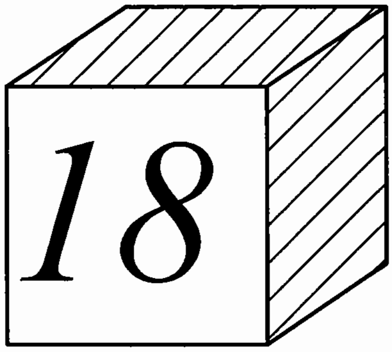
مراجعه STeel
م/رمزى شالله
(2)



FINAL REVISION

(2)

Exam 2010
Exam 2011

AIN SHAMS UNIVERSITY — FACULTY OF ENGINEERING
STRUCTURAL ENGINEERING DEPARTEMENT
THIRD YEAR CIVIL — STRUCTURAL DIVISION

MAY 2011 — STEEL STRUCTURES — TIME:3.00 Hrs

Question (1) (63 marks)

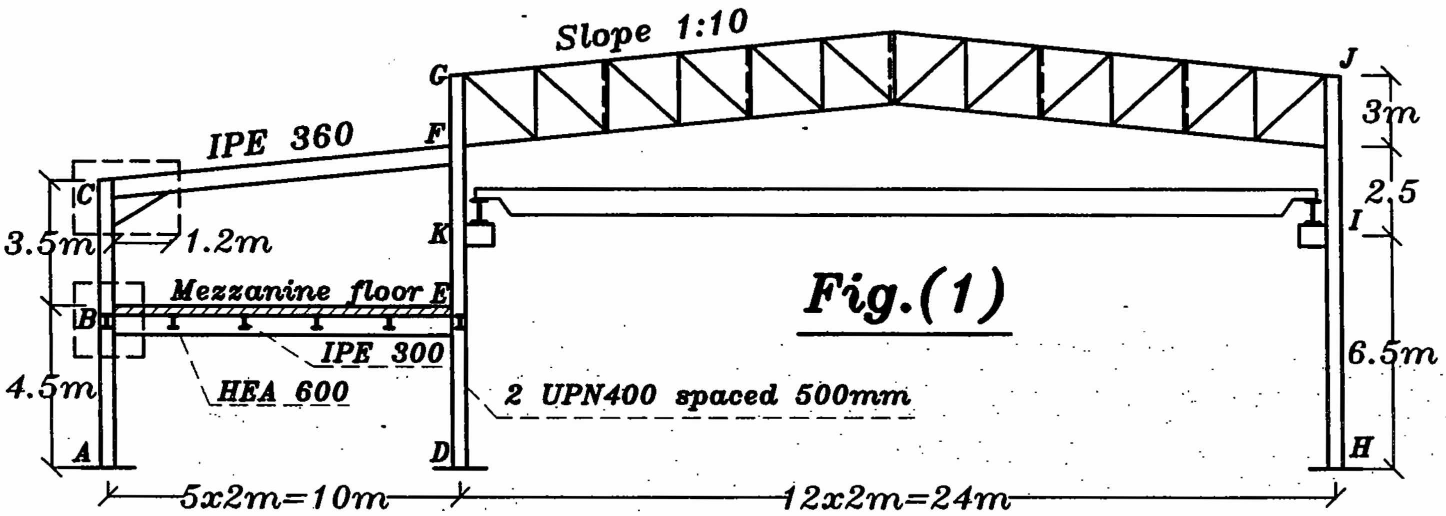


Fig.(1)

Figure 1 shows the main supporting system for an industrial building with a total area of $34x54m^2$. The spacing between main systems is 6.0 m. The main bay of the building has a crane bridge as shown in figure The side bay contains a two story administration building.

1) Draw to scale 1:200 the vertical bracing of column ABC and DEKFG. (5Marks) .

2) Design a suitable rolled section for the hinged column ABC.
Girders BE and CE are rigidly connected to the column at B and C, respectively. The straining actions affecting column ABC are shown in figure (2) and given below. (15Marks).
For part AB:
For part BC:

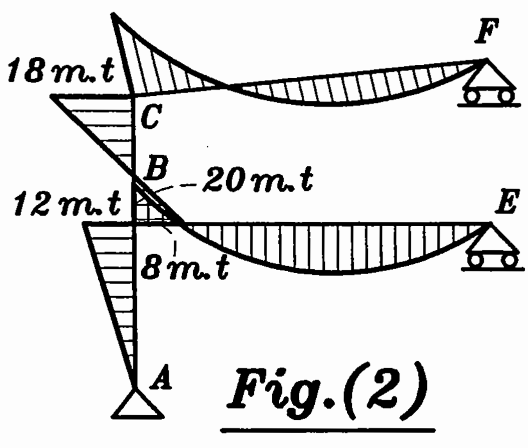


Fig.(2)

3) *Based on the crane girders reactions shown in figure (3), determine the required diameter of the bolts connecting the two crane brackets to the column at K. Each bracket is attached to the column using 6 bolts. The bolts are friction type (Pre–tensioned) bolts grade 10.9 (10Marks)*

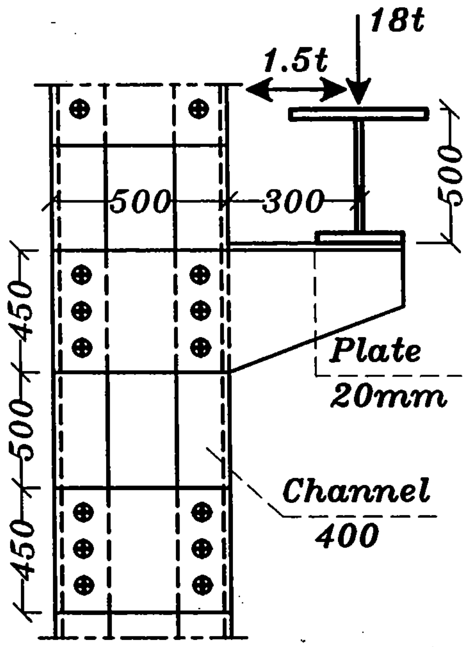


Fig.(3)

4) *Determine the required thickness for an intermediate bolted batten plates if the max. straining actions affecting the combined column are $N_D = 60t$ $Q_D = 7t$ (6Marks)*

5) *Design the column fixed base at D. A sectional plan of the base is shown in figure (4). Design should include the following items. (15Marks)*

1. *Vertical weld*
2. *Horizontal weld*
3. *Anchor bolts diameter*
4. *Base plate thickness.*

$N_D = 60t$ $Q_D = 7t$ $M_D = 25m.t$

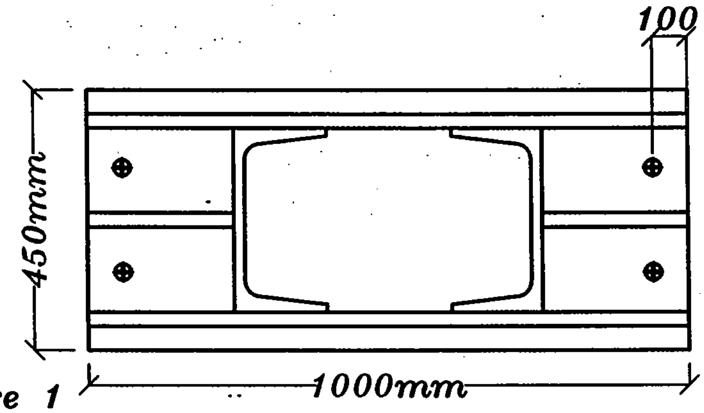
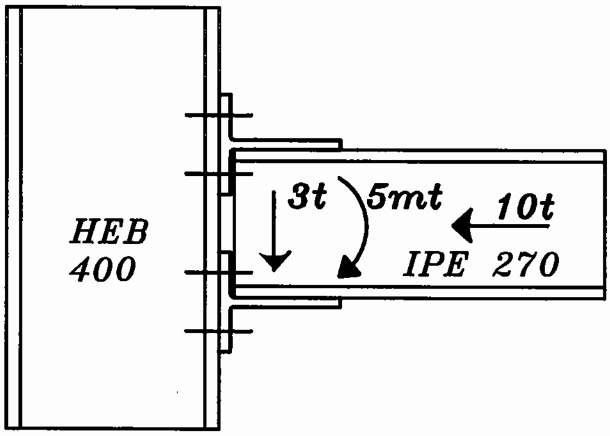


Fig.(4)

6) *Draw the two parts enclosed by dotted rectangles shown in figure 1 to scale 1:10 (minimum two views for each detail). Assume reasonable sections, bolts diameter, number of bolts and or welds sizes for any undesigned elements and indicate on your drawings. (12Marks)*

Question (2) (14 marks)



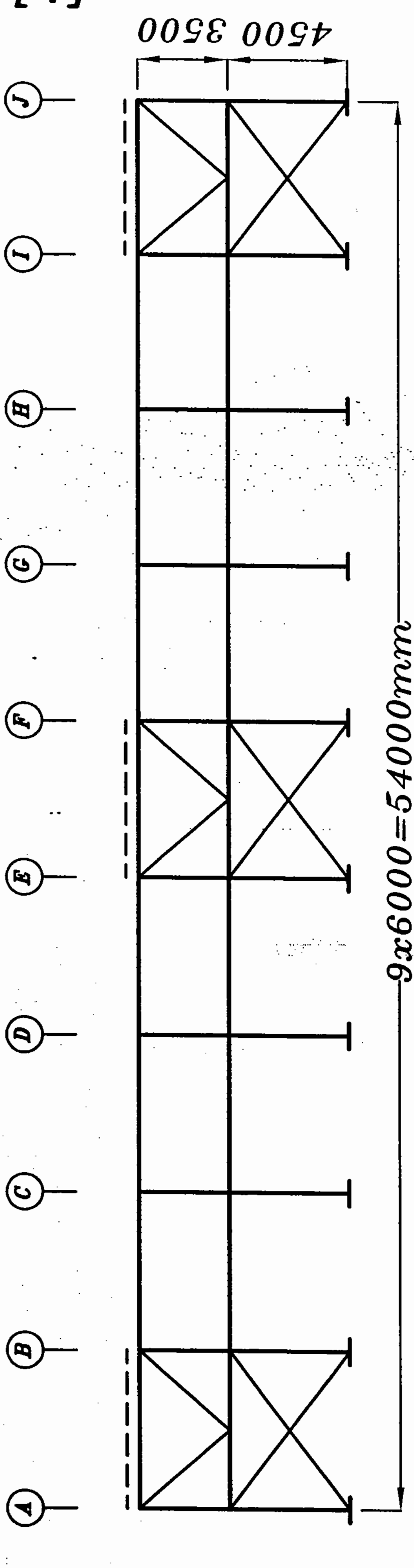
Using Tee stub moment Connection, design the shown bolted and welded connection.

1–Use M16 grade 10.9 pretensioned bearing type bolts (Threads are excluded from plane of shear).
Also calculate the Tee stub thikness

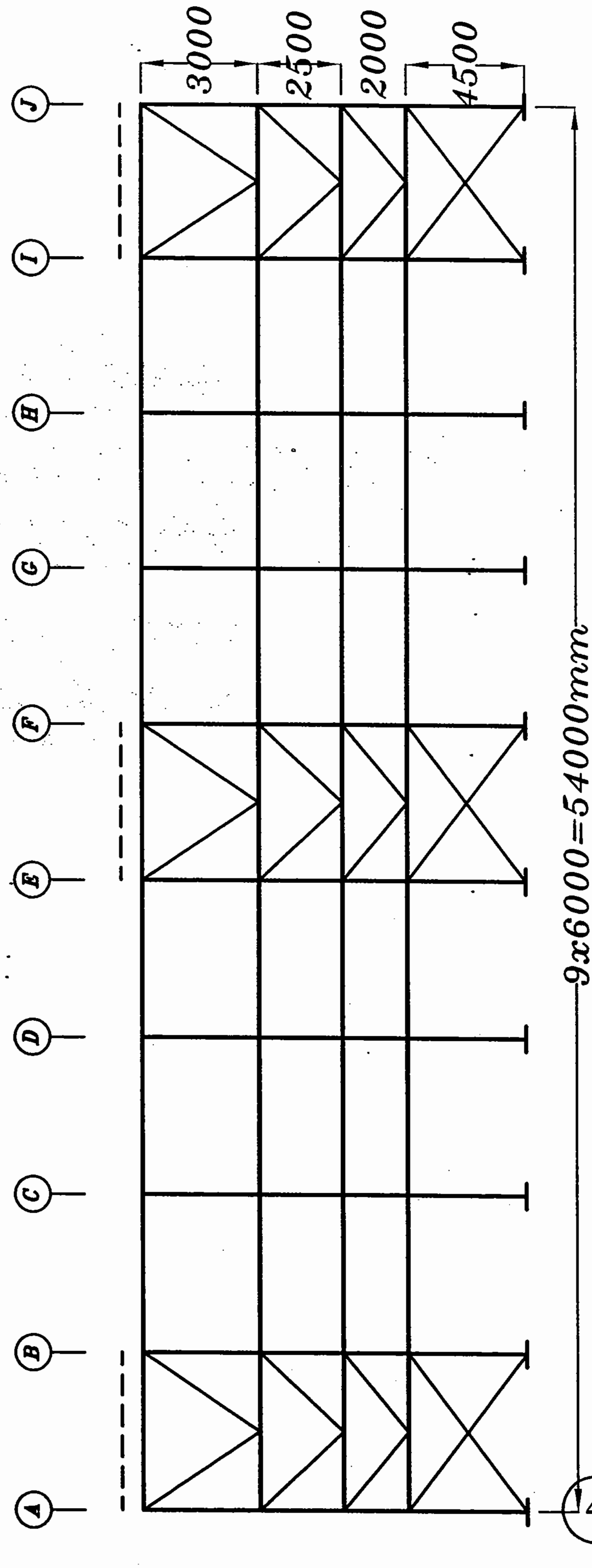
2–check the safety of the column against local stresses.

Question (1)

[1]

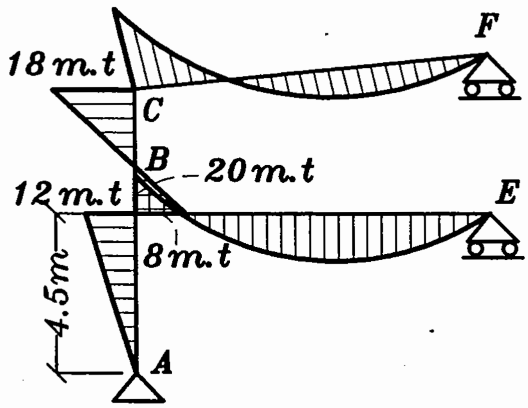


Vertical bracing for Column ABC



Vertical bracing for Column DEKFG

Question (1)



[2] $M_B = 12\ m.t$ $N_B = 40\ t$

$M_C = 18\ m.t$ $N_C = 6\ t$

نبدأ بتصميم Section (C) حيث أنه هو اللى عليه

ال moment الاكبر .

1) Suggest suitable bracing system

فى السؤال السابق .

2) Calculate the straining actions

$N = 6\ t$ $M_x = 18\ m.t$

3) Choice of section

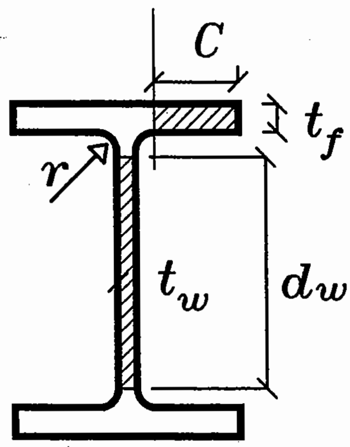
* Assume (allowable stress) $f = 1.00\ t\backslash cm^2$

$$* \ S_x = \frac{M_x}{F} = \frac{18*100}{1.0} = 1800\ cm^3$$

⟹ Choose **I.P.E 500**

4) Check Compactness

For flange Subjected to compression



$d_w = 42.6\ cm$ جداول

$t_w = 1.02\ cm$

$b_f = 20.0\ cm$

$t_f = 1.60\ cm$

$r = 2.1\ cm$

$$\frac{C}{t_f} = \frac{\frac{1}{2}(b_f - t_w - 2r)}{t_f} = \frac{\frac{1}{2}(20.0 - 1.02 - 2*2.1)}{1.60} = 9.24$$

$$\therefore \frac{C}{t_f} = 9.24 < \frac{16.9}{\sqrt{f_y}} = 10.9 \Longrightarrow \text{Compact Flange}$$

For Web

* $d_w * t_w * F_y = 42.6 * 1.02 * 2.4 = \mathbf{104.3}\, t > N = \mathbf{6}\, t$

Web ⟹ Subjected to Bending

$$* \alpha = \frac{1}{2}\left[\frac{N}{d_w * t_w * F_y} + 1\right] = \frac{1}{2}\left[\frac{6}{104.3} + 1\right] = 0.528 > 0.5$$

$$\frac{dw}{t_w} = \frac{42.6}{1.02} = 41.7 < \frac{699/\sqrt{f_y}}{13\alpha - 1} = 76.9 \Longrightarrow \text{Compact Web}$$

∴ The section is compact

5) Check Compression

$l_{b\,in}$ ⟹ Frame ⟹ G_A & G_B

$$G_B = \frac{I_c/l_c}{I_g/l_g} = \frac{48200/4.5 + 48200/3.5}{141200/10} = 1.73$$

لا نستخدم F لان الكمرة ماسكة فى عمود و رسمة ال B.M.D ليست عبارة عن Statical system و الغرض منها فقط قيم ال moments

$$G_C = \frac{I_c/l_c}{I_g/l_g} = \frac{48200/3.5}{16270/10} = 8.46$$

* From charts permitted to sway

$k = 1.9$ $\quad l_{b\,in} = 1.9 * 3.5 = \mathbf{6.65}\, m$

$l_{b\,out} = 3.5\, m$

$r_x = 20.4\, cm \qquad r_y = 4.31\, cm$

$$* \lambda_{in\,x} = \frac{l_{b\,in}}{r_x} = \frac{665}{20.4} = \mathbf{32.59} < 180$$

$$* \lambda_{out\,y} = \frac{l_{b\,out}}{r_y} = \frac{350}{4.31} = \mathbf{81.21} < 180$$

$$*F_C = 1.4 - 6.5 * 10^{-5} \lambda^2_{max.}$$

$$1.4 - 6.5 * 10^{-5} * 81.21^2 = \boxed{0.97 \ t\backslash cm^2}$$

$$*f_{Ca} = \frac{N}{A} = \frac{6.0}{116} = \boxed{0.05 \ t\backslash cm^2}$$

$$* \frac{f_{Ca}}{F_C} = \frac{0.05}{0.97} = 0.05 < 0.15 \qquad \boxed{A_1 = 1.00}$$

