S. No.	Date	Title	Page No.	Teacher's Sign / Remarks

Sumit Paudel

Chapter 1, Introduction

DATE

Outline:

→ Limitation of plain concrete
→ Properties of reinforcement & concrete
→ Analysis of forces & stresses in reinforced concrete structures.

※ Limitation of plain Concrete:

→ Plain cement concrete is a hardened mass obtained from a mixture of cement, sand, gravel & water in definite proportions.
→ The major limitations of the plain concrete are
(a) Concrete is quasi-brittle material
✓(b) Low tensile strength
(c) Concrete has low toughness & specific strength
(d) Formwork is required & long curing time
✓(e) Working with cracks
(f) Need strict quality control
✓(g) Concrete is less-ductile
(h) Weight of concrete is high compared to its strength

* Reinforced Cement Concrete:-

→ plain cement concrete has very low tensile strength. To improve tensile strength of concrete, some sort of reinforcement is needed which can take up the tensile stresses developed in the structure.
→ Thus, reinforced cement concrete is a composite material which is made up of concrete & steel reinforcement.
→ The composite action of steel & concrete depend on
(i) Bond betn steel & Concrete
(ii) prevention of corrosion of steel bar embedded in concrete
(iii) Practically equal thermal expansion of concrete & steel

※ Advantages of RCC over other construction material

→ Good strength in tension as well as compression
→ Durable if designed & laid properly. (Last upto 100 years)
→ RCC sections can be given any shape easily by properly designing the formwork. Thus, it is more suitable for architectural requirements.
→ The steel reinforcement imparts ductility to RCC structures
→ RCC is cheaper as compared to steel & prestressed concrete.
→ Raw materials for RCC are easily available & can be transported easily.
→ More fire resistance
→ Almost impermeable to moisture.
→ ~~Permeability~~ Properly designed RCC structures are extremely resistant to earthquakes.

※ Disadvantages of RCC structures

→ RCC structures are heavier than other structures.
→ RCC needs lot of formwork, centering & shuttering to be fixed, thus require lot of site space & skilled labour
→ Concrete takes time to attain its full strength. Thus, RCC structure can't be used immediately after construction unlike steel structures.
→ They require high initial cost for construction.
→ All the constituents of RCC are not locally available everywhere.
→ At the end of useful life, RC structures need to be dismantled.

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* Properties of Concrete:

The properties of concrete depends upon the proportions & type of its ingredients. A properly designed concrete mix is very durable. A good concrete mix should satisfy following requirements.

→ The concrete should be mixed throughly to form homogeneous mix.
→ Should be compacted properly.
→ Sufficient curing for developing full strength
→ W/C ratio should be appropriate considering strength & workability criteria
→ Concrete mix should be designed properly & should have all ingradients in right proportions.
→ Water used should be free from all harmful organic substances
→ Aggregate should be hard, durable & properly graded. For most RCC works, 20mm size of aggregate is suitable.
→ Cement used for RCC work should be of good quality & measure by weight only not by volume.

* For RCC works - not lower than M20
For post tensioning - M35 & above
For pretensioned prestressed - M40 & above.

→ The major properties of concrete are

i) Compressive strength (f_{ck})	vi) Poisson's Ratio (0.1-0.3)
ii) Workability	vii) Creep:
iii) Durability	viii) Shrinkage: 0.003mm/m
iv) Tensile strength, $f_{tc} = 0.7\sqrt{f_{ck}}$	ix) Freezing & thawing
v) Modulus of Elasticity ($E_c = 5000\sqrt{f_{ck}}$)	x) Sulphate Attack

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** Properties of Reinforcement:-

→ The reinforcing material should satisfy following requirements
1) Should develop a perfect bond with concrete to transfer stresses from one material to other.
2) Should have high tensile strength
3) Should be cheap, easily available & durable.
4) α reinforcing material $\approx \alpha$ concrete for good composite action
5) Should be workable, easy to cut, bend & join
6) Should not react with other ingradients of RCC.
7) Should be free from loose mill scales, loose rust etc.

→ Due to higher strength, better bond, economy, HYSD bars are preferred as reinforcement in R.C.C. over plain mild steel bars.

→

Types of steel	Grade	Yield stress or 0.2% proof stress or characteristic strength in N/mm²	% Elongation
Mild Steel	Fe250	250	23%
High Strength Deformed steel (HYSD)	Fe415	415	14.5%
	Fe500	500	12%
	Fe550	550	8%
TMT bar	Fe500	500	12%

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Chapter 2: Design Methods

Sumit Paudel

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Outline:-

→ Working stress method for design of RCC structures
→ Ultimate load method for ~~beam & slabs~~ design of RCC str.
→ Limit state method for design of RCC structures
→ Types of limit state methods
→ Characteristic loads & strength of materials
→ partial safety factors & their considerations in structural design.

✻ Working Stress Method:-

→ Oldest method, based on elastic theory & assumes that both steel & concrete are elastic & obey Hook's law.
→ In this method, the ultimate strength of concrete & yield strength or 0.2% proof stress of steel are divided by FOS to obtain permissible stresses.
→ FOS for concrete 3 & 1.78 for steel.
→ The working stress method is very simple & reliable but as per IS 456:2000, WSM is used only if it is not possible to use LSM of design.
→ The main drawbacks of WSM are
i) It assumes that concrete is elastic but concrete behaves inelastically even on low level of stresses.
ii) It uses FOS for stresses only not for loads. Hence, this method doesnot give true margin of safety wrt loads. It means, there is no provision for the uncertainties associated with estimation of loads.
iii) It doesnot account for shrinkage & creep which are time dependent & plastic in nature.
iv) This method gives uneconomical sections.

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v) It pays no attention to the conditions that arise at the time of collapse.

✻ Ultimate load Method:-

→ In this method, ultimate or collapse load is used as design load. The ultimate loads are obtained by multiplying working loads with load factors.
→ This method uses the real stress-strain curve of concrete & steel & takes into account the plastic behaviour of these materials.

$$\text{Load factor} = \frac{\text{collapse load}}{\text{Working load}}$$

→ The advantages of this method are:-
i) Realistic, takes into the account non-linear behaviour of concrete.
ii) Gives exact margin of safety in terms of loads.
iii) Gives thinner section, economic design as compared to WSM

→ The main limitations of the ultimate load method are,
(i) This method gives very thin sections which leads to excessive deformation & cracking, thus making the structure unserviceable.
(ii) No factors of safety are used for material stresses.

✻ Limit state Method of Design:-

→ Rational method
→ Combination of WSM & ULM.
→ This method is based on concept ~~that~~ of safety at

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ultimate loads & serviceability at working loads.

→ The acceptable limits of safety & serviceability requirements before failure occurs is called a limit state.

→ The two important limit states to be considered in design are.

(i) Limit state of collapse (Flexure, shear & bond, Torsion, Compression)

(ii) Limit state of Serviceability (deflection, Cracking, Vibration)

→ This method is based upon the probabilistic variation in the loads & material properties.

→ This method takes into account the uncertainties associated with load & material properties, thus uses partial factor of safety to obtain design loads & design stresses.

※ Types of Limit state:-

(a) Limit state of collapse:-

→ It corresponds to the maximum load carrying capacity.

→ Limit state of collapse is assessed from collapse of the whole or part of the structure.

→ As per this limit state, the resistance to bending, shear, torsion & axial loads at every section shall not be less than that produced by the most unfavourable condition of loads on the structure.

→ The following limit of collapses are considered in design

(i) Limit state of collapse in flexure (bending)

(ii) Limit state of collapse in compression

(iii) Limit state of collapse in shear

(iv) Limit state of collapse in torsion

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(b) Limit state of Serviceability:

→ A structure is of no use if is is not serviceable. Thus, this limit state is introduce to prevent excessive deflection & cracking

→ It ensures the satisfactory performance of structure at working loads.

→ It is estimated on the basis of elastic theory or working stress method because deformation is of significance under working load & not at collapse.

→ Limit of serviceability is of following limit states

(i) Limit state of deflection

(ii) Limit state of cracking

(iii) Limit state of vibration

※ Characteristic load

→ The characteristic load is the ultimate load that is liable to come on structure during its lifetime.

→ For simplicity, the variation in the loading is also assumed to follow a normal distribution curve, the characteristic load is given as,

$$\text{characteristic load} = \text{Mean load} + 1.645$$

$$W_u = W_m + 1.645$$

→ characteristic load is that value of load which has a 95% probability of not being exceeded during the lifetime of the structure.

→ In absence of any data, loads given in various standards shall be assumed as characteristic loads. The following standards may be used for this purpose.

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IS 875 (part 1) → for dead loads
IS 875 (part 2) → for ~~wind~~ Live loads
IS 875 (part 3) → for wind loads
IS 875 (part 4) → for snow loads
IS 1983 (part 1) → for earthquake loads

✱ Design Strength of Materials:-

→ The strength of any material obtained in structure is always less than the characteristic strength of material. It is because of workmanship & quality control in the manufacture of materials.

→ The reduced value of strength which is obtained by applying partial safety factors to the characteristic strength is called design strength of materials.

$$f_d = \frac{f}{\gamma_m}$$, f = characteristic strength of material
γ_m = partial safety factors for material.

✱ Partial Safety Factors for materials (γ_m)

→ partial safety factor is the ratio of characteristic strength of material to design strength of material.

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

$\gamma_{mc} = 1.5$ (partial safety factor for concrete)
$\gamma_{ms} = 1.15$ (partial safty factor for steel)

→ Higher value of psf for concrete has been adopted because there are greater chances of variation in strength of concrete due to improper compaction, inadequate curing & variation in the properties of ingredients.

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→ The chances of variation in the properties of steel are small, as it is fabricated in the factories where good workmanship & better quality control can be achieved. Hence, a lower value of 1.15 has been adopted for steel.

✱ Partial Safety factors for loads (γ_f)

→ The ratio of design load (f_d) to the characteristic load (F) represents the ~~load~~ partial safety factors for loads.

$$\gamma_f = \frac{F_d}{F} \Rightarrow f_d = \gamma_f \times F$$

→ Design loads are used for calculating design bending moment, shear force, thrust etc.

value of γ_f for loads.

load comb^n	Limit state of collapse			Limit state of Serviceability		
	DL	LL	WL	DL	LL	WL
DL+LL	1.5	1.5	–	1	1	–
DL+WL	1.5 or 0.9*	–	1.5	1	–	1
DL+LL+WL	1.2	1.2	1.2	1	0.8	0.8

While considering earthquake effects substitute EL for WL

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Chapter-8
Limit State Design for Beams & Slabs:-

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Outline:-

→ General design consideration
→ Analysis of singly reinforced & doubly reinforced rectangular beam section
→ Analysis of singly reinforced flanged beam section.
→ Strength & serviceability requirements for flexural members.
→ Design & detailing of simply supported & continuous beam (for flexure, shear & torsion)
→ Design & detailing of slabs spanning in one or both directions

✻ Introduction:-

→ A reinforced concrete structure has several members in the form of beams, columns, slabs & walls that are rigidly connected to form a monolithic frame. Each individual member must be capable of resisting the forces acting on it.

→ Beams are members that are primarily subjected to flexure or bending & often support slabs. Beams supports the loads applied on them by slabs & their own weight by internal moments & shears.

→ In an RC beam of rectangular ~~parts~~ cross section, if the reinforcement is provided only in tension zone, it is called singly reinforced rectangular beams whereas if the reinforcements are provided in both compression & tension zones, it is called doubly reinforced rectangular beams.
→ These beams can be designed by taking into account the contribution of slab in resisting the compression at mid span. These beams are called T-beams or L-beams.

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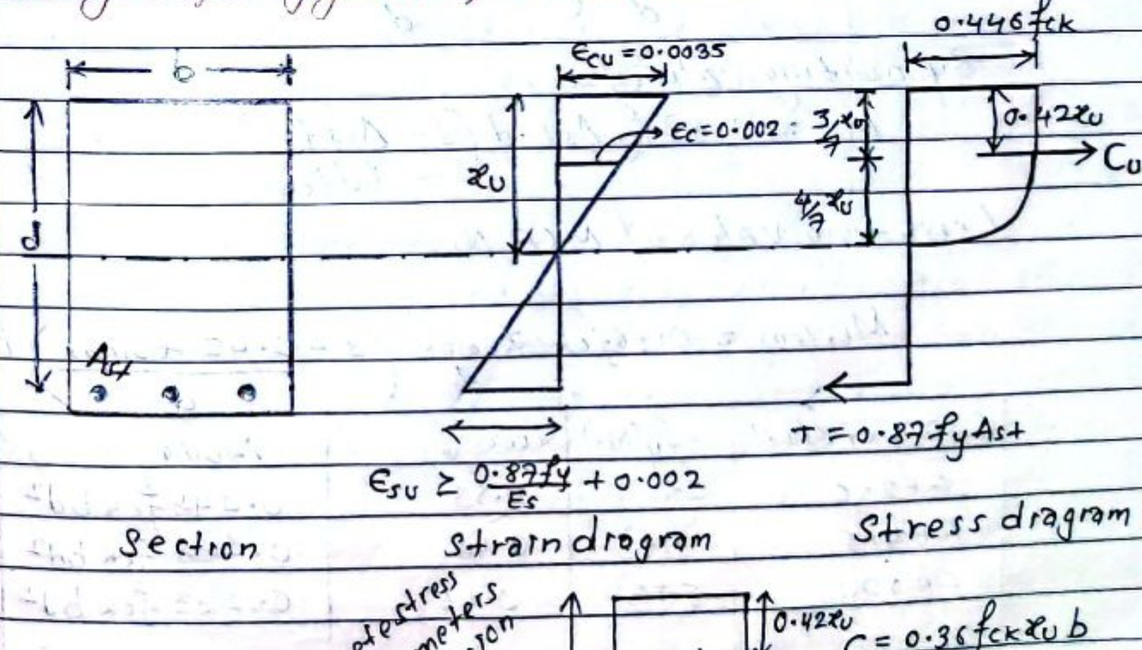
(38.1)

✻ Assumption in Limit state of Collapse in Flexure:-

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→ plane sections normal to the axis remain plane after bending.
→ The max^m strain in concrete at the outermost compression fibre is taken as 0.0035 in bending.
→ The relationship bet^n compressive stress dist^b in concrete is assumed to be parabolic.
Max^m compressive stress in concrete $= \frac{0.67 f_{ck}}{1.5} = 0.446 f_{ck}$
→ The tensile strength of concrete is ignored.
→ The stresses in the reinforcement are taken from the stress strain curve for the type of steel used. For design, $\gamma_{ms} = 1.15$
→ The max^m strain in the tension reinforcement in the section at failure shall not be less than, $\frac{f_y}{1.15 E_s} + 0.002$

✻ Analysis of Singly Reinforced beam:-



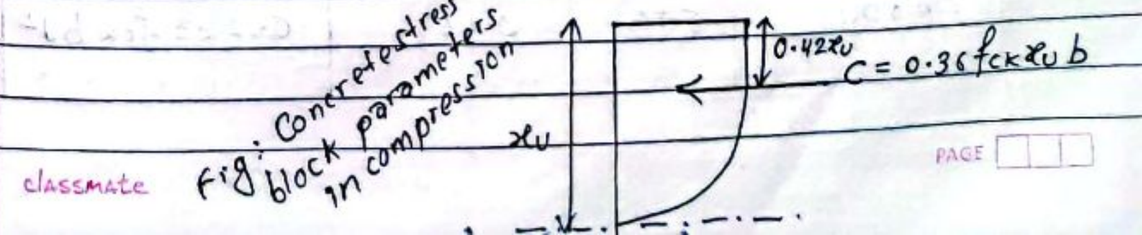


Fig: Concrete stress block parameters in compression

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* Neutral Axis Depth (x_u)

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

Limiting depth of neutral axis,

$$\frac{x_{umax}}{d} = \frac{0.87 f_y \; 0.0035}{\frac{0.87 f_y}{E_s} + 0.0055}$$

Grade of steel	f_y (MPa)	Maxm NA depth (x_{umax})
Mild steel (Fe250)	250	0.53d
Fe415	415	0.48d
Fe500	500	0.46d

* Moment of Resistance:-

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

By putting value of x_u,

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

Limiting value of MOR is,

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

Grade of steel	f_y (MPa)	x_{umax}/d	M_{ulim}
Fe250	250	0.53	$0.148 f_{ck} bd^2$
Fe415	415	0.48	$0.138 f_{ck} bd^2$
Fe500	500	0.46	$0.133 f_{ck} bd^2$

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* Percentage of steel:- (p_t)

$$p_t = \frac{0.36 f_{ck}}{0.87 f_y} \times \frac{x_u}{d}$$

$$p_{t,lim} = \frac{0.36 f_{ck}}{0.87 f_y} \times \frac{x_{u,max}}{d} \times 100\%$$

Grade of steel	f_y (MPa)	$p_{t,lim}$ (%)
Fe250	250	$0.0877 f_{ck}$
Fe415	415	$0.0478 f_{ck}$
Fe500	500	$0.088 f_{ck}$

* Balanced, Under-Reinforced and Over-reinforced Sections.

(a) Balanced section:-

In balanced section, the steel reinforcement reaches the yield strain $\frac{0.87 f_y}{E_s} + 0.002$, at the same time as the concrete reaches the ultimate strain value of 0.0035.

- $x_u/d = x_{u,max}/d$ $\quad M_{ulim} = 0.87 f_y A_{st} d \left(1 - 0.42 \frac{x_{umax}}{d}\right)$
- $p_t = p_{t,lim}$
- $M_u = M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d} \left(1 - 0.42 \frac{x_{umax}}{d}\right) bd^2$

(b) Under-Reinforced Section:-

→ In this section, steel fails first by reaching its yield strain value, although in concrete the ultimate strain has not reached.
→ steel is ductile material & it gives sufficient warning before failure, hence the under reinforced section are preferred by designers.

- $x_u/d < x_{umax}/d$
- $p < p_{lim}$
- $M_u = 0.87 f_y A_{st} d \left(1 - 0.42 \frac{x_u}{d}\right)$

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

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(c) Over-Reinforced Section

→ Strain in concrete reaches its ultimate value earlier than steel.

→ Over-reinforced beam fails by crushing failure of concrete. Concrete being brittle, fails suddenly without warning.

- $x_u/d > x_{umax}/d$
- $P > P_{lim}$
- $M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d} \left(1 - 0.42 \frac{x_{umax}}{d}\right) bd^2$

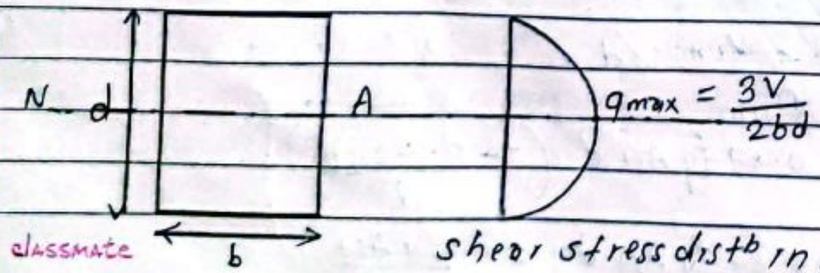
→ 25456:2000 recommends that over-reinforced section should be redesigned.

Limit state of collapse: shear

→ A beam loaded with transverse load is subjected to shear force & bending moment. The shear force results into the shear stresses across the cross-section.

$$q = \frac{V}{Ib} \times A'\bar{y}$$

q = shear stress
V = shear force at that section
I = MOI of beam section
b = width of section
$A'\bar{y}$ = First moment of area above the section about NA



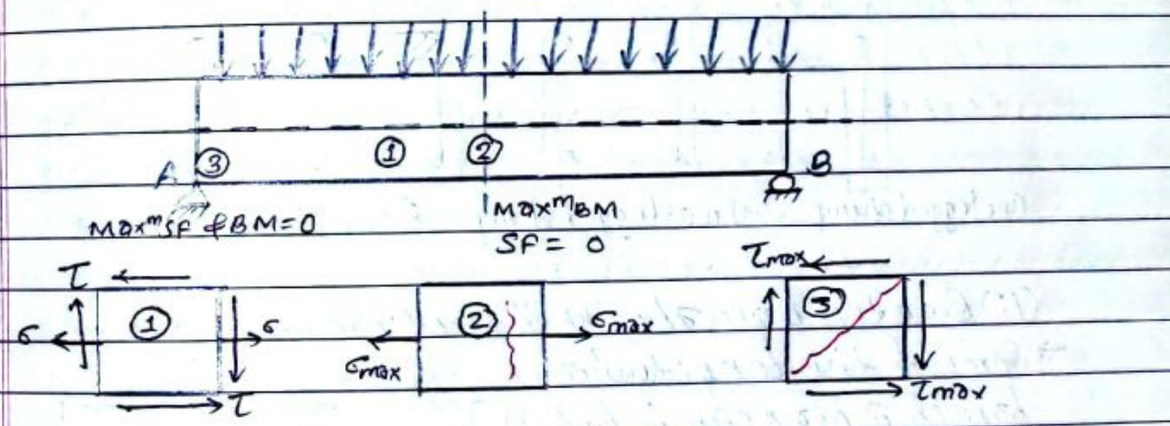
Shear stress distb in rect. section

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

V_u → shear force due to design loads
b → width of member
d → Effective depth

* Effect of shear :- diagonal tension



σ = bending stress
τ = shear stress
σ_{max} = Max^m bending stress
τ_{max} = Max^m shear stress

→ At or near the midspan, the cracks will be vertical (flexural cracks due to bending alone)

→ At or near the supports the cracks are inclined at 45° (shear or diagonal tension crack)

→ In betn the supports & midspan, the cracks inclination vary from 45° to 90° gradually. (flexural-shear cracks)

→ If point load is acting on beam, just below the point load, cracks are inclined at 45° from top of beam. This crack is diagonal compression crack.

→ So, shear reinforcement is provided in RCC beams to take up diagonal tension & prevent cracking of beam.

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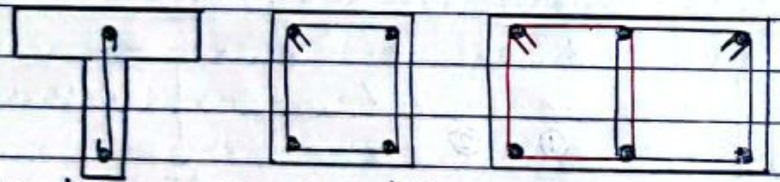
* Types of shear reinforcement:-

(i) vertical stirrups:-

→ These are the steel bars vertically placed around the tensile reinforcement at suitable spacing along the length of beam.

→ Their dia. varies from 6mm to 16mm

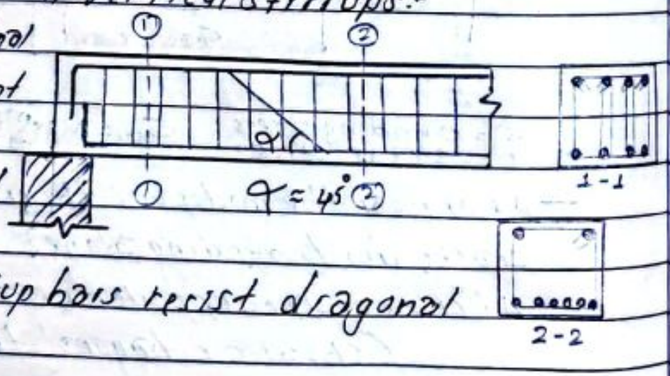
→ May be one legged, two legged, four legged etc.



One legged stirrup Two-legged stirrup Four-legged stirrup

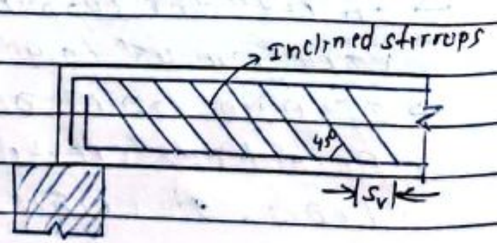
(ii) Bentup bars along with vertical stirrups:-

→ Some of the longitudinal bars in a beam can be bent up near the supports where they are not required to resist the bending moment. These bentup bars resist dragonal tension.



(iii) Inclined stirrups.

→ Inclined stirrups are also provided generally at 45° for resisting dragonal tension. They are provided throughout the length of beam.



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* Shear strength of reinforced concrete beam

→ It depends upon

(i) Grade of concrete:- Higher grade, higher shear strength

(ii) % of tensile reinforcement:- ↑ with ↑ in Ast

→ Depending upon grade of concrete & % of tensile steel, the design shear strength ~~of~~ in concrete (τ_c) in beams without shear reinforcement is given in Table 19 of IS456:2000

→ For solid slabs, Shear strength = $k * \tau_c$, 'k' is given in 40.2.1.1

→ Maxm shear ~~stress~~ stress (τ_{cmax}) (Table 20).

- If the shear strength of concrete beam is less than the nominal shear stress (τ_v) due to the loads coming on the beam, then shear reinforcement is to be provided.
- The nominal shear stress in the beam with shear reinforcement shall not exceed maximum shear stress (τ_{cmax}).
- If $\tau_{cmax} < \tau_v$, the section is to be redesigned.

* Minimum Shear Reinforcement: (26.5.1.6)

When the nominal shear stress (τ_v) is less than the design shear strength (τ_c) or shear strength of concrete, then no shear reinforcement is to be designed. But in such cases minimum shear reinforcement is to be provided in the form of stirrups such as,

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

Where,

A_{sv} = Total x-sectional area of stirrup legs effective in shear

S_v = stirrups spacing along length of member

b = width of beam

f_y = characteristic comp. strength of rebar (N/mm^2)

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* Maximum Spacing of stirrups (26.5.1.5)

→ The max^m spacing of vertical stirrups shall not exceed 0.75d or 300mm whichever is less.

→ In inclined stirrups at 45°, the max^m spacing should not be greater than 'd' or 300mm.

* Design of shear reinforcement: (40.4)

→ When $\tau_v > \tau_c$, shear reinforcement is to be designed.

→ Shear reinforcement is provided to carry a shear force equal to V_{us}.

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

V_u = shear force due to design loads

$\tau_c bd$ = shear resistance of concrete section

→ The spacing of vertical stirrups is given,

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

where,

f_y = characteristic strength of stirrups ≤ 415 MPa

A_{sv} = Total x-sectional area of stirrups legs.

S_v = spacing of stirrups

d = effective depth

→ For inclined stirrups,

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

where, A_{sv} = Area of bentup bars

α = Angle bet^n the bentup bar & member axis ($\alpha \geq 45°$)

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* Bond & Development length:-

→ The term bond refers to the adhesion between concrete & steel which resist the slipping of steel bar from concrete. It is responsible for transfer stresses from steel to concrete. The bonds develops due to setting of concrete on drying which results in gripping of steel bars.

→ The bond bet^n steel & concrete can be increased by following methods.

- Using deformed or twisted bar
- Using rich mix of concrete
- Adqvate compaction & curing of concrete for proper setting
- Providing hooks at the end of reinforcing bars.

→ There are two type of bond failure
  - ↳ Anchorage bond failure
  - ↳ Flexural bond failure

* Anchorage Bond (Development Bond)

$$L_d \geq \frac{0.87 f_y \phi}{4 \tau_{bd}}$$

where, L_d = Embedded length of steel / development length

f_y = characteristic strength of bar

ϕ = drameter of bar

τ_{bd} = Design bond stress in limit state method.

→ This embedded length of steel is called anchorage length in case of axial tension & axial compression & development length in case of flexural tension or flexural compression.

→ As per IS456:2000,

Development length (L_d) = $\frac{\phi \sigma_s}{4 \tau_{bd}}$ [26.2.1]

τ_{bd} → given in table (26.2.1.1) for plain bars in tension

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→ For deformed bar, τ_{bd} is 60% more than that of plain bars.

→ It is easier to pull a bar than to push it inside. Therefore permissible bond stress for plain & deformed bars in compression is taken as 25% more than that for the bars in tension.

Development length in compression = $\frac{0.87 f_y \phi}{4(1.25 \tau_{bd})}$

→ Development length for single bar

Type of steel bar	f_y (N/mm²)	L_d in tension			L_d in compression		
		M15	M20	M25	M15	M20	M25
plain bars	250	55ϕ	46ϕ	39ϕ	44ϕ	37ϕ	31ϕ
	415	56ϕ	47ϕ	40ϕ	45ϕ	38ϕ	32ϕ
Deformed bars	500	69ϕ	58ϕ	49ϕ	54ϕ	46ϕ	39ϕ

→ In case of bundled bars in contact, increase L_d by
(i) 10% for two bars in contact
(ii) 20% for three bars in contact
(iii) 33% for four bars in contact.

| calculate L_d for individual bar & ↑

* Flexural Bond:-

→ In a simple beam, at the critical sections i.e at the face of the support, at points of inflection & at points of high shear force, high bond stress may develop ~~needs~~ due to large variation in bending moment. These bond stresses & should be checked carefully at all critical sections.

$$\tau_{bd} = \frac{V}{\pi \phi a}$$

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Where, V = shear force at that section
ϕ = dia of bar
a = lever arm

If there are bars of different sizes, then the maximum flexural bond stress develops at the bar of large diameter.

→ As per IS 456:2000 (26.2.3.3)

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

M_1 = Moment of resistance at the section under consideration.
V = Factored shear force at that section
l_o = Sum of anchorage beyond the centre of support & equivalent anchorage value of any hook or any mechanical anchorage at support.

* Anchorage for reinforcement bars:-

(a) Anchoring bars in tension

→ Hooks are provided normally in plain bars in tension.
→ Deformed bars may be used without end anchorage if development length requirement is satisfied.
→ The hooks should always confirm to the specifications.
→ For U type hook ⇒ Anchorage value = 16ϕ
For 90° bend ⇒ Anchorage value = 8ϕ

(b) Anchoring bars in compression:-

→ The anchorage length of bars in compression is equal to the development length of bars in compression. The compression bars require no special anchorage arrangements.

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(c) Anchoring shear Reinforcement:

(a) stirrups:

→ The stirrup bar is bent through an angle of atleast 90° around a bar atleast its own diameter and is continued beyond the curve for a length of at least eight times the bar diameter. When the bar is bent through an angle of 135° & is continued beyond the curve for a length of at least six times bar diameter.

→ When the bar is bent through an angle of 180° & is continued beyond the curve for a length of atleast four times the bar diameter.

(b) Bent up bars:-

→ The development length of such bars is according to the development length of bars in tension allowing for hooks & bends if provided & measured as following

(i) In tension zone - from the end of slaping or inclined portion of bars

(ii) In compression zone - from the mid-depth of beam.

* Splicing:-

→ When the available length of bar is inadequate or has to be extended then splicing of reinforcement is done.

→ The splicing of bar means joining two reinforcement bars to transfer the forces effectively from the terminating bar to other bar.

→ Concrete at the point of splicing is subjected to high stresses which may cause cracking of concrete.

→ Splicing of reinforcement bars is done by following methods

i) Lapping of bar (most common)

ii) Mechanical joint

iii) Welding the bars

→ As per IS 456:2000 (26.2.5)

- The splicing of flexural members should not be done at sections having bending moment 50% of moment of resistance.
- Not more than 50% of bars be spliced at any given section
- The splicing of bars should be carried out for alternate bars if more than one bar is to be spliced.

* Lap splicing:-

The length of lap depends on

i) concrete strength

ii) Grade of steel bar

iii) Size of bar

iv) Spacing of bar

→ As per IS 456:2000,

1) Laps in reinforcing bars should always be staggered the centre to centre distance of laps should not be less than 1.3 times the required lap length. The bars to be lapped should be arranged either vertically one over the other or horizontally.

2) Lap length in tension $\begin{cases} 30\phi \\ 2L_d \end{cases}$ whichever is greater

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3) Lap length in flexural tension (including bends & hooks)
30ϕ } greater
L_d }

4) Lap length in compression
24ϕ } greater
L_d in compression }

5) When bars of two different diameter are to be spliced, the lap length is based on the basis of diameter of smaller bar.

6) Lap splicing of bars more than 36mm in diameter is to be avoided. If needed to be spliced, these bars should be welded. If welding is not practicable then additional spirals should be provided.

7) When bundled bars are to be spliced by lapping, one bar is to be spliced at a time & splicing is staggered.

✱ Design of Singly Reinforced Beam:-

→ In singly reinforced beam, steel reinforcement is provided only in tension zone.

→ While designing RCC beams, following important rules should be kept in mind.

(a) Effective Span:-

→ For simply supported beam,
clear distance + effective depth of beam
centre to centre distance betn supports } less

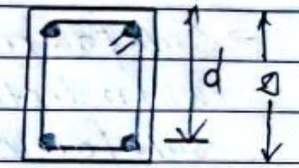
→ For continuous & cantilever beam, follow clause 22.2 of IS 456:2000

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(b) Effective depth (d)

→ Effective depth of a beam is the distance between the centroid of area of tension reinforcement & topmost compression fibre.

$d = D - \text{effective cover}$
$= D - (\text{Clear cover} + \text{dia of stirrup} + \frac{\text{dia of main bar}}{2})$



(c) Control of Deflection:-

→ For span upto 10m,

(i) For simply supported beam, $\frac{\text{Span}}{\text{Effective depth}} < 20$

(ii) For cantilever beam, $L/d < 7$

(iii) For continuous beam, $L/d < 26$

→ For span > 10m, values given above should be multiplied by 10/span, except for cantilever for which is to be calculated the exact deflection.

→ Depending upon area & stress of steel for tension reinforcement, the values above shall be modified by multiplying with the modification factor obtained from figure 4.

$$f_s = 0.58 * f_y * \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

(d) Reinforcement:-

→ Minm Reinforcement:- [26.5.1.1 a)]

$$A_s/bd = 0.85/f_y$$

$A_s \rightarrow$ Minm area of tension steel

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→ Maxm Reinforcement :- [26.5.1.1.b]
The maxm area of tension reinforcement shall not exceed 0.04bD.

→ Side face reinforcement
When depth of web in beam > 750mm, deep beam. So, side face reinforcement should be provided along two faces. For detail, follow clause 26.5.1.3

→ Spacing of Reinforcement bar.
(i) The horizontal distance betn two parallel main bar shall not be less than the greatest of the following
- Dia meter of bar if bars are of same diameter.
- Dia of larger bar if dia are unequal.
- 5mm more than the nominal maximum size of coarse aggregate.

(ii) When the bars are in rows, they should be vertically inline & minm vertical distance betn the bars shall be greater of following
- 15mm
- 2/3rd of nominal maxm size of aggregate
- Maxm diameter of bar

(e) Nominal Cover:
→ depth of concrete cover to all steel reinforcement.
→ should not be less than dia. of bar in any case.
→ Nominal cover is provided in RCC design for following purposes.

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- To protect the reinforcement against corrosion
- To provide cover against fire
- To develop ~~the~~ the sufficient bond strength along surface area of steel bar.

→ IS 456:2000, Table 16 for nominal cover to meet durability requirements.

* Types of problems in design of singly reinforced beam
(i) Determining area of tensile steel if b, d & Mu are given
(ii) Determining the dimensions (Cross-sectional) & area of tensile reinforcement.

Type I: Determining area of tensile steel if b, d & Mu are given
1) Determine $\frac{x_{umax}}{d}$ for given type of steel (Pu code, 37)

2) Determine limiting moment of resistance, (Cl. 1.1.(c))
$$M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d} \left(1 - 0.42 \frac{x_{umax}}{d}\right) bd^2$$

3) Compare Mu & Mulim
(i) Mu = Mulim, balance section, Area of steel can be found as
$$A_{st} = \frac{M_u}{0.87 f_y (d - 0.42 x_{umax})}$$

(ii) Mu < Mulim, under reinforced section, (Cl. 1.1(b))
$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{bd f_{ck}}\right)$$

(iii) Mu > Mulim, the section is over-reinforced & hence should be redesigned or design a doubly reinforced section.

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Type II: Determining dimensions of beam & area of tensile reinforcement

1) Determine design bending moment (M_u)

i) Assume suitable value of depth & width

depth of beam (D) = $l/12$ to $l/15$

width of beam (b) = $\frac{D}{2}$ to $\frac{D}{3}$

(ii) Calculate self weight of beam and determine total load (w) by adding imposed loads to self weight

(iii) Determine design or factored load (w_u)

$w_u = w * \gamma_f$.

($\gamma_f = 1.5$ for DL+LL)

(iv) Calculate factored Moment (M_u)

For simply supported beam, $M_u = \frac{w_u \cdot l^2}{8}$

For cantilever beam, $M_u = \frac{w_u \times l^2}{2}$

2) Calculate the limiting value of bending moment ($M_{u lim}$) by using Annex 'G' of IS 456:2000.

3) If $M_{u lim} > M_u$, go to next step otherwise increase the depth of beam & repeat the step.

4) Calculation of area of reinforcement using following equation,

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

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Calculate number of bar & diameter of bar.

5) Check for minimum area of reinforcement.

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

6) Check for maxm area of reinforcement

$A_{st max} = 4\%$ of bD

7) Design of shear reinforcement

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

$\tau_c \rightarrow$ Table 19 of IS 456:2000

→ When $\tau_v < \tau_c$, provide nominal reinforcement consisting of two legged 8mm or 6mm bars at not more than $0.75d$ or 30mm whichever is less.

→ Find $\tau_{c max} \rightarrow$ Table 20

→ If $\tau_c < \tau_v < \tau_{c max}$, then design shear reinforcement.

$\tau_v > \tau_{c max}$, revise the section

8) Check for deflection

9) Calculation of development length (L_d)

10) Design Summary with b, d, D & section diagram.

Q. A reinforced concrete beam is 300mm × 700mm is subjected to a bending moment of 150kNm. Determine the area of reinforcement if M20 concrete & Fe415 steel is used. Take effective cover as 40mm.

→ Given,

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width of beam (b) = 300 mm
Depth of beam (D) = 700 mm
Bending moment (M) = 150 kNm
$f_{ck} = 20 N/mm^2$ & $f_y = 415 N/mm^2$
Now, Effective depth (d) = 700 - 40 = 660 mm
For Fe415 steel, $\frac{x_{umax}}{d} = 0.48$
Limiting moment of resistance,

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

$$= 0.36 \times 20 \times 0.48 (1 - 0.42 \times 0.48) \times 300 \times 660^2$$
$$= 360.58 \text{ kNm}$$

Design moment, $M_u = 150 \times 1.5 = 225$ kNm
Here, $M_{ulim} > M_u$, so section is underreinforced.
Area of steel can be calculated as,

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

$$\Rightarrow 225 \times 10^6 = 0.87 \times 415 \times A_{st} \times 660 \left(1 - \frac{415 \times A_{st}}{20 \times 300 \times 660}\right)$$

$$\Rightarrow A_{st} = 1062.53 \text{ mm}^2$$

∴ Required area of reinforcement = 1062.53 mm^2

Q// A rectangular reinforced concrete beam is simply supported on two masonry walls 230 mm thick & 6m apart (centre to centre). The beam is carrying an imposed load of 15 kN/m. Design the beam with all necessary checks. Use M25 concrete & Fe415 steel.

→ Given,

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Thickness of masonry wall = 230 mm
centre to centre distance = 6m
Imposed load = 15 kN/m
$f_{ck} = 25 N/mm^2$ & $f_y = 415 N/mm^2$
Assuming, depth of beam = $(l/12 \text{ to } l/15) = (500 \text{ to } 400)$ mm
Take,
D = 500 mm

$$b = D/2 = 250 \text{ mm}$$

Assume, effective cover, ~~d~~ = 50 mm,
d = D - 50 = 450 mm
Effective span = Min^m of (c/c distance or clear distance (span) + d)
= Min^m of (5.77 + 0.23 or 5.77 + 0.45)
$l_{eff} = 6m$

→ Design load Calculation,
self wt of beam = 25 * 0.25 × 0.5 = 3.125 kN/m
Imposed load = 15 kN/m
Total load (w) = 15 + 3.125 = 18.125 kN/m
$w_u = 1.5 \times 18.125 = 27.2$ kN/m

$$M_u = \frac{w_u \times l_{eff}^2}{8} = \frac{27.2 \times 6^2}{8} = 122.4 \text{ kNm}$$

→ Limiting B.M (M_{ulim})

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

For Fe415, $\frac{x_{umax}}{d} = 0.48$

Then, $M_{ulim} = 0.36 \times 25 \times 0.48 (1 - 0.42 \times 0.48) \times 250 \times 450^2$
$= 174.61$ kNm
Since, $M_{ulim} > M_u$, section can be designed as singly reinforced section.

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Now, Calculation of area of reinforcement

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

$$\Rightarrow 122.4 \times 10^6 = 0.87 f * 415 \times A_{st} \times 450 \times \left(1 - \frac{A_{st} \times 415}{250 \times 450 \times 25}\right)$$

$$\Rightarrow A_{st} = 863.33 \text{ mm}^2$$

→ Minimum area of steel,

$$A_{st\,min} = \frac{0.85 bd}{f_y} = \frac{0.85 \times 250 \times 450}{415} = 230.42 \text{mm}^2 < A_{st}$$

→ Maximum area of steel,

$$A_{st\,max} = 4\% \text{ of } bD = 0.04 \times 250 \times 500 = 5000 \text{ mm}^2$$

→ Provide 20mm dia bars, then

$$\text{number of bar required} = \frac{863.33 \times 4}{\pi \times 20^2} = 2.748 \simeq 3$$

provide three bars of 20mm dia.

$$A_{st,pro} = 3 \times \frac{\pi \times 20^2}{4} = 942.47 \text{ mm}^2$$

→ Design of shear reinforcement:-

$$V_u = \frac{w_u \times l_{eff}}{2} = \frac{27.2 \times 6}{2} = 81.6 \text{ kN}$$

$$\tau_v = \frac{V_u}{bd} = \frac{81.6 \times 10^3}{250 \times 450} = 0.725 \text{ N/mm}^2$$

$$p_t = \frac{100 A_{st}}{bd} = \frac{100 \times 942.47}{250 \times 450} = 0.837\%$$

From table 19 of IS456:2000

0.75		0.57
0.837		?
1		0.64

$$\tau_c = 0.57 + \frac{0.64 - 0.57}{1 - 0.75} \times (0.837 - 0.75)$$

$$\tau_c = 0.589 \text{ N/mm}^2$$

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From Table 20, For M25 grade of concrete, $\tau_{cmax} = 3.1 \text{ N/mm}^2$

Here, $\tau_v > \tau_c$ but $\tau_v < \tau_{cmax}$. Hence, shear reinforcement is required

Shear to be carried by reinforcement, = V_{us},

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

$$V_{us} = 81600 - 0.589 \times 250 \times 450 = 15225 \text{ N}$$

Using 8mm ϕ, 2 legged stirrups,

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.53 \text{ mm}^2$$

Spacing of stirrups, $S_v = \frac{0.87 f_y A_{sv} d}{V_{us}} = \frac{0.87 \times 415 \times 100.53 \times 450}{15225}$

$$S_v = 1072.79 \text{ mm}$$

Max^m spacing of stirrup;

$$S_{v,max} = \frac{0.87 A_{sv} f_y}{0.4 b} = \frac{0.87 \times 100.53 \times 415}{0.4 \times 250}$$

$$S_{vmax} = 362.96 \text{mm}$$

The spacing should be least of the following also,

i) $0.75 \times d = 337.5$ mm

ii) 300 mm

So, $S_v = 300$ mm

Provide 8mm ϕ, two legged @ 300mm c/c throughout the length of beam.

