31081 - Structural Engineering & Concrete Structures

UNIT-I

Introduction to working Stress and Limit State Method

Reinforced Cement Concrete:

(i) Concrete ஐ வலுவூட்டுவதற்கு எஃகு கம்பிகள் உபயோகப்படுத்தப்படுகின்றன

(ii) Rcc ஆனது Structural Memberல் ஒரு பகுதியாக அமைகிறது

Materials used in Rcc

(i) Cement → ordinary, Rapid Hardning, white cement

(ii) Aggregates → Coarse, Fine Aggregates

(iii) Steel → Mild, Medium, Hard Rolled Deformed bars

(iv) Water

purpose of providing Reinforcement:

Steel Bars

Concrete ஆனது tensionல் weak ஆக இருப்பதால்

பயன்படுத்தப்படுகின்றன.

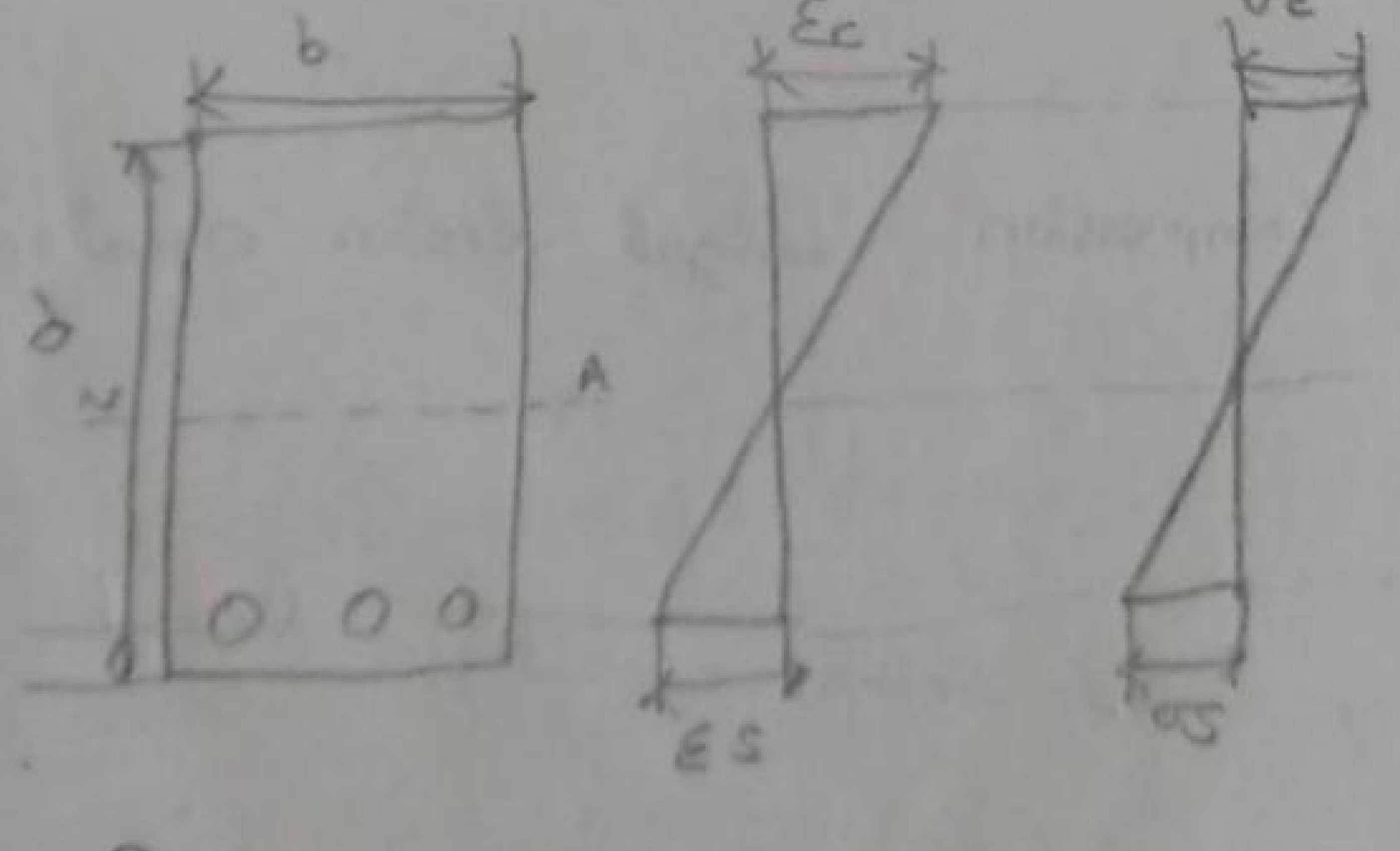
Different types, grades of cement:

(i) 33 grade of cement IS 269

(ii) 43 grade of cement IS 8112

(iii) 53 grade of cement IS 12269

Stress - Strain Distribution Diagram:



Modular Ratio:

It is defined as the Ratio b/w Elasticity of Steel to the Elasticity of Concrete.

$$\text{Modular Ratio } (m) = \frac{E_s}{E_c}$$

Equivalent Area of R.C. Sections:

$$A_c = M \cdot A_s$$

Different types of loads on Structure As per IS : 875 - 1987

(i) Dead Loads

(ii) Live Loads

(iii) Dynamic Effects

(iv) Wind Loads

(v) Impact Loads

Assumptions Made in the Working Stress Method:

(i) நேர் தள குறுக்கு வெட்டு பரப்பு bending க்கு முன்பும் பின்பும் plane ஆக இருக்கும்.

(ii) அனைத்து tensile stress-ம் steel ஆல் மட்டும் எதிர்த்து தாங்கப்படுகிறது.

(iii) Modular Ratio 'm' ன் மதிப்பு $\frac{280}{3\sigma_{cbc}}$

under Reinforced section: $x_u < x_{umax}$

balanced Section : $x_u = x_{umax}$

over Reinforced Section : $x_u > x_{umax}$

Lever Arm:

$$L = jd$$

Lever arm என்பது Compression மற்றும் tension விசைகள் புள்ளிகளுக்கு இடையேயான தூரம்.

Moment of Resistance (M_u):

(i) Moment of Resistance of the Section with Respect to Compressive force

$$M_u = C \times Z = 0.5\,\sigma_{cbc}\, b x\, (d - x/3)$$

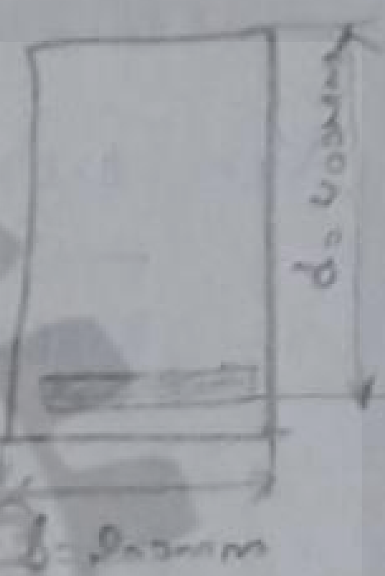
(ii) Moment of Resistance of the Section with Respect to tensile force

$$M_u = T \times Z = \sigma_{st}\, A_{st}\, (d - x/3)$$

problems:

Singly Reinforced Beam:

1. A singly Reinforced beam 200 mm wide and 400mm deep to the centre of tensile Reinforcements. Determine the Limiting Moment of Resistance of the Beam Section and also the Limiting area of Reinforcement. Use M20 Concrete and Fe 250 Steel.



Given:

$b = 200\ MM$

$d = 400\ MM$

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

To find:

(i) $M_{u\,lim}$

(ii) $A_{st\,max}$

Solution:

(i) Ultimate Moment $M_{u\,lim}$:

Formula: $M_{u\,lim} = Q_u \times b \times d^2 = 2.97 \times 200 \times 400^2 = 95.04\ kN.M$

(ii) Limiting Moment for Area of Steel ($A_{st\,max}$):

Formula:

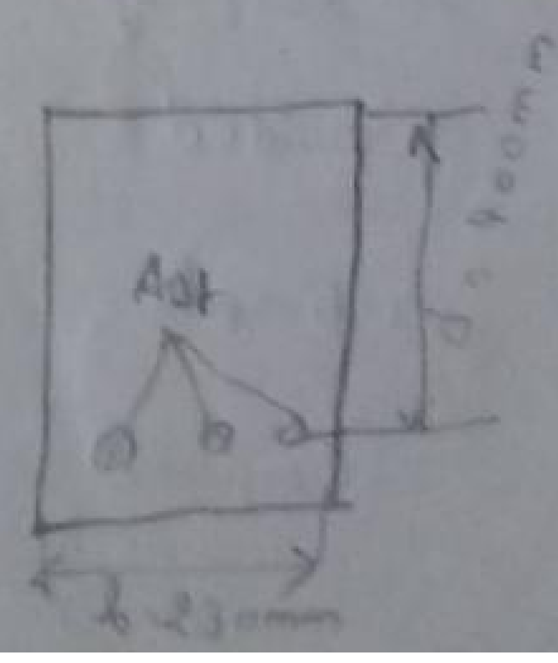
$$A_{st\,max} = \frac{\%\,A_{st}}{100} \times bd = \frac{1.754}{100} \times 200 \times 400 = 1403.2\ mm^2$$

Result:

(i) Ultimate Moment $M_{u\,lim} = 95.04\ kN.M$

(ii) Limiting Area of Steel $A_{st\,max} = 1403.2\ mm^2$

2. A beam 230 mm wide 400mm effective depth is Reinforced with 3 bars of 16mm ϕ Mild Steel in tension. Use M15 grade of Concrete Fe 250 grade of Steel.



given:

$b = 230\ mm$

$d = 400\ mm$

Reinforcement = 3 bars of $16mm\ \phi$

To find:

Moment of Resistance $M_u = ?$

Solution:

Formula:

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

(ii) Type of Section (i) $x_u > x_{umax}$ (ii) $x_u = x_{umax}$ (iii) $x_u < x_{umax}$

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

$A_{st} = \frac{\pi \times d^2}{4} \times$ No. of Bars

$= \frac{\pi \times 16^2}{4} \times 3 = 603.186 \, mm^2$

$x_u = \frac{0.87 \times 250 \times 603.186}{0.36 \times 15 \times 230} = 105.63 \, mm$

$\frac{x_{umax}}{d} = 0.53$, $x_{umax} = 0.53 \times d = 0.53 \times 400 = 212 \, mm$

$x_u < x_{umax}$ $105.63 < 212$

The Section is under Reinforced Section

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

$= 0.87 \times 250 \times 603.186 \times 400 \left(1 - \frac{250 \times 603.186}{15 \times 230 \times 400}\right)$

$M_u = 46.74 \times 10^6 \, N.mm$

Result:

(i) Type of section - under Reinforced Section

(ii) Moment of Resistance - 46.74×10^6 N.MM

Doubly Reinforced Beams:

1. A doubly Reinforced beam section is 250 mm × 500 mm and provided with 2 bars of 12mm φ as Compression steel and 4 bars of 25 mm φ tension steel. These Reinforcements are provided at an effective cover of 40mm. Determine the ultimate Moment of Resistance of the beam Section. Use M20 Concrete and Fe 415 steel.

Moment of Resistance ...

Solution:

Formula:

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

(ii) Type of Section (i) $x_u > x_{umax}$ (ii) $x_u = x_{umax}$ (iii) $x_u < x_{umax}$

$A_{st} = \frac{\pi \times d^2}{4} \times$ No. of bars

$= \frac{\pi \times 16^2}{4} \times 3 = 603.186\ mm^2$

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

$x_u = \frac{0.87 \times 250 \times 603.186}{0.36 \times 15 \times 230} = 105.63\ mm$

$\frac{x_{umax}}{d} = 0.53$, $x_{umax} = 0.53 \times d = 0.53 \times 400 = 212\ mm$

$x_u < x_{umax}$ $\quad 105.63 < 212$

The Section is under Reinforced Section

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

$= 0.87 \times 250 \times 603.186 \times 400 \left(1 - \frac{250 \times 603.186}{15 \times 230 \times 400}\right)$

$M_u = 46.74 \times 10^6\ N.mm$

Result:

(i) Type of Section - under Reinforced Section

(ii) Moment of Resistance - 46.74×10^6 N.mm

Doubly Reinforced Beams:

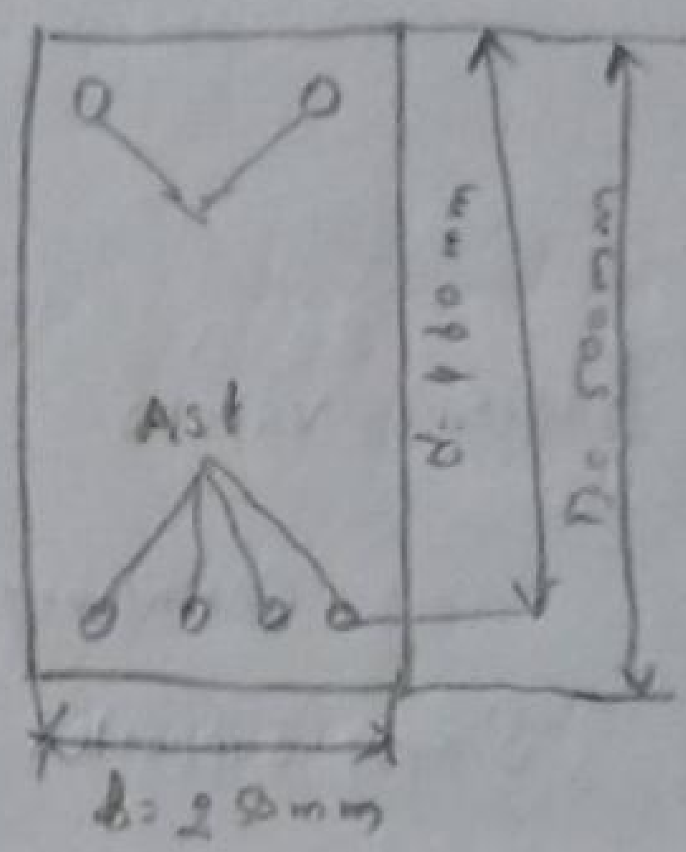
1. A doubly Reinforced beam section is 250 mm × 500 mm and provided with 2 bars of 12mm φ as Compression steel and 4 bars of 25 mm φ tension steel. These Reinforcements are provided at an effective cover of 40mm. Determine the ultimate Moment of Resistance of the beam Section. Use M20 Concrete and Fe 415 steel.

given Data:

$b = 250mm$, $D = 500 mm$

$d' = 40mm$, $d = 460 mm$

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



Reinforcement Compression - 2 Nos of 12mm φ

Tension - 4 Nos of 25mm φ

To find out:

Ultimate Moment of Resistance (M_u)

Solution:

Area of Reinforcement Ast:

Compression ($A_{st\,c}$)

$A_{sc} = \pi/4 \times d^2 \times$ No.of bars

$= \pi/4 \times 12^2 \times 2 = 226.19 mm^2$

Tension Ast:

$A_{st} = \pi/4 \times d^2 \times$ No.of bars

$= \pi/4 \times 25^2 \times 4 = 1963.49 mm^2$

To find fsc:

$f_{sc} = \frac{d'}{d} = \frac{40}{460} = 0.09$, $f_{sc} = 353.4 N/mm^2$

0.05 = 355
0.09 = 353.4
0.10 = 353

$$X_u = \frac{0.87 \times 415 \times 1963.49 - 353.4 \times 226.19}{0.36 \times 20 \times 250} = 349.43 mm$$

$\frac{X_{umax}}{d} = 0.48$, $X_{umax} = 0.48 \times 460 = 220.8 mm^2$

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

$= 0.36 \times 20 \times 250 \times 220.8 (460 - 0.42 \times 220.8) + 353.4 \times 226.19 (460-40)$

$= 179.53 \times 10^6$ N.mm

Result:

$M_u = 179.53 \times 10^6$ N.mm

1.2 Design of Rectangular Beams for Flexure by Limit State Method

Basic values:

Cantilever - 7

Simply supported - 20

Continous - 26

Development length:

கம்பிகளில் தேவைப்படும் axial stress களை எதிர்கொள்ள கம்பிகள் தேவைப்படும் critical புள்ளிக்கு அப்பாலும் concrete க்குள் அதிக நீளத்திற்கு கம்பி பதிக்கப்பட வேண்டும்.

$$L_d = \frac{\phi \, f_s}{4 \tau_{bd}}$$

Anchorage values:

For (i) 45° bend - 4ϕ

(ii) 90° bend - 8ϕ

(iii) 135° bend - 12ϕ

(iv) 180° bend and above - 16ϕ

Problem:

Simply supported beams:

A simply supported rectangular beam of 4m span carries a uniformly distributed characteristic load including self weight of 20 kN/m. The beam section is 230mm x 450mm overall. Design the beam using M20 grade concrete and Fe 415 steel.