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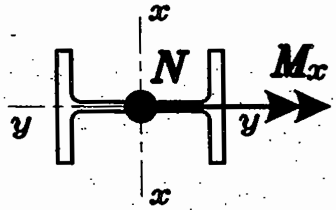
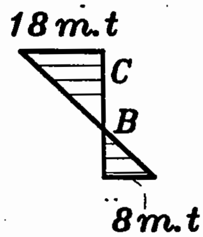
6) Check Bending



$$* f_{b(act.)x} = \frac{M_x}{S_x} = \frac{1800}{1930} = \boxed{0.933 \ t\backslash cm^2}$$

$$* l_{u_{act.}} = 350 \ cm$$

$$* l_{u \ max.} = \begin{cases} \frac{20 \ b_f}{\sqrt{f_y}} = \frac{20 * 20.0}{\sqrt{2.4}} = 258.2 \ cm \\ \frac{1380 A_f}{d * F_y} C_b \end{cases}$$ لا نحتاج الى حسابه



$$l_{u_{act.}} > l_{u_{max.}} \Rightarrow \text{LTB Occurs}$$

$$* C_b = 1.75 + 1.05 \alpha + 0.30 \alpha^2 = 2.27 \qquad \alpha = \frac{8}{18} = 0.44$$

$$* F_{ltb1} = \frac{800 * A_f}{l_u * d} C_b = \frac{800 * (20.0 * 1.60)}{350 * 50} * 2.27 = 3.32 \ t\backslash cm^2$$

$$\leqslant 0.58 \ F_y = 1.4$$

$$F_{bcx} = F_{ltb} = \sqrt{(F_{ltb1})^2 + (F_{ltb2})^2} \leqslant 0.58 \ F_y$$

$$F_{bcx} = \boxed{1.40 \ t\backslash cm^2}$$

7) Check Interaction equation

$$\frac{f_{Ca}}{F_C} + \frac{f_{bx\,(act.)}}{F_{bcx}} * A_1 + \frac{f_{by\,(act.)}}{F_{bcy}} * A_2 < 1.0$$

(the $\frac{f_{by\,(act.)}}{F_{bcy}}$ term is struck through)

$$0.050 + \frac{0.932}{1.40} * 1.0 = 0.72 < 1.0 \Rightarrow \textbf{SAFE}$$

Check on section at point B

$M_B = 12\ m.t \qquad N_B = 40\ t$

5) Check Compression

$$G_B = \frac{I_c/l_c}{I_g/l_g} = \frac{48200/4.5 + 48200/3.5}{141200/10} = 1.73$$

لا نستخدم *F* لان الكمرة ماسكة فى عمود و رسمة ال *B.M.D* ليست عبارة عن *Statical system* و الغرض منها فقط قيم ال *moments*

$G_A = 10$ (Hinged base)

* From charts permitted to sway

$\boxed{k = 2.1}$ $\qquad l_{b\,in} = 2.1 * 4.5 = \mathbf{9.45}\ m$

$l_{b\,out} = 4.5\ m$

$r_x = 20.4\ cm \qquad\qquad r_y = 4.31\ cm$

$$* \lambda_{in\,x} = \frac{l_{b\,in}}{r_x} = \frac{945}{20.4} = \mathbf{46.32} < 180$$

$$* \lambda_{out\,y} = \frac{l_{b\,out}}{r_y} = \frac{450}{4.31} = \mathbf{104.4} < 180$$

$$* F_C = 1.4 - 6.5 * 10^{-5} \lambda^2_{max.}$$

$$1.4 - 6.5 * 10^{-5} * 104.4^2 = \boxed{0.71\ t\backslash cm^2}$$

$$* f_{Ca} = \frac{N}{A} = \frac{40.0}{116} = \boxed{0.34 \quad t\backslash cm^2}$$

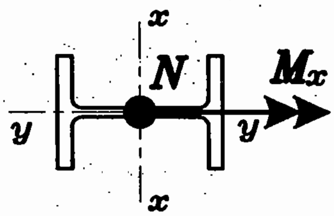
$$* \frac{f_{Ca}}{F_C} = \frac{0.34}{0.71} = 0.48 < 0.15$$

$$* F_{Ex} = \frac{7500}{\lambda_X^2} = \frac{7500}{46.3^2} = 3.49$$

Permitted to sway

$$A_1 = \frac{C_{mx}}{[1 - \frac{f_{ca}}{F_{Ex}}]} = \frac{0.85}{[1 - \frac{0.480}{3.49}]} = 0.99 \nless 1.0 \qquad \boxed{A_1 = 1}$$

6) Check Bending

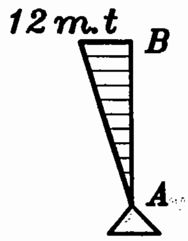


$$* f_{b(act.)x} = \frac{M_x}{S_x} = \frac{1200}{1930} = \boxed{0.620\, t\backslash cm^2}$$

$$* l_{u\,act.} = 450 \text{ cm}$$

$$* l_{u\,max.} = \begin{cases} \frac{20\, b_f}{\sqrt{f_y}} = \frac{20 * 20.0}{\sqrt{2.4}} = 258.2 \text{ cm} \\ \frac{1380\, A_f}{d * F_y} C_b \end{cases}$$

لا نحتاج الى حسابه



$$l_{u\,act.} > l_{u\,max.} \Rightarrow \textbf{LTB Occurs}$$

$$* C_b = 1.75 + 1.05\,\alpha + 0.30\,\alpha^2 = 1.75 \qquad \alpha = \frac{0}{12} = 0$$

$$* F_{ltb1} = \frac{800 * A_f}{l_u * d} C_b = \frac{800 * (20.0 * 1.60)}{450 * 50} * 1.75 = 2.02 \quad t\backslash cm^2$$

$$\leqslant 0.58\, F_y = 1.4$$

$$F_{bcx} = F_{ltb} = \sqrt{(F_{ltb1})^2 + (F_{ltb2})^2} \leqslant 0.58\, F_y$$

$$F_{bcx} = \boxed{1.40 \quad t\backslash cm^2}$$

7) Check Interaction equation

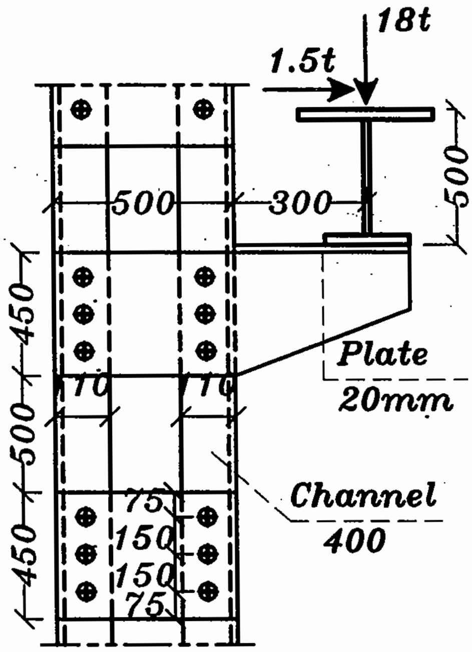
$$\frac{f_{Ca}}{F_C} + \frac{f_{bx\,(act.)}}{F_{bCx}} * A_1 + \frac{f_{by\,(act.)}}{F_{bCy}} * A_2 < 1.0$$

(The $\frac{f_{by\,(act.)}}{F_{bCy}} * A_2$ term is struck through.)

$$0.480 + \frac{0.620}{1.40} * 1.0 = 0.92 < 1.0 \Rightarrow \textbf{SAFE}$$

Question (1)

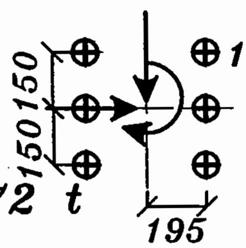
[3]



Straining actions

- $Q_x = \frac{P_x}{2} = \frac{1.5}{2} = 0.75\,t$
- $Q_y = \frac{P_y}{2} = \frac{18}{2} = 9\,t$
- $M_t = 9 * (0.3 + 0.25)$
  $+ 0.75\left(0.50 + 0.02 + \frac{0.45}{2}\right) = 5.51\ m.t.$

Calculate the force on critical Bolt and Check Bolt 1



$$\sum r^2 = \sum (x^2 + y^2) = n x^2 + 4(y_1^2 + y_2^2 + \cdots)$$

$$\sum r^2 = 6 * (19.5)^2 + 4 * (15)^2 = 3182\ cm^2$$

$$R_x = \frac{Q_x}{n} + \frac{M_t}{\sum r^2} * y_1 = \frac{0.75}{6} + \frac{5.51 * 100}{3182} * 15 = 2.72\ t$$

$$R_y = \frac{Q_y}{n} + \frac{M_t}{\sum r^2} * x_1 = \frac{9.0}{6} + \frac{5.51 * 100}{3182} * 19.5 = 4.87\ t$$

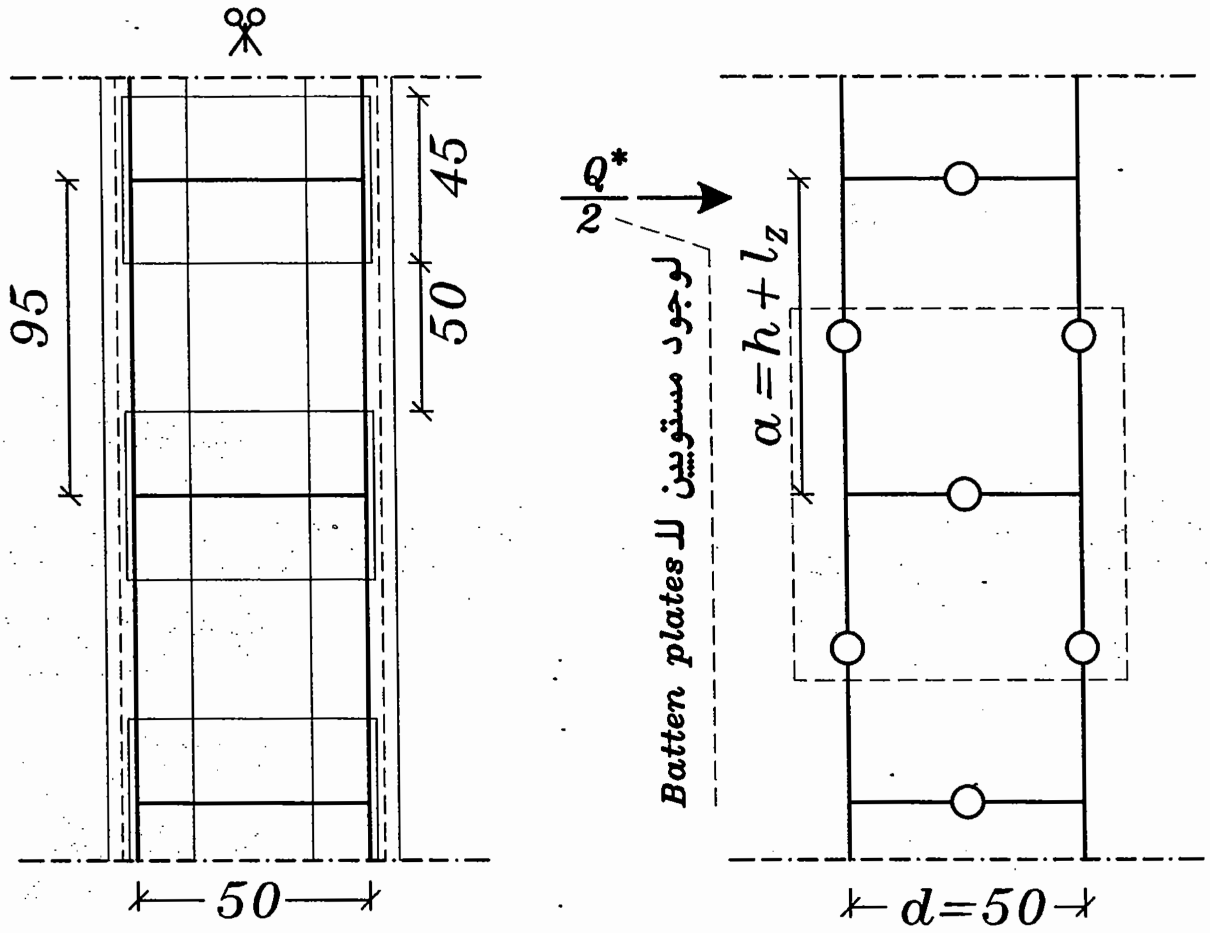
$$R_1 = \sqrt{2.72^2 + 4.87^2} = 5.58\ t$$

$$P_S = R_1 = 5.58\ t$$

Use **M 22** with $P_S = 9.25\,t$

Question (1)

[4] Design of Batten plates



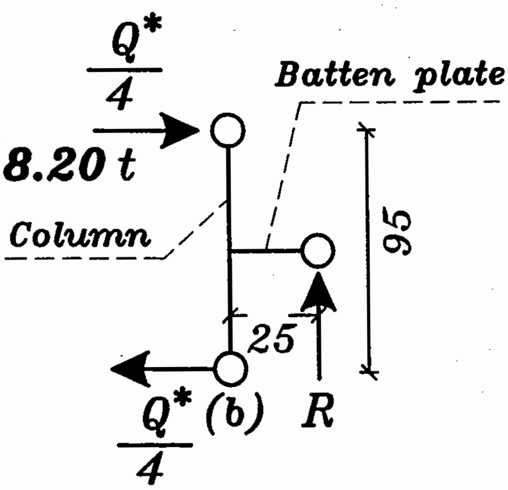
Dimensions of Batten plates

* $h = 50$ cm   Given

* $l_Z = 45$ cm   Given

* $t = \frac{d}{50} = (8 \Rightarrow 14)\ mm \Rightarrow = 1\ cm$

Forces on Batten plates



* $Q^* = Q + \frac{2}{100} N = 7.0 + \frac{2}{100} * 60 = \mathbf{8.20\ t}$

* $R = \frac{Q^* * a}{2 * d} = \frac{8.20 * 95}{2 * 50} = 7.79\ t$

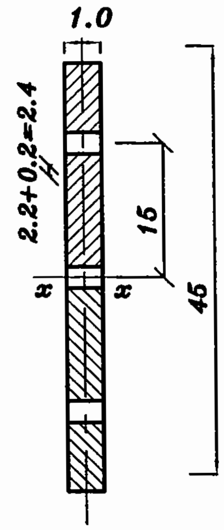
Design of batten plate

1)Straining actions

* $Q = 7.79\ t$ $M = 7.79 * 25 = 195\ cm.t$

2- Properties of area

عدد المسامير معطى فى المسألة و هو٣مسامير فى كل ناحية



$$*A_{net} = 45 * 1.0 - (2.4 * 3) * 1.0 = \mathbf{37.8}\ cm^2$$

$$*I_{X\,gross} = \frac{1.0 * 45^3}{12} - 2\,[\,1.0 * 2.4 * 15^2\,]$$

$$= \mathbf{6514}\ cm^4$$

3-Check Stresses

Normal Stresses

$$f = \frac{M_X}{I_{X_{net}}} Y = \frac{195}{6514} * 22.5 = \boxed{0.67\ t\backslash cm^2} < 1.4\ t\backslash cm^2$$

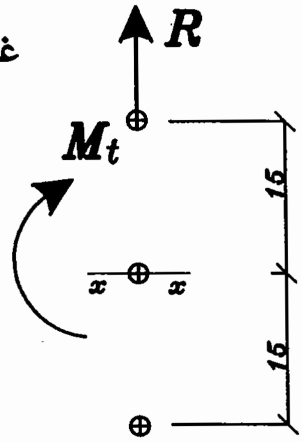
Shear stresses

$$q = \frac{3}{2} * \frac{Q}{A_{gross}} \ngtr 0.35\,F_y$$

$$q = \frac{3}{2} * \frac{7.79}{37.8} = \boxed{0.21\ t\backslash cm^2} < 0.84\ t\backslash cm^2$$

4- Check Bolts

غير مطلوب فى هذه المسألة



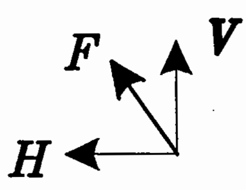
$$* \textbf{Shear} = 7.79\,t \qquad * \textbf{Torsion} = 195\ cm.t$$

$$* \Sigma y^2 = 2 * (15^2) = 450\ cm^2$$

$$* H = \frac{M_t}{\Sigma y^2}(y_1) = \frac{195}{450} * 15 = 6.50\,t$$

$$* V = \frac{R}{n} = \frac{7.79}{3} = 2.60\ t$$

$$F = \sqrt{V^2 + H^2} = 7.0\ t$$



Single shear

$$* R_{S.S} = 0.20 * 10 * \frac{\pi\ 2.2^2}{4} * 1 = 7.60\,t$$

$$* R_b = 0.8 * 3.6 * 1.0 * 2.2 = 7.77\ t$$

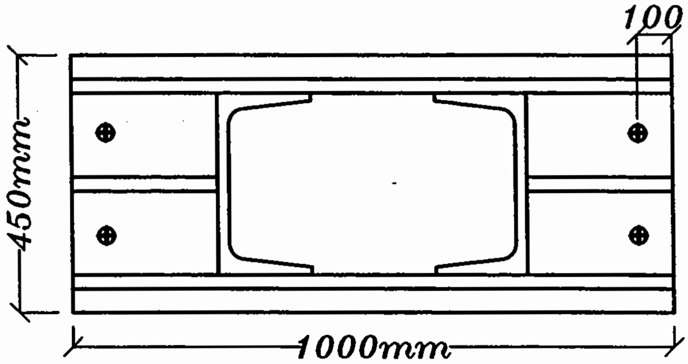
$$* R_{least} = 7.60\,t \qquad F = 7.0\ t < R_{least} = 7.60\,t$$

$\Rightarrow$ **SAFE**

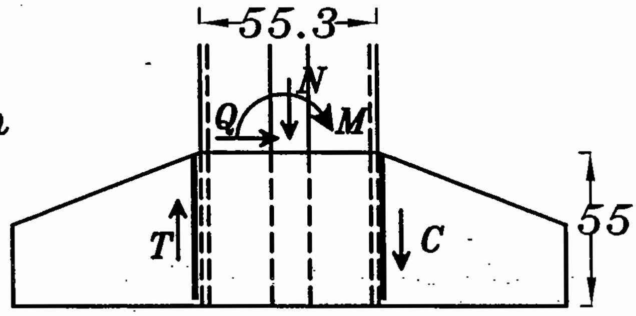
Question (1)

[5]

$M = 25\,m.t \qquad N = 60\,t \qquad Q = 7\,t$



1) Design of vertical weld



* Assume size of weld = 6 mm

  $l = 50\ cm$

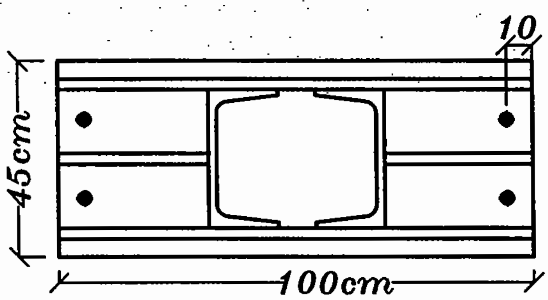
* $C = -\frac{N}{2} - \frac{M}{d}$

  $= -\frac{60}{2} - \frac{25}{0.553} = 75.2\,t$

  $\Rightarrow 0.6\,C = 45.1\,t$

  $T = -\frac{N}{2} + \frac{M}{d}$

  $= -\frac{60}{2} + \frac{25}{0.553} = 15.2\,t$



Plan

* Check stress

$$q = \frac{T \text{ or } 0.6C}{2 * l * s} = \frac{45.1}{2 * 55 * 0.6} = 0.68\ t\backslash cm^2 \not> 0.72\ t\backslash cm^2$$

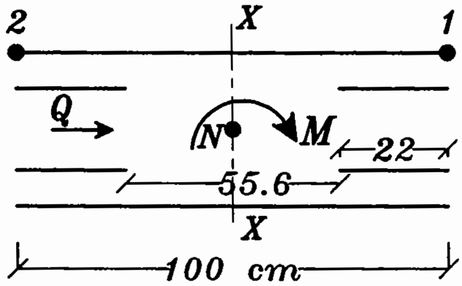
$$f = \frac{Q}{4 * l * s} = \frac{7}{4 * 55 * 0.6} = 0.05\ t\backslash cm^2$$

و دائما تكون قيمة ال Normal stress نتيجة ال **Q** صغيرة لذلك من الممكن اهمالها

و فى حالة عدم اهمالها نعمل ال Check التالى

$$f_R = \sqrt{f^2 + 3\,q^2} \not> 0.2\,F_u * 1.1 = 0.72 * 1.1\,t\backslash cm^2$$

2) Design of horizontal weld



* Assume size of weld = 8 mm

* $A_{w\,Tot.} = 4 * 0.8 * 22 + 2 * 0.8 * 100$

$= 230\ cm^2$

$$I_X = 2 * \frac{0.8 * 100^3}{12} + 4\left[\frac{0.8 * 22^3}{12} + 0.8 * 22 * (39.0)^2\right] = 243251\ cm^4$$

* $f_1 = 0.6\left[\frac{-60}{230} - \frac{2500}{243251} * 50\right] = 0.77\ t\backslash cm^2 \Rightarrow$ Comp.

