MOHAMED SATHAK AJ COLLEGE OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

CE8703 STRUCTURAL DESIGN AND DRAWING

(VII SEMESTER R-2017)

COURSE MATERIAL

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UNIT I

RETAINING WALL

Reinforced concrete cantilever and counter fort retaining wall - Horizontal backfill with surcharge - Design of shear key -Design and drawing

Course outcome : Design and draw Reinforced concrete cantilever and counter fort retaining wall

Design for cantilever Retaining wall

Example 1 :

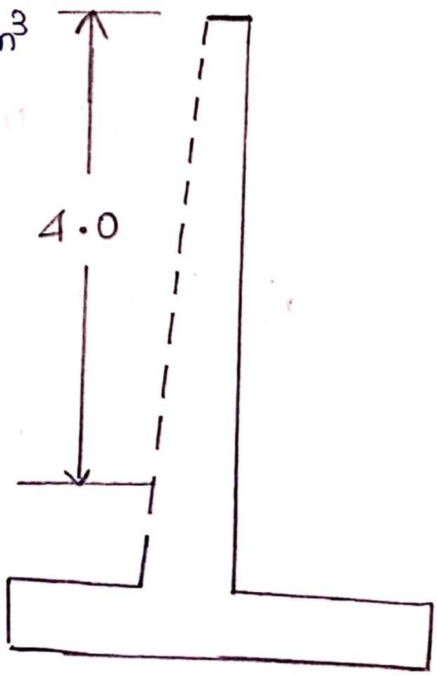
Design a cantilever retaining wall to retain an earth embankment with a horizontal top 4m above ground level. Density of earth = 18 kN/m^3. Angle of internal friction ϕ = 30 degree. SBC of Soil = 200 kN/m^2. coefficient of friction between soil and concrete = 0.5 Adopt M20 grade concrete and Fe415 HYSD bars.

Given data :

Density of earth 'γ' = 18 kN/m^3

Angle of Internal friction ϕ = 30

SBC of soil 'q' = 200 kN/m^2



Step 1 :

(a) Depth of foundation $= q/\gamma \, (1 - \sin\phi / 1+\sin\phi)^2$

$= 200/18 \, (1 - \sin 30 / 1 + \sin 30)^2$

$= 1.2m$

b) Overall depth of wall $= 4 + 1.2$

$H = 5.2m$

$= 5200mm$

c) Thickness of base slab

$\frac{H}{12} = \frac{5200}{12}$

433mm ~ 450mm

d) Height of stem 'h' $= 5200 - 450$

$= 4750mm$

$= 4.75 m$

e) width of base slab

'b' = 0.5H to 0.6H

= 2600 to 3120

= 3000mm

Step 2 : Design of system

a) Max BM at base

'M' = ka ($\gamma h^3/6$)

ie ka = (1 − sin ϕ / 1+ sin ϕ)

= (1/3) (18 x 4.75^3 /6)

= 107.2 kNm

Factored moment 'Mu' = 107.2 x 1.5

= 161 kNm

= 161×10^6 Nmm.

b) Effective depth required

ie) $b = 1000mm$

$= 10^3 mm$

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

$$d = \sqrt{\frac{Mu}{0.138 \times f_{ck} \times b}}$$

$$d = \sqrt{\frac{161 \times 10^6}{0.138 \times 20 \times 10^3}}$$

$d = 241.5 \sim 242 mm$

b) Effective depth at base of Stem

over all depth 'D' = 450mm

cover = 50mm

effective depth 'd' = D - 50

'd' = 450 - 50

= 400mm

c) Find Ast

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

page no: 96 IS 456.2000

$$161 \times 10^6 = (0.87 \times 415 \times A_{st} \times 400)$$

$$\left[\left(1 - 415 \times A_{st}\right)/(1000 \times 400 \times 20)\right]$$

$$161 \times 10^6 = (144.42 \times 10^3 A_{st})$$

$$\left[1 - 5.187 \times 10^{-5} A_{st})\right]$$

$$161 \times 10^6 = (144.42 \times 10^3 A_{st}) - (7.49 A_{st}^2)$$

$$161 \times 10^6 = (144.42 \times 10^3 A_{st}) + (7.49 A_{st}^2) = 0$$

using calculator) mode > Eqn > degree > 2

$a = 7.49 \quad b = -144.42 \times 10^3 \quad c = 161 \times 10^{6}$

$x_1 = 18093 \text{ mm}^2 \quad x_2 = 1188 \text{ mm}^2$

$A_{st} = 1188 \text{ mm}^2$

Find Spacing

Provide 16mm dia bars

Spacing $= 1000 \times [(\pi d^2/4)/Ast]$

$= 1000 \times [\pi \times 16^2/4)/1188]$

$= 169.24 \sim 170mm$

Provide 16mm dia bars at 170mm c/c

Find distribution reinforcement.

Ast (dist) $= (0.12/100) \times bD$

$= (0.12/100) \times 1000 \times 450$

$= 540mm^2$

Provide 10mm dia bars at 145mm c/c

Spacing $= 1000 \times (\pi d^2/4)/Ast$

$= 1000 \times [(\pi \times 10^2/4)/540]$

$= 145mm.$

Provide 10mm dia. bars at 145mm C/c

Provide 10mm dia bars at 290mm C/c on both faces.

Step 3 :

Stabillity calculation.

a) Find load.

$W_1 = (b \times d \times \gamma_c) + (1/2 \times bh \times \gamma_c)$

$= (0.2 \times 4.75 \times 24) + (1/2 \times 0.25 \times 4.75 \times 24)$

$= 22.80 + 14.25$

$= 37.05$ kN.

$W_2 = b \times d \times \gamma_c$

$= 3 \times 0.45 \times 24$

$= 32.40$ kN.

$W_3 = b \times d \times \gamma_s$

$= 1.55 \times 4.75 \times 18$

$= 132.50\ kN$

Total load $= W_1 + W_2 + W_3$

$= 201.95\ KN$

b) Find moment @ a

$M_1 = W_1 \times$ length

$= (22.80 \times 1.65) + (14.25 \times 1.83)$

$= 37.62 + 26.07$

$= 63.69\ kNm.$

$M_2 = W_2 \times$ length.

$= 32.40 \times 1.5 = 48.60\ kNm$

$M_3 = W_3 \times$ length

$132.50 \times 0.78 = 103.35\ kNm$

$M_4 = 107.2$ kNm (moment at base)

Total moment

$M = M_1 + M_2 + M_3 + M_4$

$= 322.81$ kNm.

point of application @ a

$$Z = \frac{\Sigma M}{\Sigma W} = \frac{322.81}{201.95}$$

$Z = 1.6$ m

Eccentricity

$e = (Z - b/2) \quad e < b/6$

$e = (1.6 - 3/2) = 0.1$ m

$(b/6) = (3/6) = 0.5$

$e < b/6$

Maximum and minimum pressures at the base.

$$\sigma_{\substack{max \\ min}} = \frac{\Sigma w}{b}\left(1 \pm \frac{be}{b}\right)$$

$$= \frac{201.95}{3}\left(1 \pm \frac{6 \times 0.1}{3}\right)$$

$$\sigma_{max} = 67.32 \times (1 + 0.2)$$

$$= 80.76 \ kN/m^2$$

$$\sigma_{min} = 67.32 \times (1 - 0.2)$$

$$\sigma_{min} = 53.84 \ kN/m^2$$

Step 4 :

Design of heel Slab

Find load :

w_1 Self weight = L x B x D x unit weight concrete

$w_1 = 1.55 \times 4.75 \times 18$

$= 132.5$ kN

w_2 weight of soil = L x B x D x unit wt soil

$w_2 = 1.55 \times 0.45 \times 24$

$= 16.7$ kN

Find moment

$M_1 = w_1 \times$ length.

$M_1 = 132.5 \times 0.775 = 102.68$ kNm

$M_1 = 102.68$ kNm.

$M_2 = W_2 \times \text{length}$

$M_2 = 16.7 \times 0.775 = 12.94$

$M_2 = 12.94$ KNm

total moment $M = M_1 + M_2$

$M = 102.68 + 12.94$

$M = 115.62$ KNm.

Deduction for upward pressure 'abih'

$W_3 = \text{pressure} \times \text{length}$

$= 53.84 \times 1.55$

$W_3 = 83.45$ KN

upward pressure 'ghi"

$W_4 = 1/2\, bh$

$= 1/2 \times 1.55 \times 13.9$

$W_4 = 10.77$ KN.

"abih" moment deduction $= W_3 \times \text{length}$

$= 83.45 \times 0.775$

$= 64.67$ KNm.

"Ghi" moment deduction $= W_1 \times \left(2/3 \times \frac{\text{length}}{2}\right)$

$= 10.77 \times \left(2/3 \times \frac{1.55}{2}\right)$

$= 10.77 \times (0.667 \times 0.775)$

$= 10.77 \times 0.517$

$= 5.56$ kNm.

Total deduction $M_d = 64.67 + 5.56$

$M_d = 70.2$ kNm.

Maximum service BM in heel slab at 'b'

$M = M - M_d$

$= 115.62 - 70.22$

$M = 45.40$ kNm

Factored moment $= M_u \times 1.5$

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

Scanned with CamScanner

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

IS 456.2000

Clause G-1.1

$$(68.1 \times 10^6) = (0.87 \times 415 A_{st} \times 400)\left[1 - \frac{415 A_{st}}{10^3 \times 400 \times 20}\right]$$

$A_{st} = 484 \, mm^2$

Provide 12mm dia bar at 200mm

($A_{st} = 565 \, mm^2$)

Distribution reinforcement =

$(0.0012 \times 1000 \times 450) = 540 mm^2$

Provide 12mm dia bar at 200mm c/c

($A_{st} = 565 \, mm^2$)

Step 5 :

Design of Toe Slab

Find load

w_1 upward pressure "cdjf" = pressure x length

$= 71.78 \times 1 = 71.78$

$= 71.78$ KN

w_2 upward pressure "jfe" = $1/2 \times b \times h$

$= 1/2 \times 1 \times 8.98 = 4.49$

$= 4.49$ KN

Total $= w_1 + w_2$

$W = 71.78 + 4.49$

$W = 76.27$ KN.

Deduction

w_3 self weight of toe slab

$= L \times B \times D \times$ unit wt of concrete

$= 1 \times 1 \times 0.45 \times 24$

$= 10.8$ KN.

W_4 Self wt of soil $= L \times B \times D \times \gamma_s$

$= 0.75 \times 1 \times 18 = 13.5$

$= 13.5$ KN.

Moment :

$M_1 = w_1 \times$ length.

$M_1 = 71.78 \times 0.5$

$M_1 = 35.89$ KNm.

$M_2 = W_2 \times (2/3 \times \text{length}) + \text{length}.$

$= 4.49 \times 2/3$

$M_2 = 3.00$ kNm

$M = M_1 + M_2$

$= 35.89 + 3.00$

$= 38.89$ KNm.

$M_{d1} = w_3 \times \text{length} / 2$

$= 10.8 \times 0.5 = 5.40$ kNm

$M_{d2} = w_4 \times length/2$

$= 13.5 \times 0.5$

$M_{d2} = 6.75$ kNm

$M = M - M_d$

total deduction $M_d = M_{d1} + M_{d2}$

$= 5.40 + 6.75$

$M_d = 12.15$ kNm.

Max load BM in toe slab

$M = M - M_d$

$= 38.89 - 12.15$

$= 26.74$ kNm.

Factored BM $= M_u \times 1.5$

$= 26.74 \times 1.5 = 40.11$ kNm

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

$$40.11 \times 10^6 = (0.87 \times 415\, Ast \times 400)\left[1 - \frac{415\, Ast}{10^3 \times 400 \times 20}\right]$$

$Ast = 275\, mm^2 < Ast$ (minimum)

Hence provide minimum reinforcement of 0.12 percent.

$$Ast\,(\text{minimum}) = (0.0012 \times 1000 \times 450)$$

$$= 540\, mm^2$$

Provide 12mm dia bars at 200 mm c/c

($Ast = 565\, mm^2$)

Distribution reinforcement is the same as in heel slab comprising 12mm dia bar at 200mm c/c

Step 6 :

Check for safety against sliding

Horizontal earth pressure

$$P = Ka \cdot \frac{WH^2}{2}$$

$$P = 1/3 \times 18 \times \frac{5.2^2}{2}$$

$$P = 81.12 \text{ kN}.$$

Assuming co.eff of friction $\mu = 0.5$

max possible frictional force is

$$\mu W = (0.5 \times 201.95)$$

$$= 100.975 \text{ kN}.$$

factor safety against sliding

$$= \frac{\mu W}{P} < 1.5$$

$$= \frac{100.975}{81.12} = 1.25 < 1.5$$

(according to jain and Jai krishna[19], Reynolds and Steedman[20])

Since the wall is unsafe against sliding, a shear key is to be designed below the stem.

Step 7 :

Design of Shear key

Intensity of pressure in shear key front

P_p = kp x pressure in Shear key Front

$$kp = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1}{Ka} = 3$$

where p = soil pressure just in front of shear key = 71.78 kN/m²

P_p = (3 x 71.78) = 215.34 kN/m²

If a = depth of shear key = 450mm

Total passive force $P_p = P_p \cdot a$

= 215.34 x 0.45 x 1

= 97 kN.

Total passive force $P_P = P_p a$

$= 215.34 \times 0.45 \times 1$

$= 97 \text{ KN}$

Therefore factor of safety against sliding is computed as

$$F.S = \left(\frac{\mu W + P_P}{P}\right) = \left(\frac{100.975 + 97}{81.12}\right)$$

$= 2.4 > 1.5$

Minimum percentage of Reinforcement 0.3 %

$= \frac{0.3}{100} \times b \times d$

Mini $= (0.003 \times 450 \times 1000)$

$= 1350 \text{ mm}^2.$

Provide 16mm dia bar at 140mm c/c

Step 8 :

Shear stress at junction

Networking shear force = $V = (1.5 \Sigma P - \mu W)$

$= (1.5 \times 81.12) - 100.975$

$V = 20.7 \, KN.$

Factored shear force = V_u

$V_u = V \times 1.5$

$= 20.7 \times 1.5$

$V_u = 31.05 \, KN$

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

$\tau_v = [(31.05 \times 10^8) / (1000 \times 400)]$

$\tau_v = 0.077 \, N/mm^2$

$\left(\frac{100 \, Ast}{bd}\right) = \left(\frac{100 \times 1350}{1000 \times 400}\right) = 0.335$

IS 456 - 2000 (Table 19)

$\tau_c = 0.40 \ N/mm^2 > 0.077 \ N/mm^2$

Hence shear stresses are within safe permissible ~~shear~~ limits.

The rein forcement details in retaining wall.

Example : 2

Design example (counterfort retaining wall)

Example : 2

Design a counterfort retaining wall based on the following data.

Hight of wall above ground level = 6m

SBC of soil = 160 KN/m^2

Angle of internal friction ϕ = 33°

Density of soil = 16 KN/m^3

Spacing of counterforts = 3 m c/c

Adopt M20 grade concrete and Fe 415 HYSD bars.

Soln

Step 1 :

Diamensions of retaining wall

minimum depth of foundation $= \frac{P}{w}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$

$= \frac{160}{16}\left(\frac{1}{3}\right)^2$

$= 1.11 m$

Provide depth of foundation = 1.2m

∴ overall height of wall $H = (6+1.2)$

$H = 7.2m$

Thickness of base slab = 2LH cm

$= 2 \times 3 \times 7.2$

= 43.2 cm

provide 450mm thick base slab

Base width = 0.6H to 0.7H

(0.6×7.2) = 4.32 m

(0.7×7.2) = 5.04 m.

Adopt base width = 4.5 m

Toe projection = $(1/4 \times 4.5)$

= 1.1 m

Step 2 :

Design of Stem

pressure intensity at base $= wh\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$

where $h = (7.2 - 0.45)$

$= 6.75m$

pressure intensity $= \left(16 \times 6.75 \times \frac{1}{3}\right)$

$= 36\ kN/m^2$

maximum working moment =

$= \left(\frac{36 \times 3^2}{12}\right) = 27\ kNm.$

Factored moment $= M_u = (1.5 \times 27)$

$= 40.5\ kNm.$

Effective depth required for balanced section is

$$d = \sqrt{\frac{M_u}{(0.138 f_{ck} b)}} = \sqrt{\frac{40.5 \times 10^6}{(0.138 \times 20 \times 10^3)}}$$

$= 121mm.$

adopt an overall thickness of 220 mm

con-stant up to the top.

Effective depth = d = 175 mm

The reinforcement in the stem are computed

using the relation.

