LECTURE NOTES

ON

STRUCTURAL DESIGN-I

Diploma in Civil Engineering

By

Mrs. SUSHREE SOURAVI ROUT

Lecturer, Civil Engineering Department

DEPARTMENT OF CIVIL ENGINEERING

Govt. Polytechnic, Jajpur

Ch-1 Working Stress Method (WSM)

(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 stress 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.

(3) 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 less 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.

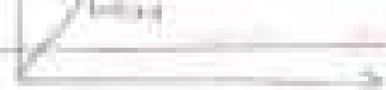
(ii) ULM :

- 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.



(iii) 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.

(12) 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.

$$\boxed{F_k = F_m + K S_d}$$

where,

F_k = characteristic load

F_m = mean load

K = constant = 1.645 ≈ 1.65

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.

$$\boxed{FOS = \frac{\text{ultimate stress}}{\text{working stress}}}$$ (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 Pg - 80) Plane sect plane before 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 material.

(iv) All tensile stresses are taken by reinforcement & not by concrete.

(v) The strength 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, vibration, 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.

(19) 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.

(i) Simple method
(ii) Larger section provided
(iii) better serviceability [illegible]
[illegible] deflection
(v) Only method to investigate

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 uncertainties.

(i) Real strength & str. of str. not given by WSM, factor of safety gives more FoS value failure
(ii) Large % of steel & uneconomical
(iii) E.c changes
(iv) Can't classify load types

(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.

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 to 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 uncertainties.

(i) Real strength & f.o.s not found, WSM gives true f.o.s under failure
(ii) Large % of steel & uneconomical
(iii) E_c changes
(iv) Can't classify load distinct

(2) 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.

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.

(i) Real strength & str. not given by WSM ... failure
(ii) larger % of steel i.e. uneconomical
(iii) E.c changes
(iv) can't classify load direct

(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.

(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.

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

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

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

(ii) Durability
(iv) Fire resistance
(v) 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 maximum 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 member is known as singly reinforced section.

(i) Durability
(ii) Fire resistance
(iii) Vibration

(25) 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 maximum 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 member is known as singly reinforced section.

(3) Doubly Reinforced Section.

- steel reinforcements are provided on both the flexural tension & compression zone of 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 $\boxed{M_u > M_{u,lim}}$ then ⇒ Doubly reinforced section.

(36) 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 Span upto 10m.

Cantilever	7
Simply Supported	20
Continuous	26

(37) Define Collapse state

The limit state of collapse of the structure or part of the structure could be assessed from rupture of one or more critical sections & from buckling due to elastic or plastic instability or overturning.

(35) Define Gross section, Transformed section, cracked section

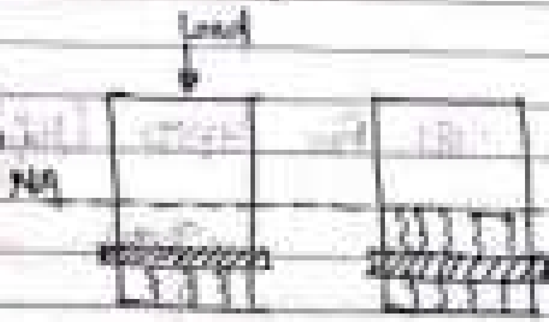
(Ref: IS 456 - 2000 Pg. 28)

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.

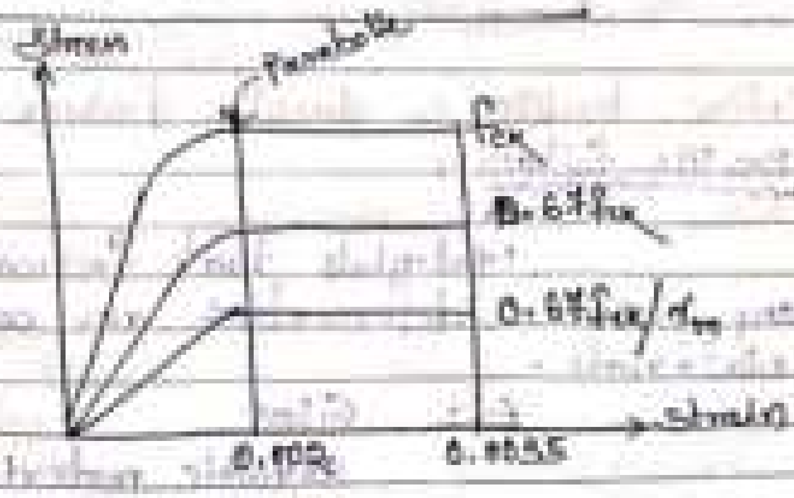
Cracked Section:

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

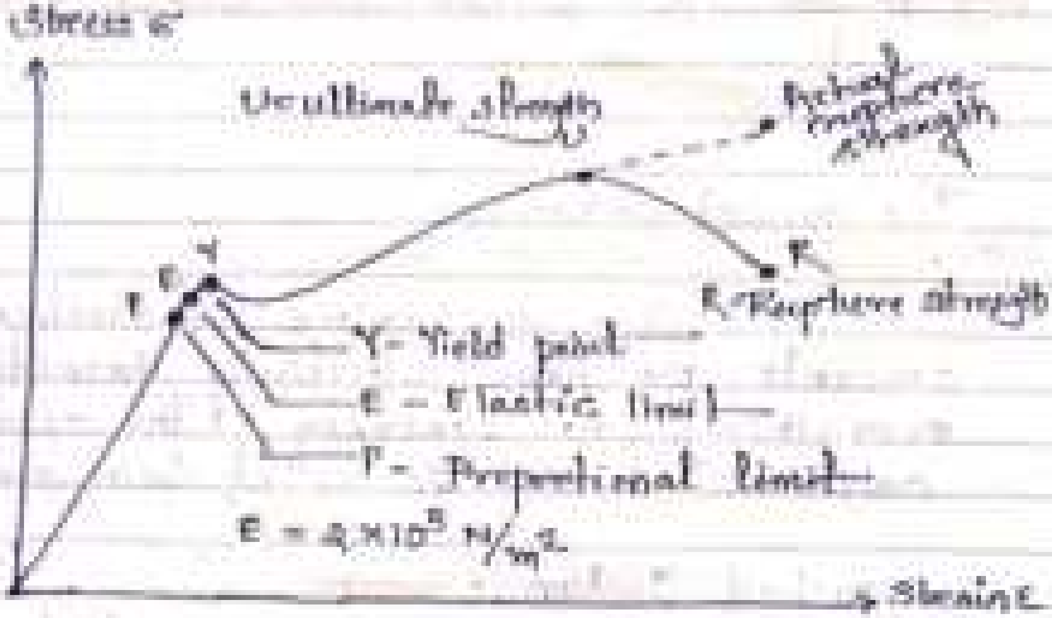


(36) Draw the stress - strain curve for concrete, mild steel bars & HYSD.

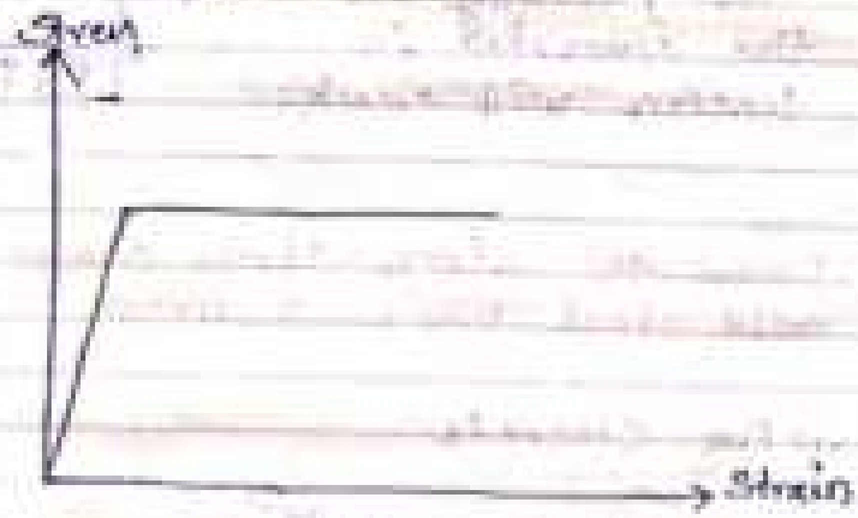
(i) For Concrete



(i) For mild steel



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



37 Define brittle & ductile failure

Brittle failure :

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

Ex: Glass

ceramic materials

Ductile failure :

Materials ∴ undergo plastic deformation before fracture is called ductile material

Ex : Aluminum
Copper
Steel & many metals

Polyethylene, nylon & many polymers

(58) Clear Cover

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

(59) Effective cover

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

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

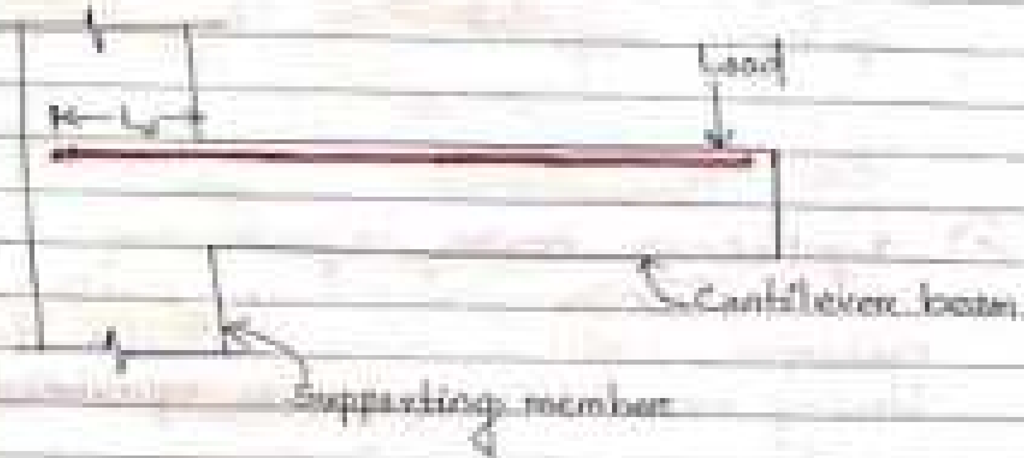
(60) Grades of concrete & steel

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

Grade of steel Fe415, Fe500, Fe550

Grade of cement 33grade, 43grade & 53grade

① What do you understand by development length of bars?
- The reinforced bar must extend in the anchorage ~~zone~~ 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 & area 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 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.

⑨ 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^m bending moment occurs
ii → Point of curtailment
iii → Point of inflation

⑩ 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 shear or variations in bending moment, which in turn causes a variation in axial tension along the length of a reinforcing	It arises over the length of anchorage provided for a bar or near the end of a reinforcing bar.

(13) 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%.

(14) 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 any one of the following three forms. (Refer IS456:2000 Pg-72)

(i) Vertical stirrups

(ii) Inclined stirrups

(iii) Bent-up bars along with stirrups

(15) Write down the value of design bond stress for M20 grade of concrete.

Design bond stress in LSM for plain bars (mild steel) in tension $\tau_{bd} = 1.2\, N/mm^2$

(16) 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.

(17) 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.

(21) Types of slab

(1) One way slab $l_y/l_x > 2$

(2) Two way slab $l_y/l_x < 2$

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.

(22) 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 :

$$A_{st} = \frac{0.15}{100} bD \text{ (For mild steel)}$$

$$A_{st,min} = \frac{0.12}{100} bD \text{ (For HYSD bars)}$$

Spacing = Min^m of { 3d ; 300mm (horizontal distance b/w parallel main reinforcement bars) }

Spacing = Min^m of (2°) { 5d ; 450mm (horizontal distance b/w parallel reinforcement bar provided against shrinkage & temperature) }

(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.

(27) What do you understand by flanged beam
The concrete in the slabs which is on the compression forces & ~~the steel in the~~ 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 c/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

(ii) It is reinforced with longitudinal & lateral ties

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

(iv) 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

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

short column $\lambda < 12$

Long column $\lambda > 12$

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.

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

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 rod 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 ≯ 6.0% of the gross cross sectional area of column
[0.8% − 6.0%]

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

The pitch of the transverse reinforcement shall be not more than the least of following distance

min^n 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 foundations?
   (i) Deep foundation
   (ii) Shallow foundation.

2. What are the types of shallow foundation?

Shallow foundation
- Combined footing
- Isolated footing
- Mat or raft footing
- Spread or strip footing
- Strap or cantilever 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 soils.

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 axial 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 &/or 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. The 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 applications for the limit state of collapse

14. Define staircase.

- Staircase flights are generally designed as slabs spanning between walls 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 : 200mm to 250mm

17. List the various types of staircases.

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

(iv) multi-flight stair
(v) Open well 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 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 stairs.

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

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 on 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 1 kN/m²)

Live Loads:

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

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

27. Explain structural behaviour of stair cases.

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 :-

[illegible]

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

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

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

(1) Force of compression $= 0.36 \sigma_{ck} b . x$

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

Force of tension $= 0.87 \sigma_y A_t$

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

$\Rightarrow 1080x = 204900$
$\Rightarrow x = 190mm$

$x_m = 0.53d$ (for $\sigma_y = 250$)
$= 0.53 \times 400$
$= 212mm > 190mm$ (O.K.)

$\therefore$ Depth of neutral axis = 190mm

(2) Force of compression $= 0.36 \sigma_{ck} b . x$

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

Force of Tension $= 0.87 \sigma_y A_t$

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

$= 340108N$

$1440x = 340100$

$\Rightarrow x = 236\,mm$

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

$= 0.48 \times 400$

$= 192\,mm < 236\,mm$

$\therefore$ It is an over-reinforced section

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

③ Determine the lever arm for section shown in fig. If effective cover assume to
(i) $f_{ck} = 20\,N/mm^2$ and $f_y = 250\,N/mm^2$

(ii) $f_{ck} = 25\,N/mm^2$ and $f_y = 415\,N/mm^2$

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

3 bars each of 16mm diameter

(i) Force of compression

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

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

$= 1800x\ N$

Force of tension

$T = 0.87 f_y A_t$

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

$= 131100\ N$

$1800x = 131100\ N$

$\Rightarrow x = 72.9\,mm$

$x_m = 0.53d$ for $f_y = 250$

$= 0.53 \times 300$

$= 159.9\,mm > 72.9\,mm$ (O.K)

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

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

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

$= 0.36 \times 25 \times 250\,x$

$= 2250x$ 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

$2250x = 217700$

$\Rightarrow x = 96.8$ mm

$x_m = 0.48\,d$ for $f_y = 415$

$= 0.48 \times 360$

$= 172.8$ mm > 96.8 mm (O.K.)

$\therefore$ Depth of neutral axis $= 96.8$ mm

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

② Determine the moment of resistance for the section shown in fig. [illegible]

[illegible] $f_{ck} = 20$ N/mm², $f_y = 415$ N/mm²

[illegible]

(i) Force of compression

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

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

$= 1800x$ N

Force of tension

$T = 0.87 f_y A_{st}$

$= 0.87 \times 415 \times \left(3 \times \frac{\pi}{4} \times 12^2\right)$

$= 122480$ N

$C = T$

$\Rightarrow 1900x = 129920$

$\Rightarrow x = 68mm$

$$x_m = 0.48d \quad (\text{for } f_y = 415)$$

$$= 0.48 \times 310$$

$$= 148.8\ mm > 68mm \quad (O.K.)\ U.R$$

$\therefore$ Depth of NA $= 68mm$

Lever arm $= 310 - 0.42 \times 68$

$z = 281mm$

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

Moment of resistance with respect to steel

= tensile force $\times z$

$$M_u = 0.87 f_y A_t z$$

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

$$= 34.0\ kN\text{-}m$$

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

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

$$= 1800x\ N$$

$$A_t = 3 \times \left(\frac{\pi}{4} \times 12^2\right) = 340mm^2$$

Force of tension $T = 0.87 f_y A_t$

$$= 0.87 \times 500 \times 3 \times 113 = 147470\ N$$

$\therefore C = T$

$\Rightarrow 1800x = 147470$

$\Rightarrow x = 82mm$

$$x_m = 0.46d \quad (\text{for } f_y = 500 N/mm^2)$$

$$= 0.46 \times 310$$

$$= 142.6mm > 82mm \quad (O.K.)$$

This is an under-reinforced section

Depth of NA $= 82mm$

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

Moment of resistance w.r.t steel.

$$M = 0.87 f_y A_t z$$
$$= 0.87 \times 500 \times 3 \times 113 \times 275.6\,mm$$
$$= 40.65\ kN\text{-}m$$

Q) Design a rectangular beam to resist a bending moment equal to 75 kNm using
(i) M25 mix & Fe415 grade steel
(ii) M25 mix & Fe500 grade steel

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 factor × BM
= 1.5 × 75
= 112.5 kNm

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

$$M_u = 0.36 f_{ck} b (0.48d)\{d - (0.42 \times 0.48d)\}$$
$$= 0.36 \times 25 \times b \times 0.48 \times d^2 (1 - 0.42 \times 0.48)$$
$$= 3.45\, bd^2$$

Since $D/b = 1.5$ $\Rightarrow$ $\frac{d}{b} = 1.5$ or,

$$b = \frac{d}{1.5}$$

$M_u = 2.48\,bd^2 = 2.48 \times \left(\frac{d}{1.5}\right)d^2 = 2.5\,d^3$

$2.5\,d^3 = 112.5$ kNm

$\Rightarrow 2.5\,d^3 = 112.5 \times 10^6$ N-mm

$\Rightarrow d = 365.7$mm

Adopt $D = 400$mm and $b = \frac{400}{1.5} = 267$mm $\simeq 300$mm (say)

Effective cover = 35mm (say)

$d = 400 - 35 = 365$mm

Area of tensile steel $A_{st} = \frac{\text{Factored BM}}{0.87 f_y (d - 0.42 x_m)}$

$= \frac{112.5 \times 10^6}{0.87 \times 415 \times (d - [illegible])}$

$= \frac{112.5 \times 10^6}{0.87 \times 415 \times ([illegible] \times 365)}$

$= 1070\,mm^2$

$= 10.7\,cm^2$

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

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

$= 224\,mm^2 < 1070\,mm^2$ (o.k)

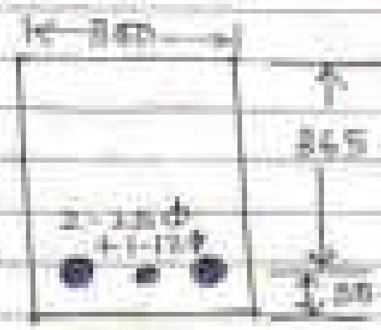
In beams, the diameter of main reinforcing bars is usually selected b/w 12mm & 25mm. Provide 2-25mm and 1-12mm bars giving total area

$= 2 \times 491 + 113$

$= 1095\,mm^2 > 1070\,mm^2$ (O.K.)

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

Reinforcement detail :



(ii) Moment of resistance $M = 0.36 f_{ck}\, b\, x_m (d - 0.42 x_m)$

For Fe500 steel,

$$x_m = 0.44 d$$

$$M_u = 0.36 \times 25 \times b \times 0.44 d \,(d - 0.42 \times 0.44 d)$$
$$= 3.25\, b d^2$$

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

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

Adopt : $D = 410$mm & $b = 275$mm $\left(= \frac{410}{1.5} = 273.33\right)$

if effective cover = 35mm

effective depth = 375mm

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

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

$$= \frac{112.5 \times 10^6}{0.87 \times 500 \times \left(373 - 0.42 \times 0.44 \times 373\right)}$$

$$= 743.21 \text{ mm}^2$$
$$= 7.43 \text{ cm}^2$$

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

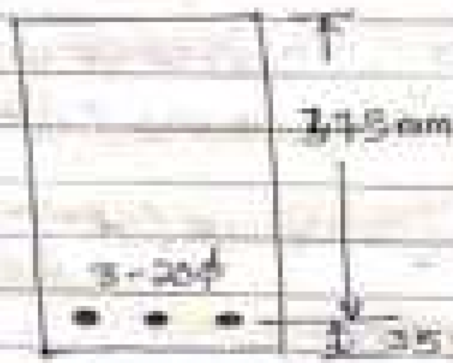
$= \frac{0.85 \times 300 \times 375}{550}$

$= 172.94\ mm^2 < 773.21\ mm^2$ (o.k.)

In beams, the diameter of main reinforcing bars is usually selected between 15mm – 32mm.

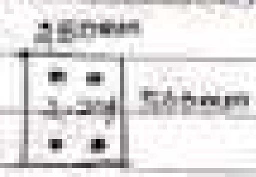
Provide 3-20mm bars, giving total area = $942\ mm^2$ $> 773.21\ mm^2$ (o.k.)

Reinforcement details :-



DOUBLY REINFORCED SECTION

Q. Find the moment of resistance of a beam section 25cm x 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

Let $\left\{\begin{array}{l} \text{depth of NA } x = x_m = 0.48d \text{ (for Fe415)} \\ = 0.48 \times 450 \\ = 216mm \\ d' = 50mm \end{array}\right.$

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

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

Stress in compression steel f'_{sc} = 350 N/mm² (Table - 4.6)

$f_{sc} = \dfrac{(351.8 - 342.8)}{(0.00276 - 0.00241)} \times (0.00267 - 0.00241) + 342.8$

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

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

Total force of compression $C = C_1 + C_2$

(i) Force of Compression in concrete

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

$= 0.36 \times 20 \times 250 \times 216$

$= 388800\ N$

(ii) Force of Compression in steel

$C_2 = (f'_{sc} - f_{cc}) A_{sc}$ → area 20 mm in concrete

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

$= (350 - 0.446 \times 20) \times 628$

$= 214198\ N$

$C = 388800 + 214198$

$= 603000\ N$

Force of tension $T = 0.87 f_y A_t$

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

< Force of compression

Hence, It is an under-reinforced section.

