MOHAMED SATHAK A.J. COLLEGE OF ENGINEERING
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CE8501 DESIGN OF REINFORCED CEMENT CONCRETE ELEMENTS

III CIVIL

Mrs.V.JANAKI
Assistant Professor

DEPARTMENT OF CIVIL ENGINEERING

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SYLLABUS

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UNIT II DESIGN OF BEAMS 9+ 6

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UNIT III DESIGN OF SLABS AND STAIRCASE 9+6

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UNIT IV DESIGN OF COLUMNS 9+6

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UNIT V DESIGN OF FOOTINGS 9+6

Concepts of Proportioning footings and foundations based on soil properties-Design of wall footing – Design of axially and eccentrically loaded Square, Rectangular pad and sloped footings – Design of Combined Rectangular footing for two columns only.

CE8501 DESIGN OF RC ELEMENTS

UNIT- I
INTRODUCTION

Reinforced concrete :

Reinforced concrete is a composite material in which concrete's relatively low tensile strength and ductility are counteracted by the inclusion of reinforcement having higher tensile strength (or) ductility.

Objectives of structural design :

↳ Stability to prevent overturning, sliding (or) buckling of the structure, (or) parts of it under the action of loads.

↳ Strength to resist safely the stresses induced by the loads, including environmental loads, in the various structural members & its connections.

↳ Serviceability to ensure satisfactory performance under service load condition which implies providing adequate stiffness and reinforcements to contain deflections, crack-widths and vibrations within acceptable limits.

Steps in RCC structural design process :

i) Structural planning
ii) Load calculation
iii) Methods of Analysis
iv) Member design
v) Detailing, Drawing & Preparation of Schedules.

Types of Loads:

- Dead Load
- Live Load
- wind Load
- Snow Load
- Earthquake forces

~~Dead~~ Load of Material: (IS 875 - part 1)

* IS - Indian Standard code

Type of Material	unit weight
i) Brick Masonry	18.85 to 22 KN/m^3
ii) Stone Masonry	20.4 to 26.50 KN/m^3
iii) cement concrete	22 to 23.5 KN/m^3
iv) Reinforced concrete	22.75 to 26.50 KN/m^3
v) cement mortar	20.40 KN/m^3
vi) plain concrete (IS: 456-2000)	24 KN/m^3
vii) Reinforced concrete	25 KN/m^3
viii) Asbestos cement sheeting	0.118 to 0.130 KN/m^3

Live Load on Structures: - IS 875 - part 2

* FOR Residential building - 2 to 3 KN/m^2

* FOR commercial building - 4 to 5 KN/m^2.

Wind Loads: - (IS 875- part 3)

For multistorey structures like water tanks, chimneys and other types of tall structures wind loads should be considered in the design.

* The design of wind pressure is given by

$$\boxed{P_z = 0.6 V_z^2}$$

where,

V_z = velocity

$$\boxed{V_z = K_1 K_2 K_3 V_b}$$

where,

K_1 -> Risk coefficient

K_2 -> coefficient based on terrain, height & structure size

K_3 -> Topography factor

V_b -> Velocity of wind pressure

Snow Loads: (IS 875 - part 4)

The minimum snow load on a roof of the building.

The minimum snow load on a roof area (or) any other area above ground which is subjected to snow accumulation is obtained by

$S = \mu S_0$

where,

S -> Design snow load on plan area of roof

μ -> shape coefficient

S_0 -> Ground snow load.

Load combinations:

The load combination by NBC are

(* NBC - National Building code)

1. Dead Load (DL)
2. Dead load + Imposed load (D.L + I.L)
3. Dead load + wind load (D.L + W.L)
4. Dead load + Earthquake load (D.L + E.L)
5. Dead load + Imposed load + Wind load (D.L + I.L + W.L)
6. Dead load + Imposed load + Earthquake load (D.L + I.L + E.L)

Code practices and specifications:

code book for loads:

IS 875 - 1987 - code of practice for design

IS 875 - part 1 - Dead load

IS 875 - part 2 - Imposed load

IS 875 - part 3 - wind Load

IS 875 - part 4 - Snow load.

IS 875 - part 5 - Special load (creep, shrinkage temperature, soil & fluid pressure)

Earthquake resistance design of structure:

IS 1893 part 1 - General provision & buildings

IS 1893 part 2 - Liquid retaining Tanks

IS 1893 part 3 - Bridges & retaining walls.

IS 1893 part 4 - Industrial Structures

IS 1893 part 5 - Dams & Embankments.

Methods of design of RCC Structures :

1. Working stress method
2. Limit state method
3. ultimate load method

Concept of working stress method :

↳ It is also known as elastic stress method and as modular ratio method.

↳ It basically assumes that the structural material ~~both~~ behaves in a linear elastic manner. (i,e) the material both the concrete and steel are considered to behave in a linear elastic manner.

↳ The factor of safety value for concrete is 3 to 4 and steel is 1.8 to 2.

$$\text{Factor of safety} = \frac{\text{Yield Stress}}{\text{working stress}}$$

Limitation :

↳ It deals only with elastic behaviour of the member.

↳ Modular ratio design approach respects in larger percentage of compression steel.

↳ As concrete members have variable moment of inertia due to variable cracking along their length and varying area of reinforcement.

↳ This method doesnot give correct picture of the safety of the member.

Advantages:

↳ This is the only method available for the design of retaining structures, high rise buildings, complex structures.

↳ The knowledge of working stress method is essential as it forms as part of limit state of design.

Ultimate Load method:

↳ This method is also known as load factor method.

↳ It involves the analysis of section in failure.

↳ Service loads are multiplied with load factor to get the ultimate design load and bending moment and shear and torsion are evaluated.

↳ As the load factor method, it has not constant mix design.

↳ For a mix design concrete a load factor of 1.5 may be considered and for a nominal stress it can be 1.6mm.

↳ In ultimate load design the strength of the member must be more than the ultimate load acting on the member.

Limit State method:

↳ In this method of design based on limit state concept.

↳ The structures shall be design to withstand safety.

↳ This method is also satisfy serviceability requirement such as prevention of excessive deflection, excessive cracking and excessive vibration.

↳ The acceptable limit for safety and serviceability requirements before failure occurs is called limit state.

↳ Properties of concrete:

1. compressive strength
2. Tensile strength
3. Elastic deformation
4. Shrinkage
5. creep of concrete
6. Thermal expansion
7. Poisson's ratio

8. Durability of concrete
9. Fire resistance
10. Modulas of ratio rupture

Compressive strength:

* Compressive strength primarily depends on age of concrete, Cement content & water cement ratio.

* The characteristic strength are based on the strength at 28 days.

Tensile strength:

Tensile strength shall be obtained as described in IS 516 & IS 5816 respectively.

Formula used for Estimating tensile strength is,

$$= 0.7\sqrt{f_{ck}} \ N/mm^2$$

$f_{ck} \rightarrow$ characteristic compressive strength,

Elastic Deformation:

The modulus of elasticity is normally related to the characteristic compressive strength of concrete.

$$E_c = 5000\sqrt{f_{ck}} \ N/mm^2 \quad (\text{short term})$$

Long term:

$$E_{ce} = \frac{E_c}{1+\theta}$$

Shrinkage :

* This is caused by evaporation of water from concrete.

* The total shrinkage of concrete depends upon the constituents of concrete, size of the member and environmental conditions.

* The approximate value of total shrinkage strain for design may be taken as 0.003.

Creep of concrete:

↳ It is the gradual increase in deformation (or) strain with time in a member subjected to sustained loads.

Thermal expansion:

↳ Concrete expands with rise in temperature and contracts with fall in temperature.

↳ To limit the development of temperature stresses, expansion joints are to be provided.

Poisson's ratio:

It is the ratio of lateral strain to the longitudinal strain under uniform axial stress.

The value of Poisson's ratio is 0.15 to 0.25.

Fire resistance :

Fire resistance of RCC depends upon the type of aggregate, thickness of various parts of compressing member and cover of concrete.

Modulus of rupture:

* The tensile resistance of concrete in bending is termed as modulus of rupture.

* This is normally neglected in the design of ordinary structural members but is taken in account in the design of liquid retaining Structures.

Steel reinforcement:

Steel bars are mainly used to reinforced concrete in the tension zone.

↳ Mild steel bar - Fe 250 - IS:432 (Part 1)

↳ HYSD bar - Fe 415, Fe 500 - IS-1786-1985.

↳ Hard drawn steel wire fabric - IS-1566-1982

Limit state philosophy as detailed in IS code:

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

Types:

↳ strength limit state
↳ Serviceability limit state
↳ Durability limit state

Safety and serviceability requirements:

In LSM, the structure is designed to withstand safely all the loads liable to act on it throughout its life and also to satisfy the serviceability requirements such as limitations to deflection and cracking at service loads.

i) Limit state of collapse safety requirement:

↳ The limit state of collapse of the structure are part of the structure could be assessed from the rupture of one (or) more critical sections and from buckling due to elastic (or) plastic instability (or) over turning.

↳ The resistance to bending, shear, torsion and axial loads at every section shall not be less than the appropriate value at that section produced by the propable most unfavourable combination of loads on the structure using the appropriate partial safety factor.

Advantages of Limit state method over other method

↳ Working stress method gives satisfactory performance of the structures but it is unrealistic at ultimate state of collapse.

↳ Ultimate load method provides a realistic assessment of safety but it doesn't gurantee the satisfactory performance at service loads.

* An ideal method which takes into account not only the ultimate strength of structure but also the serviceability and durability requirements in limit state method.

* It satisfies all these requirements simultaneously.

* It is a combination of working stress and ultimate load methods avoiding demerits of both.

Limit state of collapse considered in design are

i) Limit State of collapse in flexure } beam & Slabs
ii) Limit State of collapse in shear }
iii) Limit state of collapse in compression - columns
iv) Limit state of collapse in torsion - Joint.

Limit state of collapse depends upon ultimate Strength.

ii) Limit state of Serviceability [serviceability requirement]
It consists of
i) Excessive deflection
ii) premature (or) Excessive cracking
iii) corrosion
iv) Excessive vibrations.

↳ The deflection of a structure are part there there of shall not adversely affect the apperance (or) efficiency of a structure or finishes (or) partitions.

↳ Cracking of concrete should not adversely affect the appearance (or) durability of the structure.

↳ The limit state of excessive deflection and crack width is applicable with service loads & estimated on the basis of elastic analysis.

Design of Singly reinforced beam:

The aim of the design should be to provide economical design consistent with safety and serviceability.

Selection of cross sectional dimensions:

i) overall depth to width should be in the range of 1.5 to 2.

ii) Minimum side covers of 20mm to the links.

iii) Minimum No. of bars on tension face should not be less than 2 & not more than 6 in anyone layer.

iv) In flanged beams, the depth of the slab is generally taken as 20% of the overall depth.

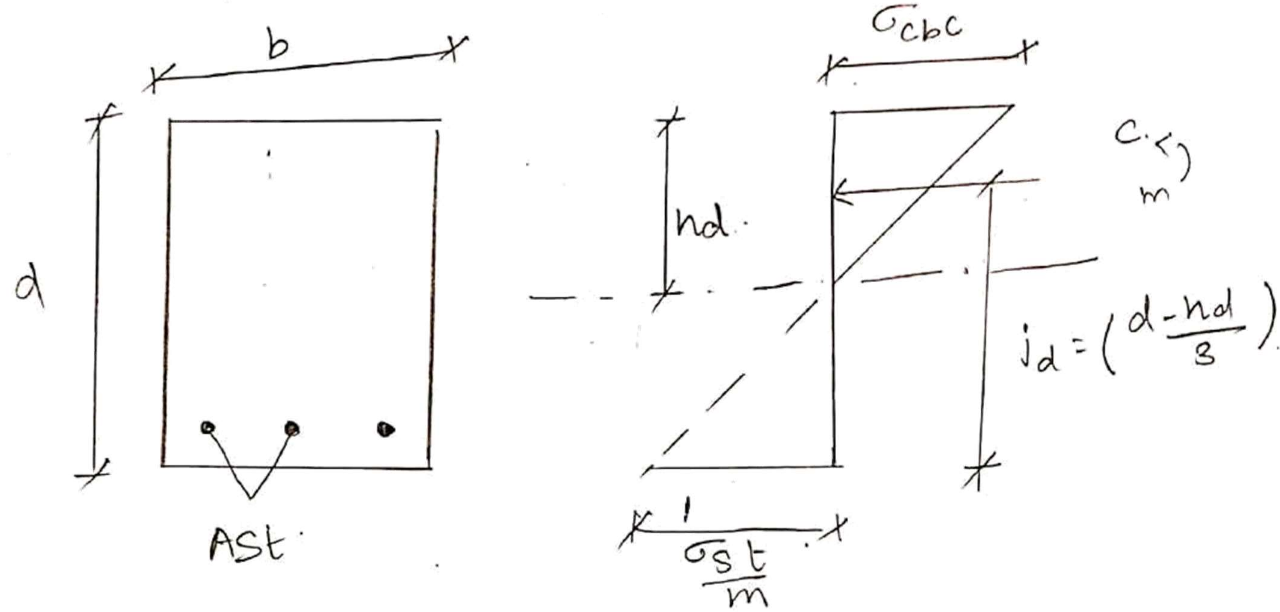
Span / depth ratio for trial section:

S.NO	Span range	Loading	Span / depth
1.	3 - 4 m	Light	15 to 20
2	5 - 10 m	Medium to heavy	12 to 15.
3	5 - 10 m	Heavy	10 to 12.

Analysis of Singly reinforced rectangular beam:

↳ Steel reinforcement is provided only at the tension zone is called singly reinforced beam.

↳ A rectangular section subjected to moment M under service loads.



where,

σ_{cbc} = Compressive stress in concrete

σ_{st} = Tensile stress in steel

Ast = Area of tension reinforcement

d = Effective depth

b = width

n = neutral axis depth factor

m = modular ratio = $\left(280/3\sigma_{cbc}\right)$

j = Lever arm coefficient $\left(1 - n/3\right)$

M = Moment of resistance

Neutral axis depth & Moment of Resistance of section:

↳ Since concrete is weak in tension, concrete below the neutral axis is neglected in computations.

↳ Below the neutral axis the steel area is converted into equivalent area of concrete by multiplying the steel area by modular ratio & this area contributes to the tensile force for equilibrium of the section.

Refering the stress distribution the following relation

$$\left[\frac{\sigma_{cbc}}{(\sigma_{st}/m)}\right] = \left[\frac{nd}{d-nd}\right] = \left[\frac{n}{1-n}\right]$$

$$n = \left[\frac{1}{1+(\sigma/m)\left(\frac{\sigma_{st}}{\sigma_{cbc}}\right)}\right]$$

From moment equilibrium of the section.

$$M = C\left[d - \left(\frac{nd}{3}\right)\right]$$

$$C = \frac{\sigma_{cbc}}{2} \cdot n \cdot b \cdot d$$

$$= \frac{\sigma_{cbc}}{2} \, b \cdot nd \cdot d\left[1-\left(n/3\right)\right]$$

$$j = (1 - n/3)$$

$$M = \frac{\sigma_{cbc}}{2} , b \cdot n \cdot j \cdot d^2$$

$$M = Qbd^2$$

$$Q = \frac{\sigma_{cbc}}{2} \cdot n \cdot j$$

$$d = \sqrt{M/Q.b}$$

$$M = \sigma_{st} \cdot A_{st} \cdot \left[d - \frac{nd}{3}\right]$$

$$M = \sigma_{st} \cdot A_{st} \cdot j \cdot d$$

$$\boxed{\therefore A_{st} = \left[\frac{M}{\sigma_{st} \cdot j \cdot d}\right]}$$

i) Balanced, under reinforced & over reinforced section:

Balanced section:

↳ A reinforced concrete section is called balanced section when a maximum allowable stress in concrete & steel are reached simultaneously.

↳ It is practically not possible.

Resisting moment of balanced section

Tension zone = Compression zone

$$\sigma_{st} \cdot A_{st} \cdot \left[d - \frac{nd}{3}\right] = \frac{1}{2} \cdot \sigma_{cbc} \cdot n \cdot b \cdot d \left[d - \frac{nd}{3}\right]$$

$$C = \frac{\sigma_{cb}}{2} \cdot n \cdot d \cdot b$$

$$T = \sigma_{st} \cdot A_{st}.$$

Equating $C = T$

$$\frac{\sigma_{cbc}}{2} n \cdot b \cdot d = \sigma_{st} \cdot A_{st}$$

% of steel reinforcement in balanced section

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

$$A_{st} = \frac{Pbd}{100}$$

$$\frac{\sigma_{cbc}}{2}\, n \cdot d \cdot b = \sigma_{st} \times \frac{Pdb}{100}$$

$$P = \frac{100}{2} \cdot n \cdot \frac{\sigma_{cbc}}{\sigma_{st}}.$$

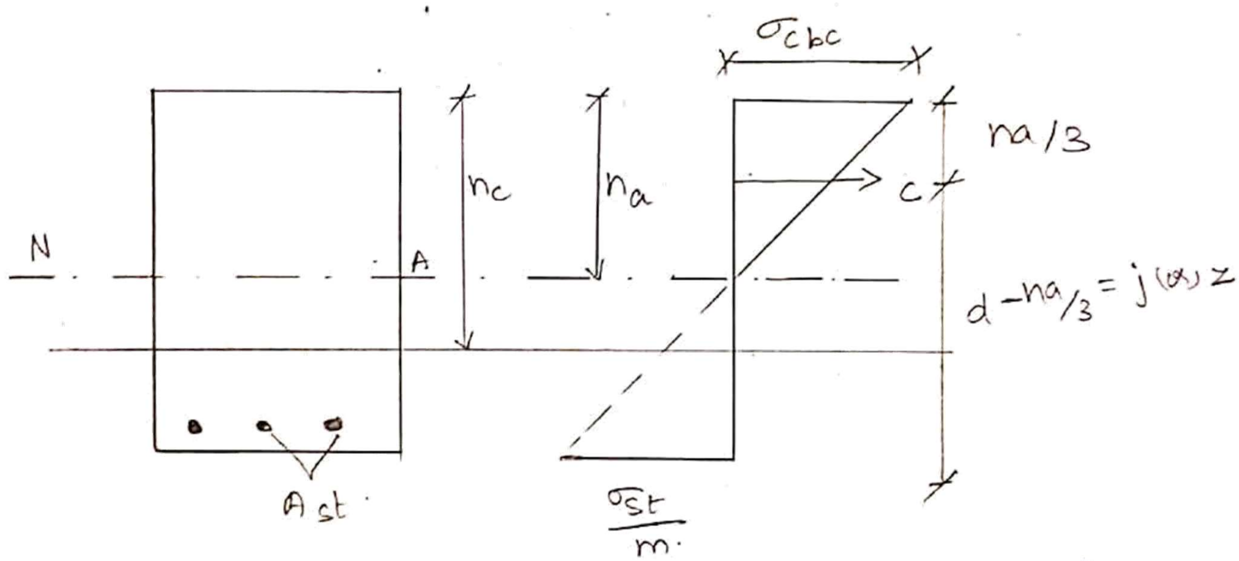
$$M_r = \sigma_{st} \cdot A_{st} \cdot d \left[1 - n/3\right]$$

(or)

$$M_r = \sigma_{st} \cdot A_{st} \cdot j \cdot d$$

under reinforced Section:

In actual neutral axis less than critical neutral axis then the section is called under reinforced section.



In under reinforced sections, the tension steel reaches yield strain at loads lower than the load at which concrete reaches the failure strain.

The moment of resistance is computed from the tension side with steel reaching the max. Permissible stress σ_{st}.

M – Moment of resistance

$M = T_z$

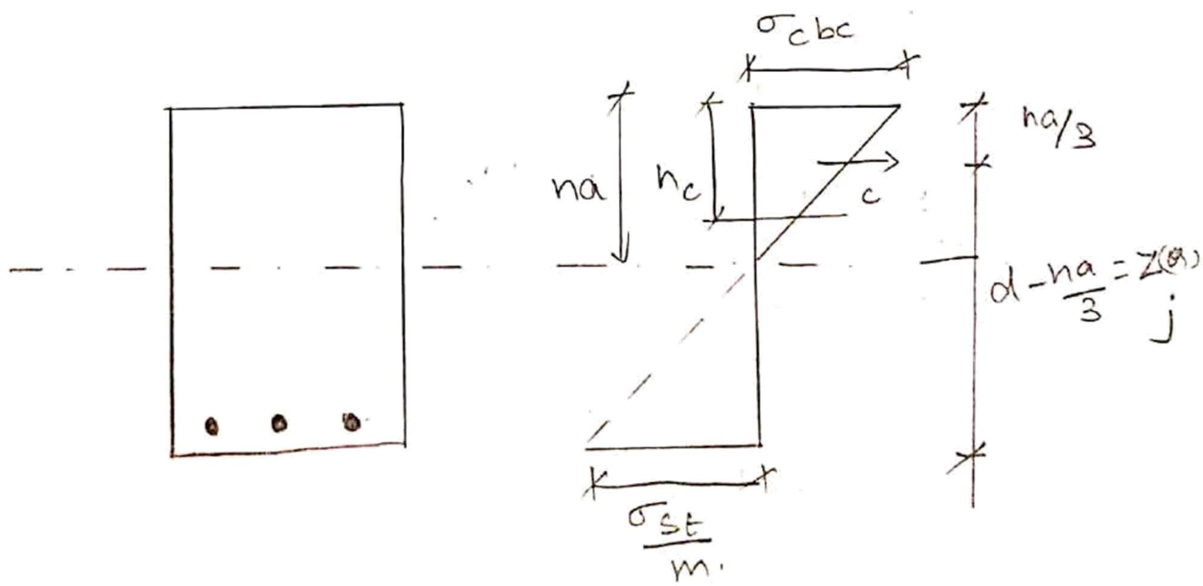
$M = \sigma_{st} \cdot A_{st} \left[d - na/3 \right]$

$\left.\begin{array}{l} \sigma_{st} = 140\ N/mm^2 \text{ – For grade I – mild steel} \\ \sigma_{st} = 230\ N/mm^2 \text{ – HYSD bar} \end{array}\right\}$ IS456 –2000

Over reinforced section :

In which concrete reaches the yield strain earlier than that of steel.

Over reinforced beam fail by compression failure of concrete without much warning & with very few cracks & negligible deflections.



It means the concrete will fail first & then the steel fails (or) $n_a > n_c$.

$$M = \frac{\sigma_{cbc}}{2} \, n_a \, b \left[d - \frac{n_a}{3} \right]$$

Moment of resistance problem:

1. Determine the position of neutral axis & the moment of resistance of a beam 300 mm wide & 500 mm effective depth. It is reinforced with 3 bars of 16mm ϕ. use M20 grade conc. & Fe415 HYSD bar. Adopt working stress method.

Given:

$b = 300mm$

$d = 550mm$.

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

$\sigma_{st} = 230 N/mm^2$

Solution:

Step 1: Actual Neutral Axis:

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

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

$$= \frac{280}{3 \times 7}$$

$$= 13.33$$

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

$$= 603.18\ mm^2$$

$$\frac{300 \times na^2}{2} = 13.33 \times 603.18 \times (500 - na) = 0.$$

$$\frac{300 \times na^2}{2} - 13.33 \times 603.18 \times (500 - na) = 0$$

$$150 na^2 + 8042.2 na - 4.42 \times 10^6 = 0.$$

$$na = 146.94\ mm.$$

Step 3: Critical Neutral axis:

$$n_c = \left[\frac{1}{1 + (1/m)\left(\frac{\sigma_{st}}{\sigma_{cbc}}\right)}\right] d$$

$$= \left[\frac{1}{1 + 13.33\left(\frac{230}{7}\right)}\right] \times 550$$

$$= 158.76\ mm.$$

$$na < nc$$

∴ The section is under reinforced section.

Step 4: Moment of resistance:

$$M = \sigma_{st} \cdot Ast \left[d - na/3\right]$$

$$= 230 \times 603.18 \left[550 - \frac{146.94}{3}\right]$$

$$= 69.5\ KNm.$$

2) A rectangular R.C Section having a width of 350mm is reinforced with 2 # Of 28mm ϕ at an effective depth of 700mm. Adopt M20 grade concrete & Fe 415 HYSD bars. Determine the ultimate Moment of resistance of the section.

Given:

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

$\sigma_{st} = 230 N/mm^2$

$b = 350mm$

$d = 700mm$

$A_{st} = 2 \times \pi/4 \, (28)^2$

$= 1230.88 \, mm^2$

Solution:

Step 1: Actual Neutral axis:

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

$$\frac{350 \, na^2}{2} = 13.33 \times 1230.88 \, (700 - na).$$

$$na = 213.56 \, mm.$$

Step 2: Critical Neutral axis:

$$n_c = \left[\frac{1}{1 + (1/m)\left(\frac{\sigma_{st}}{\sigma_{cbc}}\right)}\right] \cdot d$$

$$= \left[\frac{1}{1 + \left(\frac{1}{13.33}\right)\left(\frac{230}{7}\right)}\right] \times 700$$

$$n_c = 202.31 \, mm$$

$$na > n_c.$$

$\therefore$ The Section is over reinforced Section.

Step 3: Moment of Resistance :

$$MR = \sigma_{st} \cdot A_{st} \cdot \left[d - \frac{n_a}{3} \right]$$

$$= 230 \times 1230.88 \left[700 - \frac{213.56}{3} \right]$$

$$MR = 164.5 \text{ KNm}$$

3) Derive the expression for the depth of neutral axis & the moment of resistance of a rectangular singly reinforced balanced beam section under flexure & obtain design constant K, Q, j for M20 grade conc. & Fe 415 HYSD bar. use working stress method.

Given :

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

$$\sigma_{st} = 230 N/mm^2$$

Solution :

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.33$$

$$k = \left[\frac{1}{1 + (1/m)\left(\frac{\sigma_{st}}{\sigma_{cbc}}\right)} \right]$$

$$= \left[\frac{1}{1 + \left(\frac{1}{13.33}\right)\left(\frac{230}{7}\right)} \right]$$

$$= 0.289.$$

$$j = 1 - K/3$$

$$= 1 - \frac{0.289}{3}$$

$$= 0.90.$$

Moment of resistance of balanced section:

$$M = \frac{\sigma_{cbc}}{2} \cdot b \cdot K \cdot d \left[1 - K/3\right] d$$

$$= 7/2 \times 0.289 \times \left[1 - \frac{0.289}{3}\right] \cdot bd^2$$

$$M = 0.914\, bd^2$$

$$M = Q\, bd^2$$

$$Q = 0.914.$$

Design of singly reinforced beam by working stress method:

1. Design a rectangular reinforced concrete beam simply supported on a masonry wall 300 mm thick with an effective span of 5m to support a service load of 8 KN/m and dead load of 4 KN/m in addition to its own weight. Adopt M_{20} grade concrete and Fe-415 HYSD bars. width of support of beams = 300 mm.

Given:

$L = 5m$

$b = 300mm$

$L.L = 8 \, KN/m$

$D.L = 4 \, KN/m$

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

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

step 1: Allowable stresses:

$\sigma_{cbc} = 7 \, N/mm^2 \rightarrow$ Table 21 (IS456-2000)

$\sigma_{st} = 230 \, N/mm^2 \rightarrow$ Table 22 (IS456-2000)

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.33$$

Neutral axis depth,

$$K = \frac{1}{1 + \frac{1}{m}\left(\frac{\sigma_{st}}{\sigma_{cbc}}\right)}$$

$$= \frac{1}{1 + \left(\frac{1}{13.33}\right)\left(\frac{230}{7}\right)} = 0.288$$

$Q = \frac{1}{2} \cdot \sigma_{cbc} \cdot K \cdot j$

Lever arm factor

$j = 1 - K/3$

$= 1 - \frac{0.288}{3}$

$= 0.90$

$Q = \frac{1}{2} \times 7 \times 0.288 \times 0.90$

$= 0.91$

Step 2: Cross sectional dimensions :

$d = \frac{span}{10} = \frac{5000}{10} = 500\,mm$

$D = d + d'$

d → Effective depth

D → Overall depth.

