A LECTURE NOTE

ON

STRUCTURAL DESIGN-I

(TH-1)

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(1) Define reinforced concrete

      - Reinforced cement concrete is a composite material is made of concrete & steel reinforcement

      - The concrete may be assumed to work purely in compression whereas the reinforcement is predominately subjected to tension.

(2) What is the purpose of using reinforced cement concrete?

(i) Plain cement concrete has very low tensile strength. The tensile strength of concrete is about one-tenth of its compressive strength. As a result, a plain concrete beam fails suddenly as soon as the tension cracks start to develop.

(ii) To improve the tensile strength of concrete, some sort of reinforcement is needed which can take up the tensile stren developed in the structure.

(iii) It is not only increases the strength but also in preventing the temperature & shrinkage.

(iv) Therefore, reinforcing steel is added in the tension zone to carry all the developed tensile stresses.

② What are the advantages of RCC when compared with other building materials?

(i) Concrete is workable when fresh & strong when hardens.

(ii) It can be molded into any required shape & size.

(iii) The raw materials required are easily available.

(iv) Skill is not required for casting concrete elements.

(v) Concrete is durable, fire resisting & rigid.

(vi) Concrete requires less maintenance.

④ What are the disadvantages of RCC when compared with other building materials?

(i) The self-weight of the structural elements will be more while concrete is used.

(ii) Concrete has a very low tensile strength. Hence cracks will form in the tension zone if reinforcement is not provided properly.

(iii) Cracks develop in concrete, also due to shrinkage, creep, temperature etc. which permit seepage of water into the concrete. This causes corrosion of steel reinforcement & thereby peeling of concrete.

(iv) Concrete has poor insulating property.

(v) Dismantling & reusing of concrete elements are mostly not possible.

(vi) Concrete is brittle in nature & hence has

low impact resisting capacity

⑤ What are the uses of reinforced concrete?

It is used for the construction of
(i) building
(ii) Bunkers & silos
(iii) Chimneys & towers
(iv) flyovers
(v) Retaining walls
(vi) Roads & railway bridges
(vii) Water tanks

⑥ What are the types of load on RCC structures?

(i) Dead load
(ii) Live load or Imposed load
(iii) Wind load
(iv) Snow load
(v) Earthquake load or
Seismic load.

⑦ What are the elements of structure?

(i) Beam
(ii) Column
(iii) Floor
(iv) Foundation
(v) Slab
(vi) Staircase

⑧ What are the methods of design?

(i) Modular Ratio Method/working stress method (WSM)/
Elastic Method of design
(ii) Load factor method/ Ultimate load Method (ULM)/
ultimate strength method

(ii) Limit state method (LSM)

(i) WSM :
- Elastic behaviours of materials are used in WSM.

- The working stress method of design of a structure is defined as a method which limits the structural usefulness of the material of the structure, upto a certain load at which the maximum stress in extreme fibre reaches the characteristic strength of material in bending)

(ii) ULM :

- This method is otherwise known as load factor method

- This method is based on the ultimate strength, when the design member would fail.

- In this method factors are taken into account only on loads are load factors.

- The method of ultimate design of a structure is defined as a method which limits the structural usefulness of the material of the structure upto ultimate load.

(ii) LSM :

- The limit state method is defined as a method which limits the structural usefulness of the material of the structure upto a certain load at which acceptable limit of safety & serviceability are applied so that the failure of structure does not occur.

- It is the combination of WSM & ULM

- In this method partial factor of safety is considered on both loads & stresses.

- This method is advance over other methods. Since safety & serviceability are considered.

② Define characteristic load.

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

$$F_k = F_m + KS_d$$

where,
F_k = characteristic load
F_m = mean load
K = constant = 2.015 ≈ 2.65
S_d = standard deviation for the load

(13) Define permissible stress.

It is defined as the ratio of yield stress to the factor of safety.

$$\text{Permissible Stress} = \frac{\text{Ultimate or yield strength of material}}{\text{Factor of safety}}$$

(14) Define factor of safety.

- It is defined as the ratio of ultimate stress to working stress for brittle materials or yield stress to working stress for ductile material

$$\boxed{FOS = \frac{\text{ultimate stress}}{\text{working stress}}} \quad (\text{for brittle material})$$

- It accounts all uncertainties such as material defects, unforeseen loads, manufacturing defects, unskilled workmanship & temperature effects etc.

(15) Define modular ratio

- It is defined as the ratio of elastic modulus of steel to that of concrete.

- It is used to transform the composite section into an equivalent concrete section.

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

16. What is the expression recommended by the IS 456 - 2000 for modulus of elasticity?

Modulus of elasticity = $\boxed{E_c = 5000 \sqrt{f_{ck}}}$

17. State the assumption made for design of RC members in working stress method. (Refer IS456 : 2000 Eg - 80) Plane section plane before bending with plane after bending remain

(i) Plane section before bending will plane after bending.
(ii) Bond between steel and concrete is perfect within elastic limit of steel.
(iii) The steel & concrete behaves as linear elastic materials.
(iv) All tensile stresses are taken by reinforcement & not by concrete.
(v) The strain in steel & concrete are related by a factor known as "modular ratio".
(vi) The stress - strain relationship of steel & concrete is straight line under working load.

18. What are the advantages in limit state method?

(i) Ultimate load method only deals with on safety such as strength, overturning & sliding, buckling, fatigue.
(ii) Working stress method only deals with serviceability such as crack, vebration, deflection etc.

(iii) But, limit state method advances than other two methods. Hence by considering safety at ultimate load & serviceability at working load.

(iv) The process of stress redistribution & moment redistribution are considered in the analysis & more realistic factor of safety values are used in the design. Hence, the design by limit state method is found to be more economical.

(v) The overall sizes of flexural members arrived by limit state method are less & hence they provide better appearance to the structure.

(9) Advantages & Disadvantages of WSM.

Advantages :

(i) The design usually results in relatively large section of structural members, compared to ultimate load. Due to that structures designed by working stress method gives better serviceability performance under working load.

(ii) This method is only the method available when one has to investigate the reinforced concrete section for service stresses & for the serviceability state of deflection & cracking.

Disadvantages:

(i) The WSM doesn't show the real strength nor gives the true factor of safety of the structure under failure.

(ii) The modular ratio design results in larger % of compression steel than that given by the limit state design, thus leading to uneconomical design.

(iii) Because of creep & non-linear stress-strain relationship, concrete doesn't have definite modulus of elasticity.

(iv) The WSM fails to discriminate between different types of loads that act simultaneously but have different uncertainities.

(21) Define advantages & disadvantages of ultimate load method.

Advantages:

(i) While the WSM uses only the nearly linear part of stress-strain curve, the ULM uses fully the actual stress-strain curve.

(ii) The load factor gives the exact margin of safety against collapse.

(iii) The method allows using different load factors for different types of loads & the combination thereof.

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(ii) The load factor gives the exact margin of safety against collapse.

(iii) The method allows using different load factors for different types of loads & the combination thereof.

(iv) The failure load computed by ULM matches with the experimental results.

(v) The method is based on the ultimate strain as the failure criteria.

(vi) The method utilizes the reserve of strength in the plastic region.

Disadvantages :

(i) The method does not take into consideration the serviceability criteria of deflection & cracking.

(ii) The use of high strength reinforcing steel & concrete results in increase of deflection & crack width.

(iii) The method does not take into consideration the effects of creep & shrinkage.

(iv) In the ULM, the distribution of stress resultants at ultimate load is taken as the distribution at service loads magnified by the load factor. This is erroneous since significant redistribution of stress resultants takes place as the loading is increased from service loads to ultimate load.

(13) What are the factors considered in limit state of collapse ?

(i) Flexure.
(ii) Compression
(iii) Shear
(iv) Torsion

(14) What are the factors considered in limit state of serviceability ?
(i) Cracking
(ii) Deflection

(ii) Durability

(iii) Fire resistance

(iv) Vibration

(26) What are the factor of safety in limit state?

Partial safety factor for concrete $\gamma_c = 1.5$

for steel $\gamma_s = 1.15$

for load $\gamma_f = 1.5$

(27) What is under-reinforced section?

Steel reaches maximum permissible stress earlier than concrete due to external loads is called under-reinforced section.

(28) Over-reinforced section

Concrete reaches maximum permissible stress earlier than steel due to external load is called over-reinforced section.

(29) Balanced Section

Concrete & steel reaches minimum permissible stress simultaneously due to external load is called balanced section.

(30) Singly reinforced section

Steel reinforcements are provided only on tension zone of RC flexural members is known as singly reinforced section.

(ii) Durability

(iii) Fire resistance

(iv) Vibration

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Steel reinforcements are provided only on tension zone of RC flexural member is known as singly reinforced section

③ Doubly Reinforced Section.

- Steel reinforcements are provided on both tension & compression zone of the flexural member is known as doubly reinforced section.

- In some situations it becomes essential for a beam to carry BM more than that it can resist as a balanced section.

- In this case additional reinforcement is provided in compression zone such beams reinforced in both compression & tension zone are known as doubly reinforced section.

- When $M_u > M_{u,lim}$ then ⇒ Doubly reinforced section.

③ⓑ Write down the basic values of span to effective depth ratio for the different types of beam.

Basic values of Span-to-Depth ratio for spans upto 10m.

Cantilever	7
Simply Supported	20
Continuous	26

③ⓒ Define Collapse state.

The limit state of collapse of the structure or part of the structure could be assessed from rapture of one or more critical sections & from buckling due to

elastic or plastic instability or overturning.

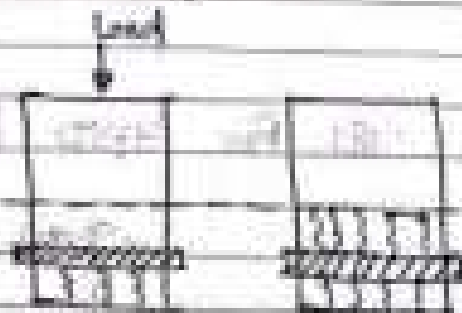(25) Define Stress Section, Transformed Section, cracked section

(Refer ... ... 456 = saree fig. 20)

Transformed Section:

When we replace the steel with equivalent concrete, we have effectively transformed everything to concrete. The resulting all concrete beam is called transformed section.
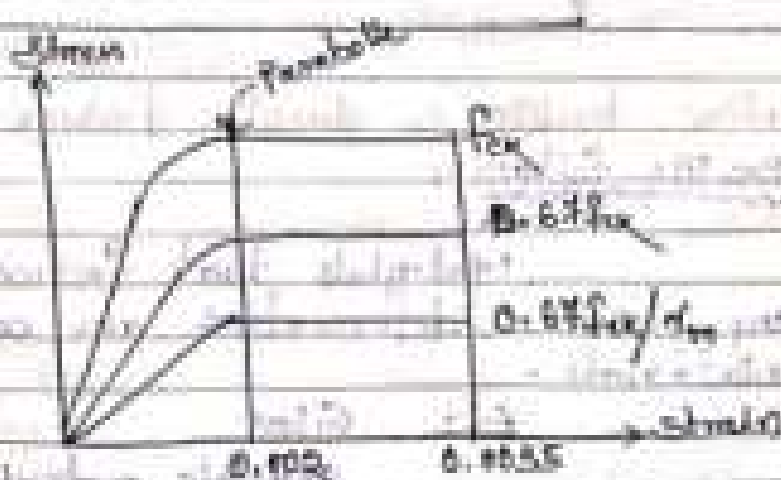
Cracked Section:

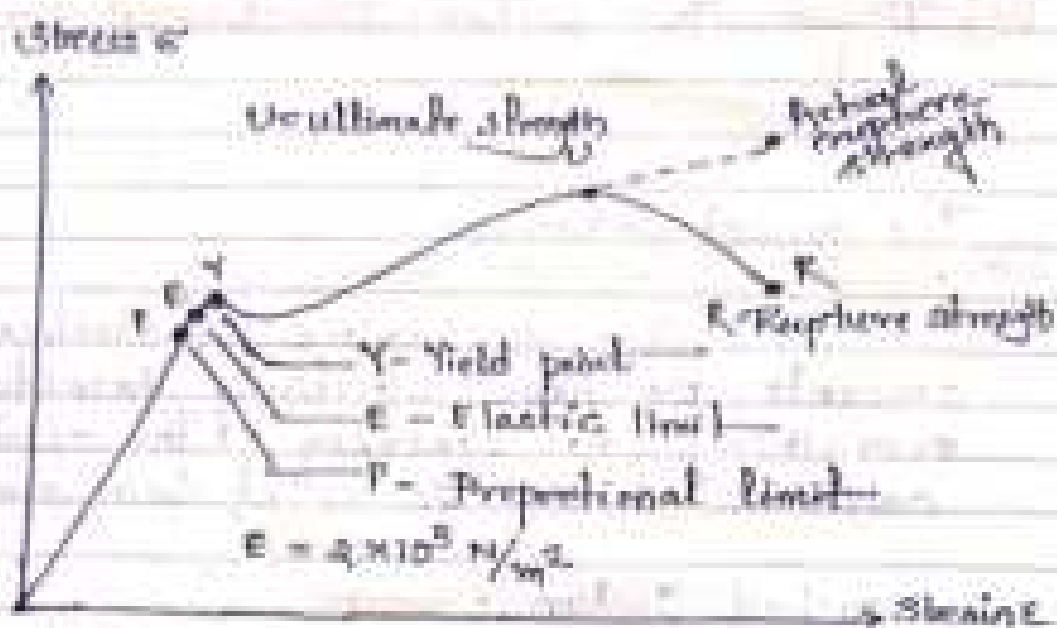As the Load increases, it cracks at the bottom first, finally all the concrete in tension will crack.

(26) Draw the stress-strain curve for concrete, mild steel bars & HYSD.
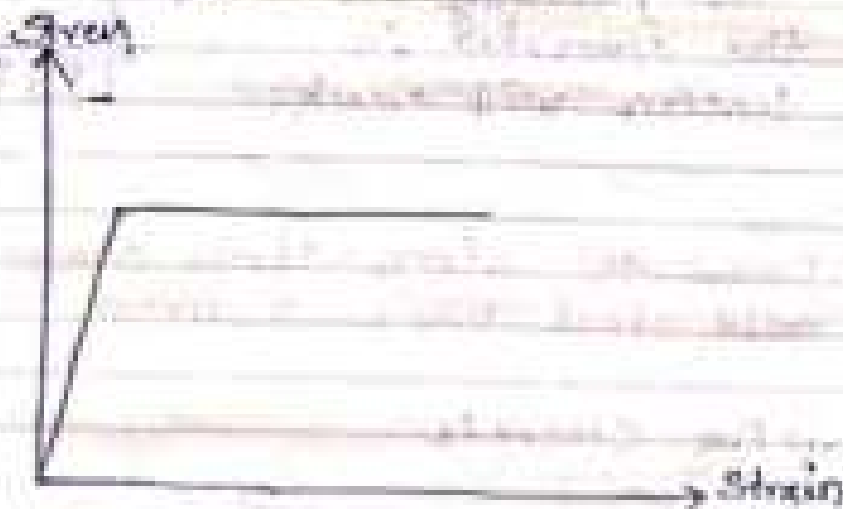
(i) For Concrete

f_{ck}

$0.67 f_{ck}$

$0.67 f_{ck}/\gamma_m$

0.002   0.0035 Strain

(ii) For mild steel

Stress σ

U-ultimate strength

Fracture rupture strength

R-Rupture strength

Y - Yield point
E - Elastic limit
P - Proportional limit

$E = 2 \times 10^5 \ N/m^2$

Strain ε

(ii) For HYSD (High yield strength deformed) bars

Stress

Strain

(37) Define brittle & ductile failure

Brittle failure :

Materials that facture without any plastic deformation are called brittle materials.

Ex: Glass
    ceramic materials

Ductile failure :

Materials :- uniform plastic deformation before fracture is called ductile material

Ex : Aluminium
Copper
steel & many metals

Polyethylene, nylon & many polymers

(59) Clear Cover

The distance between the bottom of the bars & bottom most edge of the beam is called clear cover.

(60) Effective Cover

The distance between the centre of the reinforcement bars & the bottom edge of the beam is called effective cover.

$$\boxed{\text{Effective cover} = \text{clear cover} + \frac{\text{diameter of bar}}{2}}$$

(61) Grades of concrete & steel

Grade of concrete M5, M7.5, M10, M15, M20, M25

Grade of steel Fe415, Fe500, Fe550

Grade of cement 33 grade, 43 grade & 53 grade

(1) What do you understand by development length of bars?
- The reinforced bar must extend in the anchorage zone of concrete sufficiently, to develop the required stress. The extended length of bar inside the face of the support is known as development length.
- It is denoted by the symbol L_d
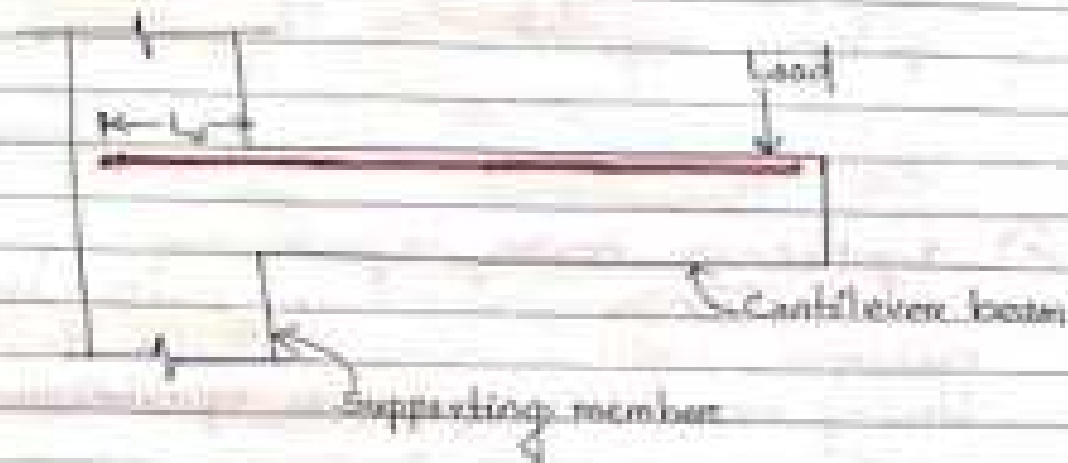
(2) Define anchorage length.
Anchorage length is defined as embedded portion of the bar to concrete, but not subjected to any flexural bond.

(3) Define anchorage bond.
- All types of reinforcement must be anchored within the concrete section, in order that the anchorage bond should be sufficient to develop the stress in the bars.
- The anchorage depends on the bond between the bar & concrete areas of contact.

④ Define curtailment of bars.

- In flexural members, design of reinforcement is done based on bending moment along the span.

- As the magnitude of bending moment on a beam decreases along its length, that case the area of bending reinforcement may be reduced by curtailing bars as they are no longer required.

⑤ Equilibrium Torsion

Torsion induced by eccentric loading & equilibrium condition alone sufficient to determine twisting moments is known as equilibrium torsion.

⑥ Torsion

Equal & opposite moments applied at both ends of a structural element or its part about its longitudinal axis is called torsion. It is also called as torsional moment / Twist / Torque.

⑦ Compatibility Torsion

Torsion induced by application of angle of twist & the resulting moment depends on the torsional stiffness of the member is known as compatibility torsion.

⑧ How can torsional resistance of RC members be enhanced?

Increasing strength of concrete & the amount of longitudinal as well as transverse reinforcements over & above those required for bending & shear can enhance the torsional resistance

of a member.

(8) Name the location In beam where the development lengths of tension bars should be checked.

At beams, development lengths should be checked at the sections where,

i → Max bending moment occurs
ii → Point of curtailment
iii → Point of inflation

(9) Write down the effect of torsion in RC beams.

- RC members may be subjected to torsion in combination with bending & shear. longitudinal & transverse reinforcement shall be provided for RC beams to resist torsion.

- Torsional reinforcement is not calculated separately from that required for bending & shear. Instead, the total longitudinal reinforcement is determined for a fictitious bending moment which is a function of actual bending moment & torsion.

(11) Write about local bond & anchorage length.

- All types of reinforcement must be anchored within the concrete section, in order that the anchorage bond should be sufficient to develop the stress in the bar.

- Anchorage length is defined as embedded portion of the bar in concrete, but not subjected to any flexural bond.

(12)

Flexural Bond	Development Bond
It arises in flexural members on account of change of variations in bending moment, which in turn causes a variation in axial tension along the length of a reinforcing [bar]	It arises over the length of anchorage provide the a bar or near the end of a reinforcing bar.

(12) Why is bond stress more in compression bars than in tension bars?

(i) Deformed bars subjected to tension, τ_{bd} values shall be increased by 60%.

(ii) Deformed bars subjected to compression, τ_{bd} values shall be increased by 25%.

(13) What are the types of reinforcement used to resist shear & write down the expressions for to shear resistance offered by the type?

  – Shear reinforcement is necessary if the nominal shear stress (τ_v) exceeds the design shear stress (τ_c).

  – In general, shear reinforcement is provided in only one of the following three forms. (Refer IS456:2000 fig 72)

   (i) Vertical stirrups

   (ii) Inclined stirrups

   (iii) Bent-up bars along with stirrups

(14) Write down the value of design bond stress for M30 grade of concrete.

  Design bond stress in LSM for plain bars (mild steel) in tension $\tau_{bd} = 1.5 \text{ N/mm}^2$

(15) What is RC slab?

  – Reinforced concrete slabs are used in roofs of buildings. Slab is a flexural member; transmits imposed & dead load to the supports.

  – Supports may be a wall, beam or column.

(16) Reinforced concrete slabs are generally safe & don't require shear reinforcement? why?

  Normally the thickness of slab is so chosen that the shear can be resisted by concrete

itself & the slab doesn't need extra shear reinforcement

⑩ Types of slab
 (i) One way slab $l_y/l_x > 2$
 (ii) Two way slab $\dfrac{l_y}{l_x} < 2$

    When the slab is supported only on two opposite sides, the slab bends in one direction only. Hence it is called one way slab.

    When the slab is supported on all four sides, the slab bends in both directions. Hence it is called two way slab.

⑫ Name two types of two-way slabs. Explain their difference in the design of slabs.
 (i) Slabs simply supported on the four edges with corners not held down & carrying UDL.
 (ii) Slabs simply supported on the four edges with corners held down & carrying UDL.
 (iii) Slabs with edges fixed or continuous & carrying UDL.

㉓ What are the codal provisions for a minimum reinforcement to be provided as main & secondary reinforcement in slab & their maximum spacing?

Minimum Reinforcement :

$$A_{st} = \dfrac{0.15\ bD}{100} \quad \text{(for mild steel)}$$

$$A_{st\ min} = \dfrac{0.15\ bD}{100} \quad \text{(for HYSD bars)}$$

Spacing = "min" $\left\{ \begin{array}{l} 3d \\ \\ 300mm \text{ (horizontal distance b/w parallel} \\ \text{main reinforcement bars)} \end{array} \right.$

Spacing = "min" of $\left\{ \begin{array}{l} 5d \\ \\ 450mm \text{ (horizontal distance b/w} \\ \text{parallel reinforcement bar} \\ \text{provided against shrinkage &} \\ \text{temperature.)} \end{array} \right.$

(24) Why is secondary reinforcement provided in one way RC slab?

  Secondary reinforcement is provided running perpendicular to the main reinforcement, in order to take the temperature & shrinkage stresses.

  It is otherwise called as distribution or temperature reinforcement.

(25) Explain the purposes of lintel beams in buildings.

  Lintels are provided over the openings of doors, windows etc. Generally, they support the load of the wall over it & sometimes also the live loads are transferred by the sub-roof of the room.

  Lintels takes the masonry load over the openings & distributes to the masonry located sides of opening.

(26) What type of slab usually used in practice, under-reinforced or over reinforced section?

  The depth of slab chosen from deflection requirements will be usually greater than the depth

required for balanced design

-Hence the area of steel required will be less than the balanced amount

-So the slab is designed as under reinforced section.

(28) What do you understand by flanged beam

The concrete in the slabs, which is on the compression forces & the steel in the steel in the tension side of the beam can carry the tension. These combined beam & slab units are called flanged beam.

(28) Define shear strength

The resistance to sliding offered by the material of beam is called shear strength.

(29) What are the important factors affecting the shear resistance of a reinforced concrete member without shear reinforcement?

(i) Characteristic strength of concrete
(ii) % of longitudinal steel
(iii) Shear span to depth ratio
(iv) Axial compressive/tensile force
(v) Effect of s/s
(vi) Effect of two way action

① Define column

A column, in general, may be defined as a member carrying direct axial load which causes compressive stresses of such magnitude that these stresses largely controls its design.

(i) It transmits load coming from beam or slab & distributes to the foundation usually columns are square, rectangle, circular & 'I' shaped in c/s

(2) It is reinforced with longitudinal & lateral ties.

(3) Load carrying capacity of column is depending upon longitudinal steel & c/s area of the column.

(4) lateral ties are giving lateral support to the longitudinal steel. The columns are analyzed for axial force & moment.

② Differentiate b/w long & short column

Based on slenderness ratio(λ) columns can be classified into long & short

$$\text{Slenderness ratio } \lambda = \frac{\text{Effective length}}{\text{least lateral dimension}}$$

short column λ < 12
long column λ > 12

③ Differentiate b/w uni-axial & bi-axial bending

Axial load & bending moment along one direction are applied simultaneously on the column is called uni-axial bending

Axial load & bending moment along two direction are applied simultaneously on the column is called bi-axial bending

4. According to IS code all columns should be designed for minimum eccentricity. Justify the statement

        Lateral loads such as wind & seismic loads are not considered in design
    (i) Misalignment in construction
    (ii) Slenderness effects not considered in design
    (iii) Accidental lateral or eccentric loads

5. Write down the formula for calculating minimum eccentricity.

$$e_{min} = \frac{L}{500} + \frac{D}{30}$$

        subjected to a minimum of 20mm
where,
    L = unsupported length of the column
    D = lateral dimension of the column

6. What is spiral column?
    For a circular column, longitudinal steel with closely spaced helix are called as spiral column.

7. What is the minimum & maximum % of reinforcement can be provided for a column?

    The c/s area of longitudinal reinforcement shall be not less than 0.8% not more than 0.6% of the gross cross sectional area of column
              $[0.8\% - 0.6\%]$

(8) What are the specifications for pitch of lateral ties in columns?

The pitch of the transverse reinforcement that be not more than the least of following distances

min of
{
(i) Least lateral dimension of the compression member

(ii) 16 times the smaller diameter of the longitudinal reinforcement bar to be tied

(iii) 300 mm
}

(9)

Braced Column	Unbraced Column
(i) In most of the cases columns are subjected to horizontal loads like wind, earthquake etc. If lateral supports are provided at the end of the column, the lateral loads are borne entirely by the lateral supports. Such columns are known as braced column	(i) Other columns, where the lateral loads have to be resisted by them, in addition to axial loads & end moments are considered as unbraced columns
(ii) It is not subjected to side sway	(ii) It is subjected to side sway

(10) What is pedestal?

Pedestal is a compression member, the effective length of which doesn't exceed three times the least lateral dimension

(11) What is slender column?

— If slenderness ratio of the column about either axis is greater than 12, is classified as long column

— Long column should be designed as slender column.

(12) Mention the functions of the transverse reinforcement in a RC column.

(i) To prevent longitudinal buckling of longitudinal reinforcement

(ii) To resist diagonal tension caused due to transverse shear due to moment / transverse load

(iii) To hold the longitudinal reinforcement in position at the time of concreting

(iv) To confine the concrete, thereby preventing its longitudinal splitting

(v) To impart ductility to the column

(vi) To prevent sudden brittle failure of the column

(13) Classify the column according to the material
   (i) prestressed concrete
   (ii) Reinforced cement concrete
   (iii) Stone
   (iv) Timber

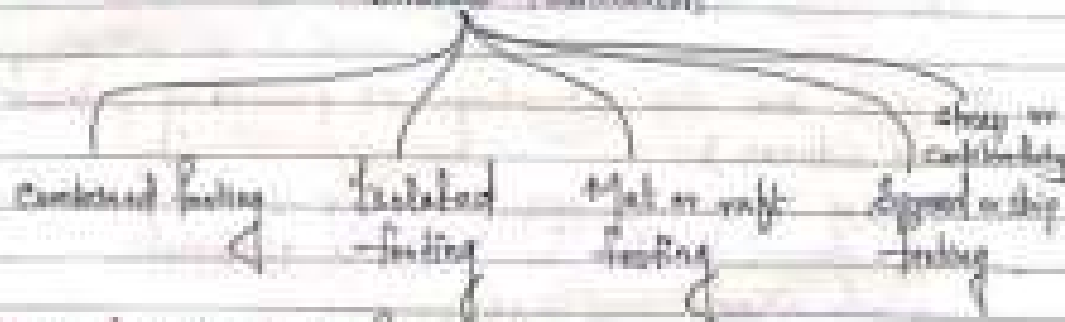(14) Classify the column according to transverse reinforcement
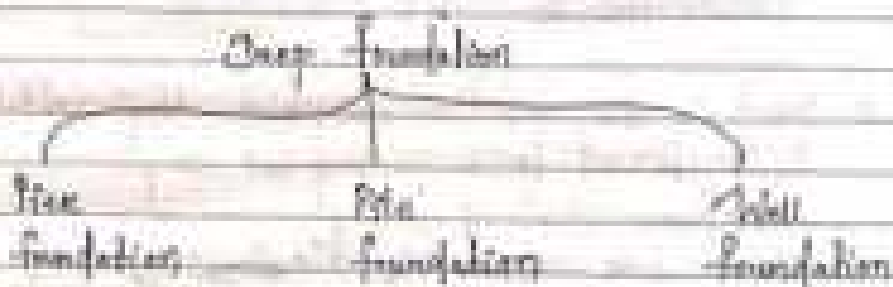   (i) Spiral or helical
   (ii) Tied

① what are the types of foundtion?
    (i) Deep foundation
    (ii) Shallow foundation.