Given Data:

$l = 4m$, $w = 20$ kN/m

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

To find out:

Design the beam

Solution:

Material properties

$f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$, $Q_u = 2.76\ bd^2$, $p_t = 0.96\%$

Approximate size of beam:

(i) Effective length $l_e = l + d$

$= 4000 + 415$

$= 4415$ mm

(ii) Effective length $l_e = l + b_w/2 + b_w/2$

$= 4000 + 150 + 150$

$= 4300$ mm

Take least value $l_e = 4300$ mm $= 4.3$ m

Load Calculation:

Total Load $w = 20$ kN/m

Factored Load $w_u = 1.5 \times 20 = 30$ kN/m

Bending Moment Calculation:

$$M_u = \frac{w_u \times l_e^2}{8} = \frac{30 \times 4.3^2}{8} = 69.34 \text{ kN.m}$$

Required Effective depth:

$$M_u = Q_u \times b \times d^2$$

$$69.34 \times 10^6 = 2.76 \times 230 \times d^2$$

$$d = 330.50 \text{mm}$$

$$d_{req} < d_{provi}$$

$330 < 415$ mm

Hence Safe

Area of Steel Ast:

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

Scanned by TapScanner

$$69.34 \times 10^6 = 0.87 \times 415 \times Ast \times 415 \left(1 - \frac{415 \times Ast}{20 \times 230 \times 415}\right)$$

$$69.34 \times 10^6 = 149.63 \times 10^3 \, Ast - 32.57 \, Ast^2$$

$$Ast = 522 \text{ mm}^2$$

Hanger bars:

$$Ast_h = 20\% \text{ of } Ast = \frac{20}{100} \times 522 = 104.4 \text{ mm}^2$$

Negative Reinforcement:

$$Ast_{-ve} = 35\% \text{ of } Ast = \frac{35}{100} \times 522 = 182.70 \text{ mm}^2$$

No of bars:

Main Reinforcement:

$$\text{No. of bars} = \frac{Ast}{ast}$$ (ast = 20mm φ bars assume)

$$= \frac{522}{\pi/4 \times 20^2} = 1.66 \simeq 2 \text{ nos}$$

$$\text{Hanger bars} = \frac{Ast_h}{ast_h}$$ (ast_h = 10mm φ bars assume)

$$= \frac{104.4}{\pi/4 \times 10^2}$$

$$= 1.33 \simeq 2 \text{ bars}$$

Negative Reinforcement:

$$\text{No. of bars} = \frac{Ast_{-ve}}{ast_{-ve}}$$ (ast_{-ve} = 12mm φ bars assume)

$= 182.79$

$\frac{}{\pi/4 \times 12^2}$

$= 1.62 \approx 2$ bars

Curtailment:

50% of bars are curtailed at 0.08l from the face of support

1 bar is curtailed at 844mm from the face of the support.

Negative Reinforcement bars are provided at 1075mm from the face of the support.

Result:

Cantilever beams:

A cantilever beam of breadth 255mm and depth 430mm is Reinforced with 793mm², steel fe500 in tension side. Assuming M20 grade of concrete, find the factored udl/m Run that can be permitted. The span of the beam is 3m

given data:

cantilever beam

breadth (b) = 255 mm

d = 430mm

Reinforcement Ast = 793 mm²

fy = 500 N/mm²

fck = 20 N/mm²

Span $l = 3m = 3000$ mm

To find out:

Factored udl permetre (wu)

Solution:

Moment of Resistance $Mu = 0.87 \times fy \times Ast \times d \left[1 - \frac{fy \times Ast}{fck\, bd}\right]$

$= 0.87 \times 500 \times 793 \times 430 \left[1 - \frac{500 \times 793}{20 \times 255 \times 430}\right]$

$= 121.51 \times 10^6$ N.mm

factored (udl):

$Mu = \frac{wu \times le^2}{2} = 121.51 \times 10^6 = \frac{wu \times 3000^2}{2}$

$wu = \frac{121.51 \times 10^6 \times 2}{300^2} = 27$ N/mm

Result:

factored udl $wu = 27$ N/mm (or) 27 kN/m

A cantilever beam is subjected to a factored bending moment of 90kN.m The breadth of the beam is 250mm. Determine the effective depth and the area of tension steel Required for the beam. if M15 grade concrete and grade steel fe415 used.

given:

Factored Bending moment $Mu = 90$ kN.m

Breadth (b) = 250mm

$fck = 15$ N/mm²

$fy = 415$ N/mm²

To find:

(i) effective depth (d)

(ii) Area of Steel (Ast)

Solution:

effective depth (d):

$Mu = Mulim$

$Mulim = Q_u \times b \, d^2$

$$90 \times 10^6 = 2.07 \times 250 \times d^2$$

$$d^2 = \frac{90 \times 10^6}{2.07 \times 250}$$

$$d = \sqrt{\frac{90 \times 10^6}{2.07 \times 250}}$$

$$d = 420 \text{ mm}$$

Area of steel:

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

$$90 \times 10^6 = 0.87 \times 415 \times A_{st} \times 420 \left[1 - \frac{415 \times A_{st}}{15 \times 250 \times 420}\right]$$

$$90 \times 10^6 = 151.64 \times 10^3 A_{st} - 39.95 A_{st}^2$$

$$A_{st} = 736.36 \text{ mm}^2$$

Result:

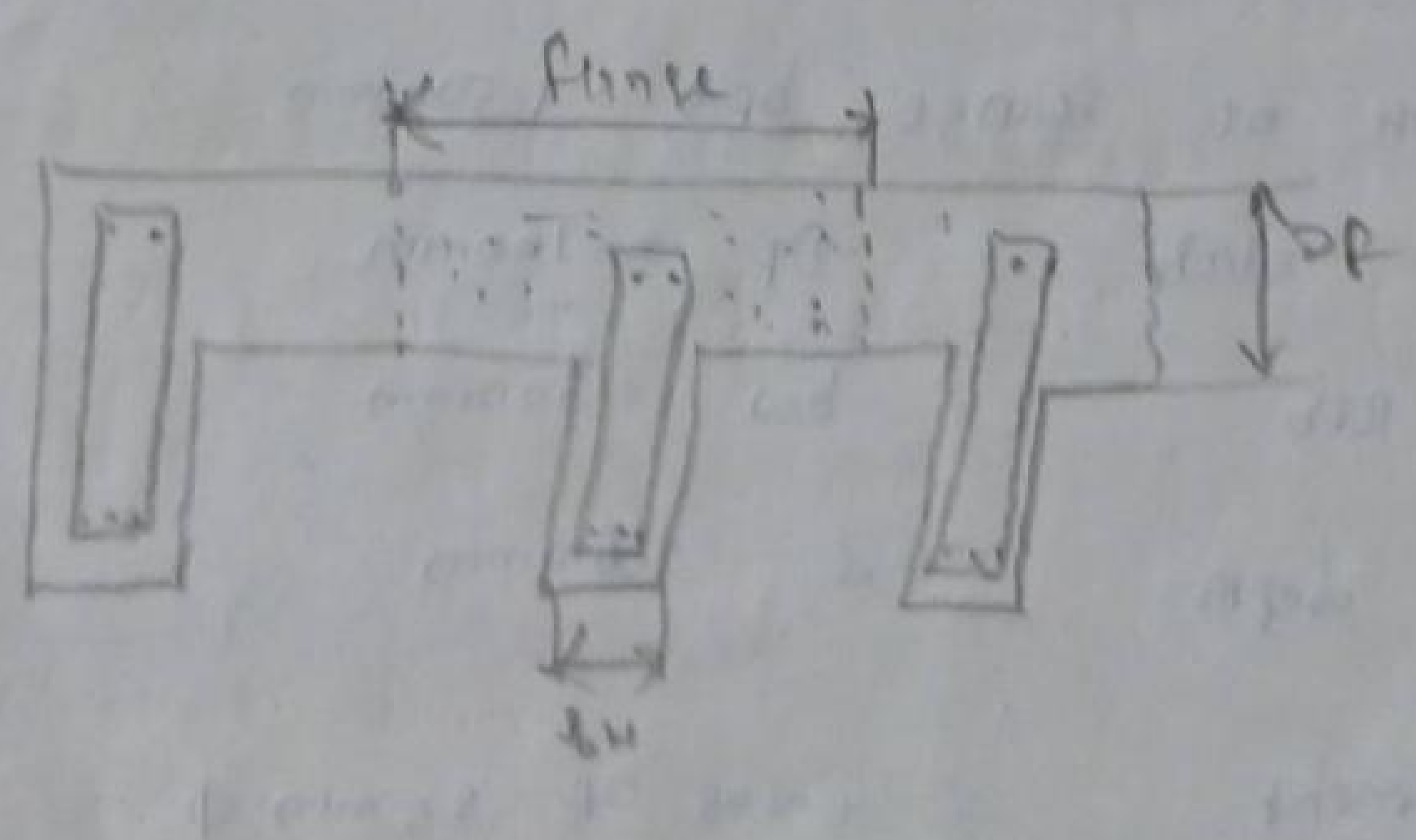
effective depth $(d) = 420$ mm

Area of Reinforcement $(A_{st}) = 736.36 \text{ mm}^2$

UNIT-II

2.1 Design of T Beams and Lintels for Flexure By Limit State Method

Cross Section of Tee beam:



T-beam:

Floor slab மற்றும் Beam ஐ Rcc கொண்டு ஒரு Mono lithic கொண்டு அமைப்பது.

Effective width of flange:

(i) for T-beams $b_f = \frac{l_o}{6} + b_w + 6D_f$

(ii) for L beams $b_f = \frac{l_o}{12} + b_w + 3D_f$

Neutral axis:

ஒரு beam (or) Shape-ன் ஒருபகுதி அழுத்த பகுதியாக அமையும் அதை neutral axis என்பர்.

Moment of Resistance:

Compressive Stress Diagram ன் மையத்தில் செயல்படும் total Compressive force மற்றும் Reinforcement ன் மையத்தில் செயல்படும் Total tensile force இவற்றால் உருவாக்கப்படும் Moment, Moment of Resistance எனப்படும்.

Design of Singly Reinforced T-beams

A T beam of effective flange width 1500mm, thickness of slab 100mm width of Rib 300mm and effective depth 560mm is reinforced with 4 nos of 25mm ϕ bars. Calculate the moment of Resistance. M20 grade concrete and Fe 415 grade steel are used.

Given Data:

Effective width of flange $b_f = 1500mm$

Thickness of slab $D_f = 100mm$

width of Rib $b_w = 300mm$

Effective depth $d = 560mm$

Reinforcement = 4 nos of 25mm ϕ

$A_{st} = \text{No. of bars} \times \frac{\pi d^2}{4}$

$= \frac{\pi \times 25^2}{4} \times 4 = 1963.50 \text{ mm}^2$

To find:

moment of Resistance

Solution:

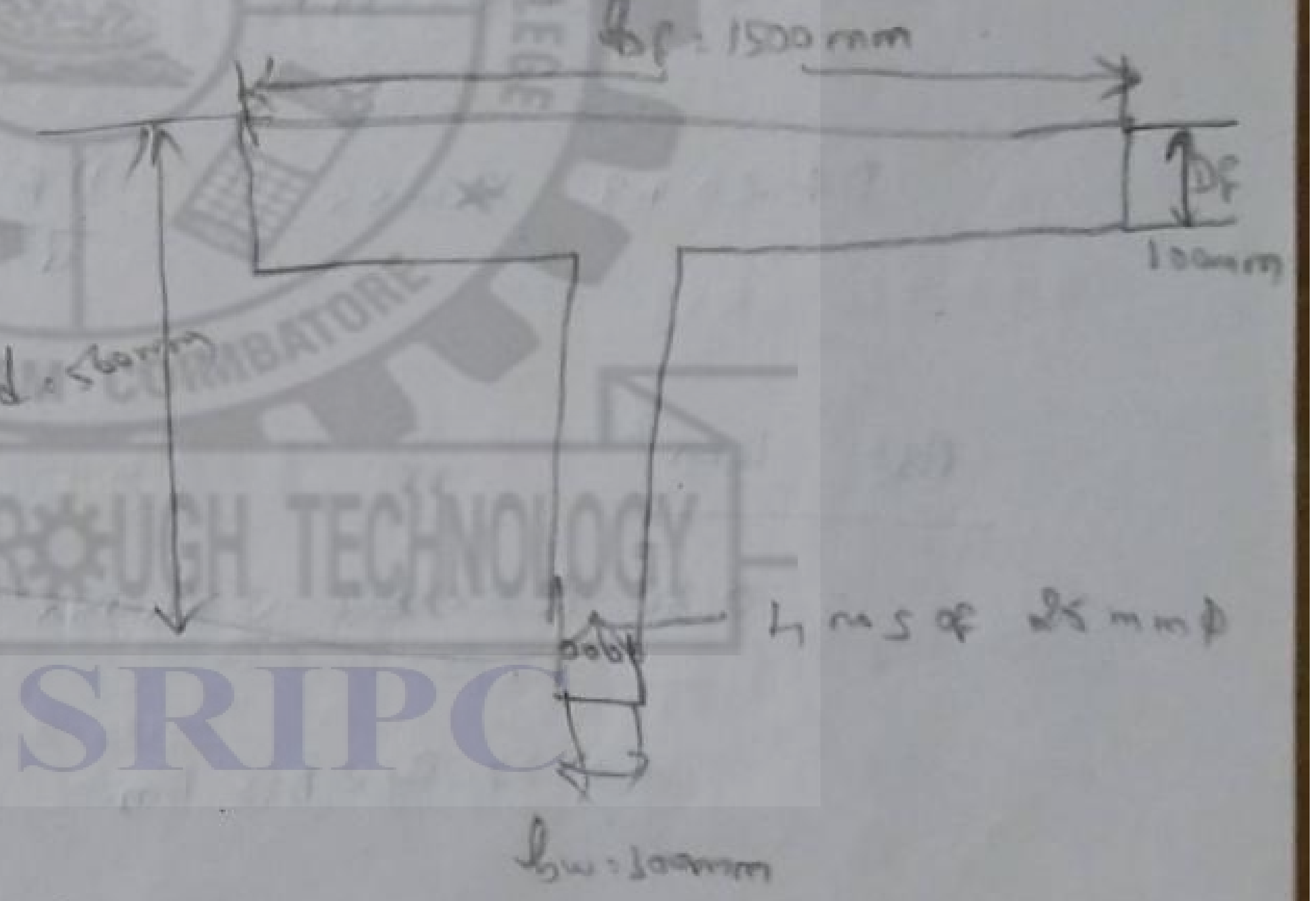
material properties. $f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$, $X_{umax} = 0.48d$

Compute the neutral axis!

$$X_u = \frac{0.87 \times f_y \times A_{st}}{0.36 \times f_{ck} \times b_f} = \frac{0.87 \times 415 \times 1963.50}{0.36 \times 20 \times 1500} = 65.64 mm$$

$X_u = 65.64 mm < D_f$

The section is under Reinforced section.



