Design of Reinforced Cement Concrete Structures (3-2-1)

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Text Books:

1. Jain, A.K. Reinforced concrete: Limit state Design, Roorkee: Nem chand and co.
2. Jai, Krishna and Jain, O.P. Design of R.C.C structure.

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Chapter:- One Introduction.

1.1) Limitation of plain concrete.

plain cement concrete is a hardened mass obtained from a mixture of cement, sand, gravel and water in definite proportions. These ingredients are mixed together to form a plastic mass which is poured into desired shape molds called as forms.

Plain cement concrete has good compressive strength but very little tensile strength, thus limiting its use in construction. It is used where good compressive strength and weight are the main requirements and tensile stresses are very low.

Limitations.

a) plain concrete is quasi-brittle materials.

b) plain concrete has low tensile strength i.e tensile strength is $(\frac{1}{10})$th of its compressive strength.

c) concrete has low toughness i.e only (1-2) % of toughness of steel.

d) concrete has low specific strength.

e) concrete has long curing time. i.e concrete attains specified compressive strength in 28 days.

1.2) Properties of reinforcement and concrete.

Types of reinforcement with their properties are as follows:-

i) Mild steel Reinforcement :- Mild steel bars are also known as Fe-250 because the yield strength of this steel is $250 N/mm^2$. The stress-strain curve for mild steel is given in figure. Its shows a clear, definite yield point. Although they are very ductile, they are not preferred over high yield strength deformed bars because of their less strength

WSM	LSM
The design is based on deterministic approach.	The design is based on probabilistic approach.
The design is governed by elastic theory	The design is governed based on limit states of safety and serviceability.
It does not provide a realistic member of the actual factor of safety underlying a design. It is a traditional method of design.	It provides a realistic member of the actual factor of safety. It is modern method of design.
permissible stress is obtained from the yield stress devided by FOS under working load.	Separate partial factor of safety for loads and strength are to be used to get design loads and design strength.

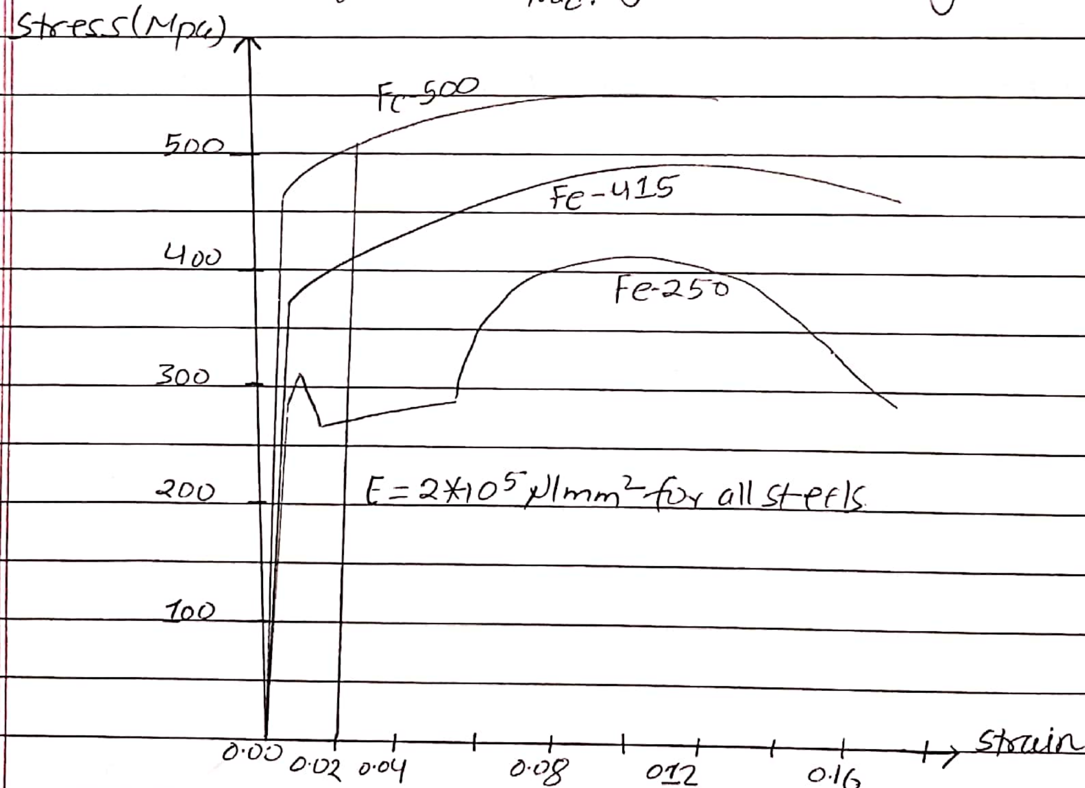

fig:- Typical stress-strain curves for various types of steel.

and weak bond. It's modulus of elasticity is $2*10^5$ N/mm². They are used as lateral ties in columns and at places where nominal reinforcement is required.

ii) High yield strength Deformed bars:- They have higher percentage of carbon as compared to mild steel. It's strength is higher than that of mild steel, but the yield point is not clearly defined as shown in figure.
It is available in two types
a) Hot rolled high yield strength bars.
b) Cold worked high yield strength bars.
Cold worked high yield strength bars a.k.a Cold Twisted Deformed (CTD) bars or Tor steel are available in two grades
i) Fe 415 or Tor 40   ii) Fe 500 or Tor 50.
A twisted deformed bar has about 50% higher yield stress than plain bars.
HYSD are preferred as reinforcement in R.C.C over plain mild steel bars because of
* Higher strength:- It has yield strength, higher than that of plain mild steel bars.
* Better Bond:- It has better bond with concrete due to corrugation on ribs on the surface of the bars.

iii) Thermo-Mechanically Treated (TMT) steel bars:- They are manufactured by passing hot rolled steel bars through cold water. By doing this the outer surface of the bar becomes harder while the inner core is still softer. It has following advantages
*) High yield strength   *) Better weldability
*) Excellent ductility   *) Superior corrosion resistance.

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properties of concrete.

a) compressive strength:- The compressive strength of concrete is determined by the cube test. It is defined as the compressive strength of 15cm cubes of 28 days in N/mm² below which not more than 5% of the test samples are expected to fall. It is represented by fck. The strength of concrete is greatly affected by water-cement ratio (w/c). As per Abram's law

$C = \frac{A}{B^X}$ where C = compressive strength
A & B are constant
X = w/c ratio

b) workability :- The workability of concrete is defined as the ease with which the concrete can be mixed, handled, placed, compacted & finished. A workable concrete should not bleed or segregate.
workability of concrete depends on.
↳ w/c ratio ↳ size & shape of aggregate
↳ Ratio of fine to coarse aggregate.

c) Durability :- The concrete should be durable to the environment when it is exposed to the surrounding. It is defined as the ability to resist weathering action, chemical attack, abrasion or any other process of deterioration. A durable concrete will retain its original form, quality and serviceability when exposed to its intended service environment.

d) Tensile strength :- The tensile strength of concrete can be correlated with the characteristic compressive strength of concrete. IS 456 gives the following correlation which can be used in the design.

$f_t = 0.7\sqrt{F_{ck}}$

e) Modulus of Elasticity:- The term static modulus of elasticity can also be expressed in terms of the characteristic compressive strength and may be written as follows $E_c = 5000\sqrt{f_{ck}}$

f) Poisson's Ratio:- It is defined as the ratio of lateral strain to the longitudinal strain. It can be taken as 0.2 for all design calculation purposes

g) Creep:- It is defined as the increase in strain of the concrete element with time under sustained load after taking consideration the time-dependent deformation not associated with stress i.e shrinkage swelling. It depends upon stress level, Age of loading, Duration of loading

Age of loading	creep coefficient
7	2.2
28	1.6

h) Shrinkage:- Shrinkage in concrete is generally caused by the loss of water through evaporation or by hydration of cement. It is also affected by the fall of temperature and carbonation. Various types of shrinkage are plastic, Drying, Autogenous, carbonation shrinkage. Factor affecting the shrinkage are
↳ effect of w/c ratio ↳ Type of aggregate ↳ relative humidity
↳ time.

1.3) Analysis of forces and stresses in reinforced concrete structure
Structures are designed to withstand various types of loads. The various type of loads expected on a structure are as follows.
↳ Dead loads ↳ live loads ↳ wind loads ↳ snow loads ↳ earthquake loads

• Dead load:- They are due to self wt. of structure which are permanent in nature. It depends upon the unit wt. of materials. It includes the self wt. of walls, floors, beams, column etc.
code IS 875 (part-I)-1987

Unit wt. of column Building Materials

Materials	Unit wt kN/m^3
plain cement concrete.	24
Reinforced Cement concrete steel.	25
steel	78.5
Brick Masonry (Cement plaster)	20

• Live Loads:- Live loads on floors and roofs consists of all the loads which are temporarily placed on the structure. These loads keep on changing from time to time. Various types of imposed loads coming on the structure are given in IS 875 (part-2). 1987.

• Wind Loads:- The force exerted by the horizontal component of wind is to be considered in the design of buildings. It depends upon the velocity of wind, shape and size of the buildings. It's design criteria are provided in IS 875 part-3 1987.

• Snow Loads:- The building which are located in the regions where snowfall is common, are to be considered for snow load. The code IS875 part 4: 1987 deals with snow load on roofs of the building.

• Earthquake Loads:- It depends upon the place where the building is located.

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Chapter:- Two Design Methods

2.1) Working stress method for design of RCC. structures

This method of design was the oldest one. It is based on the elastic theory and assumes that both steel and concrete are elastic and obey Hook's Law. It means that the stress is directly proportional to strain upto the point of collapse.

Assumptions

- plane section before bending remains plane even after bending.
- All tensile forces are taken by reinforcement unless otherwise stated.
- stress strain curve is always linear.
- The moduli of elasticity of steel (E_s) & concrete (E_c) are constant.
- There are no initial stresses in steel and concrete.
- The modular ratio $m = \frac{280}{3\sigma_{cbc}}$ (ratio betn elastic moduli of steel & concrete & is denoted by 'm')

Drawbacks

- It uses factor of safety for stresses only and not for load.
- It does not use any factor of safety with respect to loads.
- It does not account for shrinkage and creep which are time dependent and plastic in nature.
- This method gives uneconomical sections.
- It pays no attention to the condition that arise at the time of collapse.

* Balanced, Under-Reinforced and Over-Reinforced sections as per WSM

a) Balanced section :- A balance section is that in which concrete and steel reach their permissible value at the same time. The percentage of steel corresponding to this section is called as balanced steel and the neutral axis is called as critical neutral axis (n_c)

$$\frac{m.\sigma cbc}{\sigma st} = \frac{n_c}{d-n_c}$$

For a balanced section, the moment of resistance is calculated as under

$$M_R = \frac{\sigma cbc}{2} b.n_c.\left(d-\frac{n_c}{3}\right) = R.b.d^2$$

OR

$$M_b = \sigma st.Ast.\left(d-\frac{n_c}{3}\right)$$

b) Under-Reinforced Section :- In an under-reinforced section, the percentage of steel provided is less than that provided in balanced section. So, the actual neutral axis will shift upward i.e $n_c > n$. The moment of resistance of this section is calculated as

$$M_r = \sigma st.Ast.\left(d-\frac{n}{3}\right)$$

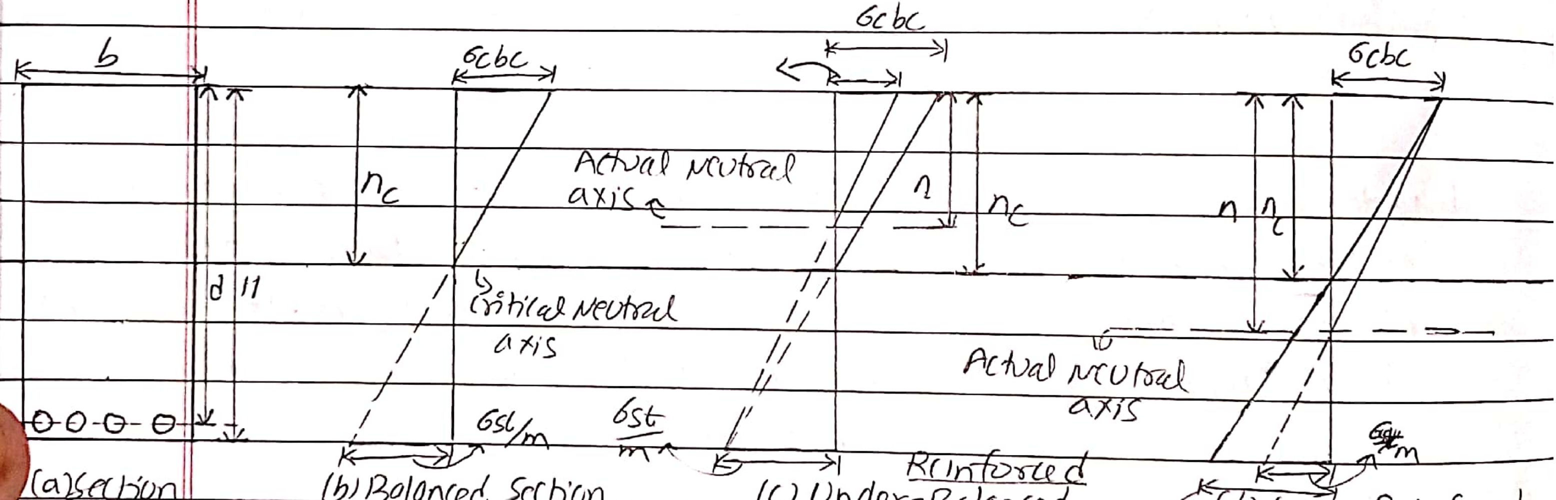
Features

*) Steel is fully stressed while concrete is not (i.e stress in steel is σst (permissible) but stress in concrete is less than σcbc.
*) The actual neutral axis lies above the critical neutral axis ($n_c > n$).
*) Ductile failure.
*) The moment of resistance is less than balanced section.

c) Over-Reinforced Sections :- In an over-reinforced section the percentage of steel provided is greater than the balanced section. So the actual neutral axis shift downwards i.e $n > n_c$. As steel is not fully utilized, the section is uneconomical.

Features

- concrete is fully stressed while steel is not (i.e the stress in concrete is at its permissible value σ_{cbc} but stress in steel is less than σ_{st})
- The actual neutral axis is below the critical neutral axis. i.e $n > n_c$.
- Sudden failure.
- The moment of resistance of over-reinforced section is calculated as $M_r = \frac{1}{2}\sigma_{cbc}\cdot b\cdot n\left(d - \frac{n}{3}\right)$



(a) Section (b) Balanced Section ($n = n_c$) (c) Under-Balanced Reinforced Section ($n < n_c$) (d) Over-Reinforced Section ($n > n_c$)

Design procedure Reinforced beam

1. For the given grade of concrete and steel, determine the permissible stresses i.e σ_{cbc} & σ_{st} from code IS 456:2000 table 21 and 22
2. Calculate modular ratio m $m = \frac{280}{3\sigma_{cbc}}$
3. Determine critical neutral axis (n_c). $\frac{m\cdot\sigma_{cbc}}{\sigma_{st}} = \frac{n_c}{d - n_c}$

(4) Determine the actual neutral axis $\frac{b.n^2}{2} = m.Ast.(d-n)$

(5) compare n & nc

i) If $n = nc$, the section is balanced and the moment of resistance can be calculated by any of the following equation.

$$M_B = \frac{\sigma cbc . b . nc}{2} . \left(d - \frac{nc}{3}\right) = Rb.d^2$$

OR

$$M_B = \sigma st . Ast . \left(d - \frac{nc}{3}\right)$$

ii) If $n < nc$ the section is under reinforced and the moment of resistance is calculated as

$$M_r = \sigma st . Ast\left(d - \frac{n}{3}\right)$$

iii) If $n > nc$, the section is over-reinforced and the moment of resistance is calculated as

$$M_r = \frac{1}{2} \sigma cbc . b . n . \left(d - \frac{n}{3}\right)$$

(*) Sometimes it is required to find out the safe load (w) which the beam can carry. For this, the maximum bending moment due to the loads is calculated and equated to the moment of resistance of the section.

The maximum bending moment value for some beams are :-

a) simply supported beam for u.d.l $= \frac{wl^2}{8}$ (sagging), $\frac{wl}{4}$ (point load) or, $\frac{wa.b}{2}$

b) cantilever beam for u.d.l $= \frac{wl^2}{2}$ (Hogging), $w.l$ (point load)

where l is the effective span of the beam

2.3) Ultimate load method for design of RCC structures

In this method ultimate or collapse load is used as design load. The ultimate loads are obtained by increasing the working loads suitably by some factors. These factors which are multiplied by the working loads to obtain ultimate loads are called as load factors. These method uses the real stress–strain curve of concrete and steel and takes into account the plastic behavior of these materials.

$$\text{Load factors} = \frac{\text{collapse load}}{\text{working load}}$$

The load factors provide a clear margin of safety and one can easily tell the load at which the structure fails, which is not clear from the working stress concept of permissible.

Advantages

- This method is more realistic as compared to working stress method because ultimate load method takes into account the non-linear behavior of the concrete.
- This method gives exact margin of safety in terms of load unlike working stress method which is based on the permissible stresses.
- This method is economical as compared to WSM.

Limitation

- This method gives very thin sections which leads to excessive deformations and cracking.
- No factors of safety are used for material stresses.

As serviceability requirement are not satisfied, limit state method are used which takes into account the strength as well as serviceability requirements.

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2.3) Limit state method for design of RCC structures

This is the most rational method which takes into account the ultimate strength of the structure and also the serviceability requirements. It is a judicious combination of working stress and ultimate load method of design. This method is based on the concept of safety at ultimate loads (ultimate load method) and serviceability at working load (working stress method). The two important limit states to be considered in design are i) Limit state of collapse ii) Limit state of serviceability

2.4) Types of Limit state methods

1) Limit state of collapse.

The limit state corresponds to the strength of the structure and categorized into following types.

- Limit state of collapse → flexure
  → shear and bond
  → Torsion
  → compression

From code IS456:2000. CL 35.2
Pg 67

2) Limit state of serviceability

The limit state corresponds to the serviceability requirements i.e. deflection cracking and vibration.

A) Deflection control

a) factors affecting Deflection

- Magnitude of load and their distribution.
- Span and type of span.
- Cross-sectional characteristics of structural member.
- Types of concrete.
- Stress in steel reinforcement.
- Amount and extend in cracking

b) Methods of Deflection control

i) Theoritical method

$\delta \le \delta$ permissible

ii) emperical method (method of sufficient stiffness)

$\frac{l}{d} \le \alpha.\beta.\gamma.\delta.\lambda$ (CL. 32.2.1 code 456:2000 p.g 37)

where

l = effective length, d = effective depth

α = basic value (boundary condition) (23.2.1-(a))

β = depend upon length of member (23.2.1-(b))

γ = depend upon tensile Reinforcement (23.2.1-(c))

δ = depend upon shape of member

λ = depend upon compressive reinforcement

B) Crack Control

a) Causes of cracks

- Due to settlement of plastic concrete.
- Due to uneven volumetric change of concrete.
- Due to external loading

b) Methods of Crack control

i) Theoritical method (Annex F IS 456:2000 p.g 95)

$\Delta c \le \Delta per$

Δc = calculated crack width

Δper = permissible crack

ii) Emperical method (Rule of proper detailing)

Cracking is controlled by proper detailing.

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2.5) characteristics loads and strength of materials

characteristics strength of materials

The term characteristics strength means that value of the strength of the materials below which not more than 5 percent of the test results are expected to fall.

Cl. 36.1 of IS 456-2000 p.g 67.

characteristics Load

The term characteristics load means that value of load which has 95% probability of not being exceeded during the life of the structure

Cl. 36.2 of IS 456-2000 p.g 67

2.6) partial safety factors and their considerations in structural design.

a) Materials:-

The design strength of the materials Fd is given by

$$F_d = \frac{F}{\gamma_m}$$ Cl. 36.3 IS 456-2000 p.g 68

where F = characteristic strength of the material → Cl. 36.1 IS 456-2000 p.g 67

γ_m = partial safety factor appropriate to the material and the limit state being considered. 1.5 for concrete & 1.15 for steel.

b) Loads

The design load Fd is given by

$$F_d = F\gamma_f$$

where F = characteristics load → Cl. 36.2 IS 456-2000 p.g 67.

γ_f = partial safety factor appropriate to the nature of loading and the limit state being considered.

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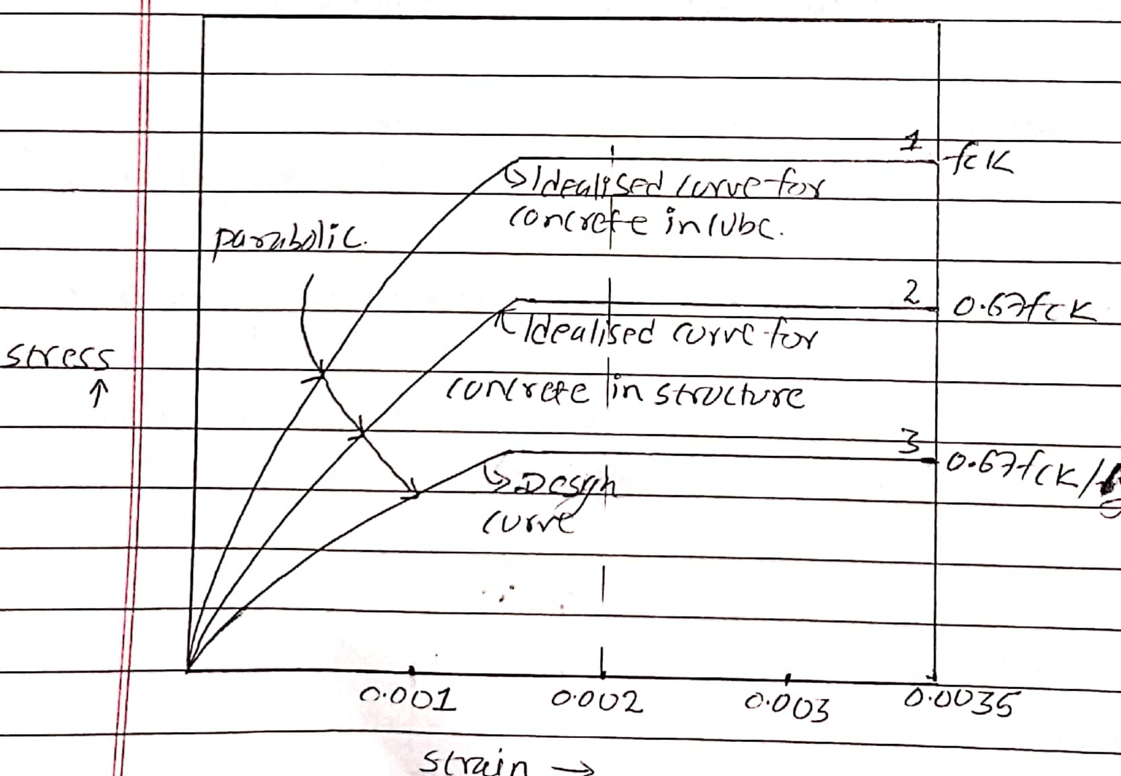
Chapter: Three — Limit state Design for Beams and slab.

3.1 General design consideration.

Basic Assumption.

- plane section normal to the axis of the member remains plane after bending.
- The maximum strain in concrete at the outermost compression fiber is 0.0035.
- The tensile strength of concrete is ignored.
- The strain in the tension reinforcement is to be not less than $\frac{0.87 f_y}{E_s} + 0.002$

or, $\frac{f_y}{1.15 E_s} + 0.002$ — IS 456:2000 P.g 82 CL. 38.1



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

Chapter 1 Remaining.

Plain cement concrete (P.C.C) :- concrete is an composite material composed of fine & coarse aggregate combined with fluid cement that hardens over time. when aggregate is mixed with dry cement and water it forms a cement slurry which is easily poured and moulded into shape. cement reacts chemically with water and other ingredients to form a complex matrix which binds all the material together to form a strong durable stone like material which has many uses.

Reinforced cement concrete (R.C.C) :- concrete is a mixture of cement, fine aggregate, coarse aggregate (crushed stone) and water, the behaviour of concrete is that it is very strong in compression but weak in tension. Steel behaves opposite to that of concrete i.e it is very strong in tension and weak in compression. when concrete or steel along is used, it is not able to withstand all compressive and tensile forces. So, concrete and steel along is used, it is not able to withstand all compressive and tensile forces. So, concrete and steel bar are combinely used to form a structural element. so that it will be strong and safe against all forces. Such structural elements are called reinforced cement concrete (R.C.C structures)

R.C.C. structure = compressive force of concrete + tensile force of steel

Properties of P.C.C/advantages.

- Strength & durability
- Affordability
- Low maintenance cost.
- Locally produced and used
- Fire resistante
- Energy efficient in production.
- Albedo (reflection of heat & light) i.e it doesn't absorb energy and hence temperature does not increase.

Limitation of RCC

- concrete is a brittle material.
- It has low toughness, low tensile strength.
- It has low specific strength.
- Formwork is required.
- Long curing time.
- Working with cracks
- Demands strict quality control

Chapter :- 2 Remaining

Methods of design of R.C.C structures.

- Working stress method (WSM)
- Ultimate stress method (USM)
- Limit stress/state method (LSM)

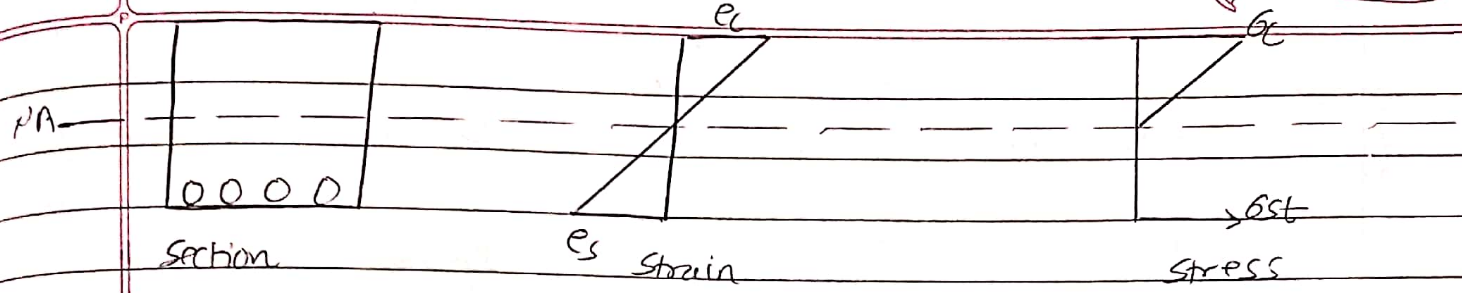
1) Working stress method (WSM)

It is the traditional method of design where it is assumed that concrete is elastic and concrete and steel both acts together elastically. The relationship between loads stress is linear upto the collapse of the structure. The basis of this method is that the permissible stress for concrete and steel are not exceeded anywhere in the structure even in the application of worst combination of load. The members are designed in accordance with elastic theory of bending assuming both concrete and steel obeys Hooke's law. It assume linear variation of stress and strain.

permissible stress is provided by assuming suitable factor of safety to allow for the uncertainities in the estimation of working load and variation in strength of material.

concrete fos = 1.5

Steel fos = 1.15



i.e $\mu R > L$, R = Resistance of material.

L = working load.

μ = inverse of factor of safety

less than 1 ($\mu < 1$)

Drawbacks

- concrete is not elastic
- Factor of safety is provided only to the strength of material
- Difficult to consider creep and strength.

2) Limit state method (LSM)

It is derived from plastic theory of structure. The object of design based on limit state concept is to achieve an acceptable probability that a structure will not deform on service in its life times for use which it is intended i.e will not reach limit state.

The limit state concept of design of R.C.C structure takes into account of the probabilistical and structural variation in material propagate, loads, and safety factors. LSM design can be expressed as

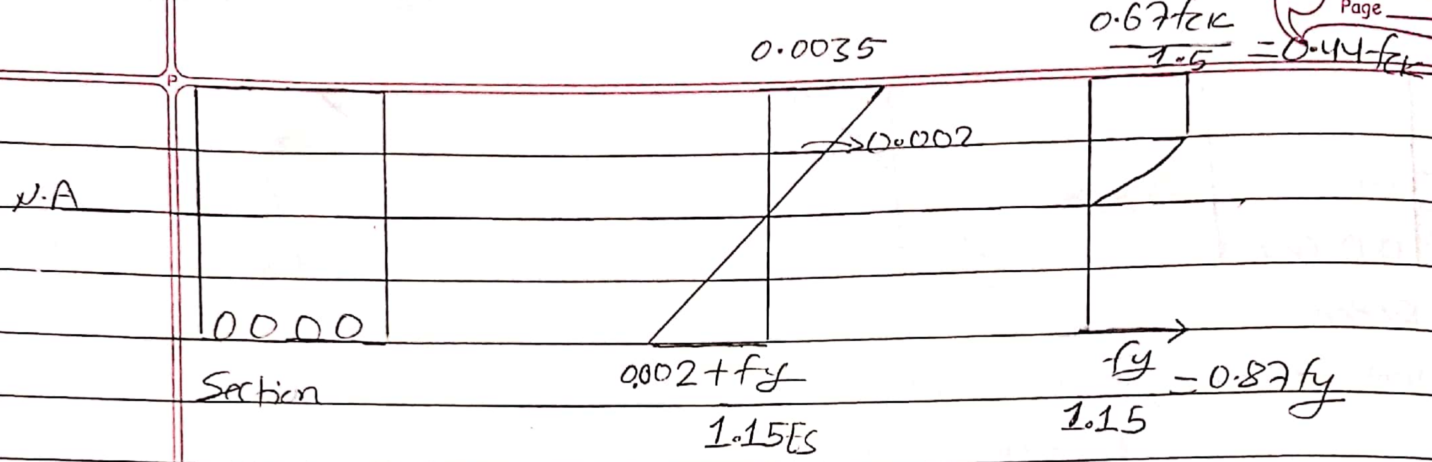
$$\mu R > \sum_{i=1}^{n} \lambda_i L_i$$

R = resistance of material μ & λ = safety factors for material and load.

L = working load

LSM assume that maximum compressive strength in outermost fiber of concrete is 0.35% and maximum tensile strength in steel is not less than $0.002 + \frac{f_y}{1.15 E_s}$ f_y = yield stress of steel

E_s = young's modulus of elasticity



Strain diagram Stress diagram

Partial factor of safety is considered to account for variation and uncertainities for the strength of material and calculation of loads.

Types of limit states.

- Limit state of collapse :- It is concerned with safety of people and safety of structre that all the loads acting on the structure can be withstand by given section. It is concerned with
  - flexure or tension
  - Shear
  - Compression
  - torsion.

Ultimate limit state for collapse.

- Loss of equilibrium
- excessive deformation.
- Rupture
- Loss of stability
- fatigue.

- Limit state of serviceability :- This state is concerned with the functioning of the structure under normal use or comfort of people or appearance of structure. It corresponds to
  - deformation that affects appearance, comfort of user & functioning of structure.
  - Vibration that correspond to people & functional effectiveness
  - Damage that is likely to affect appearance durability, functioning of structre.

Limit state for serviciability considering deflection can be expressed as $\frac{\delta}{L} \leq \frac{1}{\lambda}$ δ = deflection L = span length.
λ = non-dimensional number.

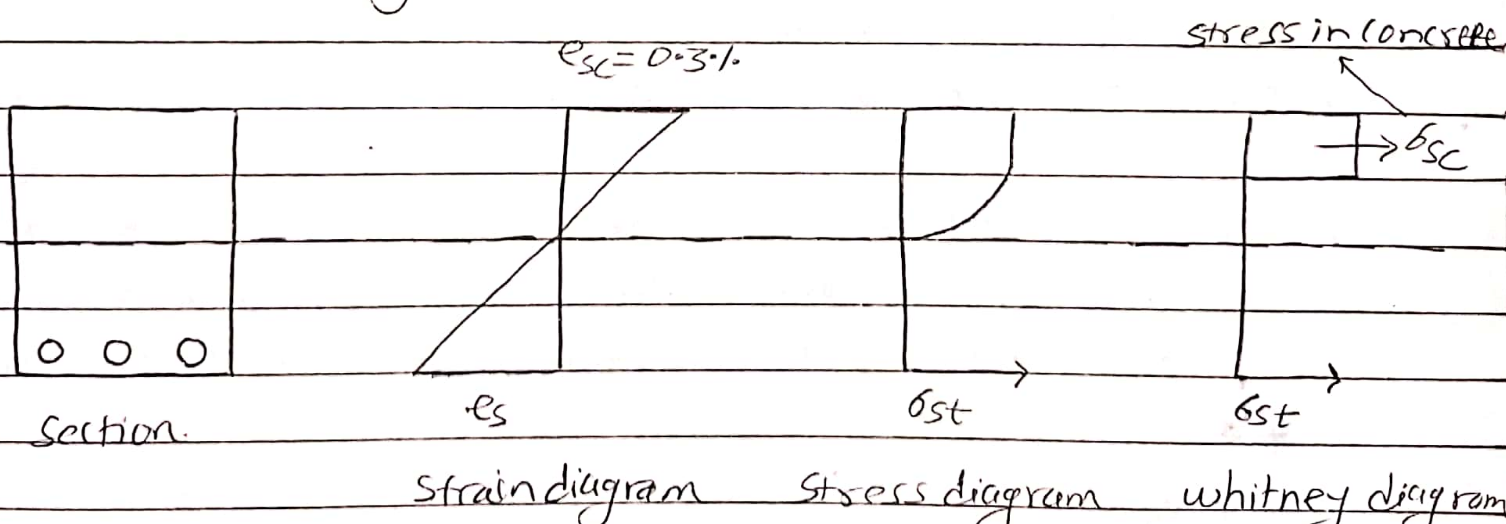
For calculation of deflection only, service loads are used.

Ultimate state method (USM)

In USM, loads are increased by suitable factor and structure is designed to withstand ultimate load. The factor is called load factor. This method uses the non-linear behavior of concrete.

$$\text{Load factor} = \frac{\text{Ultimate load or collapse load}}{\text{working load}}$$

For ultimate load method, mostly whitney's principle is used which assumes maximum compressive strength in concrete is 0.3% and stress corresponding to strain.



Ultimate state method can be expressed as $R > \lambda L$
where
R = resistance of material λ = Load factor.
L = working load

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Drawbacks

- factor of safety is only provided for loads.
- complete disregard for control of deflection.
-

Partial factor of safety

- Partial factor of safety for material, μ_m
- partial factor of safety for load, μ_L

- partial factor of safety for material (μ_m):- It is the factor incorporated to account for the possible unfavorable deviation from the strength of material, from the characteristics value, from the possible unfavourable deviation of sectional dimension, accuracy of calculation procedure and risk to the life and economy.

* Partial factor of safety for concrete = 1.5
* partial factor of safety for steel = 1.15

- partial factor of safety for load (μ_L):- It is the factor which is taken into consideration while calculating load to account for
  - unusual increase in load beyond that used for deriving characteristic value.
  - Unforeseen stress redistribution
  - Inaccurate assesment of the effect of loading.

Characteristic strength (f_{ck}) :- It is the strength below which not more than 5% of the test results are expected to fail.

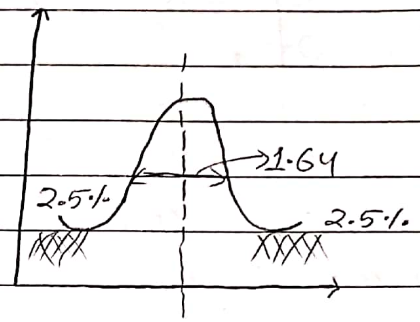
characteristics strength

$$f_{ck} = \bar{X} + 1.64\sigma$$

frequency

$\bar{X}$ = mean strength of sample

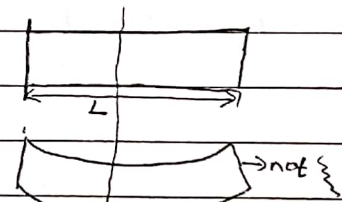
σ = Standard deviation.



