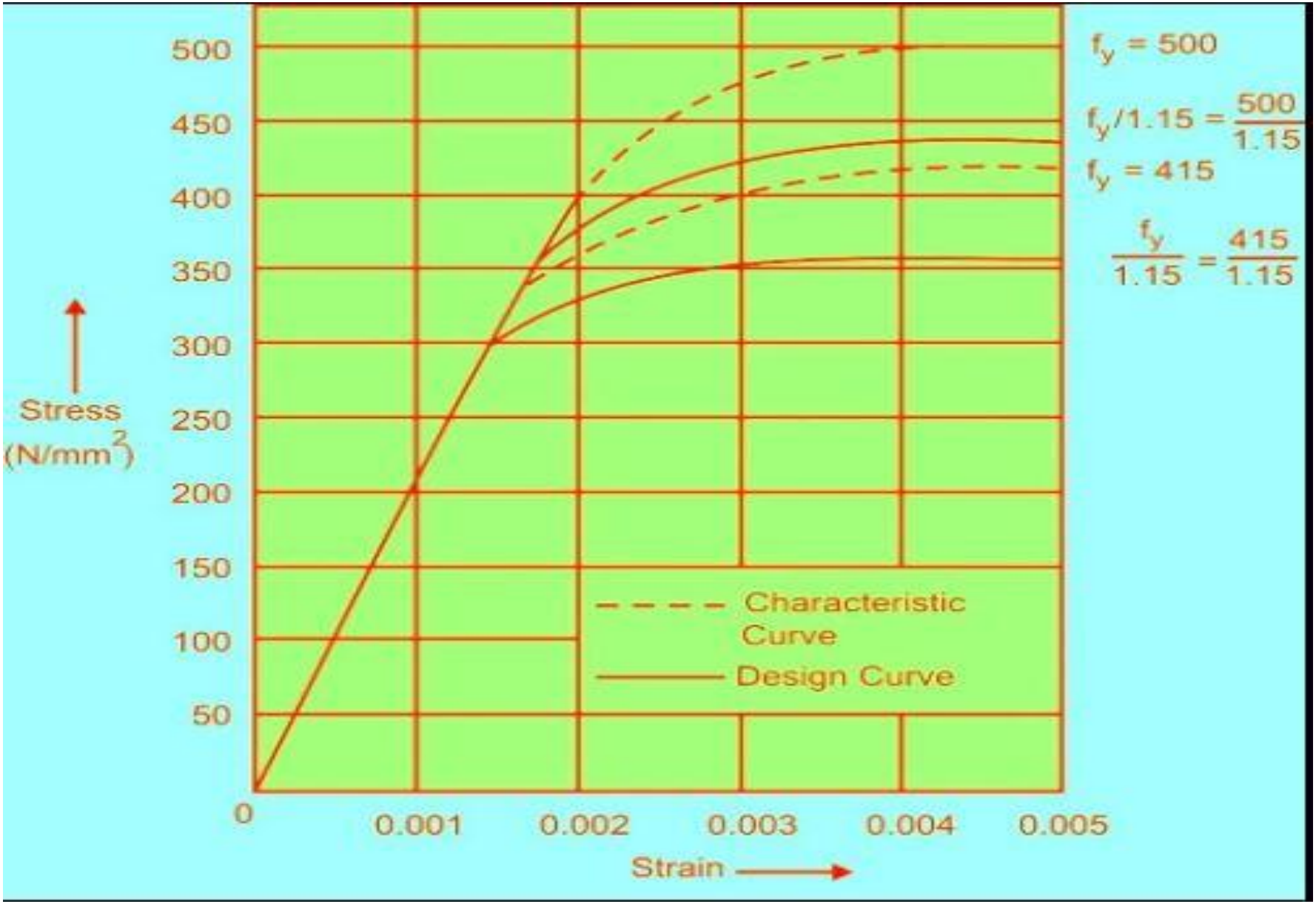


Stress – Strain curve for Mild Steel

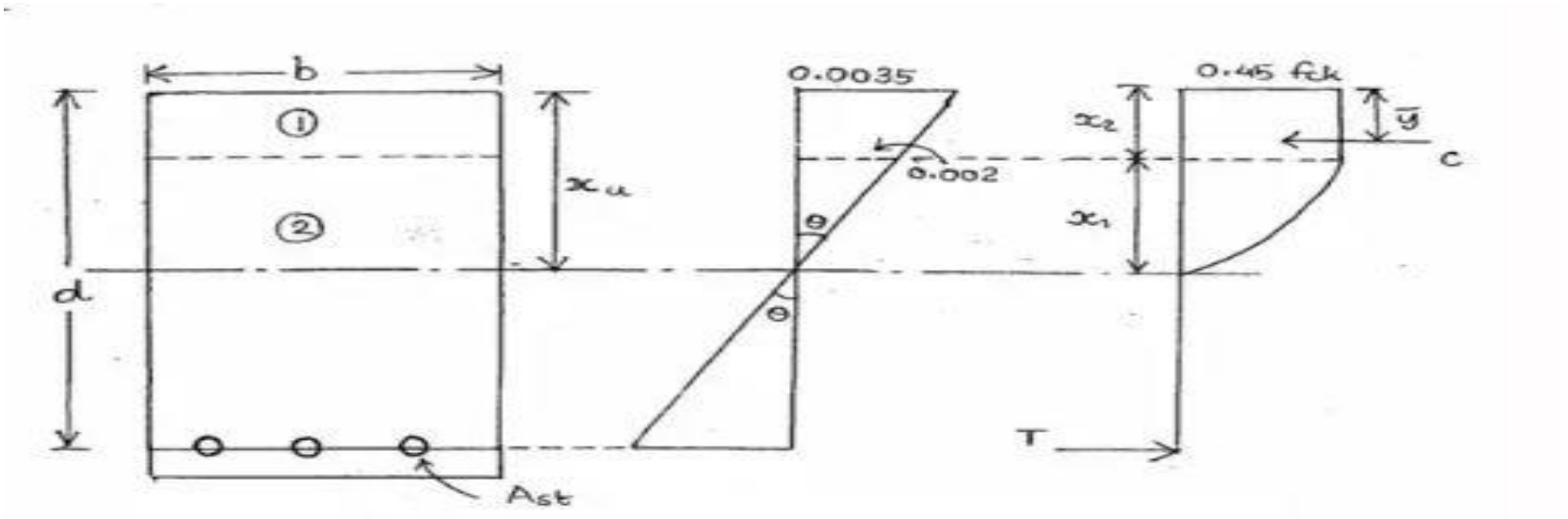


Mild Steel Bars

Stress – Strain curve for HYSD Bars Fe415 & Fe500



Stress block Parameters



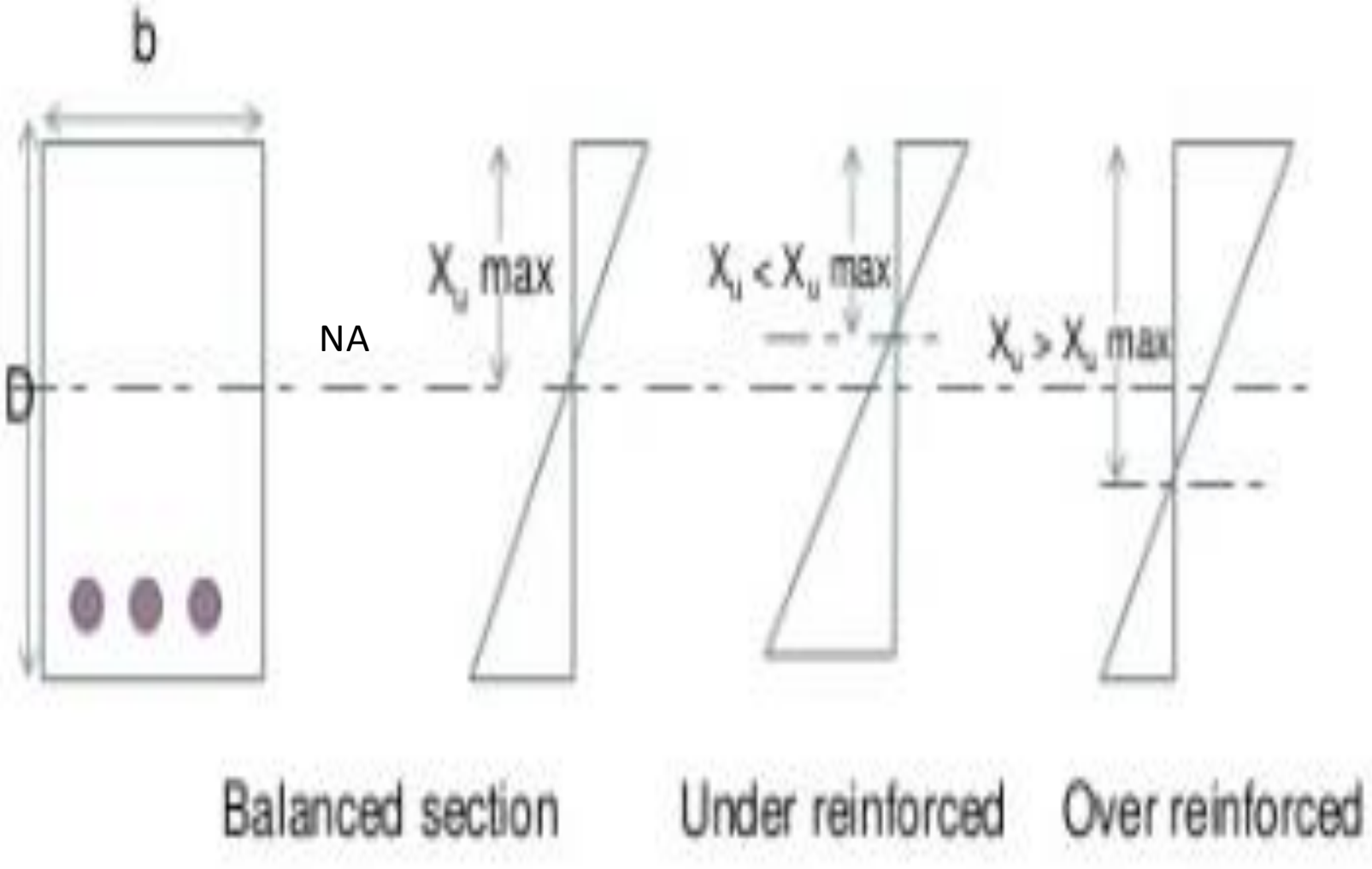
section strain Stress block

The block, which shows the variation of stress across the depth of a beam is known as a " Stress Block".

The stress block according to limit state method is rectangle up to X_2 from upper fiber and parabolically reaches the zero value at the neutral axis.

The above figure shows variation of strain along the depth and the stress block.

Balanced, Under reinforced and Over reinforced section



Balanced, Under reinforced and Over reinforced section

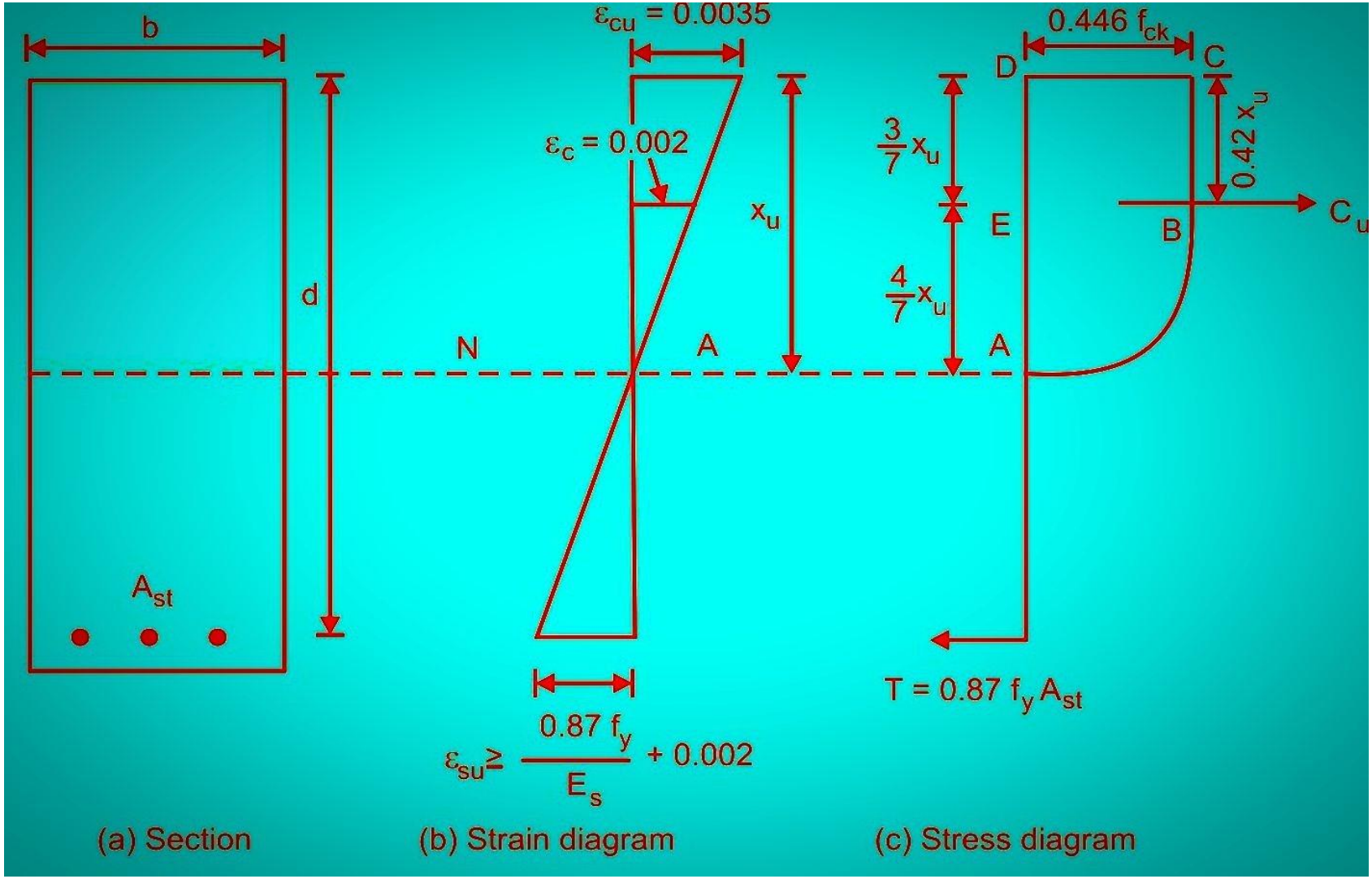
Balanced Section	Under Reinforced Section	Over Reinforced Section
Here strain in steel and in concrete reach their maximum values simultaneously.	In under reinforced section the strain in steel reaches the yield point before concrete.	In over reinforced section the strain in concrete reaches the yield point before steel.
The depth of neutral axis Xu =Xu.max.	Here the actual neutral axis is above balanced NA Xu < Xu.max.	Here the actual neutral axis is below balanced NA Xu > Xu.max.
The percentage of steel in balanced section is known as critical or limiting steel percentage.	In under reinforced section, the steel percentage is less than critical or limiting percentage.	In over reinforced section, the steel percentage is more than critical or limiting percentage.
	The under reinforced sections give warning before failure. Hence this type of structures are preferred.	The over reinforced section fails suddenly without any warning. The code recommends the beam should be redesigned.

Limit State of Collapse : Flexure

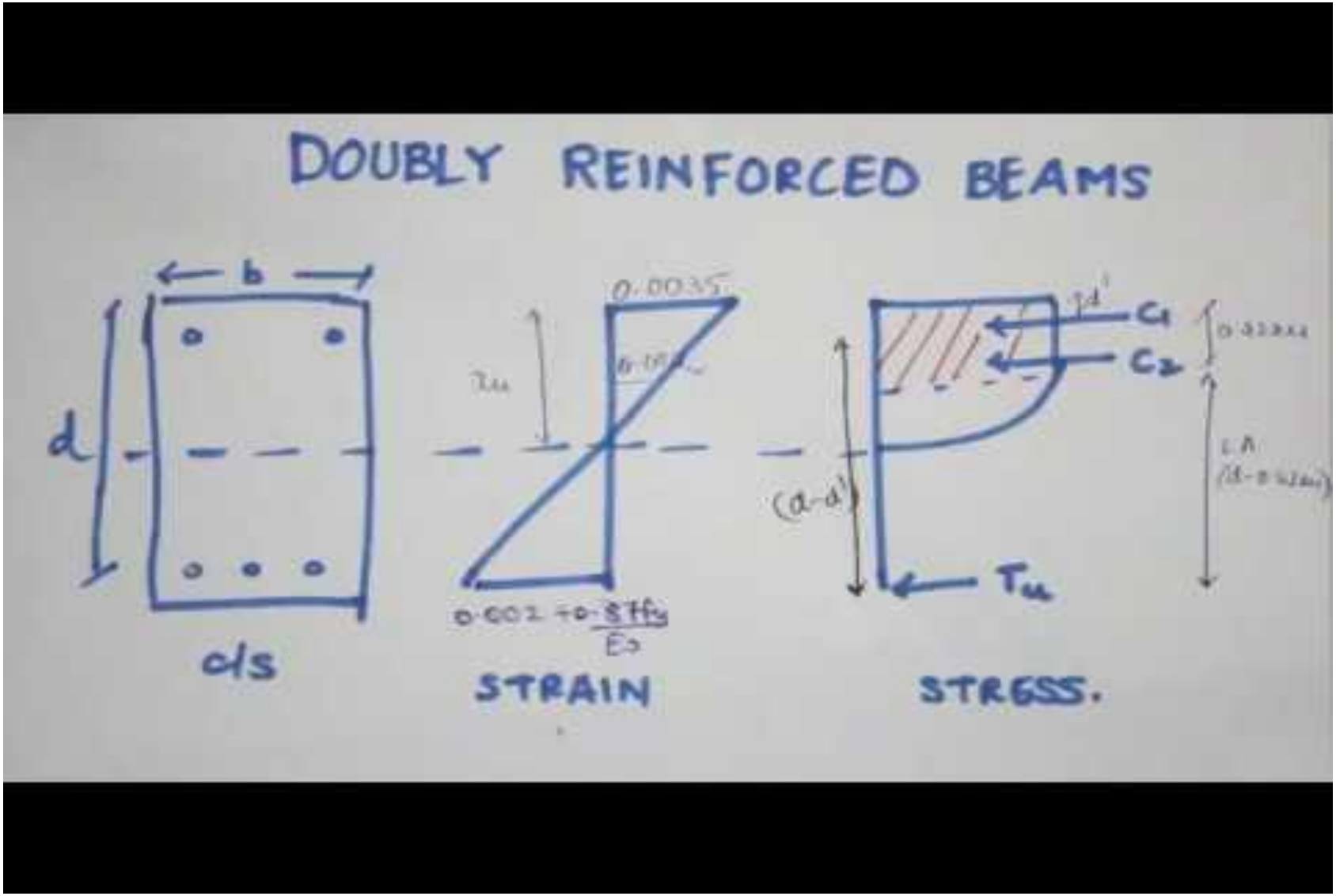
Assumptions (IS Code 456 cl. 38.1) :

1) Plane section normal to the axis remain plane after bending.
2) Maximum strain in concrete at the outermost compression fibre is taken as 0.0035 in bending.
3) For design purposes. the compressive strength of concrete In the structure shall be assumed to be 0.67times the characteristic strength. The partial safety factor y. = 1.5 shall be applied in addition to this.
4) The tensile strength of the concrete is ignored.
5) The stresses in the reinforcement are derived from representative stress-strain curve for the type of steel used. For design purposes the partial safety factor Ym equal to 1.15 shall be applied.
6) The maximum strain in the tension reinforcement in the section at failure shall not be less than: fy/1.15Es + 0.002.

Singly Reinforced Sections



Doubly Reinforced Sections



Flanged Beams

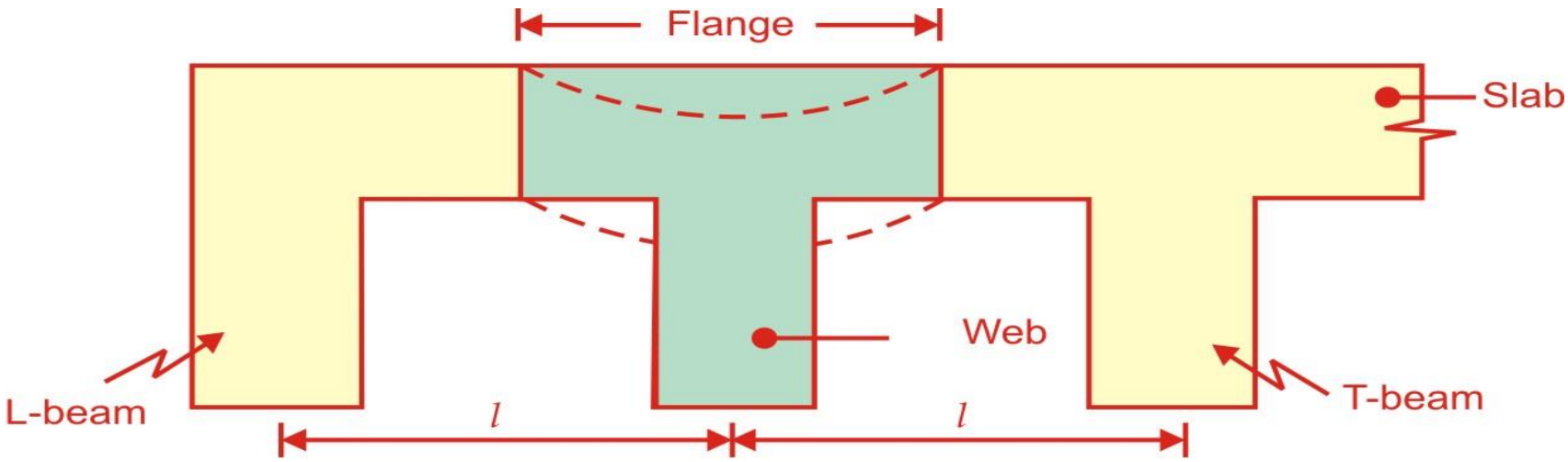
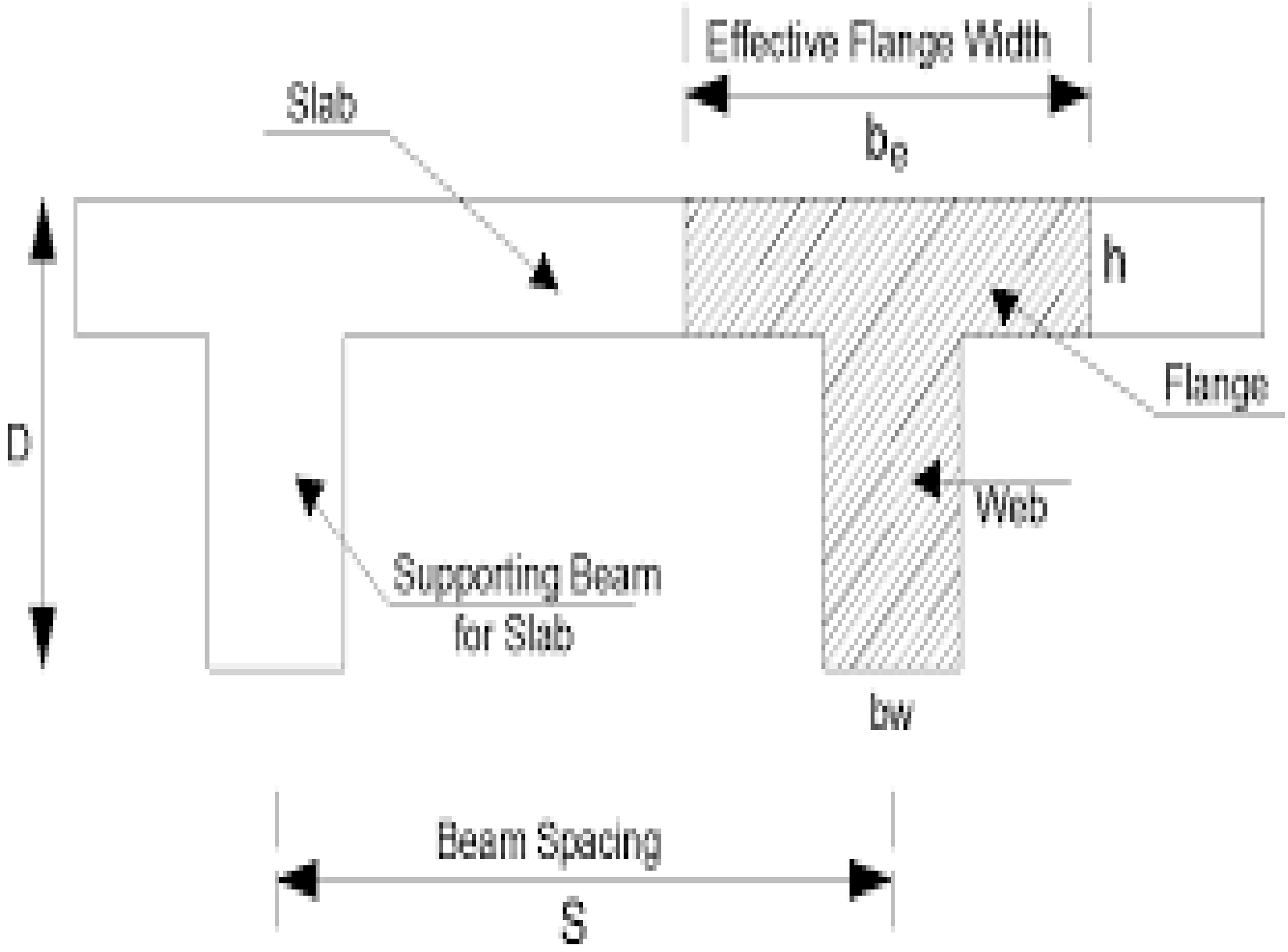


Fig. 2.11. T-beams and L-beams.


Effective Flange Width
Slab
b_e
h
D
Flange
Web
Supporting Beam
for Slab
bw
Beam Spacing
S

Problem Solving. MHKHAN

Singly Reinforced Beams.

Prob1 Find the moment of resistance of a singly reinforced beam 200mm width and 400mm effective depth. It is reinforced with 3 bars of 16mmϕ Fe 415 steel. Take Concrete M20. Use IS code method.

