GOVT. POLYTECHNIC, JAGATSINGHPUR

CIVIL ENGINEERING DEPARTMENT

LEARNING MATERIAL OF **STRUCTURAL DESIGN 1**

4^{TH} SEMESTER

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Dt - 05/01/2019

Structural Design-1 (R.C.C Structural)
Structural Design-2 (Steel Design Structure)

R.C.C Structure :-

IS code = IS:456-2000
(SP-16)

Methods for Design and analysis of R.C.C :-

1. WSM (working stress method) IS:456-1978
2. LSM (limit stress method) IS:456-2000
3. ULM (ultimate load method)

Analysis :-

finding force, moment, S.F, B.M, Torsion from the given designed section.

Design :-

Determination of Area of steel, dimension of the section from given load or moment.

1. Working stress method (1978-2000) (IS:456-1978) :-

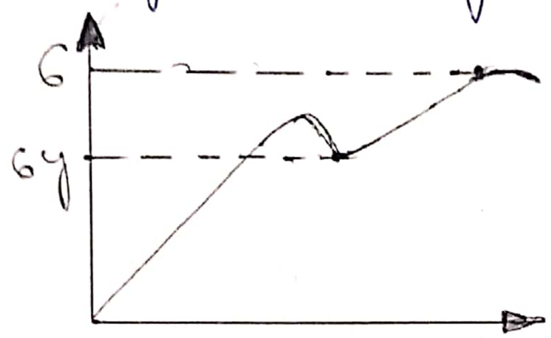
→ It is the traditional method of designing
→ Working stress method is based on linear elastic theory.
→ WSM not only applied for R.C.C structure but also applied for steel and timber structure.
→ It is the 1st method accepted by all national countries.
→ It is also called as modular ratio method.
→ WSM is a stress governing method.
→ The F.O.S of steel and concrete are 1.8 and 3 respectively.

$$F.O.S = \frac{\text{ultimate stress}}{(\text{Permissible or working or safe})\ \text{stress}} \Rightarrow (\text{for Brittle material})$$

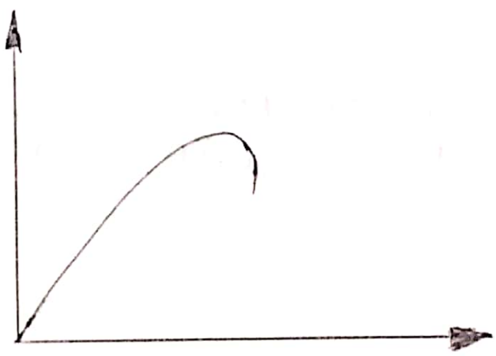
FOS = Yield stress / (permissible or working or safe) stress => (For Ductile material)

MS bar = Mild Steel steel

HYSD = High Yeild strength Deformed bar) = Fe415, Fe500, Fe550 — Yeild stress, Formula



(Mild steel)



Draw-backs (or) disadvantages of WSM :-

-> WSM neither shows the true strength nor gives true factor of safety (FOS)

-> WSM results in large compression steel as compared to LSM.

-> Because of non-linear stress strain curve and creep of concrete there is no definate modulus of elasticity to calc in the design.

Assumptions of WSM :-

(i) At any cross section, Plane section before bending remains plane after bending.

(ii) All tensile stresses are taken up by main reinforcement and none by concrete ->

Tensile strength of concrete is ignored.

(iii) The stress - strain relationship of steel and concrete under working load is a straight line

(iv) There exists a perfect bond between steel and of concrete. $\boxed{\text{C.T.E concrete} = 12\times10^{-6}/°C \rightarrow \text{bamboo}}$

(v) The modular ratio has the value $\frac{280}{3\sigma_{cbc}}$

when σ_{cbc} is permissible compressive stress due to bending in concrete in N/mm2

* □□□ in fine c.s exist and every c.s will be remain plain.

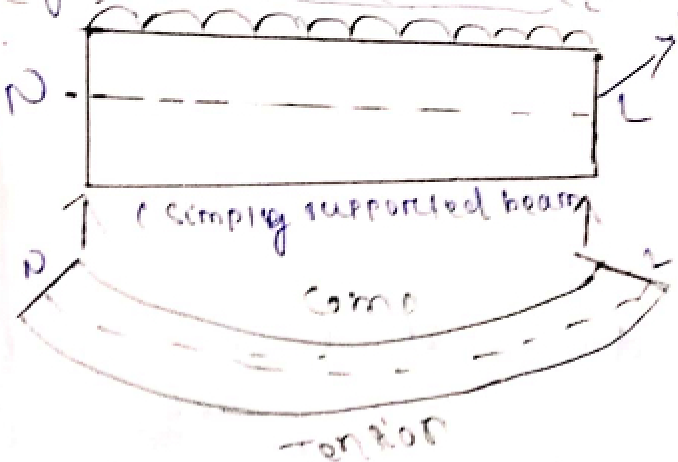
* Tensile → take by steel only why n/t concrete → because. $f_{cr} \rightarrow$ tensile strength of concrete $= 0.7\sqrt{f_{ck}}$

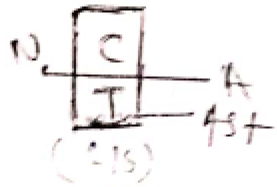
$M_{20} \Rightarrow f_{ck} = 20\ \frac{N}{mm^2} \Rightarrow f_{cr} = 0.7\sqrt{f_{ck}} = 0.7\times\sqrt{20} = 3.13\ mm$

Brittle material
Strength in compression
weak in tension
moderate in shear Ex:- c.c, brick, glass cast, iron

Ductile material :-
strong in tension
weak in shear
moderate in compression Ex - steel, aluminium.

Single reinforced beam :-





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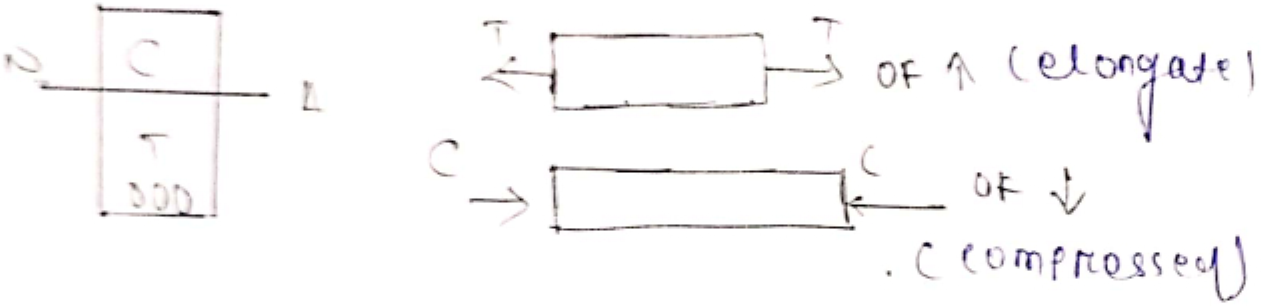
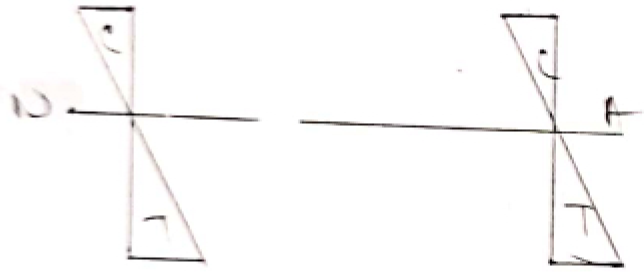
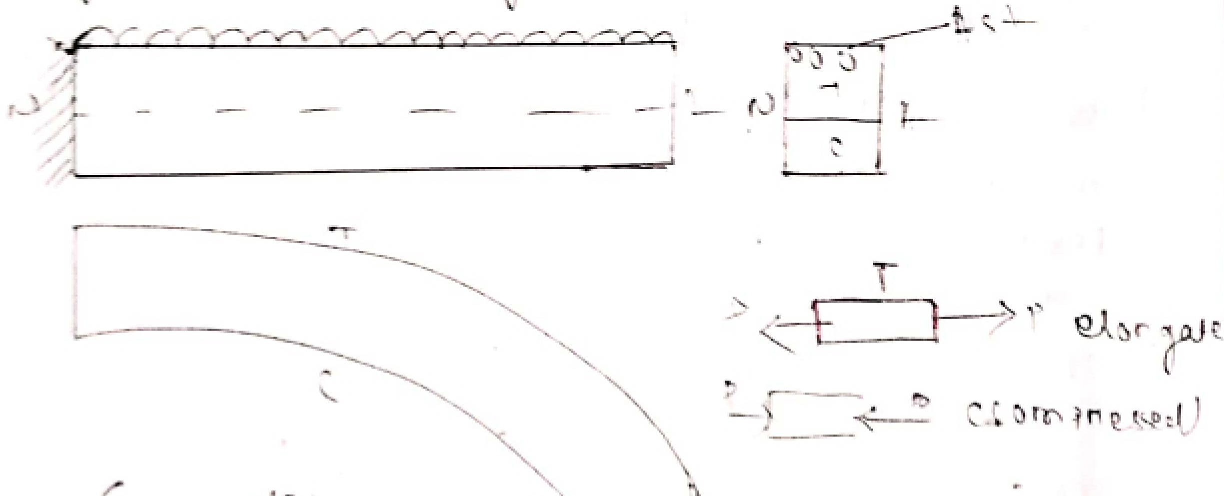




Stress dgm Stress dgm



(cantilever beam)

Singly Reinforced beam :-

-> If the steel reinforcement shall be provided only on tension zone in the beam, then the beam is called as singly Reinforced beam.

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Dt :- 07/07/2026

Basic Definitions :-

Engineering mechanics :-

→ It is the physical science which deals with the forces and moments either the bodies are in rest condⁿ or motion.

Force :-

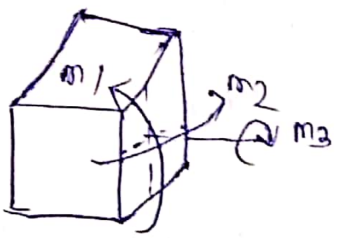
→ It is an agency which changes or tends to change the original position of a body.

Moment :-

→ It is the measure of rotational effect.

1. Due to rotational effect in the member if bending then it is called Bending moment
2. Due to rotational effect in the member it twisting occurs then is called torsional moment.

Note



→ Bending moment causes the rotation of the cross section about its N.A.

→ Torsional moment causes rotation of cross-section about longitudinal axis.

Centre of mass :-

→ It is a point through which the resultant mass is passing

Centre of gravity :-

→ It is a point through which the resultant weight is passing

Centroid :-

→ It is similar to centre of mass & centre of gravity but applicable only for plane figure, i.e. whose geometry is already known.

Note :-

→ First moment of area is also known as section modulus.

MOI :-

It is defined as the second moment of area

Strength of material :-

→ It is the study of behaviour of materials and members, under various loading condition

Strength :-

→ Resistance against failure is called strength

Stiffness :-

Resistance against deformation is called stiffness

Stress :-

→ It is defined as an internal resistance developed on a body per unit area, unit kPa, MPa, GPa

kPa	MPa	GPA
10^3 Pa	10^6 Pa	10^9 Pa
$10^3 N/m^2$	$10^6 N/m^2$	$10^9 N/m^2$
$\frac{10^3}{10^6} N/mm^2$	$\frac{10^6}{10^6} N/mm^2$	$\frac{10^9}{10^6} N/mm^2$
$10^{-3} N/mm^2$	$1 N/mm^2$	$10^3 N/mm^2$

Strain :-

It is defined as the ration between change in dimension to the original dimension.

SF :-

→ It is the vertical unbalanced force of a particular section either considering from left of the section or right the section.

SFD :-

It is the graphical representation drawn for calculated shear force values at salient point.

B.M :-

→ It is defined as the unbalanced moment at a particular section either considering from left of the section or right of the section,

BMD :-

→ It is the graphical representation drawn for calculated B.M value at salient points.

Reaction :-

It is the self adjusting force which is developed due to externally applied loads

Strength :-

It is the maximum stress developed in a body just before failure.

Moment of resistance :-

→ It is the capacity of section which can resist the bending moment caused due to external loads

(or)

→ It is the max^m BM developed in a body just before failure.

Structure :-

→ It is an arrangement of structural element in such a way that the loads are tranfered from one to another safely.

Analysis :-

Determination of forces and moments for the given structural member.

Design :-

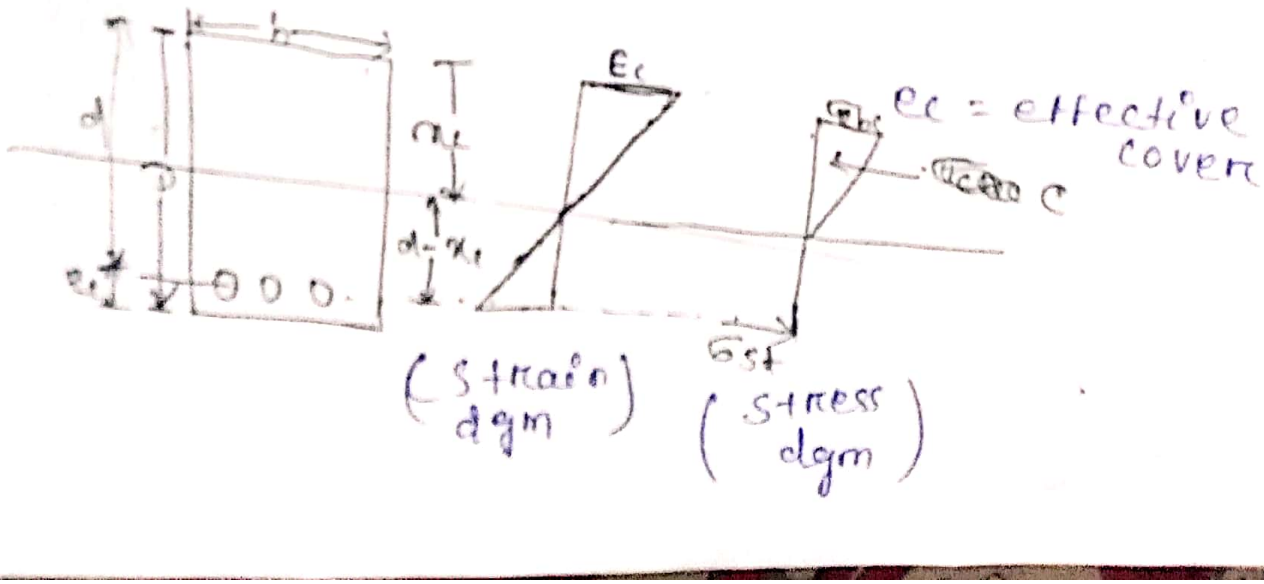
→ Determination of the dimensions for the analyzed structural ~~membe~~ member is called design

Design of concrete structures :-

→ In this design means determination of dimension as well as calculation of area of steel which includes material, R.C.C designs, etc.

Working stress method :-

Analysis of singly Reinforced section :-



Creep :- the defformation of a memberr due to constant and ~~long term load~~ longterm load

(1) ~~Critical depth of N.A (Xc)~~ :-

(b) b = width of beam

D = overall depth of beam

$d = D - e_c$ = Effective depth of beam

e_c = Effective cover

ϵ_c = Strain at extreme compression fibre

Ferrous ϵ_s = Strain at the level of tension steel.

For ferro (or) mild steel $\Rightarrow f_y = 250\ N/mm^2$ for Fe415 (HYSD bar) $\Rightarrow f_y = 415\ N/mm^2$

yield stress of steel.

M20 $\Rightarrow f_{ck} = 20\ N/mm^2$

mix f_{ck}

M30 $\Rightarrow f_{ck} = 30$ MPa

M25 = f_{ck} = 25 mpa

f_{ck} = Characteristic stress of concrete.

Permissible Stresses :-

For concrete :-

$$\text{fos} = \frac{\text{ultimate stress } (f_{ck})}{\text{Permissible stress } (\sigma_{cbc})}$$

$$3 = \frac{f_{ck}}{\sigma_{cbc}} = 3 \times \sigma_{cbc} = f_{ck}$$

$$\Rightarrow \boxed{\sigma_{cbc} = \frac{f_{ck}}{3}}$$

σ_{cbc} = Permissible stresses in concrete in bending with compression.

