B. E. CIVIL ENGINEERING
Choice Based Credit System (CBCS) and Outcome Based Education (OBE)
SEMESTER - VII

DESIGN OF RCC AND STEEL STRUCTURES

Course Code	**18CV72**	CIE Marks	40
Teaching Hours/Week(L:T:P)	(3:0:0)	SEE Marks	60
Credits	03	Exam Hours	03

Course Learning Objectives: This course will enable students to

1. Provide basic knowledge in the areas of limit state method and concept of design of RC and Steel structures
2. Identify, formulate and solve engineering problems in RC and Steel Structures
3. Give procedural knowledge to design a system, component or process as per needs and specifications of RC Structures like Retaining wall, Footing, Water tanks, Portal Frames and Steel Structures like Roof Truss, Plate Girder and Gantry Girder.
4. Imbibe the culture of professional and ethical responsibilities by following codal provisions in the analysis, design of RC and Steel Structures.
5. Provide factual knowledge on analysis and design of RC Structural elements, who can participate and succeed in competitive examinations.

Module -1

Footings: Design of rectangular slab, slab-beam type combined footing.
Retaining Walls: Design of cantilever Retaining wall and counter fort retaining wall.
Water Tanks: Design of circular water tanks resting on ground (Rigid and Flexible base). Design of rectangular water tanks resting on ground. **As per IS: 3370 (Part IV).**
Design of portal frames with fixed and hinged based supports.

Module -2

Roof Truss: Design of roof truss for different cases of loading, forces in members to given.
Plate Girder: Design of welded plate girder with intermediate stiffener, bearing stiffener and necessary checks
Gantry Girder: Design of gantry girder with all necessary checks.

Course Outcomes: After studying this course, students will be able to:

1. Students will acquire the basic knowledge in design of RCC and Steel Structures.
2. Students will have the ability to follow design procedures as per codal provisions and skills to arrive at structurally safe RC and Steel members.

Question Paper Pattern:

- Two questions shall be asked from each module. There can be maximum of three subdivisions in each question, if necessary.
- One full question should be answered from each module.
- Each question carries 50 marks.
- Code books – IS 456, IS 800, IS 3370 (Part IV), SP-16, SP (6) – Steel Tables, shall be referred for designing. The same will be provided during examination.

Textbooks:

1. N Krishna Raju, **"Structural Design and Drawing of Reinforced Concrete and Steel"**, University Press
2. Subramanian N, **"Design of Steel Structures"**, Oxford university Press, New Delhi
3. K S Duggal, **"Design of Steel Structures"**, Tata McGraw Hill, New Delhi

Reference Books:

1. Charles E Salman, Johnson & Mathas, **"Steel Structure Design and Behavior"**, Pearson Publications
2. Nether Cot, et.al, **"Behavior and Design of Steel Structures to EC -III"**, CRC Press
3. P C Verghese, **"Limit State Design of Reinforced Concrete"**, PHI Publications, New Delhi
4. S N Sinha, **"Reinforced Concrete Design"**, McGraw Hill Publication

Eg:-1 Type-I Roof Truss

Line diagram of a roof truss with External load and forces in each member along with nature are shown in figure. Design the various members of the roof truss along with their end connections with gusset plate [Welded (or) Bolted].

Also design the supports consisting of shoe angles and bearing plate for the support reaction.

Also design Anchor bolts for an uplift force of 15 kN at each support. Take M_{20} Concrete for the Column. The right support may be considered as anchoring with sliding provision. The left support may be considered as only anchoring support.

Draw to a Suitable Scale;

(i) Elevation of truss greater than half span,
(ii) Enlarged view of apex joint of the truss,
(iii) Enlarged view of the left support joint.

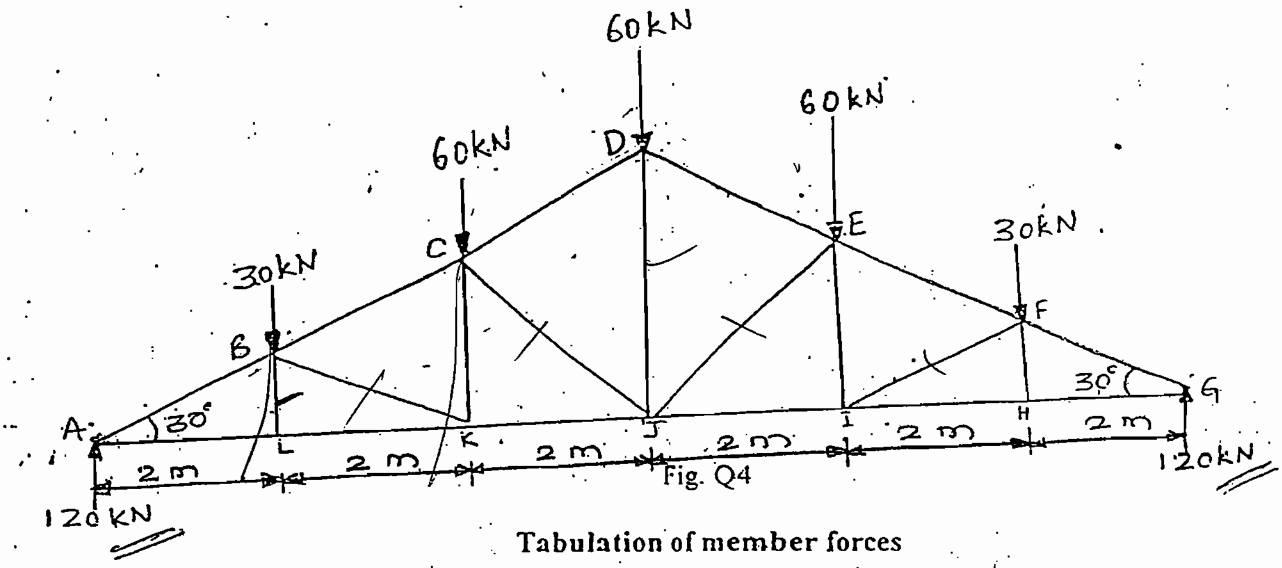


Fig. Q4

Tabulation of member forces

Members	Length (m)	Force (kN)	Nature of Force
AB, GF	2.31	240.00	Compression
BC, FE	2.31	210.00	Compression
CD, ED	2.31	160.04	Compression
AL, GH	2.00	207.84	Tension
LK, HI	2.00	207.84	Tension
KJ, IJ	2.00	181.82	Tension
BL, FH	1.154	0.00	-
BK, FI	2.31	30.00	Compression
CK, EI	2.31	15 kN	Tension
CJ, EJ	3.05	66.05	Compression
DJ	3.46	60.00	Compression

Ⓐ Design of Top chord Member:

[AB, BC and CD]

Select maximum force and max. length from top chord members.

$\therefore$ Force = 240 kN (Compression), $\boxed{l = 2.31\text{m}}$

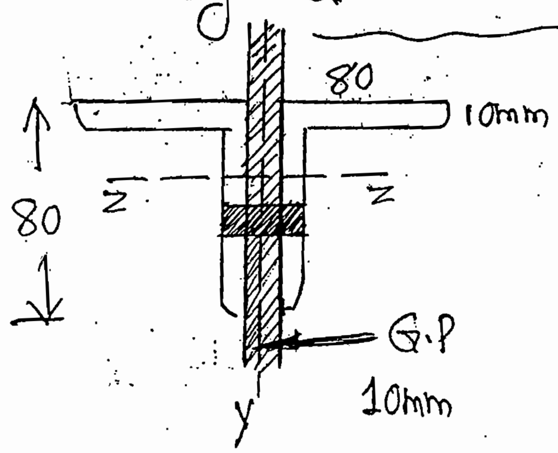
$\therefore \boxed{\text{Factored Force} = 360\,\text{kN}}$ $\therefore \boxed{l_e = 0.85\,l} = 1.9631$

Do the design as Compression Member

(i) Assume $f_{cd} = 120\,\text{N/mm}^2$

$$\therefore \text{Area})_{Req} = \frac{\text{Force}}{f_{cd}} = \frac{360 \times 10^3}{120} = 3000\,\text{mm}^2 = 30\,\text{cm}^2$$

Try 2 ISA 80mm X 80mm X 10mm



Area = $30.10\,\text{cm}^2 = 3010\,\text{mm}^2$

$r_{xx} = r_{zz} = 2.41\,\text{cm}$

$r_{yy} = 3.73\,\text{cm}$ (For 1.0cm gap)

$\therefore r_{min} = 24.1\,\text{mm}$

$$\left.\begin{array}{l}\text{Slenderness}\\ \text{Ratio}\end{array}\right\} = \lambda = \frac{l_e}{r_{min}} = \frac{1963}{24.1} = \boxed{81.45}$$

From Table 9(c), Page 42 IS-800

$\therefore f_{cd} = 133.8\ N/mm^2$

$\therefore$ Design compressive strength $\boxed{P_d = f_{cd} \cdot Area}$

$$P_d = (133.8)(3010) = 402.7 \times 10^3\ N$$
$$> 360\ kN\ (Safe)$$

(ii) Bolted Connection :

Using M22, Property class 5.6 Black Bolts.

Shear strength : $d = 22mm,\ d_o = 24mm$

$f_u(Bolt) = 500\ N/mm^2$

$f_u(plate) = 410\ N/mm^2$ $f_y = 250$

Edge distance $= \boxed{e = 1.7 d_o} = 1.7 \times 24 \approx 45mm$

Pitch $\boxed{p = 2.5d} = 2.5 \times 22 = 55mm$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left[\frac{f_u}{\sqrt{3}}\left(n_n \cdot A_{nb} + \cancel{n_s A_{sb}}\right)\right]$$

$$= \frac{1}{1.25}\left[\frac{500}{\sqrt{3}}\left(2 \times 0.78 \times \frac{\pi}{4}(22)^2 + 0\right)\right] = \underline{\underline{136.9\text{ kN}}}$$

<u>Bearing Strength</u> :

$$k_b = \frac{e}{3d_0} = \frac{45}{3 \times 24} = 0.625$$

$$= \left(\frac{p}{3d_0} - 0.25\right) = \left(\frac{55}{3 \times 24} - 0.25\right) = 0.514$$

$$= \left(\frac{500}{410}\right) = 1.22 \quad \& \quad 1.0$$

$$\therefore \boxed{k_b = 0.514}$$

$$\therefore V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left[2.5\, k_b \cdot d \cdot t \cdot f_u\right]$$

$$= \frac{1}{1.25}\left[2.5 \times 0.514 \times 22 \times 10 \times 410\right] = \underline{\underline{92.72\text{ kN}}}$$

$$\therefore \boxed{\text{Bolt Value} = 92.72\text{ kN}}$$

$$\therefore \text{No. of Bolts} = \frac{360\text{ kN}}{92.72} \simeq 4$$

Provide $\boxed{\text{2 ISA } 80 \times 80 \times 10\text{mm along with } 4\text{-}\#\,22\text{mm Bolts.}}$

(ii) Design of Bottom chord Member :

[AL, LK & KJ]

Taking Max. Force = 207.84 kN [Tension]

length = 2.0 m ∴ $\boxed{\text{Factored} = 311.76\,kN}$

(a) For Preliminary sizing

$$\boxed{T_{dn} = \frac{\alpha \cdot A_n \cdot f_u}{\gamma_{m1}}} \longrightarrow \text{Page } 33$$

$\alpha = 0.8$ For 4-Bolts (Assumed)

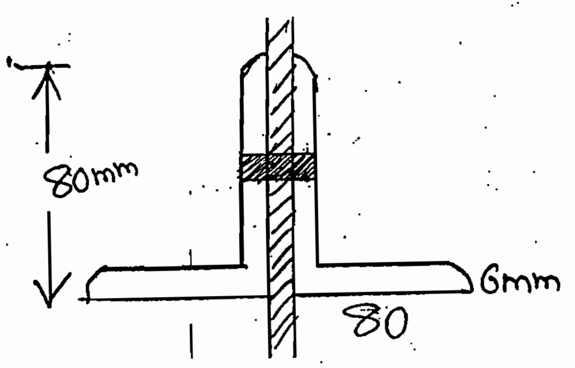
$$\therefore 311.76 \times 10^3 = \frac{0.8 \times A_n \times 410}{1.25} \qquad \therefore A_n = 1188.11\,mm^2$$

Increase approximately by 30%

$= 1.30 \times 1188.11 = 1544.5\,mm^2 = 15.44\,cm^2$

From steel Table Try 2 ISA 80 X 80 X 6 mm

Area = $18.58 \times 100\,mm^2$



(b) Connection :

Using M20, Property class 8.8 "HSFG" Bolts

Page (76) – IS-800

$\mu_f = 0.55$, $K_h = 1.0$, $n_e = 2$

$F_o = A_{nb} \cdot f_o = 0.78 \frac{\pi d^2}{4} \times 0.7 f_{ub}$

$= 0.78 \times \frac{\pi (20)^2}{4} \times 0.7 \times 800 = 137.22 \times 10^3 \, N$

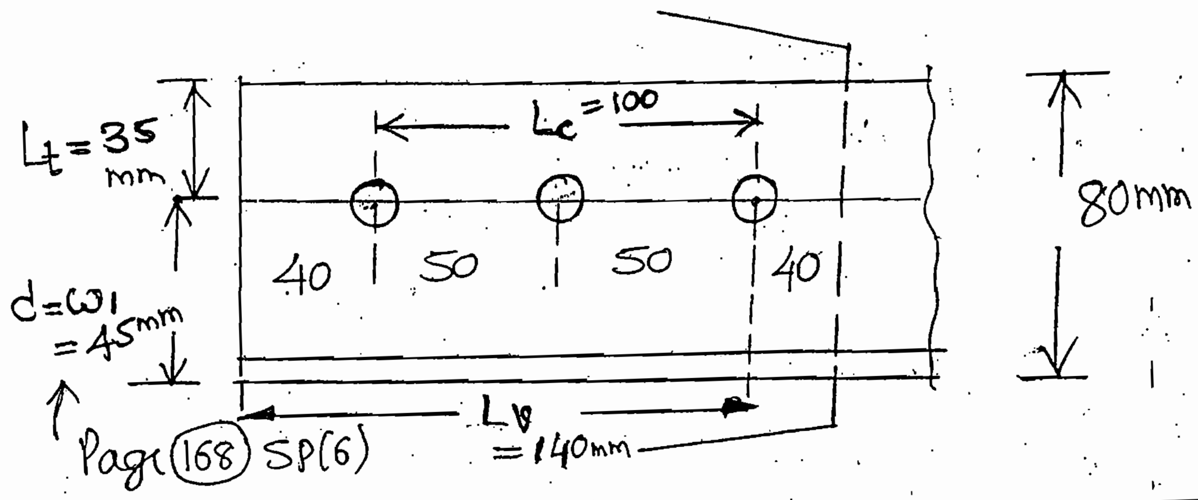
$\therefore V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{1}{\gamma_{mf}} \left[\mu_f \cdot n_e \cdot K_h \cdot F_o \right]$

$= \frac{1}{1.25} \left[0.55 \times 2 \times 1 \times 137.22 \right] = \boxed{120.75 \, kN} = B.V$

$\therefore$ No. of Bolts $= \frac{Force}{B.V} = \frac{311.76}{120.75} \approx 3$

Pitch $= 2.5 \times d = 2.5 \times 20 = 50mm$

$e = 1.7 \times d_o = 1.7 \times 22 \approx 40mm$



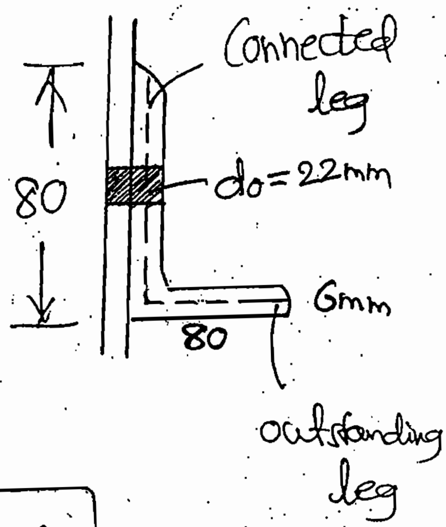
(c) Check for Rupture : [Page 33]

ω = outstanding = 80mm, $t = 6$ mm

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

$b_s = \omega + \omega_1 - t = 80 + 45 - 6 = 119$ mm

$L_c = 100$ mm



$\therefore \boxed{\beta = 0.664}$

$A_{go} = (80 - 6/2)6 = 462 \text{ mm}^2$

$A_{nc} = \left(80 - \frac{6}{2} - 22\right)6 = 330 \text{mm}^2$

$$\therefore \boxed{T_{dn} = 2\left[\frac{0.9 A_{nc} \cdot f_u}{\gamma_{m1}} + \beta \cdot \frac{A_{go} f_y}{\gamma_{m0}}\right]}$$

Double angle

$= \underline{334.27 \text{ kN}} > 311.76$ kN (safe)

(d) Check for Block Shear :

$L_v = 140$, $L_t = 35$mm, $t = 6$mm

$\therefore A_{vg} = L_v \times t = 140 \times 6 = 840$

$A_{vn} = (840 - 2.5 \times 22 \times 6) = 510$

$A_{tg} = L_t \times t = 35 \times 6 = 210$

$A_{tn} = 210 - 0.5 \times 22 \times 6 = 144$

$\therefore\ T_{db} = 2\left[\qquad\qquad\right] = 305.46\times10^3\,N$

$T_{db} = 2\left[\qquad\qquad\right] = 269.3\times10^3\,N.$

$< 311.76\ kN$ (Un-safe)

Hence Revise the section

Take 2ISA 80X80X 8mm and check

only Black shear.

$\therefore\ L_v = 140,\ L_t = 35,\ t = 8mm$

$A_{vg} = L_v \times t = 1120,\quad A_{vn} = 680$

$A_{tg} = L_t \times t = 280,\quad A_{tn} = 192$

$\therefore\ T_{db} = 2\left[\qquad\qquad\right] = 407.28\ kN$

$T_{db} = 2\left[\qquad\qquad\right] = 360\ kN$

$> 311.76\ kN$ (Safe).

$$\boxed{\text{Hence Provide 2 ISA 80X80X8mm with 3-}M_{20}\text{ HSFG 8.8 Property class Bolts.}}$$

(C) Design of Inner Compression Member :-

Members $\rightarrow$ BK, CJ & DJ.

Max. Force = 66.05 kN $\therefore$ $\boxed{\text{Factored} = 99.07\ kN}$

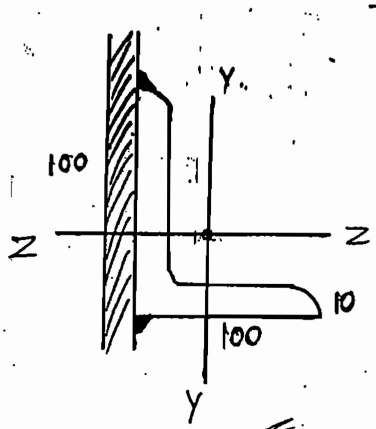
Max. Length = 3.46m $\therefore$ $\boxed{l_e = 0.85\,l = 2.94m}$

(a) Assume $f_{cd} = 60\ N/mm^2$

$\therefore$ Area Required = $\dfrac{99.07 \times 10^3}{60} = 1651.1\ mm^2$
$= 16.51\ cm^2$

Select "Single Angle" "ISA 100X100X10mm"

Area = $1903mm^2$, $r_{vv} = 19.4mm$



★ "Loaded Through One Leg" ★

For welded and assume all the joints are pin jointed (Hinged)

$\boxed{k_1 = 0.7,\ \ k_2 = 0.6,\ \ k_3 = 5}$ $\rightarrow$ Page 48

$$\lambda_{vv} = \frac{\left(\frac{l_e}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{2940}{19.4}\right)}{1\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = 1.704$$

$$\lambda_{\phi} = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(100+100)/2\times10}{1\sqrt{\frac{\pi^2 \times 2\times10^5}{250}}} = 0.112$$

$$\therefore \lambda_e = \sqrt{K_1 + K_2 \cdot \lambda_{vv}^2 + K_3 \lambda_\phi^2}$$

$$= \sqrt{0.7 + 0.6 \times 1.704^2 + 5 \times 0.112^2} = \boxed{1.58}$$

From Page 34 & 35.

$\boxed{\alpha = 0.49}$ — Buckling class 'c'

$$\therefore \phi = 0.5\left[1 + \alpha(\lambda_e - 0.2) + \lambda_e^2\right]$$

$$= 0.5\left[1 + 0.49(1.58 - 0.2) + 1.58^2\right] = \boxed{2.086}$$

$$\therefore f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left[\phi^2 - \lambda_e^2\right]^{0.5}} = \frac{(250/1.10)}{2.086 + \left[2.086^2 - 1.58^2\right]^{0.5}}$$

$$\boxed{f_{cd} = 65.91\ N/mm^2}$$

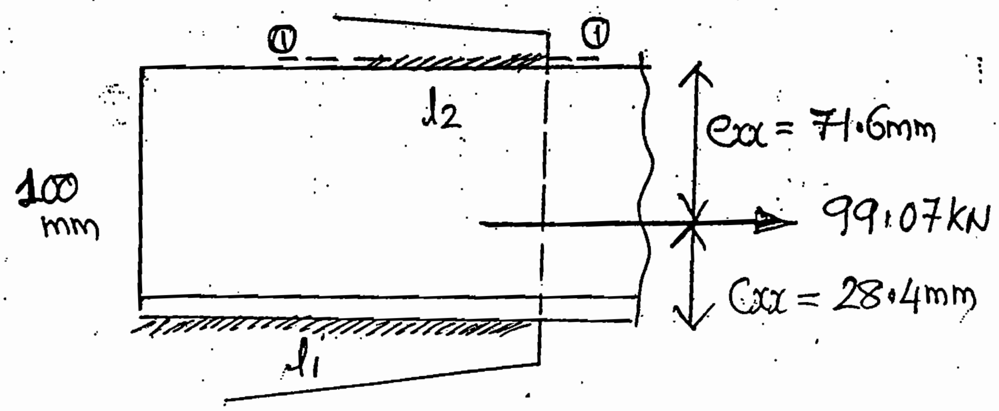
$\therefore$ Compressive load $P_d = f_{cd} \times \text{Area}$

$$P_d = (65.91)(1903) = 125.4 \times 10^3\ N,$$

> 99.07 kN

(safe)

(2) Welded connection :



Size of the weld $\boxed{s = \frac{3}{4} \times \text{Angle thickness}}$

$s = \frac{3}{4} \times 10 = 7.5$ mm Take $\boxed{s = 6\text{mm}}$

Force = Strength of the weld

$$99.07 \times 10^3 = (0.7s)(l)\left(\frac{f_u}{\sqrt{3} \cdot \gamma_{mw}}\right)$$

$$= (0.7 \times 6)(l)\left(\frac{410}{\sqrt{3} \times 1.25}\right)$$

$$\therefore l = l_1 + l_2 = 125\text{mm} \rightarrow (i)$$

& Taking moment about ①-①

$$(99.07 \times 10^3)\, 71.6 = \left[(0.7 \times 6)(l_1)\left(\frac{410}{\sqrt{3} \times 1.25}\right)\right] 100$$

$$\therefore \boxed{l_1 = 90\text{mm}}$$

Provide $\boxed{\text{ISA } 100 \times 100 \times 10\text{mm with welding } l_1 = 90\text{mm} \ \& \ l_2 = 35\text{mm}}$

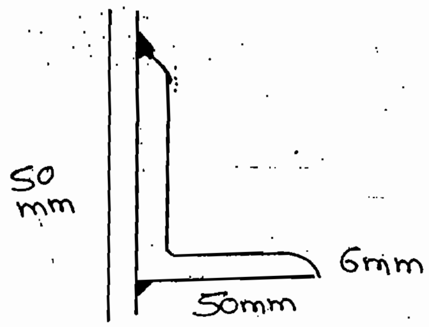
(D) Design of Inner "Tension" Member :

Member → CK

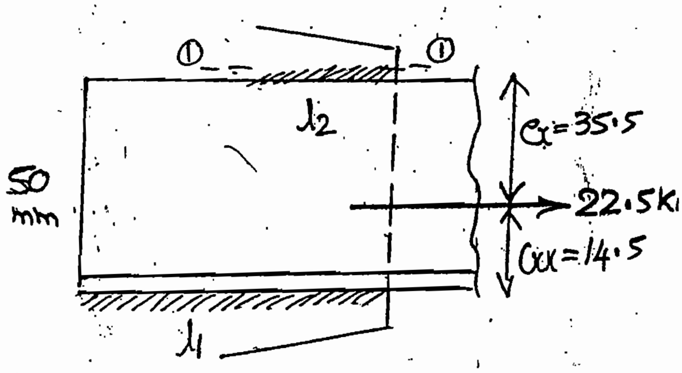
Force = 15 KN ∴ $\boxed{\text{Factored} = 22.5\,kN}$ (T)

(a) Since force is very small, Take "Min. size"

"ISA 50mm X 50mm X 6mm" Area = $568\,mm^2$



(b) Welded Connection :



$$S = \frac{3}{4} \times 6mm = 4.5mm$$

Take $\boxed{S = 4mm}$

(i) $22.5 \times 10^3 = 0.7 \times 4 \times l \times \dfrac{410}{\sqrt{3} \times 1.25}$

∴ $l = l_1 + l_2 = 45mm$ —(i)

(ii) Taking Moment

$$(22.5 \times 10^3)\,35.5 = \left[0.7 \times 4 \times l_1 \times \frac{410}{\sqrt{3} \times 1.25}\right] 50$$

∴ $\boxed{l_1 = 30mm}$ & $\boxed{l_2 = 15mm}$

(c) Check for Rupture:

$\omega = 50mm, \quad t = 6mm, \quad \boxed{b_s = \omega} = 50mm$

$L_c = l_1 + l_2 = 45mm$

$\therefore \boxed{\beta = 0.97}$

$\therefore$ ~~Tdn~~ $A_{go} = A_{nc} = (A - t/2)t$

$= (50 - 6/2)6 = 282 mm$

$\therefore T_{dn} = 145.4 kN > 22.5 kN$ (Safe)

(d) Check for "Block Shear":

$L_v = l_1 + l_2 = 45mm, \quad L_t = 50mm, \quad t = 6mm$

$A_{vg} = A_{vn} = L_v \times t = 270 mm^2$

$A_{tg} = A_{tn} = L_t \times t = 300 mm^2$

$$\left.\begin{aligned} \therefore T_{db} &= 123.9 kN \\ T_{db} &= 114.2 kN \end{aligned}\right\} > 22.5 kN \text{ (Safe)}$$

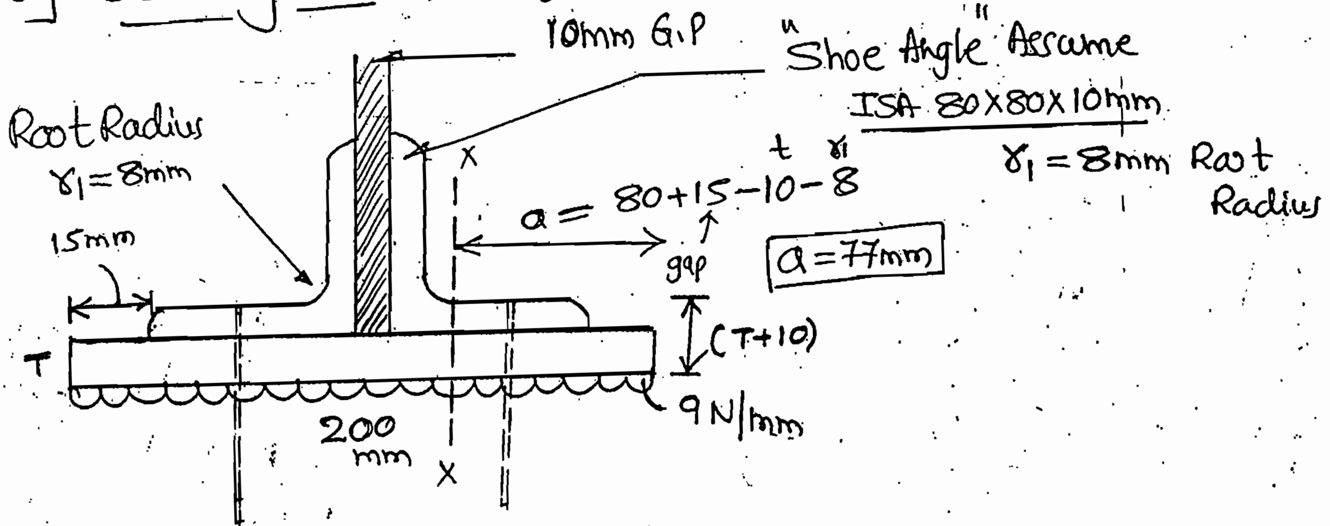
(E) Design of "Support":

(a) $\text{Reaction} = \frac{\text{Total load}}{2} = \frac{2\times30+3\times60}{2} = \boxed{120\,kN}$

$\therefore$ Factored $= 180\,kN$

Bearing Pressure of concrete $= \boxed{0.45 f_{ck}} = 0.45 \times 20 = 9\,N/mm^2$

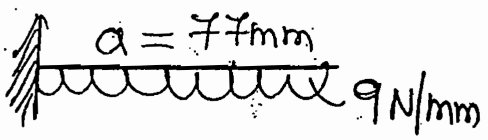
1] Bearing Plate Design:



$\text{Area of Bearing Plate} = \frac{\text{Reaction}}{0.45 f_{ck}} = \frac{180\times10^3}{9} = 20000\,mm^2$

Provide square plate $= \sqrt{20000} = 141.4\,mm$

Provide $\boxed{\text{Min. } 200mm \times 200mm}$



$M_x = 9 \times 77 \times \frac{77}{2}$

$$\boxed{M = 26.7\times10^3\,N\text{-}mm}$$

Using Design Bending strength

$$\boxed{M_d = 1.2\, Z_e \cdot \frac{f_y}{\gamma_{mo}}} \text{ — P-53}$$

$$Z_e = \left(\frac{I}{y}\right) = \frac{bd^2}{6} = \frac{(1mm)(T+10)^2}{6}$$

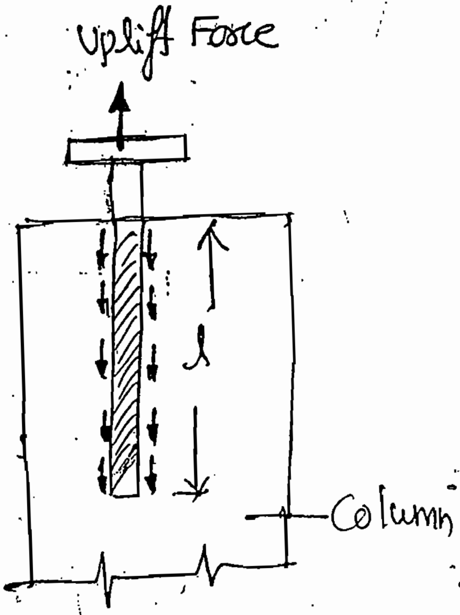
$$\therefore \quad 26.7\times10^3 = 1.2\left(\frac{1\times(T+10)^2}{6}\right)\frac{250}{1.10}$$

$$\therefore \quad T = 14.23mm \approx \underline{16mm}$$

Provide $\boxed{\text{Bearing Plate } 200mm \times 200mm \times 16mm}$

(b) "**Anchor Bolt Design**":

Using 16mm ϕ Bolt



Given Uplift Force = 15 kN at each support

Using Two anchor Bolt at each end.

$$\therefore \text{ Uplift Force for one Bolt} = \frac{15}{2} = 7.5 kN$$

$$\therefore \boxed{\text{Factored Force} = 11.25 kN}$$

Using <u>IS456 — Page 43.</u>

For M20 concrete $\longrightarrow$ Bond stress = $1.2\,N/mm^2$

For Rough surface increase by <u>60%.</u>

$$\therefore \boxed{\text{Uplift Force} = (\text{Circumference})(\text{Length})(\text{Bond stress})}$$

$$11.25 \times 10^3 = (\pi \times 16mm\,\phi)(l)(1.20\,N/mm^2 \times 1.60)$$

Increased

$$\therefore l = 116.5\,mm$$

Provide 16mm ϕ, 120mm length Anchor Bolt

$=\!=\!= \times =\!=\!=$

Date 3/4/18

Type-II ROOF TRUSS :

[With Wind load, Live Load & D.L]

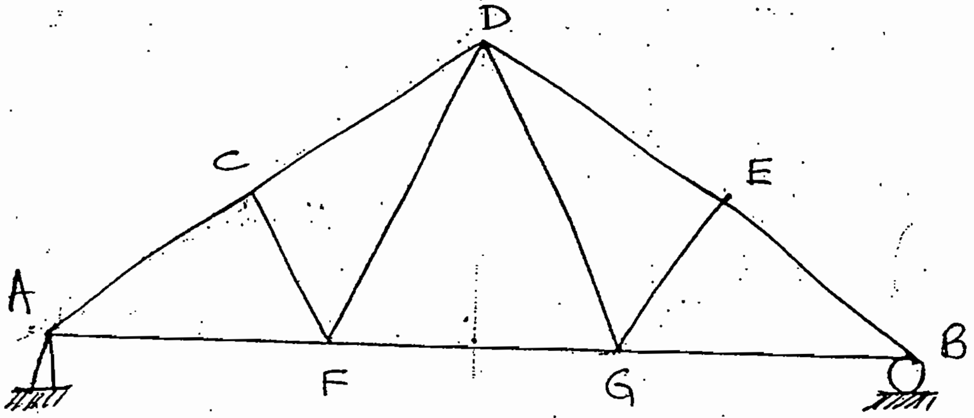
Eg:-2]

Force in the member as DL and LL and also ~~is~~ wind load is given below take posi as tension and negetive as compression. Des the truss and support given upward react at support is equal to 180 KN, uplift force is equal to 50 KN. Use M16 bolt for connection. Draw the following to suitable scal

i) Half elevation of the truss.
ii) Enlarged view of apex joint
iii) Enlarged " " end "
iv) " " intermidiate joint.

Member	Dead load	Live load	Wind load	Length (m).
AC	+9.4	−30	50.4	3.46
CD	−15.7	−23.8	43.4	3.46
CF	−6.93	−10.4	19.9	4
DF	+3.74	+5.64	−11.4	4
AF	+17.35	+26.4	−42.8	2
FG	+10.39	+15.8	−21.5	4

+ve sign → Tension
−ve sign → Compression.



Load Combination :

Member	DL+LL	DL+WL	Take Max. combination		
AC	−20.6	59.8 ✓	39.5 kN (C)	l_{max} = 3.46m	Top chord
CD	−39.5 ✓	27.70	59.8 kN (T)		
CF	−17.33 ✓	12.97 ✓	17.33 kN (C)	l_{max} = 4.0m	Inne
DF	9.38	−7.66	12.97 kN (T)		
AF	43.75 ✓	−25.45 ✓	25.45 kN (C)	l_{max} = 4m	Bottom chor
FG	26.19	−11.11	43.75 kN (T)		

Bolted Connection

(A) Design of Top chord Member :

Members → AC & CD

Forces → 39.50 kN (C) → 59.25 kN (C) ✓

59.80 kN (T) → 89.70 kN (T) ✓

∴ Factored

$l_{max} = 3.46m$ ∴ $\boxed{l_e = 0.85 l} = 2.94m$

Note:- Since Tension force is more, start the design like a tension member and later check for compression.

(a) Area Required

$$\boxed{T_{dn} = \frac{\alpha \cdot A_n \cdot f_u}{\gamma_{m1}}} \rightarrow P\text{-}33$$

Taking $\alpha = 0.6$

$$89.70 \times 10^3 = \frac{0.6 \times A_n \times 410}{1.25}$$ ∴ $A_n = 455.8 mm^2$

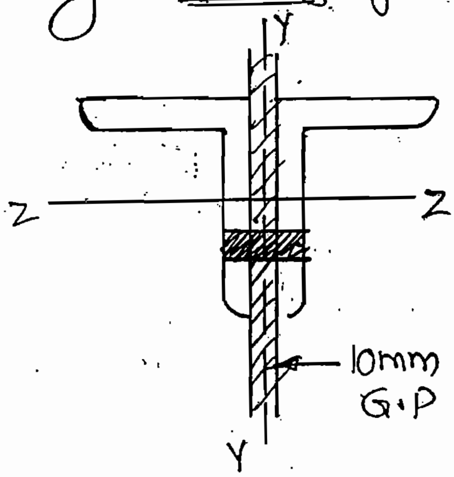
Increase the above area by 30%.

∴ Area $= 1.30 \times 455.8 = 592.54 mm^2$

$= 5.92 cm^2$

∴ From steel Table

Try Min size of angle 2 ISA $50 \times 50 \times 6mm$



Area $= 1136 mm^2$

$\gamma_{min} = 15.1 mm$

(b) Connections :

Using M16, P. Class 5.6 Bolts

Shear :

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{1.25}\left[\frac{500}{\sqrt{3}}\left(2 \times 0.78 \times \frac{\pi}{4}(16)^2 + 0\right)\right]$$

$$= 72.43 kN$$

Bearing :

$e = 1.7 d_0 = 1.7 \times 18 \approx 35 mm$

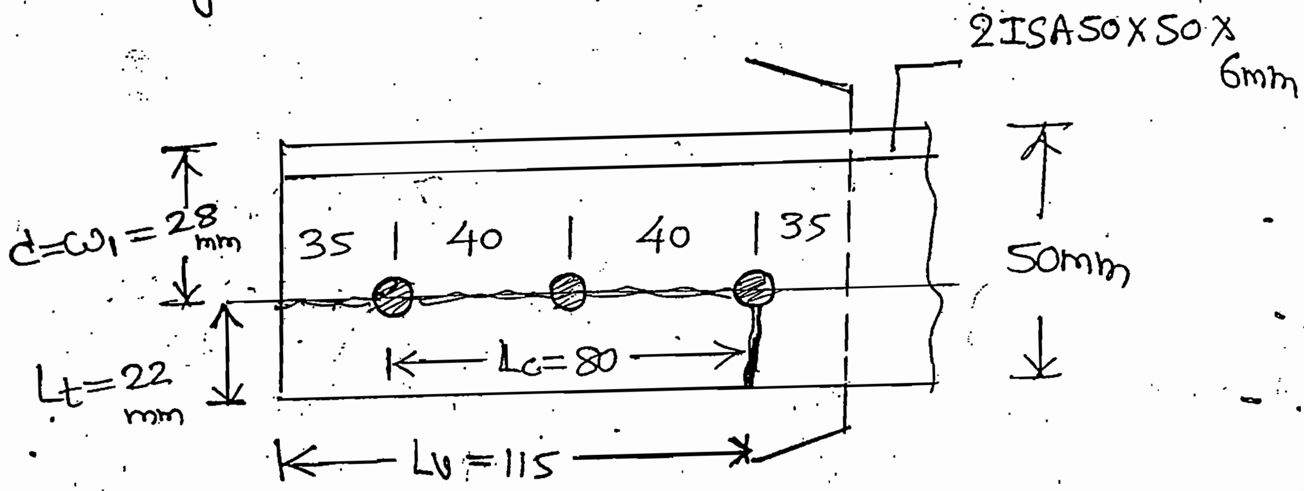
$p = 2.5d = 40mm$

$\boxed{K_b = 0.49}$

$$\therefore V_{dpb} = \frac{1}{1.25}\left[2.5 \times 0.49 \times 16 \times 6 \times 410\right] = 38.57 kN$$

$\therefore \boxed{\text{Bolt Value} = 38.57 kN}$

$$\text{No. of Bolts} = \frac{84.70}{38.57} \approx 3$$



(C) Check for "Rupture":

$\omega = 50mm$, $t = 6mm$, $L_c = 80mm$

$\boxed{b_s = \omega + \omega_1 - t} = 50 + 28 - 6 = 72mm$

$A_{go} = (B - t/2)t = (50 - 6/2)6 = 282$

$A_{nc} = (A - d_o - t/2)t = (50 - 18 - 6/2)6 = 174$

$\therefore \boxed{\beta = 1.05}$

$\therefore T_{dn} = 2[118.0] = 236\,kN > 89.70\,kN$ (safe)

Check for "Block Shear":

$L_v = 115mm$, $t = 6mm$, $L_t = 22mm$, $d_o = 18mm$

$A_{vg} = L_v \times t = 690$

$A_{vn} = (690 - 2.5 \times 18 \times 6) = 420$

$A_{tg} = L_t \times t = 22 \times 6 = 132$

$A_{tn} = (132 - 0.5 \times 18 \times 6) = 78$

$\therefore T_{db} = 2 \left[113.50 \right] = 227.0 \text{ kN}$

$T_{db} = 2 \left[101.6 \right] = 203 \text{ kN}$

$> 89.7 \text{ kN}$ (Safe).

(d) Check for "Compression" :

$\text{Force} = 59.25 \text{ kN (C)}$

Section $\rightarrow$ 2 ISA $50 \times 50 \times 6$mm

$\text{Area} = 1136 \text{mm}^2$, $r_{min} = 15.10$mm

$l_e = 2940$mm

$\therefore$ Slenderness Ratio $= \lambda = \dfrac{l_e}{r_{min}} = \dfrac{2940}{15.10} = \boxed{194.7}$

From Table 9(c) . P-42

$\therefore f_{cd} = 38.10 \text{ N/mm}^2$

$\therefore P_d = (f_{cd})(\text{Area}) = (38.10)(1136)$

$\boxed{P_d = 43.28 \text{ kN}} < 59.25 \text{ kN}$ (Un-Safe)

Hence Revise the section and Re-check only Compression.

Try 2 ISA 55 X 55 X 8mm $\left\{ \begin{array}{l} A_{ca} = 1636 mm^2 \\ r_{min} = 16.4 mm \end{array} \right.$

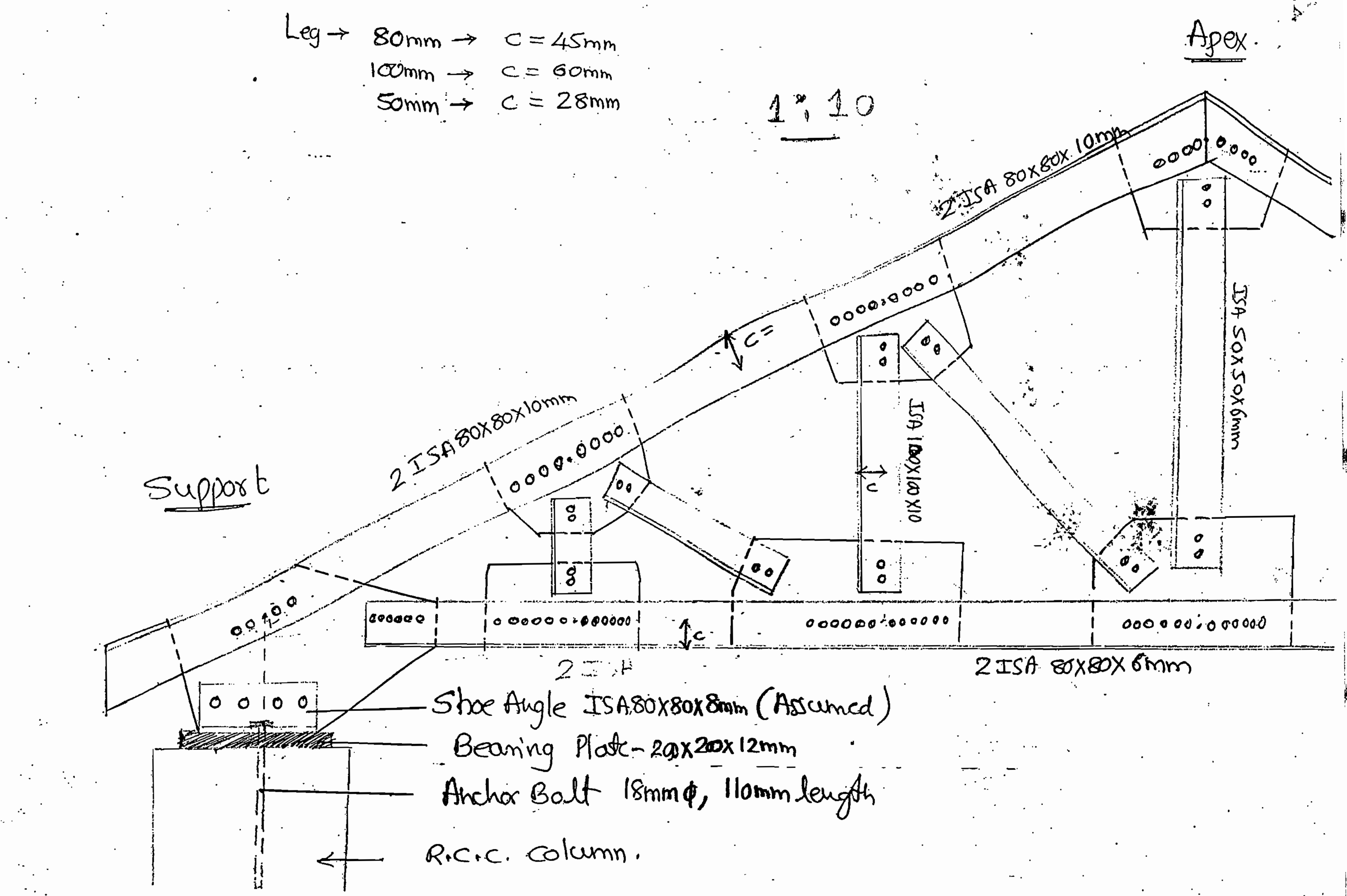
$\therefore \lambda = \frac{le}{r_{min}} = \frac{2940}{16.4} = \cancel{180} \; \underline{\underline{179.2}}$

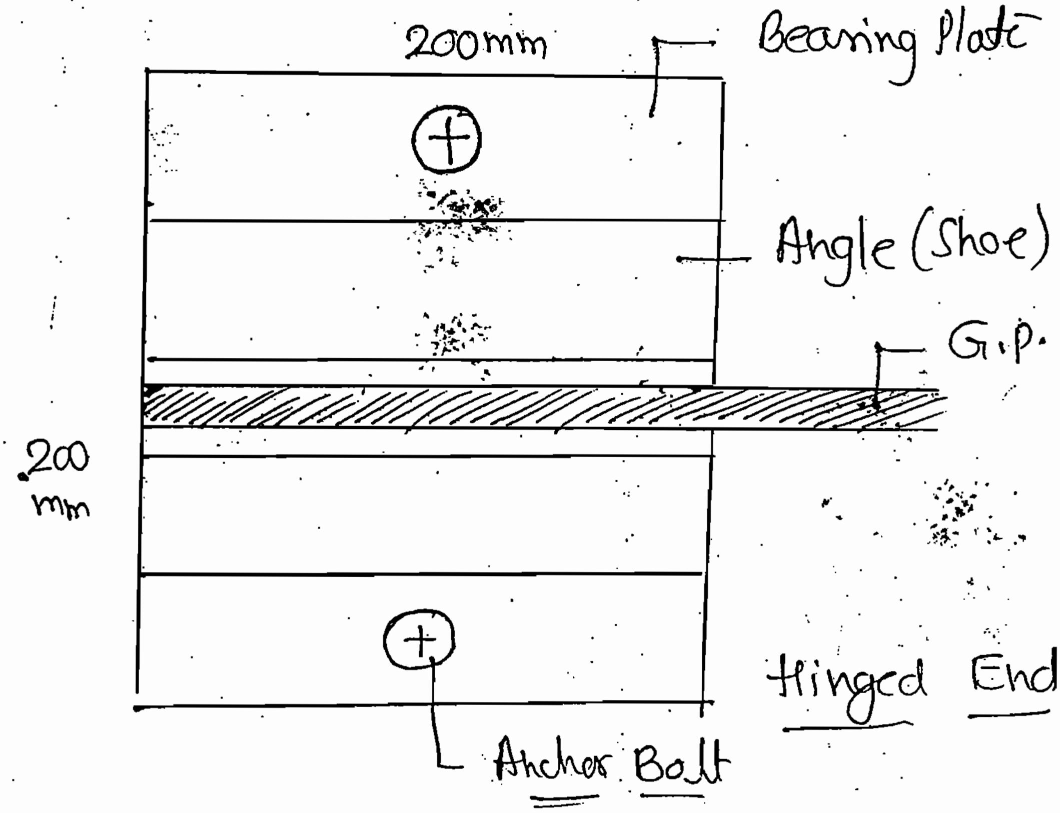
$\therefore f_{cd} = 43.6 \, N/mm^2$

$\therefore P_d = (43.6)(1636) = 71.32 \, kN > 59.25 \, kN$ (safe)

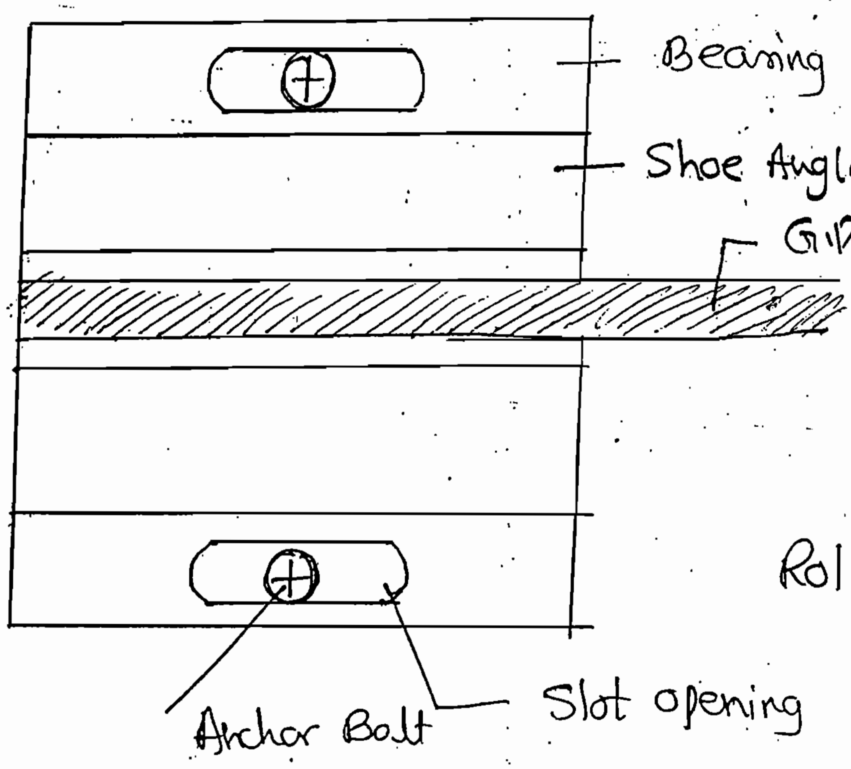
$\boxed{\text{Hence Provide 2 ISA 55X55X8mm with 3-16mm } \phi \text{ For Top chord Member}}$

Repeat the same procedure for ~~as~~ other two members. Every member should check for both Tension & Compression.


Leg → 80mm → c = 45mm
100mm → c = 60mm
50mm → c = 28mm
Apex.
1 : 10
2 ISA 80X80X 10mm
ISA 50X50X6mm
c=
ISA 100X100X10
c
2 ISA 80X80X10mm
Support
2 ISA 80X80X 6mm
Shoe Angle ISA 80X80X8mm (Assumed)
Bearing Plate - 200X200X12mm
Anchor Bolt 18mm ϕ, 110mm length
R.C.C. column.



Hinged End



Roller End

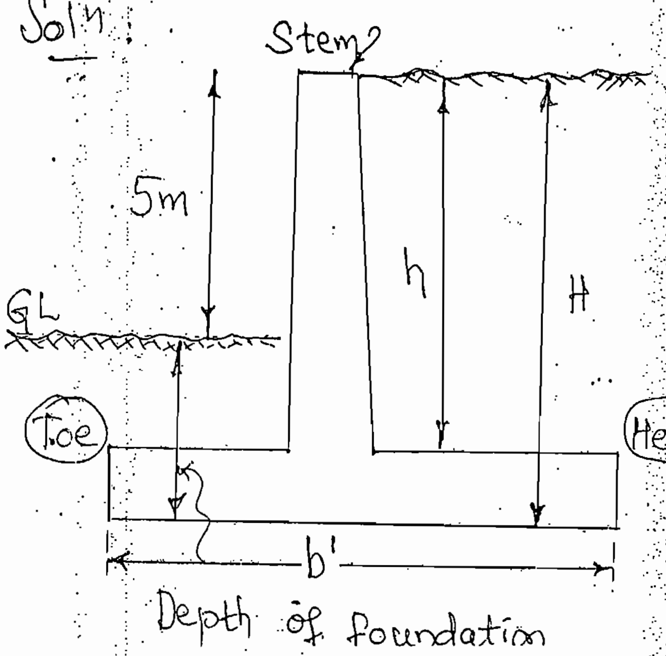
1 : 5

MODULE # 1. RETAINING WALLS. 2/10

CANTILEVER TYPE RETAINING WALL

Problem # 1. Design a Cantilever type retaining wall to retain an earth embankment 5m high above the ground level. The density of earth is 18 kN/m³ and the angle of repose is 30°. The embankment is to be horizontal at the top. The safe bearing capacity of the soil may be taken as 200 kN/m² and the Co-eff of friction between the concrete and the soil is 0.50. Adopt M_{20} grade concrete and Fe 415 grade steel.

Soln:



Density = γ = 18 kN/m³

Angle of repose = ϕ = 30°

SBC = 200 kN/m²

$\boxed{\mu = 0.5}$

M_{20} and Fe 415

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

Step # 1. Depth of Foundation :

$$\text{Depth} = \frac{SBC}{\text{Density}}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2 \quad \text{or Min. } 1\,m$$

$$\text{Depth} = \frac{200}{18}\left(\frac{1-\sin 30^\circ}{1+\sin 30^\circ}\right)^2 = 1.23 \quad \text{Say } 1.25\,m$$

$\therefore$ Provide $\boxed{\text{Depth of Foundation} = 1.25\,m.}$

$\therefore$ Total height of Retaining Wall = $H = (5.00 + 1.25)\,m$

$$\boxed{H = 6.25\,m.}$$

(a) Retaining Wall dimensions:

(i) Top width of Stem = $\boxed{150\ mm}$ → (minimum)

(ii) Base Slab width = $b' = 0.6H = 0.6 * 6.25$

$$\boxed{b' = 3.75\,m}$$

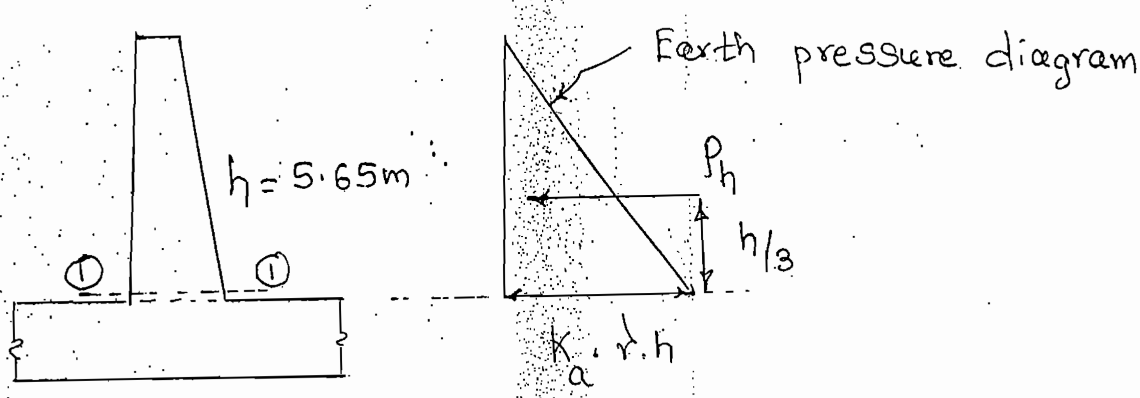
(iii) Toe projection = $\boxed{\frac{b'}{3} \text{ or minimum } 1\,m.}$

$$= \frac{3.75}{3} = \boxed{1.25\,m.}$$

(iv) Base Slab thickness = $\boxed{H}$ = 6.25 = 0.52 m Say 0.(

$\therefore$ height of Stem $= h = 6.25 - 0.60 = \boxed{5.65\,m}$

(V) Stem bottom thickness:



K_a = Co-eff of active Earth pressure $= \dfrac{1-\sin\phi}{1+\sin\phi}$

$$K_a = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = \frac{1-0.5}{1+0.5} = \frac{0.5}{1.5} = \frac{1}{3}$$

Earth pressure for stem = area of earth pr. diagram

$$P_h = \frac{1}{2} * \text{base} * \text{height}$$

$$= \frac{1}{2} * (K_a \cdot \gamma \cdot h) * h$$

$$P_h = \frac{1}{2} * \frac{1}{3} * 18 * 5.65 * 5.65$$

$$P_h = 95.77\ kN\ (\leftarrow)$$

acting @ $\dfrac{h}{3} = \dfrac{5.65}{3} = 1.88\,m$ from base

Moment about ① - ① $= P_h * \frac{h}{3}$.

