BASIC CONCEPT OF RCC

STRUCTURAL ENGINEER

↓

ANALYSIS

* SOM
* SA

DESIGN

* RCC
* DSS
* PSC

SOM	Strength Of Materials
SA	Structural Analysis
RCC	Reinforced Concrete Structures
DSS	Design of Steel Structures
PSC	Prestressed Concrete

Structural Engineer

* Structural Engineer is a Speciality within Civil Engineering

* Structural Engineer are responsible for

  * Analysis
  * Planning
  * Design
  * Construction

* Maintanence

* Rehabilitation & Demolition

* Structural Engineers work closely with

  * Client

  * Architect

  * Contractors

* If you are interested in the design and structure of building and have a lot of creative ideas (or) creative projection

* Entry-level structural Engineers may design the individual elements of a structure

  * Beam

  * Column

  * Slab

* More experience engineers may be responsible for the structural design and integrity of entire system such as building.

* The structural Engineer responsibility to calculate the load such as

  * Snow

  * Wind

  * Earthquake

* Structural Engineers generally are Involved with the design of New building, they are sometimes Involved in the demolition or dismantling of a Structure, either permanently or in order to repair it.

* Because the work of Structural Engineers is Closely tied to public safety.

* The Various Structure Can Include

  * Residential building
  * Industrial building
  * Tall Structure
  * Bridges
  * Transmission line and Microwave Towers
  * Chimney
  * Elevated and Underground water Tank
  * Silos
  * Bunkers
  * Footing
  * Space Frame such as domes
  * Multi layer Grid and Folded Plate
  * Shell
  * Cooling Towers
  * offshore plateform

* Sky Scrapers
* Ships
* Aeroplane and Crane

* Structural Engineering work in the
    * Design of Structure
    * Strength Calculation
    * Detailing
    * Making Structural drawing
    * Analysing blueprint
    * Estimating the Cost and Quantities of Materials
    * Inspecting project Site
    * preparing report

* Strength of Materials
* Structural Analysis

→ Analysis

Name of the material not mention

* Design of Concrete Structures
* Design of Steel Structures

→ Design

Name of the material mention

Difference between Structural Analysis & Structural design

Structural Analysis

* It is a process by which we find out how a structure or a member of a structure behave under applied loads.

* That is means find out Internal Forces

  * Axial Forces
  * Shear Force
  * Moment
  * Stress
  * Strain
  * Deflection

We Need this data for a Structural design

Example

Given :

* Breadth of beam (b)
* Depth of beam (d)
* Area of steel reinforcement (Ast)

To Find

Load = ?

Structural design

* It is the process by which we find out safe and economical specifications of a structure or a member of the structure.

* That is means finding out member steel section, cross sectional dimension, amount of reinforcement etc..,

* To withstand the Internal Forces that we have got from structural analysis

Example

Given

Load

To Find

Breadth of beam (b) = ?

Depth of beam (d) = ?

Area of steel reinforcement } (Ast) = ?

CONCRETE

• Definition

Concrete is a mixture of Cement, Fine aggregate, Coarse aggregate and Water

Concrete →
- → Cement (Binder)
- → Coarse aggregate (Strength)
- → Fine aggregate (Voids Filler)
- → Water (Hydration + Workability)

Introduction

* Concrete is one of the most important and useful materials for construction work

* Concrete is generally strong in compression and weak in Tension

Cement → Powder

Cement + Water → Cement paste

Cement paste + Fine Aggregate → Mortar

Mortar + Coarse Aggregate → Concrete

* Cement → Typically (10-15 percent of Volume of concrete)
  Water → 15-20 percent
  Aggregate → (FA + CA) – 60-70 percent

* Concrete has high Compressive Strength

* It means it can hold a lot of weight without getting crushed.

* Concrete are mainly used in

    * Residential building
    * Industrial building
    * Multistorey building
    * Bridges
    * Highway
    * Skyscrapers
    * Road
    * Side walk

* Excellent material for building and Road Foundation

* Due to some advantage technology now concrete are mostly prepared at batching plant and then transport to construction place.

* Some other Ingrediant are also used such as

    * Admixture
    * Pigment
    * Fiber
    * Polymer

Cement and Lime → Generally used as Binding material

Sand → Used as Fine aggregate

Crushed Stone, Gravel, Broken bricks } → Used as Coarse aggregate

Advantage of Concrete

* Compressive Strength is very high
* More Economical
* Ability to be cast
* More Energy efficient
* Excellent resist to Water
* High Temperature resistance
* Fire resistance
* Aesthetic properties
* Ability to work with reinforcing Steel.

Disadvantage of Concrete

* Tensile Strength is very low
* Lower ductility
* Volume Instability
* Formwork is Needed
* Low Toughness
* Long Curing Time

Grade of Concrete

* Grade of concrete is defined as the minimum strength of concrete after 28 days of construction with proper Quality Control.

* As per IS: 456 - 2000

We use different Grade of Concrete for different part of building.

* There are various Grade of Concrete available in the market based on these ratio

Example

Grade of Concrete	Mix proportion
M20	1 : 1.5 : 3

The concrete Grade is represented by the symbol (Mx), Where

M 20

M → Mix

20 → Characteristic Compressive Strength (fck) of the Concrete in N/mm^2 @ 28 days

The Mix proportion of concrete is expressed as
(Quantity of cement : Qty of FA : Qty of CA)

1 : 1.5 : 3

1 → Cement

1.5 → Fine Aggregate

3 → Coarse Aggregate

Unit: N/mm^2 (or) MPa, Its symbol is fck

As per IS:456-2000 the Concrete of different grades has the following proportion of Cement, Sand and Aggregate

TYPE OF CONCRETE	CONCRETE GRADE	MIX RATIO	CHARACTERISTIC COMPRESSIVE STRENGTH OF CONCRETE AT 28 DAYS IN N/mm^2	NATURE OF THE WORK
Ordinary Concrete	M5	1 : 5 : 10	5 N/mm^2	Heavy Wall, Foundation, Footing
	M7.5	1 : 4 : 8	7.5 N/mm^2	Mostly used in PCC Work
	M10	1 : 3 : 6	10 N/mm^2	Paving slabs, Footpath Works
	M15	1 : 2 : 4	15 N/mm^2	RCC Work, Slab, Beam, Columns, Walls
	M20	1 : 1.5 : 3	20 N/mm^2	Water Retaining Structures, RCC Structure
Standard Concrete	M25	1 : 1 : 2	25 N/mm^2	Heavy load RCC, National highway
	M30	Design Mix	30 N/mm^2	For Post-Tensioned of Prestress Concrete
	M35	" "	35 N/mm^2	For Pre-Tensioned of Prestress Concrete
	M40	" "	40 N/mm^2	Piling Work, Septic Tank, Foundation
	M45	" "	45 N/mm^2	
	M50	" "	50 N/mm^2	
	M55	" "	55 N/mm^2	
High Strength Concrete	M60	" "	60 N/mm^2	
	M65	" "	65 N/mm^2	
	M70	" "	70 N/mm^2	
	M75	" "	75 N/mm^2	
	M80	" "	80 N/mm^2	

* Actually H20 is a standard Mix

Means it is Mix proportion according to standard materials to be used.

* H 20 → Standard working on site

Minimum Grade of concrete recommended by IS Code

* For simple construction like

1 Storey } Building 1:2:4 is adopted
2 Storey }

* Home construction (Range of Mix ratio start from)

(1:5:10 to 1:1.5:3)

Type of Concrete Mix

There are two types of Concrete Mix

* Nominal mix
* Design mix

Nominal mix

* Nominal mix is for small projects
* Nominal mix consider OPC : Grade 33 which is not used Now a days
* What we use at site is either Grade 43 or 53

* It is used in ordinary Concrete involving Concrete grade not higher than M20

* Nominal mix specifies the proportion of cement, sand and aggregate

* Trail mix concept is consider

* No laboratory test is Conducted

* There is no scope for any deviation by the designer

* Grade of Concrete (Nominal mix)

  M5, M7.5, M10, M15, M20.

* Nominal mix has volumetric batching

Design Mix

* Design mix is for Major project

* It is adopted Higher Grade Concrete

* Design mix specifies the proportion of Cement, Fine aggregate Coarse aggregate and Admixtures (Each and every Ingrediants)

* Not much consideration Trail mixes

* Laboratory test Conducted

* Design mix are mixed as per the requirement of Engineer

* Grade of Concrete (Design Mix)

  M25, M30, M35, M40 ..., M80

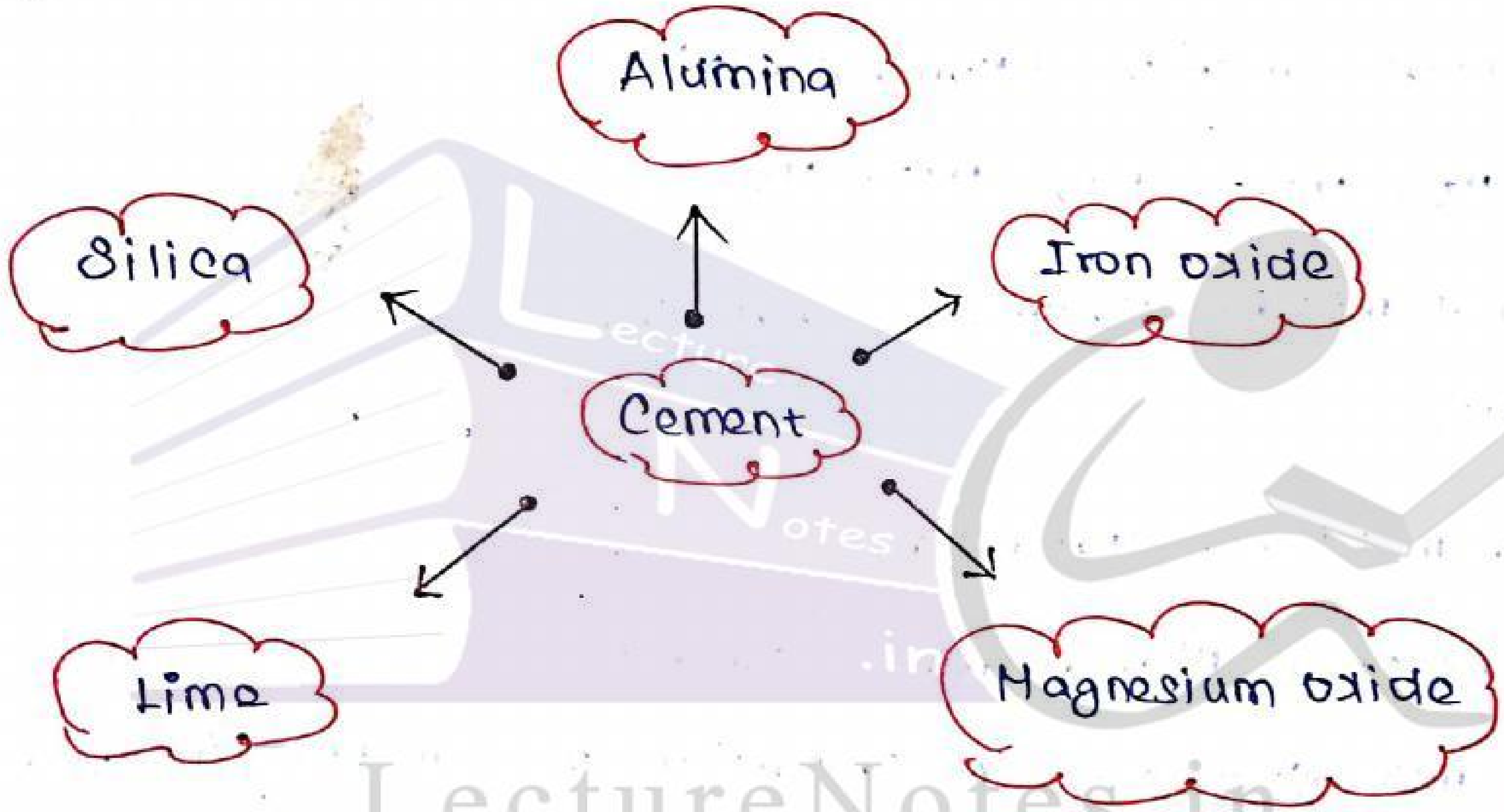
* Design mix has Weigh batching

CEMENT

Introduction

* Cement is one of the most Import building material which binds the materials together

* Cement is a mixture



* Cement are used with

→ Fine Aggregate + Cement = Mortar

→ Cement + Fine aggregate + Coarse aggregate = Concrete

History of Cement

→ **1756 - John Smeaton**

Hydraulic lime can resist the action of Water which can be obtained from the hard time

→ **1796 - Joseph parker**

Hydraullic cement made from the burning of argillaceous limestone, which was led to the invention of portland cement

Argillaceous limestone → Hydraullic cement → Portland cement

Portland Cement

portland cement is the most common type of cement in general use Today

→ **1824 - Joseph Aspin**

* He was trying to fire the mix of Finely ground Limestone and clay with water in his kitchen

(Finely Ground Limestone) → Clay → Water

* After a while, he noticed that the mixture was set and Hard
* He found that it was Cement which bonded and hardens the mixture
* He considered his cement resembles the Jurassic portland Stone of Dorest England.

Cement → Jurassic portland Stone Of Dorest England

* He Named the cement as portland cement

→ 1828 - Thomas River Tunnel

First engineering structure made of cement was Thomas River Tunnel in 1928

Type of Cement

* Ordinary Portland Cement [OPC]
* Portland Pozzolona Cement [P.PC]
* Special Cement

Ordinary Portland Cement

This type of cement gives enough compressive strength after soaking in water for 3 days, 7 days and 28 days. This is suitable for all type of civil engineering constructions

* The OPC is popularly known as Grey cement
* Indian Standard has classified OPC in three Grades based on the strength of cement

These Grades are

Grade - 33

Grade - 43 → Plastering Walls, Non- Rcc structures, Pathways

Grade - 53

Note:

53 Grade Cement are used for Fast placed construction were Initial strength is to be achieved quickly

53 Grade cement has Fast setting compared to 43 Grade cement

Grade attain 27 MPa in 7 days

23 MPa for 43 Grade

If we use higher the Grade of cement, it gives the greater would be the economy, durability and Technical advantages. More.ver construction time is also provided

Portland Pozzoloana Cement

* This cement is provided by adding 10 to 25% pozzolanic materials to the opc clinker then Grinding Together
* It is cheaply manufactured because id uses Fly ash, burnt clay, coal waste as the main ingrediant
* PPC has a Lower heat of hydration, which is of advantage in preventing cracks where large volumes are being cast

Difference between OPC Cement And PPC Cement

* Portland pozzolana cement is a variation of OPC. Pozzolana materials namely Fly ash, Volcanic ash, are added to the OPC So that it becomes PPC. pozzolana materials are added to the cement in the ratio of 15% to 35% by weight
* Both are ecofriendly materials but pozzolana Cement uses Natural and Industrial waste thus reducing the environmental pollution.
* OPC is available in 3 Grades namely Grade 33, 43, 53. Pozzolana is available in one Grade and its Strength matches the strength of Grade 33 OPC after curing
* OPC is the most commonly used Cement in construction. PPC is highly resistant to sulphate attacks hence its prime use is in construction of dams, Foundation, buildings near the sea shore, reservoirs, marine construction to name a few
* PPC has low Initial setting Strength compared to OPC but hardens over a period of time with proper curing
* PPC is cheaper than OPC → (The time at which cement paste loss its plasticity is called Initial setting Time)
  (i) Initial Setting Time – 30 min
  (ii) Final setting Time – 10 hours → (Cement paste completely loss its Plasticity)

Preparation of Cement

JURASSIC PORTLAND STONE OF DOREST ENGLAND

Stone rock

↓

Crushed

↓

Calcining

↓

Burning

↓

Fine powder

↓

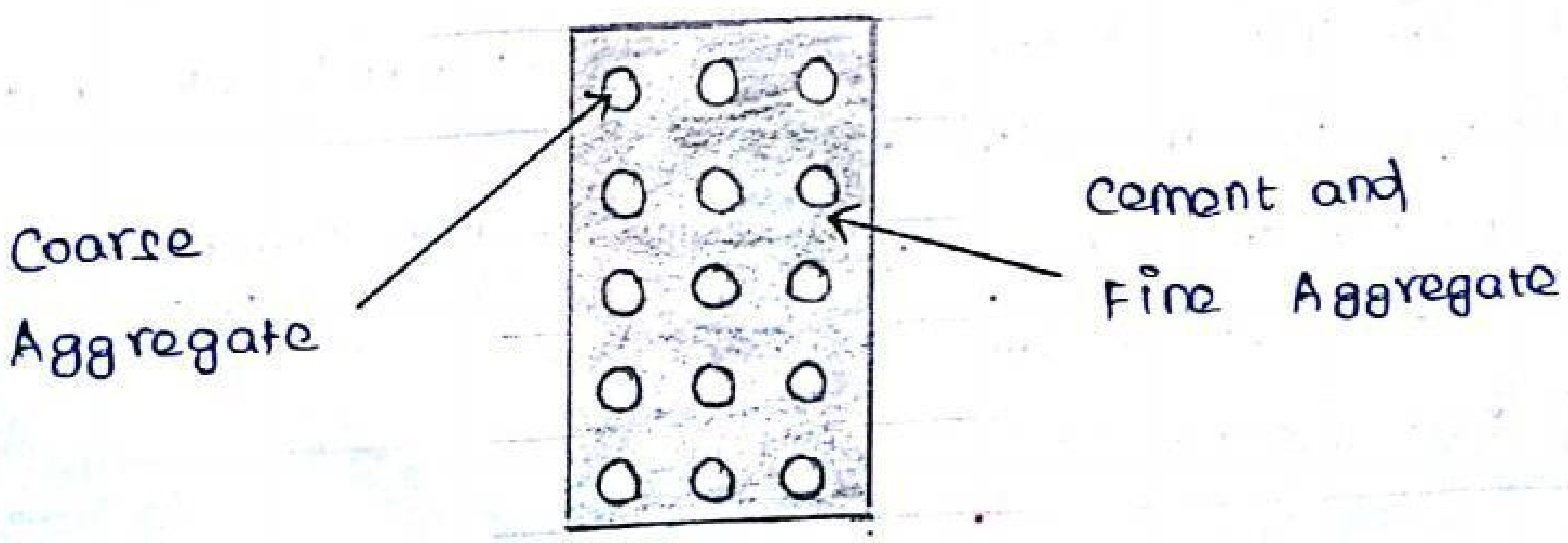
Cement

↓

Portland Cement

The Role of Aggregate in Concrete

* Strength * cost



Aggregates are the Major Component of any concrete it constitutes about 70-75 percent of the Volume of concrete.

* They are easily available and they are economical as compared to cement
* Aggregate provide additional strength to the concrete
* Aggregates help in binding the cement properly
* If the concrete is made only with cement it will shrink more and relatively more water is required for the cement to form workable paste so more amount of water means decrease in strength, and shrinkage causing cracks in the concrete So it is necessary to use aggregate as they decreases the cement content and so water content and increasing the its strength

Size of Aggregate

* Other types of aggregates such as slag and crushed overburnt brick or Tile, which may be found suitable with regard to strength, Durability of concrete
* According to IS:456-2000, the size of coarse aggregate shall not exceed $\frac{1}{4}$ th of the Thinnest structural member. At the same time, the size of stone chips or aggregate shall be 5mm less than the minimum spacing of steel bars
* Generally the size of aggregates is taken as 20mm for general RCC, which may be reduced to 10mm for Heavily reinforced section likes bridges. The size of stone chips may be taken as 40mm for mass concreting.

MORTAR

It is a combination of

* Cement
* Sand
* Water

Use of Mortar

* Masonry work
* Brick masonry
* Stone masonry
* Plastering Wall
* Column etc...

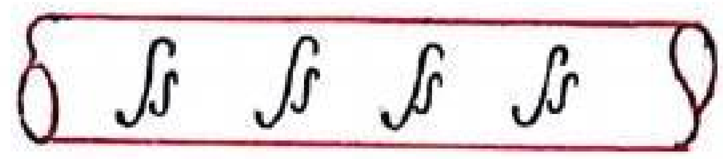
Mortar Mix Ratio

MIX RATIO	NATURE OF THE WORK	THICKNESS
1:3 or 1:4	Rcc Surfaces or Rcc Walls	6mm
1:3 or 1:4	Underside of Rcc slabs	6mm to 10mm
1:3 to 1:6	Masonry Walls	9mm to 12mm
1:3	Damp proof Course (DPC) Vertical	20mm
1:3	Plastering	1.5 mm
1:4	Ceilings	10mm
1:4	Smooth Wall Surfaces	12mm
1:4	Rough Side Surfaces	4.5mm or 15mm

STEEL BAR

Steel →
- → Mild Steel (Fe 250)
- → HYSD (Fe 415, Fe 500)

MS PLAIN
Only 6mm ϕ available



HYSD (Ribbed)
Min 8mm, 10mm, 12mm, 16mm, 20mm
25 mm, 32mm, 36mm...,

Grade of Steel

Fe 250 – Tensile Strength 250 N/mm^2

Fe 415 – Tensile Strength 415 N/mm^2

Fe 500 – Tensile Strength 500 N/mm^2

Fe 250

Fe → Ferrous material (Iron)

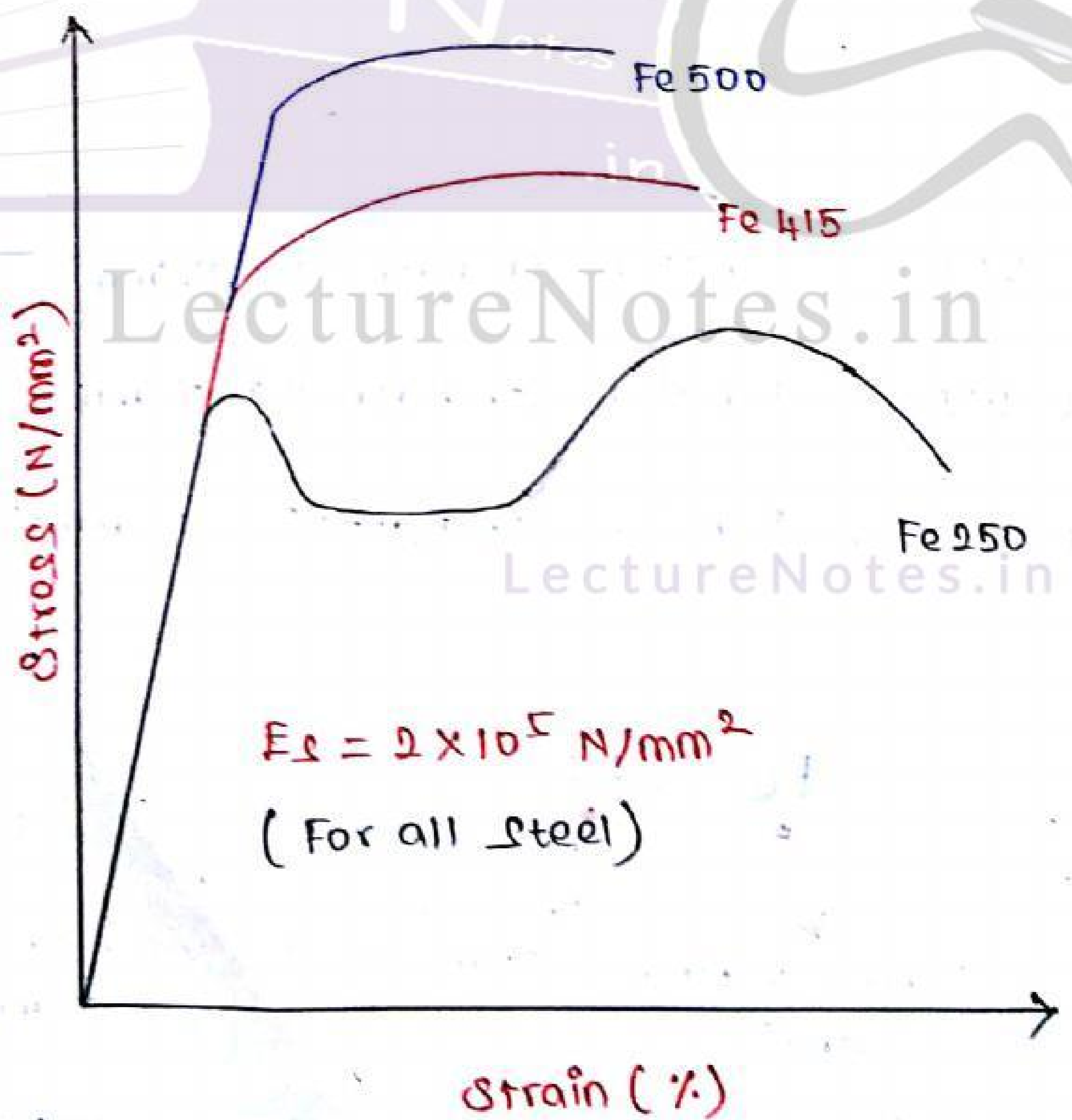
250 → characteristic yield strength of Steel in N/mm^2

Some additional Grade of Steel

* Fe 415 D
* Fe 500 D
* Fe 550 and Fe 550 D $\downarrow$ (Recently Introduced)

Where,

D - Higher ductility

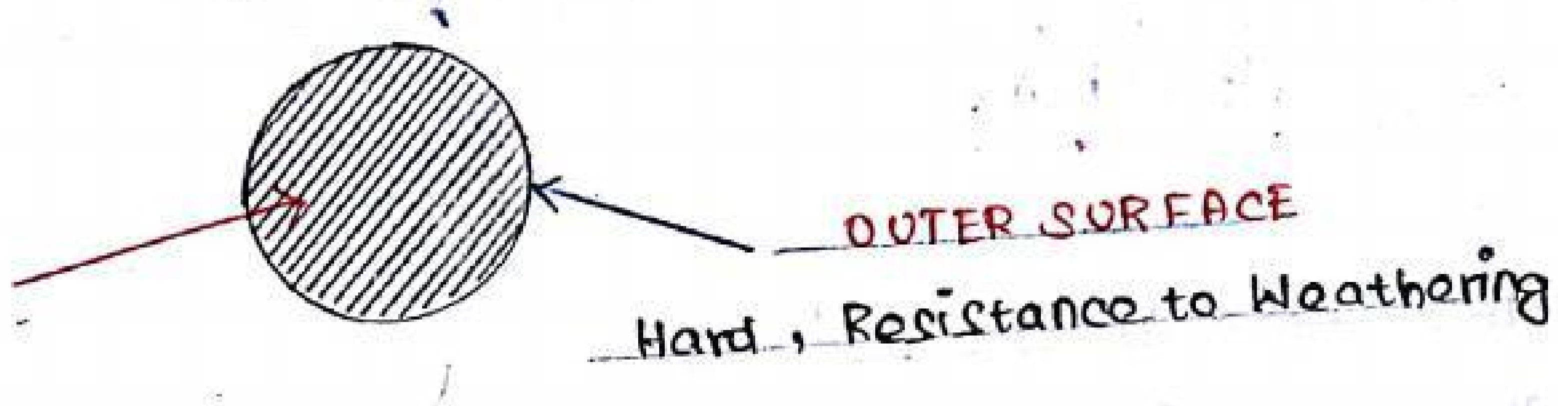


E_s - Young's Modulus

<u>STRESS - STRAIN CURVE FOR REINFORCING STEEL</u>

* Some advanced technology for also steel prevent in corrosion

TMT Bars: Thermo Mechanically Treated



Inner core : Soft and ductile

Outer core : Very high Tensile Strength.

STEEL

* Carbon Steel is an alloy consisting of Iron and carbon. Several other elements are allowed in carbon steel with low maximum percentages.
* Based on the carbon content in steel, it can be classified into 3 as

			Uses
(1)	Low carbon steel (or) Mild steel	- upto 0.25%	Motory body, Sheet metal, Structural steel etc..,
(2)	Medium carbon steel	- 0.25% - 0.7%	Making Springs, Tyres Rails
(3)	High carbon steel	- 0.7% - 1.5%	Hammers, Drills, Tools, axes.

* Steel is used Manufacturing Factories control condition
* Quality control of steel is much easier and better than concrete

* Mostly we use word

Rebars → Reinforcement bars

↓

Steel bars

Type of Reinforcing Bars

There are Two types

(1) Mild steel (or) plane steel - Fe 250

(2) HYSD bars (or) Tor Steel bar (or) Ribbed bars (or) Deformed steel bars } Fe 415, Fe 500

Mild Steel

* Round section, which are made up of mild steel and it has more ductile (Able to drawn into Long thin wire without breaking)
* But the present trend in India is to go Tor. Steel bars
* Mild Steel is the common form of steel available for Fabrications
* Ductility is high due to Low carbon content without any alloying elements

  Carbon content - 0.05% - 0.25%

* Mild steel bars are plain bars. They have less bond stress properties
* The Typical yield strength of these bars is 250 Mpa
* Mild steel have good strength, Hard and can be bent, worked or can be welded easily
* Mild used for vehicles (Like cars and ship) to building materials
* It causes wide cracks due to suddenly yielding of steel at yielding point
* High amount of energy (strain) can be absorbed by them

* Elastic strain is high
* Building Industry frequently use Mild steel in construction because of its ductility and Malleability

High yield Strength Deformed bars (HYSD)

* These bars have ribs or projections on their surface and they are produced by controlled cold twisted of hot - rolled bars
* Each bar is to be Twisted Individually and it is tested to confirm the standard requirements
* The HYSD bars are available in sizes Varying from 6mm to 50 mm diameter
* The size of Steel bar are (6, 8, 10, 12, 16, 20, 25, 28, 32, 36, 40, 45, 50)
* HYSD is the latest and advanced form of steel used in the construction Industry
* It has High carbon content
* It is costlier, more strong and more hard
* The typical yield strength of these bars is 415 MPa, 500 MPa
* Higher Tensile strength
* It is used for heavy structures in civil construction
  Eg: Multi storey building
* Very less Elastic strain

Grade of Steel

S.No	GRADE OF STEEL	YIELD STRESS (f_y) CHARACTERISTIC STRENGTH OF STEEL	BOND EFFECT WITH CONCRETE
1	Fe 250	250 N/mm^2 or Mpa	Normal
2	Fe 415	415 N/mm^2 or Mpa	60% more bond
3	Fe 500	500 N/mm^2 or Mpa	60% more bond

Fe - Ferrous Material

* The Standard length of a Steel bar (or) a threaded Steel bar is 40 Feet (or) 12m
* Weight of Steel bar in(Kg/m) = $\frac{D^2}{162}$

Where D - Diameter of bar

Unit Weight

1	Unit Weight of RCC	2500 Kg/m^3
2	Unit Weight of PCC	2400 Kg/m^3
3	Cement	2440 Kg/m^3
4	Cement Mortar	2000 Kg/m^3
5	Steel	7850 Kg/m^3
6	Fine Aggregate	2350 Kg/m^3
7	Coarse Aggregate	2300 Kg/m^3
8	Brick work	1800 Kg/m^3

Difference between Pcc and Rcc

PLAIN CEMENT CONCRETE	REINFORCED CEMENT CONCRETE
1. Pcc is the mixture of cement, Sand, aggregate and Water in a proportion. Pcc is Strong in compression and weak in tension	For increasing tensile strength of concrete reinforcing material is provided in concrete. So concrete is equally strong in taking compressive and tensile
2. Plain cement Grade (Grade) M5, M7.5, M10, M15	Reinforced cement concrete (Grade) M20, M25, M30, M35, M40, M45 etc
3. Usage: Pcc is used as bed concrete * Footing * plinth beam * Flooring	Usage: Rcc is used as Structural work * column * Footing * Beam * Slab * plinth beam * Retaining Wall
4. Thickness: pcc is normally between 50-75mm	Thickness: Rcc thickness depend as per design requirement
5 Concrete Ratio: Normally used proportion are 1:2:4, 1:3:6, 1:4:8	Concrete Ratio: It can be Nominal mix or design mix depends on site requirement
6 Unit: Cubic meter (Cum)	Unit: Cubic meter (Cum)

Purpose of Steel used in Concrete than other Materials

* Practically equal the co-efficient of Thermal expansion on both concrete and Steel
* cost and cheap and widely available

* Fire Resistance
* Low Maintanence

Different Types of RCC Structures

* Multi - Storey building
* Dams
* Sleepers
* Water Tank
* Bunkers and silos
* Shells
* Harbours
* Retaining walls
* pipes
* Towers
* Culverts

Main Components of RCC Structural Members

* Beam
* Column
* Slab
* Foundation and Footings

Load and Forces

1) Dead Load (DL) IS: 875 part-I (1987)
2) Live load (or) Imposed load (LL) part-II
3) Wind Load (WL) part-III 4) Snow Load Part-IV
4) Earthquake load (EL) IS: 1893-84 2002

Dead Loads (DL)

Self weight of the structural members

Imposed Loads (or) Live Load (or) Service Load

These are loads that change with respect to Time

* Live load Include the loads due to people occupying the Floor and those due to materials stored

Wind Load (WL)

Wind load have to be considered in the design of multi storey building, Towers and poles

Earthquake Loads

Seismic (or) earthquake forces have to be considered in the design of structure located in seismic zone according to IS: 1893 : 84 2002

Type of Forces acting on a Structural member (Internally)

* Tensile force
* Compressive force
* Shear force

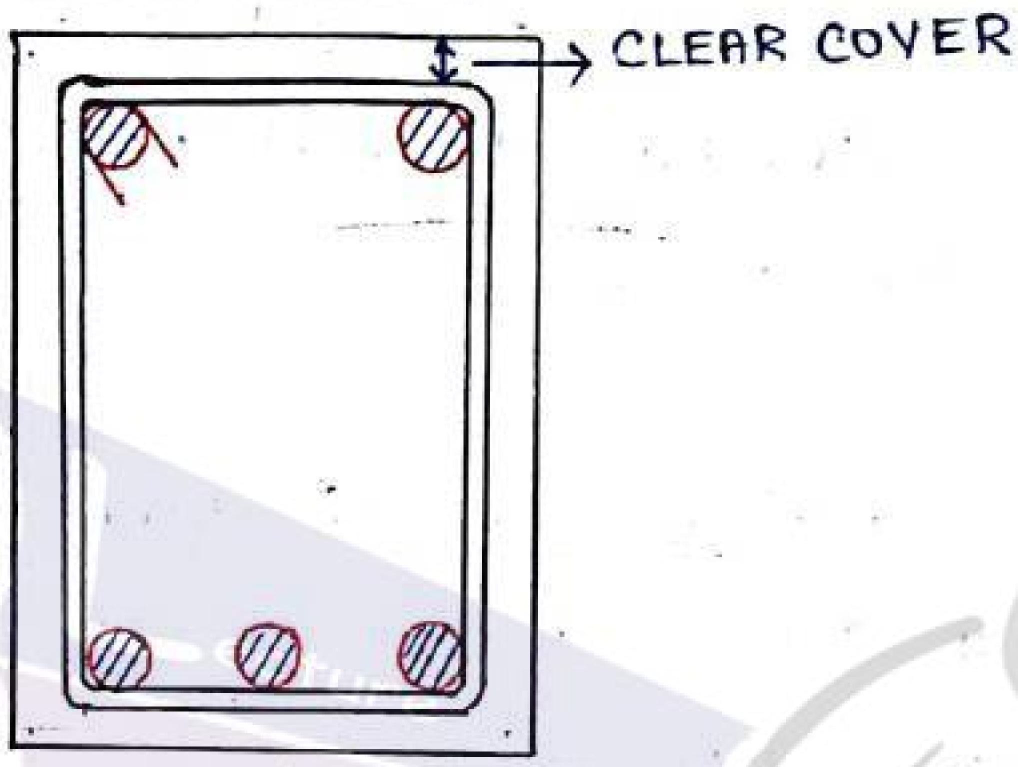
Concrete Cover

Cover

Distance b/w the Face of Concrete & Rebar

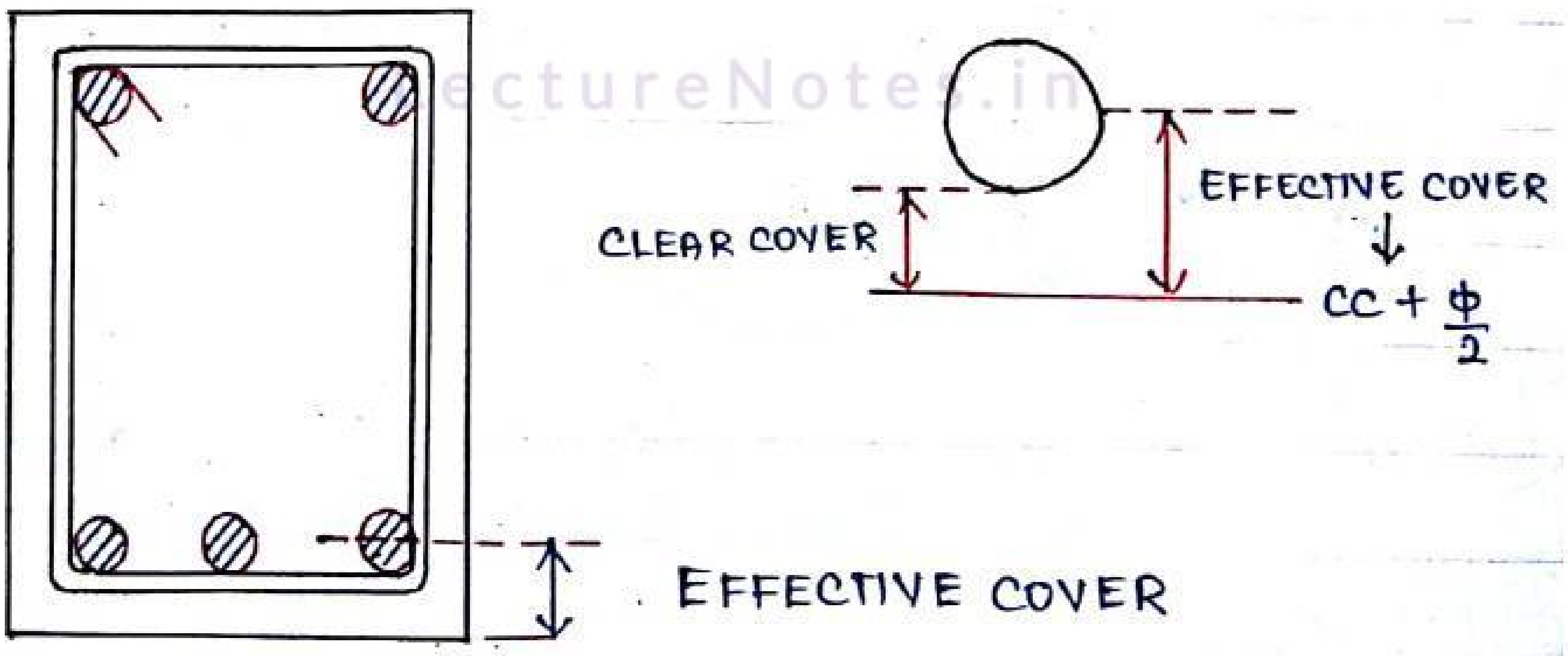
- Concrete cover for reinforcement is required to protect the rebar against corrosion and to provide resistance against fire

Clear Cover



Clear Cover (CC)
- → (From) End of the bar
- → (To) Bottom of Slab (or) Beam

Effective Cover



Effective cover
- → (From) Half of the Dia bar
- → (To) End of the beam

CLEAR COVER TO MAIN REINFORCEMENT

1	RCC Slab	15mm
2	RCC Beam	25mm
3	RCC Column	40mm
4	RCC Footing	50mm
5	Water Tank	25mm
6	Staircase	15mm
7	Retaining Wall	40mm

Column → Min. Reinforcement – 12mm φ , Max. Reinforcement – 50mm φ

Slab → Main. Reinforcement – 8mm φ (or) 12mm φ , Dist. Reinf – 8mm φ

Stirrups → Min. Reinforcement – 6mm φ , Max. Reinforcement – 8mm φ

INDIAN STANDARD CODE FOR CIVIL ENGINEERING

* If you are Civil Engineer you dont Know IS Code
* you are not able to any construction work in Civil Engineer
* IS Code Can gave more Knowledge about our project in Civil Engineering
* Every Engineers Should be Learn
  * Site Engineer
  * Design Engineer
  * Site Supervisor
  * Sub Engineer
* The Given IS code are used for Civil Engineering, for the purpose of designing and analysis of civil Engineering Structure like
  * Building
  * Road
  * Dam
  * Airport
  * Highway etc..,

IS CODES FOR R.C.C

For the design of R.C.C members the Ingrediants of R.C.C Should confirm to the Standard Specified in the relevant Codes by the Bureau of Indian Standard. The codes are

1. Plain and Reinforced Concrete
   IS : 456 - 2000

2) IS Codes for Cement

The cement used in R.C.C work may be confirming to any of the following

* IS - 8112 - (43 Grades Ordinary Portland Cement)
* IS - 12269 - (53 Grades Ordinary Portland Cement)

There are many other varieties of cements available for use in specific situations but for ordinary R.C.C work any of the Two cement are used

3) IS Codes for Aggregate

* The coarse Aggregate and Fine Aggregate used in R.C.C work shall conform IS : 383
* For most of the R.C.C work Coarse aggregate of Nominal size 20mm is used.
* Fine aggregate used shall be the one passing through IS sieve of 4.75 mm size

4) IS Codes for Steel Reinforcement

The steel reinforcement used in R.C.C shall be any of the following

* Mild steel and Medium Tensile steel bars } IS : 432 (Part-1)
* High strength Deformed Steel bar } IS : 1786

UNIT-1

DESIGN OF BEAM BY LIMIT STATE METHOD

CONCEPT OF RC DESIGN

* Reinforced Cement Concrete
* Use of Rcc
* Advantage and Disadvantage of Rcc
* Materials used in Rcc
* Properties of Rcc
* Type of Structural Elements
* Method of RC design

REINFORCED CEMENT CONCRETE

* It is a Combination of Concrete and Steel
* Concrete is very strong in Compression and weak in Tension
* To Increase Tensile strength of Concrete, the reinforcement steel bar is inserted on the Tension zone of the member as steel has got very Tensile Strength
* The Composite action of Steel and Concrete in a reinforced Concrete section is depend upon Following Factor
    * The bond between steel and concrete
    * preventation of corrosion of steel bars embeded in steel
    * practically equal thermal expansion on both Concrete and Steel

Use of Rcc

It is used in the Construction

* Column
* Beam
* Slab
* Footing
* Lintel
* Chajjas
* Roof
* Piles

It is used in Storage Structures

* Dams
* Water Tank
* Tunnel
* Bins
* Silos and Bunkers

It is used to Build Heavy Structures

* Bridges
* Walls
* Towers
* Under Water Structures
* Dock and Harbours

It is used in Tall Structures

* Sky Scrapers
* Chimney
* Multi Storey building

It is used in Pavement

* Road
* Airport

It is used for Precast

* Railway Sleeper
* Electric poles

Advantage and Disadvantage of Rcc

Advantage of Rcc members

* High Compressive Strength
* More Economical
* The Materials used in Rcc Construction are easily available
* Fire and Weather Resistance
* less deflection
* Low Maintanence Cost
* Aesthetic properties
* Mouldability : Rcc Section can be given any shape easily by the property designing formwork
* Seismic Resistance : Properly designed Rcc Structure are extremly resistance to Earthquake

Disadvantage of RCC members

* Rcc needs lot of Formwork, Centering and shuttering to be Fixed, required lot of Site Space and Skilled Labours
* Concrete take time to attain in full Strength. These Rcc Structure can't be used immediately after Construction unlike Steel Structure
* Rcc Structure are heavier than Structure of other material like Steel, Wood and Glass etc..,

Materials Used in Rcc

The Materials used in Rcc Construction are

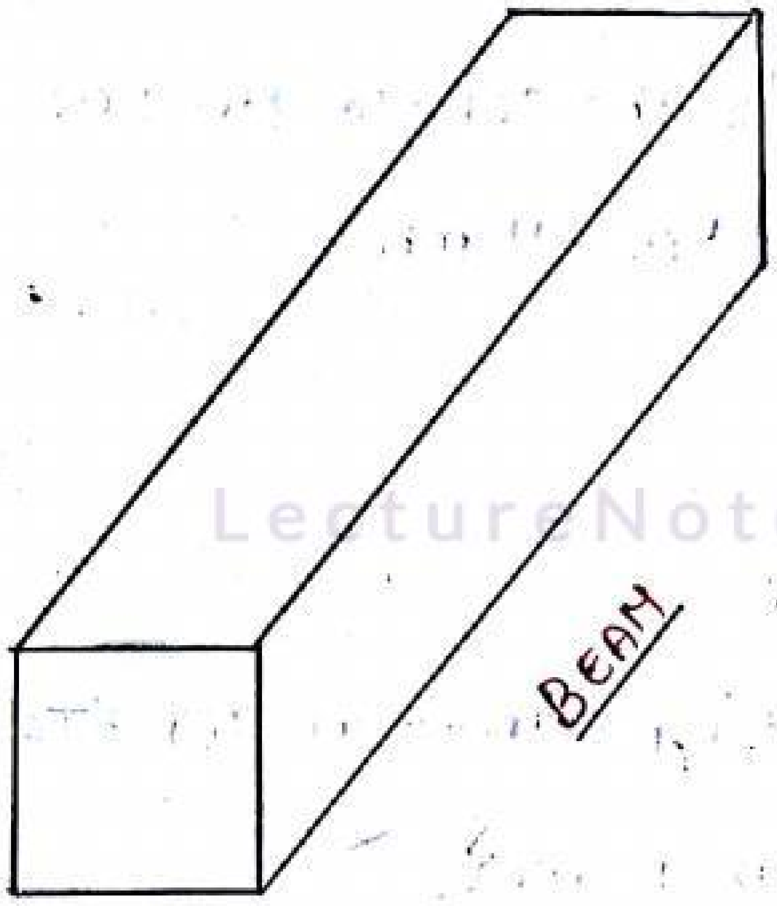
S.NO	MATERIALS	NATURE OF THE WORK
1	Cement	Acts as binding materials
2	Coarse Aggregate	This acts as a Filler material and generally made of crushed Stone in Graded Sizes
3	Fine Aggregate	This also acts as a Filler materials and generally it consists of River Sand
4	Water	The Main function of Water in Concrete is to react with Cement and Lubricate the aggregate particles

5	Steel Reinforcement	This consists of steel bars of various diameter provided to resist the Tensile stresses in the Rcc Member

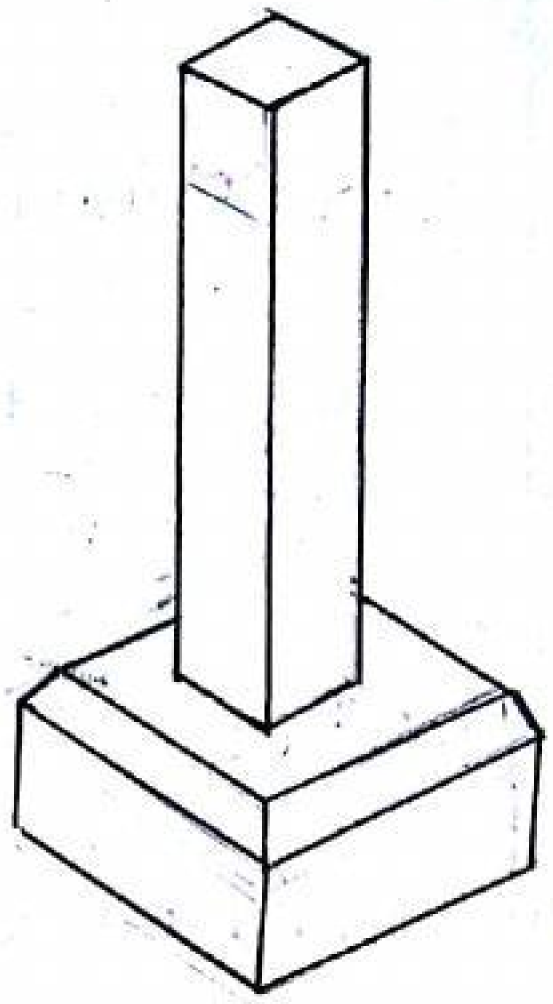
Properties of Rcc

* It should be capable of Resisting expected Tensile, Compressive bending and shear forces
* It should not show excessive deflection and spoil serviceability requirement
* There should be proper cover to the reinforcement, so that the corrosion is prevented
* The Hair crack developed should be within the permissible limit
* It is a Good Fire Resistant material
* Durability is very Good
* Rcc structure can be designed to take any load

DESIGN OF DIFFERENT TYPE OF STRUCTURAL ELEMENTS

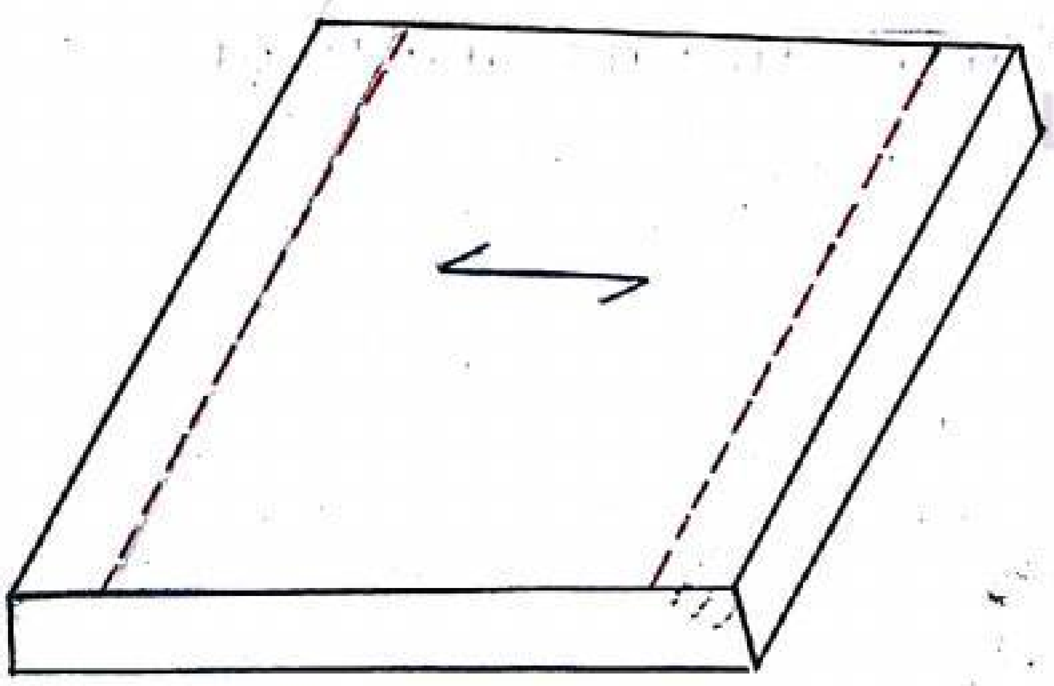


BEAM

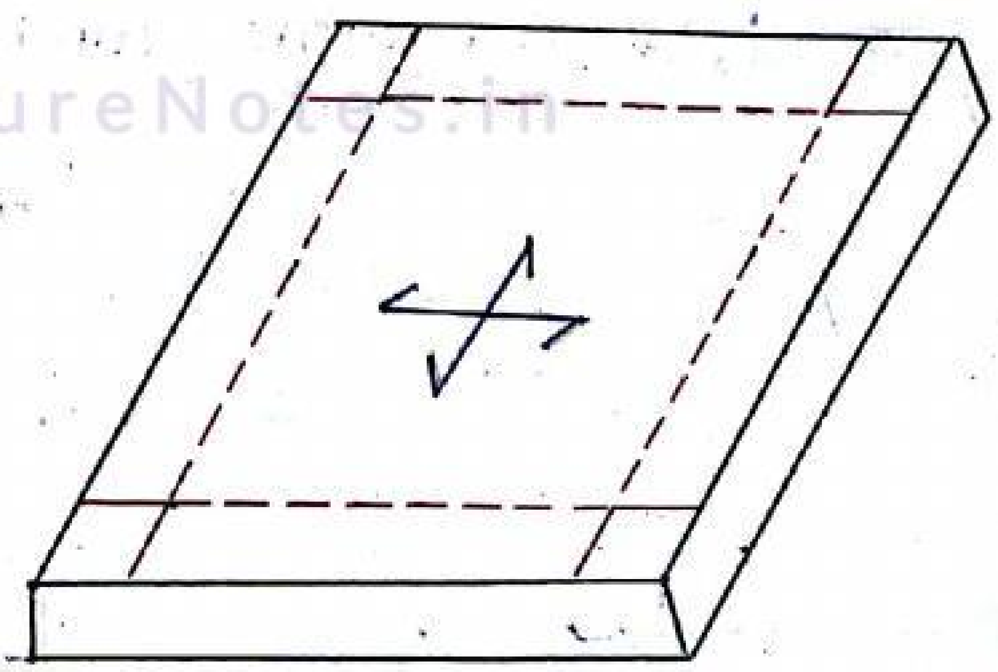


FOOTING

COLUMN



ONE WAY SLAB



TWO WAY SLAB

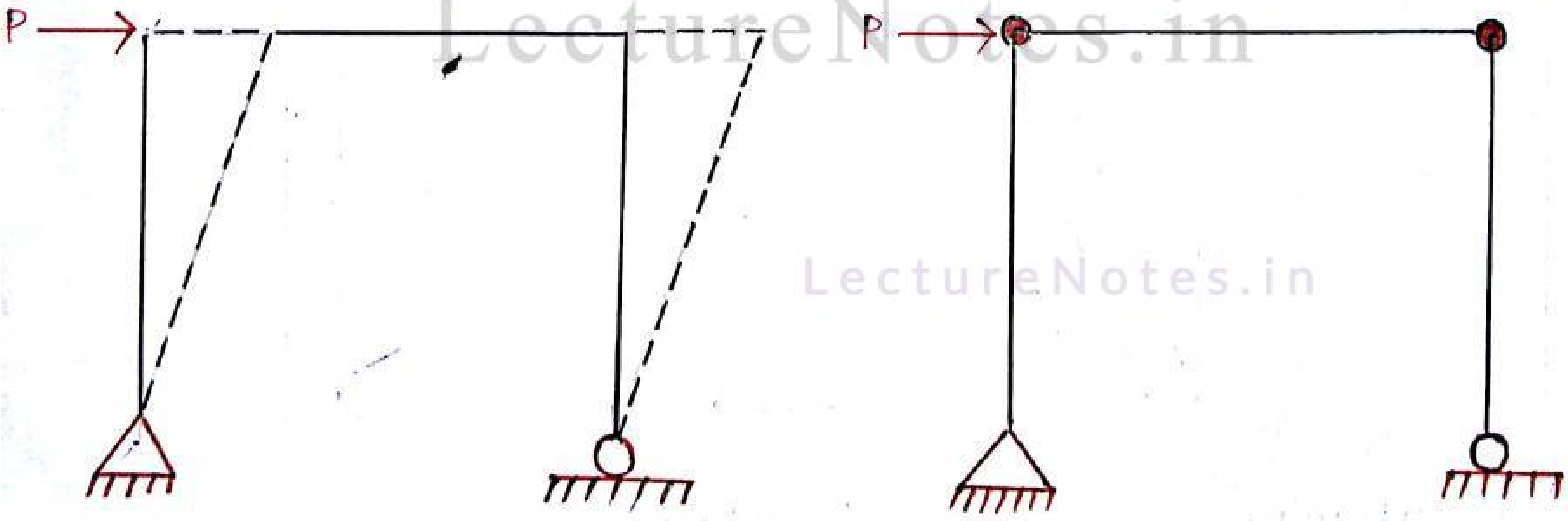
Objectives of Structural Design

- The design of a structure must satisfy basic requirements

  * Stability
  * Safety
  * Serviceability
  * Durability
  * Economic
  * Aesthetic

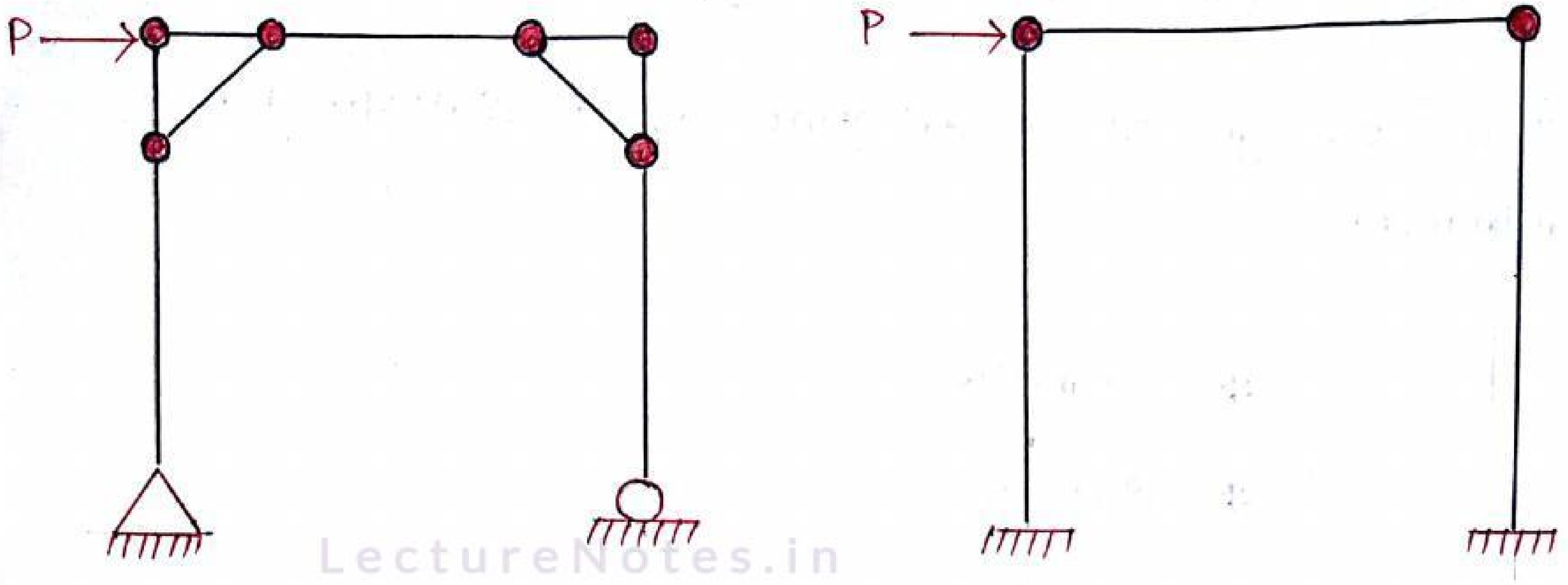
Stability

The structure should be stable. Stable means it should resist overturning or Buckling



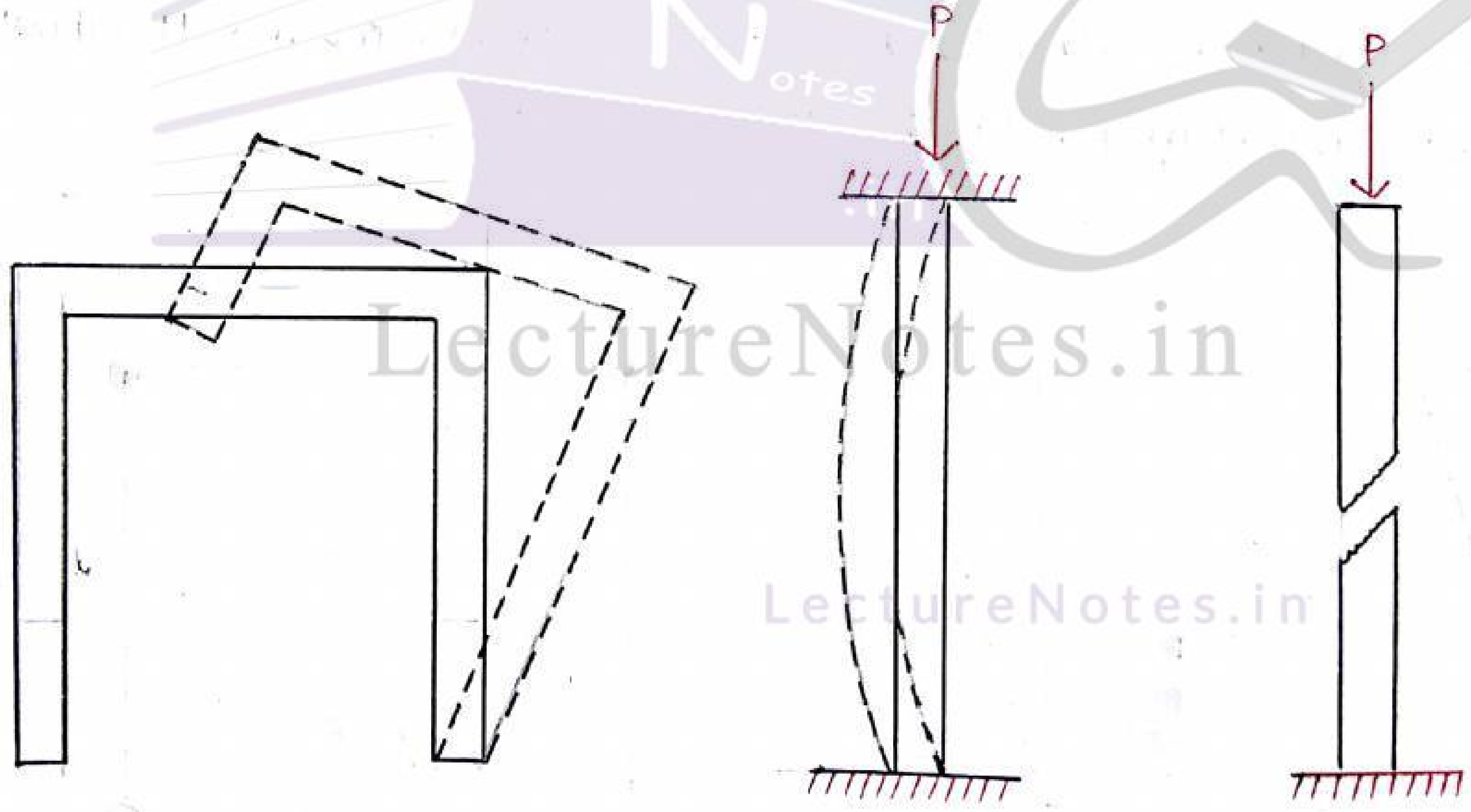
Unstable

Stable



STABLE AND UNSTABLE STRUCTURES

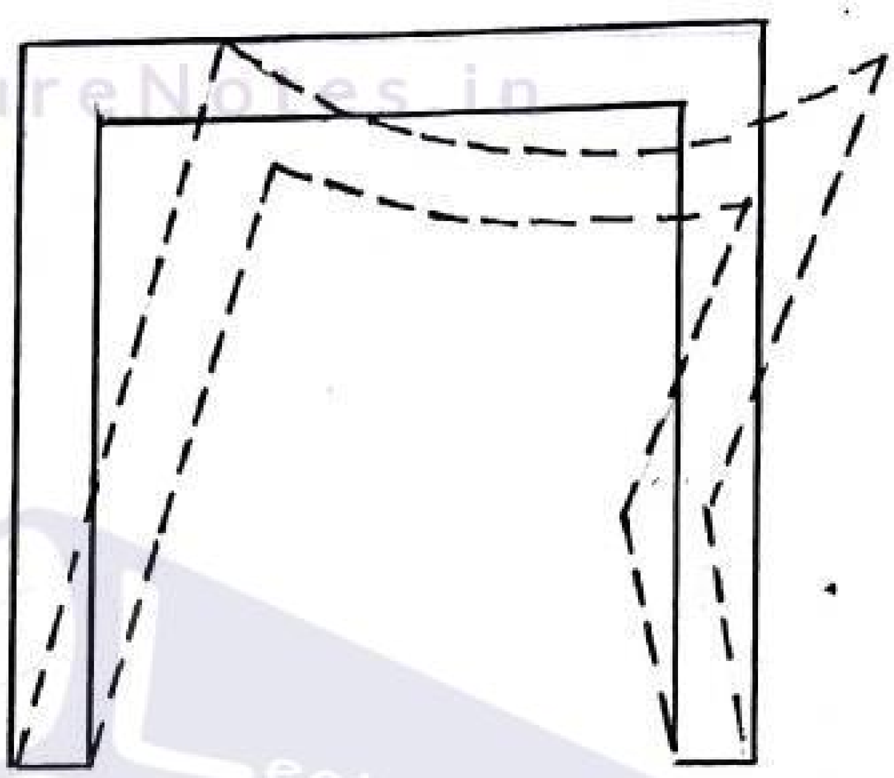
* The Structure Should not move (By Sliding or overturning)
* Stability to prevent overturning, Sliding or Buckling of the Structure under the expected load.



Overturning Buckling Slide

Safety

- The structure should be able to carry all Expected load safely, without failure, that is without breaking or collapsing under the load.



Weak Structure

Serviceability

Structure should provide the service for which it is constructed up to its whole life under service load.

Durability

Structure should sustain loading for which it was design and should perform well with safety and serviceability up to its Intended life

The structure should last for a reasonable period of Time

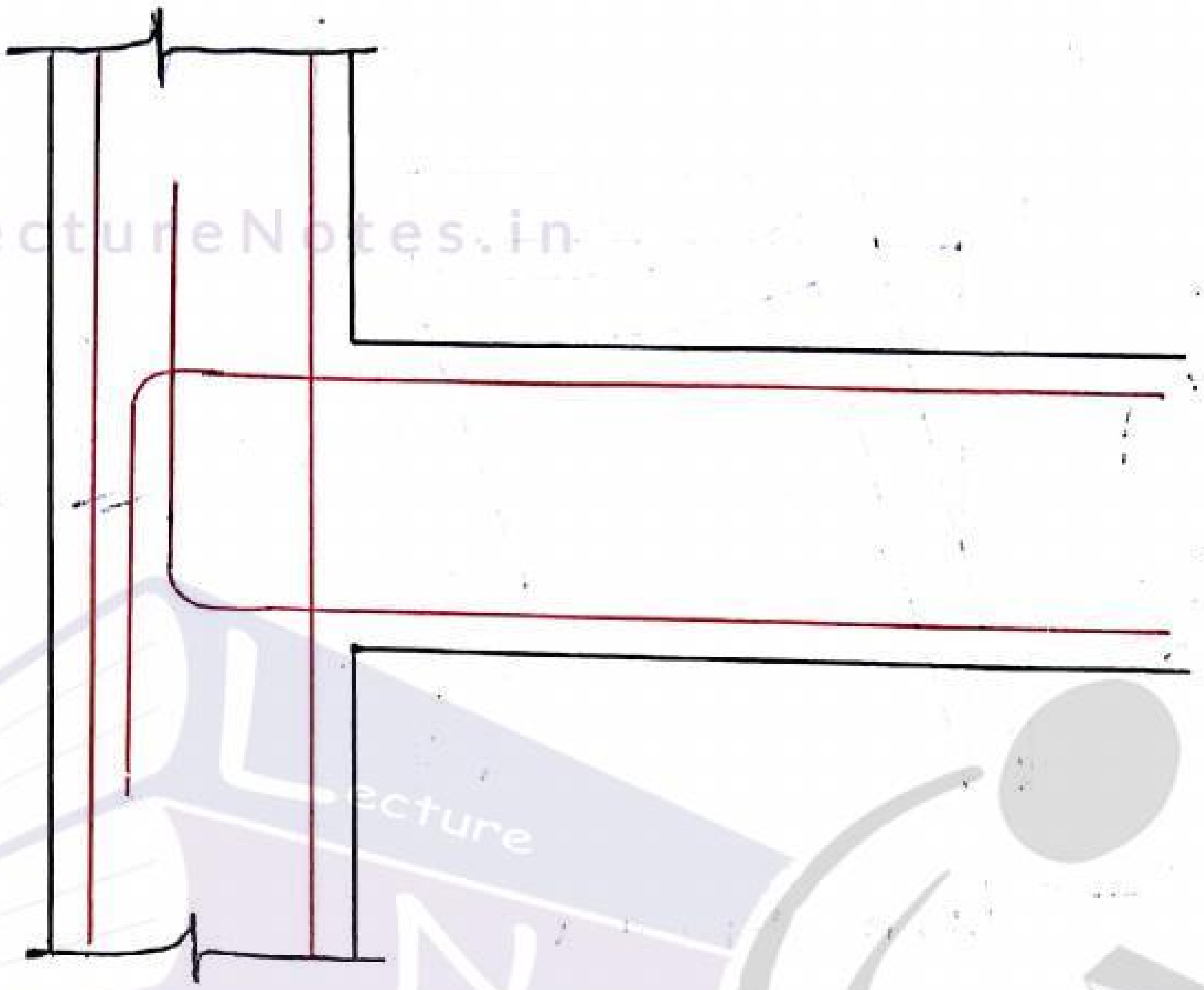
Aesthetic

When Huge Investment is involved with design and construction of a structure, then aesthetics affects design of structure.

Example:

Considering a RCC Beam

Safety



Longitudinal Reinforcement

* These bars are provided in the beam to resist bending cracks.

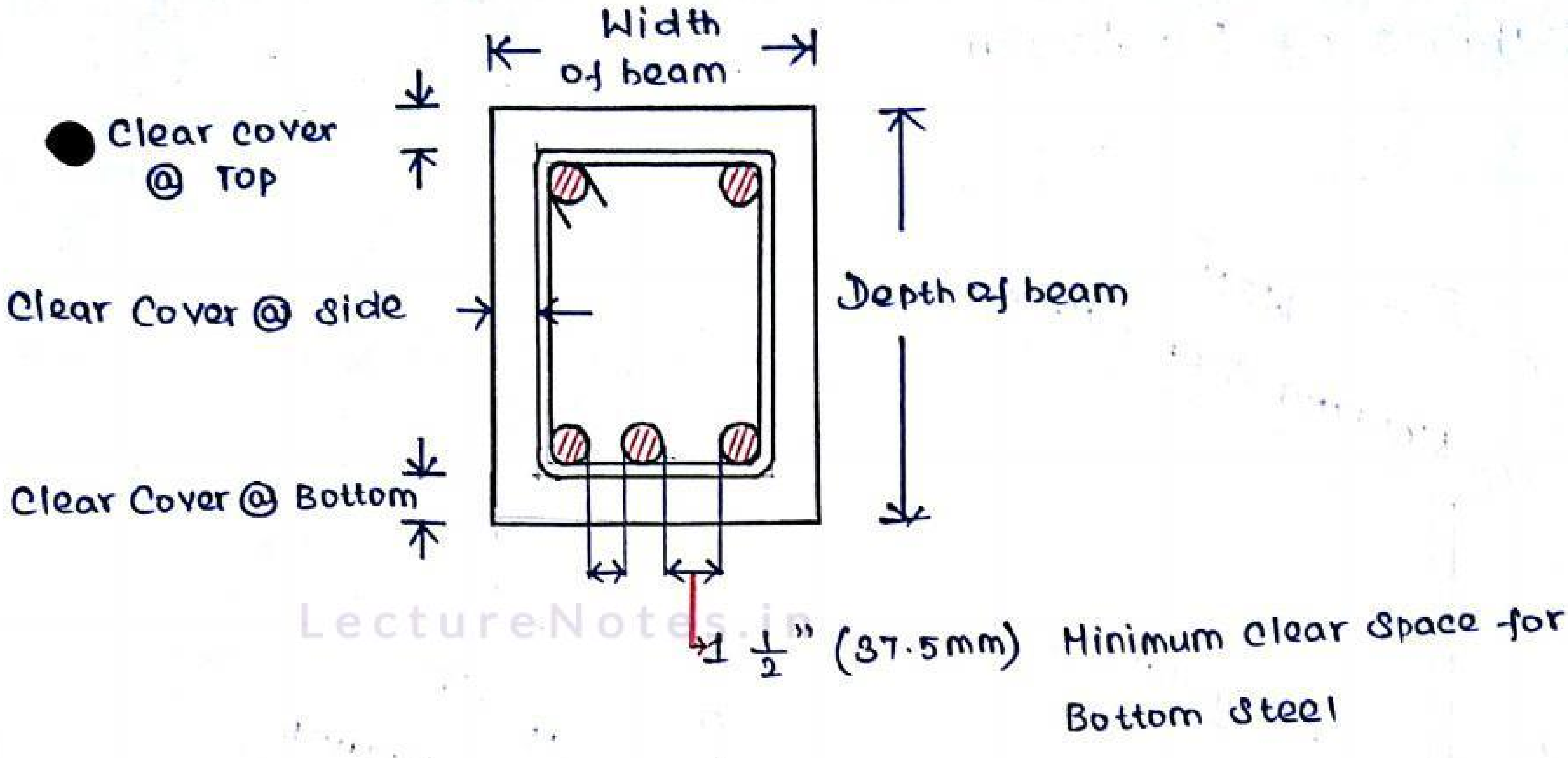
* Top and Bottom Reinforcement shall consist of at least two bars throughout the member length

Serviceability

Double reinforced in place of single reinforcement to reduce depth

Durability

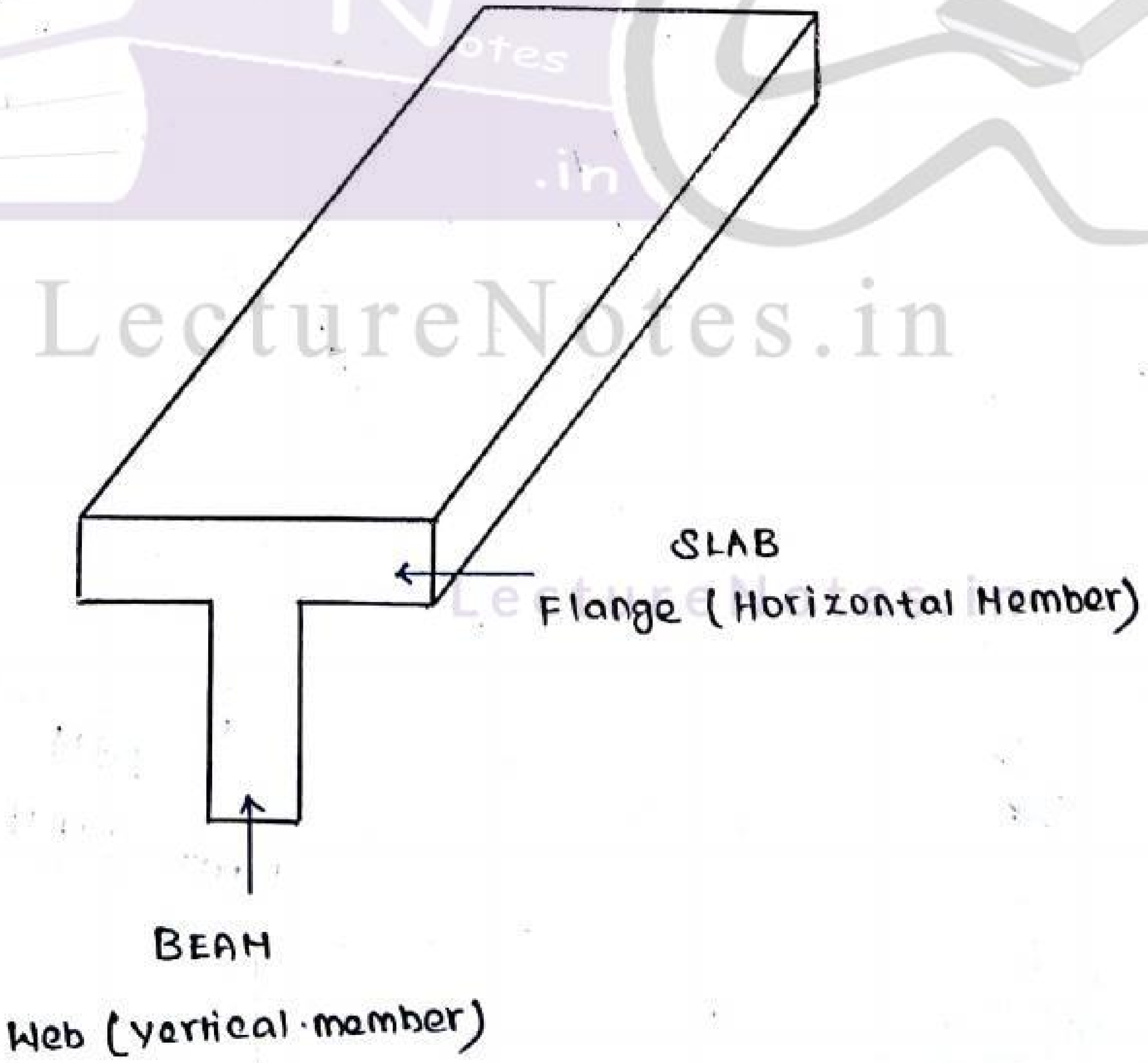
Nominal Cover to protect reinforcement from corrosion material of beam.



Instead of using <u>Clear Cover</u> use the word <u>Nominal Cover</u>

<u>Economy</u>

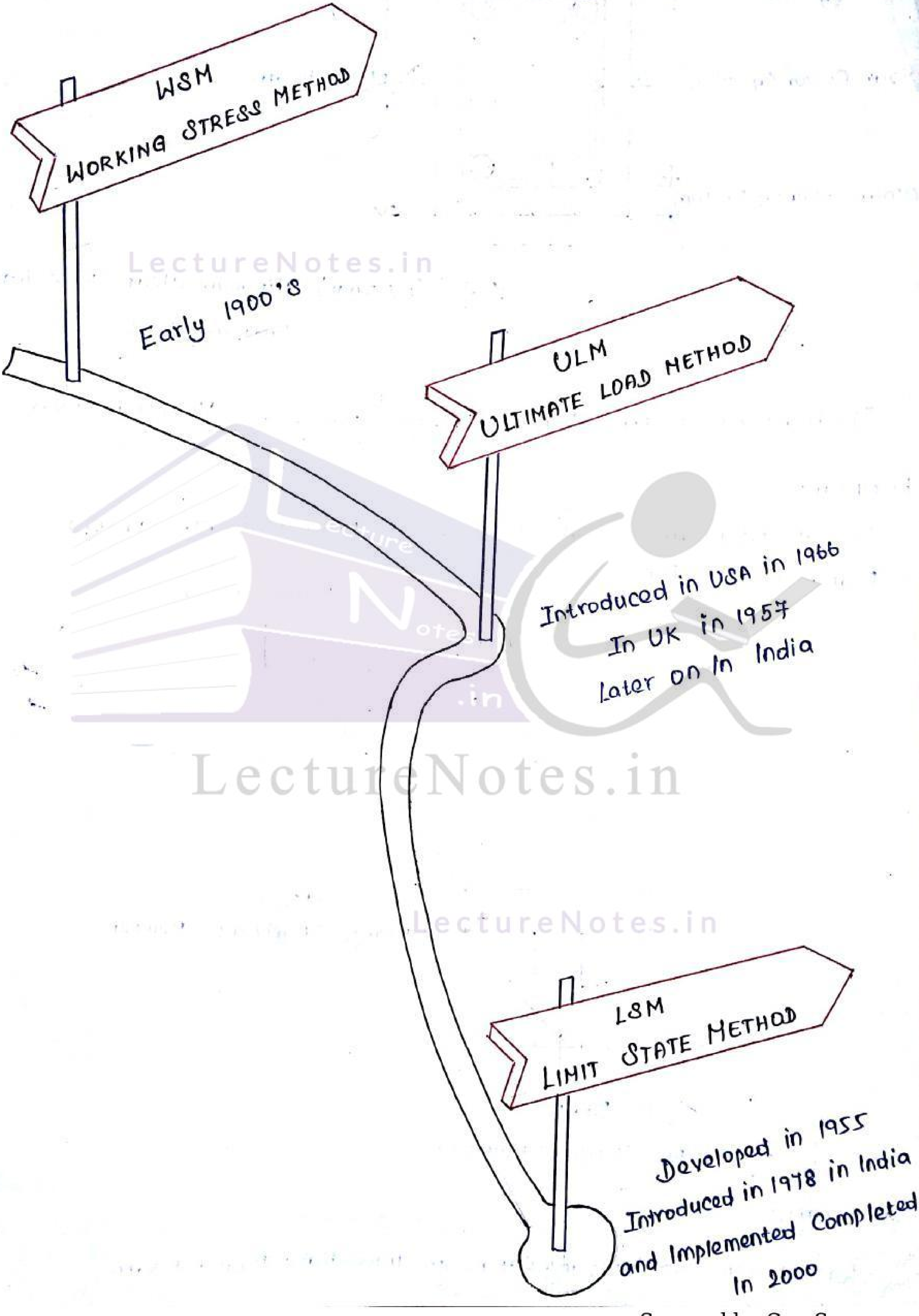
Design of beam, slab system as Monolithically Casted Flanged section



<u>Aesthetics</u>

Conversion of Rectangular beam Into Half Round beam

Methods of Rc Design



Methods of RC Design

- * Working Stress Method
  * Ultimate Load Method
  * Limit State Method

Working Stress Method

* This method of design was the oldest one
* Consider, strength point of view
* It is based on Elastic design
* WSM also assume that both the concrete and steel act together and perfectly Elastic at all stages
* Hence the modular ratio can be used to determine the stresses in concrete and steel

$$\left.\begin{array}{c}\text{Design stresses / Allowable stresses/}\\ \text{permissible stresses}\end{array}\right\} = \frac{\text{Characteristic Strength of Material}}{\text{Factor of Safety}}$$

Factor of Safety

$\boxed{\text{Concrete} = 3}$ $\boxed{\text{Steel} = 1.78}$

* In this method Serviceability is not considered.
  (For such a low level loads serviceability is not considered)

* WSM is the Traditional method of design
* Now days, only used in
    * Design of Water Tank
    * chimney

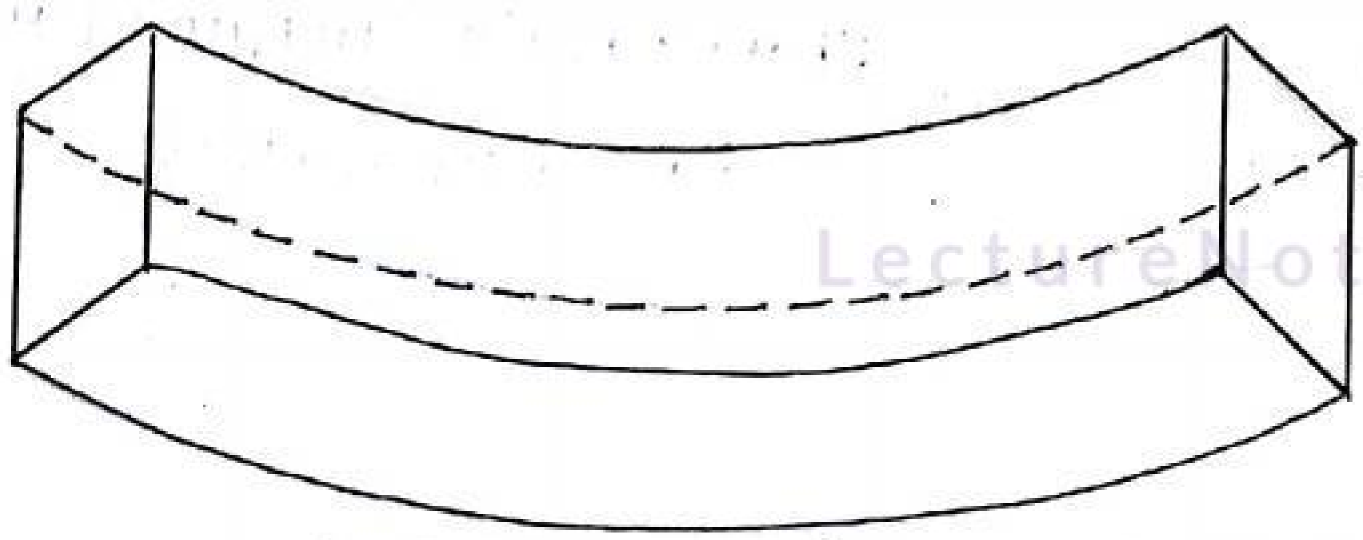
and other leak proof structures

Assumption /

Refer IS : 456 - 2000, B - 1.3, Pg. No : 80

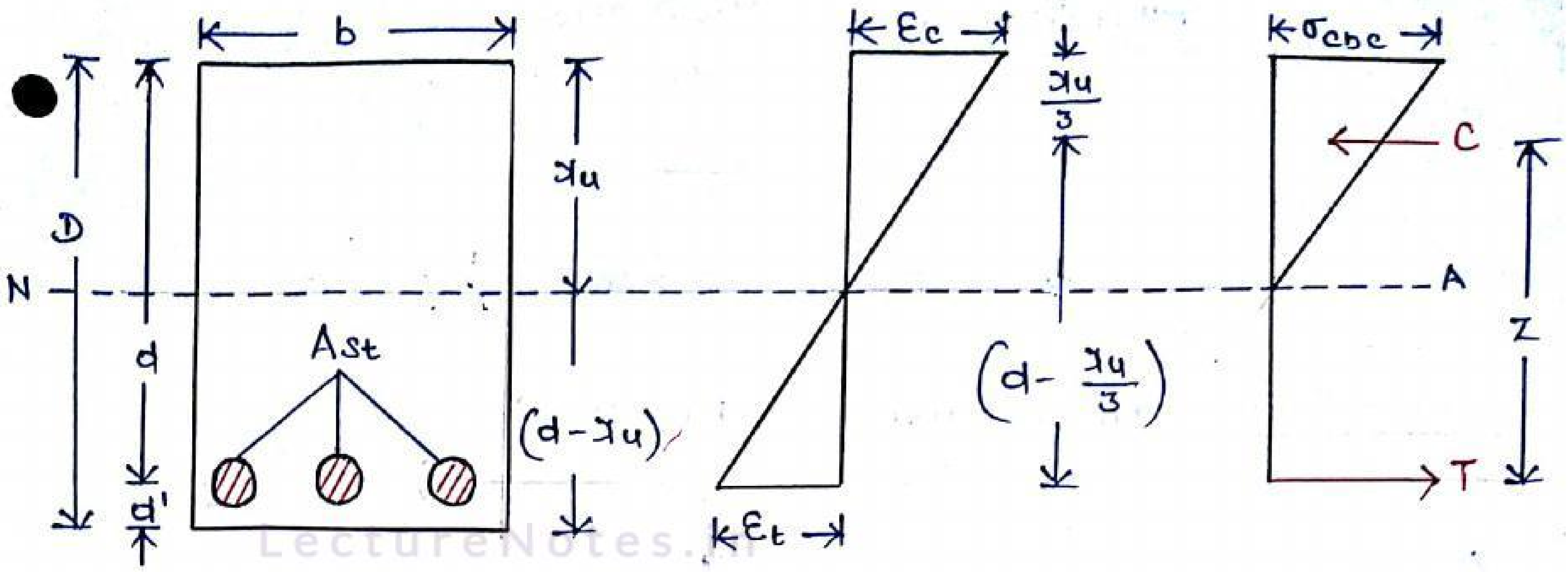
* Plane Section before bending remain plain after bending

BEFORE BENDING



AFTER BENDING

* All Tensile stresses are taken up by reinforcement and none by Concrete. (Area of Concrete in Tension zone is Neglected)



CROSS SECTION

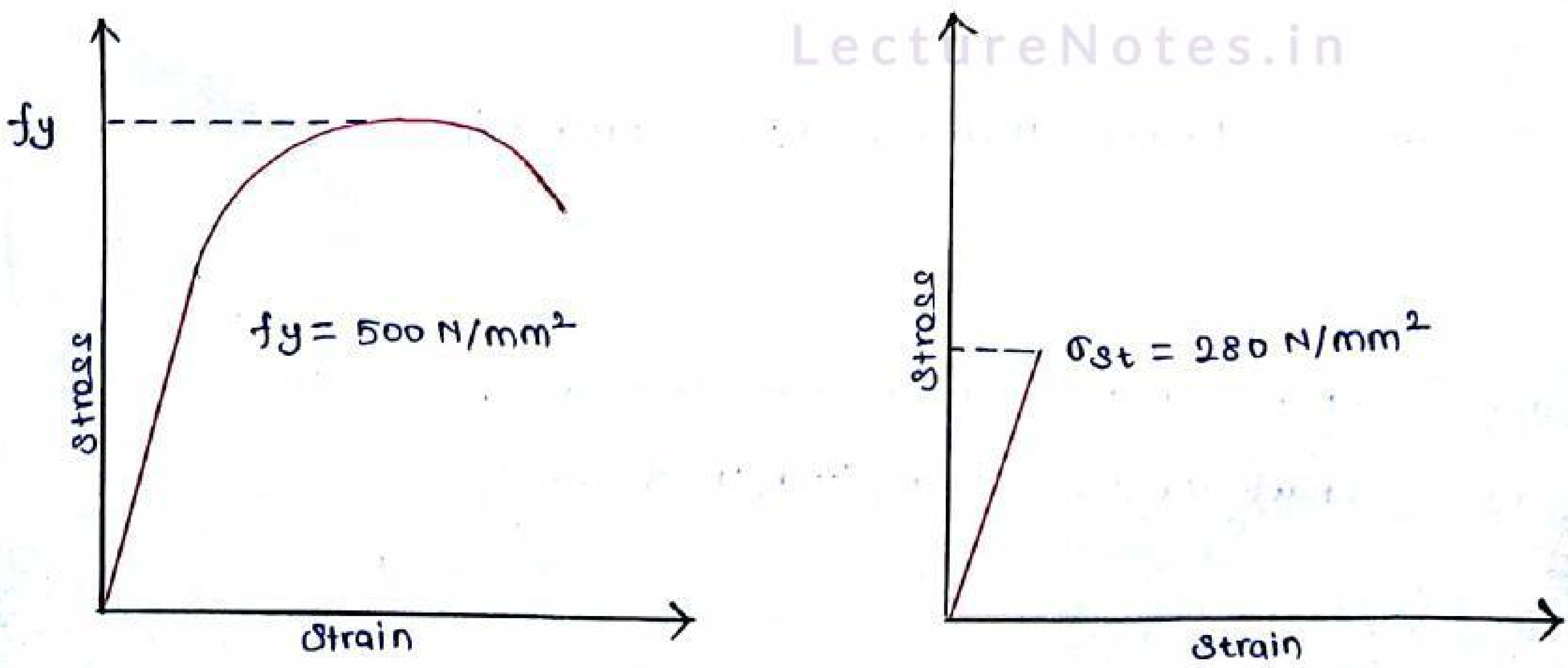
STRAIN DISTRIBUTION

STRESS DISTRIBUTION

Where,

- D – Overall depth
- d – Effective depth
- d' – Effective cover
- b – Breadth
- C – Max. Compression
- T – Max. Tension
- x_u – Actual depth of NA
- Z – Lever Arm
- A_{st} – Area of Tensile Reinforcement
- σ_{cbc} – Permissible compressive bending stress in concrete in N/mm^2
- σ_{st} – Permissible tensile stress in steel in N/mm^2

* The stress - strain relationship of steel and concrete, under working load is a straight line



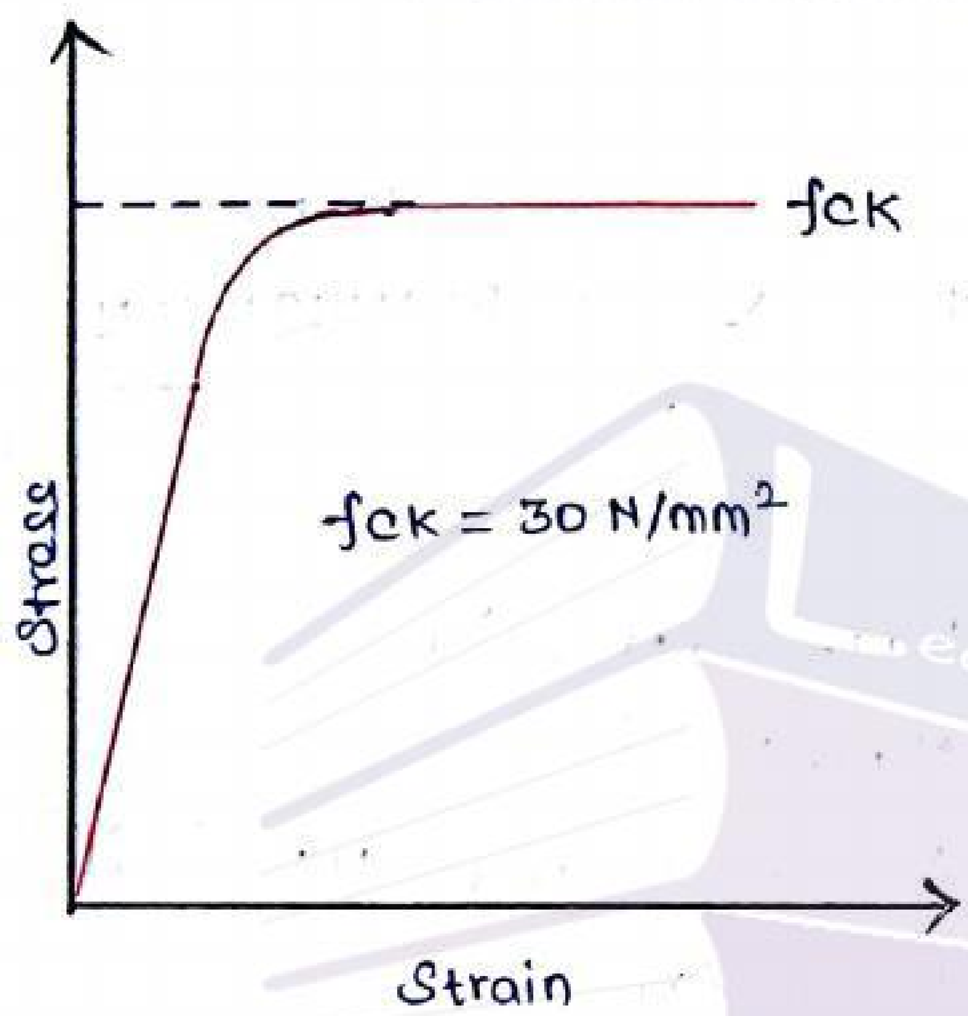
STRESS STRAIN CURVE FOR STEEL

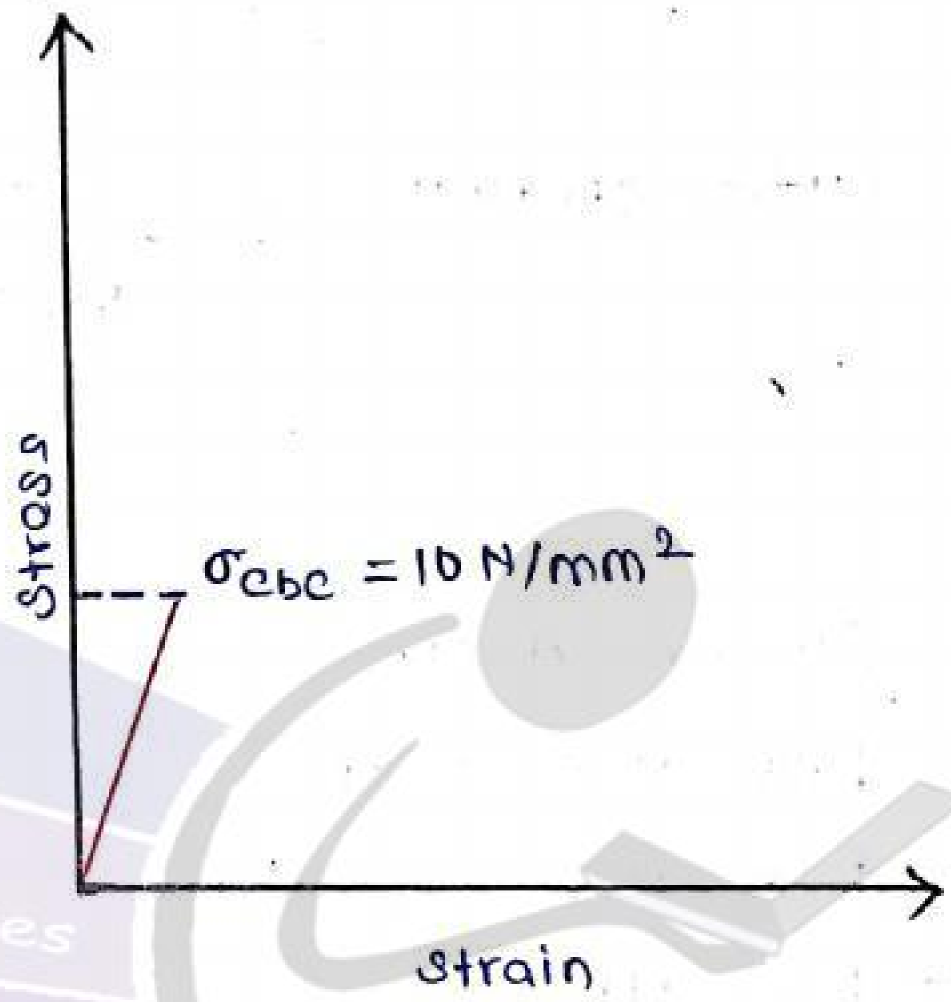
$$\text{Allowable Stress (or) permissible} = \frac{\text{Yield Stress}}{\text{FOS}}$$

$$= \frac{500}{1.78}$$

$$\boxed{\sigma_{st} = 280\ N/mm^2}$$

Yield Stress - Ductile Materials (i.e Steel)





$$\text{Allowable Stress (or) permissible} = \frac{\text{Ultimate Stress}}{\text{FOS}}$$

$$= \frac{30}{3}$$

$$\boxed{\sigma_{st} = 10\ N/mm^2}$$

Ultimate Stress - Brittle Materials (i.e. Concrete)

Where,

f_{ck} = characteristic compressive strength of Concrete

f_y = characteristic yield strength of steel

* The Modular Ratio m, has the value $\frac{280}{3\sigma_{cbc}}$

Drawback

* Concrete is not Elastic
* The Inelastic behaviour of Concrete start right from very low stresses
* The Actual stress distribution in a Concrete section cannot be described by a Triangular stress diagram
* Same factor of safety is used for different types of Loads.
* The Failure mode cannot be observed
* Warning before failure cannot be studied precisely
* This method give uneconomical section
* This method serviceability not consider

Modular Ratio of RCC

* Modular ratio is defined as the ratio between modulus of Elasticity of steel and modulus of Elasticity of concrete
* Reinforced section is made up of both steel and concrete while steel is made to resist Tension load and Compression load take by the concrete.

$\boxed{m = \frac{E_s}{E_c}}$ (If Temperature and shrinkage effect are not taken into consider)

Where,

Es - Young's modulus of steel

$\boxed{E_s = 2 \times 10^5\ N/mm^2}$

Ec - Young's modulus of concrete

$\boxed{E_c = 5000\sqrt{f_{ck}}}$

<u>According to IS:456-2000</u>

$\boxed{m = \frac{280}{3\sigma_{cbc}}}$ (If Temperature and shrinkage Effect are taken into consider)

Where,

σ_{cbc} - permissible compressive Bending stress in concrete in N/mm^2

When load is applied on RCC member to carry same load but strain produced in its different

* The Modular Ratio is not constant for all Grade of concrete
* It varies with the Grade of concrete
* $\frac{Es}{Ec}$ is generally not used to calculate Modular Ratio for reinforced concrete design

Calculation of Modular Ratio Values for different Grade of Concrete

Grade of Concrete	Modular Ratio
M 10	$m = \frac{280}{3 \times 3} \Rightarrow 31.44$
M 15	$m = \frac{280}{3 \times 5} \Rightarrow 18.66$
M 20	$m = \frac{280}{3 \times 7} \Rightarrow 13.33$
M 25	$m = \frac{280}{3 \times 8.5} \Rightarrow 10.98$
M 30	$m = \frac{280}{3 \times 10} \Rightarrow 9.33$

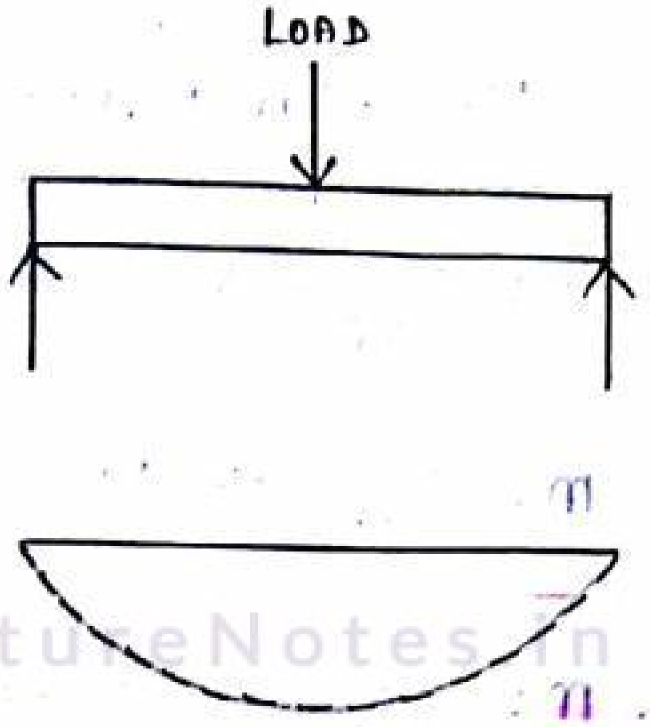
Permissible Stresses in Concrete

The stress in steel and concrete are not allowed to exceed some specified values of stresses are known as permissible stresses (or) Allowable stresses

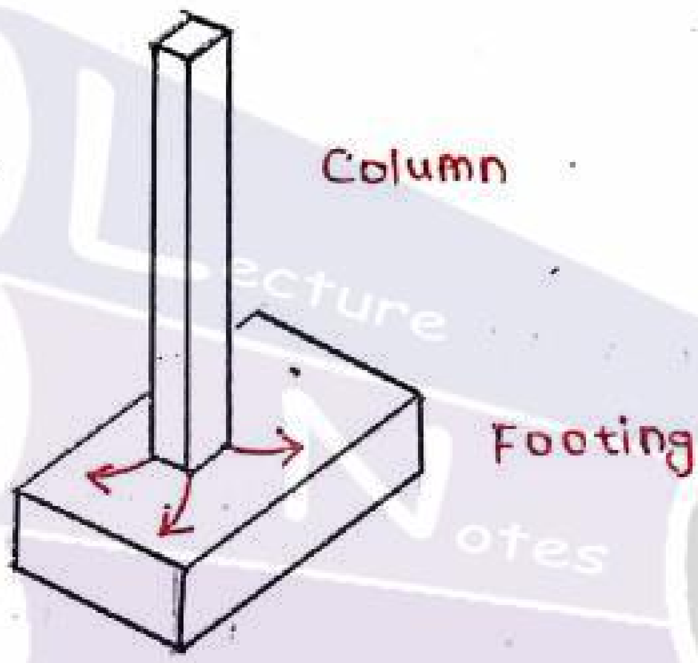
$$\text{Permissible Stresses} = \frac{\text{Characteristic strength of concrete}}{\text{Factor of safety}}$$

Nature of Work	FOS	Permissible Stresses
Bending Compression	3	σ_{cbc}
Direct Compression	4	σ_{cc}
Average Bond Shear	25 - 35	τ_{bd}

- Bending Compression → (Beam is bending)



- Direct Compression → Column is placed at a particular point



Type of Steel	Grade of Steel	Permissible stresses in Bond
MILD STEEL (Plain bars)	Fe 250	100%
HYSD (Deformed bars)	Fe 415	Bonding strength 60% higher than Mild steel (100% + 60%) 1+0.6 ⇒ 1.6
	Fe 500	

Refer IS : 456-2000
Table No: 21
Pg. No : 81

TABLE : 21
PERMISSIBLE STRESSES IN CONCRETE

GRADE OF CONCRETE	PERMISSIBLE STRESS IN COMPRESSION		PERMISSIBLE STRESS IN BOND (AVERAGE)	
	BENDING	DIRECT	MS (PLAIN BARS)	HYSD (DEFORMED BARS)
	σ_{cbc}	σ_{cc}	τ_{bd}	τ_{bd}
M 10	$\left(\frac{10}{3}\right) \Rightarrow 3 \cdot 0$	$\left(\frac{10}{4}\right) \Rightarrow 2 \cdot 5$	$\left(\frac{10}{25}\right) \Rightarrow 0 \cdot 4$	$(1+0 \cdot 6) \times 0 \cdot 4$ 0·64
M 15	5·0	4·0	0·6	0·96
M 20	7·0	5·0	0·8	1·28
M 25	8·5	6·0	0·9	1·44
M 30	10·0	8·0	1·0	1·6
M 35	11·5	9·0	1·1	1·76
M 40	13·0	10·0	1·2	1·92
M 45	14·5	11·0	1·3	2·08
M 50	16·0	12·0	1·4	2·24

NOTE:

* The bond stress given in column 4 shall be increased by 25 percent for bars in compression
* In the case of Deformed bars conforming to IS 1786 the bond stress given in Table 21 may be increased 60%.

Refer IS:456-2000
Pg.No: 82
Table No: 22

TABLE-22

PERMISSIBLE STRESSES IN STEEL REINFORCEMENT

S.NO	TYPE OF STRESS	Permissible Stress in N/mm^2			
		MS bar Grade I and Deformed MS Bars	Medium Tensile Steel & Deformed medium Tensile Steel	High yield Strength Deformed bars	
				Fe 415	Fe 500
1	Tension σ_{st} & σ_{sv} a) Upto 20 mm b) Above 20 mm	140 130	$0.5 f_y$ (or) 190 (Maximum)	230	275
2	Column in — Compression σ_{sc}	130	130	190	190
3	Compression in bars in beam & slab	The Calculated Compressive stress in the Surrounding Concrete multiplied by 1.5 times the Modular Ratio or σ_{sc} whichever is slower			

Ultimate Load Method

- This is also known as Load Factor method or Ultimate Strength method
- In this method Ultimate Load is used as design Load
- Ultimate Load is obtained by multiplying Working Load with Load Factor
- Consider Non-Linear behaviour of the material
- The Load Factor gives exact margin of safety in Terms of Loads
- This method gives very Thin section
- Load Factor depends on Load Combination

$$\boxed{\text{Load factor} = \frac{\text{Ultimate Load}}{\text{Working Load}}}$$

Factor of Safety

$\boxed{\text{Concrete} = 1.5}$ $\boxed{\text{Steel} = 1.15}$

Drawback

- This method is not all used by designer
- In this method Serviceability is not consider
- It has not become popular

Limit State Method

* In this method of design , structure shall be design to withstand safely all Load Liable to act on it throughout it Life

* It shall also satisfied all serviceability requirement such as Limitation of deflection and cracking etc...

* This method is advanced over Traditional design method

* It is a plastic design method

* The acceptance limit for safely and serviceability requirement before failure occurs is called Limit State

Consider

Safety – Ultimate Load

Serviceability – Working Load

TYPE OF LIMIT STATE

- LIMIT STATE OF COLLAPSE
- LIMIT STATE OF SERVICEABILITY

Limit State of Collapse

* Flexure
* Shear
* Compression
* Torsion

Partial Safety Factor γ_m

- $\boxed{\gamma_c = 1.5}$ $\boxed{\gamma_s = 1.15}$

Where,

γ_c - PSF for Concrete

γ_s - PSF for Steel

Limit State of Serviceability

* Deflection
* Cracking
* Vibration
* Durability

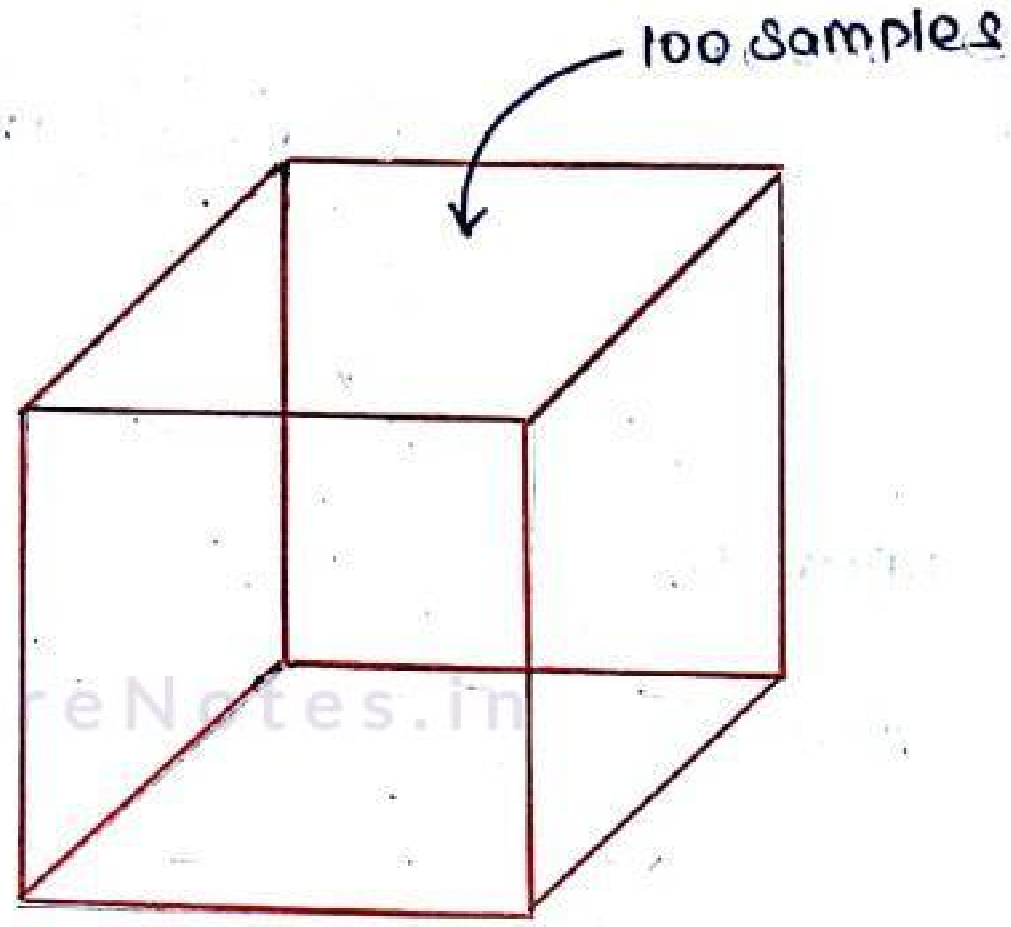
Partial Safety Factor

$\boxed{\gamma_c = 1}$ $\boxed{\gamma_s = 1}$

CHARACTERISTIC STRENGTH

'Characteristic strength' means that value of strength of the material below which not more than 5% result are expected to fall.

Concrete Cube



Size of the Cube = 150mm x 150mm

No. of Specimen = 100 No's

During Compression Test on cube in Laboratory

Let assume,

Grade of concrete = M 25

f_{ck} = 25 N/mm²

Test Results

$\boxed{23.8}$ (Below 25 N/mm²)

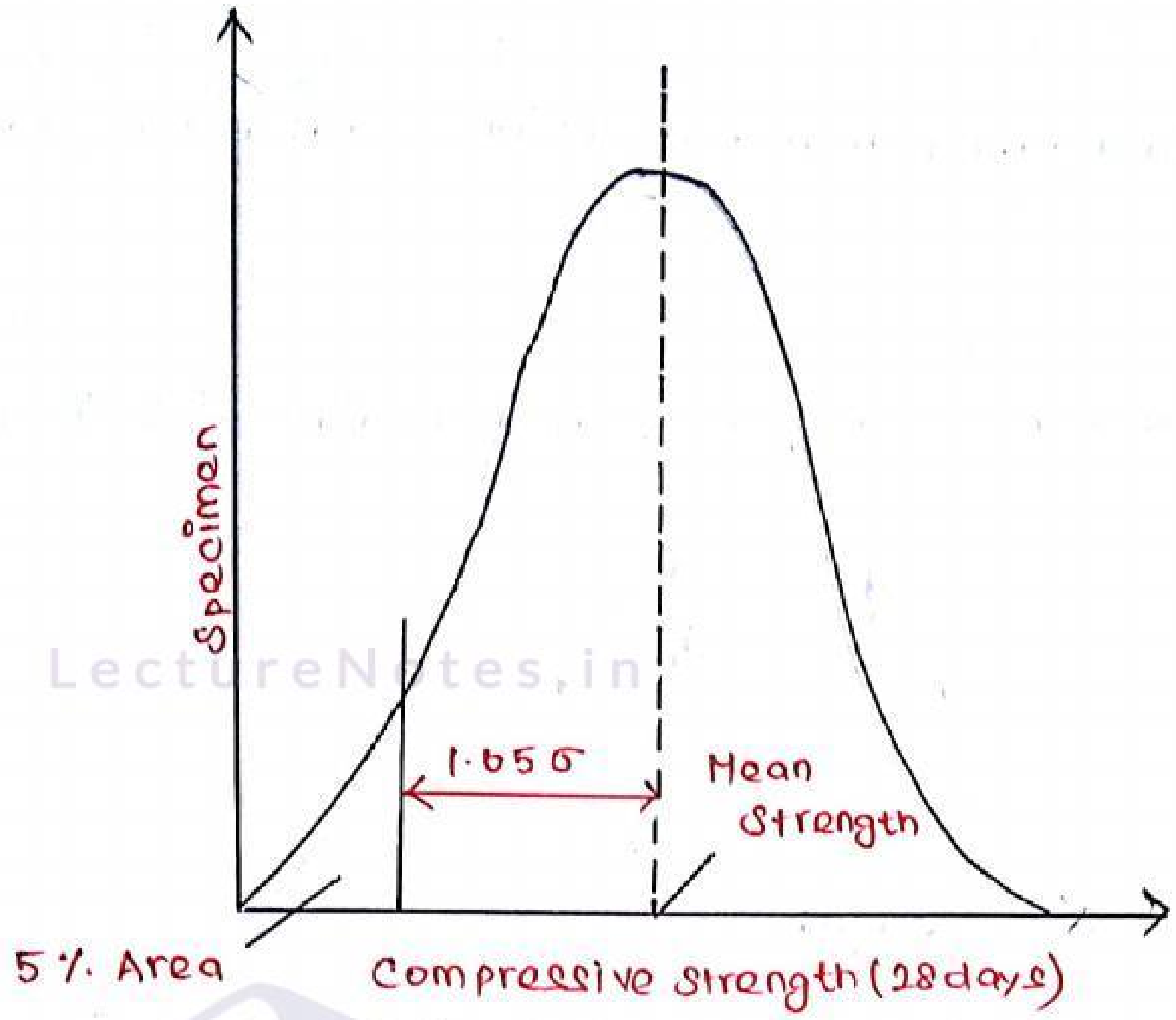
25.1

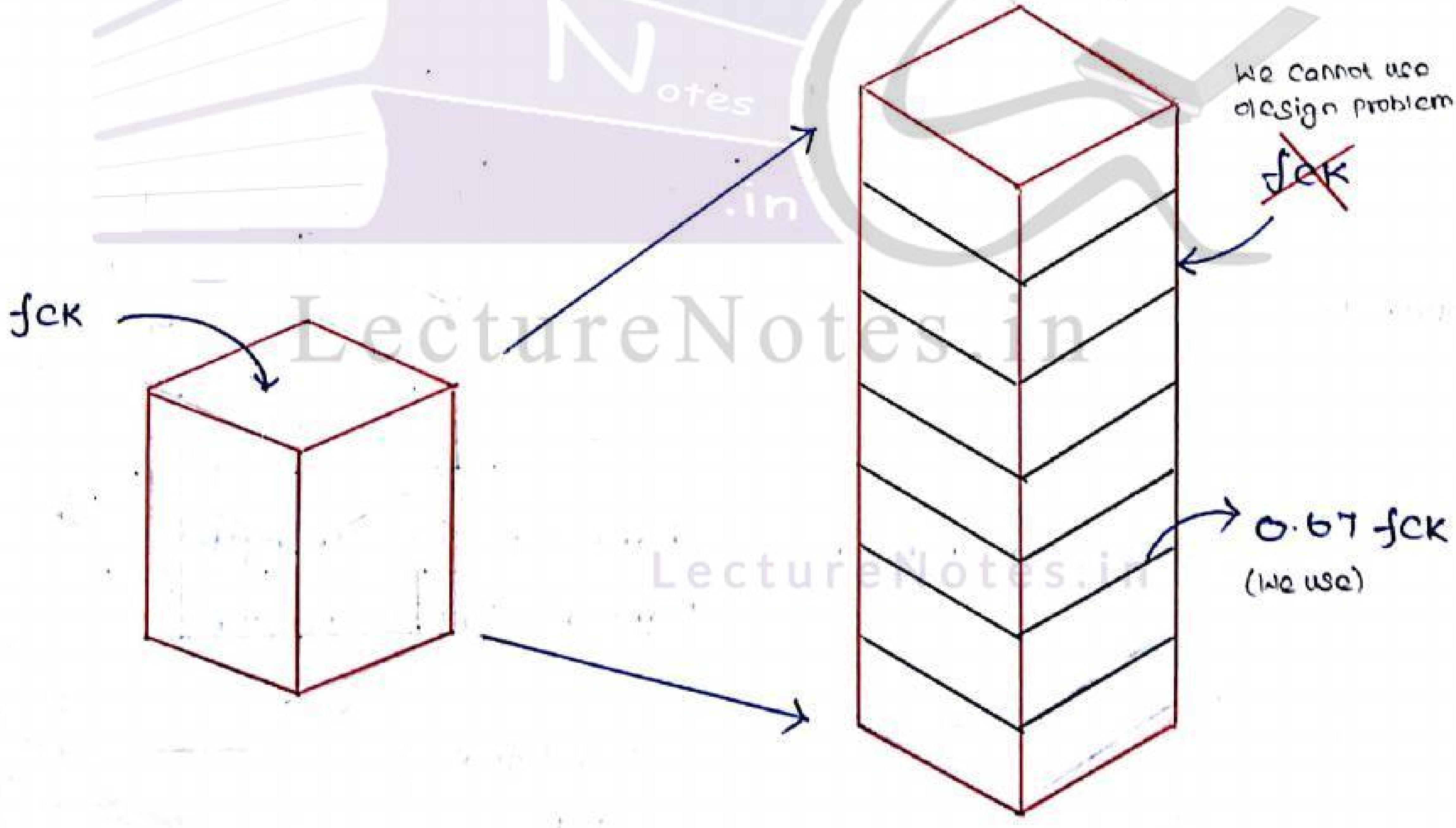
25.4

26.9

27.3

27.6 ....,





0.67 fck – Characteristic strength for a structure
(Actual value)

- Design Strength

It is the strength of material which is taken for design

- Materials

The design strength of the materials f_d is given by

$$\boxed{f_d = \frac{f}{\gamma_m}}$$

Where,

f_d = Design strength

f = Characteristic strength of Material

γ_m = Partial Safety Factor appropriate to the material and the LS being considered

1.5 (For concrete)

1.15 (For Steel)

Concrete

$$\text{Design Strength} = \frac{\text{Characteristic Strength}}{\text{Partial Safety Factor}}$$

$$= \frac{0.67 f_{ck}}{1.5}$$

$$\boxed{f_d = 0.45 f_{ck}}$$

Steel

-

$$\text{Design Strength} = \frac{\text{Characteristic Strength}}{\text{Partial Safety Factor}}$$

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

$$\boxed{f_d = 0.87 f_y}$$

CHARACTERISTIC LOAD

'Characteristic Load' means that value of Load which has a 95% probability of not being exceeded during the Life of the Structure.

Partial Safety Factor

S.NO	LOAD COMBINATION	LIMIT STATE OF COLLAPSE			LIMIT STATE OF SERVICEABILITY		
		DL	LL	WL/EL	DL	LL	WL/EL
1	DL + LL	1.5	1.5	–	1	1	–
2	DL + WL/EL	1.5 OR 0.9	–	1.5	1	–	1
3	DL + LL + WL/EL	1.2	1.2	1.2	1	0.8	0.8

NOTE

* If we consider design WL, then there is no need to consider WL
* 0.9 is taken when stability against overturning / sliding

* Structure is made safe for Ultimate Load / Factor Load and it is made serviceable for characteristic Load / Working Load / Service Load

Where

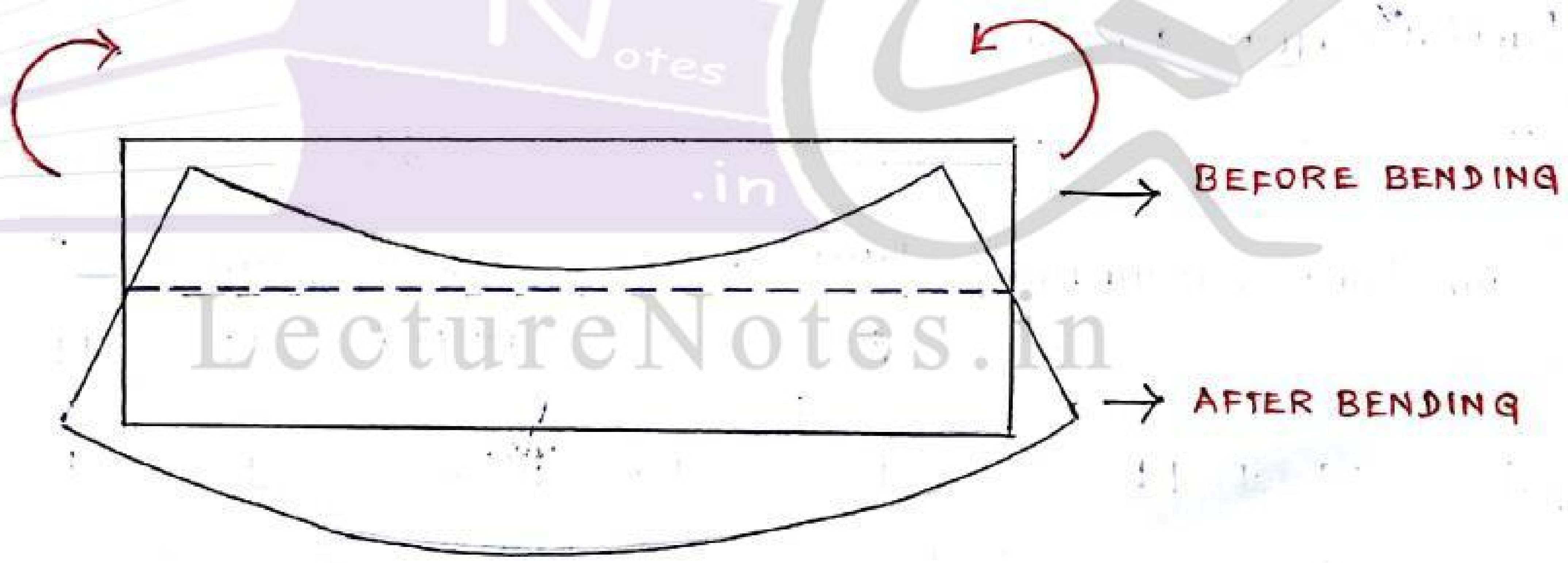
DL = Dead Load   WL = Wind Load

LL = Live Load   EL = Earthquake Load

LIMIT STATE OF COLLAPSE: FLEXURE

Assumptions:

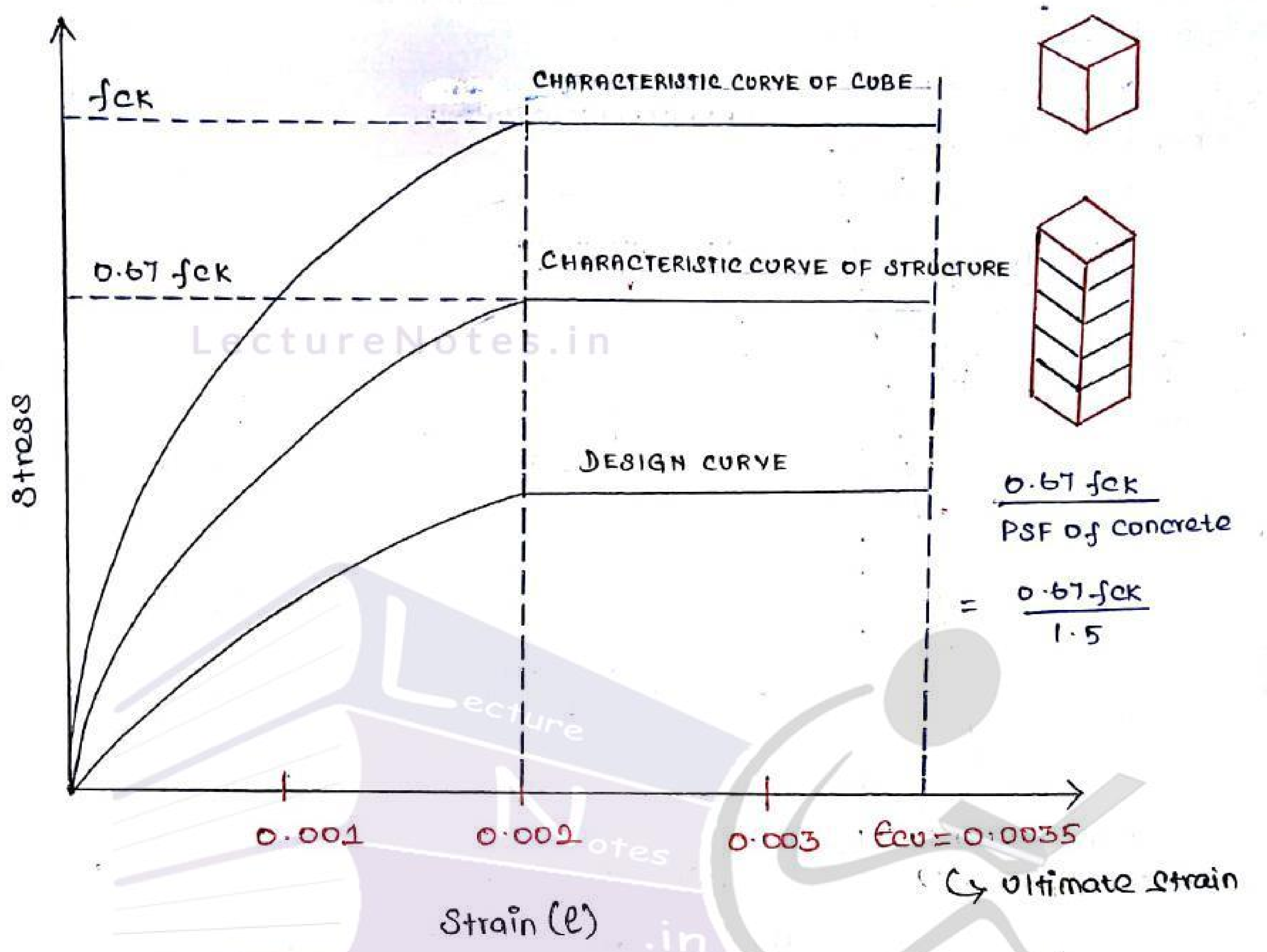
* Plane sections normal to the axis remain plan after bending



* The maximum strain in concrete at the outermost compression Fibre is taken as 0.0035 in bending

* The relationship between the stress - strain distribution in concrete is assumed to be parabolic

STRESS - STRAIN CURVE FOR CONCRETE



* The Stress - Strain Curve for Concrete is having parabolic shape upto 0.002 strain and the constant upto Limit State of 0.0035

* However IS Code does not prevent using other shapes Like Rectangle and Trapezoidal

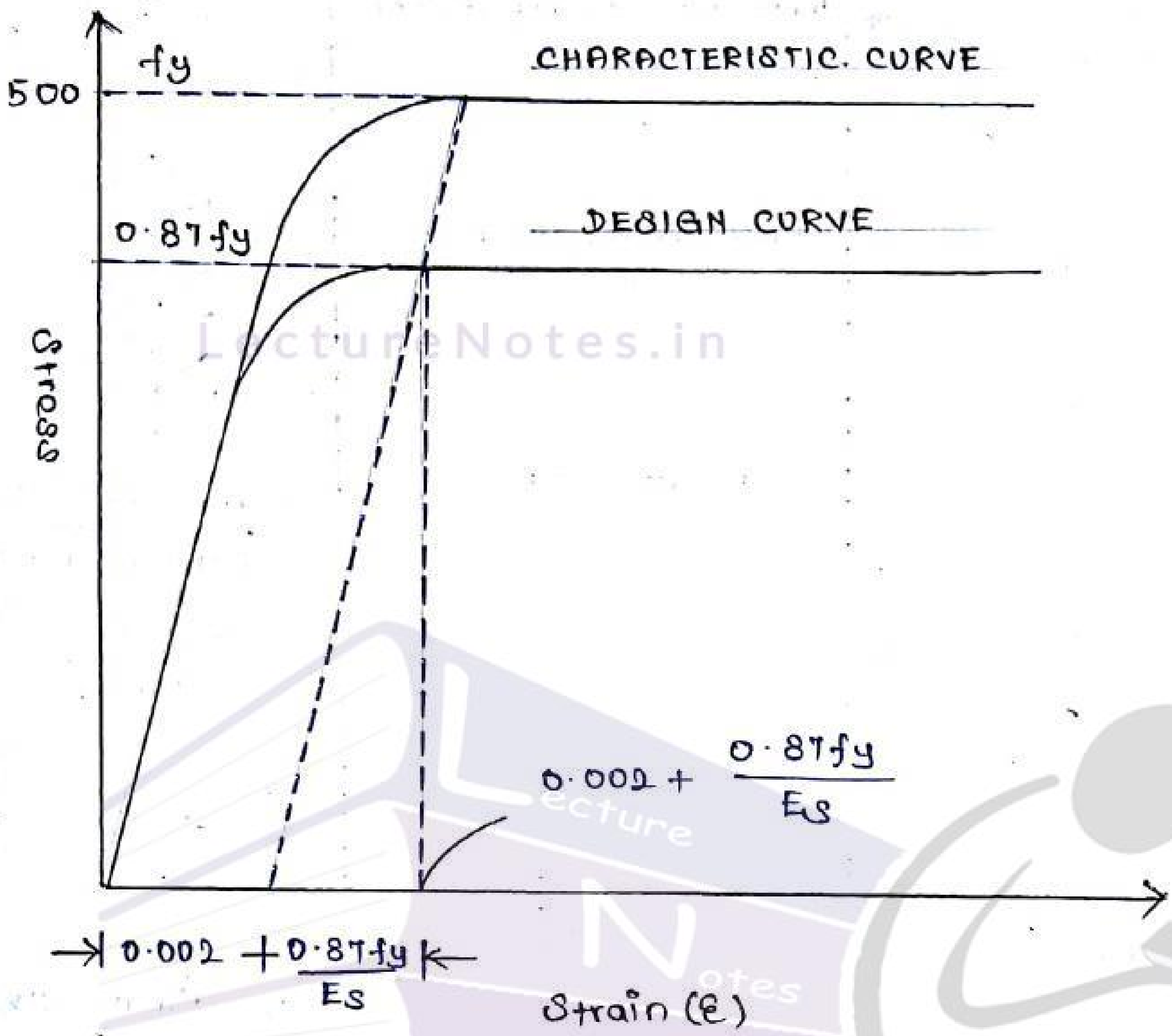
* For Design purpose , the Compressive strength of concrete is assumed to be parabolic

* For design purpose the compressive strength of concrete in the structure is assumed to be 0.67 times the characteristic strength of concrete.

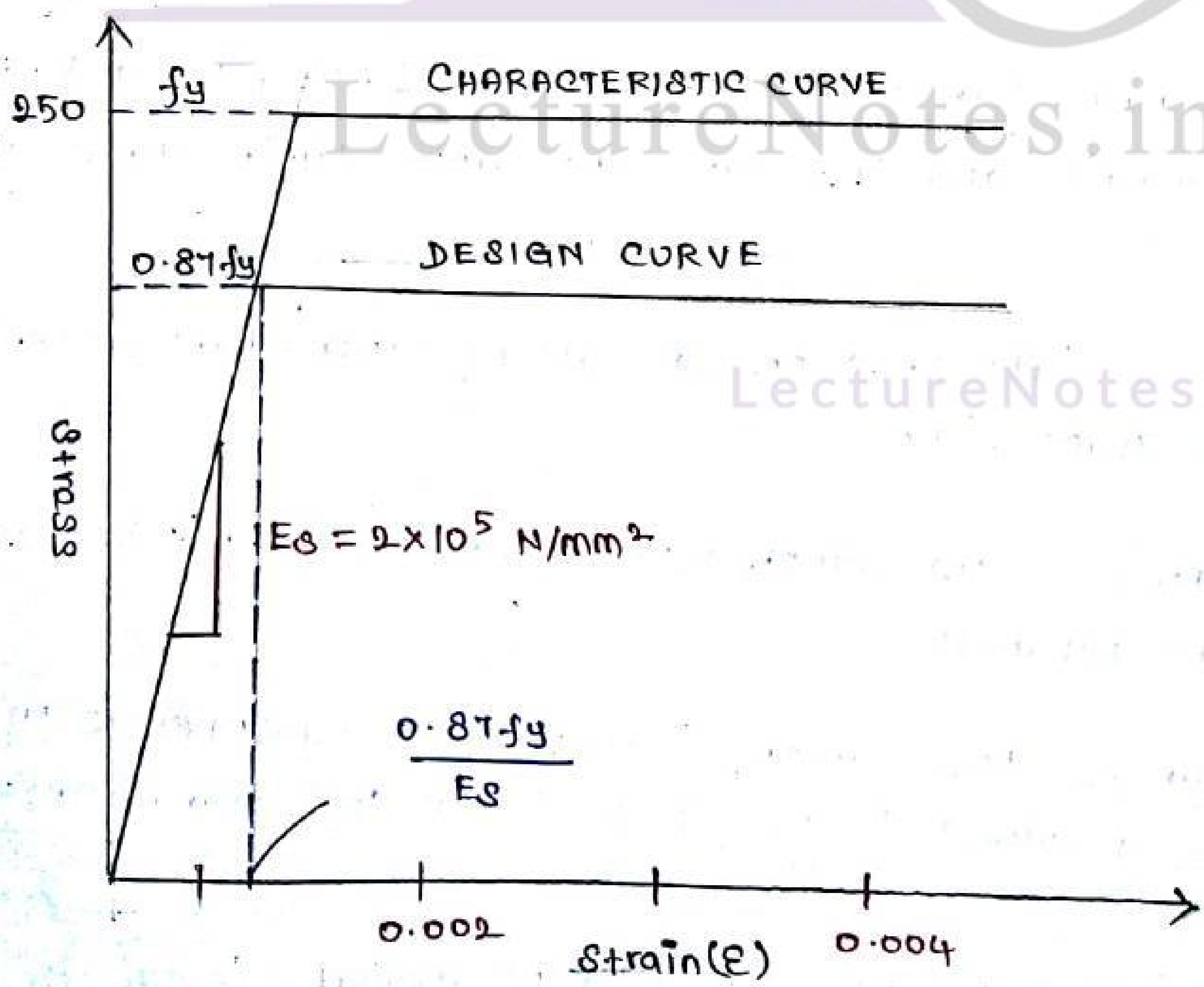
* The partial Safety factor $\gamma_m = 1.5$ Shall be applied

* The tensile strength of the concrete is ignored

STRESS - STRAIN CURVE FOR STEEL



HYSD Bars



Mild Steel Bars

* The stresses in the reinforcement are taken from the stress - strain curve for the type of steel used.

* For design purpose the PSF $\gamma_m = 1.5$ shall be applied

* Maximum strain in the Tension reinforcement should not less than

$$\boxed{\frac{f_y}{1.15 E_s} + 0.002}$$

Where

f_y = characteristic strength of steel

E_s = Modulus of Elasticity of steel

Mild steel

$$Fe250 = \frac{250}{1.15 \times (2 \times 10^5)} + 0.002 = 0.00309$$

HYSD

$$Fe415 = \frac{415}{1.15 \times (2 \times 10^5)} + 0.002 = 0.00380$$

$$Fe500 = \frac{500}{1.15 \times (2 \times 10^5)} + 0.002 = 0.00417$$

BEAM

* Beam is the horizontal member of a structure, carrying Transverse Loads
* Beam is Rectangular in cross section
* Beam carry the Floor slab or the Roof slab.
* Beam transfer all the Loads Including its self weight to the columns or Walls

R.C.C BEAM

* R.C.C Beam is Subjected to Bending moments and Shear Force
* Due to vertical Load, bending Compresses the top Fibres of the beam and Tensile elongates the bottom Fibers.
* The strength of RCC beam depends on the Composite action of Concrete and Steel

There are various types of materials used for beam

* Wood
* steel
* Aluminum etc..,

But the most Common material for beam is Reinforced Cement Concrete (RCC)

Rcc beam can be various types depending upon different criteria

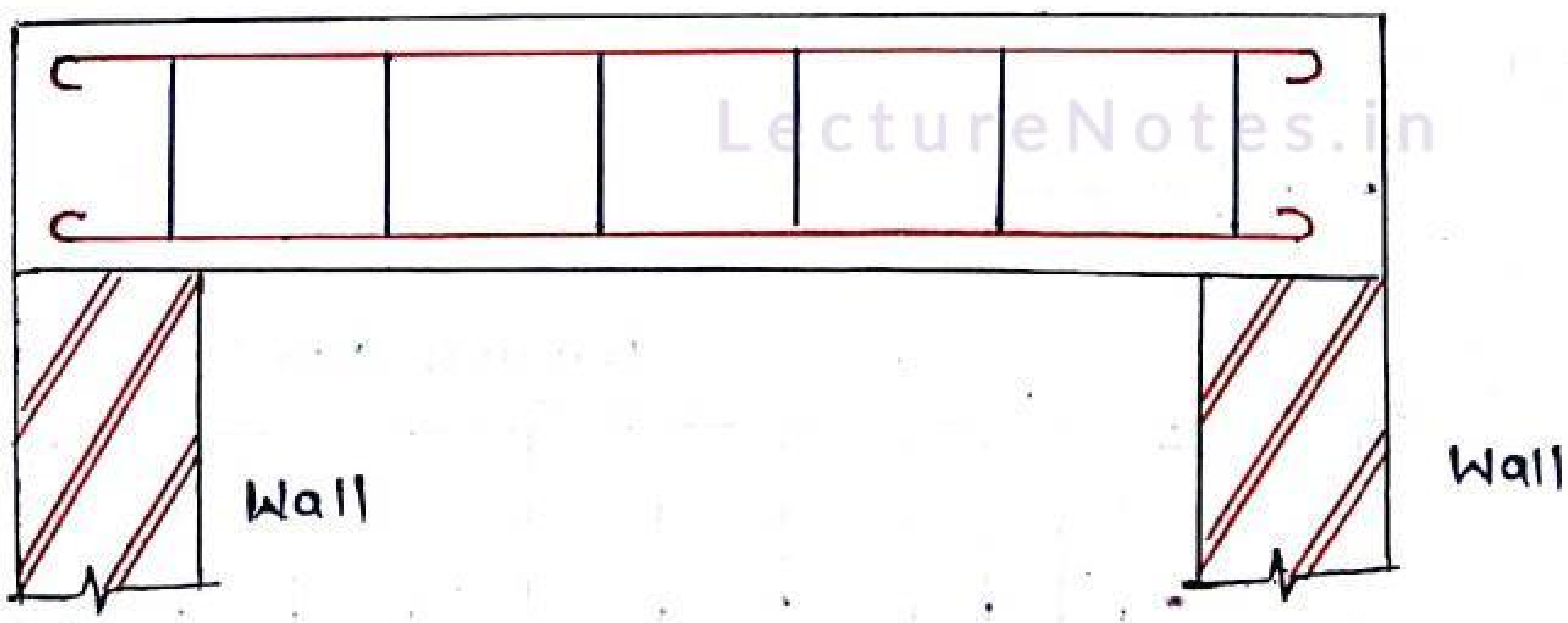
* Depending upon shape beam can be

  * Rectangular
  * T- Beam
  * L - Beam

* Type of Rcc beam depending upon their supporting systems

  * Simply Supported beam
  * Fixed beam
  * Cantilever beam
  * Continuous beam
  * Overhanging beam

Simply Supported beam

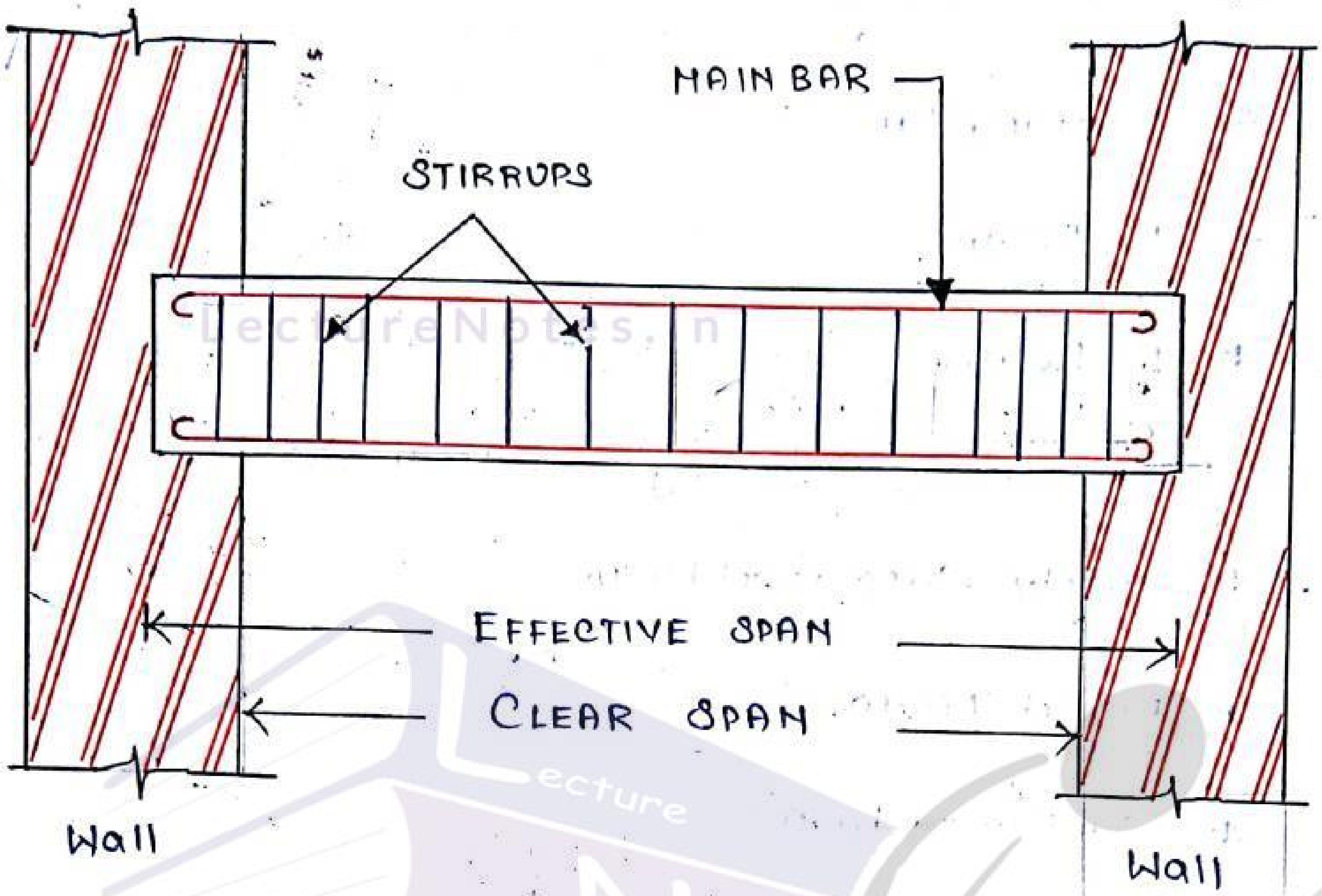


It is a beam supported freely at the Two Ends on Walls or columns.