M20 $\Rightarrow f_{ck} = 20$ mpa $\Rightarrow \sigma_{cbc} = \frac{f_{ck}}{3} = \frac{20}{3} = 7$ mpa

M15 $\Rightarrow f_{ck} = 15$ mpa $\Rightarrow \sigma_{cbc} = \frac{f_{ck}}{3} = \frac{15}{3} = 5$ mpa

M25 = f_{ck} = 25 mpa = $\sigma_{cbc} = \frac{f_{ck}}{3} = \frac{25}{3} = 8$ mpa

M30 = f_{ck} = 30 mpa = $\sigma_{cbc} = \frac{f_{ck}}{3} = \frac{30}{3} = 10$ mpa

For steel :-

$$F.O.S = \frac{\text{Yield stress } (f_y)}{\text{Permissible stress } (\sigma_{st}) \text{ in steel}}$$

$$1.8 = \frac{f_y}{\sigma_{st}} \Rightarrow \boxed{\sigma_{st} = \frac{f_y}{1.8}}$$

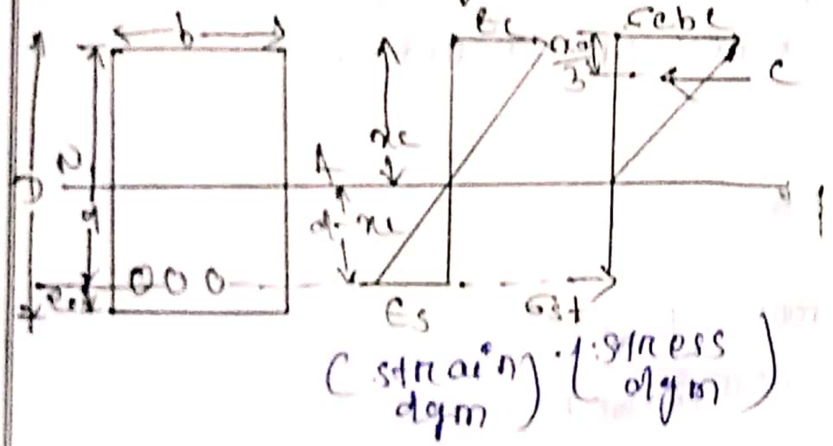
$Fe250 \Rightarrow f_y = 250\ N/mm^2$

$$\sigma_{st} = \frac{250}{1.8} = 140\ mpa$$

$$Fe415 \Rightarrow f_y = 415\ N/mm^2 \Rightarrow \sigma_{st} = \frac{415}{1.8} = 230\ mpa$$

$\boxed{\text{Direct permissible stress } (\sigma_{cc}) = \frac{f_{ck}}{4}}$:-

Analysis of single Reinforced section :-



(1) Critical Depth N.A. (x_c)

For strain dgm by similar triangle law

$$\frac{d-x_c}{x_c} = \frac{e_s}{\epsilon_c} \Rightarrow \frac{d-x_c}{x_c} = \frac{e_s}{\epsilon_c}$$

$$\Rightarrow \frac{d}{x_c} - 1 = \frac{\frac{\sigma_{st}}{E_s}}{\frac{\sigma_{cbc}}{E_c}} = \frac{\sigma_{st}}{E_s} \times \frac{E_c}{\sigma_{cbc}}$$

$$\Rightarrow \frac{d}{x_c} - 1 = \frac{\sigma_{st}}{\sigma_{cbc}} \cdot \frac{1}{\frac{E_s}{E_c}} \qquad \boxed{\frac{E_s}{E_c} = m}$$

$$\Rightarrow \frac{d}{x_c} - 1 = \frac{\sigma_{st}}{m\,\sigma_{cbc}}$$

$$\Rightarrow \frac{d}{x_c} = 1 + \frac{\sigma_{st}}{m\sigma_{cbc}} \Rightarrow \frac{d}{x_c} = \frac{m\sigma_{cbc} + \sigma_{st}}{m\sigma_{cbc}}$$

$$\Rightarrow \frac{x_c}{d} = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$$

$$\Rightarrow \frac{x_c}{d} = \frac{m}{m + \frac{\sigma_{st}}{\sigma_{cbc}}} \Rightarrow \boxed{\frac{x_c}{d} = \frac{m}{m+r}} \quad \boxed{r = \frac{\sigma_{st}}{\sigma_{cbc}}}$$

r = stress ratio $= \frac{\sigma_{st}}{\sigma_{cbc}}$

$$\frac{x_c}{d} = K \Rightarrow \boxed{x_c = Kd} \quad K = \frac{m}{m+r} \quad \frac{280}{3\sigma_{cbc}} \quad \frac{\sigma_{st}}{\sigma_{cbc}}$$

K = N.A depth factor.

(2) Actual depth of N.A (x_a) :-

↳ Always find x_a from 1st moment of Area.

$$\boxed{(b \times x_a) \cdot \frac{x_a}{2} = mA_{st}(d - x_a)}$$

mA_{st} = Transformed concrete section

(3) compressive force (C) :-

(Force = Area of stress dgm)

$$C = \frac{1}{2}\, \underset{N/mm^2}{\sigma_{cbc}} \cdot \underset{mm}{x_a} \cdot b)$$

$$\Rightarrow \boxed{C = \frac{1}{2}\sigma_{cbc}\, b x_a} \; @ \frac{x_a}{3} \text{ from top fiber}$$

(4) Tensile force :- (T)

$\boxed{T = \sigma_{st} A_{st}}$ @ d from top fibre.

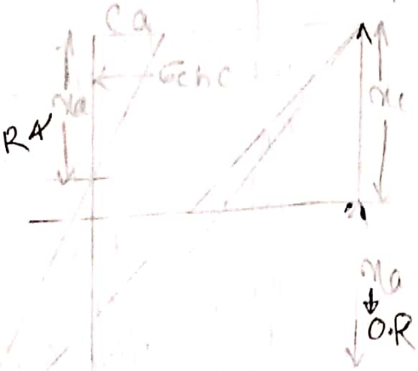
Lever arm (Z) :-

↳ It is the distance between compressive force and tensile force.

$$\boxed{Z = d - \frac{x_a}{3}}$$

Types of section:-

(1) Balanced section
(2) under-reinforced section
(3) over reinforced section.



(1) Balanced section:-

-> It is the type of the section in which the stresses in the permissible stress simultaneously.

-> It is assumed that both the material will fail simultaneously

-> mathematically $x_a = x_c$

(2) under reinforced section:-

-> It is the type of the section in which the area of steel shall be provided less than that of balanced section, hence the stresses in steel will reach its permissible stress first.

-> It is the safest section among all, because it will give prior warning before failure (i.e the nature of failure is gradual)

Mathematically $x_a < x_c$

(3) over reinforced section:-

-> It is the type of the section in which the area of steel shall be provided more than that of balanced section, hence the stresses in steel will reach its permissible stress first.

-> It is n't the safest section because the nature of failure is sudden and brittle because concrete fail first.

-> Mathematically $x_a > x_c$

(5) Moment of Resistance:-

$\boxed{MOR = \text{Moment of Resistance}}$

Balanced section:-

$MR = C \times Z$ (or) $T \times Z$

$M.R = C \times Z = \frac{1}{2}\,\sigma_{cbc}\, b x_c \left(d - \frac{x_c}{3}\right)$ — comp

$= \frac{1}{2}\sigma_{cbc} \cdot b \cdot kd\left(d - \frac{kd}{3}\right) = \frac{1}{2}\sigma_{cbc}\, b\, kd \cdot d\left(1 - \frac{k}{3}\right)$

$= \frac{1}{2}\sigma_{cbc} \cdot bd^2 j \cdot k = \left(\frac{1}{2}\sigma_{cbc} \cdot jk\right) bd^2$

$\boxed{x_c = kd}$

$\Rightarrow \boxed{M.R = Q\, bd^2}$ $\boxed{Q = M.O.R\ \text{factor} = \frac{1}{2}\sigma_{cbc}\, jk}$

$MR = T \times Z = \sigma_{st} A_{st}\left(d - \frac{x_c}{3}\right) = \sigma_{st} A_{st}\left(d - \frac{kd}{3}\right)$

$= \sigma_{st} A_{st}\, d\left(1 - \frac{k}{3}\right)$

$\Rightarrow \boxed{M.R = \sigma_{st} A_{st}\, j d}$ $\boxed{\Rightarrow A_{st} = \frac{MR}{\sigma_{st}\, j d}}$

Under reinforced section:-

$M.R = T \times Z$ (∵ failure 1st governed by tensile steel)

$$\boxed{\Rightarrow M.R = \sigma_{st} A_{st} \left(d - \frac{x_a}{3}\right)}$$

Ductile ⇒ Gradual failure ⇒ Gives warning

Brittle ⇒ Sudden failure ⇒ Doesn't give warning

Over-reinforced section:-

$M.R = C \times Z$ (∵ failure governed by concrete)

$$\Rightarrow \boxed{M.R = \frac{1}{2}\sigma_{cbc} \cdot b x_a \left[d - \frac{x_c}{3}\right]}$$

Constants:-

① N.A depth factor $(k) = \frac{m}{m+r}$ $\left(m = \frac{280}{3\sigma_{cbc}}, r = \frac{\sigma_{st}}{\sigma_{cbc}}\right)$

② Lever arm constant $(j) = 1 - \frac{k}{3}$

③ M.O.R constant $(Q) = \frac{1}{2}\sigma_{cbc}\, j k$

(6) Percentage of tensile steel:- (P_t)

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

$$M.R = \sigma_{st} A_{st} \left(d - \frac{x_c}{3}\right)$$

$$= \sigma_{st} A_{st} \left(d - \frac{kd}{3}\right) = \sigma_{st} A_{st}\, d\left(1 - \frac{k}{3}\right)$$

$$M.R = \sigma_{st} A_{st}\, j d$$

$$\Rightarrow A_{st} = \frac{M.R}{\sigma_{st}\, j d} = \frac{\frac{1}{2}\sigma_{cbc} \cdot \cancel{j} k \cdot b \cancel{d^2}\, d}{\sigma_{st}\, \cancel{j}\cancel{d}}$$

$$= \frac{\frac{1}{2}\sigma_{cbc} \cdot k \cdot bd}{\sigma_{st}}$$

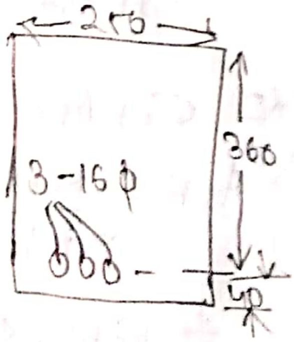
$$\Rightarrow P_t = \frac{100 A_{st}}{bd} = \frac{\cancel{100}^{50}}{\cancel{bd}} \times \frac{1}{\cancel{2}} \frac{\sigma_{cbc}\, k\, b\cancel{d}}{\sigma_{st}}$$

c) Ot $\boxed{p_t = 50 \left[\frac{\sigma_{cbc}}{\sigma_{st}}\right] k}$

Under reinforced section	Balanced section	Over-reinforced section.
$C_a < \sigma_{cbc}$ $T_a = \sigma_{st}$	$C_a = \sigma_{cbc}$ $T_a = \sigma_{st}$	$C_a = \sigma_{cbc}$ $T_a < \sigma_{st}$
$x_a < x_c$	x_c	$x_a > x_c$
$A_{st} < (A_{st})_{bal}$	$A_{st} = (A_{st})_{bal}$	$A_{st} > (A_{st})_{bal}$
$M.R < (M.R)_{bal}$	$M.R = (M.R)_{bal}$	$M.R > (M.R)_{bal}$
$z > z_{bal}$	$z = z_{bal}$	$z > (z)_{bal}$.

Problem : 1

Determind the lever arm for section shown in figure if effective cover equal to 40mm and the material used are ~~M20 and fe 250~~ m20 concrete and fe 250 steel.



Ans :-

Given data :-

$M20 \Rightarrow \sigma_{cbc} = 7\,MPa$

$M250 \Rightarrow \sigma_{st} = 140\,mpa$

$d = 360\,mm$

$A_{st} = 3 \times \left(\frac{\pi}{4} \times 16^2\right) = 603\,mm^2$

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

$b x_a \left(\frac{x_a}{2}\right) = m\, A_{st} (d - x_a)$

$= 250 \frac{x_a^2}{2} = 13.33 \times 603 (360 - x_a)$

$= 125\, x_a^2 = 2893676.4 - 8037.99\, x_a$

$\Rightarrow 125\, x_a^2 + 8037.99\, x_a - 2893676.4 = 0$

$\Rightarrow x_1 = 123.35$

$\cancel{x_2 = -187.66}$

$x_a = 123.35 mm$

$\therefore$ lever arm $= d - \frac{x_a}{3}$

$= 360 - \frac{123.35}{3}$

$= 318.88 mm$

2mp

Problem - 2 :-

A single reinforced rectangular beam of width 250mm and 460mm effective depth is reinforced with 3 number 20mm diameter bars. Find out the moment of resistance of this section. This materials are M20 concrete and Fe415 steel.

Ans :-

Given data :-

width of beam = 250mm

eff. depth = d = 460mm

$A_{st} = 3 \times \frac{\pi}{4} \times 20^2 = 942 mm^2$

$M20 = f_{ck} = 20 \Rightarrow \sigma_{cbc} = 7 mpa$

$Fe415 = f_{ck} = 415 \Rightarrow \sigma_{st} = 230 mpa$

Find out the types of section

$x_a = b x_a \cdot \frac{x_a}{2} = mA_{st}(d - x_a)$

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

$x_a = b x_a \cdot \frac{x_a}{2} = mA_{st}(d - x_a)$

$\overset{125}{\cancel{250}} \cdot \frac{x_a^2}{\cancel{2}} = 13.33 \cdot 942 (460 - x_a)$

$= 125 x_a^2 = 5776155.6 - 12556.86 x_a$

$\Rightarrow 125 x_a^2 + 12556.86 x_a - 5776155.6$

$x_1 = 170.52$

$\cancel{x_2 = -270}$

$x_a = 170.52$

$x_c = kd$

$k = \frac{m}{m+r}$

$r = \frac{\sigma_{st}}{\sigma_{cbc}}$

$\sigma_{st} = \frac{f_y}{1.8}$ $(fe = 415 = f_y)$

$= \frac{415}{1.8} = 230.55$

$r = \frac{\sigma_{st}}{\sigma_{cbc}} = \frac{230.55}{7} = 32.93$

$k = \frac{m}{m+r} = \frac{13.33}{13.33 + 32.93}$

$= 0.288$

$x_c = kd$

$= 0.288 \times 460 = 132.48$

$x_a > x_c$ = over-reinforced section

$M.R = \frac{1}{2} \sigma_{cbc} \cdot b x_c (d - \frac{x_c}{3})$

$= \frac{1}{2} 7 \cdot 250 \cdot 132.48 (460 - \frac{132.48}{3})$

$= 48204172.8$ N.mm

$$kN.m = \frac{48204172.8}{1000 \times 1000}$$

$= 48.204$ kN.m

date

$\boxed{09/01/2020}$

Problem 6-3

beam of size (230mm x 500mm) overall depth is reinforced with 4 numbers 12mm diameter bars. find the shape UDL self w.t on a span of 4.5m materials are M20 and mild steel Fe 250

Given data :-

$b = 230$ mm

$d = ?$

$D = 500$ mm

effective depth

$d = D - e_c = 500 - 40$ (assume effective cover = 40mm)

Area of tension steel $A_{st} = 4 \times \frac{\pi}{4} \times 12^2$

$A_{st} = 452\,mm^2$

simple supported span = 4.5m

$$M = \frac{wl^2}{8} = \frac{w \times 4.5^2}{8} = \frac{20.25}{8} w \text{ kN.m}$$

M.R = B.M of Beam

find out type of section

$x = kd$

$$k = \frac{m}{m+r}$$

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

$$r = \frac{\sigma_{st}}{\sigma_{cbc}} = \frac{f_y}{\sigma_{cbc}} = \frac{140}{7} = 20$$

$$= \frac{13.33}{13.33 + 20} = 0.4$$

Scanned with CamScanner

$\Rightarrow x_c = 0.4 \times 460 = 184mm$

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

$= \overset{115}{230} \frac{x_a^2}{2} = 13.33 \times 412 (460 - x_a)$

$= 115 x_a^2 = 2541731.6 - 6025.16 x_a$

$= 115 x_a^2 + 6025.16 x_a = 2541731.6$

$x_1 = 131.24$

$x_2 = -183.63$

$x_a = 131.24$

$x_a < x_c$ = under - reinforced section

$M.R = \sigma_{st} A_{st} (d - \frac{x_a}{3})$

$\sigma_{st} = 140 \times 412 (460 - \frac{131.24}{3})$

$= 24340510.93$

$= 24.34 \, kN.m$

Again given simply supported of span = 4.5

Bending moment $= \frac{wL^2}{8}$

$= \frac{w \times 4.5^2}{8} = \frac{(20.25) w}{8}$

$24.34 = \frac{20.25 w}{8}$

$w = \frac{24.34}{2.53125} = 10.4059$

$w = \frac{24.34 \times 8}{4.5^2} = 10.4059$

$\boxed{Dt\ 6 - 13/01/2020}$

A simple supported beam of 6m span carries a UDL of 12kN/m inclusive of its self weight. The beam is 230mm wide and effective depth is of 580mm. Find the steel area. The material are ~~include~~ M20 concrete and Fe415 reinforcement grade.

Given data :-

$l = 6m$

$UDL = 12 kN/m$

width of the beam (b) = 230mm

effective depth of the beam = 580 mm

$M20 \Rightarrow f_{ck} = 20 N/mm^2 \quad \sigma_{cbc} = 7 N/mm^2$

$Fe415 \Rightarrow f_y = 415 N/mm^2 \Rightarrow \sigma_{st} = 230 N/mm^2$

maximum bending moment $= \frac{wL^2}{8}$

$= \frac{12 \times 6^2}{8} = 54 \, kN.m$

$max^m \, M.R = \frac{1}{2} \sigma_{cbc} \, b x_c (d + \frac{x_c}{3})$

$x_c = kd \, , \, k = \frac{m}{m+r}$

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

$k = \frac{m}{m+r} = \frac{13.33}{13.33 + \frac{230}{7}} = 0.29$

$= \frac{1}{2} \times 7 \times 230 \times (0.29 \times 580)\left(580 - \frac{0.29 \times 580}{3}\right)$

$= 70.94 \, kN.m$

$\Rightarrow$ As $BM < (M.R)_{bal} \Rightarrow$ It is a under reinforced section

$M.R = \sigma_{st} A_{st} (d - \frac{x_a}{3})$

$= 54 \times 10^6 = 230 \, A_{st} (580 - \frac{x_a}{3})$

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

$\overset{115}{\cancel{230}} \frac{x_a^2}{\cancel{2}} = 13.33 \, A_{st} (580 - x_a)$

$\Rightarrow 8.62\, xa^2 - 580\, A_{st} + A_{st}\, xa = 0$

$\Rightarrow \dfrac{54 \times 10^6}{230} = 580\, A_{st} - \dfrac{A_{st}\, xa}{3}$

$= \cancel{\dfrac{234782.6087}{580 - xa}} = \cancel{580\, A_{st}} - \cancel{\dfrac{A_{st}\, xa}{3}}$

$= \dfrac{234782.6087}{580 - \frac{xa}{3}} = A_{st}$

$\Rightarrow 8.62\, xa^2 - 580\, A_{st} + A_{st}\, xa = 0$

$\Rightarrow 8.62\, xa^2 - 580 \left(\dfrac{234782.6087}{580 - \frac{xa}{3}} \right) + \dfrac{234782.6}{580 - \frac{xa}{3}} \cdot xa = 0$

$= 8.62\, xa^2 - 580 \left(\dfrac{234782.6087}{\frac{1740 - xa}{3}} \right) + \left(\dfrac{2347862.6}{\frac{1740 - xa}{3}} \right) xa = 0$

$\Rightarrow 8.62\, xa^2 - 580 \left(\dfrac{234782.6 \times 3}{1740 - xa} \right) + \left(\dfrac{2347862.6 \times 3}{1740 - xa} \right) xa = 0$

$\Rightarrow 8.62\, xa^2 - 580 \left(\dfrac{704347.8}{1740 - xa} \right) + \left(\dfrac{704347.8}{1740 - xa} \right) \times xa = 0$

$\Rightarrow 8.62\, xa^2 - \dfrac{580 \times 704347.8}{1740 - xa} + \dfrac{704347.8\, xa}{1740 - xa} = 0$

$\Rightarrow \cancel{8.62\, xa^2 (1740 - xa) - 580 \times 704347.8}$

$\Rightarrow \dfrac{8.62\, xa^2 (1740 - xa) - 580 \times 704347.8 + 704347.8 \times xa}{1740 - xa} = 0$

$= -\underset{a}{8.62}\, xa^3 + \underset{b}{14998.8}\, xa^2 - \underset{c}{408521724} + \underset{d}{704347.8}\, xa = 0$

$x_a = 148\,mm$

$$m.R = 230 \times \frac{234782.6087}{\left(580 - \frac{148}{3}\right)}\left(580 - \frac{148}{3}\right)$$

$$54 \times 10^6 = 230 \times \frac{234782.6087}{580}$$

$$MR = \sigma_{st} \cdot A_{st}\left(d - \frac{x_a}{3}\right)$$

$$54 \times 10^6 = 230\, A_{st}\left(580 - \frac{148}{3}\right)$$

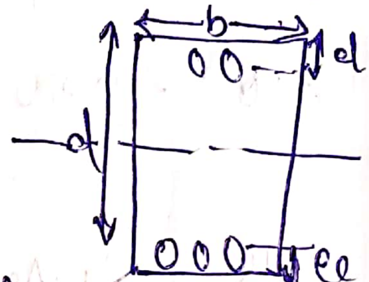
$$54 \times 10^6 = 230\, A_{st}\,(530.667)$$

$$\Rightarrow \frac{54 \times 10^6}{230 \times 530.667} = A_{st}$$

$$\Rightarrow A_{st} = 442.42\,mm^2$$

DOUBBLE RAINFORCED BEAM :—

→ If the steel reinforcement shall be provided on tensile as well as compression zone in a beam then the beam is called as ~~double~~ the rainforced beam double



NECESSITY OF DUBBLE RAINFORCED BEAM :—

When a bending movement due to external load is greater than movement of resistance of the beam (capacity of the beam), then double rainforced beam is provided otherwise beam will be become over rain forced section.