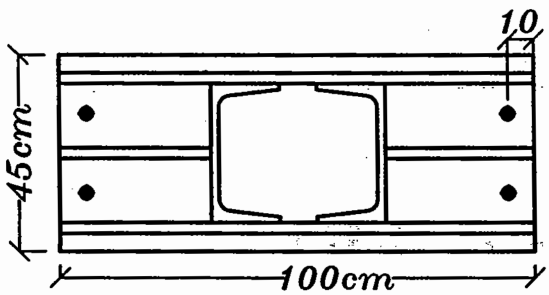
$f_2 = \frac{-60}{230} + \frac{2500}{243251} * 50 = 0.25\ t\backslash cm^2 \Rightarrow$ Tension

* $q = \frac{7}{230} = 0.030\ t\backslash cm^2$

* $f_R = \sqrt{f^2 + 3q^2} = \sqrt{0.77^2 + 3 * 0.030^2} = 0.77\ t\backslash cm^2$

$\ngtr 0.2 F_u * 1.1 = 0.72 * 1.1\ t\backslash cm^2$

3) Check Bearing stresses



Plan

* Assume $f_{cu} = 250\ Kg\backslash cm^2$

* $f_b = \frac{N}{B * L} + \frac{6M}{B * L^2}$

$= \frac{60 * 10^3}{45 * 100} + \frac{6 * 25 * 10^5}{45 * 100^2}$

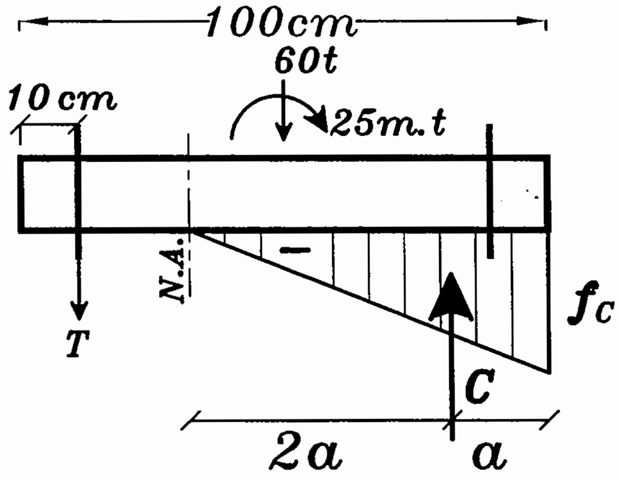
$= 46.66\ kg\backslash cm^2 \ngtr 75\ Kg\backslash cm^2$

4) Design of anchor bolts

$$\frac{M}{N} = \frac{25\ m.t}{60\ t} = 0.42\ m > \frac{L}{6} = \frac{1\ m}{6} = 0.16\ m$$

$\Rightarrow$ Large bending moment $\Rightarrow$ Tension on bolts

* assume $f_c = 0.05 \ t\backslash cm^2$



$$*\ C = \left(\frac{3a}{2} * 0.05\right) * B$$

$$= 3.375 \ a \ \text{ton}$$

$$*\ \Sigma M \ @T = 0$$

$$60 * (50-10) + (25 * 100)$$
$$- 3.375a * (100 - 10 - a) = 0$$

$$3.375a^2 - 303.75\,a + 4900 = 0$$

$B = 45$ cm

Solving the equation :

$$a = 68.94 \text{ cm} \qquad a = 21.06 \text{ cm} \ ✓✓✓$$

$$*\ \Sigma Forces = 0 \Rightarrow \Sigma Y = 0$$

$$N + T = C \Rightarrow 60 + T = 3.375\,a = 32.88 \Rightarrow \boxed{T = 11.08\,t}$$

$$*\ n = \frac{T}{0.85\,R_t} = \frac{11.08}{0.85 * 0.33 * 5.2 * 0.78 * \frac{\pi \phi^2}{4}}$$

assume $n = 2 \Rightarrow \phi = 2.49$ cm $\Rightarrow$ Use $\boxed{2\,\phi\,25}$

$$*\ R_{sh} = q_b * A_S * 1$$

$$= 0.25\,F_{ub} * \frac{\pi \phi^2}{4} * 1 = 0.25 * 5.2 * \frac{\pi * 2.5^2}{4} * 1 = \mathbf{6.38}\,t$$

$$*\ R_t = 0.33\,F_{ub} * \frac{\pi \phi^2}{4} * 0.78$$

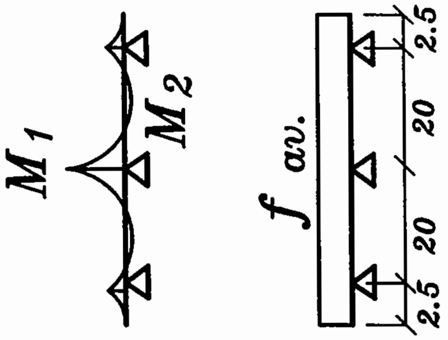
$$= 0.33 * 5.2 * \frac{\pi * 2.5^2}{4} * 0.78 = \mathbf{6.57}\,t$$

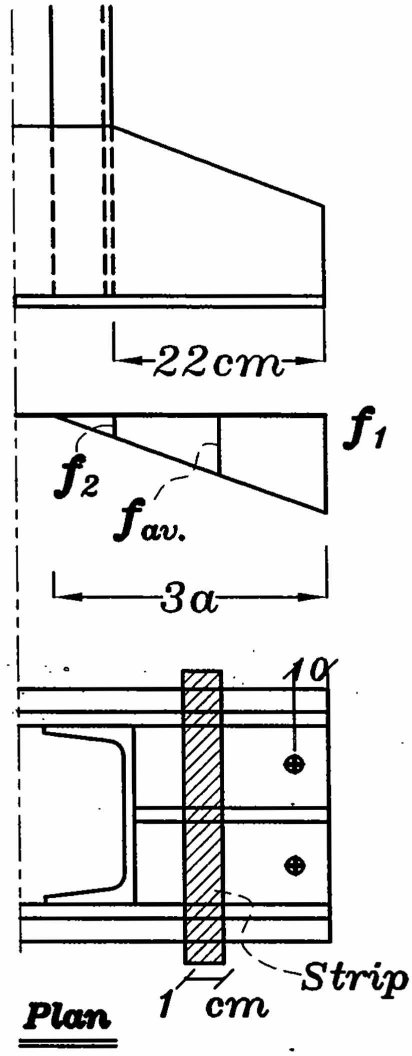
$$\left[\frac{Q\backslash 2n}{R_{sh}}\right]^2 + \left[\frac{T\backslash n}{R_t}\right]^2 < 0.85$$

$$\left[\frac{7\backslash 4}{6.38}\right]^2 + \left[\frac{11.08\backslash 2}{6.57}\right]^2 = 0.79 < 0.85 \Rightarrow Safe$$

5) Get thickness of Base plate

* assume $f_{av.} = 0.025 t\backslash cm^2$





للتسهيل نهمل ال cantilever moment و نحلها على انها
continuous beam بال Empirical values

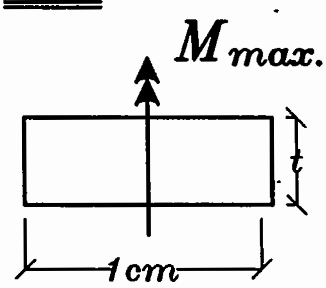
$$* \; M_1 = f_{av.} * \frac{20^2}{8} = 1.25 \; cm.t$$

$$* \; M_2 = f_{av.} * \frac{20^2}{10} = 1.0 \; cm.t$$

$$M_{max.} = 1.25 \; cm.t$$

$$* \; t = \sqrt{\frac{6 M_{max.}}{0.72 \, F_y}} = \sqrt{\frac{6 * 1.25}{0.72 * 2.4}} = 2.08 \; cm$$

($M_{max.}$ in t.cm, F_y in $t\backslash cm^2$)

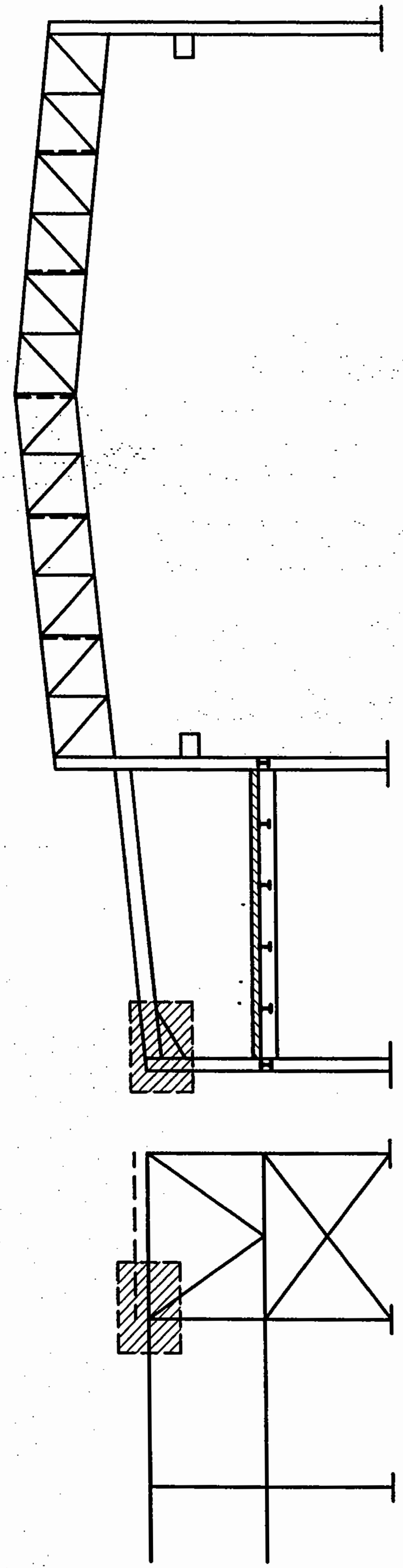


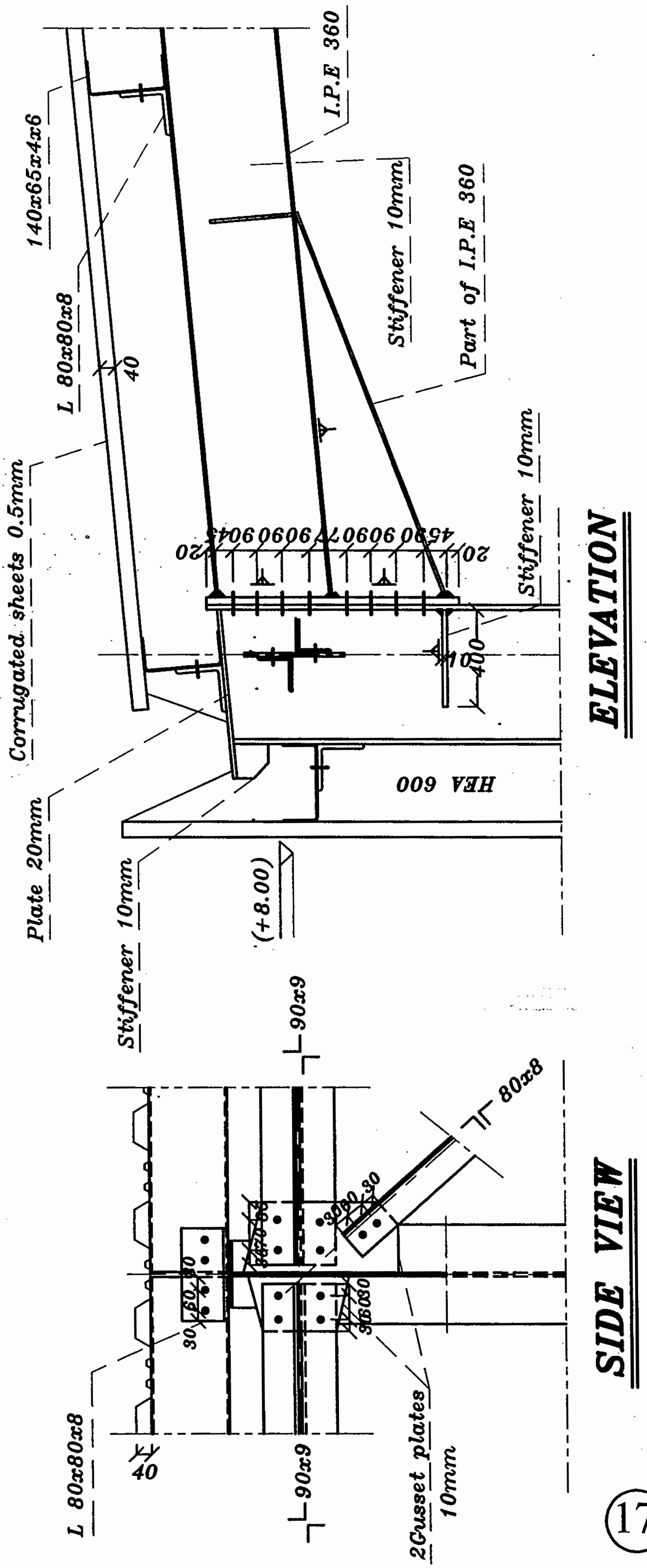
Critical section

⇨ Use $\boxed{t_{min.} = 22 \; mm}$

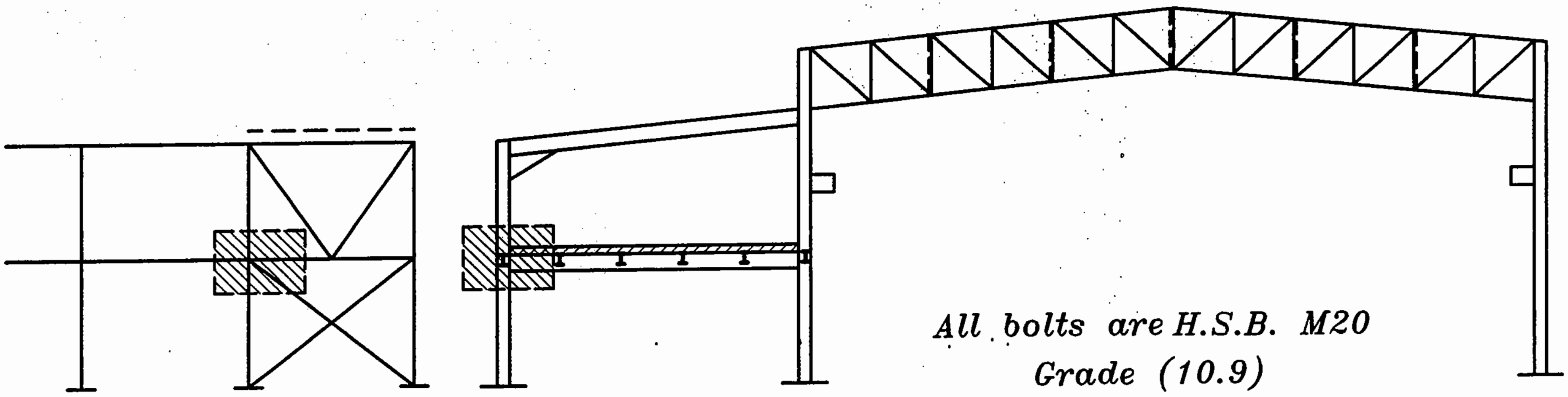
Question (1)

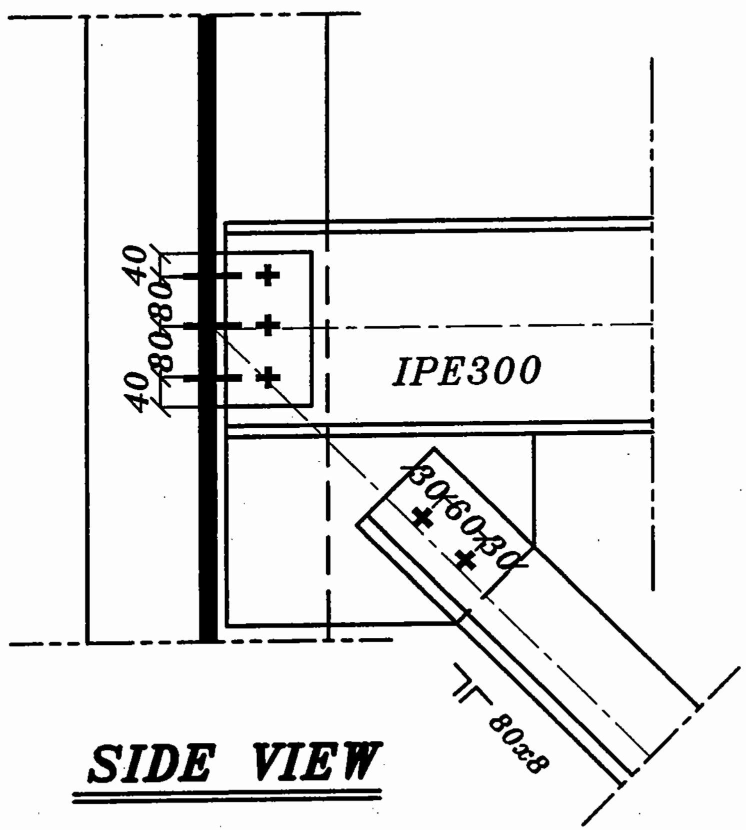
[6]