② what are the types of shallow foundation?

Shallow foundation

combined footing   Isolated footing   Mat or raft footing   strap or cantilever spread strip footing

③ what are deep foundation?

Deep foundation

Pier foundation   Pile foundation   Well foundation

④ What are the factors governing to decide the depth of footing.

    The footing is generally to resist the bending moments & shear forces developed due to soil reactions. The main purpose of the footing is to effectively support the super structure.

⑤ Define safe bearing capacity of soil.

    It is the maximum intensity of load or pressure developed under the foundation without causing failure of soil. Unit for safe bearing capacity of soil is kN/m^2. Safe bearing capacity of soil is determined by the plate load test at the site.

6. What is punching or two way shear in RCC footing?

Punching shear is a type of shear failure occurs in reinforced concrete footing due to partial load from the column & upward soil thrust from the ground.

7. What are the advantages of providing pedestals to columns?

- When pedestals are providing & full force is transferred to the footing without additional reinforcement

- Pedestal provides a plane surface for the convenience of column construction

8. What is the situation in which trapezoidal shape is preferred to a rectangular shape for a two column — combined footing?

If the one column is carrying load is much larger than the other one, trapezoidal combined footing is preferred.

9. When combined footings are adopted?

(i) When two or more columns/walls are located close to each other &/or if they are relatively heavily loaded & rest on soil with low safe bearing capacity

(ii) An exterior column located along the periphery of the building is so close to the property line that an isolated footing can't be symmetrically placed without extending beyond the property line

10. Under what circumstances rectangular shape preferred for a two column combined footing ?

When loads are equal & no restriction on sides, the footing will be rectangular with equal overhang on both sides.

11. Under what circumstances combined footing is preferred.

(i) When isolated footings for individual columns are touching or overlapping each other

(ii) When the columns are located near the boundary lines or expansion joints

12. What is meant by eccentric loading on a footing & under what situation does this occur ?

The load P acting on a footing may act eccentrically w.r.t the centroid of the footing base. This eccentricity may result from one or more of the following effect

(i) The column transmitting a moment in addition to the vertical load

(ii) The column carrying a vertical load offset w.r.t the centroid of the footing

(iii) The column or pedestal transmitting a lateral force located above the foundation level in addition to vertical load

13. Write down the formula for calculating maximum & minimum soil pressures for a rectangular footing carrying eccentric point load

The structural design of the footing which includes the design of the depth & reinforcement, is done for factored loads using the relevant safety factors applicable for the limit state of collapse.

14. Define staircase.

- Staircase flights are generally designed as slab spanning between wall supports or landing beams or as cantilever from a longitudinal / inclined beam.

- The staircase fulfils the function of access between the various floors in the building

- Generally the flight steps consist of one or more landings b/w the floor levels

15. What are the components of stairs?

The component of stairs are

(i) Baluster
(ii) Flight
(iii) Going
(iv) Landing
(v) Rise
(vi) Riser
(vii) Soffit
(viii) Step
(ix) Tread
(x) Winders

16. What are the normal range of tread & rise values of steps of a staircase in residential building

As per IS456:2000 the normal range of tread & rise values of steps of a staircase in residential building are,

Rise : 150mm to 180mm
Tread : 200 mm to 250mm

17. List the various types of stair cases

(i) Bifurcated stairs
(ii) Dog-legged stairs
(iii) Geometrical stair such as circular, spiral

(iv) multi-flight stair
(v) Open newel stair with quarter space landing
(vi) Quarter turn stairs
(vii) straight stairs
(viii) Three quarter turn stair

21. How the effectively span of a stair is decided when the landing slab spans in the same direction as the stair?

When the landing slab spans in the same direction as the stairs, they should be considered as acting together to form a single slab & the span is determined at the distance centre to centre of the supporting beams or walls, the going being measured horizontally

22. Give the guidelines of the size of rise & tread as per IS code norms.

The following guidelines may be followed while deciding the size of rise & tread of a stair.

$$400mm < rise + tread < 450mm$$
$$580mm < rise + tread < 650mm$$

23. How the load is distributed in the case of an open well stairs?

In the case of stairs with open wells where spans partly crossings at right angles occur, the load in areas common to any two such spans may be taken as one-half in each direction.

24. How the load is distributed when flights or landings are embedded into walls?

Where flights or landings are embedded

into walls for a length not less than × 110mm & designed to span in the direction of the flight, a 150mm strip may be deducted from the loaded area & effective breadth of the section increased to 75mm for the purpose of design.

25. Define depth of section.

The depth of section shall be taken as the minimum thickness perpendicular to the soffit of the staircase.

26. What are the loads acting on staircase? Explain.

Dead load :

Self weight of stair slab which includes the weight of slab, tread-rise, etc. Self-weight of finishes (0.5 to 1kg/m²)

Live loads :

- IS 875 part II specifies the load to be considered as UDL of intensity 5kN/m² for public buildings & 3kN/m² for residential buildings.

- Where the specified floor do not exceed 2 kN/m² & the staircases are should not liable for overcrowding.

27. Explain structural behaviour of staircases.

staircase can be grouped depending upon the support conditions & the direction of major bending of the slab component under the following categories.

(i) staircase slab spanning horizontally (along the slope line)

(ii) Staircase slab spanning transversely (slab width wise with central or side supports)

Singly Reinforced beams :-

Width = 200mm = b
Overall depth = 450mm = D
Effective depth d = 410mm

There are three bars
each of 20mm diameter
(ϕ = bar diameter in mm)

(i) force of compression = $0.36 f_{ck} b x$

$$= 0.36 \times 15 \times 200 \times x = 1080 x \ N$$

Force of tension = $0.87 f_y A_t$

$$= 0.87 \times 250 \times \left(3 \times \frac{\pi}{4} \times 20^2\right) = 204900 N$$

$\Rightarrow 1080 x = 204900$

$\Rightarrow x = 190mm$

$$x_m = 0.53 d \quad (for\ f_y = 250)$$
$$= 0.53 \times 400$$
$$= 212 mm > 190mm \ (o.k.)$$

$\therefore$ Depth of neutral axis = 190mm

(ii) Force of compression = $0.36 f_{ck} b x$

$$= 0.36 \times 20 \times 200 \times x$$
$$= 1440 x \ N$$

Force of tension = $0.87 f_y A_t$

$$= 0.87 \times 415 \times \left(3 \times \frac{\pi}{4} \times 20^2\right)$$

$$= 340109 N$$

$1410x = 340100$

$\Rightarrow x = 256\,mm$

$$x_m = 0.48d \quad (\text{for } f_y = 415\,\text{N/mm}^2)$$

$$= 0.48 \times 400$$

$$= 192\,mm \quad < \quad 256\,mm$$

$\therefore$ It is an over reinforced section

$\therefore$ Depth of N.A. $= 192\,mm$

② Determine the lever arm for section shown in fig. if effective cover shown for $b = 250\,mm$ and $f_y = 250\,\text{N/mm}^2$

$= 0.36 \frac{f_{ck}}{} \times 250 \times \qquad f_y = 415\,\text{N/mm}^2$

Lever arm $\boxed{z = d - 0.42x}$

3 bars each of 16mm diameter

(i) Force of compression

$$C = 0.36\, f_{ck}\, b\, x$$

$$= 0.36 \times 20 \times 250 \times x$$

$$= 1800x\ N$$

Force of tension

$$T = 0.87\, f_y\, A_t$$

$$= 0.87 \times 250 \times \left(3 \times \frac{\pi}{4} \times 16^2\right)$$

$$= 131180\ N$$

$1800x = 131180\ N$

$\Rightarrow x = 72.9\,mm$

$$x_m = 0.53d \qquad \text{for } f_y = 250$$

$$= 0.53 \times 300$$

$$= 159\,mm > 72.9\,mm \quad (o\cdot k)$$

$\therefore$ Depth of N.A. $= 72.9\,mm$

lever arm $= 300 - 0.42 \times 72.9 = 269.4\,mm$

(i) Force of compression $C = 0.36 f_{ck} b. x$

$$= 0.36 \times 25 \times 250 x$$
$$= 2250 x \ N$$

Force of Tension $T = 0.87 f_y A_{st}$

$$= 0.87 \times 415 \times \left(3 \times \frac{\pi}{4} \times 16^2\right)$$
$$= 217700 \ N$$

$2250 x = 217700$

$\Rightarrow x = 96.8 \ mm$

$$x_m = 0.48 \ d \qquad for \ f_y = 415$$
$$= 0.48 \times 360$$
$$= 172.8 \ mm > 96.8 \ mm \ (O.K.)$$

∴ Depth of neutral axis $= 96.8 \ mm$

lever arm $= 360 - 0.42 \times 96.8 = 319.35 \ mm$

(2) Determine the moment of resistance for the section shown in fig.

(i) Force of compression

$$C = 0.36 f_{ck} b. x$$
$$= 0.36 \times 25 \times 200 x$$
$$= 1800 x \ N$$

Force of tension

$$T = 0.87 f_y A_{st}$$
$$= 0.87 \times 415 \times \left(3 \times \frac{\pi}{4} \times 12^2\right)$$
$$= 122400 \ N$$

$C = T$

$\Rightarrow 1900x = 128470$

$\Rightarrow x = 68mm$

$$x_u = 0.48d \quad (\text{for } f_y = 415)$$
$$= 0.48 \times 310$$
$$= 148.8 \text{ mm} > 68 \text{ mm} \quad (O.K.) \quad U.R$$

$\therefore$ Depth of NA = 68mm

Lever arm $= 310 - 0.42 \times 68$

$z = 281 mm$

Since this is an under-reinforced section, moment of resistance is governed by steel.

Moment of resistance with respect to steel
$= $ tensile force $\times z$

$$M_u = 0.87 f_y A_{st} z$$

$$= 0.87 \times 415 \times \left(3 \times \frac{\pi}{4} \times 12^2\right) \times 281 \text{ N·mm}$$

$$= 34.4 \text{ kN·m}$$

(ii) Force of compression $= 0.36 f_{ck} b x$
$$= 0.36 \times 20 \times 250 x$$
$$= 1800x \text{ N}$$

$$A_{st} = 3 \times \left(\frac{\pi}{4} \times 12^2\right) = 312 mm^2$$

Force of tension $T = 0.87 f_y A_{st}$
$$= 0.87 \times 300 \times 3 \times 415 = 149470 \text{ N} ✓$$

(iii) $C = T$

$\Rightarrow 1800x = 149470$

$\Rightarrow x = 82mm$

$$x_m = 0.48d \quad (\text{for } f_y = 500 \text{ N/mm}^2)$$
$$= 0.46 \times 310$$
$$= 142.6 \text{ mm} > 82 \text{ mm} \quad (O.K.)$$

This is an under-reinforced section

Depth of NA = 82mm

Lever arm $= 310 - 0.42 \times 82 = 275.6 mm$

Moment of resistance w.r.t steel.

$$M = 0.87 f_y A_t z$$
$$= 0.87 \times 8.50 \times 5 \times 113 \times 295.66 mm$$
$$= 40.85 \text{ kN-m}$$

The beam will be designed so that under the applied moment both materials reach their maximum stresses. Let us assume ratio of overall depth to breadth of the beam equal to 1.5

For a balanced design,

      Factored BM = Moment of resistance w.r.t concrete
                = Moment of resistance w.r.t steel
                = Load factors × BM
                = 1.5 × 75
                = 112.5 kN-m

i) Moment of resistance $M_u = 0.36 f_{ck} b x_m (d - 0.42 x_m)$
      For Fe415 steel, $x_m = 0.48 d$

$$M_u = 0.36 f_{ck} b (0.48 d) \{ d - (0.42 \times 0.48 d) \}$$

$$= 0.36 \times 25 \times b \times 0.48 \times d^2 (1 - 0.42 \times 0.48)$$

$$= 3.45 \, b d^2$$

Since $\boxed{D/b = 1.5}$ $\Rightarrow \dfrac{d}{b} \approx 1.5$ or,
$$b = \dfrac{d}{1.5}$$

$$M_u = 2.76 \, bd^2 = 2.76 \times \left(\dfrac{d}{1.5}\right) d^2 = 2.5 \, d^3$$

$$2.3 \, d^3 = 112.5 \text{ kN·m}$$

$$\Rightarrow \quad 2.5 \, d^3 = 112.5 \times 10^6 \text{ N·mm}$$

$$\Rightarrow \quad d = 385.7 \text{ mm}$$

Adopt $D = 400 \text{ mm}$ and $b = \dfrac{400}{1.5} = 267 \text{ mm} \approx 300 \text{ mm}$ (say)

Effective cover = 35 mm (say)

$$d = 400 - 35 = 365 \text{ mm}$$

Area of tensile steel $A_t = \dfrac{\text{factored B.M}}{0.87 \, f_y \left(d - 0.42 x_m\right)}$

$$= \dfrac{112.5 \times 10^6}{0.87 \times 415 \times \left(d - \dfrac{0.42 \times 0.48}{0.48 \, d}\right)}$$

$$= \dfrac{112.5 \times 10^6}{0.87 \times 415 \times \left(1 - 0.7034 \times \dfrac{1}{0.48}\right) \times 365}$$

$$= 1070 \text{ mm}^2$$

$$= 10.7 \text{ cm}^2$$

Min^m area of steel $A_s = \dfrac{0.85 \, bd}{f_y}$

$$= \dfrac{0.85 \times 300 \times 365}{415}$$

$$= 224 \text{ mm}^2 < 1070 \text{ mm}^2 \text{ (O.K)}$$

In beams, the diameter of main reinforcing bars is usually selected b/n 12mm & 22mm. Provide 2–25mm and 1–12mm bars giving total area.

$$= 2 \times 490 + 113$$

$$= 1093 \text{ mm}^2 > 1070 \text{ mm}^2 \quad (O.K.)$$

The bars are placed symmetrically in the beam. It means 1–12mm bars will be placed in the middle.

Reinforcement details :
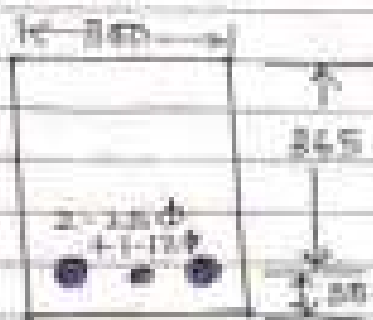
(ii) Moment of resistance $M_u = 0.36 \, f_{ck} \cdot b \cdot x_m (d - 0.42 x_m)$
for Fe 500 steel.

$$x_m = 0.448 \, d$$

$$M_u = 0.36 \times 25 \times b \times 0.448 \, d \, (d - 0.42 \times 0.448 \, d)$$
$$= 3.25 \, b \, d^2$$

$$M_u = 3.25 \left(\frac{d}{1.5}\right) d^2 = 2.167 \, d^3$$

$$\therefore 2.167 \, d^3 = 112.5 \times 10^6 \text{ N.mm}$$
$$\Rightarrow d = 373 \text{ mm}$$

Adopt : $D = 410$ mm & $b = 275$ mm $\left(\frac{410}{1.5} = 273.33\right)$
$\therefore$ effective cover = 35 mm
effective depth = 375 mm

Area of tensile steel $A_t = \dfrac{\text{factored BM}}{0.87 \, f_y (d - 0.42 x_m)}$

$$= \frac{112.5 \times 10^6}{0.87 \times 500 \times (373 - 0.42 \times 373)}$$

$$= \frac{112.5 \times 10^6}{0.87 \times 500 \times \left(373 - 0.1750044 \times \frac{373}{478}\right)}$$

$$= 773.21 \text{ mm}^2$$
$$= 7.75 \text{ cm}^2$$

Min^m area of steel $A_s = \dfrac{0.85\,bd}{6y}$

$$= \dfrac{0.85 \times 300 \times 575}{580}$$

$$= 172.94\ mm^2 < 773.21\ mm^2$$

(o.k.)

In beams, the diameter of main reinforcing bars is usually selected between 12 mm – 32 mm.

Provide 3 – 20 mm bars, giving total area = 942 mm^2

$> 773.21\ mm^2$ (o.k)

Reinforcement details :-

445 mm

3 – 20 φ

1 35
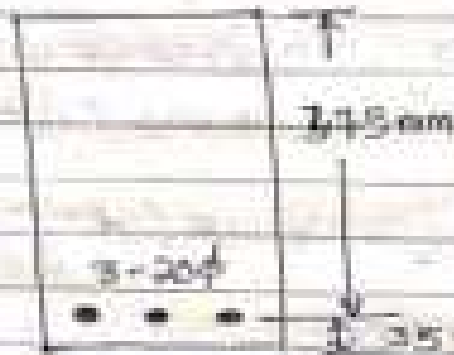
DOUBLY REINFORCED SECTION

Find the moment of resistance of a beam section 25cm × 50cm deep if it is reinforced with 2-20mm bars in compression & tension, each at an effective cover of 50mm. Use M20 mix & Fe415 grade steel.

(A) First Trial :

250mm

2-20φ 500mm

Let, depth of NA $x = x_m = 0.48d$ (for Fe415)

$$= 0.48 \times 450$$

$$= 216\ mm$$

$d' = 50\ mm$

Strain at the level of compression steel $= 0.0035 \left(\dfrac{x_m - d'}{x_m}\right)$

$$= \dfrac{0.0035 (216 - 50)}{216} = 0.00269$$

Stress in compression steel = 350 N/mm² (Table 4.6)

$f_{sc} = \left[351.8 - 342.8 \left(\dfrac{0.00276 - 0.00267}{0.00276 - 0.00241}\right)\right]$

(Table — Points on design stress-strain curve for HYSD grade bars)

Strain	Stress
0.00241	342.8 N/mm²
0.00267	? (350)
0.00276	351.8 N/mm²

Total force of compression $C = C_1 + C_2$

(i) Force of compression in concrete

$C_1 = 0.36 f_{ck} \, b \cdot x$

$= 0.36 \times 25 \times 280 \times 216$

$= 388800$ N

(ii) Force of compression in steel

$C_2 = (\sigma_{sc} - \sigma_{cc}) A_{sc}$ (Ages 20 mm in concrete)

$= (350 - 0.446 f_{ck}) \times A_{sc}$

$= (350 - 0.446 \times 25) \times 628$

$= 214192$ N

$C = 388800 + 214192$

$= 605000$ N

Force of tension $T = 0.87 f_y A_{st}$

$= 0.87 \times 415 \times 628 = 226730$ N

$<$ Force of compression

Hence, it is an under-reinforced section.

Depth of neutral axis should be reduced to equalise force of compression with force of tension.

(B) Second Trial.

Let, depth of N/A = 67.5 mm

Strain at the level of compression steel

$$\varepsilon_{sc} = 0.0035\left(\frac{x_m - d'}{x_m}\right)$$

$$= 0.0035\left(\frac{67.5 - 50}{67.5}\right)$$

$$= 0.00091$$

Stress in compression steel (Table 4)

σ_{sc}

strain	stress N/mm^2
—	—
0.00091	?
0.00144	288.7

$$= \frac{0.00091}{0.00144} \times 288.7$$

$$= 181.9 \text{ N/mm}^2$$

Force of compression C $\begin{cases} C_1 = 0.36 f_{ck} \cdot b \cdot x \\ \quad = 0.36 \times 20 \times 250 \times 67.5 \\ \quad = 121500 \text{ N} \\ C_2 = (\sigma_{sc} - \sigma_{cc}) A_{sc} \\ \quad = (181.9 - 0.446 \times 20) 628 \\ \quad = 108600 \text{ N} \end{cases}$

$$C = C_1 + C_2 = 121500 + 108600$$
$$= 230100$$

Force of tension $T = 0.87 f_y A_t$
$$= 0.87 \times 415 \times 628$$
$$= 226700 \text{ N}$$

$C \approx T$ O.K Hence depth of NA 67.5

* For more accuracy the depth of NA can be further reduced

Moment of resistance $M_u = C_1(d - 0.42x_u) + C_2(d - d')$

$$= 181500 \times (450 - 0.42 \times 67.3)$$
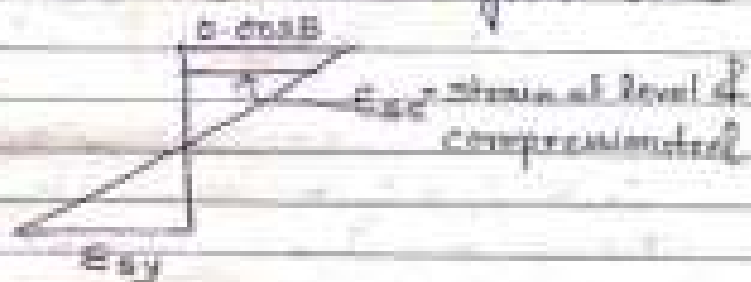$$+ 188600 \times 400$$
$$= 94.67 \times 10^6 \, N\cdot mm$$
$$= 94.67 \, kN\cdot m$$

(8) Find the moment of resistance of a beam 25cm × 50cm deep if it is reinforced with 2-12mm bars in compression zone & 2-20 mm bars in tension zone, each at an effective cover of 40mm as shown in fig. Assume (i) M20 mix & Fe415 grade steel. (ii) M25 mix & Fe500 grade steel.
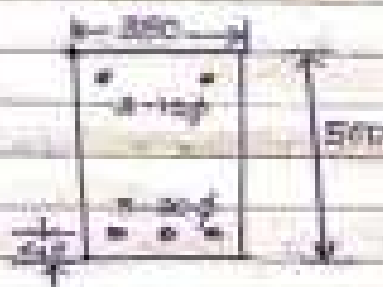
(Section)   (Strain)

(i) For M20 mix & Fe415 steel

The depth of NA is unknown

First trial:
Let, depth of NA $x = x_u = 0.48d$, d=460m
$$= 0.48 \times 460$$
$$= 221 \, mm$$

$\varepsilon_{sc} = 0.0035 \left(\dfrac{x - d'}{x} \right) = 0.0035 \left(\dfrac{221 - 40}{221} \right)$

$= 0.00286$

$\sigma_{sc} = ?$ For Fe415 grade steel

0.00276	351.8 N/mm²
0.00286	?
0.00380	360.9 N/mm²

$$\sigma_{sc} = 351.8 + (361.9 - 351.8)\left[\frac{0.00286 - 0.00276}{0.00380 - 0.00276}\right]$$

$$= 352.7 \ \text{N/mm}^2$$

$(2-12\phi) \quad A_{sc} = 2 \times 1.13 = 2.26 \ cm^2 = 226 \ mm^2$

Total force of compression $C = C_1 + C_2$

$$C_1 = 0.36 \sigma_{ck}^{\prime} \cdot b \cdot x$$
$$= 0.36 \times 20 \times 250 \times 221$$
$$= 397500 \ N$$

$$C_2 = (\sigma_{sc} - \sigma_{cc}) A_{sc}$$
$$= (352.7 - 0.446 \sigma_{ck}) \times 226$$
$$= (352.7 - 0.446 \times 20) \times 226 = 77691 \ N$$

$C = C_1 + C_2 = 475490 \ N$

Force of tension $T = 0.87 \sigma_y A_t^{3-20\phi}$

$$= 0.87 \times 415 \times 3 \times 314$$
$$= 340110 \ N < \text{force of compression}$$

Hence reduce the depth of NA so that force of tension is equal to force of compression.

Second Trial :

Let depth of NA = 150mm & $d' = 40$

$$\varepsilon_{sc} = 0.0035\left(\frac{150 - 40}{150}\right)$$

$$= 0.00257$$

The corresponding stress in steel is
$$\sigma_{sc} = 346.9 \ MPa$$

For 415 grade steel ε_{sc} | σ_{sc}
$$0.00241 \longrightarrow 342.8 \ \text{N/mm}^2$$
$$0.00257 \longrightarrow \ ?$$
$$0.00276 \longrightarrow 351.8 \ \text{N/}$$

$$C = C_1 + C_2 \qquad C_1 = 0.36\sigma_{ck}\,b\,x_u = 270780\,N$$

$$= 346380\,N \qquad C_2 = (\sigma_{sc} - \sigma_{cc})A_{sc}$$

$$\therefore \text{force of} \qquad = (\sigma_{sc} - 0.446\,\sigma_{ck})n_{sc}$$

$$\text{tension 346380} \qquad = 76280\,N$$

$\therefore$ Moment of resistance $= C_1(d - 0.42x_u) + C_2(d - d')$

$$= 139.2 \times 10^6\,N\text{mm}$$

$$= 139.2\,kN\text{-}m$$

If the load factor is 1.5

$$\text{Safe working moment} = \frac{139.2}{1.5} = 92.8\,kN\text{-}m$$

(ii) For M25 mix & Fe 500 grade steel :

First Trial :

Let, depth of NA $x = x_m = 0.46d$

$$= 0.46 \times 460$$

$$= 212\,mm \quad \&$$

$d' = 40mm$

Strain at the level of compression steel

$$\varepsilon_{sc} = 0.0035\left(\frac{x_m - d'}{x_m}\right)$$

$$= 0.0035\left(\frac{212 - 40}{212}\right)$$

$$= 0.00283$$

Corresponding stress in compression steel can be obtained by interpolation

For 500 grade Steel,

0.00277	$\longrightarrow$	347.8 N/mm²
0.00283	$\longrightarrow$	?
0.00380	$\longrightarrow$	369.9 N/mm²

$$\sigma_{sc} = 413 + (423.9 - 48)\left[\frac{0.00283 - 0.00277}{0.00312 - 0.00277}\right]$$

$$= 415.1\,N/mm^2$$

Total force of compression $C = C_1 + C_2$

$$C_1 = 0.36 \, f_{ck} \, b \, x$$
$$= 0.36 \times 25 \times 250 \times 212$$
$$= 477000 \, N$$

$$C_2 = (f_{sc} - f_{cc}) \, A_{sc}$$

$$= (415.1 - 0.446 \times 25) \times 226$$
$$= 91290 \, N$$

$$C = C_1 + C_2 = 568290 \, N$$

Force of Tension $T = 0.87 \, f_y \, A_t$

$$= 0.87 \times 500 \times 3 \times 314$$
$$= 409770 \, N \quad < \text{force of compression}$$

$\Rightarrow$ Hence the depth of NA is to be reduced.

Second Trial: $C_1 + C_2$

$$= C_1 + 91290 = 0.36 \times 25 \times 250x = 91290$$

$$\boxed{C_1 + C_2 = T}$$

$$\Rightarrow 0.36 \times 25 \times 250x + 91290 = 409770$$

$$\rightarrow x = 141.8 \, mm \quad (\text{take})$$

$$f_{sc} = 0.0035 \left(\frac{141.5 - 40}{141.5} \right)$$

$$= 0.00251$$

$$f_{sc} = ?$$

From Sub grade

$$0.00226 \longrightarrow 391.3 \, N/mm^2$$
$$0.00251 \longrightarrow ?$$
$$0.00277 \longrightarrow 413 \, N/mm^2$$

$$f_{sc} = 391.3 + (413 - 391.3) \left(\frac{0.00251 - 0.00226}{0.00277 - 0.00226} \right)$$

$$= 402 \, N/mm^2$$

Force of compression $C = C_1 + C_2 = 405.3$ tons [?]

$C_1 = 0.36 f_{ck} \cdot b \cdot x$
$= 0.36 \times 25 \times 200 \times 111.5$

$C_2 = (f_{sc} - f_{cc}) A_{sc}$
$= (402 - 0.446 \times 25) 26$

Thus moment of resistance
$= C_1 (d - 0.42x) + C_2 (d - d')$
$= C_1 (460 - 0.42 \times 111.5) + C_2 (460 - 4)$
$= 184.6$ kN-m

③ Design a rectangular beam for an effective span of 6m. The superimposed load is 60 kN/m & size of the beam is limited to 30cm × 60 cm overall. Use M20 mix & Fe415 grade steel.