$\therefore M = 95.77 * \frac{5.65}{3} = 180.37$ KN-m.

$\therefore$ factored Moment for Stem $= M_u = 1.5 M$

$\therefore (M_u)_{Stem} = 1.50 * 180.37 = 270.55$ KN-m

Using $(M_u)_{lim} = 0.138\ bd^2 f_{ck}$, b = 1m = 1000 mm.

$\Rightarrow 270.55 * 10^6 = 0.138 * 1000 * d^2 * 20$

$$d^2 = \frac{270.55 * 10^6}{0.138 * 1000 * 20} = 98.03 * 10^3$$

$d = \sqrt{98.03 * 10^3} = 313$ Say 315 mm.

Assuming an effective Cover of 60 mm,

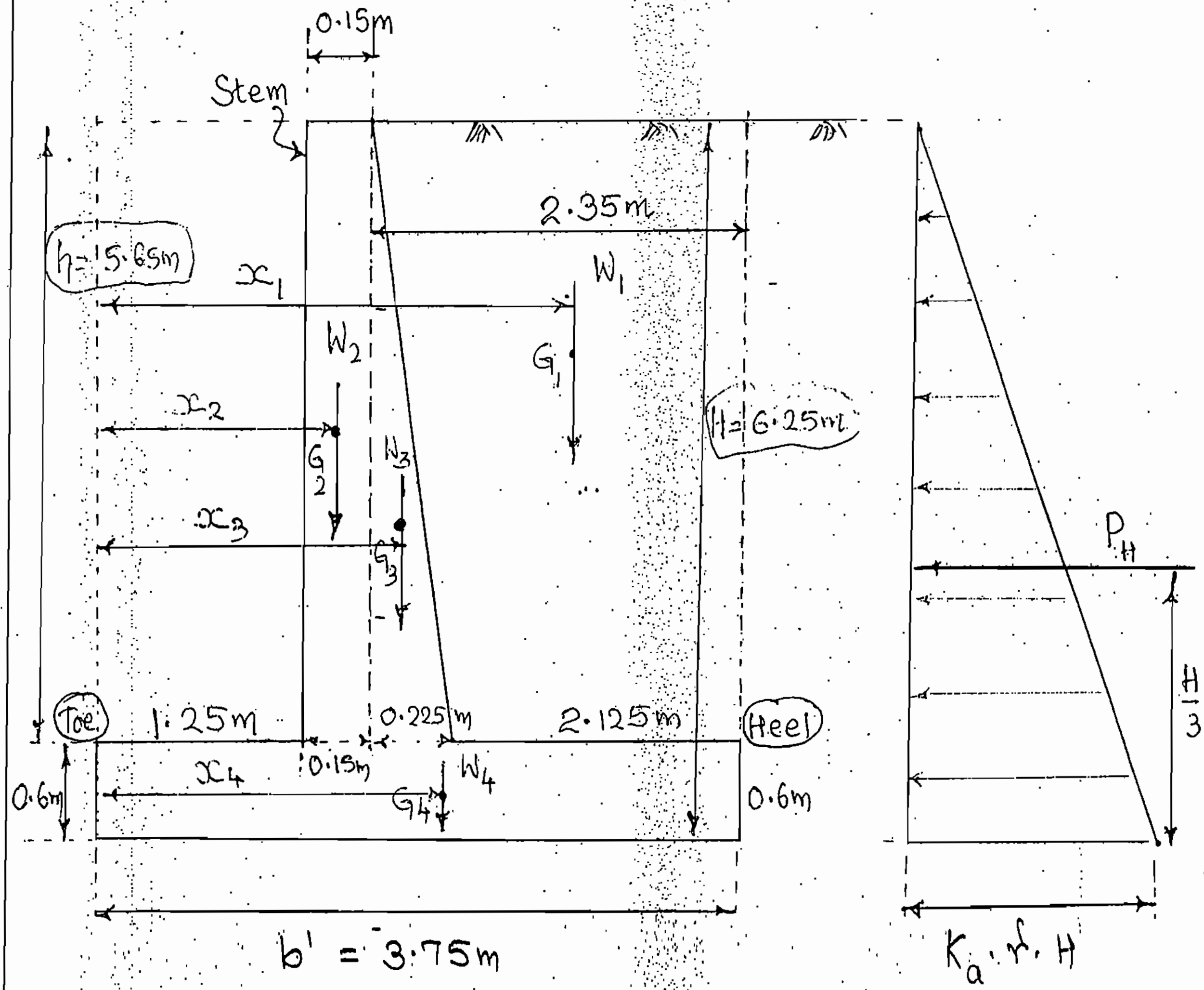
D = d + 60 = 315 + 60 = 375 mm. (Stem width at bottom)

Take $\boxed{D = 375\text{ mm and } d = 315\text{ mm}}$

$\therefore$ Heel projection $= b' - D -$ Toe projection

$\therefore$ Heel projection $= 3.75 - 0.375 - 1.25$

$= 2.125\,m.$



Step # 2: Stability of Retaining Wall:

Consider 1m length of the wall perpendicular to the plane of paper.

Density of earth $= \gamma = 18\,kN/m^3$

Density of Concrete $= 24\,kN/m^3.$

Components	Weight (kN)	Distance upto toe (m)	Moment about Toe (kN-m)
1. Back fill: $W_1 = (2.35 * 5.65 * 1 * 18)$ Density of earth	238.99	$x_1 = \frac{2.35}{2} + 0.15 + 1.25$, $x_1 = 2.575\,m$	615.40
2. Stem: $W_2 = (0.15 * 5.65 * 1) * 25$ Density of concrete; $W_2 * (25-18)$ = Concrete density − Earth density; $W_3 = \left(\frac{1}{2} * 0.225 * 5.65 * 1\right) * (25-18)$	21.19; 4.45	$x_2 = 1.25 + \frac{0.15}{2} = 1.325\,m$; $x_3 = 1.25 + 0.15 + \frac{0.225}{3}$, $x_3 = 1.475\,m$	28.07; 6.56
3. Base Slab: $W_4 = (3.75 * 0.60 * 1) * 25$ Concrete density	56.25	$x_4 = \frac{3.75}{2} = 1.875\,m$	105.47
ΣW	320.88	$\Sigma MR =$	755.50

Total Weight $= \Sigma W = 320.88$ kN

Total Resisting Moment $= MR = 755.50$ kN-m

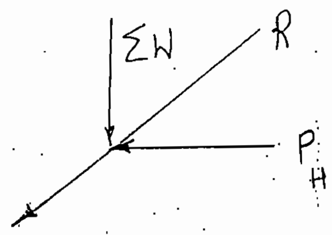
Total Earth pressure $= P_H = \frac{1}{2} k_a \cdot \gamma H \cdot H$

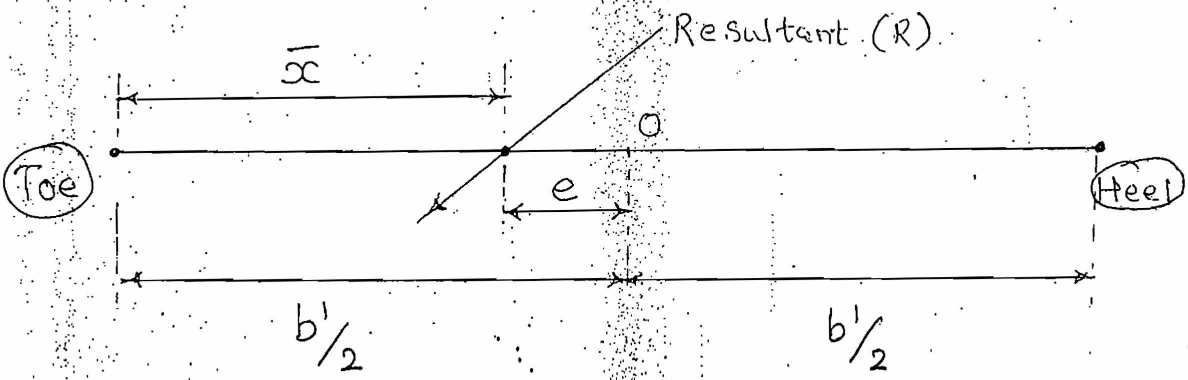
$$P_H = \frac{1}{2} * \frac{1}{3} * 18 * 6.25 * 6.25$$

$$P_H = 117.20 \text{ kN} \ @ \ \frac{H}{3} = \frac{6.25}{3} m$$

Over turning Moment $= M_o = P_H * \frac{H}{3}$

$$= 117.20 * \frac{6.25}{3} = 244.17 \text{ kN-m}.$$





ⓐ <u>check for bottom Soil pressure:</u>

Line of action of Resultant $= \bar{x} = \frac{MR - M_O}{\Sigma W}$

$$\bar{x} = \frac{755.50 - 244.17}{320.88} = 1.59 \text{ m}.$$

$$e = \frac{b'}{2} - \bar{x} = \frac{3.75}{2} - 1.59 = 0.28 \text{m}$$

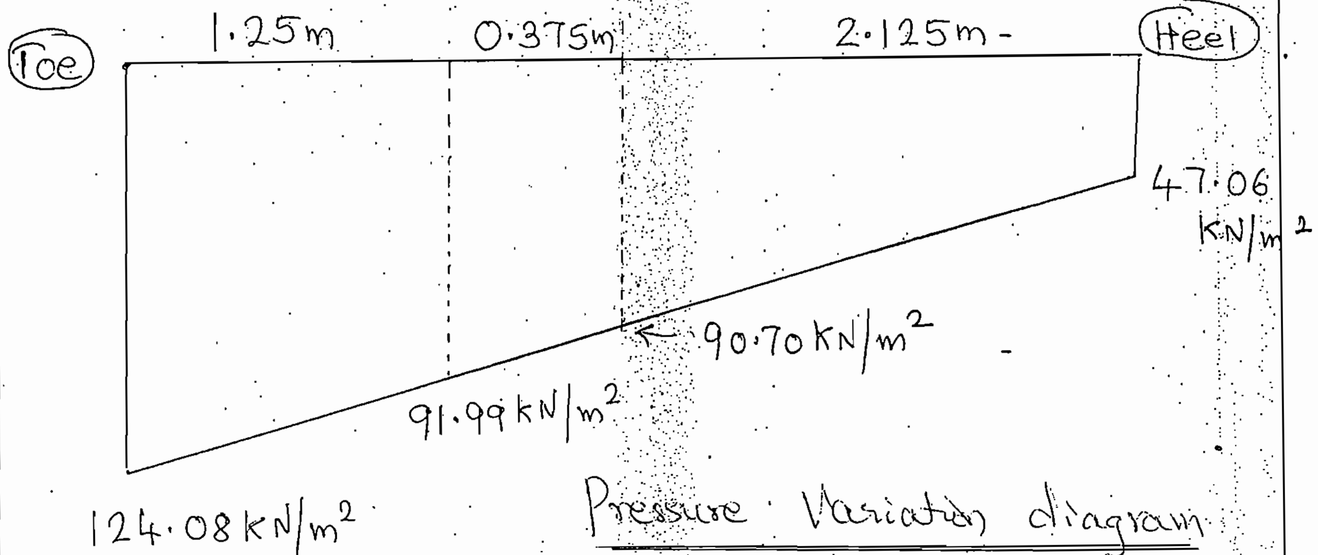
$$\boxed{e = 0.28 \text{m}} < \frac{b'}{6} \quad \text{ok.}$$

Max & min upward soil pressure $\Big\} = \boxed{f = \dfrac{\Sigma W}{b'}\left[1 \pm \dfrac{6e}{b'}\right]}$

$$f = \frac{320.88}{3.75}\left[1 \pm \frac{6 * 0.28}{3.75}\right] = 85.57\left[1 \pm 0.45\right]$$

$$\therefore f_{max} = 85.57\left[1 + 0.45\right] = 124.08 \text{ kN/m}^2$$

$$f_{min} = 85.57\left[1 - 0.45\right] = 47.06 \text{ kN/m}^2$$



Pressure Variation diagram

★ By Interpolation, find Stresses:

Proceed from Toe:

0 m ---- 124.08 kN/m^2

1.25 m ---- ?

$$\text{Stress} = 124.08 + \left(\frac{47.06 - 124.08}{3.75 - 0}\right)(1.25 - 0)$$

$$= 91.99 \ kN/m^2$$

again, 0m --- $124.08 \ kN/m^2$

1.625 m --- ?

3.75m --- $47.06 \ kN/m^2$

$$\text{Stress} = 124.08 + \left(\frac{47.06 - 124.08}{3.75 - 0}\right)(1.625 - 0)$$

$$= 90.70 \ kN/m^2$$

(b) Check for over turning:

$$\left.\begin{array}{l}\text{factor of Safety against}\\ \text{Over turning}\end{array}\right\} = \frac{M_R}{M_O} \geq 1.50.$$

$$= \frac{755.50}{244.17} = 3.09 > 1.50$$

(Safe)

(c) check for sliding:

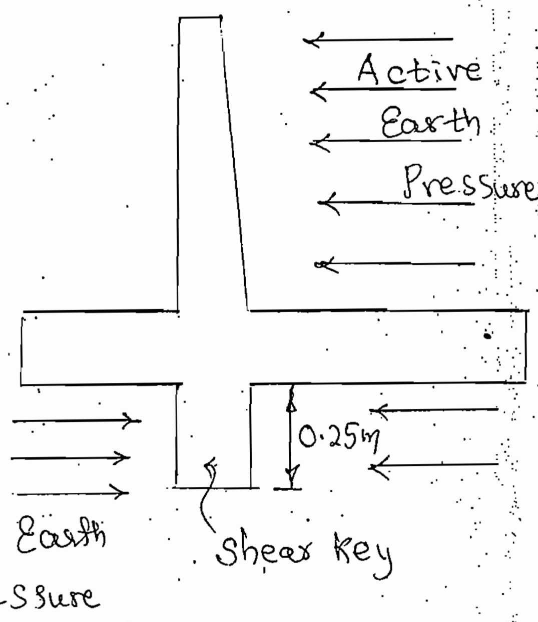
$$\left.\begin{array}{l}\text{factor of Safety against}\\ \text{Sliding}\end{array}\right\} = \frac{\mu \sum W}{P_{..}} \geq 1.50$$

$$= \frac{0.5 * 320.88}{117.20} = 1.37 < 1.50 \text{ (un Safe)}$$

* To prevent Sliding, provide shear key.

(d) Design of Shear Key:

* Assume the depth of Shear key as 0.25 m.



K_p = Co-eff of passive Earth pressure $= \frac{1+\sin\phi}{1-\sin\phi}$

$$K_p = \frac{1+\sin 30^0}{1-\sin 30^0} = \frac{1+0.5}{1-0.5} = 3.$$

Upward Soil pressure at Toe junction = 91.99 kN/m²

Resisting force due to Shear key $\Big\}$ = K_p * Depth of Shear key * Pressure @ Toe Junction

∴ Resisting force due to Shear key = 3 * 0.25 * 91.99

$= 68.99\ kN$

$$\therefore F.O.S. = \frac{\mu \Sigma W + 68.99}{P_H} = \frac{0.5 * 320.88 + 68.99}{117.20}$$

$F.O.S. = 1.96 > 1.50$ (Safe)

★ Provide same reinforcement as that of Stem Steel.

Step # 3 Design of Stem:

$(M_u)_{Stem} = 270.55\ kN\text{-}m$, $D = 375\ mm$

$d = 315\ mm$

Using Eqn' of IS 456 - 2000, Clause G.1.1;

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

$b = 1m = 1000\ mm$,

$$270.55 * 10^6 = 0.87 * 415 * A_{st} * 315 \left[1 - \frac{A_{st} * 415}{1000 * 315 * 20}\right]$$

$$270.55 * 10^6 = 113.73 * 10^3\ A_{st}\left[1 - 65.873 * 10^{-6} A_{st}\right]$$

$$2373.88 = A_{st} - 65.87 * 10^{-6} (A_{st})^2$$

$$\Rightarrow 65.87 * 10^{-6} (A_{st})^2 - A_{st} + 2373.88 = 0.$$

$$\Rightarrow (A_{st})^2 - 15180.72\, A_{st} + 36.04 * 10^6 = 0.$$

$$\therefore A_{st} = \frac{+15180.72 \pm \sqrt{(15180.72)^2 - 4(36.04 * 10^6)}}{2}$$

$$A_{st} = \frac{15180.72 - 9289.71}{2} = 2945.50 \text{ mm}^2$$

* choose minimum of 16 mm ϕ bars.

$$\text{Spacing of } 16\text{mm } \phi \text{ bars} = \frac{\text{Area of 1 bar}}{A_{st}} * 1000 \quad \{\text{In meter}$$

$$= \frac{\frac{\pi}{4} * (16)^2}{2945.50} * 1000$$

$$= 67.56 \text{ mm c/c. Say } 70 \text{ mm c/c}$$

$\boxed{\text{Spacing of } 16\text{mm } \phi \text{ bars @ } 70\text{mm c/c}}$ < 3d or 300mm **

(i) <u>Distribution Steel: for Stem:</u>

$$\text{Area} = \frac{0.12}{100} * \overset{b}{1000} * \overset{D}{375} = 450 \text{ mm}^2.$$

* choose minimum of 10mm ϕ distribution bars.

$$\text{Spacing of 10mm } \phi \text{ bars} = \frac{\text{Area of 1 bar}}{A_{st}} * 1000 \quad \{1m\}$$

$$= \frac{\left(\frac{\pi}{4}\right) * (10)^2}{450} * 1000 = 174.5 \text{ mm c/c}$$

Say 180 mm c/c

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 180 mm c/c}}$ < 5d or 450 mm)

(ii) Check for shear in stem:

Shear force V = Earth pressure for stem = 95.77 KN

$\therefore V = P_h = 95.77$ KN

Factored Shear force $= V_u = 1.50 V$

$V_u = 1.50 * 95.77 = 143.66$ KN

$b = 1000$ mm, $d = 315$ mm, $A_{st} = 2945.50 \text{ mm}^2$

Nominal Shear Stress $= \tau_v = \frac{V_u}{bd} = \frac{143.66 * 10^3}{1000 * 315}$

$$\tau_v = 0.46 \ N/mm^2.$$

$$\frac{100 A_{st}}{bd} = \frac{100 * 2945.50}{1000 * 315} = \boxed{0.935} \ \& \ M_{20}$$

From IS 456 - 2000, Table 19, Pg 73,

$\frac{100 A_{st}}{bd}$	M_{20} (τ_c)
0.75	0.56
0.935	?
1.00	0.62

$$\tau_c = 0.56 + \left(\frac{0.62 - 0.56}{1.00 - 0.75}\right)(0.935 - 0.75)$$

$$\tau_c = 0.604 \ N/mm^2 \ \text{(Permissible)}$$

Comparing τ_v and τ_c, $\boxed{\tau_c > \tau_v}$ (Safe)

(iii) Curtailment of Stem Reinforcement:

*** Earth pressure is maximum at the bottom of the stem and therefore the moment developed is also maximum.

As we go towards the top, the earth pressure decreases and hence the moment developed also decreases.

** From economy point of view, we can decrease the quantity of steel by 50 %.

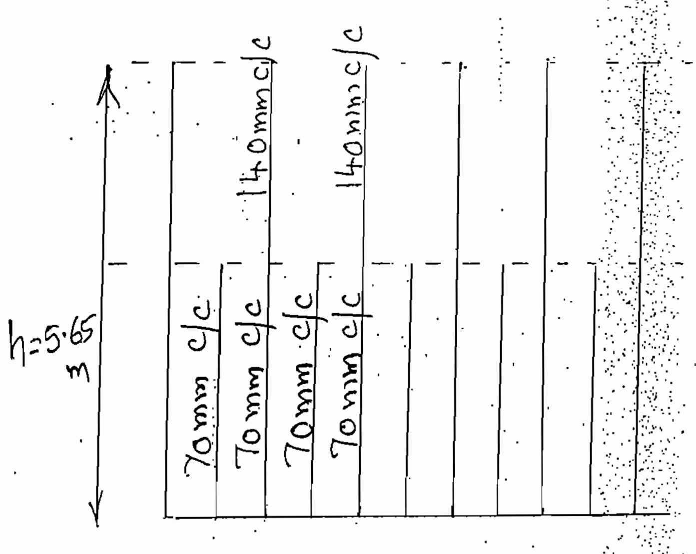
$$\therefore \quad \text{Area} = \frac{2945.50}{2} = 1472.75 \text{ mm}^2$$

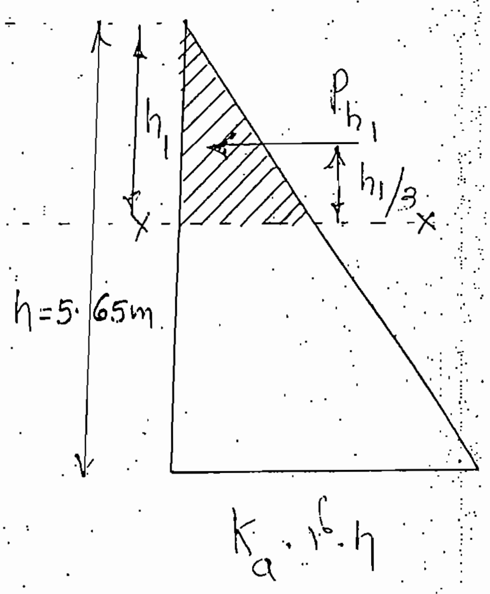
$$M_u = \frac{270.55}{2} = 135.275 \text{ kN-m.}$$

$$\text{Working moment } (M) = \frac{M_u}{1.50} = \frac{135.275}{1.50} = 90.18 \text{ kN-m}$$

See figure in Next Page (back)!

at Section x – x,





Moment at X − X = $M_{XX} = P_{h_1} * \frac{h_1}{3}$

$$M_{XX} = \frac{1}{2} * K_a * \gamma * h_1 * h_1 * \frac{h_1}{3}$$

$$M_{XX} = \frac{1}{2} K_a \frac{\gamma h_1^3}{3} = \frac{K_a \cdot \gamma h_1^3}{6}$$

Equating M_{XX} to working moment M,

$$M_{XX} = \frac{K_a \cdot \gamma h_1^3}{6} = 90.18$$

$$\Rightarrow \frac{1}{3} * \frac{18}{6} * h_1^3 = 90.18$$

$$\text{or} \quad h_1 = (90.18)^{1/3} \doteq 4.485\text{m}$$

$\therefore$ from bottom $= 5.65 - 4.48 = 1.17\ m.$

** But as per IS Code, extend the bars upto 'L_d':

$L_d = 47\phi$

$= 1.17\ m + L_d$

$\phi = 16\ mm$

$= 1.17 * 1000\ mm + 16 * 47$

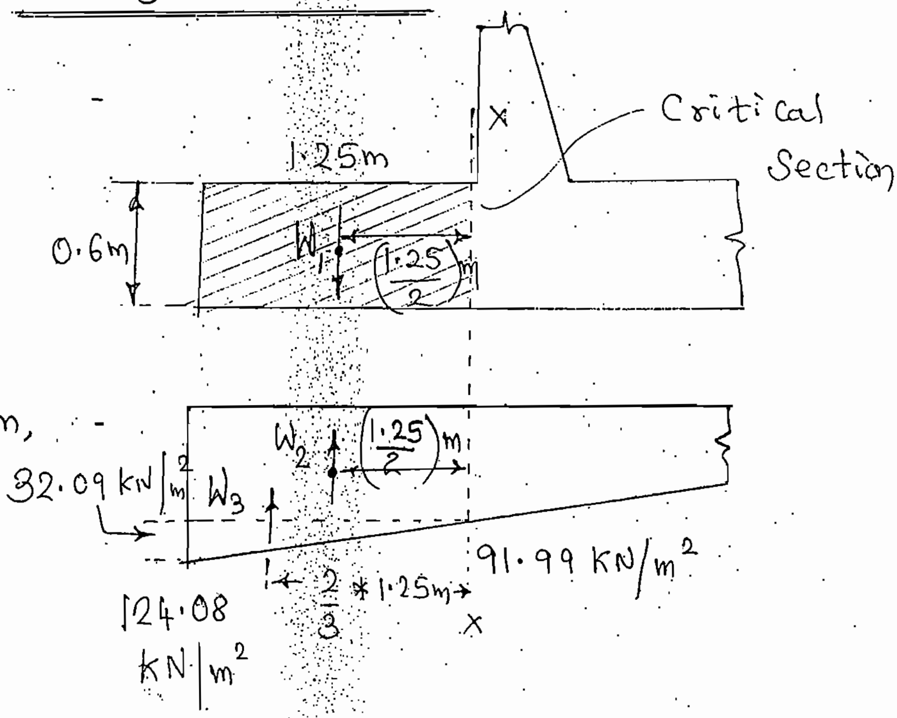
$= 1170 + 752 = 1922\ mm \simeq 1925\ mm \simeq 1.95\ m.$

Step # 4: Design of Toe:

D = 600 mm.

Assuming an effective Cover of 60 mm,

d = 540 mm.



$\therefore$ D = 600mm, d = 540 mm

Concrete density

$M_1 = W_1 x_1 = (1.25 * 0.6 * 1) * 25 * \frac{1.25}{2} = 11.72\ kN\text{-}m\ (\text{C})$

$M_2 = W_2\, x_2 = (1.25 * 1 * 91.99) * \frac{1.25}{2} = 71.87$ kN-m (↺)

$M_3 = W_3\, x_3 = \left(\frac{1}{2} * 32.09 * 1.25\right) * \frac{2}{3} * 1.25 = 16.71$ kN-m (↺)

Nett Moment $\underline{M} = M_2 + M_3 - M_1$

$\underline{M} = 71.87 + 16.71 - 11.72 = 76.86$ kN.m (↺)

factored Moment $M_u = 1.5 M = 1.5 * 76.86$

$$\boxed{M_u = 115.28 \text{ kN-m.}}$$

Shear force $(V) = W_2 + W_3 - W_1$

$\therefore V = (1.25 * 1 * 91.99) + \left(\frac{1}{2} * 32.09 * 1.25\right)$

$\ominus \left(\frac{1}{2} * 0.6 * 1 * 25\right)$

$V = 127.54$ kN.

factored Shear force $= V_u = 1.50\,V$

$$\therefore \boxed{V_u = 1.50 * 127.54 = 191.32 \text{kN.}}$$

(i) <u>Check for depth:</u>

$b = 1m$

$$(115.28 * 10^6) = 0.138 * (1000)(d^2)(20)$$

$$\therefore d^2 = \frac{115.28 * 10^6}{0.138 * 1000 * 20} = 41768.12$$

$$d = \sqrt{41768.12} = 204.37 \text{ mm}$$

$$d = 204.37 \text{ mm} < 540 \text{ mm}, \quad \text{Safe.}$$

(ii) Area of Steel (A_{st}):

Using eqn of IS 456 - 2000, Clause G.1.1

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

$b = 1m = 1000$ mm

$$115.28 * 10^6 = \left[0.87 * 415 * A_{st} * 540\right]\left[1 - \frac{A_{st} \cdot 415}{1000 * 540 * 20}\right]$$

$$115.28 * 10^6 = A_{st} \cdot 194.97 * 10^3 \left[1 - 38.43 * 10^{-6} A_{st}\right]$$

$$591.36 = A_{st} - 38.43 * 10^{-6} (A_{st})^2$$

$$\Rightarrow 38.43 * 10^{-6} (A_{st})^2 - A_{st} + 591.36 = 0$$

Solving, $A_{st} = 605.45 \text{ mm}^2$.

Provide 12mm ϕ bars.

$$\text{Spacing of 12mm } \phi \text{ bars} = \frac{\text{Area of 1bar}}{A_{st}} * 1000 \text{ mm}$$

$$= \frac{\left(\frac{\pi}{4}\right) * 12^2}{605.45} * 1000$$

$$= 186.80 \quad \text{Say } 180 \text{ mm c/c}$$

$\boxed{\text{Spacing of 12mm } \phi \text{ bars @ 180mm c/c}}$ < 3d or 300 mm

(iii) Distribution Steel for Toe:

$$\text{Area} = \frac{0.12}{100} * (\underset{b}{1000}) * (\underset{D}{600}) = 720 \text{mm}^2$$

Provide 10mm ϕ bars.

$$\text{Spacing of 10mm } \phi \text{ bars} = \frac{\text{Area of 1bar}}{A_{st}} * 1000 \text{mm}$$

$$= \frac{\left(\frac{\pi}{4}\right) * 10^2}{720} * 1000 = 109.08 \text{ mm}$$

Say 120mm c/c

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 120mm c/c}}$ < 5d or 450mm

(iv) <u>check for shear in Toe:</u> b = 1000 mm

Nominal Shear stress = $\tau_v = \frac{V_u}{bd}$

$$\tau_v = \frac{191.32 * 10^3 N}{1000 * 540} = 0.354 \ N/mm^2$$

$$\frac{100 A_{st}}{bd} = \frac{100 * 605.45}{1000 * 540} = \boxed{0.112} \text{ and } M_{20}$$

As per Table # 19, Pg 73 of IS 456 - 2000,

Permissible $\tau_c \leq 0.15 = 0.28 \ N/mm^2$

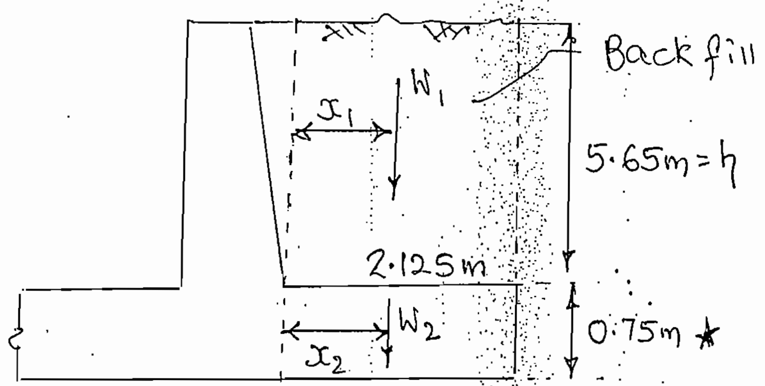
Hence increase the thickness of the Toe Slab using $(\tau_v) = \boxed{\frac{V_u}{bd} = \tau_c}$

$$\frac{191.32 * \cancel{10^3}}{\cancel{1000} * d} = 0.28$$

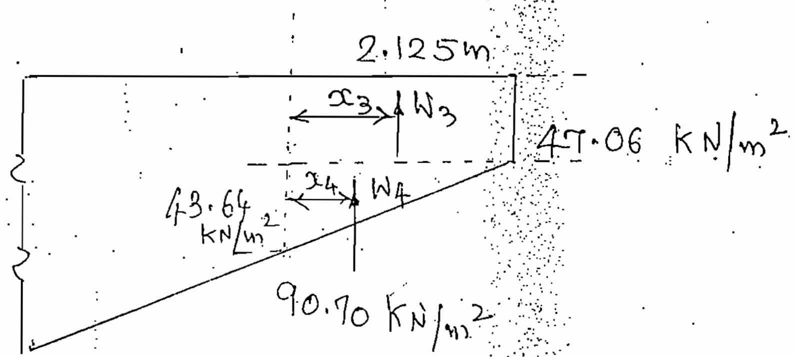
$$\Rightarrow d = \frac{191.32}{0.28} = 683.28 \text{ Say } 690mm.$$

∴ Provide $\boxed{d = 690\,mm \;\&\; D = 750\,mm}$ for Toe slab.

Step # 5. Design of Heel Slab:



D = 750 mm
d = 690 mm



Pressure Variation diagram

density of earth

$$M_1 = W_1 x_1 = (2.125 * 5.65 * 1) * 18 * \frac{2.125}{2}$$

$$= 229.62 \text{ KN-m} \;(\circlearrowleft)$$

$$M_2 = W_2 x_2 = (2.125 * 0.75 * 1) * 25 * \frac{2.125}{2}$$

$$= 42.33 \text{ KN-m} \;(\circlearrowleft)$$

$$M_4 = W_4 x_4 = \left(\frac{1}{2} * 43.64 * 2.125 * 1\right) * \frac{1}{3} * 2.125$$

$$= 32.84 \text{ kN-m} \quad (\circlearrowleft)$$

$$\text{Nett Moment} = M = (M_1 + M_2) - M_3 - M_4$$

$$M = 229.62 + 42.33 - 106.25 - 32.84$$

$$M = 132.86 \text{ kN-m}$$

$\therefore$ factored moment $M_u = 1.5M = 1.5 * 132.86$

$$\boxed{M_u = 199.30 \text{ kN-m}}$$

Shear force $V = W_1 + W_2 - W_3 - W_4$

$$V = (2.125 * 5.65 * 1 * 18) + (2.125 * 0.75 * 1 * 25) - (2.125 * 1 * 47.06) - \left(\frac{1}{2} * 43.64 * 2.125 * 1\right)$$

$$\therefore V = 216.11 + 39.84 - 100.00 - 46.37$$

$$V = 109.58 \text{ kN.}$$

Factored Shear force = $V_u = 1.50\,V$

$$\boxed{V_u = 1.5 * 109.58 = 164.37 \text{ kN}}$$

(i) <u>Check for depth:</u>

Using $(M_u)_{lim} = 0.138\, b d^2 f_{ck}$, $b = 1m = 1000\,mm$

$$199.30 * 10^6 = 0.138 * 1000 * d^2 * 20$$

$$\therefore d^2 = \frac{199.30 * 10^6}{0.138 * 1000 * 20} = 72210.14$$

$$d = \sqrt{72210.14} = 268.72 \text{ mm}$$

$$d = 268.72\,mm < 690\,mm \quad \text{Safe.}$$

(ii) <u>Area of Steel (A_{st}):</u>

Using Equ' of IS 456 - 2000, Clause G.1.1

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

$$199.30 * 10^6 = \left[0.87 * 415 * A_{st} * 690\right]\left[1 - \frac{A_{st} * 415}{\quad}\right]$$

$$199.30*10^6 = 249.12*10^3 A_{st}\left[1 - 30*10^{-6} A_{st}\right]$$

$$\therefore 800.00 = A_{st} - 30*10^{-6}(A_{st})^2$$

$$\therefore 30*10^{-6}(A_{st})^2 - A_{st} + 800.00 = 0.$$

Solving, $A_{st} = 820.18 \text{ mm}^2$

Provide 12 mm ϕ bars.

$$\text{Spacing of 12 mm } \phi \text{ bars} = \frac{\text{Area of 1 bar}}{A_{st}} * 1000$$

$$= \frac{\left(\frac{\pi}{4}\right)*12^2}{820.18} * 1000$$

$$= 138 \text{ mm c/c. Say } 130 \text{ mm c/c}$$

$\boxed{\text{Spacing of 12 mm } \phi \text{ bars @ 130 mm c/c}}$ < 3d or 300 mm

(iii) Distribution Steel for Heel:

$$\text{Area} = \frac{0.12}{100} * \underbrace{1000}_{b} * \underbrace{750}_{D} = 900 \text{ mm}^2.$$

Provide 12 mm ϕ bars.

Spacing of 12mm ϕ bars $= \frac{\text{Area of 1bar}}{A_{st}} * 1000$

$$= \frac{\left(\frac{\pi}{4}\right) * 12^2}{900} * 1000 = 125.66 \text{ Say } 120\text{mm c/c}$$

$\boxed{\text{Spacing of 12mm } \phi \text{ bars @ 120mm c/c}} < 5d \text{ or } 450 \text{ mm}$

(iv) <u>check for Shear in Heel:</u> $b = 1000$ mm.

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

$$\tau_v = \frac{164.37 * 10^3}{1000 * 690} = 0.24 \text{ N/mm}^2$$

$$\frac{100 A_{st}}{bd} = \frac{100 * 820.18}{1000 * 690} = \boxed{0.12} \text{ \& } M_{20}$$

As per Table # 19, Pg 73 of IS 456-2000,

$\frac{100 A_{st}}{bd}$	τ_c for M_{20}
≤ 0.15	0.28

Comparing τ_c and τ_v $\tau_c > \tau_v$, Safe

$$\text{Nominal} = \tau_v = \frac{V_u}{bd} = \frac{164.37 \times 10^3}{1000 \times 6.0} = 0.24\ N/mm^2$$

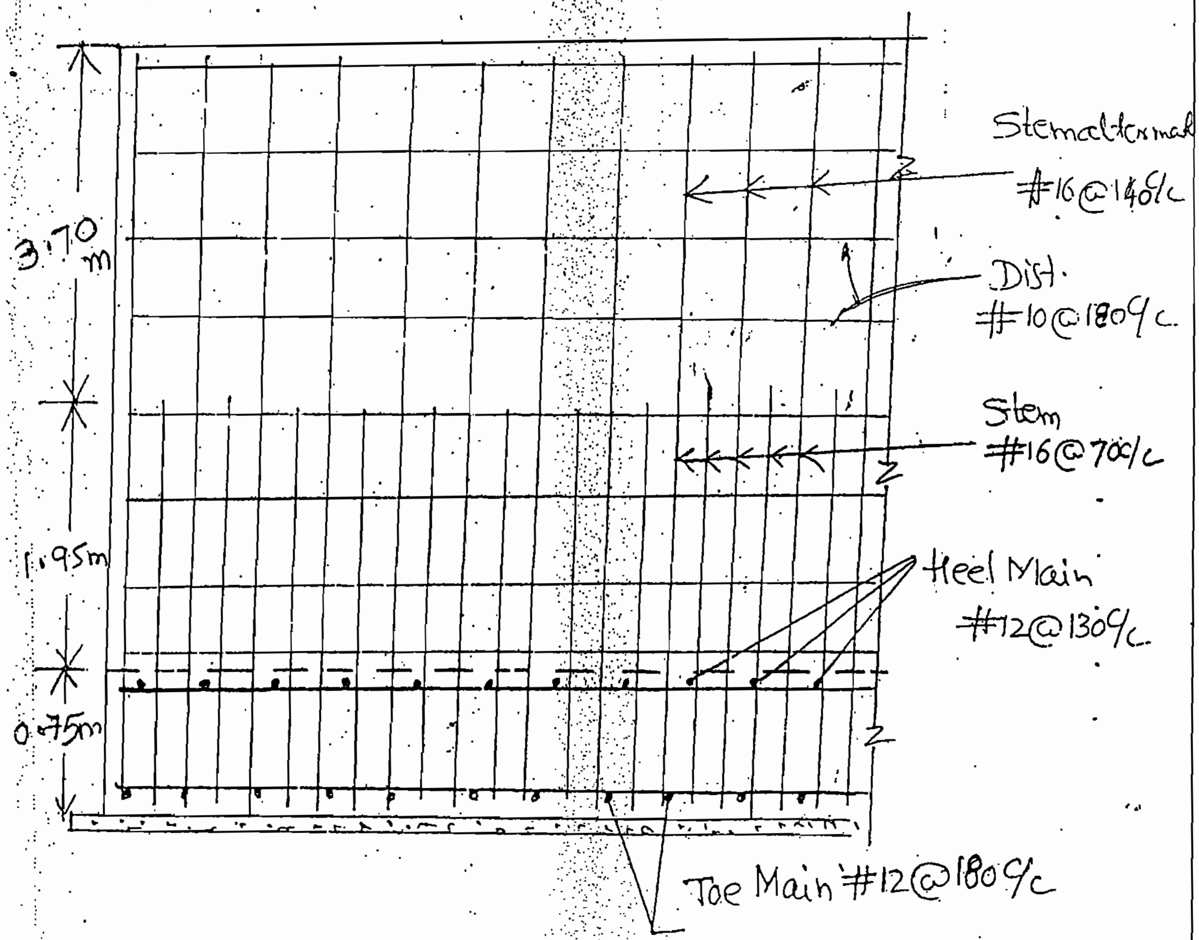
$$\frac{100 A_{st}}{bd} = \frac{100 \times 873.4}{1000 \times 650} = \boxed{0.12}\ \&\ M_{20}$$

Permissible $\tau_c = 0.28\ N/mm^2$ (Table 19)

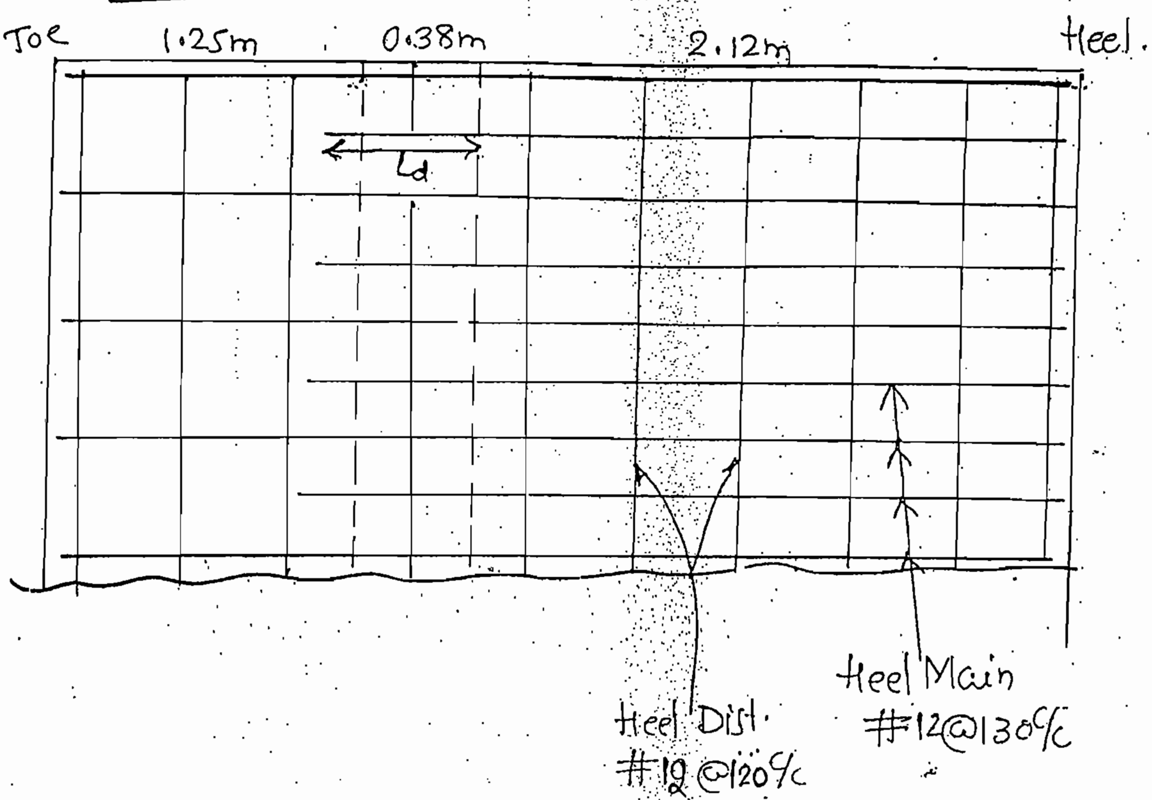
Comparing, $\tau_c > \tau_v$ (Safe)

— x —

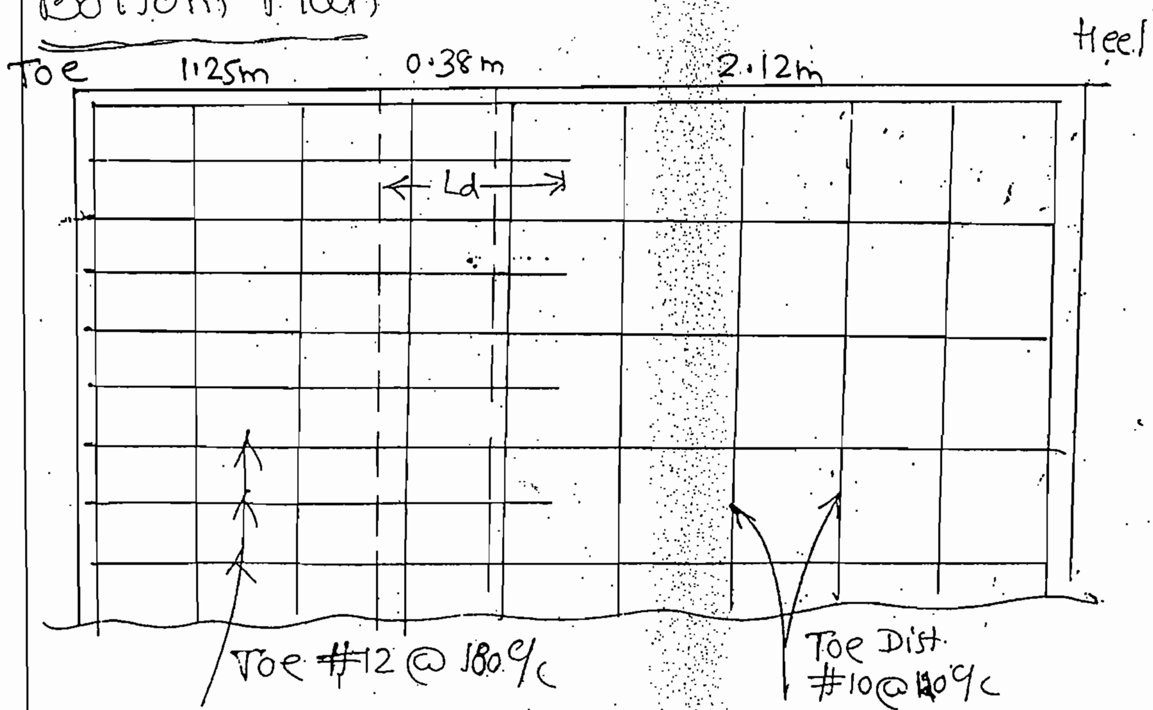
L/S Elevation : (Longitudinal Sectional Elevation).



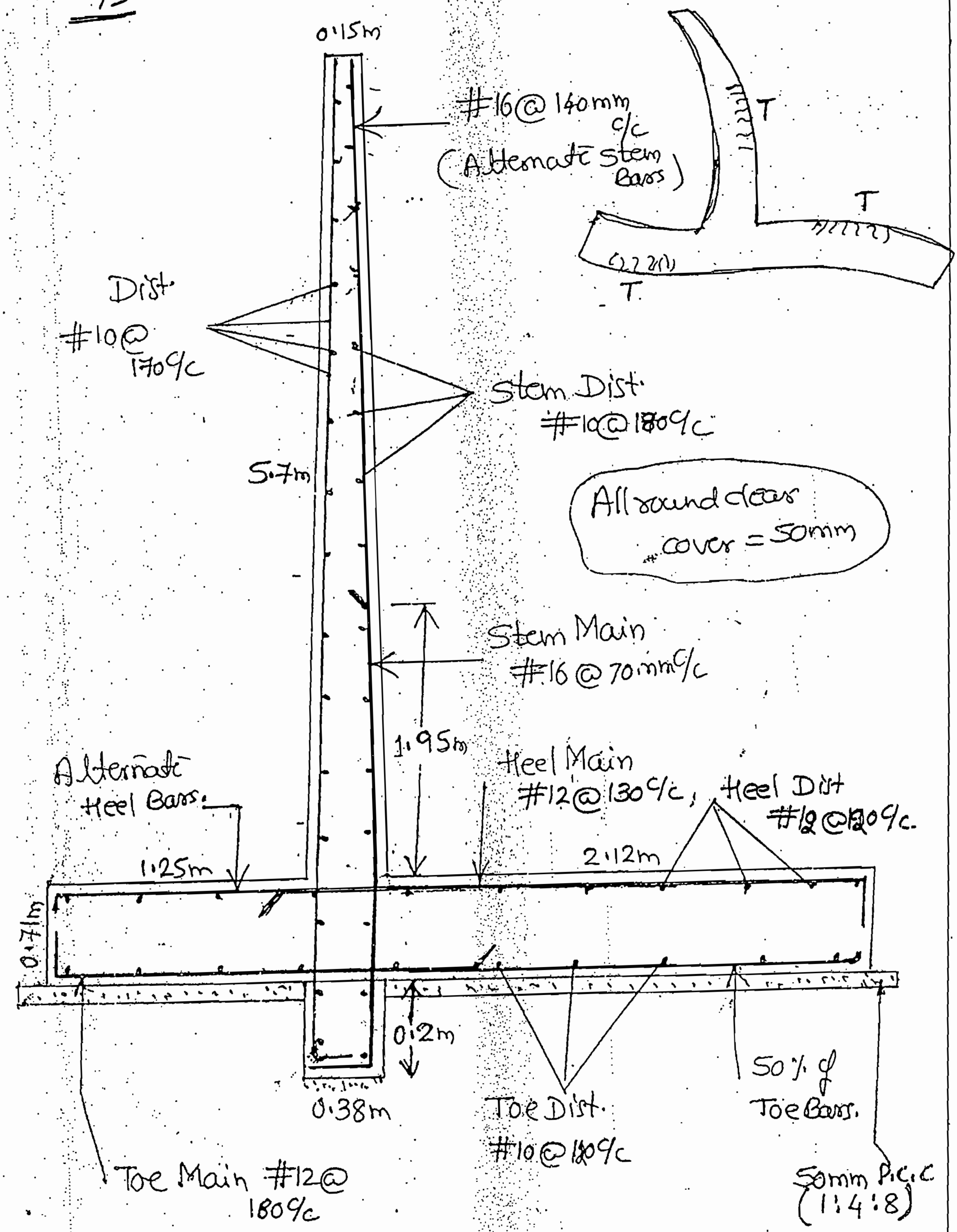
Base Slab Top Plan :



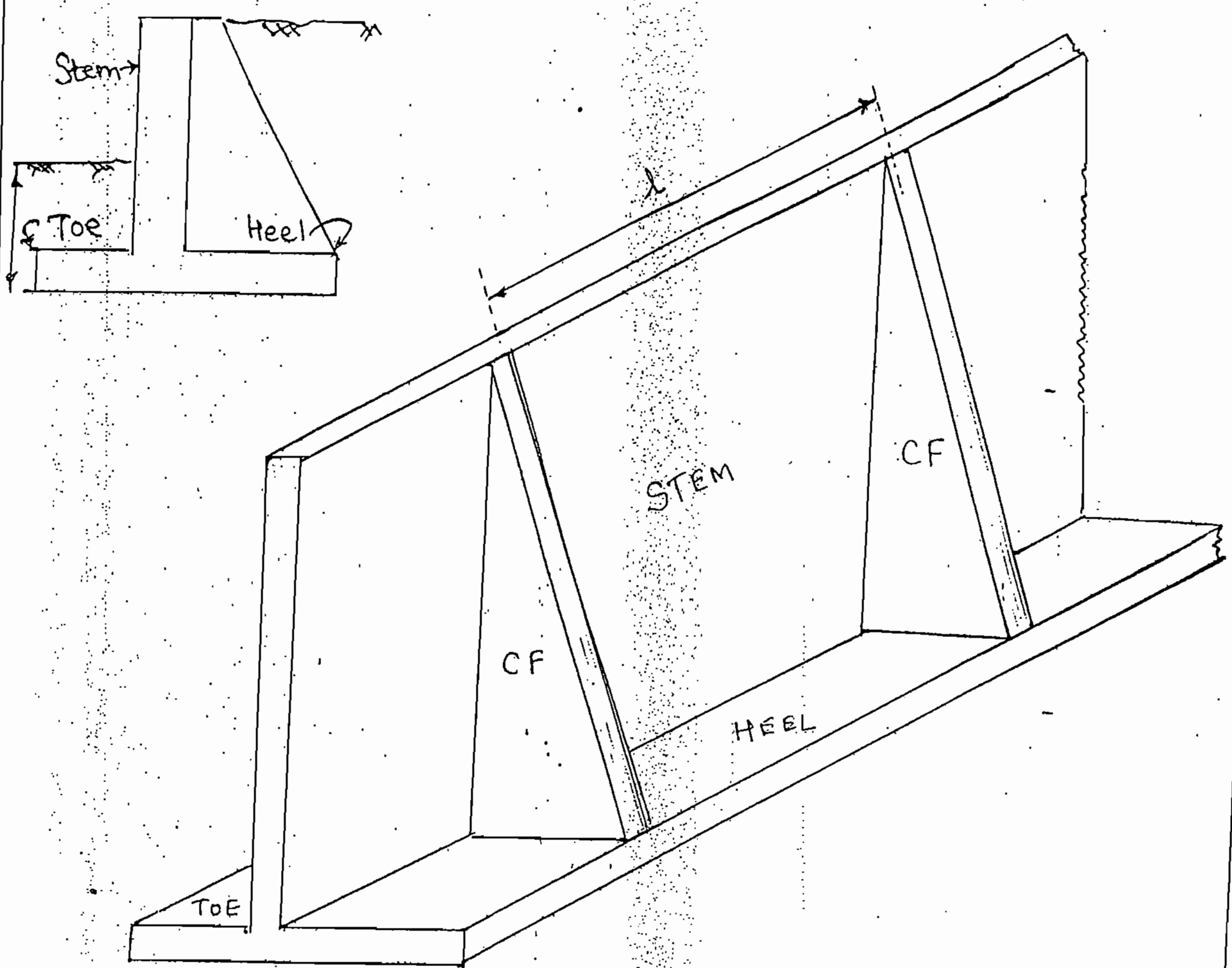
Bottom Plan



C/S: $\boxed{1:20}$


0.15m
#16@140mm c/c
(Alternate Stem Bars)
T
T
T
Dist
#10@ 170c/c
Stem Dist
#10@180c/c
5.7m
All round clear cover = 50mm
Stem Main
#16 @ 70mm c/c
1.95m
Alternate Heel Bars.
Heel Main
#12@130c/c,
Heel Dist
#12@120c/c.
1.25m
2.12m
0.71m
0.2m
0.38m
Toe Dist.
#10@120c/c
50% of Toe Bars.
Toe Main #12@ 180c/c
50mm P.C.C (1:4:8)

MODULE # 1. COUNTER FORT RETAINING WALL



Note: (a) The "Stem" and "Heel" Slabs are Connected together by using Counter forts at regular intervals. Hence, the Stem and the Heel slab are designed like a Continuous slab. (one way type)

$$\therefore \boxed{\text{Max BM} = \frac{wl^2}{10} \quad \text{and} \quad \text{Max. S.F} = 0.6\,wl}$$

(b) Spacing of Counter Forts = $\left\{ l = 3.5\left(\frac{H}{\gamma}\right)^{1/4} \right\}$

where; H = Total height of retaining wall

γ = Density of the Soil.

★ The above relation to be used if the spacing of Counter forts is not given. ★★

Problem #1. Design a Counter fort type retaining wall given the following data:

(i) Height of Wall above GL = 6.00 m.

(ii) SBC of the Soil = 160 KN/m²

(iii) Angle of internal friction = 33°

(iv) Density of the Soil = 16 KN/m³

(v) Spacing of Counterforts = 3m c/c

(vi) Use of M_{25} Concrete and Fe 415 steel.

Soln : Step # 1: Dimensions of Retaining Wall:

(i) Depth of Foundation = $\frac{SBC}{Density}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2$

$\therefore$ Depth = $\frac{160}{16}\left[\frac{1-\sin 33^\circ}{1+\sin 33^\circ}\right]^2$ = 0.87m. or 1m ★ min

∴ Provide Depth of foundation as 1 m.

(ii) h_1 = height of Wall above GL = 6 m.

∴ H = Overall height of Wall = $6 + \underbrace{1}_{\text{Fd}^n\text{ depth.}} = 7$m

(iii) l = Spacing of Counter forts = 3m c/c.