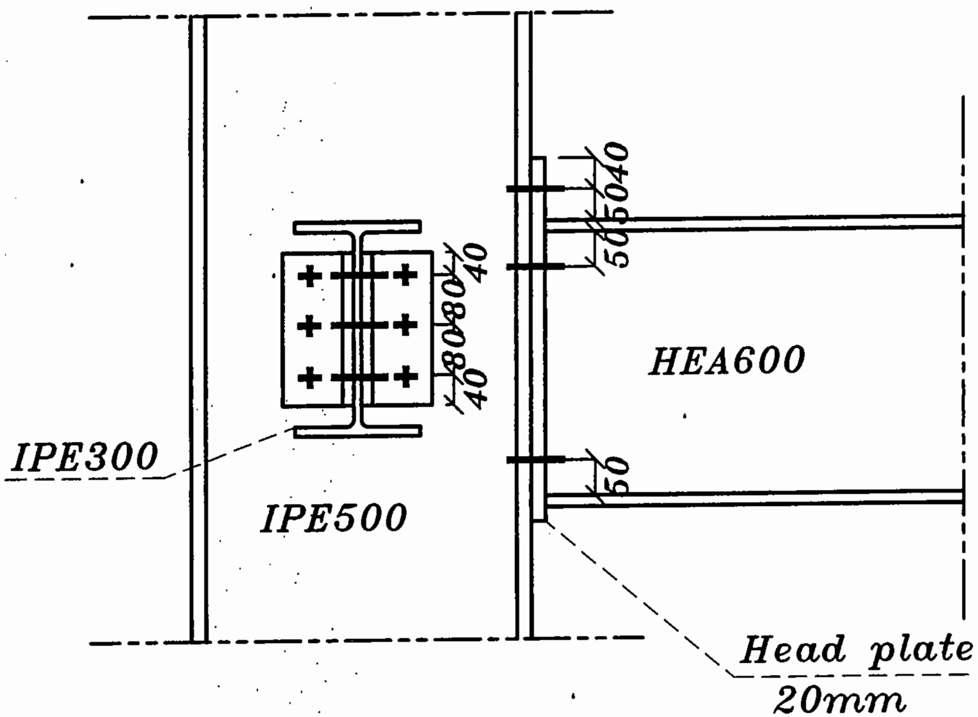


I.P.E 360
140x65x4x6
Stiffener 10mm
Part of I.P.E 360
L 80x80x8
40
Corrugated sheets 0.5mm
Stiffener 10mm
20 45 90 90 90 90 90 90 90 45 20
10
400
HEA 600
Plate 20mm
Stiffener 10mm
(+8.00)
ELEVATION
Stiffener
90x9
80x8
30 30
30 60 30
30
L 80x80x8
40
90x9
2Gusset plates 10mm
SIDE VIEW





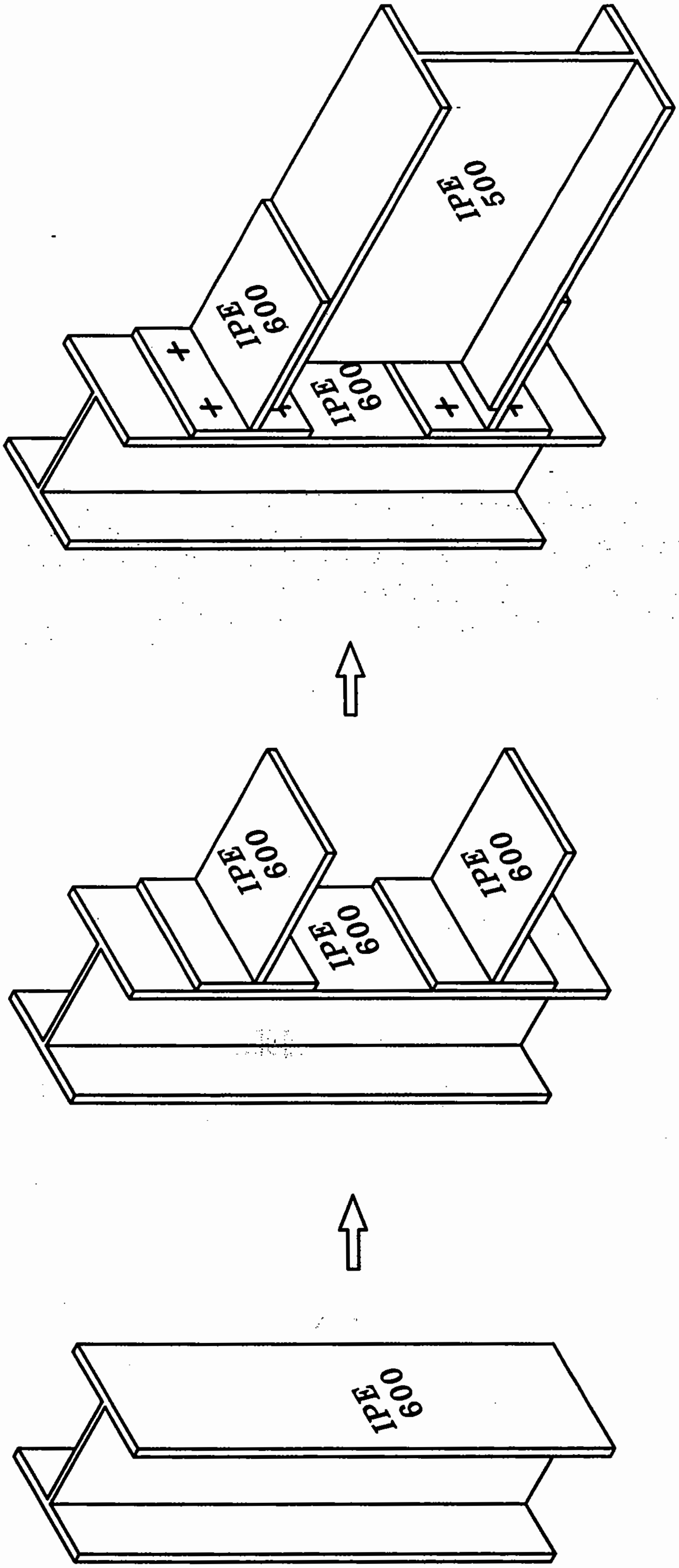
SIDE VIEW



ELEVATION

Question (2)

[1]

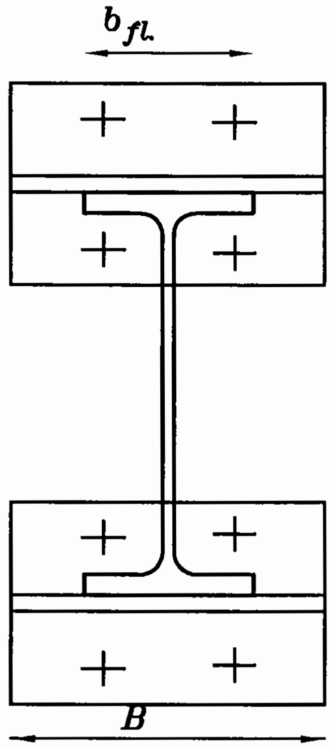


1–Bearing type

1–get straining actions [given]

Shear Force =3t   Normal Force = 10   Bending Moment =5

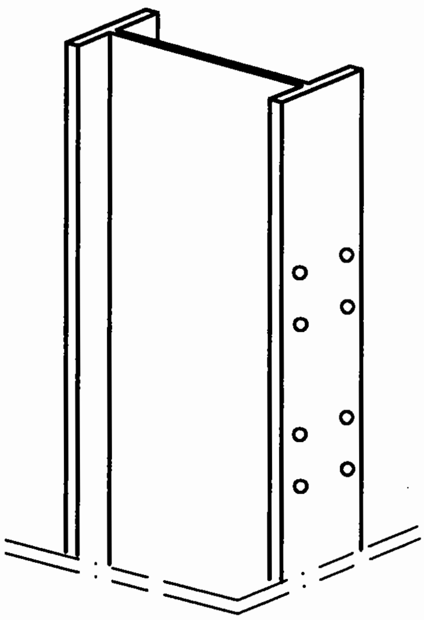
2–assume dimns. of head plate

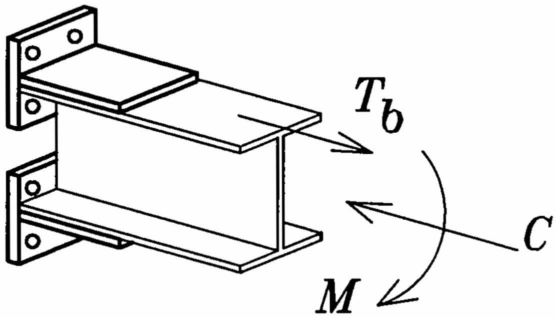


- $B = b_{fl.} + (0.0-4.0)cm$
- $B = 13.5 + 1.5Cm = 15Cm$

3–use 4 bolts only around T.Flange

4–Calculate T_b





$T_{ext,b}$ = Tension force in one bolt

$$= \frac{T_b}{4} \leqslant 0.6T \text{ (pretensioned)}$$

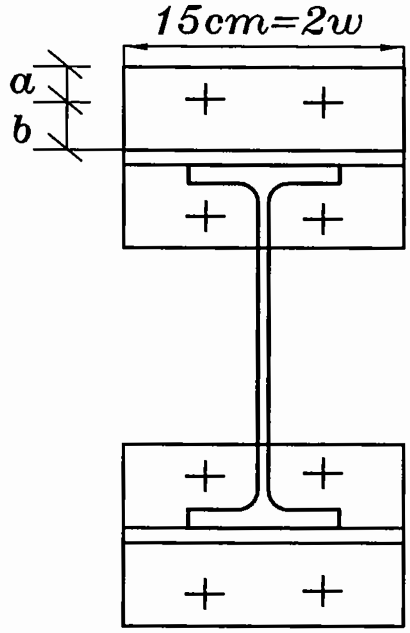
$$\bullet\ T_b = \frac{M}{d_b} - \frac{N}{2} = \frac{5*100}{25.98} - \frac{10}{2} = 14.24 \text{ ton}$$

$$T_{ext,b} = \frac{T_b}{4} = \frac{14.24}{4} = 3.56 \ t < 0.6T = 0.6*9.89 = 5.934 \ t$$

(safe)

5-Calculate the prying force P

- $a = 3.2Cm$ • $b = 3.2Cm$



$$t_p = 0.613\sqrt{\frac{M(2b+2S+t_b)}{d_b * w * F_b}} = \ldots . Cm$$

$$t_p = 0.613\sqrt{\frac{500(2*3.2+2*0+1.02)}{(27-1.02)7.5*0.72*2.4}}$$

$t_p = 2.03Cm$ use $24mm$

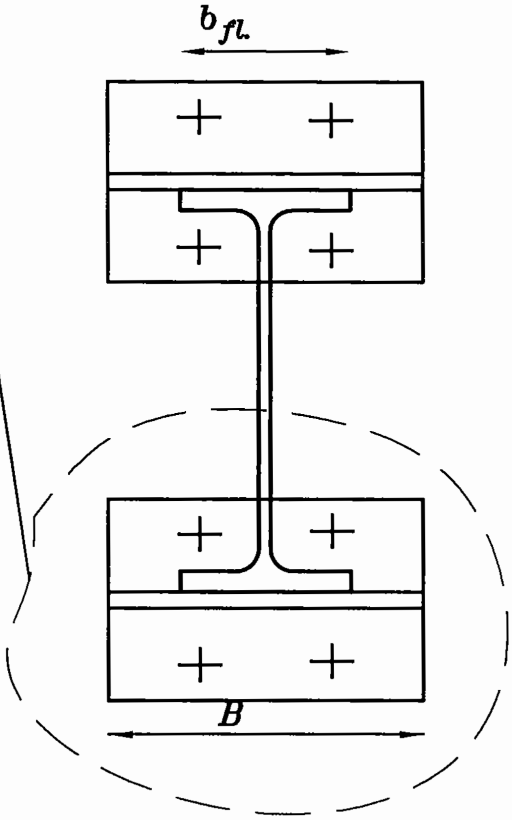
$$P = \frac{\frac{1}{2} - \frac{w\, t_p^4}{30 a b^2 A_s}}{(\frac{3a}{4b})(\frac{a}{4b}+1) + \frac{w\, t_p^4}{30 a b^2 A_s}} (T_{ext,b})$$

$$P = \frac{\frac{1}{2} - \frac{7.5*2.4^4}{30*3.2*3.20^2*1.57}}{(\frac{3*3.2}{4*3.20})(\frac{3.20}{4*3.20}+1) + \frac{7.5*2.4^4}{30*3.2*3.20^2*1.57}} (5.934)$$

$$P = 1.83 \ t$$

$$T_{ext,b} + P = 5.93 + 1.83 = 7.76 < 0.8T = 0.8*9.89 = 7.912 \quad (safe)$$

6-Check of bolt stresses



فى هذه الحالة يجب الملاحظة ان المسامير الموجودة فى ال **Tee Stub** المعرضة الى ضغط هى فقط التى تتحمل قوى القص

$$\frac{Q}{n} = \frac{3}{4} = 0.75t \leq$$

$$R_{sh} = 0.20*10*2.01*1 = 4.02t$$

SAFE

2-Check all the local stresses of the beam column connection

I) Tension zone

$$t_{fc}=2.40cm > 0.4\sqrt{b_b t_b} = 0.4\sqrt{13.5*1.02} = 1.48Cm$$

⇨ **Local bending is prevented**

II) Compression zone

Web local crippling

$$t_{wc}=1.35cm > \frac{b_b t_b}{(t_b+2t_p+5k)} = \frac{13.5*1.02}{(1.02+2*1.8+5*5.1)} = 0.457$$

⇨ **Web local crippling is prevented**

III) Shear zone

$$t_{wc}=1.35cm > \frac{(M \backslash d_b)}{(0.35F_y)*h_c} = \frac{500\backslash(27-1.02)}{(0.35*2.4)*40} = 0.572$$

⇨ **distorsion to the column web is prevented**

AIN SHAMS UNIVERSITY FACULTY OF ENGINEERING
STRUCTURAL ENGINEERING DEPARTEMENT
THIRD YEAR CIVIL STRUCTURAL DIVISION

MAY 2010 STEEL STRUCTURES TIME:3.00 Hrs

Question (1) (42 marks)

Two bays steel trussed frames are adopted as the main structural system for a steel factory as shown in figure 1, The spacing between main frames is 7.0m. The intermediate column (ABC) is subjected to the following straining actions :

Load Case	Column Base A			Column at point B		
	B.M.	N.F.	S.F.	B.M.	N.F.	S.F.
Dead Load	0.0 m.t	−4.0 t	0.0 t	0.0 m.t	−4.0 t	0.0 t
Live Load left bay	0.0 m.t	−4.0 t	+1.5 t	+0.9 m.t	−4.0 t	+1.5 t
Live Load right bay	0.0 m.t	−4.0 t	−1.5 t	−0.9 m.t	−4.0 t	−1.5 t
Wind Load	0.0 m.t	+5.0 t	−0.3 t	−1.8 m.t	+5.0 t	−0.3 t

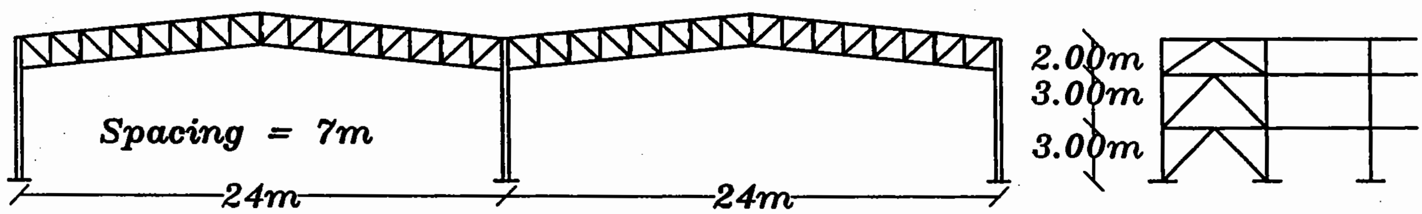
It is required to :

1) Design an intermediate side girt unstiffened cold-formed section if the walls are covered by double layer steel sheets of weight a5 $kg\backslash m^2$ and the spacing between the side girts is 1.75m.

2) Calculate the forces acting on the two marked bracing members and design them. The spacing between the main frame is 7.0m.

3) From the above table, calculate the maximum straining actions (design values) acting on figure (1). Check the safety of the column if it is composed of 4Ls 60x60x6 as shown in figure (1). Design the lacing bars connecting the angles.

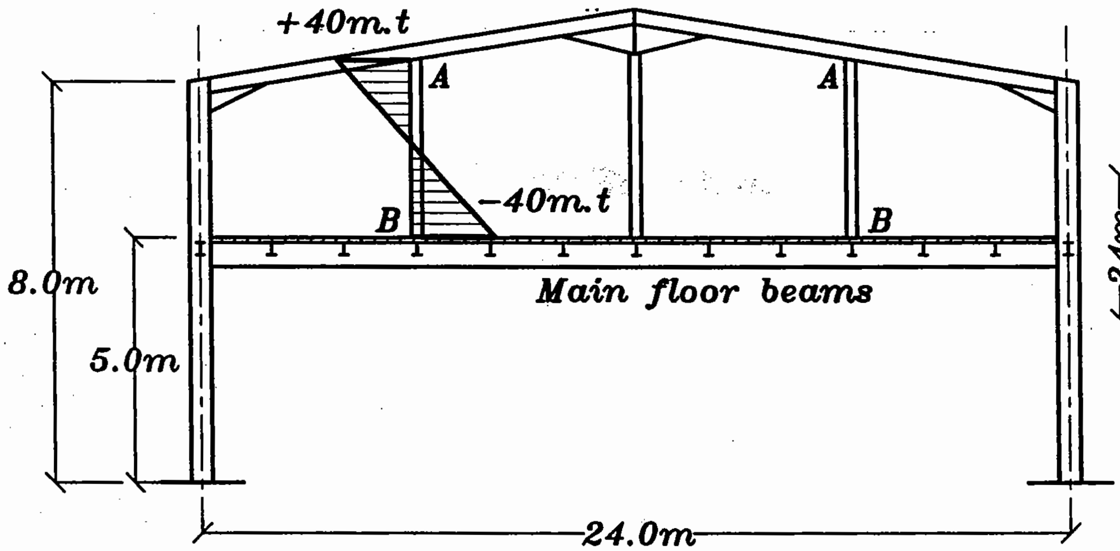
4) Design the column hinged base at point (A).



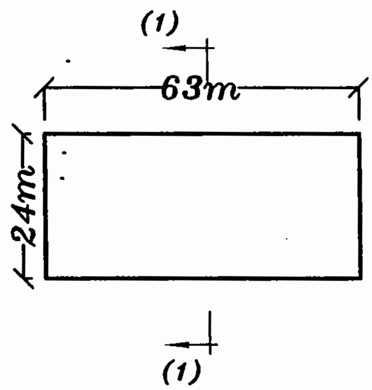
Question (2) (28 marks)

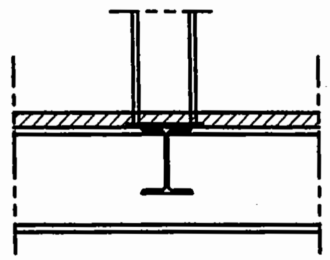
The portal frame shown in figure 2 is used as the main structural system covering a 24x63m steel factory. The frame height is 8.0m. Columns are only allowed along the area perimeter shown as solid lines in figure (2). Intermediate floor at level +5.00m is added for offices of the factory administration. No columns except for outer columns are allowed within production area existing in the ground floors. It is required to:

1) *Draw a plan of the floor at level +5.00 and an elevation for the vertical bracing system of the frame columns. Use a scale of 1:200 for your drawings and suggest suitable sections for all bracing members and the floor beams.*
2) *Design a suitable HEB section for the hanger AB if the design bending moment acting on the hanger at point B is +40m.t and the design tensile force acting on the hanger at the same location is +6.0t.*
3) *Design the welded connection of column AB to head plate at point (B)*
4) *Design a suitable HEB section for the main floor beam based on LRFD Concept if Mf=60m.t and Qf=15.0t.* ملغى هذا العام

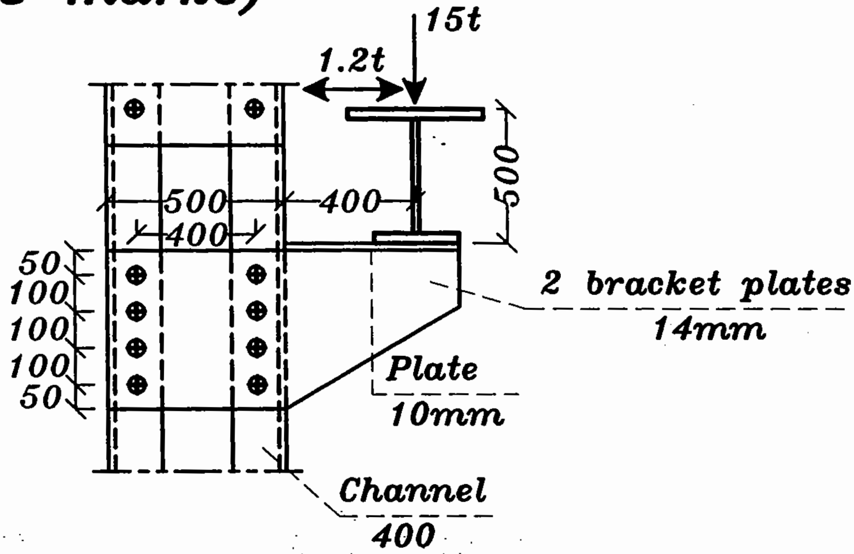


Section (1–1)





Question (3) (8 marks)



1) Check the safety of the group of bolts connecting the two brackets supporting the crane girder to the column as shown in figure 3. The bolts are M16 non-pretensioned bolts grade 8.8.