$d' = 50\,mm$

$D = 500 + 50$

$= 550\,mm$

Step 3: Load calculation :

Self weight = b × D × Density of concrete

= ~~300~~

$= 0.3 \times 0.55 \times 25$

$= 4.125$ KN/m.

Dead load = 4 KN/m

Live load = 8 KN/m

Finishes = 0.975 KN/m

Total load = 17 KN/m.

Step 4: Bending Moment & Shear force:

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

$$= \frac{17 \times 5^2}{8} = 53 \text{ KNm.}$$

$$V = \frac{wl}{2}$$

$$= \frac{17 \times 5}{2} = 43 \text{ KN}$$

Step 5: check for depth:

$$d = \sqrt{M/Qb}$$

$$= \sqrt{\frac{53 \times 10^6}{0.9 \times 300}}$$

$$= 440 \text{ mm} < 500 \text{mm}$$

Hence, it is adequate.

Step 6: Main reinforcement in tension zone:

$$M = \sigma_{st} \cdot A_{st} \cdot j \cdot d$$

$$A_{st} = \frac{M}{\sigma_{st} \cdot j \cdot d}$$

$$= \frac{53 \times 10^6}{230 \times 0.9 \times 500} = 512 \text{ mm}^2$$

$$a_{st} = \pi/4 \; d^2$$

$$= \pi/4 \times 20^2 = 314 \text{ N/mm}^2$$

No. of bar $= 1.6 \simeq 2$ $\quad \frac{A_{st}}{a_{st}} = \frac{512}{314} = 1.6 \simeq 2$

Provide 2 # of 20 mm dia bar

Step 7: Shear stress & reinforcement:

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

$$= \frac{43 \times 10^3}{300 \times 500}$$

$$= 0.28\ N/mm^2$$

$$\left[\frac{100 A_{st}}{bd}\right] = \left[\frac{100 \times (2 \times 314)}{300 \times 500}\right]$$

$$= 0.418$$

From table 23, IS:456-2000

$$\tau_c = 0.25 N/mm^2$$

$$\tau_c < \tau_v$$

Hence shear reinforcements is required. Hence shear reinforcements are provided in the form of stirrups.

Nominal shear reinforcement is given by

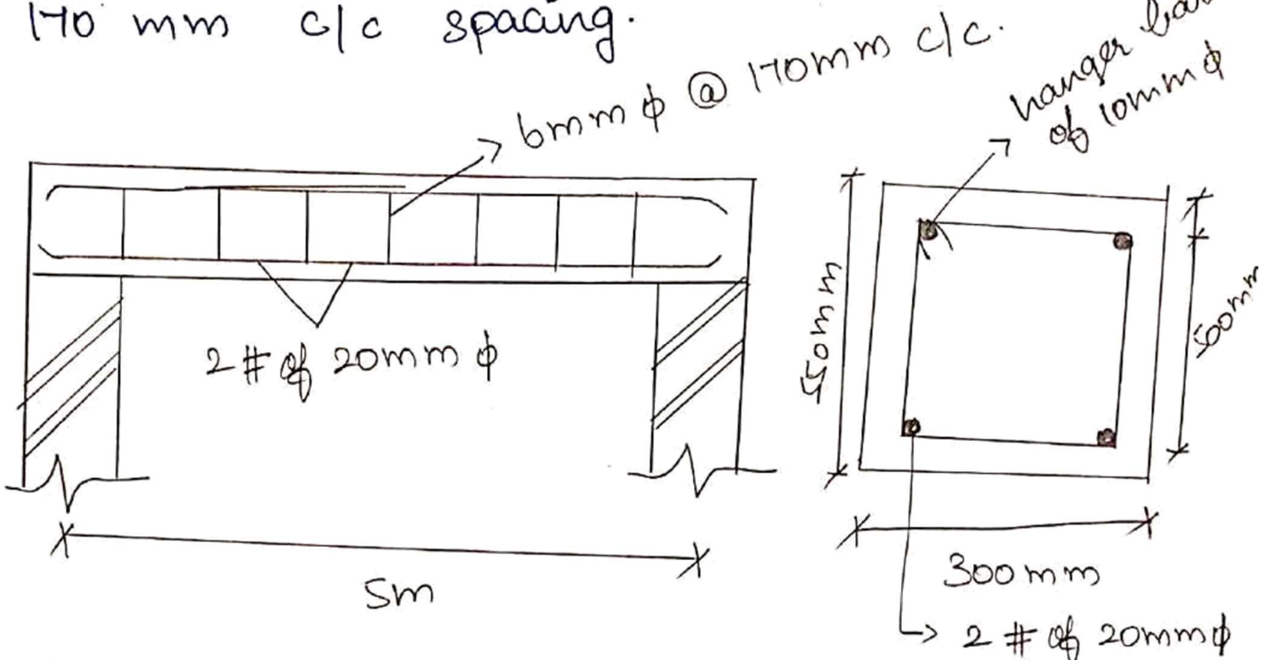
$$S_v = \frac{A_{sv} \cdot 0.87 f_y}{0.4 \cdot b}$$

provide 2 legged stirrups of ~~8~~ 6 mm ϕ

$$A_{sv} = 2 \times \pi/4\ (6)^2$$

$$S_v = \frac{2 \times \pi/4\ (6)^2 \times 0.87 \times 415}{0.4 \times 300} = 168 \simeq 170 mm.$$

provide 2 legged stirrups of 6mm ϕ @ 170 mm c/c spacing.



2) A beam, simply supported over an effective span of 8m carries a live load of 15KN/m. Design the beam, using M20 concrete & Fe 415 grade steel. Keep the width equal to half the effective depth. use working stress method of design.

Given :

$l = 8m$

$L.L = 15KN/m$

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

$f_y = 415 N/mm^2$

$b = d/2$

step 1: Permissible stresses:

$\sigma_{cbc} = 7N/mm^2$ — Table 21 - IS456-2000

$\sigma_{st} = 230N/mm^2$ — Table 22 - IS456-2000

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3\times7} = 13.33$$

$$K = \frac{1}{1+(1/m)\left(\frac{\sigma_{st}}{\sigma_{cbc}}\right)}$$

$$= \frac{1}{1+(1/13.33)\left(\frac{230}{7}\right)}$$

$$= 0.288$$

$$Q = 1/2 \cdot \sigma_{cbc} \cdot k \cdot j$$

$$j = 1 - K/3$$

$$= 1 - \frac{0.288}{3}$$

$$= 0.90$$

$$Q = 1/2 \times 7 \times 0.288 \times 0.90$$

$$= 0.91$$

Step 2: Cross sectional dimensions :

$$d = \frac{span}{10}$$

$$= \frac{8000}{10} = 800mm$$

$$D = d + d'$$

$$= 800 + 50$$

$$= 850 \text{ mm}$$

$$b = d/2 = \frac{800}{2} = 400\text{mm}.$$

Step 3: Load calculation.

Self weight of beam = b × D × density of concrete

$= 0.4 \times 0.85 \times 25$

$= 8.5\ KN/m$

Live load = 15 KN/m

Total load = 23.5 KN/m

Step 4: Bending Moment and shear force:

$M = \frac{wl^2}{8} = \frac{23.5 \times 8^2}{8} = 188\ KNm.$

$V = \frac{wl}{2} = \frac{23.5 \times 8}{2} = 94\ KN$

Step 5: Check for depth:

$d = \sqrt{M/Qb}$

$= \sqrt{\frac{188 \times 10^6}{0.9 \times 400}}$

$= 718.67 < 800mm$

Hence it is adequate.

Step 6: Main reinforcement:

$M = \sigma_{st} \cdot A_{st} \cdot j \cdot d$

$A_{st} = \left[\frac{M}{\sigma_{st} \cdot j \cdot d}\right]$

$= \left[\frac{188 \times 10^6}{230 \times 0.9 \times 800}\right]$

$= 1135.27\ mm^2$

$a_{st} = \pi/4 \, (20)^2$

$= 314.16 \, mm^2$

No. of bars $= \dfrac{A_{st}}{a_{st}}$

$= \dfrac{1135.27}{314.16}$

$= 3.6 \simeq 4$

Provide 4 # of 20 mm ϕ bars.

Step 7: check for shear stress:

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

$= \dfrac{94 \times 10^3}{400 \times 800}$

$= 0.29 \, N/mm^2$

$\dfrac{100 A_{st}}{bd} = \dfrac{100 \times (4 \times 314.16)}{400 \times 800}$

$= 0.40$

From table 23, IS 456

$\tau_c = 0.22 + \dfrac{(0.33 - 0.22)}{(0.5 - 0.25)} \times (0.4 - 0.25)$

$= 0.268 \, N/mm^2$

$\tau_c < \tau_v$

Hence shear reinforcement is required in the form of stirrups.

$A_{st} = 2 \times \pi/4 \times 8^2$

$= 100.48\ mm^2$

$S_v = \frac{\sigma_{st} \cdot A_{sv} \cdot d}{V_{us}}$

$V_{us} = V - \tau_c\, b \cdot d$

$= 94 \times 10^3 - 0.268 \times 400 \times 800$

$= 8240N.$

$S_{v_1} = \frac{230 \times 100.48 \times 800}{8240}$

$= 2248.73mm.$

Maximum value of spacing should be less than (or) equal to 300mm.

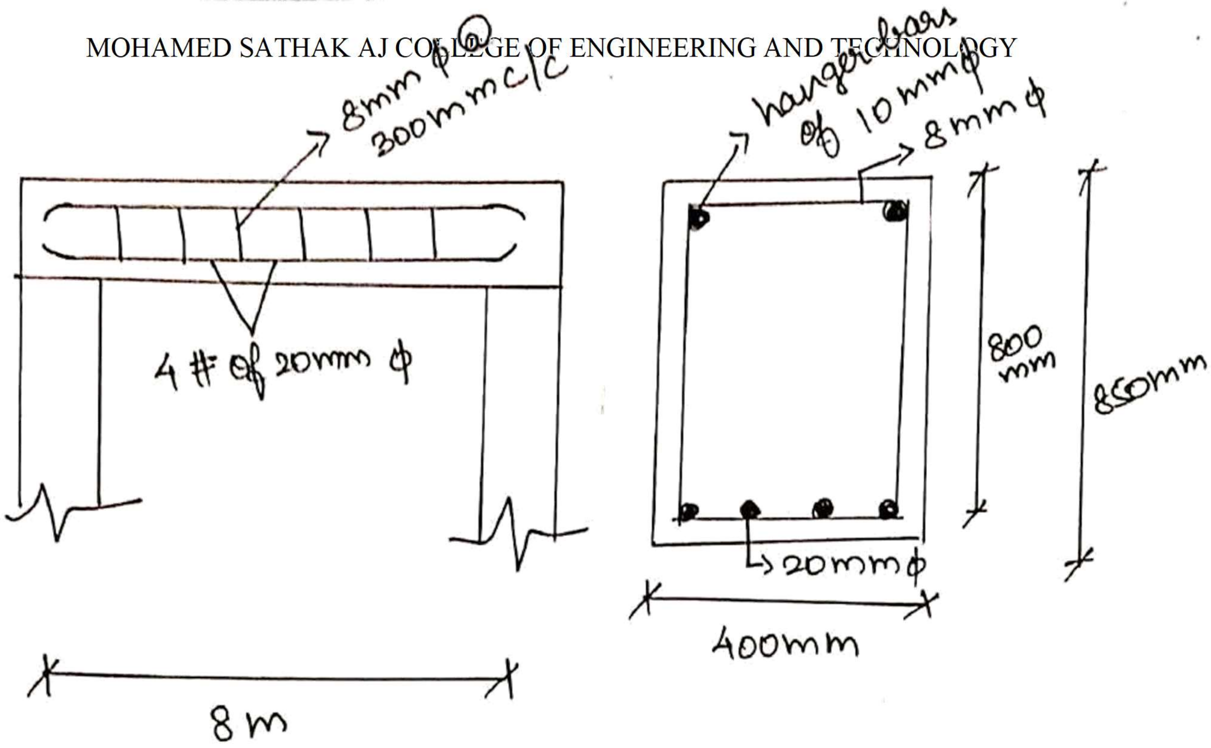
$S_{v_2} = 0.75d$

$= 0.75 \times 800$

$= 600mm.$

Both the spacing S_{v_1} & S_{v_2} doesn't satisfy the design criteria, so we have to provide maximum spacing of 300mm.

∴ provide 2 legged stirrups of 8mm ϕ @ 300mm c/c spacing.


8mm ϕ @
300mm c/c
4 # of 20mm ϕ
8 m
hanger bars
of 10 mm ϕ
8 mm ϕ
800
mm
850mm
20mm ϕ
400mm

1) Design a singly reinforced concrete beam to suit the following data.

clear span = 4 m

width of supports = 300 mm

Service load = 5 kN/m.

Materials : M20 grade concrete & Fe 415 HYSD bar

Step 1: Stresses:

$f_{ck} = 20 N/mm^2$ → Table 2 - IS 456 - 2000

$f_y = 415 N/mm^2$

Load factor = 1.5.

Step 2: Cross sectional dimensions:

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

$$= \frac{4000}{20} = 200 mm$$

$D = d + d'$

$d' = 50 mm$ (cover)

$D = 200 + 50$

$= 250 mm$

Step 3: Effective span:

l = clear span + centre to centre support

$$= 4 + \frac{0.3}{2} + \frac{0.3}{2}$$

$= 4.3 m$

l = clear span + effective depth

$= 4 + 0.2 = 4.2 m$

$l = 4.2 m$ (take least value)

Step 4: Load calculation:

Self weight $= b \times D \times$ density of conc.

$= 0.2 \times 0.25 \times 25$

$= 1.25$ KN/m

Live load $= 5$ KN/m

Total load $= 6.25$ KN/m

Ultimate load $= 6.25 \times 1.5$

$W_u = 9.375$ KN/m

Step 5: Bending Moment & Shear force:

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

$$= \frac{9.375 \times 4.2^2}{8}$$

$= 20.67$ KNm.

$$V = \frac{W_u l}{2}$$

$$= \frac{9.375 \times 4.2}{2}$$

$= 19.68$ KN

Step 6: Tension reinforcement:

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

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

$= 220.8 \times 10^6$

$= 22.08$ KNm.

$M_u < M_u$ limit

$\therefore$ Section is under reinforced section.

$$M_u = 0.87\, f_y\, A_{st}\, d \left[1 - \frac{A_{st}\, f_y}{f_{ck}\, b d}\right] \rightarrow [\text{IS } 456^{19} - 200 \text{ pg. } 96]$$

$$20.67 \times 10^6 = 0.87 \times 415 \times A_{st} \times 200 \left[1 - \frac{200 \times 415}{20 \times 200 \times 200}\right]$$

$$A_{st} = 350\, mm^2$$

$$a_{st} = \pi/4 \times 16^2$$
$$= 201.14\, mm^2$$

$$\text{No. of bars} = \frac{A_{st}}{a_{st}}$$
$$= \frac{350}{201.14}$$
$$= 1.7 \simeq 2$$

provide 2 # of 16 mm ϕ bar

Step 7: check for shear stresses :

$$\tau_v = \frac{V_u}{bd}$$
$$= \frac{19.68 \times 10^3}{200 \times 200}$$
$$= 0.49\, N/mm^2$$

$$P_t = \frac{100\, A_{st}}{bd} = \frac{100 \times (2 \times 201.14)}{200 \times 200} = 1.005$$

From Table 19 of IS 456 - 200

$$\tau_c = 0.63\, N/mm^2$$

$$\tau_c > \tau_v$$

Hence provide a nominal shear reinforcement.

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

$$A_{sv} = 2 \times \pi/4 \times 6^2$$

$$= \frac{0.87 \times 415 \times 2 \times 28}{0.4 \times 200}$$

$$= 152 mm$$

But

$$S_v \geq 0.75 d$$

$$0.75 \times 200$$

$$= 150 mm.$$

Provide 2 legged stirrups of 6mm ϕ @ 150mm c/c.

Step 8: Check for deflection:

$$P_t = 1.005.$$

Fig 4 in IS 456 - 2000

$$K_t = 1.05.$$

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

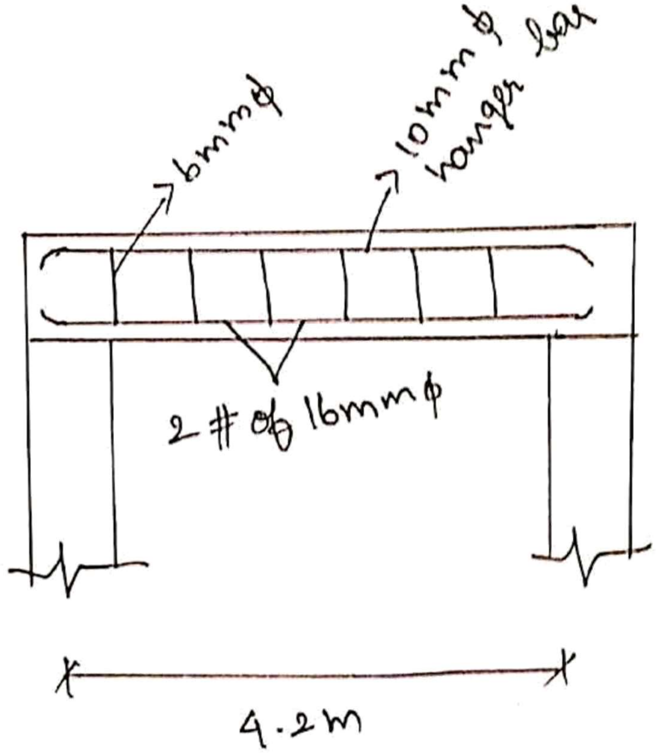
$$= (20 \times 1.05 \times 1 \times 1)$$

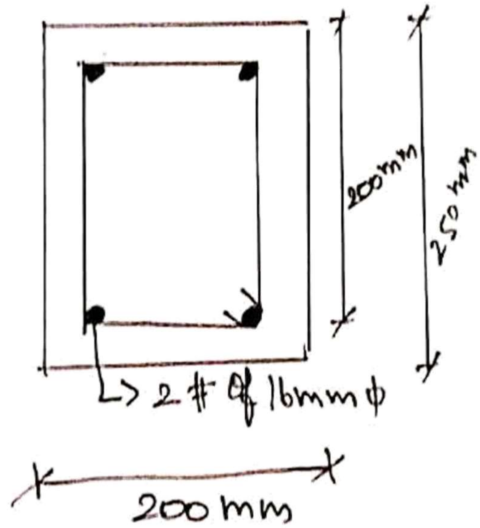
$$= 21$$

$$(L/d)_{actual} = \left(\frac{4200}{200}\right) = 21$$

$$(L/d)_{max} = (L/d)_{actual}$$

deflection control is satisfactory.





Doubly reinforced Beam:

1. Design a reinforced concrete beam of rectangular section using the following data.

Effective span = 5m

width of beam = 250mm

overall depth = 500mm

service load (D.L+L.L) = 40KN/m

effective cover = 50mm

Materials: M20 grade concrete

Fe 415 HYSD bars.

Step 1: ultimate moment & shear forces:

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

$$= \frac{(40 \times 1.5) \times 5^2}{8} = 187.5 \text{ kNm}$$

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

$$= \frac{(40 \times 1.5) \times 5}{8} = 150 \text{ kN}$$

Step 2: Main reinforcement:

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

$$= (0.138 \times 20 \times 250 \times 450^2)$$

$$= 140 \text{ kNm}.$$

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

∴ Design a doubly reinforced section.

$$(M_u - M_{u,lim}) = (187.5 - 140) = 47.5 \text{ kNm}.$$

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

$$= \left\{ \frac{0.0035 [(0.48 \times 450) - 50)}{(0.48 \times 450)} \right\} 2 \times 10^5.$$

$$= 538 \text{ N/mm}^2$$

But

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

$$= 0.87 \times 415$$

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

$$\therefore A_{sc} = \left[\frac{(M_u - M_{u\,lim})}{f_{sc}\,(d - d')} \right]$$

$$= \left[\frac{47.5 \times 10^6}{361 \times 400} \right]$$

$$= 329\ mm^2$$

$$a_{sc} = \pi/4 \times 16^2$$

$$= 201.1$$

$$\text{No. of bars} = \frac{A_{sc}}{a_{sc}}$$

$$= \frac{329}{201}$$

$$= 1.6 \simeq 2$$

provide 2 # of 16mm ϕ bars.

$$A_{st_2} = \left[\frac{A_{sc} \cdot f_{sc}}{0.87\ f_y} \right]$$

$$= \left[\frac{329 \times 361}{0.87 \times 415} \right]$$

$$= 329\ mm^2$$

$$A_{st_1} = \left[\frac{0.36\ f_{ck}\ b\ x_{u,lim}}{0.87\ f_y} \right]$$

$$= \left[\frac{0.36 \times 20 \times 250 \times 0.48 \times 450}{0.87 \times 415} \right]$$

$$= 1077\ mm^2$$

Total tension reinforcement,

$$A_{st} = (A_{st_1} + A_{st_2})$$
$$= 1077 + 329$$
$$= 1406\, mm^2$$

$$a_{st} = \pi/4 \times 25^2$$
$$= 491.071\, mm^2$$

$$\text{No. of bars} = \frac{A_{st}}{a_{st}}$$
$$= \frac{1406}{491.071}$$
$$= 2.8$$
$$\simeq 3$$

provide 3 # of 25mm ϕ bars.

Step 3: check for shear:

$$\tau_v = \frac{V_u}{bd}$$
$$= \frac{150 \times 10^3}{250 \times 450} = 1.83\, N/mm^2$$

$$P_t = \frac{100\, A_{st}}{bd}$$
$$= \frac{100 \times 4 \times 25}{250 \times 450}$$
$$= 1.3$$

From table 19, IS : 456 - 2000

$\tau_c = 0.68 \; N/mm^2$

$\tau_v > \tau_c$

$\therefore$ Shear reinforcement is required

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

$$S_v = \left[\frac{0.87 f_y A_{sv} d}{V_{us}}\right]$$

$$= \left[\frac{0.87 \times 415 \times \left(2 \times \frac{\pi}{4}(8^2)\right) \times 450}{150 - (0.68 \times 250 \times 450)}\right]$$

$= 221.51$

$\simeq 200 mm$

provide 2 legged stirrups of 8 mm ϕ @ 200 mm c/c.

step 4 : check for deflection :

$$(L/d)_{actual} = \frac{5000}{450} = 1.1$$

$$(L/d)_{max} = \left[(L/d)_{basic} \times K_t \times K_c \times K_f\right]$$

From fig 3 → IS 456, $K_t = 0.93$

Fig 4 → IS 456, $K_c = 1.10$

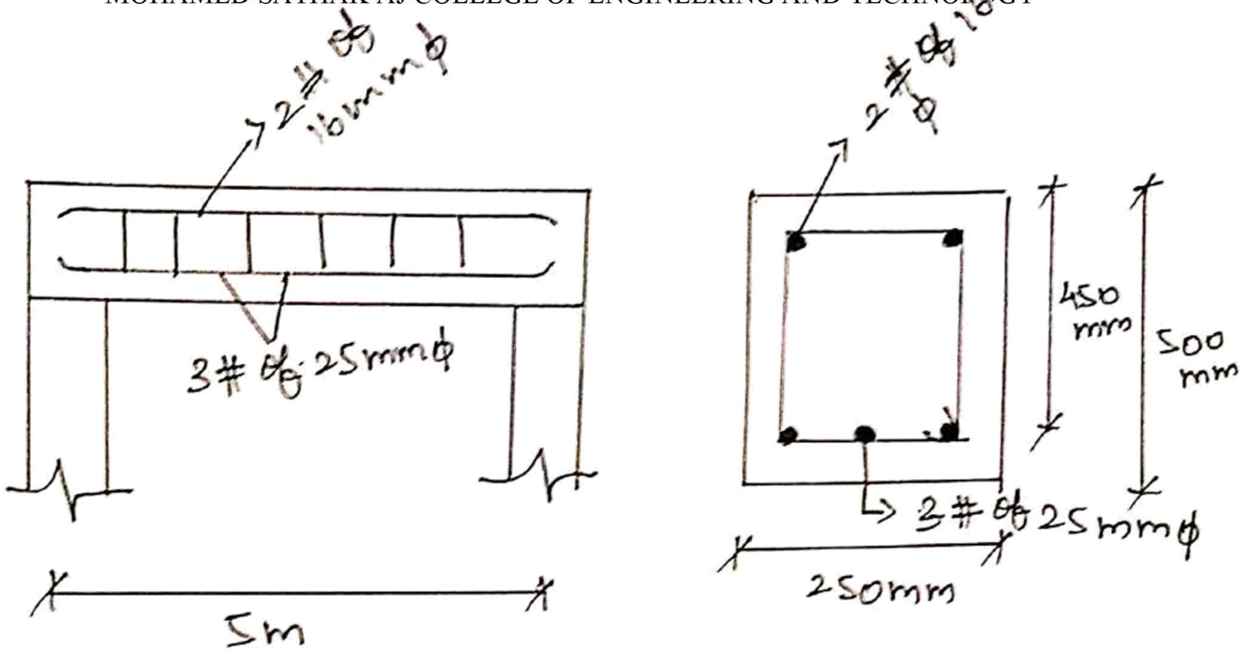
Fig 5, → IS 456, $K_f = 1$

$$(L/d)_{max} = \left[(20) \times 0.93 \times 1.10 \times 1\right]$$

$= 20.46$

$(L/d)_{max} > (L/d)_{actual}$

Hence deflection control is satisfied.


2# 10mm φ
3# 25mm φ
5m
2# 10mm φ
450 mm
500 mm
3# 25mm φ
250mm

UNIT - II

DESIGN OF BEAMS:

24 copies Analysis & Design of Flanged beams:

Design Parameters of Tee beams:

1

→ The most common type of RC floor & roof System comprises of concrete slabs monolithically cast with floor in beams in the span range of 5 to 10 m.

→ In such cases compressive flange is made up of the width of rib & a portion of the slab length on either side of the rib referred to the effective width of flange.