Self weight of beam = volm × density
$= (0.50 \times 0.60) \times 25$ kN/m³
$= 4.5$ kN/m

Super imposed load $= 60$ kN/m
∴ Total load $= 64.5$ kN/m
Factored load $= 1.5 \times 64.5$
$= 96.75$ kN/m

Max° factored BM $= \dfrac{wl^2}{8}$

$= \dfrac{96.75 \times 6^2}{8}$

$= 435.37$ kN-m

Let, effective cover $= 0.1D$
$= 0.1 \times 600$
$= 60$ mm
$d = D - d' = 600 - 60 = 540$ mm

For Fe415 $x_m = 0.48 d$
$= 0.48 \times 540$
$= 260$ mm

Limiting bending moment (from table) for Fe415 grade steel $M_{lim} = 0.138 f_{ck} b d^2$

$$= 0.138 \times 20 \times 300 \times 540^2$$
$$= 241.5 \text{ kN.m}$$

(a) Area of tension steel corresponding to this moment :

$$0.87 f_y A_{t1} = 0.36 f_{ck} b x_m$$

$$\Rightarrow 0.87 \times 415 \times A_{t1} = 0.36 \times 20 \times 300 \times 260$$

$$\Rightarrow A_{t1} = 1556 \text{ mm}^2$$

(b) The remaining bending moment has to be resisted by a couple consisting of compression steel & the corresponding tension steel

From table 5.1 : if $d'/d = 0.1$, $f_{sc} = 355 \text{ N/mm}^2$
f_{sc} in compression reinforcement

$$\Rightarrow \frac{d'}{d} = 0.1$$

$$\Rightarrow d' = 0.1 \times 540 = 54 \text{ mm}$$

$$M - M_{lim} = (f_{sc} A_{sc} - f_{cc} A_{sc})(d - d')$$
$$\text{i.e } f_{sc}(d'-d)$$

$$\Rightarrow A_{sc} = \frac{(435.37 - 241.5) 5.10^6}{(355 - 0.446 \times 20)(540 - 56)}$$

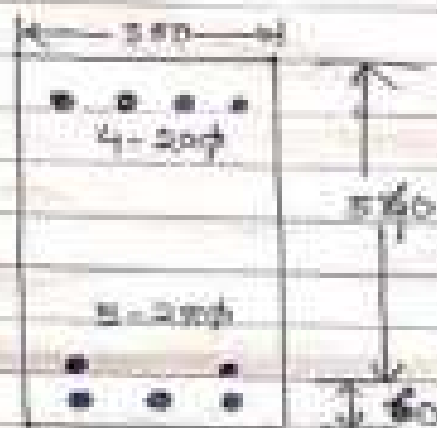$$= 1160 \text{ mm}^2$$

Corresponding tension steel $A_{t2} =$

$$0.87 f_y A_{t2} = f_{sc} A_{sc}$$

$$\Rightarrow 0.87 \times 415 \times A_{t2} = 355 \times 1160$$
$$\Rightarrow A_{t2} = 1140 \text{ mm}^2$$

Total tension steel $A_t = A_{t_1} + A_{t_2}$

$$= 1556 + 1134$$
$$= 2690 \, mm^2$$

$$\& \; A_{sc} = 1160 \, mm^2$$

Provide. 5 - 28 ϕ in tension $(A_t = 3078 \, mm^2 > 2690$

4 - 20 ϕ in compression $(A_t = 1256 \, mm^2 > 1160$

4 - 20ϕ

5 - 28ϕ

350

560

60

Check. Max^m tension steel $= 0.04 \, bD$

$$= 0.04 \times 350 \times 600$$
$$= 7800 \, mm^2$$
$$> 3078 \, mm^2 \; (O.K.)$$

Flanged Beams (L & T)

- Reinforced concrete slabs used in floors, roofs and decks are mostly cast monolithic from the bottom of the beam to the top of slab
- Such rectangular beams having slab on top are different from others having no slab (like in elevated tanks, lintels etc) or having disconnected slabs as in some pre cast system
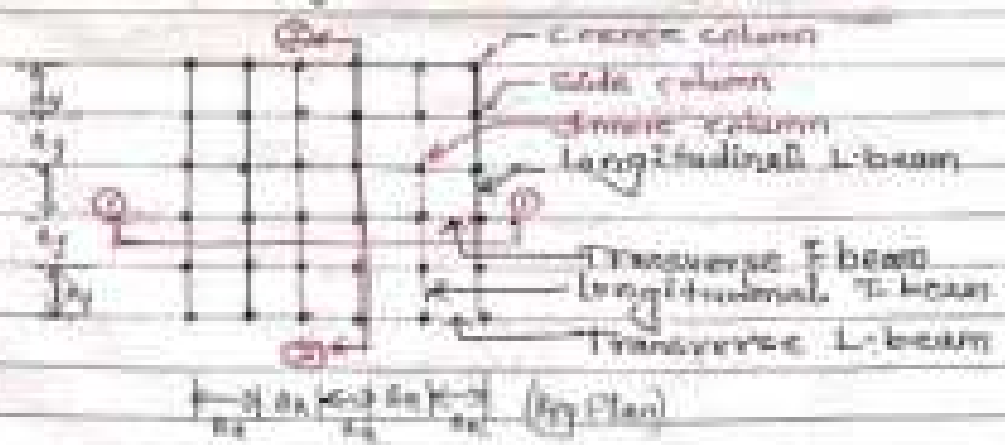

Precast slab
rectangular beam

- Due to monolithic casting, beams and a part of slab act together
- Under the action of positive bmg i.e between the supports of a continuous beam, the slab, upto a certain width greater than the width of the beam, forms the top part of the beam
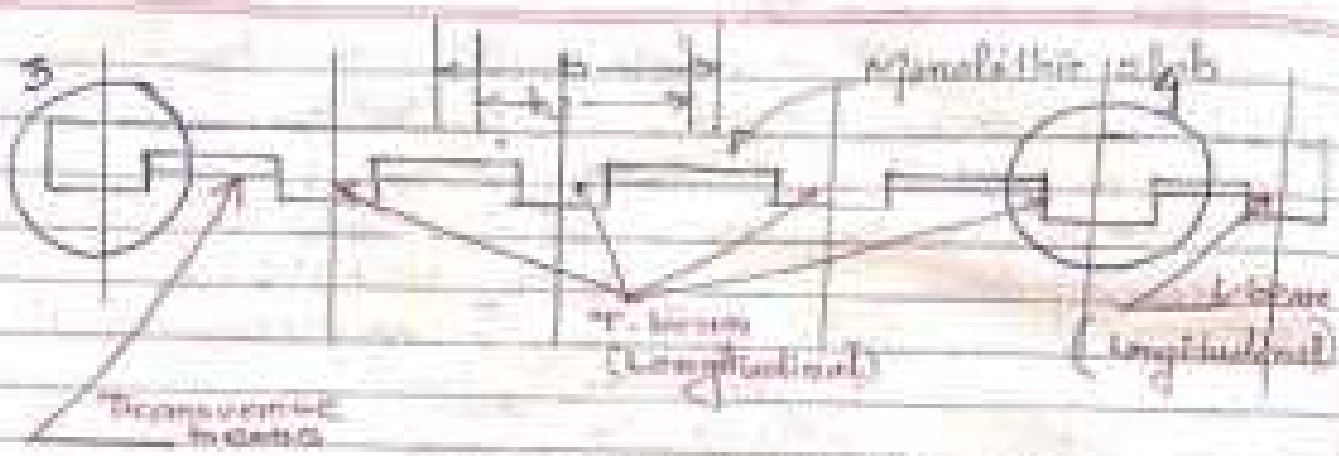- Such beams having slab on top of the rectangular rib are designated as the _flanged beam_ either T or L type depending on whether the slab is on both sides or on one side of the beam.
- Over the supports of a continuous beam, the bending moment is negative and the slab, therefore, is in tension while a part of the rectangular beam (rib) is in compression
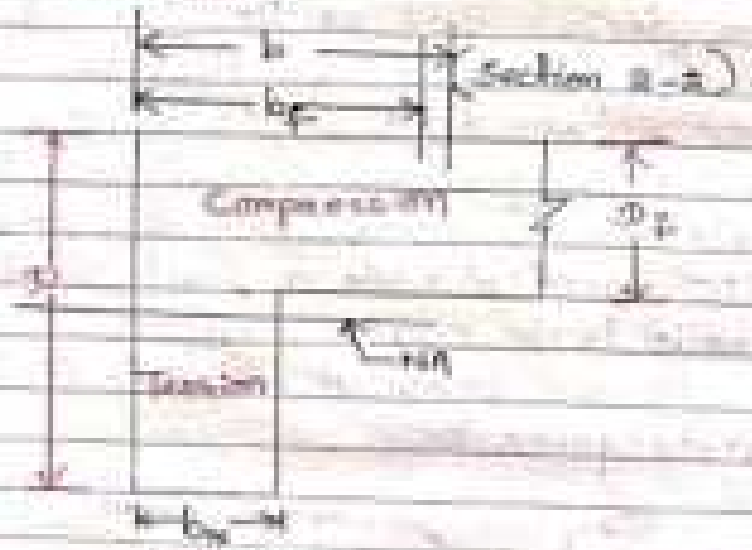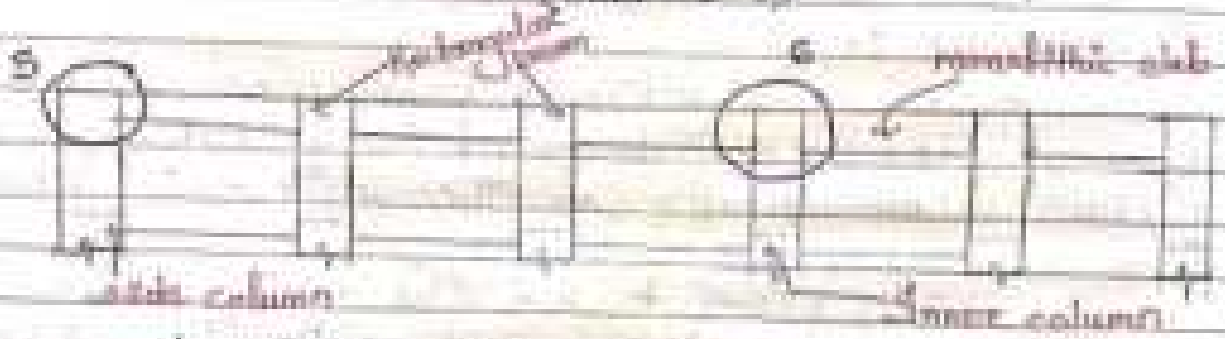- The continuous beam at support is thus equivalent to rectangular beam

corner column
side column
inner column
longitudinal L-beam
Transverse T beam
longitudinal T beam
Transverse L-beam
(Plan)

- CL of beam & column
$a_x = c/c$ distance of longitudinal beam
$a_y = c/c$ " " transverse beam

5

Monolithic slab

T-beam
(Longitudinal)

(Longitudinal)

Transverse
beam

b = actual width of slab
b_f = effective width of flange

(Sec^n A-A)

5   Rectangular beam   6   monolithic slab

side column   inner column

b_f

b_f

(Section B-B)

Compression

D_f

NA

Tension

b_w

Notations:

b = Actual width of flange
b_f = Effective width of flange
b_w = width of web
D_f = Depth of flange
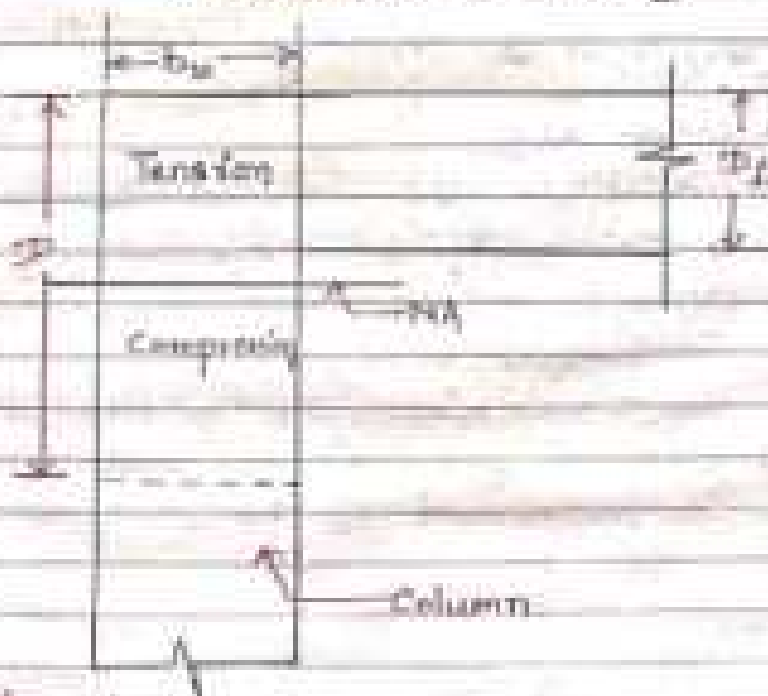NA = neutral axis

(Detail of a T-beam)

Notations :

b = actual width of flange.

b_f = Effective width of flange.

b_w = width of web

D_f = depth of flange

NA = neutral axis

(Detail of a T-beam)

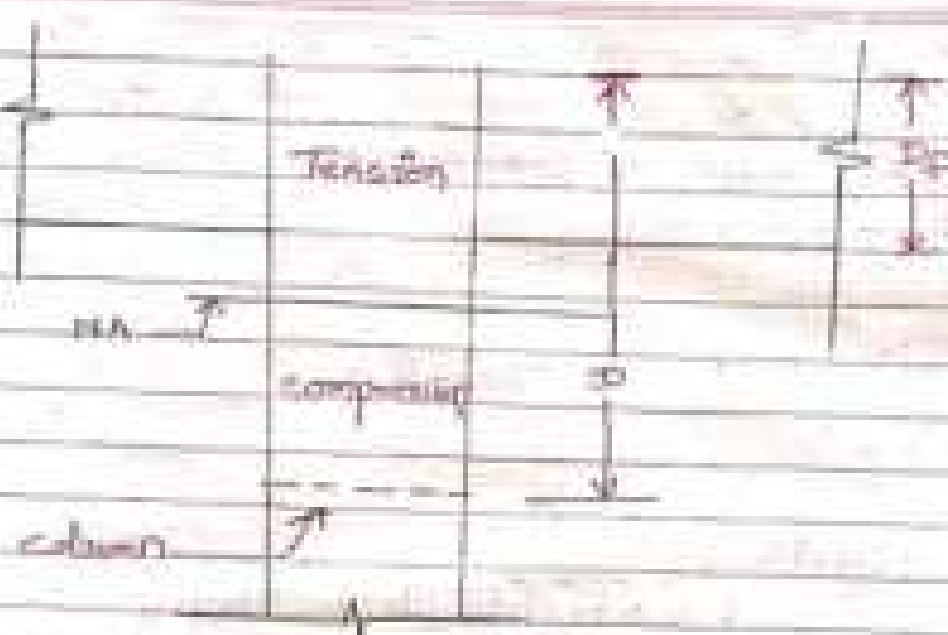
Notation

b_f = Effective width of flange

b_w = width of web

D_f = Depth of flange

NA = neutral axis

(Detail of a Rectangular beam)

Tension

NA

compression

column
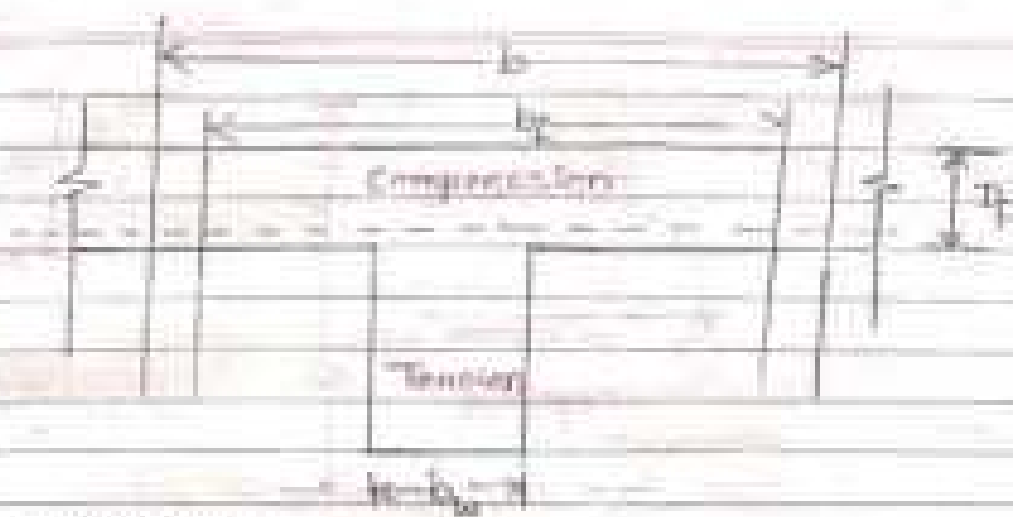
Notation.

b_f = Effective width of flange
b_w = width of web
D_f = Depth of flange
NA = neutral axis

(conduit at of rectangular beam)

- The actual width of the flange is the spacing of the beam which is the same as the distance b/w the middle points of the adjacent spans of the slab as shown in fig. a
- However In flanged beam, a part of the width less than the actual width, is effective to be considered as a part of the beam. This width of slab is designated as the effective width of flange
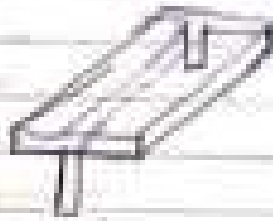
T-beam :-

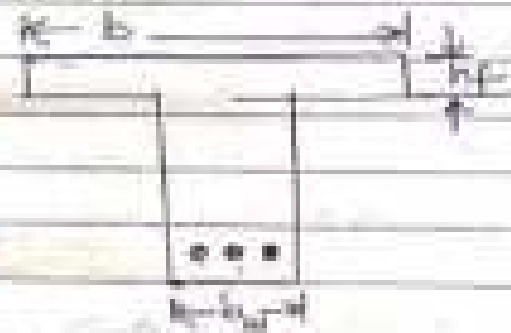Concrete beams are often casted integrally with the slab and formed T-shaped beam.

- Very effective

- Here slab portion carries the compressive load & web portion carries the tension

- The slab forms the beam flange, while the part of the beam projecting below the slab forms in what is called web or stem.

- effective flange width
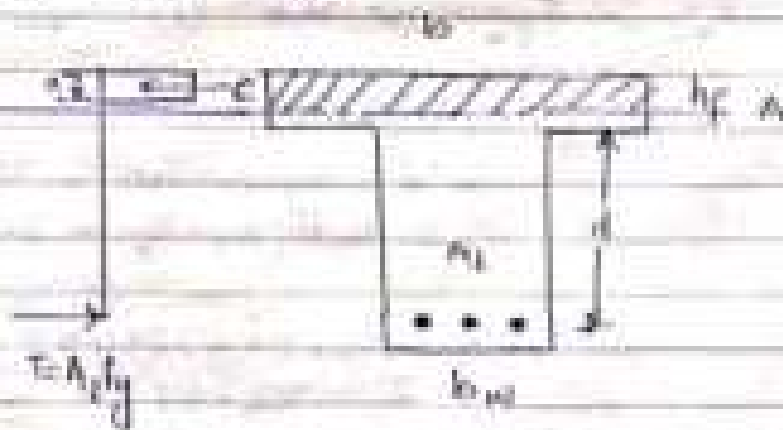- for isolated T-beam
  a) $b_{eff} \leq 4b_w$
 b) $h_f \geq b_w/2$

Strength Analysis

1st Case (NA is with in the flange)

- Analysis as a rectangular beam of width
$$b = b_{eff}$$
$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

Shear

$T = A_s f_y$

A tee beam of effective flange width 1200mm, thickness of ... 100mm, width of rib 300mm & effective depth of 560 mm is reinforced with 4 no. 25mm diameter bars. Calculate the factored moment of resistance. The materials are M20 grade concrete & HYSD reinforcement of grade Fe415.
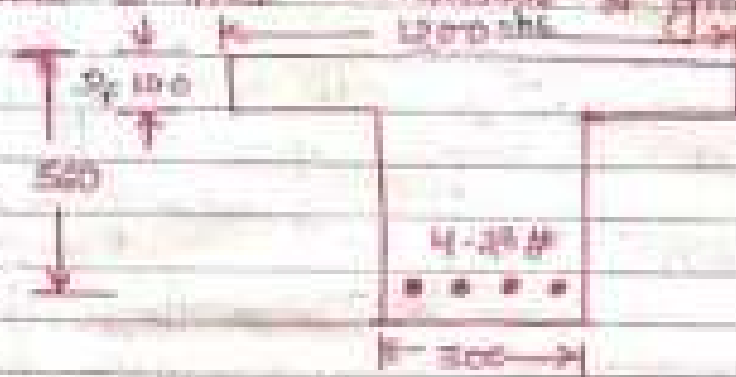
$A_{st} = 4 \times 491 = 1964\,mm^2$

To find whether the NA lies in the flange or the web, flange compression & tensile force are compared.

$$C = F_c = 0.36\,f_{ck}\,b_f\,D_f$$

$$= 0.36 \times 20 \times 1200 \times 100$$

$$= 864\,kN$$

$$T = F_t = 0.87\,f_y\,A_{st}$$

$$= 0.87 \times 415 \times 1964 \times 10^{-3}$$

$$= 709\,kN$$

$F_c > F_t \Rightarrow$ N.A. lies in flange

Equating the forces

Total compression = Total tension

$\Rightarrow 0.36\,f_{ck}\,b_f\,x_u = 0.87\,f_y\,A_{st}$

$$\Rightarrow 0.36 \times 20 \times 1200 \times x_u = 0.87 \times 415 \times 1964$$

$$\Rightarrow x_u = 82.07\,mm < 100\,mm \quad (ok)$$

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

$$= 0.48 \times 560 = 268.8\,mm$$

$x_u < x_{u,max} \Rightarrow$ section is under-reinforced

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

$$= 0.87 \times 415 \times 1964 (550 - 0.42 \times 82.07) \times 10^{-6}$$

$$= 372.65 \text{ kN.m}$$

$$Or,$$

$$M_u = 0.36 f_{ck} b_f x_u (d - 0.42 x_u)$$

$$= 0.36 \times 20 \times 1240 \times 82.07 (550 - 0.42 \times 82.07) \times 10^{-6}$$

$$= 372.65 \text{ kN.m}$$

NA in Web

(*) Find the ultimate moment of resistance for the section above if it is reinforced with five 25mm diameter bars.

$$A_{st} = 5 \times (491) = 2455 \text{ mm}^2$$

To find the position of NA

$$C = F_c = 0.36 f_{ck} b_f D_f$$

$$= 0.36 \times 20 \times 1240 \times 100 \times 10^{-3}$$

$$= 804 \text{ kN}$$

$$T = F_t = 0.87 f_y A_{st} = 0.87 \times 415 \times 2455 \times 10^{-3}$$
$$= 886.4 \text{ kN}$$

$C < T \Rightarrow$ NA lies in web (out-of-syllabus)

(*) Calculate the moment of resistance of a T beam assuming M20 mix & Fe415 grade steel

Let us assume that it is an under-reinforced section & the NA lies within the flange (case 1)
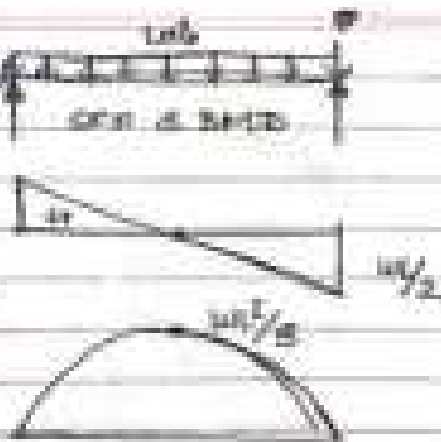
$$0.36 \, f_{ck} \, b_f \, x = 0.87 \, f_y \, A_{st}$$

$$\Rightarrow x = \frac{0.87 \times 415 \times 3600}{0.36 \times 2n \times 750} = 2.81 \, mm > D_f$$

As the value of x is more than 120 mm, NA lies in web.

- As the concrete is weak in shear & it supports the shear force is maximum as there is a chance of sliding or shear failure.

- So steel is provided to bear this shear force to prevent shear failure or slide in vertical direction

- Shear reinforcement is to provide the resistance against shear forces to which a beam is subjected to
- It is usually in the form of stirrups which also cover the purpose of holding the main tensile & compression reinforcement in place
  (i) Shear reinforcement is provided to hold longitudinal reinforcement
  (ii) To resist diagonal crack

Types of shear reinforcement
   1. Vertical stirrups
   2. Bent up bars along with stirrups
   3. Inclined stirrups.

Vertical stirrups

S_v is spacing stirrups

Bent up bar

Inclined stirrups
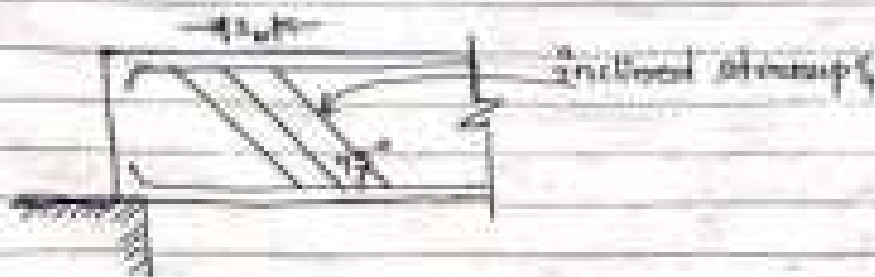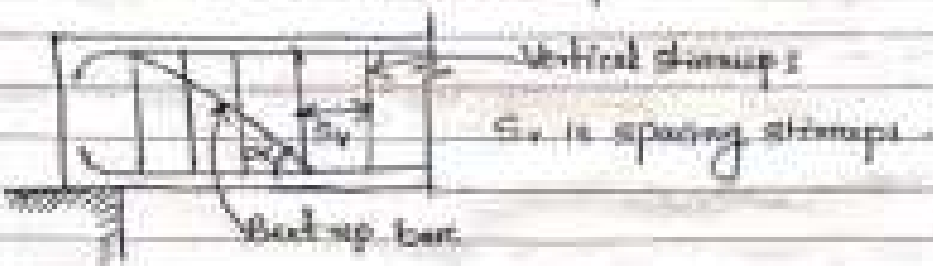
Bond

- The most important assumption made in the theory of RCC is that there is a perfect bond between the concrete & steel.

- Perfect bond in the sense the bond should act in such a way that there is no slip b/w steel & concrete.

- This bond helps in transferring force from steel to concrete & concrete to steel.

Bond stress :

- The stress which is acting on the outer interface of steel to the surrounding concrete is called bond stress.

- This stress helps in keeping bond between reinforcement & concrete together.

- Bond stress resists any force that tries to pull out the rods from the concrete.

- When you try to pull out the reinforcement bar from hardened concrete, then this bond stress resists the bar to come out.

- Different grades of concrete has different bond stress.

- These bonds are classified into two types
  1. Anchorage bond (development length)
  2. Flexural bond or Local bond

Local Bond

- It is defined as the magnitude of the bond stress at any point on the structural element between the reinforcement & the concrete.

- The value will vary depending on the variation of bending moment along the section of the element.

Let, dx = distance over the length of RCC beam.

T = Increase in tension

$$T = \frac{dM}{jd} \qquad ①$$

u = local bond stress

Σo = Perimeter of steel

$$T = u^* \Sigma o^* dx \qquad ②$$

Equating,

$$u^*(\Sigma o)^* dx = \frac{dM}{jd}$$

$$u = \frac{\left(\frac{dM}{dx}\right)}{\Sigma o \cdot jd} = \frac{V}{\Sigma o \cdot jd}$$

$$\text{Local Bond stress} = \frac{V}{\Sigma o \cdot jd}$$

Anchorage bond / D.L.

        — This bond is seen when a bar carrying certain force is removed (pull out test). In such cases, it is necessary to transfer this force in the bar to the surrounding concrete over a certain length.

        — This length of bar required to transfer the force in the bar to the surrounding concrete through bond is called development length (D.L.)

        — Reinforcing bar is embedded in concrete & subjected to a pull T.

        T = Design stress × area of bar
$$= 0.87 f_y \times \frac{\pi}{4} d^2$$

        — This force must be transferred from Steel to concrete through bond acting over the interface (perimeter) of the bar over a length (D.L)

        — If τ_{bd} = Avg. Design bond stress
then, ultimate bond force = Pull out force

$$\Rightarrow \tau_{bd} (\pi d) D.L = 0.87 f_y \frac{\pi}{4} d^2$$

$$\Rightarrow D.L = \frac{0.87 f_y d}{4 \tau_{bd}}$$

- Hence all the bars should extend to a distance of $2L$ beyond the section where they are required to take their design force.

- It is not possible to provide straight bars at all the times due to lack of space at supports. In such a scenarios we provide them as hooks & bends.

- The anchorage value (hook length) = $16d$
- The anchorage value (bend length) = $4d$ (45° angle)

Shear

Q. A RC beam has an effective depth of 500mm &
a breadth of 350mm. It contains 4-25mm bars.
If (i) $f_{ck} = 20 \text{ N/mm}^2$, $f_{sy} = 250 \text{ N/mm}^2$
(ii) $f_{ck} = 25 \text{ N/mm}^2$, $f_{sy} = 415 \text{ N/mm}^2$ (stirrup)

Calculate the shear reinforcement needed for
a factored shear force of 350KN.

% area of longitudinal steel $p = \dfrac{100 A_s}{b.d}$

$$= \dfrac{100 \times 3/4 \times 25^2 \times 4}{350 \times 550}$$

$$= 1.12\%$$

(a) Design shear stress of concrete
$$\tau_c = 0.64 \text{ N/mm}^2 \text{ (from table)}$$

Nominal shear stress $\tau_v = \dfrac{V_u}{bd}$

$$= \dfrac{350 \times 1000}{350 \times 550}$$

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

Maximum shear stress $\tau_{c,max} = 2.8 \text{ N/mm}^2$ (Table)

$$0.64 \text{ N/mm}^2 < 2 \text{ N/mm}^2 < 2.8 \text{ N/mm}^2$$

$\Rightarrow$ Provide shear reinforcement

(i) Using Fe250 grade steel

Net design shear strength $\Rightarrow V_{us} = V_u - \tau_c bd$

$$= (350 \times 10^3) - \left(0.64 \times \dfrac{}{350 \times 550}\right)$$

$$= 223000 \text{ N}$$

Adopt 8mm - 2 legged vertical stirrups

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.5 \, mm^2$$

Spacing of shear reinforcement x is given by

$$\boxed{x = \frac{0.875 \, A_{sv} \, d}{V_{us}}}$$

$$= \frac{0.87 \times 250 \times 100.5 \times 540}{248940}$$

$$= 48 mm$$

∴ The code requires that $x \not> 300mm$

$$\not> 0.75d = 0.75 \times 540 = 225mm$$

Min^m area of shear reinforcement

$$A_s \geq \frac{0.4b}{0.87 f_y} = \frac{0.4 \times 250 \times 48}{0.87 \times 250} = 30 mm^2$$

* Although all requirements of the code are satisfied by 8mm - 2 legged M.S bars @ 48mm c/c, but it is suggested that minimum spacing of stirrups be limited to 100mm in order to permit space for compaction of the concrete.