Depth of NA axis should be reduced to equalize force of compression with force of tension

(B) Second Trial

Let, depth of NA = 97.5 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

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

$$= 181.9\ N/mm^2$$

Force of compression

$$C_1 = 0.36 f_{ck} \cdot b \cdot x$$
$$= 0.36 \times 20 \times 250 \times 67.5$$
$$= 121500\ N$$

$$C_2 = (\sigma_{sc} - \sigma_{cc}) A_{sc}$$
$$= (181.9 - 0.446 \times 20)\, 628$$
$$= 108600\ N$$

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

Force of tension $T = 0.87 f_y A_t$

$$= 0.87 \times 415 \times 628$$
$$= 226700\ N$$

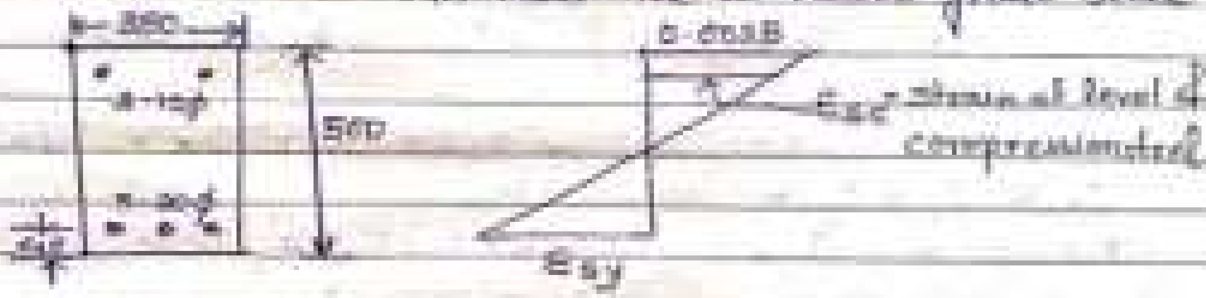
$C \approx T$ O.K. Hence depth of NA = 67.5

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

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

$= 121500 \times (350 - 0.42 \times 67.5)$
$+ 128800 \times 400$
$= 94.67 \times 10^6$ Nmm
$= 94.67$ kN m

(8) Find the moment of resistance of a beam 25cm × 50cm deep if it is reinforced with 2-12mm bars in compression zone & 3-20 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 $x = x_m = 0.48d$ & d'=40mm
$= 0.48 \times 460$
$= 221$ mm

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

$= 0.00286$

For Fe415 grade steel

$f_{sc} = ?$

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

$f_{sc} = 351.8 + (360.9 - 351.8)\left[\frac{0.00286 - 0.00276}{0.00380 - 0.00276}\right]$

$= 352.7\ N/mm^2$

(3-12B) $A_{sc} = 2 \times 1.13 = 2.26\ cm^2 = 226\ mm^2$

Total force of compression $C = C_1 + C_2$

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

$= 0.36 \times 20 \times 250 \times 221$

$= 397800\ N$

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

$= (352.7 - 0.446 f_{ck}) \times 226$

$= (352.7 - 0.446 \times 20) \times 226 = 77691\ N$

$C = C_1 + C_2 = 475490\ N$

Force of tension $T = 0.87 f_y A_t$ (3-20P)

$= 0.87 \times 415 \times 2 \times 314$

$= 340110\ N$ < force of compression

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

Second Trial :

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

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

$= 0.00257$

The corresponding stress in steel is

f_{sc} = 346.9 MPa

For 415 grade steel ε_{sc} — f_{sc}

0.00241 ⟶ 342.8 N/mm^2

0.00257 ⟶ ?

0.00276 ⟶ 351.8 N/mm^2

$C = C_1 + C_2$
$= 546380\,N$
= force of tension steel

$C_1 = 0.36 f_{ck}\, b\, x = 270750\,N$
$C_2 = (f_{sc} - f_{cc}) A_{sc}$
$= (f_{sc} - 0.446 f_{ck}) A_{sc}$
$= 275630\,N$

∴ Moment of resistance $= C_1 (d - 0.42x) + C_2 (d - d')$ $\;\;$ 460-40
$= 139.2 \times 10^6\,Nmm$
$= 139.2\,kN\text{-}m$

If the load factor is 1.5

Safe working moment $= \dfrac{139.2}{1.5} = 92.8\,kNm$

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

First Trial:

Let, depth of NA $x = x_m = 0.46d$
$= 0.46 \times 460$
$= 212\,mm$ &

$d' = 40\,mm$

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,

$0.00277 \longrightarrow 392.8\,N/mm^2$
$0.00283 \longrightarrow ?$
$0.00380 \longrightarrow 365.9\,N/mm^2$

$$f_{sc} = 413 + (423.7 - 413) \left[\frac{0.00283 - 0.00277}{0.00312 - 0.00277} \right]$$

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

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 $T = 0.87 f_y A_{st}$

$= 0.87 \times 500 \times 3 \times 314$

$= 409770 N$ < force of compression

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

Second Trial: $C_1 + C_2$

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

$\boxed{C_1 + C_2 = T}$

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

$\Rightarrow x = 141.5 mm$ (Take)

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

$= 0.00251$

$f_{sc} = ?$

For 500 grade

$0.00226 \longrightarrow 391.3 N/mm^2$

$0.00251 \longrightarrow ?$

$0.00277 \longrightarrow 413 N/mm^2$

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

$= 402 N/mm^2$

Force of compression $C = C_1 + C_2 = 406310\text{N} = $ force

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

$= 0.36 \times 25 \times 250 \times 141.5$

$= \ldots$

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

$= (412 - 0.446 \times 25) \times 36$

$= \ldots$

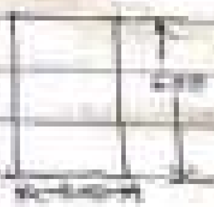
Thus moment of resistance

$= C_1 (d - 0.42x) + C_2 (d - d')$

$= C_1 (460 - 0.42 \times 141.5) + C_2 (460 - 46)$

$= 184.6 \text{ kN-m}$

(5) 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 × 60cm overall. Use M20 mix & Fe415 grade steel.



Self weight of beam = volm × density

$= (0.30 \times 0.60) \times 25 \text{ kN/m}$

$= 4.5 \text{ kN/m}$

Superimposed load $= 60 \text{ kN/m}$

∴ Total load $= 64.5 \text{ kN/m}$

Factored load $= 1.5 \times 64.5$

$= 96.75 \text{ kN/m}$

Max^n factored BM $= \dfrac{Wl^2}{8}$

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

$= 435.37 \text{ kN-m}$

Let, effective cover $d' = 0.1D$

$= 0.1 \times 600$

$= 60 \text{ mm}$

$d = D - d' = 600 - 60 = 540\text{mm}$

For Fe415 $x_m = 0.48d$

$= 0.48 \times 540$

$= 260 \text{ mm}$

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,

$$0.87 f_y A_{st_1} = 0.36 f_{ck} \cdot b \cdot x_m$$

$$\Rightarrow 0.87 \times 415 \times A_{t_1} = 0.36 \times 20 \times 300 \times 260$$

$$\Rightarrow A_{t_1} = 1550 \text{ mm}^2$$

(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 $d'/d = 0.1$, $f_{sc} = 353 \text{ N/mm}^2$

f_{sc} in compression reinforcement

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

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

$$M - M_{lim} = (f_{sc} A_{sc} - f_{cc} A_{sc})(d - d')$$

ie $f_{cc} = 0.446 f_{ck}$

$$\Rightarrow A_{sc} = \frac{(435.35 - 241.5) \times 10^6}{(353 - 0.446 \times 20)(540 - 54)}$$

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

Corresponding tension steel A_{t_2} =

$$0.87 f_y A_{t_2} = f_{sc} \cdot A_{sc}$$

$$\Rightarrow 0.87 \times 415 \times A_{t_2} = 353 \times 1160$$

$$\Rightarrow A_{t_2} = 1134 \text{ mm}^2$$

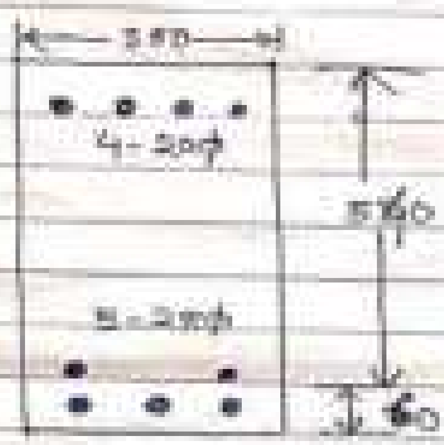
Total tension steel $A_t = A_{t_1} + A_{t_2}$

$= 1556 + 1154$

$= 2670\ mm^2$

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

$\boxed{\text{Provide}}$ 5 - 28 ϕ in tension ($A_t = 3078\ mm^2 > 2670$)
4 - 20 ϕ in compression ($A_t = 1256\ mm^2 > 1160$)



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

$= 0.04 \times 300 \times 600$

$= 7200\ mm^2$

$> 3078\ mm^2$ (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 (bracing of elevated tanks, lintels etc) or having disconnected slabs as in some pre-cast system

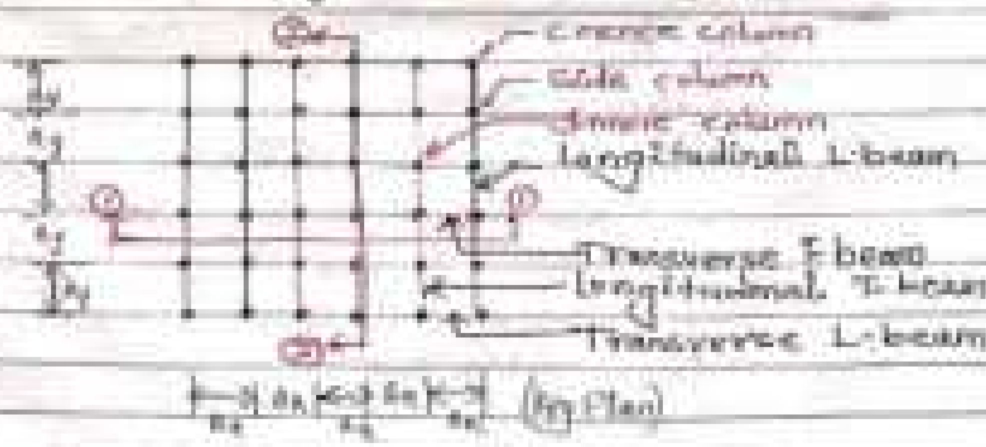
— Due to monolithic casting, beams and a part of slab act together

— Under the action of positive BM 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.

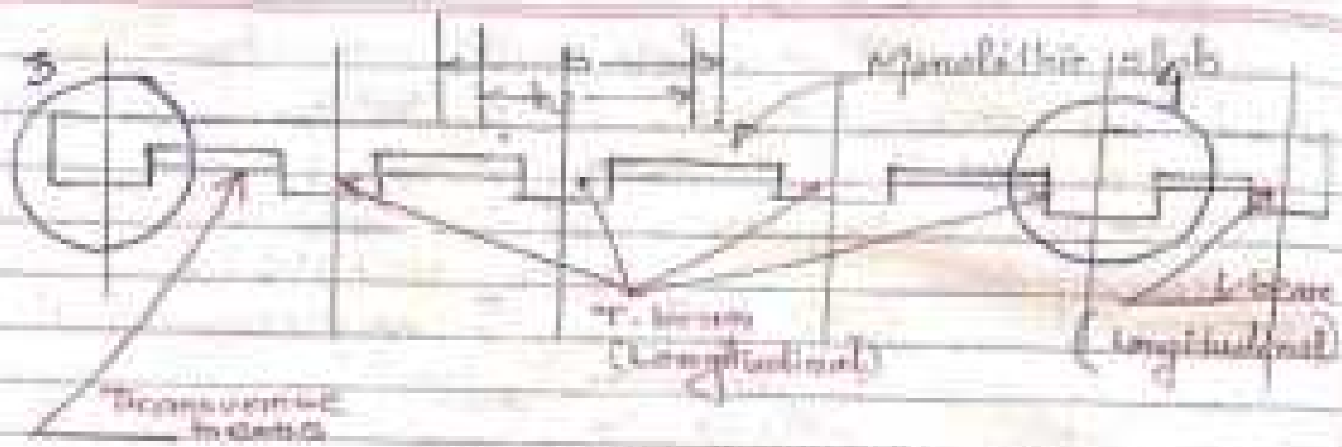
— Over the supports of a continuous beam, the bending moment is negative and the slab, therefore, is in tension while a part of the rectangular beam (rib) is in compression

— The continuous beam at support is thus equivalent to rectangular beam

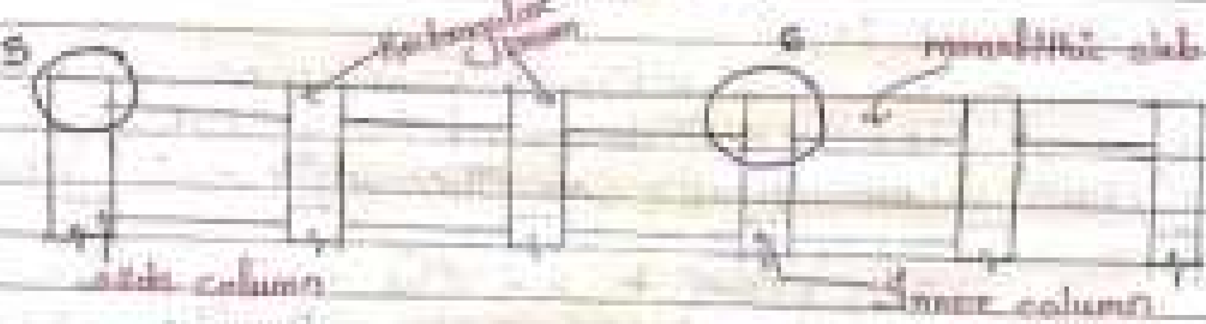


(b) Plan

— C L of beam, • column
a_x = c/c distance of longitudinal beam
a_y = c/c " of transverse beam



b = actual width of slab
b_f = effective width of flange
(Sect^n 1-1)



Notations:

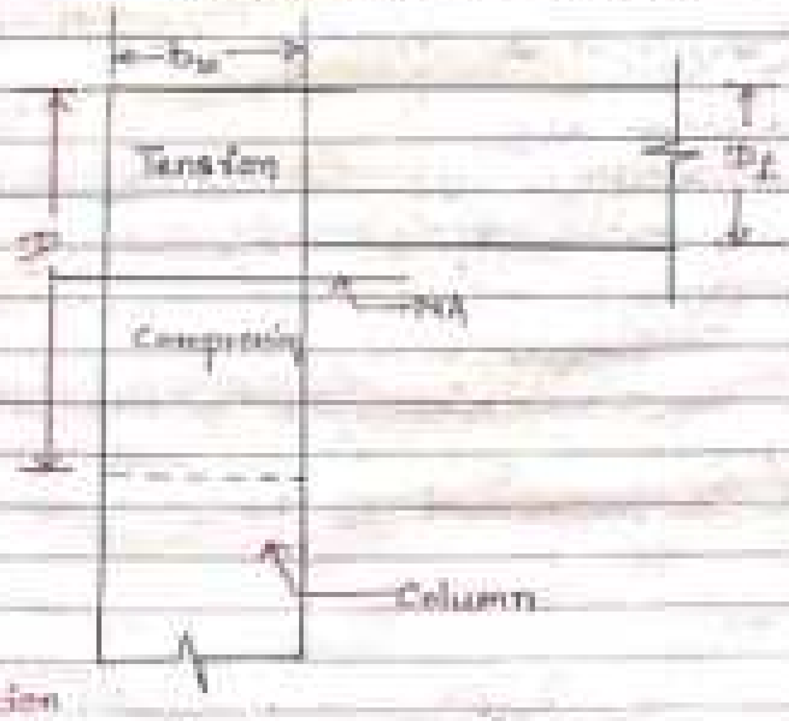
b = Actual width of flange
b_f = Effective width of flange
b_w = width of web
D_f = Depth of flange
NA = neutral axis

(Detail of 3 L-beam)

Notations:

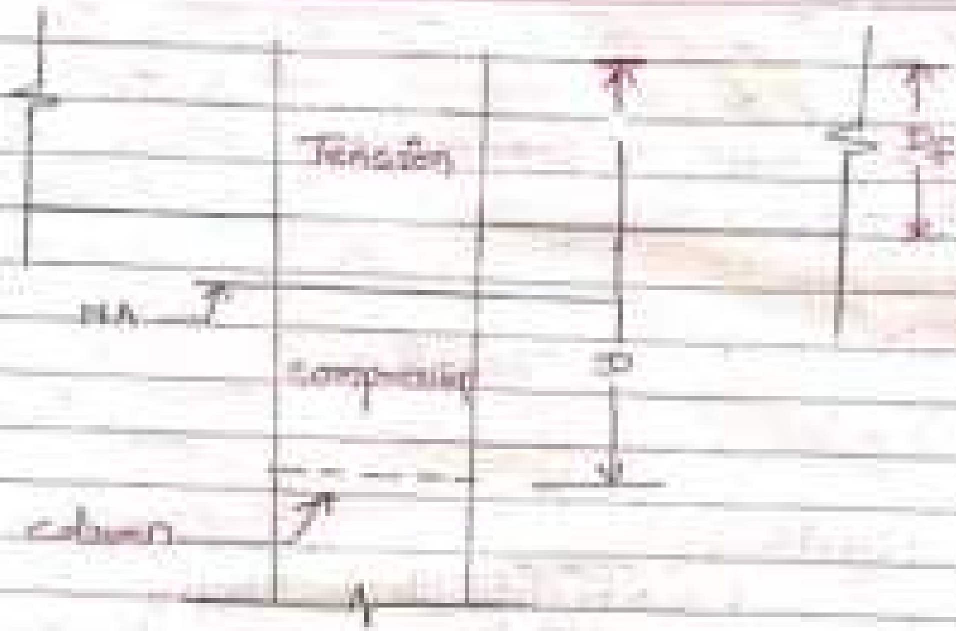
b = actual width of flange
b_f = Effective width of flange
b_w = width of web
D_f = depth of flange
NA = neutral axis

(Detail at 4 T-beam)



Notation

b_f = Effective width of flange
b_w = width of web
D_f = Depth of flange
NA = neutral axis
(Detail at 5 Rectangular beam)



Notation

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

(instead of a rectangular beam)

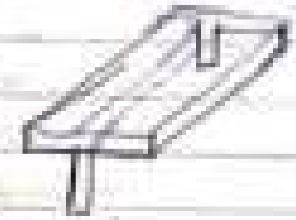
- The actual width of the flange is the spacing of the beam, which is the same as the distance b/w the middle points of the adjacent spans of the slab as shown in fig. a.
- However in flanged beam, a part of the width less than the actual width, is effective to be considered as a part of the beam. This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

N. P.

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 is what is called web or stem.
- effective flange width
- for isolated T-beam

a) $b_{eff} \le 4b_w$

b) $h_f \ge b_w/2$

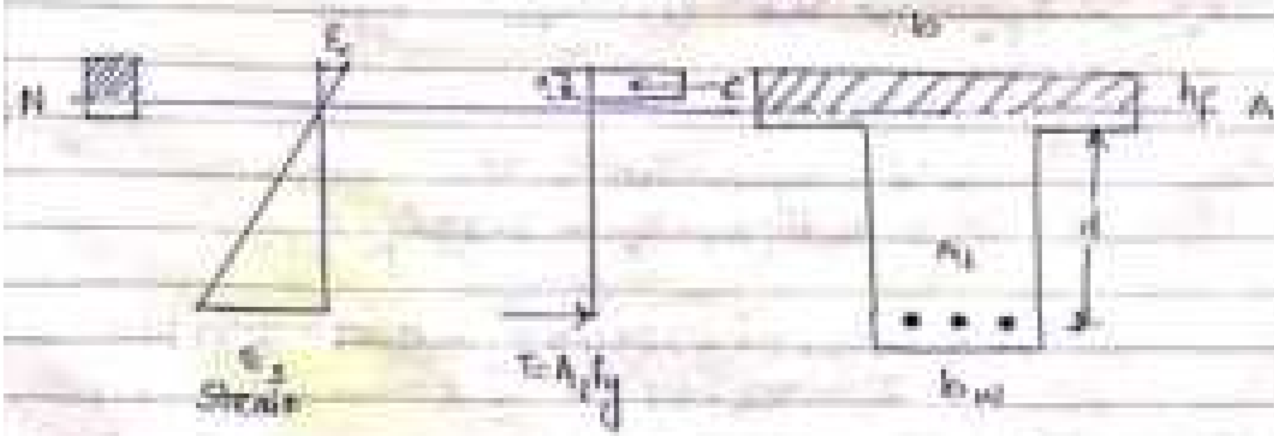
Strength Analysis

1st Case (NA is with in the flange)

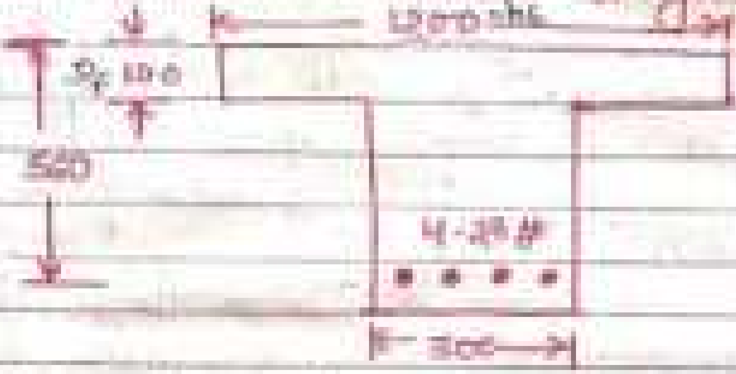
- Analysis as a rectangular beam of width

$$b = b_{eff}$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$



A tee beam of effective flange width 1200mm, thickness of slab 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.

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

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

$= 864 kN$

$T = F_{st} = 0.87 f_y A_{st}$

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

$= 709 kN$

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

Equating the forces

Total compression = Total tension

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

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

$\Rightarrow x_u = 82.07 mm < 100mm$ (OK)

$x_{u,max} = 0.48 d$

$= 0.48 \times 560 = 268.8 mm$

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

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

$= 0.87 \times 415 \times 1964 (560 - 0.42 \times 82.07) \times 10^{-6}$

$= 372.65$ kN.m

Or,

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

$= 0.36 \times 20 \times 1200 \times 82.07 (560 - 0.42 \times 82.07) \times 10^{-6}$

$= 372.65$ kN-m

NA in Web

(*) Find the ultimate moment of resistance for the section above if it is reinforced with 5 no 25mm diameter bar.