(iv) Base Slab width = $b' = 0.7 H$

$$b' = 0.7 * 7 = \boxed{4.90\text{m.}}$$

(V) Toe projection = $\frac{b'}{4}$ or Min 1 m.

$$= \frac{4.90}{4} = 1.22 \text{ m or } 1 \text{ m.}$$

Provide $\boxed{1.25 \text{ m.}}$

(Vi) Base Slab thickness = $(20 * l * H)$ mm ★★

$$= 20 * 3m * 7m = 420 \text{ mm} \quad \text{Say } 500 \text{ mm.}$$

Provide $\boxed{500 \text{ mm.}}$

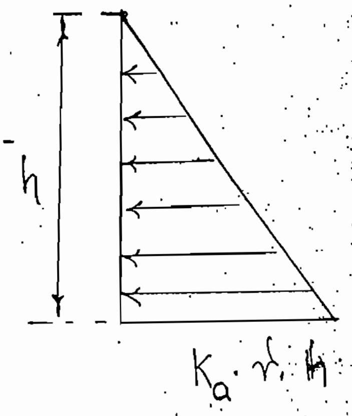
(Vii) Height of stem = (h) = $H - \underbrace{0.50}_{\text{Base Slab thickness}} = 7 - 0.50$

$$= 6.50\text{m}$$

(Viii) Stem thickness: The Stem and the heel

Slabs are Connected by Counter forts at regular intervals. Hence, the Stem and the heel Slabs are designed like a one way Continuous slab.

K_a = Co-eff of active earth pressure



$$K_a = \left(\frac{1-\sin\phi}{1+\sin\phi}\right) = \frac{1-\sin 33^\circ}{1+\sin 33^\circ}$$

$K_a = 0.2948$, $\gamma = 16\ kN/m^3$, $h = 6.50m$.

The earth pressure at bottom of the stem

$= \boxed{\omega = K_a \cdot \gamma \cdot h}$

$\omega = 0.2948 * 16 * 6.50$

$\omega = 30.66\ kN/m^2$

$\omega = 30.66\ kN/m$, per meter depth $\perp^{er}$ to plane of paper.

$\therefore \boxed{\omega = 30.66\ kN/m}$

Max BM for Continuous slab $= M = \frac{\omega l^2}{10}$

$\therefore M = \frac{30.66 * 3^2}{10} = 27.59$ kN-m.

Factored Moment $M_u = 1.50\, M = 1.50 * 27.59$

$(M_u)_{Stem} = \boxed{M_u = 41.40 \text{ kN-m.}}$

Max Shear force $V = 0.6\, \omega l = 0.6 * 30.66 * 3$

$V = 55.19$ kN.

Factored Shear force $= V_u = 1.50 V = 1.50 * 55.19$

$\boxed{V_u = 82.78 \text{ kN}}$ for stem

Using $(M_u)_{lim} = 0.138\, b d^2 f_{ck}$, $b = 1000mm = 1m$

$41.40 * 10^6 = 0.138 * 1000 * d^2 * 25$

$\therefore d^2 = \frac{41.40 * 10^6}{0.138 * 1000 * 25} = 12000$

$d = \sqrt{12000} = 109.55$ mm, Provide 60 mm effective cover

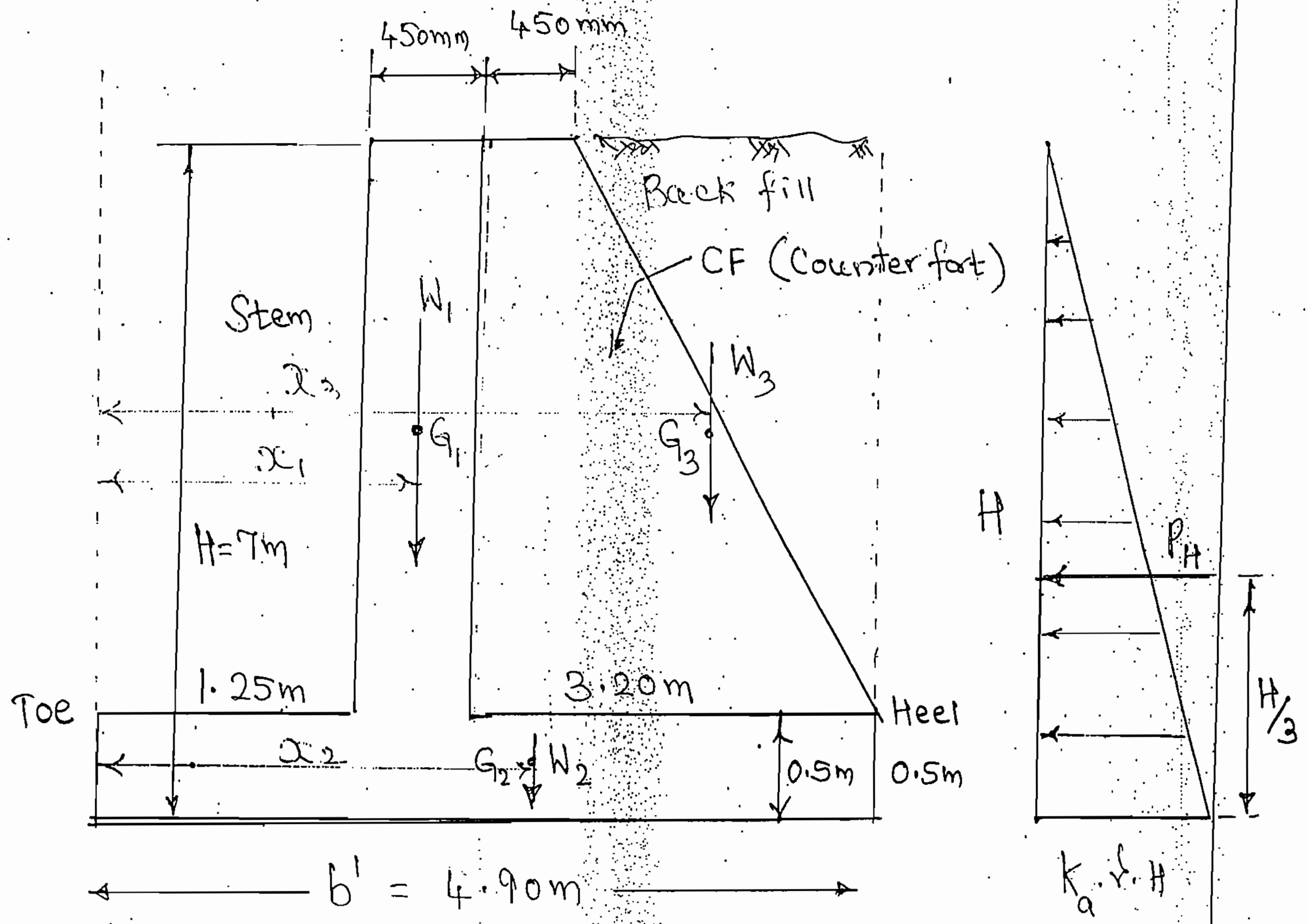
$\therefore$ Over all Depth $= D = 109.55 + 60 = 169.55$ mm

★ Increase the above depth by 2.50 times from shear Consideration.

Take $D_S = 2.50 * 169.55 = 423.86$ Say 450 mm.

$\therefore d = 390$ mm.

Provide $\boxed{D_S = 450 \text{ mm and } d = 390 \text{ mm.}}$



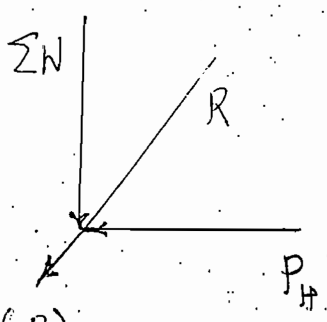
Step # 2: Stability of Retaining wall:

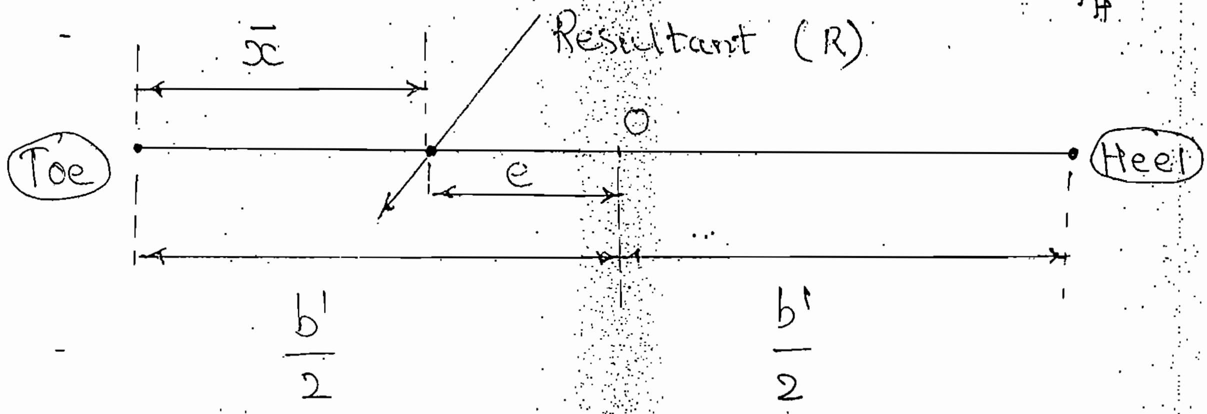
Consider 1m length of the wall perpendicular to the plane of the paper.

$P_H = \frac{1}{2} * 0.2948 * 16 * 7 * 7 = 115.56 \text{ KN} @ \frac{H}{3} = \frac{7}{3} \text{ m}$ from base.

Over turning Moment $= M_o = P_H * \frac{H}{3} = 115.56 * \frac{7}{3}$

$M_o = 269.64 \text{ KN-m}.$





(i) Check for bottom Soil pressure:

Line of action of Resultant $= \bar{x} = \frac{MR - M_o}{\Sigma W}$

$\bar{x} = \frac{1358.17 - 269.64}{467.18} = 2.33 \text{ m}.$

$e = \frac{b'}{2} - \bar{x} = \frac{4.90}{2} - 2.33 = 0.124 \text{ m} < \frac{b'}{6}$ ok

Density of earth = 16 kN/m^3

Density of Concrete = 25 kN/m^3

Components	Weight (kN)	Distance upto toe (m)	Moment about Toe (kN-m)
1. Backfill: $W_3 = (3.20 * 1 * 6.50) * 16$ (Earth Density)	332.80	$x_3 = 1.25 + 0.45 + \frac{3.20}{2}$; $x_3 = 3.30m$	1098.24
2. Stem: $W_1 = (0.45 * 1 * 6.50) * 25$ (density of Concrete)	73.13	$x_1 = 1.25 + \frac{0.45}{2}$; $x_1 = 1.475m$	107.87
3. Base Slab: $W_2 = (4.90 * 1 * 0.5) * 25$ (Concrete density)	61.25	$x_2 = \frac{4.90m}{2}$	150.06
$\Sigma W =$	467.18	$\Sigma M_R =$	1356.17

Total weight = ΣW = 467.18 kN

Total Resisting Moment = MR = 1356.17 kN-m

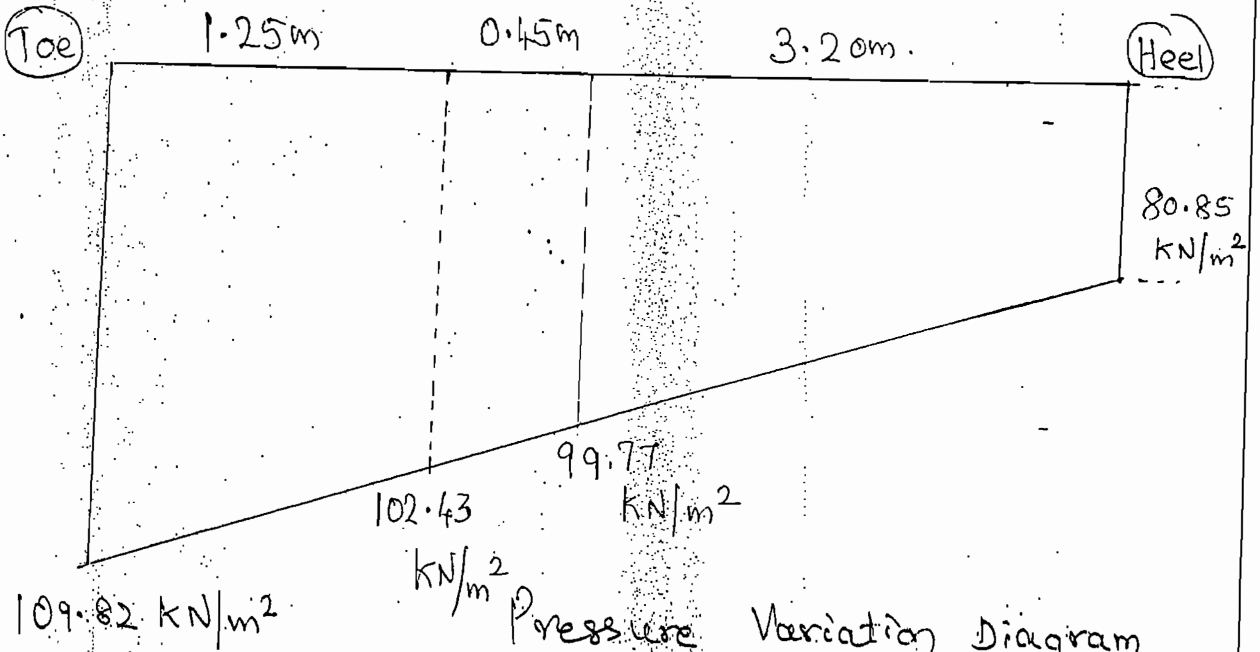
Total Earth pressure = $P_H = \frac{1}{2} K_a \cdot \gamma H \cdot H$

$$\left.\begin{array}{l}\text{Maximum and minimum} \\ \text{Upward Soil pressure}\end{array}\right\} = f = \frac{\Sigma W}{b'}\left[1 \pm \frac{6e}{b'}\right]$$

$$f = \frac{467.18}{4.90}\left[1 \pm \frac{6 * 0.124}{4.9}\right] = 95.34\left[1 \pm 0.152\right]$$

$$f_{max} = 95.34\left[1 + 0.152\right] = 109.82 \ kN/m^2$$

$$f_{min} = 95.34\left[1 - 0.152\right] = 80.85 \ kN/m^2$$



Pressure Variation Diagram

* By Interpolation, find Stresses:

Proceed from Toe.

0 m	---	$109.82 \ kN/m^2$
1.25 m	---	?
4.90 m	---	$80.85 \ kN/m^2$

$$\text{Stress} = 109.82 + \left(\frac{80.85 - 109.82}{4.90 - 0.00}\right)(1.25 - 0.0)$$

$$= 102.43 \text{ kN/m}^2$$

again,

0m	109.82 kN/m²
1.70m	?
4.90m	80.85 kN/m²

$$\text{Stress} = 109.82 + \left(\frac{80.85 - 109.82}{4.90 - 0.00}\right)(1.70 - 0.00)$$

$$= 99.77 \text{ kN/m}^2$$

(ii) Check for over turning:

$$\left.\begin{array}{l}\text{Factor of Safety against}\\ \text{Over turning}\end{array}\right\} = \frac{M_R}{M_O} \geq 1.50$$

$$= \frac{1356.17}{269.64} = 5.03 > 1.50 \text{ (Safe)}$$

(iii) Check for Sliding:

Assume $\mu = 0.50$

$$\left.\begin{array}{l}\text{Factor of Safety against}\\ \text{Sliding}\end{array}\right\} = \frac{\mu \Sigma W}{P} \geq 1.50$$

$$= \frac{0.5 * 467.18}{115.56} = 2.02 > 1.50 \text{ (Safe)}$$

Step # 3 : Design of Stem :

$(M_u)_{Stem} = 41.40$ kN-m, $D_s = 450$ mm $d_s = 390$ mm.

Using Eqn' of IS 456 : 2000, clause G.1.1.

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

$b = 1m = 1000$ mm,

$$41.40 * 10^6 = \left[0.87 * 415 * A_{st} * 390\right]\left[1 - \frac{A_{st} * 415}{1000 * 390 * 25}\right]$$

$$41.40 * 10^6 = 140.81 * 10^3 A_{st}\left[1 - 42.54 * 10^{-6} A_{st}\right]$$

$$294.01 = A_{st} - 42.54 * 10^{-6} (A_{st})^2$$

$$\therefore 42.54 * 10^{-6} (A_{st})^2 - A_{st} + 294.01 = 0$$

Solving, $A_{st} = 297.78 \text{ mm}^2$.

Check for minimum Steel:

Area = 0.12 % of gross area

$$\text{Area} = \frac{0.12}{100} * \underset{b}{1000} * \underset{D}{450} = 540\ mm^2.$$

* Choose minimum of 12mm ϕ bars.

$$\text{Spacing of 12mm } \phi \text{ bars} = \frac{\text{area of 1 bar}}{A_{st}} * \underset{1\text{ meter}}{1000}$$

$$= \frac{\frac{\pi}{4} * 12^2}{540} * 1000$$

$$= 209.44\ mm\ c/c \quad \text{Say } 200\ mm\ c/c.$$

$\boxed{\text{Spacing of 12mm } \phi \text{ bars @ 200mm c/c}} < 3d$ or 300 mm

(i) <u>Distribution Steel for Stem:</u>

$$\text{Area} = \frac{0.12}{100} * b * D = \frac{0.12}{100} * 1000 * 450$$

$$= 540\ mm^2.$$

* Choose minimum of 10mm ϕ bars.

$$\text{Spacing of 10mm } \phi \text{ bars} = \frac{\text{Area of 1 bar}}{A_{st}} * 1000$$

$$= \left[\left(\frac{\pi}{4} * 10^2\right) / 540\right] * 1000$$

$= 145.44$ Say 140 mm c/c.

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 140 mm c/c}}$ < 5d or 450 mm

(ii) Check for shear in stem:

$V_u = 82.78$ KN (already calculated)

$b = 1m = 1000$ mm, $d = 390$mm, $A_{st} = 540mm^2$

M_{25} and Fe 415.

Nominal Shear Stress $= \tau_v = \dfrac{V_u}{bd} = \dfrac{82.78 * 10^3}{1000 * 390}$

$$\tau_v = 0.212 \ N/mm^2.$$

$$\frac{100 A_{st}}{bd} = \frac{100 * 540}{1000 * 390} = 0.138$$

from IS 456 - 2000, Table 19, Page 73,

for $\dfrac{100 A_{st}}{bd} \leq 0.15$, M_{25},

(τ_c) Permissible $= 0.28 \ N/mm^2.$

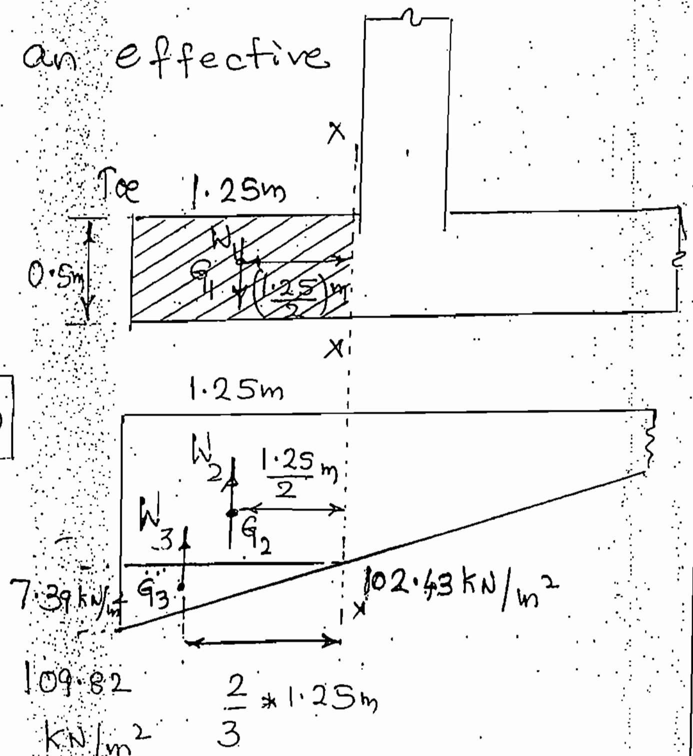
Comparing, $\tau_c > \tau_v$. Hence Safe.

<u>Step # 4:</u> <u>Design of Toe Slab:</u>

D = 500mm, Assuming an effective Cover of 60mm,

d = 440 mm.

$\boxed{D = 500\,mm, \quad d = 440\,mm}$



$$M_1 = W_1 x_1 = (1.25 * 0.5 * 1) * 25 * \frac{1.25}{2} = 11.72\ kN\text{-}m$$

$$M_2 = W_2 x_2 = (1.25 * 102.43 * 1) * \frac{1.25}{2} = 80.02\ kN\text{-}m$$

$$M_3 = W_3 x_3 = \left(\frac{1}{2} * 7.39 * 1.25 * 1\right) * \frac{2}{3} * 1.25 = 3.85\ kN\text{-}m$$

Nett Moment $= M = M_2 + M_3 - M_1$

$$M = 80.02 + 3.85 - 11.72 = 72.15\ kN\text{-}m$$

Factored moment $= M_u = 1.5M = 1.5 * 72.15$

$$\boxed{M_u = 108.23 \text{ kN-m}}$$

Shear force $V = W_2 + W_3 - W_1$

$$V = (1.25 * 102.43 * 1) + \left(\frac{1}{2} * 7.39 * 1.25 * 1\right)$$

$$\ominus (1.25 * 0.5 * 1 * 25)$$

$$V = 117.03 \text{ kN}.$$

factored Shear force $V_u = 1.5V$

$$\boxed{V_u = 1.5 * 117.03 = 175.55 \text{ kN}}$$

(i) <u>Check for depth:</u>

Using $(M_u)_{lim} = 0.138\, bd^2 f_{ck}$, $b = 1m = 1000mm$

$$108.23 * 10^6 = 0.138 * 1000 * d^2 * 25$$

$$d^2 = \frac{108.23 * 10^6}{0.138 * 1000 * 25} = 31371.01$$

$$d = \sqrt{31371.01} = 177.11 \text{ mm} < 440 \text{mm}$$ Safe.

(ii) Area of Steel (A_{st}):

Using Eqn' of IS 456-2000, Clause G.1.1

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

$b = 1m = 1000 \text{ mm}$

$$108.23 * 10^6 = 0.87 * 415 * A_{st} * 440 \left[1 - \frac{415 * A_{st}}{1000 * 440 * 25}\right]$$

$$108.23 * 10^6 = 158.86 * 10^3 A_{st} \left[1 - 37.73 * 10^{-6} A_{st}\right]$$

$$681.30 = A_{st} - 37.73 * 10^{-6} (A_{st})^2$$

or $$37.73 * 10^{-6} (A_{st})^2 - A_{st} + 681.30 = 0$$

Solving, $A_{st} = 699.78 \text{ mm}^2$

Provide 12 mm ϕ bars.

Spacing of 12mm ϕ bars $= \dfrac{\text{Area of 1 bar}}{A_{st}} * 1000 \text{ mm}$

$$= \frac{\frac{\pi}{4} * (12)^2}{699.78} * 1000 = 161.62 \text{ mm c/c}$$

Say 160 mm c/c.

$\boxed{\text{Spacing of 12mm } \phi \text{ bars @ 160 mm c/c}}$ < 3d or 300 mm

(iii) Distribution Steel for Toe:

$$\text{Area} = \frac{0.12}{100} * (\overset{b}{1000}) * (\overset{D}{500}) = 600 \text{ mm}^2$$

* Provide 10 mm ϕ bars.

$$\text{Spacing of 10 mm } \phi \text{ bars} = \frac{\text{Area of 1 bar}}{A_{st}} * 1000 \text{ mm}$$

$$= \frac{\left(\frac{\pi}{4}\right) * (10)^2}{600} * 1000 = 130.90 \text{ mm}$$

Say 130 mm c/c.

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 130 mm c/c}}$ < 5d or 450 mm

(iv) Check for shear in Toe:

b = 1000 mm.

$$\text{Nominal Shear Stress} = \tau_v = \frac{V_u}{bd}$$

$$\tau_v = \frac{175.55 * 10^3}{1000 * 440} = 0.399 \ N/mm^2$$

$$\frac{100 A_{st}}{bd} = \frac{100 * 699.78}{1000 * 440} = \boxed{0.16} \ \& \ M_{20}$$

As per table # 19, pg 73 of IS 456-2000

$\frac{100 A_{st}}{bd}$	τ_c (M_{25})
0.15	0.29
0.16	?
0.25	0.36

$$\tau_c = 0.29 + \left(\frac{0.36 - 0.29}{0.25 - 0.15}\right)\left(0.16 - 0.15\right)$$

$$\tau_c = 0.297 \ N/mm^2.$$

Comparing, $\tau_c < \tau_v$ (unsafe)

* Hence, increase the thickness of the Toe slab using $\tau_v = \frac{V_u}{\ } = \tau_c$

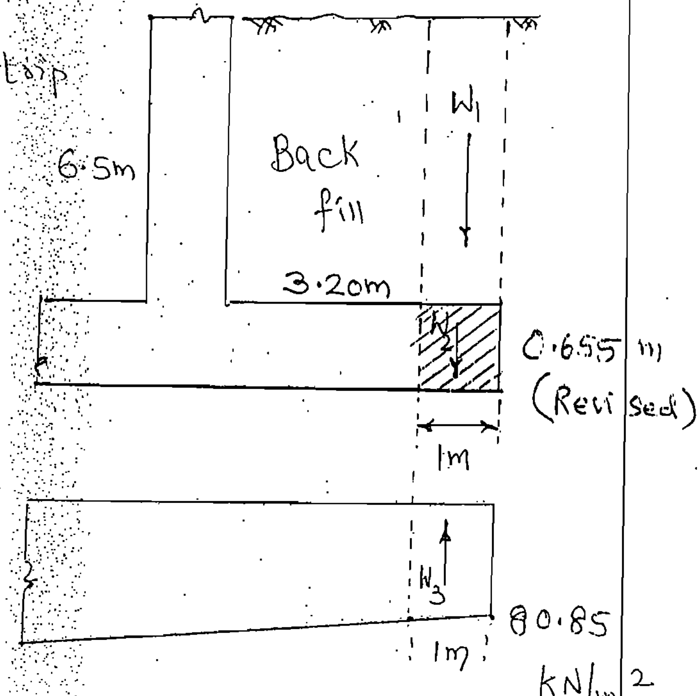
$$\frac{175.55 * 10^3}{1000 * d} = 0.297$$

$$\therefore d = \frac{175.55}{0.297} = 591 \text{ mm} \quad \text{Say } 595 \text{ mm}$$

$\therefore$ $\boxed{\text{Provide } d = 595 \text{ mm } \bar{c} \ D = 655 \text{ mm}}$ for Toe Slab

Step #5: Design of Heel Slab:

* Consider 1 m wide Strip of the Heel Slab near the outer edge. Due to Counter fort, the Heel Slab is designed as a Horizontal one way Continuous Slab.



Pressure Variation diagram

Now, $W = W_1 + W_2 - W_3$

Density of earth

$$W = (1 * 6.50 * 1 * 16) + (1m * 0.655 * 25) - (80.85 * 1) = 39.53 \text{ kN/m}$$

Spacing of Counterfort

Max BM $= M = \frac{wl^2}{10} = \frac{39.53 * 3^2}{10}$

$M = 35.57$ kN-m.

Factored Moment $(M_u) = 1.50\ M$

$(M_u)_{Heel} = 1.50 * 35.57 = 53.34$ kN-m.

$$\boxed{(M_u)_{Heel} = 53.34 \text{ kN-m.}}$$

- Shear force $V = 0.6\ wl$

$= 0.6 * 39.53 * 3$

$V = 71.15$ kN

Factored Shear force $V_u = 1.50\ V$

$\therefore (V_u)_{Heel} = 1.50 * 71.15 = 106.73$ kN.

$$\boxed{(V_u)_{Heel} = 106.73 \text{ kN}}$$

(i) Check for depth:

Using $(M_u)_{lim} = 0.138\, bd^2 f_{ck}$; $b = 1000mm$

$$53.34 * 10^6 = 0.138 * 1000 * d^2 * 25$$

$$d^2 = \frac{53.34 * 10^6}{0.138 * 1000 * 25} = 15460.87$$

$$d = \sqrt{15460.87} = 124.34 \text{ mm} < 595 \text{mm}$$

Safe.

(ii) Area of Steel (A_{st}):

Using eqn' of IS 456-2000, clause G.1.1.

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

$$(53.34) * 10^6 = \left[0.87 * 415 * A_{st} * 595\right]\left[1 - \frac{A_{st} * 415}{1000 * 595 * 25}\right]$$

$$53.34 * 10^6 = 214.82 * 10^3 A_{st}\left[1 - 27.90 * 10^{-6} A_{st}\right]$$

$$248.30 = A_{st} - 27.90 * 10^{-6} (A_{st})^2$$

$$27.9 * 10^{-6} (A_{st})^2 - A_{st} + 248.30 = 0.$$

Solving, $(A_{st}) = 250.04\ mm^2$.

Minimum Steel $= \frac{0.12}{100} * b * D$

$= \frac{0.12}{100} * 1000 * 655 = 786\ mm^2$.

Provide 12mm ϕ bars.

Spacing of 12mm ϕ bars $= \frac{\text{Area of 1 bar}}{A_{st}} * 1000m$

$= \frac{\left(\frac{\pi}{4}\right) * 12^2}{786} * 1000$

$= 143.90$ mm Say 140 mm c/c.

$\therefore$ $\boxed{\text{Spacing of 12mm } \phi \text{ bars @ 140mm c/c}}$ < 3d or 300 mr

(iii) Distribution Steel for Heel Slab:

Area $= \frac{0.12}{100} * b * D = \frac{0.12}{100} * 1000 * 655$

$= 786\ mm^2$.

* Provide the Spacing Same as main bars.

$\boxed{\text{Spacing of 12mm } \phi \text{ bars @ 130 mm c/c}}$ < 5d or 450 mm

(IV) Check for Shear in Heel: b = 1000 mm

Nominal Shear Stress = $\tau_v = \dfrac{V_u}{bd}$

$$\tau_v = \frac{106.73 * 10^3}{1000 * 595} = 0.18 \text{ N/mm}^2$$

$$\frac{100 A_{st}}{bd} = \frac{100 * 786}{1000 * 595} = 0.132.$$

from IS 456 - 2000, Table # 19, Pg 73,

for $\dfrac{100 A_{st}}{bd} \leq 0.15$, $(\tau_c) = 0.29 \text{ N/mm}^2$

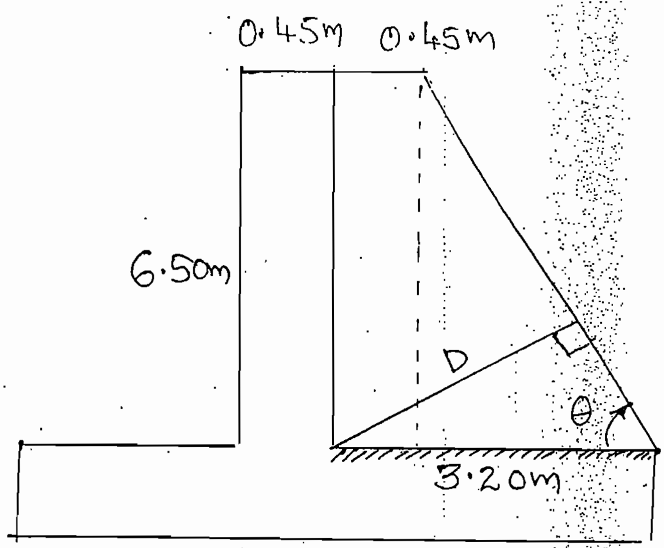
Comparing, $\tau_c > \tau_v$ (Safe).

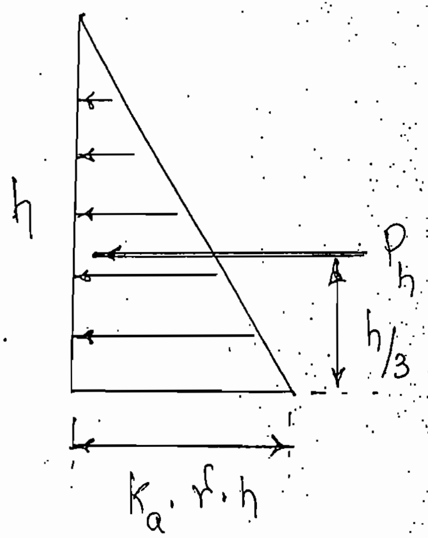
Step # 6: Design of Counter forts (CF):

The Counter fort is designed like a Cantilever beam Subjected to Soil or earth

Pressure.

* Assume thickness of CF = Stem thickness = 450 mm





$$\tan\theta = \frac{6.50}{(3.20 - 0.45)} = 2.3636$$

$$\theta = \tan^{-1}(2.3636) = 67.067^\circ$$

$$\sin\theta = \frac{D}{(3.20)\,m} \quad \text{or} \quad D = (3.20)\sin\theta$$

$$D = 3.20 * \sin(67.067^\circ) = 2.95\,m \text{ Say } 3m.$$

$$\therefore D = 3000\ mm = 3m.$$

Assuming an effective Cover of 60 mm

$d = D - 60 = 3000 - 60 = 2940\text{ mm}$

$\boxed{\text{Provide } D = 3000\text{ mm } \& \ d = 2940\text{ mm}}$

* $P_h = (K_a \cdot \gamma \cdot h) * \frac{1}{2} * h$, K_a = Co-eff of active earth pr.

$= \frac{1}{2} * 0.2948 * 16 * (6.50)^2$

$P_h = 99.64$ kN @ $\frac{h}{3}$ from base.

$\boxed{\text{* BM for Counter fort} = \left[P_h * \frac{h}{3}\right] * L}$ ← Spacing of CF

$\therefore M = \left[99.64 * \frac{6.50}{\cancel{3}}\right] * \cancel{3} = 647.68\text{ kN-m}$

Factored Moment $= (M_u) = 1.50\,M$

$\boxed{(M_u)_{CF} = 1.50 * 647.68 = 971.51\text{ kN-m}}$

(i) Area of Steel (A_{st}):

Using eqn of IS 456 - 2000, clause G.1.1.

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

$$971.51 * 10^6 = [0.87 * 415 * A_{st} * 2940]\left[1 - \frac{A_{st} * 415}{450 * 2940 * 2}\right]$$

$$971.51 * \cancel{10^6} = 1.06 * \cancel{10^6} A_{st}\left[1 - 12.55 * 10^{-6} A_{st}\right]$$

$$971.51 = 1.06 \; A_{st}\left[1 - 12.55 * 10^{-6} A_{st}\right]$$

$$\therefore \; 12.55 * 10^{-6} (A_{st})^2 - A_{st} + 916.52 = 0.$$

Solving, $A_{st} = 927.31 \text{ mm}^2$.

★ Provide minimum 20 mm ϕ bars.

$$\text{No of bars} = \frac{A_{st}}{\text{Area of 1 bar}} = \frac{927.31}{\frac{\pi}{4} * 20^2} = 2.95 \simeq$$

(Not safe)

Check for minimum steel:

By Page 47 of IS 456 - 2000, For beams,

for beams, $\boxed{\dfrac{A_{st}}{b \cdot d} = \dfrac{0.85}{f_y}}$

$$\frac{A_{st}}{450 * 2940} = \frac{0.85}{415}$$

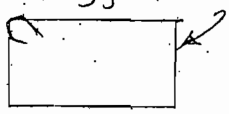
$$\therefore A_{st} = \frac{0.85 * 450 * 2940}{415} = 2709.75\ mm^2$$

No of 25mm ϕ bars $= \dfrac{2709.75}{\frac{\pi}{4} * 25^2} = 5.5$ say 6.

$\boxed{\text{Provide 6 bars of 25 mm } \phi.}$

Step # 7. Design of links:

(i) Design of horizontal links:



★ It is used to connect the CF & Stem.

Load taken for design of Stem $= \omega = 30.66\ kN/m$

$\therefore$ Factored load $= \omega_u = 1.5\omega = 1.5 * 30.66$

$= 45.99 \approx 46\ kN/m.$

Spacing of CF

* Load for link = $(w_u) * l$ = $46 * 3$ = 138 kN/m

Area of Steel = $\boxed{A_{st} = \frac{\text{force}}{0.87 f_y}}$ = $\frac{138 * 10^3}{0.87 * 415}$

A_{st} = 382.22 mm^2.

Provide 10mm ϕ 2 legged links.

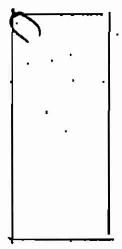
Spacing = $\frac{\text{Area of 1 bar}}{A_{st}} * 1000$ mm

= $\frac{\left(\frac{\pi}{4}\right) * 10^2}{382.22} * 1000$ = 205.48 Say 200 mm c/c

$\boxed{\text{Provide 10mm}\phi\text{, 2legged links @ 200mm c/c.}}$

(ii) Design of Vertical links:

* It is used to Connect the CF and Heel.



2legged, 10mmϕ

* Load taken for design of Heel = 39.53 kN/m

$\therefore$ w = 39.53 kN/m.

$\therefore W_u = 1.50\ W = 39.53 * 1.5 = 59.30\ kN/m$

$$A_{St} = \frac{\text{Force}}{0.87 f_y} = \frac{W_u * (\text{Spacing of CF})}{0.87 f_y}$$

$$A_{St} = \frac{10^3 * 59.30 * 3}{0.87 * 415} = 492.69\ mm^2.$$

Provide 10mm ϕ 2 legged links.

$$\therefore \text{Spacing} = \frac{\text{Area of 1 bar}}{A_{St}} * 1000$$

$$= \frac{\frac{\pi}{4} * 10^2}{492.69} * 1000$$

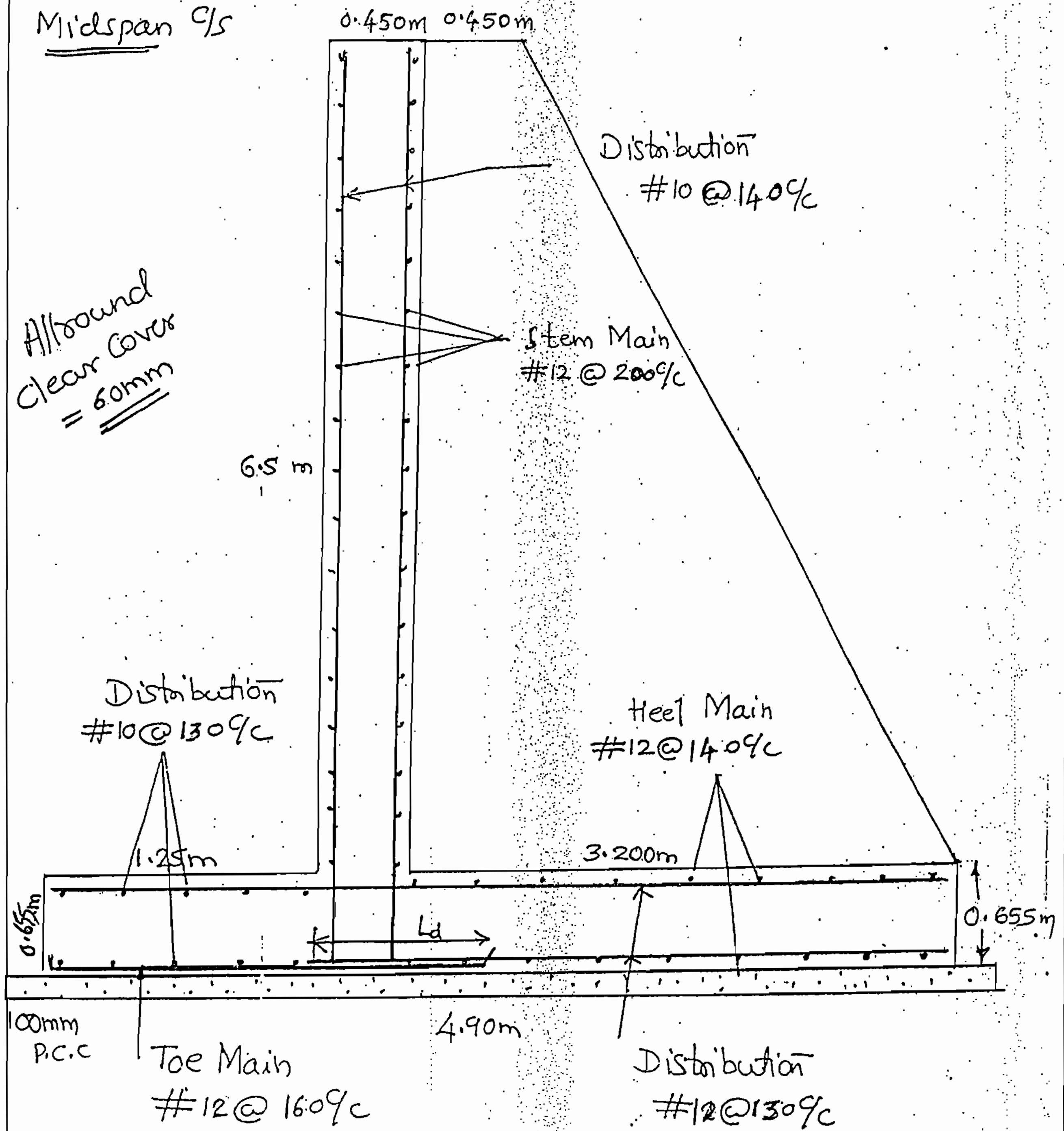
$= 159.4$ Say 150 mm c/c.

Provide 10mm ϕ, 2 legged links @ 150mm c/c

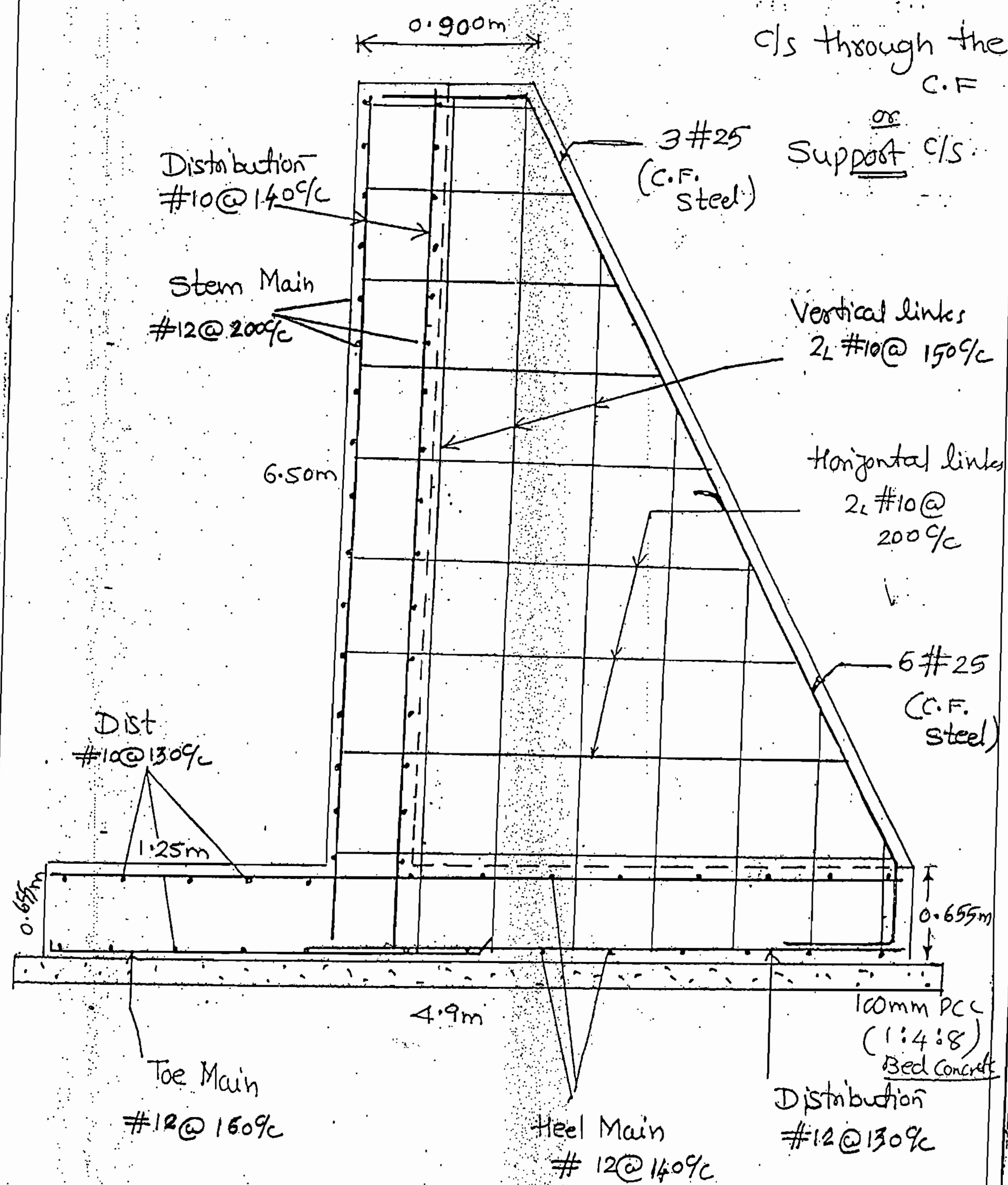
L_d = development length or Bond length

= $40\ \phi$ for M_{25} Concrete grade.

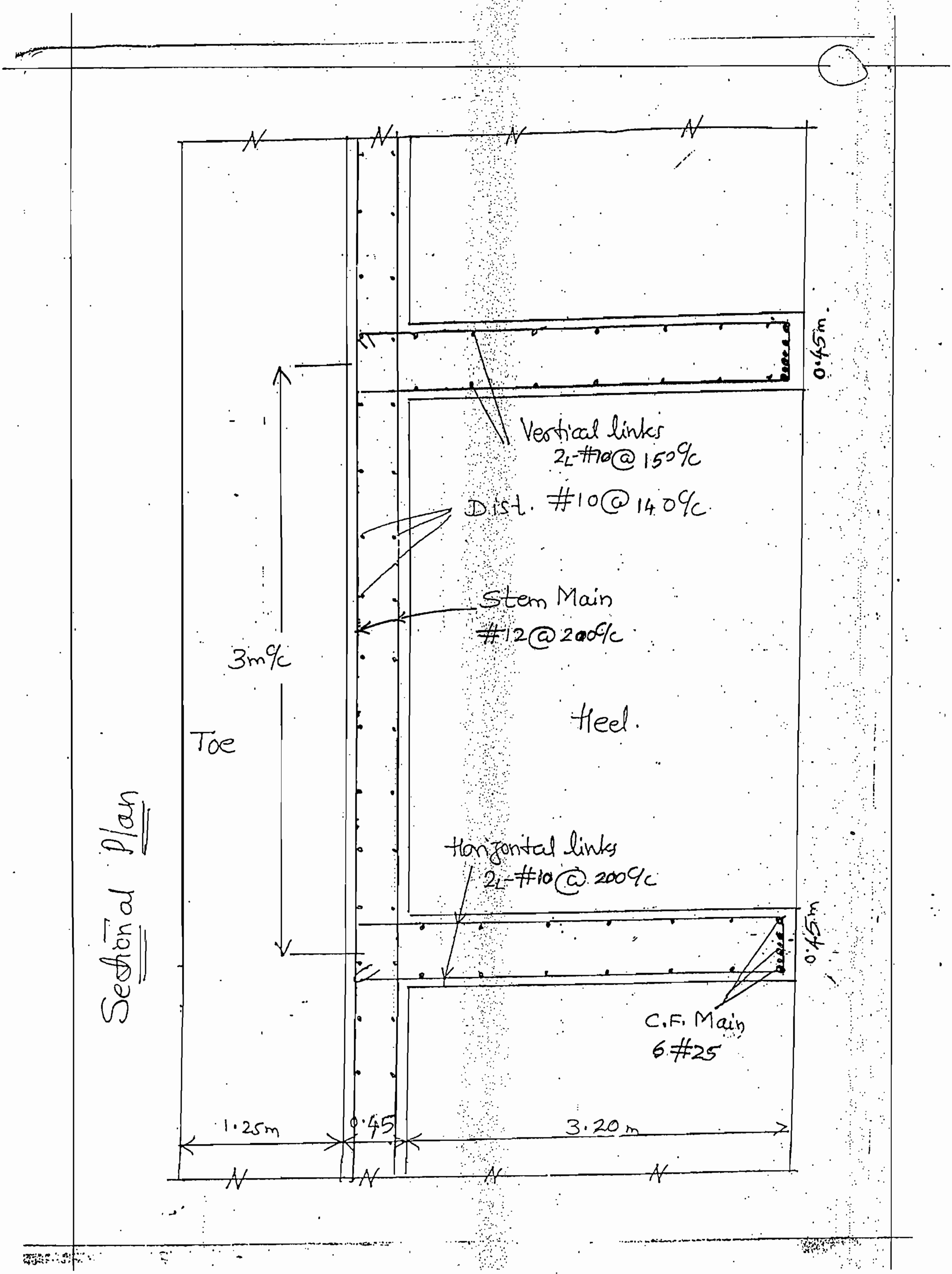
C/S Inbetween C.F
or
Midspan C/S



$L_d = 40\phi$ (For M25)



C/s through the C.F or Support C/s
0.900m
3#25 (C.F. steel)
Distribution #10@140c/c
Stem Main #12@200c/c
Vertical links 2L #10@ 150c/c
6.50m
Horizontal links 2L #10@ 200c/c
6#25 (C.F. steel)
Dist #10@130c/c
1.25m
0.655m
0.655m
4.9m
100mm PCC (1:4:8) Bed Concrete
Toe Main #12@ 160c/c
Heel Main #12@140c/c
Distribution #12@130c/c



Vertical links
2L-#10@150c/c
Dist. #10@140c/c
Stem Main
#12@200c/c
3m c/c
Toe
Heel.
Horizontal links
2L-#10@200c/c
0·45m
0·45m
C.F. Main
6#25
1·25m
0·45
3·20m
Sectional Plan

MODULE # 1. DESIGN OF WATER TANKS

Working Stress Method

σ_{cbc} = Compressive Stress in Concrete in bending

σ_{ct} = tensile stress in Concrete

σ_{st} = tensile Stress in Steel

	σ_{cbc}	σ_{ct}
M_{20}	$7\ N/mm^2$	$1.20\ N/mm^2$
M_{25}	$8.5\ N/mm^2$	$1.30\ N/mm^2$

Hysd bars or Fe 415

$\Rightarrow \sigma_{st} = 150\ N/mm^2$ or $190\ N/mm^2$

Constants:

m = modular ratio $= \dfrac{280}{3\sigma_{cbc}}$

k = Stiffness of member $= m\sigma_{cbc}$

j = lever arm = $\left(1 - \frac{k}{3}\right)$

$Q = R = \frac{1}{2} * \sigma_{cbc} * k * j$ = Resisting Moment Co-eff.

Effective depth = $d = \sqrt{\frac{M}{Qb}}$

Using 40 mm effective Cover,

$D = T = (d + 40)$ --- mm.

Area of Steel (A_{st}):

(i) If Moment is known, $\boxed{A_{st} = \frac{M}{\sigma_{st} * j * d}}$

(ii) If force is known, $\boxed{A_{st} = \frac{Force}{\sigma_{st}}}$

Approximate "Thickness" of Tank Wall (T):

(i) $\boxed{T = 30H + 50}$, H = depth of tank in mete

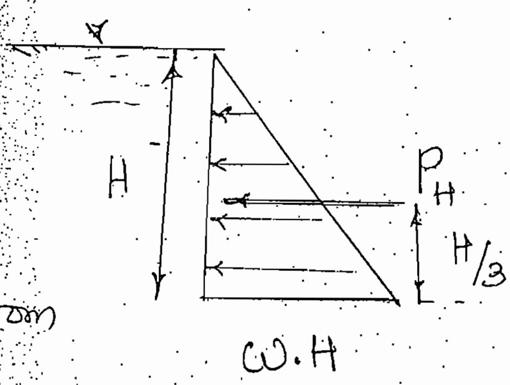
(ii) $$\boxed{\sigma_{ct} = \frac{\text{Max. Hoop's Tension}}{1000\,T + (m-1)\,A_{st}}}$$

ω = Unit weight of Water = $10\,kN/m^3$

$$\boxed{\text{Minimum Steel in Tank} = 0.3\,\%\text{ of Gross Area}}$$

$P_H = \frac{1}{2}(\omega H)\,H$

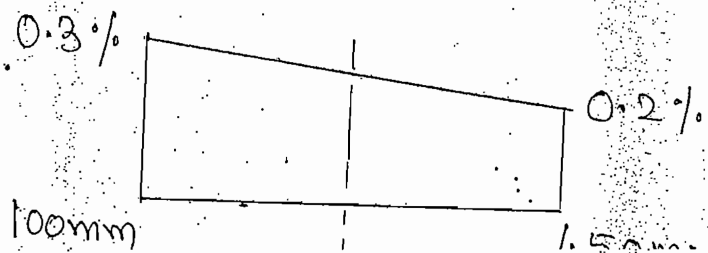
$P_H = \frac{1}{2}\,\omega H^2$ @ $\frac{H}{3}$ from base, $\omega = 10\,kN/m^3$.



Minimum Reinforcement:

Upto 100 mm thick wall ⟶ 0.3 % of gross Area

from 100 mm to 450 mm thick wall, ⟶ reduces to 0.2 % of gross Area

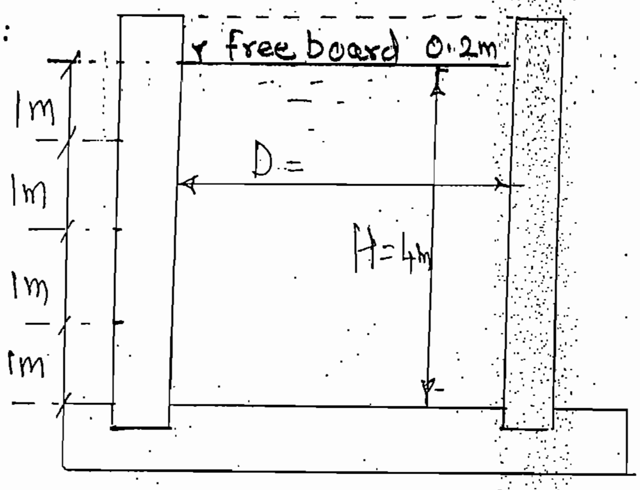


★ Calculate by inter

(I) FLEXIBLE BASE CIRCULAR WATER TANK (On Ground)*

Problem # 1. Design a circular water tank with flexible base for a capacity of $4*10^5$ liters. The depth of water in the tank is to be 4m with a free board of 200 mm. Use M_{25} grade concrete and fe 415 steel.

Soln:



$1 m^3 = 1000$ lit

Capacity = Volume

1 lit $= \left(\frac{1}{1000}\right) m^3$

$\sigma_{st} = 150 N/mm^2$

$$4*10^5 \text{ lit} = \frac{\pi D^2}{4} * H$$

$$\therefore \left(\frac{4*10^5}{1000}\right) m^3 = \frac{\pi * \cancel{4}}{\cancel{4}} * D^2$$

$$\therefore D^2 = \frac{400}{} \quad \text{or} \quad D = \sqrt{\frac{400}{}}$$

$D = 11.28$ Say $11.30\,m.$

Step # 1. Design of Bottom 1m depth:

* $H = 4m$ from Top.

* $\boxed{\text{Max Hoop Tension} = \omega * H * \frac{D}{2}}$ KN

$= 10 * 4 * \frac{11.30}{2} = 226\ KN/m$

$= 226\ KN/m$ thickness

$\boxed{\text{Max Hoop tension} = 226\ KN}$ in the Bottom 1m height.

Area of Steel = $A_{St} = \frac{Force}{\sigma_{St}} = \frac{226 * 10^3}{150}$

* (Hoop Steel, Circular rings)

$A_{St} = 1506.67\ mm^2.$

Provide 12mm ϕ Circular rings.

