STeel مراجعة

19

FINAL REVISION

PART (1)

EXAM 2014

EXAM 2010

EXAM 2013

EXAM 2002

AIN SHAMS UNIVERSITY — FACULTY OF ENGINEERING

STRUCTURAL ENGINEERING DEPARTEMENT

THIRD YEAR CIVIL — STRUCTURAL DIVISION

January 2014 — STEEL STRUCTURES — TIME:3.00 Hrs

Material of construction is steel 37.

Question (1) (16Marks)

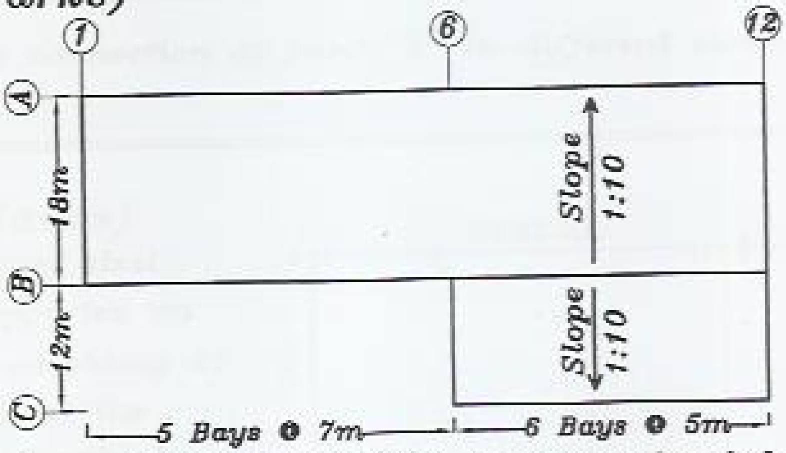


Fig.(1)

Fig.(1) illustrates a plan of an industrial building to be constructed using a series of steel trusses. From Axes (1) to (6) the spacing between the trusses is 7m. However, from Axes (6) to (12) the spacing between is 5m. The required minimum clear height under the lower chord is 7m.

Draw to scale 1:100 a complete general layout of the building showing the main supporting elements and the bracing systems. (Structural roof plan, Elevations at Axes (A), (C), and longitudinal bracing. Cross section at Axes (1) and (6) .

Question (2) (34Marks)

The field connections of the shown Truss are at joints "e" and "f". It is required to:

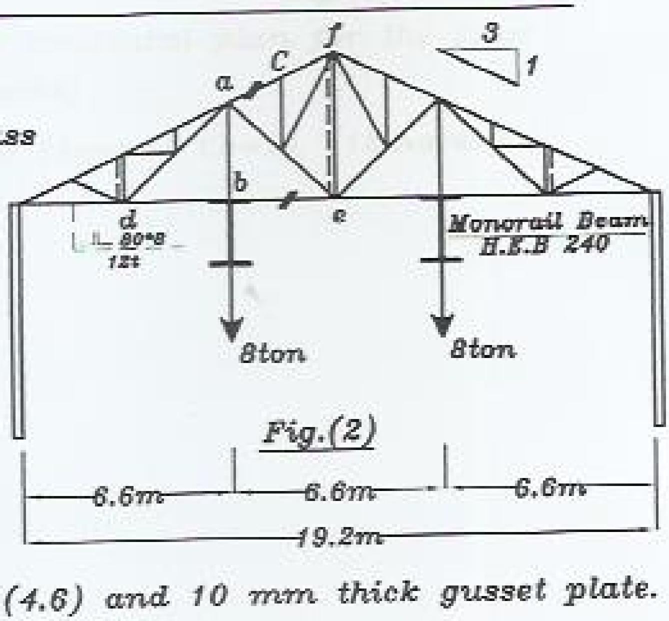


Fig.(2)

2.1) Design a suitable section for the marked members (a–C) and (b–e), given that

F_{ac} =−28ton (D.L+L.L+Wind Load)

F_{be} =+30ton (D.L+L.L)

Use non-pretensioned bolts M20 grade (4.6) and 10 mm thick gusset plate. (8marks)

2.2) Check the stresses and the suitability of using a single angle section 70x70x7 for member (a–b) (4marks)

2.3) Design the welded connection of the truss members meeting at joint (b). (8marks)

2.4) Design the bolted connection between the monorail beam (HEB240) and the lower chord of the truss at joint "b". Use pretensioned bolts M20 (8.8) ($As=2.45Cm^2$, $Ps=3.45t$, $T=10.8t$) (4marks)

2.5) Draw to scale 1:10 the connection at joint "b" in different views. (10marks)

Question (3) (20Marks)

Fig.(3) shows a PLAN view of steel platform (7.50mx8.0m) supported on four columns. A machine weighting 20 ton is to be located on top of the platform as illustrated in Fig.(3). The total deal load of the platform including steel grating (flooring), floor beams & horizontal bracing (if required) = 200kg\m2. The live load = 400kg\m2. HEB 500 section is used for columns and main girders of platform. It is required to :

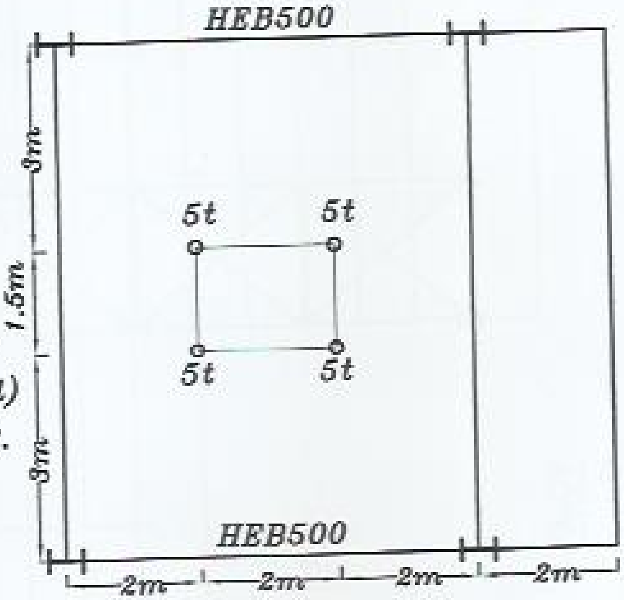


Fig.(3)

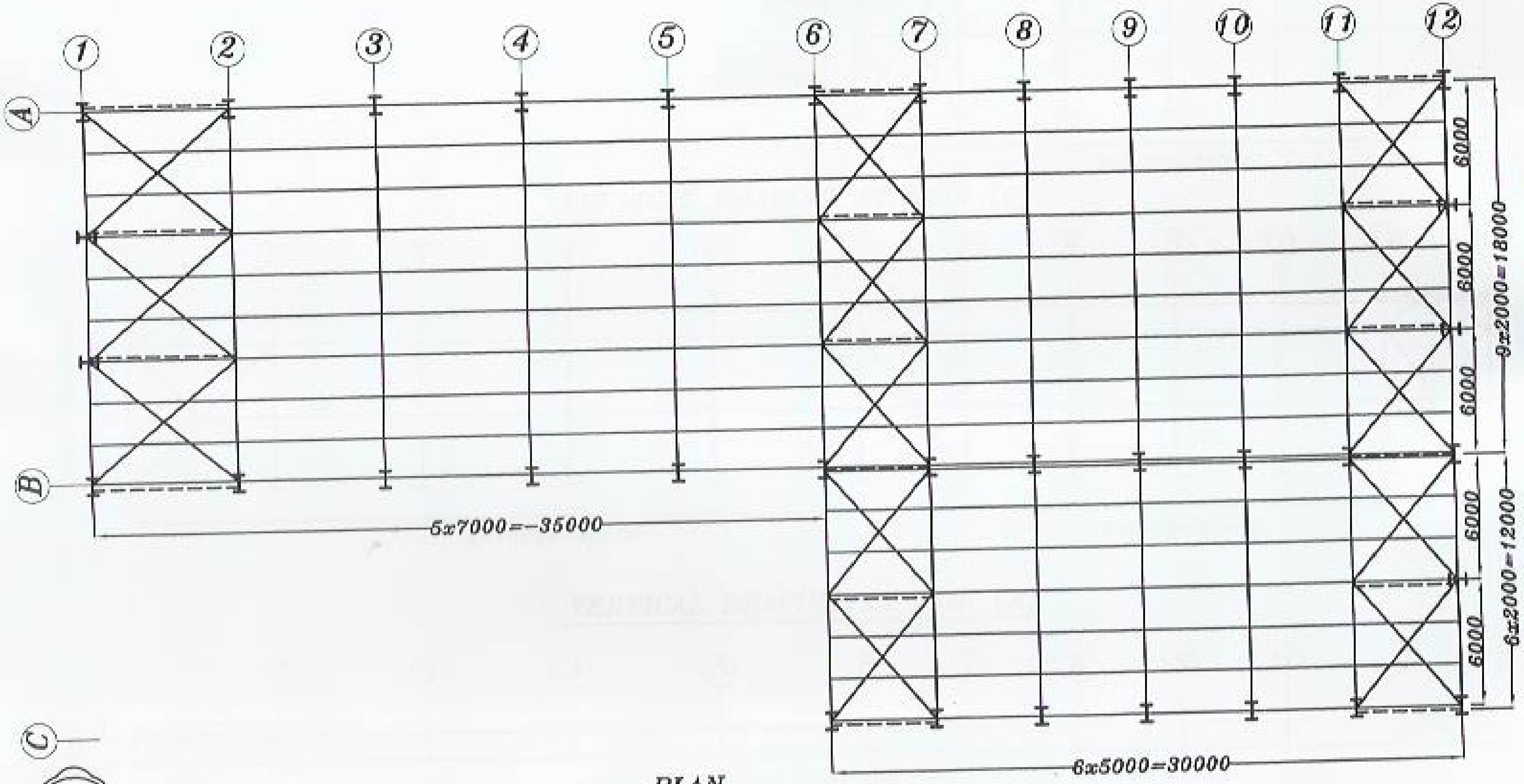
(3.1) Suggest and draw to scale 1:50 a structural plan for the floor beams and the horizontal bracing. (8marks)

(3.2) Design the critical secondary beam (Assume Cb=1). (12marks)

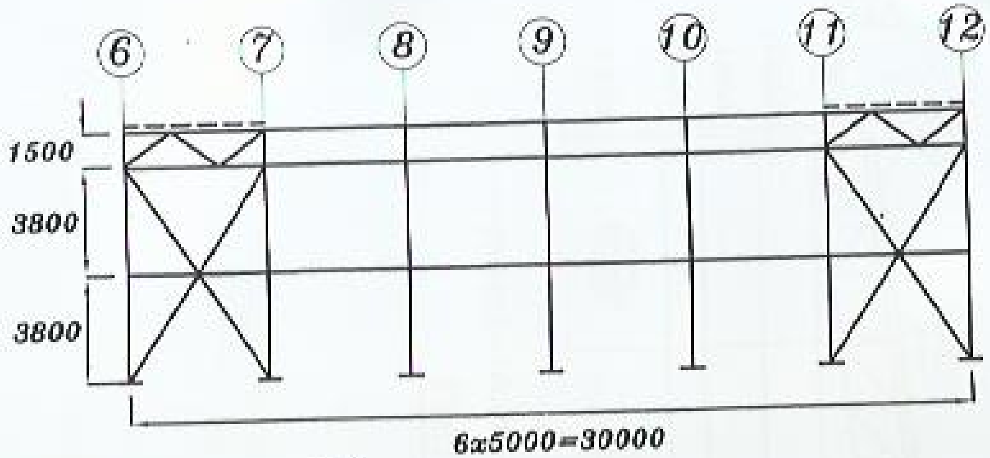
Question (1)

$$H = \frac{Span\ (B)}{12 \Rightarrow 16} = \frac{18}{12 \Rightarrow 16} = 1.13 \Rightarrow 1.50m$$

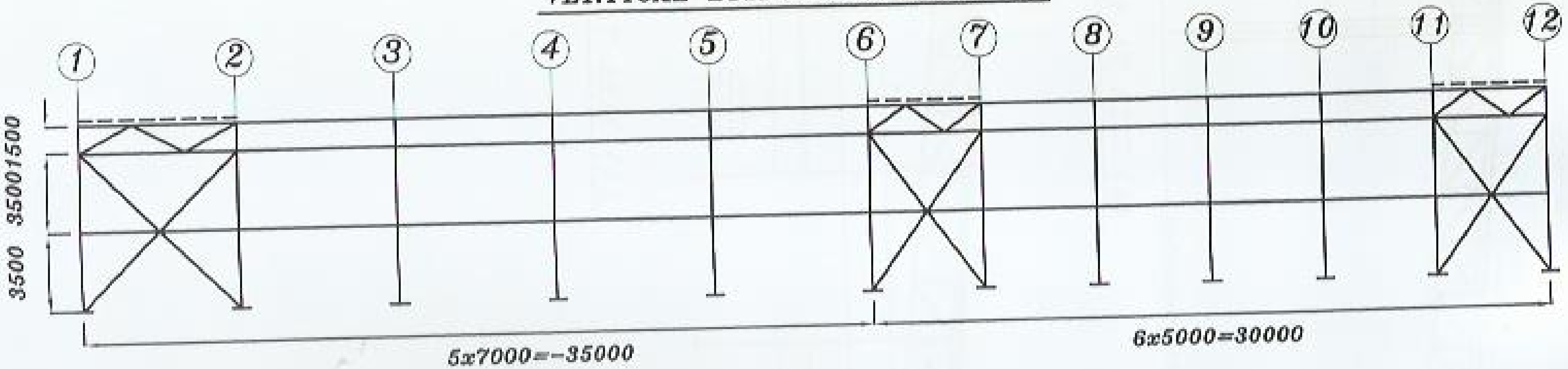
Take $H = 1.50\ m$



PLAN

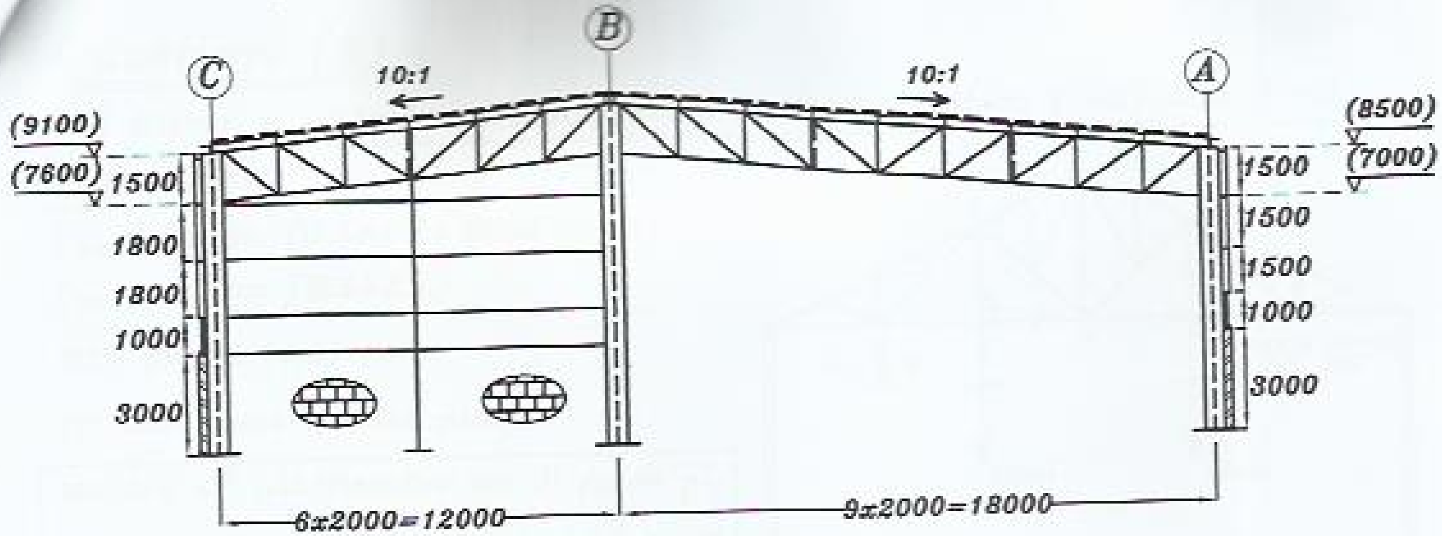


VERTICAL BRACING AT AXIS (C)

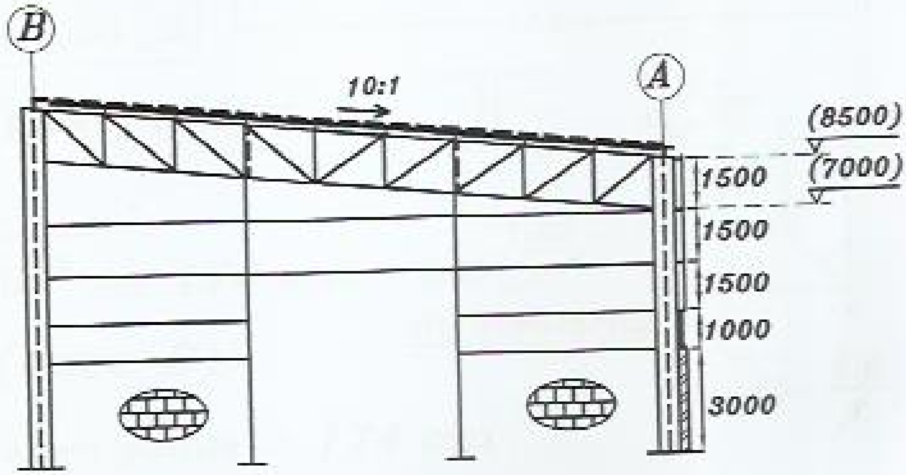


VERTICAL BRACING AT AXIS (A)

LONGITUDINAL BRACING



SECTION AT AXIS (6)



SECTION AT AXIS (1)

Question (2)

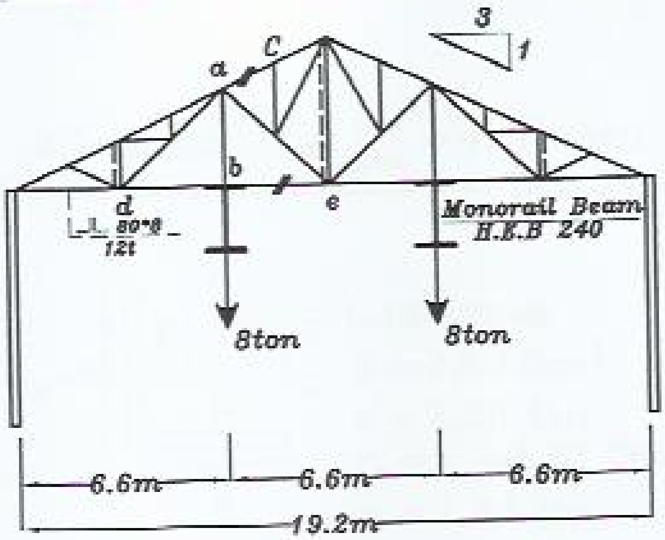
2.1) Design a suitable section for the marked members (a–C) and (b–e), given that

F_{ac} =−28ton (D.L+L.L+Wind Load)

F_{be} =+30ton (D.L+L.L)

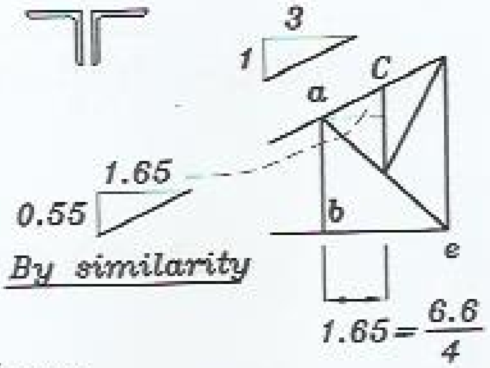
M20 grade (4.6)

10 mm thick gusset plate.



يتم تصميم الـ member ac على أنه welded
لان نهايتيه بعيدتان عن الـ filed conn.
يتم تصميم الـ member be على أنه bolted
لان احدى نهايتيه (e) filed conn.

Member (ac) ⇨ **Upper chord** ⇨



1) Data

* Length $= \sqrt{55^2 + 165^2} = 174$ cm
* Force $= -28$ ton (Case B)
* $l_{b\,in}$ = Distance between joints = 174 cm
* $l_{b\,out}$ = Distance between Purlins = 174 cm

2) Choice of section

From stresses

* assume $F_C = 0.75\ t\backslash cm^2$

$$\therefore A_{g\,\lrcorner\llcorner} = \frac{force}{F_C * 1.2} = \frac{28}{0.75 * 1.2} = 31.11\ cm^2$$

$$\therefore A_{g\,\llcorner} = \frac{A_{g\,\lrcorner\llcorner}}{2} = \frac{31.11}{2} = 15.55\ cm^2$$

Choose $\xrightarrow{tables}$ ∟100*100*10

From buckling

* assume $\lambda_{out} = \lambda_{in} = 100$

$$\therefore 100 = \frac{l_{b\,in}}{r_x} = \frac{174}{0.30 a_2}$$

$\Rightarrow a_2 =$ 5.80 cm

$$\therefore 100 = \frac{l_{b\,out}}{r_y} = \frac{174}{0.45 a_3}$$

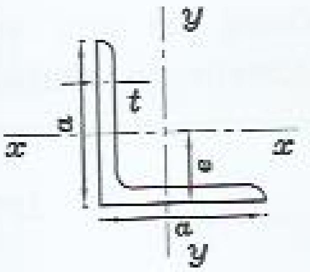
$\Rightarrow a_3 = 3.86$ cm

$$a_{av} = \frac{a_1 + (a_2 \overset{\text{الاكبر}}{Or} a_3)}{2} = \frac{10+5.80}{2} = 7.90\ cm$$

Choose ⅃L $80*80*8$

> minimum welded angle $45x45x5$ $\quad a_{min} = 4.5\ cm$

3) Checks



L $80*80*8$
$A = 12.3\ Cm^2$
$e = 2.26\ Cm$
$r_x = r_y = 2.42\ Cm$
$r_v = 1.55\ Cm$

$r_{x_{⅃L}} = r_{x_L}$ من الجدول $= 2.42\ cm$

assume $t_{cp} = 1\ cm$

$$r_{y_{⅃L}} = \sqrt{r_{yL}^2 + (e + \frac{t_{cp}}{2})^2} = \sqrt{2.42^2 + (2.26 + \frac{1.0}{2})^2} = 3.67\ cm$$

a) Buckling

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x_{⅃L}}} = \frac{174}{2.42} = 71.90 < 180 \Longrightarrow \text{(Safe)}$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{y_{⅃L}}} = \frac{174}{3.67} = 47.41 < 180 \Longrightarrow \text{(Safe)}$$

b) Stress

$$\lambda_{max.} = 71.09 \leqslant 100$$

$$* F_C = 1.2 * [1.4 - 6.5 * 10^{-5} \lambda^2_{max.}] = 1.2 * [1.4 - 6.5 * 10^{-5} (71.09)^2]$$

$$= 1.28\ t \backslash cm^2$$

$$* f_C = \text{actual stress} = \frac{force}{2 * A_{gL}} = \frac{28}{2 * 12.3} = 1.13\ t \backslash cm^2$$

$$\leqslant F_C \Longrightarrow \text{(Safe)}$$

$$* \frac{f_C}{F_C} = \frac{1.13}{1.28} = 0.89 \Longrightarrow \text{(Safe and economic)}$$

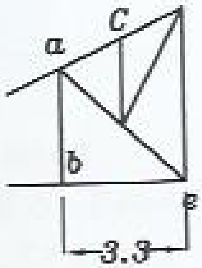
Design of tie plate

$\lambda_v \leq \lambda_{max.}$

$$\frac{l'}{r_{vL}} = \frac{l'}{1.55} \leq 71.90 \Rightarrow l' \leq 1.55 * 71.09 = 112 \text{ cm}$$

$l' \leq 112$ cm $> \frac{l}{2}$ ⇒ Use one tie plate at the middle of member

Member (be) ⇒ Lower chord ⇒ ⅃L



1) Data

* Length = 330 cm
* Force = + 30 ton (Case A)
* $l_{b\,in}$ = Distance between joints = 330 cm
* $l_{b\,out}$ = Distance between longitudinal bracing = 660 cm

2) Choice of section

a – From Stress Condition

$$A_{g_{⅃L}} = \frac{\text{Force ton}}{0.85 * F_t\ (t \backslash cm^2)} = \frac{30}{0.85*1.4} = 25.21\ cm^2$$

Rolted

$$A_{g_L} = \frac{25.21}{2} = 12.61\ cm^2$$

من الجدول ⇒ **Choose ⅃L 90 * 90 * 9** $a_1 = 9.0$ cm

b - From Slenderness Condition

assume $\lambda_{out} = \lambda_{in} = 300$

$$\therefore 300 = \frac{l_{b\,in}}{r_{x_{\rfloor\lfloor}}} = \frac{330}{0.30a} \Rightarrow a = 3.67\,cm$$

$$\therefore 300 = \frac{l_{b\,out}}{r_{y_{\rfloor\lfloor}}} = \frac{660}{0.45a} \Rightarrow a = 4.88\,cm$$

نأخذ الاكبر $a_2 = 4.88\,cm$

C – From Construction Condition Bolted

minimum angle $a_3 = 1.1 * 3\phi = 1.1 * 3 * 1.6 = 5.28\,cm$

ثم نختار من الجداول ال Angle الاكبر من a_1 & a_2 & a_3

Choose ⅃L 90*90*9

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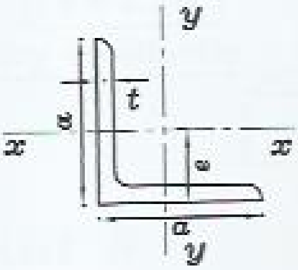
3) Checks



∟ 90*90*9

$A = 15.5\ Cm^2$

$e = 2.54\ Cm$

$r_x = r_y = 2.74\ Cm$

$$A_{net} = 2[A_{gross\,L} - (\phi + 0.2\,cm) * t_L]$$

$$= 2[15.50 - (2.0 + 0.2\,cm) * 0.9] = 27.04\,cm^2$$

a) Stress

$$* f_t = \frac{Force}{A_{net}} = \frac{30}{27.04} = 1.11\ t\backslash cm^2$$

مساحة ال angles التى تم حسابها

$$\leqslant F_t = 1.40\ t\backslash cm^2\ \text{(Safe)}$$

b) Slenderness

$r_{x_{\rfloor\lfloor}} = r_{x_L}$ من الجدول $= 2.74\ cm$

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x_{\rfloor\lfloor}}} = \frac{330}{2.74} = 120.44 < 300 \Rightarrow \text{(Safe)}$$

$$r_{y_{\rfloor\lfloor}} = \sqrt{r_{y_L}^2 + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.74^2 + (2.54 + \frac{1.0}{2})^2} = 4.09\ cm$$

* $\lambda_{out} = \frac{l_{b\,out}}{r_{y_{⌋⌊}}} = \frac{660}{4.09} = 161.37 < 300 \Longrightarrow$ (Safe)

C) Length to depth ratio (Deflection)

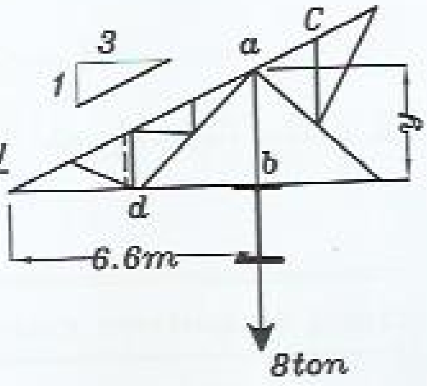
* $\frac{L}{d} = \frac{330\ cm}{a} = \frac{330\ cm}{9.0} = 36.67 \leqslant 60 \Longrightarrow$ (Safe)

2.2) Check the stresses and the suitability of using a single angle section 70x70x7 for member (a–b)

$$\frac{3}{1} = \frac{6.6}{y}$$

$$y = 2.20$$

By similarity

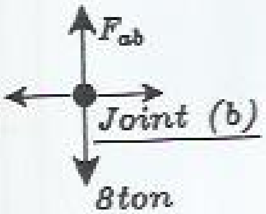


Member (AB)

1) Data

* Length = **220 cm**
* From equilibrium of joint B

  Force = **+8.0** ton (Case A)



* $l_{b\,in}$ = Distance between joints = **220 cm**
* $l_{b\,out}$ = Distance between joints = **220 cm**

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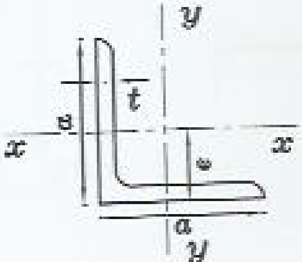
3) Checks

Check **∟ 70*70*7**



∟ 70*70*7

$A = 9.40\ Cm^2$

$r_v = 1.37\ Cm$

يتم عمل Check على member ab على أنه welded

لان نهايتيه بعيدتان عن الـ filed conn.

$A_{net} = A_{gross\,∟} = 9.40\ cm^2$

a) Stress

$$* \; f_t = \frac{Force}{A_{net}} = \frac{8.0}{9.40} = 0.85 \quad t \backslash cm^2$$

مساحة الـ angle التي تم حسابها

$$\leqslant F_t = 1.40 \; t \backslash cm^2 \quad \text{(Safe)}$$

b) Slenderness

r_{v_L} من الجدول $= 1.37 \; cm$

$$* \; \lambda_{out} = \frac{l_{b\,out}}{r_{v_L}} = \frac{220}{1.37} = 160.6 \quad < 300 \Longrightarrow \text{(Safe)}$$

C) Length to depth ratio (Deflection)

لا نحتاج الى عمل هذا الـ Check لان هذا الـ member هو Vertical member

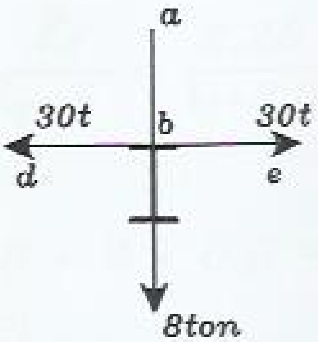
└ 70*70*7 is Safe

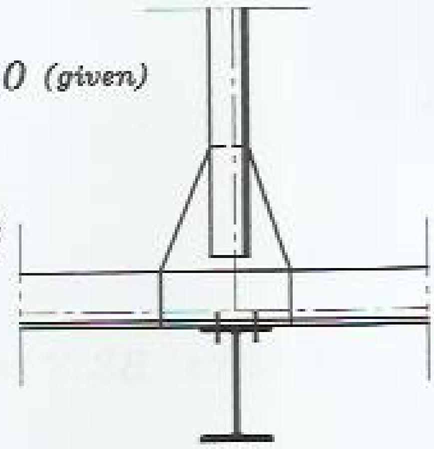
2.3) Design the welded connection of the truss members meeting at joint (b).

$\Sigma F_x = 0$

$F_{ba} = F_{be} = 30$ (given)

من اتزان (b) Joint نجد أن الـ Force
فى (be) member مساوية لـ Force
فى (bd) member و بالتالى عند تصميم
(b) Connection نصمم الـ Lower chord
على أنه Continuous member





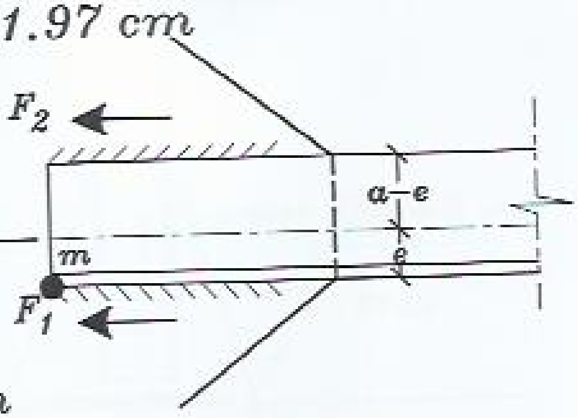
assume size of weld = **5 mm**

$l_{min} = 5 \; cm$

$l_{max} = 70\,S = 70 * 0.5 = 35 \; cm$

Member (ba) $\llcorner 70*70*7 \Rightarrow e = 1.97\ cm$

Force $= +8.0$ ton



$$* F_1 = F * \left(\frac{a-e}{a}\right)$$

$$= 8.0 * \left(\frac{7-1.97}{7}\right) = 5.74\ ton$$

$$* F_2 = F - F_1 = 8.0 - 5.74 = 2.26\ ton$$

Weld (1) Subjected to F_1

$$Stress = 0.72\ t\backslash cm^2 = \frac{F_1}{l_1 * S} = \frac{5.74}{l_1 * 0.5}$$

$$l_{1\,eff} = 15.94\ cm$$

$$* l_{1\,act.} = l_{1\,eff} + 2\,S = 15.94 + 2 * 0.5 = 16.94\ cm$$

$\Longrightarrow$ Take $\boxed{l_{1\,act.} = 17\ cm}$

Weld (2) Subjected to F_2

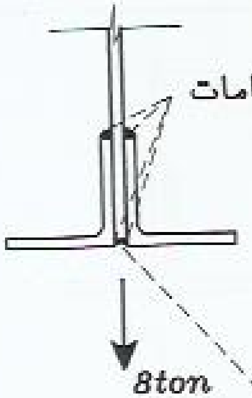
$$Stress = 0.72\ t\backslash cm^2 = \frac{F_2}{l_2 * S} = \frac{2.26}{l_2 * 0.5}$$

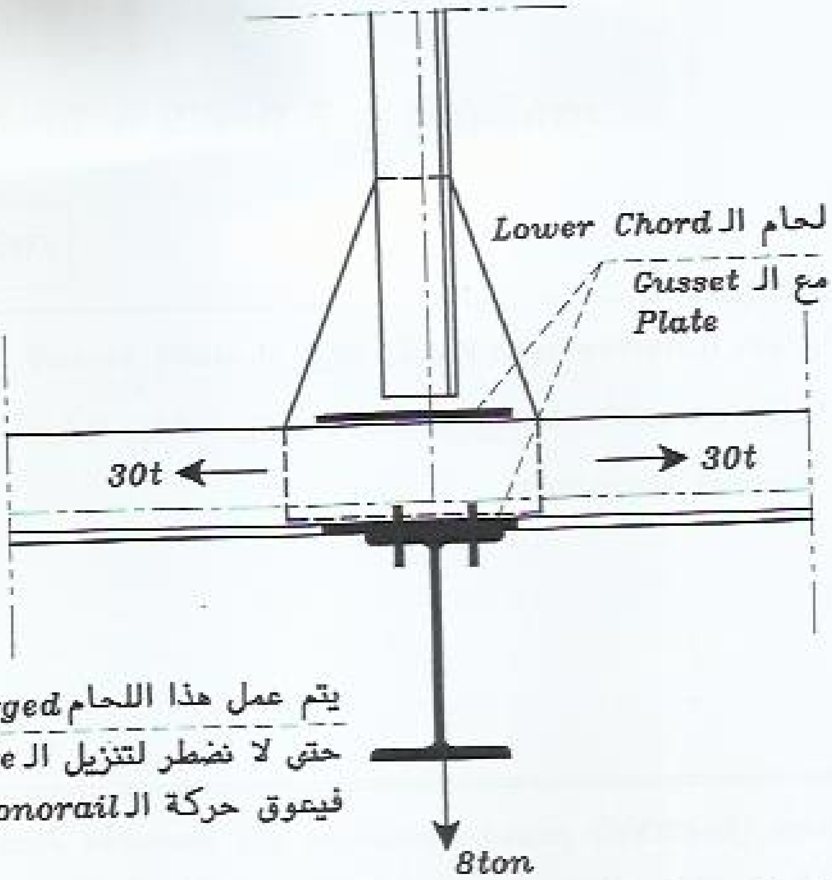
$$l_{2\,eff} = 6.28\ cm$$

$$* l_{2\,act.} = l_{2\,eff} + 2\,S = 6.28 + 2 * 0.5 = 7.28\ cm$$

$\Longrightarrow$ Take $\boxed{l_{2\,act.} = 8\ cm}$

Lower Chord





يتم عمل هذا اللحام Submerged
حتى لا نضطر لتنزيل الـ Plate لاسفل
فيعوق حركة الـ monorail

الهدف من لحام الـ Lower Chord مع الـ Gusset Plate هو نقل الـ Forces الموجودة فى الـ Lower Chord الى الـ Gusset Plate و هذه الـ Forces كالتالى :

١ - محصلة القوى فى الـ Lower Chord و هى $F_1 = 30 - 30 = 0$ و هذه القوة موازية للحام و بالتالى تسبب Shear على اللحام .

٢ - الـ monorail force حيث أنها تنتقل من الـ monorail beam الى الـ Lower Chord عن طريق المسامير ثم تنتقل من الـ Lower Chord الى الـ Gusset Plate عن طريق اللحام و هذه القوة عمودية على اللحام و بالتالى تسبب Tension على اللحام .

و حيث أن القوة الاولى التى تسبب الـ Shear على اللحام تساوى صفر يكون اللحام عليه Tension فقط و ينتقل لـ Gusset Plate عن طريق ٤ لحامات .

* assume size of weld = 5 mm

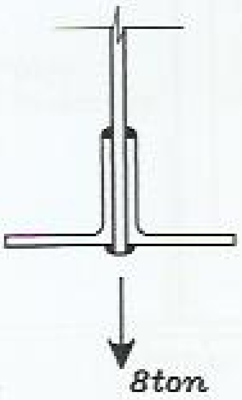
$$S_{max} = \frac{t_{G.p}}{2} = \frac{1}{2} = 0.5\ cm \Longrightarrow \text{For submerged weld}$$

$$Stress = 0.72\ t\backslash cm^2 = \frac{F}{l_1 * S * n} = \frac{8.0}{l_1 * 0.5 * 4}$$

$l_{1\,eff} = 5.55\ cm$

$* \ l_{1\,act.} = l_{1\,eff} + 2\,S = 5.55 + 2 * 0.5 = 6.55\ cm$

$\Longrightarrow$ Take $\boxed{l_{1\,act.} = 7\ cm}$



فى هذه الـ Connection لا يمكن تنزيل الـ Gusset Plate أسفل الـ Lower Chord للحامه لاننا لن نستطيع تركيب الـ monorail beam .

2.4) Design the bolted connection between the monorail beam (HEB240) and the lower chord of the truss at joint "b". Use pretensioned bolts M20 (8.8) ($As=2.45Cm^2$, $Ps=3.45t$, $T=10.8t$)

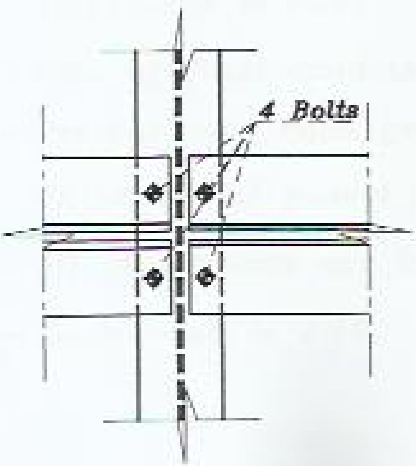
$T = 10.8\ t$

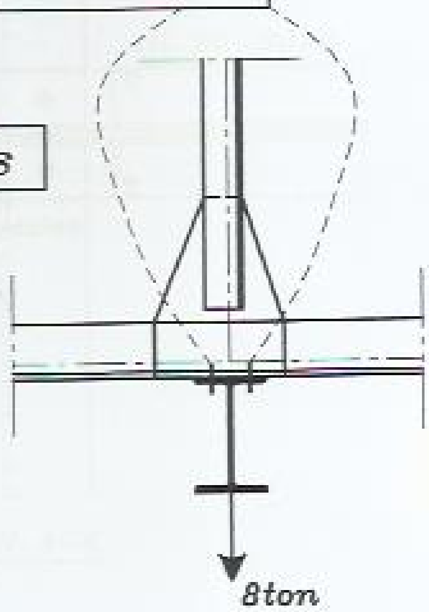
Bolts connecting upper flange of mono rail to lower chord of truss

$\Longrightarrow$ Subjected to $\Longrightarrow$ **Tension = 10t**

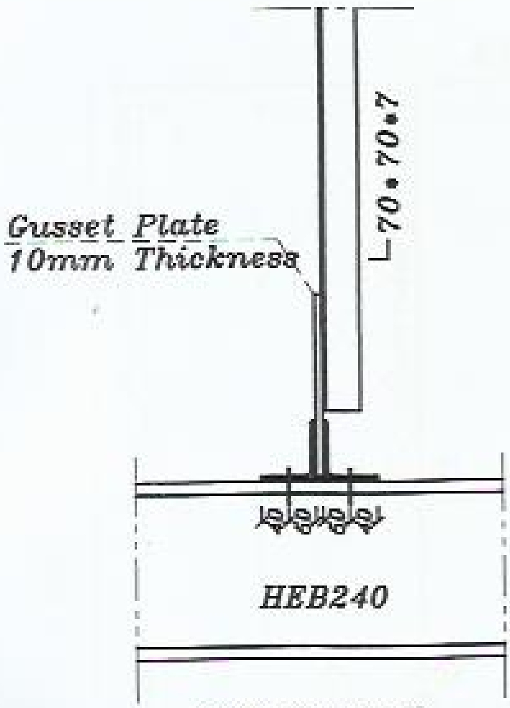
$* \ n = \dfrac{F}{0.6T} = \dfrac{8}{0.6 * 10.8} = 1.23 = \boxed{4\ Bolts}$

نأخذهم ٤ مسامير حتى يتم وضع مسمار مع كل زاوية .