In actual practise, no beam rest freely on the supports

(walls or columns)

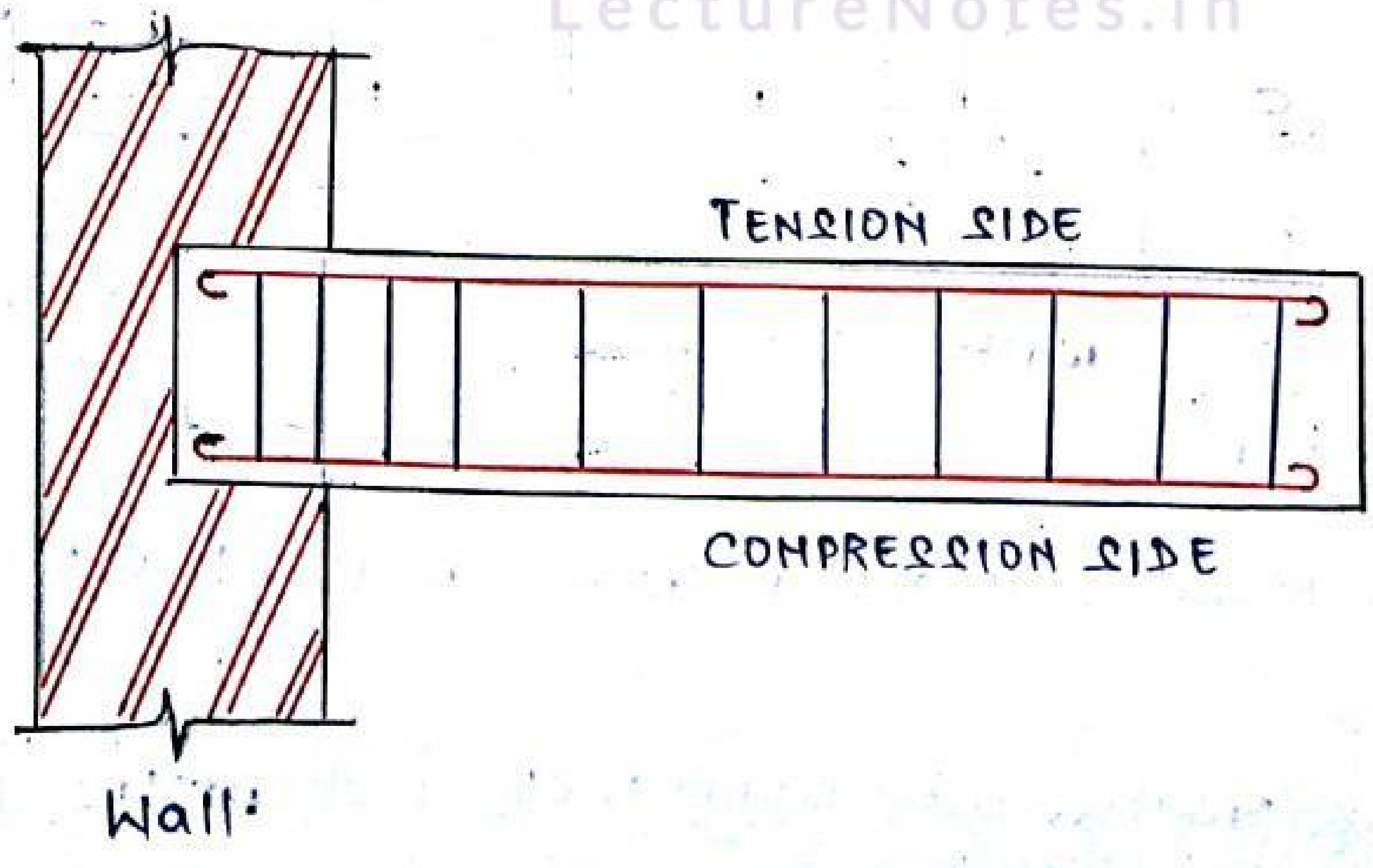
Fixed beam



* In this beam, both ends of the beam are rigidly fixed into the supports.

* Also Main reinforcement bars and stirrups are provided

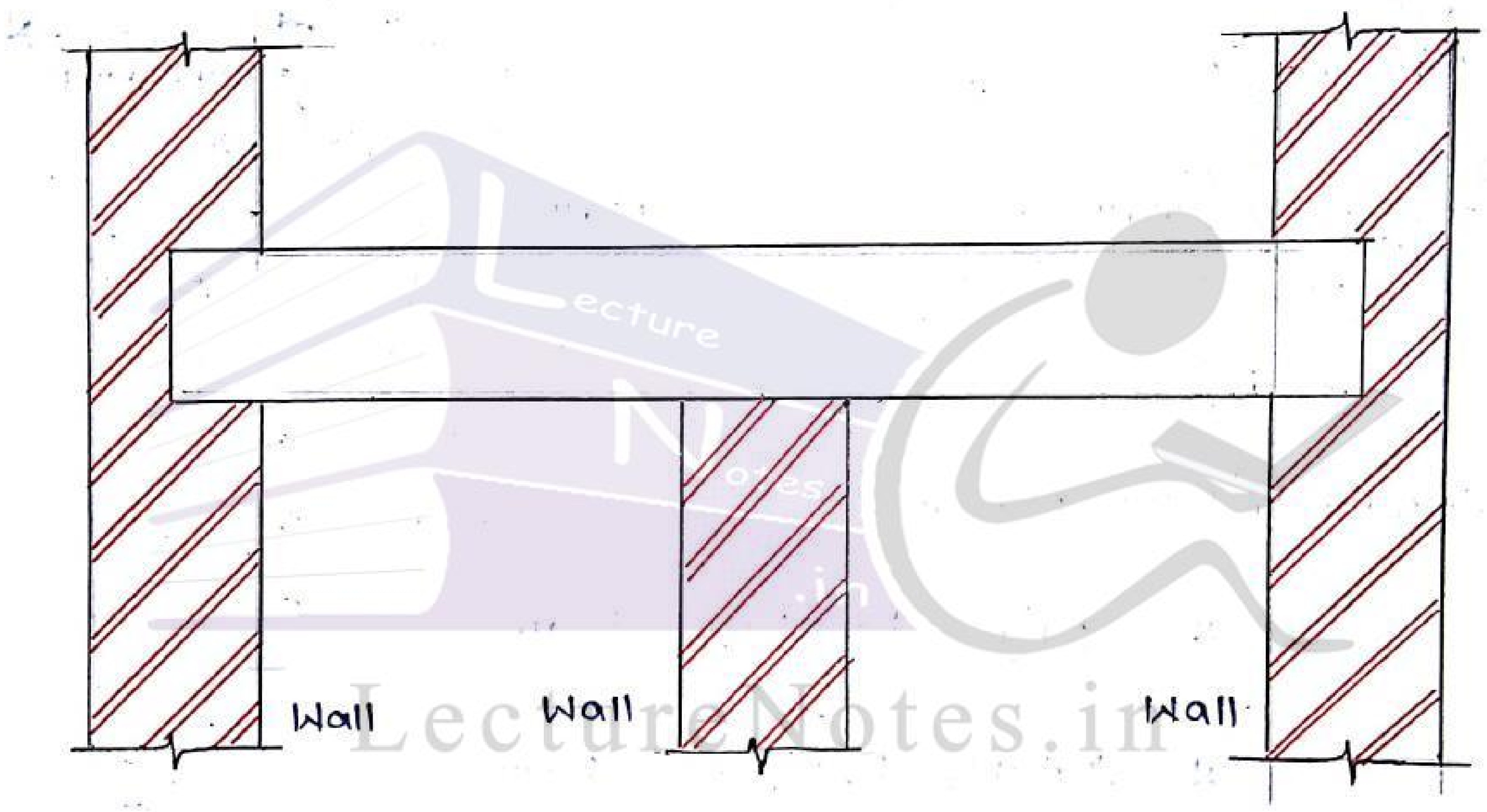
Cantilever beam:



* It is fixed in a wall or column at one end and the
* other end is free it is called cantilever beam.

* It has Tension zone in the Top side and compression in the bottom side

Continuous beam



* It is supported on more than two supports

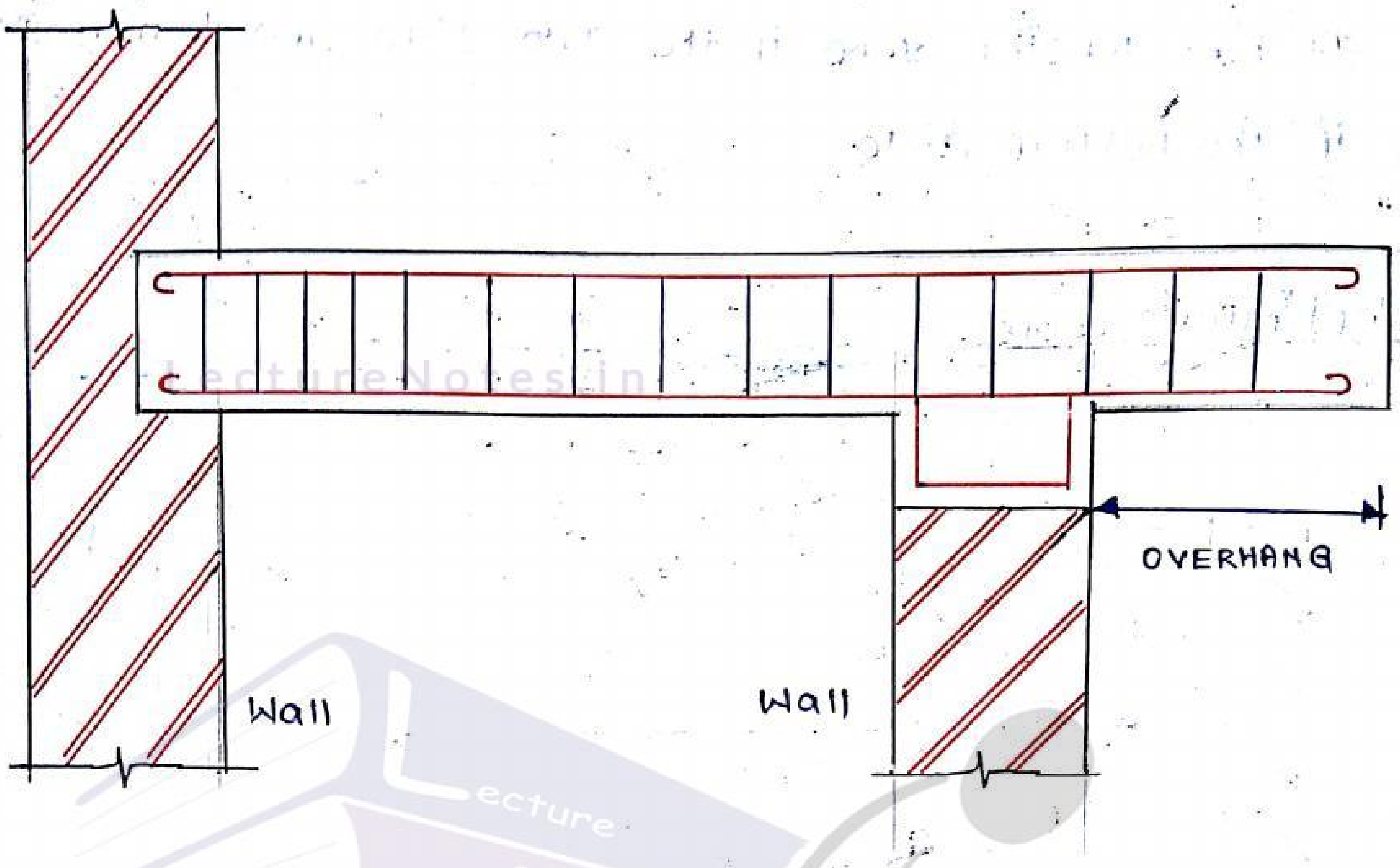
* This beam is more economical for any span length

Overhanging beam

* In Overhanging beam, Its end extends beyond the Wall or Column support

* Overhanging of the beam is the unsupported portion of the beam

* It may be one side or both the sides of the supports.



* Depending upon placement of Reinforcement

    * Singly Reinforced beam

    * Doubly Reinforced beam

Singly Reinforced Beam

•

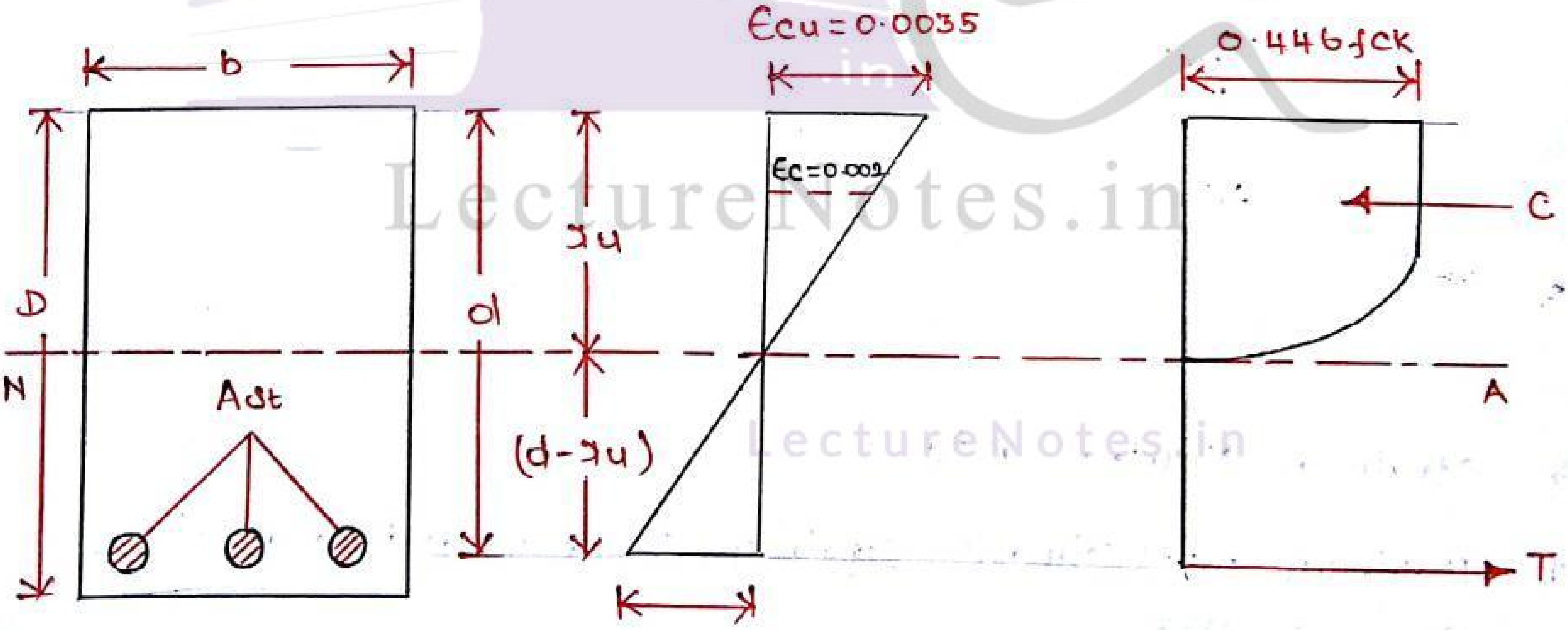
The beam that is Longitudinally rein-forced only in Tension zone it is known as singly reinforced beam

* In such beams, the Ultimate bending moment and the Tension due to bending are carried by the reinforcement while the compression is carried by the concrete

* Practically, it is not possible to provide reinforcement only in the Tension zone, because we need to Tie the Stirrups.

* Therefore two rebars are utilized in the compression zone to tie the stirrups and the rebars acts as False members just for holding the stirrups

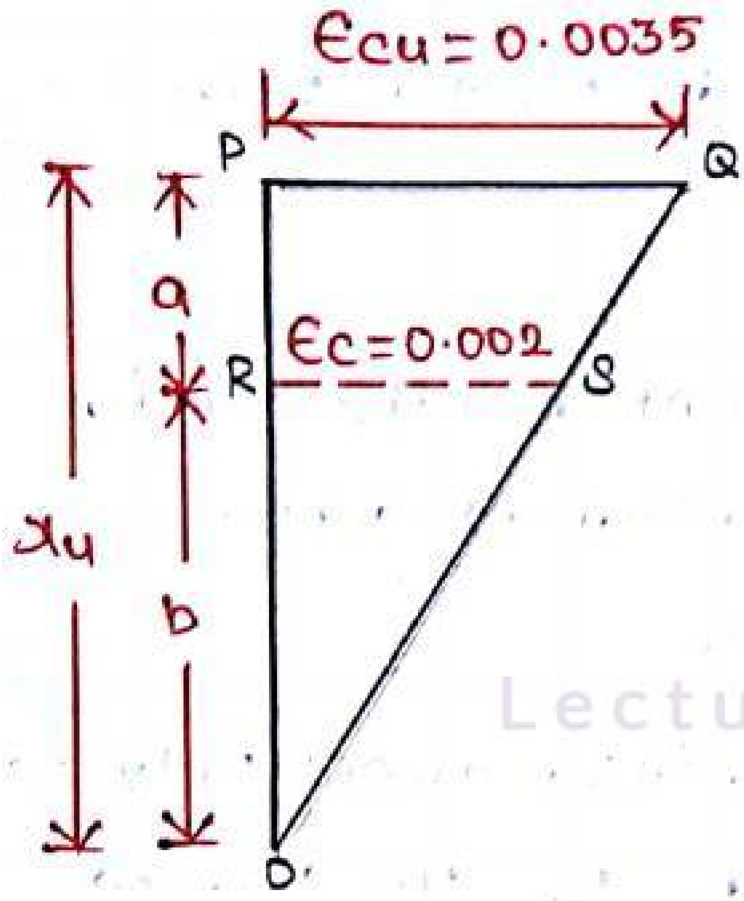
Stress Block Parameters



$$E_{su} \geq \frac{0.87 f_y}{E_s} + 0.002$$

Cross Section — Strain Diagram — Stress Diagram

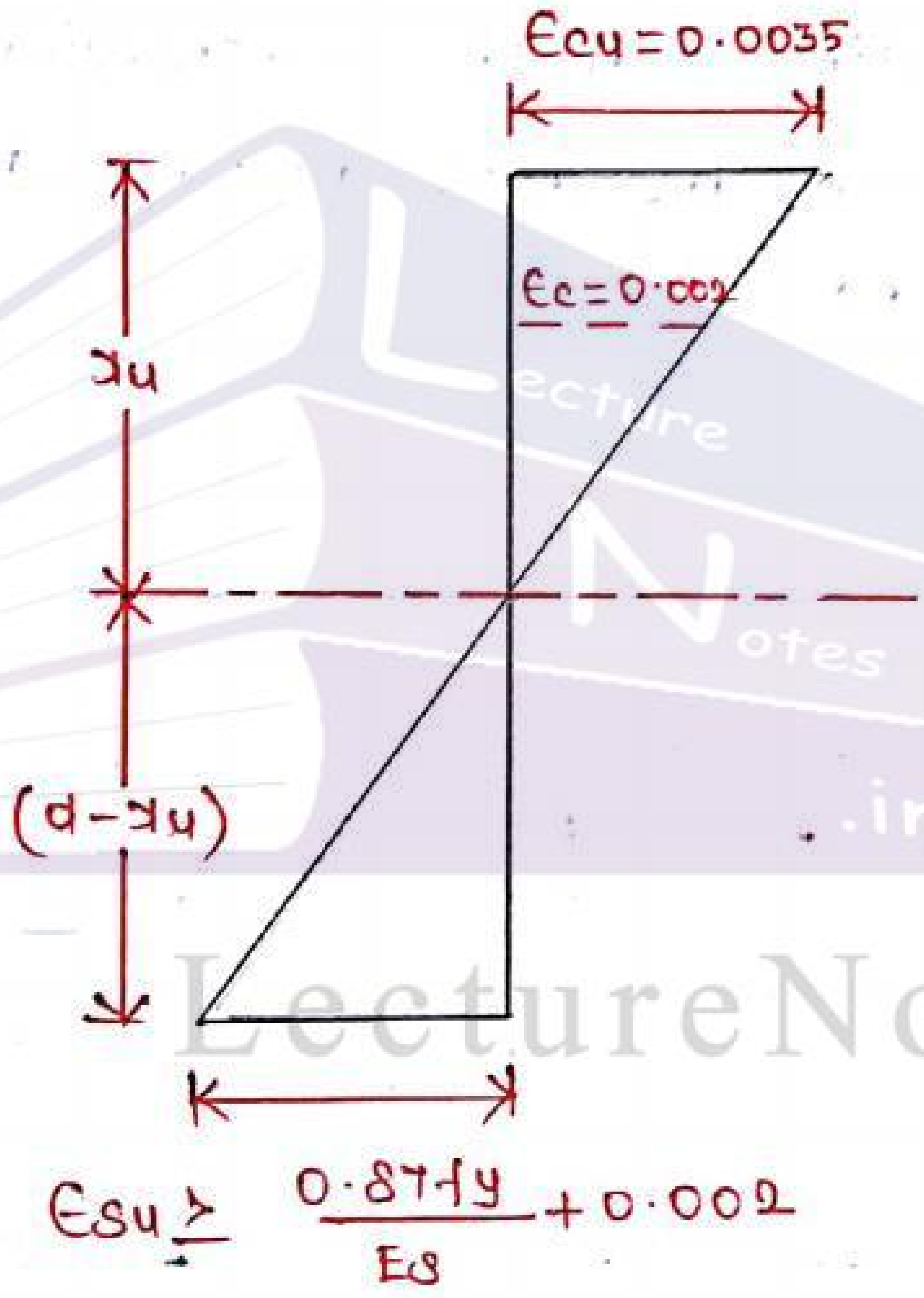
Strain distribution



$\Delta RSO \sim \Delta PQO$

$$\frac{0.002}{b} = \frac{0.0035}{x_u}$$

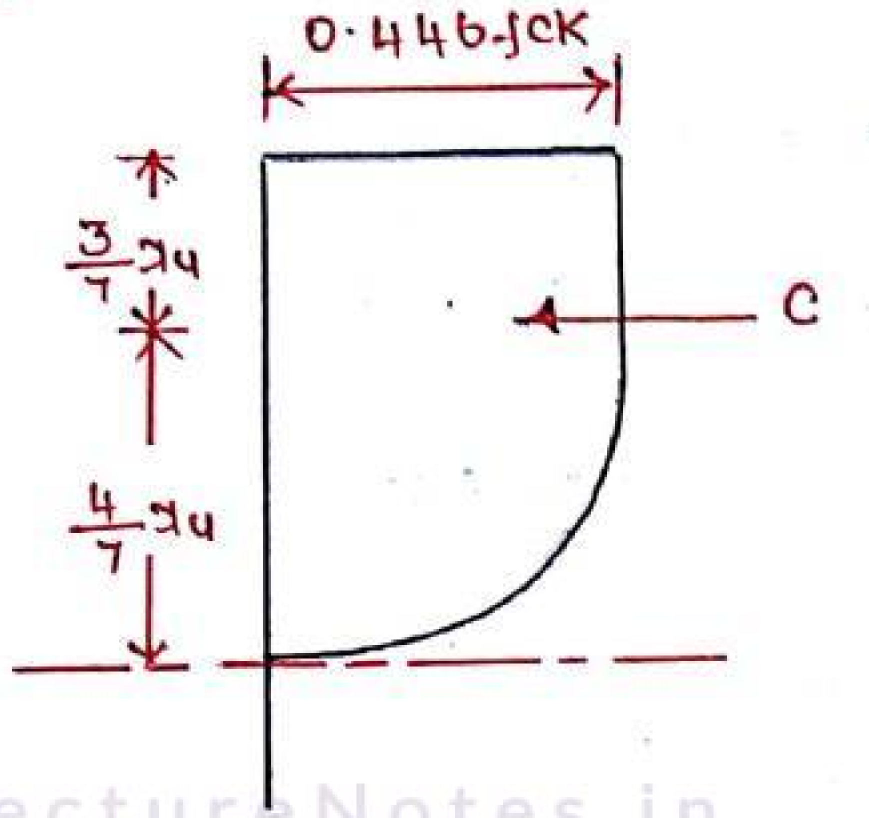
$$\boxed{b = \frac{4}{7} x_u}$$



* Strain at Neutral axis = 0
* Maximum or Ultimate Strain in Concrete at Extreme fibre $\epsilon_{cu} = 0.0035$
* Strain at constant stress of $0.67 f_{ck}$, $\epsilon_c = 0.002$
* Ultimate strain in steel corresponding to maximum stress at failure

$$\epsilon_{su} \geq \frac{0.87 f_y}{E_s} + 0.002$$

Stress Block



In the stress block

Depth of Rectangular portion $= \frac{3}{7}x_u$

Parabolic portion $= \frac{4}{7}x_u$

Area of Stress block

Area of Rectangular portion $\} = b \times \frac{3}{7}x_u \times 0.446 f_{ck}$

$A_1 = 0.191 f_{ck}\, x_u \cdot b$

Area of parabolic portion $\} = b \times \frac{2}{3} \times \frac{4}{7}x_u \times 0.446 f_{ck}$

$A_2 = 0.169 f_{ck}\, x_u \cdot b$

Area of Stress block $= A_1 + A_2$

$= 0.191 f_{ck}\, x_u \cdot b + 0.169 f_{ck}\, x_u \cdot b$

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

Total Compressive Force (concrete)

Total compression $=$ Area of stress block

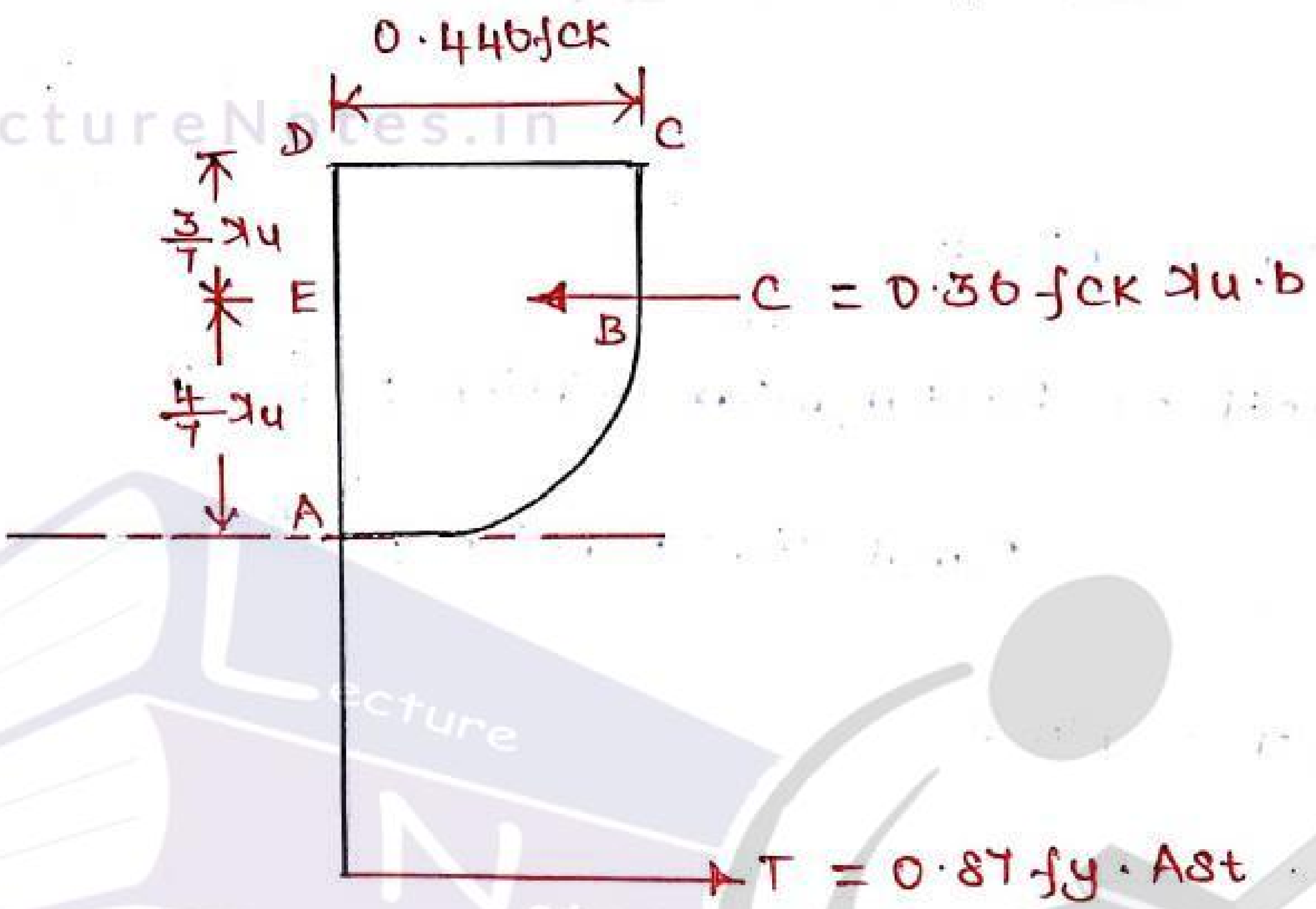
$$\boxed{C = 0.36 f_{ck}\, x_u \cdot b}$$

Total Tensile Force (Steel)

Tensile Force (steel) = $0.87 f_y$

Total Tensile = $0.87 f_y \cdot A_{st}$

$$\boxed{T = 0.87 f_y \cdot A_{st}}$$



* It has a parabolic shape from A to B and the Linear from B to C above the Neutral axis

* Stress at Neutral axis (point-A) = 0

* Stress at 0.002 strain (point-B)

$$\frac{0.67 f_{ck}}{1.5} = 0.446 f_{ck}$$

* Stress at Extreme fibre (point-C) = $0.446 f_{ck}$

* Below the Neutral axis, the Concrete is assumed to be cracked and Maximum stress in steel

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

* Area of stress block = $0.36 f_{ck} x_u \cdot b$

The depth of Neutral axis for a singly Reinforced beam is calculated by Equillibrium of tensile and compressive forces

Equating ① & ②

x_u - Depth of Neutral axis

$$C_u = T_u$$

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

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

The centroid of area of stress block $\bar{x}$ is obtained by taking moment of areas of Rectangular portion and parabolic portion of stress block about the top of stress block

Rectangular portion (centroid)

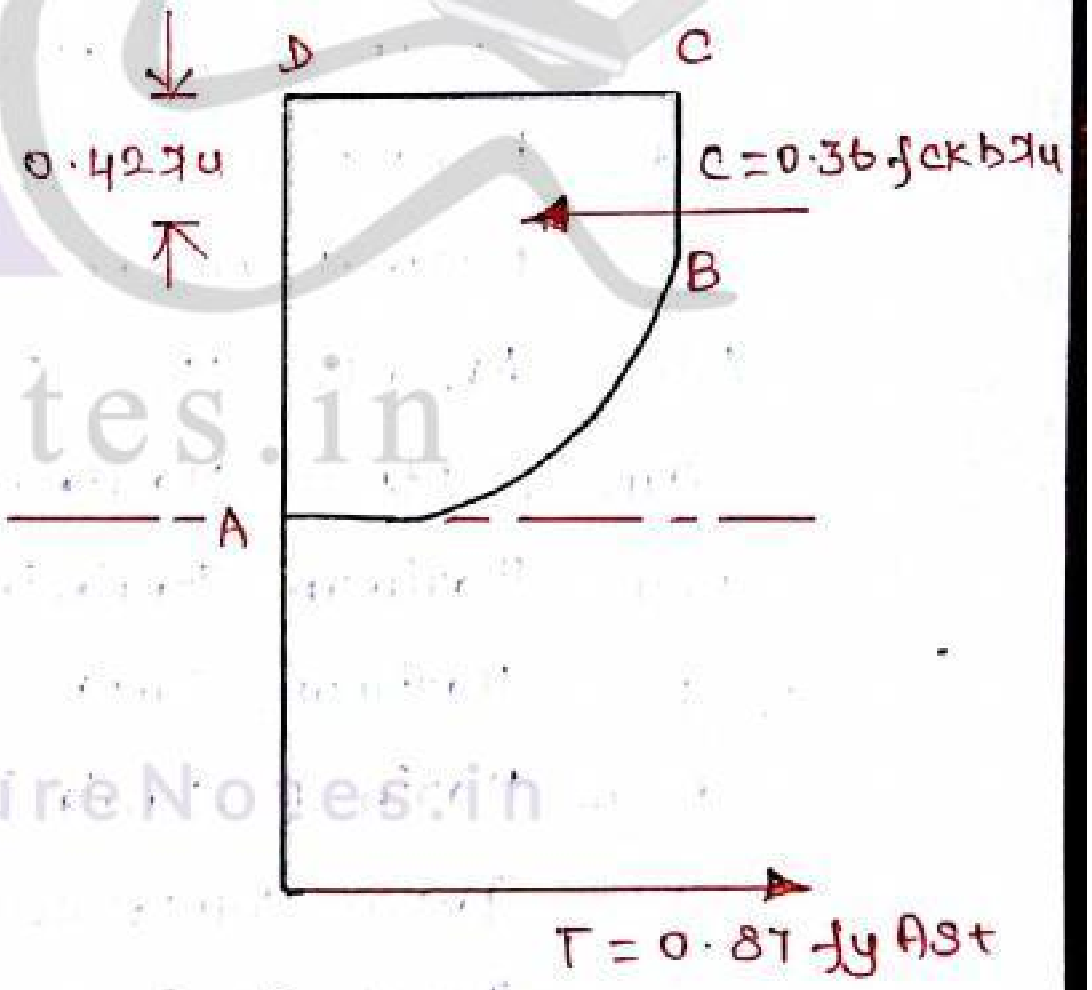
$$= \frac{3}{7} x_u \times \frac{1}{2}$$

$$= \frac{3}{14} x_u$$

Parabolic portion (centroid)

$$= \frac{3}{14} x_u + \frac{3}{7} x_u$$

$$= \frac{9}{14} x_u$$

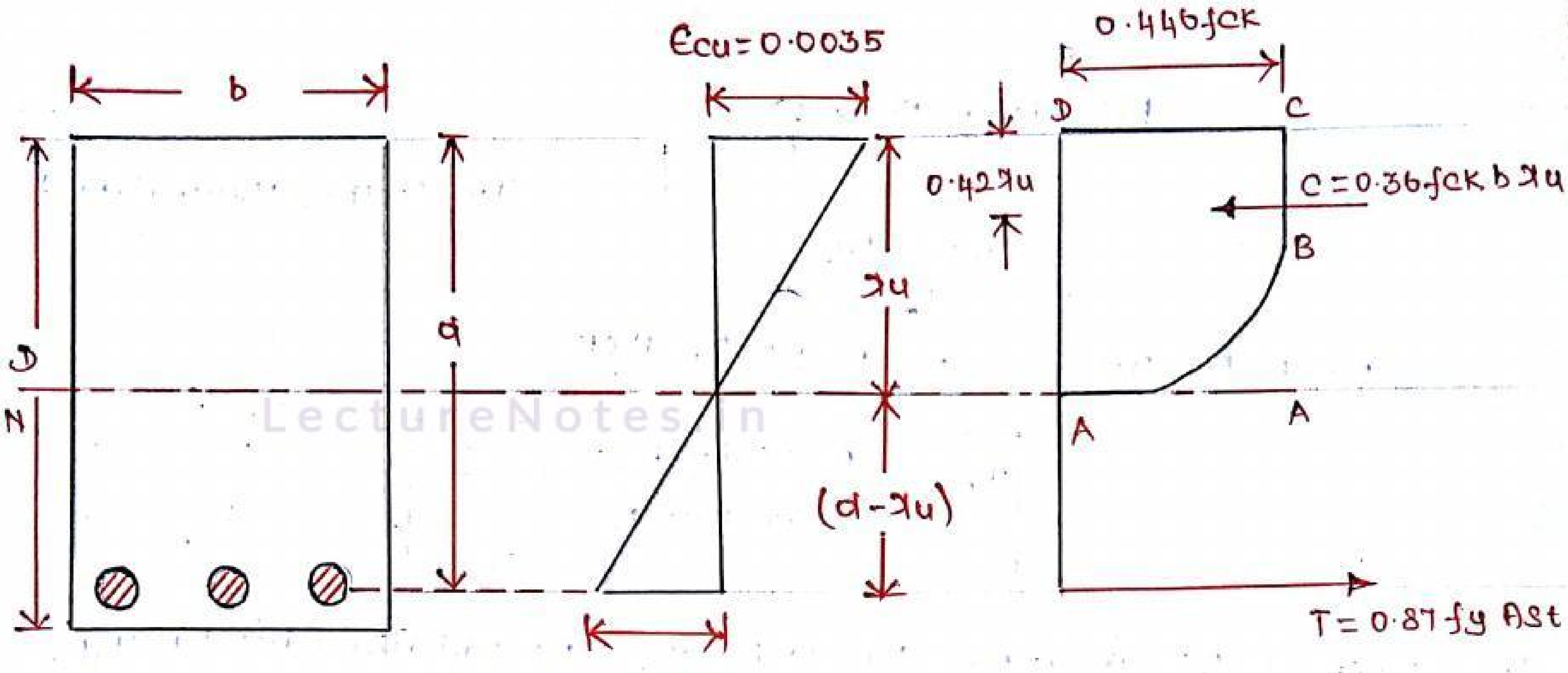


$$\bar{x} = \frac{A_1 x_1 + A_2 x_2}{\text{Area of Compression}}$$

$$= \frac{\left(0.1914\, f_{ck}\, x_u \cdot b \times \frac{3}{14} x_u\right) + \left(0.169 \times f_{ck}\, x_u \cdot b \times \frac{9}{14} x_u\right)}{0.36\, f_{ck}\, x_u \cdot b}$$

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

Analysis of Singly Reinforced Rectangular Section

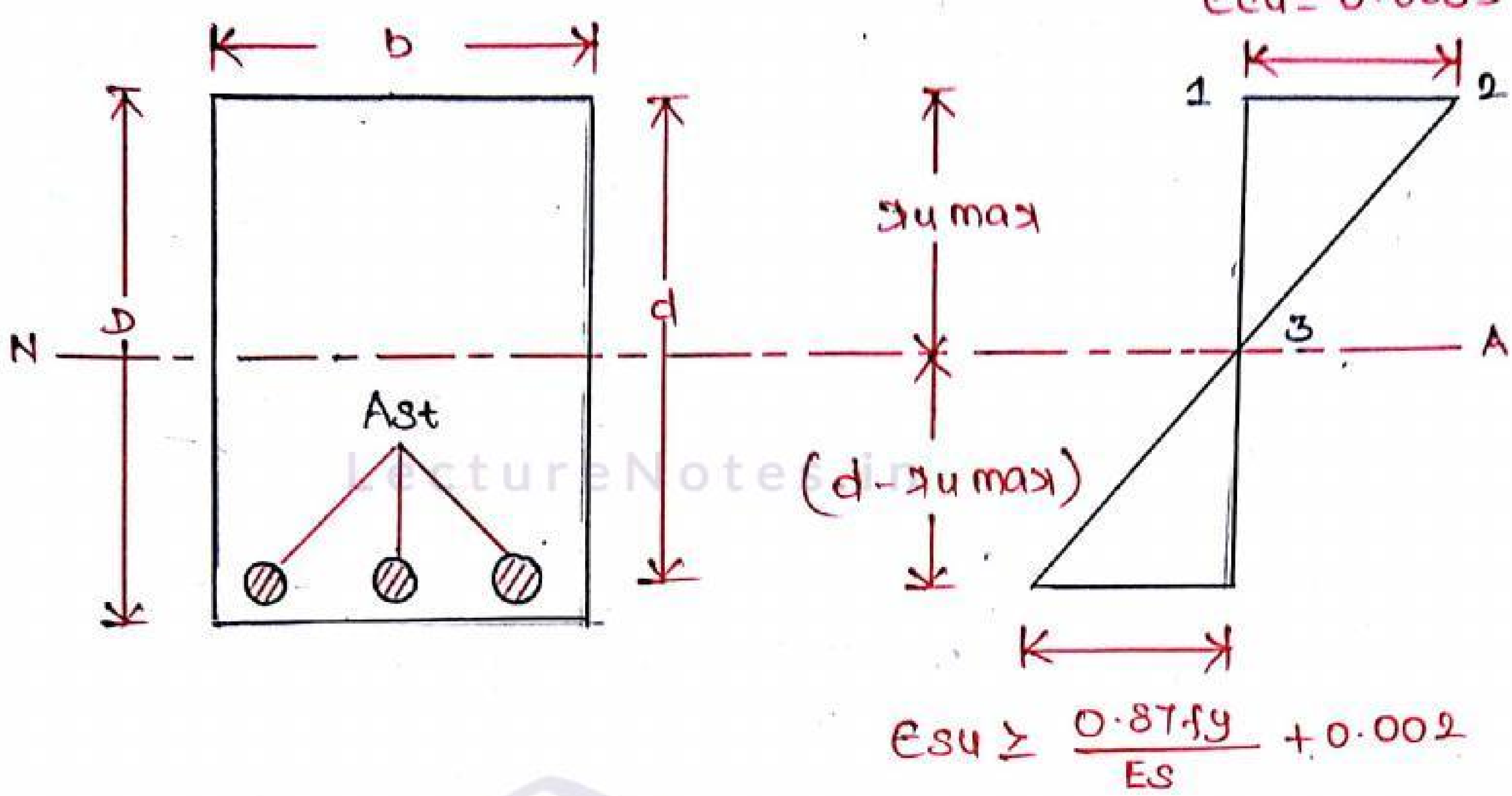


$$\epsilon_{su} \geq \frac{0.87 f_y}{E_s} + 0.002$$

CROSS SECTION   STRAIN DIAGRAM   STRESS DIAGRAM

- b – Width of Section
- d – Effective depth of beam
- D – Overall depth
- A_{st} – Area of Steel Reinforcement
- x_u – Depth of Neutral axis
- ϵ_{cu} – Maximum Strain in Concrete at Extreme fibre
- ϵ_c – Maximum Compression
- ϵ_{su} – Maximum Strain in Steel
- C – Total Compression
- T – Total Tension

Maximum depth of Neutral axis

- 

CROSS SECTION | STRAIN DIAGRAM

$$\boxed{x_u = x_u max}$$

By Comparing Triangles

$$x_u max = 0.0035$$

$$d - x_u max = 0.002 + \frac{0.87 f_y}{E_s}$$

$$\frac{d - x_u max}{x_u max} = \frac{0.002 + \frac{0.87 f_y}{E_s}}{0.0035}$$

$$\frac{d}{x_u max} - 1 = \frac{0.002 + \frac{0.87 f_y}{E_s}}{0.0035}$$

$$\frac{d}{x_u max} = \frac{1 + 0.002 + \frac{0.87 f_y}{E_s}}{0.0035}$$

$$x_{umax} = \frac{0.0035}{0.0035 + \left(\frac{0.87 f_y}{2 \times 10^5} + 0.002\right)}$$

Mild Steel

Fe 250

$$x_{umax} = \left[\frac{0.0035}{0.0035 + \left(\frac{0.87 \times 250}{2 \times 10^5} + 0.002\right)}\right] d$$

$$\boxed{x_{umax} = 0.531\, d}$$

HYSD

Fe 415

$$= \left[\frac{0.0035}{0.0035 + \left(\frac{0.87 \times 415}{2 \times 10^5} + 0.002\right)}\right] d$$

$$\boxed{x_{umax} = 0.479\, d}$$

Fe 500

$$x_{umax} = \frac{0.0035}{0.0035 + \left(\frac{0.87 \times 500}{2 \times 10^5} + 0.002\right)}$$

$$\boxed{x_{umax} = 0.456\,d}$$

GRADE OF STEEL	MAXIMUM STRAIN IN STEEL (ϵ_s)	MAXIMUM STRAIN IN CONCRETE (ϵ_c)	x_{umax}
Fe 250	0.00309	0.0035	0.53 d
Fe 415	0.00380	0.0035	0.48 d
Fe 500	0.00417	0.0035	0.46 d

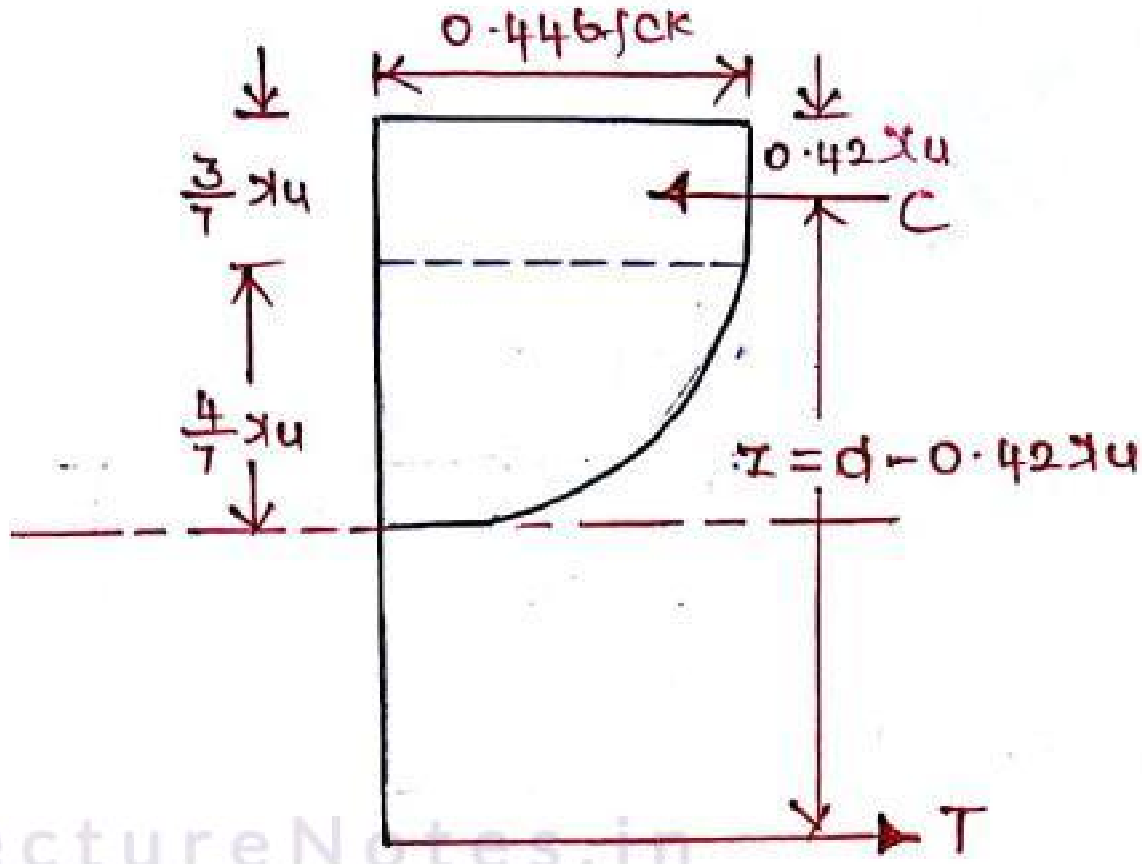
Moment of Resistance

It is the capacity of the section which can resist the moment coming due to External Loads

Lever arm (z)

It is the distance between the centroid of compressive force to the centroid of Tensile Force

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



Moment of Resistance for different Section

(i) Balanced Section ($x_u = x_{u\,max}$)

$$M_u = \text{Total Compression} \times \text{Lever arm}$$

$$= C \times z$$

$$= 0.36\, f_{ck}\, b\, x_{u\,max} \times (d - 0.42\, x_{u\,max})$$

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

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

Refer IS: 456-2000
G-1.1 (c)
Pg. No: 96

(OR)

$$M_u = \text{Total Tensile} \times \text{Lever arm}$$

$$= T \times z$$

$$= 0.87\, f_y\, A_{st} \times (d - 0.42\, x_{u\,max})$$

$$\boxed{M_u = 0.87\, f_y\, A_{st\,lim} \times (d - 0.42\, x_{u\,max})}$$

As per IS: 456-2000

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

In above equ^n sub. $\left(\frac{x_{u\,max}}{d}\right)$

Fe 250

$$M_{u\,lim} = 0.36 f_{ck} b d^2 \times 0.53 (1 - 0.42 \times 0.53)$$

$$= 0.149 f_{ck} b d^2$$

Fe 415

$$M_{u\,lim} = 0.36 f_{ck} b d^2 \times 0.48 (1 - 0.42 \times 0.48)$$

$$= 0.138 f_{ck} b d^2$$

Fe 500

$$M_{u\,lim} = 0.36 f_{ck} b d^2 \times 0.46 (1 - 0.42 \times 0.46)$$

$$= 0.133 f_{ck} b d^2$$

GRADE OF STEEL	$M_{u\,lim}$
Fe 250	$0.149 f_{ck} b d^2$
Fe 415	$0.138 f_{ck} b d^2$
Fe 500	$0.133 f_{ck} b d^2$

(ii) Under Reinforced Section ($x_u < x_{umax}$)

$$M_u = \text{Total Tensile} \times \text{Lever arm}$$

$$= T \times Z$$

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

(iii) Over Reinforced Section ($x_u > x_{umax}$)

$$M_u = \text{Total Compression} \times \text{Lever arm}$$

$$= C \times Z$$

$$\boxed{M_u = 0.36\, f_{ck}\, b\, x_{umax} \times d \left(1 - 0.42 \frac{x_{umax}}{d}\right)}$$

Compare actual depth of Neutral axis with $x_{u\,lim}$ and identify whether the section is Balanced, Under reinforced section, over reinforced section

$x_u = x_{umax} \rightarrow$ Balanced Section

$x_u < x_{umax} \rightarrow$ Under Reinforced Section

$x_u > x_{umax} \rightarrow$ Over Reinforced Section

Percentage of Steel

$$\boxed{P_t = \frac{100\, A_{st}}{bd}}$$

Area of Minimum Reinforcement

$$A_{st} > A_{st\,min}$$

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

Type of RCC Beam Section

* Balanced section
* Under Reinforced section
* Over Reinforced section

Balanced Section

* A balanced section is that in which stress in concrete and steel reach their permissible value at the same time
* It is assumed that both concrete and steel will fail at the same time
* Hence for design balanced section is taken

$$\boxed{x_u = x_{u\,max}}$$

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

(OR)

$$\boxed{M_{u\,lim} = 0.87 f_y \, A_{st\,lim} \times (d - 0.42\, x_{u\,max})}$$

Under Reinforced Section

* In an URS, the Ast provided is less than of balanced section ($A_{st} < A_{st\,bal}$)

* In URS, the stress in steel first reaches it permissible value while the concrete is under stressed

$$\boxed{x_u < x_{u\,max}}$$

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

The various features of URS are as follows

* Steel is fully stressed while concrete not (i.e stress in steel is σ_{st} (permissible) but stress in concrete is less than σ_{cbc}

* The Actual Neutral axis lies above the critical Neutral axis

* The Area of Steel is less than balanced section hence the section is economical

* The Failure nature is ductile

* The Moment of Resistance is less than balanced section

* In URS the failure is ductile because steel fails first and sufficient warning is given before collapse

* Due to ductile failures and economy, they URS are preferred by designers

Over Reinforced Section

* In an ORS the percentage of steel provided is greater than the balanced section ($A_{st} > A_{st\,bal}$)

* The Actual Neutral axis shift downward

* In this section, stress in concrete reaches its permissible value while steel is not fully stressed

* Concrete is brittle and its fails by crushing suddenly

* As steel is not fully utilised, the ORS is uneconomical (steel is much costilier than concrete)

$$\boxed{x_u > x_{u\,max}}$$

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

The various features of ORS are as follows

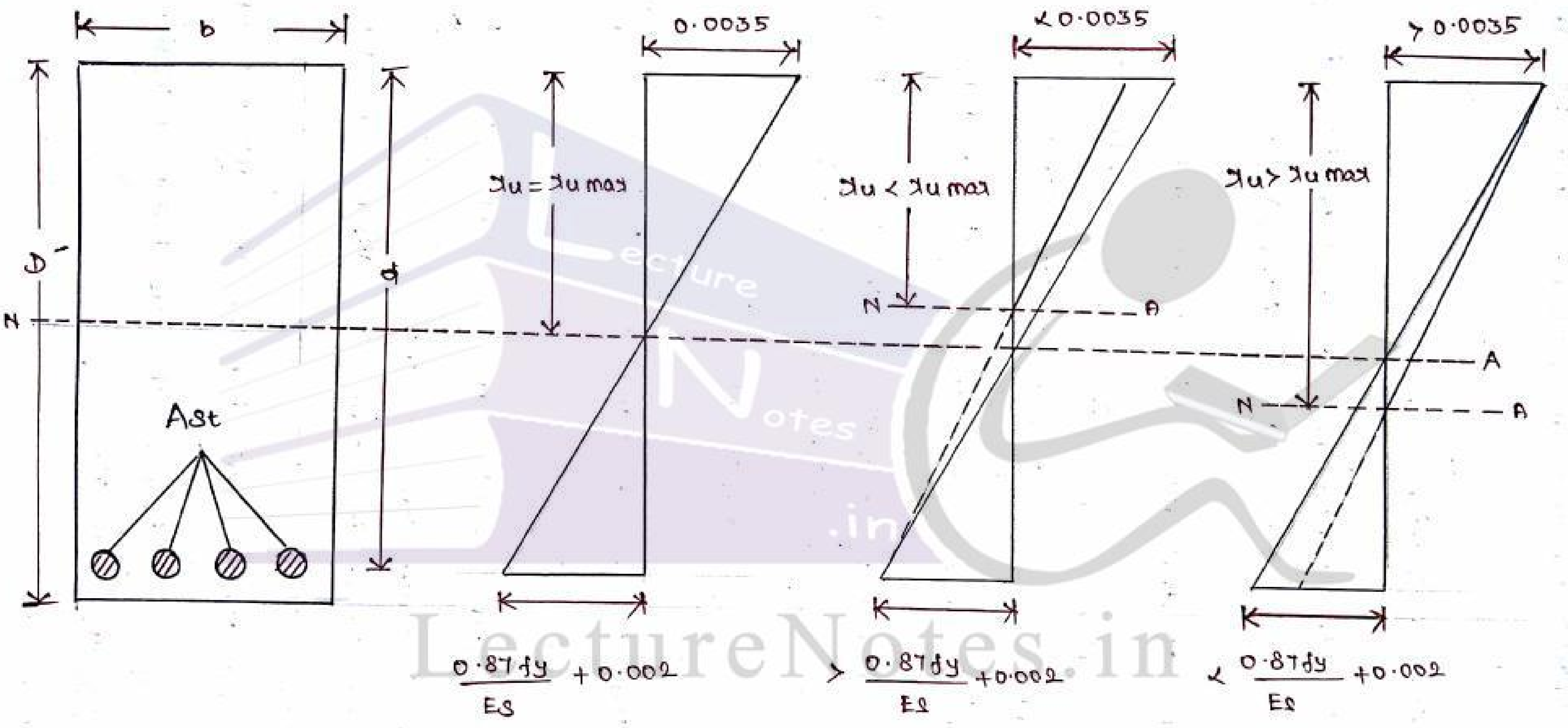
* Concrete is fully stressed while steel is not (i.e. the stress in concrete is at its permissible value σ_{cbc} but stress in steel is less than σ_{st}

* The Actual Neutral axis is below the critical Neutral axis

* The area of steel is more than the balanced section, so the section is uneconomical.

* Sudden failure.

DIFFERENT TYPES OF RCC BEAM SECTION



CROSS SECTION | BALANCED SECTION | UNDER REINFORCED SECTION | OVER REINFORCED SECTION

Problem on Analysis of Singly Reinforced Beam

Prblm. No: 1

Analyse the Rectangular beam Section of 250 mm width and 450 mm Effective depth. Determine the Ultimate Moment of Resistance for Area of Tension Steel, Ast = 3 No's of 16 mm ϕ. Consider M20 and Fe 415 Combination

Given data

Breadth of beam (b) = 250 mm

Depth of beam (d) = 450 mm

Use M20 Grade of Concrete, Fe 415 Grade of Steel

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

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

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

$$= 3 \times \frac{\pi}{4} (16)^2$$

$$= 603\ mm^2$$

To Find

$M_u = ?$

Solution

Depth of Neutral axis (x_u)

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

$$= \frac{0.87 \times 415 \times 603}{0.36 \times 20 \times 250}$$

$$\boxed{x_u = 120.95 \text{mm}}$$

Maximum depth of Neutral axis ($x_{u max}$)

For Fe 415

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

$$x_{u lim} = 0.48 \times d$$

$$= 0.48 \times 450$$

$$\boxed{x_{u max} = 216 \text{mm}}$$

$$x_u < x_{u max}$$

Hence the section is Under Reinforced section

Ultimate Moment of Resistance

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

$$= 0.87 \times 415 \times 603 \times 450 \left[1 - \frac{603 \times 415}{450 \times 250 \times 20}\right]$$

$$= 86.91 \times 10^6 \text{ N.mm}$$

$$\boxed{M_u = 86.91 \text{ KN.m}}$$

Prblm. no: 2

- A Rectangular Rcc beam of M20 Grade Concrete is 280mm wide and 500mm deep Effective. It is reinforced with 4 No's of 16mm ϕ Mild Steel bars in the Tension zone. Calculate the Moment of Resistance of the section at the Limit State of Collapse

Given data

Breadth of beam (b) = 280mm

Depth of beam (d) = 400mm

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

$f_y = 250\ N/mm^2$

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

$= 4 \times \frac{\pi}{4} (16)^2$

$A_{st} = 804.25\ mm^2$

To Find

$M_u = ?$

Solution

Actual depth of Neutral axis

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

$$= \frac{0.87 \times 250 \times 804.25}{0.36 \times 20 \times 280}$$

$$\boxed{x_u = 86.77mm}$$

Maximum depth of Neutral axis

For Fe 250

$$x_{u\,max} = 0.531\,d$$

$$= 0.531 \times 500$$

$$\boxed{x_{u\,max} = 265.5\,mm}$$

$$x_u < x_{u\,max}$$

Hence the section is Under Reinforced Section

Ultimate Moment of Resistance

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

$$= 0.87 \times 250 \times 804.25 \times 500 \left[1 - \frac{804.25 \times 250}{20 \times 80 \times 400}\right]$$

$$= 59.98 \times 10^6\ Nmm$$

$$\boxed{M_u = 59.985\ KN \cdot m}$$

Prblm. No: 3

- Determine the limiting moment of Resistance that can be obtained from the rectangular section of 300mm x 550 mm overall and also determine the area of steel required. Assume M20 & Fe 415 Grade steel

Given data

Breadth of beam (b) = 300mm

Overall depth of beam (D) = 550mm

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

$f_y = 415 \ N/mm^2$

Assume

Effective cover = 40 mm

Effective depth = overall depth - Effective cover

= 550 - 40

$d = 510mm$

To Find

$Ast_{req} = ?$

Solution

Maximum depth of Neutral axis

For Fe415

$x_{umax} = 0.48 d$

$= 0.48 \times 510$

$\boxed{x_{umax} = 244.3 \ mm}$

Area of Steel

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

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

$$= \frac{244.3 \times 0.36 \times 20 \times 300}{0.87 \times 415}$$

$$\boxed{A_{st} = 1461.5\ mm^2}$$

Limiting Moment of Resistance

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

$$= 0.36 \times 20 \times 300 \times 244.3 \left(510 - (0.42 \times 244.3)\right)$$

$$= 215.5 \times 10^6\ Nmm$$

$$\boxed{M_{u\,lim} = 215.5\ KN\cdot m}$$

Check ($M_{u\,lim}$)

$$M_{u\,lim} = 0.138 \times f_{ck}\, b d^2$$

$$= 0.138 \times 20 \times 300 \times 510^2$$

$$\boxed{M_{u\,lim} = 215.4\ KN\cdot m}$$

$$Ast_{req} = 1461.5\,mm^2$$

$$\text{No. of Bars} = \frac{Ast_{req}}{ast}$$

$$= \frac{1461.5}{\frac{\pi}{4} \times (20)^2}$$

$$= 4.65 \simeq 5\ \text{NO'S}$$

$$Ast_{prov} = n \times \frac{\pi}{4} d^2$$

$$= 5 \times \frac{\pi}{4} (20)^2$$

$$\boxed{Ast_{prov} = 1570.8\,mm^2}$$

Prblm. No: 4

A singly Reinforced beam of section 200mm x 400mm. IS reinforced with 4 bars of 16mm diameter. The beam IS simply supported over a span of 3m. Find the safe UDL the beam can carry. Use M20 Concrete and Fe 415 Steel

Given data

$b = 200mm$

$d = 400mm$

$l = 3m$

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

$f_y = 415\ N/mm^2$

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

$$= 4 \times \frac{\pi}{4} (16)^2$$

$$\boxed{A_{st} = 804.24 \ mm^2}$$

Solution

Depth of Neutral axis (x_u)

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

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

$$\boxed{x_u = 201.64 mm}$$

$$x_u > x_{umax}$$

* The Given section can be design as Over Reinforced section

As per IS: 456 - 2000

* In LSM over Reinforced section are not allowed because it wont give prior information / Warning before failure
* The Given section can be design as Balanced section

For Balanced section

$$M_u = 0.36 f_{ck} bd^2 . \frac{x_{umax}}{d} \left(1 - 0.42 \frac{x_{umax}}{d}\right)$$

For Fe 415

$$M_u\,lim = 0.138\, f_{ck}\, b d^2$$

$$= 0.138 \times 20 \times 200 \times 400^2$$

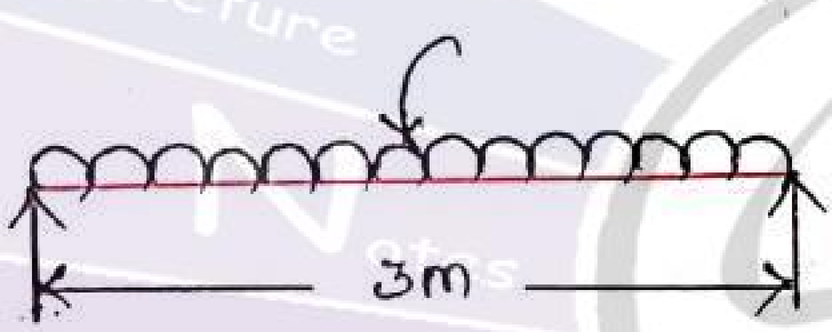
$$= 88.32 \times 10^6\ Nmm$$

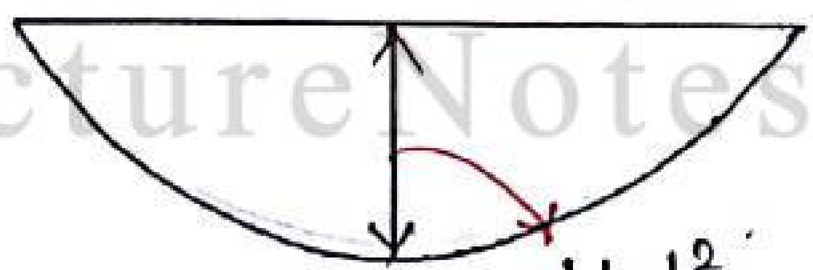
$$\boxed{M_u\,lim = 88.32\ KNm}$$

Safe Load (W)

For calculation of Load

B.M due to loading can be equated to moment of Resistance of Given Section





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

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

$$= \frac{W_u \times (3)^2}{8}$$

$$M_u = 1.125\, W_u$$

$$88.32 = 1.125\, W_u$$

$$W_u = \frac{88.32}{1.125} \Rightarrow \boxed{W_u = 78.50\ KN/m}$$

$$\text{Factor of Safety} = \frac{\text{Ultimate Load}}{\text{Safe Load}}$$

$$\text{Safe Load } (W) = \frac{\text{Ultimate Load } (W_u)}{\text{FOS}}$$

$$= \frac{78.50}{1.5}$$

$$\boxed{W = 52.33 \ KN/m}$$

Prblm.no:5:

A Singly reinforced beam 250 mm x 500 mm in section is reinforced with 4 bars of 16 mm diameter with an effective cover of 50 mm. Effective span of the beam is 6 m. Assuming M 20 concrete and Fe 250 steel. Determine the central concentrated load P than can be carried by the beam in addition to its self weight.

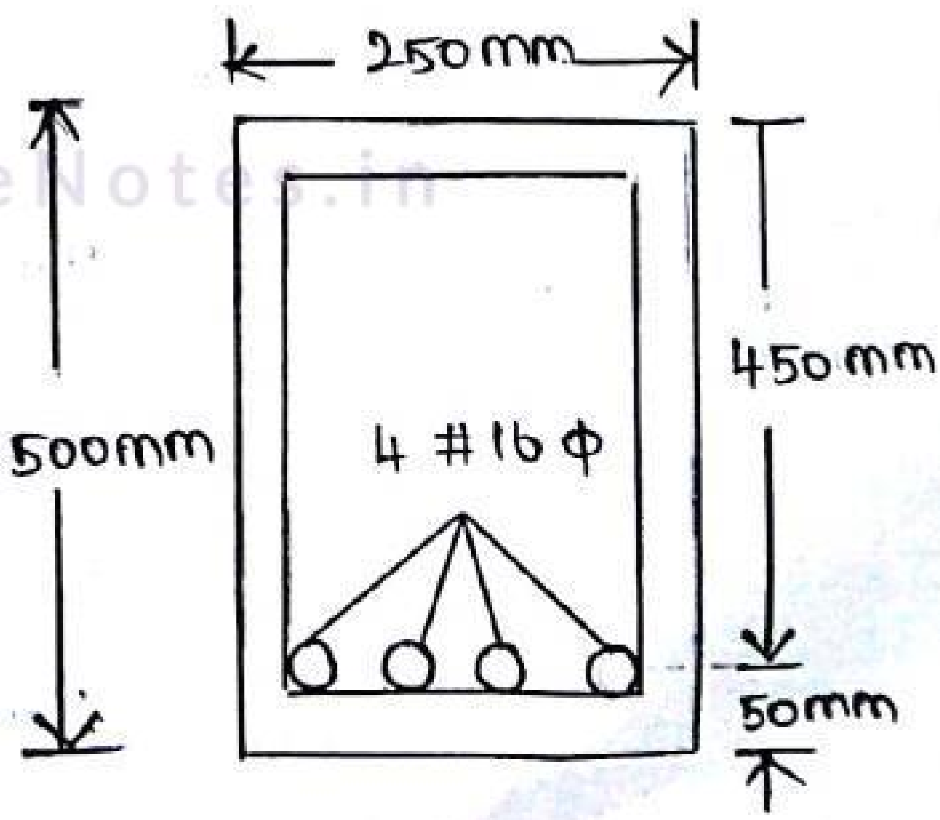
Given data

$b = 250 mm$

$D = 500 mm$

$d = 500 - 50$

$= 450 mm$



$Ast = n \times \frac{\pi}{4} d^2$

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

$Ast = 804 mm^2$

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

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

To Find

$P = ?$

Solution

Depth of Neutral axis (x_u)

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

$$= \frac{0.87 \times 250 \times 804}{0.36 \times 20 \times 250}$$

$$\boxed{x_u = 97.15 mm}$$

Maximum depth of Neutral axis ($x_{u\,max}$)

For Fe 250

$$x_{u\,max} = 0.53 d$$

$$= 0.53 \times 450$$

$$\boxed{x_{u\,max} = 238.5 mm}$$

$$x_u < x_{u\,max}$$

Hence it is under Reinforced section

Moment of Resistance

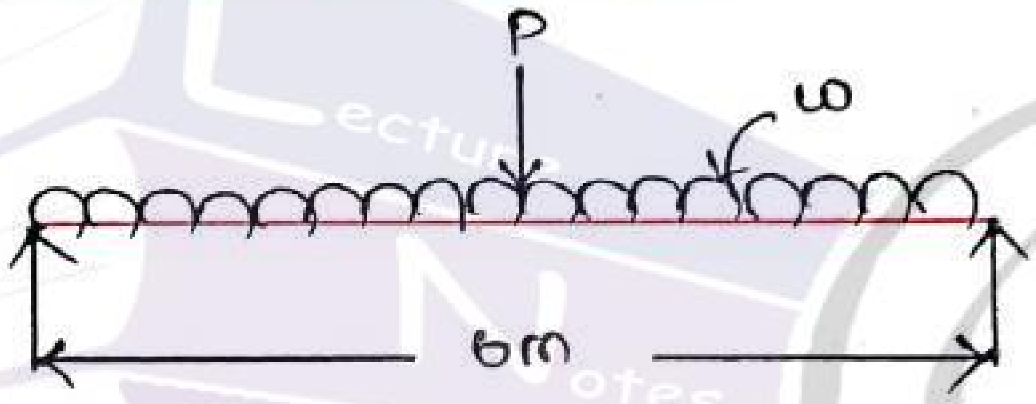
$$Mu = 0.87\,fy \cdot Ast \cdot d \left[1 - \frac{Ast\,fy}{bd\,fck}\right]$$

$$= 0.87 \times 250 \times 804 \times 450 \left[1 - \frac{804 \times 250}{250 \times 450 \times 20}\right]$$

$$= 71.609 \times 10^6 \text{ Nmm}$$

$$\boxed{Mu = 71.609 \text{ KN·m}}$$

Determine the Central point Load



Self Weight = Area of the section × Unit weight of Rcc

$$= 0.25 \times 0.5 \times 25$$

$$\boxed{\omega = 3.125 \text{ KN/m}}$$

Design Ultimate Self weight } = 1.5 × ω (PSF)

$$= 1.5 \times 3.125$$

$$\boxed{\omega u = 4.687 \text{ KN/m}}$$

Design Moment $(M_u) = \frac{w_u l^2}{8} + \frac{P_u L}{4}$

$P_u = 1.5P$

$$M_u = \frac{4.68 \times 6^2}{8} + \frac{1.5P \times 6}{4}$$

$M_u = 21.093 + 2.25P$

Equating it to Moment of Resistance of the section, we get

$$21.093 + 2.25P = 71.56$$

$$\boxed{P = 22.429\,KN}$$

DESIGN PROCEDURE OF SINGLY REINFORCED BEAM

Step: 1

Refer IS: 456-2000
Cl. no: 23.2.1 (a)
Pg. No: 37

Cross Section Dimension

The depth of beam is Fixed based on $\frac{\text{Span}}{\text{depth ratio}}$ to Satisfy the deflection requirements

$$\text{Effective depth (d)} = \frac{\text{Span}}{\text{Depth}}$$

* Effective Span (L) = Clear Span + Effective depth
* Effective Span (L) = clear Span + C/c Supports

} Whichever is Smaller

Adopt

L = ____ m

S.NO	SPAN RANGE	LOADING	SPAN/DEPTH RATIO (L/d)
1	3 to 4m	Light	15 to 20
2	5 to 10m	Medium to Heavy	12 to 15
3	> 10m	Heavy	12

Step: 2

Loads

* Self Weight = Area of Section x Unit weight of RCC

* Imposed load = ____ KN/m^2

Total Load (W) = ____ KN/m^2

Design Ultimate Load } (W_u) = PSF X W

Unit wt. of RCC = 25 $\frac{KN}{m^2}$

Partial Safety Factor } = 1.5

Step:3

Ultimate Moment and Shear Force

Ultimate Moment

$$Mu = 0.125\ Wu L^2$$

Shear Force

$$Vu = 0.5\ Wu L$$

Step:4

Tension Reinforcement

$$Mu\ lim = 0.149\ fck\ bd^2 \quad (Fe\ 250)$$

$$Mu\ lim = 0.138\ fck\ bd^2 \quad (Fe\ 415)$$

$$Mu\ lim = 0.133\ fck\ bd^2 \quad (Fe\ 500)$$

Compare

$Mu = Mu\ lim \rightarrow$ Balanced section

$Mu < Mu\ lim \rightarrow$ Under Reinforced section

$Mu > Mu\ lim \rightarrow$ over Reinforced section

Moment of Resistance

$$Mu = 0.87\ fy\ Ast\ d \left[1 - \frac{fy\ Ast}{fck\ bd}\right]$$

Refer IS:456-2000
G-1.1 (b)
Pg. No: 96

$Ast_{req} = \_\_\_\_\_ mm^2$

Minimum Ast provided

Refer IS: 456-2000
Cl. No: 26.5.1.1
Pg. No: 47

$$Ast_{min} = \frac{0.85bd}{fy}$$

Note:

* The diameter of Hanger bars should not be less than of 10mm and that of Main bars 12mm

* The Nominal diameter of bars used are 8mm, 10mm, 12mm, 16mm 20, 25 and 32mm

$$(\text{No of Bars})_{prov} = \frac{Ast}{ast}$$

$$Ast_{prov} = \_\_\_\_ mm^2$$

Step: 5

Check for Shear Stress

Nominal Shear Stress

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

Pg. No: 72
Cl. No: 40.1

Where,

V_u = Shear force due to design loads

b = Width of the beam

d = Effective depth of the beam

τ_v = Shear stress

τ_c = Design shear stress

Percentage of Tension Reinforcement

- $$P_t = \frac{100 A_{st}}{bd}$$

Read Out

Design Shear Strength of concrete

Pg. No: 73
Table : 19

Compare

$$\tau_v < \tau_c$$

Step: 6

Check for Deflection Control

$P_t =$ ______

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{basic} \times K_t \times K_c \times K_f$$

$$f_s = 0.58 \times f_y \times \frac{A_{st\,req}}{A_{st\,prov}}$$

Modification factor

$K_t \rightarrow$ Fig. No: 4
$K_c \rightarrow$ Fig. No: 5
$K_f \rightarrow$ Fig. No: 6

Pg. No: 38, 39

$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{provided}$$

Prblm. NO: 6

Design a Singly Reinforced Concrete beam to Suit the Following data

Clear Span = 3m

Width of Support = 200 mm

Working Live Load = 6 KN/m

M 20 Grade of Concrete

Fe 415 Grade of Steel (HYSD bars)

Stresses

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

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

Step: 1

Cross Sectional dimension

Refer Table: 8.1 (In IS: 456-2000) and adopt a Span/Depth ratio

$$\text{Effective depth } (d) = \frac{\text{Span}}{\text{depth}}$$

(x 1000)

3m → 3000 mm

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

$$= 150\text{mm}$$

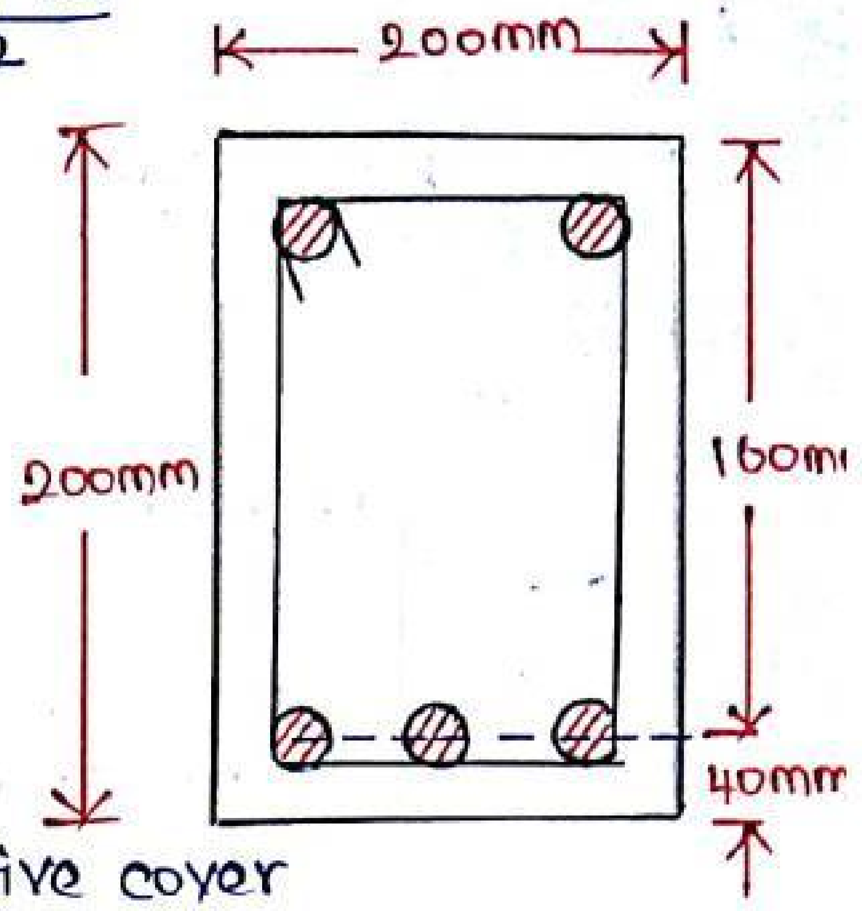
Simply Supported beam } = 20

$$\text{Effective cover } (d') = \text{clear cover} + \text{stirrups} + \frac{\text{Main bar}}{2}$$

$$= 25 + 8 + \frac{12}{2}$$

$$= 39 \simeq 40\,mm$$

$$\boxed{d' = 40\,mm}$$



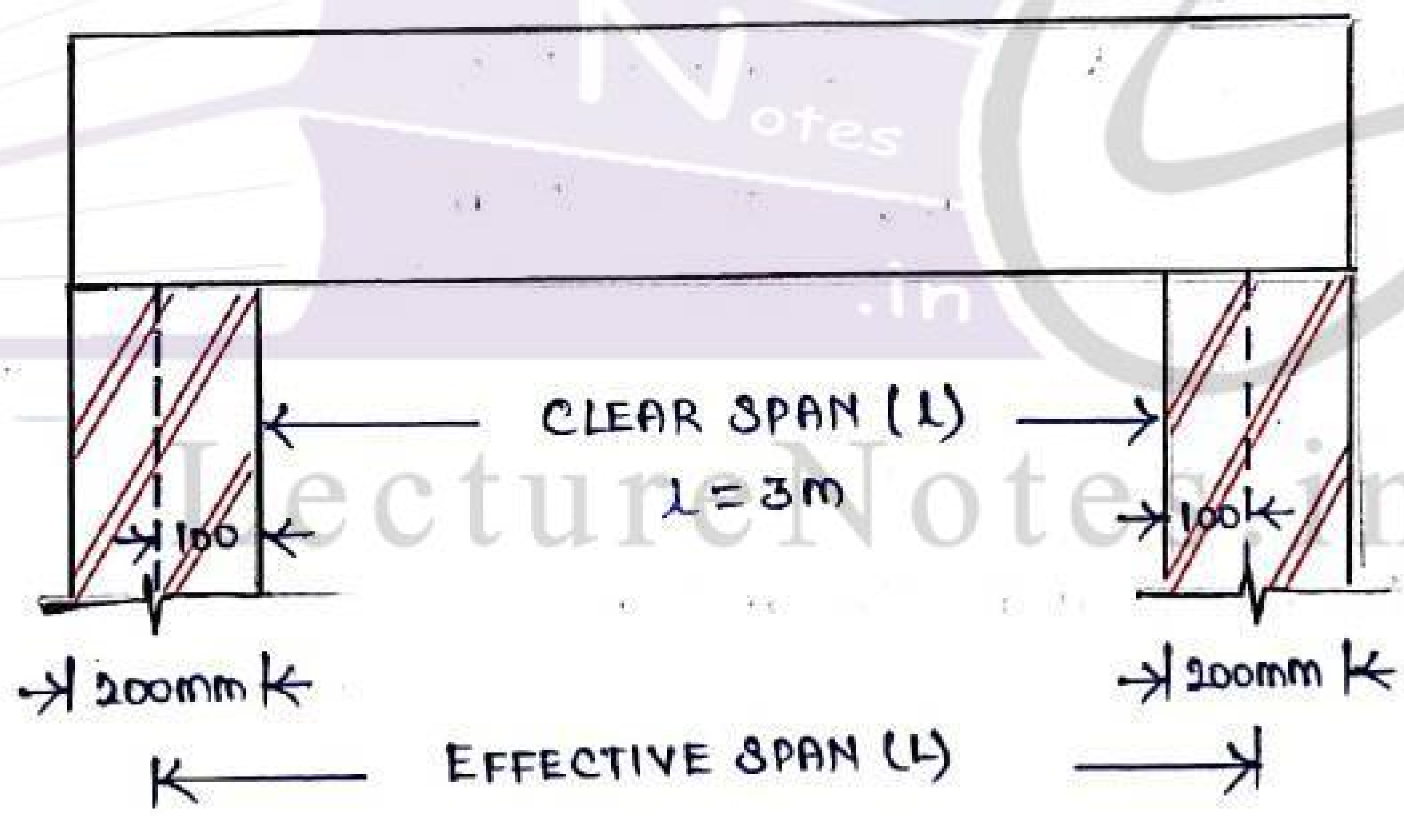
$$\text{Overall depth } (D) = \text{Effective depth} + \text{Effective cover}$$

$$= 160 + 40$$

$$\boxed{D = 200\,mm}$$

Beam Minimum clear cover $\}$ = 25mm

Effective Span (L)



$$\text{Effective span } (L) = \text{clear span} + \text{Effective depth}$$

$$= 3 + 0.16$$

$$= 3.16\,m$$

(÷1000)

160mm → 0.16m

$$\text{Effective span } (L) = \text{clear span} + \text{c/c of supports}$$

$$= 3 + (0.10 + 0.10)$$

$$= 3.2\,m$$

(÷1000m)

100mm → 0.10m

Adopt

$L = 3.16 m$

Step: 2

Loads

(÷ 1000)

200mm → 0.2m

Dead Load (D.L) = b × D × Unit wt. of RCC

= 0.2 × 0.2 × 25

= 1.00 KN/m

Live Load (L.L) = 6.00 KN/m

Total Load (W) = 7.00 KN/m

Design Ultimate Load (Wu) = PSF × W

= 1.5 × 7.00

$\boxed{Wu = 10.5 \text{ KN/m}}$

Step: 3

Ultimate Moment and Shear Force

Ultimate Moment

$Mu = 0.125 \, Wu L^2$

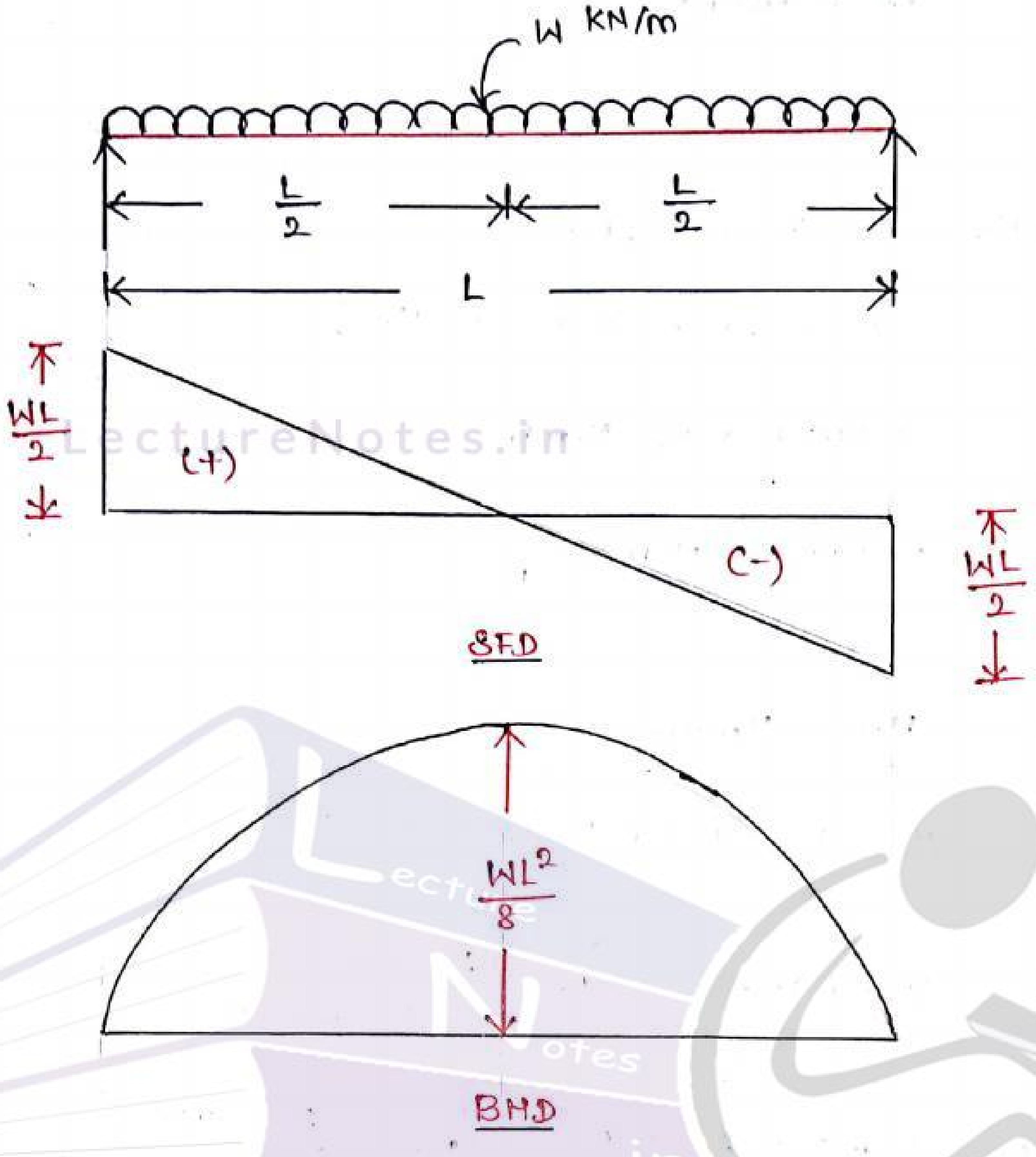
$= 0.125 \times 10.5 \times 3.16^2$

$\boxed{Mu = 13.1 \text{ KN.m}}$

(OR)

$M = \frac{WuL^2}{8}$

SHEAR FORCE AND BENDING MOMENT DIAGRAM FOR A SSB CARRYING UDL



$$= \frac{10.5 \times 3.16^2}{8}$$

$$\boxed{M_u = 13.1 \text{ KN·m}}$$

Shear Force

$$V_u = 0.5\, W_u L$$

$$= 0.5 \times 10.5 \times 3.16$$

$$\boxed{V_u = 16.59 \text{ KN}}$$

(OR)

$$V_u = \frac{W_u L}{2}$$

$$= \frac{10.5 \times 3.16}{2}$$

$$\boxed{V_u = 16.59 \text{ KN}}$$

Step: 4

Tension Reinforcement

For Fe 415

$$Mu\ lim = 0.138\ fck \cdot b \cdot d^2$$

$$= 0.138 \times 20 \times 200 \times 160^2$$

$$= 14.13 \times 10^6\ N.mm$$

$$\boxed{Mu\ lim = 14.13\ KN.m}$$

$$Mu < Mu\ lim$$

Hence the section is under Reinforced section

$$Mu = 0.87\ fy \cdot Ast \cdot d \left(1 - \frac{Ast\ fy}{b \cdot d \cdot fck}\right)$$

$$13.1 \times 10^6 = 0.87 \times 415 \times Ast \times 160 \left(1 - \frac{Ast \times 415}{200 \times 160 \times 20}\right)$$

$$= 57.78 \times 10^3\ Ast \left(1 - 6.484 \times 10^{-4}\ Ast\right)$$

$$= 57.78 \times 10^3\ Ast - 37.44\ Ast^2$$

$$37.44\ Ast^2 - 57.78 \times 10^3\ Ast + 13.1 \times 10^6 = 0$$

$$\boxed{Ast_{req} = 276.35\ mm^2}$$

Minimum Ast

$$Ast\ min = \frac{0.85\ bd}{fy}$$

$$= \frac{0.85 \times 200 \times 160}{415}$$

$$\boxed{Ast_{min} = 65.54 \ mm^2}$$

The Minimum area of Tension reinforcement shall be Not less than that

$$Ast > Ast_{min}$$

$$\text{No. of Bars} = \frac{Ast_{req}}{ast}$$

$$= \frac{276.35}{\frac{\pi}{4} \times (12)^2}$$

$$= 2.44 \simeq 3 \text{ No's}$$

$$\boxed{NOB_{prov} = 3}$$

$$Ast_{prov} = n \times \frac{\pi}{4} d^2$$

$$= 3 \times \frac{\pi}{4} (12)^2$$

$$\boxed{Ast_{prov} = 339 \ mm^2}$$

Provide 3 bars of 12mm ϕ ($Ast = 339 mm^2$)

2 Hanger bars of 10 mm ϕ on compression zone

Step:5

Check for Shear Stress

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

$$= \frac{16.59 \times 10^3}{200 \times 160}$$

(x 1000)

16.59 KN → 16.59×10^3 N

$$= \frac{16.59 \times 10^3}{32000}$$

$$\boxed{\tau_v = 0.51 \ N/mm^2}$$

Percentage of Tension Reinforcement

$$P_t = \frac{100 \ Ast}{bd}$$

$$= \frac{100 \times 339}{200 \times 160}$$

$$\boxed{P_t = 1.05\%}$$

Read Out (Design Shear Strength of Concrete)

Refer IS:456-2000
Table - 19
Pg. No: 73

P_t		τ_c
1.00	→	0.62
1.05	→	?
1.25	→	0.67

$$\frac{(1.25 - 1.00)}{(1.25 - 1.05)} = \frac{(0.67 - 0.62)}{(0.67 - x)}$$

$$\boxed{\tau_c = 0.63 \ N/mm^2}$$

$\tau_v < \tau_c$

Hence ok

Nominal shear reinforcement are provided using 6mm ϕ Two Legged Stirrups

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

S_v - Spacing

A_{sv} - Total cross section area of Stirrups

$$= \frac{2 \times \frac{\pi}{4}(6)^2 \times 0.87 \times 250}{0.4 \times 200}$$

$$\boxed{S_v = 152mm}$$

But

$S_v \ngtr 0.75 d$

$= 0.75 \times 160$

$= 120 mm$

Adopt

Spacing of Stirrups as 120mm

Step: 6

Check for Deflection Control

$P_t = 1.05$

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{Basic} \times K_t \times K_c \times K_f$$

$$\left(\frac{L}{d}\right)_{Basic} = 20$$

$$f_s = 0.58 \times f_y \times \frac{A_{st\,req}}{A_{st\,prov}}$$

$$= 0.58 \times 415 \times \frac{276.35}{339}$$

$$= 196.21 \simeq 190$$

$\boxed{f_s = 190}$ $\boxed{K_t = 1.1}$

$$\left(\frac{L}{d}\right)_{Max} = 20 \times 1.1 \times 1 \times 1$$

$$= 22$$

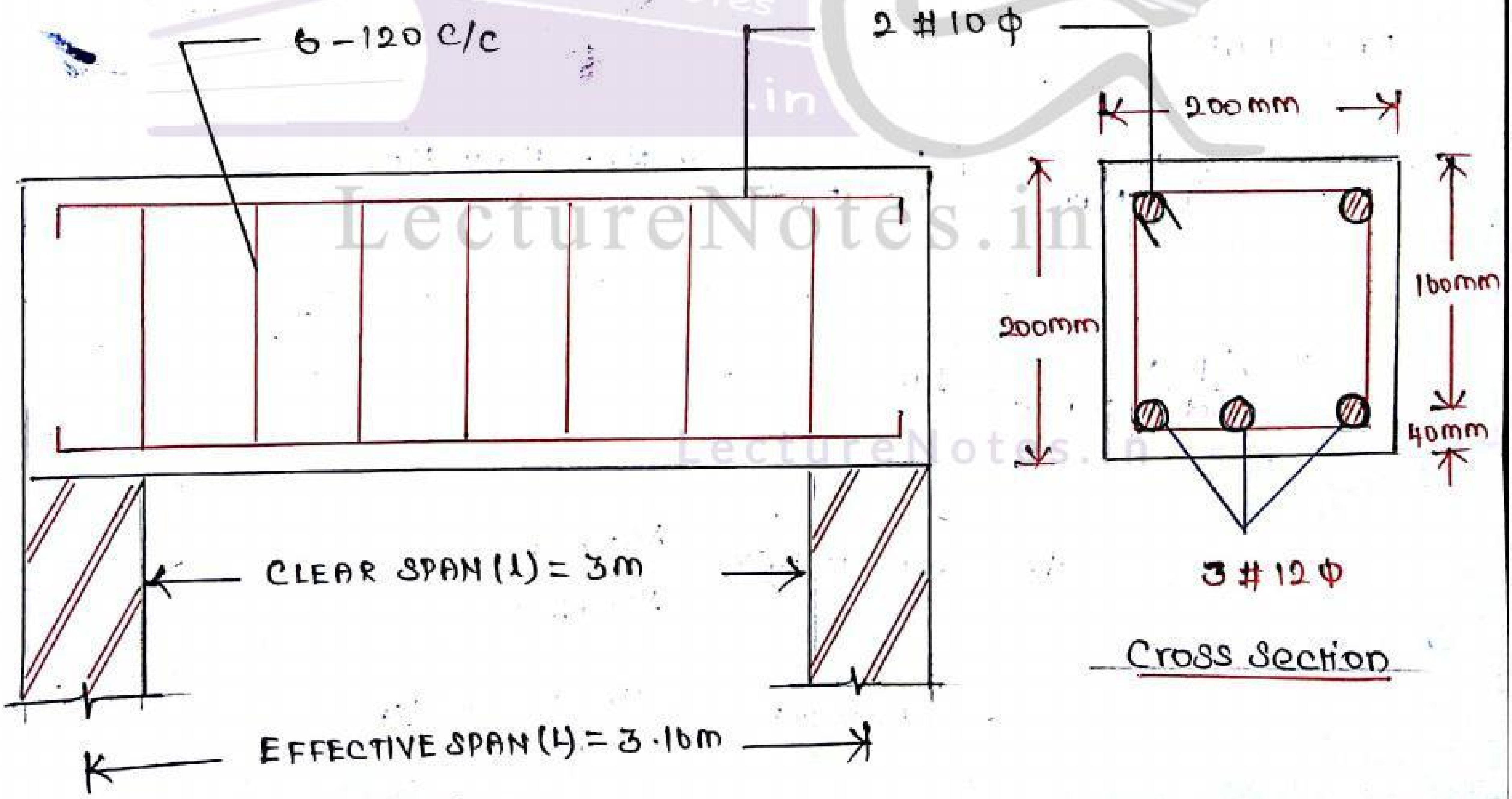
$$\left(\frac{L'}{d}\right)_{prov} = \frac{3.16}{0.16}$$

$$= 19.75$$

$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{provided}$$

Hence deflection control is satisfactory

Reinforcement Details in Singly Reinforced Rectangular beam



Longitudinal Section

Cross Section

Prblm. No: 7

- Design a singly reinforced concrete beam of clear span 5m to support a design working live load of 10 KN/m. Adopt M20 Grade concrete and Fe 415 HYSD bars

Give data

Clear span = 5m

Working live load = 10 kN/m

M20 Grade of concrete and Fe 415 HYSD Bars.

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

$f_y = 415\ N/mm^2$

Step: 1

Cross sectional dimension

$$\text{Effective depth } (d) = \frac{\text{Span}}{\text{depth}}$$

Simply Supported beam } = 20

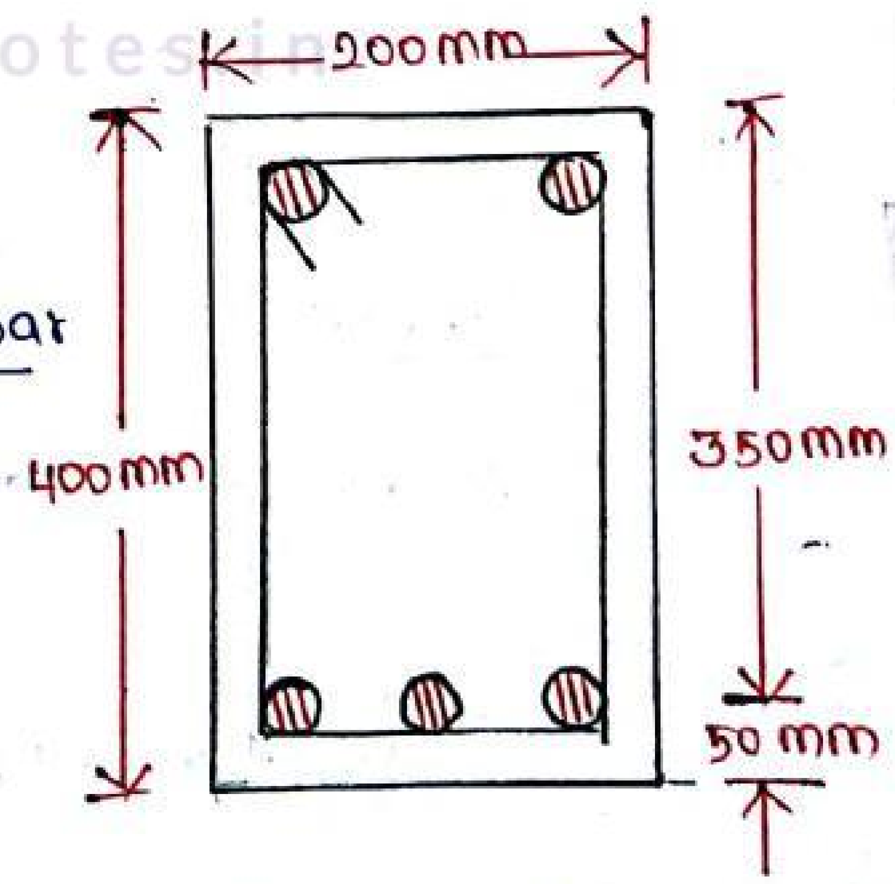
$$= \frac{5000}{15}$$

$5m \xrightarrow{(\times 1000)} 5000mm$

$$= 333mm$$

Adopt

$$\boxed{d = 350mm}$$



$$\text{Effective cover } (d') = \text{Clear cover} + \text{stirrups} + \frac{\text{Main bar}}{2}$$

$$= 25 + 8 + \frac{20}{2}$$

$$= 43 \simeq 50mm$$

$$\boxed{d' = 50mm}$$

Overall depth (D) = Effective depth + Effective cover

$= d + d'$

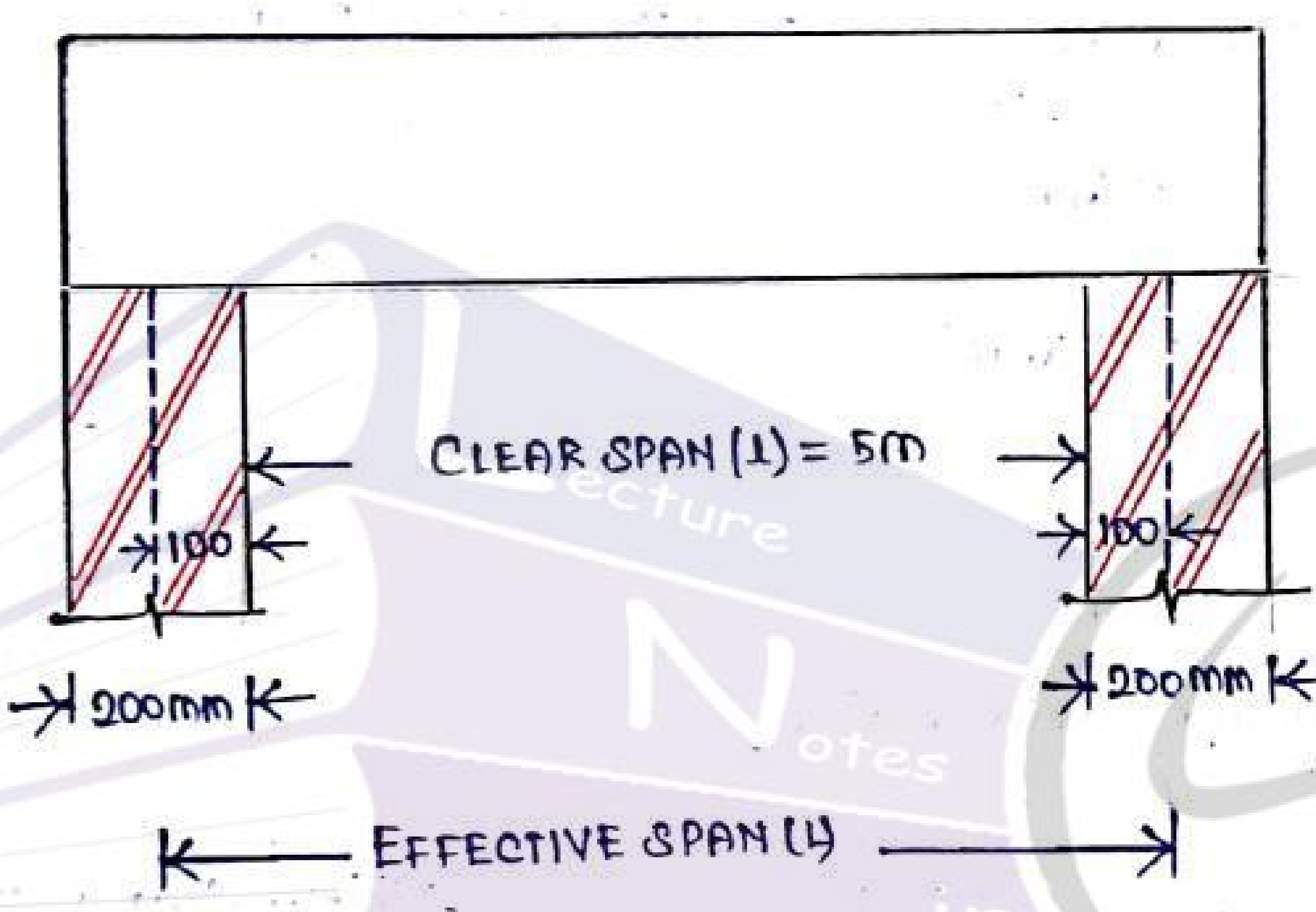
$= 350 + 50$

$\boxed{D = 400\,mm}$

Assume,

Width of beam (b) = 200mm

Effective span (L)



Effective span (L) = clear span + Effective depth

$= 5 + 0.35$

$\boxed{L = 5.35\,m}$

(÷1000)

350mm → 0.35m

Step:2

Loads

Dead Load (g) = $\overset{(b)}{0.2} \times \overset{(D)}{0.4} \times 25$

$= 2.00$ KN/m

(÷1000)

200mm → 0.2m

Unit wt of Rcc = 25 $\frac{KN}{m^2}$

(÷1000)

400mm → 0.40m

Live load (q) = 10.00 KN/m

Total Working load (W) = 12.00 KN/m

Design ultimate load (W_u) = PSF x W_u

$= 1.5 \times 12$

$\boxed{W_u = 18.00 \text{ KN.m}}$

Step:3

Ultimate Moment and Shear Force

Ultimate Moment

$M_u = 0.125\, W_u L^2$

$= 0.125 \times 18 \times 5.35^2$

$\boxed{M_u = 64.4 \text{ KN}}$

(OR)

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

$= \frac{18 \times 5.35^2}{8}$

$\boxed{M_u = 64.4 \text{ KN}}$

Shear Force

$V_u = 0.5\, W_u L^2$

$= 0.5 \times 18 \times 5.35^2$

$\boxed{V_u = 48.2 \text{ KN}}$

(OR)

$V_u = \frac{WL}{2}$

$= \frac{18 \times 5.35}{2}$

$\boxed{V_u = 48.15 \text{ KN}}$

Step: 4

Reinforcement

Limiting Moment of the Section is

$M_{u\,lim} = 0.138\, f_{ck}\, b d^2$ (Fe 415)

$= 0.138 \times 20 \times 200 \times 350^2$

$= 68 \times 10^6$ N·mm

$\boxed{M_{u\,lim} = 68 \text{ KN·m}}$

$M_u < M_{u\,lim}$

Hence the section is Under Reinforced Section

Area of Reinforcement

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

$$64.4 \times 10^6 = 0.87 \times 415 \times A_{st} \times 350 \left(1 - \frac{A_{st} \cdot 415}{200 \times 350 \times 20}\right)$$

$$= 126.367 \times 10^3 A_{st} \left(1 - 2.964 \times 10^{-4} A_{st}\right)$$

$$= 126.367 \times 10^3 A_{st} - 37.458\, A_{st}^2$$

$$37.458\, A_{st}^2 - 126.367 \times 10^3 A_{st} + 64.4 \times 10^6 = 0$$

$$\boxed{A_{st\,req} = 625.66\ mm^2}$$

$$\text{No of Bars} = \frac{A_{st\,req}}{a_{st}}$$

$$= \frac{625.66}{\frac{\pi}{4} \times (20)^2}$$

$$= 1.99 \simeq 2\ No's$$

$$\boxed{NOB_{prov} = 2}$$

$$A_{st\,prov} = n \times \frac{\pi}{4} (d)^2$$

$$= 2 \times \frac{\pi}{4} (20)^2$$

$$\boxed{A_{st\,prov} = 628\ mm^2}$$

Provide 2 bars of 20mm ϕ ($A_{st} = 628\ mm^2$)

- as Tension Reinforcement and 2 bars of 10mm ϕ as Hanger bars on compression side

Step: 5

Check for Shear Stress

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

$(\times 1000)$

48.2 kN → 48.2×10^3 N

$$= \frac{48.2 \times 10^3}{200 \times 350}$$

$$\boxed{\tau_v = 0.688\ N/mm^2}$$

Percentage of Tension Reinforcement

$$P_t = \frac{100 A_{st}}{bd}$$

$$= \frac{100 \times 628}{200 \times 350}$$

$$\boxed{P_t = 0.89\ \%}$$

Read Out (Design Shear Strength of Concrete)

P_t	τ_c
0.75 →	0.56
0.89 →	?
1.00 →	0.62

$$\frac{(1.00 - 0.75)}{(1.00 - 0.89)} = \frac{(0.62 - 0.56)}{(0.62 - x)}$$

$$\boxed{\tau_c = 0.59\ N/mm^2}$$

$\tau_v > \tau_c$

Since $\tau_v > \tau_c$ shear reinforcement are required

Balanced shear $(V_{us}) = [V_u - (\tau_c \cdot b \cdot d)]$ Pg.No:73

V_{us} - The strength of shear reinforcement V_{us} shall be calculate

$$= 48.2 - (0.59 \times 200 \times 350)\, 10^3$$

$$\boxed{V_{us} = 7\ KN}$$

Using 6mm ϕ 2 legged Mild steel stirrups

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

$$= \frac{0.87 \times 250 \times (2 \times \frac{\pi}{4} \times 6^2) \times 350}{7 \times 1000}$$

$$\boxed{S_v = 609\ mm}$$

But

$S_v \not> 0.75d$

$= 0.75 \times 350$

$= 262.5\ mm$

Adopt a spacing of 250mm

Provide 6mm ϕ 2 legged stirrups at a spacing of 250mm c/c

Step: 6

Check for Deflection Control

$P_t = 0.89$

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{Basic} \times K_t \times K_c \times K_f$$

$$\left(\frac{L}{d}\right)_{Basic} = 20$$

$$f_s = 0.58 \times f_y \times \frac{Ast\ req}{Ast\ prov}$$

$$= 0.58 \times 415 \times \frac{625.66}{628}$$

$$\boxed{f_s = 240}$$

$$\boxed{K_t = 1}$$

$$\left(\frac{L}{d}\right)_{Max} = 20 \times 1 \times 1 \times 1$$

$$= 20$$

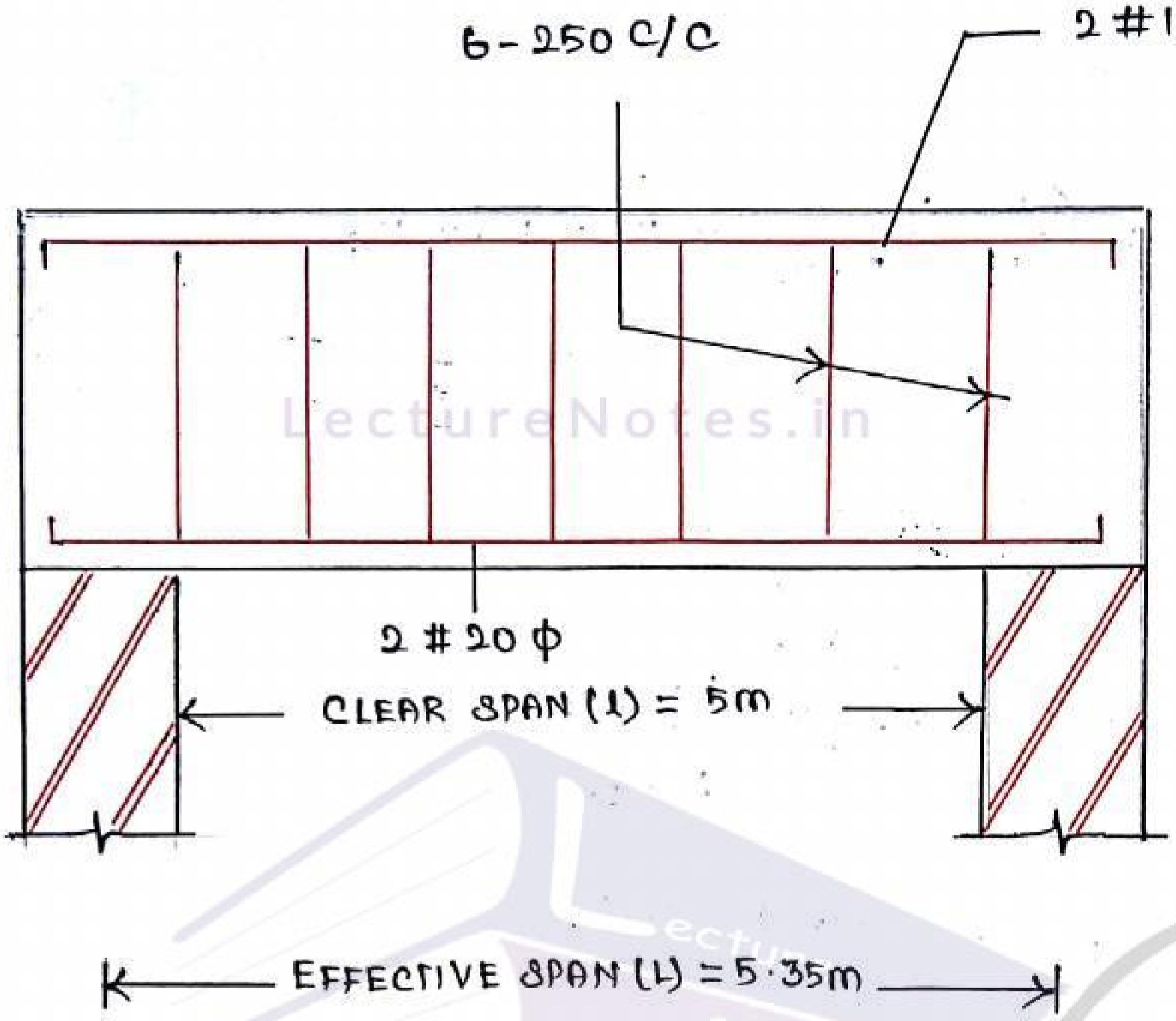
$$\left(\frac{L}{d}\right)_{prov} = \frac{5.35}{0.35}$$

$$= 15.33$$

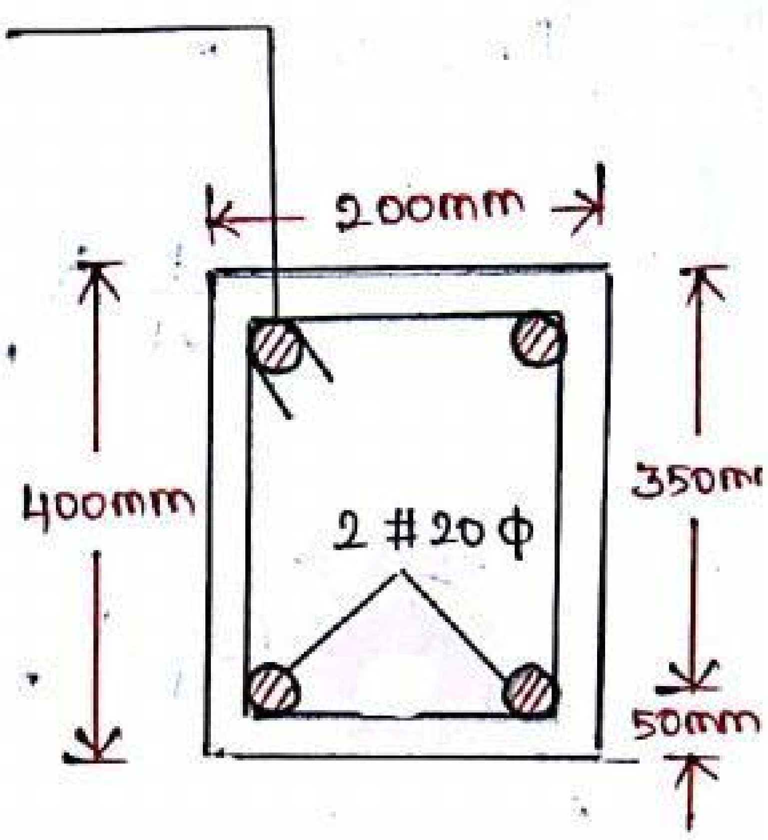
$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{provided}$$

Hence deflection control is Satisfactory

Reinforcement Detail in Singly Reinforced Rectangular beam



Longitudinal Section



Cross section

DOUBLY REINFORCED BEAM

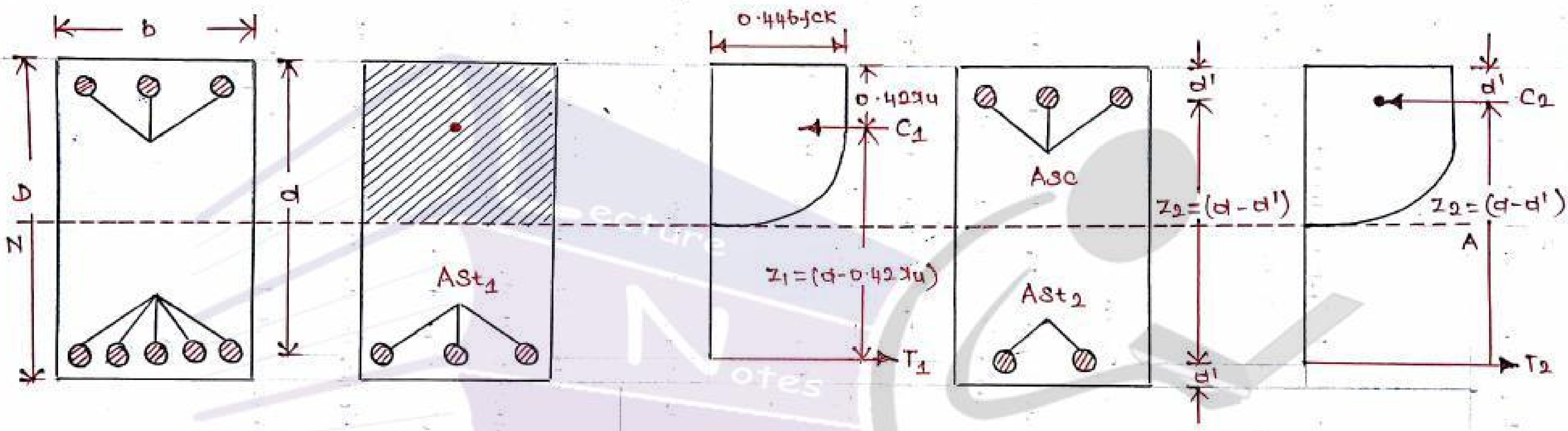
DEFINITION

A beam in which steel Reinforcement is provided at both the Tension and Compression zone is called Doubly Reinforced beam

Purpose of Doubly Reinforced Beam

* Doubly Reinforced beams are provided in order to increase the moment carrying capacity of the section
* Most important reason for providing the doubly reinforced beam is to ensure safety against reversal of stresses in the structure due to wind forces, seismic forces and Temperature stresses
* The depth of the beams may be restricted for architectural and Functional requirements
* Doubly reinforced beams are designed if such beams of restricted to resist moment more that its Mu lim
* When the member is subjected to Eccentric loads
* When the member is continuous over several supports
* When the member is subjected to shock (or) Impact Load

Analysis of Doubly Reinforced Section



SECTION

SECTION RESISTING MOMENT Mu lim | STRESS BLOCK Mu lim

LIMITING SINGLY REINFORCED SECTION

SECTION RESISTING EXTRA MOMENT EXCESS BM | STRESS BLOCK FOR EXCESS BM

ADDITIONAL (OR) EXCESS RESISTING MOMENT

Where,

b - Breadth

d - Effective depth

D - Overall depth

d' - Effective cover

A_{sc} - Area of compression reinforcement

A_{st} - Area of tensile reinforcement.

A_{st_1} - Area of steel for a Single Reinforced beam (Balanced Section)

A_{st_2} - Area of additional tension reinforcement

C_1 - Compressive force

T_1 - Tensile force

C_2 - Additional compressive force

T_2 - Additional Tensile force

M_{u1} - Maximum moment capacity of the singly Reinforced beam

M_{u2} - Additional moment of Resistance $(M_u - M_{ulim})$

M_{ulim} - Limiting moment of Resistance

f_{sc} - Design stress in Compression Steel

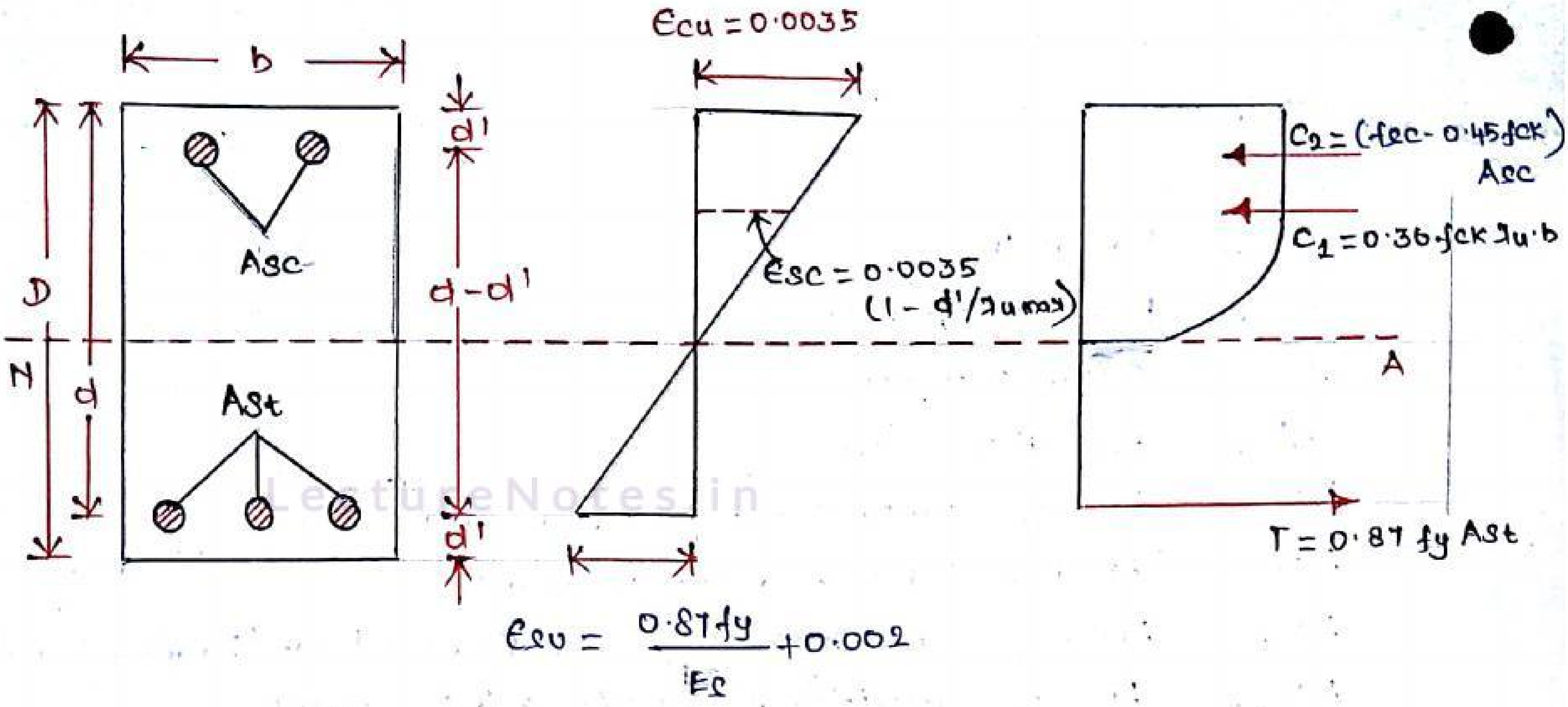
f_{ck} - characteristic strength of concrete

C - Total Compression

T - Total Tensile

ϵ_{su} - Ultimate strain in steel

ϵ_{cu} - Ultimate strain in Concrete



* In a Doubly reinforced beam the compressive force consists of
  - (i) compressive force in concrete
  - (ii) compressive force in compression steel

* compressive force in concrete 'C_1'

$$\boxed{C_1 = 0.36 f_{ck} \cdot x_u \cdot b}$$

* compressive force in compression steel 'C_2'

$$\boxed{C_2 = A_{sc}(f_{sc} - 0.45 f_{ck})}$$

Where,

Asc - Area of compression Reinforcement

fsc - Design stress in compression Reinforcement

The value of stress in compression Reinforcement (f_{sc}) depends upon the ratio $\left(\frac{d'}{d}\right)$ and the Grade of steel.

Design Stress in Compression Reinforcement (f_{sc})

S.NO	f_y N/mm^2	$\left(\frac{d'}{d}\right)$			
		0.05	0.10	0.15	0.20
1	250	217	217	217	217
2	415	355	353	342	329
3	500	424	412	395	370

Design Equation

1. Moment of Resistance of the doubly Reinforced beam

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

Where

$$M_{u1} = M_{u\,lim} \text{ (Maximum moment capacity)}$$

Fe 250 $\Rightarrow$ $M_{u\,lim} = 0.149\, f_{ck}\, bd^2$

Fe 415 $\Rightarrow$ $M_{u\,lim} = 0.138\, f_{ck}\, bd^2$

Fe 500 $\Rightarrow$ $M_{u\,lim} = 0.133\, f_{ck}\, bd^2$

$$M_{u2} = f_{sc} \cdot A_{sc}\,(d - d')$$

(a) $x_u < x_{u\,max}$

$$\boxed{M_u = 0.36\, f_{ck}\, b\, x_u\,(d - 0.42x_u) + f_{sc} \cdot A_{sc}\,(d - d')}$$

(b) $x_u > x_{u\,max}$

→ The Given section can be designed as Over reinforced Section

→ In LSM Over Reinforced section are not possible

Then Redesign the section

→ It can be designed as balanced section

Depth of Neutral axis ($x_u = x_{u\,max}$)

$$\boxed{M_u = 0.36\, f_{ck}\, b\, x_{u\,max}\,(d - 0.42\, x_{u\,max}) + f_{sc} \cdot A_{sc}\,(d - d')}$$

2. Area of Steel Reinforcement (Corresponding to $M_{u\,lim}$)

(i) A_{st} : can be calculated by

$$T \cdot F = C \cdot F$$

$$0.87\, f_y\, A_{st_1} = 0.36 \cdot f_{ck} \cdot b \cdot x_{u\,lim}$$

$$\boxed{A_{st_1} = \frac{0.36\, f_{ck} \cdot b \cdot x_{u\,lim}}{0.87\, f_y}}$$

(ii) Compression Reinforcement (A_{sc})

$$M_u = M_{u\,lim} + M_{u2} \Rightarrow M_{u2} = M_u - M_{u\,lim}$$

$$M_{u2} = f_{sc} \cdot A_{sc}\,(d - d')$$

$$\boxed{A_{sc} = \frac{M_{u2}}{f_{sc}\,(d - d')}}$$

(iii) Additional steel (A_{st_2})

$$T_2 = C_2$$

$$0.87\, f_y\, A_{st_2} = f_{sc} \cdot A_{sc}$$

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

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

- **Stress in compression steel**

$$* \; f_{sc} = 0.0035 \times \left(1 - \frac{d'}{x_{u\,max}}\right) \times E_s$$

$$* \; f_{sc} \not> 0.87 f_y$$

Depth of Neutral axis

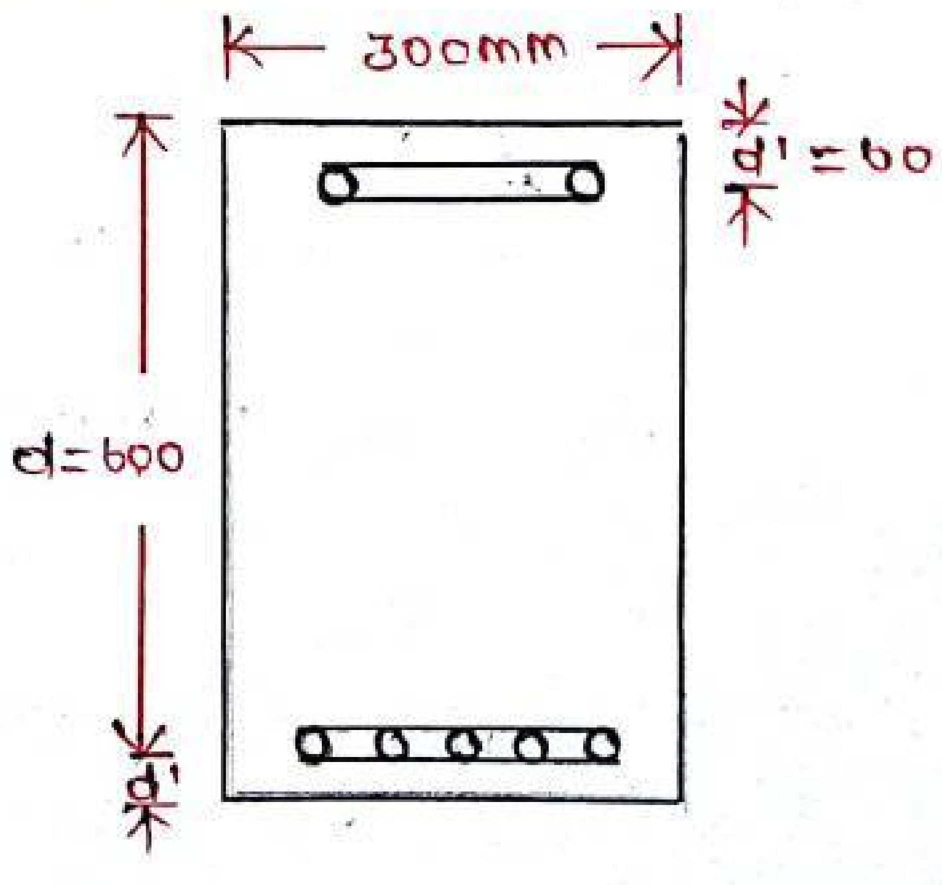
$$x_u = \frac{0.87 \, f_y \cdot A_{st1}}{0.36 \, f_{ck} \cdot b}$$

Problem on Analysis of Doubly Reinforced beam

Prblm. No: 8

Determine the Ultimate Moment of Resistance of a doubly reinforced beam of rectangular section having a width of 300mm and reinforced with 5 bars of 25mm diameter at an effective depth of 600 mm. The compression steel is made up of 2 bars of 25mm diameter at an effective cover of 60mm. Adopt M20 Grade concrete and Fe 415 HYSD bars.

Given data



$b = 300mm$

$d = 600mm$

$d' = 60 \, mm$

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

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

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

$= 5 \times \frac{\pi}{4} 25^2$

$= 2454.36 mm^2$

$A_{sc} = n \times \frac{\pi}{4} d^2$

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

$= 981.73 mm^2$

Solution

Maximum depth of Neutral axis

$x_{umax} = 0.48\, d$ (Fe 415)

$= 0.48 \times 600$

$\boxed{x_{umax} = 288 mm}$

Stress in Compression Reinforcement

$f_{sc} = 0.0035 \times \left(1 - \frac{d'}{x_{umax}}\right) \times E_s$

$= 0.0035 \times \left(1 - \frac{60}{288}\right) \times (2 \times 10^5)$

$= 554.16 \ N/mm^2$

But

$f_{sc} \not> 0.87 f_y$

$= 0.87 \times 415$

$f_{sc} = 361 \ N/mm^2$

Area of Compression Reinforcement

$$Ast_2 = \frac{fsc \cdot Asc}{0.87 \cdot fy}$$

$$= \frac{301 \times 981.73}{0.87 \times 415}$$

$$\boxed{Ast_2 = 981.59\ mm^2}$$

$$Ast = Ast_1 + Ast_2$$

$$Ast_1 = Ast - Ast_2$$

$$= 2454.36 - 981.59$$

$$\boxed{Ast_1 = 1472.77\ mm^2}$$

Depth of Neutral axis

$$x_u = \frac{0.87 \cdot fy \cdot Ast_1}{0.36 \cdot fck \cdot b}$$

$$= \frac{0.87 \times 415 \times 1472.77}{0.36 \times 20 \times 300}$$

$$\boxed{x_u = 246.17\ mm}$$

$$x_u < x_{umax}$$

Hence the section is Under Reinforced section

$$M_u = 0.87 \cdot f_y \cdot A_{st1}(d - 0.42x_u) + f_{sc} A_{sc}(d - d')$$

$$= 0.87 \times 415 \times 1472.77(600 - 0.42 \times 246.17) + 361 \times 981.73 (600 - 60)$$

$$= 264.06 \times 10^6 + 191.37 \times 10^6$$

$$= 455.43 \times 10^6 \ N \cdot mm$$

$$\boxed{M_u = 455.43 \ KN \cdot m}$$

Prblm. no: 9

A Rectangular reinforced concrete beam of width 400 mm and Effective depth 600 mm is to be designed to support an ultimate Moment of 600 KN.m. Using M20 Grade concrete and Fe 415 HYSD bars design suitable reinforcements in the beam at an effective cover of 60 mm.

Given data

$b = 400mm$

$d = 600mm$

$d' = 60\ mm$

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

$f_y = 415\ N/mm^2$

$M_u = 600\ KN \cdot m$

To Find

$A_{st} = ?$

$A_{sc} = ?$

Solution

-

Limiting Moment of Resistance

$$M_{u\,lim} = 0.138\, f_{ck}\, b d^2$$

$$= 0.138 \times 20 \times 400 \times 600^2$$

$$= 397.44 \times 10^6 \text{ N.mm}$$

$$\boxed{M_{u\,lim} = 397.44 \text{ KN.m}}$$

Reinforcement

$$A_{st_1} = \frac{0.36\, f_{ck} \cdot b \cdot x_{u\,max}}{0.87 \cdot f_y}$$

$$= \frac{0.36 \times 20 \times 400 \times (0.48 \times 600)}{0.87 \times 415}$$

$$\boxed{A_{st_1} = 2297.29 \text{ mm}^2}$$

$$A_{sc} = \frac{M_{u2}}{f_{sc}(d - d')}$$

$$M_u = M_{u\,lim} + M_{u2}$$

$$M_{u2} = M_u - M_{u\,lim}$$

$$= 600 - 397.44$$

$$\boxed{M_{u2} = 202.56 \text{ KN.m}}$$

$$f_{sc} = 0.0035 \times \left(1 - \frac{d'}{x_{umax}}\right) \times E_s$$

$$= 0.0035 \times \left(1 - \frac{60}{0.48 \times 600}\right) \times (2 \times 10^5)$$

$$f_{sc} = 540 \text{ N/mm}^2$$

But

$$f_{sc} \ngtr 0.87 f_y$$

$$= 0.87 \times 415$$

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

$$A_{sc} = \frac{202.56 \times 10^6}{361 \times (600 - 60)}$$

$$\boxed{A_{sc} = 1039 \text{ mm}^2}$$

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

$$= \frac{1039 \times 361}{0.87 \times 415}$$

$$\boxed{A_{st_2} = 1038.85 \text{ mm}^2}$$

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

$$= 2297.29 + 1038.85$$

$$\boxed{A_{st} = 3336.14 \text{ mm}^2}$$

$$\boxed{A_{sc} = 1039 \text{ mm}^2}$$

Design Procedure of Doubly Reinforced Beam

Step: 1

Cross Sectional dimension

The depth of beam is Fixed based on $\frac{\text{Span}}{\text{depth ratio}}$ to satisfy the deflection requirements

$$\text{Effective depth (d)} = \frac{\text{Span}}{\text{Depth}}$$

Span / Depth Ratios for Trial Section

S.NO	SPAN RANGE	LOADING	SPAN / DEPTH RATIO (L/d)
1	3 to 4m	Light	15 to 20
2	5 to 10m	Medium to Heavy	12 to 15
3	> 10m	Heavy	12

Step: 2

Loads Calculation

Self weight = Area of section x Unit wt. of Rcc

Live Load = ______ KN/m

Total load (W) = ______ KN/m

Design ultimate Load } (Wu) = PSF x W

Unit wt. of Rcc = 25 $\frac{KN}{m^2}$

Partial safety factor } = 1.5

Step: 3

Ultimate Moment and Shear Force

Ultimate Moment

$$M_u = 0.125\ W_u L^2$$

Shear Force

$$V_u = 0.5\ W_u L^2$$

Step: 4

Limiting Moment of Resistance

$$M_{u\,lim} = 0.149\ f_{ck}\ b d^2 \quad (Fe\ 250)$$

$$M_{u\,lim} = 0.138\ f_{ck}\ b d^2 \quad (Fe\ 415)$$

$$M_{u\,lim} = 0.133\ f_{ck}\ b d^2 \quad (Fe\ 500)$$

Compare

$M_u = M_{u\,lim} \rightarrow$ Balanced Section

$M_u < M_{u\,lim} \rightarrow$ Under Reinforced Section

$M_u > M_{u\,lim} \rightarrow$ Over Reinforced Section

Limiting Moment of Resistance ($M_{u\,lim}$) to check the sections

Step: 5

Calculation of Reinforcement details

$$A_{st1} = \frac{0.36 f_{ck} \cdot b \cdot x_{u\,max}}{0.87 \cdot f_y}$$

$$A_{st2} = \frac{A_{sc} \cdot f_{sc}}{0.87 f_y}$$

Where,

$$A_{sc} = \frac{M_u - M_{ulim}}{f_{sc}(d-d')}$$

(i) $f_{sc} = \left[0.0035 \frac{(x_{umax} - d')}{x_{umax}} \right] E_s$

(ii) $f_{sc} \ngtr 0.87 f_y$

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

$(NOB)_{prov} = \frac{A_{st\,req}}{a_{st}}$

$A_{st\,prov} = \_\_\_\_\_\_ \ mm^2$

Step: 6

Check for Shear Reinforcement

Nominal Shear Stress

Pg. No: 72
Cl. No: 40.1

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

percentage of Tension Reinforcement

Pg. No: 73
Table. No: 19
Read out Design Shear Strength of Concrete

$$\boxed{P_t = \frac{100 A_{st}}{bd}}$$

Compare

$\tau_v < \tau_c$

Step: 7

Check for Deflection Control

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{basic} \times K_t \times K_c \times K_f$$

Refer IS : 456-2000
Pg. No : 38

$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{provide}$$

Prblm. No: 10

- Design a Reinforced Concrete beam of Rectangular Section Using the Following data

Given data

Effective Span $(L) = 8\,m$

Working Live Load $= 30\,KN/m$

M20 Grade Concrete and Fe 415 HYSD bars

Assuming, Width $(b) = 300\,mm$

Effective cover $(d') = 50\,mm$

Effective depth $(d) = 600\,mm$

Overall depth $(D) = 650\,mm$

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

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

Solution

Step: 1

Loads.