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

To find the position of NA

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

$= 0.36 \times 20 \times 1500 \times 100 \times 10^{-3}$

$= 804$ kN

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

$= 886.4$ kN

$C < T \Rightarrow$ N.A. lies in web (out-of-syllabus)

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

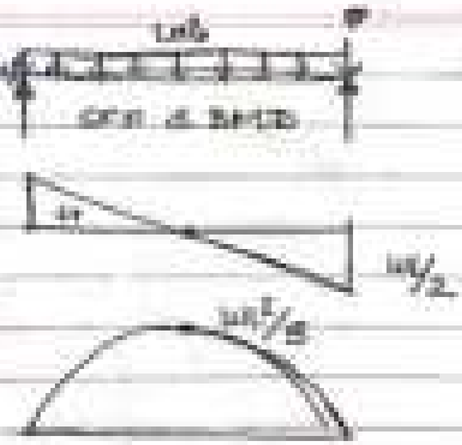
Let us assume that it is an under reinforced section & the NA lies within the flange (case 1)

$$0.36 f_{ck} \cdot b_f \cdot x = 0.87 f_y A_{st}$$

$$\Rightarrow x = \frac{0.87 \times 415 \times 3600}{0.36 \times 20 \times 750} = 241 \text{mm} > D_f$$

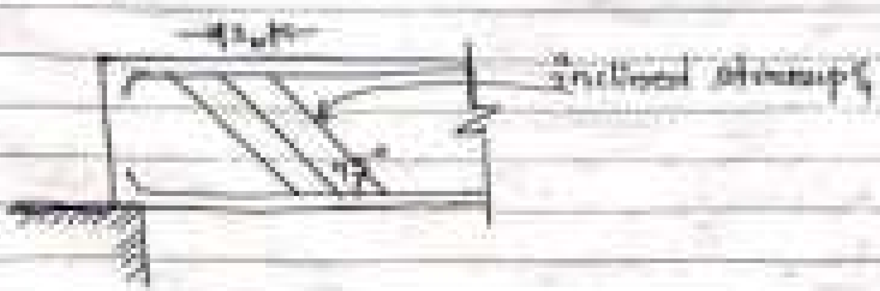
As the value of x is more than 120 mm, NA lies in web

- As the concrete is weak in shear & at supports the shear force is maximum so there is a chance of sliding or shear failure.
- So steel is provided to bear the shear force & 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 serve 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.



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 local shear 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

$$T = \frac{dM}{jd} \quad \text{--- (1)}$$

u = local bond stress

Σo = Perimeter of steel

$$T = u * \Sigma o * dx \quad \text{--- (2)}$$

Equating =>

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

$$u = \frac{dM/dx}{\Sigma o\, jd} = \frac{V}{\Sigma o\, jd}$$

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

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

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

- 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

$$\Rightarrow \tau_{bd}(\pi d)\, DL = 0.87 f_y \frac{\pi}{4} d^2$$

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

- Hence all the bars should extend to a distance of D_L beyond the section where they are required to take full 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

Q1. A RC beam has an effective depth of 500mm & a breadth of 350mm. It contains 4-25mm bars. If (i) $\sigma_{ck} = 20 N/mm^2$, $\sigma_{sv} = 250 N/mm^2$
(ii) $\sigma_{ck} = 25 N/mm^2$, $\sigma_{sv} = 415 N/mm^2$
for stirrup

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

% area of longitudinal steel $p = \frac{100 A_s}{bd}$

$= \frac{100 \times \frac{\pi}{4} \times 25^2 \times 4}{350 \times 550}$

$= 1.12\%$

(a) Design shear stress of concrete

$\tau_c = 0.64 N/mm^2$ (from table)

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

$= \frac{350 \times 1000}{350 \times 500}$

$= 2 N/mm^2$

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

$0.64 N/mm^2 < 2 N/mm^2 < 2.8 N/mm^2$

$\Leftrightarrow$ Provide shear reinforcement

(i) Using Fe250 grade steel

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

$= (350 \times 10^3) - (0.64 \times 350 \times 500)$

$= 238000 N$

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

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

$$= \frac{0.87 \times 250 \times 100.5 \times 510}{228000}$$

$$= 48mm$$

The code requires that $x \ngtr 300mm$

$\ngtr 0.75d = 0.75 \times 510 = 383mm$

Min^m area of shear reinforcement

$$A_s \geq \frac{0.4bx}{0.87 f_y} = \frac{0.4 \times 350 \times 48}{0.87 \times 250} = 30mm^2$$

* 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

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

$$= \frac{228000 \times 100}{0.87 \times 250 \times 510}$$

$$= 213mm^2$$

$\frac{\pi}{4} \times 12^2 = 113$

Area of one leg $= 213/2 = 110mm^2$

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

$(A_{sv} = 226\,mm^2)$

Chk Min^m shear reinforcement

$$A_s \geq \frac{0.4 \times 350 \times 150}{0.87 \times 250} = 65\,mm^2$$

$226\,mm^2 > 65\,mm^2$ (O.K)

(ii) Using Fe415 grade steel

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

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

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

$$= \frac{0.4 \times 350 \times 150}{0.87 \times 415}$$

$$= 58.2\,mm^2$$

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

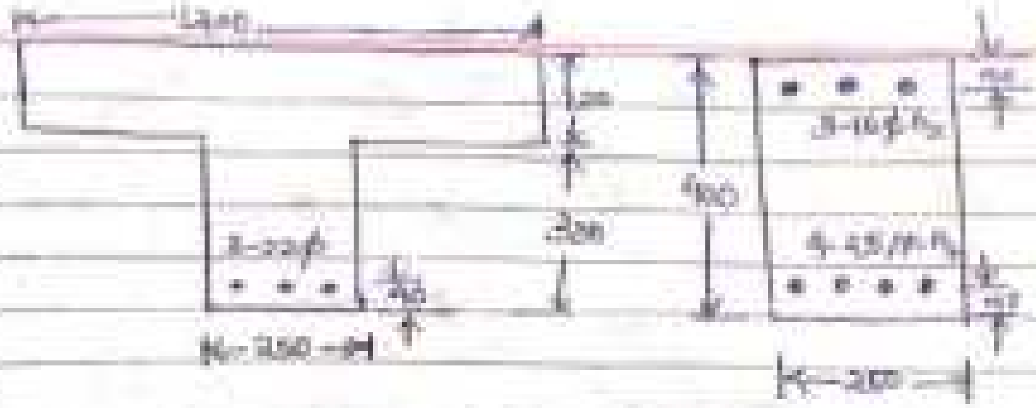
$$= \frac{238000 \times 100}{0.87 \times 415 \times 500}$$

$$= 131.8\,mm^2 > A_s$$

Area of one leg $= 131.8/2 = 66\,mm^2$

$\boxed{\text{Use}}$ 10mm - 2legged vertical stirrups @ 100mm c/c.

Q2. The beams shown in figure are subjected to factored shear force of 210kN, $\tau_c = 2\,N/mm^2$ & $f_y = 415\,N/mm^2$. Calculate the shear reinforcement



(a) T-beam

Width of web b_w = 250mm

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

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

$$= \frac{100 \times 3 \times \frac{\pi}{4} \times 22^2}{250 \times 460}$$

$$= 0.99\%$$

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

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

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

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

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

Net design shear stress $\tau_{vs} = \frac{V_u}{bd} - \tau_c$

$$= 1.74 - 0.62$$

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

Adopt 8mm - 2 legged shear stirrups giving

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

Spacing x of vertical shear stirrups

$$x = \frac{0.87 f_y A_{sv}}{\tau_{vs} \cdot b_w}$$

$$= \frac{0.87 \times 415 \times 100.5}{142 \times 250}$$

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

$$\not> 0.75d \;(= 0.75 \times 460 = 345\,mm)$$

Minimum shear reinforcement is given by

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

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

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

(ii) Doubly reinforced beam

width of section $= 250\,mm$

Effective depth $= 400 - 40 = 360\,mm$

% tension steel $p = \frac{100 A_t}{bd}$ (A_t – tension steel)

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

$$= 2.18\%$$

Design shear stress $\tau_c = 0.8\,N/mm^2$

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 $\tau_{vs} = \tau_v - \tau_c$

$$= 2.22 - 0.8$$

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

[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

$$x = \frac{0.87 f_y A_{sv}}{\tau_{uv}\, b}$$

$$= \frac{0.87 \times 415 \times 157}{1.42 \times 250}$$

$= 160 mm$

$0.75d (= 370 mm)$ O.K.

$> 100 mm$ O.K.

Min^m shear reinforcement is given by

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

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

$= 45 mm^2 < A_{sv}$ (O.K.)

Development length

Q2. A simply supported beam is 25cm × 50cm & has 2-20mm TOR bars going into the support. If the shear force at the centre of support is 110kN 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 × 110 = 165 kN
Assuming 25mm clear cover to the longitudinal bars
Effective depth = $500 - 25 - \frac{20}{2} = 465 mm$

Characteristics strength of TOR steel $f_y = 415\,N/mm^2$

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

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

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

$$= 126\,mm < x_m$$

or,

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

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

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

Bond Stress $\tau_{bd} = 1.2\,N/mm^2$ for M20 mix. It can be increased by 60% in case of TOR bars.

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

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

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

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

$$L_0 = 8\phi = 8 \times 20 = 160\,mm \text{ (for 90° bend)}$$

$L_d \le 1.3 M_1$ ... — for simply

$$47\phi \le \left\{\frac{1.3 \times (93.45 \times 10^6)}{165 \times 10^3}\right\} + 160$$

$\Rightarrow \phi \le 19mm$

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

$\phi \le 20.8mm$ ℄ of support (O.K.)

The arrangement of 90° bend is shown.

Alternatively,

Provide a U bend at the centre of support. Its anchorage value $L_0 = 16\phi = 320mm$

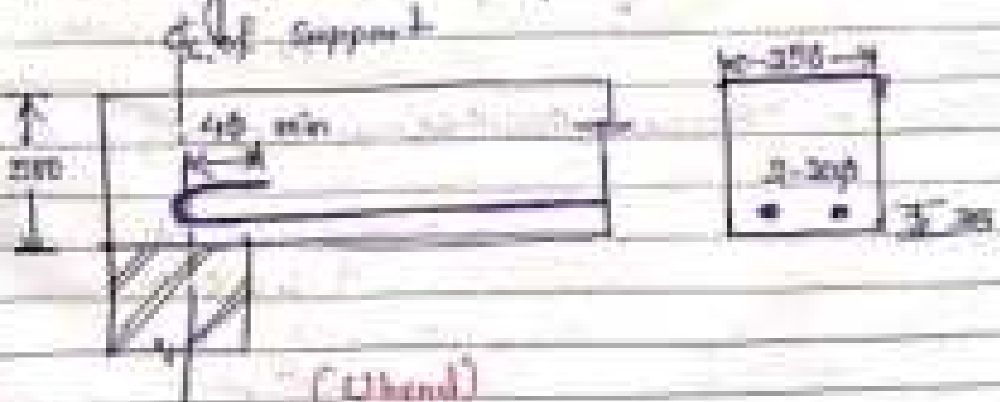
$$L_d \le \frac{1.3 M_1}{V} + L_0$$

$$\Rightarrow 47\phi \le \frac{1.3 \times 93.45 \times 10^6}{165 \times 10^3} + 320$$

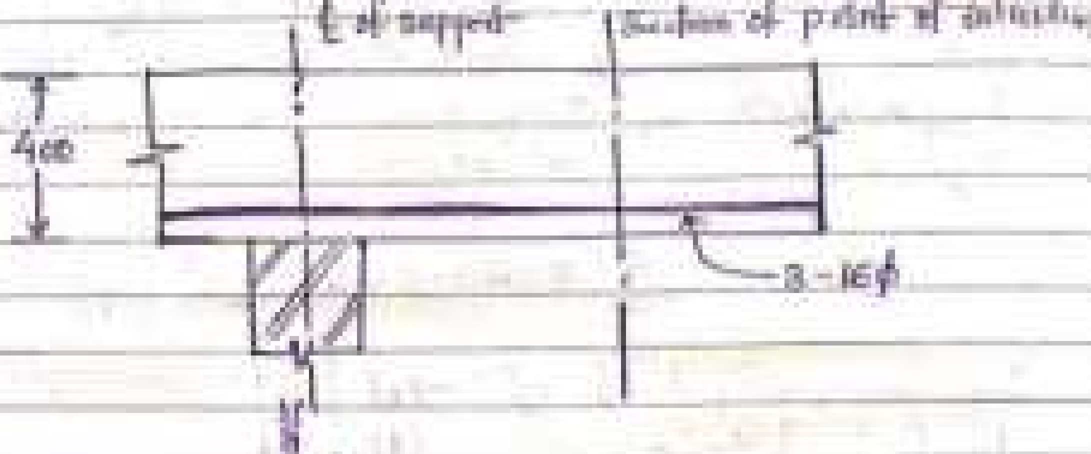
$\Rightarrow \phi \le 22.47mm$

Actual bar diameter provided is 20mm < 22.47mm OK

The arrangement of U-bend



Q2. A continuous beam 25cm × 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, $\sigma_{ck} = 20\ N/mm^2$ & $\sigma_y = 415\ N/mm^2$. Check if the beam is safe in bond?



Assuming 25mm clear cover to the longitudinal bars

Effective depth $d = 400 - 25 - \frac{16}{2}$

$= 367mm$

Depth of neutral axis

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

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

$= 120mm < x_m (= 0.48d)$ (O.K)

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

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

$= 68.9 \times 10^6\ N\text{-}mm$

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

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

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

$= 47\phi$

Anchorage length L_0 = greater of d or 12ϕ

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

= 367 mm

$$L_d \leq \frac{M_1}{V} + L_0$$

$$\Rightarrow 47\phi \leq \frac{62.9 \times 10^6}{150 \times 10^3} + 367$$

$$\Rightarrow \phi \leq 17.6 \text{mm}$$

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

Ch-7 Design of slab and stair case (LSM)

Q.1) Design a simply supported roof slab for a room 5m × 3.5m clear in size if the superimposed load is $5 kN/m^2$. Use M20 mix & Fe415 grade steel.

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

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

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

Lets adopt overall depth $D = 190 mm$

Dead Load of slab $= 0.19 \times 1 \times 25 = 4.75 kN/m$

Superimposed load $= 5 \times 1 = 5 kN/m$

$\therefore$ Total load $= 9.75 kN/m$

Factored load $= 1.5 \times 9.75 = 14.63 kN/m$

Maximum BM 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 $L = 3.5 + d$

$= 3.5 + 0.17$

$= 3.67 m$

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

$= 24.63 kN\text{-}m$

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

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

$= 25.6 kN$

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

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

Adopt effective depth $d = 130mm$
Overall depth $D = 150mm$

Area of tension steel $A_t = ?$

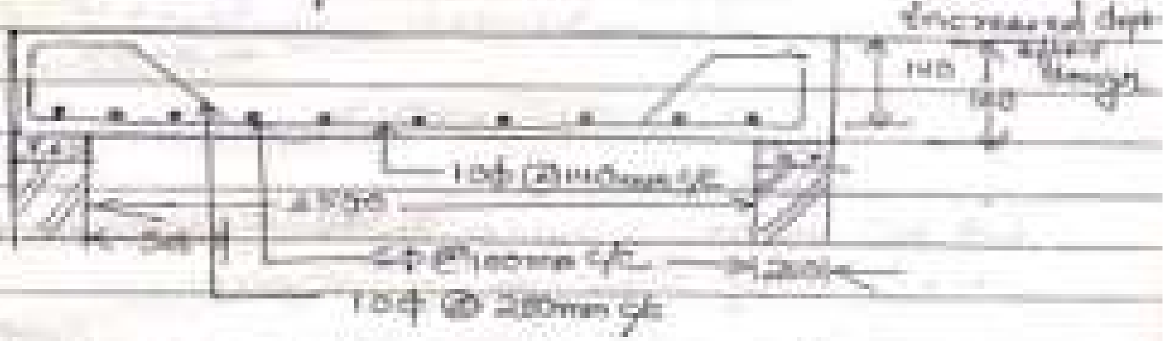
$$M = 0.87 f_y A_t \left(d - \frac{f_y A_t}{f_{ck} \cdot b}\right)$$

$$\Rightarrow 24.63\times10^6 = 0.87\times415\times A_t \left(130 - \frac{415 A_t}{20\times1000}\right)$$

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

$\boxed{\text{Use}}$ 10mm bars @ 140mm c/c giving total area $560.7mm^2$ $> 541.6\,mm^2$ (O.K.)

Bend alternate bars at $L/7$ from the face of support where moment reduces to less than half of its max^m value. Temperature



$\boxed{\text{Use}}$ 6mm MS bar @ 150mm c/c → secondary

(4) Check for shear

$$\%\text{ tension steel} = \frac{100 A_t}{bd} = \frac{100\times\frac{\pi}{4}\times10^2\times1000/140}{1000\times130}$$

$$= 0.22\%$$

% tension steel $= 0.22\%$
Shear strength of concrete for 0.22% steel $\tau_c = 0.314\,N/mm^2$
For 150mm thick slab $K = 1.3$

$\tau_c' = k\tau_c = 1.3 \times 0.34 = 0.44 \text{ N/mm}^2$

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

$$= \frac{25600}{1000 \times 130}$$

$$= 0.197 \text{ N/mm}^2 < \tau_c' \quad (\text{O.K.})$$

The slab is safe in shear.

2. Check for development length

Moment of resistance offered by 10mm bars @ 250mm c/c

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

$$= 0.87 \times 415 \times 78.5 \times \frac{1000}{250}$$

$$\times \left(130 - \frac{415 \times 78.5 \times 1000/250}{25 \times 1000}\right)$$

$$= 12.19 \times 10^6 \text{ N.mm}$$

$V = 25600 \text{ N}$

Let us assume anchorage length $L_0 = 0$

$$L_d \le 1.3\frac{M_1}{V}$$

$$\Rightarrow 40\phi \le \frac{1.3 \times 12.19 \times 10^6}{25600}$$

$$\Rightarrow \phi \le 16.75\text{mm} \qquad (\text{O.K.})$$

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

3. Check for deflection

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

$$= \frac{100 \times (78.5 \times 1000/140)}{1000 \times 130}$$

$$= 0.4\%$$

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

*For $p_t = 0.41\%$ & $f_s = 240\,MPa$ (in) Fe415 grade steel $\boxed{F = 1.32}$

$\beta = 1,\ \alpha = 1,\ \delta = 1$

Allowable $\frac{L}{d} = 20 \times 1.32 = 26.4$

$\frac{L}{d} \leq \alpha\beta\gamma\delta\lambda$

Actual $\frac{L}{d} = \frac{3670}{130} = 28.23 \nless 26.4$ (NS)

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

Foundation :-

Q1) Determine the area & depth of foundation of a square column carrying 1000 kN vertical load. The gross bearing capacity of soil = 100 kN/m² density = 17 kN/m³ & angle of repose = 29°

Load on column $W = 1000\,kN$

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

$= \frac{1000}{100}$

$= 10\,m^2$

Min^m depth of foundation is given by

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

$$= \frac{100}{17}\left(\frac{1-\sin 29^\circ}{1+\sin 29^\circ}\right)^2$$

$= 0.71\,m$

Weight of foundation including earth $= 17\times(10\times 0.71)$
$= 121\,kN$

Total load on foundation $= (100\times 10) + 121$
$= 1121\,kN$

Area of foundation required $= \frac{1121}{100} = 11.21\,m^2$

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

Weight of foundation and earth $= 17\times(11.21\times 0.71)$
$= 135\,kN$

Total load on foundation $= 1000 + 135$
$= 1135\,kN$

Area of foundation required $= \frac{1135}{100} = 11.35\,m^2$

$\boxed{\text{Adopt}}$ a foundation having an area of $11.56\,m^2$ i.e. $3.4\,m \times 3.4\,m$ at a depth of $0.71\,m$ from ground level

Qu:2 Design a square spread footing to carry a column load of 1400kN from a 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 $f_{ck} = 20\ N/mm^2$, $f_y = 415\ N/mm^2$ & load factor = 1.5

(i) Soil pressure

Axial load = 1400kN

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

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

fixed for soil

~~Total weight on soil = 1600kN~~

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

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

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

Provide 4m × 4m square footing giving total area = 16m² (O.K)

Step-1 Determining size of footing.

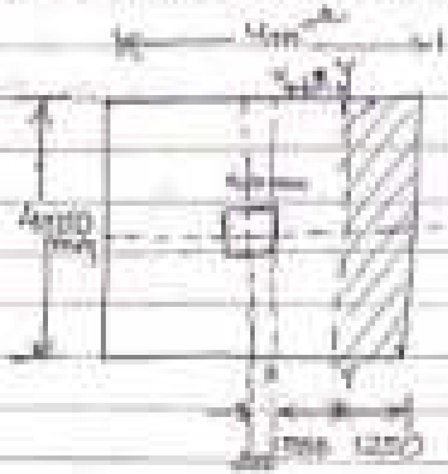
(ii) Bending moment

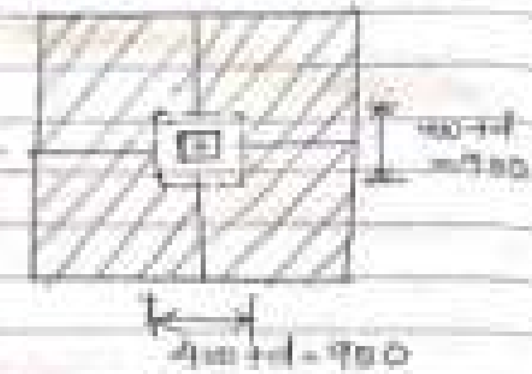
The net earth pressure acting upward due to factored load is

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

$$= 131.25\ kN/m^2$$

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)

$$M_x = 131.29\,\text{kN/m}^2 \times 4\text{m} \left(\frac{4-0.4}{2}\right)^2 \text{m}^2 \times \frac{1}{2} = \frac{wl^2}{2}$$

~~= total force × distance from critical section~~

$= 851\ \text{kN.m}$

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

$\boxed{M_u = 0.138 f_{ck} bd^2}$ $\quad d = \sqrt{\dfrac{851 \times 10^6}{0.138 \times 25 \times 4000}}$

$= 248\text{mm}$

$\boxed{\text{Adopt}}$ 550mm effective depth & 600mm overall depth, Increased depth is taken due to shear consideration.

Area of tension steel is given by BM

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

$\Rightarrow 851 \times 10^6 = 0.87 \times 415\, A_t \left(550 - \frac{415 A_t}{25 \times 4000}\right)$

$\Rightarrow A_t = 4134\ mm^2$

$\boxed{Use}$ 12mm bars @ 100mm c/c $A_t = 4520\ mm^2 > 4134\ mm^2$

(O.K.)

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

$= 0.2\%$

(ii) Shear one way action

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

shear force $V_u = 131.25 \times 4 \left[\left(\frac{4-0.4}{2}\right) - 0.55\right]$

$= w A (b - 0.55)$ shaded portion

$= 656\ kN$

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

$= \frac{656000}{4000 \times 550}$

$= 0.298\ N/mm^2$

Shear strength of M25 concrete with 0.2% steel

$\tau_c = 0.327\ N/mm^2 > \tau_v$ (O.K.)