Sol:- Given Data:-

$b = 200mm$, $d = 400mm$

$Ast = 3 \times \frac{\pi}{4} \times 16^2 = 603.18 mm^2$

$f_y = 415 N/mm^2$

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

As per IS Code:-

$$\frac{x_u}{d} = \frac{0.87 f_y Ast}{0.36 f_{ck} bd}$$

$$= \frac{0.87 \times 415 \times 603.18}{0.36 \times 20 \times 200 \times 400}$$

$$= 0.37$$

Limiting depth from code.

for $f_y = 415 N/mm^2$

$$\frac{x_{u(max)}}{d} = 0.48$$

$$\frac{x_u}{d} < \frac{x_{u(max)}}{d}$$

$0.37 < 0.48$

It is under reinforced section.

Moment of Resistance:-

for under reinforced section.

$$M_u = 0.87 f_y Ast d \left(1 - \frac{Ast f_y}{bd f_{ck}}\right)$$

$$= 0.87 \times 415 \times 603.18 \times 400 \left(1 - \frac{603.18 \times 415}{200 \times 400 \times 20}\right)$$

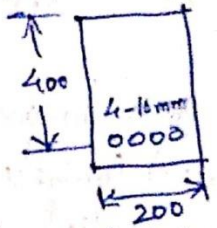
$$= 73.48 \times 10^6 Nmm.$$

Prob2. Find the moment of Resistance of singly reinforced beam 230mm width and 450mm overall depth. It is reinforced with 2 Nos 20mm dia bars. Take Fe 415 steel and M20 Grade concrete.

(Hint:- $d = D - Cover$, $Cover = 25mm$)

Problem3 Find the Moment of Resistance of a singly reinforced Concrete beam 200mm width and 400mm effective depth, reinforced with 4 Nos 16mm dia Fe 415 steel and M20 Concrete.

Sol:- Given Data:-



$b = 200mm$, $d = 400mm$.

$Ast = 4 \times \frac{\pi}{4} \times 16^2 = 804.24 mm^2$

As per IS Code Annex G.

$$\frac{x_u}{d} = \frac{0.87 f_y Ast}{0.36 f_{ck} bd}$$

$$= \frac{0.87 \times 415 \times 804.24}{0.36 \times 20 \times 200 \times 400}$$

$$= 0.50$$

Limiting depth of N.A. from code:- for f_y 415 N/mm^2

$\frac{x_{u(max)}}{d} = 0.48$

$\frac{x_u}{d} > \frac{x_{u(max)}}{d}$

$0.5 > 0.48$

The section is over reinforced section.

It should be redesigned.

Problem 4: Design a balanced singly reinforced concrete beam section for an applied moment of 60 kN-m. The width of the beam is limited to 175 mm. Use M20 concrete and Fe 415 steel bars.

Sol:- **Given Data:-**

Give Moment = 60 kNm.

Ultimate or factored Moment = 1.5 × 60
= 90 kNm.
= 90×10^6 N-mm.

b = 175 mm.

Limiting depth of N.A. from IS code 456.

for Fe 415 Steel.

$\frac{x_{u(max)}}{d} = 0.48$

From IS Code Moment of Resistance for balanced section:-

$M_u = 0.36 \frac{x_{u(max)}}{d}\left(1 - 0.42 \frac{x_{u(max)}}{d}\right) f_{ck} b d^2$

$90 \times 10^6 = 0.36 \times 0.48 (1 - 0.42 \times 0.48) 20 \times 175 \times d^2$

$d = 431.72$ mm.

Taking $d = 435$

$D = 435 + 25 = 460$ mm.

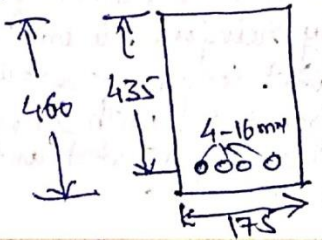
Tensile Reinforcement from eq^n - IS Code

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

$0.48 = \frac{0.87 \times 415 \times A_{st}}{0.36 \times 20 \times 170 \times 435}$

$A_{st} = 707.855 \text{ mm}^2$.

Provide 4 bars of 16 mm dia.



Problem 5: A beam simply supported over a span of 7 m carries a load of 20 kN/m including self weight. Design the beam using M25 concrete and Fe 500 steel.

Sol:- **Given data:-**

Length = 7 m.

Total load (including self wt) = 20 kN/m

factored load = 1.5 × 20
= 30 kN/m.

factored Moment = $M_u = \frac{W_u l^2}{8}$

$= \frac{30 \times 7^2}{8} = 183.75$ kNm.

$f_y = 500 \text{ N/mm}^2$

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

Assuming b = 230 mm.

From IS code :-

$\frac{x_{u(max)}}{d}$ for Fe 500

$\frac{x_{u(max)}}{d} = 0.46$.

Moment of Resistance for balanced section:-

$$= 0.36 \frac{x_{u(max)}}{d}\left(1 - 0.42 \frac{x_{u(max)}}{d}\right) b d^2 f_{ck}$$

$$183.75 \times 10^6 = 0.36 \times 0.46 (1 - 0.42(0.46)) 230 \times d^2 \times 25$$

$$d = 489.06 \text{ mm}$$

$$\simeq 490 \text{ mm}$$

$$\therefore D = 490 + 25 = 515 \text{ mm}$$

Tensile Reinforcement (Ast):-

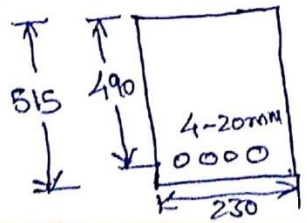
from IS Code

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

$$0.46 = \frac{0.87 \times 500 \times A_{st}}{0.36 \times 25 \times 230 \times 490}$$

$$A_{st} = 1073 \text{ mm}^2$$

Provide 4 bars of 20mm.



Problem:6 A beam simply supported over a effective span of 7m carries a live load of 20KN/m. Design beam using M20 concrete and fe415 steel. Keep width of the beam equal to half of the effective depth.

Sol:- Given Data:-

Live load = 20KN/m.

length = 7m

$f_{ck} = 20 \text{ N/mm}^2$, $f_y = 415 \text{ N/mm}^2$.

$b = \frac{d}{2}$.

Load Calculations:-

Live load = 20KN/m.

Assuming $b = 200$mm.

$d = 2b = 400$mm.

$D = 400 + 25 = 425$mm.

Self wt. of beam $= 0.2 \times 0.425 \times 25$

$= 2.125$ KN/m.

Total load = LL + DL

$= 20 + 2.125$

$= 22.125$ KN/m

Factored load $= 1.5(22.125)$

$= 33.1875$ KN/m

Bending Moment $= \frac{wl^2}{8}$

$M_u = \frac{33.1875 \times 7^2}{8}$

$= 203.27$ KNm

Limiting depth of N.A. IS Code:-

For fe 415

$\frac{x_{u(max)}}{d} = 0.48$.

Moment of Resistance for balanced section IS Code:-

$$M_u = 0.36 \frac{x_{u(max)}}{d}\left(1 - 0.42 \frac{x_{u(max)}}{d}\right) f_{ck} b d^2$$

$$203.27 \times 10^6 = 0.36 \times 0.48 (1 - 0.42 \times 0.48) \times 20 \times \frac{2b}{2} \times d^2$$

$$d = 529.39$$

$$\simeq 530 \text{ mm}.$$

$$b = \frac{530}{2} = 265 \text{ mm}.$$

$$D = 530 + 25 = 555 \text{ mm}$$

Tensile Reinforcement

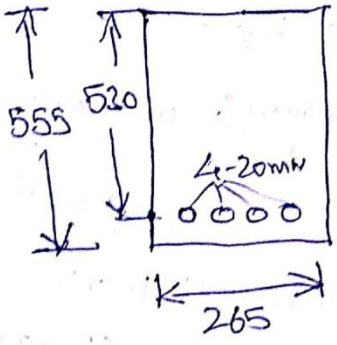
Using equation from IS code.

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

$$0.48 = \frac{0.87 \times 415 \times A_{st}}{0.36 \times 20 \times 265 \times 530}$$

$$A_{st} = 1344.\ mm^2$$

Provide 4 no's - 20mm bars.



Problem 7 Design a Singly reinforced Concrete beams to suit the following data:- clear span 3m, width of supports = 200mm. Working Live load = 6kN/m. Use M20 Concrete and Fe415 Steel.

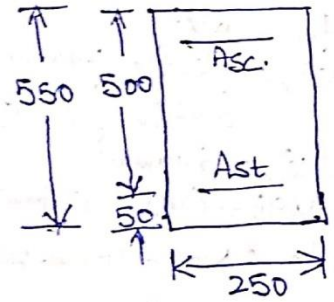
Sol:- Hint:- $l_{ef} = 3 + \frac{0.2}{2} + \frac{0.2}{3} = 3.2m$

for Self. wt. Assume $b = 230mm$
$d = 460mm$.

Doubly Reinforced Beam

Problem 1 Determine the tensile and compression reinforcement for a rectangular beam with following data overall size of beam 250 × 550 mm. Concrete M20 grade steel, Steel Fe 415. Factored Moment = 200 kNm. Effective Cover = 50 mm.

Sol:- Given Data:-



$b = 250\text{mm}$, $D = 550\text{mm}$

$d' = 50\text{mm}$, $d = 500\text{mm}$

$f_y = 415\text{N/mm}^2$, $f_{ck} = 20\text{N/mm}^2$

$M_u = 200\text{kNm}$

$= 200 \times 10^6\text{ Nmm}$

From IS Code for Fe 415 depth of N.A.:-

$$\frac{x_{u(max)}}{d} = 0.48$$

From IS Code. balanced or limiting Moment of Resistance:-

$$M_{u(lim)} = 0.36 \frac{x_{u(max)}}{d}\left(1 - 0.42 \frac{x_{u(max)}}{d}\right) \times f_{ck} b d^2$$

$$= 0.36 \times 0.48 (1 - 0.42 \times 0.48) \times 20 \times 250 \times 500^2$$

$$= 172.45 \times 10^6\text{ N mm}$$

Actual $M_u > M_{u(lim)}$.

As per IS Code 456 Compression Reinf. is given by:-

$$M_u - M_{u(lim)} = f_{sc} A_{sc} (d - d')$$

$f_{sc} = 0.87 \times 415 = 361\text{ N/mm}^2$

$$(200 \times 10^6 - 172.45 \times 10^6) = 361 \times A_{sc} (500 - 50)$$

$$A_{sc} = 169.59\text{ mm}^2.$$

Provide 2 nos 12 mm dia bars.

Tension Reinforcement A_{st}:-

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

A_{st_1} reinf. as singly reinf. beam

$$\frac{x_u}{d} = \frac{0.87 f_y A_{st_1}}{0.36 f_{ck} b d}$$

$$0.48 = \frac{0.87 \times 415 \times A_{st_1}}{0.36 \times 20 \times 250 \times 500}$$

$$A_{st_1} = 1196.51\text{ mm}^2.$$

$$A_{st_2} = \frac{A_{sc} f_{sc}}{0.87 f_y}$$

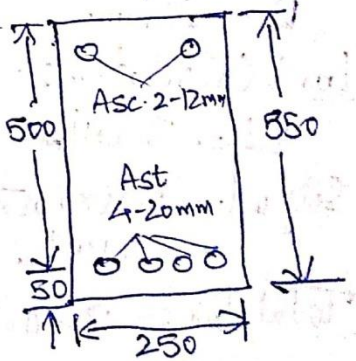
$$= \frac{169.59 \times 361}{0.87 \times 415}$$

$$= 169.59\text{ mm}^2.$$

Total $A_{st} = 1196.51 + 169.59$

$$= 1366\text{ mm}^2.$$

Provide 4 nos's – 20 mm bars



Problem 2. Design a reinforced concrete beam of rectangular section using the following data; Effective Span = 8m, Working live load = 30 KN/m. Overall depth restricted to 650mm. Use M20 grade concrete & Fe415 steel.

Sol:- **Given Data:-**

Effective Span = 8m.
LL = 30 KN/m.
D restricted to 650mm.
$f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$

Assume Dimensions:

b = 250mm.
D = 650 mm.

Load Calculations:-

LL = 30 KN/m.
Self wt. = $0.25 \times 0.65 \times 25$
= 4 KN/m.
Total load = 30 + 4 = 34 KN/m

Factored load = 1.5×34
= 51 KN/m

Bending Moment:-

$M_u = \frac{wl^2}{8} = \frac{51 \times 8^2}{8}$
= 408 KNm

from IS Code for Fe 415

$\frac{x_{u(max)}}{d} = 0.48$.

from IS Code limiting Moment of Resistance:-

$M_{u(lim)} = 0.36 \frac{x_{u(max)}}{d}\left(1 - 0.42 \frac{x_{u(max)}}{d}\right) f_{ck} b d^2$

$= 0.36 \times 0.48 (1 - 0.42 \times 0.48) \times 20 \times 250 \times 625^2$

$= 269.46 \times 10^6$ Nmm.

$M_u > M_{u(lim)}$:-

Provide compression reinforcement

Compression Reinforcement of Steel

$M_u - M_{u(lim)} = f_{sc} A_{sc} (d - d')$

$408 \times 10^6 - 269.46 \times 10^6 = 361 A_{sc} (625 - 25)$

$A_{sc} = 640 mm^2$.

Provide 4 - 16mm dia.

Tensile Reinforcement Ast:-

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

A_{st_1} Reinf. as Singly Reinf. beam

$\frac{x_u}{d} = \frac{0.87 f_y A_{st_1}}{0.36 f_{ck} b d}$

$0.48 = \frac{0.87 \times 415 \times A_{st_1}}{0.36 \times 20 \times 250 \times 625}$

$A_{st_1} = 1496 mm^2$.

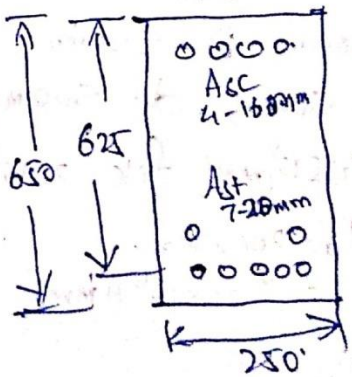
A_{st_2} from Code:

$A_{st_2} = \frac{A_{sc} f_{sc}}{0.87 f_y} = \frac{640 \times 361}{0.87 \times 415}$

$A_{st_2} = 640 mm^2$

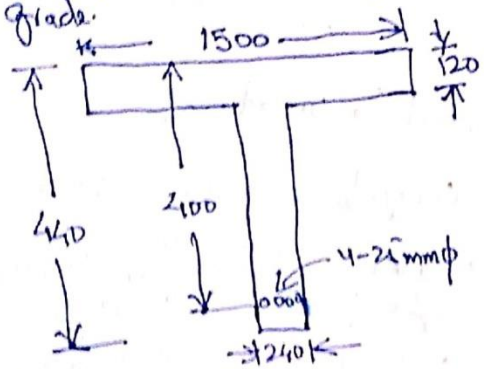
$A_{st} = 1496 + 640 = 2136 mm^2$

Provide ~~4~~ Nos - 20mm. bar



Problems on T-beam:

Problem 1. Find the moment of resistance of the T-beam section shown in fig. If concrete is of M20 grade and steel of Fe415 grade.



Sol:- Given Data:-

$b_f = 1500mm$, $D_f = 120mm$.

$b_w = 240mm$, $d = 400mm$.

$A_{st} = 4 \times \frac{\pi}{4} \times 25^2 = 1963 mm^2$.

Depth of Neutral Axis:-

Using equation

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

$$\frac{x_u}{400} = \frac{0.87 \times 415 \times 1963}{0.36 \times 20 \times 1500 \times 400}$$

$x_u = 65.65 mm$.

$x_u < D_f$ $\quad 65.65 < 120 mm$.

The neutral axis lies in flange.

The Moment of Resistance is Calculated as per Section G-1.1. of IS Code.

$$\frac{x_u}{d} = 0.164$$

$$\frac{x_{u(max)}}{d} = 0.48$$

$\frac{x_u}{d} < \frac{x_{u(max)}}{d}$ the beam is under reinforced section.

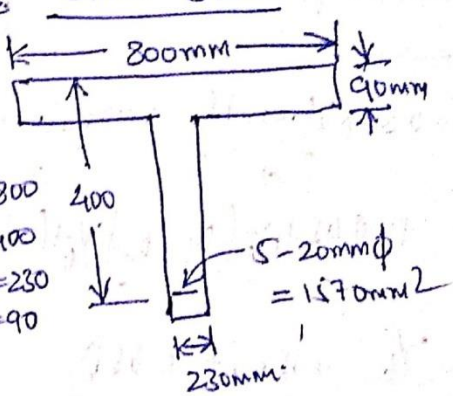
$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{f_{ck} b d}\right)$$

$$= 0.87 \times 415 \times 1963 \times 400 \left(1 - \frac{1963 \times 415}{20 \times 1500 \times 400}\right)$$

$$= 264.25 \times 10^6 \ N\,mm$$

Problem 2. A T-beam of effective flange width 800mm and thickness of slab 90mm, width of rib is 230mm, the effective depth is 400mm. The beam is reinforced with 5 bars of 20mm dia. Calculate moment of resistance, use M20 grade concrete and Fe415 steel.

Sol:- Given Data:-



Depth of Neutral Axis

using equation.

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

$$\frac{x_u}{400} = \frac{0.87 \times 415 \times 1570}{0.36 \times 20 \times 800 \times 400}$$

$x_u = 98.4 mm$

$x_u > D_f$.

Neutral Axis lies in web.

Calculate $\frac{D_f}{d}$ value.

$$\frac{D_f}{d} = \frac{90}{400} = 0.225 > 0.2$$

Moment of Resistance when $\frac{D_f}{d} > 0.2$

$$M_u = 0.36 x_{u(max)} \left(1 - 0.42 \frac{x_{u(max)}}{d}\right) f_{ck} b d^2 + 0.45 f_{ck} (b_f - b_w) y_f \left(d - \frac{y_f}{2}\right)$$

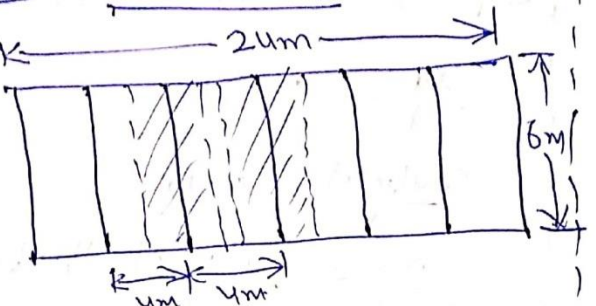
$$y_f = 0.15 x_u + 0.65 D_f$$
$$= 0.15 \times 98.4 + 0.65 \times 90$$
$$= 73.26 \text{ mm}.$$

$$M_u = 0.36 \times 0.48 (1 - 0.42(0.48)) 20 \times 230 \times 400^2 + 0.45 \times 20 (800 - 230) \times 73.26 \left(400 + \frac{73.26}{2}\right)$$

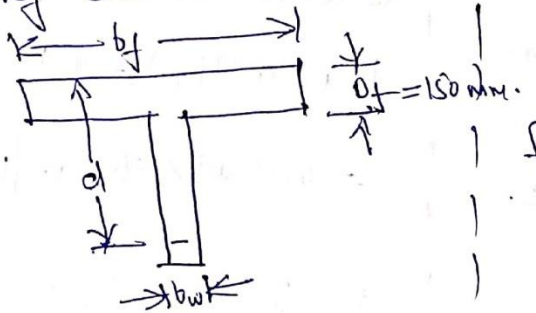
$$= 238.1 \times 10^6 \text{ Nmm}$$

Problem:- A hall of 6m x 24m is covered by RCC slab 150mm thick supported on beams spaced at 4m interval. Design the beams.

Sol:- Given Data:-



Taking Beams as T-beam.



$l_0 = 6m$, $D_f = 150mm$.

Assume $b_w = 230mm$.

As per IS code

$$b_f = \frac{l_0}{6} + b_w + 6 D_f$$
$$= \frac{6000}{6} + 230 + 6 \times 150$$
$$= 2130 \text{ mm}.$$

Assume overall depth = $\frac{1}{12}$ to $\frac{1}{15}$ of span

$$D = \frac{1}{12} \times 6000 = 500$$

Cover = 40mm.

$d = 460mm$,

Load Calculations:-

Flange load = $0.15 \times 2.13 \times 25$
$= 7.9875 \text{ kN/m}$

Web load = $0.35 \times 0.23 \times 25 = 2 \text{ kN/m}$

Floor finish = 1 kN/m

Total DL = 10.9875 kN/m

Live load = 2 kN/m

Total load = $12.9875 \approx 13 \text{ kN/m}$

factored load = 1.5×13
$= 19.5 \text{ kN/m}$.

Bending Moment = $\frac{wl^2}{8}$

$= \frac{19.5 \times 6^2}{8}$

$= 87.75 \text{ kNm}$

(Find Ast using M_u.)

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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: DESIGN OF REINFORCED CONCRETE STRUCTURES

SUBJECT CODE: 20CE5PC02

FACULTY NAME: MOHAMMED MUNEERUDDIN KHAN

SYLLABUS:

UNIT – II

Design for Shear, Bond and Torsion - Mechanism of shear and bond failure - Design of shear using limit state concept – Design for Bond –Anchorage and Development length of bars - Design of sections for torsion - Detailing of reinforcement.

DESIGN OF REINFORCED CONCRETE STRUCTURES
BTECH III-I
UNIT II

MOHD MUNEERUDDIN KHAN

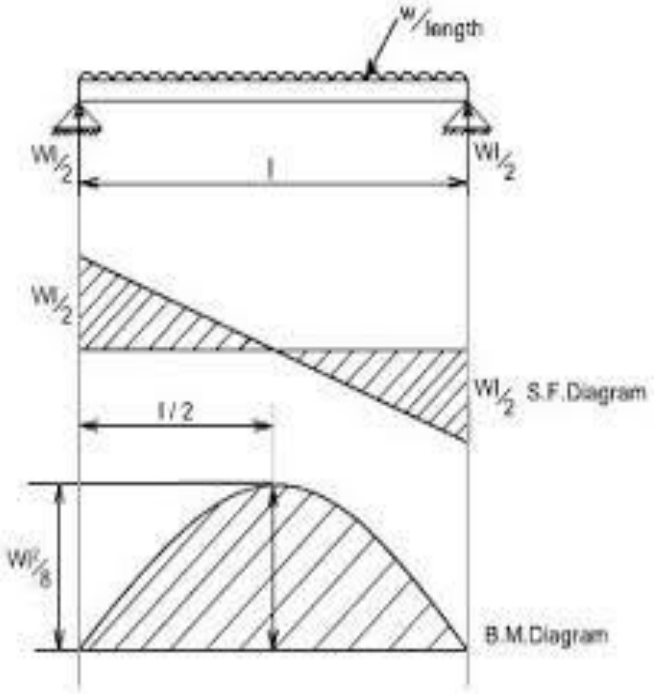
ASSISTANT PROFESSOR

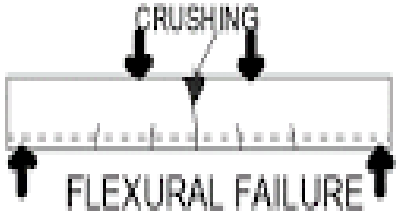
CED, TKREC

Shear stresses in RC Beams

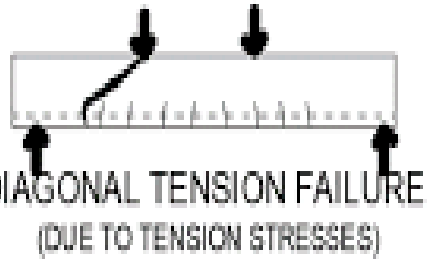
When a beam is loaded with transverse loads, the bending moment varies from section to section. Shearing stresses in the beam are caused by this variation of bending moment along the span.

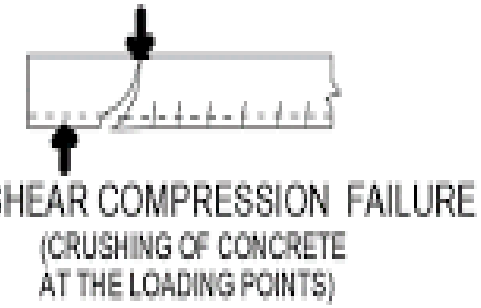
Failure of RC Beam in shear may not lead to immediate collapse, but will reduce flexural strength and ductility.