⇒ Revised area of stirrups

$$A_{sv} = \frac{V_{us} x}{0.87 \, f_y \, d}$$

$$= \frac{248940 \times 100}{0.87 \times 250 \times 540}$$

$$= 213 mm^2$$

$\frac{\pi}{4} \times 12^2 = 110$

Area of one leg = $\frac{\pi}{4} \times 12^2 = 110 mm^2$

[Use] 12mm - 2 legged vertical stirrups @ 100mm c/c

$$(A_{sv} = 220 mm^2)$$

Chk   Min^m shear reinforcement

$$A_v \geq \frac{0.4 \times 250 \times 150}{0.87 \times 260} = 65 \, mm^2$$

$$226 mm^2 > 65 mm^2 (0.$$

(ii) Using Fe415 grade steel

$$V_{us} = 238000 \, N$$

If minimum spacing is 100mm, area of shear reinforcement is given by

$$A_v \geq \frac{0.4 \, b \, x}{0.87 \, f_y}$$

$$= \frac{0.4 \times 250 \times 150}{0.87 \times 415}$$

$$= 58.7 \, mm^2$$

$$A_{sv} = \frac{V_{us} \, x}{0.87 \, f_y \, d}$$

$$= \frac{238000 \times 100}{0.87 \times 415 \times 550}$$

$$= 151.8 \, mm^2 > A_v$$

Area of one leg $= \frac{151.8}{2} = 66 \, mm^2$

 Use 10mm – 2 legged vertical stirrups @ 100mm c/c

Q2. The beams shown in figure are subjected to factored shear force of 210kN, $f_{ck} = 20 \, N/mm^2$ & $f_y = 415 \, N/mm^2$. Calculate the shear reinforcement
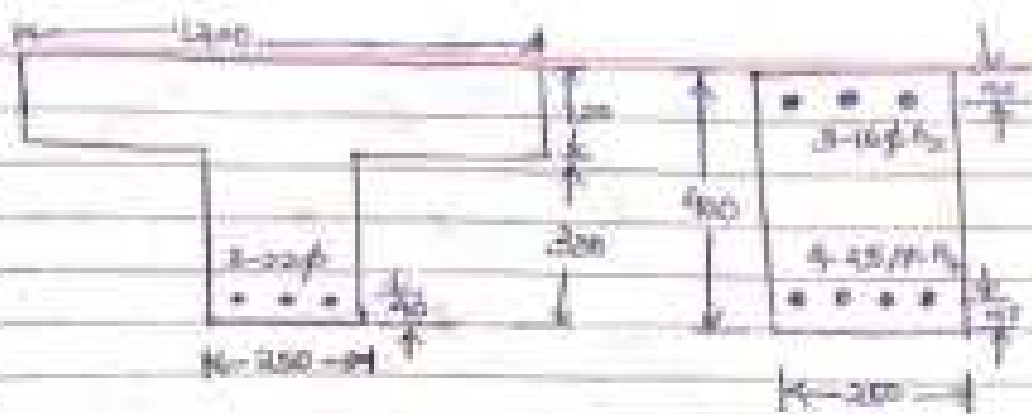
(b) T-beam

width of web $b_w = 250$ mm

Effective depth $d = 500 - 40 = 460$ mm

% tension steel $p = \dfrac{100 A_s}{b_w d}$

$$= \dfrac{100 \times 3 \times \frac{\pi}{4} \times 25^2}{250 \times 460}$$

$$= 0.99\%$$

→ Design shear stress $\tau_c = 0.62 \, N/mm^2$

Nominal shear stress $\tau_v = \dfrac{V_u}{b_w d}$

$$= \dfrac{200 \times 10^3}{250 \times 460}$$

$$= 1.74 \, N/mm^2$$

Net Design shear strength $V_{us} = V_u - \tau_c b d$

Net design shear stress $\tau_{us} = \dfrac{V_u}{b d} - \tau_c$

$$= 1.74 - 0.62$$

$$= 1.12 \, N/mm^2$$

Adopt 8mm 2 legged shear stirrups giving

$A_{sv} = 100.5 \, mm^2$

Spacing x of vertical shear stirrups

$$x = \dfrac{0.87 \, f_y \, A_{sv}}{\tau_{us} \, b_w}$$

$$= \frac{0.87 \times M15 \times 100.5}{M2 \times 250}$$

$$= 150 \, mm > 100mm \quad (O.K.)$$

$$\not> 0.75d \left(=0.75 \times 460 = 345\right)$$

Minimum shear reinforcement is given by

$$A_s > \frac{0.4 b \cdot x}{0.87 \, f_y}$$

$$= \frac{0.4 \times 250 \times 120}{0.87 \times 415}$$

$$= 33 mm^2 \quad < A_{sv} \quad (O.K.)$$

(b) Doubly Reinforced beam

width of section = 250 mm
Effective depth = 400 - 40 = 360 mm.
% tension steel $p = \frac{100 \, A_t}{bd}$ \quad (A_t = tension steel)

$$= \frac{100 \times 4 \times \frac{\pi}{4} \times 25^2}{250 \times 360}$$

$$= 2.18 \%$$

Design shear stress $\tau_c = 0.84 N/mm^2$

Nominal shear stress $\tau_v = \frac{V_u}{bd}$

$$= \frac{200 \times 10^3}{250 \times 360}$$

$$= 2.22 \, N/mm^2$$

Net design shear stress $\tau_{us} = \tau_v - \tau_c$

$$= 2.22 - 0.8$$

$$= 1.42 \, N/mm^2$$

[Adopt] 10mm - 2 legged shear stirrups giving

$$A_{sv} = 2 \times \frac{\pi}{4} \times 10^2 = 157 mm^2$$

spacing of vertical shear stirrups

$$x = \frac{0.87 \cdot f_y A_{sv}}{\tau_{us} \cdot b}$$

$$= \frac{0.87 \times 415 \times 157}{1.48 \times 250}$$

$$= 160 mm$$

$$< 0.75d (= 270mm) \quad O.K.$$
$$> 100 mm \quad O.K.$$

Min^m shear reinforcement is given by

$$A_s \geq \frac{0.4 \cdot b \cdot x}{0.87 f_y}$$

$$= \frac{0.4 \times 250 \times 160}{0.87 \times 415}$$

$$= 45 mm^2 \quad < A_{sv} \quad O.K.$$

Development length

Q2. A simply supported beam is 25cm × 50cm & has 2-25 mm TOR bars going into the support. If a shear force at the centre of support is 110 kN at working loads, determine the anchorage length. Assume M20 mix & Fe415 grade TOR steel.

For a load factor equal to 1.5, the factored

$$SF = 1.5 \times 110 = 165 kN$$

Assuming 25mm clear cover to the longitudinal bars

$$Effective \ depth = 500 - 25 - \frac{25}{2} = 465 mm$$

Characteristics strength of TOR steel $f_y = 415 \text{ N/mm}^2$

Moment of resistance $M_1 = 0.87 f_y A_t (d - 0.42x)$

$$x = \frac{0.87 f_y A_t}{0.36 f_{ck} b}$$

$$= \frac{0.87 \times 415 \times 628}{0.36 \times 20 \times 250}$$

$$= 126 \text{ mm} < x_m$$

D_{r_1}

$$M_1 = 0.87 f_y A_{st} (d - 0.42x)$$

$$= 0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 20^2 (445 - 0.42 \times 126)$$

$$= 93.45 \times 10^6 \text{ N-mm}$$

Bond stress $\tau_{bd} = 1.2 \text{ N/mm}^2$ for M20 mix

It can be increased by 60% in case of TOR bars.

Development length $L_d = \dfrac{\phi \sigma_s}{4 \tau_{bd}}$

$$= \frac{\phi (0.87 f_y)}{4 \tau_{bd}}$$

$$= \frac{0.87 \times 415 \phi}{4 \times 1.6 \times 1.2} = 47\phi$$

If the bar is given a 90° bend at the centre of support, its anchorage value

$$L_o = 8\phi = 8 \times 20 = 160 \text{ mm} \quad (\text{for 90° bend})$$

$$L_d \leq 1.3 \frac{M_1}{V} + L_o \quad \text{for simply}$$

$$47\phi \le \left\{ \frac{1\cdot3 \times (65\cdot 45 \times 10^6)}{165 \times 10^3} \right\} + 160$$

$$\Rightarrow \phi \le 19mm$$

Since actual bar diameter of 20mm is greater than 19mm, there is a need to increase the anchorage length. Let us increase the anchorage length L_0 to 240mm. It gives

$$\phi \le 20.8 \text{ mm} \quad \text{(at support (0.15))}$$

The arrangement of 90° bend is shown.

(90° bend)
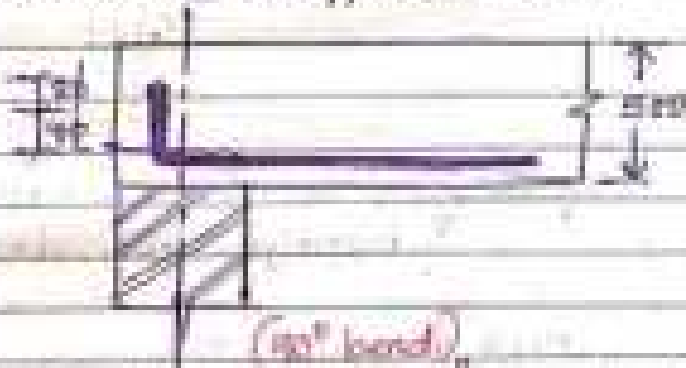
Alternatively,

Provide a U bend at the centre of support. Its anchorage value $L_0 = 16\phi = 320mm$
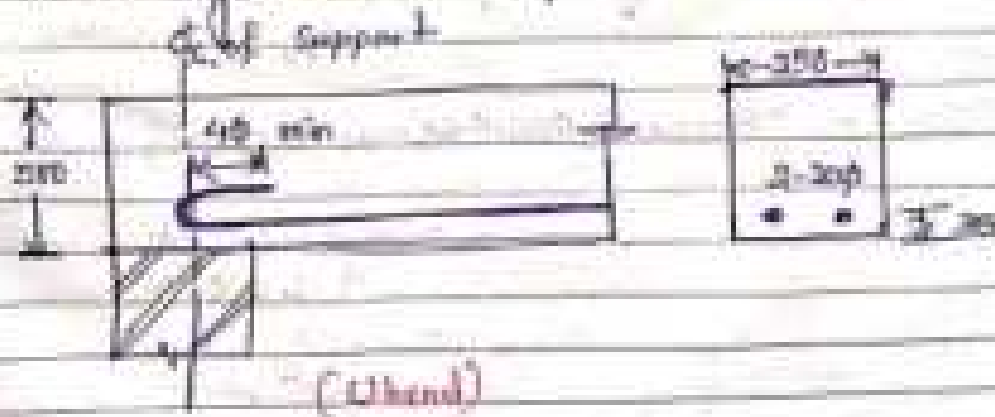
$$L_d \le \frac{1\cdot3 M_1}{V} + L_0$$

$$\Rightarrow 47\phi \le \frac{1\cdot3 \times 92.55 \times 10^6}{165 \times 10^3} + 320$$

$$\Rightarrow \phi \le 22.47mm$$

Actual bar diameter provided is 20mm < 22.47mm OK
The arrangement of U bend

(U bend)

Q2. A continuous beam 25cm x 40cm carries 3-16φ bars beyond the point of inflection in the sagging moment region as shown in fig. If the factored SF at the point of inflection is 100kn, $\sigma_{ck} = 20 \text{ N/mm}^2$ & $\sigma_y = 415 \text{ N/mm}^2$. Check if the beam is safe in bond?
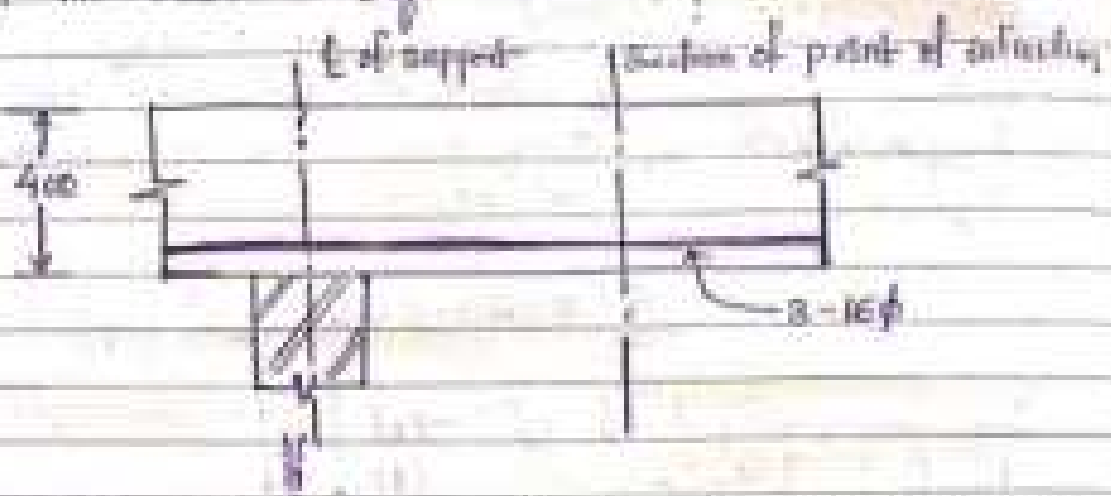
Assuming 25mm clear cover to the longitudinal bars
Effective depth $d = 400 - 25 - \frac{16}{2}$
$= 367 \text{mm}$

Depth of neutral axis

$$x = \frac{0.87 \sigma_y A_t}{0.36 \sigma_{ck} \cdot b}$$

$$= \frac{0.87 \times 415 \times 3 \times \frac{\pi}{4} \times 16^2}{0.36 \times 20 \times 250}$$

$$= 120 \text{mm} < x_m (= 0.48 d) \quad (O.K)$$

Moment of resistance $M_r = 0.87 \sigma_y A_t (d - 0.42 x)$

$$= 0.87 \times 415 \times 603 (367 - 0.42 \times 120)$$

$$= 68.2 \times 10^6 \text{ N-mm}$$

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

For M20 mix & INSDsfd Bond stress $\tau_{bd} = 1.6 \times 1.2 \text{ N/mm}^2$

$$= \frac{0.87 \times 415 \phi}{4 \times 1.6 \times 1.2}$$

$$= 47\phi$$

Anchorage length L_o = greater of or 12ϕ

= Greater of 367mm or $12 \times 16 = 192$mm

= 367mm

$$L_d \leq \frac{M}{V} + L_o$$

$\Rightarrow 47\phi \leq \dfrac{68.92 \times 10^6}{150 \times 10^3} + 367$

$\Rightarrow \phi \leq 17.6$mm

Thus, 16mm bars are safe in bond at the point of inflection.

Ch.1 Design of slab and stair case (LSM)

Qu.1) Design a simply supported roof slab for a room 5m × 3.5m clear in size if the superimposed load is 5kN/m². Use M25mix & Fe415 grade steel.

Minimum depth of slab $d = \dfrac{L}{\alpha \beta \gamma \delta \lambda}$

$$\left(\begin{array}{l} \text{let, } \alpha = 20 \\ \beta=1, \gamma=1, \delta=1, \lambda=1 \end{array}\right)$$

$\Rightarrow d = \dfrac{3500}{20} = 175 mm$

Lets adopt overall depth $D = 190 mm$

Dead Load of slab $= 0.19 \times 1 \times 25 = 4.75 kN/m$
Superimposed load $= 5 \times 1 = 5 kN/m$
∴ Total Load $= 9.75 kN/m$
Factored load $= 1.5 \times 9.75 = 14.63 kN/m$

Minimum B.M at centre of shorter span $= \dfrac{wl^2}{8}$
Assume steel consists of 10mm bars with 15mm clear cover
Effective depth $d = 190 - 15 - 5 = 170 mm$
Effective span of slab $L = 3.5 + d$
$= 3.5 + 0.17$
$= 3.67 m$

$BM = 14.63 \times \dfrac{3.67^2}{8}$

$= 24.63 kN\cdot m$

Max^m shear force $= \dfrac{wL}{2}$

$= 14.63 \times \dfrac{3.5}{2}$

$= 25.6 kN$

Depth of slab is given by $BM = 0.138 f_{ck} bd^2$

$d = \sqrt{\dfrac{24.63 \times 10^6}{0.138 \times 25 \times 1000}} = 85 mm$

Adopt effective depth $d = 130mm$
Overall depth $D = 156mm$

Area of tension steel $A_t = ?$

$$M = 0.87 \sigma_y A_t \left(d - \frac{\sigma_y A_t}{\sigma_{ck} \cdot b} \right)$$

$$\Rightarrow 24.43 \times 10^6 = 0.87 \times 415 \times A_t \left(130 - \frac{415 A_t}{28 \times 1000} \right)$$

$$\Rightarrow A_t = 541.6 mm^2$$

Use 10mm bars @ 140mm c/c giving total area $560.7 mm^2$
$$> 541.6 mm^2 (O.K)$$

Bend alternate bars at $\frac{1}{4}$ from the face of support where moment reduces to less than half of its max^m value. Temperature
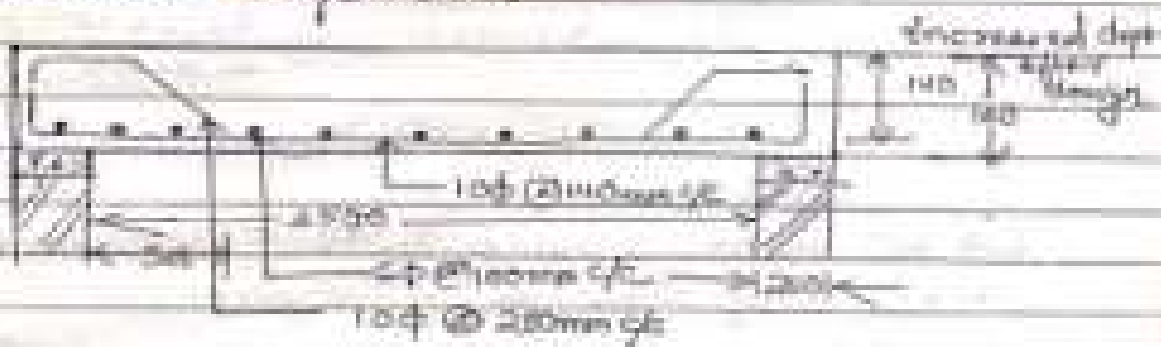
Use 6mm HS bar @ 150mm c/c → secondary

(1) Check for shear

$$\% \text{ tension steel} = \frac{100 A_t}{bd} = \frac{100 \times \frac{\pi}{4} \times 10^2 \times 1000}{1000 \times 130}$$

$$n(1)max = \frac{560.7 \times 100}{1000 \times 140}$$

$$= 0.22\%$$

% tension steel $= 0.22\%$
Shear strength of concrete for 0.22% steel $\tau = 0.32 N/mm^2$
For 150mm thick slab $k = 1.3$

$\tau_c^1 = k\tau_c = 1.12 \times 0.34 = 0.408 \ N/mm^2$

Nominal shear stress $\tau_v = \dfrac{V_u}{bd}$

$$= \dfrac{25618}{1000 \times 130}$$

$$= 0.197 \ N/mm^2 \quad < \tau_c^1 \quad (O.K.)$$

The slab is safe in shear.

2. Check for development length
Moment of resistance offered by 10mm bars @ 200mm c/c

$$M_1 = 0.87 \, f_y \, A_{st} \left(d - \dfrac{f_y \, A_{st}}{f_{ck} \, b} \right)$$

$$= \dfrac{0.87 \times 415 \times 78.5 \times 1000}{200}$$

$$\times \left(130 - \dfrac{415 \times 78.5 \times 1000/200}{25 \times 1000} \right)$$

$$= 12.14 \times 10^6 \ N.mm$$

$V = 25618 \ N$

let us assume anchorage length $L_o = 0$

$$L_d \leq 1.3 \dfrac{M_1}{V}$$

$$47\phi \leq \dfrac{1.3 \times 12.14 \times 10^6}{25618}$$

$$\Rightarrow \quad \phi \leq 16.79mm \qquad (O.K.)$$

The code requires that bars must be carried into the supports by at least $\dfrac{L_d}{3} = 155mm$

3. Check for deflection

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

$$= \dfrac{100 \times \left(78.5 \times 1000/200 \right)}{1000 \times 130}$$

$$= 0.4\%$$

From the table named " Modified factor for tension
reinforcement",

$*$ For $f_t = 0.41\%$ & $F_s = 240MPa$ (in) Fe 415
grade steel $\boxed{Y = 1.22}$
$p = 1, \alpha = 1, S = 1$

Allowable $\dfrac{L}{d} = \dfrac{\text{~~~~~~}}{\text{~~~}}$ ~~~~~~~~~~ $20 \times 1.32 = 26.4$
$\dfrac{l_y}{\text{~}} \leqslant \times 10 \times 52$

Actual $\dfrac{L}{d} = \dfrac{3470}{150} = 28.23 \not< 26.4$ (NS)

$\therefore$ There is a need to increase the depth of
slab. Well as increase the effective depth to
140mm & overall depth to 160mm

Foundation :-

Q1) Determine the area & depth of foundation of a square column carrying 1000kN vertical load. The gross bearing capacity of soil = 100 kN/m². density = 18 kN/m³ & angle of repose = 30°

Load on column $W = 1000$ kN

Approximate area of footing $= \dfrac{W}{P}$

$$= \dfrac{1000}{100}$$

$$= 10 \text{ m}^2$$

Min^m depth of foundation is given by

$$h = \dfrac{P}{\gamma}\left(\dfrac{1-\sin\phi}{1+\sin\phi}\right)^2$$

$$= \dfrac{100}{18}\left(\dfrac{1-\sin 30°}{1+\sin 30°}\right)^2$$

$$= 0.71 \text{ m}$$

Weight of foundation including earth $= 18 \times (10 \times 0.71)$

$$= 121 \text{ kN}$$

Total load on foundation $= (1000) + 121$

$$= 1121 \text{ kN}$$

Area of foundation required $= \dfrac{1121}{100} = 11.21 \text{ m}^2$

Lets, Revise the area of foundation due to increased self weight of foundation & earth.

Weight of foundation and earth $= 18 \times (11.21 \times 0.71)$

$$= 13.5 \text{ kN}$$

Total load on foundation $= 1000 + 13.5$

$$= 1185 \text{ kN}$$

Area of foundation required $= \dfrac{1185}{100} = 11.85 \text{ m}^2$

Adopt a foundation having an area of 11.56 m² ie 3.4m × 3.4m at a depth of 0.71m from ground level.

Q4:2 Design a square spread footing to carry a column load of 1400kN from a 0 400 mm square tied column containing 20mm bars as the longitudinal steel. The bearing capacity of soil is 100kN/m². Consider base of footing 1m below the ground level. The unit weight of earth is 20kN/m³. Use $\sigma_{cbc} = 25 \, N/mm^2$, $\sigma_y = 415 \, N/mm^2$ a load factor = 1.5

(i) Soil pressure

Axial load = 1400 kN

Approximate area of footing required = $\frac{1400 \, kN}{100 \, kN/m^2}$ = 14m²

Weight of footing including earth = $20 \times 1 \times 14 \, kN/m^2$
(10% of vertical load) = 200kN

~~Total weight on soil = 1600~~

~~Actual area of footing required = $\frac{1600}{100}$ = 16m²~~

Total weight on soil = axial load + weight of footing
= 1400kN + 200kN
= 1600kN

Actual area of footing required = $\frac{1600 \, kN}{100 \, kN/m^2}$ = 16m²

Provide 4m × 4m square footing giving total area = 16m² (o.k.)

Step-1 Determining size of footing

(ii) Bending moment

The net earth pressure acting upward due to factored load is

$p = \frac{1400 \, kN \times 1.5}{16}$

= 131.25 kN/m²

bending moment about axis x-x passing through the face of column
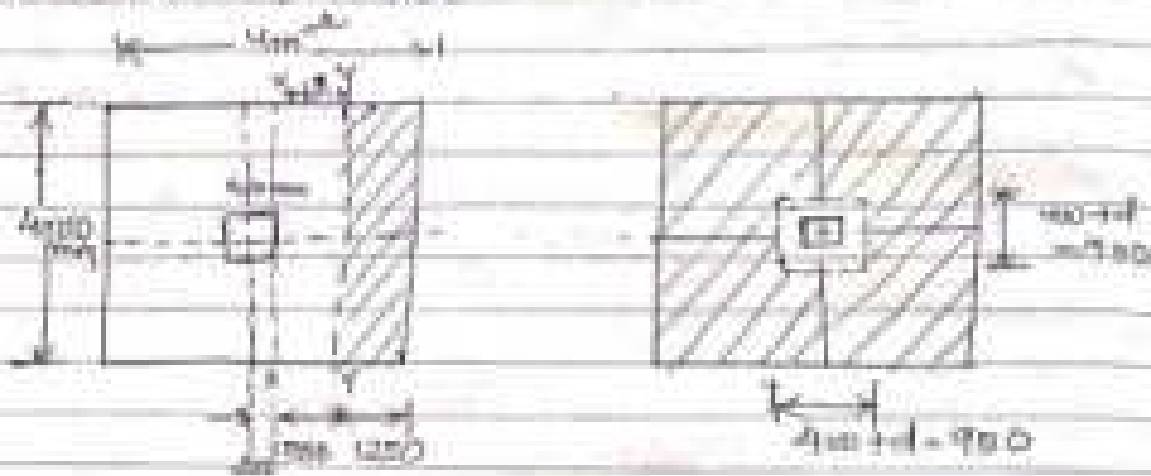
(critical section for moment & one way shear)   (Critical section for 2-way shear)

$$M_u = 131.24 \times \frac{u}{kN}\left(\frac{4-0.4}{2}\right)^2 \times \frac{1}{2} = \frac{wab^2}{2}$$

= critical force × distance from critical section

= 241 kN·m

The effective depth required is BM = $0.138 f_{ck} b d^2$

$$M_u = 0.138 f_{ck} b d^2 \qquad d = \sqrt{\frac{95 \times 10^6}{0.138 \times 25 \times 4000}}$$

= 248 mm

Adopt 550mm effective depth & 600mm overall depth. Increased depth is taken due to shear consideration.

Area of tension steel is given by BM

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

$851 \times 10^6 = 0.87 \times 415 \, A_t \left(550 - \dfrac{415 A_t}{25 \times 1000} \right)$

$\Rightarrow A_t = 4434 \, mm^2$

Use 18mm bars @ 1400mm c/c $A_t = 4520 \, mm^2 > 4434 \, mm^2$

(O.K.)

$$p = \dfrac{4520 \times 100}{4000 \times 550}$$

$$= 0.2\%$$

(iii) Shear one way action

the critical section is taken at distance 'd' away from the face of column.

Shear force $V_u = 121.25 \times 4 \left[\left(\dfrac{4 - 0.4}{2} \right) - 0.55 \right]$

$= u \, A (b - 0.55)$ ← shaded portion

$= 656.8 \, kN$

Nominal shear stress $\tau_v = \dfrac{V_u}{bd}$

$$= \dfrac{656800}{4000 \times 550}$$

$$= 0.298 \, N/mm^2$$

Shear strength of M25 concrete with 0.2% steel

$\tau_c = 0.323 \, N/mm^2 > \tau_v$ (O.K)

(iv) Shear 2-way action

The critical section taken at a distance "0.5d" away from the face of column, and also

Shear force

$V_u = 121.25 \left[16 - (0.4 + 0.55)^2 \right] = 1793 \, kN$

Nominal shear stress $\tau_v = \dfrac{V_u}{bd}$

$$= \dfrac{1782 \times 1000}{4(400 \times 550).550}$$

$$= 0.918 \, N/mm^2$$

Shear strength of M25 concrete,

$$\tau_c' = k_s \tau_c \quad , \quad k_s = (0.5 + \beta_c)$$

$\beta_c = \dfrac{\text{length of shorter side of column}}{\text{length of longer side of column}}$

$k_s = 0.5 + 1 \times 1$

$\therefore \quad k_s = 1$

$$\tau_c' = \tau_c = 0.25 \sqrt{f_{ck}} = 1.25 \, N/mm^2 > 0.918 \, N/mm^2 \quad (0.k)$$

So above that a footing having on effective depth of 208 mm wouldn't be safe in shear.

(V) Development length

Development length of 16mm bars

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

$$= \dfrac{(0.87 \times 415)\phi}{4 \times 1.6 \times 1.4}$$

$$= 40\phi$$

Where 1.6 factor due to deformed bars (+60% ↑)

$$L_d = 640mm$$

Actual embedment length provided from factor of column is $= \left(\dfrac{4000 - 400}{2} \right) - 50mm(cover)$

$$= 1750mm > L_d \quad (0.k)$$

4000mm

Main reinforcement bare

400

50mm cover

4000

Theory

RC Sections & their behaviour

(a)   (b)   (c)

(d)   (e)   (f)

Concrete Grade Table :— (IS 456 : 2000)
RCC ≥ M20

Group	Concrete Grade	Mix Ratio	Characteristic compressive stress
Ordinary concrete	M5	1:5:10	5 N/mm²
	M7.5	1:4:8	
	M10	1:3:6	
	M15	1:2:4	
	M20	1:1.5:3	
Standard concrete	M25	1:1:2	
	M30	Design Mix	
	M35	"	
	M40	"	
	M45, M50	"	
High strength concrete	M55, M60,	"	
	M65, M70	"	

* Minⁿ Grade of concrete for PCC is M15
     "   "   "   "   for RCC is M20

Grade of steel : Represented by "yield strength" of
steel

- specified yield strength may be treated as
characteristic strength f_y (Expressed in N/mm^2)

- Mild steel (Fe250): less commonly used because
of their low strength

- HYSD (high yield strength deformed) Bars e.g
Fe415 & 500

Fe415D, Fe500D, Fe550 & Fe550D

- TMT Bars - Thermo mechanically Treated
Inner core - Soft & ductile
Outer core - Very high tensile strength

- Rebars - Reinforcing bars
Size of bars
Nominal dia - 5mm to 40mm

Common Bar: 8 mm, 10mm, 12mm, 16mm, 20mm, 25mm, 32mm

* Single Reinforced section - Beam

* Analysis → Determining the strength of a beam with
given dimensions & reinforcement.

