INTRODUCTION

A steel structure is an assemblage of a group of members expected to sustain their share of applied forces and to transfer them safely to the ground.

Depending on the orientation of the member in the structure and its structural use, the member is subjected to forces either axial, bending or torsion or a combination there of.

DESIGN OF STRUCTURAL ELEMENTS

The position of elements, namely beams, columns, trusses, purlins etc. are marked on the plan provided by architects.

Various combinations of possible loads are ascertained and the members are proportional on the basis of selected deign method.

Standard specifications and codes

The bureau of Indian standards has published a number of codes, standard and handbooks

1S Handbook No.1- Properties of structural steel rolled sections

1S 875-1987- Code of practice for design loads for building and structure.

1S800-2007-Code of practice for use of structural steel in general construction.

Advantages of steel as a structural material

- They can be erected at faster rate
- It is a recyclable material
- Properly maintained steel structure has long life.

Disadvantage of steel as a structural material

- Fatigue of steel is one of the major drawbacks.

- At the places of stress concentration in the steel sections, under certain conditions, the steel may lose its ductility.
- Steel structures needs fire proof treatments treatment, which increases the cost.

UNIT-1 BEAM TO BEAM FRAMED CONNECTION

Two secondary beams ISLB-300@ 37.7kg/m and ISMB-500 are connected to main beam ISMB-600

The size of cleat angle ISA 90*90*8mm provided

3- #20 bolts for angle and ISLB-300

4- # 20 bolts for angle and ISLB-300

3 # 20 bolts per cleat angle for main beam connection.

Use pitch= 60mm, edge distance=35mm

Soln:

ISLB-300 h=300, b=150, t_f=9.4,t_w=0.7mm,g_1=60mm

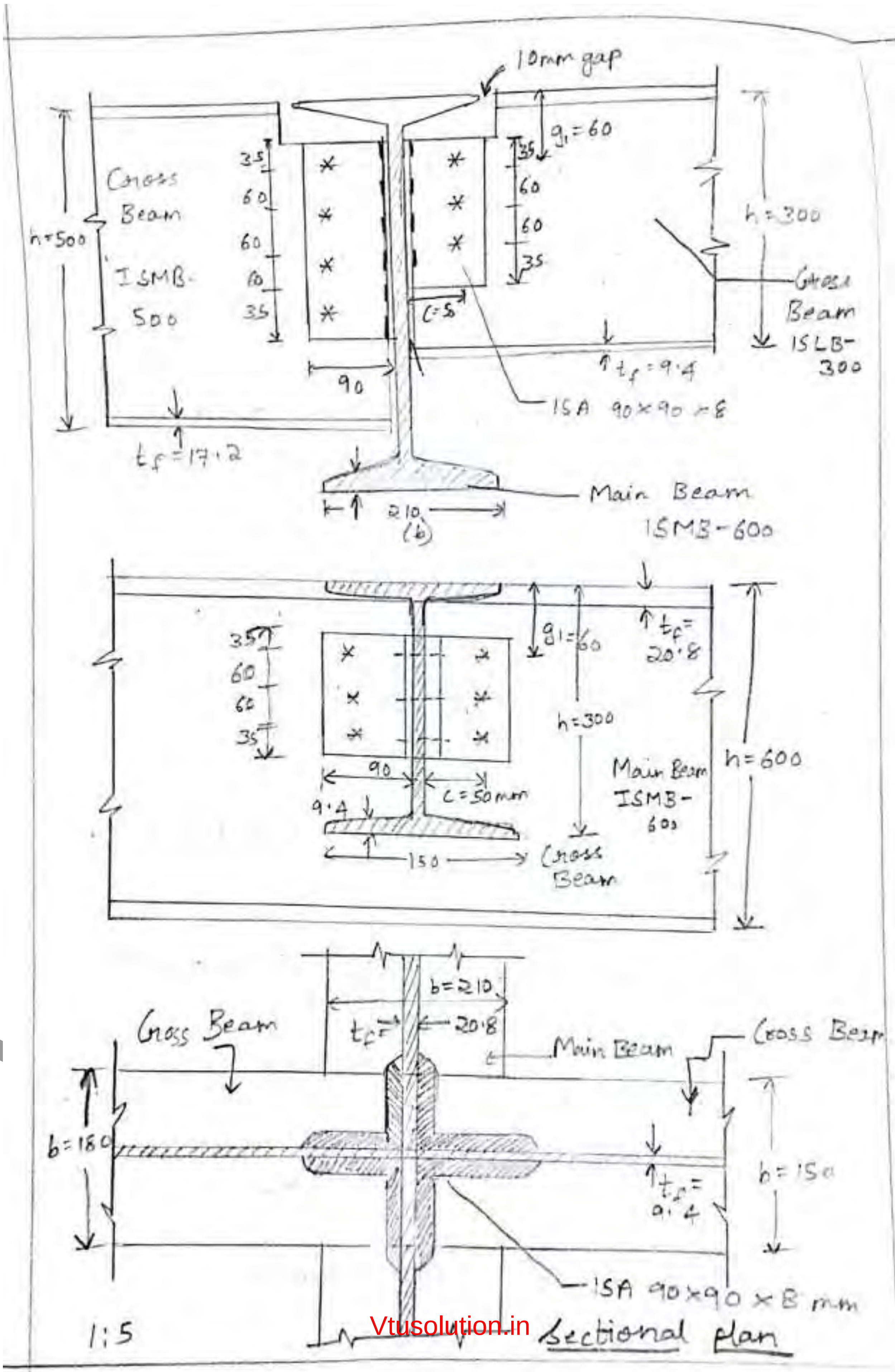
ISLB-500 h=500, b=180, t_f=17.2,t_w=10.2 mm,g_1=75mm

ISLB-600 h=600, b=210 t_f=20.8,t_w=12.0mm

Angle leg- 90mm

c=50mm

page 168 sp-6(hand book)


10mm gap
g1 = 60
Cross Beam
ISMB-500
h=500
35
60
60
60
35
h = 300
Cross Beam
ISLB-300
tf = 9·4
90
ISA 90×90×8
c=5
tf = 17·2
Main Beam
ISMB-600
210
(b)
g1 = 60
tf = 20·8
h = 300
h = 600
Main Beam
ISMB-600
9·4
c = 50mm
150
Cross Beam
b = 210
tf = 20·8
Cross Beam
Main Beam
b = 180
b = 150
tf = 9·4
ISA 90×90×8 mm
1:5
Sectional plan

Beam to column framed connection

A beam ISMB-400 @ 61.6kg/m is connected to flange of a column ISHB-400 @ 82,2kg/m using framed bolted connection size of cleat angle ISA 150*115*12mm

6- # 20mm bolt in two rows is used to connect angle and beam.

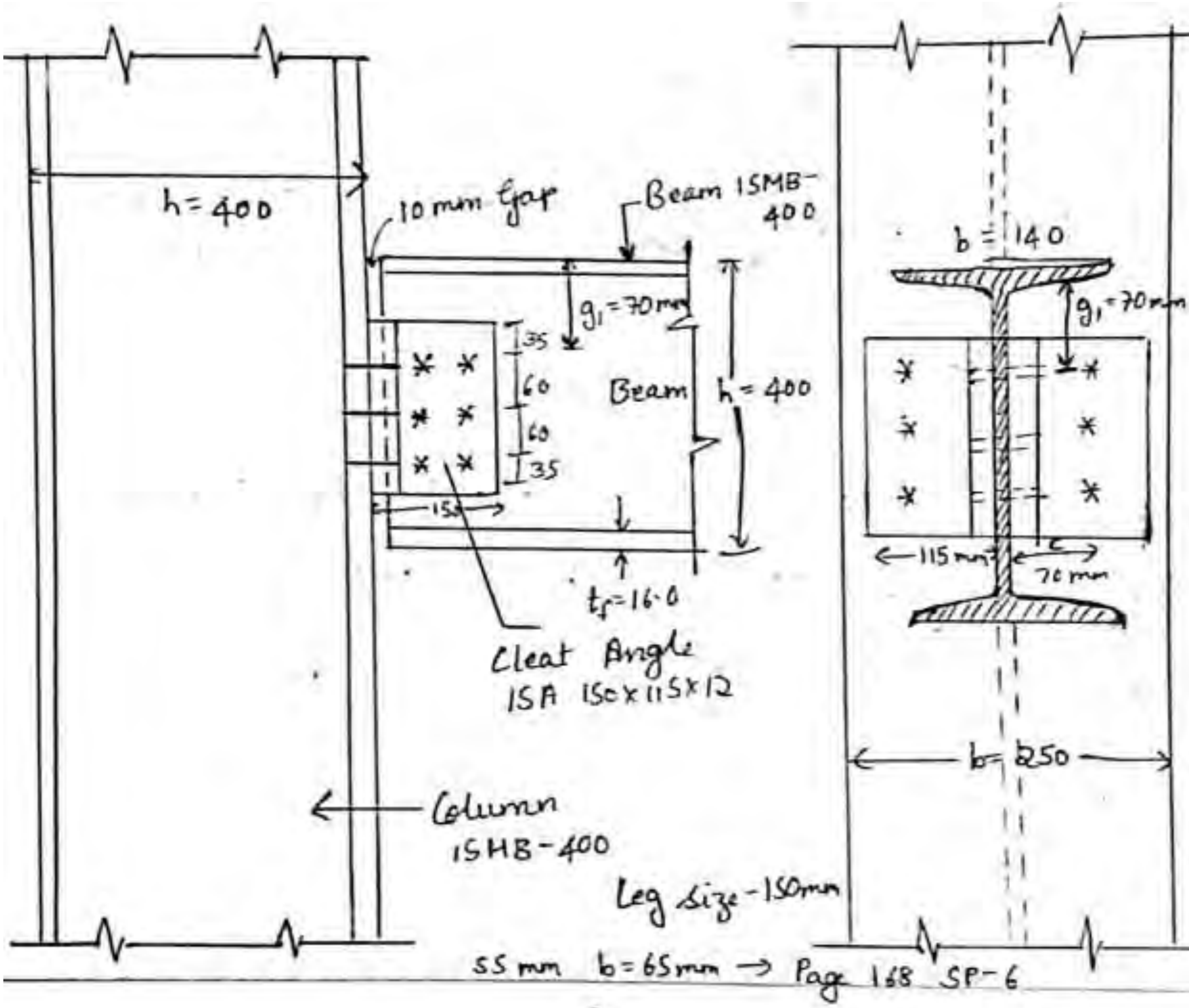
3- # 20mm bolt for each angle to connect angle and column flange.

Use pitch=60mm, edge distance=35mm

Soln:

ISBH-400 h=400, b=250, t_f=12.7,t_w=10.6mm

ISBH-400 h=400, b=140, t_f=16.0,t_w=8.9mm,g_1=70mm



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<u>BEAM TO COLUMN UNSTIFFENED SEAT CONNECTION</u>

Column – ISHB-400@82.2kg/m

Beam- ISHB-400@61.1kg/m

Seat angle- ISA 150*115*15mm

Cleat angle- 90*90*8mm

4# 22mm bolts for seat angle with “column web”

2# 20mm for remaining connections.

<u>Soln:</u>

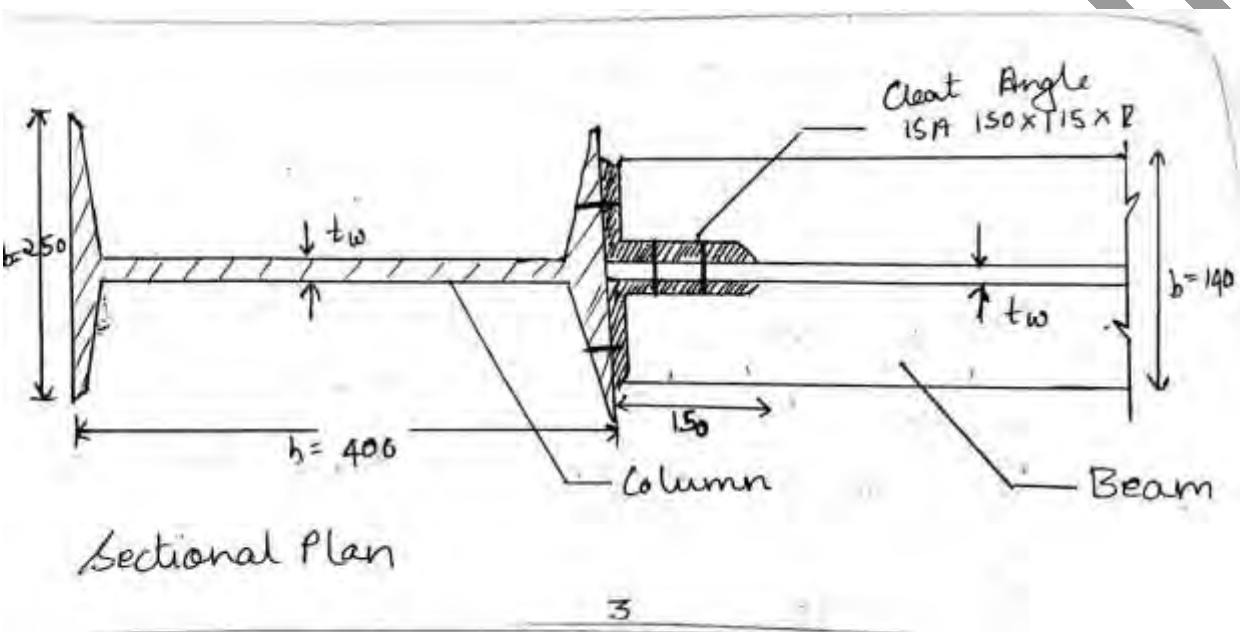
ISHB-400 h=400, b=250, t_f=12.7,t_w=10.6mm

ISMB-400 h=400, b=140, t_f=16.0,t_w=8.9mm

Angle 150mm leg- a

115mm leg-c

90mm leg-c



Beam to column stiffened seat connection

Column – ISHB-400@82.2kg/m

Beam- ISHB-400@61.1kg/m

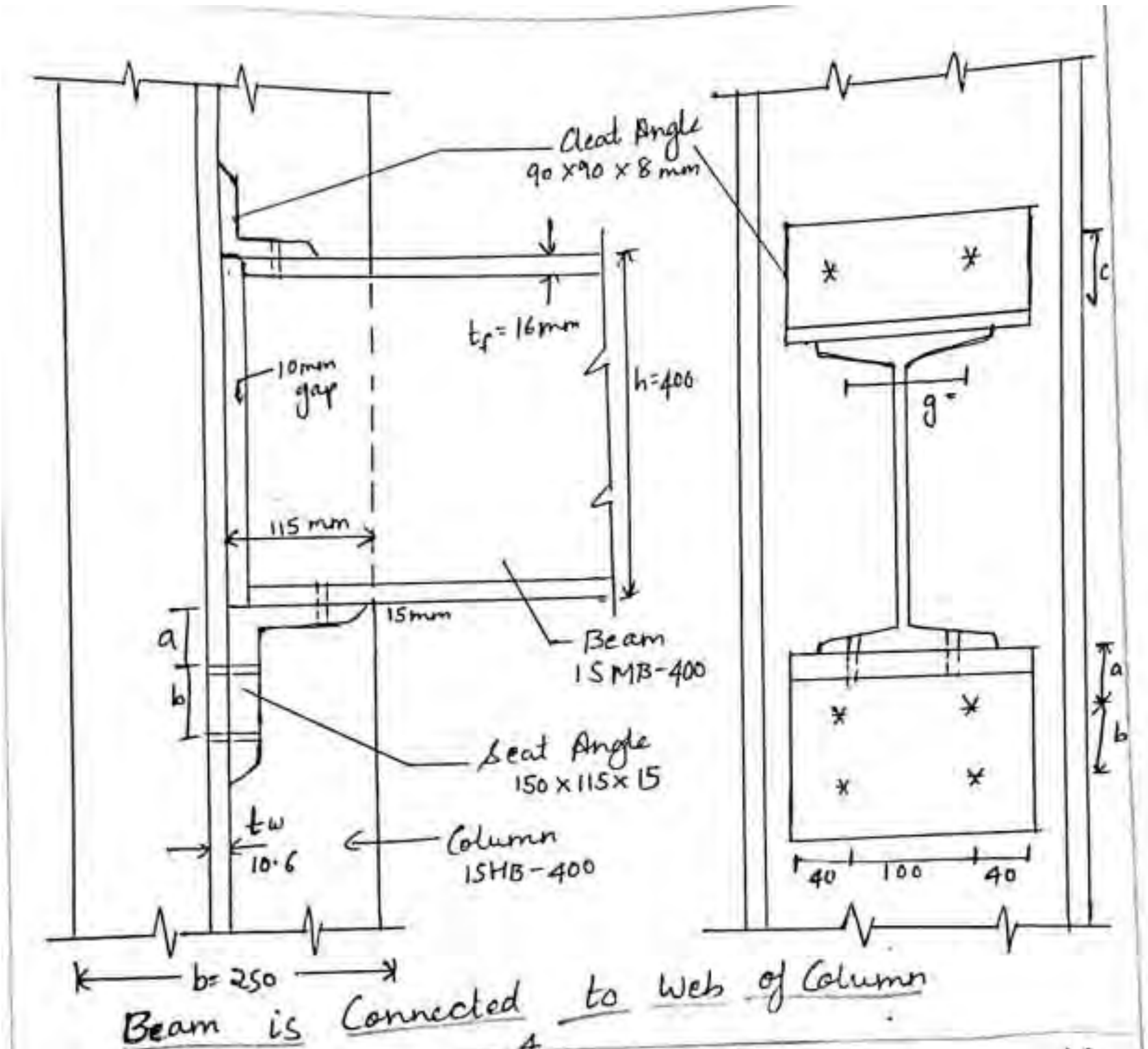
Seat angle- ISA 100*100*10mm

Cleat angle- 90*90*8mm

Pair of stiffener: 2 ISA 90*90*8mm-500mm length.