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

IS: 456.2000

clause G-1.1

$$(40.6 \times 10^6) = (0.87 \times 415 A_{st} \times 175)$$

$$\left[1 - \frac{415 A_{st}}{10^3 \times 175 \times 20)}\right]$$

$A_{st} = 700 mm^2$.

provide 12mm dia bar at 150mm c/c

($A_{st} = 754 mm^2$)

Distribution reinforcement =

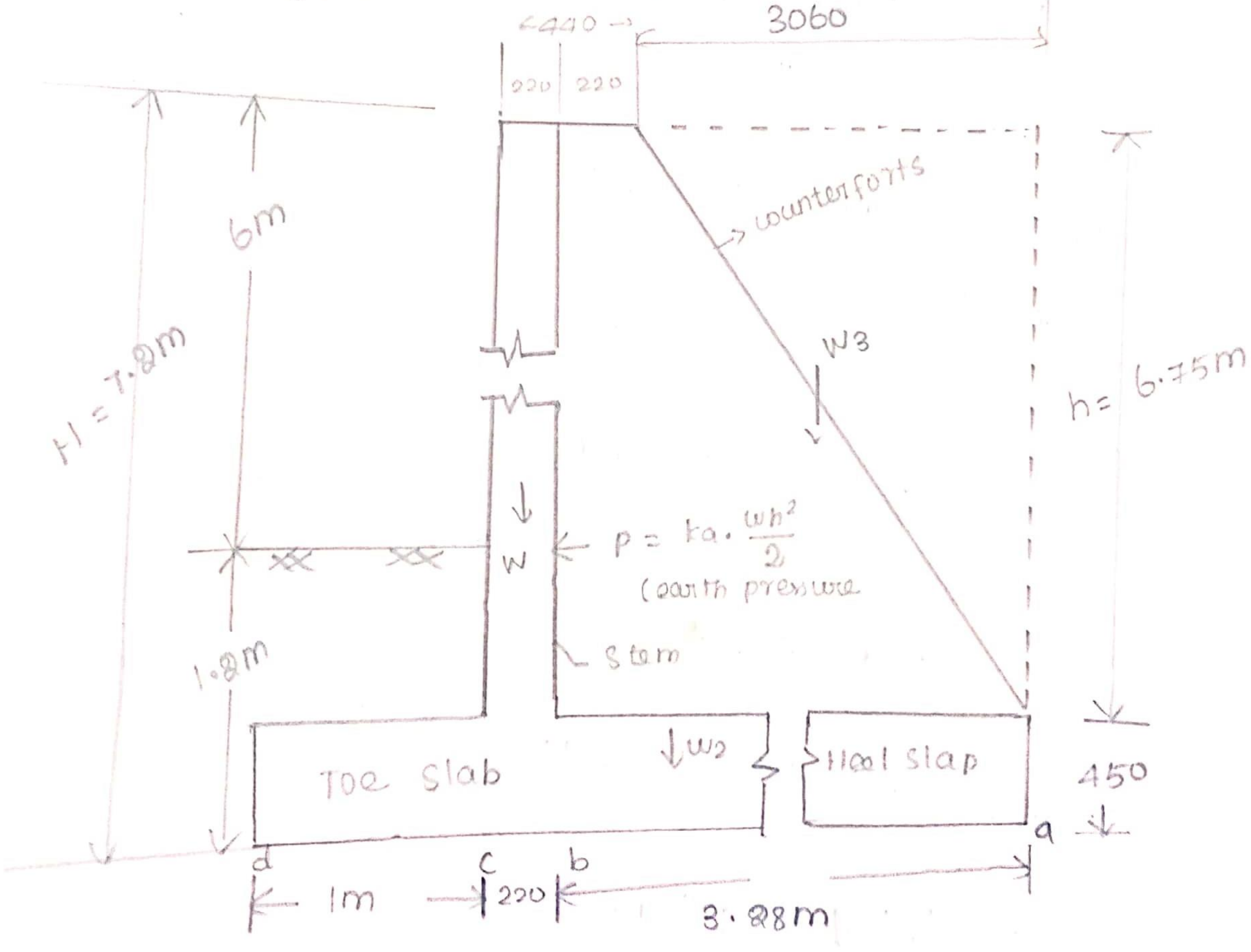
$= 0.12$ percent of section $= \frac{0.12}{100} \times b \times d$

$= (0.0012 \times 220 \times 100)$

$= 264 \text{ mm}^2/\text{m}$

Adopt 6mm dia bars at 200m c/c

($A_{st} = 283 \text{mm}^2$)



counterfor retaining wall - overall dimensions.

Step : 3

Stability calculations.

Find load :

$w_1 = b \times d \times \gamma_c$

$= 0.22 \times 6.75 \times 24$

$= 35.64$ kN.

$w_2 = b \times d \times \gamma_c$

$= 0.45 \times 4.5 \times 24$

$= 48.60$ kN

$w_3 = b \times d \times \gamma_s$

$= 3.28 \times 6.75 \times 16$

$= 354.24$ kN.

Total $= w_1 + w_2 + w_3$.

$= 35.64 + 48.60 + 354.24$

$= 438.49$ kN.

b) Find moment

$M_1 = w_1 \times \text{length}.$

$= 35.64 \times 3.39$

$= 120.80 \text{ KNm}$

$M_2 = w_2 \times \text{length}$

$= 48.60 \times 2.25$

$= 109.35 \text{ KNm}$

$M_3 = w_3 \times \text{length}$

$= 354.24 \times 1.64$

$= 580.95 \text{ KNm}$

M_4 = moment of earth pressure

$Ka = \frac{wh^3}{6} \Rightarrow \frac{1}{3} = \frac{16 \times 7.2^3}{6}.$

$= 331.77$

$M_4 = 331.77$ (moment at base)

Total moment

$$M = M_1 + M_2 + M_3 + M_4$$

$$M = 120.80 + 109.35 + 580.95 + 331.77$$

$$M = 1142.87 \text{ kNm.}$$

Distance of the point of application of the resultant from point a is

$$z = \frac{\Sigma M}{\Sigma W} = \left(\frac{1142.87}{438.49}\right)$$

$$= 2.66 m.$$

Eccentricity $e = (z - b/2)$

$$= (2.66 - 4.5/2) = 0.41 m$$

but $(b/6) = \left(\frac{4.5}{6}\right) = 0.75 m$

$$e < (b/6)$$

maximum and minimum pressure at the base are given by.

$$\sigma_{max/min} = \frac{\Sigma W}{b}\left[1 \pm \frac{6e}{b}\right]$$

$$\sigma_{max} = \frac{438.49}{4.5}\left[1 + \frac{6 \times 0.41}{4.5}\right]$$

$$= 150\ kN/m^2$$

$$\sigma_{min} = \frac{438.49}{4.5}\left[1 - \frac{6 \times 0.41}{4.5}\right]$$

$$= 45\ kN/m^2$$

The maximum intensity of pressure does not exceed the ~~pre~~ permissible value of 160 kN/m²

Step 4 : Design of toe slab.

W_1 = LBD unit wt of concrete

$W_1 = 126.6 \times 1 = 126.6\ kN.$

$W_2 = LBD \times$ unit wt soil

$W_2 = 0.5 \times 1 \times 23.4$

$W_2 = 11.7$ kN.

Find moment

$M_1 = W_1 \times$ length

$= 126.6 \times 0.5$

$= 63.30$ kNm.

$M_2 = W_2 \times$ length

$= 11.7 \times 0.67 = 7.84$ kNm

total moment $M = M_1 + M_2$

~~126.6~~

$= 63.30 + 7.84$

$= 71.14$ kNm

Deduct for self wt of toe slab

$W_3 = (1 \times 0.45 \times 24)$

$W_3 = 10.8$ KN

Deduct for wt of soil above toe slab

$W_4 = 0.75 \times 1 \times 16$

$W_4 = 12.0$ KN.

moment deduction $Wd_1 = W_3 \times$ length

$= 10.8 \times 0.5 = 5.40$ KNm.

$Wd_2 = W_4 \times$ length

$= 12.0 \times 0.5$

$= 6.00$ KNm.

Total deduction $Md = Md_1 + Md_2$

$= 5.40 + 6.00$

$= 11.40$ KNm.

Maximum working moment in toe slab

$M = M - M_d$

$= 71.14 - 11.4 = 59.74$ kNm

Forced moment $M_u = 1.5 \times M_u$

$= 1.5 \times 59.74 = 89.61$ kNm.

Effective depth of toe slab = 400 mm

Reinforcements in toe slab

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

$$89.61 \times 10^6 = (0.87 \times 415 A_{st} \times 400) \left[1 - \frac{415 A_{st}}{(10^3 \times 400 \times 20} \right]$$

$A_{st} = 644\ mm^2$

Provide 12mm dia bars at 150mm c/c

($A_{st} = 754\ mm^2$)

Distribution bars $= (0.0012 \times 1000 \times 450)$

$= 540\ mm^2$.

provide 10mm dia bar at 280mm c/c on both faces ($A_{st} = 561\ mm^2$)

Step 5 : Design of heel slab

considering 1m wide strip of heel slab near heal end a, upward soil pressure = 45 kN/m2

weight of soil on strip = (16 x 6.75)

= 108.00 kN/m2

Self weight of strip = (1 x 0.45 x 24)

= 10.80 kN/m2

Total = 108.00 + 10.80

= 118.80 kN/m2

Deduct for downward pressure

= -45.00 kN/m2

Net downward pressure = 73.80 kN/m2

Spacing of counter forts = 3m.

Max negative service BM at counter forts

$$M = \left(\frac{73.80 \times 3^2}{12}\right) = 55.35 \text{ kNm.}$$

Factored moment = Mu × 1.5

= 1.5 × 55.35 = 83 kNm

Reinforced in heel slab

IS 456. 2000 Clause G-1.1

$Mu = (0.87 f_y A_{st} d) \left[1 - A_{st} f_y / b d f_{ck}\right]$

$83 \times 10^6 = (0.87 \times 415 A_{st} \times 400) \left[1 - \frac{415 A_{st}}{(1000 \times 400 \times 20)}\right]$

$A_{st} = 600 mm^2$

provide 12 mm dia bar at 150 mm c/c

($A_{st} = 754 mm^2$)

Distribution bar = 0.12 percent of cross section

$= 0.0012 \times 1000 \times 450 = 540 mm^2$

provide 10 mm dia bar at 280 mm centres on both faces ($A_{st} = 561 mm^2$)

Step 6 :

Design of counterforts :

Thickness provide at the top = (220 + 220) = 440mm

Thickness of counterforts = 440mm.

Max working momoment in counterforts is

$$M = \left(ka \cdot \frac{wh^3}{6} \cdot L \right) = \left(\frac{1}{3} \times \frac{16 \times 6.75^3}{6} \times 3 \right)$$

M = 820.12 kNm.

Factored moment = Mu × 1.5 = 820.12 × 1.5

= 1230 kNm.

Reinforcement at the bottom of counterforts is computed using the relation

$$(1230 \times 10^6) = (0.87 \times 415 \, A_{st} \times 4400) \left[1 - \frac{415 \, A_{st}}{440 \times 4400 \times 20)} \right]$$

A_{st} = 800mm²

But minimum reinforcement as per IS 456 – 2000

$$A_{st} = \left(\frac{0.85\,bd}{f_y}\right) = \left[\frac{0.85 \times 440 \times 4400}{415}\right]$$

$$= 3965\ mm^2$$

provide 5 bars of 32mm dia ($A_{st} = 4020\ mm^2$)

Step I : curtailment of bars.

h_1 = depth at which 1 bar can be curtailed

Then $\left(\frac{5-1}{5}\right) = \left(\frac{h_1}{6.75^2}\right)$

h_1 = 6m from top.

h_2 = depth at which 2 bars are curtailed

Then $\left(\frac{5-2}{5}\right) = \left(\frac{h_2}{6.75^2}\right)$

h_2 = 5.2 m from top.

h_3 = depth at which 3 bars are curtailed

Then $\left(\frac{5-3}{5}\right) = \left(\frac{h_3}{6.75^2}\right)$

h_3 = 4.2 m from top.

The remaining two bars are taken right up to the top.

Step 8. connection b/w counterforts and upright slab.

considering the bottom 1m height of the upright slab, pressure on this strip

$= 36 \, kN/m^2$

Total working load pressure transferred to the counterfort for 1m height

$= 36(3 - 0.44) = 91.8 \, kN$

factored force $= (1.5 \times 91.8)$

$= 138 \, kN$

Reinforcement required per metre height =

$$= \left(\frac{138 \times 10^3}{0.87 \times 415}\right) = 382 \, mm^2$$

minimum reinforcement $= (0.0012 \times 10^3 \times 440)$

$= 528 \, mm^2$

spacing of 10mm dia bars $= \left(\frac{78.5 \times 1000}{528}\right)$

$= 148.6 \, mm$.

This amount of reinforcement is provide as two. legged horrizontal links of 10mm dia at 280mm c/c

Step 9: Connection b/w counterforts and heel slab.

working tension transfered in 1m width of the counterforts near the heel end

$a = 73.80\ (3 - 0.44) = 189\ kN$

factored tension $= (1.5 \times 189)$

$= 283.5\ kN.$

Reinforcement required in 1 m width

$$= \left(\frac{283.5 \times 10^3}{0.87 \times 415}\right) = 785\ mm^2/m$$

Spacing of 10mm dia two legged links

$$= \left(\frac{2 \times 78.5 \times 1000}{785}\right) = 200\ mm$$

Provide 10mm dia two-legged links at 200mm c/c.

UNIT II

FLAT SLAB AND BRIDGES

Design of flat slab with and without drops by direct design method of IS code - Design and drawing - IRC specification and loading - RC solid slab bridge - Steel foot over bridge - Design and drawing

Course outcome : Design and draw Reinforced flat slab as per code provisions

Design example interior panel · 1

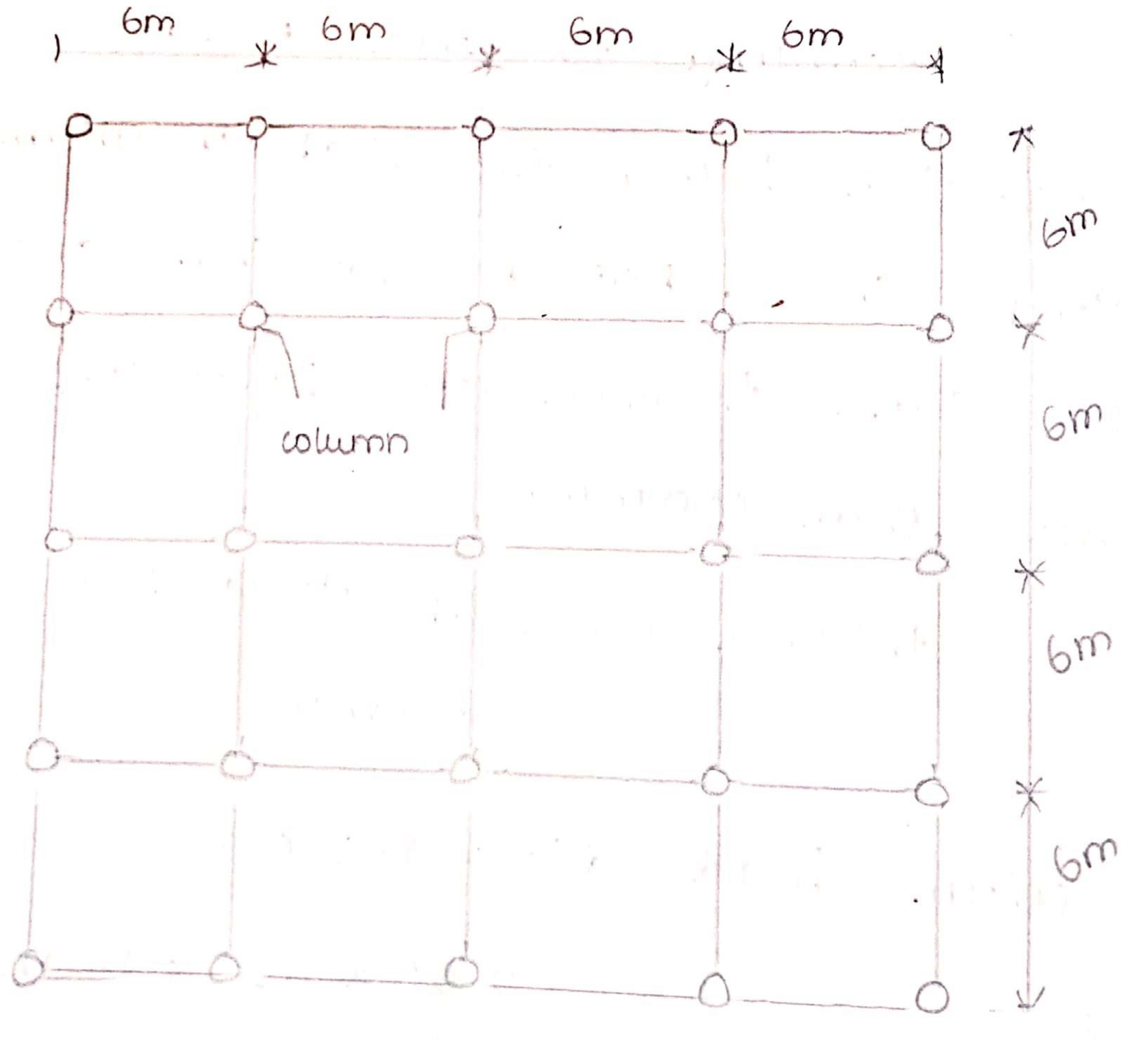
Prblm 1

Design the interior panel of a flat - slab floor system for a ware house 24m × 24m divided into panels of 6m × 6m

loading class = 5 kN/m^2

materials : M20 grade concrete
Fe 415 HYSD bars

column size = 400mm diameter.



Step 1

Diamonsion of a flat slab.