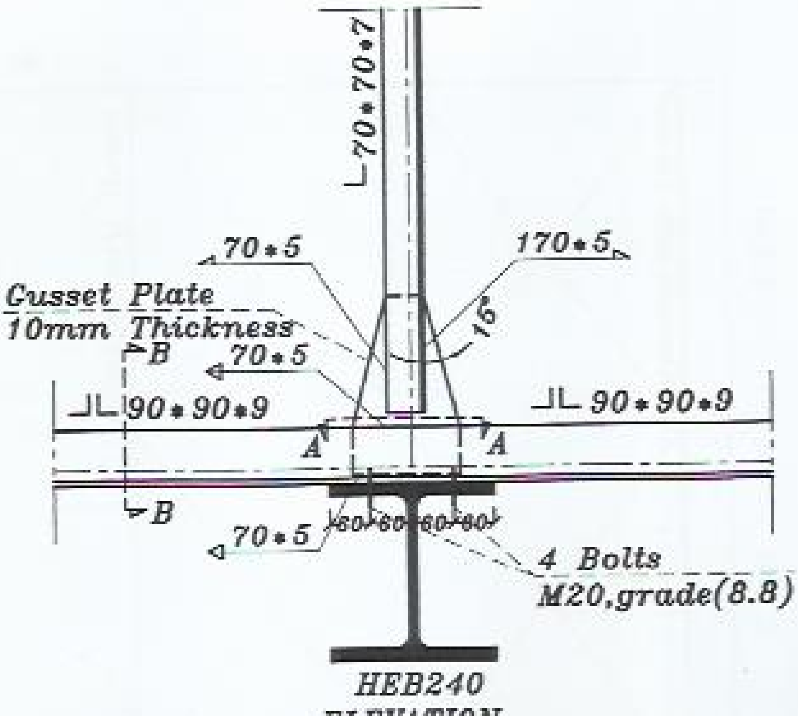




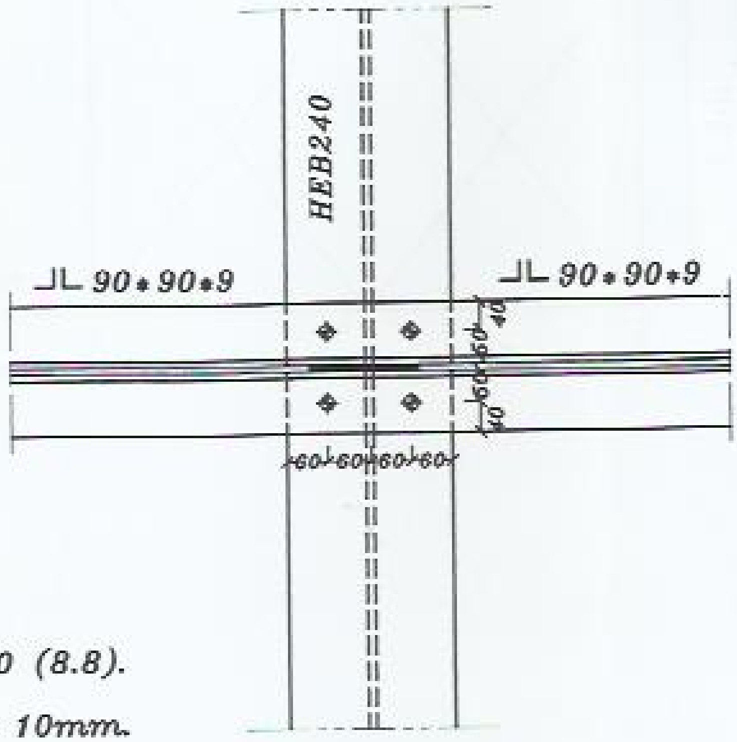
2.5) *Draw to scale 1:10 the connection at joint "b" in different views.*



SECTION B–B



ELEVATION



SECTION A–A

Notes

1- Steel used is st.37

2- Size of fillet weld is 5mm.

3- Pretensioned bolts grade M20 (8.8).

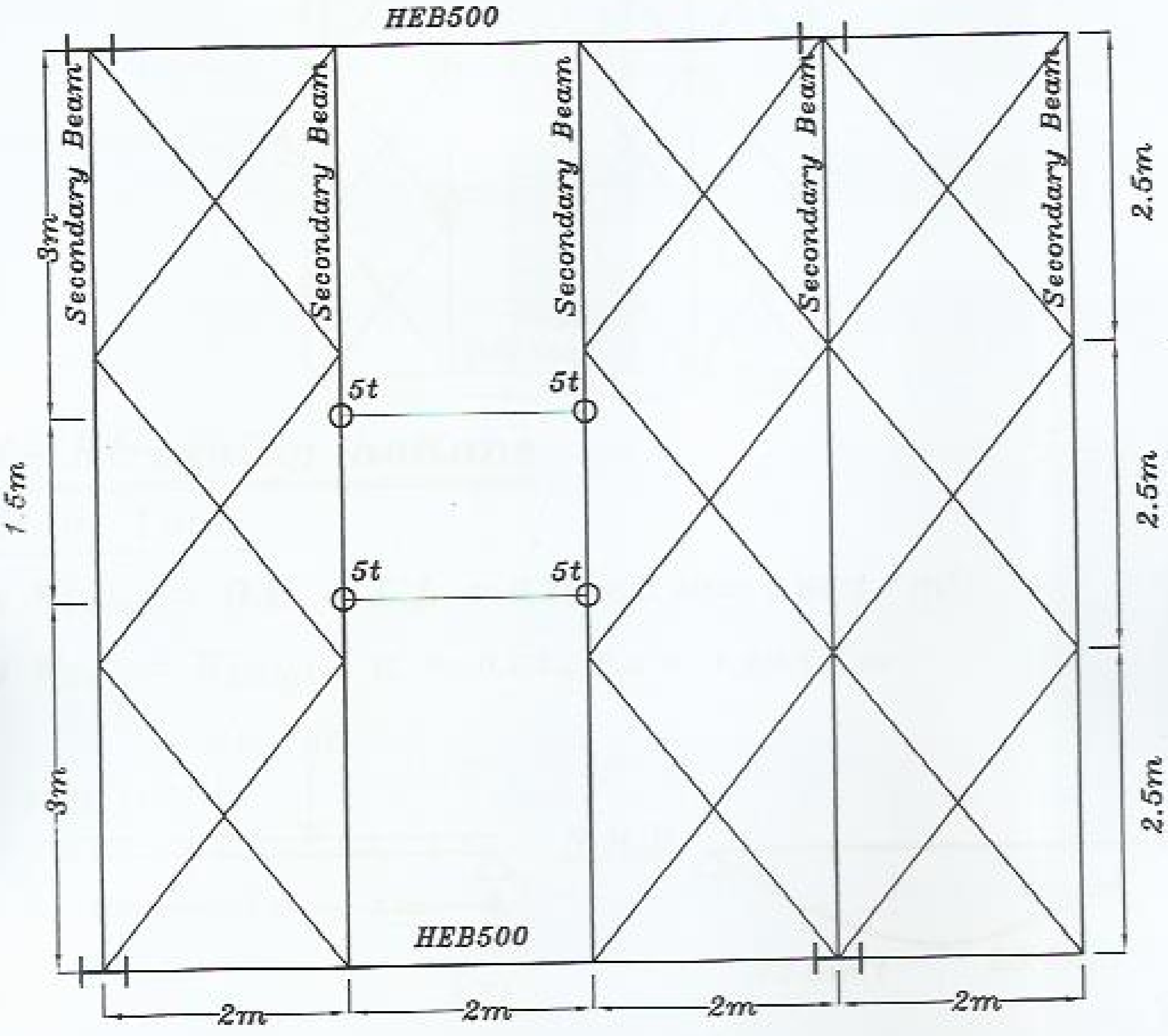
4- Thickness of gusset plate is 10mm.

5- All dimensions are in mms.

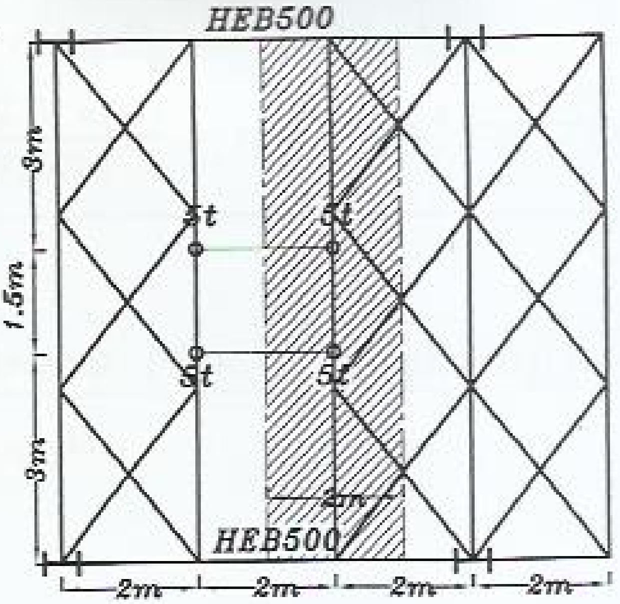
6- Scale used is 1:10.

Question (3)

(3.1) Suggest and draw to scale 1:50 a structural plan for the floor beams and the horizontal bracing.



(3.2) Design the critical secondary beam (Assume Cb=1).

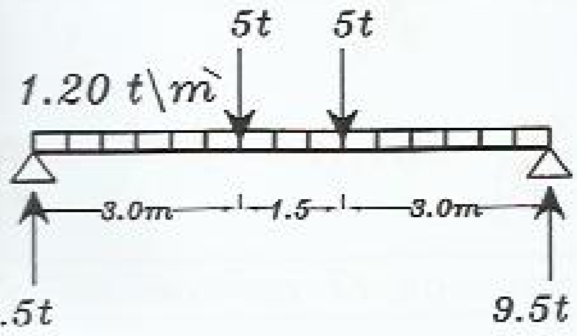


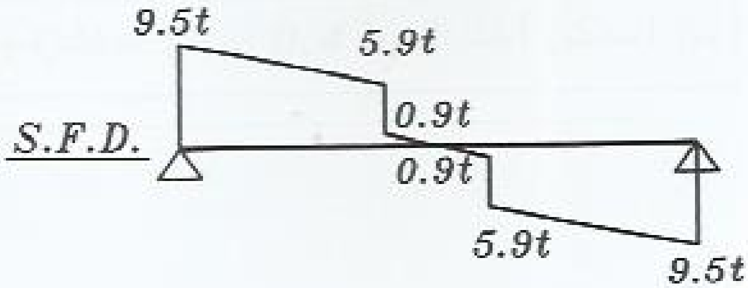
1- Straining actions

Total Load

\# W_{Total} = **D.L. + L.L** $= 0.20 + 0.40 = 0.60\ t/m^2$

\# $W_{Sec.} = W_{Total} * a = 0.60 * 2.0 = 1.20\ t/m$





\# $M_{Total} = 23.5\ m.t$

\# $Q_{Total} = 9.5\ t$

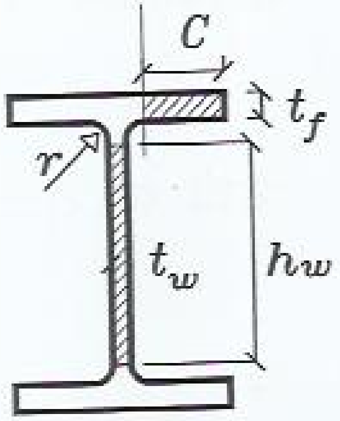
2- Choice of section

assume $F_b = 0.64\,F_y = 1.536\ t \backslash cm^2$

$$S_X = \frac{M_X}{1.536} = \frac{23.5 * 100}{1.536} = 1530\ Cm^3 \xrightarrow{Tables} \text{Use } \boxed{I.P.E\ 500}$$

3- Checks

3a-Compactness (Local buckling)



$h_w = 42.6\,cm$ جداول

$t_w = 1.02\,cm$

$b_f = 20\ cm$

$t_f = 1.60\,cm$

$r = 2.1\,cm$

$$\frac{C}{t_f} = \frac{\frac{1}{2}(b_f - t_w - 2r)}{t_f} = \frac{\frac{1}{2}(20 - 1.02 - 2 * 2.1)}{1.60} = 4.62$$

$$\therefore \frac{C}{t_f} = 4.62 < \frac{16.9}{\sqrt{f_y}} = 10.9 \Longrightarrow \text{Compact Flange}$$

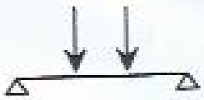
$$\frac{h_w}{t_w} = \frac{42.6}{1.02} = 41.76 < \frac{127}{\sqrt{f_y}} = 82 \Longrightarrow \text{Compact Web}$$

$\therefore$ The section is compact

ملحوظة

من الممكن أخذ ال $C = 0.4\,b_f$ مباشرة بدلا من حسابها

b-Lateral Torsional Buckling



$C_b = 1.0$ (given)

$L_{U_{act.}} = 250$ cm (braced at locations of hz. bracing)

$$L_{U_{max.}} = \begin{cases} \dfrac{20\, b_f}{\sqrt{f_y}} = \dfrac{20 * 20}{\sqrt{2.4}} = 258.2 \text{ cm} \\[2ex] \dfrac{1380\, A_f}{d * F_y} C_b = \dfrac{1380 * (20 * 1.60)}{50 * 2.4} * 1.0 = 368 \text{ cm} \end{cases}$$

نأخذ الاصغر

$$\boxed{L_{U_{max.}} = 258.2 \text{ cm}}$$

$L_{U_{act.}} = 250$ cm $< L_{U_{max.}} = 258.2$ cm $\Longrightarrow$ NO LTB

Compact Section $\Longrightarrow F_b = \boxed{1.536\ t\backslash cm^2}$

3a-Check bending stresses

$$f_{act.} = \frac{M_X}{S_X} = \frac{23.5 * 100}{1930} = 1.22\ t\backslash cm^2 < F_{bC} = 1.536$$

(Safe)

(1930) من الجداول

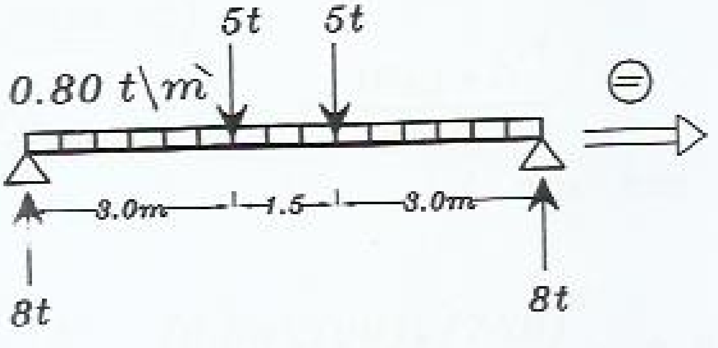
3b-Check Shear stresses

$$q_{act.} = \frac{Q}{A_{Web}} = \frac{Q}{h * t_w} = \frac{9.50}{50 * 1.02} = 0.19\ t\backslash cm^2 < 0.35\ F_y = 0.84$$

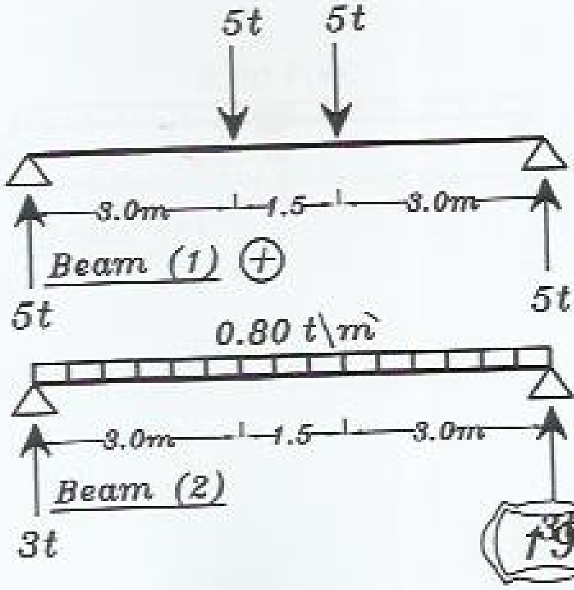
(Safe)

3C-Check deflection

$W_{Sec.} = W_{L.L.} * a = 0.40 * 2.0 = 0.80\ t/m$



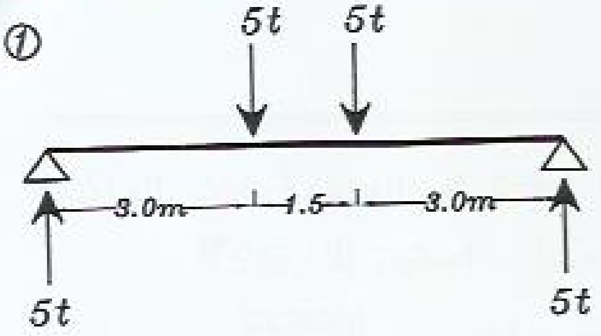
Live Load نعتبر حمل الماكينة كله



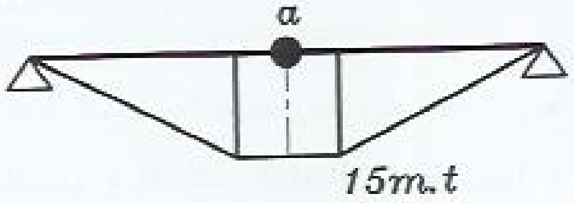
للتسهيل قسمنا الكمرة الى كمرتين و نحسب الـ *deflection* لكل كمرة من الاثنان ثم نجمعهم .

Beam (1)

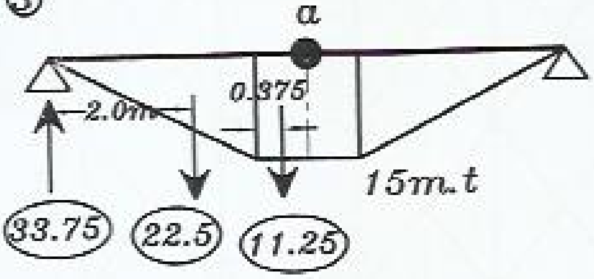
①



②



③



Elastic loads & Elastic reactions

④ نحسب الـ*Elastic moment* فى منتصف الكمرة و نقسمه على *EI* فيكون هو الـ *deflection* .

$$M_{Elastic} = 33.75*3.75 - 22.5*1.75 - 11.25*0.375 = 86.72\ m^3t$$

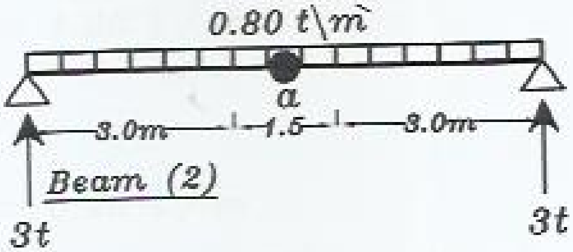
$$\triangle_{act._1} = \frac{157.1}{EI} = \frac{86.72*10^6}{2100*48200} = 0.86\,cm$$

(m^3t ; $t\backslash cm^2$; cm^4)

Beam (2)

$$\triangle_{act._2} = \frac{5}{384} * \frac{w_{L.L} * S^4}{E * I_X}$$

($t\backslash cm$; cm)



$$= \frac{5}{384} * \frac{(0.80\backslash 100)*(750)^4}{2100*48200} = 0.33\,cm$$

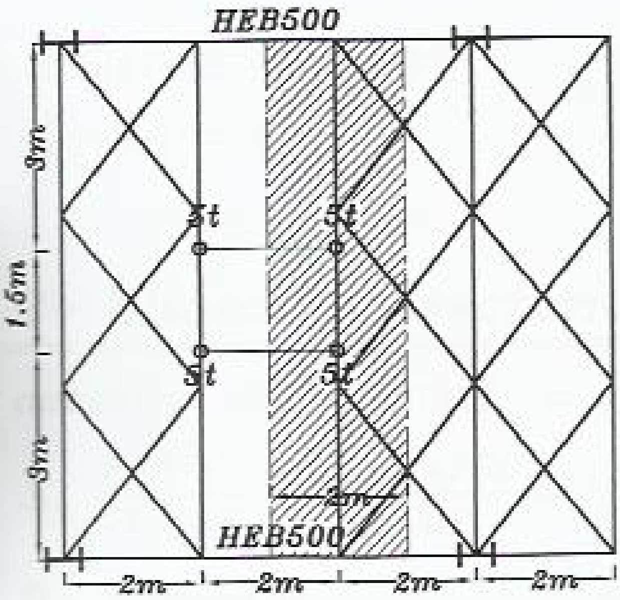
($t\backslash cm^2$; cm^4)

$$\triangle_{Total} = \triangle_{act._1} + \triangle_{act._2}$$

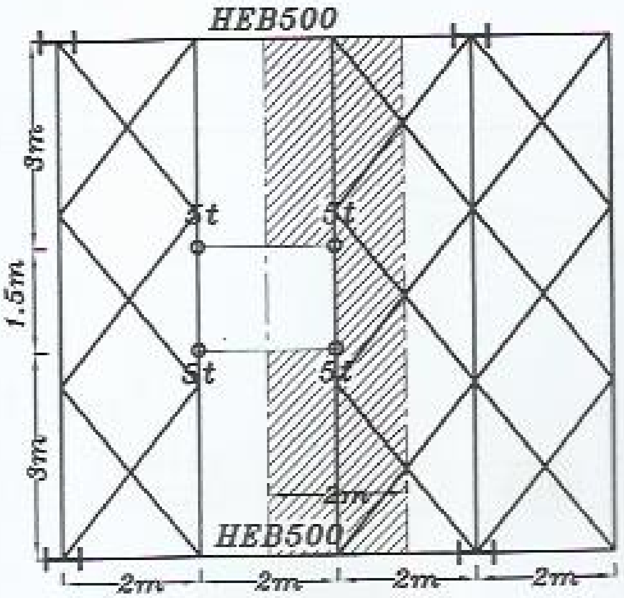
$$= 0.86\,cm + 0.33\,cm = 1.19\,cm < \frac{Span}{300}$$

$$< \frac{750}{300} = 2.50\,cm \quad (Safe)$$

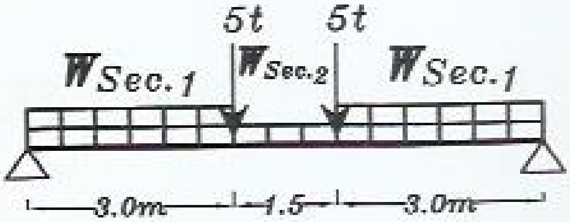
من المفترض فى هذه المسألة أنه لايوجد *Live load* فى المنطقة تحت الماكينة و لكن يوجد *Dead Load* و بالتالى الاصح هو حساب قيمتين للـ $W_{Sec.}$



D.L.



L.L.



\# $W_{Total_1} = D.L. + L.L = 0.20 + 0.40 = 0.60\ t/m^2$

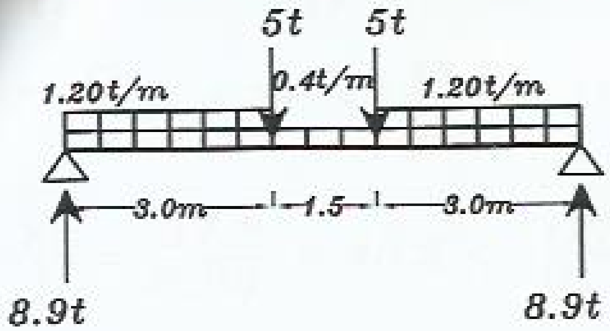
\# $W_{Sec._1} = W_{Total_1} * a = 0.60 * 2.0 = 1.20\ t/m$

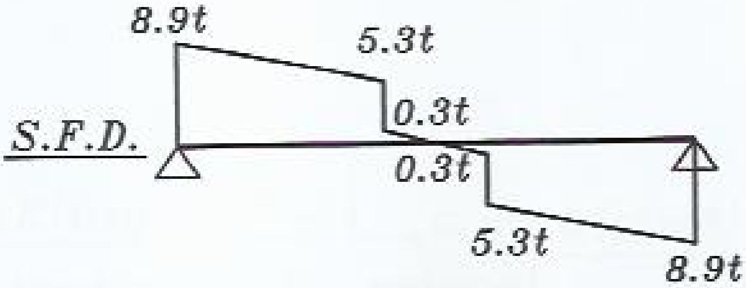
\# $W_{Total_2} = D.L. = 0.20\ t/m^2$

\# $W_{Sec._2} = W_{Total_2} * a = 0.20 * 2.0 = 0.40\ t/m$

و يكون التصميم كالتالى

1- Straining actions





\# $M_{Total} = 21.4$ m.t

\# $Q_{Total} = 8.9$ t

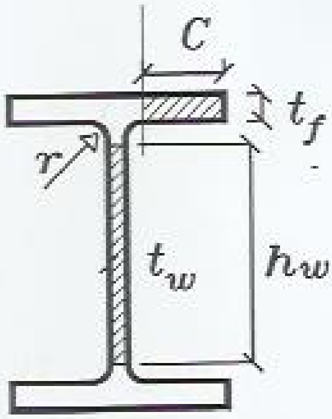
2- Choice of section

assume $F_b = 0.64\, F_y = 1.536\ t \backslash cm^2$

$$S_X = \frac{M_X}{1.536} = \frac{21.4 * 100}{1.536} = 1397\ Cm^3 \xrightarrow{Tables} \text{Use } \boxed{I.P.E\ \ 450}$$

3- Checks

3a-Compactness (Local buckling)



$h_w = 37.8\,cm$ جداول

$t_w = 0.94\,cm$

$b_f = 19\ cm$

$t_f = 1.46\,cm$

$r = 2.1\,cm$

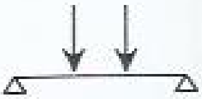
$$\frac{C}{t_f} = \frac{\frac{1}{2}(b_f - t_w - 2r)}{t_f} = \frac{\frac{1}{2}(19 - 0.94 - 2*2.1)}{1.46} = 4.62$$

$$\therefore \frac{C}{t_f} = 4.74 < \frac{16.9}{\sqrt{f_y}} = 10.9 \Longrightarrow \text{Compact Flange}$$

$$\frac{hw}{t_w} = \frac{37.8}{0.94} = 40.2 < \frac{127}{\sqrt{f_y}} = 82 \Longrightarrow \text{Compact Web}$$

∴ The section is compact

b-Lateral Torsional Buckling



$C_b = 1.0$ (given)

$L_{U_{act.}} = 250$ cm (braced at locations of hz. bracing)

$$L_{U_{max.}} = \begin{cases} \frac{20\, b_f}{\sqrt{f_y}} = \frac{20*19}{\sqrt{2.4}} = 246 \text{ cm} \\ \frac{1380 A_f}{d*F_y} C_b = \frac{1380*(19*1.46)}{45*2.4} *1.0 = 354 \text{ cm} \end{cases}$$

نأخذ الاصغر

$$\boxed{L_{U_{max.}} = 246.0 \text{ cm}}$$

$$L_{U_{act.}} = 250 \text{ cm} > L_{U_{max.}} = 246.0 \text{ cm} \Longrightarrow \text{LTB occurs}$$

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

$$F_{ltb1} = \frac{800*A_f}{l_u*d} C_b = \frac{800*(19*1.46)}{250*45} *1.0 = 1.97 \ t\backslash cm^2$$

$$> 0.58\, F_y = 1.4$$

$$\Longrightarrow F_b = \boxed{1.40\, t\backslash cm^2}$$

3a-Check bending stresses

$$f_{act.} = \frac{M_X}{S_X} = \frac{21.4 * 100}{1500} = 1.42\ t\backslash cm^2 > F_{bC} = 1.40$$

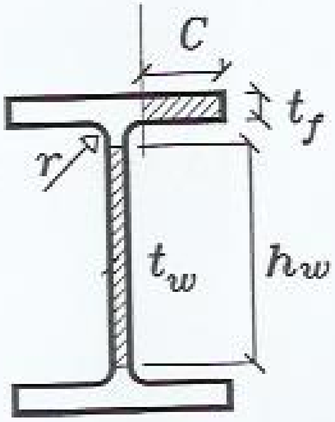
(Unsafe)

من الجداول

⟹ Use **I.P.E 500**

3- Checks

3a-Compactness (Local buckling)



$h_w = 42.6\,cm$ جداول

$t_w = 1.02\,cm$

$b_f = 20\ cm$

$t_f = 1.60\,cm$

$r = 2.1\,cm$

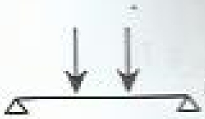
$$\frac{C}{t_f} = \frac{\frac{1}{2}(b_f - t_w - 2r)}{t_f} = \frac{\frac{1}{2}(20 - 1.02 - 2 * 2.1)}{1.60} = 4.62$$

$$\therefore \frac{C}{t_f} = 4.62 < \frac{16.9}{\sqrt{f_y}} = 10.9 \Longrightarrow \text{Compact Flange}$$

$$\frac{h_w}{t_w} = \frac{42.6}{1.02} = 41.76 < \frac{127}{\sqrt{f_y}} = 82 \Longrightarrow \text{Compact Web}$$

∴ The section is compact

b-Lateral Torsional Buckling



$C_b = 1.0$ (given)

$L_{U\,act.} = 250$ cm (braced at locations of hz. bracing)

$$L_{U\,max.} = \begin{cases} \dfrac{20\,b_f}{\sqrt{f_y}} = \dfrac{20*20}{\sqrt{2.4}} = 258.2\text{ cm} \\ \dfrac{1380\,A_f}{d*F_y}C_b = \dfrac{1380*(20*1.60)}{50*2.4}*1.0 = 368\text{ cm} \end{cases}$$

نأخذ الاصغر

$$\boxed{L_{U\,max.} = 258.2\text{ cm}}$$

$L_{U\,act.} = 250$ cm $< L_{U\,max.} = 258.2$ cm $\Longrightarrow$ NO LTB

Compact Section $\Longrightarrow F_b = \boxed{1.536\ t\backslash cm^2}$

3a-Check bending stresses

$$f_{act.} = \frac{M_X}{S_X} = \frac{21.4\ *100}{1930} = 1.10\ t\backslash cm^2 < F_{bC} = 1.536$$

من الجداول (1930)

(Safe)

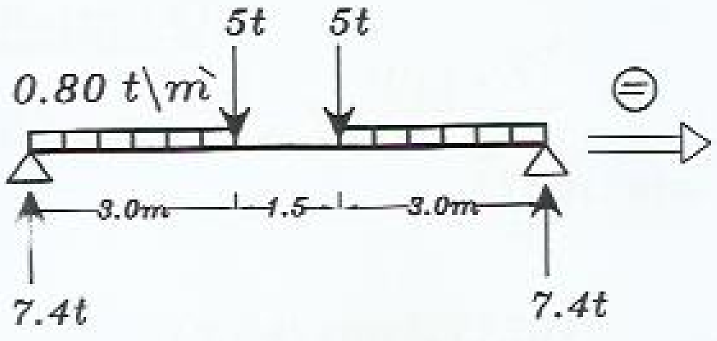
3b-Check Shear stresses

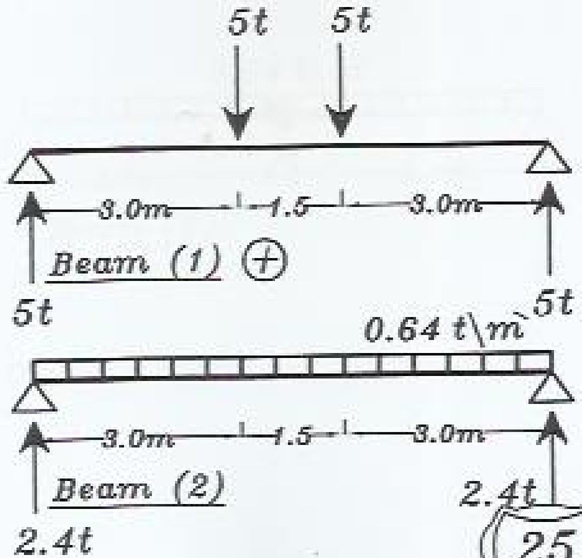
$$q_{act.} = \frac{Q}{A_{Web}} = \frac{Q}{h*t_w} = \frac{8.90}{50*1.02} = 0.17\ t\backslash cm^2 < 0.35\ F_y = 0.84$$

(Safe)

3C-Check deflection

\# $W_{Sec.} = W_{L.L.} * a = 0.40 * 2.0 = 0.80\ t/m$





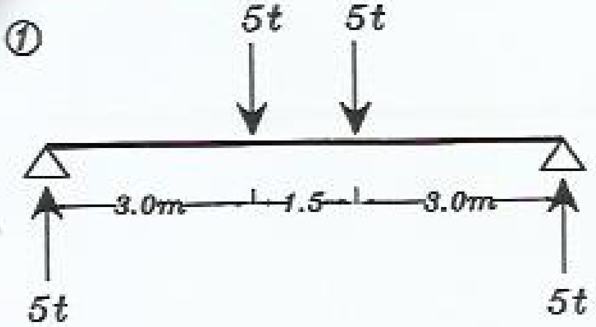
نعتبر حمل الماكينة كله Live Load

ونسيح الحمل الحي على الكمرة كلها للتسهيل

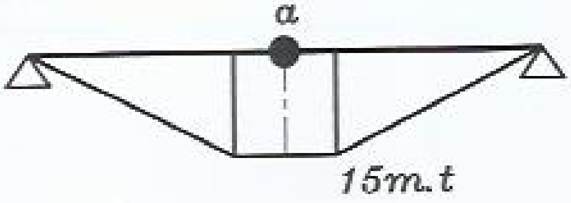
للتسهيل قسمنا الكمرة الى كمرتين و نحسب الـ *deflection* لكل كمرة من الاثنان ثم نجمعهم .

Beam (1)

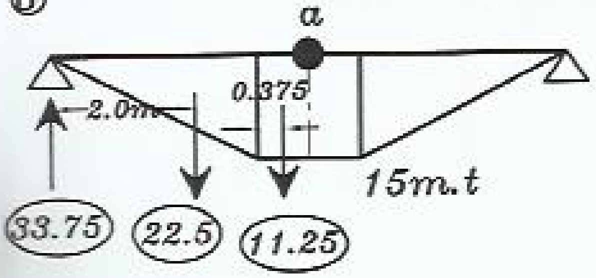
①



②



③



Elastic loads & Elastic reactions

④ نحسب الـ *Elastic moment* في منتصف الكمرة و نقسمه على *EI* فيكون هو الـ *deflection* .

$$M_{Elastic} = 33.75 * 3.75 - 22.5 * 1.75 - 11.25 * 0.375 = 86.72\ m^3t$$

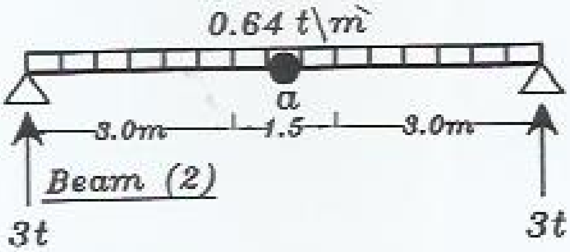
$$\Delta_{act._1} = \frac{157.1}{EI} = \frac{86.72 * 10^6}{2100 * 48200} = 0.86\ cm$$

($86.72 * 10^6$: m^3t; 2100: $t\backslash cm^2$; 48200: cm^4)

Beam (2)

$$\Delta_{act._2} = \frac{5}{384} * \frac{w_{L.L} * S^4}{E * I_X}$$

($w_{L.L}$: $t\backslash cm$; S: cm)



$$= \frac{5}{384} * \frac{(0.64\backslash 100) * (750)^4}{2100 * 48200} = 0.26\ cm$$

(2100: $t\backslash cm^2$; 48200: cm^4)

$$\triangle_{Total} = \triangle_{act._1} + \triangle_{act._2}$$

$$= 0.86\ cm + 0.26\ cm = 1.12\ cm < \frac{Span}{300}$$

$$< \frac{750}{300} = 2.50\ cm \quad (Safe)$$

AIN SHAMS UNIVERSITY | مهم جدا | FACULTY OF ENGINEERING

STRUCTURAL ENGINEERING DEPARTEMENT

THIRD YEAR CIVIL — STRUCTURAL DIVISION

January 2010 — STEEL STRUCTURES — TIME:3.00 Hrs

Material of construction is steel 37.

Question (1) (14Marks)

An exhibition hall is to be constructed over the area shown in figure (1). It is required to use steel trusses to cover the hall. The clear height of the used trusses is 7.00m. An opening is required in roof at the inner solid lines. Columns are allowed along solid lines only.

It is required to :

1) Draw to scale 1:200 a complete general layout for the building, showing all necessary views and bracing systems.

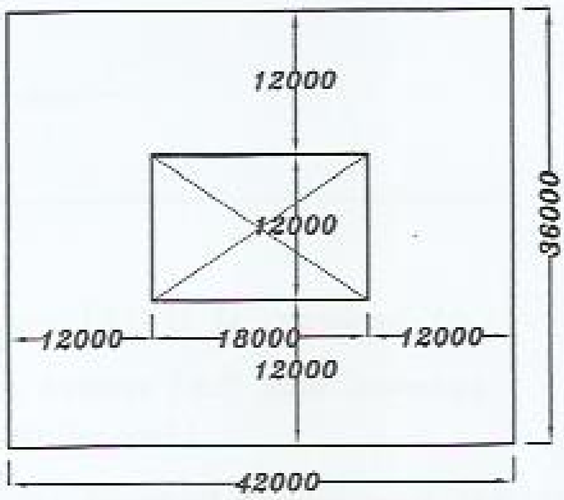


Fig.(1)

2) Write brief notes on the following :

a- How the wind forces are transmitted to the foundations .

b) Function of end gable columns & their ideal spacing.

Question (2) (38Marks)

For the part of typical truss, shown in figure (2), it is required to :

مؤجل للترم الثانى

1) Design an intermediate roof purlin, using hot-rolled channel section (Spacing between trusses=7.50m, weight of roof covering= 8 kg/m²)

2) Design the truss members, intersecting at the marked joint. Use back to back channels for chord members and back to back angles for the web members (Use H.S.B M20 (8.8) with threads excluded from the planes of shear and 15mm thick gusset plates).

3) Design the non-pretensioned bearing type bolted connection for the marked joint, considering the monorail reaction.

4) Draw to scale 1:10, the designed connection in different views.

Note Monorail capacity = 6.00t

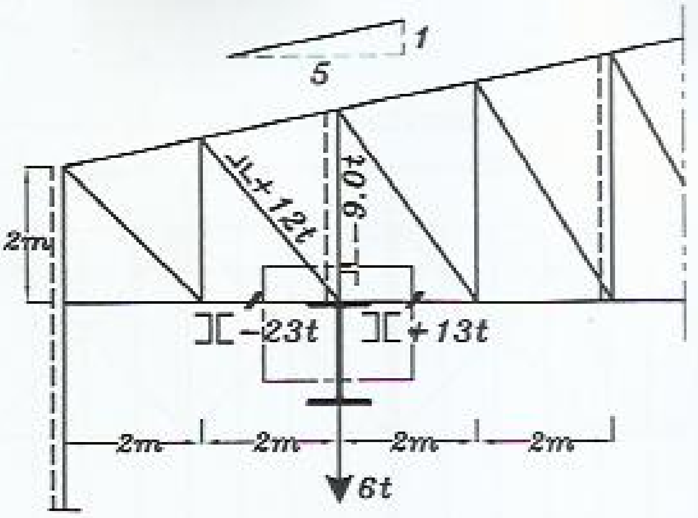


Fig.(2)

Question (3) (23Marks)

For the shown plan of floor beams in figure (3), it is required to :

1) Design the marked secondary and main beams (R.C slab average thickness = 12cm, F.C= 150kg/m^2 & L.L= 400kg/m^2)

2) Design the marked connection between the marked beams and the column using slip-critical connection and M16 bolts (grade 10.9 with As=1.57 Cm^2, T=9.89 ton and Ps=3.16 ton) الدكتور ذكر أنها لن تأتي بالامتحان

3) Draw to scale 1:10, the marked connection in different views.