Provide 2-10mm ϕ anchor bars in compression zone.

→ Check for deflection,

Here, $p_t = 0.837\%$

$$f_s = 0.58 f_y \times \left[\frac{A_{st,req}}{A_{st,pro}}\right] = 0.58 \times 415 \times \left[\frac{863.33}{942.47}\right]$$

$$f_s = 220.48 \text{ N/mm}^2$$

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For $f_s = 220.48\ N/mm^2$ & $p_t = 0.837\%$, from fig 4,

190	1.23
220.48	k_t
240	1.02

$$k_t = 1.23 + \frac{1.02 - 1.23}{240 - 190} \times (220.48 - 190) = 1.101$$

$$\left(\frac{l}{d}\right)_{max} = 20 \times 1.101 = 22.04 \quad \text{(for simply supported)}$$

$$\left(\frac{l}{d}\right)_{pro} = \frac{6000}{450} = 13.33 < \left(\frac{l}{d}\right)_{max}, \text{ OK}$$

→ Check for development length,

From clause 26.2.3.3, $\frac{M_1}{V} + l_o > L_d$

Here, $M_1 = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$

$= 0.87 \times 415 \times 942.47 \times 450 \left(1 - \frac{942.47 \times 415}{250 \times 450 \times 25}\right)$

$= 131.83\ KNm$

$V = 81600\ N$

Using no bend or hook, $l_o = 0$

$$\frac{M_1}{V} + l_o = \frac{131.83 \times 10^6}{81600} + 0 = 1615.57\ mm$$

Also,

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 20}{4 \times (1.6 \times 1.4)} = 805.91\ mm$$

($\tau_{bd} = 1.4\ N/mm^2$ for M25 concrete, 26.2.1.1)

$\frac{M_1}{V} + l_o > L_d$, Hence, codal requirements are satisfied.

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Design Summary,

Beam size = 250mm × 500mm

Main tensile bars = 3 - 20mm ϕ of Fe415 grade

Shear stirrups = 8mm ϕ 2 lgd @ 300mm c/c

Anchor bars = 2 - 10mm ϕ

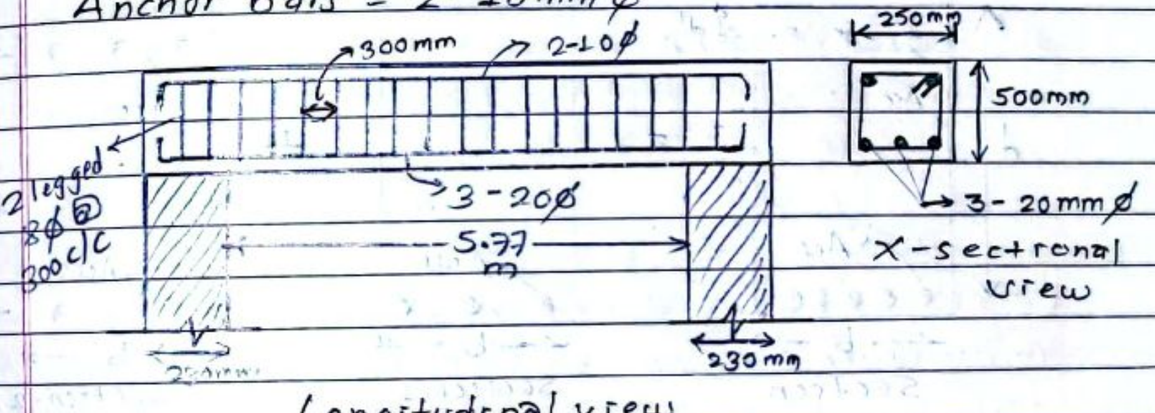


Fig: Detail of reinforcements in beam (250×500mm)

Design of Doubly Reinforced Beam:

→ The R.C.C beams in which the steel reinforcement is placed in tension as well as compression zone are called doubly reinforced beams.

→ Sometimes due to head room constraints or architectural considerations, the size of beam is restricted & same beam (b×d) is required to resist a greater moment. It can be done in two ways

i) By using an over-reinforced section

ii) By using a doubly reinforced section

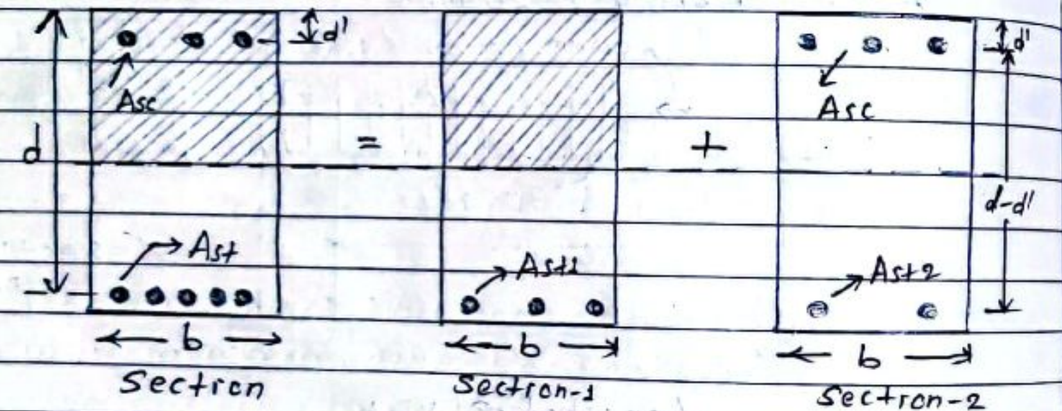
→ Option (i) is not a good choice because over-reinforced sections are uneconomical & the failure of these beams is sudden without warning. Therefore, it is better to use doubly reinforced beam section in such circumstances.

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* Analysis of Doubly Reinforced Beam

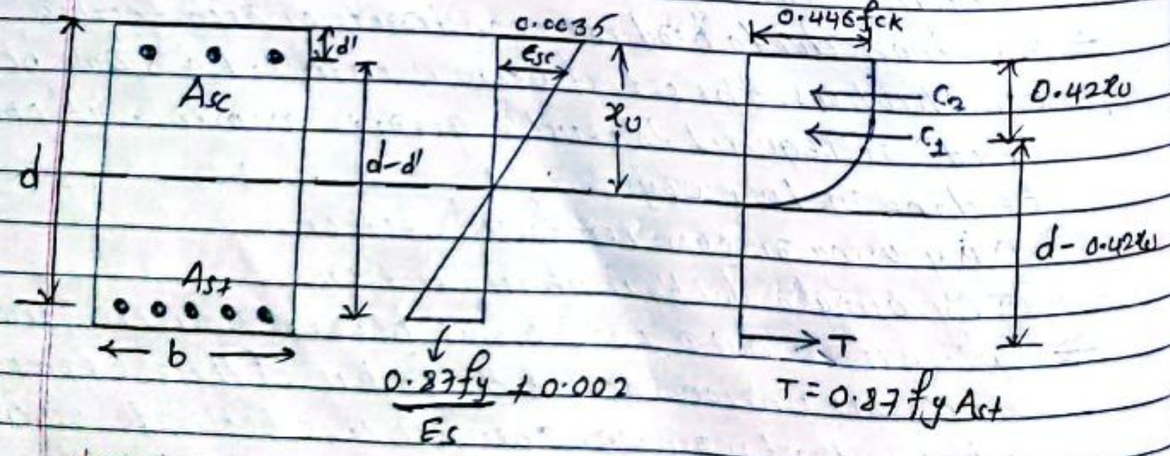
→ A doubly reinforced beam has moment of resistance greater than that of balanced section. Therefore, a doubly reinforced beam subjected to moment (Mu) can be analysed by considering it to consist of two sections.



→ Section-1 consists of a singly reinforced balanced section having area of steel Ast1 & Moment of resistance Mulim.

→ Section 2 consists of compression steel Asc & additional tensile steel Ast2 corresponding to Asc. The moment of resistance of this section is Mu2 such that,

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



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Here, b = width of beam, xu = depth of neutral axis
d = Effective depth of beam
fsc = Stress in compression steel
d' = Effective cover to compression steel
fcc = Stress in concrete at level of steel
Asc = Area of compression steel
Ast = Area of tension steel

→ Depth of Neutral Axis:-

Total compression (C) = $C_1 + C_2$

C_1 = force carried by concrete area = $0.36 f_{ck} b x_u$

C_2 = force carried by Compression steel = $f_{sc} A_{sc} - f_{cc} \cdot A_{sc}$

Then,

$$C = 0.36 f_{ck} b x_u + f_{sc} A_{sc} - f_{cc} A_{sc}$$

$$C = 0.36 f_{ck} b x_u + (f_{sc} - f_{cc}) A_{sc}$$

Total tension (T) = $0.87 f_y A_{st}$

Equating compression & tension force,

$$0.36 f_{ck} b x_u + (f_{sc} - f_{cc}) A_{sc} = 0.87 f_y A_{st}$$

$$x_u = \frac{0.87 f_y A_{st} - (f_{sc} - f_{cc}) A_{sc}}{0.36 f_{ck} b}$$

Since, f_{cc} is very small as compared to f_{sc}. So,

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

→ Moment of resistance,

$M_{ulim} = C_1 * \text{lever arm}$

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

$M_{u2} = C_2 * \text{lever arm}$

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

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

$= 0.36 f_{ck} b x_u (d - 0.42 x_u) + (f_{sc} - f_{cc}) A_{sc} (d - d')$

If loss of concrete area is neglected.

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

Also, from similar Δ,

$$\frac{0.0035}{\varepsilon_{sc}} = \frac{x_u}{x_u - d'}$$

$$\varepsilon_{sc} = 0.0035 \left(1 - \frac{d'}{x_u}\right)$$

Corresponding to this ε_{sc} value, stress (f_{sc}) in compression steel can be obtained from stress-strain curve of given type of steel or from following table (Table F of Design Aids)

Grade of steel (N/mm^2)	d'/d			
	0.05	0.10	0.15	0.20
250	317.4	317.4	317.4	317.4
415	355	353	342	329
500	424	412	359	370
550	458	441	419	380

* Determination of Ast

$$A_{st1} = \frac{M_{ulim}}{0.87 f_y (d - 0.42 x_{umax})}$$

$$A_{st2} = \frac{M_{u2}}{0.87 f_y (d - d')}, \quad M_{u2} = 0.87 f_y A_{st2} (d - d')$$

$M_{u2} = (f_{sc} A_{sc} - f_{cc} A_{sc}) * (d - d')$

&

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

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→ Determination of A_{sc}

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

We know, $M_{u2} = (f_{sc} A_{sc} - f_{cc} A_{sc}) (d - d')$

Neglecting loss of concrete area,

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

* Design steps of doubly reinforced beam

1) Assume suitable size of section. (Assumption criteria is same as in design of singly reinforced rectangular section.)
2) Calculate factored load & factored bending moment (M_u).
3) Calculate limiting value of depth of neutral axis (x_{umax}) by using clause 38.1 of IS 456:2000
4) Calculate the limiting value of moment of resistance (M_{ulim}) by considering beam is singly reinforced (Annex G, G-1.1.(c))
5) If $M_u < M_{ulim}$, singly reinforced beam is to be designed.
   $M_u > M_{ulim}$, doubly reinforced beam is to be designed.
6) Determine area of tensile reinforcement (A_{st1}) required to resist M_{ulim}.

$$A_{st1} = \frac{M_{ulim}}{0.87 f_y (d - 0.42 x_{umax})}$$

7) Determine M_{u2} & A_{st2}

$M_{u2} = M_u - M_{ulim}$

$$A_{st2} = \frac{M_{u2}}{0.87 f_y (d - d')}$$

7) Find A_{st}, $A_{st} = A_{st1} + A_{st2}$
Choose suitable diameter of bar & provide them.

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8) Determine area of compression steel,

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

determine value of f_{sc} for d'/d ratio. (page 38 of pucl) (Table F - Design Aid)

9) Provide A_{sc} by choosing suitable diameter of bar

10) Check for deflection control,

For $p_t = \frac{100 A_{st}}{bd}$ & $f_s = 0.58 f_y \left[\frac{A_{st,pro}}{A_{st,req}}\right]$, find k_t (Frg 4 IS)

$p_c = \frac{100 A_{sc}}{bd}$ & find k_c (frg 5)

$\left(\frac{l}{d}\right)_{max} = 20 \times k_t \times k_c$ (simply supported)

$= 7 \times k_t \times k_c$ (Cantilever)

$\left(\frac{l}{d}\right)_{max} > \left(\frac{l}{d}\right)_{provided}$, ok otherwise redesign

11) Check for shear

12) Check for development length

$$\frac{M_1}{V_u} + l_o > L_d$$

13) Design summary & a sketch showing reinforcement detailing.

Q Design a rectangular beam 230mm × 600mm over an effective span of 5m. The superimposed load on the beam is 50 kN/m. Effective cover to reinforcement is taken as 50mm. Use M20 concrete & Fe415 steel.

→ Given, $b = 230mm$

$D = 600mm$, $l = 5m$

$d = 600 - 50 = 550mm$

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

→ Calculation of factor load & factored BM

self wt $= (0.23 \times 0.6) \times 25 = 3.45 kN/m$

Live load $= 50 kN/m$

Total load $= 50 + 3.45 = 53.45 kN/m$

Design load / factored load $(w_u) = 1.5 \times 53.45 = 80.175 kN/m$

$$M_u = \frac{w_u \times l^2}{8} = \frac{80.175 \times 5^2}{8} = 250.54 kNm$$

$$V_u = \frac{w_u \times l}{2} = \frac{80.175 \times 5}{2} = 200.43 kNm$$

→ For Fe415 steel, $x_{umax}/d = 0.48$

→ $M_{u lim} = 0.138 f_{ck} b d^2$

$= 0.138 \times 20 \times 230 \times 550^2$

$= 192.027 kNm$

Since, $M_u > M_{u lim}$, So design doubly reinforced beam.

→ $A_{st1} = \frac{M_{u lim}}{0.87 f_y (d - 0.42 x_{umax})} = \frac{192.027 \times 10^6}{0.87 \times 415 (550 - 0.42 \times 0.48 \times 550)}$

$A_{st1} = 1211.18 mm^2$

$M_{u2} = M_u - M_{u lim} = 250.54 - 192.027 = 58.513 kNm$

$A_{st2} = \frac{M_{u2}}{0.87 f_y (d - d')} = \frac{58.513 \times 10^6}{0.87 \times 415 \times (550 - 50)}$

$A_{st2} = 324.12 mm^2$

So, $A_{st} = 1211.18 + 324.12 = 1535.3 mm^2$

Using 20mm dia bars, $a = \pi \times 20^2/4 = 314.16 mm^2$

No. of bar $= \frac{1535.3}{314.16} = 4.88 \approx 5$

Provide 5 - 20mm dia bars.

distance of $0.08 L = 400 mm$ from the supports.

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Here, $d'/d = 50/550 = 0.0909 \approx 0.1$

For Fe415, $f_{sc} = 353\ N/mm^2$

Area of compression steel,

$$A_{sc} = \frac{M_{u2}}{f_{sc}(d-d')} = \frac{52.513\times10^6}{353(550-50)} = 331.51\ mm^2$$

Using 16 mm dia bars,

$$\text{no. of bar} = \frac{331.51}{\pi\times16^2/4} = 1.648 \approx 2$$

Provide 2-16 mm dia bars.

So,

$A_{st,pro} = \pi/4 \times 20^2 \times 5 = 1570\ mm^2$

$A_{sc,pro} = \pi/4 \times 16^2 \times 2 = 402\ mm^2$

→ Check for deflection,

$$p_t = \frac{100A_{st}}{bd} = \frac{100\times1570}{230\times550} = 1.24\%$$

$$p_c = \frac{100A_{sc}}{bd} = \frac{100\times402}{230\times550} = 0.32\%$$

$$f_s = 0.58\times f_y \times \left(\frac{A_{st,req}}{A_{st,pro}}\right) = 0.58\times415\times\left(\frac{1535.3}{1570}\right)$$

$$= 235.4\ N/mm^2$$

For $f_s = 235.4\ N/mm^2$ & $p_t = 1.24\%$.

190	1.12
235.4	k_t=?
240	0.95

$$k_t = 1.12 + \frac{0.95-1.12}{240-190}\times(235.4-190)$$

$$= 0.965$$

classmate $k_c = 1.1$

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So, $(l/d)_{max} = 20\times0.965\times1.1 = 21.23$

$(l/d)_{pro} = (5000/550) = 9.09 < (l/d)_{max}$, OK

→ Design of shear,

$V_u = 200.43\ kN$

$$\tau_v = \frac{V_u}{bd} = \frac{200.43\times10^3}{230\times550} = 1.58\ N/mm^2$$

From Table 20, $\tau_{cmax} = 2.8\ N/mm^2 > \tau_v$ (ok)

A_{st} at support $= 3\times\frac{\pi}{4}\times20^2 = 942\ mm^2$ (2 bars are curtailed)

$$p_t = \frac{100\cdot A_{st}}{bd} = \frac{100\times942}{230\times550} = 0.75$$

For M20 concrete & $p_t = 0.75\%$, $\tau_c = 0.56\ N/mm^2$

Here, $\tau_v > \tau_c$, so, shear reinforcement is to be provided.

$$V_{us} = V_u - \tau_c \times bd = 200.43\times1000 - 0.56\times230\times550$$

$$= 129590\ N$$

Using 2 legged 8 mm ϕ vertical stirrups,

$$A_{sv} = 2\times\pi/4\times8^2 = 100.5\ mm^2$$

Spacing of stirrups, $S_v = \frac{0.87 f_y A_{sv} d}{V_{us}}$

$$S_v = \frac{0.87\times415\times100.5\times550}{129590} = 154\ mm \approx 150\ mm$$

Maxm spacing of stirrup,

$$S_{vmax} = \frac{0.87 A_{sv} f_y}{0.4b} = 394\ mm$$

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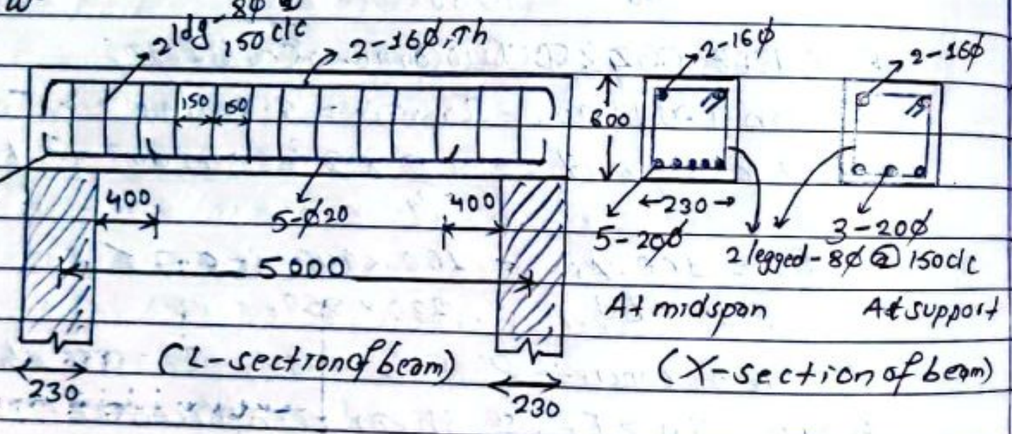
The maxm spacing should be less of following.

i) $0.75 \times d = 412.5$ mm

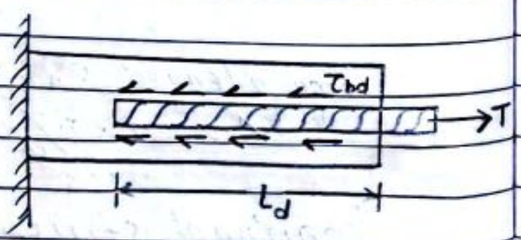
ii) 300 mm

∴ Provide 2 legged 8 mm ϕ stirrups @ 150 mm c/c.

→ The arrangement of reinforcement bars is shown below



* Prove that: $L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}}$



Let us consider a steel bar embedded in concrete. The bar is subjected to tensile force 'T'. Due to this tensile force, the steel bar will tend to come out & slip out of concrete. This tendency of slipping is resisted by the bond stress developed over the surface of bar.

To avoid slipping $T \leq \tau_{bd} \times \left(2\pi \frac{\phi}{2} \times L_d\right)$ --(i)

(surface area)

Also, Tensile force (T) is, $T = 0.87 f_y \times \frac{\pi}{4} \times \phi^2$ --(ii)

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From eqn (i) & (ii),

$$0.87 f_y \times \frac{\pi}{4} \times \phi^2 \leq \tau_{bd} \times 2\pi \times \frac{\phi}{2} \times L_d$$

$$L_d \geq \frac{0.87 f_y \phi}{4 \tau_{bd}}$$

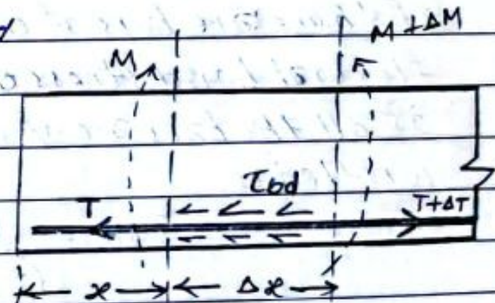
where, L_d = development length of steel bar

f_y = characteristic strength of bar

τ_{bd} = Design bond stress

ϕ = Diameter of bar.

* Prove that, $\frac{M_1}{V} + L_0 \geq L_d$



Consider a beam subjected to flexural loading. Consider two sections at a distance x & $x+\Delta x$ along the length of beam subjected to moment 'M' & 'M + ΔM' resp. Fig.: Flexural bond stress

Let T & T + ΔT be the tension developed at x & $x+\Delta x$ section respectively.

If lever arm = 'a'

M = Tension × Lever arm $= T \times a$ --(i)

$M + \Delta M = (T + \Delta T) a$ --(ii)

Eqn (ii) − Eqn (i)

$$\Delta M = \Delta T a$$

$$\frac{\Delta M}{a} = \Delta T \quad \text{-----(iii)}$$

If τ_{bd} the bond stress developed along the ~~perif~~ periphery then

$$T + \tau_{bd} \times 2\pi \frac{\phi}{2} \times \Delta x = T + \Delta T$$

$$\Delta T = \tau_{bd} \pi \phi \Delta x$$

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From eqn (iii),

$$\frac{\Delta M}{\partial} = \tau_{bd} \times \pi \phi \Delta x$$

$$\frac{\Delta M}{\Delta x} = (\tau_{bd} \times \pi \phi) \times \partial$$

$$\Rightarrow V = \tau_{bd} \times \pi \phi \partial$$

$$\Rightarrow \tau_{bd} = \frac{V}{\pi \phi \partial} \quad ---- (iv)$$

This eqn (iv) gives the flexural bond stress in the tension reinforcement at any section of the beam. If there are bars of different sizes, then the maximum flexural bond stress develops at the bar of large diameter. If all the bars are of same diameter, Eqn (iv) can be written as,

$$\tau_{bd} = \frac{V}{(\Sigma 0) \partial} \quad ---(v)$$

Where, $\Sigma 0$ = Total perimeter of all bars in tension
For 'n' number of bar at that section,

$$\Sigma 0 = n(\pi \phi)$$

Also,

$$\tau_{bd} = \frac{0.87 f_y A_{st}}{L_d \times \Sigma 0} \quad [\text{eqn for development length}] \quad ---(vi)$$

Equating eqn (v) & (vi),

$$\frac{V}{(\Sigma 0) \partial} = \frac{0.87 f_y A_{st}}{L_d (\Sigma 0)}$$

$$L_d = \frac{0.87 f_y A_{st}}{V}$$

$$L_d = M_1 / V$$

If l_0 be the sum of anchorage beyond the centre of support & equivalent anchorage value of any hook. Then,

$$L_d \leq \frac{M_1}{V} + l_0 \quad ----(vii)$$

Eqn (vii) is the required expression.

Q A simply supported beam 300mm x 600 mm (effective) is reinforced with 5 bars of 25mm diameter. It carries a uniformly distributed load of 80 kN/m (including its own weight) over an effective span of 6m. Out of the 5 main bars, two bars can be bent up safely near the supports. Design the shear reinforcement for the beam. Use M20 grade of concrete & Fe 415 steel.

→ Given,

$b = 300$ mm, $d = 600$ mm
$w = 80$ kN/m, $f_{ck} = 20$ N/mm²
$f_y = 415$ N/mm², $l_{eff} = 6m = 6000$ mm

Factored shear force (V_u)

Assuming 400mm bearing,

clear span (L) = $6000 - 400 = 5600$ mm $= 5.6$ m

Factored load = $80 \times 1.5 = 120$ kN/m

$$V_u = \frac{W_u \times L}{2} = \frac{120 \times 5.6}{2} = 336 \text{ kN}$$

$$\text{Nominal shear stress } (\tau_v) = \frac{V_u}{bd} = \frac{336 \times 10^3}{300 \times 600} = 1.86 \text{ N/mm}^2$$

$\tau_{c,max} = 2.8$ N/mm² [for M20 concrete]

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

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Design shear strength of Concrete

Area of steel available near support is 3 bars of 25mm diameter

$$A_{st} = 3 \times \frac{\pi}{4} \times 25^2 = 1472.6 \text{ mm}^2$$

$$p_t = \frac{100 A_{st}}{bd} = \frac{100 \times 1472.6}{300 \times 600} = 0.82\%$$

For M20 concrete & $p_t = 0.82\%$

$\tau_c = 0.58 \text{ N/mm}^2$

$\tau_v > \tau_c$, hence shear reinforcement is to be designed

Shear taken by shear rebar

$V_{us} = V_u - \tau_c b d$

$= 336 \times 1000 - 0.58 \times 300 \times 600 = 231600 \text{ N}$

$\frac{V_{us}}{2} = 115800 \text{ N}$

→ Bentup Bars:

Shear resistance of bentup bars = V_{us}'

$= 2 (0.87 f_y . A_{sv} . \sin\alpha)$

$= 2 \times 0.87 \times 415 \times \frac{\pi}{4} \times 25^2 \times \sin 45^\circ$

$V_{us}' = 250641 \text{ N} > \frac{V_{us}}{2}$

But $V_{us}' \neq \frac{V_{us}}{2}$

$V_{us}' = \frac{V_{us}}{2} = 115800 \text{ N}$

Balance shear force to be carried by vertical stirrups is,

$V_{us} = 231600 - 115800$

$= 115800 \text{ N}.$

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→ Design of vertical Shear stirrups,

Assuming 8mm ϕ 2-legged stirrups,

$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.54 \text{ mm}^2$

Spacing of stirrups, $S_v = \frac{0.87 f_y A_{sv} d}{V_{us}}$

$= \frac{0.87 \times 415 \times 100.54 \times 600}{115800} = 188 \text{ mm}$

$S_v = 180 \text{ mm (say)}$

Spacing corresponding to minimum reinforcement,

$S_v = \frac{0.87 f_y A_{sv}}{0.4} = \frac{0.87 \times 415 \times 100.5}{0.4 \times 300}$

$S_v = 302 \text{ mm}$

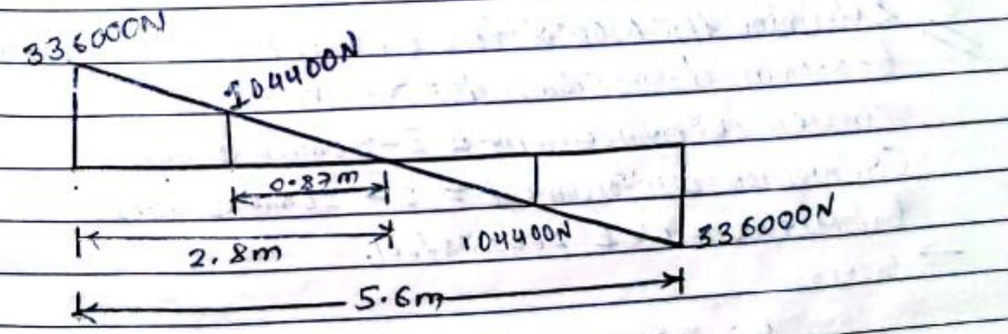
→ Maximum spacing

The maximum spacing of stirrups shall be least of following.

i) $0.75d = 0.75 \times 600 = 50 \text{ mm}$

ii) 300 mm iii) 180 mm iv) 302 mm

Provide 8mm diameter, 2 lgd stirrups @ 180mm c/c.



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Zone for nominal shear reinforcement

Shear resistance of concrete = $\tau_c \times bd$

$= 0.58 \times 300 \times 600$

$= 104400\ N$

$$\frac{336000}{2.8} = \frac{104400}{x}$$

$x = 0.87\ mm$

→ Provide 8 mm ϕ 2 lgd stirrups @ 180 mm c/c upto a distance of $(2.8 - 0.87) = 1.93$ m from the support & 8 mm ϕ 2 lgd stirrups @ 300 mm c/c in the remaining middle portion.

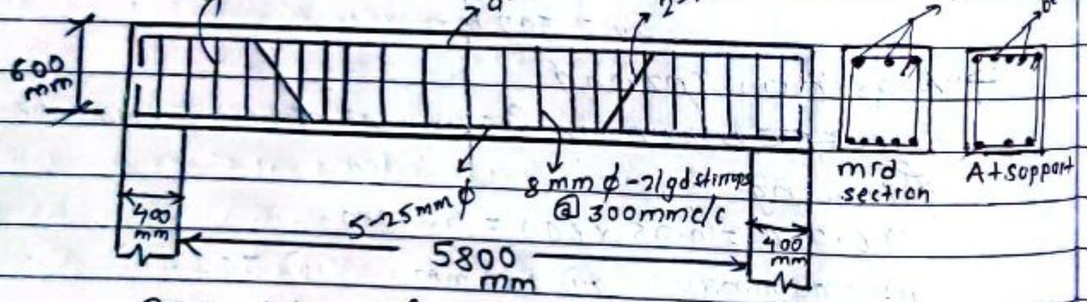


Fig: Details of rebar in beam (L-section).

Q Determine the MOR of the beam having following data.

$b = 350mm$, $d = 900\ mm$, $d' = 50mm$

Tension reinforcement = 5-20mm ϕ bars

Compression reinforcement = 2-20mm ϕ bars

Use M15 concrete & Fe415 steel.

→ Given,

$b = 350mm$ $\quad d = 900\ mm$ $\quad d' = 50mm$

$A_{st} = 5 \times \frac{\pi \times 20^2}{4} = 314.16 \times 5 = 1570.79\ mm^2$

$A_{sc} = 2 \times \frac{\pi \times 20^2}{4} = 628\ mm^2$

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

Here, $\frac{d'}{d} = \frac{50}{900} = 0.05$

For $d'/d = 0.05$ & Fe415, $f_{sc} = 355\ N/mm^2$

→ Depth of neutral axis (x_u)

$$x_u = \frac{0.87 f_y A_{st} - f_{sc} \cdot A_{sc}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times 1571 - 355 \times 628}{0.36 \times 15 \times 350}$$

$= 182.15\ mm$

$x_{umax} = 0.48d = 0.48 \times 900 = 432mm$

$x_u < x_{u,max}$. Hence, the section is under-reinforced.

→ The moment of resistance (M_u) is,

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

$= 0.36 \times 15 \times 350 \times 182.15 (900 - 0.42 \times 182.15) + 355 \times 628 \times (900 - 50)$

$= 472.99\ kNm$

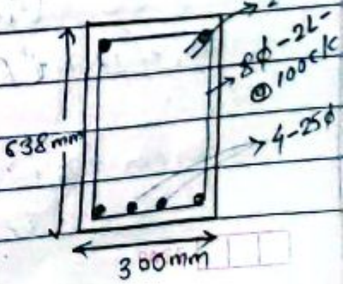
Q A RCC Beam 300 x 600 mm effective is reinforced with 2-20mm ϕ at top & 4-25 mm ϕ at bottom. Again, 8 mm ϕ 2-legged stirrups @ 100 mm c/c is provided at the full length of the beam. Calculate the flexural & shear capacity of the beam for mild exposure condition for M20 grade of concrete & Fe500 steel.

→ Given,

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

$f_y = 500\ N/mm^2$

effective depth (d) = 600 mm



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overall depth (D) = 600 + 25 + 25/2 ≈ 638mm
(clear cover), ϕ_{bottom}

$d' = 25 + 20/2 = 35$

$\frac{d'}{d} = \frac{35}{600} = 0.058$

For Fe500 steel & $d'/d = 0.058$, $f_{sc} = 412\ N/mm^2$

Depth of neutral axis, (x_u)

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

$A_{st} = 4 \times \frac{\pi \times 25^2}{4} = 1964\ mm^2$

$A_{sc} = 2 \times \pi \times 20^2/4 = 628.32\ mm^2$

$$= \frac{0.87 \times 500 \times 1964 - 412 \times 628.32}{0.36 \times 20 \times 300}$$

$= 275.68\ mm$

For Fe500 steel, $x_{umax} = 0.46d$

$= 0.46 \times 600$

$= 276\ mm$

Since $x_{umax} > x_u$, under reinforced section & M_u can be calculated as,

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

$= 0.36 \times 20 \times 300 \times 275.68 (600 - 0.42 \times 275.68) + 412 \times 628.32 \times (600 - 35)$

$= 434.58\ kNm$

This is the flexural capacity of beam.

For shear capacity of beam,

$V_u = V_{us} + \tau_c \times bd$

% of steel (p_t) $= 100 \times \frac{A_{st}}{bd} = \frac{100 \times 1964}{300 \times 600} = 1.091\%$

From Table 19,

	1	0.62
$\tau_c = 0.638\ N/mm^2$	1.091	?
	1.25	0.67

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For vertical stirrups, $V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$

$$V_{us} = \frac{0.87 \times 500 \times (2 \times \pi \times 8^2/4) \times 600}{100} \times 10^{-3}$$

$V_{us} = 262.38\ KN$

∴ Shear capacity of beam (V_u) = $V_{us} + \tau_c b d$

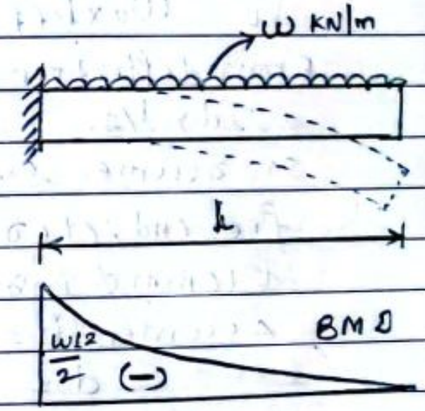
$= 262.38 + 0.638 \times 300 \times 600 \times 10^{-3}$

$= 377.22\ KN$

#

Design of cantilever beam

→ When cantilever beam is subjected to loads, it bends downwards as shown in figure.



→ Due to this upper fibres are subjected to tensile stresses & lower ones to compressive stresses. Therfore, the main steel is to be placed at upper face.

→ Following points are to be considered while designing a cantilever beam.

1) The main tensile reinforcement is to be placed in zone above the neutral axis.
2) The BM in a cantilever varies from zero at free end to maxm at fixed end. Hence, the depth of section is also kept maximum at the fixed end & minm (150 mm) at free end.
3) The width of beam is kept uniform throughout the length of beam

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Q. Design a cantilever beam having an effective span of 3m. The beam is carrying a load of 14kN/m, including its own weight. Use M20 concrete & Fe415 steel.

→ Given,

$L_{eff} = 3m$

$w = 14 kN/m$, $f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$

$w_u = 14 \times 1.5 = 21 kN/m$

$M_u = \frac{w_u \times l_{eff}^2}{2} = 94.5 kNm$

$V_u = w_u \times l_{eff} = 63 kN$

From deflection criteria, $l/7 = 3000/7 = 428.57 mm$

$\therefore d > l/7$

So, assume, $D = 450 mm$ at fixed end & $150 mm$ at free end, clear cover = 20 mm, 8 mm ϕ shear stirrups & 16mm ϕ main tensile bars.

Assume, $b = 250 mm$

$d = 450 - 20 - 8 - 8 = 414 mm$ at fixed end.

Check for d:

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

$94.5 \times 10^6 = 0.138 \times 20 \times 250 \times d^2$

$d_{req} = 370 mm = d_{req.} < d_{provided}$ (ok)

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

⇒ $94.5 \times 10^6 = 0.87 \times 415 \times A_{st} \times 414 \left(1 - \frac{415 \times A_{st}}{20 \times 250 \times 414}\right)$

⇒ $A_{st} = 742.84 mm^2$

For min^m steel, $\frac{A_{st}}{bd} = \frac{0.85}{f_y}$

$\therefore A_{st,min} = \frac{0.85 \times 250 \times 414}{415} = 211.98 mm^2 < A_{st(ok)}$

Using 16mm ϕ bar, no. of bar $= \frac{742.84}{\pi \times 16^2/4} = 3.69 \approx 4$

provide 4-16mm ϕ bars as tensile rebar.

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

→ Design of shear reinforcement,

$\tau_v = \frac{V_u}{bd} = \frac{63 \times 10^3}{250 \times 414} = 0.61 N/mm^2$ $\left[\tau_v = \frac{V_u \pm \frac{M_u}{d} \tan\beta}{bd}\right]$

$p_t = \frac{100 \times A_{st}}{bd} = \frac{100 \times 804}{250 \times 414} = 0.77\%$

$\tau_c = 0.5648 N/mm^2$	0.75	0.56
$\tau_v > \tau_c$. So shear rebar is to be designed.	0.77	τ_c
	1	0.62

$V_{us} = V_u - \tau_c b d = 63 - 0.5648 \times 250 \times 414 \times 10^{-3}$

$= 4.54 kN$

Using 8mm ϕ – two legged stirrups,

$S_v = \frac{0.87 f_y A_{sv} d}{V_{us}} = \frac{0.87 \times 415 \times 2 \times 50.26 \times 414}{4540} = 3309 mm$

$S_{v,max} = \frac{0.87 f_y A_{sv}}{0.4b}$ (clause 26.5.1.6)

$= \frac{0.87 \times 415 \times 2 \times 50.26}{0.4 \times 250} = 362.9 mm$

Max^m spacing, i) $0.75 \times d = 0.75 \times 414 = 310.5 mm$

ii) 300mm

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

Also provide 2-10mm ϕ anchor bars.

→ Check for deflection.

For $p_t = 0.77\%$ & $f_s = 0.58 \times 415 \times \frac{742.84}{804} = 222.4\ N/mm^2$

$K_t = 1.2$

$(l/d)_{pro} = 3000/414 = 7.24$

$(l/d)_{per} = K_t \times 7 = 1.2 \times 7 = 8.4$

$(l/d)_{permissible} > (l/d)_{provided}$ (OK, safe in deflection)

→ Curtailment of bars,

Required development length for 16 mm ϕ bar,

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

$$= \frac{0.87 \times 415 \times 16}{4 \times 1.6 \times 1.2} = 752\ mm$$

Out of '4' bars, 2 bars can be curtailed at a distance of 1.7m from face of support. (curtailed distance > $L/2$)

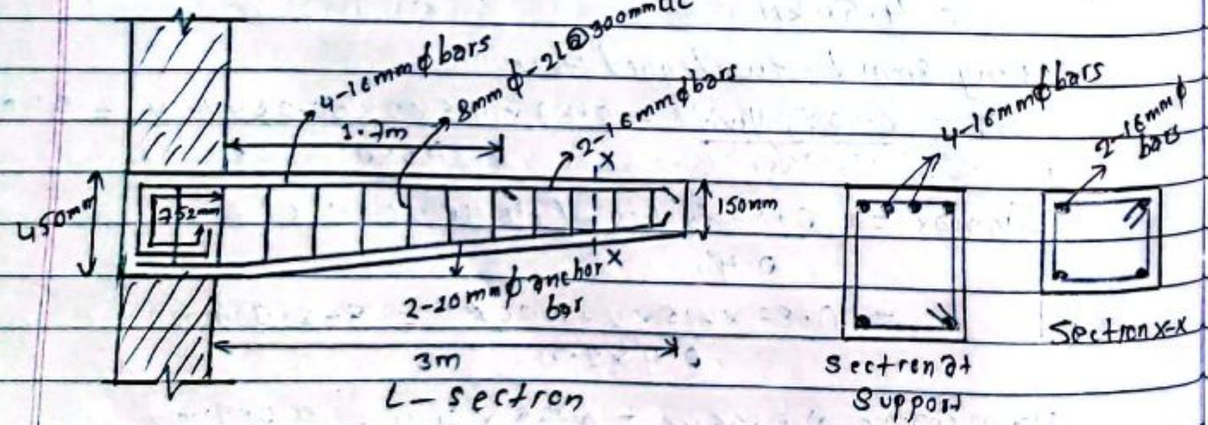


Fig: Reinforcement details of cantilever beam.

✱ Design of Continuous beam:

→ slabs & beams are cast monolithically in reinforced concrete construction. This makes the slab continuous over the supports to enhance structural integrity & continuity.

→ Continuous beams are preferred in building construction because of the better load carrying capacity, less deflection & thinner section, leading to economic design.

→ Continuous beams subjected to UDL over three or more spans, may be designed by using bending moment coefficients & shear force coefficients from Table 12 & 13 of IS 456:2000. However, these values can be used only if the spans don't differ by 15% of longer.

Q. Design a rectangular beam, continuous over four column supports of effective spans 6m. The beam is subjected to an imposed dead load of 10 kN/m & live load of 15 kN/m. Use M20 concrete & Fe 415 steel.