Strength (f_{ck})

Characteristic load (w_{ck}) :- The load which has 95% expectation of not exceeding during the life time of the structure.

$$w_{ck} = \bar{X} + 1.64\sigma$$



Limit state method (LSM)

Basic assumption

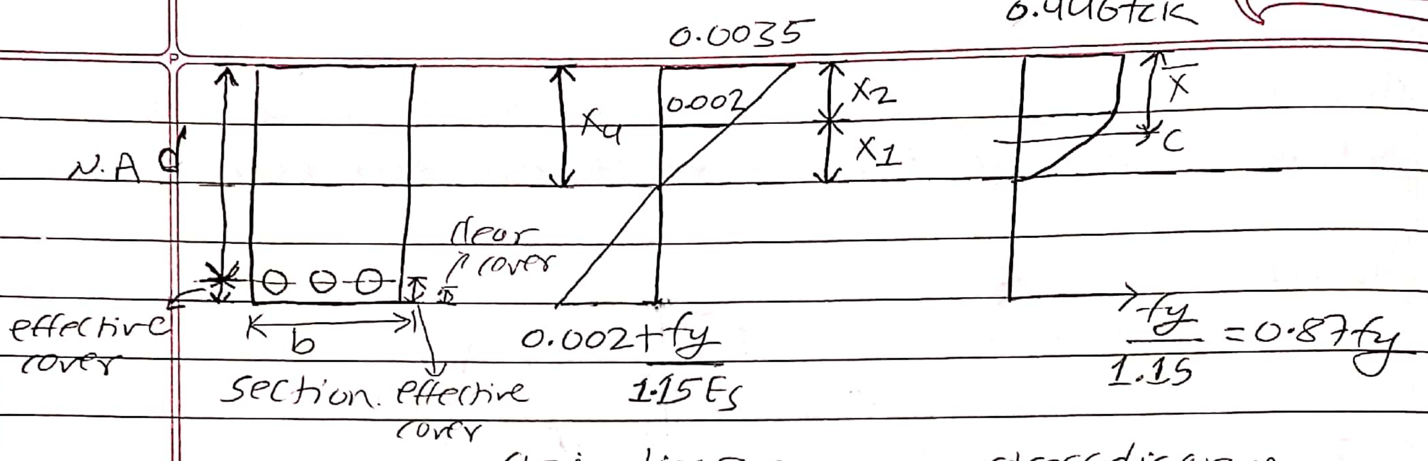
1) plane section normal to the axis of beam remains plane even after bending.

2) Tensile strength of concrete is neglected.

3) Stress-strain diagram for concrete is rectangular-parabolic. with maximum compressive at outermost fibre of concrete is $\frac{0.67 f_{ck}}{1.5} = 0.446 f_{ck}$

4) Maximum compressive strain at outermost fibre of concrete is taken as 0.35%.

5) maximum tensile strain in steel at ultimate limit is taken as not less than $0.002 + \frac{f_y}{1.15 E_s}$



b = breadth of beam.

d = effective depth (distance from extreme fiber of centre to the centre of tension reinforcement.

Xu = depth of neutral axis.

From similar Δ's

$$\frac{X_1}{X_u} = \frac{0.002}{0.0035} = \frac{4}{7}$$

or, $X_1 = \frac{4X_u}{7}$

$$X_2 = X_u - X_1 = X_u - \frac{4X_u}{7} = \frac{3X_u}{7}$$

compressive force of rectangular block $C_2 = 0.446 f_{ck} . X_2 . b$

$$= 0.446 . f_{ck} . \frac{3X_u}{7} \times b$$

$$= 0.191 f_{ck} b X_u$$

compressive force of parabolic block $C_1 = \frac{2}{3} \times 0.446 f_{ck} . X_1 . b$

$$= \frac{2}{3} \times 0.446 f_{ck} \times \frac{4X_u}{7} \times b$$

$$= 0.17 f_{ck} . X_u . b$$

parabola $\bar{x} = \frac{3a}{8}$

Total compressive force of concrete, $C = C_1 + C_2 = 0.19 f_{ck}.b.x_u + 0.17 f_{ck}.b.x_u$

$C = 0.36 f_{ck}.b.x_u$

To find the line of action of total compressive force C

$$\bar{x} = \frac{C_1\bar{x}_1 + C_2\bar{x}_2}{C_1 + C_2} = \frac{C_1\left(x_2 + \frac{3x_1}{8}\right) + C_2\frac{x_2}{2}}{C_1 + C_2}$$

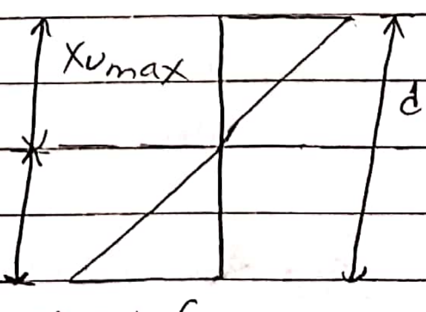
$$= \frac{0.17 f_{ck}\, b.x_u\left(\frac{3x_u}{7} + 3.\frac{4x_u}{7*8}\right) + 0.19 f_{ck}.b.x_u.\frac{3x_u}{7*2}}{0.17 f_{ck}\, b.x_u + 0.19 f_{ck}.b.x_u}$$

$$\left[\frac{3}{7} + \frac{3*4}{7*8} = \frac{9}{14}\right] \qquad \left[\frac{3}{7*2} = \frac{1.5}{7}\right]$$

$$= \frac{f_{ck}.b x_u^2\left(0.17 \times \frac{9}{14} + 0.19 \times \frac{1.5}{7}\right)}{0.36 f_{ck}.b.x_u}$$

$$= \frac{0.15\, x_u}{0.36} = 0.4166 x_u \simeq 0.42\, x_u$$

To find the neutral axis depth for balanced section or limiting depth of neutral axis, $x_{u,max}$



From similar Δ's

$$\frac{d - x_{u,max}}{x_{u,max}} = \frac{0.002 + \frac{f_y}{1.15 E_s}}{0.0035}$$

or, $$\frac{d}{x_{u,max}} - 1 = \frac{0.002 + 0.87\frac{f_y}{E_s}}{0.0035}$$

$$\frac{d}{x_{u,max}} = \frac{0.0055 + 0.87 f_y/E_s}{0.0035}$$

$$\therefore x_{u,max} = \frac{0.0035d}{0.0055 + 0.87\frac{f_y}{E_s}}$$

$E_s = 2\times10^5 \, N/mm^2$

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

for

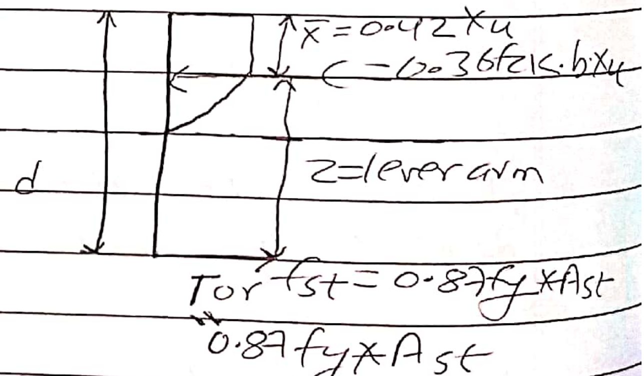
Fe250, $x_{u,max} = \dfrac{0.0035d}{0.0055 + \dfrac{0.87\times250}{2\times10^5}} = 0.53d.$

For Fe 415, $x_{u,max} = \dfrac{0.0035d}{0.0055 + \dfrac{0.87\times415}{2\times10^5}} = 0.48d.$

For Fe500, $x_{u,max} = \dfrac{0.0035d}{0.0055 + \dfrac{0.87\times500}{2\times10^5}} = 0.46d$ From code page 33

To find the depth of neutral axis, x_u

Total compressive force = total tensile force N.A



$C = T$

or, $0.36 f_{ck}.b.x_u = 0.87 f_y . A_{st}$

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

A_{st} = Area of steel.

Lever arm(z) = distance between total compressive force of concrete and tensile force of steel

$= d - \bar{x}$

$= d - 0.42x_u$

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Page

moment of resistance (MOR) in term of concrete = C.Z = $0.36 f_{ck} \cdot b x_u \cdot (d - 0.42 x_u)$

MOR in term of steel = T.Z = $0.87 f_y A_{st} \left(d - 0.42 \times \frac{0.87 f_y A_{st}}{0.36 f_{ck} \cdot b}\right)$

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

Percentage of steel:- (P%) = $\frac{A_{st}}{b \cdot d} \times 100\%$ = $\frac{0.36 f_{ck} \cdot b \cdot x_u}{0.87 f_y \cdot b \cdot d} \times 100\%$

d → effective depth

$= \frac{0.36 \times f_{ck} \cdot x_u}{0.87 \; f_y \; d} \times 100\%$

Minimum percentage of steel = $\frac{0.85}{f_y} \cdot b \cdot d$

maximum percentage of steel = 4% of $b \cdot D$ → D overall depth.

$= 0.004 bD$

code page 16 C.L 26.5.1.2

moment of resistance (MOR) in term of concrete $= C \cdot z = 0.36 f_{ck} \cdot b x_u \cdot (d - 0.42 x_u)$

MOR in term of steel $= T \cdot z = 0.87 f_y A_{st} \left(d - \frac{0.42 \times 0.87 f_y \cdot A_{st}}{0.36 f_{ck} \cdot b}\right)$

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

Percentage of steel :- $(P\%) = \frac{A_{st}}{b \cdot d} \times 100\%$ $= \frac{0.36 f_{ck} \cdot b \cdot x_u}{0.87 f_y \cdot b \cdot d} \times 100\%$

d → effective depth

$= \frac{0.36}{0.87} \times \frac{f_{ck}}{f_y} \cdot \frac{x_u}{d} \times 100\%$

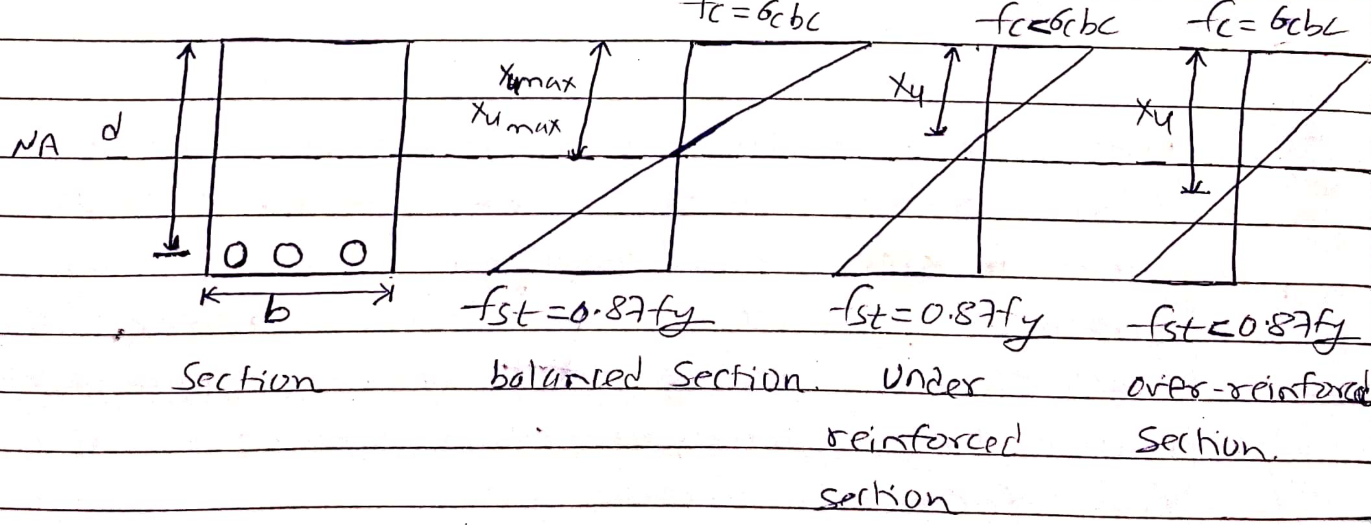
Minimum percentage of steel $= \frac{0.85}{f_y} \cdot b \cdot d$

maximum percentage of steel $= 4\%$ of $b \cdot D$ → overall depth.

$= 0.004 bD$

code page 16 C.L 26.5.1.2

Balanced, Under reinforced and over reinforced section



For balanced section, $X_u = X_{u,max}$
For under-reinforced section, $X_u < X_{u,max}$
For over-reinforced section, $X_u > X_{u,max}$ (always avoid design of over-reinforced section)

Balanced section is a section at which area of tension steel is such that at which ultimate limit state, two limiting conditions reach simultaneously i.e tensile strain in steel reaches yield strain which compressive strain of outermost fiber of concrete reaches ultimate strain. Similarly permissible tensile stress in steel and permissible compressive stress at outermost fibre of concrete reaches simultaneously.

Failure of balanced section is expected with simultaneous initiation of crushing of concrete (brittle failure) and yielding of steel (ductile failure)

Under reinforced section is a section at which area of tension steel is such that at ultimate limit state, tensile strain in steel reaches yield strain before compressive strain at outermost fiber of concrete reaches ultimate strain. Similarly permissible tensile stress in steel reaches before permissible compressive stress at outermost fiber of concrete reaches.

Failure of under-reinforced section is ductile in nature due to the failure of steel by yielding.

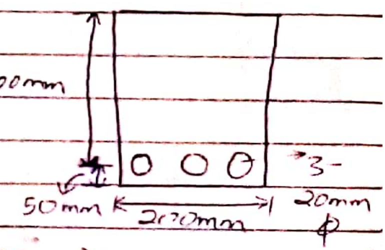
Over-reinforced section is a section at which area of tension steel is such that at ultimate limit state, compressive strain at outermost fibre of concrete reaches ultimate strain before tensile strain at steel reaches yield strain. Similarly, permissible compressive stress at outermost fibre of concrete reaches before permissible tensile stress of steel reaches.

Failure of over-reinforced section is brittle in nature due to the failure of concrete by crushing.

* Numericals

1. Determine moment of resistance of section as shown. Use M15 concrete and Fe 250 steel.

Soln



b = 200mm
effective depth (d) = 400mm
$f_y = 250$ MPa
$f_{ck} = 15$ MPa

$A_{st} = 3 \times \frac{\pi d^2}{4} = 3 \times \frac{\pi \times 20^2}{4} = 942.478\ mm^2$

Limiting depth of neutral axis, $X_{u,max} = 0.53d$ for Fe250
$= 0.53 \times 400$
$= 212mm$

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

$$= \frac{0.87*250*942.478}{0.36*15*200}$$

$= 189.805$ mm $< x_{u,max}$

(under-reinforced section)

Moment of resistance in term of concrete, MOR

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

$= 0.36*15*200*189.805*(400 - 0.42*189.805)$

$= 65654394.51 * 10^{-6}$ KNm

$= 65.654$ KNm

in term of steel, MOR $= 0.87 f_y A_{st}(d - 0.42 x_u)$

$= 0.87*250*942.478*(400 - 0.42*189.805) * 10^{-6}$

$= 65.654$ KNm

dia(ϕ)		Area (Ast)
8mm	$\frac{\pi d^2}{4}$	50.265 mm², 50mm²
10mm		78.539mm², 78mm²
12mm		113.097 mm², 113mm²
16mm		201.061mm², 201mm²
20 mm		314.159mm², 314mm²
25 mm		490.874mm², 490mm²
32mm		804.247mm², 840mm²
28mm		615.75 mm², 615 mm²

2. Design a rectangular beam to resist bending moment equal to 45 KNm use M15 mix & Fe 415 steel.

soln

Given, $f_y = 415$ Mpa, $f_{ck} = 15$ Mpa

Bending moment (M) = 45 KNm

For beam $d/b = 1-3$

Assume $d/b = 2$, $d = 2b$

factored Bending moment (M) = 45*1.5 = 67.5 KNm

Limiting depth of neutral axis, $x_{u,max} = 0.48d$ for Fe415 $= 0.48d$

For economy, design balanced section,

Now,

MOR $= 0.36 f_{ck}.b.x_u (d - 0.42 x_u)$, $x_u = x_{u,max} = 0.48d = 0.48*2b = 0.96b$

or, $1.5*45*10^6 = 0.36*15*b*0.96b(2b - 0.42*0.96b)$

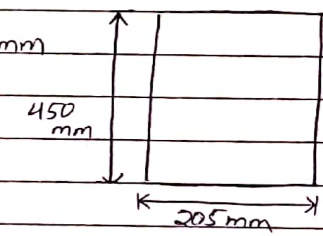
on solving,

$b = 201.278$ mm

adopt (b) = 205mm & d = 2b = 410mm

Assume, effective cover 40mm,

overall depth (D) = 410+40 = 450mm



To find area of steel,

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

or, $67.5*10^6 = 0.87*415*A_{st}*\left(410 - \frac{415 A_{st}}{15*205}\right)$

Generally effective cover = clear cover + d/2 = 35 to 40mm

on solving

$A_{st} = 558.756$ mm²

provide 3-16mmϕ rebar

$(A_{st})_{provided} = 3 \times 201 = 603 mm^2 > 558.756 mm^2$

minimum Ast to be provided $(A_{st})_{min} = \frac{0.85\, b.d}{f_y}$

$= \frac{0.85 \times 205 \times 410}{415} mm^2$

$= 172.151 mm^2$

max^m Ast to be provided, $(A_{st})_{max} = 0.04 b.D$

$= 0.04 \times 205 \times 450\ mm^2$

$= 3690\ mm^2$

check for neutral axis depth, $x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck}.b}$ → Always use provided Ast

$= \frac{0.87 \times 415 \times 603}{0.36 \times 15 \times 205}$

$= 196.667 < x_{u\,max}$ ok

$x_{umax} = 0.48.d = 0.48 \times 410 = 196.8 mm$

3) Design a rectangular beam for 4m effective length subjected to dead load of 15kN/m and live load 12kN/m. use M25 mix and Fe500 steel.

Given,

effective length (le) = 4m

$f_{ck} = 25 Mpa$, $f_y = 500 Mpa$

assume $\frac{le}{d} = 10 \Rightarrow d = \frac{le}{10} = \frac{4000}{10} = 400 mm$

Generally $\frac{le}{d} = 10 - 30$

$d/b = 2 \Rightarrow b = d/2 = \frac{400}{2} = 200 mm$

concrete $25 kN/m^3$ to change into per meter length
$\frac{25 kN \times b.d}{1.b.d}$

calculation of load,

live load = 12 kN/m

Dead load = 15 kN/m

Self wt = $25 \times 0.2 \times 0.4\ kN/m$ {tentative figure}

= 2 kN/m (considering only effective depth but we have to take overall depth)

Total load = 29 kN/m

factored load $(w_u) = 1.5 \times 29 = 43.5 kN/m$

maximum bending moment $(M_u) = \frac{w_u le^2}{8} = \frac{43.5 \times 4^2}{8} = 87 kNm$

Now,

for economy, design balanced section, $x_u = x_{u,max} = 0.46d = 0.46 \times 400$
$= 184 mm$

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

or, $87 \times 10^6 = 0.36 \times 25 \times \frac{d}{2} \times 0.46d\,(d - 0.42 \times 0.46 d)$

on solving we get,

$d = 373.47 \simeq 400 mm$

so, d = 400mm & $b = \frac{d}{2} = 200 mm$

provide 200mm × 400mm rectangular beam.

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

or, $87 \times 10^6 = 0.87 \times 500 \times A_{st}\left(400 - \frac{500 \times A_{st}}{25 \times 200}\right)$

on solving we get,

$A_{st} = 585.756 mm^2$

provide 3-16mmϕ rebar

A_{st}, provided $= 3 \times 201 = 603\,mm^2 > 585.786\,mm^2$

minimum A_{st} provided, $= \frac{0.85}{f_y} bd$

$= \frac{0.85 \times 200 \times 400}{500}$

$= 136\,mm^2$ (min^m A_{st} to be provided)

max^m A_{st} provided $= 0.04\,b.D = 0.04 \times 200 \times 440$

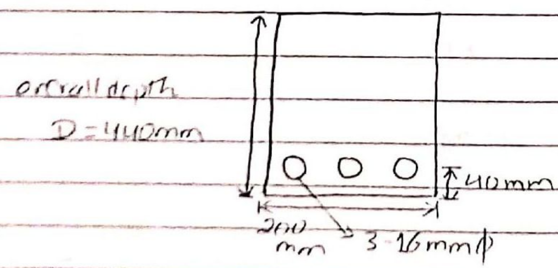
$= 3520\,mm^2$

$D = d + 40 = 440\,mm$

$x_{u,max} = 0.46d = 0.46 \times 400 = 184\,mm$

$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck}.b} = \frac{0.87 \times 500 \times 603}{0.36 \times 25 \times 200} = 145.725\,mm < x_{u,max}$

ok



Note:

If $x_u \le x_{u,max}$ = 'balanced'

$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck}.b}$ < 'under-reinforced'

If $x_u > x_{u,max}$ over-reinforced section

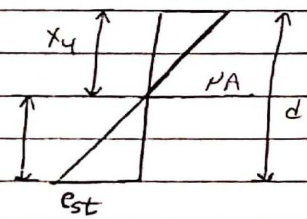
stress in steel $f_{st} \ne 0.87 f_y$ because steel doesnot yield

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

f_{st} can be found from code corresponding to strain in steel, e_{st} =

$\frac{0.0035(d - x_u)}{x_u}$

$\therefore e_{st} = 0.0035\left(\frac{d}{x_u} - 1\right)$



$\frac{e_{st}}{0.0035} = \frac{d - x_u}{x_u}$

$\therefore e_{st} = 0.0035\left(\frac{d - x_u}{x_u}\right)$

code pg. 36 Table A

4) Determine the neutral axis depth and MUR of beam section as shown. use M20 mix and Fe415 steel

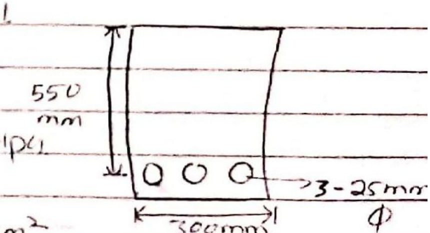
Sol^n

Here,

$b = 300\,mm$, $d = 550\,mm$, $f_{ck} = 20\,Mpa$

$f_y = 415\,Mpa$

$A_{st} = 4 \times \pi \times \frac{25^2}{4} = 1963.495\,mm^2$



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

$= 0.48 \times 550 = 264\,mm$

Assuming $x_u \le x_{u,max}$, $x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck}.b} = \frac{0.87 \times 415 \times 1963.495}{0.36 \times 20 \times 300}$

$= 328.2943\,mm$

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Now, $x_u = \frac{f_{st} A_{st}}{0.36 f_{ck} \cdot b} = \frac{1963.495 \times f_{st}}{0.36 \times 20 \times 300}$

$x_u = 0.909 f_{st}$ —(i)

First trail,

assume $x_u = \frac{264+328}{2} = 296mm$

strain in steel, $e_{st} = 0.0035\left(\frac{d}{x_u} - 1\right) = 0.0035\left(\frac{550}{296} - 1\right)$

$= 0.003$

stress in steel, $f_{st} = 351.8 + \frac{(360.9-351.8)(300-276)}{380-276}$

$= 353.9$

strain (e_{st})	stress f_{st}	code pg 36 table A
0.00276 x_1	351.8 y_1 at 415 N/mm²	
0.003 x	? y	$y - y_1 = \frac{y_2 - y_1}{x_2 - x_1}(x - x_1)$
0.00380 x_2	360.9 y_2	

यो value लाई ×10⁵ गरेर use गरेको हो

$y - 351.8 = \frac{(360.9 - 351.8)}{(0.00380 - 0.00276) \times 10^5} \times (0.003 - 0.00276) \times 10^5$

$x_u = 0.909 \times 353.9 = 321.695$ mm

Second trail,

$x_u = \frac{296+321}{2} = 308.5mm$

$e_{st} = 0.0035\left(\frac{550}{308.5} - 1\right) = 0.00273$

$\simeq 0.00270$

strain (e_{st})	stress f_{st} at 415 N/mm²
0.00241 x_1 →	342.8 y_1
0.00273 x →	? y
0.00276 x_2 →	351.8 y_2

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

or, $y - 342.8 = \frac{(351.8 - 342.8)}{(0.00276 - 0.00241) \times 10^5} \times (0.00273 - 0.00241) \times 10^5$

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From linear interpolation

270 use गर्दा यो आउ / ए यो Ans आउछ

$f_{st} = 342.8 + \frac{351.8 - 342.8}{(276 - 241)}(273 - 241) = 350.257$ Mpa

$x_u = 0.909 \times 350.275 = 318.385mm$

Third trial,

$x_u = \frac{308+318}{2} = 313mm$

$e_{st} = 0.0035\left(\frac{550}{313} - 1\right) = 0.00265$

do convert in stress & apply

$f_{st} = 342.8 + \frac{(351.8 - 342.8)}{(276-241)}(265 - 241) = 348.97$ Mpa.

$x_u = 0.909 \times 348.97 = 317.214mm$

so,

fourth trial

neutral axis depth $x_u = \frac{313+317}{2} = 315$

$e_{st} = 0.0035\left(\frac{550}{315} - 1\right) = 0.0026$

$f_{st} = 342.8 + \frac{351.8 - 342.8}{(276-241)}(260 - 241) = 347.685$

$x_u = 0.909 \times 347.685 = 316.045$ mm

so neutral axis depth (x_u) = 315mm

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

$= 0.36 * 20 * 300 * 315 * (550 - 0.42 * 315) * 10^{-6}$ kNm

$= 281.203$ kNm

in term of steel, $M_{UR} = f_{st} \cdot A_{st} (d - 0.42 x_u)$

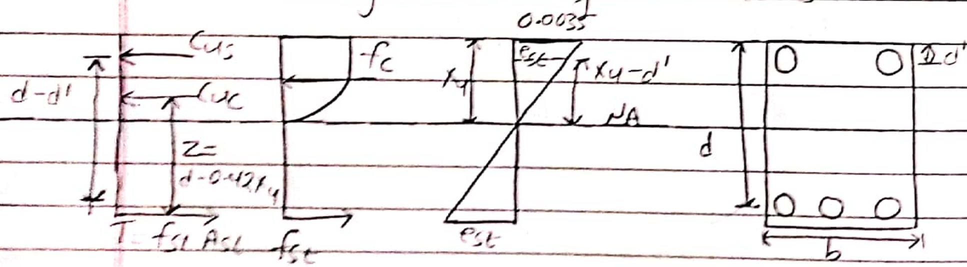
$= 347.685 * 1963.495 (550 - 0.42 * 315) * 10^{-6}$

$= 285.154$ kNm.

Doubly reinforced beam

In doubly reinforced section, reinforcement is provided both in tension and compression zone. Doubly reinforced section is required when

1) size is restricted by architectural or aesthetic point of view.
2) There is application of reversal of load.
3) In continuous monolithic cast of beam and slab, the section is designed as doubly reinforced section.



stress diagram strain diagram

d' = effective cover to compression steel

e_{sc} = strain in compression steel.

$= 0.0035 \dfrac{(x_u - d')}{x_u} = 0.0035 \left(1 - \dfrac{d'}{x_u}\right)$

f_{st} = stress in tension steel.

C_{uc} = compressive force by concrete $= 0.36 f_{ck} \cdot b \cdot x_u$

C_{us} = compressive force by compression steel $= (f_{sc} - f_{cc}) A_{sc}$

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

f_{sc} = stress in compression steel.

A_{sc} = area of compression steel.

T = Tension force by steel $= f_{st} A_{st}$.

at equilibrium

tension force = compressive force.

$T = C_{uc} + C_{us}$

or, $f_{st} A_{st} = 0.36 f_{ck} \cdot b \cdot x_u + (f_{sc} - 0.446 f_{ck}) \cdot A_{sc}$

or, $x_u = \dfrac{f_{st} A_{st} - (f_{sc} - 0.446 f_{ck}) A_{sc}}{0.36 f_{ck} \cdot b}$

$$\therefore x_u = \frac{f_{st} A_{st} - (f_{sc} - 0.446 f_{ck}) A_{sc}}{0.36 f_{ck} \cdot b}$$

For under-reinforced or balanced section i.e. $x_u \le x_{u,max}$

$f_{st} = 0.87 f_y$

$$x_u = \frac{0.87 f_y A_{st} - (f_{sc} - 0.446 f_{ck}) A_{sc}}{0.36 f_{ck} \, b}$$

$$\text{yield strain} = \frac{f_y}{1.15E_s} \quad \text{for Fe250}$$

$$= 0.002 + \frac{f_y}{1.15E_s} \quad \text{for Fe415 and Fe500}$$

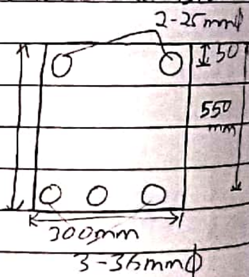
If yield strain $\leq \varepsilon_{sc}$
Then, compression steel yields and
$f_{sc} = 0.87f_y$

Moment of resistance,

$$MOR = C_{uc}z + C_{us}(d-d')$$

$$= 0.36 f_{ck}\, b\, x_u (d - 0.42x_u) + (f_{sc} - 0.446 f_{ck}) A_{sc}(d-d')$$

* Determine the ultimate MOR of doubly reinforced beam as shown. Use M20 mix and Fe250 steel.



Soln

$b = 300mm$

$d = 550mm$

$d' = 50mm$

$f_{ck} = 20 Mpa$

$f_y = 250 Mpa$

$A_{st} = \frac{3 * \pi * 36^2}{4} = 3053.628 mm^2$

$A_{sc} = \frac{2 * \pi * 25^2}{4} =$ ~~982.747 mm²~~ $= 981.747 mm^2$

$x_{u,max} = 0.53d$ for Fe250 $= 0.53 * 550 = 291.5 mm$

assuming $x_u \leq x_{u,max}$ & $f_{sc} = 0.87f_y$

neutral axis depth $x_u = \frac{f_{st}A_{st} - (f_{sc} - 0.446f_{ck})A_{sc}}{0.36 f_{ck}.b}$

$$x_u = \frac{0.87f_yA_{st} - (0.87f_y - 0.446f_{ck})A_{sc}}{0.36f_{ck}.b}$$

$$= \frac{0.87*250*3053.625 - (0.87*250 - 0.446*20)*981.747}{0.36*20*300}$$

$= 212.681mm < x_{u,max} = 291.5mm$

So, our assumption of $x_u \leq x_{u,max}$ i.e. $f_{st} = 0.87f_y$ is justified.

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

$$= 0.0035\left(1 - \frac{50}{212.681}\right)$$

$= 0.00267$

yield strain $= \frac{f_y}{1.15E_s}$ for Fe250

$$= \frac{250}{1.15*2*10^5}$$

$= 0.00108 < \varepsilon_{sc} = 0.00267$ ok

So, our assumption $f_{sc} = 0.87f_y$ is also justified.

And calculated neutral axis depth is also correct.

neutral axis depth $(x_u) = 212.681 mm$

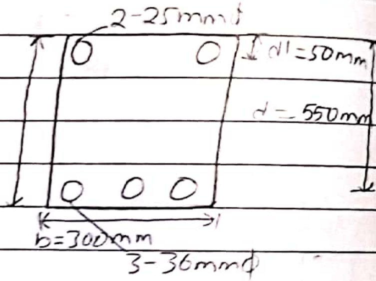
Moment of resistance (MOR) $= 0.36f_{ck}.b\,x_u(d - 0.42x_u) + (f_{sc} - 0.446f_{ck}) A_{sc}(d-d')$

$= 0.36*20*300*212.681*(550-0.42*212.681) + (0.87*250 - 0.446*20)\,481.747\,(550-50)$

$= 314015856.5 * 10^{-6}$

$= 314.015\ kNm$

* Determine the ultimate MOR of doubly reinforced beam as shown in figure. Use M20 mix and Fe415 steel.



$A_{st} = \frac{3*\pi*36^2}{4}$

$= 3053.628\ mm^2$

$A_{sc} = \frac{2*\pi*25^2}{4} = 981.746\ mm^2$

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

$= 0.48*550 = 264mm$

assuming $x_u \le x_{u,max}$ and $f_{sc} = 0.87 f_y$

$x_u = \frac{f_{st} A_{st} - (f_{sc} - 0.446 f_{ck}) A_{sc}}{0.36 f_{ck} b}$

$= \frac{0.87*415*3053.628 - (0.87*415 - 0.446*20)*981.746}{0.36*20*300}$

$= 350.374 > x_{umax} = 264mm$

so, our assumption $x_u \le x_{u,max}$ is wrong

first trial,

$x_u = \frac{264+350.374}{2} = 307.187\ mm$

$e_{st} = 0.0035\left(\frac{d}{x_u} - 1\right) = 0.0035\left(\frac{550}{307.187} - 1\right) = 0.00276$

$e_{sc} = 0.0035\left(1 - \frac{d'}{x_u}\right) = 0.0035\left(1 - \frac{50}{307.187}\right) = 0.00293$

$f_{st} = 351.8$

$f_{sc} = 351.8 + \frac{360.9 - 351.8}{380 - 276}(293 - 276)$

$= 353.287\ MPa$

$x_u = \frac{f_{st}*3053.628 - f_{sc}*981.746 + 0.446*20*981.746}{0.36*20*300}$

$= 1.413 f_{st} - 0.454 f_{sc} + 4.054$

so, $x_u = 1.413 f_{st} - 0.454 f_{sc} + 4.054$

so, $x_u = 1.413*351.8 - 0.454*353.287 + 4.054 = 340.755$

second trial,

$x_u = \frac{307+340}{2} = 323.5mm$

$e_{st} = 0.0035\left(\frac{550}{323.5} - 1\right) = 0.00245$

$e_{sc} = 0.0035\left(1 - \frac{50}{323.5}\right) = 0.00295$

$f_{st} = 342.8 + \frac{(351.8 - 342.8)}{(276 - 241)}(245 - 241)$

$= 343.828\ MPa$

$f_{sc} = 351.8 + \frac{(360.9 - 351.8)}{(380 - 276)}(295 - 276)$

$= 353.541\ MPa$

$X_u = 1.413 \times 348.828 - 0.454 \times 353.541 + 4.054$

$= 329.375$ mm

Third trial, $X_u = \frac{328+329}{2} = 326$ mm

$e_{st} = 0.0035\left(\frac{550}{326} - 1\right) = 0.0024$

$e_{sc} = 0.0035\left(1 - \frac{50}{328}\right) = 0.00296$

$f_{st} = 342.8$ Mpa.

$f_{sc} = 351.8 + \frac{(360.9 - 351.8)}{380-276}(296-276)$

$= 353.55$ Mpa

$X_u = 1.413 \times 342.8 - 0.454 \times 353.55 + 4.054 = 327.918$

So, neutral axis depth $(X_u) = \frac{326+327.918}{2} = 326.95$ mm

and,

Moment of Resistance

$= 0.36 f_{ck} b X_u (d - 0.42 X_u) + (f_{sc} - 0.446 f_{ck}) A_{sc} (d-d')$

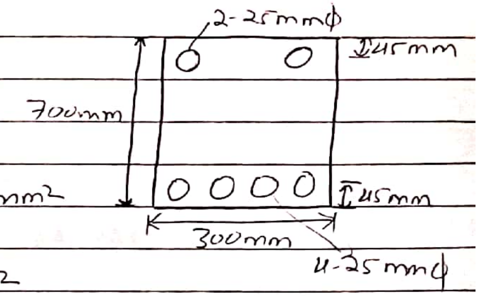
$= 0.36 \times 20 \times 300 \times 326.95 (550 - 0.42 \times 326.95)$
$+ (353.55 - 0.446 \times 20) \times 981.746 \times (550-50)$

$= 460609836.4 \times 10^{-6}$

$= 460.61$ kNm

* Determine the MoR of given section. use M20 mix and Fe415 steel.