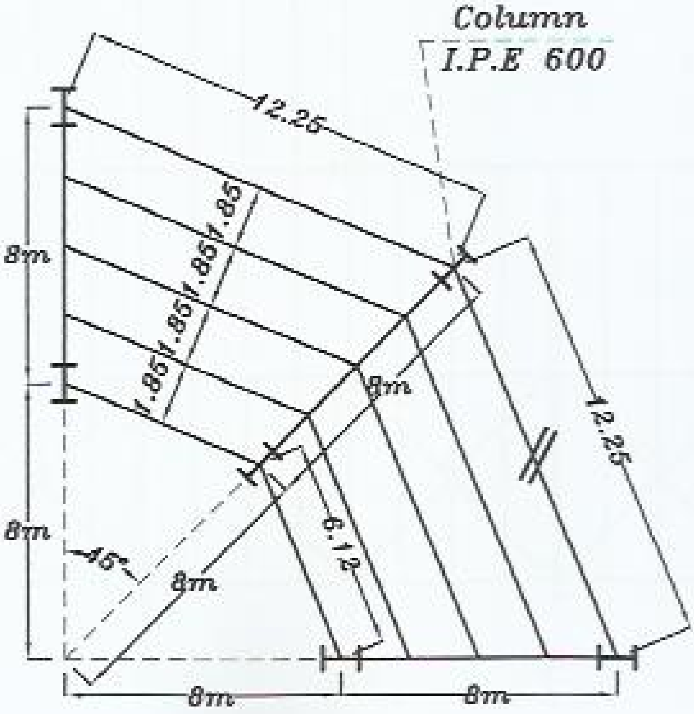
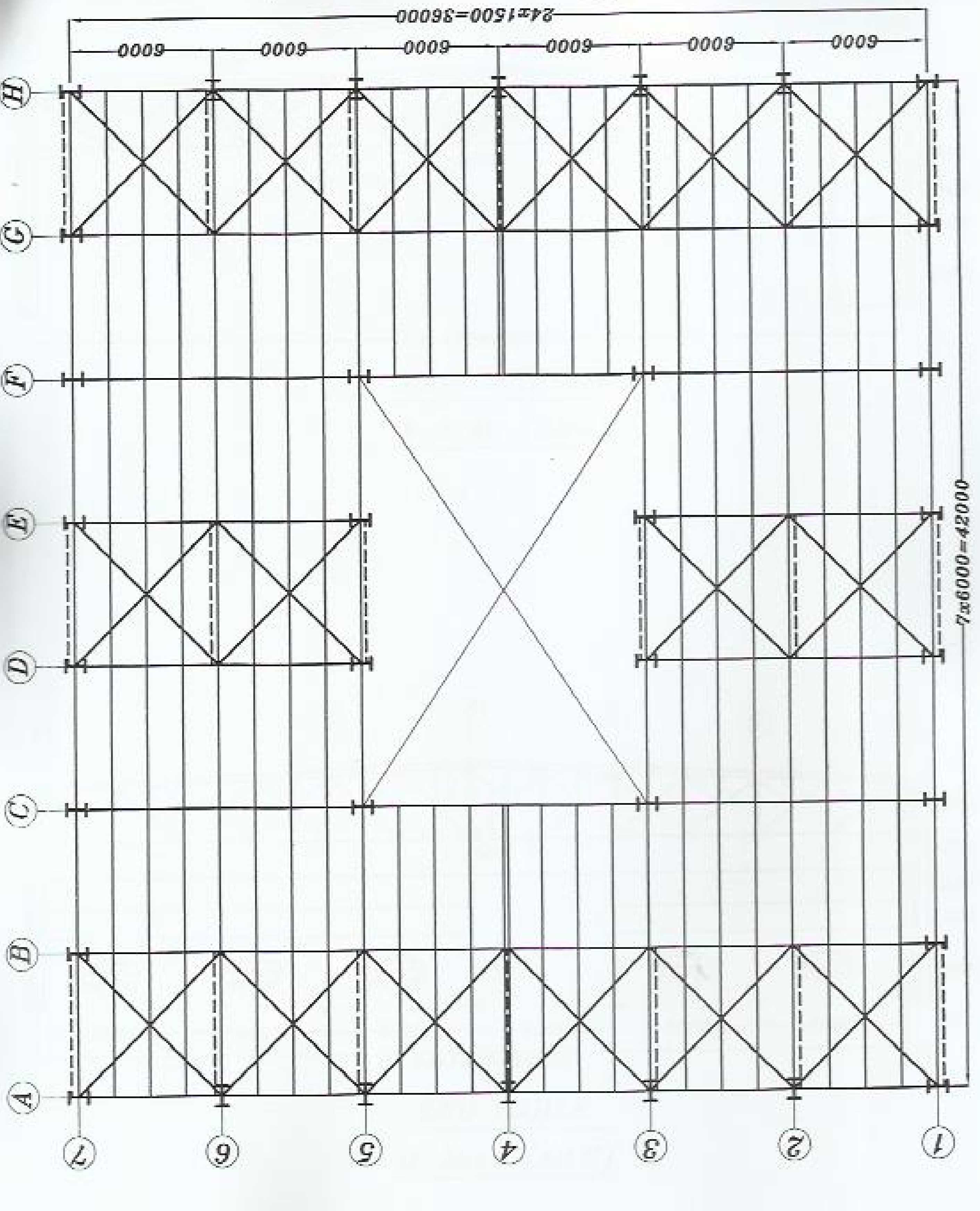
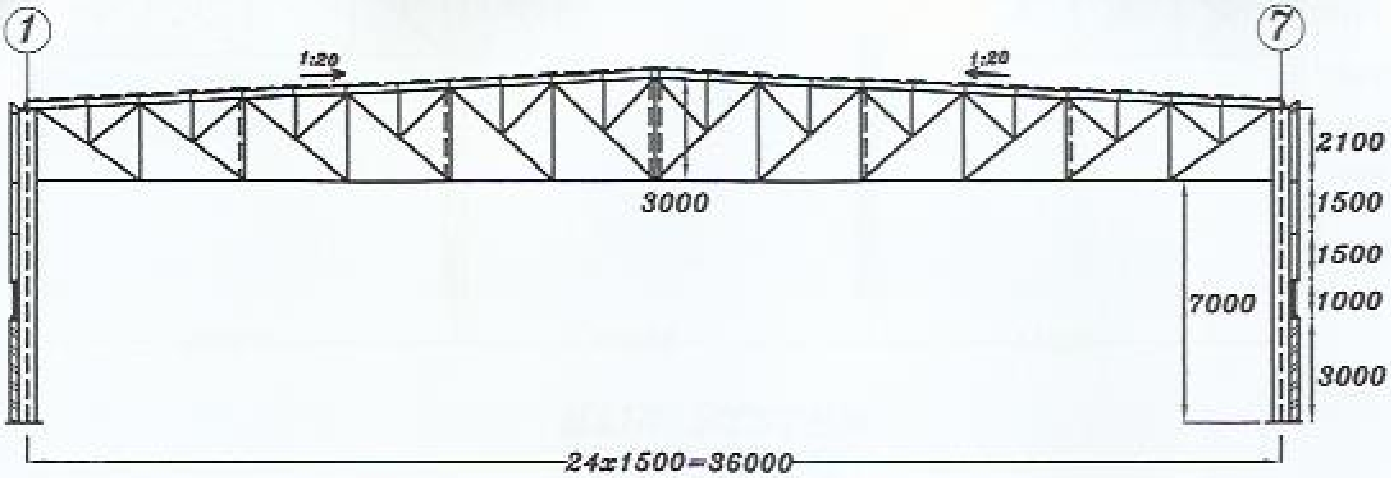


Fig.(3)

Question (1)

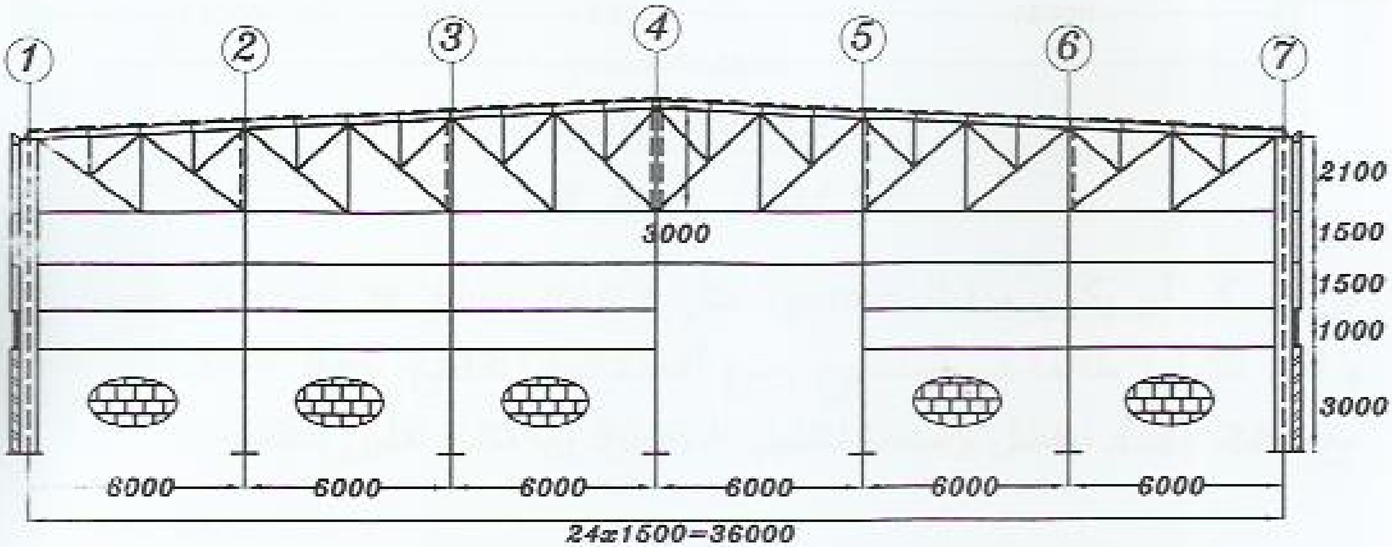
PLAN





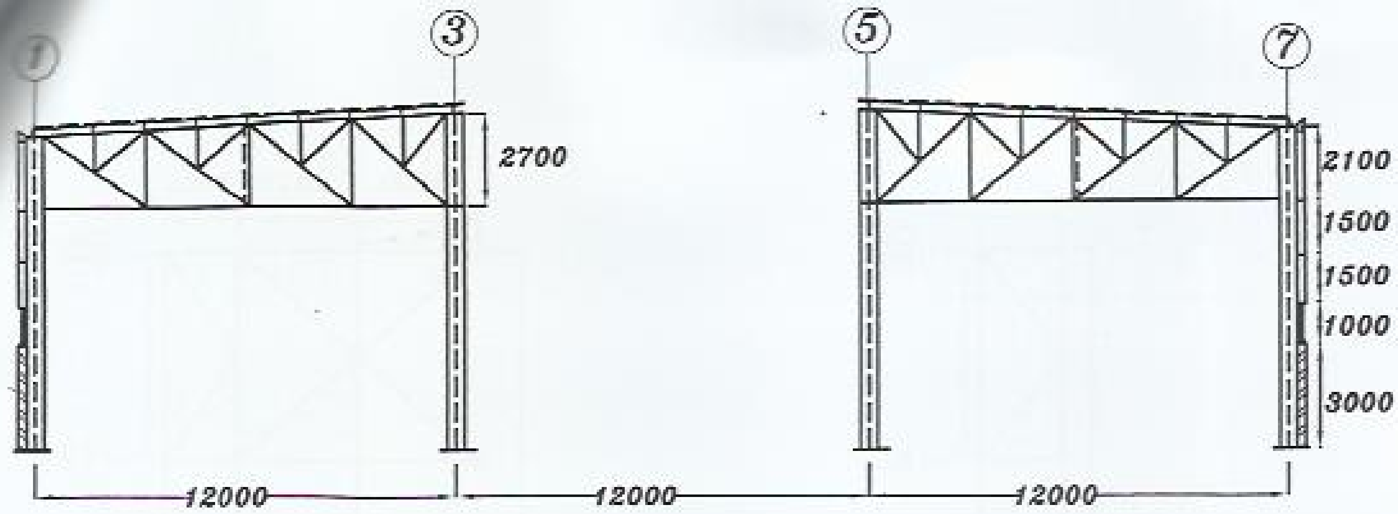
MAIN SYSTEM

❷ Axes (B&G)



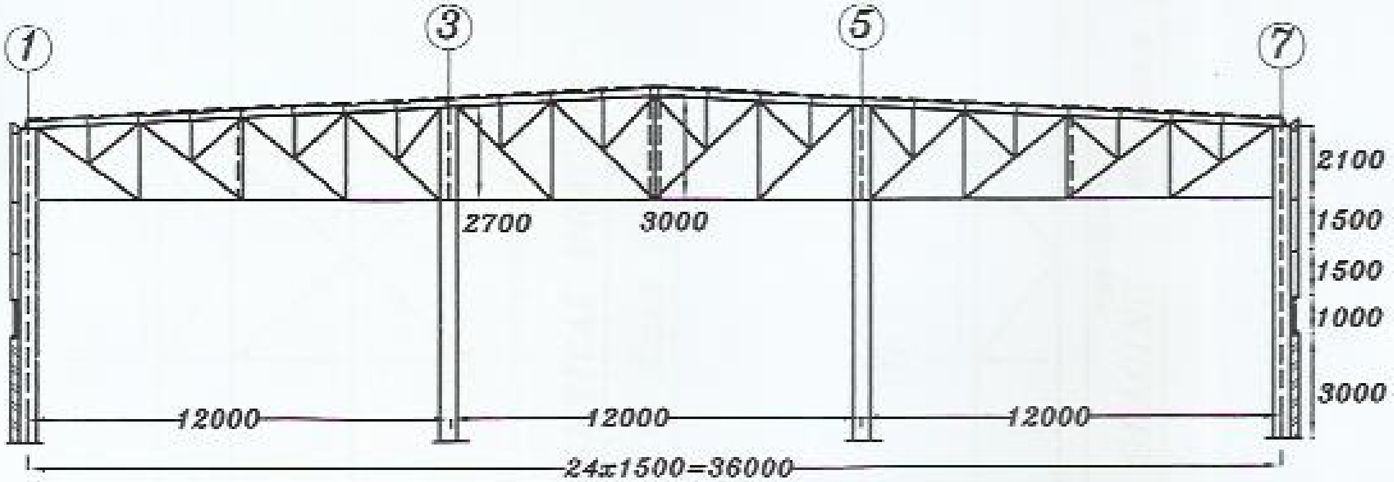
END GABLE

❶ Axes (A&H)



MAIN SYSTEM

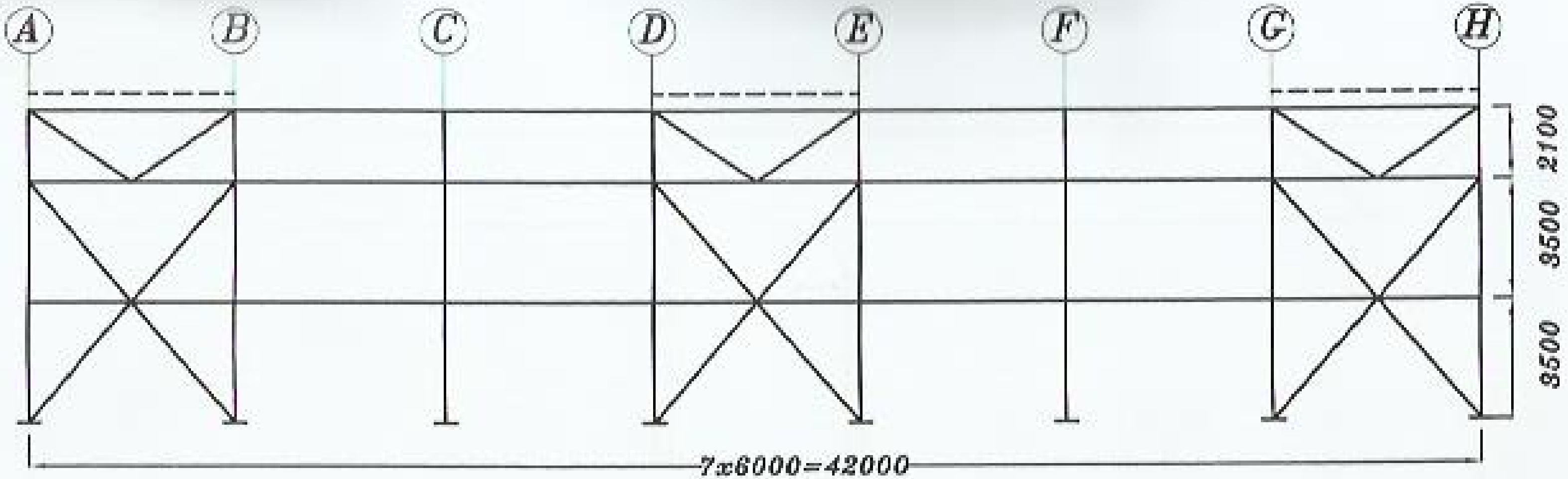
❶ Axes (D&E)



MAIN SYSTEM

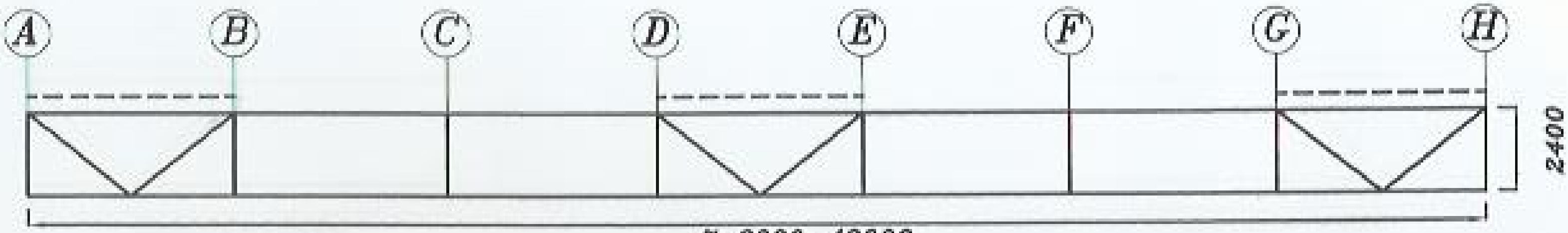
❷ Axes (C&F)

من الممكن أن يكون هذا الـ *System* مثل الـ *Main System @ Axes B&G*
و لكن للتوفير طالما من المسموح وضع أعمدة عند تقاطع *Axes B&G*
مع *Axes 3&5* فيفضل وضعها لتقليل الـ *Span* وبالتالى تقليل التكلفة .

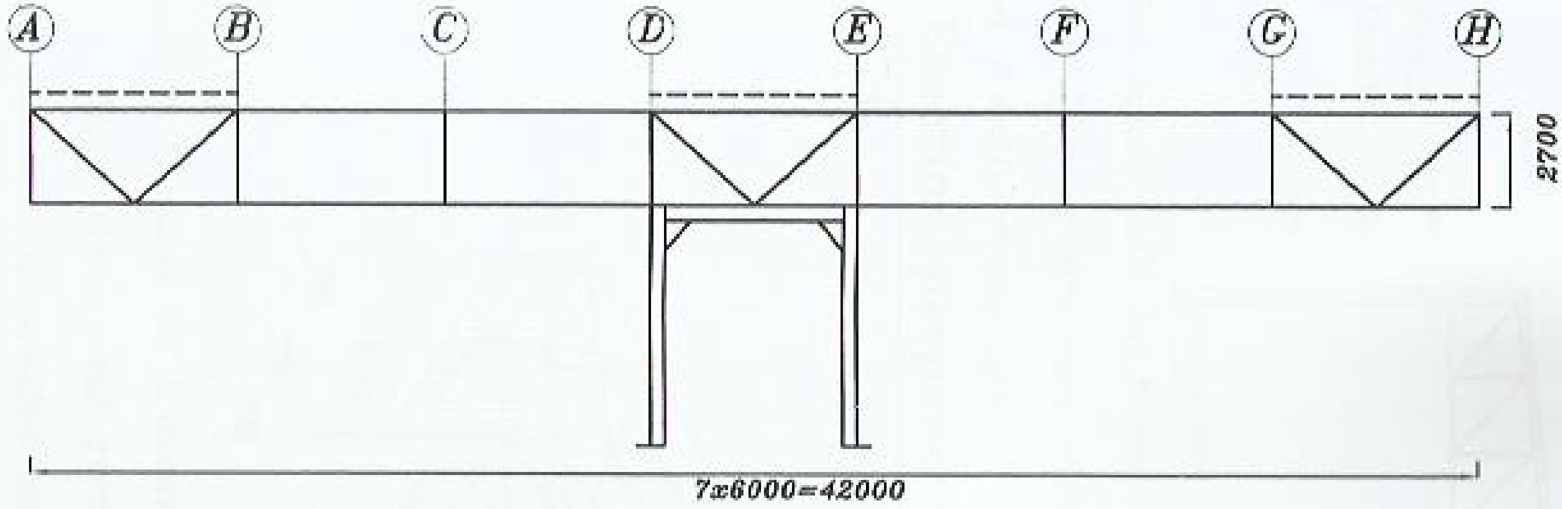


VERTICAL BRACING

@ Axes (1&7)



LONGITUDINAL BRACING *@ Axes (2&6)*

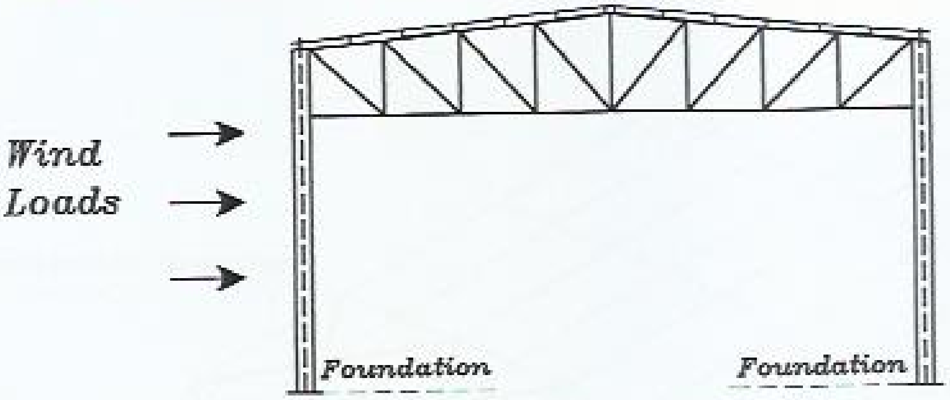


LONGITUDINAL BRACING @ Axes (3&5)

LONGITUDINAL BRACING @ Axis (4)

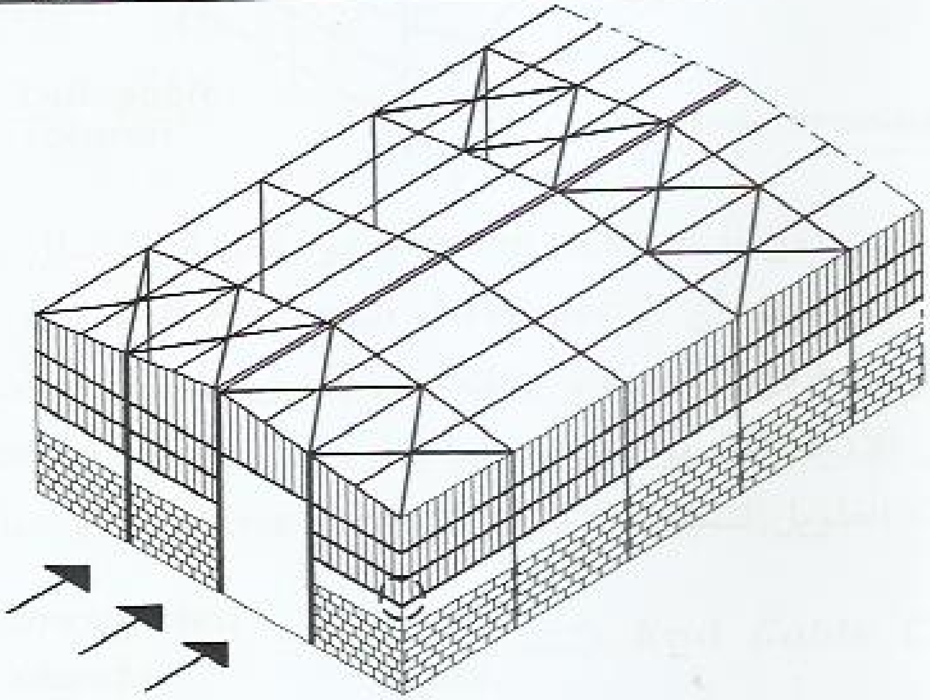
Transition of wind loads

1) Direction parallel to frames

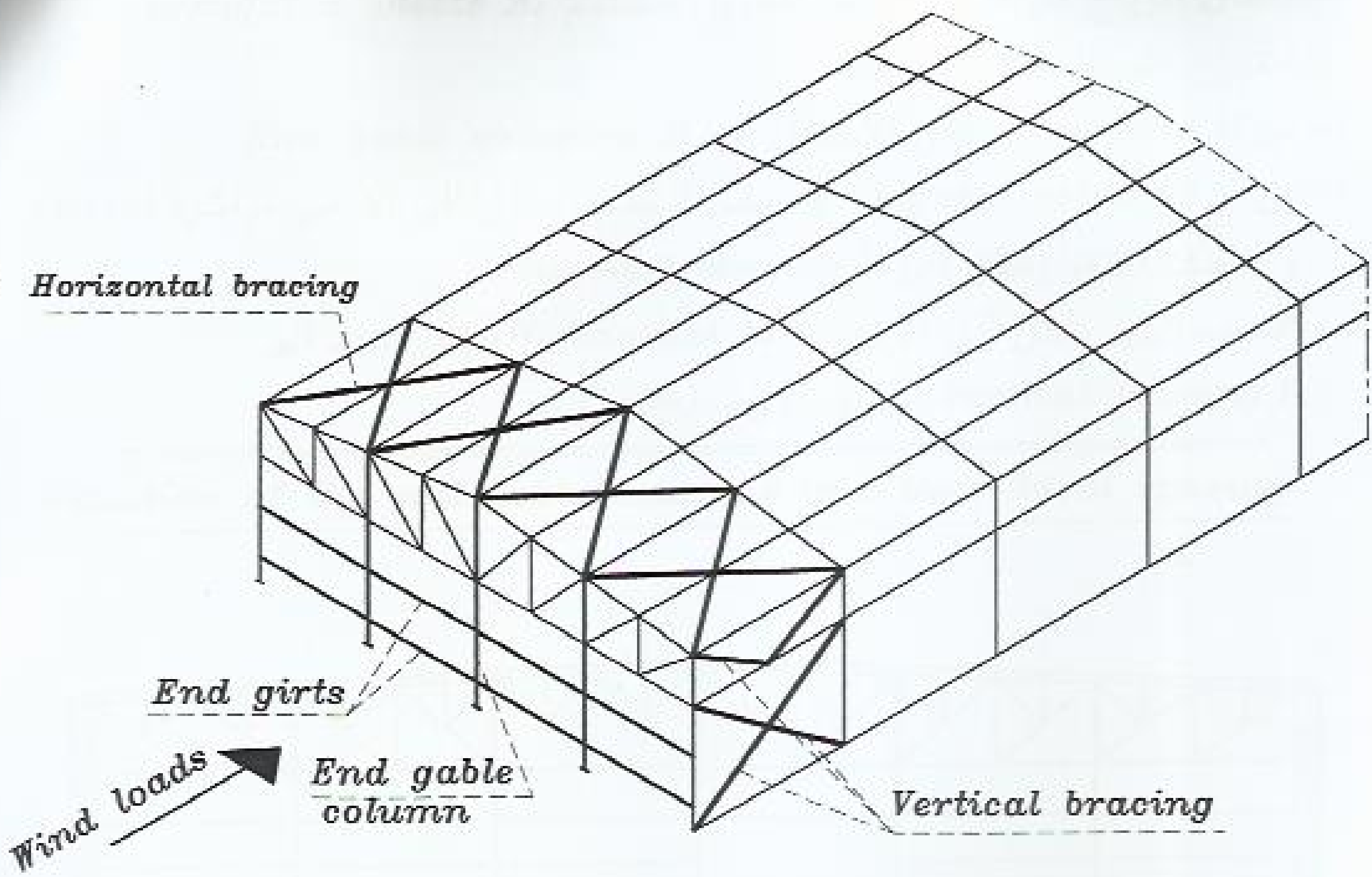


Wind loads will be transmitted from corrugated sheets to side girts. Then from side girts to the columns of the main system and finally from columns to the foundations.

2) Direction perpendicular to frames



Wind loads will be transmitted from corrugated sheets to end girts. Then from end girts to the end gable columns. Half of load will be transmitted by the end gable column to foundations directly. And the other half will be transmitted to the hz. bracing then to the vertical bracing then to the foundations.



الهدف الرئيسى من الـ *Bracing system* هو مقاومة الاحمال فى الاتجاه الطويل للمبنى حيث أن الـ *Trusses* أو الـ*Frames* لا تستطيع تحمل سوى القوى الموجودة فى اتجاهها و بذلك فهى تتحمل الاحمال الرأسية و أحمال الرياح فى الاتجاه القصير للمبنى بينما يقاوم الـ *Bracing system* أحمال الرياح فى الاتجاه الطويل للمبنى بالاضافة للـ *Braking force* فى حالة وجودها .

Wind Loads ⇨ *Corrugated sheets* ⇨ *End Girts* ⇨ *End Gable Columns*

- $\frac{1}{2}$ ⇨ *Horizontal bracing* ⇩ *Vertical bracing* ⇩ *Base*
- $\frac{1}{2}$ ⇨ *Base*

توضيح لمسار أحمال الرياح فى الاتجاه الطويل للمبنى حتى تصل الى الارض

مسار الـ *Wind loads* فى الاتجاه الطويل للمبنى

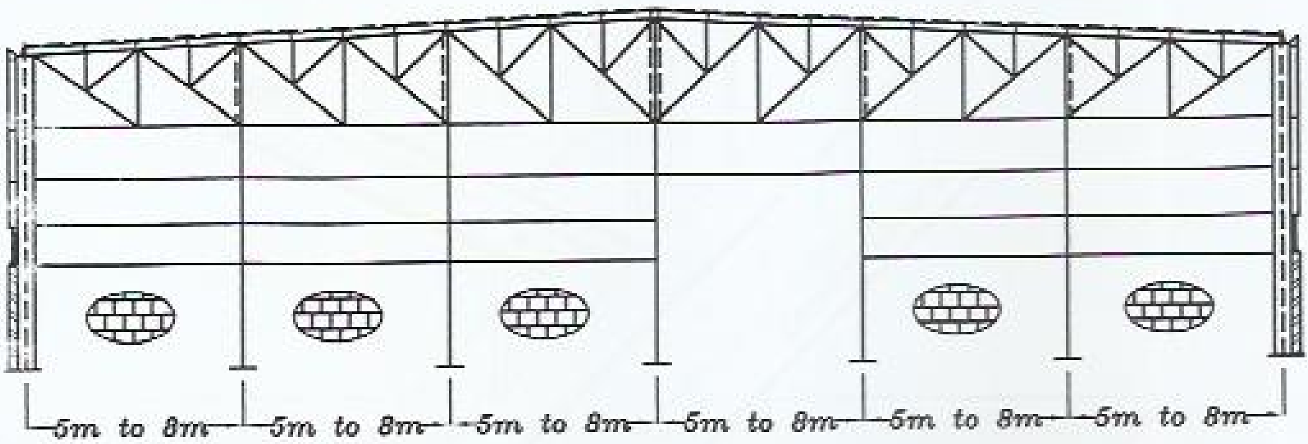
١- احمال الرياح فى اتجاه المبنى الطويل تصطدم بالـ *Corrugated sheets* فتتوزع على الـ *End girts* .

٢- من الـ *End girts* تنتقل الاحمال الى الـ *End gable columns* .

٣- من الـ *End gable columns* نصف الاحمال تنتقل الى الارض مباشرة (base) و النصف الاخر ينتقل الى الـ *Horizontal bracing* .

٤- النصف الذى ينتقل الى الـ *Horizontal bracing* ينتقل بعده الى الـ *Vertical bracing* ثم الى الارض (base) .

Function of the end gable columns and their ideal spacing



1- *Support end girts which support corrugated sheets.*

2- *Transmit half of wind load to the foundations.*

3- *Can be used as a support for the truss in case of no future extension.*

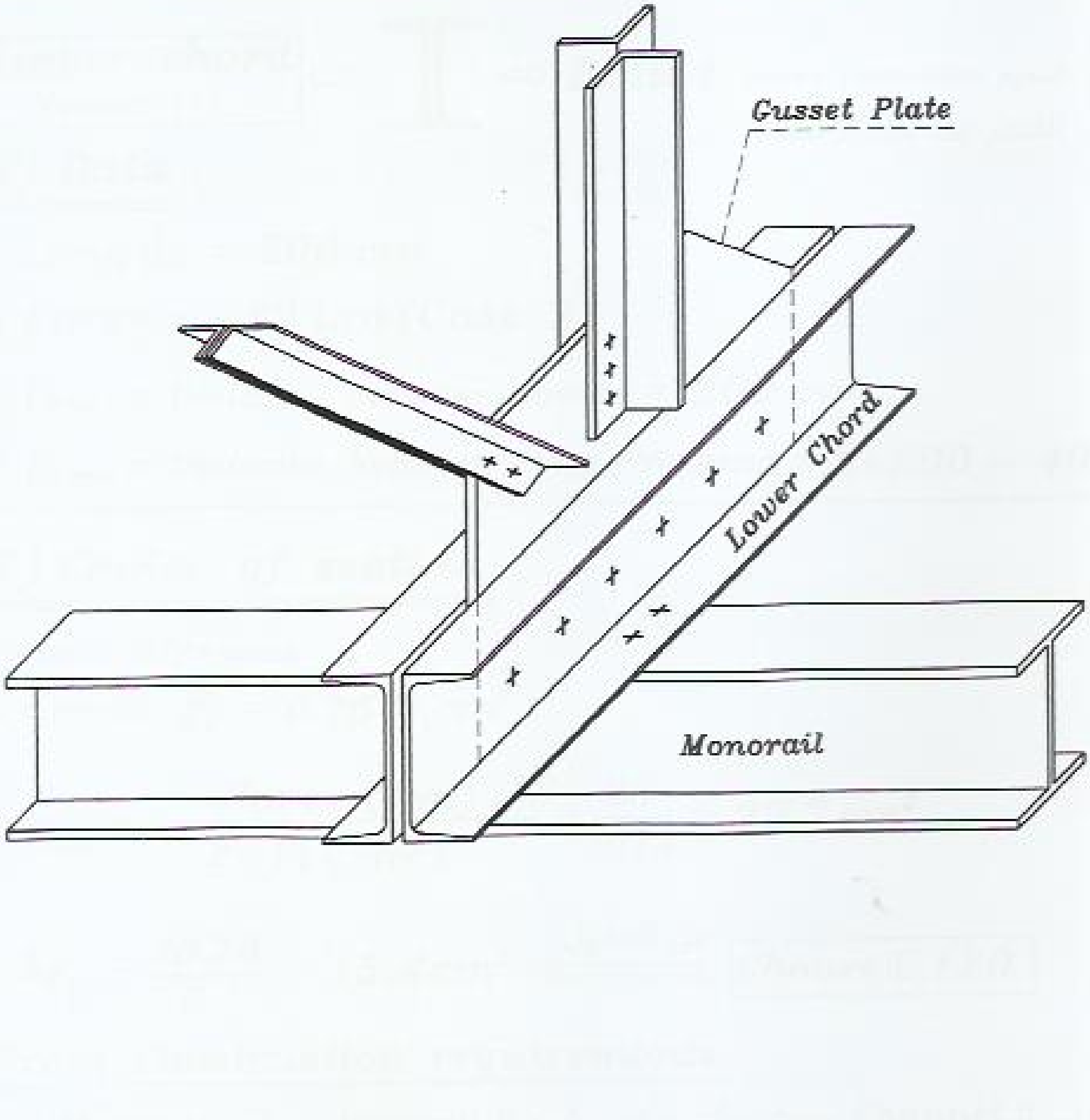
4- *Ideal spacing is from 5.0m to 8.0m .*

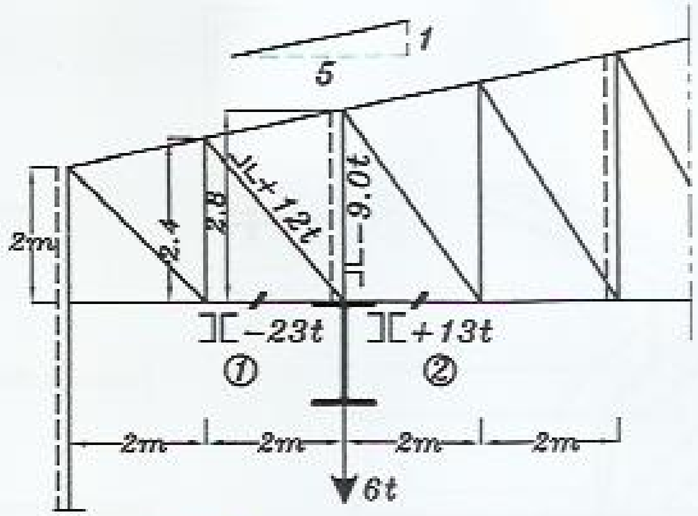
Question (2)

1) Design an intermediate roof purlin, using hot-rolled channel section (Spacing between trusses=7.50m, weight of roof covering= 8 kg/m^2)

مؤجل للترم الثانى

2) Design the truss members, intersecting at the marked joint. Use back to back channels for chord members and back to back angles for the web members (Use H.S.B M20 (8.8) with threads excluded from the planes of shear and 15mm thick gusset plates).





Lower chord Member (1) ⇨][⇨ ***Bolted*** لانه مطلوب تصميم Bolted Connection في المطلوب القادم عند هذه ال Joint

1) Data

* Length = **200 cm**
* Force = **−23** ton (Case A)
* $l_{b\,in}$ = Distance between joints = **200 cm**
* $l_{b\,out}$ = Distance between long. bracing = 2*200 = **400 cm**

2) Choice of section

From Stresses

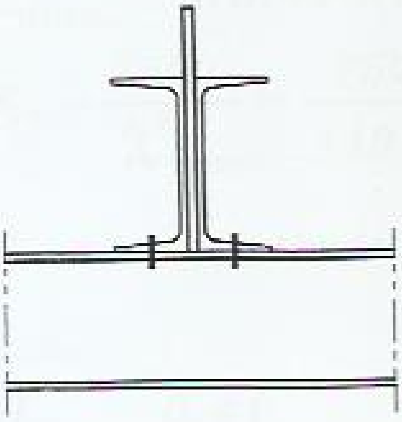
* assume $F_C = 0.75\ t\backslash cm^2$

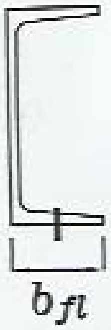
$$A_{g\,][} = \frac{Force\ (ton)}{F_C\ (t\backslash cm^2)} = \frac{23}{0.75} = 30.7\ cm^2$$

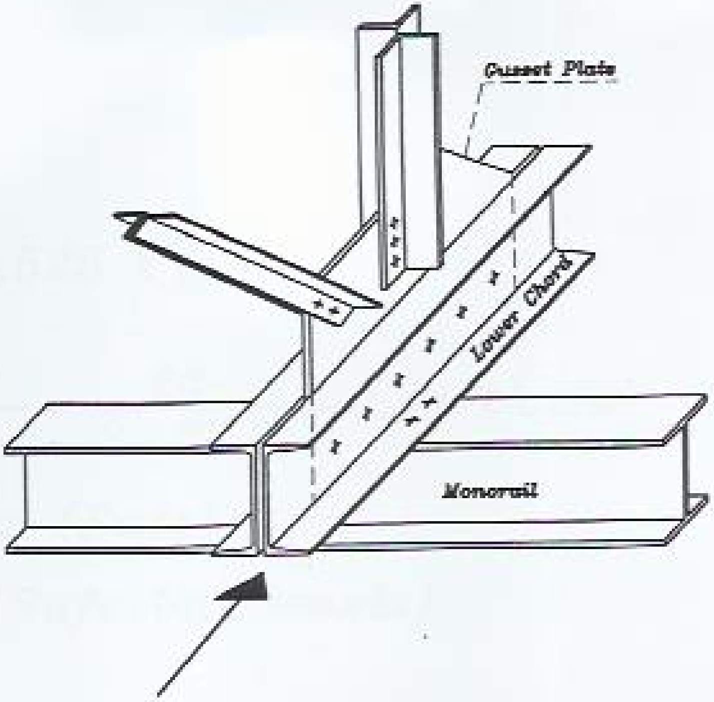
$$A_{g\,[} = \frac{30.70}{2} = 15.4\ cm^2 \xrightarrow{\text{من الجدول}} \boxed{\text{Choose } [\ 120}$$

From Construction requirements

لابد من اختيار ال Channel بحيث يمكن تركيب كمرة ال Monorail بمسامير في Flange ال Channel .





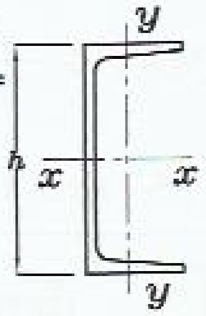


$* \, b_{min.} = 1.1 * 3\,\phi = 3.3 * 16 = 66.0\ mm$

لابد من اختيار الـ Channel بحيث لا يقل عرض الـ flange بها عن 66.0 mm

Choose][180

3) Checks



$b = 7\ Cm$

$h = 18\ Cm$

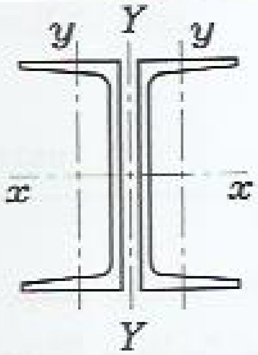
$r_x = 6.95\ Cm$

$r_y = 2.02\ Cm$

$A = 28.0\ Cm^2$

$e = 1.92\ Cm$

عند وضع الـ Web رأسي



a) Buckling (Slenderness)

$r_{x_{][}} = r_{x_{[}}$ من الجدول $= 6.95\ cm$

Given $t_{Gp} = 1.5\ cm$

$$r_{y_{][}} = \sqrt{r_{y_{[}} + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.02^2 + (1.92 + \frac{1.5}{2})^2} = 3.34\ cm$$

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x_{][}}} = \frac{200}{6.95} = 28.77 < 180 \Longrightarrow \textbf{(Safe)}$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{y_{][}}} = \frac{400}{3.34} = 119.47 < 180 \Longrightarrow \textbf{(Safe)}$$

5) Stress

$$\lambda_{max.} = 119.47 > 100$$

$$* F_C = \frac{7500}{\lambda^2_{max.}} = \frac{7500}{119.47^2} = 0.525 \ t \backslash cm^2$$

$$* f_C = actual\ stress = \frac{force}{A_{g\,][}} = \frac{23}{2 * 28.0} = 0.41 \ t \backslash cm^2$$

$$> F_C \Longrightarrow (Safe)$$

$$* \frac{f_C}{F_C} = \frac{0.41}{0.525} = 0.78 \Longrightarrow (Safe\ but\ waste)$$

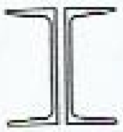
Design of tie plate

المحور الضعيف فى ال Channel هو محور y و بالتالى هو الذى ندرس حدوث Buckling لل Channel الواحدة حوله .

$$\lambda_y \leqslant \lambda_{max.}$$

$$\frac{l'}{r_{y[}} = \frac{l'}{2.02} \leqslant 119.47 \Longrightarrow l' \leqslant 2.02 * 119.47 = 241\ cm$$

$$l' \leqslant 241\ cm > l \Longrightarrow No\ need\ for\ tie\ plates$$

Lower chord
Member (2) ⇒  ⇒ **Bolted** لانه مطلوب تصميم Bolted Connection فى المطلوب القادم عند هذه ال Joint

1) Data

* Length = **200** cm
* Force = + **13** ton (Case A)
* $l_{b\,in}$ = Distance between joints = **200** cm
* $l_{b\,out}$ = Distance between long. bracing = 2 * 200 = **400** cm

2) Choice of section

From Stresses

$* F_t = 1.40\ t\backslash cm^2$

$$A_{g\,][} = \frac{Force\ (ton)}{0.85 * F_t\ (t\backslash cm^2)} = \frac{13}{0.85*1.40} = 10.92\ cm^2$$

Symmetric Bolted

$$A_{g\,[} = \frac{10.92}{2} = 5.46\ cm^2 \xrightarrow{\text{من الجدول}} \boxed{Choose\ [\ 40}$$

From Construction requirements

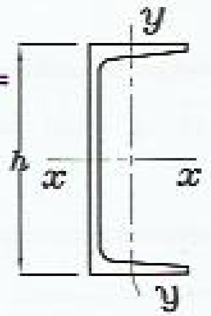
لابد من اختيار ال Channel بحيث يمكن تركيب كمرة ال Monorail بمسامير فى Flange ال Channel .