→ Given,

$l_{eff} = 6m$

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

Imposed dead load = 10 kN/m

Live load = 15 kN/m

From preliminary design criteria,

effective depth = $l/15 = \frac{6000}{15} = 400\ mm$

Trying a total depth of 500 mm & width as 300 mm

$d = 500 - 30$

$= 470\ mm$ (assuming 30 mm effective cover)

→ Load Calculation

Self wt of beam = 0.3 x 0.5 x 25 = 3.75 kN/m

Imposed dead load = 10 kN/m

Total dead load = 13.75 kN/m

Factored dead load = 1.5 x 13.75 = 20.625 KN/m

Factored live load = 1.5 x 15 = 22.5 kN/m

→ Bending Moment & shear force coeffrerents

BM Coeff.	Dead load	+1/12	-1/10	+1/16	-1/10	+1/12	
	Live load	+1/10	-1/9	+1/12	-1/9	+1/10	
Shear coeff	DL	0.4	0.6	0.55	0.55	0.6	0.4
	LL	0.45	0.6	0.6	0.6	0.6	0.45

→ Maxm Moment at support = $\frac{-1}{10} \times 20.625 \times 6^2 - \frac{1}{9} \times 22.5 \times 6^2$ (-ve)

= -164.25 kNm

Maxm Moment at midspan = $\frac{1}{12} \times 20.625 \times 6^2 + \frac{1}{10} \times 22.5 \times 6^2$ (+ve)

= 142.875 kNm

Maxm SF ⇒ $V_u = 0.6 \times (20.625 \times 6) + 0.6 (22.5 \times 6)$

= 155.25 KN

→ Check for depth,

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

⇒ $164.25 \times 10^6 = 0.138 \times 20 \times 300 \times d^2$

$d = 445.38 mm < 470 mm$ (OK)

→ $M_{ulrm} = 0.138 f_{ck} b d^2$

$= 0.138 \times 20 \times 300 \times 470^2$

$= 182.9 kNm$

$M_{ulrm} > M_{max}$, so singly reinforced section can be designed.

→ For -ve moment at support,

$164.25 \times 10^6 = 0.87 \times 415 \times A_{st} \times 470 \left(1 - \frac{415 \times A_{st}}{20 \times 300 \times 470}\right)$

$A_{st} = 1169 mm^2$

$A_{st,min} = \frac{0.85 bd}{f_y} = \frac{0.85 \times 300 \times 470}{415} = 288 mm^2$

$A_{st,max} = 4\% of bD = \frac{4}{100} \times 300 \times 500 = 6000 mm^2$

∴ provided A_{st} is in range.

Using 20mm ϕ bars,

no. of bar = $\frac{1169}{314.16} = 3.72 \approx 4$

Provide 4-20mm ϕ longitudinal bars at intermediate supports. $A_{st,pro} = 314.16 \times 4 = 1256 mm^2$

→ for +ve moment at midspan,

$142.875 \times 10^6 = 0.87 \times 415 \times A_{st} \times 470 \left(1 - \frac{415 \times A_{st}}{20 \times 300 \times 470}\right)$

$A_{st} = 985 mm^2 > A_{st,min}$ & $< A_{st,max}$ (Ok)

Using 20mm ϕ bars, no. of bars = $\frac{985}{314.16} = 3.13 \approx 4$

Provide 4-20mm ϕ bars at midspan as +ve reinforcement.

$A_{st,pro} = 1256 mm^2$

→ Shear design,

$V_u = 155.25\ kN$

$\tau_v = \frac{V_u}{bd} = \frac{155.25 \times 10^3}{300 \times 470} = 1.1\ N/mm^2$

$\%P_t = \frac{100 \times A_{st}}{bd} = \frac{100 \times 1256}{300 \times 470} = 0.89\%$

For M20 concrete, $\tau_c = 0.61\ N/mm^2$

$\tau_v > \tau_c$, so shear rebar is to be designed.

$V_{us} = 155.25 - 0.61 \times 300 \times 470 \times 10^{-3}$

$= 69.24\ kN$

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

Let us provide 8mm ϕ - 2L stirrups.

$S_v = \frac{0.87 \times 415 \times (50.26 \times 2) \times 470}{69.24 \times 10^3} = 246.35\ mm$

$S_{vmax} = \frac{0.87 f_y A_{sv}}{0.4 b} = \frac{0.87 \times 415 \times 50.26 \times 2}{0.4 \times 300} = 302.4\ mm$

Maxm spacing should be minm of

i) $0.75d = 0.75 \times 470 = 352\ mm$

ii) 300 mm

Hence, provide 8mm ϕ - 2L stirrups @ 230 mm c/c.

→ check for deflection,

$f_s = 0.58 \times 415 \times \frac{985}{1256} = 188.76\ N/mm^2$

$p_t = \frac{100 \times 1256}{300 \times 470} = 0.89\ N/mm^2$

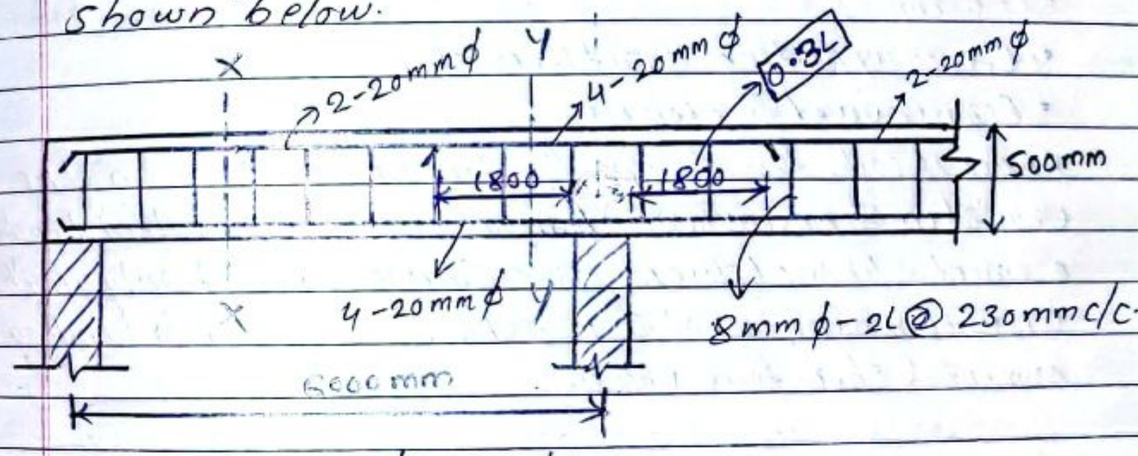
For $f_s = 188.76\ N/mm^2$ & $p_t = 0.89\ N/mm^2$

$k_t = 1.23$

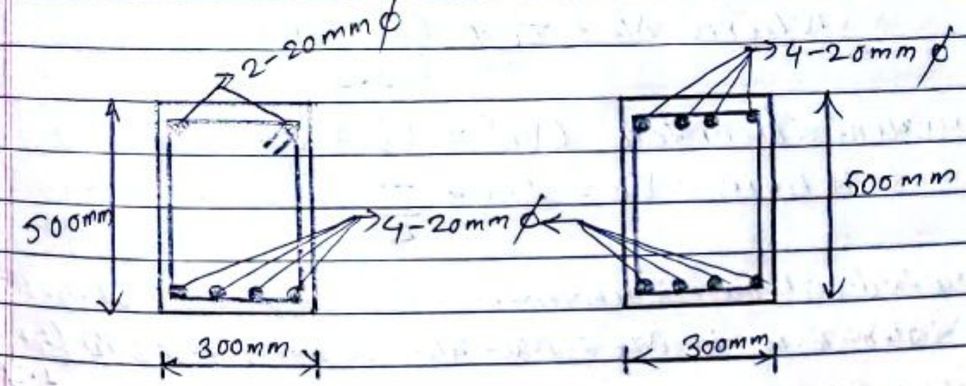
$(l/d)_{permissible} = 1.23 \times 26 = 31.98\ mm$

$(l/d)_{provided} = \frac{6000}{470} = 12.76\ mm < (l/d)_{permissible}$ (ok)

→ The reinforcement details of continuous beam are shown below.



L-section



Section x-x Section y-y

Cross section

Fig: Details of Reinforcement in continuous beam.

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* Design for Torsion:-

→ Reinforced concrete sections are also subjected to torsional moments which cause twisting or warping of section.

→ Some common examples of structures subjected to torsional moments are as follows

- Ring beam provided at bottom of elevated circular water tank.
- L-beams
- Beams supporting a cantilever slab
- Beams curved in plan etc.

→ As per IS code, the design for torsion is based on the calculation of an equivalent shear force & equivalent bending moment. This philosophy gives adequate result only if the torsional moment is not appreciable as compared to bending moment & shear force values.

Equivalent Bending Moment (M_e) = $M_u + M_T$

where, $M_T = T_u * \frac{(1+D/b)}{1.7}$

Equivalent shear force (V_e) = $V_u + V_T$

where, $V_u = 1.6 * \frac{T_u}{b}$

* Longitudinal Reinforcement.

→ Equivalent B.M ⇒ $M_e = M_u + M_T$ where, $M_T = T_u \frac{(1+D/b)}{1.7}$

→ If $M_u > M_T$, designed as singly reinforced

Tension Reinforcement only

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

→ If $M_u < M_t$ ⇒ designed as doubly reinforced,

Tension Reinforcement: $M_{e1} = M_u + M_t$

$$A_{st} = \frac{M_{e1}}{0.87 f_y (d - 0.42 x_m)} + \frac{M_{e2}}{0.87 f_y (d - d_c')}$$

Compression Reinforcement: $M_{e2} = M_t - M_u$

$$A_{sc} = \frac{M_{e2}}{(f_{sc} - f_{cc})(d - d_c')}$$

→ Longitudinal reinforcement shall be placed as close as is practicable to the corners of the cross section & in all cases, there shall be at least one longitudinal bar in each corner of the ties. If the cross sectional dimension of the member exceeds 450mm, additional longitudinal bars shall be provided to satisfy the requirements of min^m reinforcement & spacing given in 26.5.1.3

→ 26.5.1.3 Side face reinforcement

When the depth of the web in a beam exceeds 750mm, side reinforcement shall be provided along the two faces. The total area of such reinforcement shall not be less than 0.1% of the web area & shall be distributed equally on two faces at a spacing not exceeding 300mm or web thickness whichever is less.

→ For T & L beams, see clause 26.5.1.8.

* Transverse Reinforcement:-

→ Two legged closed hoops enclosing the corner longitudinal bars shall have an area of cross-section (A_{sv}) given by

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

but the total transverse reinforcement shall not be less than

$$A_{sv} \geq \frac{(T_{ve} - T_c) b S_v}{0.87 f_y}$$

Where, T_u = Torsional Moment
V_u = Shear Force
b_1 = c/c distance betⁿ corner bars in the direction of width
d_1 = c/c distance betⁿ corner bars in the dirⁿ of depth.
τ_{ve} = Equivalent shear stress
τ_c = Shear strength stress of concrete (Table 19)

→ τ_{ve} can be calculated as,

$$\tau_{ve} = \frac{V_e}{bd}, \quad V_e = V_u + \frac{1.6 T_u}{b}$$

If $\tau_{ve} > \tau_{cmax}$ (Table 20), section is to be redesigned.
If $\tau_{ve} < \tau_{cmax}$, section is designed as,
1) If $\tau_{ve} < \tau_c$, nominal shear reinforcement is provided in the form of stirrups.

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

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

2) If $\tau_{ve} > \tau_c$, both longitudinal & transverse rebar shall be provided in accordance with 41.4.

→ Distribution of transverse reinforcement (cls 26.5.1.7)
The transverse reinforcement for torsion shall be rectangular closed stirrups placed perpⁿ to axis of member. The spacing of stirrups shall not exceed the least of
i) x_1 (ii) $(x_1 + y_1)/4$ (iii) 300 mm
where, x_1 = short dimension of stirrups = b - 2x side cover
y_1 = long dimension of stirrups = D - 2x side cover

Q) Determine the reinforcement required for a beam of size 300mm x 600mm subjected to a factored BM of 150kN, factored shear force of 100kN & factored torsional moment of 50kNm. Use M20 concrete & Fe415 steel.

→ Given,
Size of beam = 300mm x 600mm
factored B.M = 150kNm
factored S.F = 100kN
factored torsional moment (T_u) = 50kNm
$f_{ck} = 20 N/mm^2$ & $f_y = 415 N/mm^2$

Assuming, CC = 20mm
$d = 600 - 20 - 10 = 570$ mm (10mm → 20mm ϕ)

Equivalent shear, $V_e = V_u + \frac{1.6 T_u}{b} = 100 + \frac{1.6 \times 50}{0.3}$
$= 100 + 266.67$
$= 366.67$ kN

Equivalent Moment, $M_e = M_u + M_t$ → $\frac{T_u (1 + D/b)}{1.7}$
$= 150 + \frac{50 (1 + 0.6/0.3)}{1.7}$
$= 150 + 88.23$
$= 238.23$ kNm

Since, $M_u > M_t$, it is designed as singly reinforced section.

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

$$\Rightarrow 238.23 \times 10^6 = 0.87 \times 415 \times A_{st} \times 570 \left(1 - \frac{415 \times A_{st}}{20 \times 300 \times 570}\right)$$

$\Rightarrow A_{st} = 1393 mm^2$

Providing 5- bars of 20 mm ϕ ($A_{st,pro} = 5 \times 314.16 = 1570 mm^2$)
In addition to this, provide two 12mm ϕ bars on compressive side.

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Equivalent shear stress (τ_{ve}) $= \frac{V_e}{bd} = \frac{366.67 \times 10^3}{300 \times 570} = 2.14\ N/mm^2$

For M20 concrete, $\tau_{cmax} = 2.8\ N/mm^2 > \tau_{ve}$ (ok)

$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 1570}{300 \times 570} = 0.92\%$

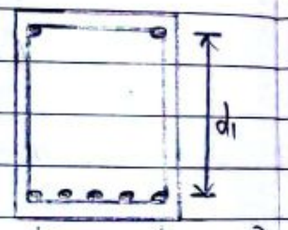
$\tau_c = 0.6\ N/mm^2$	0.75	0.56
$\tau_{ve} > \tau_c$, so we need to	0.92	τ_c
design the shear reinforcement.	1	0.62

→ Spacing of stirrups, (cls 41.4.3)

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

⇒ Assuming 8mmϕ stirrups (2L)

$A_{sv} = 2 \times 50.26 = 100.52\ mm^2$



$b_1 = 300 - 20 - 20 - 16 - 20$ (8+8 stirr)
$= 224\ mm$

$d_1 = 600 - 20 - 20 - 16 - 10 - 6$
$= 528\ mm$

$$100.52 = \frac{50 \times 10^6 \times S_v}{224 \times 528 (0.87 \times 415)} + \frac{100 \times 10^3 \times S_v}{2.5 \times 528 \times (0.87 \times 415)}$$

$S_v = 72.8\ mm$

Also,

$$A_{sv} \geq \frac{(\tau_{ve} - \tau_c) b S_v}{0.87 f_y}$$

$$100.52 \geq \frac{(2.14 - 0.6) \times 300 \times S_v}{0.87 \times 415}$$

$S_v \leq 78.55\ mm$

The spacing should be less than

i) $x_1 = 300 - 2 \times 20 = 260\ mm$

ii) $\frac{x_1 + y_1}{4} = \frac{260 + 560}{4} = 205\ mm$

iii) 300 mm

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Hence, provide 2 legged - 8mmϕ stirrups @ 70 mm c/c
[70mm spacing is very small, so you can also redesign the shear stirrups taking 10 mmϕ]

→ Side face reinforcement

Area of side face reinforcement = 0.05% on each face

$= \frac{0.05}{100} \times 300 \times 600$

$= 90\ mm^2$

Provide one bar of 12mmϕ on each face in the middle.

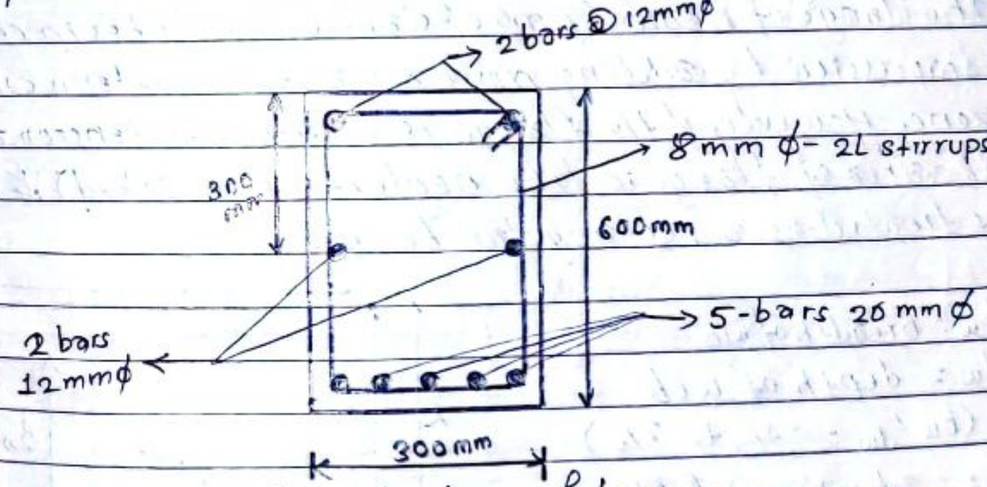


Fig: Cross section of beam.
(Reinforcement details)

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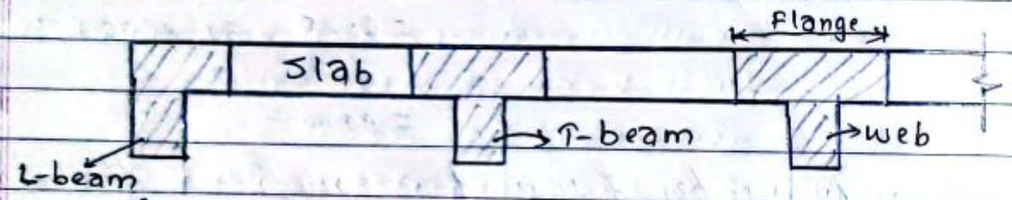
Analysis & Design of Flanged Beam

Sumit Paudel

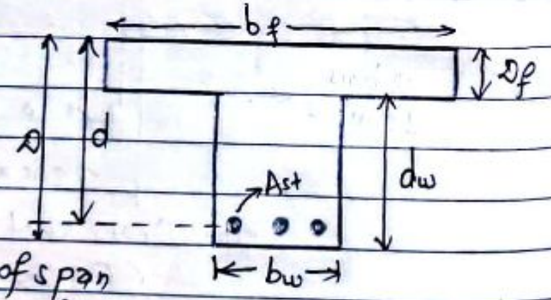
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→ In RCC construction, slabs & beams are cast monolithically. The portion of the slab that acts integrally with beam to resist loads is called flange. The portion of the beam below the flange is called web.

→ The intermediate beams supporting the slab are called T-beams & end beams are called L-beams.



→ The flange of beam (part of slab) contributes in resisting compression by adding more area of concrete in compression zone. However, if the flange is located in tension zone, the concrete area of flange is to be neglected (cracked) & beam is treated as a rectangular beam.



→ b_w = breadth of web
d_w = depth of web
(b_w/d_w = 1/3 to 2/3)
D_f = Thickness of flange
D = overall depth = 1/12 to 1/15 of span (in case of simply supported beam)

In continuous beam,
D → (1/15 − 1/20) of span, for light loads
(1/12 − 1/15) of span, for medium loads
(1/10 − 1/12) of span, for heavy loads.

b_f = Effective width of flange
(cl.s 23.1.2 of IS 456:2000)

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* Effective Width of Flange:-

(a) For T beams, $b_f = \frac{l_o}{6} + b_w + 6D_f \leq b_w + \frac{1}{2} * \text{clear span on both sides}$

(b) For L beams, $b_f = \frac{l_o}{12} + b_w + 3D_f \leq b_w + \frac{1}{2} * \text{clear span on both sides}$

(c) For isolated beams

(i) For T beam, $b_f = \frac{l_o}{(l_o/b + 4)} + b_w < b$

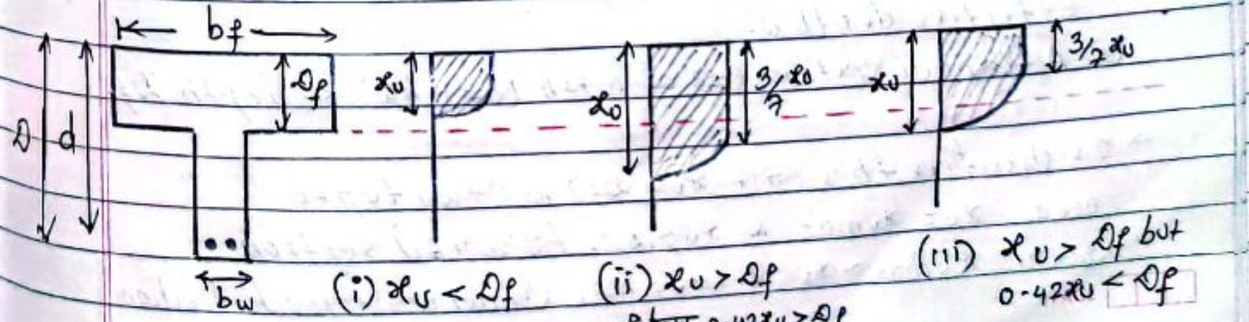
(ii) For L-beam, $b_f = \frac{0.5 l_o}{b_o/b + 4} + b_w < b$

Where, b_f = effective width of flange,
b = actual width of flange,
b_w = breadth of web
D_f = Thickness of flange
l_o = distance betn points of zero moments (for continuous beams, l_o is taken as 0.7 * leff)

Analysis of T beam:-

→ Three cases are occured in analysis of T-beam

i) $x_u < D_f$ ii) $x_u > D_f$ → $0.42x_u > D_f$ / $0.42x_u < D_f$



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(a) When $x_u < D_f$,

→ Section will behave as rectangular section having width equal to b_f.

→ Depth of neutral axis, $x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f}$

→ Moment of Resistance (M_u)

a) When $x_u < x_{umax}$, underreinforced section

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

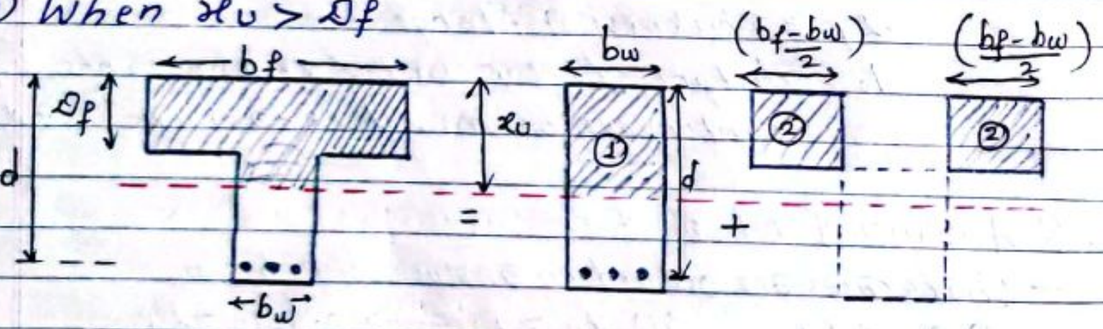
b) When $x_u = x_{umax}$, balanced section

$$M_u = 0.36 f_{ck} b_f x_{umax} (d - 0.42 x_{umax}) \quad \text{or}$$

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

c) When $x_u > x_{umax}$, overreinforced section (redesign)

(b) When $x_u > D_f$



The idealized 'T' beam section consists of

i) A singly reinforced rectangular section (1) of width b_w & effective depth 'd'.

ii) A flanged section (2) of with $(b_f - b_w)$ & depth D_f.

→ IS classifies this case ($x_u > D_f$) into two types

Case 1, $x_u = x_{umax}$ & $x_u > D_f$, balanced section

Case 2, $x_{umax} > x_u$ & $x_u > D_f$, Underreinforced Section

1) When $x_u = x_{umax}$ & $x_u > D_f$, Balanced Section

→ The moment of resistance of the section depend on value of D_f/d ratio. (G 2.2, IS456)

→ The IS456:2000 has given a simplified approach for analysing such cases.

It recommends a single value of $D_f/d = 0.2$

(a) When $D_f/d \leq 0.2$, $x_{umax} = x_u$, $x_u > D_f$ & $D_f < 0.42 x_u$

$$M_u = 0.36 \frac{x_{umax}}{d}\left(1 - 0.42 \frac{x_{umax}}{d}\right) f_{ck} b_w d^2 + 0.45 f_{ck} (b_f - b_w) \times D_f \times (d - D_f/2) \quad \text{---(i)}$$

(b) When $D_f/d > 0.2$, $x_{umax} = x_u$, $x_u > D_f$ & $D_f > 0.42 x_u$

$$M_u = 0.36 \frac{x_{umax}}{d}\left(1 - 0.42 \frac{x_{umax}}{d}\right) f_{ck} b_w d^2 + 0.45 f_{ck} (b_f - b_w) y_f (d - y_f/2) \quad \text{---(ii)}$$

where, $y_f = 0.15 x_u + 0.65 D_f$ but not greater than D_f

2) When $x_{umax} > x_u$ & $x_u > D_f$, Underreinforced section

→ The moment of resistance of section depends on D_f/x_u.

(a) When $D_f/x_u < 0.43$, $x_u > D_f$ & $x_{umax} > x_u$

put x_u in place of x_{umax} in eqn (i)

(b) When $D_f/x_u > 0.43$, $x_u > D_f$ & $x_{umax} > x_u$

Put x_u in place of x_{umax} in eqn (ii)

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Q A T beam floor system has 120mm thick slab supported on beams. The width of beam is 300mm & effective depth is 580mm. The beam is reinforced with 8 bars of 20mm diameter. Use M20 grade of concrete & Fe415 steel. The beams are spaced 3m c/c. The effective span of beam is 3.6m

→ Given,

$b_w = 300\,mm$

$d = 580\,mm$

$D_f = 120\,mm$

$A_{st} = 8 \times \pi/4 \times 20^2 = 2513\,mm^2$

c/c spacing of beam = 3m

Effective span = 3.6m

Effective width of flange,

$$b_f = l_0/6 + b_w + 6D_f$$

For simply supported beam, $l_0 = l_{eff} = 3600\,mm$

$$b_f = \frac{3600}{6} + 300 + 6 \times 120 = 1620\,mm$$

$b_w + \frac{1}{2}$ (clear span on both sides)

$$= 300 + \frac{1}{2}[(3000-300)+(3000-300)]$$

$$= 3000\,mm$$

$b_f <$ ($b_w + \frac{1}{2}$ x clear span on both sides) Ok.

Depth of neutral axis can be calculated as,

Assuming neutral axis fall in flange,

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 \times 415 \times 2513}{0.36 \times 20 \times 1620}$$

$x_u = 77.8\,mm < D_f$. Hence NA lies in the flange.

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For Fe415 steel, $x_{umax} = 0.48d = 0.48 \times 580 = 278.4\,mm$

$x_{umax} > x_u$, section is under reinforced.

So,

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

$$= 0.87 \times 415 \times 2513 \times 580 \left(1 - \frac{2513 \times 415}{1620 \times 580 \times 20}\right)$$

$$= 496.61\,KNm$$

Q A floor consists of 150mm thick slab monolithically constructed with 300mm wide beams spaced at 3.6m c/c spacing. The effective span of beam is 5m. The slab is subjected to a superimposed load of 5 kN/m². Design an intermediate beam for flexure. Use M20 concrete & Fe415 steel.

→ Given,

$b_w = 300\,mm$

$l_{eff} = 5m$

$w_L = 5\,kN/m^2$ $f_{ck} = 20\,N/mm^2$ & $f_y = 415\,N/mm^2$

Effective width of flange,

$$b_f = \frac{l_0}{6} + b_w + 6D_f = \frac{5000}{6} + 300 + 6 \times 150 = 2033.3\,mm$$

Available flange width = 3600 mm > 2033.33 mm (ok)

So, Effective width of flange (b_f) = 2033.3 mm

Assuming depth of beam as, $l/12 = 5000/12 = 400\,mm$

Effective depth, $d = 400 - 30 = 370\,mm$

(assuming effective cover = 30 mm).

Depth of web = 400 − 150 = 250 mm

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→ Load Calculations,

Dead load of slab = $0.15 \times 25 = 3.75\ kN/m^2$

Superimposed load = $5\ kN/m^2$

total load on slab = $3.75 + 5 = 8.75\ kN/m^2$

Load transfer from slab to beam = $8.75 \times 3.6 = 31.5\ kN/m$.

Self weight of web = $0.25 \times 0.3 \times 25 = 1.875\ kN/m$

total load on beam = $31.5 + 1.875 = 33.375\ kN/m$

Factored load = $33.375 \times 1.5 = 50.06\ kN/m$

Factored bending moment = $\frac{wl^2}{8} = \frac{50.06 \times 5^2}{8} = 156.5\ kNm$

→ Assuming NA lies in flange,

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 \times 415 \times A_{st}}{0.36 \times 20 \times 2033.3}$$

$$x_u = 0.0246 A_{st}$$

For an underreinforced section,

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

$$156.5 \times 10^6 = 0.87 \times 415 \times A_{st} \times (370 - 0.42 \times 0.0246 A_{st})$$

$$\Rightarrow A_{st} = 1213\ mm^2$$

Also,

$$x_u = 0.0246 \times 1213 = 30\ mm < D_f (150\ mm)$$

Hence, our assumption that NA lies in flange is valid.

→ Using 20mm ϕ bars,

$$\text{No. of bar} = \frac{1213}{314.16} = 3.86 \simeq 4$$

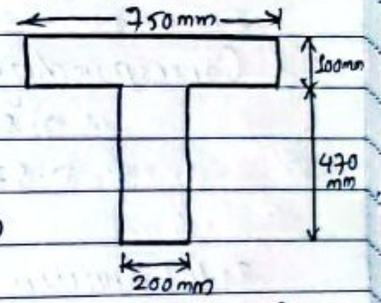
$$A_{st,pro} = 4 \times 314.16 = 1256\ mm^2$$

$$A_{st,min} = \frac{0.85 b d}{f_y} = \frac{0.85 \times 300 \times 370}{415} = 228\ mm^2$$

$$A_{st,max} = \frac{4}{100} \times bD = \frac{4}{100} \times 300 \times 400 = 4800\ mm^2$$

So, provide 20mm ϕ - 4 bars as tension rebar.

Q Calculate the amount of steel required in a T-beam to develop a moment of resistance of 300kNm at working loads. The dimensions of the beam section are given in figure. Use M20 mix & Fe415 steel.



→ Here,

Factored BM = $1.5 \times 300 = 450\ kNm$

$$\frac{D_f}{d} = \frac{100}{500} = 0.2 \quad \& \quad x_{umax} = 0.48 d$$

$$= 0.48 \times 500$$

$$= 240\ mm$$

(Assuming effective cover to tension steel = 70 mm)

Let us assume that section is balanced & NA lies in the web.

$$M_u = 0.138 f_{ck} b_w d^2 + 0.446 f_{ck} (b_f - b_w) \times D_f \times (d - D_f/2)$$

$$= 0.138 \times 20 \times 200 \times 500^2 + 0.446 \times 20 \times 550 \times 100 (500 - 50)$$

$$= 358 \times 10^6\ Nmm < \text{Factored BM}$$

Hence compression steel is required. It means the assumption of depth of neutral axis greater than the depth of flange is correct.

Area of tension steel corresponding to moment equal to 358kNm,

$$358 \times 10^6 = 0.87 \times 415 \times A_{st1} \times 500 \left(1 - \frac{415 \times A_{st1}}{20 \times 200 \times 500}\right)$$

~~A_{st}~~

$$358 \times 10^6 = 0.87 \times f_y \times A_{st1} (d - 0.42 x_{umax})$$

$$358 \times 10^6 = 0.87 \times 415 \times A_{st1} \times (500 - 0.42 \times 240)$$

$$A_{st1} = 2484\ mm^2$$

The remaining moment to be resisted = $450 - 358 = 92\ kNm$

This moment is to be resisted by compression steel A_{sc} & additional tension steel A_{st2}. If effective cover to compression steel is 50 mm,

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$d'/d = 50/500 = 0.1$ & $f_{sc} = 353\ N/mm^2$

Additional Moment = $f_{sc} \times A_{sc} \times (d-d')$

$92 \times 10^6 = 353 \times A_{sc} \times (500-50)$

$A_{sc} = 579\ mm^2$

Corresponding tensile steel is given by,

$0.87 f_y A_{st2} = f_{sc} \times A_{sc}$

$0.87 \times 415 \times A_{st2} = 353 \times 579$

$A_{st2} = 566\ mm^2$

Total tension steel $\Rightarrow A_{st} = A_{st1} + A_{st2}$

$= 2484 + 566$

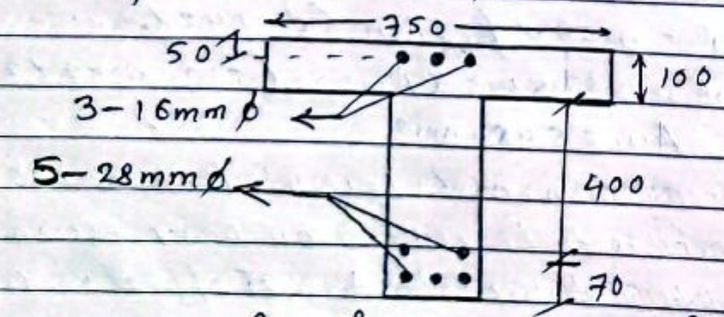
$= 3050\ mm^2$

Provide 5 - 28mm bars in tension giving total area as $3079\ mm^2$ at an effective cover of 70mm & 3-16mm bars in compression giving total area as $603\ mm^2$ at an effective cover of 50mm.

$A_{st,min} = \frac{0.85 b_w d}{f_y} = \frac{0.85 \times 200 \times 500}{415} = 210\ mm^2 < 3079\ mm^2$

$A_{st,max} = \frac{4}{100} \times 200 \times (500+70) = 4560\ mm^2 > 3079\ mm^2$

Max^m compression steel $= 0.04 b_w D = 4560\ mm^2 > (603\ mm^2)$



Reinforcement details of T section.

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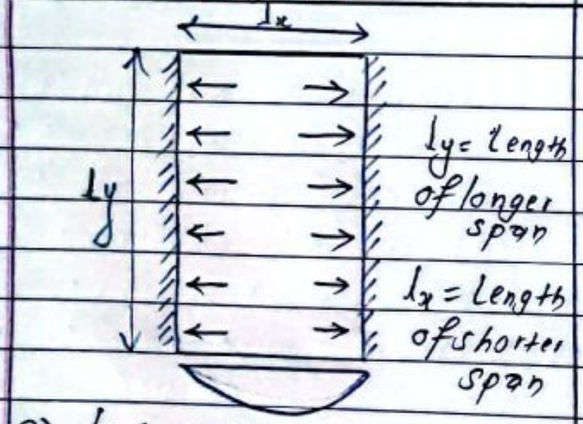
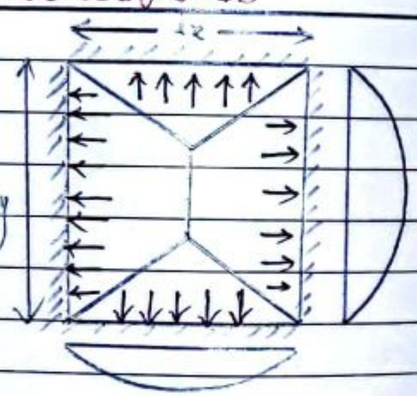
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Slab Design

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→ Slab is two dimensional or planar element, used in all types of structures such as floors & roof coverings.
→ The thickness of slab is very small as compared to its length & width.
→ Slabs are classified into two types
  i) One way slab
  ii) Two way slab

→ Difference betn one way & Two way slab:

One way slab	Two way slab
	
a) $l_y/l_x > 2$	a) $l_y/l_x \leq 2$
b) Bending takes place in one direction (shorter span)	b) Bending takes place in both directions.
c) Depth reqd is more	c) Depth reqd is less
d) Main steel reinforcement is provided along shorter span	d) Main steel reinforcement is provided along both directions
e) Less economical as thickness is more & the amount of steel is also more.	e) More economical as the thickness of slab is less & the amount of steel required is less.
f) The deflected shape of one way slab is cylindrical.	f) The deflected shape of the two way slab is dish or saucer like shape.

g) Crank is provided in two directions	g) Crank is provided in four directions.

✱ IS 456:2000 Recommendations for design of slabs:-

(a) Effective span: (cls 22.2)
→ For simply supported beam/slab
  l_{eff} = Min^m of { clear span + effective depth of slab ; c/c distance betn supports }
→ For continuous beam/slab, watch in code cls 22.2 (b).

(b) Deflection Control (cls 23.2)
→ For span upto 10m, l/d ratio

cantilever	< 7
Simply Supported	< 20
Continuous	< 26

→ For slab having span > 10m, above value may be multiplied by 10/span, except for cantilever, for which exact deflection calculations should be made.
→ For slabs spanning in two directions, the shorter of two spans shall be used for calculating l/d ratio.

(c) Reinforcement in slab
→ Min^m Reinforcement > 0.15% of total C.S.A in mild steel rebars
  > 0.12% of total CSA in high strength deformed bars
→ Max^m dia of rebar = $\frac{\text{To total depth of slab (D)}}{8}$

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(c) Distribution Reinforcement:

→ Distb reinforcement is provided in the longer span of one way slab. This steel is as per the minm reinforcement criteria.

→ The function of distb steel are

1) To distribute the concentrated loads coming on slab.
2) To protect against shrinkage & temp stresses.
3) To keep the main steel bar in position.

→ The distb bar is kept above the main steel & is not provided with hook at the ends.

(d) Spacing of Reinforcement:

→ Minm distance betn bars

1) The minimum horizontal distance betn two parallel main bars shall not be less than
   - Largest diameter of bar
   - 5 mm more than the nominal maximum size of coarse aggregate.
2) The vertical distance betn two layers of main reinforcement shall be more than
   - 15mm or
   - 2/3rd the nominal maxm size of aggregate
   - Maximum size of the bar.

→ Maxm Distance betn the bars in tension

1) The spacing of main steel in slab shouldnot exceed the following
   - 3 times the effective depth of slab
   - 300 mm

2) The spacing of bars provided to act as distribution steel or bars provided for preventing tempr & shrinkage stresses shall not exceed the following.
   i) five times the effective depth of slab
   ii) 450 mm

(e) Cover:-

→ Nominal cover to be provided in slab is 20mm & other values of cover for different environmental conditions are given in Table 16 of IS456:2000.

(f) Bentup Bars:-

→ Some of the main reinforcement in slabs are generally bentup near the supports to takeup negative moment which may develop due to partial fixity.

→ Generally alternate bars are bentup at a distance of 0.15 l (1/7) from centre of supports. The bar available at the upper face should be more than (1/10) (0.1l) from centre of support.

→ The reinforcing detailing of one way slab is shown below.

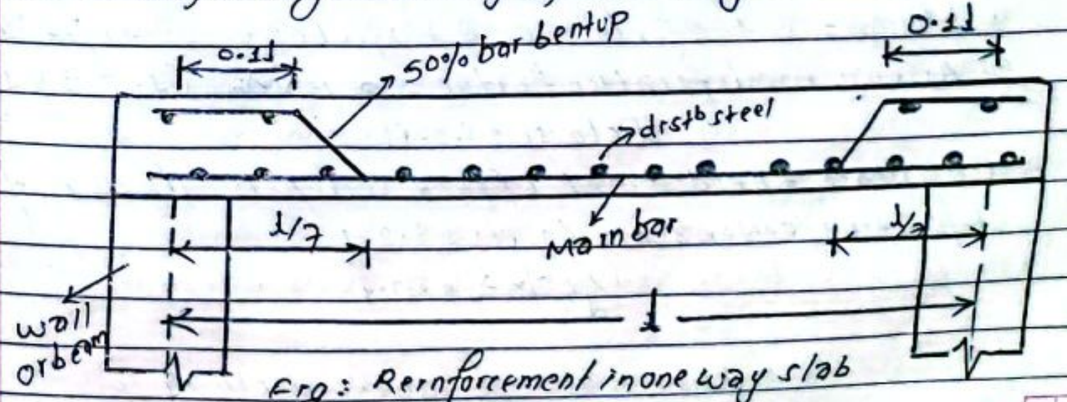


Fig: Reinforcement in one way slab

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g) Curtailment of Bars:-
→ The bars in a slab may be curtailed as per the recommendation given in curtailment of bars in beams. But in practice, the bars are bent & not curtailed in slabs.

h) Shear Design:-
→ slabs are safe in shear (nominal shear stress is very low since b is large) therefore no shear reinforcement is provided in slabs except that the alternate bars are bentup near the supports.

* Loads on slab
→ Self weight of slab
→ Live loads as per use
→ Finishing & partitioned loads.

* Design of one way slab:-
1) Calculate design parameter for given slab with given grade of concrete & steel by considering balanced section.
   i) $x_{u,lim} = 0.53d / 0.48d / 0.46d$ a/c to given f_y
   ii) $M_{u,lim} = 0.133 f_{ck} bd^2$ or $0.138 f_{ck} bd^2$ or $0.149 f_{ck} bd^2$
   iii) Assume modification factor = β_{YNS} (1.2 to 1.4) [clause 23.2.1]
2) Calculate the assumed effective depth 'd assumed' from deflection criteria a/c to cls 23.2.1

$$\frac{l}{d} \neq \alpha * m.f$$

α = Basic values of span to depth ratio

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3) Calculate the total design load by using $W_u = 1.5(DL + LL)$ for ss ~~beam~~ slab & cantilever slab but for continous slab calculate separate factored DL & LL
4) Calculation of Maxm Bending Moment, According to type of slab.
For simply supported, $BM_{max} = \frac{W_u l_{eff}^2}{8}$

For cantilever slab, $BM_{max} = \frac{W_u \times l_{eff}^2}{2}$

For continuous slab use table 12 from IS 456:2000
5) Calculate actual effective depth of slab by using, $M_{u,lim} = M_{max}$ & check $d_{assumed} > d_{calcu}$
6) Calculate area of main reinforcement,

$$M_u = 0.87 f_y A_{st\,assumed} * \left(1 - \frac{f_y}{f_{ck}} \times \frac{A_{st}}{bd}\right)$$

Check for minimum reinforcement (cls 26.5.2.1)
($A_{st} > A_{st,min}$)
Calculate spacing of main bar, $S_v = \frac{b A_\phi}{A_{st}}$, $A_\phi = \frac{\pi \phi^2}{4}$

& check it with cls 26.3.3 of IS 456:2000
7) Area of distribution reinforcement is provided as per minimum reinforcement criteria. [cls 26.5.2.1]

$A_{st,distb} = 0.12\%$ of bD

Spacing (S_v) = $\frac{b \times A_\phi}{A_{st,distb}}$ [Assume $\phi \rightarrow$ 8mm/10mm]

& check it with cls 26.3.3 of IS 456:2000
8) Check for shear,
Shear is to be checked only. No shear reinforcement is provided.

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$V_{umax} = \frac{w_u \times l}{2}$ → clear span

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

For percentage of tensile reinforcement, $p_t = \frac{100 A_{st}}{bd}$, calculate permissible shear stress according to Table 19. (τ_c)

Correct value of τ_c by multiplying it with 'k' which is given in cls 40.2.1.1 according to overall depth of slab

$\tau_{cslab} = \tau_c \times k$

Check, $\tau_{cslab} > \tau_v$, No shear reinforcement is reqd.