Design → Creating a beam that will carry a
specified load or combination of loads

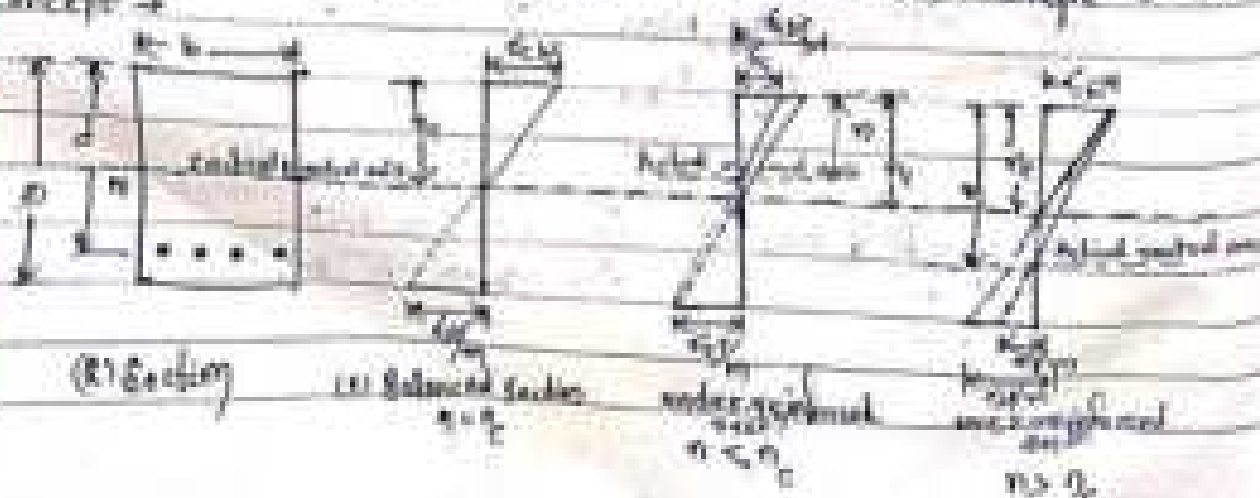* Flexural Strength of a beam is the max amount of
bending it can withstand

* First Principle Analysis → is based on the loads
acting on the undeformed geometry of the str
2nd Principle → based on deformed shape of str

concept →

(a) section   (b) Balanced section   under reinforced   over reinforced

∴ A [balanced section] is that in which stress in concrete & steel reach their permissible value at same time.

* The % of steel corresponding to this section is called as balanced steel & the neutral axis is called as critical neutral axis n_c.

[Under-reinforced section]

→ In an U.R sec^n, the % of steel provided is less than that provided in balanced section. So the actual N.A will shift upwards to n_{na}

- In U.R sec^n the stress in steel 1st reaches its permissible value while the conc. is under-stressed

Features

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

ii) The actual N.A. lies above the critical N.A. $n < n_c$

iii) The % of steel is less than the balanced section hence the section is economical

iv) Ductile failure.

v) The moment of resistance is less than balanced section.

* In U.R. section the failure is ductile because steel fails first & sufficient warning is given before collapse. Due to ductile failure & economy the U.R, sec^n are preferred by designers.

[Over reinforced section]

- In an O.R sec^n the % of steel provided is greater than the balanced section, so the actual N.A. shift downward.

- In O.R sec^n the stress in concrete reaches the permissible value while steel is not fully stressed.

- concrete is brittle & it fails by crushing suddenly

- As steel is not fully utilised the o.r sec^n is uneconomical (steel is much costlier than concrete)

Features

i) the actual N.A. is below the critical N.A $n > n_c$

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

*) the % of steel is more than the balanced section, so the section is uneconomical
*) Sudden failure

* A beam bends under bending moment, resulting in a small curvature. At the outer face (tensile face) of the curvature the concrete experiences tensile stress while at the inner face (compressive face) it experiences compressive stress

• Design of reinforced concrete elements for flexure involves
  (i) sectional design
  (ii) Member detailing

  – Sectional design includes the determination of cross-sectional geometry & the required longitudinal reinforcement

  – Member detailing includes the determination of bar lengths, locations of cut-off points & detailing of reinforcement as governed by the development, splice & anchorage length requirements

  Flexural members are slender members that deform primarily by bending moments caused by concentrated couples or transverse forces.

  First Principle Analysis is based on the loads acting on the undeformed geometry of the structure.

Def^n — Limit State method refers to the method which considers the ultimate strength of the material at failure (which is ignored in WSM) & also ensures that the str. is serviceable for its design period.

— Limit states are the acceptable limits for the safety & serviceability requirements of the str. before failure occurs

Limit state of collapse	Limit state of Serviceability
— It deals with the Strength & Stability of str. subjected to max^m design load.	— It deals with deflection & cracking of str. under service load.

Sl. No	WSM	LSM
01	Based on elastic theory i.e concrete & steel are elastic & stress-strain curve linear for them	— Based on actual stress-strain curves of steel & concrete. For concrete stress strain curve is non-linear
02	FOS is applied to yield stresses to get permissible stresses	— Partial safety factors are applied to get design values of stresses
03	No FOS for Loads	— Design loads are obtained by multiplying partial safety factors of load to working loads
04	Exact margin of safety is not known	— Exact margin of safety is known
05	This method gives thicker sections, so less economical	— This is more economy as it gives thinner sections
06	This method assumes that the actual load, permissible stresses & FOS are known so it is called deterministic method.	— This method based on probabilistic approach which depends upon the actual load or experience hence it is called non-deterministic method

Is code suggestion → The LSM is defined as a method which limits the stress condition of the material of the str upto a certain load at which acceptable limit of safety & serviceability are applied so that the failure of str doesn't occur.

* Spacing of reinforcement in slab :—

Let, Amount of steel we need or the area of steel we need in a slab: $A_s \geq 492 \, mm^2/m$ i.e. per mt. length
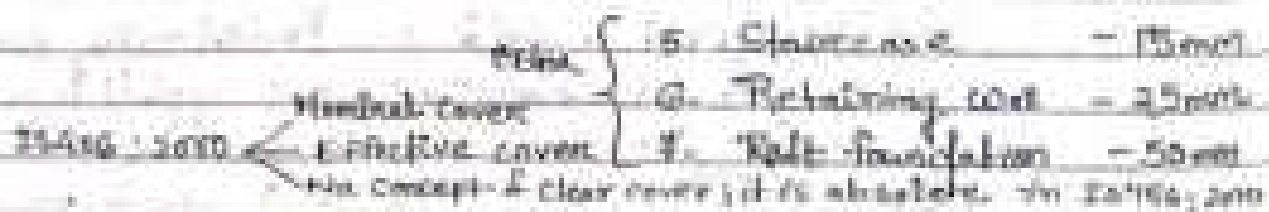

|←——— 1m of slab ———→|

$492 \, mm^2$
$\phi 12$ bars

$$Spacing = \frac{Area\ of\ Single\ bar}{Total\ Area} \times Per\ mt\ length\ of\ slab$$

$$= \frac{(\pi \times 12^2)/4}{492} \times 1000 = 22.9 \, mm \approx 220$$

* Cover to reinforcement :

Concrete cover in 1. Footing/Foundation — 50mm
                          2. Column   — 40mm
                          3. Beam   — 25mm
                          4. Slab   — 15mm
                          5. Staircase   — 15mm
                          6. Retaining wall   — 25mm
                          7. Raft foundation   — 50mm

IS456 : 2000 ← Nominal cover / Effective cover
No concept of clear cover, it is absolute. In IS456 : 2000
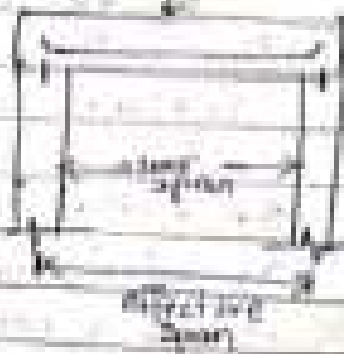
* Min. reinforcement

Beam   Slab   Column

* Effective span of beam & slab (IS456 1978 Cl. 22.2)

Clear Span + Effective depth of beam
or
— Beam/slab centre to centre of support

|← span →|
|← effective span →|

Spacing of reinforcement in slab

Q. 16 mm φ bars 300 mm spacing is mentioned but in site 12 mm φ bar available for slab design. Then what will be the placing?

Ans.

Let Slab size = 3m × 4m

no. of main bar = ?

Main bars

4m

300 mm c/c
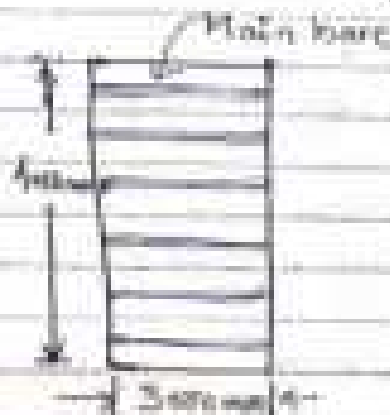
$$\boxed{\text{No. of main bar} = \frac{\text{Total length}}{\text{spacing}} + 1}$$

$$= \frac{4000}{300} + 1$$

$$= 13.3 + 1$$

$$= 14 \text{ nos}$$

weight $= 14 \times 3000 \times 1.580$ 16 mm φ = 1.58 kg/m

$$= \boxed{66.36 \text{ kg}}$$

12 mm φ :-

spacing ?
no. of bars ?
weight ?

$$\boxed{\text{Spacing formula} = \frac{\text{Change dia}^2 \times \text{spacing}}{\text{Original dia}^2}}$$

$$= \frac{12^2 \times 300}{16^2}$$

$$= 168.75 \text{ mm (spacing)}$$

No. of bars $= \frac{4000}{168.75} + 1$

equal wt (a.k)

$$= 25 \text{ nos}$$

weight $= 25 \times 3m \times 0.888 = \boxed{66.6 \text{ kg}}$
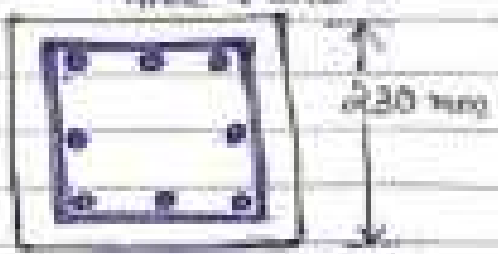
Ex-1 Min^m & Max^m Reinforcement Required in Column
As per IS456:2000   (Thumb Rule)

→ Area of steel for column should lie between
0.8% to 6% of gross cross sectional area of column.

→ $\dfrac{\text{Area of steel}}{\text{Area of column}} = 0.8\% - 6\%$

Let, column c/s Area → 230 × 550
      Steel bars → 8 - 16ϕ

Area of column → 230 × 550
                = 80,500 mm^2

Area of steel = $8 \times \left(\dfrac{\pi}{4} \times 16^2\right)$
              = 1607.4 sq mm^2
              = 1608 mm^2

check :

$\dfrac{\text{Area of steel}}{\text{Area of column}}$ %

= $\dfrac{1608}{80500} \times 100\%$

= 1.97%

So, 0.8% < 1.97% < 6% (O.K)

Min^m Reinforcement B/E
Min^m A$_s$ of steel

Slab   beam   Column

$\dfrac{Min^m}{steel}$ = 0.15% c/s area Fe250   Tension R/F $\dfrac{Min^m}{steel}$ 0.8% c/s

0.12% c/s " Fe500 $\dfrac{As}{bd} = 0.85$
                              $\dfrac{}{fy}$

                         Shear R/F (stirrups)
                         $\dfrac{Asv}{b Sv} = \dfrac{0.4}{0.87 fy}$
                              Spacing $\rightarrow$

Lapping Length or Reinforcement lapping

① Lap length is the length of the overlap of bar required to safely transfer stress from one bar to another.

Development length are provided to create a bond b/w steel & concrete

lap   Development   Anchorage
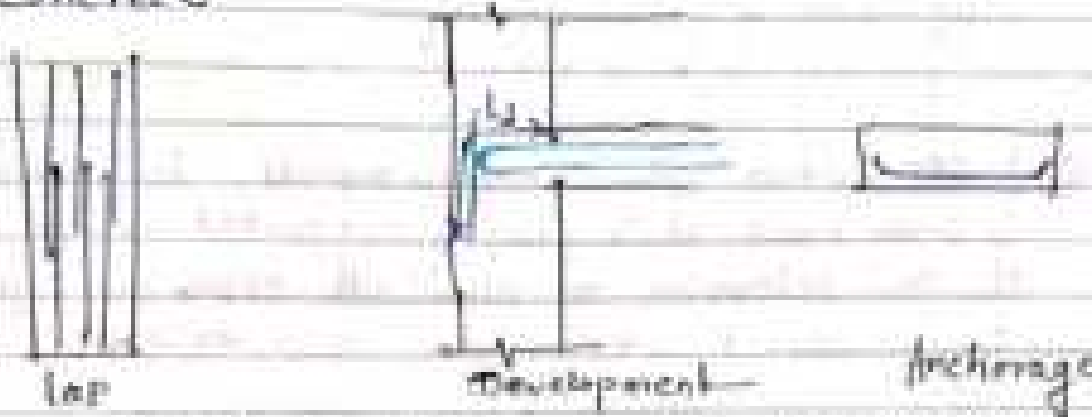
② Anchorage length is the equivalent length of the reinforcement bar which is considered to be available when a straight bar is bent through some angle

③ Development length is the length of bar that is surrounded by the concrete beyond any section in an RCC beam, which would be capable enough to resist the applied pulling force. If not provided then the steel reinforcement wouldn't be able to resist the pull so it will come out which lead to failure

$4 \times d$ — $180°$ bend
$6d$ — $135°$ "
$8d$ — $90°$ bend bar
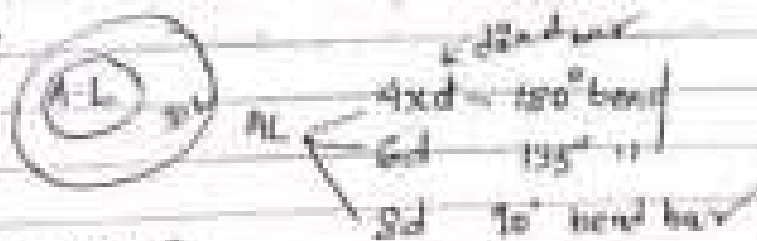
#① analysis ⇒ M.R., 2 xs.M. ?
② design → Dimensions & Reinforcement ? In beam
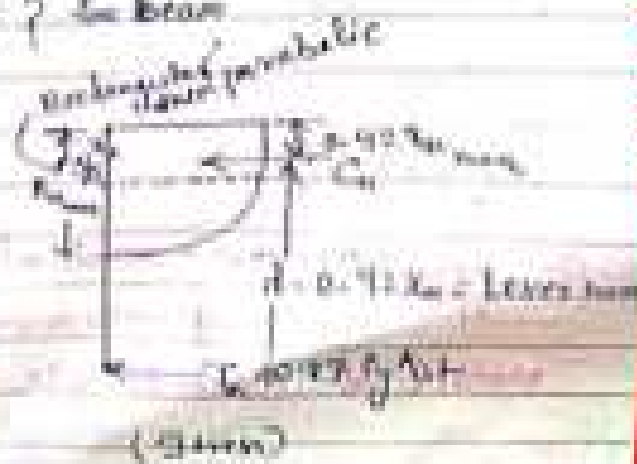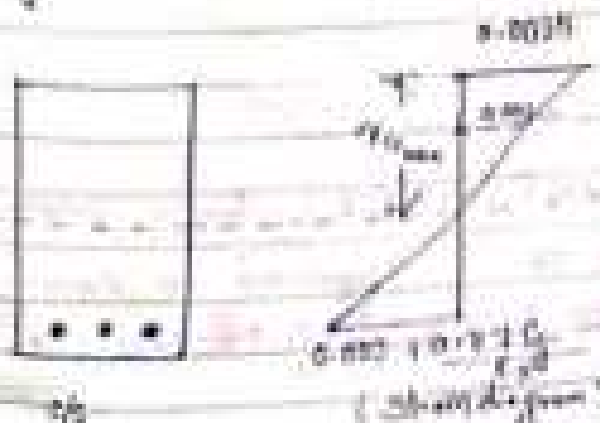
Long section: $x_u < x_{u,max}$ if $x_u = x_{u,max}$

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

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

Ultimate moment of resistance

$$M_u = 0.36 f_{ck} \cdot x_u \cdot b \,(d - 0.42 x_u) \longleftarrow \text{Comp. force} \times \text{Comp. arm}$$

$C_u = T_u$ ———— ②

comp. force in conc. = Tensile force in steel

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

Q. A RCC section 250 × 600mm overall is reinforced with 4-25mm bars it is simply supported on an effective span of 5m. Determine the max^n udl beam can carry. Use M20, Fe415

Area of steel $A_{st} = 4 \times \frac{\pi}{4} \times 25^2 = 1963.49 \, mm^2$

e = 60mm ∅ cover

d = 600 - ? = 560mm

1. $x_{u,max} = 0.48 d = 0.48 \times 560 = 268.82 \, mm$

2. $C_u = T_u$

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

b) $0.36 \times 20 \times x_u \times 250 = 0.87 \times 560 \times 1963.2$

⇒ $x_u = 316.49 \, mm$

As $x_u > x_{u,max}$ use $x_{u,max}$

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

= 334.74 kN·m

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

⇒ $334.74 = \frac{w \times 5^2}{8}$

⇒ $w = 75.49$ kN (factored loads)

working load $w = \frac{75.49}{1.5} = 50.77$ kN

Q. Calculate the area of steel required for a beam of 250 × 390mm overall it is required to carry a moment of 60kN·m. Use M20 Fe500. Take all cover as 40mm

$$d = 390 - 40 = 350 \, mm$$

I. $x_{u max} = 0.46d$

$= 0.46 \times 350$

$= 161mm$

II. Calculation of actual x_u (can't use $c_u = T_u$ as annealed/not given)

$M = 0.36 f_{ck} x_u b (d - 0.42 x_u)$

$\Rightarrow 50 \times 10^6 = 0.36 \times 20 \times x_u \times 220 (350 - 0.42 x_u)$

$\Rightarrow x_u = 97.12 mm$

$x_u < x_{u max}$ Beam is U/R

III. Method 1

$A_{st} = \dfrac{0.5 f_{ck}}{f_y} \left[1 - \sqrt{1 - \dfrac{4.6M}{f_{ck} b d^2}} \right] bd$

Method 1

$c_u = T_u$

$= \dfrac{0.5 \times 20}{500} \left[1 - \sqrt{1 - \dfrac{4.6 \times 50 \times 10^6}{20 \times 220 \times 350^2}} \right] \times 220 \times 350$

$= 381.4 mm^2$

Ch-3   Analysis & Design of Single & Simple Reinforced
          section by LSM

Methods of Solving Numericals

Analysis Type   Design Type

Given   To find   Given   Design
1. c/s, A_{st} MR   1. M_d b, D, A_{st}
2. $x_{u, \lim}$ P_t, Mu   2. b, M_d D, A_{st}, A_{sc}
                                                              $A_{st, req}$
3. c/s $M_{u, \lim}$, A_{st} 3. Leff, $\frac{d}{b}$ ratio   b, d, A_{st}
4. M_u A_{st} 4. clear span   b, d, D, A_{st}
                                          support
5. c/s,   W (UDL)   bearing

※ M_u = ultimate moment
    M_d = design Moment

Steel (f_y)	$x_{u, max}$	$M_{u, \lim}$	$P_{t, \lim}$
Fe 250	0.53d	$0.149 f_{ck} b d^2$	$0.0217 f_{ck}$
Fe 415	0.48d	$0.138 f_{ck} b d^2$	$0.0419 f_{ck}$
Fe 500	0.46 d	$0.133 f_{ck} b d^2$	$0.0385 f_{ck}$

Q. Find the moment of resistance if steel provided is
   4 bars of 18mm diameter in a beam 300 × 550 mm
   effective concrete M20 & steel Fe 500 are used.
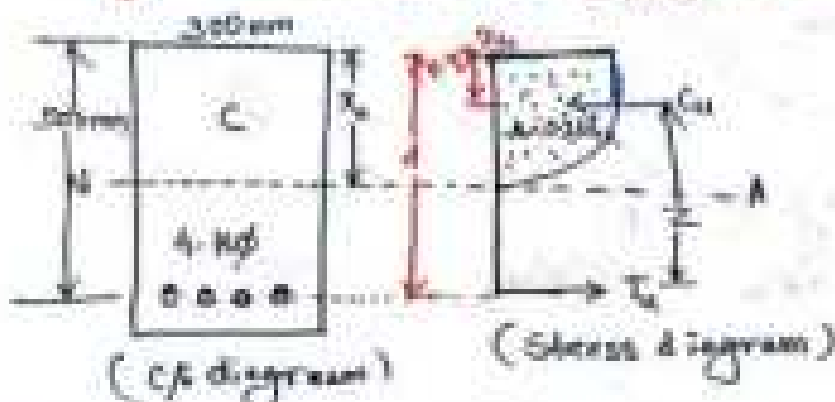
(c/s diagram)   (Stress diagram)

[unclear heading] ___ 2-A)

① Concrete = steel

$\Rightarrow (\sigma \times A)_c = (\sigma \times A)_{st}$

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

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

$A_{st} = 4 \times \frac{\pi}{4} \times 16^2$
$= 804 \cdot mm^2$

$= \dfrac{0.87 \times 500 \times A_{st}}{0.36 \times 20 \times 245}$

$x_u = 161.96\,mm$

$x_{u.max} = 0.46\,d = 0.46 \times 500 = 230\,mm$

$x_u < x_{u.max} \Rightarrow$ Section is under-reinforced

$M_R = T_u \times z = C_u \times z$

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

$= 0.87 \times 500 \times 804.25 \times (500 - 0.42 \times 161.96)$

$= 151.126 \times 10^6\, N \cdot mm$

$= 151.126\, kN\cdot m$

∴ Moment of Resistance is 151.126 kN-m (Ans)

Q2. If x_u is limited to 0.34, calculate p_t & M_u
Assume M20-Fe415 concrete & steel.

$x_{u.max} = 0.34\,d$ ——①

$x_{u.max} = \dfrac{0.87 f_y\, A_{st}}{0.36 f_{ck} b}$ ——②

Equating ① & ②

$0.34\,d = \dfrac{0.87 f_y A_{st}}{0.36 f_{ck} b}$

$$\Rightarrow \frac{0.34 \times 0.45\, f_{ck}}{0.87 f_y} = \frac{A_{st}}{bd}$$

$$\Rightarrow \frac{A_{st}}{bd} = \frac{0.34 \times 0.45 f_{ck}}{0.87 f_y} = 0.278 \times 10^{-2}$$

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

$$= 0.278 \times 10^{-2} \times 100$$

$$= 0.278\% \ (Ans)$$

$$M_u = C_u \times z$$

$$= (0.36 f_{ck} b \cdot x_u)(d - 0.42 x_u)$$

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

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

$$= 0.36 \times 20 \times b \times 0.39 \left(1 - 0.42 \times 0.32\right) d^2$$

$$= 2.342\, b\, d^2$$

$$M_u = 2.342\, b\, d^2 \ (Ans)$$

Q.₃ Find limiting Moment of resistance & steel required for a beam 200 mm × 600 mm overall depth and effective cover = 25 mm. Fe 250/Fc M5 used

$$M_{u,lim} = ?$$
$$A_{st} = ?$$

b = 200 mm   D = 600 mm
d' = 25 mm   d = D - d' = 575 mm
$f_{ck} = 20$ Fe = 415

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

$$= 0.138 \times 20 \times 200 \times 575^2$$

$$= 242.17 \times 10^6 \ N\cdot mm$$

$$= 242.17 \ kN\cdot m \ (Ans)$$

$$C_u = T_u$$

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

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

$$x_u = x_{u,max} = 0.48d$$

$$= 0.48 \times 575$$

$$= 276 mm$$

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

$$= \frac{0.36 \times 25 \times 300 \times 276}{0.87 \times 415}$$

$$= 2063.98\ mm^2 \quad (Ans)$$

Q. Calculate the area of steel required for a Singly R/F Concrete beam 220mm wide & 400mm deep to resist an ultimate moment of 60 KN·m. Use Concrete Mix M20 & Fe 500 grade steel, effective Cover = 30mm.

$b = 220mm$, $D = 400mm$, $d = D - cover = 400 - 30 = 370mm$,

$M_u = 60\ KN\cdot m$

$A_{st} = ?$

We have to first determine U/R, B/R or O/R

1) $M_{u,lim} = 0.133 f_{ck}\, b\, d^2$ (for Fe500)

$$= 0.133 \times 20 \times 220 \times 370^2$$

$$= 83.78 \times 10^6\ N\cdot mm$$

$$= 83.75\ KN\cdot m$$

Since $M_u < M_{u,lim} \Rightarrow$ section is under reinforced

or $x_u < x_{u,max}$ not to calculate as M_u given

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] b \cdot d \qquad \text{for U/R sec}^n$$

$$= \frac{0.5 \times 20}{500}\left[1 - \sqrt{1 - \frac{4.6 \times 60 \times 10^6}{20 \times 235 \times 340^2}}\right] \times 235 \times 370$$

$$A_{st} = 426.38 \text{ mm}^2$$

* A_{st} can be calculated in 3 ways
  - ① Above formula
  - ② From $P_t \%$
  - ③ Equating $C_u \& T_u$

Q5. A Singly Reinforced beam 230 mm × 500 mm effective c/s is simply Supported & has 6m effective span. Beam is R/F with 4 nos. of 16mm φ bars in tension. Calculate UDL (Including Self weight) it can carry over entire span. Use M20/Fe 415.

$b = 230mm$; $d = 500mm$; $L_{eff} = 6m$

$A_{st} = 4 \times \left(\frac{\pi}{4} \times 16^2\right) = 804.24 \text{ mm}^2$

$w = ?$

$F_{ck} = 20 \text{ MPa}$

$F_y = 415 \text{ MPa}$

→ determine y/R, o/R, o/R ?

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times 804.24}{0.36 \times 20 \times 230} = 175.34 mm$$

$$x_{u,max} = 0.48 d = 0.48 \times 500 = 240 mm$$

$x_u < x_{u,max} \Rightarrow$ section is under-reinforced

$$M_u = T_u \times z \quad \text{or} \quad C_u \times z$$

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

$$= 0.87 \times 415 \times 804.24(500 - 0.42 \times 175.34)$$

$$= 125.88 \times 10^6 \text{ N·mm}$$

$$= 125.88 \text{ kN·m}$$

Simply Supported beam

$M_u = \dfrac{w_d l_e^2}{8}$

$\Rightarrow \quad w_d = \dfrac{8\,M_u}{l_e^2}$

$\Rightarrow \quad w_d = \dfrac{8 \times 127.8 \times 10^6}{6000^2\ mm}$

$\quad = \dfrac{8 \times 127.8\ kN\text{-}m}{6\ m^2}$

$\quad = 27.84\ kN/m$

Load without factor of safety or working load

$w = \dfrac{w_d}{1.5}$

$\quad = \dfrac{27.51}{1.5}$

$\quad = 18.34\ kN/m \quad (Ans)$

Q. Design a beam to carry working moment of 80 kN-m, using M20 grade concrete & Fe415 steel

* Design a beam $\Rightarrow$ b, d, D, A_{st}, No. of bar

* Working Moment $M = 80\ kN\text{-}m$

$f_{ck} = 20$

$f_y = 415$

Step-1. Factored moment $= 1.5 \times M$ (considered in LSM for future possible....)

$M_u = 1.5 \times 80$

$\quad = 120\ kN\text{-}m$

$\quad = 120 \times 10^6\ N\text{-}mm$

Step-2. Equating $M_{u,lim}$ to M_d

$M_{u,lim} = M_d$

$\Rightarrow 0.138\, f_{ck}\, b d^2 = 120 \times 10^6$

Assume: $b = \dfrac{d}{2}$

$\Rightarrow 0.138 f_{ck} \, b \, d^2 = 120 \times 10^6$

$\Rightarrow 0.138 \times 20 \times \dfrac{d}{2} \times d^2 = 120 \times 10^6$

$\Rightarrow d^3 = \dfrac{120 \times 10^6 \times 2}{0.138 \times 20}$

$\Rightarrow d = 442 mm$

Round off the above value

$d = 450 mm$

$b = \dfrac{d}{2} = \dfrac{450}{2} = 225 mm$ $\left(\Rightarrow b = \dfrac{450}{2} = 831 mm \right)$

$\Rightarrow b = 230 mm$

$D = d + d'$

$= 450 + 30$ mm cover assume

$= 480 mm$

$d = 450 mm , \ D = 480 mm , \ b = 230 mm$

Step 2 Calculate Area of steel Reinforcement

$\boxed{P_{t,lim} = 0.96 f_{ck}} = 0.96 \times 20 \times 0.48 \%$
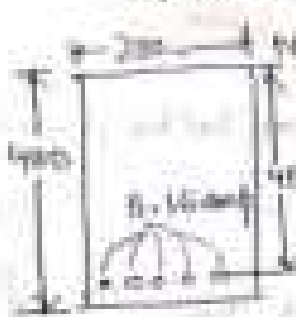
$A_{st} = \dfrac{P_{t,lim} \times bd}{100}$

$= \dfrac{0.96 \times 230 \times 450}{100}$

$= 993.6 \, mm^2$

Assume 16mm diameter bar

No. of bars $= \dfrac{\text{Total Area}}{\text{Area of 1 bar}}$

$= \dfrac{993.6}{\frac{\pi}{4} \times 16^2}$

$= 4.94 \approx 5$

Provide $5 - 16mm\phi$

Q1. A singly R/F rectangular beam of width 230mm is subjected to a B.M. of 40kN·m at working loads. Using LSM find the overall depth of the beam & area of R/F. Take M20/Fe 415. Check the Section for minimum & max^m area of tensile steel.