8 # 20mm botts for stiffener to column in two rows.

2 # 20mm for remaining connection.



Framed connection

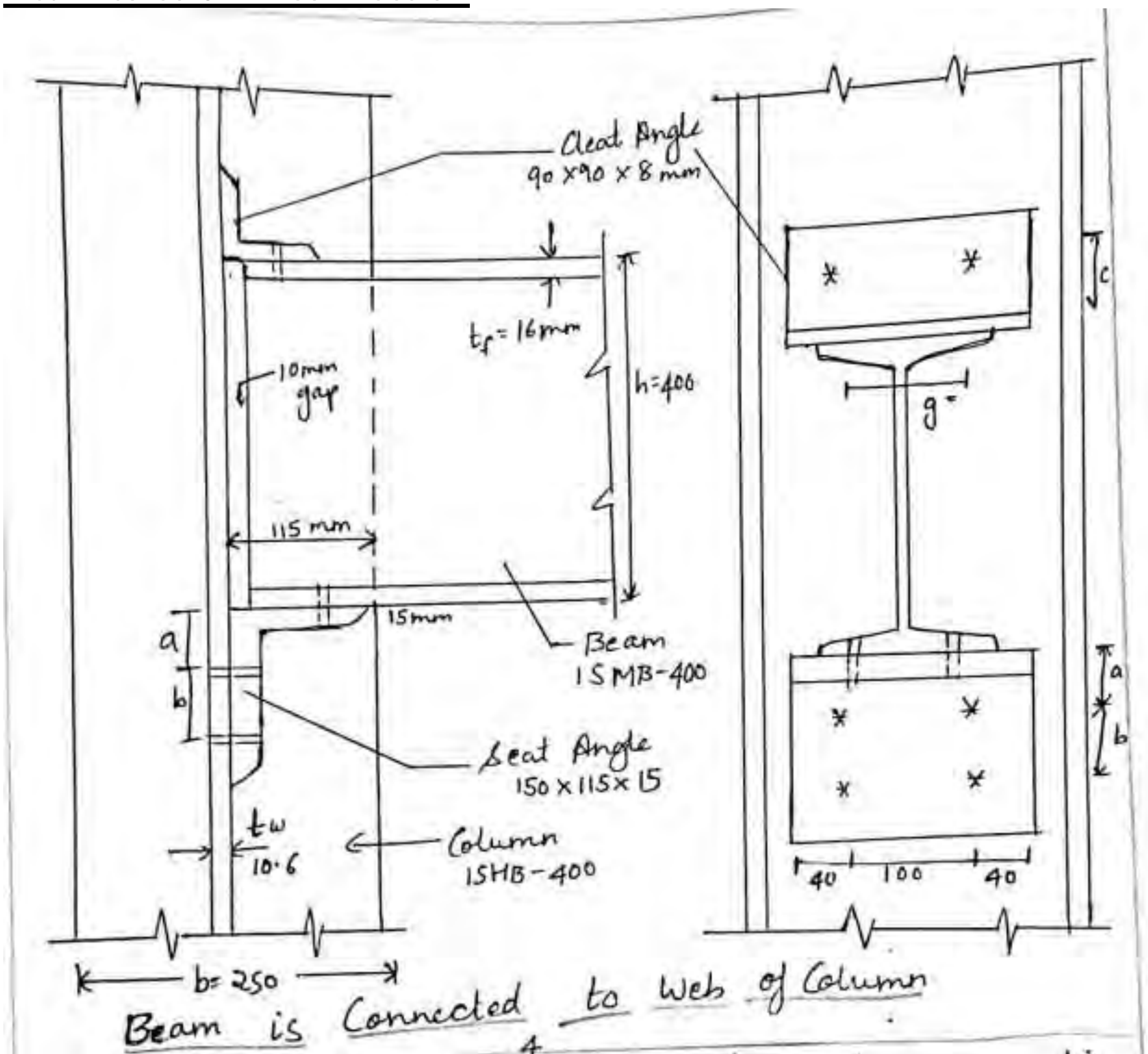
Beam ISMB-400 61.6kg/m, column ISHB-300@58.8kg/m, plate- 50mm width, 200mm depth and 12mm thick

b. Cross beam   ISMB-300@44.2kg/m

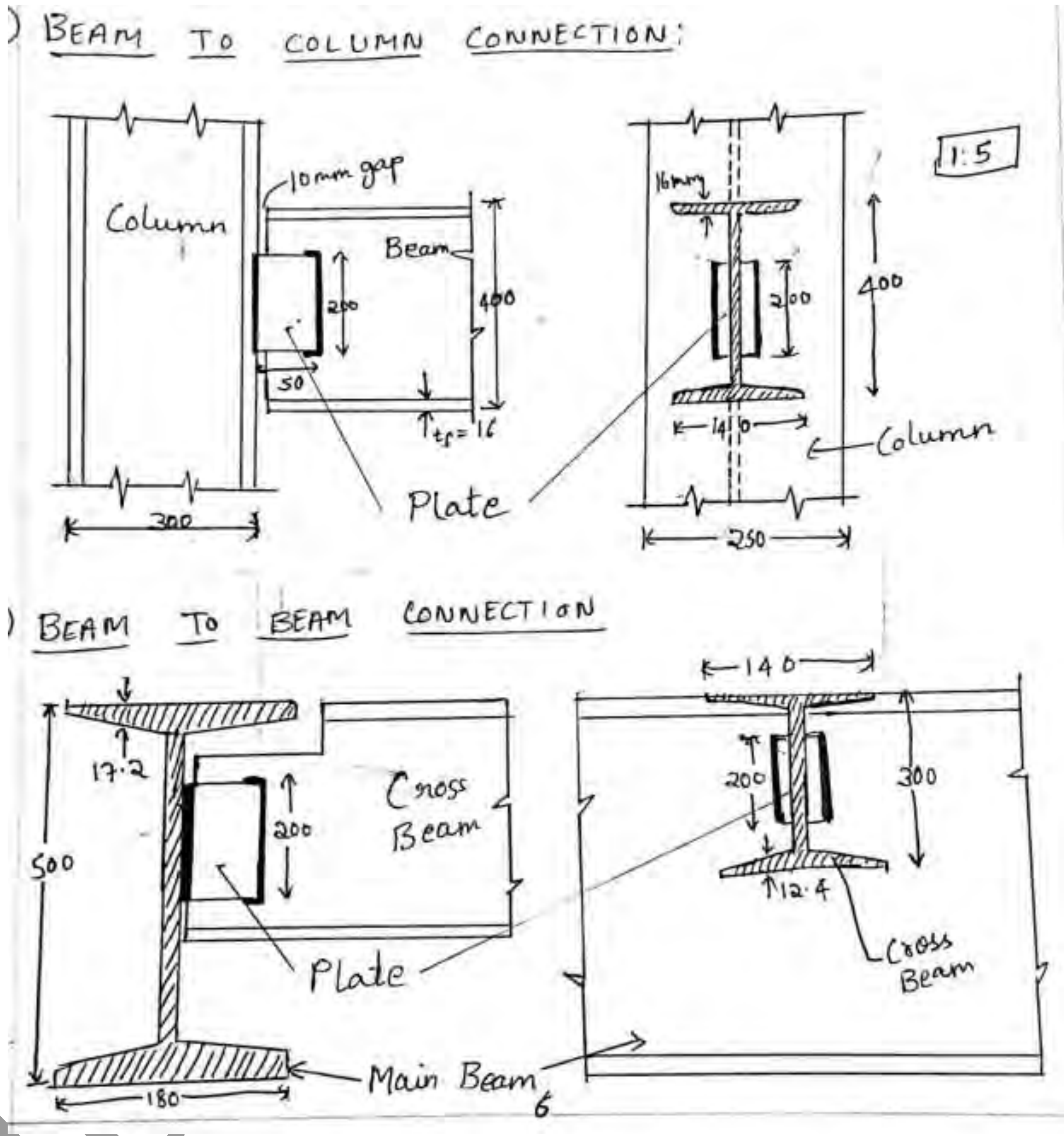
Main beam   ISMB-500@86.9kg/m

Plate-50mm width, 200mm depth and 10mm thick

a. Beam to column connection



b. Beam to beam connection



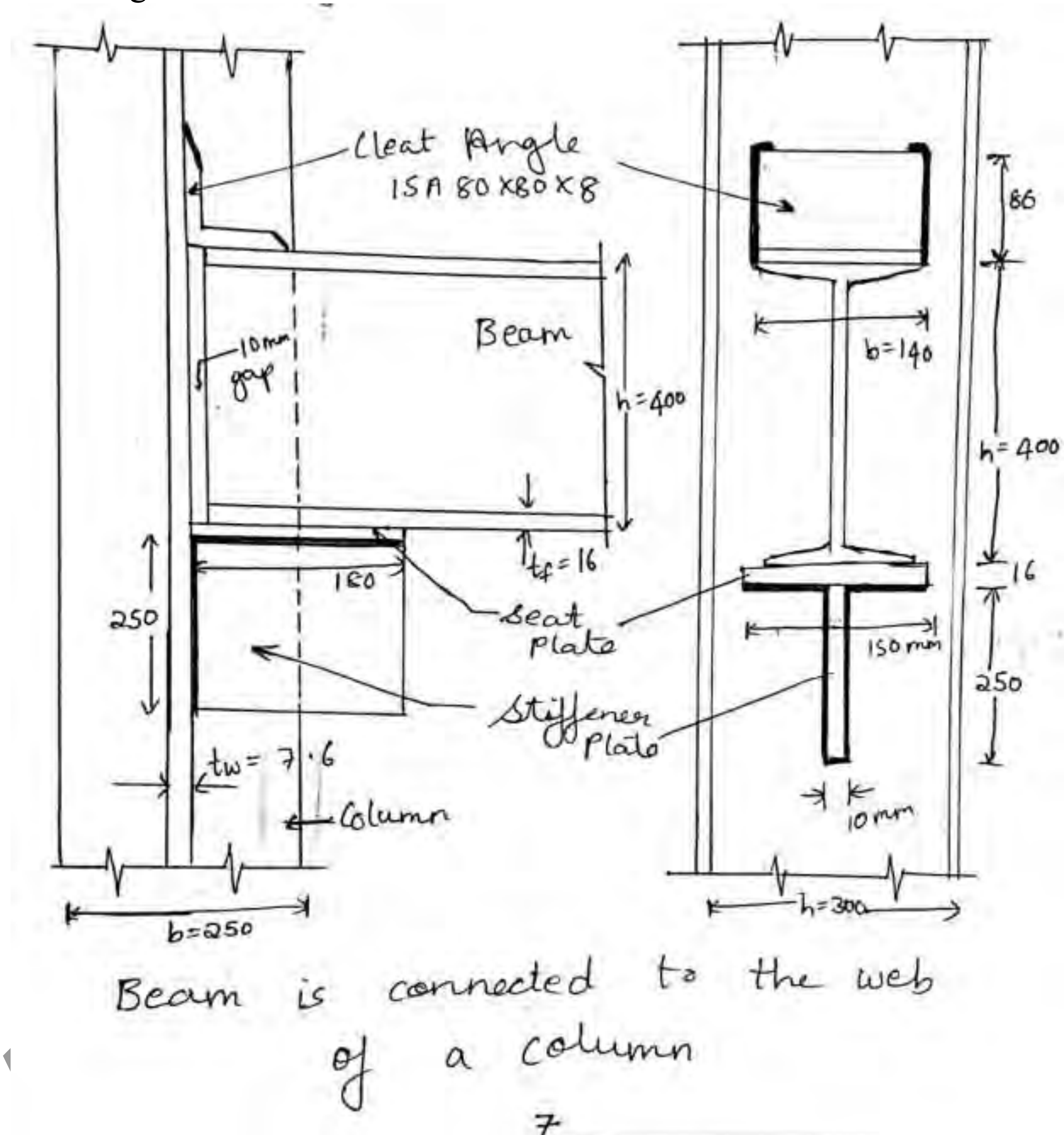
Beam-column shiftened seat connection:

Beam ISMB-400 @61.6 is connected to web of a column ISHB 300@58.8 kg/m using stiffened seat connection.

Seat connection-180mm*150mm*16mm

Stiffened plate-180mm width,250mm depth and 10mm thick.