9) Check for ~~deflection~~ development length, (26.2.1)

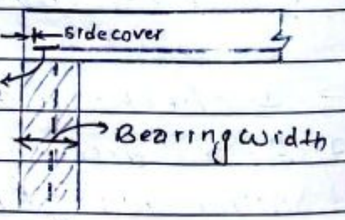
$L_d \leq \frac{1.3 M_1}{V} + L_o$

$L_d = \frac{0.87 f_y \phi}{4 \times (1.6 \tau_{bd})}$ → $\left(\frac{A_{st,pro,main\,bar}}{2}\right)$

$M_1 = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck}\, bd}\right)$

$V = V_u$

$L_o = \frac{\text{Bearing width}}{2} - \text{side cover}.$



↳ Sum of anchorage beyond the centre of support

10) Check for deflection:-

For $p_t = (100 A_{st,pro,main})/bd$, find k_t from fig'4.

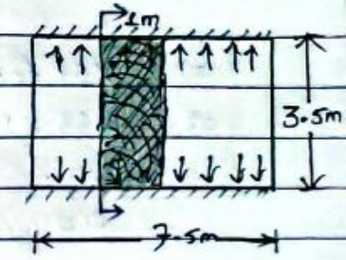
$(l/d)_{max} = 20 * k_t$ (simply supported)

$= 7 * k_t$ (For cantilever)

$= 26 * k_t$ (For continuous)

$(l/d)_{provided} < (l/d)_{max}$

Q. Design a simply supported roof slab for a room 3.5m x 3.5m clear in size. The slab is carrying an imposed load of 5 kN/m². Use M20 mix & Fe415 steel.



→ Given,

Size of room = 3.5m x 3.5m

Live load = 5 kN/m²

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

Here,

$\frac{l_y}{l_x} = \frac{3.5}{3.5} > 2$, hence, it is a one way slab.

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

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

modification factor (m.f) = $\beta \times \gamma \times \lambda \times \delta$ = 1.3 (assumed) (1 · 1.3 · 1 · 1)

Now,

$(l/d) \leq \alpha \times m.f$

Assume bearing width = 250mm, $l_{eff} = 3.5 + 0.25 = 3.75m$

$\frac{3750}{d} \leq 20 \times 1.3$

$d \geq 144.23\ mm$

$d_{assumed} = 150mm$

→ Total design factored load,

$w_u = 1.5(DL + LL)$ (Assuming effective cover = 20mm (with 10mm ϕ & c.c = 15mm))

$= 1.5[25 \times 1 \times (0.15 + 0.2) + 5 \times 1 + 24 \times 1 \times 0.03]$

$= 14.955\ kN/m$

$M_{umax} = \frac{w_u \times l_{eff}^2}{8} = \frac{14.955 \times 3.65^2}{8} = 27.9\ kNm$

l_{eff} = Minm of [c/c distance betn support = 3.75m; clear span + depth = 3.5 + 0.15 = 3.65m]

$l_{eff} = 3.65m$

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→ Calculate actual effective depth of slab,

$M_{u,lim} = M_{u,max}$

$\Rightarrow 0.138 f_{ck} b d^2 = 27.9 \times 10^6$

$\Rightarrow 0.138 \times 20 \times 1000 \times d^2 = 27.9 \times 10^6$

$d = 100.54 \text{ mm}$

$d_{assume}\ (150\text{mm}) > d_{calc}\ (100.54\text{mm})$ ∴ $d_{adopt} = 150\text{mm}$, $D = 170\text{mm}$

→ Calculate area of main bar,

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

$\Rightarrow 27.9 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \left(1 - \frac{415}{20} \times \frac{A_{st}}{1000 \times 150}\right)$

$\Rightarrow A_{st} = 558.279 \text{ mm}^2$

$A_{st,min} = 0.12\%$ of bD [cls 26.5.2.1]

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

$= 204 \text{ mm}^2 > A_{st}$ (ok)

Spacing of main bar, $S_v = \frac{b \times A_\phi}{A_{st,req}} = \frac{1000 \times \pi \times 10^2}{4 \times 558.279}$

[Assuming 10mm ϕ ≠ main bar]

$S_v = 140.682 \text{ mm}$

Check, (cls 26.3.3)

$S_v \neq 3d$ or 300 mm whichever is less

$S_v \neq 3 \times 150$ or 300mm. So, $S_v \neq 300\text{mm}$

provide 10mm ϕ @ 130mm c/c

Now,

$A_{st,pro,main} = \frac{b \times A_\phi}{S_v} = \frac{1000 \times \pi \times 10^2}{4 \times 130}$

$= 604.15 \text{ mm}^2$

→ Calculation of area of dist^b bar,

$A_{st,dist^b} = 0.12\%$ of bD

$= 204 \text{ mm}^2$

Let us provide 8mm ϕ distribution bar,

Spacing of dist^b bar ⇒ $S_v = \frac{b A_\phi}{A_{st,dist}} = \frac{1000 \times \pi \times 8^2}{4 \times 204}$

$\Rightarrow S_v = 246.39 \text{ mm}$

$\Rightarrow S_v \simeq 240 \text{ mm}$

For dist^b bar,

$S_v \neq 5d$ or 450mm whichever is less (26.3.3)

$\neq 5 \times 150$ or 450mm ⇒ 450mm

∴ Provide 8mm ϕ dist^b bar @ 240 mm c/c

$A_{st,dist^b,pro} = \frac{b \times A_\phi}{S_v} = \frac{1000 \times \pi \times 8^2}{4 \times 240} = 209.44 \text{ mm}^2$

→ Check for shear,

$V_{u,max} = \frac{w_u \times l}{2} = \frac{14.955 \times 3.5}{2} = 26.17 \text{ kN}$

$\tau_v = \frac{V_{u,max}}{bd} = \frac{26.17 \times 10^3}{1000 \times 150} = 0.174 \text{ N/mm}^2$

$p_t = \frac{100 \times A_{st}}{bd} = \frac{100 \times (604.15/2)}{1000 \times 150} = 0.2\%$ → A_{st} provided for shear at support

For $p_t = 0.2\%$	0.15	0.28
$\tau_c = 0.28 + \frac{0.36 - 0.28}{0.25 - 0.15} \times (0.2 - 0.15)$	0.2	τ_c
	0.25	0.36

$= 0.32 \text{ N/mm}^2$

$\tau_{c,slab} = k * \tau_c = 1.24 \times 0.32$ [For overall depth of 170 mm]

$= 0.39 \text{ N/mm}^2$

Here, $\tau_v < \tau_{c,slab}$. So, No need to design shear reinforcement.

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→ Check for ~~shear~~ anchorage development length

$$L_d \not> \frac{1.3 M_1}{V} + L_o$$

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 10}{4 \times (1.6 \times 1.2)} = 470.117 \text{ mm}$$

$$M_1 = 0.87 f_y A_{st} d \left(1 - \frac{f_y \times A_{st}}{f_{ck}\, bd}\right)$$

$$= 0.87 \times 415 \times \left(\frac{604.15}{2}\right) \times 150 \left(1 - \frac{415 \times (604.15/2)}{20 \times 1000 \times 150}\right)$$

$$= 15.67 \text{ kNm}$$

$V = 26.17 \text{ kN}$

$$L_o = \frac{\text{bearing width}}{2} - \text{side cover} = \frac{250}{2} - 15 = 110 \text{ mm}$$

$$\frac{1.3 M_1}{V} + L_o = \frac{1.3 \times 15.67 \times 1000}{26.17} + 110 = 888.41 \text{ mm}$$

$$L_d \not> \frac{1.3 M_1}{V} + L_o \quad (\text{OK})$$

→ Check for deflection:

$$p_t = \frac{100 A_{st}}{bd} = \frac{100 \times 604.15}{1000 \times 150} = 0.403\%$$

$$f_s = 0.58 \times f_y \left[\frac{A_{st,req}}{A_{st,prov}}\right] = 0.58 \times 415 \times \left[\frac{558.279}{604.15}\right] = 222.42 \text{ N/mm}^2$$

1.32	240	
$k_t = ?$	222.42	$k_t = 1.32 + \frac{1.67 - 1.32}{190 - 240}(222.42 - 240)$
1.67	190	

$$= 1.443$$

$$\left(\frac{l}{d}\right)_{max} = 1.443 \times 20 = 28.86$$

$$\left(\frac{l}{d}\right)_{pro} = \frac{3650}{150} = 24.33 < \left(\frac{l}{d}\right)_{max}$$

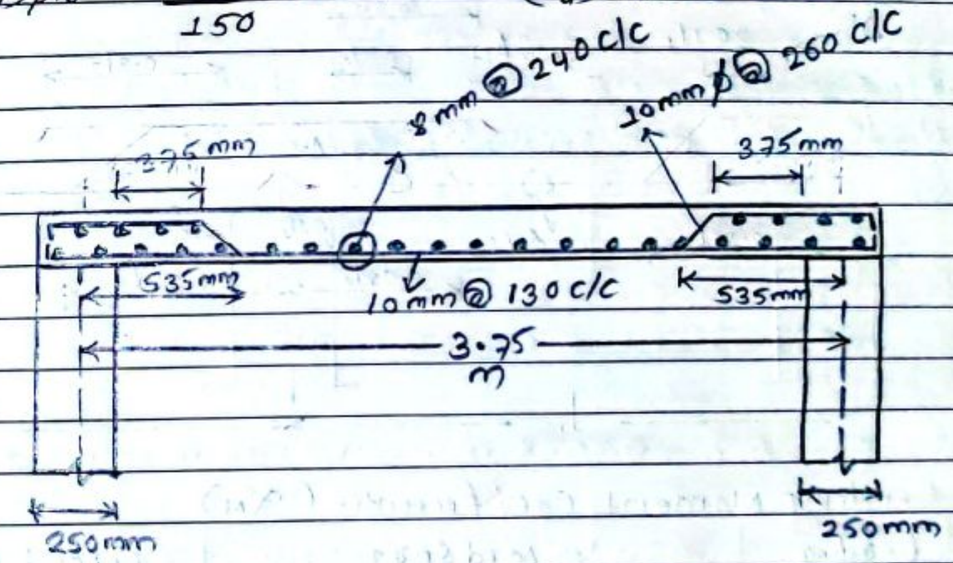


Fig: Reinforcement Details in one way slab

✱ Design of Continuous Slabs:-

→ The beams & slabs are cast monolithic in reinforced concrete construction. The 'T' beams are spaced at regular intervals & the slab is continuous over these supports.

→ The design of such one way slab can be done by moment & shear coefficients methods given in IS 456:2000.

→ These coefficients give the moment & shear force values at critical sections. These coefficients are applicable only in

a) The slab is continuous over three or more spans which donot differ by more than 15% of the longest spans & loads are mainly uniformly distributed.

b) It is to be noted that redistribution of moments is not permitted when moments are determined using moment coefficients.

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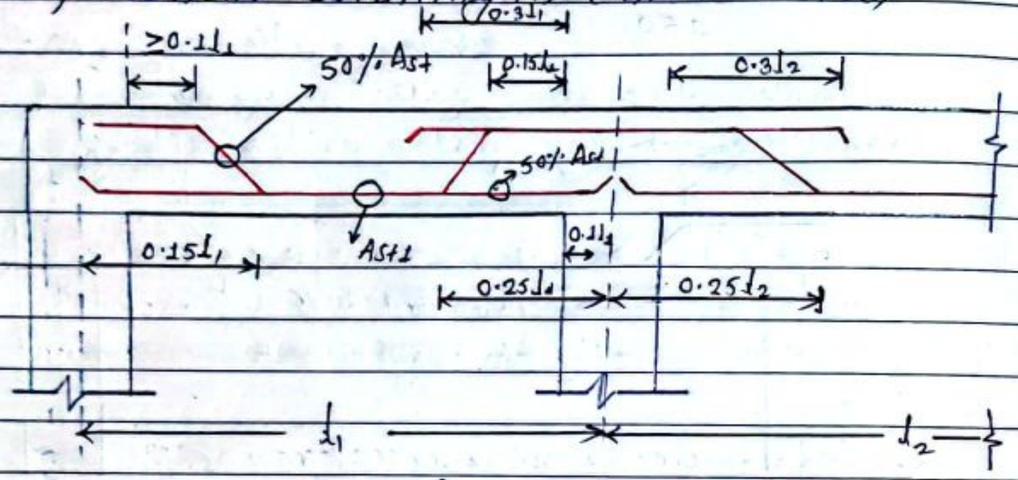
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→ Reinforcement Detailing in continuous slab,



→ Bending Moment Coefficients. (α_M)

Loading	Mid Span		Support Moments	
	End Span	Interior Span	At support next to end support	Interior Support
DL+LL(fix)	+1/12	+1/16	-1/10	-1/12
LL(not fixed)	+1/10	+1/12	-1/9	-1/9

$BM = \alpha_M \times w_u \times l_{eff}^2$

→ Shear force coefficients (α_F)

Loading	End Support	Support end to next support		Other Interior supports
		Outer face	Inner face	
DL+LL(fixed)	0.4	0.6	0.55	0.5
LL (not fixed)	0.45	0.6	0.6	0.6

$SF = \alpha_F \times w_u$

→ When a member is built into a masonry wall which doesn't provide fully fixity but develops only partial restraint, the negative moment at the face of support is taken as Wl/24 In this case, the SF coefficient for end support may be increased by 0.05.

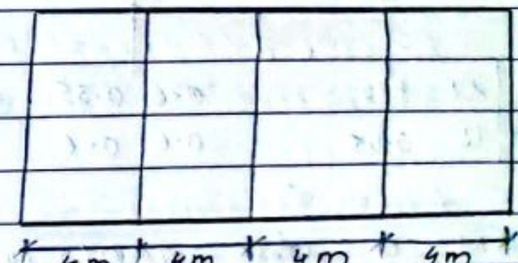
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Q Design a reinforced concrete slab for a hall measuring 8m * 16m. The slab is supported on RCC beams 250mm wide & spaced at 4m c/c. The supersimposed load is 4 kN/m². Use M20 concrete & Fe415 steel. Bearing of beams is 200mm.

→ Given,

Size of hall = 8m * 16m

$\frac{l_y}{l_x} = \frac{8}{4} = 2 \geq 2$



(One way continuous slab)

Here,

$R_{ulrm} = 0.48d$ for Fe415

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

modification factor (mf) = $\beta \times \gamma \times \lambda \times \delta$ (1, 1.3, 1, 1) = 1.3 (assumed)

Now,

$(l/d) \neq \alpha \times m.f$

$\left(\frac{4000}{d}\right) \neq 26 \times 1.3$

$d \neq 118.34$

$\therefore d_{assumed} = 120$ mm

$D = 120 + 15 + 10/2 = 140$ mm

→ Load calculation:

Self weight of slab = $0.14 \times 25 \times 1 = 3.5$ kN/m

Finishing load = $1 \times 1 = 1$ kN/m (assumed)

Total dead load = 4.5 kN/m

Live load = $4 \times 1 = 4$ kN/m

Factored dead load = $1.5 \times 4.5 = 6.75$ kN/m

Factored Liveload = $1.5 \times 4 = 6$ kN/m

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→ Bending Moment & SF coefficients.

BM	DL	0	+1/12	-1/10	+1/16	-1/12	+1/16	-1/10	+1/12	0
	LL	0	+1/10	-1/9	+1/12	-1/9	+1/12	-1/9	+1/10	0
SF	DL	0.4		0.6	0.55	0.5	0.5	0.55	0.6	0.4
	LL	0.45		0.6	0.6	0.6	0.6	0.6	0.6	0.45

BM (KNm)	0	17.86	-20.61	14.16	-18.88	14.16	-20.61	17.86	0
SF (kN)	21.16		29.98	28.66	27.34	27.34	28.66	29.98	21.16

Leff ⇒ [22.2(b)]

Width of support = 200 mm

Clear span = 4000 − 200 = 3800 mm

1/12 of clear span = 316.67 mm

width of support < 1/12th of clear span

l_{eff} = 3800 + 120 = 3920 mm

→ Maxm moment occurs at support next to end support = −20.61 kNm

Maxm shear force occurs at outer side of support next to end support = 29.98 kN

→ Calculate actual effective depth of slab

$M_{u lim} = M_{max}$

$\Rightarrow 0.138 f_{ck} b d^2 = 20.61 \times 10^6$

$\Rightarrow 0.138 \times 20 \times 1000 \times d^2 = 20.61 \times 10^6$

d = 86.41 mm < $d_{assumed}$.

So, d_{adopt} = 120 mm ∅

D = 140 mm.

Area of steel reinforcement,

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

$20.61 \times 10^6 = 0.87 \times 415 \times A_{st} \times 120 \left[1 - \frac{415}{20} \times \frac{A_{st}}{1000 \times 120}\right]$

$\Rightarrow A_{st} = 523\ mm^2$

$A_{st,min} = \frac{0.12}{100} \times 1000 \times 140 = 168\ mm^2 > A_{st}$ ok.

→ positive moment reinforcement req'd for M_u = 17.86 kNm,

$17.86 \times 10^6 = 0.87 \times 415 \times A_{st} \times 120 \left[1 - \frac{415}{20} \times \frac{A_{st}}{1000 \times 120}\right]$

$A_{st} = 446.73\ mm^2$

→ Distribution steel

$A_{st} = \frac{0.12}{100} \times 1000 \times 140 = 168\ mm^2$

Using 8mm dia bars, spacing req'd = $\frac{1000 \times \pi \times 8^2}{4 \times 168} = 299.19$ mm

Hence, provide 8mm ∅ dist⁶ bars @ 290 mm c/c.

→ A_{st} req at midspan = 446.73 mm²,

Using 10mm dia bars @, spacing = $\frac{1000 \times \pi \times 10^2}{4 \times 446.73} = 175.81$ mm

Hence, providing 10 mm dia bars @ 170 mm c/c at midspan bending half of the bars.

Now,

Area of steel available at supports

= 10mm @ 340 mm c/c + 10mm @ 340 c/c

= $\frac{1000}{340} \times \frac{\pi \times 10^2}{4} \times 2 = 461.99\ mm^2$

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Extra steel reqd at support for negative BM
= 523 - 461.99
= 61.01 mm²

Spacing reqd = $\frac{1000 \times \pi \times 8^2}{4 \times 61.01}$ = 823.88 mm

Provide additional 8 mm diameter bars @ 800 mm c/c at supports.

→ Check for shear,

V_u = 29.98 kN

$\tau_v = \frac{V_u}{bd} = \frac{29.98 \times 1000}{1000 \times 120}$ = 0.25 N/mm²

$p_t = \frac{100 \times 523}{1000 \times 120}$ = 0.43 %

For p_t = 0.43 %,	0.25	0.36
τ_c = 0.44 N/mm²	0.43	?
For slab $\tau_{slab} = k_s \times \tau_c$	0.5	0.48

For overall depth of 140 mm,

k_s = 1.3 (cls 40.2.1.1)

τ_{slab} = 1.3 × 0.44 = 0.57 N/mm²

τ_{cmax} = 0.5 × 2.8 (cls 40.2.3.1)
= 1.4 N/mm² > τ_v

$\tau_v < \tau_c$ [OK]

Hence, shear reinforcement is not required

→ Check for deflection,

$(l/d)_{max} = k_t * 26$

$f_s = 0.58 \times f_y \times \frac{Ast_{req}}{Ast_{pro}}$

$Ast_{pro} = \frac{1000 \times \pi \times 10^2}{4 \times 170} + \frac{1000 \times \pi \times 8^2}{4 \times 800} = 524.83 mm^2$

f_s = 239.86 N/mm²

For p_t = 0.43 % & f_s = 239.86 N/mm², k_t = 1.35 [Fig 4]

$(l/d)_{max} = 1.35 \times 26 = 35.1$

$(l/d)_{pro} = \frac{3920}{120} = 32.6$

$(l/d)_{max} > (l/d)_{pro}$; Hence ok.

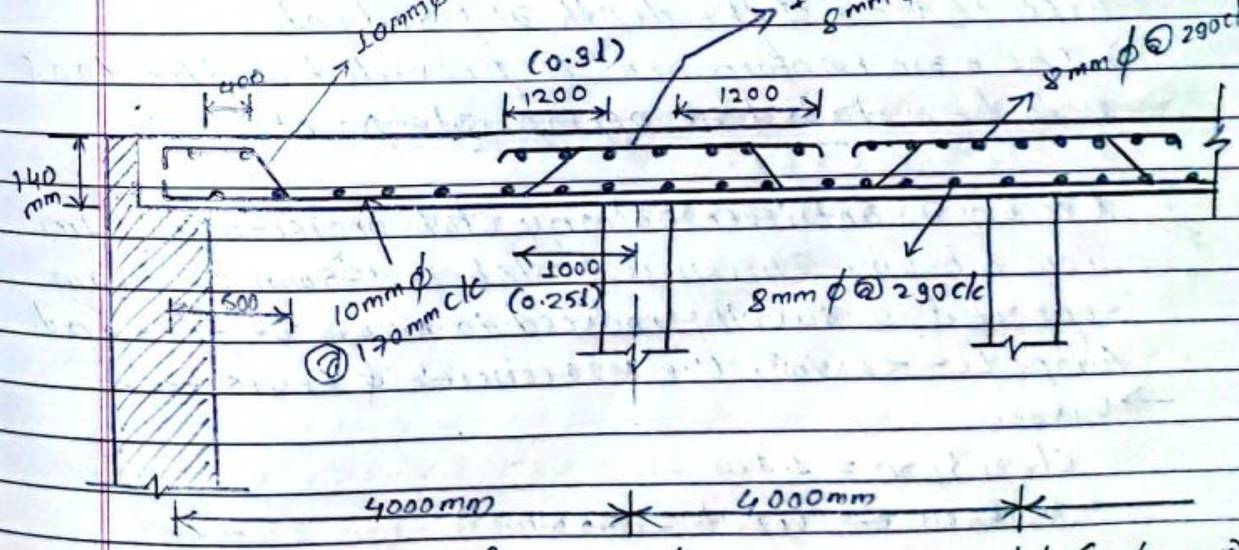


Fig: Reinforcement details in one way slab (continuous)

※ Design of Cantilever Slab:-

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→ One way cantilever slab or chajja is designed as a cantilever beam of one meter width.

→ The points to be considered in design of one way cantilever slab are,

1) The effective span of cantilever slab shall be taken as its length up to the face of the support plus half the effective depth except where it forms the part of continuous slab where length from c/c of supports is taken.

2) The effective depth at fixed end is maximum & is assumed to be about span/10 to span/12 (from deflection consideration)

3) The depth required at free end is minimum & is kept 1/2 to 1/3 of the depth at fixed end.

4) The main reinforcement is provided at the top & is to be curtailed at appropriate point.

Q Design a cantilever balcony slab projecting 1.1m from a beam. Thickness of slab is 175mm at beam support & gradually reduced to 100mm at free end. Adopt LL = 3 kN/m². Use M20 concrete & Fe415 steel.

→ Given,

Clear Span = 1.1 m

Thickness at support = 175 mm

Thickness at free end = 100 mm

LL = 3 kN/m²

f_{ck} = 20 N/mm²

f_y = 415 N/mm²

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Modification factor = $\beta \times \gamma \times \lambda \times \delta$ = 1 × 1.3 × 1 × 1 = 1.3 (assumed)

$\left(\frac{l}{d}\right) \neq 7 \times m.f$

$\Rightarrow \frac{1100}{d} \neq 7 \times 1.3$

$\Rightarrow d \neq 120.87$ mm

Adopt d = 150 mm,

Assuming effective cover of 25 mm, D = 175 mm

Now,

Self weight of slab = $\left(\frac{0.175 + 0.1}{2}\right) \times 1.1 \times 1 \times 25$

= 3.8 kN acting at $\frac{1.1}{3}$ m from support.

Moment due to dead load at support = $\frac{3.8 \times 1.1}{3}$ = 1.4 kNm

Moment due to live load = $\frac{w_u \times l^2}{2} = \frac{3 \times 1.1^2}{2}$ = 1.82 kNm

Total moment = 1.4 + 1.82 = 3.22 kNm

Factored Moment (Mu) = 1.5 × 3.22 = 4.83 kNm

Shear force at support = 3.8 + 3 × 1.1 = 7.1 kN

Factored shear force (Vu) = 1.5 × 7.1 = 10.65 kN

→ Area of steel (Ast)

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

$4.83 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \times \left[1 - \frac{415 \times A_{st}}{20 \times 1000 \times 150}\right]$

A_{st} = 91 mm²

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This Ast should be more than minimum area of reinforcement (0.12% of A)

$\frac{0.12}{100} \times 1000 \times 175 = 210\ mm^2 > 91\ mm^2$

So, $A_{st} = 210\ mm^2$

Using 8 mm ϕ diameter bars, $A_\phi = 50.3\ mm^2$

Spacing of 8 mm bar = $\frac{1000 \times 50.3}{210} = 239\ mm \approx 220\ mm$

Max^m Spacing is limited to

i) $3d = 3 \times 150 = 450\ mm$

ii) 300 mm.

Hence, providing 8 mm diameter bars @ 220 mm c/c as main steel & also as distribution steel in other direction.

→ Check for shear:-

$V_u = 10.65\ kN$

$A_{st,pro} = \frac{1000 \times 50.3}{220} = 228\ mm^2$

$\tau_v = \frac{V_u}{bd} = \frac{10.65 \times 10^3}{1000 \times 150} = 0.071\ N/mm^2$

$p_t = \frac{100 A_{st}}{bd} = \frac{100 \times 228}{1000 \times 150} = 0.15\%$

For $p_t = 0.15\%$ & M20 concrete from IS code

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

Here, $\tau_c >> \tau_v$

No need to design shear rebar.

→ Check for development length:-

The reinforcement must extend at least L_d into support.

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

For M20 concrete & Fe415,

$\tau_{bd} = 1.92\ N/mm^2$

$L_d = \frac{0.87 \times 415 \times 8}{4 \times 1.92} = 376\ mm$ say $400\ mm$

→ Check for deflection:-

As per code, $\left(\frac{l}{d}\right)_{max} = 7 k_t$

From code, for $p_t = 0.15\%$ & $f_s = 0.58 \times 415 \left(\frac{210}{228}\right)$

$k_t = 2$

Then, $\left(\frac{l}{d}\right)_{max} = 2 \times 7 = 14$

$\left(\frac{l}{d}\right)_{pro} = \frac{1100}{150} = 7.33 < 14$ (ok)

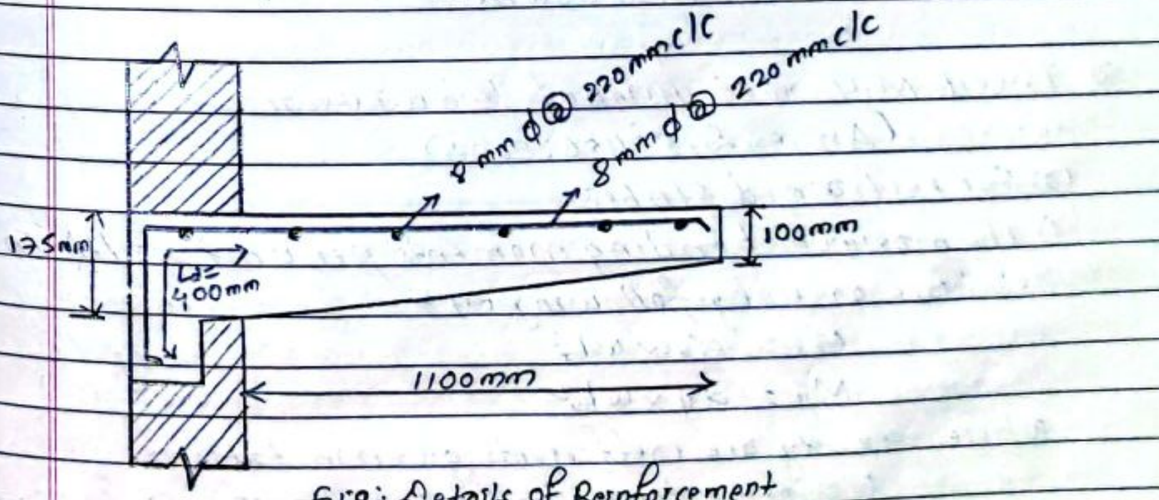


Fig: Details of Reinforcement

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✻ Two Way Slab:

→ Two way slabs are those slabs which are supported on all the four edges & having $l_y/l_x \leq 2$. In these type of slabs bending occurs in both the directions.

→ There are two kinds of two way slabs.

(a) Restrained slabs:

→ When a two way slab is loaded, the corners get lifted up. These corners can be prevented from lifting by providing fixidity at the supports by beams or walls. Such type of slabs in which the corners are prevented from lifting are called as restrained slabs.

→ In these slabs special torsion reinforcement is provided at the edges to prevent cracking of the corners. These are also called as slabs with corners held down.

(b) Unrestrained slabs:

→ The slabs in which corners are not prevented from lifting are called as unrestrained slabs or slabs with corners not held down.

✻ IS Code Method of design of two way slab

(Annex D, IS 456: 2000)

(a) For restrained slabs:

1) The maximum bending moments per unit width in a slab are given by following eqns

$$M_x = \alpha_x \times w l_x^2$$

$$M_y = \alpha_y \times w l_x^2$$

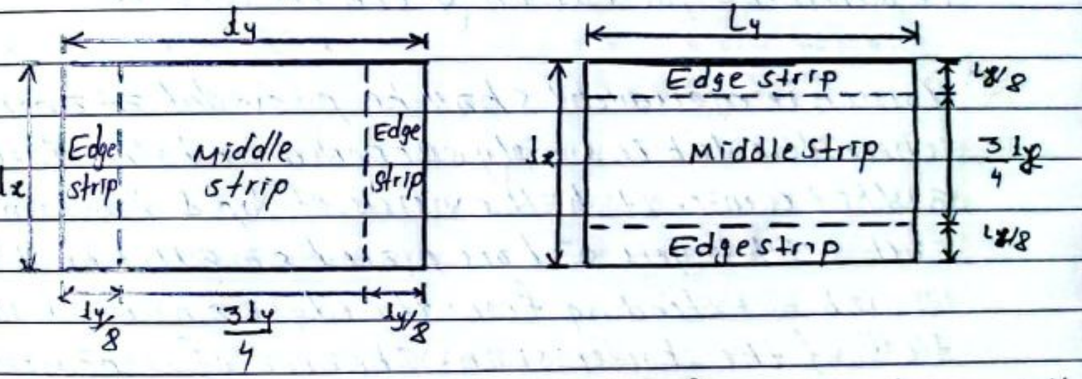
where, α_x, α_y are coefficients given in table 26.

w = total design load per unit area.

M_x, M_y = Moments on strips of unit width spanning l_x & l_y resp.

l_x, l_y = length of shorter span & longer span resp.

2) Slabs are considered as divided in each direction into middle strips & edge strips as shown in fig 25, the middle strip being three quarters of the width & each edge strip being ~~three quarters~~ of one eigth of the width.



3) The maximum moments calculated in 1) apply only to middle strips & no redistribution shall be made.

4) Tension reinforcement provided at midspan in the middle strip shall extend in the lower part of slab to within $0.25l$ of a continuous edge or $0.15l$ of a discontinuous edge.

5) Over the continuous edges of a middle strip, the tension reinforcement shall extend in the upper part of slab a distance of $0.15l$ from the support, & at least 50% shall extend a distance of $0.3l$.

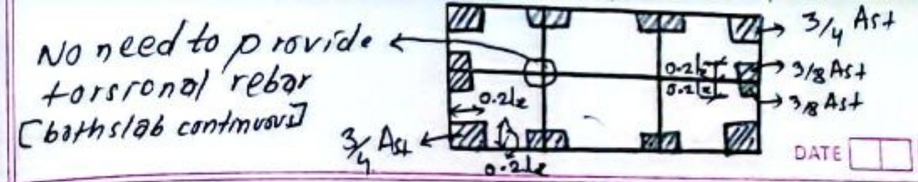
6) At a discontinuous edge, negative moments may arise

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they depend on the degree of fixity at the edge of slab but, in general, tensile reinforcement equal to 50% of that provided at midspan extending 0.1l into the span will be sufficient.

7) Reinforcement in edge strip, parallel to that edge, shall comply with the minimum given in section 3 & the requirements for torsion given in (8) to (10)

8) Torsion reinforcement shall be provided at any corner where the slab is simply supported on both edges meeting at that corner. It shall consists of top & bottom reinforcement, each with layers of bars placed parallel to the sides of slab & extending from the edges a minm distance of 1/5th of the shorter span. The area of reinforcement in each of these four layers shall be three quarters of the area required for the maximum mid span moment in the slab.

9) Torsion reinforcement equal to half that described in (8) shall be provided at a corner contained by edges over only one of which the slab is continuous.

10) Torsion reinforcements need not be provided at any corner contained by edges over both of which the slab is continuous.

11) ~~Torsion~~ where $l_y/l_x > 2$, the slabs shall be designed as spanning one way.

(b) For Simply Supported slabs:-

1) When simply supported slabs donot have adequate provision to resist torsion at corners & to prevent the corners from lifting, the maxm moments per unit width are given by following eqn.

$$M_x = \alpha_x w l_x^2$$

$$M_y = \alpha_y w l_x^2$$

α_x, α_y → Moment coeffrcients in table 27.

2) At least 50% of the tension reinforcement provided at mid span should extend to the supports. The remaining 50% should extend to within 0.1l_x or 0.1l_y of the support, as appropriate.

* Note:- For a two way slab, the depth is assumed on the basis of span/depth ratios based on deflection control. (taking shorter span into consideration)

* Shear check in two way slab:-

Maxm shear force in two way slab: $V_u = w_u \times \frac{l_x}{2}$

The nominal shear stress acting on slab is taken as

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

d → effective depth along longer span

Note:- The effective depth in longer direction is less because the reinforcement along longer span is provided above the reinforcement provided along shorter span.

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Q. Design a reinforced concrete slab for a room of clear dimensions 4m x 5m. The slab is supported on a walls of width 300mm. The slab is carrying a live load of 4kN/m² & floor finish 1kN/m². Use M20 concrete & Fe415 steel. The corners of slab are held down.

→ Given,

Clear dimensions of slab = 4m x 5m

width of wall = 300mm

Live load = 4kN/m, floor finish = 1kN/m²

f_{ck} = 20N/mm², f_y = 415 N/mm²

Corners held down ⇒ (Restrained slab)

Here, $\frac{l_y}{l_x} = \frac{5}{4} = 1.25 < 2$, hence slab is to be designed as two way slab.

For Fe415 steel,

$x_{u lim} = 0.48d$

$M_{u lim} = 0.138 f_{ck} bd^2$

→ Calculation of effective depth of slab

$L/d < \alpha \times m.f$

$d > \frac{4000}{20 \times 1.3} \Rightarrow d > 153.86$

Adopt D = 180mm

$d = 180 - 15 - 8/2 = 161$ mm

→ Effective span,

Effective span in x direction

clear span + d or clear span + support width

(4000 + 161 or 4000 + 300) mm

↳ l_x = 4.161m

So, l_y = 5.161m

→ Design load

self wt of slab = 25 x (0.18 x 1) = 4.5 kN/m

FF = 1 x 1 = 1 kN/m

Live load = 4 x 1 = 4 kN/m

Total load (w) = 9.5 kN/m

Factored load w_u = 9.5 x 1.5 = 14.25 kN/m

→ Since, the slab is supported on all four sides & its corners are held down. It corresponds to case '9' by Table 26.

→ Design Moment & shear,

$M_{ux} = \alpha_x \times w_u \times l_x^2$ & $M_{uy} = \alpha_y \times w_u \times l_x^2$

Here,

$\frac{l_y}{l_x} = \frac{5.161}{4.161} = 1.24$

$\alpha_y = 0.056$ & 1.2 — 0.072; 1.24 — α_x=? ; 1.3 — 0.079 ⇒ α_x = 0.0748

$M_{ux} = 0.0748 \times 14.25 \times 4.161^2 = 18.5$ kNm

$M_{uy} = 0.056 \times 14.25 \times 4.161^2 = 13.81$ kNm

&

Shear force $(V_u) = \frac{w_u \times l_x}{2} = \frac{14.25 \times 4.161}{2} = 29.65$ kN

→ Design of main reinforcement.

(a) Along shorter span (x-direction) (middle strip)

width of middle strip = 3/4 x l_y = 3/4 x 5.161 = 3.9m

d = 161 mm

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

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$18.5\times10^6 = 0.87\times415\times A_{st}\times161\left(1-\frac{415\times A_{st}}{20\times1000\times161}\right)$

$A_{st} = 332.6\,mm^2$

Using 8mm ϕ bars, Spacing $= \frac{1000A_\phi}{A_{st}} = \frac{1000\times50.26}{332.6} = 151.1mm$

Adopt, Spacing = 150mm ($<3d$ or 300) Ok.

$A_{st,min} = 0.12\%\times bD = \frac{0.12}{100}\times1000\times180 = 216\,mm^2$

$A_{st,pro} = \frac{1000\times50.26}{150} = 335\,mm^2$

provide 8 mm ϕ @ 150 mm c/c in middle strip of width 3.9m

(b) Along longer Span in y-direction (middle strip)

width of strip $= 3/4\,l_x = 3/4\times4.161 = 3.12m$

Effective depth (d) $= 161-4-4 = 153mm$

Now,

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

$\Rightarrow 13.81\times10^6 = 0.87\times415\times A_{st}\times153\left(1-\frac{415\times A_{st}}{20\times1000\times153}\right)$

$\Rightarrow A_{st} = 260\,mm^2 > A_{st,min}\,(216\,mm^2)$

Spacing $= \frac{1000\times50.26}{260} = 193.3\,mm$ ($<3d$ or 300mm)

So, provide 8mm ϕ bars @ 190 mm/cc in middle strip of width 3.12m.

→ Reinforcement in edge strip,

$A_{st,min} = 216\,mm^2$

Using 8mm ϕ bars,

Spacing $= \frac{1000\times50.26}{216} = 232mm$ (less than 5d or 450mm)

Width of strip in x direction $= \frac{1}{2}(5.161-3.9) = 0.63m$ ($l_y/8$)

width of strip in y direction $= l_x/8 = 4.161/8 = 0.52m$

provide 8mm ϕ bars @ 230 mm c/c in edge strip

→ Check for shear,

Nominal shear stress $(\tau_v) = \frac{V_u}{bd} = \frac{29.65\times10^3}{1000\times153} = 0.194\,N/mm^2$

$p_t = \frac{100A_{st}}{bd} = \frac{100\times395}{1000\times161} = 0.208\%$

$\tau_c = 0.326\,N/mm^2$

For 180mm thick slab,

$k = 1.2 + \frac{1.25-1.2}{175-200}(180-200) = 1.24$

0.15	0.28
0.208	τ_c
0.25	0.36

200	1.2
180	k
175	1.25

Then,

$\tau_c = 1.24\times0.326 = 0.404\,N/mm^2$

Hence, $\tau_v < \tau_c$, so, not necessary to design shear rebar.

→ Check for deflection,

$p_t = 0.208\%$,

$f_s = 0.58f_y\left[\frac{A_{st,req}}{A_{st,pro}}\right] = 0.58\times415\left(\frac{332.6}{335}\right) = 238.97\,N/mm^2$

For $p_t = 0.208\%$ & $f_s = 238.97mm^2$, $K_t = 1.61$,

$(l/d)_{max} = 20\times1.61 = 32.2$

$(l/d)_{pro} = 4161/161 = 26 < (l/d)_{max}$ (Ok)

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→ Torsional Reinforcement at Corners:-

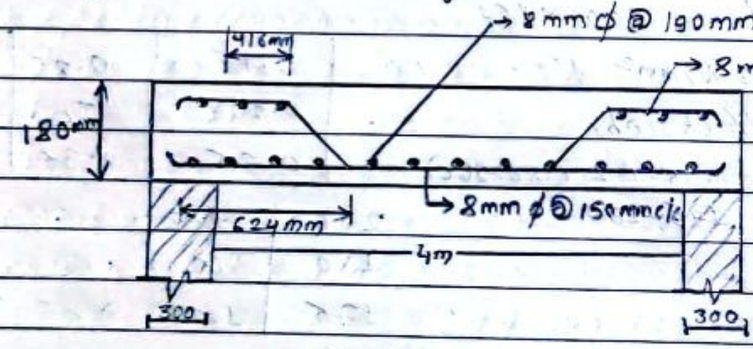
Mesh size = $l_x/5 = 4.161/5 = 0.832$ mm say 840 mm

Area of torsional reinforcement = $3/4 \times 335 = 251\ mm^2$

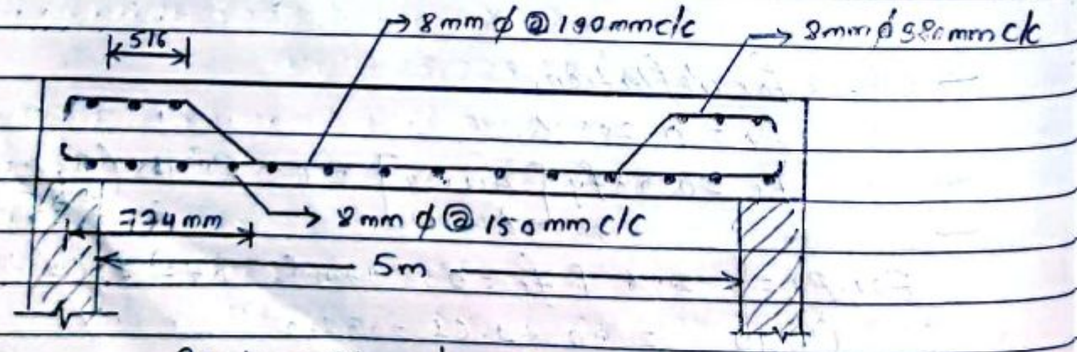
Using 8mm ϕ bars, spacing = $\frac{1000 \times 50.26}{251} = 200.23$ mm ≈ 200 mm

Provide 8 mm ϕ torsional rebar @ 200 mm c/c.

→ Reinforcement detailing:-



Sectron Along short span



Sectron along long span.