2) If you are asked to strengthen the connection to be able to support higher capacity for a crane girder, mention two proposals of the strengthening and which proposal is preferable? state why?

Question (1)

[1]

1) Loads and straining actions:

a) Dead load assume

i) Own weight = w_g = 20 kg\m²

ii) weight of the steel sheets = w_c = 15 kg\m²   Given

* $W_{DL} = O.W + W_c * a$

  = 20 + 15 * 1.75 = 46.25 kg\m

b) Live load

Neglect this case

C) Wind load

***wind load = $C_e * k * q$ = ... kg\m²**

C_e = 1.1 (Pressure) C_e = 0.5 (Suction)

K = 1.0 for h ≤ 10m

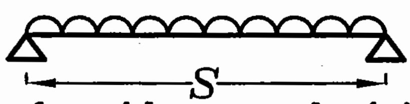
q = 70 kg\m² in cairo

* $W_w = (C * k * q) * a$ = ... kg\m

  = (1.1 * 1 * 70) * 1.75 = 135 kg\m = 0.135 t\m

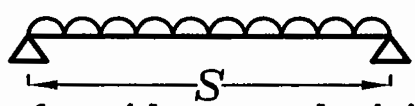
** Starting actions

W_{DL}

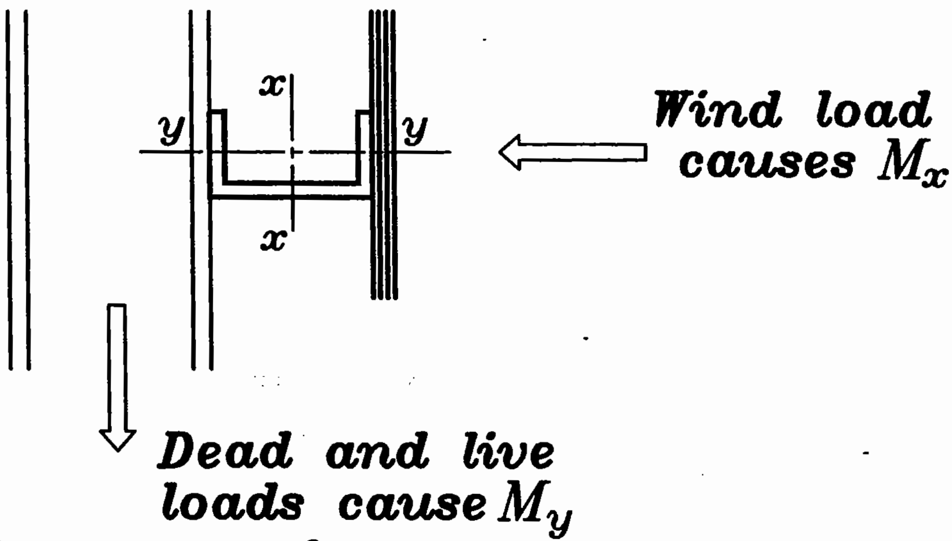


for side or end girt

W_w



for side or end girt



$$*M_x = W_w * \frac{S^2}{8} = 0.135 * \frac{7^2}{8} = 0.827 \ m.t$$

$$*M_y = W_{DL} * \frac{S^2}{8} = 0.046 * \frac{7^2}{8} = 0.28 \ m.t$$

Using two tie rods

$$*M_y = W_{DL} * \frac{(S/3)^2}{8}$$

$$= 0.046 * \frac{2^2}{8} = 0.031 \ m.t$$

$M_x = 0.827 \ m.t$
$M_y = 0.031 \ m.t$

---

2) Choice of section:

For Cold formed channels

$$\frac{M_x}{S_x} + \frac{M_y}{S_y} = F_b$$

$\Longrightarrow$ assume $S_x = 6S_y$

$\Longrightarrow$ channels are non compact or slender so assume

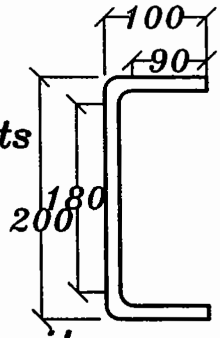
$$F_b = 1.4 \ t\backslash cm^2$$

$$\therefore \frac{82.7 + 6*3.1}{Sx} = 1.4 \Longrightarrow S_x = 72.36 \ cm^3$$

$\Longrightarrow$ Choose C 200x100x4x6

3) Checks:

a) Check code limits for slender sections



$* \; h = H - 2r - 2t = 200 - 2*6 - 2*4 = 180$

$\frac{h}{t} = \frac{180}{4} = 45 < 200 \Longrightarrow$ Web satisfies code limits

$* \; b = B - r - t = 100 - 6 - 4 = 90$

$\frac{b}{t} = \frac{90}{4} = 22.5 < 40 \Longrightarrow$ Flange satisfies code limits

Determine the effective parts of section

For flange

Flange $\Longrightarrow$ Compression Unstiffened $\Longrightarrow \psi = 1$

$* \; K_\sigma = 0.43$

$* \; b = 90$ mm

$$* \; \lambda_P = \frac{b\backslash t}{44}\sqrt{\frac{F_y}{K_\sigma}} = \frac{90\backslash 4}{44}\sqrt{\frac{2.4}{0.43}} = 1.21$$

$$* \; \rho = \frac{(\lambda_P - 0.15 - 0.05\psi^2)}{\lambda_P^2}$$

$$= \frac{(1.21 - 0.15 - 0.05*1)}{1.21^2} = 0.689 < 1$$

$* \; b_e = \rho * b = 0.689 * 90 = 62.09$ mm

For Web

Web $\Longrightarrow$ Bending Stiffened $\Longrightarrow \psi = -1$

$* \; K_\sigma = 23.9$

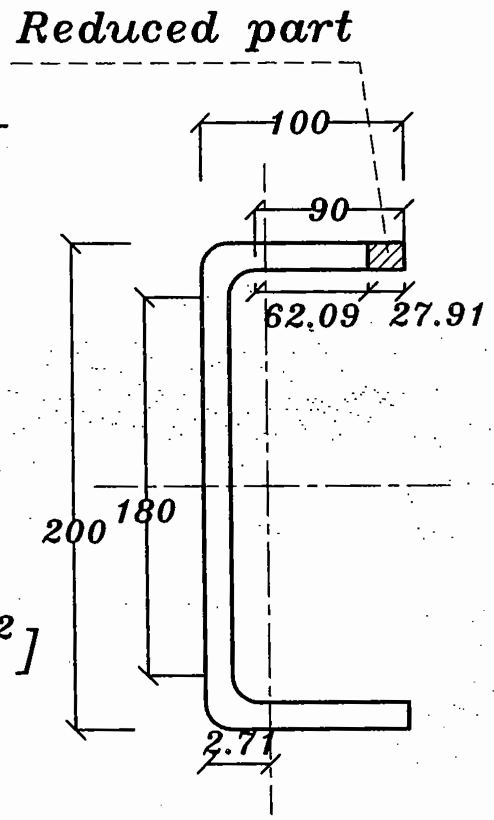
$* \; h = 180$ mm

$$* \; \lambda_P = \frac{b\backslash t}{44}\sqrt{\frac{F_y}{K_\sigma}} = \frac{180\backslash 4}{44}\sqrt{\frac{2.4}{23.9}} = 0.32$$

$$* \ \rho = \frac{(\lambda_P - 0.15 - 0.05\psi^2)}{\lambda_P^2}$$

$$= \frac{(0.32 - 0.15 - 0.05 * -1)}{0.32^2} = 2.15 > 1 \Longrightarrow \text{Fully effective}$$

$$* \ h_e = \rho * h = 1 * 180 = 180 \ mm$$



$$* \ I_{x_{eff}} = I_{x_{table}} - I_{x_{reduced\ part}}$$

$$= 969.7 - 2.791 * 0.4 * (10 - 0.2)^2$$

$$= \mathbf{862.5} \ cm^4$$

$$* \ I_{y_{eff}} = I_{y_{table}} - I_{y_{reduced\ part}}$$

$$= 154.37 - \left[\frac{0.4 * 2.791^3}{12} + 2.791 * 0.4 * \left(10 - 2.71 - \frac{2.791}{2}\right)^2 \right]$$

$$= \mathbf{114.8} \ cm^4$$

b) Check stress

$(l_{u(act.)} = 0 \Longrightarrow$ corrugated sheets)

$F_{bcx} = \boxed{1.40 \ t \backslash cm^2}$ $F_{bcy} = \boxed{1.40 \ t \backslash cm^2}$

$$F_b = \frac{M_x}{I_{x\,new}} * y + \frac{M_y}{I_{y\,new}} * X \ \ngtr \mathbf{1.4} \ t \backslash cm^2$$

$$\frac{82.7}{862.5} * 10 + \frac{3.10}{114.8} * (10 - 2.71) = 1.16 \ t \backslash cm^2$$

$$< \mathbf{1.4} \ t \backslash cm^2 \Longrightarrow \textbf{Safe}$$

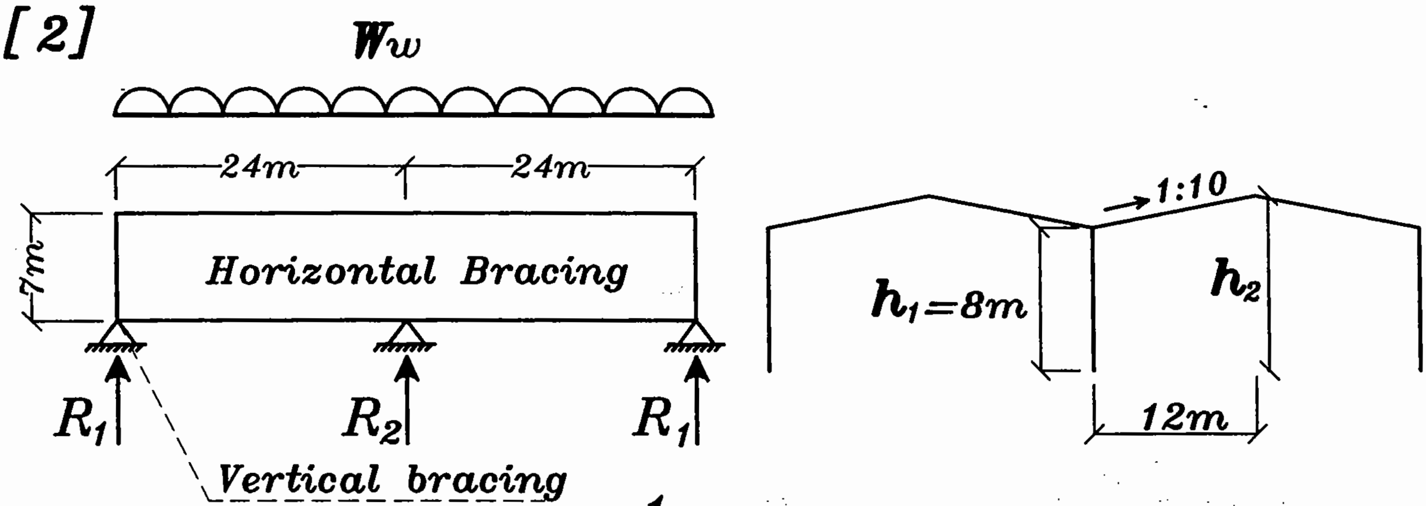
Suction Case

و لن نستخدمها فى الامتحان و لكن نكتب فقط

$C_e = 0.5$ $(l_{u(act.)} = 600 \ Cm)$

Question (1)

[2]



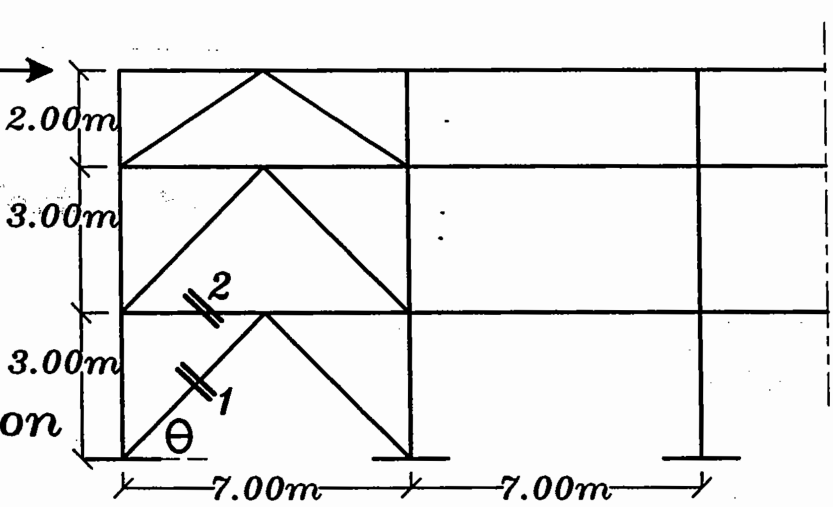
$* h_1 = 8\ m \qquad * h_2 = 8 + \frac{1}{10} * 12 = 9.2\ m$

$* h_{av} = \frac{8+9.2}{2} = 8.6\ m$

$* W_w = (C * k * q) \frac{h_{av.}}{2}$

$= 1.1 * 1 * 70 * \frac{8.6}{2} = 331.1\ kg \backslash m$

$* R_1 = \frac{331.1 * 24}{2} = 3973.2\ kg = 3.973\ ton$



$* \Theta = \tan^{-1} \frac{3}{3.5} = 40.60^\circ$

$* F_1 = \pm \frac{R_1}{2\cos\Theta}$

$= \frac{3.973}{2\cos 40.60} = \pm 2.62\ ton$

* equilibrium of joint

$F_2 = \frac{R_1}{2} = \pm 1.99\ ton$

$* l_{b\,in_1} = 4.61\ m \qquad l_{b\,out_1} = 1.2 * 4.61 = 5.53\ m$

$* l_{b\,in_2} = 3.50\ m \qquad l_{b\,out_2} = 7.0\ m$

Design of member (1) ⅃L

* Choice of sec:

assume $\lambda = 200$

$$200 = \frac{l_{b\,in}}{r_x} = \frac{l_{b\,out}}{0.30\,a} = \frac{461}{0.30\,a} \Longrightarrow a = 7.68 \text{ cm}$$

$$200 = \frac{l_{b\,out}}{r_y} = \frac{l_{b\,out}}{0.45\,a} = \frac{553}{0.45\,a} \Longrightarrow a = 6.14 \text{ cm}$$

$\Longrightarrow a = 6.67$ cm $\Longrightarrow$ use ⅃L 80*80*8

* Design as Compression member:

1)Buckling assume $t_g = 1$ cm

$$r_{x\,⅃L} = r_{x\,L} = 2.42 \text{ cm}$$

$$r_{y\,⅃L} = \sqrt{(r_x)^2 + (e + \frac{t_g}{2})^2} = \sqrt{(2.42)^2 + (2.26 + \frac{1}{2})^2}$$

$$= 3.67 \text{ cm}$$

$$* \lambda_{in} = \frac{l_{b\,in}}{r_x} = \frac{4.61*100}{2.42} = 190.5 < 200$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_y} = \frac{5.53*100}{3.67} = 150.7 < 200$$

2)Stresses $\lambda_{out} > 100$

$$* F_C = \frac{7500}{\lambda^2_{out}} = \frac{7500}{190.5^2} = 0.207$$

$$* F_{act} = \frac{force}{2A_L} = \frac{2.62}{2*12.3} = 0.107 \text{ t}\backslash\text{cm}^2 < F_c \quad (Safe)$$

* Design as Tension member:

3)Deflection

$$* \frac{l}{d} = \frac{l}{a} = \frac{461}{8} = 57.6 \leqslant 60$$

4)Stresses

لا نحتاج الى عمل هذا الـ Check لان الـ *allowable tensile stresses* كبيرة جدا و تساوى $1.4 t\backslash cm^2$ و بالتالى أكيد ستكون *Safe*

Design of member (2) ⅃L

* Choice of sec:

assume $\lambda = 200$

$$200 = \frac{l_{b\,in}}{r_x} = \frac{l_{b\,out}}{0.30\,a} = \frac{350}{0.30\,a} \Rightarrow a = 5.86 \ cm$$

$$200 = \frac{l_{b\,out}}{r_y} = \frac{l_{b\,out}}{0.45\,a} = \frac{700}{0.45\,a} \Rightarrow a = 7.77 \ cm$$

$\Rightarrow a = 6.67 \ cm \Rightarrow$ use ⅃L $80*80*8$

* Design as Compression member:

1)Buckling assume $t_g = 1\ cm$

$$r_{x\,⅃L} = r_{x\,L} = 2.42\ cm$$

$$r_{y\,⅃L} = \sqrt{(r_x)^2 + (e + \frac{t_g}{2})^2} = \sqrt{(2.42)^2 + (2.26 + \frac{1}{2})^2}$$

$$= 3.67\ cm$$

$$*\ \lambda_{in} = \frac{l_{b\,in}}{r_x} = \frac{3.50*100}{2.42} = 144.6 < 200$$

$$*\ \lambda_{out} = \frac{l_{b\,out}}{r_y} = \frac{7.00*100}{3.67} = 190.7 < 200$$

2)Stresses $\lambda_{out} > 100$

$$*\ F_C = \frac{7500}{\lambda^2_{out}} = \frac{7500}{190.7^2} = 0.205$$

$$*\ F_{act} = \frac{force}{2A_L} = \frac{1.99}{2*12.3} = 0.081\ t\backslash cm^2 < F_c \quad (Safe)$$

* Design as Tension member:

3)Deflection

$$* \frac{l}{d} = \frac{l}{a} = \frac{350}{8} = 43.8 < 60$$

4)Stresses

لا نحتاج الى عمل هذا الـ Check لان الـ allowable tensile stresses كبيرة جدا و تساوى $1.4t \backslash cm^2$ و بالتالى أكيد ستكون Safe

---

Question (1)

[3]

Load Case	Column Base A			Column at point B		
	B.M.	N.F.	S.F.	B.M.	N.F.	S.F.
Dead Load	0.0 m.t	−4.0 t	0.0 t	0.0 m.t	−4.0 t	0.0 t
Live Load left bay	0.0 m.t	−4.0 t	+1.5 t	+0.9 m.t	−4.0 t	+1.5 t
Live Load right bay	0.0 m.t	−4.0 t	−1.5 t	−0.9 m.t	−4.0 t	−1.5 t
Wind Load	0.0 m.t	+5.0 t	−0.3 t	−1.8 m.t	+5.0 t	−0.3 t
$D+L_{left}$	0.0 m.t	−8.0 t	+1.5 t	+0.9 m.t	−8.0 t	+1.5 t
$D+L_{right}$	0.0 m.t	−8.0 t	−1.5 t	−0.9 m.t	−8.0 t	−1.5 t
D+W	0.0 m.t	+1.0 t	−0.3 t	−1.8 m.t	+1.0 t	−0.3 t
$D+W+L_{left}$	0.0 m.t	−3.0 t	+1.2 t	−0.9 m.t	−3.0 t	+1.2 t
$D+W+L_{right}$	0.0 m.t	−3.0 t	−1.8 t	−2.7 m.t	−3.0 t	−1.8 t
CASE (A)	0.0 m.t	−8.0 t	+1.5 t −1.5 t	+0.9 m.t −0.9 m.t	−8.0 t	+1.5 t −1.5 t
CASE (B)	0.0 m.t	+1.0 t −3.0 t	+1.2 t −1.8 t	−1.8 m.t −2.7 m.t	+1.0 t −3.0 t	+1.2 t −1.8 t

For column (at point B)

Case of maximum moment

$M_B = -2.7 m.t$ $\quad M_A = -0.9 m.t$ $\quad \frac{M_B}{M_A} > 1.2$

و بالتالى سوف نصمم على Case B و يكون

ال Critical فى Case B هو ال .Comp حيث أنه أخطر من ال Tension

بالاضافه أن معه Moment كبير .