Spacing of 12mm ϕ rings = $\frac{\text{Area of 1bar}}{A_{..}} * 1000\,m$

$$= \frac{\left(\frac{\pi}{4}\right) * 12^2}{1506.67} * 1000 = 75.06 \text{ mm c/c}$$

$\boxed{\text{Provide 12mm } \phi \text{ rings @ 75 mm c/c}}$ in the bottom 1m height. (Hoop's Steel)

Step # 2: Design of tank at H = 3m from Top.

$$\text{Hoop tension} = \left(\frac{\omega H D}{2}\right) \text{ kN}$$

$$= \frac{10 * 3 * 11.30}{2} = 169.50 \text{ kN}$$

$$A_{st} = \frac{\text{Force}}{\sigma_{st}} = \frac{169.50 * 10^3}{150} = 1130 \text{ mm}^2$$

$$\text{Spacing of 12mm } \phi \text{ bars} = \frac{\frac{\pi}{4} * 12^2}{1130} * 1000$$

$$= 100.08 \text{ mm c/c}$$

$\boxed{\text{Spacing of 12mm } \phi \text{ bars @ 100 mm c/c}}$

at H = 3m from top. (Hoop's Steel)

Step # 3: Design of tank at H = 2m from top:

$$\text{Hoop's Tension} = \left(\frac{WHD}{2}\right) \text{ kN}$$

$$= \frac{10 * 2 * 11.30}{2} = 113 \text{ kN}$$

$$A_{st} = \frac{\text{Force}}{\sigma_{st}} = \frac{113 * 1000 \text{ N}}{150} = 753.33 \text{ mm}^2$$

$$\text{Spacing of 12mm } \phi \text{ bars} = \frac{\left(\frac{\pi}{4}\right) * 12^2}{753.33} * 1000$$

$$= 150.13 \text{ mm c/c Say } 150 \text{ mm c/c}$$

$\boxed{\text{Spacing of 12mm } \phi \text{ bars @ 150 mm c/c}}$

at H = 2m from Top (Hoop's Steel)

Step # 4 : Tank wall thickness:

$M_{25} \Rightarrow \sigma_{cbc} = 8.50 \ N/mm^2$

$\sigma_{ct} = 1.30 \ N/mm^2$

$m = \text{modular ratio} = \dfrac{280}{3\sigma_{cbc}} = \dfrac{280}{3 * 8.50}$

$\boxed{m = 10.98}$

(i) $\boxed{T = 30H + 50}$ mm $= 30 * 4 + 50$

$\boxed{T = 170 \text{ mm}}$

OR

(ii) $\sigma_{ct} = \dfrac{\text{Max Hoop Tension}}{1000\,T + (m-1)\,A_{st}}$

* Max Hoop tension = 226 KN, $\rightarrow A_{st} = 1506.67 \ mm^2$

$1.30 = \dfrac{226 * 10^3}{1000\,T + (10.98-1)\,1506.67}$

$$1.30\left[1000T + 9.98 * 1506.67\right] = 226 * 10^3$$

$$1.30 * \cancel{10^3}\left[T + 15.04\right] = 226 * \cancel{10^3}$$

$$T = 173.85 - 15.04 = 158.81 \text{ mm}$$

Take $\boxed{T = 170 \text{ mm}}$

Step # 5: Design of top 1m Height;

H = 1m from top.

$\boxed{\text{Provide minimum Steel} = 0.3\ \%\ \text{of gross Area.}}$

$$\text{Area} = \left(\frac{0.3}{100}\right) * \underset{b}{1\text{m}} * \underset{\text{thickness}}{170 \text{ mm}}$$

$$= \frac{0.3}{100} * 1000 \text{ mm} * 170 = 510 \text{ mm}^2$$

$$\text{Spacing of } 12\text{mm } \phi \text{ bars} = \frac{\left(\frac{\pi}{4}\right) * 12^2}{510} * 1000$$

$$= 241.12 \text{ Say } 240 \text{mm c/c}$$

$\boxed{\text{Spacing of 12mm } \phi \text{ bars @ 240 mm c/c}}$

at H = 1m from top (Hoop's steel)

Step # 6: Design of base slab:

* Since the slab is resting on hard ground, there is no design for base slab.

However, provide a minimum thickness of 150 mm and a mesh of 10mm ϕ bars at 150 mm c/c in the top and bottom.

Step # 7: Design of Vertical Steel:

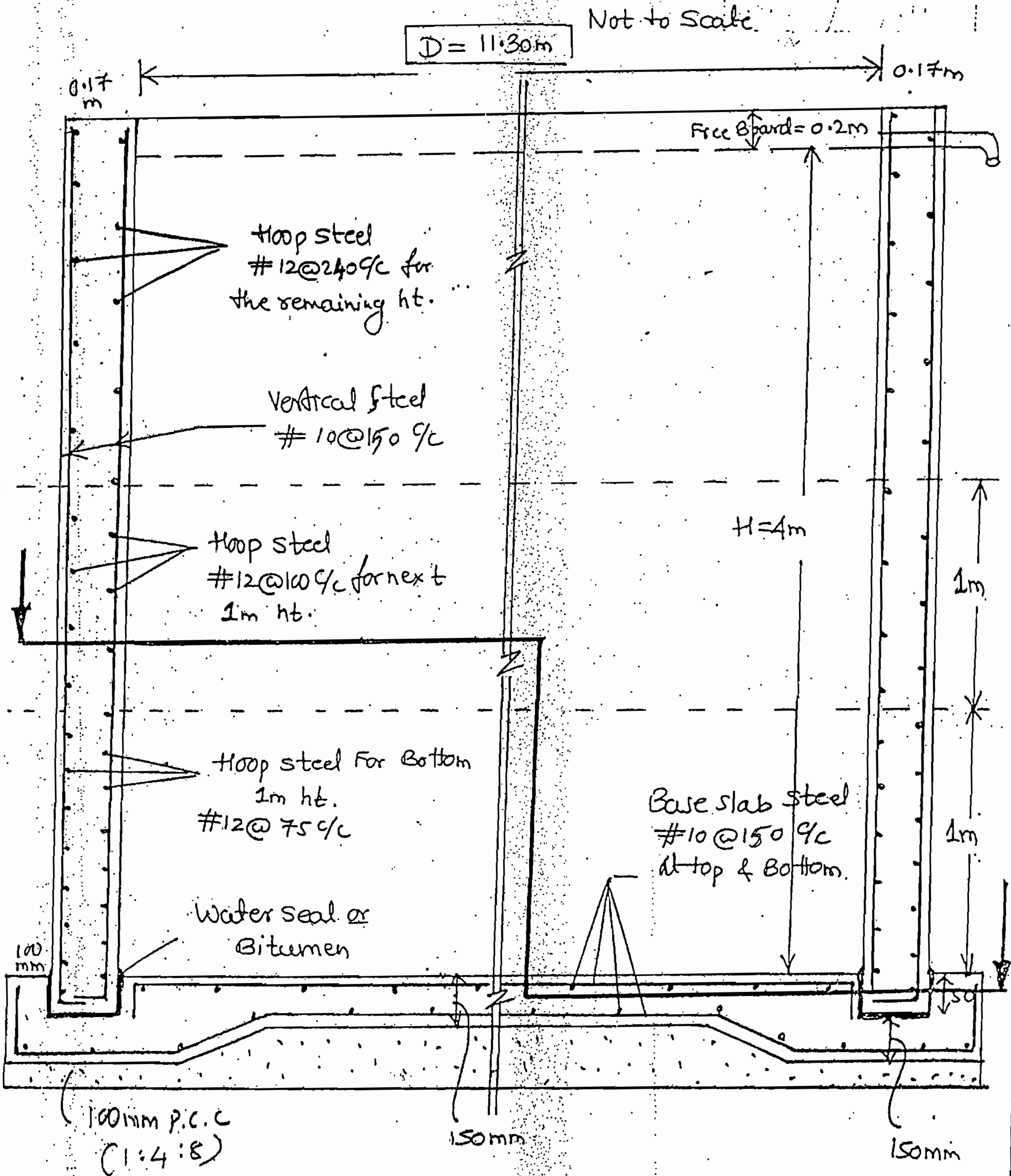
Provide minimum steel of 0.3 % of gross Area

$$\text{Area} = \frac{0.3}{100} * \overset{b=1m}{1000\,mm} * \overset{t}{170} = 510\,mm^2.$$

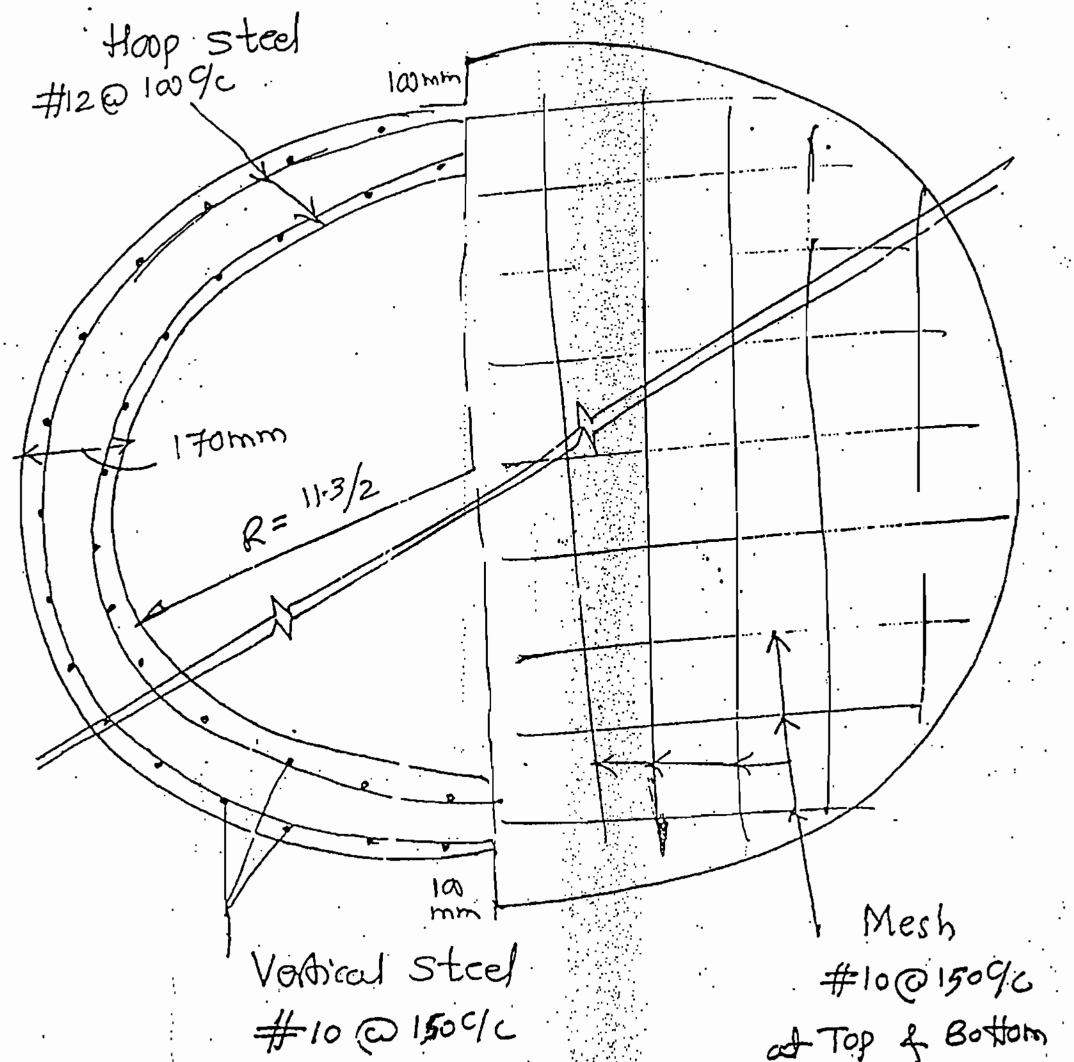
$$\text{Spacing of 10mm } \phi \text{ bars} = \frac{\frac{\pi}{4} * 10^2 * 1000}{510}$$

$\boxed{\text{Spacing of 10mm } \phi \text{ bars} = 154 \text{ Say } 150\,mm \text{ c/c.}}$

1:25



Allround Clear Cover = 30mm.

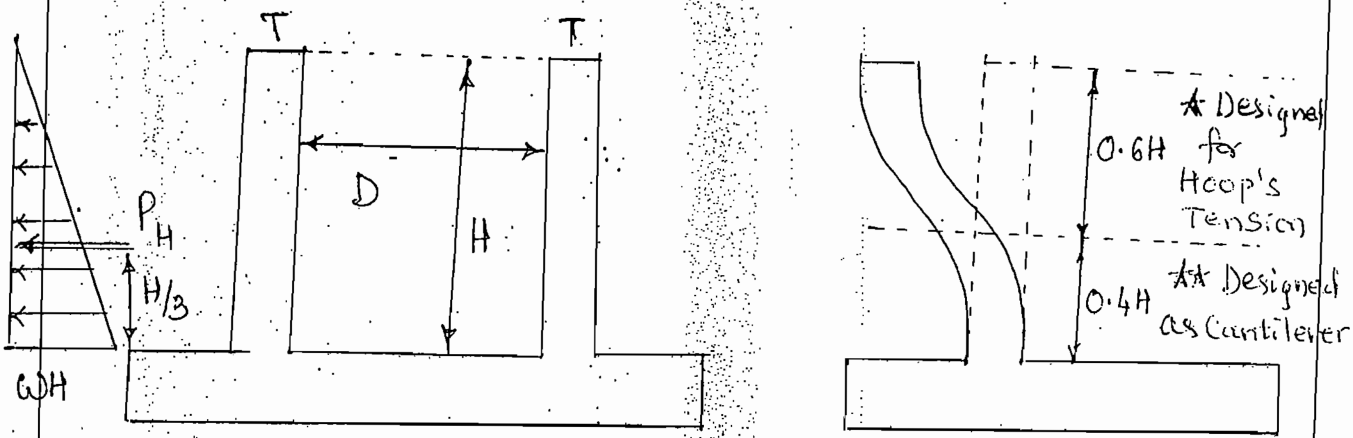


Half Plan through the wall
&
Half Plan through the Base Slab.

(II) "Rigid base" Circular Water tank

★ (on ground)

(IS 3370 Part IV)



(1) Bending Moment:

$$\boxed{M = (\text{Co-efficient}) * (\omega H^3)} \quad \text{kN-m.}$$

(2) Hoop tension:

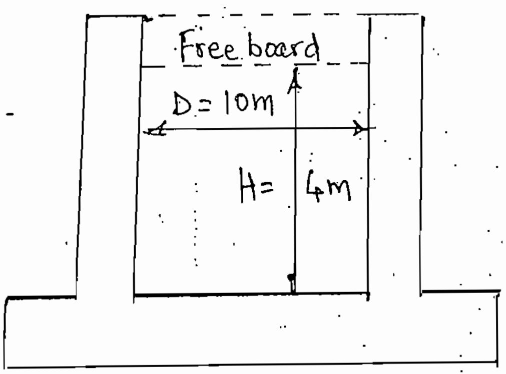
$$= (\text{Co-efficient}) * \left(\omega H \frac{D}{2}\right) \quad \text{kN.}$$

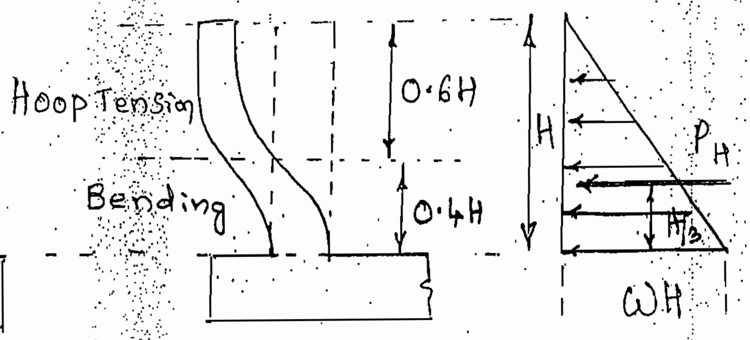
★ Use IS 3370, Part IV, Table (9) & (10) Pages 35 & 36 to get Co-efficient.

Problem # 1. Design a Circular tank of internal diameter 10 m and height 4 meter

With the Walls being restrainted at the base.
Use IS Code method, M_{25} Concrete and Fe 415 Steel.

Soln:





Step # 1. Constants:

M_{25} Concrete $\longrightarrow$ $\sigma_{cbc} = 8.5\ N/mm^2$, $\sigma_{ct} = 1.3\ N/mm^2$

Fe 415 $\longrightarrow$ $\sigma_{st} = 150\ N/mm^2$.

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3*8.50} = 10.98$$

$$K = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} = \frac{10.98*8.50}{10.98*8.50+150} = 0.38.$$

$$j = \left(1 - \frac{k}{3}\right) = \left(1 - \frac{0.38}{3}\right) = 0.87$$

$$Q = \frac{1}{2} * \sigma_{cbc} * k * j = \frac{1}{2} * 8.50 * 0.38 * 0.87$$

$$\boxed{Q = 1.421}$$

Step # 2: Thickness of tank wall:

(i) $T = (30H + 50)$ mm $\qquad H = 4$m.

$T = (30 * 4 + 50) = 170$mm $= 0.17$m

Using effective cover of 50 mm,

$d =$ effective depth $= T - 50$

$= 170 - 50 = 120$mm

$$\boxed{d = 120\text{mm}.}$$

(ii) Hoop's tension & Bending moment:

★ Use IS 3370, Part IV, Table (9) & (10)

Pages 35 & 36

$$\text{Ratio} = \left(\frac{H^2}{Dt}\right), \quad D = \text{dia of Tank}$$

$$\text{Ratio} = \frac{4^2}{10 * T} = \frac{4^2}{10 * 0.17} = \boxed{9.41}.$$

From Table (9) of I.S. 3370 - Part IV

Page 35,

8	---	0.575
$\boxed{9.41}$	---	?
10	---	0.608

Search for Max Value. It will occur at 0.6H.

$$\text{Co-eff} = 0.575 + \left(\frac{0.608 - 0.575}{10 - 8}\right)(9.41 - 8)$$

$$\boxed{\text{Co-eff} = 0.598}$$

Specific Wt of Water = 10 kN/m^3

$$\boxed{\text{Hoop Tension} = \left(\text{Co-eff} * \omega H * \frac{D}{2}\right)} \text{ kN}$$

$$= 0.598 * 10 * 4 * \frac{10}{2}$$

$$\boxed{\text{Hoop Tension} = 119.66 \text{ kN.}}$$

from Table 16 of IS 3370 - Part IV, Pg 36

8	----	0.0146
9.41	----	?
10	----	0.0122

Search for max Value. It will occur at 1H.

$$\text{Co-eff} = 0.0146 + \left(\frac{0.0122 - 0.0146}{10 - 8}\right)(9.41 - 8)$$

$$\boxed{\text{Co-eff} = 0.0129}$$

$\omega = 10 \text{ kN/m}^3$

Bending Moment $\boxed{M = \text{Co-eff} * \omega H^3}$ kN-m

$$\therefore M = 0.0129 * 10 * (4)^3 = 8.26 \text{ kN-m}.$$

$$\therefore \boxed{\text{Max BM} = 8.26 \text{ kN-m}}$$

Step # 3: Area of Steel (A_{st}):

(i) Hoop's Steel for Hoop tension:

$$A_{st} = \frac{\text{Hoop Tension}}{\sigma_{st}} = \frac{119.66 * 10^3}{150}$$

$$= 797.73 \text{ mm}^2$$

Provide 16mm ϕ bars.

Spacing of 16 mm ϕ bars $= \dfrac{\text{Area of 1 bar}}{A_{st}} * 1000$

$$= \frac{\frac{\pi}{4} * (16)^2}{797.73} * 1000$$

$= 252$ mm Say 250 mm c/c

$\boxed{\text{Spacing of 16 mm } \phi \text{ bars @ 250 mm c/c}}$

(ii) <u>Bending or Cantilever steel:</u>

$$A_{st} = \frac{M}{\sigma_{st} * j * d} = \frac{8.26 * 10^3}{150 * 0.87 * 120}$$

$A_{st} = 527.46\ mm^2 > 0.3 * b * \leftarrow 170\ mm$ (Safe)

1000 mm

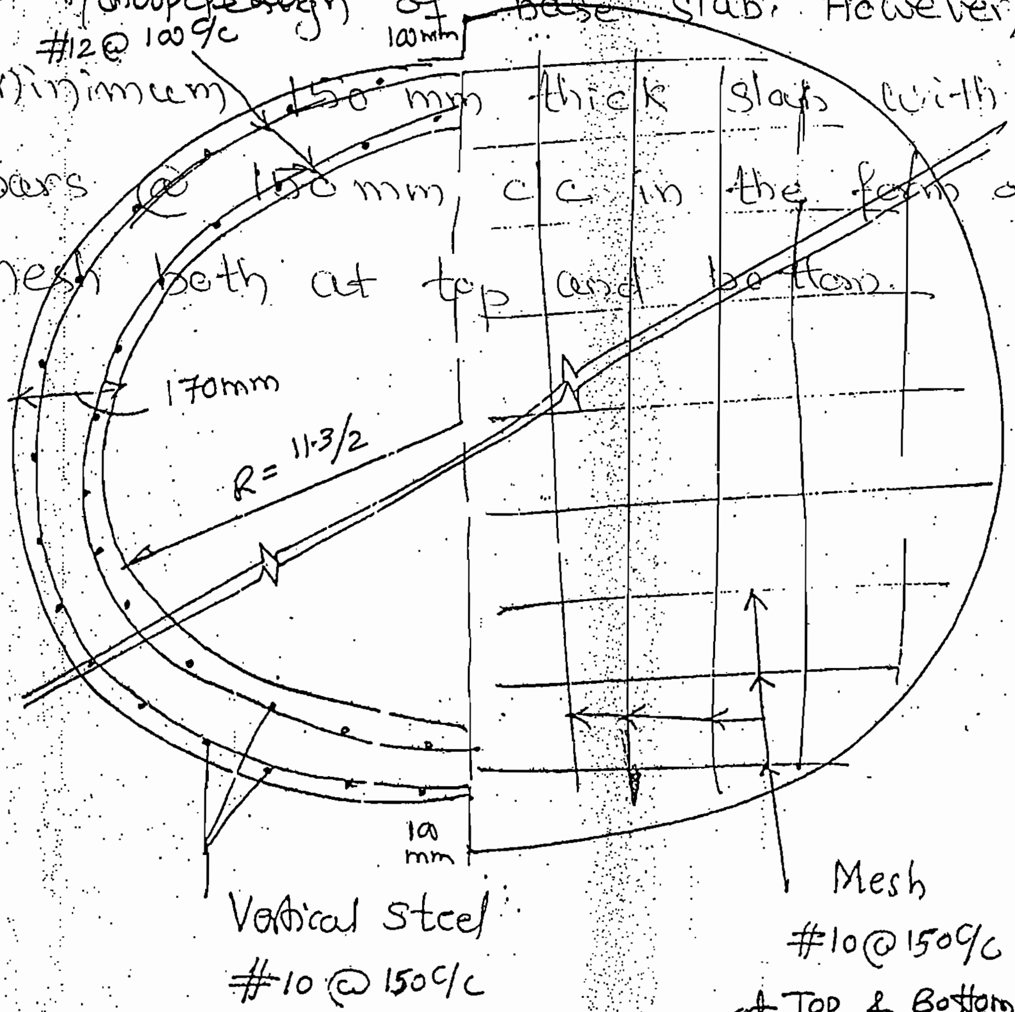
Provide 10mm ϕ bars.

Spacing of 10mm ϕ bars $= \dfrac{\frac{\pi}{4} * 10^2}{527.46} * 1000 = 148.9$mm

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 140 mm c/c.}}$

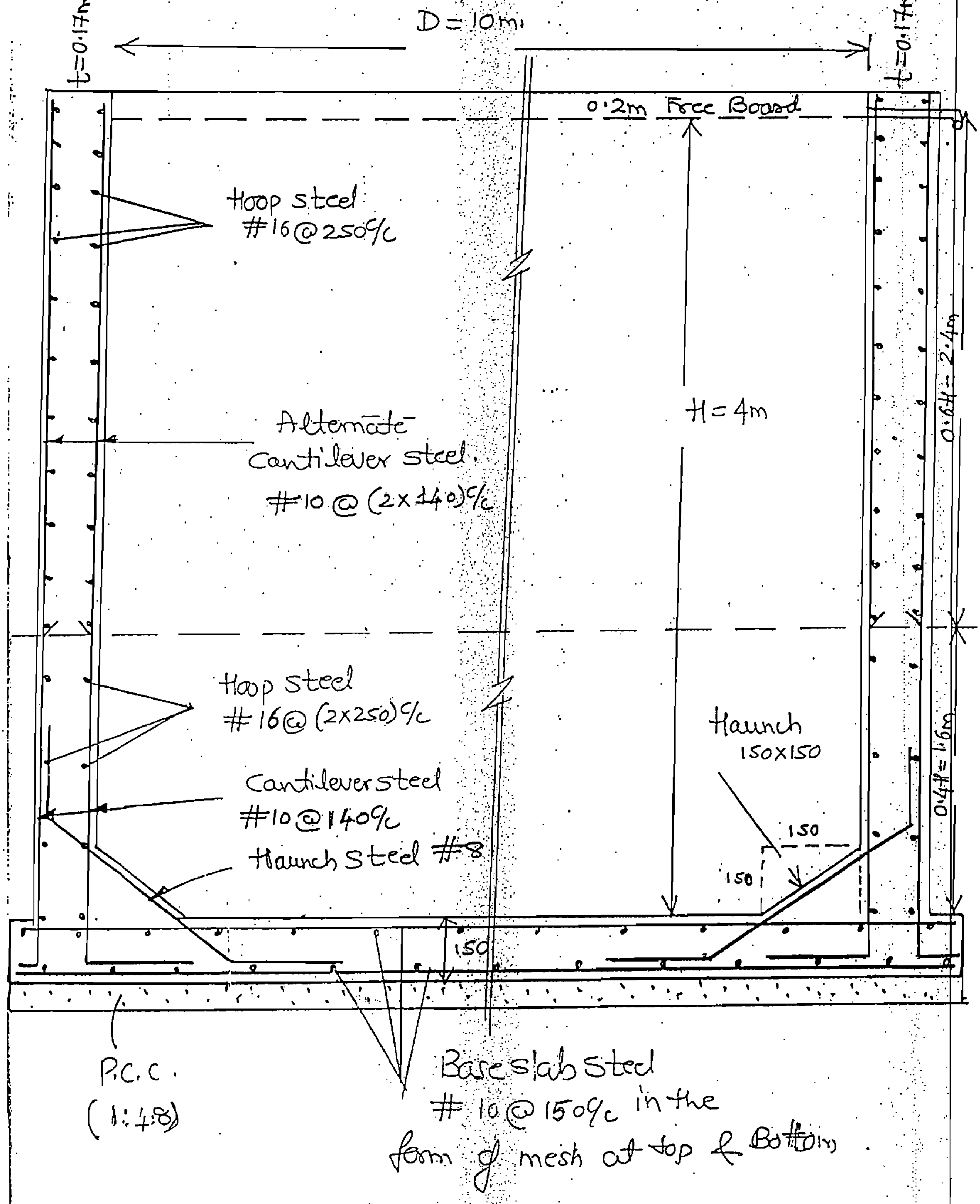
(iii) <u>Design of Base slab:</u>

* Since the slab is resting on ground, there is no design of base slab. However, provide minimum 150 mm thick slab with 10mm φ bars @ 150 mm c/c in the form of a mesh both at top and bottom.



Half Plan through the wall
&
Half Plan through the Base Slab.

* Drawing of Half plan same as previous problem!



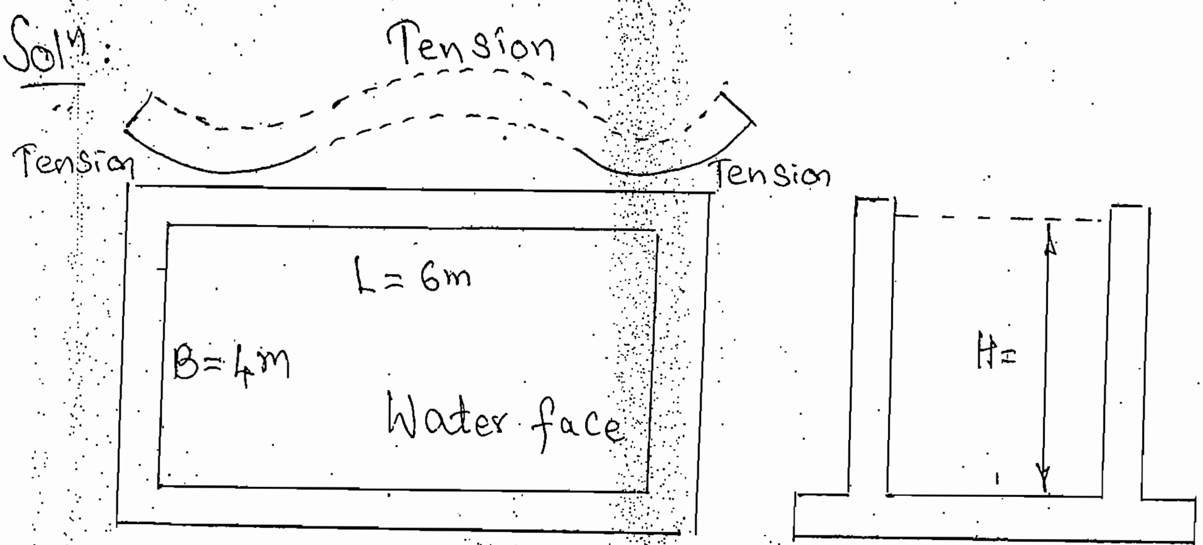
D = 10m
t = 0.17m
t = 0.17m
0.2m Free Board
Hoop steel
#16@250 c/c
Alternate
Cantilever steel
#10 @ (2x140) c/c
H = 4m
0.6H = 2.4m
Hoop steel
#16@ (2x250) c/c
Haunch
150X150
Cantilever steel
#10@140 c/c
Haunch steel #8
150
150
150
0.4H = 1.6m
P.C.C.
(1:4:8)
Base slab steel
10 @ 150 c/c in the
form of mesh at top & Bottom

MODULE # 1. RECTANGULAR WATER TANK

★ (ON GROUND)

Problem # 1. A rectangular water tank with an open top is required to store 80000 liters of water. The inside dimensions of the tank may be taken as 6m x 4m. Design the tank by IS-method using M25 Concrete and Fe 415 Steel.

Soln:



Step # 1: Constants:

$$M_{25} \longrightarrow \sigma_{cbc} = 8.50 \ N/mm^2$$

$$\sigma_{ct} = 1.30 \ N/mm^2$$

Fe 415 $\longrightarrow$ $\sigma_{st} = 150 \ N/mm^2$

Modular ratio $(m) = \dfrac{280}{3\sigma_{cbc}} = \dfrac{280}{3 * 8.50}$

$m = 10.98$

K = Stiffness factor $= \dfrac{m\,\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$

$K = \dfrac{10.98 * 8.50}{10.98 * 8.50 + 150} = 0.38$

$j = \left(1 - \dfrac{K}{3}\right) = \left(1 - \dfrac{0.38}{3}\right) = 0.87$

$Q = \dfrac{1}{2} * \sigma_{cbc} * K * j = \dfrac{1}{2} * 8.50 * 0.38 * 0.87$

$\boxed{Q = 1.405}$

$1 \ m^3 = 1000$ lit.

1 lit $= \left(\dfrac{1}{1000}\right) m^3$

Capacity of tank $= 80.000$ lit $= L * B * H_1$

$= \left(\dfrac{80.000}{\ }\right) m^3 = 6m * 4m * H_1$

$$H_1 = \frac{80}{24} = 3.33\,m$$

Using 0.17m Free board,

$$H = H_1 + 0.17 = 3.33 + 0.17 = 3.50\,m.$$

Step #2: Moment Calculations:

(i) Moment Calculation for long Wall:

a = height of Wall = 3.50 m

b = Width of Wall = 6.00 m

$$\text{Ratio} = \left(\frac{b}{a}\right) = \frac{6.00}{3.50} = \boxed{1.71}$$

★ Using IS 3370 - Part IV, (Table 3)

Page 16,-

	M_x	M_y
1.75	0.074	0.052
$\boxed{1.71}$	?	?
1.50	0.060	0.044

★ Select the largest Value from Table.

★ Ignore Sign ★

$$M_x = 0.074 + \left(\frac{0.060 - 0.074}{1.50 - 1.75}\right)(1.71 - 1.75)$$

$$\boxed{M_x = 0.072}$$

$$M_y = 0.052 + \left(\frac{0.044 - 0.052}{1.50 - 1.75}\right)(1.71 - 1.75)$$

$$\boxed{M_y = 0.051}$$

Horizontal Moment $= \boxed{M_y * \omega * a^3}$

ω = Specific weight of water = $10\ kN/m^3$

a = height of wall = H = 3.50 m.

Horizontal Moment $= 0.051 * 10 * (3.50)^3$

$\boxed{\text{Horizontal Moment} = 21.75\ kN\text{-}m.}$ ✓

Vertical Moment $= \boxed{M_x * \omega * a^3}$

Vertical Moment $= 0.072 * 10 * (3.50)^3$

$\boxed{\text{Vertical Moment} = 30.70\ kN\text{-}m.}$ ✓

(ii) Moment Calculation for short wall:

b = width of wall = 4.00m.

$$\text{Ratio} = \left(\frac{b}{a}\right) = \frac{4.00}{3.50} = \boxed{1.14}$$

* Using IS-3370 - Part IV, Table 3, Page 16,

	M_x	M_y
1.25	0.047	0.037
$\boxed{1.14}$	?	?
1.00	0.035	0.029

* Select the largest value from the Table.

* Ignore Sign *

$$M_x = 0.047 + \left(\frac{0.035 - 0.047}{1.00 - 1.25}\right)(1.14 - 1.25)$$

$$\boxed{M_x = 0.042}$$

$$M_y = 0.037 + \left(\frac{0.029 - 0.037}{1.00 - 1.25}\right)(1.14 - 1.25)$$

$$\boxed{M_y = 0.0335}$$

Horizontal Moment = $\boxed{M_y \cdot w \cdot a^3}$

$= 0.0335 * 10 * (3.50)^3$

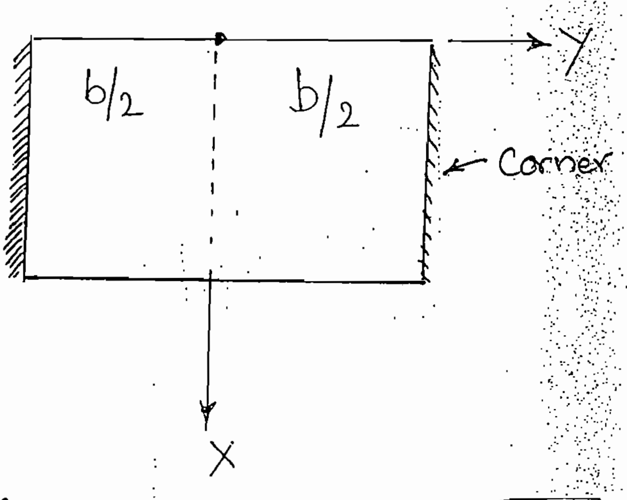
$\boxed{\text{Horizontal Moment} = 14.35 \text{ kN-m}}$ ✓

Vertical Moment = $\boxed{M_x \cdot w \cdot a^3}$

Vertical moment = $0.042 * 10 * (3.50)^3$

$\boxed{\text{Vertical Moment} = 18.00 \text{ kN-m}}$ ✓

Step # 3: Moment for long Wall Corner:



$\left(\frac{b}{a}\right) = \left(\frac{6}{3.50}\right) = \boxed{1.71}$ and $\boxed{y = \frac{b}{2}}$

★ Using I.S 3370 - Part IV Table (3)

Page 16,

	M_x	M_y
1.75	0.010	0.052
$\boxed{1.71}$	?	?
1.50	0.009	0.044

Select the largest Value from the table for $\boxed{y = \frac{b}{2}}$ Condition. Ignore Sign.

$$M_x = 0.010 + \left(\frac{0.009 - 0.01}{1.50 - 1.75}\right)(1.71 - 1.75)$$

$$\boxed{M_x = 0.0098 \simeq 0.01}$$

$$M_y = 0.044 + \left(\frac{0.044 - 0.052}{1.50 - 1.75}\right)(1.71 - 1.75)$$

$$\boxed{M_y = 0.043}$$

Horizontal Moment = $\boxed{M_y \cdot w \cdot (a)^3}$

$$= 0.043 * 10 * (3.50)^3$$

$\boxed{\text{Horizontal Moment} = 18.32 \text{ KN-m.}}$ ✓

Vertical Moment = $\boxed{M_x \cdot \omega a^3}$

$= 0.01 * 10 * (3.50)^3$

$\boxed{\text{Vertical Moment} = 4.29 \text{ kN-m.}}$

Step # 4: Tank wall thickness (D):

* Max BM from Step #2 and Step #3

After Comparison = 30.70 kN-m.

$Q = 1.405, \quad b = 1\,m = 1000\ mm.$

effective depth $\boxed{(d) = \sqrt{\dfrac{M}{Qb}}}$

$$d = \sqrt{\frac{30.70 * 10^6}{1.405 * 1000}} = 147.82\ mm.$$

Using 50 mm effective cover,

$D = d + 50 = 147.82 + 50 = 197.82\ mm$

$\boxed{\text{Provide } D = 200\,mm \ \& \ d = 150\,mm.}$

Step # 5: Pull in each Wall (T):

$$\text{Pull in long Wall} = \frac{\omega * H * B}{2}$$

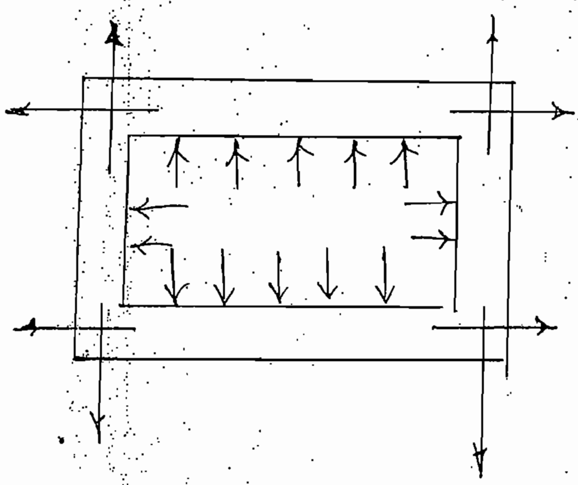
$$= \frac{10 * 3.50 * 4}{2}$$

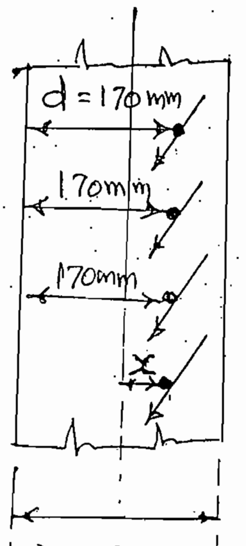
$$\boxed{\text{Pull in Long Wall} = 70\ kN}$$

$$\text{Pull in Short Wall} = \frac{\omega * H * L}{2}$$

$$= \frac{10 * 3.5 * 6}{2}$$

$$\boxed{\text{Pull in Short Wall} = 105\ kN}$$





$$x = \left(d - \frac{D}{2}\right) = \left(170 - \frac{200}{2}\right) = 70\ mm$$

Step # 6. Design of long wall:

(i) Hoop Steel or Horizontal steel:

$$\boxed{A_{st} = \frac{M - T\cdot x}{\sigma_{st}\cdot j\cdot d} + \frac{T}{\sigma_{st}}}$$

M = Horizontal Moment = 21.75 kN-m

T = Pull in Long wall = 70 kN.

x = 70 mm.

$$A_{st} = \frac{21.75 * 10^6 - 70 * 70 * 10^3}{150 * 0.87 * 170} + \frac{70 * 10^3}{150}$$

$$\boxed{A_{st} = 1226.20\ mm^2}$$

Provide 16mm ϕ bars.

$$\text{Spacing of 16mm } \phi \text{ bars} = \frac{\frac{\pi}{4} * (16)^2}{1226.20} * 1000$$

= 163.97

Say 160 mm c/c

(ii) Design of Vertical Steel:

M = Vertical Moment = 30.70 kN-m.

$$\boxed{A_{st} = \frac{M}{\sigma_{st} \cdot j \cdot d}} = \frac{30.70 * 10^6}{150 * 0.87 * 170}$$

$$\boxed{A_{st} = 1383.82 \text{ mm}^2}$$

Provide 12mm ϕ bars.

Spacing of 12mm ϕ bars $= \frac{\frac{\pi}{4} * (12)^2}{1383.82} * 1000$

$= 81.72$ Say 80 mm c/c.

$$\boxed{\text{Provide 12mm ϕ bars @ 80 mm c/c.}}$$

Step # 7 : Design of Short Wall:

(i) Hoop Steel or Horizontal Steel.

$$\boxed{A_{st} = \frac{M - T \cdot x}{\sigma_{st} \cdot j \cdot d} + \frac{T}{\sigma_{st}}}$$

M = Horizontal Moment = 14.35 KN-m

T = pull in short wall = 105 KN

x = 70 mm

$$A_{st} = \frac{14.35 * 10^6 - 105 * 10^3 * 70}{150 * 0.87 * 170} + \frac{105 * 10^3}{150}$$

$$\boxed{A_{st} = 1015.53 \text{ mm}^2.}$$

Provide 16mm ϕ main bars.

$$\text{Spacing of 16mm ϕ bars} = \frac{\frac{\pi}{4} * 16^2}{1015.53} * 1000$$

$= 197.98$ Say 190 mm c/c.

$\boxed{\text{Provide 16 mm ϕ bars @ 190 mm c/c}}$

(ii) Vertical Steel

M = Vertical moment = 18 KN-m.

$$A_{st} = \frac{M}{\ldots} = \frac{18 * 10^6}{150 * 0.87 * 170}$$

$= 811.36 \text{ mm}^2$

$$\boxed{A_{st} = 811.36 \text{ mm}^2}$$

Provide 12mm ϕ bars.

Spacing of 12mm ϕ bars $= \dfrac{\frac{\pi}{4} * 12^2}{811.36} * 1000$

$= 139.39$ Say 130 mm c/c.

$$\boxed{\text{Spacing of 12 mm } \phi \text{ bars @ 130 mm c/c}}$$

Step # 8: Corner design:

M = Max Moment @ Corner = 18.32 KN-m.

$$\boxed{A_{st} = \frac{M}{\sigma_{st} \cdot j \cdot d}} = \frac{18.32 * 10^6}{150 * 0.87 * 170}$$

$$\boxed{A_{st} = 825.78 \text{ mm}^2}$$

Provide 16mm ϕ bars.

Spacing of 16mm ϕ bars $= \dfrac{\frac{\pi}{4} * 16^2}{825.78} * 1000$

$= 243.48$ Say 240 mm c/c.

$\boxed{\text{Spacing of 16mm } \phi \text{ bars @ 240mm c/c}}$

Step # 9 : Design of Base slab:

Since the slab is continuously supported on ground, provide 150 mm thick base slab 0.3 % of the steel in the form of mesh at top and bottom shall be provided as reinforcement.

$$\text{Area of Steel} = \frac{0.3}{100} * \overset{b}{1000} * \overset{D}{150}$$

$$= 450 \text{ mm}^2.$$

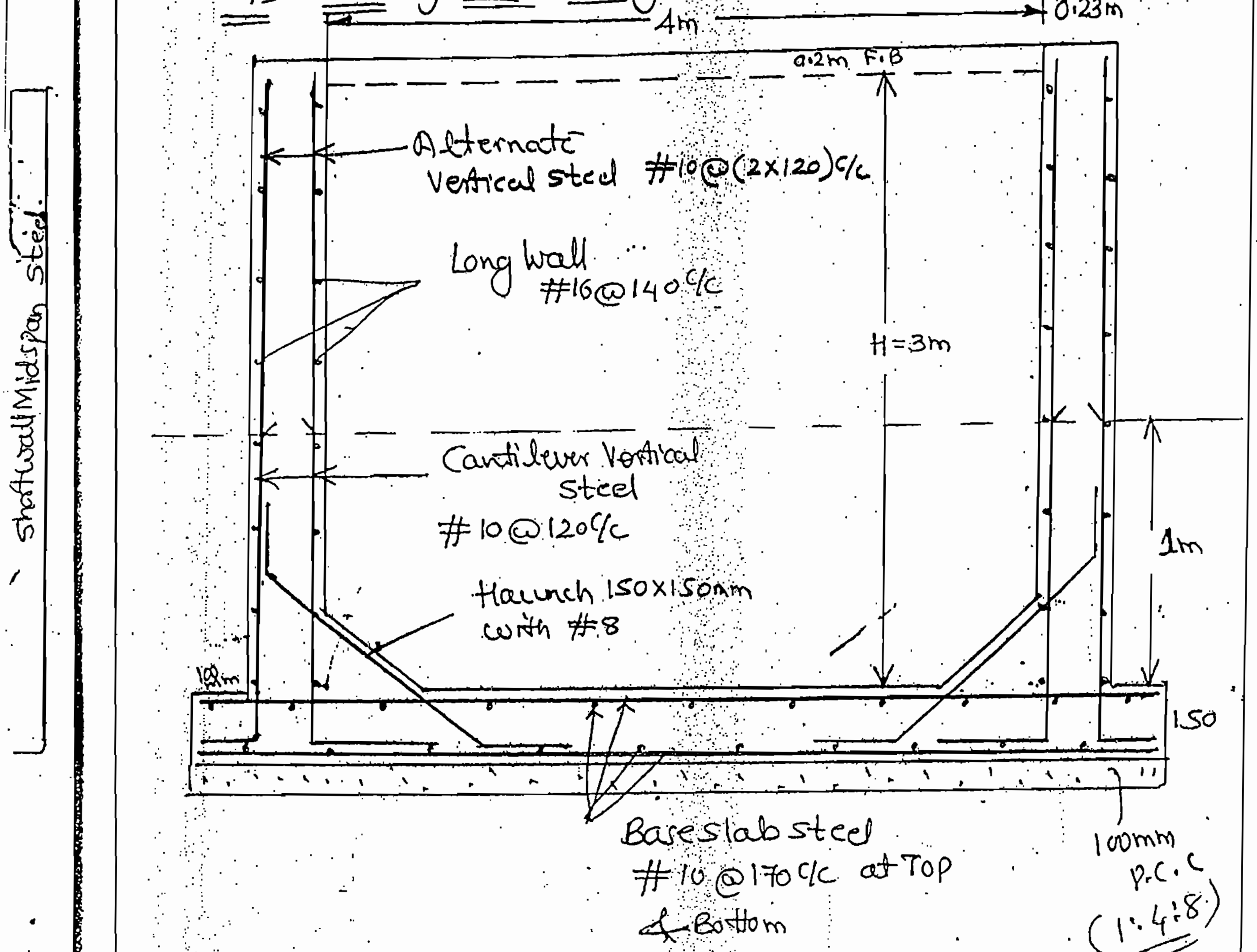
Provide 10mm ϕ bars.

$$\text{Spacing of 10mm } \phi \text{ bars} = \frac{\frac{\pi}{4} * 10^2}{450} * 1000$$

$$= 174.5 \text{ Say } 170 \text{mm c/c}$$

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 170mm c/c}}$

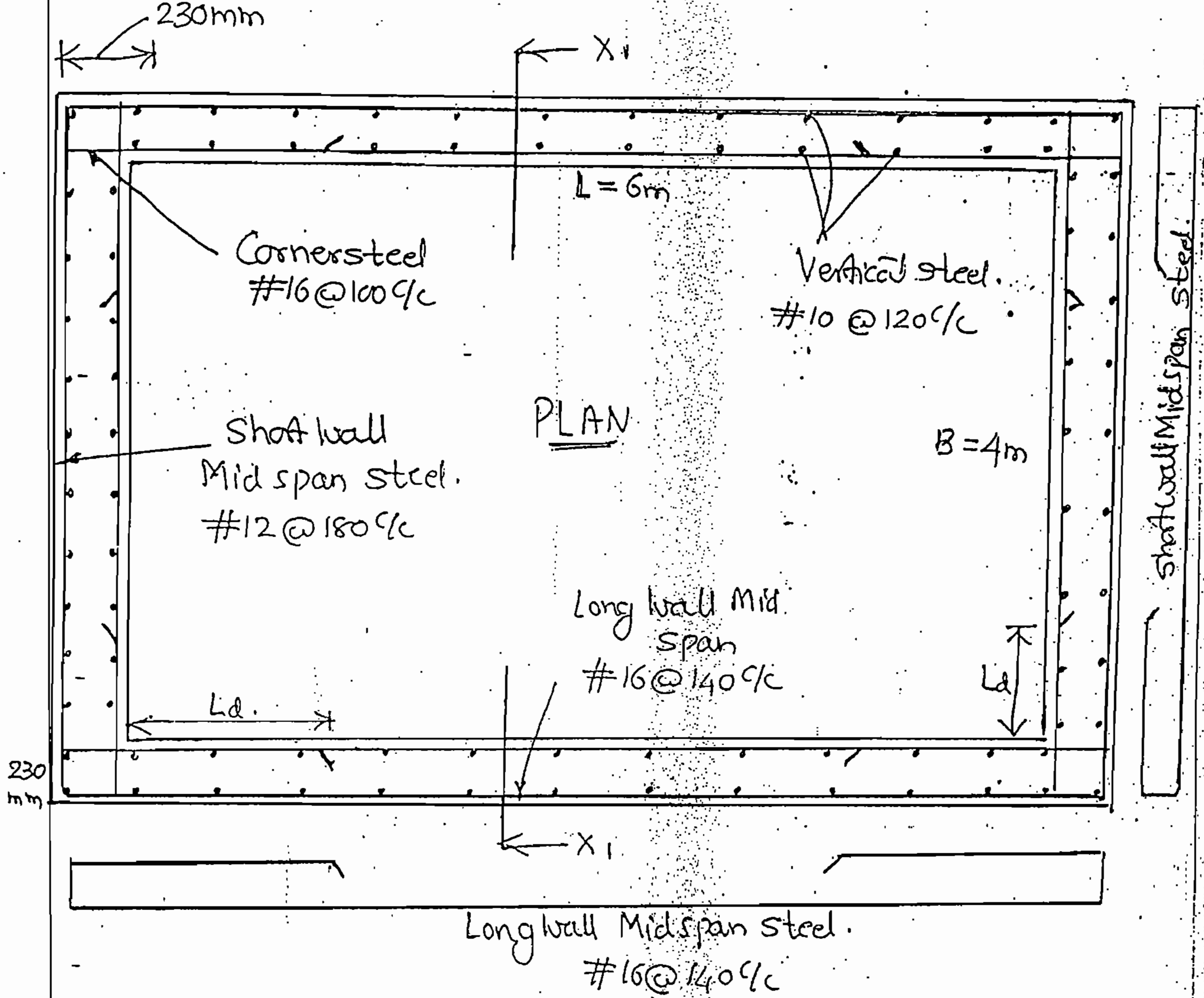
C/s Through the Long Wall



Allround Clear cover = 30mm

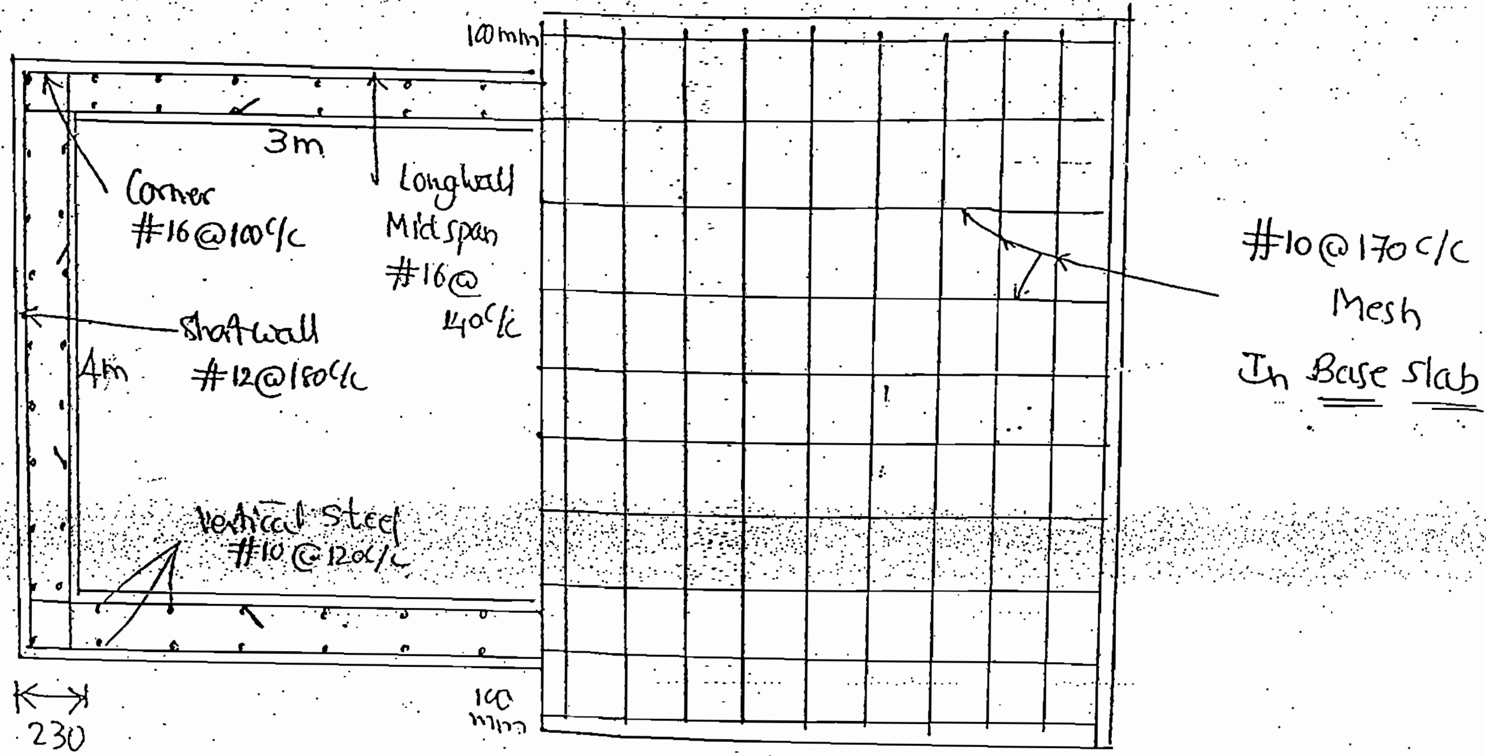
43

1 : 25



230mm
X
L = 6m
Cornersteel
#16@100 c/c
Vertical steel.
#10 @120 c/c
Short wall
Mid span steel.
#12@180 c/c
PLAN
B = 4m
Short wall Midspan steel.
Long wall Mid
span
#16@140 c/c
Ld.
Ld
230
mm
X1
Long wall Midspan steel.
#16@140 c/c

Half Plan Through the Wall and Half Plan Through Base Slab



1:2

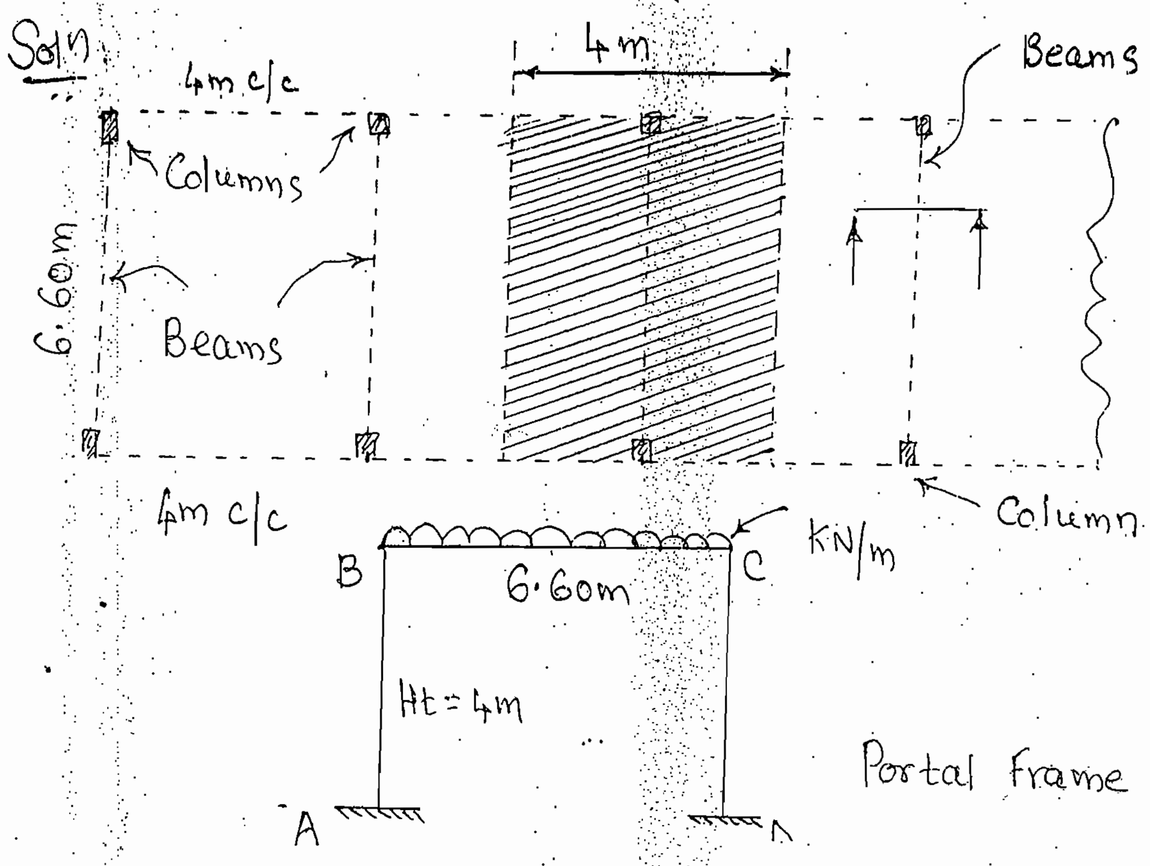
r = 2.98m

MODULE # 1.