Hence the overall span-to-depth

ratio = (26 × 1.3) = 33.8

Thickness of slab at mid span

= (600 / 33.8) = 177.5 mm.

Adopt an effective depth of 170mm and overall depth of 200mm.

According to code A.C.I - 318 the projection below the slab in column strip should not be less than one-fourth of slab thickness and preferably not less than 100 mm.

thickness of slab at drop = (200 + 100)

= 300mm.

colum head dia = D ≠ 0.25 L =

= (0.25 × 6) = 1.5 m.

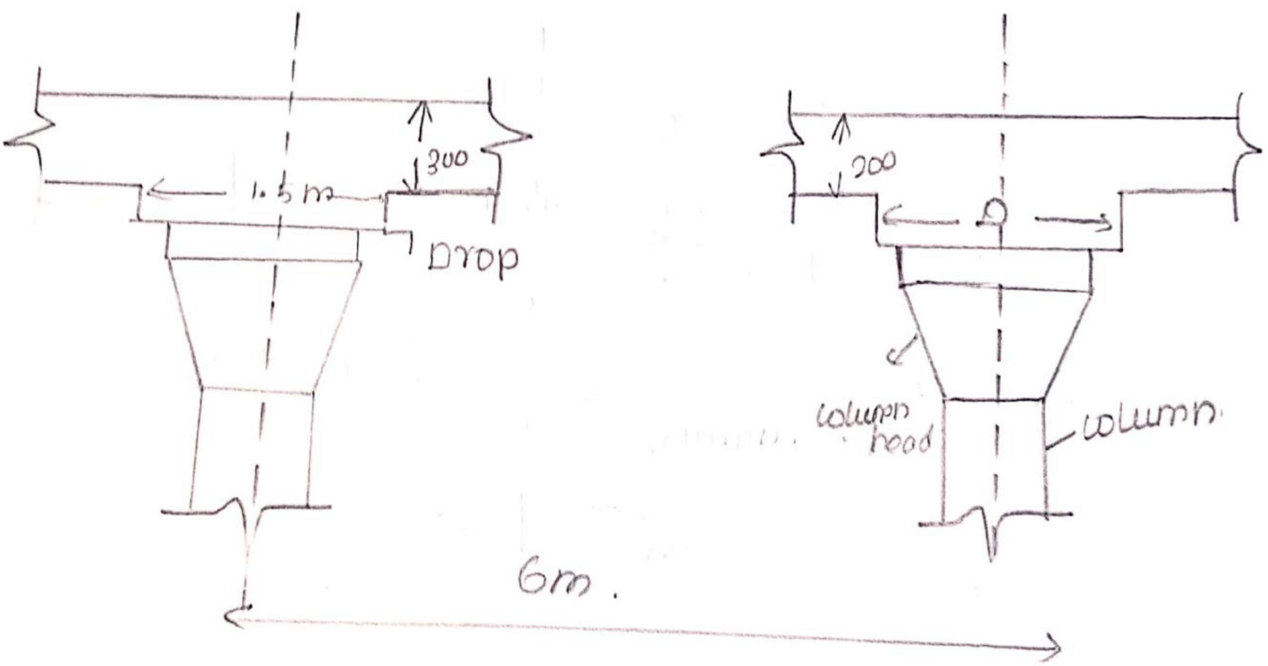
length of drop ≮ (L/3) ≮ (6/3).

= 2m

Adopt drop width = 3m.

∴ column strip = drop width = 3m

width of middle strip = 3m



Step 2 : loads.

live load at $5 kN/m^2$ = $5.00\ kN/m^2$

Dead load of slab = 0.5 (0.3 + 0.2725)

= 6.25.

Floor finishes, etc. = 0.75

Total service load = w = $12.00\ kN/m^2$

Factored load = W_u = (1.5 × 12)

W_u = $18\ kN/m^2$

Step 3 : factored bending moments:

IS 456:2000

Clause 31.4.2.2

Total moment = M_o = $\left(\frac{W L_n}{8}\right)$

L_n = (6 − 1.5) = 4.5 m

L_1 = L_2 = 6 m

W = ($W_u L_2 L_n$)

= 18 × 6 × 4.5

W = 486 kN.

Therefore

$$Mo = \left(\frac{486 \times 4.5}{8}\right) = 274 \text{ kNm}$$

a) column strip moment

negative BM = 49%

$Mo = (0.49 \times 274) = 134$ kNm.

positive BM = 21%

$Mo = (0.21 \times 274) = 58$ kNm

b) middle strip moment :

negative BM = 15%

$Mo = (0.15 \times 274) = 41$ kNm

positive BM = 15%.

$Mo = (0.15 \times 274) = 41$ kNm.

Step 4 : Check for thickness of slab.

For balanced section, $M_u = 0.138 f_{ck} bd^2$

$$\therefore d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$$

a) Thickness of slab near drops, (column strip)

$$d = \sqrt{\frac{134 \times 10^6}{0.138 \times 20 \times 3 \times 10^3}} = 187mm$$

b) Thickness of slab in middle strips:

$$d' = \sqrt{\frac{41 \times 10^6}{0.138 \times 20 \times 3 \times 10^3}} = 70mm.$$

From shear consideration, the slab thickness required will be more than the based on bending moments

overall depth near drops = 300mm

effective depth = 270mm.

Overall depth (middle strips) = 200mm.

effective depth = 170mm.

Step : 5 : Check for Shear Stresses

Shear stress is checked near the column head at a section (D+d) near the column head.

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Total load on the circular area with (D+d) as diameter is given by,

$$W_1 = \frac{\pi}{4}(D+d)^2 \cdot W_u$$

$$= \frac{\pi}{4}(1.5+0.27)^2 \times 18 = 44.3\,kN.$$

Shear force = total load − load on circular area.

$$= (18 \times 6 \times 6) - 44.3$$

$$= 603.7\,kN.$$

Shear force per meter of perimeter.

$$= \left[\frac{603.7}{(D+d)}\right] = \left[\frac{603.7}{(1.77)}\right] = 108.8\,kN.$$

$$\text{Shear stress} = \left(\frac{108.6 \times 10^3}{1000 \times 270}\right) = 0.40\,N/mm^2$$

permissible shear stress $= k_s \tau_c$

where $k_s = (0.5 + \beta_c)$

and $\beta_c = (L_1/L_2) = 1$

$\therefore$ $k_s = (0.5+1) = 1.5$ but not greater than 1.0.

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Hence $\tau_c = 0.85\sqrt{f_{ck}}$

$= 0.85\sqrt{20} = 1.118 \, N/mm^2$

$k_s \tau_c = (1 \times 1.118) = 1.118 \, N/mm^2$

The actual shear stress of $0.4 \, N/mm^2$ is within the safe permissible limits

Step 6 : Reinforcements.

a) column strip : (IS 456:2000, clause G-1.1)

Ast for negative BM

$$M_u = 0.87 \times 415 \times A_{st} \times d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

$$134 \times 10^6 = 0.87 \times 415 A_{st} \times 270 \times \left[1 - \frac{415 A_{st}}{3 \times 10^3 \times 270 \times 20}\right]$$

$A_{st} = 1485 mm^2$

Ast per metre $= (1485/3)$

$= 495 mm^2$

Ast for positive BM

$$(58 \times 10^6) = 0.87 \times 415 \times Ast \times 170 \times \left(1 - \frac{415\,Ast}{3 \times 10^3 \times 270 \times 20}\right)$$

$Ast = 968\ mm^2$

Ast per metre $= (968/3) = 323\ mm^2$

For column strip provide 12 mm dia bars at 200mm centres ($Ast = 565\ mm^2$).

for negative moment and at 300mm centres for positive bending moment ($Ast = 377\ mm^2$)

b) middle strip :

Ast for positive and negative bending moment.

$$41 \times 10^6 = 0.87 \times 415\,Ast \times 170 \times \left[1 - \frac{415\,Ast}{3 \times 10^3 \times 170 \times 20}\right]$$

$Ast = 689\ mm^2$

$\therefore$ Ast per metre $= (689/3)$

$= 230\ mm^2$

minimum reinforcement is given by

A_{st} (minimum) = $(0.0012 \times 10^3 \times 200)$

$= 240 mm^2$

provide 10mm dia bar at 300mm centre for both positive and negative moments ($A_{st} = 262 mm^2$)

<u>Step : 7</u>

Check for deflection control.

For middle strip, $P_1 = \left(\frac{100 A_{st}}{bd}\right)$

$= \left(\frac{100 \times 262}{1000 \times 70}\right) = 0.15$ (IS 456.2000 clause 23.2.1

the modification factor for tension reinforcement is read out as $k_1 = 1.8$

Hence $(L/d)_{max} = (k_1 \times 26)$

$= 1.8 \times 26 = 46.8$.

$(L/d)_{provided} = (6000/170)$

$= 35.2 < 46.8$.

Example : 2 Design example (exterior panel)

Design the exterior panel of a flash - slab floor system for a ware house 24 m by 24 m divided into panels of 6m by 6m.

loading class = 5kN /m²

materials = M20 grade concrete

Fe 415 HYSD bars

column size = 400mm dia

Height of storey = 3m

Thickness of slab in colum strip = 300mm.

Thickness of slab in middle strip = 200mm.

Step 1 : Diamension of the flat slab.

width of middle strip =

width of column strip = drop width

= 3m.

Step 2 : Stiffness computations :

Stiffness of column is given by.

$$k_c = \left(\frac{4E_c I_c}{L} \right) = \left(\frac{4E_c (400)^4}{64 \times 3000} \right)$$

$$= (1.67 \times 10^6) E_c$$

Assuming columns both at top and bottom

$$k_c = 2(1.67 \times 10^6) E_c.$$

Stiffness of slab is given.

$$k_s = \left[\frac{4(6000)(300)^3}{12(6000)} \right] = (9 \times 10^6) E_c$$

$$\alpha_c = \left(\frac{\Sigma k_c}{\Sigma k_s} \right) = \frac{2(1.67 \times 10^6) E_c}{(9 \times 10^6) E_c}$$

$$\alpha_c = 0.37$$

(From Table of IS 456 : 2000, the value of coefficient.)

$$\alpha_{c,min} = 0.7 = 0.7 \text{ for } (L_2/L_1) = 1$$

Hence $= \left(1+\frac{1}{\alpha_c}\right) = \left(1+\frac{1}{0.7}\right) = 2.43$

and $L_n = (6-1.5) = 4.5$ m.

Step 3 : Bending moments:

Total load $= W = (W_u L_2 L_n)$

$= (18 \times 6 \times 4.5) = 486$ kN.

$\therefore M_o = \frac{W L_n}{8} = \frac{486 \times 4.5}{8} = 274$ kNm

Interior negative design moment.

IS 456.2000
clause 31.4.3:3

$= \left(0.75 - \frac{0.10}{1+1/\alpha_c}\right) M_o.$

$= \left(0.75 - \frac{0.10}{2.43}\right) 274 = 194$ kNm

Positive ~~deg~~ design moment.

$= \left(0.63 - \frac{0.28}{1+1/\alpha_c}\right) M_o$

$$= \left(0.68 - \frac{0.28}{2.43}\right) 274$$

$= 141$ kNm.

Exterior negative ~~deg~~ design moment

$$= \left(\frac{0.65}{1 + 1/\alpha_c}\right) M_o = \left(\frac{0.65}{2.43}\right) 274$$

$= 73$ kNm.

The moments in the column and middle strip are obtained as given,

Interior negative design moment in

column strip $= (0.75 \times 194) = 146$ kNm.

Middle strip $= (0.25 \times 194) = 48$ kNm.

Exterior negative design moment in.

column strip $= 55$ kNm.

middle strip $= 0$.

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possitive moment in column strip for each Span = (0.60 × 141) = 85 kNm

positive moment in middle strip for each span = (0.40 × 141) = 56 kNm.

Step 4 : Thickness of slab :

Thickness of slab required near drop

$$d = \sqrt{\frac{Mu}{0.138 f_{ck} b}} = \sqrt{\frac{146 \times 10^6}{0.138 \times 20 \times 300}}$$

$= 133$ mm.

Thickness of slab required in middle strips.

$$d = \sqrt{\frac{56 \times 10^6}{0.138 \times 20 \times 3000}} = 83 \text{mm}.$$

Adopt overall depth = 300mm

effective depth = 270 mm in column strip

In middle strips:

Overall depth = 200 mm.

effective depth = 170 mm.

Step 5: Reinforcement Strip (interior)

~~Ast~~ a) column strip (interior)

Ast for negative bending moment.

$$Mu = 0.87 f_y \, Ast \, d \left(1 - \frac{Ast \, f_y}{b \, d \, f_{ck}}\right)$$

$$(146 \times 10^6) = 0.87 \times 415 \, Ast \times 270 \times \left[1 - \frac{415 \, Ast}{(3 \times 10^3 \times 270 \times 20)}\right]$$

$Ast = 1539 \, mm^2$.

Ast per metre = $(1539/3)$

$= 513 \, mm^2$

Ast for positive bending moment.

$$(85 \times 10^6) = (0.87 \times 415 \, Ast \times 170)\left(1 - \frac{415 \, Ast}{(3000 \times 170 \times 20)}\right)$$

$Ast = 1479 \, mm^2$

Ast per metre $= (1479 / 3) = 493 \, mm^2$

provide 12 mm diameter bar at 200mm centre. ($Ast = 565 \, mm^2$) for negative moment and positive moment in the column strip.

b) middle strip.

$$(56 \times 10^6) = (0.87 \times 415 \, Ast \times 170) \times \left(1 - \frac{415 \, Ast}{(3000 \times 170 \times 20)}\right)$$

$Ast = 1530 \, mm^2$

As per metre $= (1530 / 3) = 510 \, mm^2$

provide 12 mm dia bar at 200mm centre, ($Ast = 565 \, mm^2$)

c) column strip (exterior)

$$(55 \times 10^6) = (0.87 \times 415\, A_{st} \times 270)\left(1 - \frac{415\, A_{st}}{(3000 \times 170 \times 20)}\right)$$

Solving $A_{st} = 1510\,mm^2$.

A_{st} per metre $= (1510/3) = 503\,mm^2$

Provide 12mm dia bar at 200mm centres

($A_{st} = 565\,mm^2$)

A_{st} for positive bending moment.

Minimum reinforcement $= (0.0012 \times 1000 \times 200)$

$= 240\,mm^2$.

provide 10mm dia bars at 300mm centre

($A_{st} = 262\,mm^2$)

UNIT III

LIQUID STORAGE STRUCTURES

RCC water tank - On ground, Elevated circular, underground rectangular tanks - Hemispherical bottomed steel water tank - Design and drawing

Course outcome : Design and draw Reinforced concrete and steel water tanks

Design example (circular water tank)

Example 1)

Design an RC circular water tank resting on the ground with a flexible base and a spherical dome for a capacity of 500,000 litres. The depth of storage is to be 4m. Free board = 20mm. use M20 grade concrete and Fe 250 grade I steel. Permissible stresses should comply with the value recommended in IS 456.1978, Clause 44.1 and IS : 3370 (PART II)-1975 clause 33.1 and 34.2. Draw the following views.

* cross section of the tank showing reinforcement details in dome, tank walls and floor slab.

* plan of the tank showing reinforcement details.

Step 1: Data

Capacity of circular tank = 5,00,000 l

Depth of water = 4m

Free board = 200mm

Materials : M20 Grade concrete

Fe 250 grade I steel

Step 2 : permissible stresses.

$\sigma_{ct} = 1.2 \, N/mm^2$ (for tank wall)

$= 2.8 \, N/mm^2$ (for dome ring beam)

IS : 3370 (Part II)

$\sigma_{cc} = 5 \, N/mm^2$ — Table 1, clause 3.3.1 &

$\sigma_{st} = 115 \, N/mm^2$ — IS 456.2000

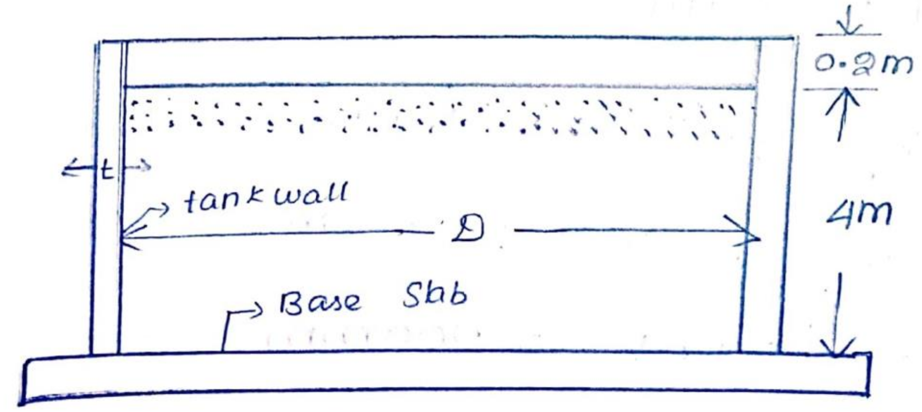
$m = 13.$ — Clause B-2.1.1 & Table 21.