Soln



$b = 300$ mm, $d = 700 - 45 = 655$ mm

$d' = 45$ mm

$f_{ck} = 20$ Mpa, $f_y = 415$ Mpa.

$A_{st} = 4 \times \frac{\pi}{4} \times 25^2 = 1963.495$ mm²

$A_{sc} = 2 \times \frac{\pi}{4} \times 25^2 = 981.747$ mm²

$X_{u\,max} = 0.48d$
$= 0.48 \times 655$
$= 314.4$ mm

assuming $X_u \le X_{u,max}$ & $f_{sc} = 0.87 f_y$

$X_u = \frac{f_{st} A_{st} - (f_{sc} - 0.446 f_{ck}) A_{sc}}{0.36 f_{ck} \cdot b}$

$= \frac{0.87 \times 415 \times 1963.495 - (0.87 \times 415 - 0.446 \times 20) \times 981.747}{0.36 \times 20 \times 300}$

$= 168.156$ mm $< X_{u\,max} = 314.4$ mm

so, our assumption of $f_{st} = 0.87 f_y$ is justified.

strain in compression steel $= e_{sc} = 0.0035\left(1 - \frac{d'}{X_u}\right)$

$= 0.0035\left(1 - \frac{45}{168.156}\right)$

$= 0.00256$

yield strain $= 0.002 + \frac{f_y}{1.15 E_s} = 0.002 + \frac{415}{1.15 \times 2 \times 10^5} = 0.0038 > e_{sc}$

so, our assumption of $f_{sc} = 0.87 f_y$ is not justified.
and calculated value of neutral axis depth is not correct.

First trial, $x_u = \frac{314+168}{2} = 241 mm$

$e_{sc} = 0.0035\left(1-\frac{d'}{x_u}\right) = 0.0035\left(1-\frac{45}{241}\right) = 0.00284$

$f_{sc} = 351.8 + \frac{(360.9-351.8)(384-276)}{380-276}$

$= 352.5 Mpa$

$x_u = \frac{0.87*415*1963.495 - (f_{sc} - 0.446*20)*981.747}{0.36*20*300}$

$x_u = 332.257 - 0.454 f_{sc}$ —(1)

$x_u = 332.257 - 0.454*352.5 = 172.222$

Second trial,

$x_u = \frac{241+172}{2} = 206.5$

$e_{sc} = 0.0035\left(1-\frac{45}{206.5}\right) = 0.00273$

$f_{sc} = 342.8 + \frac{351.8-342.8}{276-241} \times (273-241)$

$= 351.028 Mpa.$

$x_u = 332.257 - 0.454*351.028 = 172.890 mm$

3rd trial,

$x_u = \frac{206+172}{2} = 189$

$e_{sc} = 0.0035\left(1-\frac{45}{189}\right) = 0.00266$

$f_{sc} = 342.8 + \frac{(351.8-342.8)}{276-241} * (266-241)$

$= 349.228 Mpa.$

$x_u = 332.257 - 0.454 * 349.228 = 173.707 mm$

so, neutral axis depth (x_u) = 173.707 mm

and, Moment of resistance

$M_{uR} = 0.36 f_{ck} \cdot b \cdot x_u (d - 0.42 x_u) + (f_{sc} - 0.446 f_{ck}) A_{sc} (d-d')$

$= 0.36*20*300*173.707(655 - 0.42*173.707)$
$+ (349.228 - 0.446*20)*981.747(655-45)$

$= 422185478.7 \times 10^{-6}$

$= 422.185 KNm$ Ans

First trial, $x_u = \frac{314+168}{2} = 241\,mm$

$e_{sc} = 0.0035\left(1-\frac{d'}{x_u}\right) = 0.0035\left(1-\frac{45}{241}\right) = 0.00284$

0.00276 → 351.8
0.00380 → 360.9
0.00284 y = 352.5

$f_{sc} = 351.8 + \frac{(360.9-351.8)(384-276)}{380-276}$

$= 352.5\ MPa$

$x_u = \frac{0.87 \times 415 \times 1963.495 - (f_{sc} - 0.446 \times 20) \times 981.747}{0.36 \times 20 \times 300}$ (Ast, fck, Asc)

$x_u = 332.257 - 0.454 f_{sc}$ —— (1)

$x_u = 332.257 - 0.454 \times 352.5 = 172.222$

Second trial,

$x_u = \frac{241+172}{2} = 206.5$

$e_{sc} = 0.0035\left(1-\frac{45}{206.5}\right) = 0.00273$

$f_{sc} = 342.8 + \frac{351.8-342.8}{276-241} \times (273-241)$

0.00241 → 342.8
0.00276 → 351.8
0.00273 y = 351.028

$= 351.028\ MPa$

$x_u = 332.257 - 0.454 \times 351.028 = 172.890\,mm$

3rd trial,

$x_u = \frac{206+172}{2} = 189$

$e_{sc} = 0.0035\left(1-\frac{45}{189}\right) = 0.00266$ ∴ code page 36

0.00241 → 342.8
0.00276 → 351.8
0.00266 y = 349.228 MPa

$f_{sc} = 342.8 + \frac{(351.8-342.8) \times (266-241)}{276-241}$

$= 349.228\ MPa$

$x_u = 332.257 - 0.454 \times 349.228 = 173.707\,mm$

So, neutral axis depth $(x_u) = 173.707\,mm$

and, Moment of Resistance

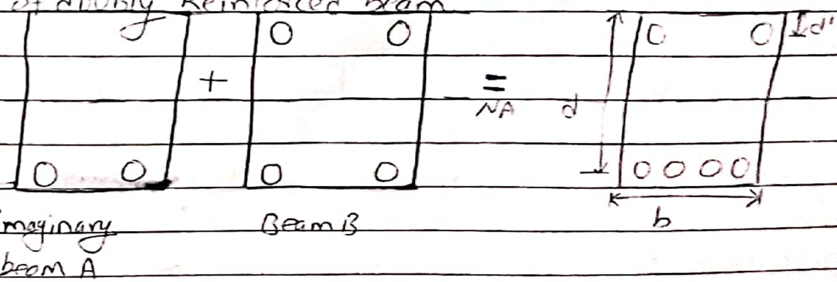
$M_{uR} = 0.36 f_{ck} \cdot b \cdot x_u (d - 0.42 x_u) + (f_{sc} - 0.446 f_{ck}) A_{sc} (d-d')$

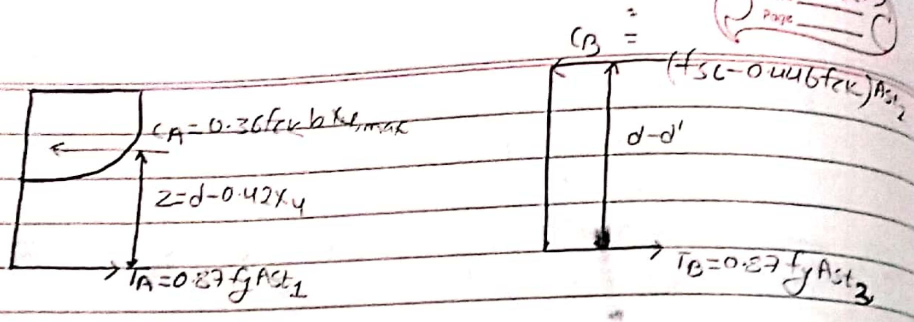
$= 0.36 \times 20 \times 300 \times 173.707 (655 - 0.42 \times 173.707) + (349.228 - 0.446 \times 20) \times 981.747 (655-45)$

$= 422185478.7 \times 10^{-6}$

$= 422.185\ kNm$ Ans

Design of doubly Reinforced beam





1) Calculate the moment of resistance of singly reinforced balanced section

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

2) If applied moment is greater than MOR of singly reinforced balanced section i.e. $M_u > M_{uL}$, then design doubly reinforced section is designed as

$M_u - M_{uL} = 0.87 f_y A_{st_2} (d-d') = (f_{sc} - 0.446 f_{ck}) A_{sc} (d-d')$

3) Calculate area of tension steel A_{st1} corresponding to singly reinforced balanced section

$C_A = T_A \Rightarrow 0.36 f_{ck} b x_{u,max} = 0.87 f_y A_{st1}$

4) Calculate the area of tension steel A_{st2}, corresponding to additional moment.

$A_{st_2} = \dfrac{M_u - M_{uL}}{0.87 f_y (d-d')}$

5) Total tension steel required, $A_{st} = A_{st1} + A_{st2}$

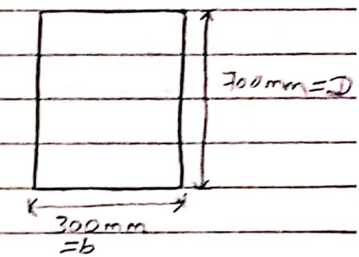
6) Calculate the area of compression steel

OR $M = \dfrac{wl^2}{4}$ जस्तै आउँछ moment ← given moment (Mu) भन्दा calculated MuL of singly reinforced balanced section कम छ भने मात्र doubly reinforced section design गर्ने

$A_{sc} = \dfrac{M_u - M_{uL}}{(f_{sc} - 0.446 f_{ck})(d-d')} = \dfrac{0.87 f_y A_{st2}}{(f_{sc} - 0.446 f_{ck})}$

8) Design a rectangular beam of effective length 6m. The superimposed load is 80 kN/m. Size of beam is restricted to 30cm × 70cm overall. Use M20 mix and Fe415 steel.

Soln



effective length (le) = 6m

calculation of load

Superimposed load = 80 kN/m

γA = Self wt = 25 × 0.3 × 0.7 kN/m

25 kN/m³ for RCC = 5.25 kN/m

Total load = 85.25 kN/m

(wu) factored load = 1.5 × 85.25 = 127.875 kN/m

factored moment (Mu) $= \dfrac{w_u l_e^2}{8} = \dfrac{127.875 \times 6^2}{8} = 575.437$ kNm

assume $d' = 10\%$ of $D = 0.1D = 0.1 \times 700 = 70$mm

effective depth (d) = 700 − 70 = 630mm

$x_{u,max} = 0.48d = 0.48 \times 630 = 302.4$mm — for Fe415

MOR of singly reinforced balanced section, $M_{uL} = 0.36 f_{ck} b x_{u,max} (d - 0.42 x_{u,max})$

$= 0.36 \times 20 \times 300 \times 302.4 (630 - 0.42 \times 302.4) \times 10^{-6}$ kNm

IMP = 328.546 kNm

Since, $M_u > M_{uL}$, doubly reinforced section has to be designed area of tension steel corresponding to singly reinforced balanced section. i.e CA=TA

$$Ast_1 = \frac{0.36 f_{ck}\, b\, x_{u,max}}{0.87 f_y}$$

$$0.36 f_{ck}\, b\, x_{u,max} = 0.87 f_y\, Ast_1$$

$$\therefore Ast_1 = \frac{0.36 f_{ck} \cdot b \cdot x_{u,max}}{0.87 f_y}$$

$$= \frac{0.36 * 20 * 300 * 302.4}{0.87 * 415}$$

$$= 1809.123\ mm^2$$

area of tension steel corresponding to additional moment

$$Ast_2 = \frac{M_u - M_{uL}}{0.87 f_y (d-d')}$$

$$= \frac{(575.437 - 328.546) * 10^6}{0.87 * 415 (630-70)} = 1221.096\ mm^2$$

Total tension steel, $Ast = Ast_1 + Ast_2$

$$= 1809.123 + 1221.096$$

$$= 3030.219\ mm^2$$

provide 5-28mm ϕ rebar

$Ast, provided = 5 * 615 = 3075\ mm^2 > 3030.219\ mm^2$ OK

Minimum Ast to be provided

$$Ast,min = \frac{0.85\, bd}{f_y}$$

$$= \frac{0.85 * 300 * 630}{415} = 387.108\ mm^2 < 3075\ mm^2 \quad OK$$

max^m Ast that can be provided, $Ast,max = 0.04\, bD$

$$= 0.04 * 300 * 700$$

$$= 8400\ mm^2 > 3075\ mm^2 \quad OK$$

area of compression steel, $Asc = \frac{M_u - M_{uL}}{(f_{sc} - 0.446 f_{ck})(d-d')}$

$$= \frac{575.437 - 328.546}{(f_{sc} - 0.446*20)(630-70)}$$

To find f_{sc}

$\frac{d'}{d} = \frac{70}{630} = 0.11$ code page 38

f_{sc} 415	0.10	0.15
	353	342

$0.11\ y = 350.8$

$$f_{sc} = 353 + \frac{(342-353) * (0.11-0.1)}{(0.15-0.1)}$$

$$= 350.8\ Mpa$$

now

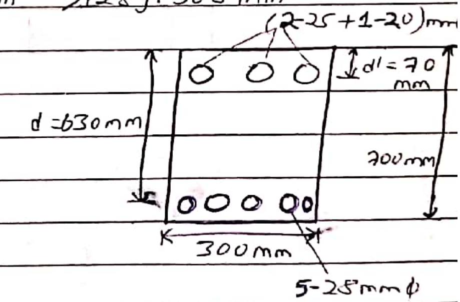
$$Asc = \frac{(575.437 - 328.546) * 10^6}{(350.8 - 0.446*20)(630-70)}$$

$$= 1289.565\ mm^2$$

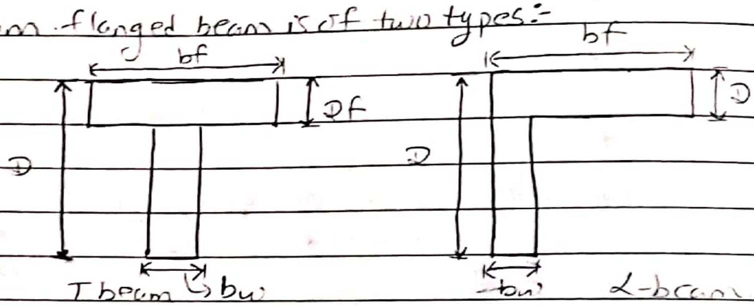
provide (2-25 + 1-20)mm ϕ rebar

$Asc, provided = 2*490 + 1*314$

$= 1294\ mm^2 > 1289.565\ mm^2$



Flanged beam :- In most construction, beam and slab are cast monolithically. web portion called beam, bears tensile force where as flange portion slab bears compressive force. such combination of beam with web and flange is called flanged beam. flanged beam is of two types :-



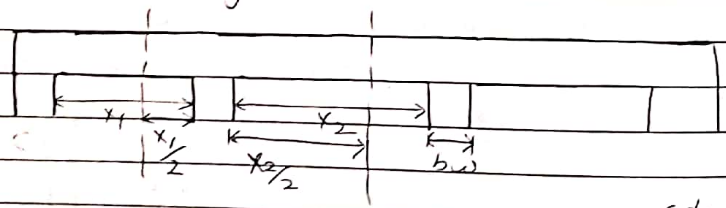
bf = width of flange

bw = width of web/rib.

Df = Depth of flange.

D = overall depth.

effective width of flange, bf



$bf = \frac{L_0}{6} + bw + 6Df$
$= bw + \frac{x_1}{2} + \frac{x_2}{2}$
} lesser of two values

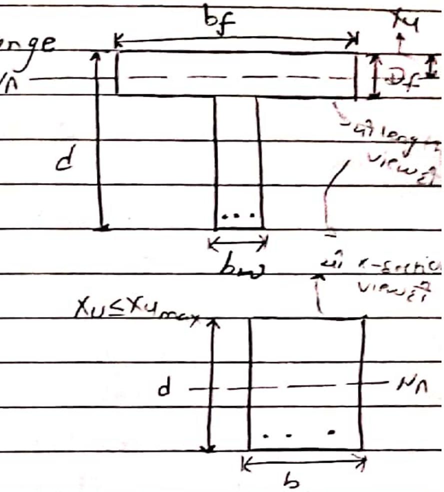
code page 6
C.L 23.1.2

Types of problem

1) when neutral axis lies within flange
depth of neutral axis, $(x_u \le Df)$

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

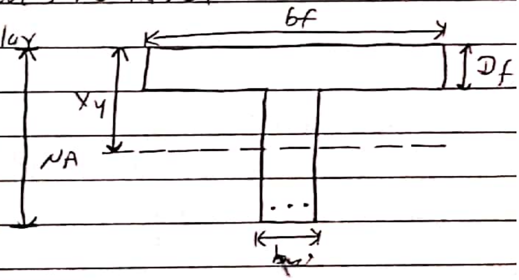
$$MOR = 0.36 f_{ck} \cdot bf \cdot x_u (d - 0.42 x_u)$$
$$= 0.87 f_y A_{st} \left(d - \frac{f_y A_{st}}{f_{ck} \cdot bf}\right)$$



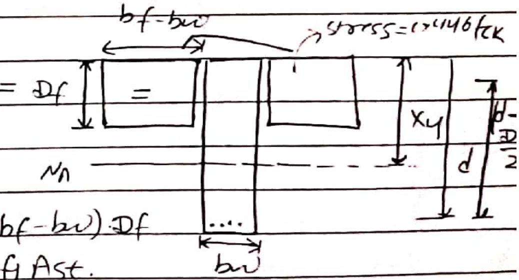
T figure का bf और rect. b same है।

2) when neutral axis lies within web i.e $x_u > Df$

case I) when depth of rectangular stress block diagram is greater than depth of flange
i.e. $\frac{3x_u}{7} > Df \Rightarrow \frac{Df}{x_u} < 0.428$



Total compressive force
(C) = compressive force by rectangular area $(bw \cdot x_u)$
+ compressive force by rectangular area
$(bf - bw) * Df$



or, $C = 0.36 f_{ck} \cdot bw \cdot x_u + 0.446 f_{ck} (bf - bw) \cdot Df$

Total tensile force $(T) = 0.87 f_y A_{st}$.

$x_u \le x_{u,max}$

as C=T

as $0.36 f_{ck} b_w x_u + 0.446 f_{ck} (b_f - b_w) D_f = 0.87 f_y A_{st}$

or $x_u = \frac{0.87 f_y A_{st} - 0.446 f_{ck} (b_f - b_w) D_f}{0.36 f_{ck} b_w}$

$MoR = 0.36 f_{ck} b_w x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) D_f * \left(d - \frac{D_f}{2}\right)$

$x_u > D_f$

$x_u > \frac{D_f}{0.428}$

$x_u \le x_{u,max}$

Case III when depth of rectangular stress block diagram is less than depth of flange i.e.

$\frac{3x_u}{7} < D_f \Rightarrow \frac{D_f}{x_u} > 0.428$

In this case, depth of flange D_f is replaced by equivalent depth of flange (y_f) where

$y_f = 0.15 x_u + 0.65 D_f \not> D_f$ (should not be greater than D_f)

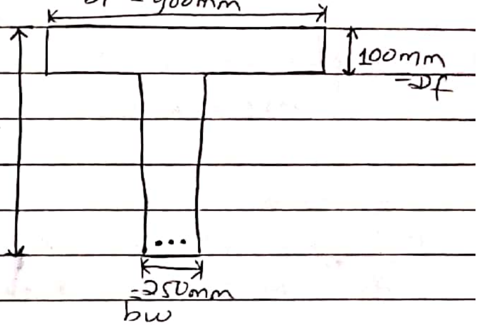
now, C=T

$0.36 f_{ck} b_w x_u + 0.446 f_{ck} (b_f - b_w) y_f = 0.87 f_y A_{st}$

$x_u = \frac{0.87 f_y A_{st} - 0.446 f_{ck} (b_f - b_w) y_f}{0.36 f_{ck} b_w}$

$MoR = 0.36 f_{ck} b_w x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) y_f \cdot (d - 0.5 y_f)$

Q7 A T beam of flange width 900 mm (b_f =), thickness 100 mm (D_f =) rib width 250 mm (b_w =) has effective depth 525 mm (d =) is reinforced with steel area of 4909 mm^2 (A_{st}) find the ultimate moment of resistance of section use M20 and Fe250 steel.



Soln

$b_f = 900mm$, $D_f = 100mm$

$b_w = 250mm$, $d = 525mm$

$A_{st} = 4909 mm^2$

$x_{u,max} = 0.53d$ for Fe250

$= 0.53 * 525$

$= 278.25 mm$

Assuming neutral axis lies within flange i.e. $x_u < D_f$

$\Rightarrow x_u \le 100mm$

and $x_u < x_{u,max} \Rightarrow x_u < 278.25mm$

$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 * 250 * 4909}{0.36 * 20 * 900} = 164.77 mm < 100mm$ not OK

So our assumption is wrong.

assume $x_u \le x_{u,max}$ and $\frac{D_f}{x_u} < 0.428 \Rightarrow x_u > \frac{D_f}{0.428}$

Assuming neutral axis lies within web, ($x_u > D_f$)

$\Rightarrow x_u > \frac{100}{0.428} = 233.645 mm$

$x_u = \frac{0.87 f_y A_{st} - 0.446 f_{ck} (b_f - b_w) D_f}{0.36 f_{ck} b_w}$

$= \frac{0.87 * 250 * 4909 - 0.446 * 20 (900 - 250) * 100}{0.36 * 20 * 250}$

$= 271.059 mm > 100mm$ | $x_u > D_f$

$> 233.645 mm$ | $x_u > 233.645$ i.e. $> \frac{D_f}{0.428}$

$< 278.25 mm$ | $x_u \le x_{u,max}$

OK

So, neutral axis depth (x_u) = 271.059 mm.
and Moment of resistance

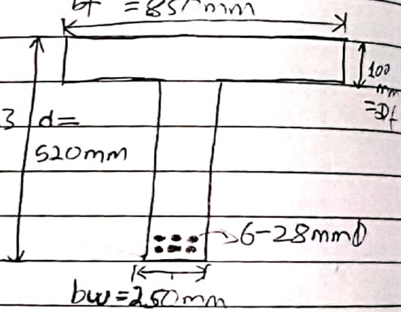
$= 0.36 f_{ck} \cdot b_w \cdot x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) D_f (d - 0.5 D_f)$

$= [0.36 * 20 * 250 * 271.059 (525 - 0.42 * 271.059) + 0.446 * 20 * (900 - 250) * 100 (525 - 0.5 * 100)] * 10^{-6}$

$= 476.01$ KNm

10) Determine the ultimate moment of resistance of T-section as shown. Use M20 mix and Fe250 steel.

Soln



$b_f = 850$mm, $D_f = 100$mm, $b_w = 250$mm

$d = 520$mm $A_{st} = 6 * \frac{\pi}{4} * 28^2 = 3694.513$ mm^2

$x_{u,max} = 0.53d = 0.53 * 520 = 275.6$mm

assuming neutral axis lies within flange i.e $x_u \le D_f \Rightarrow x_u \le 100$mm & $x_u \le x_{u,max}$

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

$= \frac{0.87 * 250 * 3694.513}{0.36 * 20 * 850}$

$= 131.30$mm > 100mm $= D_f$

not OK

assuming neutral axis lies in web that is $x_u > 100$mm & $\frac{D_f}{x_u} < 0.428 \Rightarrow x_u > \frac{100}{0.428} = 233.645$ mm, $x_u \le x_{u,max}$

$x_u = \frac{0.87 f_y A_{st} - 0.446 f_{ck} (b_f - b_w) D_f}{0.36 f_{ck} b_w}$

$= \frac{0.87 * 250 * 3694.513 - 0.446 * 20 * (850 - 250) * 100}{0.36 * 20 * 250}$

$= 149.087$ mm < 233.645 mm ($x_u > 233.645$ mm) not satisfied

(not OK)

assuming neutral axis lies in web i.e $x_u > 100$ mm & $\frac{D_f}{x_u} > 0.428$

$\Rightarrow x_u < \frac{100}{0.428} \Rightarrow x_u < 233.645$mm & $x_u < x_{u,max}$

equivalent depth of flange (y_f) $= 0.15 x_u + 0.65 D_f \ngtr D_f$

$= 0.15 x_u + 0.65 * 100$

$= 0.15 x_u + 65$

$C = T$

or, $0.36 f_{ck} b_w x_u + 0.446 f_{ck} (b_f - b_w) y_f = 0.87 f_y A_{st}$

or, $0.36 * 20 * 250 x_u + 0.446 * 20 (850 - 250)(0.15 x_u + 65) = 0.87 * 250 * 3694.513$

on solving we get

$x_u = 175.071$ mm > 100mm ($x_u > D_f = 100$ OK)

< 233.645mm ($x_u < \frac{D_f}{0.428} = 233.645$ OK)

$\le x_{u,max} = 275.6$mm ($x_u \le x_{u,max} = 275.6$mm OK)

So, neutral axis depth (x_u) = 175.071 mm

$y_f = 0.15 x_u + 65 \ngtr D_f$

$= 0.15 * 175.071 + 65 \ngtr 100$

$= 91.261$mm $\ngtr 100$mm OK

So, Moment of resistance $= 0.36 f_{ck} b_w x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w) y_f (d - 0.5 y_f)$

$= [0.36 * 20 * 250 * 175.071 * (520 - 0.42 * 175.071) + 0.446 * 20 * (850 - 250) * 91.261 * (520 - 0.5 * 91.261)] * 10^{-6}$

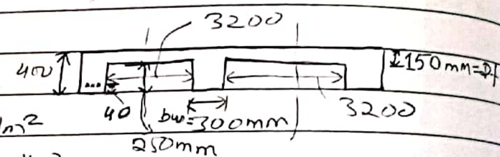
$= 372.341$ KNm

1) A T-beam floor consists of 15cm thick slab monolithic with 30cm wide beams. The beams are spaced at 3.5m centre to centre and effective span length 6m. If superimposed load on slab is 5kN/m². Design an intermediate beam. Use M20 mix and Fe250 steel.

soln

$D_f = 150mm$, $b_w = 300mm$

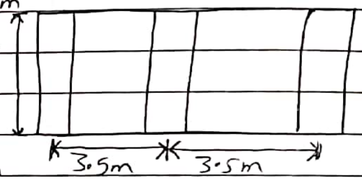
superimposed load = 5 kN/m²



self wt. of slab = 25 × 0.15 kN/m²

$\gamma \times D_f = kN/m^2$

= 3.75 kN/m²



Total load = 8.75 kN/m²

load on beam = 8.75 × 3.5 kN/m

= 30.625 kN/m

assuming overall depth of beam 400mm with effective cover 40mm

effective depth (d) = 400 − 40 = 360 mm

self wt. of portion of beam = 25 × 0.3 × 0.25 kN/m

$\gamma_{concrete}$ = 1.875 kN/m → $b_w (d - D_f)$ = 400 − 150 = 250mm

Total load = 32.5 kN/m → (30.625 + 1.875)

factored load (w_u) = 32.5 × 1.5 = 48.75 kN/m

Factored moment (M_u) = $\frac{w_u l^2}{8} = \frac{48.75 \times 6^2}{8}$ = 219.375 kNm

assuming neutral axis lies within flange $x_u \le 150mm$ and $x_u \le x_{u,max}$

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

or, $219.375 \times 10^6 = 0.87 \times 250 \times A_{st}\left(360 - \frac{250 \times A_{st}}{20 \times 2200}\right)$

on solving we get. A_{st} = 2937.95 mm²

$= \frac{x_1}{2} + \frac{x_2}{2} + b_w$ breadth of flange $b_f = \frac{l_0}{6} + b_w + 6D_f$

$= \frac{3200}{2} + \frac{3200}{2} + 300$ = 3500 mm $= \frac{6000}{6} + 300 + 6 \times 150$ = 2200mm

so small b_f = 2200mm

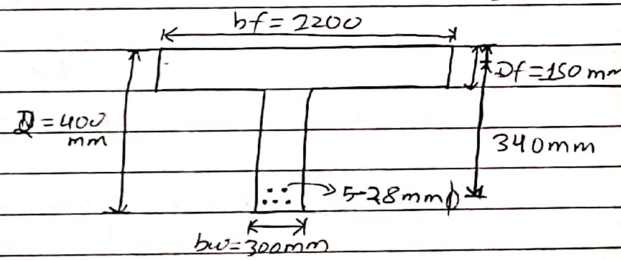
provide 5-28mmϕ rebars, A_{st}, provided = 5 × 615 = 3075mm² > 2937.95mm² OK

$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 \times 250 \times 3075}{0.36 \times 20 \times 2200}$ = 42.223 mm < 150mm < $x_{u,max}$

$x_u < D_f$ हुँदा OK

$A_{st,min} = \frac{0.85 b_w d}{f_y} = \frac{0.85 \times 300 \times 360}{250}$ = 367.2mm² < 3075mm² OK

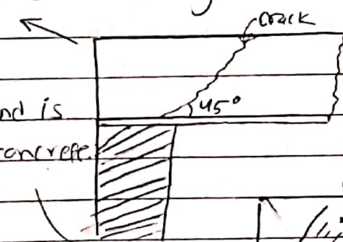
$A_{st,max}$ = 4% of $b_w \cdot D$ = 0.04 × 300 × 400 = 4800mm² > 3075mm² OK



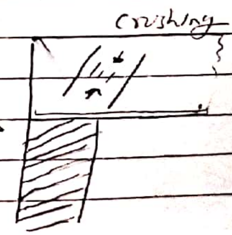
Design for shear.

Types of shear failure.

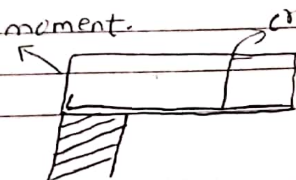
1) Diagonal tension failure:- occurs under large shear force and less bending moment.



2) Diagonal compression failure:- occurs under large shear force and is characterised by crushing of concrete.



3) flexural tension failure occurs under large shear force and large bending moment.



Shear strength of beam = shear strength of concrete + shear strength of shear reinforcement

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

V_u = factored shear force.

V_{us} = shear force to be resisted by shear reinforcement

V_{uc} = shear force resisted by concrete $= \tau_c b d$

where

τ_c = design shear stress of concrete

τ_c depends upon % of steel and grade of concrete (code page 40 – table 19)

shear reinforcement is needed when $\tau_c < \tau_u < \tau_{c,max}$

τ_u = ultimate shear stress $= \frac{V_u}{b.d}$.

$\tau_{c,max}$ = Maximum shear stress of concrete (table 20, page 40)

shear force to be resisted by shear reinforcement,

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

$= V_u - \tau_c.b.d$

* Spacing of vertical stirrups or shear reinforcement

S_v = spacing of vertical stirrups

d = effective depth.

A_{sv} = area of one vertical stirrups

shear force resisted by shear reinforcement,

$V_{us} = \frac{f_y}{1.15} \times \frac{A_{sv}.d}{V_{us}}$

$S_v \to A_{sv}$

$d \to \frac{A_{sv}.d}{S_d}$

$\therefore \frac{A_{sv}.d}{S_v}$

code page 40A

For inclined stirrups or a series of bars bent-up at different cross-section

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

α = angle of inclined stirrups or bar to the axis of beam.

For single bar or single group of parallel bars, all bent-up at the same cross section:

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

Minimum area of shear reinforcement

$A_{sv} \geq \frac{0.4 b S_v}{0.87 f_y}$ (CL 26.5.1.6, page 48)

spacing. $S_v \geq 100mm$

$< 0.75d$

$< 300mm$

12) A RCC beam has effective depth of 500mm and breadth 350mm. It contains 4-25mm φ bars. Calculate shear reinforcement needed for a factored shear force of 350kN for (i) M20 & Fe250 (ii) M25 and Fe415

Soln

i) factored shear force (V_u) = 350kN, d = 500mm, b = 350mm

% of steel $= \frac{A_{st} \times 100\%}{b.d} = \frac{4 \times \frac{\pi}{4} \times 25^2}{350 \times 500} = 0.011 = 1.121\%$ ↓ code page 40 for M20

Case I) Design shear stress of concrete (τ_c) =

$0.62 + \frac{(0.67 - 0.62)(1.121 - 1)}{(1.25 - 1)} = 0.644 N/mm^2$

$1.0 \to 0.62$

$1.25 \to 0.67$

$1.121 \to y = 0.644 N/mm^2$

Ultimate shear stress,

$\tau_u = \frac{V_u}{b.d} = \frac{350 \times 10^3}{350 \times 500} = 2 N/mm^2$

code pg 40 table 20

$\tau_{c\,max}$ for M20 = 2.8 N/mm²

Date ____ Page ____

Since, $\tau_c < \tau_v < \tau_{c\,max}$, shear reinforcement should be provided shear force to be resisted by shear reinforcement

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

$= 350 - 0.644 \times 350 \times 500 \times 10^{-3}$

$= 237.3$ kN

Providing 2-legged 8mm ϕ vertical stirrups, $A_{sv} = 2 \times \frac{\pi \times 8^2}{4}$

$= 100.53 mm^2$

Spacing

$S_v = \frac{0.87 f_y A_{sv} \cdot d}{V_{us}} = \frac{0.87 \times 250 \times 100.53 \times 500}{237.3 \times 10^3}$

$= 46.07 mm < 100 mm$

not OK

Providing 100mm spacing

$A_{sv} = \frac{V_{us} \cdot S_v}{0.87 f_y d} = \frac{237.3 \times 10^3 \times 100}{0.87 \times 250 \times 500} = 218.206 mm^2$

or, $\frac{2 \times \pi \times \phi^2}{4} = 218.206$

$\therefore \phi = \sqrt{\frac{2 \times 218.206}{\pi}} = 11.786 \simeq 12mm$

minimum Area of shear reinforcement, $A_0 = \frac{0.4 b \cdot S_v}{0.87 f_y}$

$= \frac{0.4 \times 250 \times 100}{0.87 \times 250}$

$= 64.36 mm^2 < 226.19 mm^2$

OK

$\frac{2 \times \pi d^2}{4}$, 12mm

So, provide 2-legged 12mm ϕ vertical stirrups at spacing 100mm c/c.

पाठशाला

Date ____ Page ____

ii) factored shear force (V_u) = 350 kN, d = 500mm, b = 350mm

% of steel $= \frac{A_{st}}{bd} \times 100\% = \frac{4 \times \frac{\pi}{4} \times 25^2}{350 \times 500} = 0.011 = 1.121\%$

code page 40 for M25

Design shear stress of concrete (τ_c)

1.00 → 0.64

1.121 → ?

1.25 → 0.70

1.121 ŷ = 0.669 N/mm²

From interpolation

$\tau_c = 0.669 N/mm^2$

ultimate shear stress

$\tau_v = \frac{V_u}{bd} = \frac{350 \times 10^3}{350 \times 500} = 2 N/mm^2$

$\tau_{c\,max}$ for M25 = 3.1 N/mm² code pg 40 table 20.

Since, $\tau_c < \tau_v < \tau_{c\,max}$, shear reinforcement should be provided. Shear force to be resisted by shear reinforcement.

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

$= 350 - 0.669 \times 350 \times 500 \times 10^{-3}$

$= 232.925$ kN.