(iii) Shear 2-way action

The critical section taken at a distance '0.5d' away from the face of column, ~~as the~~

Shear force

$V_u = 131.25 \left[16 - (0.4 + 0.55)^2\right] = 1981\ kN$

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

$= \frac{1982 \times 1000}{4(400+550)850}$

$= 0.61 N/mm^2$

Shear strength of M25 concrete,

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

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

$k_s = 0.5 + 1 \not> 1$

$\therefore \quad k_s = 1$

$\tau_c' = \tau_c = 0.25\sqrt{f_{ck}} = 1.25 N/mm^2 > 0.61 N/mm^2$ (O.K)

To show that a footing having an effective depth of 248 mm wouldn't be safe in shear

(V) Development length

Development length of 10mm bars

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

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

$= 40\phi$

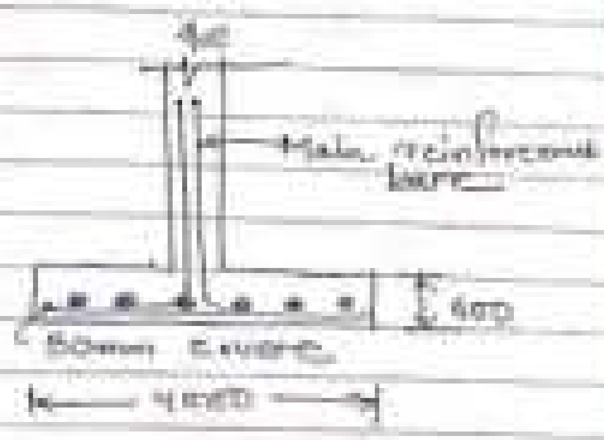
where 1.6 factor due to deformed bars (+60% ↑)

$L_d = 640mm$

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

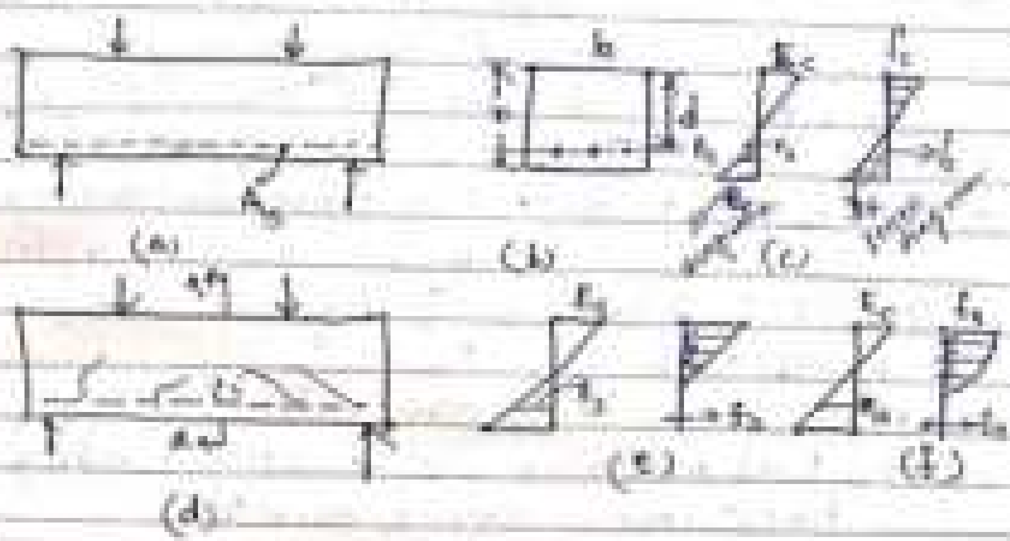
$= 1750mm > L_d$ (O.K)


4000mm


Main reinforcement bars
400
50mm cover
4000

Theory

RC Sections & their behaviour



Concrete Grade Table :— (IS 456 : 2000)

RCC ≥ M20

Group	Concrete Grade	Mix Ratio	Characteristic compressive stress
Ordinary concrete	M5	1:5:10	5 N/mm^2
	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	"	

* Min^m 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 50mm

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

* Single Reinforced Section - Beam

RC Beam

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

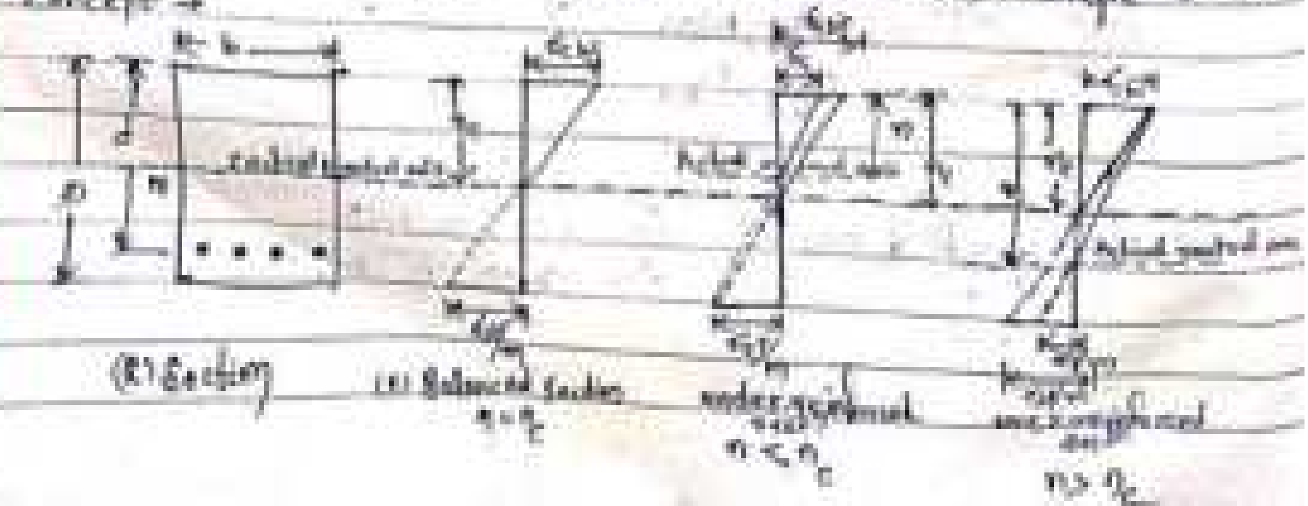
Design → Creating a beam that will carry a specified load or combination of loads

* Flexural strength of a beam is the max^m 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 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. secⁿ, the % of steel provided is less than that provided in balanced section. So the actual N.A will shift upwards to ...
- In U.R. secⁿ 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. secⁿ are preferred by designers.

Over Reinforced Section

- In an O.R. secⁿ the % of steel provided is greater than the balanced section. So the actual N.A. shift downward.
- In O.R. secⁿ the stress in concrete reaches its 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. secⁿ 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 σ_{st}

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

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

Flexural Design
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 ~~c/s geometry~~ 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.

2.1 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
- It deals with the strength & stability of str. subjected to max^m design load.

Limit state of serviceability
- 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 both	- 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 is used 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 loads, permissible stresses & FOS are known so it is called deterministic method.	- This method based on probabilistic approach which depends upon the actual data or experience hence it is called non-deterministic method

IS code suggestion → The LSM is defined as a method which limits the str. usefulness 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 ~~used~~ need or the area of steel we need in a slab : $A_s = 497\,mm^2/m$ i.e. per mt. length of slab

Spacing = $\frac{\text{Area of single bar}}{\text{Total Area}} \times$ Per 1m length of slab

$= \frac{(\pi \times 12^2)/4}{497} \times 1000 = 227\,mm \approx 220$

* Cover to reinforcement :-

Concrete cover in

	1. Footing/foundat^n	– 50mm
	2. Column	– 40mm
Exposure of building	3. Beam	– 25mm
	4. Slab	– 15mm
Extra	5. Staircase	– 15mm
	6. Retaining wall	– 25mm
	7. Raft foundation	– 50mm

IS456 : 2000 ← Nominal cover, Effective cover

No concept of clear cover, it is absolete in IS456 : 2000

* Min^m reinforcement

Beam, Slab, Column

* Effective span of beam & slab (IS456 : 2000 Cl. 22.2)

Effective length → Clear span + Effective depth of beam

or

— Beam/slab centre to centre of support

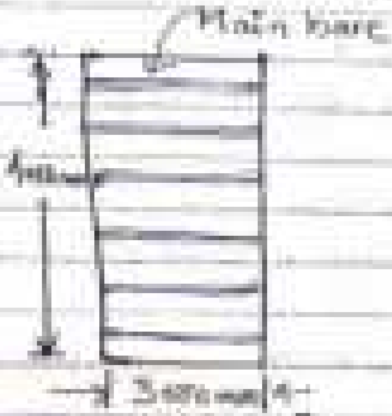
Spacing of reinforcement in slab

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

Ans

Let, slab size = 3m x 4m

no of main bar = ?



$$\boxed{\text{No of Main bar} = \frac{\text{Total length}}{\text{Spacing}} + 1}$$

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

$$= 13.3 + 1$$

$$= 14 \text{ nos}$$

weight $= 14 \times 3000 \ (3m) \times 1.580$ 16mm ϕ = 1.58 kg/m

$= 66.36$ kg

12mm ϕ –

spacing ?

no of bars ?

weight ?

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

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

$= 168.75$ mm (spacing)

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

$= 25$ nos

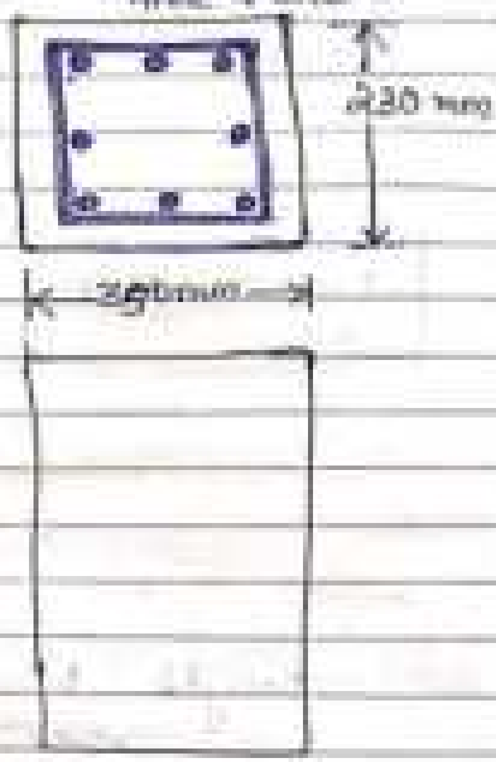
weight $= 25 \times 3m \times 0.888 = 66.6$ kg

equal wt (O.K)

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.

$\Rightarrow \frac{\text{Area of steel}}{\text{Area of column}} = 0.8\% - 6\%$



Let, column c/s Area → 230 × 350

Steel bar → 8 - 16φ

Area of column → 230 × 350

$= 80500 \text{ mm}^2$

Area of steel $= 8 \times \left(\frac{\pi}{4} \times 16^2\right)$

$= 1607 \text{ mm}^2$

$= 1608 \text{ mm}^2$

Check :

$\frac{\text{Area of steel}}{\text{Area of column c/s}}$

$= \frac{1608}{80500}$

$= 1.97\%$

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

Min^m Reinforcement R/F

Min^m % of steel

Slab: Min^m steel $= 0.15\%$ c/s area Fe250; 0.12% c/s " Fe415

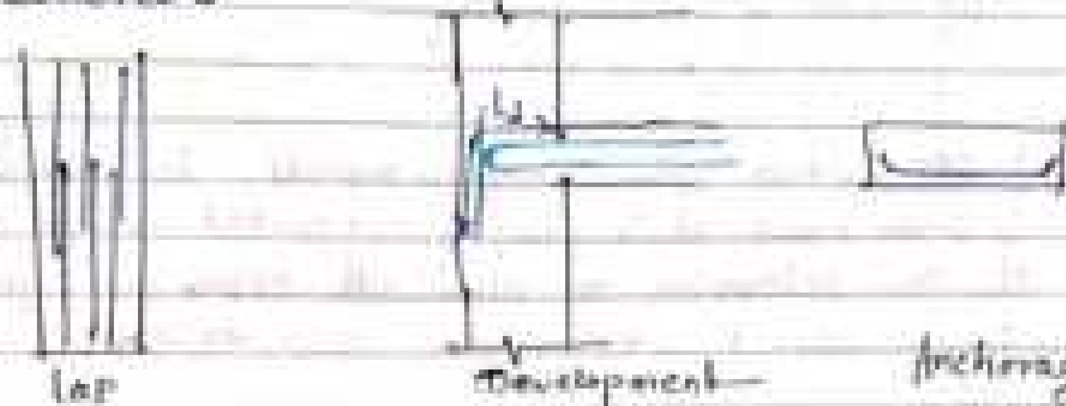
Beam: Tension R/F $\frac{A_s}{bd} = \frac{0.85}{f_y}$; Shear R/F (Stirrups) $\frac{A_{sv}}{bS_v} = \frac{0.4}{0.87 f_y}$, S_v → Spacing of stirrups

Column: Min^m steel 0.8% c/s

Lapping Length or Reinforcement Lapping

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

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



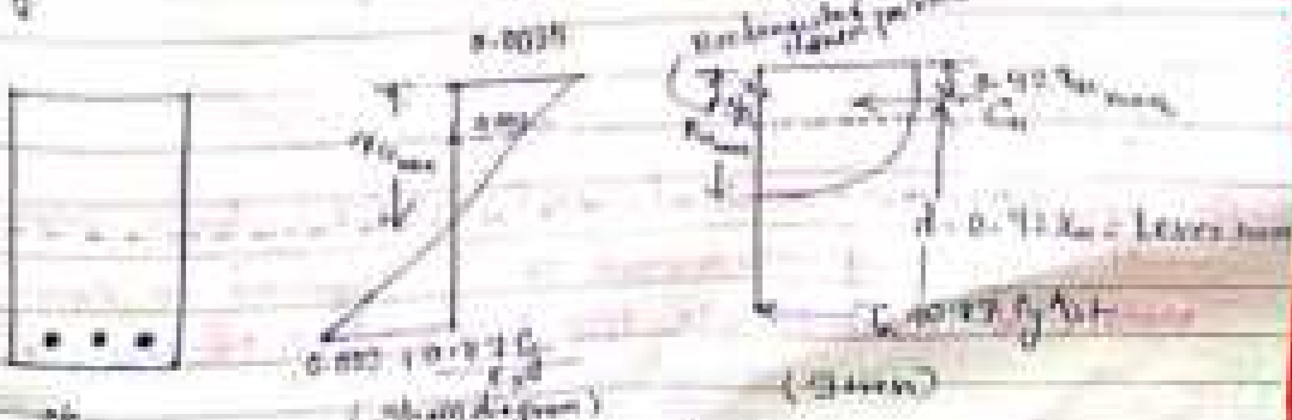
(2) Anchorage length is the equivalent length of the reinforcement bar which is considered to be available when a straight bar is bent through some angle.

(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 & it will come out which lead to failure

A.L

AL
- $4 \times d$ = 180° bend (d = dia of bar)
- $6d$ — 135° "
- $8d$ — 90° bend bar

Analysis → M.R = ? , B.M = ?
Design → Dimension ? Reinforcement ? for beam



Long. Design: $x_u < x_{u\,max}$ or $x_u = x_{u\,max}$

$C_u = 0.36 f_{ck} x_u b$

$T_u = 0.87 f_y A_{st}$

ultimate moment of resistance

$M_u = 0.36 f_{ck} x_u b \, (d - 0.42 x_u)$ — comp force × lever arm

$C_u = T_u$ — (1)

comp force in conc = Tensile force in steel

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

Q. A RCC section 250 × 600mm overall is reinforced with 4-25mm bars it is simply supported on an effective span of 6m. Determine the max^m udl beam can carry. Use M30, Fe500

Area of steel $A_{st} = 4 \times \frac{\pi}{4} \times 25^2 = 1963.47\,mm^2$

c = 40mm eff cover

d = 600 − 40 = 560mm

1. $x_{u\,max} = 0.46d = 0.46 \times 560 = 257.6\,mm$

2. $C_u = T_u$

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

$\Rightarrow 0.36 \times 30 \times x_u \times 250 = 0.87 \times 500 \times 1963.5$

$\Rightarrow x_u = 316.34\,mm$

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

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

$= 339.79\,kN{\cdot}m$

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

$\Rightarrow 339.79 = \frac{w \times 6^2}{8}$

$\Rightarrow w = 75.49\,kN$ (factored load)

working load $w = \frac{75.49}{1.5} = 50.33\,kN$

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 eff cover as 40mm

d = 390 − 40 = 350mm

I. $x_{u\,max} = 0.46d$
$= 0.46 \times 350$
$= 161mm$

II. Calculation of actual x_u (Since axial not given) $C_u = T_u$ as singly reinforced

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

$\Rightarrow 50\times10^6 = 0.36 \times 20 \times x_u \times 230 (350 - 0.42x_u)$
$\Rightarrow x_u = 97.12mm$

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

III. Method 2
$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6M}{f_{ck}bd^2}}\right]bd$

Method 1
$C_u = T_u$

$= \frac{0.5 \times 20}{500}\left[1 - \sqrt{1 - \frac{4.6 \times 50\times10^6}{20\times230\times350^2}}\right] \times 230 \times 350$

$= 391.4mm^2$

Ch-5 Analysis & Design of Single & Double Reinforced Section by LSM

Methods of Solving Numericals

Analysis Type

Given	To find
1. C/S, A_{st}	MR
2. $x_{u,lim}$	P_t, M_u
3. C/S	$M_{u,lim}$, A_{st}
4. M_u	A_{st}
5. C/S,	w (UDL)

Design Type

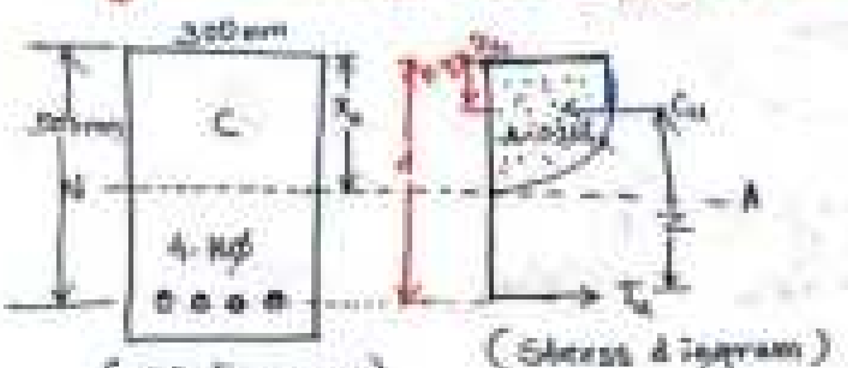
Given	Design
1. M_d	b, D, A_{st}
2. b, M_d	D, A_{st}, $A_{st,min}$, $A_{st,max}$
3. L_{eff}, $\frac{d}{b}$ ratio	b, d, A_{st}
4. clear span, support, bearing	b, d, D, A_{st}

* M_u = ultimate moment
  M_d = design Moment

Remember

Steel (f_y)	$x_{u,max}$	$M_{u,lim}$	$P_{t,lim}$
Fe 250	0.53d	$0.149 f_{ck} bd^2$	$0.088 f_{ck}$
Fe 415	0.48d	$0.138 f_{ck} bd^2$	$0.048 f_{ck}$
Fe 500	0.46d	$0.133 f_{ck} bd^2$	$0.038 f_{ck}$

Q. Find the moment of resistance if steel provided is 4 bars of 16mm diameter in a beam 300 × 500 mm effective. Concrete M20 & Steel Fe 500 are used.



(C/S diagram) (Stress diagram)

$\Rightarrow$ Concrete = Steel

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

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

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

$A_{st} = 4 \times \frac{\pi}{4} \times 16^2 = 804.25 \text{ mm}^2$

$= \dfrac{0.87 \times 500 \times A_{st}}{0.36 \times 20 \times 300}$

$x_u = 161.76$ mm

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

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

$MR = T_u \times z = C_u \times z$

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

$= 0.87 \times 500 \times 804.25 \times (500 - 0.42 \times 161.76)$

$= 151.126 \times 10^6$ N-mm

$= 151.126$ kN-m

$\therefore$ Moment of Resistance is 151.126 kN-m (Ans)

Q2. If x_u is limited to 0.39, calculate p_t & M_u. Assume M20-Fe415 concrete & steel.

$x_{u,max} = 0.39d$ —— (1)

$x_{u,max} = \dfrac{0.87 f_y A_{st}}{0.36 f_{ck} b}$ —— (2)

Equating (1) & (2)

$0.39d = \dfrac{0.87 f_y A_{st}}{0.36 f_{ck} b}$

$$\Rightarrow \frac{0.36 \times 0.48 f_{ck}}{0.87 f_y} = \frac{A_{st}}{bd}$$

$$\Rightarrow \frac{A_{st}}{bd} = \frac{0.36 \times 0.48 f_{ck}}{0.87 f_y} = 0.278 \times 10^{-2}$$

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

$= \cancel{0.278 \times 10^{-2} \times 100}$

$= 0.278\%$ (Ans)

$M_u = C_u \times z$

$= (0.36 f_{ck} b \cdot x_u) \times (d - 0.42 x_u)$

$= \left[0.36 \; f_{ck} \cdot b \cdot \frac{x_{u\,max}}{d} \times d\right] \times \left[\left(\frac{d}{d} - 0.42 x_u\right) \times d\right]$

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

$= 0.36 \times 20 \times b \times 0.53 (1 - 0.42 \times 0.53) d^2$

$= 2.942 \, b d^2$

$M_u = 2.943 bd^2$ (Ans)

Q. 2 Find Limiting Moment of resistance & steel required for a beam 300 mm × 600 mm overall depth and effective cover = 25 mm M25/Fe415 used

$M_{u,lim} = ?$

$A_{st} = ?$

$b = 300$ mm $\quad D = 600$ mm

$d' = 25$ mm $\quad d = D - d' = 575$ mm

$f_{ck} = 25 \quad f_y = 415$

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

$= 0.138 \times 25 \times 300 \times 575^2$

$= 342.17 \times 10^6$ N-mm

$= 342$ kN-m (Ans)

$C_u = T_u$

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

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

= –

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

$= 0.48 \times 575$

$= 276\text{mm}$

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

$= \dfrac{0.36 \times 25 \times 300 \times 276}{0.87 \times 415}$

$= 2063.92\ \text{mm}^2$ (Ans)

Q4. Calculate the area of steel required for a singly R/F concrete beam 250mm wide & 400mm deep to resist an ultimate moment of 60 kN-m. Use concrete mix M20 & Fe500 grade steel, effective cover = 30mm.