Step 3 : Diamension of tank

If D = diameter of tank

$$\left(\frac{\pi D^2}{4} \times 4 \right) = \left(\frac{5{,}00{,}000 \times 10^3}{10^6} \right)$$

$$D = 12.6 \, m.$$



Step 4: Design of spherical dome

Base dia of dome = 12.6m

Central rise of dome = $[(1/5) \times \text{diameter}]$

$= [(1/5) \times 12.6] = 2.5m$

R = radius of the dome,

$(R - 2.5)^2 = R^2 - 6.3^2$

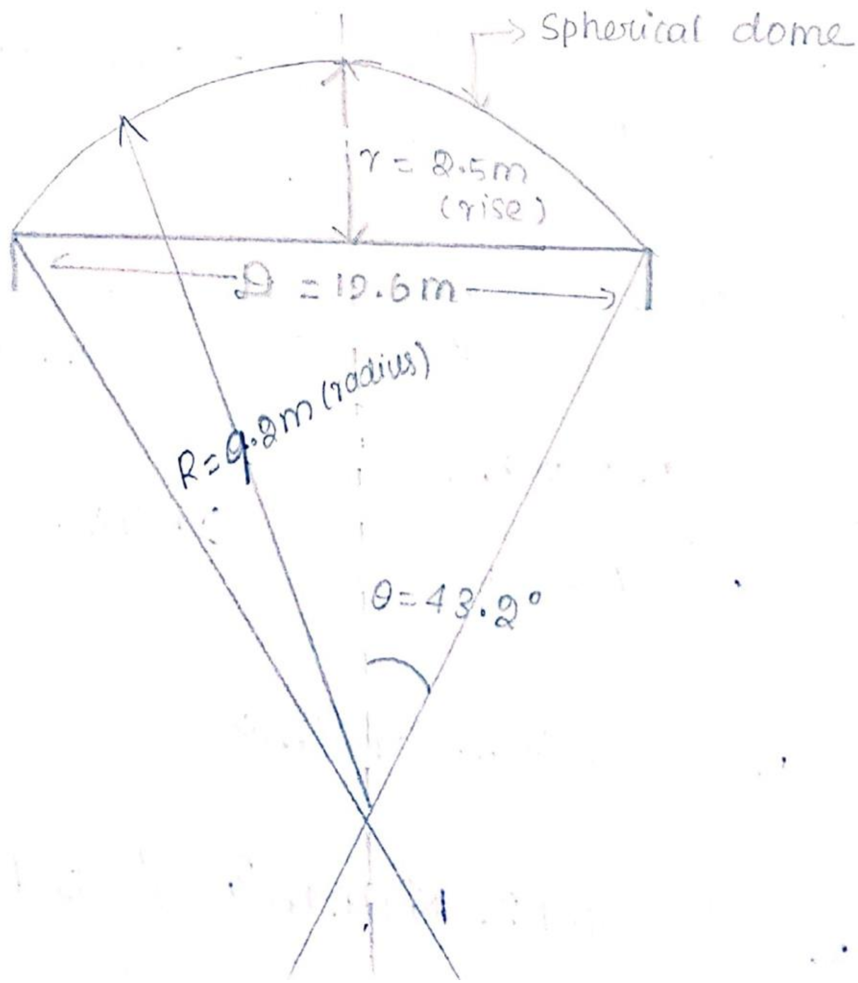
$\therefore R = 9.2m$

Semi - central angle = 43.2°

$\therefore \sin\theta = 0.6847$

$\cos\theta = 0.7289$

Assume thickness of dome t = 100 mm



a) loads :

Self weight of dome $= (0.1 \times 24) = 2.4\ kN/m^2$

live load and finishes $= 2.0\ kN/m^2$

Total load $w = 2.4 + 2.0$

$w = 4.4\ kN/m^2$

b) stress in dome :

Meridional thrust $T_1 = \left(\frac{wR}{1+\cos\theta}\right)$

$= \left(\frac{4.4 \times 9.2}{1+0.7289}\right) = 23.41\ kN/m$

meridional compressive stress

$= \left(\frac{23.41 \times 10^3}{1000 \times 100}\right) = 0.2341\ N/mm^2 < 5 N/mm^2$

Hoop stress $= \frac{wR}{t}\left(\cos\theta - \frac{1}{1+\cos\theta}\right)$

$= \left(\frac{4.4 \times 9.2}{0.1}\right)\left(0.7289 - \frac{1}{1.7289}\right)$

$= 60.72\ kN/m^2$

$= 0.06072\ N/mm^2 < 5\ N/mm^2$

Stress within safe limits

c) Reinforcement in dome

Since the stresses are very low, a nominal reinforcement of 0.3% of the cross sectional area is provided.

$$Ast = \left(\frac{0.3 \times 1000 \times 100}{100}\right) = 300\ mm^2.$$

$$\text{Spacing of 8mm dia bars} = \left(\frac{100 \times 50}{300}\right)$$

$$= 166\ mm.$$

Provide 8mm dia bars at 160mm c/c both meridionally and circumferentially.

d) Ring beam :

$$\text{Horizontal component of thrust} = T_1 \cos\theta$$

$$= (23.41 \times 0.7289) = 17.06\ kN/m$$

$$\text{Hoop tension in ring beam} = \left(\frac{17.06 \times 12.6}{2}\right)$$

$$= 107.47\ kN$$

$$Ast = \left(\frac{107.47 \times 10^3}{115}\right) = 935\ mm^2$$

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Provide 4 bars of 20mm dia

($A_{st} = 1256 mm^2$)

A_c = cross sectional area of the ring beam, allowing tensile stress of $2.8 N/mm^2$ in concrete, we have the relation.

IS 456.2000

Clause B-2.1.1

$$\left(\frac{F_1}{A_c + (m-1) A_{st}} \right) = 2.8$$

$$\frac{107.47 \times 10^3}{A_c + (13-1) 1256} = 2.8$$

$$\therefore A_c = 23310 \, mm^2$$

Adopt a ring beam of size 20mm x 200mm with 4 bar of 20mm dia as hoop reinforcement and stirrups of 6mm dia at 150mm c/c

Step 5 : Reinforcement in tank walls.

maximum hoop tension $= (0.5\ WHD)$

$= (0.5 \times 10 \times 4.2 \times 12.6)$

$= 264.6\ kN.$

Tension reinforcement per metre of height

$$A_{st} = \left(\frac{264.6 \times 10^3}{115}\right) = 2300\ mm^2$$

using 16mm dia bar on both faces.

$$Spacing = \left(\frac{1000 \times 201 \times 2}{2300}\right) = 174\ mm$$

Provide 16mm dia bar at 150mm c/c at base section on either face of the wall.

Step 6 Thickness of tank wall.

If t = thickness of tank wall, from cracking considerations.

$$\left(\frac{0.5\ WHD}{1000t + (m-1)A_{st}}\right) = \sigma_{ct}$$

$$\left(\frac{246.6 \times 10^3}{1000t + (13-1)\ 2646}\right) = 1.2$$

∴ t = 188.7 mm . Adopt 190mm thick

tank walls.

Step 7 : curtailment of reinforcement in tank walls.

Spacing of hoops is increased towards the top. minimum at the top = 0.3%.

$$A_{st} = \left(\frac{0.3 \times 1000 \times 190}{100}\right) = 570\,mm^2$$

∴ spacing of hoops (using 12mm dia bar on ~~too~~ both faces.

$$= \left(\frac{1000 \times 113 \times 2}{570}\right) = 396\ mm.$$

maximum spacing ≯ 3 times thickness of wall

≯ 3 × 190

≯ 570mm.

Spacing at a depth of 2m below the top

$$A_{st} = \left(\frac{0.5 w H D}{115}\right) = \left(\frac{(0.5 \times 10 \times 2 \times 12.6)\, 10^3}{115}\right)$$

$$= 1095 \text{ mm}^2$$

Spacing of 16mm dia bar on both faces

$$= \left(\frac{1000 \times 201 \times 2}{1095}\right) = 367 \text{ mm c/c} \not> 300 \text{mm c/c}$$

Area of Vertical reinforcement = 0.3%

$$= \left(\frac{0.3 \times 1000 \times 190}{100}\right) = 570 \text{ mm}^2$$

Spacing of 10mm diameter bar on both faces.

$$= \left(\frac{1000 \times 78.5 \times 2}{570}\right) = 274 \text{mm}.$$

Use 10mm dia bar at 270mm c/c

Step 8 :

provide nominal thickness of 150mm for the base slab

minimum area of reinforcement

$$Ast = 0.3\%$$

$$= \left(\frac{0.3 \times 150 \times 1000}{100}\right) = 450 mm^2$$ in each direction.

$$\therefore \quad Ast \text{ for each face} = \frac{450}{2} = 225 mm^2$$

$$\text{Spacing of 8mm dia bars} = \left(\frac{1000 \times 50}{225}\right)$$

$$= 220mm.$$

use 8mm dia bar at 200mm c/c. at the top and bottom faces of the tank floor slab.

Design example (rectangular water tank)

Example 2

Design a rectangular R.c water tank (resting on ground) with an open top for a capacity of 80000 l. The inside diamension of the tank may be taken as 6m x 4m. Design the side wall of the tank using M20 grade concrete and Fe 250 grade I mild steel.

Draw the following views.

* Cross sectional elevation of the tank showing reinforcement details in tank walls
* plan of the tank showing reinforcement details.

Step 1 : Data

Capacity of tank = 80000 l

Size of tank = 6m x 4 mm

Free board = 150 mm.

Materials : M_{20} grade concrete

Fe 250 grade I mild steel

$\sigma_{cb} = 7 N/mm^2$

$\sigma_{st} = 115 N/mm^2$ (on faces near water face)

$\sigma_{st} = 125 N/mm^2$ (on faces away from water faces)

$m = 13$ $Q = 1.41$, $j = 0.84$

Step 2 : Diamension of tank

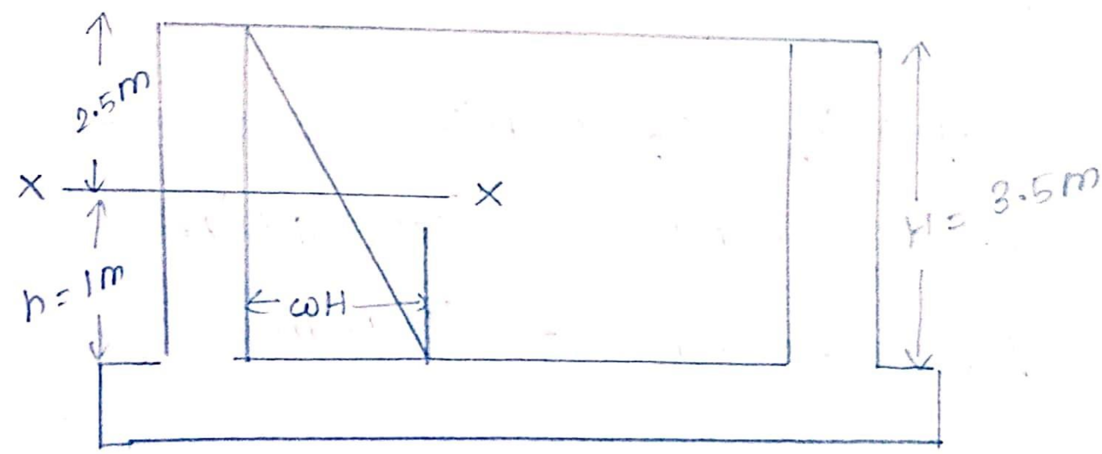
Height of water $= \left(\frac{80000 \times 10^3}{600 \times 400}\right) = 3.35\ m$

Free board $= 150\ mm$.

Height of side walls $H = (3.35 + 0.15)$

$H = 3.5 m$

$(L/B) = 6/4 = 1.5 < 2$



∴ Therefore intensity of pressure $p = \omega(H-h)$ at xx

$= (10 \times 2.5) = 25\ kN/m^2$

Alternately, the design table of IS : 3370 (part iv) - 1967, Clause 2.2 can be used for the computation of moment in tank walls.

Step 3 :

The moment in side wall are determined by moment distribution. $L = 6m$, $B = 4m$.

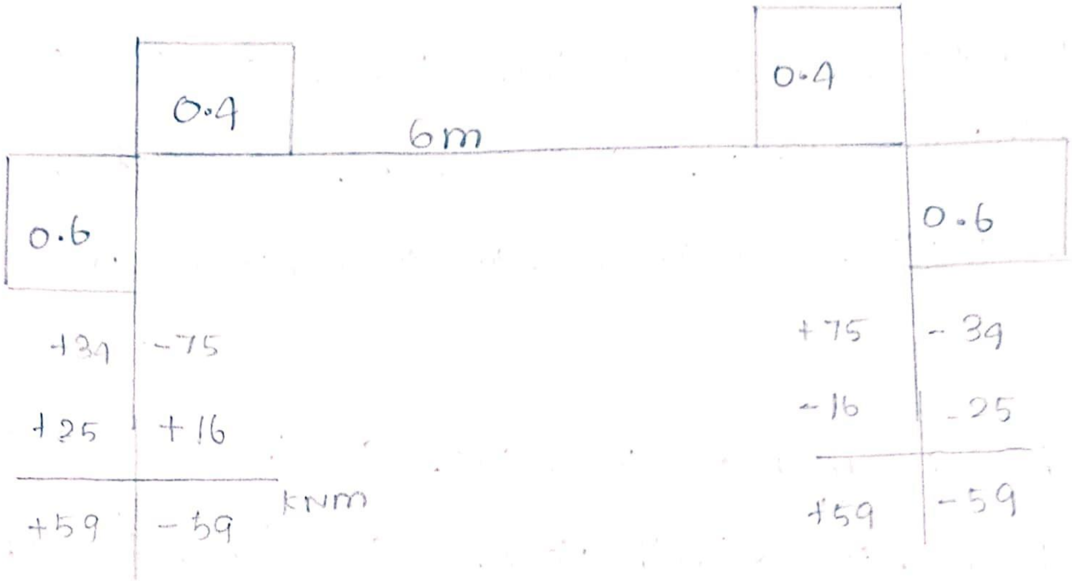
$$\left(\frac{pL^2}{12}\right) = \left(\frac{25 \times 6^2}{12}\right) = 75\ kNm.$$

$$\left(\frac{pL^2}{8}\right) = \left(\frac{25 \times 6^2}{8}\right) = 112.5\ kNm.$$

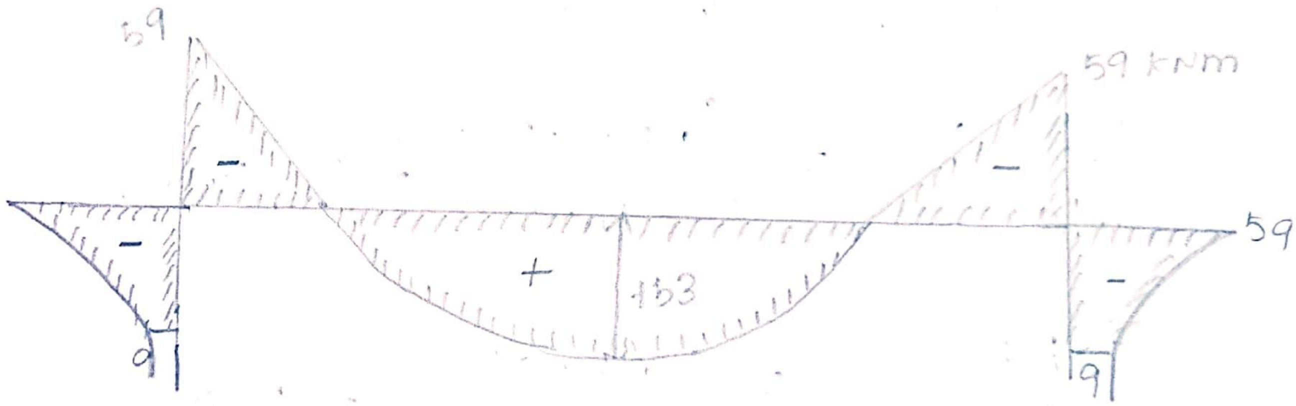
$$\left(\frac{pB^2}{12}\right) = \left(\frac{25 \times 4^2}{12}\right) = 34\ kNm.$$

$$\left(\frac{pB^2}{8}\right) = \left(\frac{25 \times 4^2}{8}\right) = 50\ kNm.$$

a) moment distribution :



b) BM diagram :



moment at Support = 59 kNm

moment at centre (long wall) = (112 - 59) =

= 53 kNm.

moment at centre (Shot walls) = (50 - 59)

= - 9 kNm.

Step 4 : Design of long and Shot walls

maximum design moment = 59 kNm.

$$d = \sqrt{\frac{59 \times 10^6}{1.41 \times 1000}} = 204 \text{ mm}.$$

Adopt effective depth = 215 mm

overall depth = 250 mm

Direct tension in long wall T = (0.5 × 25 × 4)

= 50 kN.