$M = 40$ kN·m

$M_d = 1.5 \times M = 1.5 \times 40 = 60$ kN·m

$M_{u,lim} = M_d$

$\Rightarrow 0.138 \, f_{ck} \, b d^2 = 60 \times 10^6$

$\Rightarrow 0.138 \times 20 \times 230 \times d^2 = 60 \times 10^6$

$\Rightarrow d = 307$ mm ≈ 310 mm

$D = d + cover$

$= 310 + 30$mm

$= 340$mm

A$_{st}$ Calculation →

① $T_{u,lim} = 0.87 \, f_y \sqrt{} \rightarrow A_{st}$

②
$C_u = T_u \sqrt{} \rightarrow A_{st}$

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

$\Rightarrow A_{st} = \dfrac{0.36 \, f_{ck} \, b \cdot x_{u,max}}{0.87 \, f_y}$ & $x_{u,max} = 0.48 \, d$

$= \dfrac{0.36 \times 20 \times 230 \times 0.48 \times 310}{0.87 \times 415}$

$= 682.489$ mm^2 (A is to be provided)

Check: A$_{st,min}$ formula for rectangular beam $= \dfrac{0.85 \, bd}{f_y}$

$\Rightarrow A_{st,min} = \dfrac{0.85 \times 230 \times 310}{415}$

$= 146.03$ mm^2

$A_{st,max}$ is 4% of c/s

$$= 0.04 \ bD$$
$$= 0.04 \times 230 \times 348$$
$$= 3202 \ mm^2$$

Q.# A simply supported rectangular beam of effective span 4m carries a working load of 20KN/m. Determine size of beam & calculate no. of tension bars required. Use 16mm ∅ bars. Use concrete of grade M20 & Fe500. Assume ratio of depth to width of beam as 1.8 & 50mm effective cover.

w = 20KN/m
L_e = 4m

16mm ∅
M20 , Fe415

$\frac{d}{b} = 1.8$

$\Rightarrow b = \frac{d}{1.8}$

Step 1 $M_u = 1.5 \times M$ $M = \frac{wL_e^2}{8}$

$$= 1.5 \times 60$$
$$= 90 \ KN\text{-}m$$

Step-2 $M_{u, \lim} = M_d$

$$0.138 \ f_{ck} \ bd^2 = 90KN\text{-}m$$

$\Rightarrow 0.138 \times 20 \times \frac{d}{1.8} \times d^2 = 90 \times 10^6$

$\Rightarrow d^3 = \frac{90 \times 10^6 \times 1.8}{0.138 \times 20}$

$\Rightarrow d = 393mm \approx 400mm$

$b = \frac{d}{1.8} = \frac{400}{1.8} = 222 \approx 250mm \ (235 < 250)$

$D = 400 + 50$
$$= 450mm$$

Step 8

$$k_{t,lim} = 0.685 f_{ck}$$

$$= 0.685 \times 20$$

$$= 0.767$$

$$A_{st} = \frac{k_t \cdot bd}{100}$$

$$= \frac{0.767 \times 250 \times 450}{100}$$

$$= 699.2 \, mm^2$$
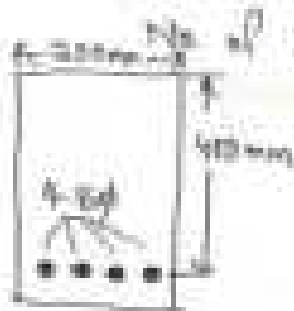
No. of bars $= \dfrac{\text{Total Area}}{\text{Area of 1 bar}}$

$$= \frac{699.2 \, mm^2}{\frac{\pi}{4} \times 16^2 \, mm^2}$$

$$= 3.4 \simeq 4$$

Provide 4 ⬤ 16mm ø

Q.9. Calculate depth & area of steel at midspan of a Simply Supported beam over a clear span 6m. This beam is carrying all inclusive load 20 kN/m. Assume 300mm Bearings. Use M20 & Fe500
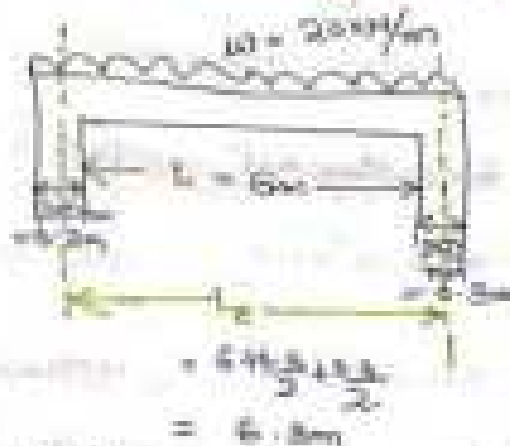
$$= \frac{6+0.3}{2} + \frac{0.3}{2}$$

$$= 6.3m$$

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

$$= \frac{20 \times 6.3^2}{8}$$

$$= 99.225 \, kNm$$

Step 1 $M_d = 1.5 M_u$

$= 1.5 \times 99.236$

$= 148.8535 \text{ kN-m}$

Step 2 $M_{u \lim} = M_d$ & Assume $b = \dfrac{d}{2}$

$\Rightarrow 0.138 \, f_{ck} \, bd^2 = 148.8535 \times 10^6$

$\Rightarrow 0.138 \times 20 \times \dfrac{d}{2} \times d^2 = 148.84 \times 10^6$

$\Rightarrow d^3 = \dfrac{148.84 \times 10^6 \times 2}{0.138 \times 20}$

$\Rightarrow d = 441.9 \text{ mm} \simeq 500 \text{ mm}$

$b = \dfrac{d}{2} = \dfrac{500}{2} = 250 \text{ mm}$

$D = d + cover$

$= 500 + 30$

$= 530 \text{ mm}$

Step 3 $p_{t \lim} = 0.035 \, f_{ck} = 0.035 \times 20 = 0.7 \%$

$A_{st} = \dfrac{p_{t \lim} \, bd}{100}$

$= \dfrac{0.7 \times 250 \times 500}{100}$

$= 950 \text{ mm}^2$

Use 16 mm ϕ bars

No. of bars $= \dfrac{\text{Total area}}{\text{Area of 1 bar}}$

$= \dfrac{950}{\frac{\pi}{4} \times 16^2}$

$= 4.72$

$\simeq 5 \text{ nos}$

Provide 5 - 16mmϕ

Check : $A_{st,min} = \dfrac{0.85\ bd}{f_y}$

$$= \dfrac{0.85 \times 250 \times 245}{3100}$$

$$= 212.5\ mm^2$$

$A_{st,max} = 0.04\ bD$

$$= 0.04 \times 250 \times 530$$

$$= 5300\ mm^2$$

Provided $A_{st} = 950\ mm^2$ (Hence O.K.)

Design of Doubly Reinforced Section

Q: Design a longitudinal R.C. beam with effective span of 4.75m. The beam is carrying

    - 10 kN/m from thick slabs
    - Live load of 5 kN/m
    - Floor finish 3.5 kN/m
    - Load of wall = 20 kN

or, directly factored moment

Size of beam is calculated to (250 × 400) mm. Assume mild exposure condition. Use M_{20} & F_e415.

$b = 250\ mm$

$D = 400\ mm$

For mild exposure cond.", effective cover $d' = 35\ mm$ on both edges. no steel to be provided on both edges

($15\ 4mc = 200$)

Now, $d = D - d'$

$$= 400 - 35$$

$$= 365\ mm$$

$L_{eff} = 4.75\ m$

Step 1: Effective width & depth
$$d = 365\ mm$$
$$b = 250\ mm$$

(ii) Effective length
$$l_{eff} = 4.75\ m$$

(iii) Calculation of load
$$Self\ wt = \rho_{rcc} \times b \times D$$

$$= 24 \times 0.25 \times 0.4 \checkmark$$

$$= 2.5\ kN/m \checkmark$$

$$Total\ load = 2.5 + 10\ kN/m + 5 + 2.5 + 10$$

$$= 31\ kN/m$$

$$Factored\ load = 1.5 \times 21 = 46.5\ kN/m$$

(iv) Calculation of M_{max} & MOR

Maximum bending moment (v) Moment of Resistance of sect^n

$$M_{max} = \frac{wl^2}{8}$$

$$= \frac{46.5 \times 4.75^2}{8}$$

$$= 131.14\ kN\text{-}m \quad \longleftarrow \text{Factored Moment Given in Some Questions}$$

For Fe415 $M_{ulim} = 0.138\ f_{ck}\ bd^2$

$$= 0.138 \times 20 \times 250 \times 365^2$$

$$= 91.925\ kN\text{-}m$$

$$M_{ulim} < M_u \Rightarrow M_{ur} = 3 \wp \sqrt{3m}\ dc\ \text{doubly reinforced section}$$

$$MOR < M_{max} \Rightarrow \text{Beam has to be designed as doubly reinforced}$$
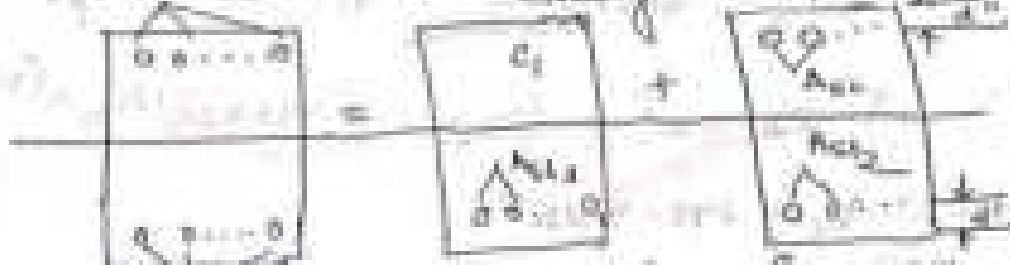
Step 5(v) Calculation of Area of Steel

(a) For Tension = A_{st_1} to provided to balance compression c_1 from Tension

$$M_{OR} = 0.87 f_y A_{st}(d - 0.42 x_u), \quad T_u = ?$$

$$\Rightarrow 91.925 \times 10^6 = 0.87 \times 415 \times A_{st_1}(d - 0.42 x_u)$$

For Fe415, $x_u = 0.48 d$
$$= 0.48 \times 365$$
$$= 175.2 mm$$

$$\Rightarrow 91.925 \times 10^6 = 0.87 \times 415 \times A_{st_1}(365 - 0.42 \times 175.2)$$

On solving, $A_{st_1} = 893.60 mm^2$

(b) For Remaining moment

$$M_{remain} = 131 \cdot M - 91.925$$
$$= 39.22 kN \cdot m$$

For A_{st_2}:
$$M_R = 0.87 f_y A_{st_2} (d - d')$$

$$\Rightarrow 39.22 \times 10^6 = 0.87 \times 415 \times A_{st_2} \times (365 - 35)$$

$$A_{st_2} = 309.132 mm^2$$

For A_{sc}: c_2 = compression force
$$M_R = (f_{sc} - f_{cc}) A_{sc} (d - d')$$

For f_{sc} (Is456 - Table F) Fe415

$$\frac{d'}{d} = \frac{35}{365} = 0.095 \approx 0.1$$

$$f_{sc} = 353 N/mm^2 \qquad f_{cc} = 0.446 f_{ck}$$

So,
$$39.22 \times 10^6 = (353 - 0.446 \times 20) A_{sc} (365 - 35)$$

$$\Rightarrow A_{sc} = 348.4 mm^2$$

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

$$= 329 \cdot 186 + 873 \cdot 68$$

$$= 1202 \cdot 88 \text{ mm}^2$$

Reinforcement:

(a) For tensile:

Provide bar size $\phi = 20\text{ mm}$

$$A_{\phi d} = \frac{\pi \times 20^2}{4} = 314 \cdot 159 \text{ mm}^2$$

No. of bars $= \dfrac{A_{st}}{(A_{st})_{individual}}$

$$= \frac{1202 \cdot 88}{314 \cdot 159} \approx 3 \cdot 8 \simeq 4 \text{ nos}$$

$$A_{st \text{ provided}} = 314 \cdot 159 \times 4 = 1256 \cdot 64 \text{ mm}^2$$

(b) For compression:

$$\phi = 16\text{ mm}$$

$$(A_{sc})_{individual} = \frac{\pi}{4} \times 16^2 = 201 \cdot 06 \text{ mm}^2$$

No. of bars $= \dfrac{A_{sc}}{(A_{sc})_{individual}}$

$$= \frac{245 \cdot 365}{201 \cdot 06}$$

$$= 1 \cdot 2 \simeq 2 \text{ nos}$$

$$A_{sc \text{ provided}} = 2 \times 201 \cdot 06 = 402 \cdot 12 \text{ mm}^2$$

Step 4 (iv) Check for reinforcement:

$$A_{st \, min} = \frac{0 \cdot 85 \, b d}{f_y}$$

$$= \frac{0 \cdot 85 \times 250 \times 365}{415}$$

$$= 186 \cdot 84 \text{ mm}^2 < A_{st \, provided} \; (o.k)$$

$A_{sc,max}$ = 4% of gross area

$= 0.04 \times b \times D$

$= 0.04 \times 250 \times 400$

$= 4000 \, mm^2 > 1286.626 \, (o.k)$

for compression

$(A_{sc})_{max} = 0.04 \, bD$

$= 0.04 \times 250 \times 400$

$= 4000 \, mm^2 > 412.12 \, mm^2 \, (o.k)$

Step-8 (a) Reinforcement Detail
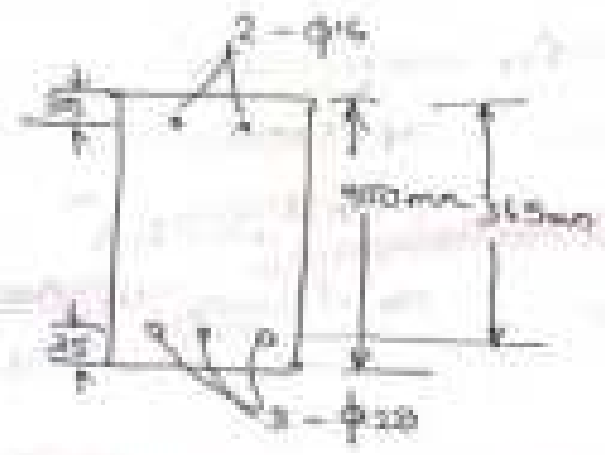
Ch-4   Shear Reinforcement

Q.1 A R.C.C. beam 250mm × 400mm effective d carrying a uniformly distributed load of 15 kN/m. A beam is reinforced with 4 nos of 22mm diameter bars. The clear span of the beam is 4m. Design the shear reinforcement. Use M20/Fe250 i.e. mild steel.

Soln:

- Given:
  - $b = 250$ mm
  - $d = 400$ mm
  - $w = 15$ kN/m
  - $L = 4$ m
  - $A_{st} = 4 \times \frac{\pi}{4} \times 22^2 = 1520.5 \, mm^2$
  - $f_{ck} = 20 \, N/mm^2$ & $f_y = 250 \, N/mm^2$

Step-1   Find out factored Shear force

$$Load = 15 \, kN/m$$
$$Factored \; Load = 1.5 \times 15 = 22.5 \, kN/m$$
$$Factored \; Shear \; Force = \frac{wl}{2}$$
$$= \frac{22.5 \times 4}{2}$$
$$= 45 \, kN$$
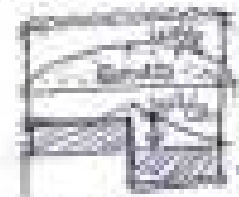
Step-2   Nominal Shear Stress τ_v

$$\tau_v = \frac{V_u}{bd}$$
$$= \frac{45 \times 10^3}{250 \times 400}$$
$$= 0.45 \, N/mm^2$$

$\tau_{c\,max}$ for M 20 = 2.8 N/mm² [IS 456 Table 20]

$\tau_v < \tau_{c\,max} \rightarrow$ sec^n safe (OK)

& if $\tau_v > \tau_{c\,max} \rightarrow$ sec^n unsafe → Redesign the sec^n

Step-3 <u>Design shear strength of concrete (τ_c)</u>

It depends on P_t % & Grade of concrete

i) $P_t = \dfrac{A_{st}}{bd} \times 100$

$= \dfrac{1526.5}{250 \times 447} \times 100$

$= 1.3\%$

& M20

$\Rightarrow \tau_c = 0.72 \, \%_{mm^2}$

(refer IS)

$\tau_c > \tau_v \Rightarrow$ No shear reinforcement is require
but IS code recommends that
"nominal shear reinforcement" ma
be provided.

Step-4 <u>Nominal Shear Reinforcement</u>

$\dfrac{A_{sv}}{b \, s_v} \geq \dfrac{0.4}{0.87 \, f_y}$ (26.5.1.6)
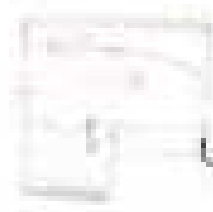
Using 8mm ϕ bar & 2 legged stirrups

$A_{sv} = \underset{leg}{2} \times \dfrac{\pi}{4} \times 8^2$

$= 100.5 \, mm^2$

$\Rightarrow S_v = \dfrac{A_{sv} \times 0.87 \, f_y}{0.4 \times b} = \dfrac{100.5 \times 0.87 \times 250}{0.4 \times 250}$

$= 218 \, mm$

Step-5 <u>Check for spacing of shear reinforcement</u>

Use Vertical Stirrups
Spacing should be least of-

* Vertical / bent-up depend
on us to choose
type of shear rft

$S_v^{min} = \text{of} \begin{cases} ⓐ & 0.75d = 0.75 \times 447 = 335 mm \\ ⓑ & 300 mm \\ ⓒ & S_v = 218 mm \end{cases}$

∴ Provide Size of bar, 2 legged stirrup @ 210mm
spacing c/c through out the length of beam.

Q2. A simply supported R/C beam, 250mm wide &
450mm deep (effective) is reinforced with a
4-18mm diameter bars. Design the Shear reinforced
if M20 grade of concrete & Fe415 steel is used
& beam is subjected to a shear force of 150kN at
service load.

Solⁿ:

Given_

$b = 250\,mm$

$d = 450\,mm$

$A_{st} = 4 \times \dfrac{\pi}{4} \times 18^2 = 1018\,mm^2$

$f_{ck} = 20\,N/mm^2$, $f_y = 415\,N/mm^2$, $V = 150\,kN$

Step-1   **Factored Shear Force**

$$V_u = 1.5\,V$$

$$= 1.5 \times 150$$

$$= 225\,kN$$

Step-2   **Nominal Shear Stress τ_v**

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

$$= \frac{225 \times 10^3}{250 \times 450}$$

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

$\tau_{c,max} = 2.8\,N/mm^2$ for M20 (IS456 - Table 20)

$\tau_{c,max} > \tau_v \Rightarrow$ Hence (O.K.)

Step-3   **Design Shear Strength of concrete (τ_c)**

τ_c depends on ① P_t% &

② Grade of concrete

(IS456 Table 19)

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

$$= \frac{1018}{250 \times 450} \times 100$$

$$= 0.9\%$$

M20

From table - 19, Interpolation

$$
\begin{array}{lcl}
0.75 & \text{------} & 0.56\ N/mm^2 \\
\text{\textcircled{0.9}} & \\
1 & \text{------} & 0.62\ N/mm^2
\end{array}
$$

$$\tau_c = 0.56 + \frac{0.62 - 0.56}{1 - 0.75}\left(0.9 - 0.75\right)$$

$$= 0.596\ N/mm^2$$

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

compare $\tau_v > \tau_c$, hence shear reinforcement is to be provided.

Step - IV **Design of Shear Reinforcement**

Shear taken by stirrups = V_{us} = design shear force

$$V_{us} = V_u - V_c$$

$$= \text{factored} - \text{Shear resisting}$$
$$\quad\quad\quad \text{shear force} \quad\quad\quad \text{capacity of sec}^n$$

$$= V_u - \tau_c \cdot bd$$

$$= 225000 - 0.59 \times 250 \times 450$$

$$= 157750\ N$$

Per vertical stirrups - 8mm ϕ bar 2 legged

$$V_{us} = 0.87 f_y \frac{A_{sv} \cdot d}{S_v} \quad\quad A_{sv} = \frac{\pi}{4} d^2$$
$$\quad\quad\quad = 100$$

∴ $S_v = 103\ mm$

Spacing of Nominal Reinforcement :

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

$$\Rightarrow S_v = 362mm$$

step - V   Check for Spacing

For vertical Stirrups, spacing should be
minimum of following :

$$= \min \begin{cases} \text{(i)} & 0.75d = 0.75 \times 450 = 337mm \\ \text{(ii)} & 250mm \\ \text{(iii)} & S_v = 103mm \quad \text{from design s/e} \\ \text{(iv)} & S_v = 362m \quad \text{from nominal s/e} \end{cases}$$

$$= 103mm$$

∴ Provide 2 legged Stirrups of
  8mm ⌀ bar @ 103 mm spacing c/c.

Ch-5 Analysis & Design of T-Beam

effective flange width

Flange

depth of flange/
thickness of flange

effective
depth

dw — depth of web

width of web/rib
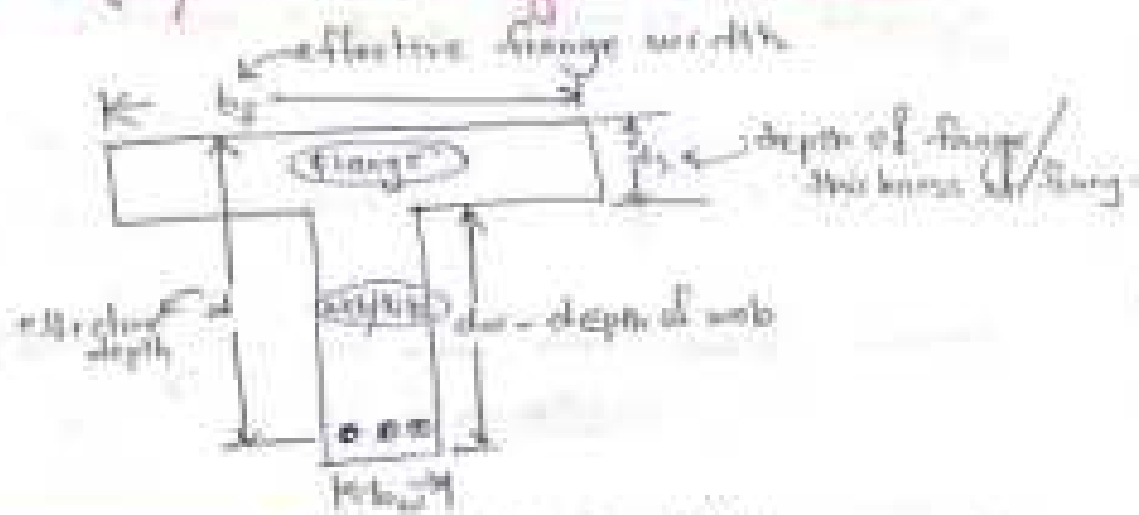
[Effective flange Width] **Type-1 Numerical**

Case-1 Intermediate T-beam

Eqns for Beam casted monolithic with slab

$$\text{①} \quad b_f = \frac{l_o}{6} + b_w + 6\, d_f$$

$$\text{②} \quad b_f = \frac{l_1}{2} + \frac{l_2}{2} + b_w$$

$$\text{min}^m \text{ of ① & ②} = b_f$$

Case-2   Isolated T-beam

$$b_f = \frac{l_o}{\frac{l_o}{b} + 4} + b_w$$

Moment of Resistance : Type-II numerical

Case-I   Case-II   Case-III

Case-I
→ NA lies in flange   ※ NA lies in web + part in flange
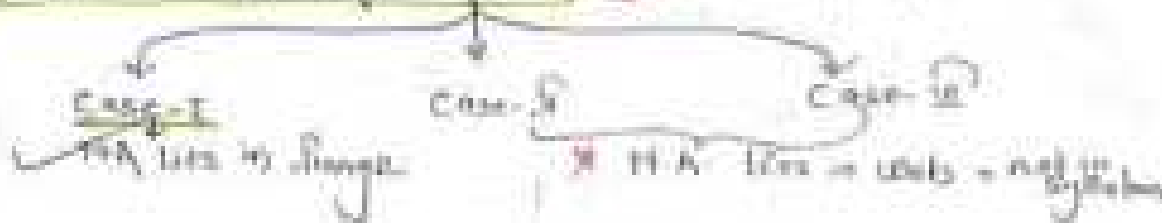
Case-I   When NA lies in flange section

$$x_u < d_f$$
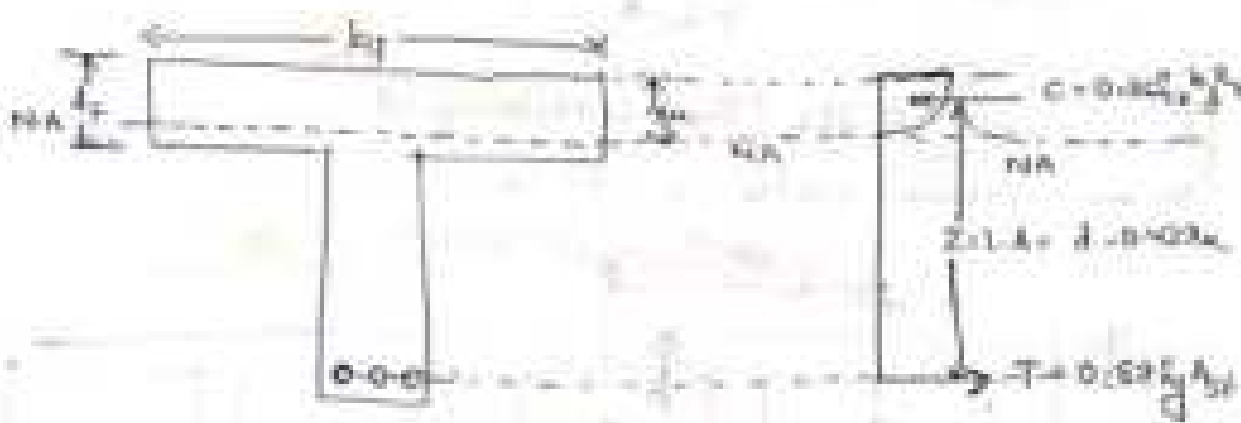
· Actual Depth of NA

$$C = T$$

$$\Rightarrow 0.36 f_{ck} \, b_f \, x_u = 0.87 f_y A_{st}$$

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

· Critical depth

$$Fe\ 250 \longrightarrow x_{u,max} = 0.53 d$$
$$Fe\ 415 \qquad\qquad\quad = 0.48 d$$
$$Fe\ 500 \qquad\qquad\quad = 0.46 d$$

Find:-   D/R , O/R or B/R sec[?]