$M = -2.7 m.t \quad N = -3t$

Case B

Case of maximum Normal

$N_B = -3\,t$ $\quad N_A = -8\,t$ $\quad \frac{N_B}{N_A} < 1.2$

و بالتالى سوف نصمم على Case A

$M = -0.9 m.t \quad N = -8t$

Case A

و سوف نقوم بالتصميم على ال Maximum moment case و عمل Check على ال Maximum Normal case

---

Case of maximum moment

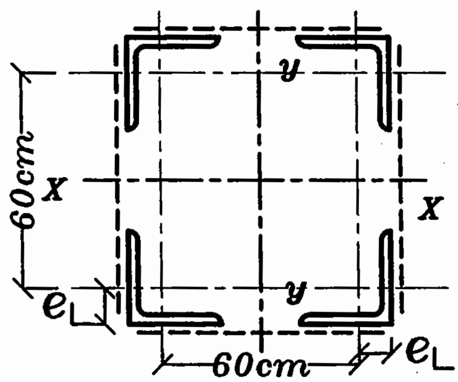
1) Suggest suitable bracing system

لا نحتاج لانه معطى فى المسألة

2) Calculate the straining actions

$N = 3t \qquad M_{in} = 2.7 m.t$

3) Choice of section



$* \; A_{⌐¬} = 4 A_L = 4 * 6.91 = 27.64 cm^2$

$* \; I_{X ⌐¬} = 4 \left[I_{X_L} + A_L \left(\frac{h}{2}\right)^2 \right]$

$= 4 \left[22.8 + 6.91 \left(\frac{30}{2}\right)^2 \right]$

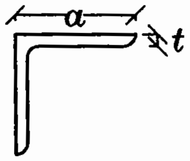
$= 6310 \; cm^4 = I_{y ⌐¬}$

$$* \; r_{x[\,]} = \sqrt{\frac{I_{x[\,]}}{A_{[\,]}}} = \sqrt{\frac{6310}{27.64}} = 15.11 cm^4 = r_{y[\,]}$$

4) Check Compactness

Flange subjected to comp.

$$* \; \frac{C}{t_f} = \frac{a}{t} = \frac{6}{0.6} = 10 < \frac{23}{\sqrt{f_y}} = 14.8$$



∴ The section is Non-compact

5) Check Compression

$l_{b\,in} = 2.0 \; (6 + \frac{2}{2}) = 14.0 m$ $\qquad$ $l_{b\,out} = 6.0 \, m$

$r_x = 15.11 \; cm$ $\qquad$ $r_y = 15.11 cm$

$$* \; \lambda_{in} = \frac{l_{b\,in}}{r_y} = \frac{1400}{15.11} = 92.65$$

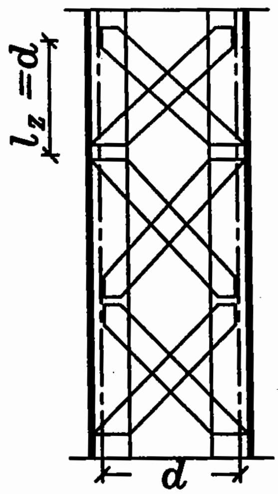
$$* \; \lambda_{out} = \frac{l_{b\,out}}{r_x} = \frac{600}{15.11} = 39.7$$

$*$ assume $l_Z = 60$ cm $\qquad r_Z = r_{v\,L} = 1.17$ cm

$$* \; \lambda_Z = \frac{l_Z}{r_Z} = \frac{60}{1.17} = 51.3$$

$$\ngtr 60$$

$$\ngtr \frac{2}{3} \lambda_{max.} \begin{cases} \lambda_{out} \\ \lambda_{in} \end{cases} = 70.6 \quad (105.9)$$



$$* \; \lambda_{in} = \sqrt{\lambda_{in}^2 + (k \lambda_Z)^2}$$

Lacing Bars

$$= \sqrt{92.65^2 + 1.00 * 51.3^2} = 105.9 < 180$$

$$* \quad \lambda_{out} = \sqrt{\bar{\lambda}^2_{out} + (k\,\lambda_Z)^2}$$

Lacing Bars

$$= \sqrt{39.7^2 + 1.00 * 51.3^2} = 64.87 < 180$$

$$* F_C = 1.4 - 6.5 * 10^{-5} \lambda^2_{max.}$$

$$= 1.4 - 6.5 * 10^{-5} * 105.9^2 = \boxed{0.67 \ t\backslash cm^2}$$

$$* f_{Ca} = \frac{N}{A} = \frac{3}{27.64} = \boxed{0.109 \, t\backslash cm^2}$$

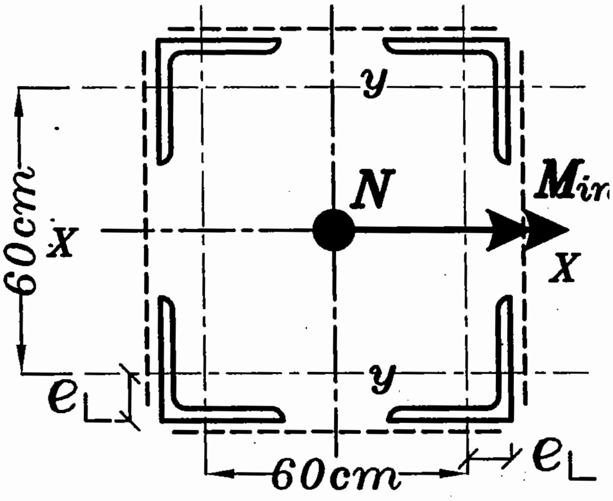
$$* \frac{f_{Ca}}{F_C} = \frac{0.109}{0.67} = 0.161 > 0.15$$

$$* F_{Ey} = \frac{7500}{\lambda^2_y} = \frac{7500}{64.87^2} = 1.78$$

Permitted to sway

$$A_2 = \frac{C_{my}}{[1 - \frac{f_{ca}}{F_{Ey}}]} = \frac{0.85}{[1 - \frac{0.161}{1.78}]} = 0.99 \not< 1.0 \qquad \boxed{A_2 = 1}$$

6) Check Bending



$$f_{b(act.)in} = \frac{M_{in}}{I_y}\left(\frac{d}{2} + e_L\right) = f_{by}$$

$$= \frac{270}{6310}\left(\frac{60}{2} + 1.17\right) = \boxed{1.33 \ t\backslash cm^2}$$

$F_{bcy} = 0.58\,F_y$ No LTB

Non-Compact تكون angle لان ال

$$\boxed{F_{bcy} = 1.40 \ t\backslash cm^2}$$

7) Check Interaction equation

$$\frac{f_{Ca}}{F_C} + \frac{f_{bx\,(act.)}}{F_{bCx}} * A_1 + \frac{f_{by\,(act.)}}{F_{bCy}} * A_2 < 1.2 \Rightarrow \text{Case } B$$

(the f_{bx} term is struck through)

$$0.161 + \frac{1.33}{1.40} * 1.0 = 1.11 < 1.2 \Rightarrow \textbf{SAFE}$$

Case of maximum Normal

5) Check Compression

$* \lambda_{in} = 105.9$ $\qquad * \lambda_{out} = 64.87$

$$* F_C = 1.4 - 6.5 * 10^{-5} \lambda^2_{max.}$$
$$= 1.4 - 6.5 * 10^{-5} * 105.9^2 = \boxed{0.67\ t\backslash cm^2}$$

$$* f_{Ca} = \frac{N}{A} = \frac{8}{27.64} = \boxed{0.29\ t\backslash cm^2}$$

$$* \frac{f_{Ca}}{F_C} = \frac{0.29}{0.67} = 0.430 > 0.15$$

$$* F_{Ey} = \frac{7500}{\lambda_y^2} = \frac{7500}{64.87^2} = 1.78$$ Permitted to sway

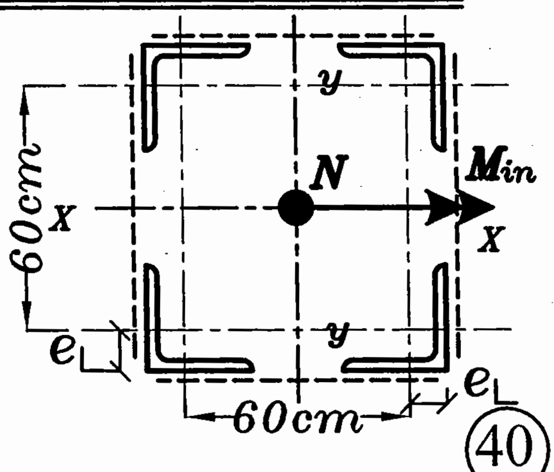
$$A_2 = \frac{C_{my}}{[1 - \frac{f_{ca}}{F_{Ey}}]} = \frac{0.85}{[1 - \frac{0.29}{1.78}]} = 1.02 \nless 1.0 \qquad \boxed{A_2 = 1.02}$$

---

6) Check Bending

$$f_{b(act.)in} = \frac{M_{in}}{I_y}\left(\frac{d}{2} + e_L\right) = f_{by}$$

$$= \frac{90}{6310}\left(\frac{60}{2} + 1.17\right) = \boxed{0.44\ t\backslash cm^2}$$



$F_{bcy} = 0.58 F_y$ No LTB

Non-Compact تكون angle لان ال

$F_{bcy} = 1.40\ t \backslash cm^2$

7) Check Interaction equation

$$\frac{f_{Ca}}{F_C} + \frac{f_{bx\,(act.)}}{F_{bcx}} * A_1 + \frac{f_{by\,(act.)}}{F_{bcy}} * A_2 < 1.0 \Rightarrow \text{Case } A$$

$$0.430 + \frac{0.44}{1.40} * 1.02 = 0.75 < 1.0 \Rightarrow \text{SAFE}$$

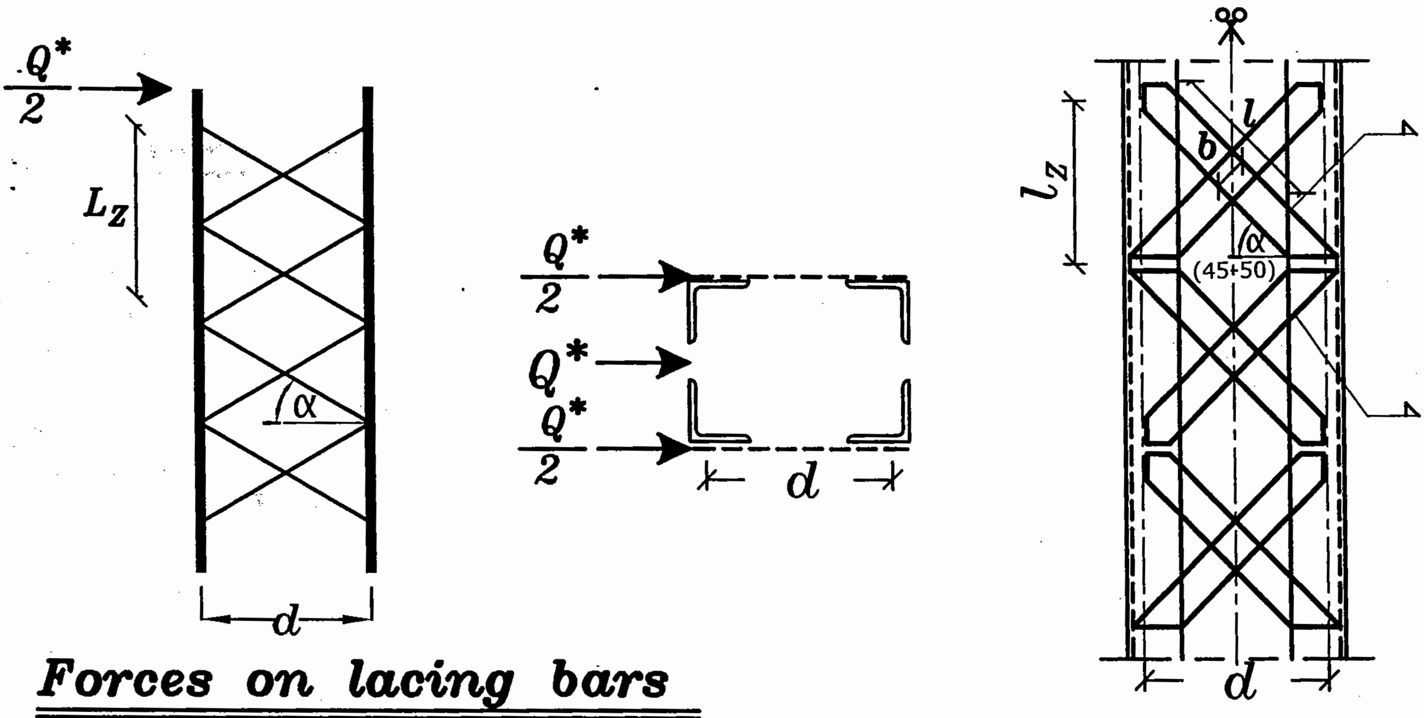
Design of lacing bars

From table :

$Q_B = -1.8\ t$ $\quad Q_A = -1.5\ t$ $\quad \frac{N_B}{N_A} \leqslant 1.2$

$Q = -1.5t \quad N = -8\ t$ Case A

و بالتالى سوف نصمم على Case A



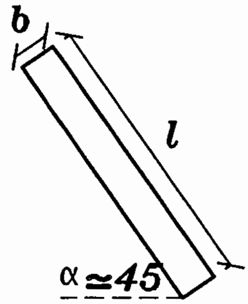
Forces on lacing bars

$$* Q^* = Q + \frac{2}{100} N = 1.87 + \frac{2}{100} * 8 = 2.03\ t$$

$$* \text{ Force in lacing bar} = \pm \frac{Q^*}{4 \cos \alpha}$$

$$= \pm \frac{1.93}{4 \cos 45} = 0.72\ t$$

Dimensions of lacing bars



$$* \ L \simeq \frac{d}{\cos \alpha} = \frac{60}{\cos 45} \simeq 85\ cm$$

$$* \ b = \frac{L}{10} = \frac{85}{10} = 8.5\ cm$$

* assume thickness of lacing bar = $t = 2$ cm

Check on section

1)As compression member

$$* \ L_{b_x} = L_{b_y} = 0.75\,L = 64\ cm$$

$$* \ r_X = \frac{t}{\sqrt{12}} = \frac{2}{\sqrt{12}} = 0.577\,cm$$

Check buckling

$$* \ \lambda_X = \frac{l_X}{r_X} = \frac{64}{0.577} = 111 < 140 \Rightarrow \text{Safe}$$

Check Stresses

$$* F_C = \frac{7500}{\lambda^2_{max.}} = \frac{7500}{111^2} = \boxed{0.608\ t\backslash cm^2}$$

$$* f_{Ca} = \frac{\text{Force}}{\text{Area}} = \frac{0.72}{8.5 * 2} = \boxed{0.04\ \ t\backslash cm^2} < F_C$$

2)As Tension member

Check Stresses

$$* f_{act.} = \frac{\text{Force}}{\text{Area}} = \frac{0.72}{8.5 * 2} = \boxed{0.04\ \ t\backslash cm^2} < 1.4\ t\backslash cm^2$$

Check deflection

$$\frac{L}{b} = \frac{85}{8.5} = 10 \not> 60 \Rightarrow Safe$$

Assume welded connection

* assume size of weld $S = 8\ mm$

$$L_{act.} = \frac{Force}{0.4 F_u * S} + 2S = \frac{0.72}{2 * 0.72 * 0.80} + 2 * 0.60 = 1.79 cm$$

* use min. length $\boxed{L_{act.} = 5\ cm}$

Question (1)

[4] From table :

القاعدة يؤثر عليها Normal & Shear و بالتالى نحتاج الى دراسة 2-Cases

Case of maximum Normal

$N_B = -3\,t$ $N_A = -8\,t$ $\frac{N_B}{N_A} < 1.2$

و بالتالى سوف نصمم على Case A

$\boxed{N = -8t \quad Q = -1.5\,t}$ Case A

Case of maximum Shear

$Q_B = -1.8\,t$ $Q_A = -1.5\,t$ $\frac{N_B}{N_A} \leq 1.2$

و بالتالى سوف نصمم على Case A

$\boxed{N = -8t \quad Q = -1.5\,t}$ Case A

و بالتالى سوف نصمم على Case واحدة فقط

1) Check Weld

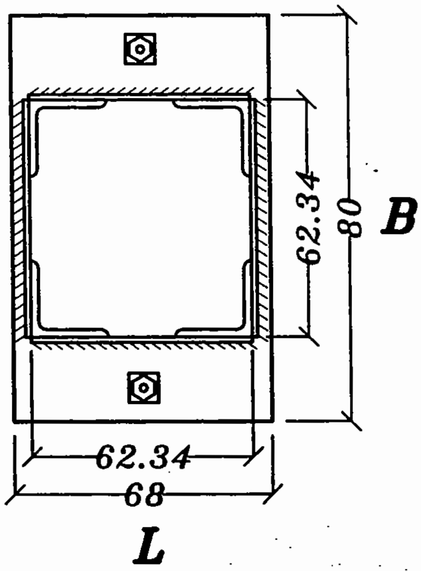
* Assume size of weld = 6 mm

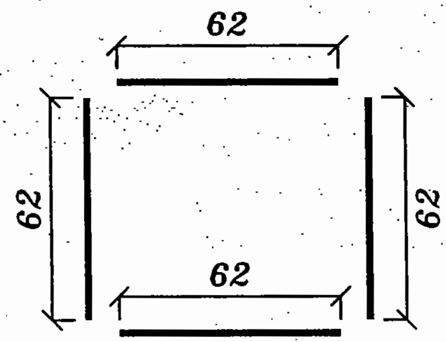
* $A_{w_{hz}} = 2 * 0.6 * 62 = 74.4\ cm^2$