Direct tension in Shot wall T = (0.5 × 25 × 6)

= 75 kN.

$$A_{st} \text{ (long wall corners)} = \left(\frac{M - Tx}{\sigma_{st}\, jd}\right) + \left(\frac{T}{\sigma_{st}}\right)$$

$$A_{st} = \left[\frac{(59 \times 10^6) - (50 \times 10^3 \times 90)}{100 \times 0.84 \times 215}\right] + \left[\frac{50 \times 10^3}{100}\right]$$

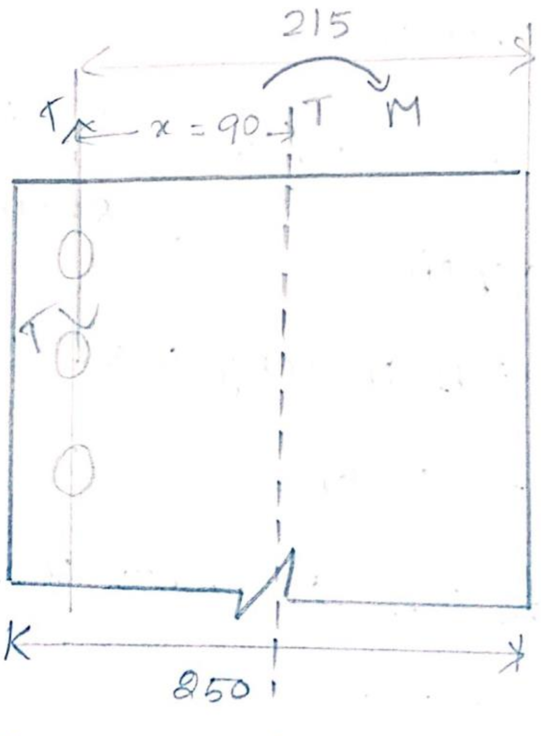
$$= 3480\ mm^2$$

Spacing of 20mm dia bars $= \left(\frac{1000 \times 314}{3480}\right)$

$= 90mm\ c/c$

Adopt 20mm dia bar at 80mm c/c

($A_{st} = 3928 mm^2$)



T = pull in Steel

Net moment $= (M - Tx)$

Reinforcement at centre of span (long wall)

$$= \left[\frac{(53 \times 10^6) - (50 \times 10^3 \times 90)}{185 \times 0.84 \times 215}\right] + \left[\frac{50 \times 10^3}{125}\right]$$

$$= 2500 \text{ mm}^2.$$

Step 5 : Reinforcement for cantilever moment.

For 1m height from the bottom.

$$\text{Cantilever moment} = (3.5 \times 10 \times 1/2 \times 1/3)$$

$$= 5.833 \text{ kNm}.$$

$$\therefore \text{Ast} = \left(\frac{5.833 \times 10^6}{100 \times 0.84 \times 215}\right) = 323 \text{ mm}^2$$

minimum reinforcement = 0.3%.

$$= \left(\frac{0.3 \times 1000 \times 250}{100}\right) = 750 \text{mm}^2.$$

Reinforcement of each face = (0.5 × 750)

$$= 375 \text{ mm}^2.$$

Spacing of 8mm dia bars = $\left(\frac{1000 \times 50}{375}\right)$

= 130mm c/c

Adopt 8mm dia bar at 130 mm c/c on both face.

Step 6 Base slab.

The base slab rest on ground.

Provide 200mm base slab with 10mm dia bar at 300mm c/c, both way on each face.

UNIT IV

INDUSTRIAL STRUCTURES

Structural steel framing - Steel roof trusses - Roofing elements - Beam column - Code provisions - Design and drawing

Course outcome : Design and draw steel trusses as per code provisions

Design example (Steel roof truss)

Example : 1

Design a steel roof truss to suit the following data.

Span of the truss = 10m

type of truss = pan - type

Roof cover = Galvanized Corrugated (GC) Sheeting.

Materials = Rolled - Steel angles.

spacing of Roof truss = 4.5 m.

wind pressure pd = 1.0 kN/m²

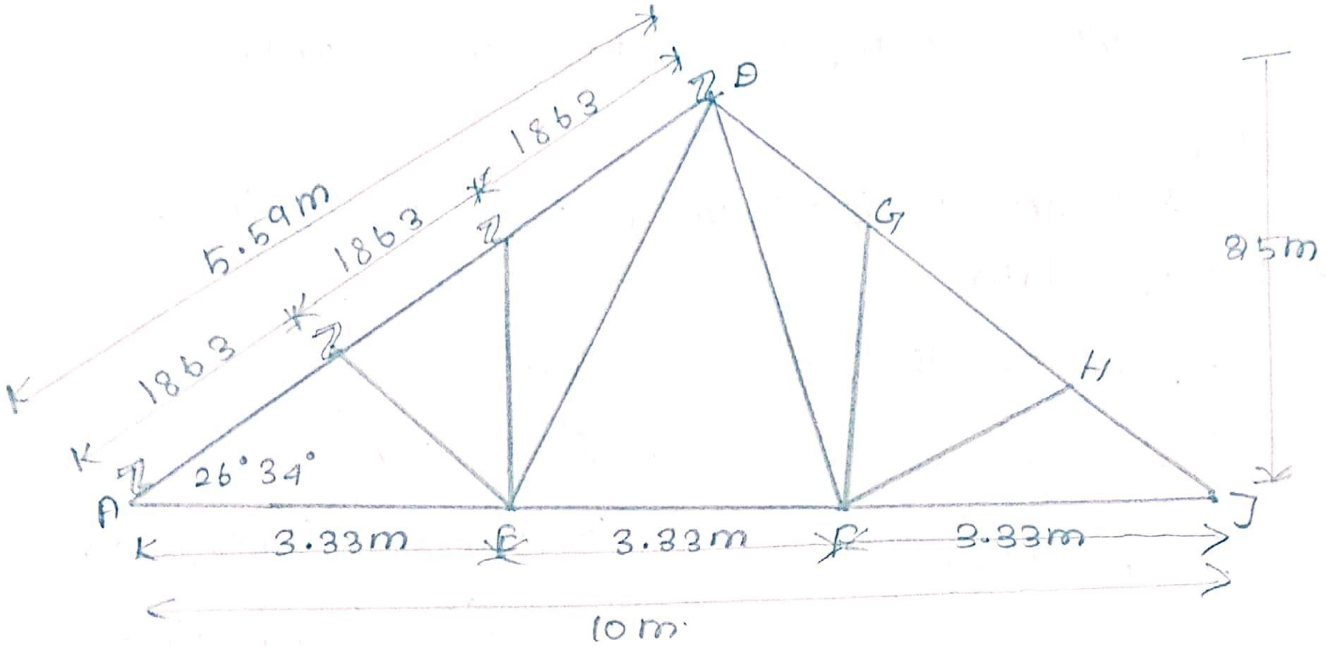
Draw the elevation of the roof truss and the details of joints.

Step 1 : Diamension of truss :

$$\text{central rise} = \left(\frac{\text{Span}}{4}\right) = \left(\frac{10}{4}\right)$$

$= 2.5\text{m}.$

purlines are provided at intervals of 1.863 m on the principal rafter AD.



Step 2 : Dead loads

Self weight of G.I.C. sheeting per purlin at 0.18 kN/m²

$= (0.18 \times 1.863) = 0.335\ kNm.$

Self wt of purlin at 0.1 kN/m = 0.10 kN/m

Total dead load = 0.435 kN/m.

Step 3 live loads

Slope of the truss = 26°34′

Live load of the truss = $0.75 - (10 \times 0.01 + 6.5 \times 0.02)$

$= 0.52\ kN/m^2.$

live load per purlin per metre $= (0.52 \times 1.836 \times \cos 26°34')$

$= 0.87$ kN.

<u>Step 4</u> : wind loads.

$$F = (C_{pe} - C_{pi}) A p_d$$

C_{pe} → external pressure co.efficient

C_{pi} → internal pressure co.efficient

A → Surface area of structural element or cladding unit

p_d → design wind pressure.

Sloping angle $\theta = 26°34'$, $C_{pe} = -0.7$

$C_{pi} = 0.2$.

$$F = (-0.7 - 0.2)\, p_d = -0.9\, p_d$$

$$= -(0.9 \times 1) = -0.9 \text{ kN/m}^2.$$

maximum wind load per purlin per metre

$= (-0.9 \times 1.863 \times \cos 26°34°) = 1.5$ kN.

Scanned with CamScanner

Step 5 : loads combinations:

(Dead load + live load) = (0.435 + 0.87)

= 1.305 kN/m.

(Dead load + wind load) = (0.435 − 1.50)

= − 1.065 kN/m.

Step 6 : Design of purlin

for continuous purlin, the max. factored bending moment and shear force are computed as follows.

$$M = \left(\frac{1.5 \times 1.305 \times 4.5^2}{10}\right) = 3.96 \text{ kN.m}$$

$$V = \left(\frac{1.5 \times 1.305 \times 4.5}{2}\right) = 4.4 \text{ kN}.$$

Adopt ISA 100 × 75 × 8 mm having section properties given below.

$Zx = (4.38 \times 10^4)\ mm^3$, $D = 100\ mm$

$b = 75\ mm$, $t = 8\ mm$.

IS 800 - 2007

Clause 3.7

a) Check for section classification is done by computing the rations.

$$b/t = \frac{75}{8} = 9.37 < 9.4$$

Hence the section considerd as plastic

b) Check for shear capacity.

$$A_v = (100 \times 8) = 800 \text{ mm}^2$$

Clause 8.4.1

$$\left(\frac{A_v f_{yw}}{\sqrt{3}\, \gamma_{m0}}\right) = \left(\frac{800 \times 250}{\sqrt{3} \times 1.10 \times 10^3}\right)$$

$$= 105 \text{ KN} > 4.40 \text{ KN}.$$

The shear capacity of the section is very large compared to the applied shear force.

c) Check for moment capacity.

M_d = The design moment capacity.

$$M_d = \left(\frac{\beta_b Z_x f_y}{\gamma_{mo}}\right) = \left(\frac{1 \times 4.38 \times 10^4 \times 250}{1.1 \times 10^6}\right)$$

$= 9.95$ kNm > 3.96 kNm

Step 7 : load on truss

a) Dead load

Sloping length of rafter AD $= \sqrt{5^2 + 2.5^2}$

$= 5.59$ m.

Spacing of trusses $= 4.5$ m c/c

Weight of GC sheeting on half truss (plan area) at 0.18 kN/m^2

$= (4.5 \times 5.0 \times 0.18)$

$= 4.05$ kN

Weight of purlins (4 nos) at 0.10 kN/m

$= (4 \times 0.1 \times 4.5) = 1.8$ kN.

Self weight of roof truss.

$$= \left(\frac{span}{300} + 0.05\right) = \left(\frac{10}{300} + 0.05\right)$$

$= 0.083$ kN/m^2

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Weight of half - roof truss = (0.083 × 5 × 4.5)

= 1.86 kN.

∴ total load on half roof truss.

= (4.05 + 1.8 + 1.86) = 7.71 kN.

Dead load on intermediate - panel point

= (7.71 / 3) = 2.57 kN.

Dead load on end panel point = (2.57 / 2)

= 1.285 kN.

b) live loads :

Live load on half truss = (0.52 × 5 × 4.5)

= 11.7 kN.

Live load on intermediate panel point

= (11.7 / 3) = 3.9 kN.

Live load on end - panel point = $\left(\frac{3.9}{2}\right)$

= 1.95 kN.

c) wind loads

maximum wind load acting perpendicular to the sloping surface.

$= -(0.9 \times 4.5 \times 5.59) = -22.63\,kN$

wind load on intermediate - panel point

$= -\left(\frac{22.68}{3}\right) = -7.5\,kN.$

wind load on end - panel point

$= \left(-\frac{7.5}{2}\right) = -3.75\,kN.$

Step 8 : Design of truss members.

a) Members, AB, BC and CD

maximum service load compressive force = 36.17 kN
= 36.17 kN

max. factored compressive force = (1.5 × 36.17)
= 54.25 kN

max. service load tensile force
= 22.95 kN.

max. factored tensile force $= (1.5 \times 22.95)$

$= 34.42$ kN.

length (L) $= 1.863$ m

Effective length (KL) $= (0.7 \times 1.863)$

$= 1.304$ m.

Try two angles ISA 50 x 50 x 6 mm placed back to back.

Area (A) $= 1136\ mm^2$

minimum radius of gyration (r_{min}) $= 15.1$ mm

Slenderness ratio $= \left[KL / r_{min}\right] = \left[\frac{1304}{15.1}\right]$

$= 86.3 < 180$

Stress reduction factor χ for column buckling class (c) corresponding to the slenderness ratio 86.3 and $f_y = 250\ N/mm^2$

$\chi = 0.56$.

∴ Design compressive stress is computed as

$$f_{cd} = \left(\frac{\chi f_y}{\gamma_{m0}}\right) = \left(\frac{0.56 \times 250}{1.25}\right) = 112\ N/mm^2$$

Design compressive force is given by

$$P_d = [A f_{cd}] = \frac{1136 \times 112}{1000}$$

$$= 127\ kN > 54.25\ kN.$$

b) <u>member DE</u>

maximum service load tension = 12.83 kN

maximum factored load tension = (1.5 × 12.83)

= 19.24 kN

maximum service load compression = 9.57 kN

maximum factored load compression =

= (1.5 × 9.57) = 14.35 kN.

Effective length = 3m

Try a single angle ISA 50×50×5 mm

connected by 6mm thick gusset plate

the junction, with two bolts of 16mm at 50mm

Gross Area (A) = 479 mm^2 , r_{min} = 15.2mm

using 16mm dia bolts,

$$A_{nc} = [50 - 18]\,5 = 160\ mm^2$$

$$A_{go} = [50 - 5]\,5 = 225\ mm^2$$

$$A_g = 479\ mm^2$$

(a) Strength governed by yielding of gross section.

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{479 \times 250}{1.10} \times 10^{-3} = 108.8\ kN$$

b) strength governed by rupture of critical section.

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

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

$$= 1.4 - 0.076 \left(\frac{50}{5}\right)\left(\frac{250}{410}\right)\left(\frac{50+25}{50}\right)$$

$$= 0.70$$

$$T_{dn} = \frac{0.9 \times 160 \times 410}{1.25} + \frac{0.70 \times 225 \times 250}{1.10} \times 10^{-3}$$

$$= 83.02 \text{ KN} = T_0$$

c) Strength governed by block shear.

$$A_{vg} = 5[50+50] = 500 \text{ mm}^2$$

$$A_{vn} = 5[50+50] - [1.5 \times 18] = 473 \text{ mm}^2$$

$$A_{tg} = [5 \times 25] = 125 \text{ mm}^2$$

$$A_{tn} = [(5 \times 25) - (0.5 \times 18)] = 116 \text{ mm}^2$$

The block shear strength is the smaller of the value of T_{db1} and T_{db2} as computed using the equations given below.

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\, \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$= \left[\frac{500 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 116 \times 410}{1.25}\right] \times 10^{-3}$$

$$= 99.92 \text{ KN}$$

$$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\, \gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 473 \times 410}{\sqrt{3} \times 1.25} + \frac{125 \times 250}{1.10} \times 10^{-3}$$

$$= 109.12 \text{ kN}$$

Hence $T_{db} = 109.12$ kN

The design shear strength is the least of the three value computed under (a), (b) and (c) which are 108.8, 83.02, 109.12 kN.

The design tensile strength of angle

$$= 83.02 \text{ kN} > 19.24 \text{ kN}$$

c) member BC and EB.

Service load compressive force = 6.95 kN

factored compressive force = (1.5 × 6.95)

$$= 10.42 \text{ kN}$$

Service load tensile force = 6.38 kN

factored tensile force = (1.5 × 6.38)

$$= 9.57 \text{ kN}$$

Effective length $= KL = (0.7 \times 1.6) = 1.12\,m$

Use minimum size angle ISA $50 \times 50 \times 5$ mm

Area $A = 479\ mm^2$, $r_{min} = 9.7$ mm

Slenderness ratio, $\lambda = (1120 / 9.7) = 115$

the stress reduction factor χ corresponding to $f_y = 250\ N/mm^2$ and $\lambda = 115$

$\chi = 0.39$

Design compressive stress is computed as

$$f_{cd} = \left(\frac{\chi f_y}{\gamma_{mo}}\right) = \left(\frac{0.39 \times 250}{1.25}\right) = 78\ N/mm^2$$

Design compressive force is given by

$$P_d = [A f_{cd}] = \left[\frac{479 \times 78}{1000}\right]$$

$$= 37.36\ kN > 10.42\ kN$$

d) member EA and EF.

max. service load tension = 32.21 KN

Factored tension = (1.5 × 32.21) = 748.31 KN

max. service load compression = 18.84 KN

Factored compression = (1.5 × 18.84) = 28.26 KN

length of member = 3.33 m

Effective length (KL) = (0.7 × 3.33)

= 2.331 m

Try minimum to angles ISA 50 × 50 × 6 mm connect by guest plate 6 mm thick with two 16 mm dia bolts spaced at 50 mm.

$A = (2 \times 568) = 1136\ mm^2$

$r_{min} = 15.1\ mm$

i) Design strength due to yielding of cross Section:

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{1136 + 250}{1.10} \times 10^{-3}$$

$$= 258\ KN.$$

ii) Design Strength governed by tearing at net Section

$$T_{dn} = \alpha A_n \frac{f_u}{\gamma_{m1}}$$

Assume a single line of 16mm. dia bolts of two number spaced 50mm apart ($\alpha = 0.6$)

$$A_n = [(50 - 18)(6 \times 2)] = 384.\text{mm}^2$$

$$T_{dn} = [0.6 \times 384 \times (410/1.25)] \times 10^{-3}$$

$$= 75.5\text{ kN} > 48.31\text{ kN}$$

Hence the angle section designed for the truss can safely resist the factored loads.