Moment of Resistance

O/R : $MR_c = C \times LA$
$$= 0.36 f_{ck} \, b_f \, x_u \, (d - 0.42 x_u)$$

U/R : $MR_T = T \times LA$
$$= 0.87 f_y A_{st} (d - 0.42 x_u)$$

Ch-6 Analysis & Design of Slab & Staircase

- Slabs are of 2 types:

① One-way slab
- spanning in one direction
- Tension Reinforcement provided only in shorter span
- Deflects in 1 dir"

② Two-way slab
- Spanning in two direction"
- Provided in both direction"
- Slab deflects in both dir"

* Aspect Ratio of slab determines the type of slab.

$$A_r = \frac{\text{Longer Span}}{\text{Shorter Span}} = \frac{l_y}{l_x} > 2 \quad \text{(One way slab)}$$

$$\frac{l_y}{l_x} \leq 2 \quad \text{(Two way slab)}$$

Q. Design a slab of size 3m × 6.2m for a living room a residential building. Take floor finish as 1.5 kN. Use M20 concrete & Fe415 steel.

$$\text{Aspect Ratio} = \frac{l_y}{l_x} = \frac{6.2}{3} = 2.067$$

$$\frac{l_y}{l_x} > 2 \Rightarrow \text{One way slab}$$

Step-1 Depth of slab (init deflection)

$$\frac{l}{d} = 20 \quad \text{or} \quad d = \frac{\text{Span (shorter)}}{\text{Basic Value} \times \text{Modification Factor}} \quad \text{Cl. 23.2.1}$$

$$\frac{l}{d} = 30000$$

$$= \frac{3000}{(20) \times 1.5 \text{ (assume)}}$$

$$= 100 \text{ mm}$$

$$\leq 12.5 \text{ mm}$$

$$D = 125 + 5 + 20$$

$$= 150 \, mm$$

Step-II **Effective span of slab** (cl. 22.2)

$$L_{eff_1} = \text{Shorter span} + \text{eff. depth}$$

$$= 3m + 0.125m$$

$$= 3.125m$$

$$L_{eff_2} = \text{Span of slab} + \text{size of end support}$$

$$= 3 + 0.3$$

$$= 3.3 m$$

$$L_{eff} = \text{min}^{n} \text{ of } \begin{cases} \text{(i) } L_{eff_1} \\ \text{(ii) } L_{eff_2} \end{cases}$$

$$\boxed{L_{eff} = 3.125m}$$

Step-III **Load Calculation**

$$\text{Dead load} = \gamma_c \times D = 25 \times 0.15 = 3.75 \, kN/m^2$$

$$\text{Floor finish} = \quad 1.5 \, kN/m^2$$

$$\text{Live load} = \text{Assume} \quad 4 \, kN/m^2$$

Load per metre $= 9.25 \, kN/m$ Total = $9.25 \, kN/m^2$

$$\Rightarrow w = 9.25 \, kN/m$$

Step-IV $M = \dfrac{wl^2}{8}$ **Calculation of Moment**

$$= \dfrac{9.25 \times 3.125^2}{8}$$

$$= 11.287 \, kN \cdot m$$

$$M_d = 1.5 \times M$$

$$= 16.93 \, kN \cdot m$$

(BFD)
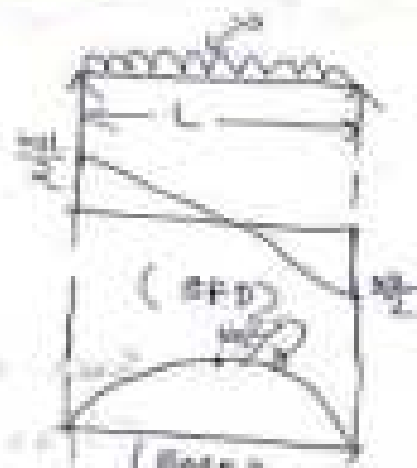
Step-V (Checks for Depth
(Calculate Depth of Slab using Bending Moment)
Consideration)

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

$$x_{u,max} = 0.48 \, d \quad \text{for} \quad Fe415$$

Assume $f_{ck} = 20$ MPa

$$\Rightarrow 16.93 \times 10^6 = 0.36 \times 20 \times 0.48 \, d \times 1000$$
$$\times (d - 0.42 \times 0.48 \, d)$$

$$\Rightarrow d = 78 \, mm < 125 \, mm \quad (\text{Safe})$$

Step-VI   Calculation of Steel Area

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

$$= \frac{0.5 \times 20}{415} \left[1 - \sqrt{1 - \frac{4.6 \times 16.93 \times 10^6}{20 \times 1000 \times 125^2}} \right]$$
$$\times 1000 \times 125$$

$$= 402.16 \, mm^2$$

$$\text{Spacing} = \frac{\text{Area of 1 bar}}{\text{Area reqd}} \times 1000 \quad \text{width}$$
per unit width of slab

$$= \frac{\frac{\pi}{4} \times 10^2}{402.16} \times 1000$$

$$= 195 \, mm \quad (\text{Safe})$$

Cl. 26.3.3
$$\text{Spacing} < \begin{cases} \text{①} \; 3d = 3 \times 125 = 375 \\ \\ \text{②} \; 300 \, mm \end{cases}$$

$$195 \, mm \; (O.K)$$

Cl. 26.5.1.1
$$A_{st,min} = \frac{0.85 \, bd}{f_y}$$

$$= \frac{0.85 \times 1000 \times 125}{415}$$

$$= 256 \, mm^2$$

$$A_{st,max} = 0.04 \, bD)$$

A_{st} provided $= 402 \cdot 16 \, mm^2$ (O.K.)

Part - B   Checks

Deflect^n check   Development   Shear
                        length check   check

① Deflect^n check   Cl 22·2·1

$$f_s = 0.58 f_y \left[\frac{A_{st \, req}}{A_{st \, prov}} \right] \quad Cl \; 23·2·1$$

depends on change of spacing

$$= 0.58 \times 415 \times \frac{402 \cdot 16}{402 \cdot 16}$$

$$= 240.7 \, N/mm^2$$

$$P_t = \frac{A_{st}}{b \cdot d} \times 100$$

$$= \frac{402 \cdot 16}{1000 \times 125} \times 100$$

$$= 0.197$$

Use Graph :   MF = 1.7 (modification factor)

$$d = \frac{Span}{20 \times M.F.}$$

$$= \frac{3050}{20 \times 1.7} = 89 \cdot 82 \, mm < 115 \, mm \; (grt)$$

if greater than than slab it not safe for deflect^n

② Check for Shear   (Cl - 40·2·1·1)

Shear $V = \frac{w_u L}{2} = \frac{9.25 \times (3)}{2}$

Here, we have to take coefficient of w_u (for shear force)

$$V_u = 1.5 \, V$$

$$= 1.5 \times$$

$$= 20.81 \, kN$$

$$\text{Nominal shear stress} \quad \tau_v = \frac{V_d}{b d} = \frac{20.81 \times 10^3}{1000 \times 125} = 0.16 \, N/mm^2$$

$$P_t = 0.19\% \text{ for central part of slab}$$

design shear stress $\tau_c = 0.28 \, N/mm^2$

$$\left\{ P_t = \frac{1}{2} \times \text{central, for support its half} \right.$$
$$= \frac{1}{2} \times 0.19\%$$
$$P_t = 0.095\%,$$

$$M25$$

For 150mm slab $K = 1.3$ Cl. 40.2.1.1

$$K\tau_c = 1.3 \times 0.28$$
$$= 0.364 \, N/mm^2$$

$$\tau_v < K\tau_c \quad (\text{safe in shear})$$

③ **Check for Development length** Cl 26.2.3.3c

$$\frac{M}{V} + L_o > L_d \quad \text{or} \quad \frac{1.3 M}{V} + L_o > L_d$$

$M \rightarrow$ Moment
$V \rightarrow$ Shear force
$L_o \rightarrow$ length available from the centre of support to the face of support
$L_d \rightarrow$ development length   Cl 26.2.1

$$L_d = \frac{\phi \left(0.87 f_y \right)}{4 \tau_{bd}} \quad \substack{\leftarrow \text{bond stress} \\ \text{from table } 3.2.45}$$

$$L_o = \min \left\{ \begin{array}{l} d = 125 \\ 12\phi = 120 \end{array} \right.$$

$$= \frac{10 \times 0.87 \times 415}{4 \times 1.2 \times 1.6} \quad \leftarrow \text{increased by 60\%}$$

$$= 470.11 \, mm$$

$$A = 8.34$$

$$\frac{16.45 \times 10^6}{20.15 \times 10^3} + L_o = 388.490 > L_d$$
$$(\text{assume})$$

$$\therefore \text{safe in } L_d$$

Last Step - Distributed bar Steel Area.

Area of Steel Reqd to "Distributed" bar

$\left(\begin{array}{l} \text{fe250} = 0.15\% \ bd \\ \\ \rightarrow \text{fe415/fe500} = 0.12\% \ bd \end{array} \right.$

$A_{st_d} = \dfrac{0.12}{100} \times bd$

$= \dfrac{0.12}{100} \times 10^3 \times 125$

$= 150 \ mm^2$

Assume Provide bar

Spacing
for 8ϕ

$= \dfrac{\text{Area of 1 bar}}{A_{st_d}} \times 1000$

$= \dfrac{\frac{\pi}{4} \times 8^2}{150} \longrightarrow 1000$

$= 335.10 \ mm$

Check $< \left\{ \begin{array}{l} 5d \\ \\ 450 mm \end{array} \right.$ (OK)

Cantilever Slab < Chajjas
                    Balcony

cond^n ← first span = l_c + d/2
       eff. dep^n 20% to
       1/8 to 1/10 of span
       Ref^l of dep
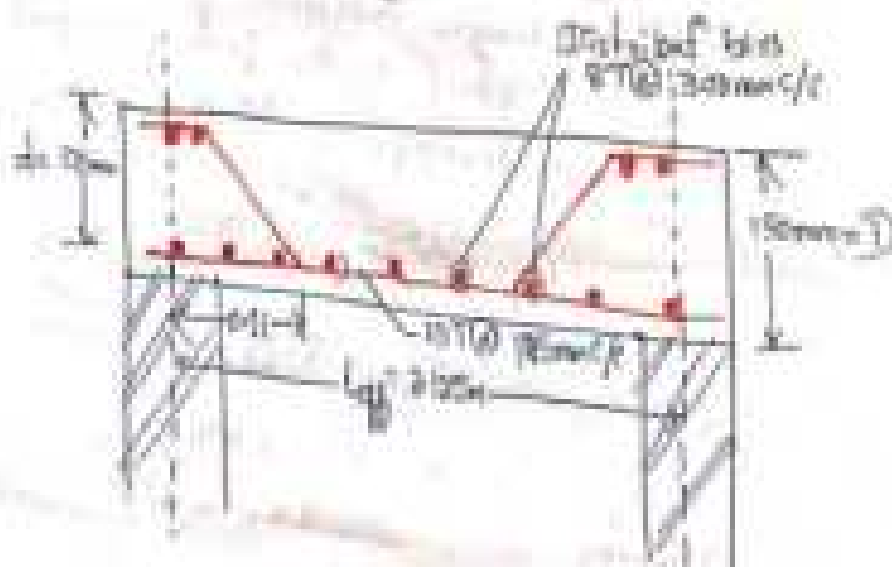
Q. Design a cantilever slab for an overhang 2.1m. The live load on slab consists of 1 KN/m2 & floor finish is 3.0 KN/m2. Use M25 & Fe415

Step-1 Depth of Slab

effective depth = $\left[\dfrac{Span}{10} \text{ to } \dfrac{Span}{12}\right]$ @fixed end

$d = \dfrac{Span}{12} = \dfrac{2100}{12} = 175mm$

$D = d + \text{clear cover} + \frac{1}{2}\,dia$

  $= 175 + 20 + \frac{1}{2} \times 10$

  $= 200mm$

Step-2   Effective Span

Effective span = $\dfrac{\text{Effective depth}}{2} + \text{clear span}$

  $= \dfrac{175}{2} + 2100$

  $= 2187.5mm$

  $= 2.18m$
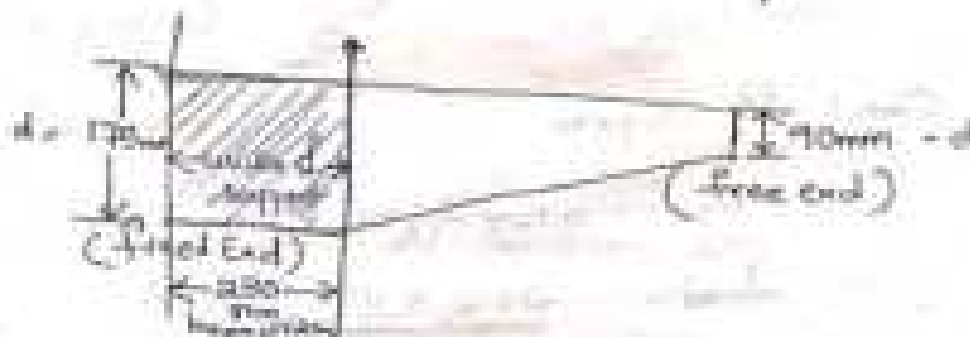
$= \dfrac{1}{2} \text{ to } \dfrac{1}{3} \times \text{effective depth @ fixed end}$

$= \dfrac{1}{2} \times 175$

$= 87.5mm$

$\approx 90mm$

Step-II Total Load Calculation

$$\text{Self weight of slab} = \gamma_{conc} \times 0.2 \times 1$$

$$= 25 \times 0.2 \times 1$$

$$= 5 \, kN/m$$

$$L.L = 1 \, kN/m^2 = 1 kg/m^2 \times 1m = 1 kg$$

$$F.F = 0.8 \, kN/m^2 = 0.8 kg/m^2 \times 1 = 0.8 kg$$

Working Load Total $= 6.8 \, kN/m$

Factored Load $= 1.5 \times 6.8 = 10.2 \, kN/m$

Step-IV Max. B.M & max S.F.

$$\text{Max}^m \, BM = \frac{wl^2}{2}$$

$$M_u = \frac{10.2 \times 2.15^2}{2}$$

$$= 24.2 \, kg \cdot m$$

$$\text{Max}^m \, SF = w \left(L \right) \quad \text{clear span}$$

$$= 10.2 \times 2.1$$

$$= 21.42 \, kN$$

Check for depth

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

$$\Rightarrow d = \frac{24.2 \times 10^6}{}$$
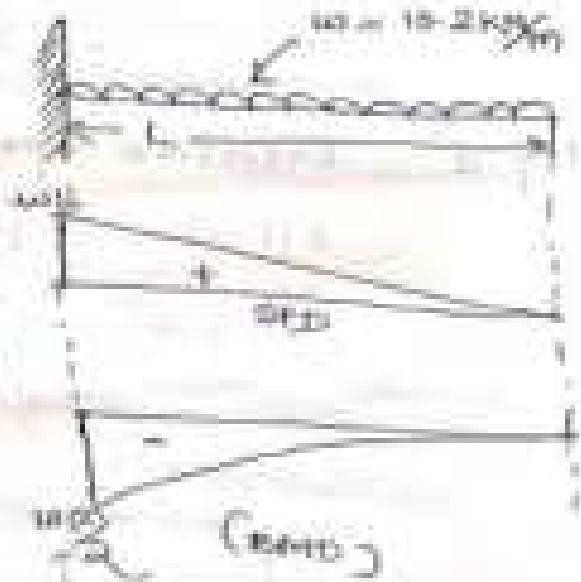
Step. V Check for Depth
$$M = 0.36 f_{ck} \cdot \frac{x_{u,max}}{d} \cdot b \cdot (d - 0.42 x_{u,max})$$

$$\Rightarrow 24.298^6 = 0.36 \times 25 \times 0.48 d \times b \left(d - \overline{0.42 \times 0.48 d}\right)$$

$$\Rightarrow 24.298^6 = 0.36 \times 25 \times 0.48 d^2 \times 1000 \left(1 - \overline{0.42 \times 0.48}\right)$$

$$\Rightarrow 24.298^6 = 3449.02 \, d^2$$

$$\Rightarrow d = 83.76 mm < 175 mm^{max} \text{ or } 90 mm^{min} \quad (O.k)$$
$$\text{consider both value} \quad Safe$$

Step. VI Calculation of steel Area
$$A_{st} = 0.5 \frac{f_{ck}}{f_y} \left[1 - \sqrt{1 - \frac{4.6 M}{f_{ck} b d^2}}\right] b d \quad 175$$

$$= \frac{0.5 \times 25}{415} \left[1 - \sqrt{1 - \frac{4.6 \times 24.298^6}{25 \times 1000 \times 175^2}}\right] 175 mm$$

$$= 385 mm^2 \quad A_{st \, req\, d}$$

$$A_{st \, min} = 0.12\% \, b D \rightarrow \text{for Distribution steel of } f_y 415$$

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

$$= 240 mm^2 < A_{st} \quad (O.k)$$

Take ϕ 8mm, $a_{st} = \frac{\pi}{4} \times 8^2 = 50.3 mm^2$

$$\text{Spacing} = \frac{a_{st}}{A_{st}} \times width$$

$$= \frac{50.3}{385} \times 1000$$

$$= 130.64 mm \approx 130 mm$$

Provide ϕ 8mm @ 130mm c/c

$$A_{st} \, Provided = \frac{a_{st}}{Spacing} \times 1000 \quad width$$

$$= \frac{50.3}{130} \times 1000$$

Check for Spacing

$$\text{min}^{n}\ S \begin{cases} ① & 3d \; , \; 3 \times 125 = 525\ mm) \\ & 3 \times 70 = 270\ mm \\ ② & 300\ mm \end{cases}$$

$$= 270\ mm \quad ☞$$

Spacing 150 mm < 270 mm (O.k)

"Bar as main reinforcement & distribution reinforcement
8 mm #"

Step-VI Check for Shear

$$V_u = 21.42\ KN$$

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

$$= \frac{21.42 \times 10^3}{1000 \times 185}$$

$$= 0.12 \quad N/mm^2$$

$$\tau_c \searrow \begin{array}{c} P_t = \dfrac{A_{st}}{bd} \times 100 = \dfrac{100 \times 286.9}{1000 \times 185} \\ \searrow M25 \\ \text{Table 19} \end{array} = 0.221\%$$

P_t	τ_c
0.15	0.29
0.221 ——————— ???	
0.25	0.36

By Interpolation $\tau_c = 0.349\ N/mm^2$

$$\therefore \tau_c > \tau_v \quad (o.k)$$

No Shear R/F Reqd

Step - VII Check for deflection

Max^n dcltn^n $\left(\dfrac{l}{d}\right)_{max}$ = $7 \times MF(2.9476)$

$$f_s = 0.58 \ f_y \ \dfrac{A_{st \ reqd}}{A_{st \ prov}}$$

$$= 0.58 \times 415 \times \dfrac{285}{386.72}$$

$$= 239.5 \ N/mm^2$$

From Graph,

$$M.F = 2$$

$$\left(\dfrac{l}{d}\right)_{max} = 7 \times 2$$

$$= 14 \ mm$$

$$\left(\dfrac{l}{d}\right)_{provided} = \dfrac{Clear \ Span}{eff. \ depth}$$

$$= \dfrac{2100}{175}$$

$$= 12 \ mm$$

$$\left(\dfrac{l}{d}\right)_{max} > \left(\dfrac{l}{d}\right)_{prov} \quad \text{———— (s.k.)}$$

$$D = d + cover + \dfrac{\phi}{2}$$

$$= 90 + 25 + \dfrac{10}{2}$$

Main Reinforcement :- At Top of slab in case of cantilever beam

Development length

$$L_d = \dfrac{0.87 f_y \ \phi}{4 \ \tau_{bd}} \qquad \underset{PETIS \to}{\overset{IS}{\bigodot}} \to 14$$

$$= \dfrac{0.87 \times 415 \times 8}{4 \times 1.4 \times 1.6}$$

$$= 322 \ mm$$

$h = b d^2 \, 2a - 25 = 210mm$

$1 \times b d^2 \, 1d, \dfrac{1}{150mm}$

$\text{Provide } L_d = 250mm \quad \text{to the support}$

Q. Design a RCC slab for a room 6.3m × 4.5m. The slab to be cast monolithically over the beams with its sides simply supported. It has to be carry a characteristic load of 10kN/m² in addition to the own weight. Use M20 concrete & Fe415.

$$\text{Aspect Ratio } \lambda_y = \dfrac{l_y}{l_x}$$

$$= \dfrac{6.3}{4.5}$$

$$= 1.4 \le 2 \, (\text{Two way slab})$$

Step 1 Depth of Slab

$$f\left(\dfrac{l}{d} = \alpha \times \beta\right) \, f_y = 415$$

$$d = \dfrac{Span \, (Shorter)}{\underset{\text{Basic value}}{(20)} \times \underset{\text{Modification factor } [1.2 - 1.3]}{MF}}$$

$$= \dfrac{4500}{20 \times 1.8}$$

$$= 150mm \le 171mm \quad \text{Take greater value}$$

effective cover = 25mm

$$D = 175 + 25 = 200mm$$

Step-II Span of the slab
cl. 22.1.a → Pg 34

$= \min \text{ of } \begin{cases} \ell_{eff} = \ell_x + \text{depth} = 4500 + 175 = 4675m \\ \ell_{eff} = \ell_x + \text{s/c between support} (\\ \ell_{eff} = 4.675m \end{cases}$

Step-III Load Calculation

$DL = 25 \times 0.2$
$= 5 \times 5$
$= 5\ KN/m^2$

$LL = 10\ KN/m^2$

_____

Total $= 15\ KN/m^2$ $= 15\ KN/m$
       (1m wide)

Factored load $= 1.5 \times 15 = 22.5\ KN/m$

Step-IV Calculation of Moments (Way 9 B-2)

$\alpha_x = 0.099$
$\alpha_y = 0.051$

$M_x = \alpha_x w \ell_x^2$ $M_y = \alpha_y w \ell_x^2$

$= 0.099 \times 22.5 \times 4.675^2$ $= 0.051 \times 22.5 \times 4.675^2$

$= 48.68\ KN\cdot m$ $= 25.07\ KN\cdot m$

Design for Max^m moment
$M_d = 48.68\ KN\cdot m$

Step-V Check for depth
$M_d = 0.36 f_{ck} x_{u,max} b(d - 0.42 x_{u,max})$

$\Rightarrow 48.68 \times 10^6 = 0.36 \times 25 \times 0.48 d \times 1000$
$(d - 0.42 \times 0.48d)$

$\Rightarrow d = 118 mm < 175 mm \quad (o.k.)$

Step-10 : Calculation of steel in shorter span

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

$$A_{st_x} = \frac{0.5 \times 25}{415} \left[1 - \sqrt{1 - \frac{4.6 \times 48.67 \times 10^6}{25 \times 1000 \times 155^2}} \right]$$

$$= 500 \, mm^2$$

Let, 12ϕ

$$Spacing = \frac{Area \, of \, 1 \, bar}{A_{st \, req}} \times 1000$$

$$= \frac{\frac{\pi}{4} \times 12^2}{500} \times 1000$$

$$= 219 \, mm$$

$$= 140 \, mm \, < \, min^d \begin{cases} 3d = 465 \\ 300 \end{cases}$$

check 3d≤300

Provide 12ϕ @ 140mm %c   (o.k)

Step-11 : Calculation of steel in longer span

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

$$= \frac{0.5 \times 25}{415} \left[1 - \sqrt{1 - \frac{4.6 \times 25 \times 10^6}{25 \times 1000 \times 143^2}} \right]$$

$$= 1000 \times 143$$

effective depth in y dir^n = 200 - 5 - 4 - 20 = 143 mm

bottom of slab

$$= 418 \, mm^2$$

Adt 12φ, Spacing $= \dfrac{\frac{\pi}{4} \times 12^2}{445} \times 1000 = 253\,mm$

$\simeq 250\,mm$

Provide 12φ @ 250mm c/c

* Provide Torsion R/F at the corner to prevent slab from rising up from the support.
Provision of Torsion R/F

Fig 90 (D - 8)

Ast provision $= \dfrac{3}{4} A_{st,x}$

$= \dfrac{3}{4} \times 680$

$= 505\,mm^2$

mesh like R/F

Size of mesh $= \dfrac{L}{5}$

$= \dfrac{4675}{5}$

$= 935\,mm$

Use 10mm bar

Spacing $= \dfrac{\frac{\pi}{4} \times 10^2}{605} \times 1000$

$= 130mm$

main steel
12T @ 140mm c/c

Shrinkage steel

#14@200mm

#18@200mm

longer span

12T @ 1cm c/c

#14@200mm

#4 mesh
for Torsion R/F

12T @ 200mm

* All three Chunks are same. One way slab

Design of Dog legged Staircase

- Stair facilitates movements from one level to the another level in a building.

→ Risers : The vertical pieces which are the solid ends to the treads

→ Treads : These are simply the steps you walk on

Design a Dog legged staircase for an office building for a room measuring 3m × 6m. Floor to floor height is 3.5m. Stairs are supported on brickwalls 230mm thick at the end of landings. Use M20 & Fe415

Use: IS 456:2000
IS 875 (Part-2)
SP-16

Step 1: Proportioning of Dimension

∗ width = 3m (Take the minimum one as width of staircase)

Breadth = 6m

- Available width = 3m

Considering 2 flights, Let us assume width of each flight = 1.35m

Space b/w 2 flights = 3000mm − (1350mm × 2)

= 300mm

Floor to floor height = 3.5m

Each flight will have a height = $\frac{3.5}{2}$ = 1.75m

- Assuming height $\pm$ 0.5m

of Riser −150mm (for public building)

Wait [150mm − 175mm]

No. of risers $= \dfrac{1.75m}{0.15m} = \dfrac{ht}{riser}$

$= 11.67$

$\approx 12 \text{ nb}$

$\text{risers} = 12 \times 2.24$
$\text{total tread} = 24 - 1.25$

No. of Treads $= $ No. of risers $- 1$

$= 12 - 1$

$= 11 \text{ no}$

Let, width of each tread $= 300mm$

Total going $= 11 \times 300 = 3300\,mm = 3.3m$

Total distance $= 6m$

width of each landing $= \dfrac{6 - 3.3}{2} = 1.35m = 1350mm$

Step-I : Effective Span

Effective span $=$ clear span $+ \%$ of bearings

$= 6 + \dfrac{0.23}{2} + \dfrac{0.23}{2}$

$= 6.23m$

$= 6230mm$

Step-II : Thickness of waist slab

Thickness $= \dfrac{1}{20}$ of span (approx.) Ref SP 16

$D = \dfrac{6230}{20}$

$\approx 311.5\,mm$ (some higher)

Take $\boxed{D = 325\,mm}$ $d = 300\,mm$

effective cover $= 25mm$

$\boxed{D = 0.325m}$
$d = 0.3m$

Step-IV   Loads

Loads

Stairs load — Landing load

(A) Stairs loading:

weight of waist slab = $\dfrac{R}{T}\sqrt{1 + \dfrac{R^2}{T^2}} \times (25) \times \left(1 = 0.325\right)$ Riser, Tread

$= 0.325\sqrt{1 + \dfrac{(0.15)^2}{(0.3)^2}} \times 25$

$= 9.14 \, kN/m$

weight of steps $= \dfrac{25 R T}{2 T}$

$= \dfrac{25 \times 0.15 \times 0.3}{2 \times 0.3}$

$= 1.875 \, kN/m$

Total D.L = $9.14 + 1.875 = 10.925 \, kN/m$

LL = $5 \, kN/m^2 \times 1m = 5 \, kN/m$ (15.875)

(+)
_____

Total Load = 15.975 kN/m

Factored load = 1.5×15.975

$= 24.5 \, kN/m$

(b) Landing loading:

DL = $0.225 \times 1 \times 25 = 8.125 \, kN/m$

(+) LL = $5 \times 1 = 5 \, kN/m$
_____

Total load = 13.125 kN/m

Factored load = $1.5 \times 13.125 = 17.9 \, kN/m$

Design Moment

$$R_A + R_B = 19.7 \times 1.465 + 24 \times 3.3 + 19.7 \times 1.465$$

$$\Rightarrow R_A + R_B = 156.921 \, kN \quad\text{————}①$$

Taking Moment at B:

$$\Sigma M_B = 0$$

$$\Rightarrow R_A \times 6.23 - 19.7 \times 1.465 \times \left(\frac{1.465}{2} + \frac{3.3}{2} + 1.465\right)$$

$$- 24 \times 3.3 \times \left(\frac{3.3}{2} + 1.465\right) - 19.7 \times 1.465 \times \frac{1.465}{2}$$

$$= 0$$

$$\Rightarrow R_A = 68.5 \, kN \quad\text{————}②$$

$$R_B = 156.92 - 68.5 = 88.41 \approx 68.5$$

Bending Moment at mid Span

$$M_C = 68.5 \times (1.465 + 1.65)$$

$$- 19.7 \times 1.465 \times \left(\frac{1.465}{2} + 1.65\right)$$

$$- 24 \times 1.65 \times \frac{1.65}{2}$$

$$= 111.99 \, kN\text{-}m$$

$$\approx 112 \, kN\text{-}m$$

Step-V : Area of Reinforcement

$$M_u = 0.87 f_y A_{st} d\left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right) \quad \text{Ans} \quad \begin{array}{l} f_y \\ f_{ck} \\ A_{st} \end{array}$$

$$\Rightarrow 112 \times 10^6 = 0.87 \times 415 \times A_{st} \times 345\left(1 - \frac{A_{st} \times 415}{1000 \times 345 \times 20}\right)$$

$$\Rightarrow A_{st} = 917 \, mm^2$$

Take 16 mm ϕ bar

$$\text{Spacing} = \frac{\text{Area of 1 bar}}{\text{total area}} \times \text{width}$$

$$= \frac{\frac{\pi}{4} \times 16^2}{1117} \times 1000$$

$$= 179.9$$

$$\approx 180 \, mm$$

∴ Provide 16 mm ϕ @ 180 mm c/c (main bar)

Distribution steel = 0.12% bD

$$= \frac{0.12 \times 1000 \times 225}{100}$$

$$= 390 \, mm^2$$

Take 10 mm ϕ bar

$$\text{Spacing} = \frac{\text{Area of 1 bar}}{\text{total area}} \times \text{width}$$

$$= \frac{\frac{\pi}{4} \times 10^2}{390} \times 1000$$

$$= 201.2$$

$$\approx 200 \, mm$$

∴ Provide 10 mm ϕ @ 200 mm c/c

Step-II Development length (for counterattack → B.M) of main bar)

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

Cl-26.2.1 , IS456

$$= \frac{0.87 \times 415 \times 16}{4 \times 1.6 \times 1.2} \quad (\tau_{bd} \text{ for } M20)$$

$$= 752 \text{mm}$$

Provide 800mm length of bars at point where L_d is required so that check will not occur due to moment

Plan:

Section:
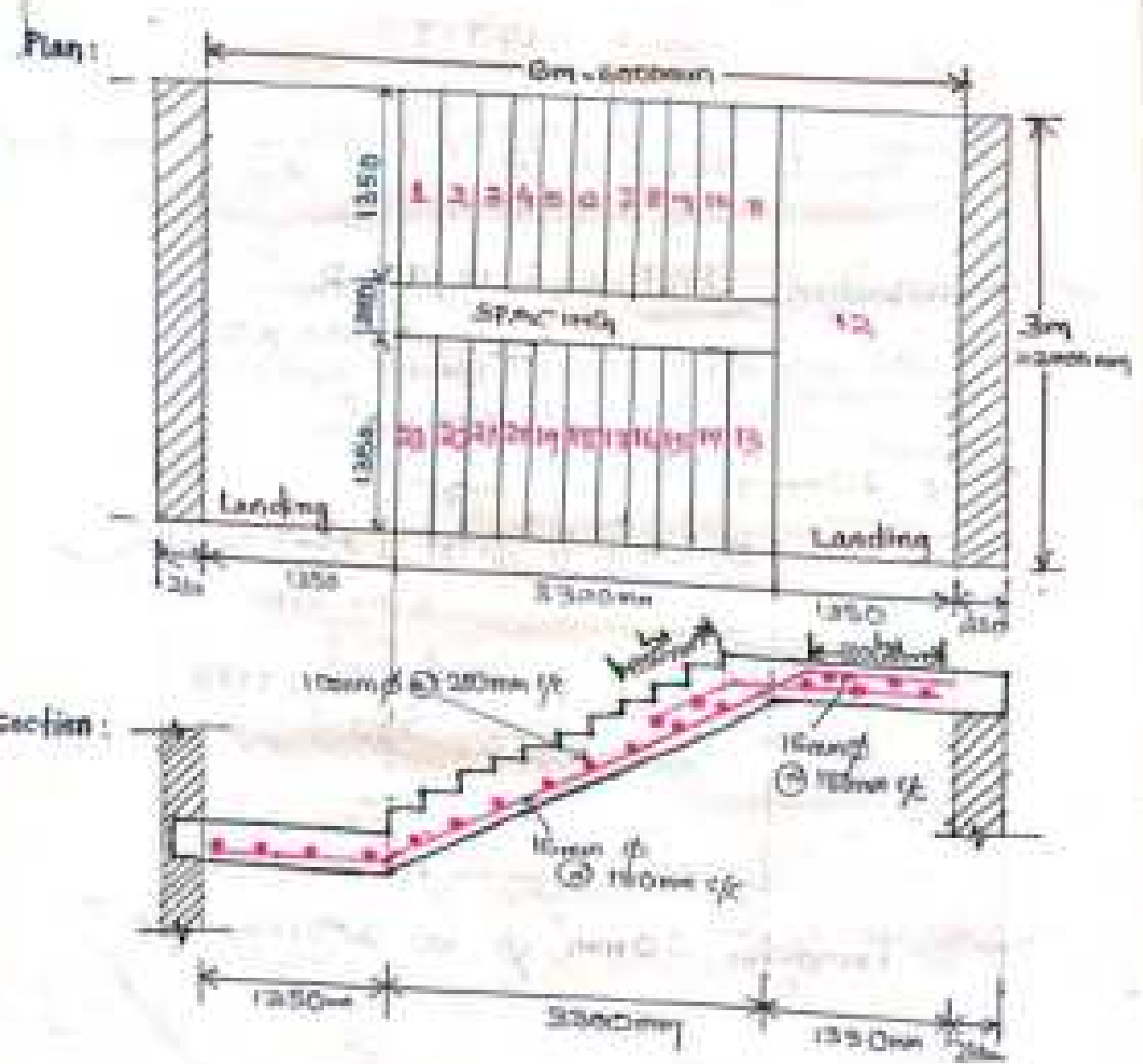
Ch-7   Design of Axially loaded Columns & Footings

Q1 Design a short rcc column to carry an axial load of 1600kN. It is 4m long, effectively held in position & restrained against rotation at both ends. Use M20 & Fe415


```
                    Column
                  /          \
            Short              Long
            Column             Column

         L_eff                  L_eff
         ───── ≤ 12             ───── > 12
          D                      D
```

Given :   L = 4m
          P = 1600kN
          M20, Fe415

Step-1   Effective Length   (Tbl 28 Ann-E, IS456-2000)

$$L_{eff} = 0.65L$$
$$= 0.65 \times 4m$$
$$= 2.6m$$
$$L_e = 2600mm$$

Step-II   Factored Load

$$P_u = 1.5P$$
$$= 1.5 \times 1600$$
$$= 2400kN$$

Step-III   Determine size of Column

$$b^2 = A_g \; ; \; gross \; area \; of \; column$$
$$b = \sqrt{A_g}$$

Let, A_g → Gross area (C + R)
                          Concrete + reinforcement
     A_{sc} → Area of steel reinforcement (s.d.a)
     A_c → Area of concrete

Clause 25.5.3.1   Pg 48 ; Assume 1% steel in
$[0.87 - 6/]$ gross area.