Types of Failure In Beam

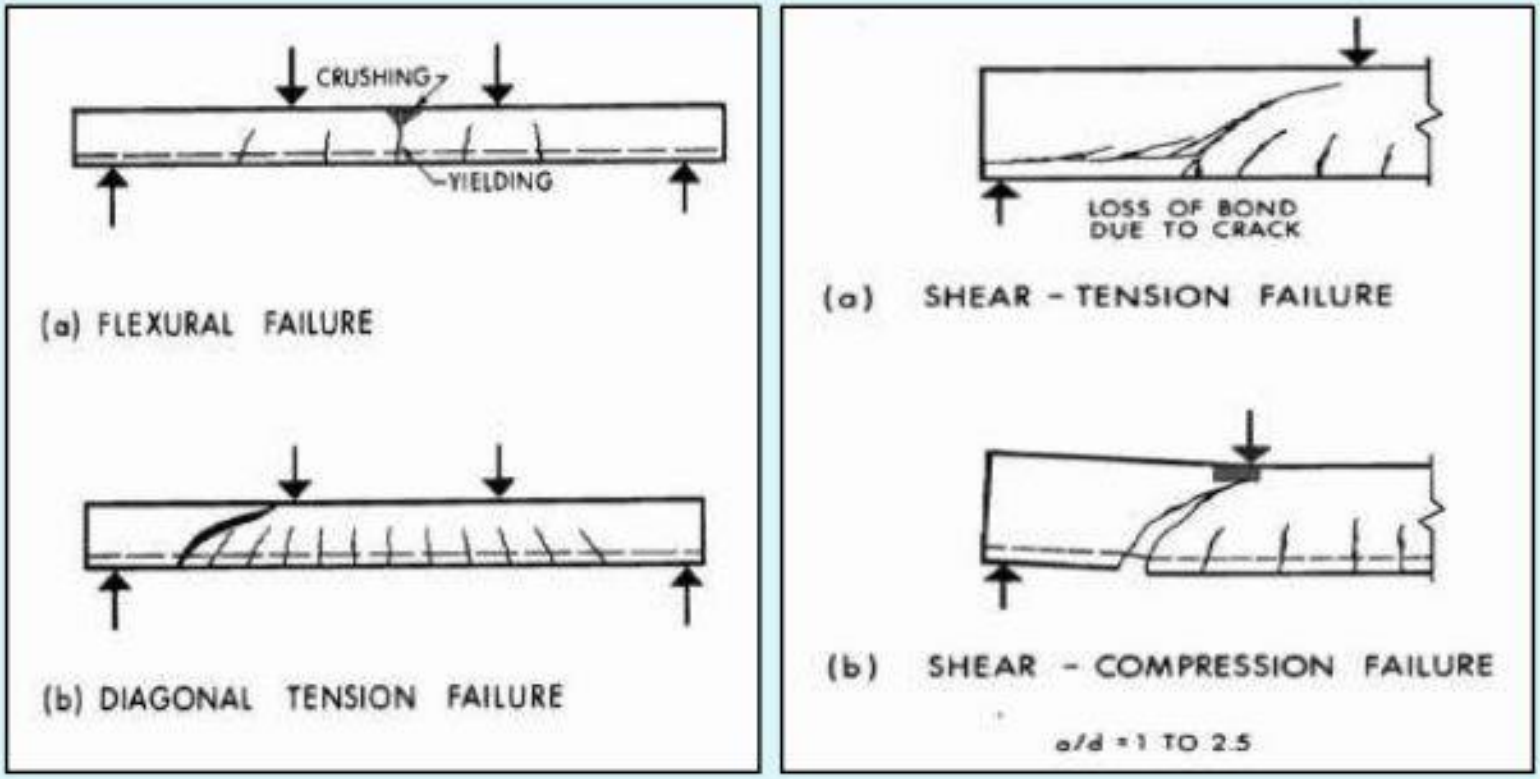
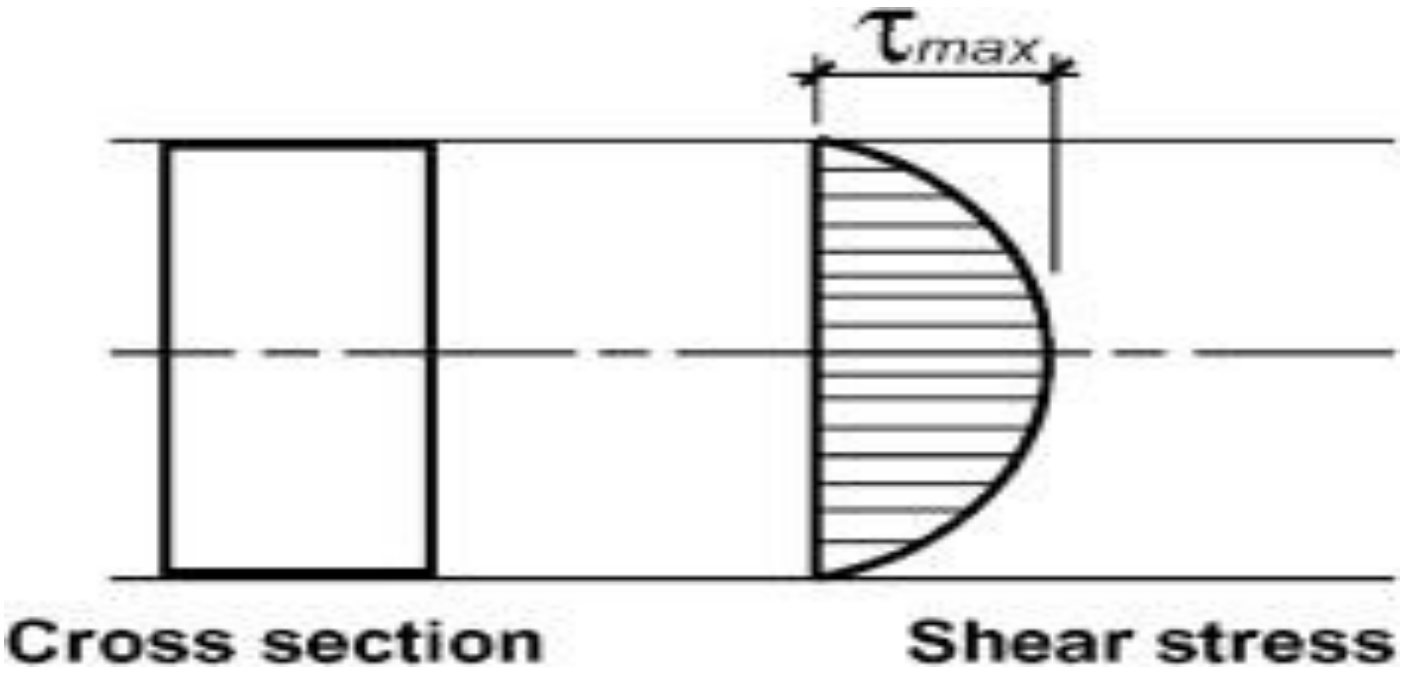


Fig.Beam failure modes (from ACI-ASCE Committee 426, 1973)

Shear Stresses in homogeneous beams

In case of a homogeneous beam (i.e. Concrete beam without reinforcement) the shear stress τ at any point is given by

$$\tau = V/I_b \; AY$$



Shear Stress distribution in RCC beam

As per IS Code 456, the shear stress is called nominal shear stress, and is given by equation

$\tau_v = V_u/bd$

Where Vu is shear force, b is breadth, d effective depth.

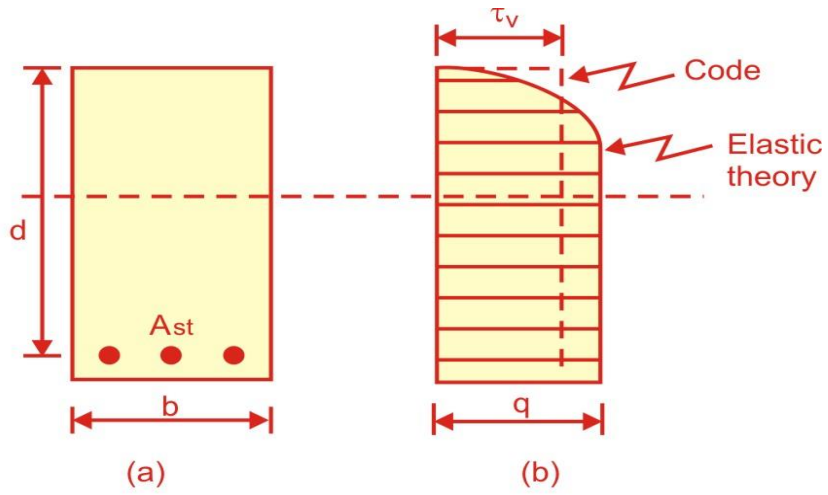
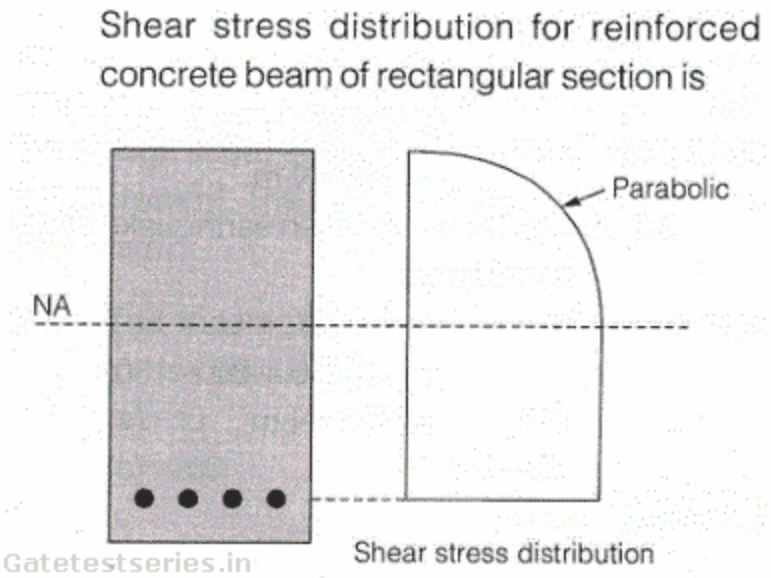


Fig. 5.2. Shear-stress distribution in RCC beam.



Nominal Shear Stress

40.1 Nominal Shear Stress

The nominal shear stress in beams of uniform depth shall be obtained by the following equation:

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

where

V_u = shear force due to design loads;

b = breadth of the member, which for flanged section shall be taken as the breadth of the web, b_w; and

d = effective depth.

40.1.1 ***Beams of Varying Depth***

In the case of beams of varying depth the equation shall be modified as:

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

where

τ_v, V_u, b and d are the same as in **40.1**,

M_u = bending moment at the section, and

β = angle between the top and the bottom edges

Design Shear Stress/Strength

Clause 40.2 :

The design shear strength of concrete τ_c is given in Table 19 of IS Code 456.

Table 19 Design Shear Strength of Concrete, τ_c, N/mm²

(*Clauses* 40.2.1, 40.2.2, 40.3, 40.4, 40.5.3, 41.3.2, 41.3.3 *and* 41.4.3)

$100\frac{A_s}{bd}$	Concrete Grade					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤ 0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.50	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.66	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.00 and above	0.71	0.82	0.92	0.96	0.99	1.01

NOTE — The term A_s is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at support where the full area of tension reinforcement may be used provided the detailing conforms to **26.2.2** and **26.2.3**

Maximum Shear Stress

Clause 40.2.3:

Under no circumstances, even with shear reinforcement, shall the nominal shear stress Ƭv in beams exceed Ƭcmax, given in table 20 of IS Code 456.

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm²

(Clauses 40.2.3, 40.2.3.1, 40.5.1 and 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0

Minimum Shear Reinforcement

Clause 40.3 : When τ_v is less than τ_c given in Table 19, minimum shear reinforcement shall be provided in accordance with clause 26.5.1.6

26.5.1.6 ***Minimum shear reinforcement***

Minimum shear reinforcement in the form of stirrups shall be provided such that:

$$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87 f_y}$$

where

A_{sv} = **total cross-sectional area of stirrup legs effective in shear,**

s_v = **stirrup spacing along the length of the member,**

b = **breadth of the beam or breadth of the web of flanged beam, and**

f_y = **characteristic strength of the stirrup reinforcement in N/mm² which shall not be taken greater than 415 N/mm².**

Design of Shear Reinforcement

Clause 40.4:

When τ_v exceeds τ_c given in Table 19, shear reinforcement shall be provided in the following forms :

a) Vertical Stirrups

b) Bent up bars along with stirrups

c) Inclined stirrups

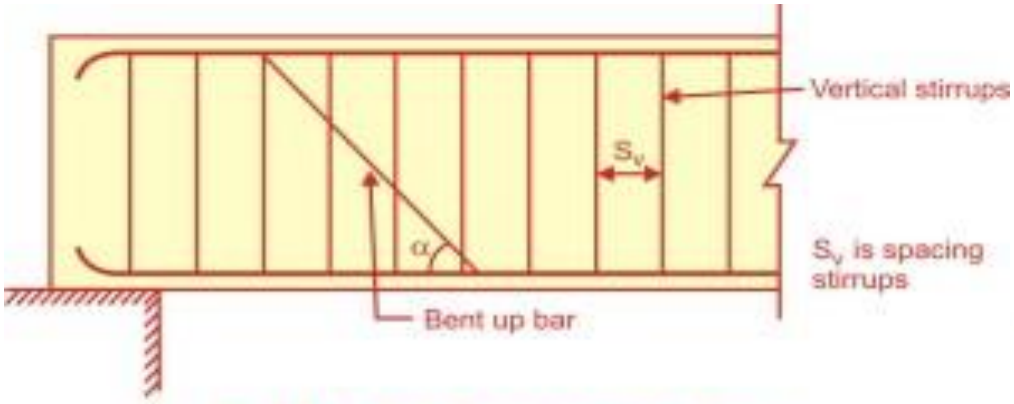


Fig. 5.6. Bent up bars alongwith stirrups.

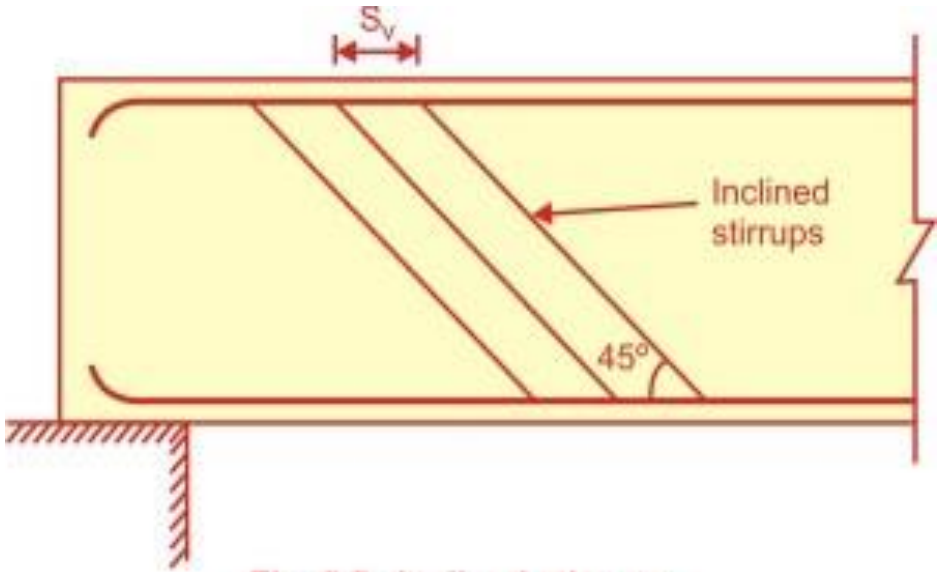
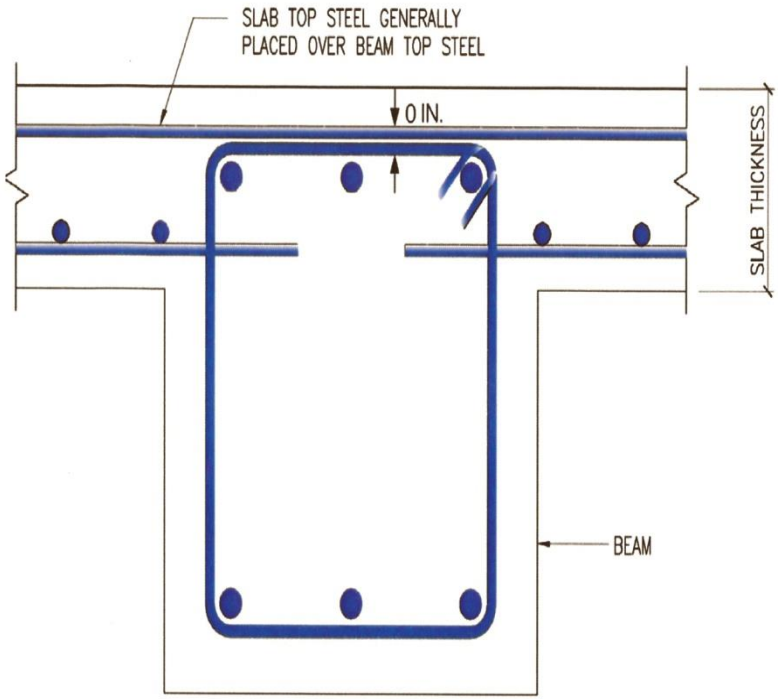
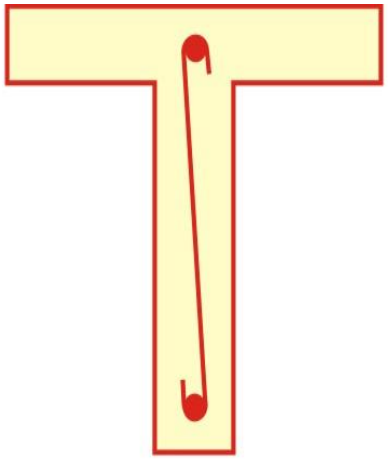
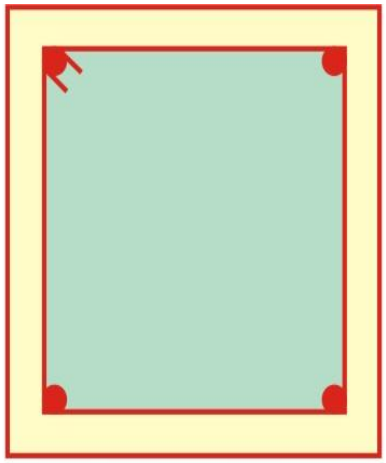


Fig. 5.7. Inclined stirrups.

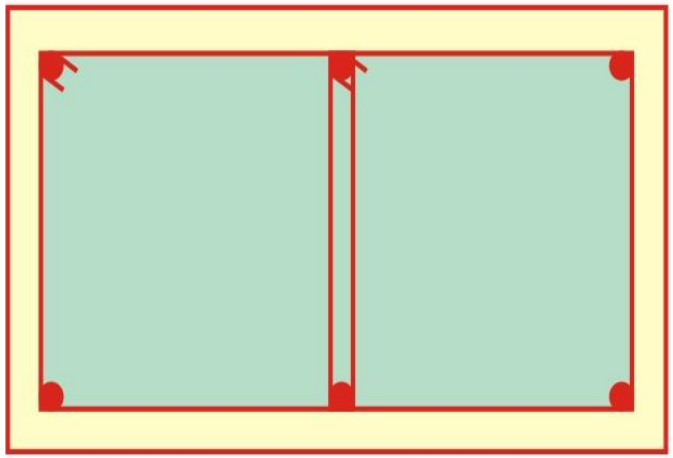




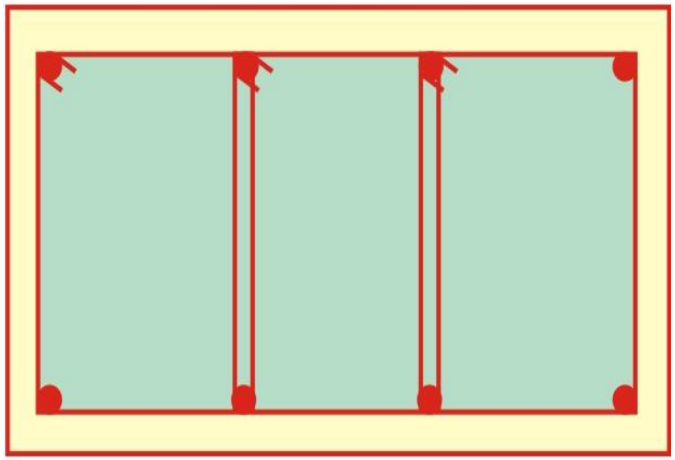
Single Legged Stirrup



Two Legged Stirrup



Four Legged Stirrup



Six Legged Stirrup

Fig. 5.5. Vertical stirrups

P1) A reinforced concrete beam 250mm wide and 430mm overall depth, is subjected to a factored shear force of 45Kn at the support. Use M20 concrete and Fe 415 steel. The beam is provided with tensile steel reinforcement of 684mm^2 at a effective cover of 50mm.

Find a) nominal shear stress b) Design shear stress and c) maximum shear stress.

Sol: <u>Given Data :</u> b =250mm, D = 430mm

d =D-Cover = 430 – 50 = 380mm.

F_{ck} = 20N/mm2, F_y = 415N/mm^2

Factored shear force = 45KN = 45x10^3N

Area of tensile reinforcement Ast = 684mm^2

Percentage of tensile reinforcement Pt = 100Ast/bd

= 100*(684)/(250*380) = 0.72

1) **Nominal Shear Stress (cl. 40.1)** = τv = Vu/bd

= 45x10^3/250*380

<u>τv = 0.474N/mm^2</u>

2) Design shear stress τ_c (table 19 IS Code)

For M20 concrete and Pt = 0.72

from table 19

Pt	M20
0.50	0.48
0.72	?
0.75	0.56

Interpolation :

$\tau_c = 0.48 + ((0.56-0.48)/(0.75-0.50))*(0.72-0.50)$

$\tau_c = 0.5504 N/mm^2$

3) Maximum shear stress Ƭc(max) (table 20)

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm²

(Clauses 40.2.3, 40.2.3.1, 40.5.1 and 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0

For M20 Concrete

Ƭc(max) = 2.8N/mm²

Ƭv < Ƭc < Ƭc(max)

P2) A reinforced concrete beam 250x400mm effective depth, is subjected to a shear force of 100KN at the support. Use M25 concrete and Fe 415 steel. The beam is provided with 2 bars of 25mm tensile reinforcement.

Find a) nominal shear stress b) Design shear stress and c) maximum shear stress.

(Practice)

P3) A reinforced concrete beam 240mm wide and 410mm effective depth, is subjected to a shear force of 30Kn at the support. Use M20 concrete and Fe 415 steel. The beam is provided with tensile steel reinforcement of 700mm^2 . Check for shear reinforcement.

Sol: <u>Given Data :</u> b =240mm, d = 410mm

F_{ck} = 20N/mm2, F_y = 415N/mm^2

Shear force = 30KN

Factored Shear force = 1.5*30 = 45KN

Area of tensile reinforcement Ast = 700mm^2

Percentage of tensile reinforcement Pt = 100Ast/bd

= 100*(700)/(240*410) = 0.711

1) **Nominal Shear Stress (cl. 40.1)** = $\tau v = Vu/bd$

$= 45x10^3/240*410$

$\tau v = 0.457 N/mm^2$

2) Design shear stress τc (table 19 IS Code)

For M20 concrete and Pt = 0.711

from table 19

Pt	M20
0.50	0.48
0.711	?
0.75	0.56

Interpolation :

$\tau c = 0.48+((0.56-0.48)/(0.75-0.50))*(0.711-0.50)$

$\tau c = 0.5475 N/mm^2$

3) Maximum shear stress Ƭc(max) (table 20)

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm²

(Clauses 40.2.3, 40.2.3.1, 40.5.1 and 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0

For M20 Concrete

Ƭc(max) = 2.8N/mm²

Since Ƭv(0.457) < Ƭc(0.5475) < Ƭc(max)(2.8)

Nominal shear reinforcement is provided.

Minimum shear reinforcement as per clause 26.5.1.6

$$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87 f_y}$$

Asv : Considering 8mm two legged stirrups

$= 2*(\prod/4)*8^2 = 100.48mm^2$

Sv = Spacing of stirrups

100.48/(240*Sv) = 0.4/(0.87*415)

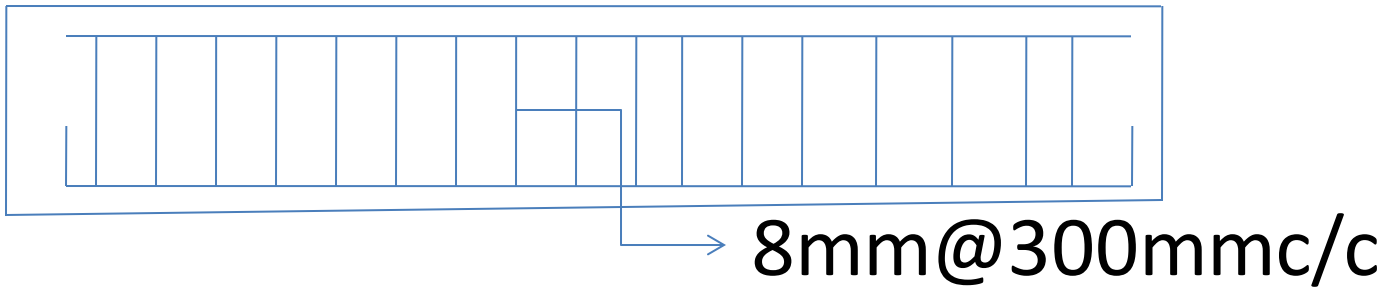
Sv = 377.89mm.

Spacing of shear reinforcement shall not exceed 0.75d or 300mm which ever is less.

0.75d = 0.75*410 = 307.5mm

300mm.

Therefore providing shear reinforcement @300mm c/c.



P4) A reinforced concrete beam 230mm wide and 450mm overall depth, is subjected to a load of 10Kn/m including self weight over a span of 6m. Use M25 concrete and Fe 415 steel. The beam is provided with 3 bars of 16mm diameter tensile steel. Check for shear reinforcement.

(Practice)

P5) A reinforced concrete beam 200mm wide and 400mm effective depth, carries a udl working load of 25Kn/m over a span of 8m. The tensile reinforcement consists of 4 bars of 25mm dia placed in two rows. Use M20 grade concrete and Fe415 steel. Design the shear reinforcement for the beam.

Sol: <u>Given Data :</u> b =200mm, d = 400mm

F_{ck} = 20N/mm2, F_y = 415N/mm^2

Shear force = wl/2 = 25*8/2 = 100Kn

Factored Shear force = 1.5*100 = 150Kn

Area of tensile reinforcement Ast =4*(∏/4)*25^2
1962.5mm^2

Percentage of tensile reinforcement Pt = 100Ast/bd

= 100*(1962.5)/(200*400) = 2.45

1) **Nominal Shear Stress (cl. 40.1)** = $\tau v = Vu/bd$

$= 150x10^3/200*400$

$\tau v = 1.875 N/mm^2$

2) Design shear stress τc (table 19 IS Code)

For M20 concrete and Pt = 2.45

from table 19

Pt	M20
2.25	0.81
2.45	?
2.5	0.82

Interpolation :

$\tau c = 0.81+((0.82-0.81)/(2.5-2.25))*(2.45-2.25)$

$\tau c = 0.818 N/mm^2$

3) Maximum shear stress Ƭc(max) (table 20)

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm²

(Clauses 40.2.3, 40.2.3.1, 40.5.1 and 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0

For M20 Concrete

Ƭc(max) = 2.8N/mm²

Since Ƭv(1.875) >Ƭc(0.818) < Ƭc(max)(2.8)

Shear reinforcement is to be designed.

4) Design shear force

$Vus = Vu - Vuc = Vu - \tau c\ bd$

$= 150x10^3 - 0.818*200*400$

$= 84560N$

5) For Vertical Stirrups :

Taking 8mm dia bar $Asv=2*(\pi/4)8^2 = 100.48mm^2$

$Vus = 0.87*Fy*Asv*d/Sv$

$84560 = 0.87*415*100.48*400/Sv$

$Sv = 171.6mm$

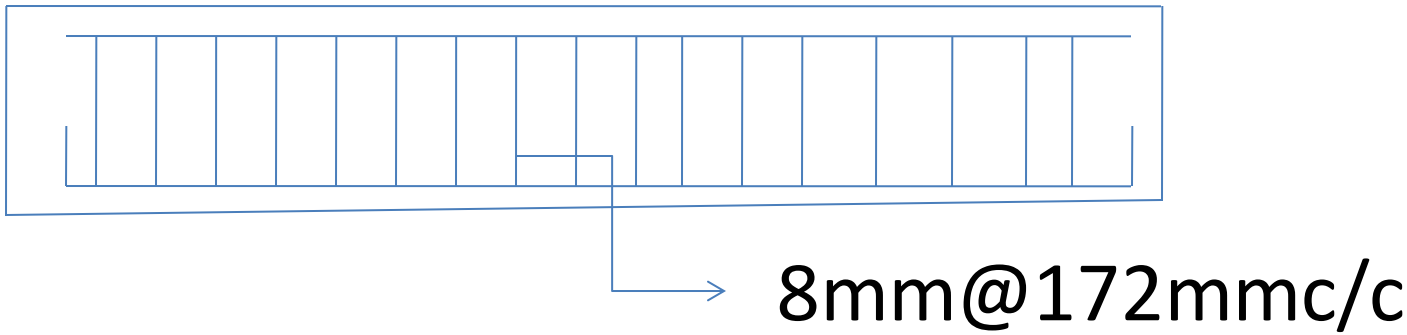
Spacing of shear reinforcement shall not exceed 0.75d or 300mm which ever is less.