Example : 2

A beam column is to be designed to support a factored axial load of 500 kN. (tension). Factored moments Mx. acting at top and bottom of the column are 30 and 50 kNm respectively. Effective length of column may be taken as 3.2 m. Assuming $f_y = 250 \, N/mm^2$, design the beam column section and check the same to conform to the specification of the Indian Standard code IS 800 : 2007

Step 1 : Data :

Factored axial load = 600 kN (tension

Bending moment at top = 30 kNm.

Bending moment at bottom = 50 kNm.

Yield stress of steel = 250 N/mm²

Step 2: Selection of beam column section.

$$M_{dx} = \frac{Z_e f_y}{\gamma_{mo}} = \frac{62 \times 10^4 \times 250}{1.1 \times 10^6}$$

$$= 140.7 \, kN.m.$$

$$T_{dg} = \left(\frac{f_y A_g}{\gamma_{mo}}\right) = \left(\frac{250 * 6500}{1.10 \times 1000}\right)$$

$$= 1477.3 \text{ kN}$$

Design strength due to rupture of critical section :

$$T_{dn} = \left(\frac{0.9 f_y A_n}{\gamma_{m1}}\right) = \left(\frac{0.9 \times 415 \times 6500}{1.25 \times 1000}\right)$$

$$T_{dn} = 1942.2 \text{ kN}.$$

The design strength $T_d = 1477.3$ kN.

Step 3 : Check for resistance of cross-section to combined effects.

using the interaction equation.

$$\left[\frac{N}{N_d} + \frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}}\right] \leq 1.0$$

$$N_d = \frac{A_g f_y}{\gamma_{mo}} = \frac{6500 \times 250}{1.1 \times 1000}$$

$$= 1477.3 \text{ kN}.$$

$M_x = 50$ kNm and $M_{dx} = 140.7$ kNm.

$$\therefore \left[\frac{600}{1477.3} + \frac{50}{140.7}\right] = 0.756 < 1$$

Step 4 Check for lateral torsional buckling resistance.

The value calculated in Example 18.2 is 127.3 kN.m

Reduced effective moment is computed as

$$M_{eff} = [M - \psi T\, Z_{ec}/A] \leq M_d$$

$$= \left[(50 \times 10^6) - \frac{0.8 \times 600 \times 10^3 \times 619 \times 10^3}{6500}\right]$$

$$= (4.3 \times 10^6)\ \text{Nmm} = 4.3\ \text{kNm} < 127.3\ \text{kN}$$

Step 5 : Check for overall buckling strength

$$\left(\frac{P}{P_{dx}} + \frac{M_{eff}}{M_{dx}}\right) \leq 1.0$$

$$\left(\frac{600}{1477.3} + \frac{4.3}{127.3}\right) = 0.439 < 1.0$$

$$= 0.439 < 1.0$$

UNIT V

GIRDERS AND CONNECTIONS

Plate girders - Behavior of components - Design of welded plate girders - Design of industrial gantry girders - Design of eccentric shear and moment resisting connections

Course outcome : Design and draw plate girders and gantry girders as per code provisions

Design example (plate girder with thick web plate)

Example 1

Design a welded plate girder of 20m span to support a uniformlly distribution live load of 75 KN/m over the span using the following data;

Step 1 Data:

Effective span of the girder = 20m

Distributed live load = 50 KN/m

yield stress of steel = 250 N/mm²

Step 2

factored load on girder = (1.5 × 70 × 20)

= 2100 KN

Assume self-weight (g) = [Total load/200]

= [2100 /200] ≃ 10 KN/m.

Total factored load (70 + 10)

= 80 KN/m.

Step 3 Bending moments and shear forces

$$M_d = (0.125 \times 80 \times 20^2) = 4000 \text{ kNm}$$

$$V_d = (0.5 \times 80 \times 20) = 800 \text{ kN}$$

Step 4: cross section of girder.

Economical depth of the plate girder is given by the relation.

$$D = \left[\sqrt{\frac{Mk}{f_y}}\right]^{0.33}$$

M = design moment and

$$k = [d/t_w] \leq 200\varepsilon$$

d = depth of web and t_w = thickness of web

$$\varepsilon = \left[\frac{250}{f_y}\right] = \left[\frac{250}{250}\right] = 1$$

$$k = (200 \times 1) = 200$$

$$D = \left[\sqrt{\frac{4000 \times 10^6 \times 200}{250}}\right] \simeq 1500 \text{ mm}$$

Adopt overall depth (D) = 1500 mm

Allowing for 40mm flange plate

Depth of web = d = (1500 − 800) = 1420mm

If t_w = thickness of web.

$$\left(\frac{d}{t_w}\right) \leq 200\varepsilon$$

$$t_w \geq (d/200) = \left(\frac{1420}{200}\right) = 7.1 \text{ mm}$$

Shear buckling consideration, $\left(\frac{d}{t_w}\right) > 67\varepsilon$

$$\therefore \quad t_w \geq \left(d/\cancel{200}\ 67\right) = \left(\frac{1420}{67}\right) =$$

$$= 21.2 \text{ mm}.$$

Adopt 20mm thick by 1420mm deep

web plate.

Width of flange = approximately 0.2 to 0.3

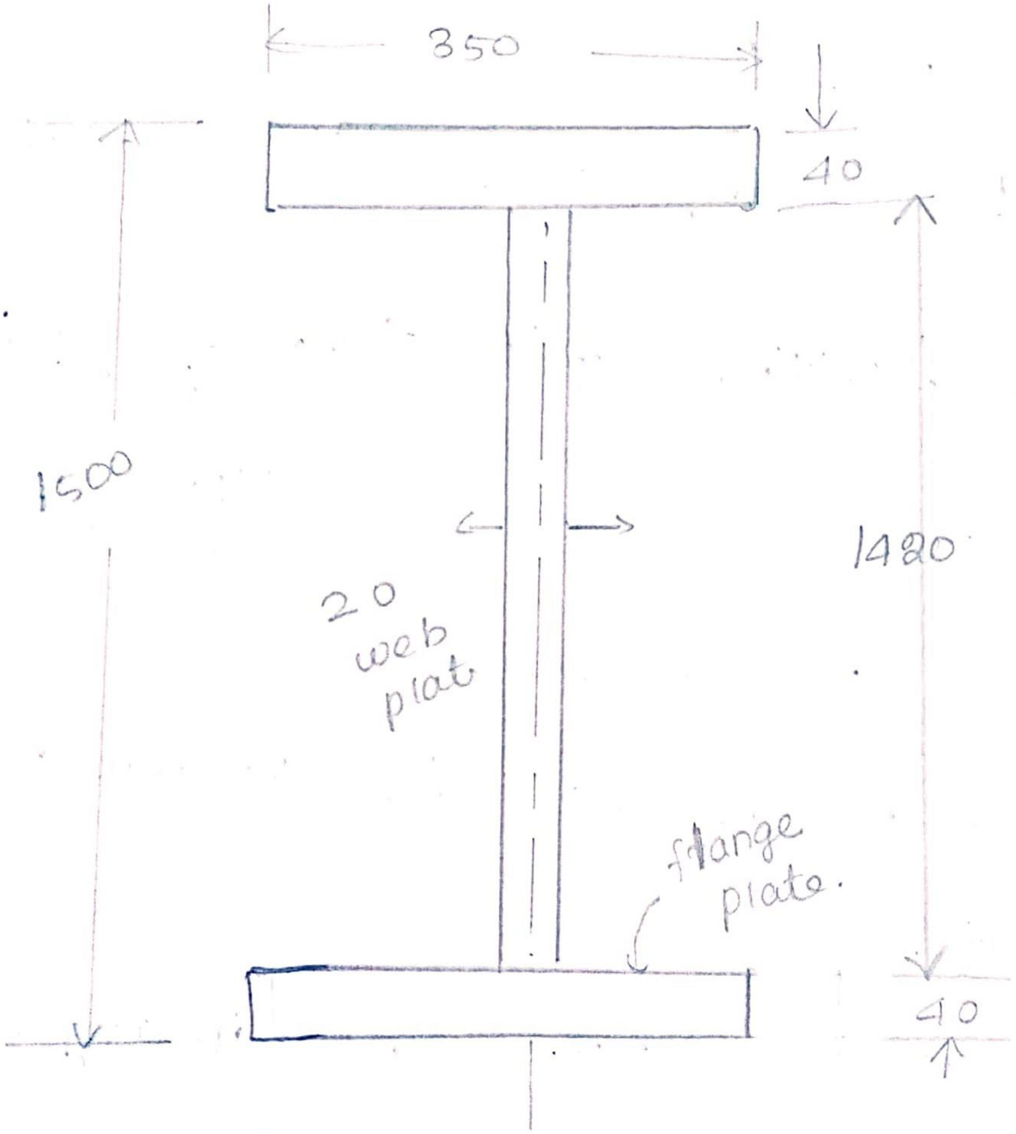
time the depth.

$$b_f = (0.2 \times 1420) = 288 \text{ mm to}$$

$$(0.3 \times 1420) = 426 \text{ mm}.$$

Adopt the width of flange = b_f = 350mm

The cross-section of the plate girder is made up of 1420 × 20mm web plate and flange plate of 350 × 40mm.



Cross sectional detail of a plate girder.

For plastic and compact section, the ratio

$$\frac{b_f}{t_f} < 8.4\,\varepsilon \text{ or } 9.4\,\varepsilon \text{ and } \varepsilon = 1$$

$f_y = 20\ N/mm^2$

$$\frac{b_f}{t_f} = \frac{350}{40} = 8.7$$

The ratio satisfies the plastic section requirement

<u>Step 5</u> : moment capacity

The moment capacity of the plate girder section is

$$M_d = \left[\beta_b Z_p f_y / \gamma_{mo}\right]$$

where $\beta_b = 1.0$ for plastic section

$$\text{plastic modulus} = Z_p = \left[\frac{2\, b_f\, t_f\, (D - t_f)}{2} + \frac{t_w d^2}{4}\right]$$

$$= \left[\frac{(2 \times 350 \times 40 \times 1460)}{2} + \frac{(20 \times 1420^2)}{4}\right]$$

$$= (30.52 \times 10^6)\ mm^3$$

$$M_d = \left(\frac{1 \times 30.52 \times 10^6 \times 250}{1.10 \times 10^6} \right)$$

$= 6936 \text{ kNm} > 4000 \text{ kN.m}$

Hence the section is safe to resist the applied moment.

<u>Step 6</u> : Shear capacity

Nominal plastic shear resistance =

$$V_n = V_p = \left(\frac{A_v f_{yw}}{\sqrt{3}} \right)$$

For welded sections, $A_v = (d\, t_w)$

Design shear strength, $V_d = \left(\frac{V_n}{\gamma_{m0}} \right)$

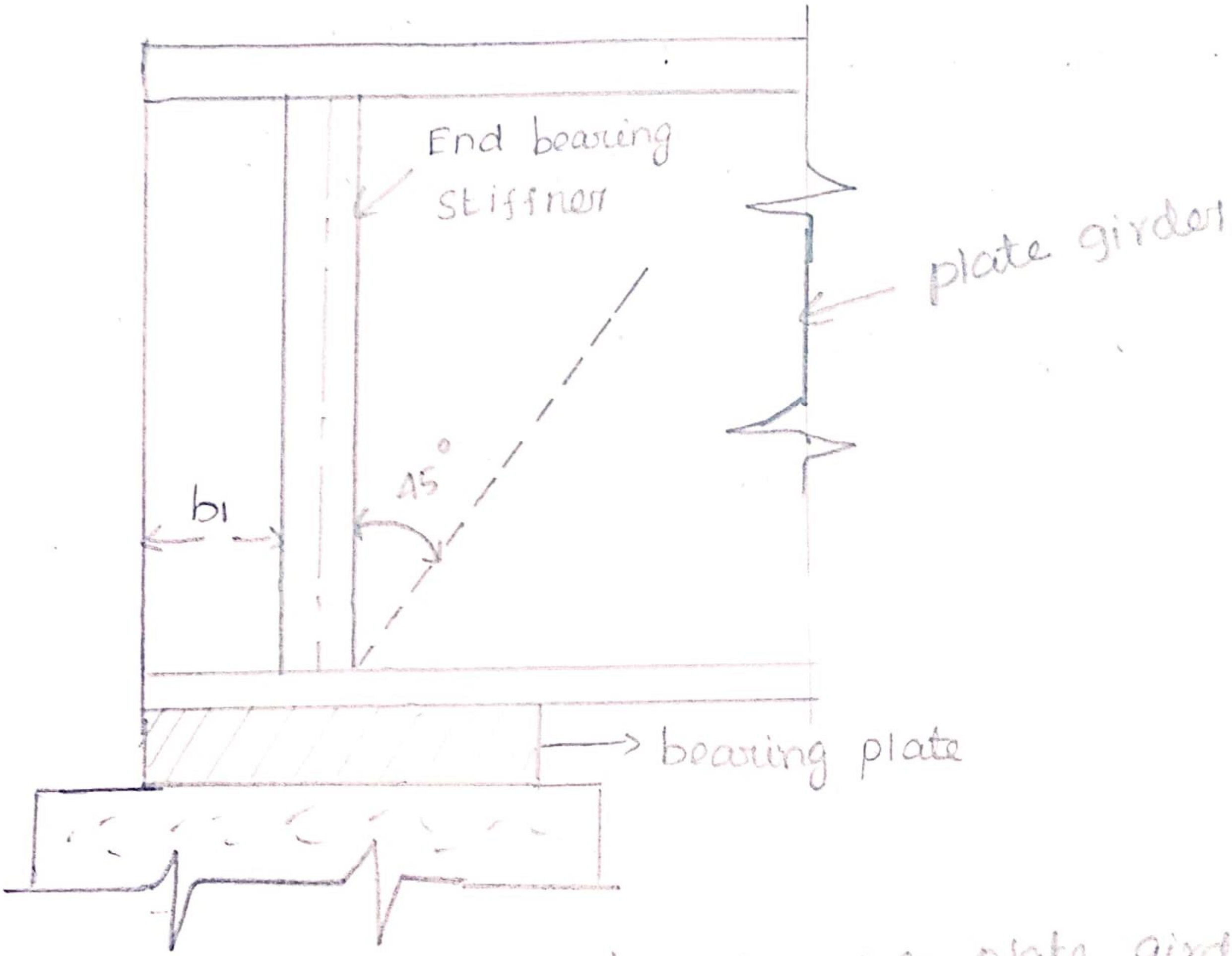
$$= \left[\frac{d\, t_w\, f_{yw}}{\sqrt{3}\, \gamma_{m0}} \right]$$

$$= \left[\frac{1420 \times 20 \times 250}{\sqrt{3} \times 1.10 \times 10^3} \right] = 3730 > 800 \text{ kN}$$

Hence the section designed is safe against shear forces.