$* b_{min} = 1.1*3\,\phi = 3.3*20 = 66.0\ mm$

لابد من اختيار ال Channel بحيث لا يقل عرض ال flange بها عن 66.0 mm

$$\boxed{Choose\ ][\ 180}$$

3) Checks



$b = 7\ Cm$

$h = 18\ Cm$

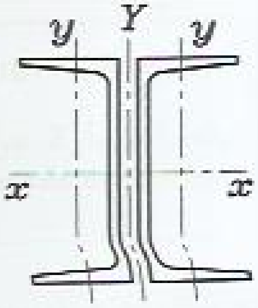
$r_x = 6.95\ Cm$

$r_y = 2.02\ Cm$

$A = 28.0\ Cm^2$

$e = 1.92\ Cm$

عند وضع الWeb رأسى



$$A_{net} = 2[\,A_{gross\,[} - (\phi + 0.2_{cm}) * t_{w\,[}\,]$$

$$= 2[\,28.0 - (2.0 + 0.2_{cm}) * 0.80\,] = 52.48\ cm^2$$

a) Stress

$$* f_t = \frac{Force}{A_{net}} = \frac{Force}{2 * A_{[}} = \frac{13}{52.48} = 0.25\ t\backslash cm^2$$

مساحة ال channel التى تم حسابها

$< F_t = 1.40\ t\backslash cm^2$ **(Safe)**

b) Slenderness

$r_{x\,][} = r_{x\,[}$ من الجدول $= 6.95\ cm$

Given $t_{Gp} = 1.5\ cm$

$$r_{y\,][} = \sqrt{r_{y\,[} + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.02^2 + (1.92 + \frac{1.5}{2})^2} = 3.34\ cm$$

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x\,][}} = \frac{200}{6.95} = 28.77 < 180 \Longrightarrow (Safe)$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{y\,][}} = \frac{400}{3.34} = 119.47 < 180 \Longrightarrow (Safe)$$

C) Deflection

$$* \frac{L}{d} = \frac{200\ cm}{18\ cm} = 11.1 \leqslant 60 \Longrightarrow (Safe)$$

(d) البعد الرأسى للقطاع

ملحوظة هامة

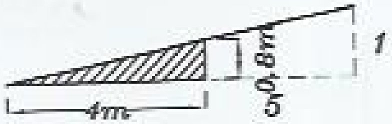
الـ Two Lower Chords لهما نفس القطاع 180][و لهذا نعتبر الوصلة بينهما عبارة عن Continuous Joint .

Vertical ⇨ ⅃L ⇨ **Bolted** لانه مطلوب تصميم Bolted Connection

من الطبيعى أن يكون الـ Vertical عند أماكن الـ Longitudinal Bracing عبارة عن Star shaped

و لكن هذا الـ memeber مطلوب تصميمه على أساس أنه back to back angles

1) Data



* Length = **280** cm
* Force = − **9.0** ton (Case A)
* $l_{b\,in}$ = Distance between joints = **280** cm
* $l_{b\,out}$ = **280** cm

2) Choice of section

From stresses

* assume $F_C = 0.75\ t\backslash cm^2$

$$\therefore A_{g\,\lrcorner\llcorner} = \frac{force}{F_C} = \frac{9.0}{0.75} = 12.0\ cm^2$$

$$\therefore A_{g\,\llcorner} = \frac{A_{g\,\lrcorner\llcorner}}{2} = \frac{12.0}{2} = 6.0\ cm^2$$

Choose $\xrightarrow{tables}$ ∟ 60*60*6

From buckling

* assume $\lambda_{out} = \lambda_{in} = 100$

$$\therefore 100 = \frac{l_{bin}}{r_x} = \frac{280}{0.30a_2}$$

$\Longrightarrow a_2 =$ **9.33 cm**

$$\therefore 100 = \frac{l_{bout}}{r_y} = \frac{280}{0.45a_3}$$

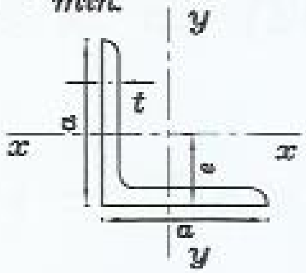
$\Longrightarrow a_3 = 6.22\ cm$

لا نحتاج هنا لحساب ال a_3

$$a_{av} = \frac{a_1 + (\overset{\text{الاكبر}}{a_2\ Or\ a_3})}{2} = \frac{6.0+9.33}{2} = 7.66 cm$$

Choose ⅃∟ 80*80*8

> minimum angle $a_{min} = 1.1*3\phi = 1.1*3*2.0 = 6.60\ cm$



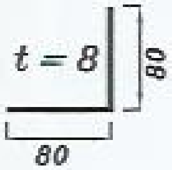
∟ 80*80*8
$A = 12.3\ Cm^2$
$e = 2.26\ Cm$
$r_x = r_y = 2.42\ Cm$
$r_v = 1.55\ Cm$

3) Checks

a) Class. of section (Compactness)

$$\frac{b}{t} = \frac{80}{8} = 10 < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$$

$\Longrightarrow$ The section is npn-compact (Code page 12)



$r_{x_{\rfloor\lfloor}} = r_{x_{\lfloor}}$ من الجدول $= 2.42\ cm$

Given $t_{Gp} = 1.5\ cm$

$$r_{y_{\rfloor\lfloor}} = \sqrt{r_{y\lfloor}^2 + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.42^2 + (2.26 + \frac{1.5}{2})^2} = 3.86\ cm$$

b) Buckling

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x_{\rfloor\lfloor}}} = \frac{280}{2.42} = 115.7 < 180 \Longrightarrow \text{(Safe)}$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{y_{\rfloor\lfloor}}} = \frac{280}{3.86} = 72.53 < 180 \Longrightarrow \text{(Safe)}$$

لا نحتاج هنا لحساب ال λ_{out} لانها بالتاكيد ستكون أصغر من λ_{in}

c) Stress

$\lambda_{max.} = 115.7 > 100$

$$* F_C = \frac{7500}{\lambda^2_{max.}} = \frac{7500}{115.7} = 0.56\ t \backslash cm^2$$

$$* f_C = \text{actual stress} = \frac{force}{2 * A_{g\lfloor}} = \frac{9.0}{2 * 12.3} = 0.36\ t \backslash cm^2$$

$$\leqslant F_C \Longrightarrow \text{(Safe) but waste}$$

$$* \frac{f_C}{F_C} = \frac{0.36}{0.56} = 0.64 \Longrightarrow \text{(Safe but waste)}$$

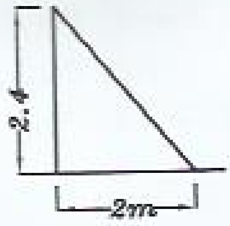
Design of tie plate

$\lambda_v \leqslant \lambda_{max.}$

$$\frac{l'}{r_{v\lfloor}} = \frac{l'}{1.55} \leqslant 115.7 \Longrightarrow l' \leqslant 1.55 * 115.7 = 179.3\ cm$$

$$l' \leqslant 179.3\ cm > \frac{l}{2} \Longrightarrow$$ Use one tie plate at the middle of member

Diagonal ⇨ ⅃L ⇨ **Bolted** Bolted Connection لانه مطلوب تصميم



1) Data

* Length $= \sqrt{200^2 + 240^2} = 312$ cm
* Force $= +12$ ton (Case A)
* $l_{b\,out}$ = Distance between joints = 312 cm
* $l_{b\,out} = 312$ cm

2) Choice of section

1) From stress condition

$$A_{g_{⅃L}} = \frac{\text{Force ton}}{0.85 * F_t\ (t \backslash cm^2)} = \frac{12}{0.85 * 1.4} = 10.08\ cm^2$$

Bolted

$$A_{g_{L}} = \frac{10.08}{2} = 5.04\ cm^2$$

من الجدول ⟹ **Choice ⅃L 55x55x5** $a_1 = 5.50$ cm

2) From buckling condition

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x_{⅃L}}} = \frac{312}{0.30\,a} = 300 \Longrightarrow a = 3.47\ cm$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{x_{⅃L}}} = \frac{312}{0.45\,a} = 300 \Longrightarrow a = 2.31\ cm$$

$a_2 = 3.47$ cm

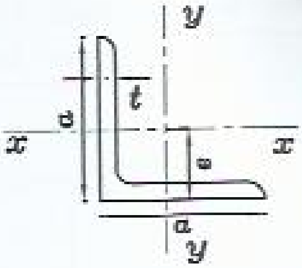
3) From construction condition

minimum angle $a_{min.} = 1.1 * 3\ \phi = 1.1\ \ 3 * 2.0 = 6.60$ cm

$a_3 = 6.60$ cm   نختار أكبر angle من السابق

Choose ⅃L 70x70x7

3) Checks



$\llcorner 70*70*7$
$A = 9.40 \ Cm^2$
$e = 1.97 \ Cm$
$r_x = r_y = 2.12 \ Cm$

$$A_{net} = 2[A_{gross \llcorner} - (\phi + 0.2_{cm}) * t_{\llcorner}]$$

$$= 2[9.40 - (2.0 + 0.2_{cm}) * 0.7] = 15.72 \, cm^2$$

a) Stress

Tension

$$* \ f_t = \frac{Force}{A_{net}} = \frac{12}{15.72} = 0.76 \ t \backslash cm^2$$

مساحة ال angles التي تم حسابها

$$\leqslant F_t = 1.40 \ t \backslash cm^2 \ \text{(Safe)}$$

b) Slenderness

$r_{x_{\lrcorner\llcorner}} = r_{x_{\llcorner}}$ من الجدول $= 2.12 \ cm$

$$* \ \lambda_{in} = \frac{l_{b\,in}}{r_{x_{\lrcorner\llcorner}}} = \frac{312}{2.12} = 147.17 < 300 \Longrightarrow \text{(Safe)}$$

$$r_{y_{\lrcorner\llcorner}} = \sqrt{r^2_{y\llcorner} + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.12^2 + (1.97 + \frac{1.5}{2})^2} = 3.44 \ cm$$

$$* \ \lambda_{out} = \frac{l_{b\,out}}{r_{y_{\lrcorner\llcorner}}} = \frac{312}{3.44} = 90.69 < 300 \Longrightarrow \text{(Safe)}$$

C) Deflection

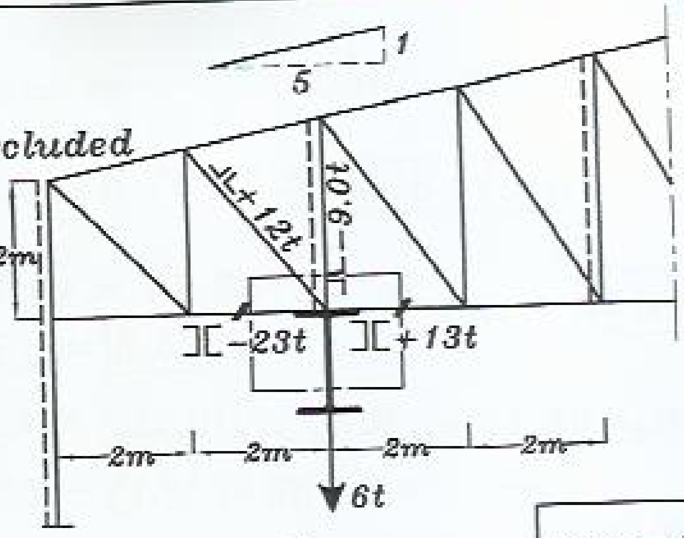
$$* \ \frac{L}{d} = \frac{312 \ cm}{a} = \frac{312 \ cm}{7.0} = 44.57 \leqslant 60 \Longrightarrow \text{(Safe)}$$

Design the non-pretensioned bearing type bolted connection for the marked joint, considering the monorail reaction.

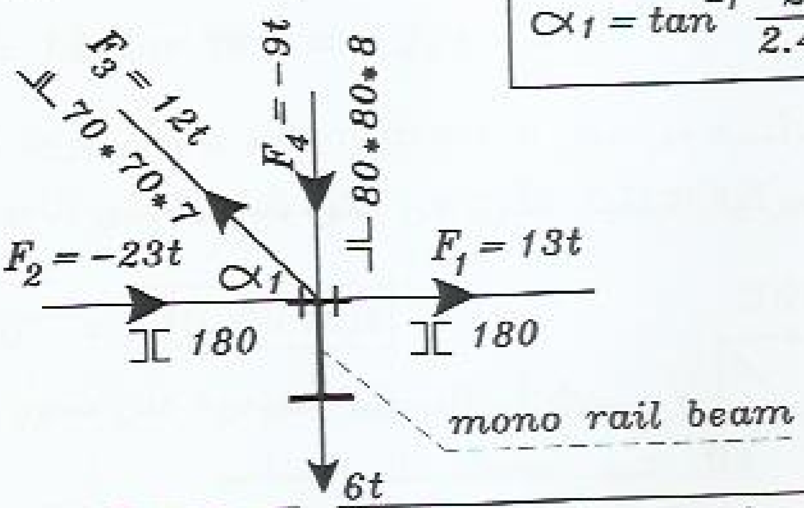
BOLTS M20

Grade 8.8

Threads are excluded



$$\alpha_1 = \tan^{-1}\frac{2}{2.4} = 39.8^\circ$$



$$R_{Shear} = q_b * A_S * n \qquad R_b = F_b * d * t_{min}$$

* $F_{ub} = 8\ t\backslash cm^2$
* $\phi = 2.0\ cm$
* $F_u \xrightarrow{for\ st.37} = 3.6\ t\backslash cm^2$
* $q_b = 0.25 F_{ub}$
* $A_S = \frac{\pi d^2}{4}$ (Threads excluded)
* Take $e > 2\phi = 4.0\ cm \Longrightarrow \alpha = 0.8$
* $R_{S.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 1 = (0.25 * 8) * \frac{\pi (2.0)^2}{4}$
  $= 6.28\ ton$
* $R_{D.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 2 = 2 * R_{S.S} = 12.56\ ton$
* $R_b = (\alpha * F_u) * d * t_{min} = 0.8 * 3.6 * 2.0 * t_{min} = 5.76\ t_{min}$

Member (1&2) ⟹ Continuous member ⟹ ⌶ 180

$$* \; t_{min} = \overset{t_{G.P}}{1.50\,cm} \quad or \quad \overset{t_{[}}{2*0.80} = 1.6\,cm \Longrightarrow \boxed{t_{min} = 1.5\,cm}$$

$$* \; R_b = 5.76\, t_{min} = 5.76 * 1.5 = \boxed{8.64\,ton}$$

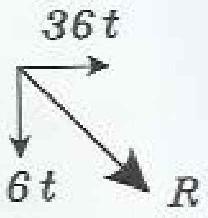
$$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = 12.56 \\ R_b = \boxed{8.64} \end{cases} \qquad R_{Least} = \boxed{8.64\,ton}$$

أولا نوجد المركبة الافقية للقوة و هى الاكبر من القيمتين التاليين

$$\left.\begin{array}{l} F_1 - F_2 = -23 - (13) = 36t \\ F_3 \; Cos\; 39.8 = 12\; Cos\; 39.8 = 9.21t \end{array}\right\} 36t$$

و المركبة الرأسية هى حمل الـ *monorail* و بالتالى نوجد محصلة المركبة الرأسية و المركبة الافقية فتكون هى القوة المؤثرة على الـ *Memeber* .

$$* \; R = \sqrt{36^2 + 6^2} = \boxed{36.49\,ton}$$



و حيث أن المحصلة عمودية على محور المسامير فانها تسبب *Shear* على المسامير .

$$* \; n_{1-2} = \frac{R_1 - R_2}{R_{Least}} = \frac{36.49}{8.64} = 4.22 \Longrightarrow \boxed{5\ Bolts}$$

No B.S.R check for continuous member

Member (3) ⟹ ⅃L 70*70*7 ⟹ Shear

$$* \; t_{min} = \overset{t_{G.P}}{1.50\,cm} \quad or \quad \overset{t_{L}}{2*0.7} = 1.4\,cm \Longrightarrow \boxed{t_{min} = 1.4\,cm}$$

$$* \; R_b = 5.76\, t_{min} = 5.76 * 1.4 = \boxed{8.06\,ton}$$

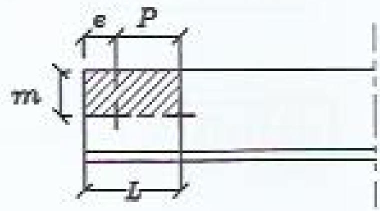
* $R_{Least} \rightarrow \begin{cases} R_{D.S} = 12.56 \\ R_b = \boxed{8.06} \end{cases}$ $\quad R_{Least} = \boxed{8.06\ ton}$

* $n_3 = \dfrac{\textbf{Force}}{R_{Least}} = \dfrac{12}{8.06} = 1.49 \Longrightarrow \boxed{2\ Bolts}$

Check block shear rupture

* Take $e = 4.0\ cm$

  $P = 6.0\ cm > P_{min} = 3\ \phi$



* $m = \dfrac{a-t}{2} = \dfrac{7-0.7}{2} = 3.15\ cm$

* $L = e + 1*P = 4.0 + 1*6 = 10.0\ cm$

* $A_{net\ Shear} = [\ L - (\ n - 0.5\ )(\phi + 0.2)] * t_L$

  $= [\ 10.0 - (\ 2 - 0.5\ )(\ 2.0 + 0.2\ )] * 0.7 = 4.69\ cm^2$

* $A_{net\ Tension} = [m - 0.5 * (\phi + 0.2)] * t_L$

  $= [\ 3.15 - 0.5 * (\ 2.0 + 0.2\ )] * 0.7 = 1.43\ cm^2$

* $\mathbf{P = 0.4 F_y A_{net\ Shear} + 0.725\ F_y A_{net\ Tension}}$

  $= 0.4 * 2.4 * 4.69 + 0.725 * 2.4 * 1.43 = 7.0\ \ ton$

* Check $\Longrightarrow \underbrace{2}_{\text{two angles}} P = 14.0\ ton > Tension\ force = 12.0\ ton$

  $\Longrightarrow$ *Safe* $\Longrightarrow$ *No B.S.R failure*

Member (4) $\Longrightarrow$ ⅃L 80*80*8 $\Longrightarrow$ Shear

$* \; t_{min} = \overset{t_{G.P}}{1.50} \, cm \quad or \quad \overset{t_L}{2*0.8} = 1.6 \, cm \Longrightarrow \boxed{t_{min} = 1.5 \, cm}$

$* \; R_b = 5.76 \, t_{min} = 5.76 * 1.5 = \boxed{8.64 \, ton}$

$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = 12.56 \\ R_b = \boxed{8.64} \end{cases} \qquad R_{Least} = \boxed{8.64 \, ton}$

$$* \; n_4 = \frac{Force}{R_{Least}} = \frac{9}{8.64} = 1.04 \Longrightarrow \boxed{2 \text{ Bolts}}$$

No B.S.R check for compression member

Bolts connecting upper flange of mono rail to lower chord of truss

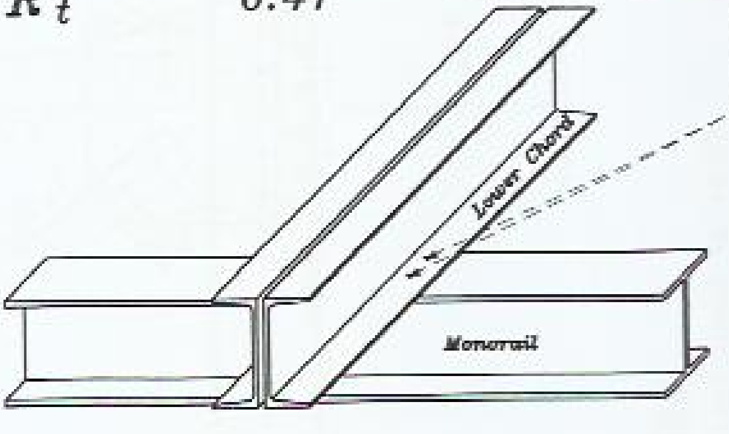
$\Longrightarrow$ Subjected to $\Longrightarrow$ **Tension** $= 6 \, t$

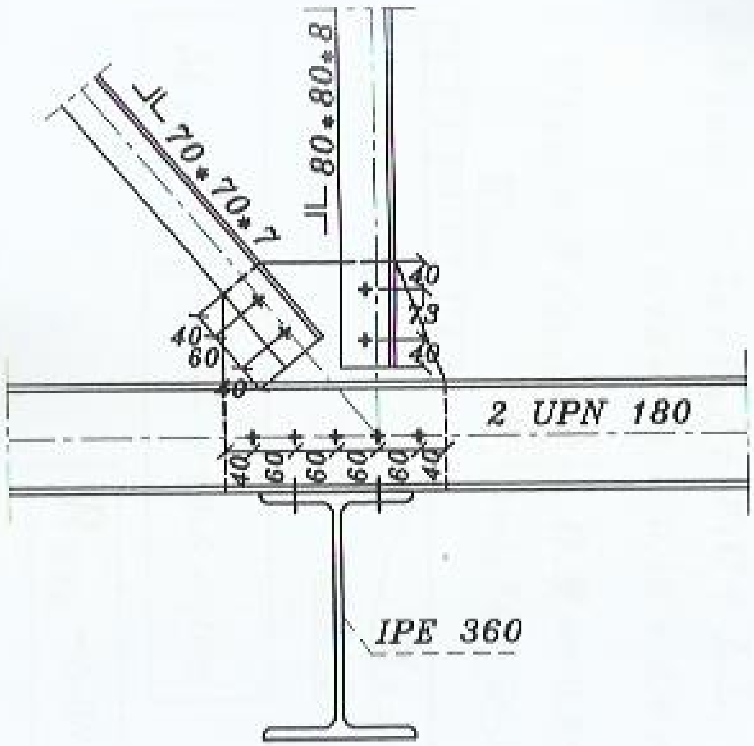
$$\boxed{R_t = \left(0.78 * \frac{\pi d^2}{4}\right) \left(0.33 * F_{ub}\right)}$$

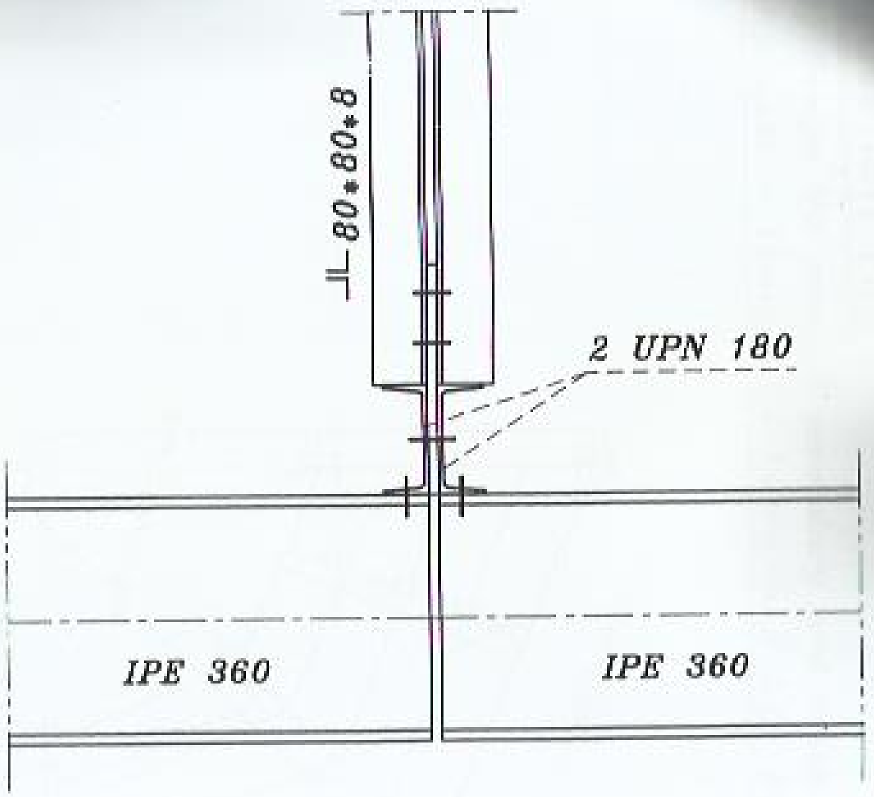
$$R_t = 0.78 * \frac{\pi (2.0)^2}{4} * 0.33 * 8 = 6.47 \, ton$$

$$* \; n = \frac{F}{R_t} = \frac{6}{6.47} = 0.93 = \boxed{4 \text{ Bolts}}$$

على الاقل مسماران هنا
و مسماران الناحية الاخرى







Notes

1- Steel used is st.37

2- Non-pretensioned bolts M20, bearing type, Grade 8.8.

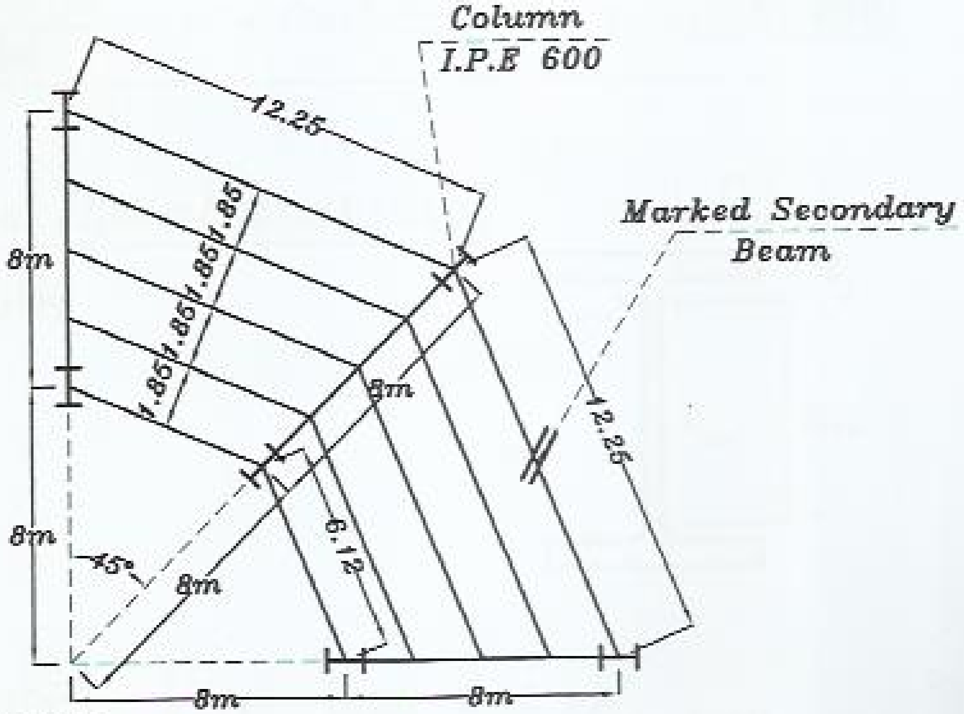
3- Thickness of gusset plate is 15mm.

4- All dimensions are in mms.

5- Scale used is 1:10.

Question (3)

1) Design the marked secondary and main beams (R.C slab average thickness = 12cm, F.C= 150kg/m² & L.L= 400kg/m²)



Secondary Beam

1- Straining actions

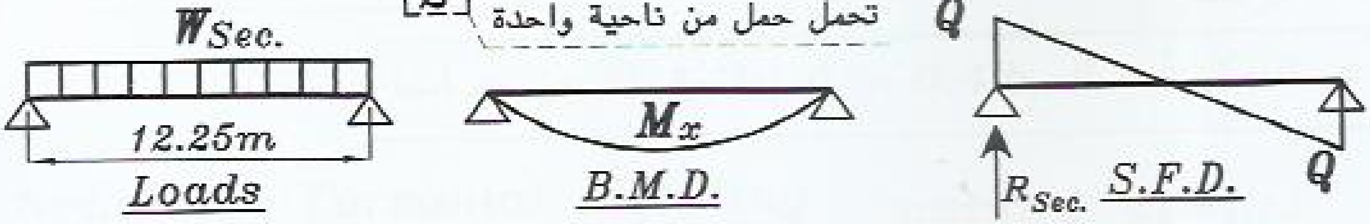
assume o.w = 100 kg/m

$$\# \; W_{Total} = t_s * \gamma_{R.C} + F.C + L.L$$

$$= 0.12 * 2.5 + 0.15 + 0.4 = 0.85 \; t/m^2$$

$$\# \; W_{Sec.} = W_{Total} * \frac{a}{2} + O.W = 0.85 * \frac{1.85}{2} + 0.10 = 0.89 \; t/m$$

تحمل حمل من ناحية واحدة



$$M_X = \frac{0.89 * 12.25^2}{8} = 16.5 \; m.ton$$

$$Q = \frac{0.89 * 12.25}{2} = 5.39 \; ton$$

$$R_{Sec.} = 5.39 \; ton$$

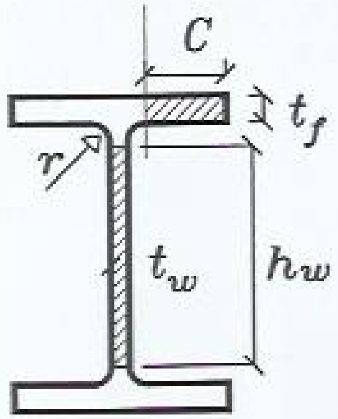
2- Choise of section

assume $F_b = 0.64 F_y = 1.536\ t \backslash cm^2$

$$S_X = \frac{M_X}{1.536} = \frac{16.5 * 100}{1.536} = 1074\ Cm^3 \xrightarrow{\text{Tables}} \text{Use } \boxed{I.P.E\ 400}$$

3- Checks

a- Compactness (Local buckling)



$h_w = 33.1\,cm$ جداول

$t_w = 0.86\,cm$

$b_f = 18\ cm$

$t_f = 1.35\,cm$

$r = 2.1\,cm$

$S_x = 1160\ cm^3$

$$\frac{C}{t_f} = \frac{\frac{1}{2}(b_f - t_w - 2r)}{t_f} = \frac{\frac{1}{2}(18 - 0.86 - 2 * 2.1)}{1.35} = 4.79$$

$$\therefore \frac{C}{t_f} = 4.79 < \frac{16.9}{\sqrt{f_y}} = 10.9 \Longrightarrow \text{Compact Flange}$$

$$\frac{hw}{tw} = \frac{33.1}{0.86} = 38.4 < \frac{127}{\sqrt{f_y}} = 82 \Longrightarrow \text{Compact Web}$$

$\therefore$ The section is compact

ملحوظة

من الممكن أخذ ال $C = 0.4\,b_f$ مباشرة بدلا من حسابها

b-Lateral Torsional Buckling

$C_b = 1.13$

$L_{U\,act.} = Zero$ (R.C slab supported the upper flange)

$\Longrightarrow$ no L.T.B

Compact Section $\Longrightarrow F_b = \boxed{1.536\,t \backslash cm^2}$

3a-Check bending stresses

$$f_{act.} = \frac{M_X}{S_X} = \frac{16.5 * 100}{1160} = 1.43\ t\backslash cm^2 < F_b = 1.536$$

1160 ← From Tables

(Safe)

3b- Check Shear stresses

$$q_{act.} = \frac{Q}{A_{Web}} = \frac{Q}{h * t_w} = \frac{5.39}{40 * 0.86} = 0.15\ t\backslash cm^2 < 0.35\ F_y = 0.84$$

(Safe)

3C- Check deflection

$W_{L.L} = L.L * \frac{a}{2} = 0.40 * \frac{1.85}{2} = 0.37\ t/m$

$$\triangle_{act.} = \frac{5}{384} * \frac{w_{L.L} * S^4}{E * I_X}$$

($w_{L.L}$: t\cm, S: cm)

$$= \frac{5}{384} * \frac{(0.37\backslash 100) * (1225)^4}{2100 * 23130} = 2.23\,cm < \frac{Span}{300}$$

(2100: t\cm², 23130: cm⁴)

$$< \frac{1225}{300} = 4.08cm$$

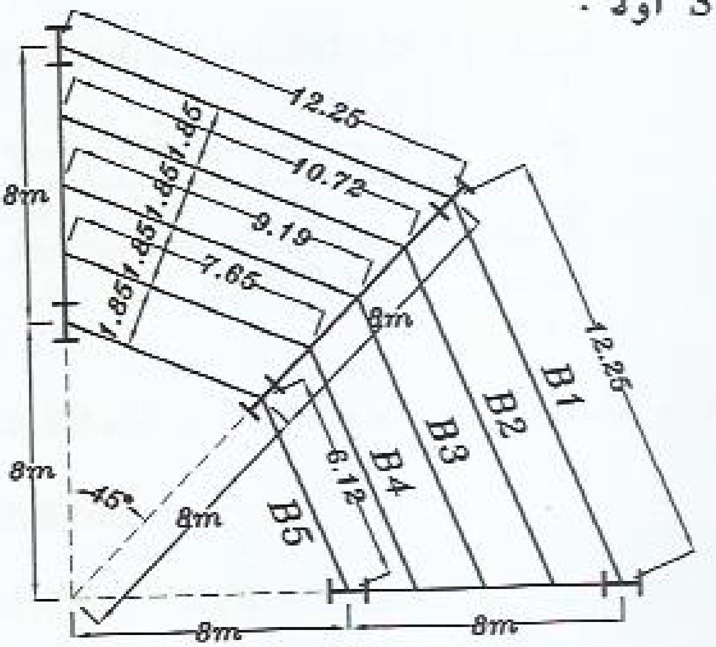
(Safe)

Main Beam

1- Straining actions

assume o.w = 150 kg/m

اذا أردنا حساب الاحمال بدقة على الكمرة الرئيسية نقوم بحساب Reactions الـ Sec. Beams أولا .



$$\# \; W_{outer\ Sec.\ beam} = W_{Total} * \frac{a}{2} + O.W$$

تحمل حمل من ناحية واحدة

$$= 0.85 * \frac{1.85}{2} + 0.10 = 0.89 \; t/m$$

$$\# \; W_{inner\ Sec.\ beam} = W_{Total} * a + O.W$$

تحمل حمل من ناحيتين

$$= 0.85 * 1.85 + 0.10 = 1.67 \; t/m$$

$$\boxed{R_{Sec.} = \frac{W * Span}{2}}$$

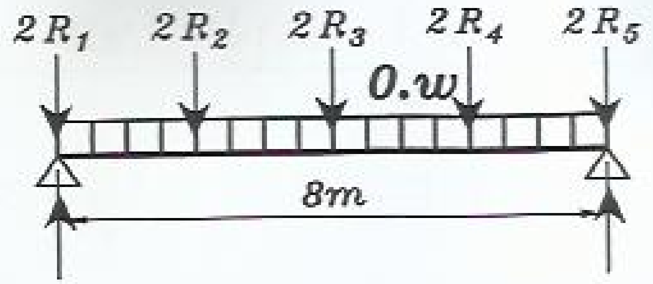
$$* \; R_1 = \frac{0.89 * 12.25}{2} = 5.39 \text{ ton}$$

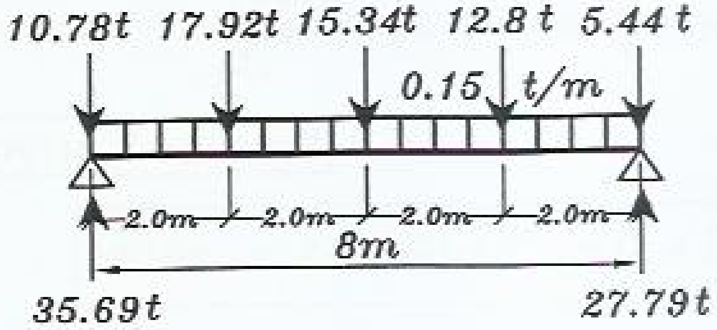
$$* \; R_3 = \frac{1.67 * 9.12}{2} = 7.67 \text{ ton}$$

$$* \; R_2 = \frac{1.67 * 10.72}{2} = 8.96 \text{ ton}$$

$$* \; R_4 = \frac{1.67 * 7.65}{2} = 6.40 \text{ ton}$$

$$* \; R_5 = \frac{0.89 * 6.12}{2} = 2.72 \text{ ton}$$





* $M_{at\ mid\ span} = 35.69 * 4 - 10.78 * 4 - 17.92 * 2 - 0.15 * 4 * 2$

$= 62.6$ m.ton

* $Q_{at\ support} = 35.69 - 10.78 = 24.9$ t

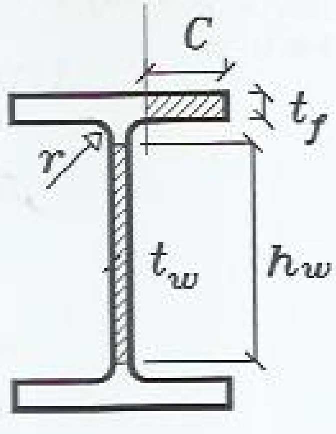
2- Choise of section

assume $F_b = 0.64 F_y = 1.536\ t \backslash cm^2$

$$S_X = \frac{M_X}{1.536} = \frac{62.6 * 100}{1.536} = 4075\ Cm^3 \xrightarrow{Tables} \text{Use } \boxed{H.E.B\ 500}$$

3- Checks

a- Compactness (Local buckling)



$h_w = 39.0$ cm جداول

$t_w = 1.45$ cm

$b_f = 30$ cm

$t_f = 2.80$ cm

$r = 2.7$ cm

$S_x = 4290\ cm^3$

$$\frac{C}{t_f} = \frac{\frac{1}{2}(b_f - t_w - 2r)}{t_f} = \frac{\frac{1}{2}(30 - 1.45 - 2 * 2.7)}{2.8} = 4.13$$

$$\therefore \frac{C}{t_f} = 4.13 < \frac{16.9}{\sqrt{f_y}} = 10.9 \Longrightarrow \text{Compact Flange}$$

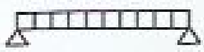
$$\frac{h_w}{t_w} = \frac{39.0}{1.45} = 26.9 < \frac{127}{\sqrt{f_y}} = 82 \Longrightarrow \text{Compact Web}$$

∴ The section is compact

ملحوظة

من الممكن أخذ ال $C = 0.4\, b_f$ مباشرة بدلا من حسابها

b-Lateral Torsional Buckling



$C_b = 1.13$

$L_{U\,act.} = Zero$ (R.C slab supported the upper flange)

$\Longrightarrow$ no L.T.B

Compact Section $\Longrightarrow F_b = 1.536\, t\backslash cm^2$

3a-Check bending stresses

$$f_{act.} = \frac{M_X}{S_X} = \frac{62.6 * 100}{4290} = 1.46\, t\backslash cm^2 < F_b = 1.536$$

4290 From Tables (Safe)

3b-Check Shear stresses

$$q_{act.} = \frac{Q}{A_{Web}} = \frac{Q}{h_w * t_w} = \frac{24.90}{50 * 1.45} = 0.34\, t\backslash cm^2 < 0.35\, F_y = 0.84$$

(Safe)

3C-Check deflection

$W_{L.L} = L.L * \frac{a}{2} = 0.40 * \frac{1.85}{2} = 0.37\, t/m$

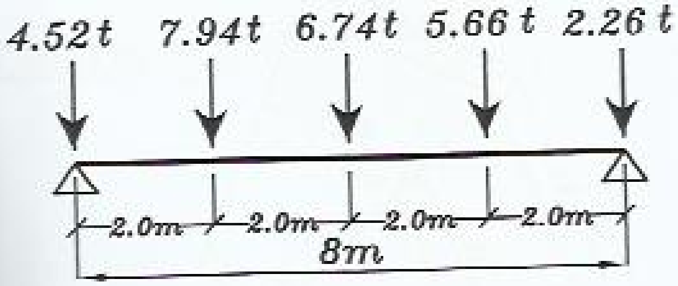
outer Sec. beam

$W_{L.L} = L.L * a = 0.40 * 1.85 = 0.74\, t/m$

inner Sec. beam

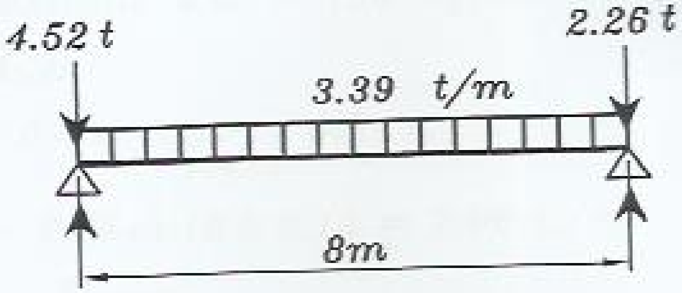
$$R_{Sec.\ L.L} = \frac{W_{L.L} * Span}{2}$$

* $R_1 = \frac{0.37 * 12.25}{2} = 2.26\ ton$
* $R_3 = \frac{0.74 * 9.12}{2} = 3.37\ ton$
* $R_2 = \frac{0.74 * 10.72}{2} = 3.97\ ton$
* $R_4 = \frac{0.74 * 7.65}{2} = 2.83\ ton$
* $R_5 = \frac{0.37 * 6.12}{2} = 1.13\ ton$



لحساب الـ Deflection فى المنتصف نحتاج الى حل الكمرة بطريقة الـ Conjugate Beam و لكنها ستأخذ وقت طويل جدا لذلك من الممكن تحويل هذه الاحمال الى حمل موزع .