0.75d = 0.75*400 = 300mm

300mm

Providing spacing of 172mm.

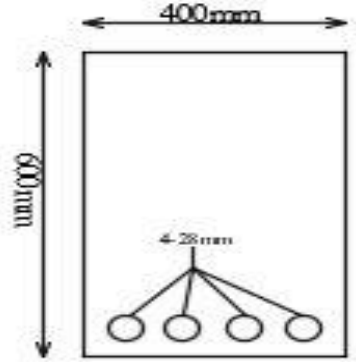
Providing 2-legged stirrups 8mm @172mm

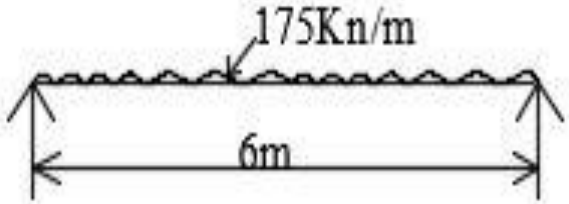


P6) A simply supported beam with clear span 6m, width 0f 400mm and overall depth of 600mm, carries a limit state load of 175Kn/m (including self weight, dead load and live load). It is reinforced with four bars of 28mm dia, tension steel which continue right into the support. Take Fck = 20N/mm^2, Fy = 250N/mm^2. Design shear reinforcement.

Sol :

Given Data :





B =400mm, D = 600mm, d = D – Cover

600 – 25 = 575mm

Fck = 20N/mm^2, Fy = 250N/mm^2

Factored or limit state load Wu = 175Kn/m

Factored shear force Vu = 175*6/2 = 525Kn

Area of tensile reinforcement Ast =4*(∏/4)*28^2

2464mm^2

Percentage of tensile reinforcement Pt = 100Ast/bd

= 100*(2464)/(400*575) = 1.07

1) **Nominal Shear Stress (cl. 40.1)** = $\tau v = Vu/bd$

$= 525x10^3/400*575$

$\tau v = 2.28 N/mm^2$

2) Design shear stress τc (table 19 IS Code)

For M20 concrete and Pt = 1.07

from table 19

Pt	M20
1.0	0.62
1.07	?
1.25	0.67

Interpolation :

$\tau c = 0.62+((0.67-0.62)/(1.25-1.0))*(1.07-1.0)$

$\tau c = 0.634 N/mm^2$

3) Maximum shear stress Ƭc(max) (table 20)

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm²

(Clauses 40.2.3, 40.2.3.1, 40.5.1 and 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0

For M20 Concrete

Ƭc(max) = 2.8N/mm²

Since Ƭv(2.28) >Ƭc(0.634) < Ƭc(max)(2.8)

Shear reinforcement is to be designed.

4) Design shear force

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

$= 525 \times 10^3 - 0.634 * 400 * 575$

$= 379180N$

5) For Vertical Stirrups :

Taking 8mm dia bar $A_{sv} = 2 * (\pi/4) 8^2 = 100.48 mm^2$

$V_{us} = 0.87 * F_y * A_{sv} * d / S_v$

$379180 = 0.87 * 250 * 100.48 * 575 / S_v$

$S_v = 33mm$ (very less spacing)

Taking 10mm dia bar Asv=2*(∏/4)10^2 =157mm^2

Vus = 0.87*Fy*Asv*d/Sv

379180 = 0.87*250*157*575/Sv

Sv = 51.78mm

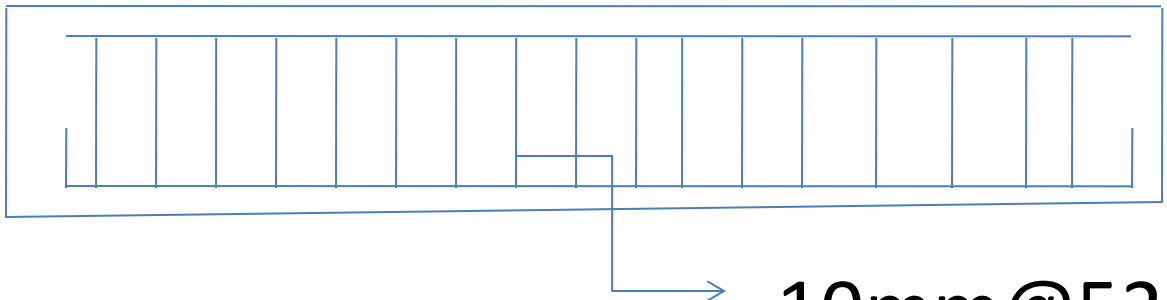
Spacing of shear reinforcement shall not exceed 0.75d or 300mm which ever is less.

0.75d = 0.75*575 = 431.25mm

300mm

Providing spacing of 52mm.

Providing 2-legged stirrups 10mm @52mm c/c


10mm@52mmc/c

BOND

One of the most important assumption in the behavior of reinforced concrete structures is that there is proper “bond” between concrete and steel.

When a R.C.C. element is loaded, the load is first borne by concrete and it is then transferred to steel reinforcement.

“The force which prevents slippage between two constituent materials is known as bond”

Mechanism of Bond Resistance

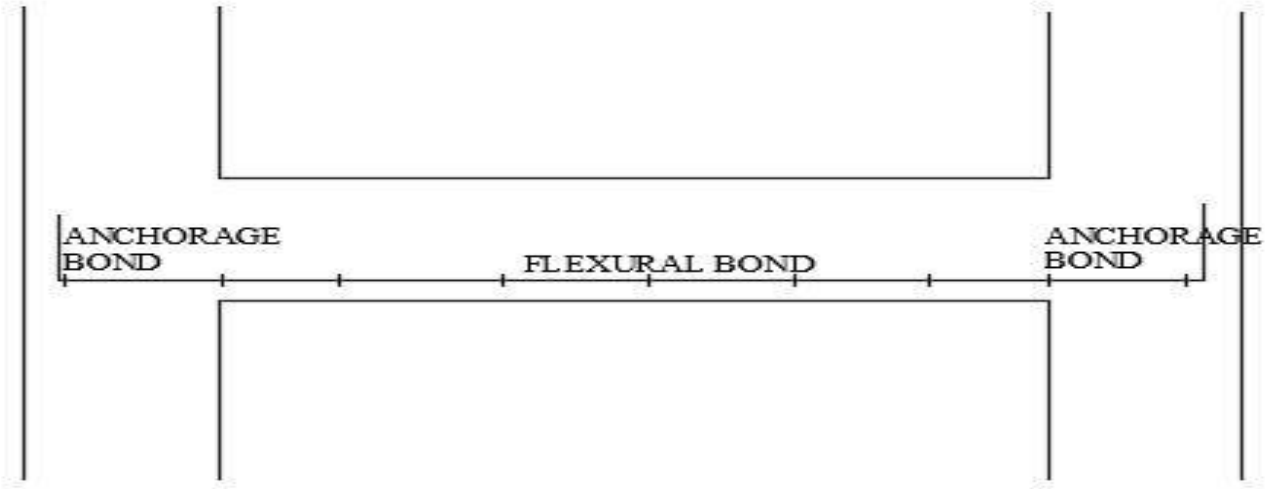
The bond between steel and concrete comprise of the following three resistances :

1) Pure adhesive resistance
2) Frictional resistance
3) Mechanical resistance, it is provided by deformed bars only.

Types of Bonds

Bond stress along the length of a reinforcing bar may be induced under two loading situations, and bonds are two types:

1) Flexural bond or local bond
2) Anchorage bond or development bond



Flexural bond or Local Bond

Flexural bond τ_{bf} is one which arises from the change in tensile force in bar, along its length.

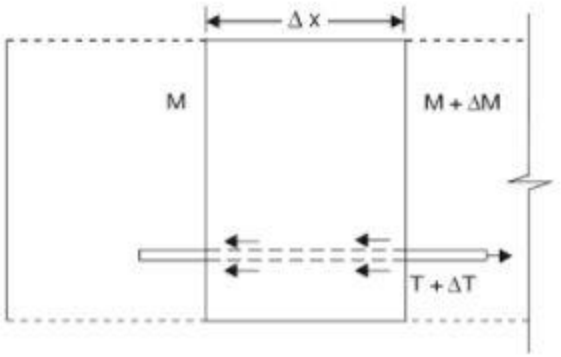
Since this occur at a particular section, flexural bond stress is known as local bond stress.

$\tau_{bf} = V/a\sum o$

a = lever arm

$\sum o$= Sum of perimeter of

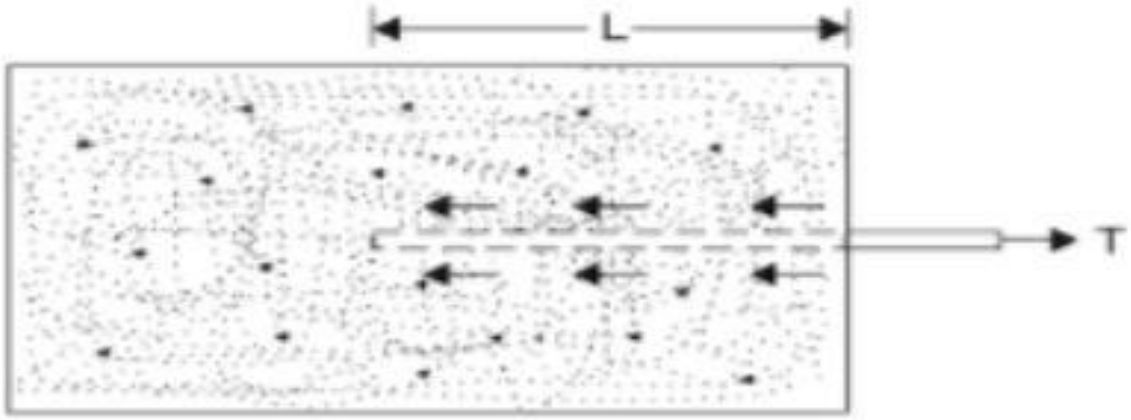
Steel bars resisting tension



Anchorage bond or Development bond

The anchorage bond τ_{bd} is that which arises over the length of anchorage provided for a bar.

The anchorage bond τ_{bd} resists the "pulling out" of the bar if it is in tension or "pushing in" of the bar if it is in compression.



26.2.1.1 Design bond stress in limit state method for plain bars in tension shall be as below:

Grade of concrete	M 20	M 25	M 30	M 35	M 40 and above
Design bond stress, τ_{bd}, N/mm^2	1.2	1.4	1.5	1.7	1.9

Development Length of bars (cl : 26.2.1)

The development length is defined as the length of the bar required to develop stress in steel through bond.

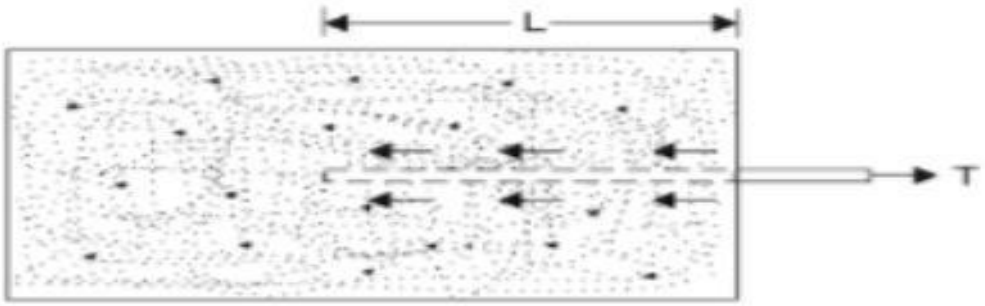
The development length Ld is given by

$$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}}$$

Ø = Nominal diameter of bar

σ_s = Stress in bar at the design load.

τ_{bd} = Design bond stress as given in 26.2.1.1

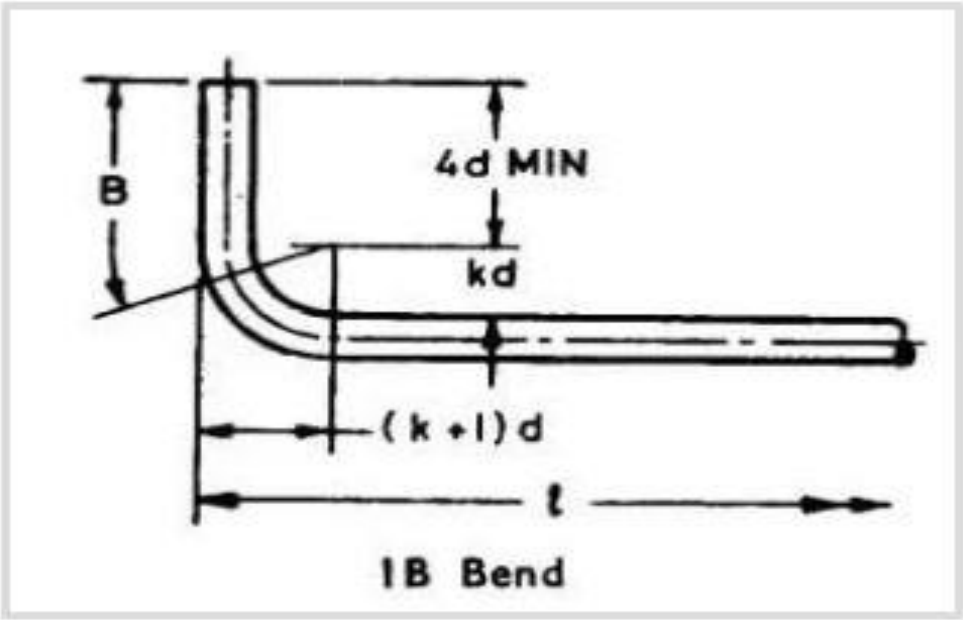


Standard Hooks and Bends for End Anchorage

- The development length required at the end of a bar is known as anchorage length.
- The space available at the end of beam is limited to accommodate the full development length Ld.
- In that case hooks and bends are provided.

Bends(26.2.2.1. – b.1)

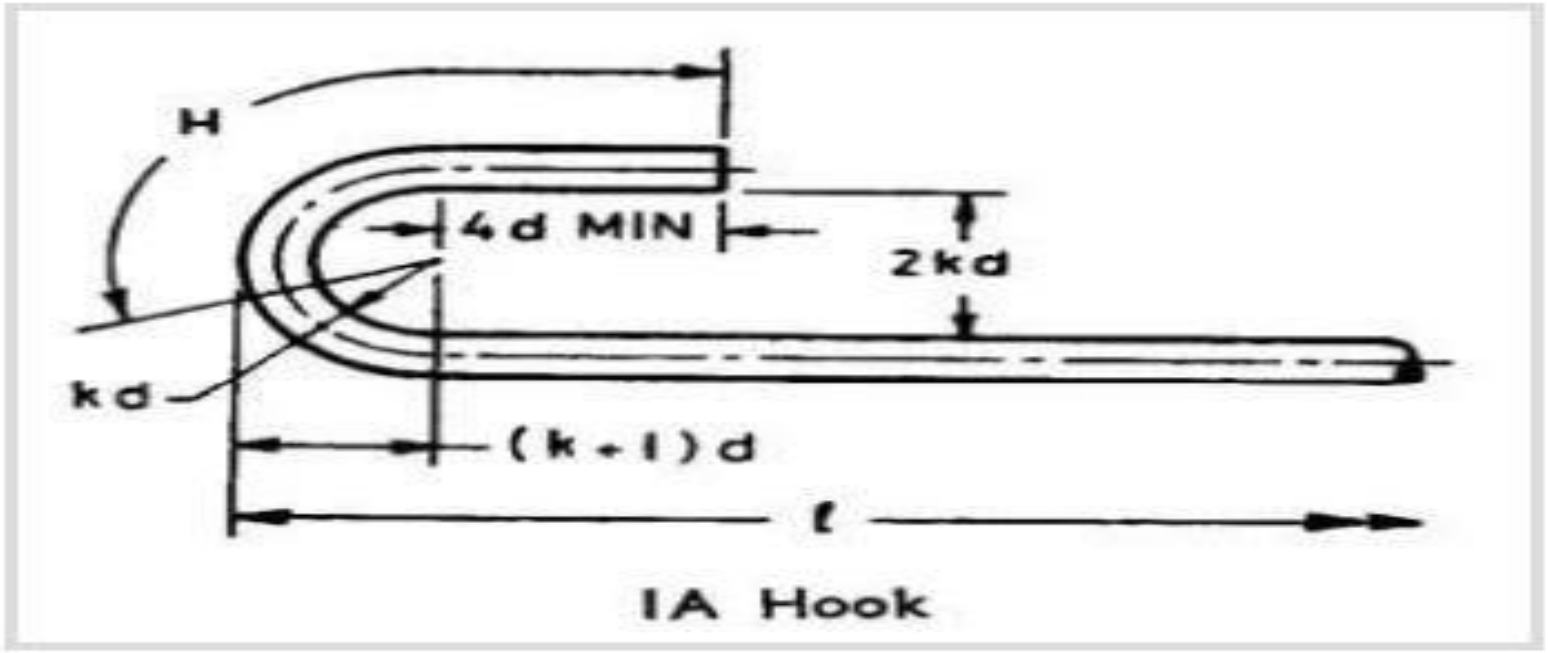
The anchorage value of bend shall be taken shall be taken as 4 times the diameter of the bar for each 45^0 bend subject to a maximum of 16times the diameter of bar.



1B Bend

Hooks (26.2.2.1. – b.2)

The anchorage value of a standard U-Type hook shall be equal to 16 times the diameter of bar.



IA Hook

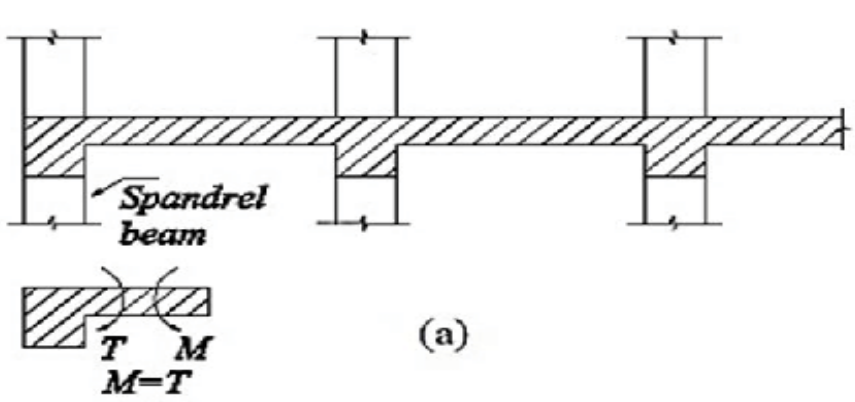
TORSION (Cl. 41)

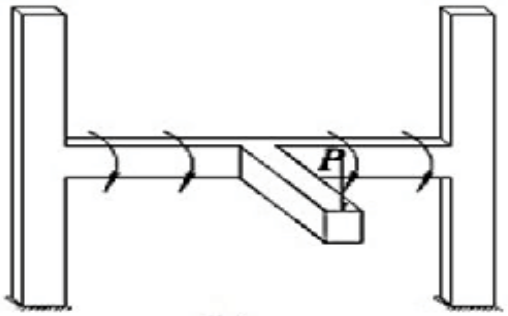
Axial loads, flexure, shear and Torsion are the basic loading situations.

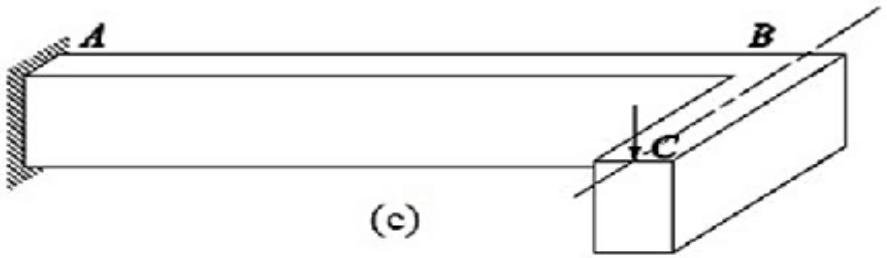
Example of Torsional loading :

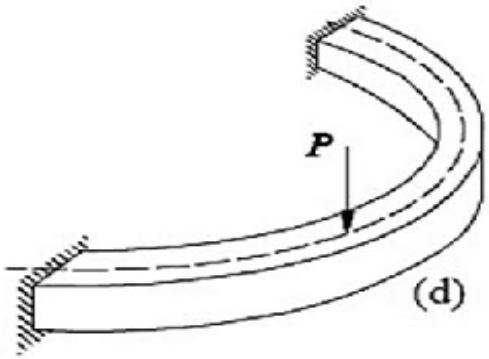
1) Peripheral beams in each floor of a multistoried building.

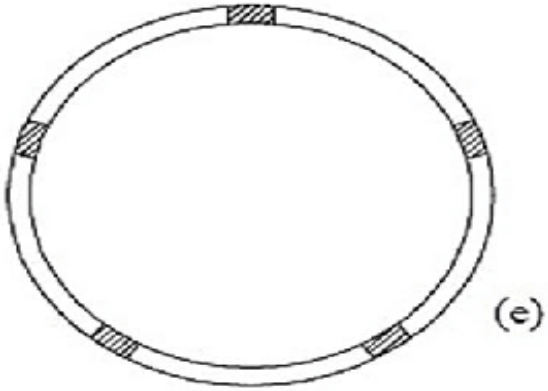
2) Ring beam at the bottom of elevated circular water tank.


Spandrel
beam
T
M
M=T
(a)

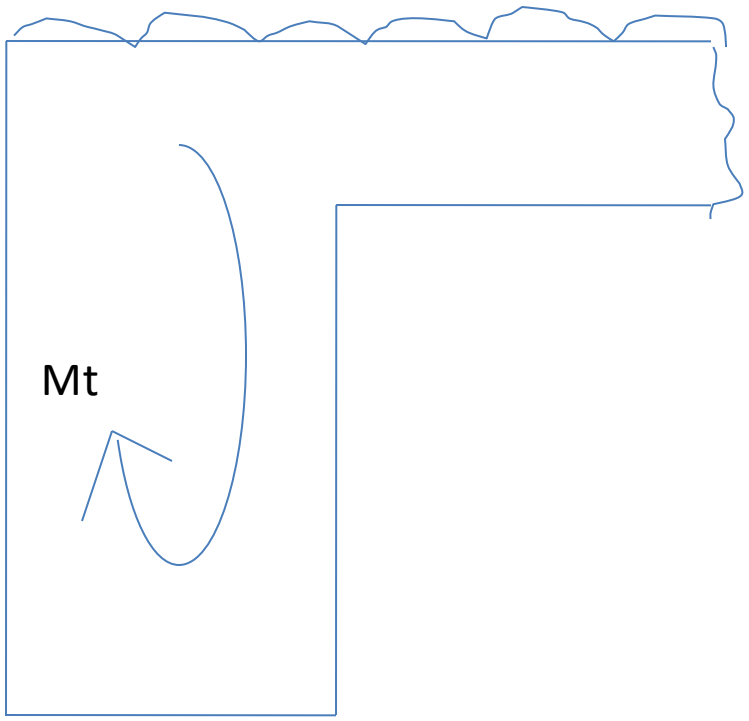

P
(b)


A
B
C
(c)


P
(d)


(e)

The figure shows L-beam, this is not symmetrically loaded. This causes torsional moment Mt. The torsional moment is resisted by the rectangular portion only.



IS Code procedure for design for torsion (Cl. 41)

1) Equivalent shear (41.3.1)

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

V_e = equivalent shear,

V_u = shear,

T_u = torsional moment, and

b = breadth of beam.

2) Equivalent nominal shear stress :

$\tau ve = Ve/bd$

3) Design shear stress τ_c (table 19 IS Code)

4) Maximum shear stress $\tau_{c(max)}$ (table 20)

5) If $\tau_{ve} < \tau_c < \tau_{c(max)}$

Minimum shear reinforcement should be provided as per as per clause 26.5.1.6.

$$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87 f_y}$$

6) If $(\tau_{ve} > \tau_c) < \tau_{c(max)}$

Longitudinal and Transverse reinforcement shall be provided in accordance with clause 41.4.

7) Longitudinal reinforcement (41.4.2)

The longitudinal reinforcement shall be provided to resist equivalent bending moment of Me1.

$$Me1 = Mu+Mt$$

Mu = Bending moment of section

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

Me1 = $$0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{bd f_{ck}} \right)$$

Aste = Ast- tensile reinf.

8) Transverse Reinforcement (41.4.3)

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

Assuming two legged stirrups.

Total transverse reinforcement shall not be less than

$$\frac{(\tau_{ve} - \tau_c) b_v s_v}{0.87 f_y}$$

P7) A rectangular section with b = 200mm and D = 350mm, is subjected to limit state shear of 80Kn and bending moment of 25Knm. Design the torsional reinforcement for torsion moment of 6Knm. Given Fck= 20N/mm^2, Fy= 250N/mm^2.

Take tensile reinforcement of 3nos -16mm dia.

Sol:

Given data : b =200mm, D = 350mm, d = D – Cover

350 – 25 = 325mm

Fck = 20N/mm^2, Fy = 250N/mm^2

Factored shear force Vu = 80Kn

Bending Moment = Mu = 25Knm

Torsional Moment = Tu = 6Knm

Area of tensile reinforcement Ast =3*(∏/4)*16^2 = 603mm^2

Percentage of tensile reinforcement Pt = 100Ast/bd

= 100*(603)/(200*325) = 0.927

1) Equivalent Shear :(41.3.1)

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

$Ve = 80x10^3 + 1.6(6x10^6/200)$

$=128000N$

2) Equivalent Nominal Shear Stress :

$\tau ve = Ve/bd = 128000/(200*325) = 1.97N/mm^2$

3) Design shear stress τc (table 19, IS Code)

For M20 concrete and Pt = 0.927

from table 19

Pt	M20
0.75	0.56
0.927	?
1.0	0.62

Interpolation :

$\tau c = 0.56+((0.62-0.56)/(1.0-0.75))*(0.927-0.75)$

$\tau c = 0.6N/mm^2$

3) Maximum shear stress Ƭc(max) (table 20)

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm²

(*Clauses* 40.2.3, 40.2.3.1, 40.5.1 *and* 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0

For M20 Concrete

Ƭc(max) = 2.8N/mm²

Since **Ƭve(1.97) >Ƭc(0.6) < Ƭc(max)(2.8)**

Longitudinal and Transverse reinforcement to be designed.