$X_{umax} = 0.48 d = 0.48 \times 560 = 265 mm$

moment of Resistance $M_u = 0.87 \times f_y \times A_{st} \left[d - \frac{f_y \times A_{st}}{f_{ck} \times b_f} \right]$

$= 0.87 \times 415 \times 1963.50 \left[560 - \frac{415 \times 1963.50}{20 \times 1500} \right]$

$= 377.74 \times 10^6$ N.MM

Result!

moment of Resistance $M_u = 377.74 \times 10^6$ N.MM

Find the area of steel Reinforcement Required and the limiting moment of Resistance for a T-beam with the following Dimensions. flange width = 1800mm, flange thickness = 90 mm, breadth of web is 300mm effective

Scanned by TapScanner

depth 600mm fck=20N/mm², fy=415 N/mm² It has to be designed as a balanced section.

Given Data:

b_f = 1500mm, D_f = 90mm, b_w = 300mm, d = 500mm

Solution:

material properties fck = 20N/mm², fy = 415 N/mm², X_{umax} = 0.48 d

Area of Steel,

X_{umax} = 0.48 d = 0.48 × 500 = 240mm

X_{umax} = 0.43 × 240 = 103.20mm

D_f < 0.43 X_{umax}, $\frac{D_f}{d} = \frac{90}{500}$ = 0.18 < 0.20

Ast lim:

$$= \frac{0.36\ f_{ck} \times b_w \times X_{umax} + 0.446\ f_{ck}\ (b_f - b_w) \times D_f}{0.87\ f_y \times Ast\ lim}$$

$$= \frac{0.36 \times 20 \times 300 \times 240 + (0.446 \times 20)\ (1500 - 300) \times 90}{0.87 \times 415}$$

Ast lim = 4104 mm²

$M_{u\,lim}$ = 0.36 fck × bw × X_{umax} (d − 0.42 X_{umax})

+ 0.446 fck (b_f − b_w) × D_f (d − D_f/2)

= 0.36 × 20 × 300 × 240 (500 − 0.42 × 240) + 0.446 × 20(1500−300)

= 645.27 kN.m

Result:

(i) Ast lim = 4104 mm²

(ii) $M_{u\,lim}$ = 645.27 kN.m

Design of lintel:

Design a Rc lintel for a door opening of 1.2m clear width using M20 grade concrete and use Fe 415 grade steel. The height of wall above the opening is 1.80m and the thickness of wall is 300 mm. Assume the bearing of lintel as 200mm on either side.

Given Data:

Door $= 1.20$ m width, Height of wall above lintel $h = 1.80$ m

Thickness of wall $= 300$mm, Bearing of lintel $= 200$mm on either side

To find:

Rc lintel

Solution:

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

Effective span (l):

Assume the width of lintel equal to width of wall $= 300$mm

Assume the thickness of lintel equal width of wall $= 150$ mm

Assume 8mm ϕ rods and 20mm clear cover

Effective depth of lintel $(d) = 150 - 20 - 8/2 = 126$mm

(i) $l_e = C/c$ of support$(l) + d$

$= 1200 + 200 = 1400$ mm

(ii) $1200 + 126 = 1326$ mm

Take least value $l_e = 1.326$m

Height of equilateral triangle:

$= 0.866 l = 0.866 \times 1.326 = 1.15$m

Height of wall above lintel $= 1.80 m > 1.15 m$

Design BM:

(i) Wt of masonry in Δ portion $W_1 = \frac{1}{2} \times 1.326 \times 1.15 \times 0.3 \times 19$

$W_1 = 4.35\ kN$

(ii) Self wt of lintel $W_2 = 1.326 \times 0.30 \times 0.15 \times 25$

$= 1.49\ kN$

Max. BM at mid span $= \frac{W_1 \times l}{6} + \frac{W_2 \times l}{8}$

$= \frac{4.35 \times 1.326}{6} + \frac{1.49 \times 1.326}{8}$

$= 1.208\ kN.m$

Design BM $= 1.5 \times 1.208 = 1.812\ kN.m = 1.812 \times 10^6\ N.mm$

Area of steel:

$M_u = 0.87 \times f_y \times A_{st} \left[d - \frac{f_y \times A_{st}}{f_{ck} \times b} \right]$

$1.812 \times 10^6 = 0.87 \times 415 \times A_{st} \left[126 - \frac{415 \times A_{st}}{20 \times 300} \right]$

$1.812 \times 10^6 = 45492.3\ A_{st} - 24.97\ A_{st}^2$

$= 24.97\ A_{st}^2 - 45.492.34\ A_{st} + 1.812 \times 10^6 = 0$

$A_{st} = 40.74\ mm^2$

provide 2 nos of 8mm φ bars at bottom as tension Reinforcement 2 nos of 6mm φ bars at top bat hanger bars.

Result:

Size of lintel 2300 mm × 150mm

Reinforcement = provide 2 nos of 8mm φ at bottom as tension

= 2 nos of 6mm φ @ top hanger bars.

2.2 Design of Continuous Beams for flexure and Shear by LSM

Methods of Continous Beams:

1. Moment Distribution Methods 2. Slope Deflection Method
3. Theorem of three moments 4. substitute frame method.

* Continous Rc Rectangular Beam of size 250mm x 450mm over all size is supported by 250mm x 250mm size masonry columns at 5.0m c/c. The Beam carries a Dead load of 25kN/m including its self wt and an imposed load of 18kN/m Using M20 concrete and fe 415 grade steel Design the beam for the Support next to end Support Section.

Given Data:

Beam size = 250mm x 450mm

Column c/c = 5.0m

DL = 25 kN/m

Masonry support = 250mm x 250mm

Clear Distance = 5.0 - 0.25 = 4.75m

Live Load = 18kN/m

To find:

The beam for the support next to the end support section

Solution:

Step 1: Design Constants:

f_{ck} = 20N/mm²

f_y = 415 N/mm²

M_{ulim} = 2.76 bd^2

x_{umax} = 0.48 d

Effective span (l)

Effective depth (d) $= 450 - 25 - \frac{20}{2} = 415$ mm

width of support $= 250$ mm

$= 1/12$ of clear span $= 1/12 \times 4750 = 395.83$ mm

width of support $<$ 1/12 of Clear span.

Effective span (l) (i) $= L + d$

(ii) $= L + \frac{bw}{2} + \frac{bw}{2}$

(i) $= 4750 + 415 = 5165$ mm

(ii) $= 4750 + \frac{250}{2} + \frac{250}{2} = 5000$ mm

Take least (l_e) $= 5000$ mm (5.0) m

Bending Moment:

BM at support next to end support

$$M_u = 1.5 \times \left[-1/10 \times 25 \times 5.0^2 - 1/9 \times 18 \times 5^2 \right]$$

$= -168.75$ kN.m

Area of tension Reinforcement at support next to end support:

$M_u = 168.75$ kN.m

$M_{u1} = M_{ulim} = 2.76\,bd^2 = 2.76 \times 250 \times 415^2$

$= 118.835$ kN.m

$M_u > M_{ulim}$ = It is designed as a Doubly Reinforced Beam

Result:

Tension Reinforcement = 5 nos of 20mm φ bars

Compression Reinforcement = 2 nos of 16mm φ bars

Limit State of Collapse in shear:

Span ல் இருக்கும் bending moment-ன் மாற்றத்தினால் shear ஏற்பட்டு இருக்கின்றது.

Assumptions:

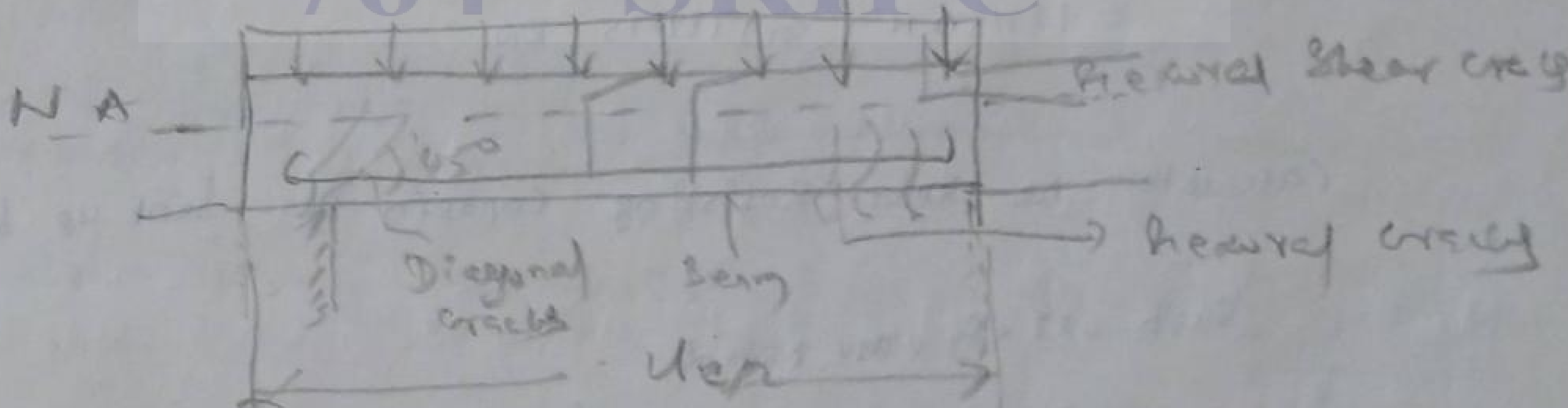
1. ஒரு வளைவு section ஆல் இருக்கும் shear failure அதனை தொடர்ந்து திண்மையுடன் இருக்கும்.

2. Shear failure ஆனது Ductile ஆக வைத்திருப்பதற்கு போதுமான Shear Resistance தேவைப்படுகின்றது.

3. over loading ன் போது load ஆனது இருக்கும் அளவினை விட flexure ஆல் கட்டுப்படுத்தப்படுகின்றது.

4. Tension Reinforcement ஆனது வலுவூட்டப்படும் bond மற்றும் anchorage ல் Shear ஆனது Minimum ஆக வைக்க உதவுகிறது.

Design Shear Strength of concrete:



Types of Stirrups:

(i) vertical
(ii) horizontal (or) inclined Stirrups 3. Bent up bars

Effects of Shear Reinforcement:

1. Stirrups ஆனது shear ஆனது ஏற்படும் போது beam ஆனது transverse ஆக Reinforce ஆனது Diagonal cracking ஐ தடுக்கின்றது.

2. Diagonal crack இருந்தாலும் Shear Reinforcement ஆல் தடுக்கப்படுகின்றது.

A Rectangular RC beam of effective size 300 mm × 615 mm is subjected to a Design Shear force of 280 kN. The Design shear Strength of Concrete is 0.5 N/mm², 2 nos of 16 mm ϕ Steel bars of Fe415 grade are bent up at 45° at end section. Determine the shear for which stirrups are to be Designed.

Given Data:

Effective size = 300 mm × 615 mm

Design shear $V_u = 280$ kN

Design shear Strength of Concrete $\tau_c = 0.5$ N/mm², $f_y = 415$ N/mm²

To find:

The suitable stirrups for shear

Solution:

Calculate the Excess Shear:

Excess shear for which shear Reinforcement $V_{us} = V_u - \tau_c b d$

$V_{us} = 280 \times 10^3 - 0.5 \times 300 \times 615$

$= 187750$ N (or) 187.75 kN

Calculate the shear Resisting Capacity of bent up bars

$V_{usb} = 0.87 \times f_y \times A_{sv} \times \sin\alpha$

$= 0.87 \times 415 \times 2 \times 201 \times \sin 45^\circ$

$= 102631$ N

$= 102.63$ kN

maximum shear Resisting capacity of bent up bars $V_{us\,lim}$

$V_{us\,lim} = \frac{V_{us}}{2} = \frac{187.75}{2} = 93.88$

Shear for which stirrups are to be

Design shear $= V_{us} - V_{us\,lim}$

$= 187.75 - 93.88$

$= 93.88$ kN

Result:

Design Shear for stirrups $= 93.88 kN$

Determine the Shear Resisting capacity of (i) 6mm dia 2 legged vertical stirrups of mild steel provided at 200mm c/c (ii) 6mm dia 2 legged inclined stirrups of mild steel provided at 200mm c/c with 60° inclination to the axis of the beam if the effective depth of the beam is 450mm. Use M20 grade concrete and Fe 250 grade of steel are used.

given Data:

$d = 450$ mm (i) 6mm ϕ 2 legged vertical stirrups @ 200mm c/c

(ii) 6mm ϕ 2 legged inclined stirrups @ 200mm c/c

inclination angle $\alpha = 60°$

material properties: $f_{ck} = 20 N/mm^2$, $f_y = 250 N/mm^2$.

To find:

Shear Resisting capacity

Solution:

Calculate the shear Resisting for vertical stirrups and inclined stirrups

Shear Resisting for vertical stirrups $V_{us} = \frac{0.87 \times f_y \times d \times A_{sv}}{S_v}$

$A_{sv} = \frac{2 \times \pi \times 6^2}{4} = 56.55 mm^2$

$V_{us} = \frac{0.87 \times 250 \times 450 \times 56.55}{200}$

$V_{us} = 27.67$ kN

Shear Resisting Capacity of inclined stirrups $V_{us} = \frac{0.87 \times f_y \times A_{sv} (\cos\alpha + \sin\alpha) \times d}{S_v}$

$= \frac{0.87 \times 250 \times 56.55 \times 450 (\cos 60° + \sin 60°)}{200}$

Result:

(i) $V_{us} = 27.67$ kN Vertical Stirrups

(ii) $V_{us} = 37.804$ kN inclined Stirrups

3.1 Design of one way slabs and Stair Cases By LSM

Slab என்பது 3D Plane Structural member ஆகும். இந்த member-களின் அமைவு ஆக இதன் தடிமன் மிக குறைவாக இருக்கும்.

Classification of Slabs

(i) one way Spanning Slabs

(ii) Two way Spanning Slabs

(iii) flat slabs supported Directly on Columns without beams

(iv) Grid slabs

(v). Circular and other Shapes

(vi) one way continous slab

(vii) Two way continous Slabs.

Design a RC loft slab of clear projection 600mm with uniform thickness to carry an imposed load of 2kN/m2. use M_{20} Concrete and Mild Steel Reinforcement. Check for Stiffness.