When B.M (external load) → M.R (capacity of beam) ⇒ ~~[illegible]~~ doubly reinforced beam provided

→ When the depth of the beam restricted from architectural point of view.

→ When the beam is subjected to reversal of stress

→ When the beam subjected to impact loads.

→ In a contineous beam at entermidiate support to counter act the negative bending movement.

LIMIT STATE METHOD :— (LSM) 03/03/2020

It is the method of design in which the structure are design for safety against collapse and checked for serviceability under working load.

Limit state :—

→ It is an acceptable limit for the safety & serviceability requirement before failure.

Limit state
↓

Limit state of collapse (L.C)

- → ~~Flexure~~ (or) Bending
- → Shear
- → Torsion
- → Compression
- → Bond

Limit state of serviceability (L.S)

(for checking whether safe or not what we have design in limit state of collapse)

- → Deflection
- → Cracking
- → Vibration and other secondary effect like shrinkage, creep, temperature chang.
- → Durability .
- → Fire resitance

Design load :— (or) (factored load) (fd) :—

$\boxed{f_d = f_{ck} \times \gamma_f}$ f_{ck} = Characteristic / working / permissible load

γ_f = partially safety factor for loads

$\boxed{f_d = f_{ck} \times \gamma_f}$ = 1.5.

Characteristic load :—

It is the value of load which has 95% of probability or not being exceeded through out ~~the~~ its design life period.

Sl no	Load combination.	Partially safety factor for load (γ_f)					
		L.C			L.S		
		DL	LL	WL	DL	LL	WL
(1)	DL + LL	1.5	1.5	—	1.0	1.0	
(2)	DL + WL / EL	1.5	—	1.5	1.0	—	1.0
(3)	DL + LL + WL / EL	1.2	1.2	1.2	1.0	0.8	0.8

(i) design strength :- (f_d)

$$\boxed{f_d = \frac{f_{ck}}{\gamma_m}}$$

f_{ck} — characteristic strength

γ_m — Partially safety factor for materials

(ii) characteristic strength :-

It is the value of the strength of the material below which not more than 5% of test results are expected to failed.

material	P.S.F (γ_m)	
	L.C	L.S
concrete	1.5	1.0
steel.	1.15	1.0

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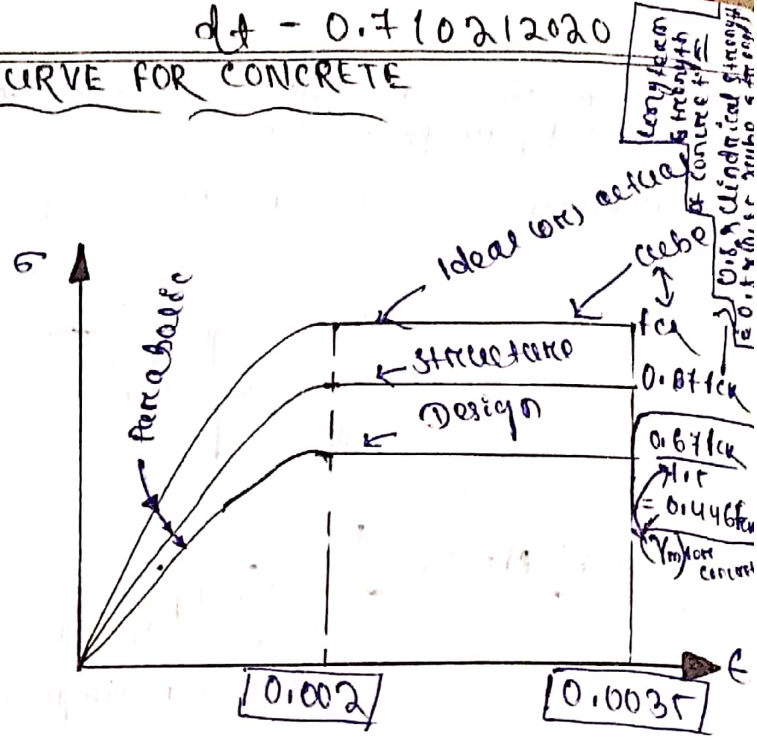
1) IDEAL STRESS - STRAIN CURVE FOR CONCRETE

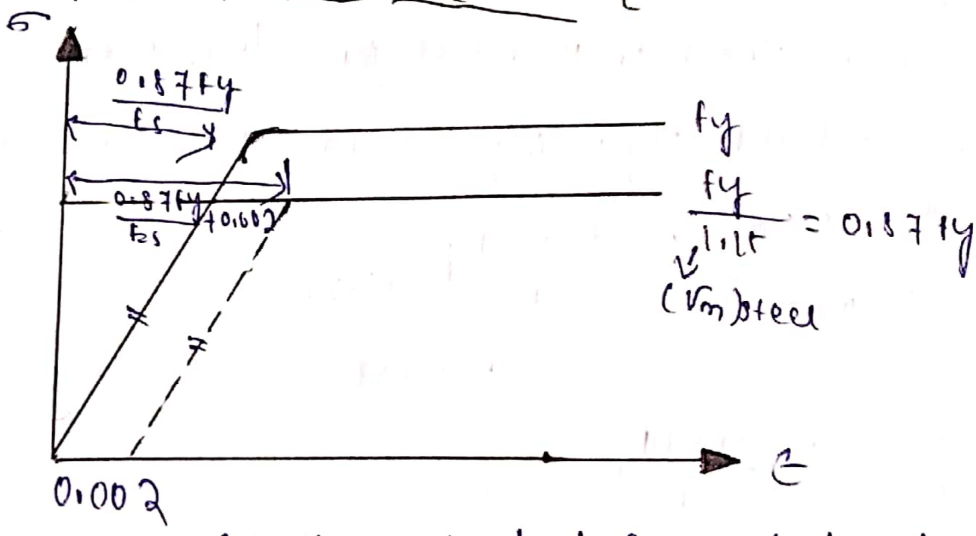


2) IDEAL STRESS - STRAIN CURVE FOR STEEL :-

$E = \frac{\sigma}{e}$

$= \frac{0.87 f_y}{E}$

$e = \frac{0.87 f_y}{E_s}$



(cold worked deformed bar)

Imp Assumption in limit state of collapse - flexure :-

(1) plane sections remains plane before bending and even after bending.

(2) Tensile strength of concrete is Ignored

(3) The maximum strength are extreme compression face in concrete is 0.0035

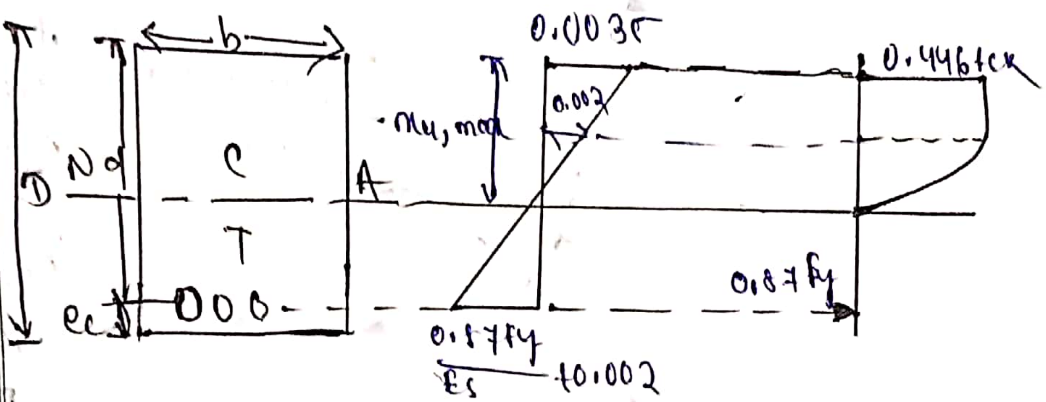
4) The strain in tension steel at the time of failure shall not be less than $\frac{0.87 f_y}{E_s} + 0.002$

5) The acceptable stress - strain curve is rectangular - parabola for concrete.

→ The partial safety factor concrete and steel are 1.5 and 1.15 respectively

⇒ The design stress in concrete and steel are $0.446 f_{ck}$ & $0.87 f_y$ respectively.

Analysis & Design of single reinforced beam :-



Analysis :-

(1) maximum depth of neutral Axis ($x_{u,max}$) :-

from strain dgm by similar triangle law

$$\frac{d - x_{umax}}{x_{umax}} = \frac{\frac{0.87 f_y}{E_s} + 0.002}{0.0035}$$

$$\Rightarrow \frac{d}{x_{umax}} - 1 = \frac{\frac{0.87 f_y}{E_s} + 0.002}{0.0035}$$

$$\Rightarrow \frac{d}{x_{umax}} = 1 + \frac{\frac{0.87 f_y}{E_s} + 0.002}{0.0035}$$

$$\Rightarrow \frac{d}{x_{umax}} = \frac{0.0035 + \frac{0.87 f_y}{E_s} + 0.002}{0.0035}$$

$$\Rightarrow \frac{x_{umax}}{d} = \frac{0.0035}{0.0055 + \frac{0.87 f_y}{2.1 \times 10^5}}$$

$$\Rightarrow \boxed{x_{umax} = \left[\frac{0.0035}{0.0055 + \frac{0.87 f_y}{2.1 \times 10^5}}\right] d}$$

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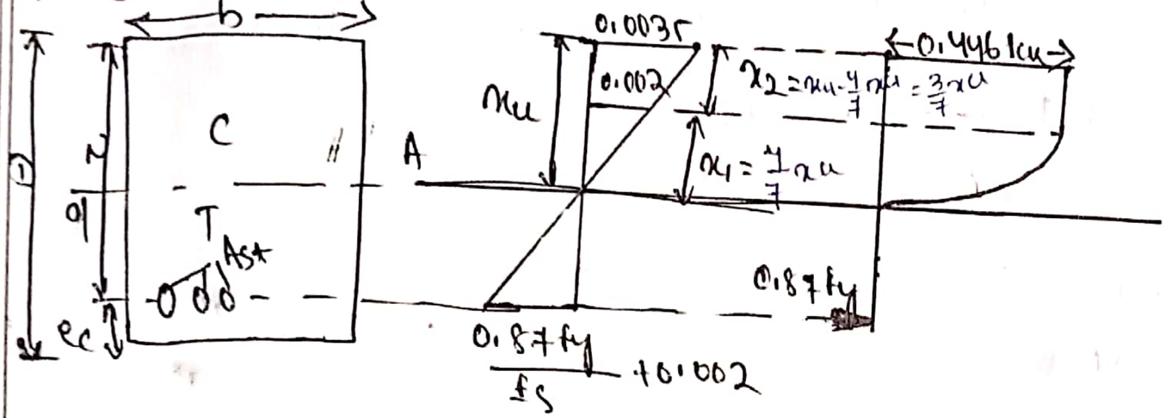
$Fe\,250 \Rightarrow f_y = 250\text{ MPa} \Rightarrow x_{u\,max} = 0.53\,d$
$Fe\,415 \Rightarrow f_y = 415\text{ MPa} \Rightarrow x_{u\,max} = 0.48\,d$
$Fe\,500 \Rightarrow f_y = 500\text{ MPa} \Rightarrow x_{u\,max} = 0.46\,d$

Stress block parameter :—

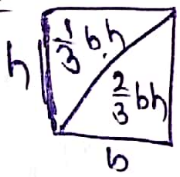


from stress diagram by similar triangle law.

$$\frac{x_1}{x_u} = \frac{0.002}{0.0035} \Rightarrow x_1 = \frac{4}{7}x_u$$

$$x_2 = x_u - x_1 = x_u - \frac{4}{7}x_u$$
$$= \frac{3}{7}x_u$$

Parabolic Area =



(1) compressive force (C) :—

C = Area of stress block
= rectangular area + parabolic area

$$= \left(0.446 f_{ck} \times \frac{3}{7}x_u\right) + \left(\frac{2}{3} \times 0.446 f_{ck} \times \frac{4}{7}x_u\right)$$

$$\Rightarrow 0.36 f_{ck} x_u$$

$$C = 0.36\, f_{ck}\, x_u$$
$N/mm^2 \times mm$

$$\left(\frac{N}{mm} \times mm = N\right)$$

$$\boxed{C = 0.36\, f_{ck}\, b\, x_u}$$

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[dt - 17/02/2020]

(ii) Distance of centroid of compressive force ($\bar{a}$) :—

$$\bar{a} = \frac{A_1 y_1 - A_2 y_2}{A_1 + A_2} = \frac{\left[(0.446 f_{ck} \times \frac{4}{7} x_u) \times (\frac{1}{2} \times \frac{4}{7} x_u)\right] + \left[(\frac{2}{3} \times 0.446 f_{ck} \times \frac{3}{7} x_u)(\frac{4}{7} x_u + \frac{3}{8} \times \frac{3}{7} x_u)\right]}{(0.446 f_{ck} \times \frac{4}{7} x_u) + (\frac{2}{3} \times 0.446 f_{ck} \times \frac{3}{7} x_u)}$$

$$= 0.416 x_u \simeq 0.42 x_u.$$

(iii) Tensile force (T) :—

T = stress $\times$ Area

$= (0.87 f_y) A_{st}$

(iv) Lever arm (z) :—

It is the distance between compressive force and tension force.

$z = d - 0.42 x_{u\,lim}$

(v) Actual depth of N.A (x_u) :—

$C = T$

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

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

Type of section :—

(1) Balanced (or) limiting section :— $x_u = x_{u\,max}$ (or) $M_u = M_{u\,lim}$ (or) $P_t = (P_t)_{lim}$

(2) Under Reinforced section :— $x_u < x_{u\,max}$ (or) $M_u = M_{u\,lim}$ (or) $(P_t < P_t)_{lim}$

(3) Over Reinforced section :— $x_u > x_{u\,max}$ (or) $M_u > M_{u\,lim}$ (or) $P_t > (P_t)_{lim}$.

NOTE :—

The design of over Reinforced sections are strictly avoided as for IS456 2000

MOMENT OF Resistance :— (M.R) :—

(1) Balanced Section (or) limiting Section :—

$M.R = C \times z$ (or) $T \times z$)

$M.R = C \times Z = (0.36 f_{ck} b x_{u,max})(d - 0.42 x_{u,max})$

$M.R = T \times Z = (0.87 f_y A_{st})(d - 0.42 x_{u,max})$

For Fe250 $\Rightarrow x_{umax} = 0.53 d$

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

$= 0.36 f_{ck} b (0.53 d)(d - 0.42 \times 0.53 d) = 0.148 f_{ck} b d^2$

$$\boxed{\begin{array}{l} \text{for Fe250} \Rightarrow M_{u\,lim} = 0.148 f_{ck} b d^2 \\ \text{for Fe415} \Rightarrow M_{u\,lim} = 0.138 f_{ck} b d^2 \\ \text{for Fe500} \Rightarrow M_{u\,lim} = 0.133 f_{ck} b d^2 \end{array}}$$

$\rightarrow \because x_{umax} = 0.53 d$

$\rightarrow \because x_{umax} = 0.48 d$

$\rightarrow \because x_{umax} = 0.46 d$

(ii) Under Reinforced section :-

$M.R = M_u = T \times Z$

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

or

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

(iii) Over Reinforced Section :- (NB ⇒ in the analysis of over reinforced section x_{umax} will be used that is (i.e) over reinforced section is limited to balanced section)

$M.R = C \times Z$

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

(2) Percentage of tension steel (P_t) :-

(i) Balanced section :-

$C = T$

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

$\Rightarrow \dfrac{0.36 f_{ck} b x_{umax}}{0.87 f_y} = A_{st}$ $\quad \left(A_{st} = \dfrac{(P_{t\,lim}) bd}{100}\right)$

$= \dfrac{0.36 f_{ck} b x_{umax}}{0.87 f_y} = \dfrac{P_{t\,lim} bd}{100}$

$$\boxed{P_{t\,lim} = \frac{0.36 f_{ck} b x_{umax} \times 100}{0.87 f_y \times bd}}$$

$\Rightarrow (P_t)_{lim} = \frac{0.36}{0.87} \cdot \frac{f_{ck}}{f_y} \cdot \frac{x_{umax}}{d} \times 100$

For Fe250 :- $x_{umax} = 0.53d$.