وعند التحويل الى حمل موزع نترك الاحمال فوق الاعمدة بدون تحويل لانها ستذهب مباشرة الى الاعمدة .



$$\# W_{main} = \frac{7.94 + 6.74 + 5.66}{3}$$

$$= \mathbf{3.39}\ t/m$$

$$\triangle_{act.} = \frac{5}{384} * \frac{w_{L.L} * S^4}{E * I_X} \quad t\backslash cm \quad cm$$

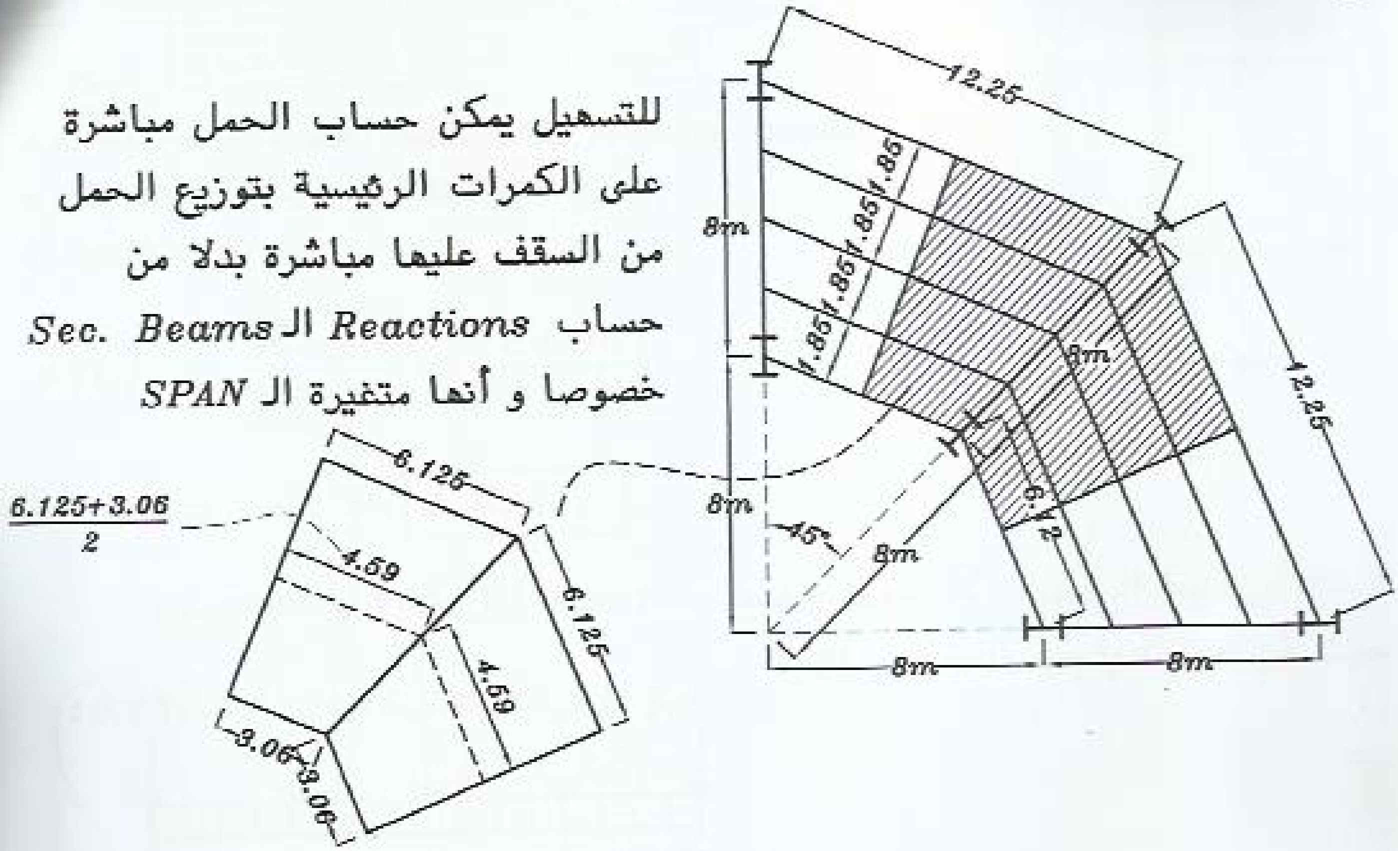
$$= \frac{5}{384} * \frac{(3.39\backslash 100) * (800)^4}{2100 * 107200} = 0.80\ cm < \frac{Span}{300}$$

$$t\backslash cm^2 \qquad cm^4 \qquad < \frac{800}{300} = 2.66cm$$

(Safe)

حل أخر أسهل لحساب الاحمال على الكمرة الرئيسية و لكنه أقل دقة

للتسهيل يمكن حساب الحمل مباشرة على الكمرات الرئيسية بتوزيع الحمل من السقف عليها مباشرة بدلا من حساب *Sec. Beams* ال *Reactions* خصوصا و أنها متغيرة ال *SPAN*



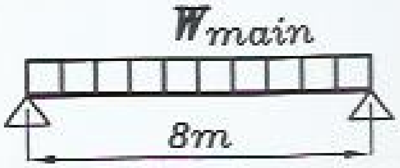
1- Straining actions

assume o.w = 150 kg/m

$$\# \; W_{Total} = t_s * \gamma_{R.C} + F.C + L.L$$

$$= 0.12 * 2.5 + 0.15 + 0.4 = 0.85 \; t/m^2$$

$$\# \; W_{main} = W_{Total} * a + O.W = 0.85 * \overset{2*4.59}{9.18} + 0.15 = 7.96 \; t/m$$



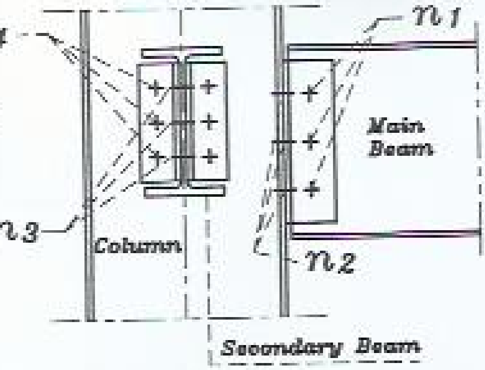
Loads

$$M_X = \frac{7.96 * 8^2}{8} = 63.6 \; m.ton$$

$$Q = \frac{7.96 * 8}{2} = 31.8 \; ton$$

و نلاحظ أن الفرق بين الحلين ليس كبيرا و لكن هذا الحل أوفر بكثير فى الوقت لذلك من الممكن استخدامه فى الامتحان .

Design the marked connection between the marked beams and the column using slip-critical connection and M16 bolts (grade 10.9 with As=1.57 Cm², T=9.89 ton and Ps=3.16 ton)



n_1

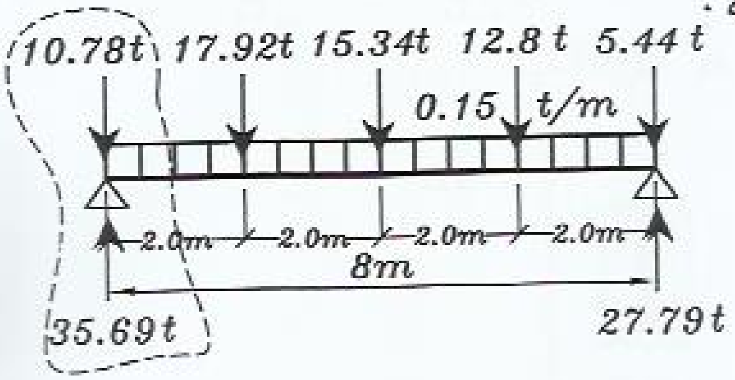
الدكتور ذكر أنها لن تأتي بالامتحان

* Connecting main beam with framing angles
* assume angles 80x80x8
* Double Shear

$$*n_1 = \frac{R_{main}}{2P_S} = \frac{24.90}{2*3.16}$$

$$= 3.94 = \boxed{4\ Bolts}$$

لاحظ أن قيمة الـ R_{main} مطروح منها Reaction أول كمرة Secondary حيث أن حملها يذهب للعمود مباشرة .



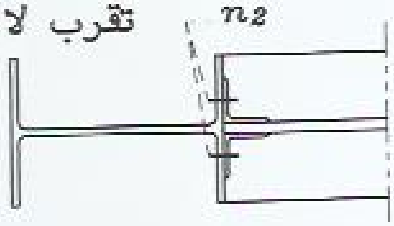
n_2

* Connecting column with framing angles of main beam
* assume angles 80x80x8
* Single Shear

$$*n_2 = \frac{R_{main}}{P_S} = \frac{24.90}{3.16} = 7.86 = \boxed{8\ Bolts}$$

تقرب لاقرب رقم زوجى بالزيادة

من المفترض أن هذا العدد يتم قسمته على اثنين و يوضع فى الناحيتين .



4 Bolts each side

$\underline{n_3}$

* Connecting secondary beam with framing angles
* assume angles 80x80x8
* Double Shear

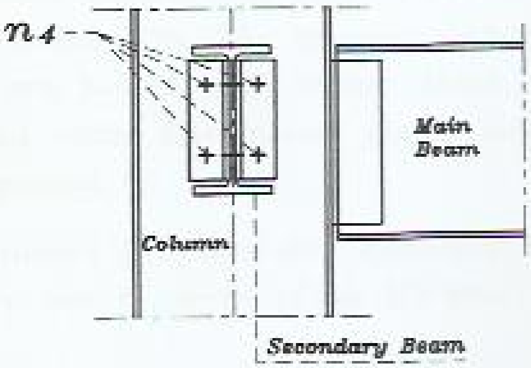
$$* n_3 = \frac{R_{Sec.}}{2P_S} = \frac{5.39}{2*3.16} = 0.85 = \boxed{2\ Bolts}$$

$\underline{n_4}$

* Connecting column with framing angles of secondary beam
* assume angles 80x80x8
* Single Shear

$$* n_4 = \frac{R_{Sec.}}{P_S} = \frac{5.39}{3.16} = 1.71 = \boxed{4\ Bolts}$$

لانه يجب وضع مسمارين على الاقل فى كل ناحية .



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

January 2013 | STEEL STRUCTURES | TIME:3.00 Hrs

Material of construction is steel 37.

Question (1) (28Marks)

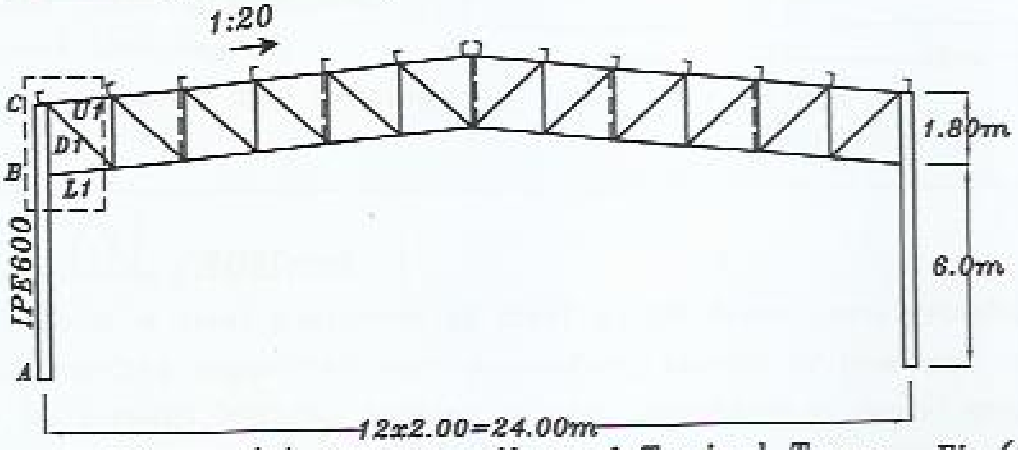


Figure (1): Cross section of Typical Truss Fig.(1)

Figure (1) shows the main supporting system for an industrial building with CL. dimensions of 24mx66m. The spacing between main systems is 6.60m. Truss members are bolted with 10mm thick gusset plate using M16 Slip critical pretensioned bolts. The gusset plates are welded to the column joints (B)&(C). it is required to:

1) Design the truss members (L1 and D1) knowing that the maximum forces in these members are F_{L1} =-3t (Case II) and F_{D1} =+9t (Case I) (6marks)

2) For the truss-to-column connection at joints (B and C), design the bolted connection between truss members and the gusset plate [use slip-critical pretensioned bolts M16 grade (8.8), As=1.57cm2, Ps=2.21t, T=7.0t] Member U1 is composed of two angles back-to-back 80x8 (F=-5.0t). Design also the weld between the gusset plates and the column (ignore any force eccentricity from the wleds C.G.). (6marks)

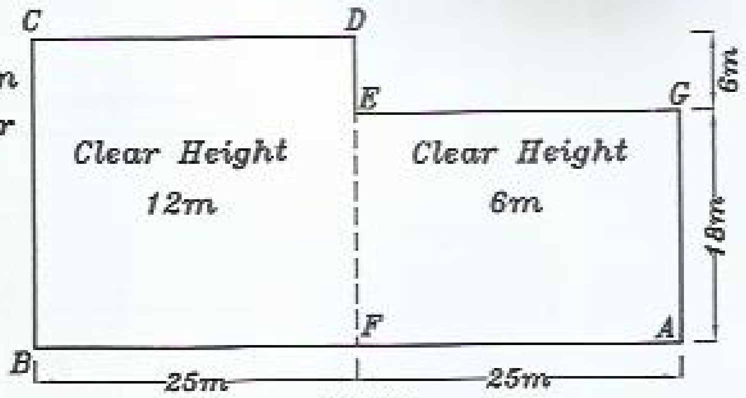
مؤجل للترم الثانى

3) Design a suitable IPE rolled section for the roof purlin. The roof is covered by corrugated sheets of weight 12kg\m2. Roof live load = 55kg\m2 (6marks)

4) Draw to scale 1:10 the part enclosed by the dotted rectangle. Assume reasonable sections for the un-designed elements. (10marks)

Question (2) (12Marks)

A factory building is to be constructed over the area shown in Figure (2). Clear height is 12m for area BCDEF, and is 6m for area AFEG. Columns are allowed along solid lines only. It is required to draw to scale 1:200 a complete general layout showing all structural components and bracing systems.



PLAN

Fig.(2)

Question (3) (30Marks)

Figure (3) shows a steel platform at level +5.00 from zero ground level composed of system supported over secondary beams of channel cross sections, A horizontal bracing system is also provided to resist any lateral loads, as shown in figure. The dead loads of grating including the weight of steel beams and bracings can be assumed 60kg\m2, and live load is 500 kg\m2. It is required to :

1) *Design an intermediate secondary beam ABC using suitable channel (UPN) cross section considering that the laterally unsupported length of cantilever part is 2.5 times the cantilever length and Cb=2.1. The Internal part of the secondary beam is laterally unsupported between braced points of horizontal bracing. Cb = 1.13 (8marks)*

2) *Design the connection enclosed by dotted rectangle at B between secondary beam and main beam of IPE 500 section, using non–pretensioned M16 bolts grade 4.6 (threads included in shear planes), knowing that the secondary beams are flush with main beam. (6marks)*

الدكتور ذكر أنها لن تأتى بالامتحان

3) *Draw to scale 1:10, the designed connection in different views. (8marks)*

4) *Design the marked member in the horizontal bracing system DE using one angle only knowing that the member has a maximum compression force –2 tons and a maximum tension force of +4 tons. (6marks)*

Suggest using neat sketches different connection profile at joint (B) (4marks)

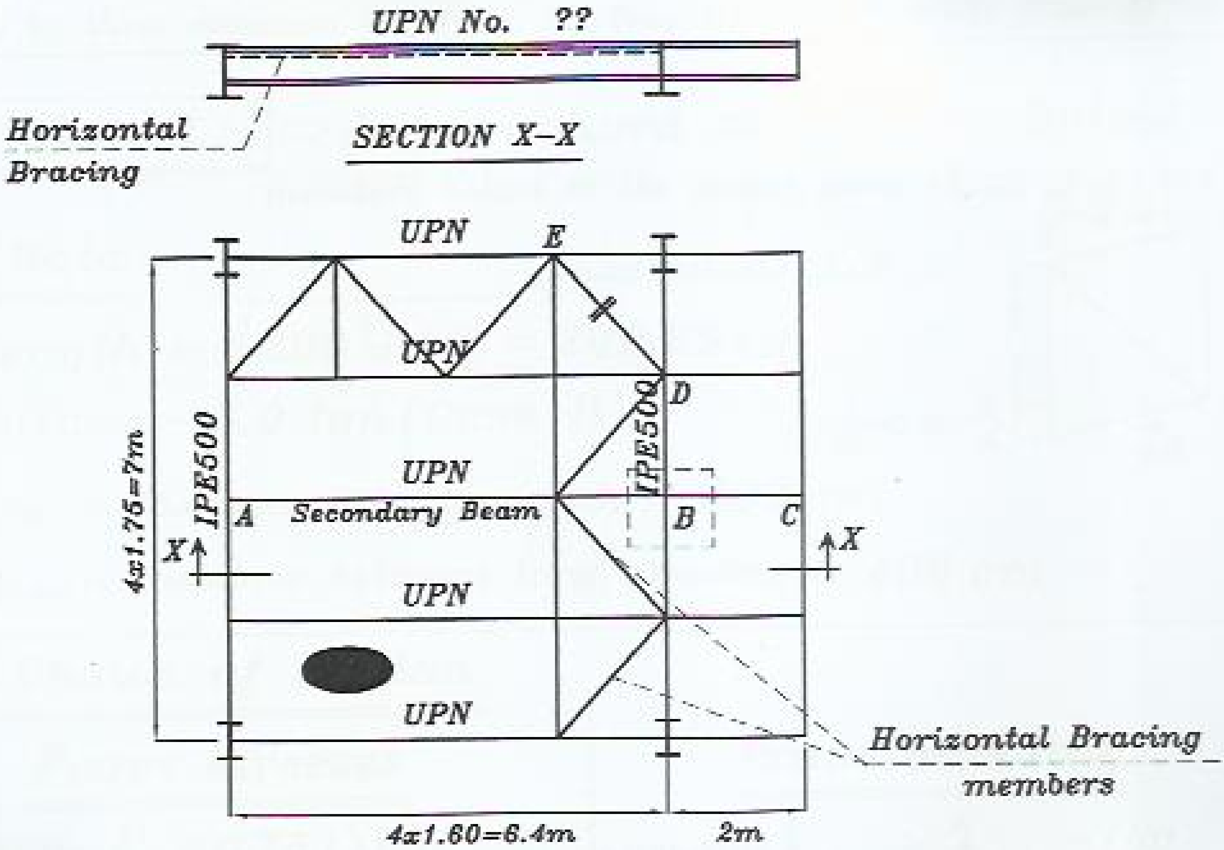


Fig.(3)

Question (1)

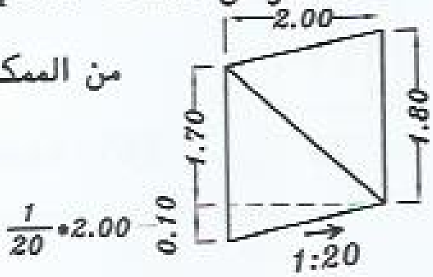
1) Design the truss members (L1 and D1) knowing that the maximum forces in these members are $F_{L1} = -3t$ (Case II) and $F_{D1} = +9t$ (Case I)

Member (L1) ⇨ **Lower chord** ⇨ ⅃L ⇨ **Bolted**

members bolted to the gusset plate لانه ذكر في المسألة

1) Data

من الممكن أخذها 200 للتسهيل



* Length $= \sqrt{200^2 + 10^2} = \mathbf{200.25}$ cm
* Force $= -\mathbf{3.0}$ ton (Case B)
* $l_{b\,in}$ = Distance between joints = **200 cm**
* $l_{b\,out}$ = Distance between long. bracing = **400 cm**

2) Choice of section

From stresses

* assume $F_C = 0.75\ t \backslash cm^2$

$$\therefore A_{g\,⅃L} = \frac{force}{F_C} = \frac{3}{0.75} = 4.0\ cm^2$$

$$\therefore A_{g\,L} = \frac{A_{g\,⅃L}}{2} = \frac{4.0}{2} = 2.0\ cm^2$$

Choose $\xrightarrow{tables}$ L 35*35*3

From buckling

* assume $\lambda_{out} = \lambda_{in} = 100$

$$\therefore 100 = \frac{l_{bin}}{r_x} = \frac{200}{0.30 a_2}$$

$$\Rightarrow a_2 = 6.67 cm$$

$$\therefore 100 = \frac{l_{bout}}{r_y} = \frac{400}{0.45 a_3}$$

$$\Rightarrow a_3 = \boxed{8.88\ cm}$$

$$a_{av} = \frac{a_1 + (a_2 \text{ Or } a_3)}{2} = \frac{3.5 + 8.88}{2} = 6.2\ cm$$

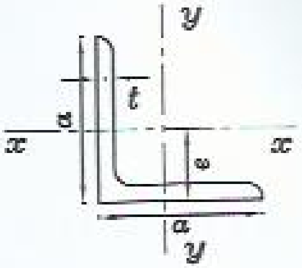
(الاكبر)

From Construction

> minimum angle $a_{min.} = 1.1*3\,\phi = 1.1*3*1.6 = 5.28\ cm$

Choose ⅃L 70*70*7

3) Checks



∟ 70*70*7

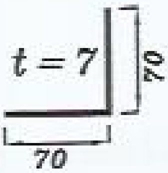
$A = 9.4\ Cm^2$

$e = 1.97\ Cm$

$r_x = r_y = 2.12\ Cm$

$r_v = 1.37\ Cm$

a) Class of section



$$* \frac{b}{t} = \frac{70}{7} = 10 < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$$

⟹ The section is non-compact (Code page 12)

b) Buckling (Slenderness)

$r_{x_{⅃L}} = r_{x_L}$ من الجدول $= 2.12\ cm$

assume $t_{Gp} = 1\ cm$

$$r_{y_{⅃L}} = \sqrt{r_{y_L}^2 + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.12^2 + (1.97 + \frac{1.0}{2})^2} = 3.26\ cm$$

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x_{⅃L}}} = \frac{200}{2.12} = 94.33 < 180 \Longrightarrow \text{(Safe)}$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{y_{⅃L}}} = \frac{400}{3.26} = 122.7 < 180 \Longrightarrow \text{(Safe)}$$

c) Stress

$$\lambda_{max.} = 122.7 > 100$$

$$* \underset{\text{CASE (B)}}{F_C} = 1.2 * \frac{7500}{\lambda^2_{max.}} = 1.2 * \frac{7500}{122.7^2} = 0.60\ t \backslash cm^2$$

$$* f_C = \text{actual stress} = \frac{force}{2 * A_{gL}} = \frac{3}{2 * 9.40} = 0.16\ t \backslash cm^2$$

$$\leqslant F_C \Longrightarrow \text{(Safe)}$$

$$* \frac{f_C}{F_C} = \frac{0.16}{0.60} = 0.27 \Longrightarrow \text{(Safe but waste)}$$

But can not be changed due to buckling condition

Design of tie plate

$\lambda_v \leqslant \lambda_{max.}$

$\frac{l^{\backprime}}{r_{v\llcorner}} = \frac{l^{\backprime}}{1.37} \leqslant 122.7 \Rightarrow l^{\backprime} \leqslant 1.37 * 122.7 = 168.1\ cm$

$l^{\backprime} \leqslant 168.1\ cm$

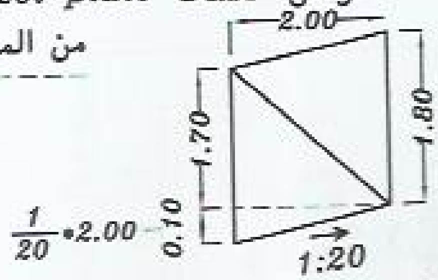
$l > l^{\backprime} > \frac{l}{2}$ ⇨ Use one tie plate at the middle of member

---

Member (D1) ⇨ **Diagonal** ⇨ ∟ ⇨ **Bolted**

members bolted to the gusset plate لانه ذكر فى المسألة

من الممكن أخذها 180 للتسهيل



1) Data

* $Length = \sqrt{200^2 + 170^2} = \mathbf{262}\ cm$
* $Force = +\mathbf{9}\ ton\ (Case\ A)$
* $l_{b\,in}$ = Distance between joints = **262** cm   لا نحتاج الى حسابها
* $l_{b\,out} = 1.2 * 262 = \mathbf{314.4}\ cm$

لان ال Lower Chord معرض ل Compression وليس Tension و بالتالى تكون ال Joint السفلية لل member عبارة عن Weak Joint و بالتالى يكون Strong-weak member

2) Choice of section

a - From Stress Condition

$$A_{g\,\llcorner} = \frac{Force\ (ton)}{0.65 * F_t\ (t\backslash cm^2)} = \frac{9.0}{0.65 * 1.4} = 9.89\ cm^2$$

Unsymmetric Bolted

من الجدول ⟹ **Choose ∟ 75*75*7** $a_1 = 7.5\ cm$

– From Slenderness Condition

assume $\lambda_{out} = 300$

$$\therefore 300 = \frac{l_{b\,out}}{r_{v_L}} = \frac{314}{0.20 a_2} \Longrightarrow a_2 = 5.24 \text{ cm}$$

C – From Construction Condition

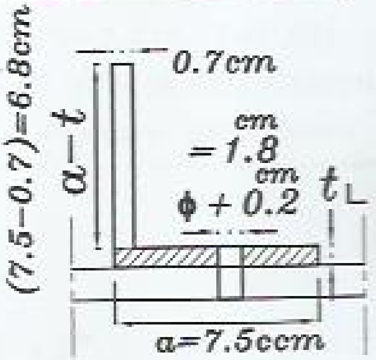
minimum angle $a_{min} = a_3 = 1.1 * 3\,\phi = 1.1 * 3 * 1.6 = 5.28 \text{ cm}$

ثم نختار من الجداول الـ Angle الاكبر من a_1 & a_2 & a_3

Choose $\llcorner$ 75 * 75 * 7

---

3) Checks



$$A_1 = [a - (\phi + 0.2\text{ cm})] * t_L$$

$$= [7.5 - (1.6 + 0.2)] * 0.7 = 3.99 \text{ cm}^2$$

$$A_2 = [a - t_L] * t_L$$

$$= [7.5 - 0.7] * 0.7 = 4.76 \text{ cm}^2$$

$$A_{net} = A_1 + A_2 \left[\frac{3A_1}{3A_1 + A_2}\right] = 3.99 + 4.76 * \left[\frac{3 * 3.99}{3 * 3.99 + 4.76}\right]$$

$$= 7.40 \text{ cm}^2$$

a) Stress

$$f_t = \frac{Force}{A_{net}} = \frac{Force}{A_L} = \frac{9.0}{7.40} = 1.22 \text{ t}\backslash\text{cm}^2$$

مساحة الـ angle التي تم حسابها

$$\leqslant F_t = 1.40 \text{ t}\backslash\text{cm}^2$$

(Safe)

b) Slenderness (Stiffness)

r_{v_L} = من الجدول = 1.45 cm

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{v_L}} = \frac{314}{1.45} = 217 < 300 \Longrightarrow \text{(Safe)}$$

C) Length to depth ratio. (Deflection)

$$* \frac{L}{d} = \frac{262\ cm}{7.5\ cm} = 34.9 \leqslant 60 \Longrightarrow \text{(Safe)}$$

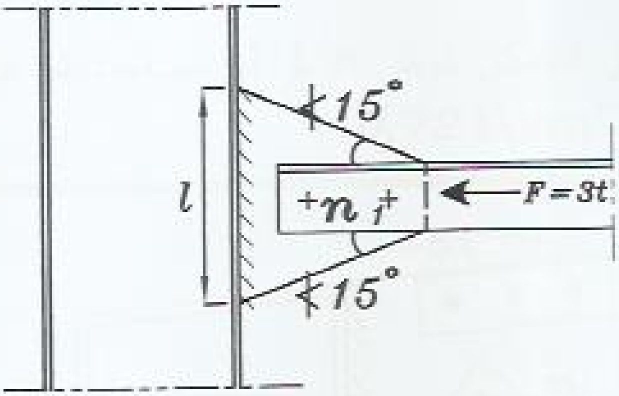
$\Longrightarrow$ **Use ∟ 75*75*7**

---

2) For the truss-to-column connection at joints (B and C), design the bolted connection between truss members and the gusset plate [use slip-critical pretensioned bolts M16 grade (8.8), As=1.57cm2, Ps=2.21t, T=7.0t] Member U1 is composed of two angles back-to-back 80x8 (F=-5.0t). Design also the weld between the gusset plates and the column (ignore any force eccentricity from the wleds C.G.).

Joint (B) Lower Chord

For n_1 ⅃∟70*70*7



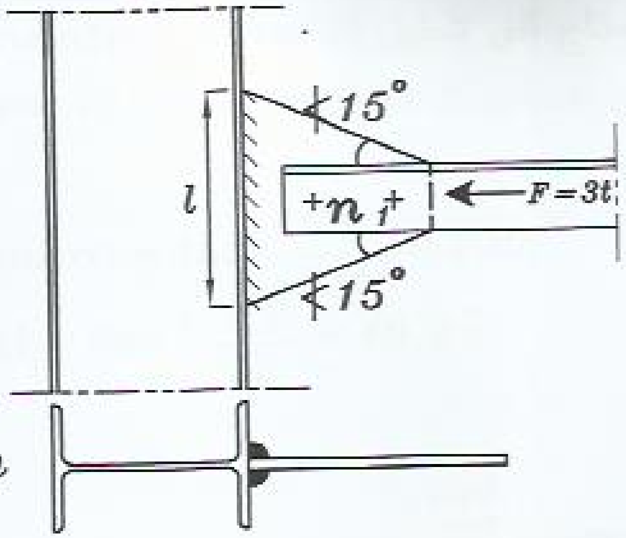
$$P_S = 2.21\ t$$

$$* n_1 = \frac{Force}{2 * P_S * 1.2}$$

CASE (B)

$$= \frac{3}{2 * 2.21 * 1.2} = 0.55 = \boxed{2\ Bolts}$$

Joint (B) : Weld subjected to compression



$$\text{Normal Stress} = f = \frac{F}{A_w}$$

$$= \frac{F}{l * S * 2} \ngtr 0.72\ t\backslash cm^2$$

assume size of weld = 5 mm

$$0.72 = \frac{3.0}{l * 0.5 * 2}$$

$$l_{eff} = 4.17\ cm < l_{eff\ min.} = 5cm$$

$$\boxed{l_{eff.} = 5\ cm}$$

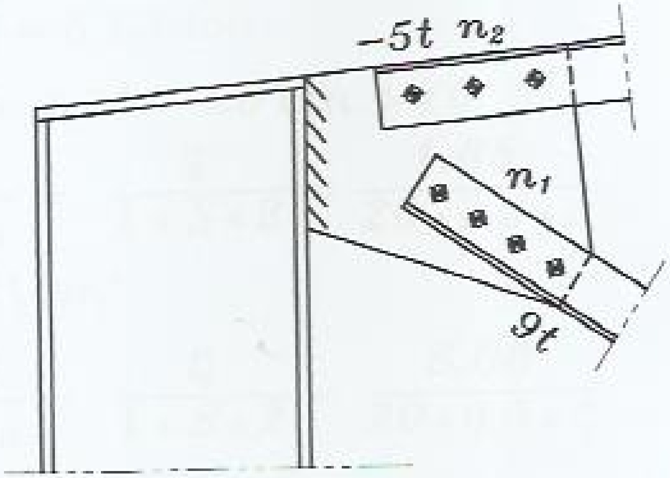
$$* l_{act.} = l_{eff} + 2\ S = 5.0 + 2 * 0.5 = 6.0\ cm$$

⟹ Take $\boxed{l_{act.} = 6\ cm}$

و من المفترض التأكد من أن الزاوية لا تقل عن 15°

و من الممكن فرض الـ l أو حسابها ثم عمل Check على الا تزيد الـ stresses عن $0.72\ t\backslash cm^2$

Joint (C)



For n_1 $\llcorner 75 * 75 * 7$

$$* n_1 = \frac{Force}{2 * P_s}$$

$$= \frac{9}{2.12} = 4.24 = \boxed{5\ Bolts}$$

For n_2 ⅂⌐ $80 * 80 * 8$

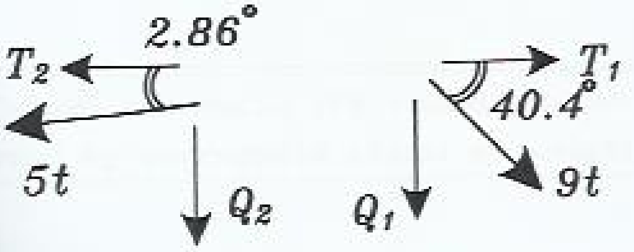
$$* n_2 = \frac{Force}{2 * P_s}$$

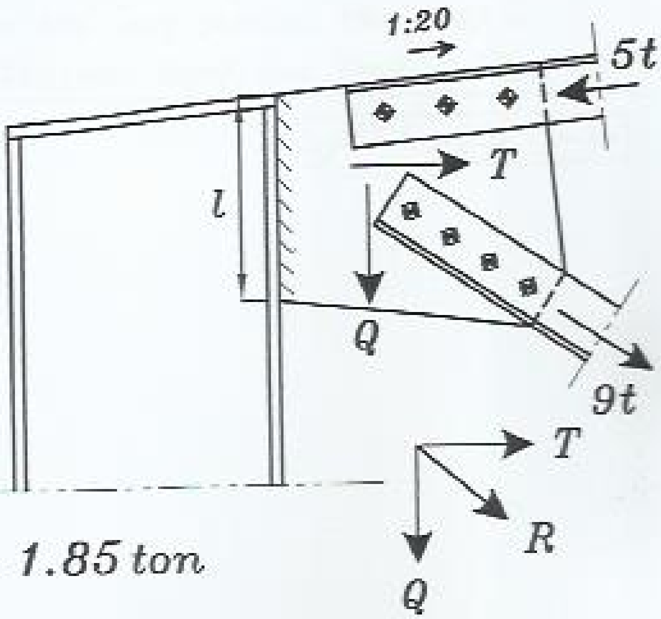
$$= \frac{5}{2 * 2.12} = 1.18 = \boxed{2\ Bolts}$$

Joint (C) : Weld subjected to Shear + Tension

نحتاج الى تحليل الـ *members forces* الى مركبات رأسية و أفقية لكى نتمكن من حساب الـ *Normal stresses + Shear stresses* .

* Inclination of the upper chord $= \tan^{-1}\dfrac{1}{20} = 2.86^\circ$
* Inclination of the diagonal $= \tan^{-1}\dfrac{1.7}{2} = 40.4^\circ$





* $Q_1 = 9 \ Sin\, 40.4 = 5.83 \ ton$
* $T_1 = 9 \ Cos\, 40.4 = 6.85 \ ton$
* $Q_2 = 5\, Sin\, 2.86 = 0.25 \ ton$
* $T_2 = 5\, Cos\, 2.86 = 5.0 \ ton$
* $T = T_1 - T_2 = 6.85 - 5.0 = 1.85\ ton$
* $Q = Q_1 + Q_2 = 5.83 + 0.25 = 6.08\ ton$
* assume size of weld = 5 mm & $l = 20\ cm \ngtr 70\ S$
* Normal Stress $= f = \dfrac{T}{A_w} = \dfrac{T}{l * S * 2} = \dfrac{1.85}{20 * 0.5 * 2}$

$$= 0.093\ t \backslash cm^2$$

* Shear Stress $= q = \dfrac{Q}{A_w} = \dfrac{Q}{l * S * 2} = \dfrac{6.08}{20 * 0.5 * 2}$

$$= 0.304\ t \backslash cm^2$$

Check using interaction equation

$$\sqrt{f^2 + 3\,q^2} \ngtr 0.2\,F_u * 1.1$$

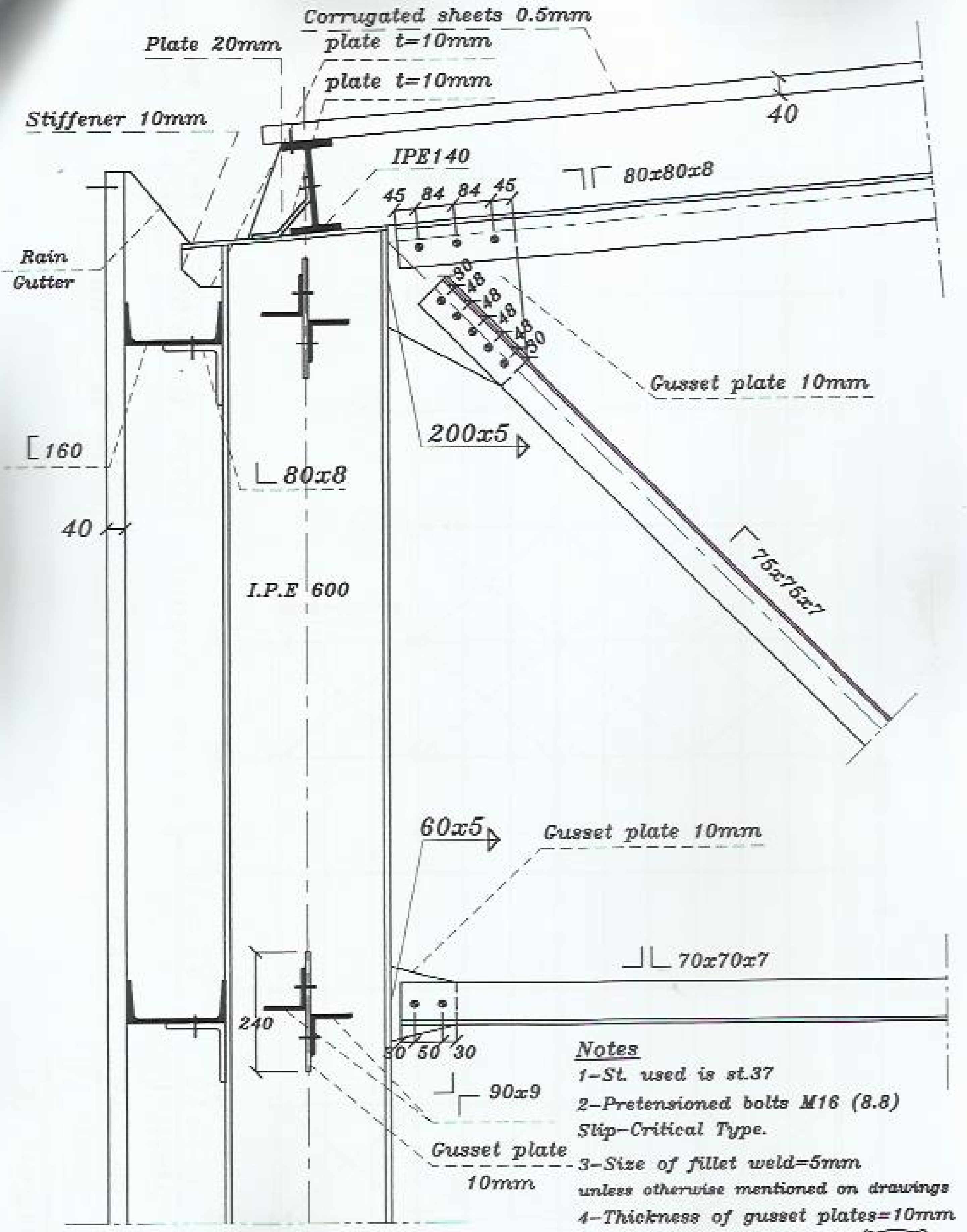
$$= \sqrt{(0.093)^2 + 3\,(0.304)^2} = 0.53\ t\backslash cm^2$$

$$> 0.72 * 1.1\ t\backslash cm^2$$

$\Longrightarrow$ ***Safe***

3) Design a suitable IPE rolled section for the roof purlin. The roof is covered by corrugated sheets of weight 12kg\m2. Roof live load = 55kg\m2

مؤجل للترم الثانى



Notes

1-St. used is st.37

2-Pretensioned bolts M16 (8.8) Slip-Critical Type.