(Unit Wt. of RCC)

Self Weight $(g) = \underset{(b)}{0.3} \times \underset{(D)}{0.65} \times 25$

$= 4.875\,KN/m$

$(\div 1000)$ 300 mm → 0.3 m

Live Load $(q) = 30.000\,KN/m$

Total Working Load $(W) = 34.875\,KN/m$

$(\div 1000)$ 650 mm → 0.65 m

Ultimate design load $(W_u) = PSF \times W$

$= 1.5 \times 34.875$

$\boxed{W_u = 52.31 \text{ KN/m}}$

Step: 2

Ultimate Moment and Shear Force

Ultimate Moment

$M_u = 0.125 \times W_u \times L^2$

$= 0.125 \times 52.31 \times 8^2$

$\boxed{M_u = 418.48 \text{ KN·m}}$

(OR)

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

$= \frac{52.31 \times 8^2}{8}$

$\boxed{M_u = 418.4 \text{ KN·m}}$

Shear Force

$V_u = 0.5 \times W_u \times L$

$= 0.5 \times 52.31 \times 8$

$\boxed{V_u = 209.24 \text{ KN}}$

(OR)

$V_u = \frac{WL}{2}$

$= \frac{52.3 \times 8}{2}$

$\boxed{V_u = 209.2 \text{ KN}}$

Step: 3

Limiting Moment of Resistance

$M_{u\,lim} = 0.138 f_{ck}\, b\, d^2$ (Fe415)

$= 0.138 \times 20 \times 300 \times 600^2$

$= 298.08 \times 10^6$

$\boxed{M_{u\,lim} = 298 \text{ KN·m}}$

$M_{u\,lim} < M_u$

Hence it is over Reinforced section

Hence the design as Doubly Reinforced section

Step:4

Main Reinforcement

$$A_{sc} = \frac{M_{u2}}{f_{sc}(d-d')}$$

$M_u = M_{u\,lim} + M_{u2}$

$M_{u2} = M_u - M_{u\,lim}$

$M_{u2} = M_u - M_{u\,lim}$

$= 418.48 - 298.08$

$$\boxed{M_{u2} = 120.4\ KN.m}$$

$$f_{sc} = \left[0.0035\ \frac{x_{umax} - d'}{x_{umax}}\right] E_s$$

$$= \left[0.0035\left(1 - \frac{50}{0.48 \times 600}\right)\right] \times (2 \times 10^5)$$

$$= [0.0035 - 6076 \times 10^{-4}] \times (2 \times 10^5)$$

$$\boxed{f_{sc} = 578.47\ N/mm^2}$$

But

$f_{sc} \not> 0.87 f_y \not> 0.87 \times 415$

$$\boxed{f_{sc} = 361\ N/mm^2}$$

$$Asc = \frac{[Mu - Mulim]}{fsc(d-d')}$$

$$= \frac{120.4 \times 10^6}{361 \times (600-50)}$$

$$\boxed{Asc_{req} = 606.39 mm^2}$$

Provide = ?

$$NOB_{prov} = \frac{Asc_{req}}{asc}$$

$$= \frac{606.39}{\frac{\pi}{4} \times (20)^2}$$

$$= 1.93 \simeq 2 Nos$$

$$Asc_{prov} = n \times \frac{\pi}{4}(d)^2$$

$$= 2 \times \frac{\pi}{4}(20)^2$$

$$\boxed{Asc_{prov} = 628.31 mm^2}$$

Provide 2 bars of 20mm ϕ $(Asc = 628.31 mm^2)$

$$Ast_2 = \frac{Asc \cdot fsc}{0.87 fy}$$

$$= \frac{628.31 \times 361}{0.87 \times 415}$$

$$\boxed{Ast_2 = 628.22 mm^2}$$

$$Ast_1 = \frac{0.36 fck \cdot b (x_{umax})}{0.87 fy}$$

$$= \frac{0.36 \times 20 \times 300 \times (0.48 \times 600)}{0.87 \times 415}$$

$$\boxed{Ast_1 = 1722.97 \, mm^2}$$

$$Ast = Ast_1 + Ast_2$$

$$= 1722.97 + 628.22$$

$$\boxed{Ast = 2351.19 \, mm^2}$$ (Required Ast)

$$NoB_{prov} = \frac{Ast_{req}}{ast}$$

$$= \frac{2351.19}{\frac{\pi}{4} \times 25^2}$$

$$= 4.78 \simeq 5 \text{ No's}$$

$$Ast_{prov} = n \times \frac{\pi}{4} d^2$$

$$= 5 \times \frac{\pi}{4} (25)^2$$

$$= 2454.26 mm^2$$ (provide Ast)

Provide 5 bars of 25mm ϕ ($\because$ Ast = $2454.26 \, mm^2$)

Step: 5

Shear Reinforcement

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

$$= \frac{209.24 \times 10^3}{300 \times 600}$$

$$= 1.162 \ N/mm^2$$

$$P_t = \frac{100 \ A_{st}}{bd}$$

$$= \frac{100 \times 2454.36}{300 \times 600}$$

$$= 1.36$$

Read out (Design Shear Strength of Concrete)

P_t		τ_c
1.25	$\rightarrow$	0.67
1.36	$\rightarrow$	?
1.50	$\rightarrow$	0.72

$$\frac{(1.50 - 1.25)}{(1.50 - 1.36)} = \frac{(0.72 - 0.67)}{(0.72 - x)}$$

$$\boxed{\tau_c = 0.69 \ N/mm^2}$$

$$\tau_v > \tau_c$$

Since $\tau_v > \tau_c$ Shear reinforcement are required

$$V_{us} = \left[V_u - (\tau_c \cdot b \cdot d)\right]$$

$$= \left[209.24 - (0.69 \times 300 \times 600) \times 10^{-3}\right]$$

$$\boxed{V_{us} = 85.04 \ KN}$$

Using 8mm ϕ 2 legged stirrups

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

$$= \frac{0.87 \times 415 \times (2 \times \frac{\pi}{4} \times 8^2) \times 600}{82.04 \times 10^3}$$

$$S_v = 262.25 mm$$

$$S_v \ngtr 0.75D = 0.75 \times 600$$

$$= 450 \ngtr 300 mm$$

$$S_v = 260 mm$$

Adopt spacing $S_v = 260mm$ Near the supports and gradually increasing to 300mm towards the centre of span

Step:6

Check for Deflection Control

$P_t = 1.36$

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{basic} \times K_t \times K_c \times K_f$$

$$P_c = \frac{100 A_{sc}}{bd} \Rightarrow \frac{100 \times 628.21}{300 \times 600} = 0.34$$

$$\left(\frac{L}{d}\right)_{basic} = 20$$

$$f_s = 0.58 \times f_y \times \frac{A_{st\,req}}{A_{st\,prov}}$$

$$f_s = 0.58 \times 415 \times \frac{2351.19}{2454.26}$$

$$= 220$$

$$\left(\frac{L}{d}\right)_{Max} = 20 \times 0.90 \times 1.1 \times 1$$

$$= 19.8$$

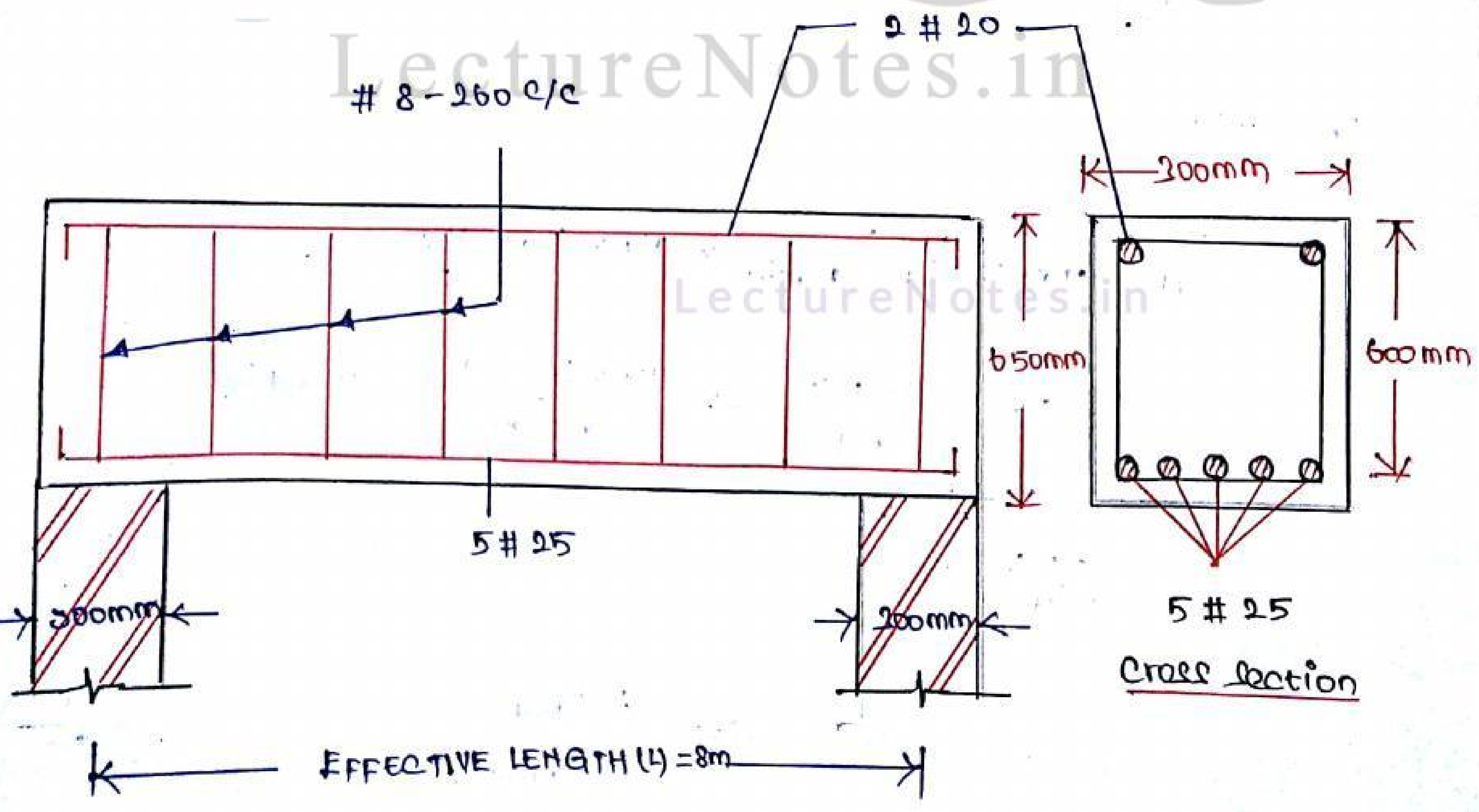
$$\left(\frac{L}{d}\right)_{prov} = \frac{8000}{600}$$

$$= 13.33$$

$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{provided}$$

Hence the Deflection control is satisfactory

Reinforcement details in Doubly Reinforced beam



Cross section

Longitudinal section

Prblm. No: 11

- Design a doubly Reinforced Rectangular simply supported at its both ends to carry a service Live load of 25 KN/m and Super Imposed dead load of 16 kN/m over a clear span of 7m. Use M25 and Fe 415 are used

Given data.

Clear span = 7m

Live Load = 25 kN/m

Dead Load = 16 KN/m

f_{ck} = 25 N/mm^2

f_y = 415 N/mm^2

Step: 1

Cross Sectional dimensions

Effective depth (d) = $\frac{7000}{12} \Rightarrow 583.33$ mm $\simeq$ 600mm

Assume

$d' = 50$mm

$D = 650$mm

The General Width of used are 150mm, 200mm, 230mm, 250mm, and 300mm

Assume, b = 230mm

Effective span (L)

Effective span (L) = Clear span + Effective depth

= 7 + 0.6 $\Rightarrow$ 7.6m

$600\text{mm} \xrightarrow{(\div 1000)} 0.6\text{m}$

Effective span (L) = clear span + c/c distance b/w the support

$(\div 1000)$

$220\,mm \rightarrow 0.22\,m$

$= 7 + 0.22$

$= 7.22\,m$

Adopt

$\boxed{L = 7.22\,m}$

Step: 2

Loads calculation

Dead Load $= 16\ KN/m$

Live Load $= 25\ KN/m$

Total Load (W) $= 41\ KN/m$

Factored load $W_u = PSF \times W$

$= 1.5 \times 41$

$\boxed{W_u = 61.5\ KN/m}$

Step: 3

Ultimate Moment (M_u) & Shear Force (V_u)

Ultimate Moment

$$M_u = \frac{W_u L^2}{8} = \frac{61.5 \times 7.22^2}{8} = 401.84\ KN \cdot m.$$

Shear Force

$$V_u = \frac{W_u L}{2} = \frac{61.5 \times 7.23}{2} = 222.32\ KN$$

Step:4

- Limiting Moment of Resistance

$$M_{u\,lim} = 0.138\, f_{ck}\, b\, d^2 \quad (Fe\,415)$$

$$= 0.138 \times 20 \times 230 \times 600^2$$

$$= 228.58 \times 10^6 \text{ N·mm}$$

$$= 228.528 \text{ KN·m}$$

$$M_u > M_{u\,lim}$$

Hence it is a over reinforced section.

So hence design as Doubly reinforced section

Step:5

Calculation of Reinforcement details

$$A_{st_1} = \frac{0.36 \cdot f_{ck} \cdot b \cdot x_{u\,max}}{0.87 \cdot f_y} \quad \therefore (x_{u\,max} = 0.48\,d)$$

$$= \frac{0.36 \times 20 \times 230 \times (0.48 \times 600)}{0.87 \times 415}$$

$$\boxed{A_{st_1} = 1320.94\,mm^2}$$

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

(i) $f_{sc} = 0.0035 \times \left[1 - \frac{d'}{x_{u\,max}}\right] \times E_s$

$$= 0.0035 \times \left[1 - \frac{50}{0.48 \times 600}\right] \times (2 \times 10^5)$$

$= 5789 \ N/mm^2$

(ii) $f_{sc} \ngtr 0.87 f_y \Rightarrow 0.87 \times f_y$

$= 0.87 \times 415$

$f_{sc} = 361.05 \ N/mm^2$

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

$M_{u2} = M_u - M_{u\,lim}$

$= 401.84 - 228.528$

$= 173.31 \ kN \cdot m$

$$= \frac{173.31 \times 10^6}{361.05\,(600-50)}$$

$A_{sc} = 872.75 \ mm^2$

Using 20mm diameter bars

$$(\text{No. of bars})_{prov} = \frac{A_{sc}}{a_{sc}}$$

$$= \frac{872.75}{\frac{\pi}{4} \times 20^2}$$

$= 2.77 \simeq 3$ No's

Provide 3 No's of 20mm dia bars

$A_{st\,prov} = n \times \frac{\pi}{4} d^2$

$= 3 \times \frac{\pi}{4} (20)^2$

$= 942.47 \ mm^2$

$$A_{st2} = \frac{A_{sc} \cdot f_{sc}}{0.87 \cdot f_y}$$

$$= \frac{942.47 \times 361.05}{0.87 \times 415}$$

$$Ast_2 = 942.47\,mm^2$$

$$Ast = Ast_1 + Ast_2$$

$$= 1320.94 + 942.47$$

$$\boxed{Ast_{req} = 2263.41\,mm^2}$$

Using 25mm diameter bars

$$(\text{No. of Bars})_{prov} = \frac{Ast_{req}}{a_{st}}$$

$$= \frac{2262.41}{\frac{\pi}{4} \times (25)^2}$$

$$= 4.61 \simeq 5 \text{ No's}$$

Provide 5 No's of 25mm dia bars

$$Ast_{prov} = n \times \frac{\pi}{4} (d)^2$$

$$= 5 \times \frac{\pi}{4} \times (25)^2$$

$$\boxed{Ast_{prov} = 2454.36\,mm^2}$$

Step: 6

Shear Reinforcement

$$\tau_v = \frac{V_u}{bd} = \frac{222.32 \times 10^3}{230 \times 600} = 1.61\,N/mm^2$$

$$P_t = \frac{100\,Ast}{bd} = \frac{100 \times 2454.36}{230 \times 600} = 1.77$$

Refer Table-19 in Pg: No: 73

P_t	τ_c
1.75 →	0.75
1.77 →	?
2.00 →	0.79

$$\frac{(2.00-1.75)}{(2.00-1.77)} = \frac{(0.79-0.75)}{(0.79-x)}$$

$$\boxed{\tau_c = 0.74\ N/mm^2}$$

$\tau_v > \tau_c$

Since $\tau_v > \tau_c$ shear reinforcement are required

$$V_{us} = V_u - (\tau_c \cdot b \cdot d)$$

$$= (222.22 \times 10^3) - (0.74 \times 250 \times 600)$$

$$\boxed{V_{us} = 120.2 \times 10^3\ N}$$

Using 8mm diameter 2 legged stirrups

(i) $$S_v = \frac{0.87 \times f_y \times A_{sv} \cdot d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times \frac{\pi}{4} \times 8^2) \times 600}{120.2 \times 10^3}$$

$= 181.18$ mm c/c

(ii) $S_v \not> 0.75d$

$= 0.75 \times 600$

$= 450$mm

(iii) $S_v = 300$ mm c/c

Provide 2 legged 8mm dia stirrups @ 200mm c/c

Step: 7

Check for Deflection Control

$P_t = 1.77$

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{basic} \times K_t \times K_c \times K_f$$

$$P_c = \frac{100 \times A_{sc}}{bd} = \frac{100 \times 942.47}{230 \times 600} = 0.68$$

$$\left(\frac{L}{d}\right)_{basic} = 20$$

$$f_s = 0.58 \times f_y \times \frac{A_{st\,req}}{A_{st\,prov}}$$

$$= 0.58 \times 415 \times \frac{2263.41}{2454.26}$$

$$= 221.94 \simeq 240$$

$$f_s = 240$$

$$\left(\frac{L}{d}\right)_{Max} = 20 \times 0.9 \times 1.18 \times 1$$

$$= 21.24$$

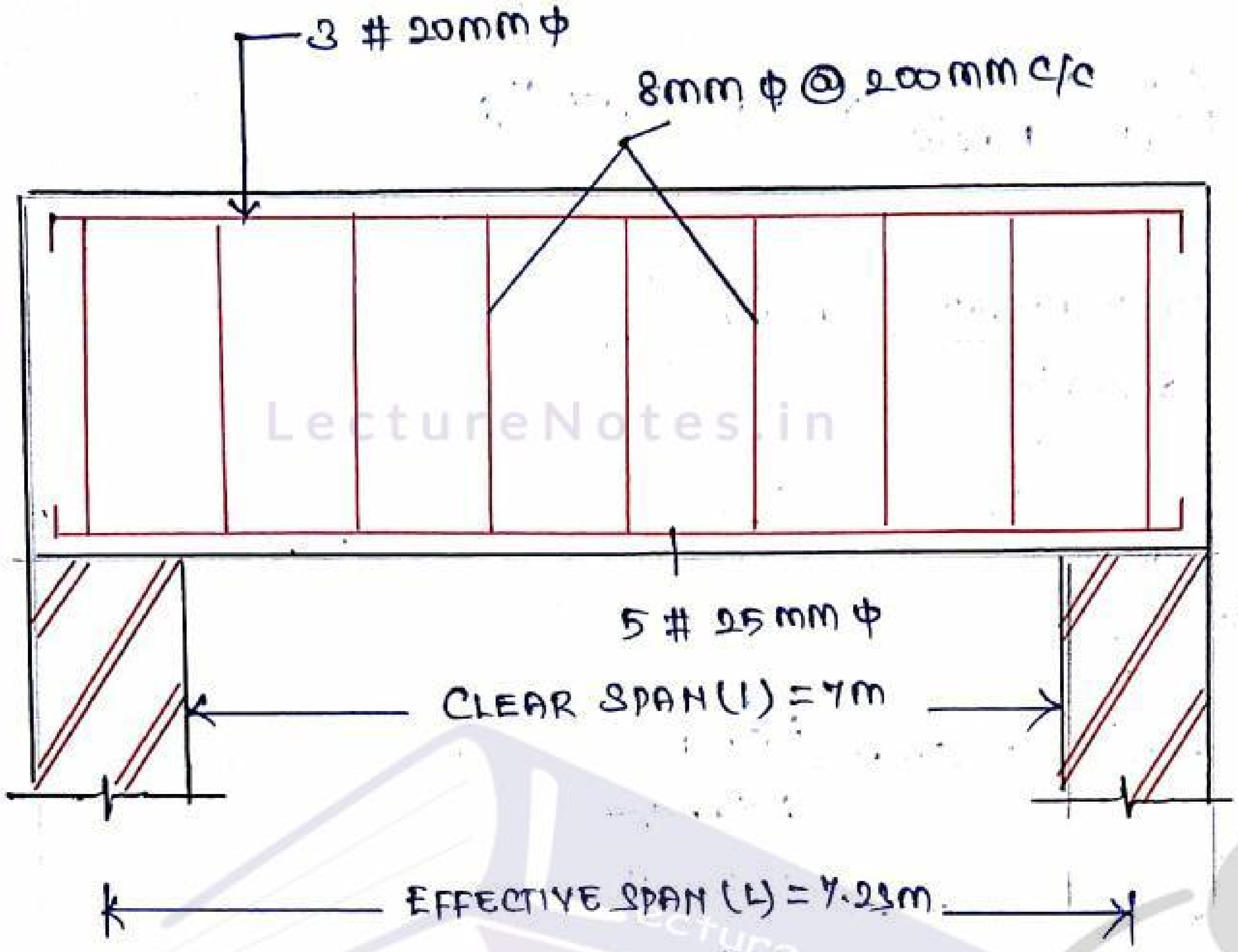
$$\left(\frac{L}{d}\right)_{prov} = \frac{7.23}{0.6}$$

$$= 12.05$$

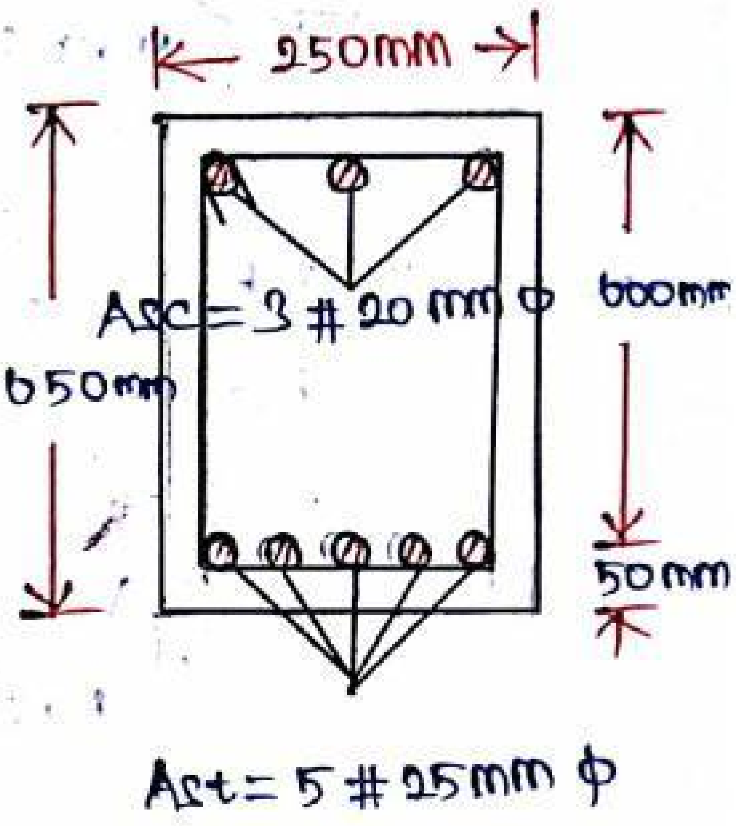
$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{provided}$$

Hence the deflection control is satisfactory

Reinforcement Details in Doubly Reinforced beams



Longitudinal Section



Cross Section

T- BEAM & L-BEAM SECTION

DEFINITION

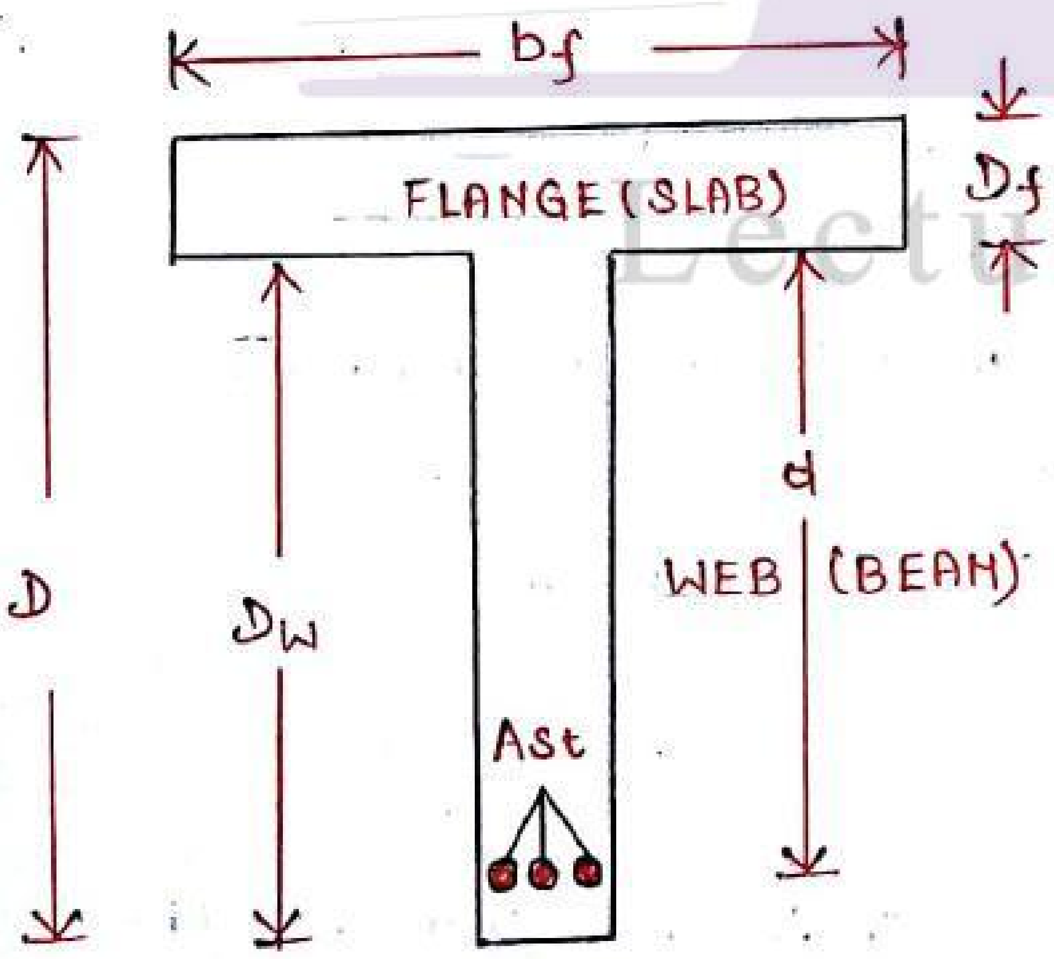
* When part of slab acts along with beam to take load then it is called as T- Beam

* T Beam & L Beam is an RCC beams which is monolithically constructed with RCC slab

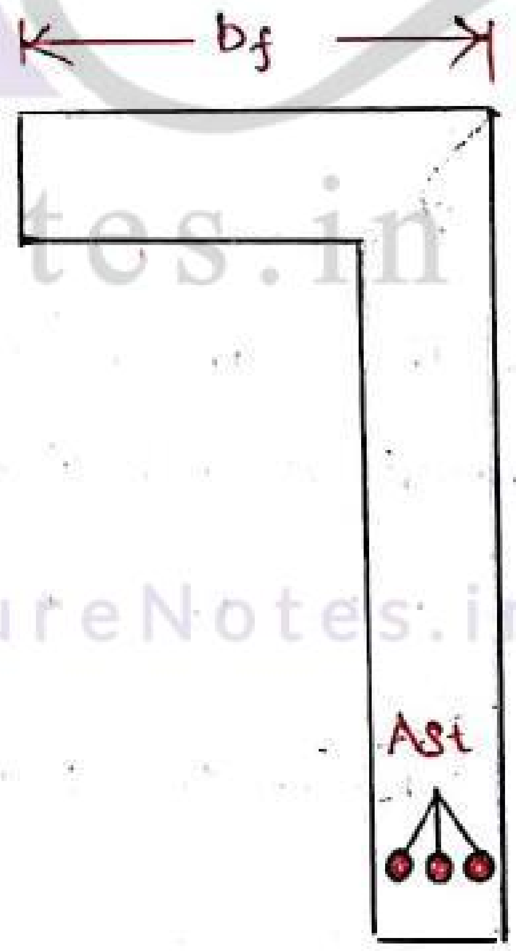
* T Beam & L Beam just same as simple rectangular beams which are casted monolithic with slab (which means the slab and beam is casted at same time)

* Due to which the portion of the slab acts as the part of beam

* T- Beam is the most economical RCC beam, which is widely used in domestic building construction



CROSS SECTION OF T-BEAM



CROSS SECTION OF L-BEAM

Where,

b_w - Breadth of Web

D_w - Depth of Web

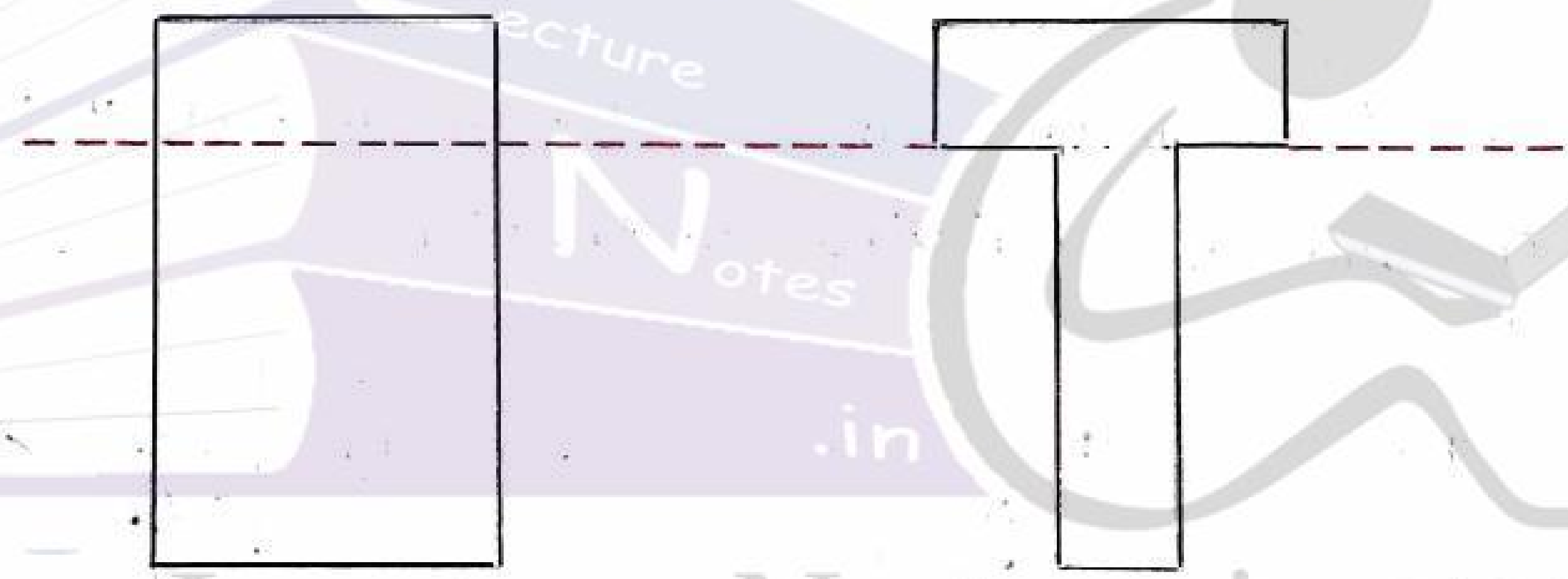
b_f - Breadth of Flange (or) Flange Width

D_f - Depth of Flange

d - Effective depth

D - Overall depth $(D_w + D_f)$

Example



* A portion of the slab acts integrally with the beam and bend in the longitudinal direction of the beam

* Slab portion is called the flange of T beam or L beam

* Beam portion below the flange (Web)

* The web is the full rectangular portion of the beam (other than the overhanging parts of the flange)

* Both the cross section will have same length

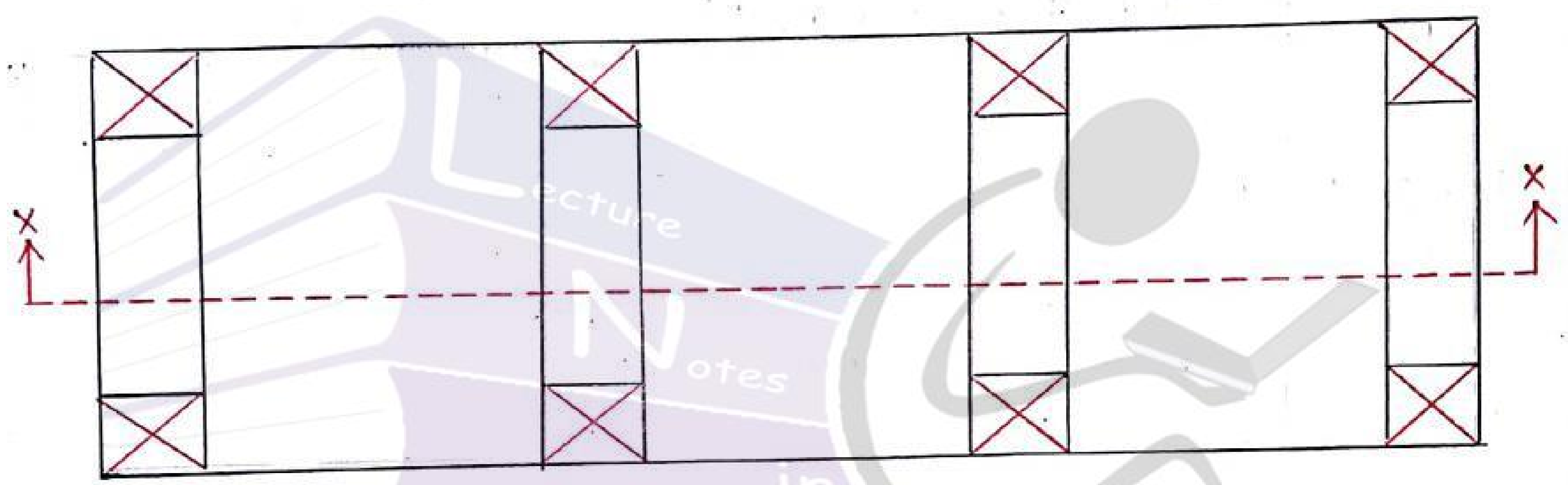
* So to save concrete T-beams

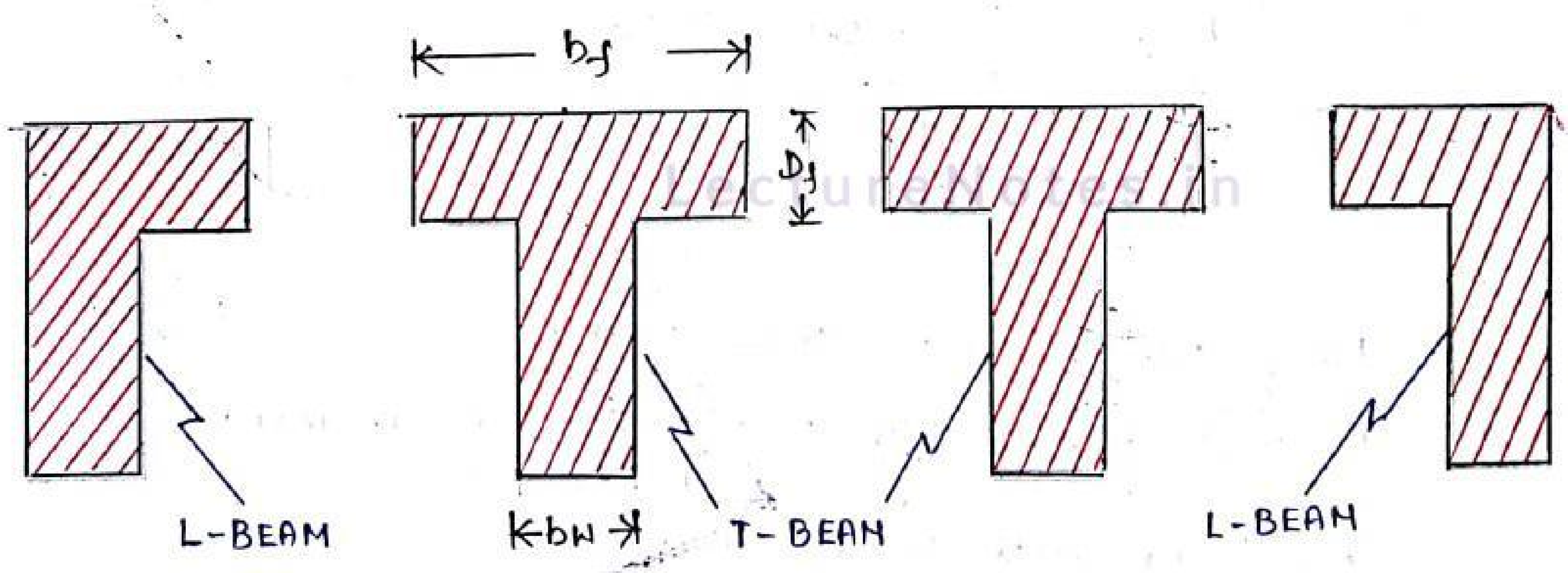
* But it is not possible to do this with precast members (bridges) as reinforcement also should be connected monolithically

DESIGN OF FLANGED BEAM

* When a Reinforced concrete slab is monolithically with the beam as in the case of beam supported floor slab system, the beam can be considered as flanged beams with slab acting as an effective flange on the compression side

* It is important to note that continuous T or L beams acts as flanges beam only between the supports where the bending moment are Negative (sagging) and the slab are on the compression side of the beam





INTERMEDIATE BEAM — S.F, B.M

END BEAM — S.F, B.M, TORSION

Advantage of Flanged beams

* When compare to the rectangular beam the Flange beam (or) resist (or) width stand more load and having more moment carrying capacity.

* In flanged beam, the slabs and beam are normally tied together by means of stirrups and bent up bars if any and then are cast form one mass of concrete

Important Formulae

Refer IS:456-2000
Pg. No: 37
Cl. 23.1.2

Effective Width of Flange (b_f)

For T- Beam

$$b_f = \left[\frac{L_o}{6} + b_w + 6D_f\right]$$

For L- Beam

$$b_f = \left[\frac{L_o}{12} + b_w + 3D_f\right]$$

Where,

b_f = Effective width of flange

L_o = Distance b/w points of zero moments

b_w = breadth of the web

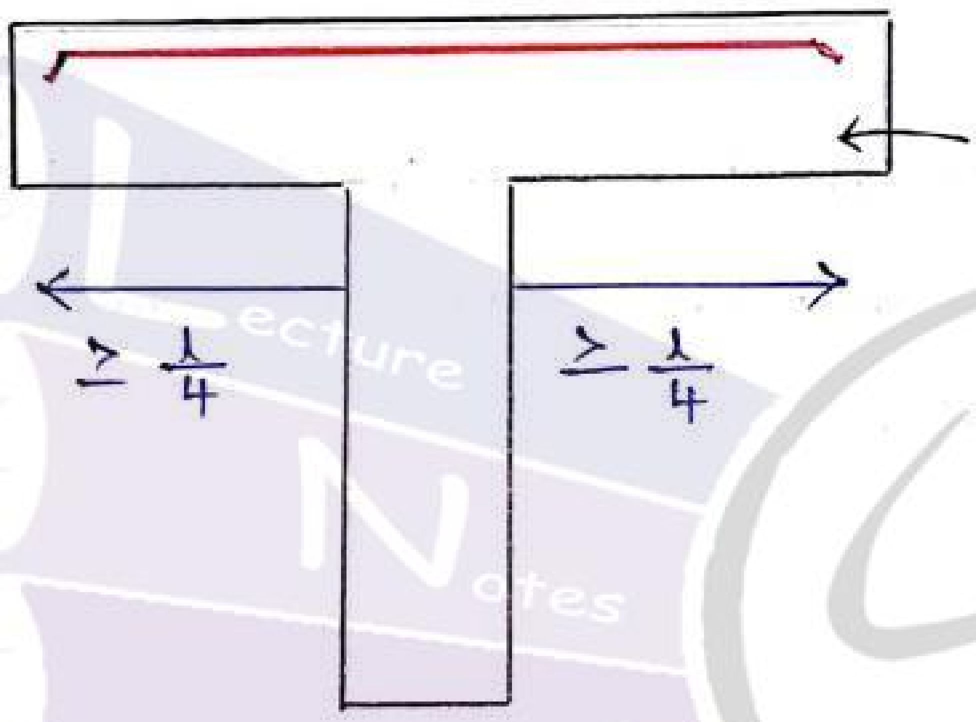
D_f = Thickness of flange

b = Actual width of flange

Note:

- According to IS 456:2000, the following condition must be satisfied

If the main reinforcement of the slab is parallel to the beam the transverse reinforcement must be provided for a distance more than (or) equal to $\frac{1}{4}$ th of the span of the slab and the transverse reinforcement shall not be less than 60% of the main reinforcement at the mid span of the slab



Transverse reinforcement in slab $\geq \frac{60}{100} \times A_{st(main)}$

Analysis of T-Beam

When T-Beams the NA lies with three different places they are.

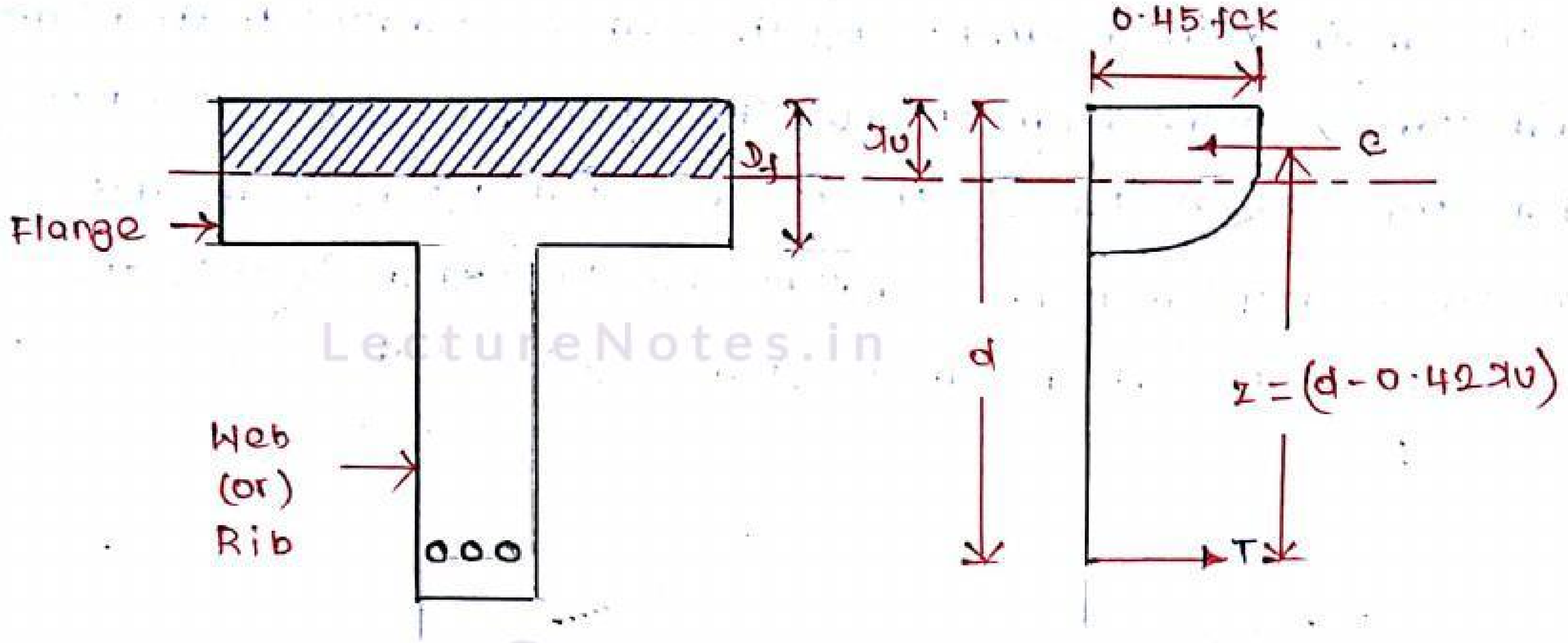
(i) Neutral axis lies within the flange

(ii) Neutral axis in the web and thickness of flange is less than (or) equal to 0.42 times depth of NA ($D_f \leq 0.42\, x_u$)

(iii) Neutral axis lies in the web and thickness of flange is greater than 0.42 times depth of NA ($D_f \geq 0.42\, x_u$)

Case - I

Neutral axis lies in the Flange ($x_u \le D_f$)



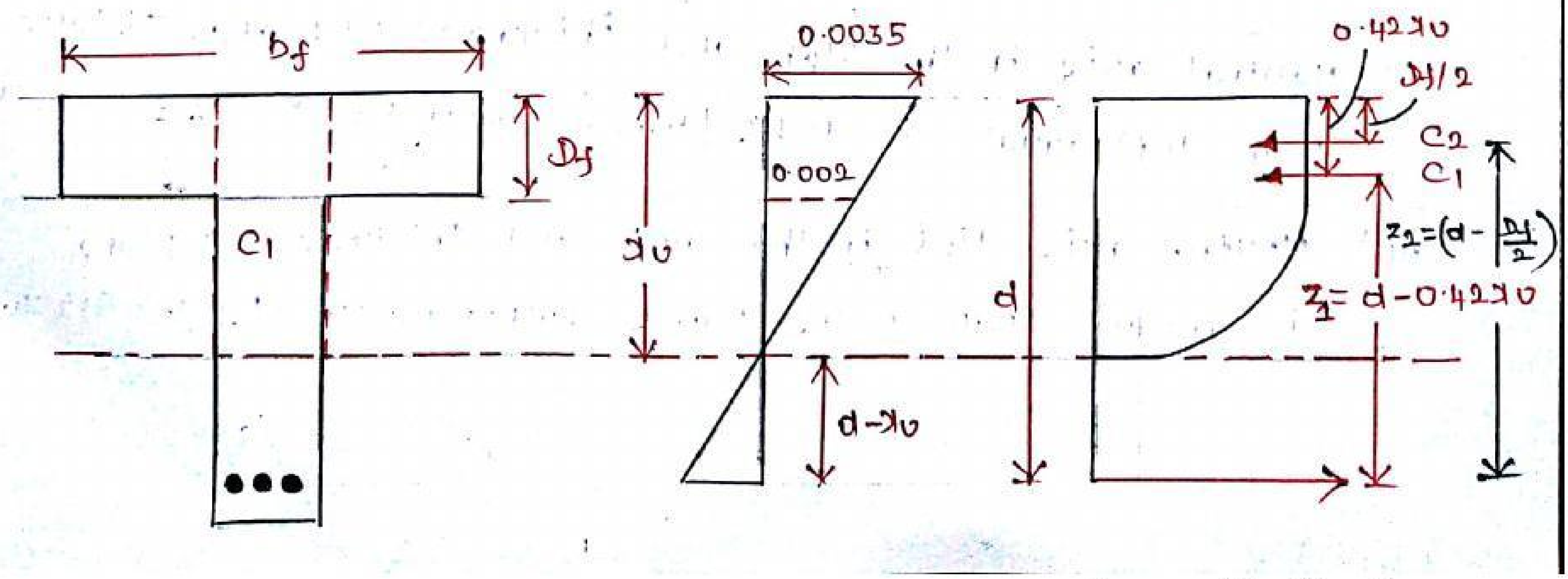
When NA lies within the Flange, the moment of resistance of the section can be calculated by the same procedure as that of Rectangular section

Depth of NA

$$\boxed{x_u = \frac{0·87 · f_y · A_{st}}{0·36 · f_{ck} · b_f}}$$

Case - II

Neutral axis lies Outside the Flange ($x_u > D_f$)



In this case, there are Two possibilities

$$\frac{D_f}{d} < 0.2 \quad (\text{or}) \quad (D_f < 0.43 x_u)$$

(i) At a section of $(b_w x_u)$ the C.F is

$$C_1 = 0.36 f_{ck} b_w \cdot x_u$$

(ii) At a section of $(b_f - b_w) D_f$ the C.F is

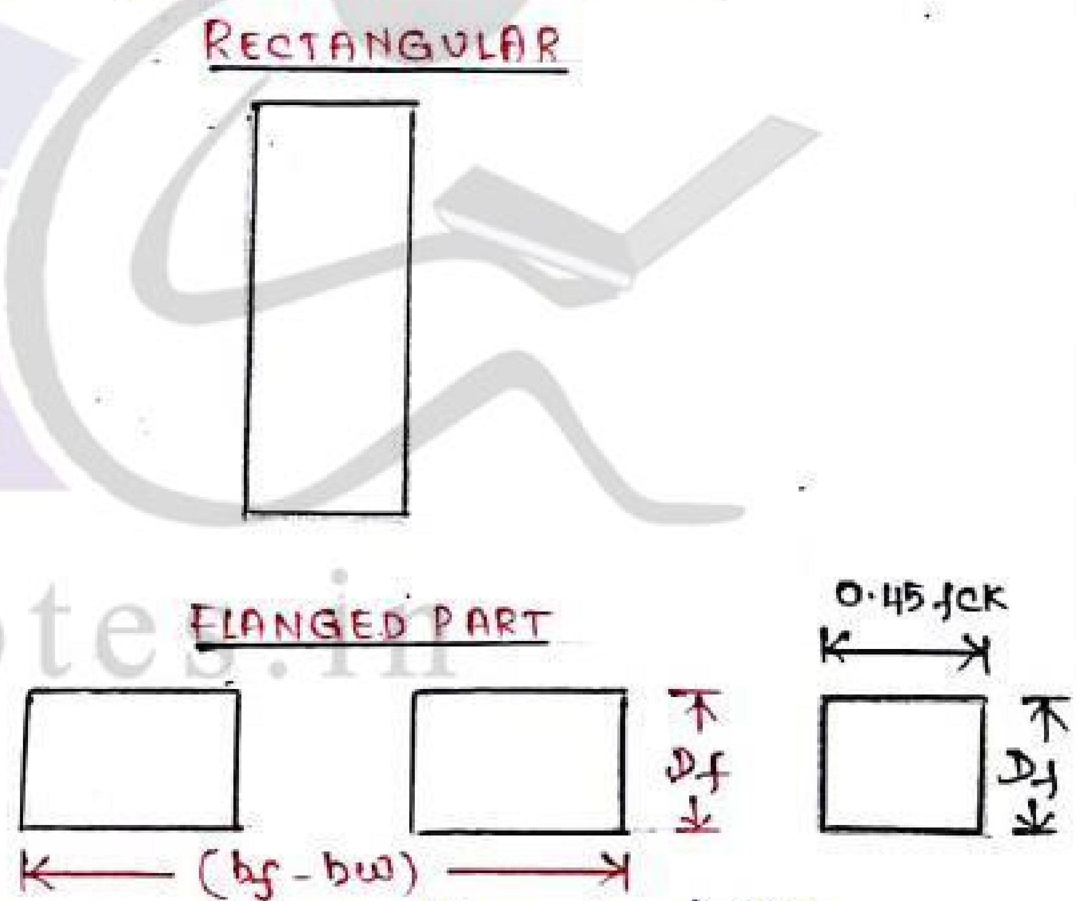
$$C_2 = 0.45 f_{ck} (b_f - b_w) D_f$$

Total compressive force $C = C_1 + C_2$

$$C = 0.36 \cdot f_{ck} \cdot b_w \cdot x_u + 0.45 f_{ck} (b_f - b_w) D_f$$

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

(1) Depth of NA



Equate the T.F & C.F

$$T = C$$

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

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

(2) Moment of Resistance (Mu)

Lever arm

$$z_1 = d - 0.42 x_u$$

$$z_2 = d - \frac{D_f}{2}$$

$$M_u = C_1 z_1 + C_2 z_2$$

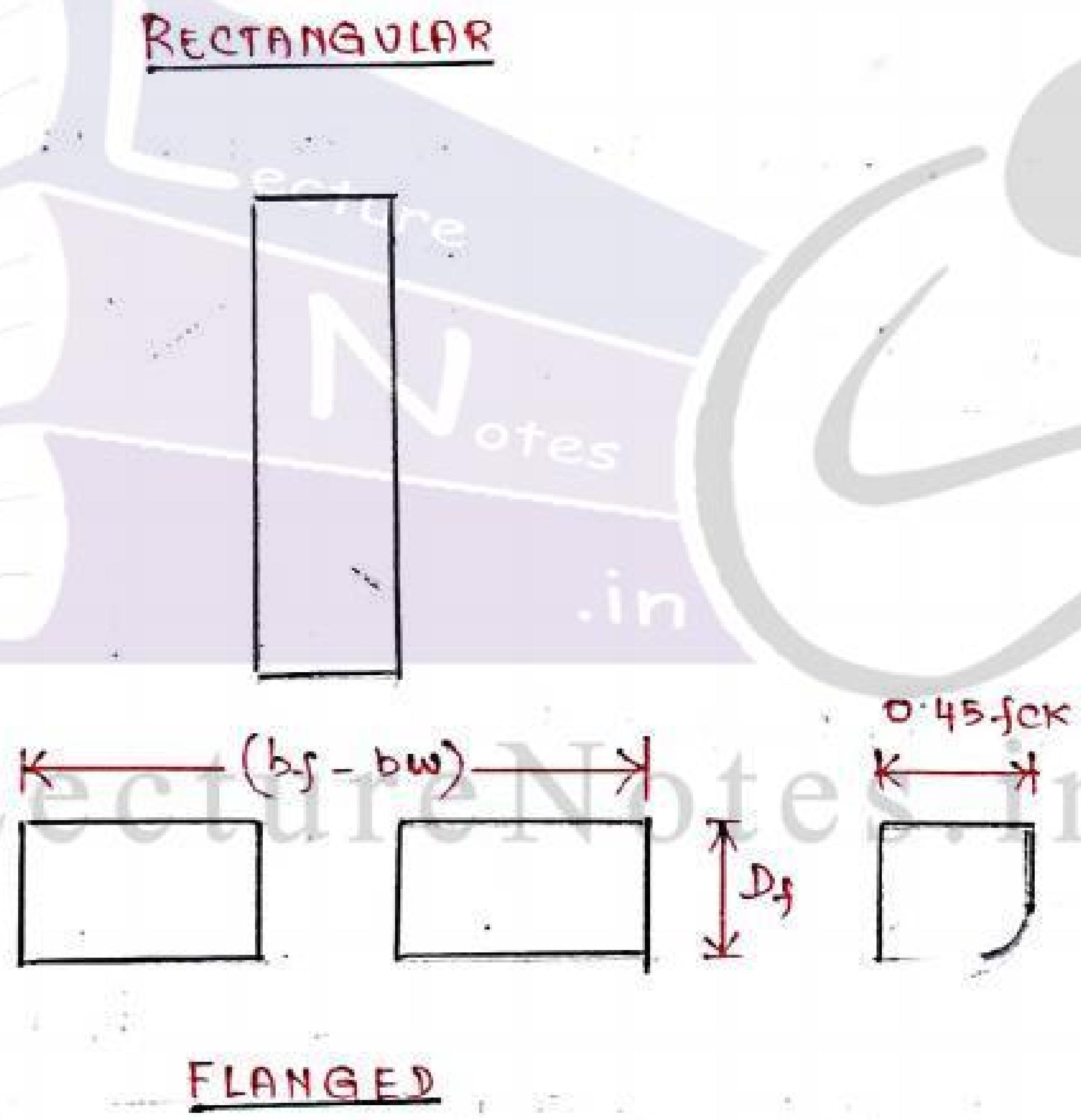
$$M_u = 0.36 f_{ck} b_w \cdot x_u (d - 0.42 x_u) + 0.45 f_{ck} (b_f - b_w) D_f \cdot \left(d - \frac{D_f}{2}\right)$$

$$M_u = 0.36\, f_{ck} \cdot \frac{x_u}{d}\left(1 - 0.42\frac{x_u}{d}\right) b_w \cdot d^2 + 0.45\, f_{ck}(b_f - b_w) D_f \cdot \left(d - \frac{D_f}{2}\right)$$

$M_{u\,lim} \rightarrow$ put x_u replace $x_{u\,max}$

(b) $D_f > 0.43\, x_u$ (or) $\frac{D_f}{d} > 0.2$

In this case because of $D_f > 0.43 x_u$, the compressive stress is various regarding the IS code says $\frac{D_f}{d} > 0.2$ the flange area is required in addition of equivalent thickness (y_f).



Where,

y_f = Equivalent depth of flange on which a uniform stress of $0.45 f_{ck}$ will be supposed to be acting.

$$y_f = 0.15 x_u + 0.65 D_f$$

Scanned by CamScanner

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

Limiting moment of Resistance

$$M_{ulim} = 0.36 f_{ck} \cdot \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right) b_w d^2 + 0.45 f_{ck} (b - b_w) y_f \left(d - \frac{y_f}{2}\right)$$

Overall depth

- For simply supported beam

  Depth ratio may be assumed as $\frac{1}{12}$ to $\frac{1}{15}$

  Continuous beam

  $\frac{1}{15}$ to $\frac{1}{20}$ of span

Moment of Resistance

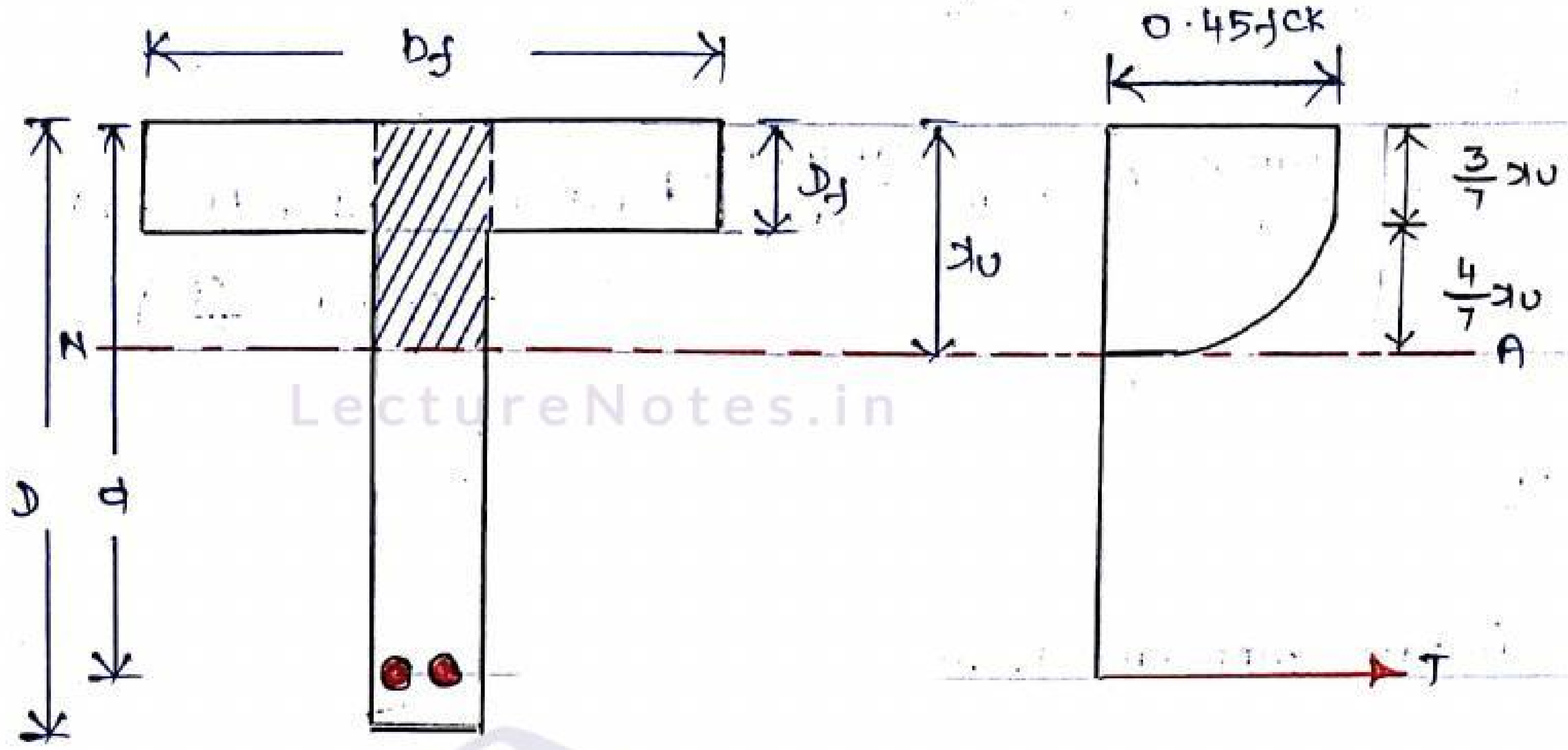
$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot d \left[1 - \frac{A_{st} \cdot f_y}{b_f \cdot d \cdot f_{ck}}\right]$$

(or)

$$M_u = 0.36 f_{ck} b_f x_u \cdot (d - 0.42 x_u)$$

Case-III

Neutral axis lies Outside the Flange ($x_u > D_f$) :



In this case there are Two possibilities

(a) $D_f < \frac{3}{7}x_u$ (or) $\frac{D_f}{d} \leq 0.2$

This case are generally when the ratio $\frac{D_f}{d} \leq 0.2$. In this case the stress in the flange is uniform

$$\left.\begin{matrix} M_{ulim} \\ (or) \\ M_u \end{matrix}\right\} = 0.36\frac{x_u}{d}\left[1 - 0.42(x_u/d)\right] f_{ck}\, b_w\, d^2$$

(or)

$$0.36 f_{ck}\, b_w\, x_{umax}(d - 0.42\, x_{umax}) + 0.45 f_{ck}(b_f - b_w) D_f\left(d - \frac{D_f}{2}\right)$$

(b) $D_f > \frac{3}{7}x_u$ (or) $\frac{D_f}{d} > 0.2$

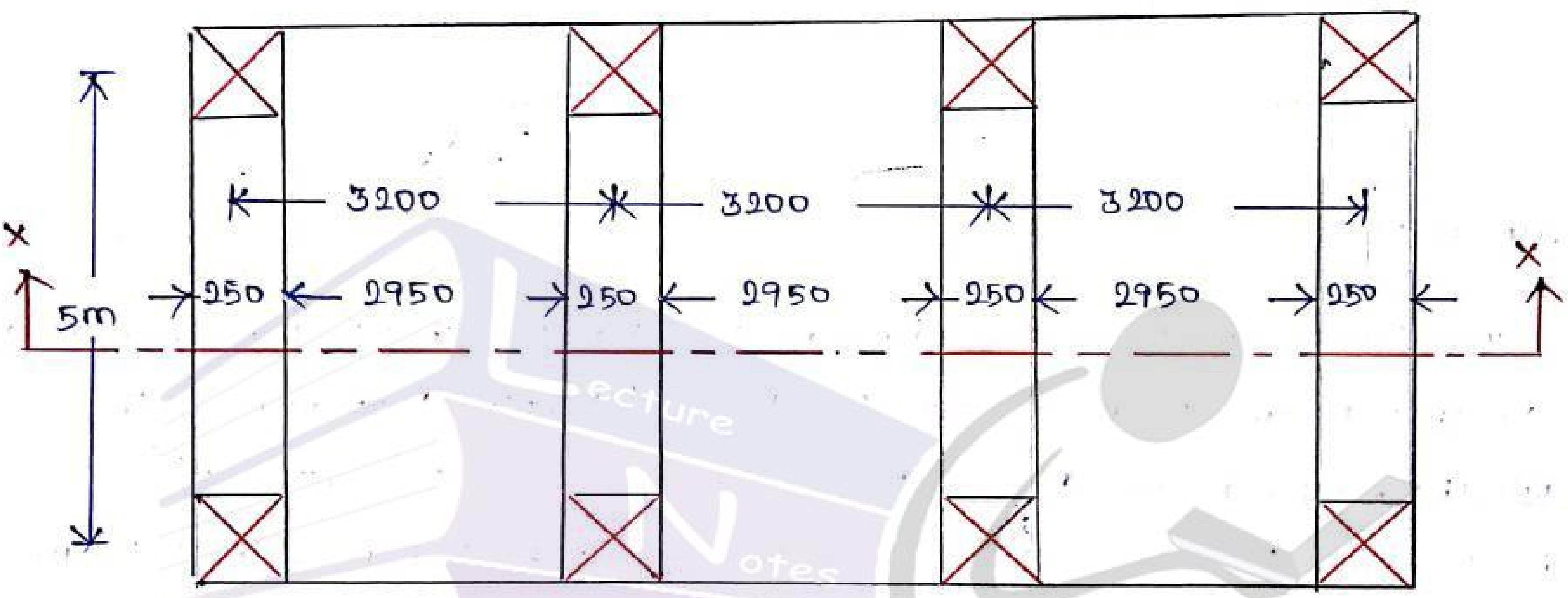
Replace D_f by y_f in the above equation

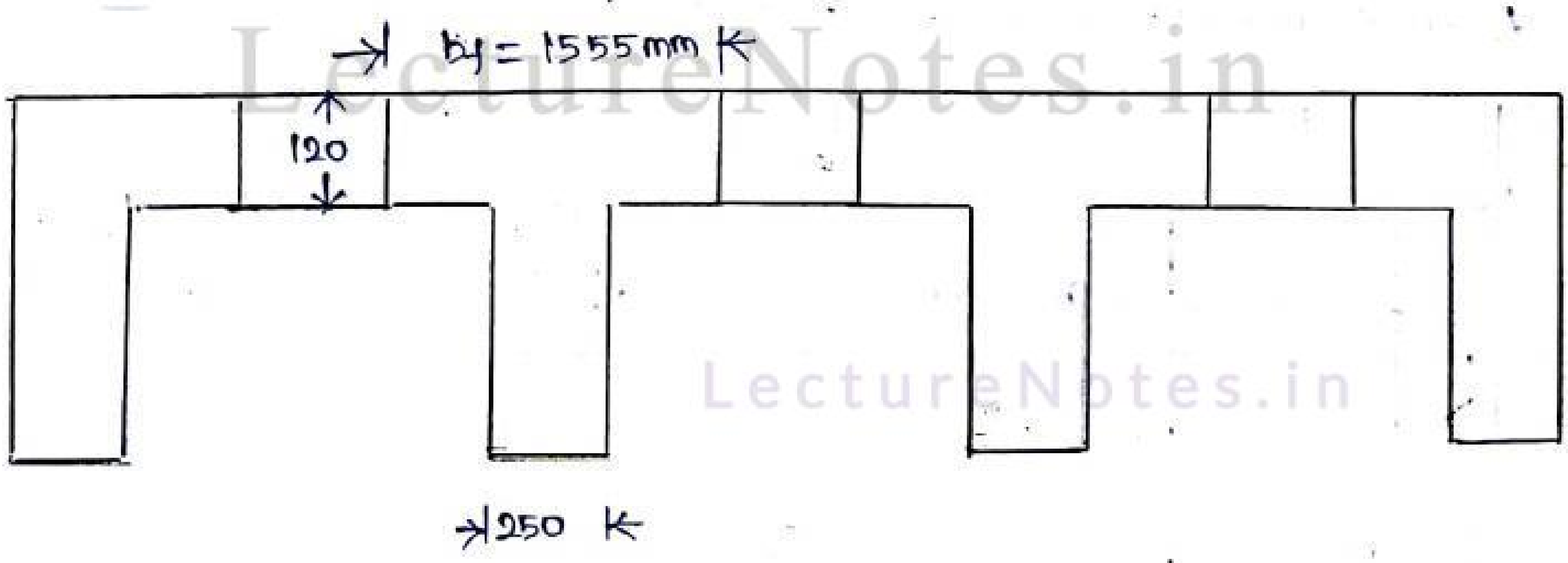
$$M_u = 0.36 f_{ck}\, b_w\, x_{umax}(d - 0.42\, x_{umax}) + (0.45 f_{ck})(b_f - b_w) y_f \left(d - \frac{y_f}{2}\right)$$

In this case the stress in the flange will not be Uniform

Prblm. no:12

- A continuous T beam is supported by RC columns at 5m centres. The centre to centre distance of adjacent panels of slab is 3200 mm. The thickness of slab and breadth of web 120mm and 250mm respectively. Determine the effective width of Flange at the T-Beam





Given data

Effective length (L) = 5000mm

l_o = Distance b/w points of zero moments

$l_o = 0.7 \times 5000$

$= 3500$ mm

To Find

$b_f = ?$

$\Rightarrow 250 + \left(\frac{2950}{\cancel{2}}\right) \times \cancel{2}$

$\Rightarrow 3200\,mm$

Solution

$$b_f = \frac{l_o}{6} + b_w + 6D_f \quad \text{(For T- Beam)}$$

$$= \frac{3500}{6} + 250 + (6 \times 120)$$

$$= 1553.33\,mm \simeq 1555\,mm$$

$1555mm < 3200mm$

Problm. No: 12

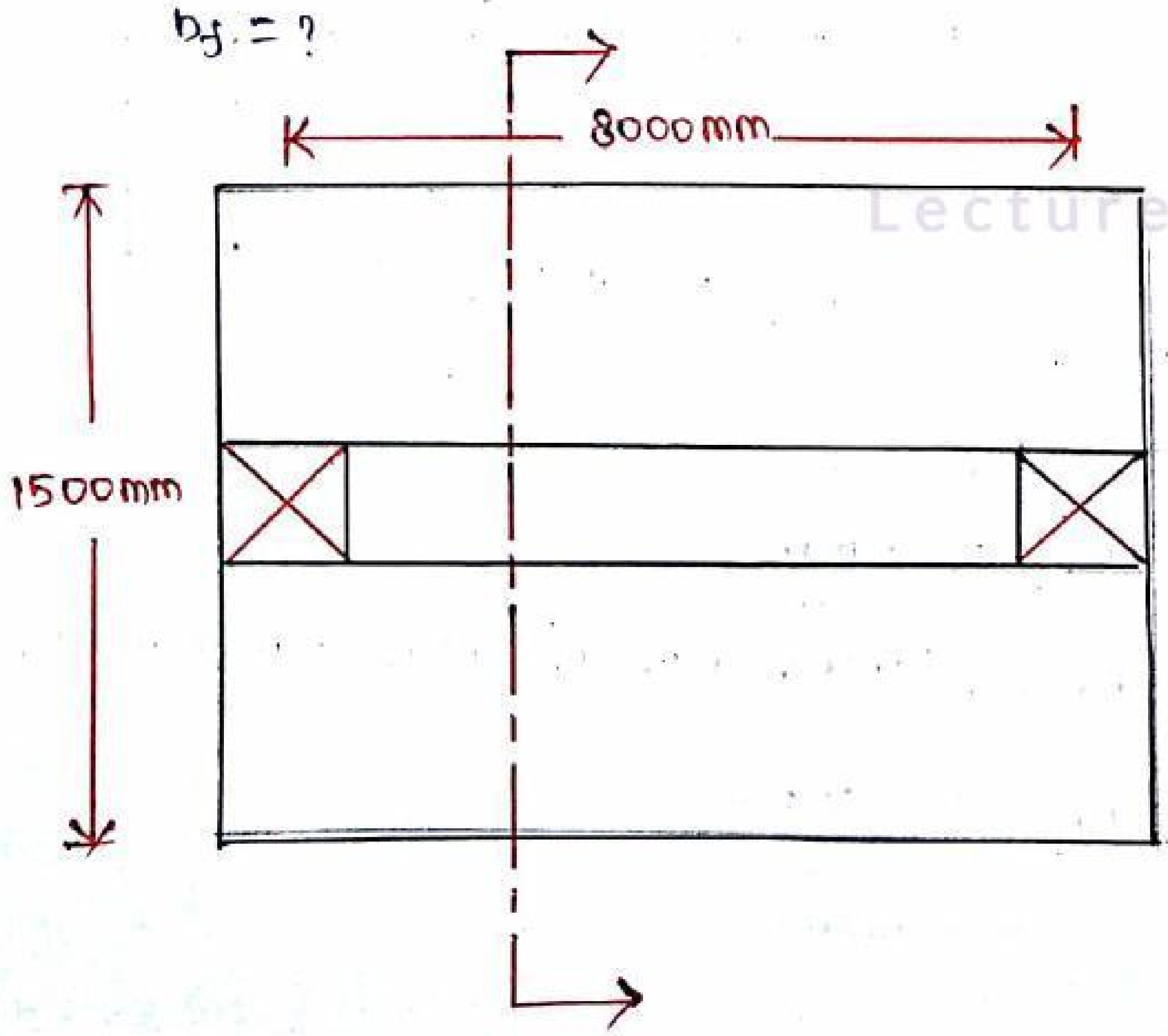
The Deckslab of a Foot bridge of 1.5m width is Supported by a central beam over a span of 8.0m (simply supported). The width of the beam (Web) is 400mm and the thickness of slab is 130mm. Determine the Effective width of Flange of the T Beam.

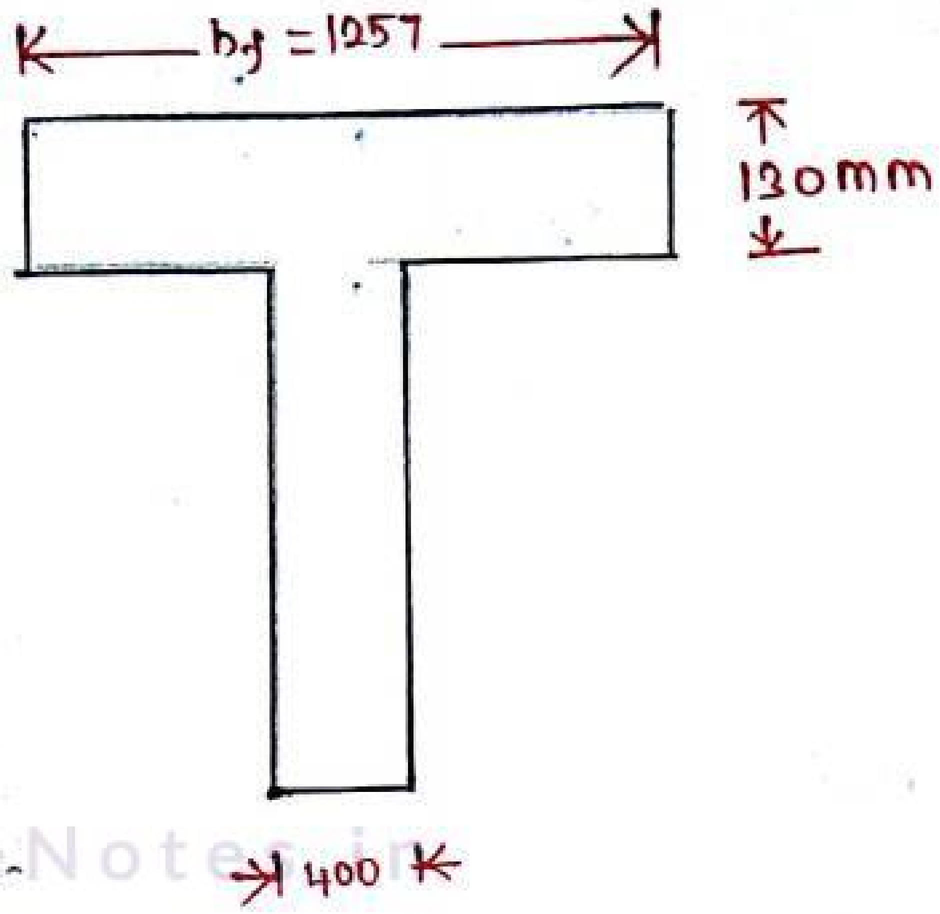
Given data

Actual width of slab $(b) = 1500\,mm$

To Find

$b_f = ?$





Solution

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

$$= \frac{8000}{\left(\frac{8000}{1500}+4\right)} + 400$$

$$= 1257.14\,mm < 1500\,mm$$

Problem on Analysis of T- Beam

Prblm. No: 14 CASE - I

A T- Beam of Flange width 850mm, Flange thickness 100 mm has an effective depth of 475mm. The rib width 275mm beam is reinforced with Four bars of 20 mm diameter. Find the Ultimate moment of Resistance. Use M20 Concrete and Fe 415 Steel

Given data

$b_f = 850\,mm$

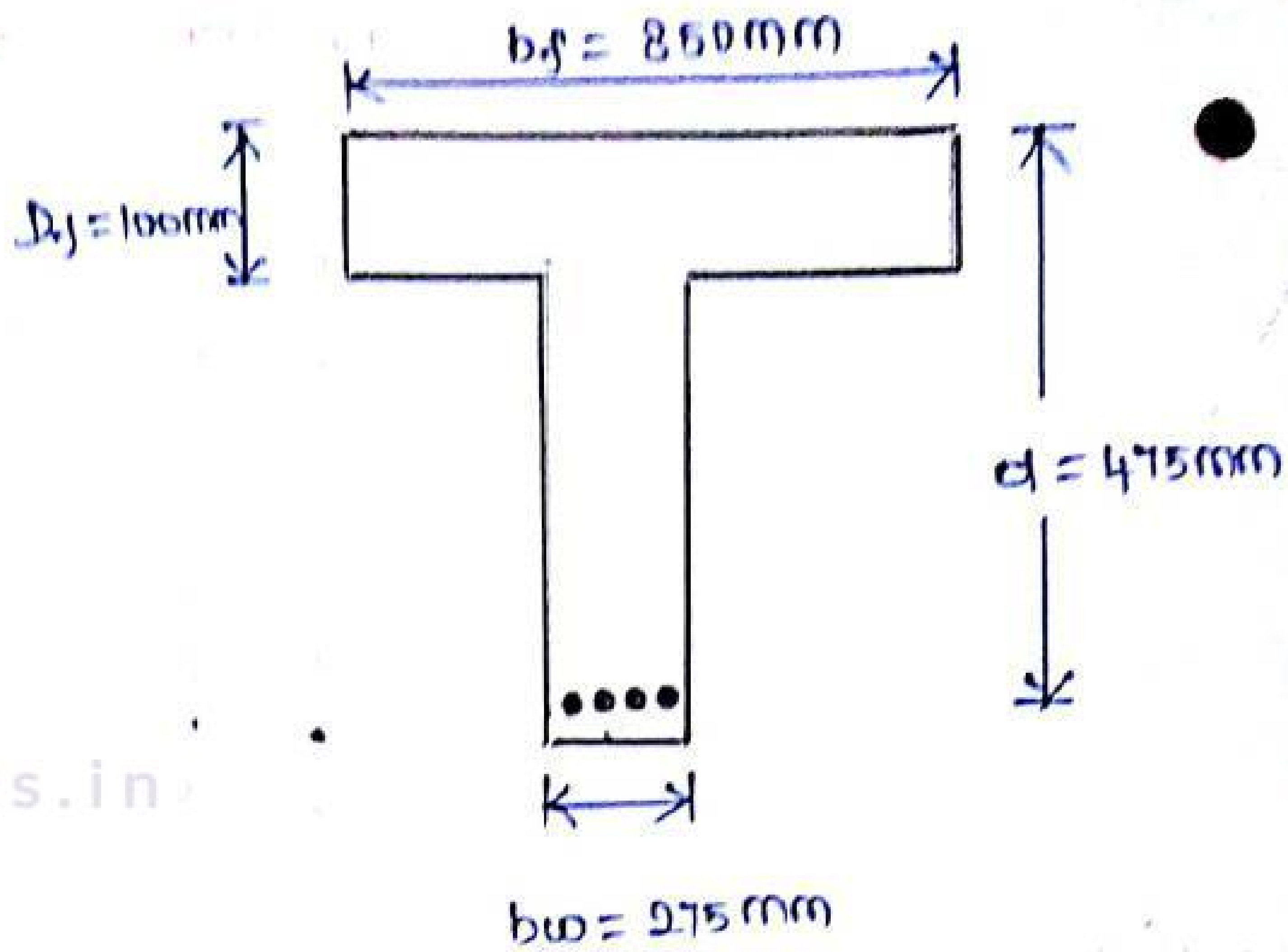
$D_f = 100\,mm$

$b_w = 275\,mm$

$d = 475\,mm$

$A_{st} = 4 \times \frac{\pi}{4}(20)^2$

$= 1256.63\,mm^2$



Solution

Assuming the NA lie within the Flange

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

$$= \frac{0.87 \times 415 \times 1256.63}{0.36 \times 20 \times 850}$$

$\boxed{x_u = 74.13\,mm} < D_f \; (850\,mm)$

Hence our assumption about the portion of the NA is correct

Also

$$x_{umax} = 0.48d$$

$$= 0.48 \times 475$$

$\boxed{x_{umax} = 228\,mm}$

$\therefore \; x_u < x_{umax}$

Hence the section is under reinforced section

Ultimate Moment of Resistance

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

$= 0.36 \times 20 \times 850 \times 74.13 (475 - (0.42 \times 74.13))$

$= 453.67 \times 10^3 (475 - 31.13)$

$= 201.37 \times 10^6$ N.mm

$\boxed{M_u = 201.37 \text{ kNm}}$

Prblm. No:15 CASE - 2(a)

Calculate the Ultimate Moment of Resistance of a T-Beam having the following section properties

Width of Flange $(b) = 1300$ mm

Thickness of Flange $(D_f) = 100$ mm

Width of Rib $(b_w) = 225$ mm

Effective depth $(d) = 600$ mm

Area of steel $(A_{st}) = 4000\ mm^2$

Use M20, Fe 415 HYSD bars

Solution

Depth of NA

Assume the NA lies within the flange

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

$$= \frac{0.87 \times 415 \times 4000}{0.36 \times 20 \times 1200}$$

$$\boxed{x_u = 154.29\,mm} > D_f\ (100\,mm)$$

Hence the assumption that $x_u < D_f$ is not correct NA lies Outside the Flange

$$\frac{D_f}{d} = \frac{100}{600} = 0.166 < 0.2$$

Recalculate the Depth of NA

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

$$= \frac{(0.87 \times 415 \times 4000) - [(0.45 \times 20)(1200 - 225)(100)]}{(0.36 \times 20 \times 225)}$$

$$\boxed{x_u = 242.18\,mm}$$

$$\frac{D_f}{D} = \frac{100}{242.18} = 0.413 < 0.42$$

$$x_{umax} = 0.48 \times d$$

$$= 0.48 \times 600$$

$$\boxed{x_{umax} = 288\,mm}$$

$$x_u < x_{umax}$$

$$M_u = \left\{0.36\left(\frac{x_u}{d}\right)\left[\left(1-0.42\left(\frac{x_u}{d}\right)\right)\right] f_{ck}\ b_w\ d\right\} + \left\{0.45 f_{ck}\ (b_f - b_w)\ D_f\ \left(d - \frac{D_f}{2}\right)\right\}$$

$$= \left\{0.36 \times \left(\frac{242.18}{600}\right)\left[\left(1 - 0.42 \times \left(\frac{242.18}{600}\right)\right)\right] \times 20 \times 325 \times 600^2\right\} + \left\{0.45 \times 20 \times (1200 - 325) \times 100 \times \left(600 - \frac{100}{2}\right)\right\}$$

$M_u = 764.84 \times 10^6$ N·mm

$$\boxed{M_u = 764.84 \text{ kN·m}}$$

Prblm · No: 16 (CASE - 2(B))

A Singly reinforced T- beam has a flange width of 900mm, thickness of Flange is 150mm, width of rib = 300mm, Effective depth = 650mm, Area of Tensile reinforcement = 4000 mm². Use M 20 Grade concrete and Fe 415 HYSD bars are used. Estimate the Ultimate Flexural strength of the section

Given data

b_f = 900mm

D_f = 150mm

b_w = 300mm

b = 650mm

A_{st} = 4000 mm²

f_y = 415 N/mm²

f_{ck} = 20 N/mm²

(a) Depth of NA

Assume NA lies within the flange

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

$$= \frac{0.87 \times 415 \times 4000}{0.36 \times 20 \times 900}$$

$$x_u = 221 mm > D_f \ (150mm)$$

Hence the assumption that $x_u < D_f$ is not correct. The NA is lies outside the flange.

$$\left(\frac{D_f}{d}\right) = \frac{150}{650} = 0.23 > 0.2$$

Recalculate the depth of Neutral axis

$$x_u = \frac{0.87 \cdot f_y \cdot A_{st} - 0.45 f_{ck} (b_f - b_w) y_f}{0.36 f_{ck} \cdot b_w}$$

$$y_f = 0.15 x_u + 0.65 D_f$$

$$= 0.15 x_u + 0.65 \times 150$$

$$\boxed{y_f = 0.15 x_u + 97.5}$$

$$C_1 + C_2 = T$$

$$C_1 = 0.36 f_{ck} b_w \cdot x_u$$

$$= 0.36 \times 20 \times 300 \times x_u$$

$$C_1 = 2160 x_u$$

$$C_2 = 0.42 f_{ck} (b_f - b_w) y_f$$

$$y_f = 0.15 x_u + 0.65 D_f$$

$$= 0.15 x_u + (0.65 \times 150)$$

$$= 0.15 x_u + 97.5$$

$$C_2 = \{0.42 \times 20 \times (900 - 200) \times (0.15 x_u + 97.5)\}$$

$$= 810 x_u + 526.5 \times 10^3$$

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

$$= 0.87 \times 415 \times 4000$$

$$T = 1.44 \times 10^6 \text{ N}$$

$$2160 x_u + 810 x_u + 526.5 \times 10^3 = 1.44 \times 10^6$$

$$2970 x_u + 526.5 \times 10^3 = 1.44 \times 10^6$$

$$x_u = \frac{(1.44 \times 10^6) - (526.5 \times 10^3)}{2970}$$

$$x_u = 309 \text{ mm} > D_f$$

$$x_{u\,max} = 0.48 d$$

$$= 0.48 \times 650$$

$$= 312 \text{ mm}$$

$$x_u < x_{u\,max}$$

Hence it is Under reinforced section

$$M_u = \left\{0.36\left(\frac{x_u}{d}\right)\left[1-0.42(x_u/d)\right] f_{ck} \cdot b_w \cdot d^2\right\} + 0.45 f_{ck}(b_f - b_w) y_f (d - 0.5 y_f)$$

$y_f = (0.15 \times 309) + (97.5)$

$y_f = 142.85 mm$

$M_u = 796.05 \times 10^6 \; N \cdot mm$

$$\boxed{M_u = 796.05 \; KN \cdot m}$$

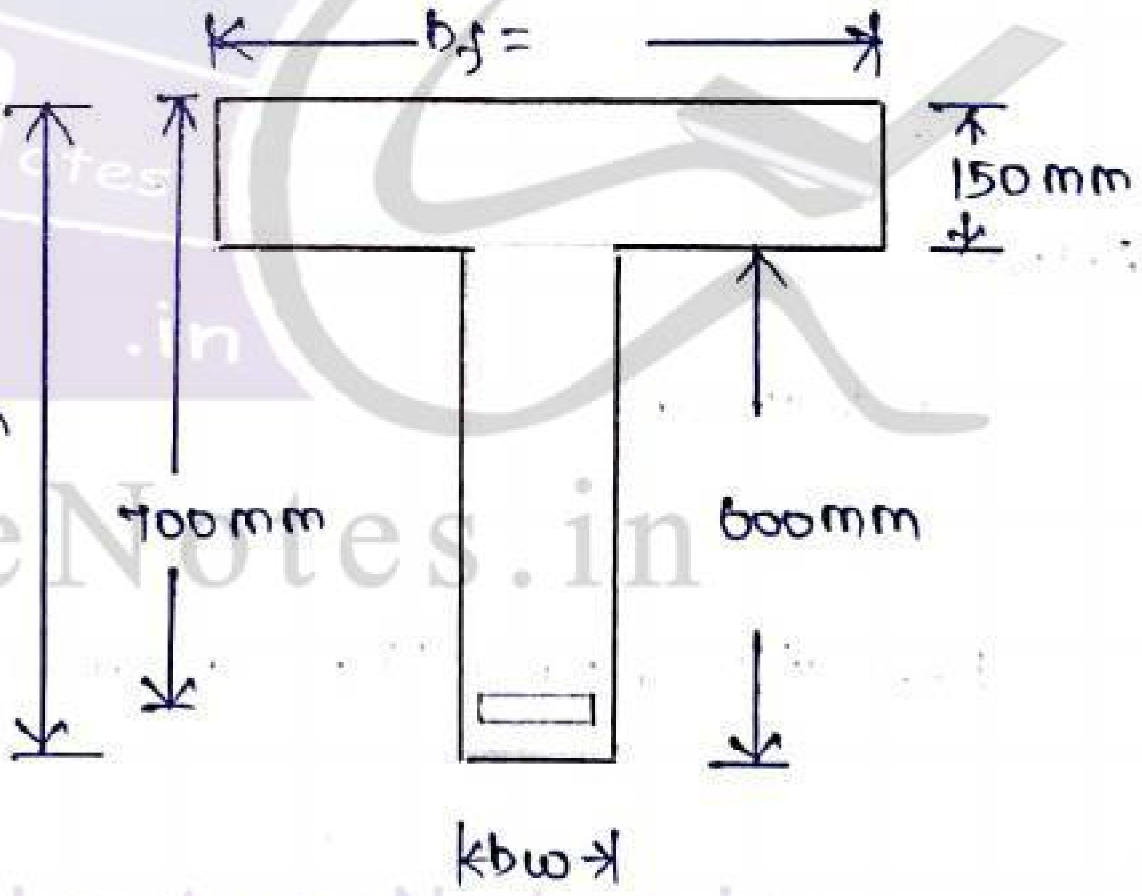
DESIGN OF T- BEAM

Prblm. No : 17

A T- Beam slab floor of Reinforced concrete has a slab 150mm thick spanning between the T- Beams which are spaced 3m apart. The beams have a clear span of 10m and the End bearing are 450mm thick walls. The Live load on the floor is 4 KN/m². Using M20 Grade Concrete and Fe 415 HYSD bars. Design one of the Intermediate T- beams

Given data

$D_f = 150\,mm$



Spacing of T- Beams $\Big\} = 3\,m$

Clear span = 10m

Bearing Thickness = 450mm

Working Live load $\Big\} = 4\,KN/m^2$

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

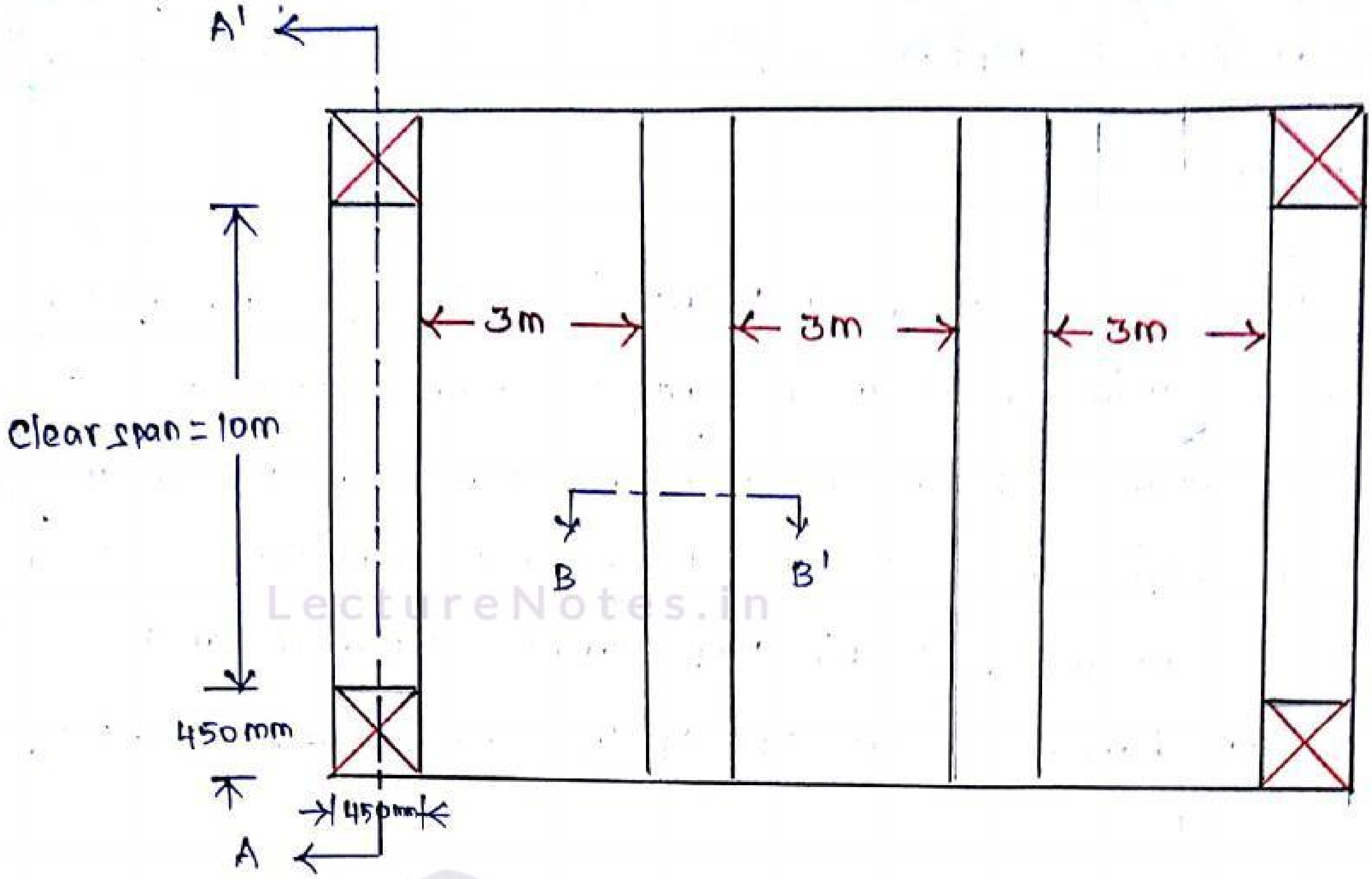
$f_y = 415\ N/mm^2$

Solution

Step : 1

Cross sectional dimensions

Assume, Effective depth $= \frac{Span}{15}$



$= \frac{10 \times 10^3}{15}$

$= 666.66 mm$

Adopt

$d = 700mm$

Effective cover $(d') =$ clear cover + Stirrups + $\frac{\text{Main bar}}{2}$

$= 25 + 8 + \frac{32}{2}$

$= 49 \simeq 50$

$\boxed{d' = 50mm}$

Overall depth (D) = Effective depth + Effective cover

$= 700 + 50$

$\boxed{D = 750mm}$

Width of rib $= \frac{2}{3}$ the depth of rib

$= \frac{2}{3} \times (750 - 150)$

$= 333.33 \text{mm}$

$\boxed{b_w = 300 \text{ mm}}$

Step: 2

Effective Span

Centre to Centre of bearing = clear span + C/C Support

$= 10 + (0.25 + 0.25)$

$= 10 + 0.45$

$= 10.45 \text{m}$

Effective span = Clear span + Effective depth

$= 10 + 0.70$

$= 10.70 \text{m}$

Step: 3

Loads

Self weight of slab $= (0.15 \times 3 \times 25) \Rightarrow 11.25$ KN/m

Self weight of Rib $= (0.3 \times 0.6 \times 25) \Rightarrow 4.5$ KN/m

Floor finish $= (0.6 \times 3) \Rightarrow 1.8$ KN/m

Total dead load $\Rightarrow 18.00$ KN/m

Total Live load $= (4 \times 3) \Rightarrow 12$ KN/m

Design ultimate load $(W_u) = 1.5\,(18+12)$

$= 45\ \text{kN/m}$

Step: 4

Ultimate Moments and Shear Force

$$M_u = (0.125 \times W_u \times L^2) \Rightarrow 0.125 \times 45 \times 10.45^2 \Rightarrow 614.26\ \text{kN·m}$$

$$V_u = (0.5 \times W_u \times L) \Rightarrow 0.5 \times 45 \times 10.45 \Rightarrow 235.12\ \text{kN}$$

Step: 5

Effective Width of Flange (b_f)

$$b_f = \left[\frac{L_0}{6} + b_w + 6D_f\right]$$

$$= \frac{10.45}{6} + 0.3 + (6 \times 0.15) = 2.94\text{m} \simeq 3\text{m}$$

$$\boxed{b_f = 3000\ \text{mm (or) } 3\text{m}}$$

Step: 6

Limiting Moment of Resistance

$$M_{u\,lim} = 0.36\, f_{ck}\, b_f\, D_f\, (d - 0.42\, D_f)$$

$$= 0.36 \times 20 \times 3000 \times 150\,(700 - 0.42 \times 150)$$

$$= 2.268 \times 10^9 - 204.12 \times 10^6$$

$$= 2063.88 \times 10^6\ \text{N·mm}$$

$$= 2063.88\ \text{kN·m}$$

$M_u < M_{u\,lim}$ (Hence the section is URS)

Step: 7

Tension Reinforcement

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

$$614.26 \times 10^6 = 0.87 \times 415 \times A_{st} \cdot 700 \left(1 - \frac{415 \times A_{st}}{2000 \times 700 \times 20}\right)$$

$$= 252.73 \times 10^3 A_{st} - 2.497 A_{st}^2$$

$$2.497 A_{st}^2 - 252.73 \times 10^3 A_{st} + 614.26 \times 10^6 = 0$$

$$\boxed{A_{st} = 2491.84 \, mm^2} \quad (\text{Required})$$

$$(\text{No. of bars})_{Prov} = \frac{A_{st\,req}}{a_{st}}$$

$$= \frac{2491.84}{\frac{\pi}{4} \times (32)^2}$$

$$= 3.09 \text{ no's}$$

$$A_{st\,prov} = n \times \frac{\pi}{4} d^2$$

$$= 2 \times \frac{\pi}{4} \times 32^2$$

$$= 1608.49 mm^2$$

$$= 2491.84 - 1608.49$$

$$= 884 \, mm^2$$

$$= \frac{Ast}{ast}$$

$$= \frac{884}{\frac{\pi}{4} \times 25^2}$$

$$= 1.80 \simeq 2 \text{ Nos}$$

Provide 2 bars of 32mm and 2 bars of 25mm diameter

Assuming the NA lies within the flange

$$x_u = \frac{0.87 \times 415 \times Ast}{0.36 \times f_{ck} \times b_f}$$

$$= \frac{0.87 \times 415 \times 2590.24}{0.36 \times 20 \times 2000}$$

$$x_u = 43.29 \text{ mm} < D_f \ (150\text{mm})$$

Hence our assumption is correct

Step:8

Shear Reinforcement

$$\tau_v = \frac{V_u}{bd} = \frac{235.12 \times 10^3}{300 \times 700} = 1.119 \ N/mm^2$$

$$P_t = \frac{100 Ast}{bd} = \frac{100 \times 2590.24}{300 \times 700} = 1.23$$

P_t	τ_c
1.00 →	0.62
1.23 →	?
1.25 →	0.67

$$\frac{(1.25 - 1.00)}{(1.25 - 1.23)} = \frac{(0.67 - 0.62)}{(0.67 - x)}$$

$\tau_c = 0.67\ N/mm^2$

$\tau_v > \tau_c$

Balanced shear $(V_{us}) = [V_u - (\tau_c \cdot b_w \cdot d)]$

$= [225.12 - (0.6 \times 10^{-3} \times 300 \times 700)]$

$V_{us} = 95\ kN$

Using 8mm dia 2 legged stirrups spacing

$$S_v = \frac{0.87 f_y \times A_s \cdot d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 700}{95 \times 10^3}$$

$= 266\ mm$

Provide a spacing of 250 mm at c/c

Step: 9

Check for Deflection Control

$$P_t = \frac{100 A_{st}}{bd} \Rightarrow \frac{100 \times 2590}{2940 \times 700} \Rightarrow 0.126$$

$$\left(\frac{b_w}{b_f}\right) = \frac{300}{2940} \Rightarrow 0.102$$

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{basic} \times k_t \times k_c \times k_f$$

$$\left(\frac{L}{d}\right)_{basic} = 20$$

$$f_s = 0.58 \times 415 \times \frac{2492}{2492}$$

$= 240.7 \simeq 240$

$f_s = 240$

$$\left(\frac{L}{d}\right)_{Max} = 20 \times 2.0 \times 1.0 \times 0.8$$

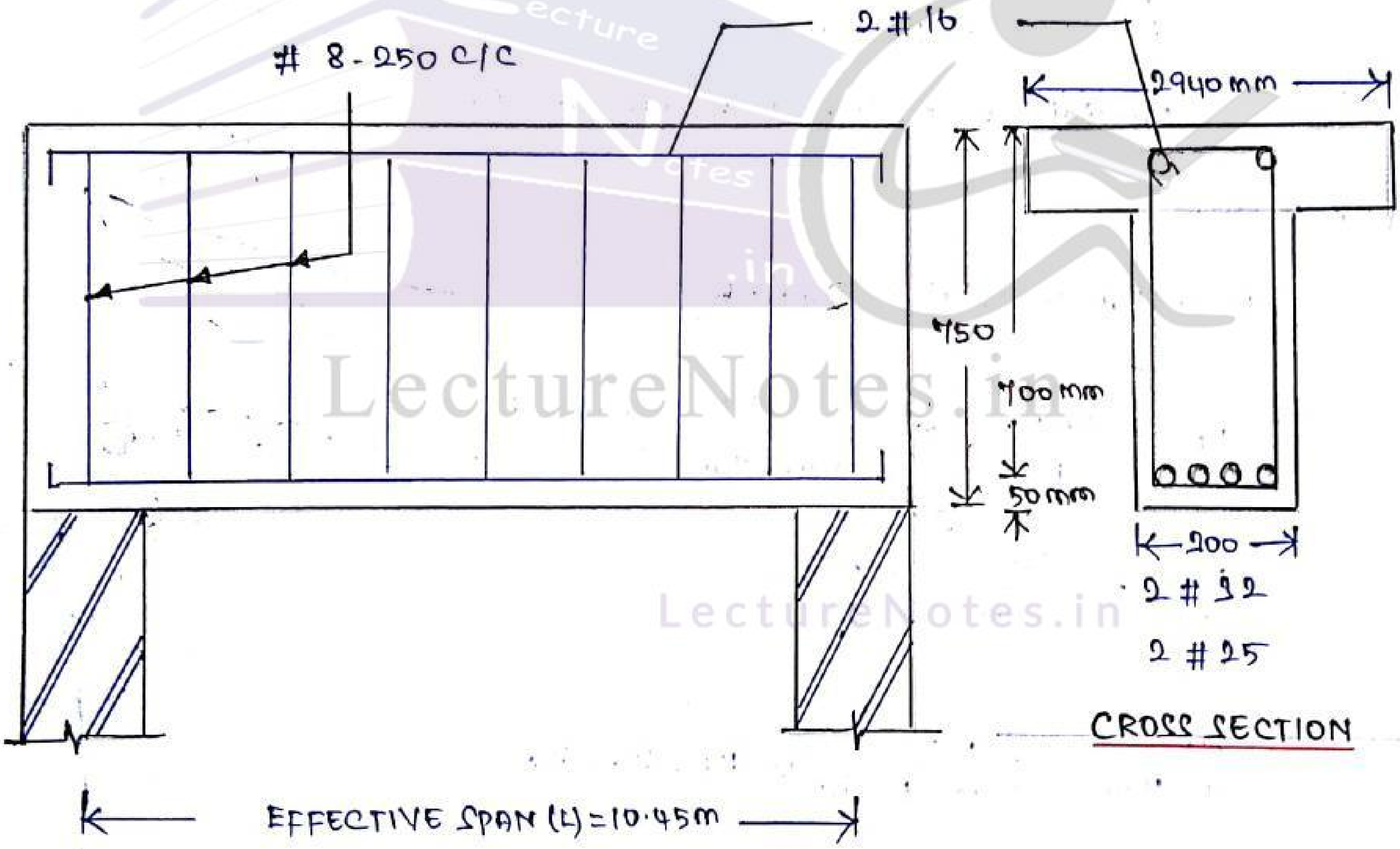
$$= 22$$

$$\left(\frac{L}{d}\right)_{prov} = \frac{10450}{700} \Rightarrow 14.92$$

$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{provide}$$

Hence, check for deflection control is satisfactory

Reinforcement Detail in Tee - Beam



LONGITUDINAL SECTION

DESIGN OF L - BEAM

Prblm. No: 18

Desiga a L- Beam for an affice room Floor to suit the Following data

Data

Clear Span = 6m

Centre to centre of Supports = 6.3m

The L- Beams are monolithic with R.C column

Spacing of beams = 2.75m c/c

Loading (office Floor) = 4 KN/m^2

Thickness of slab = 100mm

Width of column = 300 mm

Use M20 Grade of concrete Fe 415 HYSD bars

f_{ck} = 20 N/mm^2

f_y = 415 N/mm^2

Step: 1

Cross Sectional dimension

L- Beam is subjected to

* Flexure (Bending)
* Torsion

* Shear Force

Assume,

Trail section having $\left(\frac{\text{span}}{\text{depth}}\right)$ → Refer IS: 456-2000 Pg. No: 27

$$\text{Effective depth } (d) = \frac{6300}{12}$$

$$d = 525\,mm$$

Adopt

$d = 550\,mm$

$$\text{Effective cover } (d') = \text{clear cover} + \text{stirrups} + \frac{\text{Main bar}}{2}$$

$$= 25 + 8 + \frac{16}{2}$$

$$= 41 \simeq 50$$

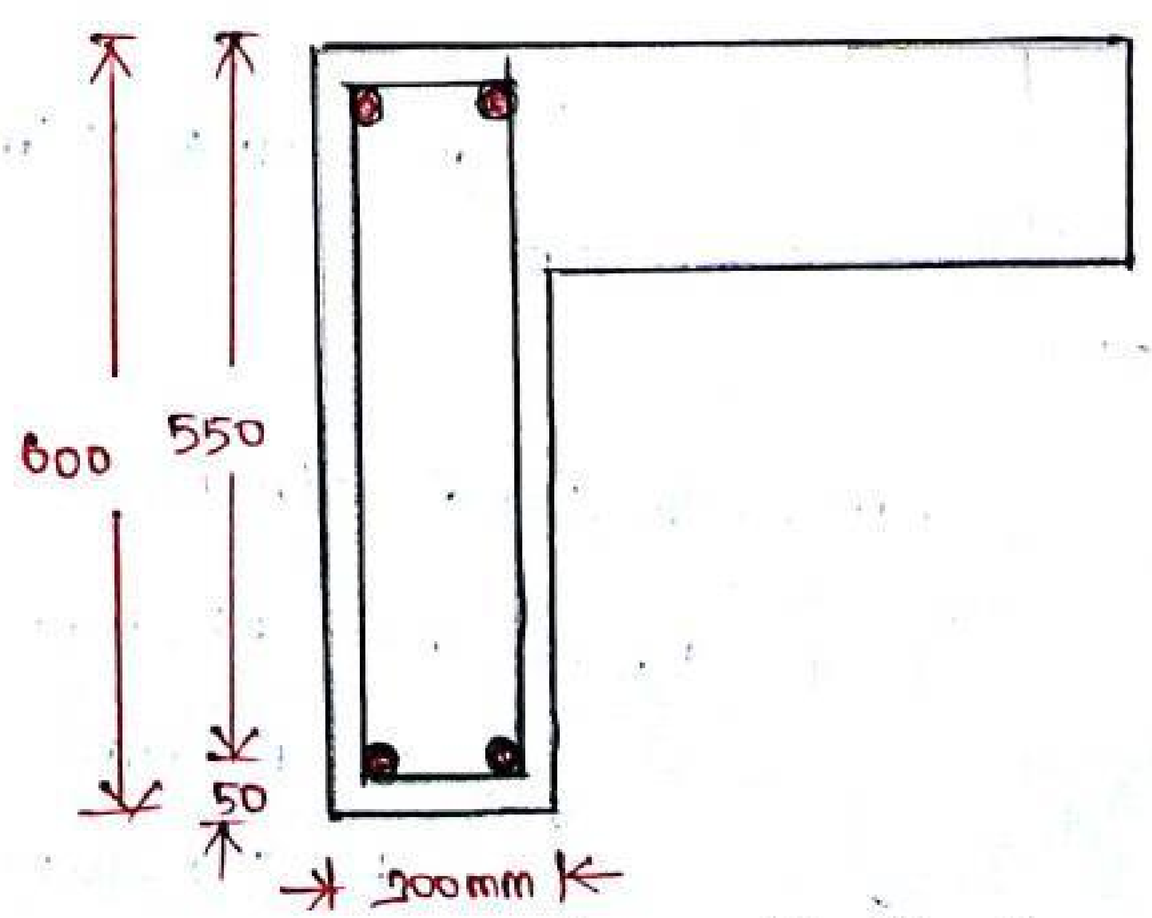
$$\boxed{d' = 50\,mm}$$

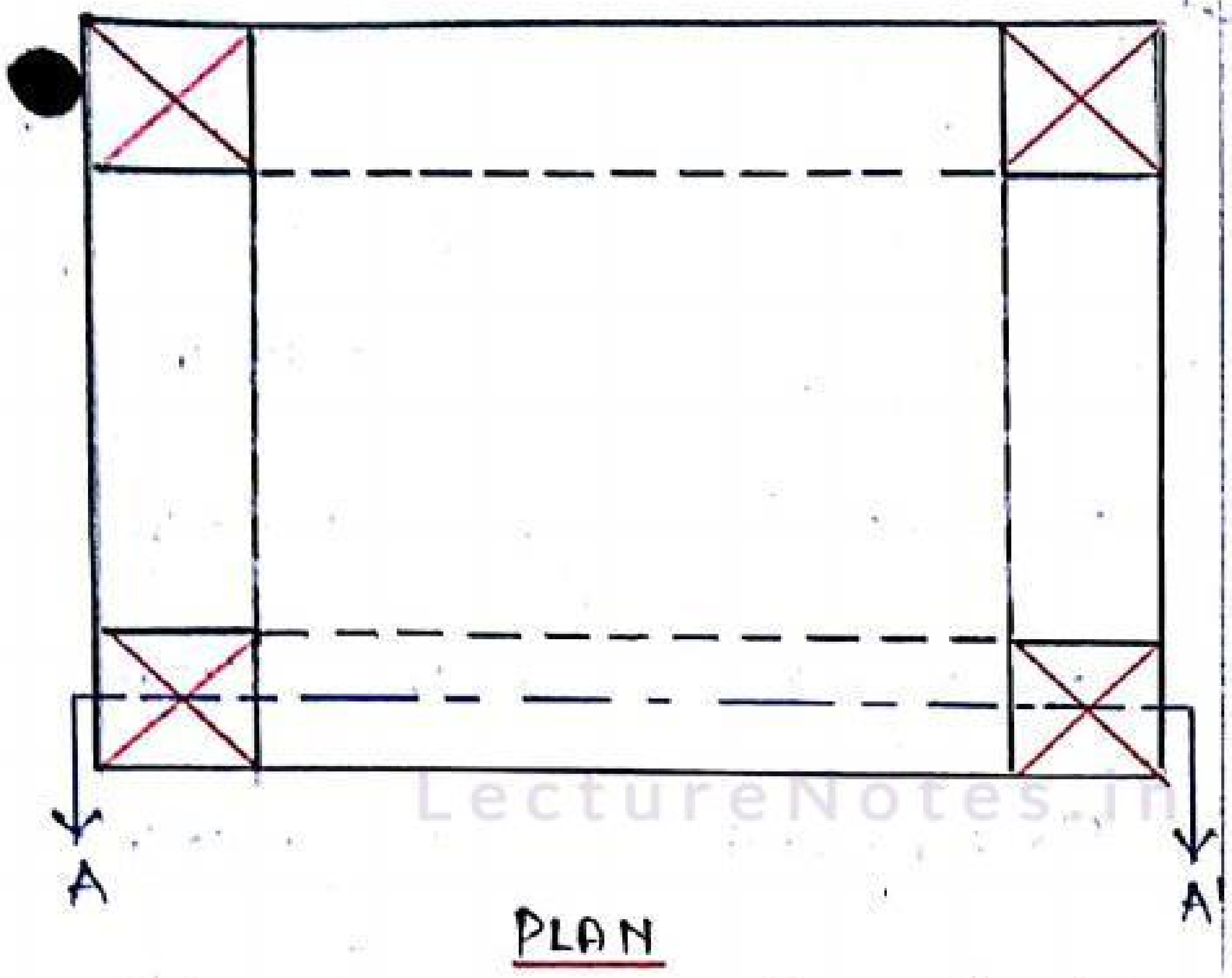
$$\text{Overall depth } (D) = \text{Effective depth} + \text{Effective cover}$$

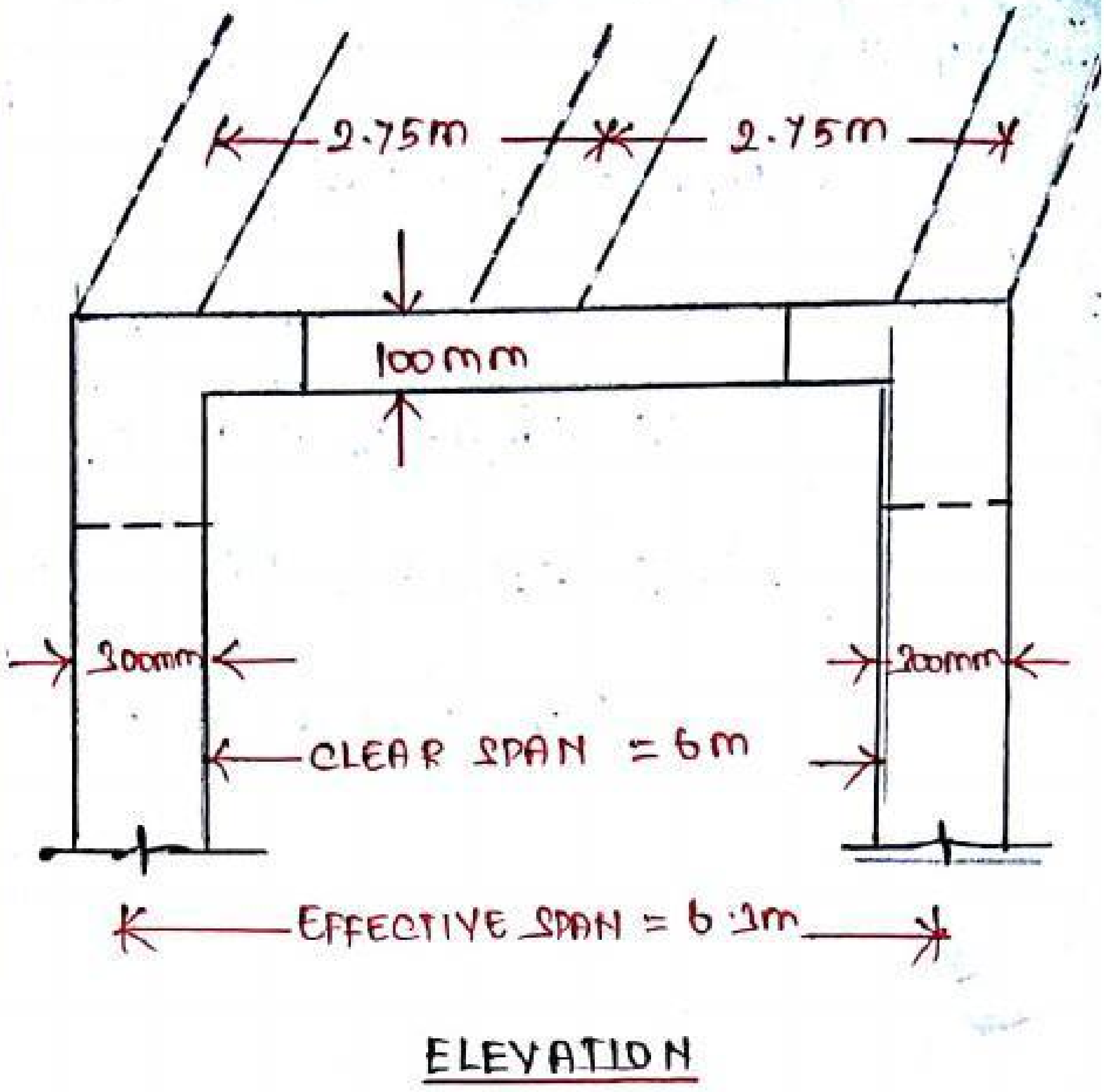
$$= 550 + 50$$

$$\boxed{D = 600\,mm}$$

Breadth of Web $(b_w) = 300\,mm$







Step: 2

Effective span

* Centre to Centre of supports = Clear span + Half of the width of column (or) Half of the breadth of the web

= 6 + (0.15 + 0.15)

= 6.3m (Least Value)

* Clear span + Effective depth = 6 + 0.55

= 6.55m

Adopt,

Effective span (L) = 6.3m

Step: 3

Loads Calculation

Self Weight of Slab $= 0.1 \times 25 \times 0.5 \times 2.75 \Rightarrow 3.43$ KN/m

Self Weight of Rib $= 0.5 \times 0.3 \times 25 \Rightarrow 3.75$ KN/m

Floor Finish $= 0.6 \times 0.5 \times 2.75 \Rightarrow 0.82$ KN/m

Plaster finishes $= 0.49$ KN/m

Total Dead Load $= 8.50$ KN/m

Live load $= 4 \times 0.5 \times 2.75 \Rightarrow 5.50$ KN/m

$W = 14.00$ KN/m

Design Ultimate Load $W_u = FOS(D.L + L.L)$

$= 1.5(8.5 + 5.50)$

$\boxed{W_u = 21 \text{ KN/m}}$

Step: 4

Effective Width of Flange

$$b_f = \left[\left(\frac{L_o}{12}\right) + b_w + 3D_f\right]$$

$$= \frac{6300}{12} + 300 + (3 \times 100)$$

$b_f = 1125$ mm (Least value)

$b_f = b_w + 0.5$ (Spacing b/w the ribs)

$= 300 + (0.5 \times 2450)$

$\boxed{b_f = 1525 \text{ mm}}$

Adopt ;

$$b_f = 1125mm$$

Step:5

Ultimate Bending moment and shear force

At Support Reaction

Refer IS:456-2000
Pg.No:36
Table.No:12

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

$$= \frac{21 \times 6.3^2}{8}$$

$$M_u = 70 kN \cdot m$$

$$V_u = \frac{W_u L}{2}$$

Pg.No:36
Table.No:13

$$= \frac{21 \times 6.3}{2}$$

$$V_u = 66 kN$$

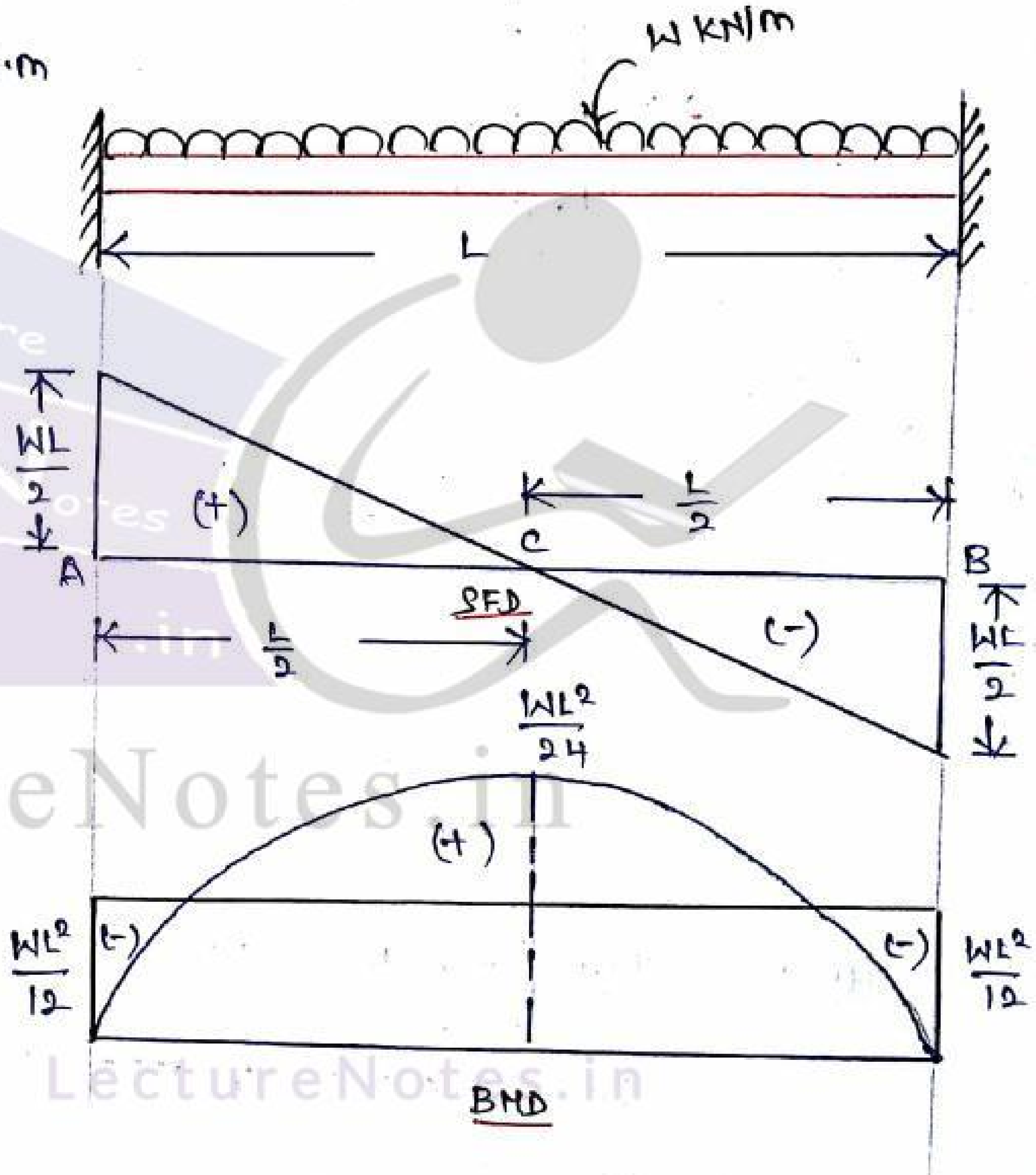
At Centre of Span

$$M_u = \frac{W_u L^2}{24}$$

Pg.No:35

$$= \frac{21 \times (6.3)^2}{24}$$

$$M_u = 35 kN \cdot m$$



Step:6

Torsional Moment at Support Reaction

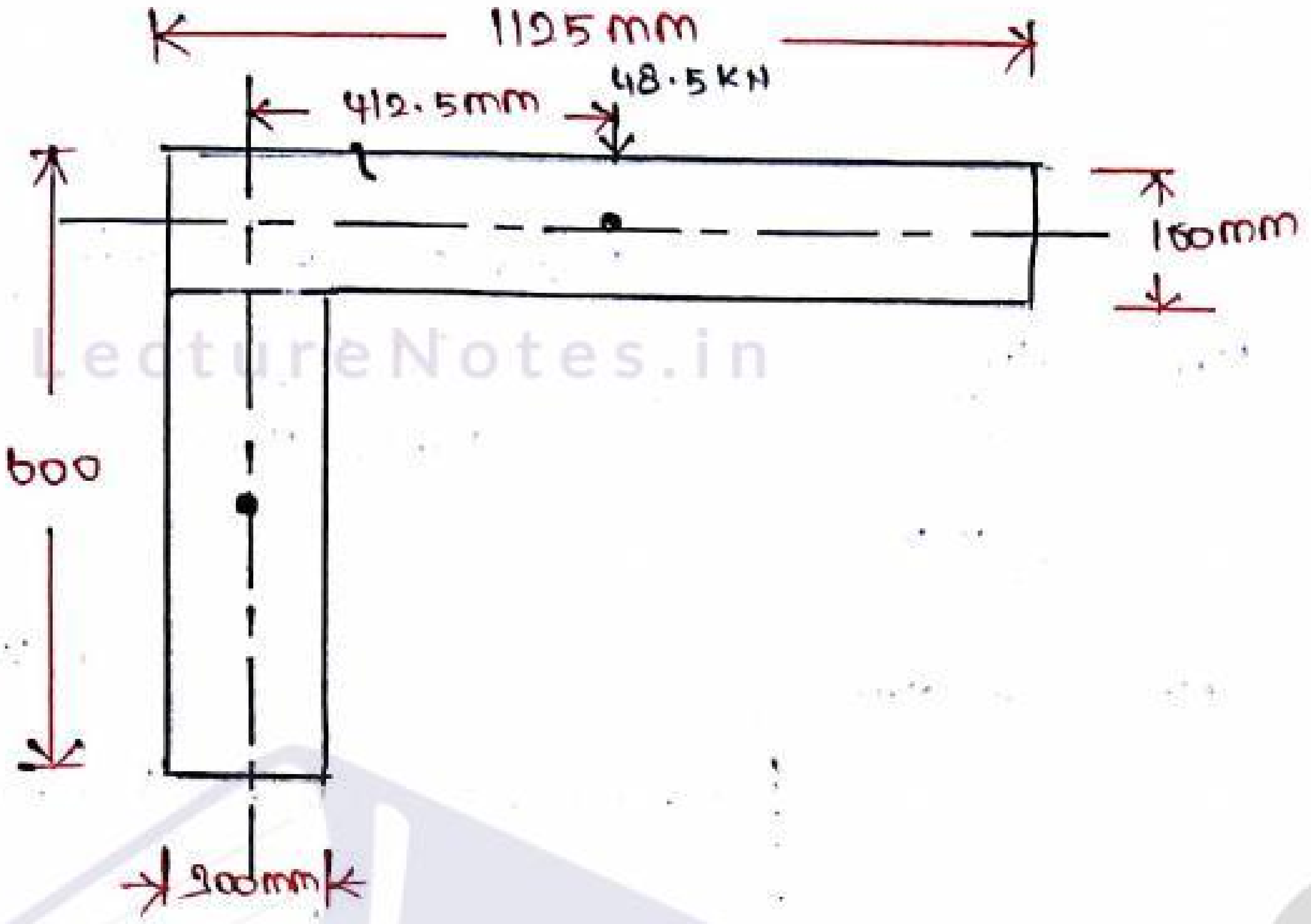
→ Torsional moment is produced due to D.L of slab and L.L on it

→ Working load/m – Web self weight ⇒ (14 – 3.75) = 10.25 kN/m

→ Total ultimate load on slab } ⇒ 0.5 (10.25 × 6.3) = 97 kN

→ Total Ultimate shear Force } ⇒ $0.5 \times 97 = 48.5$ KN

— Distance of centroid of Shear Force from the centre line of beam



$x_1 = \frac{300}{2}$

$= 150mm$

$x_2 = \frac{1125}{2}$

$= 562.5mm$

$\bar{x} = \frac{(150000 \times 150) + (112500 \times 562.5)}{(150000 + 112500)}$

$\bar{x} = 326.775mm$

$A_1 = 300 \times 500$

$= 150000$

$A_2 = 1125 \times 100$

$= 112500$

Ultimate Torsional moment (T_u)

$T_u = 48.5 \times 0.4125$

$\boxed{T_u = 20 \text{ kN·m}}$

Step: 7

Equivalent Bending Moment and Shear Force

Equivalent bending moment (At support reaction) } $M_{e1} = (M_u + M_t)$

Refer IS: 456-2000
Pg. No: 75
Cl. No: 41.4.2

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

M_u = Bending moment

M_t = Torsional moment

$$= 20 \left[\frac{1 + \left(\frac{600}{200}\right)}{1.7} \right]$$

$M_{e1} = M_u + M_t$

$= 70 + 35.2$

$M_{e1} = 105.2\ kN \cdot m$

$M_t = 35.2\ kN \cdot m$

$M_t = 35.2\ kN \cdot m < M_u = 70\ kN \cdot m$

$M_t < M_u$

Equivalent Shear force

$$V_e = V_u + 1.6 \left(\frac{T_u}{b} \right)$$

$$= 66 + 1.6 \left(\frac{20}{0.2} \right)$$

$$V_e = 172\ kN$$

Step: 8

Main Longitudinal Reinforcement

Support section is design as rectangular section to resist the Hogging equivalent moment $M_{e1} = 105.2\ kN \cdot m$

$$M_{u\,lim} = 0.138\, f_{ck}\, b d^2$$

$$= 0.138 \times 20 \times 200 \times (550)^2$$

$$M_{u\,lim} = 250\ kN \cdot m$$

$$M_{e1} = 105.2\ kN \cdot m < M_{u\,lim} = 250\ kN \cdot m$$

$$M_{e1} < M_{u\,lim}$$

∴ The section is URS

$$M_{e1} = 0.87 \times f_y \cdot A_{st}\, d \left[1 - \frac{A_{st} f_y}{b\, d\, f_{ck}}\right]$$

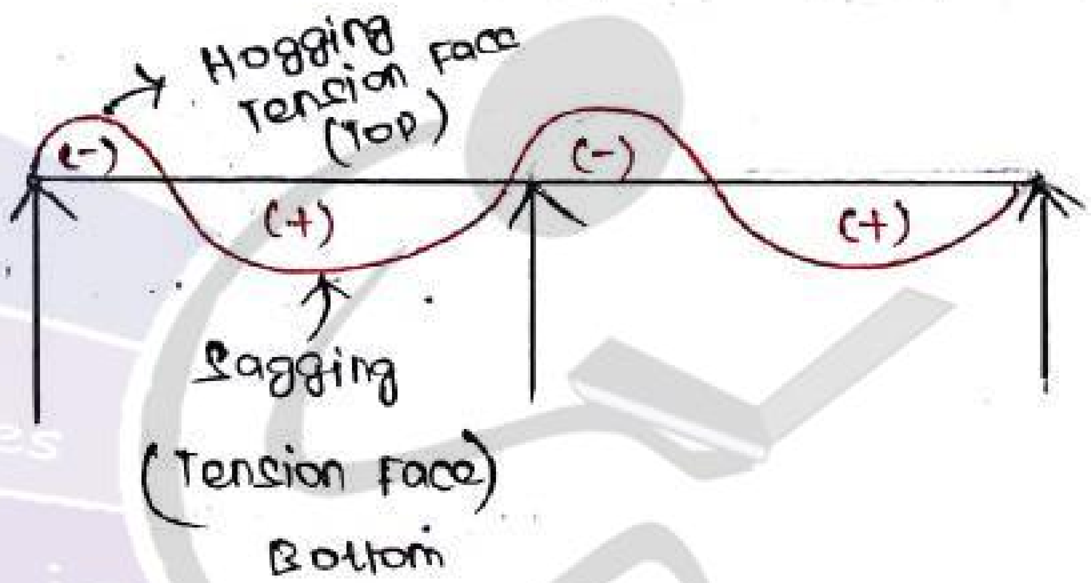
$$(105.2 \times 10^6) = 0.87 \times 415 \times A_{st} \times 550 \left[1 - \left(\frac{415 \times A_{st}}{200 \times 550 \times 20}\right)\right]$$

$$(105.2 \times 10^6) = 198577.5\, A_{st}\, [1 - 1.257 \times 10^{-4} A_{st}]$$

$$105.2 \times 10^6 = 198577.5\, A_{st} - 24.961\, A_{st}^2$$

$$24.961\, A_{st}^2 = 198577.5\, A_{st} + 105.2 \times 10^6$$

$$\boxed{A_{st} = 572\, mm^2}$$



No. of bars Tension Face (Top) Hogging $\Big\} = \frac{A_{st\,req}}{a_{st}}$

$$= \frac{572}{\frac{\pi}{4} \times 20^2}$$

$$= 1.82 \simeq 2$$

Actual area of steel $\Big\} = n \times \frac{\pi}{4} d^2$

$$= 2 \times \frac{\pi}{4} (20)^2$$

$$A_{st\,prov} = 628\, mm^2$$

Provide 2 bars of 16mm dia on the Tension face (Top) ($A_{st} = 628\, mm^2$)

Area of steel required at Centre of span section to resist a moment ($M_u = 25\, kN \cdot m$)

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

$$(25 \times 10^6) = 0.87 \times 415 \times A_{st} \times 550 \left[1 - \frac{415 \times A_{st}}{200 \times 550 \times 20}\right]$$

$$(25\times10^6) = 198577.5\,Ast\left[1-1.257\times10^{-4}\,Ast\right]$$

$$25\times10^6 = 198577.5\,Ast - 24.961\,Ast^2$$

$$24.961\,Ast^2 - 198577.5\,Ast + 25\times10^6$$

$$Ast = 180mm$$

$$Ast_{min} = \frac{0.85\,bw\,d}{fy}$$

$$= \frac{0.85\times300\times550}{415}$$

$$Ast_{min} = 338mm^2$$

$$Ast = 180mm < Ast_{min} = 338mm^2$$

No. of bars centre of span (sagging) } $(n) = \frac{Ast_{min}}{ast}$

$$= \frac{338}{\frac{\pi}{4}\times16^2}$$

$$= 1.68 \simeq 2.$$

$$(NoB) = 2\ No's$$

$$Ast_{prov} = n\times\frac{\pi}{4}(d)^2$$

$$= 2\times\frac{\pi}{4}(16)^2$$

$$Ast_{prov} = 402mm^2$$

provide 2 bars of 16mm dia at the bottom face $(Ast = 402mm^2)$

Step: 9

Side face Reinforcement

Refer IS: 456-2000, Pg. No: 48, Cl. No: 26.5.1.7

Area of tension reinforcement } $= \frac{0.1\times bw\times D}{100}$

$$= \frac{0.1\times300\times600}{100}$$

$A_{st} = 180\ mm^2$

No. of bars
Side face reinforcement $\Big\}$ $(n) = \frac{A_{st}}{a_{st}}$

$= \frac{180}{\frac{\pi}{4} \times 8^2}$

$= 3.58 \simeq 4$ No's

Provide 8mm dia bars 4 No's two on each face as horizontal reinforcement

Step: 10

Shear Reinforcement

$\tau_{ve} = \frac{V_e}{b_w d} = \frac{173 \times 10^3}{300 \times 550} = 1.05\ N/mm^2$

$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 628}{300 \times 550} = 0.38\ N/mm^2$

$\tau_c = 0.42\ N/mm^2$

Refer IS: 456-2000
Pg. No: 73
Table. No: 19

$\tau_c = 0.42 < \tau_{ve} = 1.05\ N/mm^2$

$\tau_c < \tau_{ve}$

Shear reinforcement required

Stirrups

Size of bar = 10mm

Two legged

Side cover = 25mm

Top & Bottom cover $\}$ = 25 + 25

= 50 mm

b_1 = 250mm

d_1 = 500mm

$A_{sv} = 2 \times \frac{\pi}{4} (10)^2$

$= 2 \times 78.5$

$A_{sv} = 157 mm^2$

Spacing of Stirrups

Pg. No: 75
Cl. No: 41.4.3

$$S_v = \frac{0.87 \times f_y A_{sv} d_1}{\left(\frac{T_u}{b_1}\right) + \left(\frac{V_u}{2.5}\right)}$$

$$S_v = \frac{0.87 \times 415 \times 517 \times 500}{\left(\frac{20 \times 10^6}{250}\right) + \left(\frac{66 \times 10^3}{2.5}\right)}$$

= 266mm

Check Sv

Pg. No: 48
Cl. No: 26.5.1.7

(i) x_1 = 250mm

(ii) $\left(\frac{x_1 + y_1}{4}\right) = \left(\frac{250 + 500}{4}\right) = 187.5$

(iii) 300mm

Adopt Minimum Spacing (Sv) = 187.5 ≈ 200mm

$\boxed{S_v = 200mm}$

Step: 11

Check for Deflection Control

$P_t = 0.38$

$P_c = \frac{100 A_{sc}}{b_w d} = \frac{100 \times 402}{200 \times 550} = 0.24$

$\left(\frac{b_w}{b_f}\right) = \left(\frac{200}{1125}\right) = 0.266$

$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{basic} \times K_t \times K_c \times K_f$

$f_s = 0.58 \times 415 \times \frac{572}{628}$

$f_s = 219$

$\left(\frac{L}{d}\right)_{basic} = 20$

$\left(\frac{L}{d}\right)_{Max} = 20 \times 1.20 \times 1.07 \times 0.80$

$= 22.2$

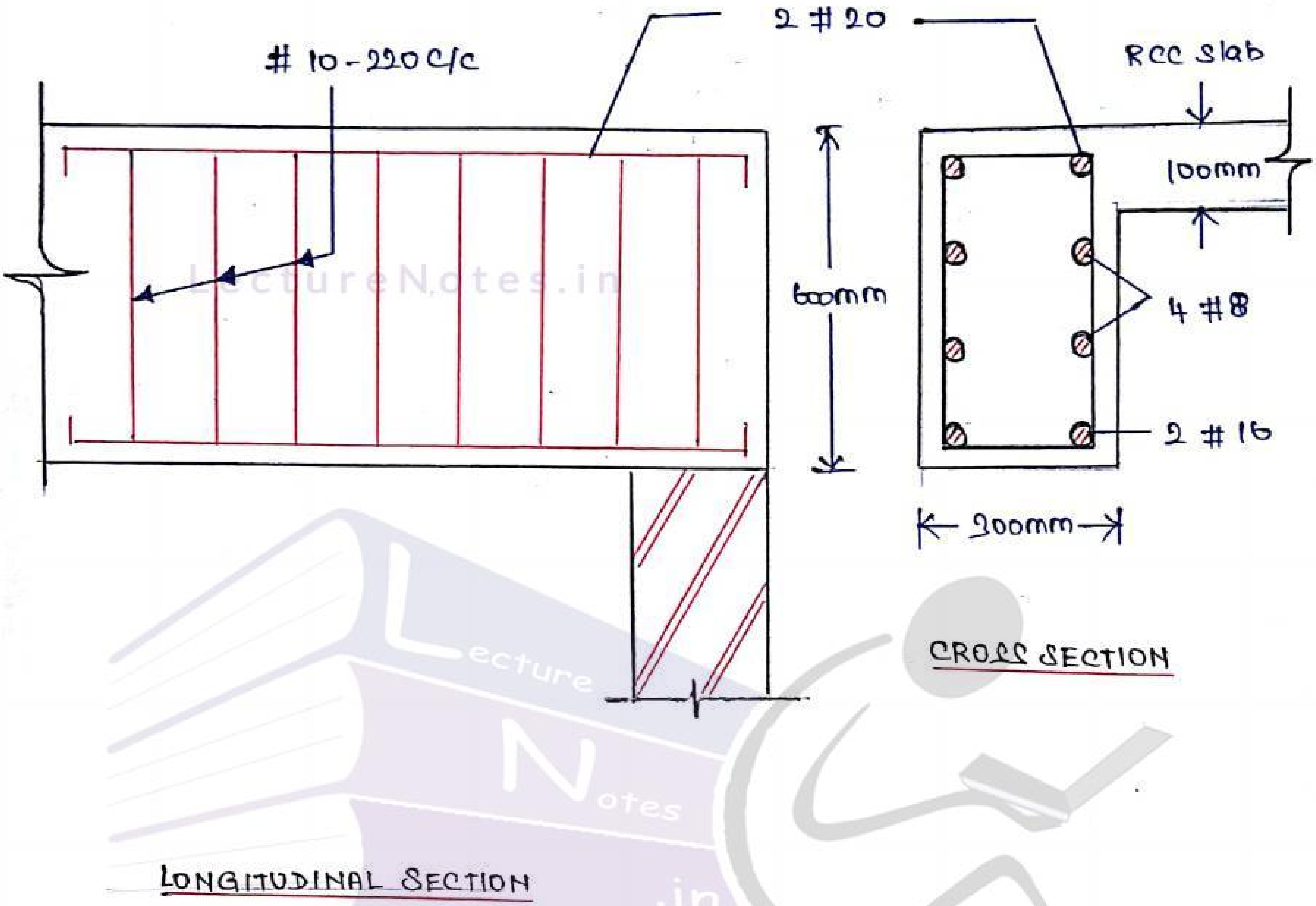
$\left(\frac{L}{d}\right)_{prov} = \left(\frac{6300}{550}\right)$

$= 11.45$

$\left(\frac{L}{d}\right)_{provided} < \left(\frac{L}{d}\right)_{Max}$

check for deflection control is satisfactory

Reinforcement detail in L-Beam



UNIT-2

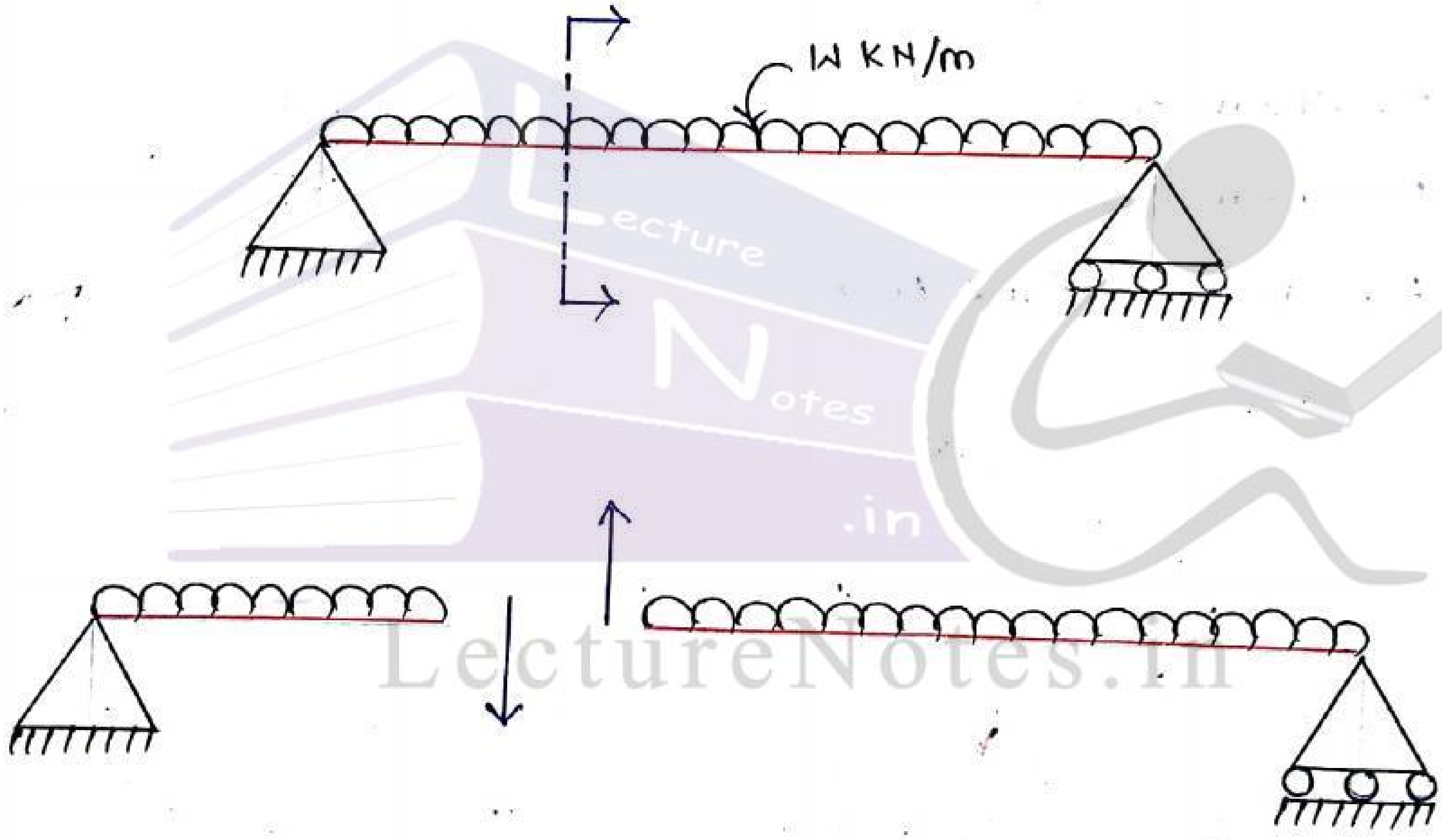
LIMIT STATE OF COLLAPSE - SHEAR, TORSION