PORTAL FRAMES (FIXED BASE)

Problem #1. A Single bay fixed portal frame has an effective Span of 6.60m and an effective height of 4m. The Spacing of portal frames is 4m c/c. Considering a live load of 1.50 kN/m² on the slab, design the Continuous slab and the portal frame using M_{20} grade Concrete and Fe 415 Steel. Take SBC of Soil as 120 kN/m².

Soln:



Step # 1: Design of one way Continuous slab

* Use $\boxed{\frac{l}{d} = (26 * M.F)}$ MF = Modification factor for steel

(MF = 1.25 assume)

$\therefore$ l = Spacing of portal frames = 4m c/c

$$\therefore \frac{4000}{d} = 26 * 1.25$$

$$\Rightarrow d = \frac{4000}{26 * 1.25} = 123.08 \text{ Say } 125 \text{ mm}$$

Using 10 mm ϕ bars with clear cover 20 mm,

$$D = d + \frac{\phi}{2} + \text{clear cover}$$

$$D = 125 + \frac{10}{2} + 20 = 150 \text{ mm.}$$

$\boxed{\text{Provide } D = 150\text{mm and } d = 125\text{mm.}}$

(i) Load Calculations:

Dead load $= \gamma_{conc} * D = 25 * 0.15 = 3.75$ kN/m

Floor finishes $\longrightarrow$ = 0.50 kN/

Factored Dead load $= (\omega_g) = 1.5\,\omega$

$$\omega_g = 1.5 * 4.25$$

$$\boxed{\omega_g = 6.38 \text{ kN/m}^2.}$$

Factored Live load $= (\omega_q) = 1.5\,\omega_L$

$$= 1.5 * 1.50$$

$$\boxed{\omega_q = 2.25 \text{ kN/m}^2.}$$

(ii) Bending Moment:

As per IS 456 – 2000, Page 36, Table (12)

(a) Span moment : (+ve Moment)

$$\boxed{M = \frac{\omega_g l^2}{12} + \frac{\omega_q l^2}{10}} = \frac{6.38 * 4^2}{12} + \frac{2.25 * 4^2}{10}$$

$$\boxed{M = 12.10 \text{ kN-m.}}$$

(b) Support moment : (– ve Moment)

$$M = -\frac{\omega_g l^2}{10} - \frac{\omega_q l^2}{9} = -\frac{6.38 * 4^2}{10} - \frac{2.25 * 4^2}{9}$$

$$= \boxed{-14.21 \ kN\text{-}m.}$$

$\therefore$ Absolute Max BM = 14.21 kN-m.

(iii) <u>check for depth:</u>

$$\bigstar \ (M_u)_{lim} = 0.36 \frac{x_{u\,max}}{d}\left[1 - \left(\frac{x_{u\,max}}{d}\right)0.42\right] bd^2 f_{ck}$$

$b = 1m = 1000mm$, $\frac{x_{u\,max}}{d} = 0.48$

fe 415 for balanced section.

$$14.21 * 10^6 = 0.36 * 0.48 \left[1 - 0.48 * 0.42\right] * 1000 * d^2 * 20$$

$$\therefore \ d^2 = \frac{14.21 * 10^6}{0.36 * 0.48 * (0.798) * 1000 * 20} = 5149.91$$

$$d = \sqrt{5149.91} = 71.76 \ mm < 125 \ mm \ (ok)$$

(IV) <u>Area of Steel:</u>

(a) <u>Mid Span Steel: (+ve Steel)</u>

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

As per IS 456-2000, clause G-1.1.

$$\therefore (M_u) = 12.10 * 10^6 = 0.87 * 415 * A_{st} * 125 \left[1 - \frac{A_{st} * 415}{1000 * 125 * 2}\right]$$

$$12.1 * 10^6 = A_{st} \cdot 45.13 * 10^3 \left[1 - 166 * 10^{-6} A_{st}\right]$$

$$\therefore 268.11 = A_{st} - 166 * 10^{-6} (A_{st})^2$$

$$\Rightarrow (A_{st})^2 - 6.024 * 10^{+3} A_{st} + 1.62 * 10^6 = 0.$$

Solving, $\boxed{A_{st} = 282.14 \text{ mm}^2}$

Provide 10mm ϕ bars.

Spacing of 10mm ϕ bars $= \dfrac{\frac{\pi}{4} * 10^2 * 1000}{282.14}$

$= 278.40$ mm c/c. Say 270 mm c/c

$\boxed{\text{Spacing of 10mm } \phi \text{ @ 270 mm c/c}}$ < 3d or 300mm

(b) Support Steel : (-ve Steel):

$$M_u = 14.21 * 10^6 = 0.87 * 415 * A_{st} * 125 \left[1 - \frac{A_{st} * 415}{1000 * 125 * 20}\right]$$

$$14.21 * 10^6 = 45.13 * 10^3 A_{st} \left[1 - 166 * 10^{-6} A_{st}\right]$$

$$\therefore 314.86 = A_{st} - 166 * 10^{-6} (A_{st})^2$$

$$\therefore 166 * 10^{-6} (A_{st})^2 - A_{st} + 314.86 = 0.$$

Solving, $\boxed{A_{st} = 333.30 \text{ mm}^2}$

* provide 10mm ϕ bars.

$$\text{Spacing of 10mm } \phi \text{ bars} = \frac{\frac{\pi}{4} * 100}{333.30} * 1000$$

$$= 235.64 \text{ Say } 230 \text{ mm c/c}.$$

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 230 mm c/c}}$ < 3d or 300 mm

(c) <u>Distribution Steel</u>

$$\text{Area} = \frac{0.12}{100} * \overset{b}{1000} * \overset{D}{150} = \boxed{180 \text{ mm}^2}$$

Provide 8mm ϕ bars.

$$\text{Spacing of 8mm } \phi \text{ bars} = \frac{\frac{\pi}{4} * 8^2}{18\ldots} * 1000 = 279 \text{ m}$$

$\boxed{\text{Spacing of 8mm } \phi \text{ bars @ 270mm c/c}}$ < 5d or 450 mm

Step # 2 : Analysis of portal frame:

★ Beam : Assume width b = 200 or 250 or 300 mm

Take $\boxed{b = 250 \text{ mm}}$

Depth $\boxed{(D) = \frac{\text{Span}}{12}} = \frac{6600}{12} = 550$ mm.

Take $\boxed{250 * 550 \text{ mm}}$

★ Column : Column width = beam width = 250 mm

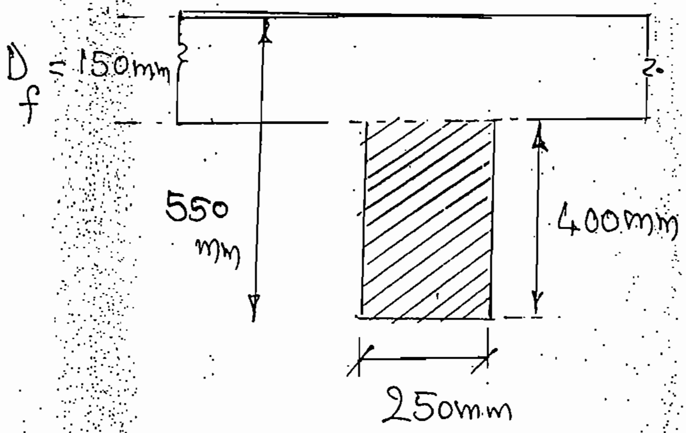
Depth : 1.50 to 2 times width

Depth = 1.50 * 250 = 375 mm or 2 * 250 = 500 mm

Take $\boxed{250 \text{ mm} * 550 \text{ mm}}$

$$I_{beam} = I_{Column} = \frac{bD^3}{12} = \frac{250 * 550^3}{12}$$

$$\boxed{I_{beam} = I_{Column} = I = 3.47 * 10^9 \text{ mm}^4}$$



Total load on Slab $= \omega_g + \omega_q = (6.38 + 2.25)\ kN/m^2$

$= 8.63\ kN/m^2$

UDL on beam from Slab $= \left\{\begin{array}{c}\text{Total load} \\ \text{on Slab}\end{array}\right\} *$ Beam Spac

$= 8.63 * 4 = 34.52\ kN/m$ --- I

Self Weight of beam $= (0.25 * 0.40) * 25$

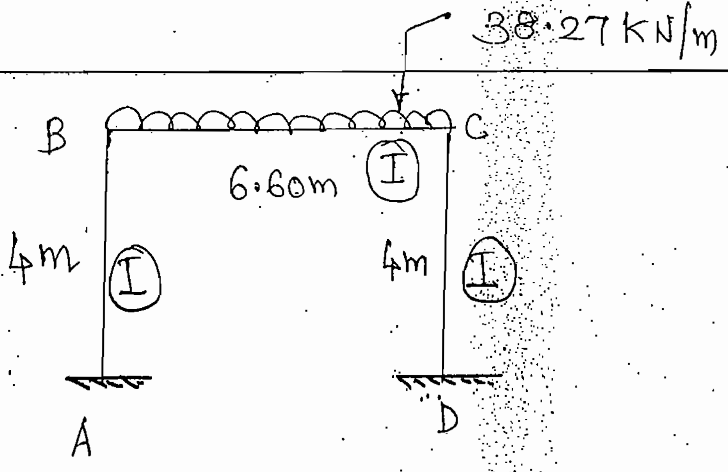
$= 2.5\ kN/m.$

Factored Self Wt on beam $= 2.50 * 1.5$

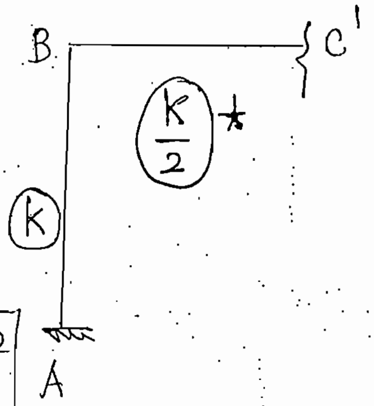
$= 3.75\ kN/m$ — II

Total udl $=$ I $+$ II $= 34.52 + 3.75$

$= \boxed{38.27\ kN/m}$



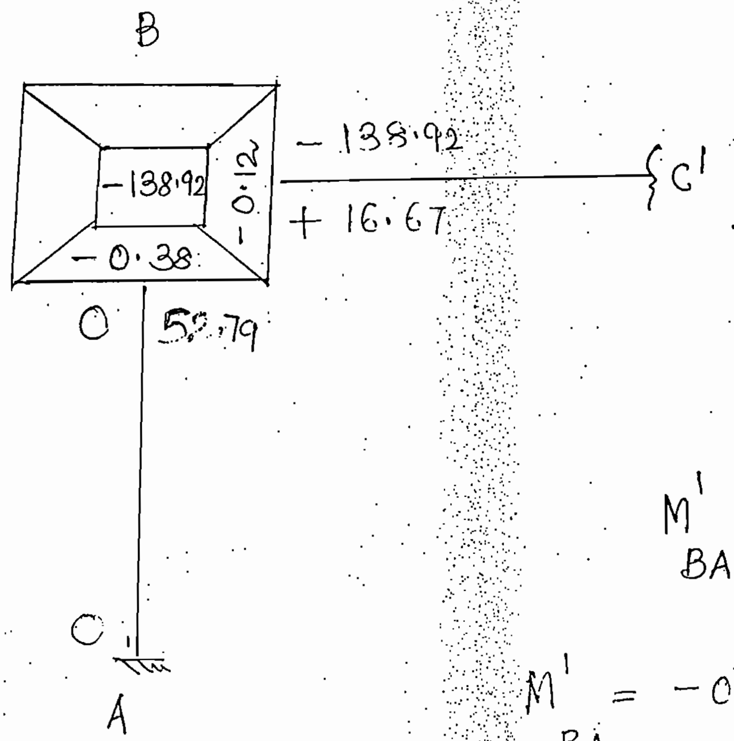
Using Kani's method :



$$M_{FBC} = -\frac{\omega l^2}{12}$$

$$M_{FBC} = -\frac{38.27 * 6.6^2}{12} = \boxed{-138.92 \text{ KN-m}}$$

Joint	Member	K	ΣK	$U = \left(\frac{-1}{2}\right)\frac{K}{\Sigma K}$
B	BA	$K = \frac{I}{l} = \frac{I}{4} = 0.25\,I$	0.33I	− 0.38
	BC'	$\frac{K}{2} = \frac{1}{2}\left(\frac{I}{l}\right) = \frac{I}{2*6.6}$ $*\,l = 6.6\text{m}$ 0.08I		− 0.12



$$M'_{BA} = U\left[\sum M_F + \sum M'_{AB}\right]$$

$$M'_{BA} = -0.38\left[-138.92 + 0\right]$$

$$= 52.79 \text{ kN-m}$$

$$M'_{BC} = -0.12\left[-138.92 + 0\right]$$

$$= 16.67 \text{ kN-m}$$

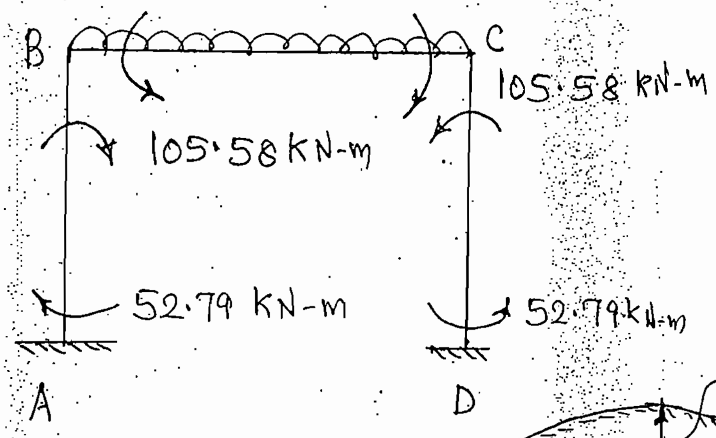
$$\boxed{\star \text{ final Moment : } M = M_f + 2\,(\text{Near}) + 1\,(\text{Far})}$$

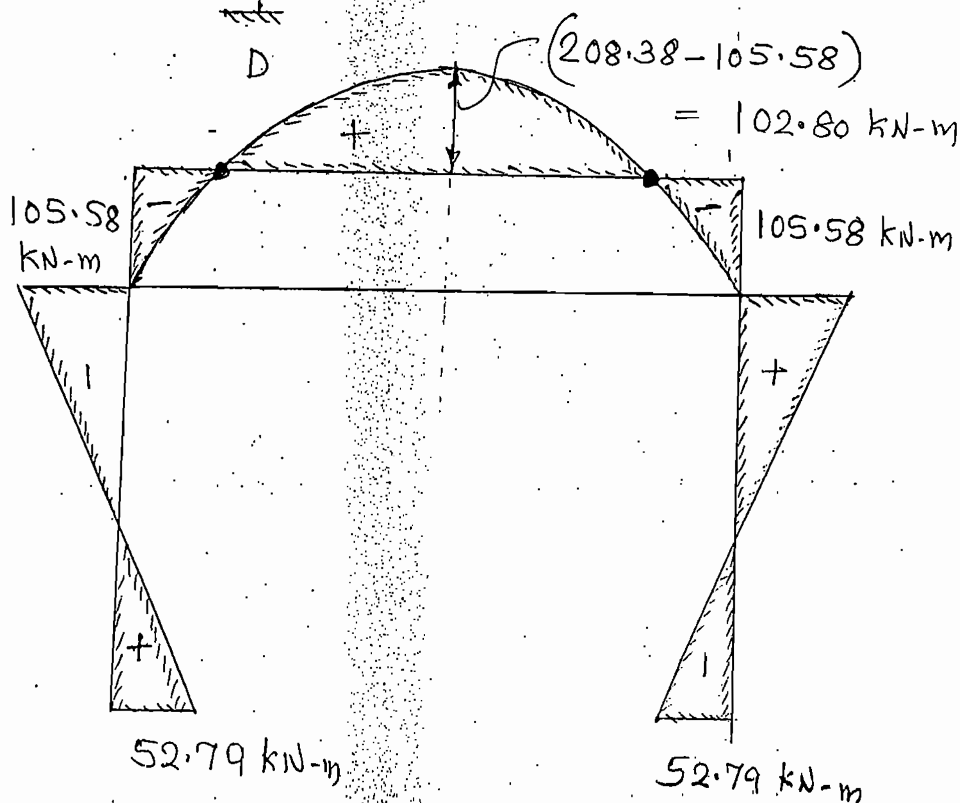
$$M_{AB} = 0 + 2(0) + 1(52.79) = 52.79 \text{ kN-m}$$

$$M_{BA} = 0 + 2(52.79) + 1(0) = 105.58 \text{ kN-m}$$

$$M_{BC} = -138.92 + 2(16.67) + 1(0) = -105.58 \text{ kN-m}$$

Max BM for BC $= \dfrac{\omega l^2}{8} = \dfrac{38.27 * 6.6^2}{8} = 208.38$ kN-m





Final design Values:

Beam : 250mm * 550 mm, Span = 6.60 m.

Load : 38.27 kN/m.

$\therefore$ Reaction = S.F. = $V_u = \dfrac{wl}{2} = \dfrac{38.27 * 6.60}{2} = 126.29 kN$

$(M_u)_{Support} = 105.58\ kN\text{-}m$, $(M_u)_{mid} = 102.80\ kN\text{-}m$.

Column: 250 * 550 mm, ht = 4 m.

Load P_u = Beam Reaction = 126.29 kN.

M_u = Support moment = 105.58 kN-m

Footing: SBC = 120 kN/m^2

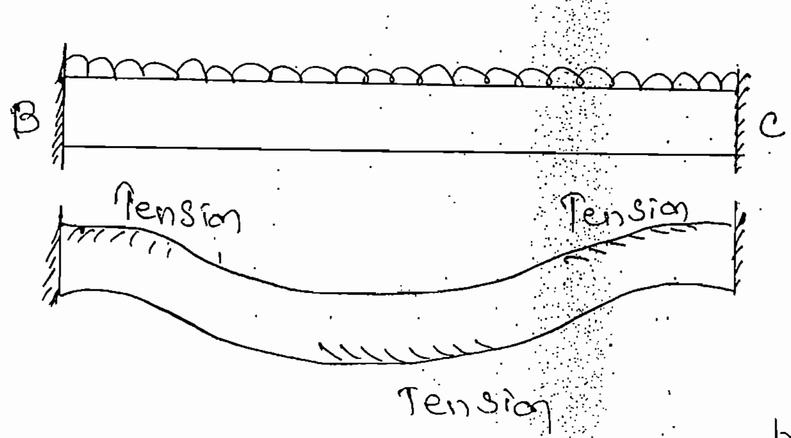
Load on Footing = Load on Column + factored Self wt of Col.

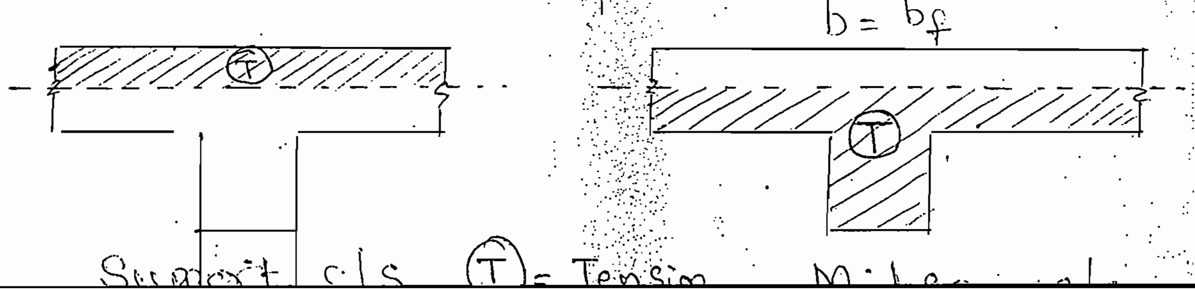
$= 126.29 + 1.50\,(0.25 * 0.55 * \overset{Ht}{4} * 25)$

$= 146.92$ kN

Moment = 52.79 kN-m

Step # 3: Design of beam:





Support c/s (T) = Tension M

* Neglecting Tension Zone,

** Support c/s is designed like a Rectangular beam taking $\boxed{b = 250\text{ mm}}$.

** Mid Span c/s is designed like a T-beam taking $\boxed{b = b_f}$.

$$\therefore \boxed{b_f = \frac{l_o}{6} + 6D_f + b_w}$$ Pg 37 IS 456-2000

$$\boxed{l_o = 0.7l}$$ → Note given in Pg 37 IS 456-2000.

$$l_o = 0.7 * 6.60 = 4.62\text{m} = 4620\text{ mm.}$$

$$b_f = \frac{4620}{6} + 6 * 150 + 250 = 1920\text{mm.}$$

A Design of Support c/s:

$M_u = 105.58$ KN-m, $b = 250$ mm, $D = 550$mm

Assuming an effective Cover of 50mm,

$$\therefore d = (550 - 50) = 500\text{ Mm.}$$

As per IS 456-2000, clause G-1.1;

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

$$105.58 * 10^6 = 0.87 * 415 * A_{st} * 500 \left[1 - \frac{A_{st} * 415}{250 * 500 * 20} \right]$$

$$105.58 * 10^6 = 180.53 * 10^3 A_{st} \left[1 - 166 * 10^{-6} A_{st} \right]$$

$$584.83 = A_{st} - 166 * 10^{-6} (A_{st})^2$$

$$A_{st}^2 - 6024.10 A_{st} \pm 3.52 * 10^6 = 0.$$

Solving, $A_{st} = \boxed{655.70 \text{ mm}^2}$

Provide 16mm ϕ bars.

$$\text{No of bars} = \frac{655.70}{\frac{\pi}{4} * 16^2} = 3.26 \simeq 4.$$

$\boxed{\text{Provide 4 bars of 16mm } \phi.}$

(B) Design of Mid. Span c/s: $\boxed{b = b_f}$

$$(M_u) = 102.80 * 10^6 = 0.87 * 415 * A_{st} * 500 \left[1 - \frac{A_{st} * 415}{\ldots} \right]$$

$$102.80 * 10^6 = 180.53 * 10^3 A_{st} \left[1 - 21.62 * 10^{-6} A_{st}\right]$$

$$\therefore 569.43 = A_{st} - 21.62 * 10^{-6} (A_{st})^2$$

$$21.62 * 10^{-6} (A_{st})^2 - A_{st} + 569.43 = 0.$$

Solving, $A_{st} = \boxed{576.62 \text{ mm}^2}$

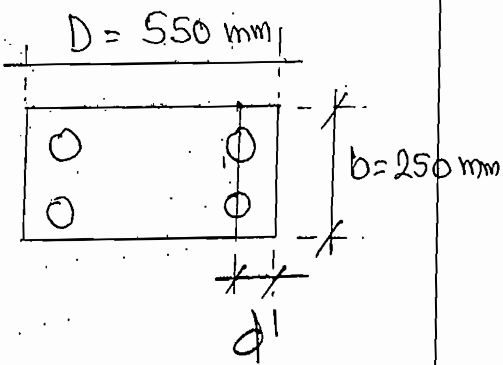
Provide 16mm ϕ bars.

$$\text{No of bars} = \frac{576.62}{\frac{\pi}{4} * 16^2} = 2.86 \simeq 3.$$

$\boxed{\text{Provide 16 mm } \phi \text{ bars of 3 No's.}}$

Step#4:

(c) Design of Column:



$P_u = 126.29$ kN

$M_u = 105.58$ kN,

Assume d' = effective cover = 50 mm

(i) Reinforcement on all 4 sides

(ii) Grade of Steel → Fe 415

(iii) Ratio $\left(\frac{d'}{D}\right) = \left(\frac{50}{550}\right) = 0.09$

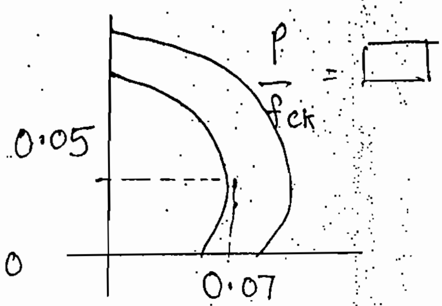
Using chart 44, Pg 128, Sp-16,

$$\frac{P_u}{f_{ck} b D} = \frac{126.29 * 10^3}{20 * 250 * 550} = \boxed{0.046}$$

$$\frac{M_u}{f_{ck} b D^2} = \frac{105.58 * 10^6}{20 * 250 * 550^2} = \boxed{0.070}$$

Using interaction Curves of Sp-16, chart 44,

$$\frac{p}{f_{ck}} = 0.05 \text{ from chart.}$$



$\therefore$ $p = 0.05 * f_{ck} = 0.05 * 20 = 1.0$

p = % of Steel reinforcement.

$$A_{st} = \left(\frac{p}{100}\right) * bD = \frac{1}{100} * 250 * 550 = 1375 \text{ mm}^2.$$

Provide 16 mm ϕ bars.

No of bars $= \dfrac{1375}{\frac{\pi}{4} * 16^2} = 6.83 \simeq 7 \simeq 8$

Design of lateral ties:

Provide 8mm ϕ lateral ties.

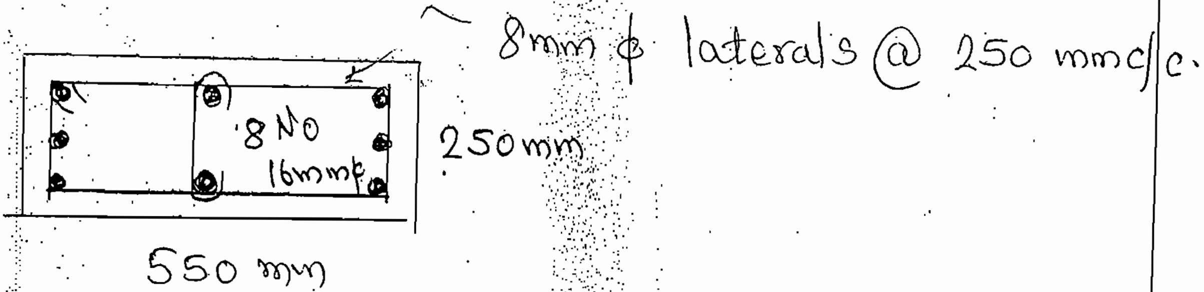
Spacing of 8mm ϕ lateral ties must be the least of the following:

(i) $b = 250$ mm

(ii) $16\phi = 16 * 16 = 256$ mm

(iii) 300 mm.

$\boxed{\text{Provide 8mm } \phi \text{ lateral ties @ 250 mm c/c}}$



Step 5 : Shear design:

Nominal Shear Stress $= \tau_v = \dfrac{V_u}{bd}$

$$\tau_v = \frac{126.29 * 10^3}{250 * 500} = 1.01 \ N/mm^2$$

$\because$ 4 bars 16mmϕ

$$\frac{100 A_{st}}{bd} = \frac{100 * \left(4 * \frac{\pi}{4} * 16^2\right)}{250 * 500} = \boxed{0.643} \ \& \ M_{20}$$

By Table # 19, Pg 73, of IS 456 - 2000,

$\frac{100 A_{st}}{bd}$		M 20
0.50	---	0.48
0.643	---	?
0.75	---	0.56

$$\tau_c = 0.48 + \left(\frac{0.56 - 0.48}{0.75 - 0.50}\right)(0.643 - 0.50)$$

$$\tau_c = 0.525 \ N/mm^2 < \tau_v.$$

Provide Shear reinforcement.

Provide 8mm ϕ 2 legged stirrups.

$$\therefore A_{sv} = 2 * \left[\frac{\pi}{4} * 8^2\right] = 100.53 mm^2$$

By IS 456 - 2000, Pg 73,

$$\boxed{V_{us} = \left(V_u - \tau_c \cdot bd\right) = \frac{0.87 * f_y * A_{sv} * d}{S_v}}$$

$$\therefore \left(126.29 * 10^3 - 0.525 * 250 * 500\right) = \frac{0.87 * 415 * 100.53 * 500}{S_v}$$

$$60.665 * 10^3 = \frac{18.15 * 10^6}{S_v}$$

$$\therefore S_v = \frac{18.15 * 10^6}{60.665 * 10^3} = 299.20 \text{ Say } 290\text{mm } c/c.$$

$\boxed{\text{Provide 8mm } \phi \text{ 2 legged Stirrups @ 290mm c/c}}$

Step # 6: Design of Footing:

SBC = 120 KN/m^2.

P_u = load on Footing = 146.92 KN

M_u = Moment on Footing = 52.79 KN-m.

Working Value of P_u = $P = \dfrac{P_u}{1.5} = \dfrac{146.92}{1.5} = 97.95$ KN

Working Value of $M_u = \frac{M_u}{1.50 \text{ (FOS)}} = \frac{52.79}{1.50} = 35.20 \text{ kN} = M$

(a) Size of Footing :

Column load + Self wt @ 10 % (10 % of load)

$= 97.95 + 9.795 = 107.75 \text{ kN} = P'$

Assume width of footing B as 1.20m or 1.50m or 1.80m or 2.00m.

Assume width $\boxed{B = 1.50 m}$

$$\boxed{\frac{P'}{B * L} + \frac{6M}{B * L^2} = SBC.}$$

$$\therefore \frac{107.75}{1.50 L} + \frac{6 * 35.20}{1.50 L^2} = 120$$

$$\frac{107.75 L + 6 * 35.20}{1.50 L^2} = 120$$

$$\therefore 180 L^2 - 107.75 L - 211.20 = 0$$

Solving, $L = 1.423 m.$

* Increase the Value by 50 %.

$\therefore L = 1.50 * 1.42 = 2.13$ Say 2.20 m.

Try $L * B = 2.20\,m * 1.50\,m$. By Trial & Error. *

$$\boxed{f = \frac{P'}{B * L} \pm \frac{6M}{BL^2} \geq 0.}$$

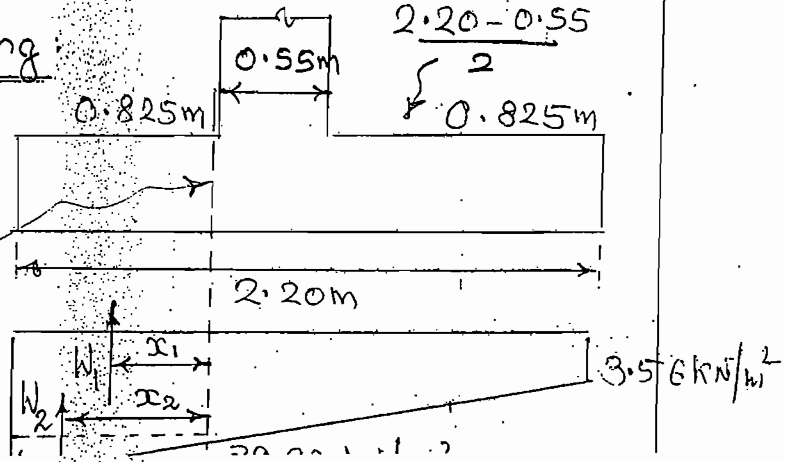
$$f = \frac{107.75}{(2.20 * 1.50)} \pm \frac{6 * 35.20 *}{(2.20)^2 * (1.50)}$$

$$f = (32.65 \pm 29.09)\ kN/m^2.$$

$$\therefore f_{max} = 32.65 + 29.09 = 61.74\ kN/m^2$$

$$f_{min} = 32.65 - 29.09 = 3.56\ kN/m^2$$

(b) <u>Thickness of footing</u>:



Pressure Intensity from one end:

0 m --- 61.74 kN/m²

0.825 m --- ?

2.20 m --- 3.56 kN/m²

$$= 61.74 + \left(\frac{3.56 - 61.74}{2.20 - 0}\right)(0.825 - 0)$$

$$= 39.92 \text{ kN/m}^2$$

$$M = (0.825 * 39.92) * \frac{0.825}{2} + \frac{1}{\cancel{2}} * 0.825 * (61.74 - 39.92) * \frac{\cancel{2}}{3} * 0.825$$

$$M = 13.585 + 4.95 = 18.535 \text{ kN-m.}$$

factored moment $= 1.5 M = 1.5 * 18.535$

$$M_u = 27.80 \text{ kN-m.}$$

Using $\left(M_u\right)_{lim} = 0.36 * \frac{x_{u\,max}}{d}\left[1 - 0.42\,\frac{x_{u\,max}}{d}\right] b d^2 f_{ck}$

$$\frac{x_{u\,max}}{d} = 0.48 \text{ for balanced Section,}$$

$b = 1000\,mm$

$$27.80 * 10^6 = 0.36 * 0.48 \left(1 - 0.42 * 0.48\right) * 1000 * d^2 * 20$$

$$27.80 * 10^6 = 2.76 * 10^3\, d^2$$

$$d = \sqrt{\frac{27.80 * 10^6}{2.76 * 10^3}} = 100.36\ mm.$$

Using 60mm effective Cover,

$$D = d + 60 = 100.36 + 60 = 160.36\ mm$$

* From Shear Consideration, double the above Value.

$$\therefore D = 160.36 * 2 = 320.72 \text{ Say } 350\ mm.$$

$$\boxed{\text{Provide } D = 350\ mm \ \& \ d = 290\ mm}$$

(c) Area of Steel (A_{st}):

As per IS 456 - 2000, Clause G-1.1,

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

$b = 1000\,mm, \quad d = 290\ mm$

$$27.80 * 10^6 = 0.87 * 415 * A_{st} * 290 \left[1 - \frac{A_{st} * 415}{1000 * 290 * 20} \right]$$

$$27.80 * 10^6 = 104.70 * 10^3 A_{st} \left[1 - 71.55 * 10^{-6} A_{st} \right]$$

$$\therefore 265.52 = A_{st} - 71.55 * 10^{-6} (A_{st})^2$$

$$\therefore (A_{st})^2 - 13976.24 A_{st} + 3.711 * 10^6 = 0$$

Solving, $\boxed{A_{st} = 270.77 \text{ mm}^2}$

Provide 10mm ϕ bars.

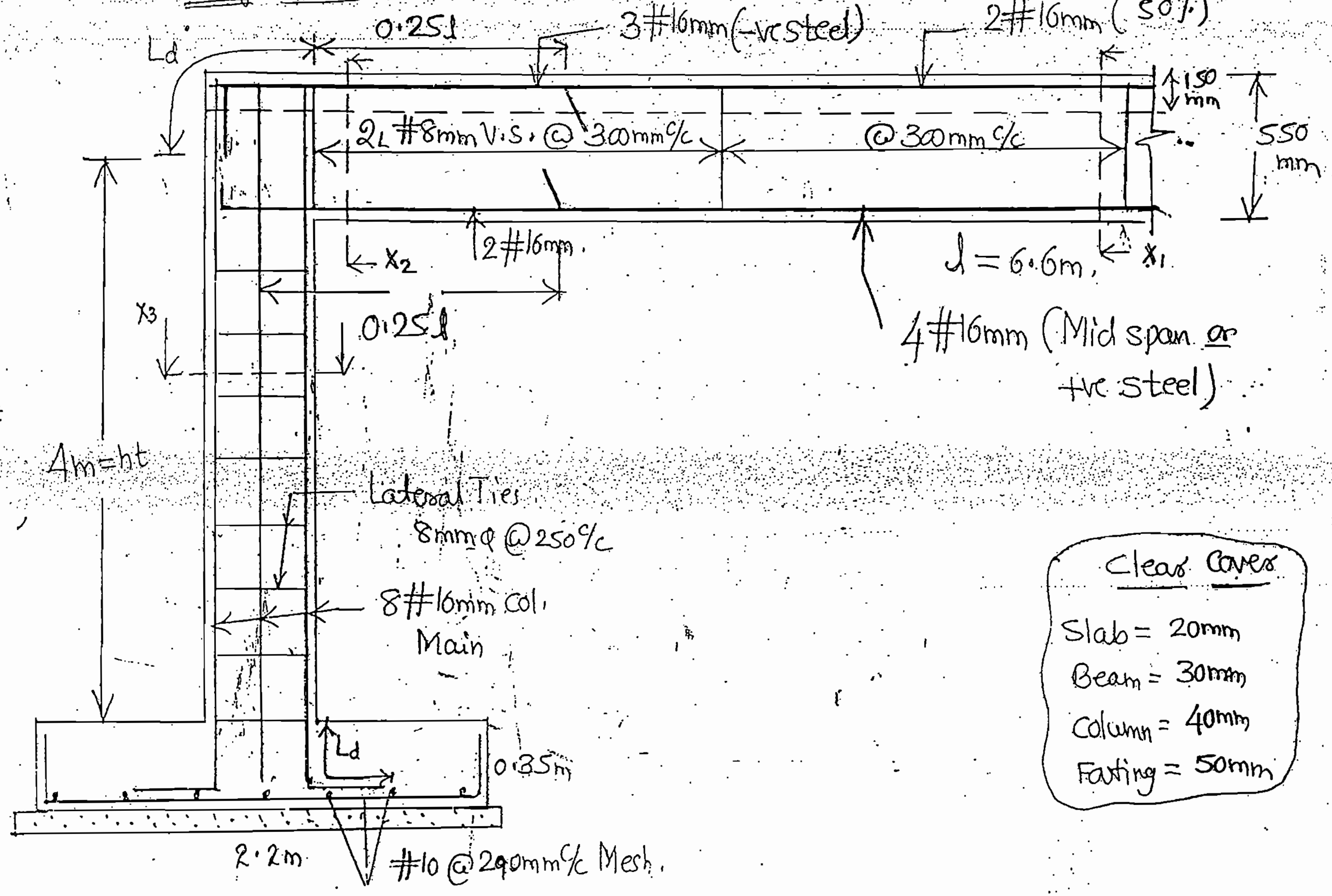
Spacing of 10mm ϕ bars $= \dfrac{\frac{\pi}{4} * 10^2}{270.77} * 1000$

$= 290.06$ mm c/c.

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 290 mm c/c.}}$

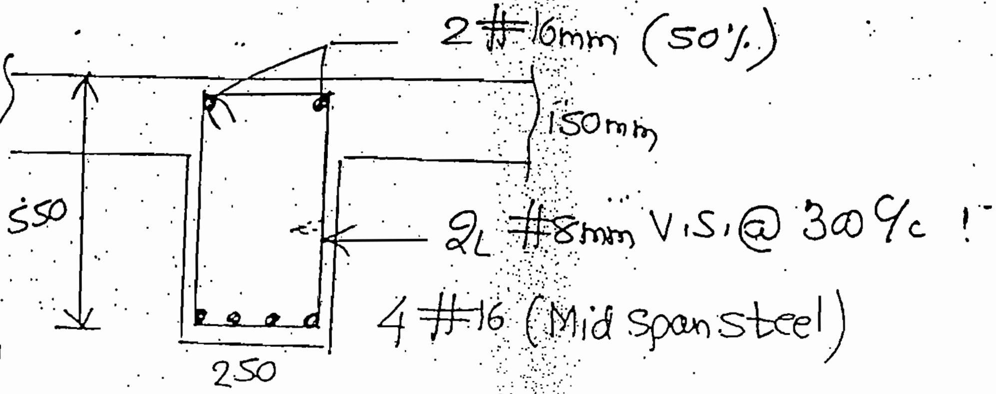
Half Sectional Elevation :

$\boxed{1:20}$

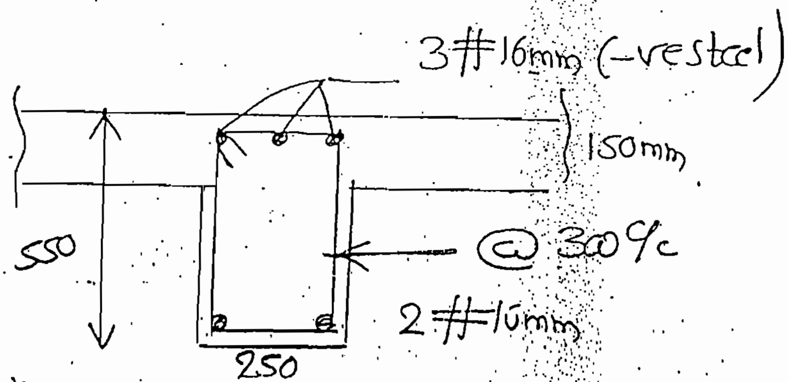


Clear Cover

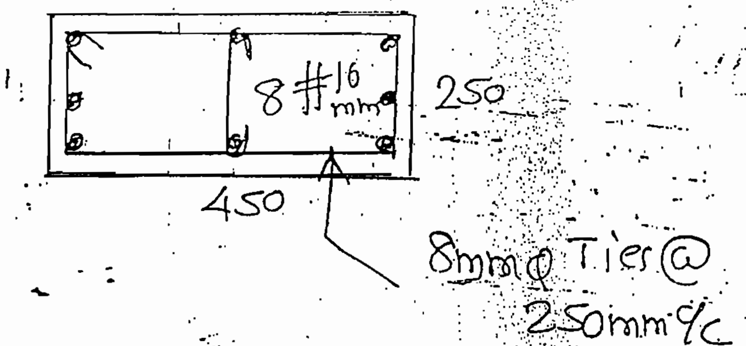
Slab = 20mm
Beam = 30mm
Column = 40mm
Footing = 50mm



X1-X1

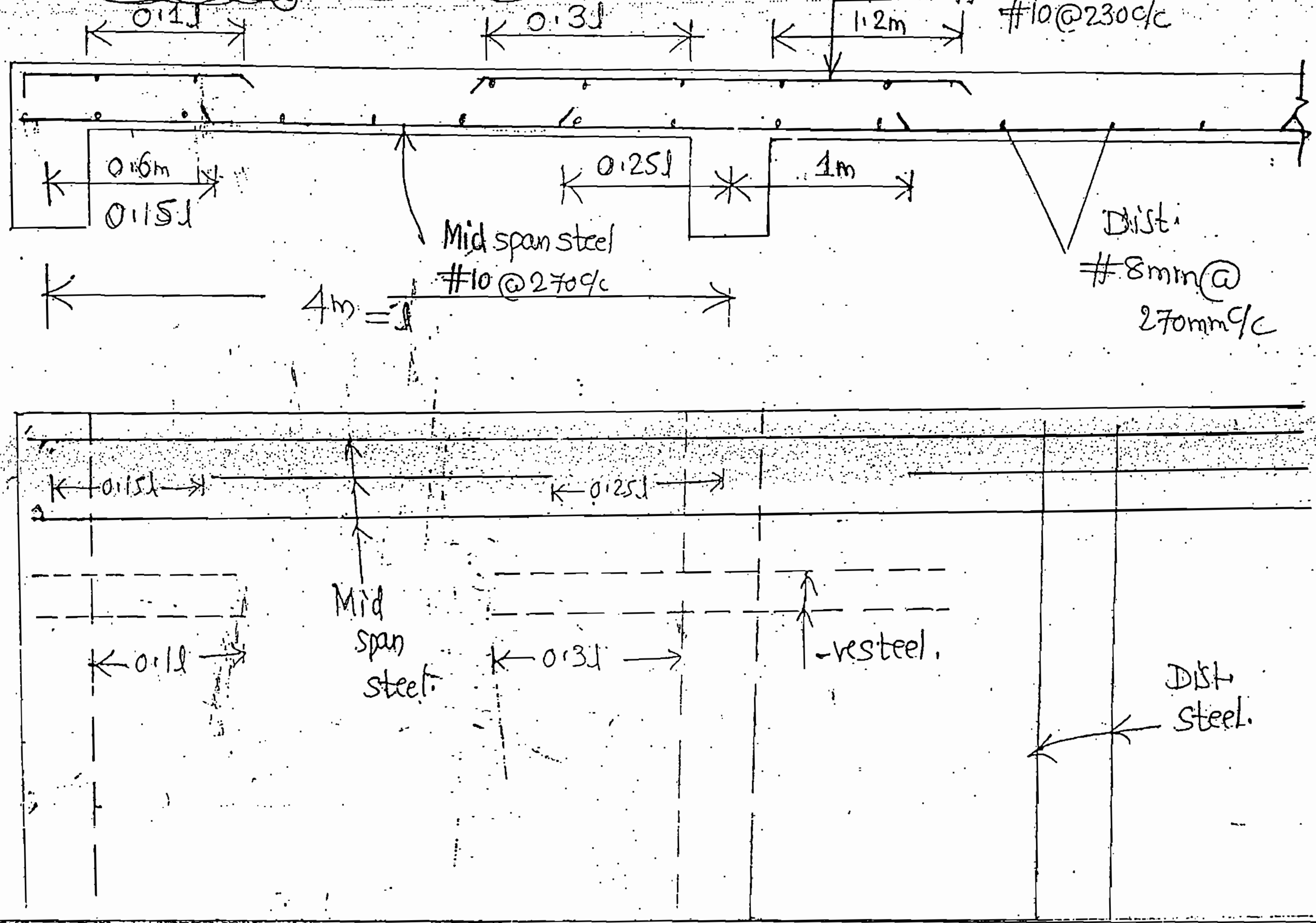


X2X2



X3X3

One-way Continuous Slab:



MODULE # 1 PORTAL FRAMES (HINGED BASE)

Problem # 1. A portal frame hinged at the base has the following data:

Spacing of portal frames = 4 m c/c

Height of Columns = 4 m

Distance b/w Column Centers = 10 m.

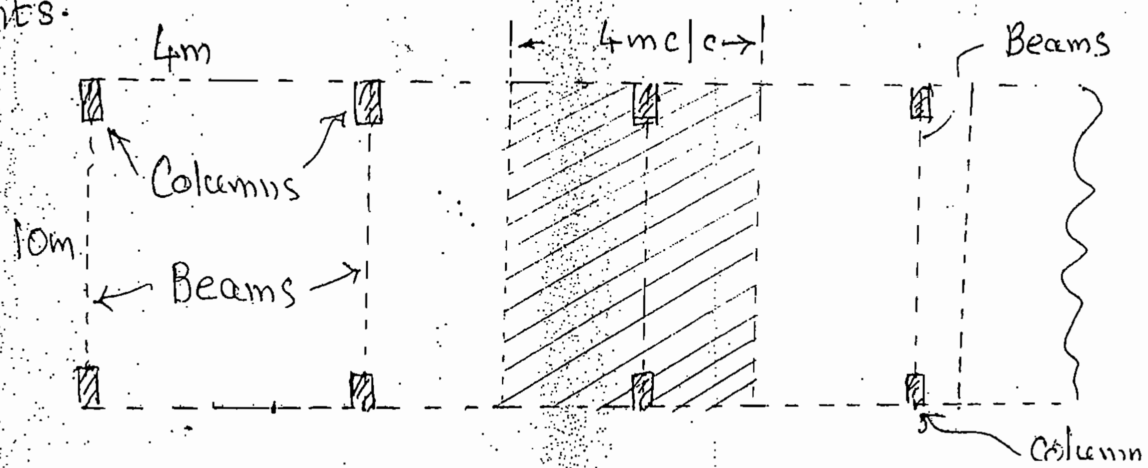
Live load on Roof = 1.50 kN/m^2

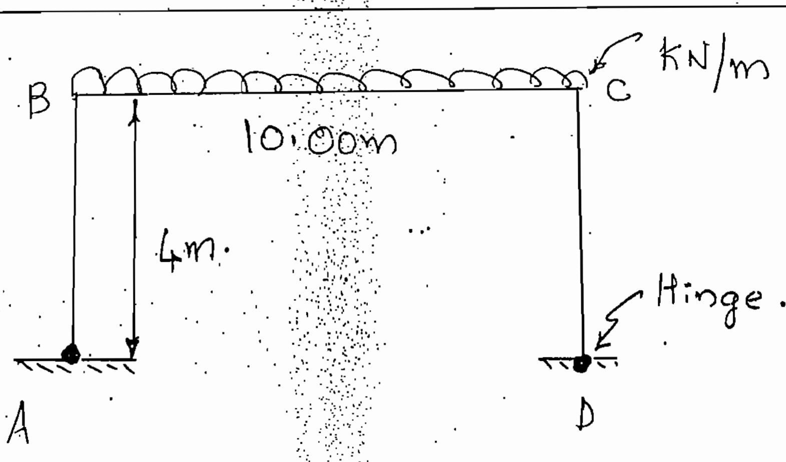
RCC slab is Continuous over portal frames.

Safe bearing Capacity of Soil = 200 kN/m^2.

Adopt M_{20} grade Concrete and Fe 415 Steel. Design the Slab, portal frames & foundation. Sketch the details of Reinforcements.

Soln:





Step # 1: Design of One Way Continuous slab:

★ Use $\boxed{\dfrac{l}{d} = 26 * M.F}$

MF = Modification factor for Steel

$\boxed{MF = 1.25}$ (Assume)

l = Spacing of portal frames = 4 m c/c.

$\therefore \quad \dfrac{4000}{d} = 26 * 1.25$

$\therefore \quad d = \dfrac{4000}{26 * 1.25} = 123.08$ Say 125 mm

Using 10mm ϕ bars with clear Cover of 20 mm,

$D = d + \dfrac{\phi}{2} +$ Clear Cover $= 125 + \dfrac{10}{2} + 20$

$\boxed{\text{Provide } D = 150\,mm \text{ and } = 125\,mm}$

(i) Load Calculations:

Dead load $= \gamma_{Conc} * D = 25 * 0.15 = 3.75\ KN/m^2$

Floor finishes $\longrightarrow \quad = \quad = 0.50\ KN/m^2$

Total $= 4.25\ KN/m^2$

Factored dead load $= \boxed{\omega_g = 1.5\,\omega}$

$\omega_g = 1.50 * 4.25$

$\therefore \boxed{\omega_g = 6.38\ KN/m^2}$

factored live load $= \boxed{\omega_q = 1.5 * \omega_l}$

$= 1.5 * 1.50$

$\boxed{\omega_q = 2.25\ KN/m^2}$

(ii) Bending Moment:

As per IS 456 – 2000, Page (36), Table #12,

(a) Span moment: (+ve moment)

$$\boxed{M = \frac{w_g l^2}{12} + \frac{w_q l^2}{10}} = \frac{6.38 * 4^2}{12} + \frac{2.25 * 4^2}{10}$$

$$\boxed{M = 12.10 \text{ kN-m.}}$$

(b) Support Moment: (–ve Moment):

$$\boxed{M = -\frac{w_g l^2}{10} - \frac{w_q l^2}{9}} = -\frac{6.38 * 4^2}{10} - \frac{2.25 * 4^2}{9}$$

$$\boxed{M = -14.21 \text{ kN-m.}}$$

∴ Absolute Max BM = 14.21 kN-m.

(iii) Check for depth:

★ $(M_u)_{lim} = 0.36 \frac{x_{u\,max}}{d} \left[1 - 0.42 \frac{x_{u\,max}}{d}\right] bd^2 f_{ck}$

$b = 1m = 1000$ mm, $\frac{x_{u\,max}}{d} = 0.48$

for Fe 415 and balanced S...

$$14.21 * 10^6 = 0.36 * 0.48 \left[1 - 0.42 * 0.48\right] * 1000 * d^2 * 20$$

$$d^2 = \frac{14.21 * 10^6}{0.36 * 0.48 * (0.798) * 1000 * 20} = 5149.91$$

$$d = \sqrt{5149.91} = 71.76 \text{ mm} < 125 \text{ mm ok}$$

(iv) Area of Steel :

(a) Mid Span Steel : (+ve Steel)

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

As per IS 456 - 2000, clause G-1.1.

$$(M_u) = 12.10 * 10^6 = 0.87 f_y \cdot A_{st} \cdot 125 \left[1 - \frac{A_{st} \cdot 415}{1000 * 125 * 20}\right]$$

$$12.10 * 10^6 = 45.13 * 10^3 A_{st} \left[1 - 166 * 10^{-6} A_{st}\right]$$

$$268.11 = A_{st} - 166 * 10^{-6} (A_{st})^2$$

$$\Rightarrow (A_{st})^2 - 6.024 * 10^3 (A_{st}) + 1.62 * 10^6 = 0.$$

Solving $\boxed{A_{st} = 282.14 \text{ mm}^2.}$

Provide 10mm ϕ bars.

Spacing of 10mm ϕ bars $= \dfrac{\frac{\pi}{4} * 10^2}{282.14} * 1000$

$= 278.40$ mm Say 270 mm c/c.

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 270mm c/c}}$ < 3d or 300 mm

(b) Support steel : (− ve Steel) :

$M_u = 14.21 * 10^6 = 0.87 * 415 * A_{st} * 125 \left[1 - \dfrac{A_{st} * 415}{1000 * 125 * 20}\right]$

$14.21 * 10^6 = 45.13 * 10^3 A_{st} \left[1 - 166 * 10^{-6} A_{st}\right]$

$314.86 = A_{st} - 166 * 10^{-6} (A_{st})^2$

$166 * 10^{-6} (A_{st})^2 - A_{st} + 314.86 = 0.$

Solving, $\boxed{A_{st} = 333.30 \text{ mm}^2.}$

★ Provide 10mm ϕ bars.

Spacing of 10mm ϕ bars $= \dfrac{\frac{\pi}{4} * 10^2}{\ldots} * 1000$

$= 235.64$ mm Say 230 mm c/c.

$\boxed{\text{Spacing of } 10\text{mm } \phi \text{ bars @ } 230 \text{ mm c/c}}$ < 3d or 300 mm

(c) Distribution Steel:

$$\text{Area} = \frac{0.12}{100} * \underset{b}{1000} * \underset{D}{150} = \boxed{180 \text{ mm}^2}$$

Provide 8mm ϕ bars.

$$\text{Spacing of } 8\text{mm } \phi \text{ bars} = \frac{\frac{\pi}{4} * 8^2}{180} * 1000$$

$= 279$ mm Say 270 mm c/c.

$\boxed{\text{Spacing of } 8\text{mm } \phi \text{ bars @ } 270 \text{ mm c/c}}$ < 5d or 450 mm

Step # 2: Analysis of portal frame:

A Beam: Assume width b = 200 or 250 or 300 mm

Take $\boxed{b = 250 \text{ mm}}$

Depth $\boxed{D = \frac{\text{Span}}{12}} = \frac{10000}{12} = 833.33$ Say 850 mm

Take $\boxed{250\,mm \times 850\,mm}$ Size for beam

* Column : Column width = Beam width = 250 mm

Depth : 1.50 to 2 times width.