3-Size of fillet weld=5mm unless otherwise mentioned on drawings

4-Thickness of gusset plates=10mm

5-Dims are in mms

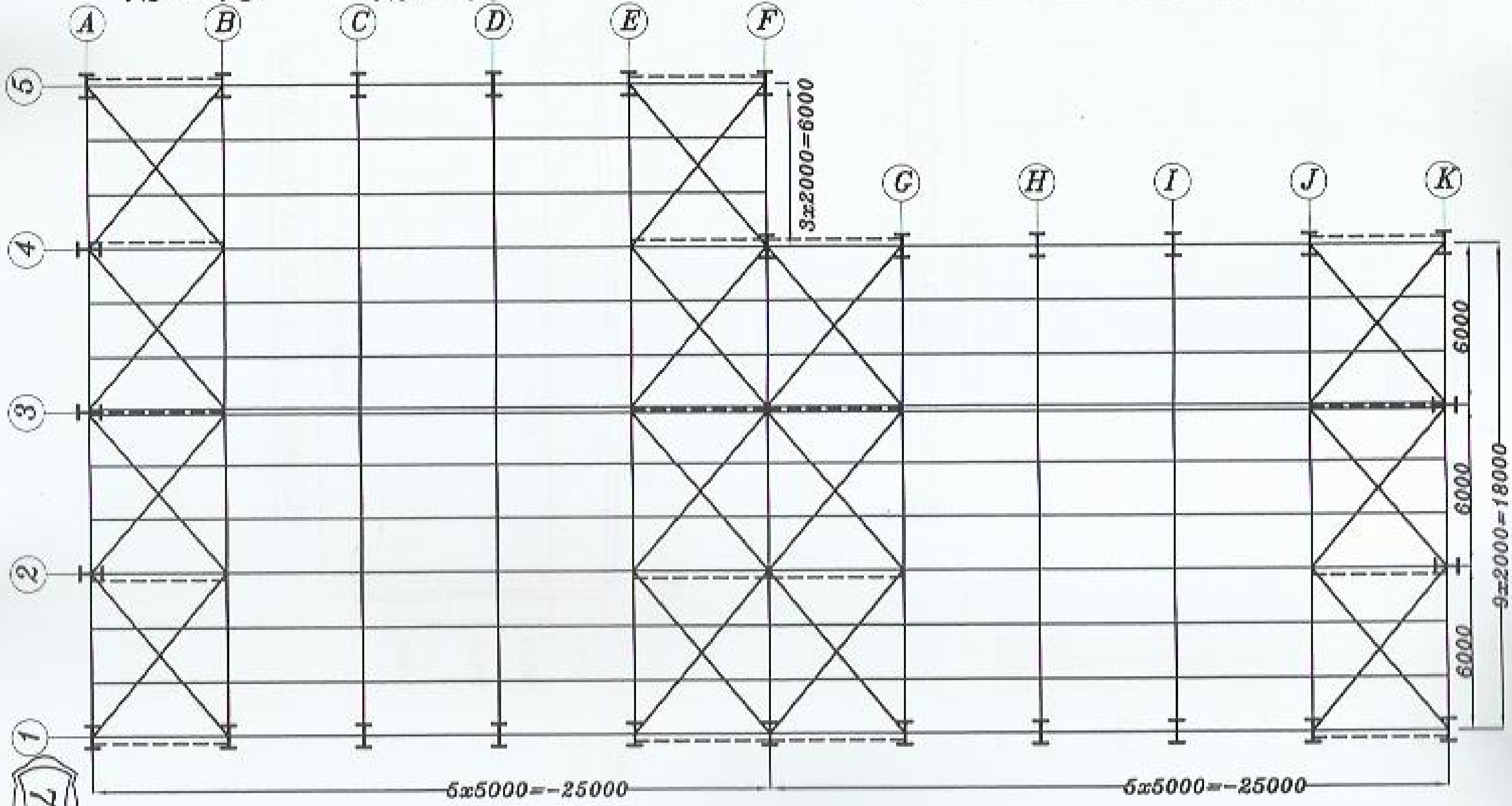
Question (2)

$$H_1 = \frac{Span\ (B)}{12 \Rightarrow 16} = \frac{24}{12 \Rightarrow 16} = 1.50 \Rightarrow 2.00m$$

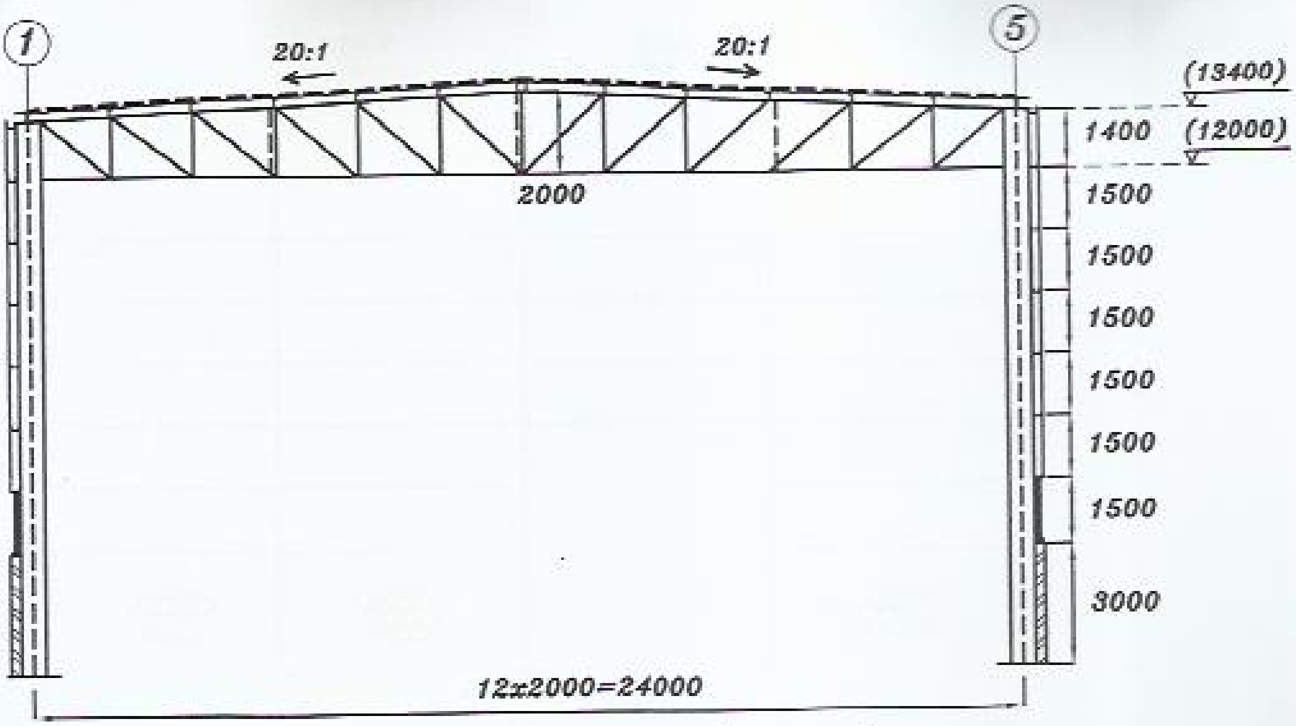
Take $H = 2.00\ m$

$$H_2 = \frac{Span\ (B)}{12 \Rightarrow 16} = \frac{18}{12 \Rightarrow 16} = 1.12 \Rightarrow 1.50m$$

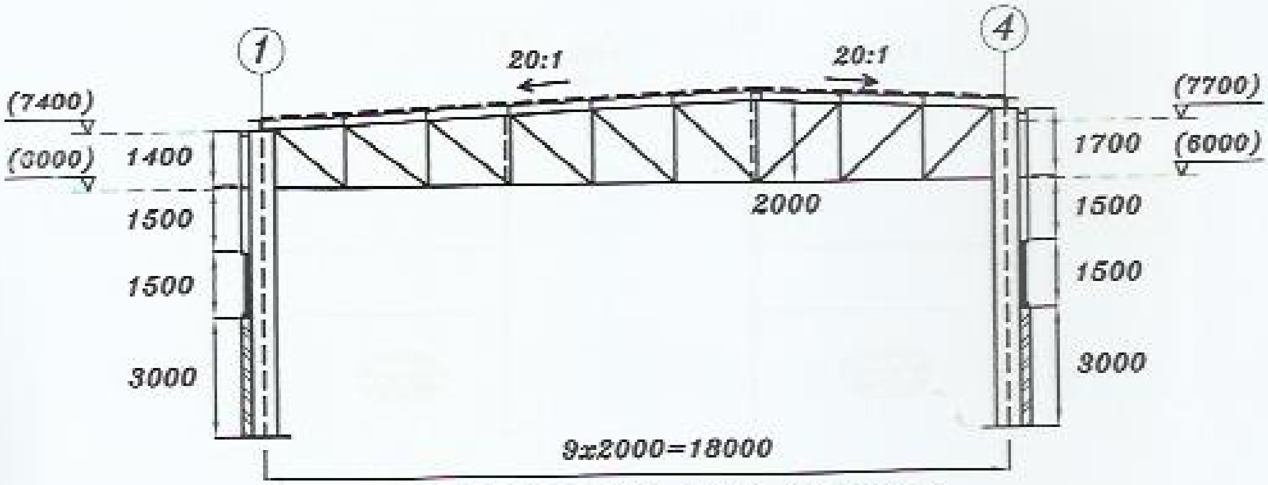
Take $H = 2.00\ m$ for h min not to be less than 1.25m



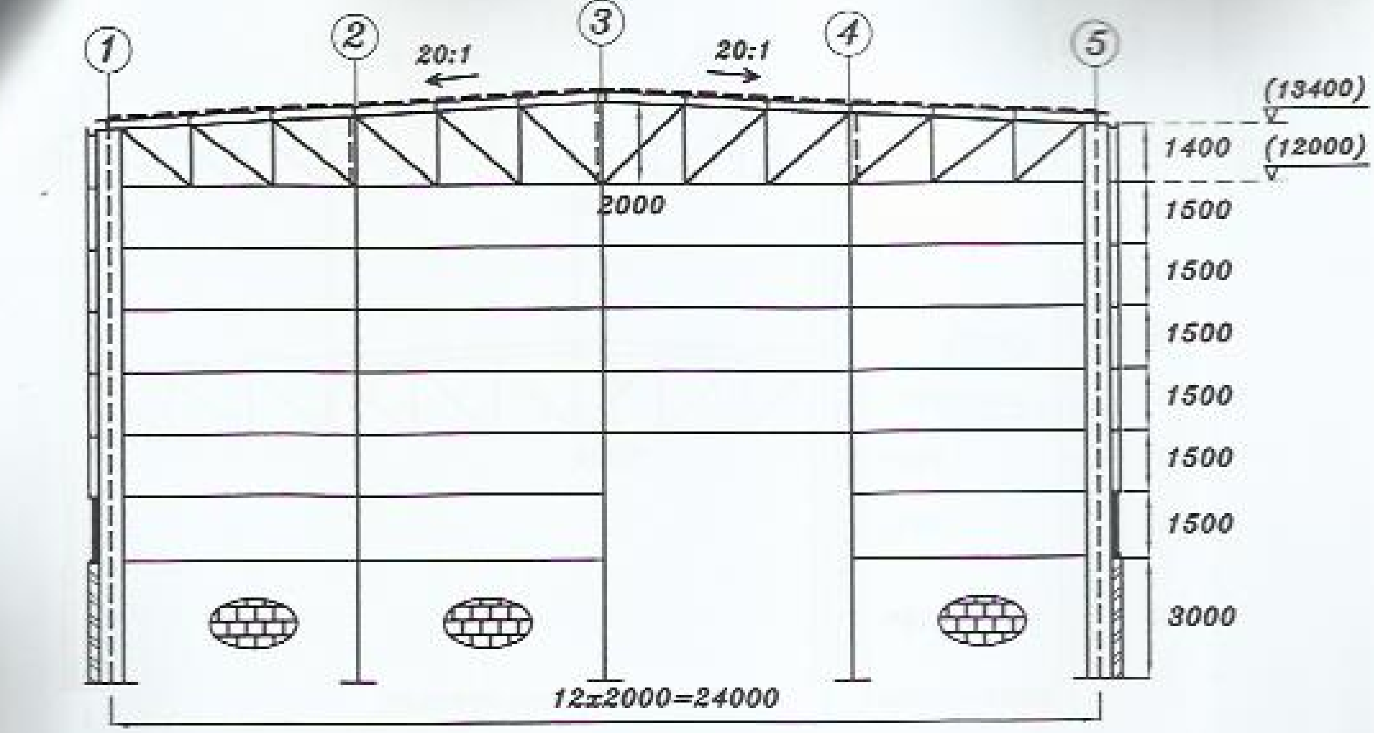
PLAN



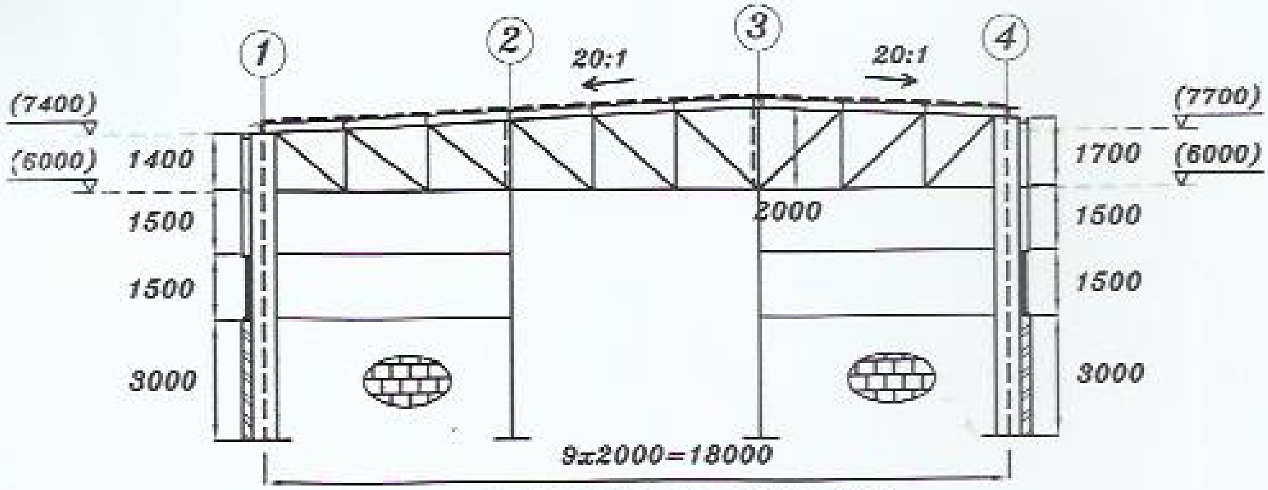
MAIN SYSTEM AT AXES (B,C,D,E)



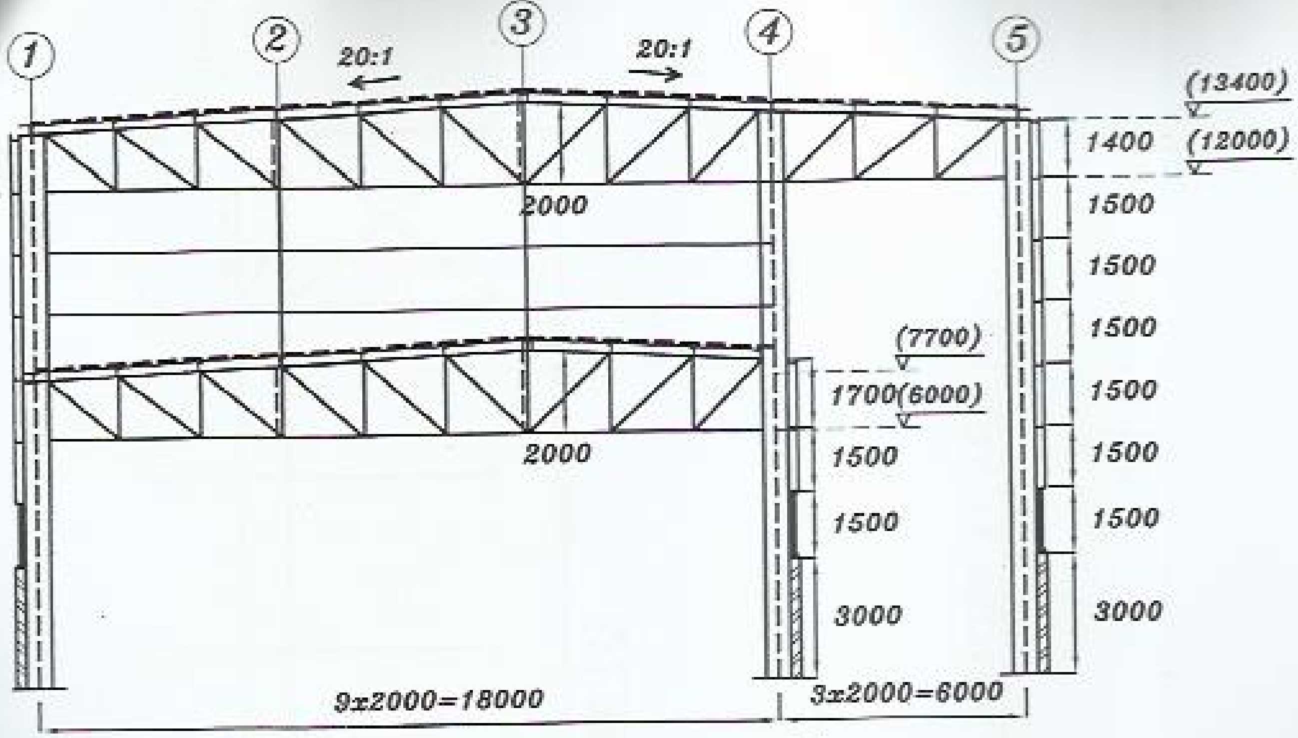
MAIN SYSTEM AT AXES (G,H,I,J)



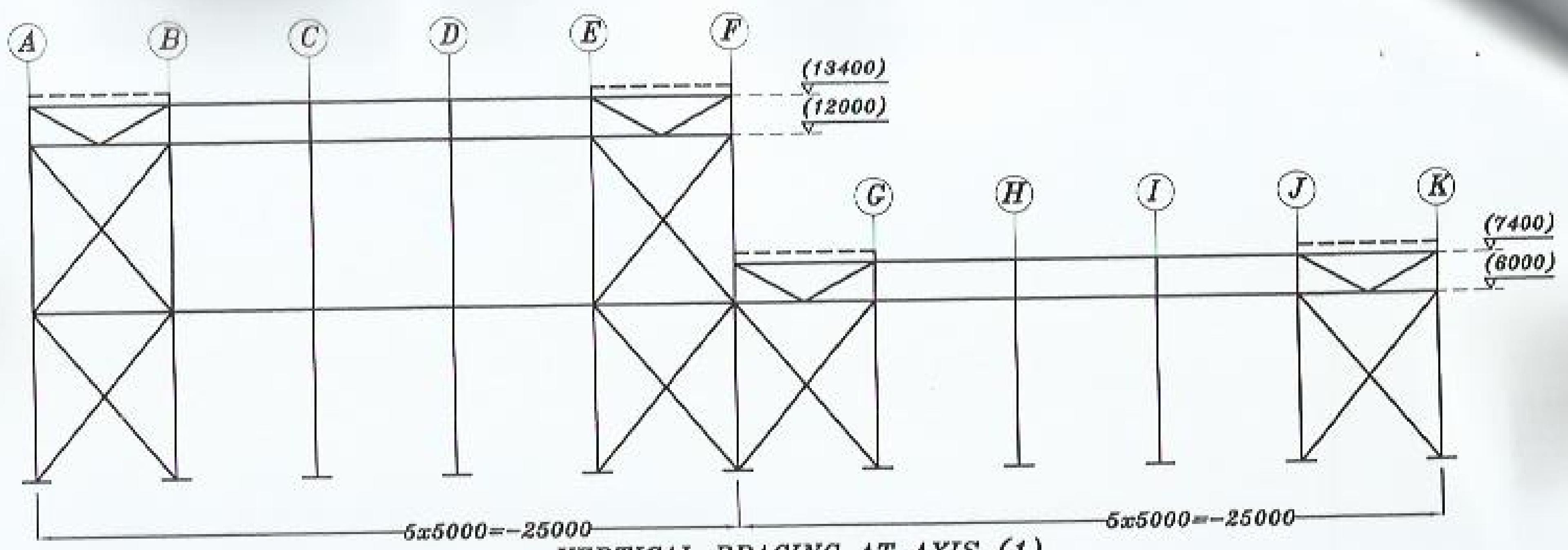
END CABLE AT AXIS (A)



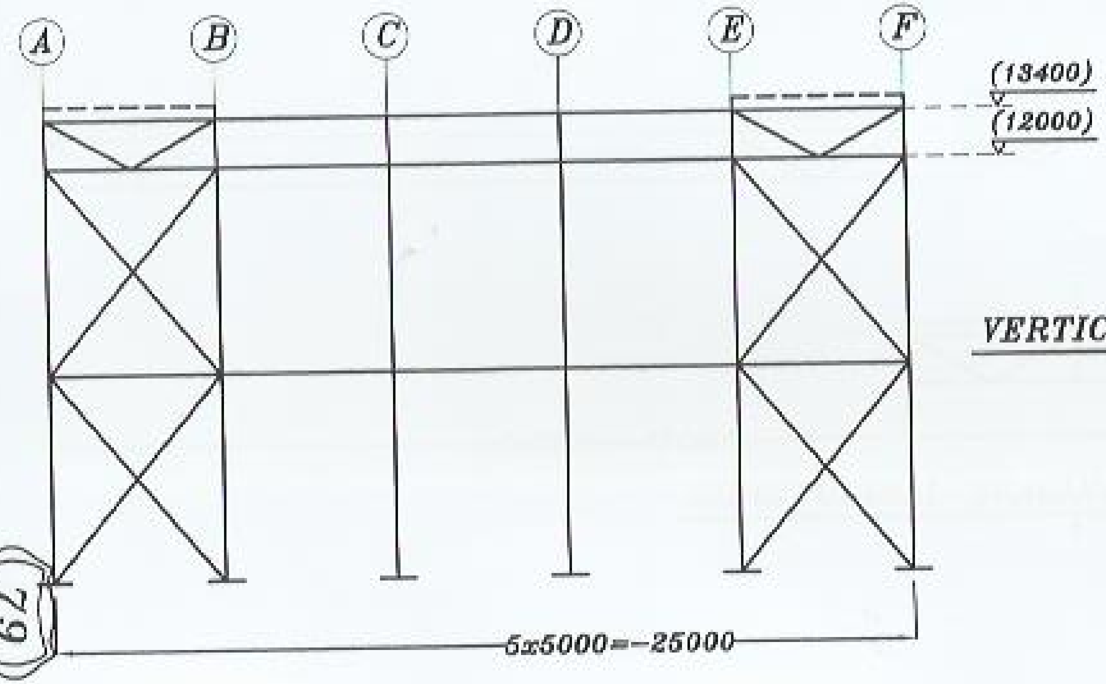
END GABLE AT AXIS (K)



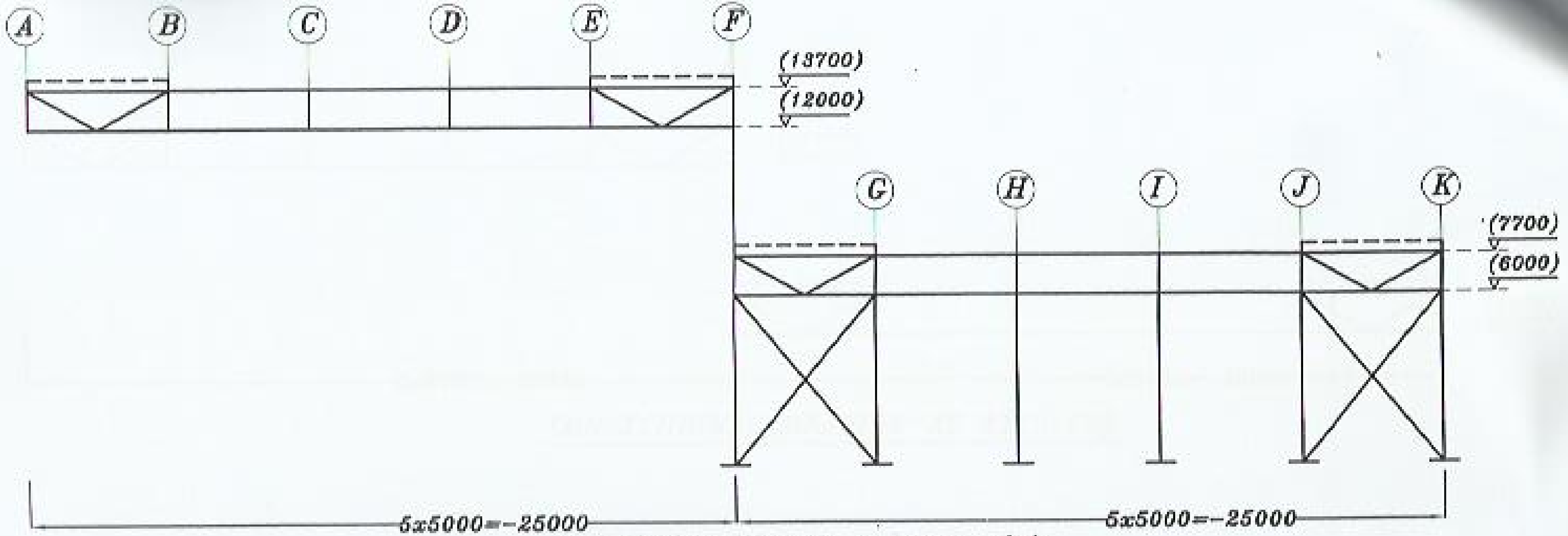
MAIN SYSTEM AT AXIS (F)



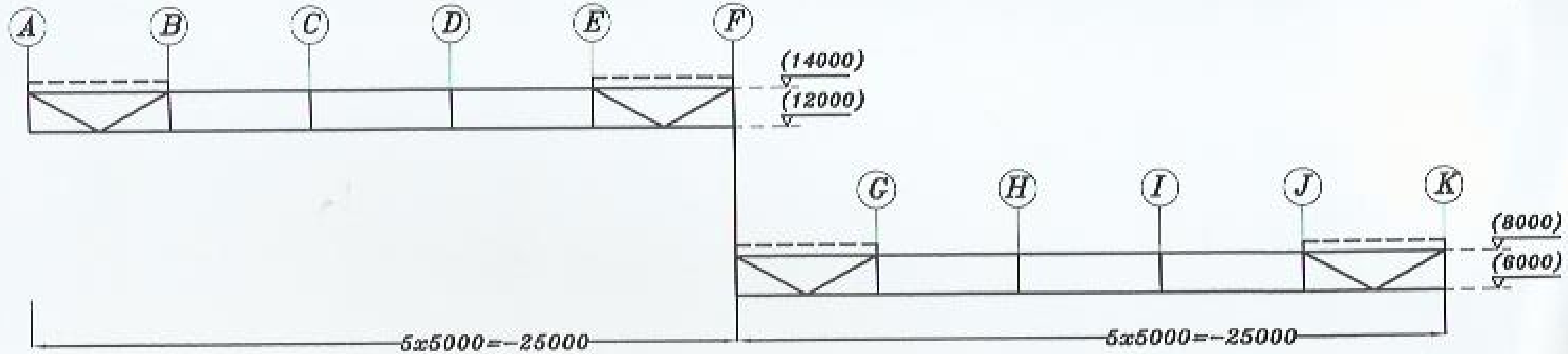
VERTICAL BRACING AT AXIS (1)



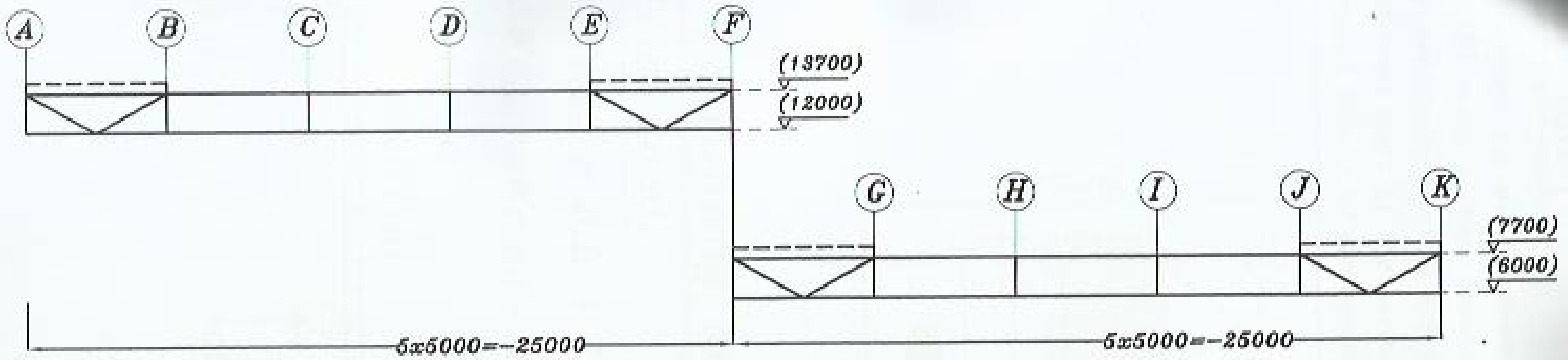
VERTICAL BRACING AT AXIS (5)



VERTICAL BRACING AT AXIS (4)



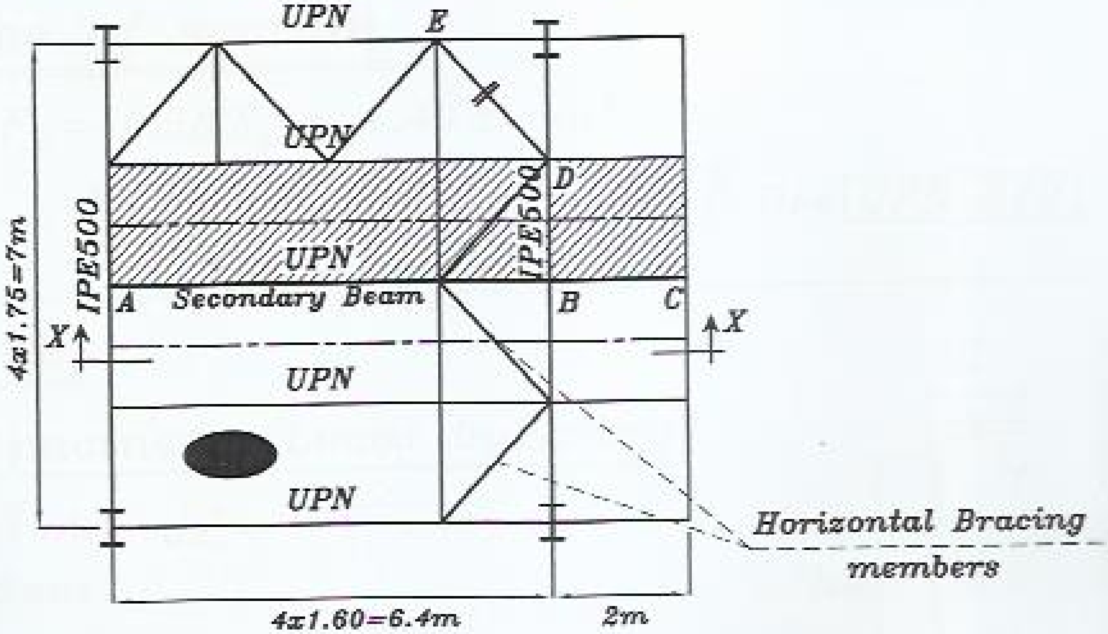
LONGITUDINAL BRACING AT AXIS (3)



LONGITUDINAL BRACING AT AXIS (2)

Question (3)

1) Design an intermediate secondary beam ABC using suitable channel (UPN) cross section considering that the laterally unsupported length of cantilever part is 2.5 times the cantilever length and Cb=2.1. The Internal part of the secondary beam is laterally unsupported between braced points of horizontal bracing. Cb = 1.13 (8marks)

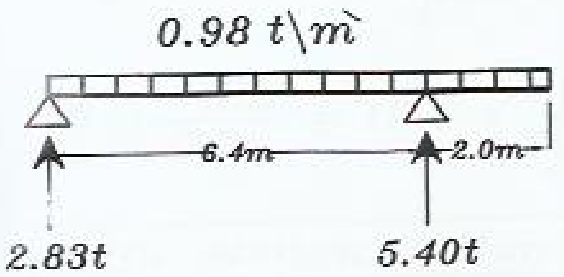


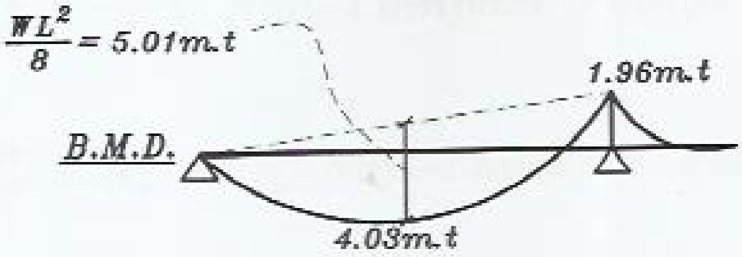
1- Straining actions

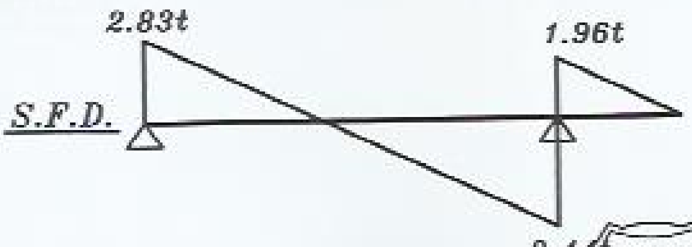
Total Load

$$\# \; W_{Total} = D.L. + L.L = 60 + 500 = 560\ kg/m^2 = 0.56\ t/m^2$$

$$\# \; W_{Sec.} = W_{Total} * a = 0.56 * 1.75 = 0.98\ t/m$$







$M_{Total\ Section\ (1)} = 4.03\ m.t$

نختار القطاع على أساس أكبر عزم مؤثر على الكمرة

$M_{Total\ Section\ (2)} = 1.96\ m.t$

$Q_{Total} = 3.44\ t$

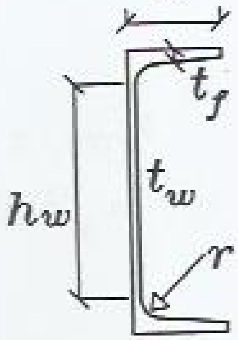
2- Choice of section

assume $F_b = 0.58\, F_y = 1.40\ t \backslash cm^2$

$$S_X = \frac{M_X}{1.40} = \frac{4.03 * 100}{1.40} = 287.9\, Cm^3 \xrightarrow{Tables} \text{Use } \boxed{UPN\ 240}$$

3- Checks

3a-Compactness (Local buckling)



$h_w = 18.5\, cm$ جداول

$t_w = 0.95\, cm$

$b_f = 8.5\, cm$

$t_f = 1.30\, cm$

$r = 1.30\, cm$

$$\frac{C}{t_f} = \frac{b_f}{t_f} = \frac{8.5}{1.30} = 6.54$$

$$\therefore \frac{C}{t_f} = 6.54 < \frac{21.0}{\sqrt{f_y}} = 13.5 \Longrightarrow \text{Non-Compact Flange}$$

$$\frac{h_w}{t_w} = \frac{18.5}{0.95} = 19.5 < \frac{190}{\sqrt{f_y}} = 82 \Longrightarrow \text{Non-Compact Web}$$

$\boxed{\therefore \text{The section is Non-Compact}}$

3-Lateral Torsional Buckling

Section (1) $C_b = 1.13$ (Given)

$L_{U\,act.} = 480$ cm (braced at hz. bracing only)

$$L_{U\,max.} = \begin{cases} \dfrac{20\, b_f}{\sqrt{f_y}} = \dfrac{20 * 8.5}{\sqrt{2.4}} = 109.7 \text{ cm} \\ \dfrac{1380\, A_f}{d * F_y} C_b = \dfrac{1380 * (8.5 * 1.30)}{24 * 2.4} * 1.13 = 265 \text{ cm} \end{cases}$$

نأخذ الاصغر

$$\boxed{L_{U\,max.} = 109.7 \text{ cm}}$$

$L_{U\,act.} = 480 \text{ cm} > L_{U\,max.} = 109.7 \text{ cm} \Longrightarrow$ LTB Occurs

$$F_b = F_{ltb_1} \leqslant 0.58\, F_y$$

$$F_{ltb_1} = \frac{800 * A_f}{l_u * d} C_b = \frac{800 * (8.5 * 1.3)}{480 * 24} * 1.13 = 0.867 \ t \backslash cm^2$$

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

$$\boxed{F_{bC} = 0.867\ t \backslash cm^2}$$

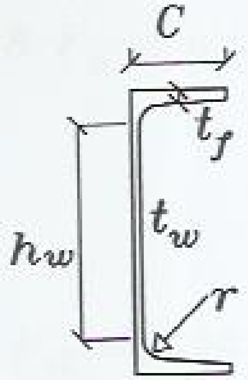
3a-Check bending stresses

$$f_{act.} = \frac{M_X}{S_X} = \frac{4.03 * 100}{300} = 1.34\ t \backslash cm^2 > F_{bC} = 0.867$$

(Unsafe)

Try **UPN 280**

3a-Compactness (Local buckling)



$h_w = 21.3$ cm جداول

$t_w = 1.00$ cm

$b_f = 9.5$ cm

$t_f = 1.50$ cm

$r = 1.50$ cm

$$\frac{C}{t_f} = \frac{b_f}{t_f} = \frac{9.50}{1.50} = 6.33$$

$$\therefore \frac{C}{t_f} = 6.33 < \frac{21.0}{\sqrt{f_y}} = 13.5 \Longrightarrow \text{Non-Compact Flange}$$

$$\frac{hw}{t_w} = \frac{21.3}{1.0} = 21.3 < \frac{190}{\sqrt{f_y}} = 82 \Longrightarrow \text{Non-Compact Web}$$

∴ The section is Non-Compact

b-Lateral Torsional Buckling

Section (1) $C_b = 1.13$ (Given)

$L_{U\,act.} = 480$ cm (braced at hz. bracing only)

$$L_{U\,max.} = \begin{cases} \dfrac{20\, b_f}{\sqrt{f_y}} = \dfrac{20 * 9.5}{\sqrt{2.4}} = 122.1 \text{ cm} \\ \dfrac{1380\, A_f}{d * F_y} C_b = \dfrac{1380 * (9.5 * 1.50)}{28 * 2.4} * 1.13 = 309 \text{ cm} \end{cases}$$

نأخذ الاصغر

$$\boxed{L_{U\,max.} = 122.1 \text{ cm}}$$

$L_{U\,act.} = 480$ cm $> L_{U\,max.} = 122.1$ cm $\Longrightarrow$ LTB Occurs

$$F_b = F_{ltb_1} \leqslant 0.58\, F_y$$

$$F_{ltb1} = \frac{800 * A_f}{l_u * d} C_b = \frac{800 * (9.5 * 1.5)}{480 * 28} * 1.13 = 0.96 \ t\backslash cm^2$$
$$\leqslant 0.58\, F_y = 1.4$$

$$\boxed{F_{bC} = 0.96 \ t\backslash cm^2}$$

3a-Check bending stresses

$$f_{act.} = \frac{M_X}{S_X} = \frac{4.03 * 100}{448} = 0.90 \ t\backslash cm^2 < F_{bC} = 0.96$$
(Safe)

Section (2) $C_b = 2.10$ (Given)

$$L_{U\,act.} = 2.50 * 200 = 500 \ cm$$

$$L_{U\,max.} = \begin{cases} \dfrac{20\, b_f}{\sqrt{f_y}} = \dfrac{20 * 9.5}{\sqrt{2.4}} = 122.1 \ cm \\[2ex] \dfrac{1380\, A_f}{d * F_y} C_b = \dfrac{1380 * (9.5 * 1.50)}{28 * 2.4} * 2.10 = 614 \, cm \end{cases}$$

نأخذ الاصغر

$$\boxed{L_{U\,max.} = 122.1 \ cm}$$

$$L_{U\,act.} = 500 \ cm > L_{U\,max.} = 122.1 \ cm \Longrightarrow LTB \ \ Occurs$$

$$F_b = F_{ltb_1} \leqslant 0.58\, F_y$$

$$F_{ltb1} = \frac{800 * A_f}{l_u * d} C_b = \frac{800 * (9.5 * 1.5)}{500 * 28} * 2.10 = 1.71 \ \ t\backslash cm^2$$

$$> 0.58\, F_y = 1.4$$

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

3a – Check bending stresses

$$f_{act.} = \frac{M_X}{S_X} = \frac{1.96 * 100}{448} = 0.44 \ t\backslash cm^2 < F_{bC} = 0.96$$

(Safe)

3b – Check Shear stresses

$$q_{act.} = \frac{Q}{A_{Web}} = \frac{Q}{h * t_w} = \frac{(3.44)}{28 * 1.0} = 0.12 \ t\backslash cm^2 < 0.35\, F_y = 0.84$$

(Safe)

3C – Check deflection

من المفترض لحساب الـ *deflection* أن نحل الكمرة بالـ *Conjugate beam* نحسب الـ *deflection* كأنها *Simple Beam* وتكون قيمة *deflection* الـ *Continous Beam* هى نفسها للـ *Simple Beam* مضروبة فى 0.8

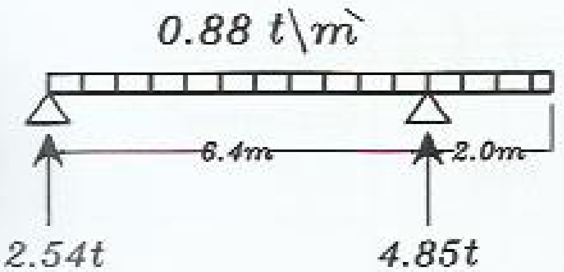
$W_{\underset{L.L}{Sec.}} = L.L * a = 0.50 * 1.75 = 0.88\ t/m$

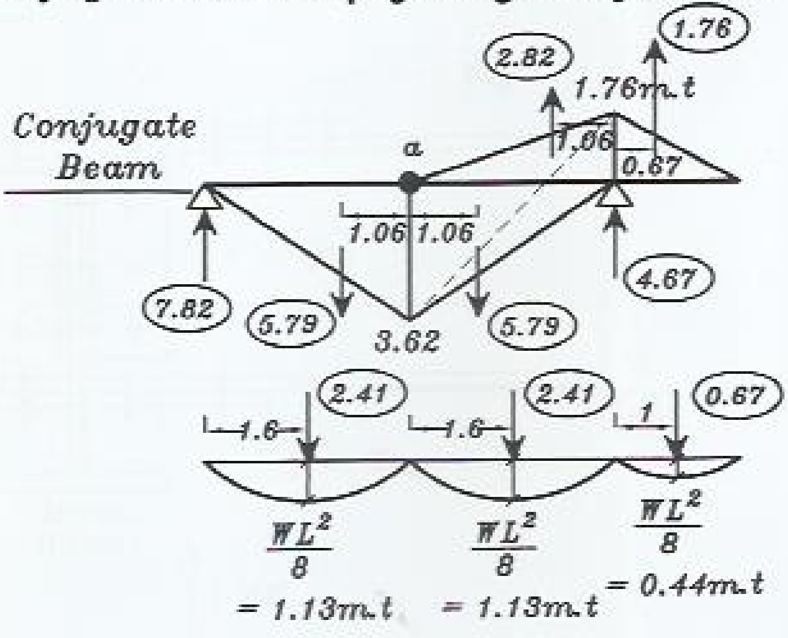
$$\triangle_{act} = 0.8 * \frac{5}{384} * \frac{w_{L.L} * S}{E * I_X} \quad (t\backslash cm,\ cm)$$

$$= 0.8 * \frac{5}{384} * \frac{(0.88\backslash 100) * (640)^4}{2100 * 6280} = 1.16\ cm < \frac{Span}{300}$$

$$(t\backslash cm^2,\ cm^4) \quad < \frac{640}{300} = 2.13\ cm$$

أما اذا اردنا حل الكمرة بالـ *Conjugate beam* فتكون كالتالى





$$M_{elastic} = 7.82 * 3.20 - 2.41 * 1.60 - 5.79 * 1.06 = 15.03\ m^3t$$

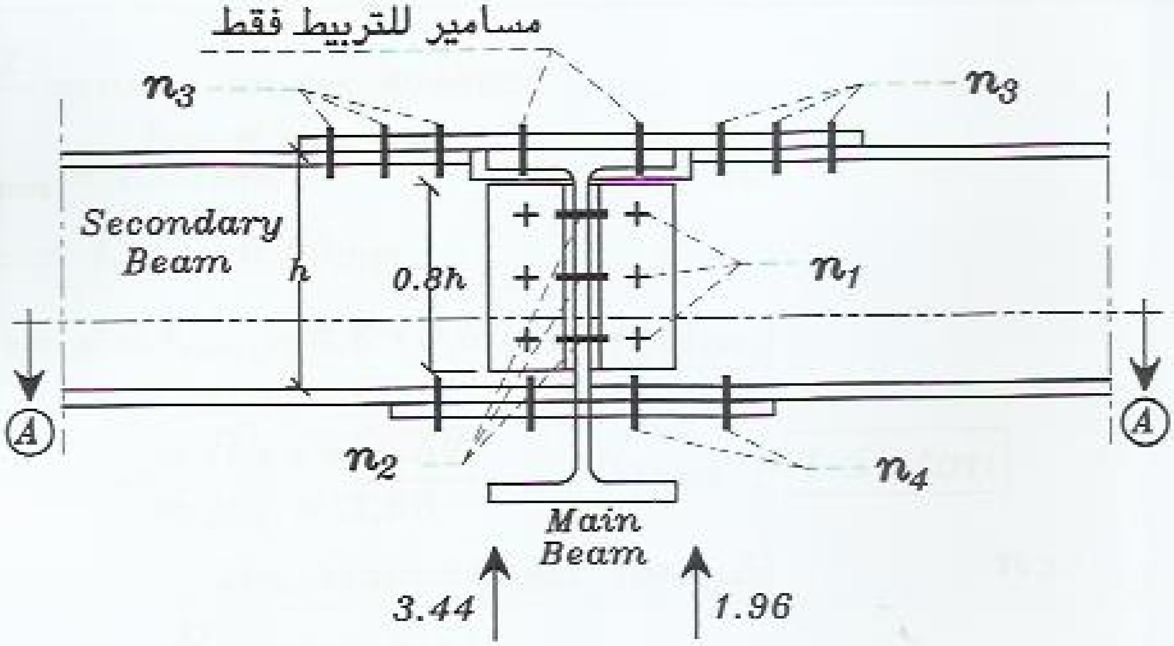
$$\triangle_{act._a} = \frac{15.03}{EI} = \frac{15.03 * 10^6}{2100 * 6280} = 1.14\ cm < \frac{640}{300} = 2.13\ cm$$

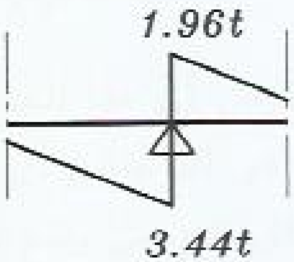
$(m^3t;\ t\backslash cm^2;\ cm^4)$

Design the connection enclosed by dotted rectangle at B between secondary beam and main beam of IPE 500 section, using non-pretensioned M16 bolts grade 4.6 (threads included in shear planes), knowing that the secondary beams are flush with main beam.