Cleat angle- ISA 80*80*8mm



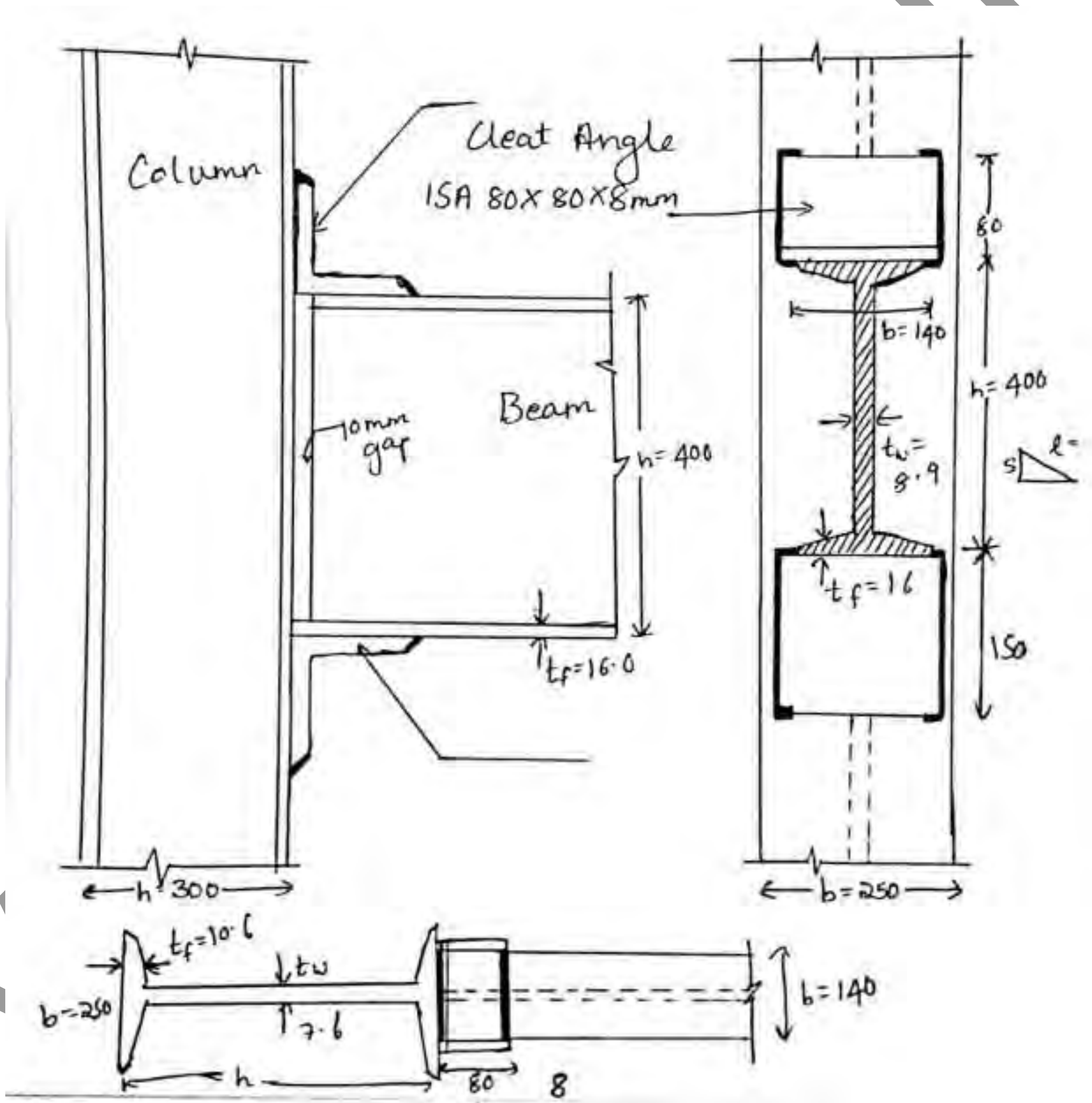
Beam-column unstiffened seat connection

Beam ISMB-400@61.6kg/m is connected to column ISHB-300@ 58.8kg/m using seat angle ISA 150*150*15.

Provide seat angle ISA 80*80*8m.

Use 12mm fillet held for main connection.

Beam is connected to flange of a column.



UNIT-2, UNIT-3

Part A: 1. Column with lacing

2. Battens

3. Column splices

4. Column gussetted base

5. Connection (batted and welded)

(a) Beam to beam and beam to column

(b) Beam to column unstiffened seat connection

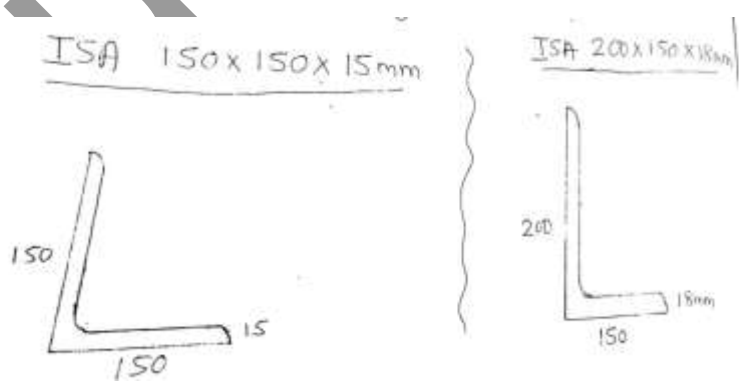
(c) Beam to column stiffened seat connection

Part B: Design and drawing

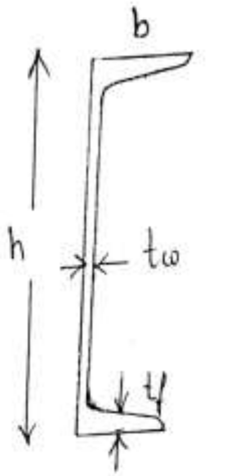
1. welded plate girder
2. bolted plate girder
3. Gantry plate girder
4. Roof truss (welded and bolted)

Rolled steel section:

a. **Rolled steel angles**



b. **Rolled steel channel section**



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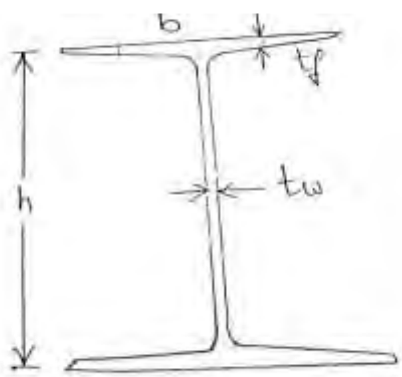
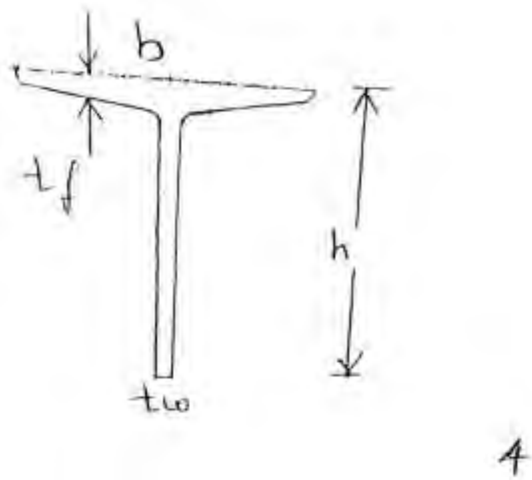
C. <u>Rolled steel beams</u>



3

<u>Rolled steel tee section</u>



A. Column with single lacing :

Build up column consists of 2ISMC-400@ 49.4kg/m back to back with a spacing 250mm

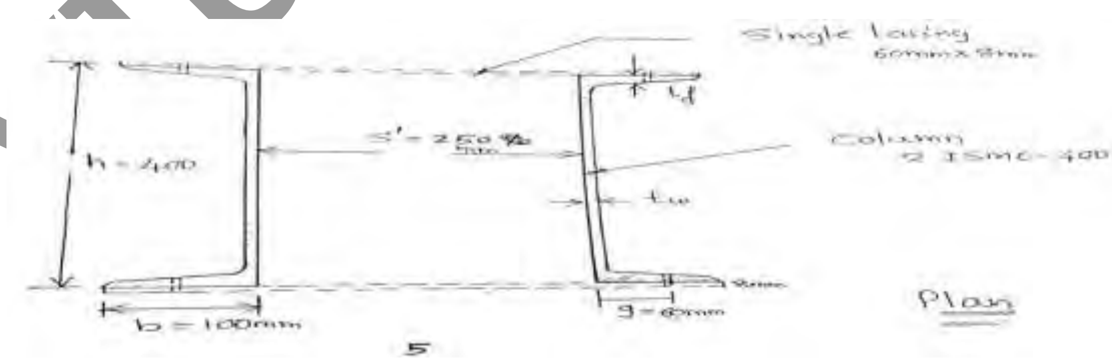
Lacing dimension-60mm*8mm

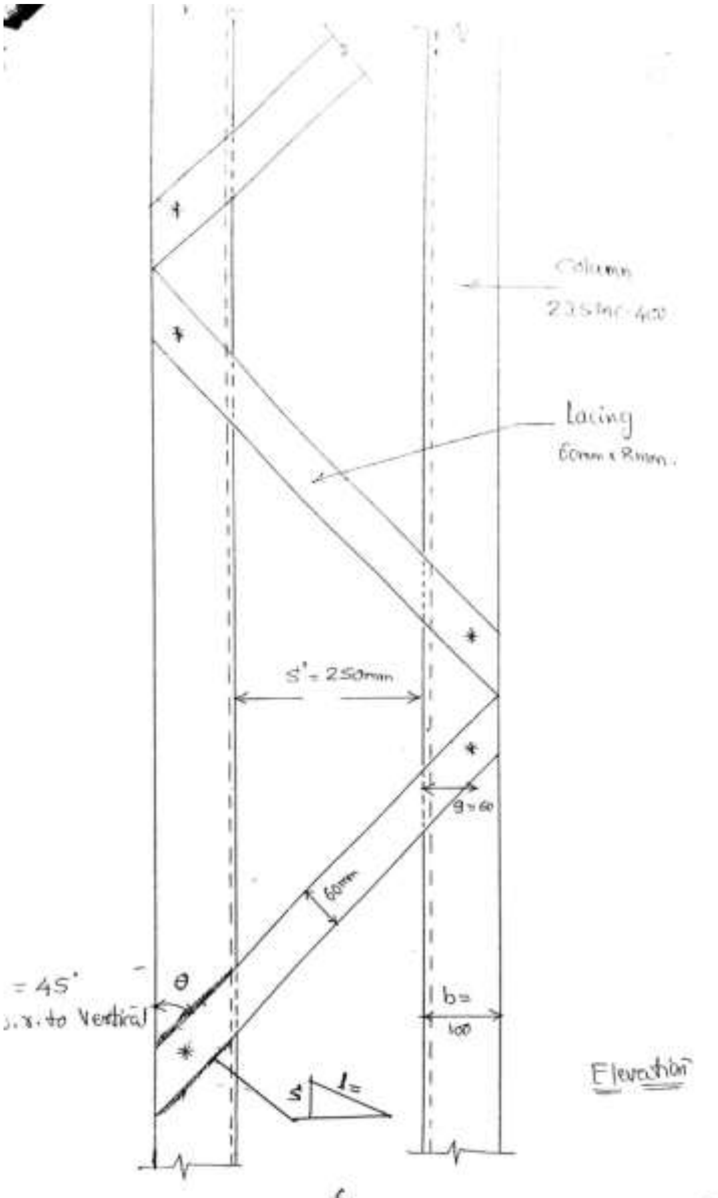
Inclination – 45degree (w.r.t vertical)

Provide one # 20mm bolts at each end.

IMSC- 400@9.9Kg/m

h=400, b=100, t_f=15.3, t_w=8.6





Column with Double lacing :

Build up column consists of 2 ISHB-300@ 63kg/m spaced at 350mm c/c

Provide double lacing 60mm*10mm

Inclination of lacing-45degree

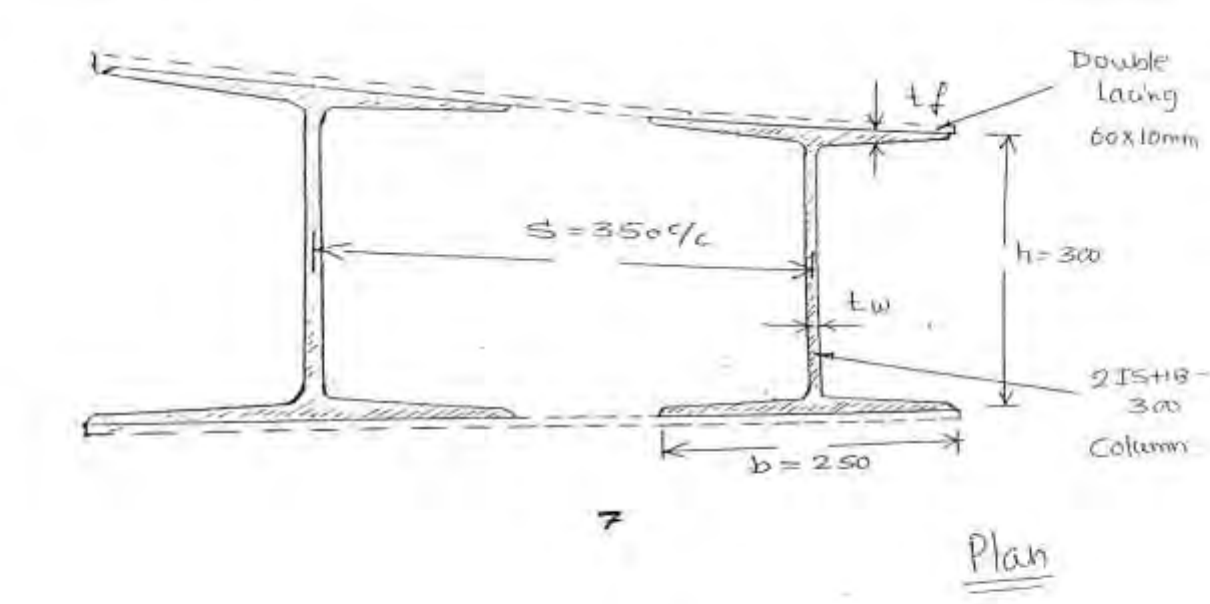
Provide 8mm size fillet weld for a length 50mm on either side.

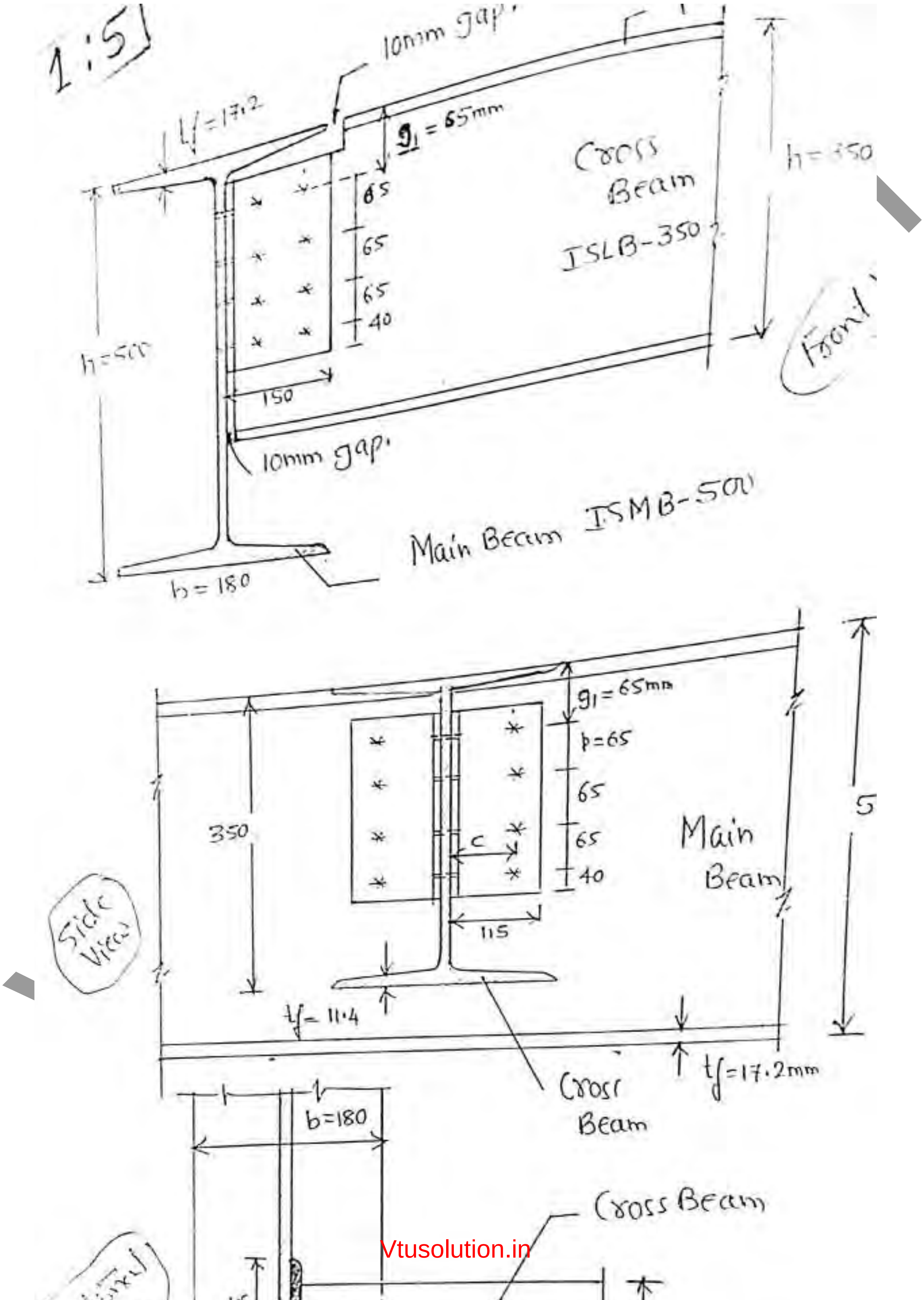
Soln: ISHB- 300@63Kg/m

h=300, b=250, t_f=10.6, t_w=9.4

Beam to column framed connection

A beam ISMB-450 is connected to "Flange of a column" ISHB-400@82.2kg/m by double angle framed connection.