Given,

Clear projection $L = 600$ mm

imposed load $W = 2$ kN/m^2

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

$f_y = 250$ N/mm^2

To find out:

The Cantilever Slab

Solution:

material properties:

$Q_u = 2.97$, $\tau_{bv} = 7$, $MF = 1.15$

depth for stiffness:

$$d_{stiff} = \frac{Span}{BV \times Mf}$$

$$= \frac{600}{7 \times 1.15}$$

$$= 74.53 \simeq 80mm$$

Assume Clear Cover as 15mm and 16mm diameter bars.

$$d' = cc + \phi/2 = 15 + 16/2 = 23mm$$

Overall Depth $D = d + d' = 80 + 23 = 116 mm$

Effective span (le):

$$le = L + d/2 = 600 + \frac{80}{2} = 640 mm$$

Load calculation:

Self wt of slab $= b \times D \times 25 = 2.75 kN/m^2$

Imposed load $= 2 kN/m^2$

Total load $= 4.75 kN/m^2$

Factored Load $Wu = W \times 1.5 = 4.75 \times 1.5 = 7.125 kN/m^2$

Moment calculation:

$$Mu = \frac{Wu \times le^2}{2} = \frac{7.125 \times 0.64^2}{2} = 1.46 \times 10^6 \; N \cdot mm$$

Effective depth

$$d_{eff} = \sqrt{\frac{Mu}{Qu \times b}} = \sqrt{\frac{1.46 \times 10^6}{2.97 \times 1000}} = 25 mm$$

Ast:

$$Mu = 0.87 \times fy \times Ast \times d \left[1 - \frac{fy \times Ast}{fck \times b \times d} \right]$$

Scanned by TapScanner

$$1.46\times10^6 = 0.87\times250\times Ast\times80\left[1-\frac{250\times Ast}{20\times1000\times80}\right]$$

$$1.46\times10^6 = 17400\,Ast - 2.72\,Ast^2$$

$$Ast = 85.04\ mm^2$$

Minimum $Ast = \frac{0.15\,bD}{100} = \frac{0.15\times1000\times110}{100}$ 2165 mm

Maximum $Ast = 0.04\,bd = 0.04\times1000\times110 = 4400\,mm^2$

No. of bars and Spacing:

No. of bars $= \frac{Ast}{ast}$

$= \frac{165}{\pi/4\times16^2}$

$= 0.82 \simeq 2$ bars

Actual Ast $=$ No. of bars $\times\frac{\pi d^2}{4}$

$= 2\times\frac{\pi\times16^2}{4} = 402.12\ mm^2$

Distributors:

No. of bars $= \frac{Ast\,min}{ast} = \frac{165}{\frac{\pi\times8^2}{4}} = 3.28 \simeq 4$ bars

Actual Ast $= 4\times\pi/4\times8^2 = 201.06\ mm^2$

Spacing for Main Reinforcement:

(i) 3d $= 240$ mm

(ii) 480mm

(iii) $\frac{ast}{Ast}\times1000 = 500$ mm

Spacing for Distributors:

(i) $5d = 5 \times 80 = 400\,mm$

(ii) $450\,mm$

(iii) $\frac{ast}{Ast} \times 1000 = 250\,mm$

Curtailment:

50 % of bars are curtailed at $l_e/2$ Distance

1 bar is curtailed at 320 mm from the face of the support.

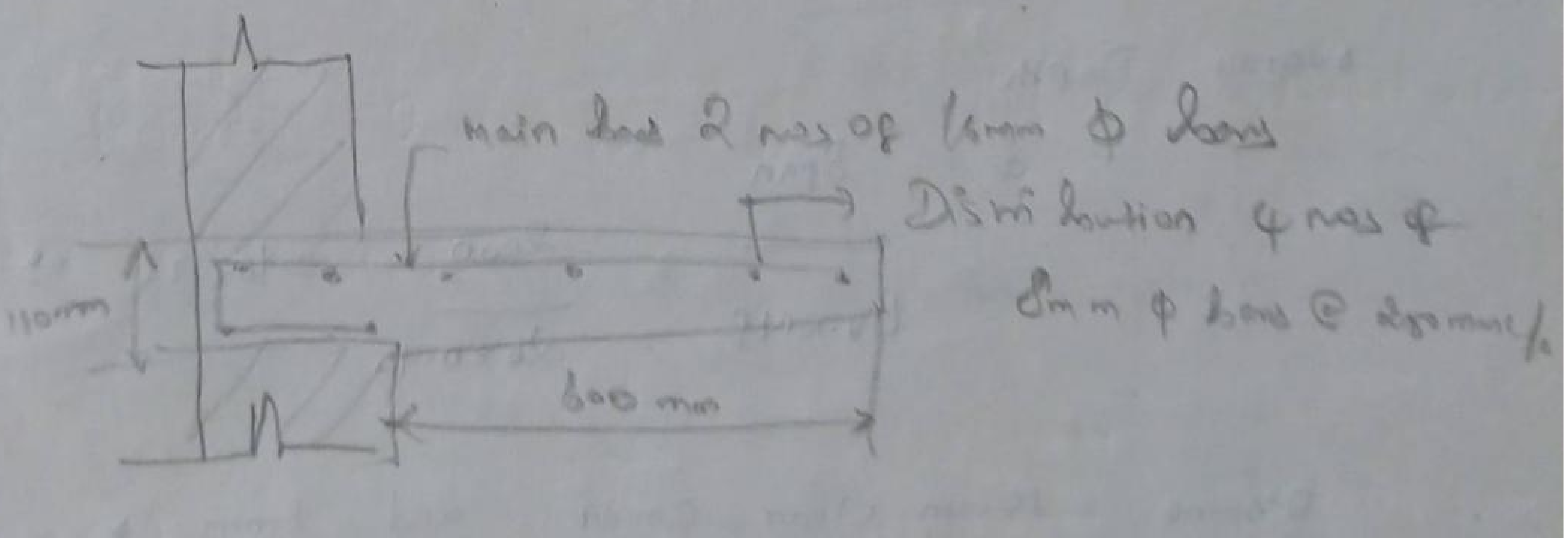
Development Length

$$L_d = \frac{0.87 \times f_y \times \phi}{4 \tau_{bd}} = \frac{0.87 \times 250 \times 16}{4 \times 1.2} = 725\,mm$$

Result:

Main Reinforcement : provide 2 nos of 16mm ϕ Fe 250 steel @ 240 mm c/c

Distributors : provide 4 nos of 8mm ϕ bars @ 250 mm c/c



Design a simply Supported Roof slab for a Room of Clear Dimensions 8m x 6m with wall thickness 200 mm Using M_{20} Concrete and Fe 415 grade steel. Wt of weathering course is 1.5 kN/m². Access is provided to the Roof.

Given Data:

Clear Dimensions 2m x 6m

Wall thickness = 200mm

f_{ck} = 20 N/mm², f_y = 415 N/mm²

Wt of weathering course = 1.5 kN/m²

Access is provided to the Roof.

To find: Simply supported Roof Slab

Solution:

Material properties and Design constants:

$Q_u = 2.76$, $B_v = 20$, $M_f = 0.95$

Types of slab $\frac{l_y}{l_x} = \frac{6}{2} = 3 > 2$

The slab is one way slab

Overall Depth:

$$d = \frac{Span}{B_v \times M_f} = \frac{2000}{20 \times 0.95} = 105.26 \simeq 110mm$$

Assume 15mm clear cover and 8mm ϕ main bar

$D = d + d' = 110 + 15 + 8/2 = 130$ mm

Effective Length:

(i) $l_e = L + d = 2000 + 110 = 2110mm$

$l_e = L + \frac{b_w}{2} + \frac{b_w}{2} = 2000 + \frac{200}{2} + \frac{200}{2} = 2200mm$

Take least value $l_e = 240$ mm

Load Calculation:

Sf wt of Slab $= b \times D \times$ unit weight $= 1 \times 0.13 \times 25$

Consider 1m width of slab $= 3.25$ kN/m

Imposed Load $= 1.50$ kN/m

Wt of weathering course $= 1.5$ kN/m

Total load $= 6.25$ kN/m

factored load $= w \times 1.5 = 6.25 \times 1.5$

$= 9.375$ kN/m

Bending moment: Mu

$$M_u = \frac{w_u \times l_e^2}{8} = \frac{9.375 \times 2.11^2}{8} = 5.22 \text{ kN.m}$$

deffective:

$$d_{eff} = \sqrt{\frac{M_u}{Q_u \times b}} = \sqrt{\frac{5.22 \times 10^6}{2.76 \times 1000}} = 43.49 \simeq 45\text{MM}$$

Area of steel: Ast:

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

$$5.22 \times 10^6 = 0.87 \times 415 \times A_{st} \times 110 \left[1 - \frac{415 \times A_{st}}{20 \times 1000 \times 110} \right]$$

$$5.22 \times 10^6 = 39715.5\ A_{st} - 7.49\ A_{st}^2$$

On Solving $A_{st} = 134.87$ mm^2

No. of Spacing:

(i) 3d $= 330$ mm

(ii) 300 mm

(iii) $\frac{ast}{Ast} \times 1000 = 250mm$

Distributors:

No. of bars $= \frac{Ast\,min}{ast\,min} \times 1000 = \frac{156}{\pi/4 \times 8^2} = 4$ bars

Spacing: (i) $5 \times d = 550mm$ (ii) $450mm$ (iii) $\frac{ast\,min}{Ast\,min} \times 1000 = 330mm$

Curtailment:

50% of bars are curtailed at 0.08 le distance from the face of support.

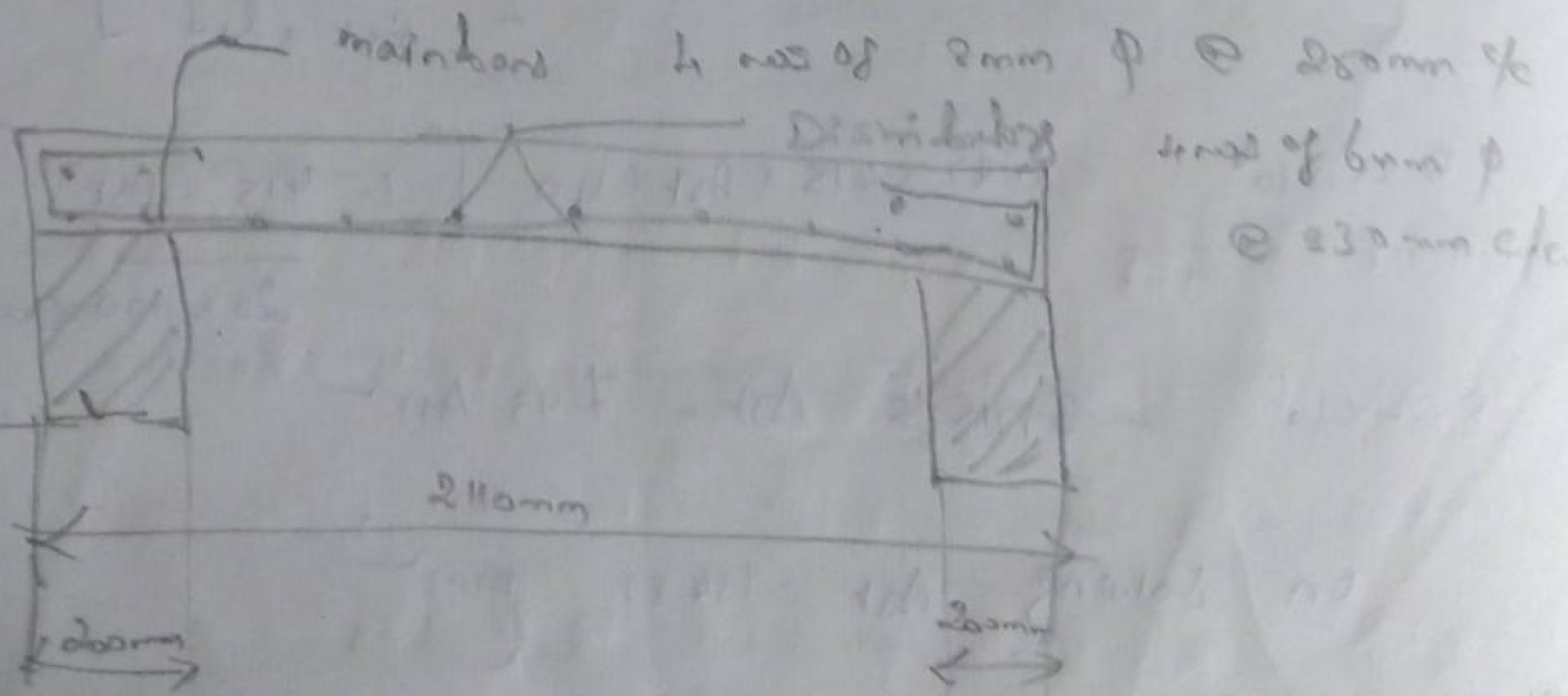
2 bars are curtailed @ 169 mm from the face of the support.

Check for shear:

$V_u = \frac{W_u \times le}{2} = \frac{9.375 \times 2.11}{2} = 9.89$ kN

$\tau_v = \frac{V_u}{bd} = \frac{9.89 \times 10^3}{1000 \times 110} = 0.09$ N/mm²

Hence safe.



Stairs:

ஒரு கட்டிடத்தில் மற்றொரு floor களை இணைப்பதற்கு பல படிகளை ஒன்றன்பின்னாக ஒன்றாக தொடர்ச்சியாக அமைக்கப்படும் படிக்கட்டுகள் தேவைப்படும்

Types of Stairs:

(i) Stairs Spanning horizontally
(ii) Stairs Spanning Longitudinally
(iii) Cantilever Stairs
(iv) Rise tread Stairs
(v) Individual Cantilever steps (spiral stairs).

The vertical height b/w two Successive floor of a Multi Storeyed Residential Building is 3m. The clear size of the Stair case Room is 2.10 x 4.25m. Plan a dog legged Staircase for the Building.

Given:

Vertical height b/w floors = 3m
Size Room = 2.10m x 4.25m

To Find:

A dog legged Stair case

Solution:

Geometrical Design

Width of stair case flight slab = 1000 mm

Width of Room = 2.10 m = 2100 mm

= 2

No. of flights

Width of opening b/w two flights = 2100 − 2x1000 = 100 mm

Width of Mid landing, landing slab = 1000 mm

Horizontal going of each flight = 4250 − 2x1000 = 2250 mm

Assuming Thread $= 250mm$

No of Tread / Steps per flight $= \frac{2250}{250} = 9$

No of Rise $= 9+1 = 10$

Height b/w ground floor to Mid landing and Mid landing to floor landing $= 1500mm$

Rise of each step $= \frac{1500}{10} = 150$

Result:

Number of flights $= 2$ nos.

Number of steps $= 9$ nos

Tread $= 250mm$

Rise $= 150mm$

3.2 Design of Two way Slabs

Two way slab:

ஒரு Slab நான்கு விளிம்புகளிலும் தாங்கியில் பொருந்தி Longer span க்கும் Shorter span க்கும் இடையே உள்ள விகிதம் 2க்கு குறைவாக இருந்தால் bending ஆனது இரண்டு span களிலும் நிகழ்கிறது. இத்தகைய slab க்கு Two way slab என்று பெயர்.

Types of slabs:

1) Slab not held down
2) Slab held down
3) Slab with edges fixed

Design a Simply Supported Roof slab for a watchman Cabin of Clear size 2m x 3m. The thickness of wall about is 200mm. Access is not provided to the Roof. The Corner of the slab are not held down. wt of weathering course is $1 kN/m^2$. use M20 grade Concrete and Fe 415 grade Steel.

Given Data:

Simply Supported Roof slab

Room size Clear = 2m x 3m.

Thickness of wall = 200mm.

Access is not provided.