SHEAR FORCE

The Shear Force at the Cross Section of the beam may be defined as the Unbalanced Vertical Force to the Right or Left of the Section

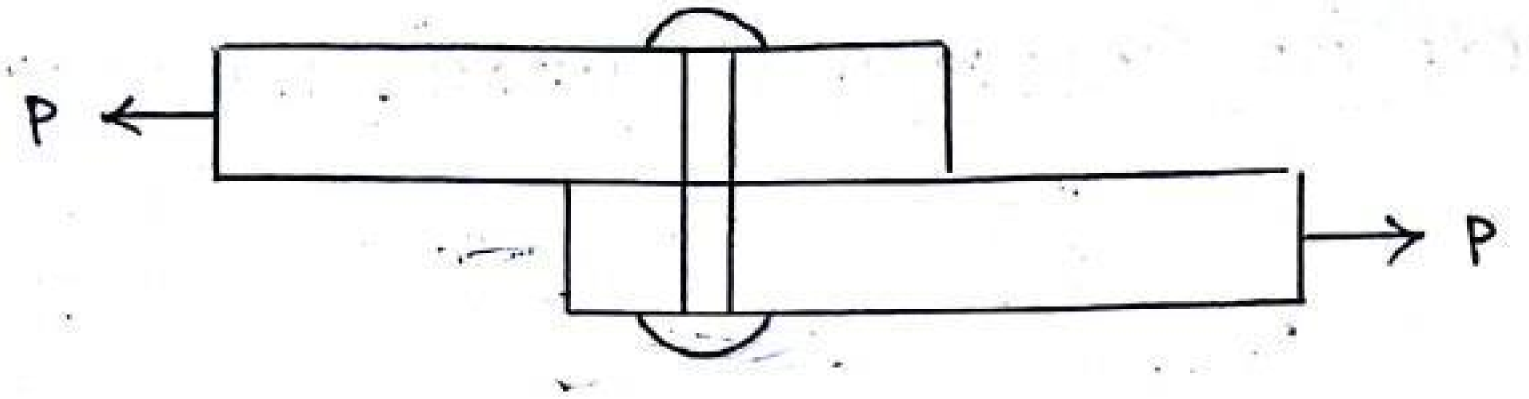
Example

Simply Supported beam with UDL



(OR)

When a body is subjected to Two Equal and opposite forces acting Tangentially across the resisting Section as a result of which the body tends to Shear off the section, then the Stress Induced is called Shear Stress (τ). The corresponding Strain is known as Shear Strain



$$\tau = \frac{F}{A}$$

$$\boxed{\tau = \frac{F}{\frac{\pi}{4}d^2}}$$

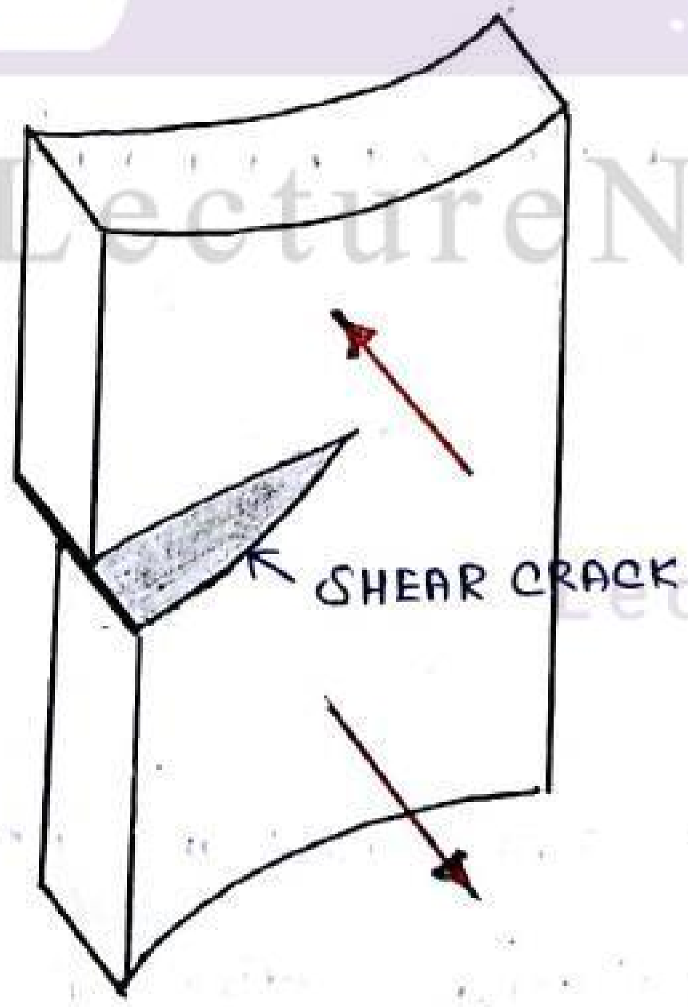
Where,

τ - Shear Stress

F - Shear Force

A - Cross Sectional Area

(OR)



When you apply a force that makes one part of an object slide over another part and thus get separated the force is called a shear force

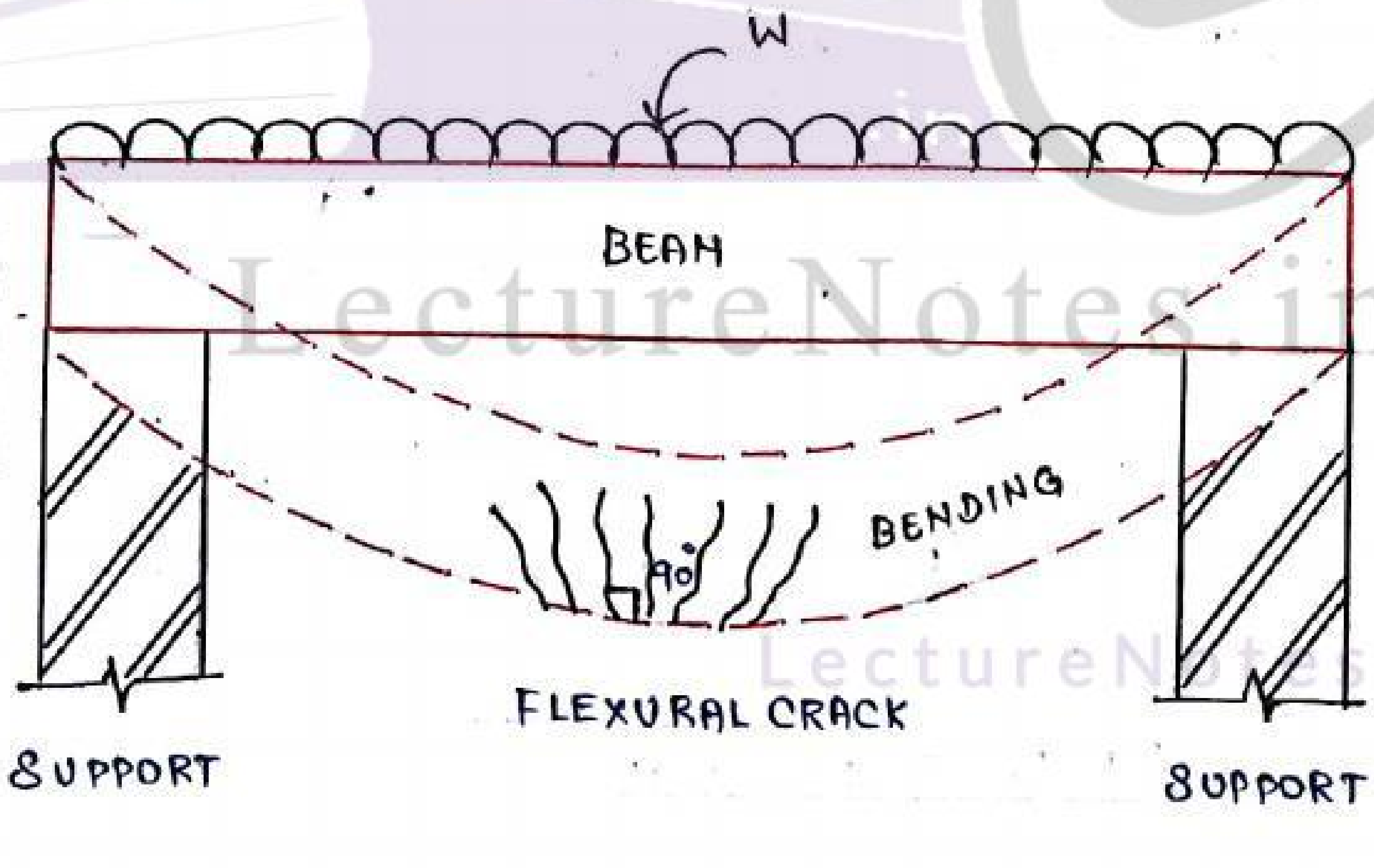
Failure occurs on Beams

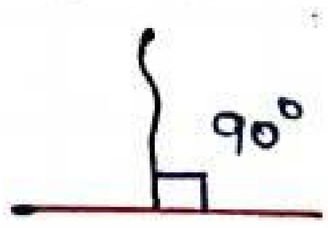
Mainly two types of failure occurs in beam

* Flexural failure (or) Bending
* Shear failure

Flexural failure

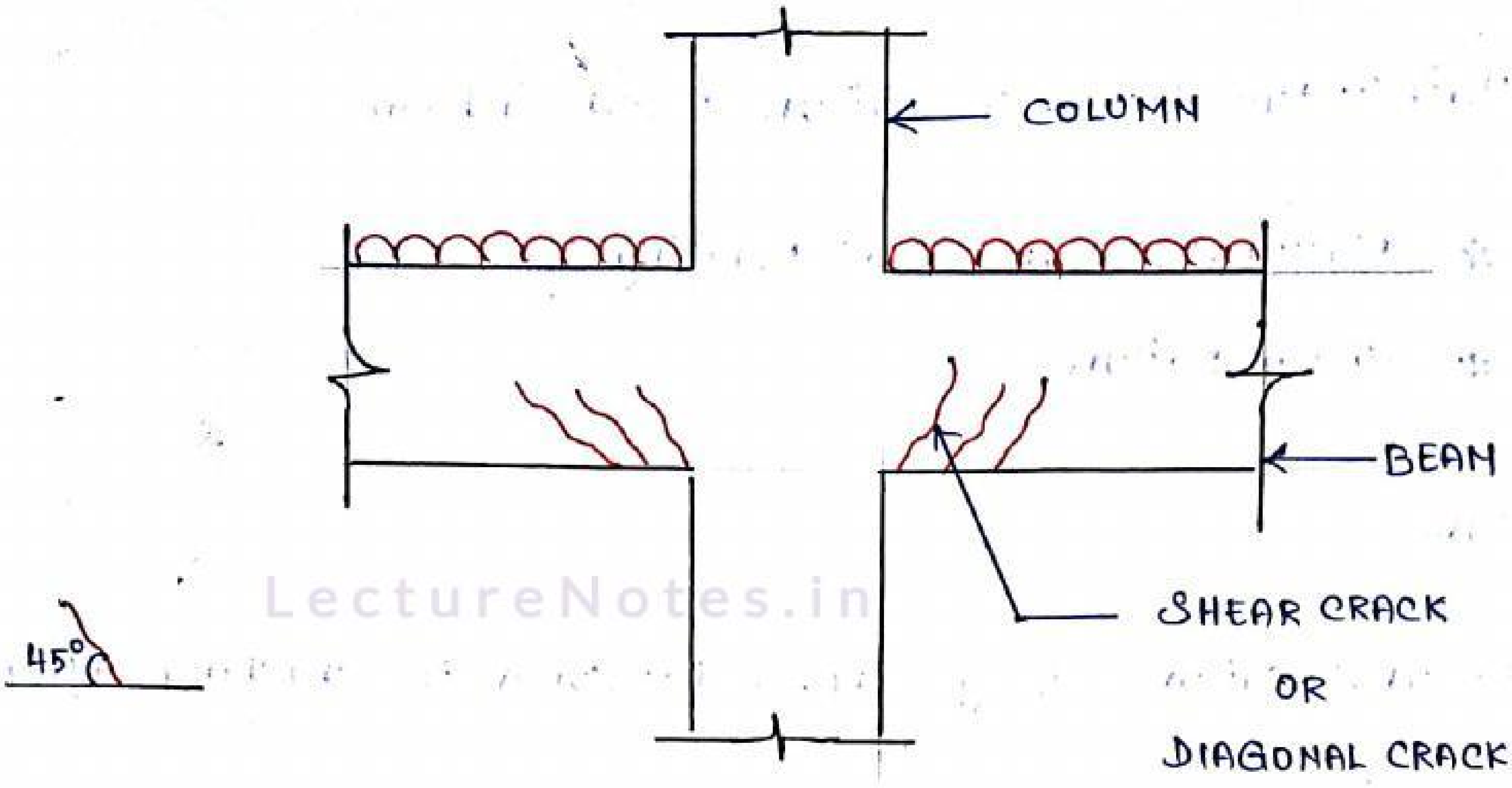
* The flexural failure is governed by Concrete Crushing after yielding of steel
* Flexural strength also known as Modulus of Rupture. Bond strength (or) Fracture strength is material property
* To avoid these type of failure, Main steel is provided at the bottom / Top of the beam Flexural failure occurs at Mid span of beam



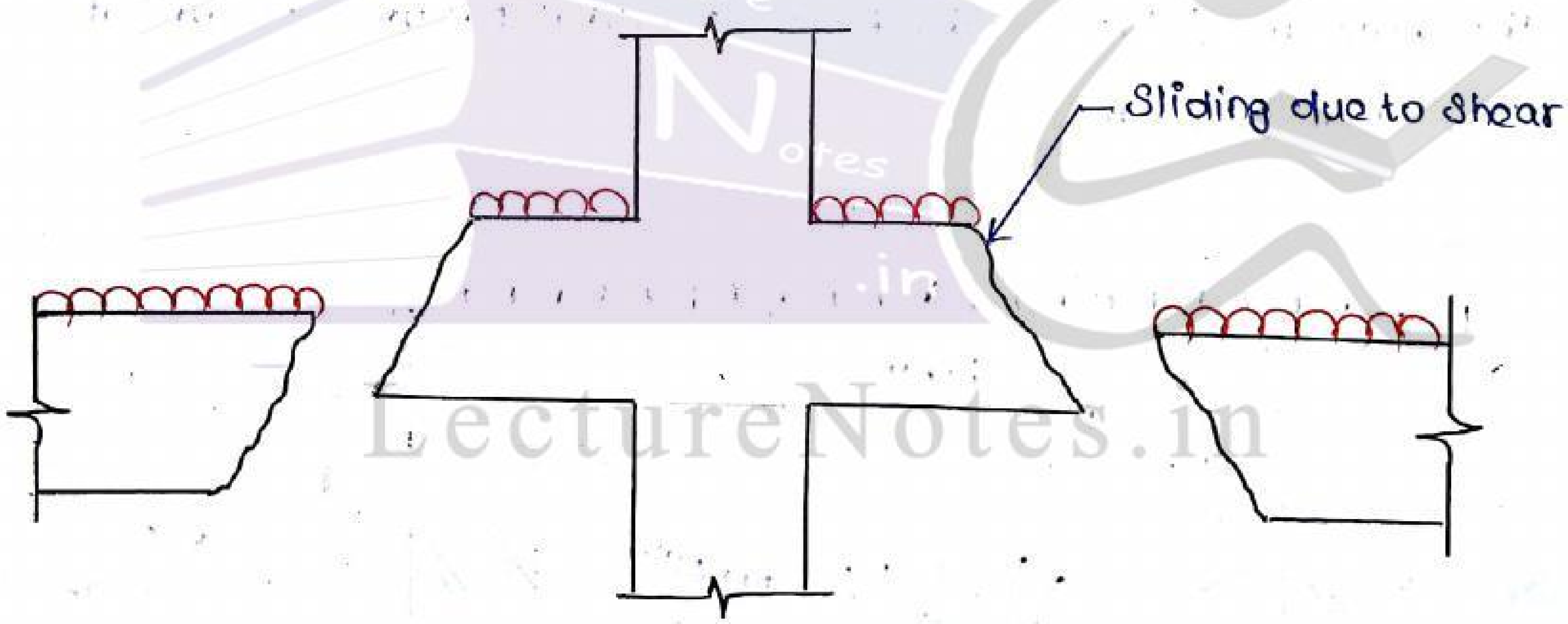


Shear Failure

Shear failure occurs on certain distance from the support. Shear strength is the strength of a material component against the type of yield or structural failure where the material (or) components fails at shear



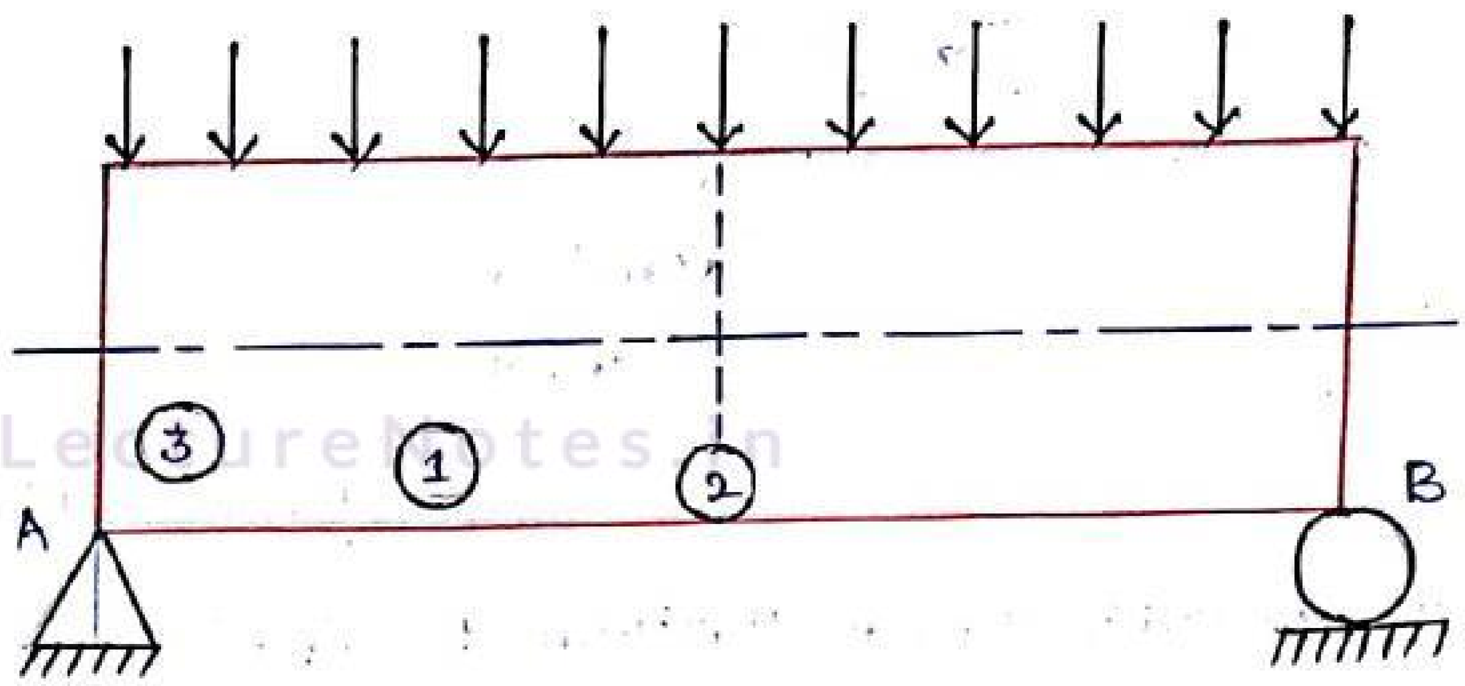
AT WARNING STAGE



AT FAILURE STAGE

* Shear Load is a force that tends to produce a sliding failure on a material along a plane that is parallel to the direction of the force
* Shear stress is maximum at 45° in the cross section of beam hence diagonal cracks in forms in shear failure and shear failure occurs at the one end of the beam where beam connected to column.
* To Avoid this type of failure stirrups are provided

Effect of shear in beams : Diagonal Tension

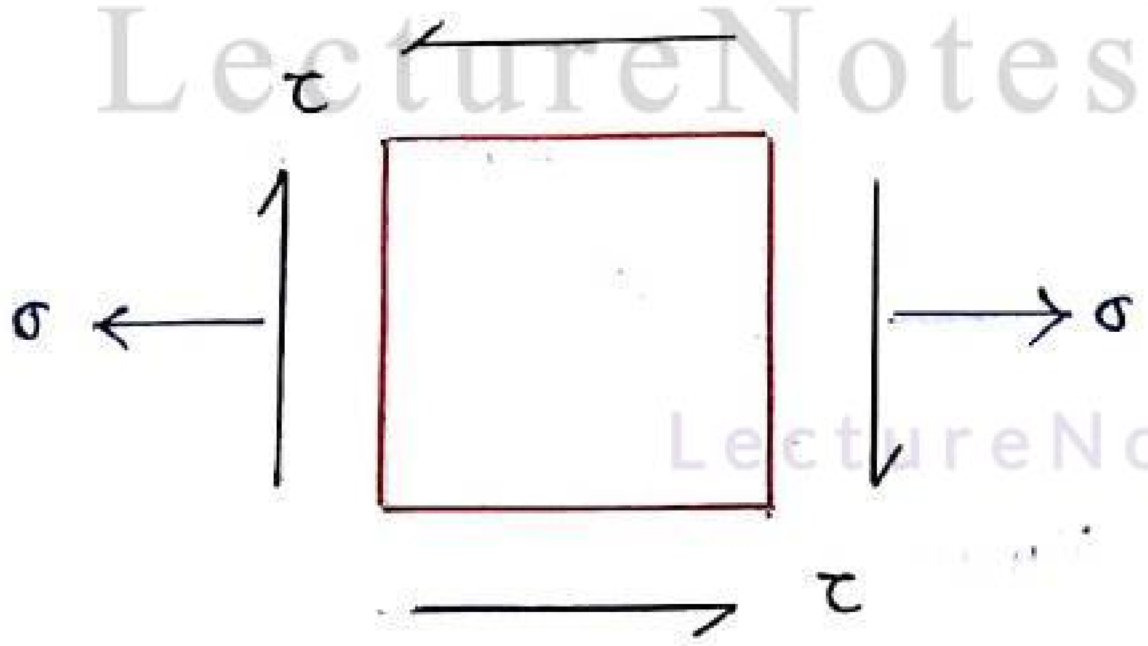


B·M = 0   MAX B·M

S·F MAX   S·F = 0

* Consider a beam AB subjected to Transverse load as show in Fig.
* The Maximum bending moment in this beam will be at Midspan
* The Maximum shear Force at Supports
* The beam is subjected to bending and shear stresses across the cross section

Element : 1

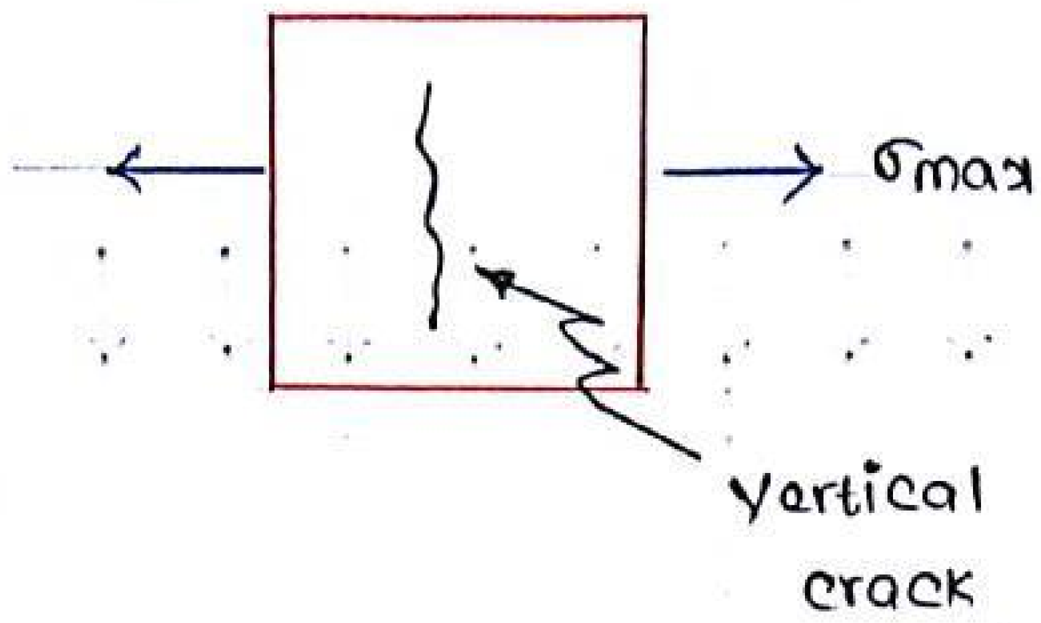


σ – Bending stress at section

τ – Shear stress at the section

* Let us consider a small element (1) from the Tensile zone of the beam
* It is subjected to Bending Tensile Stress (σ) as well as Shear Stress (τ)
* At the Midspan, the bending moment is maximum and shear force is zero

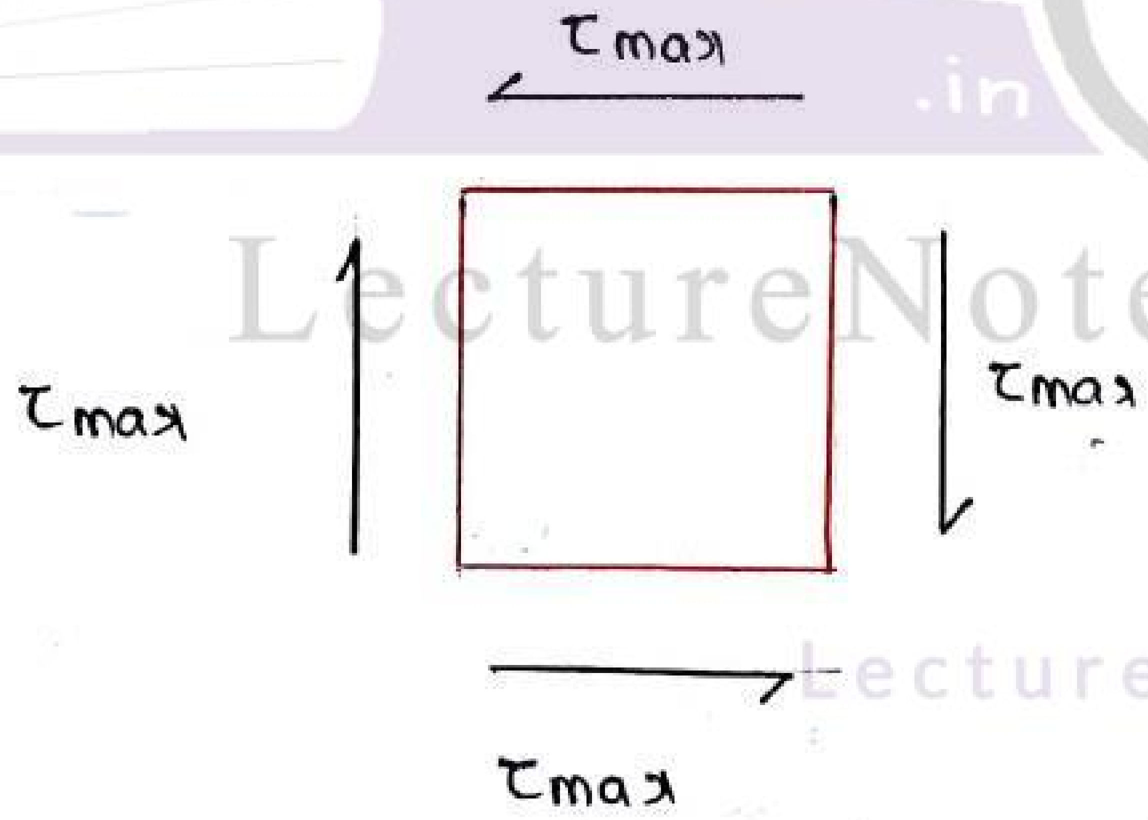
Element : 2



σ_{max} – Maximum Bending Stress

* Element (2) is subjected to Maximum Bending Tensile Stress only
* This Tensile Stress tries pull apart the section and Crack developed in vertical
* At the Support the Bending moment is zero and Shear force is maximum.

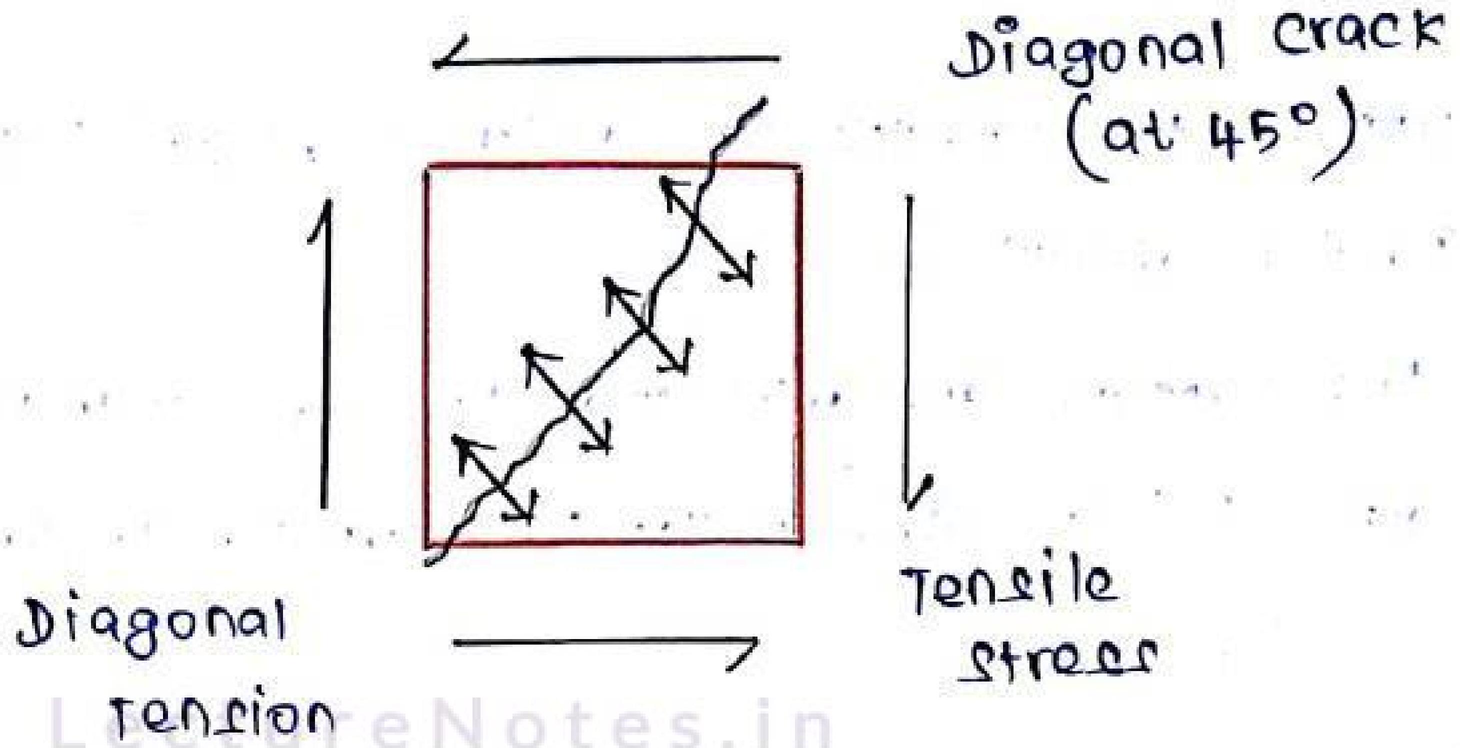
Element : 3



σ_{max} – Maximum bending Stress

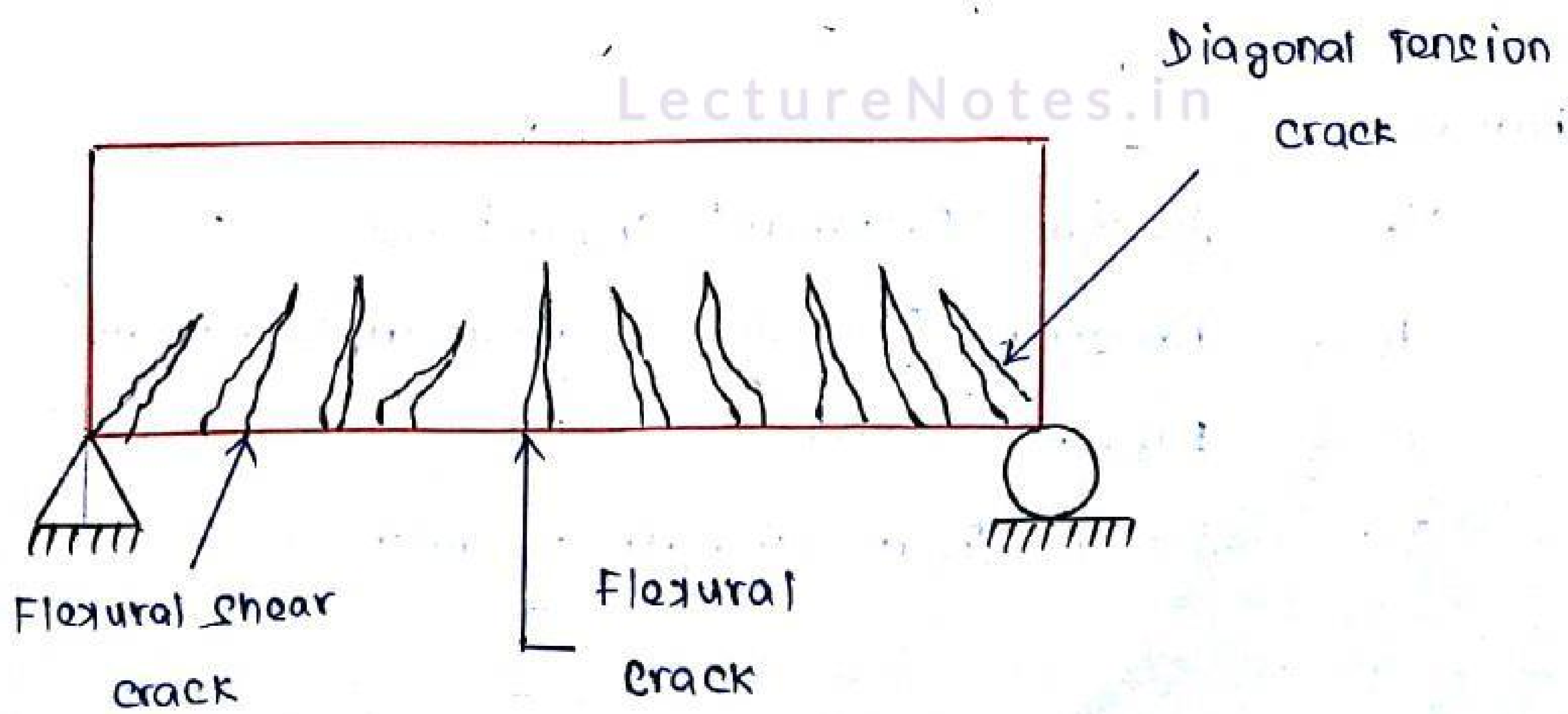
* Element (3) is subjected to Maximum Shear Stress and No bending Stresses
* Due to this stress Condition the diagonal of the element is subjected to Tensile stresses
* As the Concrete is very Weak in tension it splits along the diagonal

(at 45°) and develops crack.



* The Tension which is caused in Tensile zone of the beam due to Shear at or Near the Supports is called Diagonal Tension
* The diagonal Tension results in crack at 45°
* Concrete is quite strong in Shear but the diagonal Tension which is caused by Shear.
* Diagonal Tension cannot resisted by concrete alone
* So Shear reinforcement is provided in RCC beams to take up diagonal Tension and prevent cracking of beams.

Crack pattern in S.S.B



* Near the Mid span the crack will be vertical (Flexural crack due to bending alone

* Near the support the cracks are inclined at 45° (Shear or Diagonal Tension crack)

* In between the support and mid span the cracks inclination vary from 45° to 90° Gradually (Flexural - Shear cracks)

Logic of Stirrups

In Stirrups are provided in Rcc member, then only diagonal (between two stirrups) will be subjected to compression to arrest diagonal crack developed in Rcc members.

Nominal Shear Stress - Average Shear Stress - Adopted for Rcc design - Adopted for Shear reinforcement - Adopted for checking Shear Stress in slabs

Suggested by (τ_v)

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

Where,

V_u - Design (Factored) Shear Force

b - breadth (b_w for T-beam and L-beam)

d - Effective depth

τ_c - Design Shear Strength of concrete

(a) If $\tau_v < \tau_c$:

- $\frac{V_u}{bd} < \tau_c$ (Pg. No: 73) Values depends upon concrete grade &

Percentage of steel (Pt) $= \frac{A_{st}}{bd} \times 100$

No need to design stirrups

(b) If $\tau_v > \tau_c$

$\frac{V_u}{bd} > \tau_c$

Stirrups must be designed

LIMIT STATE OF COLLAPSE :- SHEAR

Refer IS: 456-2000 Pg. No: 72

Nominal Shear Stress

In beam of uniform depth

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

Where

τ_v = Nominal shear stress

V_u = Design shear force

d = Effective depth

b = Breadth of the beam (b_w for T-Beam and L beam)

Beams of Varying depth

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

Where

τ_v, V_u, b and d are the Same Notation

M_u = Bending Moment at the section

β = Angle between the Top and the bottom edges

Minimum Shear Reinforcement

When $\tau_v < \tau_c$ Minimum Shear Reinforcement Shall be provided

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

Pg. No: 48
Cl. No: 26.5.1.6

Where

A_{sv} = Total c/s area of Stirrups legs effective in shear.

S_v = Stirrup spacing along the length of the beams

b = breadth of the beam

f_y = characteristic Strength of the stirrup reinforcement which shall Not be taken greater than 415 N/mm^2

The Minimum shear reinforcement is provided for the following

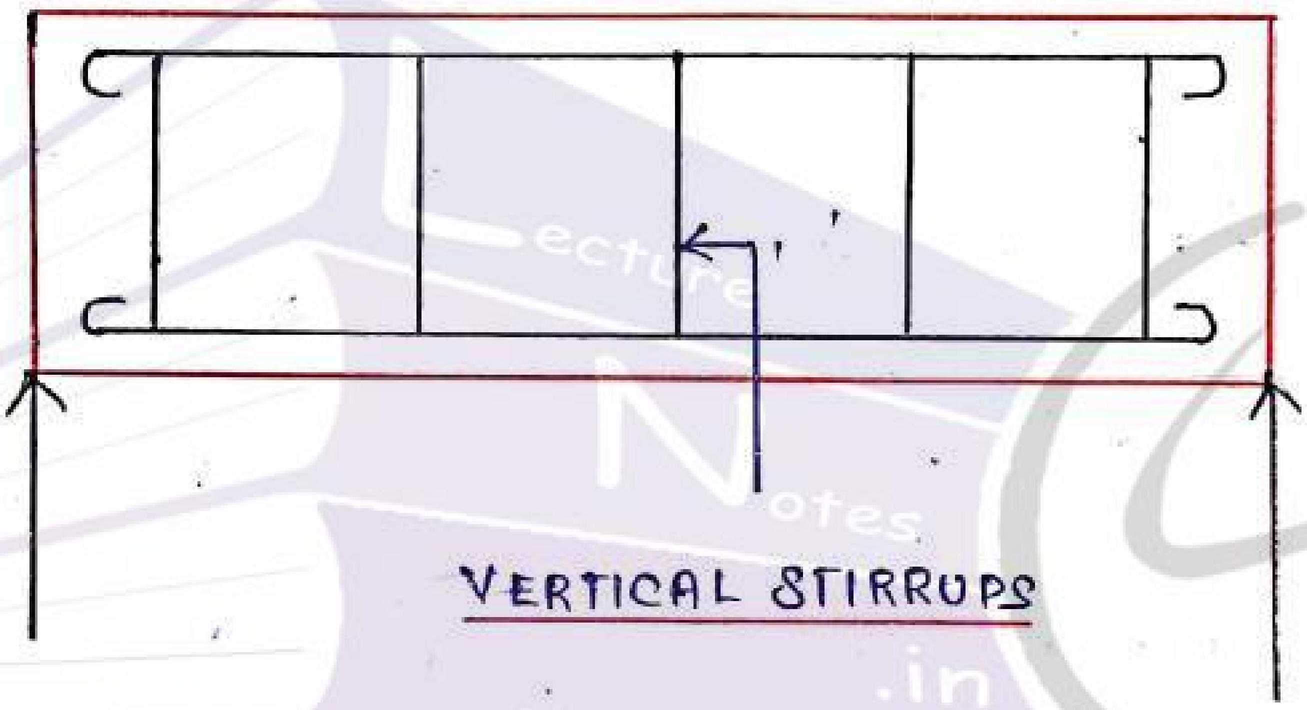
* To Hold the reinforcement in place when concrete is poured
* Any sudden failure of beams is prevented if concrete cover bursts and the bond to the Tension steel is lost

Design of Shear Reinforcement

- When $\tau_v > \tau_c$, shear reinforcement shall be provided in any of the three following forms

  * Vertical Stirrupps
  * Bent up bars along with Stirrups
  * Inclined Stirrups

Vertical Stirrups



VERTICAL STIRRUPS

(i) Cutting Length of Rectangular Stirrups

= 2(a+b) + 2 Number of Hooks − 3 No. of 90° bends − 2 No of 135° bend

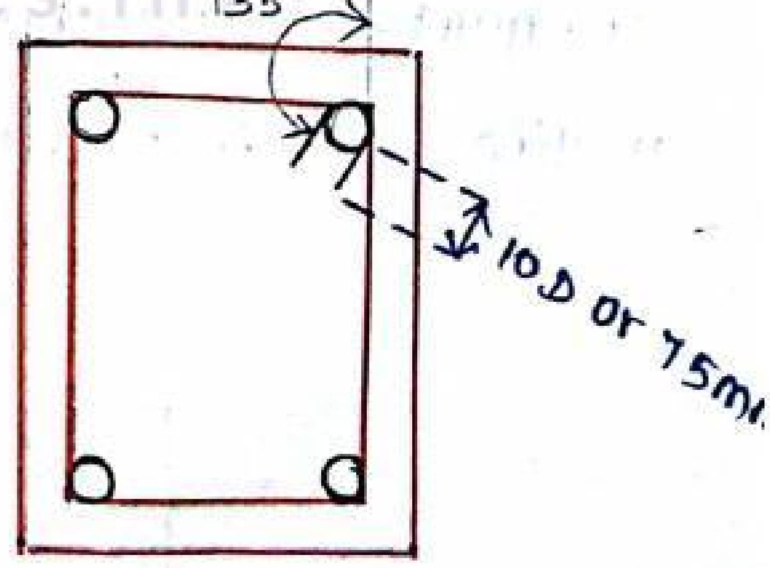
clear cover

$a = 600 - (2 \times 25) = 550$

$b = 200 - (2 \times 25) = 150$

$= 2(550 + 150) + (2 \times 10d) - (3 \times 2d) - (2 \times 3d)$

$= 1464$ mm



45° bend − 1d

90° bend − 2d

135° bend − 3d

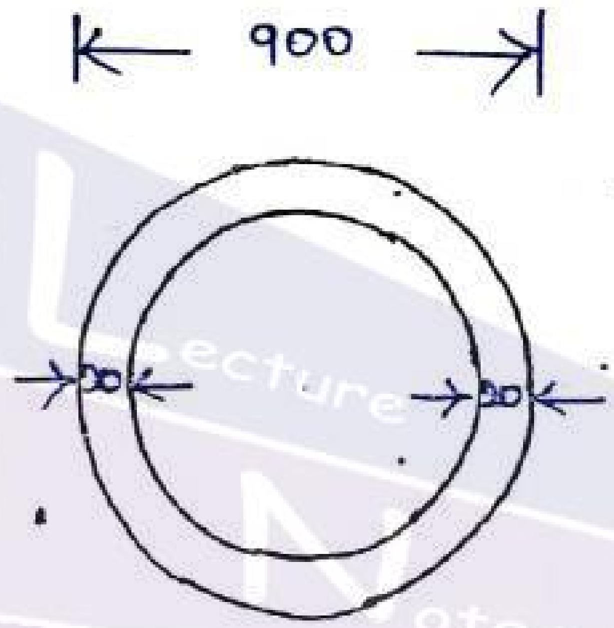
(ii) Cutting Length of Square Stirrups

$= 4a + 2$ Number of hooks $- 3$ Number of 90° $- 2$ Number of 135° bend

(iii) Cutting Length of Circular Stirrups

= Circumference of Ring + 2 Number of Hooks − 2 No. of 135° bend

$= 2\pi r + (2 \times 10d) - (2 \times 3d)$

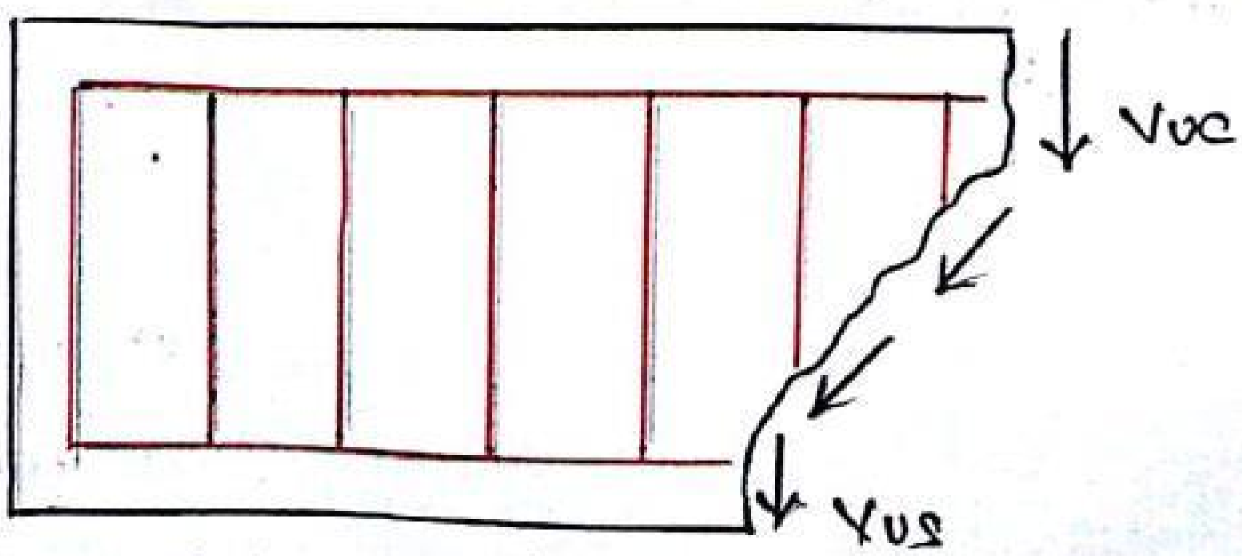


$d = 900 - 40$ $\quad \therefore \left(r = \frac{d}{2}\right)$

$= 860\,mm$

Maximum Spacing of Shear Reinforcement

The Maximum Spacing of shear reinforcement measured along the axis of the members shall not exceed , 0.75d for Vertical Stirrups and d for Inclined Stirrups at 45° where the effective depth of section



$$V_u = V_{uc} + V_{us}$$

- Where

$$V_{uc} = \tau_c \cdot b \cdot d$$

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

Where

V_{us} = Shear resisted by Concrete

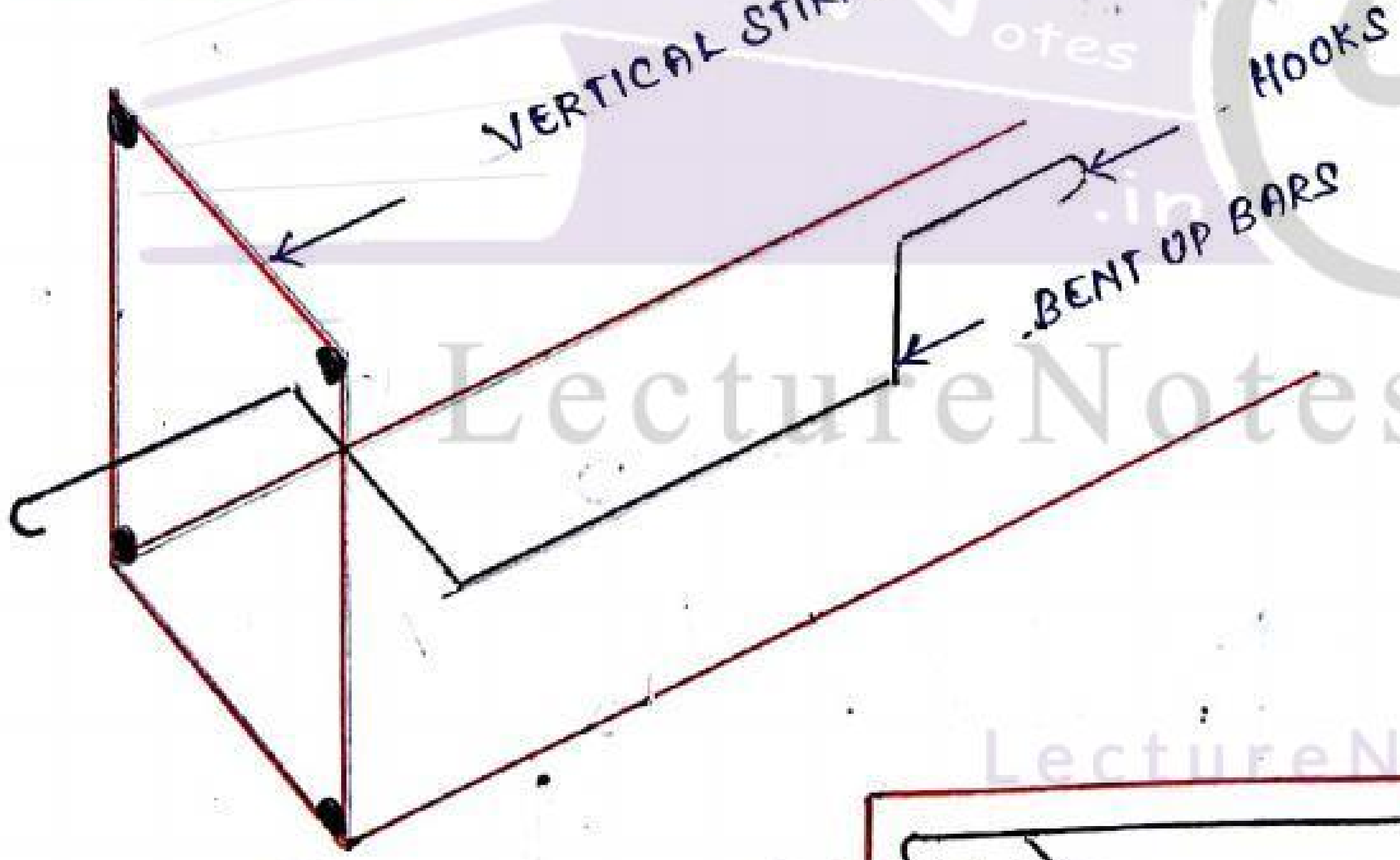
τ_c = Design Shear Strength of the Concrete

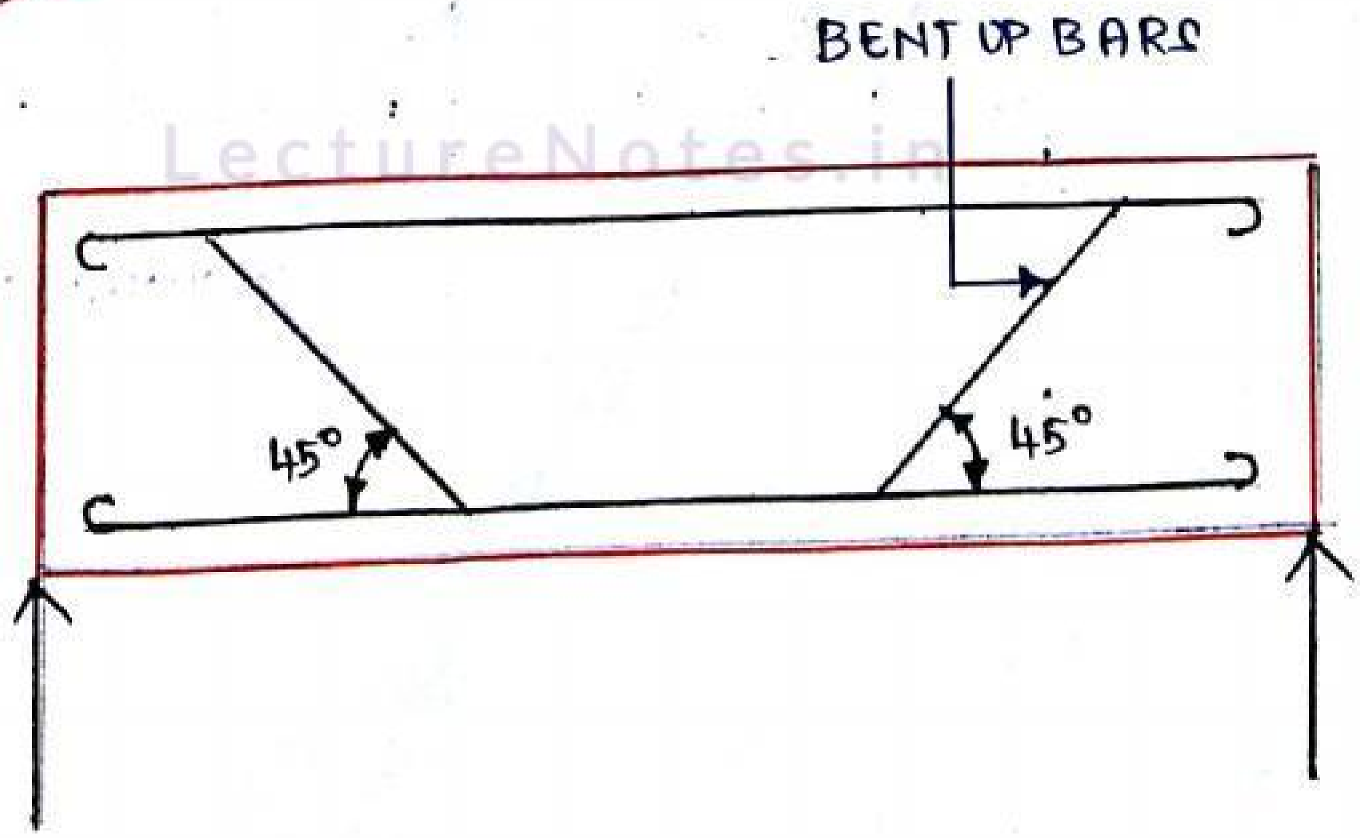
V_{us} = Balancing Shear (or) Shear resisted by Stirrups

A_{sv} = Total c/s area of Stirrups Legs (or) bent up bars

S_v = Spacing of the Stirrups along the length of the member

Bent - Up Bars





At Support bent up bars (crank up bars) are provided for Two purposes

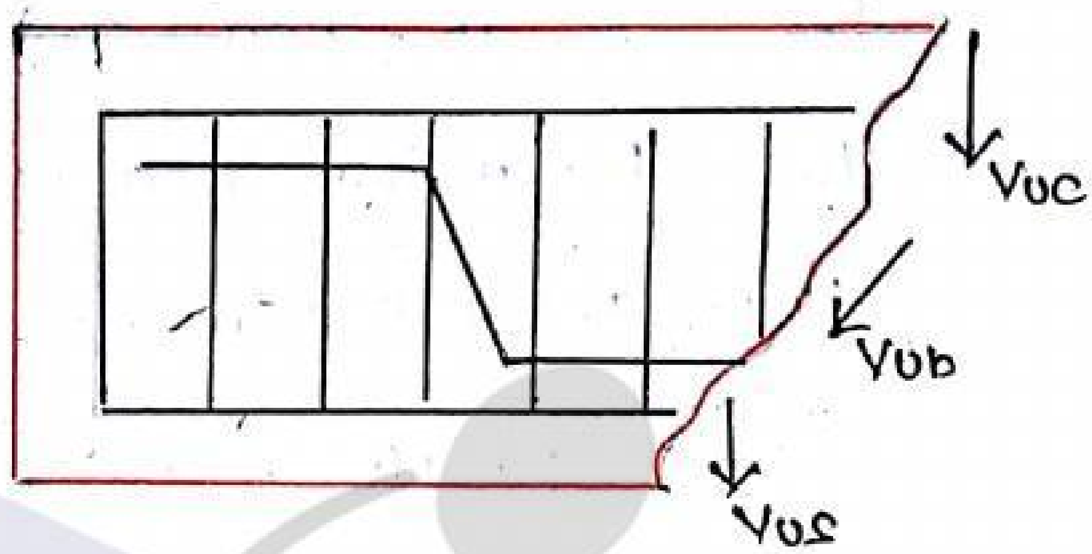
* To Resist Shear Force at Support

To resist diagonal Tension in concrete produced by principal stress along principal stress

* To Resist hogging BM at the Support due to bent up bars at Support, the Spacing of Stirrups Increases

$V_u = V_{uc} + V_{us} + V_{ub}$

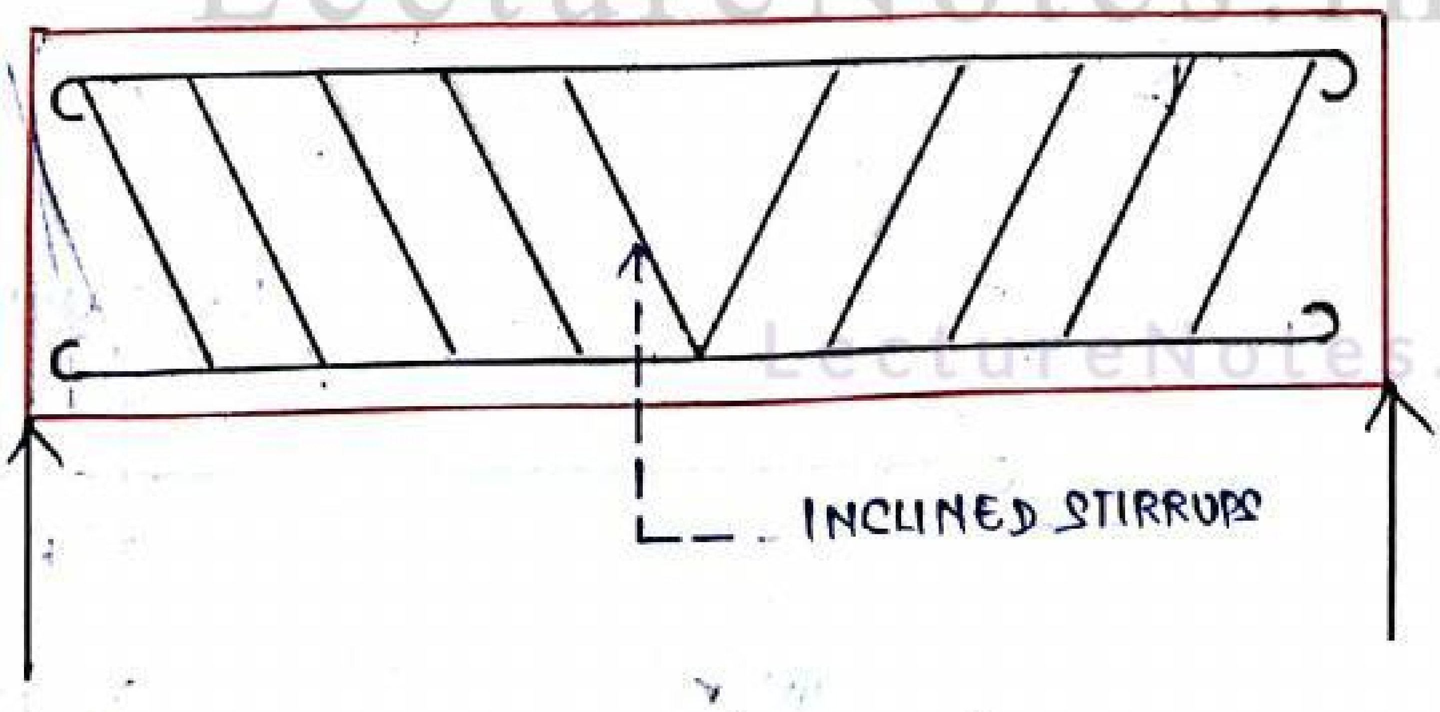
$V_{ub} = 0.87 f_y + f_y A_{sb} \sin\alpha$



Where

α = Angle b/w the Inclined Stirrups (or) Bent up bars and the axis of the member Not less than 45°

Inclined Stirrups

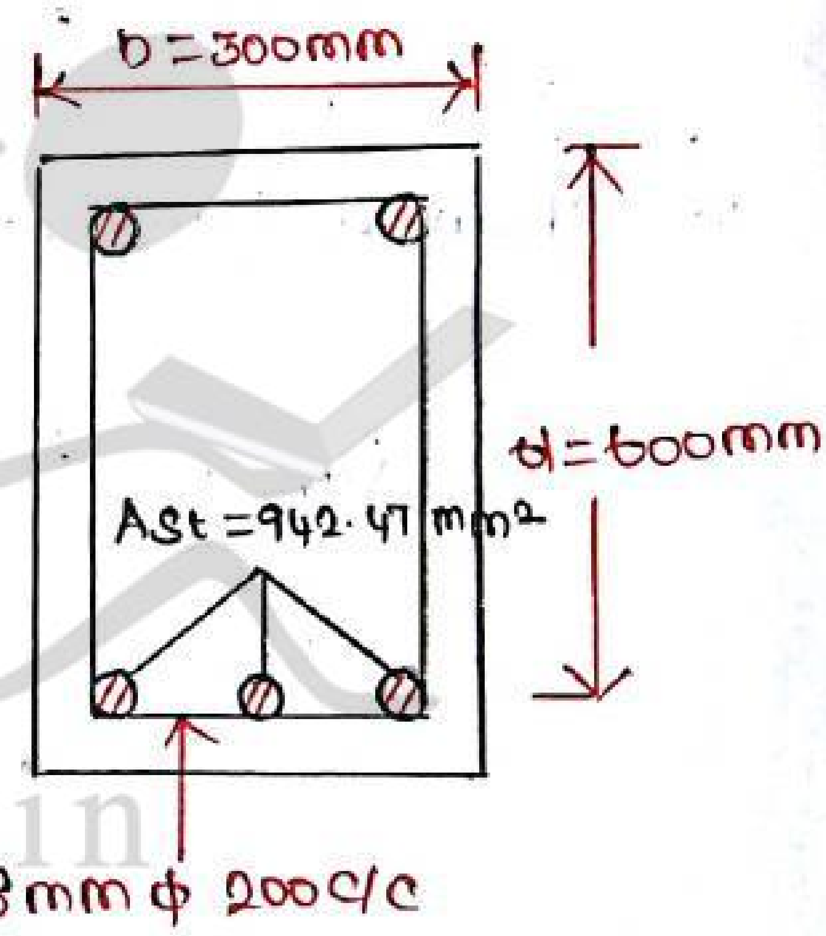


Problem on Analysis of Shear

Prblm. No: 1

A Reinforced Concrete beam has a Support Section with a width of 300mm and Effective depth of 600mm the support Section is reinforced with 3 bars of 20mm diameter at an effective depth of 600mm. 8mm diameter 2 legged Stirrups at a Spacing of 200mm is provided as shear reinforcement near supports using M20 Grade concrete and Fe 415 HYSD bars, Estimate the shear strength of a Support Section

Given data



$b = 300mm$

$d = 600mm$

$A_{st} = 3 \times \frac{\pi}{4} \times 20^2 = 942.47 mm^2$

$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.53 mm^2$

$S_v = 200mm$, $f_{ck} = 20 N/mm^2$

$f_y = 415 N/mm^2$

Solution

(a) percentage Reinforcement (P_t)

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 942.47}{300 \times 600} = 0.52$$

Refer Table-19 Pg. No: 73 IS:456

$$\frac{(0.75-0.50)}{(0.75-0.52)} = \frac{(0.56-0.48)}{(0.56-X)}$$

P_t	τ_c
0.50 →	0.48
0.56 →	?
0.75 →	0.56

$$\boxed{\tau_c = 0.48\ N/mm^2}$$

$$V_u = V_{uc} + V_{us}$$

(b) Shear resisted by concrete (V_{uc})

$$V_{uc} = (\tau_c bd) = (0.48 \times 300 \times 600) \times 10^{-3}$$

$$\boxed{V_{uc} = 86.4\ kN}$$

(c) Shear resisted by stirrups (V_{us})

$$V_{us} = \frac{A_{sv}(0.87 \cdot f_y)\, d}{S_v} = \frac{100.53 \times 0.87 \times 415 \times 600}{200} \times 10^{-3}$$

$$\boxed{V_{us} = 108.88\ kN}$$

(d) Total Shear Resistance of Support Section

$$V_u = V_{uc} + V_{us}$$
$$= 86.4 + 108.88$$
$$\boxed{V_u = 195.28\ kN}$$

Prblm. No: 2

- A Reinforced Concrete beam of rectangular Section has a width of 250mm and an effective depth of 500mm the beam is reinforced with 4 bars of 25mm diameter on the tension side. Two of the tension bars are bent up at 45° near the Support Section. In addition the beam is provided with Two Legged Stirrups. If $f_{ck} = 25\ N/mm^2$ and $f_y = 415\ \frac{N}{mm^2}$ Estimate the Ultimate Shear Strength of the Support Section

Given data

$b = 250mm$

$d = 500mm$

$A_{st} = 2 \times \frac{\pi}{4} \times 25^2 = 981.74mm^2$

$A_{sb} = 981.74mm^2$

$\alpha = 45°$

$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.53mm^2$

$S_v = 150mm$, $f_{ck} = 25\ N/mm^2$, $f_y = 415\ N/mm^2$

Solution :

(a) Percentage Reinforcement

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 981.74}{250 \times 500} = 0.78$$

Refer Table - 19 (Pg. No: 73)

P_t	τ_c
0.75 →	0.56
0.78 →	?
1.00 →	0.62

$$\frac{(1.00 - 0.75)}{(1.00 - 0.78)} = \frac{(0.62 - 0.56)}{(0.62 - x)}$$

$$\boxed{\tau_c = 0.58\ N/mm^2}$$

(b) Shear resisted by Concrete

$$V_{uc} = (\tau_c \cdot b \cdot d) = (0.58 \times 10^{-3} \times 250 \times 500)$$

$$\boxed{V_{uc} = 72.5 \text{ KN}}$$

(c) Shear Resisted by Stirrups

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

$$= \left[\frac{100.53 \times 0.87 \times (415 \times 10^{-3}) \times 500}{150}\right]$$

$$\boxed{V_{us} = 120.98 \text{ KN}}$$

(d) Shear Resisted by Bent up bars

$$V_{ub} = A_{sb} (0.87 \times f_y) \sin\alpha$$

$$= 981.74 \times 0.87 \times (415 \times 10^{-3}) \times \sin 45^\circ$$

$$\boxed{V_{ub} = 250.63 \text{ KN}}$$

(e) Total Shear Resistance of Support Section

$$V_u = V_{uc} + V_{us} + V_{ub}$$

$$= 72.5 + 120.98 + 250.63$$

$$\boxed{V_u = 441.11 \text{ KN.}}$$

Prblm. No: 5

- A Reinforced Concrete beam of Rectangular Section 300mm wide is reinforced with four bars of 25mm diameter at an effective depth of 600mm the beam has to resist a factored Shear force of 400 KN at Support section Assuming $f_{ck} = 25\ N/mm^2$ and $f_y = 415\ N/mm^2$ Design Vertical Stirrups for the section

Given data

$b = 300mm$, $d = 600mm$, $A_{st} = 4 \times \frac{\pi}{4} \times 25^2 = 1963.49\ mm^2$

$V_u = 400 KN$, $f_{ck} = 25\ N/mm^2$, $f_y = 415\ N/mm^2$

Solution

Nominal Shear Stress

$$V_u = 400\ KN$$

$$\tau_v = \frac{V_u}{bd} = \frac{400 \times 10^3}{300 \times 600} = 2.22\ N/mm^2$$

Shear Resisted by Concrete

$$P_t = \frac{100\ A_{st}}{bd} = \frac{100 \times 1963.49}{300 \times 600} = 1.09$$

Refer Table-19 Pg. No: 73 in IS: 456-2000

$\tau_c = 0.64\ N/mm^2$

$\tau_v > \tau_c$

Hence stirrups are to be designed

$V_{uc} = (\tau_c b d) = (0.64 \times 10^{-3} \times 300 \times 600)$

$\boxed{V_{uc} = 115.2\ KN}$

P_t	τ_c
1.00 →	0.62
1.09 →	?
1.25 →	0.67

$$\frac{(1.25 - 1.00)}{(1.25 - 1.09)} = \frac{(0.67 - 0.62)}{(0.67 - x)}$$

Balance shear is given by

$$V_u = V_{uc} + V_{us}$$

$$V_{us} = V_u - V_{uc}$$

$$= (400 - 115.2)$$

$$\boxed{V_u = 284.8 \text{ KN}}$$

Design of Vertical stirrups

Using 10 mm diameter 2 legged vertical stirrups spacing is given by

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

$$= \frac{0.87 \times 415 \times \left(\frac{\pi}{4} \times 10^2 \times 2\right) \times 600}{284.8 \times 10^3}$$

$$= 119.48 \text{mm}$$

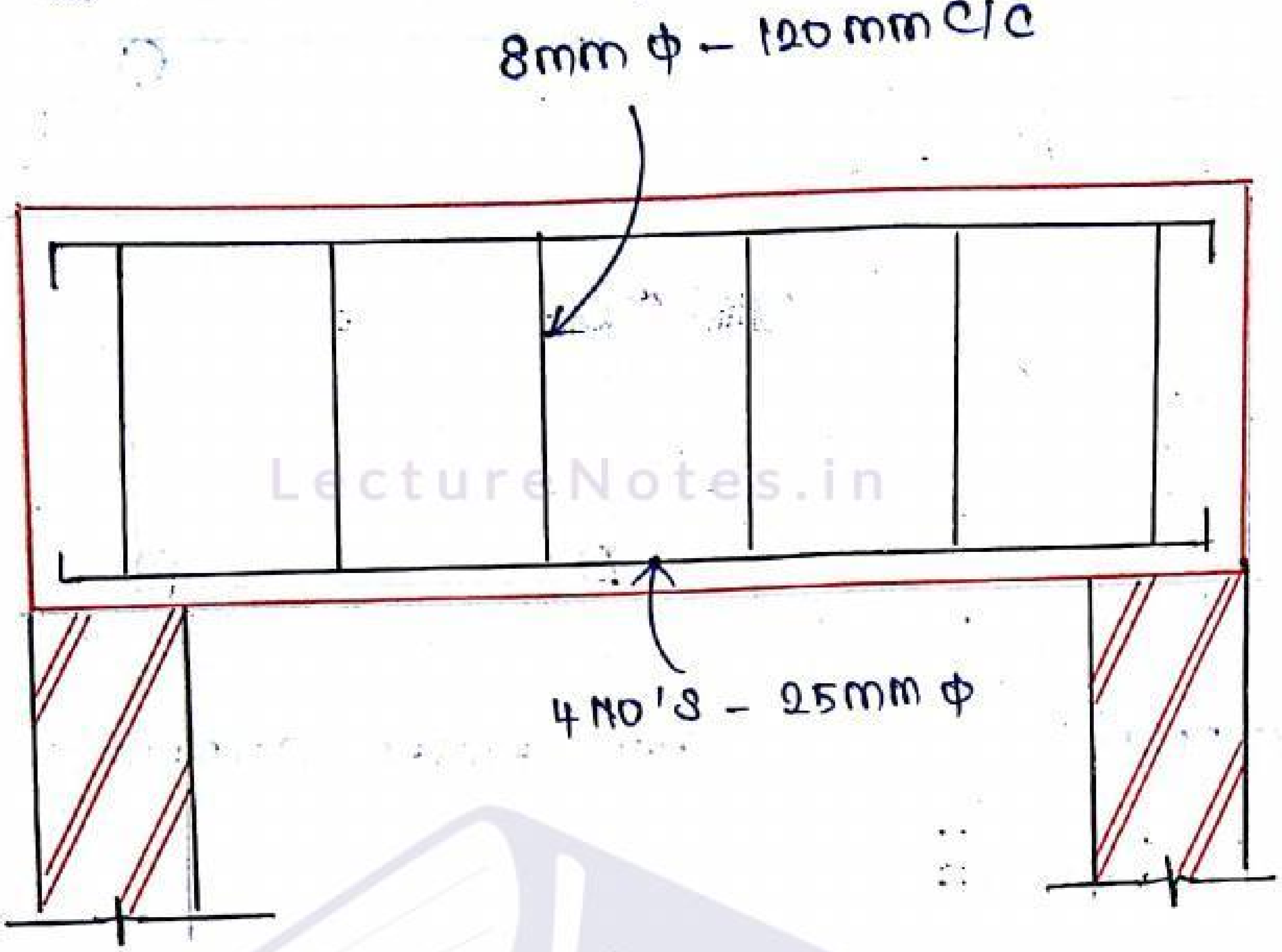
$S_{v\,max} = 0.75\,d \Rightarrow 0.75 \times 600 \Rightarrow 450\text{mm}$

Also

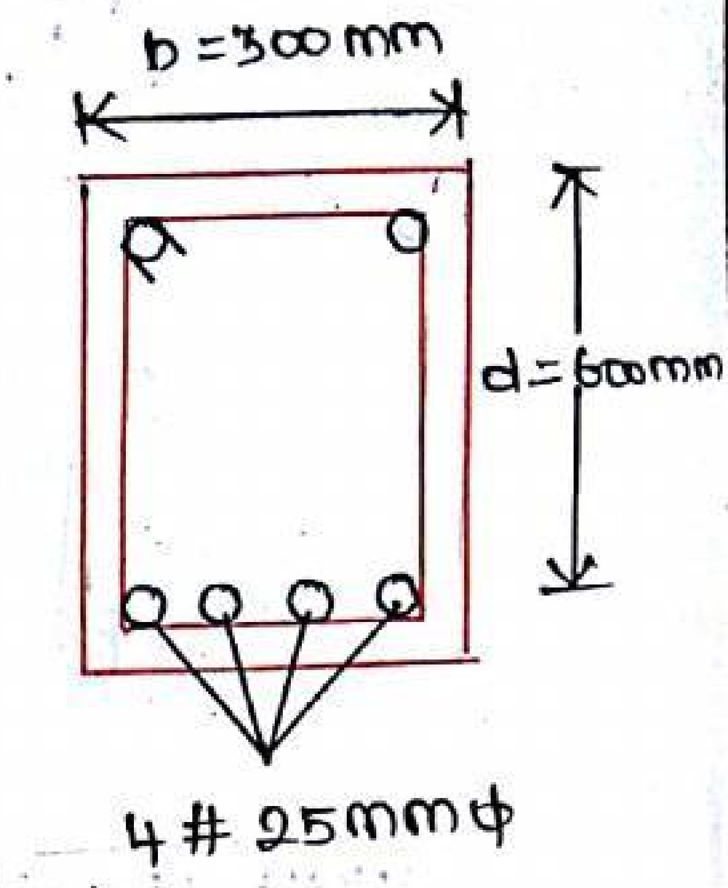
$$S_v \leq 300\text{mm}$$

Provide 10 mm dia 2 Legged vertical stirrups at 120 mm centres at Support Section.

Detail Reinforcement Of Stirrups

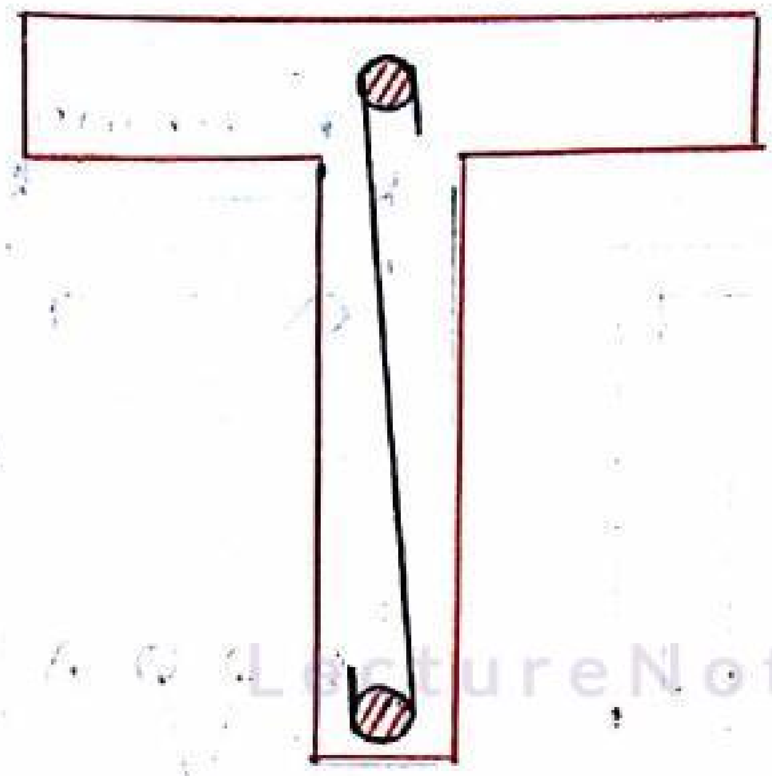


LONGITUDINAL SECTION

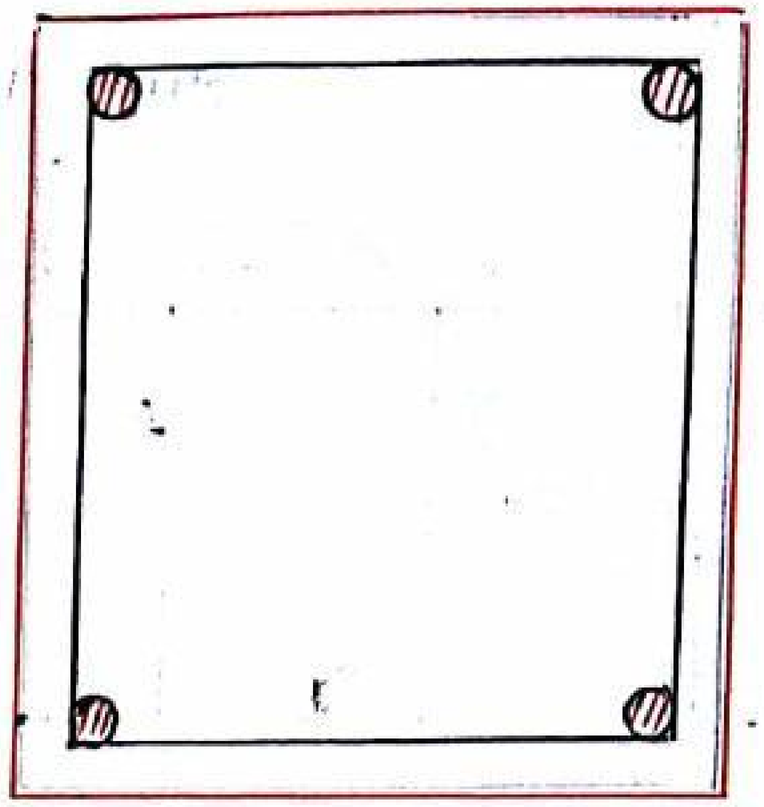


CROSS SECTION

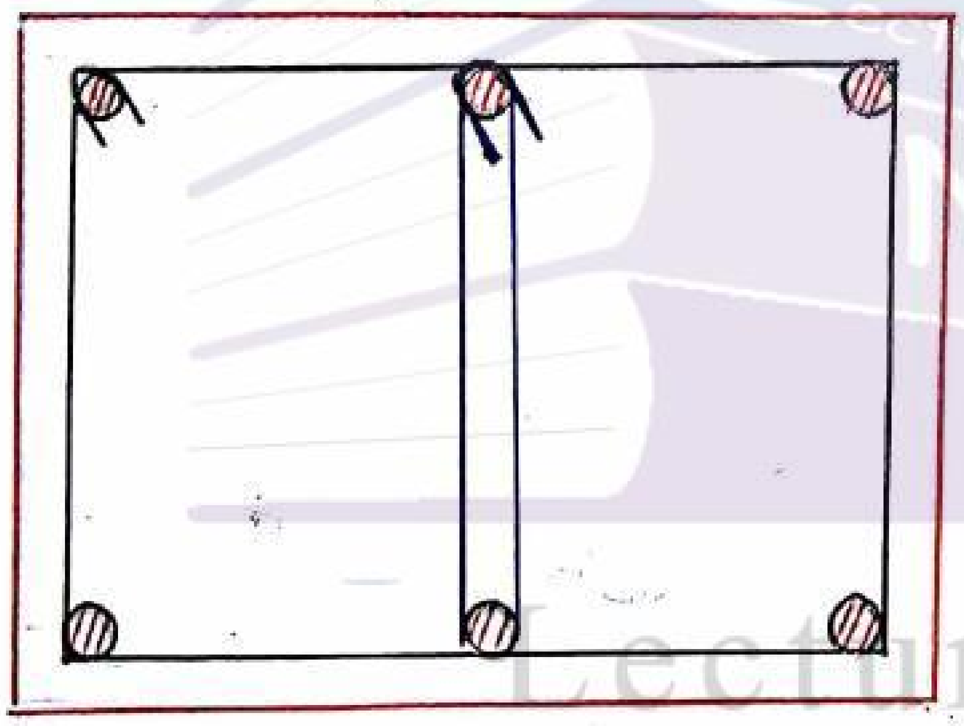
TYPES OF LEGGED STIRRUPS



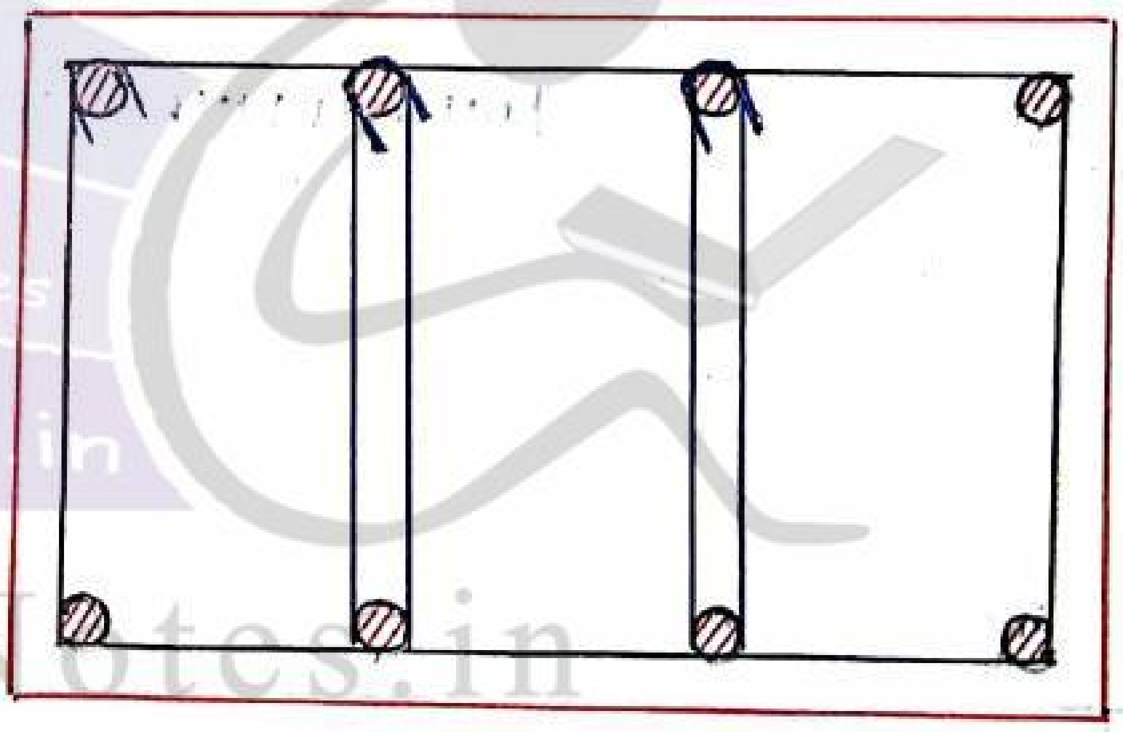
SINGLE LEGGED STIRRUPS



TWO LEGGED STIRRUPS



FOUR LEGGED STIRRUPS



SIX LEGGED STIRRUPS

Design of Shear Reinforcement

Prblm. No:4

A beam 300mm × 600mm Effective carries an UDL of 75 KN/m Including the self weight Effective span is 6m It is reinforced with 3-25 φ out of those 5 bars, 2 bars are safely bent up Design Shear Reinforcement Use M 30 Grade of Concrete and Fe 415 steel.

Given data

Breadth of beam (b) = 300mm

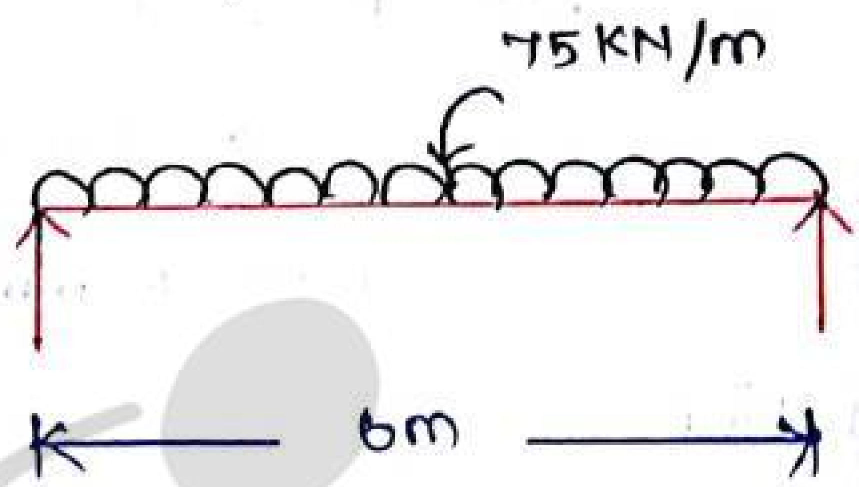
Effective depth (d) = 600 mm

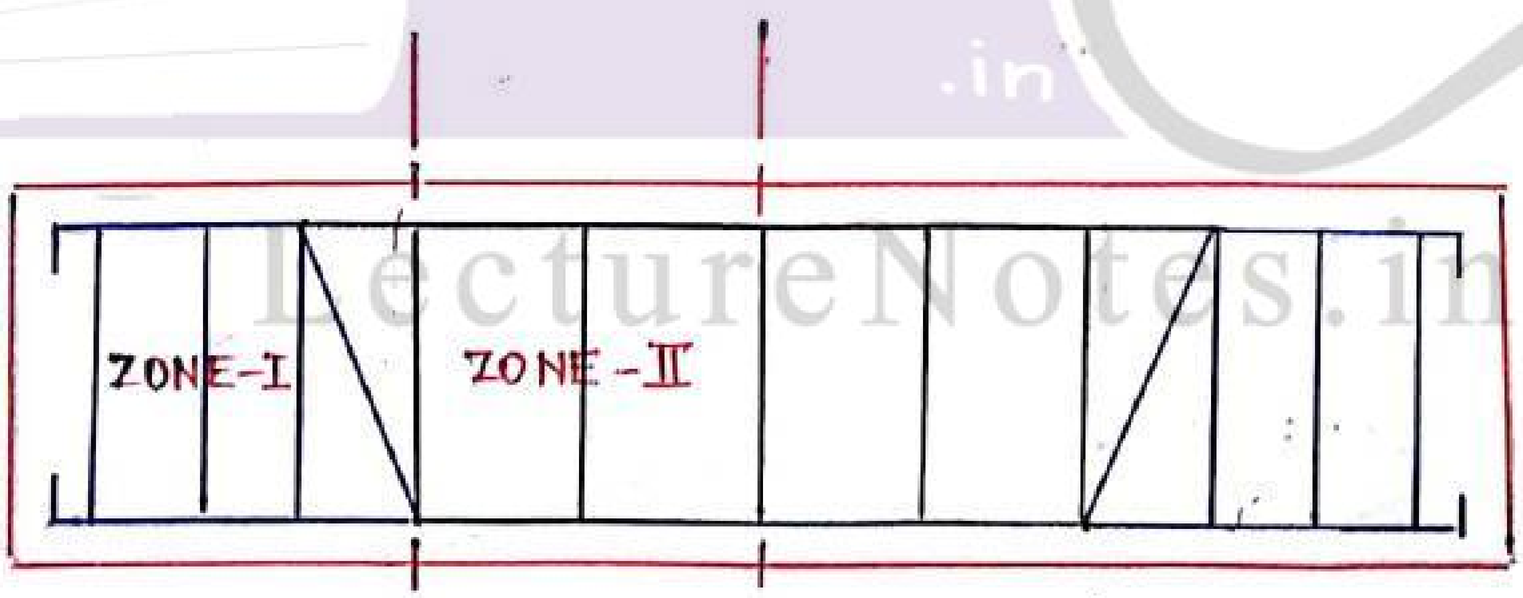
UDL (W) = 75 KN/m

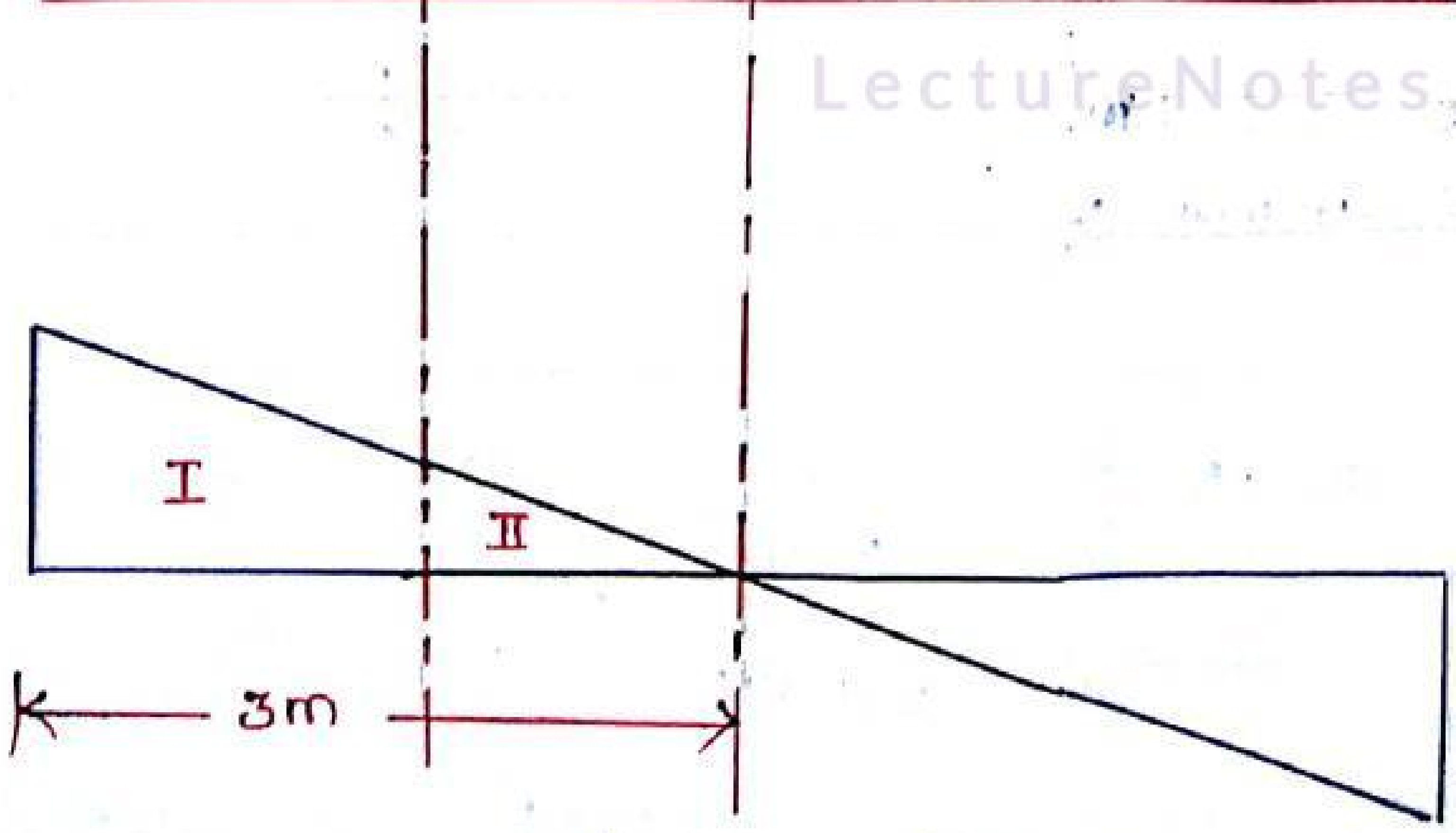
Effective span (L) = 6m

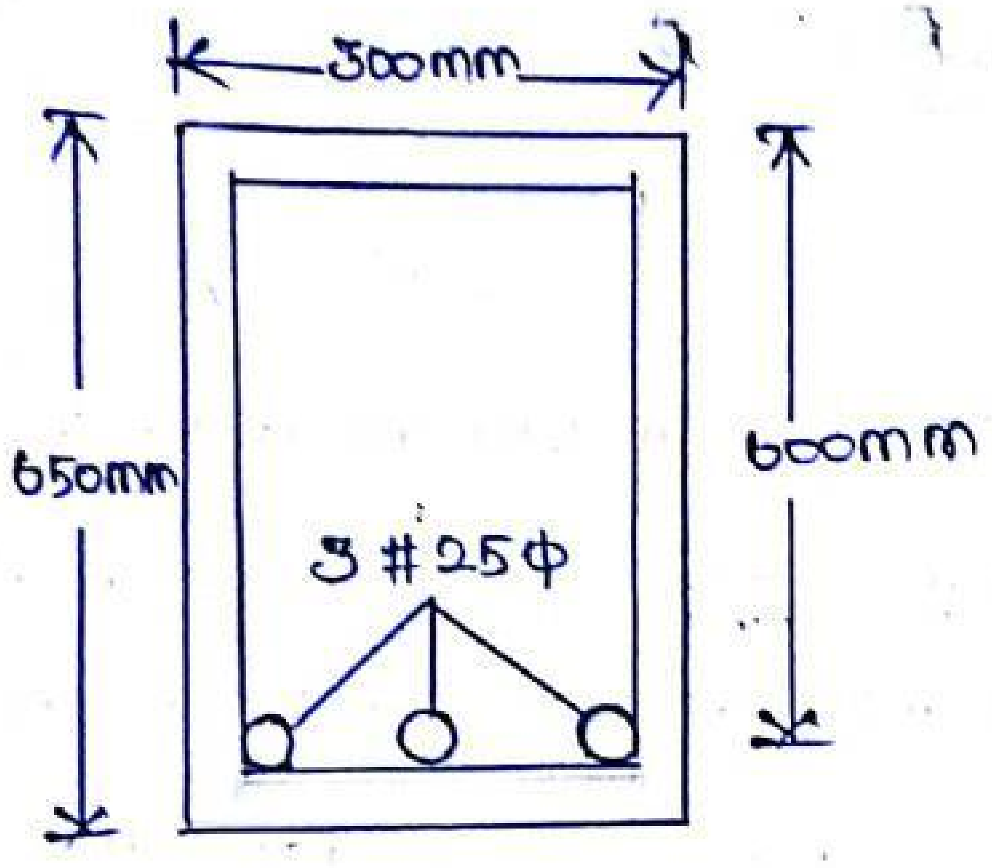
f_{ck} = 30 N/mm^2

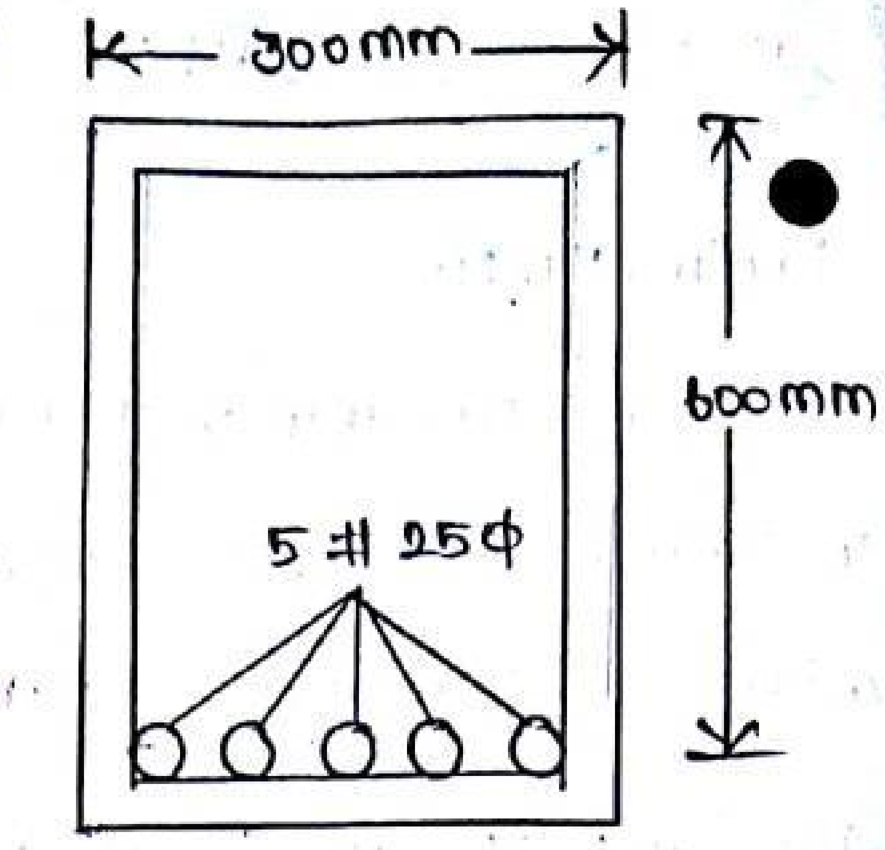
f_y = 415 N/mm^2











FOR SUPPORT SECTION

$$A_{st} = n \times \frac{\pi}{4} d^2$$
$$= 3 \times \frac{\pi}{4} (25)^2$$
$$= 1472.62 \, mm^2$$

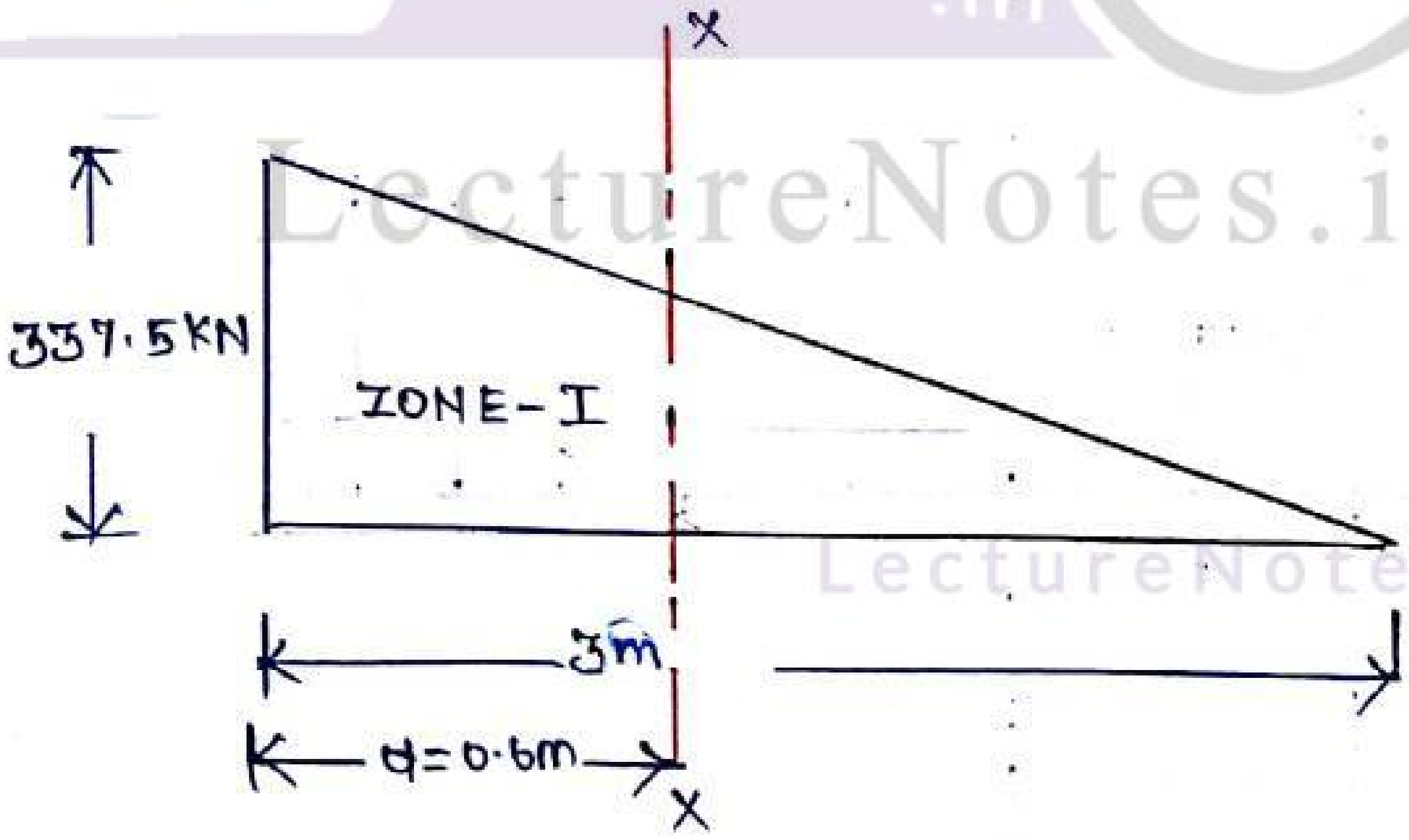
FOR MIDSPAN SECTION

$$A_{st} = n \times \frac{\pi}{4} d^2$$
$$= 5 \times \frac{\pi}{4} \times (25)^2$$
$$= 2454 \, mm^2$$

Solution

CASE-I

Let us assume the section x-x at a distance from support



$$600mm \xrightarrow{(\div 1000)} 0.6m$$

$$\frac{337.5}{3} = \frac{x}{3-0.6}$$

$$x = 270 \, KN$$

$$x = \boxed{V_{uc} = 270 \, KN}$$

$$W = 75\ KN/m$$

Design Ultimate Load $\Big\}$ $W_u = 1.5 \times 75\ KN/m$

$$= 112.5\ KN/m$$

Shear Force (V_u)

$$V_u = \frac{WL}{2}$$

$$= \frac{112.5 \times 6}{2}$$

$$\boxed{V_u = 337.5\ KN}$$

Percentage of Tension Reinforcement (P_t)

$$P_t = \frac{100 A_{st}}{bd}$$

$$= \frac{100 \times 1472.62}{300 \times 600}$$

$$\boxed{P_t = 0.81}$$

P_t	τ_c
0.75 →	0.59
0.81 →	?
1.00 →	0.66

$\boxed{\tau_c = 0.6\ N/mm^2}$ $\qquad \frac{(1.00 - 0.75)}{(1.00 - 0.81)} = \frac{(0.60 - 0.59)}{(0.60 - x)}$

Shear Force resisted by vertical Stirrups (V_{us})

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

$$= (270 \times 10^3) - 0.6 \times 300 \times 600$$

$$\boxed{V_{us} = 162\ KN}$$

Shear Force resisted by Bentup bars

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

$= 0.87 \times 415 \times 100.53 \times \sin(45°)$

$V_{us} = 256.65 \text{KN}$

(Capacity of Bent up bars)

Actual shear force resisted by Bent up bars

$= \frac{V_{us}}{2}$

$= \frac{162}{2}$

$= 81 \text{KN}$

Shear Force resist by Vertical stirrups

$= V_{us} - \frac{V_{us}}{2}$

$= 162 - 81$

$= 81 \text{KN}$

$V_{us} = 256.65 \text{KN} > \frac{V_{us}}{2} = 81 \text{KN}$

$V_{us} > \frac{V_{us}}{2}$

Bent up bar is safe

Check for Minimum Spacing

(i) $S_v = \frac{0.87 \times f_y \times A_{sv} \times d}{V_{us}}$

$= \frac{0.87 \times 415 \times 100.53 \times 600}{81 \times 10^3}$

$S_v = 268.86\text{mm} \simeq 250\text{mm}$

(ii) $S_v = \dfrac{0.87 \times f_y \times A_{sv}}{0.4 b}$

$= \dfrac{0.87 \times 415 \times 100.53}{0.4 \times 300}$

$\boxed{S_v = 302.46 mm}$

Bent up bar distance

$= d\left(1 + \dfrac{1}{\tan 45^\circ}\right)$

$= 600\left(1 + \dfrac{1}{\tan 45^\circ}\right)$

$= 1200 mm$

(iii) $S_v = 0.75 d$

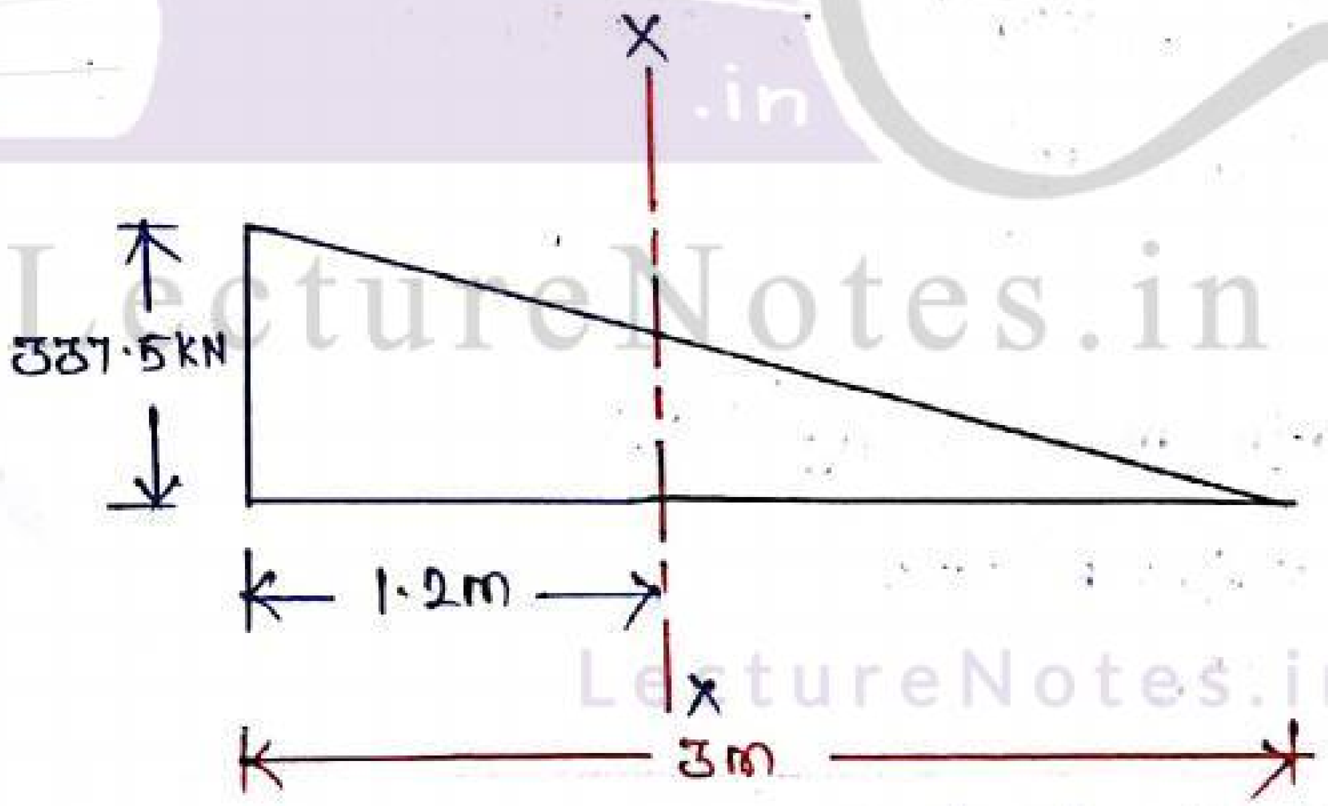
$= 0.75 \times 600$

$\boxed{S_v = 450 mm}$

Provide 2 Legged Stirrups 8mm ϕ @ 250 mm c/c

CASE-II

Let us assume the section is x-x at a distance from Mid span



$\dfrac{337}{3} = \dfrac{x}{3 - 1.2}$

$x = 202.2 KN$

$x = \boxed{V_u = 202.2 KN}$

Percentage of Tension Reinforcement (P_t)

●

$$P_t = \frac{100 A_{st}}{bd}$$

$$= \frac{100 \times 2454}{300 \times 600}$$

$$= 1.36$$

P_t	τ_c
1.25 →	0.71
1.36 →	?
1.50 →	0.76

$$\frac{(1.50-1.25)}{(1.36-1.25)} = \frac{(0.76-0.71)}{(0.76-x)}$$

$\tau_c = 0.73\ N/mm^2$

Shear resisted by Vertical Stirrups (V_{us})

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

$$= (202.2 \times 10^3) - 0.73 \times 300 \times 600$$

$$\boxed{V_{us} = 70.8\ KN}$$

Check for Minimum Spacing (S_v)

(i) $S_v = \dfrac{0.87 \times f_y \times A_{sv} \times d}{V_{us}}$

$$= \frac{0.87 \times 415 \times 100.53 \times 600}{70.8 \times 10^3}$$

$$\boxed{S_v = 307.59\ mm}$$

(ii) $S_v = \dfrac{0.87 \times f_y \times A_{sv}}{0.4b}$

$$= \frac{0.87 \times 415 \times 100.53}{0.4 \times 300}$$

$$= 302.46mm \nless 300mm$$

(iii) $S_v = 0.75d$

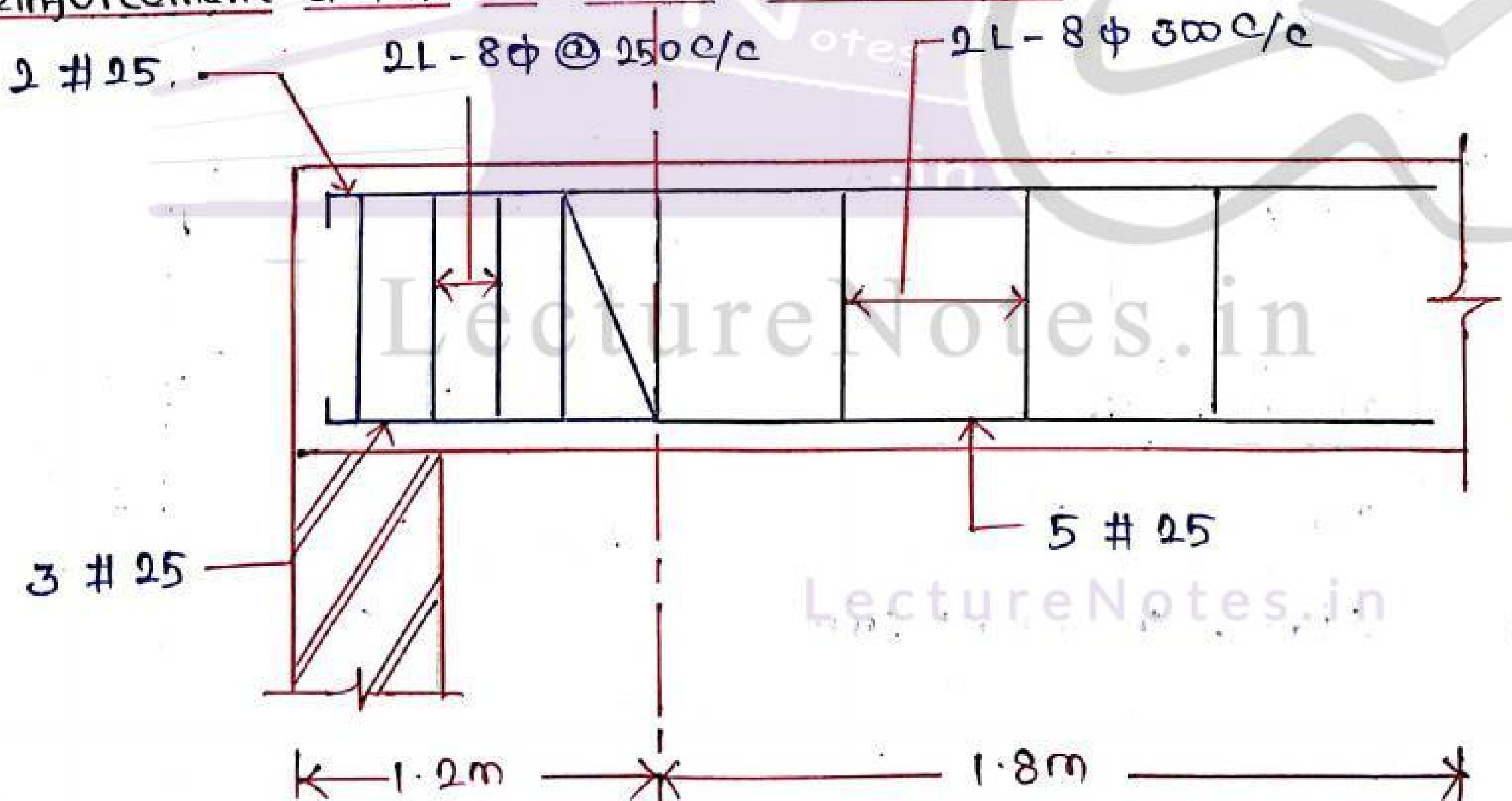
$= 0.75 \times 600$

$= 450mm$

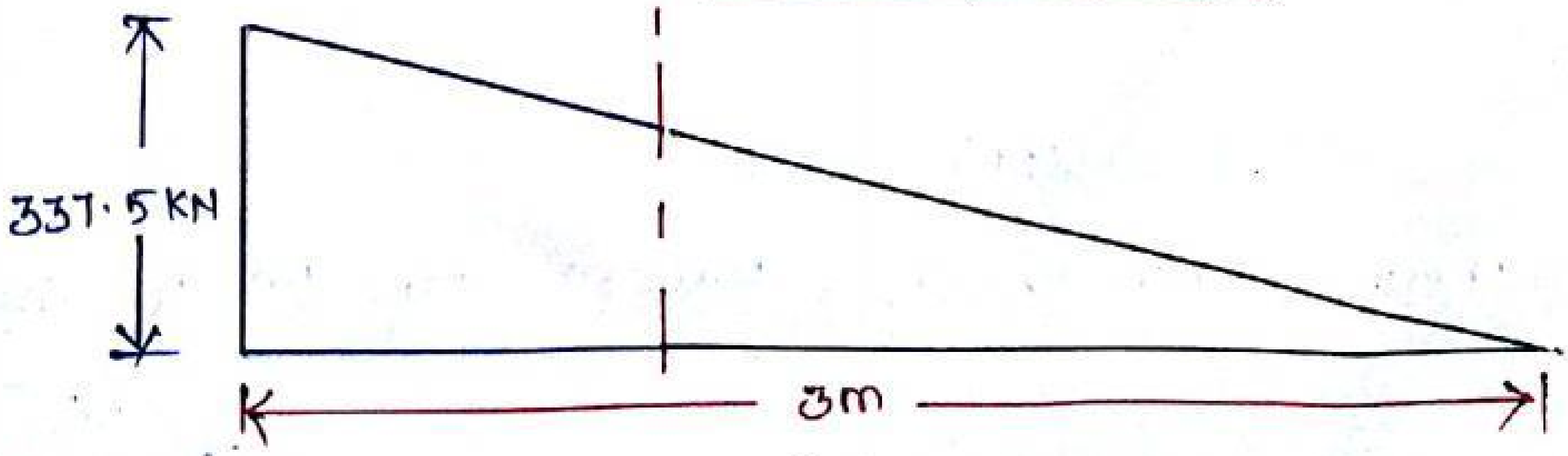
Provide 2 Legged Stirrups 8mm ϕ @ 300mm c/c

S.No	DISTANCE FROM SUPPORT	STIRRUPS	SPACING
1	0 - 1.2m	2L - 8mm ϕ	250 mm c/c
2	1.2m - 3m	2L - 8 mm ϕ	300 mm c/c

Reinforcement detail in Shear reinforcement



LONGITUDINAL SECTION



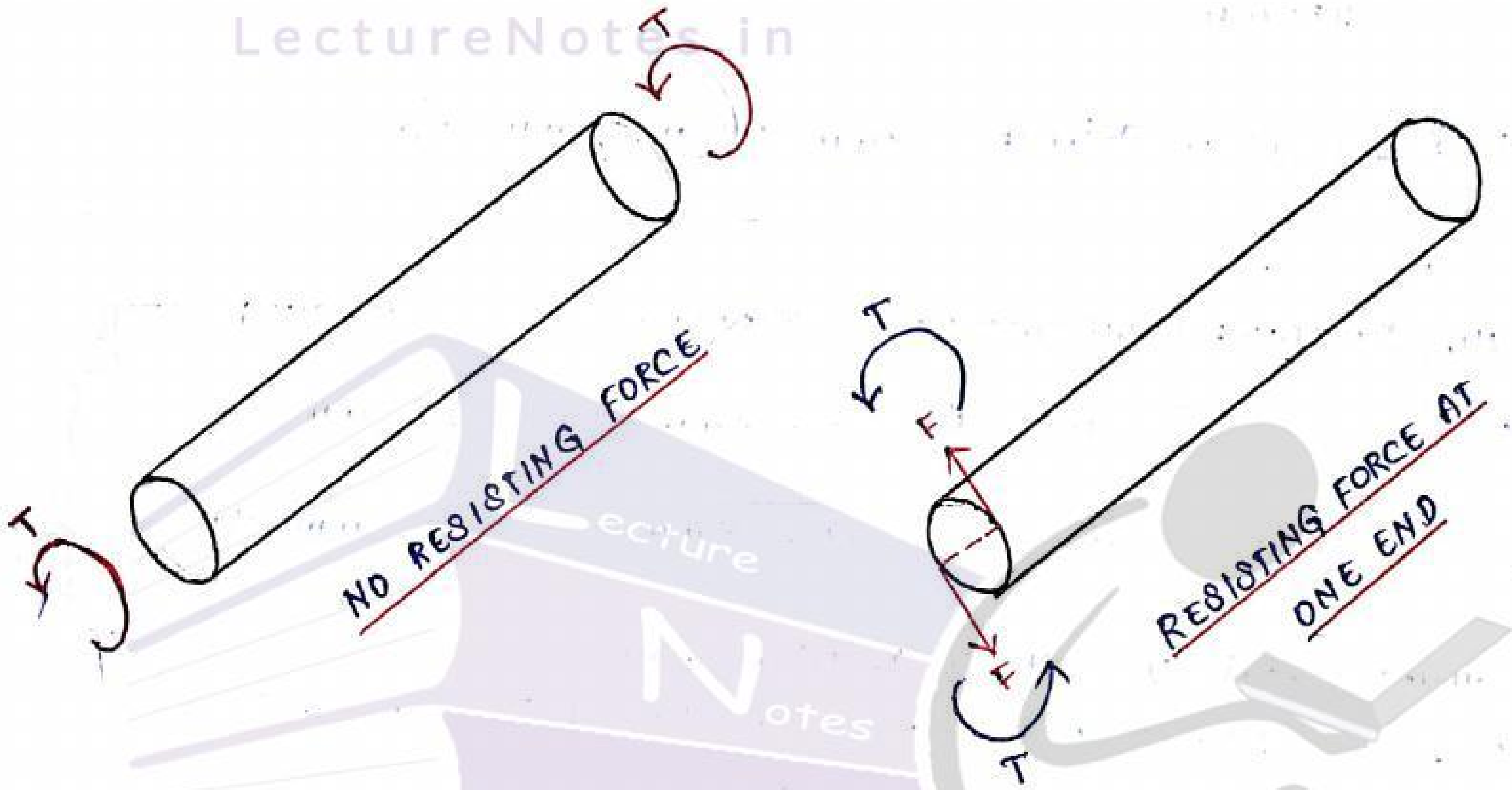
SFD

TORSION

DEFINITION

Torsion means Twisting

When a pair of forces of equal magnitude but opposite direction acting on a body, it tend to twist the body.



$$\boxed{T = F \times d}$$

Where

T – Torque (Nm).

F – Tangential force

d – Dia. of the shaft (mm)

TORQUE

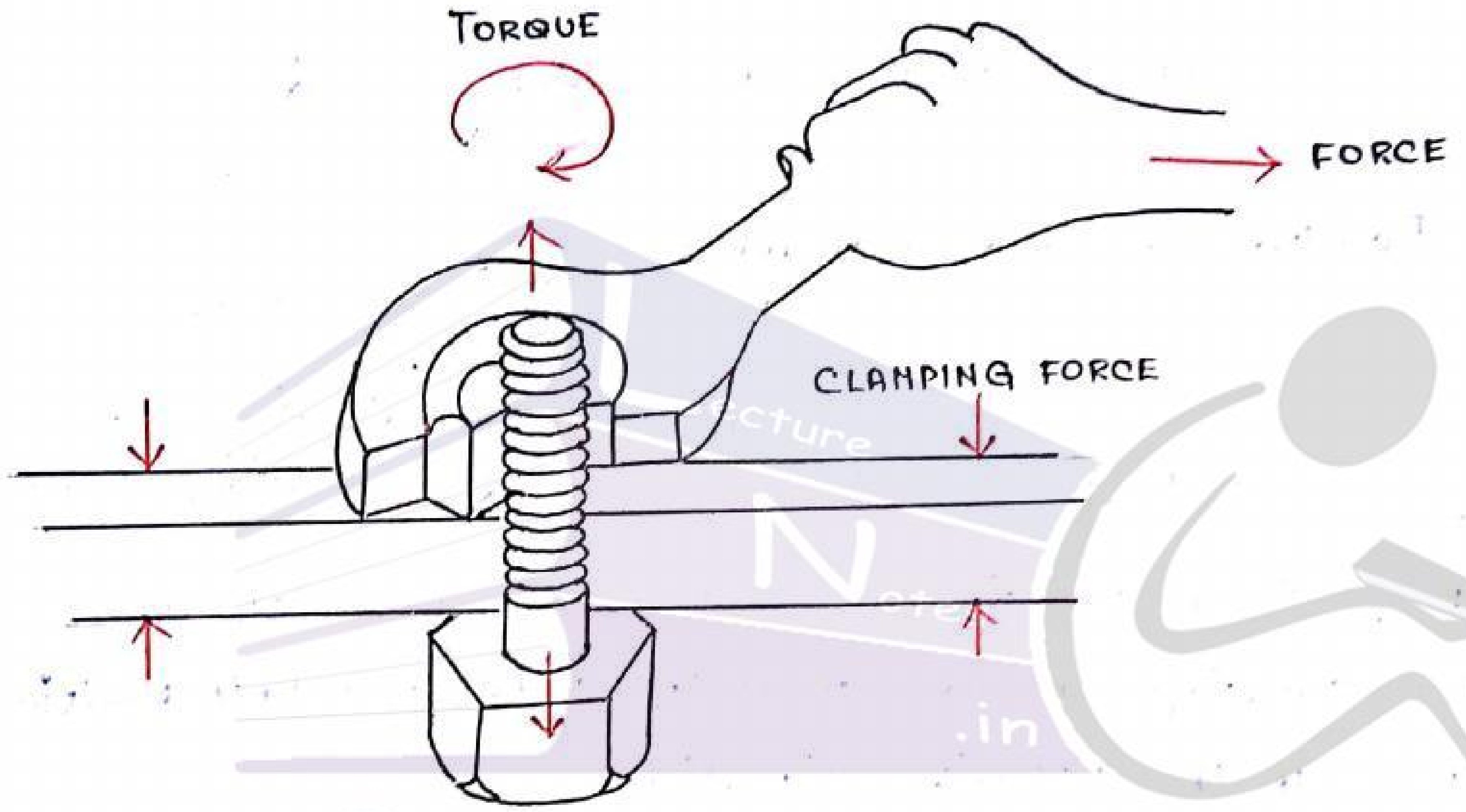
A force acts parallel to the area of cross section tension create shear stress

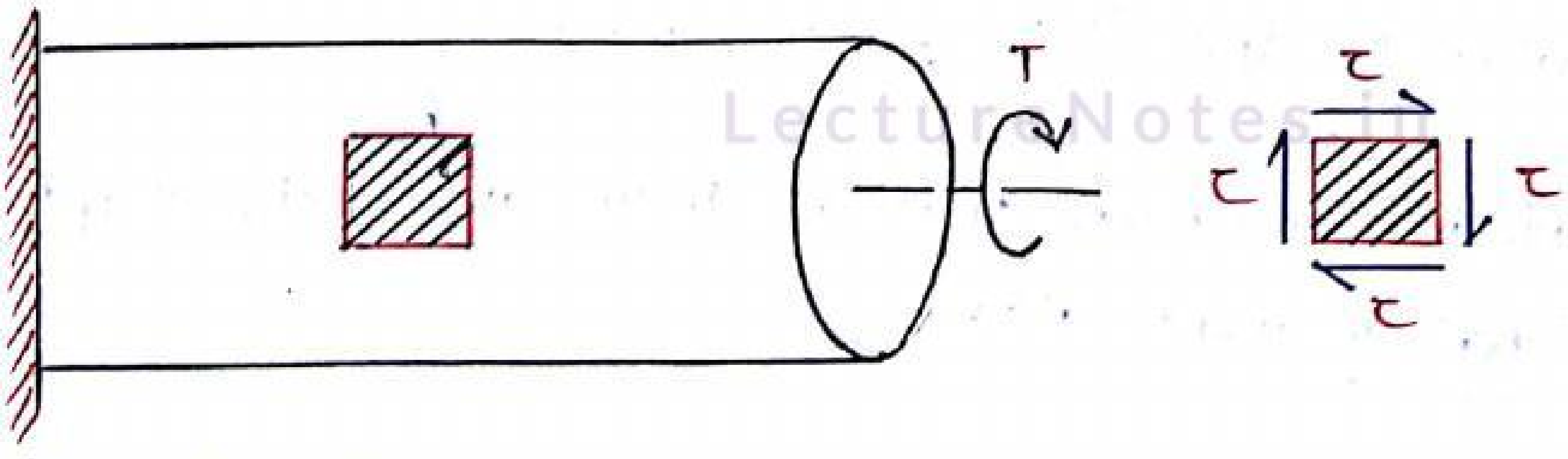
(OR)

Torque is produced due to single force. It is the turning effect of a force

Example:

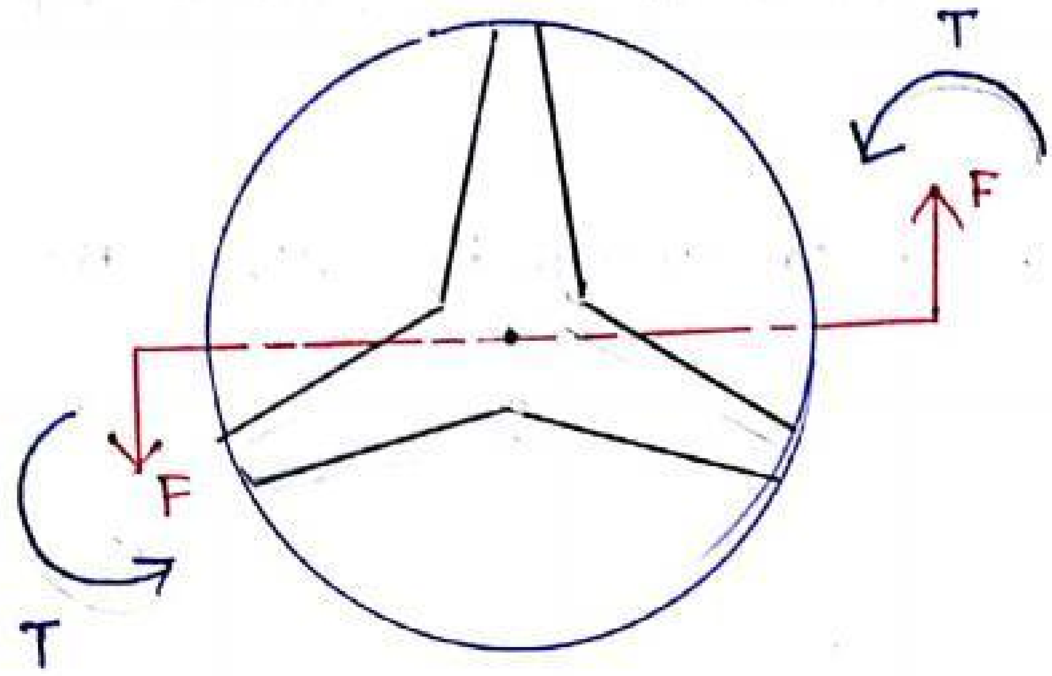
* We rotate a Spanner in a Clockwise direction to Tighten a Nut
* The Turning effect produced is called the Torque (or) Moment of Force





COUPLE

Couple is produced due to Two forces there are equal magnitude but in opposite direction (Unlike Parallel Forces) but do not have than same line of action



Example

Double arm spanner, steering wheel etc..,

Torsion in Reinforced Concrete members

Definition

A Torque is a moment of force that tends to rotate a body.

* Torsion Force is a twisting force that is applied on an object by twisting one end when the other is held in position or twisted in the opposite direction.

* Different materials have a different way of responding to torsion. Some will deform, crack (or) even breaks depending on the type of materials.

* It will be observed that all beams are subjected to some torsional moment also.

* However in many cases these values are small and designers have been carrying out only plane frame analysis

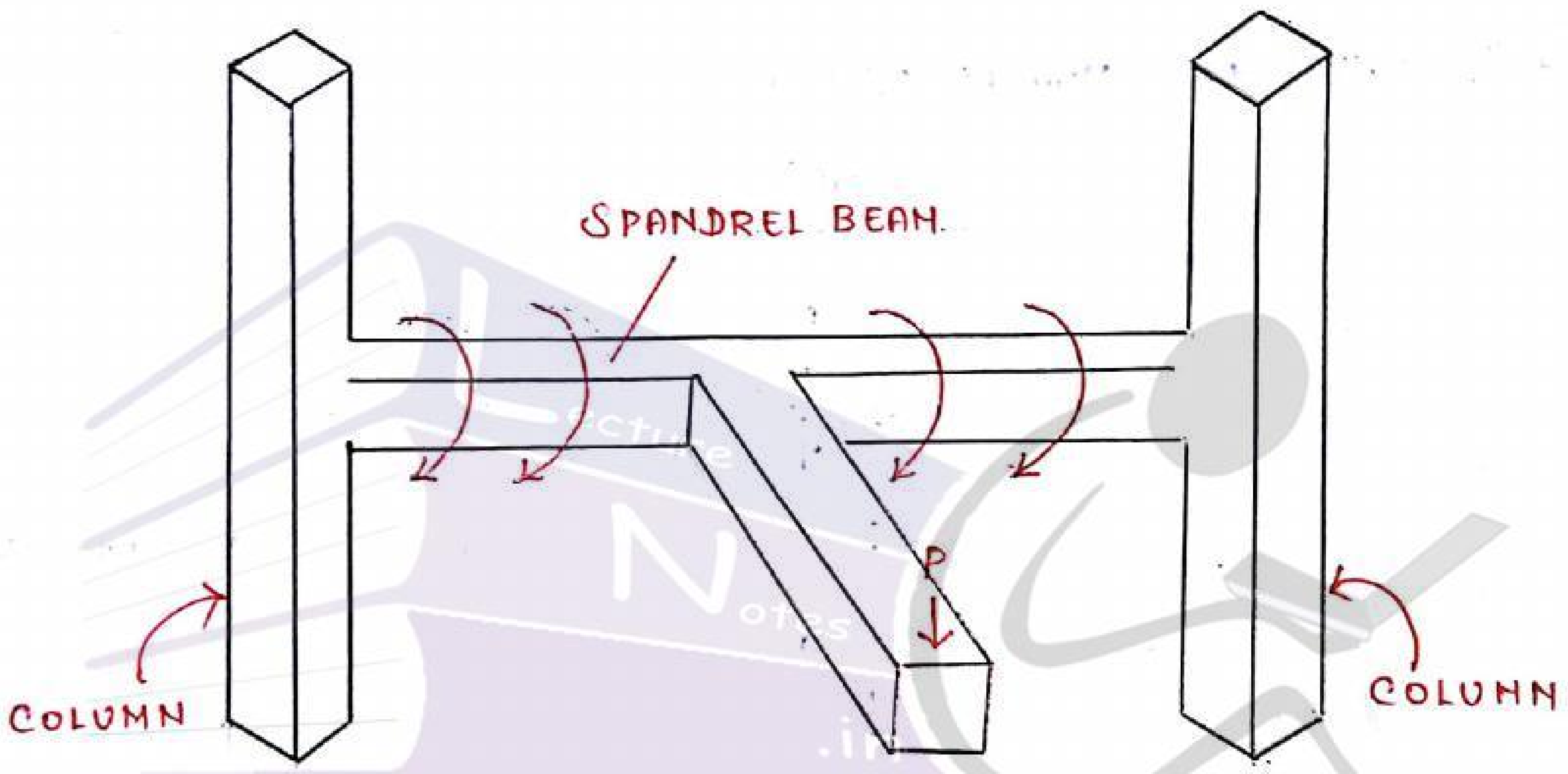
* But there are many cases in which torsional moment cannot be ignored.

(a) A beam with Cantilever Slab

(b) Balcony Girders

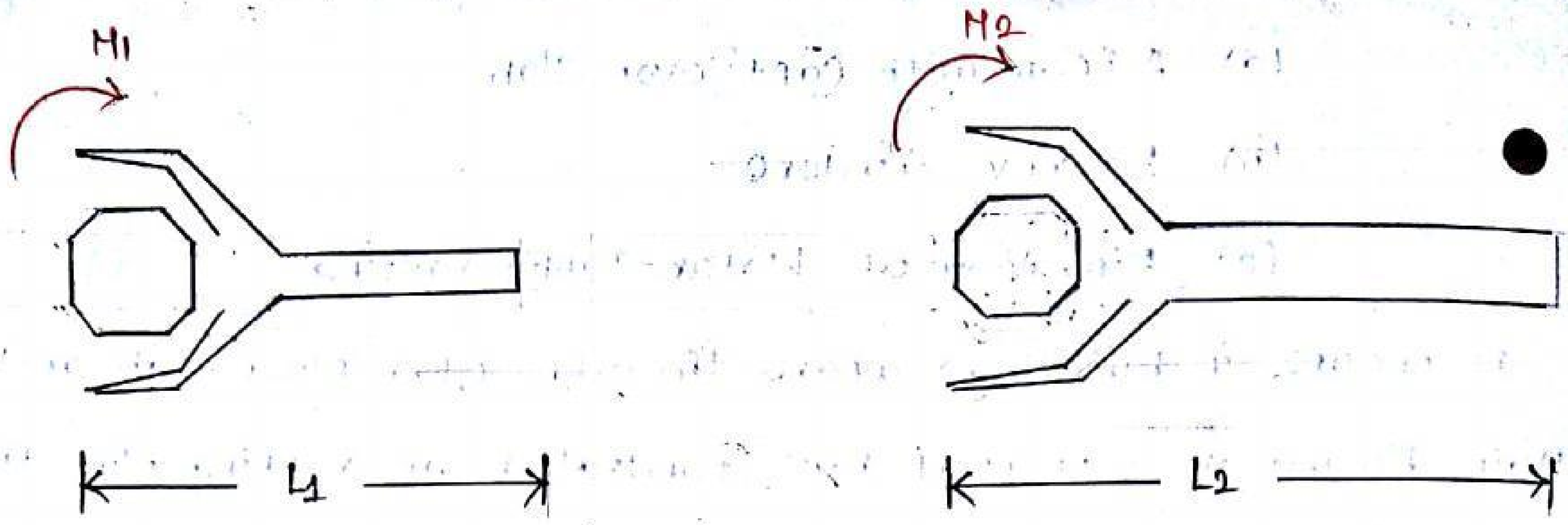
(c) Ring beam of Water Tanks resting

It occurs in buildings where the edge of a floor slab and its Main beams are supported by Spandrel beam running b/w the external Columns



How does Torsion Occur on the End of the beam

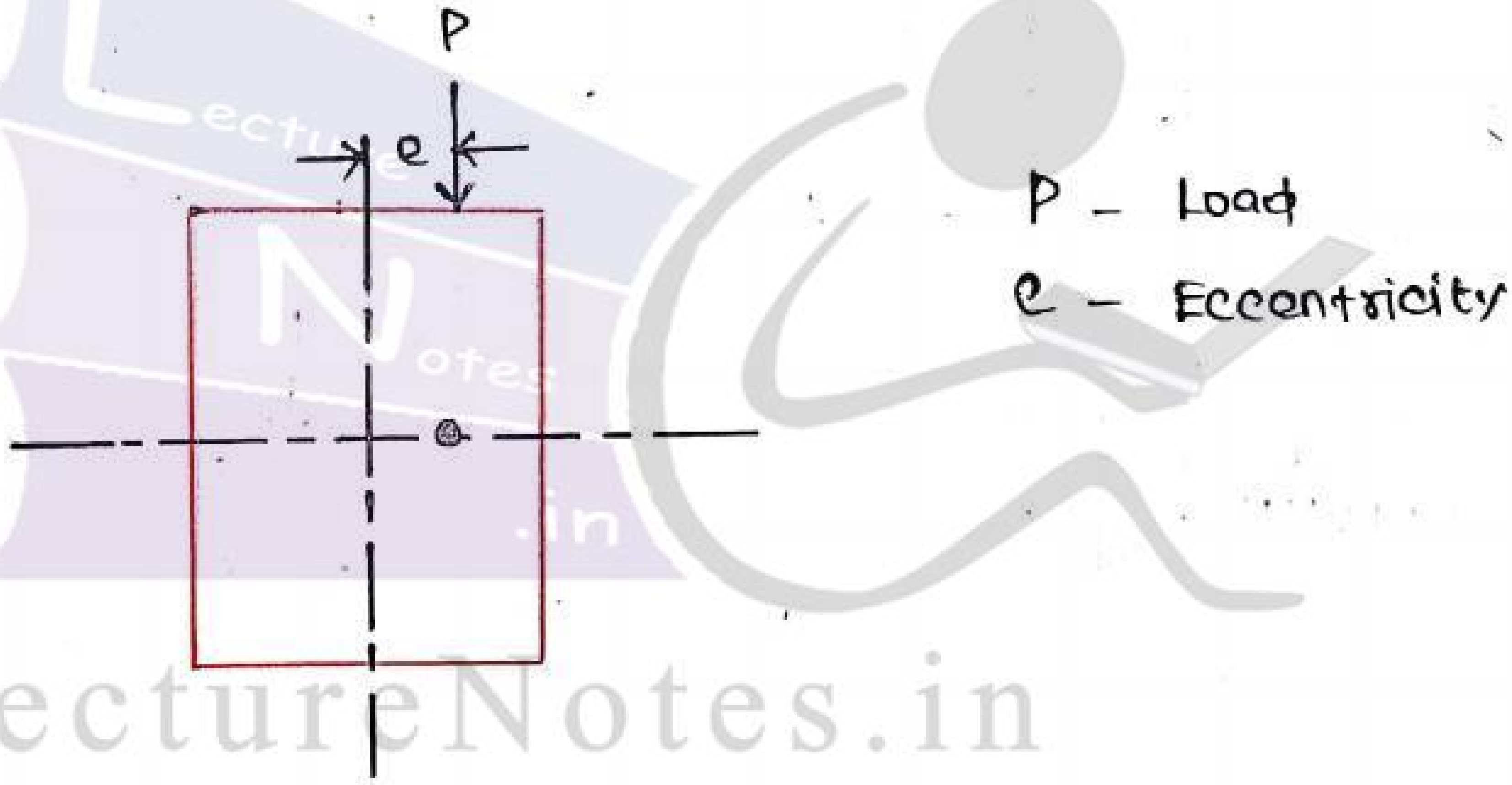
* When a beam is transverly load in such a manner that the resultant forces passes through the Longitudinal shear Centre axis, the beam, only bends and no Torsion will occurs.
* When the resultant acts away from the shear centre axis then the beam will not only bend but also Twist
* you can see the rotation is more in the second case since the lever arm is more and thus the moment is more.