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The size of the angle is ISA 100*100*10

There are 3# 20 per angle to connect cleat angle with beam web (pitch=6.5)

There are 3 # 20 per angle to connect cleat angle with column flange.

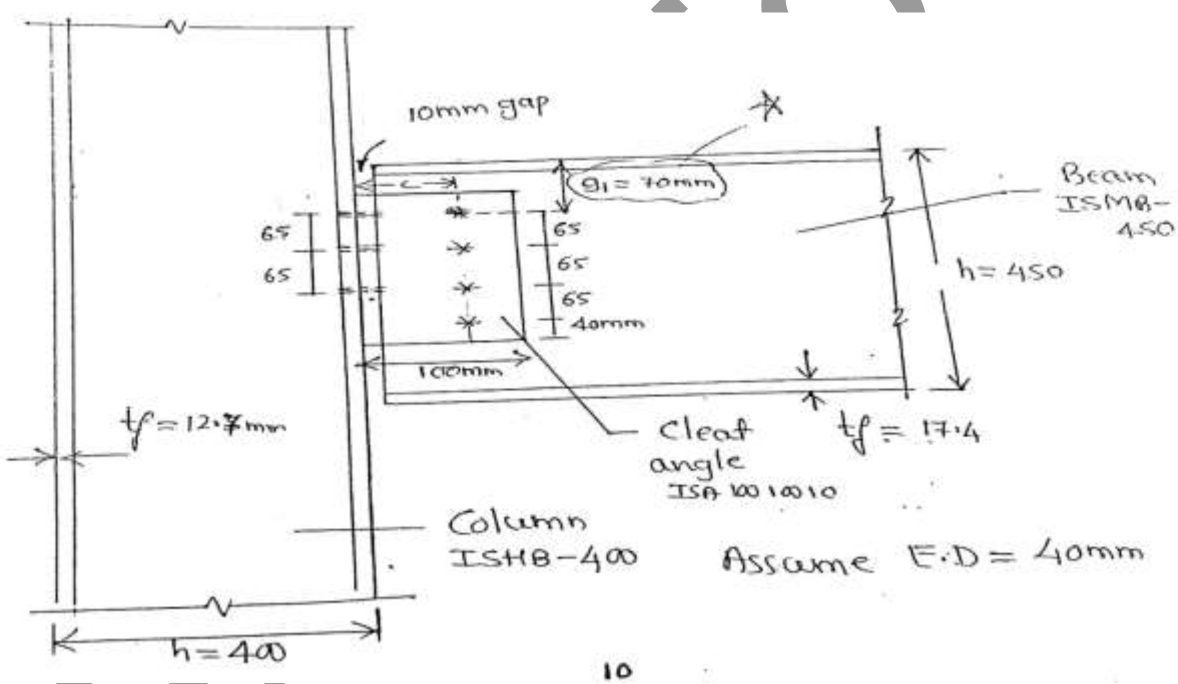
Soln: ISHB- 450

h=450, b=150, t_f=17.4, t_w=9.4,g=70

Soln: ISHB- 300

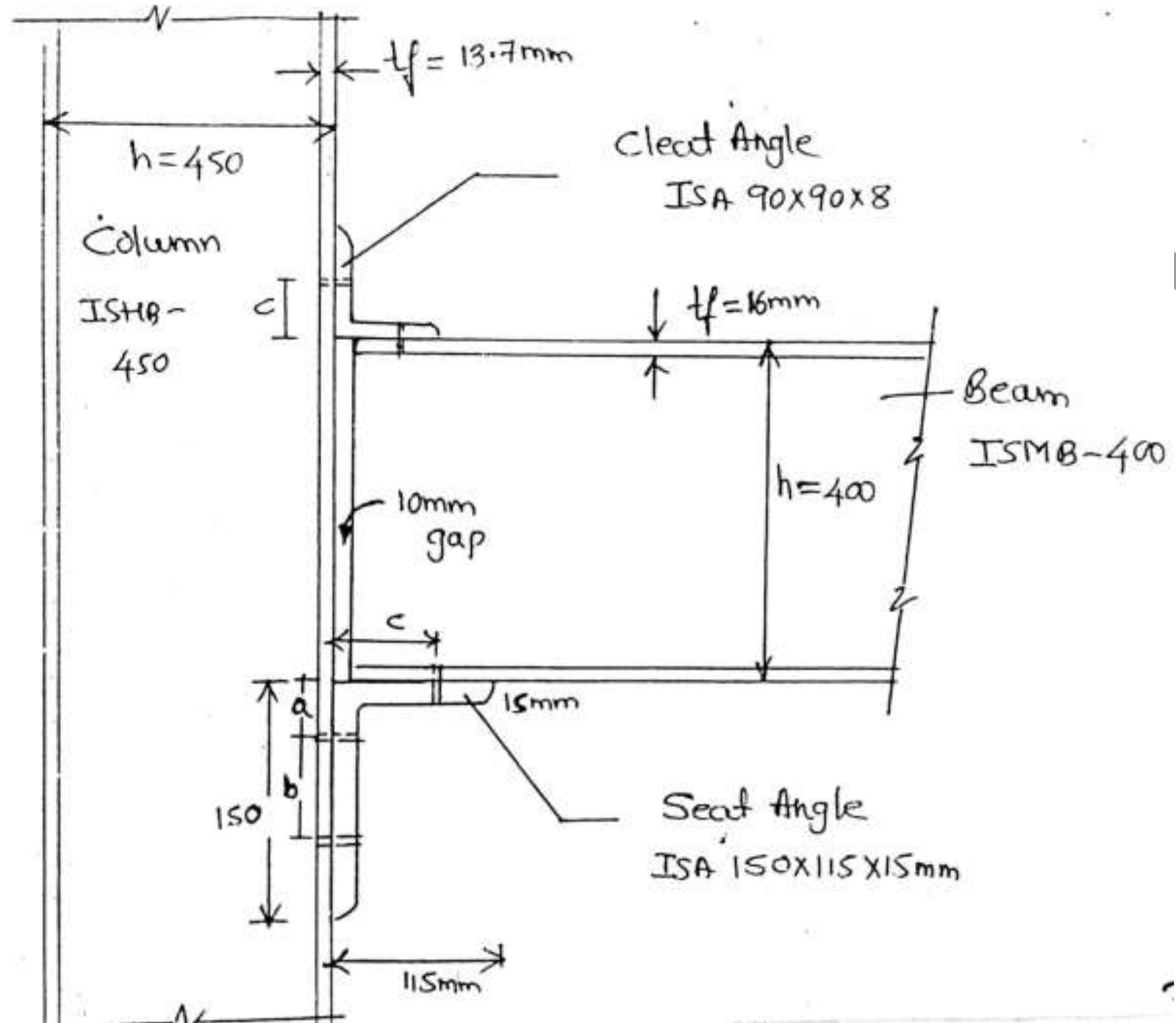
h=400, b=250, t_f=12.7, t_w=10.6

leg size=100 C=60mm



Beam to column un-stiffened seat connection

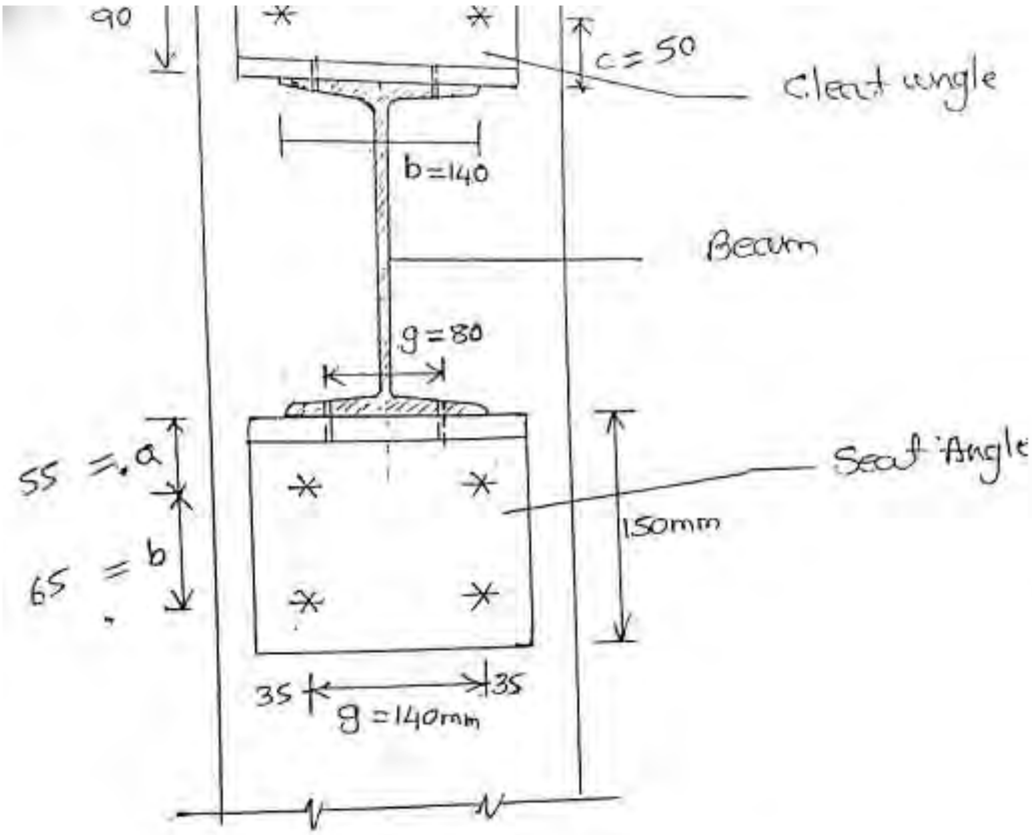
A secondary beam ISMB 400 @ 616N/m is connected to a flange of a column.

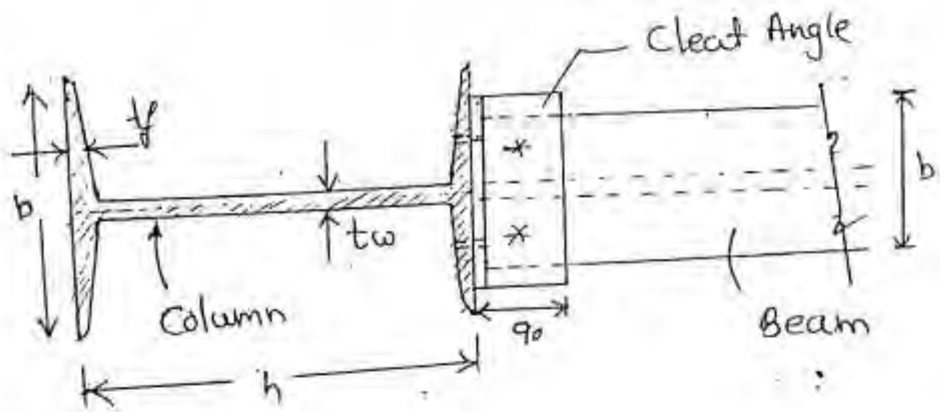


ISMB 400 @ 872 N/m by using a unstiffened seat connection.

Cleat angle is an ISA 90*90*8mm and seat angle ISA 150*115*15mm.

1.2 # 20 is used to connect cleat angle with column flange and 2 # 20 is used to connect seat angle with beam flange.





Beam to column stiffened seat connection

A secondary beam ISMB 400@ 616n/m is connected to the flange of the column

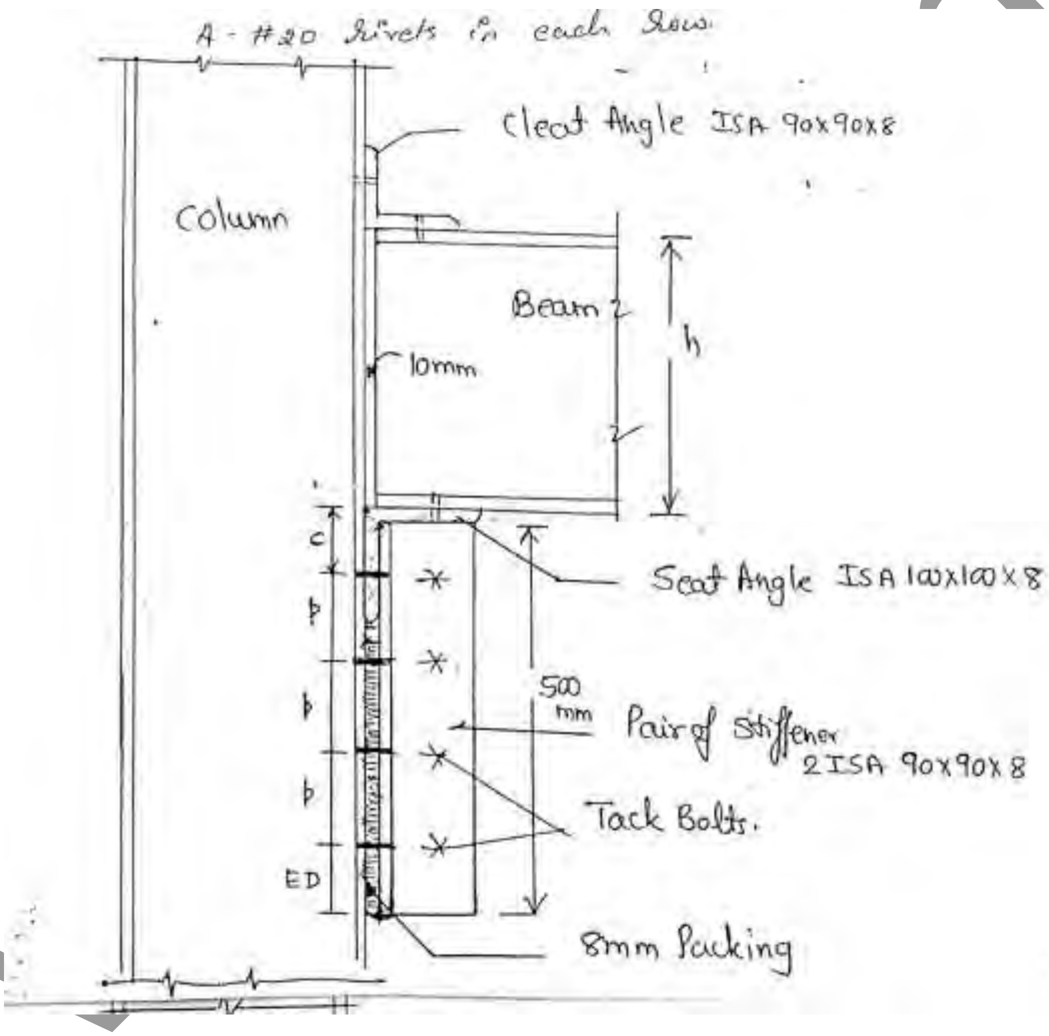
ISHB 450@ 872n/m by using stiffened seat connection.

Cleat angle is ISA 90*90*8 mm and seat angle ISA 100*100*8mm.

Use 2# 20 for all these angle connection. „

A pair of stiffener angle ISA 90*90*8mm and length 50cm

Stiffener angle are connected with two rows a # 20 rivts in each row.