Corners are not held down

Wt of weathering course = $1 kN/m^2$

$f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$, $M_{ulim} = 2.76 bd^2$

$B_v = 20$, $MF = 0.95$

To find out:

A simply supported two way slab

Solution:

Type of slab:

$$\frac{l_y}{l_x} = \frac{3}{2} = 1.5 < 2$$

The slab is Two way slab

Effective depth (d') from stiffness

d = span

$= \frac{2000}{28 \times 0.95} = 80mm$

Effective depth $d = 100 - 15 - 8/2 = 80mm$

Calculate the effective span:

Clear span + d = 2000 + 80 = 2080mm

Clear span $+ \frac{bw}{2} + \frac{bw}{2} = 2000 + \frac{200}{2} + \frac{200}{2} = 2200mm$

Effective shorter span $l_x = 2.08m$

longer:

(i) 3000 + 80 = 3080mm

(ii) $3000 + \frac{200}{2} + \frac{200}{2} = 3200mm$

longer span $l_y = 3.08m$

Calculate load:

Self wt of slab $= 0.1 \times 25 = 2.5\ kN/m^2$

wt of weathering course $= 1.0\ kN/m^2$

Access is not provided $= 0.75\ kN/m^2$

Minimum imposed load $= 1.30\ kN$

Total load $= 4.45\ kN/m^2$

1m width of slab is consider $w = 4.45\ kN/m$

Factored load $w_u = 1.5 \times 4.45 = 6.68\ kN/m$

Calculate the Design Bm:

$\frac{l_y}{l_x} = 1.481$

$\alpha_x = 0.099 + \left[\frac{0.104 - 0.099}{1.5 - 1.4}\right] \times 0.086 = 0.1033$

$\alpha_y = 0.057 - \left[\frac{0.057 - 0.0461}{1.5 - 1.4}\right] \times 0.086 = 0.0468$

Bm in shorter span $M_x = \alpha_x \times w_u \times l_x^2$

$= 0.1033 \times 6.68 \times 2.080^2$

$= 2.985$ kN.m

Bm in Longer span $M_y = \alpha_y \times w_u \times l_x^2$

$= 0.0468 \times 6.68 \times 2.080^2$

$= 1.352$ kN.m

effective depth Required:

$d = \sqrt{\frac{2.985 \times 10^6}{2.76 \times 1000}} = 32.87$ mm

$d_{provided} > d_{req}$

Hence ok

Area of steel Reinforcement:

In shorter Direction $M_{ux} = 0.87 \times f_y \times A_{stx} \times d \left[1 - \frac{f_y \times A_{stx}}{f_{ck} \times b \times d}\right]$

$2.985 \times 10^6 = 0.87 \times 415 \times A_{st} \times 80 \left[1 - \frac{415 \times A_{stx}}{20 \times 1000 \times 80}\right]$

$Ast_x = 106.27\ mm^2$

In longer Direction

$$M_{uy} = 0.87 \times f_y \times Ast_y \times d \left[1 - \frac{f_y \times Ast_y}{f_{ck} \times b \times d} \right]$$

$$1.352 \times 10^6 = 19135.65\ Ast_y - 7.49\ Ast_y^2$$

$$7.49\ Ast_y^2 - 19135.65\ Ast_y + 1.32 \times 10^6 = 0$$

$$Ast_y = 72.72\ mm^2$$

minimum $Ast_{req} = \frac{0.12}{100} \times 1000 \times 80 = 96\ mm^2$

Steel Direction $Ast_x = 106.27\ mm^2$

assuming 8mm ϕ bars Spacing $= \frac{1000 \times 50.27}{106.27} = 473\ mm$

UNIT-IV

4.1 Design of Columns by LSM

Column:

ஒரு நெடுக்குத் member ஆனது bending moment உடன் அல்லது bending moment இல்லாமலோ அச்சு அழுத்த சுமைக்கு உட்படுமானால் அவை Column என்று அழைக்கப்படுகின்றன.

Column ஆனது axial load ஐக் கொண்டும் x அல்லது y அச்சைப் பற்றி bending moment அளவைப் பொறுத்தும் வகைப்படுத்தப்படுகின்றது.

1. Axial loaded Column
2. Axial load with uniaxial bending
3. Axial load with biaxial bending

Assumptions:

1. Bending க்கு பின் உள்ள plane section ஆனது அதன் அச்சுக்கு செங்குத்தாக இருக்கும்.

2. வளைப்பு நிலையில் compression fibre ல் concrete ன் அதிகபட்ச strain ஆனது bending ல் 0.0035 சமமாக எடுத்துக் கொள்ளப்படுகிறது.

3. Concrete ன் tensile வலிமை கணக்கில் எடுத்துக் கொள்ளப்படுவதில்லை.

4. Axial compression ல் concrete ல் உள்ள அதிகபட்ச compressive strain 0.002 ஆக எடுத்துக் கொள்ளப்படுகிறது.

Unsupported length of column:

Column த்தின் end restraint களுக்கு இடைப்பட்ட தூரத்தை அதன் unsupported length (L) ஆக எடுத்துக் கொள்ள வேண்டும்.

Limiting strength of short axially loaded compression members

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

Compression members with helical reinforcement:

$$P_u = 1.05 \left[f_{ck} \times A_c + 0.67 \times f_y \times A_{sc} \right]$$

Strength of column:

1. பொருட்களின் வலிமை
2. Cross section னின் வடிவம் மற்றும் அளவு
3. நீளம்
4. இணைப்பின் degree con ஆகும் மற்றும் Directional Restraints
5. சுமை அமைக்கப்படுவது.
   Slenderness Ratio

Slenderness Ratio:

Slenderness Ratio என்பது column த்தின் unsupported நீளத்திற்கும் அதன் cross sectional area வின் குறைந்த Radius of gyration க்கும் உள்ள விகிதமாகும்.

Scanned by TapScanner

Classification of Columns:

1. Shape of Cross section
2. Slenderness Ratio
3. Type of Loading
4. Pattern of lateral Reinforcement

Short Column

Slenderness Ratio $\frac{l_{ex}}{D}$ மற்றும் $\frac{l_{ey}}{b}$ ஆகியவை 12க்கு குறைவாக இருந்தால் அவை Short column என்றழைக்கப்படுகிறது.

Long Column:

Slenderness Ratio $\frac{l_{ex}}{D}$ மற்றும் $\frac{l_{ey}}{b}$ ஆகியவை 12க்கு அதிகமாக இருந்தால் அவை Long column என்றழைக்கப்படுகிறது.

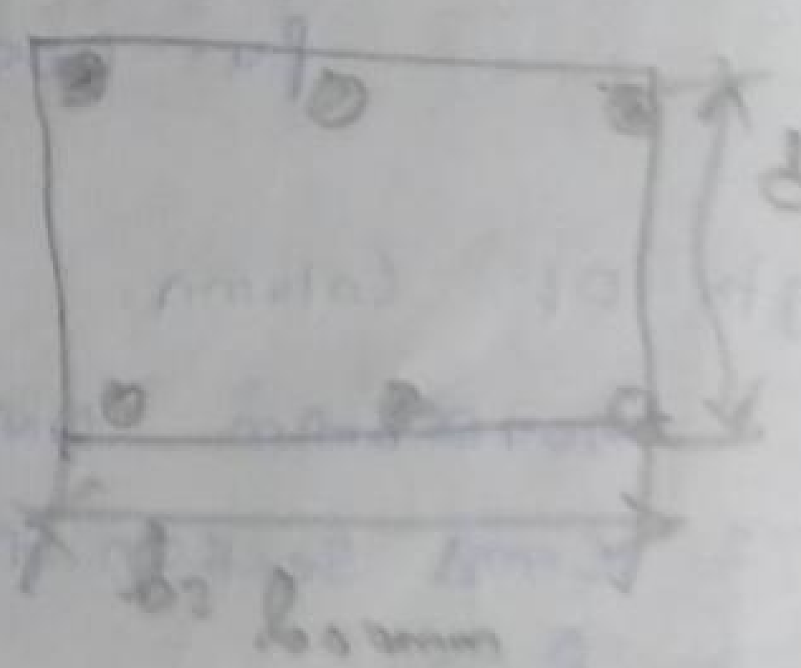
Rectangular Column:

A 600mm × 400mm Column 3m long effectively held in position and Restrained against Rotation at both ends is provided with 6 bars of 22mm φ Fe 415 steel. Determine the strength of Column if M20 grade and Fe 415 steel used.

Given Data:

Size of Column = 600mm × 400mm

Length L = 3m = 3000mm



Reinforcement = 6 nos of 22mm φ bars

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

End Condition: Effectively held in position and Restrained against Rotation.

Effective length $l_e = 0.65 \times L = 0.65 \times 3000 = 1950mm$

To determine:

Strength of Column

Solution:

Slenderness Ratio (λ)

$$\lambda_x = \frac{le}{b} = \frac{1950}{600} = 3.25 < 12 \quad @ \text{ x axis}$$

$$\lambda_y = \frac{le}{D} = \frac{1950}{400} = 4.88 < 12 \quad @ \text{ y axis}$$

Slenderness Ratio (λ) is less than 12 along both axes

$\therefore$ The column is a short column

Eccentricity (e_{min}):

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

$$= \frac{3000}{500} + \frac{600}{30} = 26$$

$= 26 \text{ mm} > 20 \text{ mm}$ Along x axis

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

$$= \frac{3000}{500} + \frac{400}{30} = 19.33$$

$= 19.33 < 20\text{mm}$ Along y axis

$e_{min} < 0.05$ times the LLD, the load may be assumed as axial.

$e_{min} < 0.05 \times b = 0.05 \times 400 = 20\text{mm}$

Hence ok.

Design of column for strength:

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

$= 0.4 \times 20 \times 237.72 \times 10^3 + 0.67 \times 415 \times 2280.79$

$= 2535.93 \times 10^3$ N

$A_c = A_g - A_{sc}$

$A_g = 600 \times 400 = 240 \times 10^3 \text{ mm}^2$

$A_{sc} = \frac{\pi d^2}{4} \times \text{No. of bars}$

$= \frac{\pi \times 22^2}{4} \times 6 = 2280.79 \text{ mm}^2$

$A_c = 240 \times 10^3 - 2280.79$

$= 237.72 \times 10^3 \text{ mm}^2$

Result:

Strength of column $P_u = 2535.93 \times 10^3$ N

Circular Column:

Design a circular column to carry an axial load of 800 kN The unsupported length of column is 3.70 m. and it is hinged at both ends. Assume circular Rings for transverse Reinforcement. Use M20 grade of concrete and Fe 415 grade of steel.

Given Data:

Axial load $P = 800\,kN$, $P_u = 800 \times 1.5 = 1200\,kN$

$f_{ck} = 20\,N/mm^2$, $f_y = 415\,N/mm^2$, $L = 3.70\,m$

To find:

The column with lateral ties

Solution:

Size of column:

$P_u = 0.4 \times f_{ck} \times A_c + 0.67 \times f_y \times A_{sc}$ $A_{sc} = 2\%$ of A_g

$A_c = A_g - A_{sc} = A_g - \frac{2}{100} \times A_g$ $A_c = 0.98\,A_g$

$1200 \times 10^3 = 0.4 \times 20 \times 0.98\,A_g + 0.67 \times 415 \times 0.02\,A_g$

$1200 \times 10^3 = 7.84\,A_g + 5.561\,A_g$

$A_g = \frac{1200 \times 10^3}{13.401} = 89.545 \times 10^3\,mm^2$

A_g for circular column $= \frac{\pi d^2}{4}$

$89.54 \times 10^3 = \frac{\pi d^2}{4}$

$D = \sqrt{\frac{89.54 \times 10^3 \times 4}{\pi}} = 337.65 \simeq 350\,mm.$

Slenderness Ratio (λ):

$\lambda = \frac{L_e}{LLD} = \frac{3700}{350} = 10.57$

Area of Reinforcement (Asc):

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

$A_g = \frac{\pi d^2}{4} = \frac{\pi \times 350^2}{4} = 96.211 \times 10^3 \ mm^2$

$1200 \times 10^3 = 0.4 \times 20 \times 96.21 \times 10^{3} - A_{sc} + 0.67 \times 415 \times A_{sc}$

$1200 \times 10^3 = 769.688 \times 10^3 - 8 A_{sc} + 278.05 A_{sc}$

$1200 \times 10^3 - 769.688 \times 10^3 = 270.05 A_{sc}$

$A_{sc} = \frac{430.12 \times 10^3}{270.05} = 1593.45 \ mm^2$

minimum A_{sc}, $A_{sc\,min} = 0.8\%$ of $A_g = \frac{0.8}{100} \times 96.21 \times 10^3$

$= 769.65 \ mm^2$

maximum $A_{sc} = 6\%$ of A_g

$= \frac{6}{100} \times 96.21 \times 10^3 = 5772.66 \ mm^2$

$A_{sc\,min} < A_{sc} < A_{sc\,max}$

$769.69 < 1593.45 < 5772.66$

Hence ok.

No. of bars $= \frac{A_{sc}}{a_{sc}}$ (assume 20mm ϕ bars)

Asc pro $= \frac{1593.45}{\frac{\pi}{4} \times 20^2} = 5.07 \simeq 6$ bars

$= 6 \times \frac{\pi}{4} \times 20^2 = 1884.96 \ mm^2$

Diameter and pitch for Lateral ties

Diameter:

(i) 6mm (ii) $1/4 \times 20 = 5mm$

pitch : (i) 300 MM (ii) LLD = 350 mm

(iii) 16 times of the Diameter of bar

$16 \times 20 = 320$ mm

$\therefore$ provide 6mm ϕ Lateral ties @ 300mm c/c

Spacing for Longitudinal Reinforcement :

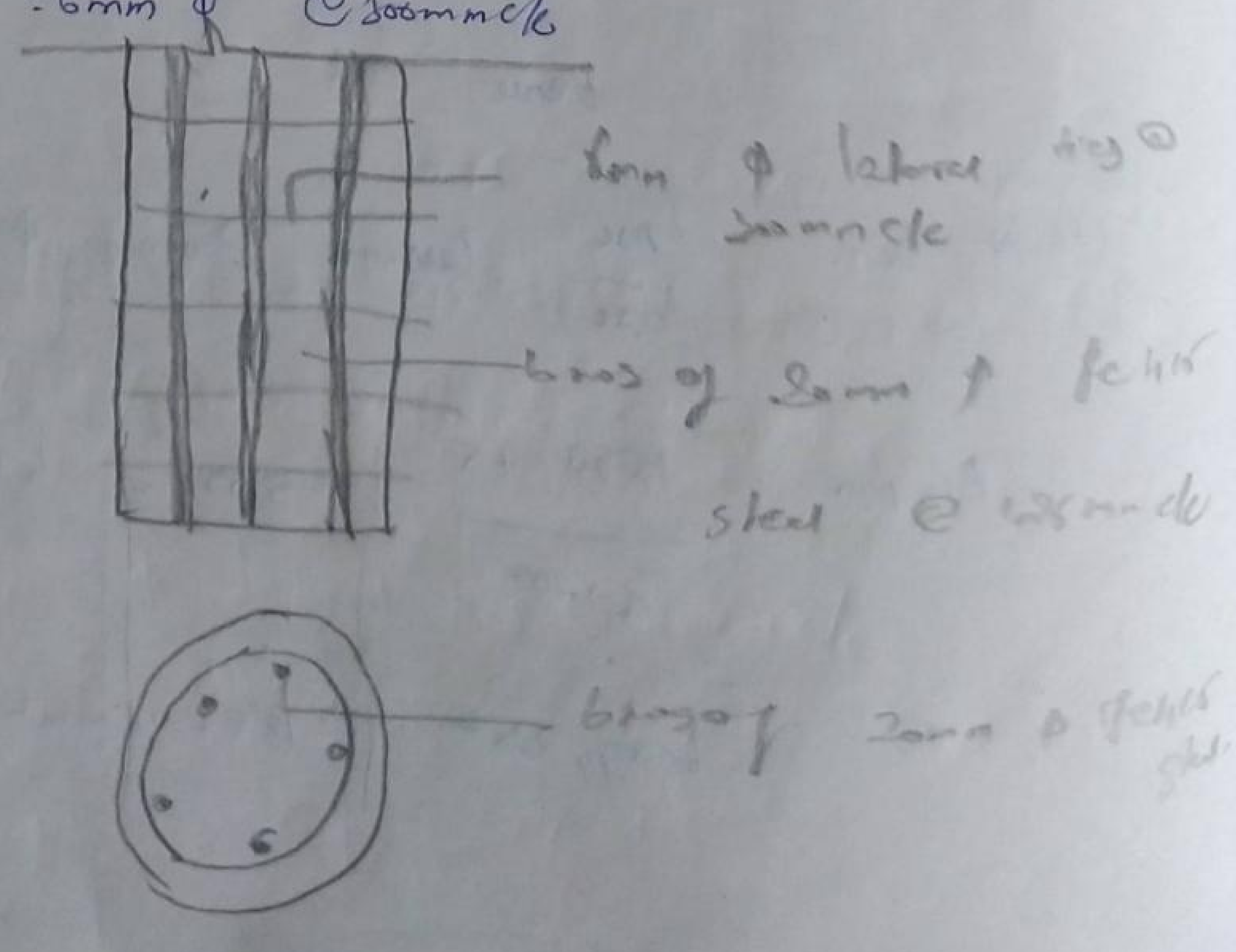
$$\text{Spacing } S = \frac{D - 2 \times cc - \phi/2 - \phi/2}{2}$$