$b = 250\text{mm}$, $D = 400\text{mm}$, $d = D - \text{cover} = 400 - 30 = 370\text{mm}$

$M_u = 60\ \text{kN-m}$

$A_{st} = ?$

We have to first determine U/R, B/R or O/R

i) $M_{u\,lim} = 0.133 f_{ck}\, b d^2$ (for Fe500)

$= 0.133 \times 20 \times 250 \times 370^2$

$= 93.75 \times 10^6\ \text{N-mm}$

$= 93.75\ \text{kN-m}$

Since $M_u < M_{u\,lim} \Rightarrow$ section is under-reinforced

or $x_u < x_{u\,max}$ not to calculate as M_u given

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] bd \quad \text{for U/R sec}^n$$

$$= \frac{0.5 \times 20}{500}\left[1 - \sqrt{1 - \frac{4.6 \times 60 \times 10^6}{20 \times 230 \times 350^2}}\right] \times 230 \times 350$$

$$A_{st} = 428.38 \text{ mm}^2$$

* A_{st} can be calculated in 3 ways
  - (1) Above formula
  - (2) from F_t x
  - (3) Equating C_u & T_u

Q5. A Singly Reinforced beam 230 mm × 500mm 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/Fe415.

$b = 230\text{mm}$; $d = 500\text{mm}$; $L_{eff} = 6\text{m}$

$A_{st} = 4 \times \left(\frac{\pi}{4} \times 16^2\right) = 804.24 \text{ mm}^2$

$w = ?$

$f_{ck} = 20 \text{ MPa}$

$f_y = 415 \text{ MPa}$

Important

Step 1- determine U/R, B/R, O/R ?

$$\boxed{x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b}} = \frac{0.87 \times 415 \times 804.24}{0.36 \times 20 \times 230} = 175.34 \text{mm}$$

$x_{u,max} = 0.48 d = 0.48 \times 500 = 240\text{mm}$

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

$M_u = T_u \times z$ or $C_u \times z$

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

$= 0.87 \times 415 \times 804.24 (500 - 0.42 \times 175.34)$

$= 123.88 \times 10^6$ N-mm

$= 123.88$ kN-m

Simply Supported beam

$M_u = \frac{w_d l_e^2}{8}$ ← design ult

$\Rightarrow w_d = \frac{8 M_u}{l_e^2}$

$\Rightarrow w_d = \frac{8 \times 137.57 \times 10^6 \, N\text{-}mm}{6000^2 \, mm^2}$

$= \frac{8 \times 137.5 \, kN\text{-}m}{6 \, m^2}$

$= 27.51 \, kN/m$

Load without factor of safety is working load

$w = \frac{w_d}{\gamma_f}$

$= \frac{27.51}{1.5}$

$= 18.34 \, kN/m$ **(Ans)**

Q6. Design a beam to carry working moment of 80 kN-m, using M20 grade concrete & Fe415 steel

* Design a beam ⇒ b, d, D, A_{st}, No. of bar
* Working Moment M = 80 kN-m

  $f_{ck} = 20$

  $f_y = 415$

Step-1. Factored moment = 1.5 × M (considered in LSM for factored load etc)

$M_d = 1.5 \times 80$

$= 120 \, kN\text{-}m$

$= 120 \times 10^6 \, N\text{-}mm$

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: $\boxed{b = \frac{d}{2}}$

$\Rightarrow 0.138 f_{ck}\, b d^2 = 120 \times 10^6$

$\Rightarrow 0.138 \times 20 \times \frac{d}{2} \times d^2 = 120 \times 10^6$

$\Rightarrow d^3 = \frac{120 \times 10^6 \times 2}{0.138 \times 20}$

$\Rightarrow d = 443mm$

Round off the above value

$d = 450mm$

$b = \frac{d}{2} = \frac{450}{2} = 225mm$ (230, 9 inch / b ≮ 225mm)

$\Rightarrow b = 230mm$

$D = d + d'$

$= 450 + 30mm$ cover assume

$= 480mm$

$d = 450mm$, $D = 480mm$, $b = 230mm$

Step 2 Calculate Area of steel reinforcement

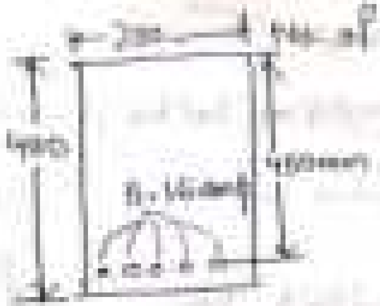
$\boxed{p_{t,lim} = 0.048 f_{ck}} = 0.048 \times 20 = 0.96\ \%$

$A_{st} = \frac{p_{t,lim} \times bd}{100}$

$= \frac{0.96 \times 230 \times 450}{100}$

$= 993.6\ mm^2$

Assume 16mm diameter bar



No. of bars $= \frac{\text{Total Area}}{\text{Area of 1 bar}}$

$= \frac{993.6}{\frac{\pi}{4} \times 16^2}$

$= 4.94 \approx 5$

Provide 5 - 16mmϕ

Q2. A singly R/F rectangular beam of width 230mm is subjected to a BM 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.

$M = 40\,kN\text{-}m$

$M_d = 1.5 \times M = 1.5 \times 40 = 60\,kN\text{-}m$

$M_{u,lim} = M_d$

$\Rightarrow 0.138 f_{ck} b d^2 = 60 \times 10^6$

$\Rightarrow 0.138 \times 20 \times 230 \times d^2 = 60 \times 10^6$

$\Rightarrow d = 307\,mm \approx 310\,mm$

$D = d + \text{cover}$

$= 310 + 30mm$

$= 340mm$

A_{st} Calculation →

① $P_{t,lim} = 0.0118 f_{ck}$ → A_{st}

or,

②

$C_u = T_u$ → A_{st}

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

$\Rightarrow A_{st} = \dfrac{0.36 f_{ck} \cdot b \cdot x_{u,max}}{0.87 f_y}$ & $x_{u,max} = 0.48d$

$= \dfrac{0.36 \times 20 \times 230 \times 0.48 \times 310}{0.87 \times 415}$

$= 682.487\,mm^2$ (A is to be provided)

Check: $A_{st,min}$ formula for rectangular beam $= \dfrac{0.85bd}{f_y}$

$\Rightarrow A_{st,min} = \dfrac{0.85 \times 230 \times 310}{415}$

$= 146.03\,mm^2$

$A_{st,max}$ is 4% of c/s

$= 0.04\,bD$

$= 0.04 \times 230 \times 340$

$= 3128\,mm^2$

Q.8 A simply supported rectangular beam of effective span 4m carries a working load of 30kN/m. Determine size of beam & calculate no. of tension bars required. Use 16mm φ bars. Use concrete of grade M20 & Fe500. Assume ratio of depth to width of beam as 1.8 & 50mm effective cover.

16mm φ

M20, Fe415

$\frac{d}{b} = 1.8$

$\Rightarrow b = \frac{d}{1.8}$

Step 1 $\quad M_d = 1.5 \times M \qquad M = \frac{wL_e^2}{8}$

$= 1.5 \times 60$

$= 90\,kN\text{-}m$

Step-2 $\quad M_{u,lim} = M_d$

$0.133 f_{ck} bd^2 = 90kN\text{-}m$

$\Rightarrow 0.133 \times 20 \times \frac{d}{1.8} \times d^2 = 90 \times 10^6$

$\Rightarrow d^3 = \frac{90 \times 10^6 \times 1.8}{0.133 \times 20}$

$\Rightarrow d = 393mm \simeq 400mm$

$b = \frac{d}{1.8} = \frac{400}{1.8} = 222 \simeq 230mm$ (IS 456:2000)

$D = 400 + 50$

$= 450mm$

Step 8 $\quad p_{t,lim} = 0.038 f_{ck}$

$= 0.038 \times 20$

$= 0.764$

$$A_{st} = \frac{p_t \cdot bd}{100}$$

$$= \frac{0.764 \times 230 \times 400}{100}$$

$= 699.2\ mm^2$

No of bars $= \frac{\text{Total Area}}{\text{Area of 1 bar}}$

$$= \frac{699.2\ mm^2}{\frac{\pi}{4} \times 16^2\ mm^2}$$

$= 3.4 \simeq 4$

Provide 4 - 16mm φ

Q.7. Calculate depth & area of steel at midspan of a simply supported beam over a clear span 6m. The beam is carrying all inclusive load 20kN/m. Assume 300mm Bearings. Use M20 & Fe500

$= 6 + \frac{0.3}{2} + \frac{0.3}{2}$

$= 6.3m$

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

$$= \frac{20 \times 6.3^2}{8}$$

$= 99.225\ kNm$

Step 1 $M_d = 1.5 \times M$

$= 1.5 \times 99.235$

$= 148.8525$ kN-m

Step 2 $M_{u,lim} = M_d$ & Assume $b = \frac{d}{2}$

$\Rightarrow 0.133 f_{ck} b d^2 = 148.8525 \times 10^6$

$\Rightarrow 0.138 \times 20 \times \frac{d}{2} \times d^2 = 148.84 \times 10^6$

$\Rightarrow d^3 = \dfrac{148.84 \times 10^6 \times 2}{0.138 \times 20}$

$\Rightarrow d = 481.9$ mm $\simeq 500$ mm

$b = \frac{d}{2} = \frac{500}{2} = 250$ mm

$D = d +$ cover

$= 500 + 30$

$= 530$ mm

Step 3 : $p_{t,lim} = 0.038 f_{ck} = 0.038 \times 20 = 0.76\%$

$A_{st} = \dfrac{p_{t,lim}\, bd}{100}$

$= \dfrac{0.76 \times 250 \times 500}{100}$

$= 950 \text{ mm}^2$

Use 16 mm ϕ bars

No. of bars $= \dfrac{\text{Total Area}}{\text{Area of 1 bar}}$

$= \dfrac{950}{\frac{\pi}{4} \times 16^2}$

$= 4.72$

$\simeq 5$ nos

Provide 5 - 16 mm ϕ

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

$= \frac{0.85 \times 250 \times 345}{415}$

$= 212.5\,mm^2$

$A_{st,max} = 0.04\,bD$

$= 0.04 \times 250 \times 530$

$= 5300\,mm^2$

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 = 10 kN/m

} or, directly factored moment given i.e. 137 kNm

Size of beam is restricted to (250 × 400) mm. Assume mild exposure condition. Assume M20 & Fe415

b = 250mm
D = 400mm

For mild exposure cond^n, effective cover $d' = 35mm$ on both edges as steel to be provided on both edges (IS 456:2000)

Then, $d = D - d'$
$= 400 - 35$
$= 365mm$

$L_{eff} = 4.75m$

Step ⑤ Effective width & depth

$d = 365\,mm$

$b = 250\,mm$

Step ⑥ Effective length

$l_{eff} = 4.75\,m$

Step ⑦ Calculation of load

$Self\ wt = \rho_{RCC} \times b \times D$

$= 25 \times 0.25 \times 0.4$

$= 2.5\,kN/m$

$Total\ load = 2.5 + 10\,kN/m + 5 + 3.5 + 10$

$= 31\,kN/m$

$Factored\ load = 1.5 \times 31 = 46.5\,kN/m$

Step ⑧ Calculation of M_{max} & MOR

maximum Bending moment (v) moment of Resistance of sect^n

$M_{max} = \frac{wl^2}{8}$

$= \frac{46.5 \times 4.75^2}{8}$

$= 131.14\,kN\cdot m$ ← Factored Moment Given if Some Question ask

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

$= 0.138 \times 20 \times 250 \times 365^2$

$= 91.925\,kN\cdot m$

$M_{u,lim} < M_u$ or M_{max} ⇒ Provide doubly reinforced section

MOR $< M_{max}$ ⇒ Beam has to be designed as doubly reinforced.

Step 5 (v) Calculation of Area of Steel

(a) For balanced beam — A_{st_1} to provided to balance compression C_1 from Tension

$$M_{u1} = 0.87 f_y A_{st_1}(d - 0.42x_u) = T_u \cdot Z$$

$$\Rightarrow 91.925 \times 10^6 = 0.87 \times 415 \times A_{st_1}(d - 0.42x_u)$$

For Fe415, $x_u = 0.48d$
$= 0.48 \times 365$
$= 175.2\ mm$

$$\Rightarrow 91.925 \times 10^6 = 0.87 \times 415 \times A_{st_1}(365 - \overline{0.42 \times 175.2})$$

On solving, $A_{st_1} = 873.68\ mm^2$

(b) for remaining of load — Remaining moment

$$M_{u_{rem}} = 131.14 - 91.925$$
$$= 39.22\ kN\cdot m$$

For A_{st_2}:

$$M_u = 0.87 f_y A_{st_2} \times (d - d')$$

$$\Rightarrow 39.22 \times 10^6 = 0.87 \times 415 \times A_{st_2} \times (365 - 35)$$

$$A_{st_2} = 329.152\ mm^2$$

For A_{sc}:

$$M_u = \overbrace{(f_{sc} - f_{cc})A_{sc}}^{C_2 - \text{compression force}}(d - d')$$

For f_{sc} (SP16 - Table F) SP 16

$$\frac{d'}{d} = \frac{35}{365} = 0.095 \approx 0.1$$

$f_{sc} = 353\ N/mm^2$ $\quad f_{cc} = 0.446 f_{ck}$

So,
$$39.22 \times 10^6 = (353 - \overline{0.446 \times 20})A_{sc} \times (365 - 35)$$

$$\Rightarrow A_{sc} = 345.4\ mm^2$$

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

$= 329.166 + 873.68$

$= 1202.82\ mm^2$

Step (v) Reinforcement

(a) For tensile,

Provide bar size $\phi = 20\ mm$

$A_{ind} = \frac{\pi \times 20^2}{4} = 314.159\ mm^2$

$\text{No. of bars} = \frac{A_{st}}{(A_{st})_{individual}}$

$= \frac{1202.846}{314.159} \approx 3.8 \approx 4\ nos$

$A_{st\ provided} = 4 \times 314.159 = 1256.64\ mm^2$

(b) For compression

$\phi = 16\ mm$

$(A_{st})_{individual} = \frac{\pi}{4} \times 16^2 = 201.06\ mm^2$

$\text{No. of bars} = \frac{A_{sc}}{(A_{st})_{individual}}$

$= \frac{245.365}{201.06}$

$= 1.2 \approx 2\ nos$

$A_{sc\ provided} = 2 \times 201.06 = 402.12\ mm^2$

Step 5 (vi) Check for reinforcement

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

$= \frac{0.85 \times 250 \times 365}{415}$

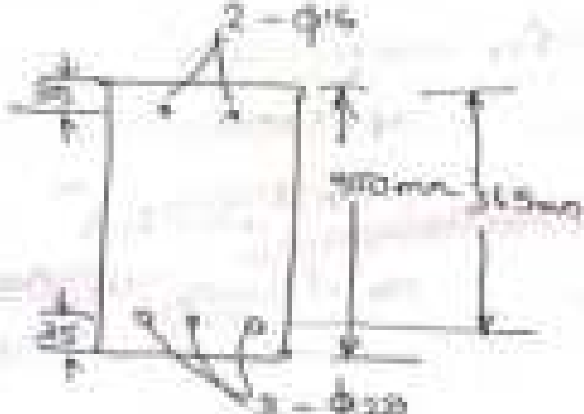
$= 186.89\ mm^2 < A_{st\ provided}$ (O.K.)

$$A_{st,max} = 4\% \text{ of gross area}$$
$$= 0.04 \times b \times D$$
$$= 0.04 \times 250 \times 400$$
$$= 4000 \text{ mm}^2 > 1056.636 \text{ (O.K.)}$$

For compression,

$$(A_{sc})_{max} = 0.04\, bD$$
$$= 0.04 \times 250 \times 400$$
$$= 4000 \text{ mm}^2 > 402.12 \text{ mm}^2 \text{ (O.K.)}$$

Step-8 (a) Reinforcement Detail:



Ch-4 Shear Reinforcement

Q.1 A R.C.C. beam 250mm × 400mm effective is 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

$b = 250mm$

$d = 400mm$

$w = 15 kN/m$

$L = 4m$

$A_{st} = 4 \times \frac{\pi}{4} \times 22^2 = 1520.5 mm^2$

$f_{ck} = 20 N/mm^2$ & $f_y = 250 N/mm^2$

Step-1 Find out factored shear force

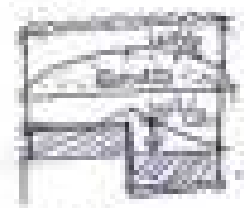
Load = 15 kN/m

Factored Load = 1.5 × 15 = 22.5 kN/m

Factored Shear Force = $\frac{wL}{2}$

$= \frac{22.5 \times 4}{2}$

$= 45 kN$



SFD

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 M20 = $2.8 N/mm^2$ [IS 456 Table 20]

$\tau_v < \tau_{c\,max}$ → Sectn safe (OK)

if $\tau_v > \tau_{c\,max}$ → Sectn unsafe → Redesign

Step-3 Design shear strength of concrete (τ_c)

It depends on P_t % & Grade of concrete

i. $P_t = \frac{A_{st}}{bd} \times 100$

$= \frac{1526.5}{250 \times 457} \times 100$

$= 1.3\%$

& M20

$\Rightarrow \tau_c = 0.72\ N/mm^2$ (Table 19, IS456)

$\tau_c > \tau_v \Rightarrow$ No shear reinforcement is required but IS code recommends that "nominal shear reinforcement" must be provided.

Step-4 Nominal Shear Reinforcement

$\frac{A_{sv}}{b S_v} \geq \frac{0.4}{0.87 f_y}$ (26.5.1.6)

Using 8mm ϕ bars & 2 legged stirrups

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

$= 100.5\ mm^2$

$\Rightarrow S_v = \frac{A_{sv} \times 0.87 f_y}{0.4 \times b} = \frac{100.5 \times 0.87 \times 250}{0.4 \times 250}$

$= 218\ mm$

Step-5 Check for spacing of shear reinforcement

Use Vertical Stirrups. * Vertical / bent-up depend on us to choose type of stirrups.

Spacing should be least of

min^m of
(i) $0.75d = 0.75 \times 457 = 342\ mm$
(ii) 300 mm
(iii) $S_v = 218\ mm$

Provide 8mm ϕ bar, 2 legged stirrups @ 300mm spacing c/c throughout the length of beam.

Q2. A simply supported RC beam 250mm wide & 450mm deep (effective) is reinforced with 4-18mm diameter bars. Design the shear reinforcement for M20 grade of concrete & Fe415 steel is used & beam is subjected to a shear force of 150kN at service load.

Soln:

Given

$b = 250mm$

$d = 450mm$

$A_{st} = 4 \times \frac{\pi}{4} \times 18^2 = 1018\ mm^2$

$f_{ck} = 20\ N/mm^2$, $f_y = 415\ N/mm^2$, $V = 150\ kN$

Step-I Factored Shear force

$$V_u = 1.5\ V$$
$$= 1.5 \times 150$$
$$= 225\ kN$$

Step-II 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)

$\tau_{c,max} > \tau_v$ → Hence (O.K.)

Step-III Design shear strength of concrete (τ_c)

It depends on (i) p_t % &

(ii) 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

0.75 ——— $0.56\ N/mm^2$

0.9

1 ——— $0.62\ N/mm^2$

$\tau_c = 0.56 + \frac{0.62 - 0.56}{1 - 0.75}(0.9 - 0.75)$

$= 0.596\ N/mm^2$

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

compare : $\tau_v > \tau_c$, hence shear reinforcement is to be provided

Step-v **Design of Shear Reinforcement**

Shear taken by stirrups = V_{us} = design shear force

$V_{us} = V_u - V_c$

= factored shear force − shear resisting capacity of sec^n

$= V_u - \tau_c \cdot bd$

$= 220000 - 0.59 \times 250 \times 450$

$= 157750\ N$

For Vertical Stirrups - 8mm φ bar 2 legged,

$V_{us} = \frac{0.87 f_y A_{sv} \cdot d}{S_v}$ $\quad A_{sv} = 2 \times \frac{\pi}{4} 8^2 = 100$

$\Rightarrow S_v = 103mm$

Spacing of nominal reinforcement

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

$\rightarrow S_v = 362mm$

step - V Check for spacing

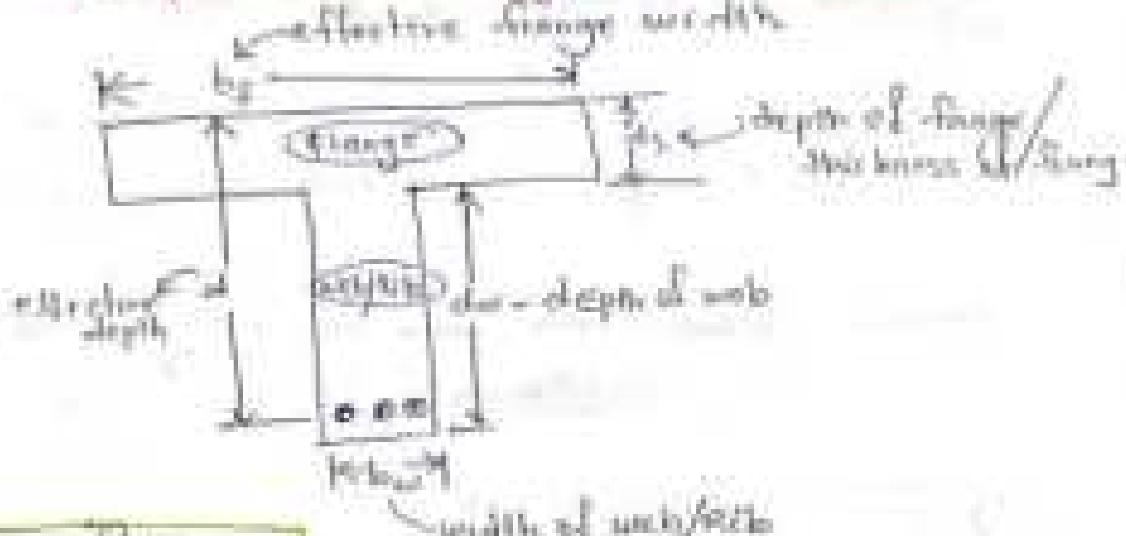
For vertical stirrups, spacing should be minimum of following

$= \min \begin{cases} \text{(i)} & 0.75d = 0.75 \times 450 = 337mm \\ \text{(ii)} & 300mm \\ \text{(iii)} & S_v = 103mm \text{ from design s/r.} \\ \text{(iv)} & S_v = 362m \text{ from nominal s/r} \end{cases}$

$= 103mm$

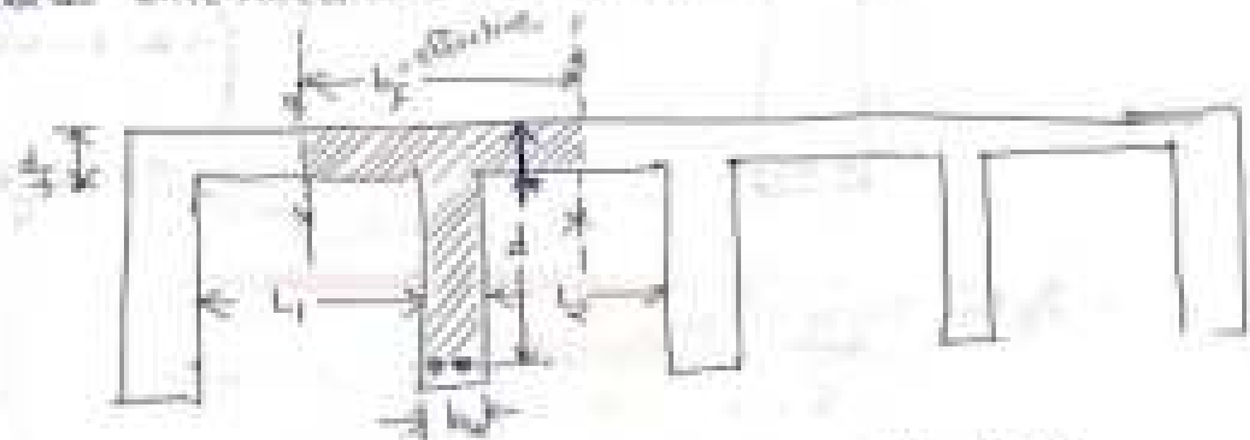
∴ Provide 2 legged stirrups of 8mm ϕ bars @ 103 mm spacing c/c.