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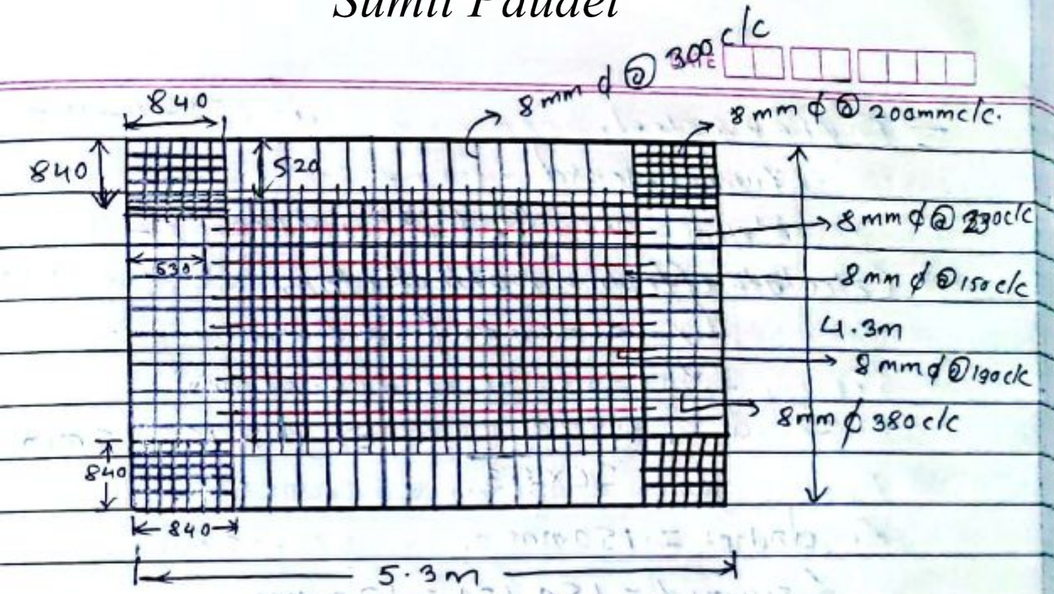


Fig: plan of bottom reinforcement.

Q. Design the panel number #1 as indicated in the typical floor as shown in figure. The clear dimension of panel is 4.5m x 5.5m & is to be rest on 250mm wide beam. It is subjected to live load of 4 KN/m² & consists of 25mm thick PCC floor finish on the slab. Adopt concrete M25 & steel Fe415. Check in shear & deflection is necessary. Show the details of reinforcement in slab after design.

→ Given,

Clear size of slab = 4.5m x 5.5m

Width of beam = 250mm

Live load = 4 KN/m²

FF = 25mm thick

$f_y = 415\ N/mm^2$

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



Here, $\frac{l_y}{l_x} = \frac{5.5}{4.5} < 2$, so, given slab is two way continuous slab.

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→ For Fe 415 steel,

$x_{ulrm} = 0.48d$

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

→ Calculate effective depth of slab,

$\frac{L}{d} < \alpha \times m.f$

$\Rightarrow d > \frac{4500}{26 \times 1.3}$ $\Rightarrow d > 133.13\,mm$

$\therefore d_{adopt} = 150\,mm$

$D_{assumed} = 150 + 20 = 170\,mm$

→ Effective length

$l_x = (4500 + 150$ or $4500 + 250)\,min^m$

$l_x = 4.65\,m$

$l_y = 5.65\,m$

→ Load Calcⁿ

self weight of slab $= 25 \times (0.17 \times 1) = 4.25\,kN/m$

$FF = 24 \times (0.025 \times 1) = 0.6\,kN/m$

Live load $= 4 \times 1 = 4\,kN/m$

Total load $= 8.85\,kN/m$

Factored load $= 1.5 \times 8.85$

$w_u = 13.275\,kN/m$

→ Bending moment & shear force,

Here given slab is restrained slab with one long edge discontinuous.

$\frac{l_y}{l_x} = \frac{5.65}{4.65} = 1.215$

	α_x	α_y	$M_x = \alpha_x w l_x^2$	$M_y = \alpha_y w l_x^2$
Negative moment at cont. edge	0.05275	0.037	15.14	10.62
Positive moment at mid span	0.03975	0.028	11.41	8.03

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→ check for 'd'

$M_{umax} = M_u$

$\Rightarrow 15.14 \times 10^6 = 0.138 \times 25 \times 1000 \times d^2$

$\Rightarrow d = 66.24\,mm < d_{assumed}$ (ok)

→ Calculation of reinforcement area.

① Along shorter span

(i) For negative moment,

$15.14 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \left(1 - \frac{415 \times A_{st}}{25 \times 1000 \times 150}\right)$

$A_{st} = 288.78\,mm^2$

$A_{st,min} = \frac{0.12}{100} \times 1000 \times 170 = 204\,mm^2 < A_{st}$ (ok)

$Spacing = \frac{1000 \times 50.26}{288.78} = 174.04\,mm < (3d$ or $300\,mm)$

(using 8 mm dia bar)

Provide 8 mm ϕ bar @ 170 mm c/c

$A_{st,pro} = \frac{1000 \times 50.26}{170} = 295.64\,mm^2$

(ii) For positive moment,

$11.41 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \left(1 - \frac{415 \times A_{st}}{25 \times 1000 \times 150}\right)$

$A_{st} = 215.83\,mm^2 > A_{st,min}\,(204\,mm^2)$ (ok)

$Spacing = \frac{1000 \times 50.26}{215.83} = 232.86\,mm$

Provide 8 mm ϕ bar @ 230 mm c/c $< (3d$ or $300\,mm)$

$A_{st,pro} = \frac{1000 \times 50.26}{230} = 218.52\,mm^2$

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Sumit Paudel

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② Along longer Span $d = 150 - 4 - 4 = 142mm$

(i) For negative moment:

$$10.62 \times 10^6 = 0.87 \times 415 \times A_{st} \times 142 \left(1 - \frac{415 \times A_{st}}{25 \times 1000 \times 142}\right)$$

$A_{st} = 212.41\ mm^2 > A_{st,min}$

$$\text{Spacing} = \frac{1000 \times 50.26}{212.41} = 236.61mm$$

Provide 8 mm ϕ bar @ 230 mm c/c

$$A_{st,pro} = \frac{1000 \times 50.26}{230} = 218.52mm^2$$

(ii) For positive moment,

$$8.03 \times 10^6 = 0.87 \times 415 \times A_{st} \times 142 \left(1 - \frac{415 \times A_{st}}{25 \times 1000 \times 142}\right)$$

$A_{st} = 159.6\ mm^2 < A_{st,min}$.

We have to provided minm A_{st}.

So,

$$\text{Spacing} = \frac{1000 \times 50.26}{204} = 246.37\ mm < (3d \text{ or } 300mm)$$

Provide 8 mm ϕ bar @ 240 mm c/c

$$A_{st,pro} = \frac{1000 \times 50.26}{240} = 209.41\ mm^2$$

→ Extra steel in ~~longer~~ shorter direction at support

$= 288.87 - 218.52$

$= 70.35\ mm^2$

$$\text{Spacing} = \frac{1000 \times 50.26}{70.35} = 714.42mm$$

So, provide 8mm ϕ additional bar @ 600mm c/c.

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→ Extra steel in longer direction at support

$= 212.41 - 209.41$

$= 3\ mm^2$

$$\text{Spacing} = \frac{1000 \times 50.26}{3} = 16753mm$$ ---very greater

So, no need to provide additional rebar in longer direction.

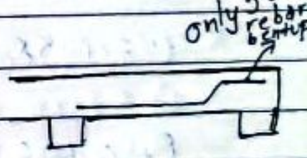
→ Check for shear,

$$V_u = \frac{w_u \times l}{2} = \frac{13.275 \times 4.5}{2} = 29.866\ kN$$

$$\tau_v = \frac{V_u}{bd'} = \frac{29.866 \times 10^3}{1000 \times 142} = 0.21\ N/mm^2$$

$$\tau_{c,slab} = \overset{1.3}{\overrightarrow{k}} * \tau_c$$

Also, $p_t = \frac{100 \times (295.64/2)}{1000 \times 142} = 0.104\%$.



For M25 grade concrete & $p_t < 0.15\%$,

$\tau_c = 0.29\ N/mm^2$

$\tau_{c,slab} = k * \tau_c = 1.3 \times 0.29 = 0.377\ N/mm^2$

$\tau_v < \tau_{c,slab}$, no need to design shear reinforcement

→ Check for development length

$L_d < \frac{1.3 M_1}{V} + L_o$

Here, $L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 8}{4 \times 1.6 \times 1.4} = 322.36mm$

$$M_1 = 0.87 \times 415 \times 295.64 \times 150 \left(1 - \frac{415 \times 295.64}{25 \times 1000 \times 150}\right)$$

$= 15.487 \times 10^6\ Nmm$

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If hook isnot provided,

L_o = bearing width/2 − side cover

= (250/2) − 15

= 110 mm.

$\frac{1.3M_1}{V} + L_o = \frac{1.3 \times 15.487 \times 10^6}{29.866 \times 10^3} + 110 = 784.11$ mm

Since, $\frac{1.3M_1}{V} + L_o > L_d$, OK.

→ Check for deflection:

$\left(\frac{L}{d}\right)_{calc} = \frac{4650}{150} = 31$

$f_s = 0.58 \times f_y \times \frac{(A_{st,req})_{mid\,span}}{(A_{st,pro})_{mid\,span}}$

$= 0.58 \times 415 \times \frac{215.83}{218.52} = 237.73\ N/mm^2$

$p_t = \frac{100 \times A_{st}}{bd} = \frac{100 \times 218.52}{1000 \times 150} = 0.145\%$.

For $f_s = 237.73\ N/mm^2$ & $p_t = 0.145\%$, $K_t = 1.9$

Then,

$\left(\frac{L}{d}\right)_{per} = 1.9 \times 26 = 49.4$ (OK)

→ Design of torsion reinforcement

ⓐ For one edge discontinuous,

Provide 8 mm ϕ bar at $0.2l_x = 0.2 \times 4.65 = 0.93$m distance from edge face.

$A_{st,torsion} = \frac{3}{8} \times 218.52$ (mid span area)

$= 81.945\ mm^2$

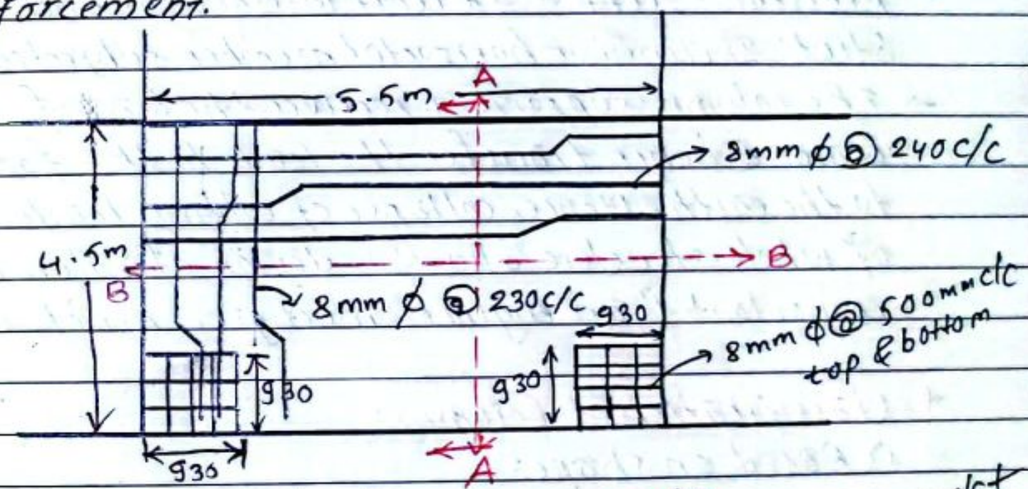
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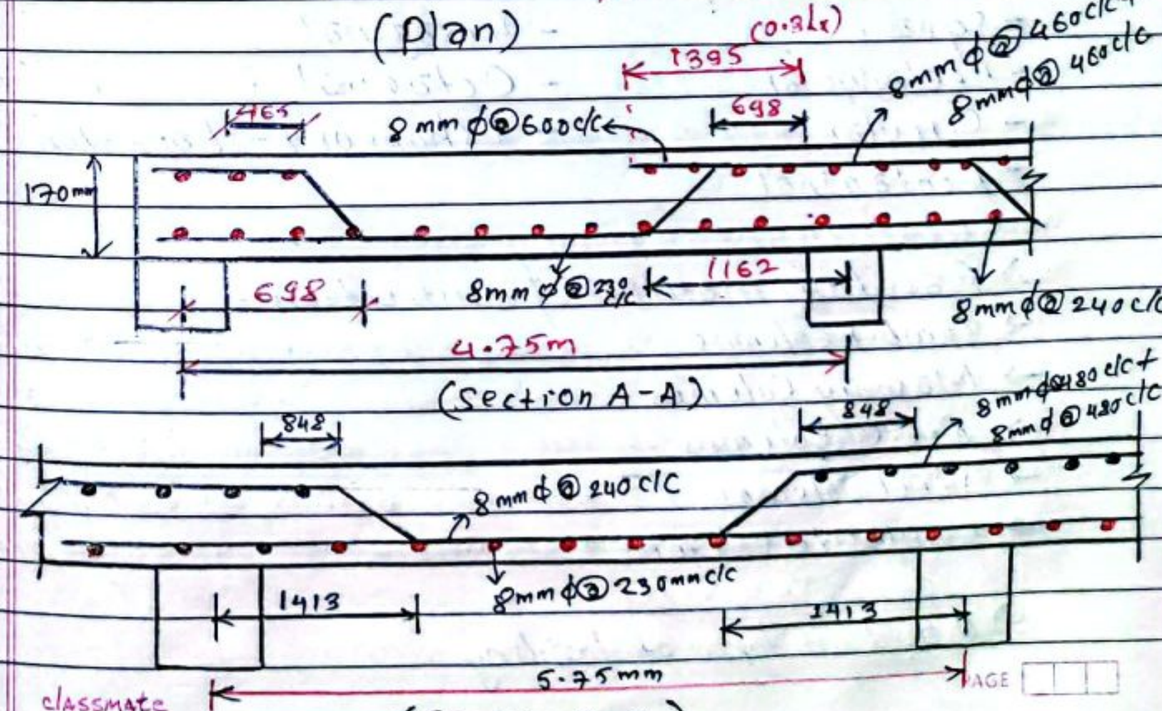
Spacing $= 930 \times \frac{50.26}{81.945} = 570.4$ mm

Provide 8 mm ϕ @ 500 mm c/c torsion bar in top & bottom.

ⓑ For continuous edge, no need to provide torsional reinforcement.



(Plan)



(Section B-B.)

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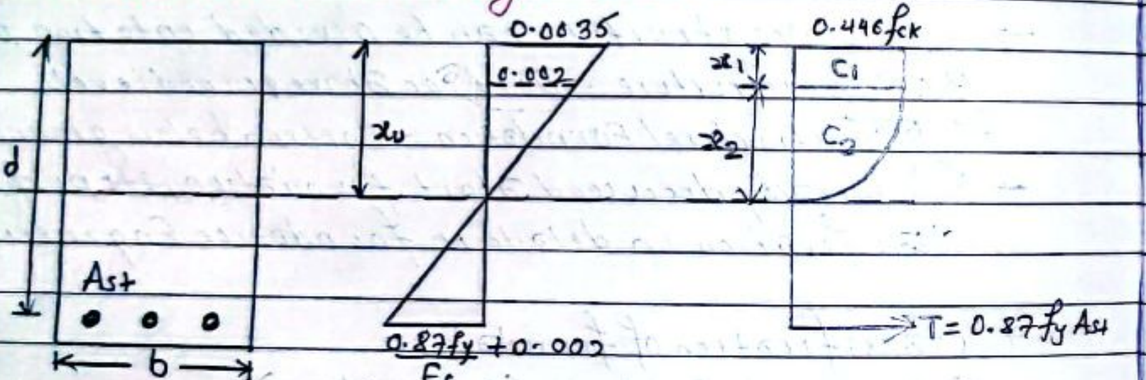
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* Choice of footing for design:-
→ Soil strata
→ Bearing capacity of soil
→ Type of structure
→ Types of load
→ Permissible settlement
→ Economy

* Review on limit state design – flexure:-



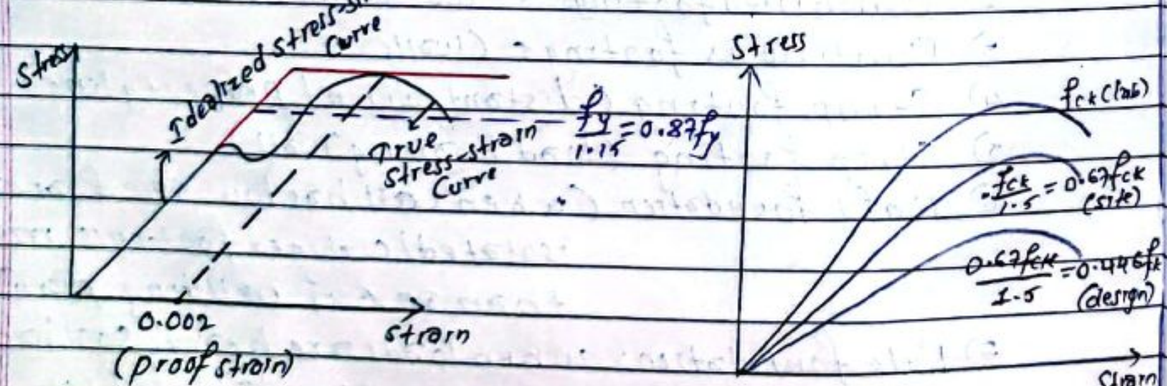


Fig: Stress-strain Curve for steel

Fig: Stress strain curve of concrete.

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From figure, $\frac{x_u}{0.0035} = \frac{x_2}{0.002} \Rightarrow x_2 = \frac{4}{7} x_u$

$x_1 = \frac{3}{7} x_u$

Tension force (T) = $0.87 f_y A_{st}$

Compression force (C) = $C_1 + C_2$

$= 0.446 f_{ck} \times (b \times x_1) + \frac{2}{3} \times x_2 \times b \times 0.446 f$

$= 0.36 f_{ck} b x_u$

Let, Total compressive force (C) acts at a distance of $\bar{x}$ from top,

Then,

$$\bar{x} = \frac{(0.446 f_{ck} \times x_1) \times \frac{x_1}{2} + \left(\frac{2}{3} \times 0.446 f_{ck} \times x_2\right) \times \left(x_1 + \frac{3}{8} x_2\right)}{(0.446 f_{ck} \times x_1) + (0.446 f_{ck} \times x_2 \times \frac{2}{3})}$$

$\bar{x} = 0.42 x_u$

(a) For balance section,

$C = T$, $x_u = x_m$

MOR = C * Z or T * Z (Z = lever arm)

Here, $Z = d - 0.42 x_u = d - 0.42 x_m$

$\therefore$

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

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

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

(b) Under-reinforced Section;

$T < C$, $x < x_m$

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

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(c) Over reinforced Sectron

$T > C, \quad x > x_m$

$M_u = 0.36 f_{ck} b x_m (d - 0.42 x_m)$

※ Maximum Depth of Neutral Axis.

$$\frac{0.0085}{x_m} = \frac{0.87 f_y / E_s + 0.002}{d - x_m}$$

$$\Rightarrow \frac{x_m}{d - x_m} = \frac{0.0035}{\frac{0.87 f_y}{E_s} + 0.002}$$

$$\Rightarrow x_m = \frac{0.0035 d}{(4.35 \times 10^{-6} f_y + 0.002 + 0.0035)}$$

x_m / d

put value of f_y for different grade of steel.

* Limiting moment,

$M_{u,lim} = 0.36 f_{ck} b x_m (d - 0.42 x_m)$

put value of x_m for different grade of steel & find $M_{u,lim}$.

For Fe415,

$x_m = 0.48 d$

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

* Covers & ϕ of bars.

Structure element	Nominal cover (mm)	dia (mm)
slab	15	8, 10
beam	25	12, 16, 20, 25, 28
column	40	16, 20, 25, 28, 32, 36
footing	50	12, 16, 20, 25
staircase	20	12, 16, 20

※ Analysis of footing: (Isolated)

→ The footing which are provided under single columns are called as isolated footings.

→ Usually square or rectangular & rarely circular.

→ Even for circular column, the area of circular column is equated to rectangular column & size of rectangular equivalent column is found out.

→ provided when loads are small & soil is not very poor (bearing capacity is sufficient)

→ Types

⇒ Uniform thickness footing

⇒ Tapered thickness footing (sloped footing)

* Codal provision for design of Isolated Footing (cl 34)

→ The thickness at edge shall not be less than 150mm for footings on soils.

→ Moment & Forces :-

→ The greatest bending moment to be used in the design of an isolated footing shall be at sections located as follows.

a) At face of column, pedestal or wall

b) Halfway betn centreline & edge of wall for footing under masonry wall

c) Halfway betn face of column or pedestal & edge of gusseted base for footing under gusseted bases.

→ For isolated footing, shear stress at critical section should not exceed $k_s \tau_c$

$\tau_v \leq k_s \tau_c$

where, $k_s = (0.5 + \beta_c)$ but $k_s \neq 1$, $\beta = \frac{\text{short dimension of col}}{\text{Long dim. of column}}$

$\tau_c = 0.25 \sqrt{f_{ck}}$ for LSM &

$0.16 \sqrt{f_{ck}}$ for WSM

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→ Shear & Bond:

(a) One way shear:

The footing acts as wide beam, the critical section for this shear shall be assumed at a distance 'd' equal to the effective depth of footing from face of column.

(b) Two way Action or Punching Shear:

The critical section for punching shear shall be at a distance $d/2$ from face of column, where 'd' is effective depth of section.

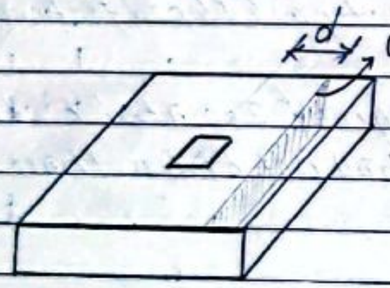


Fig: One way shear

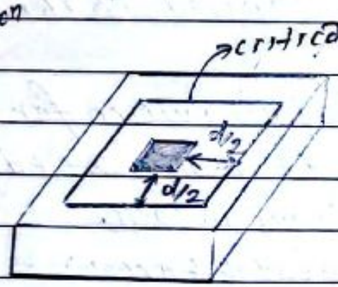


Fig: Critical section for two way Shear

→ Tensile Reinforcement:

- In a square footing, the reinforcement in each direction shall be distributed across the full width of footing
- In rectangular footing, the reinforcement in long direction shall be distributed across the full width of footing. For reinforcement in short direction, a central band equal to the width of footing is taken length of footing & reinforcement determined as per the eqn given below shall be uniformly distributed across the central band.

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$$\frac{\text{Reinforcement in central bandwidth}}{\text{Total reinforcement in short direction}} = \frac{2}{\beta+1}$$

where,

β = Ratio of long side to short side of footing. The remainder of reinforcement shall be uniformly distributed in the outer portion of footing.

→ Nominal Reinforcement:

Min^m Reinforcement = 0.12% of Bd

→ Cover to Reinforcement: '50 mm'

Design steps:

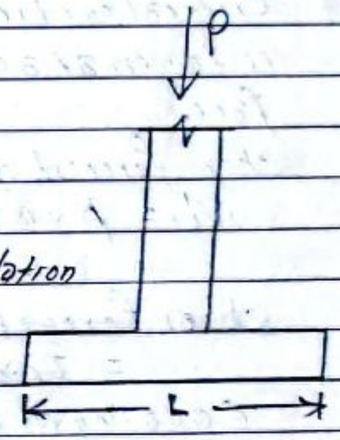
① Total load calculation,

Total load (P_{total}) = P + Self wt of foundation

(10% of 'P')

[Self wt of foundation = $V_{soil} \times A_{approx} \times b$]

$$A_{approx} = \frac{P}{SBC}$$



② Calculate area of footing,

$$A = \frac{P_{total}}{S.B.C \text{ of soil}}$$

③ Calculate size of footing

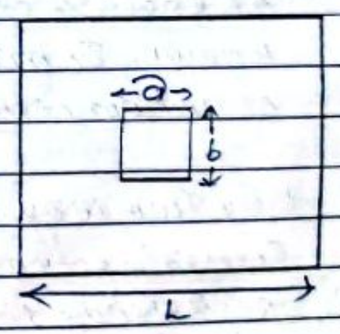
→ For square footing, $a = b = \sqrt{A}$

→ For rectangular footing,

assume length (L), then find B as, $B = A/L$

Alternatively,

find ratio of a/b for column & by taking same ratio of L/B, find L & B of footing.



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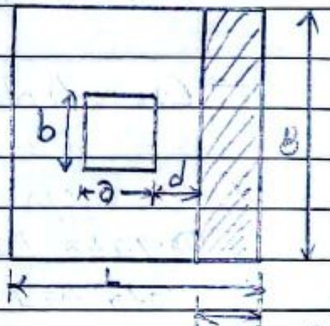
(4) Calculate the soil pressure due to factored column load only.

$$p = \frac{1.5 * P}{L \times B}$$

(5) Depth of footing is calculated by following three criteria & highest value so calculated is adopted in design.

* By one way shear Criteria:

Critical section for one way shear is taken at a distance 'd' from column's face.



shear force at critical section,

$$V_u' = p \times B \times \left(\frac{L-a}{2} - d\right) \quad --(i)$$

shear force resisted by concrete

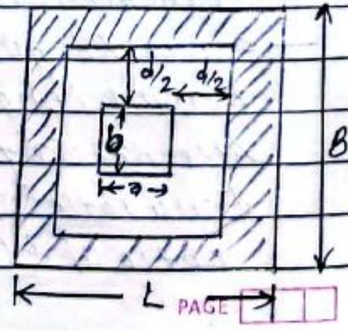
$$= \tau_c \times B \times d \quad --(ii)$$

Equating (i) & (ii), calculate depth (d) of footing.

→ As exact % of reinforcement to be provided is not yet known, τ_c may be assumed as that corresponding to minimum reinforcement (0.2%).

* By Two way shear / Punching shear Criteria:

Critical section for two way shear or punching shear is at a distance $d/2$ from face of column.



Punching shear at critical section

$$V_u'' = p * [L \times B - (a+d)(b+d)] \quad --(iii)$$

perimeter of critical section,

$$= (a+d) \times 2 + (b+d) \times 2$$
$$= 2a + 2b + 4d$$

Punching shear resisted by section

$$= \tau_c \times (2a + 2b + 4d) \times d \quad --(ii)$$

where, $\tau_c = 0.25\sqrt{f_{ck}}$

By equating eqn (i) & (ii), depth of footing (effective) (d) can be calculated.

* By bending Moment Criteria:

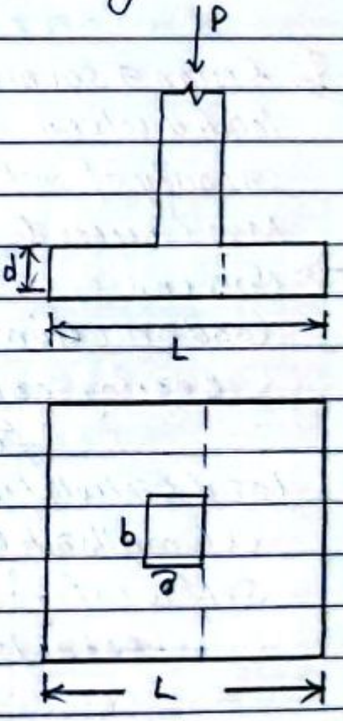
→ The critical section for bending moment is at face of column

→ BM at critical section is,

$$BM = p * B * \frac{(L-a)^2}{2} \quad --(i)$$

Also,

$$BM = p * L * \frac{(B-b)^2}{2} \quad --(ii)$$



→ Moment of resistance of section is,

$$M_{ulim} = 0.36 f_{ck} \times \frac{x_{ulim}}{d}\left[1 - 0.42\frac{x_{ulim}}{d}\right] \times Bd^2 \quad --(iii)$$

Equating (i) & (ii) with (iii), we can find out value of 'd'.

(6) Determine area of reinforcement reqd by following eqn:

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

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The reinforcement area so calculated should not be less than the minm reinforcement & distributed as per IS code provisions.

(7) Check for development length.

* For design of square footing, follow above mentioned procedure & substitute $L = B$ & $a = b$.

Q. Design a square footing of uniform thickness for an axially loaded column of 450mm * 450mm in size. The safe bearing capacity of soil is 190 kN/m². Load on column is 850 kN. Use M20 concrete & Fe415 steel.

→ Given,

Load on column (P) = 850 kN

Bearing capacity of soil = 190 kN/m²

f_{ck} = 20 N/mm² & f_y = 415 N/mm²

Load calculation,

column load (P) = 850 kN

Self wt of footing = 0.1 × 850 = 85 kN

Total load = 850 + 85 = 935 kN

Area of footing,

$$A = \frac{\text{Load}}{\text{SBC}} = \frac{935}{190} = 4.92 m^2$$

Side of square footing = $\sqrt{4.92}$ = 2.22m, ≈ 2.25m

Factored soil pressure due to column load only

$$p = \frac{850 \times 1.5}{2.25 \times 2.25} = 251.85 \text{ kN/m}^2$$

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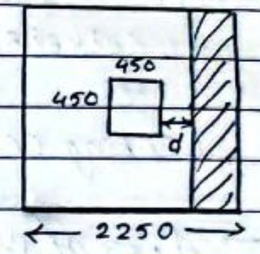
Footing depth Calculation

(a) By one way shear criteria:-

Shear force at critical section,

$V_u' = 251.85 \times 2250 \times (900 - d) \times 10^{-3}$

$= 566.6625 (900 - d)$ --(i)



Shear force resisted by concrete

$= \tau_c \times Bd$

$= 0.32 \times 2250 \times d$ [Assuming 0.2% steel, τ_c = 0.32 N/mm²]

$= 720d$ --(ii)

From (i) & (ii), $d = 396.37$ mm

(b) By Two way shear Criteria:-

Shear force at critical section,

$V_u'' = 251.85 \times 10^{-3} \times [2250 \times 2250 - (450 + d)^2]$ --(iii)

Shear force resisted by concrete

$= \tau_c' \times 4(d + a) \times d$

Here, $\tau_c' = \tau_c \times k_s$, $k_s = 0.5 + \beta_c = 0.5 + \frac{400}{400} = 1.5$

$= 0.25\sqrt{f_{ck}} \times 1$

$= 0.25\sqrt{20}$ But k_s must be less or equal to 1.

$= 1.118$ N/mm² So, $k_s = 1$

So, Shear force resisted by concrete = $4 \times 1.118 \times (d + 450)$ --(iv)

Equating (iii) & (iv), $d = 324.5$ mm

(c) By bending moment criteria,

Bending moment at critical section = $\frac{251.85 \times 10^{-3} \times 2250 \times 900^2}{2}$

$= 229.498 \times 10^6$ Nmm

$M_{u lim} = 0.138 \times f_{ck} B d^2 = 0.138 \times 20 \times 2250 \times d^2$

on solving, $d = 192.28$ mm

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From above three cisterra, Max^m value of d = 396.37mm

Let us adopt d = 400 mm

Overall depth = 400 + 8 + 50

= 458 say 460 mm

[Taking cc = 50mm & 16mm dia. bars]

→ Area of Steel Reinforcement,

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

$$229.498 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{A_{st} \times 415}{2500 \times 400 \times 20}\right]$$

$$A_{st} = 1645 mm^2$$

Min^m Reinforcement reqd, $A_{st\,min} = \frac{0.12}{100} \times 2250 \times 460 = 1242 mm^2 < A_{st}$

Using 16ϕ bars,

$$\text{Spacing} = \frac{2250 \times \pi \times 16^2}{4 \times 1645} = 275 mm$$

∴ provide 16ϕ bars @ 270 mm c/c in each direction.

→ Check for development length,

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 16}{4 \times 1.6 \times 1.2} = 752.2 mm$$

Available length of bars = $\frac{2250 - 450}{2}$ = 900 mm (ok) > L_d

→ Load transfer,

Nominal bearing stress on column concrete = $\frac{P_u}{A_c}$

$$\tau_v = \frac{1.5 \times 850 \times 10^3}{450 \times 450} = 6.29 N/mm^2$$

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column concrete grade →

Permissible bearing stress = $0.45 f_{ck}$

$\tau_v' = 0.45 \times 20 = 9 N/mm^2 > 6.29 N/mm^2$

$\tau_v' > \tau_v$, so, column can transfer load by bearing alone.

[If $\tau_v' < \tau_v$, we have to provide dowel bars]

→ The reinforcement details is shown in figure below.

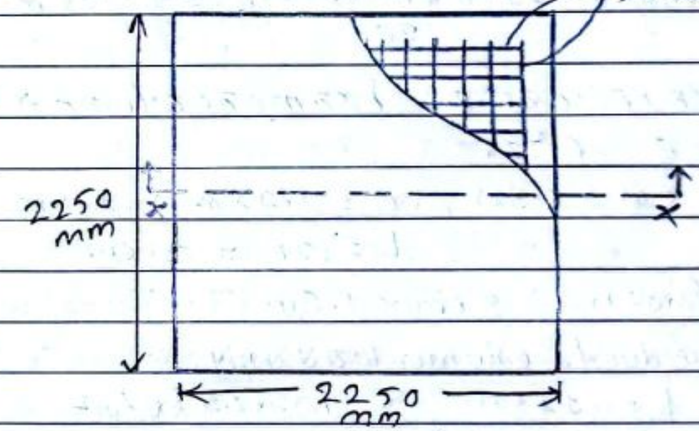
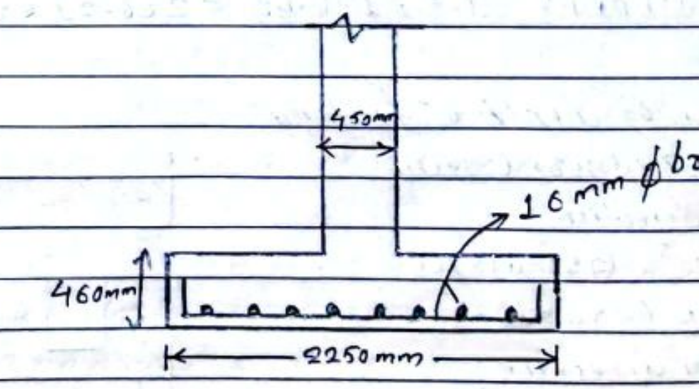


Fig: plan of footing



sectional Elevation at x-x

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Q. Design a rectangular footing of uniform thickness for an axially loaded column of size 300mm x 600mm, load on column is 1150kN. SBC of soil is 200 kN/m². Use M20 concrete & Fe415 steel.

→ Given,

Size of column = 300mm x 600 mm

Load on column = 1150 kN

SBC = 200 kN/m², f_{ck} = 20 N/mm², f_y = 415 N/mm²

→ Area of footing = $\frac{1150 + 0.1 \times 1150}{200}$ = 6.325 m²

Taking L/B ratio as 2, [same as column ⇒ 600/300 = 2]

L x B = 6.325

2B x B = 6.325, B = 1.78m ≈ 1.8m

L = 2B = 3.6m

Size of footing = 3.6m x 1.8m

→ Soil pressure due to column load only,

$p = \frac{1150}{3.6 \times 1.8}$ = 177.47 kN/m²

Factored soil press^r = 1.5 x 177.47 = 266.21 kN/m²

→ Calculation of depth of footing

(a) By one way shear Criteria

SF in longer direction

= 266.21 x 10^{-3} x (750 - d) x 3600

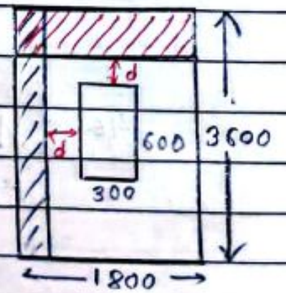
= 958.356 (750 - d)

SF in shorter direction

= 266.2 x 10^{-3} x (1500 - d) x 1800

= 479.178 x (1500 - d)

SF in shorter direction is more.



SF resisted by concrete = τ_c x Area [τ_c = 0.32 N/mm² for 0.2% steel]

= 0.32 x 1800 x d = 576d

On solving, d = 681.18 mm

(b) By two way shear Criteria:

Shear force at critical section,

$V_u'' = 266.21 \times [3600 \times 1800 - (600+d)(300+d)] \times 10^{-3}$

$= 0.26621 [6480000 - (600+d)(300+d)]$

Shear force resisted by concrete = τ_c' x Area

$\tau_c' = \tau_c \times k_s$

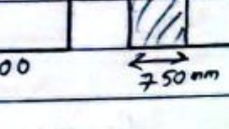
$k_s = 0.5 + \beta_c = 0.5 + \frac{600}{800} = 2.5$

k_s must be ≤ 1. So, k_s = 1

$\tau_c' = 0.25\sqrt{20} \times 1 = 1.118$ N/mm²

∴ Shear force resisted by concrete = $1.118 \times [2(600+d) + 2(300+d)] \times d$

On solving, d = 403.007 mm



(c) By bending Criteria:

BM in longer direction

$= 266.21 \times (1800 \times 1500) \times \frac{1500}{2} \times 10^{-3}$

= 539.075 x 10^6 Nmm

BM in shorter direction

$= 266.21 \times (750 \times 3600) \times \frac{750}{2} \times 10^{-3}$

= 269.54 x 10^6 Nmm.

Hence, BM in longer direction is more,

Now, $0.138 \times f_{ck} \times bd^2 = 539.075 \times 10^6$

$0.138 \times 20 \times 1800 \times d^2 = 539.075 \times 10^6$

d = 329.4mm

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Sumit Paudel

From above three criteria, highest value of depth of footing (d) = 681.18mm

Using 16mm dia bars & 50mm CC

Overall depth = 690 + 8 + 50

= 748mm say 750 mm

d = 750 - 8 - 50 = 692 mm

→ Area of reinforcement,

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

* For longer direction

$$539.075 \times 10^6 = 0.87 \times 415 \times A_{st} \times 692 \left(1 - \frac{A_{st} \times 415}{20 \times 1800 \times 692}\right)$$

$A_{st} = 2241.3 \text{ mm}^2$

$A_{st,min} = \frac{0.12}{100} \times 1800 \times 750 = 1620 \text{ mm}^2 < A_{st}$ (ok)

Spacing of 16mm ϕ bar = $\frac{1800 \times \pi \times 16^2}{4 \times 2241.3} = 161.47$ mm

So, provide 16ϕ @ 160 mm c/c

* For shorter direction,

$$269.54 \times 10^6 = 0.87 \times 415 \times A_{st} \times 692 \left(1 - \frac{A_{st} \times 415}{20 \times 3600 \times 692}\right)$$

$A_{st} = 1088.69 \text{ mm}^2$

$A_{st,min} = \frac{0.12}{100} \times 3600 \times 750 = 3240 \text{ mm}^2 > A_{st}$

As per IS provisions,

$$\frac{\text{Reinforcement in central band}}{\text{total reinforcement in shorter direction}} = \frac{2}{\beta + 1}$$

$\beta = \frac{3600}{1800} = 2$

$$\frac{\text{Reinforcement in central band}}{3240} = \frac{2}{2+1}$$

$\therefore$ Reinforcement in central band = $\frac{2}{3} \times 3240 = 2160 \text{ mm}^2$

This reinforcement shall be distributed in central 1800 mm bandwidth

Spacing of 16mm ϕ bars = $\frac{1800 \times \pi \times 16^2}{4 \times 2160} = 167.5 \approx 160$ mm

Provide 16ϕ bars @ 160 mm c/c in central band.

Balance area of steel to be distributed in outer bands

= 3240 - 2160 = 1080 mm²

Spacing of 16ϕ bars = $\frac{201}{1080} \times 1800 = 335$ mm

Maxm spacing allowed = 300 mm or d (692 mm) which is less

∴ provide 16ϕ bars @ 300mm c/c for outer bands.

→ Check for development length:-

$L_d = \frac{0.87 \times f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 16}{4 \times 1.6 \times 1.2} = 752.2$ mm

Available length in shorter direction beyond column face

$= \frac{1800 - 300}{2} = 750 \text{ mm} < 752.2 \text{ mm}$

Provide 90° bend at free end of reinforcement

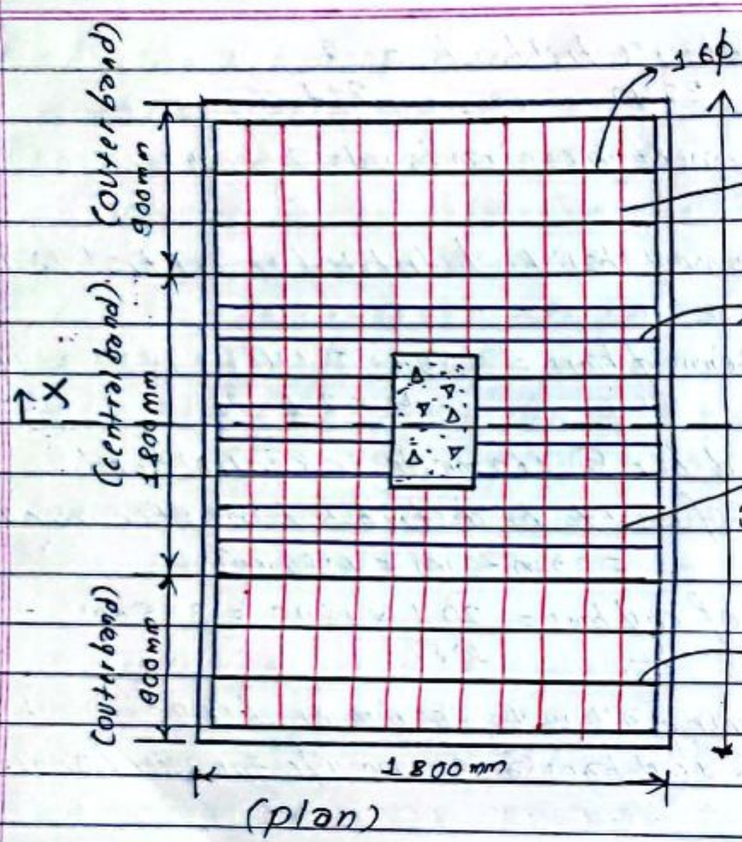
Anchorage value of bend = 8ϕ

= 8 × 16 = 128 mm

Hence, total anchorage length = 750 + 128

= 878 mm > L_d (ok).