Depth = 1.5×250 or $2 * 250$

Depth = 375 mm or 500 mm.

Take Depth = 600 mm.

Provide $\boxed{250\,mm * 600\,mm}$ Size for Column.

$$I_{beam} = \frac{bD^3}{12} = \frac{250 * 850^3}{12} = 1.28 * 10^{10}\,mm^4$$

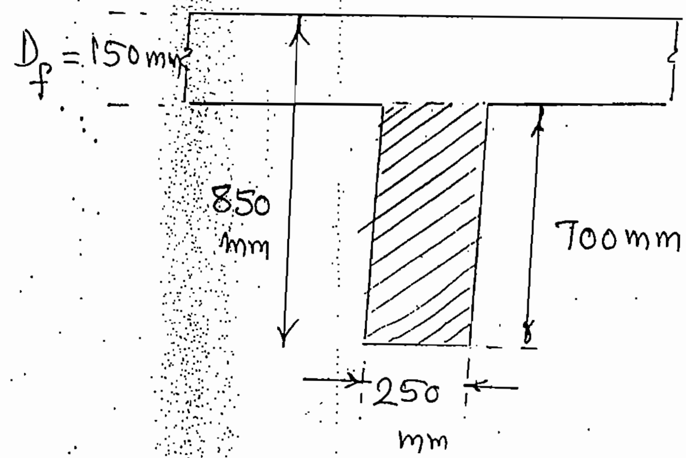
$$\therefore \boxed{I_{beam} = 1.28 * 10^{10}\,mm^4}$$

$$I_{column} = \frac{bD^3}{12} = \frac{250 * 600^3}{12} = 4.5 * 10^9\,mm^4$$

$$\therefore \boxed{I_{Column} = 4.5 * 10^9\,mm^4}$$

let $I_1 = I_{Col} = 4.5 * 10^9\,mm^4$.

$$\therefore \boxed{I_{beam} = \frac{1.28 * 10^{10}}{4.5 * 10^{9}} = 2.85\ I}$$



Total load on slab $= \omega_g + \omega_q = (6.38 + 2.25)\ kN/m^2$

$= 8.63\ kN/m^2.$

UDL on beam from slab $= \left\{ \begin{matrix} \text{Total load} \\ \text{on slab.} \end{matrix} \right\} *$ Beam Spacin

$= 8.63 * 4 = 34.52\ kN/m$ --- I

Self weight of beam $= \left(0.25 * \frac{700}{1000}\right) * 25$

$= 4.375\ kN/m.$

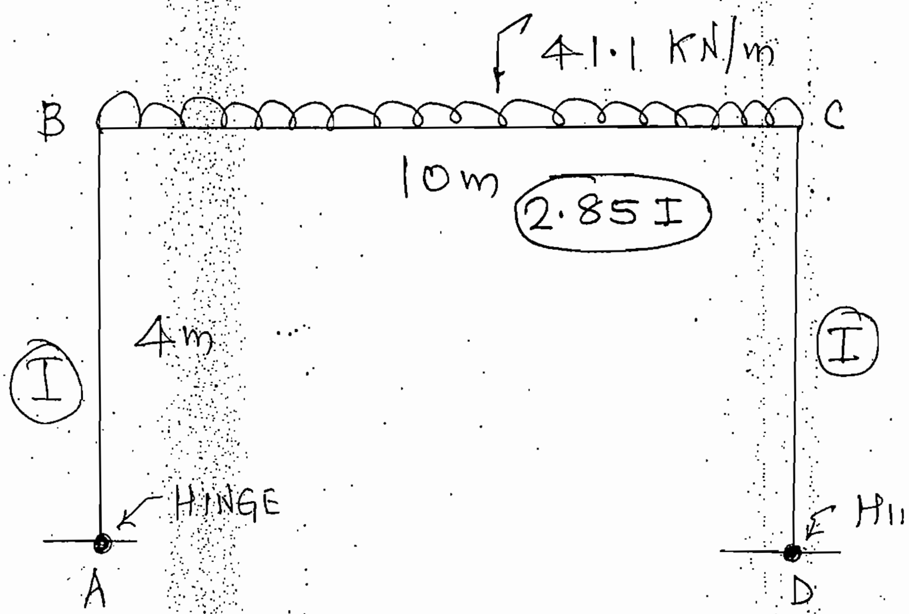
Factored self wt on beam $= 1.50 * 4.375$

$= 6.56\ kN/m$ --- II

Total udl = I + II = 34.52 + 6.5

= 41.08 ≃ 41.1 kN/m.

∴ $\boxed{\text{Total udl} = 41.1 \text{ kN/m}}$



Using Moment distribution method:

$$M_{FBC} = -\frac{\omega l^2}{12} = -\frac{41.10 * 10^2}{12} = -3$$

$$M_{FCB} = +\frac{\omega l^2}{12} = +\frac{41.10 * 10^2}{12} = +3$$

* $M_{FAB} = M_{FDC} = 0$ * Hinged support. !

Stiffness factor : $\boxed{K = \frac{I}{l}}$

$$K_{BA} = \frac{3}{4}\frac{I_{BA}}{l_{BA}} = \frac{3}{4} * \frac{I}{4} = 0.25 I. = K_{CD}$$

0.1875I

$$K_{BC} = \frac{I_{BC}}{l_{BC}} = \frac{2.85 I}{10} = 0.285 I.$$

$$\Sigma k = K_{BA} + K_{BC} = 0.25 I + 0.285 I = 0.535 I$$

0.1875I 0.4725I

Distribution factor (D.F) $= \dfrac{k}{\Sigma k}$

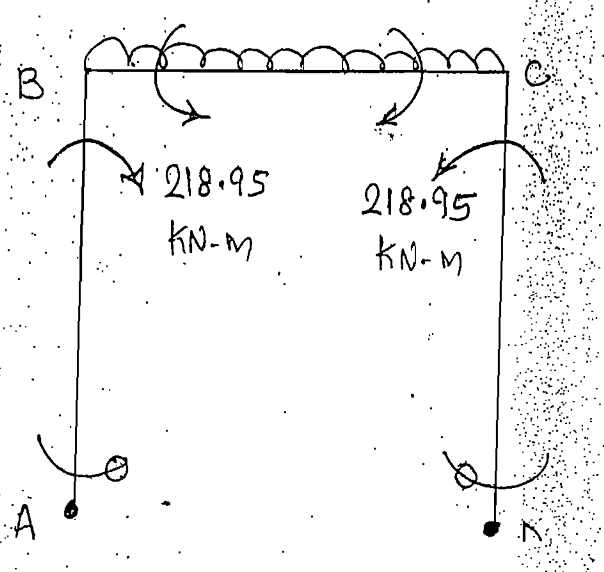
DF for BA $= \dfrac{0.25 I}{0.535 I} = 0.47$ 0.40

(0.1875I / 0.4725)

DF for BC $= \dfrac{0.285 I}{0.535 I} = 0.53$ 0.60

(0.4725)

Moment distribution Table on back page:



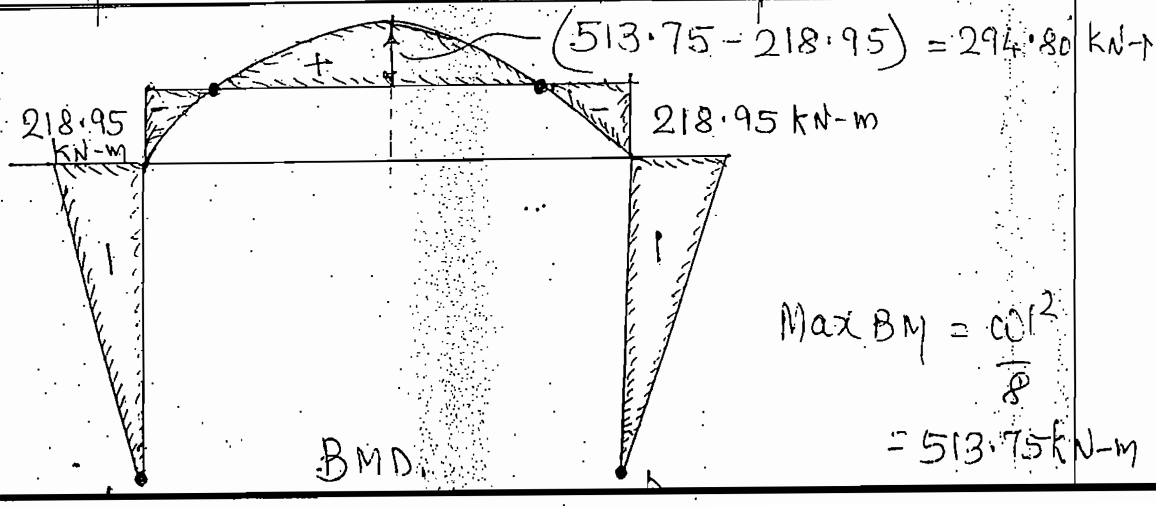
Max BM $= \dfrac{wl^2}{8}$

$= \dfrac{41.10 * 10^2}{8}$

$= 513.75$ KN-m

Moment - Distribution Table:

Joint	A	B		C		D
Memb	AB	BA 0.4	BC 0.6	CB 0.6	CD 0.4	DC
DF	0	0.47	0.53	0.53	0.47	0
FEM	0.00	0.00	−342.50	+342.50	0.00	0.00
Balance	0.00	+160.98	+181.52	−181.52	−160.98	0.00
C.O.	0.00	0.00	−90.76	+90.76	0.00	0.00
Balance	0.00	+42.66	+48.10	−48.10	−42.66	0.00
C.O	0.00	0.00	−24.05	24.05	0.00	0.00
Balance	0.00	11.30	+12.75	−12.75	−11.30	0.00
C.O.	0.00	0.00	−6.38	+6.38	0.00	0.00
Balance	0.00	+3.00	+3.38	−3.38	−3.00	0.00
C.O	0.00	0.00	−1.69	+1.69	0.00	0.00
Balance	0.00	+0.80	+0.89	−0.89	−0.80	0.00
C.O.	0.00	0.00	−0.45	0.45	0.00	0.00
Balance	0.00	+0.21	+0.24	−0.24	−0.21	0.00
End Moment	0.00	218.95	−218.95	218.95	−218.95	0.00



BMD.

$$\text{Max BM} = \frac{\omega l^2}{8}$$

$$= 513.75 \text{ kN-m}$$

** final design Values:

Beam: $250 * 850$ mm, Span = 10m.

Load: 41.10 kN/m

Reaction = S.F. = $V_u = \frac{wl}{2} = \frac{41.10 * 10}{2} = 205.5$ kN

$(M_u)_{Support} = 218.95$ kN-m, $(M_u)_{Mid} = 294.80$ kN-m.

Column: 250mm * 600 mm.

Load P_u = Beam Reaction = 205.50 kN

M_u = Support Moment = 218.95 kN-m

Footing: SBC = 200 kN/m^2.

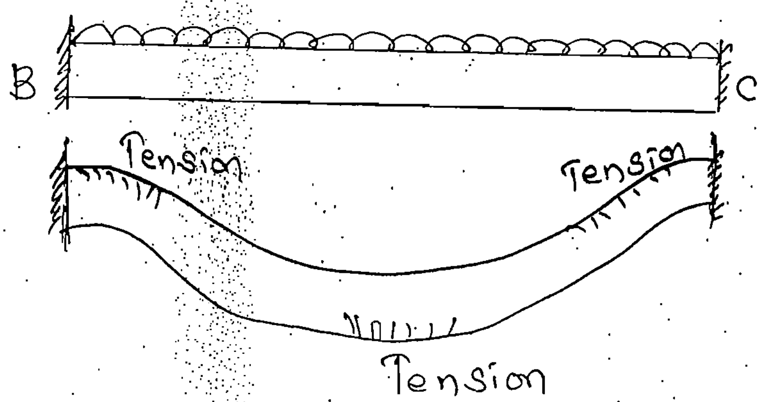
Load on Footing = Load on Column + factored Self wt of Column

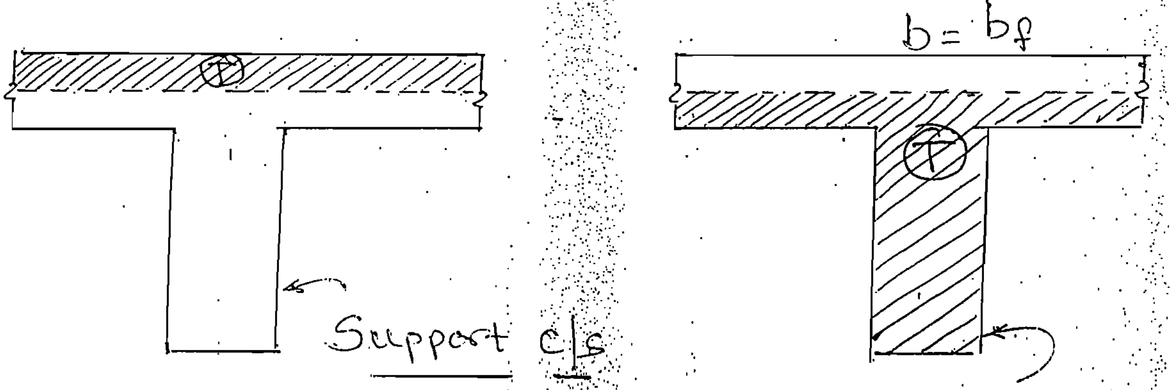
$= 205.50 + 1.50\,(0.25 * 0.6 * \overset{Ht}{4} * 25)$

$= 228$ kN

Moment $= 0$.

Step # 3: Design of beam:





(T) = Tension zone   Mid Span c/s

★ Neglecting Tension Zone,

★★ Support c/s is designed like a rectangular beam taking $\boxed{b = 250 \text{ mm.}}$

★★ Mid Span c/s is designed like a T-beam taking $\boxed{b = b_f}$

$$\therefore \boxed{b_f = \frac{l_o}{6} + 6D_f + b_w}$$ Pg (37) IS 456-2000

$$\boxed{l_o = 0.7l}$$ Pg (37) IS 456 - 2000

$l_0 = 0.7 * 10.00 = 7m. = 7000 mm.$

$b_f = \frac{7000}{6} + 250 + 6 * 150 = 2316.67$ Say 2320 mm.

(A) Design of Support c/s:

$M_u = 218.95$ kN-m, $b = 250$ mm, $D = 850$ mm

Assuming an effective Cover of 50 mm,

$d = D - 50 = 850 - 50 = 800$ mm.

As per IS 456-2000, Clause G-1.1.

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

$$218.95 * 10^6 = 0.87 * 415 * A_{st} * 800 \left[1 - \frac{A_{st} * 415}{250 * 800 * 20}\right]$$

$$218.95 * 10^6 = 288.84 * 10^3 A_{st} \left[1 - 103.75 * 10^{-6} A_{st}\right]$$

$$758.03 = A_{st} - 103.75 * 10^{-6} (A_{st})^2$$

$$(A_{st})^2 - 9638.55 A_{st} + 7.31 * 10^6 = 0.$$

Solving, $\boxed{A_{st} = 829.86 \text{ mm}^2}$

Provide 16mm ϕ bars.

$$\text{No of Bars} = \frac{829.86}{\frac{\pi}{4} * 16^2} = 4.13 \simeq 5$$

$\boxed{\text{Provide 5 bars of 16mm } \phi}$

(B) Design of Mid Span c/s : $\boxed{b = b_f}$

$$M_{LP} = 294.80 * 10^6 = 0.87 * 415 * A_{st} * 800 \left[1 - \frac{A_{st} * 4}{2320 * 80}\right]$$

$$294.80 * 10^6 = 288.84 * 10^3 A_{st} \left[1 - 11.18 * 10^{-6} A_{st}\right]$$

$$758.03 = A_{st} - 11.18 * 10^{-6} (A_{st})^2$$

or $11.18 * 10^{-6} (A_{st})^2 - A_{st} + 758.03 = 0.$

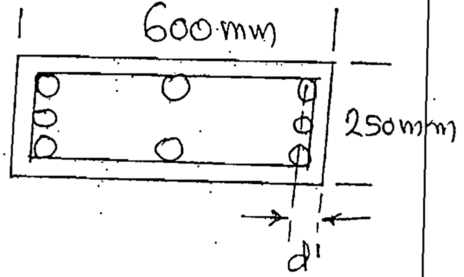
Solving, $\boxed{A_{st} = 764.57 \text{ mm}^2.}$

Provide 16mm ϕ bars.

$$\text{No of bars} = \frac{764.57}{\frac{\pi}{4} * 16^2} = 3.80 \simeq 4$$

$\boxed{\text{Provide 16mm } \phi \text{ bars of 4 No's.}}$

Step # 4: Design of Column:



$P_u = 205.50$ kN

$M_u = 218.95$ kN-m.

Assume d' = effective Cover = 50 mm,

(i) Reinforcement on all 4 sides

(ii) Grade of Steel → Fe 415

(iii) Ratio $\left(\frac{d'}{D}\right) = \left(\frac{50}{00}\right) = 0.083.$

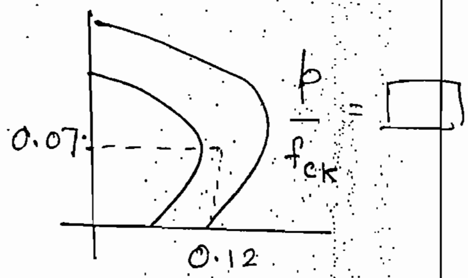
Using chart (44), Pg 128, Sp- 16,

$$\frac{P_u}{f_{ck}\, bD} = \frac{205.50 * 10^3}{20 * 250 * 600} = \boxed{0.069}$$

$$\frac{M_u}{f_{ck}\, bD^2} = \frac{218.95 * 10^6}{20 * 250 * (600)^2} = \boxed{0.122}$$

★ Using interaction Curves of Sp 16, chart (44)

$$\frac{p}{f_{ck}} = 0.07 \text{ from chart.}$$



$\therefore p = 0.07 * f_{ck}$

$p = 0.07 * 20 = 1.4$

p = Percentage of reinforcement.

$$A_{st} = \left(\frac{p}{100}\right) bD = \left(\frac{1.4}{100}\right) * 250 * 600$$

$$\boxed{A_{st} = 2100 \text{ mm}^2}$$

Provide 20mm ϕ bars.

$$\text{No of bars} = \frac{2100}{\frac{\pi}{4} * 20^2} = 6.68 \simeq 7 \text{ say } 8$$

$\boxed{\text{Provide 8 bars of 20mm } \phi.}$

Design of lateral ties:

Provide 8mm ϕ lateral ties.

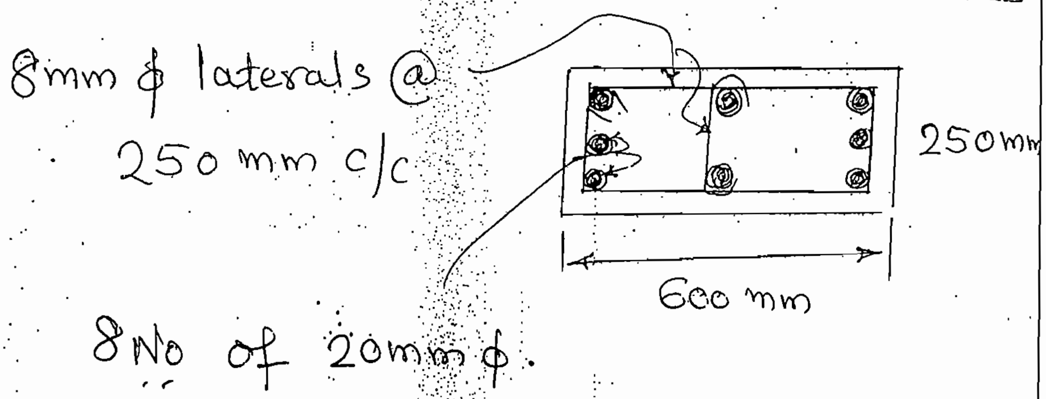
Spacing of 8mm lateral ties must be the least of the following:

(a) b = 250 mm ✓

(b) $16\phi = 16 * 20 = 320$ mm

(c) 300 mm.

$\boxed{\text{Provide 8mm } \phi \text{ lateral ties @ 250 mm c/c.}}$



Step # 5 : Design of Hinge:

★ At the Hinge portion, Concrete is under triaxial Stress and Can withstand higher permissible Stress.

★★ Permissible Compressive Stress in Concrete at the hinge $= \boxed{2 * 0.4 * f_{ck}}$

$= 2 * 0.4 * 20$

$= 16 \text{ N/mm}^2$

* Factored thrust = P_u = 205.50 KN

C/S area of Hinge reqd = $\boxed{\dfrac{P_u}{\text{Permissible Stress at Hinge}}}$

$$= \frac{205.50 * 10^3}{16} = 12843.75 \text{ mm}^2$$

** provide Concrete area $\boxed{200 \text{ mm} * 100 \text{ mm}}$ ($= 20,000 \text{ mm}^2$) at the Hinge.

** Shear force at Hinge = $\boxed{\dfrac{\text{Total Support Moment}}{\text{Column Height}}}$

$$V = \frac{218.95}{4} = 54.74 \text{ KN}.$$

factored Shear force = $1.5 V = 1.5 * 54.74$

$$V_u = 82.10 \text{ KN}.$$

* θ = inclination of bars to vertical = 45°

Factored Shear force $\boxed{V_u = 0.87 f_y \cdot A_{st} \cdot \sin\theta.}$

$$82.10 * 10^3 = 0.87 * 415 * A_{st} * \sin 45^\circ$$

$$\therefore A_{st} = \frac{82.10 * 10^3}{0.87 * 415 * 0.7071} = 321.60 \text{ mm}^2$$

* Provide min 4 bars of 16 mm ϕ.

$$\therefore A_{st} \text{ provided} = 4 * \left[\frac{\pi}{4} * 16^2\right]$$

$$= 804 \text{ mm}^2$$

$$\therefore \boxed{\text{Area of Steel for Hinge} = 804 \text{ mm}^2}$$

** Spirals of 10 mmϕ with helical reinforcement of 6mmϕ ties are provided to facilitate Hinge action.

Step # 6 : Shear design:

$$\text{Nominal Shear Stress} = \tau_v = \frac{V_u}{bd} = \frac{205.5 * 10^3}{250 * 800}$$

$$\boxed{\tau_v = 1.03 \text{ N/mm}^2}$$

5 bars of 16mm ϕ

$$\frac{100\, A_{st}}{bd} = \frac{100 * \left(5 * \frac{\pi}{4} * 16^2\right)}{250 * 800} = \boxed{0.503} \text{ \& } M_{20}$$

From Table # 19, Pg 73 of IS 456 - 2000, for $\frac{100 A_{st}}{bd} = 0.5$, M_{20}, $\tau_c = 0.48\ N/mm^2$.

$\therefore\ \tau_c < \tau_v$

Provide Shear reinforcement.

Provide 8mm ϕ 2 legged stirrups.

$$\therefore\ A_{sv} = 2\left[\frac{\pi}{4} * 8^2\right] = 100.53\ mm^2.$$

By IS 456 - 2000, Pg (73),

$$\boxed{V_{us} = (V_u - \tau_c\, b\, d) = \frac{0.87\, f_y\, A_{sv}\, d}{S_v}}$$

$$(205.50*10^3 - 0.48*250*800) = \frac{0.87*415*100.53*}{S_v}$$

$$109.5*10^3 = \frac{29.04*10^6}{S_v}$$

$$S_v = \frac{29.04*10^6}{109.5*10^3} = 265.2 \text{ Say } 260\ mm$$

Provide 8mm ϕ 2 legged Stirrups @ [illegible]

Step # 7: Design of footing:

SBC = 200 KN/m^2

P_4 = load on footing = 228 KN

M_4 = Moment on Footing = 0 (base)

Working Value of $P_4 = \frac{228}{1.50} = 152$ KN. ← fos

Working Value of $M_4 = \frac{0}{1.50} = 0$ @ base ← fos

★ Working Value = $\boxed{M = \text{S.F. at Hinge} * 1\text{ m height.}}$

$$\boxed{M = 54.74 \text{ KN} * 1\text{m} = 54.74 \text{ KN-m.}}$$

(a) Size of Footing:

Column load + 10% of Column load as Self wt

= 228 + 0.1 * 228 = 250.80 kN. = pl.

Assume width of Footing (B) as 1.20m or 1.50m or 1.80m or 2.00 m.

Assume width $\boxed{B = 1.50m.}$

$$\boxed{\frac{P'}{B*L} + \frac{6M}{B*L^2} = SBC}$$

$$\frac{250.80}{1.50\,L} + \frac{6*54.74}{1.50\,L^2} = 200$$

$$\therefore\ 250.80L + 6*54.74 = 300\,L^2$$

$$\therefore\ 250.80L + 328.44 = 300\,L^2$$

$$\text{or } 300\,L^2 - 250.80\,L - 328.44 = 0.$$

Solving, $L = 1.54m$ Say $2.00m$.

Try $\boxed{L*B = 2.00m * 1.50m.}$ ★ By Trial & error

$$\boxed{f = \frac{P'}{B*L} \pm \frac{6M}{BL^2} \geq 0.}$$

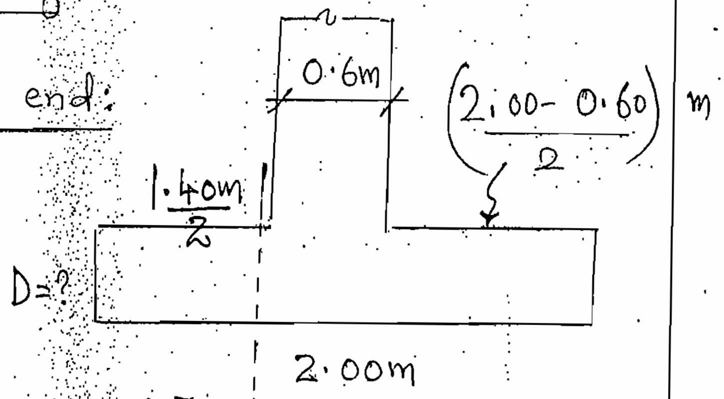
$$f = \frac{250.80}{1.5 * 2.0} \pm \frac{6 * 54.74}{1.50 * 2^2} = 83.6 \pm 54.74$$

$$f_{max} = 83.60 + 54.74 = 138.34 \ kN/m^2$$

$$f_{min} = 83.60 - 54.74 = 28.86 kN/m^2$$

(b) Thickness of Footing:

Pressure Intensity from one end:



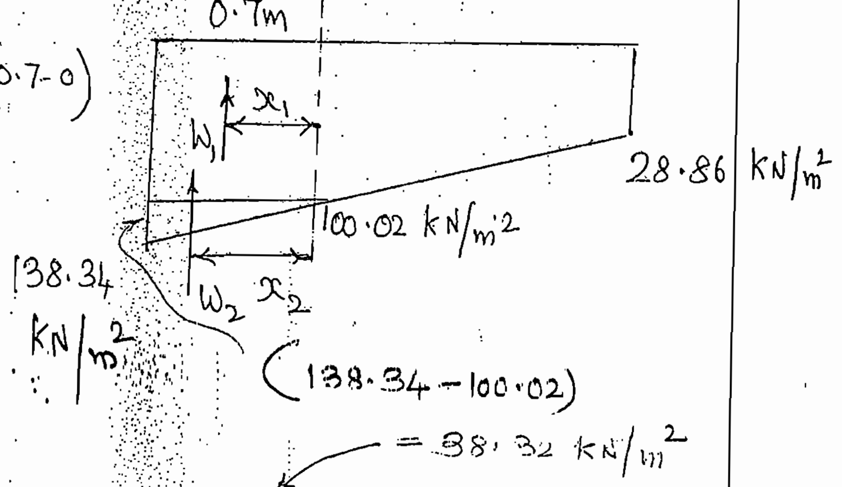
0 m ----- $138.34 \ kN/m^2$

0.7m ----- ?

2m ----- $28.86 \ kN/m^2$

$$= 138.34 + \left(\frac{28.86 - 138.34}{2 - 0}\right)(0.7 - 0)$$

$$= 100.02 \ kN/m^2$$



$$M = (0.7 * 100.02) * \frac{0.7}{2} + \frac{1}{\cancel{2}} * 0.7 * 38.32 * \frac{\cancel{2}}{3}(0.7)$$

$$M = (0.7)^2 \left[50.01 + 12.77\right] = 30.76 \ kN\text{-}m.$$

Factored Moment = $1.5 M = 1.5 * 30.76 = 46.14 \ kN\text{-}m$

$\therefore M_u = 46.14$ kN-m.

Using $(M_u)_{lim} = 0.36 \frac{x_{umax}}{d}\left[1 - 0.42 \frac{x_{u.max}}{d}\right] bd^2 f_{ck}$

$\frac{x_{u\,max}}{d} = 0.48$ for balanced Section and Fe 415,

$b = 1000$ mm,

$46.14 * 10^6 = 0.36 * 0.48 \left[1 - 0.42 * 0.48\right] * 1000 * d^2 * 20$

$46.14 * 10^6 = 2.76 * 10^3\, d^2$

$d = \sqrt{\frac{46.14 * 10^6}{2.76 * 10^3}} = 129.31$ mm.

Using an effective Cover of 60 mm,

$D = d + 60 = 129.31 + 60 = 189.31$ mm

* from Shear Consideration, double the above Value.

$\therefore D = 189.31 * 2 = 378.63$ Say 400 mm.

$\boxed{\text{Provide } D = 400 \text{ mm and } d = 340 \text{ mm.}}$

(c) Area of Steel (A_{st}):

As per IS 456 - 2000, Clause G-1.1,

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

$b = 1000$ mm, $d = 340$ mm,

$$46.14 * 10^6 = 0.87 * 415 * A_{St} * 340 \left[1 - \frac{A_{st} * 415}{1000 * 340 * 20}\right]$$

$$46.14 * 10^6 = 122.76 * 10^3 A_{st} \left[1 - 61.03 * 10^{-6} A_{st}\right]$$

$$375.86 = A_{st} - 61.03 * 10^{-6} (A_{st})^2$$

$$\therefore 61.03 * 10^{-6} (A_{st})^2 - A_{st} + 375.86 = 0.$$

Solving, $\boxed{A_{st} = 384.90 \text{ mm}^2.}$

Provide 10mm ϕ bars.

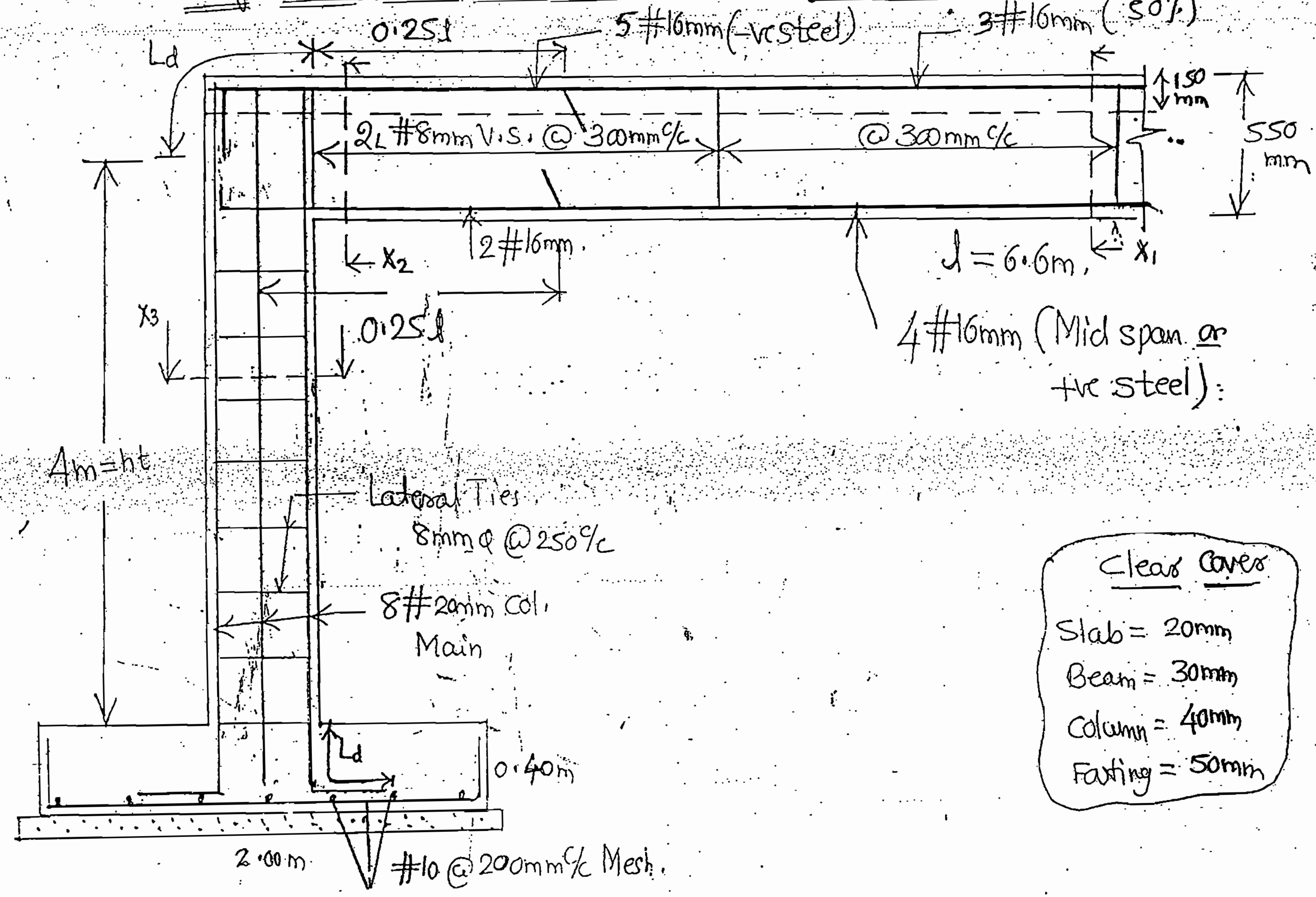
Spacing of 10mm ϕ bars $= \dfrac{\frac{\pi}{4} * 10^2}{384.90} * 1000$

$= 204.05$ mm Say 200mm c/c.

$\boxed{\text{Spacing of 10mm } \phi \text{ bars @ 200 mm c/c.}}$

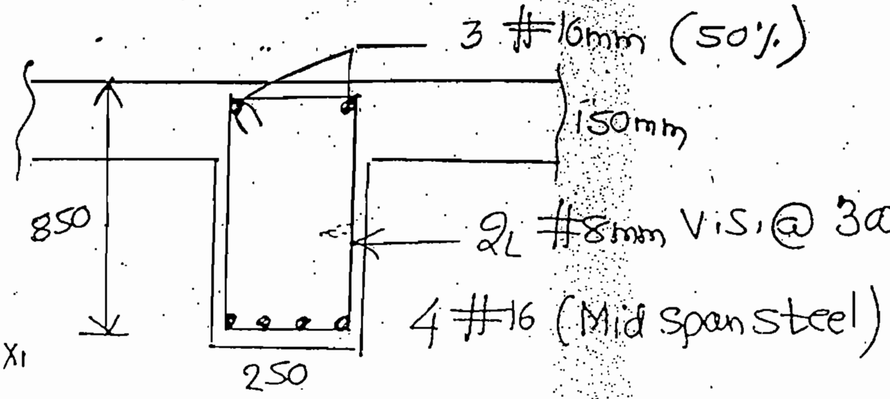
—×—

Half Sectional Elevation : 1 : 20

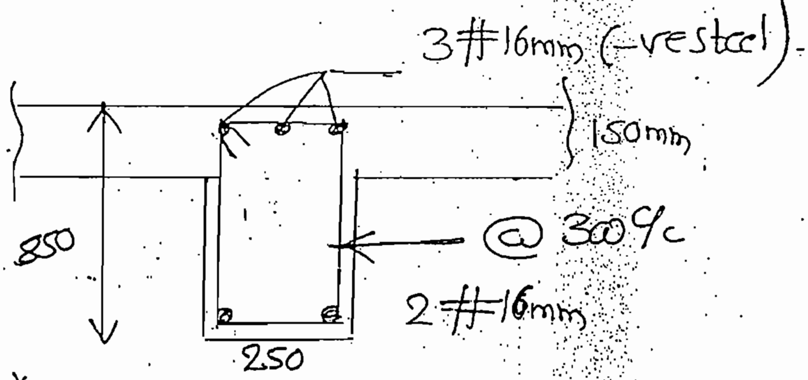


Clear Cover

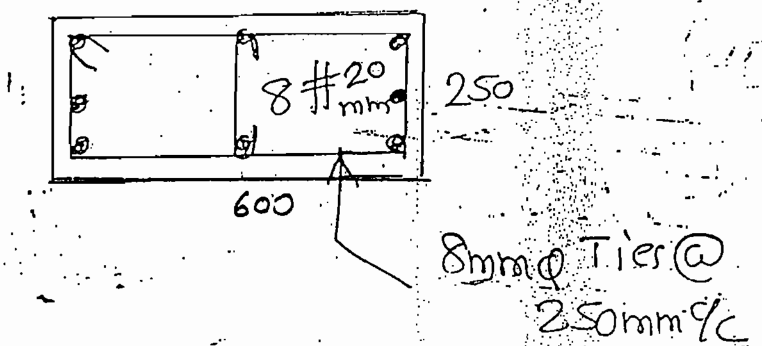
Slab = 20mm
Beam = 30mm
Column = 40mm
Footing = 50mm



X1-X1

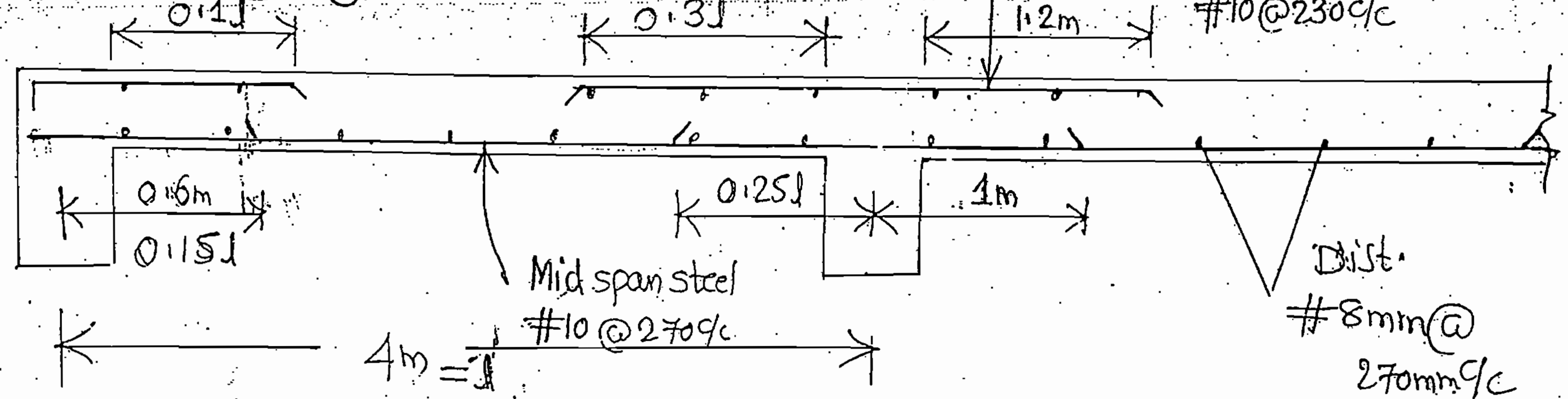


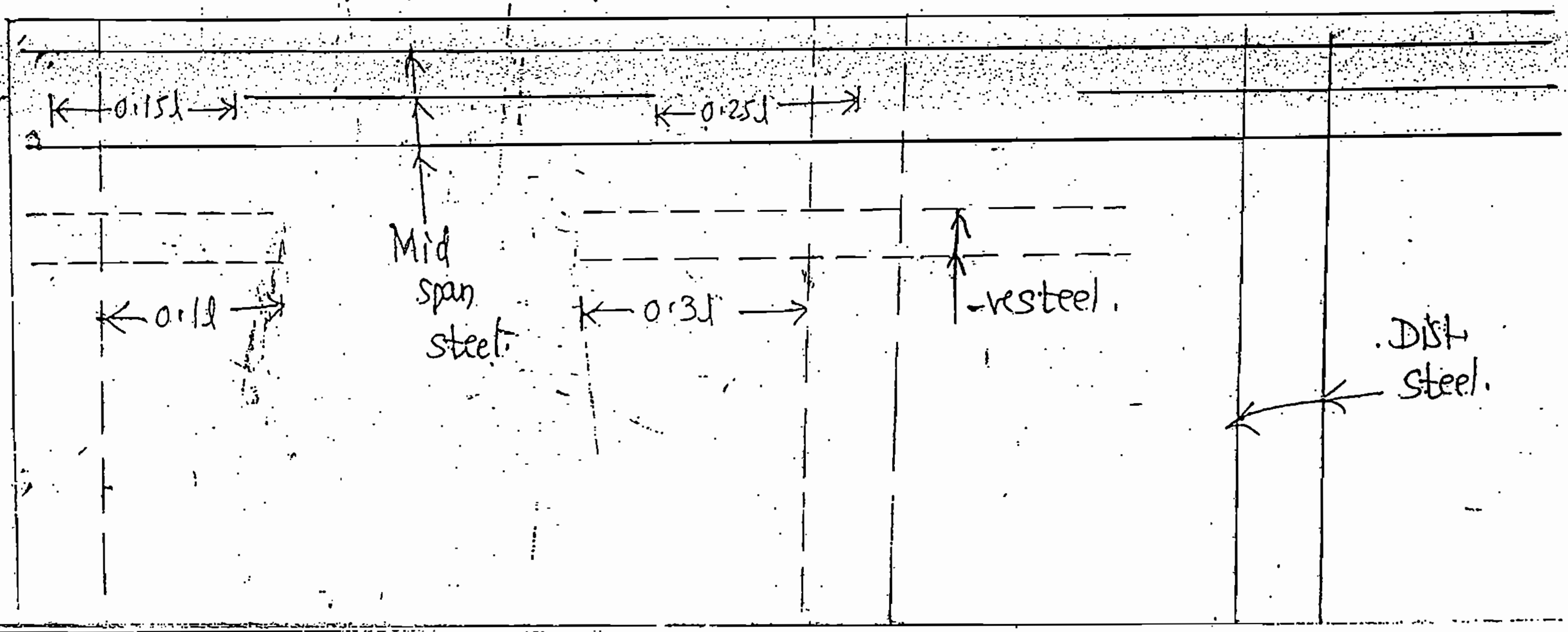
X2X2

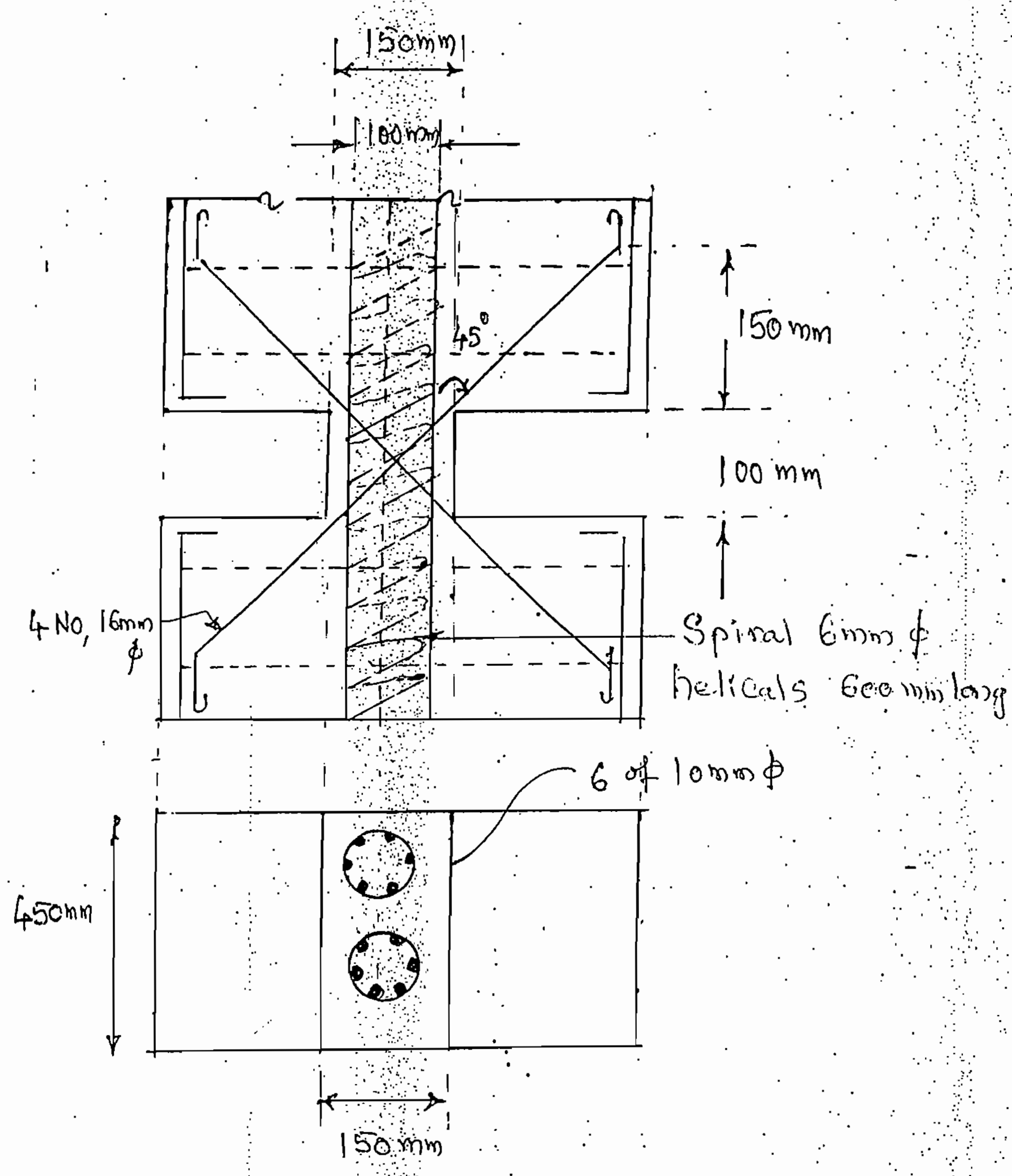


X3X3

One-way Continuous Slab:







RCC HINGE AT COLUMN BASE

Eg:-1 Type-I Roof Truss

Line diagram of a roof truss with External load and forces in each member along with nature are shown in figure. Design the various members of the roof truss along with their end connections with gusset plate [Welded ⓞ Bolted].

Also design the Supports consisting of shoe angles and bearing plate for the support reaction.

Also design Anchor bolts for an uplift force of 15 kN at each support. Take M20 Concrete for the Column. The right support may be considered as anchoring with sliding provision. The left support may be considered as only anchoring support.

Draw to a suitable scale,

(i) Elevation of truss greater than half span,
(ii) Enlarged view of apex joint of the truss,
(iii) Enlarged view of the left support joint.

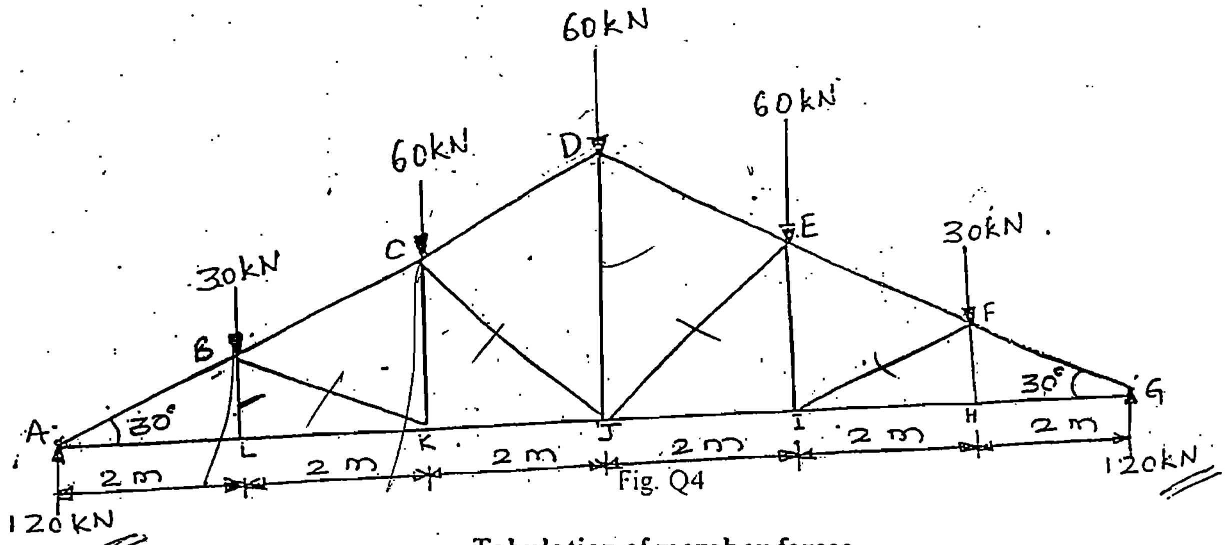


Fig. Q4

Tabulation of member forces

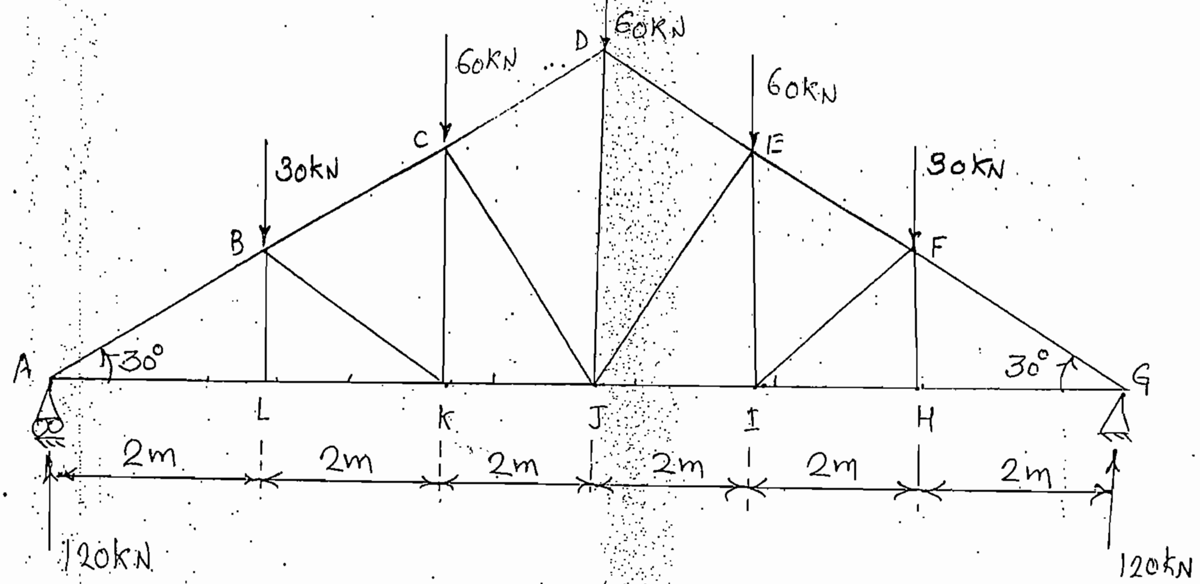
Members	Length (m)	Force (kN)	Nature of Force
AB, GF	2.31	240.00	Compression
BC, FE	2.31	210.00	Compression
CD, ED	2.31	160.04	Compression
AL, GH	2.00	207.84	Tension
LK, HI	2.00	207.84	Tension
KJ, IJ	2.00	181.82	Tension
BL, FH	1.154	0.00	-
BK, FI	2.31	30.00	Compression
CK, EI	2.31	15 kN	Tension
CJ, EJ	3.05	66.05	Compression
DJ	3.46	60.00	Compression

MODULE # 2.

ROOF TRUSS.

Problem # 1. The line diagram of a roof truss with internal loads and forces in each member are shown in the figure. Design the various members of the roof truss along with their end connection using property class 5.6 bolts.

Also design the bearing plate at the support for the reaction and anchor bolts for an uplift force of 15 kN.



(2010 scheme - Jun 17)

★ Member forces are given:

Members	Length (m)	Force (KN)	Nature
AB, GF	2.31	210.00	Compression
BC, FE	2.31	210.00	Compression
CD, ED	2.31	160.04	Compression
AL, GH	2.00	207.84	Tension
LK, HI	2.00	207.84	Tension
KJ, IJ	2.00	181.82	Tension
BL, FH	1.15	0.00	——
BK, FI	2.31	30.00.	Compression
CK, EI	2.31	15.00	Tension
CJ, EJ	3.05	66.00	Compression
DJ	3.46	60.00	Compression

Sol^n: ***Note*** :

(1) For top and bottom chord members → Use Double angle

(2) For inner members → Use Single angle

(3) Use at least 2 bolts

(4) Gusset plate thickness must be Same and uniform thro' out.

Step # 1: Design of top chord member:

Select maximum force = 240 kN (Compression)

∴ Factored force = 1.5 * 240 = 360 kN

(Factor of Safety)

Length 'l' = 2.31 m.

(a) f_{cd} = design Compressive Stress

* Assume f_{cd} (30 to 120 N/mm^2)

Assume f_{cd} = 110 N/mm^2.

$$\text{Area reqd} = \frac{\text{force}}{f_{cd}} = \frac{360*10^3 \text{ N}}{110} = 3272.73 \text{ mm}^2.$$

* Boost this area by 15 %.

∴ Area reqd = 1.15 * 3272.73 = 3763.64 mm^2

= 37.64 Cm^2.

from Steel tables,

Try 2 No' of ISA 100×100×10 mm So that

$r_x = r_z$ = 3.05 Cm = 30.5 mm,

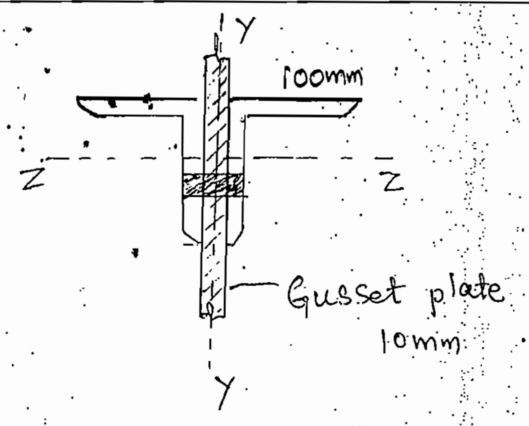
r_{yy} = 3.85 Cm = 38.5 mm

∴ r_{min} = 30.5 mm.

Area = 2* 19.03 = 38.06 Cm^2

= 3806 mm^2

(b) effective length (l_e):



$l = 2.31\ m$.

$l_e = 0.85\, l = 0.85 * 2.31$

$l_e = 1.964\ m = 1964\ mm$

$$\text{Slenderness ratio} = \lambda = \frac{l_e}{r_{min}} = \frac{1964}{30.5} = 64.38$$

Buckling class — c, Page 42, IS 800-2007,

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

f_{cd}	f_y ($= 250\ N/mm^2$)
60	168
64.38	?
70	152

} Table 9 (c) Pg 42.

for 64.38,

$$f_{cd} = 168 + \left(\frac{64.38 - 60}{70 - 60}\right)(152 - 168)$$

$$f_{cd} = 160.99\ N/mm^2$$

Design Compressive Strength $= P_d = A_c \cdot f_{cd}$

(by page 34 @ IS 800-2007)

$P_1 = 3806 * 160.99 = 612.73\ kN > 360\ kN$, Safe.