Ch-5 Analysis & Design of T-Beam



Effective flange Width Type-1 Numerical

Case-1 Intermediate T-beam



Case for Beam casted monolithic with slab

$$\text{(1)} \quad b_f = \frac{l_0}{6} + b_w + 6D_f$$

$$\text{(2)} \quad b_f = \frac{L_1}{2} + \frac{L_2}{2} + b_w$$

min of (1) & (2) $= b_f$

Case-2 Isolated T-beam

$$b_f = \frac{l_0}{\frac{l_0}{b} + 4} + b_w$$

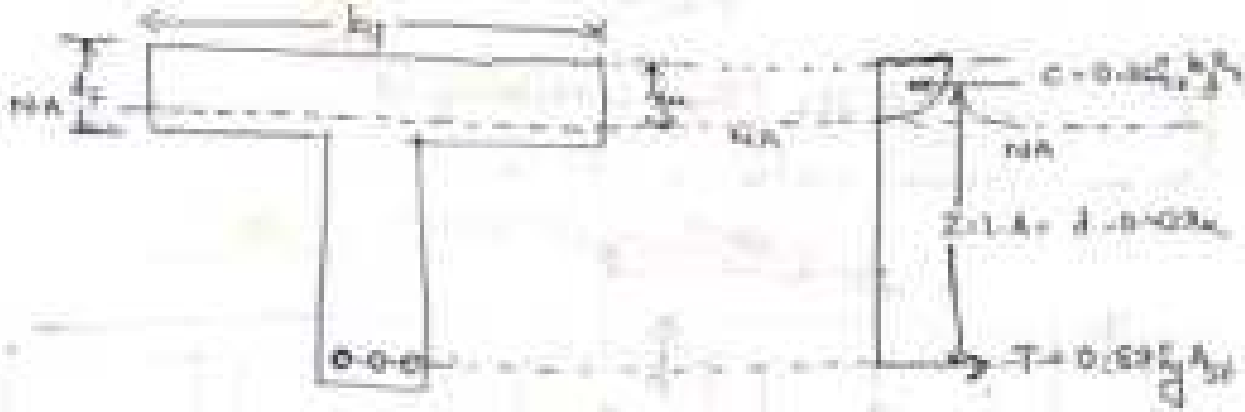
Moment Of Resistance Type-6 Numerical

Case-I — NA lies in flange

Case-II, Case-III — NA lies in web

Case-I When NA lies in flange section

$$\boxed{x_u < d_f}$$



- 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 → $x_{u\,max} = 0.53 d$

Fe 415 → $= 0.48 d$

Fe 500 → $= 0.46 d$

Find: U/R, O/R, or B/R secn

Moment of Resistance

O/R: $\boxed{\begin{aligned} MR_c &= C \times LA \\ &= 0.36 f_{ck} b_f x_u (d - 0.42 x_u) \end{aligned}}$

U/R: $\boxed{\begin{aligned} MR_T &= T \times LA \\ &= 0.87 f_y A_{st} (d - 0.42 x_u) \end{aligned}}$

Ch-6 Analysis & Design of Slab & Staircase

- Slabs are of 2 types :
  1. One-way slab
     - Spanning in one dirⁿ
     - Tension Reinforcement provided only in shorter span
     - Deflects in 1 dirⁿ
  2. Two-way slab
     - Spanning in two directⁿ
     - Provided in both directⁿ
     - Slab deflects in both dirⁿ

* Aspect Ratio of Slab determines the type of slab.

$$A_r = \frac{\text{Longer Span}}{\text{Shorter Span}} = \frac{l_y}{l_x} > 2 \text{ (One-way slab)}$$

$$\frac{l_y}{l_x} \le 2 \text{ (Two-way slab)}$$

Q. Design a slab of size 3m × 6.2m for a Living room of a Residential building. Take floor finish as 1.5 kN/m². 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 (limit deflectⁿ)

$$\frac{l}{d} = 20 \quad \text{or} \quad d = \frac{\text{Span (shorter)}}{\text{Basic Value} \times \text{Modification factor}} \quad \text{Cl. 23.2.1}$$

$$\Rightarrow \frac{l}{d} = 20 \times \text{M.F.}$$

$$= \frac{3000}{(20) \times 1.5 \text{ (Assume)}}$$

$$= 100\text{mm}$$

$$\approx 125\text{mm}$$

$D = 125 + 5 + 20$

$= 150\,mm$

Step ii) Effective span of slab (Cl 22.2)

L_{eff_1} = Shorter span + eff depth

$= 3m + 0.125m$

$= 3.125m$

L_{eff_2} = Span of slab + size of end support

$= 3 + 0.3$

$= 3.3\,mm$

$L_{eff} = \min \text{ of } \begin{cases} ① \; L_{eff_1} \\ ② \; L_{eff_2} \end{cases}$

Ans: $L_{eff} = 3.125m$

Step iii) Load calculation

Dead load $= \overbrace{25 \times 0.15}^{\gamma_c \times D} = 3.75\,kN/m^2$

Floor finish $= 1.5\,kN/m^2$

Live load = Assume $4\,kN/m^2$

Total = $9.25\,kN/m^2$

Load per metre $= 9.25\,kN/m^2 \times 1m$

$\Rightarrow w = 9.25\,kN/m$

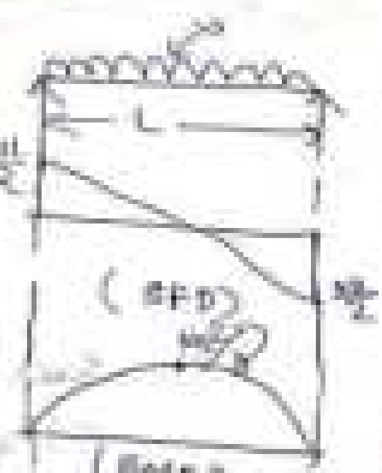
Step iv) $M = \frac{wl^2}{8}$ Calculation of Moment

$= \frac{9.25 \times 3.125^2}{8}$

$= 11.283\,kN\cdot m$

$M_d = 1.5 \times M$

$= 16.93\,kN\cdot m$



Step-V Check for Depth (Calculate Depth of Slab using Bending Moment Consideration)

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

$x_{u\,max} = 0.48\,d \rightarrow$ for Fe415

Assume 1m width

$\Rightarrow 16.93 \times 10^6 = 0.36 \times 20 \times 0.48\,d \times 1000 \times (d - 0.42 \times 0.48\,d)$

$\Rightarrow d = 78\,mm < 125\,mm$ (Safe)

Step-VI Calculation of Steel Area

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

$$= 0.5 \times \frac{20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 16.93 \times 10^6}{20 \times 1000 \times 125^2}}\right] \times 1000 \times 125$$

$= 402.16\,mm^2$

Spacing per unit width of slab $= \frac{\text{Area of 1 bar}}{\text{Area reqd}} \times 1000$ width

$= \frac{\frac{\pi}{4} \times 10^2}{402.16} \times 1000$

$= 195\,mm$ (Safe)

Cl. 26.3.3

Spacing $<$ { (1) $3d = 3 \times 125 = 375$; (2) $300\,mm$ }

$195\,mm$ (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 $=$ 402.16 mm^2 (O.K.)

Part - B $\boxed{\text{Checks}}$

Deflect^n check, Development length check, Shear check

① Deflect^n check CL 23.2.1

$$f_s = 0.58 f_y \left[\frac{A_{st\,req}}{A_{st\,prov}}\right]$$ CL 23.2.1 — depends on change of spans

$$= 0.58 \times 415 \times \frac{402.16}{402.16}$$

$$= 240.7\,N/mm^2$$

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

$$= \frac{402.16}{1000 \times 125} \times 100$$

$$= 0.197$$

Use Graph ; MF = 1.7 (modification factor)

$$d = \frac{\text{Span}}{20 \times MF}$$

$$= \frac{3500}{20 \times 1.7} = 98.02\,mm$$ < 115 mm (O.K.) if greater than then slab is not safe for deflect^n

② Check for shear (Cl - 40.2.1.1)

Shear $V = \frac{wL}{2} = \frac{9.25 \times (3)}{2}$ — # Here, we have to take clear span instead of eff (for shear force)

$V_u = 1.5\,V$

$= 1.5 \times$

$= 20.81$ kN

Nominal shear stress
$$\tau_v = \frac{V_u}{bd} = \frac{20.81 \times 10^3}{1000 \times 125} = 0.16\ N/mm^2$$

$p_t = 0.19\%$ for central part of slab

design shear strength $\tau_c = 0.28\ N/mm^2$ for M25

$p_t = \frac{1}{2} \times$ central, for support its half

$= \frac{1}{2} \times 0.19\%$

$= 0.095\%$

For 150mm slab $k = 1.3$ Cl. 40.2.1.1

$k\tau_c = 1.3 \times 0.28$

$= 0.364\ N/mm^2$

$\tau_v < k\tau_c$ (safe in shear)

(9) Check for Development length Cl 26.2.3.3

$$\frac{M}{V} + L_o > L_d \quad \text{or} \quad \frac{1}{3}\frac{M}{V} + L_o > L_d$$

M → Moment

V → Shear force

L_o → length available from the centre of support to the face of support

L_d → development length Cl 26.2.1

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

ϕ = dia of bar

τ_{bd} = bond stress from table P.43

$$= \frac{10 \times 0.87 \times 415}{4 \times 1.2 \times 1.6}$$

increased by 60%

L_o = max of (1) d = 125, (2) 12ϕ = 120

$= 470.11$ mm

$$\frac{16.93 \times 10^6}{20.15 \times 10^3} + L_o > L_d$$

$= 838.14$

L_o (125mm)

$\therefore$ Safe in L_d

Last Step - "Distribution" bar Steel Area

Area of Steel Reqd for "Distribution bar"

- Fe250 = 0.15% bd
- Fe415/Fe500 = 0.12% bd

$$A_{st_d} = \frac{0.12}{100} \times bd$$

$$= \frac{0.12}{100} \times 1000 \times 125$$

$$= 150\,mm^2$$

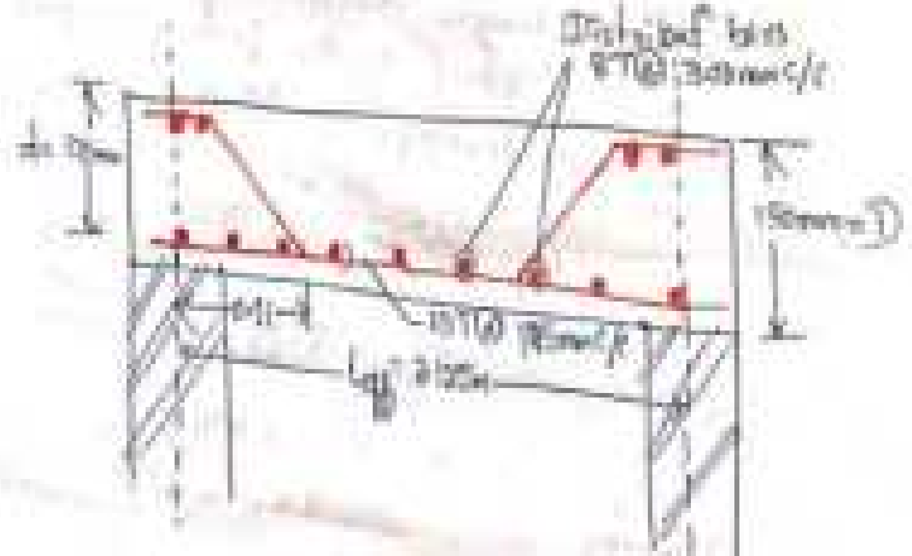
Assume 8mm bar

Spacing (for distribution) $= \dfrac{\text{Area of 1 bar}}{A_{st_d}} \times 1000$

$$= \frac{\frac{\pi}{4} \times 8^2}{150} \times 1000$$

$$= 335.10\,mm$$

Check $< \begin{cases} 5d \\ 500mm \end{cases}$ (OK)



Cantilever Slab — Chajja / Balcony

Cond^n — Cant. span ... ; eff. depth span/10 to span/12 ; 1/2 to 1/3 thickening ; Reinf at top

Q. Design a cantilever slab for an overhang 2.1m. The live load on slab consists of 3 kN/m² & floor finish is 0.5 kN/m². Use M25 & Fe415

Step-1 Depth of Slab

$$\text{effective depth} = \left[\frac{\text{Span}}{10} \text{ to } \frac{\text{Span}}{12}\right] \text{@ fixed end}$$

$$d = \frac{\text{Span}}{12} = \frac{2100}{12} = 175\text{mm}$$

$$D = d + \text{clear cover} + \frac{1}{2}\,\text{dia}$$

$$= 175 + 20 + \frac{1}{2}\times 10$$

$$= 200\text{mm}$$

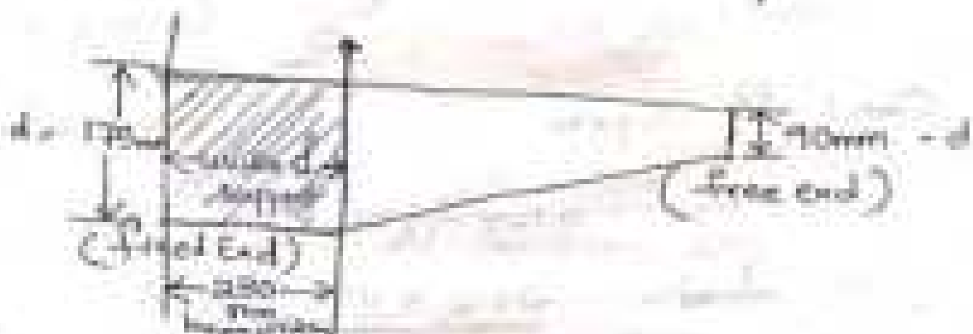
Step-2 Effective Span

$$\text{Effective span} = \frac{\text{effective depth}}{2} + \text{clear span}$$

$$= \frac{175}{2} + 2100$$

$$= 2187.5\text{mm}$$

$$= 2.18\text{m}$$



$$= \frac{1}{2} \text{ to } \frac{1}{3} \times \text{effective depth @ fixed end}$$

$$= \frac{1}{2}\times 175$$

$$= 87.5\text{mm}$$

$$\simeq 90\text{mm}$$

Step-III Total Load Calculation

Self weight of slab $= \gamma_{conc} \times 0.2 \times 1$

$= 25 \times 0.2 \times 1$

$= 5 \ kN/m$

L.L $= 1 \ kN/m^2 = 1 kN/m^2 \times 1m = 1 kN/m$

F.F $= 0.8 \ kN/m^2 = 0.8 \ kN/m^2 \times 1m = 0.8 kN/m$

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

$$\text{Max}^m \ BM = \frac{wL^2}{2}$$

$$M_u = \frac{10.2 \times 2.18^2}{2}$$

$$= 24.2 \ kN\text{-}m$$

$$\text{Max}^m \ SF = w \times L \quad (L = \text{clear span})$$

$$= 10.2 \times 2.1$$

$$= 21.42 \ kN$$

~~Check for depth~~

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

$\cancel{\Rightarrow d = 24.2 \times 10^6}$

Step V Check for Depth

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

$$\Rightarrow 24.248 \times 10^6 = 0.36 \times 25 \times 0.48d \times b (d - 0.42 \times 0.48 d)$$

$$\Rightarrow 24.248 \times 10^6 = 0.36 \times 25 \times 0.48d \times 1000 (d - 0.42 \times 0.48 d)$$

$$\Rightarrow 24.248 \times 10^6 = 3447.002 d^2$$

$$\Rightarrow d = 83.76 \text{mm} < 175 \text{mm or } 90 \text{mm (O.K)}$$

consider both value safe

Step-VI Calculation of steel Area

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

$$= \frac{0.5 \times 25}{415} \left[1 - \sqrt{1 - \frac{4.6 \times 24.248 \times 10^6}{25 \times 1000 \times 175^2}}\right] 1000 \times 175$$

$$= 385 \text{mm}^2 \quad A_{st,reqd}$$

$$A_{st,min} = 0.12\% \, bD \rightarrow \text{for Distribution steel of } fe415$$

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

$$= 240 \text{mm}^2 < A_{st} \quad (O.K)$$

Take # 8mm, $a_{st} = \frac{\pi}{4} \times 8^2 = 50.3 \text{mm}^2$

$$\text{Spacing} = \frac{a_{st}}{A_{st}} \times \text{width}$$

$$= \frac{50.3}{385} \times 1000$$

$$= 130.64 \text{mm} \approx 130 \text{mm}$$

∴ Provide # 8mm @ 130mm c/c

$$A_{st,Provided} = \frac{a_{st}}{\text{Spacing}} \times 1000$$

$$= \frac{50.3}{130} \times 1000$$

Check for Spacing

$$\text{min}^m \text{ of } \begin{cases} ① \; 3d = 3 \times 175 = 525\,mm \\ \quad\quad 3 \times 90 = 270\,mm \\ ② \; 300\,mm \end{cases}$$

$= 270\,mm$

Spacing $130\,mm < 270\,mm$ (O.K.)

* Bar as main reinforcement & distribution reinforcement 8mm #

Step. VI Check for shear

$V_u = 21.42\,KN$

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

$= \frac{21.42 \times 10^3}{1000 \times 175}$

$= 0.12 \; N/mm^2$

$\tau_c \rightarrow P_t = \frac{A_{st}}{bd} \times 100 = \frac{100 \times 386.9}{1000 \times 175}$
$\rightarrow M25$
Table-19 $= 0.221\%$

P_t	τ_c
0.15	0.29
0.221	???
0.25	0.36

By interpolation $\tau_c = 0.339\,N/mm^2$

$\tau_c > \tau_v$ (O.K.)

No Shear R/F Reqd

Step - 12 Check for deflection

Max^m deflect^n $\left(\frac{l}{d}\right)_{max}$ = 7 × M.F (IS 456)

$f_s = 0.58\, f_y \dfrac{A_{st\ reqd}}{A_{st\ prov}}$

$= 0.58 \times 415 \times \dfrac{355}{386.72}$

$= 219.5\ N/mm^2$

From Graph,

M.F = 2

$\left(\frac{l}{d}\right)_{max} = 7 \times 2$

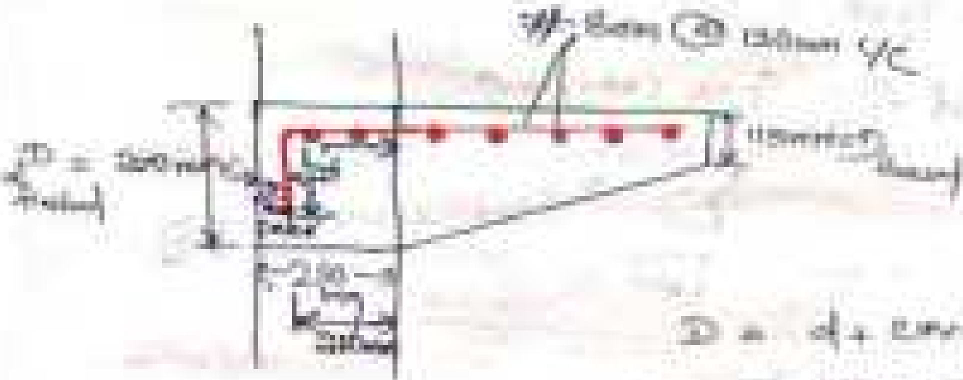
= 14 mm

$\left(\frac{l}{d}\right)_{provided} = \dfrac{\text{clear span}}{\text{eff. depth}}$

$= \dfrac{2100}{175}$

= 12 mm

$\left(\frac{l}{d}\right)_{max} > \left(\frac{l}{d}\right)_{prov}$ —— (o.k.)