Step 7:

Assume the width of support as 350mm.



stiff bearing length of a plate girder

minimum stiff bearing length provided by support $= b_1 = \left(\frac{350}{2}\right) = 175mm.$

According to code specification, assume the slope dispersion 1:2.5, the dispersion length

$= n_2 = (2.5 \times 40) = 100mm$

local shear capacity of web. $= F_w =$

$= (b_1 + n_2)\, t_w\, (f_y / \gamma_{mo})$

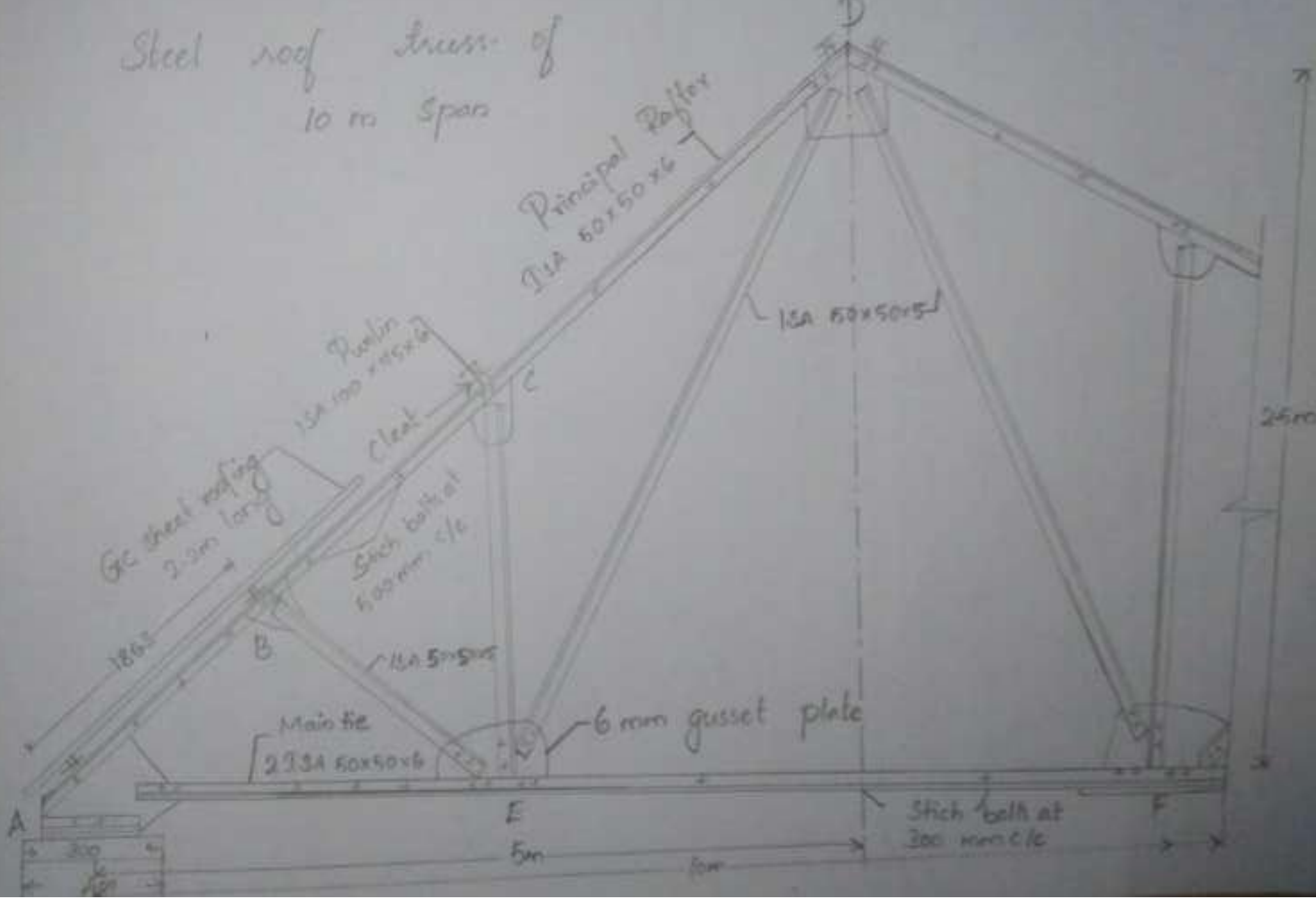
$F_w = (175 + 100)\, 20\, (250 / 1.10)$

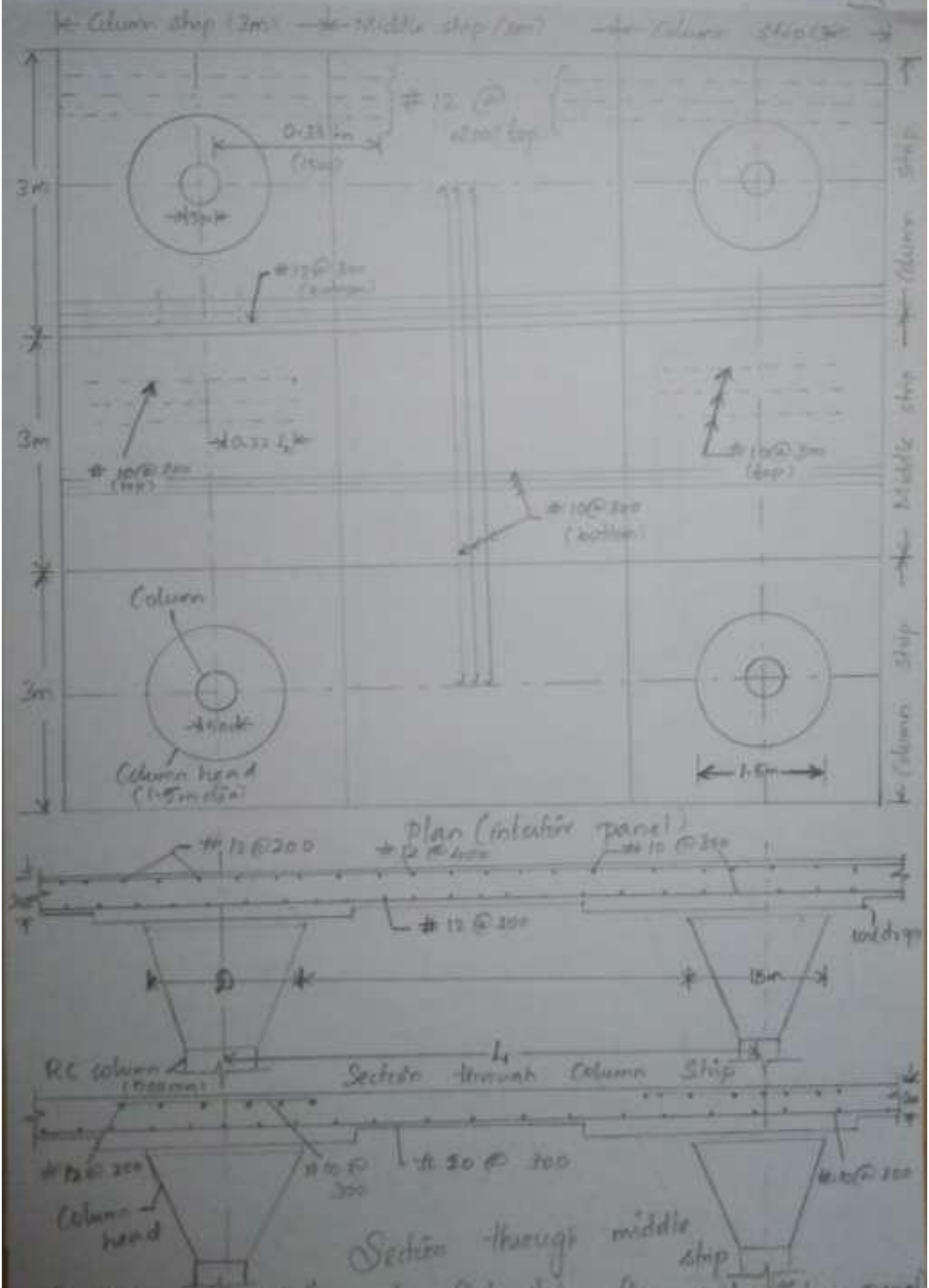
$= (1250 \times 10^3)\, N$

$= 1250\, kN >$ support reaction (V)

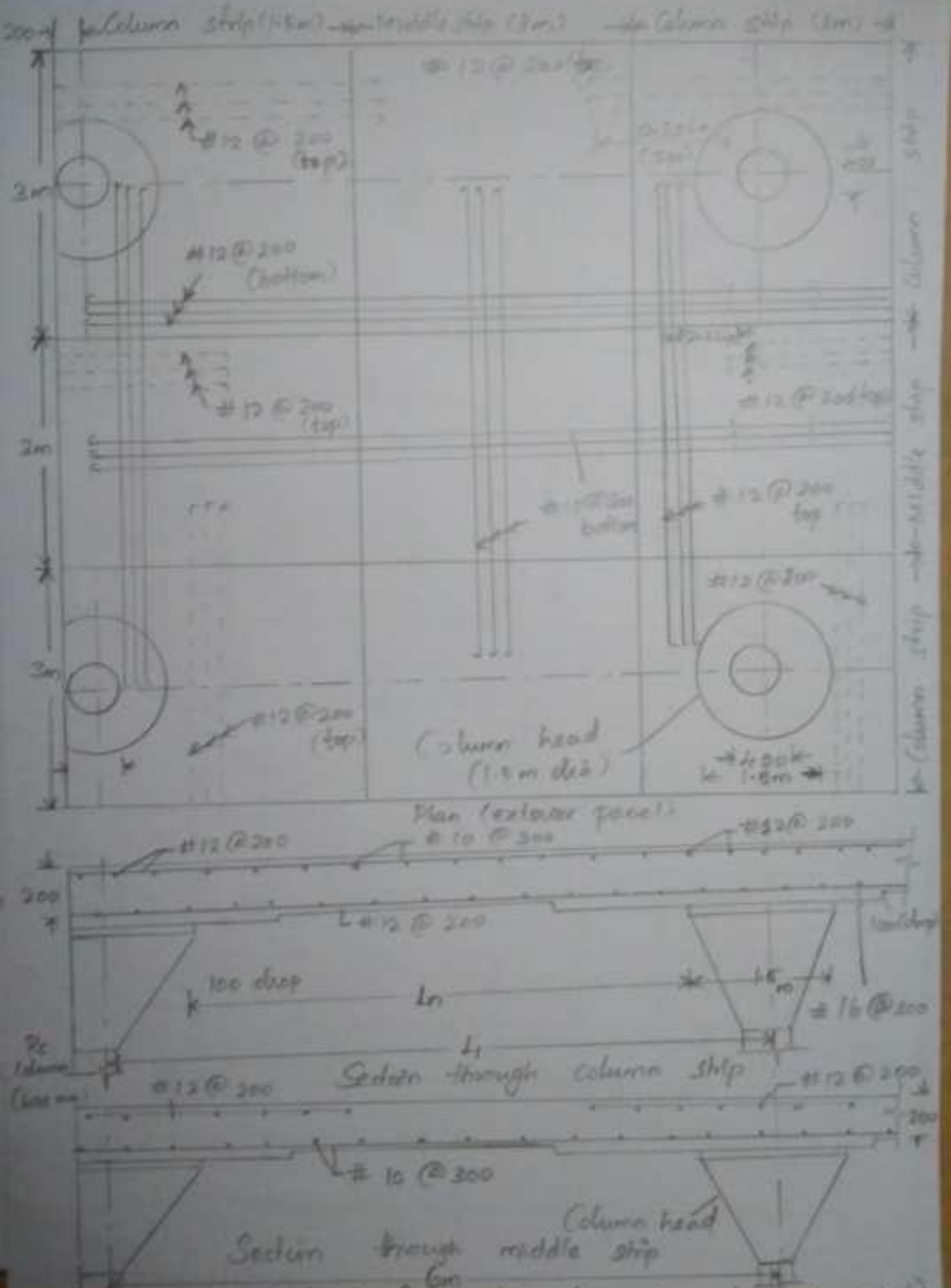
$V = 800\, kN.$

Steel roof truss of 10 m span



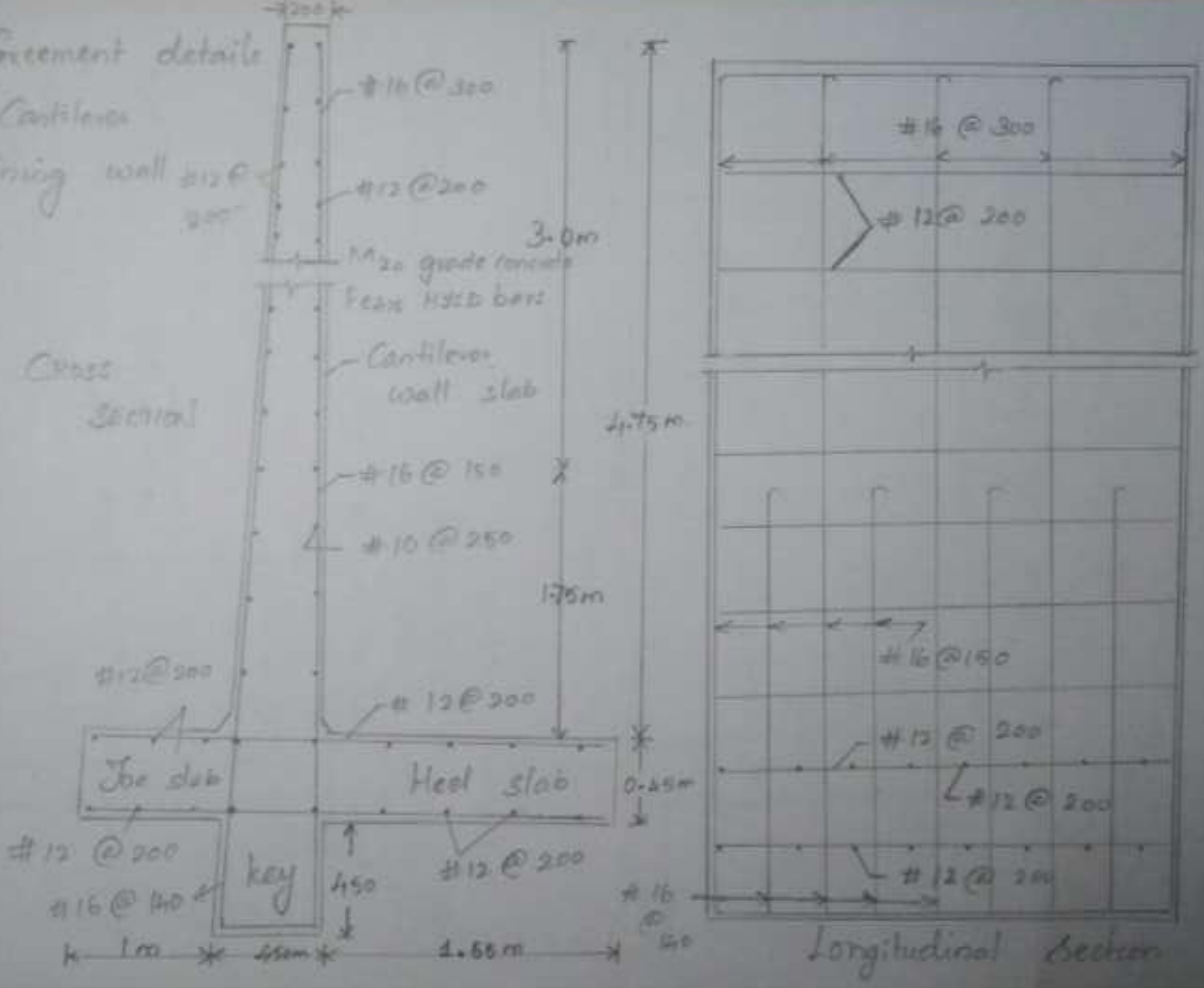


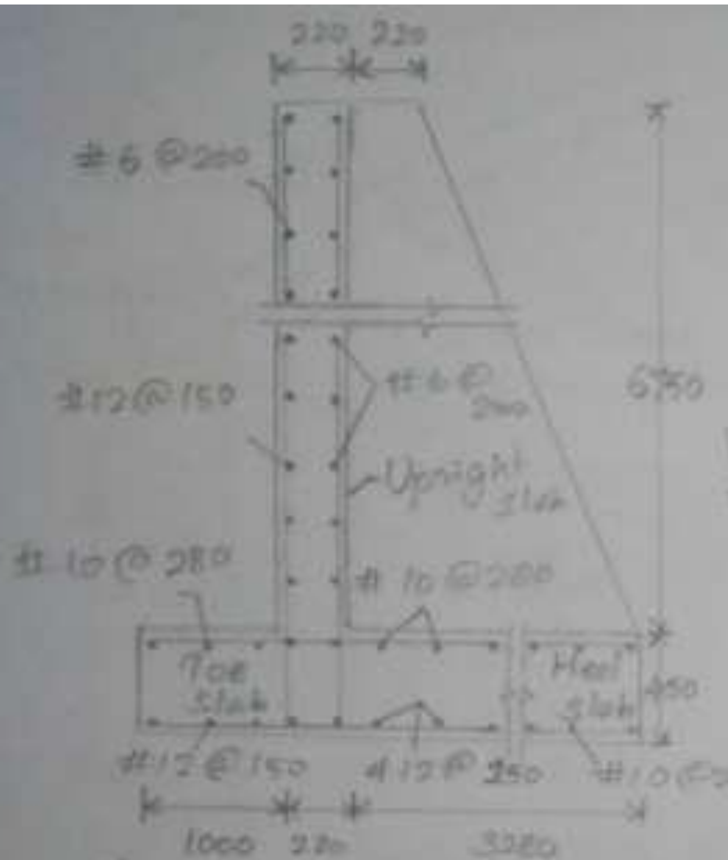
Reinforcement details in flat slab floor system (interior panel)



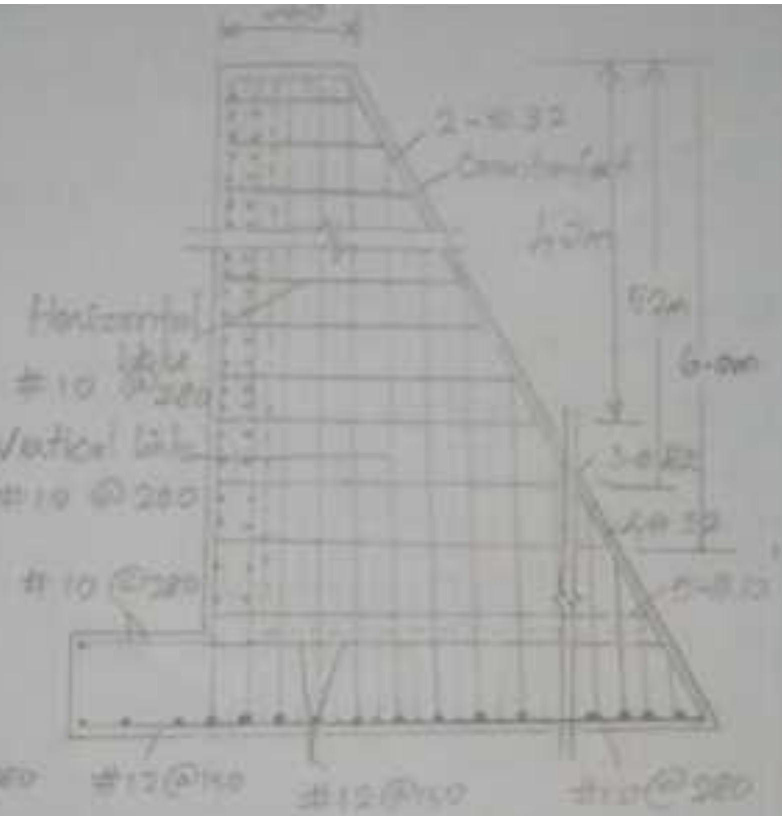
Reinforcement details in flat-slab floor system (exterior panel)