Effective width of flange: (b_f)

The effective width of flange should in no case be greater than the breadth of the web plus half the sum of the clear distance to the adjacent beam on either side.

i) For T- beams,

$$b_f = (L_0/6) + b_w + 6D_f$$

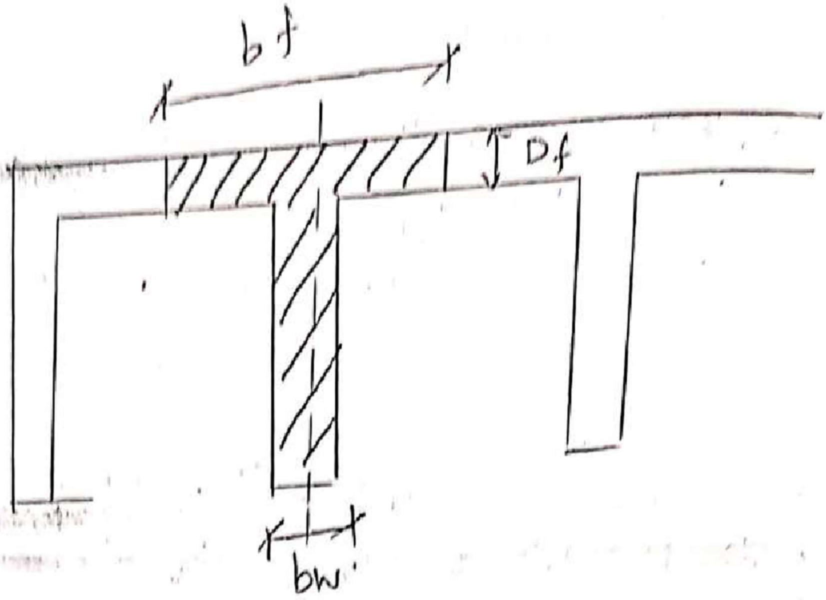
ii) For L beams,

$$b_f = (L_0/12) + b_w + 3D_f$$

For isolated flan beams, the effective flange width shall be obtained as below but in no case greater than the actual width.

T. beam, $b_f = \left[\frac{L_0}{(L_0/b)+4} + b_w\right]$

L- beam, $b_f = \left[\frac{L_0}{(L_0/b)+4} + b_w\right]$



where,

b_f → Effective width of flange

L_0 → Distance b/w points

L → Effective span

b_w → breadth of web

D_f → Flange thickness

b → actual width of flange.

ultimate flexural strength of flanged section :

case (i) Neutral axis lies within the flange ($x_u < D_f$)

$x_u < x_{u\,lim}$ (under reinforced section)

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

$x_u > x_{u\,lim}$ (over reinforced section).

$$M_u = 0.36 \frac{x_{u,lim}}{d} \left[1 - 0.42 \frac{x_{u\,lim}}{d}\right] b d^2 f_{ck}.$$

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Case (ii)

Neutral axis lies outside the flange ($x_u > D_f$)

$D_f/d < 0.2$ (URS)

$$M_R = 0.36 \frac{x_{ulim}}{d}\left(1 - 0.42\frac{x_{ulim}}{d}\right) f_{ck}\, b d^2 + 0.45 f_{ck}(b_f - b_w) D_f \left(d - \frac{D_f}{2}\right)$$

Design of Tee beam

1. A Tee beam slab floor of an office comprises of a slab of 150mm thick spanning b/w ribs spaced at 3m centre. The effective span of beam is 8m. Live load on floor is 4 kN/m². use M20 grade conc. & Fe415 HYSD bar. Design one of the intermediate tee beams.

Step 1 : Data :

$L = 8\,m$

Spacing of tee beams = 3 m

$D_f = 150\,mm$

$q = 4\,kN/m^2$

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

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

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Step 2: Cross sectional dimensions:

$b_w = 300$ mm.

$b = 3m$

$\frac{b_w}{b} = \frac{300}{3000} = 0.1$

From fig 5.3,

Reduction factor = 0.8.

$\frac{\text{Span}}{\text{depth}}$ ratio $= (20 \times 0.8)$
$= 16.$

$d = \frac{\text{span}}{16}$
$= \frac{8000}{16}$
$= 500$ mm.

$D = 500 + 50$
$= 550$ mm.

Take $D_f = 150$ mm.

Step 3: Loads:

S.W of slab $= b \times D \times \gamma$
$= 3 \times 0.15 \times 25$
$= 11.25$ KN/m.

Floor finish $= 0.6 \times 3$
$= 1.8$ KN/m.

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S.W of rib $= 0.3 \times 0.4 \times 25$
$= 3 KN/m.$

Finish $= 0.45 KN/m.$

Total dead load, $g = 16.5 KN/m$

Live load, $q = 4 KN/m$

$W_u = 1.5(16.5+4)$
$= 30.75 KN/m.$

Step 4: ultimate moment & shear force:

$$M_u = \frac{wl^2}{8}$$

Furnish $= \frac{30.75 \times 8^2}{8}$

$= 246 KNm.$

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

$= \frac{30.75 \times 8}{2}$

$= 123 KN.$

Step 5: Effective width of flange:

i) $b_f = [(L_0/6) + b_w + 6D_f]$

$= [(8/6) + 0.3 + (6 \times 0.15)]$

$= 2.53 m$

$= 2530 mm.$

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ii) Centre to centre of ribs = (3 − 0.3)
= 2.7 m.

∴ b_f = 2530 mm [least value]

Step 6: Moment capacity of flange:

$M_{uf} = 0.36\, f_{ck}\, b_f\, D_f\, (d - 0.42\, D_f)$

$= 0.30 \times 20 \times 2530 \times 150\, (500 - 0.42 \times 150)$

$= 1194 \times 10^6$ Nmm

$= 1194$ kNm.

$M_u < M_{uf}$, $x_u < D_f$.

∴ The section is under reinforced section.

Step 7: Reinforcement:

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

$246 \times 10^6 = 0.87 \times 415 \times A_{st} \times 500 \left[1 - \frac{A_{st} \times 415}{2530 \times 500 \times 20}\right]$

$A_{st} = 1417\ mm^2$

provide 25 mm ϕ bar

$a_{st} = \frac{\pi}{4}(25)^2$

$= 490.8\ mm^2$

No. of bar $= \frac{A_{st}}{a_{st}}$

$= \frac{1417}{490.8} = 2.8$

$\simeq 3$.

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provide 3# of 25mm ϕ

step 8: shear reinforcement:

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

$$= \frac{123\times10^3}{300\times500}$$

$$= 0.82 N/mm^2$$

$$Pt = \frac{(100\ Ast)}{bwd}$$

$$= \frac{100\times1472}{300\times500}$$

$$= 0.98.$$

From table 19, $\tau_c = 0.6\ N/mm^2$.

$$Sv = \frac{0.87\ fy\ Asv}{Vus}.$$

$$Vus = Vu - [\tau_c\ bwd)$$

$$= 123 - [0.6\times300\times500)]$$

$$= 33KN$$

use 8mm ϕ two legged Stirrup

$$ASV = 2\times\pi/4\ (8)^2$$

$$= 100.5\ mm^2.$$

$$Sv = \frac{0.87\times415\times100.5}{33\times10^3}$$

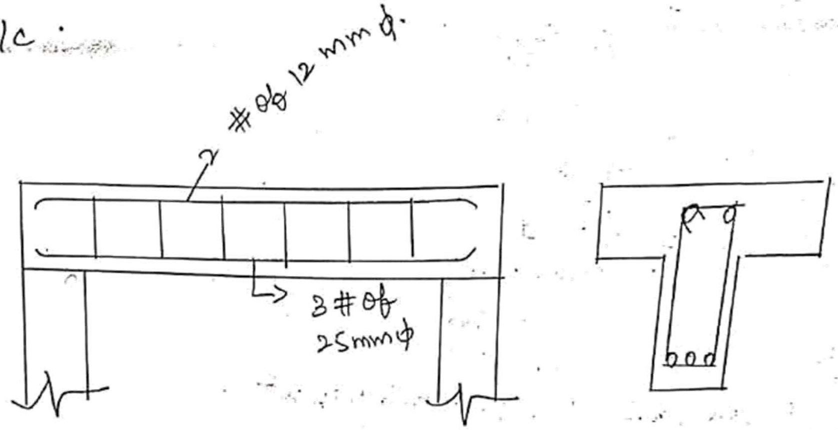
$$= 547\ mm.$$

$S_v \ngtr 0.75d$ (or) 300mm whichever is less.

$S_v = 0.75 \times 500$

$= 375mm$.

Hence provide 8mm ϕ 2legged stirrups at 300 mm c/c.



Analysis problem:

1. Determine the ultimate flexural strength of T-beam width of flange = 800 mm, Depth of flange = 150mm, width of rib = 300 mm, Effective depth = 420 mm. $A_{st} = 1470\ mm^2$. use M20 concrete & Fe 415 HYSD bar.

Step 1: Data:

$b_f = 800mm$. $f_{ck} = 20\ N/mm^2$

$D_f = 150mm$ $f_y = 415\ N/mm^2$

$b_w = 300mm$

$d = 420mm$

$A_{st} = 1470\ mm^2$

Step 2: Depth of Neutral Axis:

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

$$x_u = \frac{0.87 \times 415 \times 1470}{0.36 \times 20 \times 800}$$

$x_u = 92.14\ mm$.

$x_u < D_f$.

∴ The neutral axis lies within the flange.

$x_u < x_{u\,lim}$ [Section is URS).

Step 3: ultimate Moment of resistance:

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

$= 0.87 \times 415 \times 1470 (420 - 0.42 \times 92.14)$

$M_u = 202.37\ KNm$

2. A singly reinforced T-beam, $b_f = 950mm$, $D_f = 80mm$ $b_w = 250mm$, $d = 565mm$, $A_{st} = 1256.6mm^2$. use M15 grade concrete & Fe415 HYSD bar.

Step 1: Data:

$b_f = 950\ mm$
$D_f = 80\ mm$
$b_w = 250mm$
$d = 565mm$
$A_{st} = 1256.6mm^2$

$f_{ck} = 15 N/mm^2$
$f_y = 415 N/mm^2$

step 2: Depth of neutral axis:

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

$$x_u = \frac{0.87 \times 415 \times 1256.6}{0.36 \times 15 \times 950}$$

$x_u = 88.48mm.$

$x_u > D_f$

∴ Neutral axis lies outside of flange.

Step 3:

$$D_f/d = \frac{80}{565} = 0.14 < 0.2.$$

∴ The section is under reinforced section.

step 4: Calculation of Moment of Resistance:

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

$\frac{x_{u\,max}}{d} = 0.48 \rightarrow$ pg. 70.

$$M_u = 0.36 \times 0.48 (1 - 0.42 \times 0.48) \times 15 \times 250 + 565^2 + 0.45 \times 15 (950 - 250)\, 80 \left(565 - \frac{80}{2}\right)$$

$M_u =$

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Behaviour of RC members in Bond & Anchorage:

Bond:

↳ RC beam is a composite material which comprises of reinforcing steel & concrete with proper bond.

↳ If this bond is inadequate, slipping of the reinforcing bar will occur, which destroy the composite action.

↳ Bond b/w steel & concrete can be achieved by following method.

i) chemical adhesion:

↳ cement is used as a binder in concrete.

ii) Frictional resistance:

↳ Friction b/w surface roughness of the reinforcement & grip exerted by the concrete.

iii) Mechanical interlock:

↳ Twisted (or) deformed bars can be used to increase the bond.

Bond stress:

↳ Stress developed at the interface of steel bar & the surrounding is known as bond stress.

↳ It is classified into two types.

i) Flexure bond.

ii) Anchorage (or) Development bond.

Development length : (L_d)

↳ The reinforcement bar must extend in the anchorage zone, concrete sufficiently, to develop the required stress.

↳ The extended length of bar inside the face of the support is known as development length.

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

where,

$\phi \rightarrow$ diameter of bar.

$\sigma_s \rightarrow$ stress in bar.

$\sigma_s = 0.87 f_y$

$f_y \rightarrow$ yield stress in steel.

$\tau_{bd} \rightarrow$ Design bond stress.

Problems on Bond & Anchorage length :

1. Check for bond stress at the point of inflection of a continous beam as shown in fig. if it is subjected to ultimate shear force 300kN at the point of inflection. consider M20 grade conc & Fe415 steel, yield stress in steel.

Given :

$V_u = 300\ KN$ · $\phi = 16mm$.

$f_{ck} = 20\ N/mm^2$ $\phi = 3\#$ of $16mm\phi$.

$f_y = 415N/mm^2$.

Solution :

$$L_d = \frac{\phi\,\sigma_s}{4\,\tau_{bd}} \qquad \sigma_s = 0.87 f_y.$$

$$L_d = \frac{16 \times 0.87 \times 415}{4 \times 1.92} \qquad \left(\underline{\text{For deformed bars}}\ \tau_{bd} = 1.92\right)$$

$$= 752.19\,mm.$$

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

$$A_{st} = 3 \times \pi/4\ (16)^2$$

$$= 201\,mm^2$$

$$P_t = \frac{100 \times 201}{230 \times 450}$$

$$= 0.583\%.$$

From table 2, SP 16, $f_y = 415\ N/mm^2$

$$\frac{M_u}{bd^2} = 1.85.$$

$$M_u = 1.85\,bd^2$$

$$= 1.85 \times 230 \times 450^2$$

$$= 86.16 \times 10^6\ Nmm.$$

$$M_u = 86\,KNm.$$

$$\frac{M_u}{V_u} + l_d = \frac{86.16 \times 10^6}{250 \times 10^3} + 450.$$

$$= 794.64 > L_d \quad (L_d = 752.9mm)$$

∴ The point of inflection is within the Safe limit.

8) Determine the anchorage length of bars at the simply supported end of RC beam of overall size 300x450mm with 3-16mm ϕ @ tension zone. The beam is subjected to an ultimate shear force of 200 kN at the centre of support. Consider M20 grade conc. & Fe415 steel. width of support = 300mm.

Given :

$b = 300 mm$

$D = 450mm$.

3 # of 16mm ϕ

$V_u = 200 kN$.

$L_s = 300$

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

$f_y = 415 N/mm^2$

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sln :

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

$$\sigma_s = 0.87 f_y$$

$$= \frac{16 \times 0.87 \times 415}{4 \times 1.92}$$

$$= 752.19.$$

$$L_d \simeq 752\,mm.$$

$$d' = 25 + \frac{16}{2}$$ (consider clear cover = 25mm)

$$= 33mm$$

$$d = D - d'$$

$$= 450 - 33$$

$$= 417\,mm.$$

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

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

$$= 603\,mm^2$$

$$P_t = \frac{100 \times 603}{300 \times 417}$$

$$= 0.482\,\%.$$

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From table 2, sp 16.

Inter polation

p_t	M_u/bd^2
$x_1 = 0.477$	$y_1 = 1.55$
$x_2 = 0.494$	$y_2 = 1.60$
$x = 0.482$	$y = ?$

From table 2, sp 16.

$$y = y_1 + \frac{(y_2 - y_1)}{(x_2 - x_1)} (x - x_1)$$

$$= 1.55 + \frac{(1.60 - 1.55)}{(0.494 - 0.477)} (0.482 - 0.477)$$

$$= 1.565 \ N/mm^2$$

$$M_u/bd^2 = 1.565 \ N/mm^2$$

$$M_u = 1.565 \times bd^2$$

$$= 1.565 \times 300 \times 417^2$$

$$= 81.64 \times 10^6 \ Nmm$$

$$= 81.6 kNm.$$

$$L_{d_1} \leq 1.3 \frac{M_u}{V_u} + L_d = \frac{81.64 \times 10^6}{200 \times 10^3} +$$

$$758 \leq 1.3 \times \frac{81.64 \times 10^6}{200 \times 10^3} + L_d.$$

$$\cancel{753} \leq 530.66 + L_d.$$

$222.34 \leq L_d$

$L_d = 223$ mm.

Provide anchorage length

$L_d = 225$ mm.

Design requirement as per code:

↳ RC beam are designed to resist the shear forces resulting from external loads, after determination of bending reinforcement.

↳ In beams, combined action of flexure & shear produces principal tensile & compressive stress.

↳ When the principal tensile stress exceeds tensile stress of concrete, formation of crack occurs.

Based on experimental results on RC rectangular beam;

i) 20 - 40% shear resisted by uncracked concrete

ii) 33 - 50% shear resisted by aggregate interlock.

iii) 15 - 25% shear resisted by flexural reinforcement.

Behaviour of RC beams in shear & torsion:

The shear reinforcement provides to the strength of the beam in following way.

i) Shear reinforcement carries a part of the shear due to the truss action.

ii) It limits the diagonal tension crack.

iii) It provides support to the longitudinal bars which is being crossed by the shear crack.

iv) It increases strength of the concrete.

Types of reinforcements used to resist shear force:

Additional reinforcements to be provided to resist shear force wherever the nominal shear capacity of concrete is inadequate.

Types:

i) Vertical stirrups

ii) Inclined stirrups

iii) Bent-up bars.

Vertical stirrups:

As per clause 26.5.1.5.

Spacing b/w vertical stirrups in RC beams can be calculated by taking minimum of following:

i) S_{vmax} = Max. spacing for shear reinforcement.

$= 0.75d$ (for vertical stirrups)

$= d$ (for inclined stirrups 45°)

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ii) S_v max = 300mm (for all cases)

iii) spacing,

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

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

ii) Inclined stirrups or bent up bars:

a) For bent up at different c/s:

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

b) For single bar (or) single group of parallel bars,

$$S_v = \frac{0.87 f_y A_{sv} d \sin\alpha}{V_{us}}.$$

$\alpha = 45^\circ$

Minimum shear reinforcement:

Min. shear reinforcement = $\frac{A_{sv}}{b S_v} \geq \frac{0.4}{0.87 f_y}$.

i.e) $S_v \leq \frac{2.175 f_y A_{sv}}{b}$

where,

A_{sv} = Total c/s area of stirrups

S_v = spacing of stirrups

b = breadth of beam.

d = Effective depth of beam.

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Nominal shear stress:

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

$V_u \rightarrow$ shear force.

$b \rightarrow$ width of member.

$d \rightarrow$ effective depth

1) A rectangular beam section of 300mm width & 450mm effective depth is reinforced with 4 bars of 20mm φ. Determine the shear reinforcement required to resist shear force of 40KN. Consider M20 grade concrete & Fe 415 steel.

Given:

$b = 300mm$

$d = 450mm.$

$V = 40 KN.$

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

$f_y = 415 N/mm^2$

Solution:

Fe 415 steel $\tau_c = \frac{V_u}{bd}.$

$V_u = \tau_c \times bd.$

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

$A_{st} = 4 \times \pi/4 (20)^2$

$= 1256 mm^2.$

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From table -19, IS 456 - 2000.

P_t	τ_c (M20)
$x_1 = 0.75$	$y_1 = 0.56$.
$x_2 = 1$	$y_2 = 0.62$
$x = 0.93$	$y = ?$

$$y = y_1 + \frac{(y_2 - y_1)}{(x_2 - x_1)}(x - x_1)$$

$$= 0.56 + \frac{(0.62 - 0.56)}{(1 - 0.75)}(0.93 - 0.75).$$

$\tau_c = 0.603 \, N/mm^2$.

$V_{uc} = 0.603 \times 300 \times 450$

$= 81482 N$

$= 81.43 \, KN > V_u$

Hence minimum shear reinforcement shall be provided.

Provide 2 legged stirrups of 8mm ϕ.

$A_{sv} = 2 \times \pi/4 \, (8)^2$

$= 100.5 \, mm^2$

i) spacing, $S_v = 2.175 \, A_{sv} \, f_y / b = 2.175 \times 2 \times 50 \times \frac{415}{300}$

$= 300.87 \, mm$.

ii) $S_v = 0.75 d = 0.75 \times 450 = 337.5 mm$.

iii) $S_v = 300 mm$.

Provide 2 legged stirrups of 8mm ϕ @ 300mm c/c.

2) A rectangular beam section of 300mm width × 450 mm effective depth is reinforced with 4 bars of 200 mm diameter. Determine the shear reinforcement required to resist shear force of 140KN. Consider concrete grade M20 & steel of grade Fe415.

Given:

$b = 300mm$

$d = 450mm$.

$V = 140 KN$.

$V_u = 1.5 \times 140 \times 10^3 = 210 \times 10^3 N$.

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

$f_y = 415 N/mm^2$

S/n result

$V_u = \tau_c \times bd$

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

$A_{st} = \pi/4 \times 20^2 \times 4$

$= 1256 mm^2$

$P_t = \frac{100 \times 1256}{300 \times 450}$

S/n :

$= 0.93\%$.

From table 19, IS456.

P	τ_c (M20)
$x_1 = 0.75$	$y_1 = 0.56$
$x_2 = 1$	$y_2 = 0.62$
$x = 0.93$	$y = ?$

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Interpolation

$$y = y_1 + \frac{(y_2 - y_1)}{(x_2 - x_1)} (x - x_1)$$

$$= 0.56 + \frac{(0.62 - 0.56)}{(1 - 0.75)} (0.93 - 0.75)$$

$\tau_c = 0.603 \ N/mm^2$

$V_{uc} = 0.603 \times 300 \times 450$

$= 81.4 \ KN < V_u$

Hence shear reinforcement is required.

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

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

$= 210 - 81.4$

$= 128.56 KN$

$= 0.87 \times 418$

provide 2 legged stirrups of 8 mm ϕ

$A_{sv} = 2 \times \pi/4 \ (8)^2$

$= 100.5 \ mm^2$

$$S_v = \frac{0.87 \times 415 \times 100.5 \times 450}{\cancel{210} \ 128.56 \times 10^3}$$

$= 126.37 mm.$

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ii) $S_v = 0.75d$

$= 0.75 \times 337.5 mm$

iii) $S_v = 300 mm$

provide 2 legged stirrups of 8 mm ϕ @ 125mm c/c.

Design of torsional reinforcement beam:

RC member may be subjected to torsion in combination with bending & shear.

ii) S_v = 0 Longitudinal & transverse reinforcement shall be provided for RC beams to resist torsion

As per c.

Shear & torsion:

As per clause 41.3.

$$\text{Equivalent shear}, V_e = V_u + \left(1.6 \frac{T_u}{b}\right)$$

where,

V_u → ultimate shear force.

T_u → Torsional moment

b → width of beam.

Design based on IS 456-2000

As per clause 41.4

case (i):

If $M_u < M_{ut}$, design shall be made for equivalent moments M_{ue1} & M_{ue2}.

V_u

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$M_{ue_1} = M_u + M_{ut}$

$M_{ue_2} = M_u - M_{ut}$.

case (ii)

If $M_u > M_{ut}$.

design shall be made for equivalent Moment M_{ue_1} only.

$M_{ue_1} = M_u + M_{ut}$

$$M_{ut} = \frac{T_u(1+D/b)}{1.7}$$

where,

M_u → ultimate bending moment.

T_u → Torsional moment.

D → overall depth

b → width of beam.

1) A RC beam of overall size 230×450 mm is subjected to the following forces:

Factored bending moment = 50 KNm.

Factored torsional moment = 20 KNm.

Factored shear force = 75 KN.

use M20 grade conc. & Fe415 HYSD bar.

Determine the reinforcement required for moment & shear force. Take d = 415 mm, b_1 = 180 mm

d_1 = 380 mm.

Factored bending moment = ... KNm

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Given:

$M_u = 50 kNm$.

$T_u = 20 kNm$.

$V_u = 75 kN$

$b = 230 mm$

$D = 450 mm$.

$b_1 = 180 mm$, $d_1 = 380 mm$

$d = 415 mm$

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

$f_y = 415 N/mm^2$

Soln:

$$M_{ut} = \frac{T_u (1 + D/b)}{1.7}$$

$$= \frac{20 \times (1 + 450/230)}{1.7}$$

$$= 34.78 \text{ kNm}.$$

$M_u > M_{ut}$.

Hence design shall be made for equivalent moment M_{ue1} only.

$M_{e1} = M_u + M_{ut}$.

$M_{e1} = 34.78 + 50$

$= 84.78 kNm$.

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$$\frac{M_{e1}}{bd^2} = \frac{84.78 \times 10^6}{230 \times 415^2}$$

$$= 2.14 \, N/mm^2$$

From table 2, SP-16.,

$$P_t = 0.693 \%$$

$$\text{Ast required} = \frac{0.693}{100} \times 230 \times 415$$

$$= 661.47 \, mm^2$$

provide 20mm ϕ bar.

$$a_{st} = \pi/4 \, (20)^2$$

$$= 314 \, mm^2$$

$$\text{No. of bar} = \frac{A_{st}}{a_{st}}$$

$$= \frac{661.47}{314}$$

$$= 2.11 \simeq 3.$$

provide 3 # of 20mm ϕ bars.

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shear reinforcement:

↳ The basic origin of shear reinforcement design is based on diagonal cracking of concrete (Tension cracking concrete).

↳ At simply supports, the maximum shear force developed at the support (i.e) only shear takes place where $\sigma_x = 0$ & $\sigma_y = 0$.

↳ The shear resistance of RCC beam is based on the following principles in the absence of stirrups.

* concrete in compression can arrest diagonal tension
* Crack in a range of 20-40% of total shear.
* Interlocking among aggregates along with cement paste can develop capacity to arrest diagonal tension crack in the absence of 33-50%.
* The tension steel bars behave like dowel bar called dowel action which can arrest diagonal tension crack range of 15-25%.

providing to IS456:2000 Nominal shear stress is given by

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

compare τ_v with τ_c max.

$$\tau_{c_{max}} = 0.631\sqrt{f_{ck}}.$$

If $\tau_v > \tau_c$ max, No need to increase cross section

Purpose of providing shear reinforcement:

* To prevent the brittle shear failure
* To prevent failure that can be caused due to shrinkage & thermal stress.
* To hold main reinforcement in position.
* To protect against any sudden failure of beam.

ultimate shear force:

$V_u = V_{uc} + V_{us}$.

If suppose one of the bar is bentup then

$V_u = V_{uc} + V_{us} + V_{usb}$.

1. A RCC beam 250mm wide & 450mm deep is reinforced with 3 bars of 20mm ϕ bars of Fe415 grade steel on the tension side with an effective cover of 50mm. If the shear reinforcement of 8 mm at a spacing of 160 mm c/c is provided in a section. Determine the ultimate strength of the section.

Given:

$b = 250mm$

$D = 450mm$.

$A_{st} = 3 \times \pi/4 \times 20^2$

$= 942.47 mm^2$

$d' = 50mm$.

Stirrup $\phi = 8 mm$.

spacing of stirrup $= 160mm$ c/c.

s/n:

ultimate strength:

$V_u = V_{uc} + V_{us}$.

shear ~~stress~~ resistance by concrete:

$V_{uc} = \tau_c bd$.

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

$$= \frac{100 \times 942.47}{250 \times 400}$$

$= 0.94$

$\tau_c = 0.61 N/mm^2 \longrightarrow$ pg. 73.

$V_{uc} = 0.61 \times 250 \times 400$

$= 6100\,N.$

$V_{us} = \dfrac{0.87\, f_y\, A_{sv}\, d}{S_v}.$

$A_{sv} = 2 \times \pi/4 \times 8^2$

$= 100.53 mm^2$

$V_{us} = \dfrac{0.87 \times 415 \times 100.53 \times 400}{160}$

$= 90740.89\,N$

$V_{us} = 90.7 KN.$

$V_u = V_{uc} + V_{us}$

$= 61 \times 10^3 + 90.7 \times 10^3$

$= 151.7 \times 10^3 N.$

$V_u = 151.6 KN.$

2) The Rcc beam 250mm wide & 600mm deep is reinforced with 10mm ϕ bars with inclined stirrups at 250mm c/c with $\alpha = 60°$. longitudinal steel consist of 4 # of 20mm ϕ with cover of 40mm. use M25 grade concrete & Fe415 grade steel. Determine the ultimate strength in shear.

Given:

$b = 250 mm$

$D = 600 mm$

$A_{st} = 4 \times \pi/4 \times 20^2$

$= 1256.63 mm^2$

$d' = 40 mm$

$d = 560 mm.$

Stirrup $\phi = 10 mm.$

$\alpha = 60°$

spacing $= 250 mm\ c/c.$

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Soln:

ultimate Shear Strength, $V_u = V_{uc} + V_{us}$.

$V_{uc} = \tau_c bd$.

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

$= \frac{100 \times 1256.63}{250 \times 560}$.

$P_t = 0.89$.

$\tau_c = 0.62 N/mm^2 \rightarrow$ Table 19.