Providing 2-legged 8mm ϕ vertical stirrups, $A_{sv} = \frac{2 \times \pi \times 8^2}{4} = 100.53 mm^2$

Spacing

$S_v = \frac{0.87 f_y A_{sv} d}{V_{us}} = \frac{0.87 \times 415 \times 100.53 \times 500}{232.925 \times 10^3} = 77.93 < 100mm$

As $S_v \geq 100mm$ is not satisfied, so not OK

Providing 100mm spacing

$A_{sv} = \frac{V_{us} \cdot S_v}{0.87 f_y d} = \frac{232.925 \times 10^3 \times 100}{0.87 \times 415 \times 500} = 129.026 mm^2$

or, $\frac{2 \times \pi \times \phi^2}{4} = 129.026$ $\therefore \phi = \sqrt{\frac{129.026 \times 4}{2 \times \pi}} = 9.06 \simeq 10mm$

minimum area of shear reinforcement $A_0 = \frac{0.4 b \cdot S_v}{0.87 f_y} = \frac{0.4 \times 350 \times 100}{0.87 \times 415}$

$= 38.775 mm^2$

$\frac{2 \times \pi d^2}{4}$, 10 $= \frac{2 \times \pi \times 10^2}{4} = 157.079 mm^2$

पाठशाला

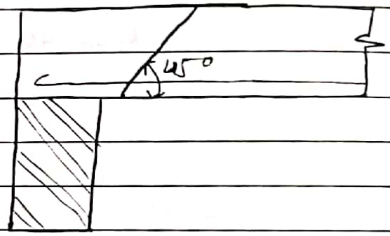
So, provide 2-legged 10mmϕ vertical stirrups at spacing of 100mm c/c.

2015 spring (5)

*) Explain in brief the type of shear failure in beam with neat sketches.

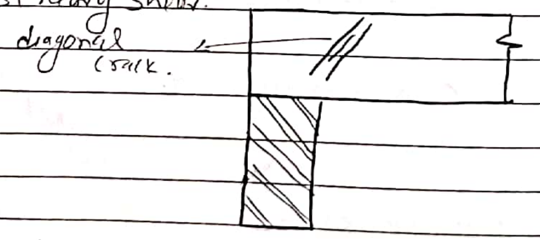
- Diagonal Tension failure.

If a section is under large shear force and small bending moment a diagonal tension crack is observed. Such cracks are normally at 45° with the horizontal.



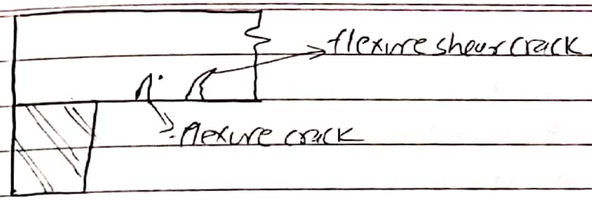
- Diagonal compression failure.

If the section is under large shear force and fails by forming a compression crack. It is indicated by crushing on concrete at support and spalling. Normally, it occurs in beam which are reinforced against heavy shear.



- Flexure shear failure

If the section is under large bending moment but small shear force, the section fails by forming a large vertical crack at around mid-span at 90°.



Mathematically $\tan 2\alpha = \frac{2\tau}{\sigma_b}$

at support.

$\sigma_b \simeq 0$

$\therefore \tan 2\alpha = \frac{2\times\tau}{0} = \infty$

or, $2\alpha = \tan^{-1}(\infty) = \frac{\pi}{2}$

$\therefore \alpha = \frac{\pi}{4} = 45°$

at mid-span: $\tau \simeq 0$

$\tan 2\alpha = \frac{2\times 0}{\sigma_b} = 0$

or, $2\alpha = \tan^{-1}(0) = 0$

$\therefore \alpha = 0$ (with vertical)

$\therefore \alpha = 90°$ (with horizontal.

2016 fall 1a) Explain about the partial safety factor & factor of safety. Derive the necessary expression for the development length (both anchorage & flexural bond).

1st part see 2.6

2nd part.

a) Anchorage bond

b) flexural bond

$\therefore \tau_{bd} = \frac{V}{\pi\phi\cdot d}$

This equation gives the flexural bond stress in the tension reinforcement at any section of the beam. If there are bars of different sizes then $\tau_{bd} = \frac{V}{\pi\phi\,d\cdot N}$

where N = no of bars

From Anchorage bond we have

$$0.87 f_y A_{st} = \tau_{bd} \cdot \pi \Phi L_d N$$

using above equation we get.

$$0.87 f_y A_{st} = \frac{V}{\Sigma \Phi \alpha \cdot N} \times \pi \Phi L_d N$$

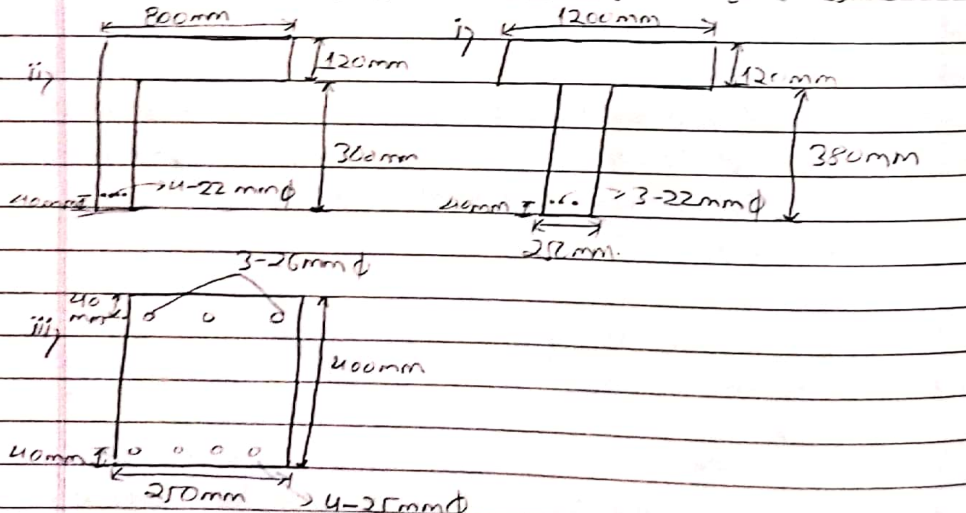
or, $$0.87 f_y A_{st} = \frac{V \cdot L_d}{\alpha}$$

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

M is further increased by 30% for compression.

Then, $$L_d = \frac{1.3 M_1}{V} + L_o$$

13) A beam shown in fig. is subjected to factored shear force of 200 kN. Calculate shear reinforcement. Use M20 mix and Fe415 steel.



i) sol^n

$d = 120 + 380 - 40 = 460$ mm

$$\% \text{ of steel} = \frac{A_{st}}{b_w d} \times 100\% = \frac{3 \times \frac{\pi}{4} \times 22^2}{250 \times 460} \times 100\% = 0.991 \simeq 1\%$$

for 1% & M20 $\tau_c = 0.62$ N/mm²

design shear stress of concrete $(\tau_c) = 0.62$ N/mm², code given

$\tau_{c,max} = 2.8$ N/mm²

ultimate shear stress $(\tau_u) = \frac{V_u}{b_w d} = \frac{200 \times 10^3}{250 \times 460} = 1.739$ N/mm²

$\tau_c < \tau_u < \tau_{c,max}$ so, shear reinforcement is required

shear force to be resisted by shear reinforcement, $V_{us} = V_u - \tau_c \cdot b \cdot d$

$$= 200 - 0.62 \times 250 \times 460 \times 10^{-3}$$

$$= 128.7 \text{ kN}$$

$V_u = V_{uc} + V_{us}$ ($V_{uc} = \tau_c b d$)

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

Providing 8mm φ 2-legged vertical stirrups, $A_{sv} = 2 \times \frac{\pi}{4} \times 8^2$

$$= 100.53 \text{ mm}^2$$

spacing, $$S_v = \frac{0.87 f_y A_{sv} d}{V_{us}} = \frac{0.87 \times 415 \times 100.53 \times 460}{128.7 \times 10^3}$$

$= 129.73$ mm $\simeq 128$ mm

≥ 100 mm

≤ 300 mm

$\leq 0.75d = 0.75 \times 460 = 345$ mm ($120 + 380 - 40 = 460$ mm)

minimum area of shear reinforcement, $$A_0 = \frac{0.4 b S_v}{0.87 f_y}$$

$$= \frac{0.4 \times 250 \times 128}{0.87 \times 415}$$

$$= 35.45 \text{ mm}^2 < 100.53 \text{ mm}^2$$

So, provide 8mm ϕ 2-legged vertical stirrup @128mm c/c.

iii) Soln

$d = 400-40 = 360mm$

$\% of\ steel = \frac{Ast}{b.d} \times 100\% = \frac{4 \times \frac{\pi}{4} \times 25^2}{250 \times 360} = 2.181\%$

design shear stress of concrete $(\tau_c) = 0.79 + \frac{(0.81-0.79)}{(2.25-2)} \times (2.181-2)$

$= 0.804 N/mm^2$

Ultimate shear stress $(\tau_u) = \frac{Vu}{bd} = \frac{200 \times 10^3}{250 \times 360} = 2.22 N/mm^2$

$\tau_c < \tau_u < \tau_{c,max}$ so, shear reinforcement is required.

$0.804 N/mm^2 \quad 2.22 N/mm^2 \quad 4.28 N/mm^2$

Shear force to be resisted by shear reinforcement,

$Vus = Vu - \tau_c b.d = 200 - 0.804 \times 250 \times 360 \times 10^{-3}$

$= 127.64 kN.$

providing 8mm ϕ 2-legged vertical stirrups, $Asv = \frac{2 \times \pi \times 8^2}{4}$

$= 100.53 mm^2$

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

$= \frac{0.87 \times 415 \times 100.53 \times 360}{127.64 \times 10^3}$

$= 102.37 mm \simeq 100mm$

$\geq 100mm$

$< 300mm$

$< 0.75d = 0.75 \times 360 = 270mm$

minimum area of shear reinforcement, $A_{sv} = \frac{0.4 b.Sv}{0.87 fy}$

$= \frac{0.4 \times 250 \times 100}{0.87 \times 415}$

$= 27.696 mm^2 < 100.53 mm^2$

So, provide 8mm ϕ 2-legged vertical stirrups @ 100mm c/c.

iv) Soln

$d = 120 + 300 - 40 = 380mm$

$\% of\ steel = \frac{Ast}{b.d} \times 100\% = \frac{4 \times \frac{\pi}{4} \times 22^2}{300 \times 380} \times 100\% = 1.33\%$

design shear stress of concrete $(\tau_c) =$

$1.25 \rightarrow 0.62$

$1.50 \rightarrow 0.72$

$1.33 \rightarrow 0.652 N/mm^2$

ultimate shear stress $(\tau_u) = \frac{Vu}{b.d} = \frac{200 \times 10^3}{300 \times 380} = 1.754 N/mm^2$

$\tau_c < \tau_u < \tau_{c,max}$. so shear reinforcement is required

$0.652 \quad 1.754 \quad 2.8$

shear force to be resisted by shear reinforcement,

$Vus = Vu - \tau_c.b.d = 200 - 0.652 \times 300 \times 380 \times 10^{-3}$

$= 125.672 kN.$

providing 8mm ϕ 2-legged vertical stirrups

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

$= 100.53 mm^2$

Spacing $Sv = \frac{0.87 fy Asv.d}{Vus} = \frac{0.87 \times 415 \times 100.53 \times 380}{125.672 \times 10^3} = 109.75$

$\simeq 110mm$

$\geq 100mm$

So provide 8mm ϕ 2-legged vertical stirrups at 100mm c/c

Design of beam for torsion.

Design for shear.

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

where V_u = ultimate shear force

(CL 41.3.1 page 112)

T_u = torsional moment

b = breadth of beam

Design for bending moment

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

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

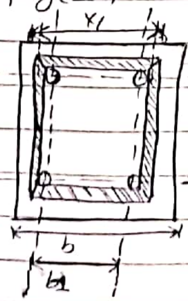
if $M_t > M_u$; longitudinal flexural reinforcement in compression zone should be provided. with equivalent moment $M_{e2} = M_t - M_u$. M_{e2} is assumed to act in opposite sense to that of of M_{e1}.

Note: If overall depth of beam exceeds 750mm i.e $D > 750$mm, side face reinforcement should be provided with area of steel not less than 0.1% of bD. (CL 26.5.1.3, page 27)

Spacing of shear reinforcement, S_v.

b_1 = centre to centre distance betwn longitudinal bars in the dirn of breadth.

d_1 = centre to centre distance betwn longitudinal bars in the dirn of depth.



x_1 = shortest dimension of centre to centre distance between legs of stirrups

y_1 = longest dimension of centre to centre distance betn legs of stirrups

$S_v = \frac{0.87 f_y A_{sv} d_1}{\frac{T_u}{b_1} + \frac{V_u}{2.5}}$ V_u = factored shear force.

$S_v \le x_1$ (CL 41.4.3 pg 42)

$\le \frac{(x_1+y_1)}{4}$

< 300mm (CL 26.5.1.7 page 18)

≥ 100mm.

minimum area of shear reinforcement $A_0 = \frac{(\tau_e - \tau_c) b S_v}{0.87 f_y}$

14) Design a section of rectangular beam 50cm wide & 70cm deep subjected to a bending moment of 200 kNm, twisting moment of 15 kNm (torsional moment) & shear force of 150 kN at ultimate. Use M20 mix and Fe415 grade steel.

Soln

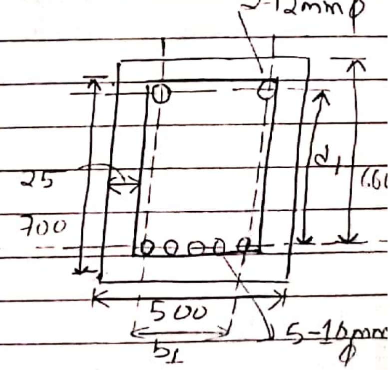
$M_u = 200$ kNm, $T_u = 15$ kNm

$V_u = 150$ kN

assuming 40mm effective cover, effective depth $(d) = 700 - 40 = 660$ mm

Design for bending moment

equivalent torsional moment



$M_t = \frac{T_u\left(1+\frac{D}{b}\right)}{1.7} = \frac{15\left(1+\frac{700}{500}\right)}{1.7} = 21.176$ kNm

200 kNm 221.176 kNm

Since $M_u > M_t$, no need of longitudinal flexural reinforcement in compression zone.

equivalent bending moment, $M_{e_1} = M_u + M_t = 200 + 21.176$
$= 221.176$ kNm

$x_{u,max} = 0.48d$ for Fe415
$= 0.48 * 660 = 316.8$ mm

MOR of singly reinforced balanced section,
$M_{ul} = 0.36 f_{ck} b\, x_{u,max} (d - 0.42 x_{u,max})$
$= 0.36 * 20 * 500 * 316.8 * (660 - 0.42 * 316.8) \times 10^{-6}$
$= 600.969$ kNm

600.969 kNm 221.176 kNm

Since, $M_{ul} > M_{e_1}$, singly reinforced section is designed.

area of reinforcement for M_{e1}

$M_{e_1} = 0.87 f_y A_{st} \left(d - \frac{f_y A_{st}}{f_{ck} b}\right)$

or, $221.176 * 10^6 = 0.87 * 415 * A_{st} \left(660 - \frac{415 A_{st}}{20 * 500}\right)$

on solving we get
$A_{st} = 989.766$ mm²

provide 5-16mm ϕ rebars, A_{st} provided $= 5 \times 201 = 1005$ mm²
> 989.766 mm²
OK

check for neutral axis depth,

$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times 1005}{0.36 * 20 * 500} = 100.793$ mm
$< x_{u,max}$
OK

minimum area of steel $(A_{st})_{min} = \frac{0.85\, bd}{f_y}$
$= \frac{0.85 * 500 * 660}{415} = 675.903$ mm²
< 1005 mm²

maxm area of steel $(A_{st})_{max} = 4\%$ of bD
$= 0.04 * 500 * 700$
$= 14000$ mm² > 1005 mm² ok

Design for shear

equivalent shear force $(V_e) = V_u + 1.6 \frac{T_u}{b} = 150 + 1.6 * \frac{15}{0.5}$ (→500mm)
$= 198$ kN

ultimate shear stress $(\tau_v) = \frac{V_e}{bd} = \frac{198 * 10^3}{500 \times 660} = 0.6$ N/mm²

% of steel $= \frac{A_{st}}{b \cdot d} \times 100\% = \frac{1005}{500 * 660} * 100 = 0.304\%$

Design shear stress of concrete, $\tau_c = 0.36 + \frac{(0.48 - 0.36)}{(0.5 - 0.25)} \times (0.304 - 0.25)$
$= 0.385$ N/mm²
$= 0.385$ N/mm²

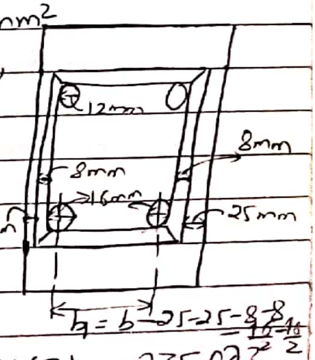
$\tau_{c,max} = 2.8$ N/mm²

$\tau_c < \tau_v < \tau_{c,max}$, so shear reinforcement is needed providing 2-legged 8mm ϕ vertical stirrups,

$A_{sv} = 2 * \frac{\pi}{4} * 8^2 = 100.53$ mm²

Assuming 2-legged 8mm ϕ vertical stirrups,
$A_{sv} = 2 * \frac{\pi}{4} * 8^2 = 100.53$ mm²

assuming 25mm clear cover on all side.



$b_1 = 500 - 25 - 25 - 8 - 8 - \frac{16}{2} - \frac{16}{2} = 418$ mm

$d_1 = 660 - 25 - 8 - 12 = 621$ mm

$x_1 = 500 - 25 - 25 - \frac{8}{2} - \frac{8}{2} = 442$ mm

$y_1 = 700 - 25 - 25 - \frac{8}{2} - \frac{8}{2} = 642$ mm

$S_v = \frac{0.87 f_y A_{sv} d_1}{\frac{T_u}{b_1} + \frac{V_u}{2.5}} = \frac{0.87 * 415 * 100.53 * 621}{\frac{15 * 10^6}{418} + \frac{150 * 10^3}{2.5}} = 235.073$
≈ 230 mm

$S_v = 230mm < x_1 = 1142mm$

$< \left(\frac{x_1+y_1}{4}\right) = \left(\frac{442+642}{4}\right) = 271\ mm$

$< 300mm$ ok

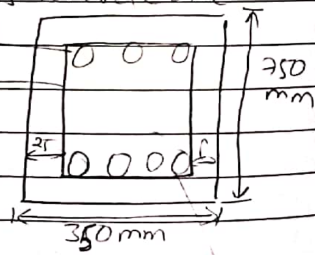
minimum area of shear reinforcement

$A_v = \frac{(τ_v - τ_c) b s_v}{0.87 f_y} = \frac{(0.6-0.385)*500*230}{0.87*415}$

$= 68.48\ mm^2 < 100.53 mm^2$ ok

so, provide 8mm ϕ 2-legged closed vertical stirrups @ 230mm c/c.

15) Design a torsional reinforcement in a rectangular beam section 350mm wide and 750mm deep subjected to ultimate twisting moment 140 KNm combined with bending moment 200 KNm and ultimate shear force 110 KN. Use M25 concrete and Fe415 steel.



Soln

$M_u = 200 KNm, V_u = 110 KN$

$T_u = 140 KNm$

assuming 40mm effective cover,

effective depth (d) = 750 - 40 = 710mm

Design for bending moment.

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

$= \frac{140\left(1+\frac{750}{350}\right)}{1.7}$

$= 258.823\ KNm$

$M_u = 200 KNm, V_u = 110 KN, T_u = 140 KNm$

Since $M_t > M_u$, flexural longitudinal reinforcement at compression zone should be provided

$M_{e1} = M_u + M_t = 200 + 258.823 = 458.823\ KNm$ (at bottom)

$M_{e2} = M_t - M_u = 258.823 - 200 = 58.823 KNm$. (at top)

$x_{u,max} = 0.48d = 0.48*710 = 340.8\ mm$

MOR for singly reinforced balanced section,

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

$= 0.36*25*350*340.8*(710 - 0.42*340.8) \times 10^{-6}$

$= 608.539 KNm$

$608.539 > 458.823\ kNm$

Since $M_{u,lim} > M_{e1}$, singly reinforced section is designed.

area of steel, A_{st1} for M_{e1}

$M_{e1} = 0.87 f_y A_{st1} \left(d - \frac{f_y A_{st1}}{f_{ck}.b}\right)$

or, $458.823 \times 10^6 = 0.87*415*A_{st1}\left(710 - \frac{415*A_{st1}}{25*350}\right)$

on solving we get,

$A_{st1} = 2078.433\ mm^2$

provide (2-28mm & 2-25mm) ϕ rebars

A_{st} provided $= 2*615 + 2*490 = 2210 mm^2$

$> 2078.433 mm^2$ ok

neutral axis depth, $x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck}.b} = \frac{0.87*415*2210}{0.36*25*350} = 253.308\ mm < x_{u,max} = 340.8mm$ ok

minimum area of steel $(A_{st})_{min} = \frac{0.85 b.d}{f_y} = \frac{0.85*350*710}{415}$

$= 508.975 mm^2 < 2210\ mm^2$ ok

max^m area of steel $(A_{st})_{max} = 4\% b.D = 0.04*350*750$

$= 10500 mm^2 > 2210 mm^2$ ok

Ast_2 for M_{t2}

$M_{t2} = 0.87 f_y Ast \left(d - \frac{f_y Ast_2}{f_{ck} \cdot b}\right)$

or, $58.823 \times 10^6 = 0.87 \times 415 \times Ast_2 \left(710 - \frac{415 \times Ast_2}{25 \times 350}\right)$

On solving, $Ast_2 = 233.097 mm^2$

provide (2-12 + 1-10) mm ϕ rebar, $(Ast)_{provided} = 2 \times 113 + 78$
$= 304 mm^2$
$> 233.097 mm^2$
ok

Design for shear

Equivalent shear

$V_e = V_u + 1.6 \frac{T_u}{b} = 110 + 1.6 \times \frac{140}{0.35} = 750 kN$

ultimate shear stress $(\tau_v) = \frac{V_e}{bd} = \frac{750 \times 10^3}{350 \times 710} = 3.018 N/mm^2$

% of steel $= \frac{Ast_L}{b \cdot d} \times 100\% = \frac{2210}{350 \times 710} \times 100\% = 0.889\%$
↳ see in code pg 40

design shear stress of concrete $(\tau_c) = 0.57 + \left(\frac{0.64 - 0.57}{1 - 0.75}\right) \times (0.889 - 0.75)$

$= 0.608 N/mm^2$

$\tau_{v,max} = 3.1 N/mm^2$ for M_{25} code pg 40 table 20.

$\tau_c < \tau_v < \tau_{v,max}$, shear reinforcement is needed providing 25mm clear cover on all sides and 8mm ϕ 2-legged vertical stirrups

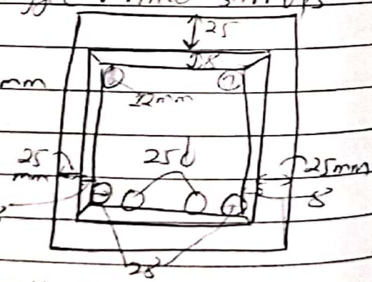
$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.53 mm^2$

$b_1 = 350 - 25 - 25 - 8 - 8 - \frac{28}{2} - \frac{28}{2} = 256 mm$

$d_1 = 710 - 25 - 8 - \frac{12}{2} = 671 mm$

$x_1 = 350 - 25 - 25 - \frac{8}{2} - \frac{8}{2} = 292 mm$

$y_1 = 750 - 25 - 25 - \frac{8}{2} - \frac{8}{2} = 692 mm$



$S_v = \frac{0.87 f_y A_{sv} \cdot d_1}{\frac{T_u}{b_1} + \frac{V_u}{2.5}} = \frac{0.87 \times 415 \times 100.53 \times 671}{\frac{140 \times 10^6}{256} + \frac{110 \times 10^3}{2.5}} = 42.41 \ngtr 100mm$

not ok

providing 12mm ϕ 2-legged vertical stirrups

$A_{sv} = 2 \times \frac{\pi}{4} \times 12^2 = 226.194 mm^2$

$b_1 = 248mm$, $d_1 = 657mm$, $x_1 = 288mm$, $y = 688mm$

$b_1 = 350 - 25 - 25 - 12 - 12 - \frac{28}{2} - \frac{28}{2} = 248mm$

$d_1 = 710 - 25 - 12 - \frac{12}{2} = 667 mm$

$x_1 = 350 - 25 - 25 - \frac{12}{2} - \frac{12}{2} = 288mm$

$y_1 = 750 - 25 - 25 - \frac{12}{2} - \frac{12}{2} = 688mm$

$S_v = \frac{0.87 \times 415 \times 226.194 \times 667}{\frac{140 \times 10^6}{248} + \frac{110 \times 10^3}{2.5}}$

$= 89.516 \approx 88mm$

$S_v = 88mm < x_1 = 288mm$

$< \frac{x_1 + y_1}{4} = \frac{288 + 688}{4} = 244mm$

$< 300mm$

Minimum area of shear reinforcement

$A_{sv} = \frac{(\tau_v - \tau_c) b S_v}{0.87 f_y} = \frac{(3.018 - 0.608) \times 350 \times 88}{0.87 \times 415}$

$= 205.589 mm^2 < 226.194 mm^2$

So provide 12mm ϕ 2-legged closed vertical stirrups @ 88mm c/c

Development length = Bond stress is defined as shear force per unit nominal area of reinforcing bar in the direction parallel to reinforcing bar at interface between reinforcing bar and surrounding concrete.

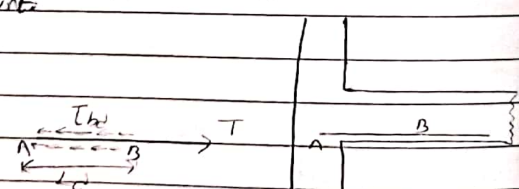
Basic assumption is that steel and concrete act together so that there is no slip of reinforcing bar with respect to surrounding concrete.

Two types of bond stress

1) **Anchorage bond** :- It is a bond which arises over the length of anchorage provided at ends, near supports and cut off points. This bond prevents steel pulling out of concrete if it is in tension or pushing out of concrete if it is in compression.

2) **Flexural bond** :- Flexural bond arises on the account of variation of shear force and bending moment which in turn causes variation in axial tension throughout the length of bar. Flexural bond is critical at a point where shear force $V = \frac{dM}{dx}$ i.e variation in bending moment is significant.

Anchorage bond.



L_d = development length

τ_{bd} = bond stress   pg 72 (cl 26.2.1.1)

For deformed bar, value of τ_{bd} given in table is increased by 60%.

At equilibrium,

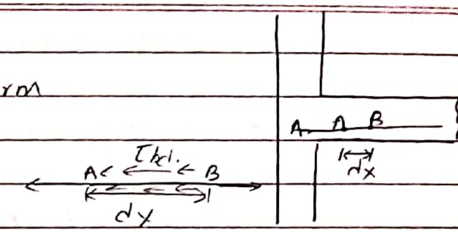
Tension in steel = shear force in surrounding steel.

or, $0.87 f_y A_{st} = \tau_{bd} \cdot \pi \phi L_d$

or, $0.87 f_y \frac{\pi \phi^2}{4} = \tau_{bd} \pi \phi L_d$

or, $L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}}$ (cl 26.2.1 pg 42)

Flexural bond



BM at A, $M_A = T_A \cdot Z$ Z = lever arm

BM at B, $M_B = T_B \cdot Z$

At equilibrium

$T_B - T_A = \tau_{bd} \cdot \pi \phi \, dx$

or, $\frac{M_B - M_A}{Z} = \tau_{bd} \cdot \pi \phi \, dx$

or, $\frac{dM}{dx} = \tau_{bd} \cdot \pi \phi \cdot Z$ where $dM = M_B - M_A$

or, $V = \tau_{bd} \cdot \pi \phi \cdot Z$ where shear force $= V = \frac{dM}{dx}$

again,

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

or, $V = \frac{0.87 f_y \phi}{4 L_d} \cdot \pi \phi Z$

or, $L_d = \frac{0.87 f_y \frac{\pi \phi^2}{4} \cdot Z}{V}$

or, $L_d = \frac{0.87 f_y A_{st} \cdot Z}{V}$ $A_{st} = \frac{\pi \phi^2}{4}$

$L_d = \frac{M}{V}$ where M = MoR in term of steel $= 0.87 f_y A_{st} \cdot Z$

V = factored shear force

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

where L_0 = sum of anchorage length beyond the centre of support

$= 12\phi$ or effective depth 'd' which ever is greater.

Above value of M/V is increased by 30% if reinforcing bar is confined by compressive reaction.

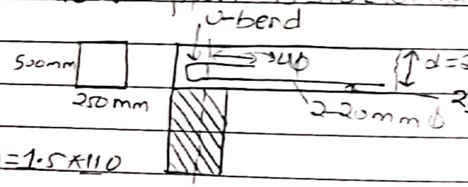
i.e $L_d \le 1.3\frac{M}{V} + L_0$

Bend or mechanical hook		Anchorage length (L_0)
45°	→	4φ
90°	→	8φ
135°	→	12φ
180 or U bend	→	16φ

pg 12 [illegible]

16) Determine the anchorage length for simply supported beam as shown. Shear force at mid of support is 110 kN. Use M20 mix and Fe415 grade steel.

Soln



$d = D - cc - \frac{\phi}{2} = 500 - 25 - \frac{20}{2} = 465$

Shear force = 110 kN

Factored shear force (V) = 1.5×110 = 165 kN

Assuming clear cover = 25mm, effective depth (d) $= 500 - 25 - \frac{20}{2} = 465$ mm

Area of steel $= 2 \times \frac{\pi}{4} \times 20^2 = 628.319\ mm^2$

$X_{u,max} = 0.48d$ for Fe415 $= 0.48 \times 465 = 223.2$ mm

$X_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times 628.319}{0.36 \times 20 \times 250} = 126.03\ mm < 223.2\ mm$

Under reinforced section

MoR (M) $= 0.87 f_y A_{st}(d - 0.42x_u) = 0.87 \times 415 \times 628.319 \times (465 - 0.42 \times 126.03) \times 10^{-6}$

$= 93.479$ kNm

$\tau_{bd} = 1.2$ increased by 60%

i.e $\tau_{bd} = 1.2 \times (1 + 0.6) = 1.2 \times 1.6$

$\tau_{bd} = 1.2$ for M20 mix (code IS 456 table 26.2.1.1)

$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87 \times 415\phi}{4 \times 1.2 \times 1.6} = 47.012\phi \approx 47\phi$

Now,

$L_d \le 1.3\frac{M}{V} + L_0$ (no hook assumed first)

$\le 1.3 \times \frac{93.479}{165} = 0.7365 \times 10^3$

or, $47\phi \le 0.7365 \times 10^3$

$\phi \le 15.67$ mm

Since actual dia used is 20mm, it is not safe in development length.

providing 90 bend

$L_0 = 8\phi$

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

or, $47\phi \le \frac{1.3 \times 93.479 \times 10^3}{165} + 8\phi$

$\phi \le 18.88$ mm (not ok)

providing U-bend $L_0 = 16\phi$

Now, $L_d \le 1.3\frac{M}{V} + L_0$

or, $47\phi \le \frac{1.3 \times 93.479 \times 10^3}{165} + 16\phi$

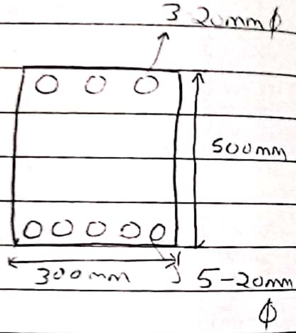
or, $\phi \le \frac{1.3 \times 93.479 \times 10^3}{165 \times (47 - 16)} \le 23.758$ mm

Since actual dia. provided is 20mm it is safe in development length arrangement is shown in figure.

Date ______
Page ______

17) A simply supported beam 30cm*50cm is provided 5-20mmϕ steel bar at bottom of mid span and 3-16mm ϕ steel bar at top. Out of 5-20mmϕ steel bar at bottom, 2-20mm ϕ bar are bent of at distance 1.5m from face of support. The beam is provided 2-legged 8-mmϕ vertical stirrups at the rate 2-20mm c/c at mid span and 1-20mmϕ centre to centre near support. Calculate the shear capacity of beam section at support and moment of resistance of beam. Use M20 mix and Fe415 grade steel.

Soln



assuming 40mm effective cover, effective depth (d) = 500 − 40 = 460mm

$x_{u,max} = 0.48d = 0.48 * 460 = 220.8mm$

assuming $x_u < x_{u,max}$ and $f_{sc} = 0.87 f_y$

$x_u = \frac{0.87 f_y A_{st} - (f_{sc} - 0.446 f_{ck}) A_{sc}}{0.36 f_{ck} b}$

$A_{st} = 5 * \frac{\pi}{4} * 20^2 = 1570.796\ mm^2$

$= \frac{0.87 * 415 * 1570.796 - (0.87*415 - 0.446*20) * 603.185}{0.36 * 20 * 300}$

$A_{sc} = 3 * \frac{\pi}{4} * 16^2 = 603.185\ mm^2$

$= 164.229\ mm < x_{u,max} = 220.8mm$

under reinforced section.

$\frac{d'}{d} = \frac{40}{460} = 0.086$

d'/d	f_{sc}
0.05 →	355
0.10 →	353
0.086	y = 353.56

$f_{sc} = 355 + \frac{(353 - 355)}{(0.1 - 0.05)} \times (0.086 - 0.05)$

$= 353.56\ MPa$

Date ______
Page ______

$MOR = 0.36 f_{ck} b x_u (d - 0.42 x_u) + (f_{sc} - 0.446 f_{ck}) A_{sc} (d - d')$

$= [0.36 * 20 * 300 * 164.229 * (460 - 0.42 * 164.229) + (353.56 - 0.446 * 20)\ 603.185 * (460 - 40)] \times 10^{-6}$

$= 226.019\ KNm$

(पो 9n12 को माथिको theory part)

shear capacity of bent up bar $(V_{us}') = 0.87 f_y A_{sv} \sin\alpha$

$= 0.87 * 415 * 628.319 * \sin 45° \times 10^{-3}$

$= 160.410\ KN$

assuming 45° inclination $\alpha = 45°$, $A_{sv} = 2 * \frac{\pi}{4} * 20^2 = 628.319\ mm^2$

(2 वटा bent अर्थात् भएको so, 5−2 = 3 वटा)

% of steel $= \frac{A_{st} * 100}{b \cdot d} = \frac{3 * \frac{\pi}{4} * 20^2 * 100}{460 * 300} = 0.683$

design shear stress of concrete $(\tau_c) = 0.48 + \frac{(0.56 - 0.48)}{(0.75 - 0.5)} * (0.683 - 0.5)$

$= 0.538\ mm^2$

code page 40
for M20

%	
0.5	0.48
0.75	0.56
0.683	y = 0.538

so shear capacity of concrete $(V_c) = \tau_c b \cdot d = 0.538 * 300 * 460 * 10^{-3}$

$= 74.244\ KN$

Shear capacity of 2-legged 8mm ϕ vertical stirrups @120mm c/c,

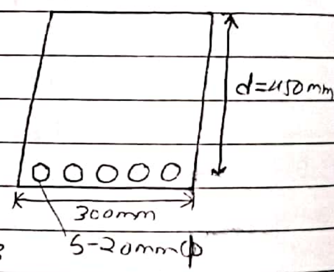
$V_{us} = \frac{0.87 * f_y A_{sv} d}{s_v}$, $A_{sv} = 2 * \frac{\pi}{4} * 8^2 = 100.53\ mm^2$

$= \frac{0.87 * 415 * 100.53 * 460}{120} = 139.136\ KN$

So, shear capacity of beam = $V_{us}' + V_{uc} + V_{us}$
$= 160.41 + 74.224 + 139.136$
$= 373.79\ kN$

18) A R.C.C. beam has an effective depth of 45cm and a breadth of 30cm it contains 5-20mm ϕ TOR steel bar out of which 2-20mm ϕ bar are bent up 30° near support. Calculate shear capacity of bent-up bar. Use M20 concrete & Fe415 steel grade. What additional vertical stirrups are needed if shear force near support is 125KN at service load.