1% of steel in gross Area (Ag)

m% of concrete in Gross Area (Ag)?

$$A_{sc} = \frac{1}{100} \times A_g$$

$$A_c = \frac{99}{100} \times A_g$$

Cl. 39.3 Pg 71.

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

$$\Rightarrow 2400 \times 10^3 = 0.4 \times 40 \times \frac{99}{100} A_g + 0.67 \times 415 \times \frac{A_g}{100}$$

$$\Rightarrow A_g = 224299 \, mm^2$$

$$b^2 = A_g$$

$$\Rightarrow b = \sqrt{224299} = 473.52 mm \simeq 500mm \quad \text{take higher value}$$

b = 500mm

Size of column : 500mm × 500mm   → check short or slender column

Step-iv   Slenderness Ratio   Cl. 25.1.2 Pg 41

$$\frac{l_{eff}}{D \, or \, b} = \frac{2600}{500}$$

   consider longer

$$= 5.2 < 12 \Rightarrow \text{Short column}$$

Step-v e_{min} eccentricity → for axial loading it arises

Cl. 25.4 $e_{min} = \frac{L}{500} + \frac{D}{30}$

   unsupported length

$$= \frac{2600}{500} + \frac{500}{30}$$

$$\Rightarrow 24.87 \, mm$$

$$\frac{e_{min}}{D} < 0.05 \Rightarrow \text{For axially loaded short column}$$

$$= \frac{24.68}{500}$$

$$= 0.049 < 0.05 \quad \text{Axially loaded short column}$$

Step-⑥ Area of Steel (Longitudinal Reinforcement)

$$A_{sc} = 1\% \text{ of } A_g$$

$$= \frac{A_g}{100} \times 224273.79$$

$$= 2243 \text{ mm}^2$$

For column min dia = 16mm

Let 20mm ϕ bar

$$\text{No. of bars} = \frac{2243}{\left(\frac{\pi}{4} \times 20^2\right)} \text{ mm}^2$$

$$= 7.14$$

$$= 8 \text{ nos}$$

$\therefore$ Provide 8 bars of 20mm ϕ

Step-⑦   Lateral Ties or Transverse Reinforcement

Cl 26.5.3.2 Part 5
i) Dia of Lateral tie $= \frac{\text{Dia of longitudinal bar}}{4}$

$$= \frac{20}{4}$$

$$= 5 \text{mm}$$

$$\not> 6 \text{ mm}$$

$\therefore$ Dia using 6mm ϕ ties

<u>Pitch of the ties:</u>

Cl 26.5.3.2 (g "4"

pitch {
(i) least lateral dimension of the
member = 500mm

(ii) 16× 20 = 320mm

(iii) 300 mm

= 300mm

∴ Provide links of 6mm φ @ 300mm c/c

← 500mm →
500mm
20mm φ
longitudinal R/P
6mm φ @ 300mm c/c
Lateral ties
clear cover = 40mm
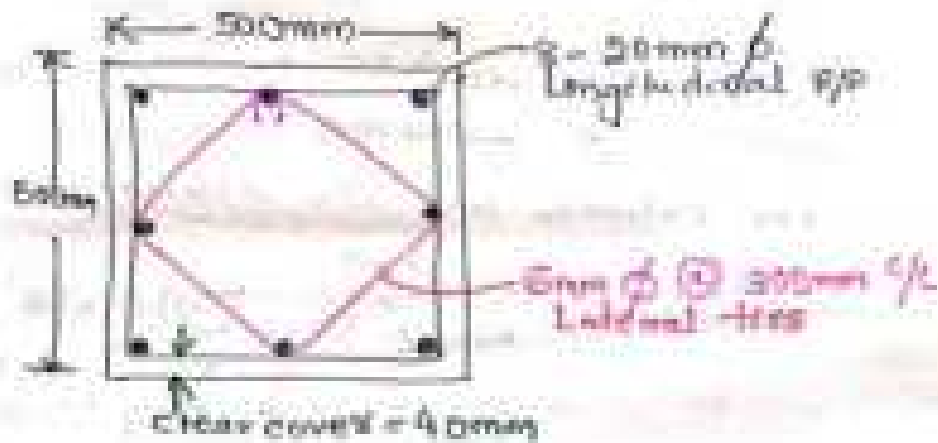
Q2. Design a column of size 450mm × 600mm having
3m unsupported length. The column is subjected to a
Load of 2000kN & is effectively held in position
but not restrained against rotation. Use M20 &
Fe415.

Step-1 Effective length L_eff

$L_{eff} = 1.0 × L$

= 1×3 m

= 3m or 3000mm

Step-5 Stiffness Ratio

$$\frac{l_{eff}}{D} = \frac{3000}{450} = 6.67 < 12$$

∴ Short Column

Step-6 Minimum Eccentricity (e_{min})
& limiting form $e_{min} < \begin{matrix} b \\ D \end{matrix}$

For D = 600mm

$$e_{min} = \frac{L}{500} + \frac{D}{30} \quad cl. 25.4 \cdot Pg. 42$$

$$= \frac{3000}{500} + \frac{600}{30}$$

$$= 26 \, mm > 20mm$$

$$\frac{e_{min}}{D} = \frac{26}{600} \quad cl. 39.3 \cdot Pg. 71$$

$$= 0.043 < 0.05 \quad (O.K.)$$

⇒ Axially loaded

For b = 450mm

$$e_{min} = \frac{L}{500} + \frac{D}{30}$$

$$= \frac{3000}{500} + \frac{450}{30}$$

$$= 21 > 20$$

$$\frac{e_{min}}{D} = \frac{21}{500} = 0.034 < 0.05 \quad (O.K.)$$

⇒ Axially loaded

∴ Axially loaded Member

Step-④ Factored load

$$P_u = P \times 1.5$$
$$= 2500 \times 1.5$$
$$= 3000 \text{ kN}$$

Step.5 Area of longitudinal Reinforcement (Asc)

$$A_g = 450 \times 600$$
$$= 270000 \text{ mm}^2$$

$$\boxed{A_g = A_{sc} + A_c}$$

$$\therefore A_c = 270000 - A_{sc}$$

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \quad \text{Cl. 39.3 Pg 71}$$

$$\Rightarrow 3000 \times 10^3 = 0.4 \times 20 \times (270000 - A_{sc})$$
$$+ 0.67 \times 415 \times A_{sc}$$

$$\Rightarrow A_{sc} = 3116.5 \text{ mm}^2$$

% of reinforcement $= \dfrac{A_{sc}}{A_g} \times 100$

$$= \dfrac{3110.5}{270000} \times 100$$

$$= 1.15\% \quad \text{Cl. 26.5.3.1 (a)}$$
$$\quad Pg 48$$
$$[0.8\% - 6\% A_g]$$

$$[1.15\% - 6\% \text{ (Hence ok)}]$$

$$A_{sc} = 3110.5 \text{ mm}^2$$

Using - 4-25mm ϕ = 4×490.6 = 1962.4 mm^2

4-20 mm ϕ = 4×314 = 1256.0 mm^2

$+$ _____

3218.4 > 2910.5

(O.K.)

∴ Provide : 4-25mmϕ in Longitudinal Reinforcement

4-20 mmϕ ~~bar bars~~

Step-5) Lateral Ties (Transverse Reinforcement)

Cl. 26.5.3-(c) Pg 49

Dia

$\left. \begin{array}{l} \text{(i) } \frac{1}{4} \times 25 = 6.25 \text{ mm} \quad \text{Least lateral dim} \\ \text{(ii) } 6 \text{ mm} \\ \end{array} \right\} \text{max}$ Not available in market so not higher value $\mathbb{I}$.

Use 8 mm ϕ Links

Pitch

$\left. \begin{array}{l} \text{(i) Lateral dimension } 4_{gro} = 450 \text{mm} \\ \text{(ii) } 16 \times 20 = 320 \\ \text{(iii) } 300 \text{min} \\ \end{array} \right\} \text{min}$

= 300 mm

∴ Provide 8 mm ϕ Links @ 300 mm c/c

4-25mm ϕ at corners

4-20mm ϕ at centres

8mm ϕ @ 300 mm c/c

Clear Cover 40mm
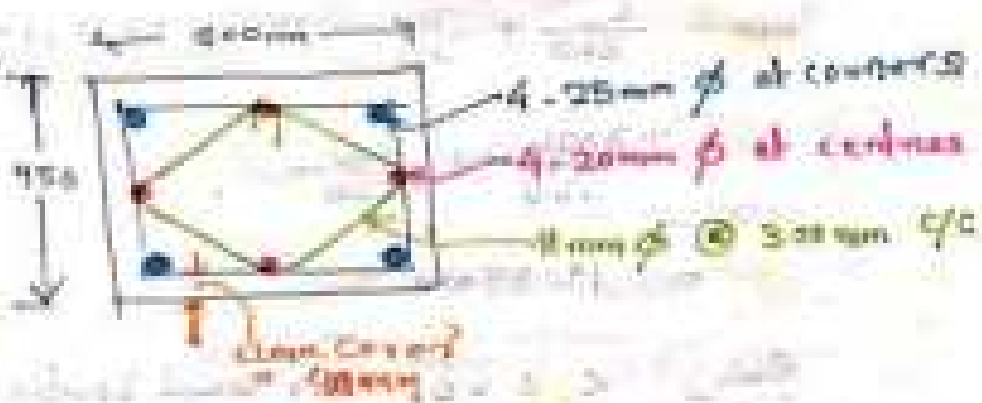
Q. Design a circular column of diameter 400mm subjected to a load of 1200 KN. The column is having spiral ties. The column is 3m long & is effectively held in position at both ends but not restrained against rotation. Use M25 & Fe 415.

Given $D = 400mm$, $P = 1200 KN$, $L = 3m$, M25 Fe415

Step-1 Effective length (L_{eff})

$$L_{eff} = 1.0 \times L \qquad \text{Table 28, Pg 94}$$

$$= 1 \times 3$$

$$= 3m \text{ or } 3000mm$$

Step-2 Slenderness Ratio

$$\frac{L_{eff}}{d} = \frac{3000}{400} = 7.5 < 12 \Rightarrow \text{Short Column}$$

$$Cl.25.1.2 \quad 3^{rd}$$

Step-3 Minimum Eccentricity ← Axial, Biaxial, Uniaxial

$$e_{min} = \frac{L}{500} + \frac{D}{30} \qquad \text{cl 25.4, Pg 42}$$

$$= \frac{3000}{500} + \frac{400}{30}$$

$$= 19.33 mm$$

$$\frac{e_{min}}{D} < 0.05 \quad \text{for axial loading}$$

$$= \frac{19.33}{400}$$

$$= 0.048$$

∴ Axially loaded member

step-ⓔ Factored load $P_u =$
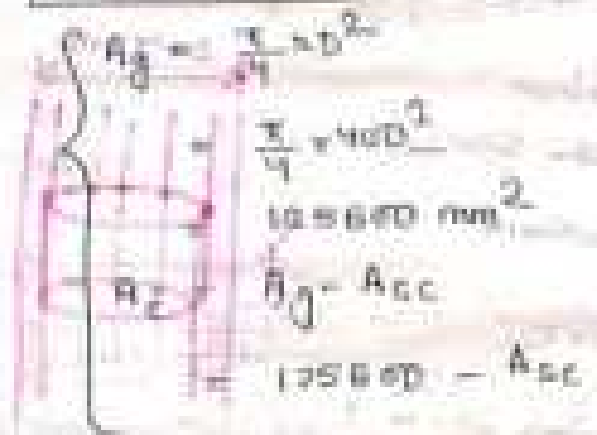
$$P_u = 1.5 \, P$$

$$= 1.5 \times 1200$$

$$= 1800 \, kN$$

$$= 1800 \times 10^3 \, N$$

step-ⓕ Area of Reinforcement (A_{sc})

cl 39.4, Eq"1.

$P_{u,min} = 1.05 \, P_u$ (short) rectangular

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

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

$\frac{\pi}{4} \times 400^2$

$125660 \, mm^2$

$A_g - A_{sc}$

$= 125660 - A_{sc}$

$\Rightarrow 1800 \times 10^3 = 1.05 \left[0.4 \times 25 (125660 - A_{sc}) \right.$

$$\left. + 0.67 \times 415 \times A_{sc} \right]$$

$\Rightarrow A_{sc} = 1793.35 \cong 1800 \, mm^2$

$$\boxed{A_{sc} = 1800 \, mm^2}$$

Now checks the A_{sc} by % of steel.

% of steel $= \frac{1800}{125660} \times 100 = 1.4\%$

∴ cl 26.5.3.1 (a), lies b/w $0.8\% \rightarrow 6\%$

$P_g - 48$ →hence (ok)

No. of bars?

Using 25 mm φ bars, their area $= \frac{\pi}{4} \times 25^2 = 314 \, mm^2$

$$\text{No. of bars} = \frac{\text{Total Area}}{\text{1 bar Area}}$$

$$= \frac{1810}{314}$$

$$= 5.7$$

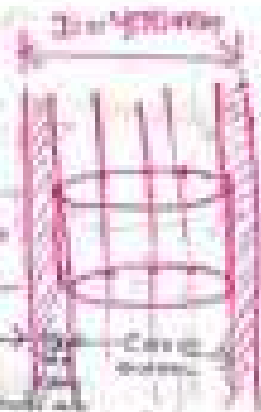$\approx$ 6 nos. (always take it to even no.)

$\therefore$ Provide 6-25 mm φ bars

$$A_{st \, provided} = 6 \times 314 = 1884 \, mm^2$$

Step-2   Helical Reinforcement (spiral ties)

Core diameter
$$= 400 - 50 \times 2$$
$$= 300 mm$$

Area of core
$$= \frac{\pi}{4} \times 300^2 = longitudinal \, reinforcement \, area \, (1884)$$
$$= 68768 \, mm^2$$

Assuming pitch = p

Volume of core $= 68766 \, p \, mm^3$

Using 8mm φ spirals;

Volume of 1 spiral $= \underbrace{\frac{\pi}{4} \times 8^2}_{Area} \times \underbrace{\pi (300-8)}_{Circumference}$

Volume of helical r/f $= 46064 \cdot 05 \, mm^2$

$$\frac{\text{Volume of helical r/f}}{\text{Volume of the core}} \geq 0.36\left(\frac{A_g}{A_c} - 1\right)$$

$$\frac{46009.85}{68746 \cdot \phi} \geqslant 0.36 \left(\frac{128650}{46064.85} - 1\right)\left(\frac{25}{415}\right)$$

$$\Rightarrow \boxed{\phi \leqslant 38 \cdot 82\,mm}$$

check the pitch

Cl. 26.5.3.2 (d) Pg 49

↳ Limits for Pitch

Max^m pitch = $\min$ of $\begin{cases} (i)\ 75mm \\ (ii)\ \dfrac{Core\ dia}{6} = \dfrac{350}{6} = 58\,mm \end{cases}$

Min^m pitch = $\min$ of $\begin{cases} (i)\ 25mm \\ (ii)\ 3\times Dia\ of\ Spiral \\ \quad = 3\times 8 \\ \quad = 24mm \end{cases}$

∴ Provide ; 8mm ϕ Spirals @ 30mm c/c

Provided
* min^m pitch = 25mm
* Max^m pitch = 75mm

8-20mmϕ

Clear Cover 50mm

Spiral ties
8mm ϕ @ 30mm c/c
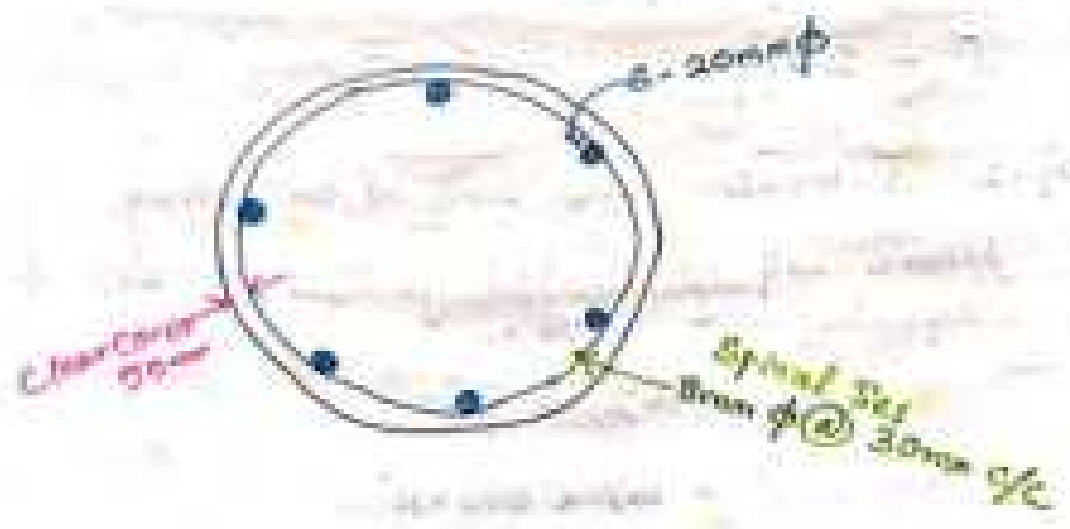
Design a isolated footing of uniform thickness of a RC column bearing a vertical load of 600KN and having a size of 500mm x 500mm. The safe bearing capacity of soil is 120 KN/m2. M20 & Fe 415

Steps involved are as follows :
1. Determine the size of footing.
2. Find out the upward pressure P_o
3. Depth of footing on basis of B.M.
4. Area of Reinforcement
5. Check for 1 way shear (Beam shear)
6. Check for 2way shear (Punching shear)

Given:

Isolated footing

$W_L = 600 KN$

Col Size = 500mm x 500mm

SBC = 120 KN/m2

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

Step-I Determine the size of the footing
Assume self wt. of footing equal to 10% of superimposed load.

$W_D = 10\% \; W_L$

$= 10\% \times 600 KN$

$= 60 KN$

New Total load $W = W_L + W_D = 600 + 60 = 660 KN$

∴ the reqd area of footing

$$A = \frac{W}{q_{oc}} \left(= \frac{\text{Total load}}{\text{Saf. pr}} \right)$$

$$= \frac{660}{120}$$

$$= 5.5 \, m^2$$

Now, for square footing

$$B = \sqrt{A}$$

$$= \sqrt{5.5}$$

$$= 2.345 \, m$$

∴ Provide a square footing of size 2.4m × 2.4m

Step-II To find out net upward pressure (p_o)

$$p_o = \frac{W}{B^2} \left(= \frac{\text{Total load}}{\text{area}} \right)$$

$$= \frac{660 \, kN}{2.4^2}$$

$$= 114.58 \, kN/m^2$$

Step-III Depth of footing on the basis of B.M.

→ The max^m B.M acts at the face of the column & is given by

$$\boxed{M = p_o \frac{B}{8} (B - b)^2}$$

Max^m arm occurs at face of the column either right or left. (Refer fig.2)

$$\rightarrow M = 114.58 \times \frac{2.4}{8} (2.4 - 0.5)^2$$

$$= 124.1 \, kN\text{-}m$$

Fig-3

Critical Section for Punching Shear (2 way shear)

Fig-2

Critical Section for Beam Shear (1way Shear)

$\left[\frac{(L-a)}{2} - d\right]$

Fig-1

Critical Section for Bending Moment

$\frac{(L-a)}{2}$

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

$BM = \frac{1}{2} P_0 B \left(\frac{L-a}{2}\right)^2$

$M_x = \frac{1}{8} P_0 L (L-a)^2$

Now ultimate Moment: $M_u = 1.5 M$

$\qquad = 1.5 \times 124 \cdot 1 \cdot$

$\qquad = 186 \cdot 15 \, kN \cdot m$

$\underline{D_{reqd}}$

$M_{u,lim} = 0.138 f_{ck} b d^2 \quad \xleftarrow{} \quad f_{u_{max}} \quad (\frac{d}{u} = 0.48 d)$

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

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

$\Rightarrow d = \sqrt{\dfrac{M_u}{0.138 \, f_{ck} \, b}}$

$\Rightarrow d = \sqrt{\dfrac{186 \cdot 15 \times 10^6}{0.138 \times 25 \times 240}}$

$\qquad = 167 \cdot 63 \, mm$

$\qquad \simeq 170 \, mm$

Provide 50mm Cover (Min^m is 50mm)

$\qquad D = d + d'$

$\qquad = 170 + 50$

$\qquad = 220 mm$

due to shear consideration: Adopt higher effective depth.

∴ Adopt 400mm as eff depth (d) & provide 50mm eff cover

∴ $d_{taken} = 400 mm$

$\qquad D = 400 + 50$

$\qquad = 450 mm$

Step-IV Area of Reinforcement

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

$$= \frac{0.5 \times 20}{415} \left[1 - \sqrt{1 - \frac{4.6 \times 196.13 \times 10^6}{20 \times 2400 \times 400^2}} \right] \frac{2400}{400}$$

$$= 1327.7 \, mm^2$$

$$P_t = \frac{A_{st}}{b d} \times 100$$

$$= \frac{1327.7}{2400 \times 400} \times 100$$

$$= 0.138 \% \quad > 0.12\% \quad (O.k.)$$

∴ At sub edge mm reinforcement criteria is 0.12%

∴ O.k.

Using 12mm φ bars in both directions

$$a_{st} = \frac{\pi}{4} \times 12^2 = 113 \, mm^2$$
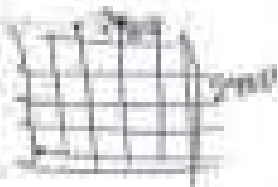
No. of bars $= \frac{A_{st}}{a_{st}} = \frac{1327.7}{\pi \times} = 11.7 \approx 12$ bars

Provide 12mm φ bars, uniformly in both direction

∴ Spacing $= \frac{B}{No. \, of \, bars}$

$$= \frac{2400}{12}$$

$$= 200 \, mm \, c/c$$

Step 3 Check for 1 way shear or Beam shear

Maxm ar. lies at a distance 'd' from face of column.

For 1way shear, critical section is located at a distance 'd' from the face of the column

Now shear force (V)

$$V = k_0 \times B \left\{ \frac{1}{2}(B-b) - d \right\}$$

V in kN
$k_0 - \text{kN/m}^2$
B - m
d (eff) - m

$$= 174.58 \times 2.4 \times \frac{1}{2}\left\{(2.4 - 0.5) - 0.49\right\}$$

$$= 151.24 \text{ kN}$$

Ultimate shear force V_u

$$V_u = 1.5V$$

$$= 1.5 \times 151.24$$

$$= 226.86 \text{ kN}$$

Nominal shear stress τ_v (Pg. 72)

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

$$= \frac{226.86 \times 10^3}{2400 \times 490}$$

$$= 0.256 \text{ N/mm}^2$$

For solid slabs (footings considered as solid slab)

Design shear strength of concrete

$$\boxed{\tau_c' = \tau_c k}$$ p.72   IS456:2000

$k = 1$ because depth > 300mm

τ_c for 0.12% steel & M20 concrete

$$\Rightarrow \tau_c' = 0.28 k = 0.28 \text{ N/mm}^2 \quad (\text{table Pg 73})$$

Now $\tau_v < \tau_c'$ ∴ (OK)

$$\frac{V_u}{bd} = K \tau_c$$

$$\Rightarrow d = \frac{V_u}{bK\tau_c}$$

$$= \frac{225.86 \times 10^3}{2.4 \times 10^3 \times 1 \times 0.28}$$

$$= 337\,mm > d_{th} = 170 \;[\text{on basis of } b.m.$$

← not considered

hence we increase effective depth

$\therefore d_{provided} = 450\,mm > 337\,mm \quad \therefore$ OK in one way shear

step- VI   Check for 2-way shear or Punching shear

→ Lies $\frac{d}{2}$ from all the faces of column column punches out footing as shear force.

For 2way shear the section lies @ a distance $\frac{d}{2}$ from the column face all around.

$\therefore$ The width $= b_0 = b + d$

$$b_0 = 450 + 450$$

$$= 900\,mm$$

Now, the net SF acting on the perimeter

$$F = \tau_v \left[B^2 - b_0^2 \right]$$

$$= \tau_v \left[B^2 - (b+d)^2 \right]$$

$$= 119.58 \left[2.4^2 - 0.9^2 \right]$$

Ultimate shear force $F_u = 1.5 F$

$$= 1.5 \times 561.171 \text{ kN}$$

$$= 841.75 \text{ kN}$$

Pg of IS456:2005 + Pg 45

$$\tau_v = \frac{F_u}{4 b_o d}$$

$$= \frac{841.75 \times 10^3}{4 \times 950 \times 450}$$

$$= 0.50 \text{ N/mm}^2$$

Now, permissible shear stress (Pg 58) $k_s \tau_c$

$$k_s = 0.5 + \beta_c$$

$$= 0.5 + 1 \qquad \text{for square column } \beta_c = 1$$

$$= 1.5 > 1$$

$$k_s = 1 \ [max]$$

Take $k_s = 1$

$$\tau_c = 0.25 \sqrt{f_{ck}} = 0.25 \sqrt{20} = 1.11 \text{ N/mm}^2$$

$$\therefore k_s \tau_c = 1 \times 1.11 = 1.11 \text{ N/mm}^2$$

$$\tau_v < k_s \tau_c \qquad \therefore \text{Safe in 2way shear}$$