$V_{uc} = 0.62 \times 250 \times 560$

$= 86800 N$.

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

$= \frac{0.87 \times 415 \times (\pi/4 \times 10^2) \times 560}{250} (\sin 60^\circ + \cos 60)$

$V_{us} = 173537.77 N$

$V_u = (86.8 + 173.5)$

$V_u = 260.33 kN$//.

3) A RC beam of Rectangular Section 350mm wide is reinforced with 4 bars of 20mm ϕ at an effective depth of 550mm out of which 2 bars are bent up near the support where a factored Shear force of 400kN is used. use M20 grade concrete & Fe 415 HYSD bars.

Given:

$b = 350mm$ $\quad f_{ck} = 20N/mm^2$

$d = 550mm$ $\quad f_y = 415N/mm^2$

$V_u = 400KN$

$A_{st} = 2 \times \pi/4 \times 20^2$

$= 628.315mm^2$

~~step~~

s/n:

$V_u = V_{uc} + V_{us}$

$V_{uc} = \tau_c bd$.

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

$$= \frac{100 \times 628.3}{350 \times 550}$$

$= 0.32 N/mm^2$

$\tau_c = 0.38 N/mm^2 \longrightarrow$ Table 19.

$V_{uc} = 0.38 \times 350 \times 550$

$= 73.15 KN$.

$V_{us} = 0.87 f_y A_{sv} \sin\alpha$ $\quad (\alpha = 45^\circ \rightarrow pg.73)$

$= 0.87 \times 415 \times \sin 45^\circ$

$= 160.41 KN$.

$V_u = 73.15 + 160.41$

$= 233.55 KN$//.

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Limit state Design : Torsion :

↳ Torsion is a twisting moment acting about z axis of the member due to external load applied on the structure.

↳ Every twisting moment are torsion develops bending moment & shear force & hence every RCC beam subject to torsion must be provided with longitudinal steel & transverse steel.

↳ There are two types of torsion

i) statically determinate torsion (Equilibrium torsion)

ii) statically indeterminate torsion (compatibility torsion)

Statically determinate torsion:

↳ It is also known as primary torsion and equilibrium torsion.

↳ It occurs mainly due to external applied load (eccentrical load).

↳ eccentrical load is resisted by torsion to maintain equilibrium of structure and hence it is known as equilibrium torsion.

↳ It mainly occurs in cantilever slab supported by longitudinal beam.

$T = P \times e$.

$P \rightarrow$ Eccentrical load

$e \rightarrow$ eccentricity

↳ It also occurs in circular beam.

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statically indeterminate torsion:

↳ It is also known as secondary torsion (or) compatibility torsion because it occurs due to secondary effect like rotation of joints and because compatibility equations are used to determine the torsion.

↳ It occurs due to continuity of joints & members.

↳ It also occurs due to stiffness of structural members & torsional rigidity of member.

According to IS456:2000, the equivalent shear force due to torsion & shear is given by.

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

equivalent nominal shear stress

$$\tau_{ve} = \frac{V_e}{bd}.$$

The equivalent B.M according to IS456:2000 is

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

$$M_t = \frac{T_v [1 + D/b]}{1.7}$$

Due to M_{e1} torsion steel shall be provided by using equation

$$M_{e1} = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{f_{ck} bd}\right]$$

If Me exceeds Mu compression steel shall be provided based on.

$$M_{e2} = M_t - M_u$$

$$= \frac{T_u (1 + D/b)}{1.7} - M_u.$$

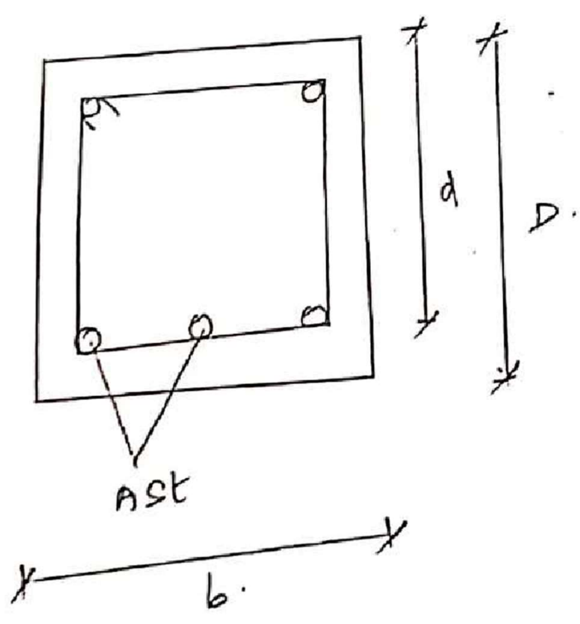
$$A_{sc} = \frac{M_{e2}}{0.87 f_y (d - d')}$$

According to IS 456:2000 pg. No 75.

If Tve exceeds Tc,
then stirrups shall be designed

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

Torsional Reinforcement:



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Determine the reinforcement required for a beam of of b=400mm, D=700mm & d=650mm subjected to a factored moment of 200 KNm & factored torsional moment is 500 KNm & factored shear force is 100 KN. use M20 grade concrete & Fe415 HYSD bar.

Given:

$b = 400mm$

$D = 700mm$

$d = 650mm$

$T_u = 50$ KNm.

$f_{ck} = 20$ N/mm²

$M_u = 200$ KNm.

$f_y = 415$ N/mm²

$V_u = 100$ KN.

step 1: calculation of equivalent shear:

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

$$= 100 + \frac{1.6 \times 50}{400}$$

$V_e = 300$ KN.

step 2: Equivalent nominal shear stress:

$$\tau_{ve} = \frac{V_e}{bd}$$

$$= \frac{300 \times 10^3}{400 \times 650}$$

$$= 1.15 \text{ N/mm}^2$$

Assume, $P_t = 0.5\%$.

$\tau_c = 0.48$ N/mm².

$\tau_{ve} > \tau_c$, Provide longitudinal & transverse reinforcement.

Step 3: Equivalent Moment (Mt):

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

$$= \frac{50 \left(1 + \frac{0.7}{0.4}\right)}{1.7}$$

$M_t = 80.88$ KNm.

$M_t < M_u$, then (Singly reinforced)

$M_{e1} = M_u + M_t$

$M_e = 200 + 80.88$

$= 280.88$ KNm.

Step 4: Longitudinal reinforcement:

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

$$280.88 \times 10^6 = 0.87 \times 415 \times A_{st} \times 650 \left[1 - \frac{A_{st} \times 415}{20 \times 400 \times 650}\right]$$

$A_{st} = 1340.17 \text{ mm}^2$

Provide 20 mm ϕ bar.

$a_{st} = \pi/4 \times 20^2$

$a_{st} = 550 \text{ mm}^2$.

No. of bar $= \frac{A_{st}}{a_{st}}$

$= 2.4 \simeq 3$

Provide 3 # of 20mm ϕ bar.

Step 5: Transverse reinforcement:

$$A_{sv} = \frac{T_u \cdot s_v}{b d (0.87 f_y)} + \frac{V_u s_v}{2.5 d (0.87 \times f_y)}$$

$$A_{sv} = 2 \times \pi/4 \times 8^2$$

$$2 \times \pi/4 \times 8^2 = \frac{50 \times 10^6 \times s_v}{200 \times 600 \times (0.87 \times 415)} + \frac{100 \times 10^3 \times s_v}{2.5 \times 600 (0.87 \times 415)}$$

$$s_v = 105.42\,mm$$

2) A RC rectangular beam B = 300mm & d = 600mm, D = 650mm subjected to a factored shear force of 70 kN assuming the % of tensile reinforcement at 0.5 determine the factored torsion moment that the section can resist if case (i) No additional reinforcement for torsion is provided case (ii) Max. steel for torsion is provided (iii) Determine the reinforcement needed for case (ii) Assume M30 grade concrete & Fe500 for longitudinal reinforcement. Fe415 for transverse reinforcement.

Given:

$b = 300\,mm$	$V_u = 70\,kN$
$d = 600\,mm$	$f_{ck} = 30\,N/mm^2$
$D = 650\,mm$	$f_y = 500\,N/mm^2 \rightarrow$ for longitudinal
$P_t = 0.5$	$f_y = 415\,N/mm^2 \rightarrow$ for transverse

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Step 1: calculation of τ_c & τ_c max:

$P_t = 0.5$

$\tau_c = 0.5\ N/mm^2 \rightarrow$ Table 19.

$\tau_{c\,max} = 3.5\ N/mm^2 \rightarrow$ Table 20

case (i) Torsional moment:

when no additional reinforcement is provided

$$\tau_{ve} = \tau_c$$

$$\tau_{ve} = 0.5\ N/mm^2$$

$$\tau_{ve} = \frac{V_e}{bd}$$

$$0.5 = \frac{V_e}{300 \times 600}$$

$$V_e = 90\ KN$$

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

$$90 = 70 + 1.6\frac{T_u}{0.3}$$

$$T_u = 3.75\ KNm.$$

case (ii) Max. Reinforcement is provided:

$$\tau_{ve} = \tau_c\ max$$

$$= 3.5\ N/mm^2$$

$$\tau_{ve} = \frac{V_e}{bd}$$

$$3.5 = \frac{V_u}{300 \times 600}$$

$$V_e = 630KN$$

$$Ve = Vu + 1.6\frac{Tu}{b}$$

$$680 = 70 + 1.6\frac{Tu}{0.3}$$

$Tu = 37.75$ KN

$Tu = 105$ KNm.

case(iii) Determination of reinforcement for case(ii)

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

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

$$0.5 = \frac{100\ Ast}{300 \times 600}$$

$Ast = 900\text{mm}^2$

$$Mu = 0.87 \times 500 \times 900 \times 600\left[1 - \frac{500 \times 900}{300 \times 600 \times 30}\right]$$

$Mu = 215.325$ KNm.

$$Mt = \frac{Tu}{1.7}\left[1 + D/b\right]$$

$$= \frac{105}{1.7}\left[1 + \frac{0.65}{0.3}\right]$$

$Mt = 195.58$ KNm.

$Mu > Mt$

$Me_1 = Mu + Mt$

$Me_1 = 215.325 + 195.58$

$Me_1 = 410.905$ KNm.

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$$410.95\times10^6 = 0.87\times 500\ Ast\ 600\left[1-\frac{500\times Ast}{300\times600\times30}\right]$$

$$Ast = 1913.29\ mm^2$$

Provide 25 mmφ

$$ast = \pi/4 \times 25^2 = 491$$

$$\text{No. of bar} = \frac{Ast}{ast}$$

$$= \frac{1913.29}{\pi/4\times25^2} = 3.9$$

$$\simeq 4.$$

Provide 4 # of 25mmφ bar.

Transverse Reinforcement:

$$Asv = \frac{T_u S_v}{b_1 d_1\, 0.87 f_y} + \frac{V_u S_v}{2.5 d_1 (0.87 f_y)}$$

$$Asv = 2\times\pi/4\times 8^2$$

$$100.53 = \frac{105\times10^6\times sv}{300\times600\times0.87\times415} + \frac{70\times10^3\times sv}{2.5\times500(0.87\times415)}$$

$$sv =$$

(ii)

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

$$100.53 = \frac{(3.5-0.5)\times300\times sv}{0.87\times415}$$

$$sv = 40.32\ mm.$$

check for spacing

i) $S_v = \frac{x_1 + y_1}{4}$

$= \frac{275 + 575}{4}$

$= 212.5$ mm.

ii) $S_v = x_1 = 275$ mm

iii) 300 mm.

side face Reinforcement :

$= 0.1\%\ bD$

$= \frac{0.1}{100} \times 300 \times 650$

$= 195 mm^2$

Provide 10mm ϕ bar

$a_{st} = \pi/4 \times 10^2$

$= 78.5 mm^2$

No. of bar $= \frac{195}{78.5}$

$\simeq 3.$

provide 3 # of 10mm ϕ bar.

UNIT- III

DESIGN OF SLABS AND STAIRCASE

Introduction :

↳ Reinforced concrete slabs constitute the most common type of structural elements used to cover floor & roofs.

↳ oneway It is classified as

* One way slab.
* Two way slab.

One way slab:

↳ oneway slabs are supported on opposite sides & the loads are transmitted in one direction.

(e.g) verandah slab.

$$L_y / L_x \geq 2.$$

↳ The ratio of longer span to shorter span is greater than 2 is classified as oneway slab.

L_y → Longer span.

L_x → Shorter span.

Two way slab:

↳ Reinforced concrete slabs supported on all the four edges with the ratio of long to short span not exceeding 2 are reffered as two way slab.

↳ In this type, the loads are transmitted to the supports in both direction with main reinforcements provided in mutually perpendicular direction.

$$L_y / L_x < 2.$$

Analysis & Design of cantilever slab:

Cantilever slab:

↳ Reinforced concrete slabs projecting from fixed end & free at other end are reffered to as cantilever slabs.

(e.g) Chajjas & balconies.

↳ In general, the depth of cantilever slab is based on span / depth ratio of specified in IS 456 : 2000.

↳ It is important to provide the required anchorage length near supports to the main reinforcement to prevent failure due to anchorage.

Design of cantilever Slab:

1. Design a cantilever chajja slab projecting 1m from the support using M20 grade concrete & Fe 415 HYSD bars. Adopt live load of 3 kN/m².

Step 1: Data:

$L = 1m$

$q = 3 kN/m^2$

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

$f_y = 415 N/mm^2$

$\tau_{bd} = 1.2 N/mm^2$ for plain bars of M20 grade concrete.

Step 2: Depth of slab:

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

$= \frac{1000}{7}$

$= 142.8mm$

$\simeq 150 mm$. [Assume cover = 25mm]

$d' = 25mm$

$D = d + d'$

$= 150 + 20$

$= 175mm$.

Adopt max. depth of 150mm at support & gradually reducing to 100mm at free end.

Step 3: Loads:

Self weight $= 0.5\,(0.15 + 0.10)\,25$

$= 3.125$ KN/m.

Live load $= 3$ KN/m.

Finishes $= 0.875$ KN/m.

Total load $= 7.000$ KN/m.

ultimate load $= 10.5$ KN/m.

Step 4: ultimate moment & shear forces:

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

$$= \frac{10.5 \times 1^2}{2}$$

$= 5.25$ KNm.

$V_u = W_u l$

$= 10.5 \times 1$

$= 10.5$ KN.

Step 5: Check for depth:

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

$= 0.138 \times 20 \times 1000 \times 150^2$

$= 62.1 \times 10^6$ N mm

$= 62.1$ KNm.

$M_u < M_{u\,lim}$.

$\therefore$ The section is under reinforced section.

Step 6: Reinforcements:

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

$$5.25 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \left[1 - \frac{A_{st} \times 415}{20 \times 1000 \times 150}\right]$$

$$A_{st} = 105 mm^2.$$

$$a_{st} = \pi/4 \, d^2$$
$$= \pi/4 \times 10^2$$
$$= 78.5 mm^2$$

$$spacing = \frac{1000 A_{st}}{0.4 b}$$
$$= \left[\frac{1000 \times 78.5}{0.4 \times 1000}\right]$$
$$= 190 mm.$$

provide 10mm ϕ @ 190 mm c/c spacing.

Step 7: Anchorage length:

$$L_d = \left[\frac{0.87 f_y \phi}{4 \tau_{bd}}\right]$$
$$= \left[\frac{0.87 \times 415 \times 10}{4 \times 1.2 \times 1.6}\right]$$
$$= 470 mm.$$

Step 8: check for deflection:

$$(L/d)_{max} = [(L/d)_{basic} \times K_t]$$

$$P_t = \frac{100\,A_{st}}{bd} = \frac{100 \times 262}{1000 \times 150}$$

$$= 0.174.$$

From fig 5.1

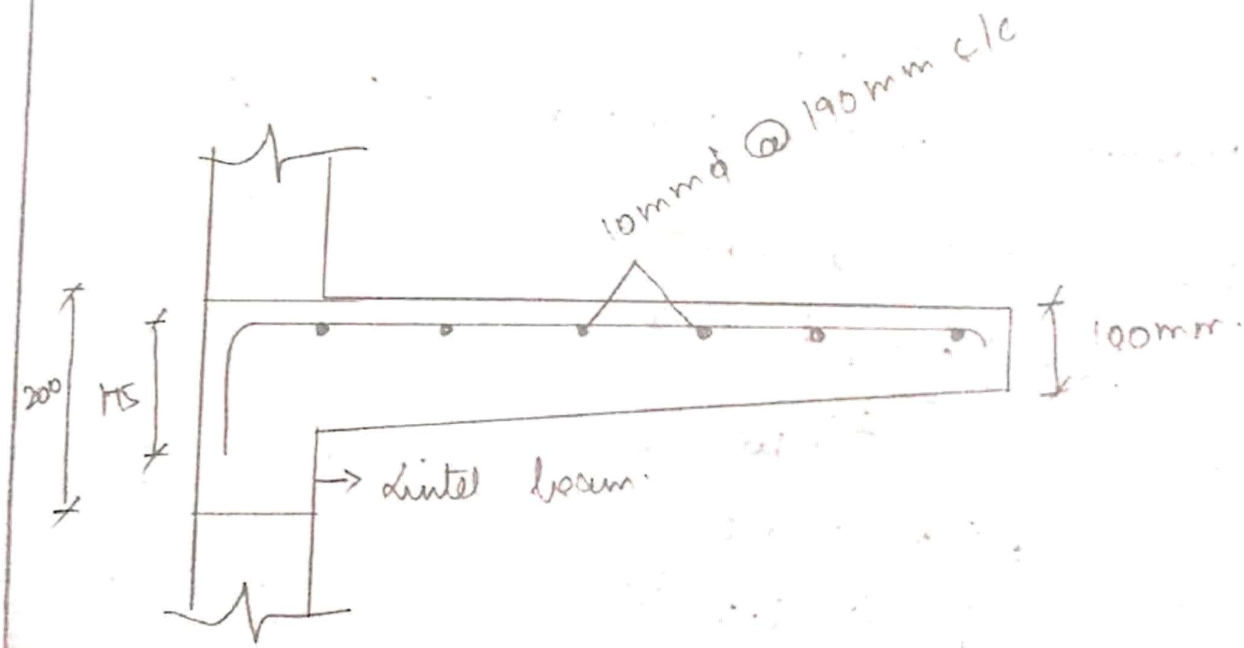
$$K_t = 1.8.$$

$$(L/d)_{max} = (9.7 \times 1.8)$$
$$= 12.6.$$

$$(L/d)_{actual} = (1000/150) = 6.66 < 12.6$$

$$(L/d)_{max} > (L/d)_{actual}.$$

∴ Hence the limit state of deflection is satisfied.



Design of one way slab:

Design a one way slab with a clear span of 3.5m, simply supported on 200 mm thick concrete masonry walls to support a live load of $4 KN/m^2$. Adopt M20 grade concrete & Fe 415 HYSD bars.

Step 1: Data

Clear span = 3.5m.

width = 200 mm

L.L $= 4 KN/m^2$

F.F $= 1 KN/m^2$

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

$f_y = 415 N/mm^2$

Step 2: Depth of slab:

$$d = \frac{span}{25}$$

$$= \frac{3500}{25}$$

$$= 140 mm.$$

Assume, $d' = 25 mm.$

$$D = d + d'$$

$$= 140 + 25$$

$$D = 165 mm.$$

Step 3: Effective span:

i) $l_{effective}$ = clear span + effective depth

$$= 3.5 + 0.14$$

$$= 3.64 m.$$

ii) l effective = Centre to Centre Of Support

$$= \frac{0.20}{2} + 8.5 + \frac{0.20}{2}$$

$$= 3.70 \, m.$$

l effective = 3.64 m [Take least value]

Step 4: Loads:

Self weight of Slab = b × D × density of concrete

$$= 1 \times 0.165 \times 25$$

$$= 4.125 \, KN/m.$$

Live load = 4

Floor finish = 1

Total load, $w = 9.125 \, KN/m.$

ultimate load. $W_u = 1.5 \times 9.125$

$$= 13.69 \, KN/m.$$

Step 5: ultimate moment & Shear force:

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

$$= \frac{13.69 \times 3.64^2}{8}$$

$$= 22.67 \, KNm.$$

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

$$= \frac{13.69 \times 3.64}{2}$$

$$= 24.92 \, KN.$$

Step 6: Limiting Moment of resistance:

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

$= 0.138 \times 20 \times 1000 \times 140^2$

$= 54 \times 10^6$ Nmm

$= 54$ KNm.

$M_u < M_{u\,lim}$

∴ The Section is under reinforced Section.

Step 7: Main reinforcement:

From pg. 96.

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

$22.67 \times 10^6 = 0.87 \times 415 \times A_{st} \times 140 \left[1 - \frac{A_{st} \times 415}{20 \times 140 \times 1000}\right]$

$A_{st} = 480\ mm^2$.

$a_{st} = \pi/4 \times 10^2$

$= 78.5\ mm^2$

$\text{Spacing} = \frac{1000 \times a_{st}}{0.4\cancel{16}}$

$= \frac{100\cancel{0} \times 78.5}{0.4 \times 1000}$

$= \cancel{104}\ 196$ mm.

spacing $= \frac{a_{st}}{\cancel{A_{st}}} \times 1000$

$= \frac{78.5}{480} \times 1000$

$= 163$ mm

provide 10 mm φ @ ~~196~~ 160 mm c/c spacing.

Step 8: Distribution reinforcement:

$$Ast = 0.12\% \; b \times D$$
$$= \frac{0.12}{100} \times 1000 \times 165$$
$$= 198 \text{ mm}^2.$$

$$ast = \pi/4 \, d^2$$
$$= \pi/4 \times 8^2$$
$$= 50 \text{ mm}^2.$$

$$\text{spacing} = \frac{ast}{Ast} \times 1000$$
$$= \frac{50}{198} \times 1000$$
$$= 252 \text{ mm}$$
$$\simeq 250 \text{ mm}.$$

Provide 8 mm ϕ @ 250 mm c/c spacing.

Step 9: Check for shear stress:

$$\tau_v = \frac{V_u}{bd}$$
$$= \frac{24.92 \times 10^3}{1000 \times 140}$$
$$= 0.178 \text{ N/mm}^2$$

$$P_t = \left[\frac{100 \, Ast}{bd}\right] 50\% \text{ of reinforcement}$$
$$= \frac{100 \times 480}{1000 \times 140} \times 0.5$$
$$= 0.17.$$

$K = 1.27$

$\tau_c = 0.28$ $\longrightarrow$ From table 19 of IS456.

$K \cdot \tau_c = 1.27 \times 0.28$

$= 0.35 \text{ N/mm}^2$

$\tau_c > \tau_v$

∴ The shear stresses are with in the safe permissible limits.

Step 10: check for deflection:

$$(L/d)_{max} = \left[(L/d)_{basic} \times K_t \times K_c \times K_f \right]$$

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

$$= \frac{100 \times 480}{1000 \times 140}$$

$$= 0.34.$$

From fig 5.1

$K_t = 1.40$

From fig 5.2

$K_c = 1$

From fig 5.3.

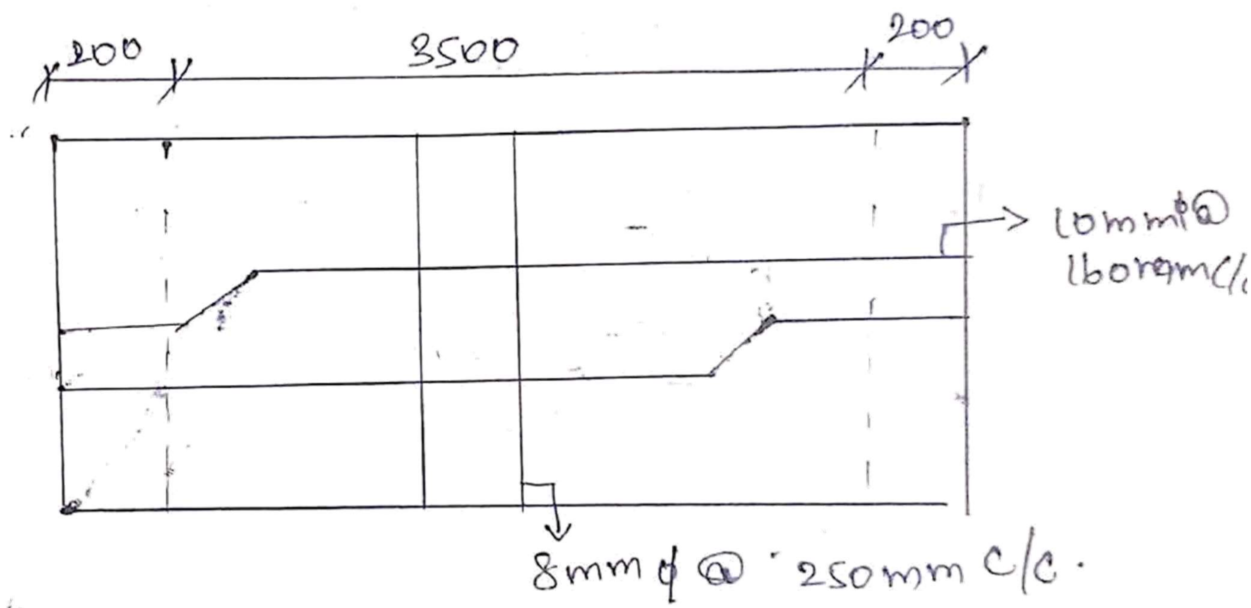
$K_f = 1$

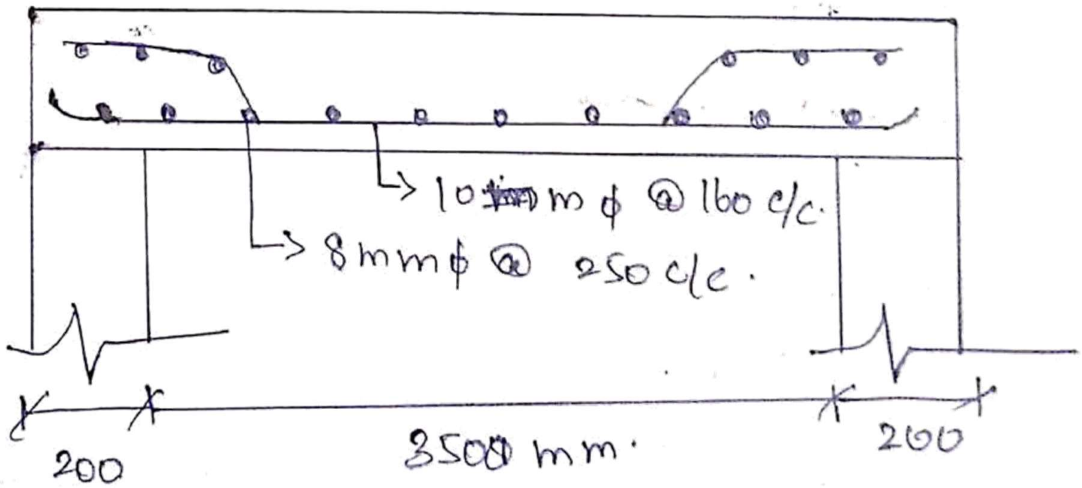
$$[L/d]_{max} = 20 \times 1.40 \times 1 \times 1$$
$$= 28$$

$$(L/d)_{actual} = \frac{3640}{140}$$
$$= 26$$

$$[L/d)_{max} > (L/d)_{actual}$$

Hence the limit state of deflection is satisfied.





continous slab:

→ In multistory buildings comprising tee beam & slab floor, the slabs are continous over the beam which are spaced at regular intervals.