Soln



Area of bent-up bar $(A_{sv}) = 2 \times \frac{\pi}{4} \times 20^2$
$= 628.319\ mm^2$

Shear capacity of bent up bar.
$(V_{us}') = 0.87 f_y A_{sv} \sin\alpha$
$= 0.87 \times 415 \times 628.319 \times \sin 30° \times 10^{-3}$
$= 113.427\ KN$

Factored shear force $(V_u) = 1.5 \times 125.000$
$= 187.5\ KN$

% of steel $= \frac{A_{st}}{bd} \times 100\%$
$= \frac{3 \times \frac{\pi}{4} \times 20^2 \times 100}{450 \times 300}$
$= 0.698\%$

code pg 40 table 19
M_{20}
$0.5 \rightarrow 0.48$
$0.75 \rightarrow 0.56$
$0.698 = 0.543\ N/mm^2$

design shear stress of concrete $(\tau_c) = 0.48 + \frac{(0.56 - 0.48)}{(0.75 - 0.5)}(0.698 - 0.5)$
$= 0.543\ N/mm^2$

Shear force to be resisted by shear reinforcement $(V_{us}) = V_u - V_{uc}$
$= V_u - \tau_c\, b \cdot d$
$= (187.5 - 0.543 \times 450 \times 300 \times 10^{-3})$
$= 114.195\ KN$

50% of total shear force to be resisted by shear reinforcement is resisted by bent-up bar and 50% by vertical stirrups. So, shear force to be resisted by vertical stirrup $(V_{us}'') = 50\%$ of 114.195
$= 57.098\ KN$

Providing 8mm ϕ 2-legged vertical stirrups, $A_{sv} = 2 \times \frac{\pi}{4} \times 8^2$
$= 100.53\ mm^2$

Spacing, $S_v = \frac{0.87 f_y A_{sv} \cdot d}{V_{us}''}$
$= \frac{0.87 \times 415 \times 100.53 \times 450}{57.098 \times 10^3}$
$= 286.058 \approx 285$ (reduce spacing can be done)
$< 300mm$
$< 0.75d = 0.75 \times 450$
$= 337.5$
$\geq 100mm$ OK

Minm area of shear reinforcement $= \frac{0.4 b S_v}{0.87 f_y} = \frac{0.4 \times 300 \times 285}{0.87 \times 415}$
$= 94.723\ mm^2$
$< 100.53\ mm^2$ OK

So provide 8mm ϕ 2-legged vertical stirrups @ 285mm c/c

Slab:- slab forms floor and roof of structural building. Generally slab is assumed to carry uniformly distributed load. In most cases, slab is designed for flexure only. Usually slab is horizontal except for staircase and ramp for storied car park. Beam and wall support the slab.

Types of slab.

1) Simply supported slab spanning in one direction.
2) Simply supported slab spanning in two direction.
3) Continuous slab.
4) Cantilever slab
5) Flat slab :- slab constructed directly over wall.

~~Thicknes~~ Depth of slab :- Depth of slab is determined from deflection criteria rather than flexural consideration. Deflection should be in permissible limit so that neither appearance nor efficiency is affected. Deflection should not exceed $\frac{\text{span mm}}{250}$

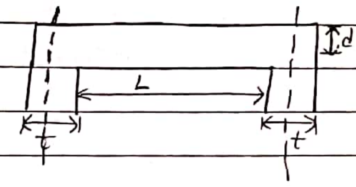
Effective depth of slab (d) = $\frac{\text{effective span length}}{(20-26) \times \text{modification factor}}$

modification factor depends upon % of steel and grade of steel (page 7)

effective span to depth ratio, $\frac{Le}{d}$ ratio (code page 8)

cantilever → 7
simply supported → 20
continuous → 26

Effective span length (Le)



L = clear span length.
t = bearing of support.
d = effective depth of slab.

$Le = L+t$ } take smaller value.
$= L+d$

Classification of slab

Ly - effective length in longer direction.
Lx - effective length in shorter direction.

1) $\frac{Ly}{Lx} > 2$ → one way slab.

2) $\frac{Ly}{Lx} < 2$ → two way slab.

Ast → 1000 mm
1 → $\frac{1000}{Ast}$

8mmϕ = $\frac{\text{one bar area}}{Ast} \times 1000$

* Minimum area of steel reinforcement = 0.12% of bD
* clear cover = 15 mm
* spacing ≤ 300 mm (main bar)
  < 3d
  > 75 mm

spacing = $\frac{\text{area of one bar} \times 1000 \text{ mm}}{\text{total area required}}$

distribution bar.
spacing < 5d
> 450 mm

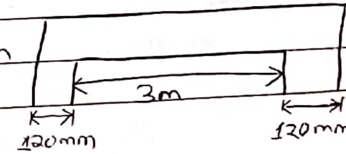
slab is assumed 1m or 1000mm wide beam.

19) Design a simply supported slab supported on a masonry wall to the following requirement, bearing on each support equals 120mm clear span 3m, Live load 4000 N/m² use M20 concrete and fe415 steel.

[मॉ oncrete slab भएनी मात्रै cosrect छ।]

Soln

effective length (le) = 3+0.12
= 3.12 m



0.038 240.7
M.F = 1.5

$0.58 * 415 = 240.7\ N/mm^2$

assuming 0.3% of steel, modification factor = 1.5 (code page 7)

effective depth of slab (d) = effective span length / (20 * modification)

simply supported case $= \frac{3120}{20 * 1.5} = 104$

assuming 8mm φ bar at clear cover 15mm, → D = d + clear cover + φ/2

effective cover = 15 + 8/2 = 19 overall depth = 104 + 15 + 8/2 = 123mm

provide 130mm overall depth, effective depth (d) = 130 − 15 − 8/2 = 111 mm

effective span length (Le) = L + t = 3 + 0.12 = 3.12 m
= L + d = 3 + 0.111 = 3.111 m

take smaller value

Le = 3.111 m

Load calculation.

Live load = 4 × 1 KN/m = 4 KN/m

Self wt = 25 × 1 × 0.13 KN/m = 3.25 KN/m

floor finish (50mm) = $24 × 1 × \frac{50}{1000}$ = 1.2 KN/m

Total load = 8.45 KN/m.

factored load (Wu) = 1.5 × 8.45 = 12.675 KN/m

maxm bending moment (Mu) = $\frac{Wu \cdot le^2}{8} = \frac{12.675 × 3.111^2}{8}$ = 15.334 KN/m

check for depth (Design balanced section) (Singly reinforce मात्र design गर्ने)

Mmax = 0.36 fck b xumax (d − 0.42 xumax)

or, 15.334 × 10^6 = 0.36 × 20 × 1000 × 0.48 d (d − 0.42 × 0.48 d)

on solving we get

d = 74.547 mm < d = 111 mm ok

area of steel in shorter direction.

Mmax = 0.87 fy Ast (d − fy Ast / (fck · b))

or, 15.334 × 10^6 = 0.87 × 415 Ast (111 − 415 Ast / (20 × 1000))

on solving we get

Ast = 414.778 mm²

provide 8mm φ bar at spacing = $\frac{\frac{\pi}{4} * 8^2}{414.778} × 1000$

= 121.186 (always reduce

= 120 mm spacing)

(could be 100)

so provide 8mm φ bar @ 120mm c/c.

Actual Ast, provided

= $\frac{\frac{\pi}{4} * 8^2}{120} × 1000$

= 418.879 mm² > 414.778 mm² ok

area of distribution bar = 0.12% of b.D

= $\frac{0.12 × 1000 × 130}{100}$ = 156 mm²

providing 8mm φ bar at spacing = $\frac{\frac{\pi}{4} × 8^2}{156} × 1000$

= 322.214 ≥ 300mm

bend alternate bar at 1/10 th distance from support.

Check for shear

shear force (Vu) = Wu × clear span / 2 = 12.675 × 3 / 2 = 19.012

ultimate shear stress (τv) = Vu / bd = $\frac{19.012 × 10^3}{1000 × 111}$ = 0.171 N/mm²

% of steel = Ast / (2 × bd) × 100% = $\frac{418.879 × 100}{2 × 1000 × 111}$

= 0.189% code page 40

design shear stress (τc) = 0.36 + (0.36 − 0.28) / (0.25 − 0.15) × (0.189 − 0.15)

= 0.39

$\tau_c' = k \cdot \tau_c$ where $k = 1.3$ for overall depth $< 150mm$

$= 1.3 * 0.39 = 0.51\ N/mm^2 > 0.188\ N/mm^2$ OK

check for development length.

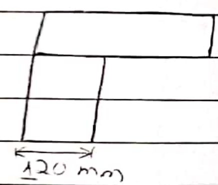
$$MOR\ (M) = 0.87 f_y \frac{A_{st}}{2}\left(d - \frac{f_y \cdot A_{st}}{2 f_{ck} \cdot b}\right)$$

$$= 0.87 * 415 * \frac{418.879}{2}\left(111 - \frac{415 * 418.579}{2*20*1000}\right) * 10^{-6}\ KNm$$

$= 8.064\ KNm$

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87*415\phi}{4*1.2*1.6} = 47.01\phi$$

L_o = anchorage length $= \frac{120}{2} - 15 = 45mm$



Now

$$L_d \le 1.3\frac{M}{V} + L_o$$

or, $47.01\phi \le \frac{1.3 * 8.064 * 10^3}{19.012} + 45$

on solving we get

$\phi \le 12.687\ mm$

Since actual dia of bar used is 8mm, it is safe in development length.

check for deflection

$$\left(\frac{L}{d}\right)_{provided} = \frac{3111}{111} = 28.027.$$

$$\left(\frac{L}{d}\right)_{permissible} = K * \text{basic value} \ (\to 20)$$

K = modification factor.

$$f_s = 0.58 f_y \times \frac{\text{area of steel required}}{\text{area of steel provided}}$$

$$= \frac{0.58 * 415 * 414.778}{418.879}$$

$= 238.343$ Mpa

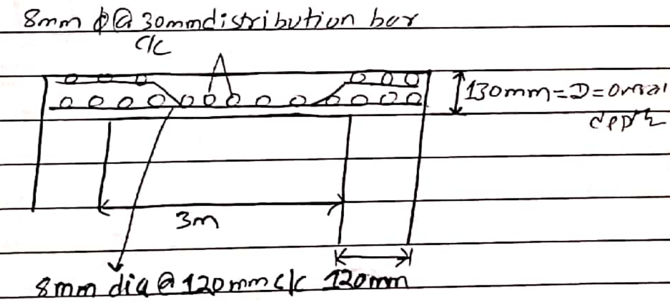
$$\%\ \text{of steel} = \frac{A_{st} * 100\%}{b \cdot d} = \frac{418.879}{1000 * 111} * 100\%$$

$= 0.377\%$

modification factor = 1.4   code page 7 for $f_s = 238.343$ Mpa & % = 0.377

so $\left(\frac{L}{d}\right)_{permissible} = 1.4 * 20$

$= 28mm \simeq 28.027mm$ OK $\left(\frac{L}{d}\right)_{provided}$



Restrained slab i.e when corners of slab are prevented from lifting i.e corners are held down.

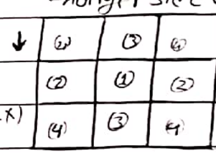
for two way slab, $\frac{L_y}{L_x} < 2$

$M_x = \alpha_x \cdot w_u \cdot L_x^2$ L_y = effective length along longer direction

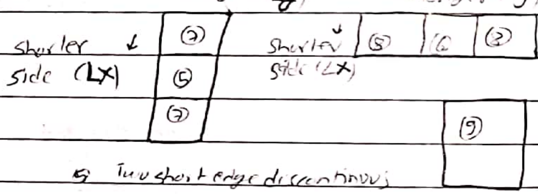
$M_y = \alpha_y \cdot w_u \cdot L_x^2$ L_x = effective length along shorter direction.

M_y = Max^m BM along longer direction M_x = max^m B.M along shorter direction per unit width

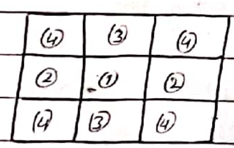
table 27 pg 54 { α_x = BM coeff along shorter direction.
α_y = BM coeff along longer direction. [code table 26 page 54]

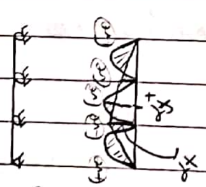


① interior panel (All edge continuous)
② (one short edge discontinuous)
③ (one long edge discontinuous)
④ (two adjacent edge discontinuous)



⑤ Two short edge discontinuous
⑥ Two long edge discontinuous
⑦ Three edge discontinuous (1 edge (long) continuous)
⑧ Three edge discontinuous (1 short edge continuous)
⑨ Four edge discontinuous — code page 54 table 26





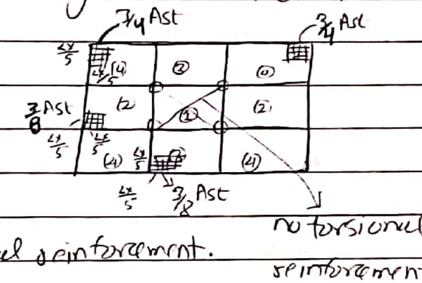
from str

α_y^+ → +ve BM coeff mid span.
α_y^- → -ve BM coeff at continous support.

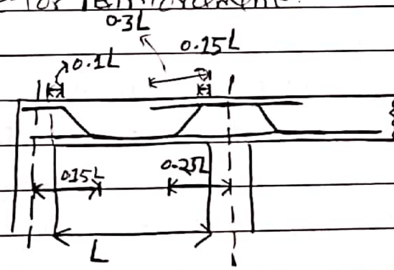
provision for torsion.

Torsional reinforcement is provided in both dir^n at top and bottom layer with area of steel = $\frac{3}{4}$ Ast where Ast = area of steel in mid span for shorter direction.
with area $\frac{Lx}{5} \times \frac{Lx}{5}$ where Lx = effective length in shorter direction.

for two edge discontinuous → $\frac{3}{4}$ Ast
for one edge discontinuous → $\frac{3}{8}$ Ast.
for all edge continous → No torsional reinforcement.

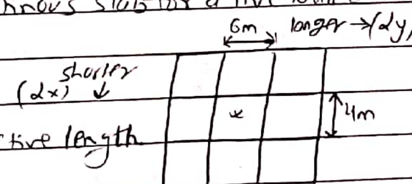


curtailment rule for reinforcement.



(all edge continous)

20) Design a interior panel of 4m×6m continous slab for a live load of 3 kN/m^2. Use M20 mix and fe415 steel.



Sol^n

assuming 230mm wide beam, effective length
= 4+0.23 = 4.23m

effective depth of slab (d) = $\frac{\text{effective length}}{26 \times \text{modification factor}}$
→ 26 for continuous beam
20 for simply supported
- for other do average $\frac{26+20}{2} \approx 23$

Assuming 0.3% steel, modification factor = 1.5

$d = \frac{4230}{26 \times 1.5} = 108.46\,mm$

providing 8mm ϕ bar at clear cover 15mm, overall depth $= 108.46 + 15 + \frac{8}{2} = 127.46\,mm$

So provide 130mm overall depth.

Effective depth $(d_x) = 130 - 15 - \frac{8}{2} = 111\,mm$

effective depth along longer dirⁿ $(d_y) = 111 - 8 = 103\,mm$

effective span length (shorter dirⁿ) $(L_x) = 4 + 0.111 = 4.111\,m$

Longer dirⁿ $L_y = 6 + 0.103 = 6.103\,m$

Load calculation

Live load $= 3 \times 1 = 3\,kN/m$

Self wt $= 25 \times 1 \times 0.13 = 3.25\,kN/m$

Floor finish (50mm) $= 24 \times 1 \times 0.05 = 1.2\,kN/m$

Total load $= 7.45\,kN/m$

factored load $(w_u) = 1.5 \times 7.45 = 11.175\,kN/m$

$\frac{L_y}{L_x} = \frac{6.103}{4.111} = 1.484 < 2$ two way slab

$\alpha_x^+ =$ $\qquad$ $\alpha_y^+ =$

$\alpha_x^- =$ $\qquad$ $\alpha_y^- =$

$\alpha_y^+ = 0.024$

$\alpha_y^- = 0.032$

$\alpha_x^+ = 0.039 + \frac{(0.041 - 0.039)}{(1.5 - 1.4)}(1.484 - 1.4)$

$= 0.0406$

$\alpha_x^+ = 1.4 \rightarrow 0.039$

$1.5 \rightarrow 0.041$

$1.484 ? = 0.0406$

$\alpha_x^- = \begin{vmatrix} 1.4 \rightarrow 0.051 \\ 1.5 \rightarrow 0.053 \end{vmatrix}$

$1.484 ? = 0.0527$

Calculation of bending moment

$M_x^+ = \alpha_x^+ \cdot w_u L_x^2 = 0.0406 \times 11.175 \times 4.111^2 = 7.667\,kNm$

$M_x^- = \alpha_x^- \cdot w_u L_x^2 = 0.0527 \times 11.175 \times 4.111^2 = 9.953\,kNm$

$M_y^+ = \alpha_y^+ w_u . L_x^2 = 0.024 \times 11.175 \times 4.111^2 = 4.533\,kNm$

$M_y^- = \alpha_y^- . w_u . L_x^2 = 0.039 \times 11.175 \times 4.111^2 = 6.043\,kNm$

(Singly reinforced मानेर design गर्ने)

check for depth

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

यो maxᵐ value लिने हो।

or, $9.953 \times 10^6 = 0.36 \times 20 \times 1000 \times 0.48 d (d - 0.42 \times 0.48 . d)$

On solving we get

$d_x = 111$ (सँग compare गर्ने)

$d = 60.06\,mm < 111\,mm$ OK

Calculation of area of steel.

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

For $M_x^+ = 7.667$ — $A_{stx}^+ = 198.688\,mm^2$

for $M_x^- = 9.953$ — $A_{stx}^- = 261.093\,mm^2$

for $M_y^+ = 4.533$ — $A_{sty}^+ = 125.044\,mm^2$

for $M_y^- = 6.043$ — $A_{sty}^- = 168.197\,mm^2$

e.g.

$7.667 \times 10^6 = 0.87 \times 415 \times A_{st} \left(111 - \frac{415 \times A_{st}}{20 \times 1000}\right)$

(111 → effective depth in x-dirⁿ)

On solving

$A_{stx}^+ = 198.688\,mm^2$

$4.533 \times 10^6 = 0.87 \times 415 \times A_{st} \times \left(103 - \frac{415 \times A_{st}}{20 \times 1000}\right)$

(103 → effective depth in y-dirⁿ)

On solving

$A_{sty}^+ = 125.044$

$A_{stx}^- \Rightarrow$

$9.953 \times 10^6 = 0.87 \times 415 \times A_{st} \left(111 - \frac{415 \times A_{st}}{20 \times 1000}\right)$

$A_{stx}^- = 261.093\,mm^2$

Min^m area of steel = 0.12% of b.D

$= \frac{0.12}{100} \times 1000 \times 130$

$= 156\ mm^2$

provide 8mm φ bar at spacing $= \frac{\frac{\pi}{4} \times 8^2}{Ast_x^+} \times 1000$, $Ast_x^+ = 198.688$

$= 252.987\ mm \approx 240\ mm$ (reduce spacing)

provide 8mm φ bar at spacing $= \frac{\frac{\pi}{4} \times 8^2}{Ast_x^-} \times 1000$, $Ast_x^- = 261.093$

$= 192.59\ mm \approx 190\ mm$

provide 8mm φ bar at spacing $= \frac{\frac{\pi}{4} \times 8^2}{Ast_y^+} \times 1000 = 322.215\ mm \approx 300\ mm$, $Ast_y^+ = 156$

must not exceed min^m value of $156\ mm^2$

here $125.044 < 156.50$, $156\ mm^2$ is used

provide 8mm φ bar at spacing $= \frac{\frac{\pi}{4} \times 8^2}{Ast_y^-} \times 1000 = 298.849\ mm \approx 290\ mm$, $Ast_y^- = 168.197$

Actual Ast provided $= \frac{\frac{\pi}{4} \times 8^2}{240} \times 1000\ mm^2$ [Always use Ast_x^+ while checking (shorter side को लागि check गर्ने)]

$= 209.439\ mm^2 > 198.688\ mm^2 = Ast_x^+$ OK [shorter को mid span को लागि check गर्ने]

[shorter side नै critical हुन्छ अब 240 लिएको हो] (shorter span → critical)

Check for shear

shear force $(V_u) = \frac{w_u \times clear\ span}{2} = \frac{11.175 \times 4}{2} = 22.35\ kN$

Ultimate shear stress $(\tau_u) = \frac{V_u}{bd} = \frac{23.35 \times 10^3}{1000 \times 111} = 0.201\ N/mm^2$

% of steel $= \frac{Ast_x^+}{bd} \times 100\%$ [धेरै spacing भएको bend गरिएन so divided by 2 [illegible]]

$= \frac{209.439 \times 100}{1000 \times 111} = 0.188\%$

Design shear stress $(\tau_c) = 0.28 + \frac{0.36 - 0.28}{(0.25 - 0.15)} \times (0.188 - 0.15)$

$= 0.31$ code pg 40 table 19 for

$\tau_c' = K.\tau_c$ where $K = 1.3$ for $D = 150mm$ M20 & % of steel = 0.188%

$= 1.3 \times 0.31 = 0.403\ N/mm^2 > 0.201\ N/mm^2$ OK

(safe no need to add stirrups)

Check for development length, $MOR = (M_1) = 0.87 f_y Ast_x^+ \left(d - \frac{f_y Ast_x^+}{f_{ck}\, b}\right)$

$= 0.87 \times 415 \times 209.439 \times \left(111 - \frac{415 \times 209.439}{20 \times 1000}\right) \times 10^{-6}$

$= 8.065\ kNm$

$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \phi}{4 \times 1.2 \times 1.6} = 47.01\phi \approx 47\phi$

τ_{bd} ← add extra 60% code pg 12

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

first check by making $L_0 = 0$ if it is unsafe then use L_0 value.

or, $47\phi \le \frac{1.3 \times 8.064 \times 10^3}{22.35}$

Since actual dia of bar provided is 8mm, it is safe in development length
< 9.97mm
OK

Check for deflection

$\left(\frac{L}{d}\right)_{provided} = \frac{4111}{111} = 37.036$

$\left(\frac{L}{d}\right)_{permissible} = K \times \text{base value}$ (26)

K = modification factor.

$f_s = 0.58 f_y \times \frac{\text{area of steel required}}{\text{area of steel provided}}$

$= 0.58 \times 415 \times \frac{148.688}{209.439}$

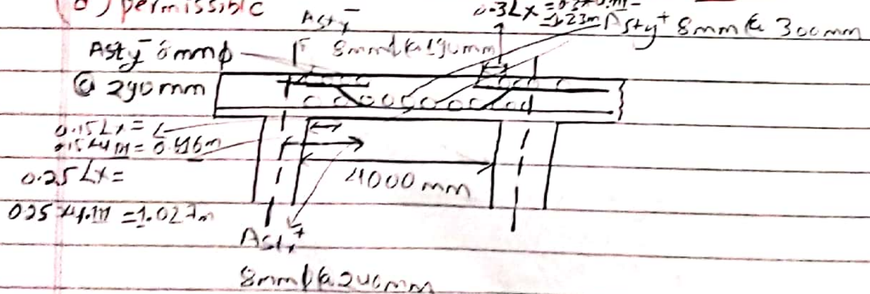
= 228.344 Mpa

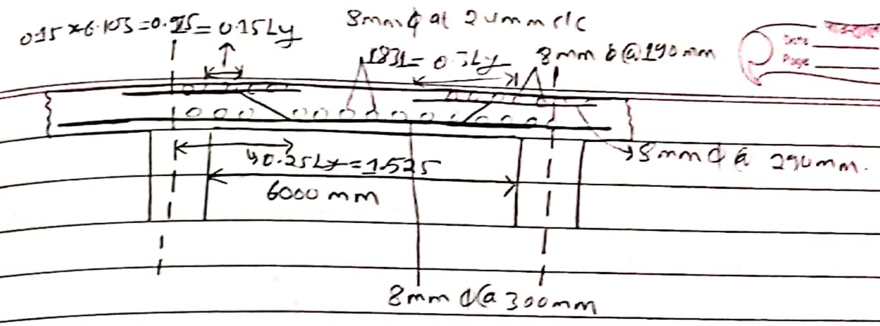
& %age of steel = 0.188%

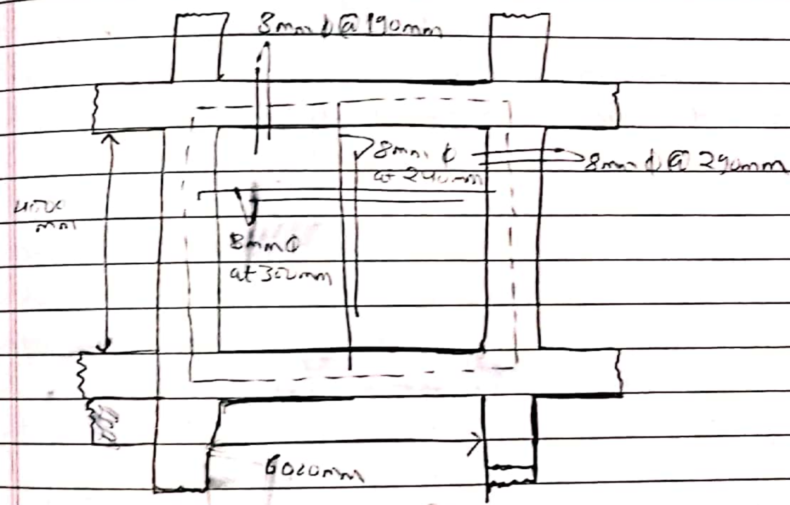
now, from code page 7 chart

K = 1.75

$\left(\frac{L}{d}\right)_{permissible} = 1.75 \times 26 = 45.5 > 37.036$ OK







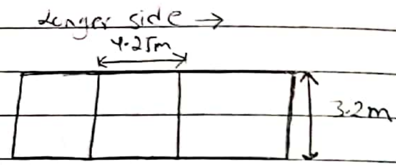
21) Design a two long edge discontinuous slab with clear 30cm side 3.2m*4.25m to support live load of 2kN/m², 40mm thick floor marble finishing and 1kN/m² partition wall load. Use M20 and Fe415 steel.

Soln

Assuming 230mm wide beam.

longer side →

shorter side ↓



effective length = 3.2+0.23 = 3.43m

effective depth of slab (d) = $\frac{\text{effective length}}{23}$ *modification factor.

$= \frac{3430}{23*1.5}$

$\frac{20+26}{2} = 23$

longer side को दुईटै continuous

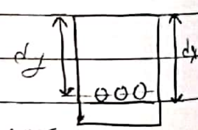
shorter side तिर दुईटै simply supported so average value लिने

∴ d = 99.42mm

providing 8mm Φ dia bar of 15mm clear cover,

overall depth = 99.42+15+$\frac{8}{2}$ = 118.42mm ≈ 121mm

provide 125mm overall depth



effective depth along shorter span (dx) = 125−15−$\frac{8}{2}$ = 106mm

effective depth along longer span (dy) = 106−8 = 98mm

effective length along shorter span (Lx) = 3.2+0.106 = 3.306m

effective length along longer span (Ly) = 4.25+0.098 = 4.348m

load calculation

Live load = 2*1 = 2kN/m

self wt = 25*1*0.125 = 3.125kN/m

floor finishing (40mm) = 24*1*0.04 = 0.96kN/m

partition wall load = 1*1 = 1kN/m

Total load = 7.085 kN/m

factored load (wu) = 1.5*7.085 = 10.627 kN/m

$\frac{Ly}{Lx} = \frac{4.348}{3.306} = 1.315 < 2$, two way slab

$\alpha_x^+ = 0.0579 = 0.058$ $\alpha_y^+ = 0.035$

1.3 → 0.057 $\alpha_y^- = 0.045$

1.4 → 0.063

1.315 → 0.0579 code pg 54 table 26

Calculation of bending moment.

$M_x^+ = \alpha_x^+ w_u L_x^2 = 0.058*10.627*3.306^2 = 6.736$ kNm

$M_y^+ = \alpha_y^+ w_u L_x^2 = 0.035*10.627*3.306^2 = 4.065$ kNm

$M_y^- = \alpha_y^- w_u L_x^2 = 0.045*10.627*3.306^2 = 5.226$ kNm

Check for depth

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

or, $6.736 \times 10^6 = 0.36*20*1000*0.48d(d-0.42*0.48d)$

On solving we get ठूलो compare गर्ने

d = 49.1108mm < dx = 106mm OK

Calculation of area of reinforcement.

minimum Ast, to be provided Ast,min = 0.12% of b.D

$= \frac{0.12*1000*125}{100} = 150$ mm²

Calculation of area of steel.

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

d = 106 mm	d = 98 mm	d = 98 mm
$M_x^+ = 6.736$ kNm	$M_y^+ = 4.065$ kNm	$M_y^- = 5.226$ kNm
$A_{stx}^+ = 182.528$ mm²	$A_{sty}^+ = 117.825$ mm² < 150 mm² → minimum लिनुपर्छ	$A_{sty}^- = 152.631$ mm²
spacing $= \frac{\frac{\pi}{4} \times 8^2 \times 1000}{182.528}$	providing 8mm φ bar $= \frac{\frac{\pi}{4} \times 8^2 \times 1000}{150}$	spacing $= \frac{\frac{\pi}{4} \times 8^2 \times 1000}{152.631}$
$= 275.385$ mm ≃ 175 mm	$= 335.103$ mm ≃ 250 mm	$= 329.326$ mm ≃ 230 mm

along shorter dirⁿ at mid span provide 8mm φ bar at 175 mm c/c
along longer dirⁿ at mid span provide 8mm φ bar at 250 mm c/c
along longer dirⁿ at continuous support provide 8mm φ bar at 230 mm c/c

actual Ast provided $= \frac{\frac{\pi}{4} \times 8^2 \times 1000}{175}$ (A_{stx}^+ को spacing 175 use गर्ने)

$= 287.231$ mm² > 182.528 mm² OK

extend 50% of bar provided at mid span (A_{stx}^+) to the support.

check for shear.

shear force $(V_u) = \frac{\text{clear span} \times w_u}{2} = \frac{10.627 \times 3.2}{2}$

$= 17.003$ kN

ultimate shear stress $(\tau_u) = \frac{V_u}{bd} = \frac{17.003 \times 10^3}{1000 \times 106} = 0.16$ N/mm²

% of steel $= \frac{A_{stx}^+}{2b \cdot d} \times 100\% = \frac{287.231 \times 100\%}{2 \times 1000 \times 106}$ (हाम्रो spacing भएर divided by 2 गरेको हो)

$= 0.135\%$

$\tau_c = 0.28$ N/mm²

for solid slab $\tau_c' = k\tau_c$ where k = 1.3 for D < 150 mm

$= 1.3 \times 0.28$

$= 0.364$ N/mm² > 0.16 N/mm² OK (τ_u)

check for development length.

$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \phi}{4 \times 1.2 \times 1.6} = 47\phi$ (5/0 do pg 12)

$MOR(M) = 0.87 f_y \frac{A_{stx}^+}{2} \left(d - \frac{f_y A_{stx}^+}{2 f_{ck} \cdot b}\right)$

or, $M = 0.87 \times 415 \times \frac{287.231}{2} \times \left(106 - \frac{415 \times 287.231}{2 \times 20 \times 1000}\right) \times 10^{-6}$ kNm

$= 5.341$ kNm

Now

$L_d \le \frac{1.3M}{V} + L_0$ ($L_0 \to 0$)

or, $47\phi < \frac{1.3 \times 5.341 \times 10^3}{17.003}$

or, $\phi < 8.688$ mm

Since actual dia of bar provided is 8mm < 8.688 mm, it is safe in development length.

check for deflection.

$\left(\frac{L}{d}\right)_{provided} = \frac{3306}{106} = 31.188$

$\left(\frac{L}{d}\right)_{permissible} = k \times \text{base value}$ (→ 23)

k = modification factor.

% of steel $= \frac{A_{stx}^+}{bd} \times 100 = 0.27\%$

$f_s = 0.58 f_y \times \frac{\text{area of steel required}}{\text{area of steel provided}}$

$= 0.58 \times 415 \times \frac{182.528}{287.231} = 152.958$ Mpa

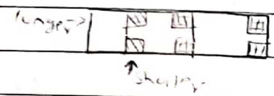
$K = 2$ from code page 7 % of steel = 0.27 %

$f_s = 152.958$

$(\frac{l}{d})$ provided

$(\frac{l}{d})_{permissible} = 2 \times 23 = 46 > 34.88$ OK

Area of steel for torsion $= \frac{3}{8} A_{st} x^{+}$

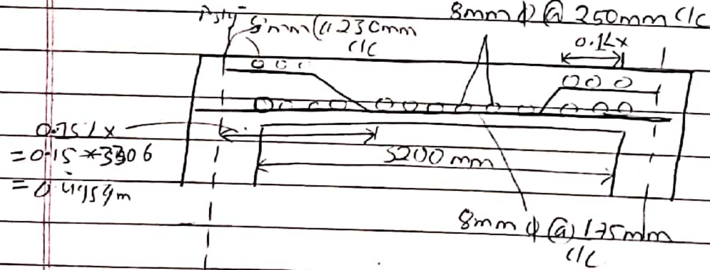


$= \frac{3}{8} \times 287.231 = 107.712$ mm²

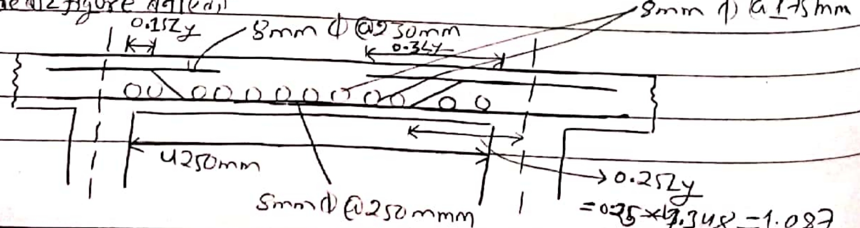
Provide 8mm ϕ bar at spacing $\frac{\frac{\pi}{4} \times 8^2}{107.712} \times 1000$ mm $= 466.665 \simeq 450$mm

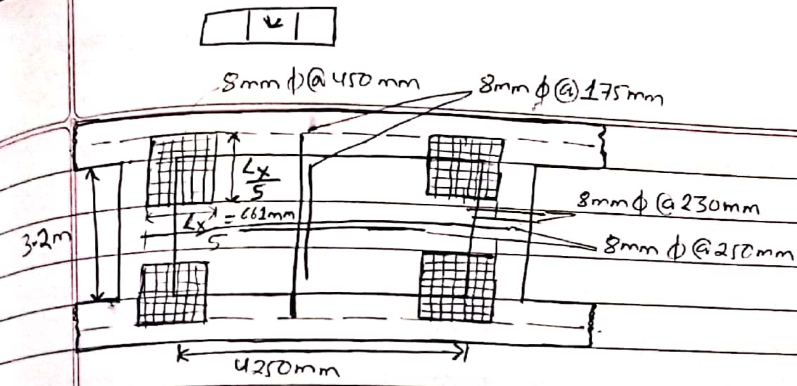
in both dirⁿ at top and bottom.

shorter side का figure बनाउँदा



Longer side का figure बनाउँदा





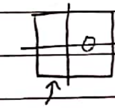
2021 December 22

Chapter:- four. Design of column and footing 30marks

Column:- Column is an important component of structural element. Column supports beam, which in term supports wall and slab. Column is a compression member whose longitudinal dimension exceeds three times lateral dimension. Compression member whose longitudinal dimension does not exceed three times its lateral dimension is known as pedestal.