Column bases

Slab base:

ISHB 400 @77.4 kg/cm^2 is supported on slab base.

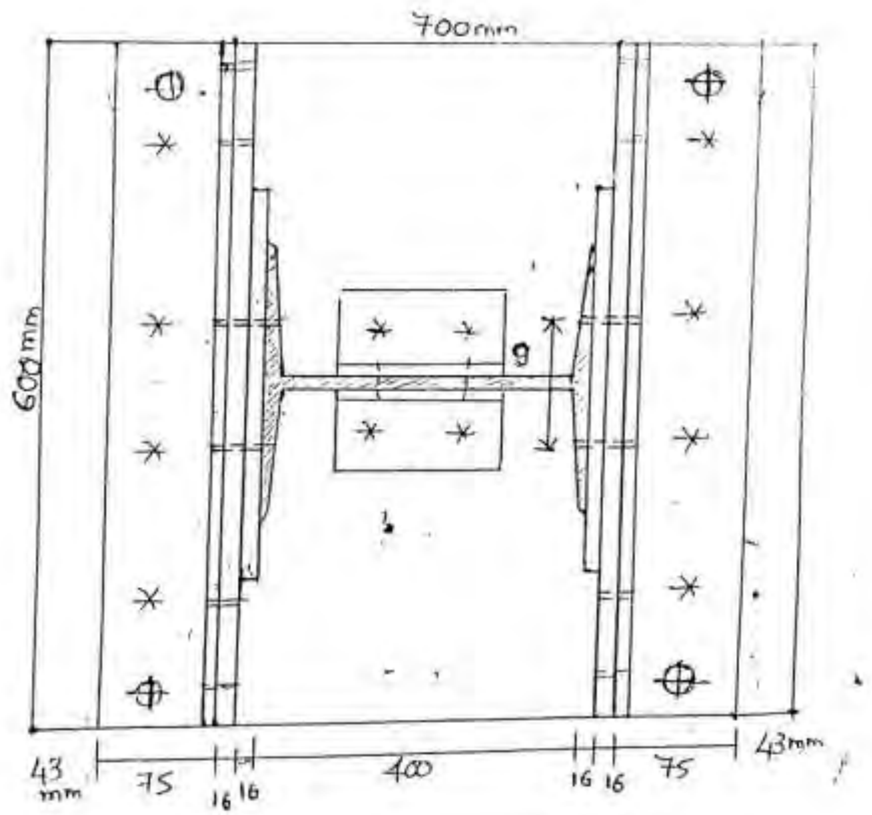
Size of the slab base 900mm*500mm*30mm

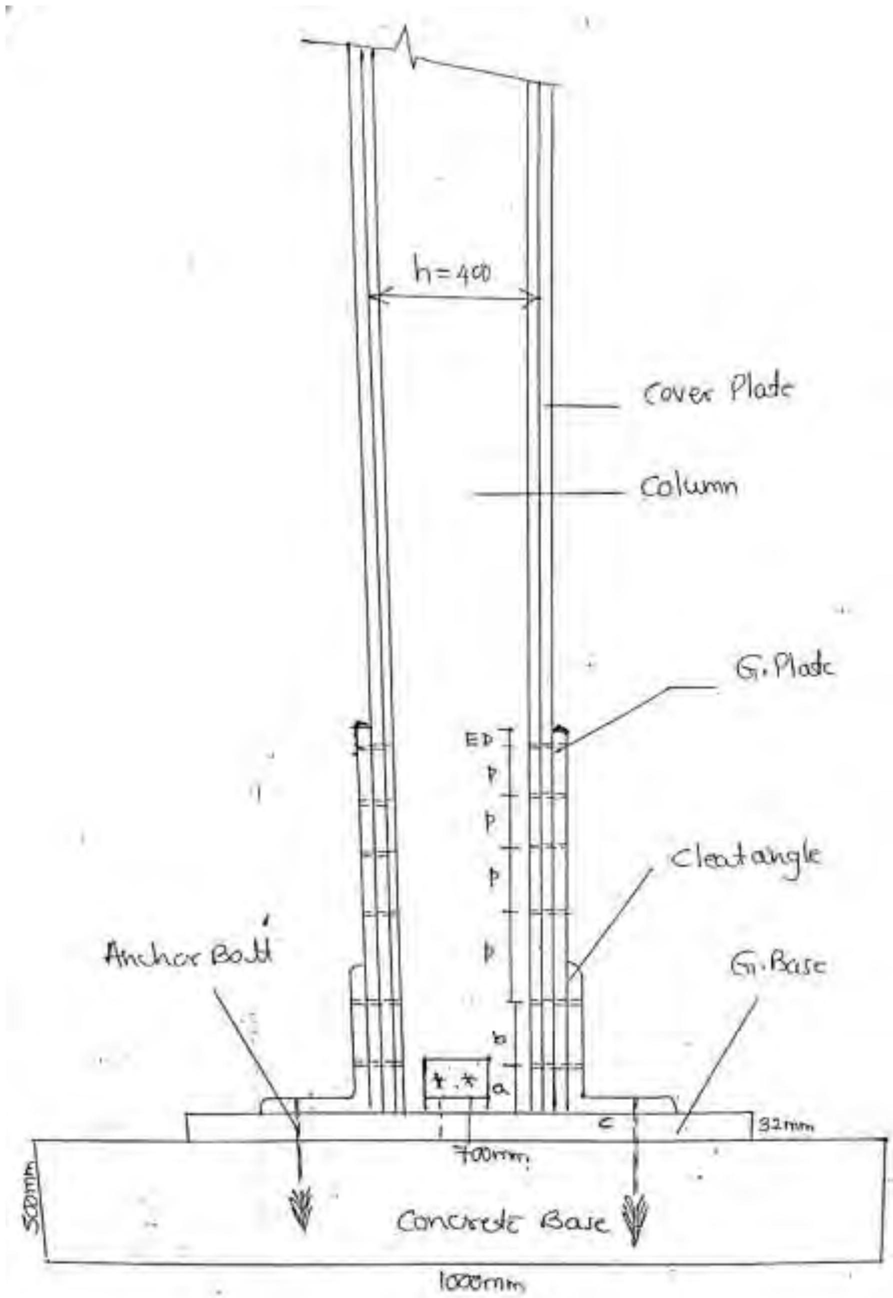
Size of the concrete base 1.5*1.5*1m

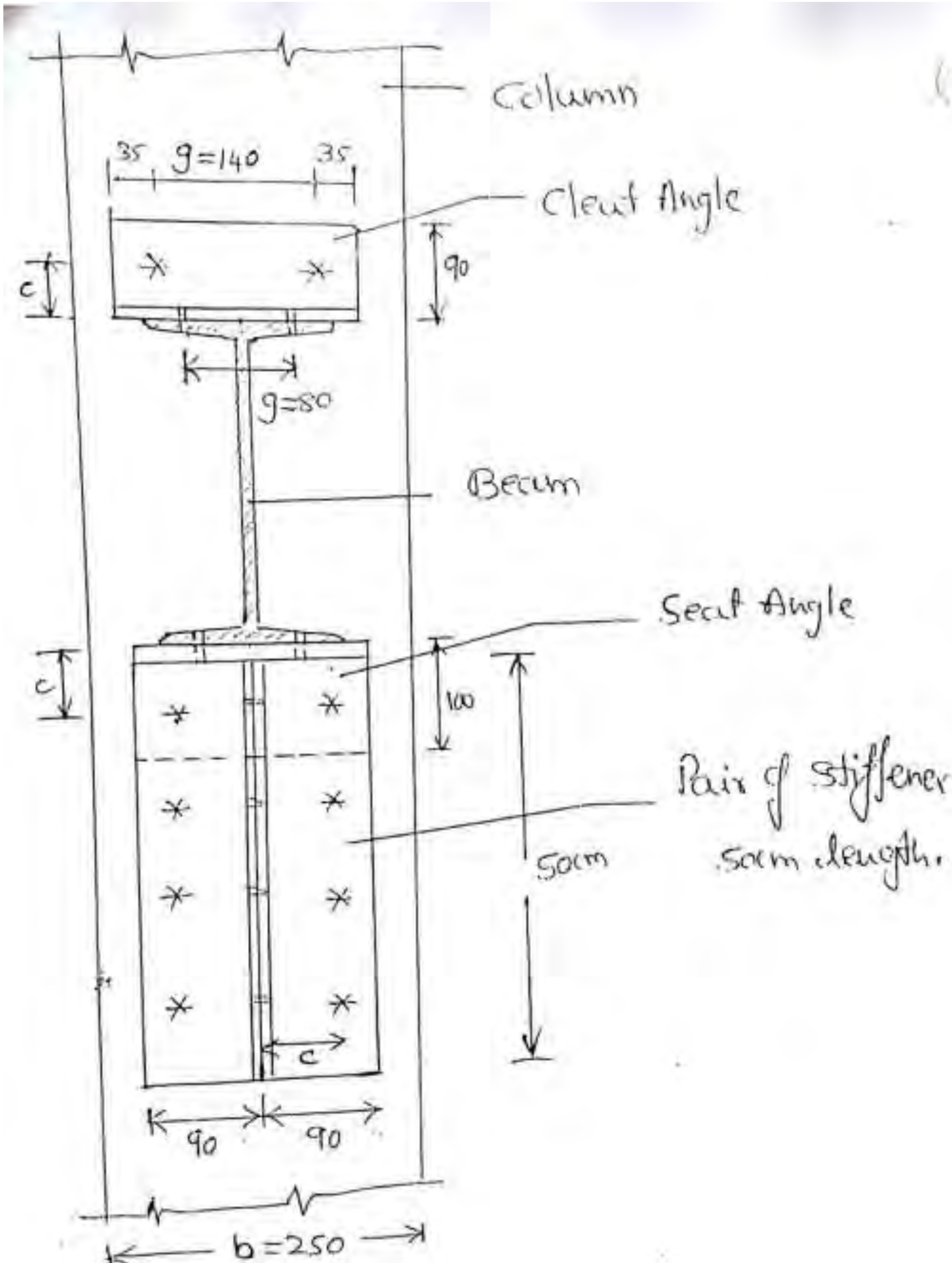
Size of the cleat angle ISA 150*115*10mm

Use 4- #24 di bolts between angle and column flange.

Provided 4 #20 dia anchor bolts.



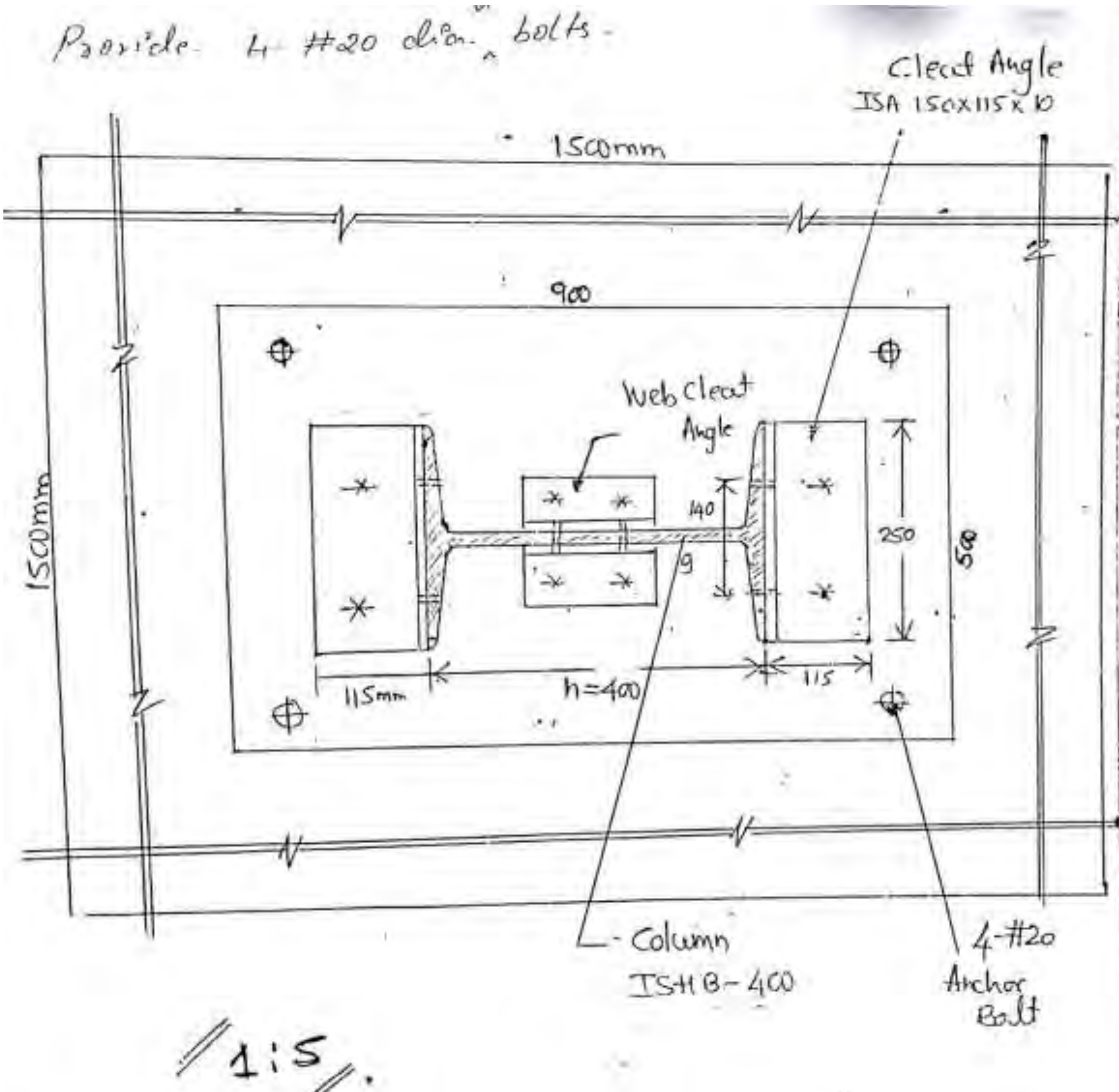

h=400
Cover Plate
Column
G. Plate
ED
p
p
p
p
Cleat angle
Anchor Bolt
G. Base
b
a
c
32mm
700mm
500mm
Concrete Base
1000mm



Column with battens :

Build up columns consists of 2 ISMC 300 @ 35.8kg/m placed face to face or toe 150m apart.

The column is connected to battens using welded connection size of the battens = 250mm* 10mm spacing @ 700mm c/c.



Column with gusseted base

Column ISHB-400@ 82.2 with cover plates 320mm*16mm is supported on gusseted base.

- The size of the gussetled base-700mm*600mm*32mm
- The thickness of gusselle plate-16mm
- The cleat angle-ISA 150*75*12

- There are 12 #20 bolt arranged.
- Two rows to connect pedestal 1m*1m*0.5m
- 4 # 16 anchor bolt are provided for connecting base plate with concrete.