$D = d + \text{cover} + \frac{\phi}{2}$

$= 90 + 20 + \frac{10}{2}$

Main Reinforcement :- At Top of slab 115 mm in case of cantilever beam

Development length

$L_d = \dfrac{0.87 f_y\, \phi}{4 \tau_{bd}}$

τ_{bd}: M25 → 1.4, Fe415 →

$= \dfrac{0.87 \times 415 \times 8}{4 \times 1.4 \times 1.6}$

= 322 mm

In width 230 - 20 = 210 mm
In 7 dm² l_d = 150 mm

Provide l_d = 350 mm in the support

Q. Design a RCC slab for a room 6.3 m × 4.5 m. The slab to be cast monolithically over the beams with the sides simply supported. It has to be carry a characteristic load of 10 kN/m² in addition to the own weight. Use M20 concrete & Fe415.

$$\text{Aspect Ratio } \lambda_y = \frac{l_y}{l_x}$$

$$= \frac{6.3}{4.5}$$

$$= 1.4 \leq 2 \text{ (Two way slab)}$$

Step 1 Depth of slab
(Cl 23.2.1) Pg 38

$$d = \frac{\text{Span (shorter)}}{B.V \times M.F}$$

Basic value (20)

Modification factor [1.2 – 1.3]

$$= \frac{4500}{20 \times 1.8}$$

$= 150$ mm ≤ 175 mm Take greater value

effective cover = 25 mm

$D = 175 + 25 = 200$ mm

Step-II Span of the slab

Cl - 22 - 2 - a - Pg 34

$$\min\text{ of }\begin{cases} l_{eff} = l_x + \text{depth} = 4500 + 175 = 4675\text{m} \\ l_{eff} = l_x + \text{c/c between support} \end{cases}$$

$$L_{eff} = 4.675\text{m}$$

Step-III Load Calculation

$$DL = 25 \times 0.2$$

$$= \gamma \times D$$

$$= 5\ \text{KN/m}^2$$

$$LL = 10\ \text{KN/m}^2$$

$$\text{Total} = \frac{15\ \text{KN/m}^2}{1\text{m width}} = 1.5\ \text{KN/m}$$

$$\text{Factored Load} = 1.5 \times 15 = 22.5\ \text{KN/m}$$

Step-IV Calculation of Moments (IS456 Pg 91 [D-2])

$$\alpha_x = 0.079$$

$$\alpha_y = 0.051$$

$$M_x = \alpha_x w l_x^2 \qquad M_y = \alpha_y w l_x^2$$

$$= 0.079 \times 22.5 \times 4.675^2 \qquad = 0.051 \times 22.5 \times 4.675^2$$

$$= 48.68\ \text{KN·m} \qquad = 25.07\ \text{KN·m}$$

Design for max^m moment

$$\therefore M_u = 48.68\ \text{KN·m}$$

Step-V Check for depth

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

$$\Rightarrow 48.68 \times 10^6 = 0.36 \times 25 \times 0.48d \times 1000 (d - 0.42 \times 0.48d)$$

$$\Rightarrow d = 118\text{mm} < 175\text{mm} \quad (\text{O.K.})$$

Step-10 Calculation of steel in shorter span

$$A_{st,x\text{-}dir^n} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M}{f_{ck} b d^2}}\right] bd$$

$$A_{st_x} = \frac{0.5 \times 25}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 40.68 \times 10^6}{25 \times 1000 \times 175^2}}\right] \times 1000 \times 175$$

$$= 800\ mm^2$$

Let, 12ϕ Spacing = $\frac{\text{Area of 1 bar}}{A_{st\ req}} \times 1000$

$$= \frac{\frac{\pi}{4} \times 12^2}{800} \times 1000$$

$$= 141\ mm$$

$$\approx 140\ mm < \min^m \text{ of } \begin{cases} 3d = \\ 300 \end{cases}$$

(O.K.)

Provide 12T @ 140mm c/c

Step-11 Calculation of steel in longer span

$$A_{st,y\text{-}dir^n} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M}{f_{ck}\, b\, (163)^2}}\right] bd$$

$$= \frac{0.5 \times 25}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 25 \times 10^6}{25 \times 1000 \times 163^2}}\right] \times 1000 \times 163$$

Remember

Effective depth in y dir^n = 200 − 5 − 6 − 20 = 163 mm

$$= 418\ mm^2$$

A_{st} reqd, Spacing $= \frac{\frac{\pi}{4}\times 12^2}{445}\times 1000 = 253\,mm$

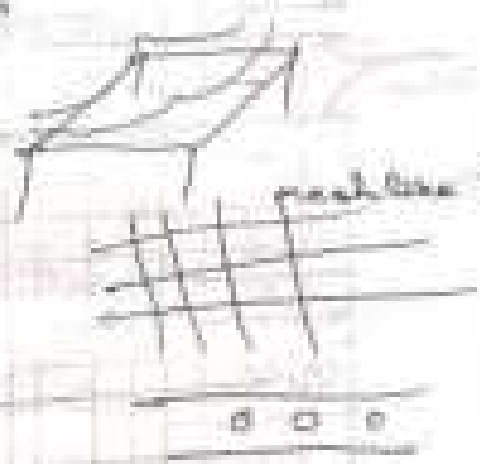
$\simeq 250\,mm$

Provide 12⌀ @ 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 10 (D-1.8)



A_{st} torsion $= \frac{3}{4} A_{st,x}$

$= \frac{3}{4}\times 880$

$= 605\,mm^2$

size of mesh $= \frac{l_x}{5}$

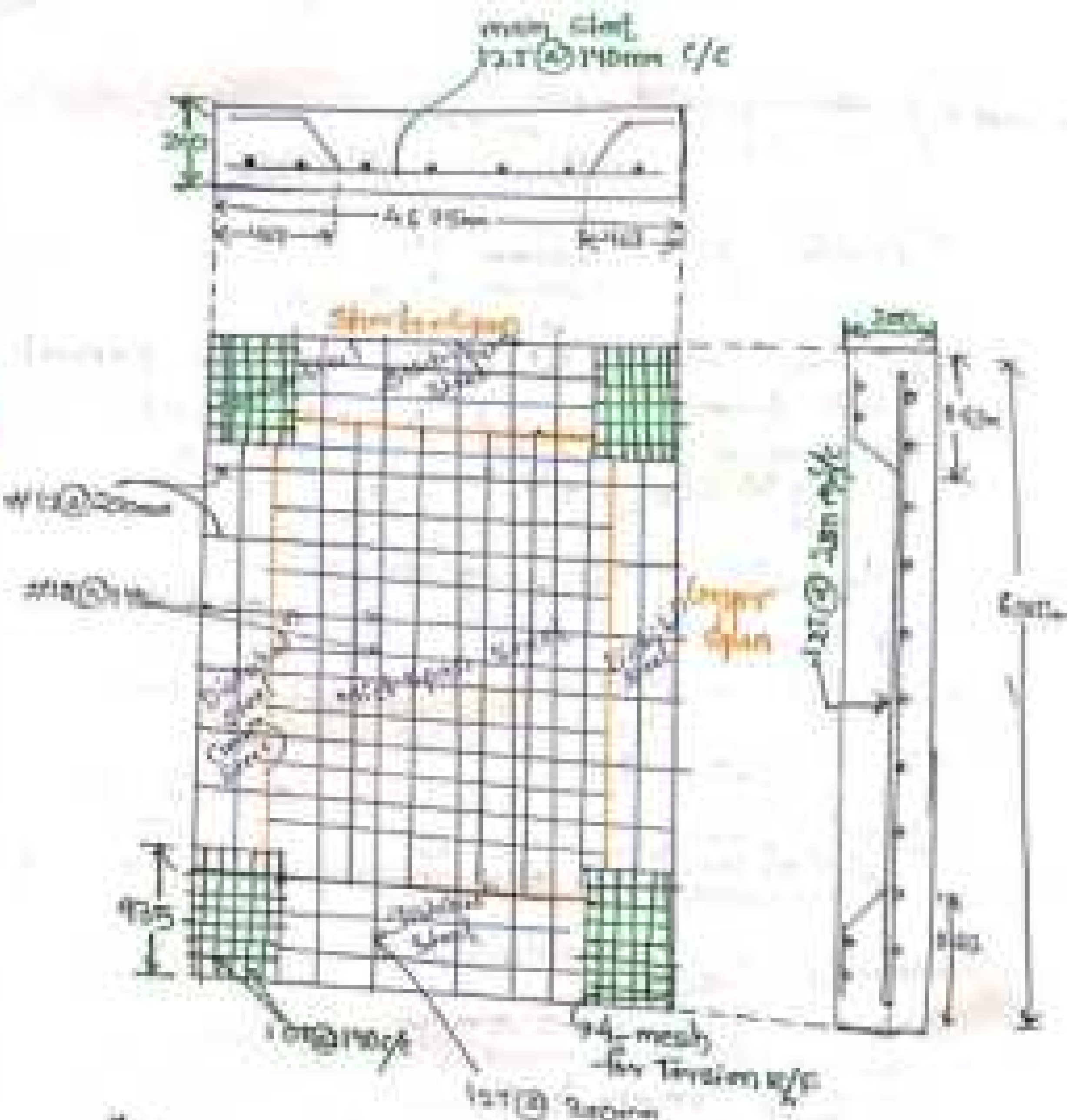
$= \frac{4675}{5}$

$= 935\,mm$

Use 10mm bar

Spacing $= \frac{\frac{\pi}{4}\times 10^2}{605}\times 1000$

$= 130\,mm$



* All three checks are same for 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 of 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 x 6m. Floor to floor height is 3.5m. Stairs are supported on brickwalls 230mm thick at the end of landings. Use M_{20} & 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 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 of Riser − 150mm (for public building)

Ideal [150mm − 175mm]

No. of risers = $\frac{1.75m}{0.15m}$ = $\frac{ht}{riser}$

= 11.67

≃ 12 no.

12+12 = 12×2 = 24
total tread = 24 − 1 = 23

No. of Treads = No. of risers − 1

= 12 − 1

= 11 no.

Let, width of each tread = 300 mm

Total going = 11 × 300 = 3300 mm = 3.3 m

Total distance = 6 m

Width of each landing = $\frac{6 - 3.3}{2}$ = 1.35 m = 1350 mm

Step-II Effective Span

Effective span = clear span + ½ of bearings

= 6 + $\frac{0.23}{2}$ + $\frac{0.23}{2}$

= 6.23 m

= 6230 mm

Step-III Thickness of waist slab

Thickness = $\frac{1}{20}$ of span (approx.) Ref. SP-16

D = $\frac{6230}{20}$

≃ 311.5 mm, ensure higher

Take: $\boxed{D = 325 \text{ mm}}$; d $\boxed{= 300 \text{ mm}}$ — waist slab

effective cover = 25 mm

D = 0.325 m

d = 0.3 m

Step-IV Loads

Loads
- Stairs load
- Landing Load

(a) Stairs loading:

Remember

weight of waist slab $= D\sqrt{1+\frac{R^2}{T^2}} \times (25)$ (D = 325 / 0.325)

Riser, Tread

$= 0.325\sqrt{1+\frac{(0.15)^2}{(0.3)^2}} \times 25$

$= 9.1\ kN/m$

weight of steps $= \frac{25RT}{2T}$

$= \frac{25 \times 0.15 \times 0.3}{2 \times 0.3}$

$= 1.875\ kN/m$

Total D.L $= 9.1 + 1.875 = 10.975\ kN/m$

LL $= 5\ kN/m^2 \times 1m = 5\ kN/m$ (10.975)

(+)

Total Load $= 15.975\ kN/m$

Factored Load $= 1.5 \times 15.975$

$= 24\ kN/m$

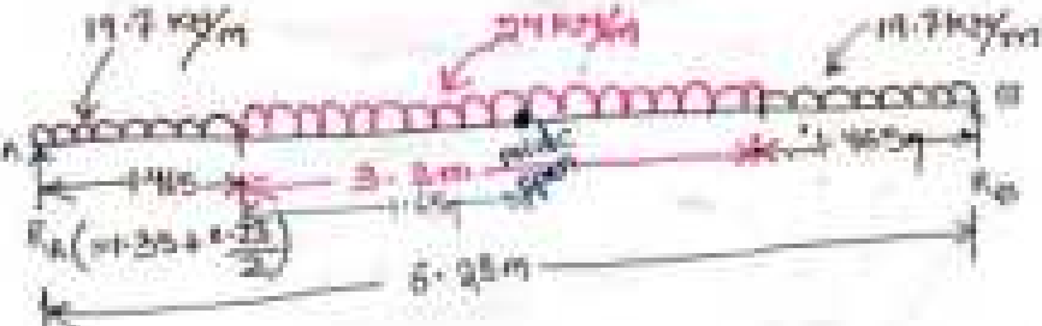
(b) Landing loading

DL $= 0.325 \times 1 \times 25 = 8.125\ kN/m$

(+) LL $= 5 \times 1 = 5\ kN/m$

Total Load $= 13.125\ kN/m$

Factored Load $= 1.5 \times 13.125 = 19.7\ kN/m$



Design Moment

$$R_A + R_B = 19.7 \times 1.465 + 24 \times 3.3 + 19.7 \times 1.465$$

$$\Rightarrow R_A + R_B = 136.92\,kN \quad \text{——— (1)}$$

Taking Moment at B.

$$\Sigma M_B = 0$$

$$\Rightarrow R_A \times 6.23 - 19.7 \times 1.465 \times \left(\frac{1.465}{2} + \overset{c.g}{3.3 + 1.465}\right)$$

$$- 24 \times 3.3 \times \left(\frac{3.3}{2} + 1.465\right) - 19.7 \times 1.465 \times \underset{c.g}{\left(\frac{1.465}{2}\right)}$$

$$= 0$$

$$\Rightarrow R_A = 68.5\,kN \quad \text{——— (2)}$$

$$R_B = 136.92 - 68.5 = 68.44 \simeq 68.5$$

Bending Moment at mid span

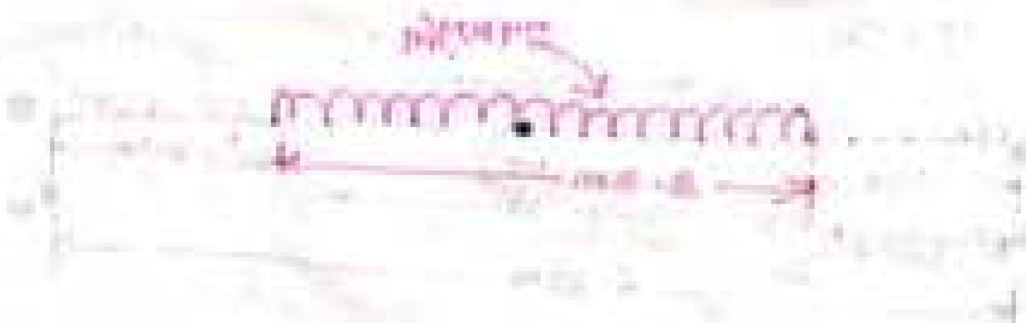
$$M_C = 68.5 \times (1.465 + 1.65)$$

$$- 19.7 \times 1.465 \times \left(\frac{1.465}{2} + 1.65\right)$$

$$- 24 \times 1.65 \times \frac{1.65}{2}$$

$$= 111.94\,kN\text{-}m$$

$$\simeq 112\,kN\text{-}m$$



step-V Area of Reinforcement

$$M_u = 0.87 f_y A_{st} d\left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$ Annex G Pg-96 IS 456

$$\Rightarrow 112 \times 10^6 = 0.87 \times 415 \times A_{st} \times 300\left(1 - \frac{A_{st} \times 415}{1000 \times 300 \times 20}\right)$$

$\Rightarrow A_{st} = 1117\,mm^2$

Take 16 mm ϕ bar

$$\text{Spacing} = \frac{\text{Area of 1 bar}}{\text{total area}} \times \text{width}$$

$$= \frac{\frac{\pi}{4} \times 16^2}{1117} \times 1000$$

$$= 179.9$$

$$\approx 180\,mm$$

$\therefore$ Provide 16mm ϕ @ 180mm c/c (main bar)

Distribution steel = 0.12% bD

$$= \frac{0.12}{100} \times 1000 \times 325$$

$$= 390\,mm^2$$

Take 10mm ϕ bar

$$\text{Spacing} = \frac{\text{Area of 1 bar}}{\text{total area}} \times \text{width}$$

$$= \frac{\frac{\pi}{4} \times 10^2}{390} \times 1000$$

$$= 201$$

$$\approx 200\,mm$$

$\therefore$ Provide 10mm ϕ @ 200mm c/c

Step-11 Development length (for [illegible] of main bar → B.M)

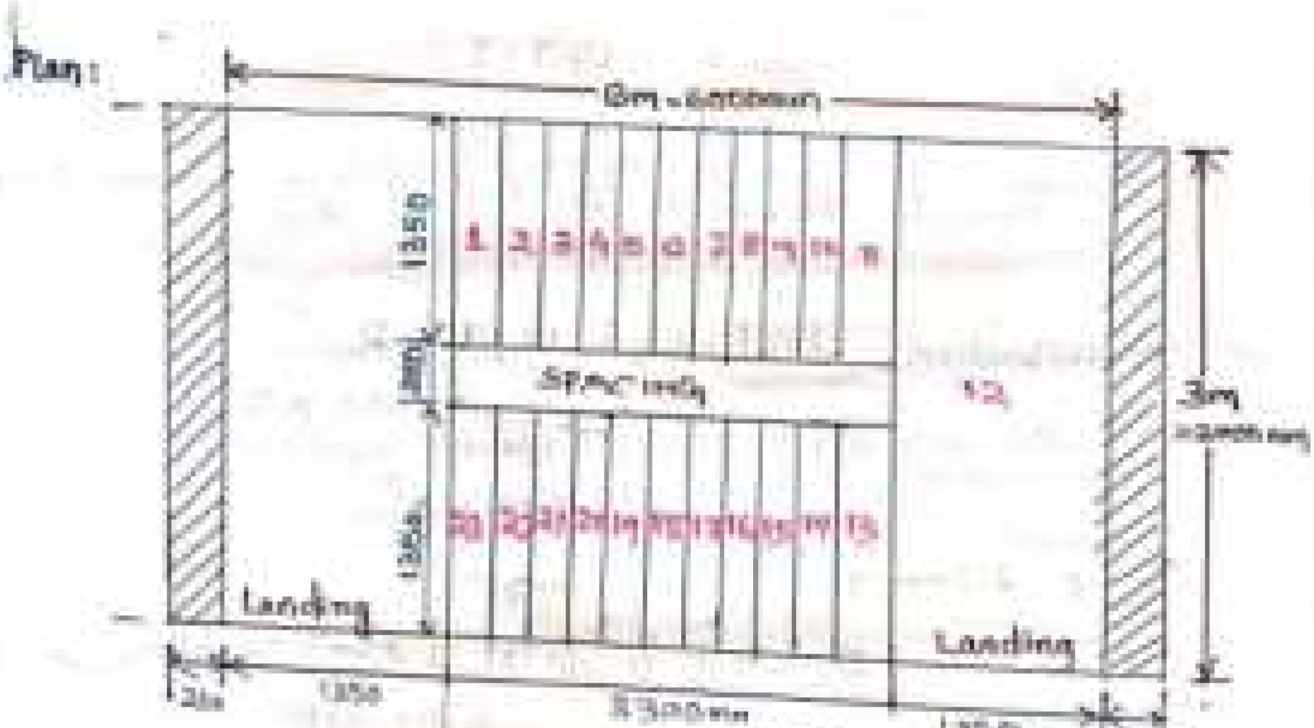
$$L_d = \frac{(0.87 f_y)\phi}{4\tau_{bd}} \qquad \text{Cl-26.2.1, IS456}$$

$$= \frac{0.87 \times 415 \times 16}{4 \times 1.6 \times 1.2} \quad \text{(main bar)}$$

$$= 752\text{mm}$$

Provide 800mm length of bars at point where L_d is required, so that crack 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 Column, Long Column

Short Column: $\frac{L_{eff}}{D} \leq 12$

Long Column: $\frac{L_{eff}}{D} > 12$

Given: $L = 4m$

$P = 1600kN$

M20, Fe415

Step-I Effective Length (Tbl 28 Ann-E, IS456-2000)

$L_{eff} = 0.65L$

$= 0.65 \times 4m$

$= 2.6m$

$= 2600mm$

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, $A_g \rightarrow$ Gross area (C + R) Concrete + reinforcement

$A_{sc} \rightarrow$ Area of steel reinforcement (total)

$A_c \rightarrow$ Area of concrete

Clause 26.5.3.1 Pg 48, Assume 1% steel in gross area.

[0.8% - 6%]

1% of steel in Gross Area (A_g)
99% of concrete in Gross Area (A_g)

$A_{sc} = \frac{1}{100} \times A_g$

$A_c = \frac{99}{100} \times A_g$

Cl. 39.3 Pg 71.

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

$$\Rightarrow 2400\,kN = 0.4 \times 20 \times \frac{99}{100} A_g + 0.67 \times 415 \times \frac{A_g}{100}$$

$$\Rightarrow A_g = 224299\,mm^2$$

$b^2 = A_g$

$\Rightarrow b = \sqrt{224299} = 473.52\,mm \simeq 500\,mm$ Take higher value

b = 500mm

Size of column : 500mm X 500mm * think about as to column

Step-iv Slenderness Ratio Cl. 25.1.2 Pg 41

$$\frac{l_{eff}}{\underset{\text{Consider Square}}{D \text{ or } b}} = \frac{2600}{500}$$

$$= 5.2 < 12 \Rightarrow \text{Short Column}$$

Step-v Minm eccentricity for axial loading it arises

Cl. 25.4 $$e_{min} = \frac{l}{500} + \frac{D}{30}$$

$$= \frac{\overset{\text{unsupported length}}{3700}}{500} + \frac{500}{30}$$

$$\Rightarrow 24.07\,mm$$

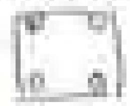
$\frac{e_{min}}{D} < 0.05 \Rightarrow$ For axially loaded short column

$= \frac{24.62}{500}$

$= 0.049 < 0.05$ Axially loaded short column

Its already given in question but we have to find it

Step-VI Area of steel (Longitudinal Reinforcement)



$A_{sc} = 1\%$ of A_g

$= \frac{1}{100} \times 224279$

$= 2243 \, mm^2$

For column min dia = 16mm

Let 20mm ϕ bars

No. of bars $= \frac{2243}{\left(\frac{\pi}{4} \times 20^2\right)}$

$= 7.14$

$= 8$ nos

$\therefore$ Provide 8 bars of 20mm ϕ

Step-VII Lateral Ties or Transverse Reinforcement

Cl 26.5.3.2 Part C (i) Dia of lateral ties $= \frac{\text{Dia of longitudinal bar}}{4}$

$= \frac{20}{4}$

$= 5mm$

$\nless 6$ mm

$\therefore$ Dia using 6mm ϕ ties

Pitch of the ties:

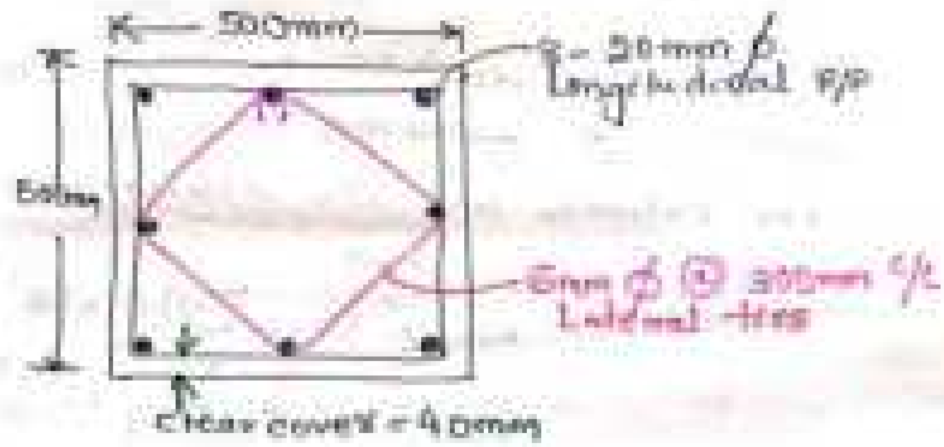
Cl 26.5.3.2 (c) of 456

pitch = least of {
(i) least lateral dimension of the member = 500mm
(ii) 16 × 20 = 320mm
(iii) 300mm
}

[illegible]

= 300mm

∴ Provide links of 6mm ϕ @ 300mm c/c



Q2. Design a column of size 450mm × 600mm having 3m unsupported length. The column is subjected to a load of 3000kN & is effectively held in position but not restrained against rotation. Use M20 & Fe415.