(C) Design of Connection:

Using M_{20}, property class 5.6 bolts,

(i) Shear Strength:

Assume the threads to be in shear plane,

n_n = number of shear planes with threads intercepting shear planes = 2

n_s = number of shear planes with out the threads intercepting shear planes = 0

A_{nb} = nett tensile c/s area of bolt = $\frac{\pi}{4} d^2 * 0.78$

$$= \frac{\pi}{4}(20)^2 * 0.78 = 245.04 \ mm^2$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left[\frac{f_u}{\sqrt{3}}\left(n_n \cdot A_{nb} + n_s * A_{sb}\right)\right]$$

where, V_{dsb} = design shear capacity of bolt

V_{nsb} = Nominal shear capacity of bolt

γ_{mb} = Partial factor of safety = 1.25

f_u = characteristic ultimate tensile stress

A_{nb} = Nett tensile cross sectional area of 1 bolt

A_{sb} = shank c/s area of 1 bolt

$$\therefore V_{dsb} = \frac{1}{1.25}\left\{\frac{500}{\sqrt{3}} * \left(2 * 245.04 + 0\right)\right\}$$

$$V_{dsb} = 113.18 * 10^{3} \text{ N} = 113.18 \text{ kN}.$$

(ii) Bearing Strength:

d_o = Outer dia = $d + (1.5\text{mm to } 2\text{mm}) = (d+2)$ mm

$d_o = (20 + 2) = 22$ mm

$e = 1.70 * d_o = 1.70 * 22 = 37.4 \text{ mm} \simeq 40 \text{ mm}$

p = pitch = $2.5 d = 2.5 * 20 = 50$ mm

$$K_b = \text{least of } \left(\frac{e}{3d_o}\right) \text{ or } \left(\frac{p}{3d_o} - 0.25\right) \text{ or } \left(\frac{f_{ub}}{f_u}\right) \text{ or } 1$$

$$K_b = \text{least of } \left(\frac{40}{3*22}\right) \text{ or } \left(\frac{50}{3*22} - 0.25\right) \text{ or } \left(\frac{500}{410}\right) \text{ or } 1$$

K_b = least of 0.61 or 0.51 or 1.22 or 1

$\therefore K_b = 0.51.$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left[2.50 * k_b * d * t * f_u\right)$$

V_{dpb} = design shear strength of 1 bolt in bearing

V_{npb} = Nominal shear strength of 1 bolt in bearing

$$V_{dpb} = \frac{1}{1.25}\left[2.50 * 0.51 * 20 * \underset{\text{angle thickness}}{10} * 410\right] = 83.64 * 10^3 N$$

V_{dpb} = 83.64 KN.

∴ Bolt Value = least of V_{dsb} or V_{dpb}

∴ Bolt Value = least of 113.18 KN or 83.64 KN

∴ Bolt Value = 83.64 KN.

$$\therefore \text{No. of Bolts} = \frac{\text{factored load}}{\text{Bolt value}} = \frac{360.00}{83.64}$$

No. of Bolts = 4.30 Say 5 bolts.

Hence provide 2 ISA 100 x 100 x 10mm for top chord members.

Step #2: Design of Bottom chord members:

[Members AL, LK, KJ]

Select maximum force = 207.84 kN (Tension)

Factored force = 1.50 * 207.84 = 311.76 kN

(factor of safety)

length 'l' = 2.00m

(a) Area reqd = $A_{reqd} = \dfrac{\text{Load} * \gamma_{mo}}{f_y}$

Where, γ_{mo} = partial factor of safety against yield and buckling = 1.10

f_y = yield stress = 250 N/mm²

$\therefore (A)_{reqd} = \dfrac{311.76 * 10^3 * 1.10}{250} = 1.372 * 10^3 \text{ mm}^2$

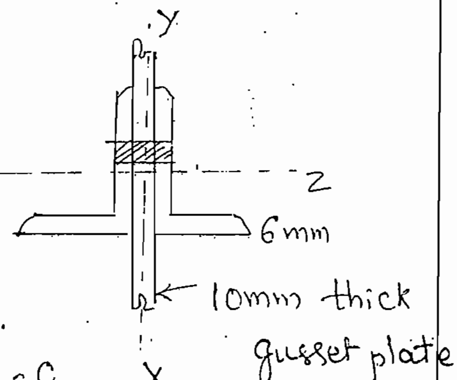
$= 1372 \text{ mm}^2$.

Increase the above area by 15 %,

$(A)_{reqd} = 1.15 * 1372 = 1577.50 \text{ mm}^2$.

From steel tables, try 2 ISA 100x100x6mm

$\therefore (A)_{avail} = 2 * 11.67 = 23.34\ cm^2 = 2334\ mm^2.$



(b) Design of Connection:

* Assume threads in shear plane.

$\therefore n_n = 2,\ n_s = 0.$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}} \left[\frac{f_u}{\sqrt{3}} \left(n_n * A_{nb} + n_s * A_{sb} \right) \right]$$

$$\therefore V_{dsb} = \frac{1}{1.25} \left[\frac{500}{\sqrt{3}} \left(2 * 245.04 \right) \right] = 113.18 * 10^3\ N$$

$$V_{dsb} = 113.18\ KN.$$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}} \left[2.50 * k_b * d * t * f_u \right]$$

angle thickness

$$V_{dpb} = \frac{1}{1.25} \left[2.50 * 0.513 * 20 * 6 * 410 \right]$$

$$V_{dpb} = 50.48 * 10^3\ N = 50.48\ KN.$$

Bolt Value = least of V_{dsb} and V_{dpb} = 50.48 kN

No of bolts = $\frac{\text{Force}}{\text{Bolt Value}} = \frac{311.76}{50.48} = 6.18$ Say 7.

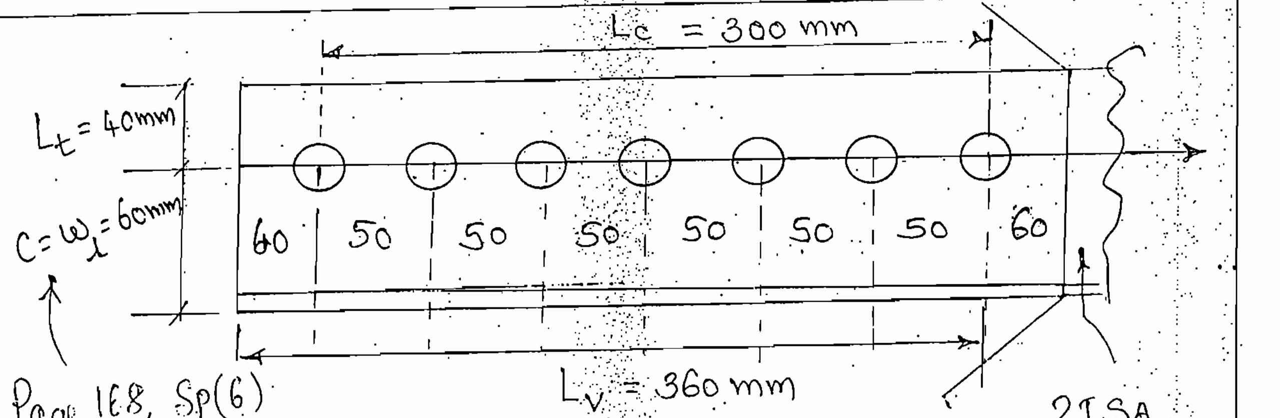


fig 1

(c) check for rupture: (Page 33 of IS 800-2007)

ω = 100mm (out standing leg), t = 6mm.

L_c = 300 mm, $b_s = \omega + \omega_l - t = 100 + 60 - 10 = 150$ mm

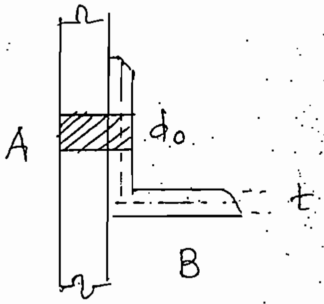
$$\boxed{\beta = 1.40 - 0.076\left(\frac{\omega}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)} \geq 0.70 \quad \leq 1.40$$

$$^{*}0.70 \leq \beta < 1.40^{*}$$

$$\beta = 1.40 - 0.076\left(\frac{100}{06}\right)\left(\frac{250}{410}\right)\left(\frac{150}{300}\right) = 1.014.$$

$$A_{go} = \left(B - \frac{t}{2}\right) * t = \left(100 - \frac{6}{2}\right) * 6$$

$$= 582 \text{ mm}^2$$



$$A_{nc} = \left(A - d_o - \frac{t}{2}\right) * t$$

$$= \left(100 - 22 - \frac{6}{2}\right) * 6 = 450 \text{ mm}^2.$$

T_{dn} = rupture strength = $2\left\{\frac{0.9 A_{nc} \cdot f_u}{\gamma_{m1}} + \beta \cdot \frac{A_{go} \cdot f_y}{\gamma_{mo}}\right\}$

For double angle.

A_{nc} = effective nett area

A_{go} = nett area of out standing leg

γ_{m1} = Partial Safety factor for ultimate stress = 1.25

γ_{mo} = Partial Safety factor against yield & buckling = 1.10

$$T_{dn} = 2\left\{\frac{0.9 * 450 * 410}{1.25} + \frac{1.014 * 582 * 250}{1.10}\right\}$$

$T_{dn} = 533.93 * 10^3 N = 533.93 \text{ KN} > 311.76 \text{ KN}$

Safe.

(d) check for block shear: (Page 33 of IS 800 - 2007)

$L_v = 360 \text{ mm}$, $L_t = 35 \text{ mm}$ (L_v & L_t from fig 1)

A_{vg} = gross sectional area in shear

$= L_v * t = 360 * 6 = 2160 \text{ mm}^2$

A_{tg} = gross sectional area in tension

$= L_t * t = 35 * 6 = 210 \text{ mm}^2$

A_{vn} = nett shear cross sectional area along the force line

$$A_{vn} = A_{vg} - (\overset{\text{No of holes in direction of force}}{n} - 0.5)\, d_0 * t$$

$$A_{vn} = 2160 - (7 - 0.5) * 22 * 6 = 1302\ mm^2$$

A_{tn} = Nett c/s area in tension ⊥er to direction of force

$$A_{tn} = A_{tg} - \left(\frac{d_0}{2}\right) t = 210 - \frac{22}{2} * 6 = 144\ mm^2$$

$$T_{db} = \underbrace{2}_{\text{double angle}} \left[\frac{A_{vg} \cdot f_y}{\sqrt{3}\ \gamma_{m0}} + \frac{0.9\ A_{tn} \cdot f_u}{\gamma_{m1}} \right] > \text{load}$$

$$T_{db} = \underbrace{2}_{\text{double angle}} \left[\frac{0.9 \cdot A_{vn} \cdot f_u}{\sqrt{3}\ \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}} \right] > \text{load}$$

$$T_{db} = 2 \left\{ \frac{2160 * 250}{\sqrt{3} * 1.10} + \frac{0.9 * 144 * 410}{1.25} \right\} = 651.88\ kN > 311.76\ kN$$

Safe

and

$$T_{db} = 2 \left\{ \frac{0.9 * 1302 * 410}{\sqrt{3} * 1.25} + \frac{1 * 210 * 250}{1.10} \right\} = 539.26\ kN > 311.76\ kN$$

Hence provide 2 ISA 100 * 100 * 6 mm.

Step # 3 : Design of Inner Compression member

[Members BK, CJ and DJ]

Taking maximum force = 66.05 KN (Compression)

Factored force = 1.5 * 66.05 = 99.07 KN

Max length 'l' = 3.46 m = 3460 mm

(a) Assume f_{cd} = 50 N/mm² [Assume f_{cd} b/w 30 to 120 N/mm²]

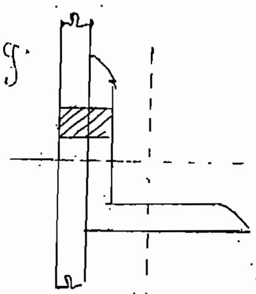
$$(\text{Area})_{\text{reqd}} = \frac{\text{Force}}{f_{cd}} = \frac{99.07 * 10^3 \text{ N}}{50} = 1981.4 \text{ mm}^2$$

$$(\text{Area})_{\text{reqd}} = 19.81 \text{ cm}^2$$

Try Single angle ISA 100 * 100 * 10 mm.

$(\text{area})_{\text{Available}}$ = 19.03 cm², r_{min} = 1.94 cm = 19.4 mm (r_{vv})

Here, Load is acting thro' one leg.



Using Page 48 of IS 800-2007

★ from table # 12, Bolts more and

Hinged Condition, $K_1 = 0.70$, $K_2 = 0.60$, $K_3 = 5$,

$l_e = 0.85\, l = 0.85 * 3.46 = 2.94\,m = 2940\,mm$

$r_w = 19.4$, $\epsilon = 1$, $E = 2.1 * 10^5\ N/mm^2$, $b_1 = b_2 = 100\,mm$

$t = 10\,mm$,

$$\lambda_{vv} = \frac{\left(l/r_{vv}\right)}{\epsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{3460}{19.4}\right)}{1 * \sqrt{\frac{\pi^2 * 2.1*10^5}{250}}} = 1.958$$

$$\lambda_{\phi} = \frac{\left(\frac{b_1+b_2}{2t}\right)}{\epsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{100+100}{2*10}\right)}{1*\sqrt{\frac{\pi^2 * 2.1*10^5}{250}}} = 0.109$$

Page 48 of IS 800 - 2007

$$\lambda_e = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_{\phi}^2}$$

$$\lambda_e = \sqrt{0.70 + 0.60 * (1.958)^2 + 5 * (0.109)^2}$$

$$\lambda_e = 1.75$$

$\alpha = 0.49 \rightarrow$ Table 7, IS 800, for buckling class - C,

$$\phi = 0.5\left[1 + \alpha\left(\lambda_e - 0.2\right) + \lambda_e^2\right]$$ Page 34 of IS 800-2007,

$$\phi = 0.5\left[1 + 0.49\left(1.75 - 0.2\right) + (1.75)^2\right]$$

$$\phi = 2.411.$$

$$f_{cd} = \frac{\left(f_y / \gamma_{mo}\right)}{\phi + \left[\phi^2 - \lambda_e^2\right]^{1/2}}$$ Page 34 of IS 800-2007

$$f_{cd} = \frac{\left(250/1.10\right)}{2.411 + \left[2.411^2 - 1.75^2\right]^{1/2}} = \frac{227.27}{4.07}$$

$$f_{cd} = 55.85 \text{ N/mm}^2$$

CR

* If we assume "load acting thro' C.G",

then $\lambda = \dfrac{l_e}{r_{min}} = \dfrac{2940}{19.4} = 151.55.$

By table # 9c, f_{cd} λ -- $f_y = 250 \text{ N/mm}^2$

150 -- 59.2

151.55 -- ?

160 -- 53.3

$$f_{cd} = 59.20 + \left(\frac{151.55 - 150}{160 - 150}\right)(53.3 - 59.2)$$

$$= 58.28 \ N/mm^2$$

Load $P_d = f_{cd} * A_c = 55.85 * 1903 = 106.28 * 10^3 N$

$P_d = 106.28 \ kN > 99.07 \ KN$ Safe.

(b) Design of Connection:

Design for shear:

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left[\frac{f_u}{\sqrt{3}}\left(n_n \cdot A_{nb} + n_s \cdot A_{sb}\right)\right]$$

$n_n = 1, \quad n_s = 0. \quad f_u = 500 N/mm^2, \quad \gamma_{mb} = 1.25$

$$V_{dsb} = \frac{1}{1.25}\left[\frac{500}{\sqrt{3}}\left(1 * \frac{\pi}{4} * (20^2) * 0.78\right)\right]$$

$$V_{dsb} = 56.59 * 10^3 N = 56.59 \ kN$$

Design for bearing Strength.

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left[2.50 * k_b * d * t * f_u\right]$$

$$V_{dpb} = \frac{1}{1.25}\left[2.50 * 0.51 * 20 * 10 * 410\right]$$

$$V_{dpb} = 83.64 * 10^3 \, N = 83.64 \, kN.$$

$\therefore$ Bolt Value = least of 56.59 kN and 83.64 kN

$\therefore$ Bolt Value = 56.59 kN.

$$\text{No of Bolts} = \frac{99.07}{56.59} = 1.75 \simeq 2$$

Hence provide ISA 100 × 100 × 10 mm.

Step #4: Design of "Inner Tension Member":

[Member CK] Force = 15 kN

$\therefore$ factored force = 1.50 * 15 = 22.5 kN

(i) Since the force is very small, provide the least minimum sized angle ISA 50 × 50 × 6 mm.

$\therefore$ A = 5.68 cm^2 = 568 mm^2.

(ii) Design of Connection:

Use M_{12}, Property class 5.6 bolts.

d_0 = dia of hole = 12 + 1 = 13 mm.

(iii) Shear Strength:

$\eta_n = 1, \quad \eta_s = 0, \quad A_{nb} = \frac{0.78\, \pi d^2}{4}$

$A_{nb} = 0.78 * \frac{\pi}{4} * 12^2 = 88.22 \text{ mm}^2$

$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}} \left[\frac{f_u}{\sqrt{3}} \left(\eta_n A_{nb} + \cancel{\eta_s A_{sb}} \right) \right]$

$V_{dsb} = \frac{1}{1.25} \left[\frac{500}{\sqrt{3}} \left(1 * 88.22 \right) \right] = 20.37 * 10^3 \text{ N}$

$V_{dsb} = 20.37 \text{ KN}$

(iv) Bearing Strength:

$e = 1.7 d_0 = 1.7 * 13 = 22.1 \text{ mm} \simeq 25 \text{ mm}$

$p = 2.5 d = 2.5 * 12 = 30 \text{ mm}$

K_b = least of $\left(\frac{e}{3d_0} \right)$ or $\left(\frac{p}{3d_0} - 0.25 \right)$ or $\left(\frac{f_{ub}}{f_u} \right)$ or 1

K_b = least of $\left(\frac{25}{} \right)$ or $\left(\frac{30}{} - 0.25 \right)$ or $\left(\frac{500}{} \right)$ or 1

k_b = least of 0.64 or 0.52 or 1.22 or 1.

$\therefore k_b = 0.52$.

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left[2.50 * k_b * d * t * f_u\right]$$

$$V_{dpb} = \frac{1}{1.25}\left[2.50 * 0.52 * 12 * 6 * 410\right]$$

$$V_{dpb} = 30.7 * 10^3 \text{ N} = 30.7 \text{ KN}$$

$\therefore$ Bolt Value = least of 20.37 KN or 30.7 KN

$\therefore$ Bolt Value = 20.37 KN.

$\therefore$ No of Bolts $= \dfrac{22.50}{20.37} = 1.10 \simeq 2$.

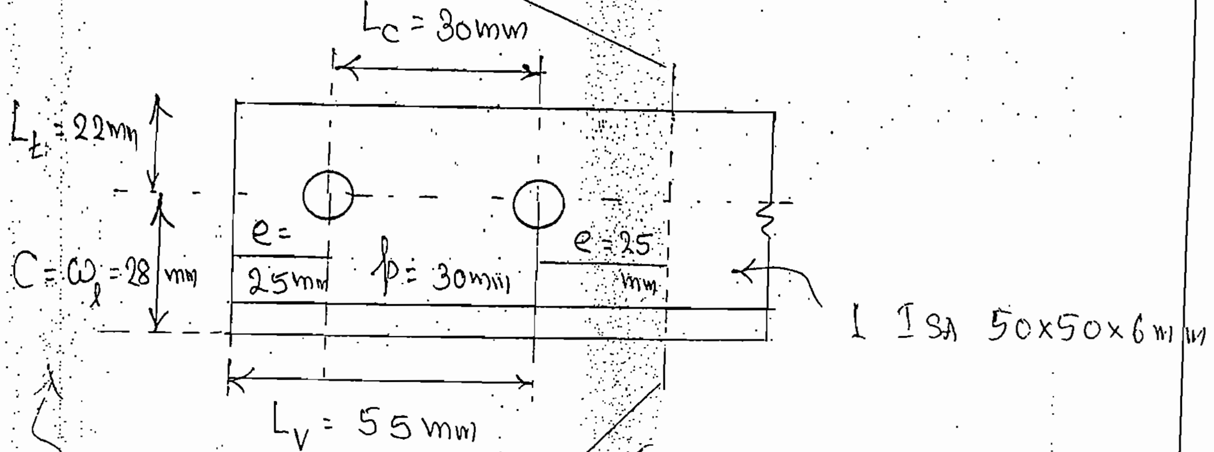


Fig 2

Page 168, Sp-6.

(c) check for rupture : (Page 33 of IS 800 - 2007)

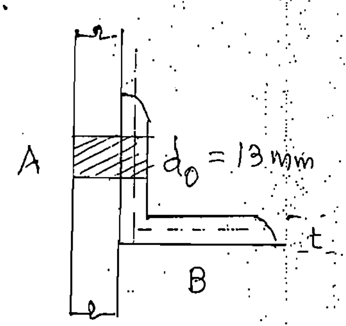
$\omega = 50\,mm$ (out standing leg), $t = 6\,mm$

$L_c = 30\,mm$, $b_s = \omega + \omega_{\ell}^{c} - t = 50 + 28 - 6 = 72\,mm$

$$\beta = 1.40 - 0.076\left(\frac{\omega}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)$$

$$\beta = 1.40 - 0.076\left(\frac{50}{6}\right)\left(\frac{250}{410}\right)\left(\frac{72}{30}\right) = 0.47$$

Since $0.7 \leq \beta < 1.40$, $\beta = 0.70$.



$$A_{go} = \left(B - \frac{t}{2}\right) * t = \left(50 - \frac{6}{2}\right) * 6$$

$$A_{go} = 282\,mm^2$$

$$A_{nc} = \left(A - d_0 - \frac{t}{2}\right)t = \left(50 - 13 - \frac{6}{2}\right) * 6 = 204\,mm^2$$

$$T_{dn} = \left\{\frac{0.9\, A_{nc} \cdot f_u}{\gamma_{ml}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{mo}}\right\}$$

$$T_{dn} = \left\{\frac{0.9 * 204 * 410}{1.25} + \frac{0.70 * 282 * 250}{1.10}\right\}$$

$T_{dn} = 105.08 \times 10^3\,N = 105.08\,kN > 92\,kN$ Safe

(d) check for Block shear: (Page 33 of IS 800 : 2007)

$L_v = 55\,mm$, $L_t = 22\,mm$ (L_v and L_t from fig 2)

$A_{vg} = L_v * t = 55 * 6 = 330\,mm^2$

$A_{tg} = L_t * t = 22 * 6 = 132\,mm^2$

No of holes in direction of force

$A_{vn} = A_{vg} - (n - 0.5) * d_0 * t.$

$A_{vn} = 330 - (2 - 0.5) * 13 * 6 = 213\,mm^2$

$A_{tn} = A_{tg} - \left(\frac{d_0}{2}\right) * t = 132 - \frac{13}{2} * 6 = 93\,mm^2$

$$T_{db} = \left[\frac{A_{vg} \cdot f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9\,A_{tn} \cdot f_u}{\gamma_{m1}}\right] > \text{load}$$

$$T_{db} = \left[\frac{330 * 250}{\sqrt{3} * 1.10} + \frac{0.9 * 93 * 410}{1.25}\right] = 70.75 * 10^3\,N$$

$= 70.75\,kN > 22.5\,kN$ (Safe)

and

$$T_{db} = \left[\frac{0.9 * A_{vn} * f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}\right] > \text{load}$$

$$T_{db} = \left[\frac{0.9 * 213 * 410}{\sqrt{3} * 1.25} + \frac{132 * 250}{1.10}\right] = 66.30 * 10^3\,N$$

$T_{db} = 66.30$ KN > 22.5 KN (Safe).

Hence provide ISA $50 \times 50 \times 6$ mm.

Step # 5: Design of Bearing plate:

Support reaction $= \frac{\text{Total load}}{2} = \frac{240}{2} = 120$ KN

$\therefore$ factored load $= 1.50 * 120 = 180$ kN

Using M_{20} Concrete,

Bearing pressure of Concrete $= 0.45 f_{ck}$

$= 0.45 * 20 = 9\ N/mm^2$

Area of Bearing plate $= \frac{\text{load}}{\text{Bearing pr.}}$

$= \frac{180*10^3}{9} = 20*10^3\ N/m^2$

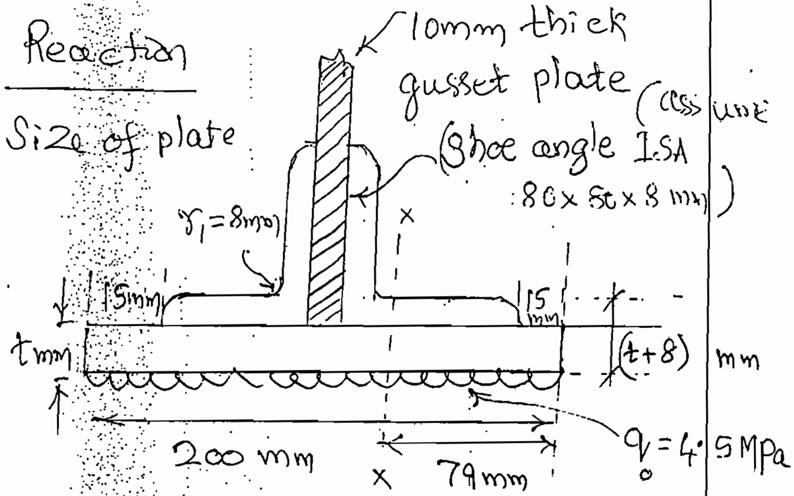
Providing a Square plate,

$A = b^2 = 20*10^3\ mm^2$

$b = \sqrt{20000} = 141.42$ mm Say 145 mm.

However provide bearing plate of size 200×200 mm 2

Upward pressure $= q_0 = \dfrac{\text{Reaction}}{\text{Size of plate}}$



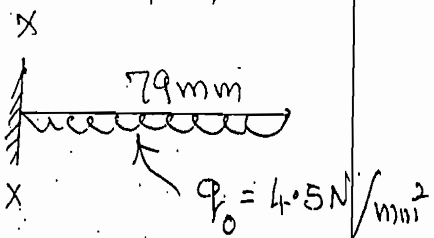
$$q_0 = \frac{180 * 10^3 \text{ N}}{200 * 200}$$

$$q_0 = 4.50 \text{ N/mm}^2$$

79mm
$80 + 15 - t - r_1$
$80 + 15 - 8 - 8$
$= 79$ mm

$$M_{xx} = \frac{wl * l}{2}$$

$$= 4.5 * 79 * \frac{79}{2}$$



$$M_{xx} = 14.04 * 10^3 \text{ N-mm}$$

Using $M = \sigma_b * Z$, — Section modulus

$$M = \left(\frac{f_y}{\gamma_{mo}}\right) * \left(\frac{1}{6} bd^2\right)$$

b = 1mm strip

$$M = \left(\frac{250}{1.10}\right) * \left(\frac{1}{6} * 1.00 * (t+8)^2\right)$$

$$14.04 * 10^3 = 37.88 * (t+8)^2$$

$10^3 * 14.04 = 37.88\ (t+8)^2$

$\therefore\ (t+8)^2 = 370.66 \Rightarrow t = 19.25 - 8 = 11.25\,mm$

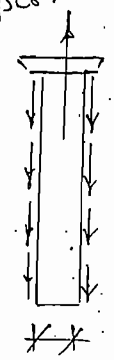
$\therefore\ t = 12\,mm$

$\therefore$ Provide a bearing plate of $200 \times 200 \times 12\,mm$.

<u>Step # 6.</u> Design of Anchor bolt:

Given: Uplift force = 15 kN at each Support.

No: of Supports = 2



20mm
assume

force on each bolt = $\frac{15}{2} = 7.5\,kN$

Factored force = $1.5 * 7.5 = 11.25\,kN$

Assume 20 mm ϕ anchor bolt

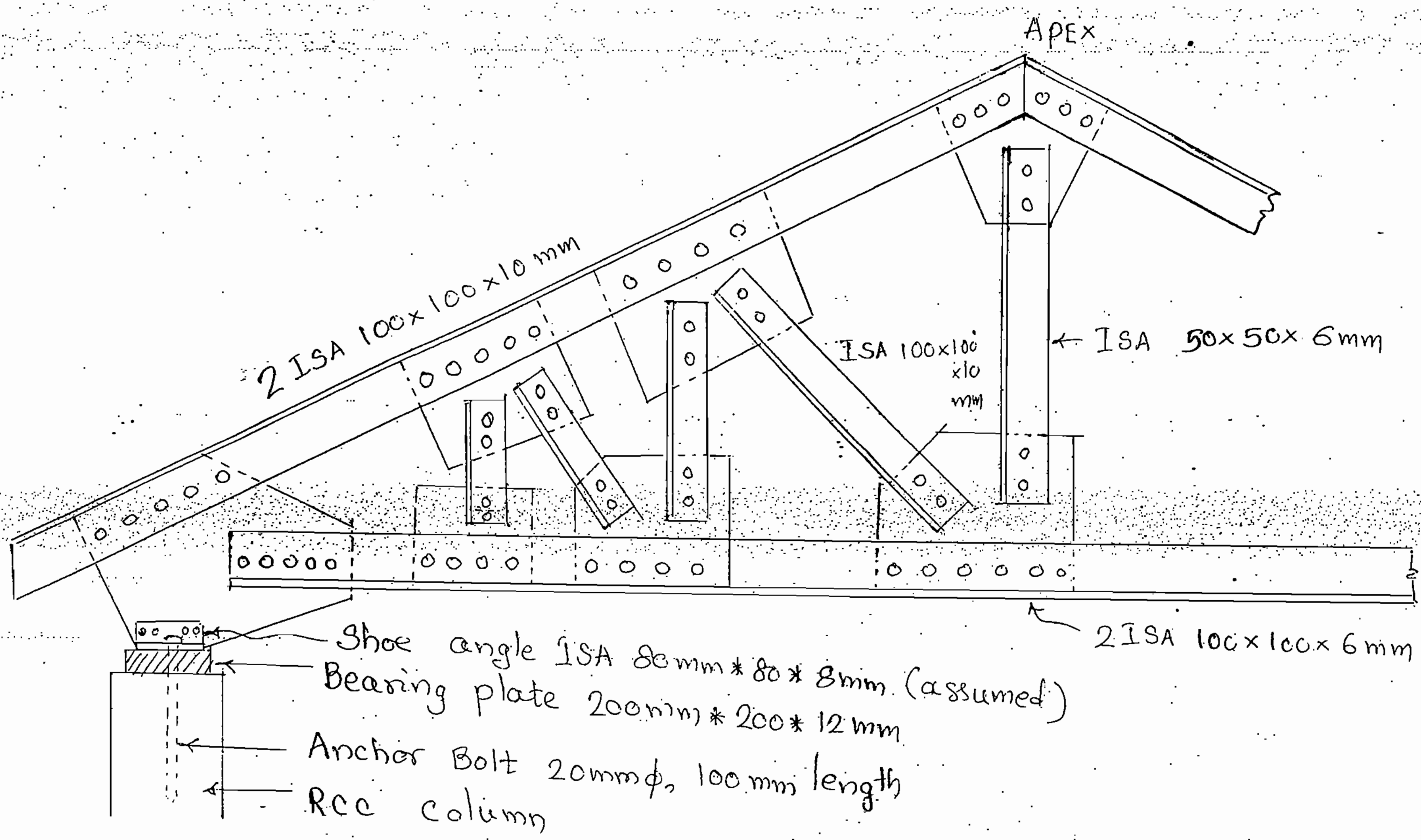
force = (Circumference of bolt) * length * Bond Stress

$11.25 * 10^3\,N = (\pi * 20) * l * (\underset{M_{20}}{1.20\,N/mm^2} * \underset{60\%\ \text{increase}}{1.60})$

$11.25 * 10^3 = 120.64 * l$ IS 456 - page 43

$\Rightarrow l = 93.25\,mm$ Say 100 mm.

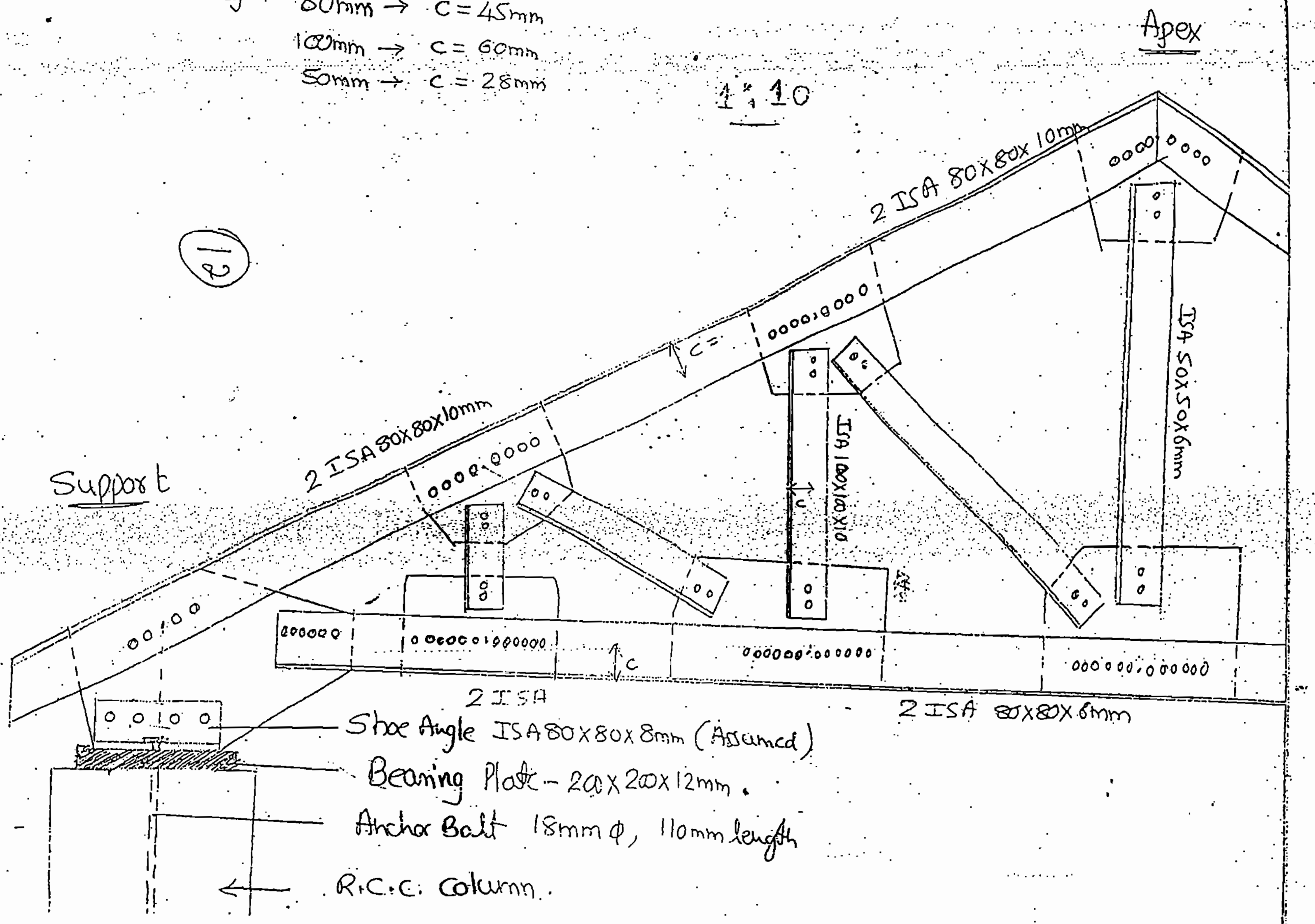
Provide two anchor bolts of 20mm dia, 100 mm length.

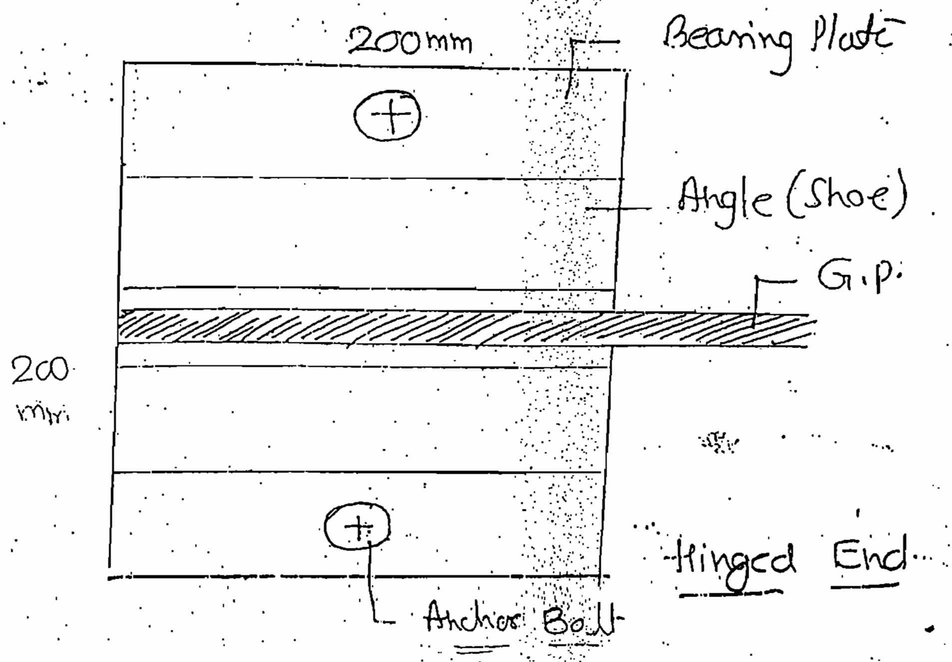

Apex
2 ISA 100×100×10 mm
ISA 100×100 ×10 mm
← ISA 50×50×6mm
2 ISA 100×100×6 mm
Shoe angle ISA 80mm * 80 * 8mm (assumed)
Bearing plate 200mm * 200 * 12 mm
Anchor Bolt 20mmϕ, 100 mm length
RCC column

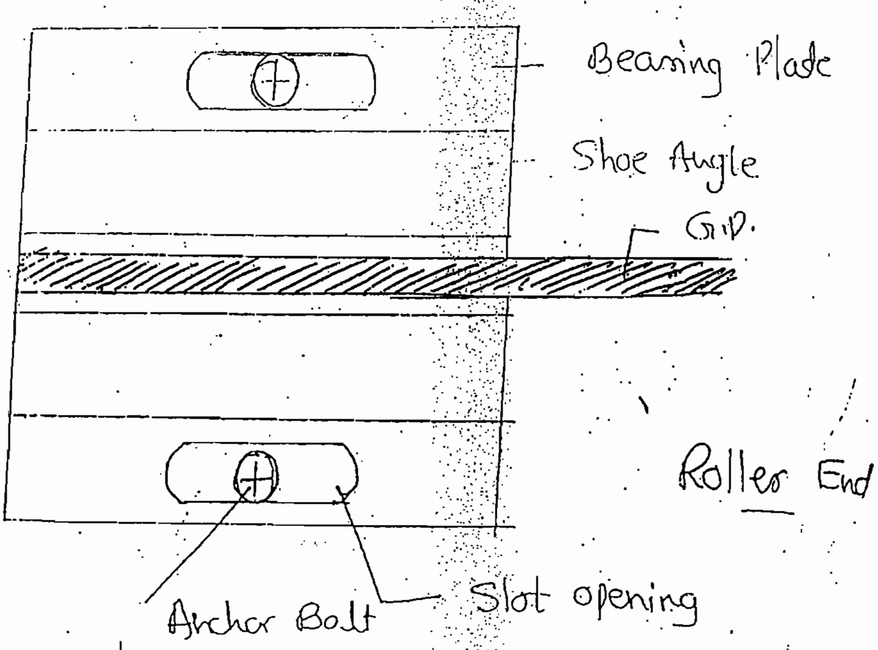
Roof Truss

12

Leg → 80mm → c = 45mm
100mm → c = 60mm
50mm → c = 28mm




200mm
Bearing Plate
Angle (Shoe)
G.P.
200 mm
Hinged End
Anchor Bolt


Bearing Plate
Shoe Angle
G.P.
Roller End
Anchor Bolt
Slot opening

1 : 5

PART B: WELDED PLATE GIRDER

Problem # 1. Design a welded plate girder for an effective span of 14m. The imposed load on the girder consists of a udl of 45 kN/m together with two point loads of 400 kN placed at a distance of 3m on either side of mid span of the girder. Design: (i) Mid span cross section (ii) Curtailment of flange plates (iii) Intermediate stiffeners and (iv) End bearing & Bearing stiffeners.

Soln: ** Note: Important components to be designed are: (i) Mid span c/s (ii) Curtailment of flange plates (iii) Intermediate stiffeners (iv) End and bearing stiffeners.

Step # 1. Design of mid span cross section:

(a) Load calculation:

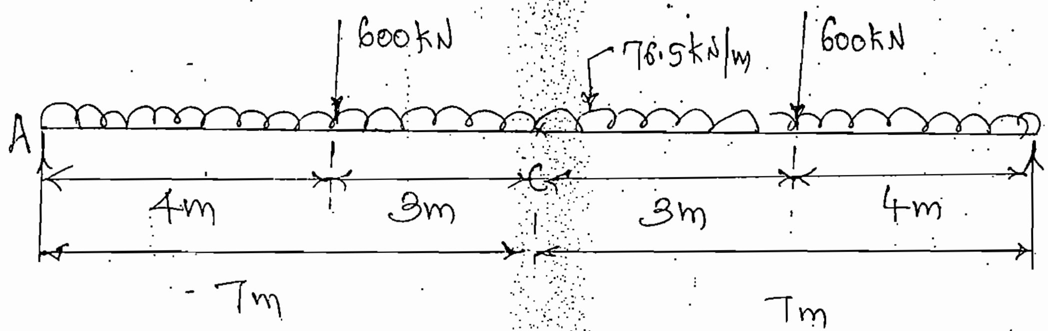
Self weight of girder $= \dfrac{\text{Total load}}{250} = \dfrac{400*2+45*14}{250}$

$= 5.72 \text{ kN/m} \simeq 6 \text{ kN/m}$

$\therefore$ Total load as udl $= (45+6) = 51 \text{ kN/m}$.

Multiply all loads by 1.5 to get factored loads.

$\therefore \; 400 * 1.5 = 600\ \text{kN}$ and $51 * 1.5 = 76.5\ \text{kN/m}$



Reaction = Shear force (factored) = $V = \dfrac{\text{Total load}}{2}$

$$\therefore \; V = \frac{600 * 2 + 76.5 * 14}{2} = 1135.5\ \text{kN}$$

Max BM at mid Span

$$= M = 1135.5 * 7 - 600 * 3 - 76.5 * \frac{7^2}{2}$$

$$M = 4274.25\ \text{KN-m}.$$

(b) Girder dimensions:

$$d_w = \left[\frac{K * M}{f_y}\right]^{0.33}, \quad K = \frac{d}{t_w} \text{ ratio}$$

regression Co-eff

[Assume b/w 150 to 200]

Assume $K = 180$.

$$d_w = \left[\frac{180 * 4274.25 * 10^6}{250} \right]^{0.33} = 1352.4 \text{ mm}$$

Take $d_w = 1400$ mm.

Web thickness $= t_w$: From table # 2, page 18 of IS 800 - 2007,

$$\left(\frac{d}{t_w} \right) \begin{matrix} < 84\,\epsilon \\ < 105\,\epsilon \end{matrix} \Bigg\} \text{ for plastic or Compact Condition}$$

where $\epsilon = \left[\frac{250}{f_y} \right]^{1/2} = 1$ $\because f_y = 250$ MPa.

Using $\frac{d}{t_w} = 84\,\epsilon$; $\boxed{\epsilon = \text{yield stress ratio}}$

$$\Rightarrow \frac{1400}{t_w} = 84 * 1 \quad \text{or} \quad t_w = \frac{1400}{84} = 16.67$$

Say 20mm.

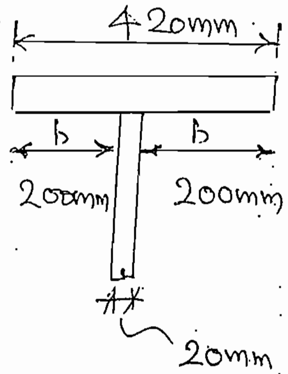
$\therefore$ Provide Web of 1400×20 mm Size.

(ii) Flange dimensions:

Width of flange $= b_f = 0.3\, d_w$ *

$\therefore b_f = 0.3 * 1400 = 420$ mm.

$\therefore\ b = \ = 200\ mm.$



Again, by Table # 2 of IS 800 : 2007, Page 18,

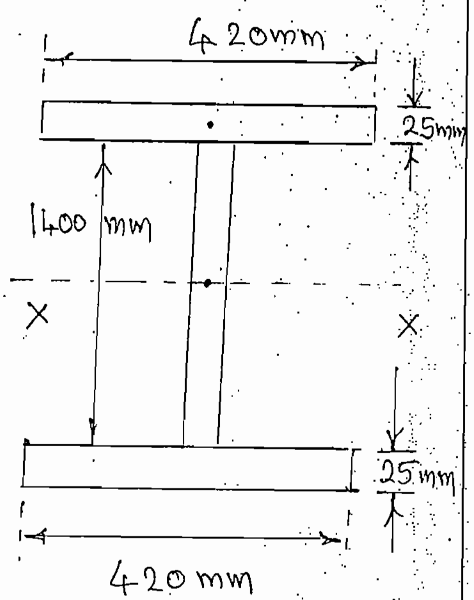
$$\left(\frac{b}{t_f}\right) \begin{matrix} \leq 8.4\epsilon \\ \leq 9.4\epsilon \end{matrix} \Bigg\}\ \text{for plastic \& Compact Condition.}$$

Using $\frac{b'}{t_f} = 8.4\epsilon \Rightarrow \frac{200}{t_f} = 8.4\epsilon = 8.4 \quad \because \epsilon = 1$

or $t_f = \frac{200}{8.4} = 23.81\ mm$ Say 25 mm

$\therefore\ t_f = 25\ mm$

$\therefore$ Provide flange $400 \times 25\ mm.$



$$I_{xx} = \frac{BD^3}{12} - 2\left[\frac{bd^3}{12}\right]$$

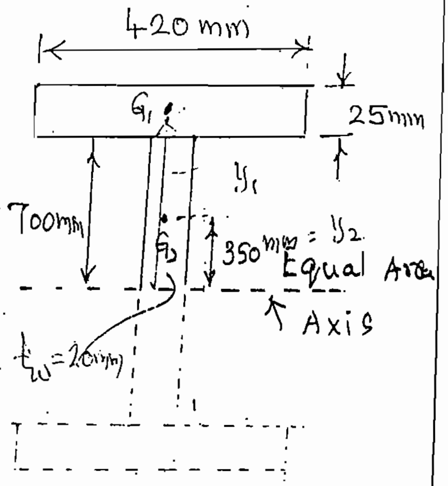
$B = 420\ mm = b_f$

$b = 200\ mm, \quad D = 1450\ mm$

$d = 1400\ mm$

$$I_{xx} = \frac{420 * 1450^3}{12} - 2\left[\frac{200 * 1400^3}{12}\right] = 1.524 * 10^{10}\ mm^4$$

Plastic Section modulus = Z_p



$$Z_p = \sum a\bar{y}$$

$$Z_p = 2\left\{420 * 25 * 712.5 + 20 * 700 * 350\right\}$$

$$Z_p = 24.763 * 10^6 \text{ mm}^3$$

(c) Check for moment of resistance:

As per IS 800 - 2007, Page 53, Clause 8.2.1.2.

Design Bending Strength = $M_d = \dfrac{\beta_b * Z_p * f_y}{\gamma_{m0}}$

★ β_b = Bending moment ratio

For plastic or Compact Condition, take $\beta = \beta_b = 1$.

$$\therefore M_d = \frac{1 * 24.763 * 10^6 * 250}{1.10} = 5.63 * 10^9 \text{ N-mm}$$

$$M_d = 5.63 * 10^3 \text{ KN-m} = 5630 \text{ KN-m} > M$$

$$M_d > M \quad [\because 5630 \text{ KN-m} > 4274.5 \text{ KN-m}]$$

Safe.

** Note: If $M_d < M$, increase flange width

γ_{mo} = ~~yie~~ Partial factor of Safety against yield stress & Buckling

by 20 % and rework Z_p, M_{dd} to ensure $M_d > M$

(d) <u>check for shear resistance:</u> From I.S Code 800 - 2007, pages 59 - 60,

Design shear force $= V_d = \dfrac{V_n}{\gamma_{mo}}$

* V_n = nominal shear strength, V_d = design shear strength

$V_n = V_{cr} = A_v * \tau_b \ (T_b)$

V_{cr} = critical shear strength corresponding to web buckling

A_v = Shear area = $d * t_w = 1400 * 20 = 28000 \ mm^2$

(for welded connection)

$T_b = \tau_b$ = shear stress corresponding to web buckling

* provide vertical stiffners at a spacing

of $\left(\dfrac{c}{d}\right) = 1.50 \rightarrow$ (Max limit 1.50)

$\therefore \dfrac{c}{d} = 1.50$

C = Spacing of transverse stiffner

d = depth of web

clause 8.4.2.2., $\boxed{Pg\ 60}$

$K_v = 5.35 + \dfrac{4}{(c/d)^2}$ for $\dfrac{c}{d} \geq 1.0$

K_v = Shear buckling Co-eff

$K_v = 5.35 + \dfrac{4}{(1.5)^2} = 7.12.$

Again, by IS sec. 2007, Page 12, $E = 2.0 * 10^5$ Mpa

μ = poisson's ratio = 0.3

$$T_{cr,e} = \frac{K_v \cdot \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$$ (Pg 60)

$T_{cr,e}$ = elastic critical shear stress of web

$$T_{cr,e} = \frac{7.12 * \pi^2 * 2*10^5}{12\left(1-0.3^2\right)\left(\frac{1400}{20}\right)^2} = 262.66.$$

λ_w = nominal web slenderness ratio

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\ T_{cr,e}}} = \sqrt{\frac{250}{\sqrt{3} * 262.66}} = 0.74$$

T_b = Shear Stress Corresponding to web buckling

(i) when $\lambda_w \leq 0.8$, $T_b = \frac{f_{yw}}{\sqrt{3}}$

(ii) when $0.8 < \lambda_w < 1.2$,

$$T_b = \left[1 - 0.8\left(\lambda_w - 0.8\right)\right] \frac{f_{yw}}{\sqrt{3}}$$

(iii) when $\lambda_w > 1.2$,

$$T_b = \left(f_{yw}\right) \div \left(\sqrt{3}\ \lambda_w^2\right)$$

IS 800-2007 Page 60

f_{yw} = yield Stress of web

$$\tau_b = \frac{f_{yw}}{\sqrt{3}} = \frac{250}{\sqrt{3}} = 144.34 \text{ N/mm}^2$$

$$V_{cr} = A_v * \tau_b = 28000 * 144.34 = 4041.45 * 10^3 \text{ N}$$

$V_{cr} = 4041.45$ KN (V_{cr} = Critical Shear Strength) $= V_n$

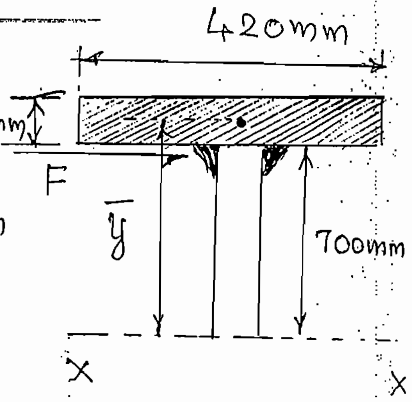
Design Shear force = $V_d = \dfrac{V_{cr}}{\gamma_{mo}} = \dfrac{4041.45}{1.10}$

$V_d = \dfrac{4041.45}{1.10} = 3674.05$ KN $>$ 1135.5 KN (External Shear force)

$V_d >$ External Shear force Hence Safe.

(e) Connection of flange to web:

Horizontal Stress at the Junction $= F = \dfrac{Va\bar{y}}{I_{xx}}$ N/mm



$$F = \frac{(1135.5*10^3)*(420*25)*712.5}{1.524*10^{10}}$$

F = 557.41 N/mm = Force per mm length.

γ_{mw} = Partial factor of safety for weld = 1.25

f_u = Ultimate. Yield Stress = 410 MPa

Equating the above force with strength of the weld per mm length,

$$\star\ 557.41\ N/mm = \left[(0.7\, S) * 1 * \left(\frac{f_u}{\sqrt{3}\,\gamma_{mw}}\right)\right] * 2$$

$$557.41 = \left[0.7\, S * 1\,mm * \frac{410}{\sqrt{3 * 1.25}}\right] * 2$$

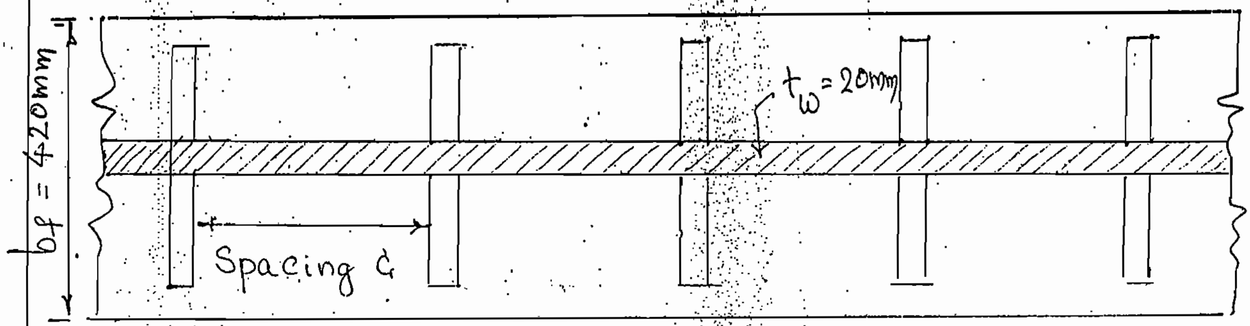
Both sides welding

$$557.41 = 2 * 132.56\, S$$

$$\therefore S = 2.10\ mm.$$

$\therefore$ Provide minimum Size of weld = 5mm.

Step # 2. Design of Intermediate Stiffeners:



(i) Ratio of $\frac{d}{t_w} = \frac{1400}{20} = 70 > 67.$

$\star$ The ratio must not exceed 67.

Hence we must provide Intermediate Stiffeners

(ii) Spacing (C):

Maximum allowable Spacing $C = 1.50\,d$

$$\therefore C = 1.50 * 1400 = 2100 \text{ mm}.$$

$\therefore$ Provide a Spacing of 2100 mm.

(iii). Size of Intermediate Stiffeners:

Ratio of $\left(\frac{c}{d}\right) = \frac{2100}{1400} = 1.50 > \sqrt{2}$.

As per IS 800 - 2007, Page 66 - 67,

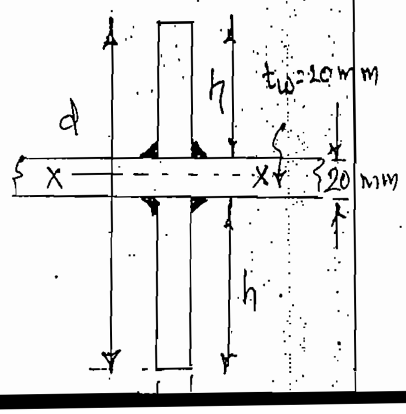
If $\left(\frac{c}{d}\right) > \sqrt{2}$, use $I_s = 0.75\, d\, t_w^3$ ✓

If $\left(\frac{c}{d}\right) < \sqrt{2}$, use $I_s = \frac{1.50\, d^3 t_w^3}{C^2}$

Using $I_s = 0.75\, d\, t_w^3$

$$\Rightarrow I_s = 0.75 * (1400) * (20)^3 = 8.4 * 10^6 \text{ mm}^4$$

$$M.I = I_{xx} = \frac{bd^3}{12} = \frac{b\,(2h+20)^3}{12} = 8.4 * 10^6$$



Using 8 mm thick plate, b = 8 mm

$$\therefore \quad \frac{8\,(2h+20)^3}{12} = 8.40 * 10^6$$

$$\Rightarrow \quad (2h+20)^3 = 12.6 * 10^6$$

$$\Rightarrow \quad (2h+20) = \left(12.6 * 10^6\right)^{1/3} = 232.70$$

$$2h = 212.70 \quad \text{or} \quad h = 106.35 \text{ mm.}$$

Provide I.S. of size 110 × 8 mm on either side of the web.