* $A_{w_{vl}} = 2 * 0.6 * 62 = 74.4\ cm^2$

* $A_{w_{Total}} = A_{w_{hz}} + A_{w_{vl}}$

$= 148.8 cm^2$





$$f_1 = \frac{0.6 N}{A_{w_{Total}}} = \frac{0.6 * 8}{148.8} = \mathbf{0.03}\ t \backslash cm^2$$

$$q_1 = \frac{Q}{A_{w_{hz}}} = \frac{1.5}{74.4} = \mathbf{0.02}\ t \backslash cm^2$$

$$f = \sqrt{f_1^2 + 3 q_1^2} = \mathbf{0.05}\ t \backslash cm^2$$

$$\not> 0.72 * 1.1 t \backslash cm^2$$

2) Check Bearing stresses

* Assume $f_{cu} = 250\ Kg \backslash cm^2$

$$f_b = \frac{N_{(Kg)}}{B * L} = \frac{8 * 10^3}{80 * 68} = \mathbf{1.47}\ kg \backslash cm^2 \not> 75\ Kg \backslash cm^2$$

3) Design anchor Bolts

$$* R_{Sh} = q_b * A_S * 1$$

$$= 0.25 F_{ub} * \frac{\pi \phi^2}{4} * 1$$

ال anchor bolts دائما نستخدملها steel 52 و تكون ال $F_{ub} = 5.2\ t \backslash cm^2$

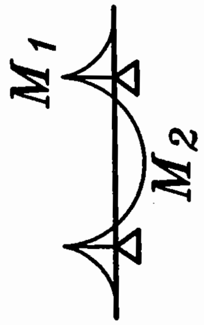
و دائما نأخذ ال $q_b = 0.25 F_{ub}$

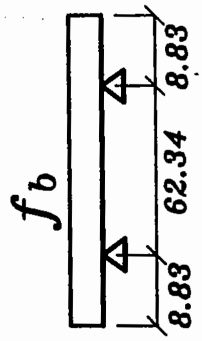
$$* n = \frac{Q}{R_{sh} * 0.85} = \frac{1.5}{(0.25 * 5.2) * \frac{\pi \phi^2}{4} * 0.85} = 2$$

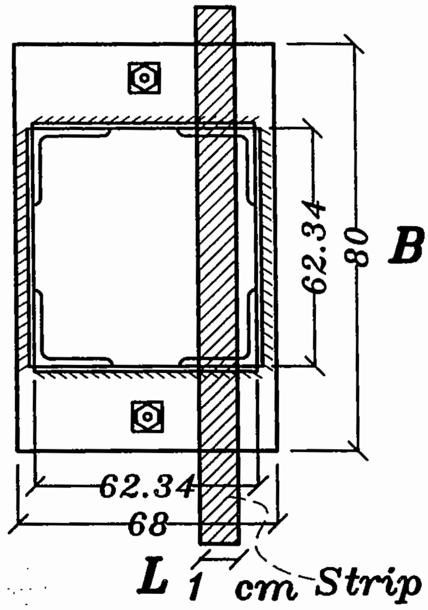
$\Rightarrow \phi = \mathbf{0.73} cm = \mathbf{7.30} mm \quad \Rightarrow$ Use $\boxed{2 \phi 20}$

4) Design of base plate

* $f_b = 0.0015\ t\backslash cm^2$







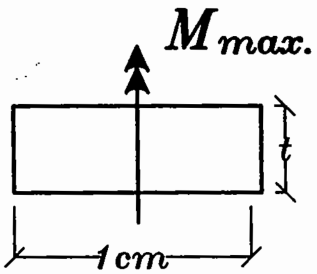
* $M_1 = f_b * \frac{8.83^2}{2} = 0.06\ cm.t$
* $M_2 = f_b * \frac{62.34^2}{8} - M_1 = \mathbf{0.67}\ cm.t$

$M_{max.} = 0.67\ cm.t$

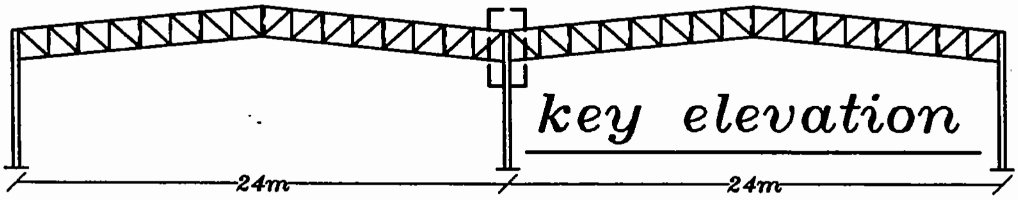
* $t = \sqrt{\frac{6 M_{max.}}{0.72\, F_y}} = \sqrt{\frac{6 * 0.67}{0.72 * 2.4}} = 1.52\ cm$

($M_{max.}$ in t.cm; F_y in $t\backslash cm^2$)

⇨ Use $t_{min.} = 20\ mm$



Critical section



key elevation
24m
24m

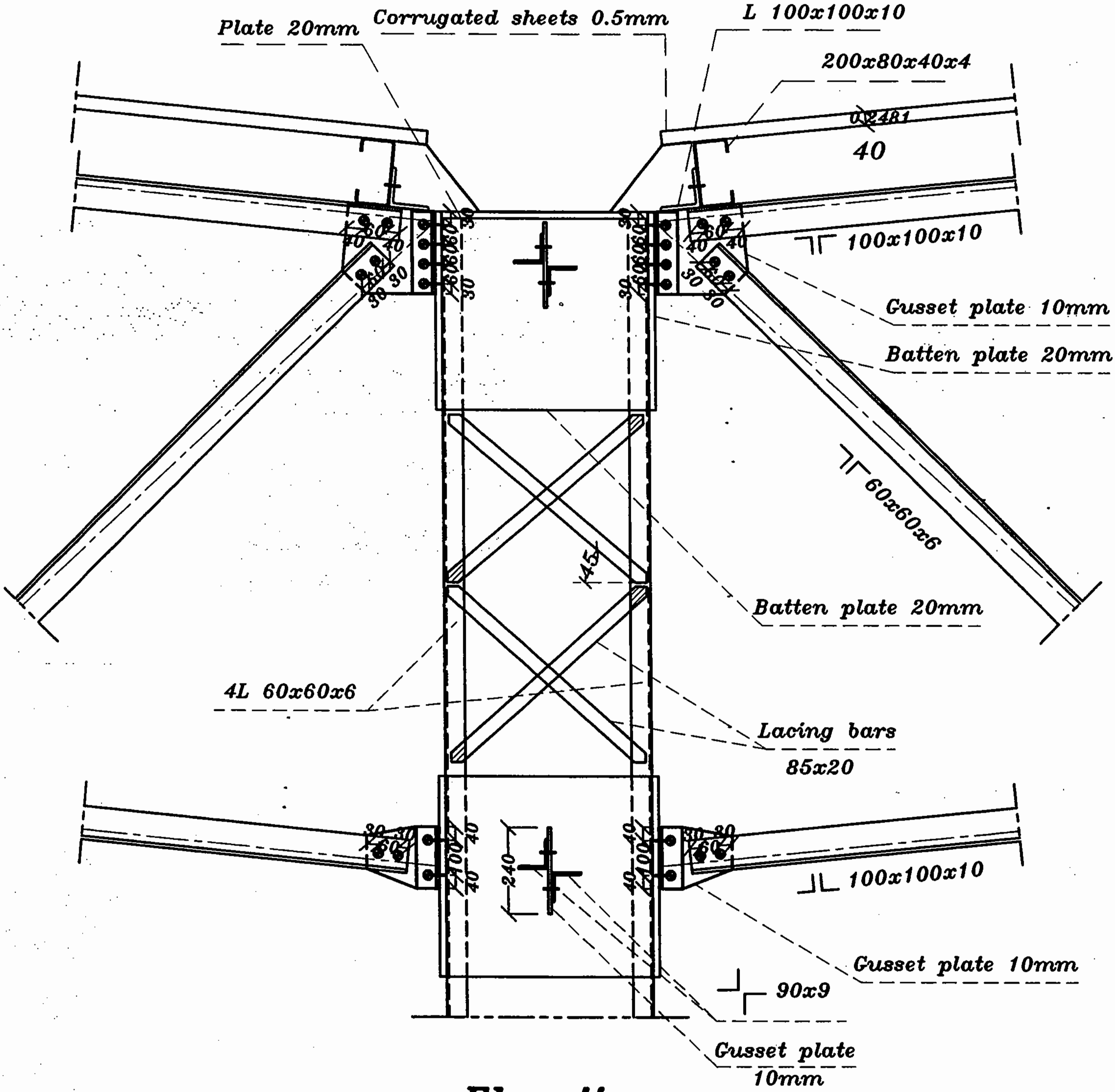
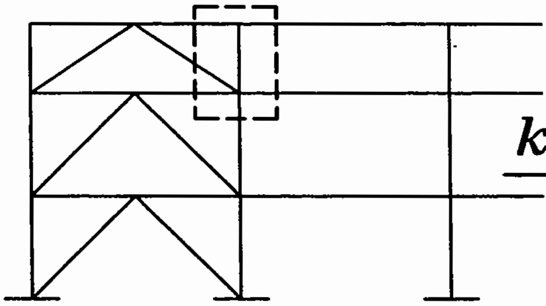
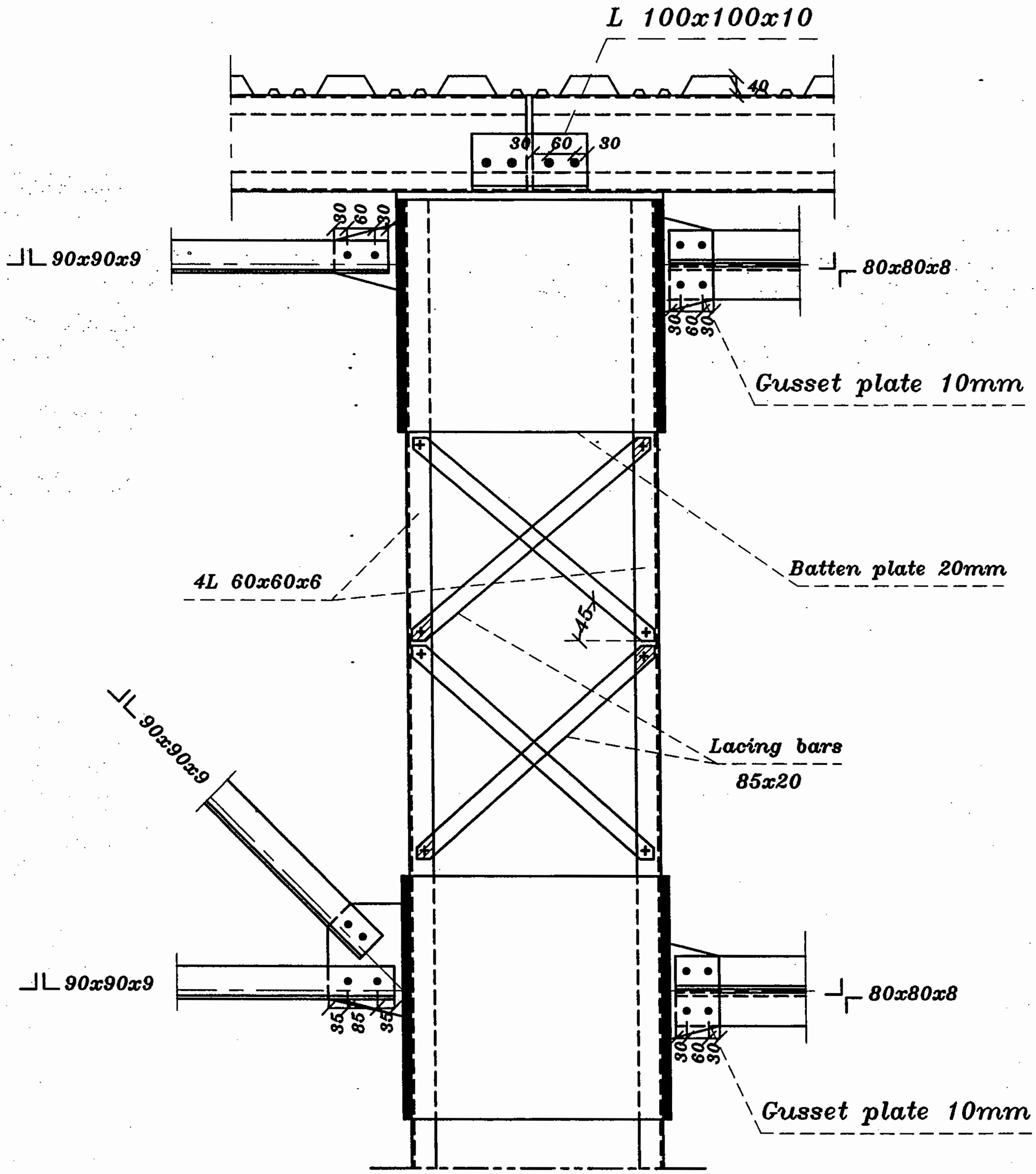


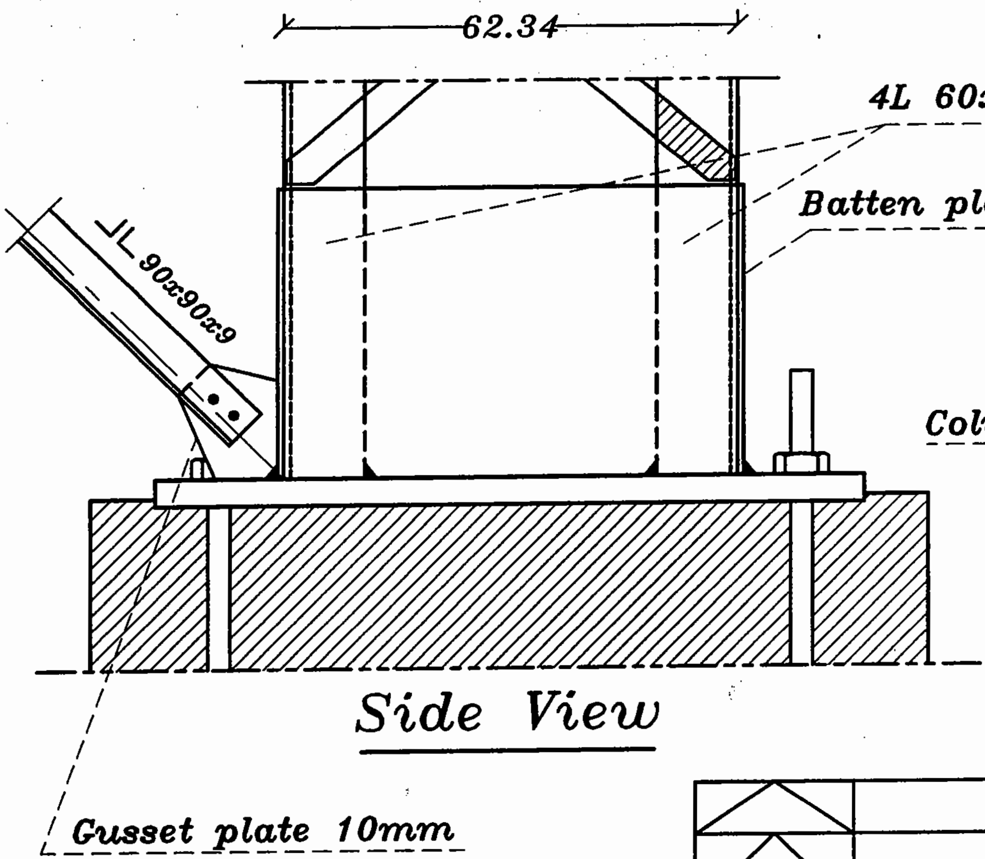
Plate 20mm
Corrugated sheets 0.5mm
L 100x100x10
200x80x40x4
40
100x100x10
Gusset plate 10mm
Batten plate 20mm
60x60x6
45
Batten plate 20mm
4L 60x60x6
Lacing bars
85x20
240
100x100x10
Gusset plate 10mm
90x9
Gusset plate
10mm