$(P_t)_{lim} = 21.93 \frac{f_{ck}}{f_y}$

For Fe415 :- $x_{umax} = 0.4d \Rightarrow (P_t)_{lim} = 19.87 \frac{f_{ck}}{f_y}$

For Fe500 $\Rightarrow (P_t)_{lim} = 19.03 \frac{f_{ck}}{f_y}$

(ii) For under Reinforced section :-

$$(P_t) = 50 \left(\frac{1 - \sqrt{1 - \frac{4.6}{f_{ck}} \cdot \frac{M_u}{bd^2}}}{\frac{f_y}{f_{ck}}} \right)$$

MINIMUM & TENSION STEEL :-

$$\boxed{\frac{A_s}{bd} = \frac{0.85}{f_y}}$$ A_s = Area of tension steel.

NO. OF BARS (n) :-

$n = \frac{\text{total area of steel}}{\text{area of one steel rod}} = \frac{A_{st}}{a_{st}}$

Analysish :-

A single reinforced rectangular beam of width 230mm and 460mm . effective depth is reinforced with 3 numbers. 20mm diameter bars, find the factor moment of resistance of this section. The materials are M20 grade concrete and Fe415 steel also find out the factor moment of resistance of it is reinforced with 4 number 20mm diameter bar.

given data :-

S.R.B

width = b = 230mm

effective depth = 460 mm

M20 $\Rightarrow f_{ck}$ = 20MPa

fe 415 ⇒ f_y = 415 mpa

(i) when 3 no. 20mm diameter :-

$A_{st} = 3 \times \frac{\pi}{4} \times 20^2 = 314 \times 3 = 942 mm^2$

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

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

$\Rightarrow x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times 942}{0.36 \times 20 \times 230} = 205.37$

$x_u < x_{u\,max} \Rightarrow$ U.R section

⇒ factored = $C \times Z$
$= 0.36 \times f_{ck} b x_u (d - 0.42 x_u)$
$= 0.36 \times 20 \times 230 \times 205.38 (460 - 0.42 \times 205.3$
$= 127109684.2$ N/mm. $= 127.1$ kN-M

(ii) When 5 nos 20mm dia bar :-

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

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

$C = T = 0.36 f_{ck} b x_u = 0.87 f_y A_{st}$

$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b x_u} = \frac{0.87 \times 415 \times 1570}{0.36 \times 20 \times 230}$

$= 342.29$ MM

⇒ factored MR = $C \times Z$
$= 0.36 \times f_{ck} b x_{u\,max} (d - 0.42 x_{u\,max})$
$= 0.36 \times 20 \times 230 \times 220.8 (460 - 0.42 \times 220.8)$
$= 134288171.8$ N-m
$= 134.288$ kN-M

Design Problem:—
A rectangular cantilever beam of size 230mm width x 500mm eff depth is subjected a bending moment of 80KN/m at working loads. find the steel area required the material M20 concrete & Fe415 reinforcement of grade fe 415.

given data:—

width $b = 230mm$
eff. depth $= d = 500mm$
working moment $= m = 80KN.m$
$M20 = f_{ck} = 20MPa$
$fe415 \Rightarrow f_y = 415 MPa$

factored moment $= M_u = 1.5 \times 80 = 120$ KN/m.

type of section:—

$M_{u,lim} = 0.138 f_{ck} bd^2 = 0.138 \times 20 \times 230 \times 500^2$
$= 158.7$ KN/m

As $M_u < M_{u\,lim} \Rightarrow$ It is a U.R Section

$$P_t = 50\left[\frac{1-\sqrt{1-\frac{4.6}{f_{ck}}\cdot\frac{M_u}{bd^2}}}{(f_y/f_{ck})}\right]$$

$$= 50\left[\frac{1-\sqrt{1-\frac{4.6}{20}\times\frac{120\times10^6}{230\times500^2}}}{\frac{415}{20}}\right]$$

$= 0.67$

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

$$\Rightarrow A_{st} = \frac{0.67 \times 230 \times 500}{100} = 770.5\,mm^2$$

A rectangular cantilever beam of size 230mm width x 500mm eff depth is subjected to a factored ~~working~~ UDL of 15KN/m is applyed throughout on a 4m span of beam find the steel area required the material are m20 concrete Fe415 reinforcement of grade fe 415.

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also find out the No of bars used in beam if 16mm diameter bars used.

given data :-

$b = 200m$

$d = 500mm$

UDL = 15 kN/m

L = 4

$M_{20} = f_{ck} = 20mpa$

$fe_{415} = f_y = 415 mpa$

$d = 16mm$

factored moment = Mu = 1.5 × working load

$$\text{Working load} = \frac{wl^2}{8} = \frac{15 \times 4^2}{8} = 120 \text{ kN.m}$$

$Mu = 1.5 \times 120 = 180 \text{ kN/m} = 180 \times 10^6 \text{ N/m}$

~~Ma~~ Type of section :-

$Mu, lim = 0.138 f_{ck} bd^3 = 0.138 \times 20 \times 200 \times 500^2$

$= 138.0 \text{kN/m}$

As Mu > Mulim = OR section

~~factored~~

factored $= \cancel{C \times Z}$

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

$\cancel{= 0.36 \times 20 \times 200 \times x_u (d}$

$$Pt = 50\left(\frac{1 - \sqrt{1 - \frac{4.6}{f_{ck}} \cdot \frac{Mu}{bd^2}}}{\frac{f_y}{f_{ck}}}\right)$$

$$= 50\left(\frac{1 - \sqrt{1 - \frac{4.6}{20} \times \frac{180 \times 10^6}{200 \times 500^2}}}{\frac{415}{20}}\right)$$

$= 1.41$

~~Assume diameter bars~~

~~$Pt = 100 \frac{Ast}{ast} = Pt$~~ $\qquad 100 \frac{Ast}{bd} = Pt$

~~$100 \frac{Ast}{ast} = 141$~~

$ast = \frac{\pi}{4} \times 12^2 = 113.09$

$100 \times \frac{Ast}{113.09} = .01591$

Ast $100 \times \frac{Ast}{200 \times 500} = 1.41$

$Ast = \frac{1.41 \times 200 \times 500}{100} = 1410$

Assume 12mm dia bar

$ast = \frac{\pi}{4} \times 12^2 = 113.09$

no of bars $\frac{Ast}{ast} = \frac{1410}{113.09} = 12.46$ Nos.

A simply supported of rectangular beam a span of 8 m carries a U.D.L of 230kN/m inclusive of its self wt determin the Reinforcedment for flexure the materials M15 concrete and . bar of Fe415.

given data :-

Design the beam = Determine or determine area of reinforcement.

U.D.L $w = 23$ kN/m

span $l = 8$ m

~~b = 270~~

~~d = 550~~

$M25 = f_{ck} = 25$

$Fe415 = f_y = 415$

Supported rectangular beam max^m Bending moment $= \frac{wL^2}{8}$

$= \frac{23 \times 8^2}{8} = 184$ kN.m

=> factored BM $= 1.5 \times 184 = 276$ kN.m $= Mu$

assume width $b = 250$mm

effective depth $d = 500 = d$

$(Mu)lim = 0.138 f_{ck} bd^2$

$= 0.138 \times 25 \times 250 \times 500^2 = 215625000$ N-mm

$= 215$ kN.m

Mu > Mulim => O.R section

as over reinforced section is avoided because it's given sudden failure of the structure show we have to redesign the beam so assume,

width $b = 270$mm

$d = 550$mm

$Mulim = 0.138 f_{ck} bd^2$

$= 0.138 \times 25 \times 270 \times 550^2$

$= 281778750$ N-mm

= 281 kN-m

=> Mu < Mulim => U.R section

$$P_t = 50\left[\frac{1-\sqrt{1-\frac{4.6}{f_{ck}}\times\frac{M_u}{bd^2}}}{f_y/f_{ck}}\right]$$

$$= 50\left[\frac{1-\sqrt{1-\frac{4.6}{25}\times\frac{276\times10^6}{270\times550^2}}}{\frac{415}{25}}\right]$$

$$= 1.15$$

$$100\frac{A_{st}}{bd} = 1.15$$

$$A_{st} = \frac{1.15\times270\times550}{100} = 1707.75\ mm^2$$

$$\text{no of steel} = \frac{A_{st}}{a_{st}}$$

$$\text{assume } a_{st} = \frac{\pi}{4}\times d^2 = \frac{\pi}{4}\times20^2$$

$$= 314.$$

$$\frac{A_{st}}{a_{st}} = \frac{1707.75}{314} = 5.43 \Rightarrow 6\ nos.$$

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Internal

dt - 17/02/2020

IS codal provision for beam :-

1. Cover :-

> It is defined as minimum distance between surface of concrete & to the outside of reinforcement.

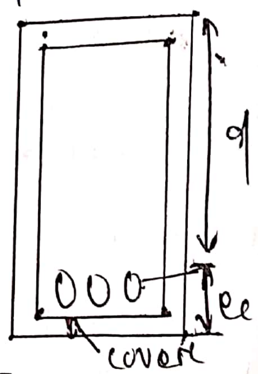
Cover is provided for following reason :-

> To protect the reinforcement from weather & fire so that the it does not corrode or melt.

> To develop the full bond stress between steel & concrete.

Note :-

Minimum cover is provided based on durability criteria.



Clear cover to main reinforcement :-

(1) Slab & stair case = 15mm
(2) Grade slab & flat slab = 20mm
(3) Beam & shear walls = 25mm
(4) column = 40mm
(5) footing = 50mm

Exposure	minimum cover (mm)
mild	20
moderate	30
severe	45
very severe	50
Extreme	75

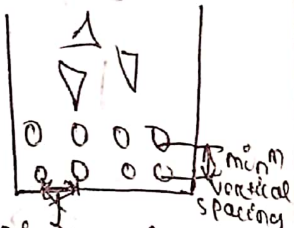
(2) Spacing of bars :-

(1) minimum horizontal spacing :-

(a) dia of bars (If all bars are of same dia)
(b) maximum dia. of bars (If bars are of different dia)

(c) $S_{max} + 5mm$ (S_{max} = max^m size of coarse aggregate)

$\therefore$ Min^m horizontal spacing = max^m of above 3 conditions (a), (b), (c)



(ii) Minimum Vertical Spacing :-

(a) 15mm

(b) $\frac{2}{3}$rd of S_{max}

(c) $\frac{2}{3}$ of ϕ_{max}

S_{max} = max^m size of coarse aggregate

ϕ_{max} = max^m size of bar.

$\therefore$ min^m vertical spacing = max^m of above 3 conditions (a), (b), (c)

(iii) Max^m spacing :-

Max^m spacing $\ngtr$ 300mm

(3) Reinforcement :-

(i) Min^m tension reinforcement :-

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

where,

A = minimum area of tension reinforcement.

b = breadth of beam or the breadth of the web of T-beam

d = effective depth, and

f_y = characteristic strength of reinforcement in N/mm^2

(ii) Max^m area of tension reinforcement :-

$$(A_{st})_{max} \ngtr 0.04bD$$

(iii) min^m compression reinforcement :-

min^m compression = 0.2% of bD

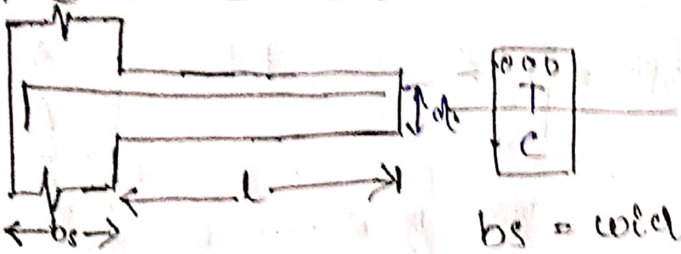
(iv) max^m area of compression reinforcement :-

$$(A_{sc})_{max} \ngtr 0.04bD$$

(v)

Effective span calculation :— (2) :—

(I) cantilever beam :—



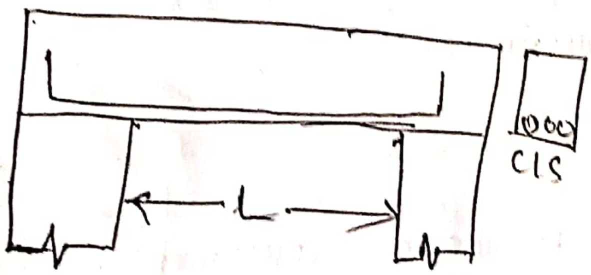
bs = width of support

L = clear span of beam

$l =$ (i) $L + \frac{bs}{2}$

$l =$ (ii) $L + \frac{d}{2}$ } whichever is smaller.

(II) simply supported beam :—



(i) $l = L + bs$

(ii) $l = L + d$ } whichever is smaller

dt - 18/02/2020

(III) for frames : eff. span (l) = centre to centre distance betn support

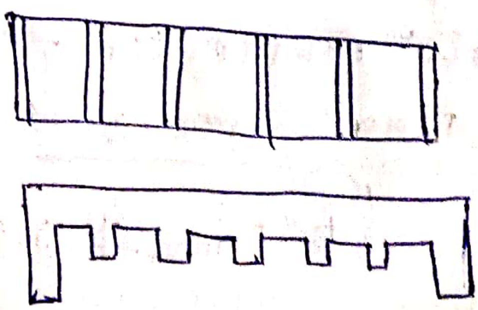
dt - 18/02/2020

ch- Analysis & Design of T-beam :—

advantages of T beam :—

Not monolithical

monolithical



→ Both the beam & slabs are casted monolithically

→ Both of plan beam & slabs are not casted monolithically → There no perfect bond betn beam & slab	→ There is some perfect bond betn beam & slab.

Note:-

The intermediate beams are considered as 'T' beams and the end beams are considered as 'L' beams

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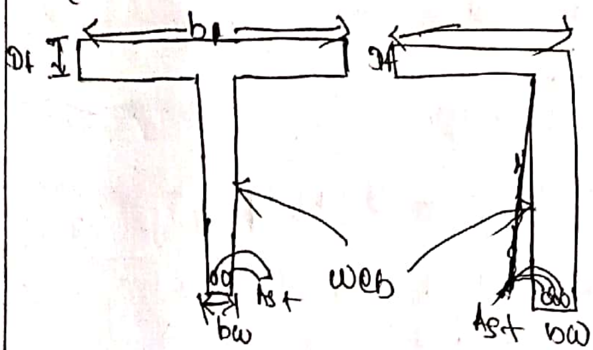
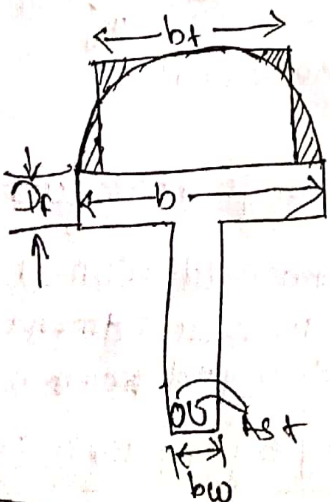


b = actual width of flange
bw = width of web
Df = depth of flange (or) depth of slab.

Ideal stress distribution in flange:-



always $\boxed{b_f \le b}$

Effective width of flange (b_f):- (as per IS: 456-2000) Page-36.
T-beam & L-beam

1) for T-beam ⇒ $b_f = \frac{l_0}{6} + b_w + 6D_f$

(2). For L beam ⇒ $b_f = \frac{l_o}{12} + b_w + 3D_f$

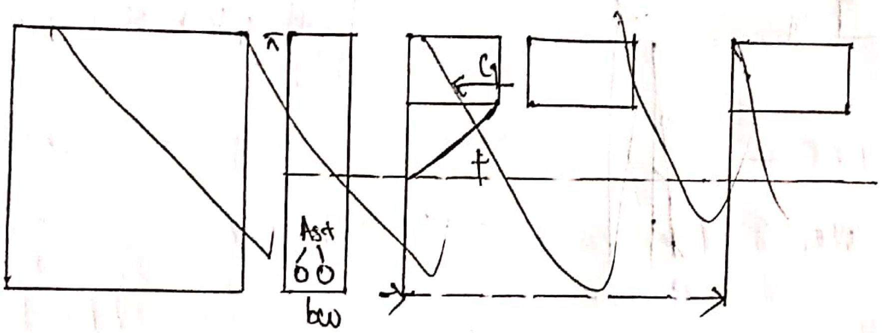
(3) For isolated T-beam ⇒ $b_f = \frac{l_o}{\left(\frac{l_o}{b}\right)+4} + b_w$

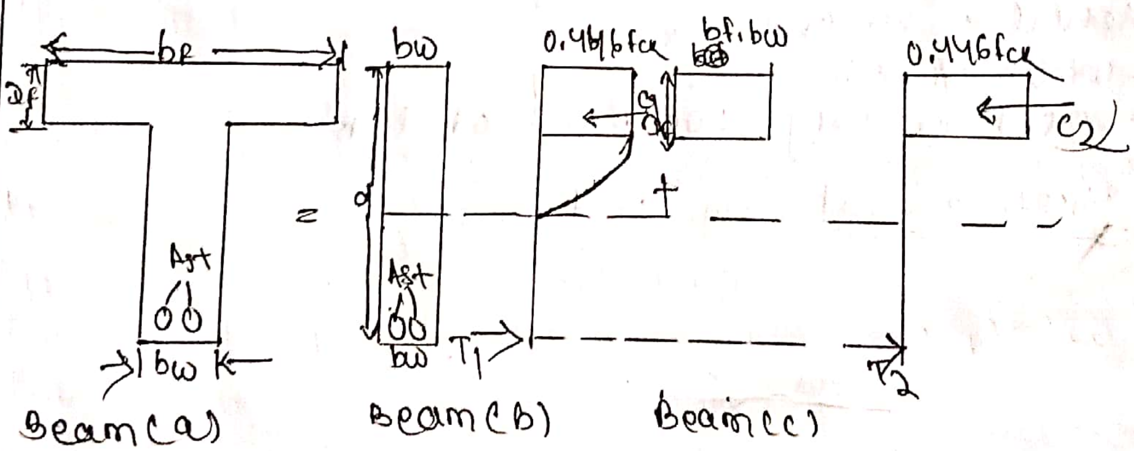
(4) For isolated L-beam ⇒ $b_f = \frac{0.5\, l_o}{\left(\frac{l_o}{b}\right)+4} + b_w$

l_o = distance between point of zero '0' moments (or) point of ~~contraflexure~~ contraflexure

Analysis of T-beam :- or flanged beam

Single reinforced beam





→ Beam(a) = Beam(b) + Beam(c)

→ The analysis & Design of 'T' beam is analogous to Doubly reinforced rectangular beam.