Thus, Here it is not the magnitude of the force but also the distance at which it is applied that t us about its effect

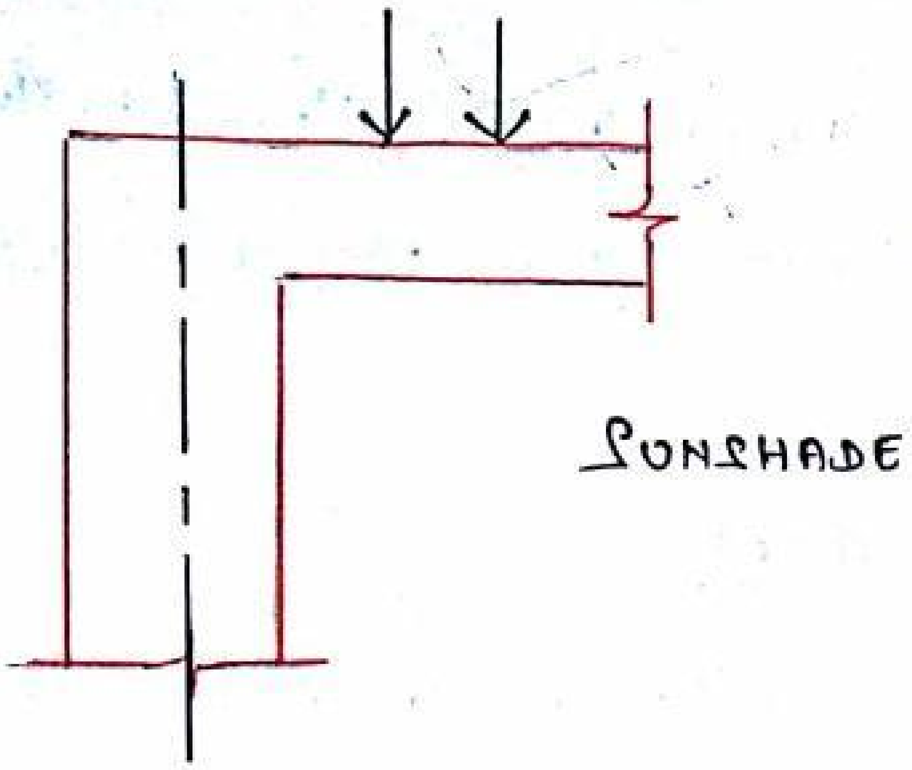
It is called the Moment of force

(OR)



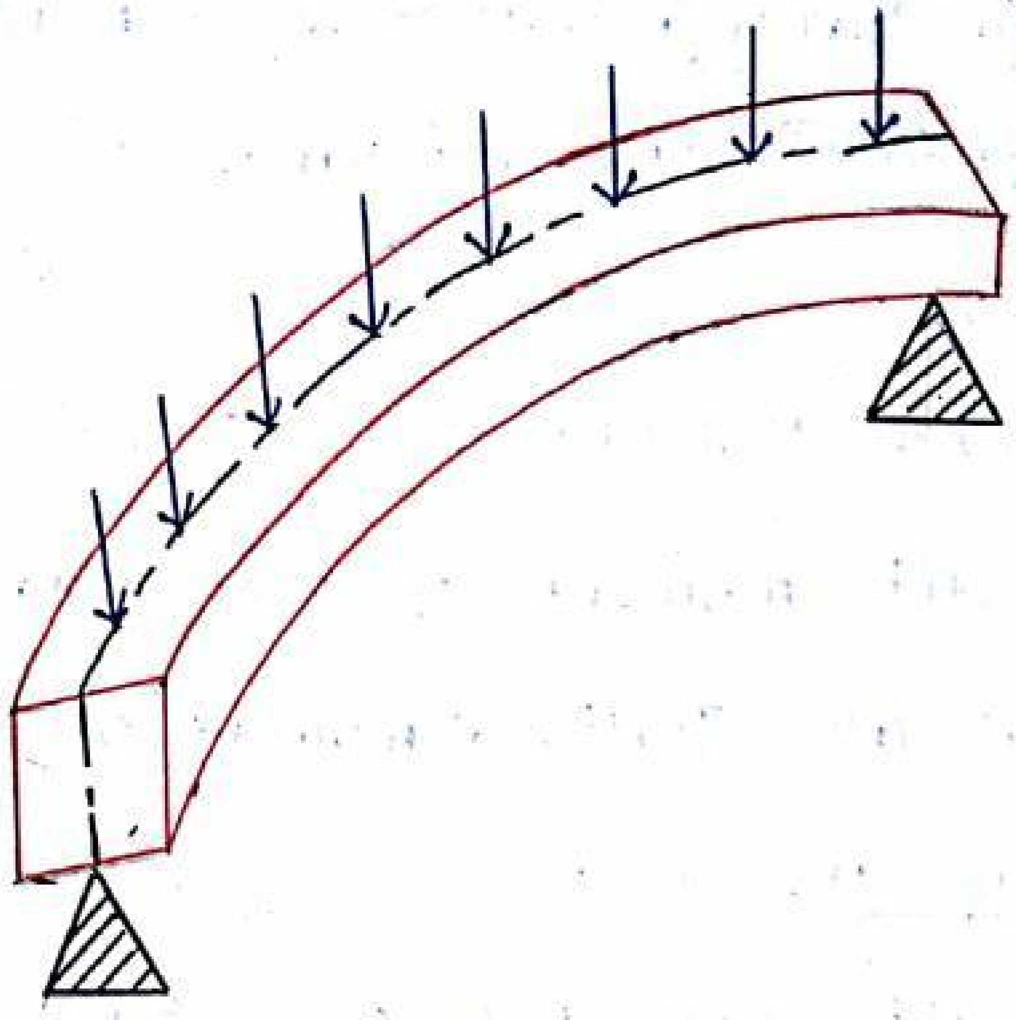
The Load acting away from the centroid

* on several Situation beams and Slab are Subjected to Torsion in addition to bending moment and shear force
* Load acting normally to the plane of bending will cause bending moment and shear force.
* However the Load away from the plane of bending will induced Torsional moment along with B.M and shear

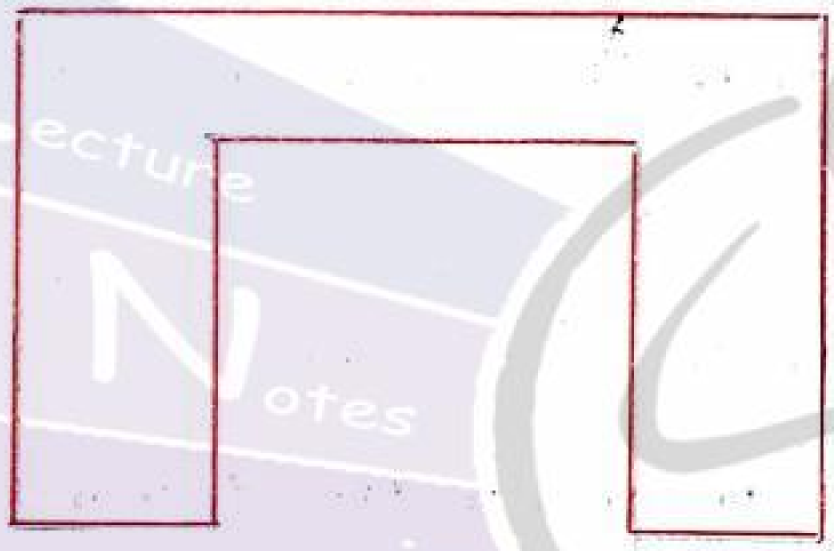


INVERTED L-BEAM

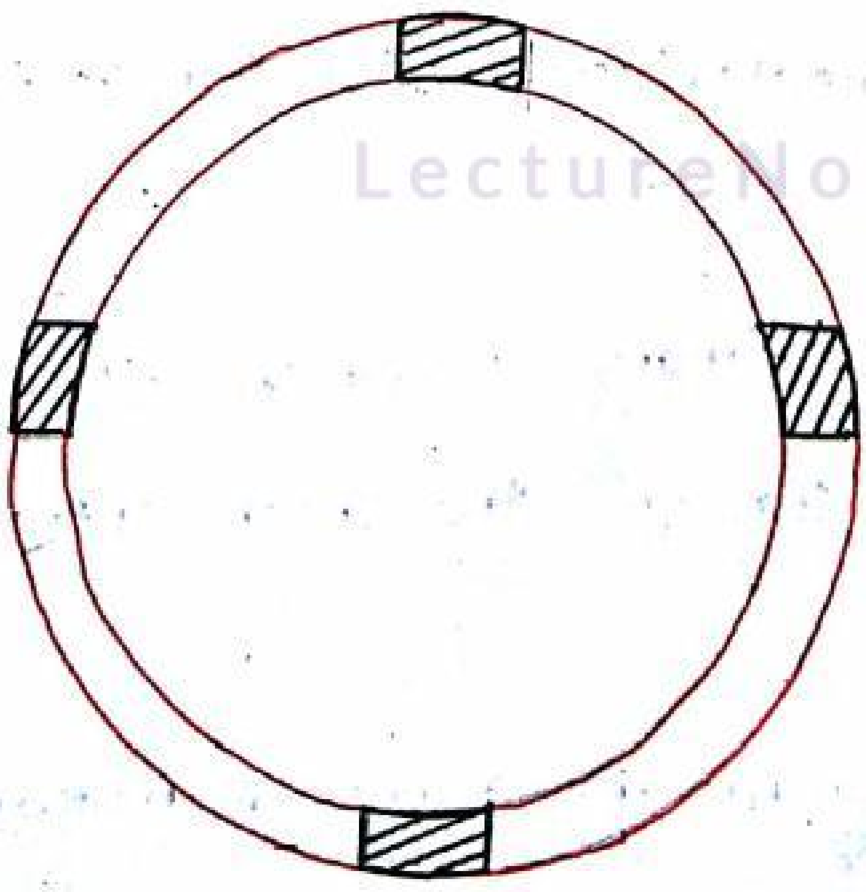
(SUPPORT SUNSHADE)



BEAM CURVED IN PLAN



EDGE BEAM IN SLAB



CIRCULAR WATER TANK

Moment

The turning effect of a Force is known as the Moment. Moment of Force is the distance of the Force from the point of Interest

Torsional Moment (or) Twisting moment

If this moment of force tries to Twist the member than we Called as Torsional moment or Twisting moment

Bending Moment

* If this moment of forces tries to bend the member than we call it as Bending Moment
* A Force applied in the longitudinal axis of the member would tend to elongate (or) Compress the member

Shear force

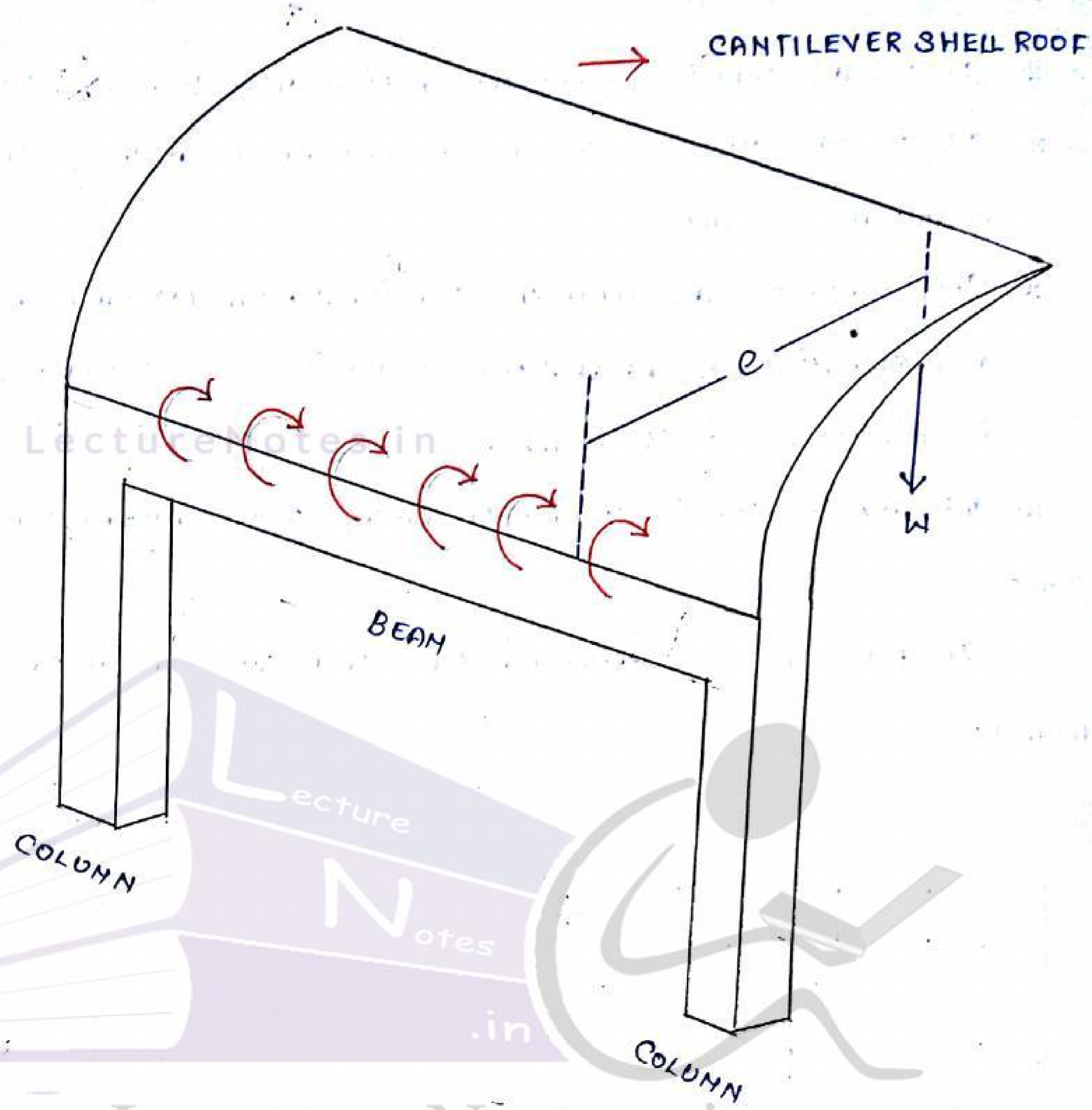
A Force applied in the Lateral axis would try to slice off the member (shear force) or would try to bend the member (Bending moment)

TYPE OF TORSION

* primary (or) Equillibrium Torsion (or) Statically determinate Torsion
* Secondary (or) Compatibility Torsion (or) Statically Indeterminate Torsion

Equillibrium Torsion

* It occurs due to primary action of loads
* load not passing through the shear centre. It occurs due to eccentricity of load
* This case occurs when the Torsional moment is required to be in equillibrium and max. Torsional moment cannot be reduced by Redistribution of moments



BEAM SUPPORT A LATERAL OVERHANG

* In this case Torsion reinforcement must be provide to resist To

* This type of Torsion is also known as primary Torsion

Example :

* An Edge beam Supporting Cantilever slab

* Canopy beam

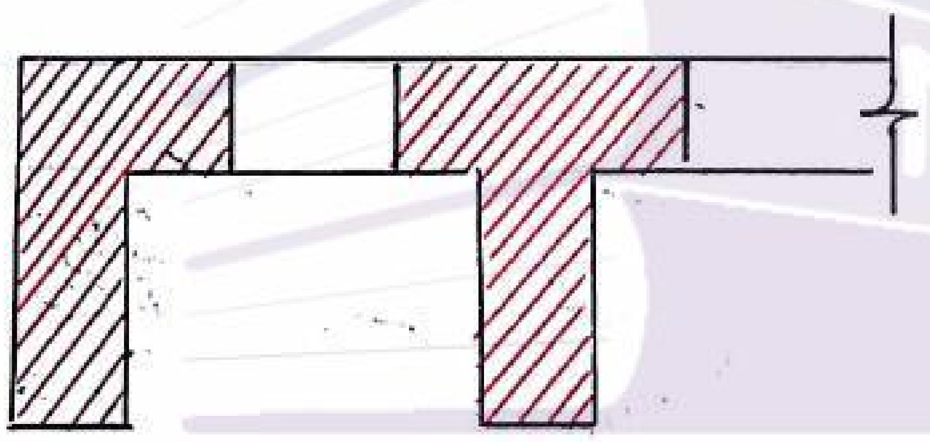
* Circular beam

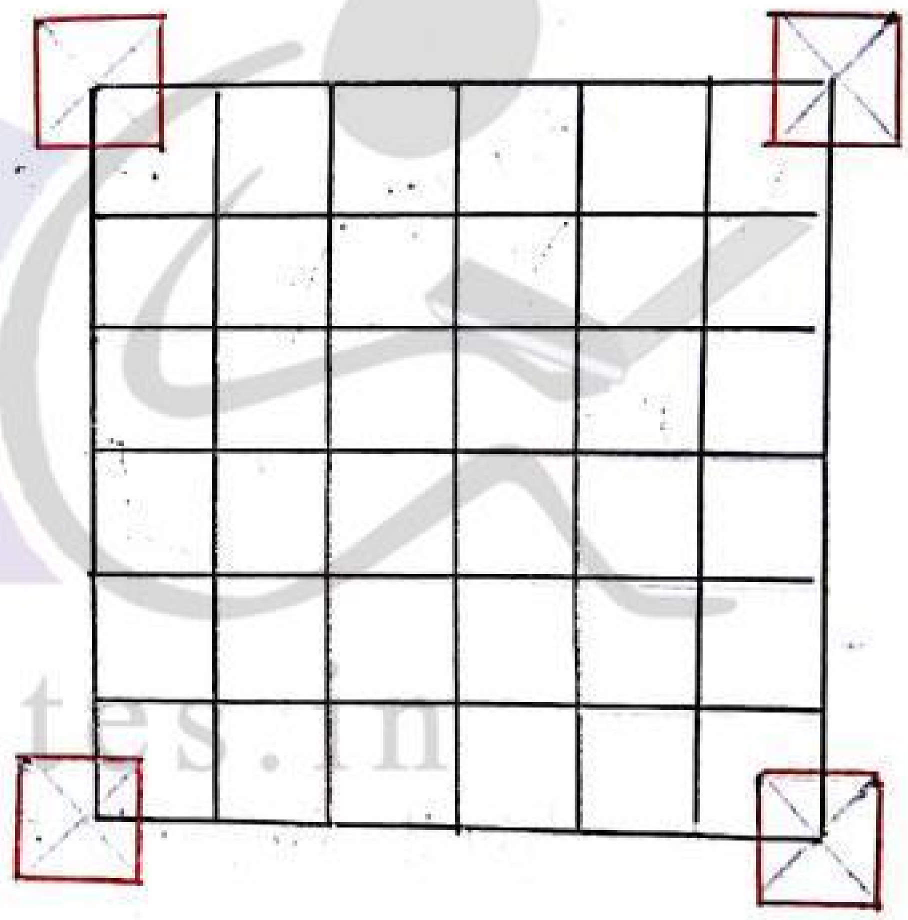
Compatibility Torsion

* It occurs due to secondary action of a loaded structure
* It occurs due to stiffness developed at the joints of beam and column
* This case occurs when the Torsional moment can be reduced by the redistribution of internal forces while Compatibility of deformation is maintained in the member
* This type of torsion is also known as Secondary Torsion

Example

An Edge beam supporting two transverse beam producing Twisting moment





GRID FLOOR

NOTE:

* Primary Torsion must be designed
* The design of secondary torsion is optional
  → If Torsional stiffness (T/θ) is considered in analysis then design for Torsion is required
  → If Torsional stiffness is not considered in analysis then no need of design for Torsion

* IS Codal provision for Torsion are taken from Skew bending Theory
* There is no separate analysis & design for Torsion
* Torsion is designed along with Shear and Flexure
* If the beam is subjected to Torsion, minimum 4 no's of bars are provided one at each corner
* If the beam is subjected to torsion spiral cracks are developed

LIMIT STATE OF COLLAPSE :- TORSION

Refer IS:456-2000

Pg. No: 75

(i) EQUIVALENT SHEAR (Ve)

$$V_e = V_u + 1.6\left(\frac{T_u}{b}\right)$$

Where

Ve – Equivalent shear

Vu – Ultimate shear force

Tu – Ultimate Torsional Moment

b – Breadth of beam

(ii) EQUIVALENT BENDING MOMENT (Me)

$$M_e = M_u + M_t$$

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

Where,

Me – Equivalent bending Moment

Mu – Ultimate Bending Moment

Tu – Ultimate Torsional Moment

D – Overall depth

b – Breadth of beam

Mt – Effective bending Moment

(iii) Ultimate TORSIONAL MOMENT

$$T_u = \frac{0.87 \times f_y \times A_{sv} \times b_1 \times d_1}{S_v}$$

Where,

b_1, d_1 denote the Centre to Centre distance between the Corner bars

S_v – Spacing

A_{sv} – c/s of stirrups

$$A_{sv} = \left[\left(\frac{T_u \cdot S_v}{b_1 d_1\ 0.87 f_y}\right) + \left(\frac{V_u \cdot S_v}{2.5 d_1\ 0.87 f_y}\right)\right]$$

(IV) TRANSVERSE REINFORCEMENT

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

(ii) $$A_{sv} = \left[\left(\frac{T_u \cdot S_v}{b_1 d_1\ 0.87 f_y}\right) + \left(\frac{V_u S_v}{2.5 d_1\ 0.87 f_y}\right)\right]$$

Problem on Analysis of Torsion

-

Prblm.No:5

A Rectangular Concrete beam has c/s 300mm x 600mm and is Subjected to the Following design forces. Bending Moment 115 KN·m, Shear Force = 95 KN, Torsional moment 45 KN. Determine the Equivalent bending Moment and Shear force for which Section is to be designed

Given data

$M_u = 115\ KN{\cdot}m$

$V_u = 95\ KN{\cdot}m$

$T_u = 45\ KN$

$D = 600mm$

$b = 300mm$

To Find

$M_e = ?$

$V_e = ?$

(a) **Equivalent Bending Moment**

$$M_e = M_u + T_u \left[\frac{1 + D/b}{1.7}\right]$$

$$= 115 + 95 \left[\frac{1 + 600/300}{1.7}\right]$$

$$\boxed{M_e = 194.41\ KN{\cdot}m}$$

(b) Equivalent Shear force

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

$$= 95 + 1.6 \times \frac{4.5}{0.3}$$

$$\boxed{V_e = 335\,KN}$$

Prblm. No: 6

A Reinforced Concrete beam of rectangular section has a width of 350mm and overall depth of 700mm. The beam is reinforced with 2 bars of 25mm diameter both on the Tension and compression face at an effective cover of 50mm. The side cover are 25mm 10mm diameter Two legged stirrups are provided at 100mm centres. The section is subjected to a factored shear forces of 200KN. If Fe 415 HYSD bars are used, Estimate the Torsional resistance of the bars using I.S code provisions

Given data

$b = 350mm$

$D = 700mm$

$A_{st} = 2 \times \frac{\pi}{4} \times 25^2 = 981.74 mm^2$

$d' = 50mm$

$A_{sv} = 2 \times \frac{\pi}{4} \times 10^2 = 157.07 mm^2$

$S_v = 100mm$, $V_u = 200\,KN$

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

Solution

- (a) Torsional Strength Considering $V_u = 0$

$$T_u = \frac{0.87 f_y A_{sv} \cdot b_1 \cdot d_1}{S_v}$$

$$= \frac{0.87 \times 415 \times 10^{-3} \times 157.07 \times 300 \times 600}{100}$$

$$\boxed{T_u = 102.07\ kN \cdot m}$$

(b) Torsional Strength Considering $V_u = 200 kN$

$$A_{sv} = \left[\left(\frac{T_u \cdot S_v}{b_1 d_1\ 0.87 f_y}\right) + \left(\frac{V_u S_v}{2.5 d_1\ 0.87 f_y}\right)\right]$$

$$157.07 = \left[\left(\frac{T_u \times 100}{300 \times 600 \times 0.87 \times 415}\right) + \left(\frac{200 \times 10^3 \times 100}{2.5 \times 600 \times 0.87 \times 415}\right)\right]$$

$$= \left(1.538 \times 10^{-6} T_u + 36.92\right)$$

$$T_u = \frac{157.07 - 36.92}{1.538 \times 10^{-6}}$$

$$= 78.12 \times 10^6\ N \cdot mm$$

(or)

$$\boxed{T_u = 78.12\ kN \cdot m}$$

Prblm. No : 7

A Reinforced Concrete rectangular beam has a breadth of 400 mm and Effective depth of 800 mm. It has a Factored Shear force of 120 KN at a particular section. Assuming that $f_{ck} = 25\ N/mm^2$ and $f_y = 415\ N/mm^2$ and percentage of Tensile steel at that section as 0.5 percent. Determine the Torsional moment the section can resist for the following cases

Case-1 :- If No additional Reinforcement for Tension is provided

Case-2 :- If the Maximum steel for Torsion is provided in the section

Given data :-

$b = 400\,mm \quad f_{ck} = 25\,N/mm^2, \quad f_y = 415\,N/mm^2$

$d = 800\,mm \quad P_t = 0.5$

$V_u = 120\,KN$

Solution :-

(a) permissible Shear stress

For, $P_t = 0.5\,\%$ and $f_{ck} = 25\ N/mm^2$

Refer Table-19

$\tau_{c(min)} = 0.49\ N/mm^2$

Case-1 :- If no additional Reinforcement for Tension is provided (Allowable Torsion)

$$V_e = V_u + 1.6\,(T/b) \qquad \left(\frac{V_e}{bd}\right) = 0.49$$

$$\frac{V_u + 1.6\,(T/b)}{bd} = 0.49$$

$$\frac{(120 \times 10^3) + 1.6\,(T/400)}{400 \times 800} = 0.49$$

Scanned by CamScanner

$$\frac{1.6\,(T/400)}{400 \times 800} = 0.49 - 0.379$$

$$1.25 \times 10^{-8}\,T = 0.111$$

$$T = 8.88 \times 10^{6}\ N{\cdot}mm$$

$$\boxed{T = 8.88\ KN{\cdot}m}$$

Case-2 :- If the Maximum Steel for Torsion in provided in the Section (Max. Torsional Capacity of the Section

(Torsion + Shear)

$$\tau_{c\,max} = 3.1\ N/mm^2$$

T - Maximum allowable Torsion

$$\frac{V_u + 1.6\,(T/b)}{bd} = 3.1$$

$$\frac{120 \times 10^{3} + 1.6\,(T/400)}{400 \times 800} = 3.1 \Rightarrow 1.27 \times 10^{-8}\,T = 2.725$$

$$T = 218 \times 10^{6}\ Nmm$$

$$\boxed{T = 218\ KN{\cdot}m}$$

Design Examples

Prblm. No: 8

An RCC Section 200 x 400mm is subjected to a characteristic Torsional moment of 2.5 KN.m and a Transverse shear of 60 KN Assuming the use of M-25 Grade concrete and Fe 415 HYSD bars, Determine the reinforcement required according to the IS 456 code provision, using the following data

Given data

$b = 200mm$ $\quad f_y = 415\ N/mm^2$

$D = 400mm$ $\quad f_{ck} = 25\ N/mm^2$

$d = 350mm$ $\quad b_1 = 150\ mm$

$T_u = 2.5\ KN.m$ $\quad d_1 = 300mm$

$V_u = 60\ KN$

(a) Equivalent Shear Force (V_e)

$$V_e = V_u + 1.6\left(\frac{T_u}{b}\right)$$

$$= 60 + 1.6 \times \left(\frac{2.5}{0.2}\right)$$

$$\boxed{V_e = 80KN}$$

(b) Equivalent Bending Moment (M_e)

$$M_e = M_u + M_t$$

$$= 0 + \frac{T_u(1 + D/b)}{1.7} \Rightarrow \frac{2.5\left(1 + \frac{400}{200}\right)}{1.7}$$

$$\boxed{M_e = 4.41\ KN.m}$$

(c) Longitudinal Reinforcement

$$x_{u\,lim} = 0.48d \Rightarrow 0.48 \times 350 = 168mm$$

$$M_{u\,lim} = 0.36 f_{ck}\, b\, x_{u\,lim}(d - 0.42\, x_{u\,lim})$$

$$= 0.36 \times 20 \times 200 \times 168 \times (350 - 0.42 \times 168)$$

$$= 67.60 \times 10^6 \text{ N·mm}$$

$$M_{ulim} = 67.60 \text{ KN·m}$$

$$M_e < M_{ulim}$$

Hence the section is Under Reinforced section

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

$$4.41 \times 10^6 = 0.87 \times 415 \times A_{st} \times 350 \left(1 - \frac{A_{st} \times 415}{200 \times 250 \times 20}\right)$$

$$= 126.36 \times 10^3 A_{st} - 37.45 A_{st}^2$$

$$37.45 A_{st}^2 - 126.36 \times 10^3 A_{st} + 4.41 \times 10^6 = 0$$

$$\boxed{A_{st} = 35.26 \text{ mm}^2}$$

Providing Minimum Reinforcement of

$$A_{st} = \frac{0.85 bd}{f_y} = \frac{0.85 \times 200 \times 350}{415}$$

$$= 143.37 \text{ mm}^2 \quad (\text{Required } A_{st})$$

Provide 2 bar of 10mm diameter as Tension Reinforcement and 2 Hanger bars of 10mm diameter on Compression side

$$\boxed{A_{st} = 157.07 \text{ mm}^2}$$

(d) permissible shear stress

$$\tau_{ve} = \frac{V_e}{bd} = \frac{80 \times 10^3}{200 \times 350} = 1.14 \text{ N/mm}^2$$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 157.07}{200 \times 350} = 0.224$$

Refer Table - 19

$$\tau_c = 0.34 \text{ N/mm}^2$$

Design Transverse Reinforcemente

(e) Transverse Reinforcement (Side Reinforcement)

Using 8mm dia 2 legged stirrups wide side cover of 25mm

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

$$= \frac{(2 \times \frac{\pi}{4} \times 8^2) \times 0.87 \times 415}{(1.14 - 0.24) 200}$$

$$S_v = 226.85 \text{mm}$$

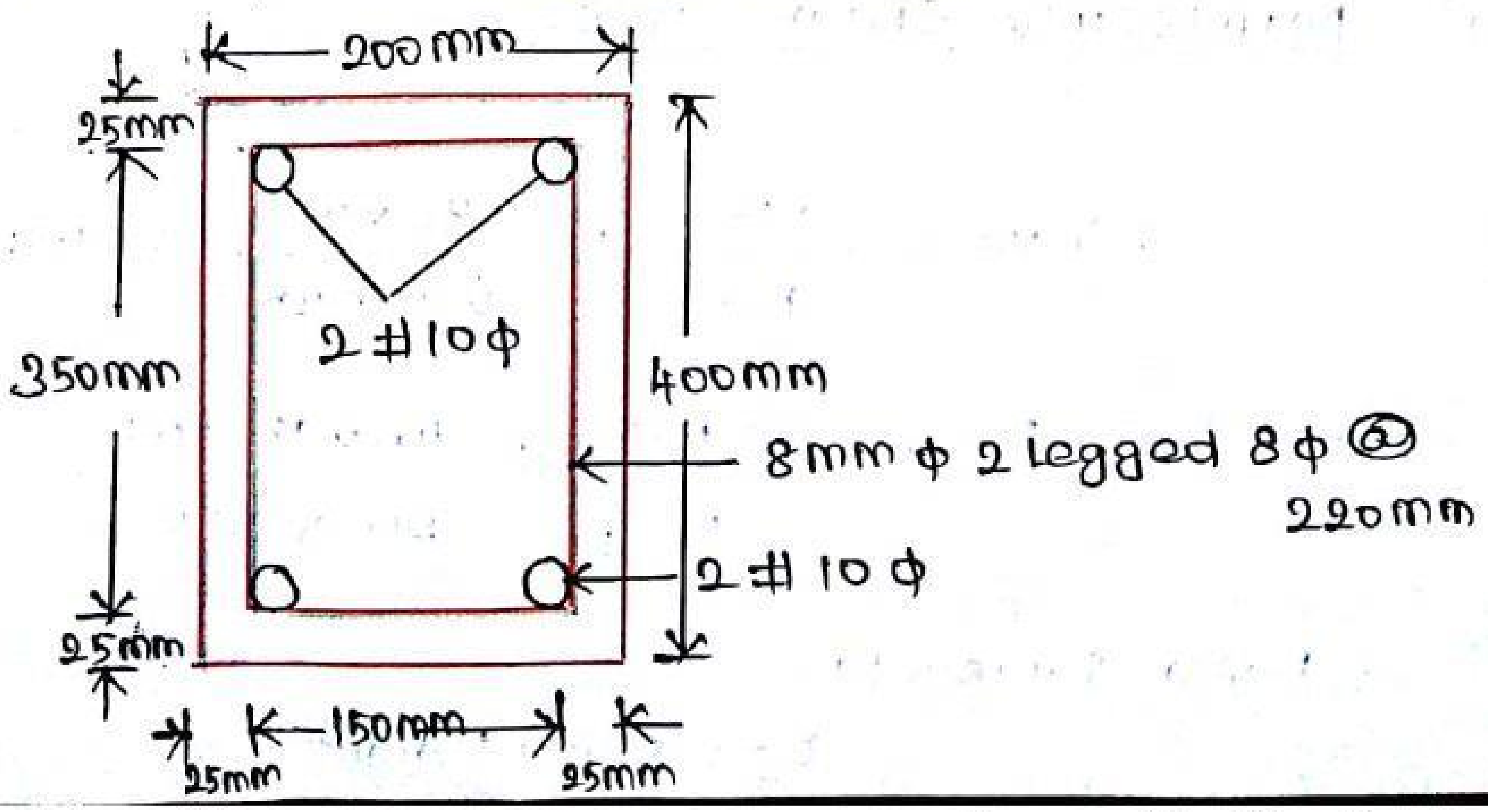
(ii) $$A_{sv} = \left[\frac{T_u S_v}{b_1 d_1 0.87 f_y} + \frac{V_u S_v}{2.5 d_1 \cdot 0.87 f_y} \right] = 100$$

$$S_v \left[\frac{2.5 \times 10^6}{150 \times 300 \times 0.87 \times 415} + \frac{60 \times 10^3}{2.5 \times 300 \times 0.87 \times 415} \right] = 100$$

$$S_v (0.37) = 100$$

$$\boxed{S_v = 266 \text{mm}}$$

Adopting the smaller of the two values. Using 8mm dia 2 legged stirrups at a spacing of 225 mm



BOND

DEFINITION

* The term bond refers to the adhesion between concrete and steel which resist the sliping of steel bar from the concrete
* It is this bond which is responsible for transfer to stresses from steel to concrete and there by providing composite action of steel and concrete in RCC
* The bond develop due to setting of concrete on drying which results in gripping of steel bars

Bond Mechanism

Bond between concrete and steel develops due to the following three mechanism

* Chemical Adhesion
* Friction b/w concrete and steel
* Mechanical resistance

Chemical Adhesion

chemical adhesion is the grip b/w developed due to the gum like property of the hydration products of cement in concrete

Friction b/w Concrete and Steel

Friction resistance developed due to the relative moyement b/w concrete and steel bars depending upon the surface characteristics of the bar and the grip developed due to shrinkage of concrete

(c) Mechanical Resistance

(The circular grip provided in deformed bar for better bond)

It is due to Mechanical Interlock developed as a consequence of surface ribs provided in deformed bars

In case of Mild Steel (Fe 250), the bond develops due to adhesion and friction only. But in case of deformed bars (Fe 415 & Fe 500), the bond develops due to adhesion, friction, and mechanical deformation and hence the bond stress increases by 60% due to the presence of deformed bars (HYSD).

Bond Stress

The bond stress is the shear stress developed in concrete at the interface of concrete and steel. It is called Bond stress and is expressed in terms of the tangential force per unit nominal surface area of the reinforcing bars.

There is a limit upto which such stress can develop and beyond that there will be slip. The design has to take care of the following two cases of bond failure

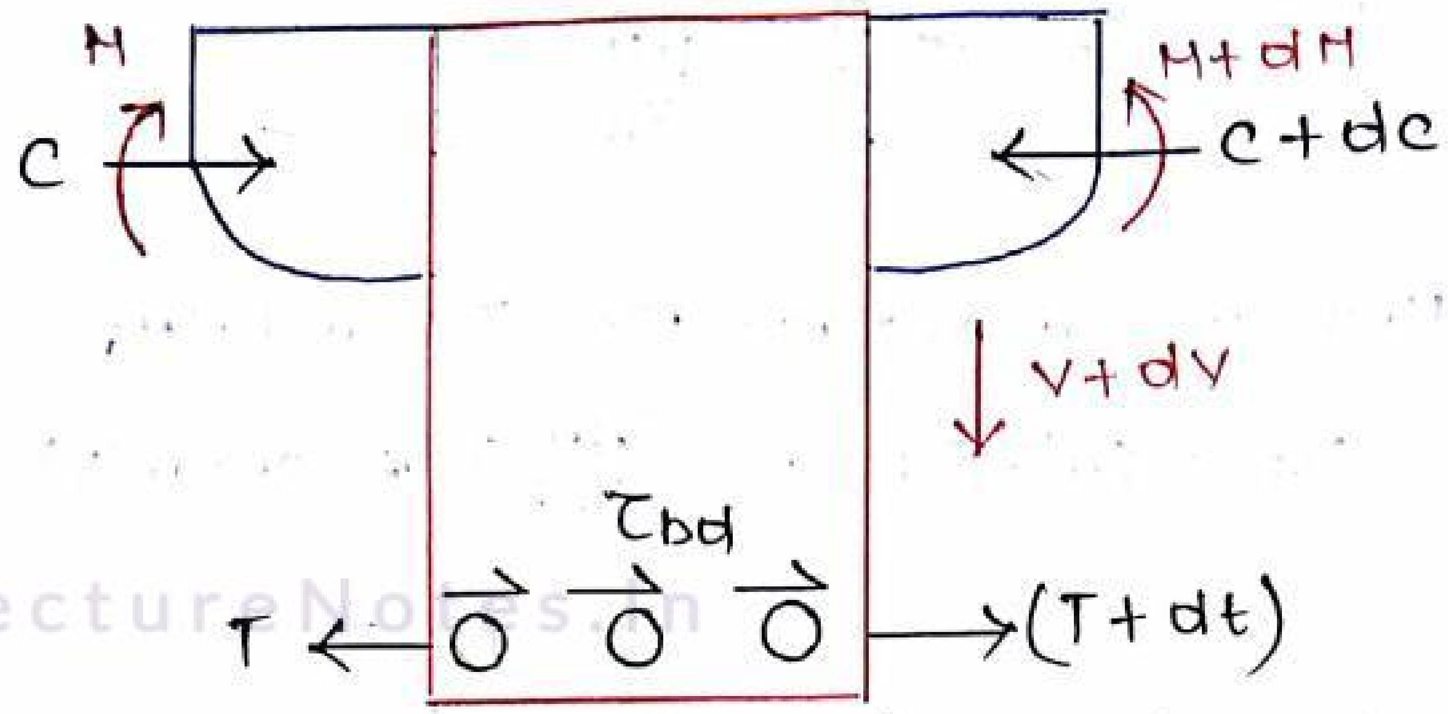
* Flexural bond
* Anchorage bond

Flexural bond (Local bond)

That occurs due to varying moment along span thereby applying different tension or bar ends

(or)

The Flexural bond is defined as the magnitude of the bond stress at any point on the structural element between reinforcement and the concrete



Consider a c/s of beam, Compression force, Tensile force,

$$M = T \cdot Z \; (\tau_d)$$

$$dM = dT \cdot Z$$

Where,

dM = Differential Moment

dT = From section x to y causes the additional tension (dT)

$$\boxed{dT = \frac{dM}{\tau_d}}$$

Now consider a steel x and y having some stress will acting on the reinforcement

Sum of perimeter + Length (dx) = Surface area of Cylinder

$$U_f \, [\Sigma o \cdot dx] = dT$$

Surface area of cylinder

Where

U_f = Amount of Stress

Stress (U_f) + Σo + dx = F
Force

$[\Sigma o \cdot dx]$ = Surface area of Cylinder

Σo = Sum of perimeter

May be bundle bar
(or)
May be No. of bar

dx = length

$$U_f\left[(\Sigma_0)\cdot dx\right] = dT$$

$$U_f\left[\Sigma_0 \cdot dx\right] = \frac{dM}{\tau_d}$$

Now you can replace the τ_d and dM

$$U_f\,(\Sigma_0)\cdot \tau_d = \left(\frac{dM}{dx}\right) \leftarrow \text{shear (V)}$$

Hence,

$$\left(\frac{dM}{dx}\right) = \text{Shear (V)} \qquad U_d = \frac{V}{(\Sigma_0)_d}$$

↓

Flexural bond

Here you can see a Number of things

$$\text{Shear } V\uparrow = U_f\, T \leftarrow \text{bond stress}$$

$$\Sigma_0 \uparrow,\ U_f \downarrow \leftarrow \text{Bond stress}$$

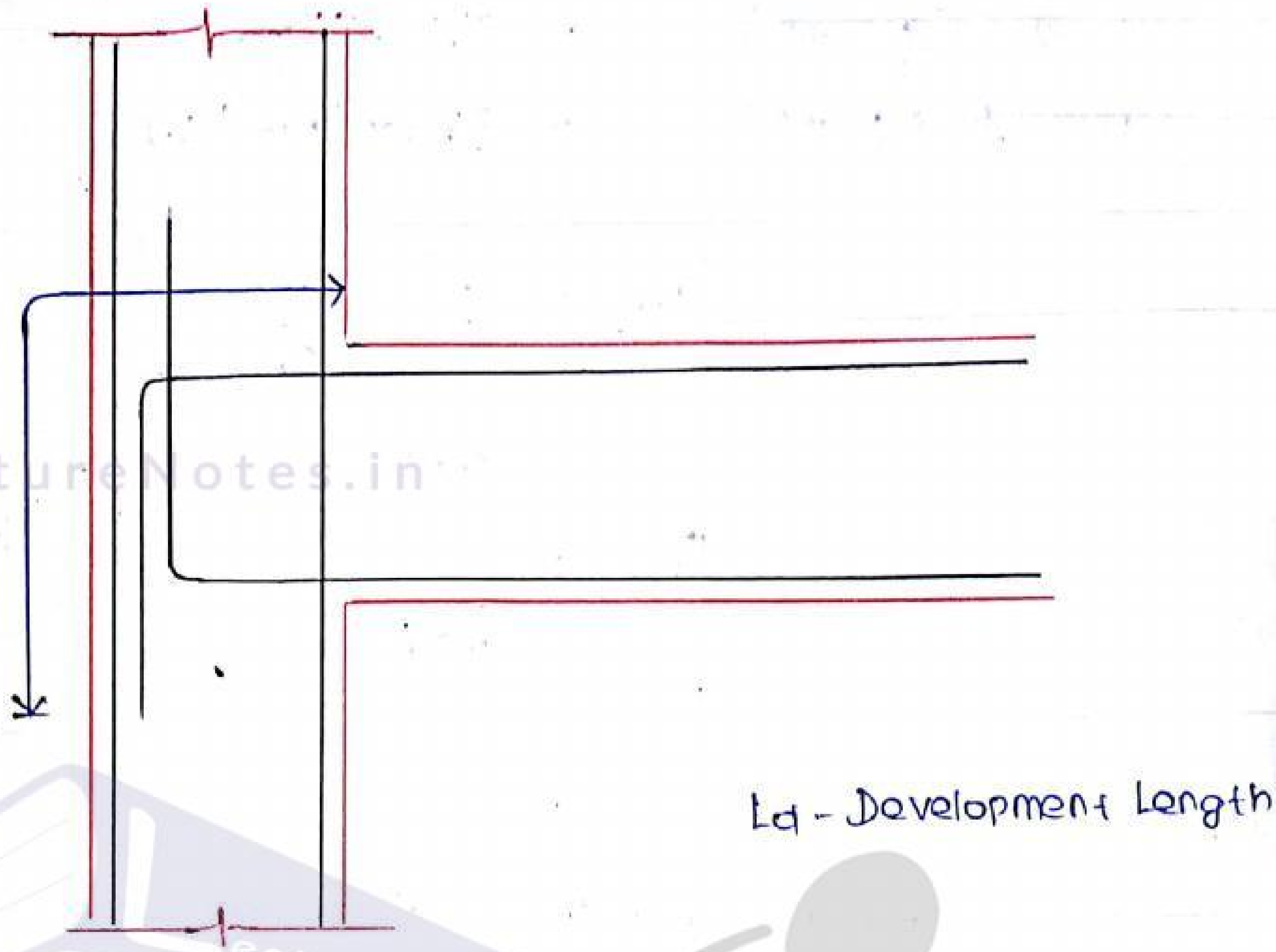
That's why we can use more number of small bar rather than big bars.

<u>Note</u>

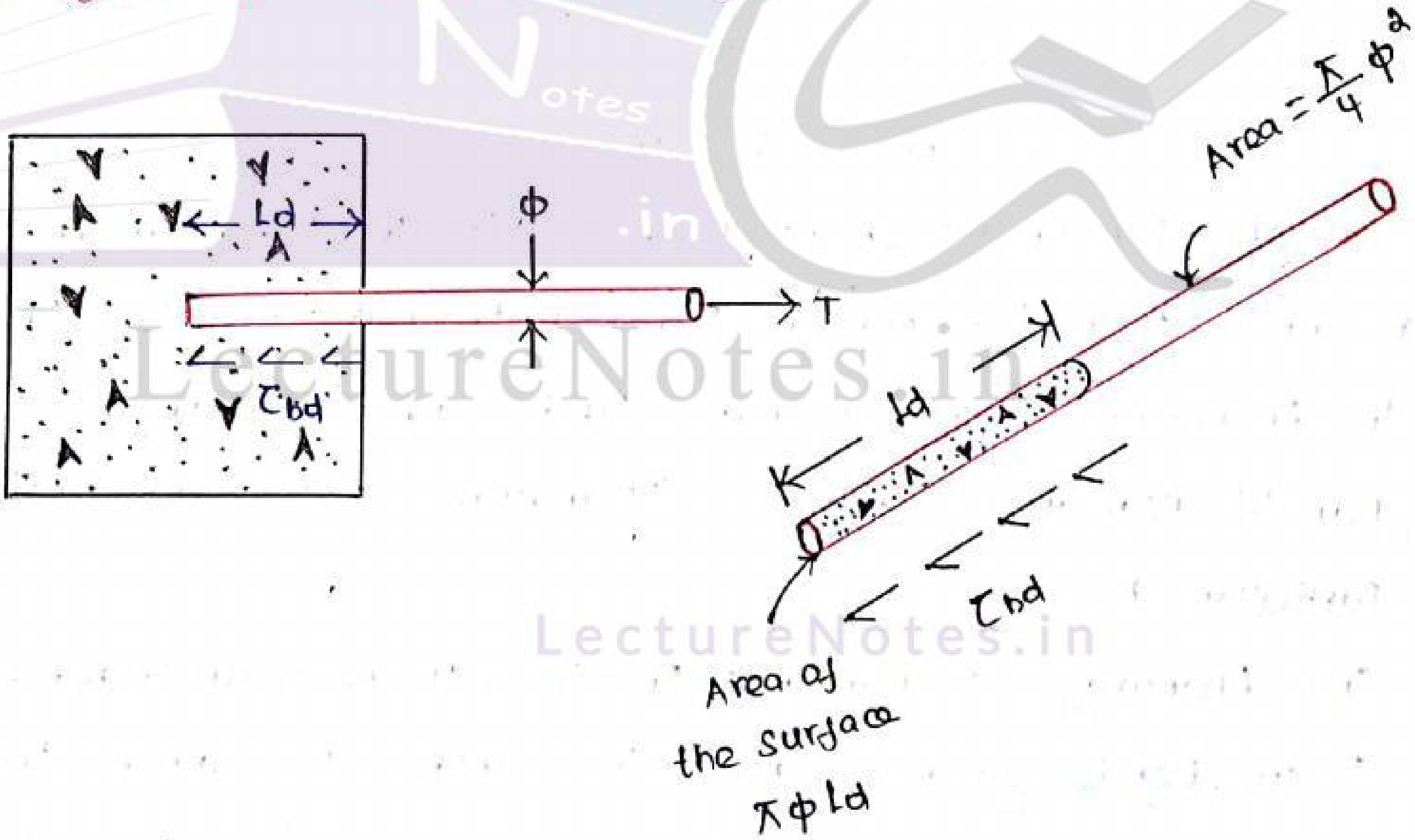
However it is to be Noted that the above expression is valid only for Mild Steel and Not for ribbed bar the reason is in ribbed bars there is considerable Increase in resistance to Flexural bond due to projection on it

In case of ribbed bars there is no Need to check for this for this Flexural bond.

Anchorage or Development Bond



The length of the bar needed to Transfer the stress to the concrete



The Bar Needs sufficient anchorage length to resist the force T is applied.

Reinforced bar is embeded in concrete and is subjected to a pull (T)

Max. pull that can be applied on bar (T) = Design stress x Area of bar

$$T = \sigma_s \times A_s$$

Max. applied force = Bond Resistance

$$\sigma_s \left(\frac{\pi}{4}\phi^2\right) = \tau_{bd} \times \text{Surface area of Embedment}$$

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

$\sigma_s = 0.87 f_y$

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

(OR)

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

Refer IS: 456-2000
Cl. 26.2.1
Pg. No: 42

Where

σ_s - Stress in bar at the section consider at Design Load

ϕ - Nominal diameter of the bar

τ_{bd} - Design bond stress

L_d - Development Length

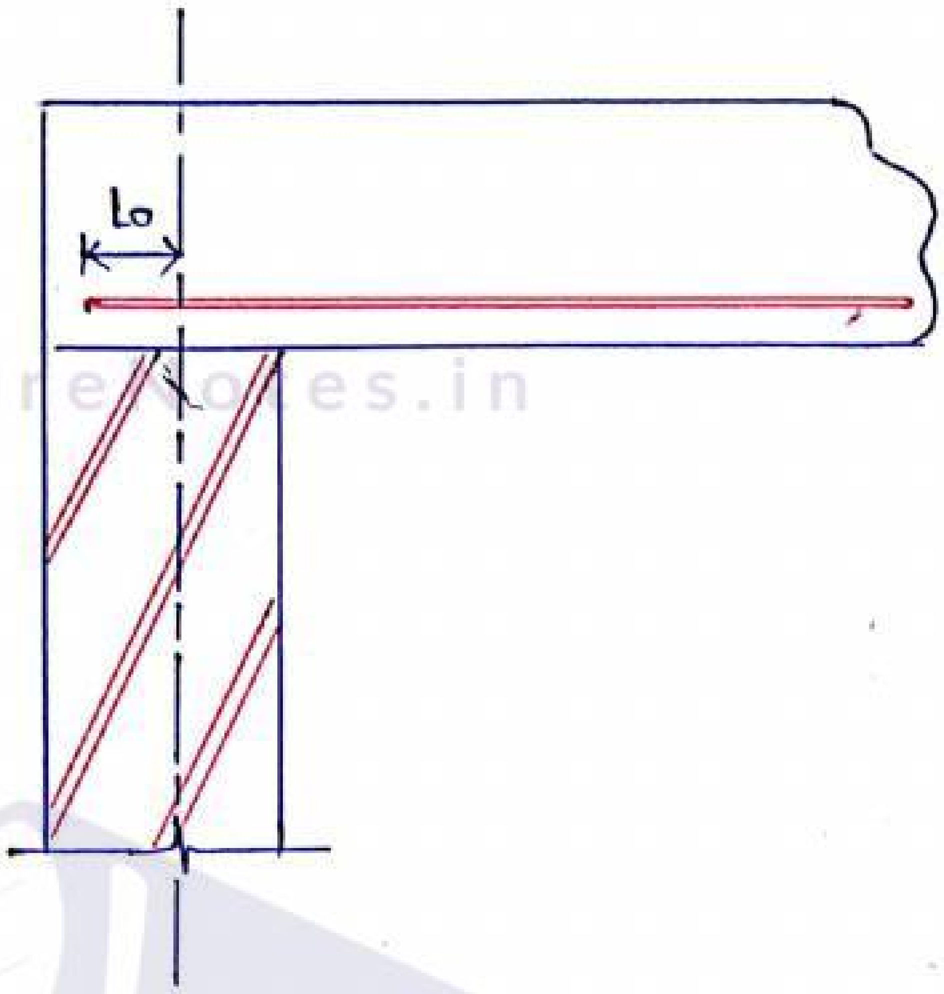
If the length is short it will slip. The Minimum anchorage length required to resist design force in the bar is called Anchorage length

* The Anchorage length of bars can be provided in the form of straight bar if available otherwise it may be pratically straight and partially Hooked

* The Anchorage is normally provided in the form of bends and Hooks
* If the bends started after the centre of support the anchorage length is at least 4ϕ bar but not greater than 12ϕ

Anchorage Values of Bend & Hook

- For every 45° bend it gives an anchorage of 4 dia, Subjected to maximum of 16 dia



The Minimum extension of Steel bar in a Structural member beyond theoretical Cut off point

Check for Anchorage length

Refer IS: 456-2000
Cl-26.2.3.3, Pg. No: 44

$$\boxed{L_d \ngtr \frac{M_u}{V} + L_o \quad \text{Fixed / Continuous}}$$

$$\boxed{L_d \ngtr 1.3\,\frac{M_1}{V} + L_o \quad \text{Simply Supported beam}}$$

Where,

M_1 - MOR of the beam c/s at a place where the bond is to be checked

V - Shear force due to External load

L_o - Original length

L_d - Development length

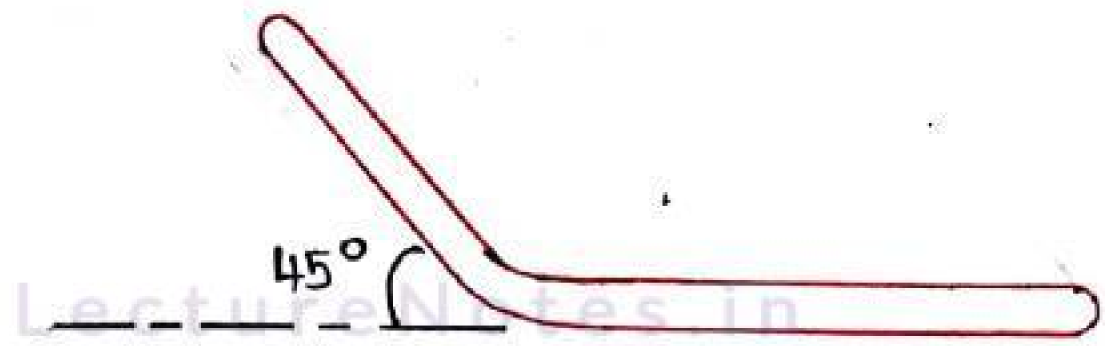
In Flexural member of beams $L_o < L_d$

(i) Bars in Tension

Refer IS: 456-2000
Cl: 26.2.2.1
Pg. No: 43

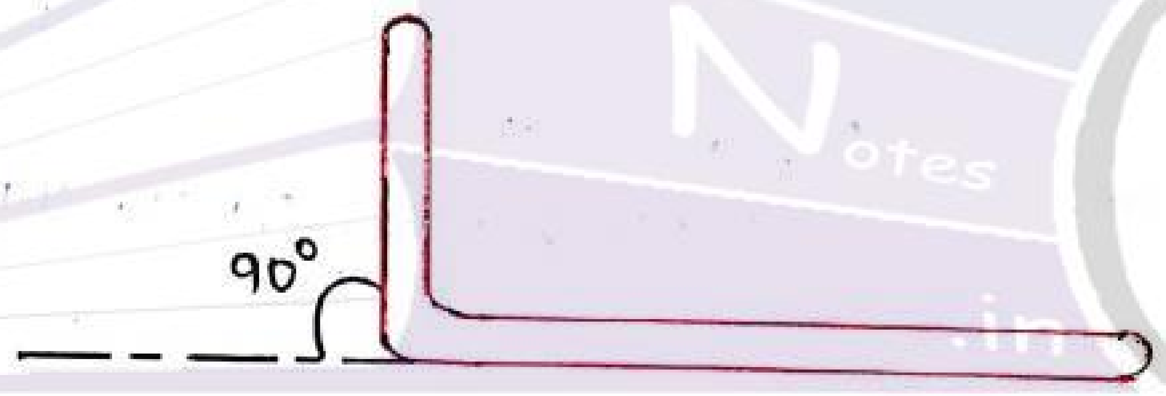
Angle of bending = 45°

Anchorage Value = 4ϕ



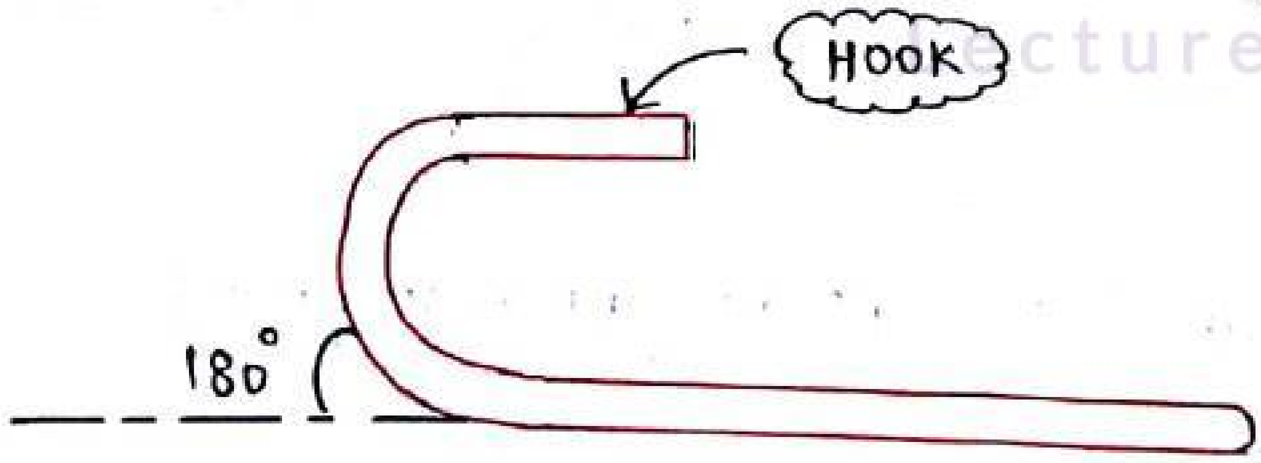
Angle of bending = 90°

Anchorage Value = 8ϕ



Angle of bending = 180°

Anchorage Value = 16ϕ



→ More than 180° bending is not allowed in concrete

→ 180° Hook is possible only with Mild steel

→ For HYSD, Maximum allowed is 135°

(ii) Bars in Compression

Refer IS : 456-2000
Cl : 26.2.2.2
Pg. No : 43

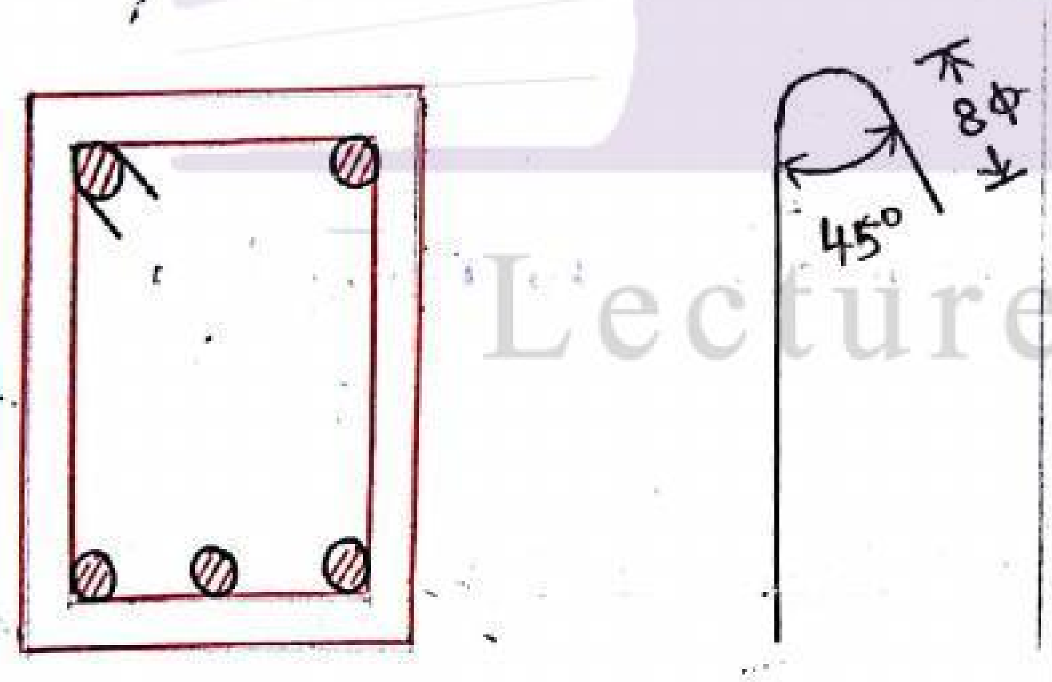
* The Anchorage length of Straight Compression bars shall be equal to its development length
* The development length shall include the projected length of Hooks, Bends and Straight length beyond bend, if provided

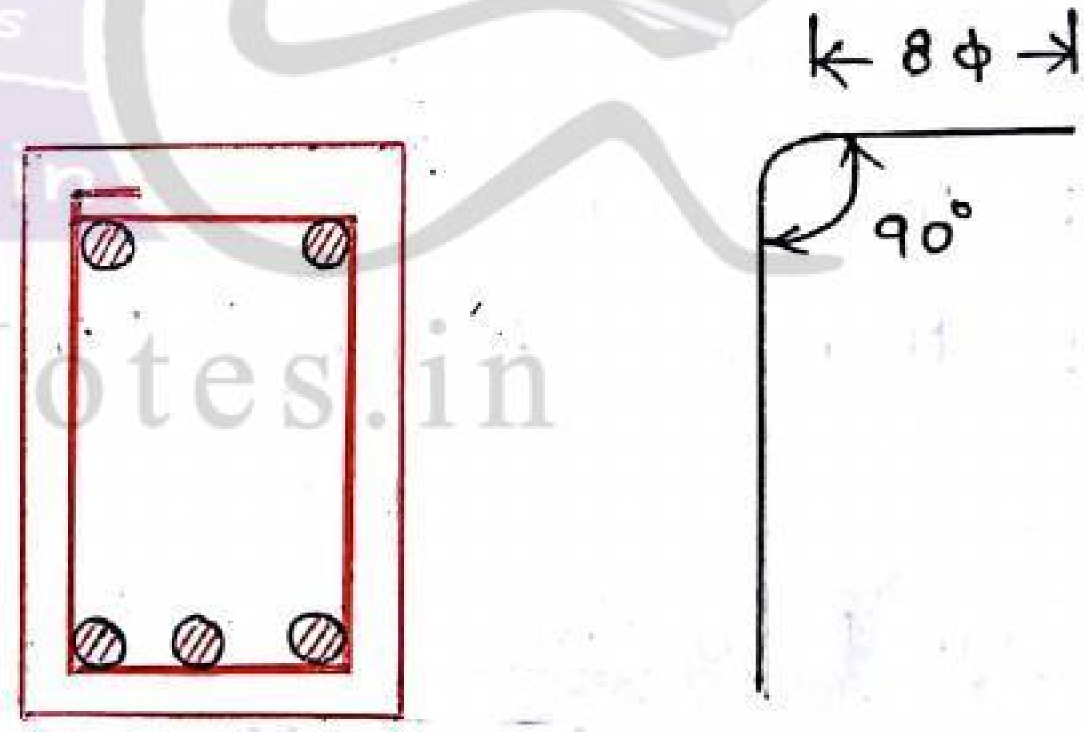
(iii) Bars in Shear

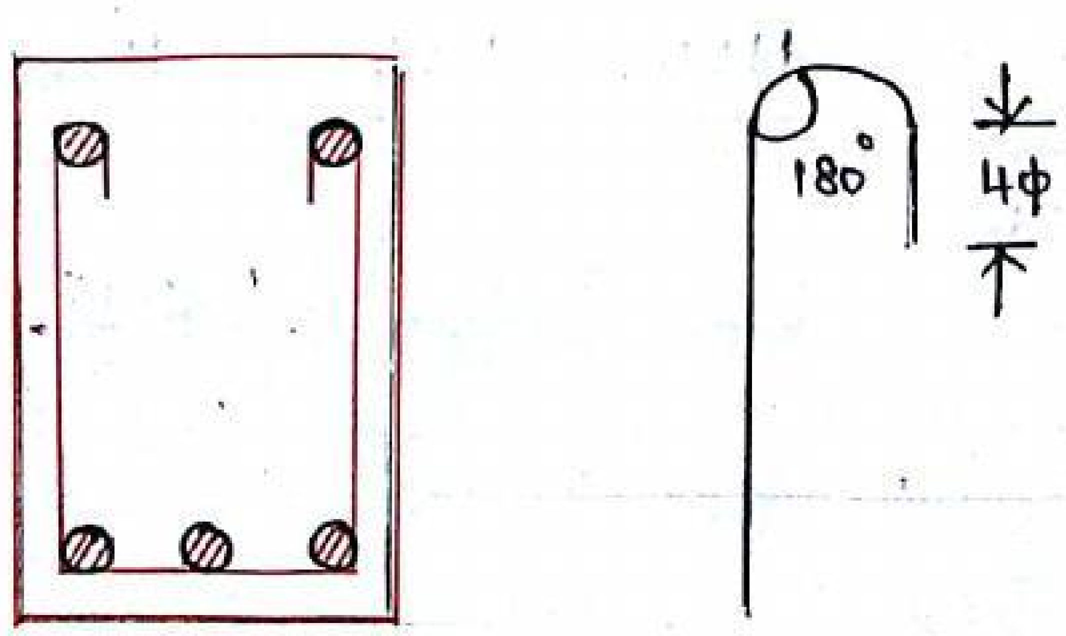
Refer IS : 456-2000
Cl. 26.2.2.4
Pg. No : 43

Refer SP : 34 - 1987
Pg. No : 61

Angle of Bend	Anchorage Value
45°	8ϕ
90°	8ϕ
180°	4ϕ







Inclined bars in Tension zone (Cl. No: 26.2.2.4 ; Pg. No: 43)

Development length equal to that of bars in tension and this length shall be measured from the end of sloping or Inclined portion of the bar

Inclined bars in Compression zone

The development Length equal together of bars in tension and this Length shall be measured from the Mid depth of the beam

How to Increase Bond Strength

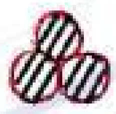
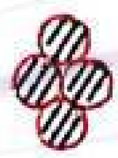
* Deformed bars (60% Increase bond stress)
* Closer Spacing of Stirrups to avoid crack
* Higher grade of Concrete (Increase bond Strength)
* Higher grade of Stirrups
* Increased Cover
* Increased bond Strength (L_d)
* Mechanical Anchorage (Use ribs, and Use Anchorage)

Design Bond Stress in LSM for PLAIN BARS In Tension

GRADE OF CONCRETE	M20	M25	M30	M35	> M40
DESIGN OF BOND STRESS (τ_{bd}) N/mm^2	1.2	1.4	1.5	1.7	1.9

TYPE OF STEEL	TYPE OF ZONE	τ_{bd} (N/mm^2)
Fe 250	Tension	τ_{bd}
	Compression	$1.25 \times \tau_{bd}$ (25% increase)
Fe 415 (HYSD) DEFORMED BARS	Tension	$1.60 \times \tau_{bd}$ (60% Increase)
	Compression	$1.25 \times \tau_{bd}$ (25% Increase)

Bundled bars

No. of bars	τ_{bd}
	Increase 10%
	20%
	33.33%

Bundled of bars becomes necessary

* When there are large number of bars required to be provided based on design
* It may not be possible to place the bars separately with necessary clearance

In such cases, there are two points

* Increase the size of the member (Column, Beams)
* Bundle the bar in group of Two, Three, Four

→ Unnecessary Cost (Implication due to the Increase in the Vol. of Concrete)

→ Engineers mostly resort (Bundle of bars)

→ IS Code also recommended not to bundled bars with dia more than 36mm

$\phi > 36mm$ (Not to bundle)

Development Length

(i) M15 Grade of Concrete
Fe 250 Grade of Steel } Tension

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

$$= \frac{0.87 \times 250 \times \phi}{4 \times 1}$$

$$\boxed{L_d = 54.37 \phi}$$

(ii) M20 Grade of Concrete
Fe 250 Grade of Steel } Compression

$$L_d = \frac{0.87 \times 250 \times \phi}{4 \times (1.2 \times 1.25)}$$

$$\boxed{L_d = 36.25 \phi}$$

(iii) M25 Grade of concrete } Tension
Fe250 Grade of Steel

$$= \frac{0.87 \times 250 \times \phi}{4 \times 1.4 \times 1}$$

$$\boxed{L_d = 38.83\,\phi}$$

(iv) M20 Grade of concrete } Compression
Fe415 Grade of Steel

$$= \frac{0.87 \times 415 \times \phi}{4 \times 1.25 \times 1.2}$$

$$\boxed{L_d = 60.175\,\phi}$$

(v) M25 Grade of Concrete } Tension
Fe415 Grade of Steel

$$= \frac{0.87 \times 415 \times \phi}{4 \times 1.6 \times 1.4}$$

$$\boxed{L_d = 40.29\,\phi}$$

(vi) M30 Grade of Concrete } Tension
Fe415 Grade of Steel

$$= \frac{0.87 \times 415 \times \phi}{4 \times 1.6 \times 1.5}$$

$$\boxed{L_d = 37.60\,\phi}$$

Design Equation

(1) Maximum Local bond stress (or) Flexural bond stress

(τ_b) (or) U_f

$$\tau_b = \left[\frac{V}{\Sigma o \cdot d}\right]$$

Where,

V = Shear force

Σo = Sum of perimeter ($\pi\phi$)

d = Effective depth

(2) Anchorage length (L_d)

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

(3) Average bond stress (τ_{bd})

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

Problem on Analysis of Bond

Prblm.No: 9

A Continuous beam having a width of 200mm and Effective depth of 300mm supports a Uniformly distributed load and is reinforced with 4 bars of 16mm diameter. If the factored total load is 80KN calculate

(a) The Max. local bond stress

(b) The Anchorage length required

(c) If the Anchorage length provided 900mm, the average bond stress assume

Use M20 Grade Concrete and Fe 415 HYSD bars

Given data

$b = 200\,mm$ $\quad$ $A_{st} = 4 \times \pi/4 \times 16^2$ $\quad$ To Find

$d = 300\,mm$ $\quad$ $= 804.24\,mm^2$ $\quad$ (i) τ_b (or) U_f

$V = 80\,kN$ $\quad$ $\Sigma o = 4(\pi \times 16) = 201\,mm$ $\quad$ (ii) L_d

$f_{ck} = 20\,N/mm^2$ $\quad$ $f_y = 415\,N/mm^2$ $\quad$ (iii) τ_{bd}

Solution

(a) Max. Local Bond Stress (τ_b (or) U_f)

$$\tau_b = \left[\frac{V}{\Sigma o \cdot d}\right] = \frac{80 \times 10^3}{201 \times 200}$$

$$\boxed{\tau_b = 1.32\,N/mm^2}$$

(b) Anchorage Length (Ld)

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

$$= \frac{0.87 \times 415 \times 16}{4 \times (1.6 \times 1)}$$

$$\boxed{L_d = 752.18 mm}$$

(c) Anchorage Bond Stress (τ_{bd})

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

$$= \frac{0.87 \times 415 \times 16}{4 \times 900} = 1.60 N/mm^2$$

$$\boxed{\tau_{bd} = 1.60 N/mm^2}$$

Prblm.no:10

Determine the Anchorage length of 3-16ϕ reinforcing the bars going into the support of the simply supported beam. The factored shear force V_u = 110 KN. Width of the column support 250mm, Depth of the beam 400mm Eff. cover = 40mm. provide hook or bond if required Use M20 Grade of concrete and Fe 415 Grade of steel.

Given data

V_u = 110 KN

b = 250 mm

D = 400 mm, d' = 40 mm

f_{ck} = 20 N/mm², f_y = 415 N/mm²

To Find

L_d = ?

Solution

(1) To Find the development length (L_d)

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

$$= \frac{0.87 \times 415 \times 16}{4 \times 1.2 \times 1.6}$$

$$\boxed{L_d = 751 mm}$$

To check

For simply supported beam

$$L_d \not> 1.3 \frac{M_1}{V} + L_0$$

(2) Moment of Resistance at NA

(i) Neutral axis

$$x_u = \frac{0.87 f_y \cdot A_{st}}{0.36 f_{ck} \cdot b} \Rightarrow \frac{0.87 \times 415 \times 603.18}{0.36 \times 20 \times 250}$$

$$\boxed{x_u = 120.98 mm}$$

(ii) Maximum depth of NA

$$x_{umax} = 0.48 d \quad (Fe\ 415)$$

$$= 0.48 \times 360$$

$$\boxed{x_{umax} = 172.88 mm}$$

$$x_u < x_{umax}$$

Hence it is under Reinforced section

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

$$= 0.87 \times 415 \times 602.18 \times 260 \left[1 - \frac{602.18 \times 415}{250 \times 260 \times 20}\right]$$

$$= 78.40 \times 10^6 - 10.90 \times 10^6$$

$$\boxed{M_u = 67.49\ KN\cdot m}$$

(3) Check for Anchorage length

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

$$\ngtr 1.3 \times \frac{67.49}{110} + 0$$

$$751 \ngtr 797\ mm$$

OK Safe

Prblm.No: 11

Determine the Anchorage value of steel at the support of a simply supported beam subjected to shear force of 300 KN. There are 3 bars of 20 mm dia at the support. The size of the beam is 300 mm × 500 mm and the support length is 220 mm. Use M20 & Fe 415

Given data

$$V = 300\ KN$$

$$A_{st} = 3 \times \frac{\pi}{4} \times 20^2$$

$b = 200mm$

$D = 500mm$

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

$d = 465mm$

Solution

To find the Development Length

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} \Rightarrow \frac{0.87 \times 415 \times 20}{4 \times 1.2 \times 1.6} = 940.3mm$$

To check

For simply supported beam

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

(i) Neutral axis

$$x_u = \frac{0.87 \times f_y \times A_{st}}{0.36 \times f_{ck} \times b} = \frac{0.87 \times 415 \times 942.47}{0.36 \times 20 \times 200}$$

$$\boxed{x_u = 157.53mm}$$

(ii) Max. depth of Neutral axis

$x_{umax} = 0.48d$ (Fe 415)

$= 0.48 \times 465$

$$\boxed{x_{umax} = 223.2mm}$$

$x_u < x_{umax}$

Hence it is under Reinforced section

(iii) Moment of Resistance

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

$$= 135\ KN \cdot m$$

$$L_d \ngtr 1.3 \frac{M_1}{V} + L_o$$

Assume No hooks are not provided

$$L_o = 0$$

$$= 1.3 \times \frac{135 \times 10^6}{200 \times 10^3} - 940$$

$$L_o = 255\ mm$$

$$940.2\ mm \ngtr 255\ mm$$

Hence Condition Not satisfied

If above condition are not satisfied then it becomes unsafe in Anchorage, Development length and diameter

$$940 - \left(1.2 \times \frac{135 \times 10^6}{200 \times 10^3}\right) = L_o$$

$$L_o = 255\ mm$$

Available

$$L_o = \frac{Ls}{2} - Cs$$

$$= \frac{220}{2} - 2\phi$$

$$= \frac{220}{2} - 2(20)$$

$$L_o = 75\ mm$$

L.S – Length of Support

CS – End Cover

↓

More than clear cover

Eg: 25 mm (or) 2ϕ

Crank up size = 355 - 75

= 280 mm

Minimum crank up = 8 ϕ

= 8 × 20

= 160 mm

Hence ok

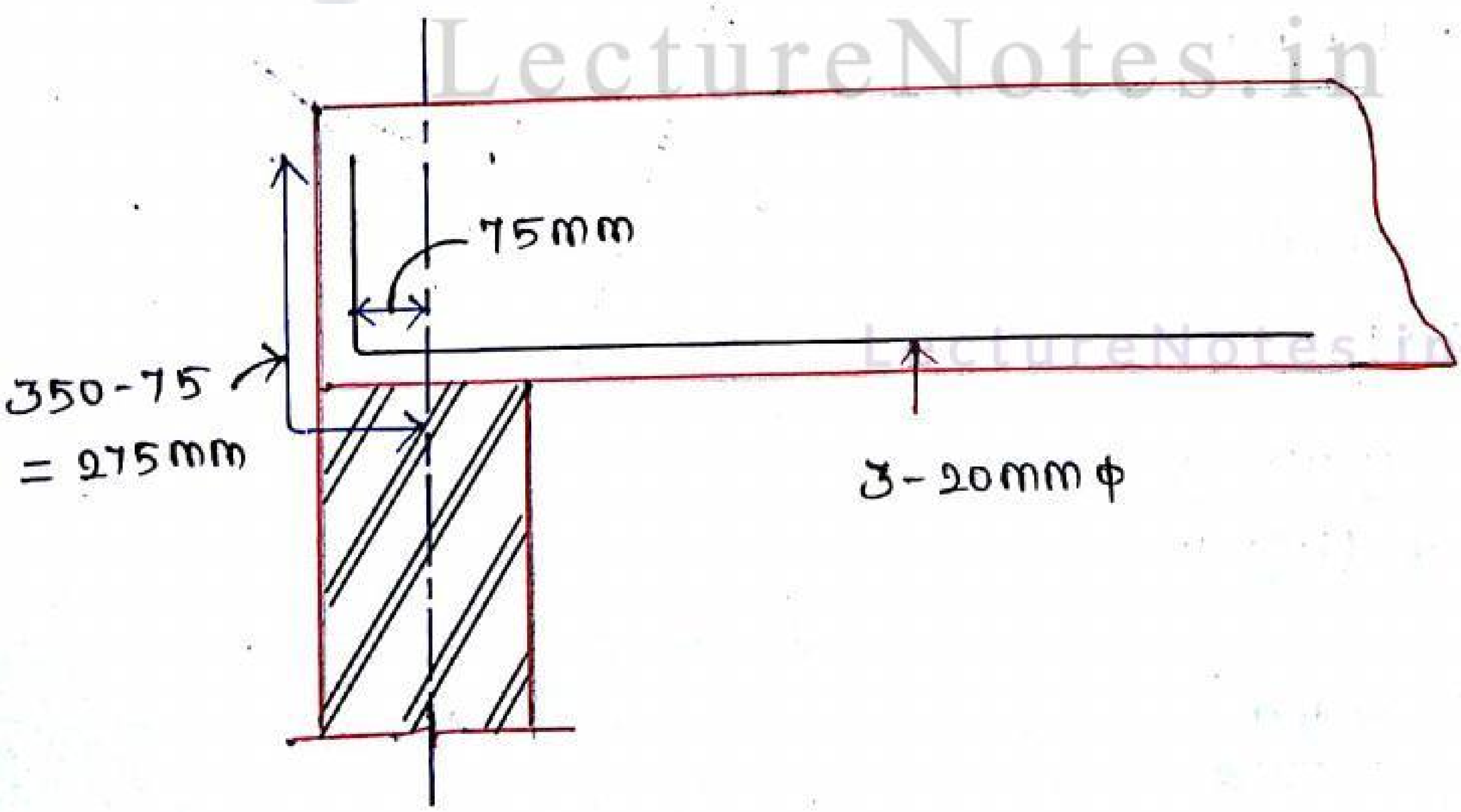
(iv) Check for Minimum Extension

$= \frac{L_d}{3} \Rightarrow \frac{940}{3} \Rightarrow 313\text{mm}$

∴ Provide Extension

$= 355 + \frac{230}{2} \Rightarrow 470\text{mm} > \frac{L_d}{3}$

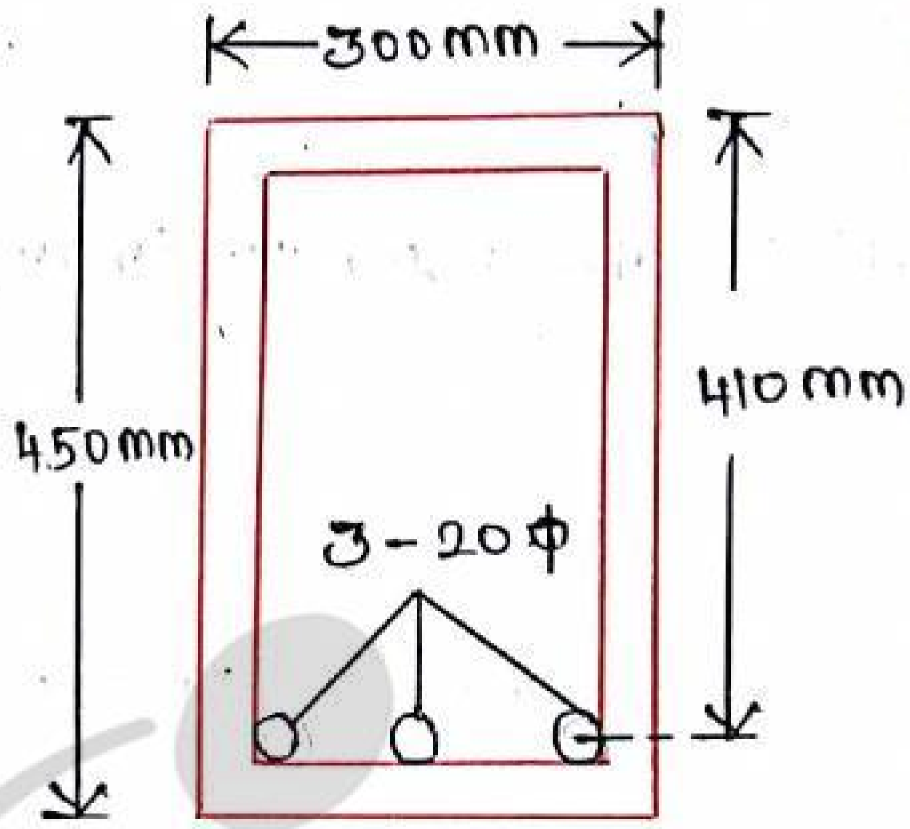
Hence ok



Prblm. No : 12

Check the bond requirement of the Continuous beam, if the Factored Shear Force is 200 KN at the point of Inflection Assume M20 Grade of concrete, Fe 415 Steel, breadth 300mm overall depth 450mm and bars of 3 - 20ϕ

Given data



$b = 300mm$

$D = 450mm$

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

$f_y = 415 \ N/mm^2$

$d = 450 - \left(25 + 8 + \frac{20}{2}\right)$

$= 407 \simeq 410mm$

$d = 410mm$

Solution

Development Length

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

$$= \frac{0.87 \times 415 \times 20}{4 \times (1.2 \times 1.6)}$$

$$\boxed{L_d = 940.23 \ mm}$$

(i) Neutral axis

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

$$= \frac{0.87 \times 415 \times 3 \times \frac{\pi}{4} \times (20)^2}{0.36 \times 20 \times 200}$$

$$= 157.53\,mm$$

(ii) Maximum depth of Neutral axis

$x_{umax} = 0.48d$ (Fe415)

$= 0.48 \times 410$

$= 196.8\,mm$

$x_u < x_{umax}$

Hence it is URS

(iii) Moment of Resistance

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

$= 0.87 \times 415 \times 2 \times \frac{\pi}{4}(20)^2\,(410 - 0.42 \times 157.07)$

$\boxed{M_u = 117.067\ KN\cdot m}$

(iv) Check for Anchorage length

$$L_d \not> \frac{M_u}{V} + L_o$$

$$940.22 \not> \frac{117.001 \times 10^2}{200} + L_o$$

$940.22 \not> 585.005$ (Not safe)

$L_o = 355.225 \simeq 355\,mm$

Available

$$L_o = \frac{L_s}{2} - C_s$$

$$= \frac{300}{2} - 2\phi$$

$$= \frac{300}{2} - 2(20) \Rightarrow 130\,mm$$

Crank up size $= 355 - 130$

$= 225\,mm$

(v) Check for Minimum Extension

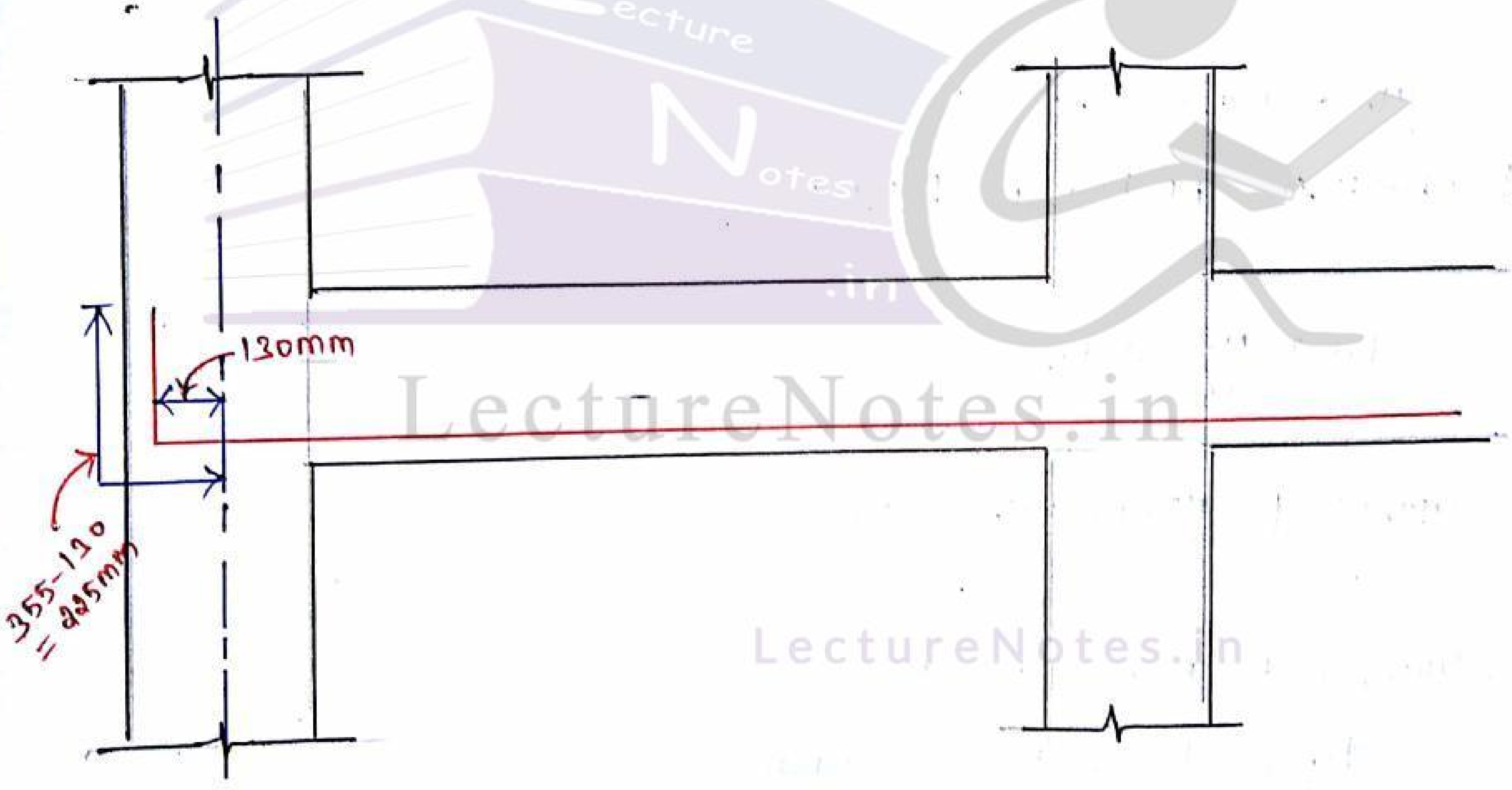
$$\frac{L_d}{2} = \frac{940.23}{2} = 313.41\,mm$$

Provide Extension

$$= 130 + \frac{300}{2} + 225$$

$$= 505 > \frac{L_d}{2}$$

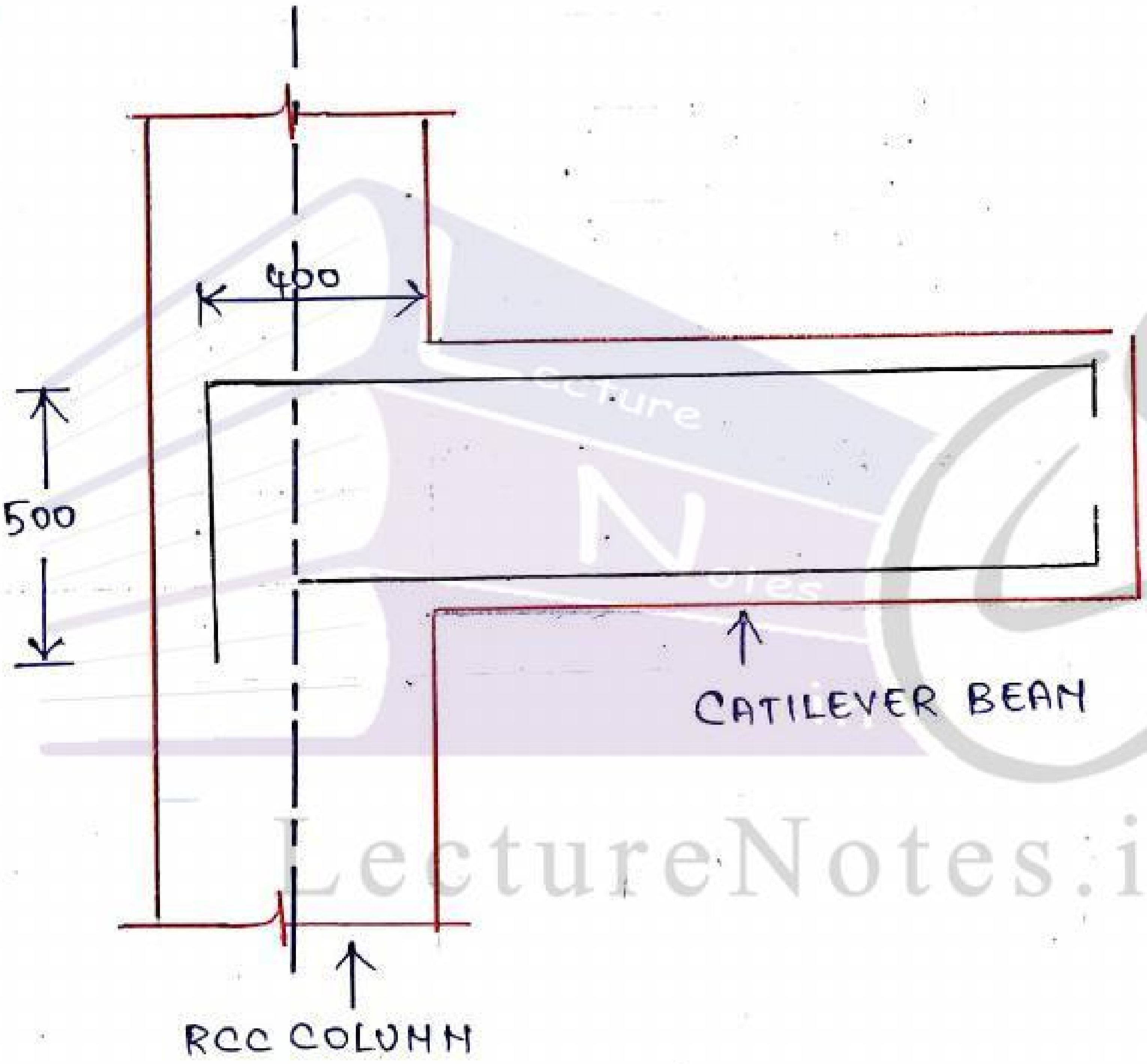
Hence ok



CANOPY BEAM

Pblm. No: 12

A Reinforced Concrete cantilever beam of rectangular section 300 mm wide by 600 mm deep is built into a column 500 mm wide as shown in Fig. The cantilever beam is subjected to a hogging moment of 200 kN·m at the function of beam and column. Design suitable reinforcement in the beam and check for required anchorage length. Adopt M20 Grade of concrete and Fe 415 HYSD bars.



Given data

$b = 300\,mm$

$d = 550\,mm$

$M_u = 200\,kN\cdot m$

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

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

(i) Limiting Moment of Resistance

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

$$= 0.138 \times 20 \times 200 \times (550)^2$$

$$= 250 \text{ KN·m}$$

$$M_u < M_{ulim}$$

Hence it is URS

(ii) Reinforcement

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

$$200 \times 10^6 = 0.87 \times 415 \times A_{st} \times 550 \left[1 - \frac{415 A_{st}}{200 \times 550 \times 20}\right]$$

$$A_{st} = 1181 \text{ mm}^2$$

$$\text{No. of bars} = \frac{A_{st}}{a_{st}}$$

$$= \frac{1181}{\frac{\pi}{4} d^2} = \frac{1181}{\frac{\pi}{4}(20)^2}$$

$$(NOB)_{prov} = 3.75 \simeq 4$$

$$A_{st\,prov} = n \times \frac{\pi}{4} \times d^2$$

$$= 4 \times \frac{\pi}{4} \times (20)^2$$

$$\boxed{A_{st\,prov} = 1256 \text{ mm}^2}$$

Provide 4 bars of 20 mm ϕ ($A_{st} = 1256\, mm^2$)

(iii) Anchorage Length

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

$$= \frac{0.87 \times 415 \times 20}{4 \times 1.6 \times 1.2}$$

$$\boxed{L_d = 940mm}$$

The bars are extended into the column to a length of 400mm with a 90° bend and 500mm length

$$\left.\begin{array}{r}\text{Anchorage length}\\ \text{provided}\end{array}\right\} = [400 + (8 \times 20) + 500]$$

$$= 1060mm$$

$$1060mm > 940mm$$

Hence ok

UNIT-4

DESIGN OF FOOTING BY LSM

FOOTING

Footing are structural members used to support column and Wall and to transmit and distribute their load to the soil in such way that the load bearing capacity of the soil is not exceed excessive settlement, differential settlement or rotation are prevented and adequate safety against overturning or sliding is maintained.

Real life Examples

* Footing is derived from word foot
* part of a Leg
* Stability of a Entire body - to stand upon it
* Footing is the part of the foundation

Structure Consists of Two parts

* Sub - Structure
* Super Structure

FOUNDATION

* Foundation is defined as that part of the structure that transfers the load from the structure as well as its own weight over a large area of soil in such a way that the load doest not exceed the ultimate bearing capacity of the soil and the settlement of the total structure remains within a tolerable limit.
* Foundation is the part of a structure on which the building stands. The solid ground on which the foundation rests is called the Foundation bed.

Type of Foundation

Foundations are divided into Two Categories such as

* Shallow Foundation
* Deep Foundation

Shallow Foundation

If the width of the foundation is greater than the depth of the foundation it is labeled as "Shallow foundation"

$$D < B$$

Deep Foundation

If the width of the foundation is smaller than the depth of the foundation it is called as "Deep Foundation"

$$D > B$$

Why a Foundation is provided

* Distribute the weight of the structure over a large area of soil.
* Avoid unequal settlement
* Prevent the lateral movement of the structure
* Increase structural stability

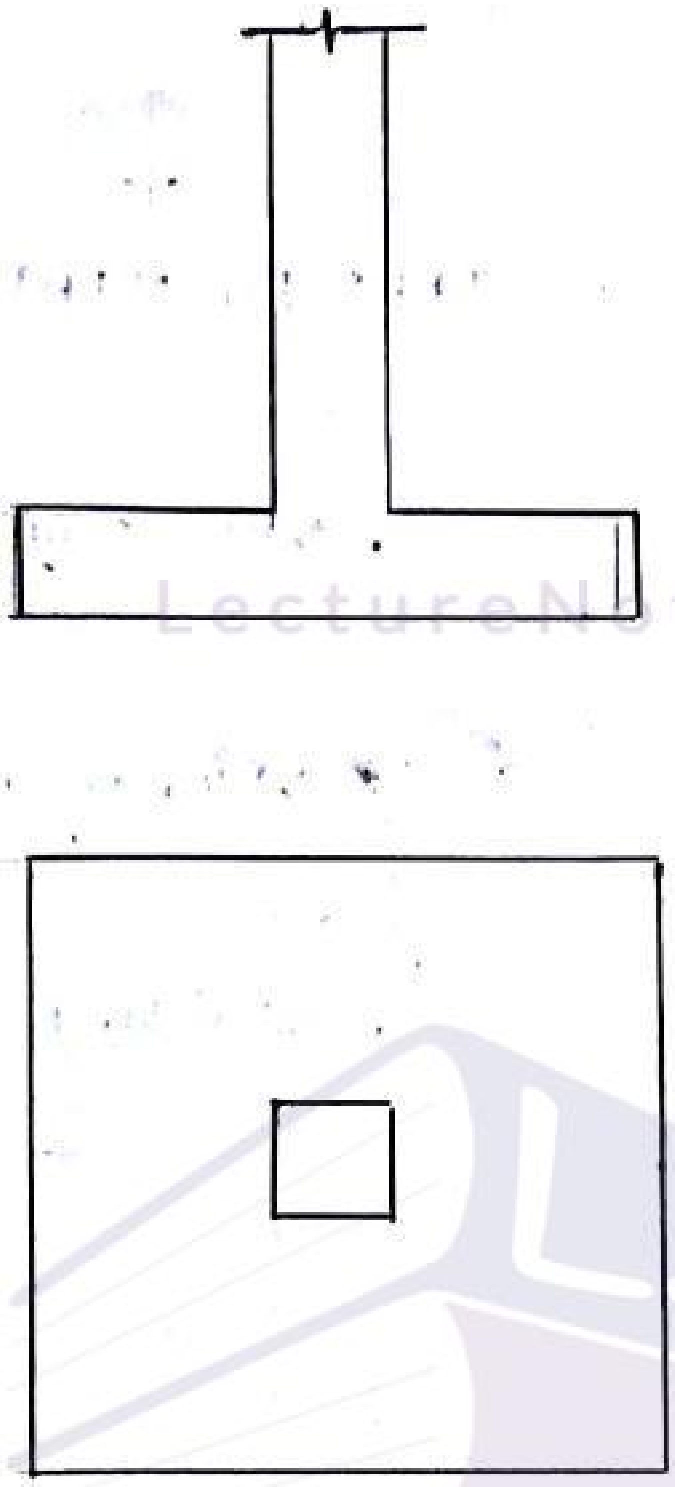
Why There are Different Types of Foundation

* There are different types of soil and bearing capacity of the soil is different for each individual type of soil
* So depending on the soil profile, size and load of the structure, Engineers choose different types of foundation

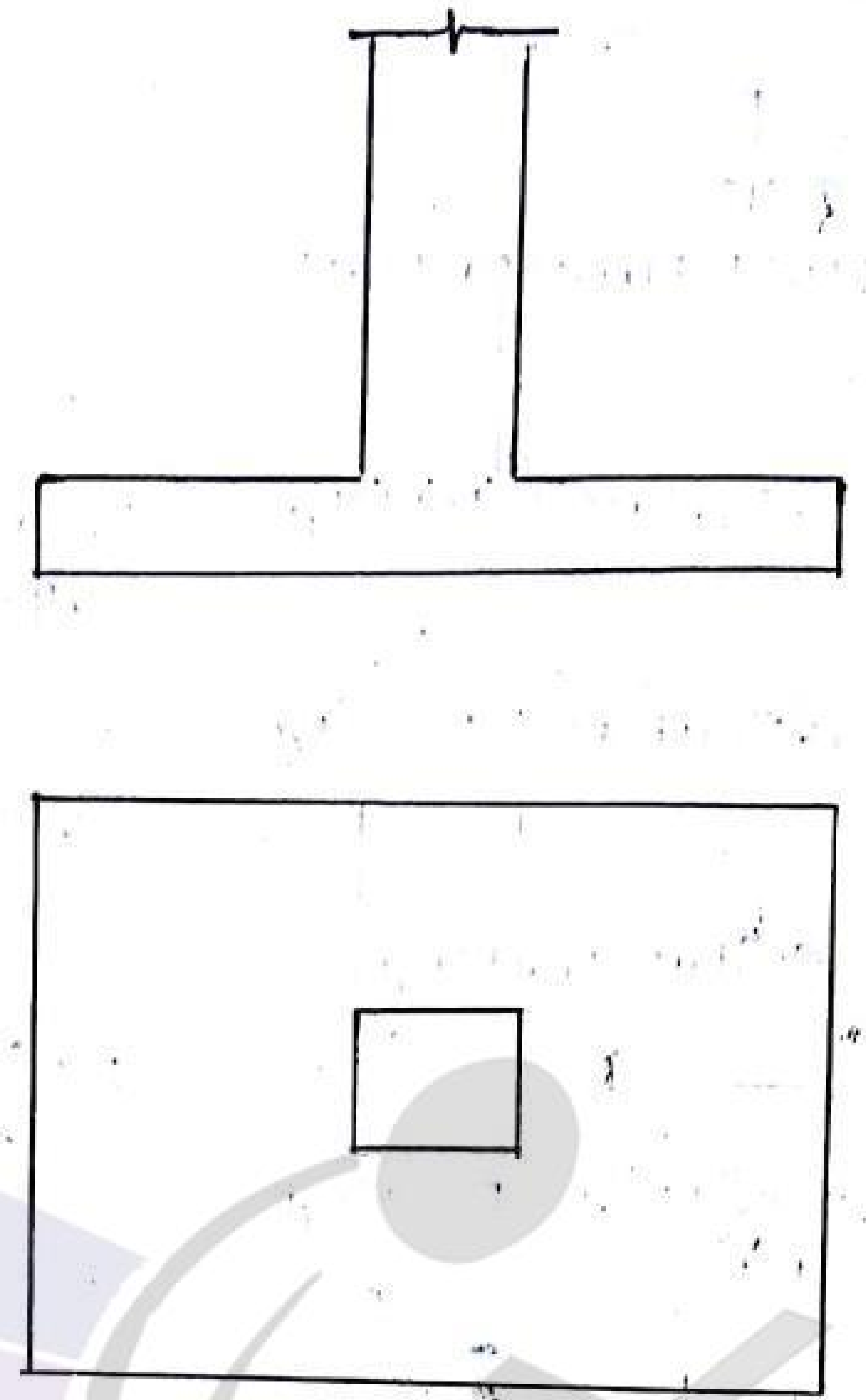
FOUNDATION

- SHALLOW FOUNDATION
  - Isolated Footing
    - Square Footing
    - Rectangular
    - Sloped
    - Circular
  - Combined Footing
  - Strip Footing
  - Spread Footing
  - Stepped Footing
  - Raft or Mat footing
  - Eccentric Footing
  - cantilever or Strap footing
- DEEP FOUNDATION
  - Pile foundation
  - Pier foundation
  - Driller foundation

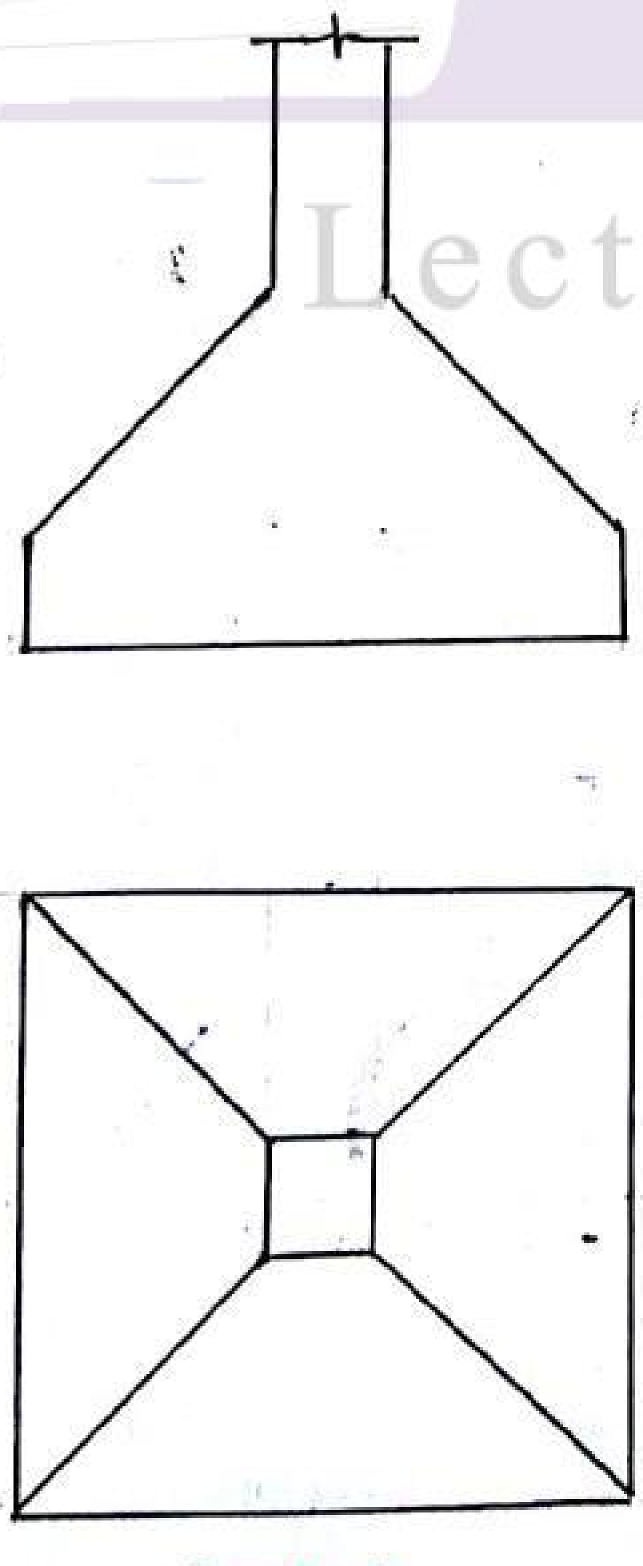
Isolated Footing



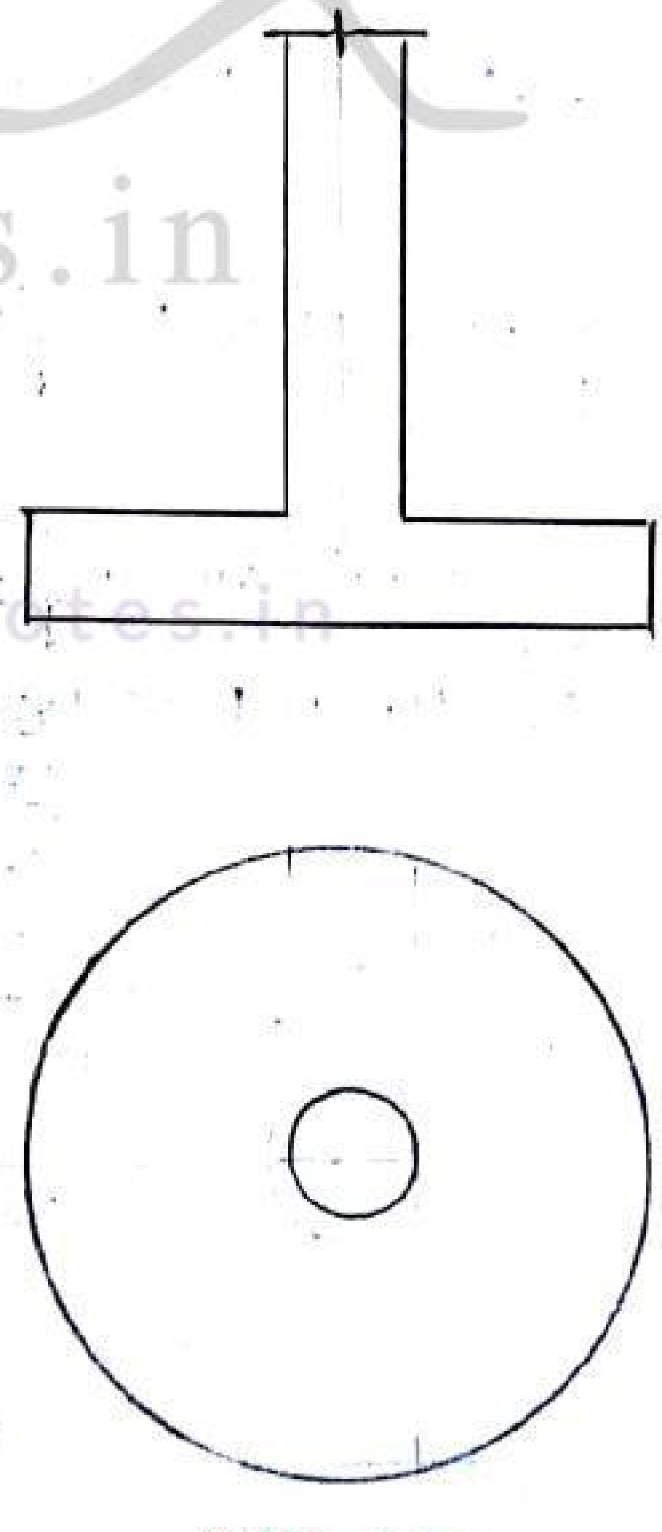
SQUARE FOOTING



RECTANGULAR



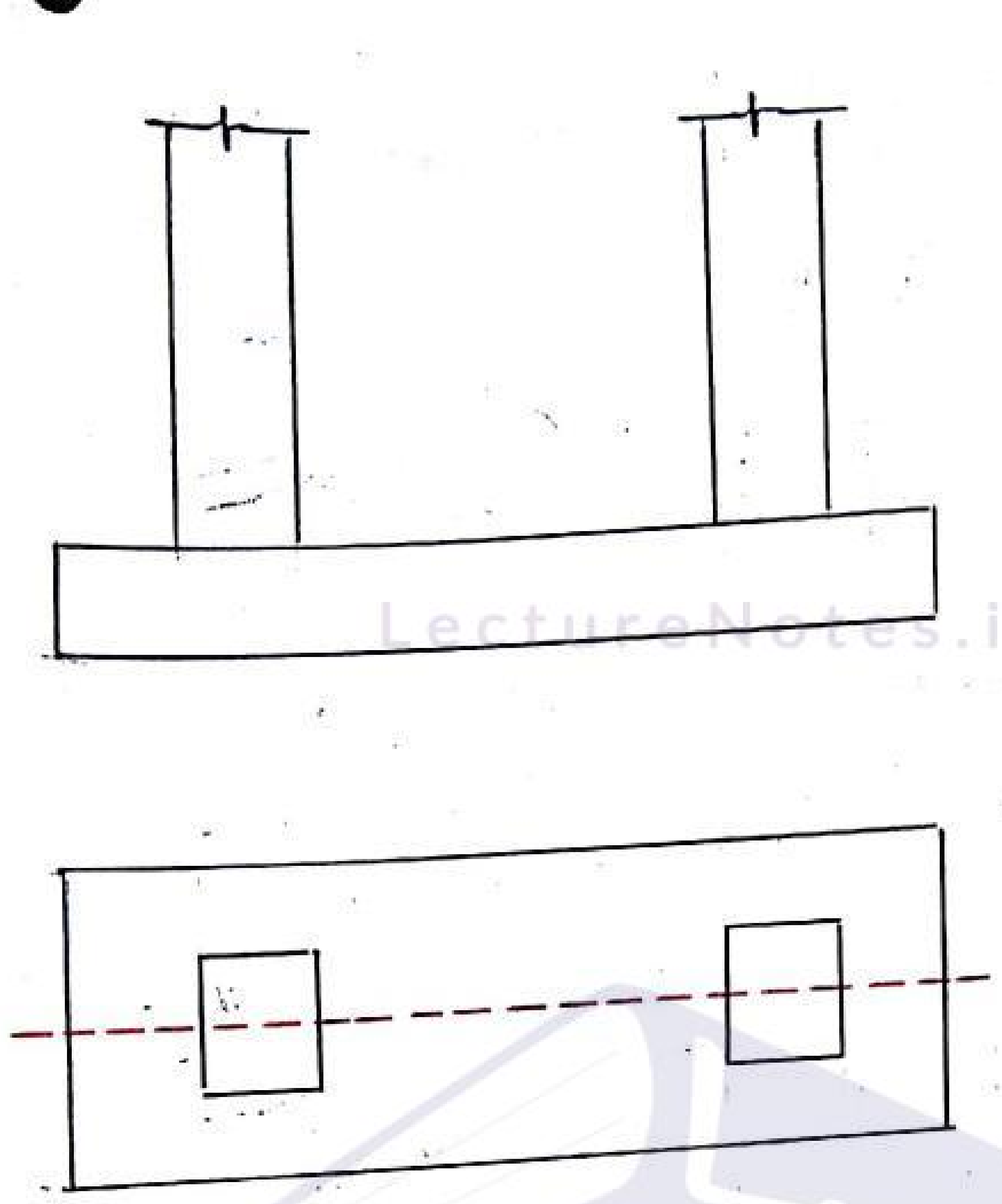
SLOPED



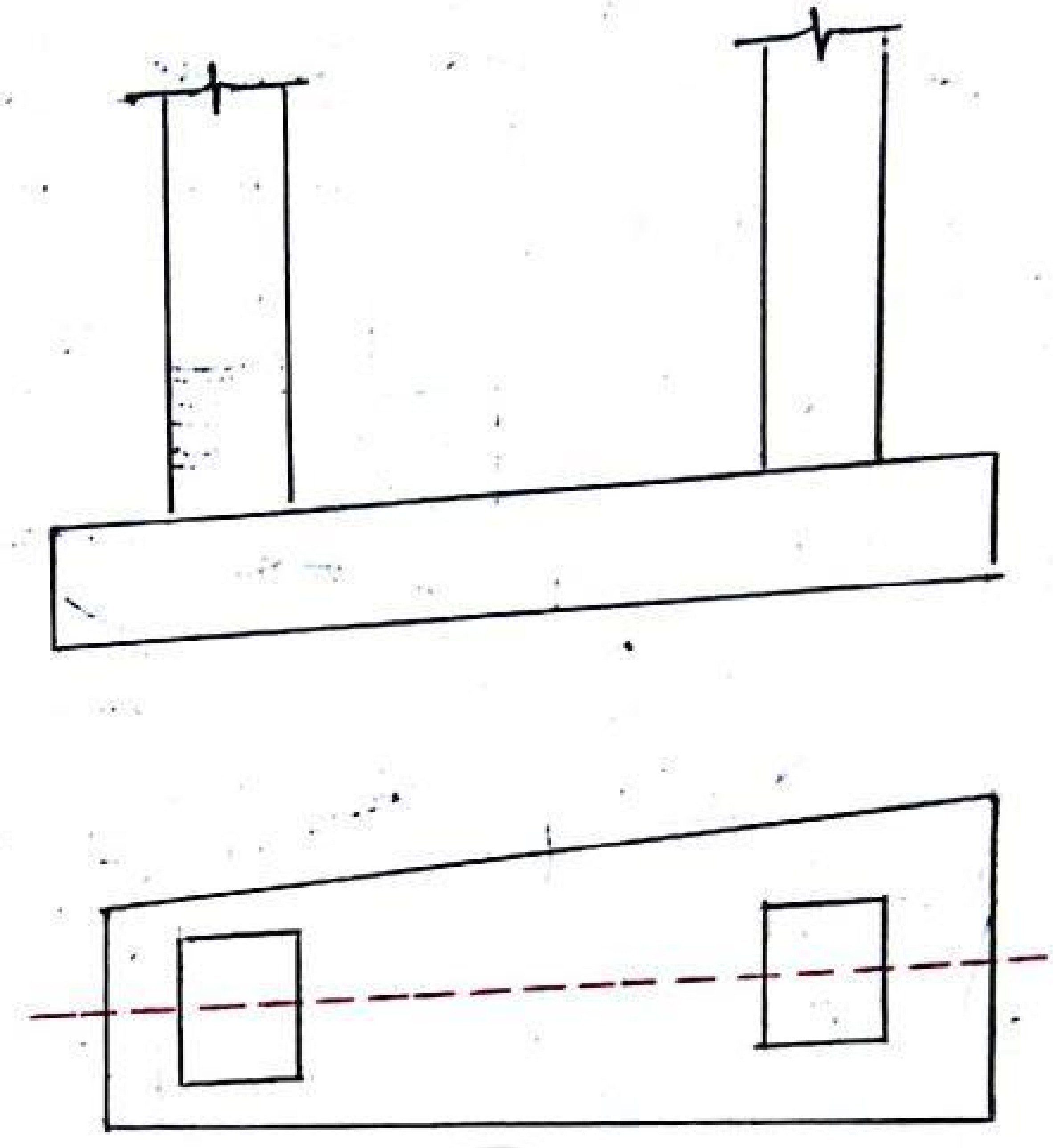
CIRCULAR

Combined Footing

•

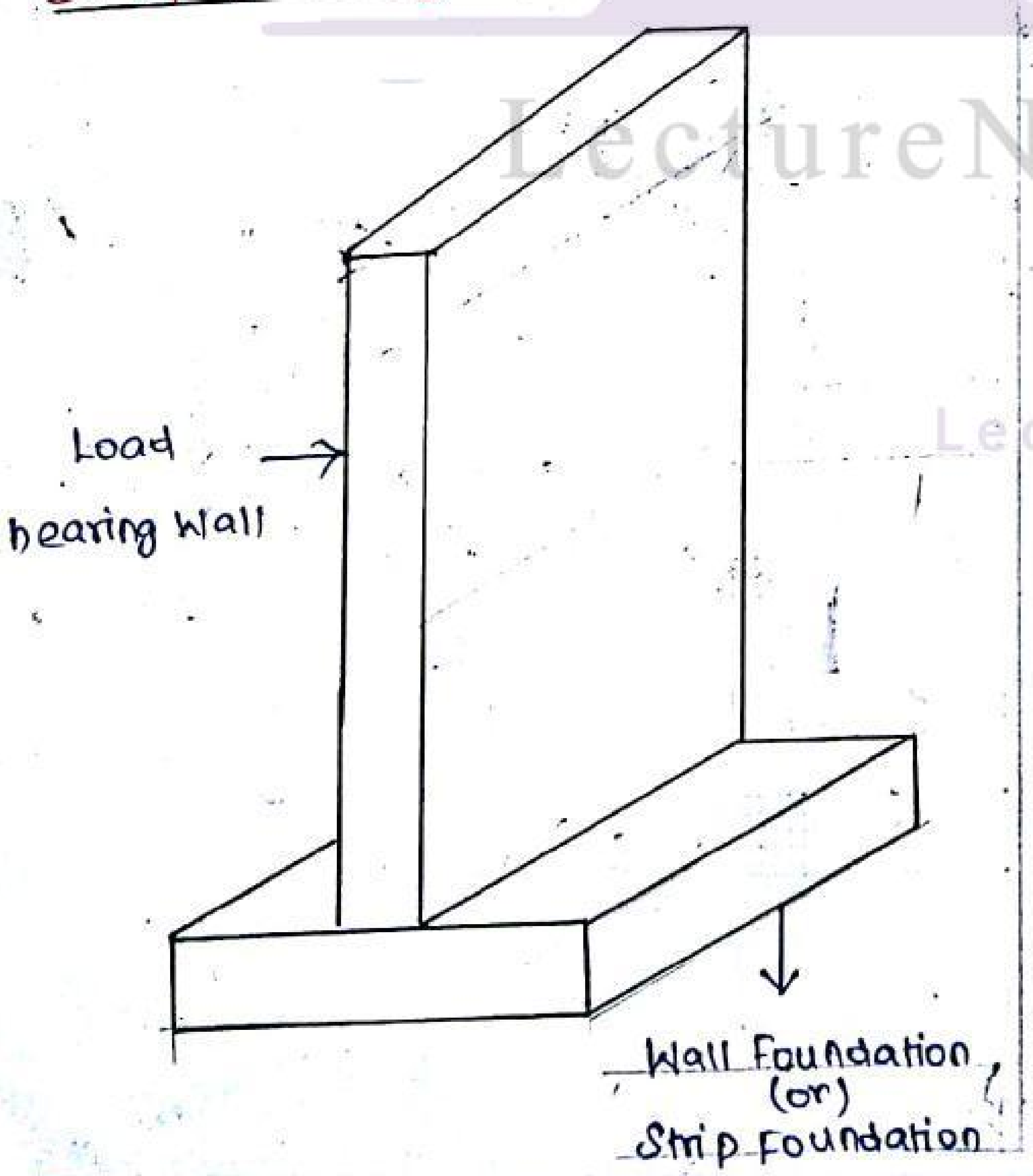


RECTANGULAR COMBINED FOOTING

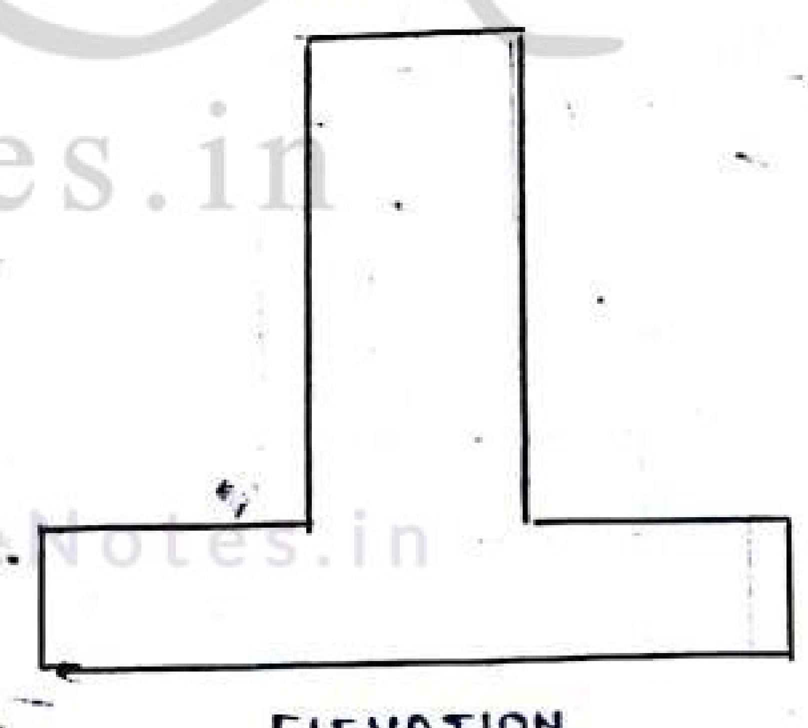


TRAPEZOIDAL COMBINED FOOTING

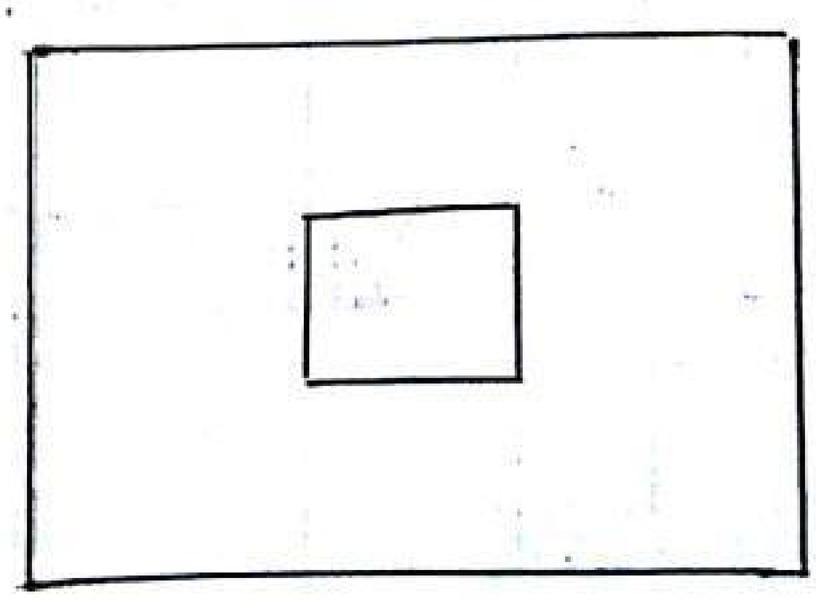
Strip Footing



Spread Footing

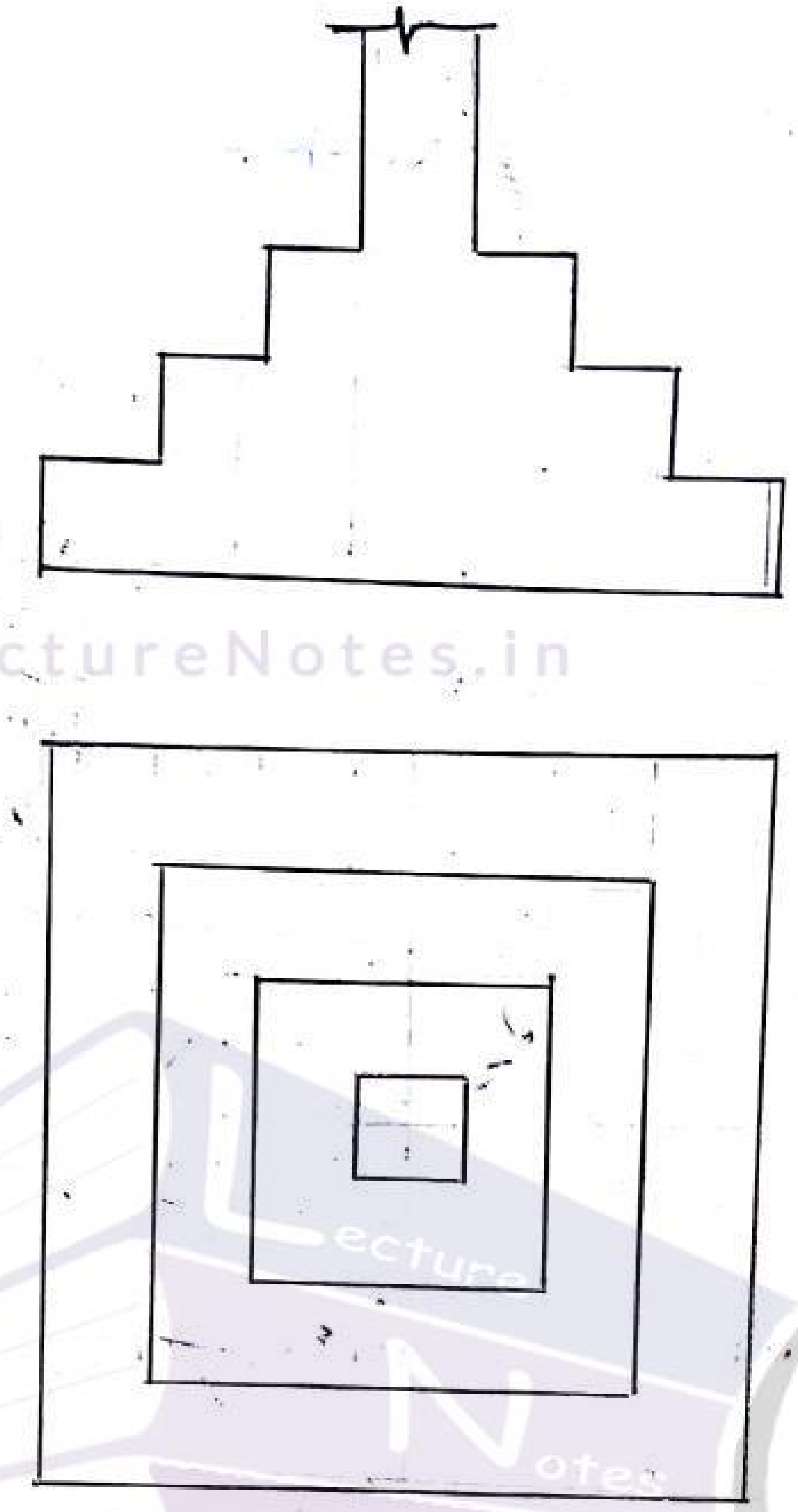


ELEVATION

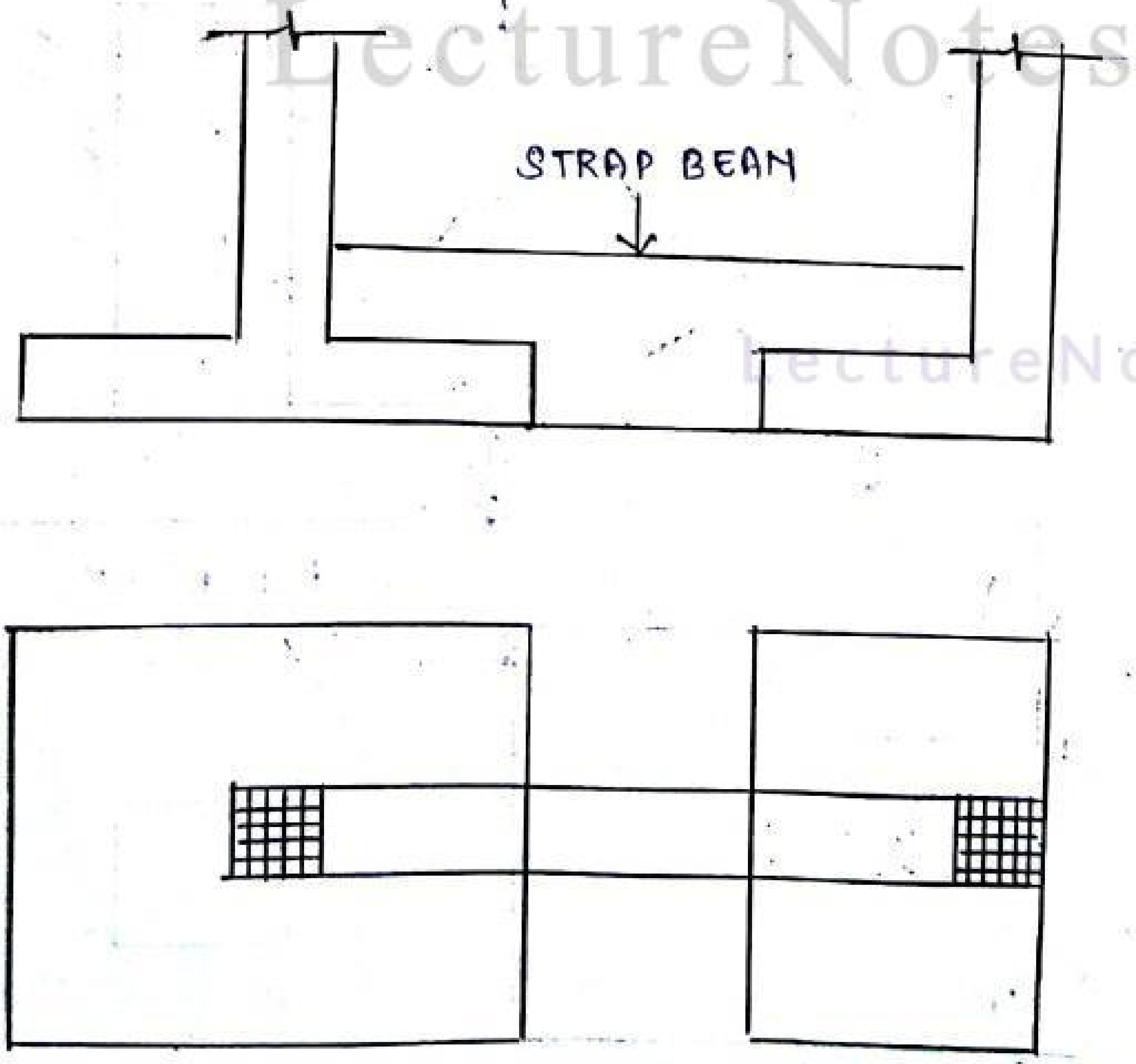


PLAN

Stepped Footing



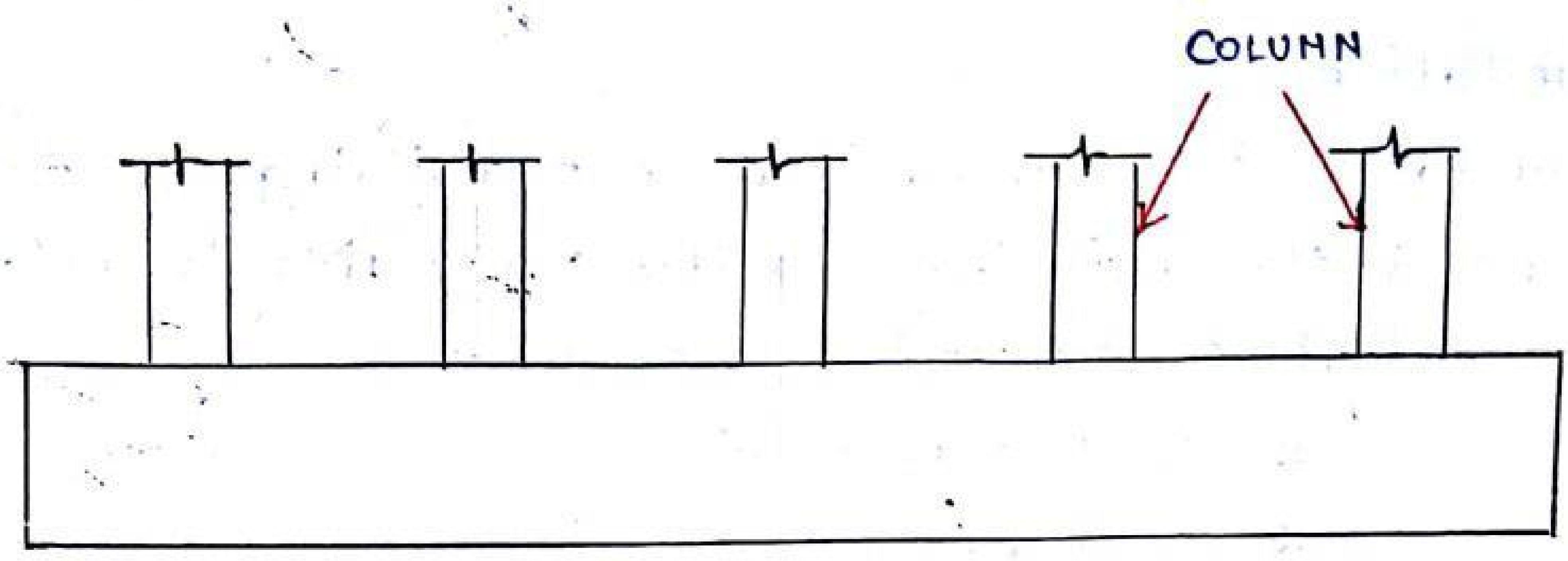
Cantilever or Strap Footing



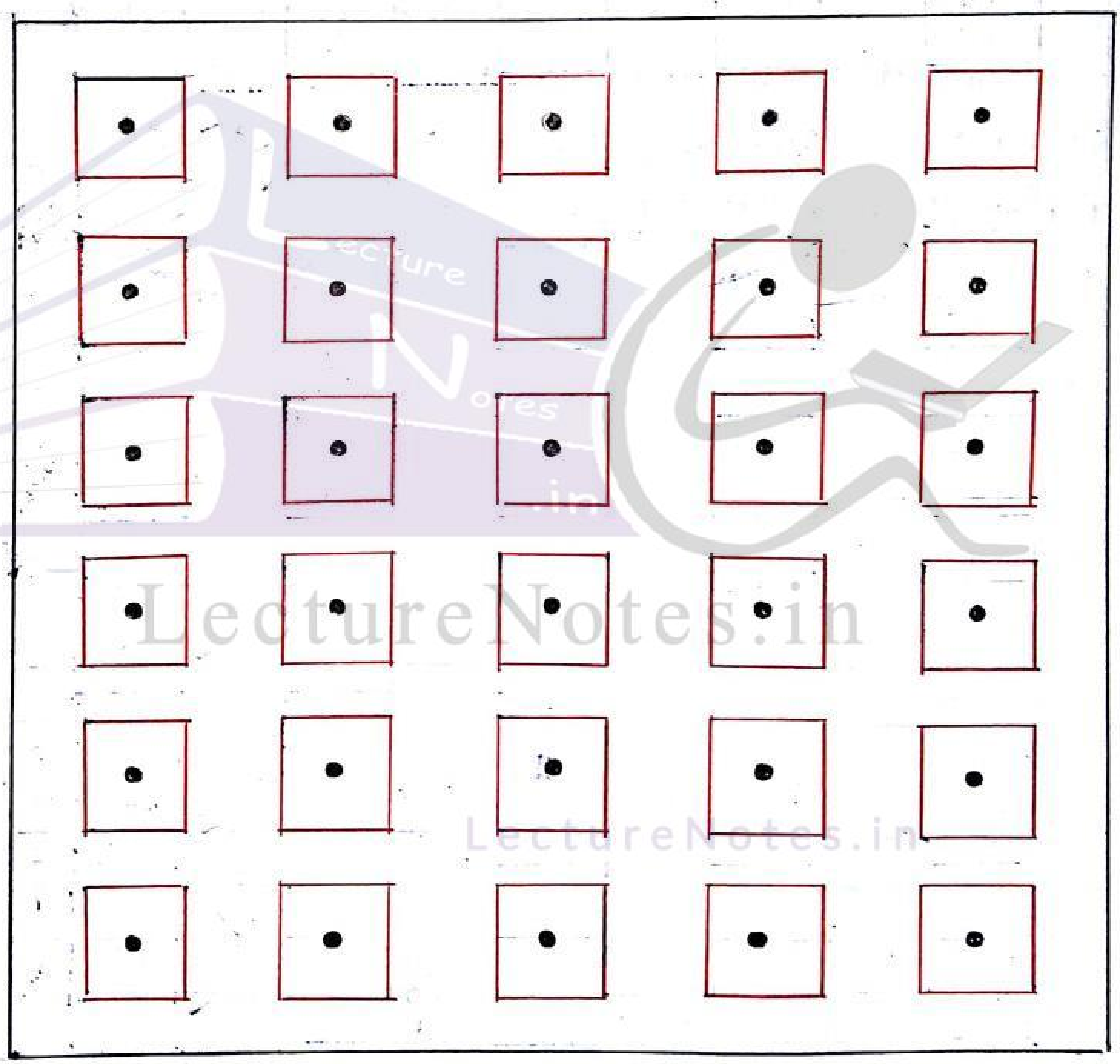
STRAP FOOTING

Raft Footing

-



SECTION



RAFT FOOTING

Difference between Foundation and Footing

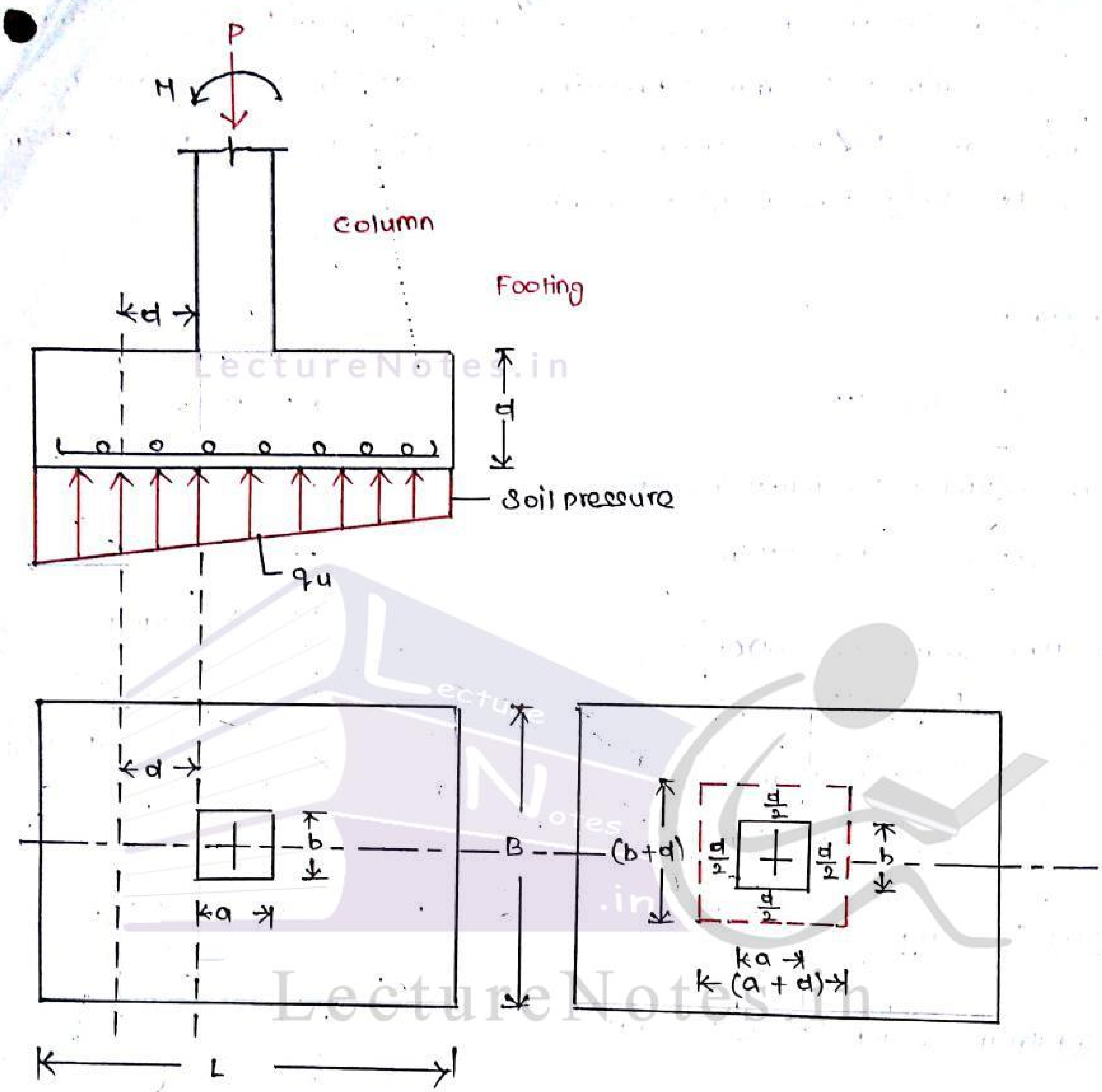
Foundation

* Foundation is a lower portion of building structure that transfer the building Gravity loads into to the earth
* Here we have two types of foundation
    * Shallow foundation
    * Deep foundation

Footing

* Footing is the lowest part of the foundation which rest on sub-soil and distributes load to the Ground
* Under the column and spread the load to a Large area which Increase the bearing capacity of soil.