<u>Column splices</u>

1. **Same column size**: Column splice is provided for two columns Ishb-250 @ 54.7 kg/m

a. Flange splice: 250mm*400*10mm"

<u>No</u> of rivets 6 # 18 for each column on each side with pitch= 60mm and E.D= 40 mm

b. Web splice: 140mm*160*10mm

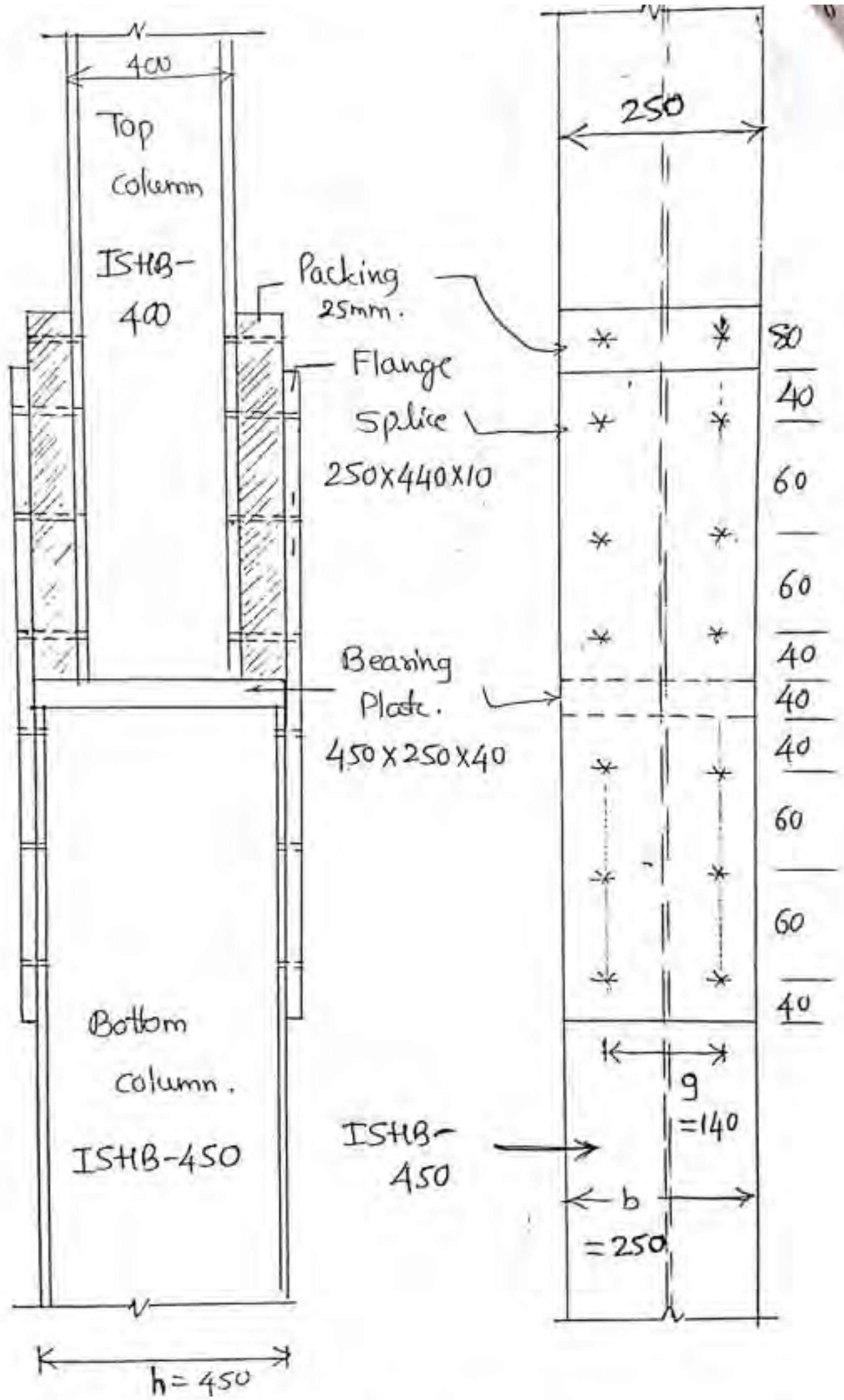
<u>No</u> of rivets 4 # 20 for each column

2. <u>Differential column sizes :</u>

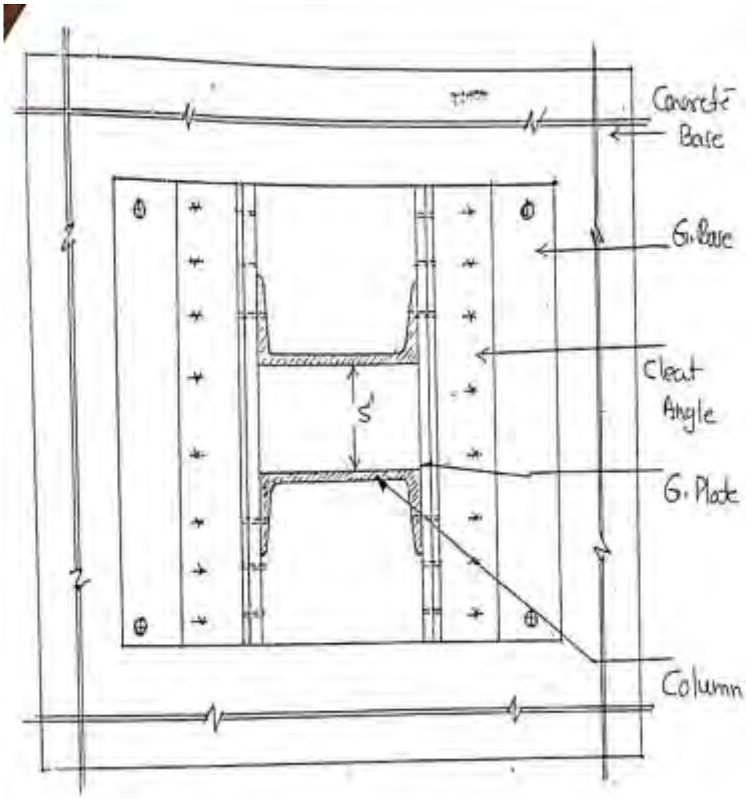
Column ISHB-400 @ 77.4kg/m is supported an another column ISHB-450 @ 87.2 kg/m

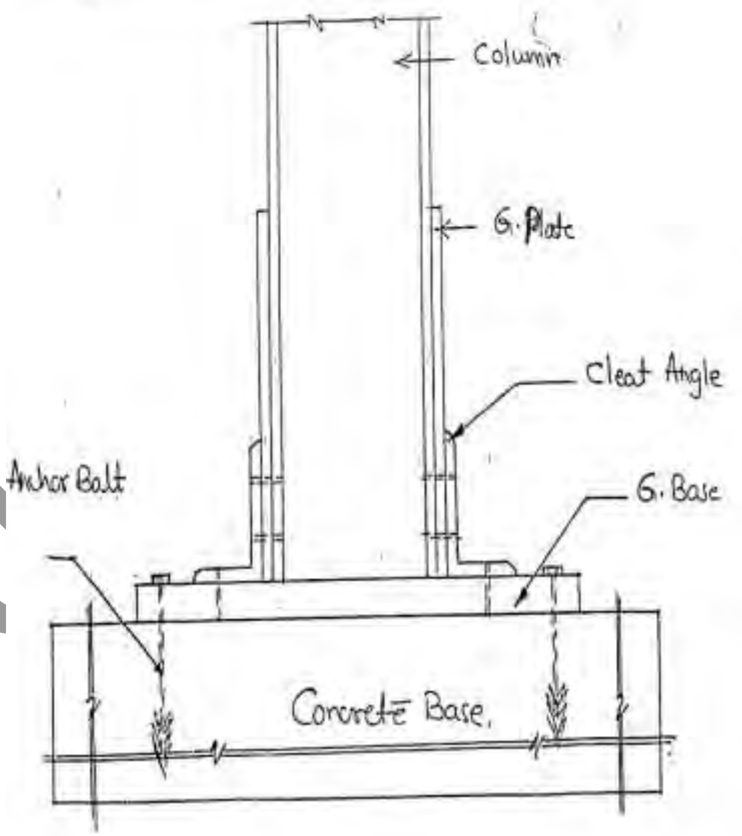
Provide suitable splices.

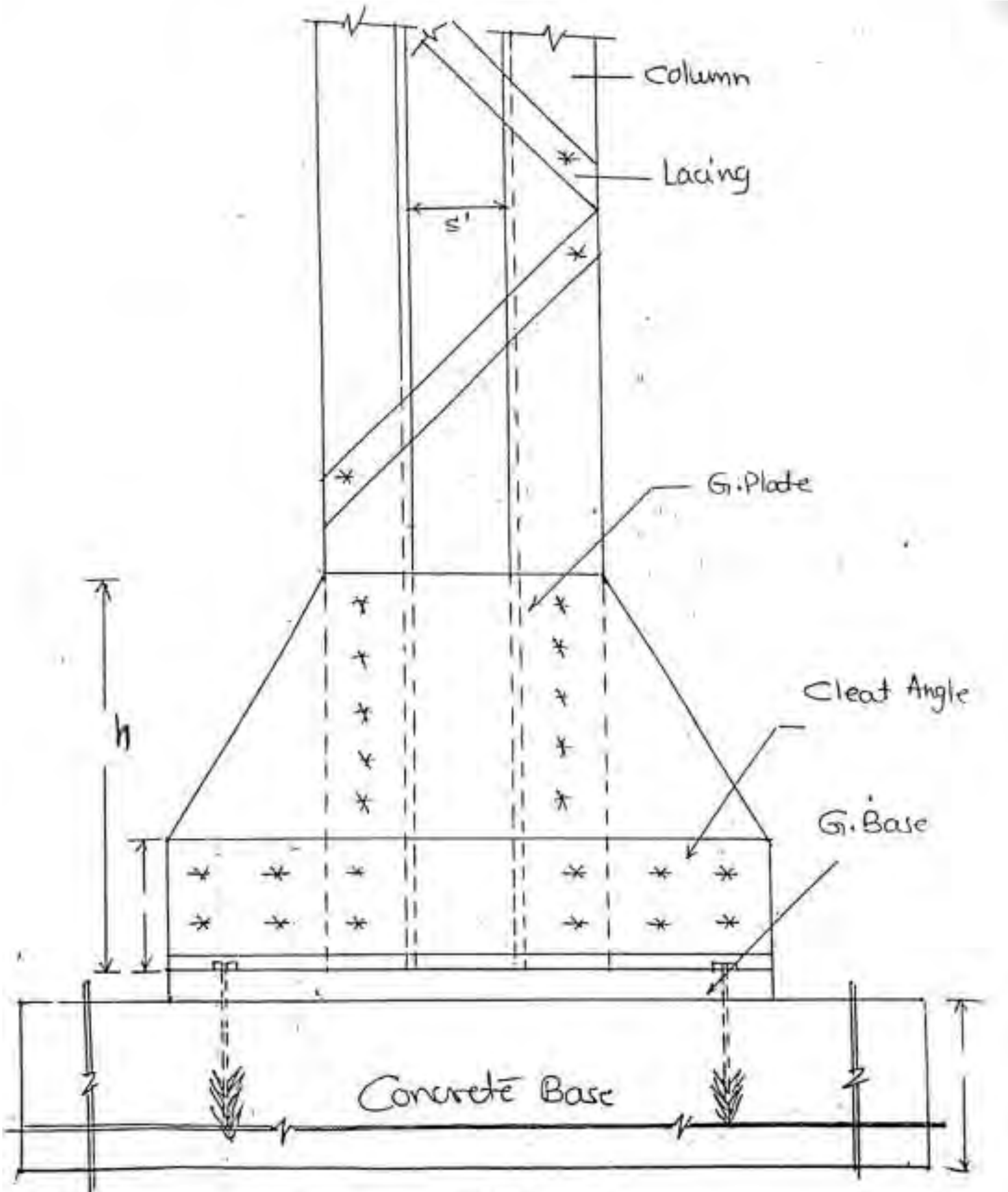
a. Depth of flange splices is 440mm and 10mm thickness.
b. Bearing plate of size 450mm*250mm*40 mm is provided in between two columns
c. There are 6 # 20 rivets for each column on each side with p=60mm and edge distance=40mm



Different size


Concrete Base
G. Base
Cleat Angle
S'
G. Plate
Column


Column
G. Plate
Cleat Angle
Anchor Bolt
G. Base
Concrete Base.



$h = a + b + 5(p) + ED$

UNIT -3 COLUMN BASE SUBJECTED TO MOMENT

Design a Slab base for a column ISHB-300@ 58.8 Kg/m to carry a load of 600 KN and moment 10 KN.

Also design suitable pedestal and the welded connection. Take compressive strength of concrete 20 N/mm^2 and SBC of soil 200 KN/m^2.

Solution: Given, P=600 KN & M=10 KN-m

Therefore, P_U=900 KN & M_U=15 KN-m

(a)Size of the plate :

For column ISHB- 300 @ 58.8 Kg/m

B=250 mm, h=300 mm

Provide plate width B=250+50mm projection on both sides

Therefore, B=350 mm

Length is calculated using

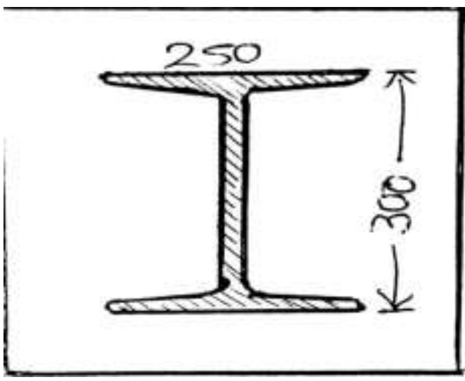
Bearing pressure, $w = \frac{p_u}{B*L} + \frac{6*M_u}{B*L^2}$

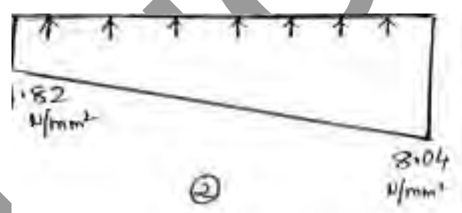
$$0.45 * 20 = \frac{900 * 10^3}{350 * L} + \frac{6 * 15 * 10^6}{350 * L^2}$$

L=364.2mm

Provide Slab Base

$$B * L = 350mm * 400mm$$





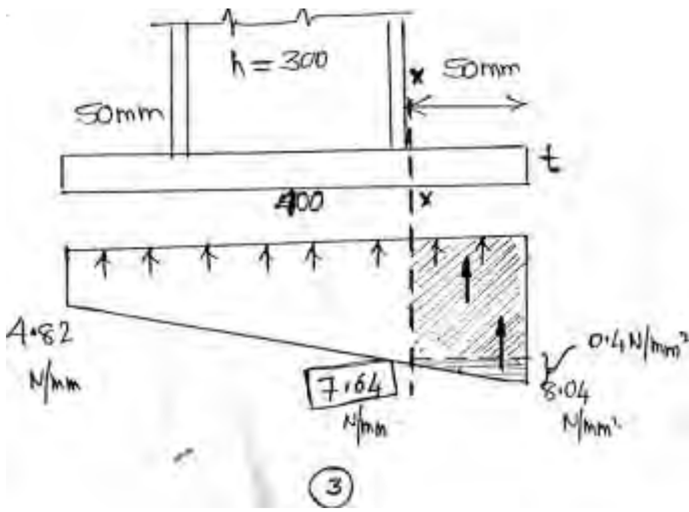
Hence with above dimensions, the Upward pressure

$$\sigma_{max} \ \& \ \sigma_{min} = \frac{p_u}{B * L} \pm \frac{6 * M_u}{B * L^2}$$