4) Eqivalent Bending Moment :

Me1 = Mu+Mt

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

= $25x10^6 + 6x10^6*(1 +350/200)/(1.7)$

= $34.705x\ 10^6$ Nmm

5) Longitudinal reinforcement: (41.4.2)

Me1 = $0.87\ f_y\ A_{st}\ d\left(1 - \frac{A_{st}\ f_y}{bd\ f_{ck}}\right)$

$34.705x\ 10^6 = 0.87*250*Ast* 325(1 – (Ast*250)/(200*325*20))$

Ast =

Aste = Ast – 603 =

6) Transverse reinforcement : (41.4.3)

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

Taking 2- legged 8mm dia

Asv = $2*(\prod/4)8^2 = 100.5mm^2$

$$100.5 = \frac{(6x10^6*Sv)}{200*325*0.87*250} + \frac{80x10^3*Sv}{2.5*325*0.87*250}$$

Sv = 114.73mm

But Sv shall not be less than

Asv = $$\frac{(\tau_{ve} - \tau_c) b . s_v}{0.87 f_y}$$

$100.5 = ((1.97 – 0.6)*200*Sv)/(0.87*250)$

$Sv = 80mm$

$114.73mm > 80mm$ OK

Providing 2-legged 8mm dia transverse reinforcement @ 115mm c/c

P7) Determine the reinforcement required for a rectangular beam section with the following data : Width of section 300mm, depth of section 500mm Factored bending moment 65Knm, Factored torsional moment 40Knm, Factored shear force 70Kn, use M20 grade concrete and Fe415 steel. Tensile reinforcement consist of 4nos 16mm dia bars.

(Practice)

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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: DESIGN OF REINFORCED CONCRETE STRUCTURES

SUBJECT CODE: 20CE5PC02

FACULTY NAME: MOHAMMED MUNEERUDDIN KHAN

SYLLABUS:

UNIT – III

Design of Two-way slabs with different end conditions, one-way slab, and continuous slab Using I S Coefficients - Design of dog-legged staircase – Limit state design for serviceability for deflection, cracking and codal provisions.

UNIT- III

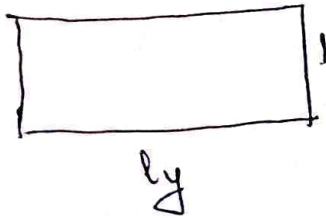
Slab:-

A Slab is a Structural element with small thickness compared to its length and breadth. The loads from slabs are transferred to beams.

Types of Slab:-

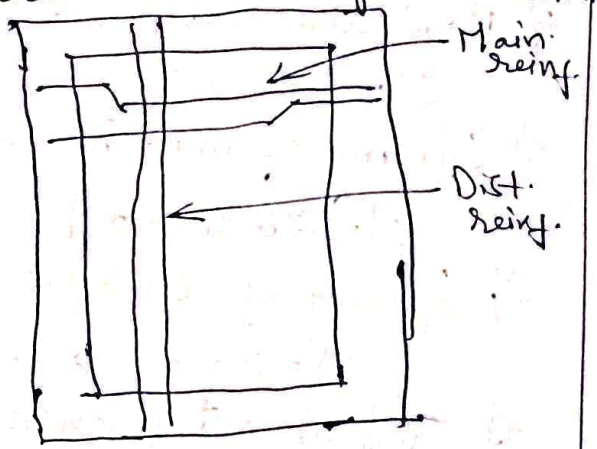
1) one-way slab
2) Two-way slab
3) Cantilever slab
4) Continous slab
5) Flat slab
6) Ribbed slab

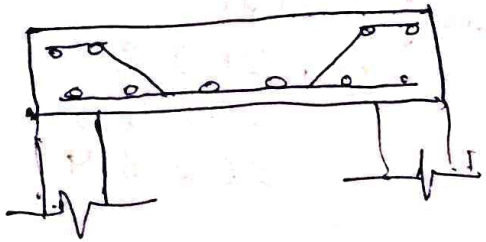
Explain about one-way slab:



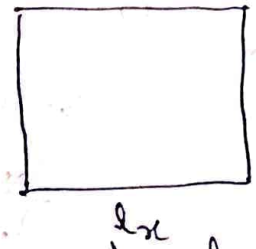
-when the ratio $\frac{ly}{lx} > 2$ it is called as one-way slab.

- The whole load is carried by short span.
- The slab bends and deflects in the direction lx only.
- The main reinforcement is provided in short span direction

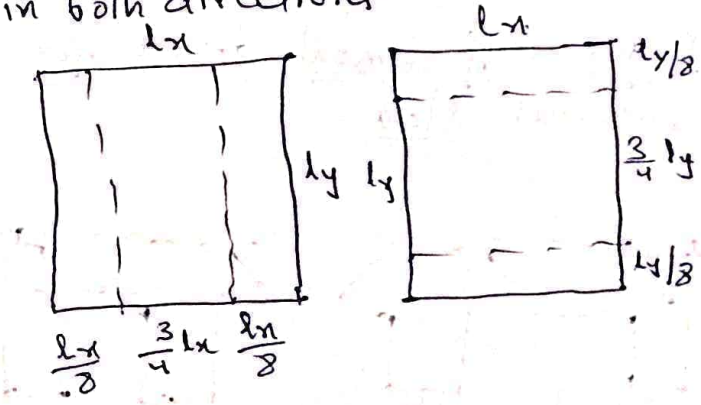


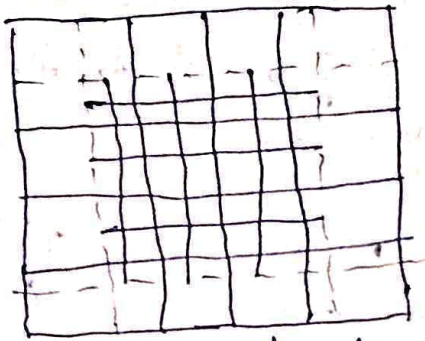


Explain about Two way Slab



- when the ratio $\frac{ly}{lx} \leq 2$ it is called as two-way slab.
- The load is carried in both directions.
- The main reinforcement is provide in both directions.



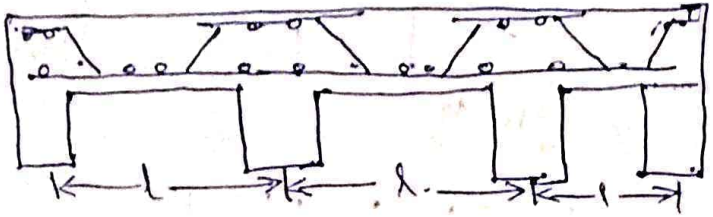


Main steel in both direction.

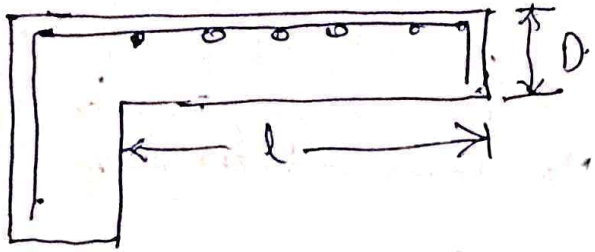
Difference between one-way & Two-way Slab

one way slab	Two way slab
$\frac{l_y}{l_x} > 2$	$\frac{l_y}{l_x} \leq 2$
load acts as shorter direction	load acts on both direction
Main reinforcement is along shorter direction	Main reinf. along both direction

Draw reinforcement details for Continous slab:-



Draw reinforcement details for Cantilever slab:-



IS Code guidelines for slabs

1) Effective Span.
   Clause 22.2
2) Control of deflection.
   Cl. 23.2
   23.2.1
3) Minimum Reinforcement.
   Cl. 26.5.2.1
4) Maximum diameter.
   Cl. 26.5.2.2
5) Spacing of main reinforcement
   Cl. 26.3.3.b.1
6) Spacing of distribution reinf.
   Cl. 23.3.3.b.2.
7) Curtailment of tension reinforcement.
   Cl. 26.2.3
   26.2.3.3
   26.2.3.4

Types of Deflection:-

1) Short term deflection.
2) long term deflection
   a) Shrinkage of Concrete
   b) Creep of concrete

Methods for Crack Control:-

Cracks can be kept in permissible limit by proper detailing of reinforcement.

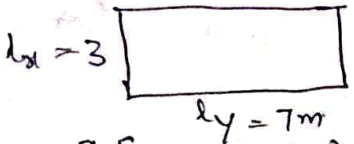
a) Spacing of reinforcement
b) Side face reinforcement
c) Percentage of Steel

Calculation of Deflection Annex 'C' IS Code 456

Calculation of Crack width. Annex F' IS Code 456.

(Q1) Design a slab for a room of internal dimensions 3x7m. The slab is resting on walls 250mm wide. Take live load 2KN/m² and floor finish 1.5KN/m². The slab is simply supported. Use M20 grade concrete and Fe415 steel.

Sol:- Given Data:-



f.f. = 1.5KN/m²
Live load = 2KN/m².
fck = 20 N/mm²
fy = 415 N/mm².
width of Support = 250mm

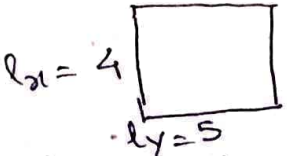
Type of slab.

$\frac{l_y}{l_x} = \frac{7}{3} = 2.33 > 2$

It is one way slab

(Q2) Design a R.C. slab for a room of clear size 4m x 5m. The superimposed load on slab is 3KN/m² and weight of finishes is 1KN/m². The corner of the slab are restrained. Width of support is 250mm. Use M25 Concrete & Fe415 steel.

Sol:- Given Data



Superimposed or live load = 3KN/m²
floor finish = 1KN/m²
Corners are restrained.
width of Support = 250mm.
fck = 25 N/mm², fy = 415N/mm²

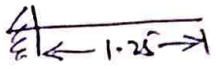
Type of slab.

$\frac{l_y}{l_x} = \frac{5}{4} = 1.25 \leq 2$

It is two way slab

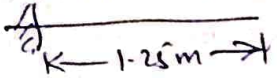
(Q3) Design a Cantilever slab for a overhang of 1.25m. Take live load of 2KN/m², floor finish of 1KN/m². Use M20 grade concrete and Fe415 steel.

Sol:- Given Data



Live load = 2KN/m².
floor finish = 1KN/m².
fck = 20N/mm²
fy = 415N/mm²

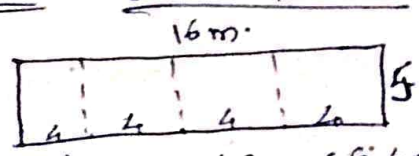
Type of slab



Cantilever slab.

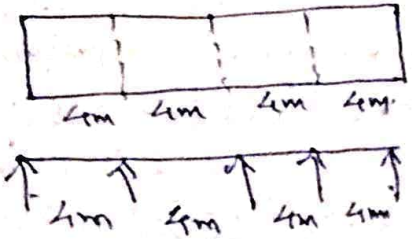
(Q4) Design a Continous R.C slab for a hall of 5x16m. The slab is resting on beam each 230mm wide. Design the slab for live load of 2KN/m² and roof finishing of 1.5KN/m². Use M20 grade concrete & Fe415 steel.

Sol:- Given Data



Dividing in 4 spans of 4m.
Live load = 2KN/m²
Roof/floor finish = 1.5KN/m²
fck = 20N/mm², fy = 415N/mm²

Type of slab.



Continous slab.

①

Effective depth:-

$\frac{l}{d} = 20$ (for simply supported)

$\frac{3000}{d} = 20$

$\therefore d = \frac{3000}{20} = 150mm$

$D = d + cover$

$= 150 + 20 = 170mm$

Effective Span:

$l_e = l_x + d$

$= 3000 + 150$

$= 3150mm$

$= 3.15m$

Load Calculation:

Dead load:-

1) Self wt. of slab $= bD \times 25$

$= 1 \times 0.17 \times 25$

$= 4.25 KN/m$

2) Floor finish $= 1.5 KN/m$

Total Dead load $DL = 4.25 + 1.5$

$= 5.75 KN/m$

Live or Superimposed load:-

$LL = 2KN/m$

Total load $= DL + LL$

$= 5.75 + 2 = 7.75 KN/m$

Factored load $= 1.5 \times 7.75 = 11.625 KN/m$

②

Effective depth:-

$\frac{l}{D} = 28$ (for two ways slab)

$\frac{4000}{D} = 28$

$D = \frac{4000}{28} = 142.85 \simeq 145mm$

$d = D - Cover$

$= 145 - 20 = 125mm$

Effective Span:

$l_{ex} = l_x + d$

$= 4 + 0.125 = 4.125m$

$l_{ey} = l_y + d$

$= 5 + 0.125 = 5.125m$

Load Calculation:

Dead load:-

1) Self wt. of slab $= bD \times 25$

$= 1 \times 0.145 \times 25$

$= 3.625 KN/m$

2) Floor finish $= 1 KN/m$

Total Dead load $= DL = 3.625 + 1$

$= 4.625 KN/m$

Live or Superimposed load:-

$LL = 3 KN/m$

Total load $= DL + LL$

$= 4.625 + 3$

$= 7.625 KN/m$

Factored load $= 1.5(DL + LL)$

$= 1.5(7.625)$

$= 11.4375 KN/m$

③

Effective depth:-

$\frac{l}{d} = 7$ (for Cantilever slab)

$\frac{1250}{d} = 7$

$d = \frac{1250}{7} = 178.57mm \simeq 180mm$

$D = d + cover$

$= 180 + 20 = 200mm$

Effective Span:

$l_e = l + d$

$= 1.25 + 0.18$

$= 1.43m$

Load Calculation

Dead load:-

1) Self wt. of slab $= bD25$

$= 1 \times 0.2 \times 25$

$= 5 KN/m$

2) Floor finish $= 1 KN/m$

Total Dead load $= DL = 6 KN/m$

Live or Superimposed load:-

$LL = 2 KN/m$

Total load $= DL + LL$

$= 6 + 2 = 8 KN/m$

Factored load $= 1.5(DL + LL)$

$= 1.5(8) = 12 KN/m$

④

Effective depth:-

$\frac{l}{d} = 26$ (for Continuous slab)

$\frac{4000}{d} = 26$

$d = \frac{4000}{26} = 153.84mm \simeq 155mm$

$D = d + cover$

$= 155 + 20 = 175mm$

Effective Span:

$l_e = l + d$

$= 4 + 0.155$

$= 4.155m$

Load Calculation

Dead load:-

1) Self wt. of slab $= bD25$

$= 1 \times 0.175 \times 25$

$= 4.375 KN/m$

2) Floor Finish $= 1.5 KN/m$

Total Dead load $= DL = 5.875 KN/m$

Live or Superimposed load:-

$LL = 2 KN/m$

Total load $= DL + LL$

$= 5.875 + 2$

$= 7.875 KN/m$

Factored load $= 1.5(DL + LL)$

$= 1.5(7.875) = 11.8125 KN/m$

①

Bending Moment

$$M_u = \frac{W_u l_{ex}^2}{8}$$

$$= \frac{11.625 \times 3.15^2}{8}$$

$$= 14.42 \text{ KNm}$$

$$= 14.42 \times 10^6 \text{ Nmm}$$

Check for depth.

$$M_u = Q f_{ck} b d^2$$

$$14.42 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$$

$$d = 72.3 < 150 \text{ mm}$$

depth is ok.

Area of Main Steel

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

$$14.42 \times 10^6 = 0.87 \times 415 A_{st} 150 \left(1 - \frac{415 A_{st}}{20 \times 1000 \times 150}\right)$$

$A_{st} =$

②

Bending Moment

$$M_{ux} = \alpha_x W_u l_{ex}^2$$

$$M_{uy} = \alpha_y W_u l_{ex}^2$$

α_x, α_y, B.M. Coefficient from Table 26.

$\alpha_x = 0.072, \alpha_y = 0.056$

$$M_{ux} = 0.072 \times 11.4375 \times 4.125^2$$

$$= 14.01 \text{ KNm}$$

$$= 14.01 \times 10^6 \text{ Nmm}$$

$$M_{uy} = 0.056 \times 11.4375 \times 4.125^2$$

$$= 10.898 \text{ KNm}$$

$$= 10.898 \times 10^6 \text{ Nmm}$$

Check for depth.

$$M_u = Q f_{ck} b d^2$$

$$14.01 \times 10^6 = 0.138 \times 25 \times 1000 \times d^2$$

$$d = 63.72 < 125 \text{ mm}$$

depth is ok.

Area of Main Steel x-direction

$$14.01 \times 10^6 = 0.87 \times 415 A_{stx} 125 \left(1 - \frac{415 A_{stx}}{25 \times 1000 \times 125}\right)$$

$A_{stx} =$

③

Bending Moment

$$M_u = \frac{W_u l_e^2}{2}$$

$$= \frac{12 \times 1.43^2}{2}$$

$$= 12.269 \text{ KNm}$$

$$= 12.269 \times 10^6 \text{ Nmm}$$

Check for depth

$$M_u = Q f_{ck} b d^2$$

$$12.269 \times 10^6 = 0.138 \times 20 \times 1000 d^2$$

$$d = 66.673 < 180 \text{ mm}$$

depth is ok.

Area of Main Steel

$$12.269 \times 10^6 = 0.87 \times 415 A_{st} 180 \left(1 - \frac{415 A_{st}}{20 \times 1000 \times 180}\right)$$

$A_{st} =$

④

Bending Moment

$$M_u = \frac{W_u l_e^2}{10}$$

$$= \frac{11.8125 \times 4.15^2}{10}$$

$$= 20.393 \text{ KNm}$$

$$= 20.393 \times 10^6 \text{ Nmm}$$

Check for depth.

$$M_u = Q f_{ck} b d^2$$

$$20.393 \times 10^6 = 0.138 \times 20 \times 1000 d^2$$

$$d = 85.96 < 155 \text{ mm}$$

depth is ok.

Area of Main Steel

$$20.393 \times 10^6 = 0.87 \times 415 \times A_{st} 155 \left(1 - \frac{415 A_{st}}{20 \times 1000 \times 155}\right)$$

$A_{st} =$

(1)

Area of distribution steel

$$Ast_d = \frac{0.12}{100} bD$$

$$= \frac{0.12}{100} \times 1000 \times 170$$

$$= 204 mm^2$$

Spacing of Main Steel

$$S = \frac{1000 \times A\phi}{Ast\ main}$$

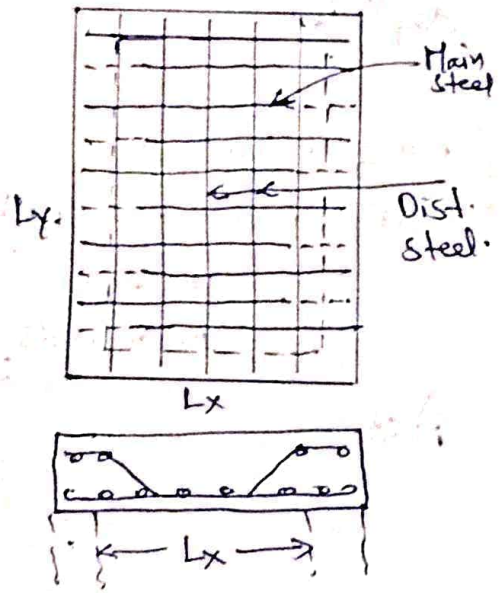
Spacing should be less than 3d or 300mm.

Spacing of distribution steel

$$S = \frac{1000 \times A\phi}{Ast_d}$$

spacing should be less than 5d or 450mm.

Reinf. details :-



(2)

Area of Main steel Y-direction

$$10.898 \times 10^6 = 0.87 \times 415 Asty\ 125 \left(1 - \frac{415\ Asty}{20 \times 1000 \times 125}\right)$$

$$Asty =$$

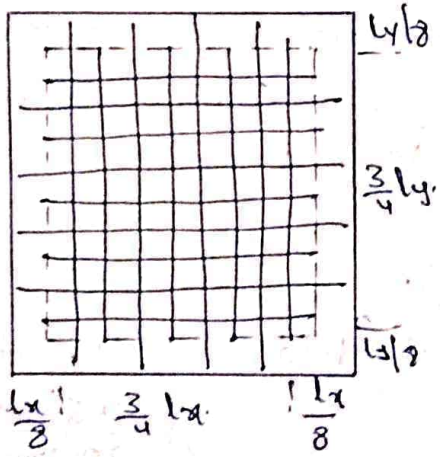
Spacing of Main steel x-direct

$$S = \frac{1000 A\phi}{Astx}$$

Spacing of Main steel Y-direction

$$S = \frac{1000 A\phi}{Asty}$$

Spacing should ~~not~~ be less than 3d or 300mm.



(3)

Area of distribution steel

$$Ast_d = \frac{0.12}{100} bD$$

$$= \frac{0.12}{100} \times 1000 \times 200$$

$$= 240 mm^2$$

Spacing of Main steel

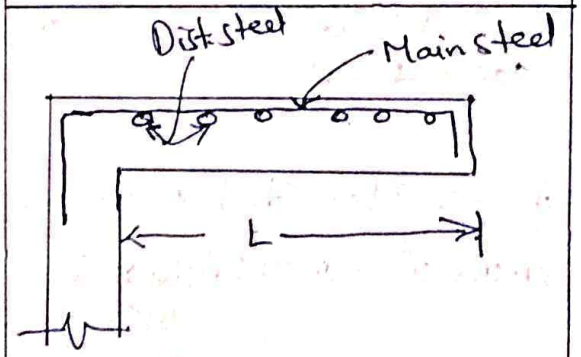
$$S = \frac{1000 A\phi}{Ast\ main}$$

Spacing should be less than 3d or 300mm.

Spacing of distribution steel

$$S = \frac{1000 A\phi}{Ast_d}$$

Spacing should be less than 5d or 450mm.



(4)

Area of distribution steel

$$Ast_d = \frac{0.12}{100} bD$$

$$= \frac{0.12}{100} \times 1000 \times 175$$

$$= 210 mm^2$$

Spacing of Main steel

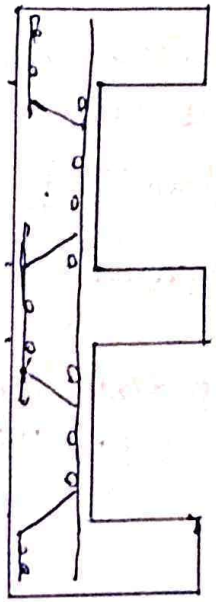
$$S = \frac{1000 A\phi}{Ast\ main}$$

Spacing should be less than 3d or 300mm.

Spacing of distribution steel

$$S = \frac{1000 A\phi}{Ast_d}$$

Spacing should be less than 5d or 450mm.



Que Explain about staircase.

Answer Staircase is an important structural element that provide access for the various floors of the building.

The staircase consists of a series of steps with landings at appropriate intervals.

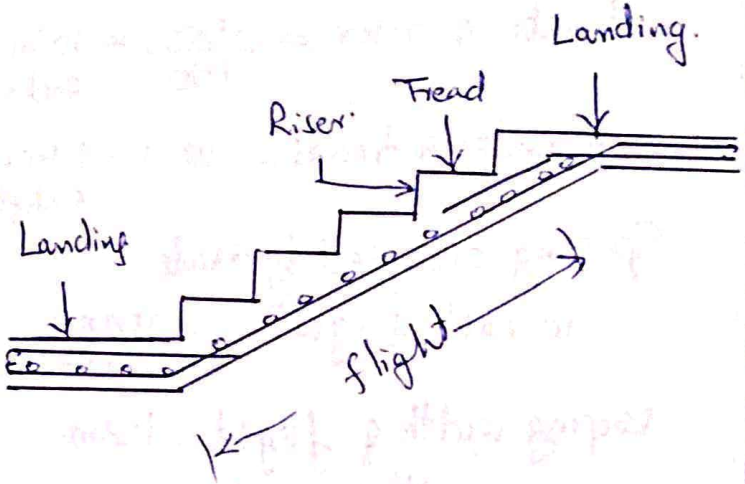
The width of staircase depends upon the type of building. It varies from 1m in residential building to 2m for public buildings.

The length of staircase situated between two landings is called flight.

The number of steps in a flight will depend on the rise and tread of the steps.

Rise varies from 150 – 300mm.

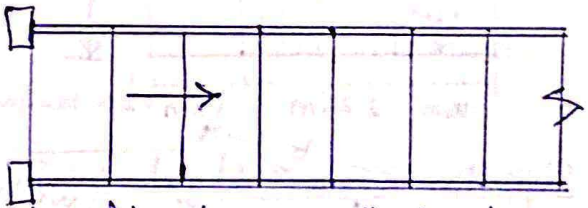
Tread varies from 200 – 300mm.



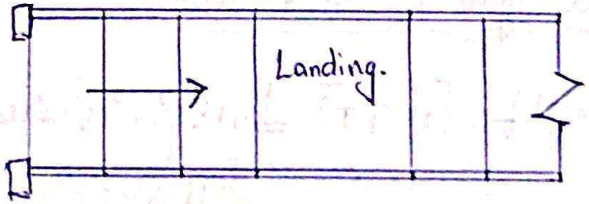
Que what are the different types of stairs?

Answer:-

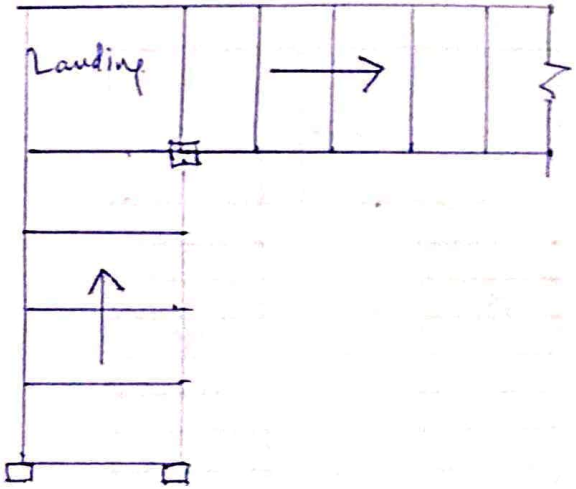
1) Straight stairs without Landing:-



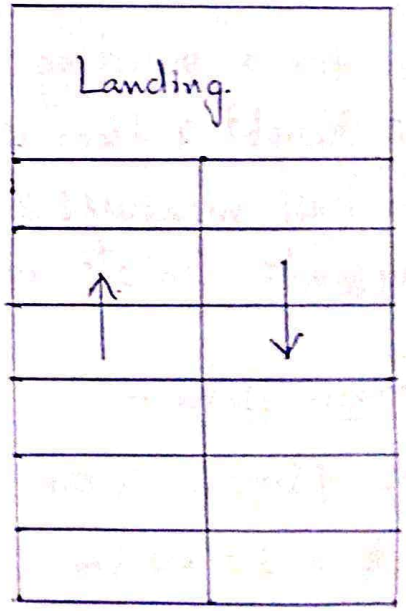
2) Straight stairs with Landing:-



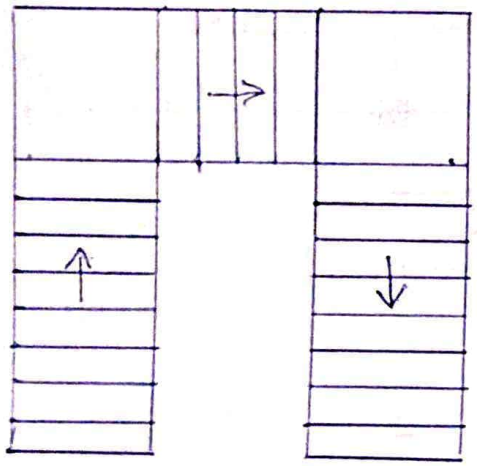
3) Quater turn Stairs:-



4) Dog-legged Stairs:-



5) Open well Stairs :-



Problem:- Design a dog legged staircase for a building in which the height of floor is 3.0m. The stair hall measures 2.5×4.5m. Use M20 grade Concrete and Fe415 steel.