Classification of column

* Based on types of loading

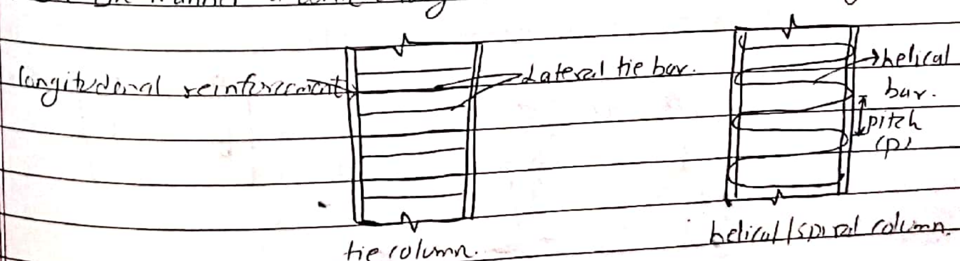


a) axially loaded column.

b) Column subjected to axial load and uniaxial moment.

c) Column subjected to axial load and biaxial moment.

* Based on manner at which longitudinal reinforcement is laterally supported



*) Based on slenderness ratio

Slenderness ratio $(\lambda) = \frac{l_{ex}}{d}$ or $\frac{l_{ey}}{b}$

l_{ex} = effective length along major axis

l_{ey} = effective length along minor axis.

d = lateral dimension along major axis (greater lateral dimension)

b = lateral dimension along minor axis (smaller lateral dimension)

i) $\lambda \le 12 \Rightarrow$ short column.

ii) $\lambda > 12 \rightarrow$ long column

effective length of column (table 28, page 55)

Minimum eccentricity e_{min}

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

$= \frac{L}{500} + \frac{d}{30}$

$= 20mm$

} greater value should be adopted (all dimension in mm)

L = unsupported length in mm

b = least lateral dimension in mm

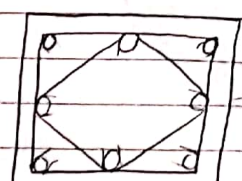
d = greatest lateral dimension in mm

* clear cover 40mm

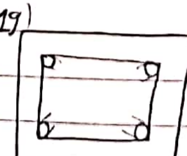
* % of steel (0.8–6%) upto 4% is preffered

% of steel of gross area.

lateral reinforcement arrangement (page 17, 18, 19)



for 8 bar



for 4 bar

diameter of lateral tie bar

$\phi > 6mm$

$> \frac{(\phi_L)_{max}}{4}$

$(\phi_L)_{max}$ – greatest diameter of longitudinal reinforcement.

Spacing of lateral tie bar < 300mm

< d

< $16(\phi_L)_{min}$

< $48\phi_L$

$(\phi_L)_{min}$ = smallest dia. of longitudinal reinforcement.

ϕ_L – dia. of lateral tie bar.

Axially loaded short column.

If minimum eccentricity, $e_{min} < 0.05b$, b = least lateral dimension

$P_u = 0.4 f_{ck} \cdot A_c + 0.67 f_y A_{sc}$

where A_c = area of concrete = $A_g - A_{sc}$

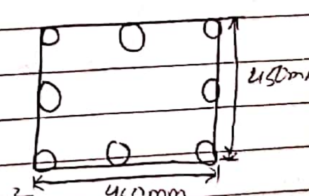
A_g – gross area. – bd

A_{sc} = area of steel.

P_u = factored axial load.

22) A short RCC column 450mm × 450mm is reinforced with 8-20mm bar. The unsupported length of column is 2.75m. find the ultimate load for column. use M20 concrete and Fe250 steel.

Soln



unsupported length (L) = 2.75m

area of steel $(A_{sc}) = 8 \times \frac{\pi}{4} \times 20^2$

$= 2513.274\ mm^2$

area of concrete (Ac) = Ag − Asc = 450*450 − 2513.274
= 199986.726 mm^2

$e_{min} = \frac{L}{500} + \frac{b}{30} = \frac{2750}{500} + \frac{450}{30} = 20.5$ mm } greater value adopted
= 20mm   ∴ e_{min} = 20.5mm

0.05b
= 0.05*450
= 22.5mm > e_{min}.

So, ultimate load, $P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$
= $(0.4*20*199986.726 + 0.67*250*2513.274) * 10^{-3}$ kN
= 2020.867 kN.

23) A R.C.C column 450mm×450mm has to carry factored axial load of 1800kN. The unsupported length of column is 2m. find the area of reinforcement required. Use M20 concrete and Fe250 steel.

Soln

- factored axial load (P_u) = 1800kN

$e_{min} = \frac{L}{500} + \frac{d}{30} = \frac{2000}{500} + \frac{450}{30} = 19 < 20$ → 400 लिदा कम [illegible] हुन्छ।

e_{min} = 20mm
0.05b = 0.05*400 = 20mm ≥ e_{min}
area of concrete (Ac) = Ag − Asc = 450*400 − Asc
= 180000 − Asc

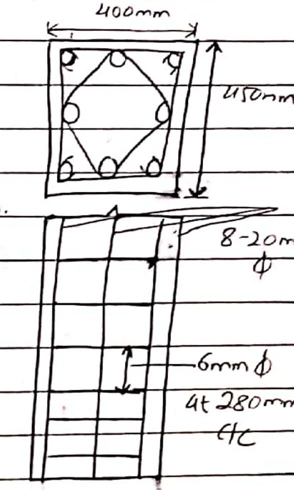
Using relation,
$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$
$1800*10^3 = 0.4*(180000 - A_{sc})*20 + 0.67*250*A_{sc}$
on solving we get,
A_{sc} = 2257.053 mm^2

provide 8-20mm φ longitudinal bar.
area of steel provided = 8*314 = 2512 mm^2 > 2257.053 mm^2
OK

Design of lateral reinforcement
dia. of lateral tie bar
φ ≥ 6mm
≥ $\frac{(\phi_L)_{max}}{4} = \frac{20}{4}$ = 5mm.

provide 6mm φ lateral tie bar.
spacing < 300mm.
< b = 400mm.
< $16(\phi_L)_{max}$ = 16*20 = 320mm.
< $48\phi_L$ = 48*6 = 288mm

So, provide 6mm φ lateral tie bar @ 280mm c/c.



24) A reinforced concrete column of 2.75m length carries an axial load of 1600 kN. Design the column using M20 concrete and Fe415 steel.

Soln

factored axial load (P_u) = (1.5) 1.5 × 1600 = 2400 kN.
assume 2% of steel   (0.8−6%)
area of steel (Asc) = 2% of gross area (Ag) = 0.02Ag
area of concrete (Ac) = Ag − Asc = Ag − 0.02Ag = 0.98Ag
assuming $e_{min} \le 0.05b$.
$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$
$2400*10^3 = 0.4*20*0.98A_g + 0.67*415*0.02A_g$

$\frac{L}{d} < 12$

$d > \frac{L}{12} = \frac{2750}{12} = 229.162mm$ [illegible] and % of steel calculate [illegible]

or, $2400 \times 10^3 = 0.4 \times 20 \times 0.98 A_g + 0.67 \times 415 \times 0.02 A_g$

on solving we get.

$A_g = 179091.1126 \, mm^2$

assuming square column, side $= b = d = \sqrt{A_g} = \sqrt{179091.1126}$

$= 423.192 \, mm$

so, provide 430mm × 430mm column.

area of steel, $A_{sc} = 0.02 A_g = 0.02 \times 430 \times 430$

$= 3698 \, mm^2$

provide 8-25mm φ longitudinal bar, $(A_{sc})_{provided} = 8 \times 490$

$= 3920 \, mm^2$

design of lateral tie bar.

dia(φ) ≥ 6mm

$\geq \frac{(\phi_L)_{max}}{4} = \frac{25}{4} = 6.25mm$

$e_{min} = \frac{L}{500} + \frac{b}{30} = \frac{2750}{500} + \frac{430}{30}$

$= 19.83mm$

$< 20mm$

$e_{min} = 20mm$

$0.05b = 0.05 \times 430$

$= 21.5 \, mm > e_{min}$

OK

provide 8mm φ tie bar.

spacing

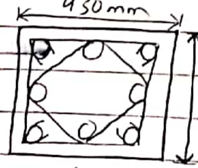
$< 300mm$

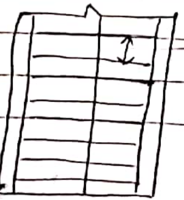
$< b = 430mm$

$< 16(\phi_L)_{min} = 16 \times 25 = 400mm$

$< 48\phi_t = 48 \times 8 = 384 \, mm$

so, provide 8mm φ lateral tie bar @ 300mm c/c





spirally reinforced circular column

$P_u = 1.05 (0.4 f_{ck} A_c + 0.67 f_y A_{sc})$

To satisfy the above relation.

$\frac{\text{volume of spiral reinforcement}}{\text{volume of core per pitch length}} = 0.36 \left(\frac{A_g}{A_c} - 1 \right) \frac{f_{ck}}{f_y}$

A_g - gross Area.

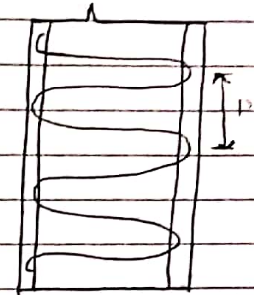
A_c - area of core $= \frac{\pi}{4} d_c^2$

d_c = dia of core

$= D - 2 \times$ clear cover.

volume of spiral reinforcement $= \frac{\pi}{4} \times \phi_{sp}^2 \, \pi \, d_c$

volume of core $= \frac{\pi}{4} d_c^2 \times P$



dia of spiral reinforcement.

$\phi \geq 6mm$

$> \frac{(\phi_L)_{max}}{4}$

pitch (p) $> 25 \, mm$

$< 75mm$ (code pg 47)

$< \frac{d_c}{6}$

$> 3\phi_{sp}$

ϕ_{sp} = dia. of spiral reinforcement

25) Design a circular column to carry axial load of 1500 kN. Using i) lateral tie ii) helical reinforcement. Use M25 and Fe415 steel.

Soln

i) factored axial load $(P_u) = 1.5 \times 1500 = 2250$ kN.

Assuming 0.8% of steel (0.8–6%)

$A_{sc} = 0.8\%$ of $A_g = 0.008 A_g$

$A_c = A_g - A_{sc} = A_g - 0.008 A_g = 0.992 A_g$

now,

$P_u = 0.4 f_{ck} \cdot A_c + 0.67 f_y \cdot A_{sc}$

or, $2250 \times 10^3 = 0.4 \times 25 \times 0.992 A_g + 0.67 \times 415 \times 0.008 A_g$

on solving

$A_g = 185270.577 \text{ mm}^2$

dia. $D = \sqrt{\frac{4A_g}{\pi}} = \sqrt{\frac{4 \times 185270.577}{\pi}} = 485.689$ mm

provide 500 mm φ circular column

$A_{sc} = 0.008 \times \frac{\pi}{4} \times 500^2 = 1570.796 \text{ mm}^2$

provide 8-16 mm φ bar.

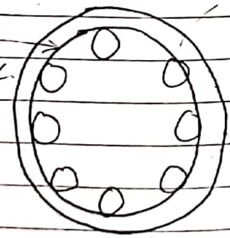
(circular भएकाले min^m 6 वटा bar राख्नुपर्छ।)

A_{sc}, provided $= 8 \times 201 = 1608 \text{ mm}^2 > 1570.769 \text{ mm}^2$

OK

8-16mm φ

6mmφ @ 250mm c/c



Design of lateral tie bar.

dia φ > 6mm

$> \frac{\phi_{max}}{4} = \frac{16}{4} = 4$ mm

provide 6mm φ lateral tie bar.

Spacing < 500 mm

< D = 500 mm

$< 16\phi_{min} = 16 \times 16 = 256$ mm ≈ 250 mm

$< 48\phi_t = 48 \times 6 = 288$ mm

So provide 6mm φ tie bar @ 250 mm c/c

ii) factored axial load $(P_u) = 1.5 \times 1500 = 2250$ kN.

assuming 0.8% of steel

$A_{sc} = 0.8\%$ of $A_g = 0.008 A_g$

$A_c = A_g - A_{sc} = A_g - 0.008 A_g$

$= 0.992 A_g$

now;

$P_u = 1.05 (0.4 f_{ck} \cdot A_c + 0.67 f_y \cdot A_{sc})$

or, $\frac{2250 \times 10^3}{1.05} = 0.4 \times 25 \times 0.992 A_g + 0.67 \times 415 \times 0.008 A_g$

On solving,

$A_g = 176448.168 \text{ mm}^2$

dia (D) $= \sqrt{\frac{4A_g}{\pi}} = \sqrt{\frac{4 \times 176.168}{\pi}} = 473.983$ mm

Provide 480 mm φ circular column

$A_{sc} = 0.008 \times \frac{\pi}{4} \times 480^2$

$= 1447.645 \text{ mm}^2$

Provide 8-16mm φ bar.

A_{sc}, provided $= 8 \times 201 = 1608 \text{ mm}^2 > 1447.645 \text{ mm}^2$

OK

Design of helical bar dia φ ≥ 6 mm

$\geq \frac{\phi_{max}}{4} = \frac{16}{4} = 4$ mm

(Satisfy नभए φ increase गर्ने अथवा core increase गर्ने)

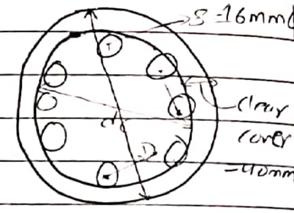
Provide 6mm φ helical bar.

To satisfy the above condition

$\frac{\text{Vol}^m \text{ of spiral reinforcement}}{\text{Vol}^m \text{ of core per pitch length}} = 0.36 \left(\frac{A_g}{A_c} - 1\right) \frac{f_{ck}}{f_y}$

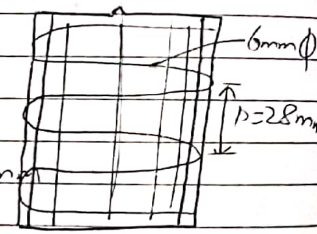
assuming 40mm clear cover
dia of core (dc) = D − 2×clear cover
$= 480 - 2\times40 = 400mm$

$$\frac{\frac{\pi}{4}\phi_{sp}^2 \times \pi d_c}{\frac{\pi}{4} d_c^2 \times p} \geq 0.36\left(\frac{\frac{\pi}{4}D^2}{\frac{\pi}{4}d_c^2} - 1\right)\frac{f_{ck}}{f_y}$$



or, $\frac{6^2 \cdot \pi}{400\times p} \geq 0.36\left(\frac{480^2}{400^2} - 1\right)\frac{25}{415}$

or, $\frac{0.2827}{p} \geq 9.542\times10^{-3}$



$p \leq 29.62mm$

from code, pitch $p \geq 25$
$\leq 75mm$
$< d_c/6 = \frac{400}{6} = 66.667mm$
$\geq 3\phi_{sp} = 3\times6 = 18mm$

So provide 6mm φ helical bar at pitch distance 28mm c/c.

26) Determine the reinforcement in spiral column 400mm dia subjected to factored load 1500KN. The column has unsupported length of 3.4m. Use M25 & Fe415 steel.

Soln

factored load (Pu) = 1500 KN.

$e_{min} = \frac{L}{500} + \frac{D}{30} = \frac{3400}{500} + \frac{400}{30} = 20.133 \approx 20mm$

$e_{min} = 20.13$ (take greater value) $A_c = A_g - A_{sc}$

$0.05D = 0.05\times400 = 20mm \approx e_{min}$

Now,

$P_u = 1.05(0.4 f_{ck} A_c + 0.67 f_y A_{sc})$

or, $1500\times10^3 = 1.05\left(0.4\times25\times\left(\frac{\pi}{4}\times400^2 - A_{sc}\right) + 0.67\times415\times A_{sc}\right)$

on solving

$A_{sc} = 641.426 mm^2$

minm area of steel = 0.8% of $A_g = \frac{0.8}{100}\times\frac{\pi}{4}\times400^2 = 1005.309mm^2$

provide 6-16mm φ bar.

A_{sc} provided $= 6\times201 = 1206mm^2 > 1005.309mm^2$
OK

Design of helical bar

dia(φ) ≥ 6mm
$\geq \frac{\phi_{max}}{4} = \frac{16}{4} = 4mm$

provide 6mm φ helical bar.

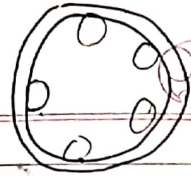
To satisfy above relation

$$\frac{\text{vol}^m \text{ of spiral reinforcement}}{\text{vol}^m \text{ of core per pitch length}} \geq 0.36\left(\frac{A_g}{A_c} - 1\right)\frac{f_{ck}}{f_y}$$

assuming 40mm clear cover.
dia of core (dc) = $400 - 2\times40 = 320mm$

$$\frac{\frac{\pi}{4}\times\phi_{sp}^2\times\pi\times d_c}{\frac{\pi}{4} d_c^2 \times p} \geq 0.36\left(\frac{D^2}{d_c^2} - 1\right)\frac{f_{ck}}{f_y}$$

or, $\frac{6^2\times\pi}{320\times p} \geq 0.36\left(\frac{400^2}{320^2} - 1\right)\times\frac{25}{415}$ (must satisfy this condition)



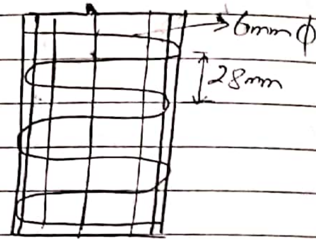
Or, $\frac{0.3534}{p} \geq 0.0121$

$p \leq 28.937mm$

from code, $p \geq 25mm$

$\leq d_c/6 = \frac{320}{6} = 53.33mm$

$> 3\phi_{sp} = 3*6 = 18mm$



So provide 6mm φ helical bar at pitch distance 28mm

27) Determine the safe load for short circular column 425mm dia, reinforcement with 6-22mm φ. It is provided with 8mm φ helical bar with pitch of 40mm. Use M20 concrete Fe250 steel.

Soln

area of steel (Asc) $= 6*\frac{\pi}{4}*22^2 = 2280.796 \, mm^2$

area of concrete (Ac) $= A_g - A_{sc}$

$= \frac{\pi}{4}*425^2 - 2280.796 = 139581.747 \, mm^2$

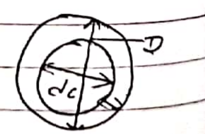
now

$P_u = 1.05(0.4 f_{ck}.A_c + 0.67.f_y.A_{sc})$

$= 1.05*(0.4*20*139581.747+0.67*250*2280.796)*10^{-3}$

$= 1573.621 \, kN$

safe load $= \frac{P_u}{1.5} = \frac{1573.621}{1.5} = 1049.081 \, kN$



To check the validity of formula

$\frac{\text{Vol}^m \text{ of spiral reinforcement}}{\text{Vol}^m \text{ of core per pitch}} \geq 0.36\left(\frac{A_g}{A_c} - 1\right)\frac{f_{ck}}{f_y}$

$d_c = D - 2 \times x$

$= D - 2x$

assuming 40mm clear cover, dia of core (dc)

$= 425 - 2 \times 40$

$= 345 mm$

$\frac{\frac{\pi}{4}\phi_{sp}^2 \times \pi \times d_c}{\frac{\pi}{4} \cdot d_c^2 \times p} \geq 0.36\left(\frac{D^2}{d_c^2} - 1\right) \times \frac{f_{ck}}{f_y}$

Or, $\frac{8^2 \times \pi}{345 \times 40} \geq 0.36\left(\frac{425^2}{345^2} - 1\right) \times \frac{20}{250}$

Or, $0.0145 \geq 0.0149$ OK

Column subjected to axial load and uniaxial moment

Design procedure

1) Determine factored axial load and factored bending moment.
2) Calculate minimum eccentricity along both axes.
3) Min^m eccentricity multiplied by factored load gives moment due to eccentricity.
   Greater of moment due to min^m eccentricity and factored moment is taken as design bending moment.
4) Size of reinforcement, distribution of reinforcement and clear cover is assumed to find $\frac{d'}{D}$, $\frac{P_u}{f_{ck}.b.D}$, $\frac{M_u}{f_{ck}.b.D^2}$
5) from graph find $\frac{p}{f_{ck}}$ where p = % of steel
6) find the area of longitudinal reinforcement.
7) Design of lateral bar.

$\frac{Lex}{d} < 12$ $\quad \frac{2600}{12}$

$d > \frac{Lex}{12}$ $\quad d > 216.66 \rightarrow 250 * 250$ mm

28) Find the reinforcement required for 450mm*450mm R.C.C column subjected for factored load 2500KN and bending moment 180 KNm. Use M25 concrete and fe415 steel.

Sol^n

factored load (Pu) = 250 KN.

factored bending moment (Mu) = 180 KNm

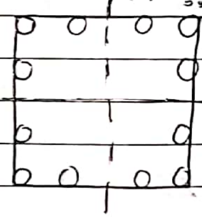
(unsupport length दिएको छैन भने emin check गर्नुपर्दैन)

assuming 20mmɸ rebar at clear 40mm distributed equally on four sides.

$d' = 40 + \frac{20}{2} = 50mm$

$\frac{d'}{d} = \frac{50}{450} = 0.111 \simeq 0.1$ choose nearest value (0.14 ≃ 0.15)

(equal rebar 4 side मा हुन्छ भने)



$\frac{Pu}{fck.b.d} = \frac{2500*10^3}{25*450*450} = 0.494 \simeq 0.493$

$\frac{Mu}{fck.b.d^2} = \frac{180*10^6}{25*450*450^2} = 0.079 \simeq 0.08$

from graph code pg 75

$\frac{P}{fck} = 0.08$

0.49 in vertz. } cross garne $\frac{P}{fck}$
0.08 in horiz

$P = 0.08*25 = 2\%$

so, area of longitudinal bar (Asc) = 2% of Ag

$= \frac{2}{100} * 450 * 450$

$= 4050 mm^2$ (code no. ni use note)

provide (4-25mm + 8-20mm)ɸ bar

Asc provided = 4*490 + 8*314 = 4472 mm² > 4050 mm²

OK

29) Design a column subjected to factored axial force and bending moment of 1000KN and 100 KNm respectively. The unsupported length of column is 4m with supports rigidly fixed and effectively held in position. Use M20 mix and fe500 steel.

Sol^n

factored load (Pu) = 1000KN.

factored bending moment (Mu) = 100KNm

Unsupported length (L) = 4m.

effective length (Le) = 0.65L (code pg 55 table 28)

= 0.65*4 (case i)

= 2.6 m

Asc = 2% of Ag
= 0.02 Ag
Ac = Ag − Asc = 0.98Ag

assuming emin < 0.05d and 2% of steel

$Pu = 0.4fck.Ac + 0.67fy.Asc$

or, $1000*10^3 = 0.4*20*0.98Ag + 0.67*500*0.02Ag$

on solving we get

$Ag = 68775.79 mm^2$

assuming square column $b = d = \sqrt{Ag} = 262.251 mm \simeq 300mm$

provide 300mm*300mm column

$emin = \frac{L}{500} + \frac{b}{30} = \frac{4000}{500} + \frac{300}{30} = 18mm < 20mm$

$0.05d = 0.05*300 = 15mm < emin$

not OK

(यो त्यो short column भए मात्र check गर्ने)

(i.e for $\frac{Lex}{d} < 12$; if $\frac{Lex}{d} > 12$ आएमा buckling हुने हुदा ल्याउने प्रयोग गर्नुपर्छ)

Provide 500mm*500mm column

$emin = \frac{4000}{500} + \frac{500}{30} = 24.66 > 20mm$

$0.05D = 0.05*500 = 25mm > emin$

OK

$\frac{Lex}{d} = \frac{2600}{500} = 5.2 < 12$ short column.

assuming 20mmϕ bar at clear cover 40mm distributed equally on far side, $d' = 40 + \frac{20}{2} = 50mm$

$\frac{d'}{d} = \frac{50}{500} = 0.1$

$\frac{P_u}{f_{ck} \cdot b \cdot d} = \frac{1000 \times 10^3}{20 \times 500 \times 500} = 2 \times 10^{-1} = 0.2$

$\frac{M_u}{f_{ck} \cdot b \cdot d^2} = \frac{100 \times 10^6}{20 \times 500 \times 500^2} = 4 \times 10^{-2} = 0.04$

From graph, code pg 79, for $\frac{d'}{d} = 0.1$, vert = 0.2, hori = 0.04

$\frac{P}{f_{ck}} = 0.01$

$P = 0.01 \times 20 = 0.2\% < 2\%$

so, provided area of steel is sufficient

$A_{sc} = 2\%$ of $A_g = \frac{2}{100} \times 500 \times 500 = 5000 mm^2$

provide 12-25mmϕ bar $490 \times 12 = 5880 mm^2 > 5000 mm^2$ (A_s =) ok

for lateral tie bar.

$\phi \geq 6mm$

$\geq \frac{\phi_{max}}{4} = \frac{25}{4} = 6.25mm$

provide 8mm ϕ bar lateral tie bar.

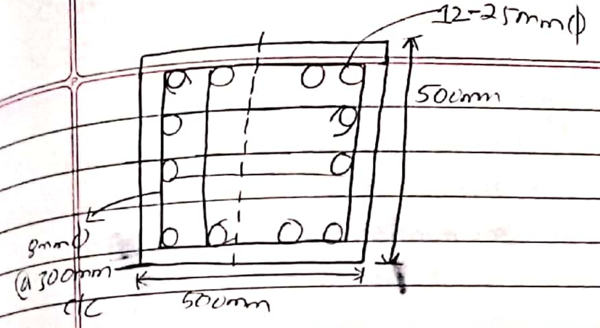
Spacing $\leq 300mm$

$\leq d = 500mm$

$\leq 16\phi_{min} = 16 \times 25 = 400mm$

$\leq 48\phi_t = 48 \times 8 = 384mm$

provide 8mm ϕ tie bar at spacing 300 c/c.



Column subjected to axial load and biaxial moment

Design procedure (% of steel is assumed firstly)

1. Calculate factored axial load and factored moment along both directions
2. Calculate minimum eccentricity along both axes.
   Minimum eccentricity multiplied by factored load gives moment due to minimum eccentricity.
3. Design moment is taken as greater value of imposed moment and moment due to minimum eccentricity.
4. Size of reinforcement, distribution of reinforcement, clear cover and % of steel is assumed to calculate $\frac{d'}{d}$, $\frac{P_u}{f_{ck} \cdot b d}$ and $\frac{P}{f_{ck}}$
5. From graph calculate M_{ux1} and M_{uy1}
6. Calculate the value of α_n from given relation.
   $\alpha_n = 0.667 + 1.667 \frac{P_u}{P_{uz}}$
   where $P_{uz} = 0.45 f_{ck} \cdot A_c + 0.75 f_y A_{sc}$
7. Check the relation
   $\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$
8. If above relation is true, design is safe otherwise increase the %age of steel and repeat the above procedure.
9. Design of lateral reinforcement.

307) Design the column for the following data.
factored axial load = 2000 KN; factored bending moment along major axis = 190 KNm; factored bending moment along minor axis = 95 KNm, size of column 300mm × 500mm, unsupported length = 3m, use M30 concrete and Fe500 steel.

Soln

factored axial load (P_u) = 2000 KN.
factored bending moment along major axis (M_{ux}) = 190 KNm
factored bending moment along minor axis (M_{uy}) = 95 KNm
Unsupported length (L) = 3m
size of column 300mm (b) × 500 mm (d)
([illegible] dimension [illegible] major (x) [illegible])

Minm eccentricity, $e_{x,min} = \frac{L}{500} + \frac{d}{30} = \frac{3000}{500} + \frac{500}{30} = 22.667 > 20mm$
$= 22.667mm$

$e_{y,min} = \frac{L}{500} + \frac{b}{30} = \frac{3000}{500} + \frac{300}{30} = 16mm < 20mm$
$= 20mm$

Moment due to minimum eccentricities $M_{ex} = P_u \cdot e_{x,min}$
$= \frac{2000 \times 22.667}{1000}$

$= 45.334 < 190 KNm$

$M_{ey} = P_u \cdot e_{y,min} = \frac{2000 \times 20}{1000} = 40 KNm < 95 KNm$

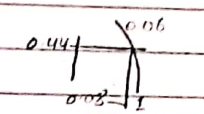
so, design moments are $M_{ux} = 190 KNm$, $M_{uy} = 95 KNm$
assume 20mmφ bar distributed equally on four sides at clear cover 40mm and 2% of steel

$d' = 40 + \frac{20}{2} = 50mm$

Moment capacity along major axis

$d'/d = \frac{50}{500} = 0.1$ $\qquad \frac{P}{f_{ck}} = \frac{2}{30} = 0.067$

$\frac{P_u}{f_{ck} \cdot b d} = \frac{2000 \times 10^3}{30 \times 300 \times 500} = 0.44$



code pg 79

from chart $\frac{M_{ux1}}{f_{ck} \cdot b \cdot d^2} = 0.08$ $\qquad \frac{M_u}{f_{ck} b D^2} = 0.08$

$M_{ux1} = 0.08 \times 30 \times 300 \times 500^2 \times 10^{-6} KNm$
$= 180 KNm < 190 KNm$
Unsafe

So, increase the % of steel to 3.5%.
Moment capacity along Major axis
$d'/d = 0.1$, $\frac{P_u}{f_{ck} \cdot b \cdot d} = 0.44$

$\frac{P}{f_{ck}} = \frac{3.5}{30} = 0.116$

from chart $\frac{M_{ux1}}{f_{ck} \cdot b d^2} = 0.135$ $\qquad$ code pg 79.

$M_{ux1} = 0.135 \times 30 \times 300 \times 500^2 \times 10^{-6} = 303.75 > 190 KNm$
OK

Moment capacity along minor axis.
$d'/b = \frac{50}{300} = 0.166 \sim 0.15$

$\frac{P_u}{f_{ck} \cdot b \cdot d} = 0.44$, $\frac{P}{f_{ck}} = 0.116 = \frac{3.5}{30} = 0.116$

from chart $\frac{M_{uy1}}{f_{ck}\, d \cdot b^2} = 0.13$ $\qquad$ code pg 80

$M_{uy1} = 0.13 \times 30 \times 300^2 \times 500 \times 10^{-6} = 175.5 > 95 KNm$
OK

$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$ (with $A_c = A_g - A_{sc}$)

$= \left[0.45 * 30 * \left(500*300 - \frac{3.5}{100} * 500 * 300\right) + 0.75 * 500 * \frac{3.5}{100} * 500 * 300\right] \times 10^{-3}$

$= 3922.875 \, kN$

$\alpha_n = 0.667 + 1.667 \frac{P_u}{P_{uz}}$

$= 0.667 + 1.667 \times \frac{2000}{3922.875}$

$= 1.513$

check the relation

$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$

or, $\left(\frac{190}{303.75}\right)^{1.513} + \left(\frac{95}{175.5}\right)^{1.513} \leq 1$

or, $0.886 < 1$

So, area of longitudinal reinforcement $A_{sc} = 3.5\%$ of A_g

$= \frac{3.5}{100} * 300 * 500 \, mm^2$

$= 5250 \, mm^2$

provide 12-25mm ϕ bar A_{sc} provided $= 12 * 490 = 5880 \, mm^2 > 5250 \, mm^2$

for lateral tie bar $\phi \geq 6mm$

$\geq \frac{\phi_{max}}{4} = \frac{25}{4} = 6.25mm$

provide 8mm ϕ lateral tie bar.

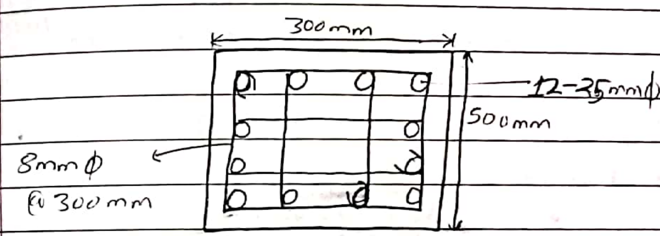
spacing $< 300mm$

$< b = 300mm$

$< 16\phi_{min} = 16 * 25 = 400mm$

$< 48\phi_t = 48 * 8 = 384mm$

So, provide 8mm ϕ lateral tie bar at 300mm c/c



31) A R.C.C carries an axial load of 1170KN accompanied by moment Mx = 120 KNm & My = 30 KNm about major & minor axis respectively. effective length along major axis = 5.25m & along minor axis = 4m, unsupported length along both axis 4.75m. Design the column section using M20 concrete and Fe415 steel.

Soln

$P_u = 1170 * 1.5 = 1755 \, KN$

$M_{ux} = 1.5 * 120 = 180 \, KNm$

$M_{uy} = 1.5 * 30 = 45 \, KNm$

assume the dimension of column so that it will be short column

$\frac{l_{ex}}{d} < 12 \Rightarrow d > \frac{l_{ex}}{12} = \frac{5250}{12} = 437.5mm$

$\frac{l_{ey}}{b} < 12 \Rightarrow b > \frac{l_{ey}}{12} = \frac{4000}{12} = 333.33 \, mm$

assume 350*500mm column (b×d)

$e_{x,min} = \frac{L}{500} + \frac{d}{30} = \frac{4750}{500} + \frac{500}{30} = 26.167\ mm > 20mm$

$e_{y,min} = \frac{L}{500} + \frac{d}{30} = \frac{4750}{500} + \frac{350}{30} = 21.167\ mm > 20mm$

moment due to minimum eccentricity $M_{ex} = P_u . e_{x,min}$

$= \frac{1755 * 26.167}{1000} = 45.92 < 180 kNm$

$M_{ey} = P_u\ e_{y,min} = \frac{1755 * 21.167}{1000}$ (180 kNm)

$= 37.042 kNm < 45 kNm$

so, design moment are $M_{ux} = 180\ kNm$

$M_{uy} = 45\ kNm$

assume 20mm φ bar distributed equally on four sides at clear cover 40mm and 2.5% of steel.

$d' = 40 + \frac{20}{2} = 50mm$

Moment capacity along major axis

$d'/d = \frac{50}{500} = 0.1$

$\frac{P_u}{f_{ck}.b.d} = \frac{1755*10^3}{20*350*500} = 0.501$

$\frac{P}{f_{ck}} = \frac{2.5}{20} = 0.125$

from chart $\frac{M_{ux1}}{f_{ck}.bd^2} = 0.12$ code pg 75

$M_{ux1} = 0.12 * 20 * 350 * 500^2 * 10^{-6} = 210\ kNm > 180 kN$

Moment capacity along minor axis

$d'/b = \frac{50}{350} = 0.143 \approx 0.15$

code pg 76

$\frac{P_u}{f_{ck}.bd} = 0.501$, $\frac{P}{f_{ck}} = 0.125$

from chart $\frac{M_{uy1}}{f_{ck}d.b^2} = 0.115$

or, $M_{uy1} = 0.115 * 20 * 500 * 350^2 * 10^{-6}$

$= 140.875 > 45\ kNm$

($A_c = A_g - A_{sc}$) (A_{sc} = 2.5% of A_g = 0.025.b.d)

$P_{uz} = 0.45 f_{ck}.A_c + 0.75 f_y A_{sc}$

$= (0.45 * 20 * 0.975 * 350 * 500 + 0.75 * 415 * 0.025 * 350 * 500) * 10^{-3}\ kN$

$= 2897.344\ kN.$

$\alpha_n = 0.667 + 1.661 \frac{P_u}{P_{uz}}$

$= 0.667 + 1.661 * \frac{1755}{2897.344} = 1.673$

check for relation

$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \le 1$

or, $\left(\frac{180}{210}\right)^{1.673} + \left(\frac{45}{140.875}\right)^{1.673} \le 1$

or, $0.92 \le 1$ OK

So, area of longitudinal bar $A_{sc} = 2.5\%$ of A_g

$= 0.025 \times 350 \times 500$

$= 4375 \text{ mm}^2$

provide 8-25mm φ bar

For lateral tie bar, $\phi \geq 6$mm

$\geq \frac{\phi_{max}}{4} = \frac{25}{4} = 7$mm

provide 8mm φ lateral tie bar.