Critical Sections for One way Shear and Two Way Shear



CRITICAL SECTION FOR ONE WAY SHEAR

CRITICAL SECTION ALL ROUND FOR TWO WAY SHER

One Way Shear

One Way shear is checked at a critical section distance 'd' from the column face

Two Way shear

Two Way shear is considered at a distance $(\frac{d}{2})$ from the periphery of the column.

Design of footings

1) A reinforced Concrete Column 400 mm × 400 mm Supports an axial Service load = 1000 KN. The Safe bearing Capacity of the Soil at site is 200 KN/m². Adopting M-20 grade and Fe415 HYSD bars. Design a Suitable footing for the Column and sketch the details of reinforcement ?

A) Given data,

Size of Column = 400 × 400 mm

Given axial service load = 1000 KN

Safe bearing Capacity = 200 KN/m²

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

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

① Determine the size of footing:-

As per IS code, the given load is increase by 10%.

$$\therefore \text{ Area of footing} = \frac{10\% \text{ of } P + P}{S.B.C}$$

$$\text{Area} = \frac{\frac{10}{100}(1000) + 1000}{200} = \frac{1100}{200} = 5.5 \, m^2$$

Here we design square footing then.

$$\text{Size of footing} = L = B = \sqrt{5.5} = 2.345 \, m \simeq 2.4 \, m$$

Adopt size of footing = 2.4 m × 2.4 m
(B × B)

(2) Calculation of soil reaction :-

According to IS Code, $(q_u)_{soil\ reaction} = \dfrac{P_u}{Area}$

where $P_u = 1.5 \times P = 1.5 \times 1000 = 1500\ kN$

$$q_u = \frac{1500}{2.4 \times 2.4} = 260\ kN/m^2$$

$$q_u = 260 \times \frac{10^3\ N}{(10^3)^2\ mm^2}$$

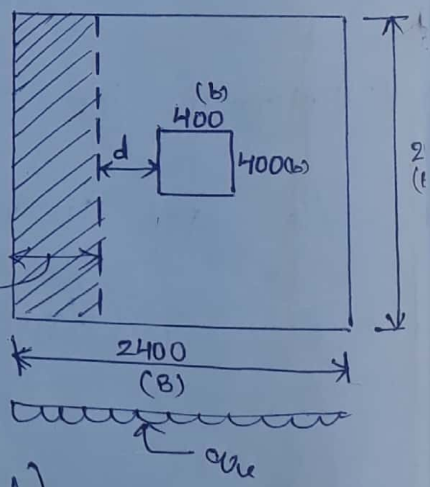
$$q_u = 0.260\ N/mm^2$$

(3) Calculation for depth of footing :-

a) check for one way shear :-

We will be found depth & check for one way shear.

The critical section is at the distance 'd' from the face of Column as per IS Code.



The design shear as per IS Code,

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

depth
$L = B = 2400\ mm$

$$= 0.26 \times 2400 \left(\left(\frac{2400 - 400}{2}\right) - d\right)$$

$$V_u = 624\ (1000 - d)$$

Assuming percentage of reinforcement in footing,

$P_t = 0.25\%$ for M20 grade.

Refer table 19 in IS 456:2000.

find $\tau_c = 0.36\ N/mm^2$

But we know, Shear stress $V_u = \tau_c B d$

$$V_u = 0.36 \times \cancel{1000}\ 2400 \times d$$

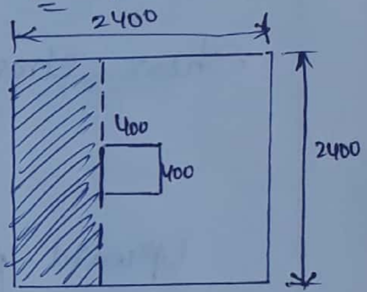
$$V_u = 864 \times d$$

Equating the both shear stress & find depth.

$$624(1000 - d) = 864\,d$$

$$624 \times 10^3 = 864\,d + 624\,d$$

$$d = 419.35\ mm \simeq 420\ mm$$



④ Check for Calculation of B.M:-

$$M_u = \frac{w_u \cdot l^2}{8}$$

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

$$= \cancel{0.168}\ 0.326 \times 2400 \times \frac{(2400-400)^2}{8}$$

$$M_u = 312\ KN.m$$

⑤ Check for depth:-

$$M_{u,lim} = 0.138\, f_{ck}\, B d^2$$

$$312 \times 10^6 = 0.138 \times 20 \times 2400 \times d^2$$

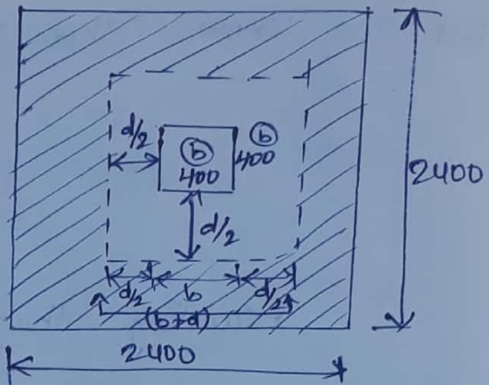
$$d = 217.02\ mm < 420\ mm$$

Hence depth is adequate.
(or)
Provided depth is sufficient.

⑥ Check for Two way Shear:-

As per IS 456:2000, critical section lies at $\frac{d}{2}$ distance from the face of column in two way shear.

Perimeter of Critical section



$= 4(b+d)$

$= 4(400+420)$

$= 3280$

Area of Critical section = Perimeter × Depth

$= 3280 \times 420$

$= 1377600 \text{ mm}^2$

Shear stress in Two way shear (τ) = $\dfrac{\text{Upward pressure in shaded area}}{\text{Area of critical sec}}$

Upward pressure in shaded area $= q_u \times \left(B^2 - (b+d)^2\right)$

$= 0.26 \times \left(2400^2 - (400+420)^2\right)$

$= 1322776.$

∴ Shear stress in two way shear (τ) $= \dfrac{1322776}{1377600} = 0.9$

But according to IS Code.

Max. permitted shear stress $= 0.25\sqrt{f_{ck}}$

$= 0.25\sqrt{20}$

$= 1.118 \text{ N/mm}^2$

Shear stress in two way shear < Max. permitted Shear stress.

Hence provided depth is sufficient.

Scanned with CamScanner

7) Calculation of Area of steel:-

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

$$312 \times 10^6 = 0.87 \times 415 \times A_{st} \times 420 \left(1 - \frac{415 \times A_{st}}{20 \times 2400 \times 420}\right)$$

$$A_{st} = 2152.9 \text{ mm}^2$$

Assume 16 mm ϕ of bar.

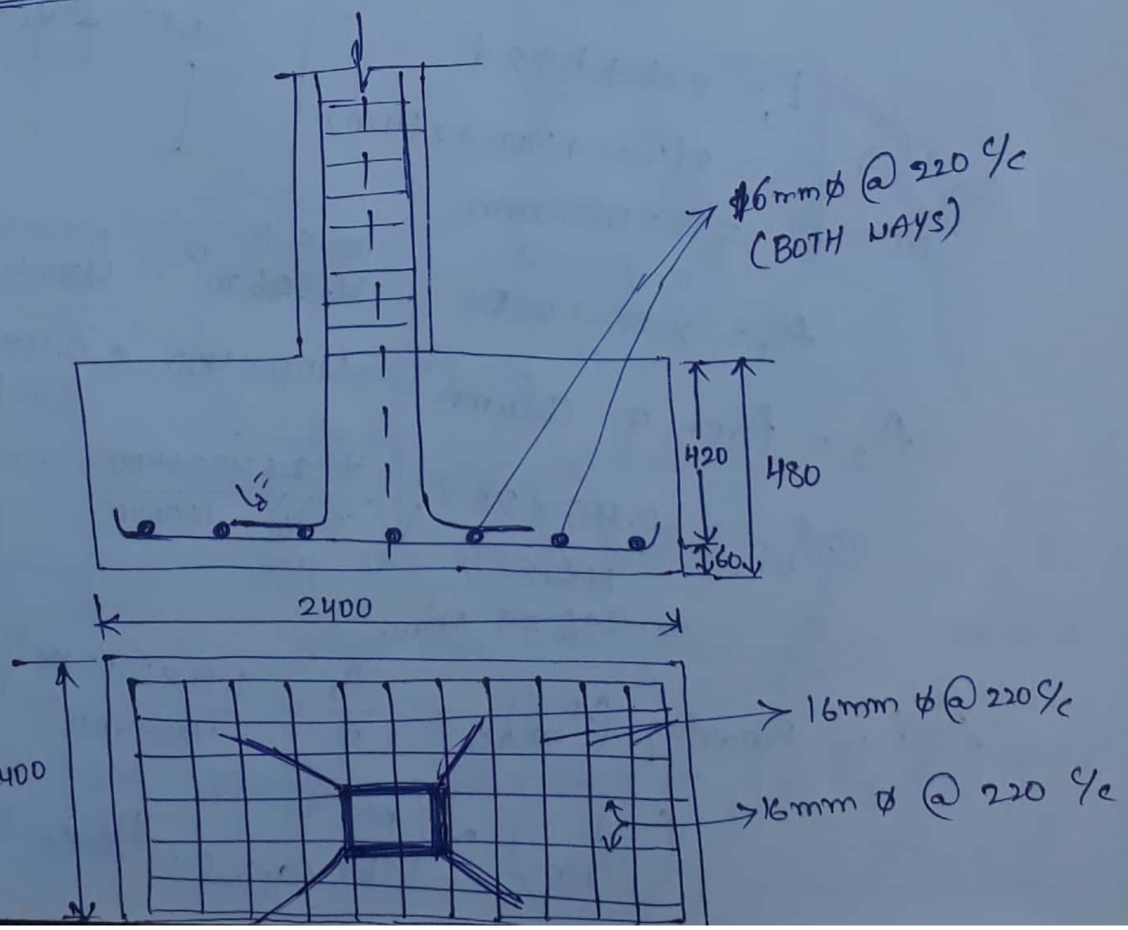
$$\text{Spacing} = \frac{a_{st}}{A_{st}} \times B = \frac{\frac{\pi}{4} \times 16^2}{2152.9} \times 2400$$

$$= 224.13 \cong 220 \text{ mm c/c}$$

Hence provide 16mm ϕ @ 220 mm c/c spacing.

Provided on both ways.

Details of Reinforcement :-



Check for development length :-

Clause - 26.2.1 (P.NO. 42)

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

~~Take $\tau_{bd} = 1.92$ for footing design~~ (M40 & above)

$\cancel{\tau_{bd}} =$ $\tau_{bd} =$ for deformed bars increase 60%..

$$L_d = \frac{0.87 \times 415 \times 16}{4 \times 1.2 \times 1.6} = 752.18 \text{ mm}$$

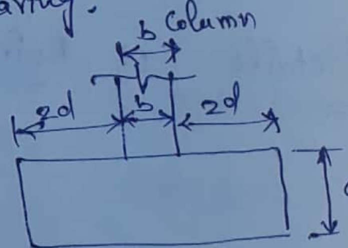
$$\text{Length available} = \frac{B-b}{2} = \frac{2400-400}{2} = \frac{2000}{2} = 1000 > L$$

Hence development length is safe.

Check for Bearing Pressure :- (uplift pressure due to on-soil)

$$f_{br} = 0.45 \times f_{ck} \times \sqrt{\frac{A_1}{A_2}}$$

$A_1 =$ Area of Load distributed under footing (or) Supporting area for bearing.



For isolated footing

$L_1 = 2d + b + 2d$

$= 2(420) + 400 + 2(420)$

$= 2080 \text{ mm}$

$A_1 = \cancel{2.08 \times 2.08}$ $2.4 \times 2.4 = \cancel{4.326}$ $5.76 \text{ m}^2 = \cancel{4326400}$ 5760000 mm^2

$A_2 =$ Area of Column $= 400 \times 400 = 0.4 \times 0.4 = 0.16 \text{ m}^2$
$= 160000 \text{ mm}^2$

$$f_{br} = 0.45 \times 20 \times \sqrt{\frac{\cancel{4.326}\ \cancel{4326400}\ 57600000}{\cancel{0.16}\ 160000}}$$

$\cancel{46.854}$

$= \cancel{46.79} \text{ N/mm}^2$

$$\text{Actual Bearing } (f_{br})_{act} = \frac{P_u}{A} = \frac{1.5 \times 1000 \times 10^3}{400 \times 400} = 9.375 \text{ N/mm}^2$$

$$(f_{br}) > (f_{br})_{actual}. \text{ Hence Safe.}$$

4) Design a rectangular footing for a rectangular Column 400 X 600 mm (4)
Carries a live load of 1700 kN. The bearing Capacity of Soil 170 kN/m².

A: Given data,

Size of Column = 400 × 600

live load $P = 1700$ KN

Safe bearing Capacity = 170 KN/m²

Assume M20 & Fe415 Steel.

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

$f_y = 415\ N/mm^2$

(1) Determine the Size of footing :-

∴ As per IS Code, the given load is increase by 10%.

$$\text{Area of footing} = \frac{10\% \text{ of } P + P}{S.B.C}$$

$$= \frac{\frac{10}{100}(1700) + 1700}{170}$$

$$= \frac{1870 \cancel{KN}}{170} = 11\ \cancel{mm^2}$$

Area of footing $= 11\ m^2$
$(l \times b)$

assume any one value.

say one side of footing = 4 m.

$$\text{other side} = \frac{\text{Area}}{4} = \frac{11}{4} = 2.75\ m \approx 3\ m$$

So, we provide ~~4m×~~ 3 m × 4 m footing size.
L × B

② Calculation of soil reaction

According to IS Code

$$q_u = \frac{P_u}{Area}$$

$$P_u = 1.5 \times P = 1.5 \times 1700 = 2550$$

$$q_u = \frac{2550}{3 \times 4} = 212.5 \ KN/m^2$$

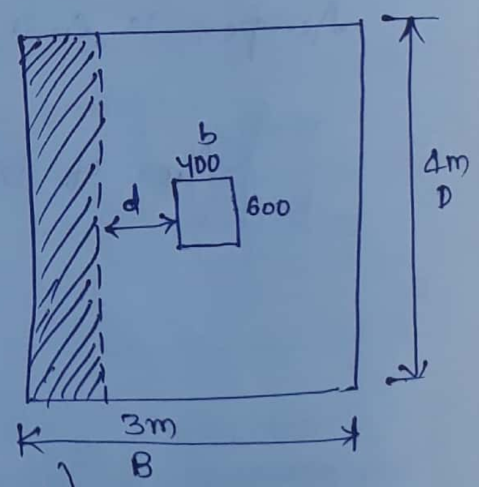
$$q_u = 212.5 \times \frac{10^3}{(10^3)^2} \ N/mm^2$$

$$q_u = 0.2125 \ N/mm^2$$

③ Calculation of depth of footing:-

we will be found depth & check for one way shear.

The critical section is at a distance 'd' from the face of Column as per IS Code



The design soil pressure as per Code

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

$$= 0.212 \times 4000 \left(\frac{3000 - 400}{2} - d \right)$$

$$V_u = 848 \times (1300 - d)$$

Assuming percentage of reinforcement in footing

$P_t = 0.25\%$ for M_{20} grade

Refer table 19 in IS 456:2000.

find $\tau_c = 0.36 \ N/mm^2$

But, we know, shear stress $V_u = \tau_c B d$

$$= 0.36 \times 3000 \times d$$

$$V_u = 1080 \times d$$

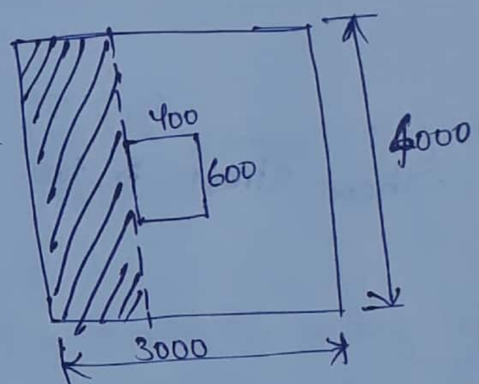
equating the both shear stress & find depth

$$848\,(1300 - d) = 1080\, d$$

$$d = 571.78 \text{ mm} \simeq 580 \text{ mm}$$

Hence we provide 580 mm depth of footing.

4) ~~Ca~~ Calculation of B.M :-



$$M_u = q_u \times B \times \frac{(B-b)^2}{8}$$

$$= 0.212 \times 4000 \times \frac{(3000-400)^2}{8}$$

$$= 537420000$$

$$M_u = 537.42 \text{ KN.m}$$

5) Check for depth :-

$$M_{u,lim} = 0.138 f_{ck} B d^2$$

$$537.42 \times 10^6 = 0.138 \times 20 \times 3000 \times d^2$$

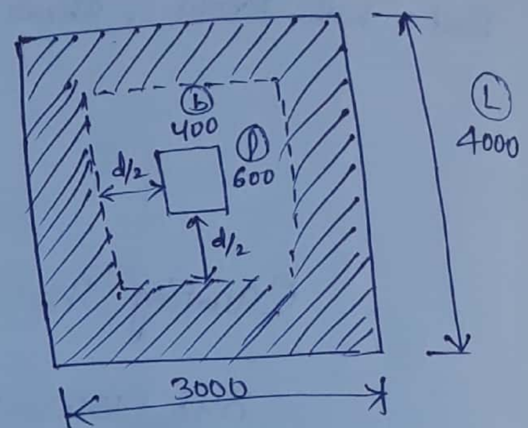
$$d = 254.76 \text{ mm} < 580 \text{ mm}$$

Hence provided depth is sufficient.

6) Check for Two way shear :-

As per IS 456:2000, Critical section lies at $\frac{d}{2}$ distance from the face of column in two way shear.

Perimeter Of Critical Section



$= 2(l+b)$

$P = 2\left((b+d)+(D+d)\right)$

$= 2\left((400+580)+(600+580)\right)$

$= 4320$

Area of Critical Section = Perimeter × Depth

$= 4320 \times 580$

$= 2505600\ mm^2$

Shear stress in two way shear $(V_{u_2}) = \dfrac{\text{Upward pressure in shaded}}{\text{Area of Critical section}}$

$(400+580)\quad(600+580)$

$= \dfrac{q_u \times \left((B \times L) - (980 \times 1180)\right)}{2505600}$

$= \dfrac{0.212 \times (3000 \times 4000 - (980 \times 1180))}{2505600}$

$V_{u_2} = 0.917\ N/mm^2$

But according to IS Code,

Max. Permitted Shear stress $= 0.25\sqrt{f_{ck}}$

$= 0.25\sqrt{20}$

$= 1.118\ N/mm^2$

Shear stress in two way shear < Max. Permitted shear stress

Hence provided depth is adequate.

⑦ Calculation of Area of steel:-

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

$$537.42 \times 10^6 = 0.87 \times 415 \times A_{st} \times 580 \left(1 - \frac{415 \times A_{st}}{20 \times 3000 \times 580}\right)$$

$$A_{st} = 2650.11 \ mm^2$$

Assume 16 mm ϕ of bar

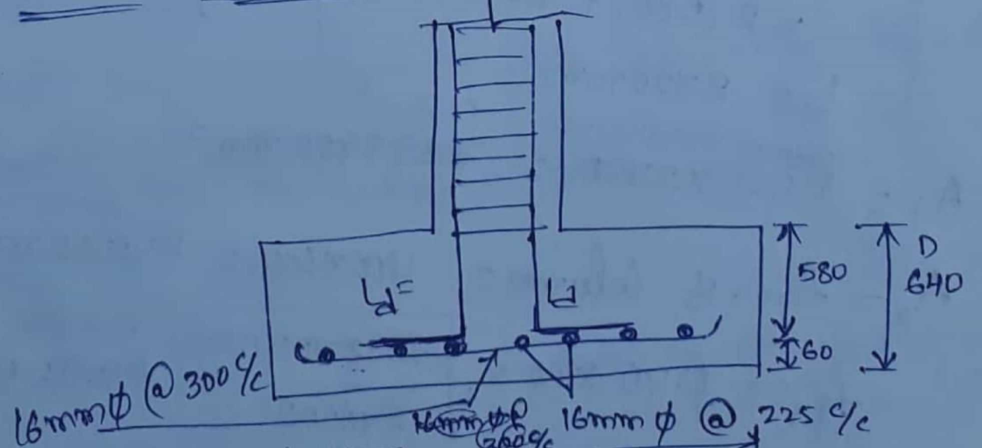
Spacing along width direction $= \dfrac{a_{st}}{A_{st}} \times B = \dfrac{\frac{\pi}{4} \times 16^2}{\cancel{2152.9}\ 2650.11} \times 3000 = 227.6 \simeq 225$ c/c

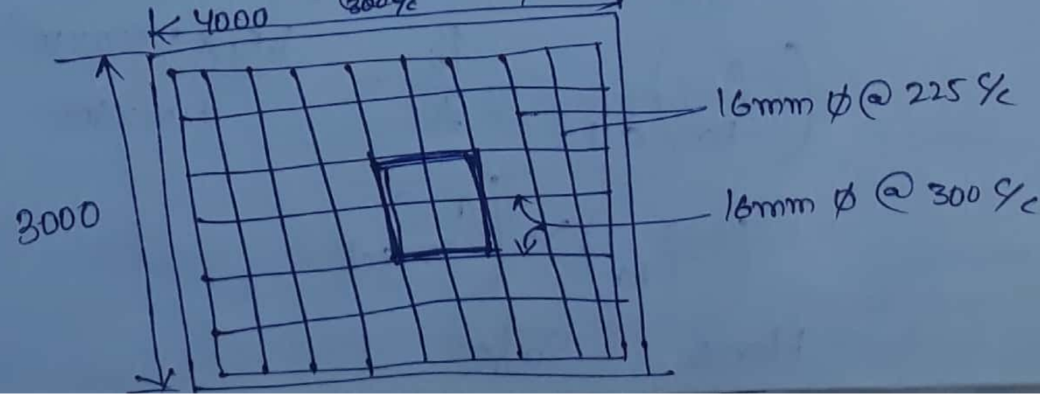
Spacing along ~~length~~ depth direction $= \dfrac{a_{st}}{A_{st}} \times \cancel{L} = \dfrac{\frac{\pi}{4} \times 16^2}{2650.11} \times 4000 = 303.47 \simeq 300$ c/c

Hence provide 16mm ϕ @ 225mm c/c along width direction

& 16mm ϕ @ 300mm c/c along Depth direction.

⑧ Details of reinforcement :-





Check for development length:-

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

for M_{20} $\tau_{bd} = 1.2$ &, for deformed bars τ_{bd} increased by 60%.

$$L_d = \frac{0.87 \times 415 \times 16}{4 \times 1.2 \times 1.6}$$

135
137
139

$$= 752.18 \text{ mm}$$

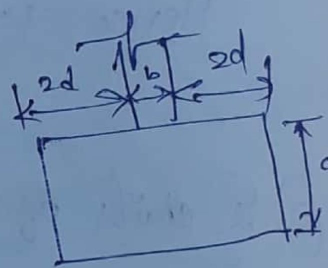
Length available $= \frac{B - b}{2} = \frac{3000 - 400}{2} = 1300 > L_d$

∴ Hence Safe.

Check for Bearing pressure:-

$$f_{br} = 0.45 \times f_{ck} \times \sqrt{\frac{A_1}{A_2}}$$

A_1 = Supporting area for bearing



for isolated footing ×

$L_1 = 2d + b + 2d$

$= 2(580) + 400 + 2(580)$

$L_1 = 2720$ mm

$A_1 = \cancel{2720 \times 2720}\ 3000 \times 4000 = \cancel{7398400}\ 12 \times 10^6 \text{ mm}^2$

A_2 = Area of Column = $400 \times 600 = 240000 \text{ mm}^2$

$$f_{br} = 0.45 \times 20 \times \sqrt{\frac{\cancel{7398400}\ 12\times10^6}{240000}} = \cancel{49.96}\ 63.63 \text{ N/mm}^2$$

$$(f_{br})_{act} = \frac{P_u}{A} = \frac{1.5 \times 1700 \times 10^3}{400 \times 600} = 10.625 \text{ N/}$$

$$(f_{br}) > (f_{br})_{act}$$

Hence ∴ Safe.

Design of Combined footing

Design a Combined Column footing with a Strap beam for two reinforced Concrete Columns of size 300mm by 300mm spaced 4m c/c and each Supporting a Service axial load of 500kN. The Safe bearing Capacity of Soil at site is 150 KN/m². Adopt M20 grade & Fe415 HYSD bars?

Given data, Size of Columns = 300x300 mm

Spacing between columns = 4 m

Working Load on each Column = 500 kN

Safe bearing Capacity of soil = 150 kN/m²

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

$f_y = 415 N/mm^2$

① Design loads & stresses:-

Ultimate load on each column = 1.5 x 500 = 750 KN.

Load Bearing Capacity of soil = 1.5 x 150
= 225 KN/m².

Total ultimate load on two Columns = 750 + 750 = 1500 kN

> Design of footing:-

~~Self wt. of footing~~ ≈ 10%

② Size of footing :-

$$\text{Area of footing} = \frac{\text{10\% of ultimate load} + \text{ultimate load}}{S.B.C}$$

$$= \frac{\frac{10}{100}(1500) + (1500)}{225} = 7.33 m^2$$

Adopt footing size = 6m x 1.5m

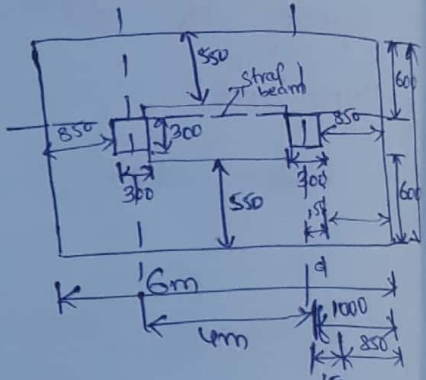
Adopt a strap beam breadth = 400mm (Here we take more than Column dimension.)

③ Design of footing :-

$$\text{Soil pressure } q_u = \frac{\text{load}}{\text{Area of footing}} = \frac{P_u}{A} = \frac{1500}{6 \times 1.5} = 166.67 \text{ kN}$$

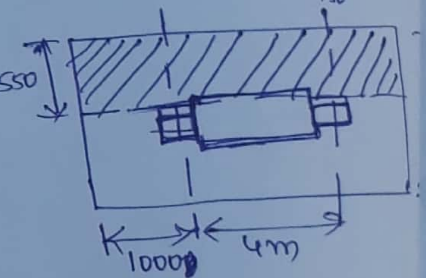
$$q_u = 0.166 \text{ N/mm}^2$$

④ Calculation of Design Moment :-



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

$$= \frac{(0.166 \times 1000) \times (1500 - 400)^2}{8}$$



$$= 25107.5 \times 10^3$$

$$M_u = 25.107 \text{ kN.m}$$

150.64

⑤ Eff. depth of footing :-

$$M_u = 0.138 f_{ck} b d^2$$

$$25.107 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$$

$$d = 95 \text{ mm}$$

Actual Take $d_{eff} = 2d = 2 \times 95 = 190 \text{ mm} \simeq 250 \text{ mm}$

Hence Adopt $d = 250$ mm (for Combined footing Take depth of footing upto 300 mm)

Total overall depth of footing $D = 250 + 50 = 300$ mm

⑥ Reinforcement :-

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

$$25.1 \times 10^6 = 0.87 \times 415 \times A_{st} \times 250 \left(1 - \frac{415 \times A_{st}}{20 \times 1000 \times 250}\right)$$

$$A_{st} = 286 \text{ mm}^2$$

Min. Area of steel $(A_{st})_{min} = 0.12\% \text{ (Gross Area)}$

$$= \frac{0.12}{100}(1000 \times 300)$$

(2)

$(A_{st})_{min} = 360\ mm^2$

$(A_{st})_{min} > (A_{st})_{req}$ so, take $(A_{st})_{min}$ as main reinforcement

Assume 10 mm ϕ of bar

Spacing $(s) = 1000 \times \frac{a_{st}}{A_{st}}$

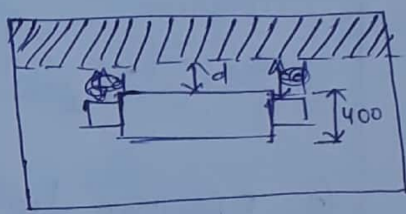
$= 1000 \times \frac{\frac{\pi}{4}(10)^2}{360}$

$= 218.16 \approx 210$ mm c/c

Provide 10 mm ϕ @ 210 mm c/c spacing.

(5) Check for one-way shear :-

The critical section will act as at a distance 'd' from the column face.



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

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

$= 0.166 \times 1000 \times \left(\left(\frac{1500-400}{2}\right) - 250\right)$

$V_u = 50$ KN

$\tau_v = \frac{V_u}{b \times d} = \frac{50 \times 10^3}{1000 \times 250} = 0.2\ N/mm^2$

$P = \frac{A_{st}}{bd} \times 100 = \frac{360}{1000 \times 250} \times 100 = 0.14\%$

Refer table 19. in IS456:2000

$\tau_c = 0.28$

$\tau_c > \tau_v$

Hence It is safe in one way shear.

Two way shear is neglected because shear reinforcement will resist the two way shear.

Design of Strap ~~Footing~~ Beam :-

Load acting on beam = Soil pressure

= 166.6 KN/m^2

Ultimate load acting on ~~soi~~beam = 1.5 × 166.6

(W_u) = 250 KN/m^2

① Max. B.M & S.F :-

$$M_u = \frac{W_u l^2}{8} = \frac{250 \times (4)^2}{8} = 500 \text{ KN.m}$$

$$V_u = \frac{W_u l}{2} = \frac{250 \times 4}{2} = 500 \text{ KN}$$

② Effective depth of beam :-

Note :- The eff. depth of strap beam can be calculated by two way

i) Eff. depth w.r. to B.M.

ii) Eff. depth w.r. to Shear stress in Concrete τ_c.

where τ_c = shear stress in concrete = 1.2 N/mm^2 (T

Here for strap beam the eff. depth is to be calculated w.r.t shear because the depth obtained with respected to shear will be more to depth w.r. to BM.

So ~~Consid~~ find the E.ff. depth of beam based the shear stress.

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

Adopt $\tau_c = 1.2$ for M20. where b = width of bean

$$d = \frac{V_u}{\tau_c b}$$

$$d = \frac{500 \times 10^3}{1.2 \times 400} = 1041 \approx ~~1050~~ \approx 1100$$

Assume d' = 1050 mm

$M_{u,lim} = 0.138 f_{ck} b d^2$

$$d = \sqrt{\frac{M_{u,lim}}{0.138 f_{ck} b}}$$

$$= \sqrt{\frac{500 \times 10^6}{0.138 \times 20 \times 400}}$$

d = 672.9 mm

Overall depth of beam = 1100 + 50 = 1150 mm

(3) Calculation of ~~As~~ Main reinforcement :-

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

$$500 \times 10^6 = 0.87 \times 415 \times A_{st} \times 1100 \left(1 - \frac{415 \times A_{st}}{20 \times 400 \times 1100}\right)$$

$$A_{st} = 1344.15 \text{ mm}^2$$

Assume 22 mm ϕ of bars.

$$\text{no. of bars} = \cancel{\frac{a_{st}}{A_{st}}} = \cancel{\frac{\frac{\pi}{4} \times 22^2}{1344.15}} \quad \frac{A_{st}}{a_{st}} = \frac{1344.15}{\frac{\pi}{4} \times 22^2} = 3.53 \simeq 4$$

Hence provide 4 No's of 22 mm ϕ of bars.

(4) Check for shear

$$\tau_v = \frac{V_u}{bd} = \frac{500 \times 10^3}{400 \times 1100} = 1.136 \text{ N/mm}^2$$

$$\%\text{ of steel} = \frac{A_{st}}{bd} \times 100$$

$$= \frac{4 \times \frac{\pi}{4} \times 22^2}{400 \times 1100} \times 100$$

$$p = 0.354\%$$

Refer Table 19 in IS 456:2000.

p	τ_c
0.25 →	0.36
0.50 →	0.48
0.354 →	?

$$\tau_c = 0.36 + \frac{0.48 - 0.36}{0.50 - 0.25}(0.354 - 0.25)$$

$$\tau_c = 0.42 \text{ N/mm}^2$$

$\tau_v > \tau_c$. So. shear reinforcement is provided

~~Hence shear stress is safe~~

Assume 2 Legged vertical stirrup of 8mm ϕ

Spacing (S_v) = ?

$$\frac{A_{sv}}{b.S_v} = \frac{0.4}{0.87 f_y}$$

$$S_v = \frac{A_{sv} \times 0.87 f_y}{0.4 b}$$

$$= \frac{2 \times \frac{\pi}{4} \times 8^2 \times 0.87 \times 415}{0.4 \times 400}$$

$$S_v = 226.85 \simeq 220 \text{ c/c}$$

(5) Side face reinforcement :-

(A_{sd}) = 0.1% (Gross c/s Area of beam)

$$= \frac{0.1}{100}(400 \times 1150)$$

$$= 460 \text{ mm}^2$$

Assume 10mm ϕ of bars

$$\text{No. of bars} = \frac{A_{st}}{a_{st}} = \frac{460}{\frac{\pi}{4} \times 10^2} = 5.85 \simeq 6$$

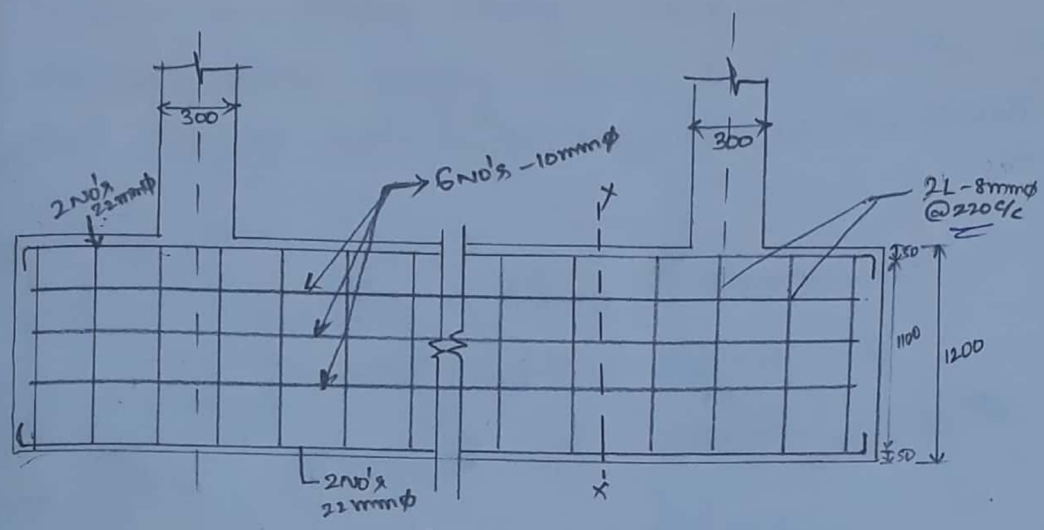
Hence provide 6 no's 10mm ϕ of bars. in equally provided as side face reinforcement

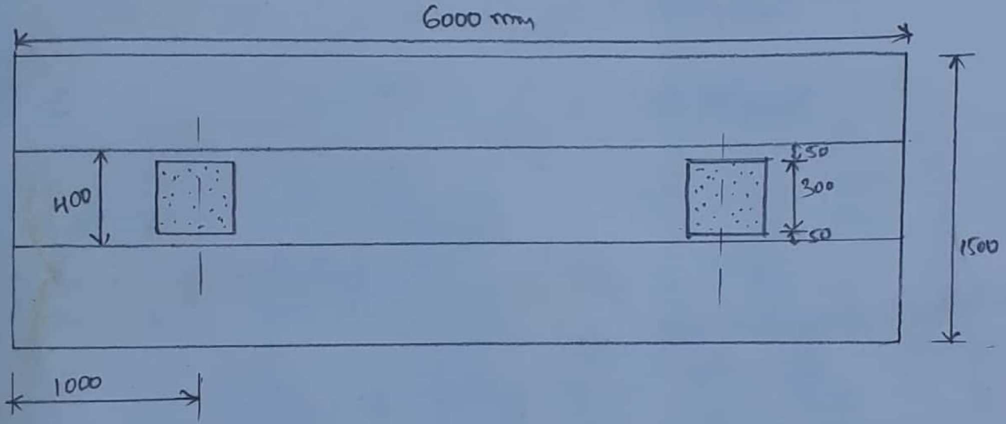
(6) Check for Development length :-

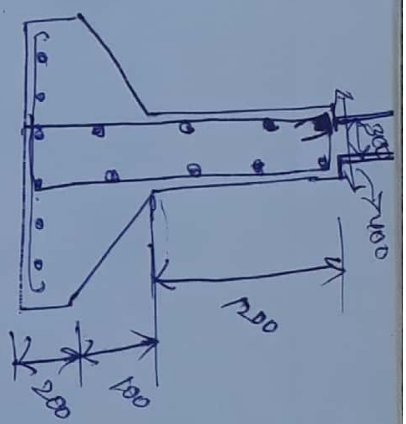
$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}}$$

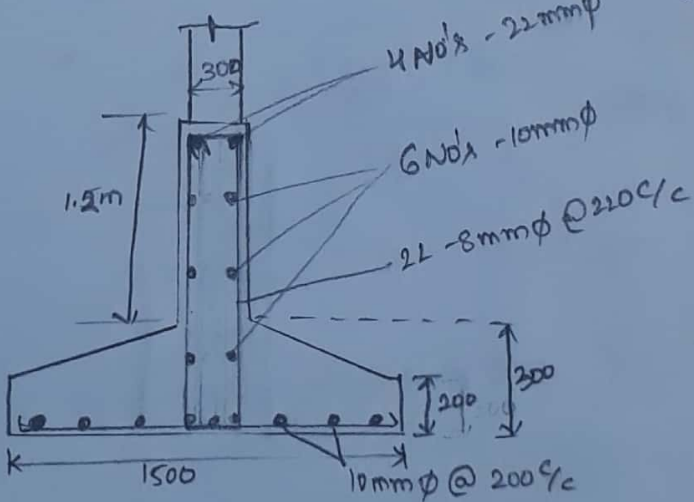
$$L_d = \frac{0.87 \times 415 \times 10}{4 \times 1.6 \times 1.2} = 470.11 \text{ mm} \simeq 470$$

Details of Reinforcement :-









Section at XX.

Design of Sloped footing :-

① Design a sloping footing for a short axial column of size 300mm x 300mm, carrying 600kN load. Use M_{20} Concrete and Fe415 steel. SBC of soil is 180 kN/m². Sketch the details of reinforcement?

Sol: Given data,

Size of Column = 300 x 300mm

axial Load (P) = 600 kN

SBC of soil = 180 kN/m²

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

$f_y = 415\ N/mm^2$

1) Size of footing :-

$$\text{Area of footing} = \frac{10\%\ \text{increase of load}(P) + P}{S.B.C}$$

$$a^2 = \frac{\frac{10}{100}(600) + 600}{180} = 3.67\ m^2$$

$$a = \sqrt{3.67} = 1.91 \simeq 2m$$

$\therefore$ Size of footing = 2m x 2m.

2) Calculation of Soil reaction :-

$$q_u = \frac{P_u}{\text{Area of footing}} = \frac{1.5(600)}{2\times 2} = 225\ kN/m^2$$

$$q_u = \frac{225 \times 10^3}{(10^3)^2}\ N/mm^2$$

$$q_u = 0.225\ N/mm^2$$

③ Calculation of Depth of footing :-

Equivalent width $b_e = b + \frac{1}{8}(B-b)$

$= 300 + \frac{1}{8}(2000-300)$

$b_e = 512.5$ mm.

Bending Moment $(M_u) = \frac{W_u \cdot l^2}{2}$

$= \frac{(q_u \times B)\,(B-b)^2}{8}$

$= 0.225 \times (2000) \times \frac{(2000-300)^2}{8}$

$M_u = 162.56$ KN.m.

But for Fe415, we know

$M_{u,lim} = 0.138\, f_{ck}\, b_e\, d^2$

$162.56 \times 10^6 = 0.138 \times 20 \times 512.5 \times d^2$

$d = 339$ mm.

Note: But for sloped footing $d_{eff} = \frac{4}{3} \times d$

$d_{eff} = \frac{4}{3} \times 339 = 452$ mm $\simeq 500$

∴ Depth of footing $D = d_{eff} + d' = 500$

$= 550$

④ Calculation of Main reinforcement :-

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

$162.56 \times 10^6 = 0.87 \times 415 \times A_{st} \times 500 \left(1 - \frac{415 \times A_{st}}{20 \times 512.5 \times}\right)$

$A_{st} = 977.92$ mm^2

Assume 12 mm ø of bars

Spacing $= 2000 \times \frac{a_{st}}{A_{st}} = 2000 \times \frac{\frac{\pi}{4} \times 12^2}{977.92} = 231.3 \simeq$

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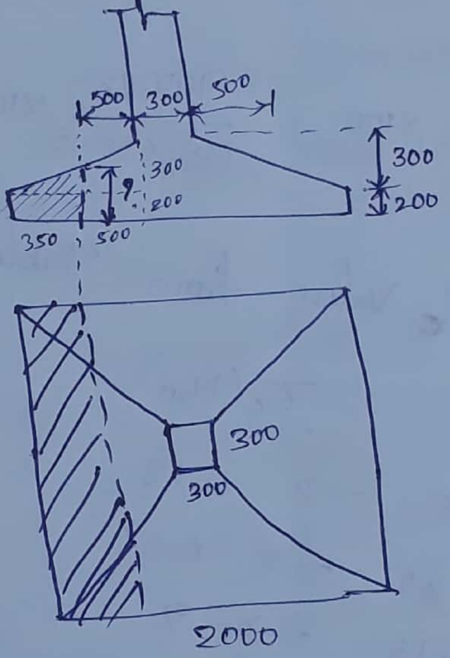
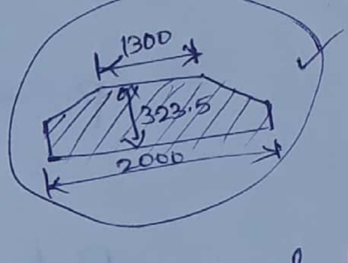
Hence Provide 12 mm ϕ of bar @ 230 mm c/c Provided in Both directions.

⑤ check for one way Shear:-

For one way sheer the Critical section lies at a distance 'd' (d = 500)

i,e



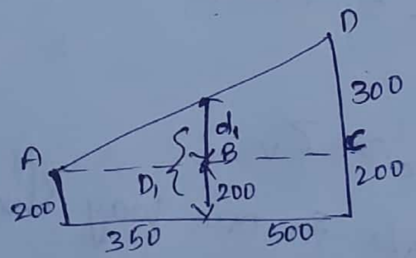


B_1 = Avg width of footing

$= \dfrac{\text{Top width} + \text{Bottom width}}{2}$

$B_1 = \dfrac{(500+300+500) + 2000}{2}$

From figure,



Depth of footing at Critical section = D_1. = $200 + d_1$

Due to similar Δ^{les} rule.

$\dfrac{d_1}{350} = \dfrac{300}{\underset{(350+500)}{850}}$ (or) $d_1 = \underset{\text{base depth}}{200} + (\underset{\text{depth}}{500} - 200) \times \dfrac{850-500}{850}$ depth.

$d_1 = 123.5$

$\therefore$ Depth of footing @ critical section $= 200 + 123.5$

$= 323.5$ mm

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

$$= (0.225 \times 2000) \times \left(\left(\frac{2000-300}{2}\right)-500\right)$$

$$V_u = 157.5 \text{ KN}$$

$$\tau_v = \frac{V_u}{B_1 \cdot D_1} = \frac{157.5 \times 10^3}{1650 \times 323.5} = 0.29 \text{ N/mm}^2$$

$$P = \frac{A_{st}}{B_1 D_1} \times 100 = \frac{977.92}{1650 \times 323.5} \times 100 = 0.18\ \%$$

Refer τ_c Value from Table-19.

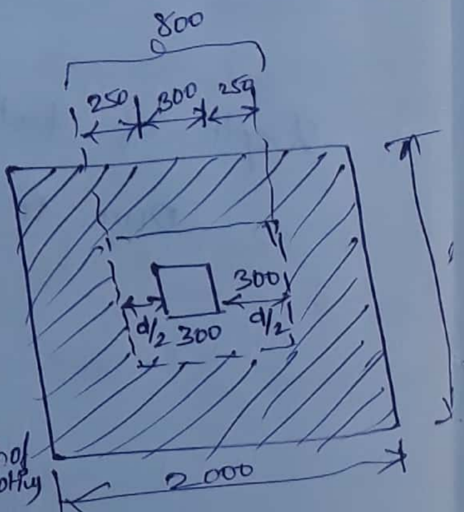
P	$\tau_c (M_{20})$
0.15 →	0.28
0.25 →	0.36
0.18 →	?

$$\tau_c = 0.304 \text{ N/mm}^2$$

$$\tau_c > \tau_v$$

Hence safe in one way shear.

(6) Check for Two way shear:-



Perimeter of critical section $= 4(b+d)$

$d_2 = 200 + (500-200) \times \left(\frac{850-210}{810}\right)$. $= 4(300+500)$

$= 3200$ mm

Perimeter x depth of footing

Area of Critical section $= 3200 \times 500$ 411.76

$= \quad = 13.17 \times 10^5 \text{ mm}^2$

Uplift pressure $= q_u (B^2 - (b+d)^2)$

$= 0.225(2000^2 - (800)^2) = 756000$

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V_u Shear stress $= \dfrac{\text{Uplift pressure}}{\text{Area of critical section}} = \dfrac{756000}{16 \times 10^5} = 0.47\ N/mm^2$

~~Critical~~
Max. Permit for Shear stress $= 0.25\sqrt{f_{ck}}$
$= 0.25\sqrt{20}$
$= 1.118\ N/mm^2$

$1.118 > 0.47$

Hence Two way Shear is Safe.

(7) Development length:-

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

$$= \frac{0.87 \times 415 \times 12}{4 \times 1.2 \times 1.6}$$

$$L_d = 564.14\ mm < \left(\frac{B-b}{2}\right) \simeq \left(\frac{2000-300}{2}\right)$$

850 mm

Hence development length is Safe.

(8) Check for bearing pressure:-

$$f_{br} = 0.45 \times f_{ck} \times \sqrt{\frac{A_1}{A_2}}$$

A_1 = Area of Load distributed under footing:

$L = 2d + b$ ~~+2d~~

$L = 2(500) + 300 = 2300$ mm

$A_1 = 2300 \times 2300 = 5290000\ mm^2$

A_2 = Area of column = $300 \times 300 = 90000\ mm^2$

$$f_{br} = 0.45 \times 20 \times \sqrt{\frac{5290000}{90000}}$$

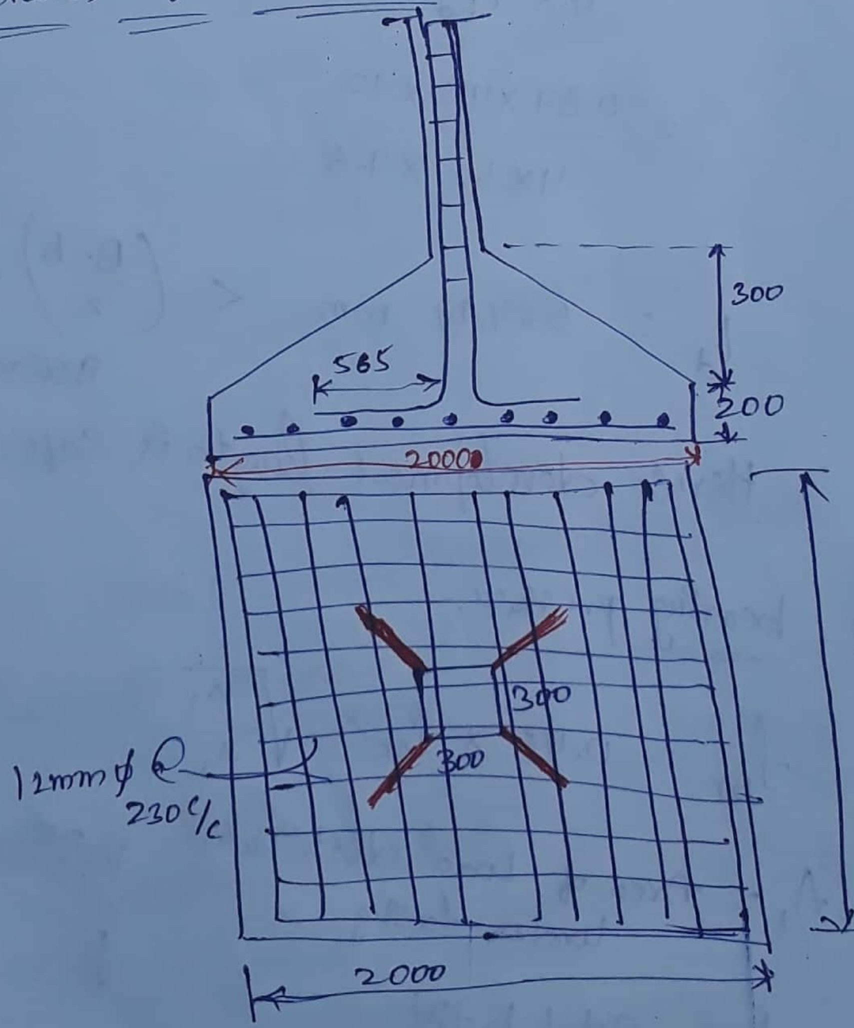
$$= 69\ N/mm^2$$

Actual bearing $(f_{br})_{act} = \frac{P_u}{A} = \frac{1.5 \times \cancel{1000}^{600} \times 10^3}{300 \times 300} = 10 <$

$$f_{br} > (f_{br})_{act}$$

Hence Bearing pressure is safe.

(9) Details of reinforcement :-



DESIGN OF STAIR CASES

Stair Case: Stair Case provide access for the Various floors of the building. The Stair Consist of series of steps with landings at appropriate intervals. The stretch between the two landings is called "Flight". The room (or) space bet where stairs are provided is called "stair case".

* Min width of stair case for residential building is = 1m.
  Min width of staircase for public building = 2 m.
* For free flow of users, The width of landing should be atleast equal to the width of stairs.
* The range of the Rise may vary from 150 to 200 mm
* The thread range may vary from 250 to 300 mm.

As per IS456, the slope (or) Pitch of the stair should be in between 25^0 to 40^0.

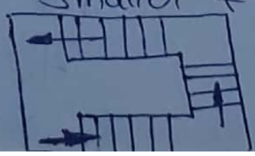
Types of Stair Cases:-

1. Single flight stair case
2. Quarter turn staircase
3. Doglegged staircase
4. Open well (or) Open Newel Stair Case
5. Spiral stair case. etc.

Dog legged staircase: The most Common type of stairs arranged with two adjacent flights running parallel with a mid landing. where less space is available at that place we can use this type of staircase.

Open well (or) Open Newel stair Case:-

In public buildings where large spaces are available, open well stair case is generally prefered due to its better assessibility & Comfort and Ventilation due to its Smaller flights with an Open well at the Centre.



Assumption Values

* Take live load as 5 KN/m^2 for public building
3 KN/m^2 for residential building

* Assume width of slab as 1m (or) 1000mm.

* Floor finishing load = 0.5 to 1 KN/m^2.

Formulas: ~~for~~ ~~to~~

Weight of waist slab $(w_1) = D\sqrt{1+\left(\frac{R}{T}\right)^2} \times 25$

Weight of steps per unit horizontal area $(w_2) = \frac{1}{2} \times R \times 25$

where R = Rise in "m."
T = thread in "m".
D = Thickness of waist slab (or) overall depth of waist slab

(1) Design a dog legged staircase for an office building in a room measuring 2.8m×5.8m clear vertical distance b/w floor is 3.6 Take live load of 3 KN/m^2. Sketch the width of flight 1.25m. Use M20 & Fe415. Assume stair are details of reinforcement. supported on edges of landing slabs.

(A) Given data,

Dimension of Stair = 2.8m × 5.8m

Height of floor = 3.6 m Assume Riser(R) = 1
Thread(T) =

width of flight = 1.25 m

live load = 3 KN/m^2

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

$f_y = 415\ N/mm^2$

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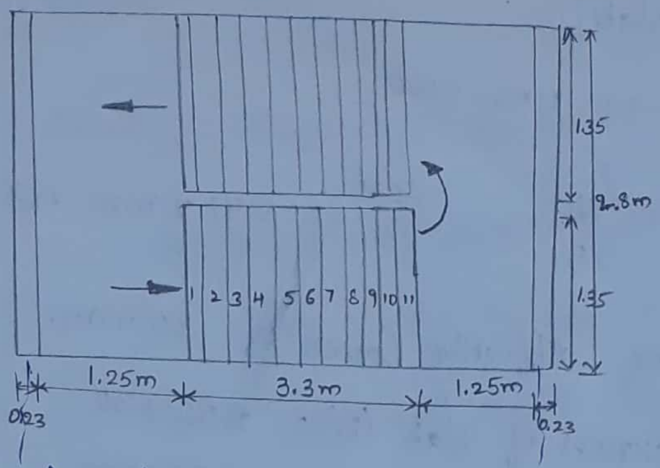

Height of floor level = 3.6 m

height of one flight = $\frac{3.6}{2}$ = 1800 mm = 1.8 m

No of risers = $\frac{\text{flight height}}{\text{length of riser}} = \frac{1800}{150}$ = 12 Nos

No of threads (T) = $\left(\text{No of risers}\right) - 1$

= 12 − 1 = 11

Total length of going = No of threads × thread width

= 11 × 300

= 3300

= 3.3 m

Width of landing (or) flight = 1.25 m (given)

If it is not given, width of landing = $\frac{(5.8 - 3.3)}{2}$ = 1.25 m

① Effective span:-

Eff. span = c/c distance of supports

$l_{eff} = \frac{0.23}{2} + 5.8 + \frac{0.23}{2} = 6.03$ m

$l_{eff} = 6.03$ m

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② Thickness of slab:-

As per IS 456:2000,

$$\text{eff. depth } (d) = \frac{l_{eff}}{25} = \frac{6030}{25} = 241.2 \text{ mm} \simeq 250 \text{ mm}$$

Assume effective cover of 30 mm.

$$\text{Over thickness of slab } (D) = 250 + 30$$
$$= 280 \text{ mm.}$$

③ Loads acting on the staircase:-

i) weight of waist slab $= D\sqrt{1+\left(\frac{R}{T}\right)^2} \times 25$

$$W_1 = 0.28 \times \sqrt{1+\left(\frac{0.15}{0.3}\right)^2} \times 25$$

$$= 7.82 \text{ kN/m}$$

ii) weight of steps $W_2 = \frac{1}{2} \times R \times 25$

$$= \frac{1}{2} \times 0.15 \times 25$$
$$= 1.875 \text{ kN/m}^2$$

iii) Live load $= 3 \text{ kN/m}^2$

iv) Floor finish $= 0.8 \text{ kN/m}^2$

Total load $= 7.82 + 1.875 + 3 + 0.8 = 13.495 \text{ kN/m}^2$

Factored load $(W_u) = 1.5 \times 13.495 = 20.24 \text{ kN/m}^2$

④ Calculation of Design Moment :-

$$M_u = \frac{W_u l_{eff}^2}{8} = \frac{20.24(6.03)^2}{8} = 91.99 \text{ kN.m}$$

(5) Check for depth:-

For Fe415, $M_u = 0.138 f_{ck} bd^2$

$$91.99 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$$

$$d = 182.56 \text{ mm} < 250 \text{ mm}$$

Hence provided depth is adequate.

(6) Calculation of Main reinforcement:-

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

$$91.99 \times 10^6 = 0.87 \times 415 \times A_{st} \times 250 \left(1 - \frac{415 \times A_{st}}{20 \times 1000 \times 250}\right)$$

$$A_{st} = 1123.99 \text{ mm}^2$$

Assume 16 mm ϕ of bars.

$$\text{Spacing of bars (S)} = \frac{a_{st}}{A_{st}} \times 1000$$

$$= \frac{\frac{\pi}{4} \times 16^2}{1123.99} \times 1000$$

$$S = 178.88 \text{ mm} \simeq 175 \text{ mm}$$

Hence provide 16mm ϕ of bar of 175 mm c/c spacing provided as main reinforcement.

(7) Distribution Reinforcement:-

For Fe415, $(A_{st})_{min} = 0.12\%$ of gross area

$$= \frac{0.12}{100} \times (1000 \times 280)$$

$$= 336 \text{ mm}^2$$

Assume 10 mm ϕ of bars.

Spacing of bars:

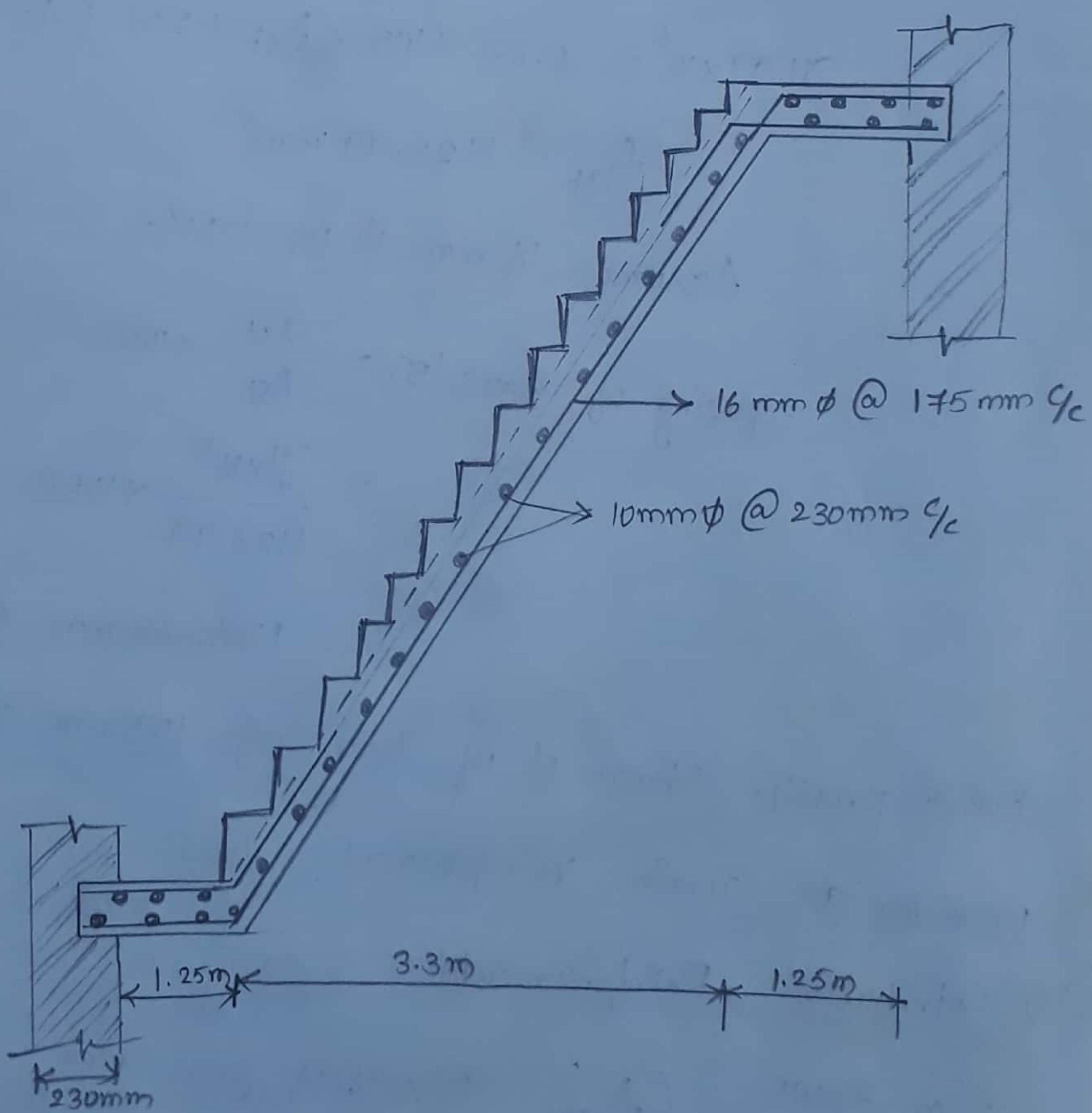
$$S = \frac{a_{st}}{A_{st}} \times 1000$$

$$= \frac{\frac{\pi}{4} \times 10^2}{336} \times 1000$$

$$S = 233.74 \text{ mm} \simeq 230 \text{ mm } c/c$$

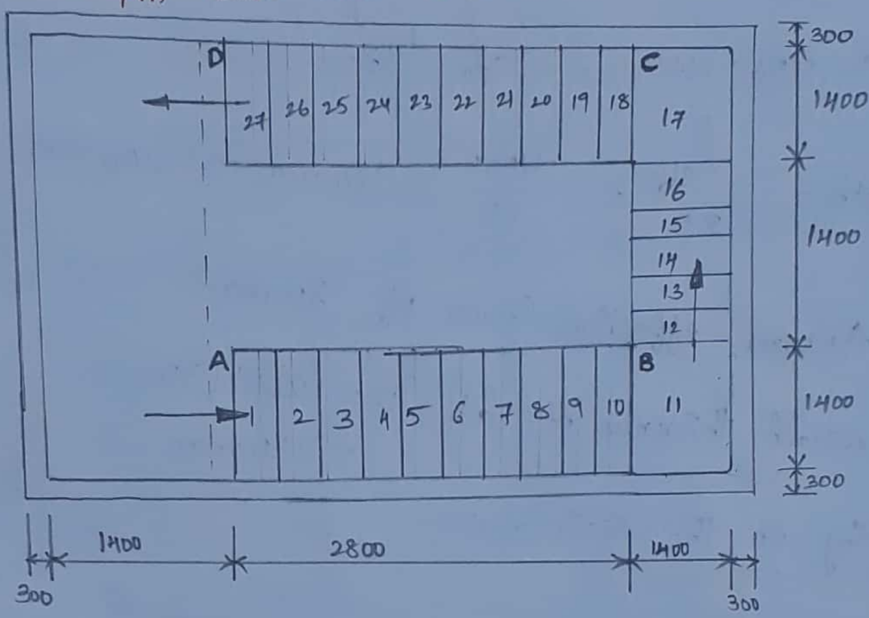
Hence provide 10 mm ϕ of bar provided at 230 mm c/c Spacing as distribution reinforcement.

Details of Reinforcement :-



Design of Open Well Stair Case :- (or) Open Newel Stair Case :-

A stairs with open well is having steps of size 150 x 280 mm. The arrangement of stairs as shown in fig. Design the stair for a live load of $3\,KN/m^2$ Use M20 & Fe415. Sketch the details of Reinforcement.



Given data,

$R = 150\,mm$

$T = 280\,mm$

Live load $= 3\,KN/m^2$

Assume floor finish $= 0.8\,KN/m^2$

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

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

① Effective Span :-

i) Along flight AB & CD :-

$$(l_{eff})_{AB} = \frac{300}{2} + 2800 + 1400 + \frac{300}{2} = \cancel{45}\,4500 = 4.5\,m$$

ii) Along flight 'BC':-

$$\left(l_{eff}\right)_{BC} = \frac{300}{2} + 1400 + 1400 + 1400 + \frac{300}{2} = 4500 = 4.5m$$

(2) Thickness of slab:-

As per IS 456 : 2000 , range $\left(\frac{l}{20} \text{ to } \frac{l}{25}\right)$

$$\text{eff. depth} = \frac{l_{eff}}{25} = \frac{4500}{25} = 180\,mm \cong 200\,mm$$

Assume effective cover of 30 mm

overall thickness of slab $D = 200 + 30$
$= 230\,mm$

(3) Loads acting on the staircases:-

i) On flight AB & CD:- a) ~~Going total~~ Load on Going :-

~~Load~~ Weight of waist slab $(W_1) = D \times \sqrt{1 + \left(\frac{R}{T}\right)^2} \times 25$

$$= 0.23 \times \sqrt{1 + \left(\frac{0.15}{0.28}\right)^2} \times 25$$

$$W_1 = 6.52\,kN/m$$

~~Load~~ Weight of steps $(W_2) = \frac{1}{2} \times R \times 25$

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

$$= 1.875\,kN/m$$

Floor finish $= 0.8\,kN/m^2$

live load $= 3\,kN/m^2$

$\therefore$ Total load acting on going $= 6.52 + 1.875 + 0.8 + 3$
$= 12.195\,kN/m$

Factored load acting on going $(W_{uG}) = 1.5 \times 12.195 = 18.29$

b) Load on Landing :-

S/w of landing = (C/s area of landing) x Unit wt of RCC

$= (0.23 \times 1) \times 25$

$= 5.75$ KN/m

Floor finish = 0.8 KN/m

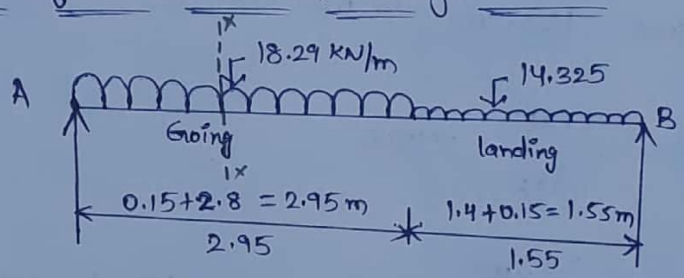
live load = 3 KN/m

∴ Total load on Landing = $5.75 + 0.8 + 3 = 9.55$ KN/m

Total factored load on landing (W_{uL}) = 1.5×9.55

$= 14.325$ KN/m.

④ Calculation of Moment in Flight AB & CD :



In this unsymmetrical loading the max. B.M Can be known at the point where shear force is '0'.

Calculation of Reactions :-

$(\Sigma V = 0)$ $R_A + R_B = 18.29(2.95) + 14.325(1.55)$

$R_A + R_B = 76.15$ KN

$(\Sigma M_B = 0)$ $R_A(1.55 + 2.95) - (18.29 \times 2.95)(1.55 + \frac{2.95}{2}) - (14.325 \times 1.55)(\frac{1.55}{2}) = 0$

$R_A = \frac{180.42}{4.5} = 40.09$ KN

$R_B = 76.15 - 40.09 = 36.06$ KN

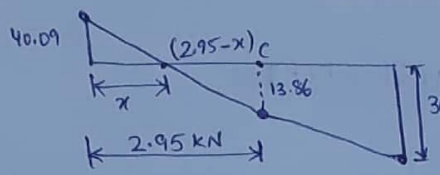
Shear forces :-

S.F @ A $\rightarrow$ (R_A) 40.09 KN

S.F @ C $\rightarrow$ $40.09 - 18.29(2.95) = -13.86$ KN.

S.F @ B → $-13.86 - 14.325(1.55) = -36.06\ kN$

→ $-36.06 + 36.06 = 0$



We know for Max B.M (where S.F = 0)
From figure at 'x' distance the S.F becomes zero

→ Due to symetry of triangles.

$$\frac{40.09}{x} = \frac{13.86}{2.95-x}$$

$$x(13.86) = 40.09(2.95-x)$$

$$x(13.86+40.09) = 40.09(2.95)$$

$$\boxed{x = 2.19\,m}$$

$$\text{Max B.M}(M_u) = R_A(x) - 18.29(x)\left(\frac{x}{2}\right)$$

$$= 40.09(2.19) - 18.29(2.19)\left(\frac{2.19}{2}\right)$$

$$\boxed{M_u = 43.936\ kN.m}$$

Calculation of Main reinforcement on flight 'AB' & 'CD' :-

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

$$43.93 \times 10^6 = 0.87 \times 415 \times A_{st} \times 200\left(1 - \frac{415 \times A_{st}}{20 \times 1000 \times 200}\right)$$

$$A_{st} = 649.3\,mm^2$$

Assume 16 mm ϕ of bars

Spacing :

$$S = \frac{a_{st}}{A_{st}} \times 1000$$

$$S = \frac{\frac{\pi}{4} \times 16^2}{649.3} \times 1000 = 309.65 \cong 300\,mm\ c/c$$

Hence provide 16mm ϕ of bars at 300mm c/c spacing provid as main reinforcement in flight 'AB' & 'CD'.

Calculation of Distribution reinforcement on flight 'AB' & 'CD' :-

$$(A_{sc})_{min} = 0.12\%\ (\text{Gross C/s area})$$

(D) → Depth (or) thickness of slab

$$= \frac{0.12}{100}(1000 \times 230)$$

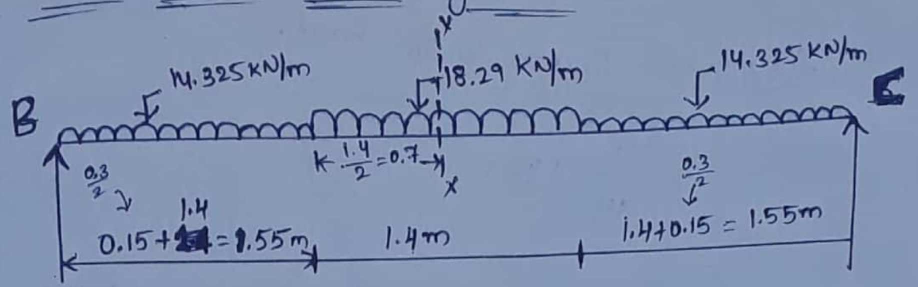
$$= 276\,mm^2$$

Assume 8 mm ϕ of bars

$$\text{Spacing }(S) = \frac{a_{st}}{A_{st}} \times 1000 = \frac{\frac{\pi}{4} \times 8^2}{276} \times 1000 = 182.12 \cong 180\,mm$$

Hence provide 8mm ϕ of bars at 180 mm c/c spacing provided as distribution rein in flight 'AB' & 'CD'.

(5) Calculation of Moment in flight 'BC' :-



Due to symetric Condition the B.M Can Calculate at Centre.

Calculation of Reaction :-

$(\Sigma V = 0)$ $V_B + V_C = 14.325(1.55) + 18.29(1.4) + 14.325(1.55)$

$V_B + V_C = 70 \text{ KN}$

$(\Sigma M_C = 0)$,

$V_B(1.55 + 1.4 + 1.55) - (14.325 \times 1.55)\left(1.55 + 1.4 + \frac{1.55}{2}\right) - (18.29 \times 1.4)\left(1.55 + \frac{1.4}{2}\right)$

$-(14.325 \times 1.55)\left(\frac{1.55}{2}\right) = 0$

$V_B = \frac{157.53}{4.5} = 35 \text{ KN}$

$V_C = 70 - 35 = 35 \text{ KN}$

(or)

For Symmetric Condition $V_B = V_C = \frac{\text{Total load}}{2} = \frac{70 \text{KN}}{2} = 35 \text{KN}.$

Bending Moment at Centre $\Rightarrow$

$M_u = V_B(0.7 + 1.55) - (14.325 \times 1.55)\left(0.7 + \frac{1.55}{2}\right) - (18.29 \times 0.7)\left(\frac{0.7}{2}\right)$

$= 35(2.25) - 32.75 - 4.48$

$M_u = 41.52 \text{ KN.m}$

Calculation of Main reinforcement on flight 'BC' :-

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

$41.52 \times 10^6 = 0.87 \times 415 \times A_{st} \times 200 \left(1 - \frac{415 \times A_{st}}{20 \times 1000 \times 200}\right)$

$A_{st} = 614.11 \text{ mm}^2$

Assume 16mm ϕ of bars.

Spacing: $S = \dfrac{a_{st}}{A_{st}} \times 1000$

$= \dfrac{\frac{\pi}{4} \times 16^2}{614.11} \times 1000 = 327.40 \simeq 320 \text{ mm}$

Hence provide 16mm ϕ of bars at 320mm c/c spacing provided as main reinforcement in flight 'BC'.

Calculation of Distribution reinforcement on flight 'BC':-

$(A_{sc})_{min} = 0.12\% \text{ (Gross c/s area)}$

$= \dfrac{0.12}{100} \times (1000 \times 230)$

$= 276 \text{ mm}^2$

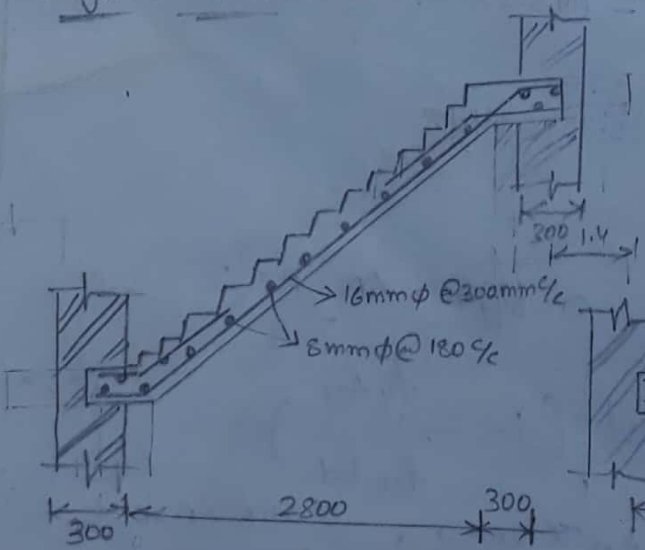
Assume 8mm ϕ of bars

Spacing: $S = \dfrac{a_{st}}{A_{st}} \times 1000 = \dfrac{\frac{\pi}{4} \times 8^2}{276} \times 1000 = 182.12 \text{ mm} \simeq 180 \text{ mm}$

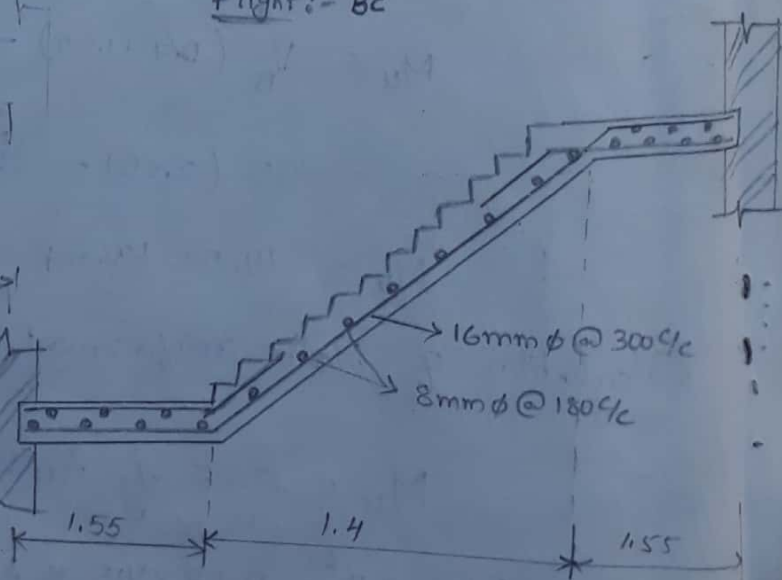
Hence provide 8mm ϕ bars at 180mm c/c spacing provided as distribution reinforcement in flight 'BC'.

(6) Details of reinforcement:-

Flight : AB & CD



Flight :- BC



UNIT-5

DESIGN OF SLAB BY LSM

Introduction

* Slab is Two dimensional or planar elements
* Slab is used in all types of structure such as Floors and Roof covering
* The Thickness of slab is very Small as Compared to its Length and Width
* Slab are classified on the basic of $\frac{l_y}{l_x}$

  l_y – Length of Longer Span

  l_x – Length of Shorter Span
* Slab are constructed to provide Flat Surfaces, usually horizontal in building.
  * Floor
  * Roof
  * Bridges

  And other types of Structures
* The slab may be supported by Walls by Reinforced concrete beam usually cast Monolithically.
* So slab is Large, Thick, Flat piece of Stone or Concrete typically Square or Rectangular in shape structure which transfer Live Load (Varying load or Movable load) and Dead Load (Structure member load like Wall, Beam, column and many other forces like Wind Load, Snow Load (At Terrace)

Slab

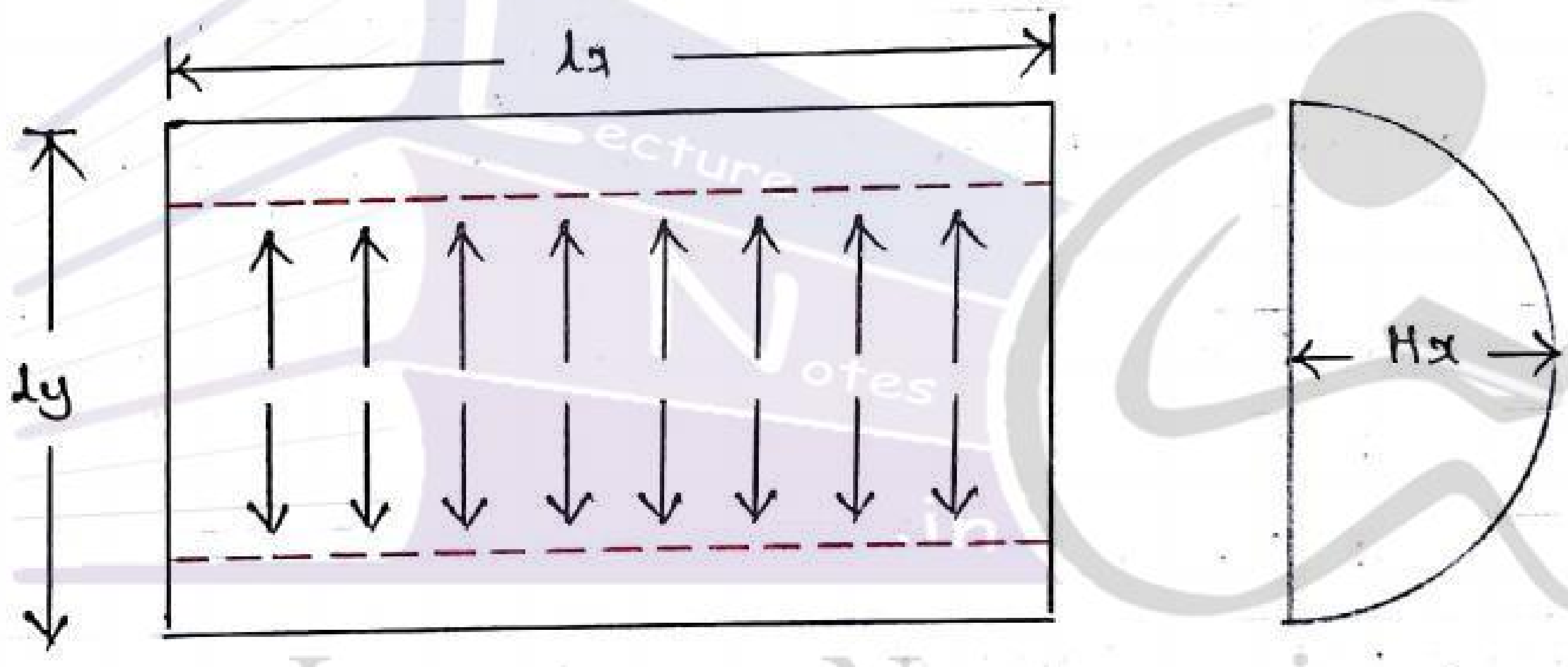
* It is a Flat horizonatal Surface.
* It is Supported by Beam and Column
* It Transfer Load to the beam

Types of Slab

Based on length and breadth of conventional slab is classified into Two Types

* One Way Slab
* Two Way Slab

One Way Slab



Bending in Shorter direction

* In One Way Slab the Slabs are Supported by the beams on the Two opposite Sides
* A Rectangular Slab is Supported on all the Four Edges
* The Load on the Slab is transferred to the Two Supporting Walls
* So this type of Slab bend only in One direction (i.e in Shorter Span) and is called One Way Slab
* Main reinforcement is provided in only One direction for One Way Slab

* The Ratio of Longer Span (L) to Shorter Span (B) is equal or greater than 2

$$\frac{\text{Longer Span}}{\text{Shorter Span}} > 2$$

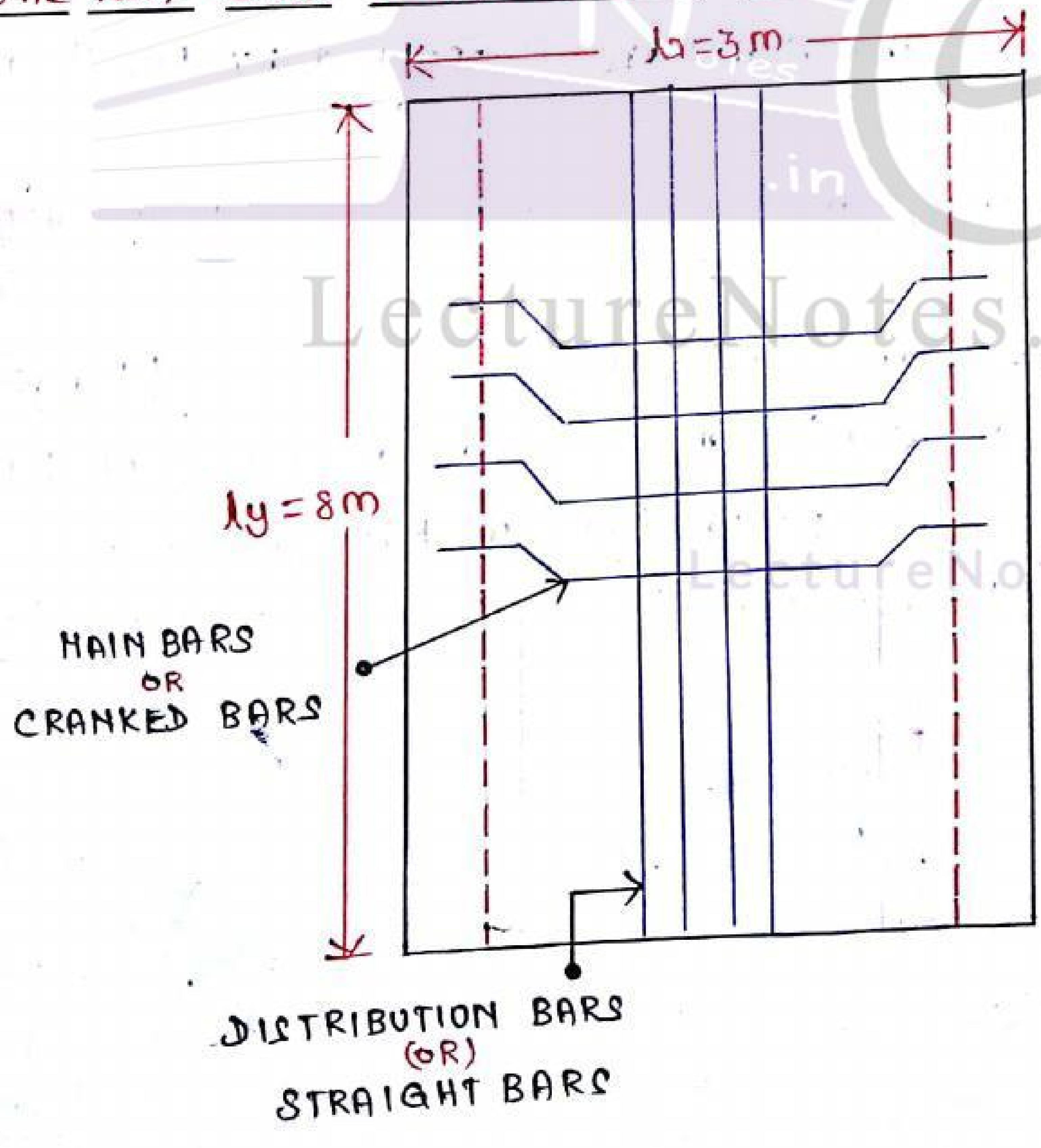
$$\boxed{\frac{l_y}{l_x} > 2}$$

* Main bars (cranked bars) are provided on Shorter side
* Distribution bars (Straight bars) are provided on Longer side
* 50% of bars of Shorter side are cranked or bent up
* Less economical (costly).
* Thickness of slab is More

Example :

Cantilever slab , Chajjas slab ; Verandahs

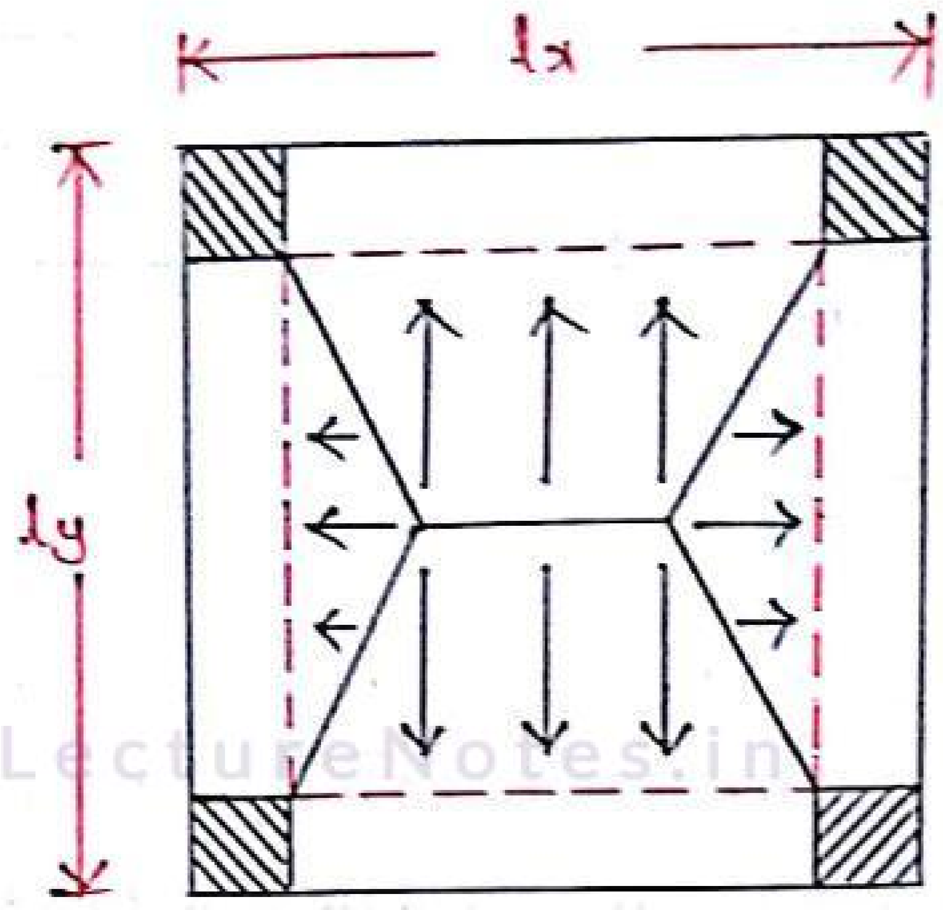
One Way Slab Reinforcement details

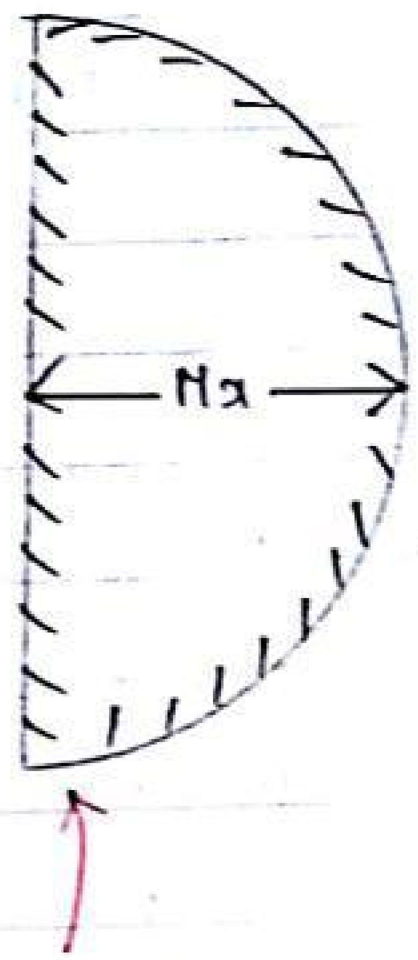


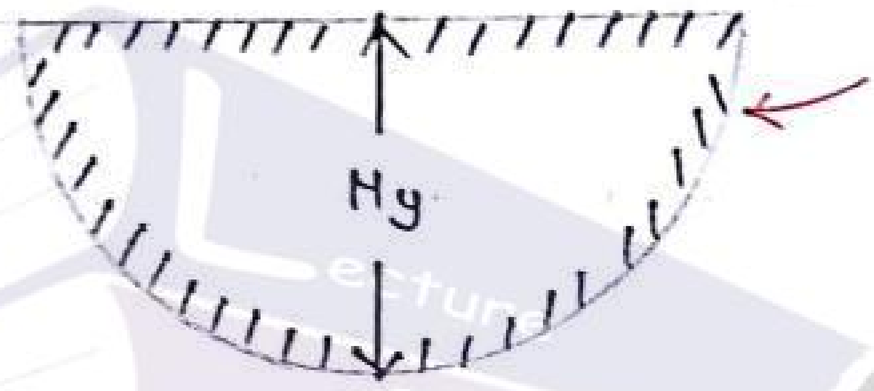
$$\frac{l_y}{l_x} = \frac{8}{3} = 2.66 > 2$$

Hence One Way Slab is adopted

Two Way Slab







Bending Take place along both direction

* In Two way Slab the Slabs are Supported by the beam on all the Four edges of both Span
* The Load on the Slab is transferred to the Four Supporting Walls
* So this type of Slab bends in Two direction (Longer Span & Shorter Span) and is Called a Two-Way Slab
* Main Reinforcement is provided in both direction
* The Ratio of Longer Span (L) to Shorter Span (B) is less than 2

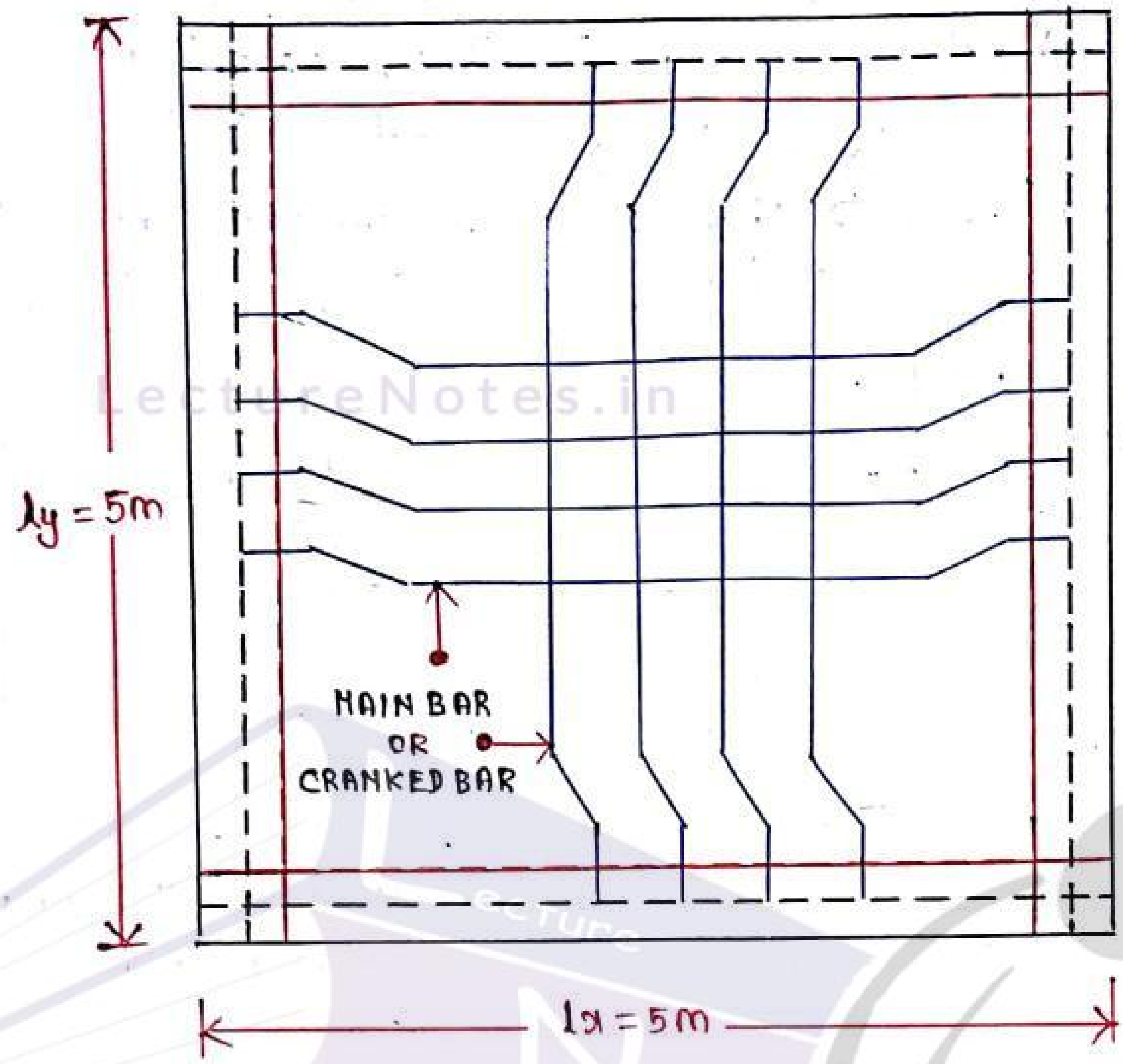
$$\boxed{\frac{l_y}{l_x} < 2}$$

* Thickness of Slab is Less
* More Economical

<u>Example</u>:

Construction Floor, Multi Storey building

Two Way Slab Reinforcement details

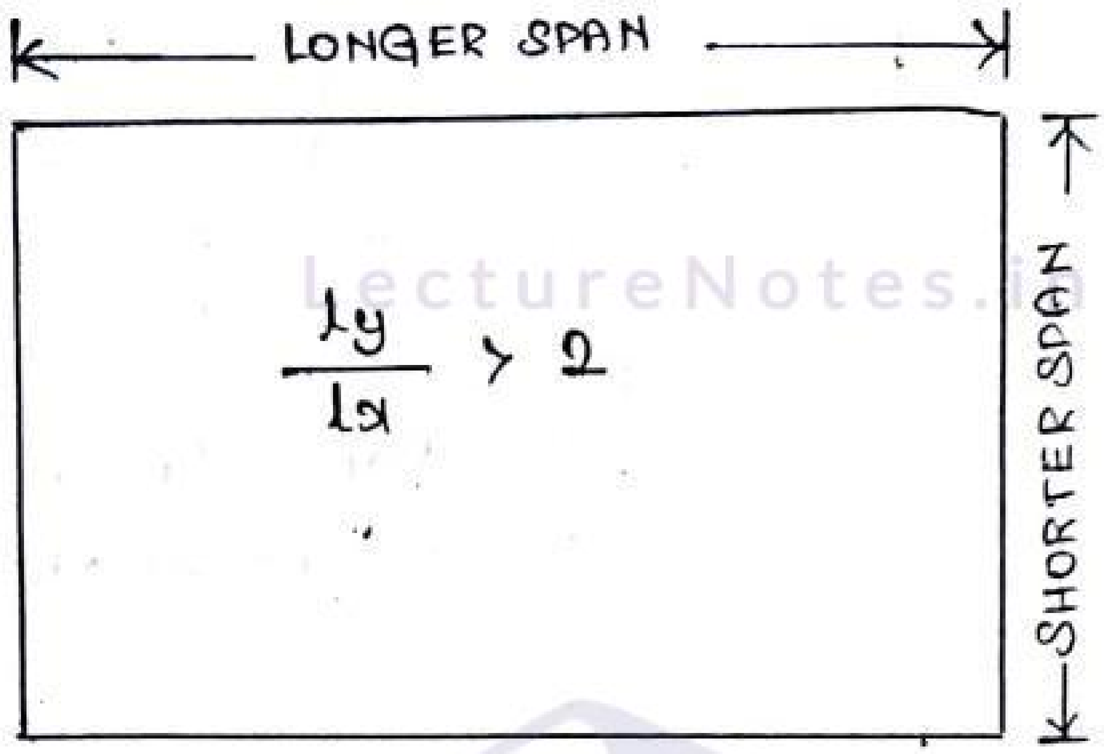


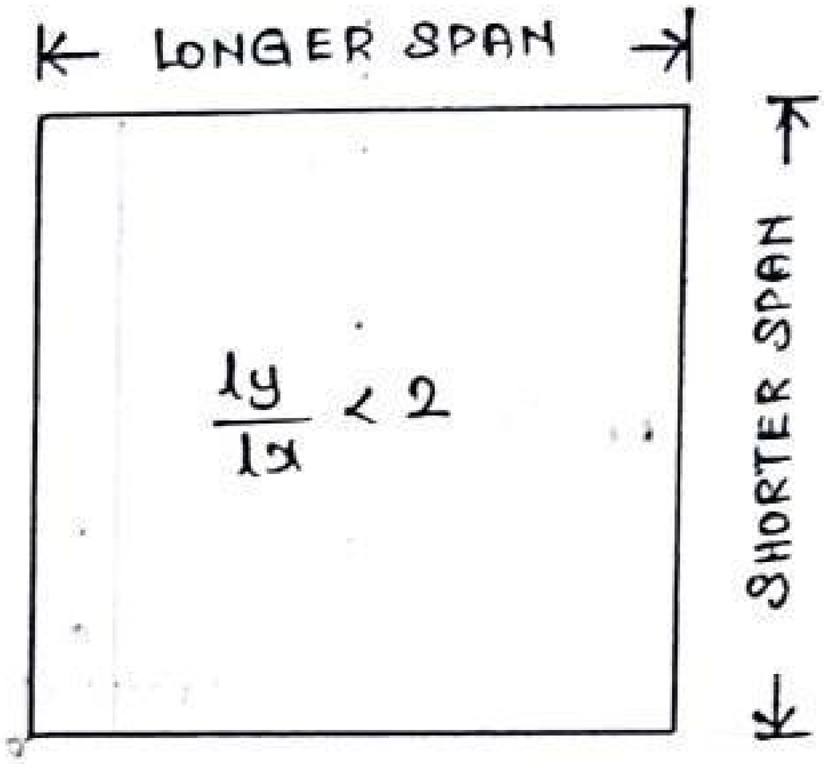
$$\frac{l_y}{l_x} = \frac{5}{5} = 1 < 2$$

Hence Two way slab is adopted

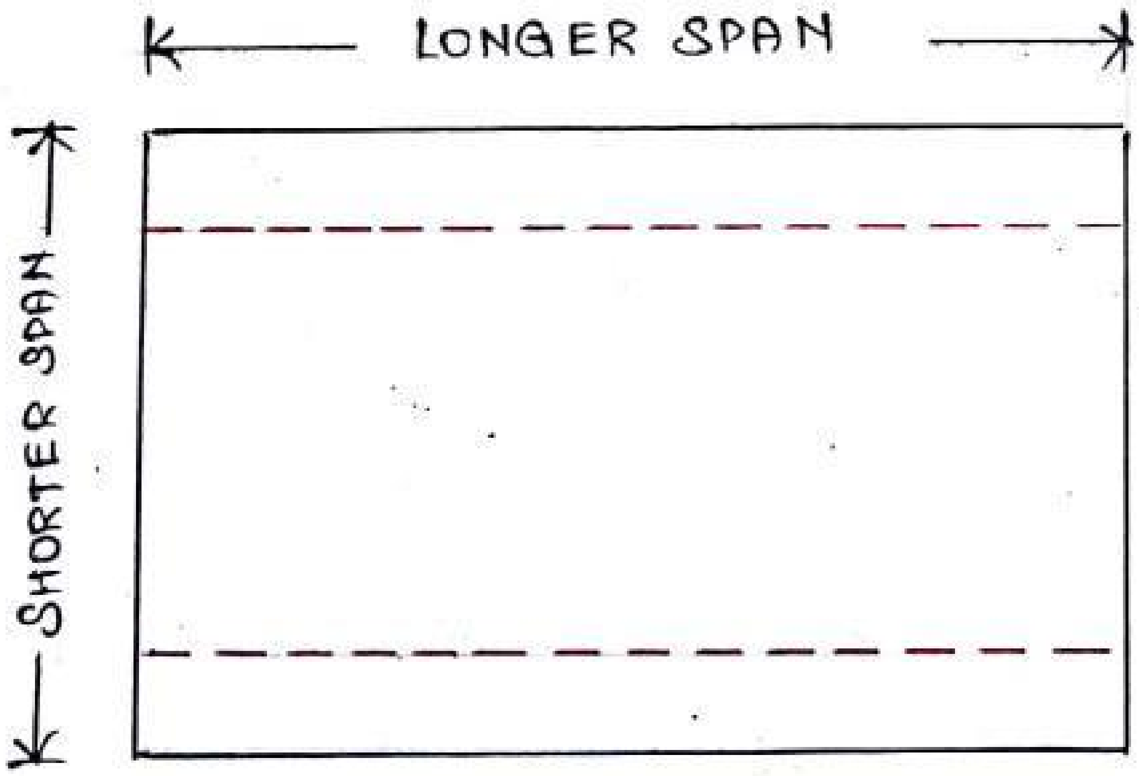
Difference between One way slab & Two way slab

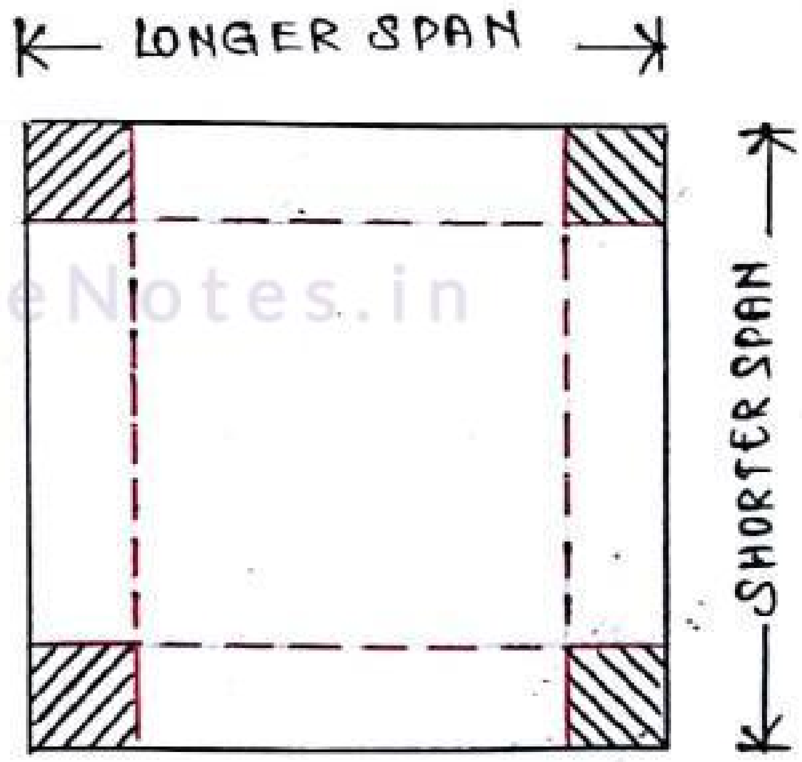
One way Slab	Two way Slab
1	



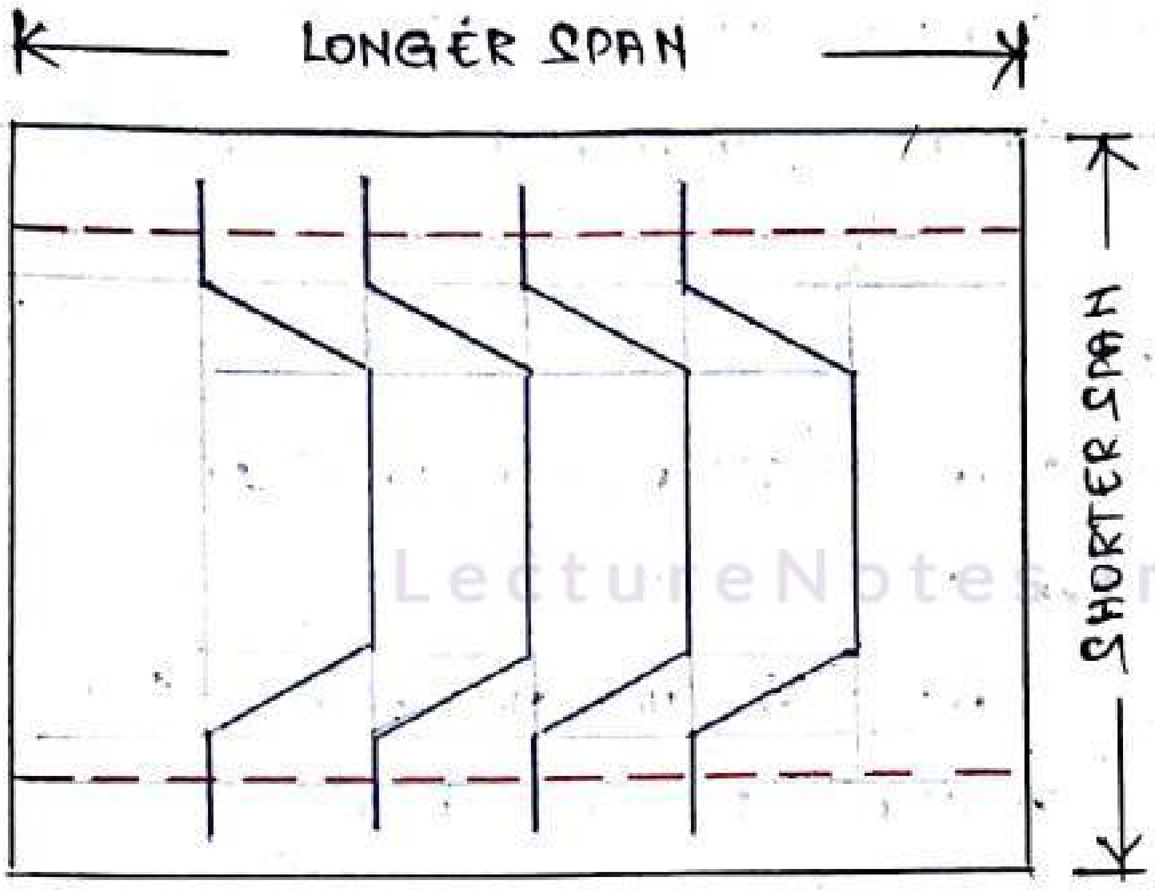


One way Slab	Two way Slab
$\frac{\text{Longer Span}}{\text{Shorter Span}} > 2m$	$\frac{\text{Longer Span}}{\text{Shorter Span}} < 2m$
$\frac{l_y}{l_x} = \frac{8}{3} = 2.66 > 2$	$\frac{l_y}{l_x} = \frac{5}{5} = 1 < 2$
2. Supported by beam from Two opposite sides	Supported by beam from Four opposite sides

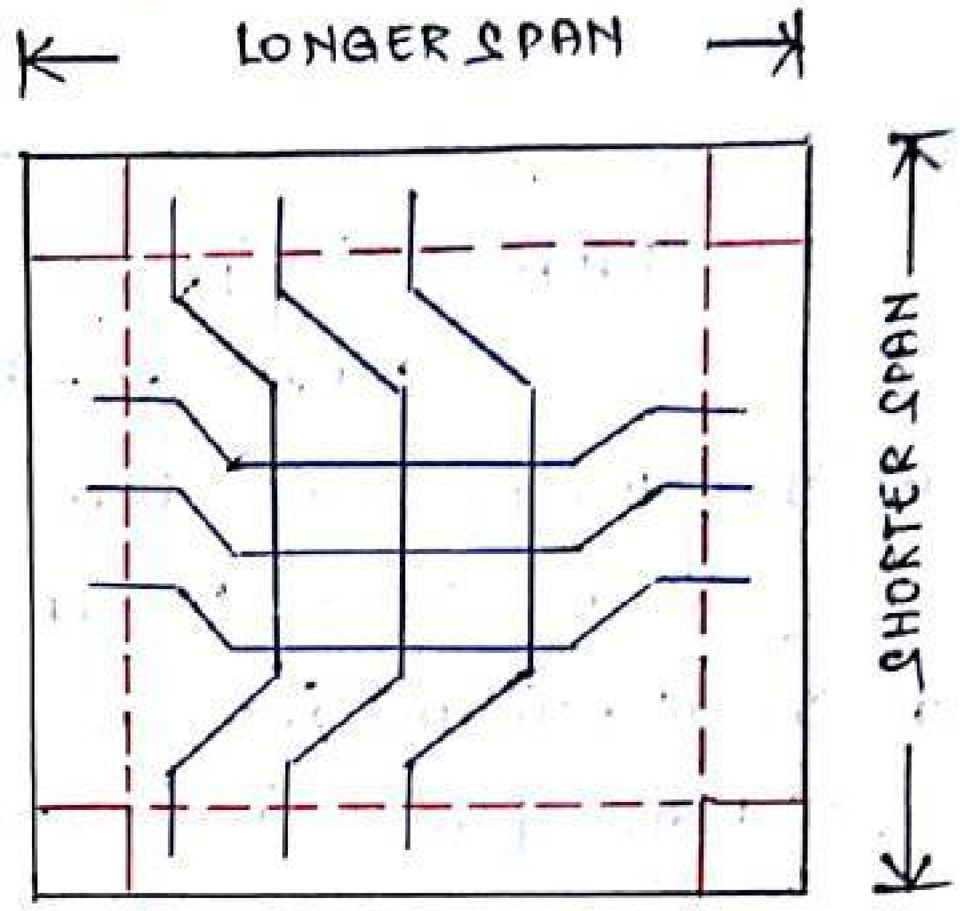




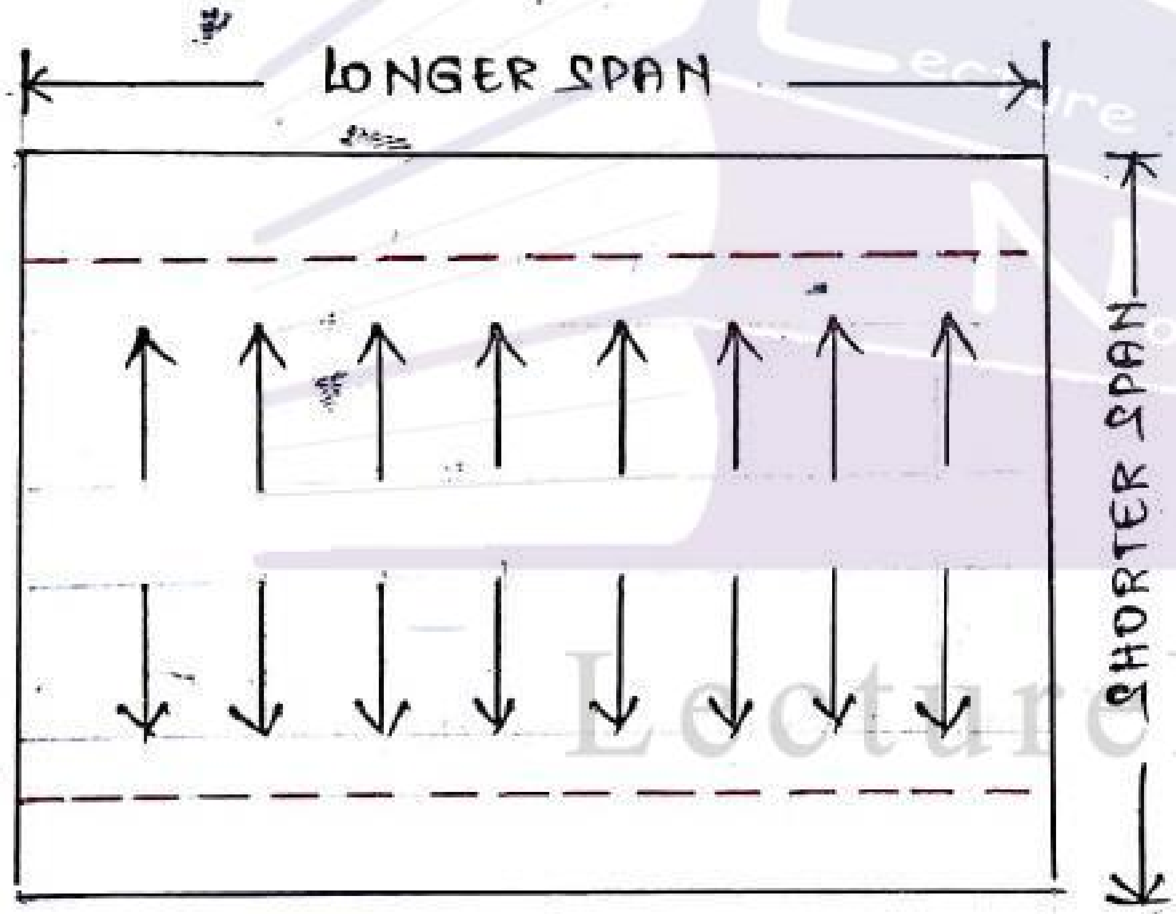
3. Main bars is provided in One direction only



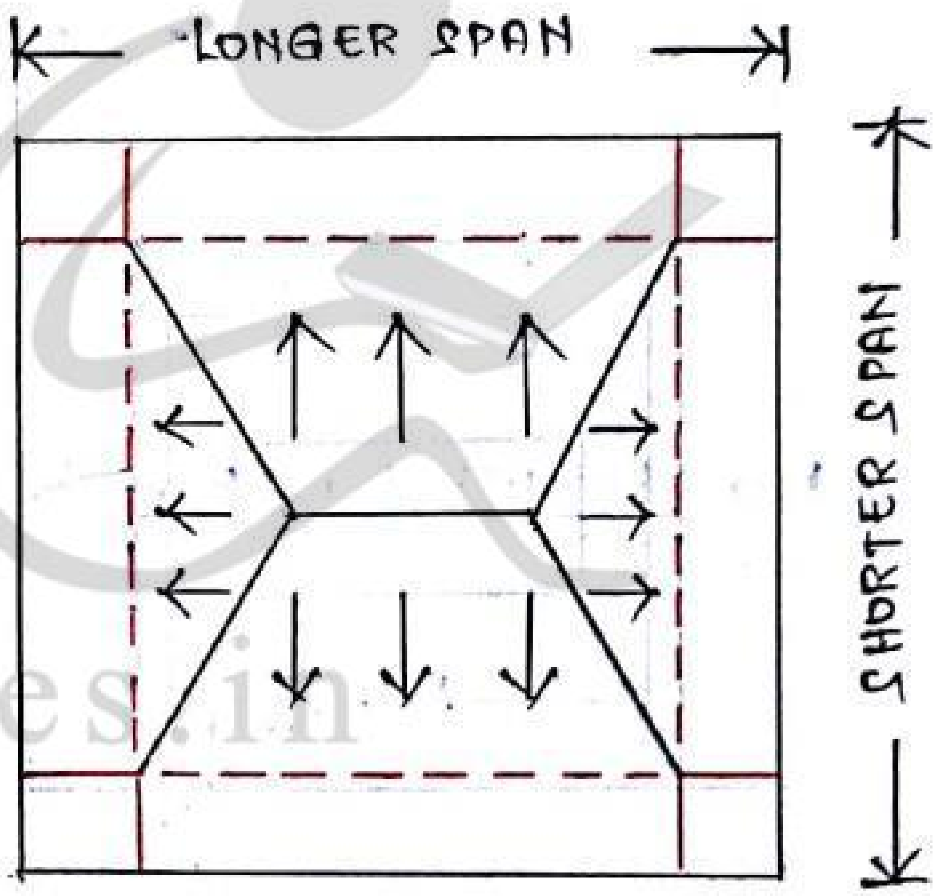
Main bars is provided in both direction only



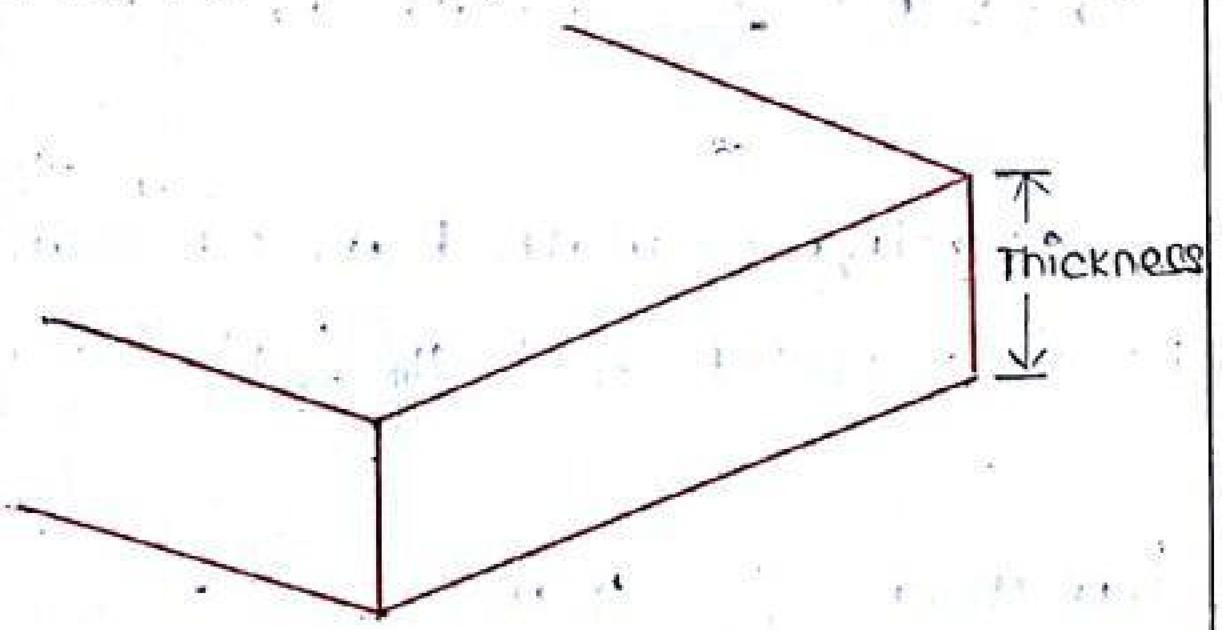
4. Thickness of Slab is More



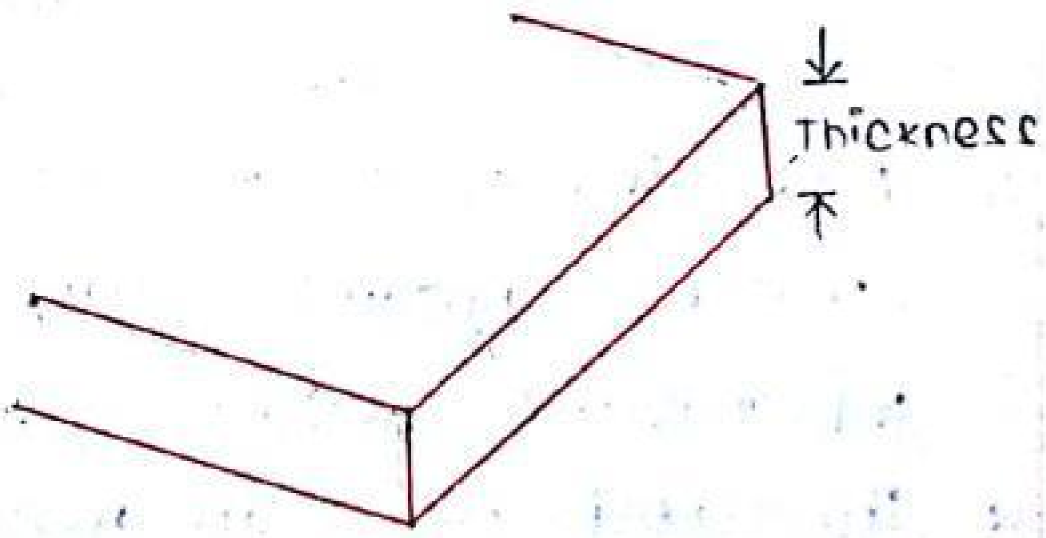
Thickness of Slab is Less



5. Thickness of Slab is more



Thickness of Slab is Less



6. Less economic (costly).

More economic

DESIGN PRINCIPLES

* The Minimum clear cover shall be 20mm for slab. Generally in slab the dia of steel bar does not exceed 12mm
* According to IS:456-2000, the clear cover may be reduced by 5mm

i.e $20 - 5 = 15mm$

* The dia of steel bar used in slab shall not exceed in $\frac{1}{8}$th of Total thickness of slab i.e $0.125D$
* The minimum area of steel used in slab in any direction (Main bar & Distribution bar) shall not be less than

$$A_{st\,(minimum)} = \frac{0.15}{100} \times A_g \text{ (Gross area of slab)}$$

$$= \frac{0.15}{100} \times (b \times D)$$

$$A_{st\,(min)} = \frac{0.15}{100} \times (1000 \times D) \leftarrow (Fe\ 250)$$

For Fe 415 (or) 500 HYSD bars

$$A_{st\,(min)} = \frac{0.12}{100} \times A_g \text{ (Gross area of slab)}$$

$$= \frac{0.12}{100} \times (b \times D)$$

$$A_{st\,(min)} = \frac{0.12}{100} \times (1000 \times D) \leftarrow (Fe\ 415 \text{ or } Fe\ 500)$$

* According to IS:456-2000, the spacing of main bars of slab shall not exceed 3 times effective depth of slab (or) 300mm whichever is lesser
* The depth of slab may be taken as $\frac{1}{25}$ span
* Find the factored moment M_u in 1m width of slab
* Determine M_{ulim} for 1000mm wide slab If $M_u < M_{ulim}$

design the singly reinforced section $M_u > M_{u lim}$, Increase the depth and design singly reinforced section.

* The required area of steel Ast is to be provided in 1000 mm width. Instead of find number of bars 1000mm width

$$S = \frac{\pi/4\ \phi^2}{A_{st}} \times 1000$$

S is the spacing of the bars using the following expression. The spacing should not be more than 3D or 300mm

* check for shear in 1000mm strip. For this find

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

* The design shear strength of concrete may be taken as $\tau_c K_s$ where K_s is given by

Refer IS: 456-2000
Cl. No: 40.2.1.1
Pg. No: 72

Hence, if $\tau_v < \tau_c$, Hence ok

If $\tau_v > \tau_c$, Increase the thickness of slab and redesign

* check for Deflection control

Since K_c, K_f the modification factor are unity

$$\left(\frac{l}{d}\right)_{Max} = K_t \times \text{Basic value}$$

Determine K_t from Fig: 4 in IS: 456

$$\left(\frac{l}{d}\right)_{Provided} < \left(\frac{l}{d}\right)_{Max}$$

Hence it is ok

Distribution Steel

A minimum of 0.15 percent of Total cross section if mild steel is used (or) 0.12 percent of Total cross Section if Fe 415 is used, is required as distribution steel

$$A_{st} = \frac{0.12}{100} bD$$

Hence Spacing of distributing steel is given by

$$S = \frac{\pi/4\ \phi^2}{A_{st}} \times 1000\,mm$$

Where ϕ is diameter of distribution steel

DESIGN PROCEDURE FOR ONE WAY SIMPLY SUPPORTED SLAB

Step : 1

Type of Slab

$$\boxed{\frac{L_y}{L_x} > 2}$$

Has to be designed as One Way Slab

Step : 2

Thickness of Slab

Assume,

Refer IS : 456-2000
Cl - 23.2.1 (a)
Pg. no: 37

(i) Effective depth (d)

$$\text{Effective depth (d)} = \left(\frac{\text{Span}}{\text{depth}}\right)$$

(ii) Overall depth (D)

$$D = \underset{(d)}{\text{Effective depth}} + \underset{(d')}{\text{Clear Cover}} + \frac{\text{Dia. of Main bar}}{2}$$

Step:3

Effective Span

Cl: 22.2
Pg. No: 34

(i) Clear Span + Effective depth
(ii) Clear span + C/C of supports

} Whichever is less

Adopt

L = ______ m

Step: 4

Loading

Consider 1m width of slab

Dead load

Self weight of slab = (Slab thickness × b × Density of RCC)

= D × 1 × 25 KN/m²

Floor finish = ______ KN/m²

Refer IS: 875-1987 Part-2

Live load = ______ KN/m²

Total Load (W) = ______ KN/m²

Design Load (Wu) = PSF × W

Step: 5

Ultimate moment and Shear Forces

(i) Ultimate moment (Mu)

$$\boxed{M_u = \frac{W_u L^2}{8}}$$

(ii) Shear Force (Vu)

$$\boxed{V_u = \frac{W_u L}{2}}$$

Step: 6

Limiting Moment of Resistance

$M_{u\,lim} = 0.148 \times f_{ck} \cdot bd^2$ (Fe 250)

$M_{u\,lim} = 0.138 \times f_{ck} \cdot bd^2$ (Fe 415)

$M_{u\,lim} = 0.133 \times f_{ck} \cdot bd^2$ (Fe 500)

$$\boxed{M_u < M_{u\,lim}}$$

Hence the section is Under reinforced

Step: 7

Main Reinforcement (G-1.1 (b)) Pg. No: 96

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

$A_{st_{req}}$ = ________ mm^2

Check for Minimum Reinforcement (Cl: 26.5.2.1 Pg. No: 48)

$$A_{st_{min}} = \frac{0.12}{100} \times b \times D$$

$A_{st_{req}} > A_{st_{min}}$

Hence ok

Spacing

$$S = \frac{a_{st}}{A_{st}} \times b$$

Check for Spacing (Cl: 26.3.3 (b) Pg. No: 46)

(i) 3d
(ii) 300

Whichever is small

Step: 8

- **Distribution reinforcement** (Cl-26.5.2.1, Pg. No: 48)

$$\boxed{Ast_{min} = \frac{0.12}{100} \times b \times D}$$

Spacing

$$\boxed{S = \frac{a_{st}}{Ast_{min}} \times b}$$

Check for Spacing (Cl: 26.3.3, Pg. No: 46)

(i) 5d
(ii) 450 mm

$\Big\}$ Whichever is small

Step: 9

Check for Shear Stress

Nominal Shear Stress (Cl: 40.1, Pg. No: 72)

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

Percentage of Tension reinforcement

$$\boxed{P_t = \frac{100\, Ast_{req}}{bd}}$$

Design Shear Strength of Concrete

Refer IS : 456 - 2000

Table No : 19

Pg. No : 73

P_t		τ_v
xx	$\rightarrow$	xx
xx	$\rightarrow$	?
xx	$\rightarrow$	xx

$\tau_c =$ ______ N/mm^2

$\tau_v < \tau_c$

Hence the slab is safe in shear

Step: 10

Check for deflection Control

$$\left(\frac{L}{d}\right)_{max} = \left(\frac{L}{d}\right)_{basic} \times K_t \times K_c \times K_f$$

$K_t \rightarrow$ (Fig. no: 4, Pg. no: 38)

$K_c \rightarrow$ (Fig. no: 5, Pg. no: 39)

$K_f \rightarrow$ (Fig. no: 6, Pg. no: 39)

$$\boxed{f_s = 0.58 \cdot f_y \times \frac{A_{st\,req}}{A_{st\,prov}}}$$

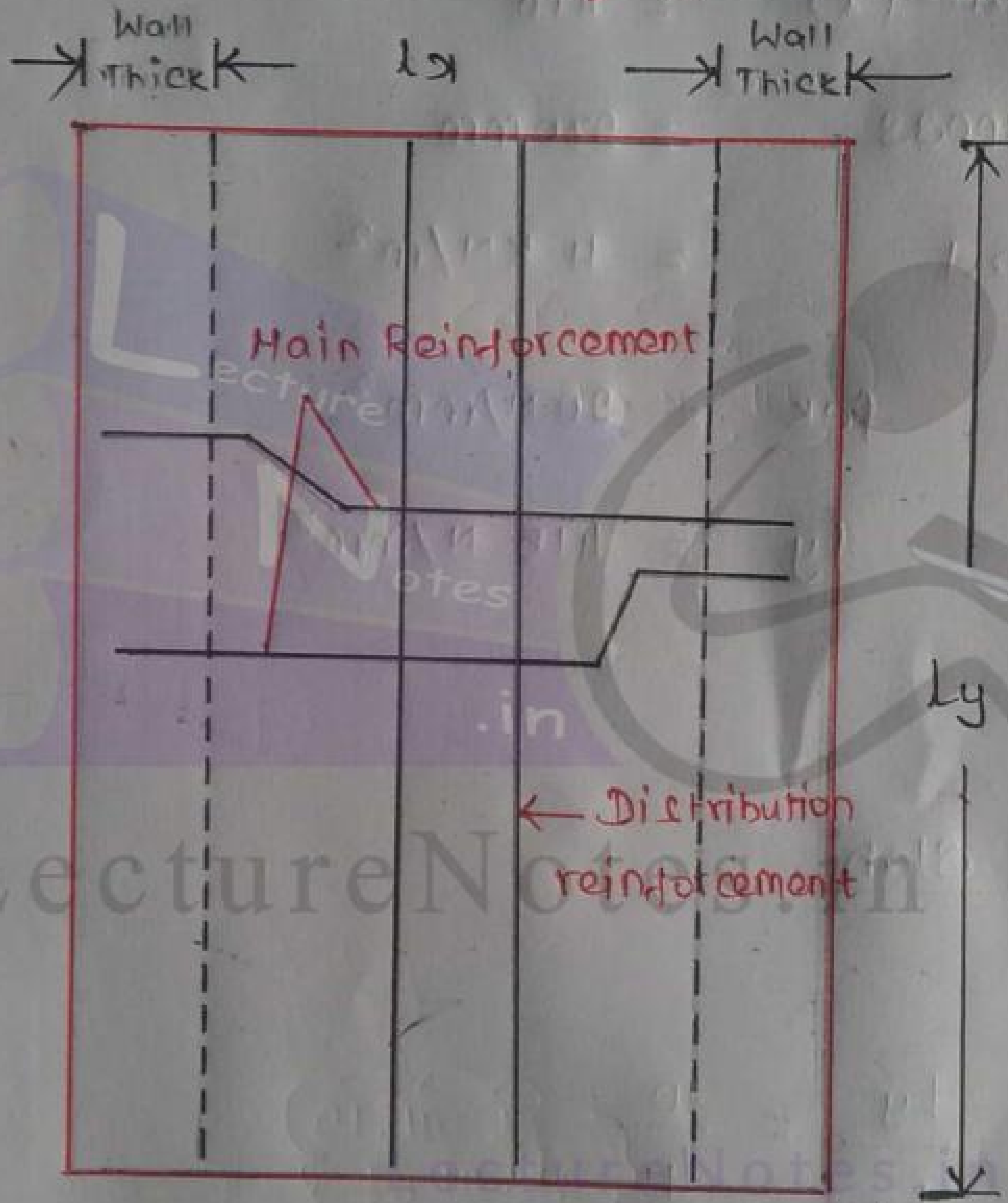
$$\left(\frac{L}{d}\right)_{provided} = xx$$

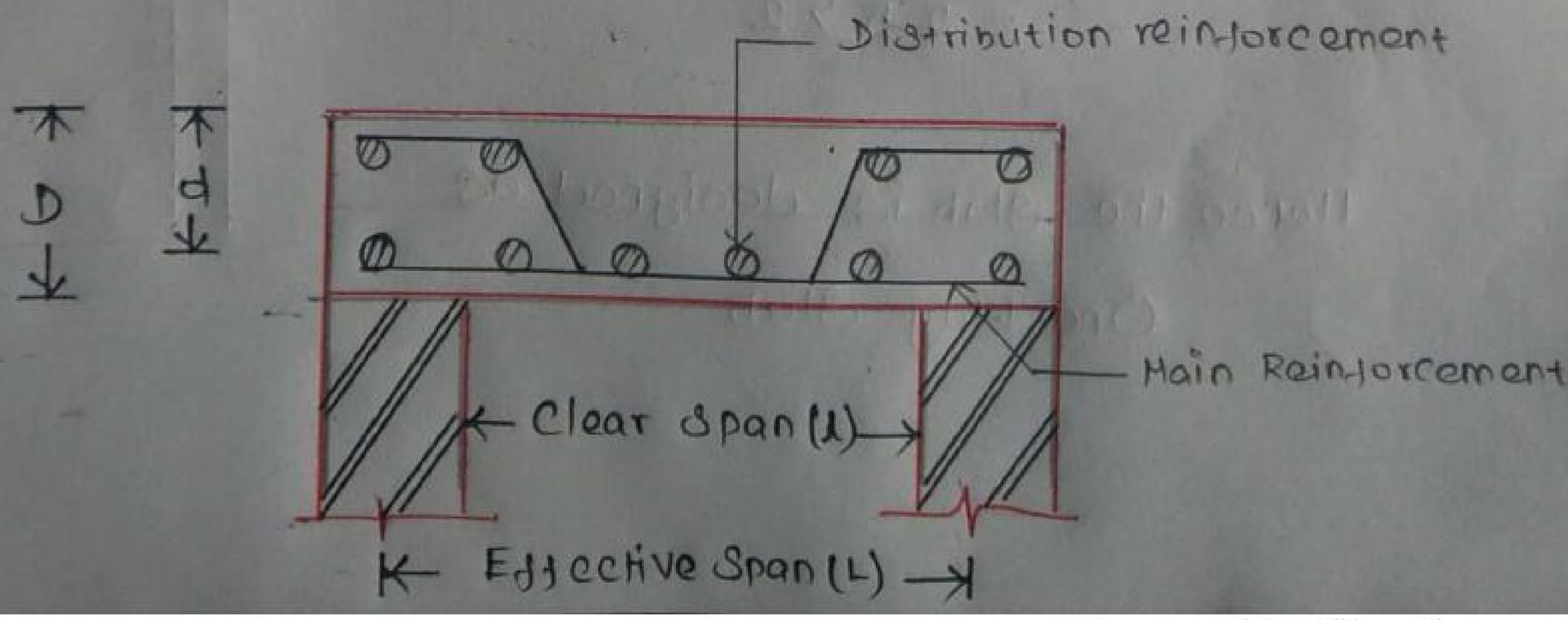
- $\left(\frac{L}{d}\right)_{prov} < \left(\frac{L}{d}\right)_{Max}$

Hence the deflection criterion is satisfied

Step: 11

Reinforcement detailing of One Way Slab





Prblm. No: 1

Design a Simply Supported Rcc slab for an Office Floor having clear dimension. of 4m by 10m with 230 walls all-round. Adopt M-20 Grade of Concrete and Fe 415 Grade of Steel

Given data

Clear Span (l) = 4m

Wall Thickness = 230 mm

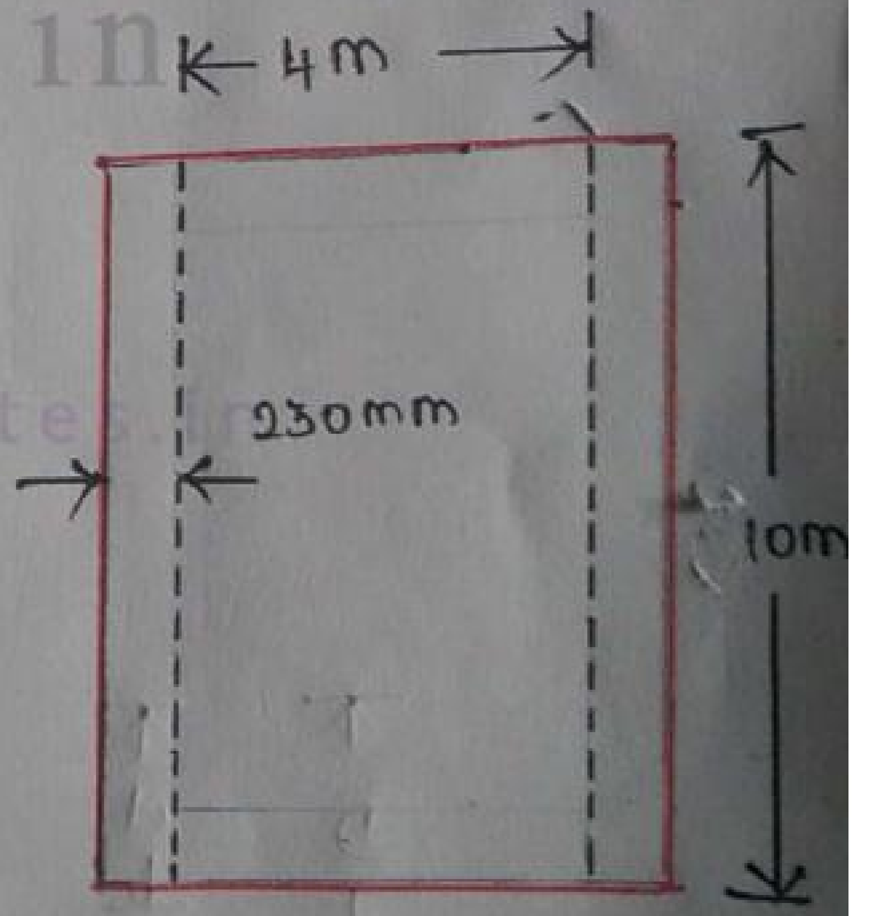
Live load = 4 KN/m^2

f_{ck} = 20 N/mm^2

f_y = 415 N/mm^2

Step: 1

Type of slab



$$\frac{L_y}{L_x} = \frac{10}{4}$$

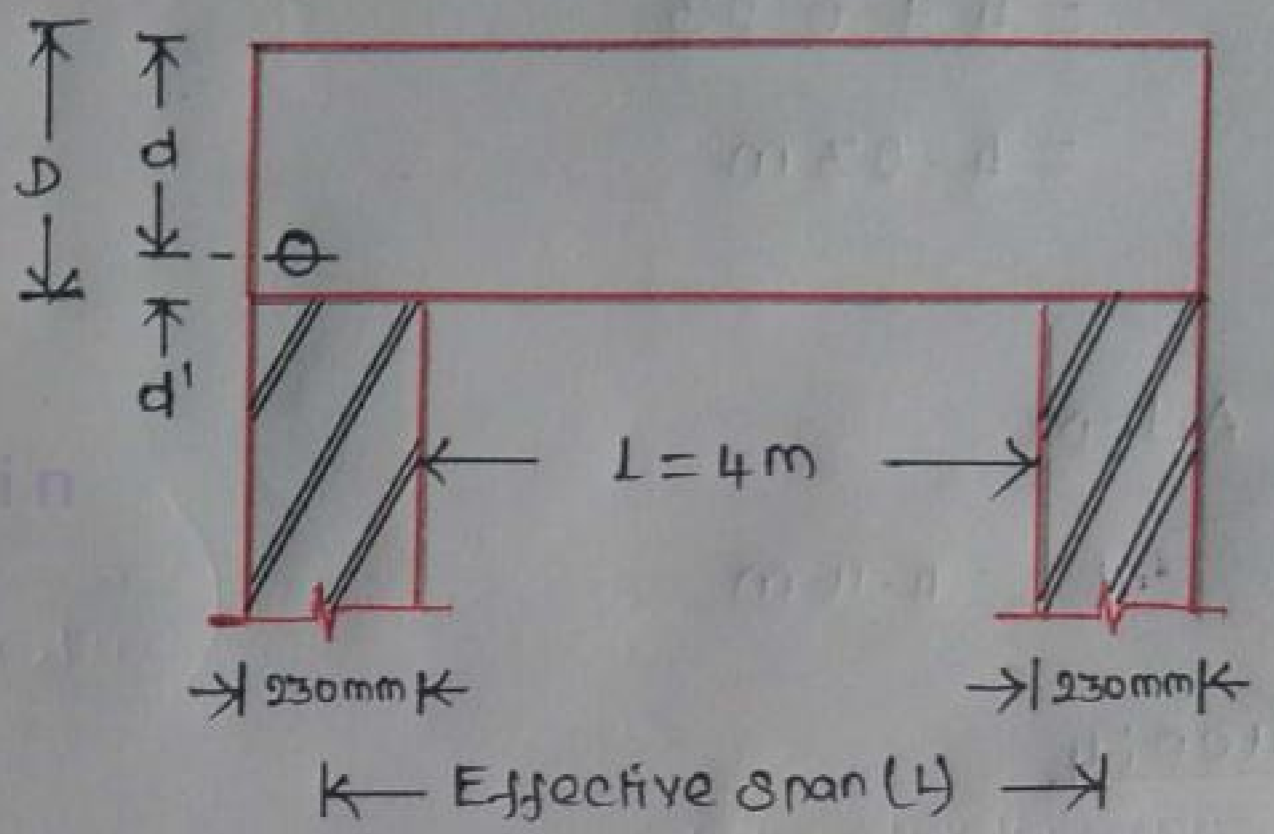
$$= 2.5 > 2$$

Hence the slab is designed as One way slab

Step: 2

- ### Thickness of Slab

(i) Effective depth (d)



Assume,

$$d = \frac{\text{Span}}{\text{depth ratio}}$$

$$= \frac{L}{d}$$

$$= \frac{4000}{25}$$

∴ Simply Supported Slab } = 25

$$\boxed{d = 160\,mm}$$

4m ⇒ 4 (m) × 1000 (mm)

= 4000 mm

(ii) Overall depth (D)

$$D = \underset{(d)}{\text{Effective depth}} + \underset{(d')}{\text{Clear Cover}} + \frac{\text{Dia. of Main bar}}{2}$$

$$= 160 + 20 + \frac{10}{2}$$

$$\boxed{D = 185\,mm}$$

Step: 3

Effective span (L)

(i) Clear span + Effective depth

= 4 + 0.16

= 4.16m

160mm (÷ 1000 m)

= 0.16m

(iii) Clear Span + Centre to Centre of Supports

$= 4 + (0.115 + 0.115)$

230mm $(\div 1000\,m)$

$= 0.23\,m$

$= 4 + 0.23$

$= 4.23\,m$

Adopt

$L = 4.16\,m$

Step: 4

Loadings

Consider 1m width of slab

Dead Load

Self Weight of slab $= D \times b \times$ Density of RCC

$= 185 \times 1 \times 25$

$= 4.625\ KN/m^2$

Floor Finish $= 1.500\ KN/m^2$

Live load $= 4.000\ KN/m^2$

Total Load (W) $= 10.125\ KN/m^2$

Design Load (Wu) $=$ W $\times$ PSF

$= 10.125 \times 1.5$

$\boxed{Wu = 15.19\ KN/m^2}$

Step:5

- Ultimate moment and Shear forces

(i) Ultimate Moment (Mu)

$$Mu = \frac{WuL^2}{8}$$

$$= \frac{15.19 \times (4.16)^2}{8}$$

$$\boxed{Mu = 32.86 \text{ KN}\cdot\text{m}}$$

(ii) Shear Force (Vu)

$$Vu = \frac{WuL}{2}$$

$$= \frac{15.19 \times 4.16}{2}$$

$$\boxed{Vu = 31.60 \text{ KN}}$$

Step:6

Limiting Moment of Resistance

$$Mu_{lim} = 0.138 \times fck \cdot bd^2 \quad (Fe\ 415)$$

$$= 0.138 \times 20 \times 1000 \times (160)^2$$

$$\boxed{Mu_{lim} = 70.65 \text{ KN}\cdot\text{m}}$$

$$Mu < Mu_{lim}$$

Since the section is Under Reinforced.