Step-1 Effective length L_{eff}

$L_{eff} = 1.0 \times L$

$= 1 \times 3m$

$= 3m$ or $3000mm$

Step - I Slenderness Ratio

$$\frac{l_{eff}}{D} = \frac{3000}{450} = 6.67 < 12$$

$\therefore$ Short Column

Step - II Minimum Eccentricity (e_{min})

e_{min} & limits one form $<$ b, D

For D = 600mm

$$e_{min} = \frac{L}{500} + \frac{D}{30} \quad Cl\ 25.4 \ Pg\ 42$$

$$= \frac{3000}{500} + \frac{600}{30}$$

$$= 26\ mm > 20mm$$

$$\frac{e_{min}}{D} = \frac{26}{600} \quad Cl\ 39.3 \ Pg\ 71$$

$$= 0.043 < 0.05 \ (O.K.)$$

$\Rightarrow$ Axially Loaded

For b = 450mm

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

$$= \frac{3000}{500} + \frac{450}{30}$$

$$= 21 > 20$$

$$\frac{e_{min}}{D} = \frac{20}{600} = 0.034 < 0.05 \ (O.K)$$

$\Rightarrow$ Axially Loaded

$\therefore$ Axially Loaded Member

Step-II Factored load

$P_u = P \times 1.5$

$= 2000 \times 1.5$

$= 3000$ kN

Step-3 Area of Longitudinal Reinforcement (A_{sc})

$A_g = 450 \times 600$

$= 270000 \text{ mm}^2$

$$\boxed{A_g = A_{sc} + A_c}$$

$\Rightarrow A_c = 270000 - A_{sc}$

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$ Cl. 39.3 Pg 71

$\Rightarrow 3000 \times 10^3 = 0.4 \times 20 \times (270000 - A_{sc})$
$+ 0.67 \times 415 \times A_{sc}$

$\Rightarrow A_{sc} = 3110.5 \text{ mm}^2$

% of reinforcement $= \frac{A_{sc}}{A_g} \times 100$

$= \frac{3110.5}{270000} \times 100$

$= 1.15\%$ Cl. 26.5.3.1 (a) Pg 48 [0.8% — 6%]

Lies b/w 0.8% — 6% (Hence ok)

$A_{sc} = 3110.5 \text{ mm}^2$

Prvd 25φ →

$4 - 25 \rightarrow 4 \times 490.6$
$= 1962 < 3110.5$ 25mm

$8 - 20 \rightarrow 8 \times 314 = 2512$

$8 - 25 \rightarrow 8 \times 490.6$
$= 3924.8 \text{ mm}^2$

Using 4-25mm φ = 4 × 490.6 = 1962.4 mm^2

4-20mm φ = 4 × 314 = 1256.0 mm^2 (+)

= 3218.4 > 2740.5 (O.K.)

∴ Provide: 4-25mm φ in longitudinal reinforcement
4-20 mm φ

Step-VI Lateral Ties (Transverse Reinforcement)
Cl 26.5.3.(1) Pg 49

Dia

$= \text{max of} \begin{cases} (i)\ \frac{1}{4} \times 25 = 6.25\text{mm} & \text{Least lateral dim} \\ (ii)\ 6\text{mm} & \end{cases}$

not available in market so use higher value 8.

Use 8mm φ Links

Pitch

$= \text{Min of} \begin{cases} (i)\ \text{Lateral dimension} = 450\text{mm} \\ (ii)\ 16 \times 20 = 320 \\ (iii)\ 300\text{mm} \end{cases}$

= 300 mm

∴ Provide 8mm φ Links @ 300mm c/c

Q1. 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 & Fe415.

Given $D = 400mm$, $P = 1200\,kN$, $L = 3m$, M25 Fe415

Step-I Effective length (l_{eff})

$$L_{eff} = 1.0 \times L \qquad \text{Table 28, Pg 94}$$

$$= 1 \times 3$$

$$= 3m \text{ or } 3000mm$$

Step-II Slenderness Ratio

$$\frac{L_{eff}}{d} = \frac{3000}{400} = 7.5 < 12 \Rightarrow \text{Short Column}$$

Cl 25.1.2 Pg 41

Step-III Minimum Eccentricity < Axial / Bi-axial / Uniaxial

$$e_{min} = \frac{L}{500} + \frac{D}{30} \qquad \text{Cl 25.4, Pg 42}$$

[illegible] $= \frac{3000}{500} + \frac{400}{30}$

$$= 19.33mm$$

$$\frac{e_{min}}{D} < 0.05 \text{ --- for axial loading}$$

$$= \frac{19.33}{400}$$

$$= 0.048 \quad (5 \ldots$$

:- Axially loaded Members

Step-III Factored load P_u =

$P_u = 1.5 P$

$= 1.5 \times 1200$

$= 1800 KN$

$= 1800 \times 10^3 N$

Step-IV Area of Reinforcement (A_{sc})

Cl 39.4, Eq^n

$P_{u_{helical}} = 1.05 P_u$ (Lateral ties) square/rectangular.

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

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

$= \frac{\pi}{4} \times 400^2$

$= 125600 \ mm^2$

$A_c = A_g - A_{sc}$

$= 125600 - A_{sc}$

$\Rightarrow 1800 \times 10^3 = 1.05 [0.4 \times 25 \times (125600 - A_{sc}) + 0.67 \times 415 \times A_{sc}]$

$\Rightarrow A_{sc} = 1793.35 \simeq 1800 mm^2$

$$\boxed{A_{sc} = 1800 mm^2}$$

Now checks the A_{sc} by % of steel.

% of steel = $\frac{1800}{125600} \times 100 = 1.4\%$

:- Cl 26.5.3.1 (a). Lies b/w 0.8% → 6%.
Pg - 48. → hence (ok)

No. of bars ?

Using 20 mm ϕ bars, 1 bar area $= \frac{\pi}{4} \times 20^2 = 314\, mm^2$

$$\text{No of bars} = \frac{\text{Total Area}}{\text{1 bar Area}}$$

$$= \frac{1850}{314}$$

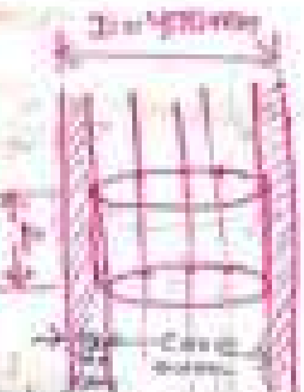
$$= 5.7$$

$\approx$ 6 nos (always take it in even no.

$\therefore$ Provide 6-20mm ϕ bars

$A_{st\ provided} = 6 \times 314 = 1884\, mm^2$

Step-VI 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)
$= 68766 mm^2$

Assuming pitch = p

Volume of core $= 68766\, p\ mm^3$

Using 8mm ϕ spirals;

Volume of 1 spiral $= \underbrace{\frac{\pi}{4} \times 8^2}_{\text{Area}} \times \underbrace{\pi (300 - 8)}_{\text{circumference}}$

volume of helical R/F $= 46064.05 mm^2$

$$\frac{\text{volume of helical R/F}}{\text{volume of the core}} \geq 0.36\left(\frac{A_g}{A_k} - 1\right)\frac{f_{ck}}{f_y}$$

(39.4.1 pg 31)

$$\frac{46009.85}{68766\,p} \geq 0.36\left(\frac{125660}{46064.05} - 1\right)\left(\frac{f_{ck}}{f_y}\right)$$

$$\Rightarrow \boxed{p \leq 38.82\text{mm}}$$

Check the pitch

Cl. 26.5.3.2 (d), Pg 49

↳ Limits for Pitch

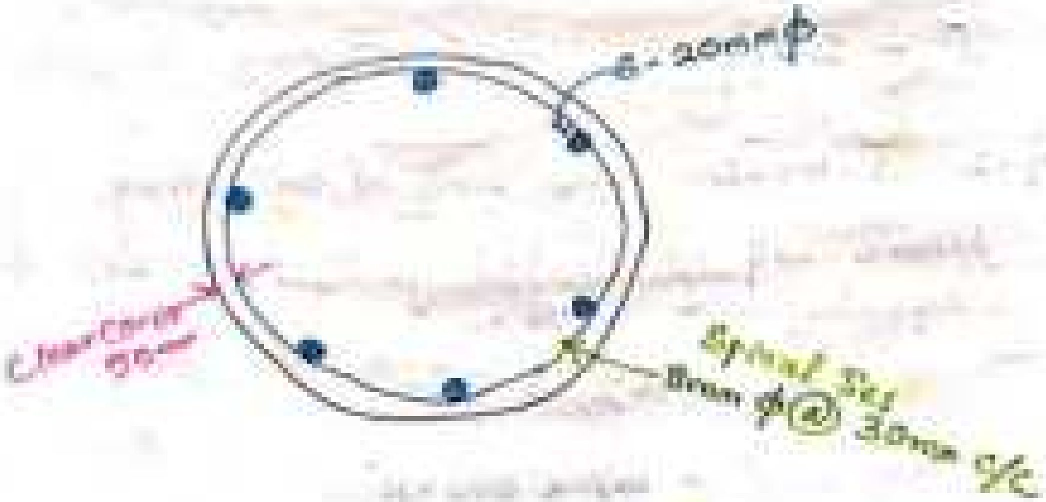
Max^m pitch = min of { (i) 75mm ; (ii) $\frac{\text{Core dia}}{6} = \frac{180}{6} = 30$mm }

Min^m pitch = max of { (i) 25mm ; (ii) 3 × Dia of Spiral $= 3 \times 8 = 24$mm }

∴ Provide 8mm ϕ Spirals @ 30mm c/c

Remember

$\boxed{\begin{array}{l} * \text{ Min}^m \text{ pitch} = 25\text{mm} \\ * \text{ Max}^m \text{ pitch} = 75\text{mm} \end{array}}$



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 2 way 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

$$A = \frac{W}{SBC} = \frac{\text{Total load}}{\text{Soil pr}}$$

$$= \frac{660}{120}$$

$$= 5.5\,m^2$$

Now for square footing

$$B = \sqrt{A}$$

$$= \sqrt{5.5}$$

$$= 2.345\,m$$

∴ Provide a square footing of size 2.4m x 2.4m

Step-II To find out net upward pressure (p_0)

$$p_0 = \frac{W}{B^2} = \frac{\text{total load}}{\text{area}}$$

$$= \frac{660\,KN}{2.4^2}$$

$$= 114.58\ KN/m^2$$

Step-III Depth of footing on the basis of B.M.

→ The maxm B.M acts at the face of the column & is given by

$$\boxed{M = p_0 \frac{B}{8} (B-b)^2}$$

Maxm BM occurs at face of the column either right or left. (Refer fig.1)

$$\Rightarrow M = 114.58 \times \frac{2.4}{8} (2.4 - 0.5)^2$$

$$= 124.1\ kN.m$$

Fig-1

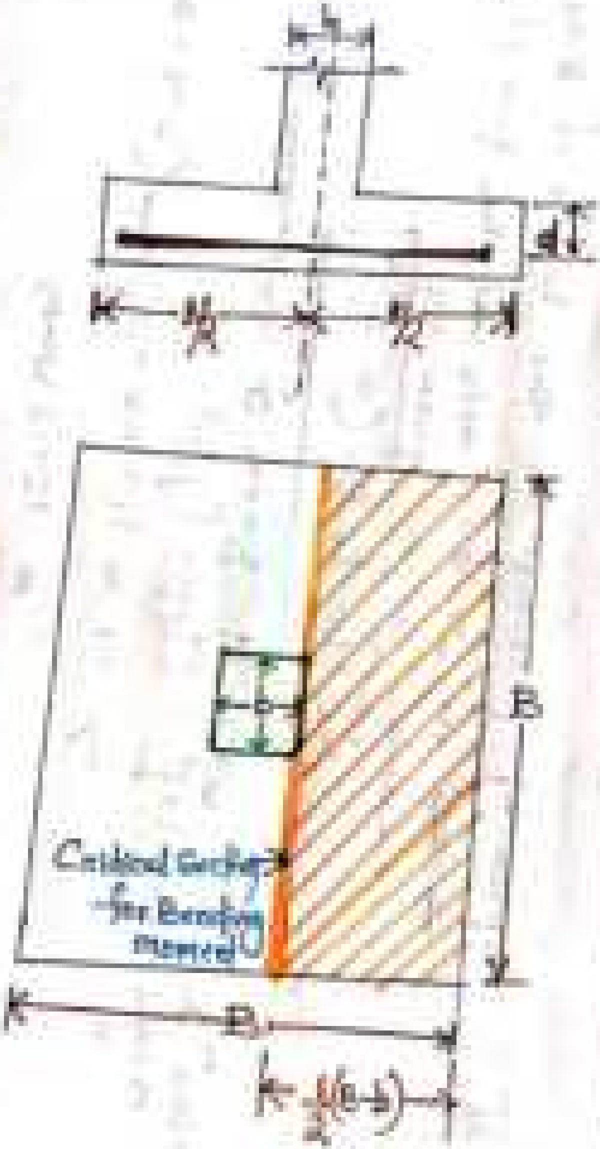


Fig-2

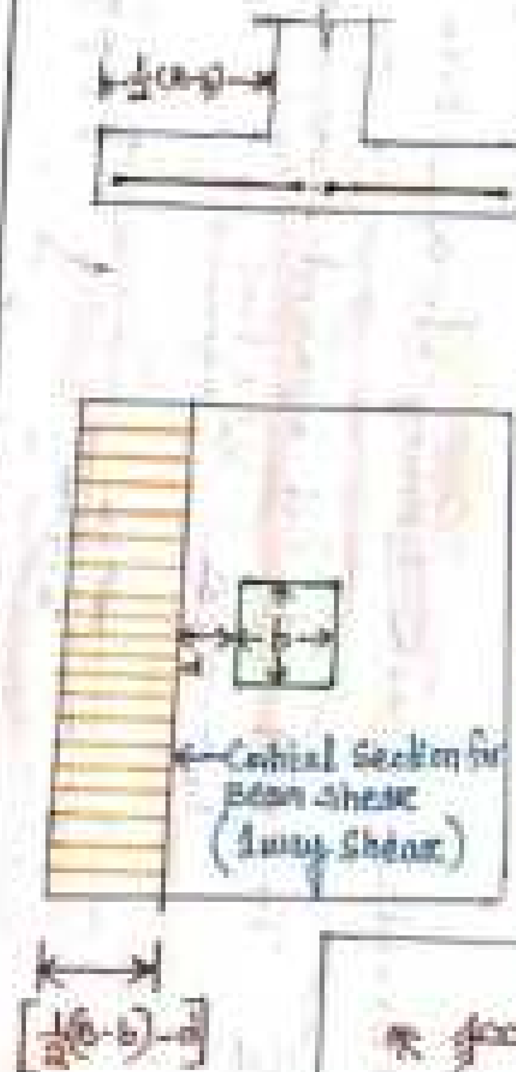
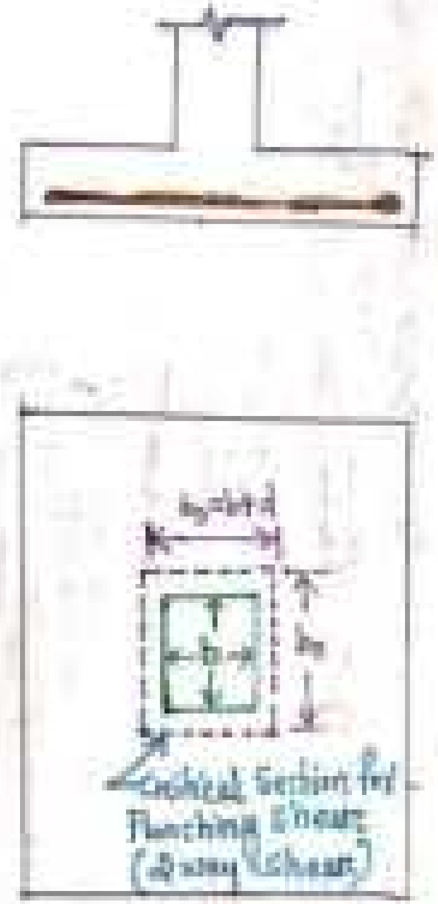
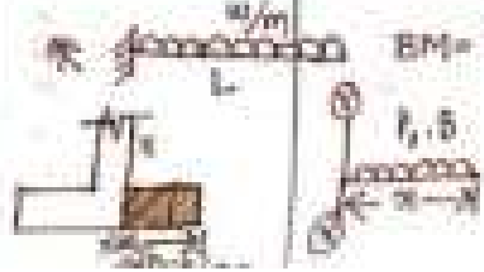


Fig-3





$BM = \frac{wL^2}{2}$

$B.M = \frac{1}{2}(p_0 B)x^2$

$M = \frac{1}{2} p_0 B \cdot \frac{1}{4}(B-b)^2$

Now ultimate Moment $M_u = 1.5 M$

$= 1.5 \times 124.1$

$= 186.15 \text{ kN-m}$

Dprov Pg 96

$M_{u_{lim}} = 0.138 f_{ck} b d^2 \leq M_{u_{max}} \left(\frac{x_u}{d} = 0.48 d\right)$

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

$d^2 = \frac{M_u}{0.138 f_{ck} B}$

$\Rightarrow d = \sqrt{\frac{M_u}{0.138 f_{ck} B}}$

$\Rightarrow d = \sqrt{\frac{186.15 \times 10^6}{0.138 \times 20 \times 2400}}$

$= 167.63 \text{mm}$

$\approx 170 \text{mm}$

Provide 50mm Cover (Min^m is 30mm)

$D = d + d'$

$= 170 + 50$

$= 220 \text{mm}$

NB Due to shear consideration Adopt higher effective depth.

$\therefore$ Adopt 400mm as eff depth (d) & provide 50mm eff cover

$\therefore d_{taken} = 400 \text{mm}$

$\therefore D = 400 + 50$

$= 450 \text{mm}$

Step (iv) Area of Reinforcement

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

$$= \frac{0.5\times 20}{415}\left[1-\sqrt{1-\frac{4.6\times 196.73\times 10^6}{20\times 2400\times 400^2}}\right] 2400\times 400$$

$$= 1327.7\ mm^2$$

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

$$= \frac{1327.7}{2400\times 400}\times 100$$

$$= 0.138\% > 0.12\%\ (O.K.)$$

∴ It satisfies min^m reinforcement criteria 0.12%

∴ O.K.

Using 12mm φ bars in both directions

$$a_{st} = \frac{\pi}{4}\times 12^2 = 113\ mm^2$$

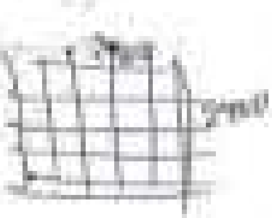
$$\text{No. of bars} = \frac{A_{st}}{a_{st}} = \frac{1327.7}{113} = 11.7 \approx 12$$

Provide 12 no. of 12mm φ bars uniformly in both directions

$$\text{Spacing} = \frac{B}{\text{No. of bars}}$$

$$= \frac{2400}{12}$$

$$= 200mm\ c/c$$



Step 8 Check for 1-way shear or Beam shear

Max^m S.F. lies at a distance 'd' from face of column.

For 1-way shear, critical sec^n is located at a distance 'd' from the face of the column

Now shear force (V)

$V = P_o B \left\{ \frac{1}{2}(B - b) - d \right\}$

$= 114.58 \times 2.4 \times \frac{1}{2}\{(2.4 - 0.5) - 0.4\}$

$= 151.24 \, kN$

Ultimate shear force V_u

$V_u = 1.5V$

$= 1.5 \times 151.24$

$= 226.86 \, kN$

Nominal Shear Stress τ_v (Pg. 72)

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

$= \frac{226.86 \times 10^3}{2400 \times 400}$

$= 0.236 \, N/mm^2$

For solid slabs (footing 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 0.128% steel & M_{20} concrete

$\Rightarrow \tau_c' = 0.28 \tau_c = 0.28 N/mm^2$ (Table, Pg 73)

Now $\tau_v < \tau_c'$ ∴ (OK)

$$\frac{V_u}{Bd} = k_1 \tau_c$$

$$\Rightarrow d = \frac{V_u}{B k \tau_c}$$

$$= \frac{226.88 \times 10^3}{2400 \times 1 \times 0.28}$$

$$= 337\,mm > d = 170$$ [on basis of ... is not considered hence we increase effective depth

∴ $d_{provided} = 450\,mm > 337\,mm$ ∴ ok for 1-way shear

Step - VI Check for 2-way shear or Punching shear

→ Lies $\frac{d}{2}$ from all the faces of column
Column punches on footing as shear.

or,

For 2 way shear the section lies @ a distance $\frac{d}{2}$ from the column face all around.

∴ The width = $b_0 = b + d$

$$b_0 = 450 + 450$$

$$= 900\,mm$$

Now, the net SF acting on the perimeter

$$F = P_0 \left[B^2 - b_0^2 \right]$$

$$= P_0 \left[B^2 - (b + d)^2 \right]$$

$$= 119.58 \left[2.4^2 - 0.9^2 \right]$$

$$= 591.92\ kN$$

Ultimate shear force $F_u = 1.5F$

$= 1.5 \times 561.171$ kN

$= 841.75$ kN

Pg 57 IS 456:2000 + Pg 25

$\tau_v = \frac{F_u}{4 b_0 d}$

$= \frac{841.75 \times 10^3}{4 \times 900 \times 400}$

$= 0.58$ N/mm^2

Now, permissible shear stress (Pg 58) $k_s \tau_c$

$k_s = 0.5 + \beta_c$

$= 0.5 + 1$ for square column $\beta_c = 1$

$= 1.5 > 1$

$k_s = 1$ [max]

Take $k_s = 1$

$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.11$ N/mm^2

$\therefore k_s \tau_c = 1 \times 1.11 = 1.11$ N/mm^2

$\tau_v < k_s \tau_c$ $\therefore$ Safe in two way shear