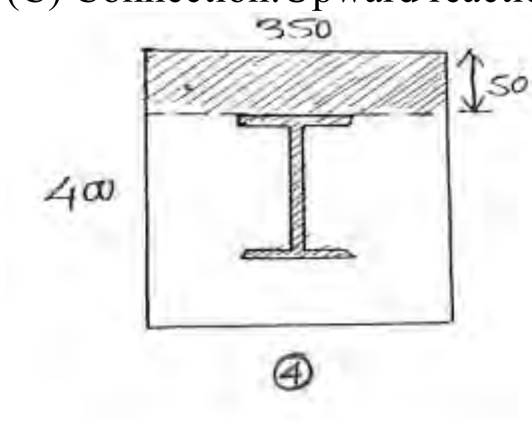
$$= \frac{900 * 10^3}{350 * 400} + \frac{6 * 15 * 10^6}{350 * 400^2}$$

$$\sigma_{max} = 8.04 \frac{N}{mm^2}$$

$$\sigma_{min} = 4.82 \frac{N}{mm^2}$$



(C) Connection:Upward reaction on shaded area



Upward reaction on shaded area = (avg upward pressure) (shaded area)

$$= (\frac{8.04 + 7.64}{2}) (350 * 50)$$

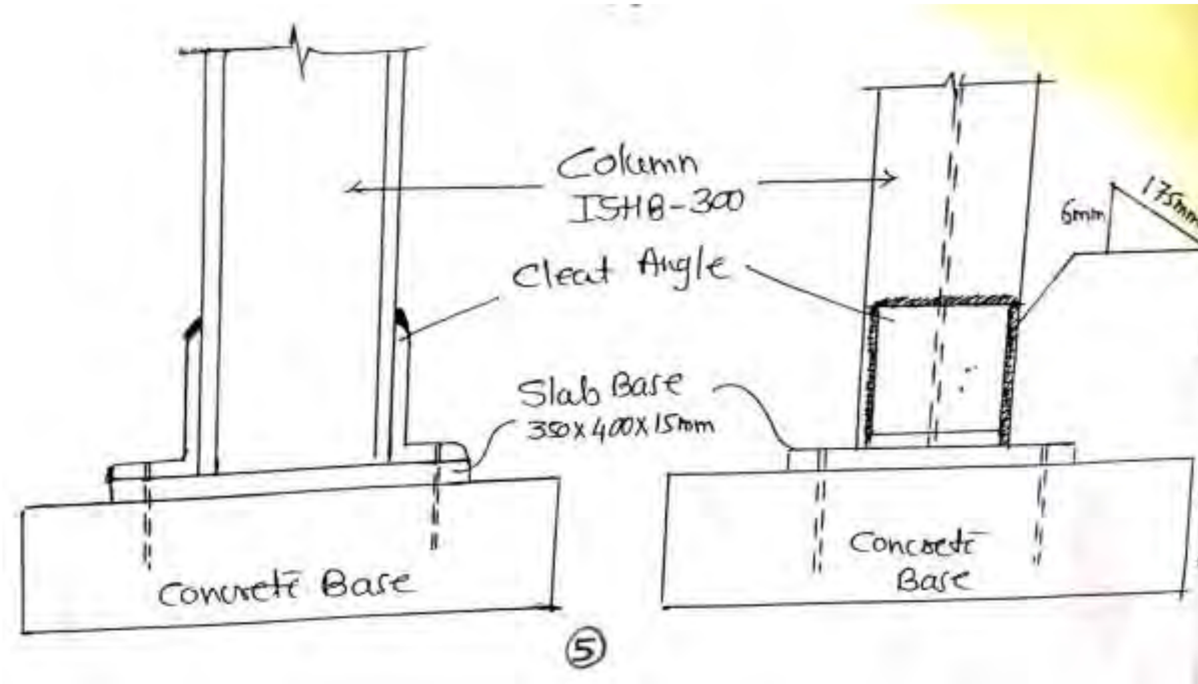
=137200 N

Using 6 mm size fillet weld,

$$Force \;=\; strength\; of\; weld$$

$$137200 = (.7 * 6\,)l * \frac{410}{\sqrt{3}*1.25}$$

Therefore, l = 175 mm



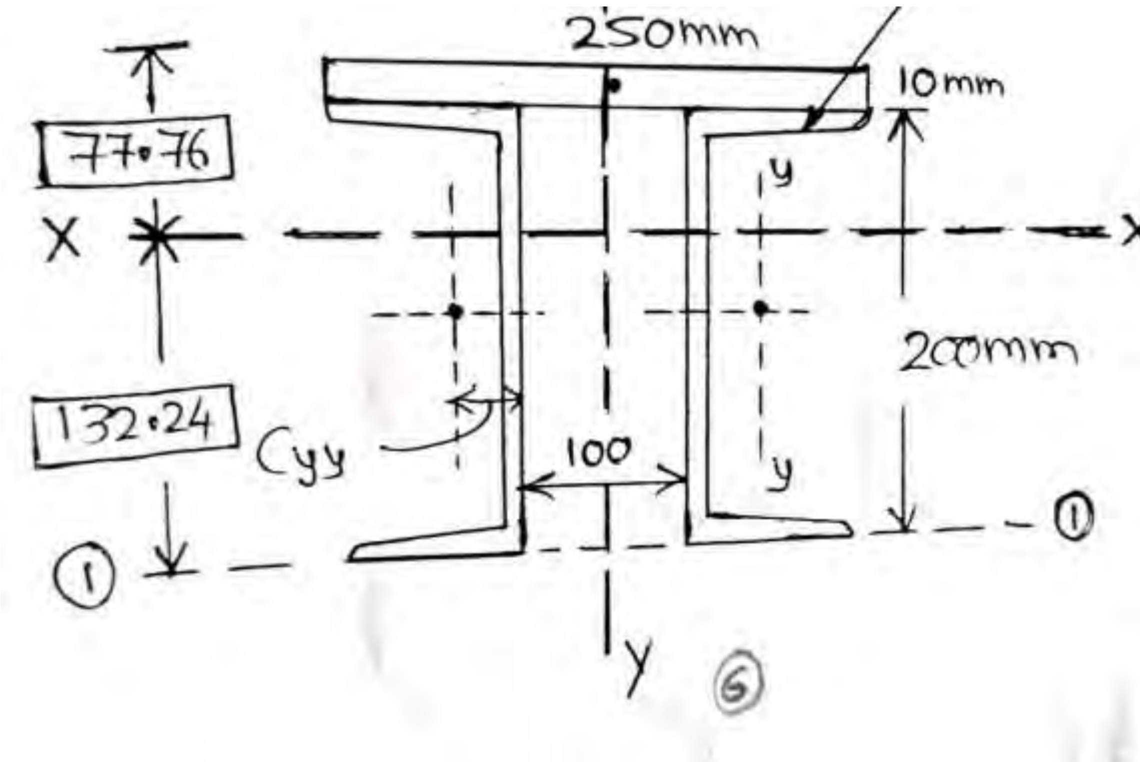
(d). Design of concrete base:

COLUMNS:

2). The top chord of a bridge truss having an effective length of 3.6m has a c/s shown. Determine the compressive load this member can carry.

Properties of one channel

ISMC- 200 @ 22.1 Kg/m



area=$28.21 * 100\ mm^2$

$$I_{xx} = 1819.3 * 10^4\ mm^4$$

$$I_{yy} = 140.4 * 10^4\ mm^4$$

$$C_{yy} = 21.70\ mm$$

Location of x-x axis :

$$y = \frac{2(2821)100 + (250 * 10)205}{2(2821) + (250 * 10\ )}$$

y =132.24 mm

$$I_{XX} = I_{xx} + ab^2$$

$$I_{XX} = 2\left[1819.3 * 10^4 + (2821)\left(132.24 - \frac{200}{2}\right)^2\right] + \left[\frac{250 * 10^3}{12} + (250 * 10)\left(77.76 - \frac{10}{2}\right)^2\right]$$

$$I_{XX} = 55.51 * 10^6$$

$$I_{YY} = 2\left[140.4 * 10^4 + (2821)\left(\frac{100}{2} + 21.70\right)^2\right] + \left[\frac{250 * \ 10^3}{12}\right]$$

$$I_{YY} = \ 44.83 \ * 10^6$$

$$I_{min} = 44.83 * 10^6 \ mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}}$$

$$r_{min} = \sqrt{\frac{44.83 * 10^6}{(250 * 10) + 2(2821)}}$$

$$= 74.20mm$$

$$\lambda = \frac{l_e}{r_{min}}$$

$$= \frac{3600}{74.20}$$

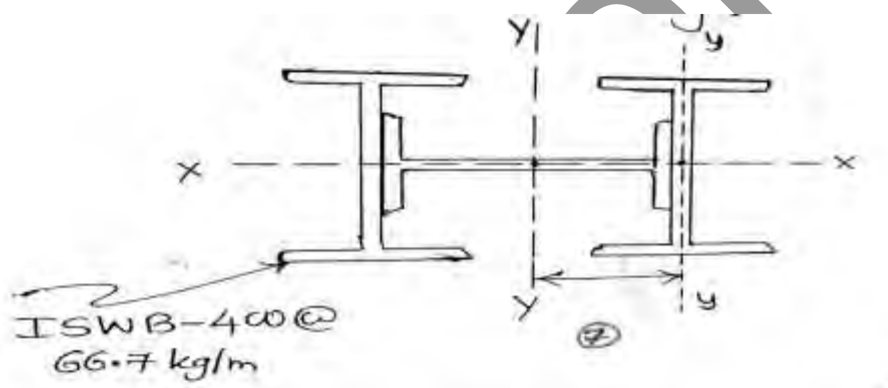
$$= 48.52$$

$$therefore \qquad f_{cd} = 185.22\frac{N}{mm^2}$$

$$therefore \qquad load\ ,p = (185.22)\big(250 * 10 + 2(2821)\big)$$

$$P = 1508.10\ KN$$

3) A built up column consist of 3-rolled steel I-beam ISWB-400 @ 66.7 kg/m and connected effectively to act as one column as shown. Take le=4m. Determine safe load the column can carry.



soln :

$$Total\ area = 3 * 8501$$

$$= 25503mm^2$$

$$I_{XX} = 2[23426.7 * 10^4 + (8501(0))] + [1388 * 10^4 + (8501(0))]$$

$$= 482.41 * 10^6 mm^4$$

$$I_{YY} = 2\left[1388 * 10^4 + (8501)\left(\frac{400}{2} + \frac{8.6}{2}\right)^2\right] + [23426.7 * 10^4]$$

$$= 971.66 * 10^6 mm^4$$

$$I_{min} = 482.41 * 10^6$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}}$$

$$r_{min} = \sqrt{\frac{482.41 * 10^6}{25503}}$$

$$= 137.53 mm$$

$$\lambda = \frac{l_e}{r_{min}}$$

$$= \frac{4000}{137.53}$$

$$= 29.08$$

$$therefore \qquad f_{cd} = 212.2 \frac{N}{mm^2}$$

$$therefore \qquad load\ ,p = (212.2)(25503)$$

$$P = 5412\ KN$$

UNIT-4 ROOF TRUSS

Points to remember:

- Design totally 4 members(2-outer,2-inner)
- Use equal angles
- Minimum size of angle- ISA 50mmX5OmmX6mm
- For outer members – Double angles

  Inner members – Single angle
- Take effective length l_e=0.85l
- Provide a minimum of 2 bolts
- Gusset plate thickness is same as angle thickness or more and it is uniform throughout.

1) Line diagram of a roof truss with external load and forces in each member along with nature are shown in fig. below. Design various members of the roof truss along with their end connections with gusset plate[welded or bolted].

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

Also design anchor bolts for an uplift of 15KN 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.

A) Design of top chord member

{AB,BC & CD Members}

Taking maxium force = 240 KN(compression)

Factored force= 360 KN

Max length = 2.31m = l

Effective length l_e=0.85l=1.964m

a) Assume f_{cd}=120N/mm^2

Area = force/f_{cd} = 360X10^3/120 = 3000mm^2 = 30cm^2

From steel table try 2ISA 80X80X10mm

Area = 30.10cm^2 = 3010mm^2

Υ_{xx} = 2.41cm, Υ_{yy} = 3.73cm

Υ_{min} = 24.1mm

$\lambda = l_e / \Upsilon_{min}$ = 81.3

From table 9(c) P-42 IS – 800

$F_{cd} = 134.05\ N/mm^2$

Design compressive load = $P_d = f_{cd}$.Area

$P_d = (134.05)(3010) = 403.4x10^3 N > 360KN$ (Safe)

b) Connection

Using M_{20}, property class 5.6 Black Bolt

i) Shear strength

$V_{dsb} = V_{nsb}/\Upsilon_{mb}\ [f_y/\sqrt{3}]$ No shank

= 113.18KN

ii) Bearing strength

P = 2.5d = 2.5X20= 50mm

e = $1.7d_o$ = 1.7 X 22 = 37.4 = 40mm

$k_b = (e/3d_o) = 0.61, (p/3d_0 - 0.25) = 0.51$

$(f_{ub}/f_u) = 500/410 = 1.22, 1.0$

$K_b = 0.51$

$V_{dpb} = V_{nsb}/\Upsilon_{mb}[2.5k_b.d.t.f_u]$

1/1.25[2.5x0.51x20x10x410]=83.64KN

Bolt value = 83.64KN

No. of bolts = Forcw/BV = $360X10^3/83.64X10^3$ = 5

B) Design of bottom chord member

[Members AL,LK and KJ]

Max. force = 207.84KN

Factored = 311.76KN

L= 2.0 m

L_e= 0.85l = 1.7m

a) For preliminary sizing

$T_{dn}=\frac{\propto An.fu}{\gamma m1}$

α = 0.8 Assuming no. of bolts more than 4

$311.76X10^3 = 0.8XA_n x410/1.25$

$A_n = 1188.1mm^2$

Increase approximately by 30%

$= 1.30X1188.1= 1544.5mm^2 = 15.44cm^2$

From steel tables try 2ISA 80X80X6mm

Area = $18.58cm^2 = 1858mm^2$

b) Connection

Let us try M_{20} P Class 8.8 HSFG Bolts

$V_{dsf} = V_{nsf} / \Upsilon_{mf} = 1/ \Upsilon_{mf} [\mu_{f.} n_e k_h F_{o]}]$

$\Upsilon_{mf} = 1.25$, $\mu_f = 0.55$, $n_e = 2$

$K_h = 1.0$

$F_O = A_{hb}.f_o = A_{hb}(0.7f_{ub})$

$= 0.78 \frac{d^2}{4}(0.7f_{ub})$

$= 137.2X10^3$

$V_{dsf} = 1/1.25[(0.55)(2)(1)(13702x10^3)]$

$= 120.75KN$

No.of bolts = 311.76/120.75 = 3

c) Check for rupture

w=80mm, t=6mm, $b_s = w + w_1 - t$

$b_s = 80+45-6 = 119mm$, $L_c = 100mm$

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

$\beta = 0.665$, Take min $\beta = 0.7$

$A_{go} = (B-t/2)t = (80 – 6/2)6 = 462$

$A_{nc} = (A-d_o-t/2)t = (80-22-6/2)6 = 330$

T_{dn}= $2[0.9\,\frac{Anc.fu}{\Upsilon m1} + \beta\,\frac{Ago.fy}{\Upsilon m0}]$ = 334.4 KN > 311.76 KN (Safe)

d) Check for block shear

L_V = 140mm, L_t = 35mm, t= 6mm

$A_{vg} = L_V$ X t = 840

A_{vn} = 840-2.5X22X6 = 510

$A_{tg} = L_t$ X t = 210

A_{tn} = 210-0.5X22X6 = 144

T_{db} = 305.46

T_{db} = 269.00

Unsafe

Hence revise the angle section

Now try 2ISA 80x80X8mm, Area = 2442 mm^2

Re check only Block shear

L_V = 140 mm, L_t= 35mm, t = 8mm

$A_{vg} = L_t$ x t = 1120, $A_{tg} = L_t$ x t = 280

A_{vn} = 1120-2.5X22X8 = 680 , A_{tn} = 280-0.5X22X8 = 192

T_{db} = 407.0 KN

T_{db} = 359 KN > 311.76KN

Hence provide 2ISA 80X80X8mm with 3-M_{20} Bolts for bottom chord.