Step: 7

Main Reinforcement

$$Mu = (0.87\ fy \cdot Ast \cdot d)\left[1 - \left(\frac{Ast \cdot fy}{b \cdot d \cdot fck}\right)\right]$$

$$32.86 \times 10^6 = (0.87 \times 415 \times Ast \times 160)\left[1 - \left(\frac{Ast \cdot 415}{1000 \times 160 \times 20}\right)\right]$$

$$= 57768\ Ast \times (1 - 1.296 \times 10^{-4}\ Ast)$$

$$32.86 \times 10^6 = 57768\ Ast - 7.49178\ Ast^2$$

$$7.49178\ Ast^2 - 57768\ Ast + 32.86 \times 10^6$$

$$\boxed{Ast_{req} = 531\ mm^2}$$

Check for Minimum Reinforcement

$$Ast_{min} = \frac{0.12}{100} \times b \times D$$

$$= \frac{0.12}{100} \times 1000 \times 185$$

$$\boxed{Ast_{min} = 222\ mm^2}$$

$$Ast_{req} > Ast_{min}$$

Spacing

$$S = \frac{a_{st}}{Ast_{req}} \times b$$

(Assume, 10 mm dia bars)

$$= \frac{\frac{\pi}{4} \times 10^2}{531} \times 1000$$

$$\boxed{S = 147\ mm}$$

Check for Spacing

- (i) $3d = 3 \times 160$

$= 480 mm$

(ii) $300 mm$

$300 mm > 147 mm$

Hence OK

Provide 10mm ϕ bars as Main reinforcement spacing @ 147mm c/c

Step: 8

Distribution Reinforcement

$$Ast_{min} = \frac{0.12}{100} \times b \times D$$

$$= \frac{0.12}{100} \times 1000 \times 185$$

$$\boxed{Ast_{min} = 220 \ mm^2}$$

Spacing

$$S = \frac{ast}{Ast_{min}} \times b$$

$$= \frac{\frac{\pi}{4} \times (8)^2}{220} \times 1000 \quad \text{(Assume 8mm } \phi \text{ bars)}$$

$$= 228.47 \simeq 230 mm$$

$$\boxed{S = 230 mm}$$

Check for Spacing

(i) 5d $= 5 \times 160$

$= 800mm$

(ii) 450 mm

450mm > 230mm

Hence OK

Provide 8mm ϕ bars as distribution reinforcement spacing @ 230 mm c/c

Step:9

Check for Shear Stress

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

$$= \frac{31.6 \times 10^6}{1000 \times 160}$$

$$\boxed{\tau_v = 0.198 \ N/mm^2}$$

Percentage of Tension Reinforcement

$$P_t = \frac{100 \ Ast_{req}}{bd}$$

$$= \frac{100 \times 531}{1000 \times 160}$$

$$\boxed{P_t = 0.33 \%}$$

P_t	τ_c
0.25 →	0.36
0.33 →	?
0.50 →	0.48

$$\frac{(0.50-0.25)}{(0.50-0.33)} = \frac{(0.48-0.36)}{(0.48-x)}$$

$$\boxed{\tau_c = 0.39\ N/mm^2}$$

$$\tau_v < \tau_c$$

Hence the slab is safe in shear

Step: 10

Check for deflection Control

$$\left(\frac{L}{d}\right)_{max} = \left(\frac{L}{d}\right)_{basic} \times K_t \times K_c \times K_f$$

$$= 20 \times 1.4 \times 1 \times 1$$

$$\left(\frac{L}{d}\right)_{max} = 29$$

$$\left(\frac{L}{d}\right)_{prov} = \frac{4160}{160}$$

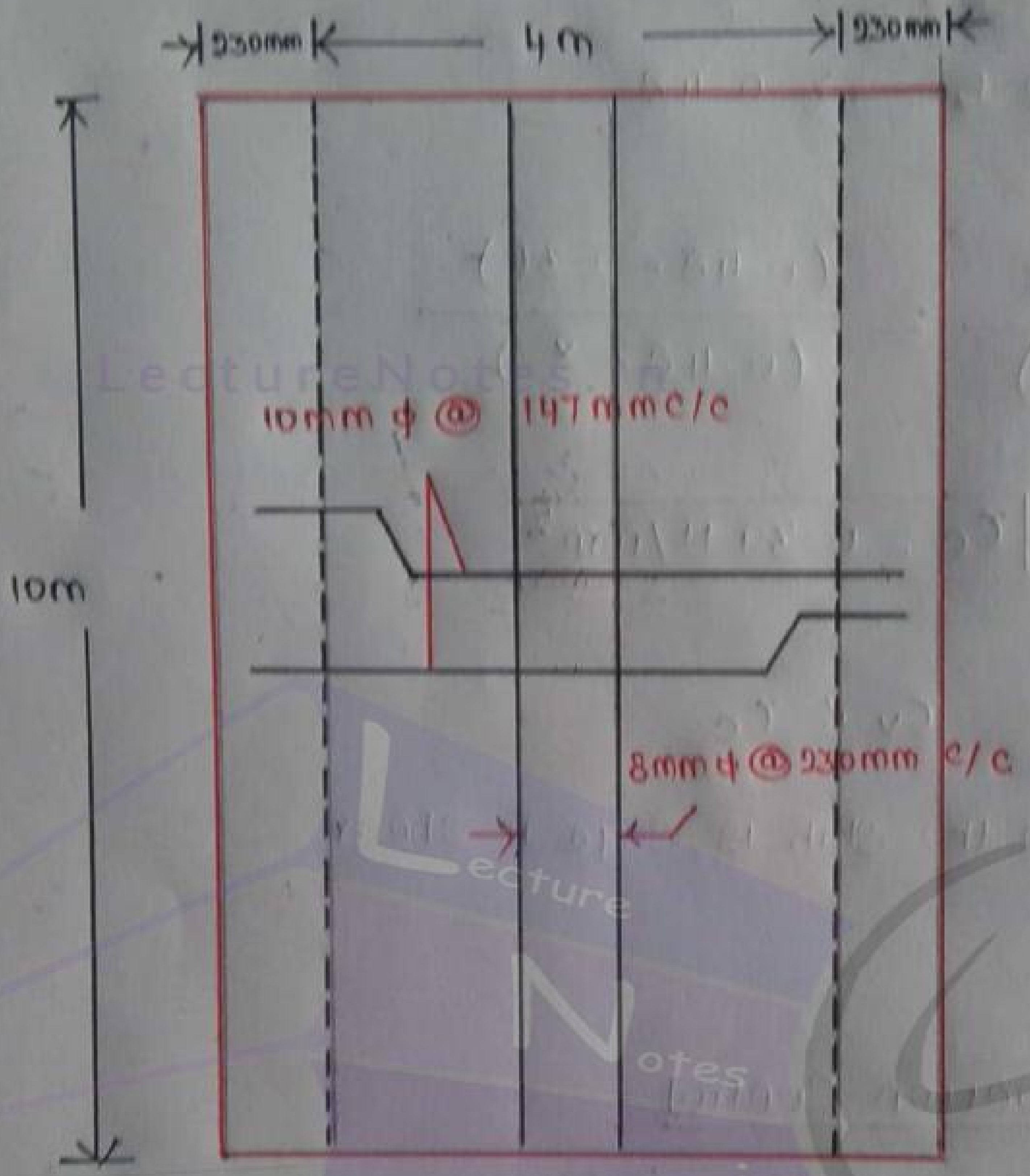
$$= 26$$

$$\left(\frac{L}{d}\right)_{prov} < \left(\frac{L}{d}\right)_{max}$$

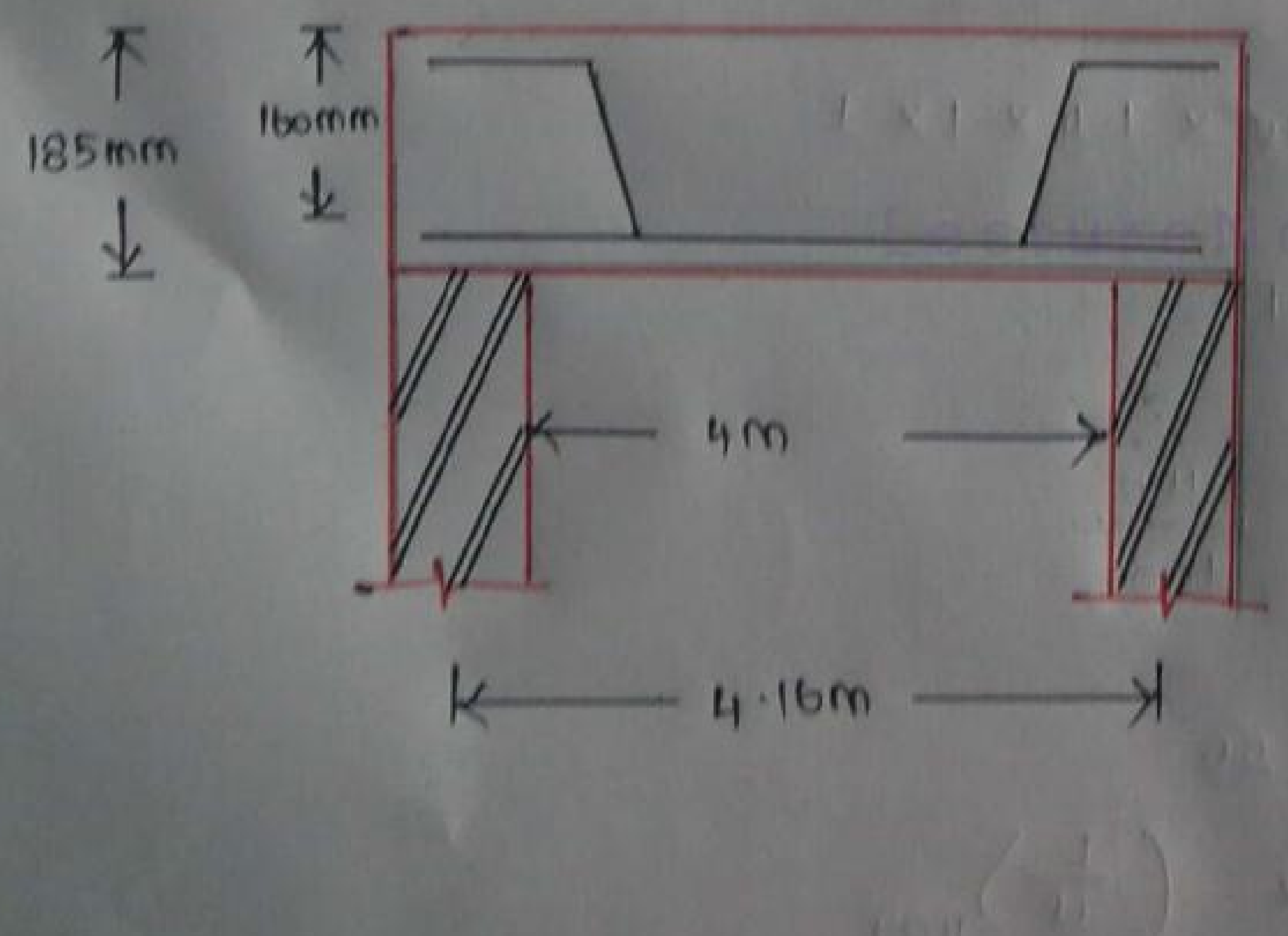
Hence the deflection Criterion is Satisfied

Step: 11

Reinforcement detailing of One Way Slab



PLAN

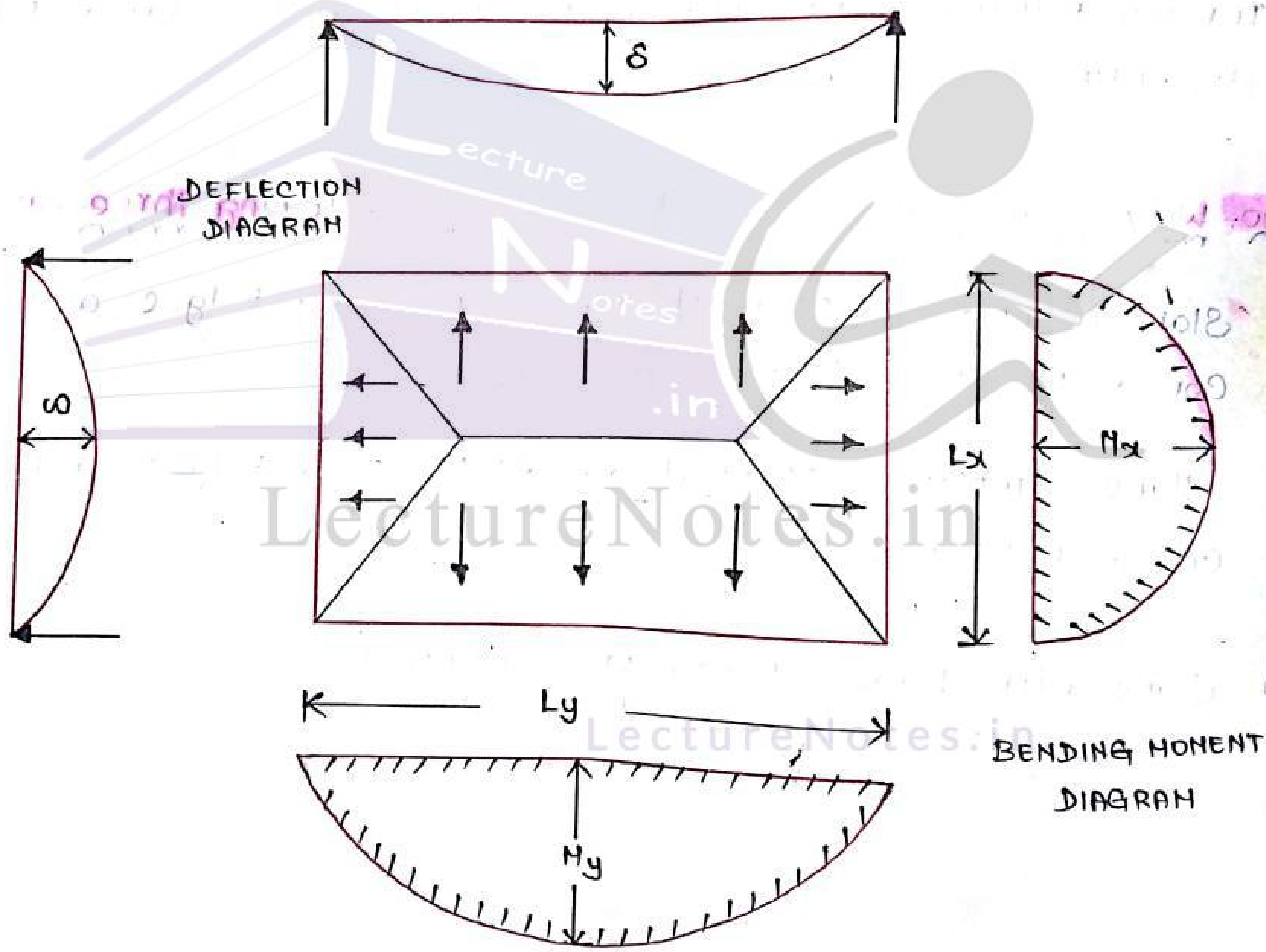


TWO WAY SLABS

Introduction

If the slabs are supported on all the four edges and if the ratio of the Long span to short span is less than two, the slab are likely to bent along the two spans and such slabs are called Two Way Slabs

$$\boxed{\frac{L_y}{L_x} < 2}$$



TWO WAY SLAB ACTION WITH MOMENT AND DEFLECTION

* Due to bending in two direction the bending moment and deflection in Two way slabs are reduced resulting in Thinner section

* But such slabs have to be reinforced in both the direction
* Two Way Slabs may have their corners held down or free to lift.
* If the corners are not held down they are lifted up under load.
* If the corners are held down against lifting the bending moment and deflection are further reduced
* When the corners are held down Torsional reinforcement has to be provided to prevent cracking of the corners
* Torsion reinforcement is to be provided at the corners having discontinuous Edges

Two Way Slab can be divided into the following Three categories

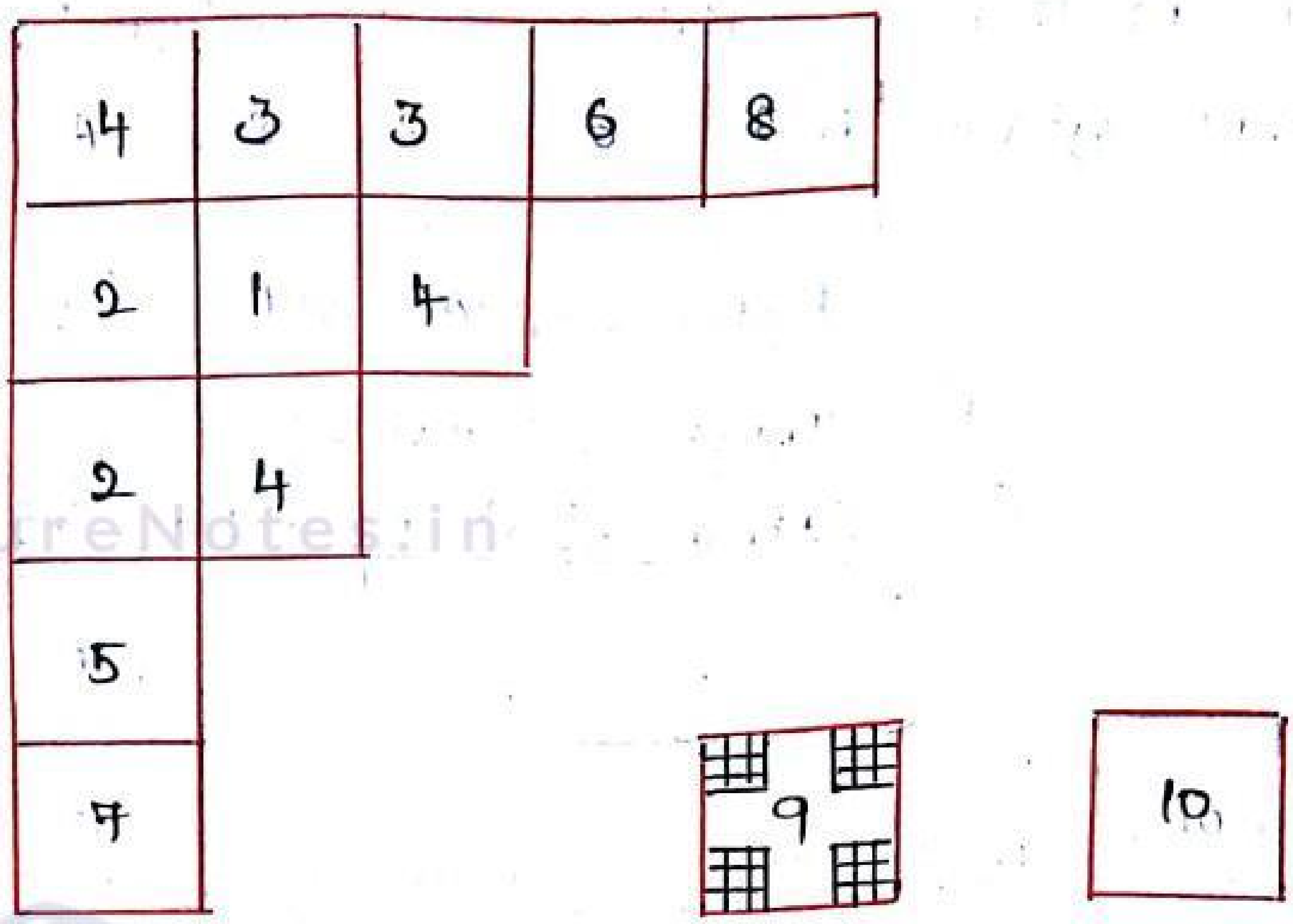
(i) Slabs simply supported on all the four Edges and corner free to lift

(ii) Slabs simply supported on all the four Edges and corner Held down

(iii) Slabs with Edges fixed or continuous

Bending Moment in the slab depend upon the following parameters

-



- The Edge Condition

1) All four Edges continuous
2) One short Edge discontinuous
3) One Long Edge discontinuous
4) Two Adjacent Edge discontinuous
5) Two short Edge discontinuous
6) Two long Edge discontinuous
7) Three Edges discontinuous and one Long edge continuous
8) Three edges discontinuous and one short edge continuous
9) Four Edges discontinuous but corners held down by providing Torsion reinforcement
10) Simply supported slab without Torsion reinforcement

DESIGN OF TWO WAY SLABS

For Two Way Slabs carrying Uniformly distributed Load the design is to be carried out using Bending moment co-efficient given in

Refer IS: 456-2000

TABLE - 26 and 27

(Appendix - D) Pg. No: 91, 92

Two Way Restrained Slabs - Corners prevented from Lifting

When the Corners of Slab are prevented from Lifting the Slab may be designed as specified below

* The maximum B.M per unit width in a slab is given by the Following Equations

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

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

Where

α_x and α_y are Co-efficient given in Table - 26

w = Total design load per unit area

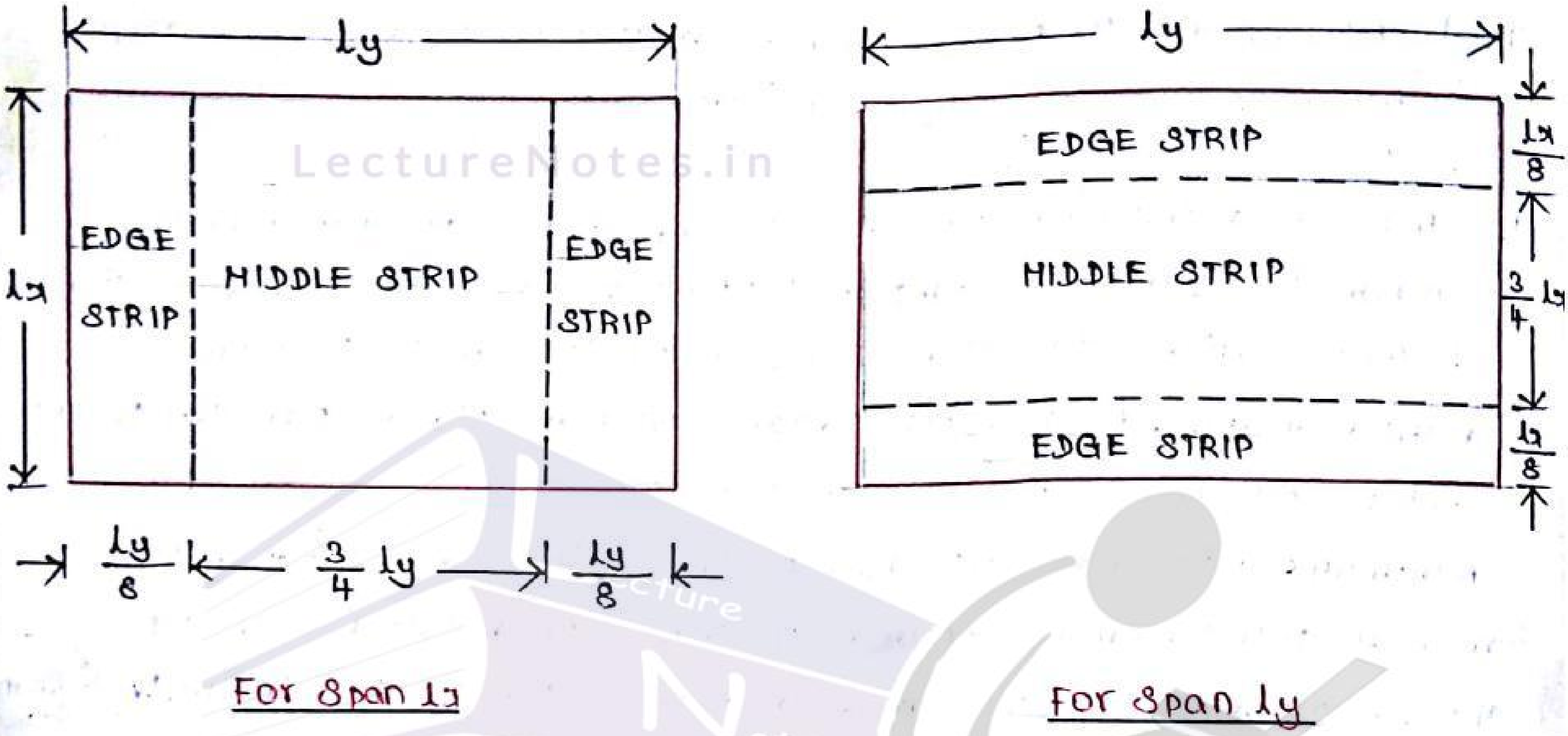
M_x, M_y = Moment on stripe of unit width spanning l_x and l_y

l_x, l_y = Length of the Shorter Span and Longer Span

* Slabs are considered as divided in each direction into

- Middle Strips and Edge Strips

* The Middle Strip being three - Quarters of the Width and each Edge Strip one - Eight of the Width



DIVISION OF SLAB INTO MIDDLE AND EDGE STRIPS

* The Maximum moment calculated apply only to the Middle Strips and No Redistribution shall be made

* Tension reinforcement provided at Mid - span in the middle strip shall Extend in the lower part of the slab to within 0.25 l of a Continuous Edge. or 0.15 l of a discontinuous edge.

* Over the continuous Edges of a Middle strip, the Tension reinforcement shall Extend in the upper part of the slab a distance of 0.15 l from the support, and at least 50 percent shall extend a distance of 0.3 l

* At a discontinuous edge, Negative moments may arise. They depend on the degree of Fixity at the Edge of the slab but, In general, Tension reinforcement equal to 50 percent of that provided at mid-span extending 0.1 l into the Span will be sufficient

* Reinforcement In Edge strip, parallel to that Edge, shall comply with the minimum given in section per the code

* Torsion reinforcement shall be provided at any corner wher the slab is simply supported on both Edges meeting at that Corner. It shall consist of Top and Bottom reinforcement, Each with layers of bars placed parallel to the sides of the slab and Extending from the edges a minimum distance of one - Fifth of the Shorter span. The area of reinforcement in each of these four layers shall be Three - Quarters of the area required for the maximum Mid-span moment in the slab.

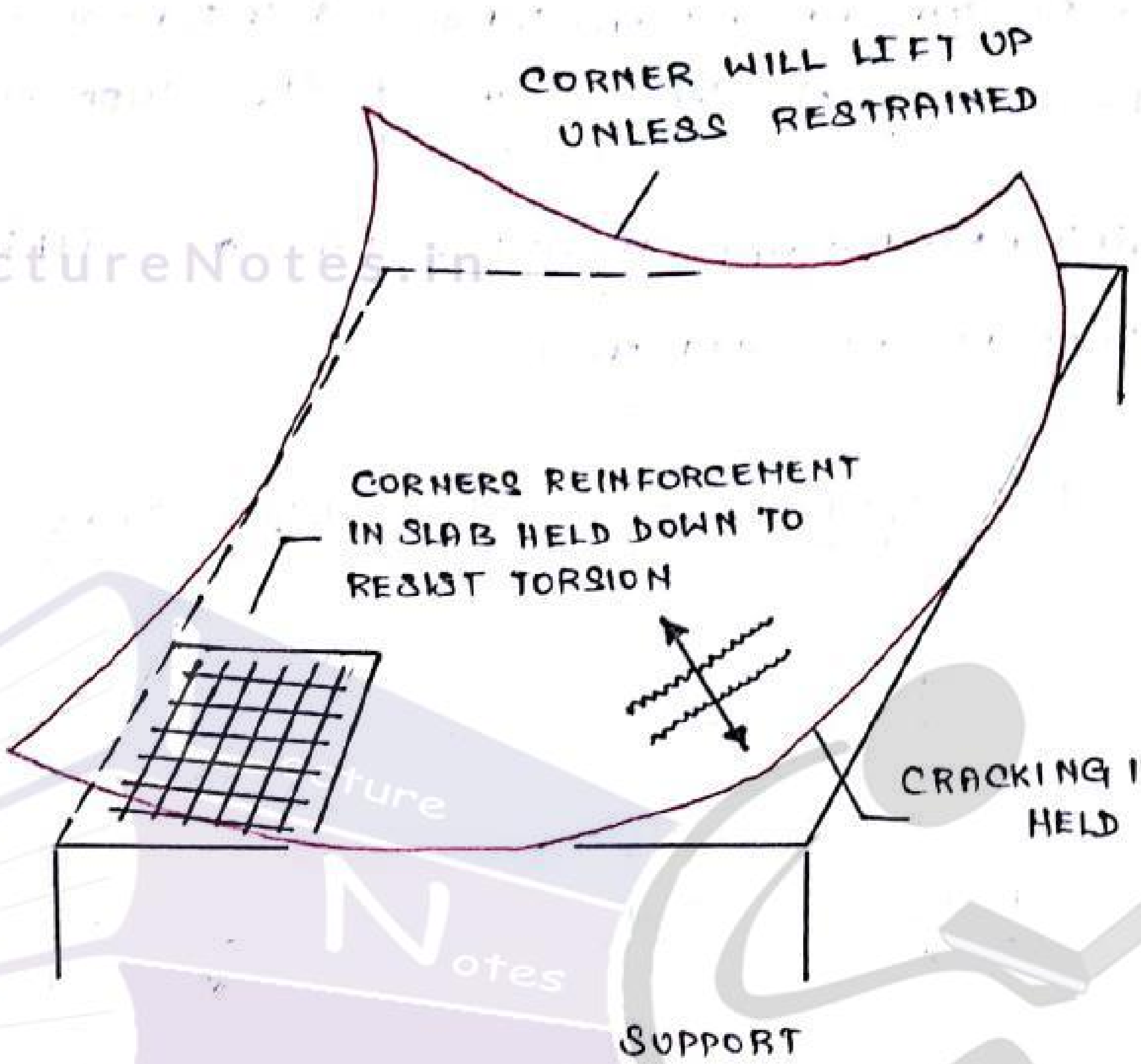
* Torsion reinforcement equal to Half that described shall be provided at a Corner contained by Edges over only one of which the slab is continuous

* Torsion reinforcement need not be provided at any Corner contained by Edges over both of which the slab is continuous

* When $\frac{l_y}{l_x}$ is Greater than 2, the slab shall be designed as spanning one way.

Two Way Simply Supported Slabs - Corners are not Held down.

- When a slab simply supported on all the four sides is subjected to transverse loads, the bending of the slab in the two principal directions causes the corners to curl and lift up



TORSION EFFECTS IN TWO WAY SLAB

* When simply supported slabs do not have adequate provision to resist Torsion at corners and to prevent the corners from lifting, the maximum moment per unit width are given by the following Equation

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

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

Where,

α_x and α_y - Moment Co-efficient given in Table 27

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M_x , M_y - Moment on strip of unit width spanning l_x and l_y.

w - Total design load per unit area

l_x , l_y - Length of the shorter span and longer span

* At least 50 percent of the tension reinforcement provided at Mid-span should extend to the supports.

* The remaining 50 percent should extend to within $0.1\,l_x$ or $0.1\,l_y$ of the support as appropriate

DESIGN PROCEDURE FOR TWO WAY SLAB

Step:1

Type of Slab

$$\boxed{\frac{L_y}{L_x} < 2}$$

Has to be designed as Two Way Slab

Step:2

Thickness of Slab

(i) Depth of Slab

It will be decided by the deflection criteria based on short span (L_x) and the total depth (D)

* The Allowable $\frac{L}{D}$ ratio for Two way slab having span upto 3.5m and subjected to Maximum Live load of 3 KN/m^2 the Code Specifies in clause 24.1

* The deflection may be assumed to be satisfied if span to overall depth ratios satisfy the values given below

Refer IS : 456-2000
Cl : 24.1
Pg. No : 39

SHORT SPAN TO OVERALL DEPTH RATIO

End Condition	L/D Ratio	
	Reinforcement	
	Fe 250	Fe 415
Simply Supported Slab	35	28
Continous Slab	40	32

* The span is more than 3.5m, Adopt a Span/Depth of ratio of 25

Step : 3

Effective Span

(i) Clear Span + Effective depth
(ii) Clear span + Centre to Centre of Supports } Whichever is less

Adopt

L = ______ m

Step: 4

Loading

Consider 1m Width of Slab

$$W_u = 1.5\,(25D + F.F + LL)\ \text{kN/m}$$

Step: 5

- Ultimate Bending Moment and Shear Force

Bending Moment

$$M_{ux} = \alpha_x \cdot w_u \cdot L_x^2$$

$$M_{uy} = \alpha_y \cdot w_u \cdot L_x^2$$

Refer IS:456-2000
Table - 27
Pg.no: 91

α_x and α_y are Co-efficient is given in Table-26 For Given ~~Loading~~ Boundary condition and the aspect ratio $\frac{l_y}{l_x}$

Shear Force

$$V_u = \frac{w_u l_x}{2}\ \text{kN}$$

Step: 6

Check for depth

$$M_{u\,lim} = 0.138\, f_{ck}\, b d^2 \quad (\text{Fe } 415)$$

$$d = \sqrt{\frac{Mu_{lim}}{0.138 fck b}}$$

$d <$ Effective depth

Hence the Effective depth Section is sufficient to resist the design Ultimate moment

Step: 7

Main Reinforcement

Calculate Area of steel required at Four different Location using

$$Mu = 0.87 fy\, Ast\, d \left[1 - \frac{Ast \cdot fy}{b\, d\, fck}\right]$$

Step: 8

Check for Shear Stress

Consider the short span L_x and Unit width of slab, the shear stress is given by

$$\boxed{\tau_v = \frac{Vu}{bd}}$$

$$\boxed{P_t = \frac{100\, Ast}{bd}}$$

Refer Table-19 (IS: 456) and read out the permissible shear stress as

$$k\tau_c > \tau_v$$

Hence the slab is safe

Step: 9

Check for Deflection

$$\left(\frac{L}{d}\right)_{max} = \left(\frac{L}{d}\right)_{Basic} \times k_t \times k_c \times k_f$$

$k_t \rightarrow$ (Fig. No: 4)
$k_c \rightarrow$ (Fig. No: 5)
$k_f \rightarrow$ (Fig. No: 6)
Pg. No: 38, 39

$$\left(\frac{L}{d}\right)_{max} = \left(\frac{L}{d}\right)_{Basic} \times k_t$$

$$\left(\frac{L}{d}\right)_{prov} = xx$$

$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{Provided}$$

Hence the deflection control is satisfied

Step: 10

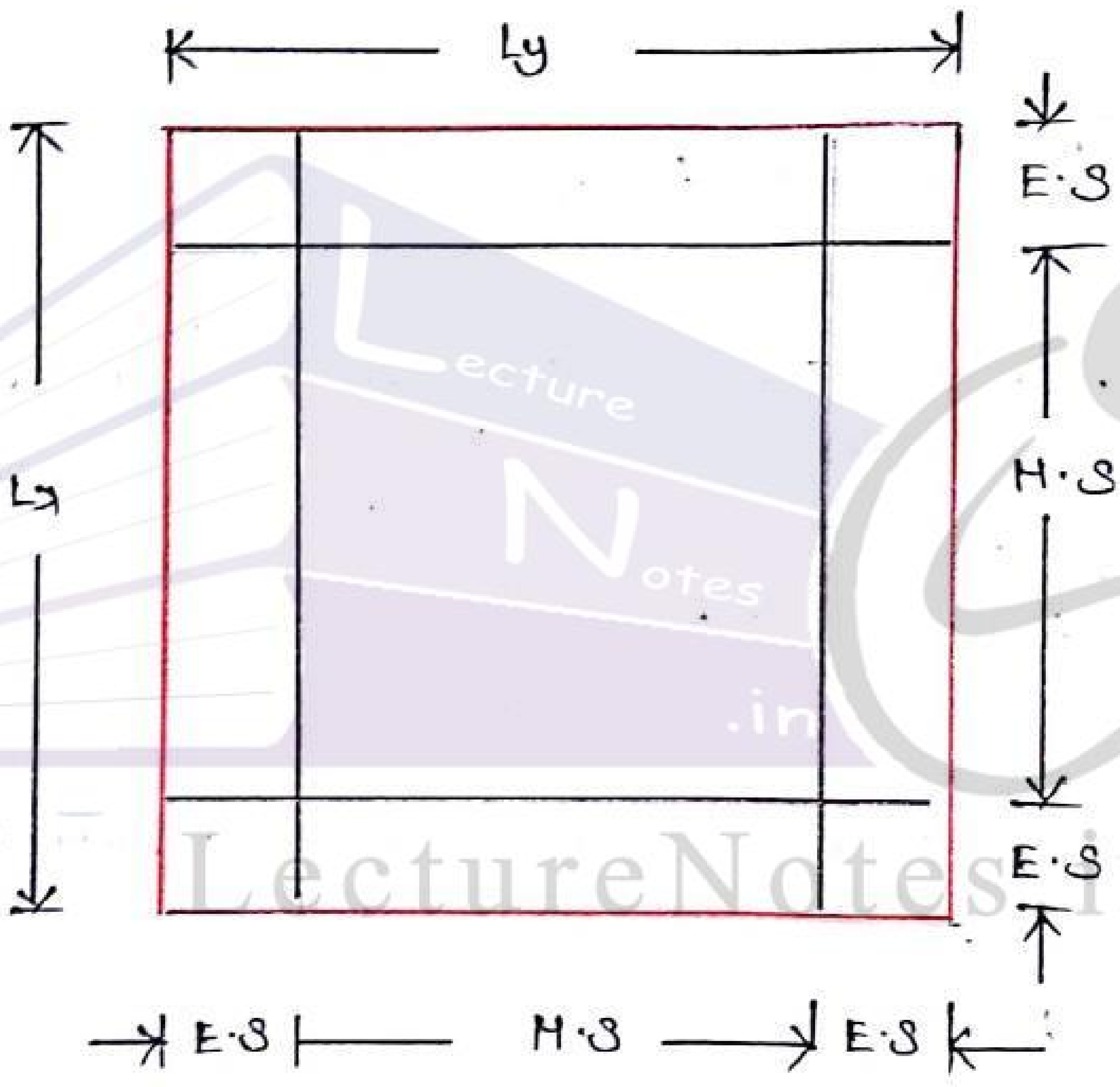
Torsion Reinforcement at Corner

* Torsion reinforcement is provided at corners where the slab

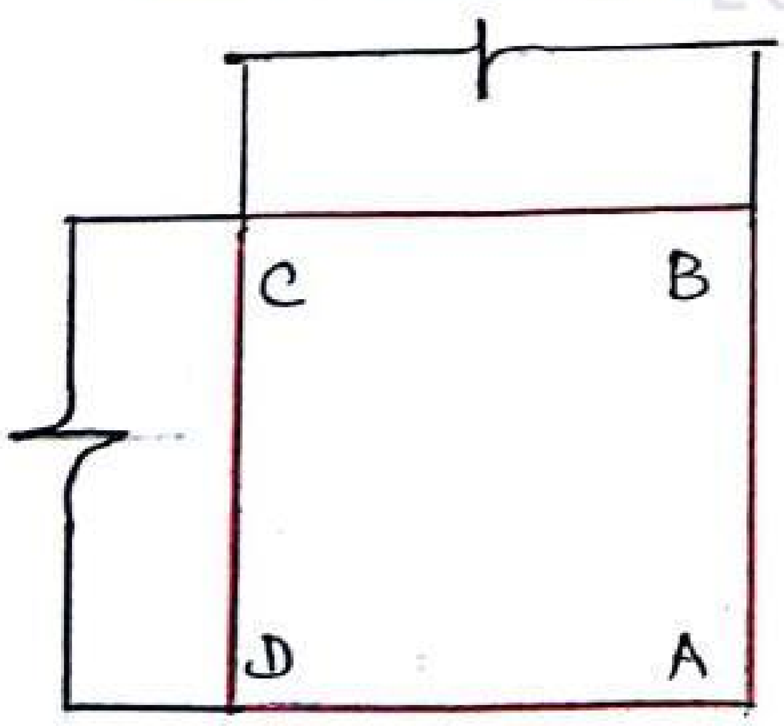
is simply supported on both edges meeting at that corner

* The Reinforcement required for the maximum mid span moment in the slab is provided in each of the four layers in the form of a mesh extending to a minimum distances of one - fifth of the shorter span (l_x)

* Full torsional steel is provided at corner A where the slab is discontinuous on both edges meeting at that corner



MIDDLE AND EDGE STRIPS IN TWO WAY SLABS



PROVISION OF TORSIONAL STEEL IN SLAB

* At Corner B where the slab is discontinuous on only one edge meeting at the corner, 50 percent of Full Torsional steel is provided

* At corner C as the slab is continuous on both Edges meeting at the corner, Torsional steel is not required

* At (A) Corners where slab is discontinous over both the Edges

$$\boxed{A_{st} = \frac{3}{4} \times A_{stx}}$$

* At (B) corner where slab is discontinuous over only c 2 Edge

$$\boxed{A_{st} = \frac{3}{8} \times A_{stx}}$$

* At Corner where slab is continuous over both the Edges

$$\boxed{A_{st} = 0}$$

Prblm.no : 2

Design a Reinforced concrete slab for a room of clear dimension 4m by 5m with discontinuous and simply supported Edges on all the side with corners prevented from Lifting to supported all around on walls of width 300m. The slab has to carry a Live load of 4 KN/m^2 and Floor Finish 1 KN/m^2. Use M-20 Grade concrete and Fe 415 HYSD Bars

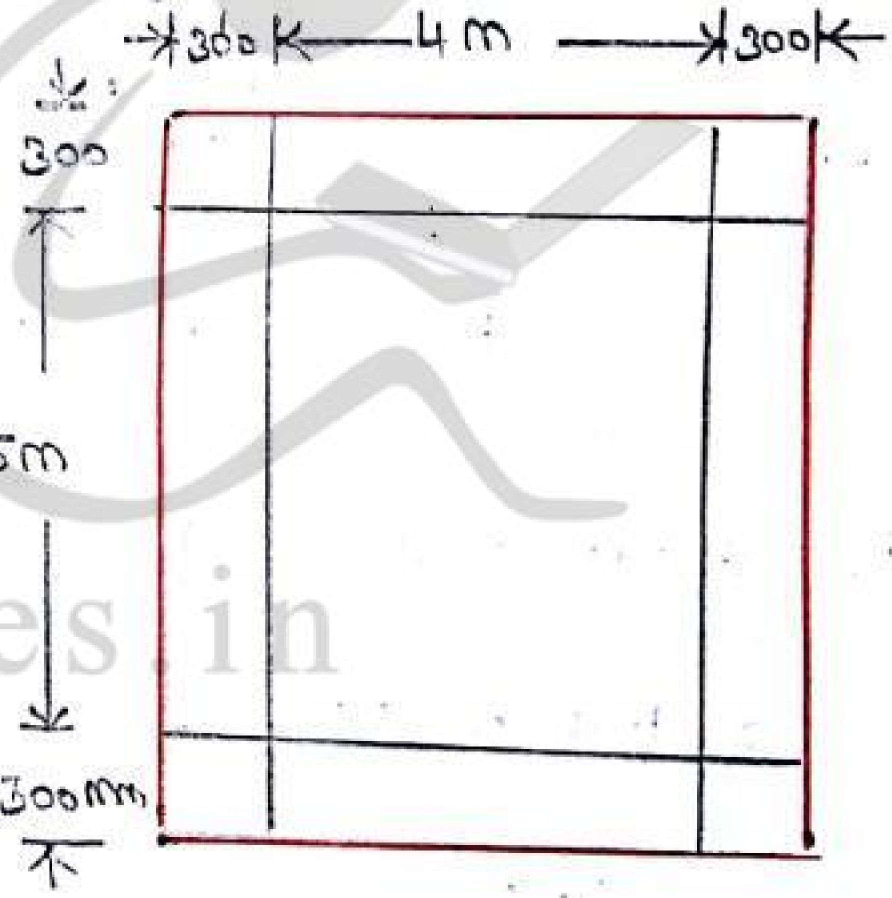
Given data

$l_x = 4m$

$l_y = 5m$

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

$f_y = 415\ N/mm^2$



Step : 1

Type of slab

$$\frac{l_y}{l_x} = \frac{5}{4}$$

$$= 1.25 < 2$$

Hence the slab is designed as Two Way slab

Step : 2

Depth of slab

The span is more than 3.5m, Adopt a span/Depth ratio of 25

(i.e) $l_x > 3.5m$

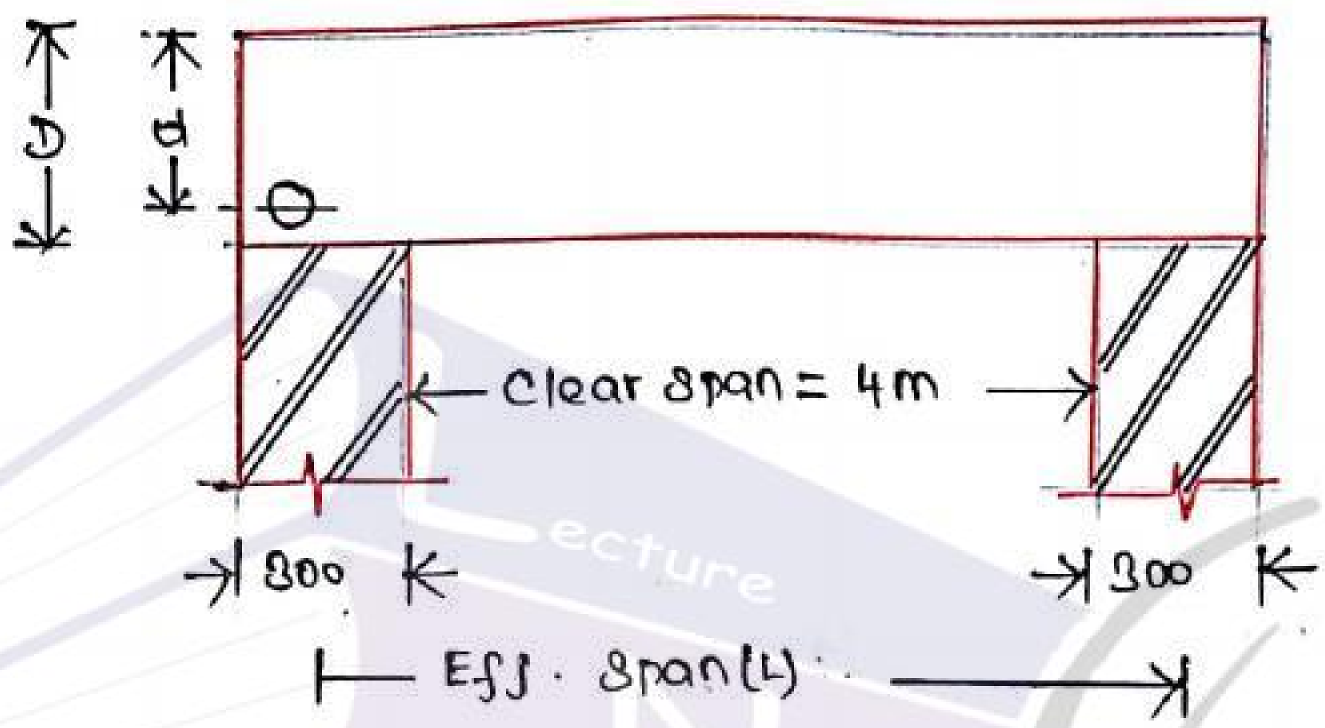
$$\text{Effective depth} = \frac{\text{Span}}{25}$$

$$d = \frac{4000}{25}$$

$$\boxed{d = 160\,mm}$$

$$\text{overall depth} = 160 + 15$$

$$\boxed{D = 175\,mm}$$



Step:3

Effective span (L)

(i) clear span + Effective depth

$= 4 + 0.175$

$= 4.175$

(ii) clear span + centre to centre of support

$= 4 + (0.15 + 0.15)$

$= 4.3\,m$

Adopt

$$\boxed{L = 4.17\,m}$$

Step:4.

Loadings

Consider 1m Width of slab

Self weight of slab = D x b x Unit Wt. of RCC

= 0.175 x 1 x 25

= 4.375 KN/m²

Floor Finish = 1.000 KN/m²

Live load = 4.000 KN/m²

Total Load (ω) = 9.375 KN/m²

Design Ultimate Load (ωu) = PSF x ω

= 1.5 x 9.375

$\boxed{\omega_u = 14.06 \text{ KN/m}^2}$

Step:5

Ultimate B.M and Shear Force

Refer Table-26 (Pg.No: 91) and read out the Moment co-efficient for $\frac{l_y}{l_x} = 1.25$

$\left(\frac{l_y}{l_x}\right)$	α_x
1.2 →	0.072
1.25 →	?
1.3 →	0.079

$$\frac{(1.3 - 1.2)}{(1.3 - 1.25)} = \frac{(0.079 - 0.072)}{(0.079 - x)}$$

$$\alpha_x = 0.075$$

$$\alpha_y = 0.056$$

$$M_{ux} = \alpha_x \cdot w_u \cdot l_x^2$$

$$= 0.075 \times 14.06 \times 4.17^2$$

$$\boxed{M_{ux} = 18.33 \text{ KN}\cdot\text{m}}$$

$$M_{uy} = \alpha_y \cdot w_u \cdot l_y^2$$

$$= 0.056 \times 14.06 \times 4.17^2$$

$$\boxed{M_{uy} = 13.69 \text{ KN}\cdot\text{m}}$$

$$V_u = \frac{w_u \cdot l_x}{2}$$

$$= \frac{14.06 \times 4.17}{2}$$

$$\boxed{V_u = 29.31 \text{ KN}}$$

Step:6

Check for depth

$M_{u_x} = 0.138 f_{ck} b d^2$ (For Fe 415)

$$d = \sqrt{\frac{Mu_x}{0.138 f_{ck} b}}$$

$$= \sqrt{\frac{18.33 \times 10^6}{0.138 \times 20 \times 1000}}$$

$$= 81.49 \text{ mm} < 160 \text{mm}$$

Hence the Effective depth selected is sufficient to resist the design Ultimate Moment

Step: 7

Main Reinforcement

(i) Reinforcement in y - direction

$$Mu_y = 0.87 \times 415 \; Ast \times d \left[1 - \frac{Ast \times 415}{bd \; f_{ck}}\right]$$

$$18.33 \times 10^6 = 0.87 \times 415 \times Ast \times 160 \left[1 - \frac{Ast \times 415}{1000 \times 160 \times 20}\right]$$

$$= 57.76 \times 10^3 \; Ast - 7.49 \; Ast^2$$

$$7.49 Ast^2 - 57.76 \times 10^3 \; Ast + 18.33 \times 10^6 = 0$$

$$\boxed{Ast = 331.6 \text{ mm}^2}$$

Spacing

$$S = \frac{ast}{Ast} \times b$$

$$= \frac{\frac{\pi}{4} \times 10^2}{331.6} \times 1000$$

$$= 236.85 \text{ mm}$$

Hence provide 10mm ϕ bars @ 230mm c/c

(ii) Reinforcement in y-direction

The Reinforcement will be placed above the reinforcement in x-direction.

$$d = 160 - 10$$

$$= 150 \text{ mm}$$

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

$$13.69 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \left[1 - \frac{415 \times A_{st}}{1000 \times 150 \times 20}\right]$$

$$= 54.157 \times 10^3 A_{st} - 7.49 A_{st}^2$$

$$7.49 A_{st}^2 - 54.157 \times 10^3 A_{st} + 13.69 \times 10^6 = 0$$

$$\boxed{A_{st} = 262.29 \text{ mm}^2}$$

Using 10mm diameter

Spacing

$$s = \frac{\frac{\pi}{4} \times 10^2}{262.29} \times 1000$$

$$= 299.4 \text{ mm}$$

Hence provide 10mm ϕ bars @ 280mm c/c

Step : 8

Check for Shear

Consider the short span l_x and unit width of slab, the shear stress is given by

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

$$= \frac{29.31 \times 10^3}{1000 \times 160}$$

$$\boxed{\tau_v = 0.183 \frac{N}{mm^2}}$$

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

$$= \frac{100 \times 331.6}{100 \times 160}$$

$$\boxed{P_t = 0.20 \%}$$

Refer Table - 19 (IS : 456) and read out the permissible shear stress as

P_t		τ_c
0.15	$\rightarrow$	0.28
0.20	$\rightarrow$	?
0.25	$\rightarrow$	0.36

$$\frac{(0.25 - 0.15)}{(0.25 - 0.20)} = \frac{(0.36 - 0.28)}{(0.36 - x)}$$

$$\boxed{\tau_c = 0.32\ N/mm^2}$$

$$K \cdot \tau_c = 1.25 \times 0.32$$

$$= 0.4\ N/mm^2$$

$$K \cdot \tau_c > \tau_v$$

Hence the slab is safe against shear forces.

Step: 9

Check for Deflection

$p_t = 0.20\%$

$$f_s = 0.58 \times f_y \times \frac{Ast\ req}{Ast\ prov}$$

$$= 0.58 \times 415 \times \frac{331.6}{331.6}$$

$$= 240.7 \simeq 240$$

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{basic} \times K_t \times K_c \times K_f$$

$$= 20 \times 1.7 \times 1 \times 1$$

$$= 34$$

$$\left(\frac{L}{d}\right)_{prov} = \left(\frac{4.16 \times 1000}{160}\right)$$

$$= 26$$

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{provided}$$

Hence deflection control is satisfactory

Step: 10

- ## Torsion Reinforcement at corners

Size of Mesh $= \frac{l_x}{5}$

$= \frac{4.16}{5}$

$= 0.832\,m \Rightarrow 832\,mm$

Area of Torsional Reinforcement

$= \frac{3}{4} \times Ast_x$

$= \frac{3}{4} \times 331.6$

$= 248.7\,mm^2$

Using 8mm bars

Spacing

$S = \frac{ast}{Ast} \times b$

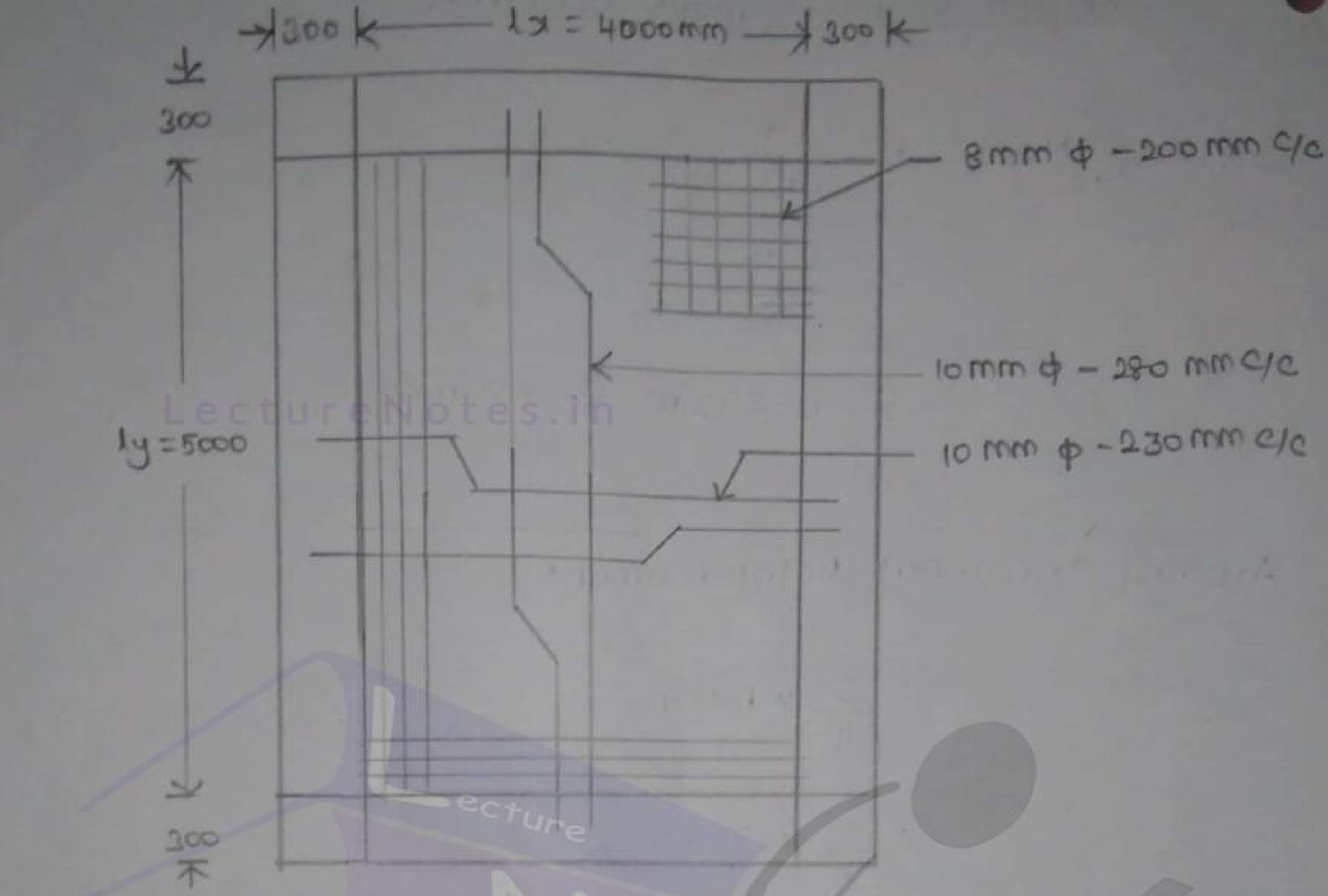
$= \frac{\frac{\pi}{4} \times 8^2}{248.7} \times 1000$

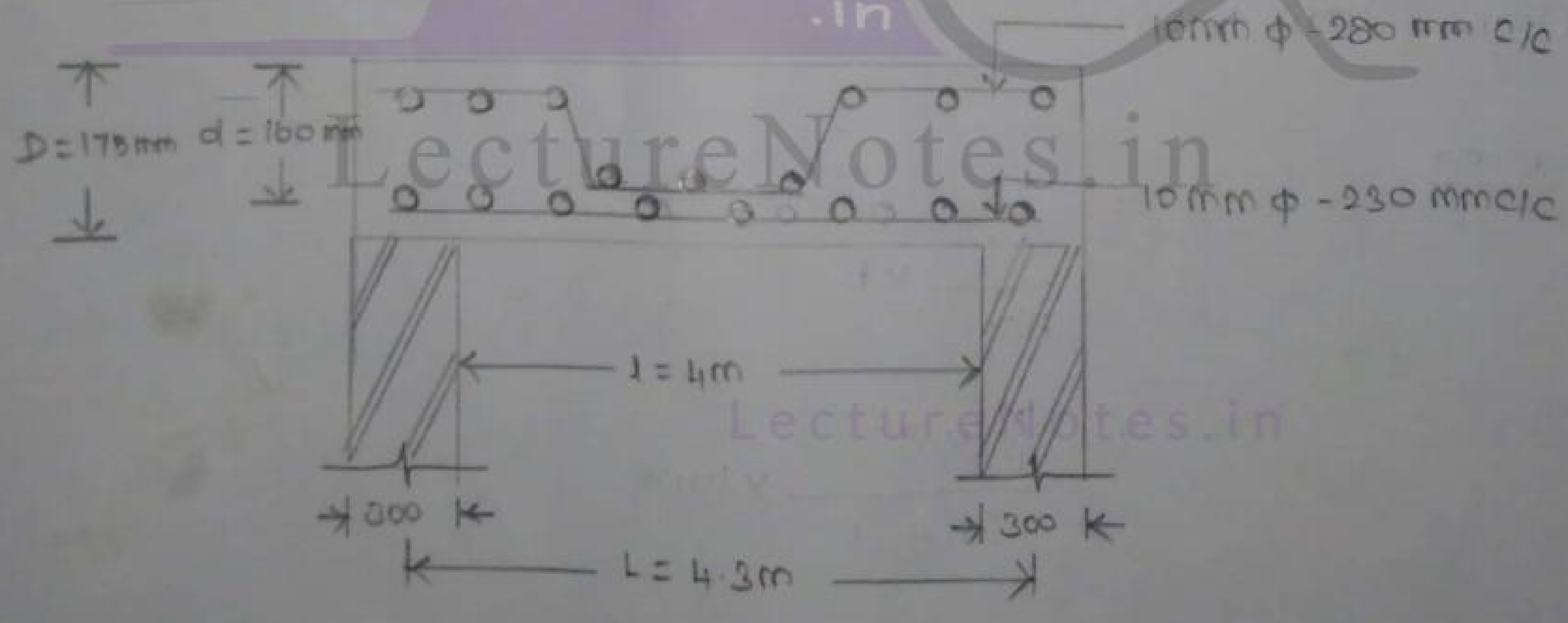
$= 201.11\,mm$

Provide 8mm ϕ bars @ 200 mm c/c spacing

Step: 11

Reinforcement Details in Two Way Slab





Prblm. No :3

Design a Two Way Slab for an office floor to Suit the Following data. Size of office Floor 4m by 6m Edge condition Two Edges discontinuous, Use M20 Grade of concrete and Fe 415 HYSD bars support width = 300mm and Floor finish = 1 KN/m², Live load = 4 KN/m²

Given data

$l_x = 4m$

$l_y = 6m$

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

$f_y = 415\ N/mm^2$

Edge condition = Two Edges discontinuous

Step:1

Depth of Slab

The slab is more than 3.5m, Adopt a span/Depth ratio of 25 (i.e) $l_x > 3.5m$

$$\text{Effective depth} = \frac{\text{Span}}{25}$$

$$= \frac{4000}{25}$$

$$d = 160mm$$

Clear cover = 15mm

overall depth (D) = $160 + 15 + \frac{10}{2}$

$D = 180m$

Step:2

Effective Span (L)

(i) Clear Span + Effective depth

$= 4 + 0.16$

$= 4.16m$

(ii) Clear Span + C/C of Support

$= 4 + (0.15 + 0.15)$

$= 4.2 m$

Adopt

$L = 4.16m$

Step:3

Loadings

Consider 1m width of slab

Self weight of slab $= 0.18 \times 1 \times 25$

$= 4.5 \, kN/m^2$

Floor Finish $= 1.0 \, kN/m^2$

Live load $= 4.0 \, kN/m^2$

Total Load (w) $= 9.5 \, kN/m^2$

Design Ultimate Load $(w_u) = 1.5 \times 9.5$

$= 14.25 \, kN/m^2$

Step: 4

Ultimate B.M and Shear Force

Refer IS: 456-2000
Table-26
Pg. No: 91

$$\frac{l_y}{l_x} = \frac{6}{4} = 1.5$$

(i) Short Span Moment Co-efficient

(-Ve) $\alpha_{x1} = 0.075$

(+Ve) $\alpha_{x2} = 0.056$

(ii) Long Span Moment Co-efficient

(-Ve) $\alpha_{y1} = 0.047$

(+Ve) $\alpha_{y2} = 0.035$

Mu_{x1} (-Ve) $= \alpha_{x1} \cdot w_u \cdot l_x^2 \Rightarrow 0.075 \times 14.25 \times 4.16^2 \Rightarrow 18.49$ kN.m

Mu_{x2} (+Ve) $= \alpha_{x2} \cdot w_u \cdot l_x^2 \Rightarrow 0.056 \times 14.25 \times 4.16^2 \Rightarrow 13.80$ kN.m

Mu_{y1} (-Ve) $= \alpha_{y1} \cdot w_u \cdot l_x^2 \Rightarrow 0.047 \times 14.25 \times 4.16^2 \Rightarrow 11.59$ kN.m

Mu_{y2} (+Ve) $= \alpha_{y2} \cdot w_u \cdot l_x^2 \Rightarrow 0.035 \times 14.25 \times 4.16^2 \Rightarrow 8.63$ kN.m

$$V_u = \frac{w_u \cdot l_x}{2} = \frac{14.25 \times 4.16}{2} = 29.64 \text{ kN}$$

Step: 5

Check for Depth

$$Mu_{lim} = 0.138 f_{ck} b d^2$$

$$d = \sqrt{\frac{18.49 \times 10^6}{0.138 \times 20 \times 1000}} \Rightarrow 81.84\text{mm} > 160\text{mm}$$

Hence the Effective depth Selected is Sufficient.

Step: 6

Main Reinforcement

(i) Short Span

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

$$18.85 \times 10^6 = 0.87 \times 415 \times A_{st} \times 160 \left(1 - \frac{A_{st} \times 415}{1000 \times 160 \times 20}\right)$$

$$18.85 \times 10^6 = 57.76 \times 10^3 A_{st} - 7.49\, A_{st}^2$$

$$7.49\, A_{st}^2 - 57.76 \times 10^3 A_{st} + 18.85 \times 10^6 = 0$$

$$A_{st x1} = 341.47\, mm^2$$

Using 10mm diameter

Spacing

$$s = \frac{a_{st}}{A_{st}} \Rightarrow \frac{\frac{\pi}{4} \times 10^2}{341.47} \times 1000 \Rightarrow 230\, mm$$

(−ve) Moment (Top of Support)

Provide 10mm φ @ 230 mm c/c

(ii) Design (+ve) Moment

Similarly

$$M_{u x 2} = 13.80\ kN \cdot m$$

$$A_{st x2} = 246.81\, mm^2$$

$$s = \frac{\frac{\pi}{4} \times 10^2}{246.81} \times 1000 \Rightarrow \overset{318}{\cancel{384}}\ mm$$

Adopt (s) = 300mm

(+ve) Moment (Centre of Span)

Provide 10mm φ @ 300 mm c/c

(ii) Long Span

(i) Design (-Ve) Moment

$$d = 160 - 10\,mm$$
$$= 150\,mm$$

$$M_{uy1} = 0.87 \times f_y\, A_{st}\, d \left(1 - \frac{A_{st} f_y}{b\, d\, f_{ck}}\right)$$

$$11.59 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \left(1 - \frac{415 \times A_{st}}{1000 \times 150 \times 20}\right)$$

$$= 54.15 \times 10^3 A_{st} - 7.49\, A_{st}^2$$

$$7.49\, A_{st}^2 - 54.15 \times 10^3 A_{st} + 11.59 \times 10^6$$

$$A_{st\,y1} = 220.7\,mm^2$$

Provide 8mm diameter

Spacing

$$S = \frac{\frac{\pi}{4} \times 8^2}{220.7} \times 1000 \Rightarrow 227\,mm$$

(Top of Support) Provide 8mm ϕ @ 227mm c/c

(ii) Design (+Ve) Moment

$$M_{uy2} = 8.62\ kN \cdot m$$

$$A_{st\,y2} = 162\ mm^2$$

$$S = \frac{\frac{\pi}{4} \times 8^2}{162} \Rightarrow 300\,mm$$

(Centre of Span) Provide 8mm ϕ @ 300 mm c/c

Location	Ast (Required)	Spacing
(i) Short Span		
(a) -Ve (Top of Support)	341.47 mm^2	230 mm c/c
(b) +Ve (centre of span)	246.81 mm^2	300 mm c/c
(ii) Long Span		
(a) -Ve (Top of Support)	220.7 mm^2	227 mm c/c
(b) +Ve (centre of Span)	162 mm^2	300 mm c/c

Step: 7

Check for Shear

$$\tau_v = \frac{V_u}{bd} = \frac{29.64 \times 10^3}{1000 \times 160} = 0.18\ N/mm^2$$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 341.47}{1000 \times 160} = 0.2\,\%$$

P_t	τ_c
0.15 →	0.28
0.20 →	?
0.25 →	0.36

$$\frac{(0.25-0.15)}{(0.25-0.20)} = \frac{(0.36-0.28)}{(0.36-x)}$$

$\tau_c = 0.32\ N/mm^2$

$K \cdot \tau_c = 1.25 \times 0.32 \Rightarrow 0.41 > \tau_v$

Hence OK

Pg. No: 72
IS: 456-2000

Step: 8

Check for Deflection Criteria

$$\left(\frac{L}{d}\right)_{Max} = K_t \times K_c \times K_f$$

$= 20 \times 1.7 \times 1 \times 1$

$\left(\frac{L}{d}\right)_{Max} = 34$

$\left(\frac{L}{d}\right)_{Prov} = \left(\frac{4.16 \times 1000}{160}\right) \Rightarrow 26$

$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{Prov}$

Hence deflection control is satisfactory

$f_s = 0.58 \times f_y \times \frac{Ast_{req}}{Ast_{pro}}$

$= 0.58 \times 415 \times \frac{241.47}{241.47}$

$f_s = 240.7 \simeq 240$

Step: 9

Torsion Reinforcement at Corners

Size of Mesh $= \frac{l_x}{5} \Rightarrow \frac{4.16}{5} \Rightarrow 832mm$

Area of Torsional Reinforcement

$= \frac{3}{4} \times Ast_{x1}$

$= \frac{3}{4} \times 241.47$

$= 256.1$

Spacing

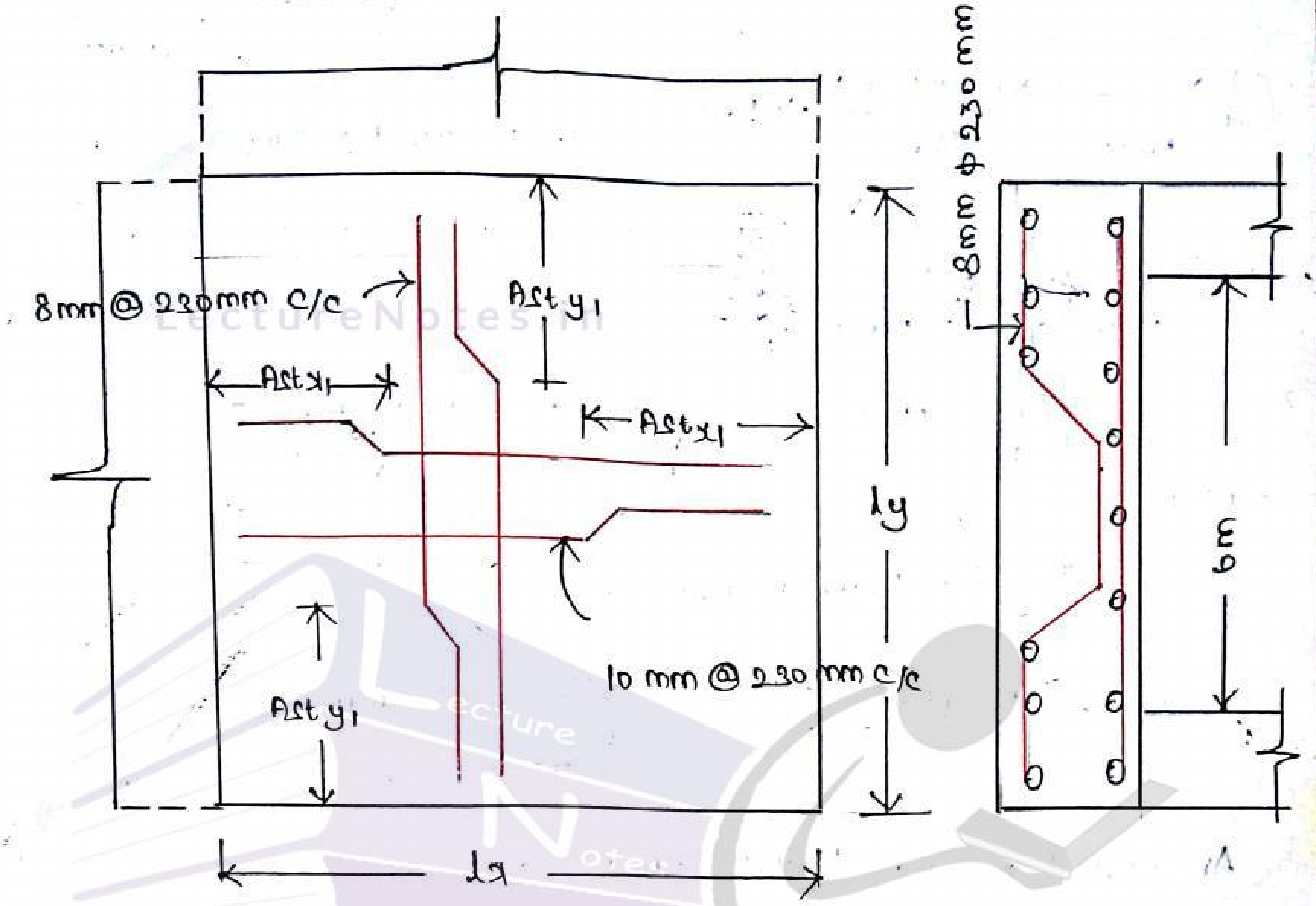
$S = \frac{\frac{\pi}{4} \times 8^2}{256.1} \times 1000$

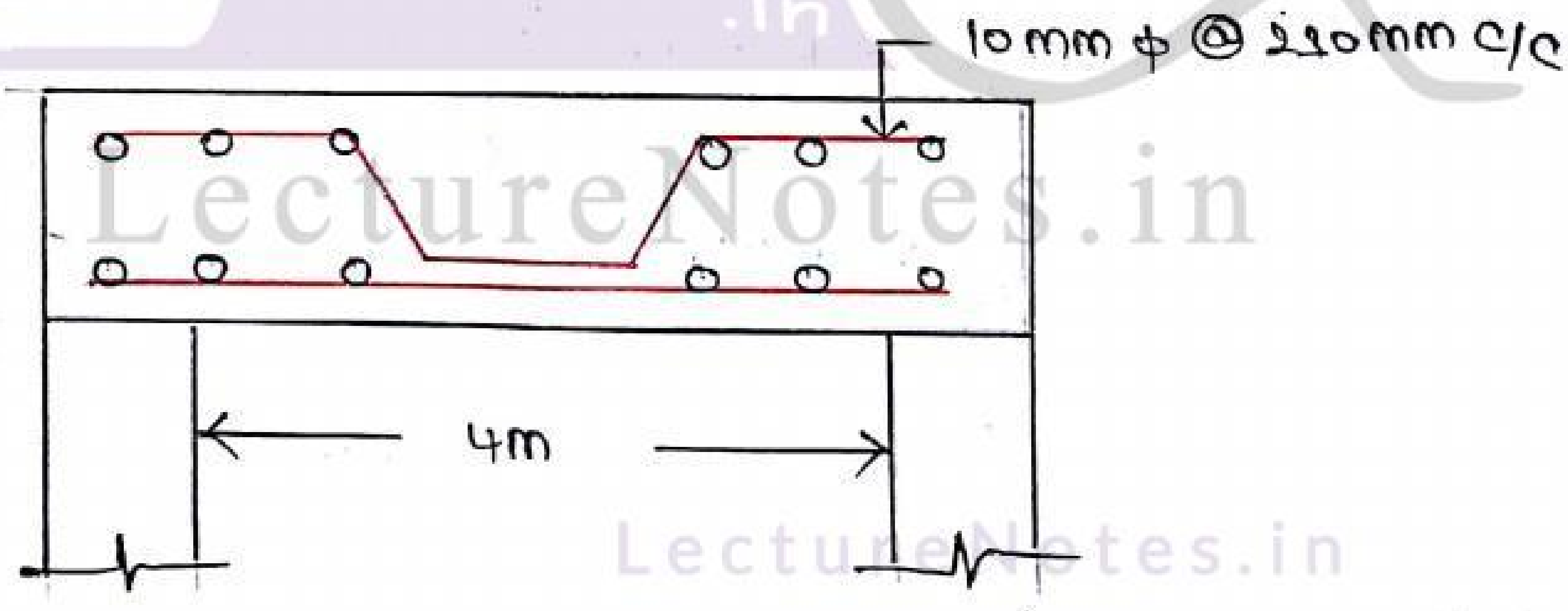
$= 200mm$

Provide 8mm ϕ @ 200mm c/c

Step: 10

REINFORCEMENT DETAIL IN TWO WAY SLAB





DESIGN OF CONTINUOUS SLAB

* In the case of T-beam and slab floors, the slab is continuous over a beams spaced at regular intervals of 2.5m to 3.5m
* Continuous slab are designed similar to that of continuous beam using moment and shear co-efficient
* Continuous slab are subjected to Negative moments at supports and to positive moments at Mid span.
* The depth of the slab is based on the basic span depth ratio of 26 and Maximum 20 recommended in the IS code
* The design of continuous slabs when the Moment and shear co-efficient specified in table-12 and Table-13 (Pg.No:36) the IS-code 456-2000 are used in design
* It may be Noted that if all supports are equal maximum moment is at support next to End support

$$(-ve)\ M_u = \frac{w_d l^2}{10} + \frac{w_L l^2}{9}$$

$$(+ve)\ M_u = \frac{w_d l^2}{12} + \frac{w_L l^2}{10}$$

V_u is at outer of

$$V_u = (0.6 w_d + 0.6 w_L) L$$

Prblm. No: 4

Design a continuous R.C slab for a class room 7m wide and 4m Long. The Roof is to be supported on R.C.C beams spaced at 3.5m Intervals. The Width of beam should be kept 230 mm The Imposed load is 3 kN/m² and Finishing Load expected in 1 kN/m². Use M20 Concrete and Fe 415 Steel.

Given data

$l_x = 7m$, $l_y = 14m$

$L = 3.5m$, $L.L = 3 \ KN/m^2$

$F.F = 1 \ KN/m^2$

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

Step: 1

Depth of slab

Effective depth $= \frac{Span}{30} \Rightarrow \frac{3500}{30} \Rightarrow 116.67mm$

$d = 120 mm$

$D = 120 + 20 + \frac{10}{2}$

$= 145mm$

Step: 2

Effective Span (L)

(i) Width of span $= 230mm$

Clear span $= 3500 - 230$

$= 3270 \ mm$

$\frac{1}{12}$th of clear span greater than width of support

$\frac{L}{12th} >$ Width of Support

Effective Span $=$ Clear Span $+$ Effective depth

$= 3270 + 120$

$= 3390$

$L = 3.39m$

Step: 3

Loadings

Self weight of slab $= 0.15 \times 1 \times 25$

$= 3.75\ kN/m^2$

Floor Finish $= 1.00\ kN/m^2$

Dead Load $(g) = 4.75\ kN/m^2$

Live load $(q) = 3.00\ kN/m^2$

Step: 4

Moment and Shear Forces

Refer to Table-12 and Table-13 of IS: 456-2000 Maximum moment occurs at Support next to the End Support

$$(-ve)\ M_u = 1.5\left[\frac{gL^2}{10} + \frac{qL^2}{9}\right]$$

$$= 1.5\left[\frac{4.75 \times 2.39^2}{10} + \frac{3 \times 2.39^2}{9}\right]$$

$$= 12.93\ kN \cdot m$$

$$(+ve)\ M_u = 1.5\left[\frac{gL^2}{12} + \frac{qL^2}{9}\right]$$

$$= 1.5\left[\frac{4.75 \times 2.80^2}{12} + \frac{3 \times 2.39}{10}\right]$$

$$= 11.99\ kN \cdot m$$

Maximum shear Force occurs at outer side of the support next to End support

$$V_u = 0.6\,(w_d + w_L)\,L$$

$$= 0.6(4.75+3) \times 2.29$$

$$V_u = 1.5 \times 15.76$$

$$V_u = 23.64 \text{ kN}$$

Step: 5

Check for Depth

$$d_1 = \sqrt{\frac{M_u}{0.138 \times 20 \times 1000}} \Rightarrow \sqrt{\frac{13.93 \times 10^6}{0.138 \times 20 \times 1000}} \Rightarrow 71.04\text{mm}$$

$d_1 < d$

Hence OK

Step: 6

Design of Main Reinforcement

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

$$13.93 \times 10^6 = 0.87 \times 415 \times A_{st} \times 120 \left(1 - \frac{A_{st} \times 415}{1000 \times 120 \times 20}\right)$$

$$13.93 \times 10^6 = 43.32 \times 10^3 A_{st} - 7.49 A_{st}^2$$

$$A_{st} = 341.45 \text{ mm}^2$$

Using 10 mm bars

$$s = \frac{\frac{\pi}{4} \times 10^2}{341.45} \times 1000$$

$$s = 230 \text{ mm}$$

Provide 10 mm ϕ at 225 mm c/c @ supports

Step: 7

Check for Shear

$$\tau_v = \frac{V_u}{bd} \Rightarrow \frac{23.64 \times 10^3}{10^3 \times 120} = 0.19 \text{ N/mm}^2$$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 249.06}{1000 \times 120} = 0.29\%$$

$\tau_c = 0.28 \ N/mm^2$

Since the slab thickness is less than 150mm

$$k\tau_c = 1.20 \times 0.28$$
$$= 0.479 > \tau_v$$

Hence the slab is safe against shear failures

Step:8

Check for Deflection control

$$\left(\frac{L}{d}\right)_{prov} = \frac{3.39 \times 1000}{120} = 28.25$$

$$\left(\frac{L}{d}\right)_{Max} = \left(\frac{L}{d}\right)_{basic} \times K_t$$
$$= 26 \times 1.55 = 40$$

$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{provide}$$

Hence the slab is safe against deflection

Step:

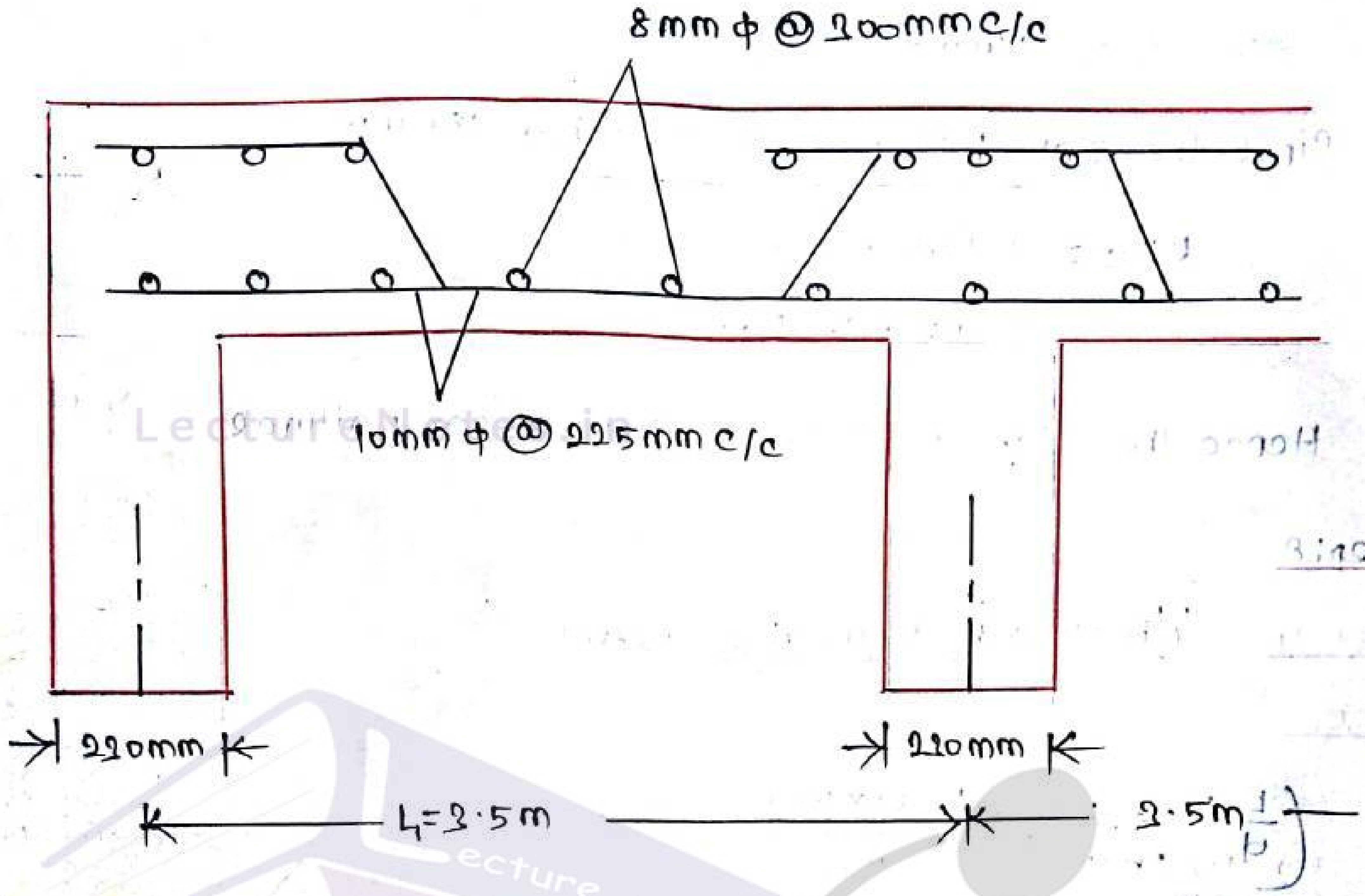
Distribution steel

$$A_{st} = \frac{0.12 \times bd}{100} \Rightarrow \frac{0.12 \times 1000 \times 145}{100}$$
$$= 174 \ mm^2$$

Use 8mm ϕ

$$S = \frac{\frac{\pi}{4} \times 8^2}{174} \times 1000 \Rightarrow 290 \ mm$$

Provide 8mm φ @ 290 mm c/c



REINFORCEMENT DETAIL IN ONE WAY CONTINUOUS SLAB

DESIGN OF CANTILEVER SLAB

Introduction

* Common example of Cantilever slabs are chajjas and balcony slabs. They may be overhanging portions of Interior slabs
* The slab are generally designed as one way slabs as a cantilever fixed (or) Continuous at the supports
* Moment is maximum at fixed / continuous end
* At Free End the moment reduces to zero
* Hence the thickness of Cantilever slab may be reduced gradually towards free End.
* Minimum thickness 75mm to 150mm maximum
* The depth is selected based on span/depth ratio of 7 recommended in IS 456 Code
* For uniformly distributed load $M_u = \frac{w_u L^2}{2}$ and $V_u = w_u L$
* Main bars determined are to be provided at Top
* There should be check for Anchorage length of Main bars

Prblm. No: 5

Design a cantilever slab projecting 2.1m from the support using M-20 Grade of concrete and Fe 415 Grade of steel. Assume Live load (Residential buildings = 2 KN/m²)

Step: 1

Depth of slab

$$\text{Effective depth} = \frac{\text{Span}}{10}$$

$$= \frac{2100}{10} \Rightarrow 210$$

$$d = 210 \text{ mm}$$

$$D = 210 + 15 + \frac{10}{2}$$

$$D = 230 \text{ mm}$$

Maximum depth of 230mm at support is gradually reduced to 120mm at Free End.

Step:2

Loadings

$$\text{Self weight of slab} = \frac{0.23 + 0.12}{2} \times 1 \times 25$$

$$= 4.37 \text{ KN/m}^2$$

$$\text{Floor Finish} = 1.00 \text{ KN/m}^2$$

$$\text{Live load} = 2.00 \text{ KN/m}^2$$

$$\text{Total Working load } (w) = 7.37 \text{ KN/m}^2$$

$$\text{Design load } (w_u) = 1.5 \times 7.37 \Rightarrow 11.05 \text{ KN/m}^2$$

Step:3

Design Moment and Shear

$$M_u = \frac{w_u l^2}{2} \Rightarrow \frac{11.05 \times 2.1^2}{2} \Rightarrow 24.36 \text{ KN.m}$$

$$V_u = w_u L \Rightarrow 11.05 \times 2.1 \Rightarrow 23.20 \text{ KN}$$

Step:4

Check for depth

$$d_1 = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$$

$$24.961 A_{st}^2 - 198577.5 A_{st} + 35 \times 10^6$$

$$A_{st} = 180 \text{ mm}$$

$$A_{st\,min} = \left(\frac{0.85 b_w d}{f_y}\right)$$

$$= \left(\frac{0.85 \times 300 \times 550}{415}\right)$$

$$A_{st\,min} = 338 \text{ mm}^2$$

$$A_{st} = 180 \text{ mm} < A_{st\,min} = 338 \text{ mm}^2.$$

$$\boxed{A_{st\,min} = 338 \text{ mm}^2}$$

No. of bar centre of span (sagging) ↳ bottom face $\Big\}$ $(n) = \frac{A_{st}}{a_{st}}$

$$= \frac{338}{\frac{\pi}{4} \times 16^2}$$

$$= 1.68 \simeq 2$$

$$n = 2$$

Actual area of steel $\Big\}$ $= n \times \frac{\pi}{4} (d)^2$

$$= 2 \times \frac{\pi}{4} (16)^2$$

$$\boxed{A_{st} = 402 \text{ mm}^2}$$

Provide 2 bars of 16 mm dia at the bottom face $(A_{st} = 402 \text{ mm}^2)$

(x) Side face reinforcement

Area of Tension reinforcement $\Big\}$ $= \frac{0.1 \times b_w \times D}{100}$

Refer IS: 456-2000
Pg. no: 48
Clause: 26.5.1.7

$$= \frac{0.1 \times 300 \times 600}{100}$$

$$A_{st} = 180 \text{ mm}^2$$

No. of bars (Side face reinforcement) $\Big\}$ $n = \frac{A_{st}}{a_{st}}$

$$= \frac{180}{\frac{\pi}{4} \times 8^2}$$

$$= 3.58 \simeq 4 \text{ No's}$$

$$n = 4$$

Provide 8mm dia bars 4 no's Two on each face as horizontal reinforcement

(XI) Shear Reinforcement

$$\tau_{ve} = \frac{V_e}{b_w d} = \frac{173 \times 10^3}{300 \times 550} = 1.05 \text{ N/mm}^2$$

Percentage of tension reinforcement

$$P_t = \frac{100 A_{st}}{b_w d} = \frac{100 \times 628}{300 \times 550} = 0.38$$

Refer IS: 456-2000 Table-19

$$\tau_c = 0.42 \text{ N/mm}^2$$

$$\tau_c = 0.42 < \tau_{ve} = 1.05 \text{ N/mm}^2$$

$$\tau_c < \tau_{ve}$$

Shear reinforcement are required

Step: 9

- Check for Deflection Control

$$\left(\frac{L}{d}\right)_{prov} = \left(\frac{2100}{210}\right) = 10$$

$P_t = 0.15$ $\qquad f_s = 0.58 \times 415 \times 1$

$K_t = 2$ $\qquad = 240$

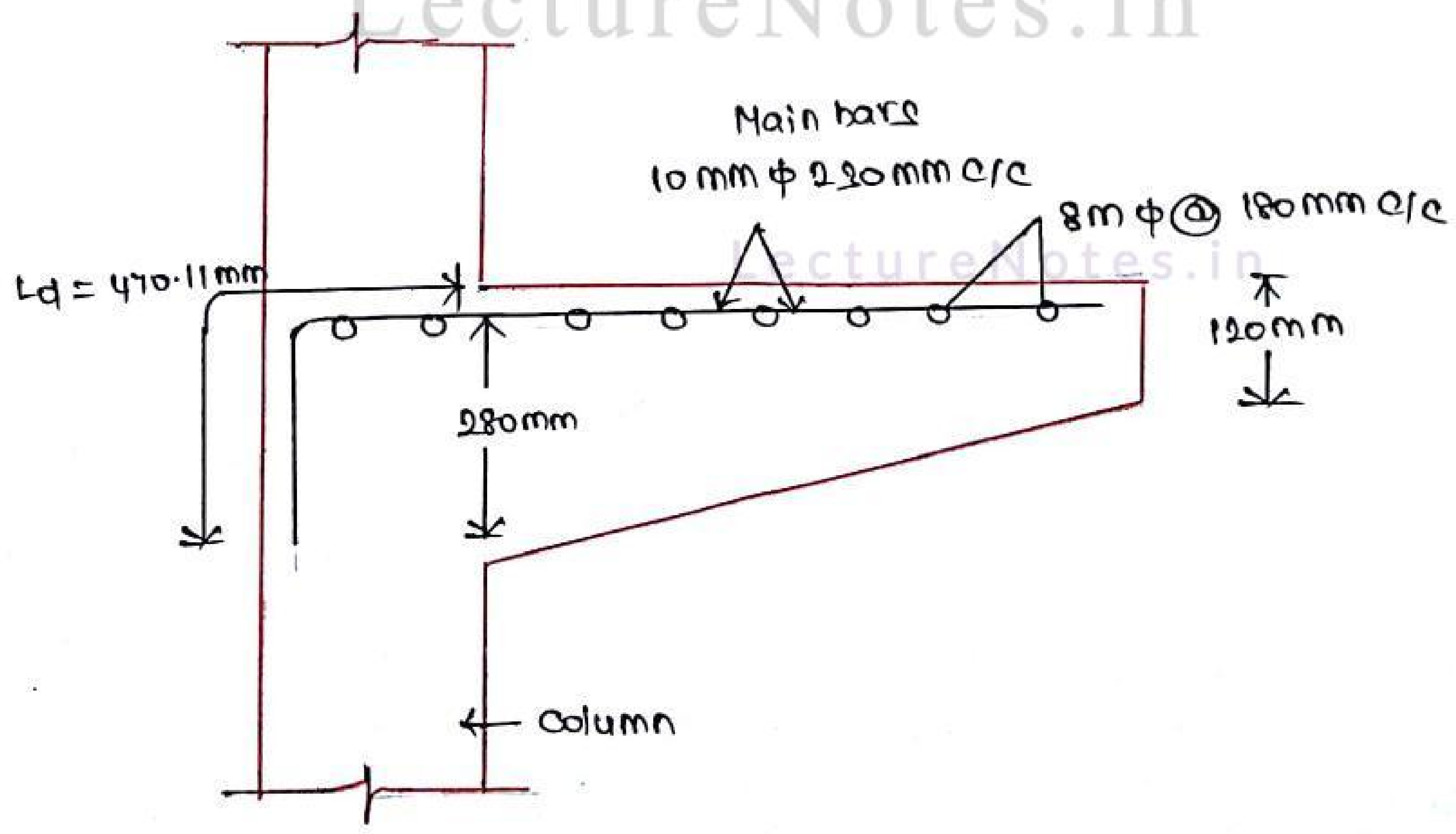
$$\left(\frac{L}{d}\right)_{Max} = 7 \times 2 \Rightarrow 14$$

$$\left(\frac{L}{d}\right)_{Max} > \left(\frac{L}{d}\right)_{provided}$$

Hence the cantilever slab satisfied the deflection control

Step: 10

Detail of Reinforcement in cantilever Slab



UNIT-3

DESIGN OF COLUMNS BY LSM

Introduction

* Column is a very Important component in a Structure
* Column is a Structural member of RCC Frame Structural building
* Column is a vertical Structural member
* Column pure compression member
* Normally columns carry Heavy compressive Loads
* We Need to build strong column, otherwise failure will occur
* It transmit the Load from
    - Ceiling / Roof Slab
    - Beam

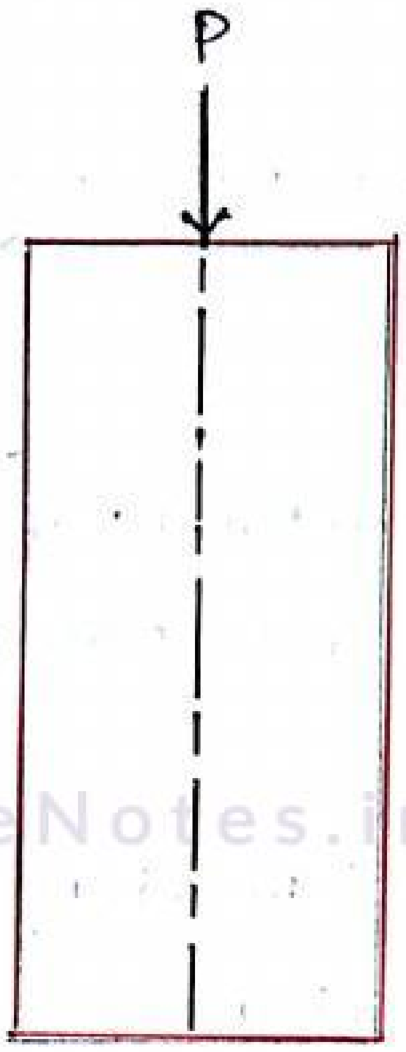
  Including its self weight to the Foundation
* These are used to transfer the load of Super Structure to the Foundation Safely
* Mainly columns, Struts and pedastal are used as compression members in building, Bridges, Supporting system of Water Tank and Factories and many more such Structures

Refer IS:456-2000
Cl·25·1·1, Pg·No: 41

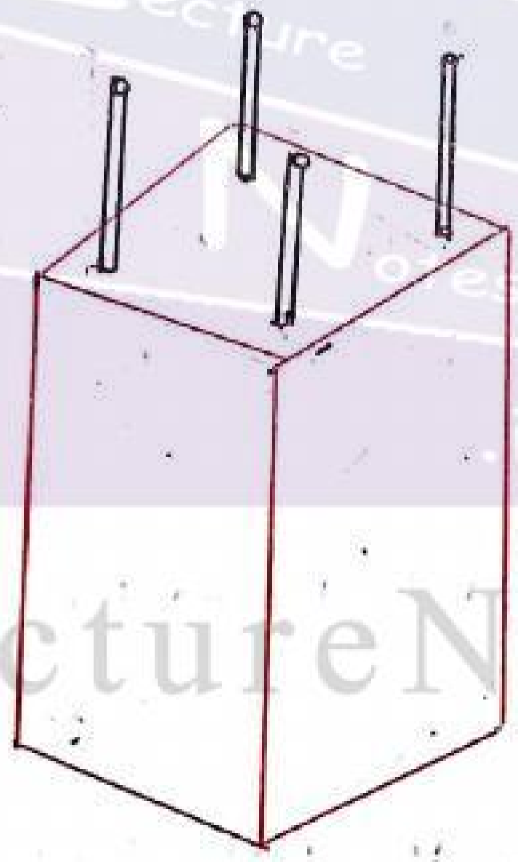
Column

A column is defined as a vertical compression member which is mainly Subjected to axial load and the Effective length of which exceed three times its least lateral dimension

$$\boxed{\frac{\text{Length}}{\text{least lateral dimension}} > 3}$$



Pedastal

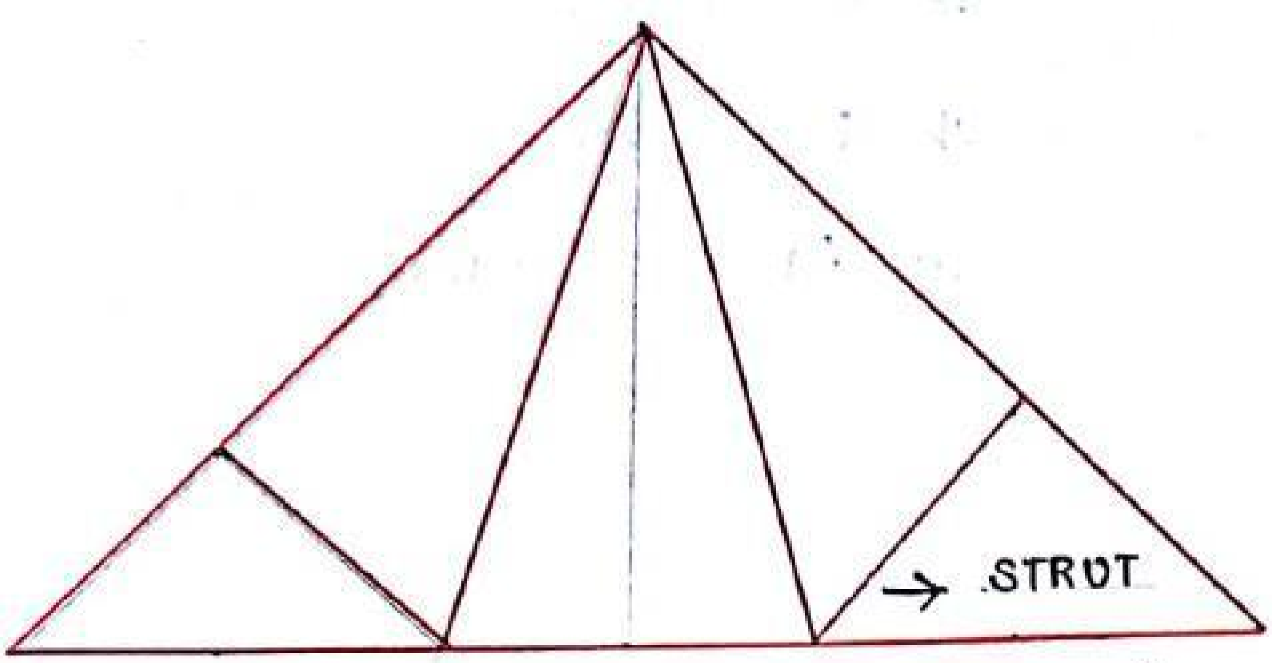


The compression member which Effective length is Less than three times its Least lateral dimension is called a pedestal

$$\boxed{\frac{\text{Length}}{\text{Least lateral dimension}} < 3}$$

Strut

•



TRUSS

The Compression member which is Inclined or horizontal and is Subjected to Axial Load is called as Strut

Strut are used in Truss

$$\boxed{\frac{\text{Length}}{\text{Least lateral dimension}} > 3}$$

Classification of Column

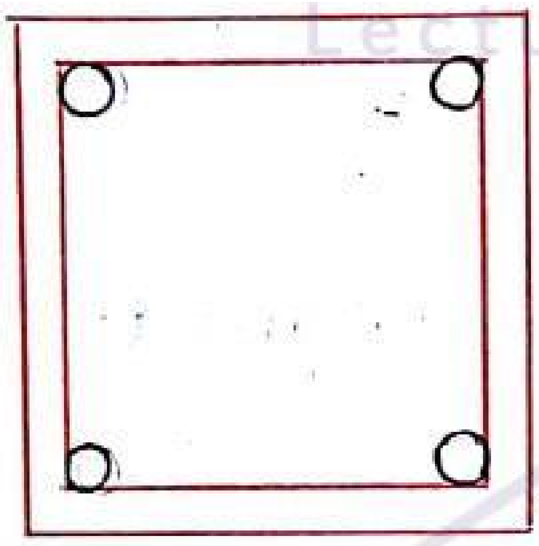
Column are classified based on different criteria such as

* Based on the Type of Shapes
* Based on the Type of Reinforcement
* Based on the Type of Loading
* Based on Slenderness ratio
* Material of Construction

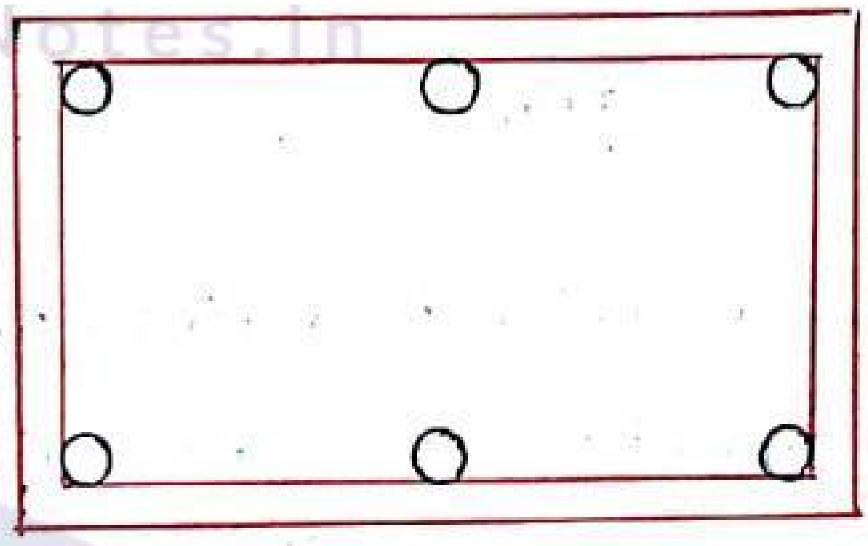
Based on the Type of Shapes

Column with Longitudinal Steel Surrounded by lateral ties or Spiral

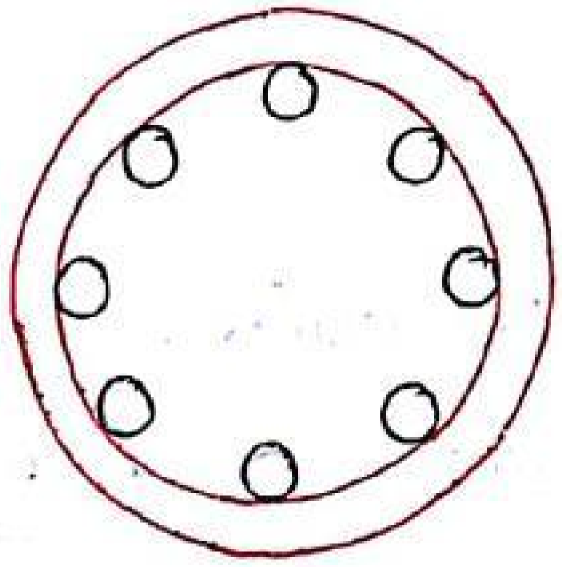
* Square
* Rectangular
* Circular
* Pentagonal
* Hexagonal
* Octagonal
* T-Shape
* L-Shape
* Arch column



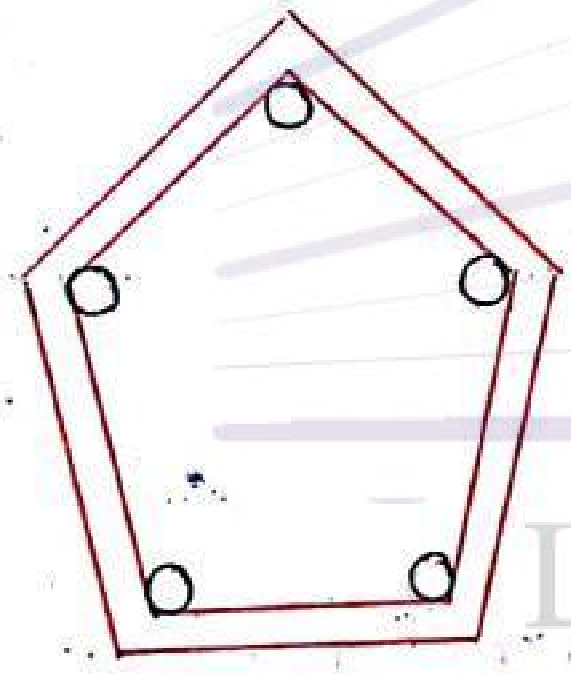
SQUARE



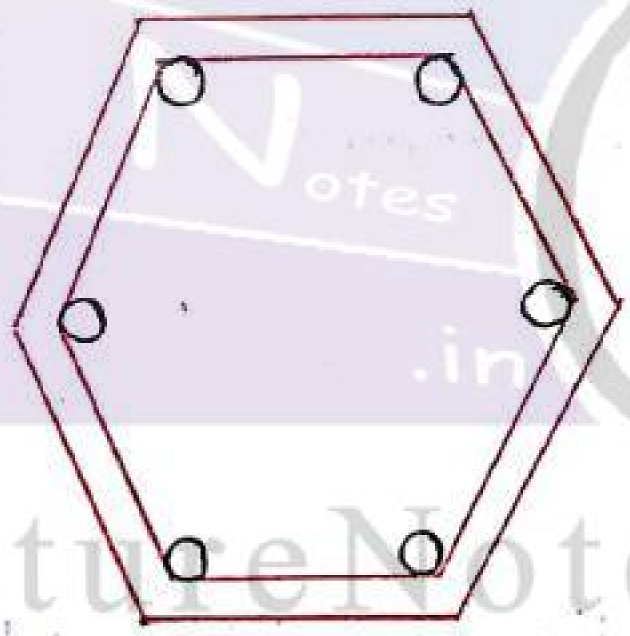
RECTANGULAR



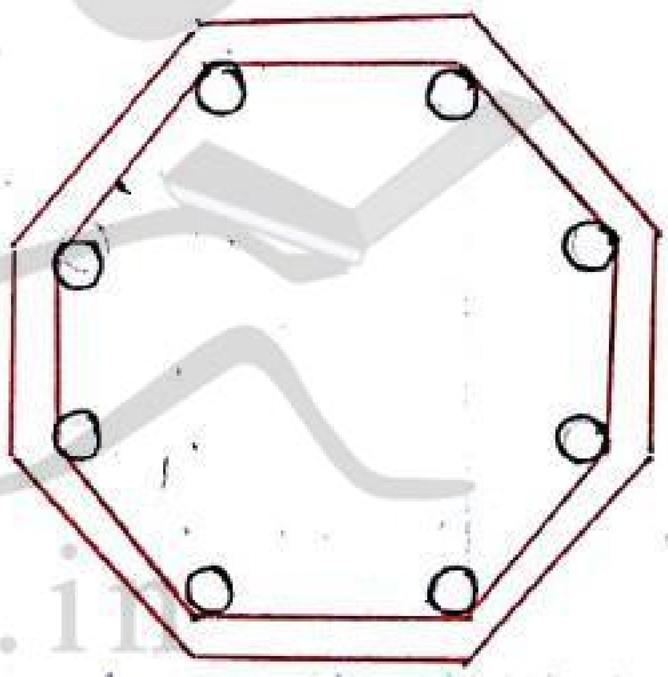
CIRCULAR



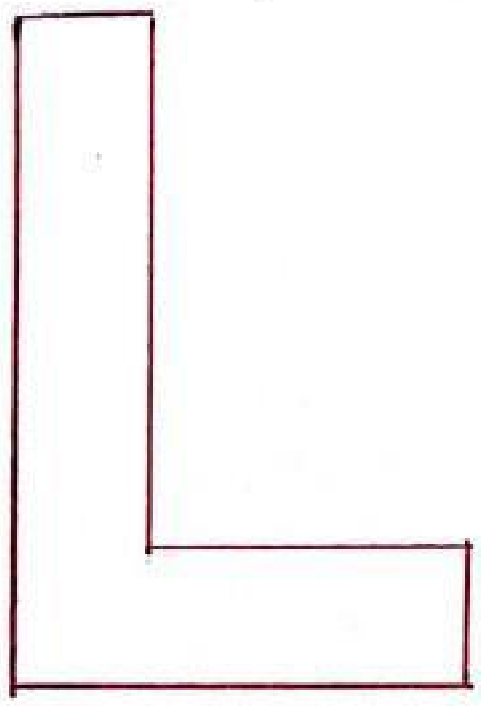
PENTAGONAL



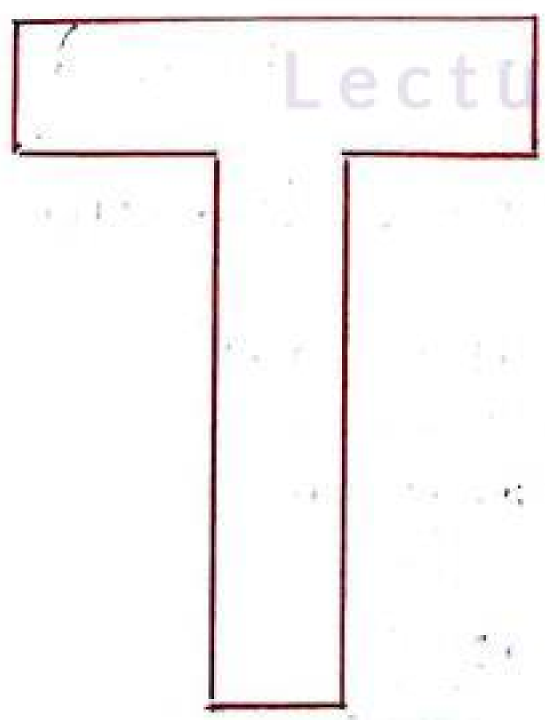
HEXAGONAL



OCTAGONAL



L-COLUMN



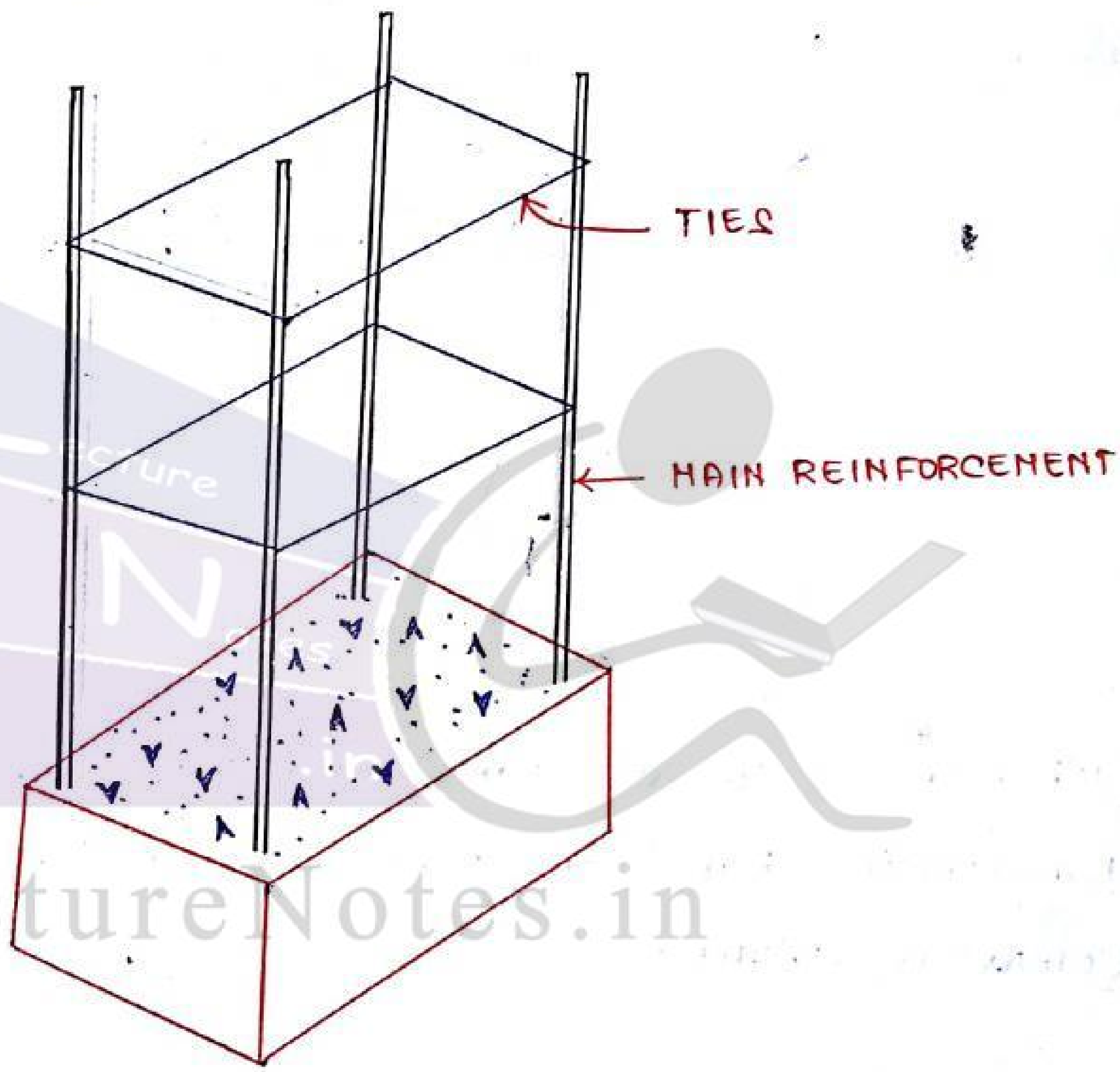
T-SECTION

ARCH COLUMN

Based on Type of Lateral Reinforcement

- Tied Column
- Spiral Column
- Composite Column
- Infilled Column

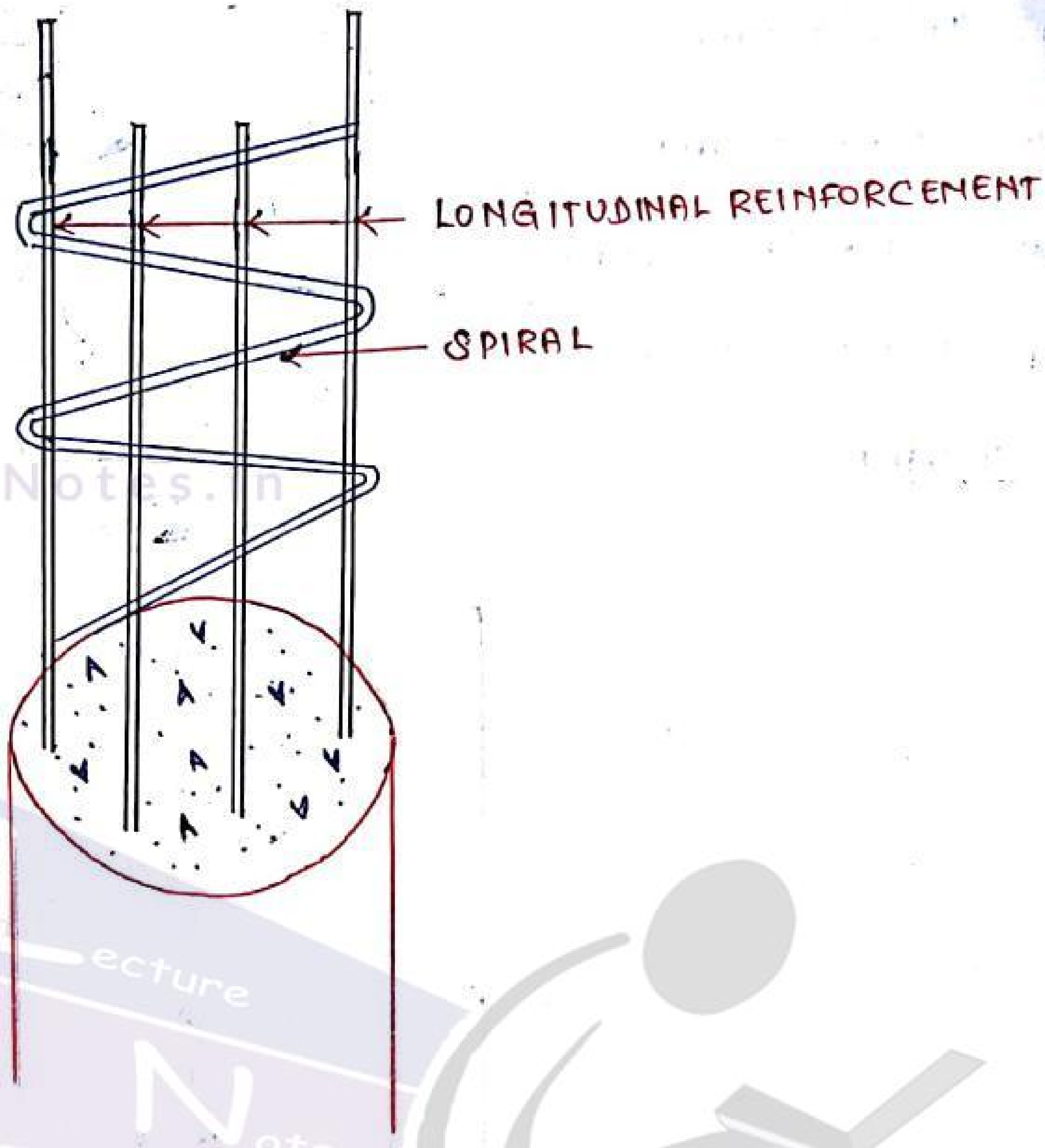
Tied Column



In which the Main Longitudinal bars are confined within closely Spaced lateral Ties

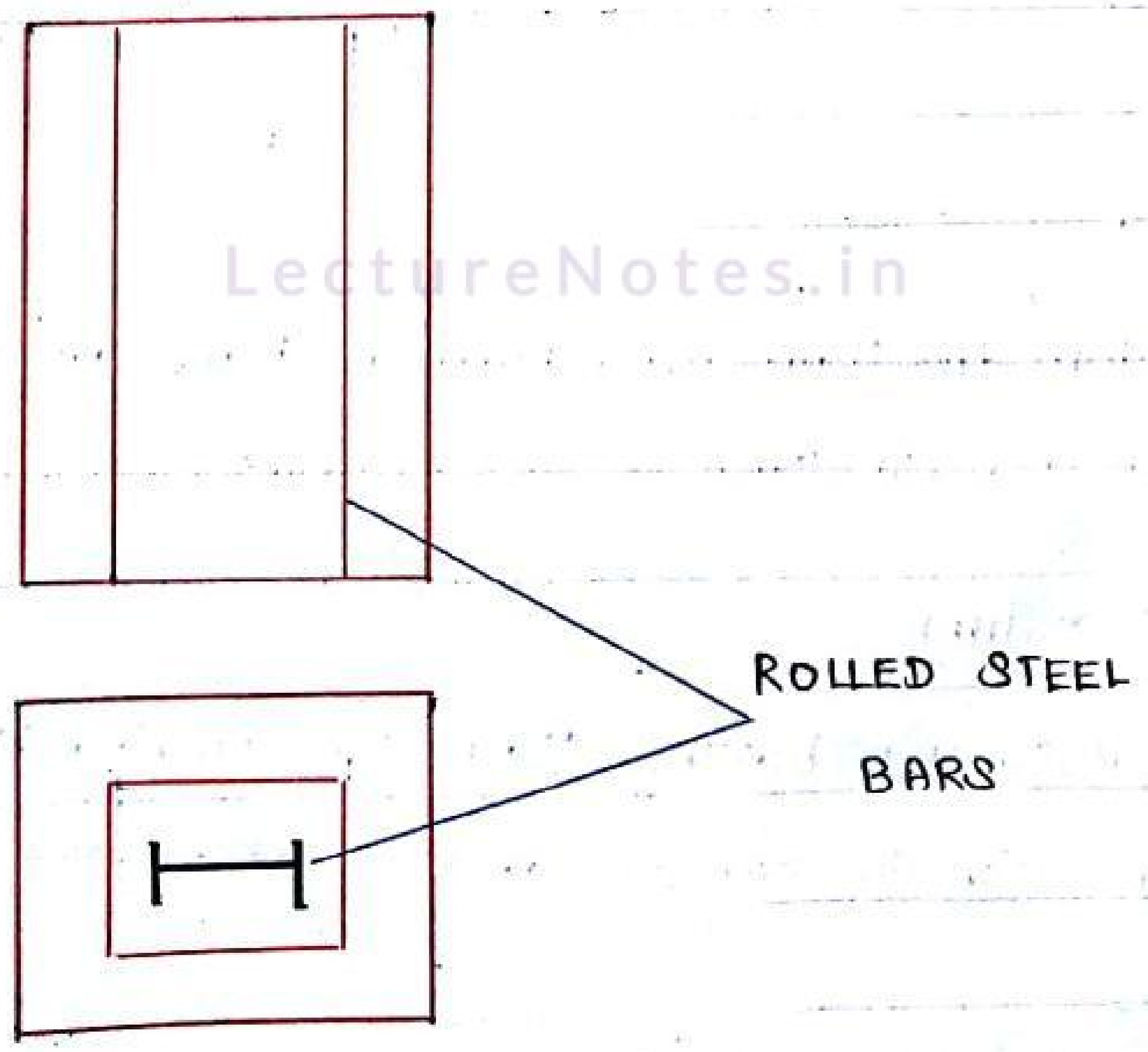
Spiral Column

Spiral Column having Main longitudinal reinforcement enclosed within closely Spaced and Continuously wound Spiral reinforcement



When a Continous bar or heavy wire is wrapped around the Longitudinal bar in the form of a helical spiral

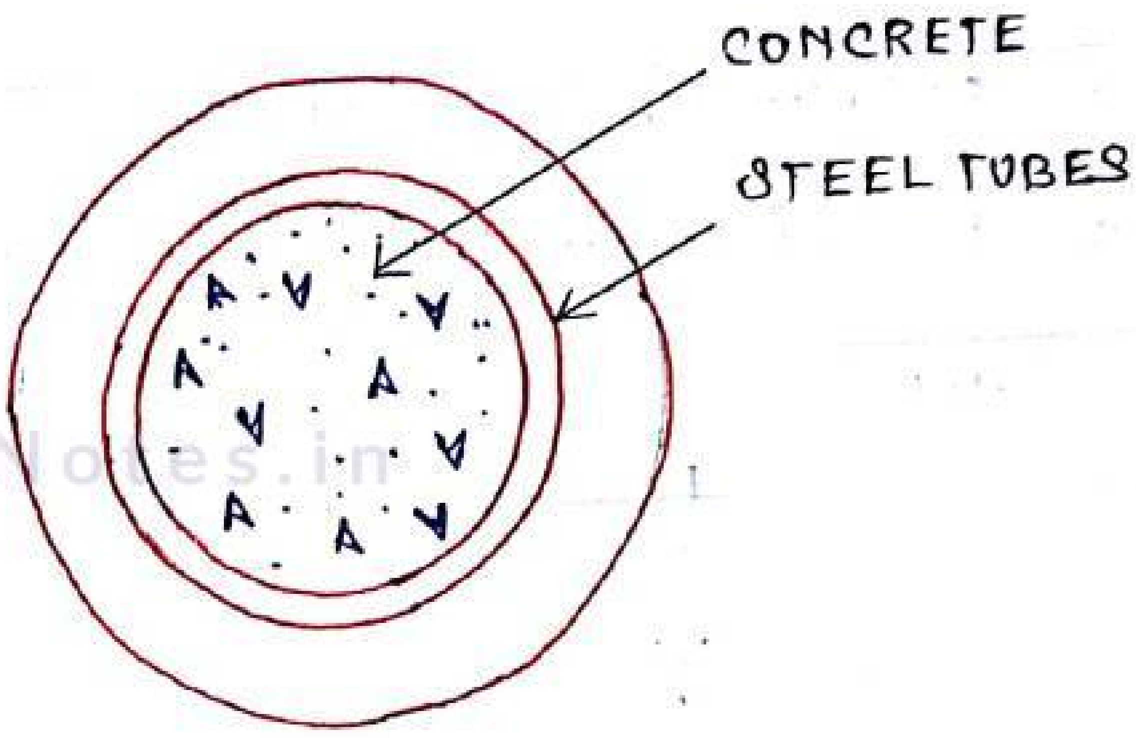
Composite Columns



In which the longitudinal reinforcement is in the form of structural

- Steel section (or) pipe

Infilled Column



CONCRETE FILLED STEEL TUBE

Purpose of Tie Bars

* Every RCC Column must be provided with transverse steel called Tie bars (lateral steel) due to following reasons

  (a) To control buckling of longitudinal bar in prematured conditions

  (b) To resist shear and torsion developed in the column

  (c) To maintain proper position of longitudinal bars during construction stage

* According to IS : 456 - 2000 (pg : No 49) the dia of Tie bar

$$\phi \geq 6\text{mm}$$

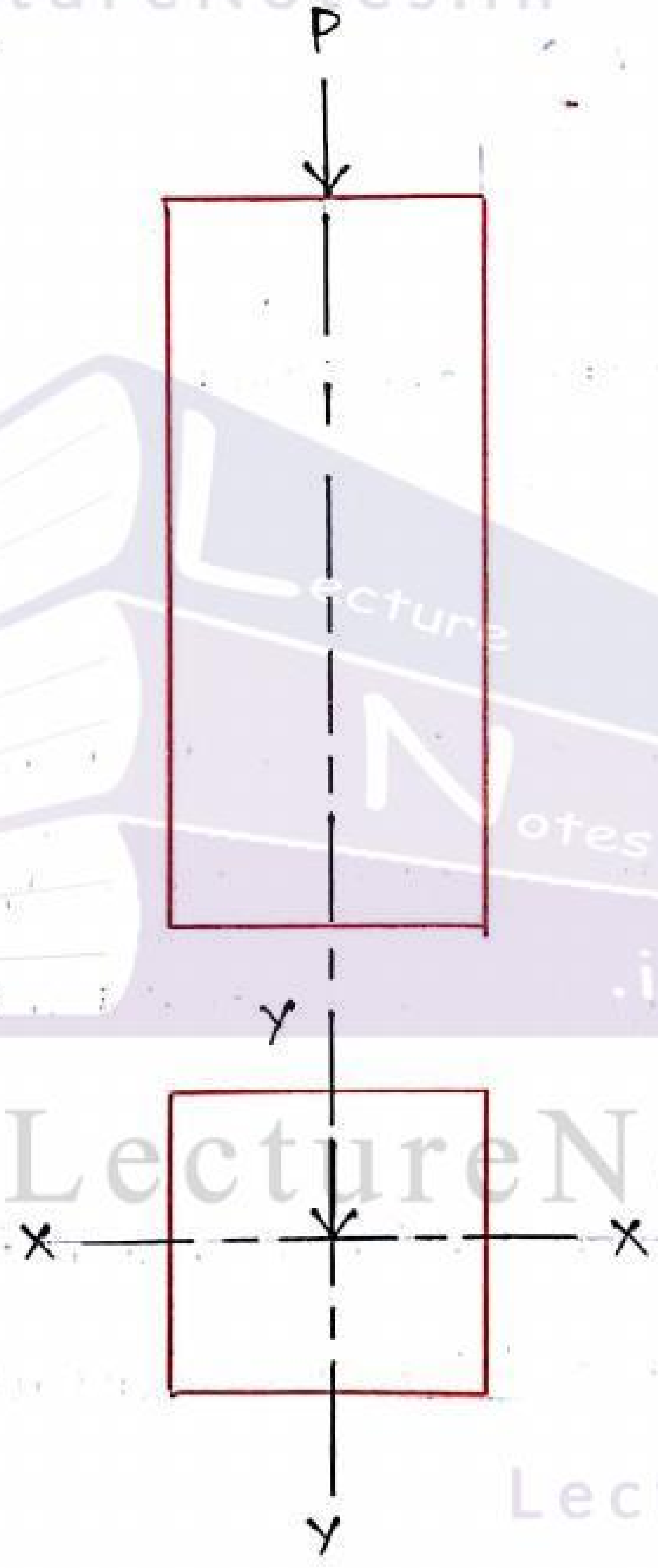
$$\phi \geq \frac{1}{4} \times \phi \text{ Long. max}^{m}$$

Based on Type of Loading

Depending upon the type of loading columns may be classified into the following Two types

* Axially Loaded column
* Eccentrically Loaded column

Axially Loaded Column



Only Axial Load

Moment = 0

* Columns subjected to Load acting along the longitudinal axis or centroid of the column section are called Axially Loaded column
* Axially Loaded column is subjected to direct compressive stress only. No bending stress developed anywhere in the column

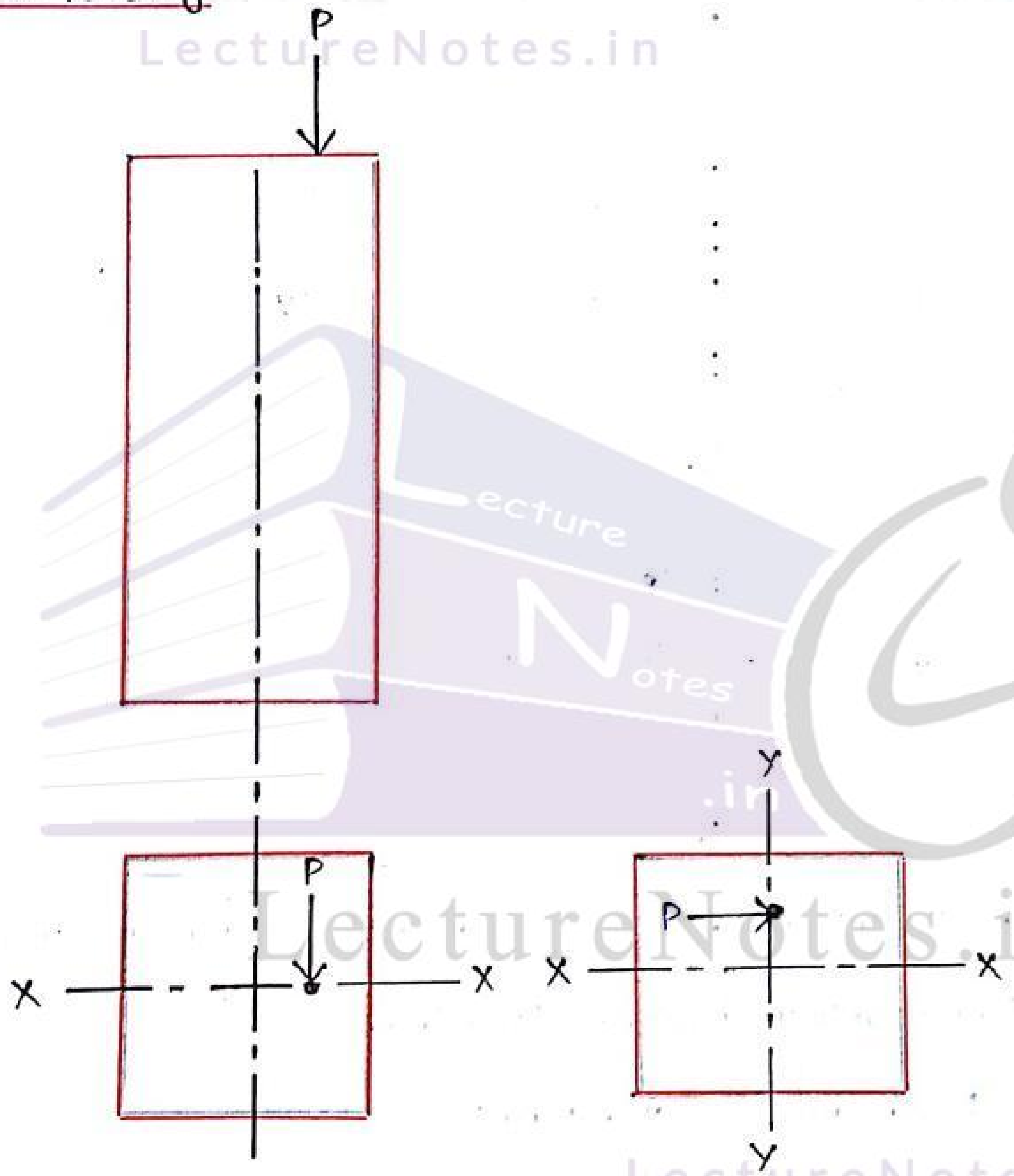
Eccentrically Loaded Column

- Column subjected to load acting away from the centroid of Gravity of column

There are two type of Eccentric loaded column

* Uniaxial Loading
* Biaxial Loading

Uniaxial loading



→ Load donot acts on the longitudinal axis of Column.

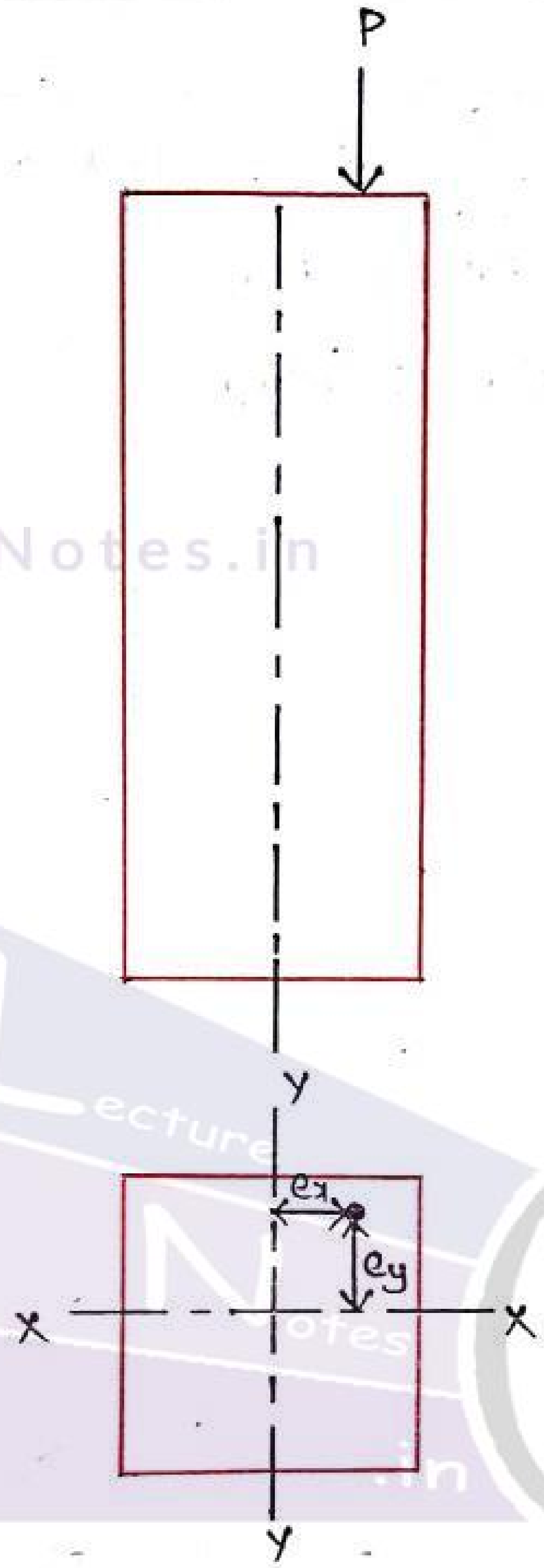
→ They are subjected to direct compressive stress and bending stress Either (x axis or y axis) M_x or M_y

M_x - Moment at x-axis

M_y - Moment at Y-axis

* Column with uniaxial eccentric loading are generally encountered in the case of columns rigidly connected to beams from one side only such as the edge column.