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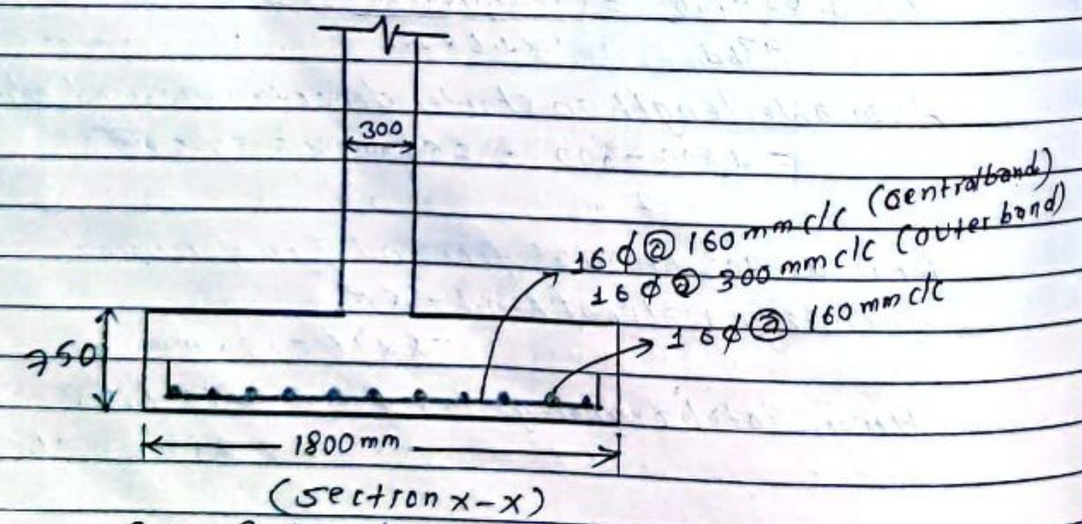


Fig: Rebar details in rectangular footing

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Q. Design a rectangular footing in which one side is limited to 2m for a rectangular column 450×300 mm reinforced with 8-25mm bars & carrying a load of 1200kN & Mmax = 100kNm. Assume soil bearing capacity of 120 kN/m², mild exposure condition for concrete & TMT steel.

→ Given,

width of ~~beam~~ footing (B) = 2m

Axial load (P) = 1200kN

Moment, M_{ux} = 100 kNm

Bearing capacity of soil = 120 kN/m²

f_{ck} = 20 N/mm² (mild exposure condition)

f_y = 500 N/mm² (TMT steel)

Now,

$$\text{Area of footing} = \frac{P + \gamma \times h \times P/Bc}{B.C}$$

$$= \frac{1200 + 20 \times 1.5 \times 1200/120}{120}$$

$$= 12.5 m^2$$

$$\text{Length of footing } (L) = \frac{12.5}{2} = 6.25m$$

Upward pressure calculation (p)

$$p = \frac{1.5 \times P}{L \times B} = \frac{1.5 \times 1200}{6.25 \times 2} = 144 kN/m^2$$

$$\text{eccentricity } (e) = \frac{M}{P} = \frac{100}{1200} = 0.08m = 80mm = 8cm$$

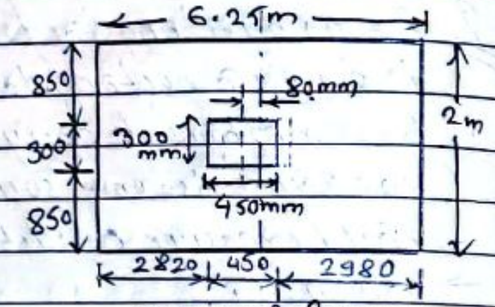
→ Calculation of depth of footing

~~Critical section at d" from face of column.~~

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(a) By one way shear Criterra,



plan of footrng

SF in longer direction
$= 144 \times [6.25 \times (0.85-d)]$
$= 765 - 900d$

SF in shorter direction
$= 144 \times [2 \times (2.98-d)]$
$= 858.24 - 288d$

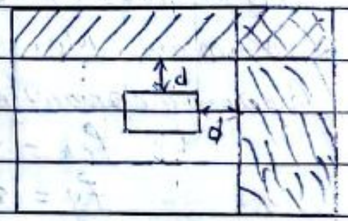
SF in shorter direction is more.

SF resisted by concrete $= \tau_c \times bd$

For 0.2% of steel rebar & M20 concrete
$\tau_c = 0.32\ N/mm^2$

Then,

SF resisted by concrete $= \frac{0.32 \times 10^{-3}}{10^{-6}} \times 2 \times d$
$= 640d$

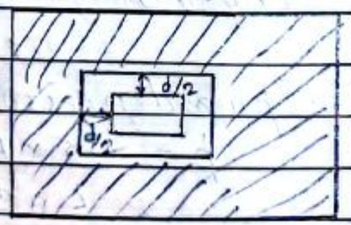


One way shear

Now, $640d = 858.24 - 288d \Rightarrow d = 0.9248m$

(b) By two way shear Criteria:

Shear force at critical section
due to shaded area,
$= 144 \times [6.25 \times 2 - \{(0.45+d) \times (0.3+d)\}]$ ---①



Punching shear resisted by section
$= \tau_c \times A$

$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.118\ N/mm^2$ Two way shear

Perimeter(p) $= 2 \times (0.45+d) + 2 \times (0.3+d)$

Then, Shear resisted by concrete $= \frac{1.118 \times [2 \times (0.45+d) + 2(0.3+d)] \times d}{\times 1000}$ ---⑪

Equating ① & ⑪,
$d = 0.457m$

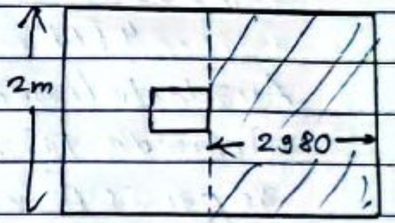
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(c) By bending moment Criteria,

BM in Longer direction
$= 144 \times (2.98 \times 2) \times 2.98/2$
$= 1278.77\ kNm$



Moment of resistance
$= 0.138 f_{ck} bd^2$
$= 0.138 \times 20 \times 10^3 \times 2 \times d^2$
$= 5820 d^2$

Then, $d = 0.49m$

→ From above three criterra, 924.8mm is highest,
Using 16mm dia & 50mm clear cover,
$D = 924.8 + 8 + 50$
$= 982.8mm$ say $990mm$
$d = 990 - 8 - 50 = 932mm$

→ Area of reinforcement
For longer direction,

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

$$1278.77 \times 10^6 = 0.87 \times 500 \times A_{st} \times 932 \left(1 - \frac{A_{st} \times 500}{20 \times 2000 \times 932}\right)$$

$A_{st} = 3300.266\ mm^2$

$A_{st,min} = \frac{0.12}{100} \times 2000 \times 990 = 2376\ mm^2 < A_{st}$ (ok)

Using 16ϕ bars,

$$\text{spacing} = \frac{2000 \times \pi \times 16^2/4}{3300.266} = 121.84\ mm$$

Provide 16ϕ bars @ 120 mm clc in each direction.

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→ Check for development length,

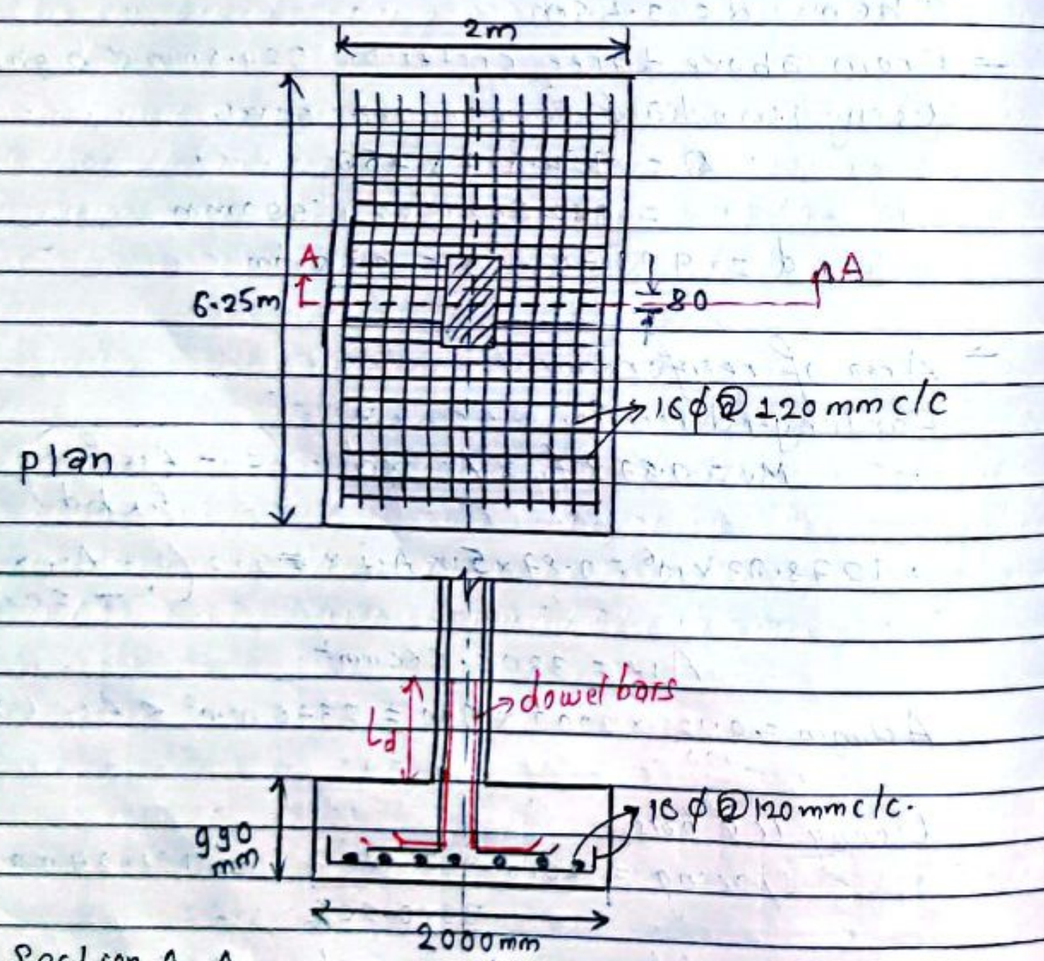
$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 500 \times 16}{4 \times 1.6 \times 1.2} = 906.25\ mm$

Available length of bars = 850mm < L_d. (Not ok)

Provide 90° bend at the free end of reinforcement, as per IS Provisions, anchorage value of 90° bend = 8ϕ.

So, Total anchorage length = 850 + 8 × 16

= 978mm > L_d (ok)



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→ Check for load Transfer:-

Nominal bearing stress on column concrete = $\frac{P_u}{A_c}$

$\tau_v = \frac{1.5 \times 1200 \times 10^3}{300 \times 450} = 13.33\ N/mm^2$

Permissible bearing stress = $0.45 \times f_{ck}$ → grade for concrete column

$= 0.45 \times 20$

$\tau_v' = 9\ N/mm^2 < 13.33\ (\tau_v)$

Here, $\tau_v' < \tau_v$, so, column cannot transfer load by bearing alone. Hence, we have to provide dowel bars.

Area of dowel bar = $\frac{(13.33 - 9) \times 300 \times 450}{0.67 \times 500}$

$= 1744.92 mm^2$

Let us use 25 mm ϕ dower bars,

No. of bars = $\frac{1744.92}{(\frac{\pi \times 25^2}{4})} = 3.55 \simeq 4$

So, provide 4 - 25 mm ϕ dra dowel bars.

* Design of Combined footing:-

→ Combined footing is provided under two or more columns. It is provided in following cases:

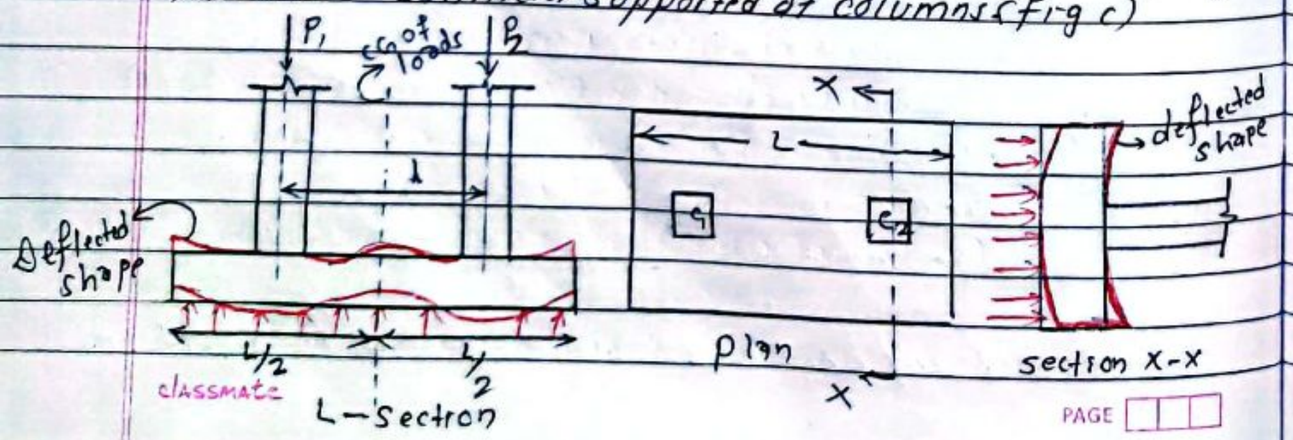
1) When two columns are close together such that the separate footing of the individual columns overlap.
2) When the bearing capacity of the soil is low enough to cause overlapping of the adjacent isolated footing.
3) When a column is too close to a property line, building or sewerline.

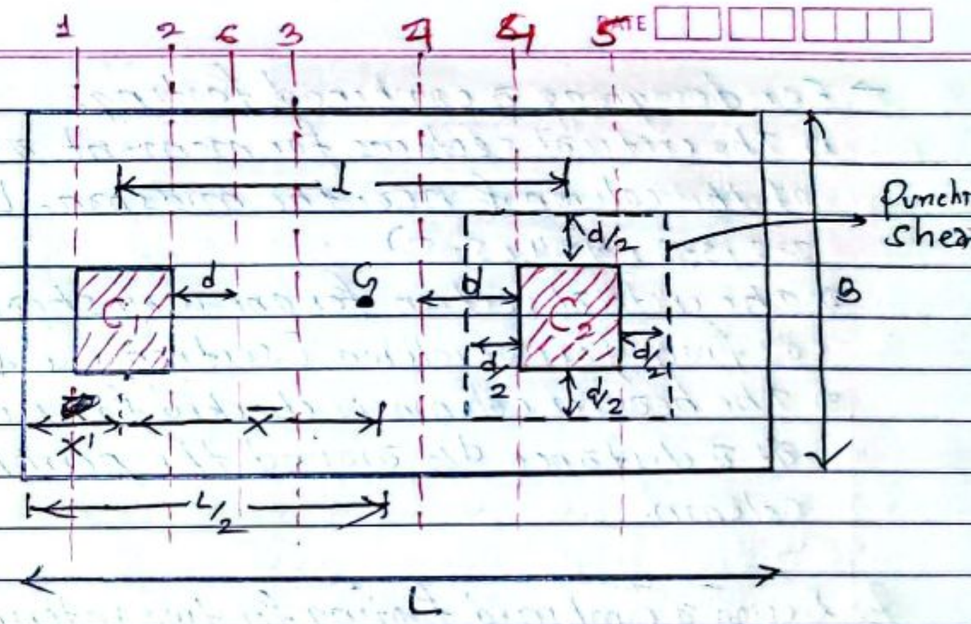
→ The combined footing may be rectangular or trapezoidal in shape.

→ The basic principle of design of a combined footing is that the centre of gravity of the footing should coincide with the centre of gravity of the loads, which results in uniform base pressure distribution.

※ Analysis of Combined Footing:-

Consider the combined footing shown in figure. The footing carries an upward base pressure & acts like a beam supported on two columns in the longitudinal direction. In the transverse direction, the footing acts like a cantilever supported at columns (fig c)





Let the column C_1 carries a load P_1 and the column C_2 carries a load P_2. The footing is proportioned in such a way that the CG of the load system coincides with the CG of the footing.

Distance of the resultant load (P_1+P_2) from column C_1

$$\bar{x} = \frac{P_2 \times L}{P_1 + P_2}$$

The point 'G' should be CG of footing.

$$x' + \bar{x} = L/2 \quad \Rightarrow L = 2(x' + \bar{x})$$

→ In the longitudinal direction, the footing is designed by considering it to be a beam of width 'B' & the maximum values of bending moment & shear force is calculated at the critical sections shown in fig.

In the transverse direction, the footing is considered as cantilever projecting from the column supports. In this case, the BM is calculated at the face of the column.

TMT ⇒ 500 N/mm² → fy
TOR ⇒ 415 N/mm²

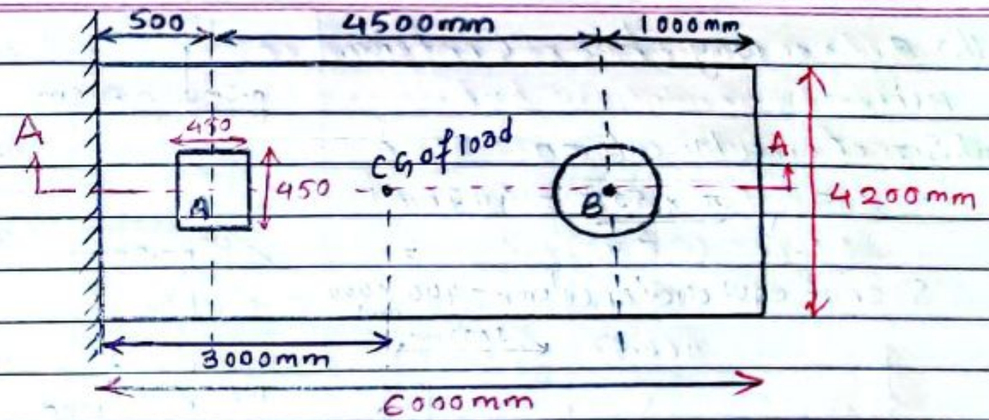
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→ For designing a combined footing:
1) The critical sections for moment are at the face of the column & near the midspan. [section 1-1, 2-2, 3-3, 4-4, 5-5)
2) The critical section for one way shear is at a distance 'd' from face of column [section 6-6 & 7-7]
3) The heavier column is checked for punching shear at a distance d/2 around the periphery of the column.

Q Design a combined footing for two interior columns carrying axial loads 1200kN & 1500kN. Column A is 450mm * 450mm & column 'B' is circular with 500mm diameter. The centroid of column 'A' is 500mm from property line. They are reinforced with 25mm ϕ bars with 4.5m spaced c/c. Bearing capacity of soil is 120kN/m². Use moderate exposure condition of concrete & TMT steel.

→ Given,
$P_A = 1200\,kN$, $P_B = 1500\,kN$
Size of column A = 450mm x 450mm
Size of column B = 500mm ϕ
centroid of column 'A' from property-line = 500mm
c/c spacing of A & B = 4500 mm
Safe bearing capacity of soil = 120 kN/m²
$f_{ck} = 25\,N/mm^2$ for moderate exposure (Table 5)
$f_y = 500\,N/mm^2$ (For TMT steel)

Let 'x' be the CG of loads from property line.



$$x = \frac{P_A \times 0.5 + P_B \times (0.5 + 4.5)}{P_A + P_B} = \frac{1200 \times 0.5 + 1500 \times 5}{1200 + 1500}$$

$x = 3m$
Length of footing (L) = 3 + 3 = 6m
Area of footing = $\frac{1.1(1200+1500)}{120} = 24.75\,m^2$
Width of footing (B) = $\frac{A}{L} = \frac{24.75}{6} = 4.125m \approx 4.2m$

Size of footing = 6m x 4.2m
Upward pressure (p) = $\frac{1.5(P_A + P_B)}{L \times B} = \frac{1.5(1200+1500)}{6 \times 4.2}$
$= 160.71\,kN/m^2$
$w = p \times B = 160.71 \times 4.2 = 675\,kN/m$

→ Depth of footing
(a) By maximum bending moment -
Maxm BM = 1500 kNm
Then, $M_{max} = M_{u\,lim}$ ($0.133 f_{ck} b d^2$ for Fe500)
$1500 \times 10^6 = 0.133 \times 25 \times 4200 \times d^2$
$d = 327.78\,mm \simeq 350\,mm$

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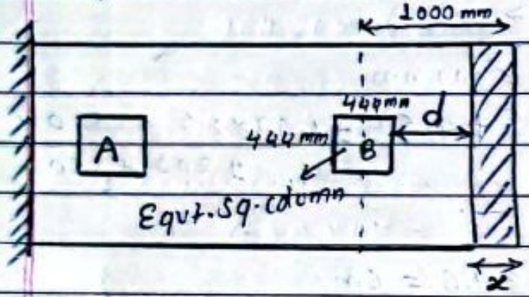
(b) By one way shear Criteria,

Here,

Eqvt. Size of circular column

$= \sqrt{\frac{\pi \times 500^2}{4}} = 444\,mm$

∴ Size of eqvt circular column = 444 × 444 mm



Shear force at 'd' distance from face of equivalent square column

$= 1575 \times 10^3 - 675 \times (222 + d)$ — (i)

Let use % of steel = 0.15%,

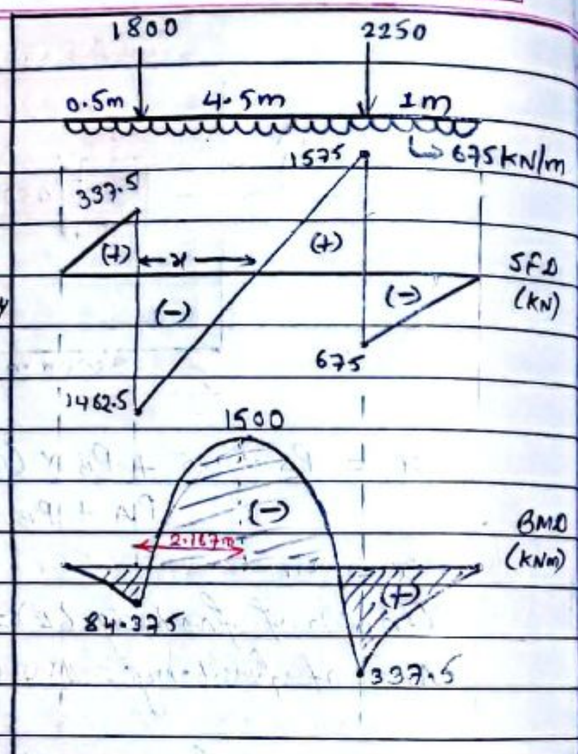
$\tau_c = 0.29\ N/mm^2$

Shear force balanced by $\tau_c = \tau_c \times b \times d$

$= 0.29 \times 4200 \times d$ — (ii)

Solving (i) & (ii),

$d = 752.85\,mm$



From SFD, $\frac{1462.5}{x} = \frac{1575}{4.5 - x} \Rightarrow x = 2.167m$

$BM_x = -1800 \times 2 + \frac{2.667^2 \times 675}{2}$

$= -1500\,kNm$

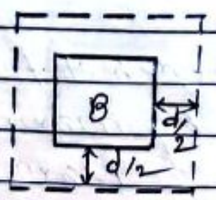
(c) By two way shear criteria,

Shear force $(V_w) = 2250 * 10^3 - (d+444)(d+444) \times 160.71 \times 10^{-3}$ — (iii)

Nominal shear stress, $\tau_v = 0.25\sqrt{f_{ck}} = 1.25\ N/mm^2$

Shear force balanced by $\tau_v = \tau_v \times b_o d$

$= 1.25 \times 4 (d + 444) \times d$ — (iv)



On solving (iii) & (iv), $d = 46.52\,mm$

From above three criteria,

Max^m depth $(d) = 752.85\,mm \approx 780\,mm$

Overall depth $(D) = 780 + 20/2 + 50 = 840\,mm$

→ Steel Calculation

(a) Longitudinal bars,

(i) Negative Moment (Hogging Moment),

$1500 \times 10^6 = 0.87 \times 500 \times A_{st}^- \times 780 \left(1 - \frac{500 \times A_{st}^-}{25 \times 4200 \times 780}\right)$

$A_{st}^- = 4547.09\,mm^2 > A_{st,min}$ (OK)

$A_{st,min} = 0.12\%\,bD = \frac{0.12}{100} \times 4200 \times 840 = 4233\,mm^2$.

Provide 16mm ϕ bars,

Spacing of bar $= \frac{4200}{4547.09} \times 201 = 185.65\,mm \approx 180\,mm$

Provide 16mm ϕ longitudinal bars @ 180 mm c/c to resist hogging moment.

(ii) Positive Moment (Sagging Moment)

$337.5 \times 10^6 = 0.87 \times 500 \times A_{st}^+ \times 780 \left(1 - \frac{500 \times A_{st}^+}{25 \times 4200 \times 780}\right)$

$A_{st}^+ = 1000\,mm^2 < A_{st,min}$.

So, Provide $A_{st}^+ = 4233\,mm^2$

Let us use 12mm ϕ bars,

Spacing of bar $= \frac{4200}{4233} \times 113 = 112.11\,mm \approx 100\,mm$

Provide 12mm ϕ bar @ 100 mm c/c to resist sagging moment.

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(b) Transverse Reinforcement:

Load intensity(w) = p × L

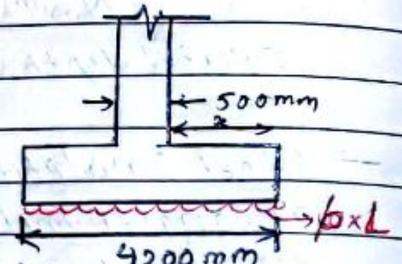
= 160.71 × 6

= 964.26 kN/m

$M_{max} = \frac{w \times x^2}{2}$

$= \frac{964.26}{2} \times \left(\frac{4200-500}{2}\right)^2 \times 10^{-6}$

= 1650 kNm



For reinforcement area,

$1650 \times 10^6 = 0.87 \times 500 \times A_{st} \times 780 \left(1 - \frac{500 \times A_{st}}{25 \times 6000 \times 780}\right)$

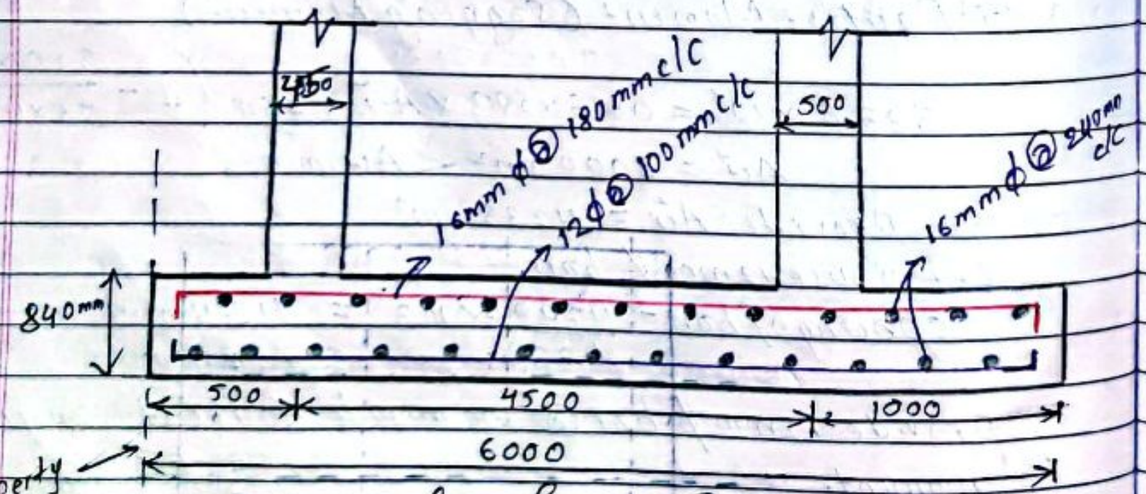
$A_{st} = 4968\ mm^2 > A_{st,min}$

Provide 16mm ϕ bar,

Spacing of bar = $\frac{6000}{4968} \times 201 = 242$ mm

≈ 240mm

Provide 16mm ϕ transverse rebar @ 240mm c/c.



Detailing of reinforcement @ Section A-A

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* Design of Raft Foundation :-

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→ If the loads transmitted by the columns in a structure are so heavy or the allowable soil pressure so small that individual footings would either overlap or cover more than about one half of the area, it may be better to provide a continuous footing under all columns & walls. Such a footing is called a raft or mat foundation.

→ Raft foundations are also used to reduce the settlement of structures located above highly compressible deposits.

* Method of Analysis :-

→ The essential task in the analysis of a raft foundation is the determination of the distribution of contact pressure, which is a complex function of rigidity of superstructure, raft itself & supporting soil.

- Rigid foundation
  - → Conventional Method.
- Flexible foundation
  - → Simplified Method
  - → General Method.

Q. Design a raft foundation for the layout as shown in fig. Net bearing capacity of soil is 65 kN/m² & column size is 30cm * 30cm. Use M30 Concrete & Fe500 steel.



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Total vertical column load = (600 + 1500×2 + 500) + (600 + 2000×2 + 1200) + (550 + 1200×2 + 550)

P_u = 13300 kN

Centroid of area of raft,

$\bar{x}_0$ (from grid 1-1) = 7 + 3.5 = 10.5 m

$\bar{y}_0$ (from grid C-C) = 6 m

Centroid of column loads,

$\bar{x}_L$ (from grid 1-1) = $\frac{(1500+2000+1200)\times 7 + (1500+2000+1200)\times 14 + (500+1200+550)\times 21}{13300}$

= 10.973 m

$\bar{y}_L$ (from grid C-C) = $\frac{(600+2\times 2000+1200)\times 6 + (500+1500\times 2+500)\times 12}{13300}$

= 6.225 m

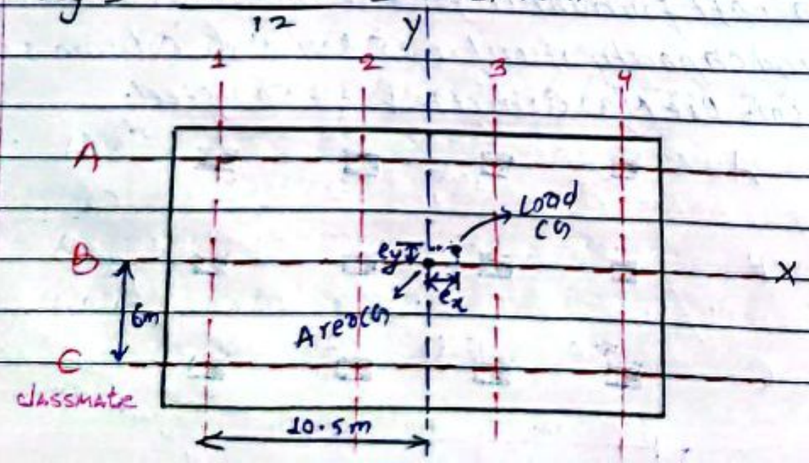
Eccentricity,

$e_x = \bar{x}_0 - \bar{x}_L = 10.5 - 10.973 = -0.473$ m

$e_y = \bar{y}_0 - \bar{y}_L = 6 - 6.225 = -0.225$ m

$I_x = \frac{21.6\times 12.6^3}{12} = 3600.67\ m^4$ Size of raft = 21.6m × 12.6m

$I_y = \frac{12.6\times 21.6^3}{12} = 10581.58\ m^4$



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$A = L \times B = 21.6 \times 12.6 = 272.16\ m^2$

$M_y = P_u \times e_x = 13300 \times 0.473 = 6290.9$ kNm

$M_x = P_u \times e_y = 13300 \times 0.225 = 2992.5$ kNm

The soil pressure at different point is,

$$\sigma = \frac{P}{A} \pm \frac{M_y}{I_y} * x \pm \frac{M_x}{I_x} * y \quad \text{---} (1)$$

$\frac{P}{A} = \frac{13300}{272.16} = 48.86\ kN/m^2$

$M_y/I_y = \frac{6290.9}{10581.58} = 0.5945$

$M_x/I_x = \frac{2992.5}{3600.67} = 0.831$

Then, eqn (1) becomes, $\sigma = 48.86 \pm 0.5945x \pm 0.831y$

(a) Corner A-4 (10.8, 6.3), $\sigma_{A4} = 60.51\ kN/m^2$

(b) Corner C-4 (10.8, -6.3), $\sigma_{C4} = 50.04\ kN/m^2$

(c) Corner A-1 (-10.8, 6.3), $\sigma_{A1} = 47.67\ kN/m^2$

(d) Corner C-1 (-10.8, -6.3), $\sigma_{C1} = 37.204\ kN/m^2$

(e) Edge B-4 (10.8, 0), $\sigma_{B4} = 55.28\ kN/m^2$

(f) Edge B-1 (-10.8, 0), $\sigma_{B1} = 42.44\ kN/m^2$

} $< 65\ kN/m^2$ (Ok)

→ Determine Maxm bending moment for tensile reinforcement

(i) Critical grid along major axis

Most critical strip is A-A grid beam with its width $6/2 + 0.3$ = 3.3m & stress on it is $\sigma_{A4} = 60.51\ kN/m^2$ & $L_x = 7$ m

$M_{ux} = \frac{\sigma L_x^2}{10} = \frac{60.51\times 7^2}{10} = 296.5$ kNm

(ii) Critical grid along minor axis,

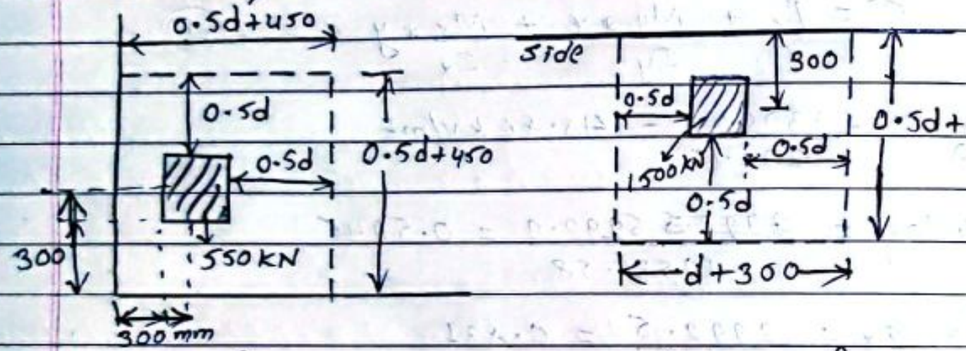
Most critical strip is 4-4 grid beam with its width $7/2 + 0.3 = 3.8$

classmate & stress on it is $\sigma_{A4} = 60.51\ kN/m^2$ & $L_y = 6$m PAGE

$M_{uy} = \frac{\sigma L_y^2}{8} = \frac{60.51\times 6^2}{8} = 272.3$ kNm

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→ The depth of raft will be governed by two way shear at the one of the exterior columns. In each case location of critical shear is not obvious, it may be necessary to check all possible locations.



Column of Corner C1 Column of Edge A-2

→ Shear strength of concrete; $\tau_c' = 0.25\sqrt{f_{ck}} = 0.25\sqrt{30}$
$= 1.37 N/mm^2$ --- (i)

For corner column (C-1)

$b_o = 2(d/2 + 450) = d + 900$

$\tau_v = \frac{V_u}{b_o d} = \frac{1.5 \times 550 \times 1000}{(d+900) \times d}$ --- (ii)

Equating (i) & (ii), $d = 447 mm$

For edge column (A-2)

$b_o = 2(0.5d + 450) + (d + 300) = 2d + 1200$

$\tau_v = \frac{1.5 \times 1500 \times 10^3}{(2d+1200) d}$ --- (iii)

Equating (i) & (iii), $d = 655 mm$.

Adopt effective depth of 670mm & overall depth of 720mm.

* Reinforcement in x-direction,

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

$\Rightarrow 295.5 \times 10^6 = 0.87 \times 500 \times A_{st} \times 670 \left(1 - \frac{500 \times A_{st}}{30 \times 1000 \times 670}\right)$

$\Rightarrow A_{st} = 1041 mm^2$

$A_{st,min} = \frac{0.12}{100} \times 1000 \times 720 = 864 mm^2 < A_{st}$ (ok)

Spacing $= \frac{1000 \times \pi \times 16^2}{4 \times 1041} = 193.14 mm$

So, provide 16 mm ϕ bars @ 180 mm c/c spacing in x-direction at top & bottom.

* Reinforcement in y-direction,

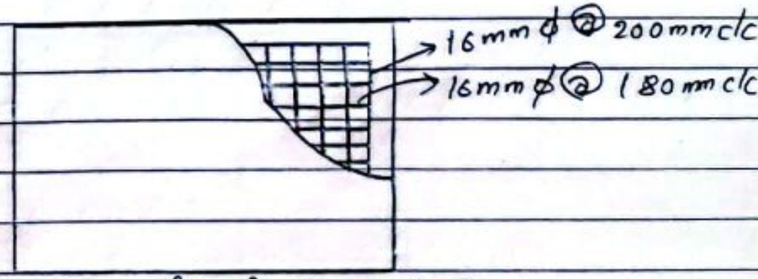
$\because M_y = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$

$\Rightarrow 272.3 \times 10^6 = 0.87 \times 500 \times A_{st} \times 670 \left(1 - \frac{500 \times A_{st}}{30 \times 1000 \times 670}\right)$

$\Rightarrow A_{st} = 957.1 mm^2 > A_{st,min}$

Spacing $= \frac{1000 \times \pi \times 16^2}{4 \times 957.1} = 210.07 mm$

Provide 16 mm ϕ bars @ 200 mm c/c in y-direction at top & bottom.



Plan of raft (top rebar)

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✱ Design of Columns:-

* Introduction:-

→ A column is defined as a vertical compression member which is mainly subjected to axial loads & the effective length of which exceeds three times its least lateral dimension.

→ Pedestal:- $l_{eff} < 3 *$ least lateral dimension

Strut: Inclined or horizontal member subjected to axial load

→ The column supports & receives the load from the beam & hence transfer the loads to the footing & then to the earth. Hence, collapse of column leads to collapse of whole structure also. So, design of column is very important & an engineer must give much importance to it.

* Classification of Columns:

1) Based on shape:-

- Square
- Rectangular
- Circular
- Pentagonal
- Hexagonal
- Octagonal
- T shape or L-shape etc.

2) Based on material of construction:-

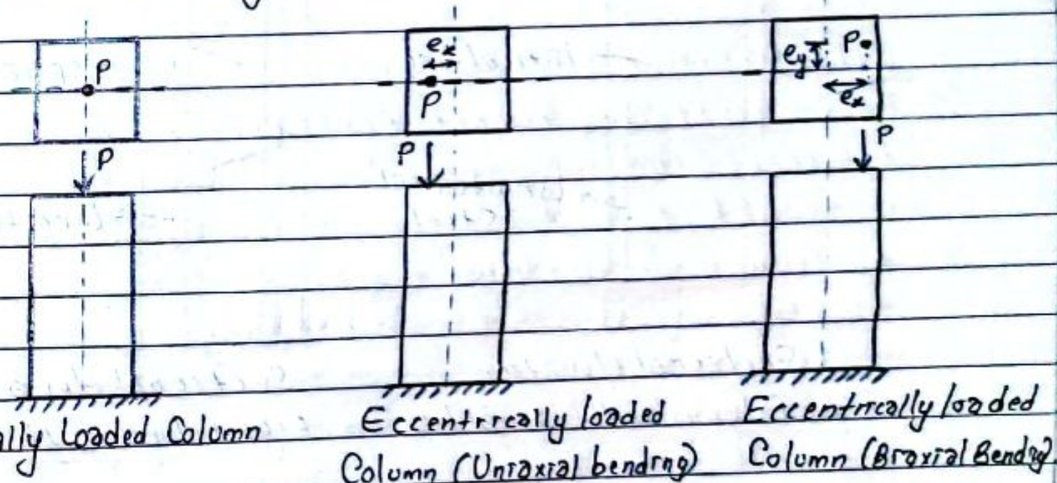
→ Timber columns
→ Masonry Columns
→ R.C.C. Columns
→ Steel Columns
→ Composite Columns

3) Based on type of loading

(i) Axially loaded Column:- columns which are subjected to loads acting along the longitudinal axis or centroid of column section. Column is subjected to direct compressive stress only.

(ii) Eccentrically loaded Columns:- Loads do not act on centroid of column. They are subjected to direct compressive stress & bending stress both.

- Uniaxial bending & axial load ($P_u + M_{ux}$ or $P_u + M_{uy}$)
- Biaxial bending & axial load ($P_u + M_{ux} + M_{uy}$)



Axially Loaded Column | Eccentrically loaded Column (Uniaxial bending) | Eccentrically loaded Column (Biaxial Bending)

4) Based on Slenderness Ratio:-

(a) Short columns: $SR \leq 12$
(b) Long columns: $SR > 12$

→ Short column generally fails by crushing of concrete due to axial force.
→ Long columns generally fail by bending effect than due to axial effect.

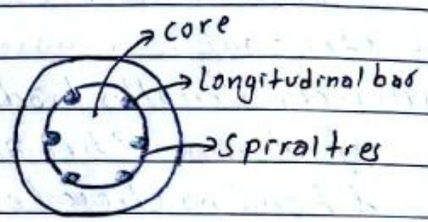
$$\text{Slenderness ratio} = \frac{l_{eff}}{\text{Least lateral dimension}}$$

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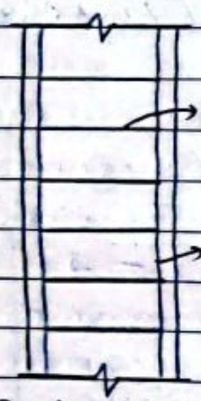
5) Based on pattern of lateral reinforcement:-
(a) Columns with lateral ties
(b) Columns with spiral ties.



Sectional plan

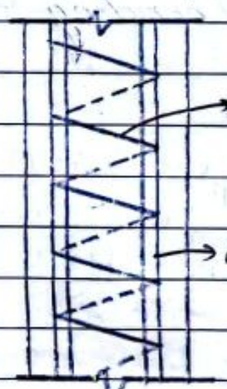


Sectional plan



Sectional elevation

Fig: Column with lateral ties



Sectional elevation

Fig: Column with helical ties.

* Reinforcement in column:-
→ A minimum area of steel is always provided in the column whether it is required for carrying load or not. It is done to resist tensile stresses which may be caused due to eccentricity of loads. It is practically impossible to have a perfectly axially loaded structure.
→ There are two types of reinforcements provided in RCC column.
a) Longitudinal rebar
b) Transverse rebar

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* Functions of longitudinal rebar:-
→ To share the compressive loads along with concrete, thus reducing the overall size of the column & leaving more usable area.
→ To resist tensile stresses developed due to any moment or accidental eccentricity.
→ To impart ductility to the column
→ To reduce the effect of creep or shrinkage due to continuous constant loading applied for long time.

* Functions of Transverse Reinforcement:-
→ To hold the longitudinal bars in position.
→ To prevent buckling of main longitudinal bars
→ To resist diagonal tension caused due to transverse shear developed because of any moment or load.
→ To impart ductility to the column
→ To prevent longitudinal splitting or bulging out of concrete by confining it in core.

* IS 456:2000 Specifications regarding Columns:-

1) Nominal Cover:-
→ The nominal cover for longitudinal reinforcing bar in a column shall not be less than any of following
a) 40mm
b) Dia. of bar
→ Also, it depends on exposure condition to meet the durability requirements.

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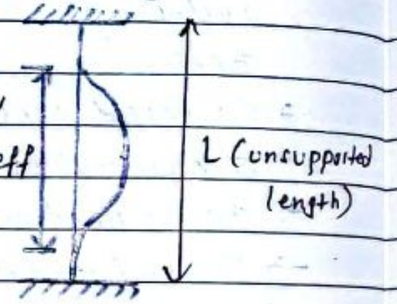
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2) Effective Length of Column:-

→ Length of column which takes part in buckling under the action of loads.