→ In doubly reinforced beam, the additional compressive force is provided by adding reinforcement (Asc) in compression zone; where as in flanged beam.

~~compression~~

compressive force is provided by the slab concrete.

$C_1 = 0.36 f_{ck} b_w x_u$

$T_1 = 0.87 f_y \cdot A_{st1}$

C_2 = Area of stress block

$= 0.446 f_{ck} (b_f - b_w) D_f$

$T = T_1 + T_2$

$T_2 = 0.87 f_y A_{st2}$

$z_1 = d - 0.42 x_u$

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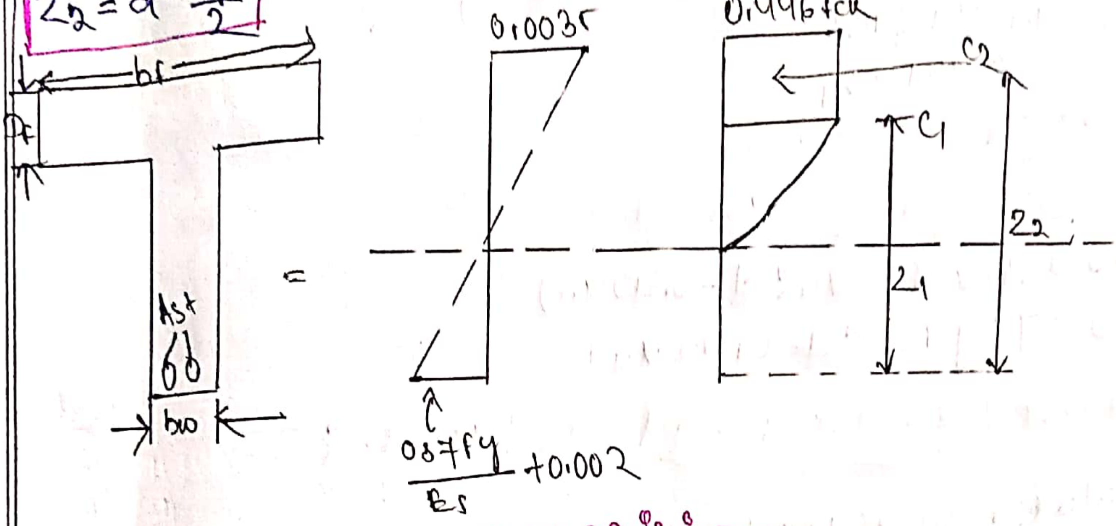
$\boxed{Z_2 = d - \frac{D_f}{2}}$



Maximum depth of Neutral axis :-

$$x_{umax} = \left(\frac{0.0035}{0.0035 + \frac{0.87 f_y}{E_s}}\right) d$$

$x_{umax} = 0.53d / 0.48d / 0.46d$
Fe250 Fe415 Fe500

Position of Neutral Axis :-

for flanged beam neutral axis may either lies
(a) in flang (b) in web

Let us opproximation Neutral axis lies a bottom of flange

Total compression (FTC) = $0.36 f_{ck} b_f D_f$

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

Then If (i) ftc > fts => NA lies in the flang.
(ii) ftc = fts = N.A lies at the bottom
(iii) ftc < fts => N.A lies at the web

(1) Actual depth of neutral Axis :-

Case-1

N.A within (or) upto the flange bottom $(x_u \leq D_f)$

When NA lies in the flange, the size of compression zone becomes $(b_f \times x_u)$. As concrete does not resist any tension, the width of tension zone has no effect on the moment of resistance of the section. there fore this beam can be through out taken as a rectangular beam of dimension $(b_f \times d)$. The formulas derive for rectangular beam shall be applied

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

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

(ii) moment of Resistance :-

for under reinforced section :-

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

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

for balanced & over reinforced section :-

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

$= 0.87 f_y A_{st} (d - 0.42 x_{u\,max})$

Problem :- 1 — dt - 03/03/2020

A T-beam of effective flanged width 1200mm, thickness of slab 100mm, width of web (rib) 300mm and effective depth of 560mm is reinforced with 4 number 25mm diameter bar. calculate ~~materials are M20 concrete. if the material~~ the factor moment of Resistance. if the materials are M20 concrete and HYSD reinforcement of fe415

Ans :-

effective flang width (b_f) = 1200

thickness of slab (or) depth of flange (D_f) = 100

$b_w = 300$

$d = 560$

$A_{st} = 4 \times \frac{\pi}{4} \times 25^2 = 1963.49$

$M20 \Rightarrow f_{ck} = 20\ N/mm^2$

$Fe415 \Rightarrow f_y = 415\ N/mm^2$

Assume N.A lies at the bottom of flange

Total compression = $F_c = 0.36 f_{ck}\, b_f \times D_f$

$\Rightarrow 0.36 \times 20 \times 1200 \times 100 = 864\ kN$.

Total tension $\Rightarrow F_{ts} = 0.87 f_y A_{st}$

$\Rightarrow 0.87 \times 415 \times 1963.49 = 708\ kN$

As $f_{tc} > f_{ts}$ ⇒ N.A lies in the flange.

$C = T$

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

$$= \frac{0.87 \times 415 \times 1963}{0.36 \times 20 \times 1200}$$

$= 82.03$

$x_{u\,max} = 0.48d = 0.48 \times 560 = 268.8$

$x_u < x_{u\,max}$ = under Reinforced section

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

$= 0.36 \times 20 \times 1200 \times 82.03 (560 - 0.42 \times 82.03)$

$= 372476043.8$

$= 372$ kN

Problem:-2

T-beam of effective flanged width 1200mm thickness of slab 100mm width of web (rib) 300mm and effective depth of 560mm is reinforced with 4 number 12mm diameter bar. calculate the factor moment of Resistance. If the materials are M20 concrete and Fe415 reinforcement of fe415.

Ans:-

effective flange width $(b_f) = 1200$

thickness of slab (or) depth of flange $(D_f) = 100$

$b_w = 300$

$d = 560$

$A_{st} = 4 \times \frac{\pi}{4} \times 12^2 = 452$ mm^2

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

$fe415 \Rightarrow f_y = 415$ N/mm^2

Assume N.A lies at the bottom of flange.

Total compression $= f_{tc} = 0.36 f_{ck}\, b_f \times D_f$

$= 0.36 \times 20 \times 1200 \times 100 = 864$ kN

Total tension $= f_{ts} = 0.87 f_y A_{st}$

$= 0.87 \times 415 \times 452 = 163$ kN

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As $f_{tc} > f_{ts}$ => N.A lies in the flange.
$C = T$

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

$$= \frac{0.87 \times 415 \times 452}{0.36 \times 20 \times 1200}$$

$$= 18.88$$

$x_{umax} = 0.48d = 0.48 \times 560 = 268.8$

$x_u < x_{umax}$ => Under Reinforced section.

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

$= 0.36 \times 20 \times 1200 \times 18.88 \times (560 - 0.42 \times 18.88)$

$= 900$ kN.

$\boxed{\text{17 - 04/03/2020}}$

Limit Stress of collapse :- Shear dt -04/03/2021

→ shear stresses are developed either due to the change in B.M (or) due to change in torsional moment.

$$\boxed{\frac{dm}{dx} = f}$$

Nominal shear stress (τ_v) :—

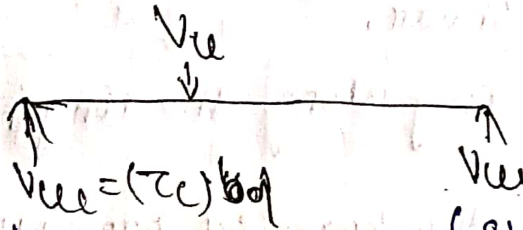
$$\boxed{\tau_v = \frac{V_u}{bd}}$$

V_u = Nominal shear force
b = width of beam
d = eff. depth of beam

→ for prismatic beam ⇒ same material & same c-s area throughout the length of beam.

$$\boxed{\tau_v = \frac{V_u \pm \frac{M_u}{d}\tan\beta}{bd}}$$ → for non-prismatic beam

Total shear strength (or) capacity of beam (V_u) :—



(shear strength of cement concrete alone)

(strength of shear reinforcement)

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

→ shear reinforcement are be design to resiste a shear force V_{us}

forms of shear reinforcement :—

(1) vertical stirrup
(2) inclined stirrup
(3) Bent-up bars along with stirrups

Primary function of stirrup :—

→ To resist a part of shear
→ To resist the growth of inclined cracks and improve aggregate interlocking
→ To tie the longitudinal bars,

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$\boxed{\text{Shear resisted by stirrups} = V_{us} = V_u - \tau_c bd}$

(I) $\boxed{V_{us} = 0.87 f_y A_{sv} \frac{d}{S_v}}$ → for vertical ~~stirr~~ stirrup

(II) $\boxed{V_{us} = 0.87 f_y A_{sv} \cdot \frac{d}{S_v} (\sin\alpha + \cos\alpha)}$ → for ~~vertical~~ inclined stirrup

(III) $\boxed{V_{us} = 0.87 f_y A_{sv} \sin\alpha}$ → for Bent up bar along with vertical stirrup

Function of Bent-up bar :-

→ It is provided to resist the diagonal tension.

→ It is also effective in resisting shear.

Minimum shear reinforcement :-

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

where

A_{sv} = total cross-sectional area of ~~stirr~~ stirrup legs effective in shear,

S_v = stirrup spacing along the length of the member,

b = breadth of the beam or breadth of the web of flanged beam, and

f_y = characteristic strength of the stirrup reinforcement in N/mm² which shall not be taken greater than 415 N/mm²

maximum ~~grade~~ greater of steel of Reinforcement is fe 415 N/mm²

Design of shear reinforcement :-

steps :-

→ calculate the factored S.F = V_u

→ calculate the nominal shear stress = $\tau_v = \frac{V_u}{bd}$

→ calculate the shear resistance of concrete to ~~along~~ - alone = $\tau_c bd$

$\left(\frac{100 A_{st}}{bd}\right)$ Grade of concrete (M_{10} ; M_{20} - - -)

Cond^n (1) => If $\tau_v < \tau_c$ => It is safe.
How ever provide min^m ~~shear reinforcement~~ shear reinforcement.

Cond^n - 2 :-

-> If $\tau_v > \tau_c$ but less than $\tau_{c,max}$ than not safe.
So design of shear Reinforcement is Required for $\boxed{V_{us} = V_u - \tau_c bd}$

STEP - 4 :-

(1) $\boxed{V_{us} = 0.87 f_y \cdot A_{sv} \frac{d}{S_v}}$ -> for vertical stirrup

(2) $\boxed{V_{us} = 0.87 f_y A_{sv} \frac{d}{S_v} (\sin\alpha + \cos\alpha)}$ -> for Inclined stirrups

(3) $\boxed{V_{usb} = 0.87 f_y A_{sv} \sin\alpha}$ -> for Bent up bar along with vertical stirrups.

$\boxed{\alpha = \text{angle of stirrup}}$

(a) If $V_{usb} \ngtr \frac{V_{us}}{2}$, then $V_u - \tau_c bd - V_{usb} = 0.87 f_y A_{sv} \frac{d}{S_v}$

(b) If $V_{usb} > \frac{V_{us}}{2}$, then $\frac{V_{us}}{2} = 0.87 f_y A_{sv} \frac{d}{S_v}$

STEP - 5 :-

If $\tau_v > \tau_{c,max}$ => Then not safe because diagonal compression failure occurs, so we have to redesign the beam.

Check :-

max^m spacing shouldn't be more than

(i) $0.75 d$ -> for vertical stirrups

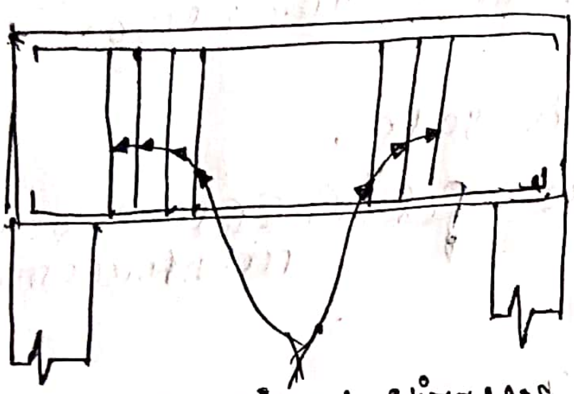
(ii) d -> for inclined stirrups

(iii) minimum S_v

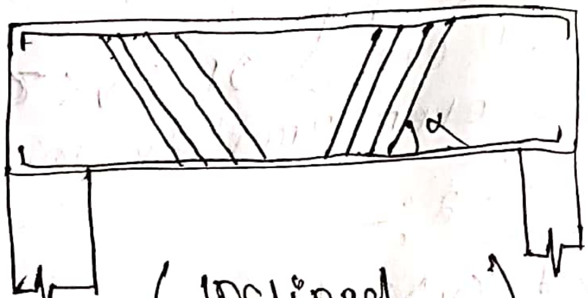
(iv) calculated design S_v

(v) 300mm.

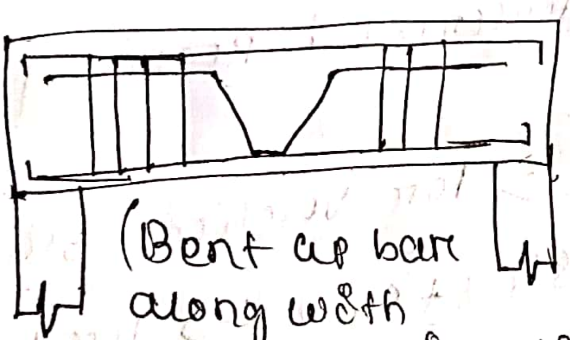
(smaller value will be taken)



Vertical stirrups

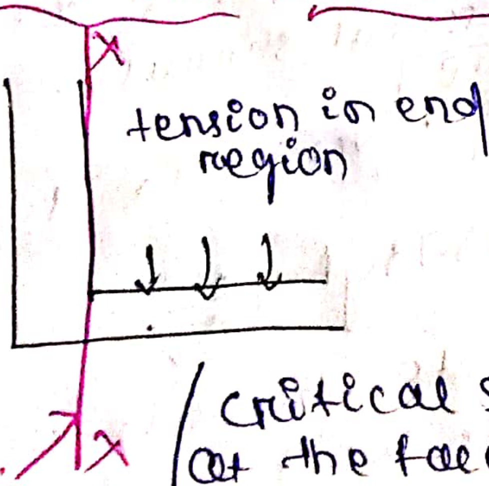


(Inclined stirrups)



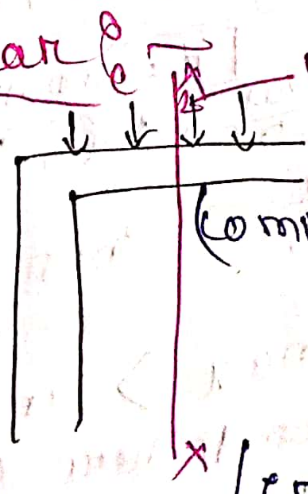
(Bent up bar along with vertical stirrups)

Critical section for shear



critical tension

(critical section at the face of the support)



(compression in end region)

(critical section at d' from the face of the support.)

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BOND AND DEVLOPMENT LENGTH :-

BOND :-

> Bond refers the adhesion between steel & concrete.
> Bond is the primary assumption of limit state method & also for working stress method of design.
> Because plain section remains plain assumption is valid only when there is a perfect bond.
> It is ~~mechanical~~ mechanisim that devlopes between any two materials (steel & concrete) so that stresses are transfered from one ~~to other~~ material to other safely without any sliping.

CLASSIFICATION OF BOND :-

(I) flexural bond or local bond
(II) Anchorage (or) devlopment bond.

flexural bond or local bond :-

This type of bond is arises when the change in bending moment along the length occurs (or) arrise when the change in tensile forced along the length occurs.

(II) Anchorage (or) development bond :-

This bond arrises over a zone of anchorage length which is due to ~~axial~~ axial tensile force in steel bar

$$\text{Devlopment length } (L_d) = \frac{\phi \sigma_{st}}{4\tau_{bd}}$$

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

τ_{bd} = Design bond stress in N/mm^2

grade of concrete	M15	M20	M25	M30	M35	M40 & above
τ_{bd} N/mm^2	1.0	1.2	1.4	1.5	1.7	1.9

→ Applied for mild steel in tension

> for deformed bar, the above τ_{bd} value will be ~~incr~~ increase by 60%.

> for bar in compression, the above τ_{bd} value will be increase ~~by~~ by ~~60%~~ 25%

Limit state meth dt- 06.03.2020

Types of steel	types of zone	τ_{bd}
mild	T	τ_{bd}
mild	C	$1.25\,\tau_{bd}$
HYSD	T	$1.60\,\tau_{bd}$
HYSD	C	$(1.25\times1.6)\,\tau_{bd} = 2\tau_{bd}$

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

$$\Rightarrow \frac{16(0.87\times415)}{4\times1.2\times1.6}$$

$$= 752\text{ mm.}$$

$$L_d = \frac{\phi\sigma_{st}}{4\tau_{bd}}$$

working state method :-

Q) M20 & Fe415, dia of bar = 16mm development length =

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

$$= \frac{16\times230}{4\times1.2\times1.6}$$

$$= 479\text{ mm}$$

Devlopment length :-

It is the minimum length of still bar embedded in concrete so that it will develop full bond stresses at the interface between steel & concrete.

Anchorage length :-

It is the develpement length provided at the end of reinforcing bar.

Note :-

In case of developement length zone the loads are transfered from concrete to steel, but in the zone of ~~chorag~~ anchorage length the loads dissipated from ~~Still~~ steel to concrete

Check for devlopment length :-

$$L_d \not> \frac{M_1}{V} + L_0$$

$$\boxed{L_d \leq \frac{M_1}{V} + L_0}$$ → As per IS:456-2000 → At continous support

$$\boxed{L_d \not> \frac{1.3M_1}{V} + L_0}$$ → for simply supported end.