Sol:- Given Data:-

Ht. of floor = 3.0m.

Stair Hall = 2.5×4.5m.

f_{ck} = 20 N/mm²

f_y = 415 N/mm².

Taking Riser = R = 150mm, Tread = T = 250mm.

Height of each flight = $\frac{3}{2}$ = 1.5m.

Number of riser = $\frac{1500}{150}$ = 10 in each flight

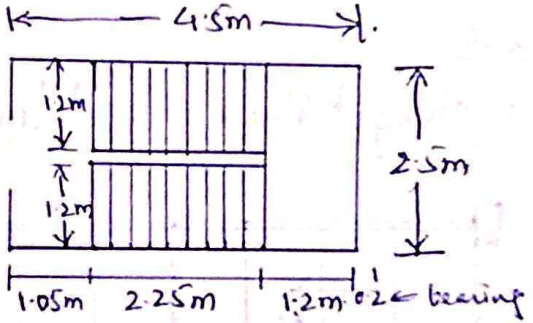
Number of treads = 10 − 1 = 9 Nos in each flight

Spacing occupied by treads in each flight = 250×9 = 2250mm.

Keeping width of flight = 1.2m.

Width of landing = 1.2m

Space for passage = 4.5 − 2.25 − 1.2 = 1.05m.



Effective Span of flight = 2.25 + 1.2 + $\frac{0.2}{2}$ = 3.55m.

Weight Calculation:-

Weight of Slab on horizontal area W_1

$= 0.15 \times 25 \times \frac{\sqrt{R^2+T^2}}{T}$

$= 0.15 \times 25 \times \frac{\sqrt{0.15^2+0.25^2}}{0.25}$

W_1 = 4.37 KN/m² ≃ 4.4 KN/m²

Dead wt. of Step for 1m. W_2

$= \frac{1}{2} \times 0.15 \times 0.25 \times 25 \times \frac{1.}{0.25}$

W_2 = 1.875 KN/m²

Total loads:

W_1 = 4.4 KN/m²

W_2 = 1.875 KN/m².

floor finish → 0.1 KN/m².

Live load = 3 KN/m².

9.375 KN/m².

Factored load = W_u = 1.5×9.375 = 14 KN/m².

Bending Moment:-

$M_u = \frac{W_u l^2}{8}$

$= \frac{14 \times 3.55^2}{8}$

≃ 22 KNm.

Check for depth:-

$M_u = Q f_{ck} b d^2$

$22 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

$\therefore d = 89.3mm$

$\therefore$ Provided Depth of 150mm ok.

Area of steel Reinforcement:-

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

$$22 \times 10^6 = 0.87 \times 415 A_{st} 150 \left[1 - \frac{A_{st} \times 415}{1000 \times 150 \times 20}\right]$$

$$406 = A_{st} - 1.383 \times 10^{-4} A_{st}^2$$

$$1.383 \times 10^{-4} A_{st}^2 - A_{st} + 406 = 0$$

$$A_{st}^2 - 7228.9 A_{st} + 2934939.76 = 0$$

Solving.

$A_{st} = 432 mm^2$

Using 8mm ϕ bars.

Number of bars in 1.2m width.

$$= \frac{432}{\frac{\pi}{4} \times 8^2} = 8.598 \simeq 9 \text{ bars}$$

Distribution Reinforcement:-

$$A_{st(dist)} = \frac{0.12}{100} bD$$

$$= \frac{0.12}{100} \times 1000 \times 150$$

$$= 180 mm^2.$$

Using 6mm dia bars.

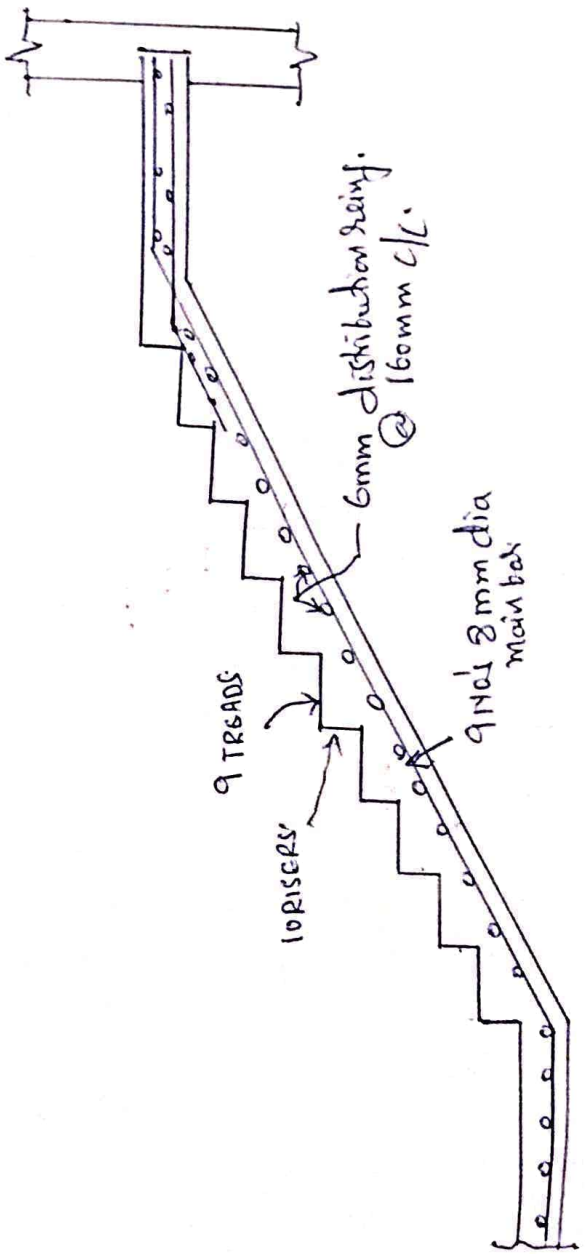
Spacing of Distribution steel.

$$A_{s(dist)} = \frac{A\phi \times 1000}{A_{st(dist)}}$$

$$= \frac{\frac{\pi}{4} \times 6^2 \times 1000}{180}$$

$$= 157mm \simeq 160mm \text{ c/c}$$

Reinforcement Details:-



TEEGALA KRISHNA REDDY ENGINEERING COLLEGE
(UGC-Autonomous)
Approved by AICTE, Affiliated by JNTUH, Accredited by NAAC- 'A' Grade
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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: DESIGN OF REINFORCED CONCRETE STRUCTURES

SUBJECT CODE: 20CE5PC02

FACULTY NAME: MOHAMMED MUNEERUDDIN KHAN

SYLLABUS:

UNIT – IV

Design of compression members - Short Column - Columns with axial loads, uni-axial and bi-axial bending – Use of design charts- long column – Design of long columns - I S Code provisions.

UNIT-IV.

(Q) Define Column.

A) A vertical member Carrying Compressive load is called as column.

A Column or Strut is a compression member, the effective length of which exceeds three times the least lateral dimension.

$l_{eff} > 3b$ (Cl. 25.1.1)

(Q) Explain about different Classification of Columns.

A) 1) Classification based on Slenderness ratio (25.1.2)

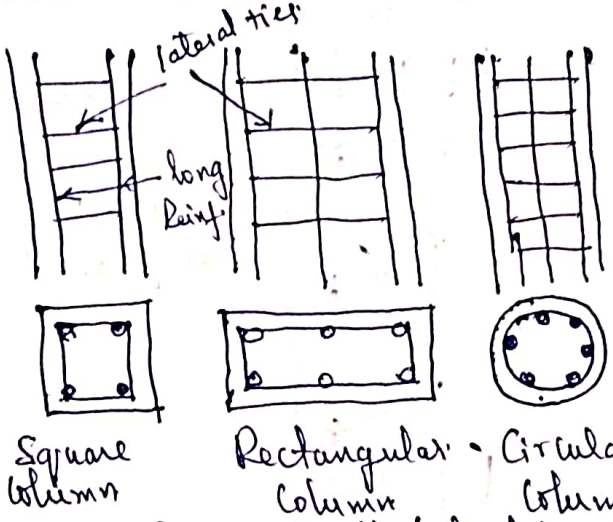
Short column: A column is considered short when when Slenderness ratio $\frac{l_{ex}}{D}$ and $\frac{l_{ey}}{b}$ are less than 12

$\frac{l_{ex}}{D} \& \frac{l_{ey}}{b} < 12.$

Long Column :- A Column is considered long when Slenderness ratio $\frac{l_{ex}}{D}$ and $\frac{l_{ey}}{b} > 12.$

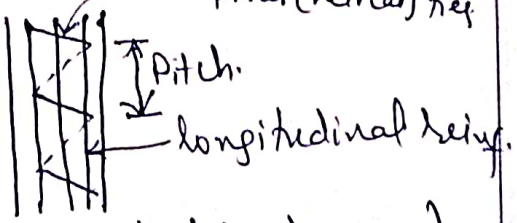
2) Classification based on Shape and Reinforcement

2.1) Columns with longitudinal steel and lateral ties



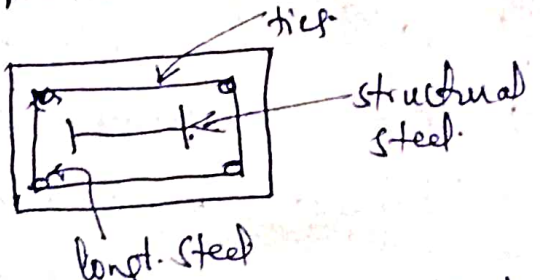
(Columns with lateral ties)

2.2) Columns with longitudinal steel & Spiral (helical) ties



(Columns with helical Reinf)

3) Composite Columns



Composite Columns are Combination of longitudinal steel, structural steel and ties.

4) Classification based on loading.

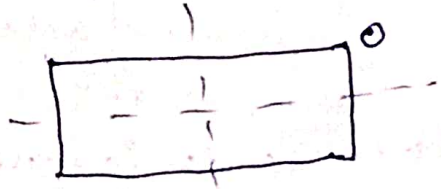
4.1) Axially loaded Column.



4.2) Axially loaded uniaxial bending.



4.3) Axially loaded biaxial bending.



Q3)

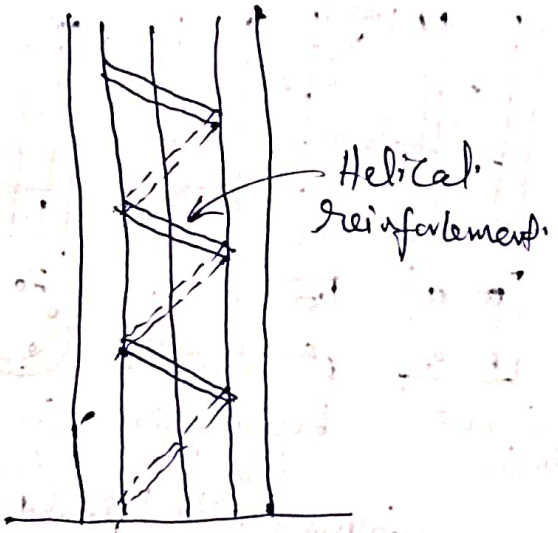
IS CODE 456: 2000 Columns Specifications :-

	Clause.	Details
A)	26.5.3 -	Columns
	26.5.3.1 -	longitudinal reinforcement
	26.5.3.2 -	Transverse reinforcement.
	39.1 -	Assumptions.
	39.2 -	Minimum eccentricity
	39.3 -	Short axially loaded in compression.
	39.4 -	Compression members with helical reinf.
	39.5 -	Members subjected to combined axial load & Uniaxial bending.
	39.6 -	Members subjected to combined axial load and biaxial bending
	39.7 -	Slender compression members.
	Annex E -	Effective length of column.

Q4) Explain about columns with helical reinforcement.

Ans: The strength of compression members with helical reinf. shall be taken 1.05 times the strength of similar member with lateral ties.

$$P = 1.05(0.4 f_{ck} A_c + 0.6 f_y A_{sc})$$



Q5) Explain the Design of column using interaction charts (SP chart)

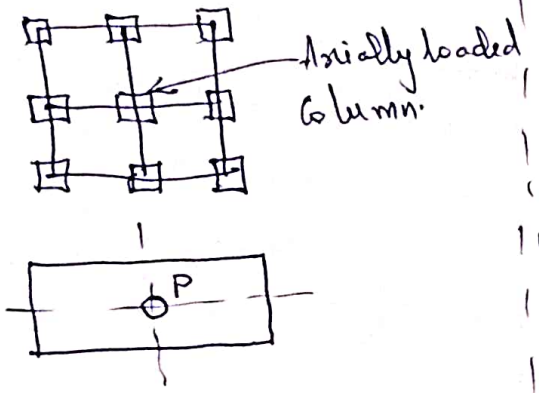
Ans: The method of use of equations is really very labrious and time consuming.
The IS code recommends the use of charts.

Steps

1) Given Data.
2) Decide column dimensions if not given.
3) Compute $\frac{d'}{D}$.
4) Compute $\frac{P_u}{f_{ck} b D}$
5) Compute $\frac{M_u}{f_{ck} b D^2}$
6) On the appropriate chart, obtain the point of intersection of $P_u/f_{ck} b D$ & $M_u/f_{ck} b D^2$.
7) Obtain P/f_{ck} value.
8) Compute area of steel.

$$A_{sc} = \frac{\text{Percent}}{100} b D.$$

Axially loaded Column.



$P_u = 0.4 f_{ck} A_c + 0.6 f_y A_{sc}$.

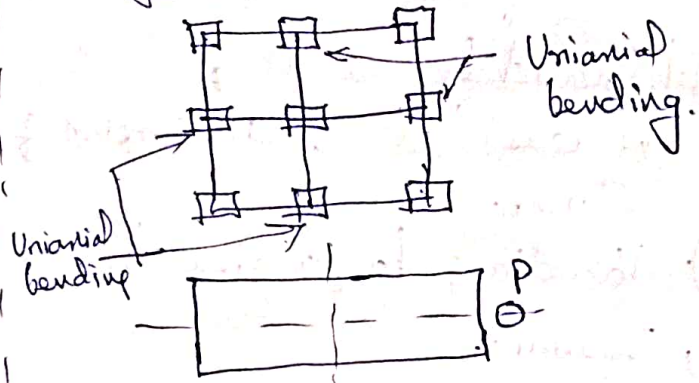
$A_{sc} = 0.01 A_g$.

$A_c = 0.99 A_g$.

for helical Reinf. Column.

$P_u = 1.05 [0.4 f_{ck} A_c + 0.6 f_y A_{sc}]$

Axially loaded Uniaxial bending.



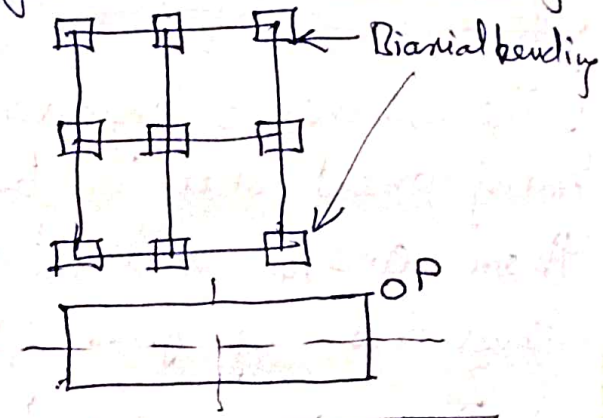
Calculate $\frac{P_u}{f_{ck} b D}$, $\frac{M_u}{f_{ck} b D^2}$

$\frac{d'}{D}$, f_y.

Refer to SP-16 Chart.

Calculate $\frac{P}{f_{ck}}$

Axially loaded Biaxial bending.



$M_u = \alpha \sqrt{M_{ux}^2 + M_{uy}^2}$

Calculate $\frac{P_u}{f_{ck} b D}$, $\frac{M_u}{f_{ck} b D^2}$

$\frac{d'}{D}$, f_y

Refer to SP-16 Chart.

Calculate $\frac{P}{f_{ck}}$.

Longitudinal Reinforcement (26.5.3.1)

1) The c/s area of longitudinal reinforcement shall not be less than 0.8% and not more than 6% of gross c/s area

2) The minimum number of bars.

Square Column = 4 no's

Rectangular Column = 4 no's

Circular Column = 6 no's

3) The minimum diameter of bar shall not be less than 12mm.

4) The diameter of lateral ties shall not be less than
a) $\frac{1}{4}$ dia of bar
b) should not be less than 6mm.

5) Pitch of lateral ties.

It should be least of.

a) The least lateral dimension of Column.
b) 16x dia of longitudinal bar.
c) 300mm.

Q1) Design a short R.C. Square column to carry an axial load of 900KN. Use Concrete of M25 grade and Fe415 steel.

Sol:- Given Data:-

Square Column. 

Axial load = P = 900KN.

$f_{ck} = 25 N/mm^2$, $f_y = 415 N/mm^2$

Factored load:-

$P_u = 1.5P = 1.5 \times 900$

$= 1350 KN$

C/S areas:-

Gross area of Section = Ag

Assuming 1% steel

Area of steel = $A_{sc} = \frac{1}{100} Ag$.

$= 0.01 Ag$

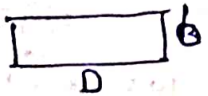
Area of Concrete = $A_c = Ag - A_{sc}$

$= Ag - 0.01 Ag$.

$A_c = 0.99 Ag$.

Q2) Design a short rectangular column of 4.5m unsupported length. Restrained in position and direction at both ends, to carry an axial load of 1200KN. Use M20 Concrete and Fe415 steel.

Sol:- Given Data:-

Rectangular Column. 

Axial load = P = 1200KN

$f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$

Unsupported length = L = 4.5

Effective length $L_e = 0.65L = 0.65 \times 4.5 = 2.925m$.

Factored load:-

$P_u = 1.5P = 1.5 \times 1200 = 1800 KN$

C/S areas:-

Gross area of section = Ag

Assuming 1% steel

Area of steel = $A_{sc} = \frac{1}{100} Ag$.

$= 0.01 Ag$

Area of Concrete = $A_c = Ag - A_{sc}$.

$= Ag - 0.01 Ag$.

$A_c = 0.99 Ag$.

Q3) Design a short circular column with lateral ties to carry an axial load of 800KN. Use M20 grade Concrete and Fe415 steel.

Sol:- Given Data:-

Circular column

Axial load = P = 800KN.

$f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$

Factored load:-

$P_u = 1.5P = 1.5 \times 800 = 1200 KN$

C/S areas:-

Gross area of section = Ag.

Assuming 1% steel.

Area of steel = $A_{sc} = \frac{1}{100} Ag$

$= 0.01 Ag$.

Area of Concrete = $A_c = Ag - A_{sc}$

$= Ag - 0.01 Ag$

$A_c = 0.99 Ag$.

① As per IS Code

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$1350 \times 10^3 = 0.4 \times 25 \times (0.99 A_g) + 0.67 \times 415 \times (0.01 A_g)$

$A_g = 106462.7 mm^2$

for square column

$A_g = a \times a = 106462.7 mm^2$

$a = \sqrt{106462.7}$

$= 326.3 mm \simeq 330 mm$

$A_g = 330 \times 330 = 108900 mm^2$

Area of Steel:-

$A_{sc} = 0.01 A_g$

$= 0.01 \times 108900$

$= 1089 mm^2$

12mm - 113 mm²
16mm - 201 mm²
20mm - 314 mm²
25mm - 490.6

No. of bars $= \frac{A_{sc}}{A\phi}$

As Taking 16mm bars.

No. of bars $= \frac{1089}{113} = 9.63 \simeq 10 Nos$

12mmϕ - 10 bars.

② As per IS Code

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$1800 \times 10^3 = 0.4 \times 20 \times (0.99 A_g) + 0.67 \times 415 \times (0.01 A_g)$

$A_g = 168216.438 mm^2$

for rectangular column, taking b = 230mm

$A_g = bD = 168216.438 mm^2$

$230 \times D = 168216.438$

$D = 731.37 mm \simeq 735 mm$

$A_g = 230 \times 735 = 169050 mm^2$

Area of Steel:-

$A_{sc} = 0.01 A_g$

$= 0.01 \times 169050$

$= 1690.5 mm^2$

No. of bars $= \frac{A_{sc}}{A\phi}$

Taking 16mm bars

No. of bars $= \frac{1690.5}{201} = 8.41 nos \simeq 8 bars$

16mmϕ - 8 bars

③ As per IS Code

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$1200 \times 10^3 = 0.4 \times 20 \times (0.99 A_g) + 0.67 \times 415 \times (0.01 A_g)$

$A_g = 112144.29 mm^2$

for circular column.

$A_g = \frac{\pi D^2}{4} = 112144.29 mm^2$

$D = 377.96 mm$

$\simeq 380 mm$

$A_g = \frac{\pi}{4} \times 380^2 = 113354 mm^2$

Area of Steel:-

$A_{sc} = 0.01 A_g$

$= 0.01 \times 113354$

$= 1133.54 mm^2$

No. of bars $= \frac{A_{sc}}{A\phi}$

Taking 16mm bars

No. of bars $= \frac{1133.54}{201} = 5.639 \simeq 6 nos$

16mmϕ - 6 bars.

Lateral Ties:- (1)

Assume 8mm dia lateral ties

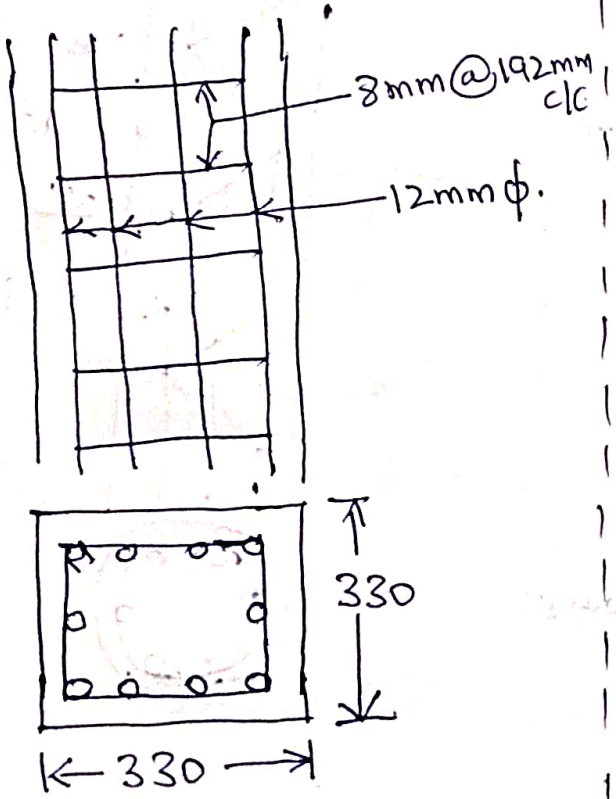
Pitch of lateral ties:-

It should be least of.

1) Least lateral dimension = 330mm
2) 16×dia of bar = 16×12 = 192mm.
3) 300mm.

Taking pitch of 192mm c/c

Reinforcement Details.



Lateral ties:- (2)

Assume 8mm dia lateral ties.

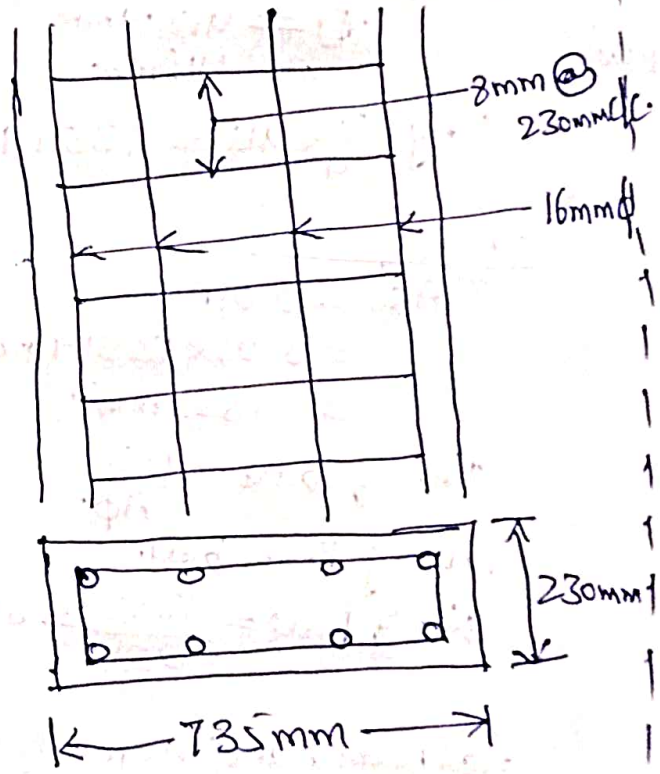
Pitch of lateral ties:-

It should be least of.

1) Least Lateral dimension = 230mm
2) 16× dia of bar = 16×16 = 256mm
3) 300mm.

Taking pitch of 230mm.

Reinforcement Details.



Lateral ties:- (3)

Assume 8mm dia lateral ties.