→ The effective length of a compression member depends on the end support condition & conditions are classified as 7 types. Leff for all cases are given in table 28 of IS456:2000.



3) Slenderness limit: (clause 25.3)

→ The unsupported length betn end restraints shall not exceed 60 times the least lateral dimension of a column

$$L \leq 60 * \text{Least lateral dimension.}$$

→ If in any given plane, one end of column is unrestrained, its unsupported length 'L' shall not exceed $\frac{100b^2}{D}$

where

b = width of member

D = Depth in respect to major axis.

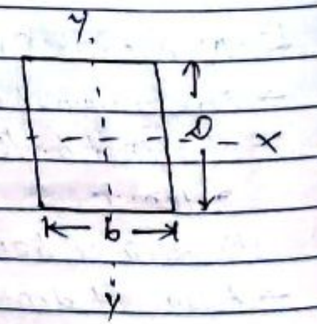
4) Minimum Eccentricity (cls 25.4)

$$e_{x,min} \geq \frac{L}{500} + \frac{D}{30}$$

$$\geq 20mm$$

$$e_{y,min} \geq \frac{L}{500} + \frac{b}{30}$$

$$\geq 20mm$$



where, L = unsupported length of column.

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* 5) Longitudinal Reinforcement. (cls 26.5.3.1)

→ The x-sectional area of longitudinal reinforcement shall not be less than 0.8% & not more than 6% of gross area of column.

→ The min^m number of longitudinal bars provided in a column shall be four in rectangular & six in circular column.

→ The bars shall not be less than 12mm in diameter.

→ A reinforced concrete column having helical reinforcement shall have at least six bars of longitudinal reinforcement within the helical reinforcement.

→ Spacing of longitudinal bars measured along the periphery of column shall not exceed 300mm.

→ In case of pedestals in which the longitudinal reinforcement is not taken in account in strength calculations, nominal longitudinal reinforcement not less than 0.15% of cross-sectional area shall be provided.

6) Transverse Reinforcement

→ The lateral or transverse reinforcement in a column may be provided in the form of lateral ties or spirals or helical reinforcement.

→ The IS code recommendations for lateral reinforcement are as follows

i) Diameter of lateral ties:

$$\phi_{\text{lateral ties}} \geq 1/4 * \phi \text{ largest longitudinal bar}$$

$$\geq 6mm$$

(ii) Pitch of lateral ties: shall not be more than least of following

- least lateral dimension of column
- 16 * dia. of smallest longitudinal bar
- 300mm

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(iii) Diameter of spiral/helical reinforcement should be greater than
- $^1/_4$th of diameter of largest longitudinal bar.
- 6mm

(iv) pitch of helical/spiral reinforcement shall not be more than
- 75mm
- $^1/_6$* Core diameter of column

pitch of helical rebar shall not be less than
- 25mm
- 3 times the diameter of steel bar forming the helix.

→ For arrangement of transverse reinforcement, see clause 26.5.3.2(b).

* Design steps for axially loaded short Columns:-

1) Calculate design load, $P_u = 1.5 \times P$ (service load)
2) Assume % of steel (0.8% to 4% of Ag)
   Generally assume 2%.
3) Calculate size of column by assuming cross section (Square / Rectangular ($D = 1.5b$), circular with ~~lateral~~ ~~helical~~ tres etc.) using,

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \quad (cls\ 39.3)$$

For circular column with helical tres,

$$P_u = 1.05 [0.4 f_{ck} A_c + 0.67 f_y A_{sc}] \quad (cls\ 39.4)$$

4) Check slenderness ratio & minimum eccentricity
- Check for short column

$$\frac{l_{ex}}{D} < 12, \quad \frac{l_{ey}}{b} < 12$$

where, l_{ex} & l_{ey} are effective length/ height of column along x & y direction.

- Check for axially loaded column

$e_{xmin} = \frac{L}{500} + \frac{D}{30}$ or 20mm whichever greater $\neq 0.05D$

$e_{ymin} = \frac{L}{500} + \frac{b}{30}$ or 20mm whichever greater $\neq 0.05b$ (cls 25.4)

5) Calculate area of steel by using,

A_{sc} = % of steel assumed * A_{gross}

& number of bar by assuming dia of longitudinal bar

$n = \frac{A_{sc,req}}{A_{\phi,long}}$ > 4 for rectangular column
> 6 for circular column

6) Design of tres

(a) For lateral tres
- Dia of lateral tres, ϕ_{lat} = $\phi_{largest\ long.bar}/4$ or 6mm whichever greater.
- Spacing/pitch:- least lateral dimension of column, 16 * small dia of longitudinal bar or 300mm whichever is less.

(b) For Helical tres
- pitch of helical reinforcement

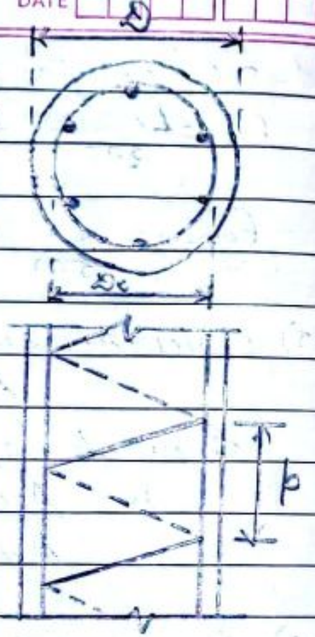
Maximum pitch $\neq$ 75mm or $D_c/6$ whichever is less
Minimum pitch $\neq$ 25mm or 3 * dia of spiral (ϕ_s) whichever is less.

From clause 39.4.1,

$$\frac{\text{volume of helical reinforcement}}{\text{Volume of Core}} > 0.36 \left(\frac{A_g}{A_c} - 1\right) \times \frac{f_{ck}}{f_y}$$

Where, A_g = Gross area of section
A_c = Area of core of helically reinforced column measured to the outside diameter of helix.

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Here,

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

$A_c = \frac{\pi}{4} \times D_c^2$

D_c = D - 2 * end cover

D' = D - 2 * (end cover) - ϕ_s

→ Volume of helical reinforcement per mm height of column

= $\frac{\text{circumference of spiral} * \text{Area of spiral}}{\text{pitch of spiral}}$

= $\frac{\pi \times D' \times \pi/4 \times \phi_s^2}{p}$

→ Volume of core per mm height of column = $\frac{\pi}{4} \times D_c^2 \times 1$

2/ Design a column of size 450 mm * 600 mm & having 3m unsupported length. The column is subjected to a load of 2000kN and is effectively held in position but not restrained against rotation. Use M20 concrete & Fe415 steel.

→ Given,

b = 450 mm f_{ck} = 20 N/mm²

D = 600 mm f_y = 415 N/mm²

P_u = 2000 kN × 1.5

= 3000 kN

Effective length (Leff) = 1L [Table 28]

= 1 × 3

= 3m

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Slenderness ratio = $\frac{\text{Leff}}{\text{Least lateral dimension}} = \frac{3000}{450} = 6.67 < 12$

Hence, it is a short column.

→ Minimum eccentricity (e_{min})

$e_{x,min} = \frac{L}{500} + \frac{D}{30} = \frac{3000}{500} + \frac{600}{30} = 26\text{mm} > 20\text{mm}$

$\therefore e_{x,min} = 20\text{mm} \leq 0.05D = 0.05 \times 600 = 30\text{mm}$ (ok)

$e_{y,min} = \frac{L}{500} + \frac{b}{30} = \frac{3000}{500} + \frac{450}{30} = 21\text{mm} > 20\text{mm}$

$e_{y,min} = 20\text{mm} \leq 0.05b = 0.05 \times 450 = 22.5\text{mm}$ (ok)

→ Area of longitudinal reinforcement

$A_g = 450 \times 600 = 270000\ \text{mm}^2$

$A_c = A_g - A_{sc} = 270000 - A_{sc}$

For axially loaded short column,

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

$3000 \times 10^3 = 0.4 \times 20 \times (270000 - A_{sc}) + 0.67 \times 415 \times A_{sc}$

$A_{sc} = 3110.5\ \text{mm}^2$

% of reinforcement provided = $\frac{3110.5}{270000} \times 100 = 1.152\%$.

(In bet^n 0.8 to 4%) Ok.

Using 4-25mm ϕ bars = 4 × 490.6 = 1962.5 mm²

4-20 mm ϕ bars = 4 × 314 = 1256 mm²

$A_{sc,provided}$ = 3218.5 mm² > $A_{sc,req}$ (Ok)

→ Design of lateral ties

The diameter of lateral ties should be,

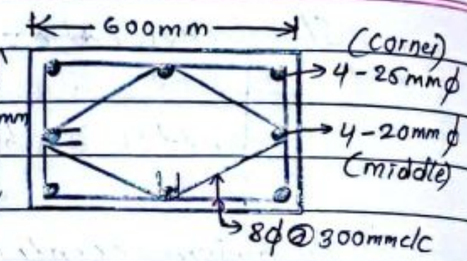
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greater than

(i) $\frac{1}{4} \times 25 = 6.25$ mm

(ii) 6 mm



So, use 8 mm dia. tres.

The pitch of tres shouldnot exceed the following

i) Least lateral dimension = 450 mm

ii) $16 * \phi = 16 \times 20 = 320$ mm

iii) 300 mm

Provide 8 mm ϕ tres @ 300 mm c/c.

The arrangement of reinforcement is shown in figure.

Q Design an axially loaded column of circular section for axial compressive load, $P_u = 3000$ kN, unsupported length (L) = 3.25m, effective length along major & minor axis are $l_{ex} = 3$m & $l_{ey} = 2.75$m resp. Use M20 concrete & Fe415. Use spiral tres for circular column.

→ Given,

Factored load (P_u) = 3000 kN

Unsupported length (L) = 3.25m

Effective length along major axis (l_{ex}) = 3m

Effective length along minor axis (l_{ey}) = 2.75m

$f_{ck} = 20$ N/mm²

$f_y = 415$ N/mm²

Assume, percentage of steel = 2%

$A_{sc} = 0.02 A_g$

$A_c = 0.98 A_g$

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Now, for a column with helical tres,

$$P_u = \frac{0.4 f_{ck} A_c + 0.67 f_y A_{sc}}{1.05}$$

$$\Rightarrow 3000 \times 10^3 = 1.05 [0.4 \times 20 \times 0.98 A_g + 0.67 \times 415 \times 0.02 A_g]$$

$$\Rightarrow A_g = 213203.7 \text{ mm}^2$$

Size of column, $A_g = \frac{\pi D^2}{4} \Rightarrow D = \sqrt{\frac{4A_g}{\pi}} = 521.02$ mm

Adopt, drameter of circular column (D) = 530 mm

→ Check for slenderness ratio:-

$\frac{l_{ex}}{D} = \frac{3000}{530} = 5.66 < 12$

$\frac{l_{ey}}{D} = \frac{2750}{530} = 5.18 < 12$

} Short column

$e_{x\min} = e_{y\min} = \frac{L}{500} + \frac{D}{30} = \frac{3250}{500} + \frac{530}{30} = 24.17$ mm > 20mm

So, $e_{x\min} = e_{y\min} = 20$ mm $\leq 0.05D = 0.05 \times 530 = 26.5$ mm (OK)

→ Area of Reinforcement,

$A_{sc} = 0.02 \times \frac{\pi \times 530^2}{4} = 4412 \text{ mm}^2$

Provide 8 − 28 mm ϕ bar,

$A_{sc,pro} = \frac{\pi \times 28^2}{4} \times 8 = 4926 \text{ mm}^2$

$A_{sc,min} = 0.8\%$ of $A_g = \frac{0.8}{100} \times \frac{\pi \times 530^2}{4} = 1765 \text{ mm}^2$

$A_{sc,max} = 4\%$ of $A_g = \frac{4}{100} \times \frac{\pi \times 530^2}{4} = 8825 \text{ mm}^2$

$A_{sc\min} < A_{sc,pro} < A_{sc,max}$ (OK)

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Minimum no. of bar = 6 but provided = 8 (ok)
Minm dia of bar = 12mm but provided = 28 mm (ok)

→ Design of Helical Reinforcement:-

(a) Dia of spiral tres,
→ $\phi_t = \frac{1}{4} \times 28 = 7mm$
→ $\phi_t = 6mm$
∴ provide 8 mm spiral tres.
Assume clear cover = 40mm

(b) Spacing (pitch) of helical tres)
Diameter of core (D_c) = 530 − 2×40 = 450mm
$D' = 450 - 8/2 - 8/2 = 442mm$

Volm of helical reinforcement/unit height = $\frac{\pi D' \times \pi \times \phi_s^2/4}{p}$

$= \frac{\pi \times 442 \times \pi \times 8^2/4}{p} = \frac{69797.84}{p}$

Volume of core/height = $\frac{\pi D_c^2}{4} \times 1 = 159043.13 mm^2/m$

Now, $\frac{69797.84}{159043.13p} \geq 0.36\left(\frac{A_g}{A_c} - 1\right) \times \frac{f_{ck}}{f_y}$

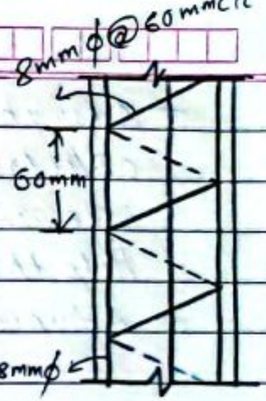
$\Rightarrow \frac{0.4388}{p} \geq 0.36\left(\frac{\pi/4 \times 530^2}{\pi/4 \times 450^2} - 1\right) \times \frac{20}{415}$

$\Rightarrow p \leq 65.32mm$

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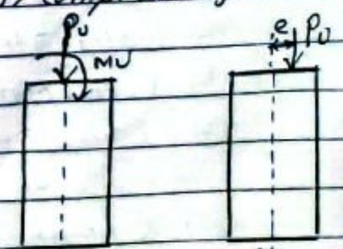
Maxm pitch = 75mm or 450/6 = 75mm
∴ p_{max} = 75mm
Minm pitch = 25mm or 3×8 = 24mm
∴ p_{min} = 24 mm
∴ provide 8mmϕ spiral tres @ 60mm c/c.



Column subjected to axial compression & uniaxial bending Moment

→ As per IS456:2000, all columns should be designed for minimum eccentricity of load as it is not possible to have perfectly axial load.
→ Due to imperfections in construction & non-homogeneity of materials some eccentricity will always be there which will result in bending of column.
→ If $e_{min} \leq 0.05D$ or $0.05b$, column is designed as axially loaded column but if $e_{min} > 0.05D$ or $0.05b$, some moment is coming on section, then column should be designed for bending in addition to compression.
→ The maxm compressive strain at the highly compressed extreme fibre in concrete subjected to axial compression & bending & when there is no tension on section shall be 0.0035 − 0.75* strain at the least compressed fibre.

$\epsilon_{max} = 0.0035 - 0.75 * \epsilon_{min}$



→ Consider a column subjected to axial load P_u & Moment M_u. The effect of these two forces is equivalent to load P_u acting at eccentricity e' such that,

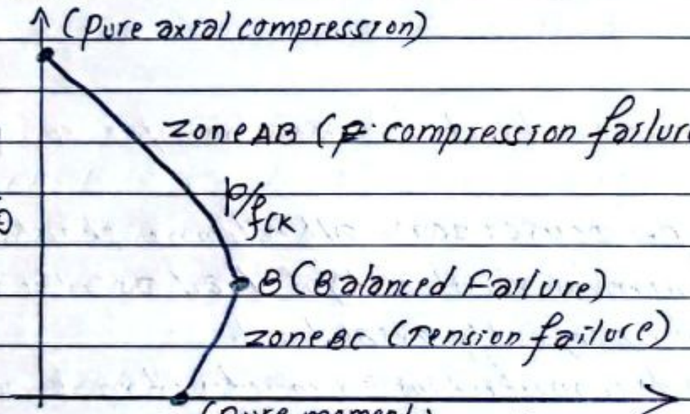
$M_u = P_u * e$

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→ Design of such column involve complicated & lengthy calculation. Based on exact analysis of such column, interaction curves have been developed. These curves help the designer & are given in SP-16.

→ A typical interaction curve is shown below.



→ These charts are given in SP:16 for rectangular and circular columns subjected to axial compression & bending. Charts have been given for three grades of steel & four values of d'/d.

* Design steps for column subjected to compression & Uniaxial bending

① Calculate d'/D & based on d'/D & fy select the chart to be used.

② Calculate Pu/fck b D & Mu/fck b D²

③ Corresponding to these values, get the value of p/fck

④ Calculate Asc & provide bars. $A_{sc} = \frac{p}{100} \times A_g$

⑤ Design lateral tres as previous.

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Q Design an reinforced concrete square column of 500mm side to carry an ultimate load of 2000kN at an eccentricity of 180mm. Use M20 concrete & Fe415 steel.

→ Given,

$b = D = 500\,mm$ $\quad f_{ck} = 20\,N/mm^2$

$P_u = 2000\,kN$ $\quad f_y = 415\,N/mm^2$

$e = 180\,mm$

Assuming: effective cover (d') = 50mm

Now,

$\frac{P_u}{f_{ck} b D} = \frac{2000 \times 10^3}{20 \times 500 \times 500} = 0.2$

$\frac{M_u}{f_{ck} b D^2} = \frac{2000 \times 10^3 \times 180}{20 \times 500 \times 500^2} = 0.144$

Also, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 32, $\frac{p}{f_{ck}} = 0.065 \quad \therefore p = 0.065 \times 20 = 1.3\%$

$A_{sc} = \frac{1.3}{100} \times 500 \times 500 = 3250\,mm^2$

Using 4-25mm drameter bars & 2-30 mm drameter bars (2-25mm bars & 1-30 mm bars on each face),

$A_{sc,pro} = 4 \times \frac{\pi}{4} \times 25^2 + 2 \times \frac{\pi}{4} \times 30^2 = 3377\,mm^2$

→ Design of lateral tres.

(a) Drameter of tres,

- 6mm
- $\phi_L/4 = 30/4 = 7.5mm$.

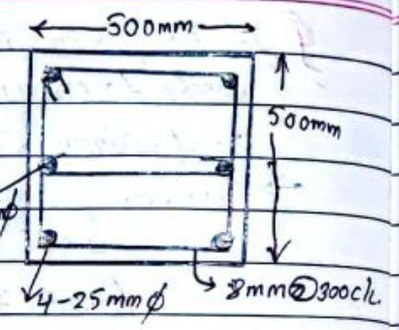
So, provide 8 mm ϕ tres.

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(b) pitch of tres,

- least lateral dimension = 500 mm
- 16 * $\phi_{smaller}$ = 16 x 25 = 400 mm
- 300 mm.

provide 8mm tres @ 300 mm c/c.



Q Design the reinforcement for a circular column of diameter 500mm subjected to an ultimate load of 1600kN & an Ultimate ~~load~~ Moment of 125 kNm about the major axis. Use M20 concrete & design helical reinforcement. (Fe250-steel)

→ Given,

diameter of column (D) = 500 mm

Ultimate load (P_u) = 1600 kN

Ultimate Moment (M) = 125 kNm

f_{ck} = 20 N/mm²

f_y = 250 N/mm²

Assume, effective cover (d') = 50 mm

$\frac{d'}{D} = \frac{50}{500} = 0.1$

$\frac{P_u}{1.05 f_{ck} b D} = \frac{P_u}{1.05 f_{ck} D^2} = \frac{1600 \times 10^3}{1.05 \times 20 \times 500^2} = 0.305$

$\frac{M_u}{1.05 f_{ck} D^3} = \frac{125 \times 10^6}{1.05 \times 20 \times 500^3} = 0.048$

For f_y = 250 N/mm² & d'/D = 0.1, chart 52 is to be preferred.

For $\frac{P_u}{f_{ck} D^2} = 0.305$ & $\frac{M_u}{f_{ck} D^3} = 0.048$

$p/f_{ck} = 0.08$

$p = f_{ck} \times 0.08 = 20 \times 0.08 = 1.6\%$

$A_{sc} = \frac{1.6}{100} \times \frac{\pi}{4} \times 500^2 = 3142 mm^2$

Provide 4-25mm bars & 4-20mm dia bars symmetrically,

$\therefore A_{sc,pro} = 4 \times \frac{\pi \times 25^2}{4} + 4 \times \frac{\pi \times 20^2}{4} = 3220.13 mm^2$

→ Design of Helical Reinforcement:

(a) Dia of spiral tres,

- 1/4 * 25 = 6.25 mm
- 6 mm

So, provide 8mm dia bars for helix.

Here,

$D_c = 500 - 2 \times (50 - 4) = 408$ mm

$D' = 408 - 8 = 400$ mm

Volᵐ of helical reinforcement/unit height = $\frac{\pi D' \times \pi \times \phi_s^2/4}{p}$

$= \frac{\pi \times 400 \times \pi \times 8^2/4}{p} = 63165.4/p$

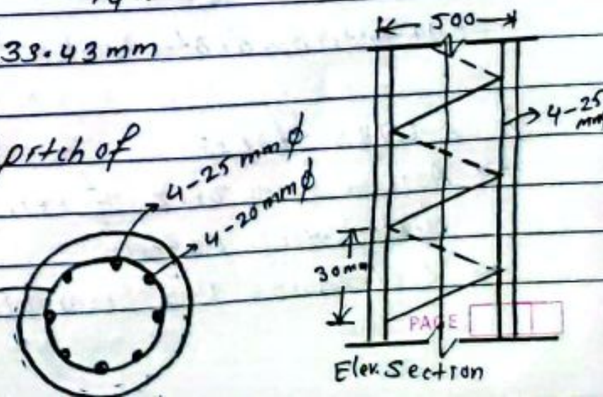
Volume of core/height = $\frac{\pi \times D_c^2}{4} \times 1 = \frac{\pi}{4} \times 408^2 = 130740.5$

$\frac{63165.4}{p \times 130740.5} \geq 0.36 \times \left(\frac{\pi/4 \times 500^2}{\pi/4 \times 408^2} - 1\right) \times \frac{20}{250}$

$p < 33.43$ mm

Minᵐ pitch = 25 mm

So, provide 8mm ϕ helix @ pitch of 30 mm c/c.



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✻ Design of columns Subjected to combined axial load & biaxial bending moment:-

→ Exact design of columns subjected to combined axial load and biaxial bending moments is very complicated & extremely laborious. To simplify the process, IS 456 permits the design of such columns by following eqn (clause 39.6)

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$$

Where, M_{ux} & M_{uy} are moments about x & y axes resp. due to design loads.

M_{ux1} & M_{uy1} are max^m uniaxial moment capacities of the column with an axial load P_u, bending about x & y resp.

$$\alpha_n = \frac{P_u}{P_{uz}}$$

where, $P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$

(chart 63 of design aids can be used to determine P_{uz})

Also,

P_u/P_{uz}	α_n
≤ 0.2	1
≥ 0.8	2

For intermediate value, linear interpolation may be done.

✻ Design Steps:-

→ Assume some area of reinforcement along with its distribution pattern.

→ Determine P_{uz} from chart 63 of sp16 or from following equation, $P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$

→ Find $\frac{P_u}{P_{uz}}$ ratio

→ Determine M_{ux1} & M_{uy1}, the ultimate uniaxial moment capacities of column with an axial load P_u from suitable design chart from SP16.

→ Calculate $\frac{M_{ux}}{M_{ux1}}$ & $\frac{M_{uy}}{M_{uy1}}$ | ⇒ check $\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$ & if ok calculate % of steel. Below steps are not necessary.

→ From chart 64, find the permissible value of $\frac{M_{ux}}{M_{ux1}}$ corresponding to $\frac{M_{uy}}{M_{ux}}$ & $\frac{P_u}{P_{uz}}$

→ If $\frac{M_{ux}}{M_{ux1}}$ is less than $\frac{M_{ux}}{M_{ux1}}$ value found from chart 64, the section is safe & design is adopted.

→ If $\frac{M_{ux}}{M_{ux1}}$ is more than $\frac{M_{ux}}{M_{ux1}}$ value found from chart 64, the section is unsafe & hence it is to be redesigned.

✻ Design a short column of 400mm * 600mm carrying factored axial load 1600kN, factored moment along major axis is 120kNm and along minor axis is 90kNm. Take M25 grade of Concrete & Fe415 steel. $d' = 60$ mm.

→ Given,

$b = 400$ mm, $D = 600$ mm

$P_u = 1600$ kN

$M_{ux} = 120$ kNm

$M_{uy} = 90$ kNm

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

$f_y = 415$ N/mm^2

$d' = 60$ mm

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Here, given column is short. so, no need to calculate slenderness ratio.

→ Check for minimum eccentricity

$$e_{x,cal} = \frac{M_{ux}}{P_u} = \frac{120\times10^6}{1600\times1000} = 75\text{mm}$$

$e_{minx} = 20\text{mm} > 0.05D\ (0.05\times600) = 30\text{mm}$

$\therefore e_{minx} = 20\text{mm}$

$e_{x,cal}\ (75\text{mm}) > e_{minx}\ (30\text{mm})$

Again,

$$e_{y,cal} = \frac{M_{uy}}{P_u} = \frac{90\times10^6}{1600\times1000} = 56.25\text{mm}$$

$e_{miny} = 20\text{mm} > 0.05b\ (0.05\times400) = 20\text{mm}$

$\therefore e_{miny} = 20\text{mm}$

Check, $e_{y,cal}\ (56.25\text{mm}) > e_{min,y}\ (20\text{mm})$

→ Assume % of steel = 1.5% & calculate M_{ux1} by using interaction chart,

$$\frac{d'}{D} = \frac{60}{600} = 0.1$$

$$\frac{p}{f_{ck}} = \frac{1.5}{20} = 0.075$$

$$\frac{P_u}{f_{ck}bD} = \frac{1600\times1000}{20\times400\times600} = 0.333$$

Type of steel ($f_y = 415\text{MPa}$) & reinforcement distributed equally on four side then,

$$\frac{M_{ux1}}{f_{ck}bD^2} = 0.112 \text{ from chart 44}$$

$$M_{ux1} = 0.112\times20\times400\times600^2\times10^{-6}$$

$$= 322.56\text{ kNm}$$

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→ Calculate M_{uy1} by using interaction chart,

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

$$\frac{p}{f_{ck}} = \frac{1.5}{20} = 0.075$$

$$\frac{P_u}{f_{ck}bD} = 0.333$$

Type of steel ($f_y = 415\text{MPa}$) & reinforcement distributed equally on four side then,

$$\frac{M_{uy1}}{f_{ck}bD^2} = 0.1 \text{ from chart 45}$$

$\therefore M_{uy1} = 0.1\times20\times600\times400^2 = 192\text{ kNm}$

→ Calculate α_n which depends on P_u/P_{uz} value,

$P_u = 1600\text{ kN}$

$P_{uz} = 0.45 f_{ck}A_c + 0.75 f_y A_{sc}$

$= 0.45\times20\times(0.985\times400\times600) + 0.75\times415\times(0.015\times400\times600)$

$= 3229.188\text{ kN}$

$$\frac{P_u}{P_{uz}} = \frac{1600}{3229.188} = 0.495$$

0.2	1
0.495	$\alpha_n = ?$
0.8	2

$\alpha_n = 1.491$

→ Check by using,

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \le 1$$

$$\Rightarrow \left(\frac{120}{322.56}\right)^{1.491} + \left(\frac{90}{192}\right)^{1.491} = 0.552 < 1 \text{ (ok)}$$

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→ Calculate area of longitudinal bar,

$$A_{sc} = 1.5\% \, bD$$
$$= \frac{1.5}{100} \times 400 \times 600$$
$$= 3600 mm^2$$

Calculate no. of bar, $(n) = \frac{3600}{\left(\frac{\pi \times 20^2}{4}\right)} = 11.46 \approx 12$

$A_{sc,min} = 0.8\%$ of $bD = 1920 mm^2$
$A_{sc,max} = 4\%$ of $bD = 9600 mm^2$
$A_{sc,min} < A_{sc,pro} < A_{sc,max}$ (ok)

→ Design of lateral ties

diameter of lateral ties
- $\frac{1}{4} \times 20 = 5mm$
- 6 mm

provide 8 mm ϕ lateral ties.

pitch of lateral ties
- Least lateral dimension = 400 mm
- $16 \times 20 = 320 mm$
- 300 mm.

so, provide 8 mm ϕ lateral ties @ 300 mm c/c.



sectional plan

Design of long Columns (Slender Column)

→ long or slender columns are those columns whose slenderness ratio (l_{ex}/D or l_{ey}/b) is greater or equal to 12. (clause 25.1.2)

→ The ultimate strength of a long slender column is less as compared to that of a short column with same cross section & reinforcement because the slender column is liable to buckle & undergoes more pronounced lateral deflection which causes additional secondary moments in the column.

→ Therefore, IS 456 recommends that for design of slender columns, an additional moment (M_{ax} & M_{ay}) given by,

$$M_{ax} = P_u \times \frac{D}{2000} \left(\frac{l_{ex}}{D}\right)^2 = P_u \times e_{ax} \quad \text{[clause 39.7.1]}$$

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

→ The values given by eqn 39.7.1 may be multiplied by following factor

$$K = \frac{P_{uz} - P_u}{P_{uz} - P_b} \leq 1$$

where, P_u = Axial load on compression member

$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$

P_b = Axial load corresponding to the condition of maximum compressive strain of 0.0035 in concrete & tensile strain of 0.002 in outermost layer of tension steel.

'P_b' is determined by using following eqns,

$$\frac{P_b}{f_{ck} bD} = k_1 + k_2 \times \frac{p}{f_{ck}} \quad \text{for rectangular section}$$

$$\frac{P_b}{f_{ck} D^2} = k_1 + k_2 \times \frac{p}{f_{ck}} \quad \text{for circular sections.}$$

value of k_1 & k_2 is given in Table 60 of SP 16.

→ The final design moments are obtained by adding the additional moments to the initial moments.

Design Steps for slender Column :-

1) Calculate slenderness ratio & check for short or long column.

$$\lambda_x = \frac{l_{ex}}{D} \quad \& \quad \lambda_y = \frac{l_{ey}}{b}$$

Check it with cls 25.1.2, IS 456-2000

If $\lambda_x \geq 12$, column is slender about x-axis

$\lambda_y \geq 12$, column is slender about y-axis

2) Calculate minm eccentricity & moment due to eccentricity.

$$e_{x,min} = \frac{L}{500} + \frac{D}{30} \text{ or } 20\text{mm whichever is greater}$$

& $M_{ux,e} = P_u * e_{x,min}$

Again,

$$e_{y,min} = \frac{L}{500} + \frac{b}{30} \text{ or } 20\text{mm whichever is greater}$$

& $M_{uy,e} = P_u * e_{y,min}$

3) Assume % of reinforcement (p_t %), then calculate area of reinforcement (A_{sc}) & area of concrete.

Assume 2% of reinforcement, then $A_{sc} = 0.02 A_g$ & $A_c = 0.98 A_g$.

4) a) Calculate moment about x-axis & about y-axis a/c to cls 39.7.1, IS 456:2000

$$M_{ax} = P_u \times \frac{D}{2000}\left(\frac{l_{ex}}{D}\right)^2 \quad \& \quad M_{ay} = P_u \times \frac{b}{2000}\left(\frac{l_{ey}}{b}\right)^2$$

b) Calculate reduction factor ($k_{x,y}$) a/c to cls 39.7.1.1,

$$k_{x,y} = \frac{P_{uz} - P_u}{P_{uz} - P_b} \leq 1$$

Where, $P_{uz} = 0.446 f_{ck} A_c + 0.75 f_y A_{sc}$

$P_b = \left(k_1 + k_2 \times \frac{p}{f_{ck}}\right) \times f_{ck} b D$ for rectangular column

$P_b = \left(k_1 + k_2 \times \frac{p}{f_{ck}}\right) \times f_{ck} D^2$ for circular column

Where, k_1 & k_2 are constant taken from table 60 of sp 16 which depends on the value of d'/D & d'/b for x & y axis resp. shape of x-section of column, f_y & arrangement of rebar.

c) Calculate adjusted (reduced) additional moment about x-axis & y-axis,

$M'_{ax} = k_x \times M_{ax}$

$M'_{ay} = k_y \times M_{ay}$

5) Calculate initial moments or intermediate moment (M_i) a/c to cls 39.7 (notes).

(a) Initial Moment for braced Column

1) For single curvature,

$M_{uxi} = 0.6 M_{ux2} + 0.4 M_{ux1} \geq 0.4 M_{ux2}$

$M_{uyi} = 0.6 M_{uy2} + 0.4 M_{uy1} \geq 0.4 M_{uy2}$

2) For double curvature,

$M_{uxi} = 0.6 M_{ux2} - 0.4 M_{ux1} \geq 0.4 M_{ux2}$

$M_{uyi} = 0.6 M_{uy2} - 0.4 M_{uy1} \geq 0.4 M_{uy2}$

(b) Initial Moment for unbraced Column

$M_{uxi} = M_{ax}(M_{ux1}, M_{ux2})$

$M_{uyi} = M_{ax}(M_{uy1}, M_{uy2})$

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6) Calculate total design moments

1) For braced System

$M_{ux} = M_{ox}(M_{ux1}, M_{ux,e}) + M'_{ox} \leq M_{ux2}$

$M_{uy} = M_{ox}(M_{uy1}, M_{uy,e}) + M'_{oy} \leq M_{uy2}$

2) For unbraced System

$M_{ux} = M_{ox}(M_{ux1}, M_{ux2}) + M'_{ox}$

$M_{uy} = M_{ox}(M_{uy1}, M_{uy2}) + M'_{oy}$

7) Calculate moment resisting capacity (M_{ux1} & M_{uy1}) similar way as biaxial bending design.

8) Calculate α_n which depends on $\frac{P_u}{P_{uz}}$ value.

where,

P_u = Given Axial load

$P_{uz} = 0.446 f_{ck} A_c + 0.75 f_y A_{sc}$

9) Check

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$$

- If not redesigned by increasing the % of steel & repeat step 3 to 9.

10) Calculate area of longitudinal bar by using $A_{sc} = p_c \times b \times D$ & calculate no. of bar provided then check min (0.8% of A_g) $< A_{sc}$ provided $< 4\%$ of A_g.

11) Design of lateral ties.

i) dia of lateral ties :- Greater of $\phi_{largest}/4$ or 6mm

ii) Spacing: Min^m of least lateral dimension, $16\phi_{smallest}$ or 300 mm

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8 Determine the reinforcement required for a column which is restrained against sway which is in double curvature, with the following data.

Size of column = 40 cm x 30 cm, Concrete Grade = M 30

Characteristic strength of reinforcement = 415 N/mm^2

Effective length for bending parallel to larger dimension (l_{ex}) = 6m

Effective length for bending parallel to shorter dim. (l_{ey}) = 5m

Unsupported length = 7m, Factored load = 1500kN

Factored moment in the direction of larger dimension: 40 kNm at top & 22.5 kNm at bottom.

Factored moment in the direction of shorter dimension 30 kNm at top & 20 kNm at bottom.

→

Step 1: Calculate slenderness ratio & check for short column/long column

$$\lambda_x = \frac{l_{ex}}{D} = \frac{6000}{400} = 15 > 12$$

$$\lambda_y = \frac{l_{ey}}{b} = \frac{5000}{300} = 16.67 > 12$$

Therefore, the column is slender about both axes.

Step 2: Calculate Min^m eccentricity & Moment due to eccentricity

$$e_{xmin} = \frac{L}{500} + \frac{D}{30} = \frac{7000}{500} + \frac{400}{30} = 27.33\,mm$$

$M_{ux,e} = P_u * e_{xmin} = 1500 \times 0.0273 = 40.95\,kNm$

$$e_{ymin} = \frac{L}{500} + \frac{b}{30} = \frac{7000}{500} + \frac{300}{30} = 24\,mm$$

$M_{uy,e} = P_u * e_{ymin} = 1500 \times 0.024 = 36\,kNm$

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Step3: Assume, percentage of reinforcemen = 3%

$A_{sc} = 0.03A_g$ & $A_c = 0.97A_g$

Step4 a) Calculate moment about x-axis & y-axis,

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

$$= 1500 \times \frac{0.4}{2000}\left(\frac{6}{0.4}\right)^2$$

$$= 67.5 \text{ kNm}$$

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

$$= \frac{1500 \times 0.3}{2000}\left(\frac{5000}{300}\right)^2$$

$$= 62.5 \text{ kNm}$$

(b) Calculate reduction factor

$$k_{x,y} = \frac{P_{uz} - P_u}{P_{uz} - P_b} \leq 1$$

$$P_{uz} = 0.446 f_{ck} A_c + 0.75 f_y A_{sc}$$

$$= 0.446 \times 30 \times 0.97 \times 300 \times 400 + 0.75 \times 415 \times 0.03 \times 300 \times 400$$

$$= 2677.93 \text{ kN}$$

$P_b = \left(k_1 + k_2 \frac{p}{f_{ck}}\right) \times f_{ck} b D$ for rectangular section

Assume $d' = 60$ mm

For $d'/D = 60/400 = 0.15$

$k_{1x} = 0.196$ & $k_{2x} = 0.203$ [Table 60, SP16]

$$P_{bx} = \left(k_{1x} + k_{2x} \frac{p}{f_{ck}}\right) f_{ck} b D$$

$$= \left(0.196 + 0.203 \times \frac{3}{30}\right) \times 30 \times 300 \times 400$$

$$= 778.68 \text{ kN}$$

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For $d'/b = 60/300 = 0.2$

$k_{1y} = 0.184$ & $k_{2y} = 0.028$ (Table 60, SP 16)

$$P_{by} = \left(0.184 + 0.028 \times \frac{3}{30}\right) \times 30 \times 300 \times 400$$

$$= 672.48 \text{ kN}$$

Now,

$$k_x = \frac{P_{uz} - P_u}{P_{uz} - P_{bx}} = \frac{2677.93 - 1500}{2677.93 - 778.68} = 0.62 < 1 \text{ (ok)}$$

$$k_y = \frac{P_{uz} - P_u}{P_{uz} - P_{by}} = \frac{2677.93 - 1500}{2677.93 - 672.48} = 0.587 < 1 \text{ (ok)}$$

c) Calculate adjusted (reduced) additional moment,

$M'_{ax} = k_x \times M_{ax} = 0.62 \times 67.5 = 41.85$ kNm

$M'_{ay} = k_y * M_{ay} = 0.587 \times 62.5 = 36.68$ kNm

Step5: Calcute Initial Moment (M_i)

For braced column with double curvature

$M_{uxi} = 0.6 M_{ux2} - 0.4 M_{ux1} \neq 0.4 M_{ux2}$

$= 0.6 \times 40 - 0.4 \times 22.5 \neq 0.4 \times 40$

$= 15$ kNm $\neq 16$ kNm

So, adopt $M_{uxi} = 16$ kNm

Again,

$M_{uyi} = 0.6 M_{uy2} - 0.4 M_{uy1} \neq 0.4 M_{uy2}$

$= 0.6 \times 30 - 0.4 \times 20 \neq 0.4 \times 30$

$= 10$ kNm $\neq 12$ kNm

So, adopt $M_{uyi} = 12$ kNm

Step6: Calculation of total design moments.

$M_{ux} = M_{ax}(M_{ux1}, M_{ux2}) + M'_{ax} \neq M_{ux2}$

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$M_{ux} = Max(16, 40.995) + 41.85 \not\leq 40\ kNm$

$= 40.995 + 41.85$

$= 82.845\ kNm \not\leq 40\ kNm$ (ok)

$M_{uy} = Max(M_{uy1}, M_{uye}) + M'_{ay} \not\leq M_{uy2}$

$= Max(12, 36) + 36.6875 \not\leq 30\ kNm$

$= 36 + 36.6875$

$= 72.6875\ kNm \not\leq 30\ kNm$ (ok)

The total design moments are,

$M_{ux} = 82.845\ kNm$

$M_{uy} = 72.6875\ kNm$

Step7: Calculation of M_{ux1} & M_{uy1}

Here, $d'/D = \frac{60}{400} = 0.15$ & $\frac{b}{f_{ck}} = \frac{3}{30} = 0.1$

$$\frac{P_u}{f_{ck} b D} = \frac{1500 \times 1000}{30 \times 300 \times 400} = 0.417$$

From chart 45,

$$\frac{M_{ux1}}{f_{ck} b D^2} = 0.106$$

$M_{ux1} = 0.106 \times 30 \times 300 \times 400^2 = 152.64\ kNm$

Also,

$d'/b = 60/300 = 0.2$, $P/f_{ck} = 0.1$ & $P_u/f_{ck} b D = 0.417$

From chart 46,

$$\frac{M_{uy1}}{f_{ck} D b^2} = 0.095$$

$M_{uy1} = 0.095 \times 30 \times 400 \times 300^2$

$= 102.6\ kNm$

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Step8: Calculate α_n which depends on P_u/P_{uz} value,

$P_u = 1500\ kN$

$P_{uz} = 0.446 f_{ck} A_c + 0.75 f_y A_{sc} = 2677.932\ kN$

$\frac{P_u}{P_{uz}} = \frac{1500}{2677.932} = 0.56$

$$\alpha_n = 1 + \frac{2-1}{0.8-0.2} \times (0.56 - 0.2)$$

0.2	1
0.56	$\alpha_n = ?$
0.8	2

$\alpha_n = 1.6$

Step9: Check,

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{82.845}{152.64}\right)^{1.6} + \left(\frac{72.6875}{102.6}\right)^{1.6}$$

$= 0.952 < 1$ (ok)

Step10: Calculate area of longitudinal rebar,

$A_{sc} = 0.03 \times 300 \times 400 = 3600\ mm^2$

No. of bar (n) $= \frac{3600}{\pi \times 25^2/4} = 7.33 \approx 8$ (Taking 25mm ϕ long. bar)

% Ast provided $= \frac{\pi \times 25^2/4 \times 8}{300 \times 400} \times 100\% = 3.27\%$

Check, $A_{st,min}$ (0.8%) $< A_{st,pro}$ (3.27%) $< A_{st,max}$ (4%)

Minimum no. of bar = 4 (we provided 8, ok)

Minimum drameter of bar = 12mm (we provided 25 mm, ok)

Step11: Design of lateral tres:

(a) Drameter of lateral tres

- ϕ largest longitudinal bar /4 = 25/4 = 6.25mm
- 6mm.

Proviide 8mm ϕ lateral tres.