V = factored shear force.

M_1 = moment of Resistance of steel bar of the section.

L_0 = sum of anchorage beyond the center support.

(i) d } whichever is more
(ii) 12 ϕ

d = effective depth

ϕ = dia of bar

As per IS:456-2000 it is given that the moment of resistance (M_1) can be increased at simply supported when the compressive reaction is confined to tension steel.

Q $b = 250mm$
$d = 400mm$
$M.R = 70 kN.m$
factored shear force $= 120 kN$
$A_{st} = 3$ nos of 12mm ϕ bars
M20 & fe415
Check for development length where continuous support exist (or) check if the beam is safe in bond (or) not, when continuous support exist?

Given data:-

$b = 250mm$
$d = 400mm$
$M.R = 70kN.m$
factored shear force $= 120kN$
$A_{st} = 3 \times \frac{\pi}{4} \times 12^2 = 340 mm^2$
M20 grade of concrete $f_{ck} = 20 N/mm^2$
fe415 grade of steel $f_y = 415 N/mm^2$
for continous support.

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

$$L_d = \frac{\phi \sigma_{st}}{4 \tau_{bd}} = \frac{\phi \times 0.87 f_y}{4 \tau_{bd}} = \frac{12 \times 0.87 \times 415}{4 \times 1.2 \times 1.6}$$

$= 564mm$

$L_0 =$ maximum of $\begin{cases} (i)\ d = 400mm \\ (ii)\ 12\phi = 12 \times 12 = 144mm \end{cases}$

$L_0 = 400mm$

$$\frac{M_1}{V} + L_0 = \frac{70 \times 10^6}{120 \times 10^3} + 400 = 983.33 mm$$

$L_d = 564 mm < \frac{M_1}{V} + L_0$ So safe in bond

Note:-

If the section is not getting satisfy in bond then the economical option is reduce the diameter & increase the no. of bars by keeping same area of steel (∵ contact surface area will be increased by steel with concrete.)

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Anchoring of reinforcing bar :-

→ For plain steel (mild steel bar) at the end, either bend or hook will be provided because it will hold the whole concrete during failure.

→ The bend or hook should be provided only for tension steel because it will yield of concrete.

(i) Anchoring of reinforcing bar in tension :-

As per IS:456-2000 on tension zone, bend or hook shall be provided. Per each 45° bend, there shall be anchorage value of four times the diameter of the bar & for each 45° increase, anchorage value will be increase by 4φ.

Angle of bend	Anchorage value (AV)
45°	4φ
90°	8φ
135°	12φ
180°	16φ

(ii) Anchoring of Reinforcing bar in compression :-

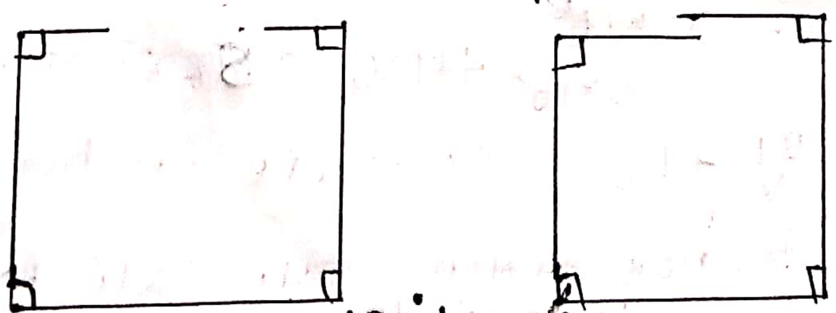
A straight bar is provided in compression zone whose length beyond cut off point shall not be less than development length.

(iii) Anchoring of Reinforcing bar in shear :-

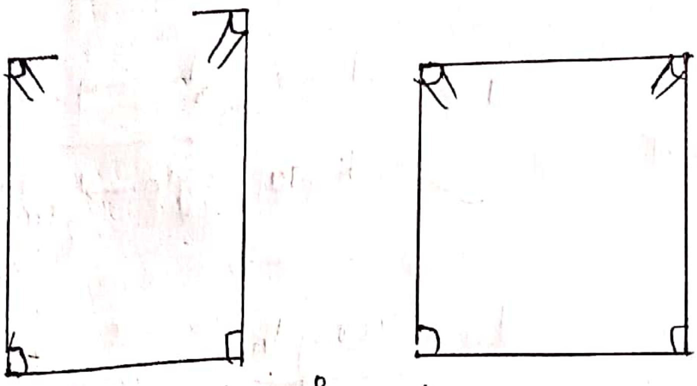
Shear reinforcement in terms of stirrup will be provided.

Angle of bend	Anchorage value
90°	8φ
135°	6φ
180°	4φ

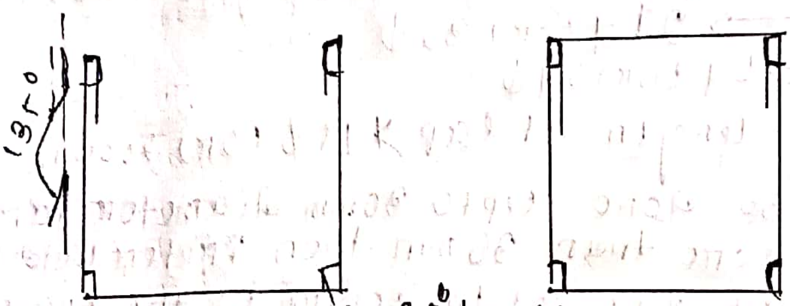
(∵ for every 45° ↑, Anchorage value will ↓ by 2φ)



(90° bend)



(135° bend)



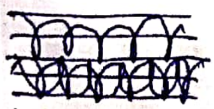
(180° bend)

Bundled bar:-

→ It is defined as tie up to two bars or more than two bars with one over the other through out its length with binding wire.

→ Bar bundling is prefer when (i) The reinforcee is congested

(ii) The sufficient diameter of bar is not available readily in the market.

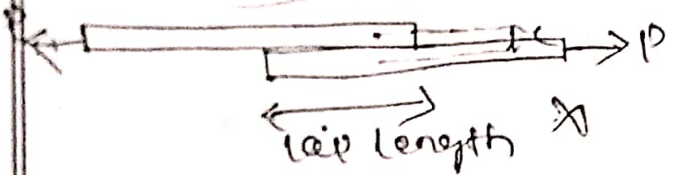
fig	no. of bars in contact	↑(Ld) for each bar
	2	10%
	3	20%
	4	33%

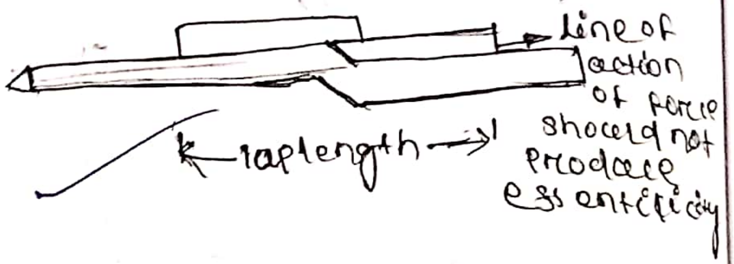
Here Ld is increases because to account for reduction in contact surface area.

Splicing:-

It is a type of joint in which two bars are tie up with wire with certain length.

So that the force in one bar transfer to another bar safely





Type	Lap length
1. flexural tension	→ Ld (or) 30φ which ever is maximum
2. Direct tension	→ 2Ld (or) 30φ
3. compression	→ Ld (or) 24φ

The straigth length OR lap ≯ 15φ (or) 200mm

→ Splicing should be done upto 36mm diameter bar If the diameter is more than 36 mm then prefer welding

→ splicing should not be done at the centre of the beam (at mid span)

→ splicing should not be done when bending moment greater than 50% of moment of Resistance $\left[BM > \frac{MR}{2}\right]$

If BM greater than 50% of moment of Resistance then splicing will not be provided,

SLAB :-

→ Slabs are considered as plane (or) plate elements which are classified on the basis of aspect ratio.

Aspect ratio :-

→ It is defined as the ratio of longer span of slab to shorter span of slab.

⇒ Aspect ratio = $\frac{L_y}{L_x}$ ($L_y \geq L_x$)

Types of slab :-

1. One way slab ⇒ If $\frac{L_y}{L_x} > 2$
2. Two way slab ⇒ If $\frac{L_y}{L_x} \leq 2$
3. Flat slab
4. Flat plate

1. One way slab :-

→ When $\frac{L_y}{L_x} > 2$

Slab, Stair	flexural member
Column, footing	compression member

→ In case of one way slab there is a uniform deflection along the shorter span.

→ In which direction there is uniform deflection in that direction main steel should be provided & perpendicular to that distribution steel should be provided.

→ The function of main steel is resisting flexure.

→ The functions of distributing steel are :

(i) To keep the main reinforcement in position.

(ii) To take care of secondary effect like creep, shrinkage, temperature change.

2. Two way slab :-

→ When $\frac{L_y}{L_x} \ngtr 2$

→ Two way slab must be rest over four support on it's edges.

→ In case of two way slab there is uniform deflection in both the direction, hence main steel should be provided in both the direction.

I.S. codal provision for slab :-

(i) minimum cover = 20mm

(ii) minimum steel :-

0.15% of bD ⇒ mild steel

0.12% of bD ⇒ HYSD bar

bD => Gross area.

(3) maximum steel :-

$\boxed{(A_{st})_{max} \not> 4\% \text{ of } bD}$ (or) $0.04bD$

(1m) (4) maximum dia of bar in slab :-

$\phi_{max} \not> \frac{1}{8th}$ of overall thickness of slab

e.g.-> Thickness of slab = 95 mm what should be the max^m diameter of bar in slab

Ans :- ~~10~~ ~~12~~ 16 20

$\phi_{max} \not> \frac{95}{8} = 11.87 \simeq 12mm$ ✗

$\phi_{max} \not> 11.87mm$

=> $\phi_{max} = 10mm$

Dt - 12.03.2020

5. maximum size of coarse aggregate :-

$(\phi_{max})_{C.A} \not> \frac{1}{4}th$ of thickness of thinner member in RCC work (slab)

Q. Thickness of slab = 95mm what is the size of coarse aggregate ?

a. 20 ; b 24 c. 28 d. 30

$\phi_{max} \not> \frac{1}{4} \times 95 = 23.75 mm$

=> $\phi_{max} = \not> 23.75$
$= 20mm$

(1m) 6(i) maximum spacing for main steel :-

1. 3d
2. 300mm } which ever is smaller.

(ii) maximum spacing for distribution steel :-

1. 5d
2. 450mm } which ever is smaller

7. ~~calculation~~ calculation of spacing :-

$\boxed{S = 1000 \frac{a_{st}}{A_{st}}}$

(8) control of

8. control of deflection :-

(1) for one way slab :- (and also for beam)

Type	Span upto 10m ( )	Span above 10m ( )
cantilever	7	
simply supported	20	$\frac{span}{10} \times 20$
continous	26	$\frac{span}{10} \times 26$

For two way slab :-

Type :	span upto 3.5m, live load upto 3kN/m2 mild steel span (L/D)	(L/D) Fe415 bar span
Simply supported	35	$0.8 \times 35 = 28$
continuous	40	$0.8 \times 40 = 32$

a_{st} = area of one steel bar in mm^2

A_{st} = Required steel area in mm^2 per meter

d = effective depth of slab

To ensure the cracking of slab is excessive so spacing of the steel should be limited

General concept one way slab :-

The reinforcement in the direction of span is known as main reinforcement or moment steel & is placed in the first layer nearest to the extream tension fibre, keeping clear cover as per requirement to get maximum effective depth. the reinforcement perpendicular to the main reinforcement is known as distribution steel which is placed in second layer

function of distribution steel :-

> Distribution steel resist temperature shrinkage stresses.

> It keep the main reinforcement in position & distributes the non uniform loads through out the slab by tranfering to main steel.

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LIMIT STATE OF COLLAPSE :- COMPRESSION

~~Colou~~

Column :-

→ It is defined as the primary structural compression member which can receive the total load from beam and transfer to the footing safely.

Classification of compression member :-

(1) **Based on material :-**

(i) Stanchion ⇒ for steel.
(ii) Post ⇒ for wood
(iii) Pillar ⇒ for masonary (stone (or) brick)
(iv) Column ⇒ R.C.C

(2) **Based on position :-**

(i) Boom ⇒ crane
(ii) Strut ⇒ Inclined truss

(3) **Based on shape :-**

(i) Square column
(ii) Rectangular column
(iii) Circular column
(iv) Polygonal column < Hexagonal, Octagonal.

(4) **Based on Loading :-**

(i) Axially loaded column
(ii) Axially loaded column with uniaxial bending
(iii) Axially loaded column with biaxial bending

IS Codal Provision :-

(1) **maximum cover :-**

min^m cover = 25 mm (If size of column ≤ 200 mm
min^m cover = 40 mm (If size of column > 200 mm)

(2) **Longitudinal reinforcement :-**

(i) **min^m steel :-**

$(A_{st})_{min} = 0.8\%$ of A_g (∵ A_g = gross area of column)

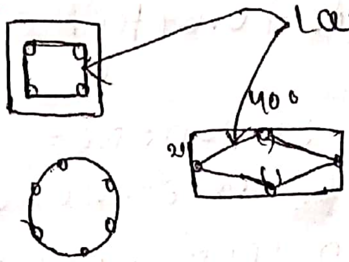
(ii) **max^m steel :-**

$(A_{st})_{max^m} = 6\%$ of A_g (However it is limited to 4% of A_g for practical difficulties)

(iii) min^m diameter :-

$(\phi)_{min}$ = 12mm (to avoid buckling of bar)

(iv) min^m no. of bar :-



(a) for square = 4

(b) for rectangular = 4

(c) for circular = 6

(d) Hexagonal = 6

(e) octagonal = 8

minimum eccentricity (e_{min}) :-

$$e_{min} = \frac{\text{unsupported length}}{500} + \frac{\text{least lateral dimension}}{30}$$

It subjected to 20mm eccentricity

$$e_{min} = \left(\frac{L}{500} + \frac{L.L.D}{30}\right) \text{ or } 20\,mm \text{ which ever is } max^m$$

L = unsupported length.

As per IS code, all the columns are design for min^m eccentricity. because in real practice perfectly axial loaded column is not possible

Q Size of column = (300 × 400) mm

unsupported Length = 3m

(i) $e_{min} = \frac{L}{500} + \frac{L.L.D}{30}$

$$= \frac{3000}{500} + \frac{300}{30} = 16\,mm$$

(ii) 20mm

Axially Loaded column :-

→ If $e_{min} \leq 0.05$ times least lateral dimension then the column will be called as axially loaded column

(If $e_{min} \leq 0.05D$ => short axially loaded column)

D = L.L.D

Load carrying capacity of Axially loaded column :-

(1) column with lateral ties :-

$$\boxed{P_u = [0.4 f_{ck} A_c + 0.67 f_y A_{sc}]} \quad \text{if } e_{min} \nleq 0.05D$$

A_c = net area of concrete = $A_g - A_{sc}$

A_{sc} = Area of longitudinal steel.

(2) column with spiral reinforcement :-

$$P_u = 1.05 [0.4 f_{ck} A_{sc} + 0.67 f_y A_{sc}]$$

where,

P_u = axial load of compression member

Transverse reinforcement :-

lateral ties :-

(1) diameter :-

(i) 6 mm
(ii) $\frac{1}{4} \times \phi_{LL}$

which ever is max^m

where, ϕ_{LL} = largest longitudinal bar diameter.

(2) pitch :- which ever is smaller

(i) L.L.D = Least Lateral diamension
(ii) $16\ \phi_{SL}$ = ϕ_{SL} = Smallest longitudinal bar diameter
(iii) 300 mm

pitch = spacing bet^n then longitudinal bar.

04-13-03-2020

fig	Support condition	effective span	
	Both the ends of a column are restrained against translation and rotation	0.65L	no translation, no rotation, fix
	one end of a column is restrained against translation and rotation and the other end restrained against translation but n't rotation. ex: electric pole with wire.	0.80L	no translation, rotation, hinge
	Both the ends of a column are restrained against translation but n't in the rotation	1.0L	→ Translation in one direction, rotation, roller
	one end of a column is restrained against ~~tra~~ translation & rotation but other end is restrained		

fig	Supported condition	effective span
	against rotation but n't in the translation	1.2L
	one end of a coloumn is restrained against translation & rotation but not the other end.	1.5L
	→ one end of a column is restrained against translation but n't in the rotation and the other end is restrained against rotation but n't in translation	Le = 2L
	one end of a column is restrained against translation and rotation and the other end is free. Ex: only electric pole.	Le = 2L

Assumptions in limit state of collapse: compression :—

→ The axial strain in a column is 0.002
→ The maximum strain at highly compressed edge due to flexure is 0.0035
→ The maximum strain at highly compressed edge due to axial load and bending moment shall be = 0.0035 − 0.75 e_m (when there is no tension)

e_m = strain at least compressed edge.
→ ~~tension~~ tensile strength of concrete is ignored
→ Max^m compressive stress at the extreme compression fibre is 0.446 fck.

dt − 13/03/2020

Analysis problem :—

Q1 A short column (400×400)mm is reinforced with 4 nos 25mm diameter bars. find the ultimate load carrying capacity of the coloumn. If the minimum eccentricity is less than the 0.05 times least lateral dimension, the materials are M_{20} grade concrete and fe415 grade steel

given data :—

Square ~~colum~~ column

L.L.D = D = 400

gross area of concrete = Ag = 400^2 = 160000 mm^2

$ASC = 4 \times \frac{\pi}{4} \times 25^2 = 1963$

$e_{min} \leq 0.05 D$

$M_{20} \Rightarrow f_{ck} = 20$

Fe415 => $f_y = 415$

Soln

Assume lateral ties used in column

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$A_c = A_g - A_{sc}$

$= 160000 - 1963$

$= 158037$

$P_u = 0.4 \times 20 \times 158037 + 0.67 \times 415 \times 1963$

$= 1810108.15$ N

$= 1810.10$ kN

Q:2 A short R.C.C column (450×450)mm is provided with 8 bars of 16mm diameter. If the effective depth of column is 2.5m find the ultimate load for the column. Used M20 and Fe415.

given data :-

Square column size = (450×450)mm

L.L.D = D = 400

$A_{sc} = 8 \times \frac{\pi}{4} \times 16^2 = 1608$

$e_{min} \leq 0.05 D$

M20 = f_{ck} = 20 mpa

$A_g = 450^2 = 202500$

Fe415 = f_y = 415 mpa

Soln

Assume lateral ties used in column.