Design of Inner compression

[Member BK,CJ and DJ]

Max. compression = 66.05

Factored load = 99.07 KN

Max. length = l = 3.46m

L_e = 0.85l = 2940mm

a) Assume f_{cd} = 70N/mm^2

Area = 99.07X10^3/70 = 1415.2 mm^2 = 14.15cm^2

From steel Tables Try single angle

ISA 80X80X10mm

Area = 1505 mm^2

Υ_{vv} = 15.5 mm

Loaded through one leg

Refer Page 48

Assume more than two bolts and hinged

K_1=0.7, k_2=0.6, k_3=5

$\Lambda_{vv} = \dfrac{l/r_{vv}}{\sqrt{\ ^2E/250}}$

$= 2.13$

$\lambda_{\Phi} = \dfrac{(b_{1+b_2})/2t}{\sqrt{\frac{2E}{250}}} = 0.09$

$\lambda_e = \sqrt{k_1 + k_2\lambda_{vv} + k_3\lambda_{\Phi}^2} = 1.86$

From Page 34

For buckling class c , $\alpha = 0.49$

$\Phi = 0.5[1+\alpha(\lambda_e-0.2)+\lambda_e^2] = 2.64$

$F_{cd} = \dfrac{fy/\Upsilon m0}{\Phi+[\Phi^2]}0.5$

$= 50.35\ N/mm^2$

Design compressive strength $P_d = f_{cd}XArea$

$P_d = (50.35)(1505) = 75.7X10^3 < 99.07KN$ (Unsafe)

Hence revise the section

Try "ISA 100X100X10mm"

Area= $1903mm^2$, Υ_{VV} = 19.4mm

Λ_{vv} = 1.70, $\lambda_{\Phi\Phi}$= 0.112, λ_e=1.58

Φ= 2.08

F_{cd}= 66.20 N/mm^2

P_d= F_{cd}XArea= 66.20 X 1903 = 126KN> 99.07 (Safe)

b) Connection : Let us provide "welded connection"

Size of the weld

S=3/4 X Angle thickness

S = 3/4 X 10 = 7.5mm

Take s= 7mm

Force = strength of the weld

$99.07 \times 10^3 = 0.7 \text{ X S X l X } \frac{f_u}{\sqrt{3X1.25}}$

= (0.7 x 7)(l) [410/√3 X 1.25]x 100

L_1= 76.4mm

Provide l_1= 80mm and l_2 = 30mm

Hence provide ISA 100X100X10mm with welded connection.

C) Design of Inner tension members

Member CK

Force = 15 Kn

Factored = 22.5 KN

a) Since the force is very small, provide min. size of angle ISA 50X50X6mm

Area = 568mm^2

b) Connection

Let us provide M_{16}, P Class , 8.8 HSFG Bolts

UNIT- 4 GANTRY GIRDER

1) Design a simply supported gantry girder to carry an electricity operation, travelling crane with the following data.

Span of crane bridge =25cm(c/c distance to gantry girder)

Column spacing =span of gantry girder=8m

Wheel base=3.5m

Crane capacity=200kn

Weight of crane bridge =150kn

Weight of trolley (crab)=75kn

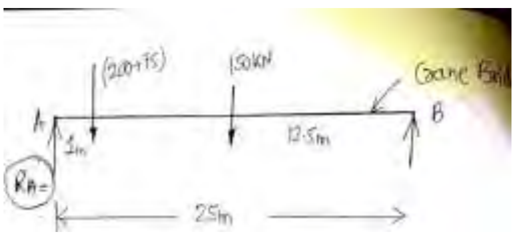
Min. hook distance-1.0m

Weight of rail=0.30kn/m

Height of rail=105mm

SOLUTION:

a) Load calculation: maximum reaction in the crane bridge occurs, when the trolley along with hook if it is towards left or right with a minimum hook distance 1m.



Σ MA=0, RA*25-150*12.5-275*24=0

RA=339kn

There are two wheels at each end of crane bridge .

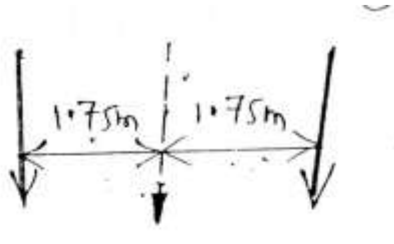
Therefore load on each wheel = 12.5*169.5=211.87kn

Take 212kn

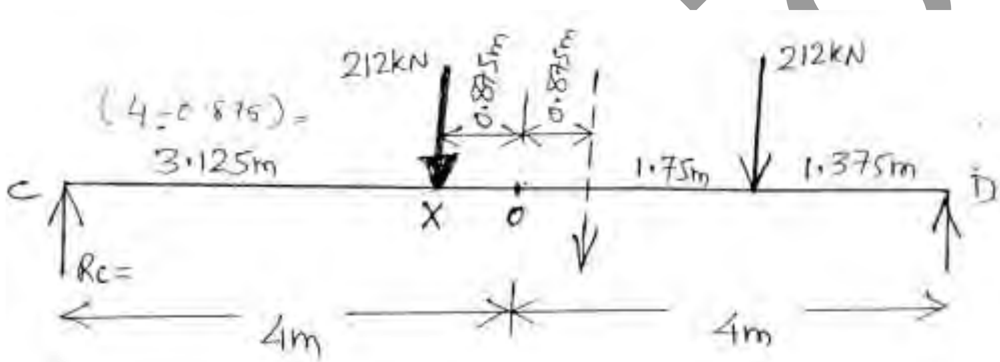
b) Now consider gantry girder:

"The arrangement of wheel load for maximum BM is, the mid span is equi-distance from resultant of two wheel loads and any one load".

“ROLLING LOAD METHOD”



“later take moment under the wheel load 4 which is very close to mid span”.



Σmd=0, rc*8-212*1.375-212(1.375+1.75+1.75)=0

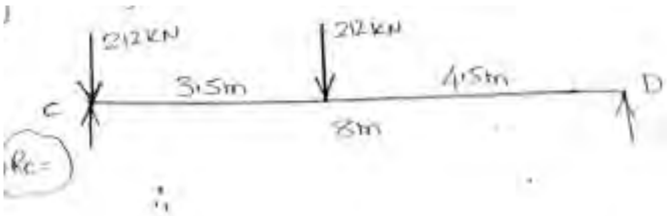
RC=165.63kn

Therefore Mmax=RC*3.125=165.63*3.125=517.6kn

Therefore factored moment M=776.4kn-m

MAXIMUM SHEAR FORCE:

“The arrangement of wheel load for maximum SF is two wheel loads is placed either complete left or right side of the span”.



$\Sigma MD=0$

(RC*8)-(212*4.5)-(212*8)=0

Therefore Vmax=RC=331.25KN

c) "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 10% of lifted weight and trolley weight.

Therefore Horizontal force = ((10/100)*(200+75))/4 wheels

=6.875KN

Therefore factored force=10.5KN

Therefore moment due to horizontal load = (776.4/212*1.5)*10.5

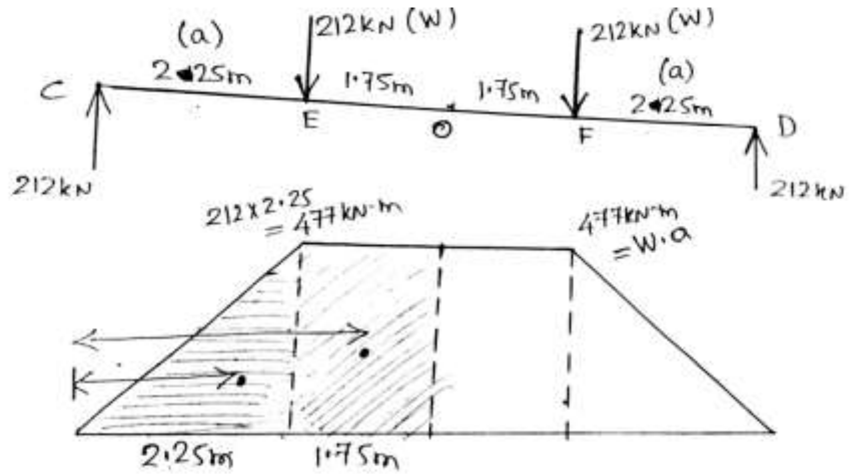
=25.64KN-M

d) <u>Trial section:</u>

The trial section is selected based on deflection condition .

The permissible deflection for electricity operated crane (upto 50t=500KN)

But actual deflection



$d)_{load}+d)_{self\ wt}$=10.67mm , E=2*10^5 N/mm*mm

using moment area method $d)_{load}$= (Area)x/EI_2

= 10.67mm

$I_Z = 1.765X10^9\ mm^4$

Increase the above value by 30% to 50% approximately

$= 1.30\ X\ 1.765\ X\ 10^9 = 2.295\ X\ 10^9\ mm^4$

$I_Z = I_X = 229500cm^4$

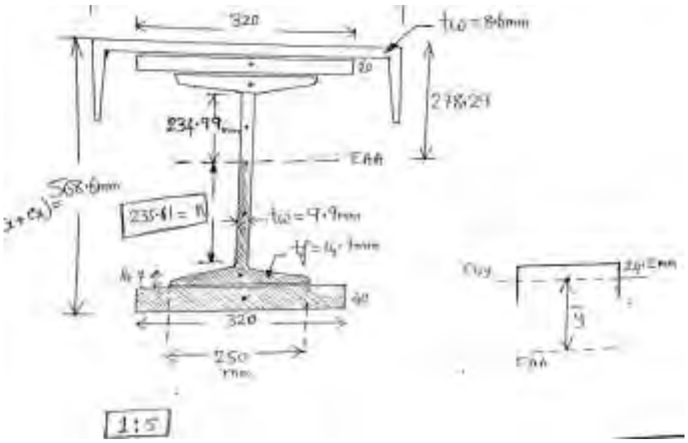
From steel table try suitable section

Try ISMB- 500 @ 95.2kg/m

ISMB- 400 @ 49.4kg/m

Top cover plate 320X20mm

Bottom cover plate 320X40mm



Overall properties (P-36 steel table)

$I_{XX}=I_{ZZ}= 230194.5X10^4mm^4$

Area = $376.15X100mm^2$

C_{xx}= 283.7mm (from top) and e_{xx}= 284.9mm(bottom)

r_{yy}= 9.57cm= 95.7mm

(Page 6) ISMC-400 : Area= $6293mm^2$, C_{yy} = 24.2mm,t_w=8.6mm

(Page 4)ISWB-500 : Area = $121.22X100mm^2$,

B= 250mm,t_f=14.7mm

<u>Location of equal area axis</u>

Area of shaded portion = ½(Total area)

(9.9Xn)+(250X14.7)+(320X40)=1/2 [376.15X100]

n=235.61mm

Plastic modulus Z_P=ay

= $9.05X10^6mm^3$

e) <u>Check for moment of resistance:</u>

For "laterally unsupported"Beam

Design Bending strength= $M_d=B_bZ_pf_{bd}$ Page 54

$E=2X10^5$ N/mm^2

L_{LT}= 8m=8000mm (gantry girder span)

Υ_y=95.7mm for whole section.

Average or Mean Thickness of flange (P-63 steel table)

Top flange = 33.8mm = t_f

Bottom flange= 51.5mm

H_f=c/c distance between flanges

H_f = overall depth-1/2 (Top and bottom mean thickness)

= 568.6-1/2(33.8+51.5)= 525.95mm

$$F_{crb}= 1.1\, f_{cr.b} = 1.1\pi^2 E/(L_{LT}/\ y)\ [1+1/20(\frac{L_{LT}/\ y}{h_f/t_f})]^{0.5}$$

= 485.65 N/mm^2

From table 13a, P-55(IS-800)

Design bending compression stress=f_{bd}=187.96N/mm^2

$M_d=B_dZ_pf_{cbd}=1X9.05X10^6X187.96$

M_d=1701.3X10^6N-mm > (776.4 Kn-m moment + 25.64Kn-m)

Safe

f) Check for shear resistance

Design shear strength $V_d=V_n/\Upsilon_{mo}$

A_V=Shear area = hXt_w – for hot rolled

h=overall depth of the section

A_v=568.6X9.9=5629.14

$V_d=(5629.14)X250/(3)^{1/2}X1.10$

=738.63X10^3 N

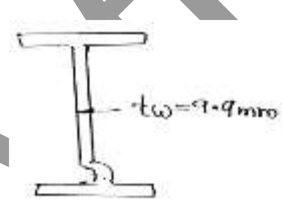
>496.88 KN Safe

g) check for web crippling

Local capacity of the web = $F_w(b_1+n_2)t_wf_yw/\Upsilon_{mo}$

B_1=100mm(assume)

H_2=2.5[40+14.7] = 136.75mm



F_w=(100+136.75)x9.9x250/1.10

F_w=532.38KN > 496.88 KN(safe)

h) Check for buckling of web

Buckling strength = $(b_1+n_1)t_w X f_{cd}$

B_1=100mm

H_1=284.9mm=e_{xx}

T_w=9.9mm

$\Lambda = kL/\Upsilon = l_e/\Upsilon_{min} = 0.7d/\Upsilon_{yy}$

Λ= 3.44

From table 9(c) page – 42

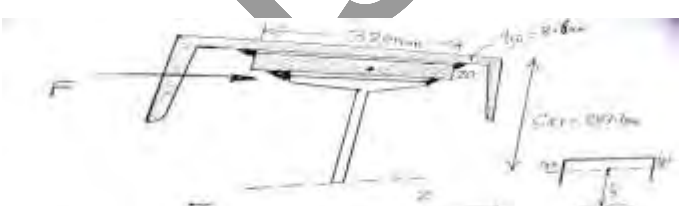
F_{cd}=227 N/mm^2

Buckling strength = (100+284.9)X9.9X227

= $865X10^3$N > 496.88KN (Safe)

Hence the above section can be used as a gantry girder.

Connection



The force at the junction = $F=Vay/I_z$ N/mm^2

F= 718.71N/mm

Equating the above force with strength of weld

718.71N/mm = 2[0.7XsX1X410/3X1.25]

S= 2.71mm

Provide min. 5mm weld , at top and bottom flange.

Design of bracket connection

a) No. of bolts = n = $\sqrt{\frac{6M}{l.p.R}}$

l= No. of lines of bolts = 4

p= pitch= 2.5X20=50mm

R=bolt value = 60.38 KN

Moment M=PXe

P= Max. SF in GG= 496.88 Kn

Assume , e= 200mm

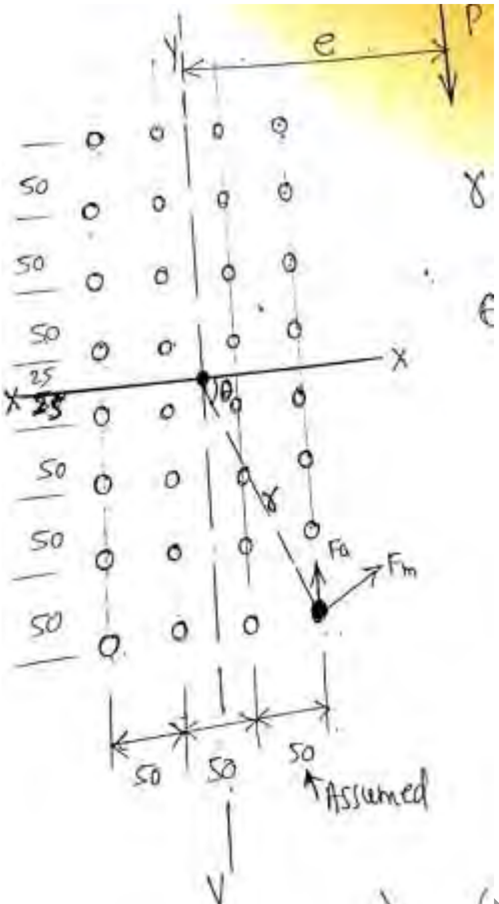
M= $(496.88X10^3)(200)=99376X10^3$ N-mm

n= $\sqrt{\frac{6*99376*1000}{4*50*60.38*1000}}$= 8 per line

b) Check

i) Fa = P/N = $496.88X10^3/8X4$

= 15.52KN



$\Upsilon = \sqrt{75^2 + 175^2} = 190.39$

$\Sigma\Upsilon^2 = 520X10^3\ mm^2$

$F_m = M\Upsilon/\Sigma\Upsilon^2 = 36.38KN$

Resultant = $R = \sqrt{f_a{}^2 + f_m{}^2 + 2f_af_m\cos\Theta}$

R= 44.82KN < Bolt value 60.38KN (Safe)