$$= \frac{350 - 2 \times 40 - \frac{20}{2} - \frac{20}{2}}{2} = 125 \text{ MM}$$

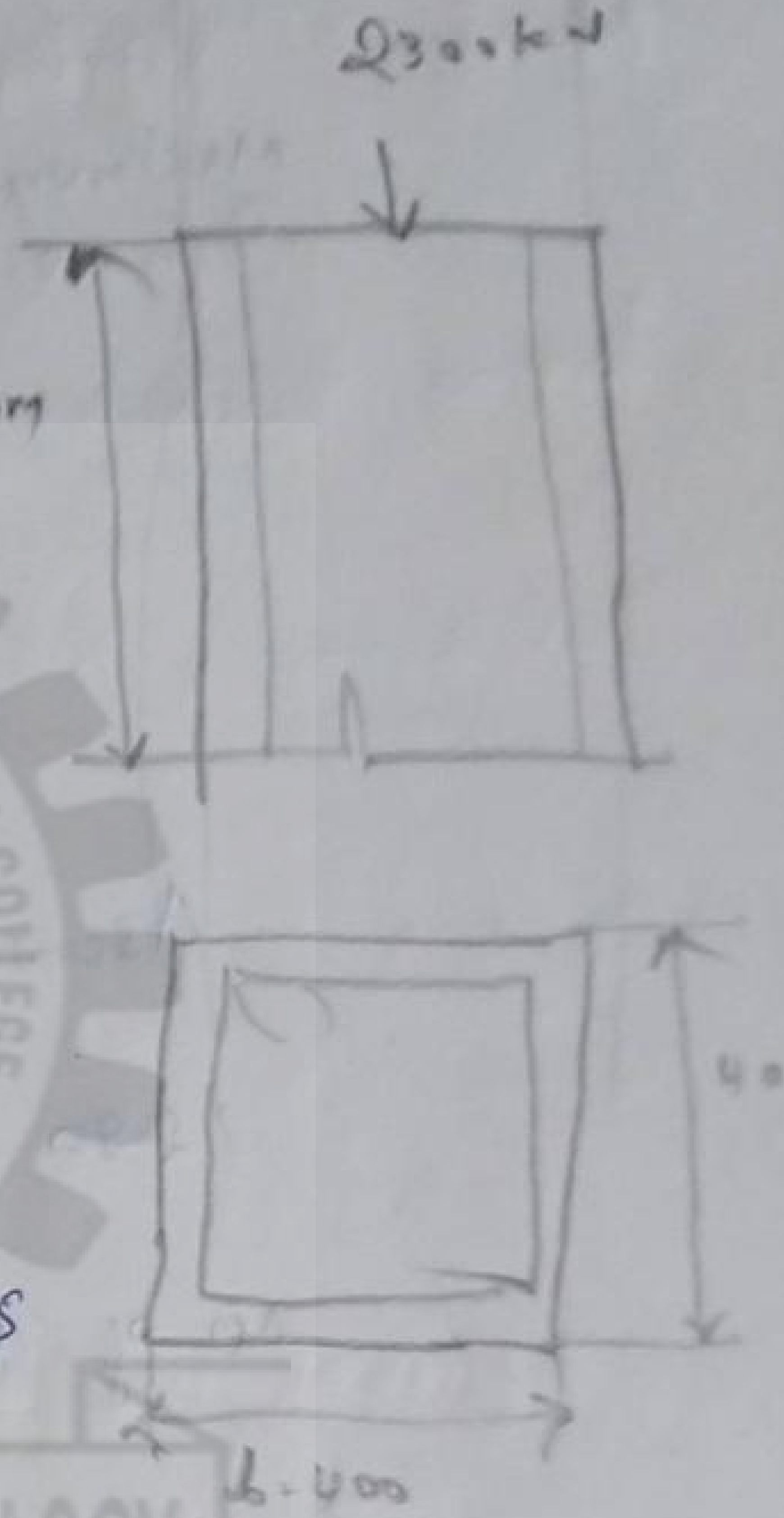
Result!

Longitudinal Reinforcement! 6 nos of 20mm ϕ Fe415 Steel @ 125mm c/c.

Lateral ties : 6mm ϕ @ 300mm c/c



Design an axially loaded column of 400 mm × 400 mm size to carry an axial ultimate load of 2300 kN. The effective length of column is 3m. M20 grade of concrete and Fe415 grade steel are to be used.



Given Data:

Column size = 400 mm × 400 mm

ultimate Load $P_u = 2300$ kN

Effective column $l_e = 3m = 3000$ mm

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

To Find:

The column with Lateral ties

Solution:

Slenderness Ratio $(\lambda) = \frac{l_e}{LD} = \frac{3000}{400} = 7.5$

$\lambda = 7.5 < 12$

It is a short column

Eccentricity (e_{min}):

$e_{min} = \frac{l}{500} + \frac{b}{30} = \frac{3000}{500} + \frac{400}{30} = 19.33 < 20mm$

It is a axially loaded column.

Area of Reinforcement:

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

$A_g = 400 \times 400$

$= 160 \times 10^3\ mm^2$

$2300 \times 10^3 = 0.4 \times 20 \times 160 \times 10^3 - A_{sc} + 0.67 \times 415 \times A_{sc}$

$1.02 \times 10^6 = 270.05\ A_{sc}$

$A_{sc} = \frac{1.02 \times 10^6}{270.05} = 3777.08\ mm^2$

Minimum $A_{sc} = 0.8\%$ of A_g

$= \frac{0.8}{100} \times 160 \times 10^3$

$= 1280\,mm^2$

Maximum $A_{sc} = 6\%$ of A_g

$= \frac{6}{100} \times 160 \times 10^3$

$= 9600\,mm^2$

$A_{sc\,min} < A_{sc} < A_{sc\,max}$

$1280 < 3727 < 9600$ Hence ok.

No. of bars $= \frac{A_{sc}}{a_{sc}}$ (assume 25mm ϕ bars)

$= \frac{3772.08}{\frac{\pi \times 25^2}{4}} = 7.69 \simeq 8$ nos

Diameter and pitch for lateral ties:

Diameter:

(i) 6mm (ii) $\frac{1}{4} \times \phi = \frac{1}{4} \times 25 = 6.25 \simeq 8mm$

pitch:

(i) 300mm (ii) LLD = 400 mm

(iii) $16 \times \phi = 16 \times 25 = 400mm$

provide 8mm ϕ lateral ties @ 300mm c/c.

Spacing:

$S = \frac{B - 2 \times Cc - \phi/2 - \phi/2}{2}$

$S = 400 - 2\times 40 - 25/2 - \frac{25}{2}$

$= 180\ mm$

25

(i) Longitudinal Reinforcement : 8 nos of 25 mm φ bars @ 180 mm c/c

Result (ii) Lateral ties 8 mm φ bars @ 300 mm c/c

4.2 Design of Column footings

Footings:

தூண்களின் சுமைகளை தாங்குவதற்காக அமைக்கப்படும் Column base அல்லது Column footings என்று பெயர்.

Basic Requirements of footings:

1. அடியின் அடிப்பகுதி, அமைப்பிற்கு ஏற்படும் எடை moment மற்றும் Reaction ஆகிய தாங்கக்கூடிய போதுமான திறனுடன் அமைக்கப்பட வேண்டும்.

2. போதுமான ஆழம் மற்றும் Reinforcement உம் கொண்டிருக்க வேண்டும்.

Types of the footings:

1. Isolated bases / footings
2. Spread footings
3. Combined footings
4. Mat foundation
5. Pile foundation
6. Strap footings
7. Continuous footings for walls
8. Cantilever footing

Minimum Depth below GL:

$$D = \frac{w_1}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\theta}\right]^2$$

Minimum Reinforcement:

Square ஆன ஒவ்வொரு Column base களில் tension Reinforcement ஒருசீராக அமைக்கப்படுகின்றது.

Distribution of Reinforcement:

Long band, Uniform Spacing

Short band Spacing in band width A & B

$$\frac{\text{Reinforcement in central band width}}{\text{Total Reinforcement in short direction}} = \frac{2}{\beta+1}$$

Design a Square footing of uniform thickness to carry an axial load of 1200 kN. Size of column is 400mm × 400mm. Safe bearing capacity of soil is 180 kN/m². Use M_{20} grade concrete and Fe 415 grade of steel.

Given Data:

Axial load $P = 1200$ kN

Size of Column $= 400$ mm × 400 mm

Safe bearing capacity $= 180$ kN/m²

To find:

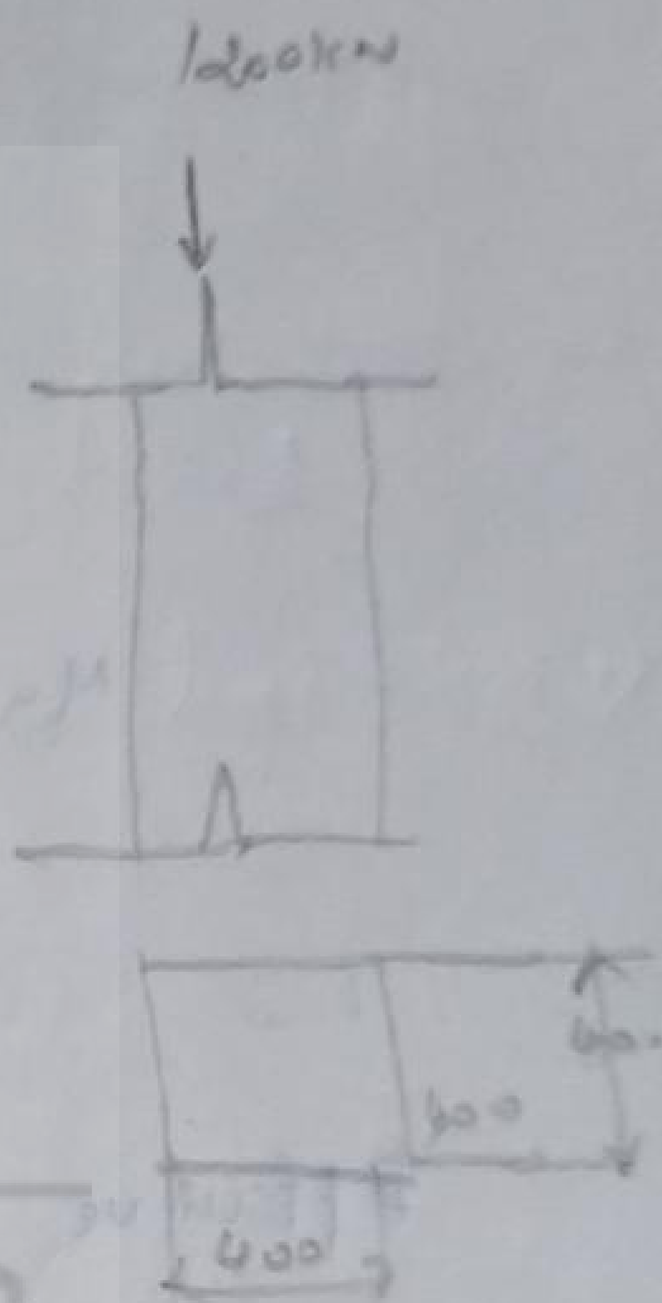
A square footing with uniform thickness

Solution:

Material properties & Design Constants

$f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$, $Q_u = 2.76$

Required area A_{req}:



$$A_{req} = \frac{\text{Load on Column + self wt}}{SBC}$$

Self wt = 10 % of Load on Column

$$= \frac{10}{100} \times 1200 = 120 \, kN$$

$$A_{req} = \frac{1200 + 120}{150} = 8.8 \, m^2$$

Size of footing:

Square footing $\sqrt{A_{req}}$

$$\sqrt{B} = \sqrt{8.8} = 2.96 \simeq 3m$$

$B = 3000mm$

Size of footing $B \times B = 3000 \times 3000$

Actual Area Required $= 9 \times 10^6 mm^2$

Upward Design pressure (Q_0)

$$Q_0 = \frac{\text{Load on Column} \times PSF}{A_{req}}$$

$$= \frac{1200 \times 10^3 \times 1.5}{9 \times 10^6} = 0.2 \, N/mm^2$$

Design Bending Moment: Mu:

Mu = Qo × (area of ▨ portion × Distance between c.g and face of column

Lp = projection length $= \frac{2000-400}{2} = 1300$

Centre of gravity $= \frac{Lp}{2} = \frac{1300}{2} = 650mm$

$Mu = 0.2 \times \left[(1300 \times 3000) \times 650 \right]$

$= 507 \times 10^6$ N.mm

Effective depth (d) & overall Depth (D)

$Mu = Q_{u} \times b \times d^2$

$d = \sqrt{\frac{Mu}{Q_u \times b}} = \sqrt{\frac{507 \times 10^6}{2.76 \times 3000}} = 247.45 \simeq 250mm$

$D = d + d' = 250 + 60 = 310mm$

Overall Depth is not sufficient to take care of shear force so increase the depth by 2 times.

$D = 2 \times D = 2 \times 310 = 620mm$

Revised effective depth $d = D - d' = 620 - 60 = 560mm$

Area of steel (Ast) & no. of bars

$Mu = 0.87 \times fy \times Ast \times d \left[1 - \frac{fy \times Ast}{fck \times b \times d} \right]$

$$507\times10^6 = 0.87\times415\times Ast\times560\left[1-\frac{415\times Ast}{20\times3000\times560}\right]$$

$Ast = 2590.30\ mm^2$

No. of bars $= \frac{Ast}{ast} = \frac{2590.30}{\pi/4\times20^2} = 8.24 \simeq 10$ bars

Actual Ast = no. of bars $\times \pi/4\times d^2 = 10\times\pi/4\times20^2 = 3141.6\ mm^2$

Development length (Ld):

$$Ld = \frac{0.87\times fy\times\phi}{4\times\tau_{bd}\times1.6}$$

$$= \frac{0.87\times415\times20}{4\times1.2\times1.6} = 940.23 \simeq 950\,mm$$

Ld < projection length, 950 < 1300

Hence safe

Check for shear:

(i) one way shear (@ d from face of column)

Shear force $Vu = q_0\times$ Area of $\boxed{\quad}$ portion.

$= 0.2\times(3000\times740$

$Vu = 444\times10^3\ N$

projection length − d

$1300-560$

$=740$

Nominal shear stress $\tau_v = \frac{Vu}{bd} = \frac{444\times10^3}{3000\times560}$

$= 0.26\ N/mm^2$

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

$\frac{100 A_{st}}{bd} = \frac{100 \times 314 \times 6}{3000 \times 560} = 0.19$

$\tau_c = 0.312\ N/mm^2$

$k = 1$ Assume

$\tau_v < k < \tau_c$

$0.26 < 1 < 0.312$

$0.26 < 0.312$

Hence safe in Local Shear

Two way Shear (@ d/2 around face of Column)

$V_u = q_0 \times$ loaded area

$= 0.2 \times 3000^2 - 960^2$

$= 1.62 \times 10^6\ N$

$\tau_v = \frac{V_u}{bd} = \frac{1.62 \times 10^6}{300 \times 560} = 0.96\ N/mm^2$

$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.12\ N/mm^2$

$K_s = 0.5 + \beta_c$

$\beta_c = \frac{\text{Short side Column}}{\text{Long side Column}}$

$= \frac{400}{400} = 1$

$K_s = 0.5 + 1 = 1.5$

$K_s \tau_c = 1.5 \times 1.12 = 1.68$

$\tau_v < K_s \tau_c$

$0.96 < 1.12$ (or) 1.68

Hence safe in two way shear

Transfer Load at column base

Nominal bearing stress < permissible bearing stress

$$\frac{P_u}{\text{Area of column}} < 0.45 \times f_{ck} \sqrt{\frac{A_1}{A_2}}$$

$$\frac{1200 \times 1.5 \times 10^3}{400 \times 400} < 0.45 \times 20 \sqrt{\frac{3000 \times 3000}{400 \times 400}}$$

$11.25 < 0.45 \times 20 \times 2$

$11.25 < 18$

Hence safe

Result:

Size of footing = 3000 mm × 3000 mm

Reinforcement provided 10 # of 20mm ϕ fe 415 steel bars on either sides.