Design of continous slab:

1. Design a one way slab for an office floor which is continous over tee beams spaced at 3.5m intervals. Assume a live load of 4 KN/m² & adopt M20 grade concrete & Fe 415 HYSD bars.

Step 1: Data:

$L = 3.5m$

$q = 4\ KN/m^2$

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

$f_y = 415\ N/mm^2$

Step 2: Depth of slab:

$$d = \frac{span}{26} \longrightarrow \text{From clause 23.2.1 of IS-456}$$

$$= \frac{3500}{26}$$

$$= 135mm$$

$$d \simeq 140mm.$$

$$D = 140 + 25$$

$$= 165\ mm.$$

Step 3: Loads:

Self weight $= 0.165 \times 25$

$= 4.125\ KN/m^2$

Finishes $= 0.875\ KN/m^2$

Total dead load $= 5\ KN/m^2$

Live load $= 4\ KN/m^2$

Step 4: Bending Moment & Shear force:

From table 12 & 13 of IS 456-200.

$$M_u(-ve) = 1.5\left[\frac{gL^2}{10} + \frac{qL^2}{9}\right]$$

$$= 1.5\left[\frac{5 \times 3.5^2}{10} + \frac{4 \times 3.5^2}{9}\right]$$

$$= 17.35\ KNm.$$

$$M_u(+ve) = 1.5\left[\frac{gL^2}{12} + \frac{qL^2}{10}\right]$$

$$= 1.5\left[\frac{5 \times 3.5^2}{12} + \frac{4 \times 3.5^2}{10}\right]$$

$$= 15\ KN/m.$$

Max. Shear force at the support section

$$V_u = 1.5 \times 0.6\,(g+q)L$$

$$= (1.5 \times 0.6)(5+4)\,3.5$$

$$= 28.35\ KN.$$

Step 5: Check for depth of slab:

$$M_{u,lim} = 0.138 f_{ck} b d^2$$
$$= 0.138 \times 20 \times 1000 \times 140^2$$
$$= 54.1 \times 10^6 \text{ Nmm}$$
$$= 54.1 \text{ KNm.}$$

$M_u < M_{u,lim}$

$\therefore$ section is under reinforced section.

Step 6: Reinforcements:

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

$$17.3 \times 10^6 = 0.87 \times 415 \times A_{st} \times 140 \left[1 - \frac{A_{st} \times 415}{20 \times 1000 \times 140}\right]$$

$A_{st} = 360 mm^2$.

Provide 10 mm ϕ bar

$$a_{st} = \pi/4 \times 10^2$$
$$= 78.5 mm^2.$$

$$\text{Spacing} = \frac{1000 A_{st}}{0.4 b}$$
$$= \frac{1000 \times 78.5}{0.4 \times 1000}$$
$$= 196.25 \simeq 200 mm$$

Provide 10 mm ϕ @ 200 mm c/c spacing.

Distribution steel = 0.12 % bD

$$= \frac{0.12}{100} \times 1000 \times 165$$

$Ast = 198\ mm^2$.

Provide 10mm ϕ bar.

$ast = \pi/4 \times \cancel{10d}^2$

$= \pi/4 \times 10^2$

$= 78.5\ mm^2$

Spacing $= \frac{1000\ Ast}{0.4\ b}$

$= \frac{1000 \times \cancel{198}\ 78.5}{0.4 \times 1000}$

$= 196$

$\simeq 200mm$

Provide 10mm ϕ bar @ 200mm c/c spacing.

Step 7: Check for shear stress:

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

$= \frac{28.35 \times 10^3}{1000 \times 140}$

$= 0.20\ N/mm^2$

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

$= \frac{100 \times 262}{1000 \times 140} = 0.187$

Table 19, IS - 456.

$K \cdot \tau_c = (1.27 \times 0.30)$

$= 0.38 \ N/mm^2$

$\tau_c > \tau_v$

∴ slab is against the shear stresses:

Step 8: check for deflection:

$[L/d)_{max} = [(L/d)_{basic} \times K_c \times K_f]$

$K_c = K_f = 1.$

$P_t = \left[\frac{100 \times Ast}{bd} \right]$

$= \left[\frac{100 \times 393}{1000 \times 140} \right]$

$= 0.28$

From fig 5.1

$K_t = 1.5$

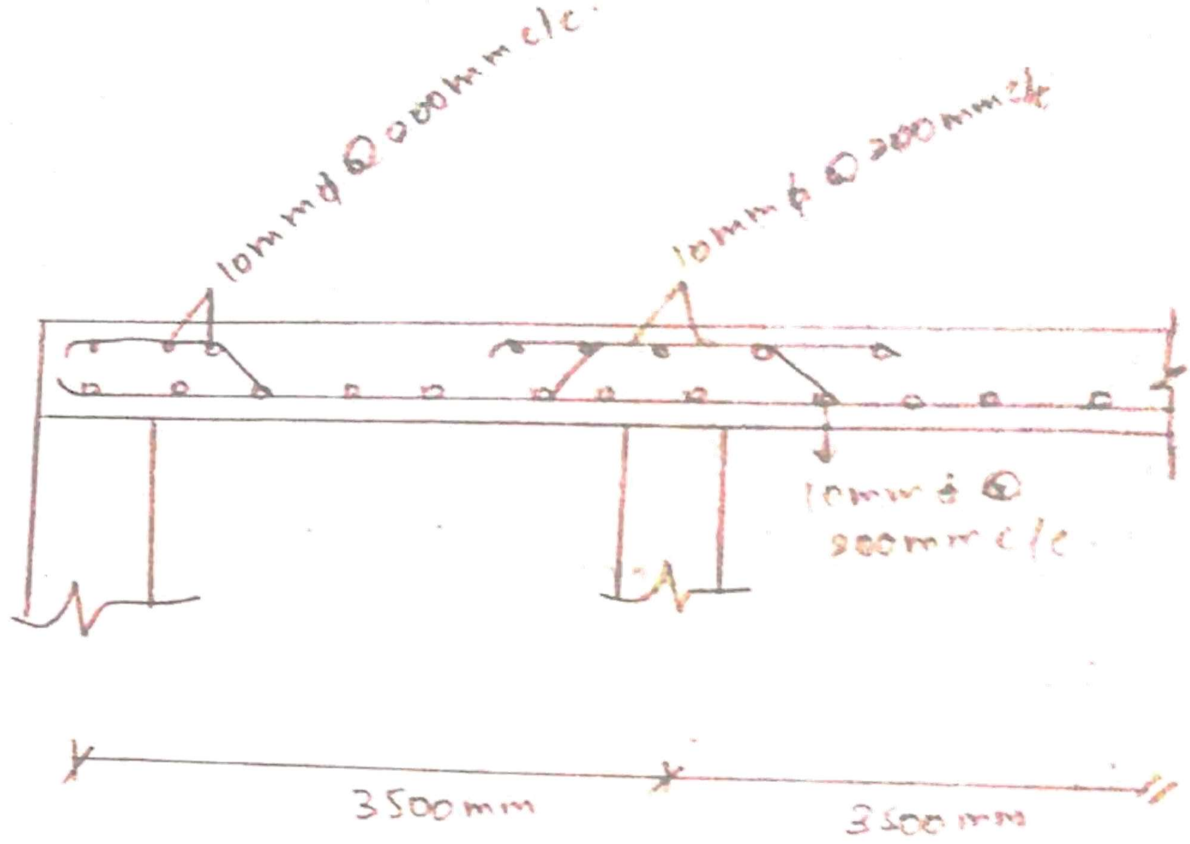
$(L/d)_{max} = \left[\frac{20+26}{2} \right] 1.5$

$= 34.5.$

$(L/d)_{actual} = \left[\frac{3500}{140} \right] = 25 < 34.5$

$(L/d)_{max} > (L/d)_{actual}$.

∴ The slab is safe against the deflection control.


10mm ϕ @ 200mm c/c
10mm ϕ @ 200mm c/c
10mm ϕ @ 200mm c/c
3500mm
3500mm

Design of Two way slab:

1. Design a two way slab for an office floor of size 3.5m by 4.5m with discontinous & simply supported edges on all the sides with corners prevented from lifting & supporting a service live load of 4 KN/m^2. Adopt M20 grade concrete & Fe 415 HYSD bars.

step 1:

$L_x = 3.5m$

$L_y = 4.5m.$

$(L_y/L_x) = (4.5/3.5)$

$= 1.28 < 2$

Hence it is a two way slab.

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

$f_y = 415 N/mm^2$

step 2: Depth of slab:

$d = \frac{span}{25}$

$= \frac{3500}{25}.$

$= 140mm.$

$D = d + d'$

$= 140 + 25$

$= 165 mm.$

Step 3: Effective span:

i) $l_{effective}$ = clear span + Effective depth

$= 3.5 + 0.14$

$= 3.64 mm$

ii) $l_{effective}$ = clear span + centre to centre of support

$= \frac{0.2}{2} + 3.5 + \frac{0.2}{2}$

$= 3.7 mm$

$l_{effective} = 3.64 mm$ [Take least value]

Step 4: Loads:

Self weight of slab = b × D × density of concrete

$= \cancel{1000}\,1 \times 0.165 \times 25$

$= 4.125\ KN/m^2$

Live load $= 4$

Floor finish $= 0.6$

Total load, $w = 8.725\ KN/m^2$

ultimate load, $w_u = 1.5 \times 8.725$

$= 13.08\ KN/m^2$.

Step 5: ultimate moment & shear force:

From Table 26 of IS 456.

$L_y / L_x = 1.28$.

$\alpha_x = 0.47$

$\alpha_y = 0.056$.

$M_{ux} = \alpha_x W_u L_x^2$

$= 0.047 \times 13.08 \times 3.64^2$

$= 13.34 \text{ KNm}.$

$M_{uy} = \alpha_y W_u L_x^2$

$= 0.056 \times 13.08 \times 3.64^2$

$= 9.70 \text{ KNm}.$

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

$= \frac{13.08 \times 3.64}{2}$

$= 23.8 \text{ KN}.$

Step 6: check for depth:

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

$d = \sqrt{\frac{13.34 \times 10^6}{0.138 \times 20 \times 10^3}}$

$= 69.52 \text{ mm} < 140 \text{ mm}.$

Hence the effective depth selected is adequate.

Step 7: Reinforcement:

Short span:

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

$13.34 \times 10^6 = 0.87 \times 415 \times A_{st} \times 140 \left[1 - \frac{A_{st} \times 415}{20 \times 1000 \times 140}\right]$

$A_{st} = 980 \text{ mm}^2.$

$a_{st} = \pi/4 \times 12^2$

$= \cancel{78.8\,mm^2}\ 113\,mm^2$

Spacing $= \dfrac{\cancel{78.8}}{}\ \dfrac{a_{st}}{A_{st}} \times 1000$

$= \dfrac{113}{980} \times 1000$

$= 115\,mm.$

Provide 12mm ϕ @ 115mm c/c spacing.

Long span:

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

$9.7 \times 10^6 = 0.87 \times 415 \times A_{st} \times 140 \left[1 - \dfrac{A_{st} \times 415}{20 \times 1000 \times 140}\right]$

$A_{st} = 213\,mm^2.$

Provide 10mm ϕ bar.

$a_{st} = \pi/4 \times 10^2$

$= 78.5\,mm^2$

Spacing $= \dfrac{a_{st}}{A_{st}} \times 1000$

$= \dfrac{78.5}{213} \times 1000$

$= 368\,mm$

$\simeq 300\,mm$

Provide 10mm ϕ @ 300mm c/c spacing.

Step 8: check for shear stress:

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

$$= \frac{23.8 \times 10^3}{1000 \times 140}$$

$$= 0.17 \ N/mm^2$$

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

$$= \frac{100 \times 315}{1000 \times 140}$$

$$= 0.225.$$

From table 19, IS:456.

$$K \cdot \tau_c = 1.27 \times 0.31$$

$$= 0.39.$$

$$K \cdot \tau_c > \tau_v.$$

Hence, the shear stresses are within safe permissible limits.

Step 9: check for deflection:

$$(L/d)_{basic} = 20$$

$$P_t = 0.225.$$

From, fig 5.1,

$$K_t = 1.6.$$

$$(L/d)_{max} = 20 \times 1.6$$

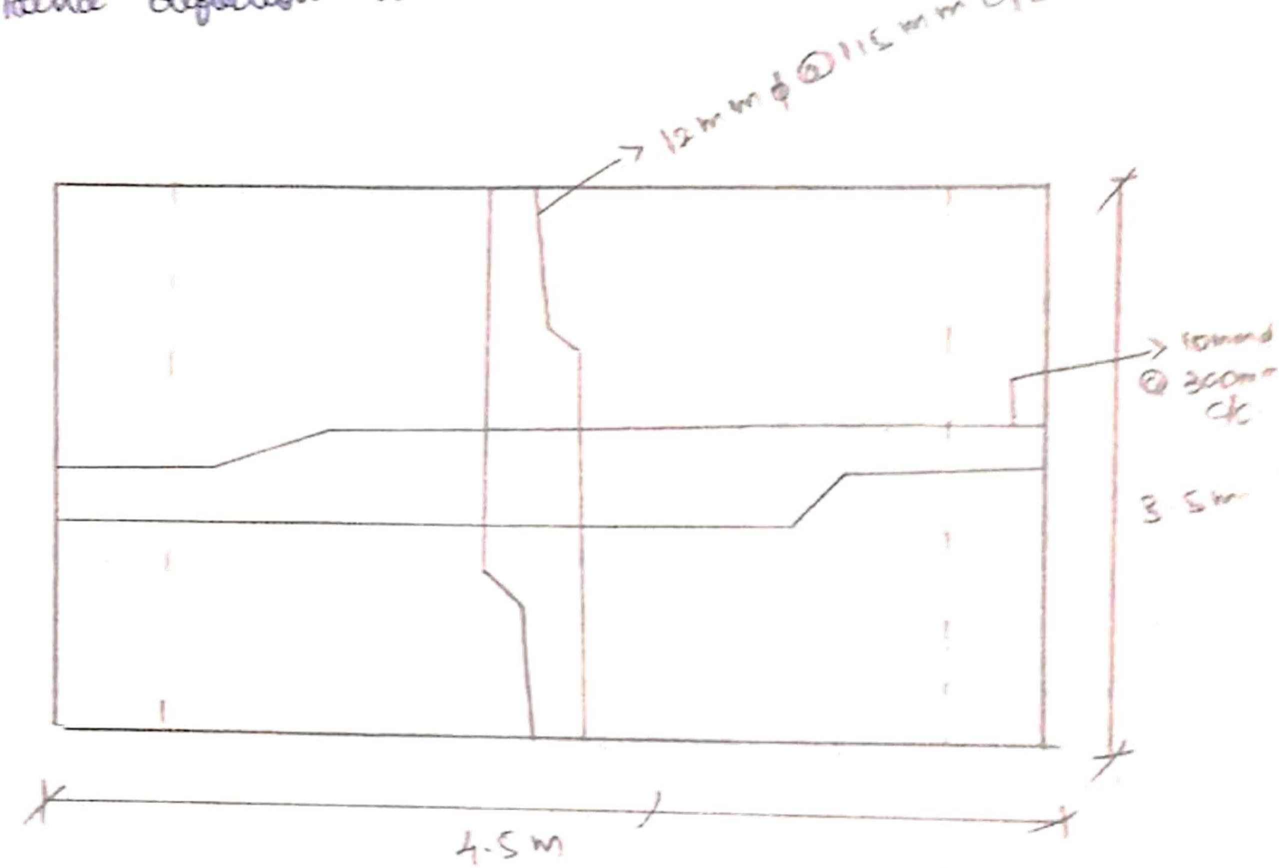
$$= 32.$$

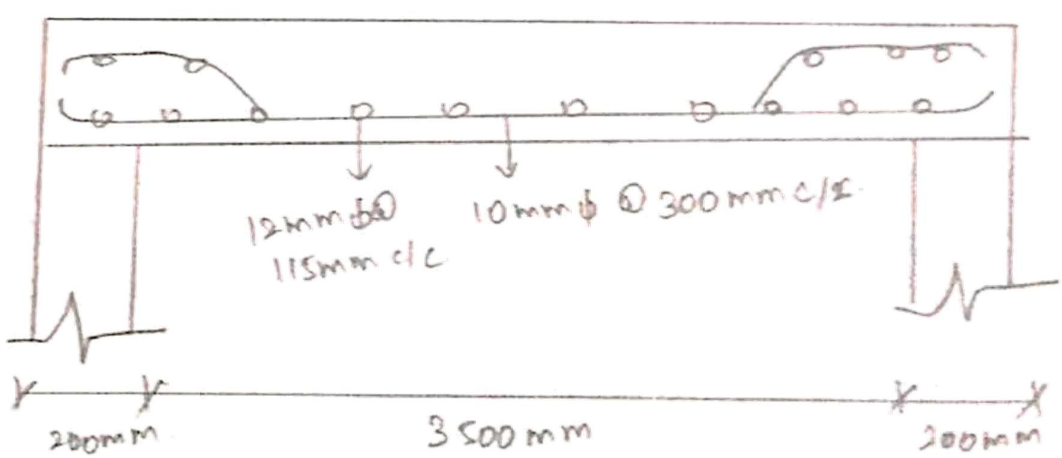
$$(L/d)_{actual} = \left(\frac{3640}{140}\right)$$

$$= 26 < 32.$$

$$(L/d)_{max} > (L/d)_{actual}.$$

Hence deflection control is satisfied.



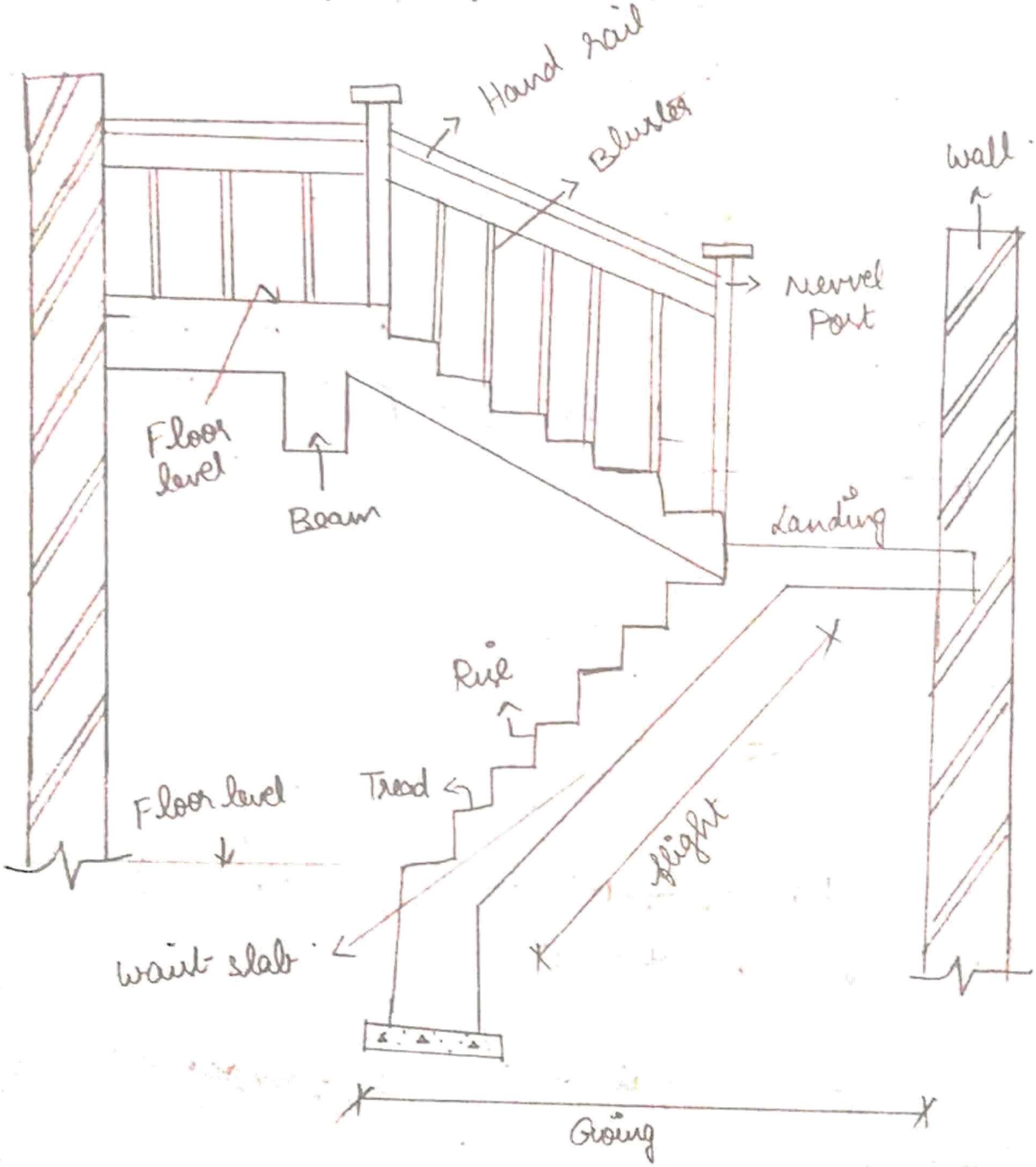


Stair case:

Stair case are provided to connect successive floors of a building & they are the means of access b/w the floor.

Structural component of a flight of a staircase:

↳ Flight of steps
↳ Landing b/w floor level.



Tread:
It is the horizontal portion of the step.

Rise:
It is the vertical distance b/w the adjacent treads.

For residential building
Tread = 200 - 250mm
Rise = 150 - 180mm.

Public building
Tread = 200 - 300mm
Rise = 120 - 150mm.

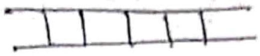
Going:
It forms the horizontal plan projection of an inclined flight of steps b/w the first and last riser.

width:
width of flight is 1-2m depending upon the type of building.

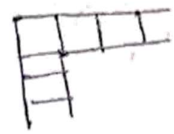
Landing:
It is a flat plat form provided b/w the flights.

Types of staircase:

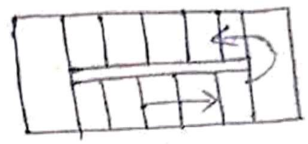
↳ straight stair



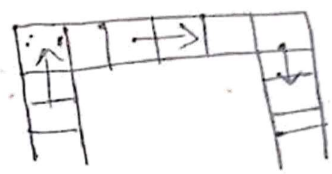
↳ Quarter turn (90°)



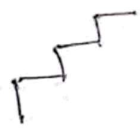
↳ Dog legged (180°)

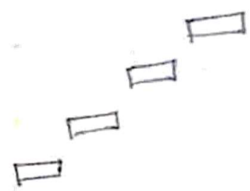


↳ open well stair case.



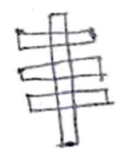
↳ Tread & riser





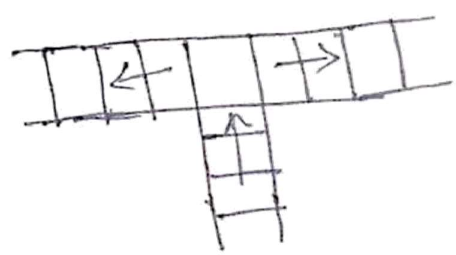
↳ Isolated cantilever staircase

↳ Double cantilever precast stair



↳ Geometrical stair case (curved stair case)

↳ Bifurcated stair case



Loads on staircase:

1) Dead load → 1. S.W of waist slab.
2. S.W of step
3. Finish load.

↳ Live load.

Self weight of slab:

$$W_s = b \times D \times \text{density of concrete}$$

Dead load of slab:

$$W = \frac{W_s \sqrt{R^2 + T^2}}{T}$$

Self weight of step:

$$= R/2 \times b \times \text{density of concrete}$$

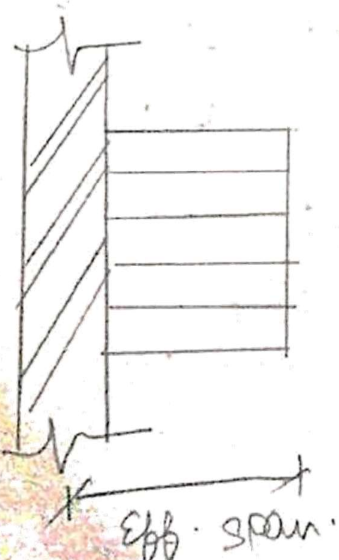
Self weight of finish = 0.5 - 0.75 kN/m.

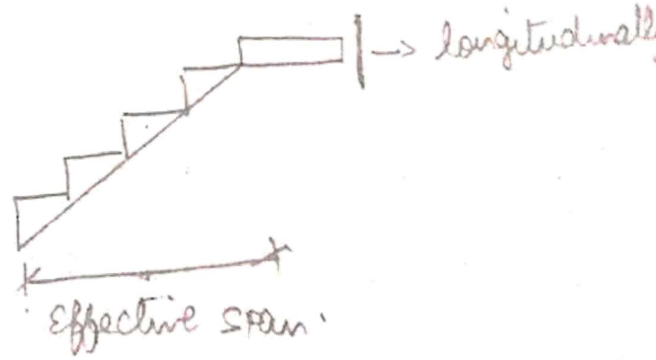
Effective span of stair:

Stair case may be divided into two categories depending upon the direction in which the stair slab span.

i) Stair slab spanning horizontally.
ii) stair slab spanning longitudinally.

Stair slab spanning horizontally:





↳ The slab is supported by side walls one side & the stringer beam on the other side.

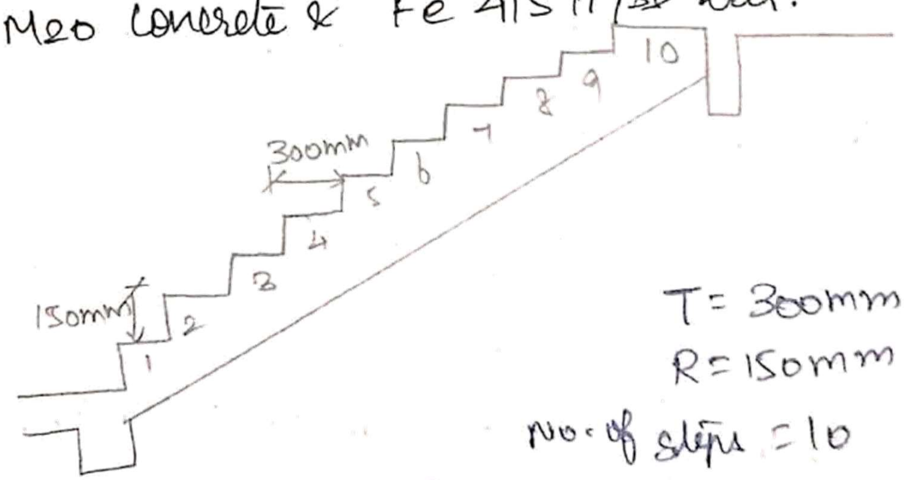
↳ The effective span l is the horizontal distance b/w the c/c of the supports.

Stair spanning longitudinally:

↳ The slab is supported @ the bottom & top of the flight & remain unsupported on the sides.

↳ The effective span of the stair without stringer beam shall be taken as per IS 456 – clause-33.1

1) Design one of the flight of a dog legged stair spanning b/w landing beams. using the following data. No. of steps in a flight 10. Tread = 300mm. Rise = 150mm. width of landing beam = 300mm. use M20 concrete & Fe 415 HYSD bar.