الدكتور ذكر أنها لن تأتى بالامتحان

* $F_{ub} = 4\ t \backslash cm^2$
* $\phi = 1.6\ cm$
* $F_u \xrightarrow{\text{for st.37}} = 3.6\ t \backslash cm^2$
* $q_b = 0.25 F_{ub}$
* $A_S = 0.78 * \frac{\pi d^2}{4}$ (Thread included)
* Take $e > 2\phi = 3.2\ cm \Longrightarrow \alpha = 0.8$
* $R_{S.S} = (0.25 F_{ub}) * 0.78 * \frac{\pi d^2}{4} * 1 = (0.25 * 4) * 0.78 * \frac{\pi (1.6)^2}{4}$
  $= 1.57\ ton$
* $R_{D.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 2 = 2 * R_{S.S} = 3.14\ ton$
* $R_b = (\alpha * F_u) * d * t_{min} = 0.8 * 3.6 * 1.6 * t_{min} = 4.6\ t_{min}$





For bolts connecting the secondary beam to the framing angles

$\underline{n_1}$ assume angles 80x80x8

$$* \; t_{min} = \overset{t_{web\ of\ sec.\ beam}}{1.00\,cm} \quad \text{or} \quad \overset{t_{L}}{2*0.8\ cm} = 1.6\ cm$$

$$\Longrightarrow t_{min} = 1.0\ cm$$

$$* \; R_b = 4.6\ t_{min} = 4.6 * 1.0 = \boxed{4.60\ ton}$$

$$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{3.14} \\ R_b = 4.60 \end{cases} \qquad R_{Least} = \boxed{3.14\ ton}$$

Sec. beam للـ Shear force أكبر

$$* \; n_1 = \frac{R_{Sec.}}{R_{Least}} = \frac{3.44}{3.14} = 1.15 = \boxed{2\ Bolts}$$

For bolts connecting the framing angles to the main beam

$\underline{n_2}$ assume angles 80x80x8

$$* \; t_{min} = \overset{t_{web\ of\ main\ beam}}{1.02\,cm} \quad \text{or} \quad \overset{t_{L}}{0.8\ cm}$$

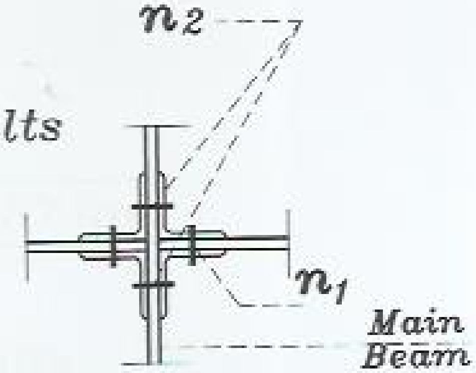
$$\Longrightarrow t_{min} = 0.80\,cm$$

$$* \; R_b = 4.6\ t_{min} = 4.6 * 0.80 = \boxed{3.68\ ton}$$

$$* \; R_{Least} \rightarrow \begin{cases} R_{S.S} = \boxed{1.57} \\ R_b = 3.68 \end{cases} \qquad R_{Least} = \boxed{1.57\ ton}$$

Sec. beam للـ Shear force أكبر

$$* \; n_2 = \frac{R_{Sec.}}{R_{Least}} = \frac{3.44}{1.57} = 2.19 = 4\ Bolts$$



من المفترض أن هذا العدد يتم قسمته على اثنين
و يوضع فى الناحيتين أى مسمار كل ناحية و لكن
على الاقل لابد من وضع مسمارين فى كل ناحية

minimum $\boxed{2\ Bolts}$ each side

For bolts connecting the upper flanges of sec. beams with the upper plate $\boxed{n_3}$

n_3 assume plate thickness = 10 mm

$$* T = C = \frac{M}{h} = \frac{0.75 M_0}{h} = \frac{0.75 * 3.44}{28} = 9.21 t$$

$$* t_{min} = \overset{t_{flange\ of\ sec.\ beam}}{1.50\ cm} \quad or \quad \overset{t_{plate}}{1.0\ cm}$$

$$\Longrightarrow t_{min} = 1.0\ cm$$

$$* R_b = 4.6\, t_{min} = 4.6 * 1.0 = \boxed{4.60\ ton}$$

$$* R_{Least} \rightarrow \begin{cases} R_{s.s} = \boxed{1.57} \\ R_b = 4.60 \end{cases} \qquad R_{Least} = \boxed{1.57\ ton}$$

$$* n_3 = \frac{T}{R_{Least}} = \frac{9.21}{1.57} = 5.86 = 6\ Bolts$$

For bolts connecting the lower flanges of sec. beams with the lower plate $\boxed{n_4}$

n_4 assume plate thickness = 10 mm

$$* T = C = 9.21 t$$

$$* t_{min} = \overset{t_{flange\ of\ sec.\ beam}}{1.50\ cm} \quad or \quad \overset{t_{plate}}{1.0\ cm}$$

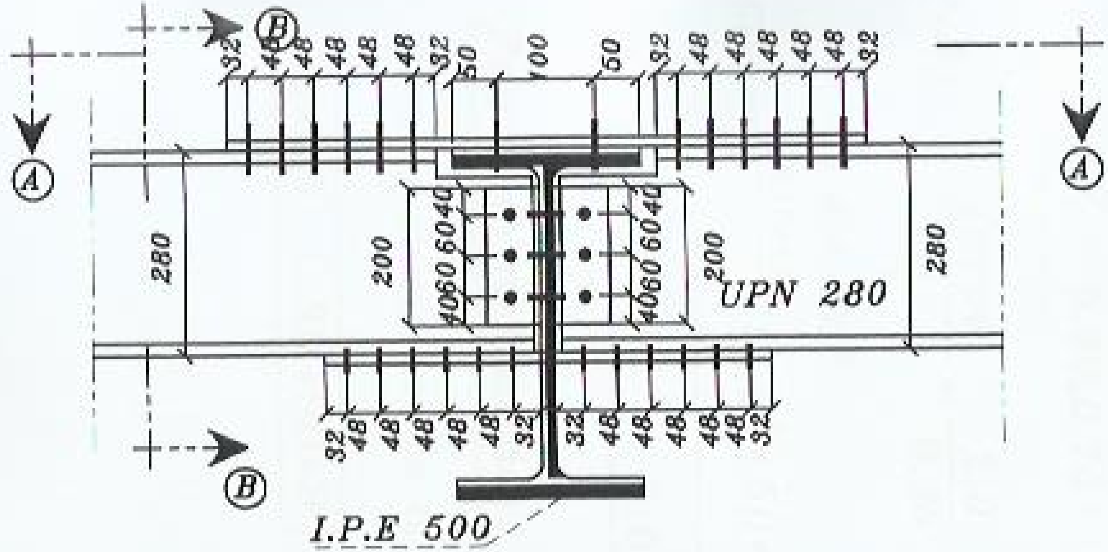
$$\Longrightarrow t_{min} = 1.0\ cm$$

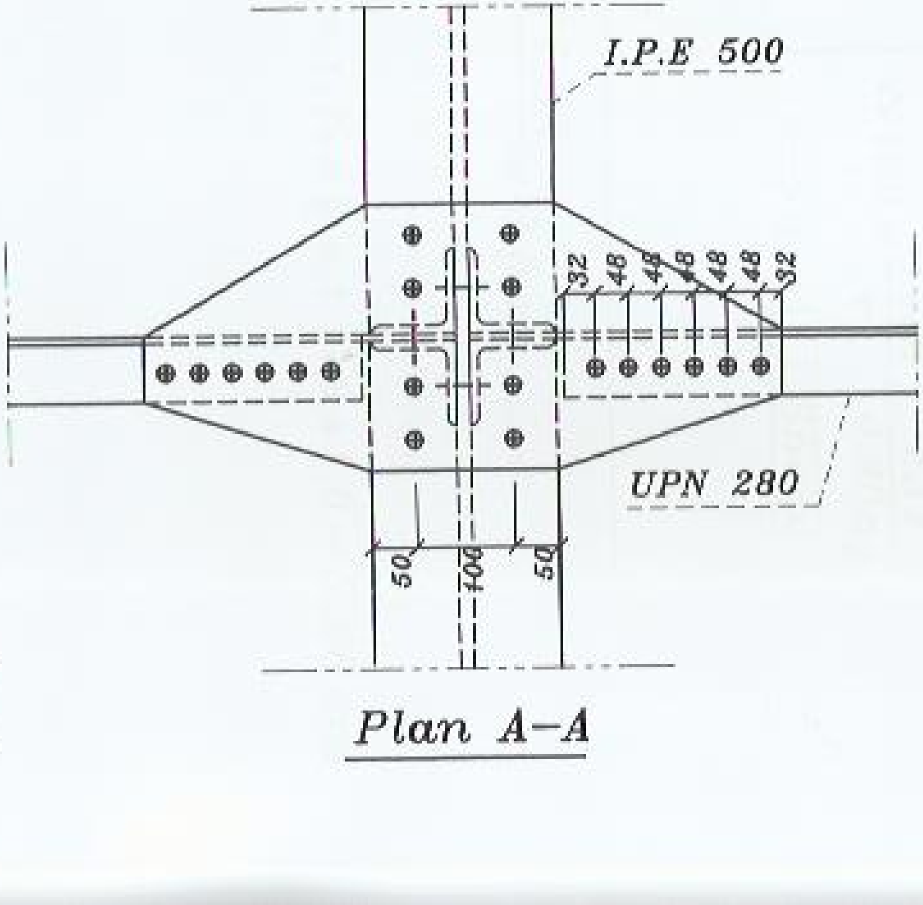
$$* R_b = 4.6\, t_{min} = 4.6 * 1.0 = \boxed{4.60\ ton}$$

$$* R_{Least} \rightarrow \begin{cases} R_{s.s} = \boxed{1.57} \\ R_b = 4.23 \end{cases} \qquad R_{Least} = \boxed{1.57\ ton}$$

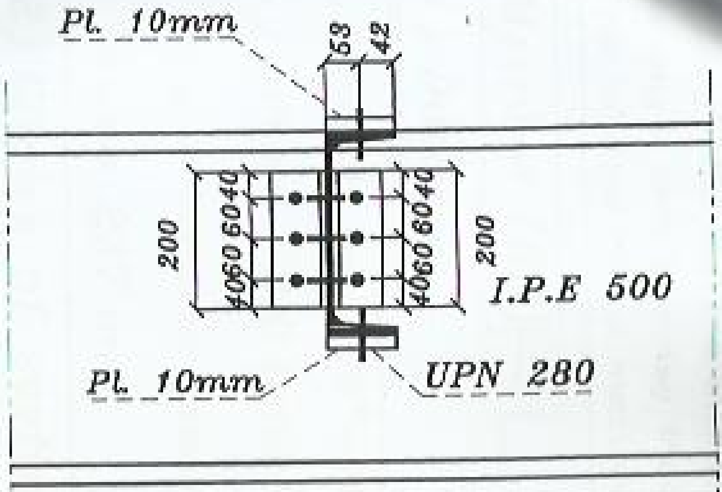
$$* n_4 = \frac{C}{R_{Least}} = \frac{9.21}{1.57} = 5.86 = 6\ Bolts$$

لاحظ أن $\boxed{n_4 = n_3}$





Plan A-A



Sec. B-B

Notes

1-St. used is st.37

2-Non-pretensioned bolts M16 (4.6)

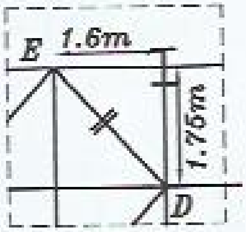
3-Framing angles are 80x80x8

4-All dimensions are in mms

ملحوظة

لا يمكن رص المسامير على صفين فى الـ Channel حيث أن عرض الـ Flange لا يسمح لانه لرص المسامير على صفان نحتاج أن تكون الـ $b_f = 6\phi * 1.1 = 10.56$ و لكن عرض الـ Flange فى الـ UPN 280 يساوى 9.5cm

Design the marked member in the horizontal bracing system DE using one angle only knowing that the member has a maximum compression force −2 tons and a maximum tension force of +4 tons.



Member (DE) ⇨ L ⇨ Bolted

1) Data

* Length $= \sqrt{175^2+160^2} = \mathbf{237}$ cm
* Force $= -\mathbf{2.0}$ ton (Case A) $\quad +\mathbf{4.0}$ ton (Case A)
* $l_{b\,in}$ = Distance between joints = **237** cm
* $l_{b\,out}$ = **237** cm

2) Choice of section

Compression

From stresses

* assume $F_C = 0.60 * 0.75$ t \ cm²

Unsymmetric section

$$\therefore A_{g\,L} = \frac{force}{F_C} = \frac{2.0}{0.45} = 4.4 \text{ cm}^2$$

Choose $\xrightarrow{tables}$ L 50*50*5

From buckling

* assume $\lambda_{out} = 100$

$$\therefore 100 = \frac{l_{b\,out}}{r_v} = \frac{237}{0.20 a_2}$$

$\Rightarrow a_2 =$ 11.85cm

$$a_{av} = \frac{a_1 + a_2}{2} = \frac{5.0 + 11.85}{2} = 8.5 \text{ cm}$$

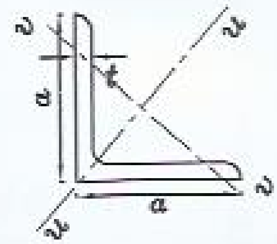
From Construction

> minimum angle $a_{min} = 1.1 * 3\phi = 1.1 * 3 * 1.6 = 5.28$ cm

Choose L 90*90*9

3) Checks

من المفترض أولا التأكد أن الـ angle تكون Non-Compact .



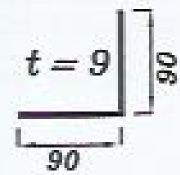
∟ 90 * 90 * 9

$A = 15.5\ Cm^2$

$r_v = 1.76\ Cm$

a) Class of section

$$* \frac{b}{t} = \frac{90}{9} = 10 < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$$



⟹ The section is non-compact (Code page 12)

b) Buckling (Slenderness)

r_{v_L} = من الجدول = 1.76 cm

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{v_L}} = \frac{237}{1.76} = 134.6 < 180 \Longrightarrow \text{(Safe)}$$

C) Stress

Compression

$$\lambda_{max.} = 134.6 > 100$$

$$* F_C = 0.6 * \frac{7500}{\lambda^2_{max.}} = 0.6 * \frac{7500}{134.6^2} = 0.41\ t \setminus cm^2$$

Unsymmetric

$$* f_C = \text{actual stress} = \frac{\text{force}}{A_{g\llcorner}} = \frac{2.0}{15.5} = 0.13\ t \setminus cm^2$$

$$\leq F_C \Longrightarrow \text{(Safe)}$$

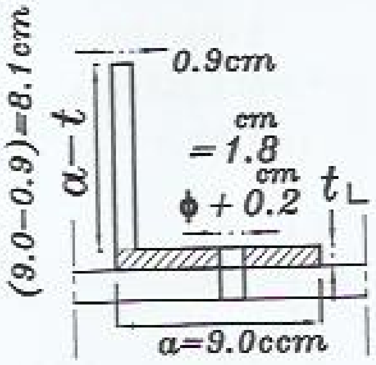
$$* \frac{f_C}{F_C} = \frac{0.13}{0.41} = 0.31 \Longrightarrow \text{(Safe but waste)}$$

و من المفترض فى هذه الحالة أن نختار angle أصغر و نعيد الـ Checks .

Design of tie plate

No tie plate ⟹ As it is one angle

d) Stress Tension



$$A_1 = [a - (\phi + 0.2\,cm)] * t_L$$

$$= [9.0 - (1.6 + 0.2)] * 0.9 = 6.48\ cm^2$$

$$A_2 = [a - t_L] * t_L$$

$$= [9.0 - 0.9] * 0.9 = 7.29\ cm^2$$

$$A_{net} = A_1 + A_2 \left[\frac{3A_1}{3A_1 + A_2}\right] = 6.48 + 7.29 * \left[\frac{3 * 6.48}{3 * 6.48 + 7.29}\right]$$

$$= 11.7\ cm^2$$

$$* \ f_t = \frac{Force}{A_{net}} = \frac{Force}{A_L} = \frac{4.0}{11.7} = 0.34\ t \backslash cm^2$$

A_L: مساحة الـ angle التى تم حسابها

$$\leqslant F_t = 1.40\ t \backslash cm^2$$

(Safe)

E) Length to depth ratio. (Deflection)

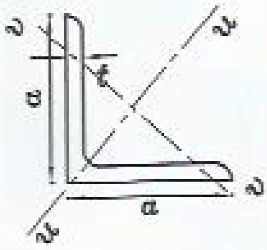
$$* \ \frac{L}{d} = \frac{237\ cm}{9.0\ cm} = 26.3 \leqslant 60 \Longrightarrow \text{(Safe)}$$

⟹ **Use ∟ 90*90*9**

و من المفترض فى هذه الحالة أن نختار angle أصغر و نعيد الـ Checks .
و ذلك لان الـ Checks كلها Safe بفارق كبير .

Try ⟹ **∟ 80*80*8**

من المفترض أولا التأكد أن ال angle تكون Non-Compact .



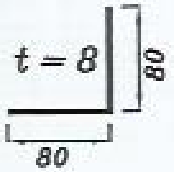
$\llcorner 80*80*8$

$A = 12.3\ Cm^2$

$r_v = 1.55\ Cm$

a) Class of section

$* \frac{b}{t} = \frac{80}{8} = 10 < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$



$\Longrightarrow$ The section is non-compact (Code page 12)

b) Buckling (Slenderness)

$r_{v_\llcorner}$ = من الجدول = 1.55 cm

$* \lambda_{out} = \frac{l_{b\,out}}{r_{v_\llcorner}} = \frac{237}{1.55} = 152.9 < 180 \Longrightarrow$ (Safe)

C) Stress Compression

$\lambda_{max.} = 152.9 > 100$

$* F_{C\,\text{Unsymmetric}} = 0.6 * \frac{7500}{\lambda^2_{max.}} = 0.6 * \frac{7500}{152.9^2} = 0.19\ t \backslash cm^2$

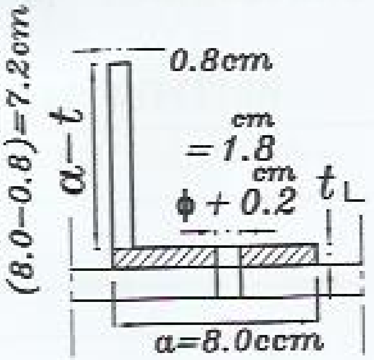
$* f_C$ = actual stress $= \frac{force}{A_{g\llcorner}} = \frac{2.0}{12.3} = 0.165\ t \backslash cm^2$

$\leqslant F_C \Longrightarrow$ (Safe)

$* \frac{f_C}{F_C} = \frac{0.165}{0.19} = 0.86 \Longrightarrow$ (Safe but waste)

d) Stress

Tension



$$A_1 = [a - (\phi + 0.2\,cm)] * t_L$$

$$= [8.0 - (1.6 + 0.2)] * 0.8 = 4.96\ cm^2$$

$$A_2 = [a - t_L] * t_L$$

$$= [8.0 - 0.8] * 0.8 = 5.76\ cm^2$$

$$A_{net} = A_1 + A_2 \left[\frac{3A_1}{3A_1 + A_2}\right] = 4.96 + 5.76 * \left[\frac{3 * 4.96}{3 * 4.96 + 5.76}\right]$$

$$= 9.11\ cm^2$$

$$* \ f_t = \frac{Force}{A_{net}} = \frac{Force}{A_L} = \frac{4.0}{9.11} = 0.44\ t \backslash cm^2$$

مساحة ال angle التى تم حسابها

$$\leqslant F_t = 1.40\ t \backslash cm^2$$

(Safe)

⟹ **Use ∟ 80*80*8**

SHAMS UNIVERSITY — FACULTY OF ENGINEERING

STRUCTURAL ENGINEERING DEPARTEMENT

THIRD YEAR CIVIL — STRUCTURAL DIVISION

January 2002 — STEEL STRUCTURES — TIME:3.00 Hrs

Question (1) (20%)

A Store Of a Factory has to be constructed over The area ABCDEF showing In Figure (1) .
A System Of Steel trusses is used to cover the area . The clear Height Of the Trusses is 6.00m Columns are allowed On the outer perimeter. It is required to :

1-Draw to scale 1:200 acomplete general layout for the building showing all necessary views and Bracing systems.

2-How to build Up the components of the bracing Sysytems (which elements are from the main truss and which are to be added) and explain how will the wind loads affect the Upper Chord of the truss and purlins .

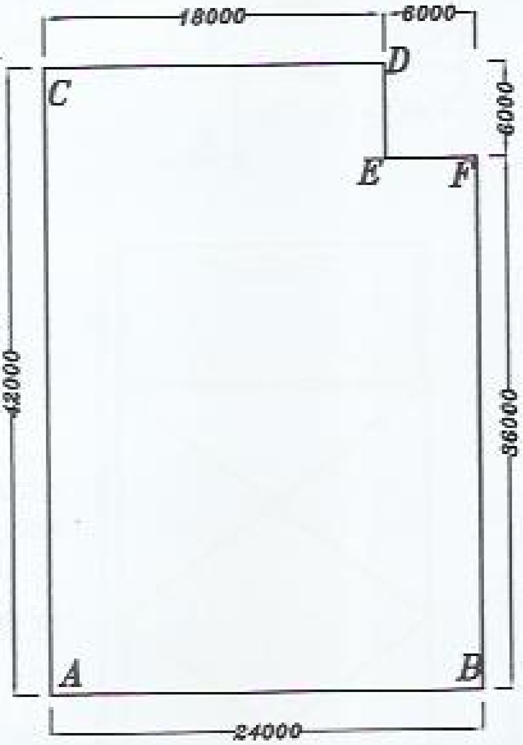


Fig.(1)

Question (2) (57%)

For the shown in figure (2) . it is requiredto:

1-Design the three-marked truss members

2-Design the connection enclosed by dotted rectangle using 16mm Diameter non-pretensioned bolts (bearing type) bolt grade 4.60 consider the end distance = 3 times the bolt diameter
It is not allowedto use a number of bolts in any member more than 6 bolts

3-Compare between the types of splices you can use to reduse the required number of bolts in chord members : The vertical splice and Horizontal splice

4-Design an intermediate roof purlin (spacing between main trusses equal 5.50m) consider the weight of sheets = 10 kg/m^2 and the live load to be 50 kg/m^2 . Will the tie rods be necessary in this case مؤجل للترم الثانى

5- Draw the part encolsed by dotted rectangle to scale 1:10

6-Draw the section X-X to scale 1:10

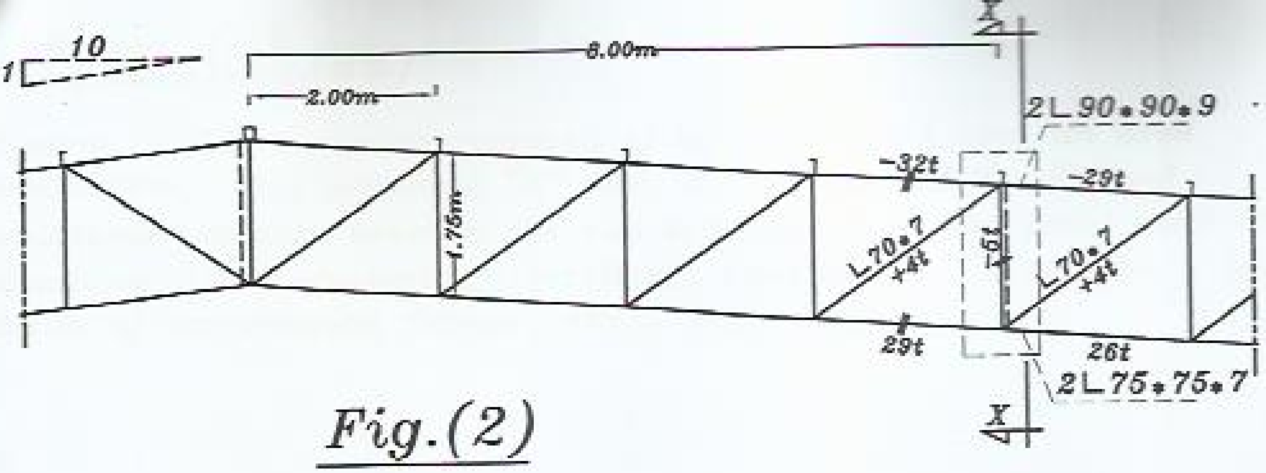


Fig.(2)

Question (3) (15%)

Figure (3) shows an elevated circular water tank of diameter 5.00m and the height 2.50m .
The tank is rested over a system of beams .
The secondary beams are supported over two main girders. Each of the two main girders is supported over 2 colmns spaced 5.00m as shown in figure .
Consider The total loads over the floor is 2.60 t/m^2

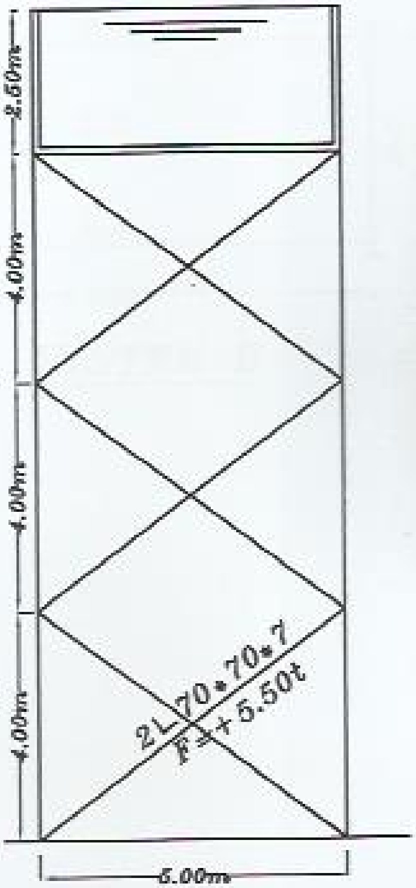
It is required to :

1- Design a suitable I.P.E Section for the secondary beams (consider the beams to be fully laterally supported and the section is a compact section i.e. $F_b = 0.64\ F_y$

2- Design the connection between the secondary beam and the main beam using non-pretensioned bolts (bearing type) bolts grade 8.80 consider the end distance = 3 times the bolt diameter . Draw the connection to scale 1:10 in different views .

الدكتور ذكر أنها لن تأتي بالامتحان

3- Check the stresses in the marked member shown in Figure (3)



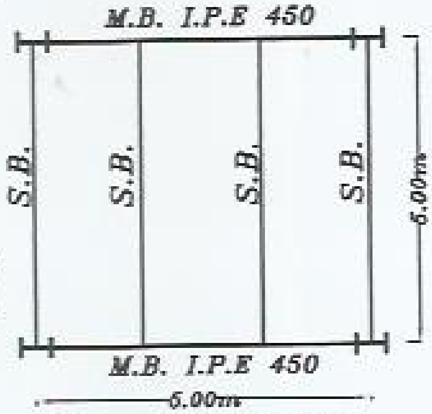
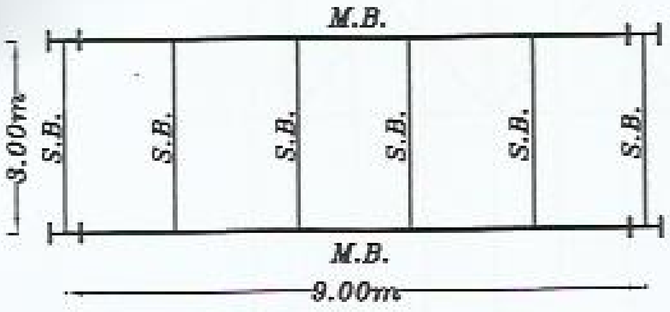


Fig.(3)

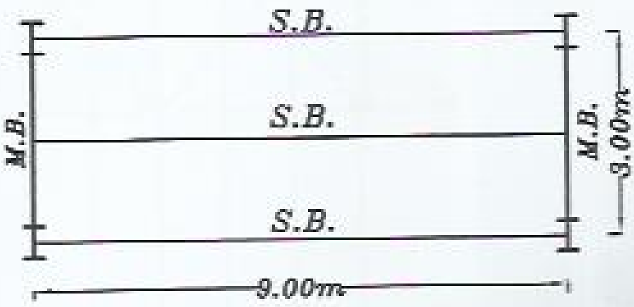
Floor Arrangement

Question (4) (8%)

A System of floor beamsis required to be constructed over an area 3.00mX9.00m . Two solutions "a" and "b" are suggested . Without calculations compare between the two solutions from the point view of: Connections , manufacturing , erection , steel weight , deflection & lateral stability of compression flange , etc..... state why ?

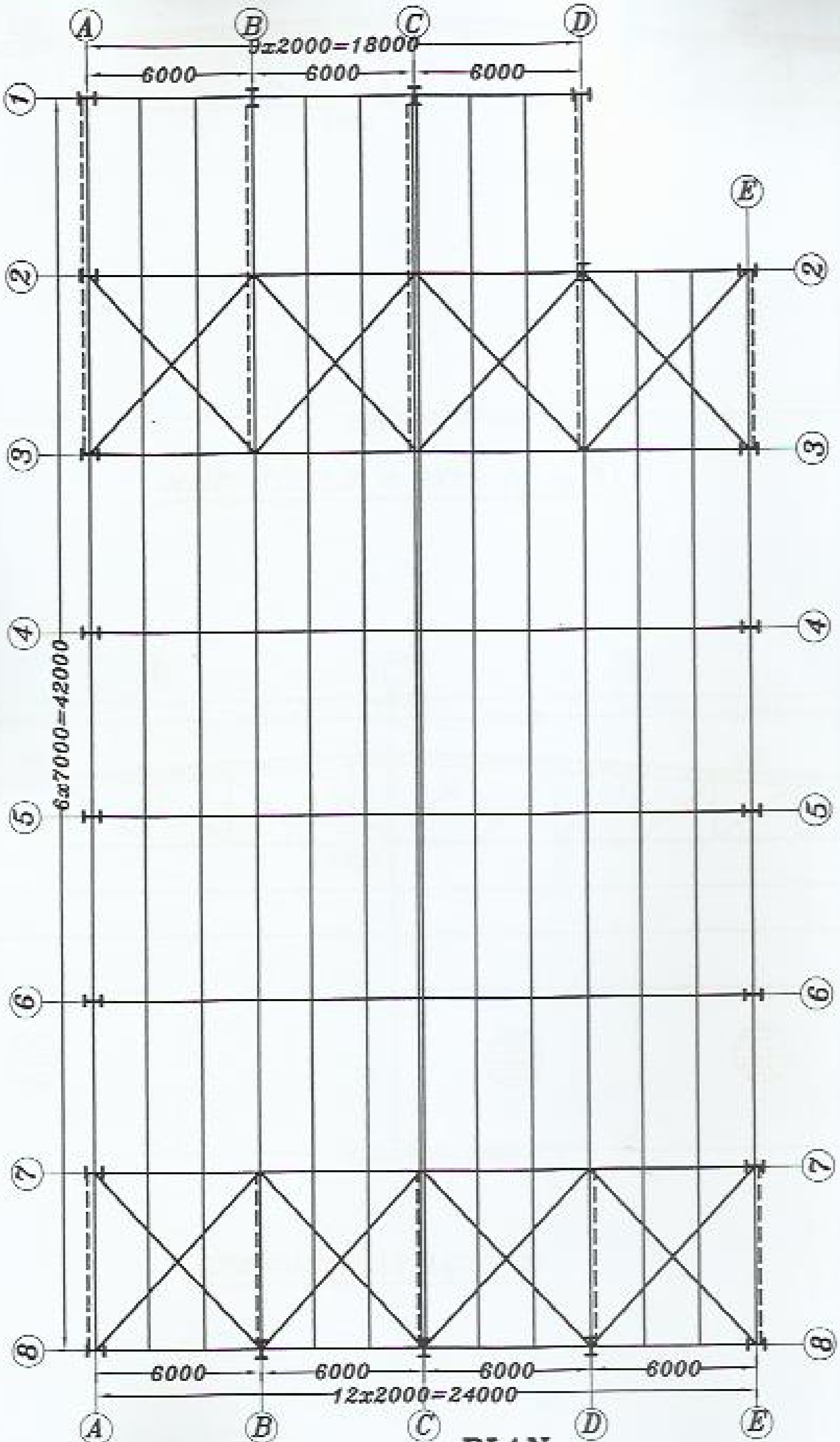


SYSTEM A

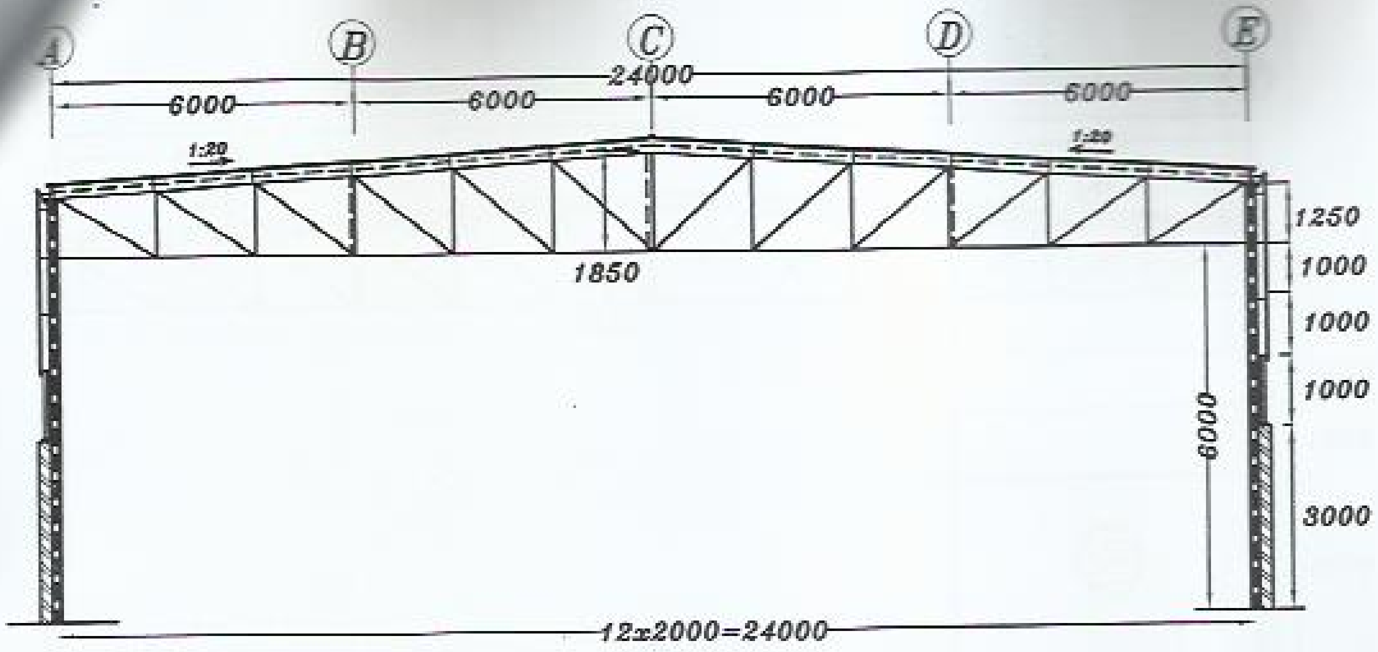


SYSTEM B

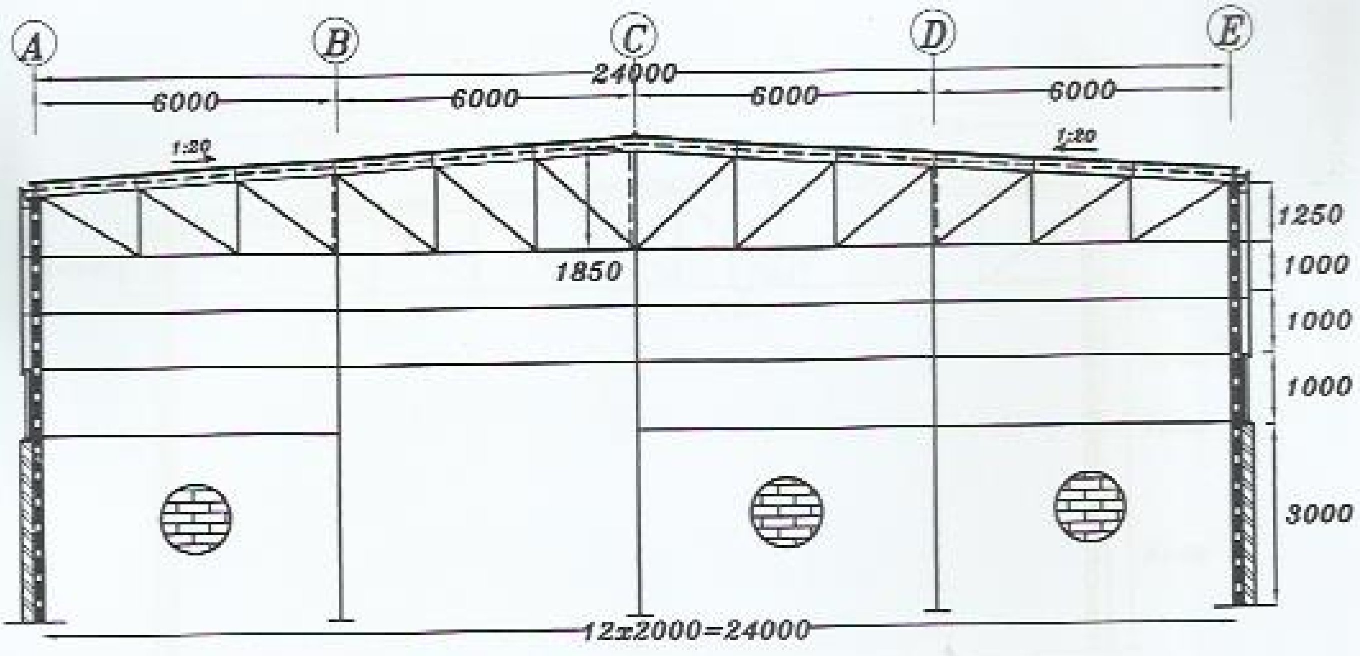
Question (1)



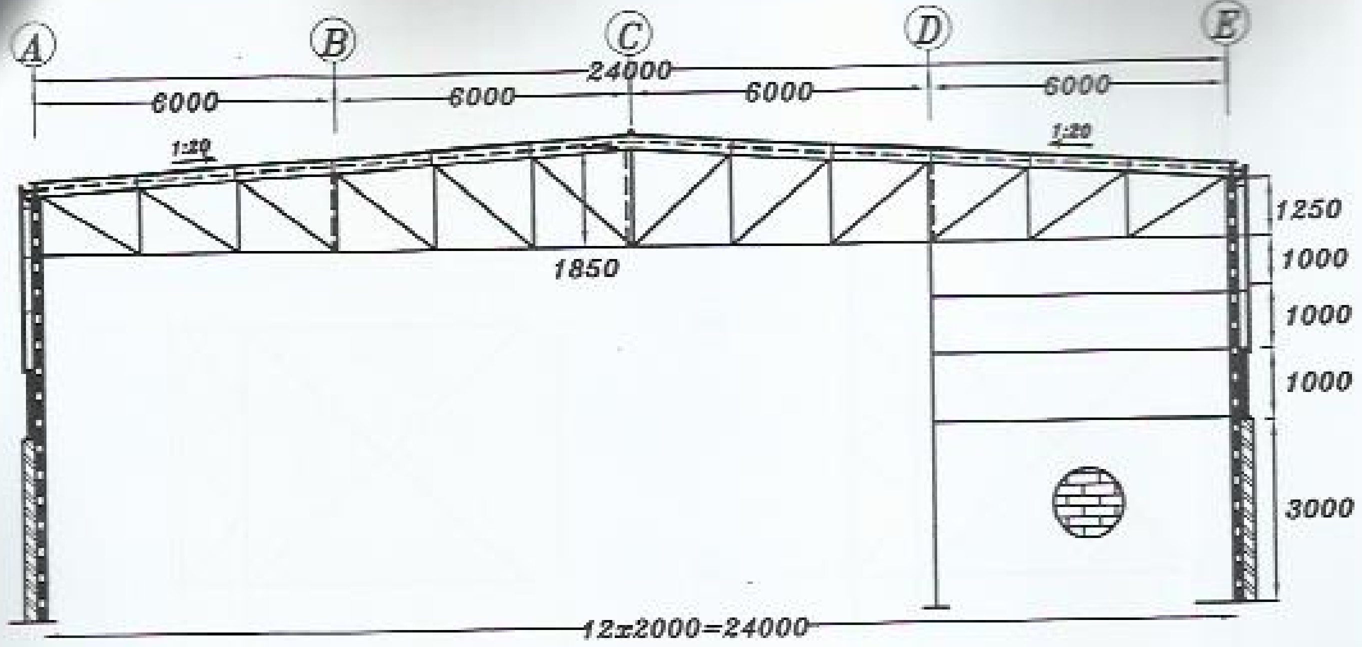
PLAN



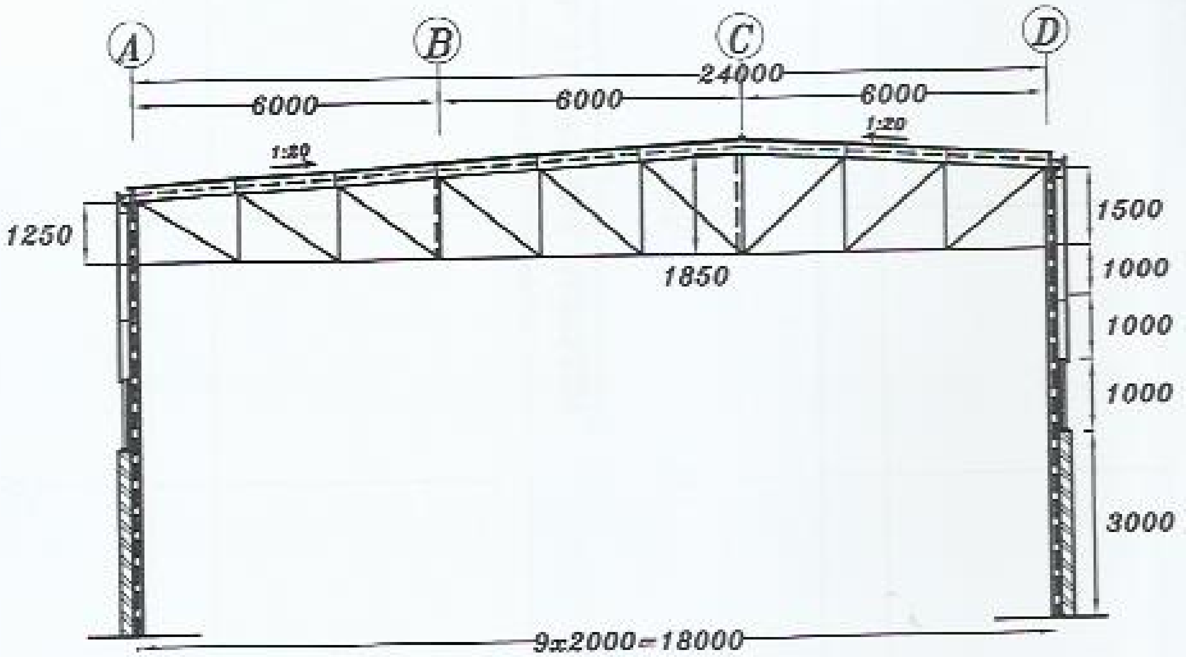
MAIN SYSTEM @ AXES (3 to 7)