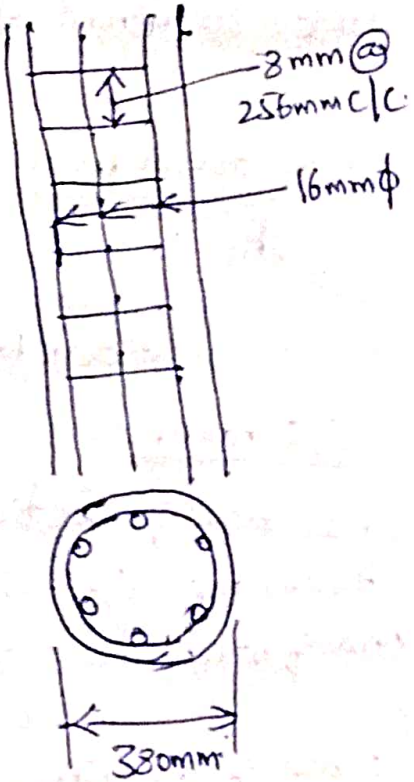
Pitch of lateral ties:-

It should be least of.

1) Least lateral dimension = 380mm
2) 16×dia of bar = 16×16 = 256mm
3) 300mm

Taking pitch of 256mm.

Reinforcement Details.



Q4) Design a Circular Column with helical Reinforcement to Carry an axial load of 1000 kN. Use M20 grade Concrete and Fe 415 steel.

Sol:- Given Data:-

Circular column (←D→)

helical reinforcement.

Axial load = P = 1000 kN

$f_{ck} = 20\ N/mm^2$, $f_y = 415\ N/mm^2$

Factored load:-

$P_u = 1.5P$

$= 1.5 \times 1000$

$= 1500\ kN$

C/s Areas:-

Gross area of section = Ag.

Assuming 1% steel.

Area of steel = $A_{sc} = \frac{1}{100} A_g$.

$= 0.01 A_g$

Area of Concrete = $A_c = A_g - A_{sc}$

$= A_g - 0.01 A_g$.

$A_c = 0.99 A_g$.

As per IS Code

$P_u = 1.05(0.4 f_{ck} A_c + 0.67 f_y A_{sc})$

$1500 \times 10^3 = 1.05(0.4 \times 20 \times (0.99 A_g) + 0.67 \times 415 \times (0.01 A_g))$

$A_g = 133505\ mm^2$.

for circular column.

$A_g = \frac{\pi}{4} D^2 = 133505\ mm^2$

$D = 412.4\ mm$

$\simeq 415\ mm$

$\therefore A_g = \frac{\pi}{4} \times 415^2 = 135196.6\ mm^2$.

Area of steel:-

$A_{sc} = 0.01 A_g$.

$= 0.01 \times 135196.6$

$= 1352\ mm^2$.

No. of bars = $\frac{A_{sc}}{A\phi}$.

Taking 16mm bars.

No. of bars = $\frac{1352}{201} = 6.7 \simeq 7$ bars.

Helical Reinforcement:-

Assume 8mm dia helical reinf.

Pitch of helical reinforcement:-

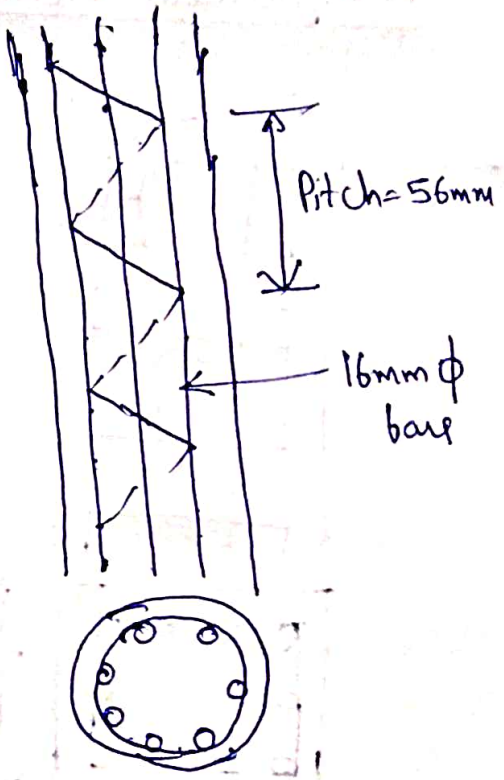
The pitch of helical reinf. should not be more than.

1) 75mm.

2) $\frac{1}{6}$ Core dia = $\frac{1}{6}(415 - 2 \times 40) = 56$ mm

Provide pitch of 56mm.

Reinforcement details:-



Problems Axial load Uniaxial Bending.

Problem 7. Design a short Square Column, with effective length 3.5m, Capable of Safely resisting factored load $P_u = 2000kN$ and $M_u = 125kN\text{-}m$ (under uniaxial eccentricity). The column is braced against side sway. Use M25 Concrete and Fe 415 steel.

Answer Given Data:-

Effective length = $l_{eff} = 3.5m$.

factored load = $P_u = 2000kN$.

factored Moment = $M_u = 125kN\,m$.

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

$f_y = 415\ N/mm^2$.

Assuming the Size of Column Square

Side = $D = 400mm$.

Cover = 40mm = d'

$$\frac{d'}{D} = \frac{40}{400} = 0.1$$

Compute $\frac{P_u}{f_{ck} b D} = \frac{2000 \times 10^3}{25 \times 400 \times 400} = 0.5$

Compute $\frac{M_u}{f_{ck} b D^2} = \frac{125 \times 10^6}{25 \times 400 \times 400^2} = 0.078$

Refering to SP16 chart for $\frac{d'}{D} = 0.1$, $f_y = 415$

$\frac{p}{f_{ck}} = 0.06$

∴ Percentage of Steel required

$p = 0.06 \times 25 = 1.5\%$.

Area of Steel required $= \frac{1.5}{100} \times b D$.

$= \frac{1.5}{100} \times 400 \times 400$

$= 2400 mm^2$

Provide 16mm dia.

No. of bars $= \frac{2400}{\frac{\pi}{4} \times 16^2} = 7.64$ Nos

Provide 8bars of 16mm

Lateral ties:-

Dia of ties should not be less than

1) $\frac{1}{4}$ x Dia of main bar = $\frac{1}{4} \times 16 = 4mm$

2) 6mm.

Provide 6mm dia lateral ties.

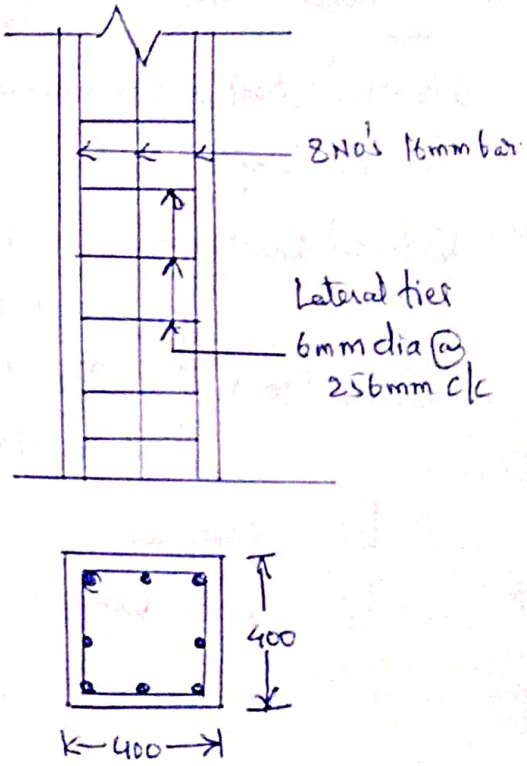
Pitch of lateral ties:- (should be least of)

1) least Lateral dimension = 400mm

2) 16x dia of main bar = $16 \times 16 = 256mm$

3) 300mm.

Provide 6mm lateral ties @ 256mm c/c



Problem 8:- Design the Reinforcement for a column of section 400mm x 400mm and effective length 4.2m. The column is subjected to a factored axial load of 1250 kN and factored moment of 125 kNm about one of the centroidal axes. Use M25 grade concrete and Fe415 steel.

Answer:-

Given Data:-

Column section = 400mm x 400mm

Effective length = 4.2

Factored axial load = P_u = 1250 kN

$= 1250 \times 10^3$ N

Factored Moment = M_u = 125 kNm

$= 125 \times 10^6$ N mm

$f_{ck} = 25$ N/mm²

$f_y = 415$ N/mm².

$D = 400$mm

Cover = $d' = 40$mm

$\frac{d'}{D} = \frac{40}{400} = 0.1$

Compute $\frac{P_u}{f_{ck} \cdot b \cdot D} = \frac{1250 \times 10^3}{25 \times 400 \times 400}$

$= 0.3125$

Compute $\frac{M_u}{f_{ck}\, b\, D^2} = \frac{125 \times 10^6}{25 \times 400 \times 400^2}$

$= 0.078 \simeq 0.08$

Refering SP16 Chart

$f_y = 415$, $\frac{d'}{D} = 0.1$, $\frac{P_u}{f_{ck} b D} = 0.3125$

$\frac{M_u}{f_{ck} b D^2} = 0.08$

$\frac{p}{f_{ck}} = 0.04$

$p = 0.04 \times 25 = 1\%$

Area of steel required = $\frac{1}{100} bD$

$= \frac{1}{100} \times 400 \times 400$

$\simeq 1600$mm².

Assuming 16mm dia bar

No. of bars = $\frac{1600}{\frac{\pi}{4}(16)^2} = 7.96$ Nos

Say 8 Nos

Provide 8 No's 16mm dia.

Lateral ties :-

Dia of lateral ties should not be less than

1) $\frac{1}{4}$ x Dia of main bar = $\frac{1}{4} \times 16 = 4$mm

2) 6mm.

Provide 6mm dia lateral ties.

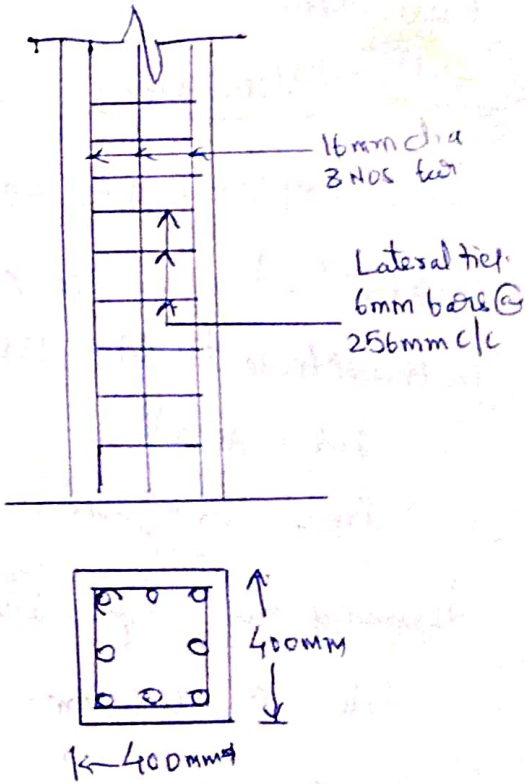
Pitch of lateral ties:- (It should be least of)

1) least lateral dimension = 400mm

2) 16x dia of main bar = $16 \times 16 = 256$mm

3) 300mm.

Provide lateral ties 6mm dia at 256mm c/c



Problem 9:- Design a reinforced concrete column, 400mm square, to carry an ultimate load of 1000KN at an eccentricity of 160mm. Use M_{20} grade concrete and fe415 steel.

Answer:-

Given Data:-

Square column $b = D = 400mm$

Ultimate load $P_u = 1000KN$

e = Eccentricity $= 160mm = 0.16m$

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

$f_y = 415N/mm^2$

Factored Moment $M_u = P_u \cdot e$

$= 1000 \times 0.16$

$= 160KNm$

Cover $d' = 40mm$

$\frac{d'}{D} = \frac{40}{400} = 0.1$

Compute $\frac{P_u}{f_{ck} b \cdot D} = \frac{1000 \times 10^3}{20 \times 400 \times 400} = 0.3125$

Compute $\frac{M_u}{f_{ck} b D^2} = \frac{160 \times 10^6}{20 \times 400 \times 400^2} = 0.125$

Referring to SP16 chart for

$f_y = 415$, $\frac{d'}{D} = 0.1$, $\frac{P_u}{f_{ck} b D} = 0.3125$

$\frac{M_u}{f_{ck} b D^2} = 0.125$

$\frac{P}{f_{ck}} = 0.06$

$P = 0.06 \times 20 = 1.2$ percent.

$\therefore$ Area of Steel Required is

$A_s = \frac{1.2}{100} bD = \frac{1.2}{100} \times 400 \times 400$

$= 1920mm^2$.

Provide 4 nos - 20mm bar

4 nos - 16mm bar.

$\therefore$ Total $A_s = 4 \times \frac{\pi}{4} \times 20^2 + 4 \times \frac{\pi}{4} \times 16^2$

$= 2059.8mm^2$.

Lateral ties:-

Dia of lateral ties should not be less than

1) $\frac{1}{4} \times$ Dia of main bar $= \frac{1}{4} \times 20 = 5mm$

2) 6mm.

Provide 8mm dia lateral ties.

Pitch of lateral ties:-

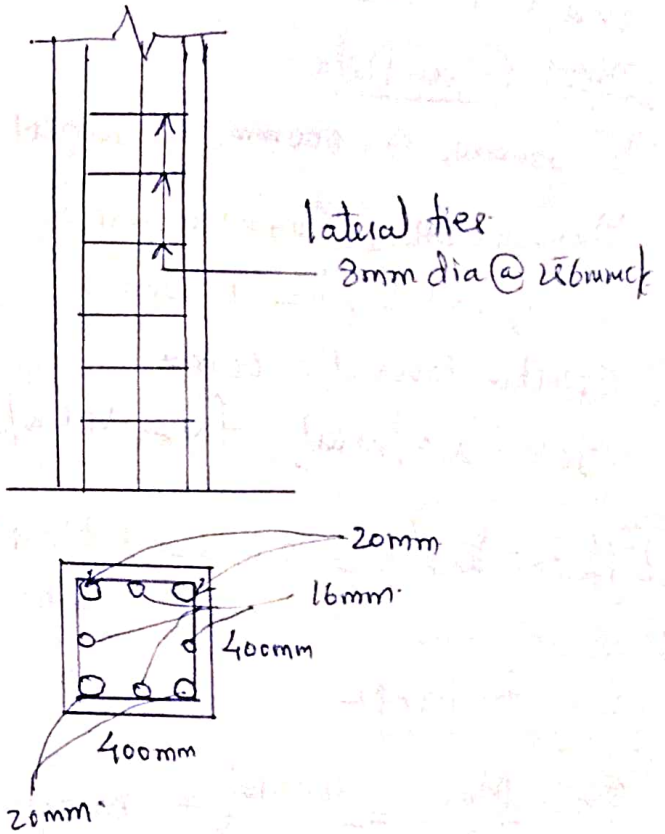
It should be least of

1) Least lateral dimension $= 400mm$

2) 16 x dia of main bar $= 16 \times 16 = 256mm$

3) 300mm.

Provide lateral ties 8mm dia at 256mm c/c.



Problem 10:- Determine the Reinforcement for a column of a braced frame for the following data:

Size of column: 400mm x 600mm.

$P_u = 2000\,KN$, $M_{ux} = 160\,KNm$, $M_{uy} = 120\,KNm$.

Unsupported length of column = 4m,

Effective cover = 60mm, Use M_{20} concrete and fe 415 Steel.

Answer **Given Data:-**

$b = 400mm$, $D = 600mm$., $P_u = 2000\,KN$

$M_{ux} = 160\,KNm$, $M_{uy} = 120\,KNm$.

Unsupported length $= L = 4m$

Effective cover $d' = 60mm$.

$f_{ck} = 20\,N/mm^2$, $f_y = 415\,N/mm^2$.

Effective length $= 0.65L \rightarrow 0.65 \times 4$.

$= 2.6m$.

Eccentricities:-

$$e_x = \frac{M_{ux}}{P_u} = \frac{160 \times 10^6}{2000 \times 10^3} = 80mm.$$

$$e_y = \frac{M_{uy}}{P_u} = \frac{120 \times 10^6}{2000 \times 10^3} = 60mm.$$

Selection of Reinforcement:-

$$M_u = a\sqrt{M_{ux}^2 + M_{uy}^2}$$

$$= 1.15\sqrt{160^2 + 120^2}$$

$= 230\,KNm$.

$$\text{Compute} = \frac{P_u}{f_{ck} b D} = \frac{2000 \times 10^3}{20 \times 400 \times 600}$$

$= 0.417$.

$$\text{Compute} = \frac{M_u}{f_{ck} b D^2} = \frac{230 \times 10^6}{20 \times 400 \times 600^2}$$

$= 0.0798 \simeq 0.08$.

$\frac{d'}{D} = \frac{60}{600} = 0.1$; $f_y = 415\,N/mm^2$.

Refer to SP-16 chart.

$\frac{p}{f_{ck}} = 0.06$.

$p = 0.06 f_{ck} = 0.06 \times 20$

$= 1.2\%$.

$\therefore$ Area of Steel required $= \frac{1.2}{100} b D$.

$= \frac{1.2}{100} \times 400 \times 600$

$= 2880\,mm^2$.

Provide 25mm bars

number of bars $= \frac{2880}{\frac{\pi}{4} \times 25^2} = 5.27\,nos$.

Provide 6 bars of 25mm.

$\therefore A_s = \frac{\pi}{4} \times 25^2 \times 6 = 2943.75\,mm^2$

Lateral ties:- Dia of ties should not be less than

1) $\frac{1}{4} \times$ Dia of main bar $= \frac{1}{4} \times 25 = 6.25mm$.

2) 6mm.

Provide 8mm dia lateral ties.

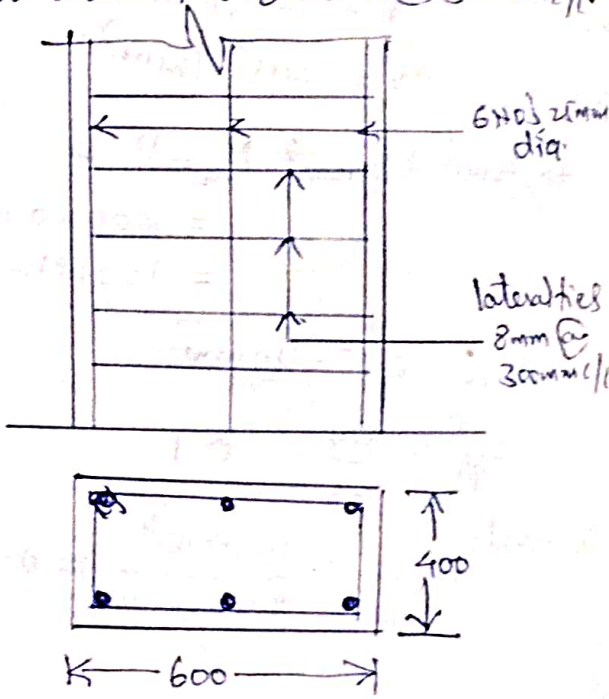
Pitching lateral ties:- (Should be least of)

1) least lateral dimension $= 400mm$

2) 16 x dia of main bar $= 16 \times 25 = 400mm$

3) 300mm.

Provide lateral ties 8mm dia @ 300mm c/c.

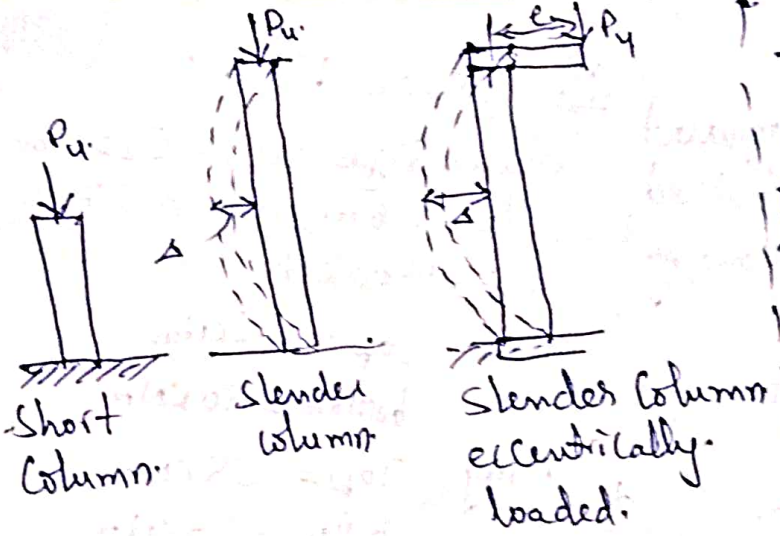


Slender Columns

- Compression member may be considered as slender or long when the slenderness ratio l_{ex}/D or l_{ey}/b are more than 12.

- If $l_{ex}/D > 12$, the column is considered to be slender for bending about x-x axis.

- If $l_{ey}/b > 12$, the column is considered to be slender for bending about Y-Y-axis.



- When a short column is loaded even with an axial load, the lateral deflection is either zero or very small.

- In slender column, with an axial load, the lateral deflection Δ, becomes appreciable.

- This make the axial load eccentric at the central section of column by a value of Δ, Subjecting the section to a B.M. equal to $P_u \Delta$. Hence separate consideration of this additional B.M. has to be made.

- The axial compression due to axial load is called primary effect, while the bending caused due to lateral deflection of slender column is called the slender effect.

Problem:-

Design the reinforcement required for a column which is restrained against sway using the following data:-

Size of column = 450 × 530 mm.
Effective length = 6.6 m.
Unsupported length = 7.7 m.
factored load = 1600 kN.
Factored Moment about Major axis = 45 kNm at top and 30 kNm at bottom.
Factored Moment about Minor axis = 35 kNm at top and 20 kNm at bottom.
Concrete grade M-25,
Steel Fe500 HYSD.
Column is bent in double curvature and reinforcement is distributed equally on all the four sides of the section.

Sol:- Given Data:-

Column Size 450 × 530 mm.
$l_e = 6.6$ m, $L = 7.7$ m.
$P_u = 1600$ kN
M_{ux} Top = 45 kNm
bottom = 30 kNm
M_{uy} Top = 35 kNm.
bottom = 20 kNm.
$f_{ck} = 25\ N/mm^2$
$f_y = 500\ N/mm^2$.

Slenderness Ratio:-

$$\frac{l_{ex}}{D} = \frac{6600}{530} = 12.45 > 12.$$

$$\frac{l_{ey}}{b} = \frac{6600}{450} = 14.67 > 12.$$

Column is slender about both axis.

Eccentricities:-

$$e_{xmin} = \frac{l_x}{500} + \frac{D}{30}$$

$$= \frac{7700}{500} + \frac{530}{30} = 33 mm$$

$$e_{ymin} = \frac{l_y}{500} + \frac{b}{30}$$

$$= \frac{7700}{500} + \frac{450}{30}$$

$$= 30.4 mm$$

Moments due to eccentricity

$M_{ix} = P_u e_{xmin}$

$$= 1600 \times \frac{33}{1000} = 52.8 kNm > 45 kNm$$

$M_{iy} = P_u e_{ymin}$

$$= 1600 \times \frac{30.4}{1000} = 48.64 kNm > 35 kNm.$$

Additional Moments (39.7.1)

$$M_{ax} = \frac{P_u D}{2000}\left(\frac{l_{ex}}{D}\right)^2.$$

$$= \frac{1600 \times 10^3 \times 530}{2000}\left(\frac{6600}{530}\right)^2$$

$$= 65.75 \times 10^6\ N mm.$$

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

$$= \frac{1600\times10^3 \times 450}{2000}\left(\frac{6600}{450}\right)^2$$

$$= 77.44\times10^6 \text{ N mm.}$$

The total Moment Column to be designed are.

$M_{ux} = 52.8 + 65.75$

$= 118.55$ kNm.

$M_{uy} = 43.64 + 77.44$

$= 126.08$ kNm

Design Moment:

$$M_u = a\sqrt{M_{ux}^2 + M_{uy}^2}$$

$$= 1.15\sqrt{118.55^2 + 126.08^2}$$

$= 199$ kNm.

SP-16 chart for steel percentage:-

$$\frac{P_u}{f_{ck} b D} = \frac{1600\times10^3}{25\times450\times530} = 0.268$$

$$\frac{M_u}{f_{ck} b D^2} = \frac{199\times10^6}{25\times450\times530^2} = 0.063$$

$f_y = 500$; $d' = 40 + \frac{25}{2} = 52.5$

$$\frac{d'}{D} = \frac{52.5}{530} = 0.099 \simeq 0.1$$

Chart 48.

$$\frac{P}{f_{ck}} = 0.02$$

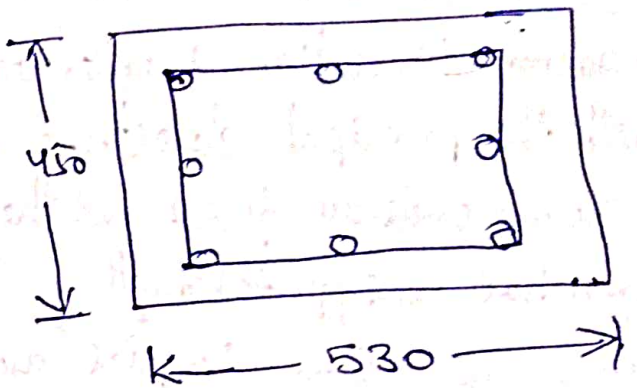
$$P = 0.02\times\frac{25}{500} = 0.5\%$$

Provide 1% steel.

$$A_{st} = \frac{1}{100}\times450\times530$$

$= 2385 \text{ mm}^2$

Provide 8 bars of 20mm.



Problem 2:- An unbraced column 300×400mm in section, is unbraced in both the principal directions. The column ends are fixed and the column has unsupported length of 3.9m. The column carries an ultimate axial load of 800KN and ultimate moments of 80KNm and 40KNm about the major axis bisecting the depth at its ends. Assuming M20 grade Concrete and Fe415 grade Steel and effective Cover of 60mm. Determine area of longitudinal Reinforcement.

Sol:- Given Data:-

$b = 300mm$

$D = 400mm$

$L = 3.9m$

Le for unbraced Condition

$Le = 1.2 \times 3.9 = 4.68m$

$P_u = 800KN$

$M_{ux} = 80KNm$

$M_{uy} = 40KNm$

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

$f_y = 415N/mm^2$

$d' = Cover = 60mm$

Slenderness Ratios:-

$\frac{Lex}{D} = \frac{4680}{400} = 11.7 < 12$

$\frac{Ley}{b} = \frac{4680}{300} = 15.6 > 12$

Hence the column is short in bending about x-axis and long in bending about Y-axis.

Eccentricities:-

$e_{x\,min} = \frac{Lx}{500} + \frac{D}{30}$

$\frac{3900}{500} + \frac{400}{30} = 21.13mm$

$e_{y\,min} = \frac{Ly}{500} + \frac{D}{30}$

$= \frac{3900}{500} + \frac{300}{30} = 17.8mm$

Moments due to eccentricities:-

$M_{ix} = P_u e_{x\,min}$

$= 800 \times 10^3 \times 21.13$

$= 16.90 \times 10^6 Nmm$

$= 16.90KNm$

$M_{iy} = P_u e_{y\,min}$

$= 800 \times 10^3 \times 17.8$

$= 14.24 \times 10^6 Nmm$

$= 14.24KNm$

Additional Moments (39.7.1):-

$M_{ax} = \frac{P_u D}{2000}\left(\frac{Lex}{D}\right)^2$

$\frac{800 \times 10^3 \times 400}{2000}\left(\frac{4680}{400}\right)^2$

The column is short for bending about x-axis. No additional moment is computed along this axis.