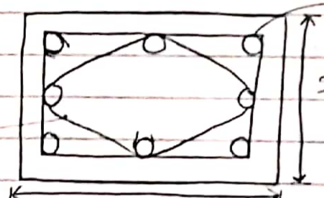
spacing < 300mm

< b = 350mm

< $16\phi_{min} = 25 \times 16 = 448$mm

< $48\phi_t = 48 \times 8 = 384$mm

provide 8mm φ lateral tie bar @ 300mm c/c



Long/slender column

Slenderness ratio $(\lambda) = \frac{L_{ex}}{d}$ or $\frac{L_{ey}}{b}$

$\lambda > 12 \rightarrow$ long/slender column

Additional moment are calculated as

$M_{ax} = P_u . e_{ax}$

$M_{ay} = P_u e_{ay}$

$$e_{ax} = \frac{d}{2000}\left(\frac{L_{ex}}{d}\right)^2$$

$$e_{ay} = \frac{b}{2000}\left(\frac{L_{ey}}{b}\right)^2$$

Additional moment further reduced by factor k.

where $k = \frac{P_{uz} - P_u}{P_{uz} - P_b} \leq 1$

$P_{uz} = 0.45 f_{ck}.A_c + 0.75 f_y.A_{sc}$

$P_b = \left(k_1 + k_2 \frac{p}{f_{ck}}\right) f_{ck}.b.d \Rightarrow k_1 + k_2 \frac{p}{f_{ck}} = \left(\frac{P_b}{f_{ck}.b.D}\right)$

where $p \rightarrow$ % of steel

$k_1, k_2 \rightarrow$ from code 34 (L.39.7)

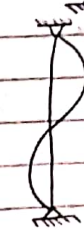
(CL.39.7.1.1 code pg 39)

(for value of P_b table 60 code pg 94)

Braced column: If lateral stability to the column as a whole is provided by buttressing wall supports

column bent in single curvature

Column bent in double curvature



Column bent in single curvature

equivalent moment $M_{ux} = 0.6 M_{ux2} + 0.4 M_{ux1} > 0.4 M_{ux2}$

$M_{uy} = 0.6 M_{uy2} + 0.4 M_{uy1} > 0.4 M_{uy2}$

M_{ux2}, M_{uy2} = greater end moment.

M_{ux1}, M_{uy1} = smaller end moment.

Total moment, $M_{tx} = M_{ux} + M_{ax} \geq M_{ux2}$

$M_{ty} = M_{uy} + M_{ay} \geq M_{uy2}$

Column bent in double curvature

equivalent moment, $M_{ux} = 0.6 M_{ux2} - 0.4 M_{ux1} \geq 0.4 M_{ux2}$

$M_{uy} = 0.6 M_{uy2} - 0.4 M_{uy1} \geq 0.4 M_{uy2}$

Total moment,

$M_{tx} = M_{ux} + M_{ax} > M_{ux2}$ (code pg 39 notes)

$M_{ty} = M_{uy} + M_{ay} \geq M_{uy2}$

32) Design the R.C.C column for the following data:-
Size of column 300mm*400mm
ultimate axial load = 1250 kN, ultimate moment at top about x-axis = 40 kNm, ultimate moment at top about y-axis 15 kNm, ultimate moment at bottom about x & y axis are 25 kNm & 15 kNm respectively unsupported length 6m. effective length along x-axis 4.75m & effective length along y-axis 4.5m. Use M20 concrete & Fe415 steel.
(Assume column bent in single curvature)
(if this condition is not given, assume suitably.)

no need to multiply by 1.5

hints $M_{ux2} = 40$ kNm $M_{uy2} = 15$ kNm
$M_{ux1} = 25$ kNm $M_{uy1} = 15$ kNm

$P_u = 1250$ kN

slenderness ratio, $\lambda = \frac{l_{ex}}{d} = \frac{4.750}{400} = 11.875 < 12$

$= \frac{l_{ey}}{b} = \frac{4.50}{300} = 15 > 12$

so, column is long/slender about minor or y-axis

equivalent moment, $M_{ux} = 0.6 M_{ux2} + 0.4 M_{ux1} \geq 0.4 M_{ux2}$

$= 0.6*40 + 0.4*25 \geq 0.4*40$

$= 34 \geq 16 = 34$ kNm

$M_{uy} = 0.6 M_{uy2} + 0.4 M_{uy1} \geq 0.4 M_{uy2}$

$= 0.6*15 + 0.4*15 \geq 0.4*15$

$= 15 \geq 6 = 15$ kNm

$e_{x,min} = \frac{L}{500} + \frac{d}{30} = \frac{6000}{500} + \frac{400}{30} = 25.33\text{mm} > 20\text{mm}$

$e_{y,min} = \frac{L}{500} + \frac{b}{30} = \frac{6000}{500} + \frac{300}{30} = 22 > 20\text{mm}$

Moment due to minimum eccentricity

$M_{ex} = P_u \cdot e_{xmin} = \frac{1250*25.33}{1000} = 31.662 \text{ kNm} < 34 \text{ kNm}$

$M_{ex} = P_u \cdot e_{y,min} = \frac{1250*22}{1000} = 27.5 \text{ kNm} > 15 \text{ kNm}$

Additional moment $M_{ay} = \frac{K \cdot P_u \cdot b}{2000}\left(\frac{l_{ey}}{b}\right)^2$

assuming 20mmϕ bar distributed equally on four sides of 40mm clear cover and 2% of steel, $d' = 40 + \frac{20}{2} = 50$mm

$P_{uz} = 0.45 f_{ck} \cdot A_c + 0.75 f_y \cdot A_{sc}$

$= (0.45*20*0.98 + 0.75*415*0.02)*300*400*10^{-3}$ kN

$= 1805.4$ kN

$d'/b = \frac{50}{300} = 0.167$

$K_1 = 0.196 + \frac{(0.184-0.196)}{(0.2-0.15)} * (0.167-0.15) = 0.191$

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$K_2 = 0.203 + \frac{(0.028-0.203)}{(0.2-0.15)}(0.167-0.15) = 0.143$

$P_b = \left(K_1 + K_2 \frac{P}{f_{ck}}\right) f_{ck}.b.d = \left(0.191 + \frac{0.143*2}{20}\right) * 20 * 300 * 400 * 10^{-3}$
$= 492.72\ kN.$

$K = \frac{P_{uz} - P_u}{P_{uz} - P_b} = \frac{1805.4 - 1250}{1805.4 - 492.72} = 0.423 < 1$

so, $M_{ay} = \frac{0.423 * 1250 * 0.3}{2000}\left(\frac{4500}{300}\right) = 17.845\ kNm$

final moment

$M_{tx} = M_{ux} + M_{ax} \geq M_{ux_2}$ (∵ $M_{ax} = 0$ cause y-axis मा मात्र Long column हुन्छ भनेर)

$= 34 \geq 40 = 40\ kNm$

$M_{ty} = M_{uy} + M_{ay} \geq M_{uy_2}$

$= 15 + 17.845 \geq 15$

$= 45.345\ kNm$

Moment capacity along major axis

$d'/d = \frac{50}{400} = 0.125 \approx 0.1$

$\frac{P_u}{f_{ck}.b.d} = \frac{1250*10^3}{20*300*400} = 0.521$

$\frac{P}{f_{ck}} = \frac{2}{20} = 0.1$

from chart, $\frac{M_{ux1}}{f_{ck}.b.d^2} = 0.09$

or, $M_{ux1} = 0.09 * 20 * 300 * 400^2 * 10^{-6}$

∴ $M_{ux1} = 86.4\ kNm$

Moment capacity along minor axis

$d'/b = 0.167 \approx 0.15$

$\frac{P_u}{f_{ck}.b.d} = 0.521$

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$\frac{P}{f_{ck}} = 0.1$

from chart $\frac{M_{uy1}}{f_{ck}.d.b^2} = 0.085$

or, $M_{uy1} = 0.085 * 20 * 400 * 300^2 * 10^{-6} = 61.2\ kNm$

$\alpha_n = 0.667 + 1.661 \frac{P_u}{P_{uz}} = 0.667 + 1.661 * \frac{1250}{1805.4} = 1.817$

check the relation

$\left(\frac{M_{tx}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{ty}}{M_{uy1}}\right)^{\alpha_n} \leq 1$

$\left(\frac{40}{86.4}\right)^{1.817} + \left(\frac{45.345}{61.2}\right)^{1.817} \leq 1$

$0.538 < 1$ OK

so, area of longitudinal reinforcement $A_{sc} = 2\%$ of A_g

$= \frac{2 * 300 * 400}{100} =$

$= 2400\ mm^2$

Provide 8-20mm ϕ bar longitudinal bar $= 8 * 314 = 2512\ mm^2$
$> 2400\ mm^2$

for lateral tie bar $\phi \geq 6mm$

$\geq \frac{\phi_{max}}{4} = \frac{20}{4} = 5mm$

Provide 6mm ϕ tie bar

spacing < 300 mm

< b = 300 mm

$< 16\phi_{min} = 16 * 20 = 320mm$

$< 48\phi_t = 48 * 6 = 288mm$

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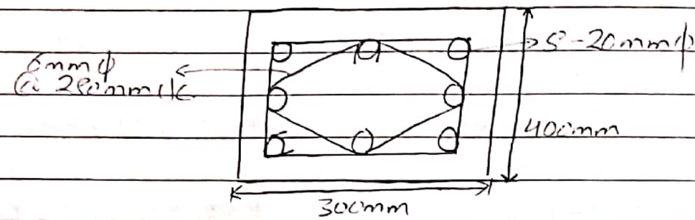
so provide 6mm φ lateral tie bar at 280mm c/c spacing



33) Design unbraced rectangular column for following data: size of column 300mm * 250mm, factored axial load = 750 kN. effective length along x-axis = 3m, effective length along y-axis = 4m, unsupported length = 5m. factored moment in larger dimension = 25 kNm, factored moment in shorter dimension = 15 kNm use M25 concrete & fe 500 steel.

($l_{ex} \rightarrow$ major)

Soln

Size of column 300mm*250mm

Slenderness ratio $\lambda = \frac{l_{ex}}{d} = \frac{4000}{300} = 13.33 > 12$ (long)

$= \frac{l_{ey}}{b} = \frac{3000}{250} = 12$ (short)

So column is long about major axis.

$e_{x,min} = \frac{l}{500} + \frac{d}{30} = \frac{5000}{500} + \frac{300}{30} = 20mm$

$e_{y,min} = \frac{l}{500} + \frac{b}{30} = \frac{5000}{500} + \frac{250}{30} = 18.33mm < 20mm$

$= 20mm$

Moment due to minimum eccentricity

$M_{ex} = P_u.e_{x,min} = \frac{750 * 20}{1000} = 15\ kNm < 25\ kNm$

$M_{ey} = P_u.e_{y,min} = \frac{750 * 20}{1000} = 15\ kNm \geq 15\ kNm$

Additional moment $M_{ax} = \frac{k.P_u.d}{2000}\left(\frac{l_{ex}}{d}\right)^2$

assuming 20mm φ bar distributed equally on four sides at 40mm clear cover and 2% of steel

$d' = 40 + \frac{20}{2} = 50mm$

$d'/d = \frac{50}{300} = 0.167$

$k_1 = 0.196 + \frac{(0.184 - 0.196)}{(0.2 - 0.15)} * (0.167 - 0.15)$

$= 0.191$

$k_2 = 0.253 + \frac{(0.04 - 0.256)}{(0.2 - 0.15)} * (0.167 - 0.15)$

$= 0.183$

$P_b = \left(k_1 + \frac{k_2.p}{f_{ck}}\right) f_{ck}.b.d = \left(0.191 + \frac{0.183 * 2}{25}\right) * 25 * 250 * 300 * 10^{-3}$

$= 385.575\ kN$

$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$

$= (0.45 * 25 * 0.98 + 0.75 * 500 * 0.02) * 300 * 250 * 10^{-3}$

$= 1389.375\ kN$

$k = \frac{P_{uz} - P_u}{P_{uz} - P_b} = \frac{1389.375 - 750}{1389.375 - 385.575} = 0.637$

So,

$M_{ax} = \frac{0.637 * 750 * 0.3 * (13.33)^2}{2000} = 12.738\ kNm$

→ greater of 25 & 15

final moments, $M_{tx} = M_{ux} + M_{ax} = 25 + 12.738 = 37.738\ kNm$

$M_{ty} = M_{uy} + M_{ay}^{0} = 15\ kNm$

Moment capacity along major axis

$d'/d = 0.167 \approx 0.15$

$\frac{Pu}{f_{ck} \cdot b \cdot d} = \frac{750*10^3}{25*250*300} = 0.4$

$\frac{P}{f_{ck}} = 2/25 = 0.08$

from chart, $\frac{Mux_1}{f_{ck} \cdot b d^2} = 0.11$

or, $Mux_1 = 25*250*300^2*0.11*10^{-6}$

$= 61.875 kNm$

Moment capacity along minor axis

$d'/b = \frac{50}{250} = 0.2$

$\frac{Pu}{f_{ck} \cdot b \cdot d} = 0.4$

$\frac{P}{f_{ck}} = 0.08$

from chart

$\frac{Muy_1}{f_{ck} d \cdot b^2} = 0.1$

or, $Muy_1 = 25*300*250^2*0.1*10^{-6}$

$= 46.875 kNm$

$\alpha_n = 0.667 + 1.661 \frac{Pu}{Pu_2}$

$= 0.667 + 1.661 * \frac{750}{1389.375}$

$= 1.564$

check relation

$\left(\frac{Mux}{Mux_1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy_1}\right)^{\alpha_n} \leq 1$

or, $\left(\frac{37.738}{61.875}\right)^{1.564} + \left(\frac{15}{46.875}\right)^{1.564} \leq 1$

or, $0.63 \leq 1$ OK

So, area of longitudinal bar $A_{sc} = 2\%$ of $Ag = \frac{2}{100}*250*300$

$= 1500 mm^2$

Provide 8-16mm ϕ longitudinal bar

for lateral tie bar.

$\phi \geq 6mm$

$> \frac{\phi_{max}}{4} = \frac{16}{4} = 4mm$

So, provide 6mm ϕ lateral tie bar.

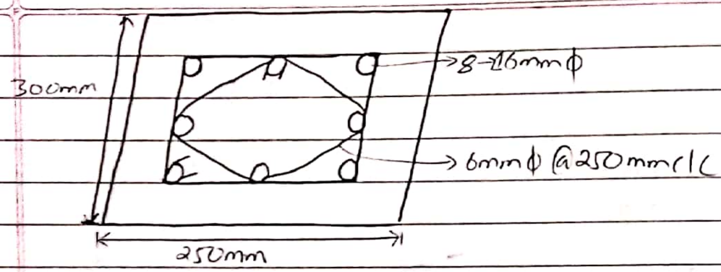
Spacing $< 300mm$

$< b = 250mm$

$\leq 16 \phi_{min} = 16*16 = 256mm$

$\leq 48\phi_t = 48*6 = 288mm$

So, provide 6mm ϕ lateral tie bar at 250mm c/c



34) Design a braced column 300mm*400mm in size subjected to following condition.

$l_{ex} = l_{ey} = 6m$

factored axial load = 1000 KN.

M_{ux} = 40 KNm at top.

M_u = 30 KNm at bottom

M_{uy} = 30 KNm at top

= 25 KNm at bottom

Use M20 mix and Fe415 steel. column bent in double curvature

Sol^n

Size of column = 300mm*400mm.

Slenderness ratio (λ) $= \frac{l_{ex}}{d} = \frac{6000}{400} = 15 > 12$

$\frac{l_{ey}}{b} = \frac{6000}{300} = 20 > 12$

so column is long/slender about both axes.

Initial moments

$M_{ux} = 0.6 M_{ux2} - 0.4 M_{ux1} \geq 0.4 M_{ux2}$

$= 0.6*40 - 0.4*30 \geq 0.4*40$

$= 12 \geq 16 = 16$ KNm

$M_{uy} = 0.6 M_{uy2} - 0.4 M_{uy1} \geq 0.4 M_{uy2}$

$= 0.6*30 - 0.4*25 \geq 0.4*30$

$= 8 \geq 12$

= 12 KNm

Minimum eccentricities

$e_{x,min} = \frac{L}{500} + \frac{d}{30} = \frac{6000}{500} + \frac{400}{30} = 25.333 > 20mm.$

$e_{y,min} = \frac{L}{500} + \frac{b}{30} = \frac{6000}{500} + \frac{300}{30} = 22mm > 20mm$

Moment due to minimum eccentricities

$M_{ex} = P_u . e_{x,min} = \frac{1000*25.333}{1000} = 25.333$ KNm > 16 KNm (M_{ux})

$M_{ey} = P_u . e_{y,min} = \frac{1000*22}{1000} = 22$ KNm > 12 KNm (M_{uy})

assuming 20mm ϕ bar distributed equally on four sides at 40mm clear cover and 3% of steel.

Date ___ Page ___

$d' = \frac{40+20}{2} = 50\,mm$

$d'/d = \frac{50}{400} = 0.125$

$K_1 = \frac{0.207+0.196}{2} = 0.201$

$K_2 = \frac{0.328+0.203}{2} = 0.265$

$P_b = \left(K_1 + K_2 \cdot \frac{P}{f_{ck}}\right) f_{ck} \cdot b \cdot d$

$= \left(0.201 + 0.265 * \frac{3}{20}\right) * 20 * 400 * 300 * 10^{-3}$

$= 577.8\,KN.$

$P_{uz} = 0.45 f_{ck} \cdot A_c + 0.75 f_y A_{sc}$

$= (0.45 * 20 * 0.97 + 0.75 * 415 * 0.03) * 300 * 400 * 10^{-3}$

$= 2168.1\,KN$

reduction factor, $K_x = \frac{P_{uz} - P_u}{P_{uz} - P_b}$

$= \frac{2168.1 - 1000}{2168.1 - 577.8}$

$= 0.735$

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so,

$M_{ax} = K_x \cdot \frac{P_u \cdot d}{2000} \left(\frac{\alpha_{cx}}{d}\right)^2$

$= \frac{0.735 * 1000 * 0.4}{2000} (15)^2$

$= 33.053\,KNm$

$d'/b = \frac{50}{300} = 0.167$

$K_1 = 0.196 + \frac{(0.184 - 0.196)}{(0.2 - 0.15)} (0.167 - 0.15)$

$= 0.192$

$K_2 = 0.203 + \frac{(0.028 - 0.203)}{(0.20 - 0.15)} * (0.167 - 0.15)$

$= 0.144$

$P_b = \left(K_1 + K_2 \frac{P}{f_{ck}}\right) f_{ck} \cdot bd$

$= \left(0.192 + 0.144 * \frac{3}{20}\right) * 20 * 300 * 400 * 10^{-3}$

$= 512.64\,KN$

reduction factor $k_y = \frac{P_{uz} - P_u}{P_{uz} - P_b}$

$= \frac{2168.1 - 1000}{2168.1 - 512.64}$

$\therefore k_y = 0.706$

additional moment $M_{ay} = k_y \cdot \frac{P_u \cdot b}{2000} \left(\frac{l_{ey}}{b}\right)^2$

$= 0.706 * 1000 * \frac{0.3}{2000} (20)^2$

$= 42.336$ kNm

final moments $M_{tx} = M_{ux} + M_{ax} \geq M_{ux2}$

$= M_{ux} + M_{ax} \geq M_{ux2}$

$= 25.333 + 33.053 \geq 40$

$= 58.386$ kNm

$M_{ty} = M_{uy} + M_{ay} = M_{uy2}$

$= 12 + 42.336 \geq 30$

$= 54.336 \geq 30$

$= 54.336$ kNm

Moment capacities along major axis

$\frac{d'}{d} = 0.125 \simeq 0.15$

$\frac{P_u}{f_{ck} b d} = \frac{1000 * 10^3}{20 * 300 * 400} = 0.416$

$\frac{P_u}{f_{ck}} = \frac{3}{20} = 0.15$

from chart

$\frac{M_{ux1}}{f_{ck} \cdot b \cdot d^2} = 0.15$

or $M_{ux1} = 0.15 * 20 * 300 * 400^2 * 10^{-6}$

$= 144$ kNm

Moment capacity along minor axis

$d'/b = 0.167 \simeq 0.15$

$\frac{P_u}{f_{ck} \cdot b \cdot d} = 0.416$

$\frac{P}{f_{ck}} = 0.15$

from chart $\frac{M_{uy1}}{f_{ck} \cdot d \cdot b^2} = 0.15$

or, $Muy_1 = 0.15 * 20 * 400 * 300^2 * 10^{-6}$ kNm

$\therefore Muy_1 = 108$ kNm

$\alpha_n = 0.667 + 1.661 * \frac{Pu}{Pu_2}$

$= 0.667 + 1.661 * \frac{1000}{2168.1}$

$= 1.433$

Check the relation

$$\left(\frac{Mtx}{Mux_1}\right)^{\alpha_n} + \left(\frac{Mty}{Muy_1}\right)^{\alpha_n} \le 1$$

or, $$\left(\frac{58.386}{144}\right)^{1.433} + \left(\frac{64.336}{108}\right)^{1.433} \le 1$$

or, $0.752 \le 1$ ok

So, area of longitudinal bar, $A_{SC} = 3\%$ of $300 * 400$

$= \frac{3}{100} * 300 * 400$

$= 3600 mm^2$

provide 12-20mm ϕ longitudinal bar $= 12 * 314$

$= 3768 mm^2 > 3600 mm^2$

for lateral tie bar

$\phi = 6mm$

$\ge \frac{\phi max}{4} = \frac{20}{4} = 5mm$

provide 6mm ϕ tie bar.

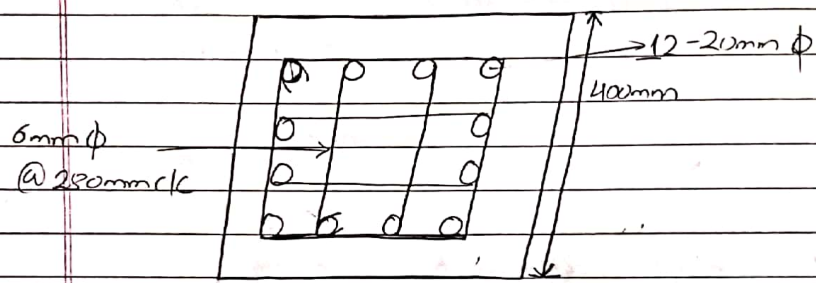
Spacing $< 300mm$

$< b = 300mm$

$< 16\phi min = 16 * 20 = 320 mm$

$< 48 \phi t = 48 * 6 = 288$ mm

So, provide 6mm ϕ lateral tie bar at 280mm c/c spacing


12-20mm ϕ
400mm
6mm ϕ
@ 250mm c/c

Design of footing

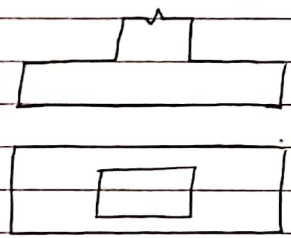
footing:- footing is a structural element that transfers load from building or individual column to soil underneath. If these loads are to be properly transmitted, footing should be designed to prevent excessive settlement, rotation to avoid differential settlement to provide safety against overturning, sliding etc.

Types of footing.

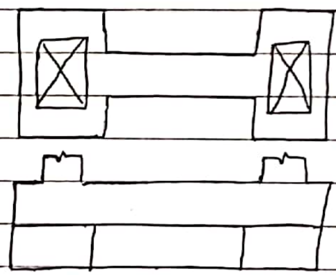
a) shallow footing
If depth of footing is less or equal to width of footing

b) deep footing
If depth of footing is greater than width of footing

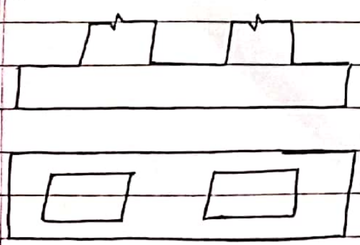
i) Isolated footing



iii) strapped footing.

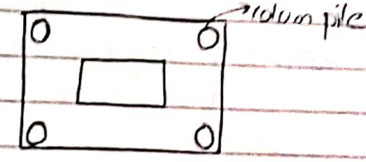


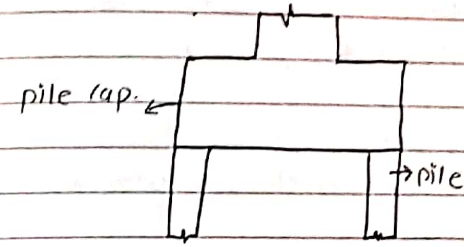
ii) combined footing



iv) Mat/raft footing :- If soil conditions are poor and differential settlement is to be avoided, mat/raft footing are used.

deep footing
1) pier/ pile footing





Design of isolated footing
Design procedure.

1) calculate the area of footing, $A_f = \frac{\text{Total load (P)}}{\text{Safe bearing capacity of soil (SBCS)}}$

Total load (P) = service load (Ps) + self wt. of footing.
If depth of foundation is not given, self wt. of footing is taken as 10% of service load.

If depth of foundation is given, self. wt. of footing is taken tentatively equal to the wt. of back fill soil.

2) calculate soil pressure intensity below footing, $w = \frac{\text{total factored load}}{\text{area of footing}}$

3) calculate max^m bending moment. critical section for bending moment is taken as faces of column.

4) calculate depth of footing $M_{max} = 0.36 f_{ck} \cdot b x_u (d - 0.42 x_u)$
(b → footing breadth.) ↳ Balanced section assumed
above calculated depth is increased by (1.5-3) times for shear consideration.

5) calculate area of reinforcement.

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

6) check for one way shear.

critical section for one way shear is taken at a distance 'd' i.e effective depth from faces of column.

7) check for two way shear.

critical section for two way shear is taken at a distance d/2 from faces of column.

8) check for development length.

35) An isolated reinforced concrete footing has to transfer a service load of 800KN from a square column 300mm*300mm inside safe bearing capacity of soil is 180KN/m². Design isolated footing. use M20 concrete & Fe415 steel.

Soln

service load (Ps) = 800KN.

safe bearing capacity of soil (SBCS) = 180KN/m²

size of column 300mm*300mm

Total load (P) = service load (Ps) + self wt. of footing

$= 1.1 P_s = 1.1 * 800 = 880 KN.$

area of footing (Af) = $\frac{\text{total load}}{SBCS} = \frac{880}{180} = 4.889\ m^2$

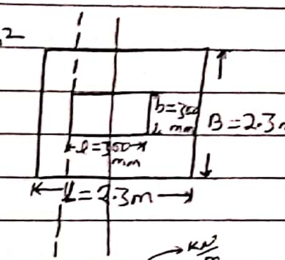
Assuming square footing, side = $\sqrt{A_f} = \sqrt{4.889} = 2.211m$

provide 2.3m*2.3m square footing

net upward soil pressure (w) = $\frac{\text{total factored load}}{\text{area of footing}}$

$= \frac{1.5 * 880}{2.3 * 2.3}$

$= 249.527\ KN/m^2$



calculation of bending moment

$$M_{max} = \frac{wB \left(\frac{L}{2} - \frac{l}{2} \right)^2}{2}$$

$$= \frac{249.527 * 2.3 * \left(\frac{2.3}{2} - \frac{0.3}{2} \right)^2}{2}$$

$= 286.956\ KNm$

$M = \frac{wl^2}{2}$ → KN/m

$M = \frac{w \cdot B \left(\frac{L}{2} - \frac{l}{2} \right)^2}{2}$ → KN/m²

calculate depth of footing → breadth of footing = 2.3m

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

or, $286.956 * 10^6 = 0.36 * 20 * 2300 * 0.48d (d - 0.42 * 0.48d)$

on solving we get,

d = 212.640 mm

(effective depth of footing)

increase the depth of footing by 2.5 times

d = 2.5 * 212.640 = 531.602 ≈ 540 mm

providing 60mm effective cover, overall depth (D) = 540+60

= 600 mm

calculate the area of reinforcement

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

or, $286.956 * 10^6 = 0.87 * 415 * A_{st} \left(540 - \frac{415 * A_{st}}{20 * 2300} \right)$

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On solving we get

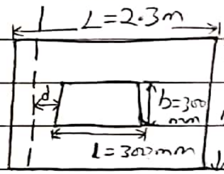
$Ast = 1509.907 mm^2$

provide 12mm ϕ at spacing $= \frac{\frac{\pi}{4} * 12^2}{1509.907} * 2300 = 172.278 \approx 170$ mm

provide 12mm ϕ bar @ 70 mm c/c.

Actual Ast, provided $= \frac{\frac{\pi}{4} * 12^2}{170} * 2300 mm^2 = 1530.140 mm^2$

Check for one way shear.



Shear force $(V_{max}) = wB\left(\frac{L}{2} - \frac{l}{2} - d\right)$

$= 249.527 * 2.3 * \left(\frac{2.3}{2} - \frac{0.3}{2} - 0.54\right)$

$= 263.999 \approx 264$ kN.

Ultimate shear stress $(\tau_u) = \frac{V_{umax}}{B.d} = \frac{264 * 10^3}{2300 * 540} = 0.213 N/mm^2$

% of steel $= \frac{Ast}{B.d} * 100\% = \frac{1530.140}{2300 * 540} * 100\% = 0.123\%$ ↳ < 0.15

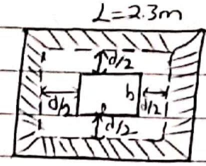
From code pg 40

$\tau_c = 0.28 N/mm^2$

$\tau_c' = k.\tau_c$ where $k = 1$ for $D > 300mm$

$= 1 * 0.28 = 0.28 N/mm^2 > 0.213 N/mm^2$ OK

Check for two way shear.



Shear force (V_{max})

$= w * [L*B - (l+d)(b+d)]$

$= 249.527 * (2.3*2.3 - (0.3+0.54) * (0.3+0.54)) = 1143.932$ kN

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Ultimate shear stress $(\tau_u) = \frac{V_{max}}{2*(l+d+b+d) * d}$ — whole perimeter → length (b_o); d → downside depth

$= \frac{1143.932 * 10^3}{2*(300+540+300+540) * 540}$

$= 0.63 N/mm^2$

now,

$\tau_c' = k_s.\tau_c$

where $k_s = 1 + \beta \leq 1$

β ↳ $\frac{\text{shorter side}}{\text{longer side}} = 1$

$k_s = 1 + 1 \leq 1$

$= 1$

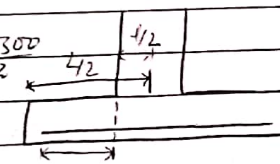
$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.118 N/mm^2 > 0.63 N/mm^2$ OK

Check for development length

$(L_d)_{required} = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87 * 415 \phi}{4 * 1.2 * 1.6} = 47\phi$

for 12mm ϕ bar, $(L_d)_{required} = 47 * 12 = 564$

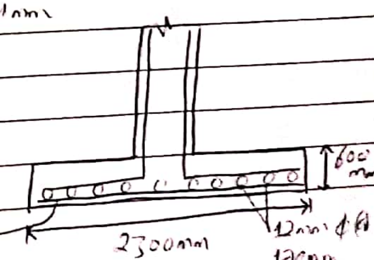
$(L_d)_{available} = \frac{L}{2} - \frac{l}{2} - \text{clear cover} = \frac{2300}{2} - \frac{300}{2} - 50$

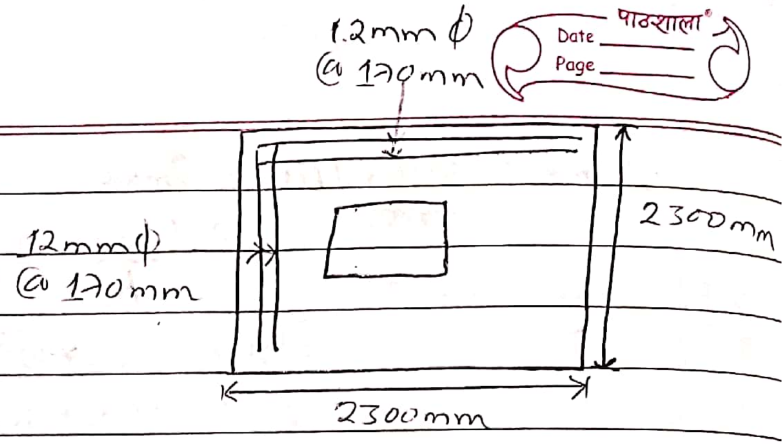


$= 950 mm$

$> 564 mm$

OK





1.2mm ϕ
@ 170mm
12mm ϕ
@ 170mm
2300mm
2300mm

(Unit wt. of soil is 18 kN/m³)

36) Design a rectangular isolated footing for column 400mm * 600mm provided with 8–25mm dia longitudinal bar carries a service load 3500 kN assume safe bearing capacity of 175 kN/m² at a depth of 1.8m below ground level. Use M20 concrete and Fe415 steel.

Soln.

[Depth of footing दिएको बेला self wt पत्ता निकाल्ने बाहेकको बेला 10% जोडेर गर्ने.]

Size of column = 400mm * 600mm

Safe bearing capacity of soil (SBCS) = 175 kN/m²

Service load (P_s) = 3500 kN.

Total load (P) = service load (P_s) + self wt of footing

$$= P_s + \frac{\gamma \times D_f \times P_s}{SBCS}$$

$$= 3500 + \frac{18 \times 1.8 \times 3500}{175}$$

$$= 4148 \text{ kN}$$

$$\text{Area of footing} = \frac{\text{total load (P)}}{\text{safe bearing capacity of soil (SBCS)}}$$

$$= \frac{3500 + 18 \times 1.8 \times 3500/175}{175}$$

$$= \frac{4148}{175} \quad (\because \text{no factoring})$$

$$= 23.703 \text{ m}^2$$

ratio of length and breadth of footing is kept same to that of column

$$\frac{L}{B} = \frac{l}{b} = \frac{600}{400} = 1.5$$

(L, B → footing; l, b → column)

$\therefore L = 1.5B$

now, Area of footing (Af) = L*B = 1.5B*B = 23.703

or, $B = \sqrt{\dfrac{23.703}{1.5}}$

$= 3.975 \simeq 4m$

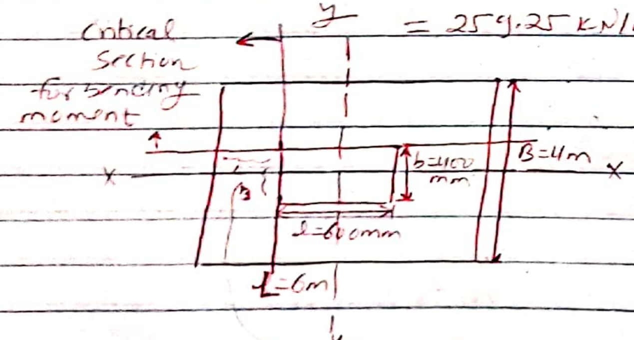
$L = 1.5B = 1.5*4 = 6m$

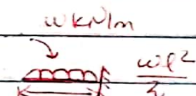
So, provide 4m*6m footing

Net upward soil pressure (w) = $\dfrac{\text{factored total load}}{4*6}$ ← area of footing

$= \dfrac{1.5*4148}{4*6}$

$= 259.25\ kN/m^2$



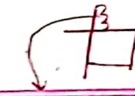


Calculation of Bending Moment.