Elevation

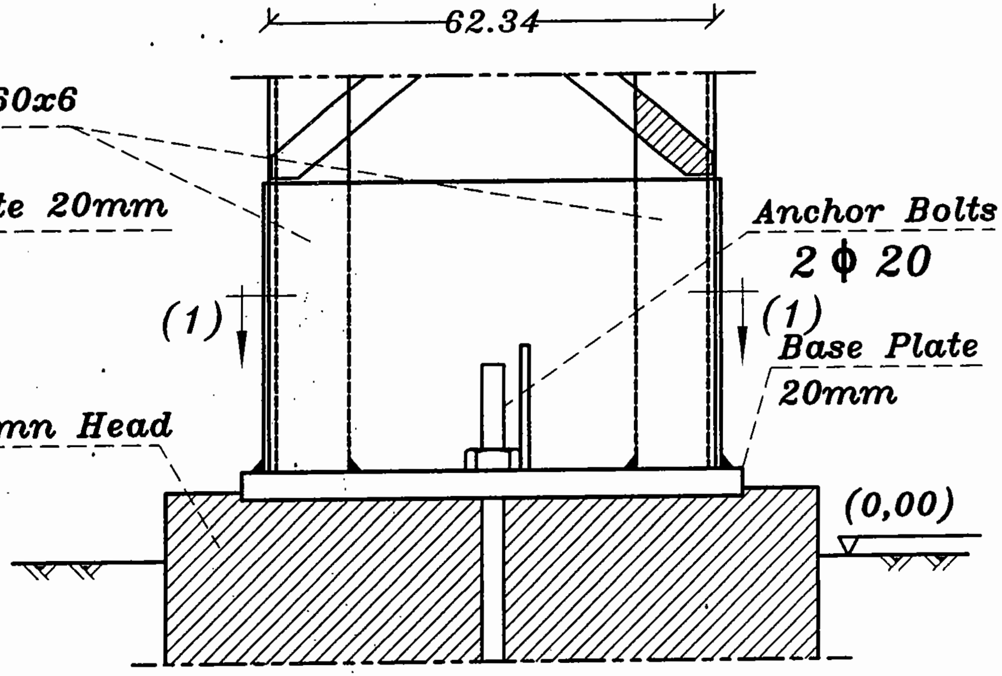


key side view

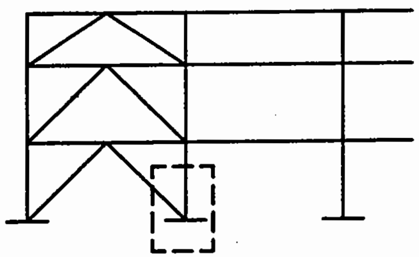




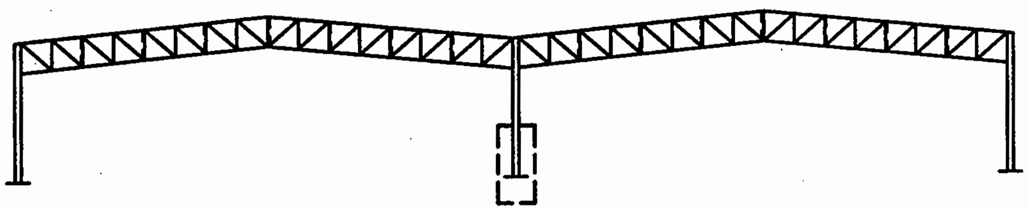
Side View



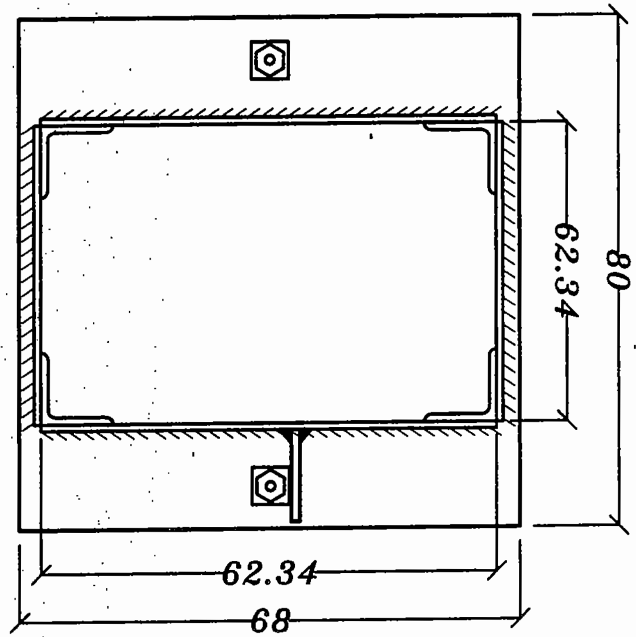
Eleation



key side view



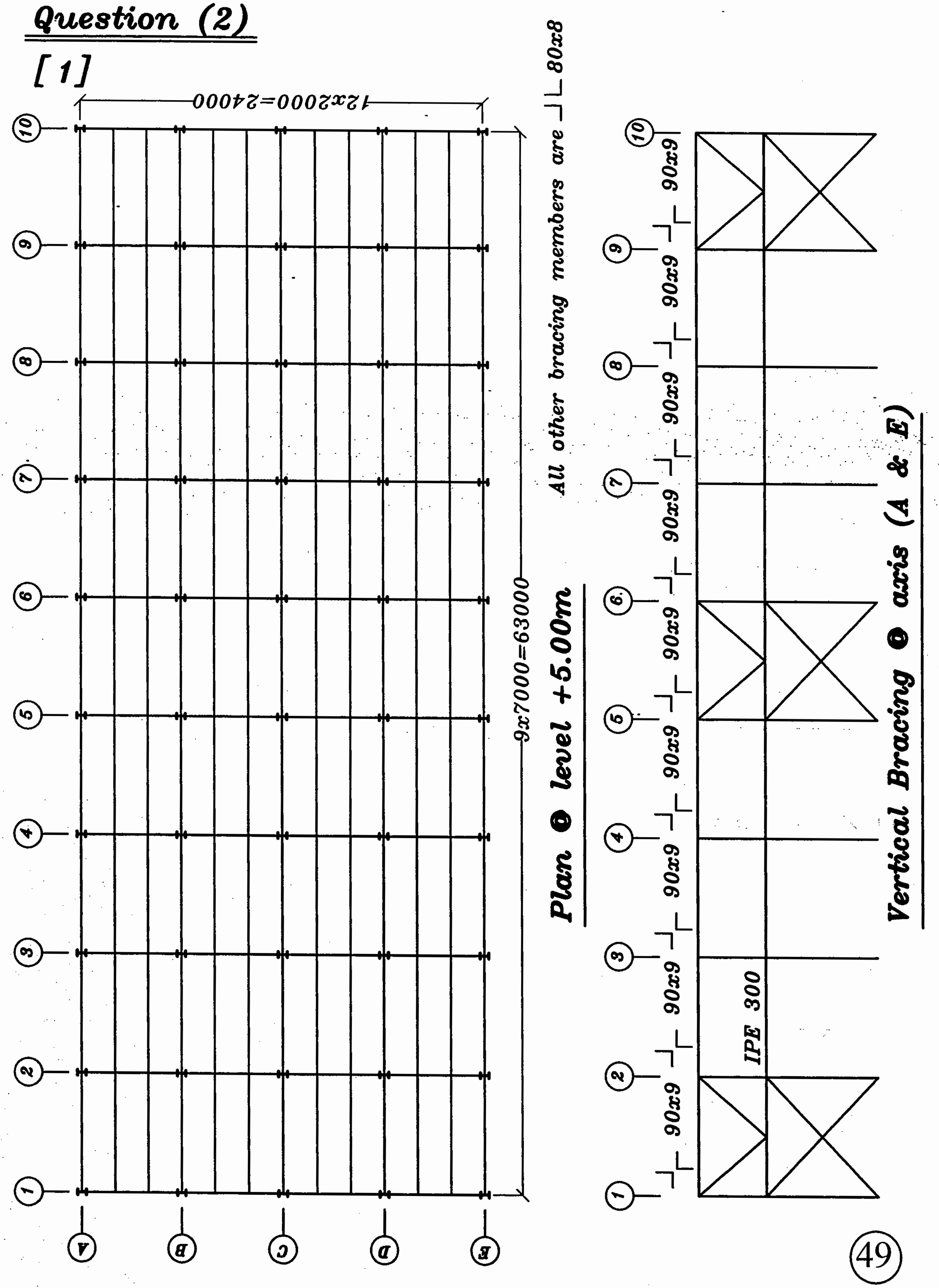
key elevation



Sec. (1–1)

Question (2)

[1]



12x2000=24000
9x7000=63000
All other bracing members are ⅃L 80x8
Plan @ level +5.00m
⅃L 90x9
IPE 300
Vertical Bracing @ axis (A & E)

Question (2)

[2]

1) Suggest suitable bracing system

. Layout في ال

2) Calculate the straining actions

$T = 6\,t \qquad M_x = 40\,m.t$

3) Choice of section

* Assume (allowable stress) $f = 1.00\ t\backslash cm^2$

* $S_x = \dfrac{M_x}{F} = \dfrac{40*100}{1.0} = 4000\ cm^3$

⇨ Choose **HEB 500**

4) Check Compactness

For flange Subjected to compression

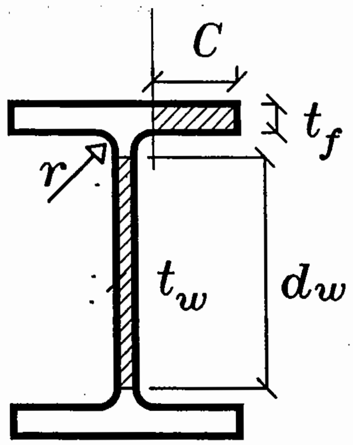
$d_w = 39.0\,cm$ جداول

$t_w = 1.45\,cm$

$b_f = 30.0\,cm$

$t_f = 2.80\,cm$

$r = 2.8\,cm$



$$\frac{C}{t_f} = \frac{\frac{1}{2}(b_f - t_w - 2r)}{t_f} = \frac{\frac{1}{2}(30.0 - 1.45 - 2*2.8)}{2.8} = 8.19$$

$$\therefore \frac{C}{t_f} = 8.19 < \frac{16.9}{\sqrt{f_y}} = 10.9 \Longrightarrow \text{Compact Flange}$$

For Web

$* \quad d_w * t_w * F_y = 39.0 * 1.45 * 2.4 = \mathbf{56.55} t > N = \mathbf{6}\, t$

Web ⟹ Subjected to Bending

$$* \alpha = \frac{1}{2}\left[\frac{N}{d_w * t_w * F_y} + 1\right] = \frac{1}{2}\left[\frac{-6}{56.55} + 1\right] = 0.44 \quad < 0.5$$

$$\frac{dw}{t_w} = \frac{39.0}{1.45} = 26.9 < \frac{\mathbf{63.6}}{\alpha \sqrt{f_y}} = 93.3 \Longrightarrow \text{Compact Web}$$

∴ The section is compact

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5) Check Tension

$l_{b\,in}$ ⟹ Frame ⟹ $G_A \& G_B$

و حيث أننا لا نملك قطاعات الكمرات و بالتالى لن نستطيع حساب ال $G_A \& G_B$

فمن الممكن فرض ال k أكبرن واحد حيث أن ال Hanger معرض لـ Sway

$l_{b\,in} = 2*3 = 6\,m$ $\qquad$ $l_{b\,out} = 3.0\,m$

$r_x = 21.2\ cm$ $\qquad$ $r_y = 7.27\ cm$

$$* \lambda_{\substack{in\\x}} = \frac{l_{b\,in}}{r_x} = \frac{600}{21.2} = \mathbf{28.3} < 300$$

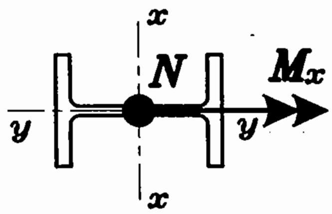
$$* \lambda_{\substack{out\\y}} = \frac{l_{b\,out}}{r_y} = \frac{300}{7.27} = \mathbf{41.27} < 300$$

$$* F_t = \boxed{1.40\ t\backslash cm^2}$$

$$* f_{ta} = \frac{T}{A} = \frac{6}{239} = \boxed{0.025\ t\backslash cm^2}$$

$*$ No compression ⟹ No Buckling ⟹ $\boxed{A_1 = 1}$

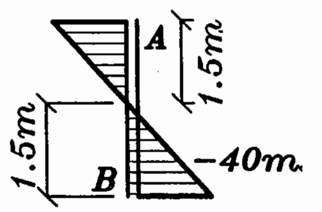
6) Check Bending



* $f_{b(act.)x} = \dfrac{M_x}{S_x} = \dfrac{4000}{4290} = \boxed{0.932\ t\backslash cm^2}$

* $l_{u\,act.} = 150\ cm$

* $l_{u\,max.} = \begin{cases} \dfrac{20\,b_f}{\sqrt{f_y}} = \dfrac{20 * 30.0}{\sqrt{2.4}} = 378\ cm \\ \dfrac{1380\,A_f}{d * F_y} C_b = \dfrac{1380 * 30 * 2.8}{50 * 2.4} * 2.3 = 2221 cm \end{cases}$



$C_b = 2.3$

$l_{u\,max.} = 378\ cm$

$l_{u\,act.} < l_{u\,max.} \Rightarrow$ **No LTB**

$F_{bcx} = \boxed{1.40\ \ t\backslash cm^2}$

$\dfrac{M_X}{S_X} \not> F_{bcx} \Rightarrow$ Safe

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7) Check the interaction equation

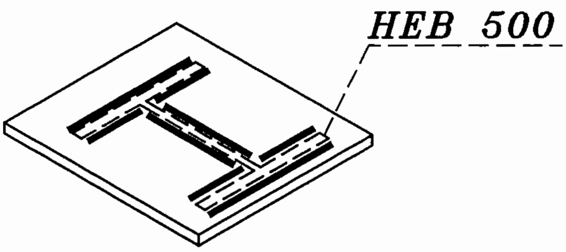
For tension side

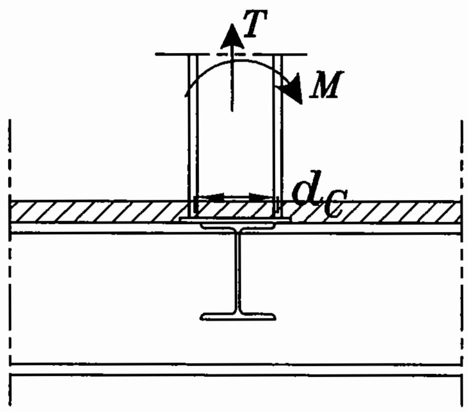
$$\boxed{\dfrac{\frac{T}{A}}{1.4} + \dfrac{f_{bx\,(act.)}}{F_{bt}} * A_1 < 1.0}$$

$$\dfrac{0.025}{1.40} + \dfrac{0.932}{1.40} * 1 = 0.68 < 1.0 \Rightarrow \text{Safe}$$

Question (2)

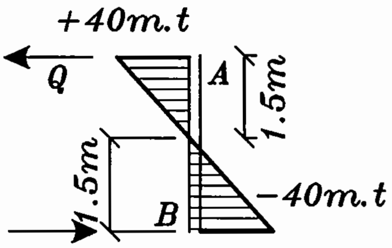
[3] لاحظ أن المطلوب فقط هو تصميم ال *Welded Connection* بين العمود و ال *Head plate* ويتم عمل تصميم مثل القواعد





$M = 40\ m.t$	$T = 6\,t$

$$Q = \frac{2M}{h} = \frac{2*40}{3.0} = 26.7\,t$$



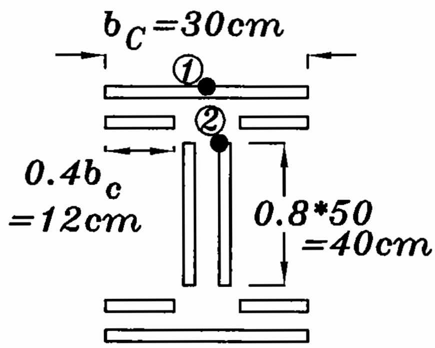
* **HEB 500 dimensions:**

• $d_C = 50cm$ • $b_C = 30cm$ • $t_C = 2.8cm$ • $w_C = 1.45cm$

weld dimensions

assume $S_f = 26\ mm < t_C = 28\ mm$

$S_w = 12\ mm < w_C = 14.5\ mm$



For one flange

$$A_{w.Hz.} = (30*2.6) + 2*(12*2.6) = 140\ cm^2$$

$$A_{w\,web} = 2*(40*1.20) = 96\ cm^2$$

$$I_{x.weld.} = \left[\left(\frac{1.2*40.0^3}{12}\right)\right]*2 + [(2.6*30)(21.3)^2]*2 + [(2.6*12)(15.9)^2]*4$$

$$= 115126.328\ cm^4$$

** Check stresses:

Point 1: (Horizontal weld)

$$f_1 = \frac{M}{I_{x.weld.}} \cdot y_1 = \frac{40*100}{115126.32}(20+2.6) = 0.785 \ t/cm^2 < 0.72 \ t/cm^2$$

(unsafe)

Point 2: (Vertical weld)

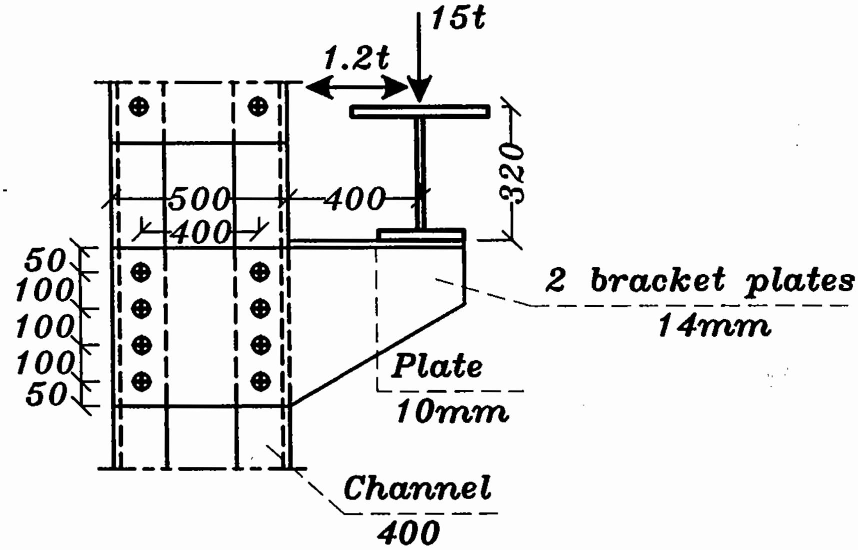
$$f_2 = \frac{M}{I_{x.weld.}} \cdot y_2 = \frac{40*100}{115126.32} * 20 = 0.69 \ t/cm^2$$

$$q_2 = \frac{Q}{A_{w_{web}}} = \frac{26.7}{96} = \mathbf{0.28}\, t \backslash cm^2$$

$$\Rightarrow \sqrt{f_2^2 + 3 q_2^2} = 0.843 t/cm^2 > 0.72 * 1.1 \quad \text{(unsafe)}$$

Question (3)

[1]

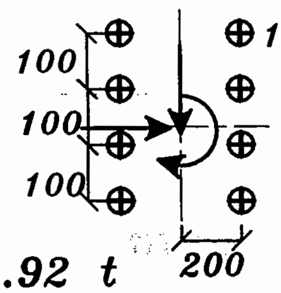


Straining actions

- $Q_x = \frac{P_x}{2} = \frac{1.2}{2} = 0.60\,t$
- $Q_y = \frac{P_y}{2} = \frac{15}{2} = 7.5\,t$
- $M_t = 7.5 * (0.4 + 0.25)$
  $+ 0.60\left(0.32 + 0.01 + \frac{0.40}{2}\right) = 5.19\, m.t.$

Calculate the force on critical Bolt and Check

Bolt 1



$\sum r^2 = \sum (x^2 + y^2) = n x^2 + 4(y_1^2 + y_2^2 + \dots)$

$\sum r^2 = 8 * (20.0)^2 + 4 * (15^2 + 5^2) = 4200\ cm^2$

$R_x = \frac{Q_x}{n} + \frac{M_t}{\sum r^2} * y_1 = \frac{0.60}{8} + \frac{5.19 * 100}{4200} * 15 = 1.92\ t$

$R_y = \frac{Q_y}{n} + \frac{M_t}{\sum r^2} * x_1 = \frac{7.5}{8} + \frac{5.19 * 100}{4200} * 20 = 3.40\ t$

$R_1 = \sqrt{3.40^2 + 1.92^2} = 3.91\ t$

$*R_{sh} = q_b * A_S * n$

$= 0.25\, F_{ub} * \frac{\pi \phi^2}{4} * 1 = 0.25 * 8 * \frac{\pi * 1.6^2}{4} * 1 = 4.02\, t$

$edge\ distance = 50\,mm > 3\phi = 48\ mm$

$\Longrightarrow \alpha = 1.2$

$$*R_b = 1.20\,F_u * \phi * t_{min} \begin{cases} t_{f_{UPN400}} = 1.80 \\ t_P = 1.40 \end{cases}$$

$$= 1.20 * 3.6 * 1.6 * 1.40 = \mathbf{9.67}\,t$$

$$R_{least} = \mathbf{4.02}\,t$$

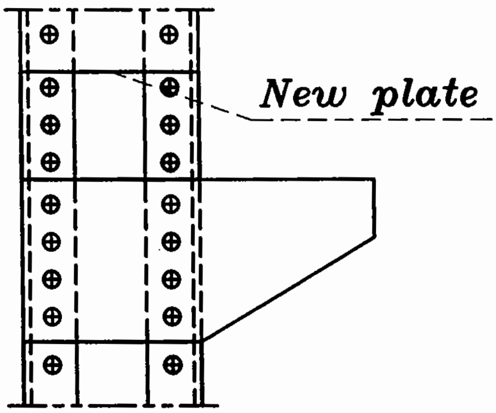
$$\mathbf{R_1 = 3.91\ t} < R_{least} = \mathbf{4.02}\,t \Longrightarrow \textbf{Safe}$$

Question (3)

[2]

الغرض هنا هو تقوية الوصلة على حالها هذا لانها *existing connection* أى أنها موجودة و نريد عمل *strengthening* لها .

١- اضافة *Plate* بين الـ *Batten Plates* و لحامه معها حتى نتمكن من اضافة بعض المسامير و بالتالى تستطيع الوصلة تحمل احمال اعلى .



٢- اضافة *Weld* مع المسامير .