T = 300mm

R = 150mm

No. of steps = 10

$$\text{Effective span} = 10 \times 300 + \frac{300}{2} + \frac{300}{2}$$

$$= 3300\ mm$$

$$= 3.3\ m$$

Step 1: Thickness of waist slab:

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

$$= \frac{3300}{20}$$

$$= 165 mm.$$

$$D = 165 + 25$$

$$= 190 mm.$$

Step 2: Loads:

i) Dead load:

$$S.W \text{ of slab } (W_s) = b \times D \times \gamma$$

$$= 1 \times 0.19 \times 25$$

$$= 4.725.$$

$$W = \frac{W_s \sqrt{R^2 + T^2}}{T}$$

$$= \frac{4.75 \sqrt{150^2 + 300^2}}{300}$$

$$= 5.34 \, kN/m.$$

ii) S.W of step $= R/2 \times b \times \gamma$

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

$$= 1.875.$$

iii) S.W of finish = 0.5 to 0.75 kN/m.

Live load $= 5\,kN/m^2$
$= 5 \times 1$
$= 5\,kN/m.$

Total load $= 5.34 + 1.875 + 5 + 0.5$
$= 12.7\,kN/m.$

Factored load $= 1.5 \times 12.7$
$= 19.03\,kN/m.$

Step 3: Bending moment:

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

$$= \frac{19.03 \times 3.3^2}{8}$$

$$= 25.9\,kNm.$$

Step 4: Check for depth:

$$d = \sqrt{\frac{Mu}{0.138\,f_{ck}\,b}}$$

$$= \sqrt{\frac{25.9 \times 10^6}{0.138 \times 20 \times 1000}}$$

$= 96.8 < d$ provided

Hence it is adequate.

Step 5: Main reinforcement:

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

$$25.9 \times 10^6 = 0.87 \times 415 \times A_{st} \times 160\left[1 - \frac{A_{st} \times 415}{20 \times 1000 \times 160}\right]$$

$A_{st} = 477.7\,mm^2$

17
3.6m.
e
rade
ed on
ng

provide 10mm ϕ

$$\text{spacing} = \frac{a_{st}}{A_{st}} \times 1000$$

$$a_{st} = \pi/4 \times 10^2 = 78.5 \text{ mm}^2$$

$$\text{Spacing} = \frac{78.5}{477.7} \times 1000$$

$$= 164.4$$

$$\simeq 160 \text{ mm.}$$

Provide 10mm ϕ @ 160 mm c/c.

Step 6: Distribution reinforcement:

$$A_{st} = 0.12\% \, bD$$

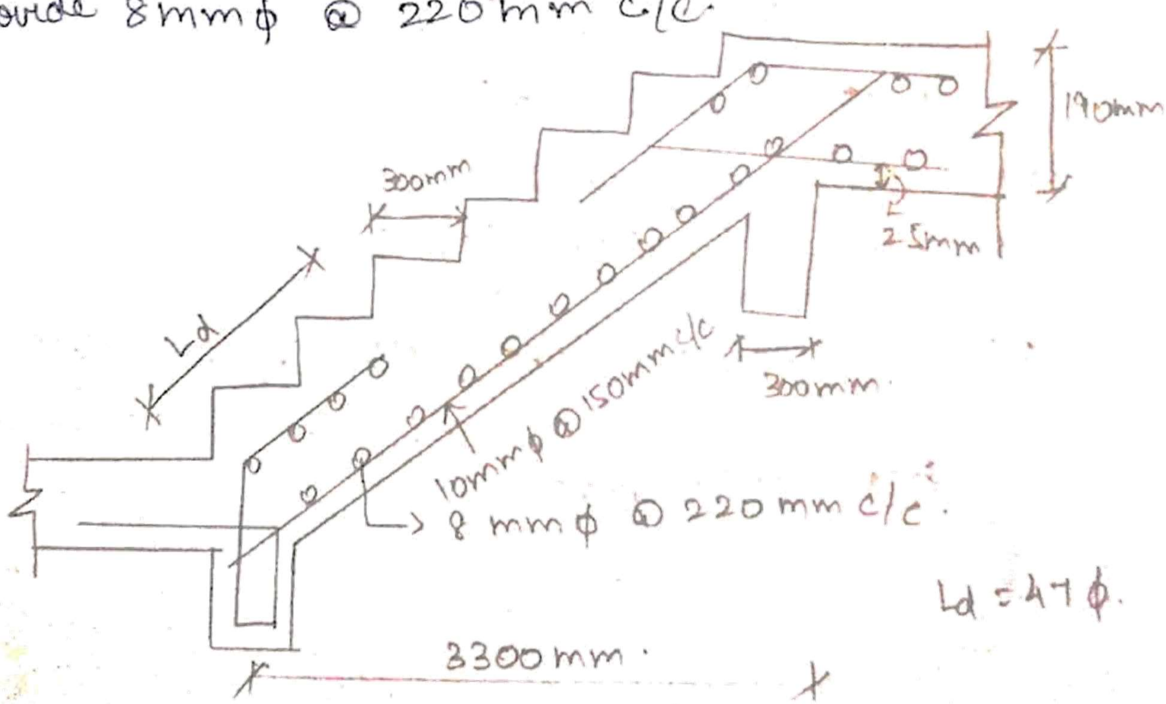
$$= \frac{0.12}{100} \times 1000 \times 190$$

$$= 228 \text{ mm}^2$$

provide 8mm ϕ

$$\text{spacing} = \frac{a_{st}}{A_{st}} \times 1000 = \frac{\pi/4 \times 8^2}{228} \times 1000 = 220 \text{mm}$$

Provide 8mm ϕ @ 220 mm c/c.



2) Design a dog legged stair for a building in which the vertical distance b/w the floor is 3.6m. The stair hall measures 2.4 m x 5m. The live load on the stair is $3KN/m^2$. Adopt M20 grade conc. & Fe 415 HYSD bar. The stair are supported on 230mm wall at an end of outer edge of landing slab.

Step 1: Dimension of stair:

Distance b/w floors = 3.6m.

Height of each flight = $\frac{3.6}{2}$ = 1.8m.

Tread = 250mm

Rise = 150mm

No. of rise = $\frac{1.8}{0.15}$ = 12 Nos.

No. of tread = 12-1

= 11 Nos.

Assume landing width = 1.25m

Effective span = $11 \times 0.25 + 1.25 + \frac{0.23}{2}$

= 4.115 m

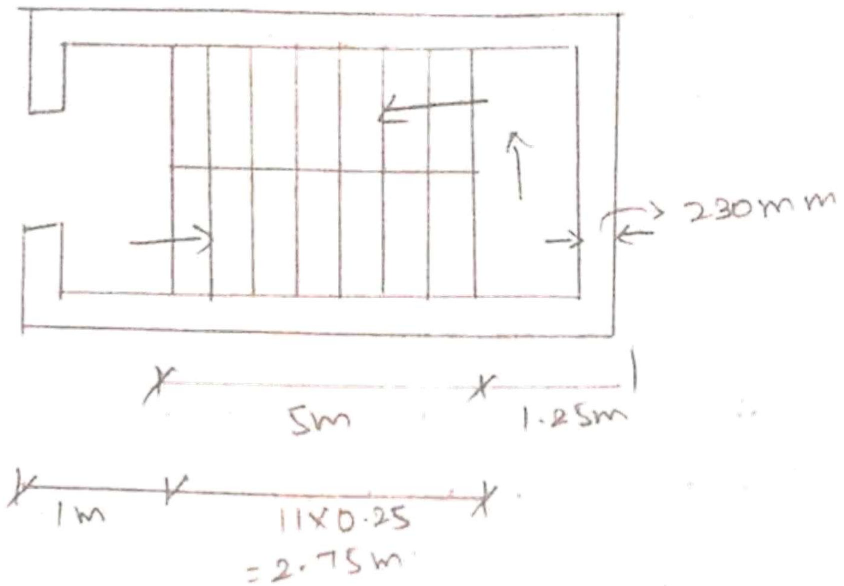
Thickness, d = $\frac{span}{20}$

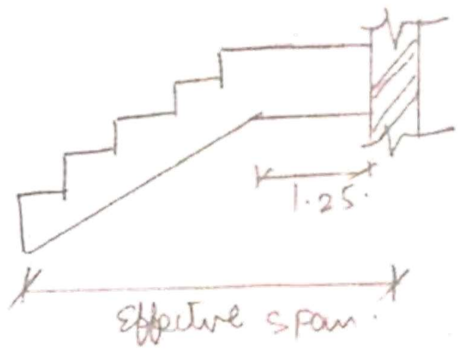
= $\frac{4115}{20}$

= 205.7mm.

$D = 205 + 25$

$= 230mm$





Step 2: Loads:

i) Dead load:

S.W of slab $(W_s) = b \times D \times \gamma$

$= 1 \times 0.23 \times 25$

$= 5.75\ KN/m.$

$$W = W_s \frac{\sqrt{R^2 + T^2}}{T}$$

$$= 5.75 \frac{\sqrt{150^2 + 250^2}}{250}$$

$= 6.70 KN/m.$

ii) S.w of step $= R/2 \times b \times \gamma$

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

$= 1.875$ KN/m.

iii) Finish load $= 0.5$ KN/m.

Live load $= 3$ KN/m^2

Total load $= 12.075$ KN/m.

Factored load $= 1.5 \times 12.075$

Step 3: $= 18.1125$ KN/m.

Bending Moment:

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

$$= \frac{18.11 \times 4.115^2}{8}$$

$= 38.32$ KNm.

Step 4: Check for depth:

$$d = \sqrt{\frac{M_u}{0.138 f_{ck}\, b}}$$

$$= \sqrt{\frac{38.32 \times 10^6}{0.32 \times 20 \times 1000}}$$

$= 117.8 <$ d provided.

Hence depth is adequate.

Step 5: Main reinforcement:

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

$$38.32 \times 10^6 = 0.87 \times 415 \times A_{st} \times 205 \left[1 - \frac{A_{st} \times 415}{20 \times 1000 \times 205}\right]$$

$$= 548\ mm^2$$

$$A_{st} \simeq 540\ mm^2$$

provide 10 mm ϕ

$$\text{spacing} = \frac{\pi/4 \times 10^2}{548} \times 1000$$

$$= 143.32$$

$$\simeq 140\ mm.$$

provide 10 mm ϕ @ 140 mm c/c.

Step 6: Distribution Reinforcement:

$$A_{st} = 0.12\ \%\ bD$$

$$= \frac{0.12}{100} \times 1000 \times 230$$

$$= 276\ mm^2$$

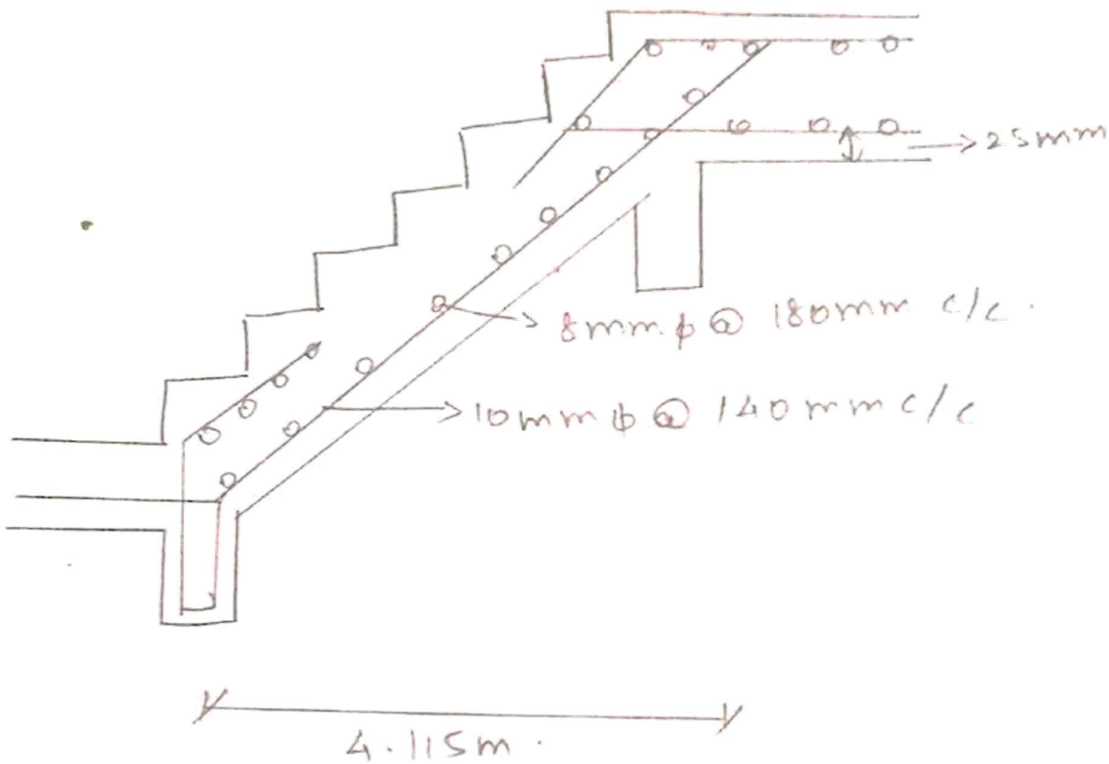
provide 8 mm ϕ

$$\text{spacing} = \frac{\pi/4 \times 8^2}{276} \times 1000$$

$$= 182.12$$

$$\simeq 180\ mm$$

provide 8 mm ϕ @ 180 mm c/c.


25mm
8mm ϕ @ 180mm c/c
10mm ϕ @ 140mm c/c
4.115m

UNIT- IV
DESIGN OF COLUMNS

columns:

↳ Members in compression are called column and struts.

↳ column transmits load coming from beam(or) slab and distributes to the foundation.

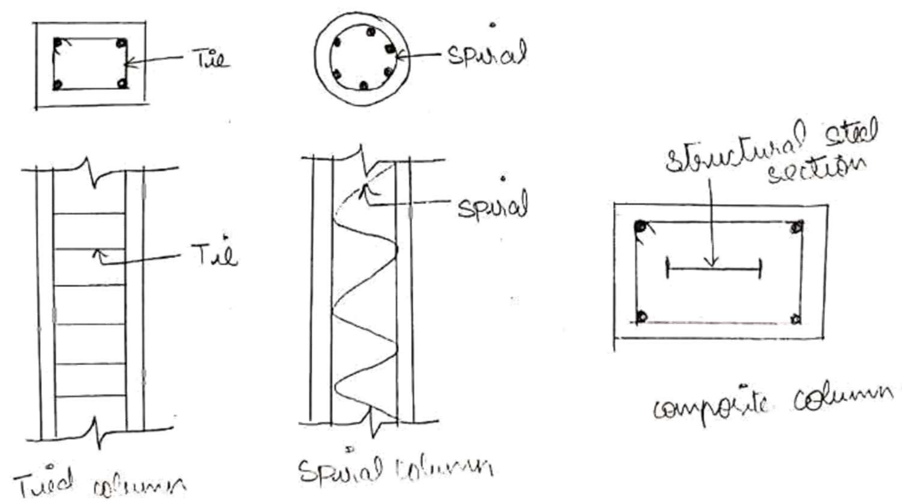
Types of column:

Based on shape:

* Square column
* circular column
* Rectangular column
* Polygonal column

Based on Type of Reinforcement:

* Tied column
* Spiral column
* composite column.



Based on type of loading :

* column with axial loading
* column with uni axial bending
* Loading with biaxial bending.

column with axial bending:

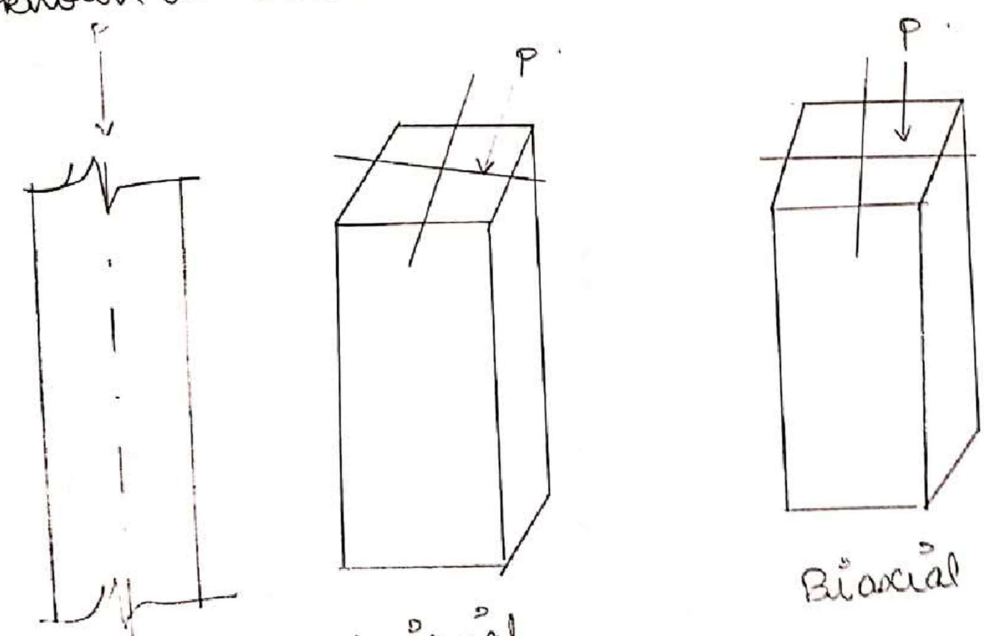
Load acting exactly at the centroid of the column is called axially loaded column.

Loading with uniaxial bending:

Axial load and bending moment along one direction are applied simultaneously on the column is known as uniaxial eccentrically loaded column.

Loading with biaxial eccentricities or bending:

Axial load and bending moment along two directions are applied simultaneously on the column is known as biaxial eccentrically loaded column.



Axial loading   uniaxial bending   Biaxial bending.

Based on slenderness ratio:

i) Short column $(l_e/D) < 12$

ii) Long column (or) $(l_e/D) > 12$

Slender column.

Design of axially loaded short columns:

1. A rectangular reinforced concrete column of cross sectional dimension 300mm by 600mm is to be designed to support an ultimate axial load of 2000 KN. Design suitable reinforcements in the column using M20 grade concrete & Fe 415 HYSD bar.

step 1): Given:

$P_u = 2000\ KN$ $\quad f_{ck} = 20\ N/mm^2$

$b = 300mm$ $\quad f_y = 415\ N/mm^2$

$D = 600\ mm$

step 2: Longitudinal reinforcement:

$P_u = 0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}$

$(2000 \times 10^3) = 0.4 \times 20 \times 300 \times 600 + [(0.6 \times 415) - (0.4 \times 20)] \times A_{sc}$

$A_{sc} = 2073\ mm^2$.

Provide 22mm ϕ bar.

$a_{sc} = \pi/4 \times d^2$

$= \pi/4 \times 22^2$

$= 380.28\ mm^2$

No. of bar $= \dfrac{A_{st}}{a_{st}}$

$= \dfrac{2073}{380}$

$\cong 6$

Provide 6 bars of 22 mmϕ bar.

Lateral ties:

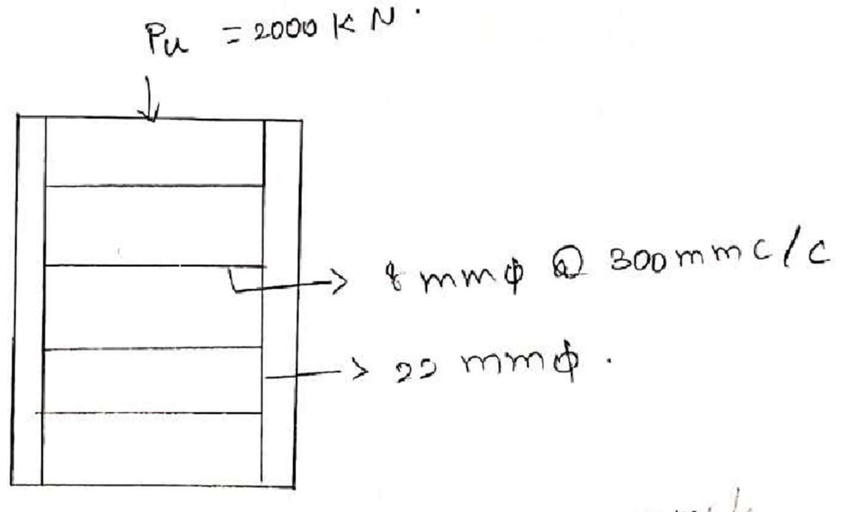
Tie diameter: $\phi_t \not< \begin{cases} (1/4) \times \phi = \frac{1}{4} \times 22 = 5.5\text{mm} \\ 6\text{ mm} \end{cases}$

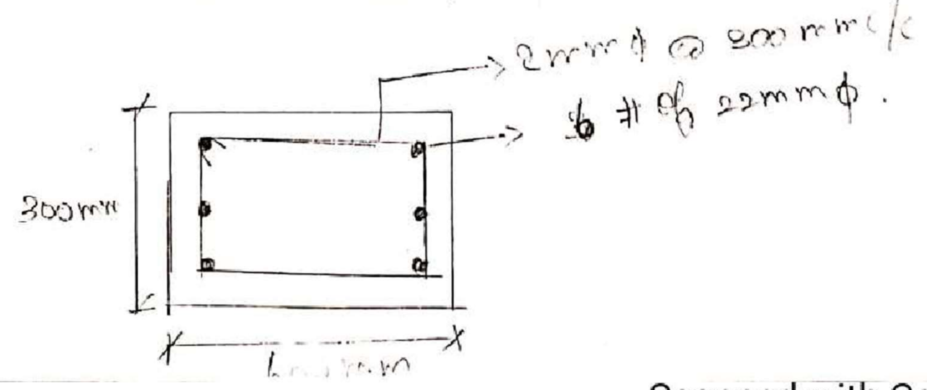
Provide 8mm ties.

Tie spacing $s_t \not> \begin{cases} 300\text{ mm} \\ (16 \times \phi) = 16 \times 22 = 352\text{ mm} \\ 48 \times 8 = 384\text{ mm} \end{cases}$

Provide 300mm.

Provide 8 mm ϕ ties @ 300 mm centres.





squre column:

2. Design an axially loaded tied column 400mm x 400 mm pinned at both ends with an unsupported length of 3 m to carry a factored load of 2300 KN. use M20 grade concrete and Fe 415 grade steel.

Step 1: Data:

$l_e = 3m$ $\quad f_{ck} = 20 N/mm^2$

$P_u = 2300$ KN. $\quad f_y = 415 N/mm^2$

$b = 400$ mm

$D = 400$ mm

Step 2: Slenderness ratio:

$$\left(\frac{L_e}{D}\right) = \left(\frac{3000}{400}\right)$$

$$= 7.5 < 12$$

Hence the column is designed as short column.

Step 3: Minimum Eccentricity:

$$e_{min} = \frac{L}{500} + \frac{D}{30}$$

$$= \frac{3000}{500} + \frac{400}{30}$$

$$= 19.33 < 20 \text{ mm}.$$

Minimum eccentricity less than (or) equal to 0.05D.

$$0.05D = 0.05 \times 400$$

$$= 20 \text{mm}.$$

Hence the eccentricity condition is satisfied.

Step 4: Main reinforcement :

$P_u = 0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}$

$2300 \times 10^3 = 0.4 \times 200 \times 400 \times 400 + (0.67 \times 415 - 0.4 \times 20) A_{sc}$

$A_{sc} = 3777 mm^2$

Provide 25mmϕ bar.

$a_{st} = \pi/4 \times 25^2$

$= 490 mm^2$

No. of bar $= \frac{A_{sc}}{a_{sc}}$

$= \frac{3777}{490}$

$= 7.6$

$\simeq 8$

Provide 8 # of 25mmϕ bar.

Step 5: Lateral ties :

Tie diameter : $\phi_t \nless \begin{cases} 1/4 \times \phi = 1/4 \times 25 = 6mm \\ 6mm \end{cases}$

Provide 8mmϕ

Tie spacing : $S_t \nless \begin{cases} 300 mm \\ (16 \times \phi) = 16 \times 25 = 400 mm \\ 48 \times 8 = 384 mm \end{cases}$

Provide spacing of 300 mm.

Provide 8 mmϕ @ 300mm c/c spacing.

3. Design the reinforcement in a column of size 450mm x 600mm, Subjected to an axial load of 2000 KN under service load & live loads. The column has an unsupported length of 3m and is braced against sideway in both direction. use M20 concrete and Fe415 steel.

Step 1: Data:

$L_e = 3m$ (or) $3000mm$. $P_u = 1.5 \times 2000$
$= 3000 KN$.

$b = 450mm$.

$D = 600 mm$.

Step 2: Slenderness ratio:

$$\frac{L_e}{D} = \frac{3000}{600}$$
$$= 5 < 12.$$

Hence, designed as a short column.

Step 3: Minimum eccentricity:

$$e_{min} = \frac{L_e}{500} + \frac{D}{30}$$
$$= \frac{3000}{500} + \frac{600}{30}$$
$$= 26 mm > 20mm$$

$$0.05D = 0.05 \times 600$$
$$= 30mm.$$

$$0.05D = 0.05 \times 450$$
$$= 21mm.$$

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step 3: Longitudinal reinforcement:

$P_u = 0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}$

$3000 \times 10^3 = 0.4 \times 20 \times 450 \times 600 + (0.67 \times 415 - 0.4 \times 20) A_{sc}$

$A_{sc} = 3111 \text{ mm}^2$

Provide 25 mm φ

$a_{st} = \frac{\pi}{4} \times 25^2 = 490.87$

No. of bar $= \frac{3111}{490.87} = 6.3 \simeq 8$

Provide 8# of 25 mm φ bar.

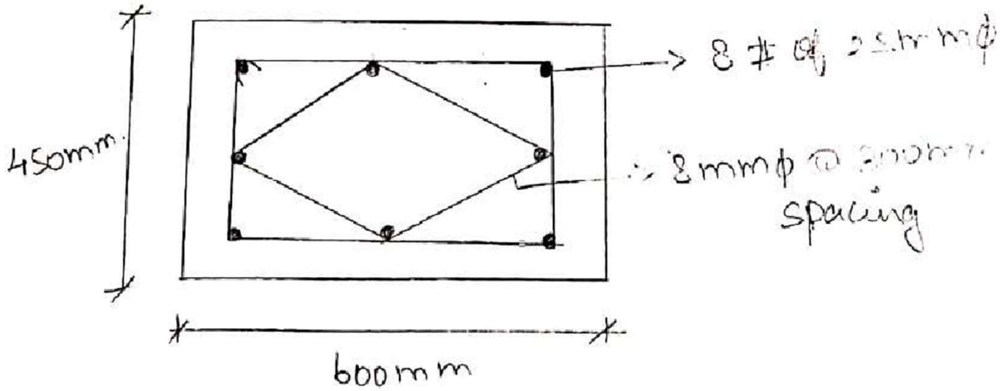
step 4: Lateral ties:

Tie diameter : $\phi_t \nless \begin{cases} \frac{1}{4} \times \phi = \frac{1}{4} \times 25 = 6.25\text{mm} \\ 6\text{mm} \end{cases}$

provide 8 mm φ.

Tie spacing : $S_t \ngtr \begin{cases} 300\text{mm.} \\ 16 \times 25 = 400\text{mm.} \\ 48 \times 8 = 384\text{mm.} \end{cases}$

provide 8mm φ @ 384 mm c/c spacing.



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4\. circular column :

Design the reinforcements in a circular column of diameter 300 mm to support a service axial load of 800 KN. The column has an unsupported length of 3m & braced against side away. The column is reinforced with helical ties. Adopt M20 grade concrete & Fe 415 HYSD bar.