Steel Structures

Unit - V Design of Tension and Compression members by Limit State method

Tension Member என்பது ஒரு Structural உறுப்பு ஆகும்.

Design strength of materials:

$$S_d = S_u/\gamma_m$$

Rolled Steel Sections:

Rolled steel section-கள் tension and compression Members-க்கு பொருத்தமான அமைப்பு கொண்டுள்ளது.

1. Steel pipes (circular, square (or) Rectangular)
2. Steel equal Angles (or) unequal
3. Steel - T-sections
4. Steel - channels such as ISLC, ISJC, ISMC
5. Steel - I sections such as IS JB, ISLB, ISWB, ISMB, ISHB

Different Forms of tension members:

1. wires and cables
2. Rods and bars
3. A single angle connected through one of its leg
4. Double Angles, back to back connected
5. T, I-sections and channels
6. Built up members

Gross area:

கணக்கிடும் பொழுது துளைகள் இல்லாத Member-ன் முழு cross section பரப்பு Gross Area என்று அழைக்கப்படுகிறது.

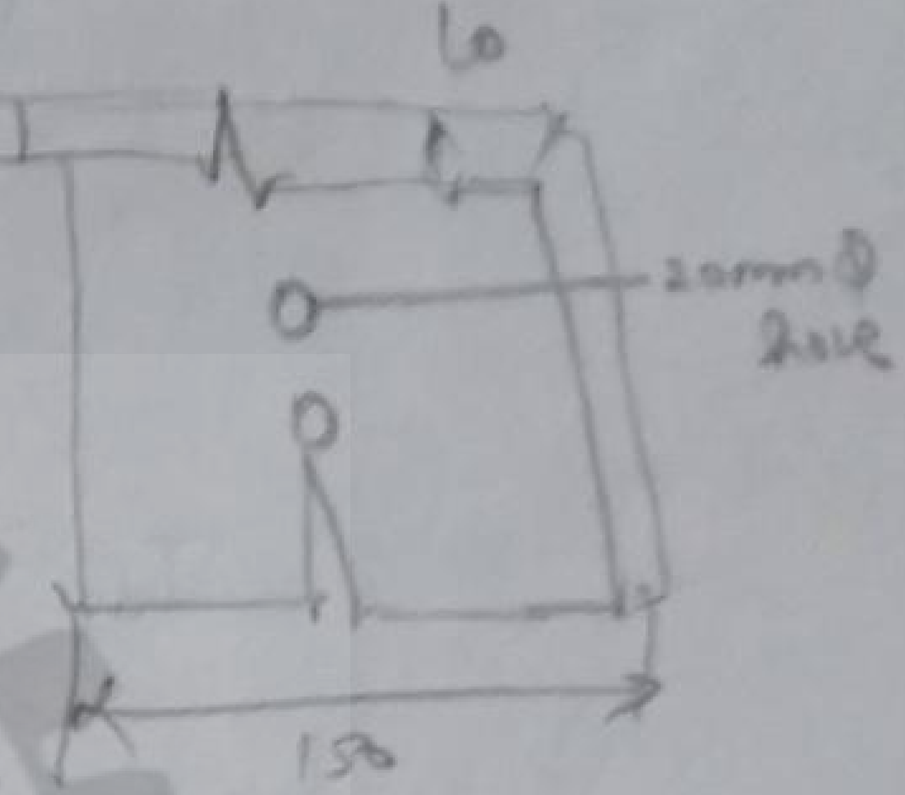
Net sectional Area:

Tension Member-களின் Design-ல் effective area ஆனது net sectional area ஆக எடுக்கப்படுகிறது.

Design of Tension Members:

Determine the Design tensile strength of the steel plate in which 2 holes of 20mm dia are available as shown in fig. width and thickness of steel plate are 150mm and 6mm Respectively. Take $f_y = 250\,N/mm^2$, $f_u = 410\,N/mm^2$

Given Data:



Steel plate with holes

Size of steel plate = 150mm × 6mm

Dia of hole = 20mm

No. of hole = 2

$f_y = 250\,N/mm^2$, $f_u = 410\,N/mm^2$

To find:

Design tensile strength

Solution:

Design strength in Tension Due to yielding of gross section T_{dg}

gross Sectional Area $A_g = 150 \times 6 = 1500\,mm^2$

partial safety factor $\gamma_{m0} = 1.10$

Design Strength in Tension $T_{dg} = \dfrac{A_g \times f_y}{\gamma_{m0}}$

$= \dfrac{1500 \times 250}{1.10}$ $= 340.90\,kN$

Design strength in Tension Due to Rupture of critical section

Net Area of section $A_n = (b - nd) \times t$

$= (150 - 2 \times 20) \times 6$

$= 1100\,mm^2$

partial safety factor $\gamma_{m1} = 1.25$

Design strength in tension $T_{dn} = \frac{0.9 \times A_{n} \times f_{u}}{\gamma_{m1}}$

$= \frac{0.9 \times 1100 \times 410}{1.25}$

$T_{dn} = 324.720 \, kN$

$T_{dg} < T_{dn}$

Lesser of the above two values T_{dg}, T_{dn}

$T_{d} = 324.720 \, kN$

Result:

Design strength in tension $T_{d} = 324.720 \, kN$

2. An ISA 80 x 50 x 10 mm is to be connected at its ends by fillet welds through one leg along three sides and used as a tension member. The thickness of gusset plate is 8mm. Yield strength and ultimate strength of the material is 250mpa and 410 mpa Respectively. length of weld along the length direction is 110mm. Determine the strength of the member if it is connected through a) its longer leg b) its shorter leg

Given Data:

ISA 80 x 50 x 10 mm

$f_y = 250$ mpa

$f_u = 410$ mpa

Thickness of gusset plate = 8mm

To Determine:

Strength of the member

Solution:

Section properties and Design constants

$A_g = 1202\ mm^2$

Partial safety factor $\gamma_{m0} = 1.10$, $\gamma_{m1} = 1.25$

Design strength in Tension Due to yielding of gross area

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1202 \times 250}{1.10} = 273.181\ kN$$

Design strength in Tension Due to Rupture of critical section

(i) when long leg is connected:

Length of outstanding leg $w = 80mm$.

Shear leg Distance $b_s = 50mm$

Length of end connection $L_c = 110mm$

Net Area of connected leg $A_{nc} = (b - t/2) \times t$

$= (80 - 10/2) \times 10 = 750mm^2$

gross area outstanding leg $A_{go} = (b - t/2) \times t$

$= (50 - 10/2) \times 10 = 450mm^2$

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} \geq 0.7$$

$$\beta = 1.4 - 0.076 \left(\frac{50}{10}\right)\left(\frac{250}{410}\right)\left(\frac{50}{110}\right) \leq \frac{410 \times 1.10}{250 \times 1.25} \geq 0.7$$

$\beta = 1.295 \leq 1.443 \geq 0.7 = 1.295$

Scanned by TapScanner

$$T_{dn} = \frac{0.9\, A_{nc}\, f_u}{\gamma_{m1}} + \frac{\beta \times A_{go} \times f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 750 \times 410}{1.25} + \frac{1.298 \times 450 \times 250}{1.10}$$

$T_{dn} = 353.843$ kN

iii) When Connected by short legs

$w = b_{s2} = 80$ mm $A_{nc} = 450\ mm^2$, $A_{go} = 750\ mm^2$

$$\beta = 1.4 - 0.076 \left(\frac{80}{10}\right)\left(\frac{250}{410}\right)\left(\frac{80}{110}\right) \leq \left(\frac{410 \times 410}{250 \times 1.25}\right) \geq 0.7$$

$\beta = 1.130 \leq 1.443 \geq 0.7$

$$T_{dn} = T_{dn} = \frac{0.9 \times A_{nc} \times f_u}{\gamma_{m1}} + \frac{\beta \times A_{go} \times f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 450 \times 410}{1.25} + \frac{1.130 \times 750 \times 250}{1.10}$$

$T_{dn} = 352.45$ kN

Design Strength in Tension Due to Block Shear

$A_{vg} = A_{vn} = 2 \times L_c \times$ plate thickness

$= 2 \times 110 \times 8 = 1760\ mm^2$

$A_{tg} = A_{tn} = 50 \times 8$

$= 400\ mm^2$

Scanned by TapScanner

$$T_{db} = \frac{A_{vg} \times f_y}{\sqrt{3} \times \gamma_{m0}} + \frac{0.90 \times A_{tn} \times f_u}{\gamma_{m1}}$$

$$= \frac{1760 \times 250}{\sqrt{3} \times 1.10} + \frac{0.90 \times 400 \times 410}{1.25}$$

$= 349.020$ kN

$$T_{db} = \frac{0.90 \times A_{vn} \times f_u}{\sqrt{3} \times \gamma_{m1}} + \frac{A_{tg} \times f_y}{\gamma_{m0}}$$

$$= \frac{0.90 \times 1760 \times 410}{\sqrt{3} \times 1.25} + \frac{400 \times 250}{1.10}$$

$= 390.873$ kN

Take lesser value $T_{db} = 349.020$ kN

Result:

Design strength in Tension $T_d = 273.18$ kN.

Design a single angle tension member to carry an axial load of 155 kN. If the yield stress of steel is 300 mpa and ultimate stress is 440 mpa. The angle is to be connected to a gusset plate through one of its leg by fillet welding.

Given Data:

Axial load $= 155$ kN

Yield stress $f_y = 300$ mpa

Ultimate stress $= 440$ mpa

To Design:

A single angle tension member

Solution:

Design load

Partial safety factor for Loads $\gamma_f = 1.5$

Design Load $T = \gamma_f \times$ Axial load

$= 1.5 \times 155 = 232.5$ kN

Selection of section

Assume the permissible Design tensile stress on gross area as $0.9\ f_y$

$f_{td} = 0.9 \times 300 = 270\ N/mm^2$

Approximate gross area required $= \frac{T}{f_{td}} = \frac{232.5 \times 10^3}{270}$

$= 861.11\ mm^2$

choose ISA $75 \times 50 \times 8$ mm, gross area of the section $A_g = 938\ mm^2$

Check for Design strength in tension due to yielding of gross area T_{dg}

Design strength in Tension $T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{938 \times 300}{1.10}$

$= 255.82$ kN

255.82 kN > 155 kN

Hence ok.

Design strength in tension due to Rupture of critical section T_{dn}

Assume the section is connected through its long leg

Net area of Connected leg $A_{nc} = (l - t/2) \times t$

$= (75 - 8/2) \times 8$

$= 568 \ mm^2$

gross area Outstanding leg $A_{go} = (b - t/2) \times t$

$= \left(50 - \frac{8}{2}\right) \times 8$

$= 368 \ mm^2$

Assume length of end Connection $L_c = 90mm$

Shear lag Distance $b_s = w = 50mm$

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \left(\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}\right)$$

$$\beta = 1.4 - 0.076 \left(\frac{50}{8}\right)\left(\frac{300}{440}\right)\left(\frac{50}{90}\right) \leq \left(\frac{440 \times 1.10}{300 \times 1.25}\right) \geq 0.7$$

$\beta = 1.22 \leq 1.29 \geq 0.7$, $\beta = 1.22$

$$T_{dn} = \frac{0.9 \times A_{nc} \times f_u}{\gamma_{m1}} + \frac{\beta \times A_{go} \times f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 568 \times 440}{1.25} + \frac{1.22 \times 368 \times 300}{1.10}$$

$T_{dn} = 302.39 \ kN$

Design strength of member T_d = least of T_{dg} and T_{dn}

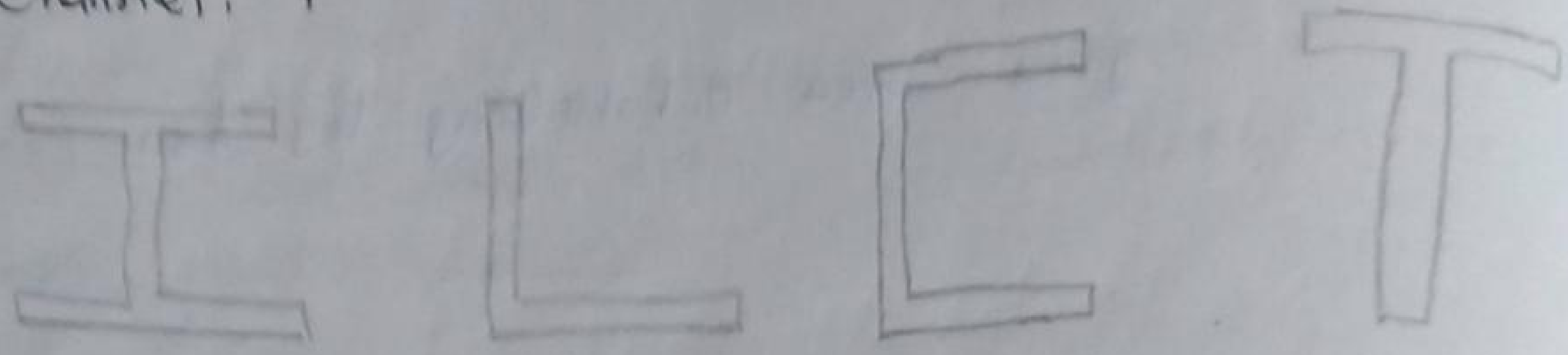
$T_d = 255.82 \ kN > 155 \ kN$

Result:

Provide ISA $75 \times 50 \times 8$ mm

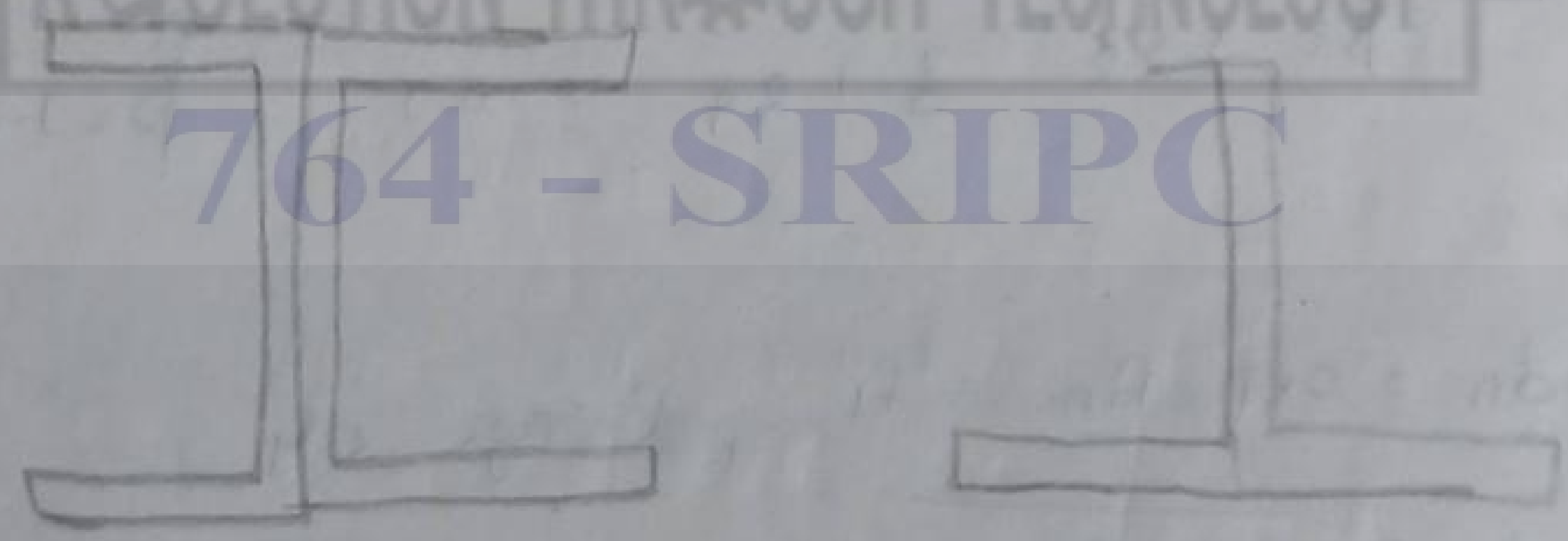
Different forms of compression members:

1. Single Rolled steel sections such as I, Equal (or) unequal angle, Channel, T.



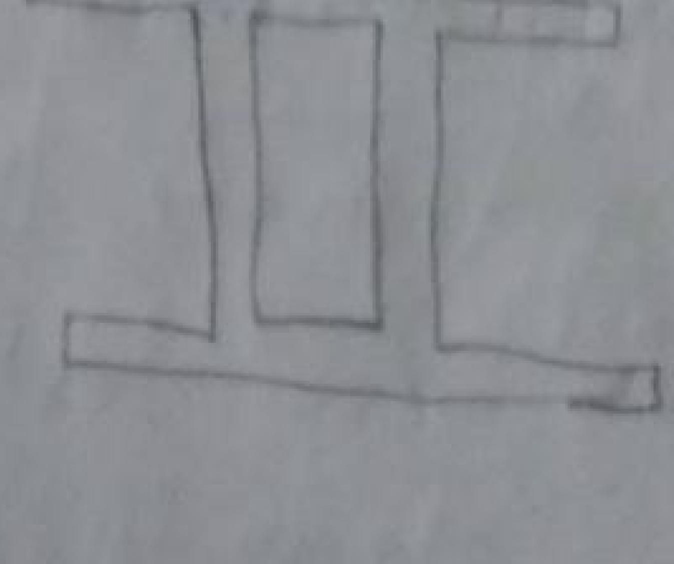
2. Hot Rolled Rectangular / circular hollow section

3. Built-up sections made up of Two (or) more Rolled section



4. Built up section with Rolled sections and plates

5. Sections with plates only



Determine the compressive strength of ISHB @ 925 N/m used as a Column for a height of 6m. Both ends are Column are effectively held in position and restrained against Rotation. Take $f_y = 250$ mpa.