(iv) Connection of I.S. to the web:

As per IS 800 - 2007, page 67,

$$\text{Force reqd for connection} = \frac{(t_w)^2}{5\, b_s} \; \ldots \text{kN/mm}$$

where b_s = length of stiffener out standing = 110 mm

$$\therefore \quad \text{Force} = \frac{(20)^2}{5 * 110} = 0.727 \text{ kN/mm} = 727.27 \text{ N/mm}$$

* Equating the above force to the strength of the weld per mm,

$$727.27 = \left[0.7 * S * \underset{1\text{ mm}}{l} * \frac{f_u}{\sqrt{3}\,\gamma}\right] * 4$$

Four side welding

$$727.27 = \left[0.7\, S * 1\,\text{mm} * \frac{410}{\sqrt{3} * 1.25}\right] * 4 = 530.24\, S$$

$$\therefore\ S = \frac{727.27}{530.24} = 1.37\ \text{mm}$$

$\therefore$ Provide minimum Size = 3mm Continuous weld.

Step # 3. Curtailment of flange plate:

*** The Bending Moment (M) is maximum at mid-Span and it decreases towards the Supports. Hence, from the economical point of view, we decrease the thickness of the flange plate as the Bending moment (M) decreases which is called Curtailment of flange plate.

*** Note: The flange plate must not be Curtailed by more than 40 % of the flange thickness as at mid Span. Thus, the flange thickness at Supports = $0.6\, t_f$

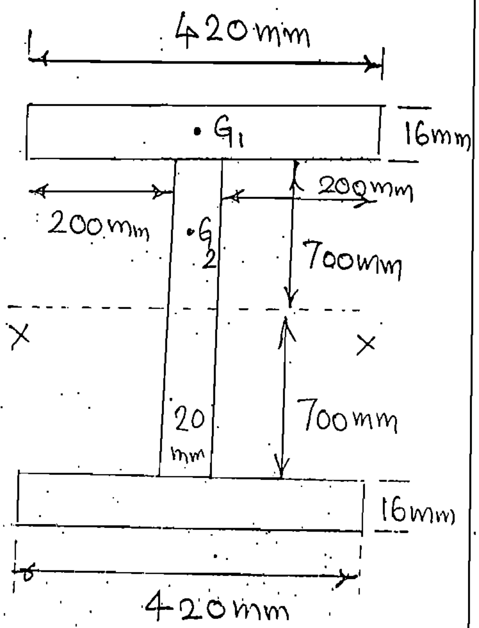
where. t_f = flange thickness at mid Span.

[Standard thickness of plate = 6mm, 8mm, 10mm, 12mm, 14mm, 16mm, 18mm, 20mm, 22mm, 24mm, 26mm]

28 mm, 30 mm, 32 mm, 40 mm].

$0.6\, t_f = 0.6 * 25 = 15\text{ mm}$ Say 16 mm.

★ Reduce the thickness of flange plate from 25 mm to 16 mm.



Plastic Section modulus

$$Z_p = \sum a\bar{y}$$

$$Z_p = 2\left\{(420*16)*\left(700+\frac{16}{2}\right) + (20*700)*\frac{700}{2}\right\}$$

Symmetry @ x-x axis

$$Z_p = 19.32*10^6\ mm^3$$

As per IS 800-2007, Page 53,

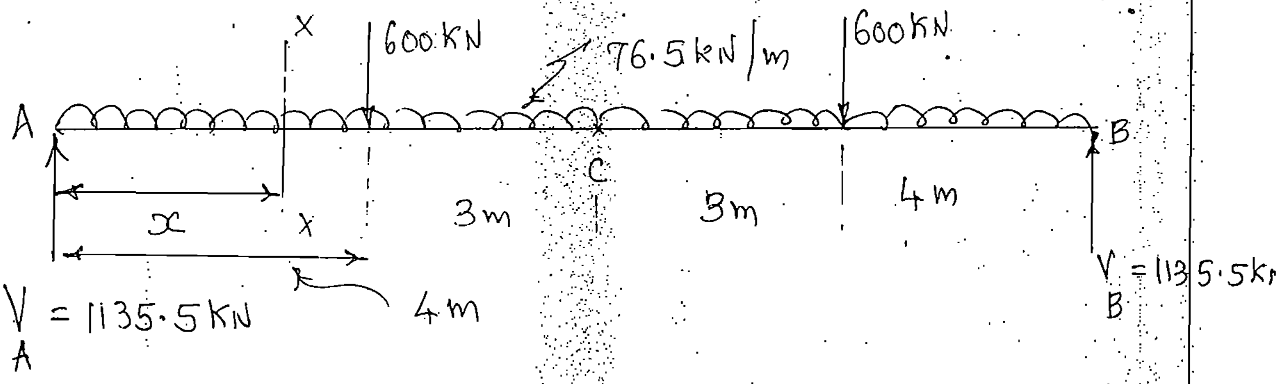
Design Bending Strength $= M_d = \dfrac{\beta_b * Z_p * f_y}{\gamma_{mo}}$

$$\beta_b = 1.$$

$$\therefore M_d = \frac{1 * 19.32*10^6 * 250}{1.10} = 4390.91*10^6\ N\text{-}mm$$

$$M_d = 4390.91\ kN\text{-}m.$$

let 'x' be the distance from the supports where the Curtailment shall end:



$$M_{XX} = V_A * x - 76.5 * x * \frac{x}{2} = 4390.91 \text{ kN-m}$$

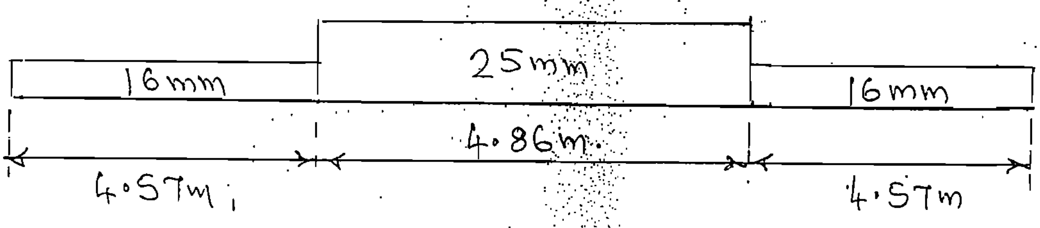
$$\Rightarrow 1135.5x - 76.5\frac{x^2}{2} = 4390.91$$

$$\Rightarrow 76.5x^2 - 2271x + 8781.82 = 0.$$

$$x^2 - 29.69x + 114.80 = 0$$

Solving for x, $x = \dfrac{29.69 \pm \sqrt{(29.69)^2 - 4(114.80)}}{2}$

x = 4.57m from the ends.



Step # 4: Design of End bearing stiffners:

** Note: Bearing Stiffeners are provided under the point load to prevent buckling due to load. If it is provided at the ends for the Support reactions, then it is called the end bearing Stiffeners (EBS). If it is provided at the external point load, then it is called bearing Stiffeners (BS).

i) End Bearing Stiffeners:

Support reaction = 1135.50 kN

Local web load Carrying Capacity:

As per IS 800 - 2007, page 67,

$$F_w = \frac{(b_1 + n_2)\, t_w\, f_{yw}}{\gamma_{m0}}$$

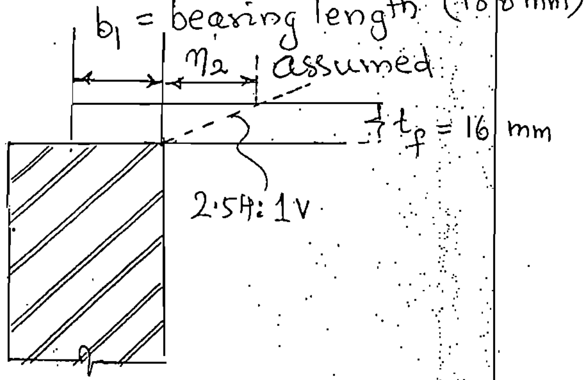
b_1 = Stiff bearing length

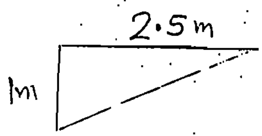
n_2 = length obtained by dispersion thro' the flange to the web junction at a slope of 2.5 H : 1 V to the plane of flange

t_w = web thickness

f_{yw} = yield stress of web = 250 N/mm²

$\eta_2 = 2.5 * 16 = 40$ mm.





$$F_w = \frac{(180 + 40) * 20 * 250}{1.10}$$

$F_w = 1000 * 10^3$ N = 1000 kN < 1135.5 kN

Hence provide End bearing Stiffners (EBS).

(ii) <u>Area of EBS:</u>

As per IS 800 - 2007, page 68,

$$F_{psd} = \frac{A_q \; f_{yq}}{0.8 \; \gamma_{mc}} \geq F_x \text{ (Reaction)}$$

where, F_{psd} = Force of bearing strength of the Stiffner

A_q = area of End Stiffner in Contact with the flange

f_{yq} = yield Stress of Stiffner

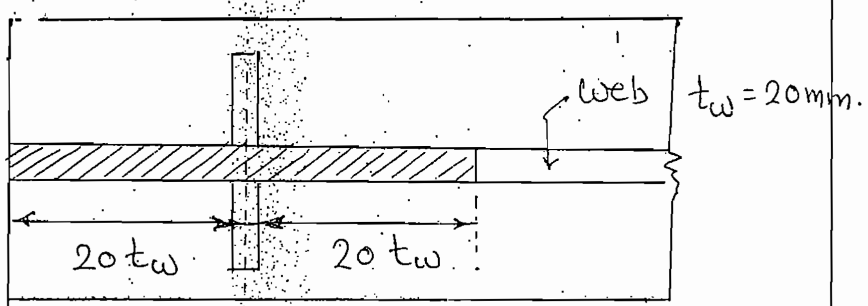
$$\therefore f_{psd} = \frac{A_q * 250}{0.8 * 1.10} = 1135.5 * 10^3$$

$$A_q = \frac{(1135.5 * 10^3) * 0.8 * 1.10}{250} = 3996.96\ mm^2$$

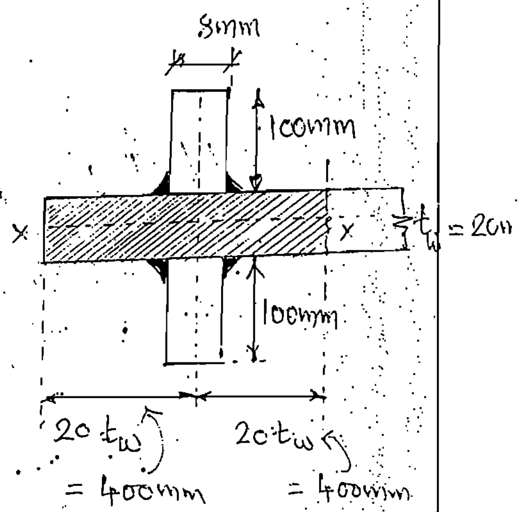
$$A_q = 4000\ mm^2 \text{ (Say)}$$

★★ The EBs are designed like a Compression members subjected to reaction and the effective height is 0.70 d.

Along with EBS plates, the part of the web ($20t_w$ on either side) also carry the reactions.



Let us provide EBS plates of Size 100mm * 8mm (assumed)



Area = $A = 2*(8*100) + 20*800$

$A = 1600 + 16000 = 17600 \text{ mm}^2$

$I_{xx} = \frac{bd^3}{12} + ah^2$

$I_{xx} = 2\left[\frac{8*100^3}{12} + (8*100)*(50+10)^2\right]$

$+ 1\left[800*\frac{20^3}{12} + (20*800)*(0)^2\right]$

$I_{xx} = 7.09*10^6 + 0.533*10^6 = 7.623*10^6 \text{ mm}^4$

$I_{xx} = I_{min} = 7.623*10^6 \text{ mm}^4$

$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{7.623*10^6}{17600}} = 20.81 \text{ mm}$

$\lambda = \frac{l_e}{r_{min}} = \frac{0.7d}{r_{min}} = \frac{0.7*1400}{20.81} = 47.09$

Buckling class - C, Page 42, IS 800-2007

f_{cd}	λ	f_{cd}
	40.00	198
	47.09	?
	50.00	183

$$f_{cd} = 198 + \left(\frac{47.09 - 40.00}{50.00 - 40.00}\right)(183 - 198)$$

$$f_{cd} = 187.37\ N/mm^2$$

f_{cd} = design Compressive stress

$$\text{Load} = P = (A_c)\, f_{cd} = 17600 * 187.37$$

$$P = 3297.63 * 10^3\ N = 3297.63\ kN > 1135.5\ kN$$

Safe.

(iii) Design of Connection:

$$\text{force per mm height of web} = \frac{\text{Reaction}}{d} = \frac{1135.5}{1400}$$

$$= 0.811\ kN/mm = 811\ N/mm$$

Equating the above force with Strength of the weld per mm,

1 mm

4 side welding

$$811 = \left[0.7\,S * l * \frac{f_u}{\sqrt{\ }}\right] * 4$$

$$811 = \left[0.7S * 1 * \frac{410}{\sqrt{3} * 1.25} \right] * 4 = 530.24\, S$$

$$S = \frac{811}{530.24} = 1.53 \text{ mm.}$$

Provide minimum Size of Weld = 3mm.

Step # 5 : De Sign of Bearing Stiffner (BS):

*** Note: End bearing Stiffners are provided to resist Support reaction at the ends, whereas Bearing Stiffners are provided to resist the external loads and hence provided under the external point loads. The design procedure for bearing Stiffeners (BS) is exactly Similar to the end bearing Stiffeners

* Replace the Support end reactions by the external Point load.*

i) Bearing Stiffeners:

Supported External point load = 600 KN

Local web load Carrying Capacity:

As per - I.S 800 - 2000, Page 67,

$$F_w = \frac{(b_1 + n_2)\, t_w\, f_{yw}}{\gamma_{m0}}$$

b_1 = Stiff bearing length = 120 mm (assumed)

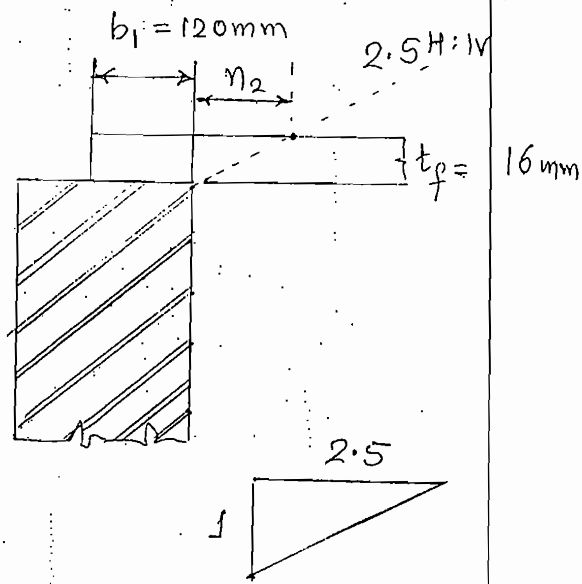
n_2 = length obtained by dispersion thro' the flange to the web junction at a slope of 2.5H : 1V to the plane of the flange.

t_w = web thickness = 20 mm

f_{yw} = yield stress of web = 250 N/mm²

$n_2 = 2.5\,(16) = 2.5\, t_f$

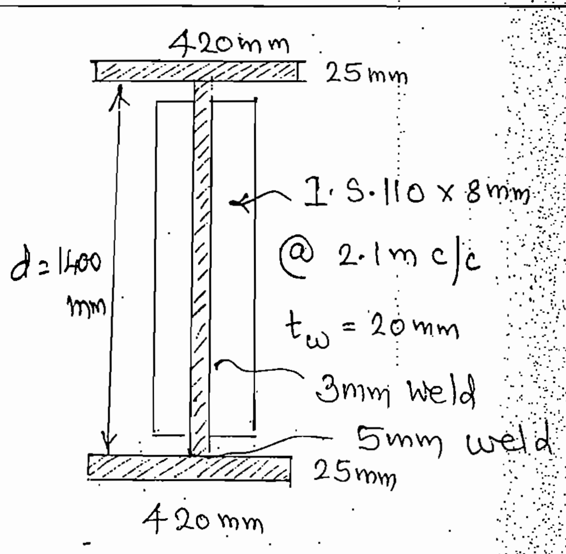
$= 40.0$ mm



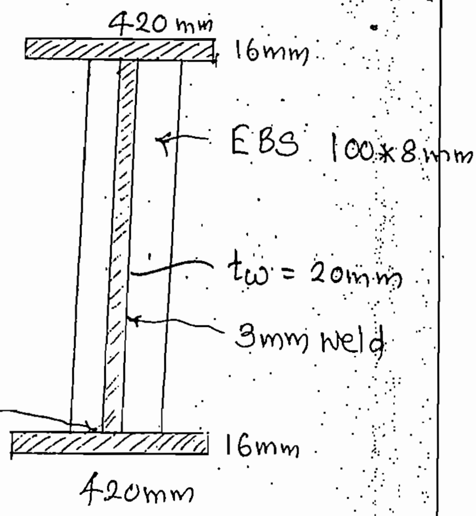
$$F_w = \frac{(120 + 40) * 20 * 250}{1.10}$$

$F_w = 727.27 * 10^3$ N $>$ 600 kN

∴ Hence bearing Stiffners are not reqd for the present problem.



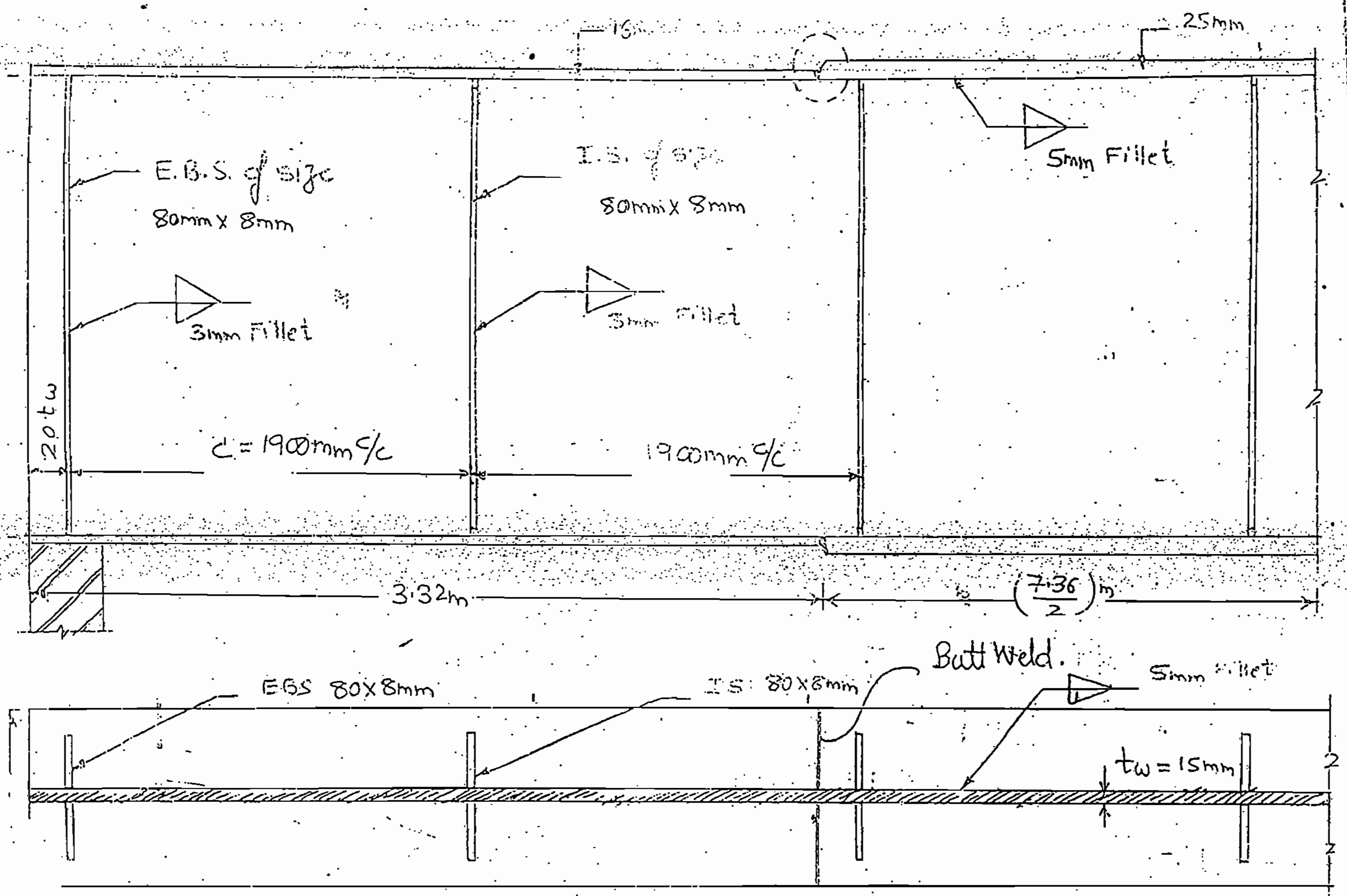
420mm
25mm
I.S. 110 x 8mm
@ 2.1m c/c
d = 1400 mm
t_w = 20mm
3mm weld
5mm weld
25mm
420 mm



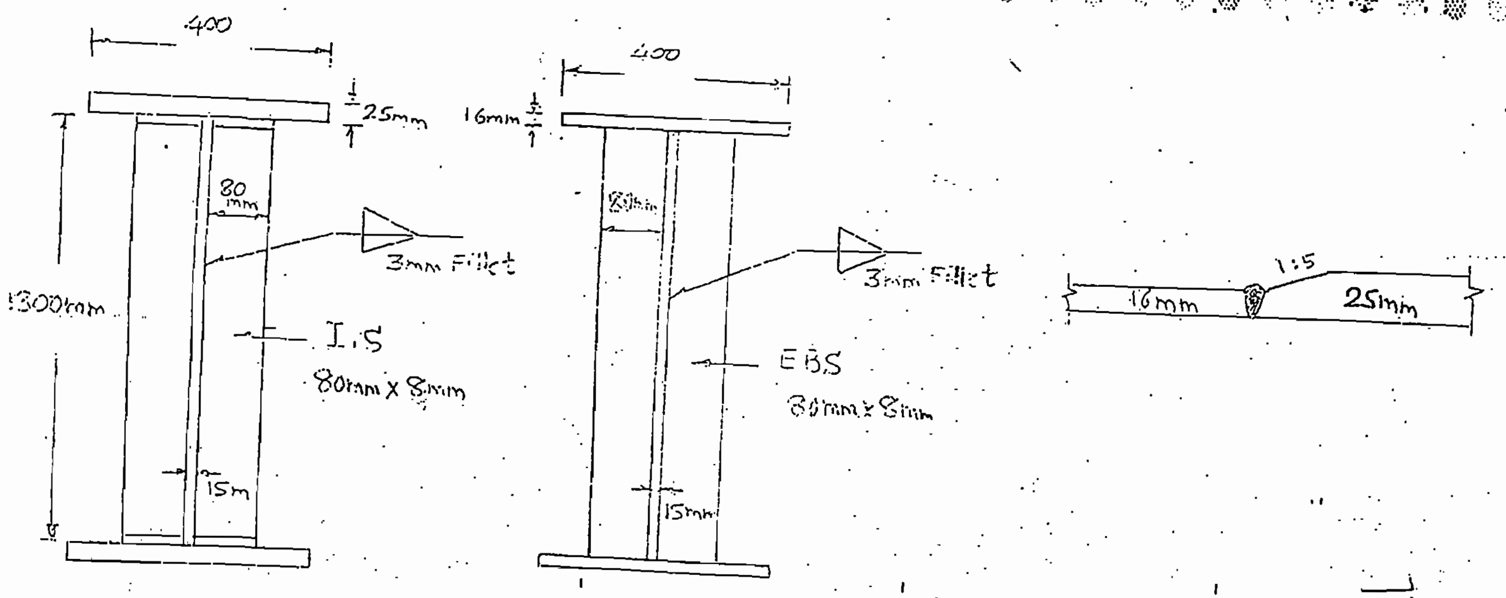
420 mm
16mm
EBS 100*8mm
t_w = 20mm
3mm weld
16mm
420mm

x x x

Half Elevation : 1:10



Sectional Plan



Mid Span c/s

Support c/s

1 : 10

NAVEEN KUMAR.B

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BMS EVENING

COLLAGE

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MODULE#2. GANTRY GIRDER

Problem # 1. Design a simply supported gantry girder to carry an electrically operated travelling crane with the following data:

(1) Span of the crane bridge = 25 m (ie. c/c distance b/w gantry girders)

(2) Column spacing = 8 m (ie span of gantry girder)

(3) Wheel base = 3.50 m

(4) Crane capacity = 200 kN

(5) Weight of crane bridge = 150 kN

(6) Weight of trolley (crab) = 75 kN

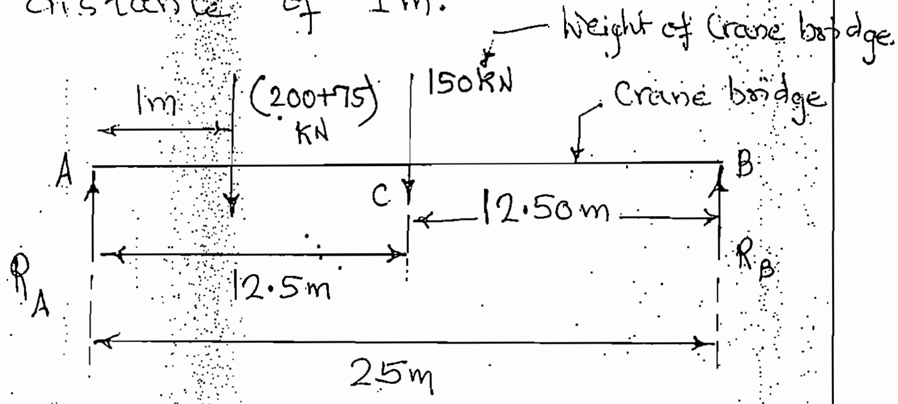
(7) Minimum hook distance = 1.0 m

(8) Weight of the rails = 0.30 kN/m

(9) Height of the rails = 105 mm.

Soln: Step # 1. Load Calculation:

Maximum reaction in the crane bridge occurs when the trolley along with hook is towards the left or right Support with a minimum hook distance of 1m.



$$\sum M_B = 0, \quad R_A * 25 - 275 * 24 - 150 * 12.50 = 0$$

$\circlearrowleft_{+}$ $\circlearrowright_{-}$

$$R_A = 339 \text{ kN.}$$

★ There are two wheels at the end of each Crane bridge.

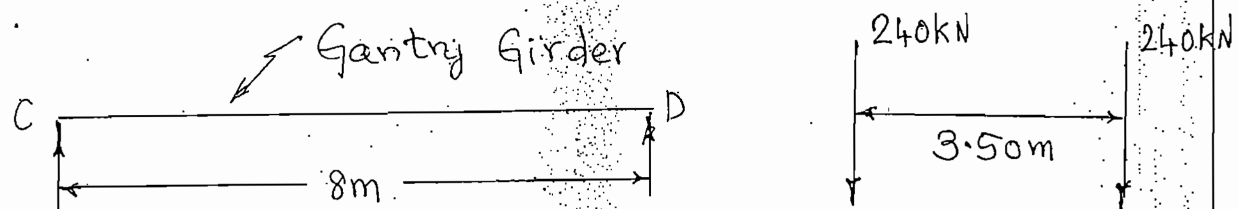
$$\therefore \text{ Load on each wheel} = \frac{339}{2} = 169.5 \text{ kN}$$

★★ Consider an Impact factor of 40 %

$$\therefore \text{ Load on each wheel} = 1.40 * 169.5 = 237.3 \text{ kN}$$

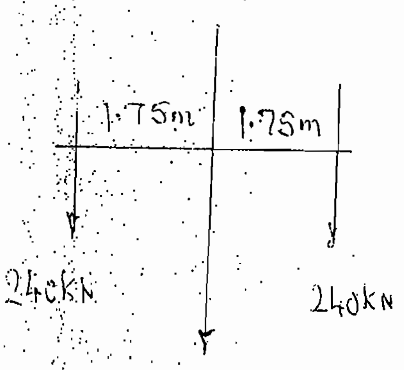
Take $\underline{\underline{240 \text{ kN}}}$.

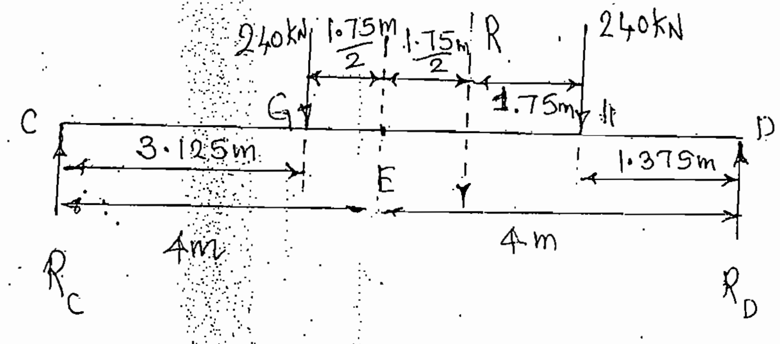
Step # 2: Shear force and BM on Gantry Girder:



ii. "The arrangement of wheel loads for max BM is that the mid span is equidistant from the resultant of the two wheel loads and any one load". "Later take BM under wheel load which is close to mid span".

Arrange any one load and resultant equidistant from mid span.





* place the resultant either to left or right of mid span by $\frac{1.75}{2}$ m ie 0.875 m and hence Calculate R_C or R_D for that position of load.

$\sum M_D = 0$, $\quad R_C * 8 - 240 * 4.875 - 240 * 1.375 = 0$

$\circlearrowleft$ + $\quad \circlearrowright$ −

$$R_C = 187.5 \text{ kN}$$

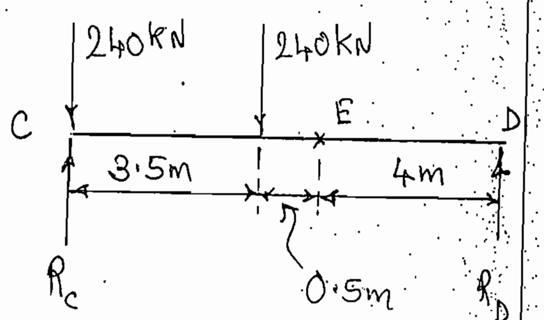
Max BM at G $= R_C * 3.125 = 187.5 * 3.125$

$= 585.94 \text{ kN-m}.$

factored Moment = $1.50 * 585.94$

$= 878.91$ kN-m.

Max Shear force: "The arrangement of wheel loads for max Shear force is that the two wheel loads are placed either Complete left or Complete right of Span".



$\Sigma M_D = 0,$

$$R_c * 8 - 240*8 - 240*4.5 = 0$$

$$R_c = \frac{240(12.5)}{8} = 375 \text{ KN}$$

$\therefore$ Factored Shear force $= V = 1.5 R_c$

$$\therefore V = 1.5 * 375 = 562.5 \text{ KN}$$

Step # 3. Horizontal load and its Moment:

A lateral load is developed due to the application of brakes or the Sudden acceleration of the trolley.

It is taken as 10 % of lifted weight + trolley weight.

Horizontal force $= 0.1 [200 + 75] = 27.5$ KN

The above force should be equally divided between the 4 Wheels

$\therefore$ Nett Horizontal force $= (H) = \frac{27.5}{4 \text{ Wheels}}$

$H = 6.875 \text{ kN} \simeq 7 \text{ kN}$

$\therefore$ factored Horizontal force $= 1.5 * 7 = 10.5 \text{ kN}$ $(\leftarrow)$

Moment due to Horizontal force $= \frac{878.91}{240 * 1.5} * 10.5$ ← factor of Safety

$= 25.64 \text{ kN-m}$

Step # 4. Trial Section:

* The trial Section is based on deflection condition.

*** Max permissible deflection for electrically operated Crane $= \boxed{\frac{\text{Span}}{750}}$

$\therefore$ Max permissible deflection $= \frac{8000 \text{ mm}}{750} = 10.67 \text{ mm}$

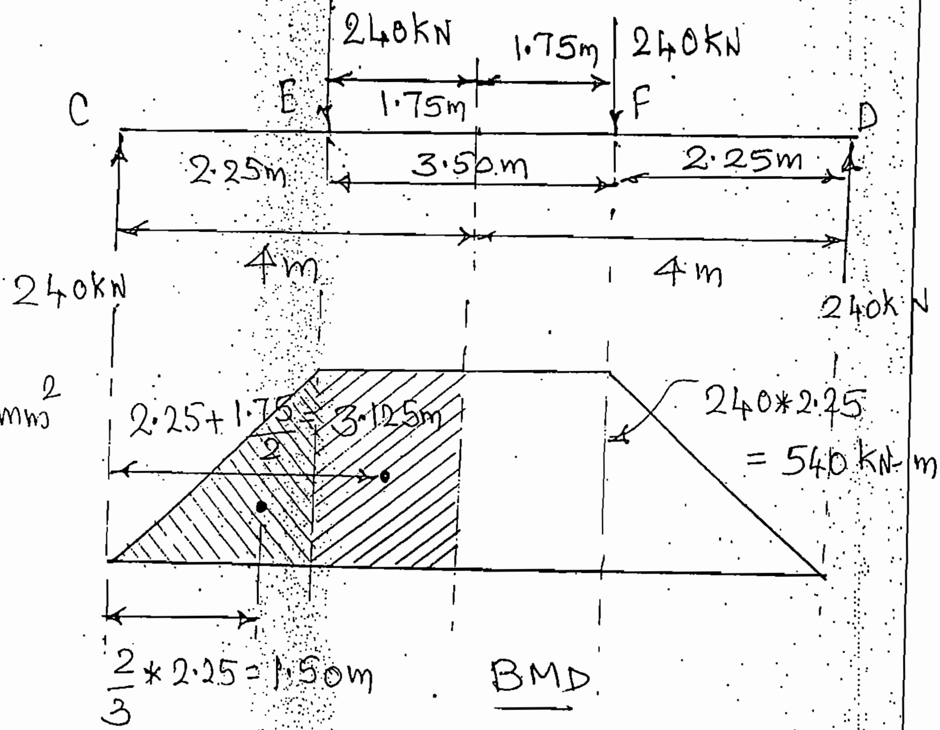
But actual deflection: The point loads are Symmetrically placed from mid Span point and max deflection is Worked out.

Max BM = Wa

$= 240 * 2.25$

$= 540$ kN-m

$E = 2*10^5 \ N/mm^2$



$\therefore \ \delta_{load} + \delta_{Self\ Wt} = 10.67$ mm

Assume 1mm

Using Moment - Area method, $\delta_{load} = \frac{Area * \bar{x}}{EI}$

$$\frac{1}{2*10^5 I}\left\{\left(\frac{1}{2}*2.25*10^3*540*10^6\right)*1.50*10^3 + 1.75*10^3*540*10^6*3.125*10^3\right\} + 1mm = 10.67 mm$$

$$\frac{1}{2*10^5 I}\left\{9.1125*10^{14} + 2.953*10^{15}\right\} + 1mm = 10.67mm$$

$$\frac{1}{2*10^5 I}\left\{3.864*10^{15}\right\} = 9.67 \text{ mm}$$

$I = 1.998 * 10^9 \ mm^4$.

Increase the above Value by 30%.

$\therefore \ I_{xx} = 1.30 * I = 1.30 * 1.998 * 10^9$

$I_{xx} = 2.598 * 10^9 \ mm^4 = 2.597 * 10^5 \ Cm^4$

From steel tables, try Suitable Section.

*** From Steel tables Single Joist With channel and plates on both flanges (Table # 12, Page 41, 42, 43 in Steel Tables by Agor).

Try ISWB 550 @ 112.5 kg/m

ISMC 400 @ 49.4 kg/m

Top Cover plate 320 * 20 mm

Bottom Cover plate 320 * 40 mm,

Total $I_z = I_x = 288010.60 \ cm^4 = 2.88 * 10^5 \ Cm^4$
$= 2.88 * 10^9 \ mm^4$

** Note: After Selecting the Sections, we must read the properties of ISWB 550 @ 112.5 Kg/m from Steel tables Table #4, to get the dimensions and properties, and Steel tables Table #5 to read the dimensions and properties of ISMC 400 @ 49.4 kg/m

The dimensions and properties to be read from Table #4 and Table #5 of Steel tables include b, t_f, t_w, h, r_1, Area, C_{xx}. These details to be incorporated to other details of the Steel table, table #12; which in Table #12, we need Area, C_{xx}, e_{xx} and r_{yy}.

Other Sectional properties include

$$\left.\begin{aligned} \text{Area} &= \text{Total area} = 398.27\ \text{cm}^2 = 39827\ \text{mm}^2 \bigstar \\ C_{xx} &= \text{Center of gravity} = 30.85\ \text{cm (From Top)} \\ e_{xx} &= \text{extreme fiber distance} = 31.01\ \text{cm (From Bottom)} \\ r_{yy} &= \text{radius of gyration} = 9.40\ \text{cm} \end{aligned}\right\} \text{Steel Table, Table \#12, Pages 44–45, Agor}$$

c/s details of ISWB 550 @ 112.5 Kg/m

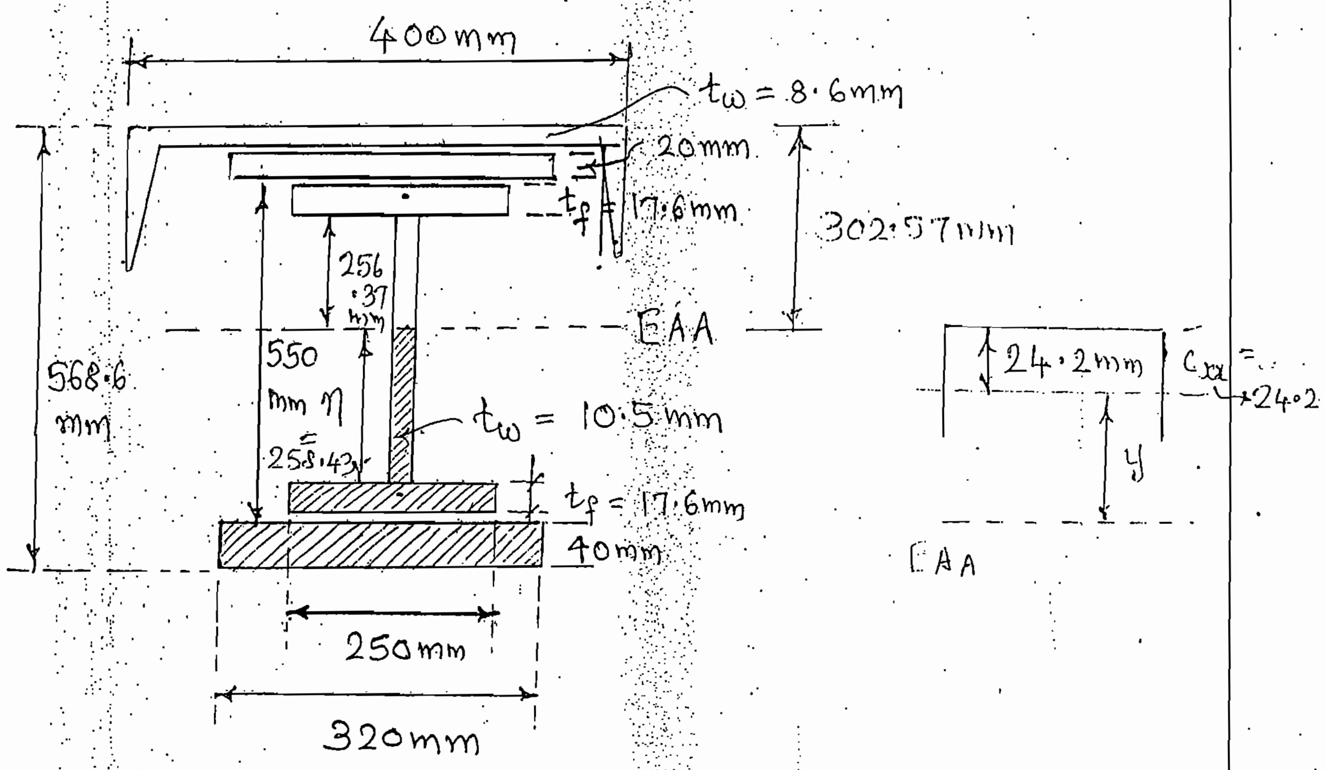
(Steel tables table #4, page 18, Agor)

$$\left.\begin{aligned} h &= \text{Depth of Section} = 550\ \text{mm} \\ b &= \text{width of Section} = 250\ \text{mm} \\ t_f &= \text{thickness of flange} = 17.6\ \text{mm} \\ t_w &= \text{web thickness} = 10.5\ \text{mm} \\ r_1 &= \text{radius at root} = 16\ \text{mm} \end{aligned}\right\} \text{Dimensions and properties}$$

C/g details of ISMC 400 @ 49.4 kg/m
(Steel tables, table # 5, Page 2c, Ager)

h = depth of Section = 400 mm
b = width of Section = 100 mm
t_f = thickness of flange = 15.3 mm
t_w = thickness of web = 8.6 mm
Area = C/S area = 62.93 cm^2
C_{xx} = Center of gravity = 2.42 cm

Dimensions and properties



Location of Equal area axis (EAA):

Area of shaded portion = $\frac{1}{2}$ * Total Area

$$\therefore \; (\eta * t_w) + (250 * t_f) + (320 * 40) = \frac{1}{2} * 39827$$

$$\eta * 10.5 + 250 * 17.6 + 320 * 40 = 19913.5$$

$$\eta * 10.5 = 2713.5$$

$$\text{or } \eta = 258.43 \text{ mm}$$

Z_p = plastic Section modulus = $\Sigma a\bar{y}$

$$Z_p = (320 * 40)\left(258.43 + 17.6 + \frac{40}{2}\right) +$$

$$+ (250 * 17.6)\left(258.43 + \frac{17.6}{2}\right) +$$

$$+ (10.5 * 258.43) * \left(\frac{258.43}{2}\right) +$$

$$+ (10.5 * 256.37) * \left(\frac{256.37}{2}\right) +$$

$$+ (250 * 17.6)\left(256.37 + \frac{17.6}{2}\right) +$$

$$+ (320 * 20)\left(256.37 + 17.6 + \frac{20}{2}\right) +$$

$$+ (6293)(302.57 - 24.2)$$

Area of channel $\qquad$ C_{xx} of channel

$$Z_p = 3.79 * 10^6 + 1.175 * 10^6 + 0.351 * 10^6 + 0.345 * 10^6 +$$

$$+ 1.166 * 10^6 + 1.817 * 10^6 + 1.752 * 10^6$$

$$Z_p = 10.397 * 10^6 \text{ mm}^3$$

Step #5: Check for Moment of Resistance:

*For Laterally "Unsupported Beam" condition,

as per IS 800 - 2007, clause 8.2.2. page 54,

Correction factor for Lateral Buckling

Design bending strength $= M_d = \beta_b \cdot Z_p \cdot f_{bd}$,

$\boxed{\alpha_{LT} = 0.21}$

For Rolled Steel Section

$\beta_b = 1$ for plastic and compact sections

$Z_p =$ plastic section modulus

$f_{bd} =$ design bending compressive stress

$E =$ Youngs modulus of steel $= 2*10^5$ N/mm^2

$L_{LT} =$ Span of gantry girder $= 8$ m $= 8000$ mm

$r_{yy} = r_y =$ radius of gyration $= 9.4$ cm $= 94$ mm

Average or mean thickness of flange:

Top flange thickness $= 35.6$ mm $= t_f$

Bottom flange thickness $= 53.8$ mm

Steel table Table #12 Page 44-45 Ager

$h_f =$ c/c distance b/w flanges

$h_f =$ overall depth $- \frac{1}{2}$ (Top and Bottom flange thickness)

$h_f = 568.6 - \frac{1}{2}(35.6 + 53.8) = 523.90$ mm

As per IS 800 - 2007, clause 8.2.2.1, page 54,

$f_{cr,b}$ = resisting Compressive Stress in bending $= \dfrac{1.10\,\pi^2 E}{(L_{LT}/r_y)^2}\left\{1+\frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right\}^{0.5}$

$$\therefore f_{cr,b} = \frac{1.10 * \pi^2 * 2*10^5}{\left(\dfrac{8000}{94}\right)^2}\left\{1+\left[\frac{(8000/94)}{(523.9/35.6)}\right]^2 * \frac{1}{20}\right\}^{0.5}$$

$$f_{cr,b} = 299.776\left\{1+\frac{33.44}{20}\right\}^{0.5} = 490.01\ N/mm^2$$

from I.S. 800 - 2007, table # 13 (a) Page 55, for $f_y = 250\ N/mm^2$,

$f_{cr,b}$	$(f_y = 250\ N/mm^2) \rightarrow f_{bd}$
$500\ N/mm^2$	$188.6\ N/mm^2$
$490.01\ N/mm^2$	?
$450\ N/mm^2$	$186.4\ N/mm^2$

$$f_{bd} = 188.60 + \left(\frac{490.01 - 500}{450 - 500}\right)(186.4 - 188.6)$$

$f_{bd} = 188.16\ N/mm^2$ = Design bending Compressive Stress

$$M_d = \beta_b * Z_p * f_{bd} = 1 * 10.397*10^6 * 188.16$$

$M_d = 1956.30 * 10^6$ N-mm = 1956.30 KN-m > 878.91 KN-m

Safe.

Step # 6 : Check for shear resistance:

V_1 - design Shear strength

As per IS 800 - 2007, page 59,

$$V_d = \frac{V_n}{\gamma_{mo}} = \left[\frac{A_v \cdot f_{yw}}{\sqrt{3}\ \gamma_{mo}} \right]$$

A_v = Shear area = $h * t_w$ (for Hot rolled)

h = over all depth of the section

$A_v = 568.6 * 10.5 = 5970.30\ mm^2$

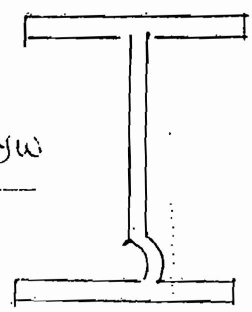
$$\therefore V_d = \frac{5970.30 * 250}{\sqrt{3} * 1.10} = 783.4 * 10^3\ N = 783.4\ kN$$

$V_d = 783.4\ kN > 562.5\ kN$ (Factored shear force)

Safe.

Step # 7. check for Web Crippling:

As per IS 800 - 2007, page 67;

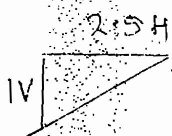


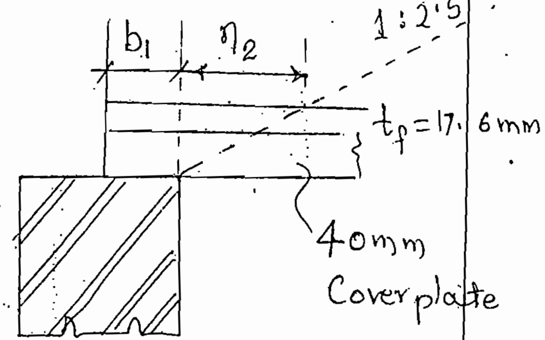
Local Capacity of web = $F_w = \dfrac{(b_1 + n_2) t_w f_{yw}}{\gamma_{mo}}$

b_1 = bearing = 120 mm (assume)

$n_2 = 2.5(40 + 17.6)$

$= 144 mm$



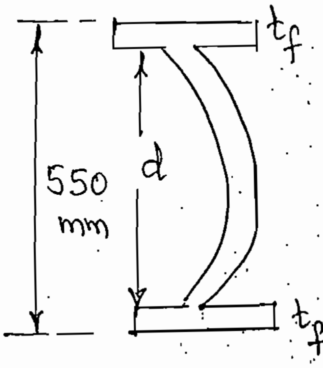


$$\therefore F_w = \frac{(120 + 144) * 10.5 * 250}{1.10}$$

$F_w = 630 * 10^3 N = 630\ kN > 562.5$ ...

Step # 8 : Check for web buckling:

As per IS 800 - 2007, Page 67,

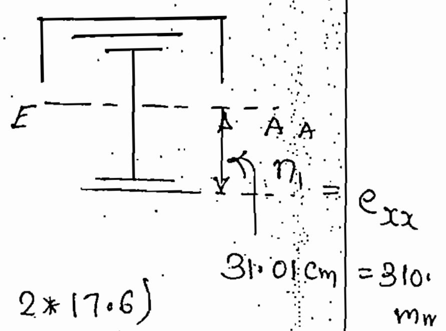


Buckling Strength = $(b_1 + n_1) * t_w * f_{cd}$

b_1 = bearing = 120 mm (assumed)

$n_1 = e_{xx} = 31.01$ cm = 310.1 mm

$t_w = 10.5$ mm, $r_{yy} = 9.4$ cm = 94 mm.



$$\lambda = \frac{KL}{r} = \frac{l_e}{r_{min}} = \frac{0.7d}{r_{yy}}$$

$$\lambda = \frac{0.7(550 - 2 * t_f)}{94} = \frac{0.7 * (550 - 2 * 17.6)}{94}$$

$\lambda = 3.83.$

As per IS 800 - 2007, Page 42, Table 9 (c),

$f_{cd} = 227 \text{ N/mm}^2$ (for $\lambda = \leq 10$) @ $f_{yw} = 250$ N/mm

$f_{cd} = 227 \text{ N/mm}^2$.

$\therefore$ Buckling Strength = $(120 + 310.1) * 10.5 * 227$

$= 1025.14 * 10^3$ N = 1025.14 KN

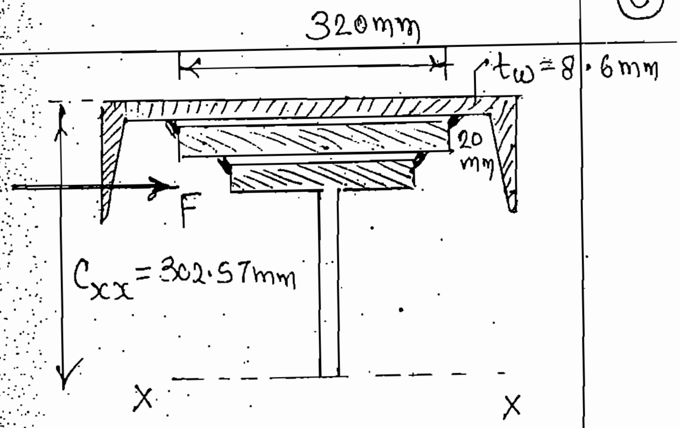
$\therefore$ Buckling Strength = 1025.14 kN $>$ 562.5 kN (factored shear force)

Safe.

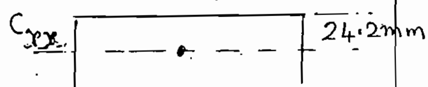
Step #9 : Design of Connection:

Force at the Junction = F = $\frac{V \cdot \Sigma a\bar{y}}{}$ N/mm

$$F = \frac{562.5 * 10^3 N}{2.88 * 10^9} \left[a\bar{y} \right]$$



$$F = 195.313 * 10^{-6} \left[a\bar{y} \right] \quad \text{---(i)}$$



Cover plate

$$a\bar{y} = 6293\,(302.57 - 24.2) + (320 * 20) * \left(302.57 - \underset{t_w}{8.6} - \underset{\text{Cover plate}}{20} - \underset{t_f}{\frac{17.6}{2}}\right)$$

$$= 1.752 * 10^6 + 1.697 * 10^6 = 3.45 * 10^6 \text{ mm}^3.$$

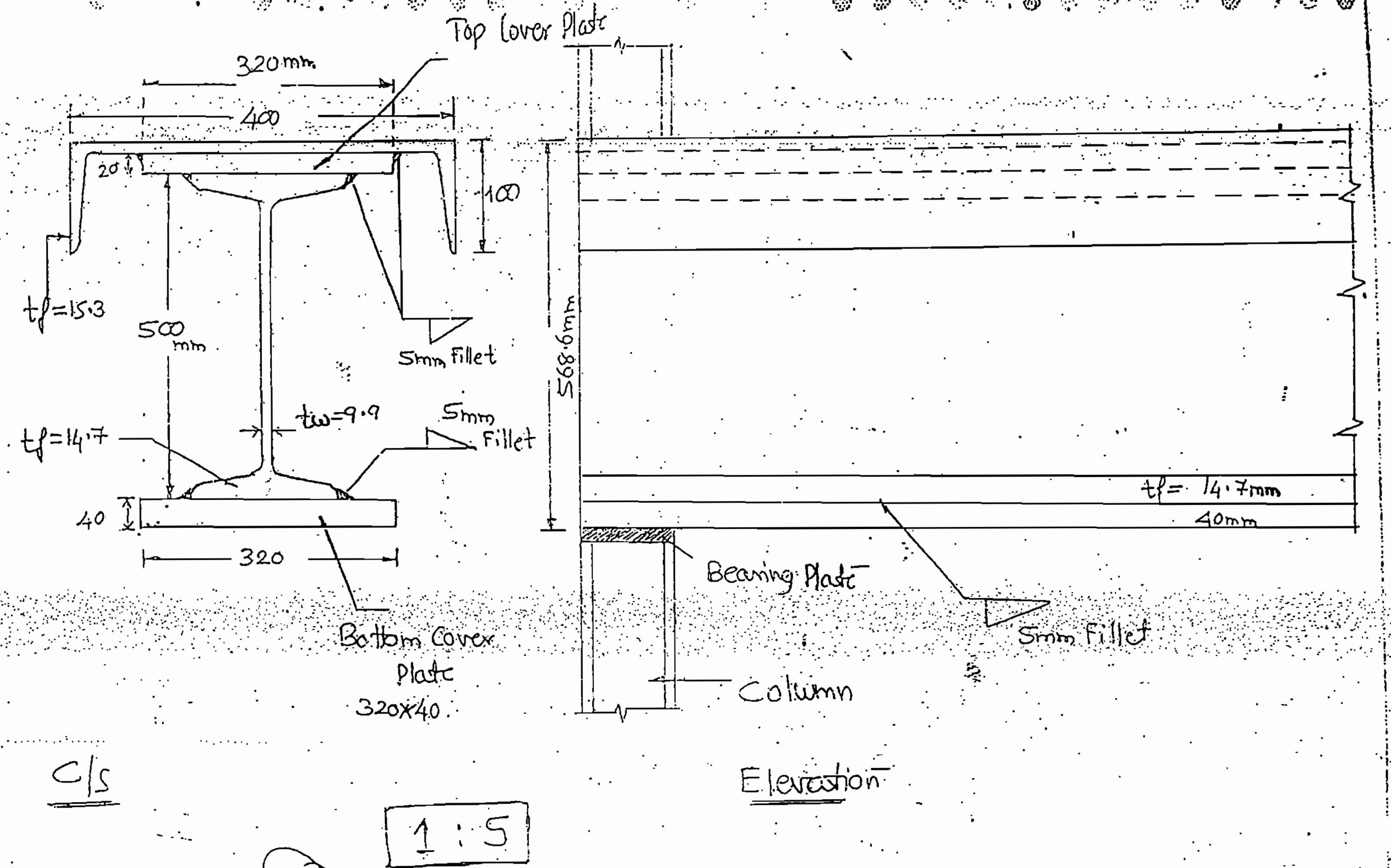
$$\therefore F = 195.313 * 10^{-6} * 3.45 * 10^6 = 673.83 \text{ N/mm}.$$

Equating the above force to the strength of weld,

$$673.83 = \underset{\text{Welding on two sides}}{2} \left[0.7 * S * \underset{\text{length} = 1mm}{1mm} * \frac{410}{\sqrt{3} * 1.25} \right]$$

$\therefore$ $S = 2.54$ mm.

Provide minimum weld size of 5 mm.


Top Cover Plate
320mm
400
20
100
tf=15.3
500 mm
5mm Fillet
tw=9.9
5mm Fillet
tf=14.7
40
320
Bottom Cover Plate 320x40
568.6mm
tf= 14.7mm
40mm
Bearing Plate
5mm Fillet
Column
C/S
Elevation
1 : 5

$(50+20) = 70$