Step 1: Data :

$L_e = 3m$ $\qquad f_{ck} = 20 N/mm^2$

$D = 300 mm.$ $\qquad f_y = 415 N/mm^2.$

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

Step 2: Slenderness ratio :

$e_{min}\ [L_e/D] = \frac{3000}{300} = 10.$

Hence design the Short column.

Step 3: Minimum eccentricity :

$$e_{min} = \left[\frac{L}{500} + \frac{D}{30}\right]$$

$$= \left[\frac{3000}{500} + \frac{300}{30}\right]$$

$$= 15mm < 20mm.$$

$$0.05D = 0.05 \times 300$$

$$= 15mm < 20mm.$$

Step 4: Main reinforcement :

$$P_u = 1.05\left[0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}\right]$$

$$1200 \times 10^3 = 1.05\left[\left[0.4 \times 20 \times \pi/4 \times 300^2\right] + (0.67 \times 415 - 0.4 \times 20) A_{sc}\right]$$

$$A_{sc} = 2139 mm^2.$$

Provide 22 mm ϕ bar.

$a_{sc} = \pi/4 \times 22^2$

$= 380 \text{ mm}^2$

No. of bar $= \frac{A_{sc}}{a_{sc}}$

$= \frac{2139}{380}$

$= 5.6$

$\simeq 6$

Provide 6 # of 22 mm ϕ bar.

Step 5: Helical reinforcement:

$d' = 50 \text{ mm}$.

core diameter $= [D - 2d']$

$= [300 - 2 \times 50]$

$= 200 \text{ mm}$.

Area of core, $A_c = [(\pi/4 \times 200^2) - (6 \times \pi/4 \times 22^2)]$

$= 29135 \text{ mm}^2$

Gross area of section, $A_g = \pi/4 \times 300^2$

$= 70685 \text{ mm}^2$

Volume of core, $V_c = 29135 \times 10^3 \text{ mm}^3$.

use 8 mm ϕ helical spiral at a pitch 'P' mm,

$V_{us} = [\pi(300 - 100 - 8)\, 50 \times 1000/P]$

$= 30159.288 \times 10^3 / P \text{ mm}^3/\text{m}$.

6

According to clause 39.4.1 of IS456.

$$(V_{us}/V_c) < 0.36 \left[A_g / A_c - 1 \right] (f_{ck} / f_y).$$

$$\left[\frac{30159 \times 10^3}{29135 \times 10^3 \, p} \right] < 0.36 \left[(70685 - 29135) - 1 \right] (20/415)$$

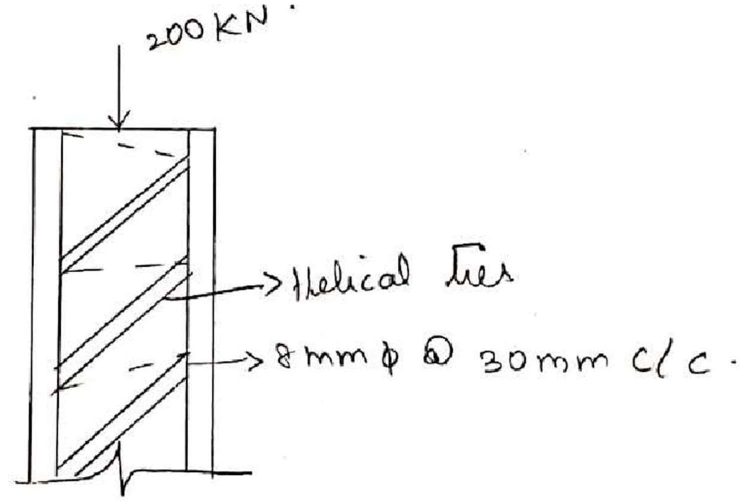
$P = 42mm.$

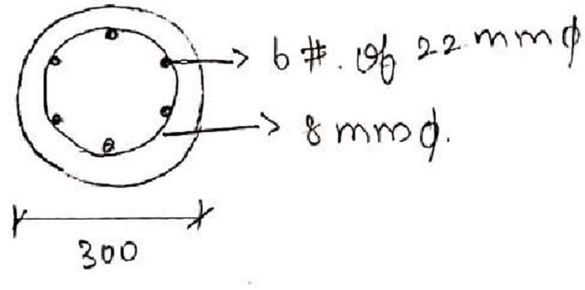
According to clause 26.5.3.2 of IS:456.

$$P < \begin{cases} 75 \, mm \\ \frac{\text{core } \phi}{6} = \frac{200}{6} = 33.6mm \end{cases}$$

$$P > \begin{cases} 25mm \\ 3 \times \phi = 3 \times 8 = 24mm. \end{cases}$$

Hence provide 8mmϕ helical spiral at a pitch of 30mm.





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Design of slender columns:

Design the reinforcements in the slender column which is restrained against sway using the following data:

Size of column = 400mm x 400mm.

concrete grade = M30, $f_{ck} = 30 N/mm^2$

$f_y = 415 N/mm^2$

Effective length of column, $L_{ex} = L_{ey} = 6m$.

unsupported length = $L = 7m$.

Factored load = $P_u = 1500 KN$.

Factored moment in mutually perpendicular direction

$M_{ux}, M_{uy} = 40$ KNm at top & 20 KNm at bottom.

Slenderness ratio:

$$(L_e/D) = (6000/400)$$

$$= 15 > 12$$

Hence design the slender column.

From table 1 - SP 16.

$$(e_x/D) = (e_y/b) = 0.113$$

Step 2: Additional moments:

$$M_{ax} = M_{ay} = (P_u e_x) = (P_u e_y) = (1500 \times 0.113) 0.4$$

$$= 67.8 KNm.$$

According to clause 39.4.1 of IS456.

$(V_{u\phi}/V_c) < 0.36\,[Ag/Ac - 1]\,(f_{ck}/f_y)$.

$$\left[\frac{20159\times10^3}{29135\times10^3\,p}\right] < 0.36\,[\,(70685-29125)-1]\,(^{20}/_{415})$$

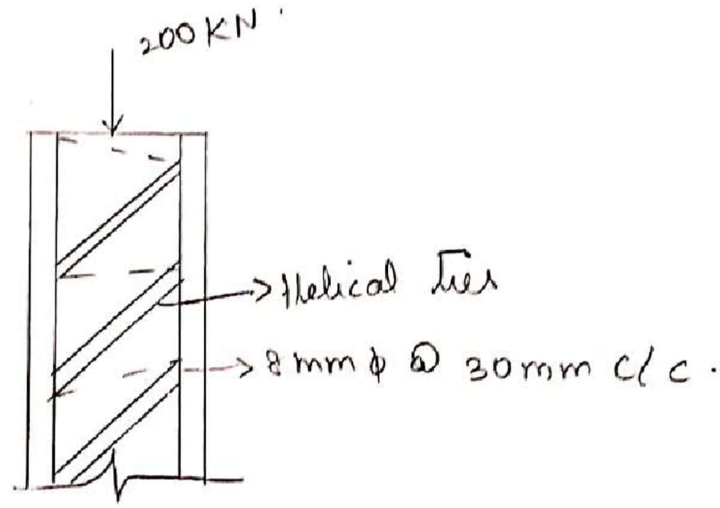
P = 42mm.

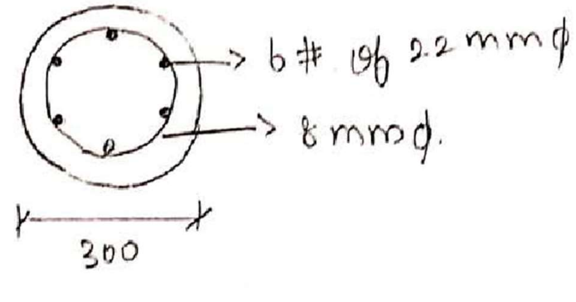
According to clause 26.5.3.2 of IS:456.

$$P < \begin{cases} 75\,mm \\ \frac{core\ \phi}{6} = \frac{200}{6} = 33.6mm \end{cases}$$

$$P > \begin{cases} 25\,mm \\ 3\times\phi = 3\times8 = 24mm. \end{cases}$$

Hence provide 8mm ϕ helical spiral at a pitch of 30mm.





step 3: Reinforcement :

Assume, $p = 3$.

$A_g = 400 \times 400$

$= 16 \times 10^4 \ mm^2$.

From chart - 63 of SP - 16.

$$\left[\frac{P_{uz}}{A_g}\right] = 22.5 \ N/mm^2$$

$$P_{uz} = \frac{(22.5 \times 400 \times 400)}{1000}$$

$= 3600 \ KN$.

step 4: Computation of P_b:

$(d'/D) = 40/400 = 0.1$

From table 60 of SP 16.

$K_1 = 0.207$, $K_2 = 0.328$.

$$P_{bx} = P_{by} = \left[K_1 + K_2 \frac{p}{f_{ck}}\right] f_{ck} bD$$

$$= \left[\left(0.27 + 0.328 \times \frac{3}{30}\right)\right] \times 30 \times 400 \times 400$$

$= 1151 \ KN$.

Step 5: Computation of Reduction factors:

$$K_x = K_y = \left[\frac{P_{uz} - P_u}{P_{uz} - P_{bx}}\right]$$

$$= \left[\frac{3600 - 1500}{3600 - 1151}\right]$$

$= 0.85$.

Hence modified additional moments are:

$M_{ux} = M_{uy} = (67.8 \times 0.85)$

$= 57.63 KNm$

The additional moments due to slenderness effects should be added to initial moments

$M_{ux} = M_{uy} = [(0.6 \times 40) - (0.4 \times 20)]$

$= 16 KNm.$

$e_x = e_y = \left[\frac{L}{500} + \frac{D}{30}\right]$

$= \left[\frac{7000}{500} + \frac{400}{30}\right]$

$= 27.3mm. > 20mm.$

So, moments due to eccentricity are computed as

$M_{ux} = M_{uy} = \left[1500 \times \frac{27.3}{1000}\right]$

$= 41 KNm.$

Total design moments are

$M_{ux} = M_y = [41 + 57.63]$

$= 98.63 KNm.$

Step 6: check for biaxial bending:

$\left[\frac{P_u}{f_{ck}\, b\, D}\right] = \left[\frac{1500 \times 10^3}{30 \times 400 \times 400}\right] = 0.3125.$

$(P/f_{ck}) = (3/30)$

$= 0.10.$

∴ Chart 44 of SP16.

$$\left[\frac{M_u}{f_{ck}\, bD^2}\right] = 0.14.$$

$$M_{ux_1} = M_{uy_1} = (0.14 \times 30 \times 400 \times 400^2)$$

$$= 268.8 \text{ KNm}.$$

$$(P_u/P_{uz}) = \left(\frac{1500}{3600}\right)$$

$$= 0.416.$$

From $\alpha_n = 1.35$.

$$\left[\frac{M_{ux}}{M_{ux_1}}\right]^{\alpha_n} + \left[\frac{M_{uy}}{M_{uy_1}}\right]^{\alpha_1} \leq 1.0.$$

$$\left[\frac{98.63}{268.8}\right]^{1.35} + \left[\frac{98.63}{268.8}\right]^{1.35} = 0.50 < 1.0.$$

Hence the assumed percentage of 3% is satisfactory

Design for axially loaded uniaxial bending column:

Design the reinforcements in a rectangular column of size 300 mm by 500 mm to support a design ultimate load of 500 KN together with a factored moment of 200 KNm. Adopt the value of $f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$.

Step 1: Data:

$b = 300 mm$ $\quad$ $f_{ck} = 20 N/mm^2$

$D = 500 mm$ $\quad$ $f_y = 415 N/mm^2$

$P_u = 500 KN$ $\quad$ $(d'/D) = 0.1$

$M_u = 200 KNm$.

step 2: Non dimensional parameters:

$$\left[\frac{P_u}{f_{ck}\, b\, d}\right] = \left[\frac{500 \times 10^3}{20 \times 300 \times 500}\right]$$

$$= 0.166.$$

$$\left[\frac{M_u}{f_{ck}\, b\, D^2}\right] = \left[\frac{200 \times 10^6}{20 \times 300 \times 500^2}\right]$$

$$= 0.132.$$

step 3: Longitudinal reinforcement:

From chart 32, SP:16.

$(P/f_{ck}) = 0.06$.

$P = 0.06 \times 20 = 1.2$.

$$A_{sc} = \left[\frac{PbD}{100}\right] = \left[\frac{1.2 \times 300 \times 500}{100}\right] = 1800 mm^2.$$

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$\therefore$ Chart 44 of SP16.

$$\left[\frac{M_u}{f_{ck}\, b D^2}\right] = 0.14.$$

$$M_{ux_1} = M_{uy_1} = (0.14 \times 30 \times 400 \times 400^2)$$

$$= 268.8 \text{ KNm}.$$

$$(P_u/P_{uz}) = \left(\frac{1500}{3600}\right)$$

$$= 0.416.$$

From $\alpha_n = 1.35$.

$$\left[\frac{M_{ux}}{M_{ux_1}}\right]^{\alpha_n} + \left[\frac{M_{uy}}{M_{uy_1}}\right]^{\alpha_1} \leq 1.0.$$

$$\left[\frac{98.63}{268.8}\right]^{1.35} + \left[\frac{98.63}{268.8}\right]^{1.35} = 0.50 < 1.0.$$

Hence the assumed percentage of 3% is satisfactory

$a_{st} = \pi/4 \times 25^2$

$= 490.8\ mm^2$

$\therefore$ No. of bar $= \frac{A_{sc}}{a_{sc}}$

$= 3.6$

$\simeq 4.$

Provide 4 # of 25mm bar.

Step4: Lateral ties:

Tie diameter, $\phi_t \nless \begin{cases} \frac{1}{4} \times \phi = 1/4 \times 25 = 6.25mm \\ 6mm \end{cases}$

provide 8mm ϕ ties

Tie spacing $s_t \ngtr \begin{cases} 16 \times \phi = 16 \times 25 = 400mm. \\ 48 \times 8 = 384mm. \\ b = 300mm \end{cases}$

Provide 8mm ϕ @ 300 mm Centres.

Design a reinforced concrete column, 400 mm square to carry an ultimate load of 1000 KN at an eccentricity of 160mm. Use M20 grade concrete & Fe 250 grade steel.

Step 1: Data :

$b = 400mm$ $\quad$ $f_{ck} = 20 N/mm^2$

$D = 400 mm$. $\quad$ $f_y = 250 N/mm^2$

$e = 160 mm$.

$P_u = 1000 KN$. $\quad$ $(d'/D) = (40/400) = 0.1$.

Step 2: Non dimensional parameters:

$$\left[\frac{P_u}{f_{ck} b d}\right] = \left[\frac{1000\times10^3}{20\times400\times400}\right]$$

$$= 0.3125.$$

$M_u = P_u \cdot e$
$= 1000\times10^3\times160.$

$$\left[\frac{M_u}{f_{ck} b D^2}\right] = \left[\frac{1000\times10^3\times160}{20\times300\times500^2}\right]$$

$$= 0.125.$$

Step 3: Longitudinal reinforcement :

From chart 28, SP:16.

$(P/f_{ck}) = \cancel{0.06}\ 0.105.$

$P = 0.105\times20$

$= 2.1$

$$A_{sc} = \left[\frac{PbD}{100}\right]$$

$$= \left[\frac{2.1\times400\times400}{100}\right]$$

$A_{sc} = 3360 mm^2.$

$a_{st} = \pi/4 \times 25^2$

$= 490.8\ mm^2$

No. of bar $= \dfrac{A_{sc}}{a_{sc}}$

$= 3.6$

$\simeq 4.$

Provide 4 # of 25mm bar.

Step4: Lateral ties:

Tie diameter, $\phi_t \not< \begin{cases} \frac{1}{4} \times \phi = \frac{1}{4} \times 25 = 6.25mm \\ 6mm. \end{cases}$

provide 8mm ϕ ties

Tie spacing $s_t \not> \begin{cases} 16 \times \phi = 16 \times 25 = 400mm. \\ 48 \times 8 = 384mm. \\ b = 300mm \end{cases}$

Provide 8mm ϕ @ 300mm centres.

$a_{sc} = \pi/4 \times 25^2$

$= 490.8 mm^2.$

No. of bar $= \frac{A_{sc}}{a_{sc}}$

$= \frac{3360}{490.8}$

$\simeq 8.$

Provide 8 # of 25mmϕ bar.

Step 4: Lateral ties:

Tie diameter, $\phi_t \nless \begin{cases} 1/4 \times \phi = 1/4 \times 25 = 6.25mm. \\ 6mm. \end{cases}$

provide 8mmd.

Tie spacing $S_t \ngtr \begin{cases} 16\times\phi = 16\times 25 = 400mm. \\ 48\times 8 = 384mm. \\ 300 mm. \end{cases}$

Hence provide 8mmϕ @ 300mm c/c spacing.

Design a reinforcements in a circular column of diameter 400mm to support a factored load of 800 KN together with a factored moment of 80 KNm. Adopt M_{20} grade concrete & Fe 415 HYSD bars.

Step 1: Data:

$D = 400mm$ $\quad f_{ck} = 20 N/mm^2$

$P_u = 800 KN$ $\quad f_y = 415 N/mm^2$

$M_u = 80 KNm$ $\quad (d'/D) = 0.10$.

$d' = 40mm$.

Step 2: Non dimensional parameters:

$$\left[\frac{P_u}{f_{ck} b D}\right] = \left[\frac{800 \times 10^3}{20 \times 400 \times 400}\right]$$

$$= 0.25$$

$$\left[\frac{M_u}{f_{ck} D^3}\right] = \left[\frac{80 \times 10^6}{20 \times 400^3}\right]$$

$$= 0.0625.$$

Step 3: Longitudinal reinforcement:

From Chart 56, SP:16

$$(P/f_{ck}) = 0.06$$

$$P = 0.06 \times 20$$

$$= 1.2.$$

$$A_{st} = \frac{P \pi D^2}{4 \times 100}$$

$$= \frac{\cancel{P}\, 1.2 \times \pi \times 400^2}{400}$$

$$= 1508\, mm^2$$

$a_{sc} = \pi/4 \times 20^2$

$= 314 mm^2$

No. of bar $= \frac{A_{sc}}{a_{sc}}$

$= \frac{1508}{314}$

$= 4.8$

$\simeq 6$

Provide 6 # of 8mm ϕ bar.

Step 4: Lateral ties:

Tie diameter, $\nless \begin{cases} 1/4 \times \phi = 1/4 \times 20 = 5mm \\ 6mm \end{cases}$

provide 6mm ϕ ties.

Tie spacing $\ngtr \begin{cases} 400 mm \\ 16 \times \phi = 16 \times 20 = 320mm \\ 48 \times \phi = 48 \times 6 = 288 mm. \end{cases}$

Provide 6mm ϕ @ 288 mm centers.

Design of axially loaded biaxial bending column:

Design the reinforcement in a short column 400x400mm at the corner of a multistoreyed building to support an axial factored load of 1500 KN, together with biaxial moments of 50KNm acting in perpendicular planes. Adopt M20 grade concrete & Fe 415 HYSD bars.

Step 1: Data :

$b = 400mm.$

$D = 400mm.$

$P_u = 1500 KN.$

$M_{ux} = M_{uy} = 50 KN.$

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

$f_y = 415 N/mm^2.$

$(d'/D) = (40/400) = 0.10.$

Step 2: equivalent Moment :

$$M_u = 1.15\sqrt{M_{ux}^2 + M_{uy}^2}$$

$$= 1.15\sqrt{50^2 + 50^2}$$

$$= 81.3 KNm.$$

Step 3: Non dimensional parameters :

$$\left[\frac{P_u}{f_{ck} b D}\right] = \left[\frac{1500 \times 10^3}{20 \times 400 \times 400}\right]$$

$$= 0.468.$$

$$\left[\frac{M_u}{f_{ck} b D^2}\right] = \left[\frac{81.3 \times 10^6}{20 \times 400 \times 400^2}\right]$$

$$= 0.063.$$

$a_{sc} = \pi/4 \times 20^2$

$= 314 mm^2$

$\text{No. of bar} = \frac{A_{sc}}{a_{sc}}$

$= \frac{1508}{314}$

$= 4.8$

$\simeq 6$

Provide 6 # of 8mm ϕ bar.

Step 4: Lateral ties:

Tie diameter, $\nless \begin{cases} 1/4 \times \phi = 1/4 \times 20 = 5mm \\ 6mm \end{cases}$

provide 6mm ϕ ties.

Tie spacing $\ngtr \begin{cases} 400 mm \\ 16 \times \phi = 16 \times 20 = 320mm \\ 48 \times \phi = 48 \times 6 = 288mm. \end{cases}$

provide 6mm ϕ @ 288 mm centers.

step 4 : ~~non~~ Longitudinal reinforcement:

From chart 44, SP:16.

$(P/f_{ck}) = 0.06$

$P = 0.06 \times 20$

$= 1.2$

$A_{sc} = \left[\frac{PbD}{100}\right]$

$= \left[\frac{1.2 \times 400 \times 400}{100}\right]$

$= 1920$

$a_{sc} = \frac{\pi}{4} \times 20^2$

$= 314 mm^2$

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

$= \frac{1920}{314}$

$\simeq 6$

Provide 6 # of 20mm ϕ bar.

$\left[\frac{M_{ux1}}{f_{ck} b D^2}\right] = 0.468$

$M_{ux1} = 0.468 \times 20 \times 400 \times 400^2$

$= 87 kNm$

$M_{ux1} = M_{uy1} = 87 kNm$

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$P_{uz} = [0.4 f_{ck} A_c + 0.75 f_y A_s]$

$= [(0.4 \times 20)(400 \times 400) + 2060] + (0.75 \times 415 \times 2060]$

$= 2062 \text{ kN}.$

$\left(\frac{P_u}{P_{uz}}\right) = \left(\frac{1500}{2062}\right)$

$= 0.72.$

$\alpha_n = 1.8.$

Step 5: Check for safety under biaxial bending:

$$\left[\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha n}\right] \leq 1$$

$$\left[\left(\frac{50}{87}\right)^{1.8} + \left(\frac{50}{87}\right)^{1.8}\right] = 0.736 < 1.$$

Hence section is safe under biaxial bending.

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UNIT- V
DESIGN OF FOOTINGS

Footings :

Footings are the structural members which spread and distribute the load carried from the load carried from the superstructure to the Soil below the ground over a large area.

They came under the category shallow foundations and are employed at places, where the soil with good bearing capacity is available within a small depth below the ground surface.

Types of footing:

1. Isolated footing
   - ↳ Square footing
   - ↳ Rectangular footing
   - ↳ Circular footing
2. combined footing
3. Strap footing
4. wall footing
5. Mat (or) Raft footing.

Forces to be considered while designing the footing.

- ↳ Earth pressure
- ↳ Seismic forces & wind forces
- ↳ Dead load
- ↳ Imposed load
- ↳ Thermal force.
- ↳ Buoyancy force in case of submerged foundations.

concept of proportioning footing and foundation based on soil properties:

Proportioning of footings:

Proportioning of footing refer to the distribution of footings in such a way that equal pressure is developed below each footings.

Objectives of proportioning footing:

↳ proportioning of footing decreases the differential settlement due to live load differences for footing on fine grained soils.

↳ All the footings are proportioned in such a manner that equal amount of pressure is developed below the footings & differential settlements gets reduced.

↳ A proportioned footing equalizes the average bearing pressure below the footing.

Procedure for proportioning of footings:

1. calculate the load from each column along with self weight of the footing.

2. Evaluate the max. live load subjected to each footings.

3. For each footings determine the ratio of maximum live load & dead load.

4. For The footing which has the highest live load to dead load ratio is considered as the governing footing.

5. Calculate the area of governing footing from the relation.

$$A_g = \frac{\text{Dead load + Live load}}{\text{Allowable bearing capacity}}$$

6. Determine the service load of all the footings.

7. The design bearing capacity (q_d) of all the footings excluding the governing footing is calculated as

$$q_d = \frac{\text{Service load for governing footing}}{\text{Area of governing footing}}.$$

8. Calculate the area of other footings from the equation

$$\text{Area of footing} = \frac{\text{service load for that footing}}{\text{Design bearing capacity}}.$$

Design of Masonry wall footing:

A 230mm thick masonry wall is to be provided with a RC footing on a site having soil with SBC, unit weight & angle of repose of $125 KN/m^2$, $17.5 KN/m^2$ & $30°$ respectively. use M20 grade of concrete & HYSD bars of grade Fe 415. Design the footing when the wall supports at service state, a load of 150 KN/m length.

Given:

Wall thickness = 230mm.

SBC, $q = 125 KN/m^2$

$\gamma_s = 17.5 KN/m^3$

$\phi = 30°$

Characteristic load = 150 KN/m.

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

$f_y = 415 N/mm^2$.

Step 1: Depth of foundation:

$$D_f = \frac{q}{\gamma_s}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2$$

$$= \frac{125}{17.5}\left[\frac{1-\sin 30}{1+\sin 30}\right]^2$$

$= 0.794m.$

Let the depth of footing be 1m.

Step 2: Footing width (B):

load = 10 % of P

$= \frac{10}{100} \times 150$

$= 15 kN/m$

For unit width of footing,

$L = \frac{\text{Total load}}{SBC}$

$= \frac{150+15}{125}$

$= 1.320 m$

Let us provide a footing length of 1.5m

Step 3: Bending Moment (BM):

Net upward pressure

$q = \frac{\text{Load without self weight of footing}}{\text{Area of footing}}$

$= \frac{150}{1 \times 1.5}$

$= 100 kN < SBC$

Hence, OK.

Max. BM occurs @ b/4 from the centre of wall

$Mu = 1.5 \times q/8 \left[(B-b)\left(B - b/4\right) \right]$

$= 1.5 \times 100/8 \left[(1.5-0.23)\left(1.5-\frac{0.23}{4}\right) \right]$

$Mu = 30.242 kNm.$

Step 4 : Thickness of footing :

$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$$

$$= \sqrt{\frac{30.242 \times 10^6}{0.138 \times 20 \times 1000}}$$

$$= 104.677 \text{mm.}$$

$$D = 104 + 50$$

$$= 154 \text{ mm}$$

$$\simeq 200 \text{ mm.}$$

∴ Let us provide an overall depth of 400 mm (i.e). (2 × 200 mm) to be safe against punching shear failure,

$$d_{provided} = 400 - 50 - \frac{16}{2}$$

$$= 342 \text{mm.}$$

Table 2,

$$P_t = \frac{0.128 + 0.43}{2}$$

$$= 0.136 \%.$$

Area of reinforcement,

$$A_{st} = P_t (bd)$$

$$= \frac{0.136}{100} (1000 \times 342)$$

$$= 465.120 \text{mm}^2.$$

As the diameter of bar is 16mm

$A_{st} = \pi/4 \times 16^2$

$= 201.06 \, mm^2$

$$\text{spacing} = \frac{a_{st}}{A_{st}} \times 1000$$

$$= \frac{201.06}{465.120} \times 1000$$

$= 432.28 \, mm$

$= 430 mm \; c/c$

As the spacing is 430mm, assume diameter of bar as 12mm

$$\text{spacing} = \frac{\pi/4 \times 12^2}{465.120} \times 1000$$

$= 230 mm$

∴ provide 12mm φ bar @ 230 mm c/c.