Biaxial Loading

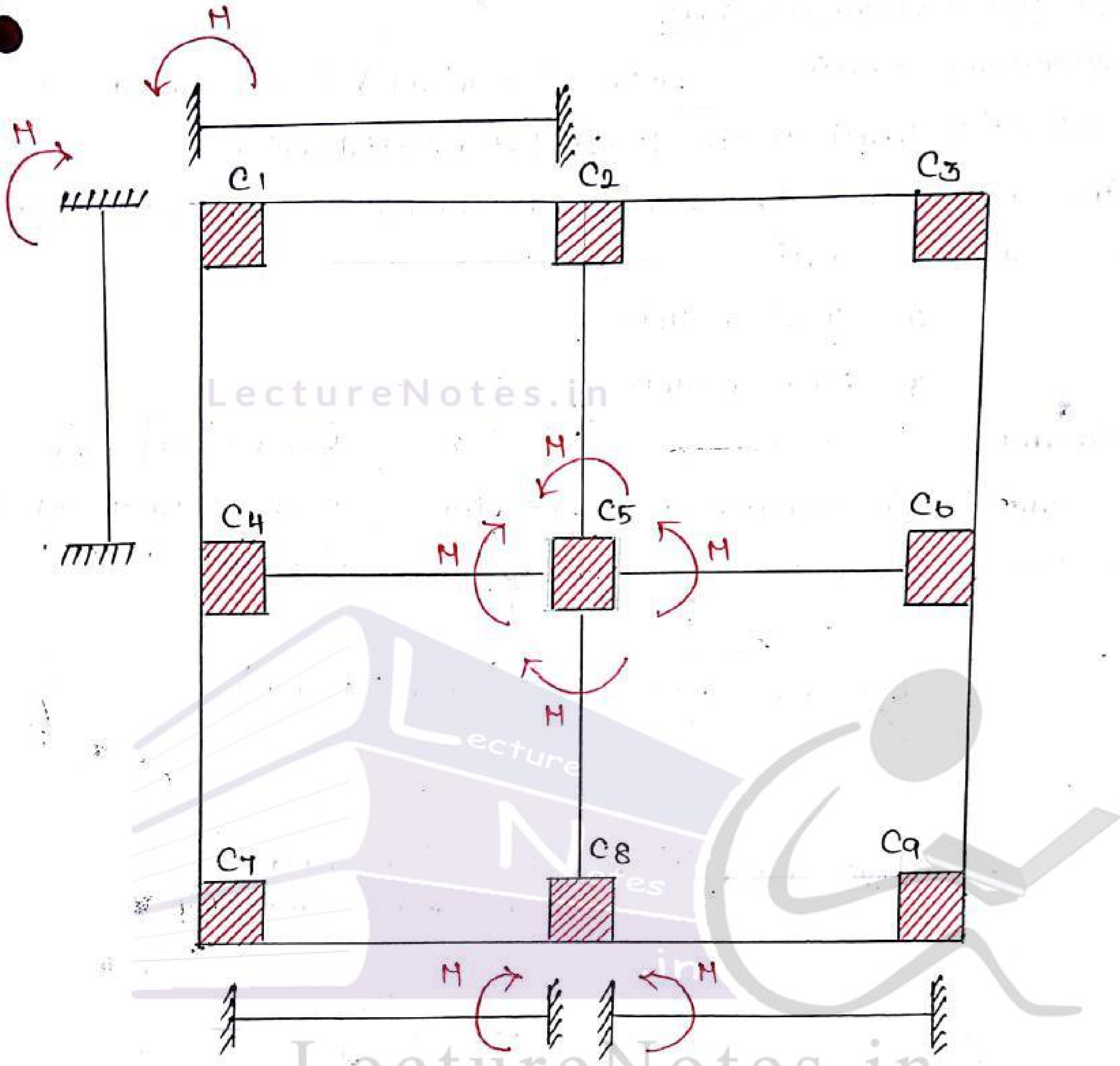


It is eccentricity is with respect to both the axis (x and y axis) the columns is said to be under biaxial loading

→ Both axis (x and y axis)

→ Mx and My

column with biaxial eccentric loading is common in corner column with beams rigidly connected at right angles on the top of the columns.



PLAN

C5 – Axial Load Column

C1, C3, C7, C9 – Biaxial Load Column

C2, C4, C6, C8 – Uniaxial Load Column

Based on the slenderness ratio

The slenderness ratio of compression member is defined as the ratio of effective length to the least lateral dimension

The column are classified as following two types depending upon the slenderness ratio

* Short column
* Long column

Short Column

Refer IS:456-2000
Cl. 25.1.2 Pg. No: 41

The column is considerered as short when both the slenderness ratio $\frac{l_{ex}}{D}$ and $\frac{l_{ey}}{b}$ are less than 12

$$\frac{l_{ex}}{D} \text{ (or) } \frac{l_{ey}}{b} < 12 \quad \text{(Short column)}$$

$$\text{Slenderness ratio } (\lambda) = \frac{\text{Effective length}}{\text{least lateral dimension}}$$

$$\boxed{\lambda = \frac{l_{eff}}{LLD} < 12}$$

Long column

If the slenderness ratio of the column is greater than 12. It is called Longer or slender column.

$$\boxed{\lambda = \frac{l_{eff}}{LLD} > 12}$$

Material of Construction

* Timber columns
* Masonry columns
* RCC columns
* Steel columns

Timber Column

- * Timber column are generally used for Light loads
- * They are used in small Trusses and Wooden houses
- * These are called as post

Masonry column

- * These are used for Light loads

Rcc columns

Rcc columns are used for mostly all type of building and other Rcc structures like water tank, Bridges etc..,

Steel columns

Steel column are used for Heavy Loads

Effective Length

The Effective length of a column is defined as the height between the points of contraflexure of the buckled column. IS 456-2000 Table 28 has given certain values of the effective height for Normal usage assuming Idealized end conditions.

The Effective length of a column depends upon the unsupported length and the boundary conditions at the ends of column.

The Effective length (l_{eff}) can be expressed in the form

$$l_{eff} = KL$$

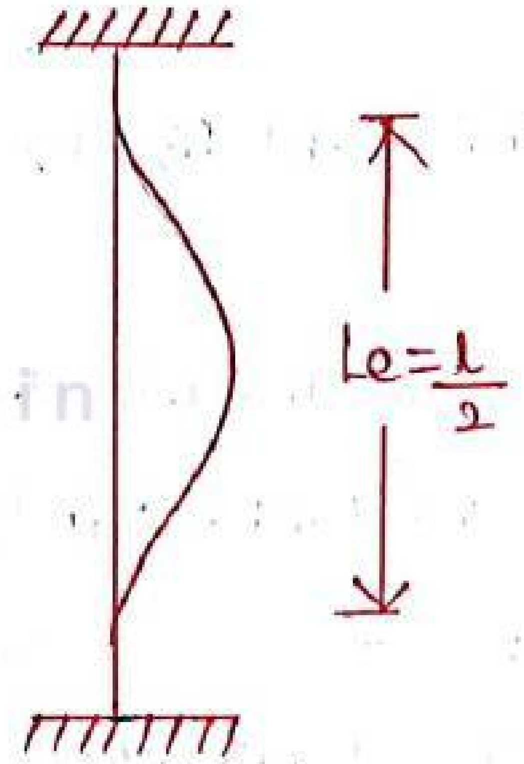
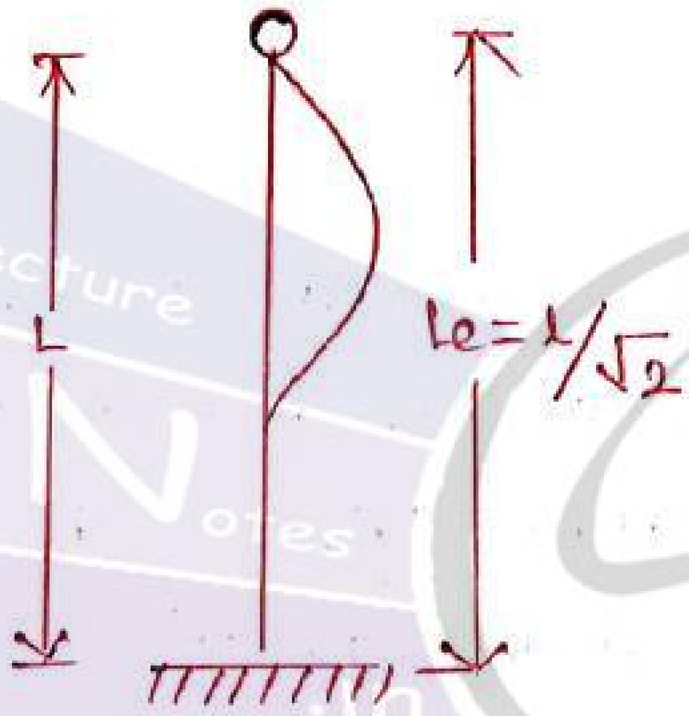
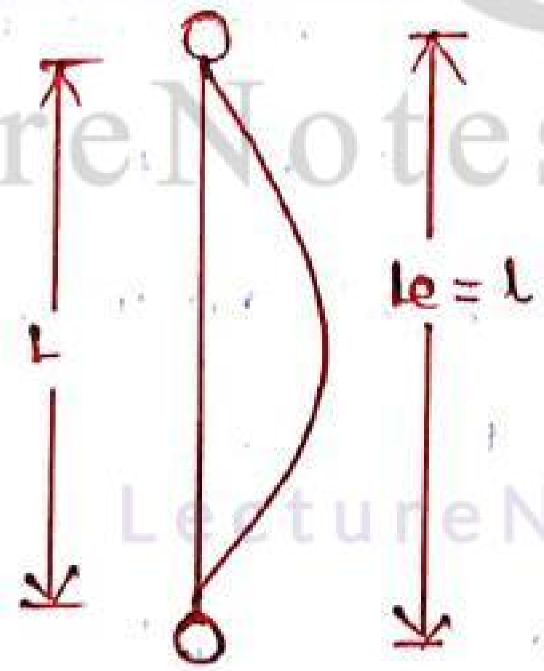
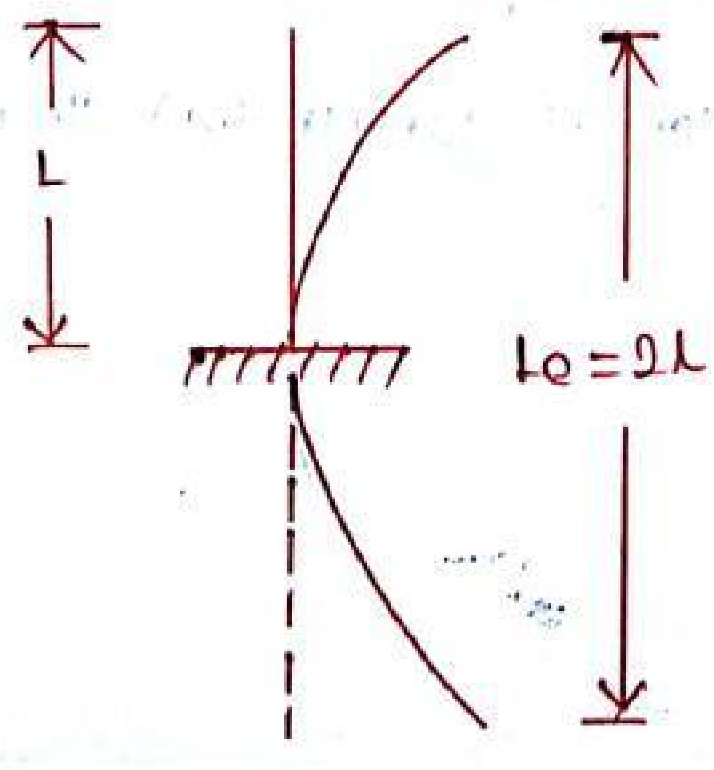
Where,

K = Effective length Ratio

L = Unsupported length (or) Clear height of the column

TABLE - 28

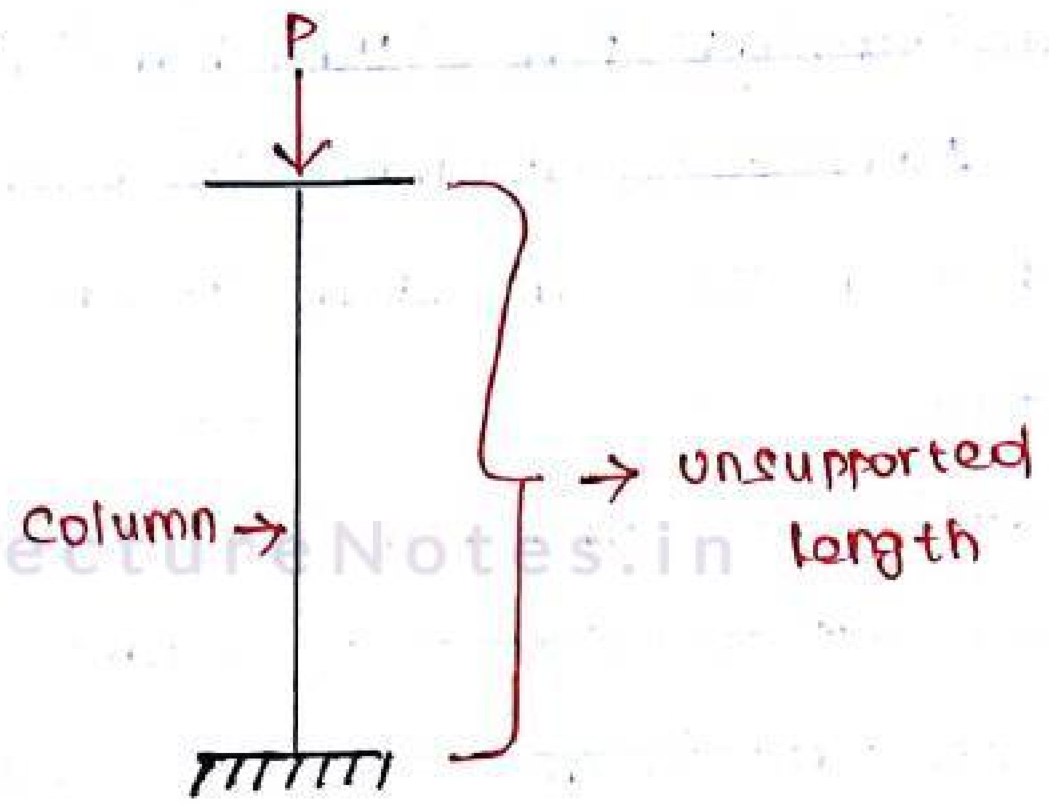
*EFFECTIVE LENGTH OF COMPRESSION MEMBERS

S.No	END CONDITION	SYMBOL	THEORETICAL VALUE EFFEC. LENGTH	RECOMMENDED VALUE. EFFEC. LENGTH
1	Both End Fixed		0.50l	0.65l
2	One End Fixed other End Hinged		0.70l	0.80l
3	Both End Hinged		1.00l	1.20l
4.	One End Fixed other End Free		2.00l	2.00l

Unsupported length of column (L)

- The unsupported length (L) of a column shall be taken as the clear distance between end Restraints

Cl. 25.1.3
Pg. No: 42



Slenderness limit for columns

The unsupported length b/w end restraints shall not exceed 60 times the least lateral dimension of the column, if any given plane one end of a column is unrestrained, it unsupported length (L) shall not exceed $\left(\frac{100b^2}{D}\right)$ (When one of its ends is free)

LIMIT STATE OF COLLAPSE :- COMPRESSION

Assumptions

* The c/s area of Longitudinal bars (Main bars) shall not be less than 0.8 % of Agross of column < Long Column, Short column, LSM, WSM but at the same time it shall not exceed 6% of Agross of Column
    - → But If lapping of column bar is required than it shall not exceed 4% of Agross of Column
    - → In pedestral minimum area of Longitudinal bars shall be 0.15 of Agross of Column
* The overall length (unsupported length) of a column shall not exceed
    - (a) If both ends are restrained $L \leq 60$ times LLD (b or D)
    - (b) If one end is restrained $L \leq 100 \frac{b^2}{D}$

Note:

Every RCC column must be provided with minimum Longitudinal bars (Main bars) due to

(a) Flexural Stress (Bending tension) developed due to eccentricity of loads

(b) To take care of stress developed by shrinkage and creep in concrete

* The dia of Longitudinal bar used in RCC column shall not be less than 12mm
* The clear cover for Longitudinal bar shall not be less than 40mm (or) Dia of bar (Greater). But in case of small size column ($b \leq 200$mm) The clear cover may be reduced 25mm
* The Minimum number of Longitudinal bars provided in a column shall be Four in Rectangular Columns and Six in Circular Columns

Minimum Eccentricity (e_{min})

- Every RCC column must be designed by considering minimum Eccentricity

$$\boxed{e_{min} = \frac{L}{500} + \frac{b}{30} \leq 20\,mm}$$

Where,

L – Unsupported length of column

b – Least Lateral dimension of the column

Minimum Eccentricity for bending about Major axis of bending x-x bisections the depth (D) of the column is given

$$\boxed{e_{minx} = \frac{L_x}{500} + \frac{D}{30} \not< 20\,mm}$$

Minimum Eccentricity for bending about Minor axis of bending y-y bisection the width (b) of the column

$$\boxed{e_{miny} = \frac{L_y}{500} + \frac{b}{30} \not< 20\,mm}$$

Note:

Where bi-axial bending is considered it is sufficient to ensure that eccentricity exceeds the minimum about on axis at a time the minimum eccentricity should be ignored.

Design of Short Columns Under Axial Compression

Design Equation and procedure

The Accordingly, Under pure Axial Loading conditions the design strength of Short Columns is expressed as

Step: 1 Effective Length (Pg. No: Table 28 Pg. No: 94)

Step: 2 Slenderness Ratio (Cl. 25.1.2, Pg. No: 41)

$\frac{l_{ex}}{D}$ and $\frac{l_{ey}}{b} < 12$ - Short Column

$\frac{l_{ex}}{D}$ and $\frac{l_{ey}}{b} > 12$ - Long Column

Step: 3 Minimum Eccentricities (Pg. No: 71)

When the Minimum eccentricity does not exceed 0.05 times the least lateral dimension

$$e_{min} \ngtr 0.05 D$$

$$e_{min} = \frac{L}{500} + \frac{D}{30} \leq 20mm$$

Step: 4 Longitudinal Reinforcement (Cl. 39.3; Pg. No: 71)

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

Where,

P_u = Axial Ultimate Load on the member

A_c = Area of Concrete

A_{sc} = Area of Longitudinal Reinforcement

$A_c = A_g - A_{sc}$

Where A_g = Gross area of the section of column

$$P_0 = 0.4 f_{ck} A_g + (0.67 f_{ck} - 0.4 f_{ck}) A_{sc}$$

Step: 5 Traverse reinforcement (Pg.No: 49)

(i) Diameter of Ties Should not be less than

* $\frac{1}{4}$ th of Longer diameter of Longitudinal bars
* 6mm

(ii) The pitch (or) Spacing of Ties Shall Not be more than

* 16 x The Dia of Smallest longitudinal bar
* 300mm
* 48d

Compression member with Helical Reinforcement

The Strength of compression with helical reinforcement Satisfy the requirement the following Shall be taken as 1.05 times the similar member with lateral Ties

$$P_0 = 1.05(0.4 f_{ck} A_c + 0.67 f_y A_{sc})$$ (Pg.No: 71)

When the ratio of the Volume of helical reinforcement to the Volume of the core shall not be less than

$$\frac{\text{Volume of Helix}}{\text{Volume of core}} \geq 0.36\left(\frac{A_g}{A_c} - 1\right)\frac{f_{ck}}{f_y}$$

Where,

A_g - Gross area of the Section

A_c - Area of the core of the Helically reinforced Column measure to the outside dia of Helix

f_{ck} - Characteristic Comp. Strength of Concrete

f_y - Characteristic Strength of the Helical reinforcement but not exceed 415 N/mm²

Design Consideration for Helical reinforcement

(i) Diameter of the Helix

Should not less than the following

* $\frac{1}{4}$ x Dia of Largest longitudinal bar
* 5mm

Cl. 26.5.3.2
Pg. No: 49

(ii) Pitch of the Helix

Case-I

* 75mm
* 1/6 x D_k

} Lesser of these Two

Case-II

* 25 mm
* 3 x Dia of helix

} Greater of these two

Problem on Analysis of Column

Prblm. No: 1

A Rectangular column of size 400mm x 550mm is Fixed at one End and Hinged at other End. Its unsupported length is 4.2m. Determine the slenderness ratio of the column and state whether it is a Long column or Short column

Given data

Unsupported Length (L) = 4200mm

Effective length $(l_{eff}) = 0.80L$

$$= 0.80 \times 4200$$

$$l_{eff} = 3660\,mm$$

Solution

(i) Slenderness ratio with respect to Major axis

$$\lambda_x = \frac{l_{eff}}{D}$$

$$= \frac{3360}{550}$$

$$\lambda_x = 6.11 < 12$$

(ii) Slenderness ratio with respect to Minor axis

$$\lambda_y = \frac{l_{eff}}{b}$$

$$= \frac{3360}{400}$$

$$\lambda_y = 8.4 < 12$$

Both of its slenderness ratio are less than 12

The column is a Short column

Prblm. No:2

The Effective Length of a rectangular RC column of 250mm x 500mm Size are found to be 4.6m and 3.8m with respect to the Major and Minor axes respectively. State whether the column is a Short or Long Column.

Given data

$l_{ex} = 4600mm$ (Eff. Length with respect to Major axis)

$l_{ey} = 3800mm$ (,, ,, Minor axis)

$D = 500mm$

$b = 250mm$

Solution

$$\lambda_x = \frac{l_{ex}}{D} = \frac{4600}{500} = 9.2 < 12 \quad \text{(Short column)}$$

$$\lambda_y = \frac{l_{ey}}{b} = \frac{3800}{250} = 15.2 > 12 \quad \text{(Long column)}$$

Since the slenderness ratio in respect of Minor axis exceed 12, it is a Long column.

Prblm. No:3

What is the maximum permitted Unsupported Length of a RCC column of size 300mm x 450mm

(i) Both Ends are Fixed

(ii) One End Fixed and one end Free

Given data

$b = 300mm$

$D = 450mm$

Solution

(i) When both ends are Fixed

$l_{max} = 60b$

$= 60 \times 300$

$= 18000 mm$

$\boxed{l_{max} = 18m}$

(ii) When one end Fixed and one End Free

$$l_{max} = \frac{100b^2}{D}$$

$$= \frac{100 \times 300^2}{450}$$

$$= 20000\ mm$$

$\boxed{l_{max} = 20m}$

Prblm. No: 4

Determine the minimum required area of Longitudinal reinforcedl in a RC column of 250 mm x 400 mm size if the concrete is to be Fully effective. Determine also the max. permitted area of Longitudinal reinforcement When

(i) The Bars are not over lapped

(ii) The Bars are overlapped

Given data

Gross area $(A_g) = 250 \times 400$

$= 100000 mm^2$

Minimum area of Longitudinal reinforcement

$A_{st\,min} = 0.8\%$ of A_g

$$= \frac{0.8}{100} \times 100000$$

$A_{st\,min} = 800 mm^2$

Solution

(i) Maximum area of Longitudinal reinforcement

$A_{sc\,max} = 6\%$ of A_g

$$Asc_{max} = \frac{6}{100} \times 100000$$

$$\boxed{Asc_{max} = 6000\, mm^2}$$

(ii) Asc max when the bars are overlapped

$$Asc_{max} = 4\% \text{ of } Ag$$

$$= \frac{.4}{100} \times 100000$$

$$\boxed{Asc_{max} = 4000\, mm^2}$$

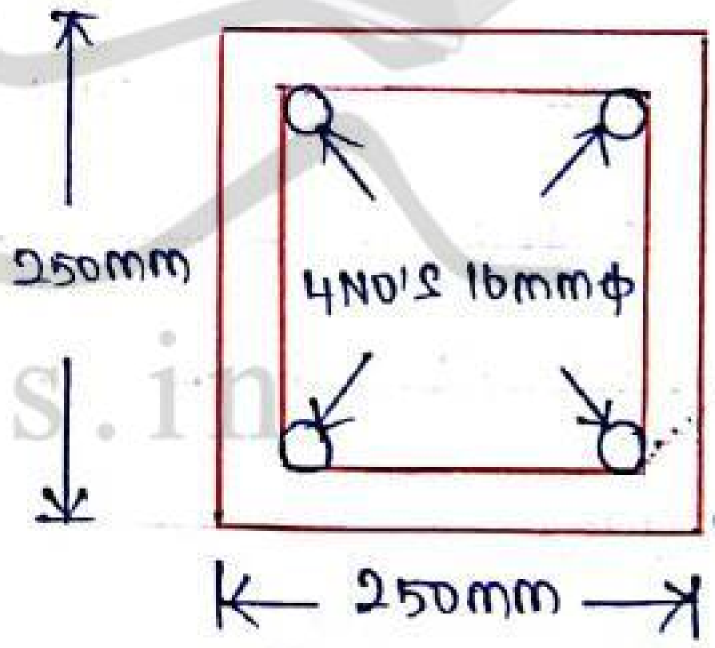
Prblm. No: 5

An RC column of 250mm x 250mm size has 4 Mild steel bars of 16mm diameter one at each corner. The Effective length of the column is 2.75m. The concrete is of M20 Grade. Determine the strength of this column at the Limit State of Collapse.

Given data

Size of column $(Ag) = 250 \times 250$

$$Ag = 62500\, mm^2$$



Effective length $(l_{eff}) = 2.75m \Rightarrow 2.75 \times 1000$

$$= 2750\, mm$$

$$Asc = n \times \frac{\pi}{4} d^2$$

$$= 4 \times \frac{\pi}{4} (16)^2$$

$$Asc = 804.25\, mm^2$$

Area of concrete $(Ac) = Ag - Asc$

$$= 62500 - 804.25$$

$$Ac = 61695.75\, mm^2$$

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

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

Solution

(i) Slenderness Ratio

$$\lambda = \frac{l_{eff}}{LLD}$$

$$= \frac{2750}{250} \Rightarrow 11 < 12$$

Hence Short column

(ii) Strength of Column

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

$$= (0.4 \times 20 \times 61695.75) + (0.67 \times 250 \times 840.25)$$

$$= 493566 + 134711.875$$

$$= 628277.875 N$$

$$P_u = 628.28 kN$$

Prblm. No: 6

A 600mm x 400mm column 3m long effectively held in position and restrained against rotation at both ends is provided with 6 bars of 22mm dia of Fe415 steel. Determine the strength, if M20 Grade is used

Given data

Size of column

$$A_g = 600 \times 400$$

$$= 240000 mm^2$$

$$A_{sc} = n \times \frac{\pi}{4} d^2$$

$$= 6 \times \frac{\pi}{4} (22)^2$$

$$= 2280.8 mm^2$$

End Condition : Both End Fixed

Effective length (l_{eff}) = $0.65L$

Solution

(i) Slenderness Ratio

$$\lambda = \frac{l_{eff}}{LLD}$$

$$= \frac{2750}{250} \Rightarrow 11 < 12$$

Hence Short column

(ii) Strength of Column

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

$= (0.4 \times 20 \times 61695.75) + (0.67 \times 250 \times 840.25)$

$= 493566 + 134711.875$

$= 628277.875 N$

$P_u = 628.28 KN$

Prblm. No: 6

A 600mm x 400mm column 3m long effectively held in position and restrained against rotation at both ends is provided with 6 bars of 22mm dia of Fe415 steel. Determine the strength, if M20 Grade is used

Given data

Size of column

$A_g = 600 \times 400$

$= 240000 mm^2$

$A_{sc} = n \times \frac{\pi}{4} d^2$

$= 6 \times \frac{\pi}{4} (22)^2$

$= 2280.8 mm^2$

End Condition : Both End Fixed

Effective length $(l_{eff}) = 0.65L$

$= 0.65 \times 3000$

$l_{eff} = 1950\,mm$

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

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

Solution

(i) Slenderness Ratio

$$\lambda = \frac{l_{eff}}{LLD}$$

$$= \frac{1950}{400} \Rightarrow 4.88 < 12$$

Hence short column

(ii) Minimum Eccentricity

Longer side

$$e_{min} = \frac{L}{500} + \frac{D}{300}$$

$$= \frac{3000}{500} + \frac{400}{30}$$

$$\boxed{e_{min} = 26\,mm}$$

$0.05D = 0.05 \times 600$

$= 30\,mm$

Shorter side

$$e_{min} = \frac{L}{500} + \frac{b}{30}$$

$$= \frac{3000}{500} + \frac{400}{30}$$

$$\boxed{e_{min} = 19.33\,m}$$

$0.05b = 0.05 \times 400$

$= 20\,mm$

Since $e_{min} < 0.05\,LLD$

(iii) Strength of Column

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

$$= [0.4 \times 20 \times \underset{(A_g - A_{sc})}{(240000 - 2280.8)}] + [0.67 \times 415 \times 2280.8]$$

$$= 1901753.6 + 634176.44$$

$$= 2535930.04 N$$

$$P_u = 2535.93 KN$$

Safe Working Load (Axial compressive load)

$$P = \frac{P_u}{PSF}$$

$$= \frac{2535.93}{1.5}$$

$$\boxed{P = 1690.62 KN}$$

Ultimate Factor Load $(P_u) = 2535.93 KN$

Safe Working (Axial load) $P = 1690.62 KN$

Prblm. No: 7

A 450mm dia reinforced concrete column 3.6m long effectively held in position at both ends but not restrained against rotation, is reinforced with 8 bars of 25mm dia. Determine the ultimate load and safe working load if M20 & Fe 500 were used

Given data

Given column : Circular

Dia. of Column (D) = 450mm

$$A_{sc} = 8 \times \frac{\pi}{4} 25^2 = 3927 mm^2$$

$$f_{ck} = 20 N/mm^2, \; f_y = 500 N/mm^2$$

End condition : Both Ends are Hinged

Solution

- $A_g = \frac{\pi}{4} D^2 \Rightarrow \frac{\pi}{4} \times 450^2 = 159043.13 mm^2$

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

$= 159043.13 - 3927 = 155116.13 mm^2$

$l_e = 1.00 L$

$= 1.00 \times 3600 = 3600 mm$

(i) Slenderness ratio (λ)

$$\lambda = \frac{l_{eff}}{LLD} = \frac{3600}{450} = 8 < 12$$

(ii) Minimum Eccentricity

$$e_{min} = \frac{L}{500} + \frac{LLD}{30}$$

$$= \frac{3600}{500} + \frac{450}{30}$$

$= 7.2 + 15$

$= 22.2 mm$

$0.05D = 0.05 \times 450$

$= 22.5 mm$

$e_{min} < 0.05D$

(iii) Strength of Column

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

$= (0.4 \times 20 \times 155116.13) + (0.67 \times 500 \times 3927)$

$= 1240929 + 1315545$

$= 2556474 N$

$= 2556.474 kN$

Ultimate Strength } $\boxed{P_u = 2556.474 KN}$

(iv) Safe (Axial Compressive) Working Load

$$P = \frac{P_u}{\text{Partial Safety Factor}}$$

$$= \frac{2556.474}{1.5}$$

$$\boxed{P = 1704.316 \text{ kN}}$$

Ultimate load = 2556.474 kN

Safe Working load = 1704.316 kN

Prblm. No: 8

A RCC Column 3.3m effective length and 400 mm dia. is reinforced with 8 bars of 20 mm dia Fe250 Steel. Find the Safe Load the Column can carry if it is wound by Spiral reinforcement with 8mm mild steel bar around the compression reinforcement at a pitch of 50mm clear cover for main bars is 40mm. Use M15 grade Concrete

Given data

Given column :- Helically Reinforced Concrete column

Effective length (l_{eff}) = 3.3m

= 3300mm

$Asc = 8 \times \frac{\pi}{4}(20)^2 = 2513.27 mm^2$

Pitch (P) = 50mm ; clear cover = 40mm

$f_{ck} = 15 N/mm^2$; $f_y = 250 N/mm^2$

Solution

(i) Slenderness Ratio (λ)

$$\lambda = \frac{l_{eff}}{LLD}$$

$= \frac{330}{400} \Rightarrow 8.25 < 12$ (Short Column)

(ii) Minimum Eccentricity (e_{min})

$$e_{min} = \frac{L}{500} + \frac{LLD}{30}$$

$$= \frac{3300}{500} + \frac{400}{30}$$

$$\boxed{e_{min} = 19.93\,mm}$$

$$0.05D = 0.05 \times 400$$

$$= 20\,mm$$

$$e_{min} < 0.05D$$

(iii) Strength of Column

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

As per Cl: 39.4
IS: 456-2000

$$\frac{\text{Volume of Helix}}{\text{Volume of Core}} \geq 0.36 \frac{f_{ck}}{f_y}\left(\frac{A_g}{A_c} - 1\right)$$

Mean dia. of Helix = Dia. of Column − Both Side clear Cover − $\frac{1}{2}$ of Helical reinforcement (Both side)

$$= 400 - (2 \times 40) - \left(\frac{8}{2} + \frac{8}{2}\right)$$

$$= 400 - 80 - \frac{16}{2}$$

$$= 312\,mm$$

Length of Helix in pitch Height $\Big\} = \sqrt{(\text{circumference})^2 + (\text{pitch})^2}$

$$= \sqrt{\pi D^2 + 50^2}$$

$$= \sqrt{\pi \times 312^2 + 50^2}$$

$$= 981.45\,mm$$

Area of helix $= \frac{\pi}{4} d^2$

$= \frac{\pi}{4} \times (8)^2 = 50.27 mm^2$

Vol. of helix per pitch height $\Big\}$ = length of helic in pitch ht (mm) $\times$ Area of Steel bar in Helix (mm^2)

$= 981.45 \times 50.27$

$= 49337.49 mm^3$

Core diameter = Dia of column − Both side clear cover

$= 400 - (2 \times 40)$

$= 320 mm$

Area of core (Ac) $= \frac{\pi}{4} d^2$

$= \frac{\pi}{4} \times 320^2$

$Ac = 80424.77 mm^2$

Vol. of core per pitch height $\Big\}$ = Area of core × P

$= 80424.77 \times 50$

$= 4021238.5 mm^3$

Gross area (Ag) $= \frac{\pi}{4} D^2$

$= \frac{\pi}{4} \times 400$

$Ag = 125664 mm^2$

$0.36 \frac{f_{ck}}{f_y} \left(\frac{Ag}{Ac} - 1\right) = 0.36 \times \frac{15}{250} \left(\frac{125664}{80424.77} - 1\right)$

$= 0.121$

$0.0121 \geq 0.122$ (Condition - I is satisfied)

Condition - II

Case - I

Maximum pitch should be less than 75 mm

$= \frac{1}{6} \times$ core dia

$= \frac{1}{6} \times 320 \Rightarrow 53.33 mm$

Case- II

- Minimum pitch 25mm

$= 3 \times$ dia of helix

$= 3 \times 8$

$= 24$ mm (condition - II also satisfied) $25 > 24$

Provide 8mm dia Spiral at a pitch of 50mm

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

$= 1.05(0.4 f_{ck}(A_g - A_{sc}) + 0.67 \times 250 \times 2513.27)$

$= 1.05(0.4 \times 15(125664 - 2513.27) + 0.67 \times 250 \times 2513.27$

$= 1.05(1159877.1)$

$= 1217871$ N

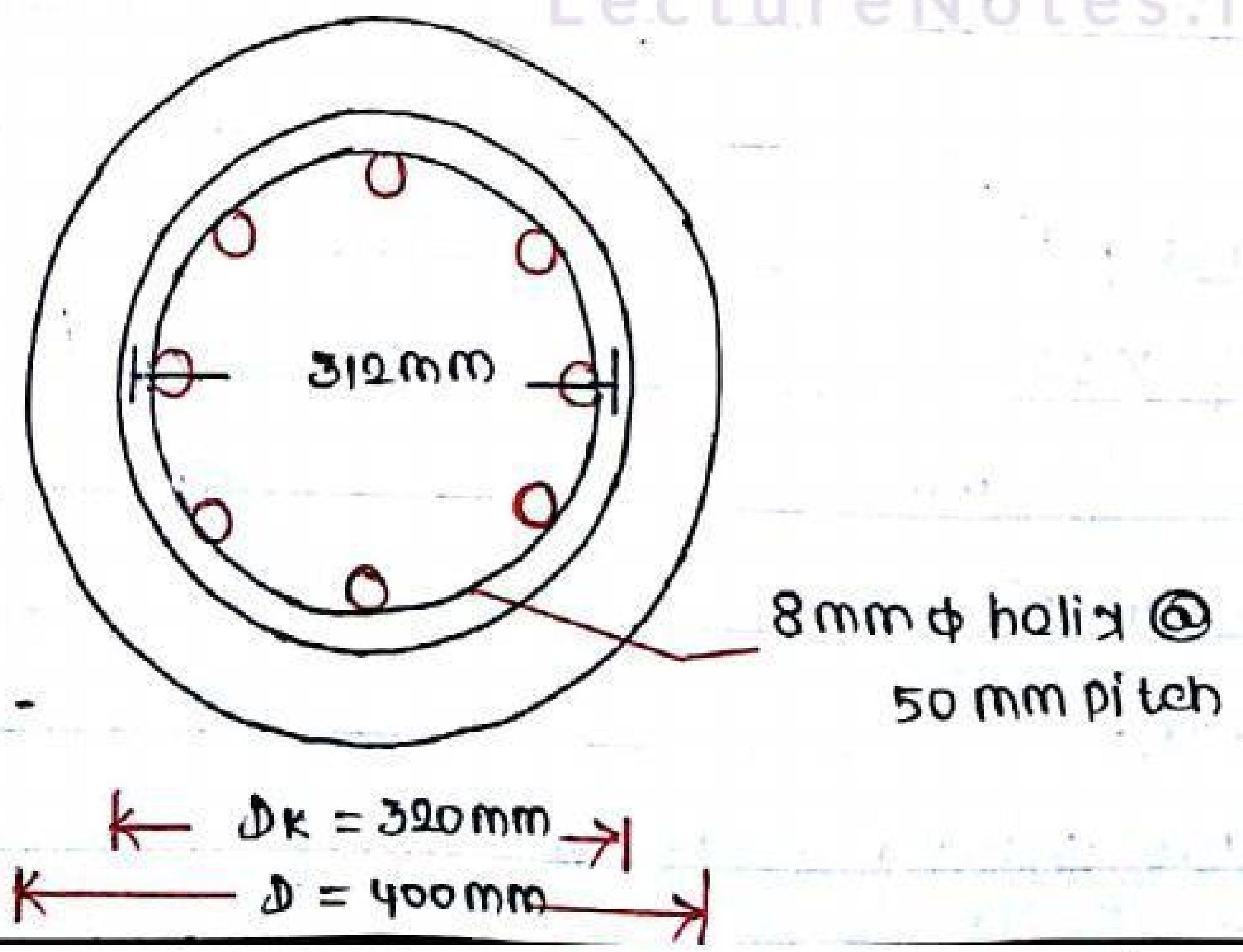
$\boxed{P_u = 1217.87 \text{ KN}}$

Safe Working Load

$$P = \frac{P_u}{PSF}$$

$$= \frac{1217.87}{1.5}$$

$\boxed{P = 811.914 \text{ KN}}$



Prblm. No: 9

A Rectangular reinforced Concrete column of c/s dimensions 300mm x 600mm is to be designed to support an Ultimate axial load Of 2000 kN. Design Suitable reinforcement in the Column Using M20 Grade Concrete and Fe 415 HYSD bars

Given data

$$P_u = 2000\ \text{kN}$$

$$b = 300\ \text{mm}, D = 600\ \text{mm}$$

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

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

Longitudinal Reinforcement

$$P_u = 0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}$$

$$2000 \times 10^3 = [0.4 \times 20 \times (300 \times 600)] + [(0.67 \times 415) - (0.4 \times 20)] A_{sc}$$

$$2000 \times 10^3 = 1.44 \times 10^6 + 270.05 A_{sc}$$

$$A_{sc} = \frac{2000 \times 10^3 - 1.44 \times 10^6}{270.05}$$

$$\boxed{A_{sc} = 2073.69\ \text{mm}^2}$$

provide 6 bars of 22mm dia

$$A_{sc\,prov} = \frac{\pi}{4} \times (22)^2 \times 6$$

$$= 2280.79\ \text{mm}^2$$

Lateral Ties

(i) The diameter ties

$$= \frac{1}{4} \times 22$$

$$= 5.5 \simeq 6\ \text{mm}$$

provide 8mm ties

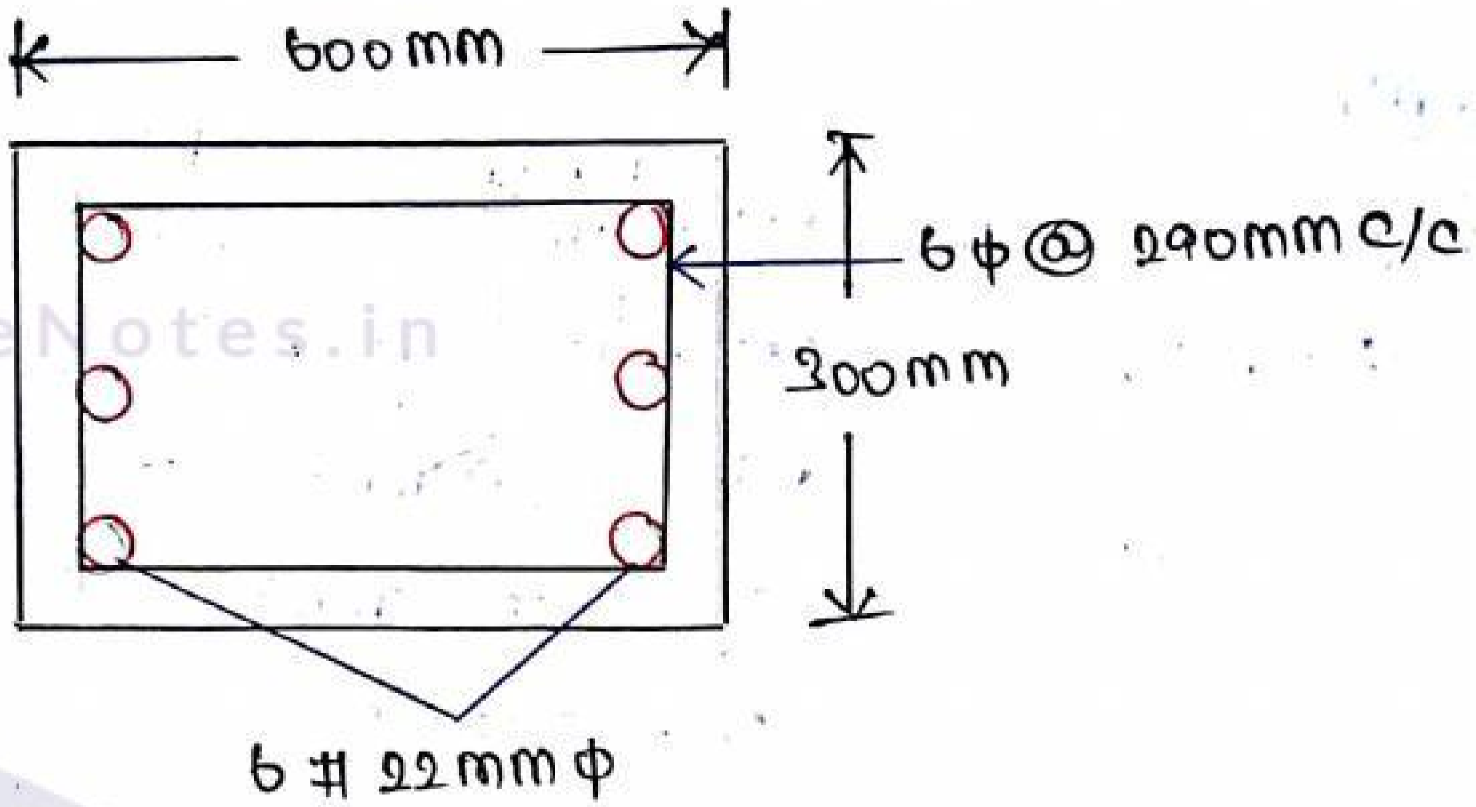
(ii) Tie Spacing

* $16\phi \Rightarrow 16 \times 22 = 352\ \text{mm}$

* $48 \times 6 = 288$ mm

* 300 mm

Provide 6mm ϕ ties at 290mm c/c



Design Problems

Prblm. No: 10

Design a Column 4m Long restrained in position and direction at both end to carry an axial load of 1600 KN. Use M20 Concrete and Fe 415 Steel

Given data

Length of column $(L) = 4m$

Axial Load $(P) = 1600\ KN$

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

$f_y = 415\ N/mm^2$

$P_u = PSF \times P$

$= 1.5 \times 1600$

$P_u = 2400\ KN$

Step: 1

Effective length (l_{eff})

$l_{eff} = 0.65L$ (End Condition: Both End Fixed)

$= 0.65 \times 4$

$= 2.6m$

Step: 2

Slenderness Ratio

Assume 1% Steel

$A_c = A_g - A_{sc}$

We Get,

$A_{sc} = 1\%$ of A_g

$= \frac{1}{100} A_g$

$= 0.01 A_g$

$A_c = A_g - 0.01 A_g$

$= 0.99 A_g$

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

$2400 \times 10^3 = (0.4 \times 20 \times 0.99 A_g) + (0.67 \times 415 \times 0.01 A_g)$

$= 7.92 A_g + 2.78 A_g$

$= 10.7 A_g$

$A_g = \frac{2400 \times 10^3}{10.7}$

$\boxed{A_g = 224.29 \times 10^3 \ mm^2}$

We know Area of Column

Since it is a square column size

Side of square column $= \sqrt{A_g}$

$= \sqrt{224.29 \times 10^3}$

$= 473.59 \ mm \simeq 500 mm$

Hence we provide 500 mm × 500 mm size of Rectangular Column

Check for Slenderness of Column

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

$\frac{l_{eff}}{b} = \frac{2600}{500} \Rightarrow 5.2 < 12$

It is a short column

Step: 3

Minimum Eccentricity

$e_{min} = \left[\frac{L}{500} + \frac{D}{30}\right]$

$= \left[\frac{4000}{500} + \frac{500}{30}\right]$

$= 24.67 mm$

$0.05D = 0.05 \times 500$

$= 25\,mm$

$e_{min} < 0.05D$

Hence design as Axially Loaded Short column is satisfactory

Step: 4

Longitudinal Reinforcement

$A_{sc} = 0.01 \times A_g$

$= 0.01 \times 214.298 \times 10^3$

$= 2242.9\,mm^2$

$$\boxed{A_{sc} = 2.24 \times 10^3\ mm^2}$$

$$(\text{No. of bar})_{prov} = \frac{A_{sc\,req}}{a_{sc}}$$

$$= \frac{2242.9}{\frac{\pi}{4} \times 20^2}$$

$(NOB)_{prov} = 7.13 \simeq 8$ No's

$$A_{sc\,prov} = n \times \frac{\pi}{4} d^2$$

$$= 8 \times \frac{\pi}{4} (20)^2$$

$$= 2513\,mm^2$$

Provide 8 bars of 20mm ϕ.

Step: 5

Lateral Ties

(i) Ties diameter

$= \frac{1}{4} \times \phi \Rightarrow \frac{1}{4} \times 20 = 5\,mm$

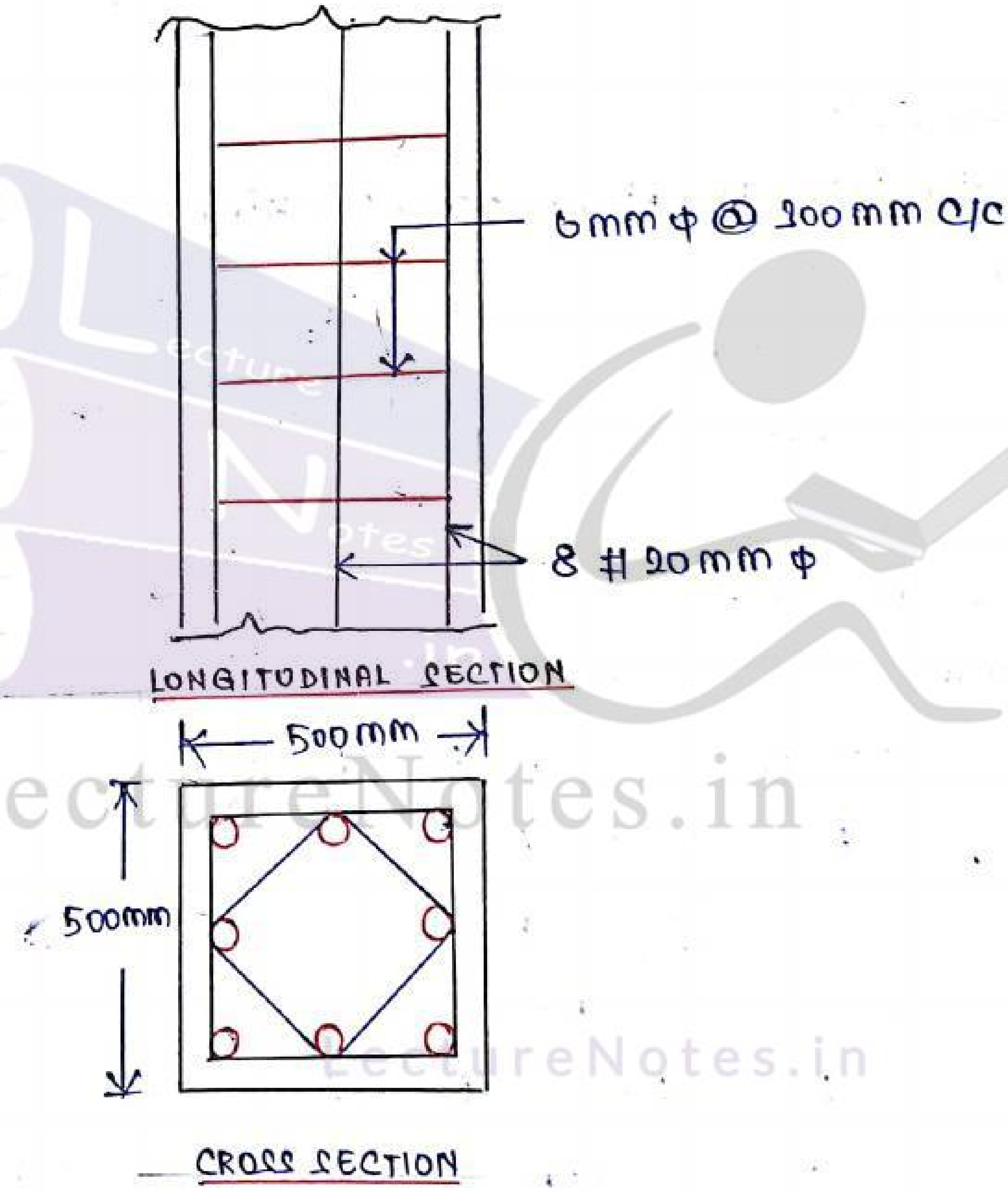
Hence provide 6mm ϕ bars

(ii) It shall be minimum of

* $16\phi = 16 \times 20 \Rightarrow 320$ mm
* $48 \times 6 \Rightarrow 288$ mm $\simeq 300$ mm
* 300 mm

Provide 6 mm ϕ lateral ties at 300 mm c/c

Reinforcement details of Square column



Prblm. No: 11

Design a Rectangular Column 4m, Long effective held in position at both ends, restrained against rotation at one end. If the one side of Column is 400mm and load over it is 2000 KN

Given data :-

Given column : Rectangular column

Length of Column (L) = 4m

Load (P) = 2000 kN

x = 400 mm

$P_u = PSF \times P$

$= 1.5 \times 2000$

$P_u = 3000$ kN

Assume,

M20 Grade of Concrete & Fe 415 Grade of Steel

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

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

Step : 1

Design Load

End Condition : One End Hinged other End Hinged

$l_{eff} = 0.8L$

$= 0.8 \times 4$

$= 3.20$ m

Step : 2

Slenderness Ratio

Assume

1% of steel , $A_c = A_g - A_{sc}$

$A_{sc} = 1\%$ of A_g

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

$= 0.01 A_g$

$A_c = A_g - 0.01 A_g$

$= 0.99 A_g$

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

$3000 \times 10^3 = 0.4 \times 20 \times 0.99 A_g + 0.67 \times 415 \times 0.01 A_g$

$$= \frac{3000 \times 10^3}{0.4 \times 20 \times 0.99 A_g + 0.67 \times 415 \times 0.01 A_g}$$

$\boxed{A_g = 280360.73 \text{ mm}^2}$

We know that Area of Rectangular

Length x Width = Ag (Gross area of Column)

$x \times y = 280360.73$

$400 \times y = 280360.73$

$y = \frac{280360.73}{400}$

$= 700.90 \simeq 700mm$

$y = 700mm$

We provide 400 x 700 mm size of Rectangular column

Check for Slenderness of Column

$\frac{l_{eff}}{D} = \frac{3200}{700}$

$= 4.57$

It is a short column

Step: 3

Minimum Eccentricity

$e_{min} = \left[\frac{L}{500} + \frac{D}{30}\right]$

$= \left[\frac{4000}{500} + \frac{700}{30}\right]$

$= 8 + 23.3$

$= 31.3mm$

$0.05D = 0.05 \times 700$

$= 35$

$e_{min} < 0.05D$

Hence design as Axially Load Short column is satisfactory

Step : 4

Longitudinal Reinforcement

$$Asc_{req} = \frac{1}{100} \times Ag$$
$$= 0.01 \times Ag$$
$$= 0.01 \times 280360.73$$
$$= 2803.60 mm^2 \quad Asc (\text{required})$$

Provide 25 mm ϕ bars

$$(NoB)_{prov} = \frac{Asc\ req}{asc}$$
$$= \frac{2803.60}{\frac{\pi}{4} \times 25^2}$$
$$= 5.70 \simeq 6\ No's$$

$$Asc_{prov} = n \times \frac{\pi}{4} \times 25^2$$
$$= 6 \times \frac{\pi}{4} \times 25^2$$
$$= 2945.24 mm^2$$

Hence provide 6 bars @ 25 mm ϕ ($Asc = 2945.24 mm^2$)

Step : 5

Lateral Ties

(i) Tie diameter

$$= \frac{1}{4} \times \phi$$
$$= \frac{1}{4} \times 25$$
$$= 6.25 \simeq 7\ mm$$

Hence provide 8mm dia bars

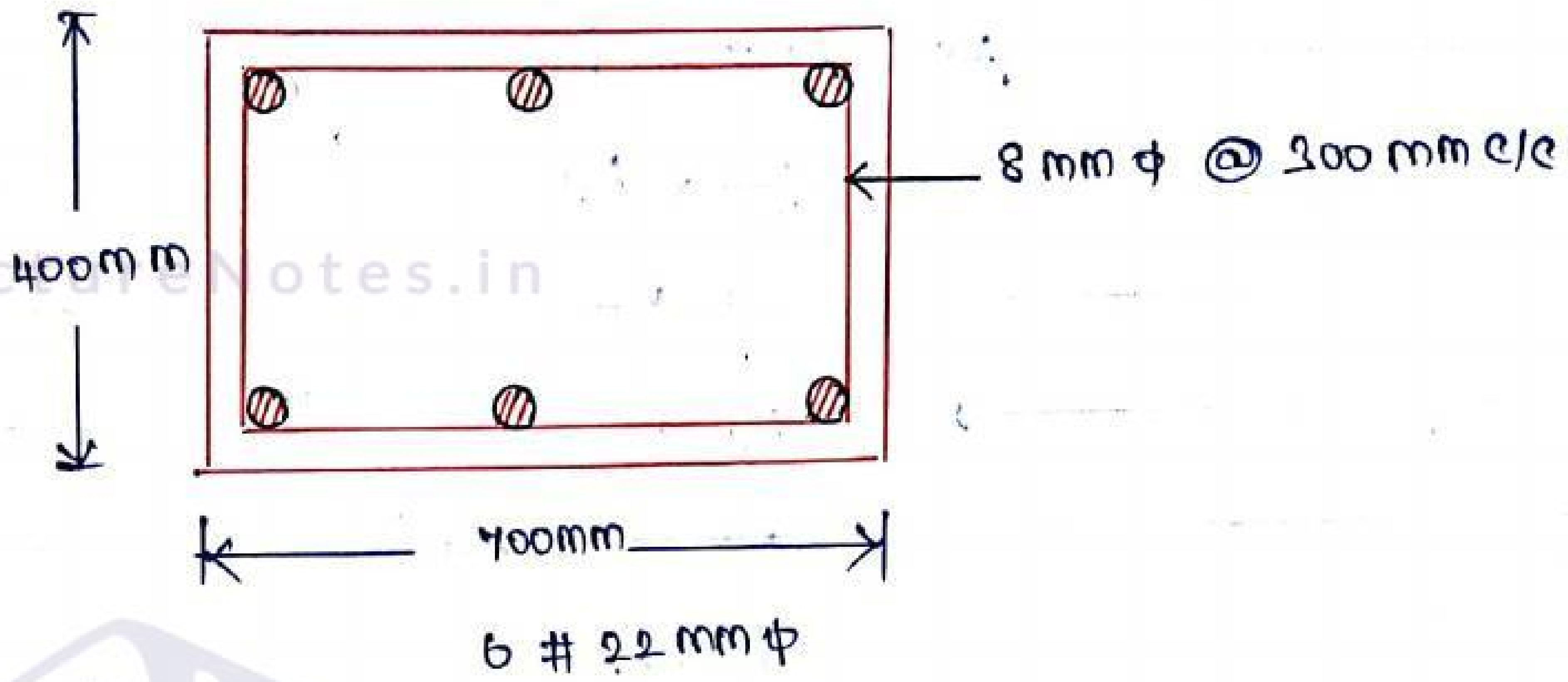
(ii) Pitch (or) Spacing

* $16 \times \phi = 16 \times 25 \Rightarrow 400 mm$

(ii) 48 x 8 = 384mm

(iii) 300mm

provide 8mm Lateral Ties at 300 mm c/c



Prblm · No: 12

Design a circular column to carry an axial load of 1500 KN. The column has an effective length of 2.5m. Use M20 Concrete and Fe 415 Steel.

Given data

Given column : Circular column

Effective length (l_{eff}) = 2.5m

Axial Load (P) = 1500KN

f_{ck} = 20 N/mm^2

f_y = 415 N/mm^2

P_u = PSF x P

= 1.5 x 1500

P_u = 2250 KN

Step: 1

Slenderness ratio

Assume 1% of Steel , $Ac = Ag - Asc$

We Get,

$$Asc = 1\% \text{ of } Ag$$
$$= \frac{1}{100} \times Ag$$
$$= 0.01Ag$$
$$Ac = Ag - Asc$$
$$= Ag - 0.01Ag$$
$$= 0.99Ag$$

$$Pu = 1.05(0.4 f_{ck} Ac + 0.67 f_y Asc)$$

$$2250 \times 10^3 = 1.05(0.4 \times 20 \times 0.99Ag) + (0.67 \times 415 \times 0.01Ag)$$
$$= 1.05(7.92 + 2.78\,Ag)$$
$$= 1.05(10.7\,Ag)$$
$$10.7\,Ag = \frac{2250 \times 10^3}{1.05}$$
$$= \frac{2142857.142}{10.7}$$

$$Ag = 200267\ mm^2 \Rightarrow 200.267 \times 10^3\ mm^2$$

We know Area of circular column

$$\frac{\pi D^2}{4} = \text{Gross area }(Ag)$$

$$0.785D^2 = 200.267 \times 10^3$$
$$= \frac{200.267 \times 10^3}{0.785}$$

$$D = \sqrt{255117.1975}$$

$505.09 \, mm \simeq 550 \, mm$

- Hence we provide a dia of 550mm for the column

Check for slenderness of column

$$\lambda = \frac{l_{eff}}{LD}$$

$$= \frac{2500}{550} = 4.54 < 12$$

It is a short column

Step: 3

Minimum Eccentricity

$$e_{min} = \frac{L}{500} + \frac{D}{30}$$

$$= \frac{2500}{500} + \frac{550}{30}$$

$$= 5 + 18.33$$

$$e_{min} = 23 \, mm$$

$$0.05D = 0.05 \times 550$$

$$= 27.5 \, mm$$

$$e_{min} < 0.05D$$

Hence design as Axially Loaded short column is satisfactory

Step: 4

Longitudinal Reinforcement

$$A_{SC} = \frac{1}{100} \times A_g$$

$$= 0.01 \times 200.267 \times 10^3$$

$$A_{SC_{req}} = 2002.67 \, mm^2$$

$$(\text{No. of bars})_{prov} = \frac{Asc_{req}}{asc}$$

$$= \frac{2002.67}{\frac{\pi}{4} \times 16^2}$$

$$= 9.96 \simeq 10 \text{ No's}$$

$$Asc_{prov} = n \times \frac{\pi}{4} d^2$$

$$= 10 \times \frac{\pi}{4} \times 16^2$$

$$= 2010.61 \text{ mm}^2$$

provide 10 No's of 16mm ϕ bars ($Asc = 2010.61 \text{ mm}^2$)

Step: 5

Design of Helical Reinforcement.

CONDITION - I

Provide 8mm ϕ bars as helical reinforcement

$$\frac{\text{Volume of helix}}{\text{Volume of core}} \geq 0.36 \frac{f_{ck}}{f_y} \left(\frac{A_g}{A_c} - 1 \right)$$

Mean dia of Helix = Dia. of colum − Clear cover (Both side) − $\frac{1}{2}$ of Helical reinforcement (Both side)

$$= 550 - (2 \times 40) - \left(\frac{8}{2} + \frac{8}{2} \right)$$

$$= 550 - 80 - \frac{16}{2}$$

$$= 462$$

length of Helix in pitch Height $\Big\} = \sqrt{c^2 + p^2}$

Assume,

Pitch (P) = 50mm

$$= \sqrt{\pi D^2 + p^2}$$

$$= \sqrt{(\pi \times 462)^2 + 50^2}$$

$$= 1452.27 \text{ mm}$$

$$\text{Area of Helix} = \frac{\pi}{4} d^2$$

$$= \frac{\pi}{4} \times 8^2$$

$$= 50.27 \text{ mm}^2$$

$$\left.\begin{array}{l}\text{Vol. of helix per} \\ \text{pitch ht}\end{array}\right\} = \text{Length} \times \text{Area}$$

$$= 1452.27 \times 50.27$$

$$= 73005.61 \text{ mm}^3$$

$$\text{Dia. of core} = \text{Dia. of col} - \text{Cover both side}$$

$$= 550 - (2 \times 40)$$

$$= 470 \text{ mm}$$

$$\text{Area of core} = \frac{\pi}{4} d^2$$

$$= \frac{\pi}{4} \times 470^2$$

$$= 173494.45 \text{ mm}^2$$

$$\left.\begin{array}{l}\text{Vol. of core} \\ \text{per pitch ht}\end{array}\right\} = \text{Area of core} \times P$$

$$= 173494.45 \times 50$$

$$= 8674722.5 \text{ mm}^3$$

$$\frac{\text{Vol. of Helix}}{\text{Vol. of core}} = \frac{73005.61}{8674722.5}$$

$$= 0.0084$$

$$0.36 \frac{f_{ck}}{f_y}\left(\frac{A_g}{A_c} - 1\right) = 0.36 \times 415 \times \left(\frac{200.267 \times 10^2}{173494.45} - 1\right)$$

$$= 0.0026$$

$$0.0084 > 0.0026$$

Hence the condition – I is satisfactory

Condition - II

Case - I

Pitch should be less than 75mm

(i) $\leq 75\,mm$

(ii) $\frac{1}{6} \times$ core dia

$= \frac{1}{6} \times 470$

$= 78.3\,mm$

Case - II

(i) $\geq 25\,mm$

(ii) $3 \times$ dia of helix

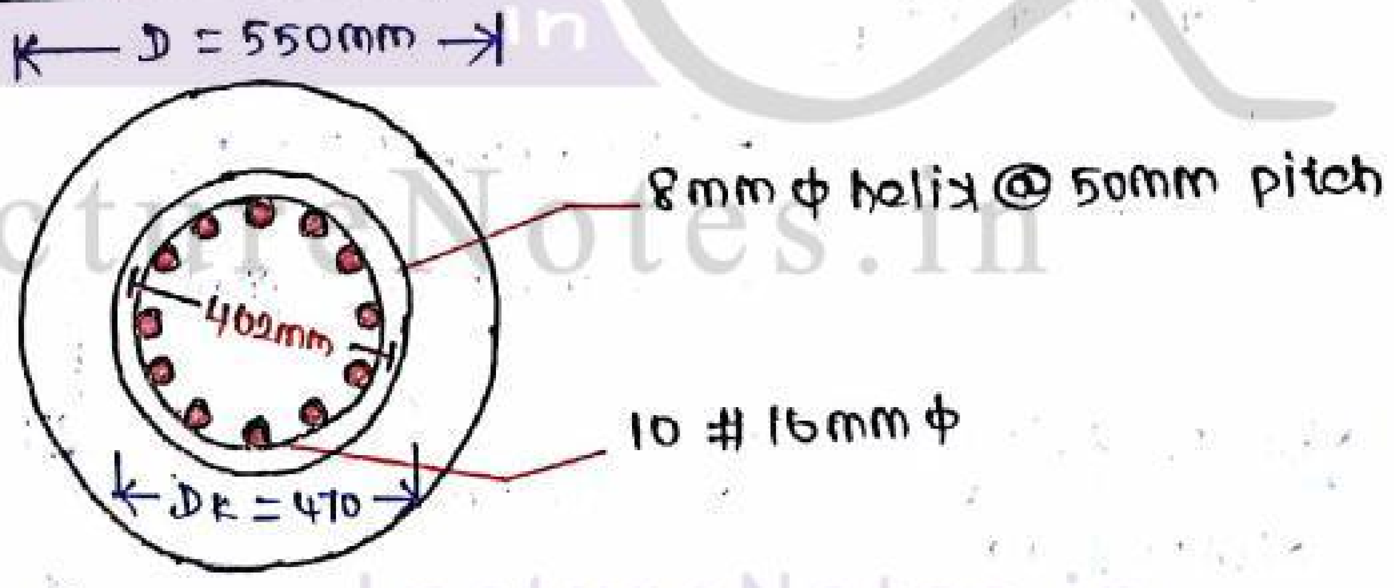
$= 3 \times 8$

$= 24\,mm$

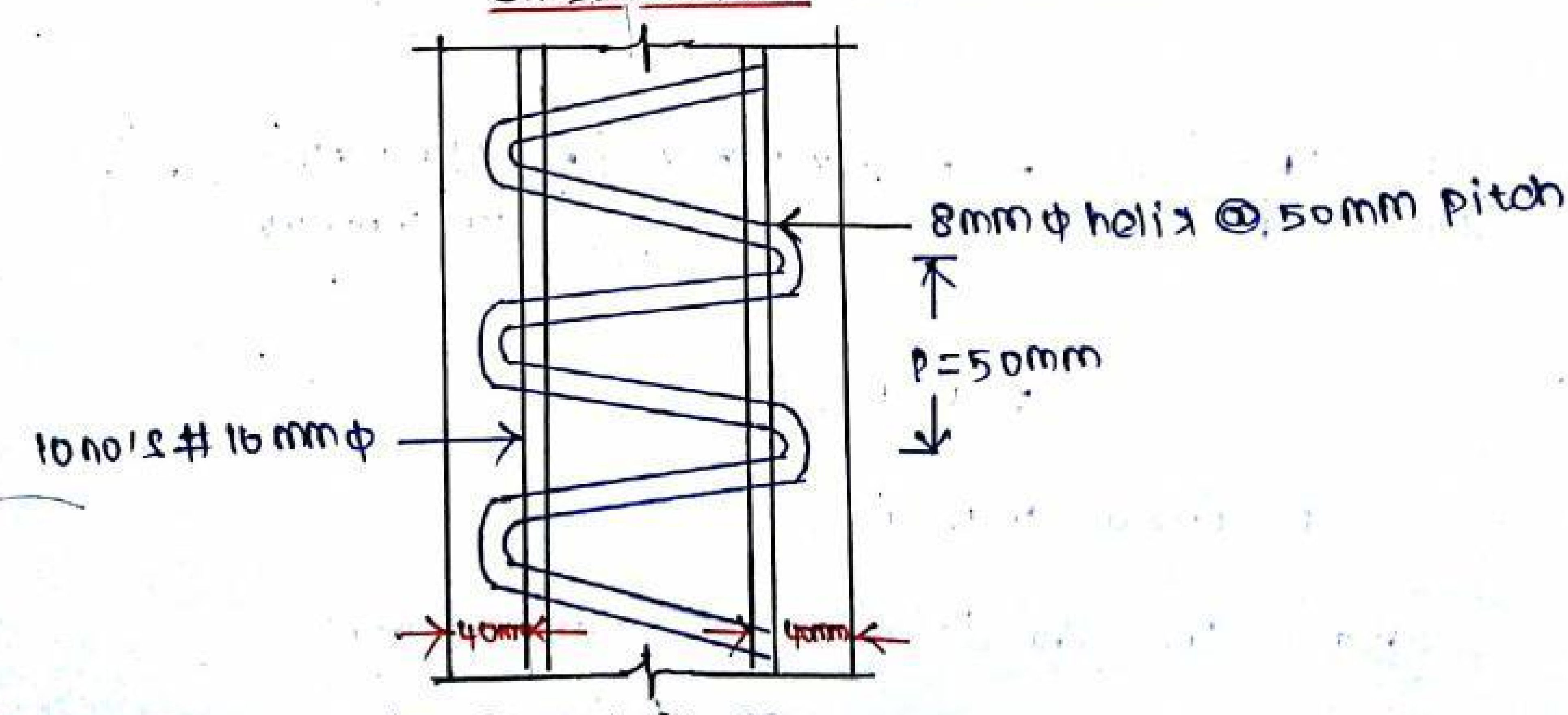
$25 > 24\,mm$

Hence condition - II also satisfied.

Reinforcement detail of circular column



CROSS SECTION



LONGITUDINAL SECTION

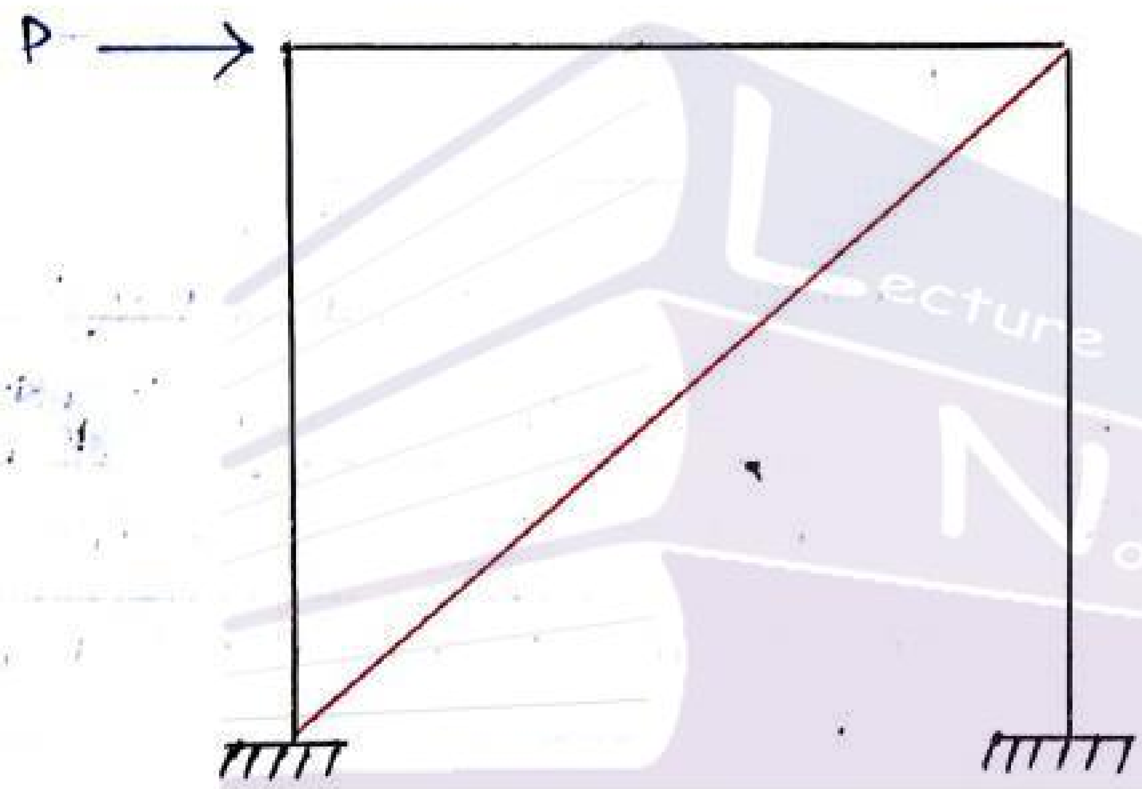
BRACED COLUMN AND UNBRACED COLUMN

Braced column

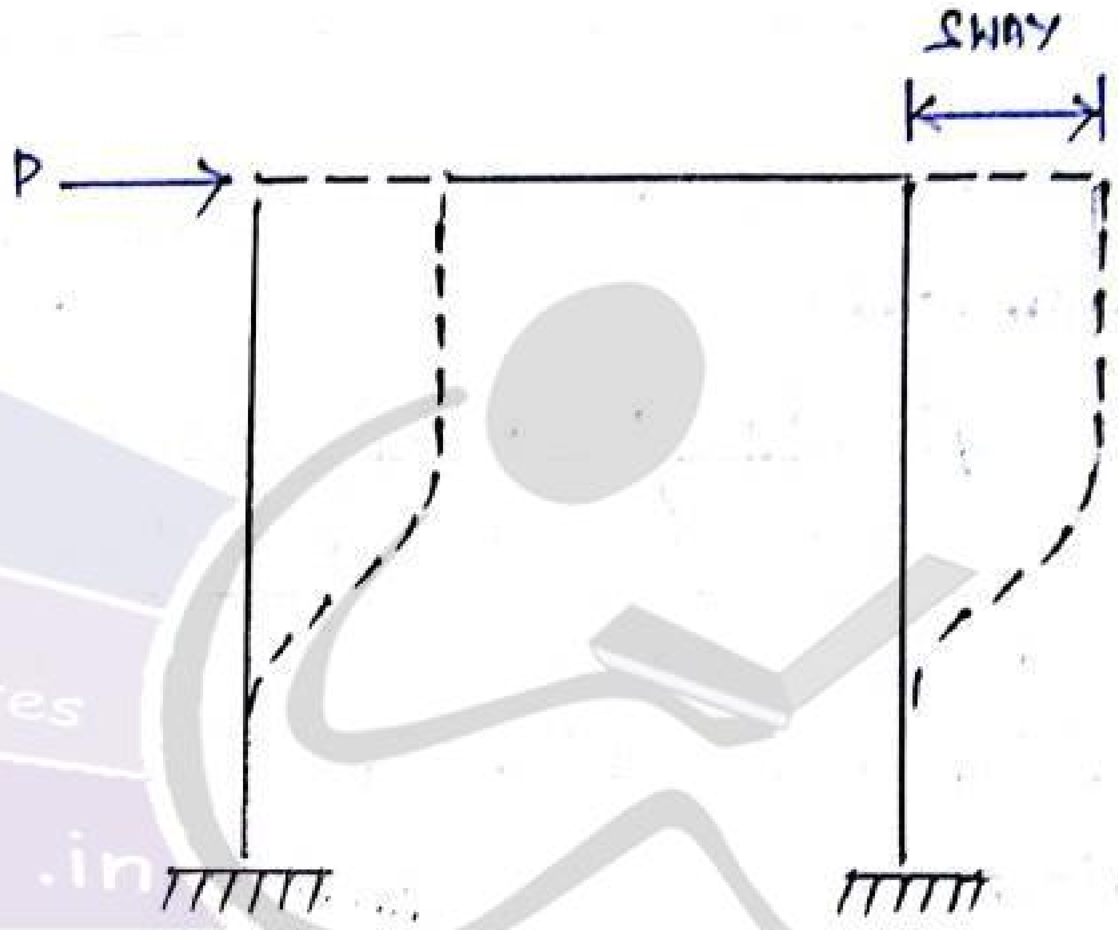
When relative transverse displacement between the upper and lower ends of a column is prevented the frame is said to be braced column

Unbraced column

When relative transverse displacement between the upper and lower ends of a column is not prevented, the frame is said to be unbraced column.



BRACED FRAME



UNBRACED FRAME

Difference between Braced and Unbraced Column

Braced and Unbraced columns are basically vertical member which go throughout the height of the structural system. From (Foundation to Terrace)

S.NO	BRACED COLUMN	UNBRACED COLUMN
1	Braced column may be considered braced in a given plan if lateral stability of the structure as a whole is provided by walls or bracing	Unbraced columns may be considered unbraced in a given plan if lateral stability of the structure as a whole is provided by column only

2.	Braced columns are not designed to resist lateral column	Unbraced columns are designed to resist lateral loads
3	Braced columns have zero value of sway	Unbraced column are subjected to sway
4	Most of steel structures are designed by this method	Most of the RCC structure are design by this method
5.	Braced column are more resistant to Earthquake than Unbraced Column	Unbraced columns are less resistant to earthquake than braced column.

DESIGN OF BRACED COLUMN

Prblm. No: 13

Design the reinforcement in a column of size 400mm by 600mm, subjected to an axial working load of 2000 KN. The column has an unsupported length of 3m and is braced against side sway in both direction. Adopt M20 Grade of Concrete and Fe415 HYSD bars.

Given data

column dimension 400mm × 600mm

Axial working load (P) = 2000 KN

(L) = 3m

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

Step: 1

Slenderness Ratio

$$\lambda = \frac{l_{eff}}{LLD} = \frac{3000}{400} = 7.5 < 12$$

Hence the column is designed as a short column

Step: 2

Minimum Eccentricity

$$e_{x\,min} = \frac{L}{500} + \frac{D}{30} \Rightarrow \frac{3000}{500} + \frac{600}{30}$$

$= 26 > 20mm$

$0.05 \times b = 0.05 \times 600$

$= 30mm$

$e_x min < 0.05b$

Hence ok

$e_y min = \frac{3000}{500} + \frac{400}{30} \Rightarrow 19.33 \; mm > 20mm$

$0.05 \times D = 0.05 \times 400$

$= 20mm$

$e_y min < 0.05D$

<u>Step:3</u>

<u>Longitudinal Reinforcement</u>

$P_u = 0.4 f_{ck} A_g + [0.67 f_y - 0.4 f_{ck}] A_{sc}$

$3000 \times 10^3 = [0.4 \times 20 \times 400 \times 600] + [(0.67 \times 415) - (0.4 \times 20)] A_{sc}$

$= 1.92 \times 10^6 + 270.05 \; A_{sc}$

$A_{sc\,req} = 4000 \; mm^2$

provide 25mm dia

No of bars $= \frac{A_{sc}}{a_{sc}} = \frac{4000}{\frac{\pi}{4} \times 25^2}$

$= 8.14 \simeq 9 \; No's$

provide 6 - 25mm diameter

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

$= 4000 - 2945.24$

$= 1054.75 \; mm^2$

Provide 20mm diameter

No. of bars $= \frac{A_{sc}}{a_{st}} = \frac{1054.75}{\frac{\pi}{4} \times 20^2} \Rightarrow 3.35 \simeq 4 \; No's$

$A_{sc\,pro} = n \times \frac{\pi}{4} \times d^2 \Rightarrow 4 \times \frac{\pi}{4} (20)^2 \Rightarrow 1256.63 \; mm^2$

$$\text{Total } A_{st\,prov} = 2945.24 + 1256.63$$
$$= 4201\,mm^2$$

provide 6 # 25mm diameter bars reinforcement ;

Provide 4 # 20mm dia bars on side reinforcement

Step : 4

Lateral Ties

(i) Ties diameter

$$= \frac{1}{4} \times \phi$$
$$= \frac{1}{4} \times 25 = 6.25\,mm$$

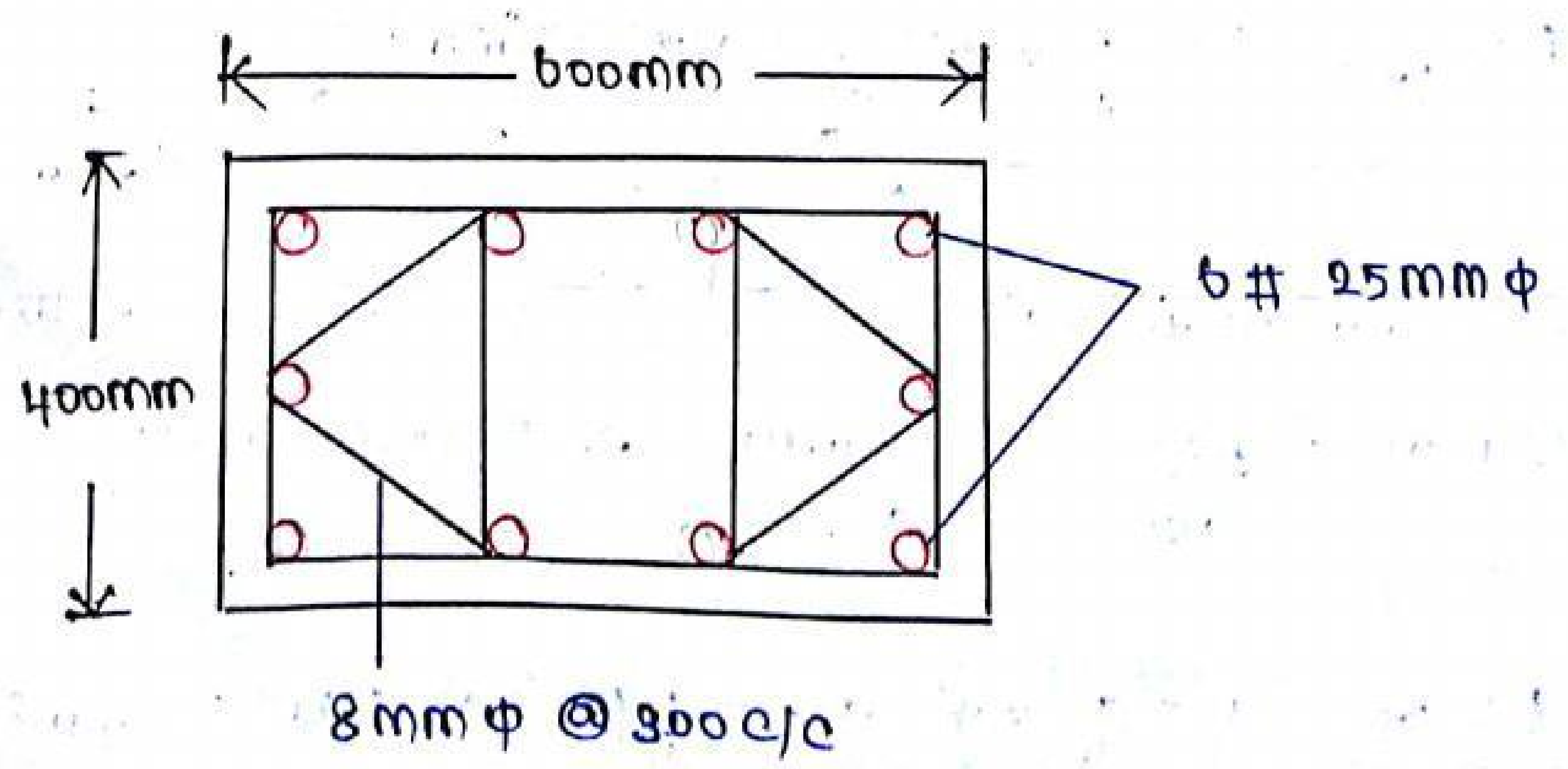
Hence provide 8mm dia tie bars

(ii) Tie spacing

* LLD = 400mm
* $16\phi = 16 \times 25$
$= 400mm$
* $48 \times \phi = 48 \times 8$
$= 384\,mm$
* 300mm

provide 8mm dia Ties at 300 mm c/c

Reinforcement detail of Rectangular column



Prblm. No:14

Design the reinforcement in a circular column of diameter 300mm with helical reinforcement to support a factor load of 1500 KN. The column has an unsupported length of 3m and is braced against sidesway. Adopt M20 Grade of Concrete and Fe 415 HYSD bars

Given data

Diameter of column (D) = 300mm

Unsupported length (L) = 3000mm

Column braced against sidesway

Factor load (Pu) = 1500 KN

f_{ck} = 20 N/mm^2

f_y = 415 N/mm^2

Step: 1

Slenderness Ratio

$$\lambda = \frac{l_{eff}}{LLD} = \frac{3000}{300} = 10 < 12$$

Hence the column design as short column

Step: 2

Minimum Eccentricity (e_{min})

$$e_{min} = \frac{L}{500} + \frac{LLD}{30} \Rightarrow \frac{3000}{500} + \frac{300}{30}$$

$e_{min} = 16 < 20mm$

$0.05D = 0.05 \times 300 = 15mm$

Step: 3

Longitudinal Reinforcement

$$P_u = 1.05 \left[(0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc} \right]$$

$$1500 \times 10^3 = 1.05 \left[0.4 \times 20 \times (\pi/4 \times 300^2) \right] + \left[(0.67 \times 415) - (0.4 \times 20) \right] A_{sc}$$

$$\frac{1500\times10^3}{1.05} = [565486.67 + 270.05\,A_{sc}]$$

$$1428571 - 565486.67 = 270\,A_{sc}$$

$$\boxed{A_{sc}\ req = 3196\ mm^2}$$

using 28 mm diameter bars.

$$(NOB)_{prov} = \frac{3196}{\frac{\pi}{4}\times28^2} = 5.19 \simeq 6\ NO's$$

$$A_{st}\ prov = 6\times\frac{\pi}{4}\times28^2$$
$$= 3694\ mm^2$$

Provide 6 bars of 28 mm ϕ $(A_{sc} = 3694\ mm^2)$

Step: 4

Helical Reinforcement (Condition - I)

Assume,

Provide 8 mm dia bars

$p = 50\,mm$

Clear cover of 40 mm

$$\frac{\text{Volume of helix}}{\text{Volume of core}} \geq 0.36\frac{f_{ck}}{f_y}\left(\frac{A_g}{A_c} - 1\right)$$

$$\text{Mean dia. of helix} = 300 - (2\times40) - \frac{16}{2}$$
$$= 212\,mm$$

$$\left.\begin{array}{l}\text{Length of helix in}\\ \text{per pitch height}\end{array}\right\} = \sqrt{C^2 + p^2}$$

$$= \sqrt{\pi D^2 + p^2}$$

$$= \sqrt{\pi\times212^2 + 50^2}$$

$= 667.89 mm$

- Area of helix $= \frac{\pi}{4} \times \phi^2$

$= \frac{\pi}{4} \times 8^2$

$= 50.27 mm^2$

Vol. of Helix per pitch height $=$ Length x Area

$= 667.89 \times 50.27$

$= 33574.83 \; mm^3$

Dia. of core $=$ Dia of column $-$ (Both side clear cover)

$= 300 - (2 \times 40)$

$= 220 \; mm$

Area of core $= \frac{\pi}{4} \phi^2$

$= \frac{\pi}{4} (220)^2$

$= 38013.27 \; mm^2$

Vol. of core per pitch height $=$ Area of core x P

$= 38013.27 \times 50$

$= 1900663.5 \; mm^3$

$$\frac{\text{Vol. of helix}}{\text{Vol. of core}} = \frac{33574.83}{1900663.5}$$

$= 0.0176$

$$0.36 \frac{f_{ck}}{f_y} \left(\frac{A_g}{A_c} - 1\right) = 0.36 \times \frac{20}{415} \times \left(\frac{70685.834}{38013.27} - 1\right)$$

$= 0.0149$

$0.0176 > 0.0149$

Hence the condition - I is satisfactory

Condition - II

Case - I

Maximum pitch Specified 75mm

$\frac{1}{6} \times \text{core dia} \Rightarrow \frac{1}{6} \times 220 = 36.6\text{mm}$

Case - II

Minimum Pitch 25mm

(i) $\geq$ 25mm

(ii) 3 x dia of helix

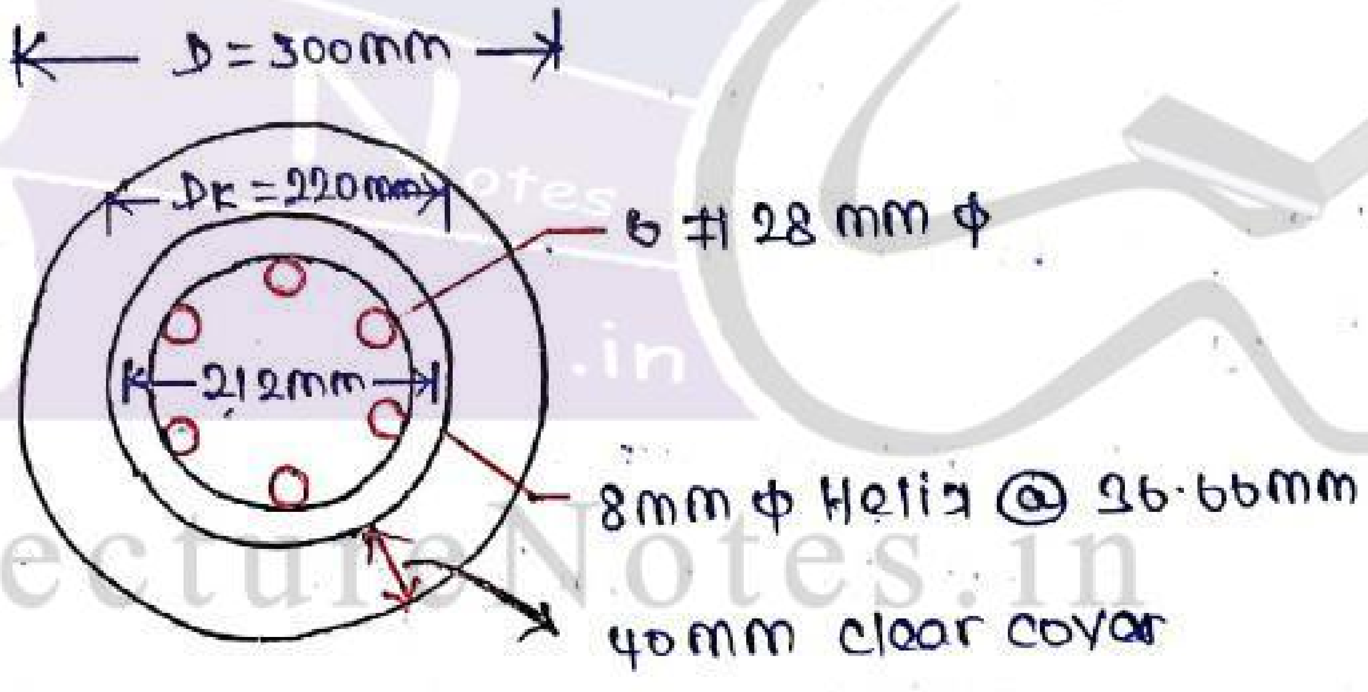
$= 3 \times 8 \Rightarrow 24$

$25 > 24$

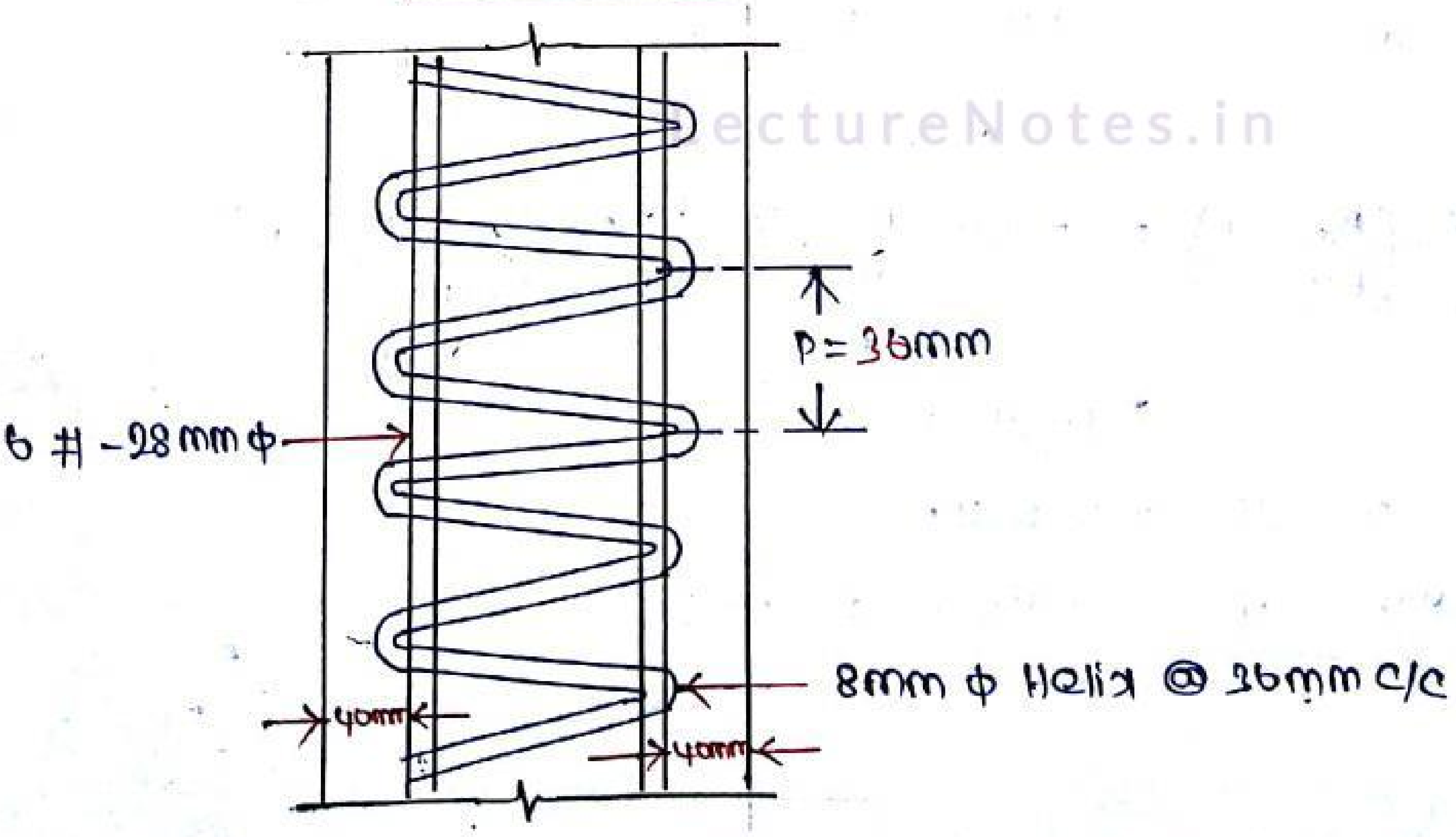
Provide 8mm φ spiral at a pitch of 36mm

Hence the condition - II also satisfactory

Reinforcement detail of circular column



CROSS SECTION



LONGITUDINAL SECTION

DESIGN OF SHORT COLUMN UNDER COMPRESSION WITH UNI-AXIAL BENDING

Uniaxially Loaded Column

When a column is subjected to either combined axial compression (P) and Moment (M) or only Axial Load (P) applied at an eccentricity $e = (M/P)$ so that the column is trying to bend about only one axis of the column cross section is known as Uniaxially loaded column

Design procedure

Step: 1

Check for slenderness Ratio

$$\lambda = \frac{l_{eff}}{L.L.D} < 12$$

Step: 2

Eccentricity $(e > e_{min})$

(i) Actual Eccentricity

$$e = \frac{M_u}{P_u}$$

(ii) Minimum Eccentricity

$$e_{min} = \frac{L}{500} + \frac{D}{30} > 20\,mm$$

Step: 3

Non-Dimensional parameters

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

y axis $\Rightarrow \frac{P_u}{f_{ck} b D}$

The design charts are given in the form of 'Interaction diameter in which $\frac{P_u}{f_{ck} b d}$ Vs $\frac{M_u}{f_{ck} b D^2}$ are plotted for different Values of P/f_{ck}, where p is the percentage reinforcement

→ SP-16 gives charts for designing rectangular section having reinforcement on two sides [Chart 27 to 28] for reinforcement on four sides [Charts 39 to 50]

→ But to use these charts minimum of 8 bars should be selected distributing equally on the four faces

→ The charts for circular section [chart 51 - 62]

Step: 4

Longitudinal Reinforcement

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

$$P = XX$$

$$A_{sc} = \frac{2}{100} \times A_g$$

Step: 5

Ties Reinforcement

(i) Tie diameter = $\frac{1}{4}$ x (Diameter of longitudinal reinforcement)

(ii) pitch

(i) $16 \times \phi$

(ii) 300mm.

Prblm. No: 15

Design the Longitudinal and Lateral reinforcement in a rectangular reinforcement Concrete Column of size 300mm by 400mm has effective length of 3.6m Subjected to a design Ultimate Load of 1200kN and Ultimate Moment of 200 kN·m with respect to the Major axis. Adopt M20 Grade of concrete and Fe 415 Grade HYSD bars

Given data

$b = 300mm, \; D = 400mm$

$P_u = 1200\,kN, \; M_u = 200\,kN \cdot m$

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

Step: 1

Slenderness Ratio

$L = 3.6m \Rightarrow 3600mm, \; D = 400mm$

$$\frac{L}{D} = \frac{3600}{400} = 9 < 12$$

Hence it can be designed as Short column

Step: 2

Eccentricity

(i) Actual Eccentricity

$$e = \frac{M_u}{P_u} = \frac{1200 \times 10^6}{1200 \times 10^3} \Rightarrow 166.67mm$$

(ii) Minimum Eccentricity

$$e_{min} = \frac{L}{500} + \frac{D}{30} \Rightarrow \frac{3600}{500} + \frac{400}{30} \Rightarrow 20.53mm$$

$e > e_{min}$

Hence design its as short column subject to axial load and Uniaxial Moment

Step: 3

Non-Dimensional Parameters

$$x\text{-axis} = \frac{Mu}{f_{ck} b D^2} = \frac{200 \times 10^6}{20 \times 300 \times 400^2} = 0.20$$

$$y\text{-axis} = \frac{Pu}{f_{ck} b D} = \frac{1200 \times 10^3}{20 \times 300 \times 400} = 0.50$$

$$\frac{d'}{D} = \frac{50}{400} = 0.125 \text{ nearly equal to } 0.15$$

Adopting an effective cover of 50mm = d'

Step: 4

Longitudinal Reinforcement

Refer Chart-33 of SP:16 and Read out the ratio $\frac{p}{f_{ck}} = 0.20$

$$\frac{p}{f_{ck}} = 0.20 \Rightarrow p = 20 \times 0.20$$

$$p = 4$$

$$A_{SC} = \frac{4}{100} \times 300 \times 400$$

$$= 4800 \text{ mm}^2$$

provide 25mm diameter of 8 bars ($A_{SC} = 3927 \text{ mm}^2$).

Step: 5

Ties Reinforcement

(i) Tie diameter

$$\frac{1}{4} \times \phi \Rightarrow \frac{1}{4} \times 25$$

$$= 6.25 \text{mm} \simeq 7 \text{mm}$$

Provide 8mm ϕ ties

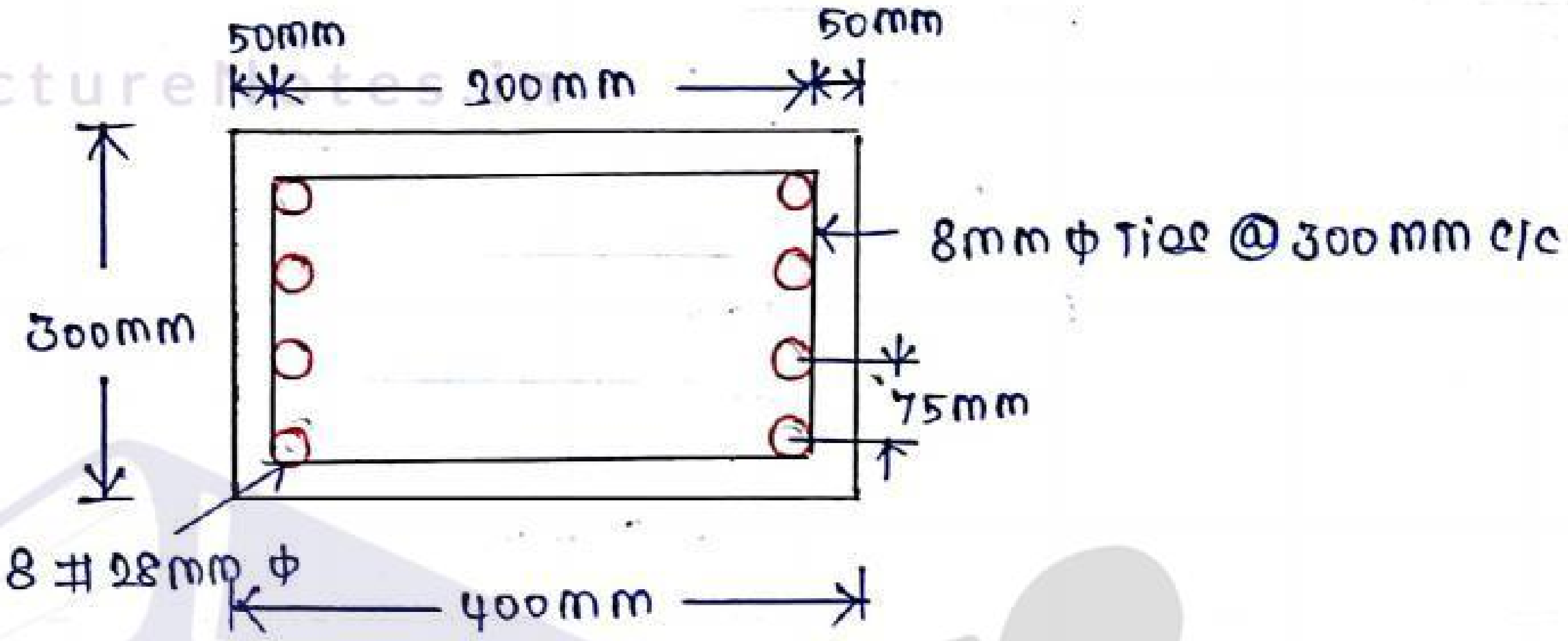
(ii) Pitch (or) Spacing

* $16\phi = 16 \times 25 \Rightarrow 400mm$

* 300mm

Hence provide 8mm dia ties at 300mm c/c

Detailing of Reinforcement



Problm: No: 16

Design a Short circular column of diameter 400mm to support a Factored axial Load of 900 kN, Together with a Factored moment of 100 kN·m. Adopt M20 Grade Concrete and Fe 415 Grade reinforcement.

Given data

$D = b = 400mm$ Assume $d' = 40mm$

$P_u = 900kN$, $M_u = 100 kN \cdot m$ $\frac{d'}{D} = 0.10$

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

Step: 1

Non-Dimensional Parameters

(Y-axis) $\frac{P_u}{f_{ck} D^2} \Rightarrow \frac{900 \times 10^3}{20 \times 400^2} = 0.28$

x-axis $\frac{Mu}{f_{ck}D^2} = \frac{100\times10^6}{20\times400^2} = 0.078$

Step: 2

Longitudinal Reinforcement

Refer chart-56, Sp-16 and read out the values of the parameter

$$\frac{P}{f_{ck}} = 0.10$$

$$= 20\times0.10$$

$$P = 2$$

$$Asc = \frac{P\pi D^2}{400}$$

$$= \frac{2\times\frac{\pi}{4}\times400^2}{400} = 2512\ mm^2$$

Provide 25mm ϕ

$$NoB = \frac{Asc}{asc} = \frac{2512}{\frac{\pi}{4}\times25^2} = 5.11 \simeq 6\ No's$$

$$Asc_{prov} = n\times\frac{\pi}{4}d^2 = 6\times\frac{\pi}{4}(25)^2 = 2945\ mm^2$$

Provide 6 No's bars and of 25mm dia ($Asc = 2945\ mm^2$)

Step: 3

Lateral Ties

(i) Tie diameter.

$$= \frac{1}{4}\times25 = 6.25\ mm$$

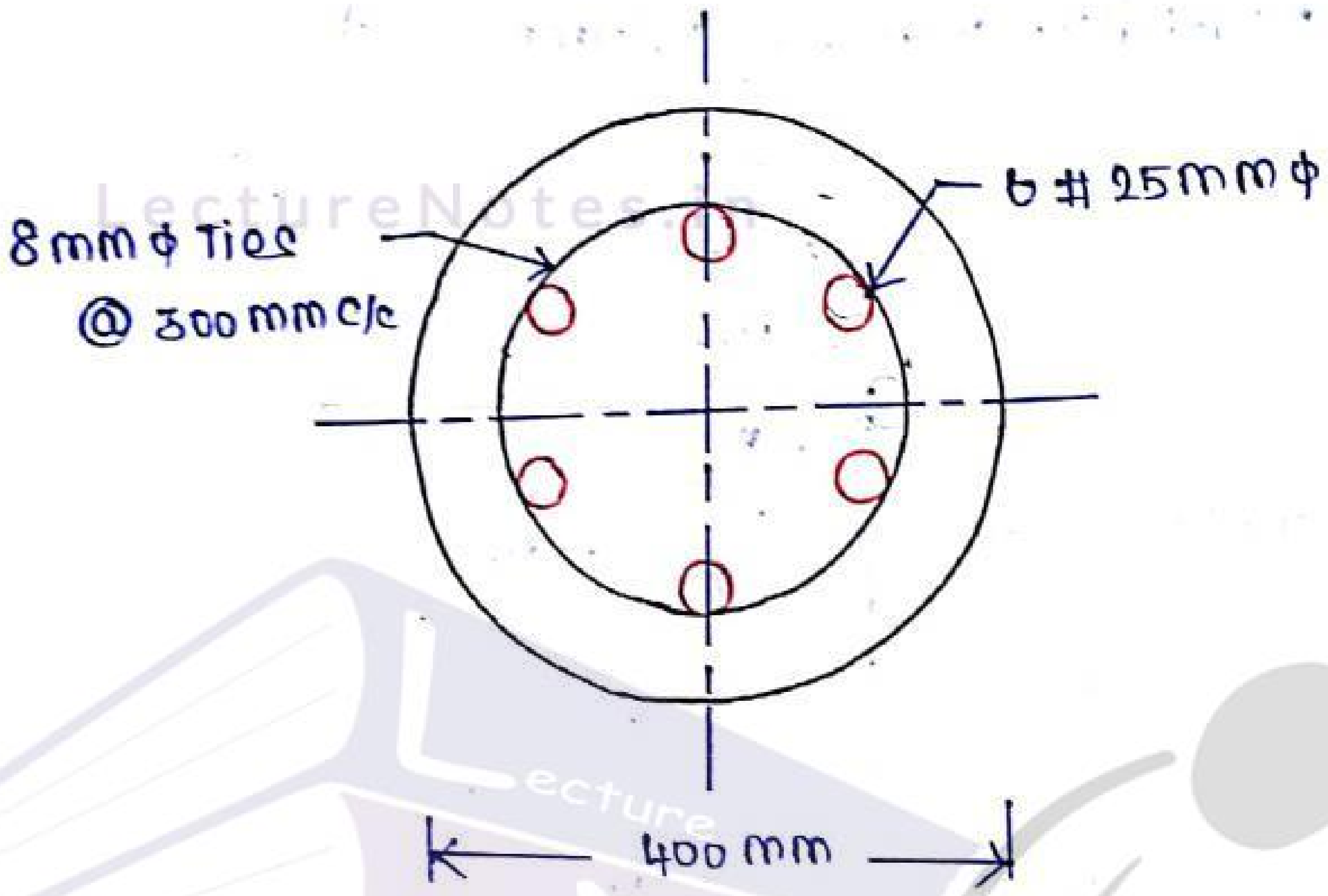
Provide 8mm diameter

(ii) Ties pitch

* $16\phi = 16 \times 25 = 400mm$

* 300mm

Provide 8mm dia ties at 300mm c/c

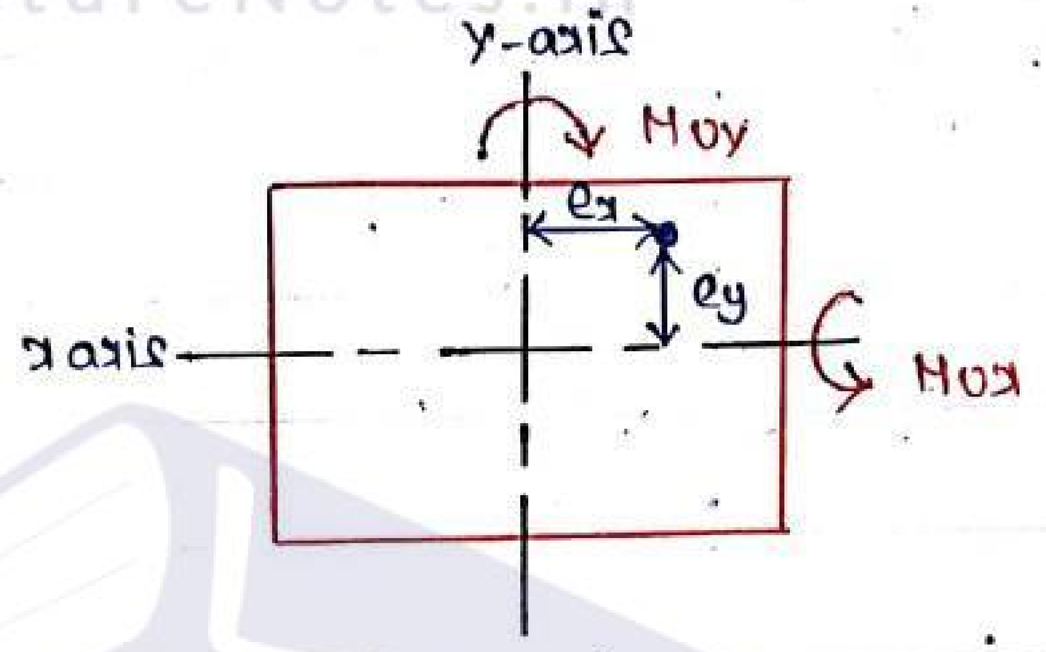


Reinforcement In Circular Column

DESIGN OF SHORT COLUMNS UNDER COMPRESSION AND BIAXIAL BENDING

DEFINITION

The column having axial load acting in such a way that the load is eccentric about both the axes in the plane of the column that is called Biaxially loaded column



$$\left[\frac{M_{ox}}{M_{ox1}}\right]^{\alpha n} + \left[\frac{M_{oy}}{M_{oy1}}\right]^{\alpha n} \le 1.0$$

Where,

M_{ox}, M_{oy} = Moment about x and y-axes due to design loads

M_{ox1}, M_{oy1} = Maximum Uniaxial moment capacities with an axial load P_u bending about x and y axes respectively

αn is related to $\frac{P_u}{P_{uz}}$

$$P_{uz} = 0.45\, f_{ck}\, A_c + 0.75\, f_y \cdot A_{sc}$$

Design Procedure

Step : 1

Equivalent Moment

The reinforcement in section is designed for the axial compressive load Pu and the equivalent moment given by the relation

$$M_u = 1.15\sqrt{M_{ux}^2 + M_{uy}^2}$$

Step: 2

Non-Dimensional parameters

x-axis = $\frac{M_u}{f_{ck} b D^2}$ $\quad d'/D$

y-axis = $\frac{P_u}{f_{ck} bd}$

Step: 3

Longitudinal Reinforcement

Refer chart of SP-16 (Equal reinforcement on all forces) with $\frac{d'}{D}$ and read out the value of P/f_{ck}

$$A_{sc\,req} = \frac{P}{100} \times bD$$

$A_{sc\,prov}$ =

$$P = \frac{A_{sc}}{bD} \text{ and Find out the } \frac{P}{f_{ck}} =$$

Provided actual 'P' provided and hence P/f_{ck} for $(P_u/f_{ck}bD)$ given and P/f_{ck} provided and determine. M_{ux1}

Step: 4

Determine Puz using the relation

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

$$A_c = A_g - A_{sc}$$

To Find α_n corresponding to P_u/P_{uz}

$\frac{P_u}{P_{uz}}$	α_n
≤ 0.2	1.0
≥ 0.8	2.0

Step: 5

Check for Safety under Biaxial Loading

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

Note:

* check whether Interaction Formula is Satisfied
* If Interaction formula is not satisfied Increase P and Pu again

Prblm. No: 17

Design the Reinforcement of Short column 400mm x 500mm with axial load $P_u = 2000$ kN, $M_{ux} = 130$ kN·m, $M_{uy} = 120$ kN·m Unsupported length = 3.2m. Use M25 Grade of Concrete and Fe415 Grade of Steel

Given data

$L = 3200$ mm ; $P_u = 2000$ kN $f_{ck} = 20$ N/mm^2

$b = 400$ mm ; $M_{ux} = 130$ kN·m $f_y = 415$ N/mm^2

$D = 500$ mm ; $M_{uy} = 120$ kN·m

Step: 1

Slenderness Ratio

$$\lambda = \frac{l_{ex}}{D} = \frac{3200}{500} = 6.4 < 12$$

$$\lambda = \frac{l_{ey}}{b} = \frac{3200}{400} = 8 < 12$$

Hence it can be design as short column

Step: 2

Eccentricity (e)

(i) Actual Eccentricity

$$e_x = \frac{M_{ux}}{P_u} = \frac{130 \times 10^6}{2000 \times 10^3} = 65mm$$

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

(ii) Minimum Eccentricity

$$e_{minx} = \frac{L}{500} + \frac{D}{30} \Rightarrow \frac{3200}{500} + \frac{400}{30} = 19.73 \simeq 20mm$$

$$e_{miny} = \frac{L}{500} + \frac{D}{30} \Rightarrow \frac{3200}{500} + \frac{500}{30} = 23.07$$

$$e > e_{min}$$

Hence OK

Step: 3

Equivalent Moment

$$M_u = 1.5\sqrt{M_{ux}^2 + M_{uy}^2}$$

$$= 1.5\sqrt{130^2 + 120^2}$$

$$\boxed{M_u = 203.456 \text{ kN·m}}$$

Step: 4

Non-Dimensional Parameters

$$x\text{-axis} \Rightarrow \frac{M_u}{f_{ck} b D^2} = \frac{203.456 \times 10^6}{25 \times 400 \times 500^2} = 0.0814$$

$$y\text{-axis} \Rightarrow \frac{P_u}{f_{ck} b D} = \frac{2000 \times 10^3}{25 \times 400 \times 500} = 0.4$$

$$\frac{d'}{d} = \frac{40}{500} = 0.08$$

Nearly equal to 0.10

Step:5

Longitudinal Reinforcement

Refer SP-16: 1987 chart No :- 45, 44

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

$$p = 0.06 \times 25$$

$$p = 1.5\%$$

$$A_{sc} = \frac{1.5}{100} \times b \times d \Rightarrow \frac{1.5 \times 400 \times 500}{100}$$

$$A_{sc\,req} = 3000\,mm^2$$

Use 20mm ϕ bars

$$\text{No. of bars} = \frac{A_{sc\,req}}{a_{sc}} \Rightarrow \frac{3000}{\frac{\pi}{4} \times 20^2} \Rightarrow 9.54 \simeq 10\text{No's}$$

$$A_{sc\,prov} = 10 \times \frac{\pi}{4} \times 20^2 \Rightarrow 3141.59\,mm^2$$

$$p = \frac{100 \times A_{sc\,prov}}{b \times D}$$

$$= \frac{100 \times 3141.59}{400 \times 500}$$

$$= 1.57\%$$

$$\frac{p}{f_{ck}} = \frac{1.57}{20} = 0.07$$

Refer chart -44 (SP-16) and read out the Ratio

$\left(\frac{M_{u x 1}}{f_{ck} b D^2}\right)$ corresponding the value of $\left(\frac{P_u}{f_{ck} b D}\right) = 0.4$ and $\frac{p}{f_{ck}} = 0.07$

Step: 6

Determine the M_{ux1} & M_{uy1}

$$\frac{M_{ux1}}{f_{ck} b D^2} = 0.08$$

$$M_{ux1} = 0.08 \times f_{ck} \times b \times D^2$$

$$= 0.08 \times 25 \times 400 \times 500^2$$

$$= 225 \text{ KN}\cdot\text{m} > M_{ux}$$

$$\frac{M_{uy1}}{f_{ck} D b^2} = 0.08$$

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

$$= 0.08 \times 25 \times 500 \times 400^2$$

$$= 180 \text{ KN}\cdot\text{m} > 120 \text{ KN}\cdot\text{m}$$

Hence ok

Step: 7

$$P_{uz} = 0.45 f_{ck} A_c + 0.45 f_y A_{sc}$$

$$= [0.45 \times 25 \times (A_g - A_{sc})] + [0.45 \times 415 \times 3141.59]$$

$$= [0.45 \times 25 \times (200 \times 10^3 - 3141.59)] + [0.45 \times 415 \times 3141.59]$$

$$= 2358.41 \times 10^3 \text{ N}$$

$$\boxed{P_{uz} = 2358.41 \text{ KN}}$$

$$\alpha_n = \frac{P_u}{P_{uz}} = \frac{2000}{2358.41} = 0.81$$

Step: 8

Check for Safety under biaxial loading

$$\left[\frac{M_{ux}}{M_{ux1}}\right]^{\alpha_n} + \left[\frac{M_{uy}}{M_{uy1}}\right]^{\alpha_n} \leq 1.0$$

$$\left(\frac{120}{225}\right)^2 + \left(\frac{120}{180}\right)^2 \leq 1.0$$

$$0.77 \leq 1.0$$

Hence it is satisfied

Step: 9

Tie Reinforcement

(i) Dia of the bar $= \frac{1}{4} \times$ Long dia

$= \frac{1}{4} \times 20$

$= 5 \leq 6\text{mm}\,\phi$

Provide 6mm ϕ as Tie bars

(ii) Pitch of the Tie bars

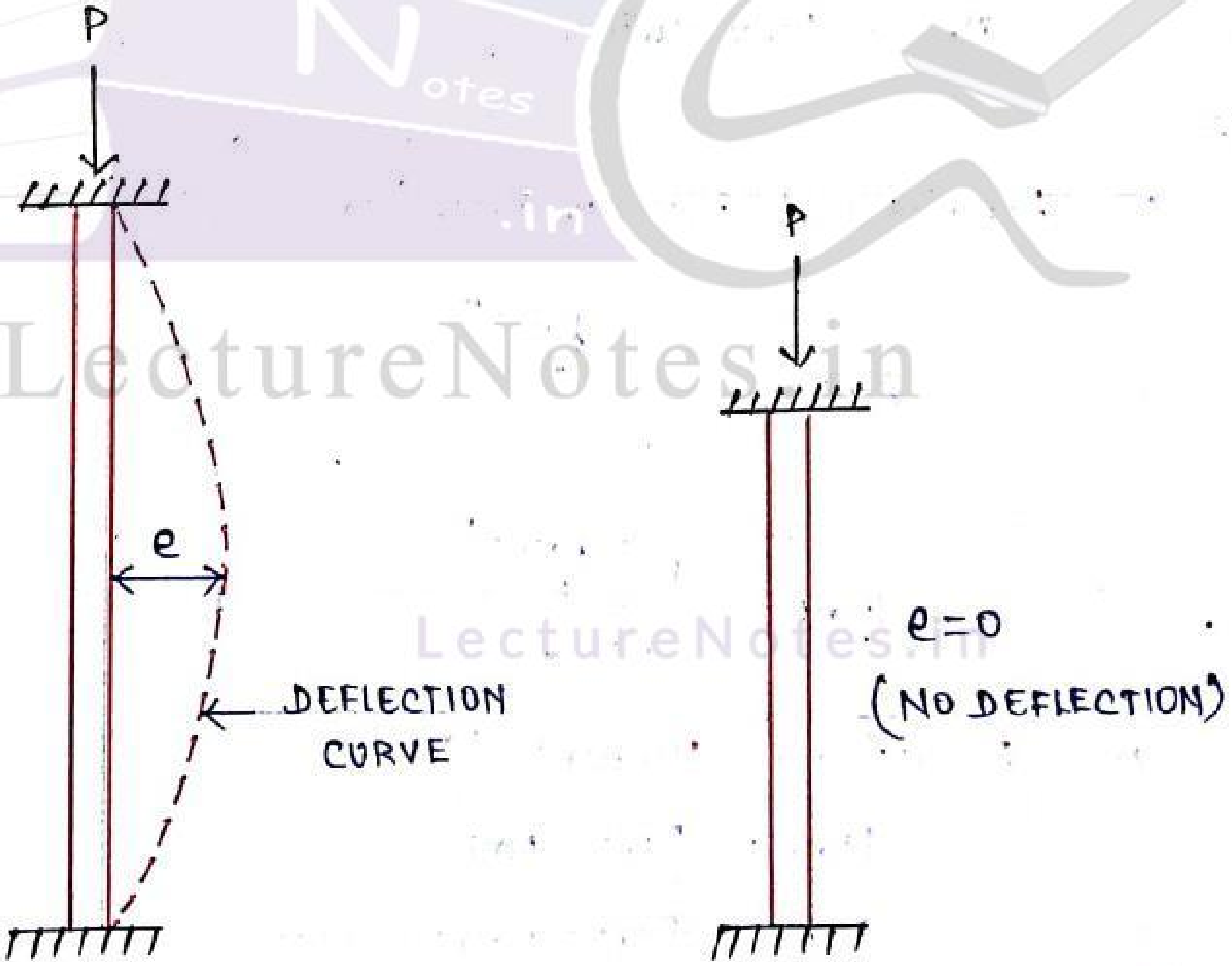
* $16\phi = 16 \times 20 = 320\text{mm}$
* $48\phi = 48 \times 6 = 288\text{mm}$
* 300mm

Provide 6mm ϕ @ 300 mm c/c spacing

SLENDER COLUMN

Introduction

* Compression members having the ratio of effective length to its least lateral dimension (slenderness ratio) exceeding 12 are categorized as slender or long columns according to IS 456-2000 code
* The deformation characteristic of slender column are significantly different from that of short column
* When slender columns are loaded even with axial loads the lateral deflection is significantly greater in comparision with short column.
* Consequently in slenders columns, the moment produced by the deflection is large and should be considered in design



LONG COLUMN SHORT COLUMN

Design Procedure of Slender Column

Step: 1

Slenderness Ratio

$$\lambda = \frac{l_{ex}}{D} > 12$$

$$\lambda = \frac{l_{ey}}{b} > 12$$

Step: 2

(i) Minimum Eccentricity (e_{min})

$$e_{minx} = \frac{L}{500} + \frac{D}{30}$$

$$e_{miny} = \frac{L}{500} + \frac{b}{30}$$

(ii) Actual Moment (M_u)

$$M_{ux} = P_u \times e_{minx}$$

$$M_{uy} = P_u \times e_{miny}$$

Step: 3

(i) Additional Eccentricity (e_a)

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

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

(ii) Additional Moment (M_a)

$$M_{ax} = P_u \times e_{ax}$$

$$M_{ay} = P_u \times e_{ay}$$

Step: 4

Determine Modification Factor (K)

The above moment have to be multiplied by Modification Factor (K)

$$\boxed{K = \frac{P_{uz} - P_u}{P_{uz} - P_b}}$$

$$P_{uz} = 0.45 f_{ck} \cdot A_c + 0.75 f_y \cdot A_{sc}$$

$$A_c = A_g - A_{sc}$$

$$A_{sc} = \frac{p}{100} \times A_g$$

(Assume, percentage of reinforcement For the First Trail.)

Calculate the P_b

$$\frac{P_{bx}}{f_{ck} bD} = K_1 + K_2 \left(\frac{p}{f_{ck}}\right)$$

Depending upon $\frac{d'}{D} = xx$ Read out the values of K_1 and K_2

$K_1, K_2 \rightarrow$ (Refer SP:16-1978 Pg. No: 71, Table. No: 60.)

$M_{ax} = xx$

$M_{ay} = xx$

Step: 5

Initial Moment acting on the Column Should be Modified

$$M_{ux} = 0.6 M_{x1} - 0.4 M_{x2}$$

$$M_{uy} = 0.6 M_{y1} - 0.4 M_{y2}$$

→ The Moment are Compared with the moment due to min. eccentricity

→ Total Moment for which the Column is to designed are

$$M_{ux} = M_{ax} + M_{ux}$$

$$M_{uy} = M_{ay} + M_{uy}$$

Step: 6

Non - Dimensional Parameter

$$\frac{P_u}{f_{ck} bD}$$

$\frac{d'}{D} = xx$ (For M_{ux} calculation)

$\frac{d'}{b} = xx$ (For M_{uy} calculation)

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

Step: 7

Determine M_{ux1}, M_{uy1}

$\frac{M_{ux1}}{f_{ck} b D^2} = xx$

$\frac{M_{uy1}}{f_{ck} b^2 D} = xx$

$\alpha_n = 0.67 + 1.65 \left(\frac{P_u}{P_{uz}}\right)$

Step: 8

Check for Biaxial bending

$$\left[\frac{M_{ux1}}{M_{ux}}\right]^{\alpha_n} + \left[\frac{M_{uy1}}{M_{uy}}\right]^{\alpha_n} \le 1$$

Step: 9

Longitudinal Reinforcement

A_{sc} req =

A_{sc} prov =

Step: 10

Lateral Ties

(i) Ties dia

$= \frac{1}{4} \times \phi$

(ii) Tie Spacing

* 16ϕ
* 300 mm

Prblm·NO:18

Design a column of size 450mm×300mm using M30 concrete and Fe 415 steel. Given l_{ex} = 6cm, l_{ey} = 5·5cm, P_u = 1600 kN. Factored Moment about Major axis = 45 kN·m at Top and 30kN·m at bottom. Factored bottom about Minor axis = 40 kN·m at Top and 25 kN·m at bottom. Column is bent in double curvature and reinforcement is distributed equally on all the Four sides of the Section.

Given data

Size of column

$D = 450mm$, $b = 300mm$

$l_{ex} = 6 \cdot 0mm$, $l_{ey} = 5 \cdot 5m$, $P_u = 1600\,kN$

$M_{ux} = M_{ux1} = 45\,kN \cdot m$, $M_{ux2} = 30\,kN \cdot m$

$M_{uy} = M_{uy1} = 40\,kN \cdot m$, $M_{uy2} = 25\,kN \cdot m$

$f_{ck} = 30\,N/mm^2$, $f_y = 415\,N/mm^2$

Step: 1

Slenderness Ratio (λ)

$$\lambda = \frac{l_{ex}}{D} = \frac{6000}{450} = 13 \cdot 33 > 12$$

$$\lambda = \frac{l_{ey}}{b} = \frac{5500}{300} = 18 \cdot 33 > 12$$

Hence the column is Slender about both axis

Step: 2

Minimum Eccentricity

$$e_{minx} = \frac{L}{500} + \frac{D}{30}$$

$$= \frac{6000}{500} + \frac{450}{30}$$

$$= 27mm > 20mm$$

$$e_{miny} = \frac{L}{500} + \frac{b}{30}$$

$$= \frac{5500}{500} + \frac{300}{30}$$

$$= 21mm > 20mm$$

(ii) Actual Moment

$M_{ux} = P_u \times e_{minx}$

$= 1600 \times 27$

$= 43.2$ KN·m

$M_{uy} = P_u \times e_{miny}$

$= 1600 \times 21$

$= 33.6$ KN·m

Step: 3

(i) Additional Eccentricity

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

$$= \frac{450\left(\frac{6000}{450}\right)^2}{2000}$$

$= 40$ mm

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

$$= \frac{300\left(\frac{5500}{300}\right)^2}{2000}$$

$= 50.41$ mm

(ii) Additional Moment

$M_{ax} = P_u \times e_{ax}$

$= 1600 \times 40$

$= 64.08$ KN·m

$M_{ay} = P_u \times e_{ay}$

$= 1600 \times 50.41$

$= 80.64$ KN·m

Step: 4

To Determine Modification Factor

The above moments have to be multiplied by Modification factor (K)

$$K = \frac{P_{uz} - P_u}{P_{uz} - P_b}$$

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

$A_c = A_g - A_{sc}$

Assume, the percentage of reinforcement $P = 3$ (For First Trial)

$A_g = 450 \times 300$

$= 135 \times 10^3$ mm^2

$$A_{sc} = \frac{p}{100} \times A_g$$

$$= \frac{3}{100} \times 135\times10^3 = 4050\,mm^2$$

$$= 135\times10^3 - 4050$$

$$A_c = 130.950\times10^3\,mm^2$$

$$P_{uz} = 0.45\times30\times(130.950\times10^3) + 0.75\times415\times4050$$

$$= 3028.39\times10^3\,N$$

$$P_{uz} = 3028.39\,kN$$

$$\frac{P_b}{f_{ck}bD} = K_1 + K_2\left(\frac{p}{f_{ck}}\right)$$

K_1 & K_2 → Depending upon $\frac{d'}{D}$ read out the values

For Table - 60 of SP-16 (pg. No: 171)

Assume,

Effec. Cover = 50mm

$$\frac{d'}{D} = \frac{50}{450} = 0.111 \simeq 0.15$$

$$K_1 = 0.196,\quad K_2 = 0.203$$

$$\frac{P_b}{f_{ck}bD} = K_1 + K_2\times\left(\frac{p}{f_{ck}}\right)$$

$$= 0.196 + 0.203\times\left(\frac{3}{30}\right)$$

$$= 0.216\times30\times300\times450$$

$$= 876.01\times10^3\,N$$

$$P_b = 876.01\,kN$$

$$K_x = K_y = \frac{3028.39 - 1600}{3028.39 - 876.01}$$

$$K_x = K_y = 0.663$$

$M_{ax} = 64.08 \times 0.662 = 42.48$ kN·m

$M_{ay} = 80.64 \times 0.668 = 53.86$ kN·m

Step: 5

Initial Moment acting on the Column Should be modified

$M_{ux} = (0.6\, M_{x1}) - (0.4\, M_{x2})$

$= (0.6 \times 45) - (0.4 \times 30)$

$M_{ux} = 15$ kN·m

$M_{uy} = (0.6\, M_{y1}) - (0.4\, M_{y2})$

$= (0.6 \times 40) - (0.4 \times 25)$

$= 14$ kN·m

→ These moment are to be compared with the moment due to moment due to Minimum Eccentricity

→ The Total Moment for which the column is to designed are

$M_{ux} = M_{ax} + M_{ux}$

$= 42.8 + 42.8$

$= 85.66$ kN·m

$M_{uy} = M_{ay} + M_{uy}$

$= 53.86 + 33.6$

$= 87.46$ kN·m

Step: 6

Non - Dimensional Parameters

$\frac{P_u}{f_{ck} b D} = \frac{1600 \times 10^3}{30 \times 300 \times 450} = 0.395$

$\frac{d'}{D} = \frac{50}{450} = 0.15$ (For M_{ux} calculation)

$\frac{d'}{b} = \frac{50}{300} = 0.2$ (For M_{uy} Calculation)

Step: 7

Longitudinal Reinforcement

$$P = 3$$

$$\frac{P}{f_{ck}} = \frac{3}{30} = 0.1$$

$$Asc_{req} = 4050\ mm^2$$

Provide 22 mm ϕ bars

$$NOB = \frac{Asc}{asc} = \frac{4050}{\frac{\pi}{4} \times (22)^2} = 12\ NO'S$$

$$Asc_{prov} = n \times \frac{\pi}{4} \times d^2 \Rightarrow 12 \times \frac{\pi}{4} \times (22)^2 = 4561\ mm^2$$

Provide 12 No's of 22 mmϕ ($Asc = 4561\ mm^2$)

Step: 8

Determine M_{ux1}, M_{uy1}

$$\frac{M_{ux1}}{f_{ck} b D^2} = 0.11$$

Refer SP : 16
Chart · No : 45

$$= 0.11 \times 30 \times 300 \times 450^2$$

$$= 200.475 \times 10^6\ N \cdot mm$$

$$= 200.475\ KN \cdot m$$

$$\frac{M_{uy1}}{f_{ck} b D^2} = 0.1$$

$$= 0.1 \times 30 \times 300 \times 450^2$$

$$= 182.25 \times 10^6\ N \cdot mm$$

$$= 182.25\ KN \cdot m$$

$$\alpha_n = 0.67 + 1.67 \left(\frac{P_u}{P_{uz}}\right)$$

$$= 0.67 + 1.67 \left(\frac{1600}{3028.39}\right)$$

$$\alpha_n = 1.547$$

Hence the interaction formula

Step: 9

Check for Biaxial bending

$$\left[\frac{85.68}{200.475}\right]^{1.547} + \left[\frac{87.46}{182.25}\right]^{1.547} \le 1$$

$= 0.828$

Hence the design is safe

Step: 10

Lateral Ties

(i) Ties dia.

$\frac{1}{4} \times \phi = \frac{1}{4} \times 22$

$= 5.5$

Provide 8mm ϕ bars

(ii) Ties Spacing

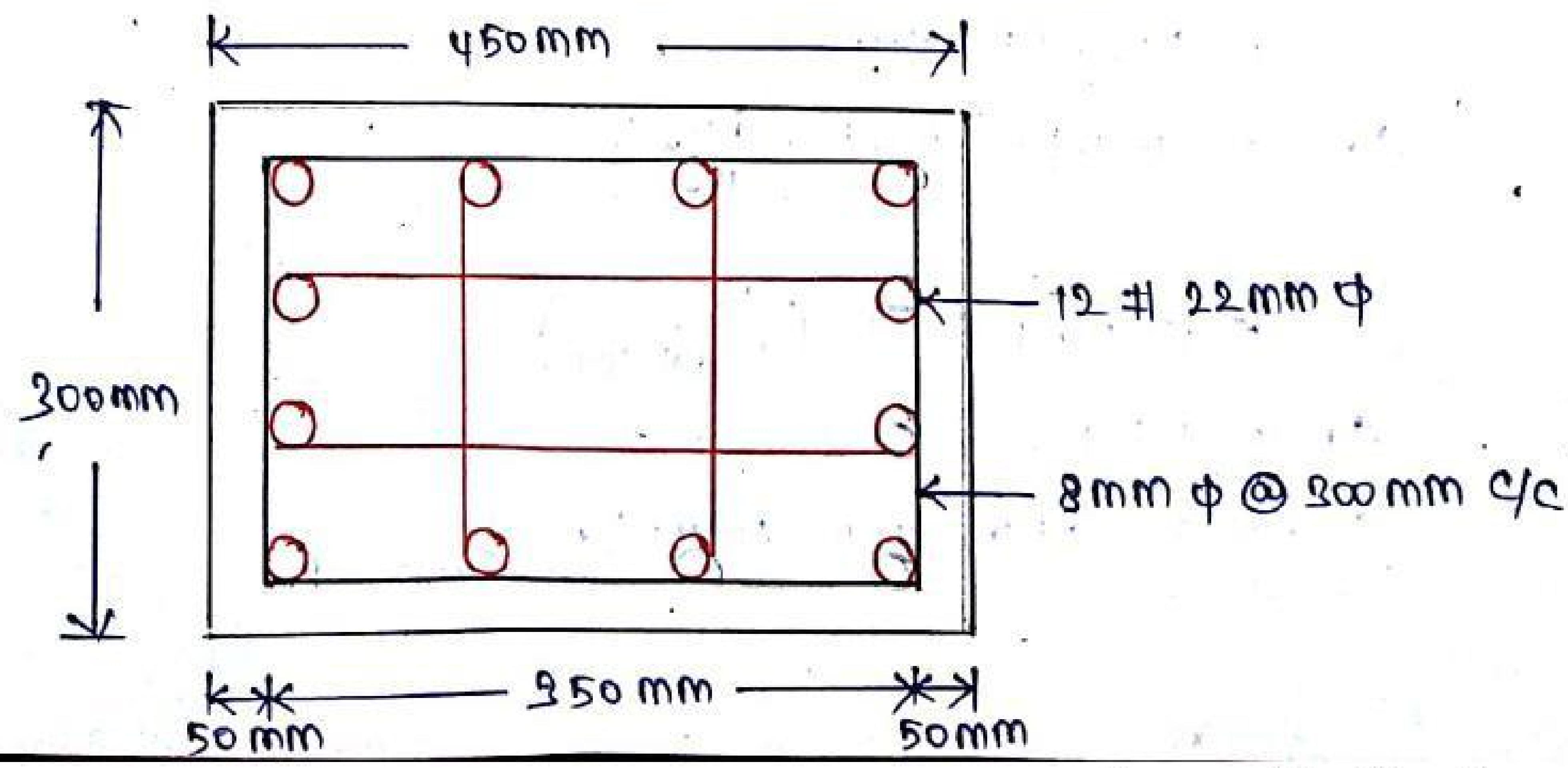
* $16\phi = 16 \times 22$

$= 352$ mm

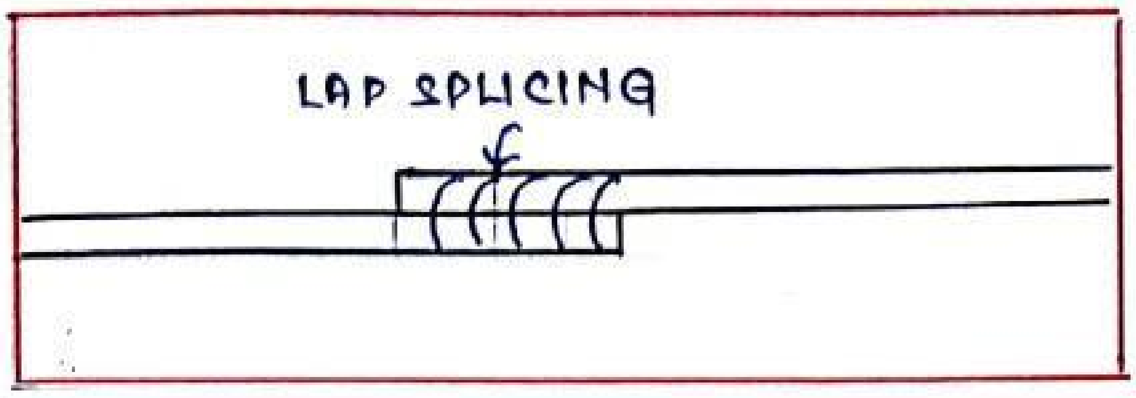
* 300mm

Provide 8mm ties at 300mm c/c

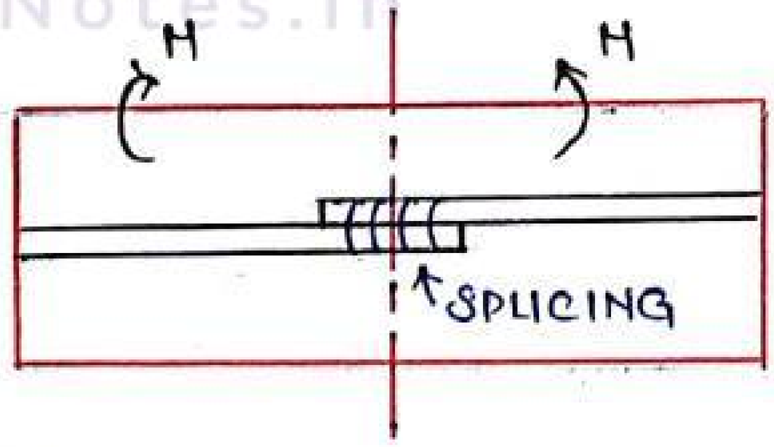
Reinforcement details of Slender Columns



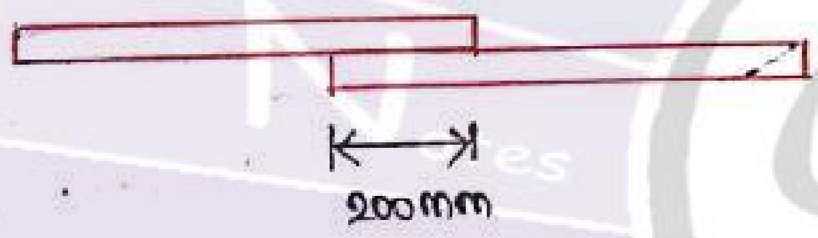
LAPPING LENGTH



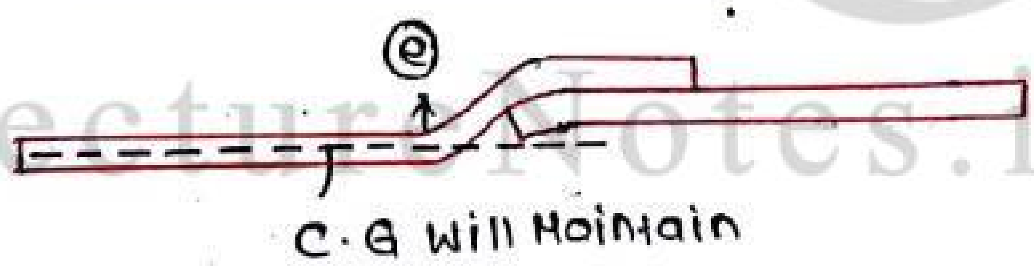
* Moment at Location of Splices should not more than 50% of capacity



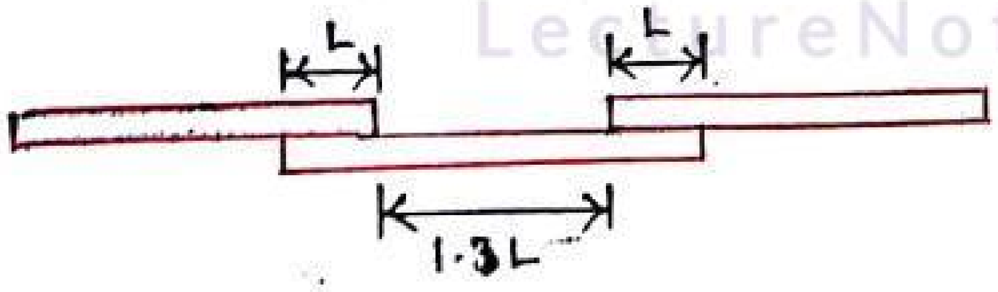
* No more than 50% bar should be spliced at same location
* Minimum lap length is 15φ or 200 mm



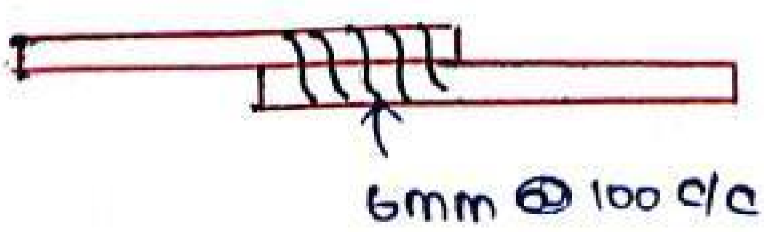
* Bars are bent slightly to main Eccentricity



* If L is the length of splice the distance b/w two splice should be greater than 1.3L



* Bars with dia>36mm cannot be spliced, but welding is used Instead



* Spiral use to Toe together has a dia of min 6 mm with 100 c/c