B.M. along Y-Y axis

$M_{Y-Y} = \dfrac{w.B(L/2 - l/2)^2}{2}$

$= \dfrac{259.25*4*(6/2 - 0.6/2)^2}{2} = 3779.865\ kNm$



Bending Moment along X-X axis

$M_{X-X} = \dfrac{w.L(B/2 - b/2)^2}{2}$

$= \dfrac{259.25*6*(4/2 - 0.4/2)^2}{2}$

$= 2519.91\ kNm$ — M_{Y-Y} is greater than M_{X-X}

Calculation of depth of footing

$M_{Y-Y} = 0.36 f_{ck}.b.x_u(d - 0.42x_u)$ — (b=1 for Mx-x) (Mx-x निकालते समय b को 1 लिया है)

or, $3779.865*10^6 = 0.36*20*4000*0.48d(d - 0.42*0.48d)$

On solving we get,

$d = 585.209mm$

increase the depth by 2 times = 585.209*2 = 1170.418 ≈ 1175mm

providing 60mm effective cover, overall depth (D) = 1175+60

= 1235 mm

Calculation of area of reinforcement

along Y-Y direction

$M_{Y-Y} = 0.87 f_y.A_{sty}(d - \dfrac{f_y.A_{sty}}{f_{ck}.b})$

or, $3779.865*10^6 = 0.87*415*(1175 - \dfrac{415.A_{sty}}{20*4000})*A_{sty}$

On solving we get

$A_{sty} = 9290.966\ mm^2$

Provide 16mm φ bar at spacing $= \dfrac{\frac{\pi}{4}*16^2}{9290.966}*4000 = 86.56\ mm$ ≃ 80mm (reduce spacing)

(X-axis के parallel along y dirn)

provide 16mm ϕ bar @ 80mm c/c

actual (Ast) provided $= \frac{\frac{\pi}{4} \times 16^2}{80} \times 4000 mm^2$

$= 10053.096 mm^2$

reinforcement along x-x dirⁿ.

$M_{x-x} = 0.87 f_y . A_{st x} \left(d - \frac{f_y . A_{st x}}{f_{ck} . L} \right)$

or, $2519.91 \times 10^6 = 0.87 \times 415 \times A_{st x} \left(1175 - \frac{415 . A_{st x}}{20 \times 6000} \right)$

on solving we get

$A_{st x} = 6047.553 mm^2$

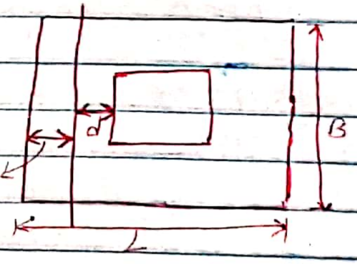
provide 12mm ϕ at spacing $= \frac{\frac{\pi}{4} \times 12^2}{6047.553} \times 6000$

$= 112.208 \simeq 110 mm$

check for one way shear.

$V_{y-y} = w . B \left(\frac{L}{2} - \frac{l}{2} - d \right)$

$= 259.25 \times 4 \left(\frac{6}{2} - \frac{0.6}{2} - 1.175 \right)$



$= 1581.425 KN$.

$V_{x-x} = w . L \left(\frac{B}{2} - \frac{b}{2} - d \right) = 259.25 \times 6 \left(\frac{4}{2} - \frac{0.4}{2} - 1.175 \right)$

$= 972.1875 KN$

$V_{yy} > V_{x-x}$

Ultimate shear stress $(\tau_u) = \frac{V_{yy}}{B . d} = \frac{1581.425}{B . d} = \frac{1581.425 \times 10^3}{4000 \times 1175}$

$= 0.336 N/mm^2$

%age of steel $= \frac{A_{st y}}{B . d} \times 100\%$

$= \frac{10053.096}{4000 \times 1175} \times 100\%$

$= 0.214\%$

Now from code pg 40

M20	
0.15	→ 0.28
0.25	→ 0.36

$\tau_c = 0.2144 = 0.331$

design shear stress $(\tau_c') = K . \tau_c$. where K for D > 300mm

$= 1 \times 0.331 = 0.331 \simeq \tau_u = 0.336$ OK
N/mm^2

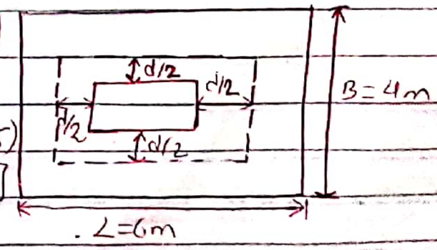
[Value of K code pg 89 . Cl. 40.2.]

If unsafe increased by 3 or 4 times d. i.e d = 3 × 585.209 or do truncate the spacing

Check for two way shear.

shear force $(V) = w \left[L \times B - (l + d)(b + d) \right]$

$= 259.25 \left[6 \times 4 - (0.6 + 1.175) \times (0.4 + 1.175) \right]$



$= 5497.234 KN$.

Ultimate shear stress $(\tau_v) = \frac{V}{b_o d} = \frac{V}{2(l+d+b+d)d}$

$= \frac{257.25 \times 10^3}{2 \times (600+1175+400+1175) \times 1175}$

$= 0.0329\ N/mm^2$

design shear stress $(\tau_c') = K_s \cdot \tau_c$

where $K_s = (0.5+\beta) \leq 1$

$\beta = \frac{\text{shorter side}}{\text{longer side}} = \frac{400}{600} = 0.667$

$K_s = 0.5 + 0.667 < 1$

$1.667 \leq 1 = 1$

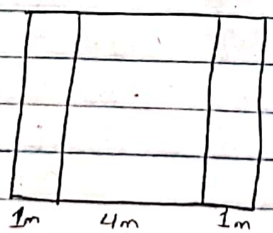
$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.118$

$\tau_c' = 1 \times 1.118 = 1.118\ N/mm^2 > 0.0329\ N/mm^2$ OK

reinforcement in central band $= \frac{2}{\beta+1} \times$ reinforcement in shorter dir^n

$\beta = \frac{\text{longer side (footing)}}{\text{shorter side (footing)}}$

$= \frac{6}{4} = 1.5$



$= \frac{2}{1.5+1} \times 10053.096$

$= 8042.476\ mm^2$

provide 16mm ϕ bar at spacing equals $\frac{\pi \times 16^2}{4 \times 8042.476} \times 4000$ mm

$= 100$ mm

Check for development length, $(l_d)_{required} = \frac{0.87 f_y \phi}{4 \tau_{bd}}$

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

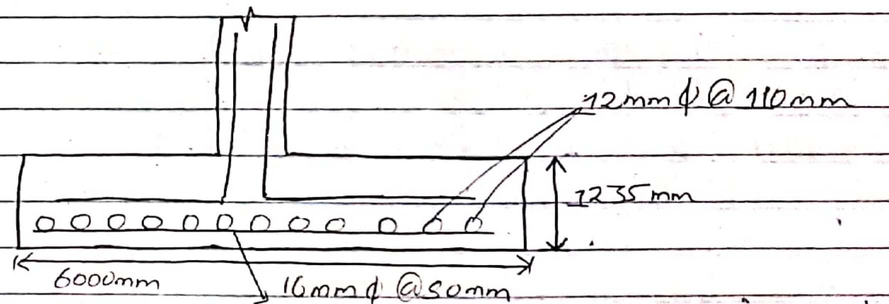
$= 47\phi$

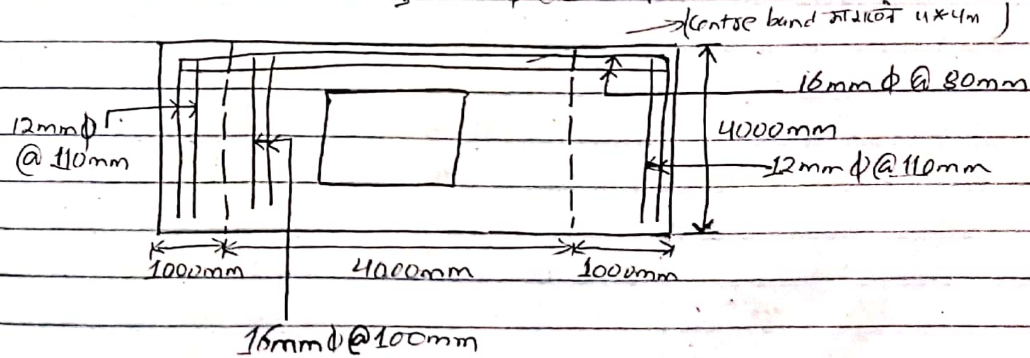
$= 47 \times 16 = 752$

$(l_d)_{available} = \frac{L}{2} - \frac{l}{2} - \text{clear cover}$

$= \frac{6000}{2} - \frac{600}{2} - 50 = 2650\ mm > 752\ mm$

OK





37) Design a footing to support a 300mm×400mm column. The column carries factored axial load of 1400kN and a factored moment of 90kNm. The safe soil pressure is 200kN/m² at a 1.5m depth. Column is reinforced with 6-22mm ϕ bar. Unit wt. of soil is 20 kN/m³. Use M20 concrete and Fe415 steel.

Sol^n

(Moment is given by axis/dir^n is not specified)

Size of column is 300mm×400mm.

Factored axial load (Pu) = 1400kN.

Factored B.M. (Mu) = 90kNm.

Safe bearing capacity of soil (SBCS) = 200kN/m²

Service load $(P_s) = \frac{P_u}{1.5} = \frac{1400}{1.5} = 933.333\,kN$.

total load (P) = service load (Ps) + self wt. of footing

$= 933.333 + \frac{\gamma \cdot D_f \cdot P_s}{SBCS}$

$= 933.333 + \frac{20 \times 1.5 \times 933.333}{200}$

$= 1073.333\,kN$.

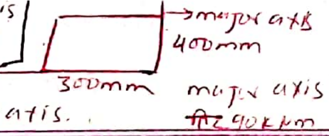
area of footing $(A_f) = \frac{\text{Total load}(P)}{SBCS} = \frac{1073.333}{200} = 5.367\,m^2$

assume square footing, side $= \sqrt{A_f} = \sqrt{5.367} = 2.317$

$\simeq 2.5m$

provide 2.5m×2.5m square footing.

[major फिर moment आएगा minor axis फिर shift जाएगी & minor axis का moment आएगा major axis फिर shift जाएगी



major axis फिर 90kNm moment आएगा X-dirn फिर shift जाएगी

Net upward soil pressure $(w) = \frac{\text{factored total load}}{\text{area of footing}}$

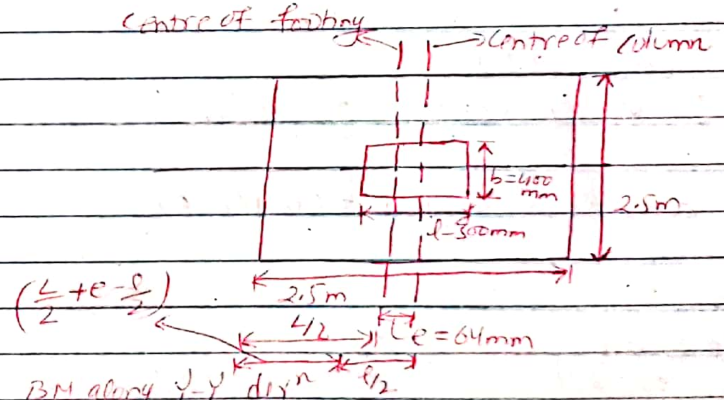
$= \frac{1073.333 \times 1.5}{2.5 \times 2.5}$

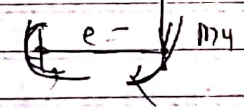
$= 257.599 \simeq 257.6\,kN/m^2$

[both side का moment आएगा both dirn मा shift जाएगी]

To compensate effect of B.M., shift the axis of footing from axis of column by eccentricity $(e) = \frac{M_u}{P_u} = \frac{90 \times 10^3}{1400}\,mm = 64.286\,mm$

$\simeq 64\,mm$





BM along Y-Y dir^n

$M_{Y-Y} = \frac{w \cdot B \left(\frac{L}{2} - \frac{l}{2} + e\right)^2}{2}$

$= \frac{257.6 \times 2.5 \times \left(\frac{2.5}{2} - \frac{0.3}{2} + 0.064\right)^2}{2}$

$= 436.271\,kNm$

$M_{x-x} = \frac{w \cdot L \left(\frac{B}{2} - \frac{b}{2}\right)^2}{2} = \frac{257.6 \times 2.5 \left(\frac{2.5}{2} - \frac{0.4}{2}\right)^2}{2}$

$= 355 \text{ kNm}$

calculation of depth of footing

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

or, $436.227 \times 10^6 = 0.36 \times 20 \times 2500 \times 0.48d (d - 0.42 \times 0.48 d)$

On solving we get

$d = 251.486 \text{ mm}$

Increase the depth of footing atleast 2 times

$d = 2 \times 251.486 = 502.972 \approx 510 \text{mm}$

provide 60mm effective cover overall depth $(D) = 510 + 60 = 570\text{mm}$

calculation of reinforcement

$M_{y-y} = 0.87 f_y A_{st} \left(d - \frac{f_y A_{sty}}{f_{ck} \cdot b}\right)$

or, $436.227 \times 10^6 = 0.87 \times 415 A_{sty} \left(510 - \frac{415 A_{sty}}{20 \times 2500}\right)$

On solving we get

$A_{sty} = 2468.488 \text{mm}^2$

provide 12mm ϕ bar at spacing $= \frac{\frac{\pi}{4} \times 12^2}{2468.488} \times 2500$

$= 114.541 \text{ mm}$

$= 100\text{mm}$

provide 12mm ϕ bar @ 100mm c/c.

Actual Ast provided $= \frac{\frac{\pi}{4} \times 12^2}{100} \times 2500 \text{mm}^2$

$= 2827.433 \text{mm}^2$

$M_{x-x} = 0.87 f_y A_{stx} \left(d - \frac{f_y A_{stx}}{f_{ck} L}\right)$

or, $355 \times 10^6 = 0.87 \times 415 \times A_{stx} \left(510 - \frac{415 \times A_{stx}}{20 \times 2500}\right)$

On solving we get

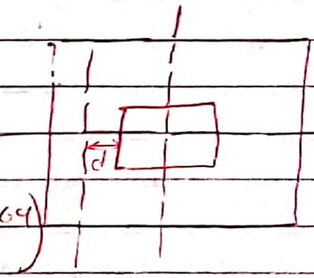
$A_{stx} = 1992.541 \text{mm}^2$

provide 12mm ϕ bar at spacing $= \frac{\pi \times 12^2}{4 \times 1992.541} \times 2500$

$= 141.9$

$= 140\text{mm}$

check for one way shear



$V_{y-y} = w \cdot B \left(\frac{L}{2} - \frac{l}{2} + e - d\right)$

$= 257.6 \times 2.5 \left(\frac{2.5}{2} - \frac{0.3}{2} - 0.510 + 0.064\right)$

$= 421.176 \text{ KN}$

$V_{x-x} = w \cdot L \left(\frac{B}{2} - \frac{b}{2} - d\right)$

$= 257.6 \times 2.5 \left(\frac{2.5}{2} - \frac{0.4}{2} - 0.570\right) = 347.76$

Ultimate shear stress $(\tau_u) = \frac{V_u}{b \cdot d} = \frac{421.176 \times 10^3}{510 \times 2500}$ 0.33 N/mm²

(510 → d, 2500 → b)

% of steel $= \frac{A_{sty}}{b \cdot d} \times 100\%$

$= \frac{2827.433}{2500 \times 510} \times 100$

$= 0.222$

from code pg 40

0.15 → 0.28

0.25 → 0.36

$\tau_u = 0.2229 = 0.3376$ N/mm²

$\tau_c' = k \times \tau_c$ where $k = 1$ for $D > 300$mm code pg 39 40.2.1.1

$= 1 \times 0.337 = 0.337$ N/mm² > 0.337 N/mm²

OK

Check for two way shear.

$V = w(L \times B - (l+d)(b+d))$

$= 257.6(2.5 \times 2.5 - (0.3+0.51)(0.4+0.51))$

$= 1420.123$ KN.

Ultimate shear $(\tau_u) = \frac{V}{b \cdot d} = \frac{V}{2(l+d+b+d) \times d}$

$= \frac{1420.123 \times 10^3}{2 \times (300+510+400+510) \times 510}$

$= 0.8094$ N/mm²

$\tau_c' = k_s \tau_c$

$k_s = 0.5 + \beta \leq 1$

$\beta = \frac{\text{shorter side (column)}}{\text{longer side}} = \frac{300}{400} = 0.75$

$k_s = 0.5 + 0.75 = 1.25 \leq 1 = 1$

$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.118$ N/mm²

$\tau_c' = 1 \times 1.118 = 1.118$ N/mm² > 0.8094 N/mm²

OK

reinforcement in central band $= \frac{2}{\beta + 1} \times$ reinforcement in shorter dirⁿ

$\beta = \frac{\text{longer side (footing)}}{\text{shorter side (footing)}} = \frac{2.5}{2.5} = 1$

$= \frac{2}{1+1} \times A_{sty} = \frac{2 \times 2827.433}{2} = 2827.433$ mm²

So, provide 12mm ϕ bar @ 100 mm

check for development length

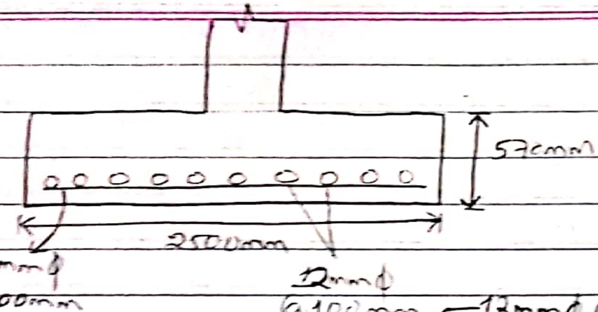
$(L_d)_{required} = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \phi}{4 \times 1.2 \times 1.6} = 47\phi = 47 \times 12$

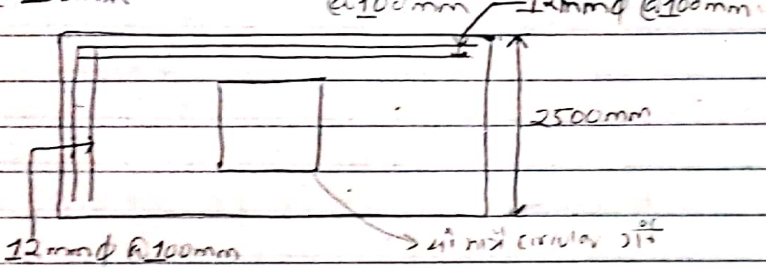
$= 564$ mm

$(L_d)_{available} = \frac{L}{2} - \frac{l}{2} - e - \text{clear cover}$

$= \frac{2500}{2} - \frac{300}{2} - 64 - 50 = 986$ mm > 564 mm

OK





Note :-

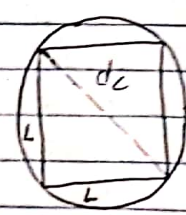
If circular column is given,

circular column is converted to equivalent square column of side $\frac{d_c}{\sqrt{2}}$ where d_c = dia of circular column

$L^2 + L^2 = d_c^2$

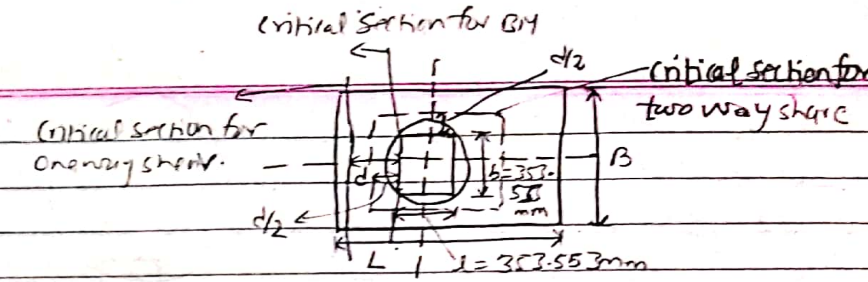
$2L^2 = d_c^2$

$\therefore L = \frac{d_c}{\sqrt{2}}$



If dia. of circular column is 500mm

side of square column $= \frac{500}{\sqrt{2}} = 353.553$ mm



38) Design an isolated footing which has 400mm dia of column with 8-20mm dia longitudinal bar and carrying a load 800KN. assume soil with safe bearing capacity of 80KN/m² at a depth 2m below ground. use M25 grade concrete for column & M20 concrete for footing and Fe415 steel. Unit wt of soil 18KN/m³

Sol:

diameter of circular column (d) = 400mm

Service load (Ps) = 800KN.

Safe bearing capacity of soil (SBCS) = 80KN/m²

Depth of footing (Df) = 2m

Total load (P) = service load (Ps) + self wt. of footing

$= 800 + \frac{\gamma \cdot D_f \cdot P_s}{SBCS}$

$= 800 + \frac{18 \times 2 \times 800}{80}$

= 1160 KN

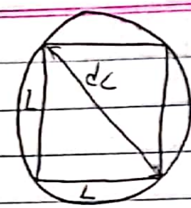
Area of footing $= \frac{\text{total load}}{\text{safe bearing capacity of soil (SBCS)}}$

$= \frac{1160}{80} = 14.5\ m^2$

Here

$L^2 + L^2 = d_c^2$

$\therefore L = \frac{d_c}{\sqrt{2}} = \frac{400}{\sqrt{2}} = 282.842\,mm$



converting circular column to equivalent square column.

Now

Area of footing $(A_f) = L*L = 14.5$

$\therefore L = \sqrt{14.5} = 3.8124\,m$

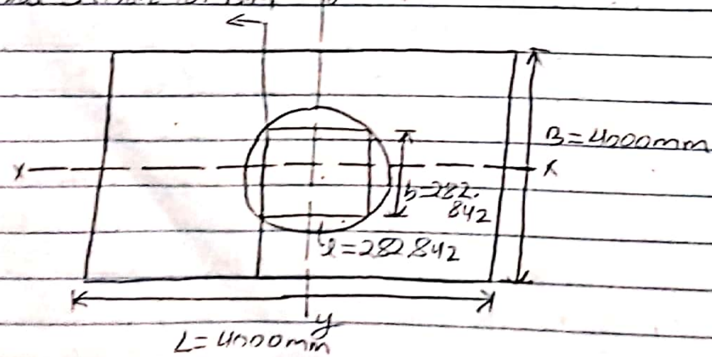
So, provide 4*4 (m*m) footing

net upward soil pressure (w) = $\frac{\text{factored total load}}{\text{area of footing}}$

$= \frac{1.5*1160}{4*4}$

$= 108.75\,kN/m^2$

Critical section for B.M



Calculation of Bending Moment.

BM along Y-Y axis.

$M_{y-y} = \frac{w.B\left(\frac{L}{2} - \frac{l}{2}\right)^2}{2}$

$= \frac{108.75*4*\left(\frac{4}{2} - \frac{0.282842}{2}\right)^2}{2}$

$= 751.3137\,kNm$

Bending Moment along X-X axis

$M_{x-x} = \frac{w.L\left(\frac{B}{2} - \frac{b}{2}\right)^2}{2}$

$= \frac{108.75*4\left(\frac{4}{2} - \frac{0.282842}{2}\right)^2}{2}$

$= 751.3137\,kNm$ $\quad M_{yy} = M_{x-x}$

Calculation of depth of footing

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

or, $751.3137*10^6 = 0.36*20*4000*0.48d * (d - 0.42*0.48d)$

on solving, we get,

$d = 260.906$

Increase the depth of footing by 2 times $d = 2*260.906 = 521.812 \approx 530\,mm$

provide 60mm effective cover, overall depth (D) = 530+60

= 590mm

Calculation of area of reinforcement

along Y-Y direction

$$M_{y-y} = 0.87 f_y A_{sty} \left(d - \frac{f_y A_{sty}}{f_{ck} \cdot b}\right)$$

or, $751.3137 \times 10^6 = 0.87 \times 415 \times A_{sty} \left(590 - \frac{415 \times A_{sty}}{20 \times 4000}\right)$

on solving we get.

A_{sty} = ~~3643.704~~ 4089.98 mm^2

provide 16mm φ bar at spacing $= \frac{\frac{\pi}{4} \times 16^2}{~~3643.704~~\ 4089.98} \times 4000$

= ~~220.736 mm~~ 196.64 mm

≈ ~~200mm~~ 180mm (reduce spacing)

provide 16mm φ bar 180mm c/c

actual (Ast) provided $= \frac{\pi \times 16^2}{4 \times 180} \times 4000$

= ~~4021.24~~ mm^2

4468.043 mm^2

reinforcement along X-X direction

$$M_{x-y} = 0.87 f_y A_{stx} \left(d - \frac{f_y A_{stx}}{f_{ck} \cdot L}\right)$$

or, $751.3137 \times 10^6 = 0.87 \times 415 \times A_{stx} \left(590 - \frac{415 \cdot A_{stx}}{20 \times 4000}\right)$

On solving

$A_{stx} = 4089.98\ mm^2$

provide 16mm φ bar at spacing $= \frac{\frac{\pi}{4} \times 16^2}{4089.98} \times 4000$

= 196.64mm

= 180mm

provide 16mm φ bar 180mm c/c.

check for one way share.

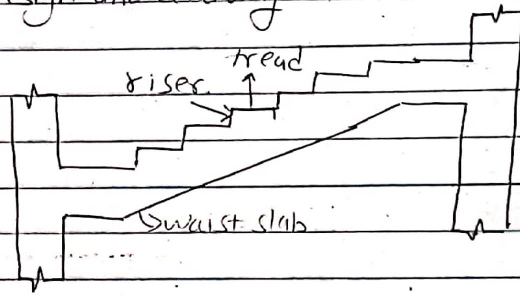
$$V_{y-y} = w \cdot B \left(\frac{L}{2} - \frac{\ell}{2} - d\right)$$

$$= 108.75 \times 4 \times \left(\frac{4}{2} - \frac{0.282842}{2} - \right.$$

2021 January 16 Sunday

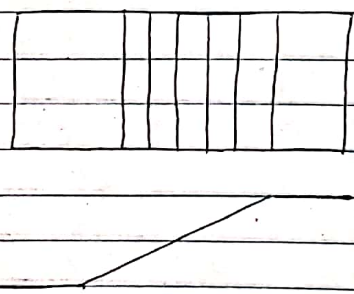
Chapter:-five. Design and detailing of miscellaneous structures

staircase.



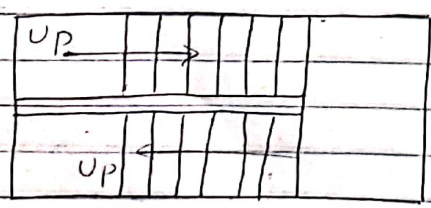
Types of staircase

1> Single flight staircase

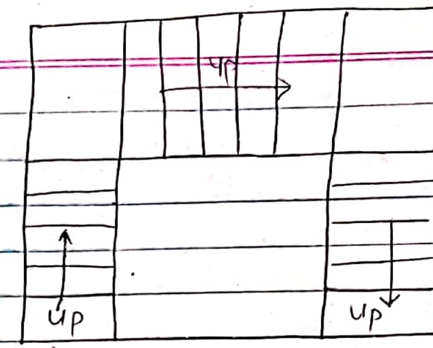


2> Double flight staircase.

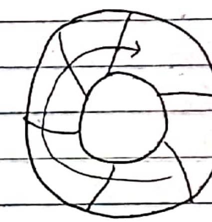
a> Dogg legged/well staircase



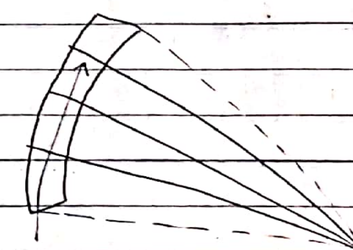
b> open legged/well staircase.



3> Spiral staircase.



4> Helicoidal staircase.



- staircase spaning longitudinally.
- Staircase spaning transversly.

-for residential/office building.

Riser R → (150-200) mm

Tread T → (250-300) mm

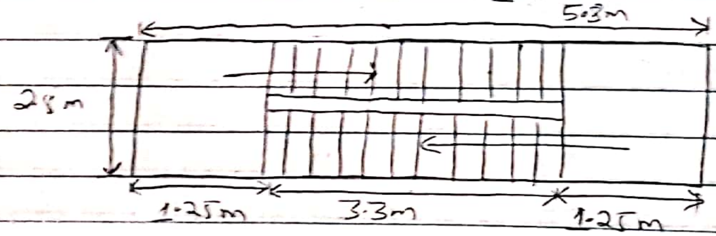
33) Design a dogged legged staircase in a room 2.8m*5.8m clear size for a office building assuming floor to floor height of 3.6m, flight width 1.2m and landing width 1.25m. Assume the stairs to be supported on 230mm thick masonry wall at the edges of landing parallel to the riser. Use M20 concrete & Fe415 steel. assume live load of 5kN/m².

soln

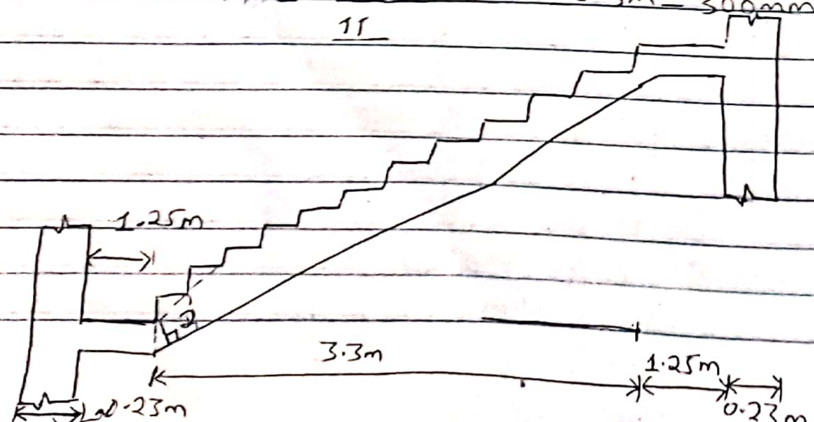
assuming two flights, hight of each flight $= \frac{3.6}{2} = 1.8m$

no of riser $= \frac{1800}{150} = 12$. (do not take in decimal)

no of tread = no of riser − 1 = 12 − 1 = 11



width of tread (T) $= \frac{5.8-2*1.25}{11} = 0.3m = 300mm$



effective length = 3.3 + 2*1.25 + 0.23 = 6.03m

waist slab thickness $= \frac{\text{effective length}}{20 * \text{modification}} = \frac{6030}{20*1.5} = 201mm$

(simply supported case)

providing 12mm ϕ bar at 15mm clear cover, overall depth $= 201 + 15 + \frac{12}{2}$

= 222mm

effective depth (d) $= 230 - 15 - \frac{12}{2}$

= 209mm

overall depth in loading is reduced to 190mm since B.M in landing is low.

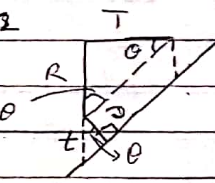
Load calculation:

On steps $\gamma = 25 kN/m^3$

Self wt. of waist slab $= 25 \cdot D * \frac{\sqrt{R^2+T^2}}{T}$ kN/m²

$= 25 * 0.23 * \frac{\sqrt{0.15^2+0.3^2}}{0.3}$

$= 6.428 kN/m^2$



$\cos\theta = \frac{D}{t}$

Self wt of steps $= 25 * \frac{1}{2} \cdot D$ (Riser only)

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

$= 1.875 kN/m^2$

$t = \frac{D}{\cos\theta} = \frac{D}{T/\sqrt{R^2+T^2}} = \frac{D\sqrt{R^2+T^2}}{T}$

also $\cos\theta = \frac{T}{\sqrt{R^2+T^2}}$

floor finish (40mm) = $\frac{40}{1000} * 24 = 0.96\ kN/m^2$

Liveload = $5\ kN/m^2$

Total load = $14.263\ kN/m^2$

factored load = $21.394\ kN/m^2$

on landing

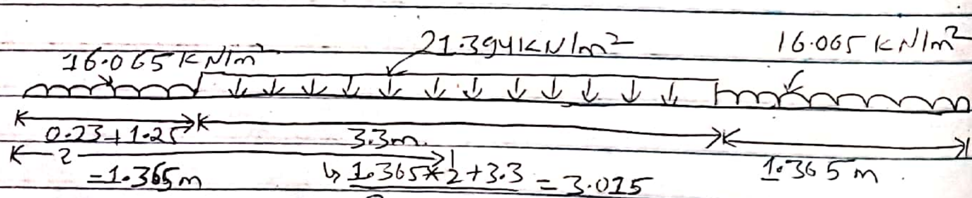
Self wt of waist slab = $25 * 0.19 = 4.75\ kN/m^2$

floor finish = $\frac{40}{1000} * 24 = 0.96\ kN/m^2$

Liveload = $5\ kN/m^2$

Total load = $10.71\ kN/m^2$

factored load = $1.5 * 10.71 = 16.065\ kN/m^2$



Reaction (R) = $16.065 * 1.365 + \frac{21.394}{2} = 57.228\ kN/m$

assuming per m

i.e 57.228 kN

Max^m B.M, $M_{max} = 57.228 * 3.015 - 16.065 * 1.365 * \left(3.015 - \frac{1.365}{2}\right) - 21.394 * \frac{3.3}{2} * \frac{3.3}{4} = 92.271\ kNm$

check for depth,

$M_{max} = 0.36 f_{ck} . X_u . b(d - 0.42 X_u)$

or, $92.271 * 10^6 = 0.36 * 20 * 0.48d * 1000 * (d - 0.42 * 0.48d)$

B = 1m

on solving we get,

$d = 182.867\ mm < 209\ mm$

OK

calculation of area of reinforcement

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

or, $92.271 * 10^6 = 0.87 * 415 * X \left(209 - \frac{415 * A_{st}}{20 * 1000}\right)$

on solving we get

$A_{st} = 1424.155\ mm^2$

provide 16mm φ bar at spacing $= \frac{\pi * 16^2}{4 * 1424.155} * 1000\ mm$

$= 141.179 \approx 140\ mm$

distribution bar = 0.12% of bD

$= \frac{0.12}{100} * 1000 * 230 = 276\ mm^2$

provide 10mm φ bar at spacing $= \frac{\pi * 10^2}{4 * 276} * 1000 = 284.56\ mm$

$\approx 280\ mm$

check for shear:

Ultimate shear stress (τ_u) (at support) $= \frac{V_u}{b d} = \frac{57.228 * 10^3}{1000 * 190} = 0.3012\ N/mm^2$

(b = landing width)

% of steel $= \frac{A_{st}}{b . d} * 100\% = \frac{1436.156}{1000 * 190} * 100\% = 0.755\ \%$

From table $\tau_c = 0.56 N/mm^2$

$\tau_c' = k.\tau_c$ where $k = 1.15$

$\tau_c' = 1.15 * 0.56 = 0.644 N/mm^2 > 0.301 N/mm^2$

OK

Check for devellopment length

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

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

Moment of reinforcement (M) $= 0.87 f_y \frac{A_{st}}{2} \left(d - \frac{f_y A_{st}}{2 f_{ck} b}\right)$

$= 0.87 * 415 * \frac{1436.156}{2} \left(209 - \frac{415 * 1436.156}{2 * 20 * 1000}\right) * 10^{-6}$ kNm

$= 50.323$ kNm

Now,

$L_d \leq 1.3 \frac{M}{V} + L_0$

or, $47\phi \leq 1.3 * \frac{50.323}{57.228} * 10^3$

or, $47\phi \leq \frac{1143.145}{47} = 24.322$

$\therefore \phi \leq 24.322$ mm

Since actual dia provided is 16mm it is safe in development length