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$A_c = A_g - A_{sc}$

$= 202500 - 1608$

$= 200892 \text{ mm}^2$

$e_{min} \leq 0.05 D$

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

$= \frac{2.5 \times 1000}{500} + \frac{450}{30} = 20$

(2) 20mm

which ever is maximum

$e_{min} = 20$

$0.05 D = 0.05 \times 450$
$= 22.5$

$e_{min} = 20 < 22.5$ so it is axially loaded column

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$= 0.4 \times 20 \times 200892 + 0.67 \times 415 \times 1608$

$= 2054240.4 \text{ N}$

$= 2054.24 \text{ kN}$.

Design problem :-

Q A short R.C.C column is to carry a factored load of 1900kN. If the column is to be a square, design the column. Assume $e_{min} \leq 0.05 D$. The materials are M20 grade concrete & fe415 steel.

given data :-

factored load $(P_u) = 1900 \text{kN}$

$e_{min} \leq 0.05 D$

$\Rightarrow \frac{e_{min}}{0.05} \leq D$

$\Rightarrow \frac{20}{0.05} \leq D \Rightarrow 400 \leq D$

$\Rightarrow D \geq 400$

$M20 = f_{ck} = 20 \text{mpa}$

$fe415 = f_y = 415 \text{ mpa}$

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Soln

Assume ~~area~~ steel area = 0.8 of Ag

net area of concrete

$\Rightarrow A_{sc} = \frac{0.8}{100} \times A_g = 0.008\,A_g$

$A_c = A_g - A_{sc} = A_g - 0.008\,A_g = 0.992\,A_g$

As $l_{min} \leq 0.05\,D$

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$\Rightarrow 1900 \times 10^3 = (0.4 \times 20 \times 0.992\,A_g) + (0.67 \times 415 \times 0.008\,A_g)$

$\Rightarrow 1900 \times 10^3 = 7.936\,A_g + 2.2244\,A_g$

$\Rightarrow 1900 \times 10^3 = 10.16\,A_g$

$\Rightarrow A_g = \frac{1900 \times 10^3}{10.16} = 187007.874\ mm^2$

As it is square column $\Rightarrow$ side of the column $= \sqrt{A_g}$

$= \sqrt{187007.874}$

$= 432.44\ mm$

let us used 430×430 mm square column

$\Rightarrow A_g = 430^2 = 184900\ mm^2$

$A_c = A_g - A_{sc} = 184900 - A_{sc}$

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$= \cancel{0.4 \times 20 \times}$

$1900 \times 10^3 = 0.4 \times 20\,(184900 - A_{sc}) + 0.67 \times 415\,A_{sc}$

$\Rightarrow 1900 \times 10^3 = 1479200 - 8A_{sc} + 278.05\,A_{sc}$

$\cancel{\Rightarrow 1900 \times 10^3 = 1479192\,A_{sc} + 278.05\,A_{sc}}$

$\cancel{\Rightarrow 1900 \times 10^3 = 1479470.05\,A_{sc}}$

$\cancel{A_{sc} = \frac{1900 \times 10^3}{1479470.05} = 1.284}$

$1900\times10^3 - 1479200 = 8\,Asc - 278.05\,Asc$

$1900\times10^3 - 1479200 = 270.05\,Asc$

$$\frac{1900\times10^3 - 1479200}{270.05} = Asc$$

$\therefore Asc = 1558.29\,mm^2$

Note :-

Spacing between longitudinal bars should not be greater than 300 mm

Use 16mm dia bar as a longitudinal steel.

$\Rightarrow a_{st} = \frac{\pi}{4}\times16^2 = 201\,mm^2$

no of bars $= \frac{Asc}{asc} = \frac{1558}{201} = 7.75 \simeq 8$ nos

Total steel area $= 8\times\frac{\pi}{4}\times16^2 = 1608\,mm^2$

Lateral ties :

(I) diameter

(i) 6mm

(ii) $\frac{1}{4}\times\phi_{LL} = \frac{1}{4}\times16mm = 4\,mm$ } which ever is maxm

$\Rightarrow$ dia of lateral ties = 6mm

(II) pitch

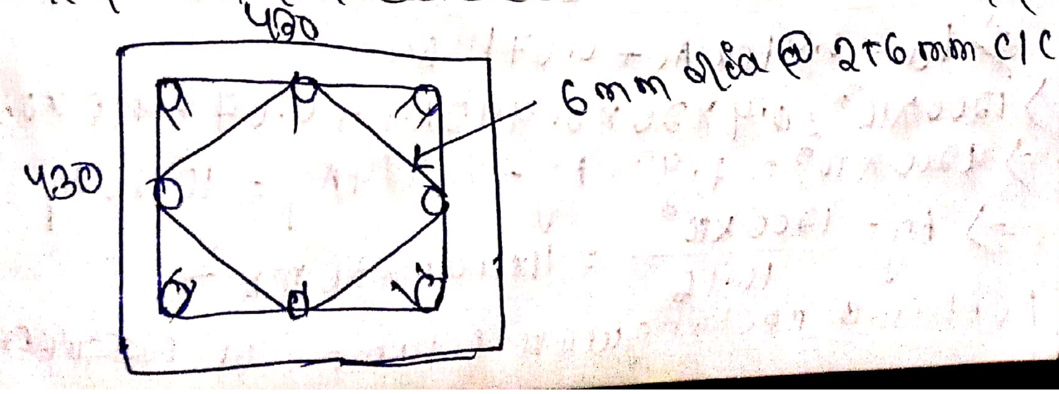
(i) LLD = 430mm

(ii) $16\,\phi_{SL} = 16\times16 = 256mm$ } which ever is smaller

(iii) 300mm

$\therefore$ pitch = 256mm

$\therefore$ Provided 6mm dia lateral ties @ 256mm c/c



STEP for design of RCC column :-

> Assum. 0.8% of Ag => $A_{sc} = 0.008 A_g$ $A_c = 0.992 A_g$

> $P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

=> $(A_g)_{required} = ?$

=> side of column =

> (A_g) provided =

> $P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$; $A_c = A_g - A_{sc}$

=> A_{sc}

=> no of bars = $\frac{A_{sc}}{a_{sc}}$ Lateral ties < (i) dia (ii) pitch

> Detailing.

Q Design a short column of rectangular section with one side as 250 mm to carry an axial load of 800kN. Determine the dimension of other side of the column and reinforcement to be provided. Give sketch of the column section clearly indicating position of longitudinal reinforcement and provision of lateral ties use M_{20} and fe415. Design by lsm.

Given data :-

Axial load = 800 k.N

factored axial load = 1.5 × 800 = 1200 k.N

$e_{min} < 0.05D$. M_{20} => $f_{ck} = 20\ N/mm^2$

fe415 => $f_y = 415\ N/mm^2$

Soln

Assume minimum %ge of steel = 0.8% of Ag

=> $A_{sc} = 0.008 A_g$

=> $A_c = A_g - A_{sc} = A_g - 0.008 A_g = 0.992 A_g$

for provision of lateral ties and as $e_{min} < 0.05D$

=> $P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

=> $1200 \times 10^3 = 0.4 \times 20 \times 0.992 A_g + 0.67 \times 415 \times 0.008 A_g$

=> $1200 \times 10^3 = 7.936 A_g + 2.224 A_g = 10.16 A_g$

=> $A_g = \frac{1200 \times 10^3}{10.16} = 118110.236\ mm^2$

As it is a rectangular column of ~~[illegible]~~ one side = 250mm

$\Rightarrow$ other side $= \dfrac{118110.236\,mm^2}{250\,mm} = 472.44\,mm \approx 480\,mm$

$\Rightarrow$ Gross area of rectangular column $= B \times L$
$= 250 \times 480 = 120{,}000\,mm^2$

$\Rightarrow A_{sc} = 0.008 \times A_g = 0.008 \times 120{,}000 = 960\,mm^2$

As it is rectangular column, $\Rightarrow$ minimum 6 bars shall be used

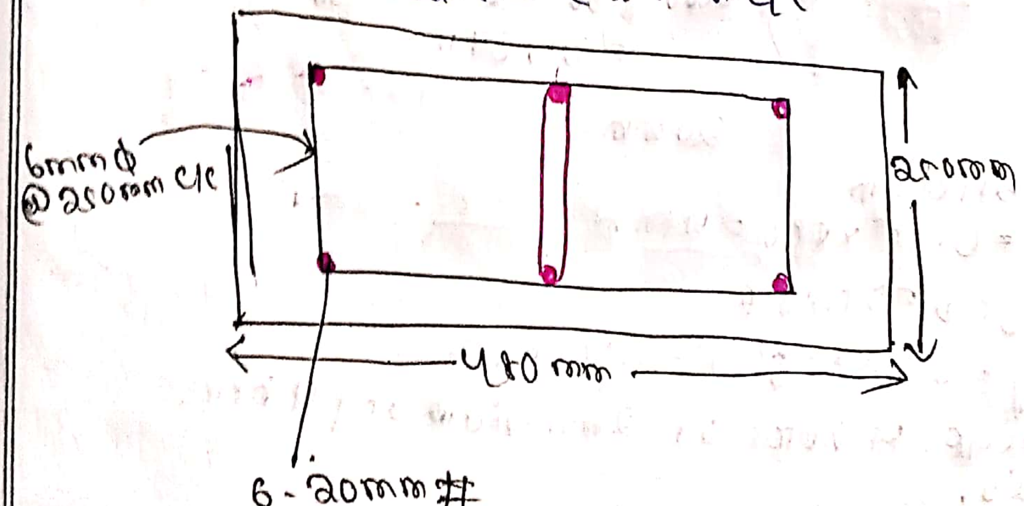
$\therefore$ let us provide 6 nos. 16mm diameter | can give 8 nos 12mm diameter

$\Rightarrow A_{sc} = 6 \times (\frac{\pi}{4} \times 16^2) = 6 \times 201 = 1206\,mm^2 \ngtr 4\%$ of A_g

Now use 6mm ϕ lateral ties. Spacing shall be lesser of (i) 250mm (min^m lateral diameter)
(ii) $16 \times \phi_L = 16 \times 16 = 256\,mm$
(iii) 300mm

i.e spacing = 250mm

$\therefore$ use 6mm ϕ lateral ties @ 250mm c/c



(3) Design a short column of square in section, to carry an axial load of 2000 kN using mild steel and M25 grade of 25 concrete. The column has effective length of 2.5m.

Given data :-

Axial load = 2000 kN

$\Rightarrow$ factored axial load $= 1.5 \times 2000 = 3000\,kN$.

$\Rightarrow$ column is square in section.

$M25 \Rightarrow f_{ck} = 25\,N/mm^2$

$Fe415 \Rightarrow f_y = 415\,N/mm^2$

Sol^n

Let the side of the square column = B

$\Rightarrow$ Gross-sectional area of the column $= A_g = B^2$

let us assume 0.8% of steel ⇒ $A_{sc} = 0.008\,A_g$

⇒ Area of concrete $= A_c = A_g - 0.008A_g = 0.992A_g$

⇒ factored load $= 1.5 \times 2000 = 3000$ kN $= P_u$

As given the column is short and let us assume the minimum eccentricity doesn't exceed $0.05D$ (D = L.L.D)

⇒ $P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

⇒ $3000 \times 10^3 = 0.4 \times 25 \times 0.992A_g + 0.67 \times 415 \times 0.008A_g$

⇒ $3000 \times 10^3 = 9.92A_g + 2.2244A_g$

⇒ $12.144A_g = 3000 \times 10^3$

⇒ $A_g = 247035.573$ mm^2

⇒ $A_g = B^2 \Rightarrow B = \sqrt{A_g} = \sqrt{247035.573} = 497$ mm

So let us provide side of square column $= 500$ mm

Now check for minimum eccentricity.

e_{min} is greater of
$$\begin{cases} \dfrac{L}{500} + \dfrac{D}{30} = \dfrac{2500}{500} + \dfrac{500}{30} \\ = 21.67 \text{ mm} \\ \\ 20 \text{ mm} \end{cases}$$

⇒ $e_{min} = 21.67$ mm

But $0.05D = 0.05 \times 500 = 25$ mm

As $e_{min} < 0.05D = 25$ mm

$A_{sc} = 0.008A_g = 0.008 \times (500)^2 = 2000$ mm^2

let us provide 4 bars of 20mm dia and 4 bars of 16mm dia.

⇒ $A_{sc} = \left(4 \times \frac{\pi}{4} \times 20^2\right) + \left(4 \times \frac{\pi}{4} \times 16^2\right) =$

⇒ $1256 + 804 = 2060$ mm^2

Now use 6mm dia of lateral ties with spacing is lesser of.

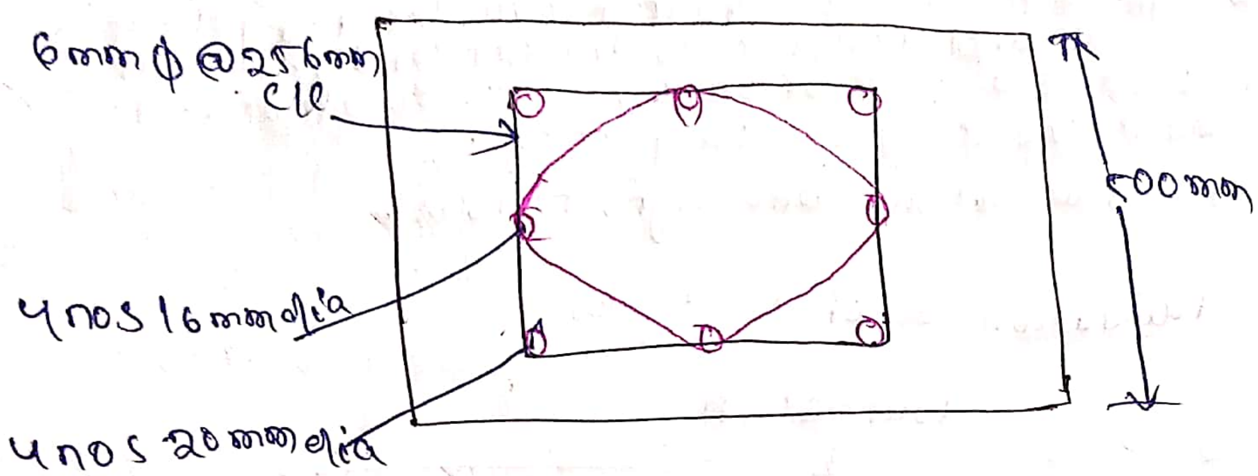
(i) 500mm (least lateral dimension)

(ii) $16 \times \phi_{sl} = 16 \times 16 = 256$ mm

(iii) 300mm

∴ spacing $= 256$ mm

∴ use 6mm ϕ lateral ties @ 256mm c/c


6mm ϕ @ 256mm c/c
4 nos 16mm dia
4 nos 20mm dia
500mm

FOOTING OR FOUNDATION :-

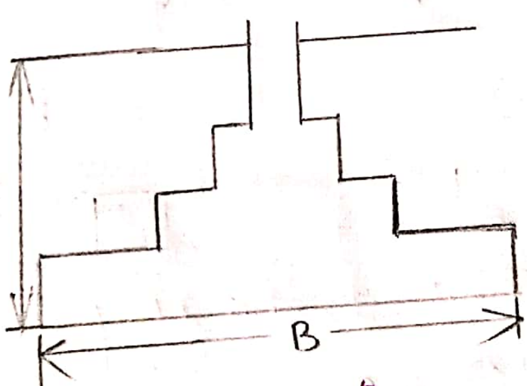
→ It is substructure which can receive the load from super structure and transfer to the soil safely. It is an individual part of a foundation which is constructed exactly under the column and work.

→ Footing and foundations are synonomous.