Minimum reinforcement:

$A_{st} = 0.12\% \, bD$

$$= \frac{0.12}{100} \times 1000 \times 400$$

$= 480 \, mm^2$

use 10mm φ bars.

$$\text{spacing} = \frac{\pi/4 \times 10^2}{480} \times 1000$$

$= 163.625$

$\simeq 150 mm$

∴ provide 10mm φ bar @ 150 mm c/c.

Step 5: Check for shear:

One way shear:

$$A_s \text{ provided} = \frac{a_{st}}{\text{spacing}} \times 1000$$

$$= \frac{\pi/4 \times 10^2}{150} \times 1000 = 523.599\,mm^2$$

$$\left[\, d' = D - \text{cover} - \phi/2\right.$$
$$\left. = 400 - 50 - 10/2 = 325\,mm \,\right]$$

% steel,

$$P_t = 100\left[\frac{A_{st}}{bd}\right]$$

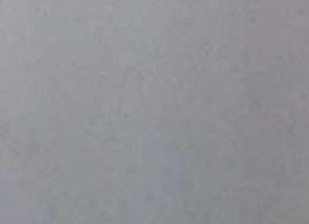
$$= \frac{100 \times 523.599}{1000 \times 325} = 0.163\%$$

From table 19,

$$P_t = 0.163\%$$
$$\tau_c = 0.29\,N/mm^2$$

$$\therefore V_{uc} = \tau_c\, b\, d$$
$$= 0.29 \times 1000 \times 325$$
$$= 94.38 \times 10^3\,N$$
$$= 94.38\,KN$$

For oneway shear, the critical section is at a distance of 'd' from the wall fall.



$$\therefore V_u = \text{Shear force}$$
$$= 1.5\,(p')\,(b \times d)$$
$$= 1.5 \times 150\,(1 \times 0.325)$$
$$= 73.12\,KN < 94.38\,KN.$$

Hence safe.

Design of sloped footing :

Design a RC circular footing for a circular column of 300mm ϕ supporting a factored axial load of 750KN. Adopt the SBC of the soil as 200 KN/m^2 & use M20 grade concrete & Fe 415 HYSD bar.

Given :

$P_u = 750$ KN

$D = 300$ mm

$SBC = 200$ KN/m^2

Step 1 : Total load :

Working load $= \frac{750}{1.5} = 500$ KN

Self weight of footing (10%) $= 500 \times \frac{10}{100} = 50$ KN

Total load $= 500 + 50 = 550$ KN

Step 2 : Size of footing :

$A = \frac{P}{\sigma_{sbc}} = \frac{550}{200} = 2.75\,m^2$

$A = \pi/4\, D_f^2$

$2.75 = \pi/4\, D_f^2$

$D_f = 1.87$ m

$\simeq 2$ m

Adopt diameter of footing $= 2$ m.

Step 3: upward soil pressure:

$$w = \frac{P_u}{\text{Area provided}}$$

$$= \frac{750}{\pi/4 \times 2^2}$$

$$= 238.8 \text{ kN/m}^2$$

$238 < (1.5 \times 200) = 300 \text{ kN/m}^2$

Hence diameter of footing is adequate.

Centre of gravity of quadrant of footing 'obc' from 'o' is

$$= 0.6 \left[\frac{R^2 + r^2 + R \cdot r}{R + r} \right]$$

$$= 0.6 \left[\frac{1000^2 + 150^2 + (1000 \times 150)}{(1000 + 150)} \right]$$

$$= 610 \text{ mm}$$

ϕ of footing
$D = 2000$ mm
$R = 1000$ mm
ϕ of column
$D = 300$ mm
$r = 150$ mm

upward load on area $bb'cc'$ is computed as

$$= \left[\pi/4 \; (R - r^2) \times w \right]$$

$$= \pi/4 \; (1^2 - 0.15^2) \times 238.6$$

$$= 183 \text{ kN}.$$

Step 4: Bending Moment:

Maximum Bending moment at face of the column, M_u = U/w load (C.G − r)

$= 183\,(0.61 - 0.15)$

$= 84.2$ KNm.

Breadth of footing at column face (for one quadrant c'b')

$= \left[\frac{\pi}{4} \times 300\right]$

$= 235$ mm

Step 5: Depth of footing:

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

$84.2 \times 10^6 = 0.138 \times 20 \times 235 \times d^2$

$d = 360$ mm

Depth required from shear consideration will be nearly 1.5 times for moment consideration.

$d = 540$ mm

$D = 600$ mm

Step 6: Reinforcement:

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

$84.2 \times 10^6 = 0.87 \times 415 \times A_{st} \times 540 \left[1 - \frac{415 \times A_{st}}{20 \times 235 \times 540}\right]$

$A_{st} = 484\ mm^2$.

Min. Ast = 0.12% bD

$= \frac{0.12}{100} \times 235 \times 600$

$= 169.2 mm^2$

~~Pro~~ $a_{st} = \pi/4 \times 12^2$

$$\text{Spacing} = \frac{a_{st}}{A_{st}} \times 1000$$

$$= \frac{\pi/4 \times 12^2}{484} \times 1000$$

$= 150$ mm.

Provide 12mm ϕ @ 150 mm centres both ways.

Step 7: check for shear stresses :

ultimate shear force at a distance of 0.54 m from the face of column is computed as

$$V_u = w\left[D_f^2 - (D+r)^2\right] \times \pi/4$$

$$= 238.8\left[2^2 - (1+0.15)^2\right] \times \pi/4.$$

$$= 238.8\left[2^2 - 1.3^2\right] \times \pi/4$$

$= 408$ kN.

Shear per meter width of perimeter is

$$\frac{V_u}{\pi \times [2(D+r)]} = \frac{408}{\pi \times 1.35}$$

$$= 96\,KN.$$

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

$$= \frac{96 \times 10^3}{1000 \times 540}$$

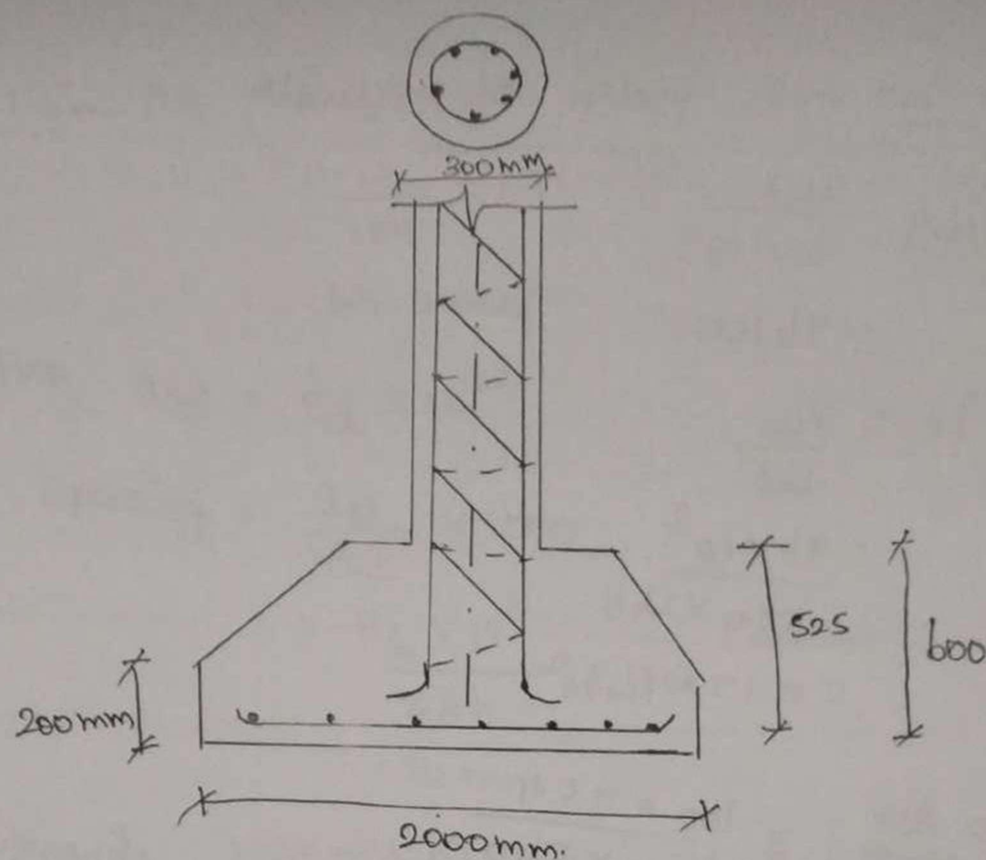
$$= 0.17\,N/mm^2$$

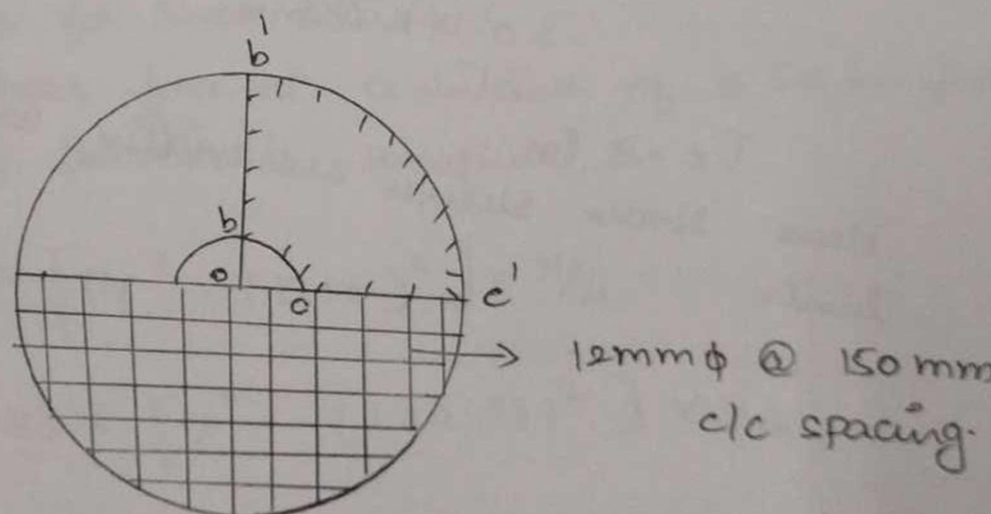
$$\frac{100\,A_{st}}{bd} = \frac{100 \times 754}{1000 \times 540}$$

$$= 0.143\,N/mm^2$$

$$\tau_c > \tau_v.$$

Hence shear stresses are within safe permissible limits.


300 mm
525
600
200 mm
2000 mm.


b'
b
o
c
c'
12mm ϕ @ 150 mm
c/c spacing.

Design a reinforced concrete footing for a rectangular column of a section 300mm × 500mm supporting an axial factored load 1500KN. The safe bearing capacity of soil at site is 185 KN/m². Adopt M20 grade concrete & Fe415 HYSD bars.

Step 1: Total load:

Working load $= \frac{1500}{1.5} = 1000\,KN$

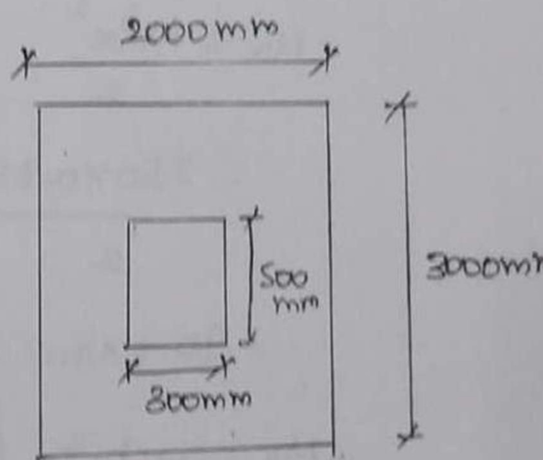
Self weight of footing (10%) $= \frac{1000 \times 10}{100}$

$= 100\,KN$

Total load $= 1000 + 100$

$= 1100\,KN$

Step 2: Size of footing:



$A = \frac{P}{\sigma_{cbc}}$

$= \frac{1100}{185}$

$= 5.9 \simeq 6m^2$

$\frac{L}{B} = \frac{500}{300} = 1.6$

$A = L \times B$

$6 = 1.6B \times B$

$B = 1.89\,m$

$L = 3.13m$

$B \simeq 2m$

$L \simeq 3m$.

Adopt rectangular footing size 2m×3m.

Step 3: Factored soil pressure (W):

$$W = \frac{P_u}{\text{Area provided}}$$

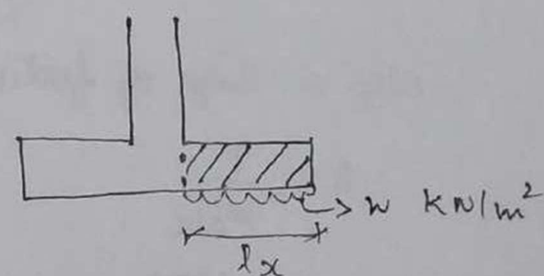
$$= \frac{1500}{2 \times 3}$$

$$= 250\ KN/m^2$$

$250 < (1.5 \times 185) = 277.5\ KN/m^2$.

Hence the footing area is adequate since the soil pressure developed @ base is less than the SBC of soil.

Step 4: Bending moment:



$$M_x = \frac{W l_x^2}{2}$$

$$= \frac{250 \times 0.85^2}{2}$$

$$= 90\ KNm.$$

$$L_x = \frac{B-b}{2} = \frac{2000-300}{2} = 850$$

$$L_y = \frac{3000-500}{2} = 1250$$

$$M_y = \frac{W l_y^2}{2}$$

$$= \frac{250 \times 1.25^2}{2}$$

$$= 195\ KNm.$$

use greater value, $M_x = M_y = 195\ KNm$.

steps: Depth of footing:

a) From moment consideration:

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

$195 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

$\boxed{d = 266 mm}$

b) From Shear consideration:

$V_{uL} = W [L_y - d]$

$= 250 (1250 - d) N$

Assuming the shear strength $\tau_c = 0.36 N/mm^2$ for M_{20} grade concrete with nominal % of reinforcement

$P_t = 0.25$

$\tau_c = \frac{V_{uL}}{bd}$

$0.36 = \left[\frac{250 (1250 - d)}{1000 \times d} \right]$

$d = 513 mm$.

$d \simeq 550 mm$

$\boxed{D = 600 mm}$

step 6: Reinforcement in footing :

a) Longer direction :

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

$$195 \times 10^6 = 0.87 \times 415 \times Ast \times 550 \left[1 - \frac{Ast \times 415}{20 \times 1000 \times 550}\right]$$

$$Ast = 1025\, mm^2$$

$$ast = \pi/4 \times 16^2$$

$$= 201\, mm^2$$

$$\text{Spacing} = \frac{ast}{Ast} \times 1000 \qquad \text{No. of bar} = \frac{Ast}{ast}$$

$$= \frac{201}{1025} \times 1000 \qquad = \frac{1025}{201}$$

$$= 5.09$$

$$\simeq 6$$

$$= 196$$

$$\simeq 200\, mm$$

provide 16 mm φ @ 200 mm c/c spacing.

b) shorter direction :

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

$$90 \times 10^6 = 0.87 \times 415 \times Ast \times 550 \left[1 - \frac{Ast \times 415}{20 \times 1000 \times 550}\right]$$

$$Ast = 465\, mm^2$$

$$ast = \pi/4 \times 12^2 = 113\, mm^2$$

$$\text{spacing} = \frac{ast}{Ast} \times 1000 = \frac{113}{465} \times 1000 = 243 \simeq 250\, m$$

Provide 12 mm φ @ 250 mm c/c spacing.

Step 7: check for shear stresses:

$$V_u = w\left[\frac{L_y}{2} + \frac{\text{col. size}}{2} - d\right]$$

$$= 250\left[\frac{2000}{2} + \frac{500}{2} - 550\right]$$

$$= 175\ \text{KN}$$

$$\frac{100\,A_{st}}{bd} = \frac{100\left(6 \times \pi/4 \times 16^2\right)}{1000 \times 550}$$

$$= 0.219.$$

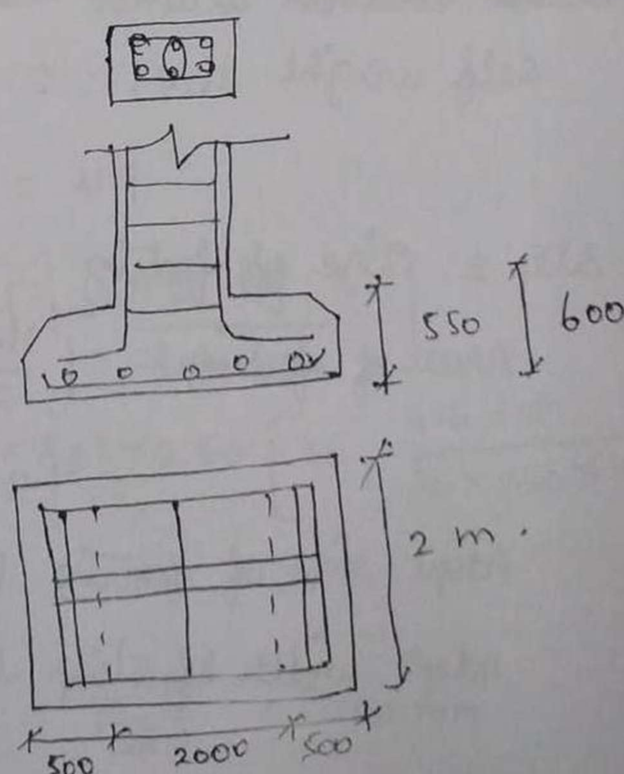
From table 19.

$$\tau_c = 0.33\ \text{N/mm}^2$$

Nominal shear stress

$$\tau_v = \frac{V_u}{bd} = \frac{175 \times 10^3}{1000 \times 550} = 0.315.$$

$\tau_v < \tau$, shear stresses are within the permissible limit.



Combined footing :

Design a combined column footing with a strap beam for two reinforced concrete columns 300mm by 300mm size spaced at 4m apart & each supporting a factored axial load of 750KN. Assume the ultimate bearing capacity of soil at site as 225 KN/m².
Adopt M20 grade concrete & Fe415 HYSD bars.

Given :

$b = 300mm$

$D = 300mm$

Spacing $= 4m$.

Load $= 750KN$ on each column

$SBC = 225 KN/m^2$

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

$f_y = 415 N/mm^2$

Step 1: Loads on footing :

Total load on column $= 2 \times 750 = 1500 KN$.

Self weight 10% $= 150 KN$.

$P_u = 1650 KN$

Step 2: Size of footing :

Area of footing $= \left(\frac{1650}{225}\right)$

$= 7.33 m^2$

Adopt size of footing 6m by 1.5m.

Adopt width of strap beam $= b = 400mm$.

Step 3: Design of footing:

Soil pressure $= P_u = \dfrac{1500}{6 \times 1.5}$

$= 166.6\ kN/m^2 < 225\ kN/m^2$.

cantilever projection of footing $= 0.5\,(1.5 \times 0.4)$

$= 0.55\,m$.

ultimate design moment $= M_u = \dfrac{P_u L^2}{2}$

$= \left(\dfrac{0.5 \times 166.6 \times 0.55^2}{2}\right)$

$= 25.2\ kNm$.

Effective depth of footing $= d = \sqrt{\dfrac{M_u}{0.138\, f_{ck}\, b}}$

$= \sqrt{\dfrac{25.2 \times 10^6}{0.138 \times 20 \times 1000}}$

$= 96\,mm$.

But the depth based on shear consideration will be nearly double than that due to moment consideration. Hence adopt

$d = 250\,mm$

$D = 300\,mm$

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

$25.2 \times 10^6 = 0.87 \times 415 \times A_{st} \times 250 \left[1 - \dfrac{415\, A_{st}}{20 \times 250 \times 1000}\right]$

$A_{st} = 287\,mm^2$.

$a_{st} = \pi/4 \times 10^2$

Spacing $= \dfrac{a_{st}}{A_{st}} = \dfrac{\pi/4 \times 10^2}{287} = 200\ mm$.

Minimum reinforcement = 0.12 % bD

$$= \frac{0.12}{100} \times 1000 \times 300$$

$$= 360 mm^2.$$

Step 4: Check for shear stress:

$$V_u = (0.55 - 0.25) 166.6$$

$$= 50 KN$$

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

$$= \frac{50 \times 10^3}{1000 \times 250}$$

$$= 0.2 N/mm^2$$

$$\frac{100 A_{st}}{bd} = \frac{100 \times 393}{1000 \times 250}$$

$$= 0.157.$$

Refer Table 19.

$$\tau_c = 0.28 N/mm^2.$$

$$k_s \tau_c = 1 \times 0.28$$

$$= 0.28 N/mm^2.$$

$$k_s \cdot \tau_c > \tau_v.$$

∴ shear stresses are within safe permissible limits.

Step 5: Design of strap beam:

Factored load on beam = $W_u = 1.5 \times 166.6$

$= 250 \, KN/m$.

Neglecting the small cantilever portion of beam:

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

$$= \frac{250 \times 4^2}{8}$$

$$= 500 \, KNm.$$

$$V_u = \frac{W_u l}{2} = \frac{250 \times 4}{2} = 500 \, KN.$$

Depth of strap beam computed based on moment.

Assume, $\tau_c = 1.2 \, N/mm^2$:

$$d = \frac{V_u}{b \cdot \tau_c}$$

$$= \frac{500 \times 10^3}{400 \times 1.2}$$

$$= 0.2 \, N/mm^2$$

Adopt,

$d = 1150 \, mm$ &

$D = 1200 \, mm$.

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

$$500 \times 10^6 = 0.87 \times 415 \times A_{st} \times 1150 \left[1 - \frac{415 \times A_{st}}{400 \times 1150 \times 20}\right]$$

$$A_{st} = 1290 \, mm^2.$$

$$a_{st} = \pi/4 \times 22^2$$

$$No. of\ bar = \frac{A_{st}}{a_{st}}$$

$$= \frac{1290}{\pi/4 \times 22^2}$$

$$\simeq 4$$

Provide 4 # of 22 mmϕ bar.

$$\tau_v = \frac{V_u}{bd} = \frac{500 \times 10^3}{400 \times 1150} = 1.09\ N/mm^2$$

$$\frac{100\ A_{st}}{bd} = \frac{100 \times 1520}{400 \times 1150} = 0.33.$$

From table 19,

$$\tau_c = 0.40\ N/mm^2 < \tau_v.$$

Hence shear reinforcements are required to resist the balanced shear force computed as:

$$V_{us} = [500 - (0.4 \times 400 \times 1150) \times 10^{-3}]$$

$$= 316\ KN.$$

using 8 mm ϕ 4 legged stirrups the spacing is

$$S_v = \frac{0.87 \times 415 \times 4 \times 50 \times 1150}{316 \times 1000}$$

$$= 262\ mm.$$

Design a combined column footing with a strap beam for two reinforced [illegible] columns [illegible] size spaced 4 m apart & each supporting a factored axial load of 750 kN. Assume the ultimate bearing capacity at site as 225 kN/m². Adopt M20 grade concrete & Fe 415 HYSD bars.

Step 1: Data:

Size of column = 300 × 300 mm

Spacing of column = 4 m

Factored load on each column = 750 kN

ultimate bearing capacity of soil = 225 kN/m²

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

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

Step 2: Loads on footing:

Total load on column = 2×750

= 1500 kN

self weight (10%) = 150 kN

Total ultimate load = Pu = 1650 kN.

Step 3: Size of footing:

$$A = \frac{Pu}{SBC} = \frac{1650}{225}$$

$$= \cancel{1.66}\ 7.3\,m^2$$

Adopt a footing of size 6m by 1.5m

Adopt width of strap beam = b = 400 mm.

Step 4: Design of footing:

Soil pressure = $P_u = \frac{\text{Load}}{\text{Area provided}}$

$= \frac{1500}{6 \times 1.5}$

$= 166.6\ KN/m^2 < 225\ KN/m^2$

cantilever projection of footing $= \frac{(1.5 \times 0.4)}{2}$

$= 0.55\ m.$

ultimate design moment = $M_u = \frac{P_u L^2}{2}$

$= \frac{166.6 \times 0.55^2}{2}$

$= 25.2\ KNm.$

Effective depth of footing = $d = \sqrt{\frac{M_u}{0.138\ f_{ck}\ b}}$

$= \sqrt{\frac{25.2 \times 10^6}{0.138 \times 20 \times 1000}}$

$= 96\ mm.$

But the depth based on shear considerations will be nearly double than that due to moment considerations.

Hence, adopt, $d = 250\ mm$

$D = 300\ mm$

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

$$25.2 \times 10^6 = 0.87 \times 415 \times A_{st} \times 250 \left[1 - \frac{A_{st} \times 415}{1000 \times 250 \times 20}\right]$$

$$A_{st} = 287 \text{ mm}^2$$

Minimum reinforcement $= 0.12\ \%\ bD$

$$= \frac{0.12}{100} \times 1000 \times 300$$

$$= 360 \text{ mm}^2$$

Adopt 10 mm ϕ @ 200mm centres.

Step 5: check for shear stresses:

V_u = (Cantilever projection - d) p_u.

$$V_u = (0.55 - 0.25)\, 166.6$$

$$= 50 \text{ KN}.$$

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

$$= \frac{50 \times 10^3}{1000 \times 250}$$

$$= 0.2 \text{ N/mm}^2$$

$$\frac{100 A_{st}}{bd} = \frac{100 \times 360}{1000 \times 250}$$

$$= 0.157.$$

From table 19,

$$\tau_c = 0.28 \text{ N/mm}^2.$$

$$\tau_c > \tau_c$$

$\therefore$ Shear stresses are within safe permissible limits.

step 6: Design of strap beam:

Factored load on beam = W_u = 1.5×166.6

$= 250\ KN/m$

Neglecting the small cantilever portion of the beam:

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

$$= \frac{250 \times 4^2}{8}$$

$$= 500\ KNm.$$

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

$$= \frac{250 \times 4}{2}$$

$$= 500\ KN$$

Depth of strap beam computed based on moment.

Assuming, $\tau_c = 1.2\ N/mm^2$

$$d = \frac{V_u}{b \cdot \tau_c}$$

$$= \frac{500 \times 10^3}{400 \times 1.2}$$

$$= \cancel{0.2\ N/mm^2}\ 1041\ mm^2$$

$d = 1150\ mm$ &

$D = 1200\ mm$

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

$$500 \times 10^6 = 0.87 \times 415 \times A_{st} \times 1150 \left[1 - \frac{A_{st} \times 415}{400 \times 1150 \times 20}\right]$$

$A_{st} = 1290\ mm^2$

$a_{st} = \pi/4 \times 22^2$

No. of bar $= \frac{A_{st}}{a_{st}}$

No. of bar = $\frac{1290}{\frac{\pi}{4} \times 22^2}$

$= 4$

Provide 4 # of 22 mm ϕ bar.

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

$= \frac{500 \times 10^3}{400 \times 1150}$

$= 1.09 \ N/mm^2$

$\frac{100 \ Ast}{bd} = \frac{100 \times 1520}{400 \times 1150}$

$= 0.33$

From table 19,

$\tau_c = 0.40 N/mm^2$

$\tau_c < \tau_v$

Hence shear reinforcements are required to resist the balanced shear force computed as:

$V_{us} = [500 - (0.4 \times 400 \times 1150)]$

$= 316$ KN.

use 8 mm ϕ 4-legged stirrups

$S_v = \frac{0.87 \ f_y \ ASV}{V_{us}}$

$= \frac{0.87 \times 415 \times 4 \times 50 \times 1150}{316 \times 10^3}$

$= 262$ mm

use 8mm ϕ 4legged stirrups at 262mm c/c spacing.