Given Data:

ISHB 450 @ 925 N/m

Height of Column $L = 6m$

End Condition: Both ends are effectively held in position and restrained against rotation, $f_y = 250$mpa

To find:

Compressive strength

Solution:

Sectional properties of ISHB 450 @ 925 N/m

$A = 11789\ mm^2$ D lon $h = 450mm$ $b_f = 250mm$

$t_f = 13.70\ mm$, $t_w = 11.30\ mm$, $r_z = 185\ mm$ $r_y = 50.9mm$

Classification of section

$$\frac{b}{t_f} = \frac{b_f/2}{t_f} = \frac{250/2}{13.70} = 9.12$$

$$9.4\ \varepsilon = 9.4 \times \sqrt{\frac{250}{250}} = 9.40$$

Class I semi compact section

$A_g = A_e = 11789\ mm^2$

Classification of buckling class of section

$$\frac{h}{b_f} = \frac{450}{250} = 1.80 > 1.2$$

$t_f = 13.70mm < 40\ mm$

$\frac{h}{b_f}$ 1.2 and $t_f < 40mm$

Design compressive stress fcd:

Effective length of column $KL = 0.65 \times l$

$= 0.65 \times 6 = 3.90m$

$= 3900mm$

Effective slenderness Ratio about major axis (a) Zz axis $= \frac{KL}{r_z} = \frac{3900}{185} = 21.08$

Effective slenderness Ratio about major axis (b) yy axis $= \frac{KL}{r_z} = \frac{3900}{50.80}$

$= 76.77$

Design compressive stress fcd considering the major axis ZZ axis

for $\frac{KL}{r_y}$ 21.08 & $f_y = 250$ mpa

$$fcd = 226 - \left[\frac{226 - 220}{(40 - 20)} \times (21.08 - 20)\right]$$

$= 225.352\ N/mm^2$

Design compressive stress fcd considering the minor axis (or) yy axis

for $\frac{KL}{r_z} = 76.77$ & $f_y = 250$ mpa

$$fcd = 166 - \left[\frac{(166 - 150)}{(80 - 70)} \times (76.77 - 70)\right] = 155.168\ N/mm^2$$

fcd lesser value is taken

fcd = 155.168 N/mm²

Design Compressive Strength $P_d = A_e \times f_{cd}$

$= 11789 \times 155.168$

$= 1829.276$ kN

Result:

Design Compressive Strength $P_d = 1829.276$ kN

5.2 Design of Simple Beams and Welded Connections by LSM

Simple Beams:

Beam என்பது ஒரு Structural member ஆகும். இவை transverse load களை உடையன்று.

Classification of steel beams:

Beams according to the function it performs

(i) purlin

(ii) Common Rafter

(iii) Lintel

(iv) Joists

(v) Shaft

Beams according to shapes

Rolled steel section களான

(i) I section

(ii) channels

(iii) angles

(iv) Tee section

(v) H-section

Effective span:

நீளத்தில் எடுக்கப்படும் Beamன் effective span என்பது Support களின் மையத்திற்கு இடையே உள்ள தூரமாகும்.

Plastic Moment of Resistance M_p:

It is the moment of Resistance of the Cross Section where the Bending Stress Reaches the yield Stress Value all over the Section.

Plastic Section Modulus Z_p

It is the Ratio b/w Plastic Moment of Resistance and the yield stress of the material.

$$Z_p = \frac{M_p}{f_y}$$

Shape factor:

It is the Ratio between the plastic moment of Resistance and the elastic moment of Resistance of the section

$$\text{Shape factor} = \frac{Z_p}{Z_e}$$

An ISLB 450 @ 653 N/m is to be used as a Simple beam on a span of 6m. Find the maximum udl the beam Can Carry if $f_y = 250$ mpa and $E = 2\times10^5$ N/mm^2

Given Data:

ISLB 450 @ 653 N/m

Span $l = 6$m

$f_y = 250$ Mpa

$E = 2\times10^5$ N/mm^2

To find

Maximum udl of the beam Can Carry

Solution:

Sectional properties

For ISLB 450 @ 653 N/m

$A = 8314$ mm^2, $D = 450$ mm, $b_f = 170$mm, $t_f = 13.4$ mm

$t_w = 8.6$ m, $I_{xx} = 275.36 \times 10^6$ mm^4, $Z_e = 1.2238 \times 10^6$ mm^3

$Z_p = 1.40 \times 10^6\ mm^3$

Limiting width to thickness Ratio

ISLB 450 @ 653 N/m

$b = b_f/2 = \frac{170}{2} = 85\ mm$

$b/t_f = \frac{85}{13.40} = 6.34$

Yield Stress Ratio $\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$

$9.4\ \varepsilon = 9.4 \times 1 = 9.4$

$b/t_f = 6.34 < 9.4$

The section is classified as class I plastic section

Design Bending strength Md

for plastic section $\beta_b = 1$

Design Bending strength $M_d = \frac{\beta_b \times Z_p \times f_y}{\gamma_{mo}}$

$= \frac{1 \times 1.40135 \times 10^6 \times 250}{1.10}$

$= 318.489 \times 10^6\ N.mm$

$M_{d\,max} = \frac{1.2 \times Z_e f_y}{\gamma_{mo}} = \frac{1.2 \times 1.22 \times 10^6 \times 250}{1.10}$

$= 333.764 \times 10^6\ N.mm$

Maximum Load with Respect to Bending Strength

Design BM at mid span $M_u = \frac{W_u \times l^2}{8}$

$= \frac{W_u \times 6^2}{8}$

$= 318.489$

$W_u = 70.78$ kN/m

Safe Load for the beam $W = \frac{70.78}{1.50}$

$= 47.19$ kN/m

Self wt of the beam $= 653$ N/m $= 0.653$ kN/m

Safe Imposed Load on beam

$= 47.19 - 0.653 = 46.537$ kN/m

Maximum Load Respect to Design Shear Strength

$t_w = 8.60$ mm

$d = h - 2 \times t_f = 450 - 2 \times 13.40 = 423.20$ mm

$\frac{d}{67\varepsilon} = \frac{423.20}{67 \times 1} = 6.31$

$t_w = 8.60$ mm $> \frac{d}{67\varepsilon}$

Design shear strength $V_d = \frac{A_v \times f_{yw}}{1.10 \times \sqrt{3}}$

Shear Area $A_v = h \times t_w = 450 \times 8.60 = 3870$ mm^2

$V_d = \frac{3870 \times 250}{1.10 \times \sqrt{3}} = 507.805 \times 10^3$ N

$= 507.805$ kN

Max Design shear force for a simply supported beam with udl $V_d = \frac{w_u l}{2}$

Equating $V_d = 507.808$ kN $= \frac{w_u l}{2}$

$W_u = 169.268$ kN/m

Safe load for the beam $w = \frac{169.268}{1.50} = 112.84$ kN/m

Self wt of beam $= 653$ N/m $= 0.653$ kN/m

Safe imposed load on beam $=$ safe load $-$ selfweight

$= 112.84 - 0.653$

$= 112.187$ kN/m

Maximum load with respect to stiffness

max deflection $= \frac{\text{Span}}{300} = \frac{6000}{300} = 20$ mm

Max deflection for a simply supported beam with udl throughout its length $\delta_{max} = \frac{5}{384} \times \frac{wl^4}{EI}$

Equating the max deflection to 20 mm

$$\frac{5}{384} \times \frac{w \times 6000^4}{2 \times 10^5 \times 275.36) \times 10^6} = 20$$

$= 65.27$ kN/m

Safe load on beam $w = 65.27 -$ self wt

$= 65.27 - 0.653$

$= 64.61$ kN/m

Result:

maximum udl beam can carry $w = 46.537$ kN/m

An ISMB 350 @ 524 N/m is to be used as a simply supported beam to carry a super imposed load of 24 kN/m. effective span of the beam is 6.5m, f_y = 300 mpa and E = 2×10^5, check the safety of the beam.

given Data:

ISMB 350 @524 N/m

Super Imposed load = 24 kN/m

Effective Span (l) = 6.5 m

f_y = 300 mpa

E = 2×10^5 N/mm^2

To find:

Safety of beam

Solution:

Sectional properties: ISMB 350 @ 524 N/m

A = 66.70 cm^2 = 6671 mm^2

D = 350 mm, t_f = 14.2mm, t_w = 8.1mm,

b_f = 140 mm

Z_e = 779×10^3 N/mm^3, Z_p = 889.2×10^3 N/mm^3

I_{zz} = 13630.3×10^4 N/mm^4

Limiting width to thickness Ratio

$b = b_f/2 = \frac{140}{2} = 70$

$b/t_f \quad \frac{70}{14.2} = 4.93$

Yield Stress Ratio $\varepsilon = \sqrt{\frac{250}{f_y}}$

$= \sqrt{\frac{250}{300}}$

$= 0.91$

$9.4\,\varepsilon = 9.4 \times 0.91 = 8.55$

Scanned by TapScanner

$b/t_f = 4.93 < 9.4\varepsilon$

The Section is classified as class I plastic Section

Check for Safety against Bending:

for plastic Section $\beta_b = 1$

Design Bending Strength $M_d = \frac{\beta_b \times Z_p \times f_y}{\gamma_{m0}}$

$= \frac{1 \times 889.57 \times 10^3 \times 300}{1.10}$

$= 242.61 \times 10^6$ N.mm

$M_{dmax} = \frac{1.2 \times Z_e \times f_y}{\gamma_{m0}} = \frac{1.2 \times 779 \times 10^3 \times 300}{1.10}$

$= 254.945$ kN.m

Design Bm on Beam $M_u = \frac{W_u \times l^2}{8}$

$W_u = 1.5\ [24 + \text{Self wt})$

$= 1.5\ (24 + 0.524)$

$W_u = 36.786$ kN.m

$M_u = \frac{36.786 \times 6.5^2}{8} = 194.276$ kN.m

$M_u = 194.276$ kN.m < 242.61 kN.m

Design Bending Strength of the Beam is greater than the Design Bending Moment of he beam is safe in Bending

Check for safety against Shear $d = D - 2 \times t_f = 350 - 2 \times 14.2$

$t_w = 8.1$ mm

$D = 321.60$ mm

$$\frac{d}{67\varepsilon} = \frac{d}{67\varepsilon} = \frac{326.60}{67\times0.91} = 5.27$$

$$t_w > \frac{d}{67\varepsilon}$$

Design shear strength $V_d = \dfrac{A_v \times f_{yw}}{1.10\times\sqrt{3}}$

Lon

$A_v = h \times t_w = 350 \times 8.1 = 2835\ mm^2$

$$V_d = \frac{2835 \times 300}{1.10 \times \sqrt{3}} = 446.396 \times 10^3\ N$$

Max Design shear force on the Beam $= \dfrac{w_u \times l}{2}$

$$= \frac{36.78 \times 6.5}{2}$$

$= 119.55$ kN.m

$119.55 < 446.396$ kN.m

Design shear strength of the Beam is greater than the Max. Design shear force of the Beam is safe in shear.

Check for safety against deflection:

Max permitted Deflection $= \dfrac{span}{300} = \dfrac{6500}{300} = 21.67\ mm$

Max. Deflection for a simply supported beam $= \dfrac{5}{384} \times \dfrac{wl^4}{EI}$

$w = 24 + 0.524 = 24.524$ kN/m

$$= \frac{5}{384} \times \frac{24.524 \times 6000^4}{2 \times 10^5 \times 13630.3 \times 10^4}$$

$$= 20.90\,mm < 21.67\,mm$$

Max deflection is less than the max permitted deflection the Beam is safe in Deflection.

Result:

The Beam is safe in Bending shear and Deflection

Welds:

Welding என்பது Structural Member களை இணைப்பதற்கு பயன்படும் ஒரு வகை இணைப்பாகும்.

Types of welds:

1. Member ன் மேல் weld ன் நிலையை பொறுத்து வகைப்படுத்தப்படுகிறது.

   a) Flat (b) overhead (c) கிடைமட்ட d) செங்குத்து

2. Weld ன் வடிவமைப்பு பொறுத்து

   a) Butt weld b) fillet weld c) slot weld d) plug weld

3. இணைப்புகளின் வடிவமைப்பு பொறுத்து

   a) Butt joint b) Lap joint c) Tee joint d) Edge joint

   e) Corner joint

4. Butt weld ன் உள்ள edge Machining முறையை பொறுத்து

   1) Square groove weld 2) Single Vee groove weld

   3) Double Vee groove weld 4) Single U groove weld

   5) Double U groove weld 6) Single J groove weld

   7) Double J groove weld

An ISMC 250 @ 304 N/m is to be used as a Simple Beam with its Major axis as the axis of Bending. Determine the capacity of the Beam to carry udl. If the Span is 4m

$f_y = 340$ mpa.

Given Data:

ISMC 250 @ 304 N/m, Span $l = 4m$, $f_y = 340$ mpa

To find:

Determine the capacity of the beam to carry udl

Solution:

Sectional properties:

$A = 38.67\ cm^2 = 3867\ mm^2$

$D = 250\ mm$ $b_f = 80\ mm$, $t_f = 14.1\ mm$, $t_w = 7.1\ mm$

$Z_e = 305.3\ cm^3 = 305.3 \times 10^3\ mm^3$

$Z_p = 356.72\ cm^3 = 356.62 \times 10^3\ mm^3$, $I_{zz} = 38.16 \times 10^4\ mm^4$

$b/t_f < 9.4\varepsilon$.

It is class 1 plastic section.

$$M_d = \frac{\beta_b \times Z_p \times f_y}{\gamma_{m0}} = 110.259\ kNm$$

$$M_{d\,max} = \frac{1.2 \times Z_e \times f_y}{\gamma_{m0}} = 113.233\ kNm$$

$M_d < M_{d\,max}$

Hence the Md value is taken

Result:

$$W = \frac{Mu}{l^2} \times 8 = 55.13\ kNm$$

$W = 36.465\ kN/m$