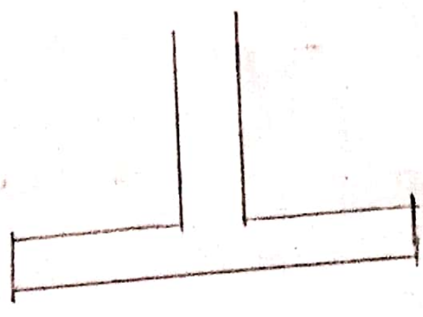
foundation is of two types

- foundation
  - Shallow foundation ($D_f \leq B$)
    - Spread foundation
    - Strap ~~foundation~~ footing
    - combined footing
    - Raft or mat footing
  - Deep foundation ($D_f > B$)
    - pile foundation
    - pier foundation
    - well foundation

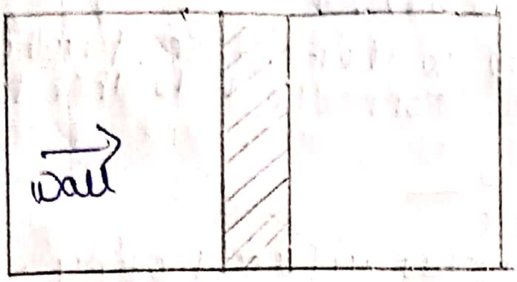
Shallow foundation :-



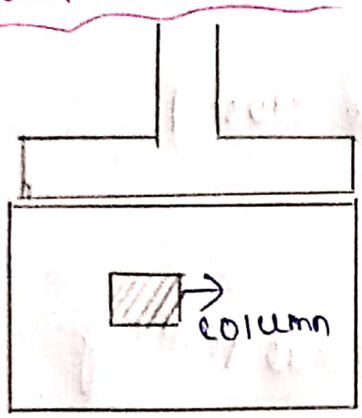
Spread foundation :-

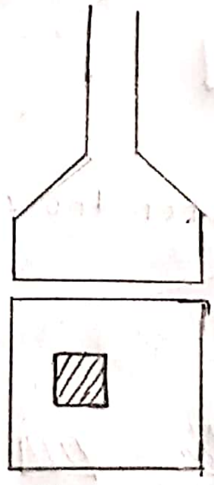


Wall footing :-

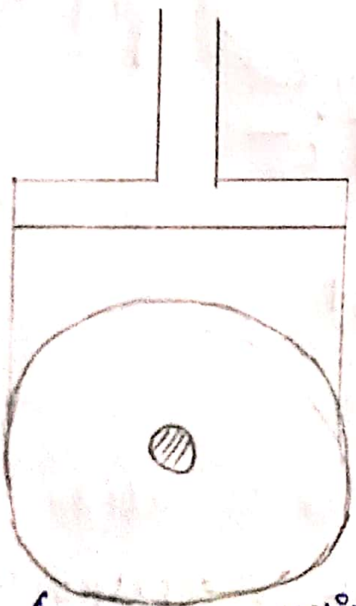


ISOLATED FOOTING :-

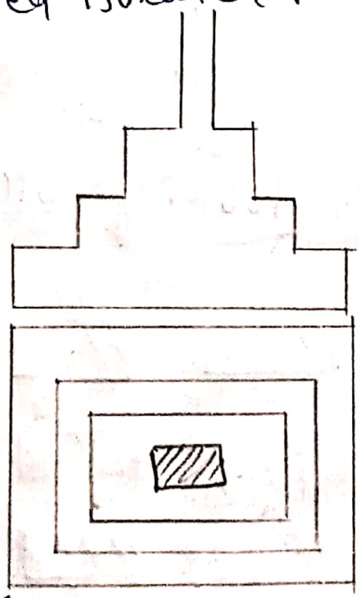




Sloped isolated footing



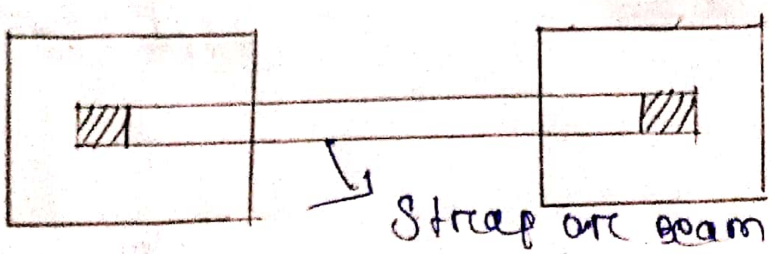
(circular footing)



(stepped footing)

→ The spread footing is used when the soil bearing capacity is strong.

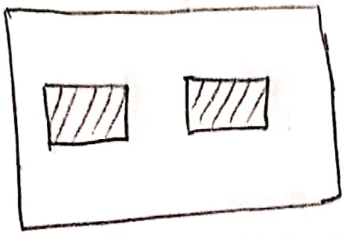
Strap footing :-



(Strap footing)

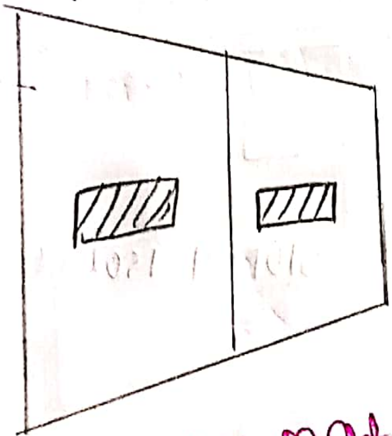
→ The strap footing is used when the soil bearing capacity under one footing is strong and under another footing it is weak.
→ To avoid differential settlement in further due to any reason a Beam is connected to this footing

Combined footing :—



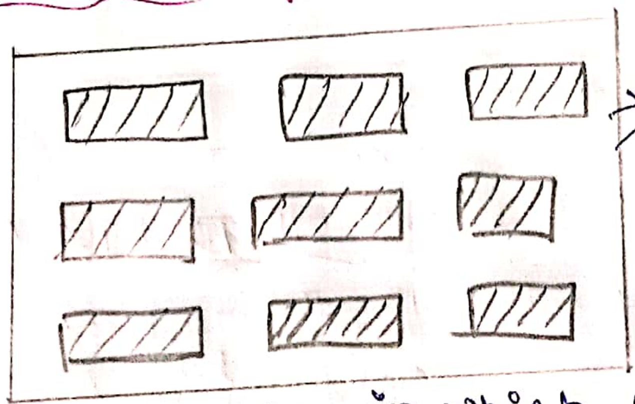
Rectangular footing

It is prefered when the coloumns are closed



Trapezoidal footing

Raft footing or mat footing :—



→ Raft

→ A Raft footing in which a slab is constructed at a shallow depth.
→ It is prefered when the soil Bearing capacity is weak even higher depth.

IS Codal provision for footing :-

(1) minimum cover :-

(50mm for any type of exposure condition)

(2) minimum % of steel :-

0.15% of bD (mild steel)
0.12% of bD (HYSD bar)

(3) minimum Depth of foundation :-

$$D_f = \frac{P_a}{\gamma_{soil}} (K_a)^2$$

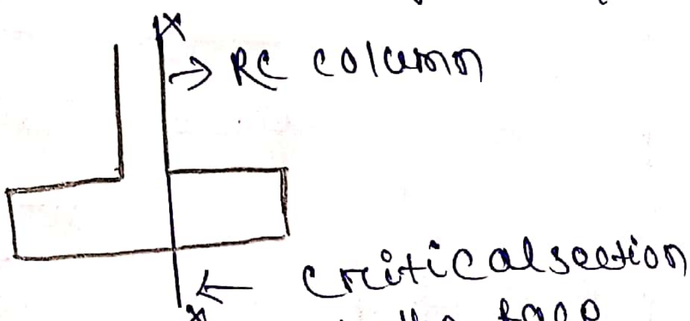
$$\Rightarrow D_f = \frac{p}{\gamma} \left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

p = S.B.C of soil
γ = unit wt. of soil.
ϕ = Angle of internal friction of soil.

(4) Critical section :-

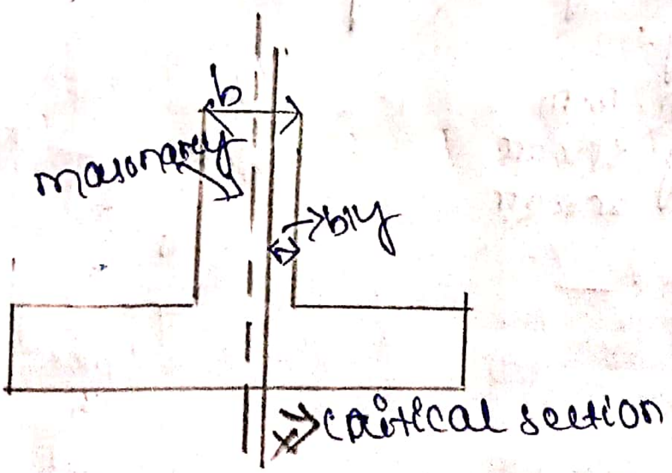
(i) for B.M :-

→ The critical section for B.M occurs at halfway between the masonary of C.C and its edge (i.e $\frac{b}{4}$) in case of masonary.



→ The critical section of B.M occurs at the face of column incase of R.C column.

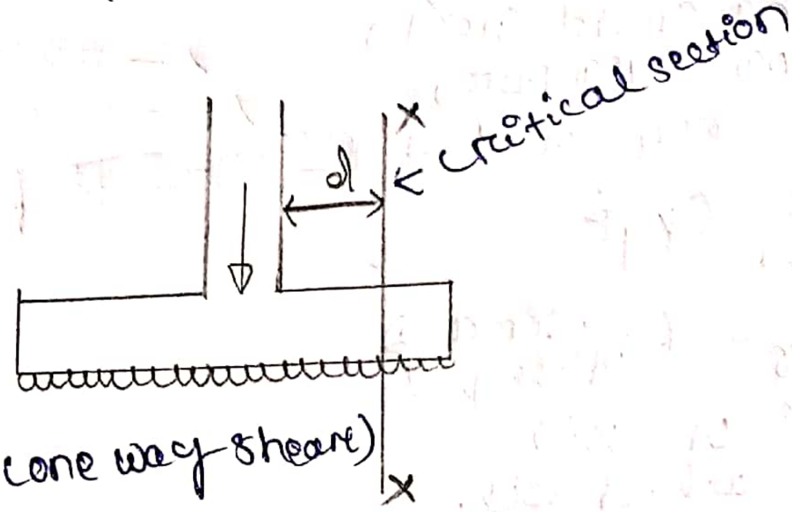
→



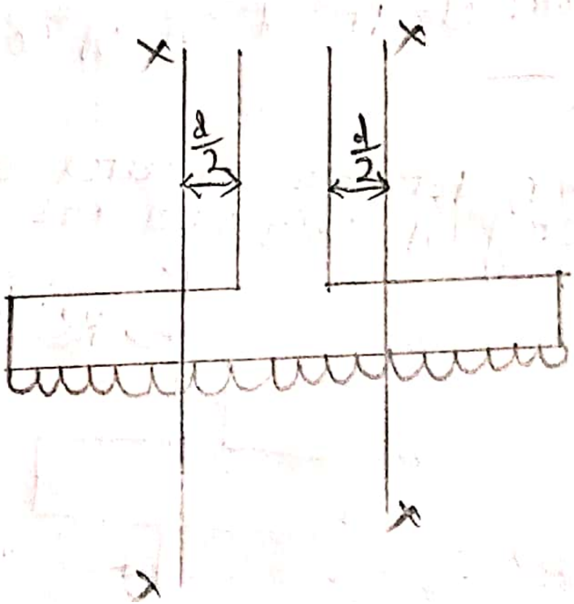
(ii) for Shear :—

→ for one way shear the critical section occurs at a distance d from the face of the column (d = effective depth)

→ The critical section for ~~way~~ Two way shear occure at a distance $\frac{d}{2}$ from the peripheral of the column



(one way shear)



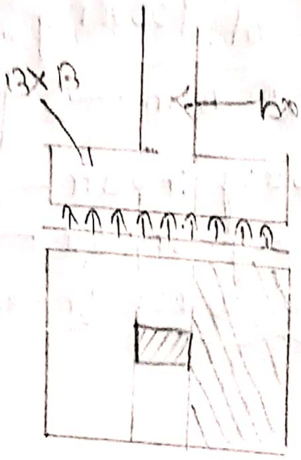
(Two way shear)

(5) Depth of the beam :-

following are the governing factors to decide the depth of footing :

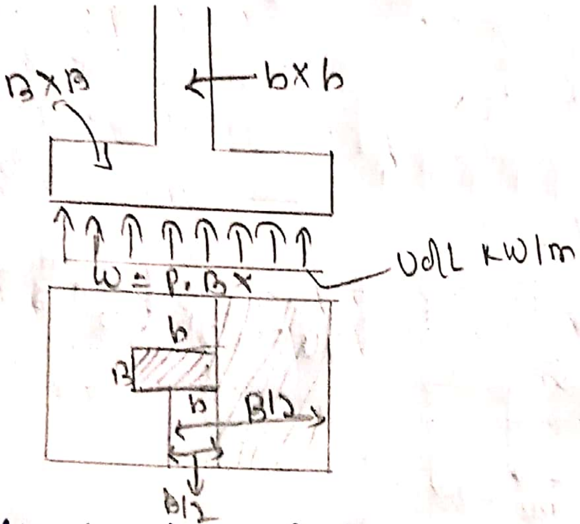
(1) maximum B.M
(2) one way shear
(3) two way shear

(1) maximum B.M :-



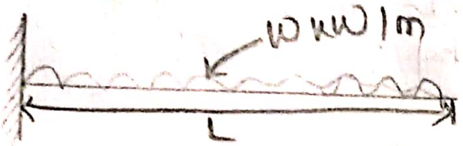
(Q) what is critical section :-

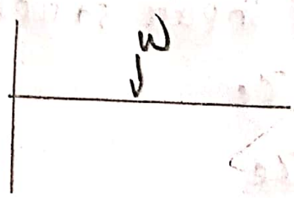
critical sections are those sections in which load carrying capacity (shear and moment) can not be increased beyond the certain limit



UdL load = $P_0 B$

$$M_u = (P_0 \cdot B)\cdot\left(\frac{B}{2}-\frac{b}{2}\right) \times \frac{1}{2}\left(\frac{B}{2}-\frac{b}{2}\right)$$





$$m = (wL) \times \frac{L}{2}$$

where, P_0 = safe bearing capacity or net upward soil pressure.

$$\boxed{M_u = \left(\frac{P_0 B}{8}\right)(B-b)^2}$$

$$\boxed{M_u = M.R = R_u b d^2}$$

R_u = 0.148 fck (fe250)
0.138 fck (fe415)
0.133 fck (fe500)

$$\Rightarrow \boxed{d = \sqrt{\frac{M_u \text{ or } M.R}{R_u b}}}$$

$\boxed{D = d + e_c}$

Overall depth of footing slab

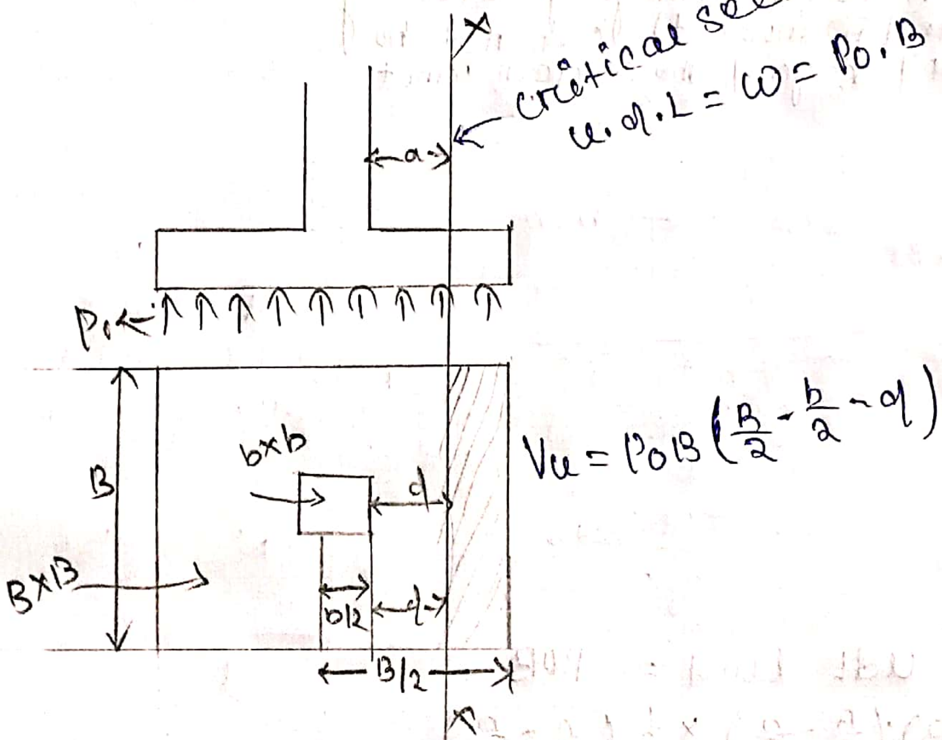
Depth of footing :-

(1) Max^m B.M $\rightarrow$ $\boxed{d = \sqrt{\frac{M_u}{R_u \cdot b}}}$

(2) 1 way shear $\rightarrow$

(3) 2 way shear $\rightarrow$

(4) One way shear :-



$V_u = P_0 B\left(\frac{B}{2} - \frac{b}{2} - d\right)$

nominal shear stress $= \tau_v = \frac{V_u}{Bd}$

$\tau_v \nless K\tau_c$

$\tau_v \leq K\tau_c$

"K" value is given by IS 456 - 1000 72 page

In general $D \geq 300mm$

$\Rightarrow \boxed{K = 1}$ $\boxed{\tau_v \leq \tau_c}$

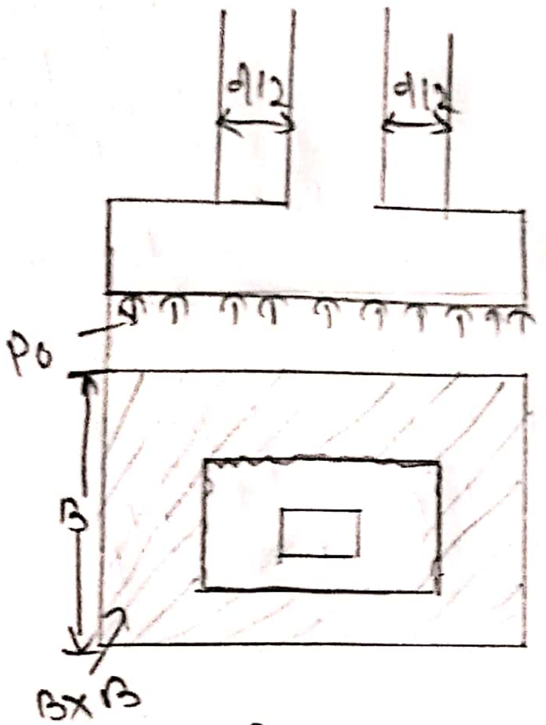
$\tau_v = \tau_c$

$\Rightarrow \frac{V_u}{BD} = \tau_c \rightarrow 100\frac{A_{st}}{BD}$ & grade of con. (table - No 19)

$\Rightarrow \frac{P_0 B\left(\frac{B}{2} - \frac{b}{2} - d\right)}{Bd} = \tau_c$

$\Rightarrow$ For one way shear critical section occurs at a distance 'd' from the face of the column.

(iii) Two way shear or Punching shear :-



→ critical section occurs at a distance '$\frac{d}{2}$' from the periphery of the column.

$$SF = V_u = P_0[(B\times B) - (b+d)(b+d)]$$

Nominal shear stress $\tau_v = \frac{V_u}{4(b+d)d}$

$$\Rightarrow \boxed{\tau_v = \frac{V_u}{b_o d}, \quad b_o = 4(b+d)}$$

$\tau_v \nless K_s \tau_c$

$K = 0.5 + \beta_c$

$$\beta_c = \frac{\text{Short side of coloumn}}{\text{long side of coloumn}}$$

$$\Rightarrow \frac{V_u}{4(b+d)d} = K_s \tau_c$$

$$\Rightarrow \frac{P_0[B^2 - (b+d)^2]}{4(b+d)d} = K_s \tau_c$$