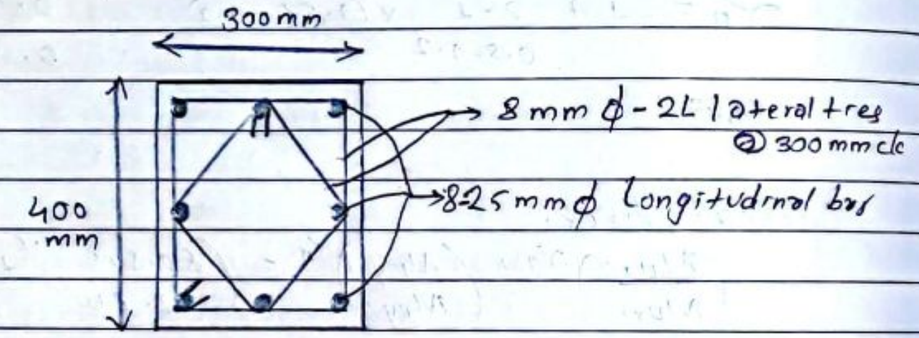
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(b) Pitch / Spacing of lateral ties.

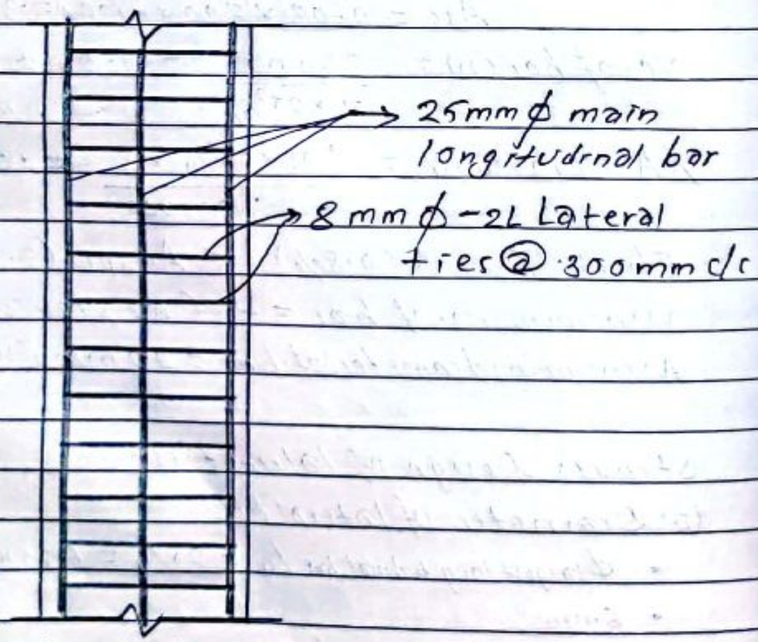
- Least lateral dimension = 300mm
- 16 * ϕ smaller long. bar = 16 x 25 = 400mm
- 300 mm

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



Sectional plan

(Reinforcement details)



Sectional elevation

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Chapter 5,
Design & Detailing of Miscellaneous Structures

DATE

Outline

→ Design & detailing of dog legged & open well staircase
→ Design concepts & detailing of underground watertank & overhead tank
→ Design concepts & detailing of domes, folded plates & Intzetank.

* Staircase :-

→ A staircase is a means of giving access to different floors or levels of a building.
→ Commonly used staircases are as follows
1) straight stairs
2) Quarter turn stairs
3) Open well type (Half turn stair)
4) Dog legged stairs
5) Circular or Spiral stairs

* Proportioning of staircase :-

a) 2 * Riser + Tread = 600 to 640 mm
b) Riser * Tread = 40,000 to 42,000 mm^2
c) Width of private staircase = 900 mm
public staircase = (1800 - 2400) mm
d) Clear height between a flight & the other vertically above it (headroom) should no be less than 2.1m
e) Angle of flight with horizontal ⇒ 25° to 40°
f) Width of landing is equal to width of stairs.

* Structural Behaviour of staircase :-

Structurally, a staircase slab behaves like an ordinary slab supported on the walls or beams. It may be supported in the direction of steps or in the direction of going depending upon the location of the supports.

→ The staircase slabs are classified into two types based on direction in which it spans.
a) Stair slab spanning horizontally
b) stair slab spanning longitudinally.

* Stair slab Spanning Horizontally

→ The stair slab supported on both sides over the walls or beam (stringer beam on one side & a beam or wall on other side) is called stair slab spanning horizontally.
→ Leff = Horizontal distance between c/c of supports.
→ Each step is designed as a slab spanning horizontally i.e. like a rectangular beam of span Leff.

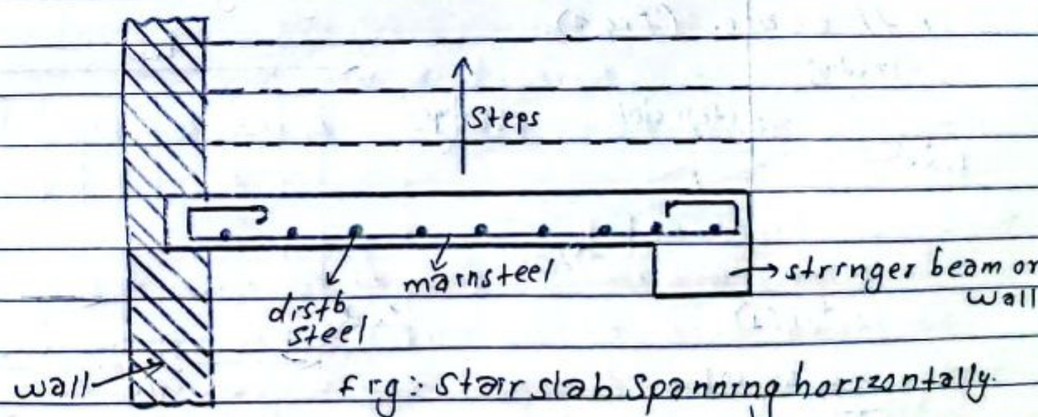


Fig: Stair slab Spanning horizontally.

* Stair slab Spanning Longitudinally :-

→ The stair slab supported at bottom & top of flight is called as stair slab spanning longitudinally. Dog legged, open well

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& quarter turn stairs come under this category.

→ Clause 33 of IS456:2000 gives the gurdeline about design of stairs.

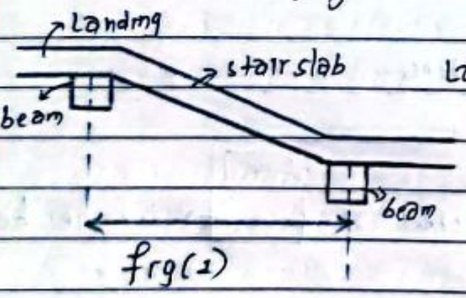
* Effective span of stairs:-

The effective span of stairs without stringer beams shall be taken as following horizontal distances.

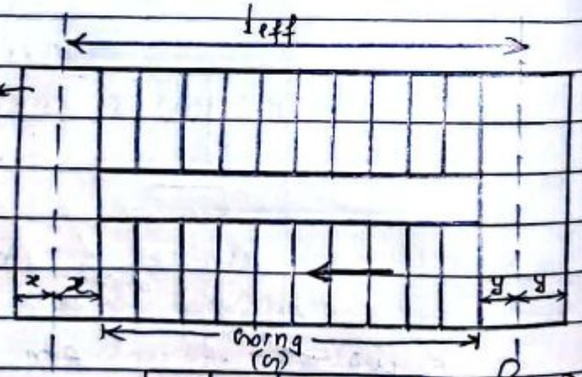
(i) The c/c distance of the beams supporting the top & bottom risers (fig 1)

(ii) When the landing spans parallel to the risers (fig 2), the effective span is taken as a distance equal to the going of the stairs plus half the width of landing on either sides or one meter whichever is smaller.

(iii) When the landing spans parallel to the stairs, the effative span is taken as equal to the c/c distance of the supporting beams or walls, the going is measured horizontally in this case. (fig 3).

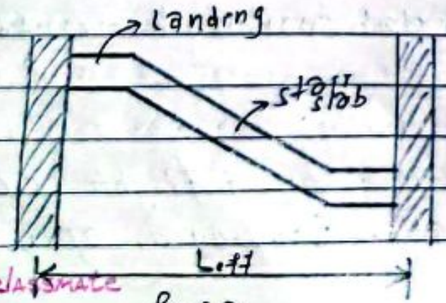


fig(1)



fig(2)

x(m)	y(m)	Span(m)
<1	<1	G+x+y
<1	≥1	G+x+1
≥1	<1	G+1+y
≥1	≥1	G+1+1



fig(3).

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* Loads on stair

(a) Live load:-

As per IS 875 part II,

stairs liable for over crowding = 5 kN/m²

Stairs not liable for over crowding = 3 kN/m²

(b) Dead load:-

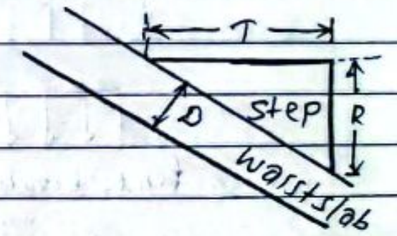
Dead load of stairs consists of i) Dead weight of steps

ii) Dead weight of waist slab.

(i) Weight of one steps (for 1m width)

W_1 = Area of steps × 1 × Density of RCC

$= \frac{1}{2} \times R \times T \times 1 \times 25 = 12.5RT$



Weight of ~~steps~~ steps per m length in plan

$w_1 = \frac{12.5RT}{T} = 12.5R$

(ii) Weight of waist slab (for 1 m width)

$w_2 = \sqrt{R^2+T^2} \times D \times 1.0 \times 25 = 25D\sqrt{R^2+T^2}$

Weight of waist slab per m length in plan

$w_2 = \frac{25D\sqrt{R^2+T^2}}{T} = 25D\sqrt{\frac{R^2}{T^2}+1}$

Total load ⇒ $w = w_1 + w_2$

* Distribution of loading on stairs:

a) In case of stairs with open wells, where spans partly crossing at right angles occur, the load on areas common to any such spans maybe taken as one half in each direction (fig 1)

b) Where the flights or landing are built into walls at a distance

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of not less than 110 mm and are designed to span in the direction of the flight, a 150 mm strip may be deduced from the loaded area & the effective breadth of the section increased by 75 mm for the purposes of design. (fig 2)

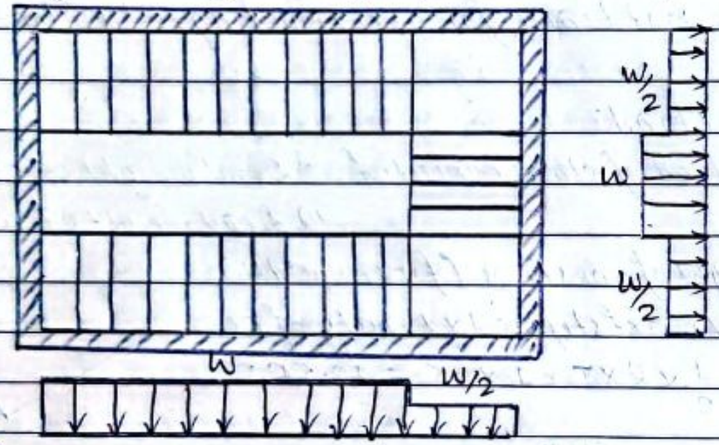


fig1: loading on stairs with open wells.

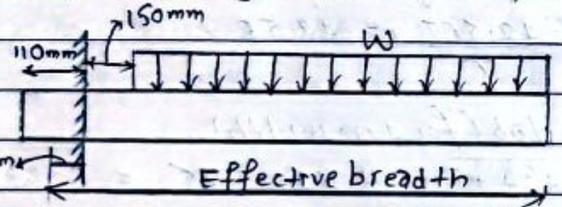


fig2: loading on stairs built into walls.

* Depth of section:

→ The depth of section (D) shall be taken as the minimum thickness perpn to the soffit of the stairs.

→ For designing the stairs, the thickness of the waist slab may be assumed as 1/20 to 1/25 of span for trial calculations.

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* Design steps for dog legged staircase:

1) Proportioning of various dimensions of staircase.
2) Calculate effective span of stair
3) Calculate thickness of waist slab
4) Calculation of design loads
5) Calculation of design moments
6) Calculation of area of reinforcement
7) Check for development length
8) Check for shear as per requirement
9) Sketch drawing of output reinforcement detail of flight & landing.

Q. Design a dog legged staircase for an office building in a room measuring 3m x 6m (clear dimension). Floor to floor height is 3.5m. The building is a public building liable to overcrowding. Stairs are supported on brick walls 230mm thick at the end of landings. Use M20 concrete Fe415 steel.

→

Proportioning of various dimensions of staircase:

Available width of staircase = 3m

Considering 2 flights of dog legged staircase, let us assume width of each flight as 1.35m.

Space bet^n two flights = $3 - 2 \times 1.35 = 0.3$m

Floor to floor height = 3.5m

Height of each flight = $3.5/2 = 1.75$m

Assume, Riser (R) = 150 mm (For public building)

Number of risers = $\frac{1750}{150} = 11.66 \approx 12$

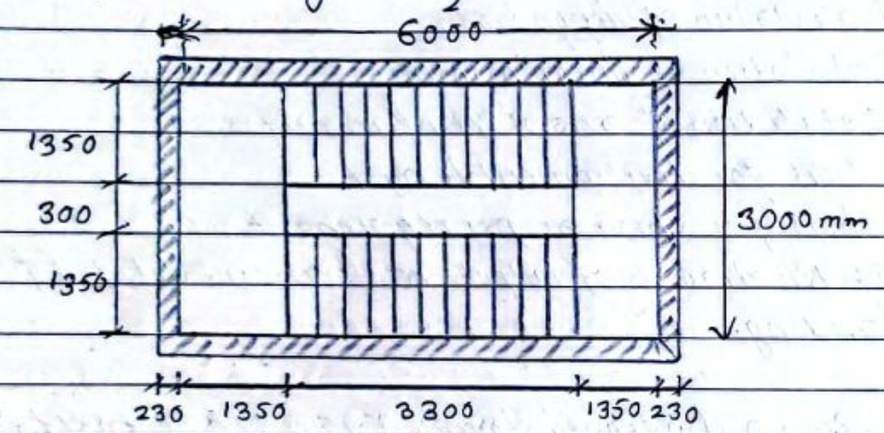
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Number of treads required = 12 - 1 = 11

Let, width of each tread = 300 mm

Total Going = 300 x 11 = 3300 mm

Width of each landing = $\frac{6 - 3.3}{2}$ = 1.35m = 1350mm



→ Design of staircase:

→ Effective span of flight = c/c distance of walls

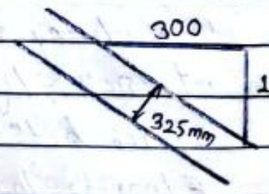
$= \frac{0.23}{2} + 6 + \frac{0.23}{2} = 6.23m$

Thickness of waist slab = $\frac{1}{20}$ of span = $\frac{1}{20} \times 6230 = 311.5mm$.

Let us use d = 300mm & D = 300 + 25 = 325mm.

→ Loads:

(i) Weight of waist slab in plan (per m width of flight)



$= D\sqrt{1 + R^2/T^2} \times 25$

$= 0.325\sqrt{1 + \frac{0.15^2}{0.3^2}} \times 25$

$= 9.1 kN/m$

(ii) Weight of steps (per m width of flight) = $12.5 \times R = 12.5 \times 0.15$

$= 1.875 kN/m$

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Total dead load = 9.1 + 1.875 = 10.975 kN/m

Live load = 5 kN/m² = 5 kN/m per m width of staircase

Total DL + LL = 15.975 kN/m say 16 kN/m

Factored load, $W_u = 16 \times 1.5 = 24 kN/m$

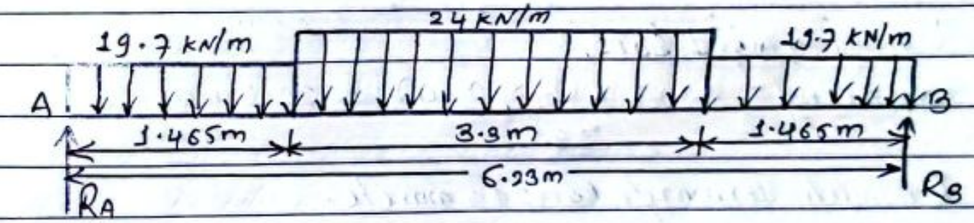
For landing, DL = 0.325 x 25 x 1 = 8.125 kN/m

LL = 1 x 5 kN/m² = 5 kN/m per m width of landing

Total DL + LL = 13.125 kN/m

Factored load = 1.5 x 13.125 = 19.7 kN/m

Load diagram of stairs shall be as follows.



Here,

$R_A = R_B = (19.7 \times 1.465 \times 2 + 24 \times 3.3)/2 = 68.5 kN$

BM at mid span, $M_u = 68.5 \times \frac{6.23}{2} - 19.7 \times 1.465 \times \left(\frac{3.3 + 1.465}{2}\right)$

$- 24 \times 3.3/2 \times 3.3/4$

$= 112 kNm$

Max^m BM allowed for a singly reinforced section with Fe415 bars,

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

$= 0.138 \times 20 \times 1000 \times 300^2 \times 10^{-6}$

$= 2484 kNm > 112 kNm$

Hence, section can be designed as singly reinforced.

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→ Area of reinforcement,

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

$$\Rightarrow 112 \times 10^6 = 0.87 \times 415 \times A_{st} \times 300 \left(1 - \frac{A_{st}}{1000 \times 300} \times \frac{415}{20}\right)$$

$$\Rightarrow A_{st} = 1117\ mm^2$$

Using 16 mm ϕ bars,

$$\text{Spacing} = \frac{1000 \times (\pi \times 16^2/4)}{1117} = 179\ mm$$

Provide 16 mm ϕ @ 170 mm c/c

Dist^b steel = 0.12% of area = $\frac{0.12}{100} \times 1000 \times 325 = 390\ mm^2$

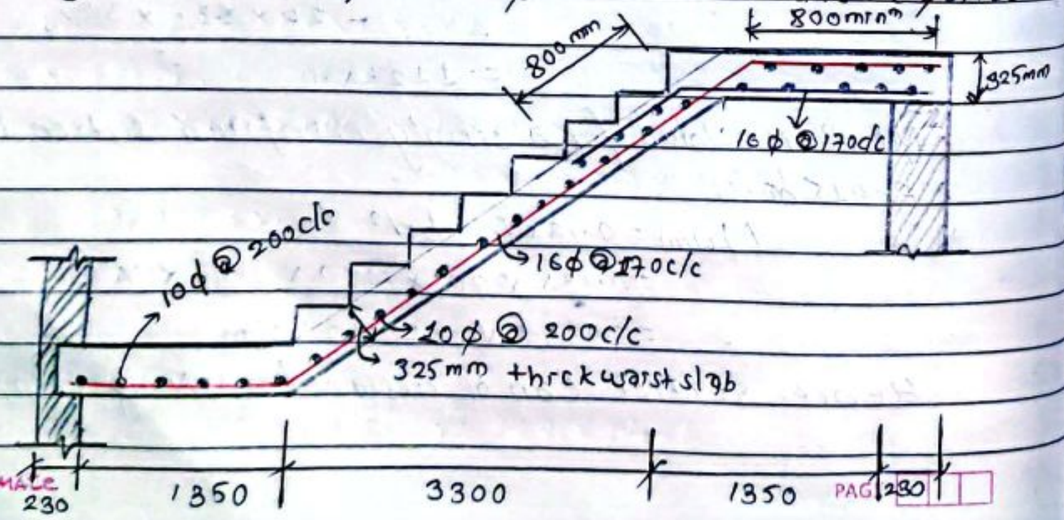
Using 10 mm ϕ bars,

$$\text{spacing} = \frac{1000 \times (\pi \times 10^2/4)}{390} = 201\ mm$$

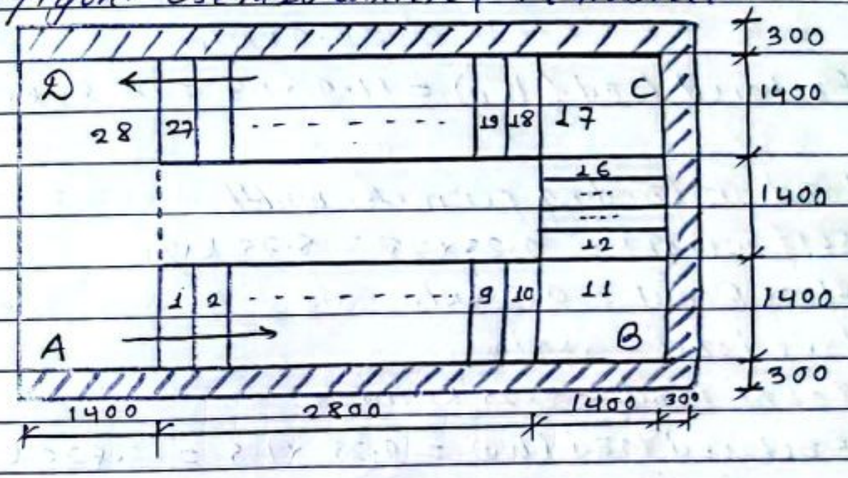
Provide 10 mm ϕ @ 200 mm c/c.

→ Development length (L_d) = $\frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 16}{4 \times 1.6 \times 1.2} = 752\ mm$

Providing 800 mm length of bars at points where L_d is required.



b) Design an open well type staircase middle flight having steps of size 280 mm × 150 mm. The live load is 3 kN/m² & floor finish is 0.5 kN/m². The arrangement of stair is as shown in figure. Use M20 concrete & Fe415 steel.



→ Given,

Riser = 150 mm

Tread = 280 mm

Live load = 3 kN/m² Floor finish = 0.5 kN/m²

f_{ck} = 20 N/mm² f_y = 415 N/mm²

We have to design middle flight (BC).

So,

$$L_{eff} = 300/2 + 1400 + 1400 + 1400 + 300/2 = 4500\ mm$$

Thickness of waist slab, $D = 1/25 \times 4500 = 180\ mm$

Adopt, D = 230 mm

d = 200 mm (assuming effective cover 30 mm)

→ Load calculation,

1) Self weight of waist slab = $D\sqrt{\left(\frac{R}{T}\right)^2 + 1} \times 25$

$$= 0.23 \times \sqrt{\left(\frac{150}{280}\right)^2 + 1} \times 25$$

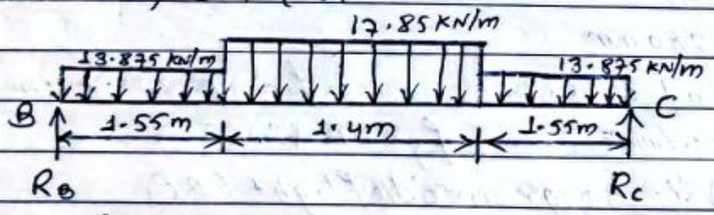
$$= 6.529\ kN/m$$

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ii) Weight of steps = 12.5 R = 12.5 × 0.15 = 1.875 kN/m
iii) Floor finish = 0.5 kN/m
iv) Live load = 3 kN/m
Total load on waist slab = 6.523 + 1.875 + 0.5 + 3
= 11.9 kN/m
Factored load (Wu) = 11.9 × 1.5 = 17.85 kN/m

→ Load on landing per meter width,
Self weight = 0.23 × 25 = 5.75 kN/m
Floor Finish = 0.5 kN/m
Live load = 3 kN/m
Total load = 9.25 kN/m
Factored load (Wu) = 9.25 × 1.5 = 13.875 kN/m

→ Calculation of BM & SF



$R_B = R_C = (13.875 \times 1.55 \times 2 + 17.85 \times 1.4)/2 = 34$ kN
BM at Mid span,
$M_{max} = 34 \times (1.55 + 0.7) - 13.875 \times 1.55 \times (0.7 + 1.55/2) - 17.85 \times 0.7 \times 0.7/2$
= 40.405 kNm
SF = 34 kN.

→ Check for section,
$M_{u lim} = 0.138 \times f_{ck} b d^2$ for Fe415 steel

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$M_{u lim} = 0.138 \times 20 \times 1000 \times 200^2 \times 10^{-6} = 110.4$ kNm
$M_{u lim} > M_{max}$
So, section is designed as singly reinforced.

→ Area of main Reinforcement,
$M_u = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b d}\right)$

$40.405 \times 10^6 = 0.87 \times 415 \times A_{st} \times 200 \left(1 - \frac{415 \times A_{st}}{20 \times 1000 \times 200}\right)$

$\Rightarrow A_{st} = 596.45\ mm^2$
Using 10mm ϕ bars,
$\text{spacing} = \frac{1000 \times (\pi \times 10^2/4)}{596.45} = 131.67$ mm
∴ Provide 10mm ϕ main bar @ 130 mm c/c.
$A_{st\,provided} = \frac{1000 \times \pi \times 10^2/4}{130} = 604.15\ mm^2$

→ Area of distribution steel,
$A_{st} = \frac{0.12}{100} \times 1000 \times 230 = 276\ mm^2$
$\text{Spacing} = \frac{1000 \times \pi \times 8^2/4}{276} = 182.12$ mm
Provide 8mm ϕ distb bar @ 180 mm c/c.

→ Check for shear,
$\tau_v = \frac{V_u}{bd} = \frac{34 \times 10^3}{1000 \times 200} = 0.17\ N/mm^2$
$P_t = \frac{100 \times 604.15}{1000 \times 200} = 0.3\%$.

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For $P_t = 0.3\%$ & M20 concrete,

$$\tau_c = 0.36 + \frac{0.48 - 0.36}{0.5 - 0.25} \times (0.3 - 0.25)$$

$$= 0.384 \text{ N/mm}^2$$

0.25	0.36	
0.3	τ_c = ?	
0.5	0.48	

$\tau_c > \tau_v$, safe in shear.

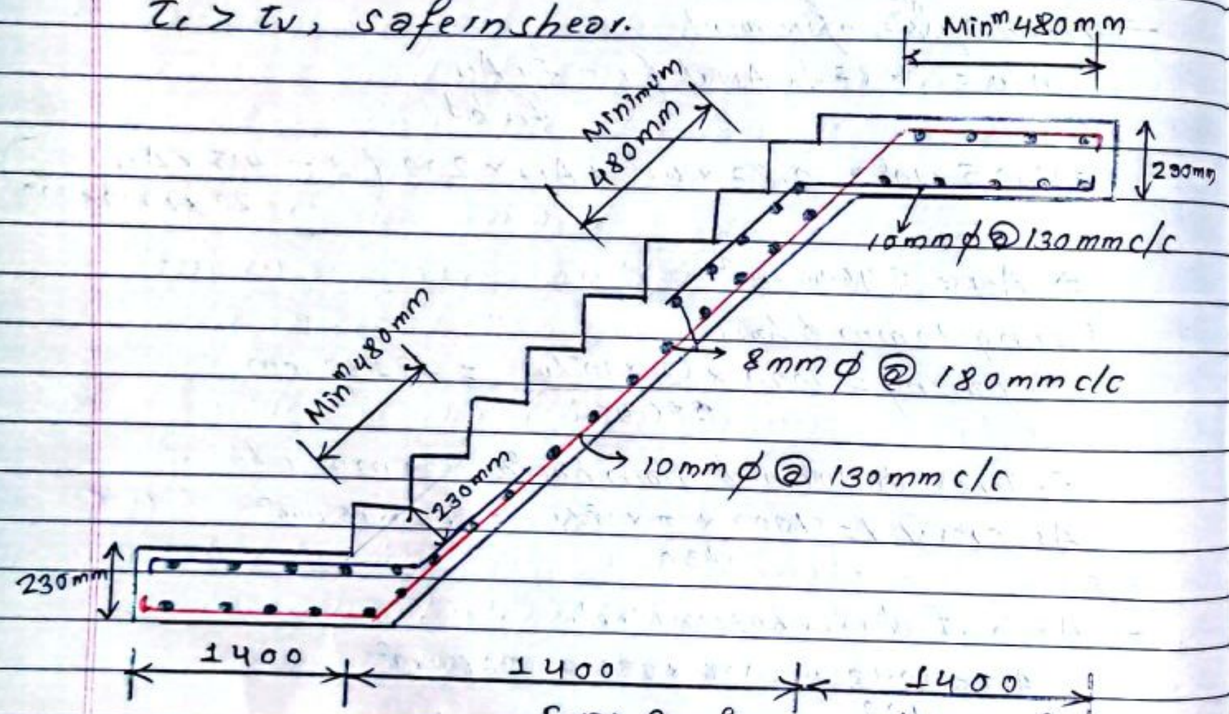


Fig: Reinforcement details of middle flight of open well staircase

→ Development length,

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 10}{4 \times 1.6 \times 1.2} = 470.11\text{mm} \approx 480\text{mm}$$

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✻ Design of water tank:-

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→ Tanks are widely used for storing liquids like water, chemicals, petroleum etc.

→ The tanks are generally circular or rectangular in shape.

→ They are broadly classified into following three types.

1) Tanks resting on ground
2) Underground tanks
3) Elevated or overhead tanks.

Examples

a) Sedimentation tanks, aeration tanks, filtration tank, clear water reservoir ⇒ Tank resting on ground

b) Underground Tanks ⇒ Septic tank, imhoff tank, water tank

c) Elevated tank → Used in water distb system.

→ IS 3370:2009 (part I to IV) is used in design of concrete structures for storage of liquids.

✻ Criteria for design of liquid retaining structures:-

1) Concrete mix lower than M30 are not to be used. Use of richer mixes results in less cracking

2) The structure retaining the liquids should be designed as "subjected to severe exposure conditions".

3) The cement content, not including fly ash & ground granulated blast furnace slag, should not exceed 400 kg/m^3

4) Cracking can be controlled by using the plasticizers & by using minimum amount of cement content which will result in reduced water content per unit of concrete mix. The minimum cement content from durability criteria is 320 kg/m^3

5) Cracking can also be controlled by reducing the steep changes in temp^r & moisture content at early age of concrete. Curing

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should be done at least for period of 14 days.

6) Correct placing of reinforcement bars, use of deformed bars, bars closely spaced & use of small sized bars will also result in reduced cracking.

7) Crack width for RCC members in direct tension & flexural tension is considered satisfactory, if steel stress under service condition does not exceed 130 N/mm² for high strength deformed bars.

8) The maximum calculated surface width of crack for direct tension & flexure should not be more than 0.2mm with specified cover.

* IS Code Recommendation regarding Detailing in water tanks

→ 1) The minimum reinforcement in walls, floors & roofs in each of two directions at right angles within each surface zone should not be less than 0.35% of cross section of surface zone for HYSD bars & 0.64% for mild steel bars.

2) The minimum reinforcement can be further reduced to 0.24% for deformed bars & 0.4% for plain bars, for tanks not having any dimension more than 15m.

3) In tank walls & slabs, having thickness less than 200mm, the reinforcement can be placed in one ~~direction~~ face only.

4) For ground / base slab having thickness less than 300mm the reinforcement should be placed on one face, as near as possible to the upper surface consistent with cover.

5) The spacing of reinforcing bars should not exceed 300mm or thickness of section whichever is less.

6) Size of bars, distance between bars, laps & bends should be as per IS 456:2000.

* Design of Rectangular Overhead Tanks:-

→ The rectangular overhead tanks are used for smaller capacities and are designed using IS 3370 (part-4) which gives the values of the coefficients of hoop tension, moments & shear forces for the walls of the tank for various boundary conditions.

→ The base slab of elevated rectangular tank is designed based on type of supporting system.

→ The base slab is designed for the condition when tank is full.

→ As the base slab is supported on a staging, it may be designed by assuming it to be fixed along the edges.

→ The top / roof slab may be designed as a simple slab. Due to the fixity provided by the top & bottom slab, the walls may be assumed to be hinged at top & fixed at bottom for the purpose of analysis.

* Design of Circular Overhead Tanks:

→ These tanks are used for large capacities. The top / roof slab of these tanks consists of a dome.

→ The base slab can be flat, spherical dome shaped or conical dome shaped.

Sumit Paudel

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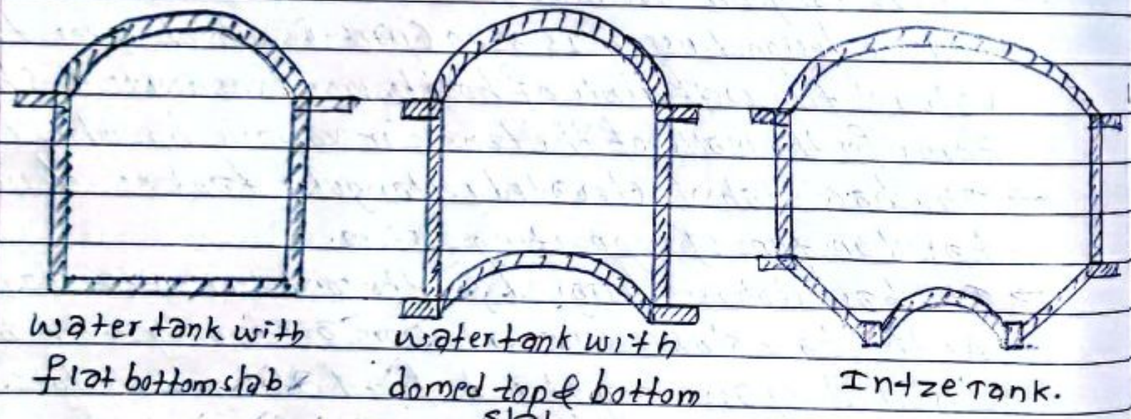
→ Ring beams are provided at the junction of tank wall & dome to take the outward thrust transferred from the dome.

→ On the basis of type of base slab, following types of circular water tanks are commonly used.

a) Water tank with domed top & flat bottom slab.

b) Water tank with domed top & bottom slab

c) Water tank with a conical dome shaped bottom slab (Intze tank)



→ The flat bottom slabs are generally not preferred for the cylindrical tanks due to economical considerations. The thickness of this type of base slab & area of steel reinforcement required for flat slab is more as compared to dome shaped bottom slab.

✻ Domes:-

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→ The roof and floor of a circular tank may be designed as a dome. A dome is a type of shell obtained by the revolution of a regular curve like parabolic, circular or elliptical curve about one of its axis.

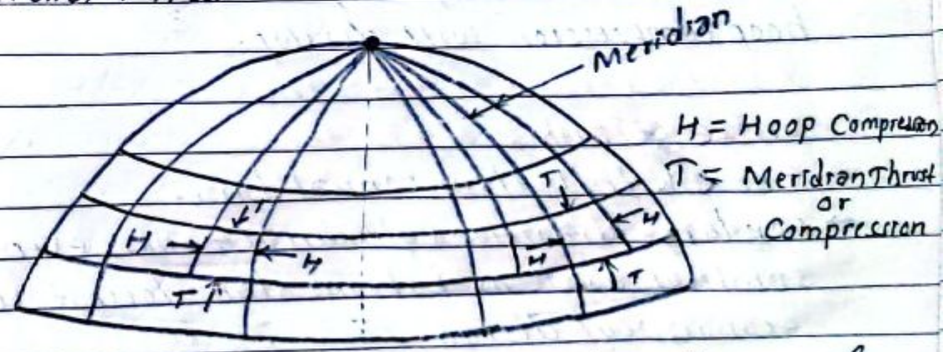
→ A dome is economical as compared to a slab as it carries the load through membrane action resulting in negligible bending moment & shear forces.

→ A dome may be analyzed by assuming it to consists of a number of horizontal rings placed one over the other with gradually reducing diameters towards top as shown in fig(a).

→ The equilibrium of each ring is considered as independent of the rings placed above it. Any section of the ring is subjected to following types of two forces.

a) Hoop or circumferential tension or compression

b) Meridional Thrust



→ The hoop tension or compression acts along the circumference of the dome & the meridonal thrust acts along the meridian direction.

→ Let us consider a spherical dome of radius 'R', subjected to a uniformly distributed load 'w'. The meridonal thrust 'T' at any section in the dome is given by following eqn,

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$$T = \frac{w \cdot R}{1+\cos\theta}$$

Where θ is the vertical angle of the section measured from vertical axis of the dome.

The value of hoop compression can be found by following equation,

$$H = w \cdot R \left[\frac{\cos^2\theta + \cos\theta - 1}{1+\cos\theta}\right]$$

If the value of H is negative, there is hoop tension.

For Hoop compression to be zero,

$$\cos^2\theta + \cos\theta - 1 = 0$$

On solving which gives

$$\theta = 51°49'$$

Hence, for sections having θ more than 51°49', hoop tension will develop & for θ less than 51°49' only hoop compression will develop.

* Intze Tank:-

→ Tank with bottom conical dome.

→ For large diameter cylindrical tanks, even providing a spherical dome as bottom slab doesnot result into an economical design.

→ For such cases, Intze tanks have proven to be more economical.

→ Intze tank is a special type of circular overhead tank in which a conical dome is provided betn the tank wall & the spherical bottom dome as shown in figure. The conical dome results in the reduction in the diameter of the bottom dome & hence reduced stresses. The design of the conical dome and the bottom spherical dome is done in such a way that the outward thrust from the bottom spherical dome is nearly balanced by the inward thrust from the conical dome. The resultant thrust is taken by the ring beam provided at the junction of the conical dome & spherical dome. This type of arrangement gives a more economical design as it results in lower values of hoop tension & other forces.

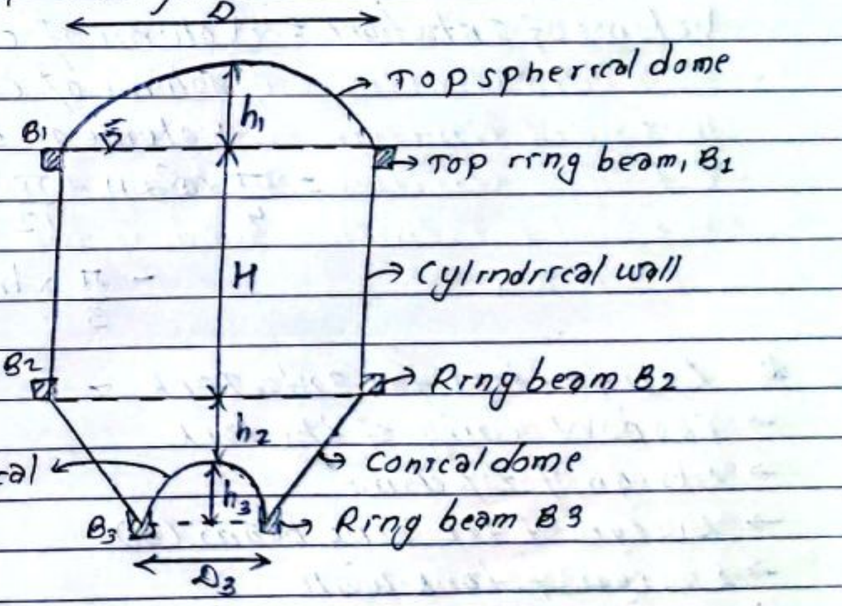


Fig:- Intze tank

* Proportioning of Intze Tank:-

The preliminary dimensions for the design of Intze tank may be taken as follows.

Diameter of tank = D

Height of Cylindrical portion (H) = 0.6D

Rise of top dome (h1) = D/7

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Height of conical dome (h_2) = 0.2D

Rise of bottom spherical dome (h_3) = D/7

Diameter of bottom circular girder/beam (D_3) = 0.6D

Calculation of radius of top ~~dome~~ dome (R_1)

$$\left(\frac{D}{2}\right)^2 = (2R_1 - h_1)\, h_1$$

Calculation of radius of bottom dome (R_3)

$$\left(\frac{D_3}{2}\right)^2 = (2R_3 - h_3)\, h_3$$

Volume of Intze tank = volume of cylindrical portion + volume of conical dome − volume of bottom spherical dome

$$= \frac{\pi}{4} \times D^2 \times H + \frac{\pi}{12} h_2 (D^2 + D_3^2 + D_3 D) - \frac{\pi}{3} \times h_3 \times (3R_3 - h_3)$$

* Design steps for Intze Tank
→ Proportioning of the tank
→ Design of top dome
→ Design of top ring beam (B_1)
→ Design of tank wall
→ Design of bottom ring beam (B_2)
→ Design of conical dome
→ Design of bottom spherical dome
→ Design of bottom girder / Ring beam (B_3)
→ Design output & Reinforcement Details.

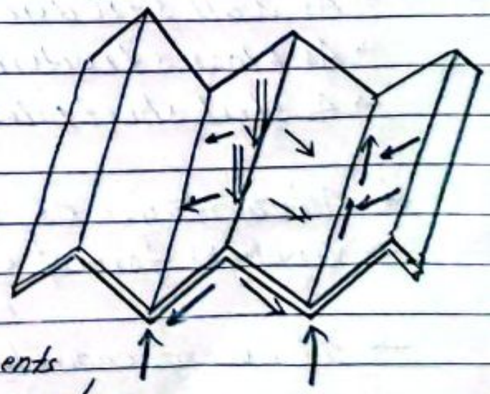
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** Folded plate structure :-

→ Folded plate are assemblies of flat plates rigidly connected together along their edges in such a way that the structural system capable of carrying loads without the need for additional supporting beams along mutual edges.
→ Engineer Eudene Freyssinet performed the first roof with the folded structure in 1923 as an aircraft hanger at Orly Airport in Paris.
→ The principle of folding as a tool to develop a general structural shape has been known for a long time. Folded structure systems which are analogous to several biological systems such as found at broadleaf-tree leaves, petals & foldable insect wings. are adopted to be employed in a new, technical way.
→ The structural characteristics of folding structure depends on
i) The pattern of folding
ii) Their geometric basic shape
iii) Its material
iv) The connection of different folding planes.
v) The design of the bearings.



* Load Distribution process :-
→ At first, the external forces are transferred to the shorter edge of one folding elements.
→ There, the reaction as an axial force is divided bet^n adjacent elements
→ Then, the forces transferred to the bearings.

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* Types of folded structure:-
→ Based on geometrical shape, folded structure can be divided into
i) Folded plate surface structures
ii) Folded plate frame structures
iii) Spatial folded plate structures

→ Based on material of construction
i) Reinforced concrete folded structure
ii) Metal folded structures
iii) Folded structures of wood
iv) Folded structures of glass
v) Folded structures of plastic materials
vi) Folded constructions made in combination of different materials.

* Applications:-
→ As Roof structure
→ As Wall structure
→ As Floor structure
→ As steel sheetpiles

* Advantages of folded plate structures:-
→ very light form of construction (Upto 30m span, shell thickness required is 60mm only)
→ The use of concrete as building material reduces both material cost & construction cost
→ Longer span can be provided.

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→ Flat shapes by choosing certain arched shapes
→ Esthetically, it looks good over other forms of construction.

* Disadvantages of folded plate structures
→ Shuttering is difficult
→ Greater accuracy in formwork is required
→ Good labor & supervision is necessary.
→ Rise of roof may be a disadvantage.

* Note:- For reinforcement detailing of underground & overhead water tank, Intze tank & Dome, prefer books of "Neelam sharma"