Reinforcement details in Cantilever retaining wall

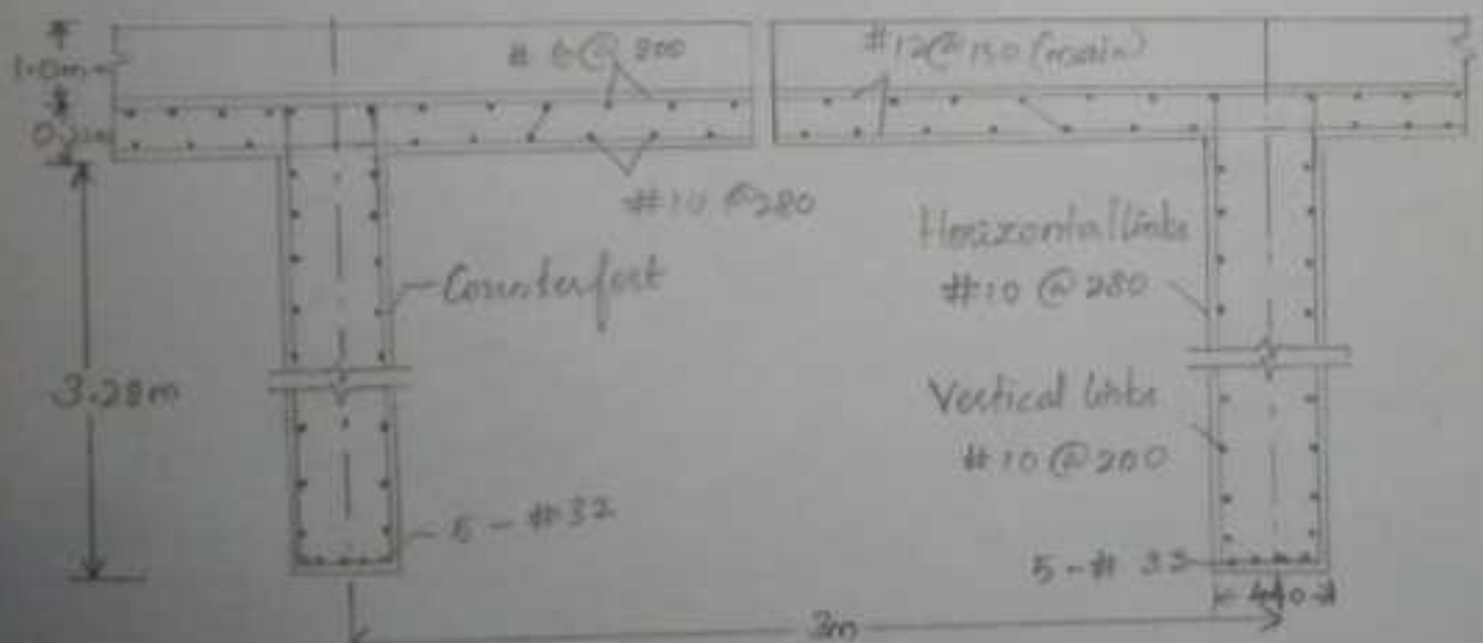




Sectional elevation midway between supports

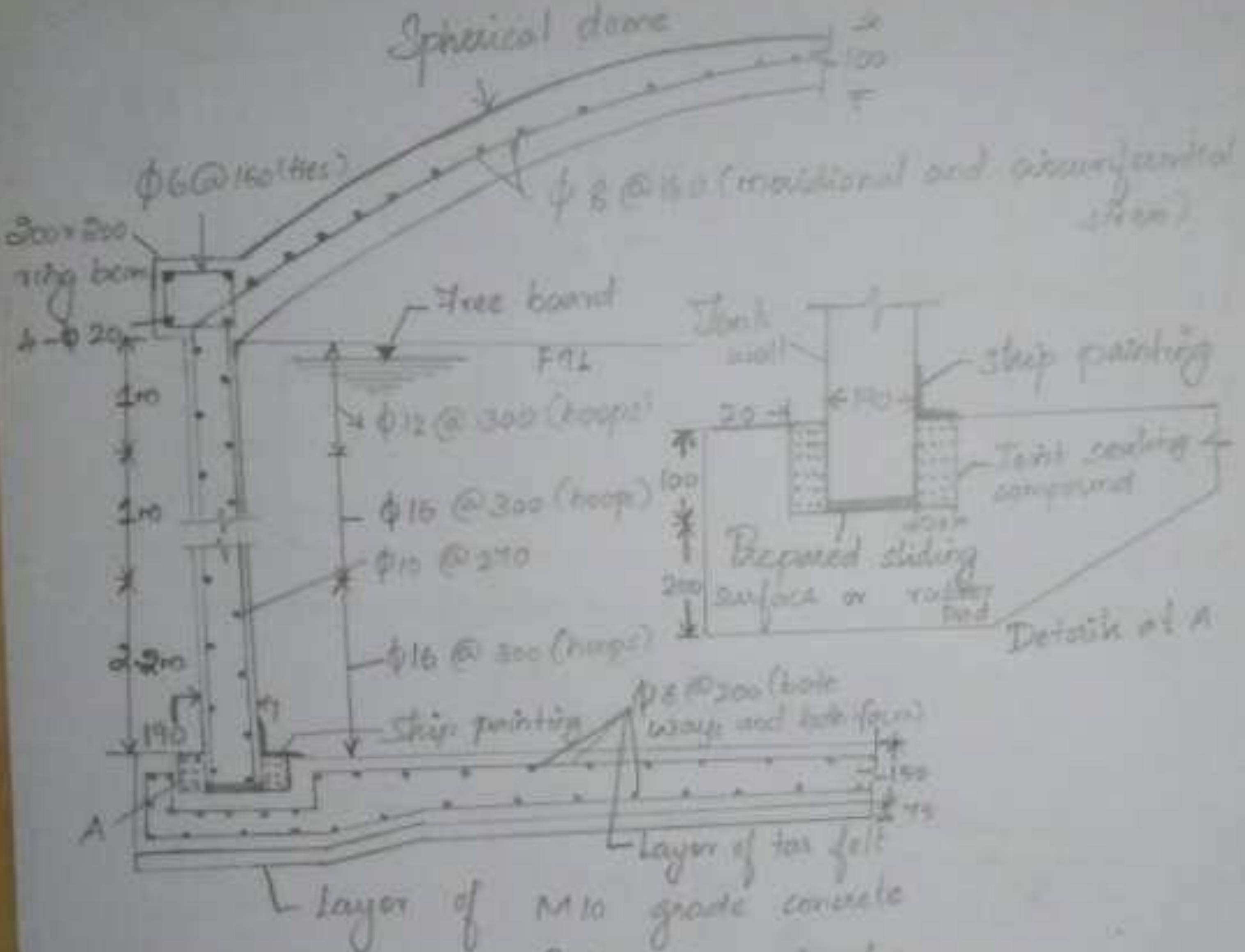


Sectional elevation at counterfort

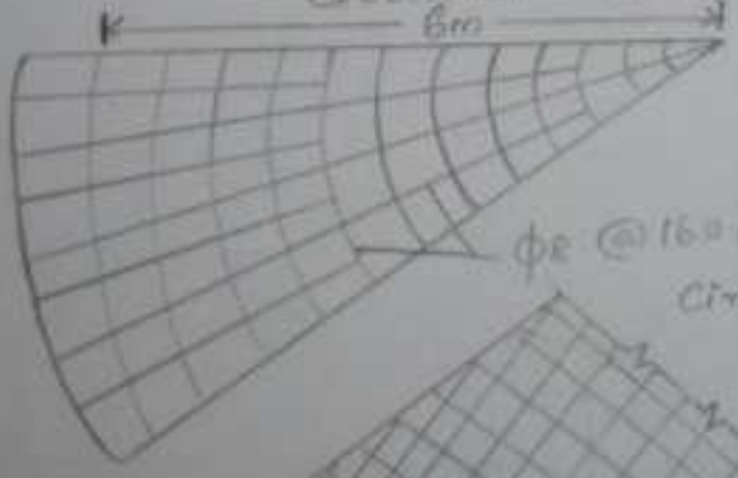


Sectional plan at base of counterforts

Reinforcement details in counterfort retaining walls

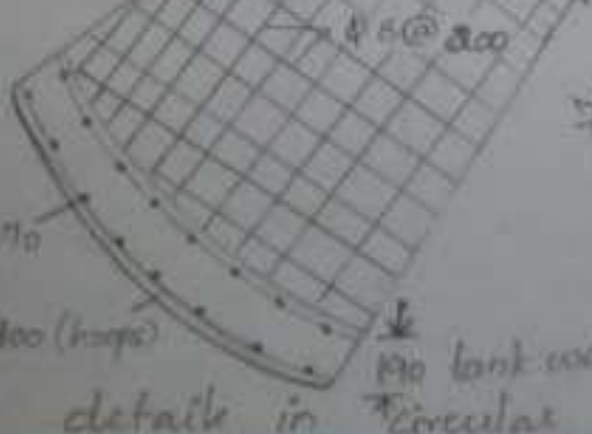


Sectional elevation



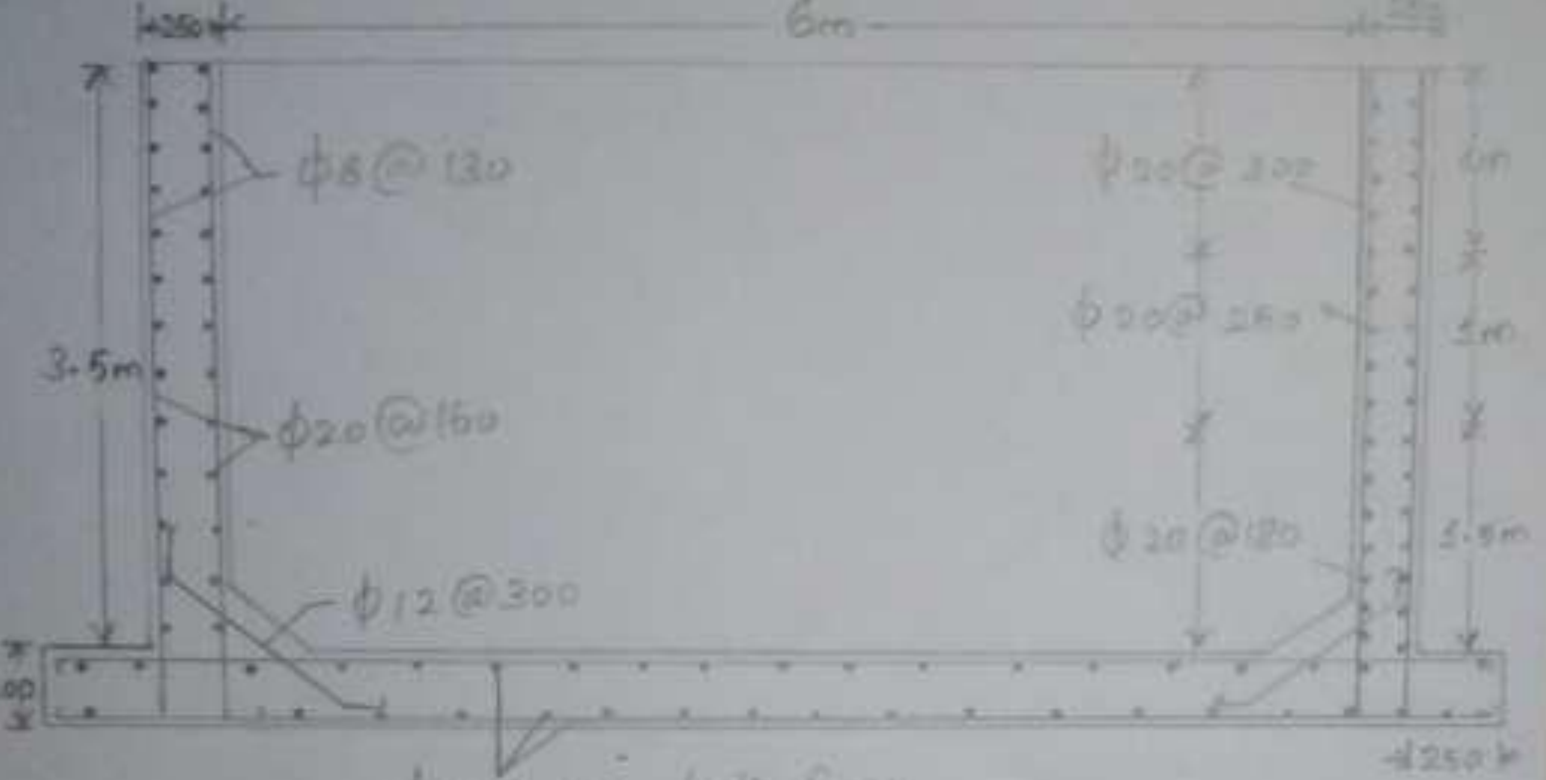
Top dome reinforcements

Plan.

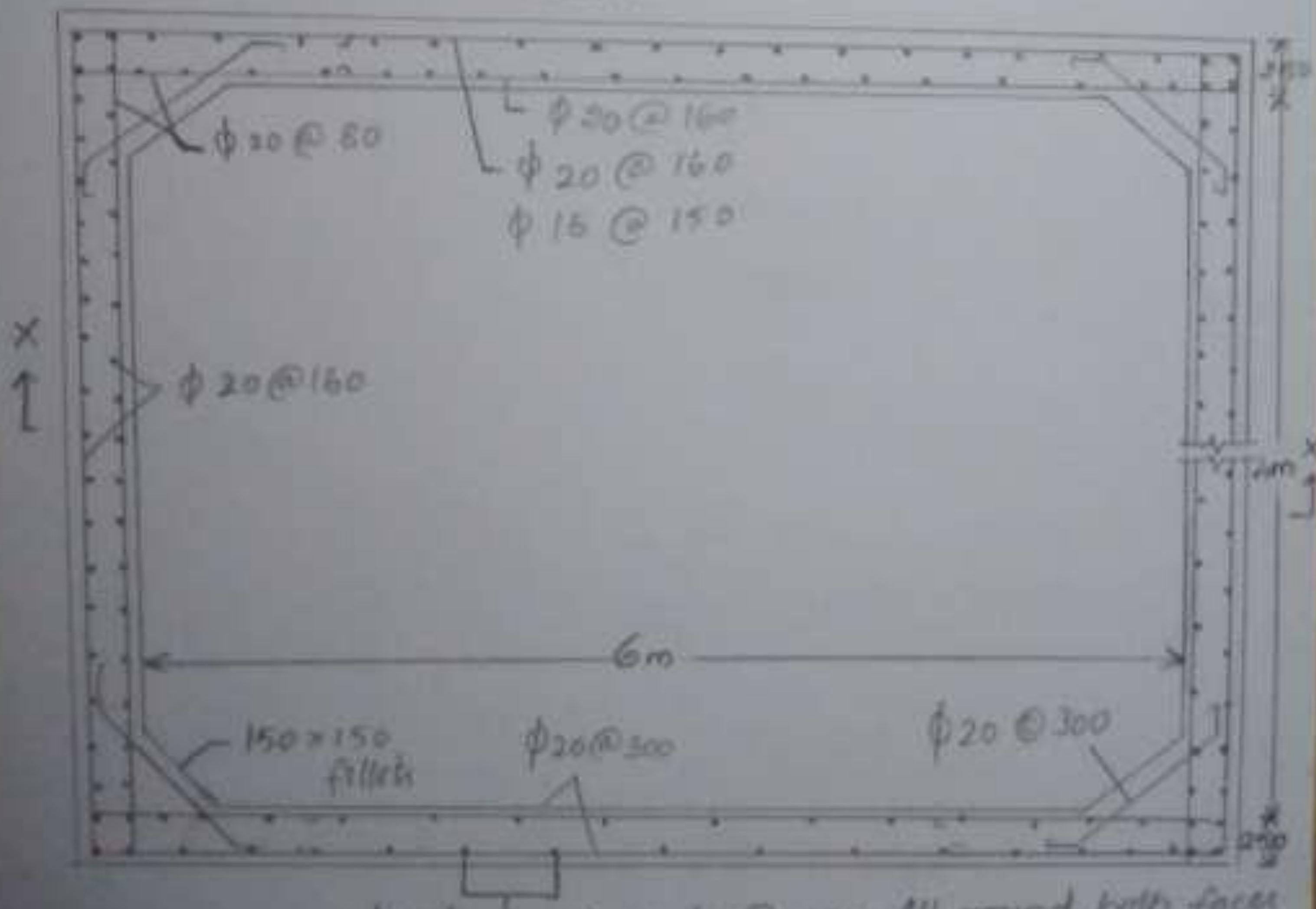


Floor slab reinforcement

Reinforcement detail in circular water tank.



Section xx



Vertical steel ϕ8 @ 130 All round both faces

Reinforcement details in rectangular tank.