About Y-axis

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

$$= \frac{800\times10^3\times300}{2000}\left(\frac{4680}{300}\right)^2$$

$$= 29.2\times10^6 \text{ N mm}$$

$$= 29.2 \text{ kN m}$$

The total Moment column to be designed are :-

$M_{ux} = 80 + 0 = 80 \text{ kN m}$

$M_{uy} = 14.24 + 29.2 = 43.44 \text{ kN m}$

Design Moment :-

$$M_u = a\sqrt{M_{ux}^2 + M_{uy}^2}$$

$$= 1.15\sqrt{80^2 + 43.44^2}$$

$$= 91.033 \text{ kN m}$$

SP-16 chart for steel percentage :-

$$\frac{P_u}{f_{ck}bD} = \frac{800\times10^3}{20*300\times400} = 0.333$$

$$\frac{M_u}{f_{ck}bD^2} = \frac{91.033\times10^6}{20\times300\times400^2} = 0.0948$$

$f_y = 415 \text{ N/mm}^2$

$$\frac{d'}{D} = \frac{60}{400} = 0.15$$

Chart - 46. SP-16

$$\frac{P}{f_{ck}} = 0.06$$

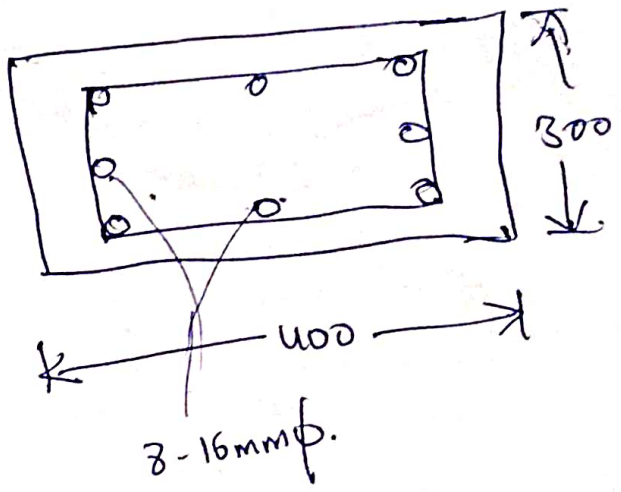
$P = 20 \times 0.06 = 1.2\%$

$A_{st} = pbD$

$$= \frac{1.2}{100}\times300\times400$$

$$= 1440 \text{ mm}^2$$

Provide 16-mm dia 8 bars



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DEPARTMENT OF CIVIL ENGINEERING
LECTURE NOTES

ACADEMIC YEAR: 2023-24

REGULATION: R20

YEAR/SEMESTER: III - I

SUBJECT: DESIGN OF REINFORCED CONCRETE STRUCTURES

SUBJECT CODE: 20CE5PC02

FACULTY NAME: MOHAMMED MUNEERUDDIN KHAN

SYLLABUS:

UNIT – V

Design of foundation – Different types of footings – Design of wall footing – Design of flat isolated square, rectangular, circular footings and combined footings for two columns.

UNIT-IV

RCSD&D NOTES & ASSIGNMENT

SE-I RCC Q&A

Que 1. Explain about footing and foundation.

Answer: Any structure is generally considered to have two main portions 1) The superstructure and 2) The substructure.

The substructure transmits the loads of superstructure to the supporting soil and is generally termed as foundation.

Footing is that portion of the foundation which ultimately delivers the load to the soil, and it is in contact with soil.

The foundation will distribute the load to the soil in such a way that the maximum pressure on the soil does not exceed its permissible bearing value, and at the same time the settlement is within permissible limit.

Que 2 What are the different types of footings.

Answer:

Foundations may be broadly classified under two heads:- Shallow foundation and deep foundation.

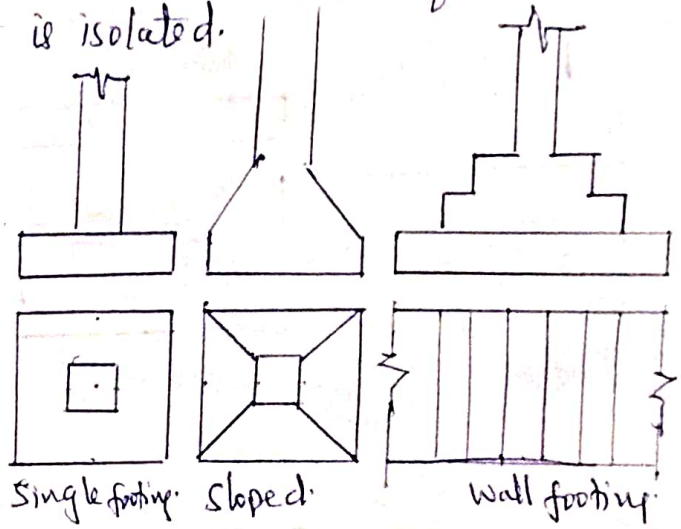
According to Terzaghi, a foundation is shallow if its depth is equal to or less than its width.

In case of deep foundation, the depth is greater than the width.

Types of Shallow foundation:-

1) Spread footing:-

A spread footing or simply footing, is a type of shallow foundation used to transmit the load of an isolated column, or that of a wall on subsoil. In case of wall, the footing is continous while in case of column it is isolated.

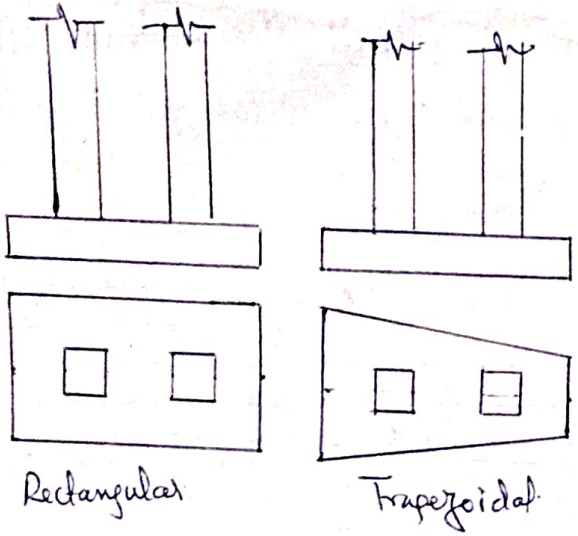


(fig) Types of Spread footing.

2) Combined footing:-

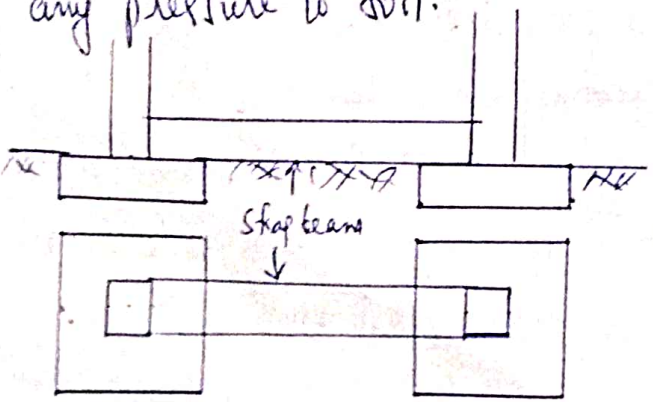
A spread footing which supports two or more columns is termed as a combined footing. Such a footing is provided when the individual footings are either very near to each other or overlap.

These footings may be rectangular or trapezoidal in shape.



3) Strap or Cantilever footing :-

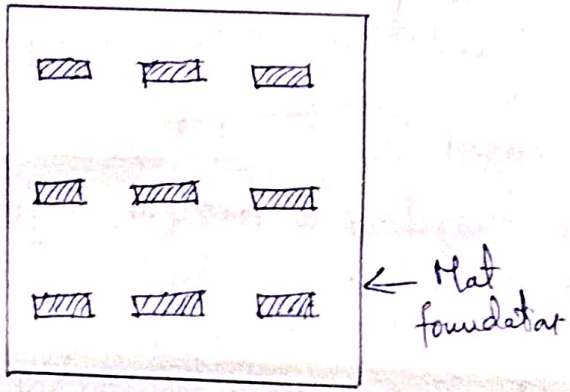
A strap footing consists of spread footings of two columns connected by a strap beam. The strap beam does not remain in contact with soil, and thus does not transfer any pressure to soil.



4) Mat or Raft foundation :-

A mat or raft is a combined footing that covers the entire area beneath a structure and supports all the walls and columns.

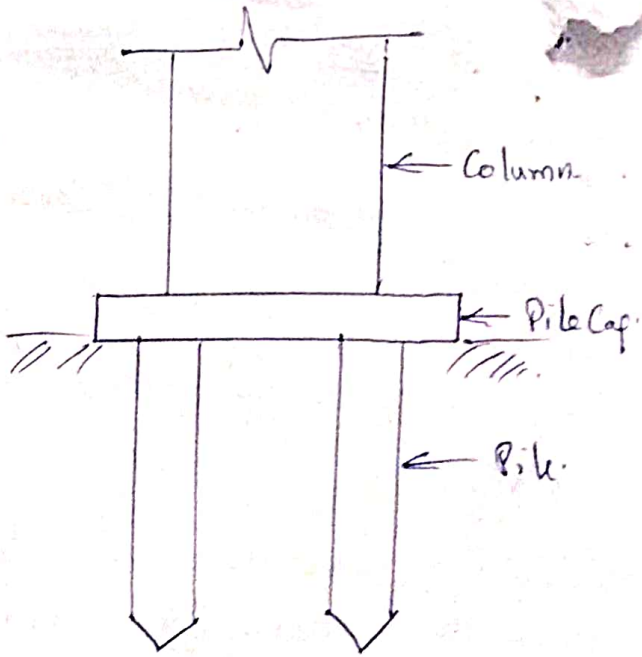
When the available soil pressure is low or building loads are heavy, it is economical to use mat or raft foundation.



Types of Deep foundation:-

1) Pile foundation:-

Pile foundation is a deep foundation used where the top soil is relatively weak. Piles transfer the load to a lower stratum by way of end bearings or skin friction.



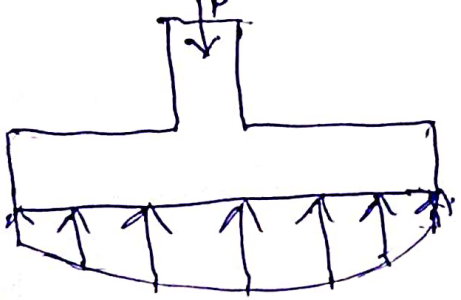
Cover To footing is 50mm.

8) Explain about Pressure distribution below the footing.

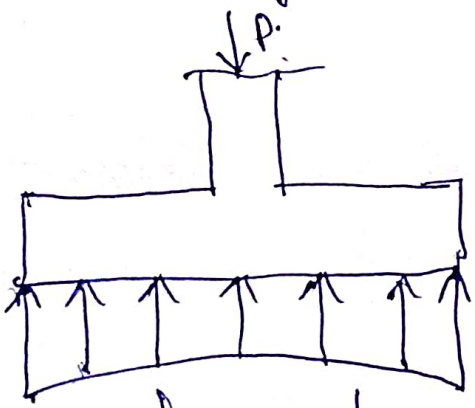
Ans: The pressure intensity depends upon

1) Rigidity of footing.
2) Type of soil.
3) Depth of foundation.

Depending on soil the pressure distribution varies

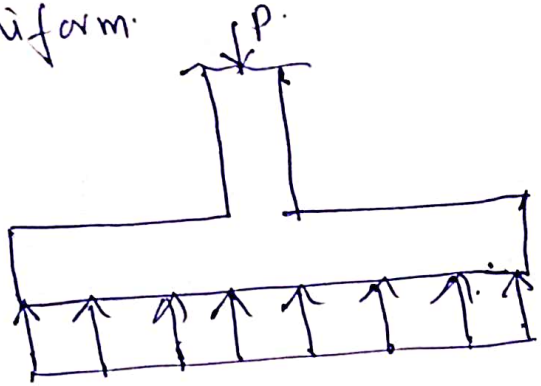


Sandy soil



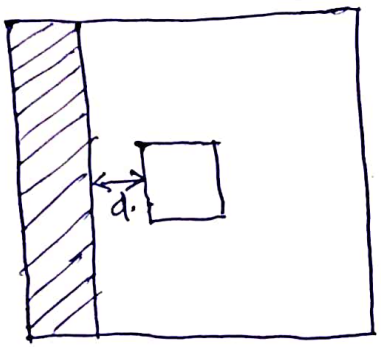
Clayey soil

In practice, the pressure distribution is assumed to be uniform.



8) Explain about one-way Shear and two-way shear.

Ans: One-way Shear



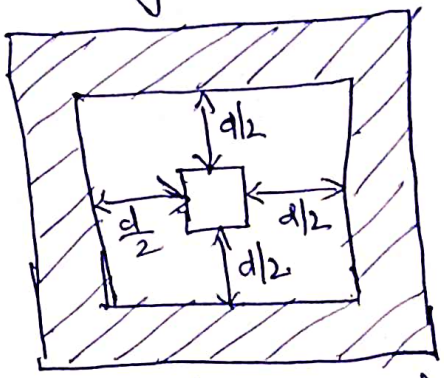
- The check for one way shear is carried out similar to the beams or slabs.
- The critical section for the one way shear is at a distance 'd' from column.

V_u = Soil pressure from shaded area.

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

$\tau_v = \frac{V_u}{bd} < \tau_c$.

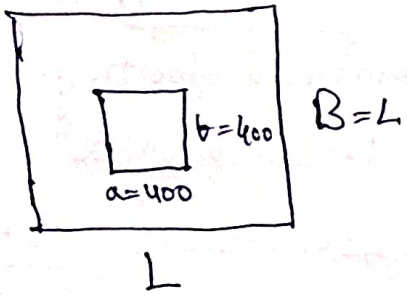
Two-way Shear:



- Two way shear is also called punching shear.
- The footing depth is less, the column may punch through the footing.
- The critical section for two-way shear is at a distance d/2 from the periphery of column.
- $\tau_p = \frac{S}{A} = \frac{\text{Punching force}}{\text{Area}} = \frac{q_u[B^2 - (b+d)^2]}{4(b+d)d}$
- $\tau_p < 0.25\sqrt{f_{ck}}$.

Que1) A R.C. Column of size 400x400mm carries a load of 1000KN. The safe bearing Capacity of soil is 200KN/m². Design a isolated Square Column of uniform thickness. Use M20 Concrete and fe415 steel.

Sol:- Given Data:-



Axial load = P = 1000KN.

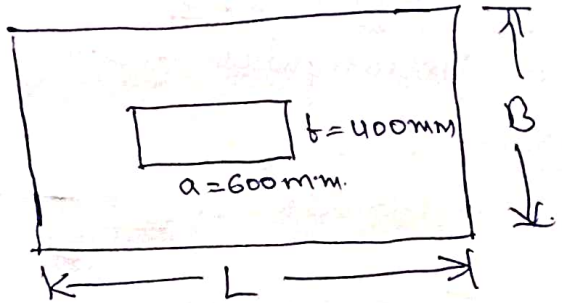
SBC = 200KN/m².

f_{ck} = 20 N/mm².

f_y = 415 N/mm².

Que2) Design a rectangular isolated footing of uniform thickness for R.C. Column bearing a vertical load of 600KN, and having a base size of 400x600mm. The safe bearing Capacity of soil is taken as 120KN/m². Use M20 grade Concrete & fe415 steel.

Sol:- Given Data:-



Axial load = P = 600KN.

Size of Column = 400x600mm.

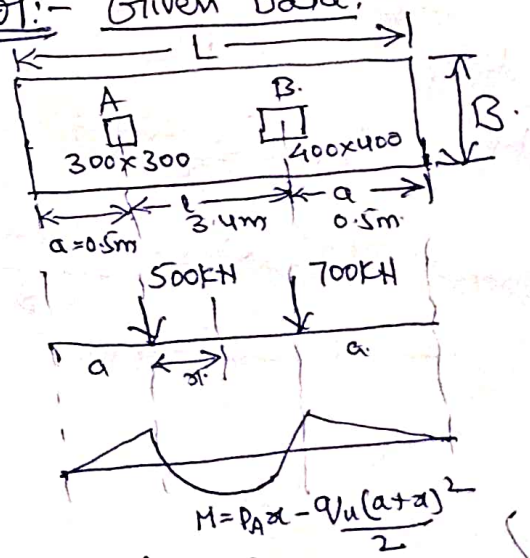
SBC = 120KN/m².

f_{ck} = 20N/mm²

f_y = 415N/mm².

Que3) Design a Combined rectangular footing for two Columns A & B, Carrying loads of 500 and 700 KN. respectively. Column 'A' is 300x300mm in size and Column B is 400x400mm in size. The centre to centre spacing of the column is 3.4m. The safe bearing Capacity of soil is 150KN/m². Use M20 Concrete and fe415 steel.

Sol:- Given Data:-



$x = \frac{l}{2} = \frac{3.4}{2} = 1.7m$.

a = 0.5m.

P_A = 500KN, P_B = 700KN.

P = $P_A + P_B$ = 500+700 = 1200KN

SBC = 150KN/m²

f_{ck} = 20 N/mm², f_y = 415N/mm²

Size of footing (A)

$$A = \frac{1.1P}{SBC} = \frac{1.1\times1000}{200}$$

$$= 5.5m^2$$

$A = L\times B$, Square, $A = L^2 = 5.5$, $L = B = 2.34m$

Upward soil pressure q_u:-

$$q_u = \frac{1.5P}{A} = \frac{1.5\times1000}{5.5}$$

$$= 272.73 KN/m^2$$

Bending Moment:-

for Square footing.

$M_{u1} = M_{u2} = M_u$.

$$M_u = q_u B \frac{(L-a)^2}{8}$$

$$= 272.73\times2.34 \frac{(2.34-0.4)^2}{8}$$

$M_u = 300.235 KNm$.

Size of footing (A)

Taking $B = \frac{2}{3}L$.

$A = L\times B = 5.5$

$= L\times\frac{2}{3}L = 5.5$

$L = 2.87$

$B = \frac{2}{3}\times2.87 = 1.91m$

$$A = \frac{1.1P}{SBC} = \frac{1.1\times600}{120}$$

$$= 5.5m^2$$

Upward soil pressure q_u:-

$$q_u = \frac{1.5P}{A} = \frac{1.5\times600}{5.5}$$

$$= 163.63 KN/m^2$$

Bending Moment:-

for rectangular footing.

$$M_{u1} = q_u B \frac{(L-a)^2}{8}$$

$$= 163.63\times1.91 \frac{(2.87-0.6)^2}{8}$$

$$= 201.3 KNm.$$

$$M_{u2} = q_u L \frac{(B-b)^2}{8}$$

$$= 163.63\times2.87 \frac{(1.91-0.4)^2}{8}$$

$$= 133.85 KNm$$

M_u = Maximum of M_{u1} or M_{u2}

$M_u = 201.3 KNm$

Size of footing (A)

$L = a+L+a$

$= 0.5+3.4+0.5$

$= 4.4m$.

$L\times B = 8.8$

$4.4\times B = 8.8$

$B = 2m$

$$A = \frac{1.1P}{SBC} = \frac{1.1\times1200}{150}$$

$$= 8.8m^2$$

Upward Soil Pressure q_u:-

$$q_u = \frac{1.5P}{A} = \frac{1.5\times1200}{8.8}$$

$$= 204.54 KN/m^2$$

Bending Moment:-

for Combined footing.

$$M_u = P_A\cdot x - q_u\frac{(a+x)^2}{2}$$

$$= 500\times1.7 - 204.54\frac{(0.5+1.7)^2}{2}$$

$M_u = 355 KNm$

Depth of footing :- (d)

$M_u = \emptyset f_{ck} B d^2$

$300.235 \times 10^6 = 0.138 \times 20 \times 2340 \times d^2$

$d = 215.6\text{mm}$

To check shear take d

$d = 2 \times 215.6 = 431\text{mm}$

$D = d + \text{cover}$

$= 431 + 50 = 481\text{mm}$

Area of steel (Ast):-

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

$300.235 \times 10^6 = 0.87 \times 415 A_{st} \times 431 \left(1 - \frac{415 A_{st}}{20 \times 2340 \times 431}\right)$

$A_{st} =$

Depth of footing (d) :-

$M_u = \emptyset f_{ck} B d^2$

$201.3 \times 10^6 = 0.138 \times 20 \times 1910 \times d^2$

$d = 195.4\text{mm}$

To check shear take 'd'

$d = 2 \times 195.4 = 391\text{mm}$

$D = d + \text{cover}$

$= 391 + 50 = 441\text{mm}$

Area of steel (Ast):-

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

$201.3 \times 10^6 = 0.87 \times 415 \times A_{st} \times 391 \left(1 - \frac{415 A_{st}}{20 \times 1910 \times 391}\right)$

$A_{st} =$

Depth of footing (d):-

$M_u = \emptyset f_{ck} B d^2$

$355 \times 10^6 = 0.138 \times 20 \times 2000 \times d^2$

$d = 253.6\text{mm}$

To check shear take 'd'

$d = 253.6 \times 2 = 507 \simeq 510\text{mm}$

$D = d + \text{cover}$

$= 510 + 50 = 560\text{mm}$

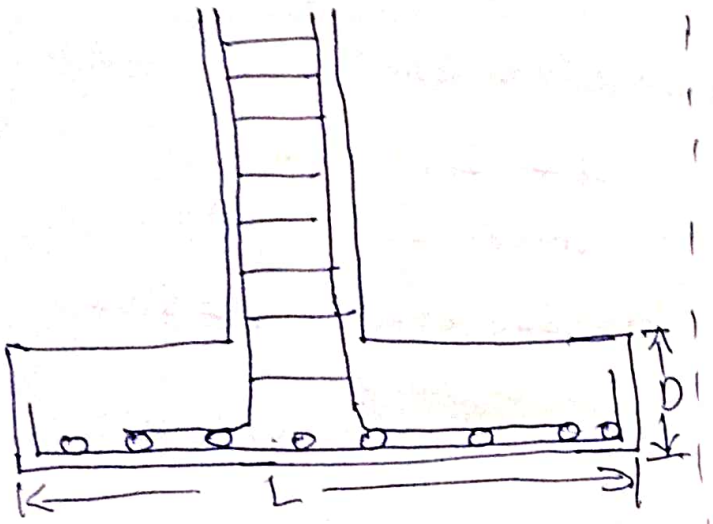
Area of steel (Ast) :-

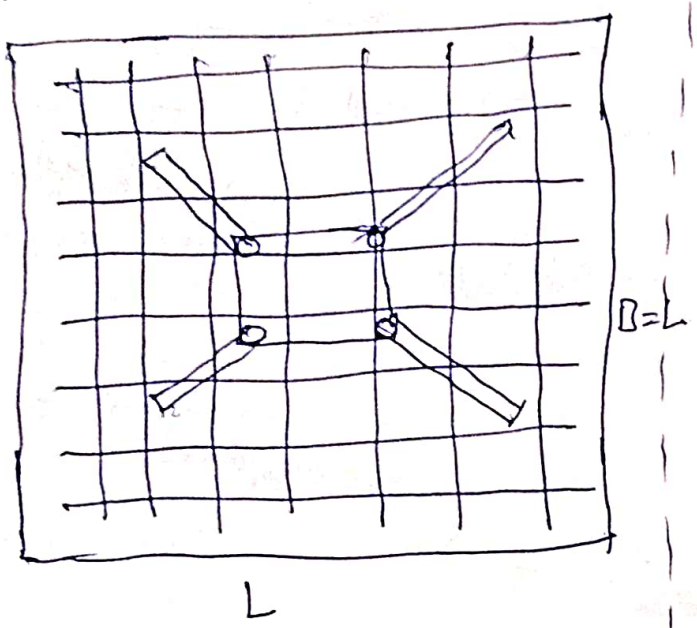
$M_u = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} B d}\right)$

$355 \times 10^6 = 0.87 \times 415 \times A_{st} \times 510 \left(1 - \frac{415 \times A_{st}}{20 \times 2000 \times 510}\right)$

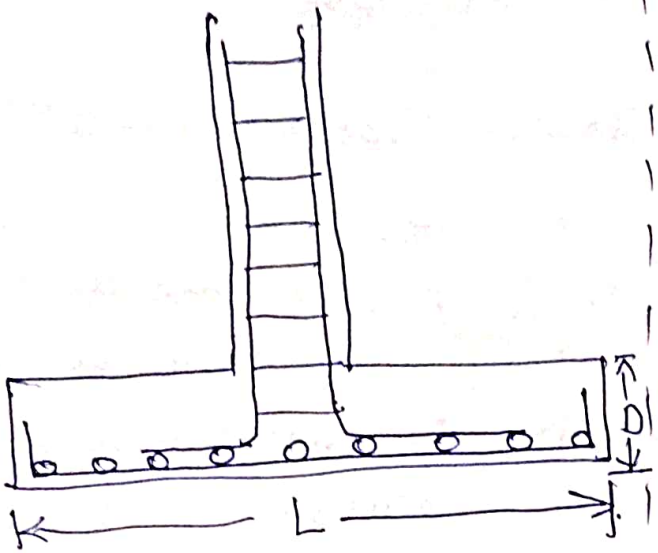
$A_{st} =$

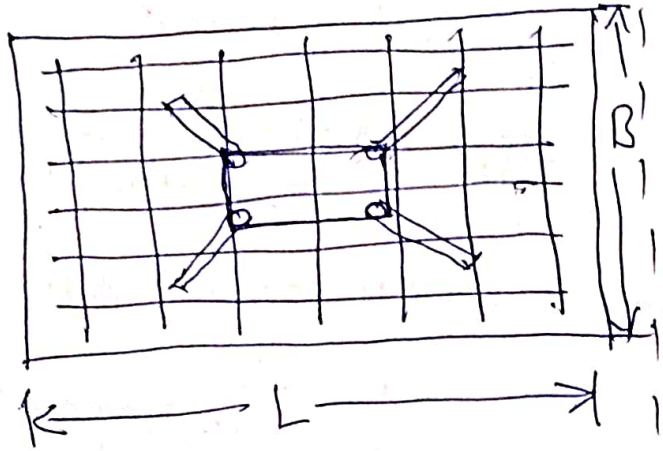
Reinforcement Details :-





Reinforcement Details :-





Reinforcement Details :-