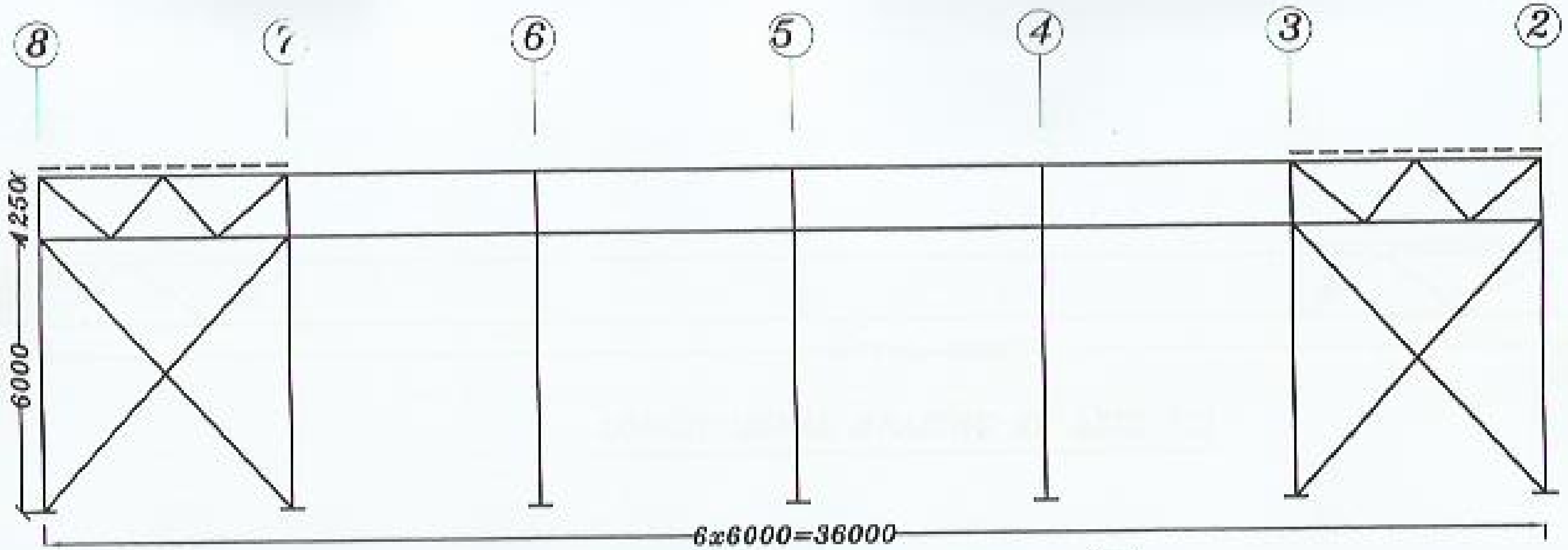
END GABLE AT AXIS (8)



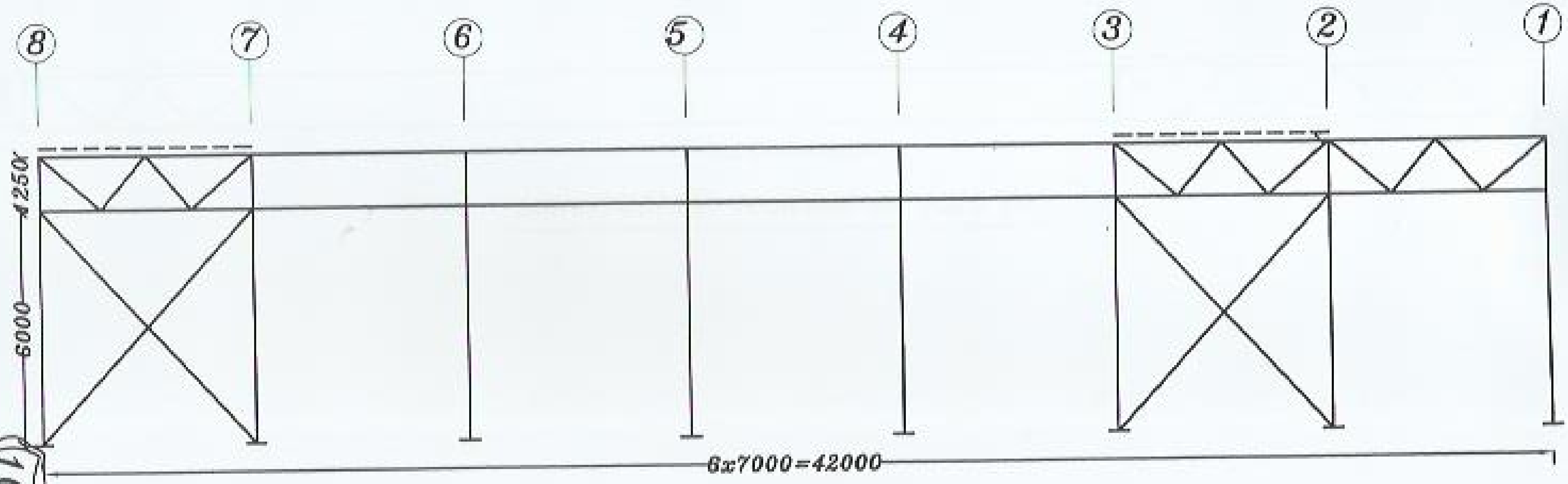
MAIN SYSTEM AT AXIS (2)



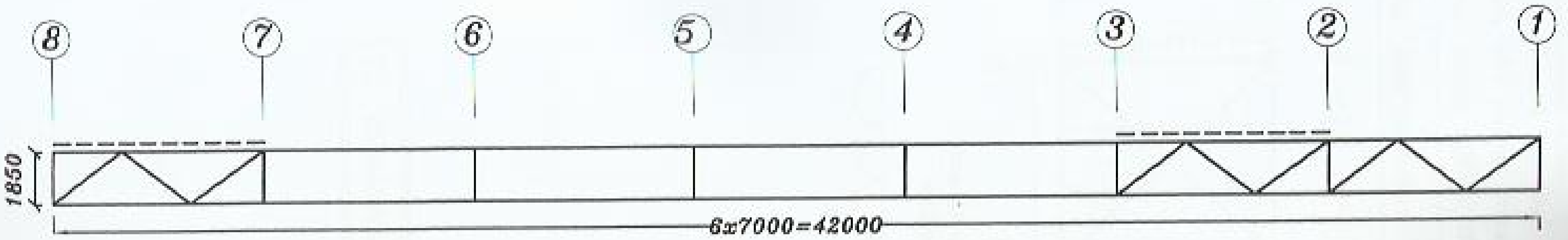
END GABLE AT AXIS (1)



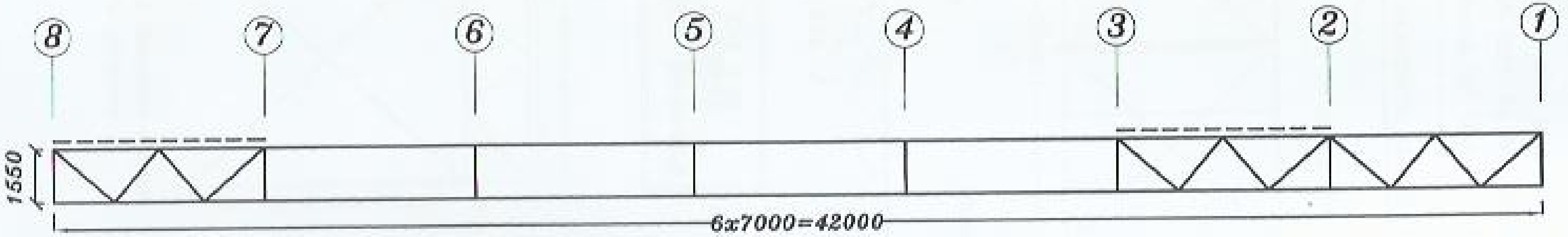
VERTICAL BRACING AT AXIS (E)



VERTICAL BRACING AT AXIS (A)



LONGITUDINAL BRACING AT AXIS (C)

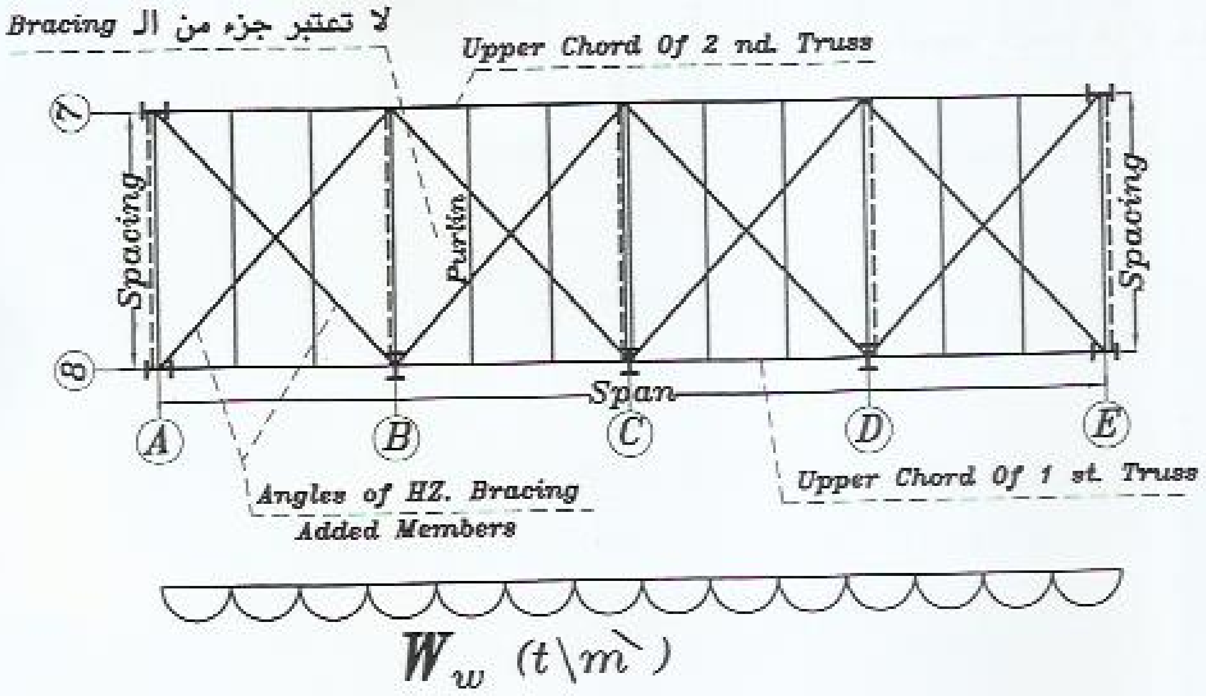


LONGITUDINAL BRACING AT AXIS (D)

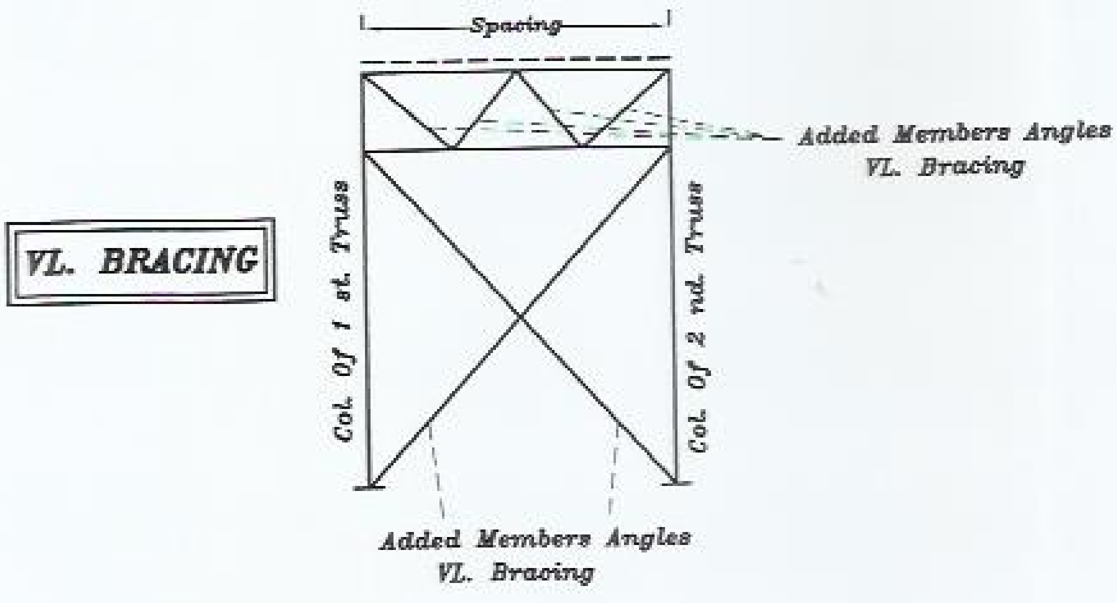
Question (1)

2-How to build Up the components of the bracing Systems (which elements are from the main truss and which are to be added), and explain how will the wind loads affect the Upper Chord of the truss and purlins .

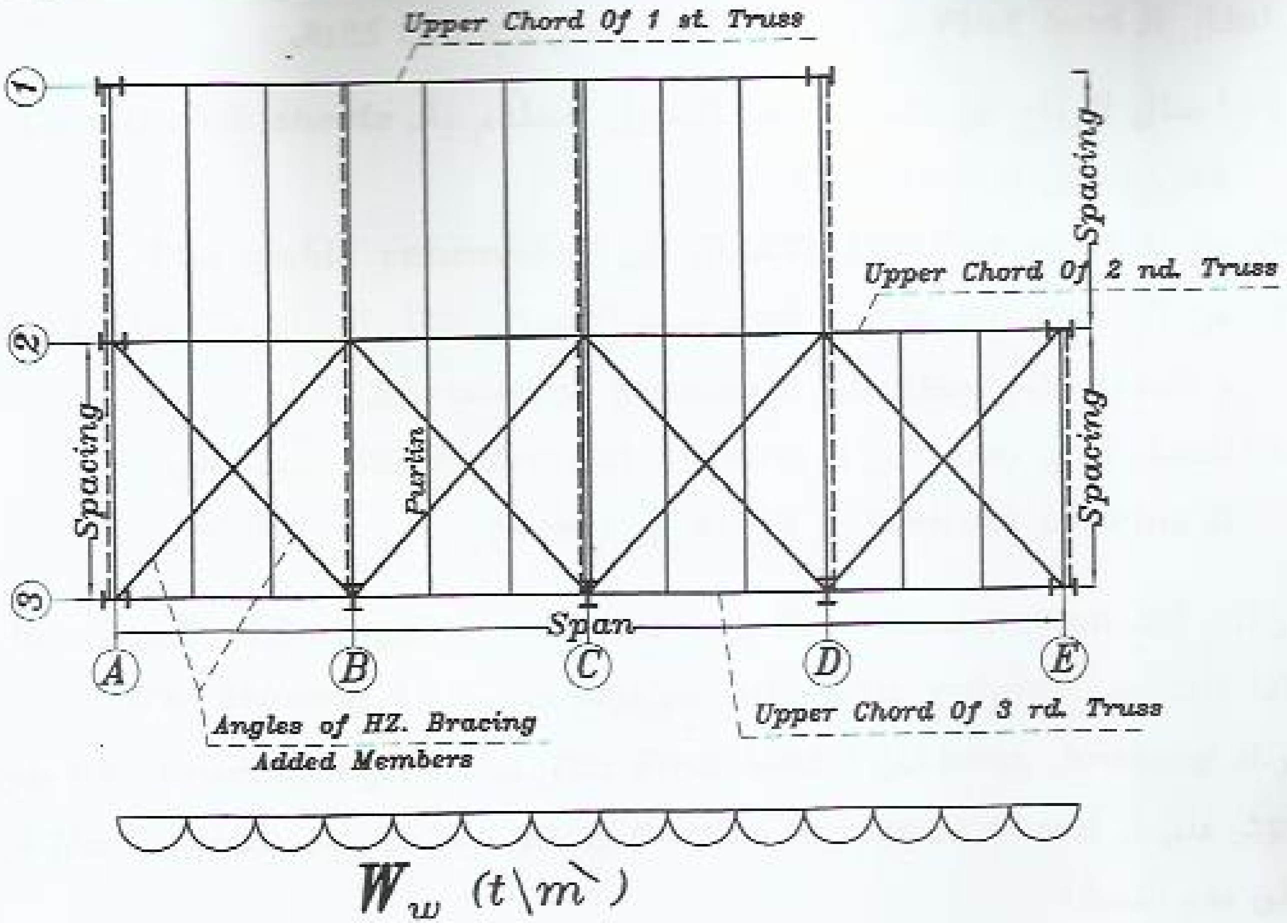
For the horizontal bracing between axes (7) and (8)



PLAN
Of HZ. Bracing



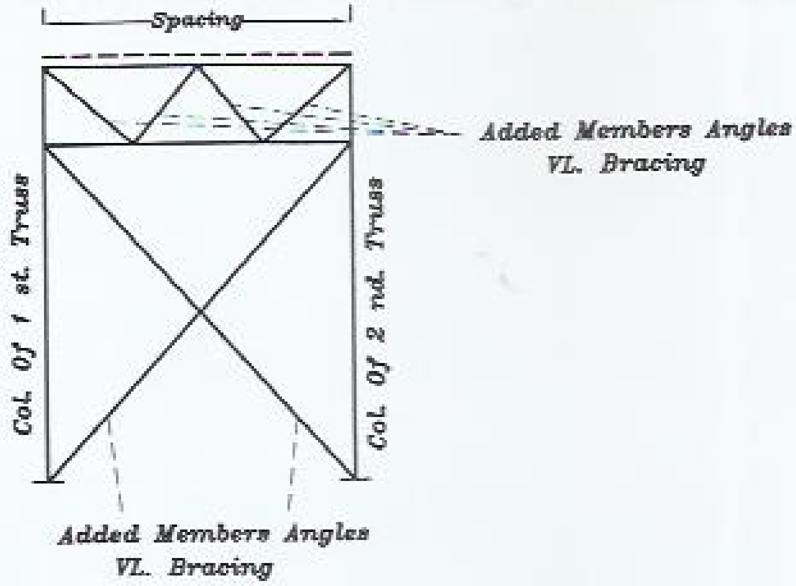
or the horizontal bracing between axes (1) and (3)



PLAN

Of HZ. Bracing

VL. BRACING



ال *Upper Chord* و ال *Purlins* تتأثر بال *Wind load* حيث أن من المفترض أن يكون انتقال ال *Wind load* فى الاتجاه العمودى على ال *Truss* كالتالى

١- احمال الرياح فى اتجاه المبنى الطويل تصطدم بال *Corrugated sheets* فتتوزع على ال *End girts* .

٢- من ال *End girts* تنتقل الاحمال الى ال *End gable columns* .

٣- من ال *End gable columns* نصف الاحمال تنتقل الى الارض مباشرة (base) و النصف الاخر ينتقل الى ال *Horizontal bracing* .

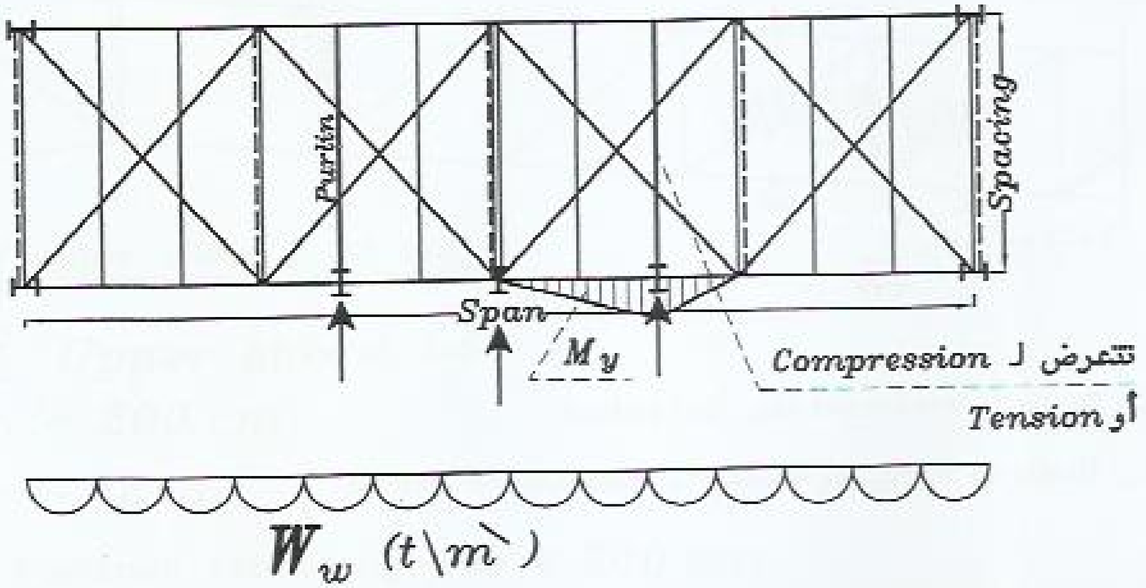
٤- النصف الذى ينتقل الى ال *Horizontal bracing* ينتقل بعده الى ال *Vertical bracing* ثم الى الارض (base) .

و لكن أثناء انتقال ال *Wind load* من ال *End gable columns* الى ال *Hz. bracing* اذا كانت ال *End gable columns* موجودة عند مكان ال *Hz. bracing* و ال *long. bracing* فان ل *Wind load* تنتقل مباشرة الى ال *Hz. bracing* بدون أن تؤثر على ال *Upper Chord* و ال *Purlins* كما هو موجود فى ال *Hz. bracing* فى فى هذه المسألة .

و لكن لان ال *Upper Chord* يعتبر جزء من ال *Hz. bracing* الذى يشبه ال *Truss* الافقى فانه يتعرض لضغط أو شد كأنه *Chord* لل *Truss* الافقى .

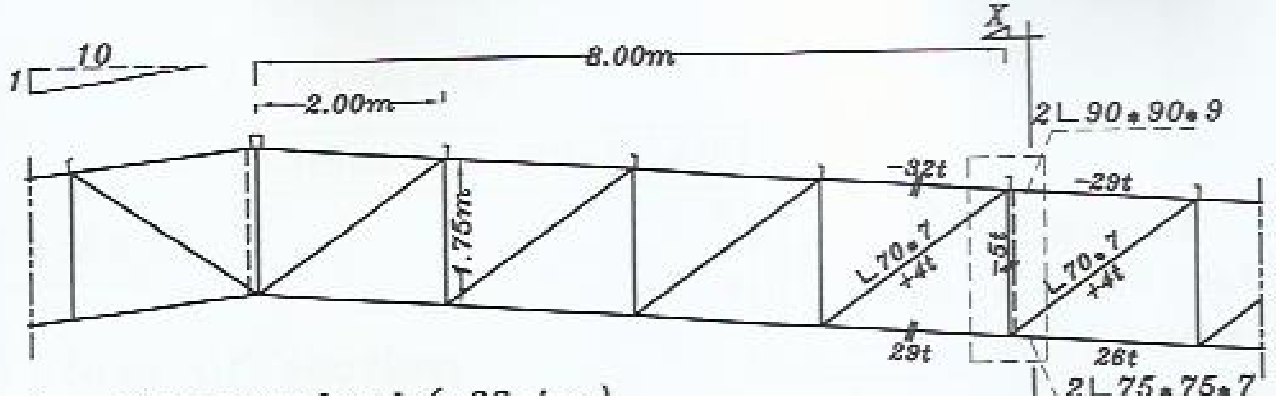
كما تتعرض ال *Purlins* ل ضغط و شد قليل نتيجة وجودها فى ال *Truss* الافقى .

اما اذا كان ال *End gable columns* غير موجودة عند مكان ال *Hz. bracing* و ال *long. bracing* فان لـ *Wind load* لكى تصل الى ال *Hz. bracing* فانها تصل عن طريق ال *Upper Chord* و ال *Purlins* مسببة *Compression or tension* فى ال *Purlins* و M_y فى ال *Upper Chord* .



Question (2)

1-Design the three-marked truss members



Design of upper chord (−32 ton)

1) Data

Upper chord ⇨ ╥

* Length = 200 cm
* Force = −32 ton
* $l_{b\,in}$ = Distance between joints = 200 cm
* $l_{b\,out}$ = Distance between Purlins = 200 cm

يتم تصميم ال members على أنها Bolted

لان المطلوب التالى هو تصميم ال Bolted Connection

2) Choice of section

From stresses

* assume $F_C = 0.75\ t \backslash cm^2$

$$\therefore A_{g\perp} = \frac{force}{F_C} = \frac{32}{0.75}$$

$$= 42.66\ cm^2$$

$$A_{g\lrcorner} = 21.33\ cm^2$$

Choose $\xrightarrow{tables}$ ⊥110*110*10

From buckling

* assume $\lambda_{out} = 100$

$$\therefore 100 = \frac{l_{bout}}{r_x} = \frac{200}{0.3\ a_2}$$

$$\Rightarrow a_2 = \boxed{6.67\ cm}$$

$$\therefore 100 = \frac{l_{bout}}{r_y} = \frac{200}{0.45 a_3}$$

$$\Rightarrow a_3 = 4.44\ cm$$

$$a_{av} = \frac{a_1 + a_2}{2} = \frac{11 + 6.67}{2} = 8.85\ cm$$

$$\boxed{\text{Choose } \perp 90*90*9}$$

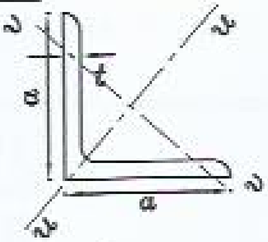
From construction

Bolted

$a_{min.} = 3\phi * 1.1\,(Bolted) = 3.30 * 16 = 52.8\ mm$

Choose ⊥ 90*90*9

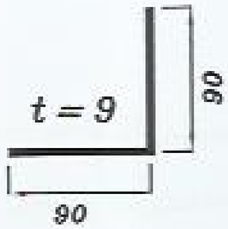
Checks



$\llcorner\ 90*90*9$

$A = 15.5\ Cm^2$

$r_v = 1.76\ Cm$



a) Class of section

$* \frac{b}{t} = \frac{90}{9} = 10 < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$

⟹ The section is non-compact (Code page 12)

b) Buckling (Slenderness)

$r_{x\perp} = 2.74\ cm$

$r_{y\perp} = \sqrt{r_{x\perp}^2 + (0.50+e)^2} = \sqrt{2.74^2 + (0.50+2.54)^2} = 4.09\,cm$

$* \lambda_{in} = \frac{200}{2.74} = 72.99 < 180 \Longrightarrow$ **(Safe)**

$* \lambda_{out} = \frac{200}{4.09} = 48.89 < 180 \Longrightarrow$ **(Safe)**

$\lambda_{max.} = 72.99$

c) Stress

$\lambda_{max.} = 79.41 < 100$

$* F_C = 1.4 - 6.5 * 10^{-5} \lambda^2_{max.} = 1.4 - 6.5 * 10^{-5}(72.99)^2$

$= 1.05\ t \backslash cm^2$

$* f_C = \text{actual stress} = \frac{force}{A_{g\perp}} = \frac{32}{2*15.5} = 1.03\ t \backslash cm^2$

$\leqslant F_C \Longrightarrow$ **(Safe & Economic)**

Design of tie plate

$$\lambda_v \leqslant \lambda_{max.}$$

$$\frac{l'}{r_{v\llcorner}} \leqslant 72.99 \Rightarrow l' \leqslant 1.76 * 72.99 = 128.5\ cm$$

$$l' > \frac{l}{2} \Rightarrow$$ Use one tie plate at the middle of member

Design of lower chord (+29t)

1) Data

Lower chord $\Rightarrow \lrcorner\llcorner$

* Length = 200 cm — يتم تصميم الـ members على أنها Bolted
* Force = +29 ton — لأن المطلوب التالى هو تصميم الـ Bolted Connection
* $l_{b\,in}$ = Distance between joints = 200 cm
* $l_{b\,out}$ = Distance between longitudinal bracing = 800 cm

2) Choice of section

1) From stress condition

$$A_{g_{\lrcorner\llcorner}} = \frac{\text{Force ton}}{1.4 * 0.85} = \frac{29}{1.4 * 0.85} = 24.37\ cm^2$$

$$A_{g_{\llcorner}} = \frac{24.37}{2} = 12.18\ cm^2$$

$\xrightarrow{\text{من الجدول}}$ **Choose $\llcorner$ 80x80x8** $a_1 = 8.0\ cm$

2) From Slenderness condition

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x\lrcorner\llcorner}} = \frac{200}{0.30\,a} = 300 \Rightarrow a = 2.22\ cm$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{x\lrcorner\llcorner}} = \frac{800}{0.45\,a} = 300 \Rightarrow a = 5.92\ cm$$

$a_2 = 5.92\ cm$

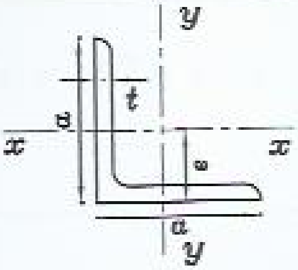
2) From Construction condition

$a_{min} = a_3 = 3\phi * 1.1 \text{ (Bolted)} = 3.30 * 16 = 52.8 \text{ mm}$

Choose ⅃L 80x80x8   نختار أكبر angle من السابق

3) Checks

a) Stress



L 80*80*8
$A = 12.3\ Cm^2$
$e = 2.26\ Cm$
$r_x = r_y = 2.42\ Cm$

$$A_{net} = 2[A_{gross\,L} - (\phi + 0.2\,cm) * t_{w\,L}]$$

$$= 2[12.3 - (1.6 + 0.2\,cm) * 0.8] = 21.72\ cm^2$$

$$* f_{act} = \frac{Force}{A_{net}} = \frac{29}{21.72} = 1.34\ t\backslash cm^2$$
$$\leqslant F_t = 1.40\ t\backslash cm^2 \text{ (Safe)}$$

b) Slenderness

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

assume $t_{Gp} = 1\ cm$

$$r_{y\,⅃L} = \sqrt{r_{y\,L}^2 + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.42^2 + (2.26 + \frac{1.0}{2})^2} = 3.67\ cm$$

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x\,⅃L}} = \frac{200}{2.42} = 82.6 < 300 \Longrightarrow \text{(Safe)}$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{y\,⅃L}} = \frac{800}{3.67} = 218 < 300 \Longrightarrow \text{(Safe)}$$

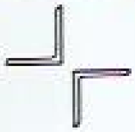
C) Length to depth ratio (Deflection)

$$* \frac{L}{d} = \frac{200\ cm}{a} = \frac{200\ cm}{8.00} = 25 \leqslant 60 \Longrightarrow \text{(Safe)}$$

Choose ⅃L 80x80x8

Design of VL. member (−5t)

1) Data

Vertical ⇨  ⇨ At position of vertical bracing

* Length = 175 cm

يتم تصميم الـ members على أنها Bolted

* Force = −5 ton

لان المطلوب التالى هو تصميم الـ Bolted Connection

* $l_{b\,in}$ = Distance between joints = 175 cm
* $l_{b\,out}$ = 175 cm

2) Choice of section

From stresses

* assume $F_C = 0.75\ t\backslash cm^2$

$$\therefore A_{g\,\perp\!\!\vdash} = \frac{force}{F_C} = \frac{5}{0.75}$$

$$= 6.66\ cm^2$$

Choose $\xrightarrow{tables}$ ∟ 45*45*5

From buckling

* assume $\lambda_{out} = 100$

$$\therefore 100 = \frac{l_{b\,out}}{r_u} = \frac{175}{0.38 a_2}$$

$$\Rightarrow a_2 = \boxed{4.54\ cm}$$

$$a_{av} = \frac{a_1 + a_2}{2} = \frac{4.5 + 4.50}{2} = 4.5\ cm$$

Choose ∟ 45*45*5

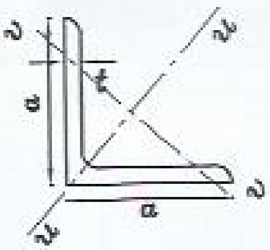
3) From construction

Bolted

$$a_{min.} = 3\phi * 1.1\ (Bolted) = 3.30 * 16 = 52.8\ mm$$

Choose ┤├ 55*55*5

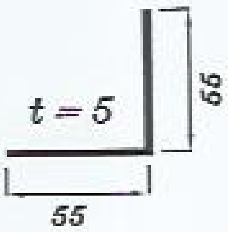
Checks



$\llcorner 55 * 55 * 5$

$A = 5.32\ Cm^2$

$r_v = 1.07\ Cm$



a) Class of section

$* \frac{b}{t} = \frac{55}{5} = 11 < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$

$\Longrightarrow$ The section is non-compact
(Code page 12)

b) Slenderness (Buckling)

$r_{u \perp} = r_{u \llcorner}$ من الجدول $= 2.09\ cm$

$* \lambda_{out} = \frac{175}{2.09} = 83.73 < 180 \Longrightarrow$ (Safe)

$\lambda_{max.} = 83.73$

c) Stress

$\lambda_{max.} = 83.73 < 100$

$* F_C = 1.4 - 6.5 * 10^{-5} \lambda^2_{max.} = 1.4 - 6.5 * 10^{-5} (83.73)^2$

$= 0.94\ t \setminus cm^2$

$* f_C = \text{actual stress} = \frac{force}{A_{g \perp}} = \frac{5}{2 * 5.32} = 0.47\ t \setminus cm^2$

$\leqslant F_C \Longrightarrow$ (Safe)

$* \frac{f_C}{F_C} = \frac{0.47}{0.94} = 0.50 \Longrightarrow$ (Safe but waste)

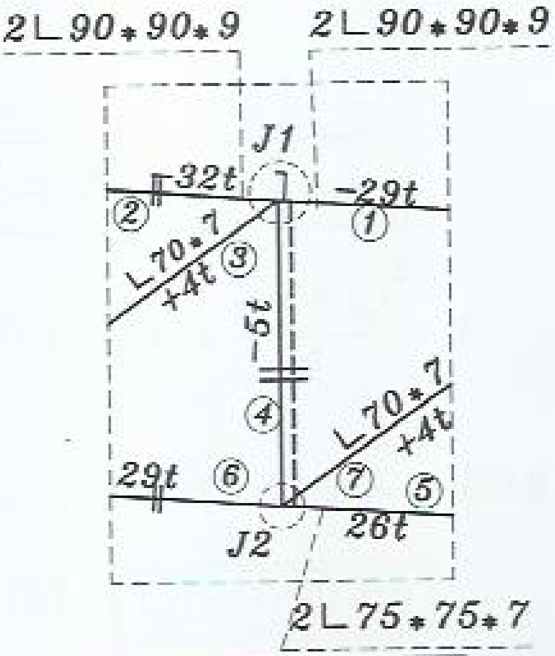
Design of tie plate

$\lambda_v \leqslant \lambda_{max.}$

$\frac{l'}{r_{v \llcorner}} \leqslant 83.73 \Longrightarrow l' \leqslant 1.07 * 83.73 = 88.59\ cm$

$l' > \frac{l}{2} \Longrightarrow$ Use one tie plate at the middle of member

-Design the connection enclosed by dotted rectangle using 16mm Diameter non-pretensioned bolts (bearing type) bolt grade 4.60 consider the end distance = 3 times the bolt diameter .
It is not allowed to use a number of bolts in any member more than 6 bolts



Design of Joint J1

يجب الملاحظة ان هذه الوصلة هى

Continous Joint

Member 1,2 $2∟90*90*9$

$$R_{Shear} = q_b * A_S * n \qquad R_b = F_b * d * t_{min}$$

$* F_{ub} = 4\ t\backslash cm^2$ $\qquad * \phi = 1.6\ cm$

$* F_u \xrightarrow{\text{for st.37}} = 3.6\ t\backslash cm^2$

$* q_b = 0.25 F_{ub}$ $\qquad * A_S = \frac{\pi d^2}{4}$

سوف يتم التعويض عن α بقيمة تساوى ١,٢٠ بدلا من ٠,٨ وذلك لانه كان قد اعطى ان الـ Edge dist.=3ϕ

$\Longrightarrow \alpha = 1.20$

$$* R_{S.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 1 = (0.25 * 4) * \frac{\pi (1.6)^2}{4} * 1 = 2.01\ ton$$

$$* R_{D.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 2 = 2 * R_{S.S} = 4.02\ ton$$

$$* R_b = (\alpha * F_u) * d * \Sigma t_{min} = 1.2 * 3.6 * 1.6 * 1.00 = 6.90\ ton$$

Σt_{min}: Min. of $2t_L = 0.9*2 = 1.80\ cm$ OR $t_{g.p} = 1.00\ cm$ govern

$* n_{1,2} = \frac{Force}{R_{Least}} = \frac{32-29=3t}{4.02} = 2.00 =$ **2 Bolts**

Member 3 $\llcorner 70 * 70 * 7$

$$R_{Shear} = q_b * A_S * n \qquad R_b = F_b * d * t_{min}$$

$* R_{S.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 1 = (0.25 * 4) * \frac{\pi (1.6)^2}{4} * 1$
$= 2.01$ ton

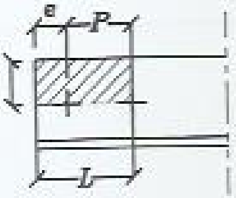
$* R_b = (\alpha * F_u) * d * \Sigma t_{min} = 1.2 * 3.6 * 1.6 * 0.70 = 4.838$ ton

Σt_{min}: Min. of $t_L = 0.70$ cm OR $t_{g.p} = 1.00$ cm
(govern: $t_L = 0.70$ cm)

$* n_3 = \frac{Force}{R_{Least}} = \frac{4}{2.01} =$ **2 Bolts**

Check block shear rupture

$*$ Take $e = 4.8$ cm
$P = 6.0 \text{ cm} > P_{min.} = 3\,\phi$



$* m = \frac{a-t}{2} = \frac{7-0.7}{2} = 3.15$ cm

$* L = e + P = 4.8 + 6 = 10.8$ cm

$* A_{net\ Shear} = [L - (n - 0.5)(\phi + 0.2)] * t_L$
$= [10.8 - (2 - 0.5)(1.6 + 0.2)] * 0.7 = 5.67 \text{ cm}^2$

$* A_{net\ Tension} = [m - 0.5 * (\phi + 0.2)] * t_L$
$= [3.15 - 0.5 * (1.6 + 0.2)] * 0.7 = 1.57 \text{ cm}^2$

$* P = 0.4 F_y A_{net\ Shear} + 0.725 F_y A_{net\ Tension}$
$= 0.4 * 2.4 * 5.67 + 0.725 * 2.4 * 1.57 = 8.17$ ton

* Check $\Longrightarrow P = 8.17$ ton $>$ Tension force $= 4$ ton

$\Longrightarrow$ Safe $\Longrightarrow$ No B.S.R failure

Member [4] 55*55*5

$$R_{Shear} = q_b * A_S * n \qquad R_b = F_b * d * t_{min}$$

$$* R_{S.S} = (0.25\, F_{ub}) * \frac{\pi d^2}{4} * 1 = (0.25 * 4) * \frac{\pi (1.6)^2}{4} * 1$$

$$= 2.01 \text{ ton}$$

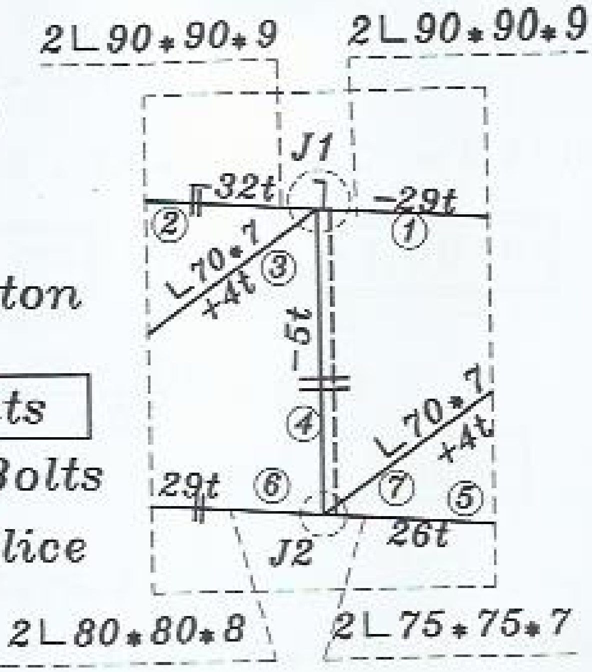
$$* R_b = (\alpha * F_u) * d * \Sigma t_{min} = 1.2 * 3.6 * 1.6 * 0.50 = 3.456 \text{ ton}$$

Σt_{min} : Min. of $\boxed{t_L = 0.50 \text{ cm}}$ OR $\boxed{t_{g.p} = 1.00 \text{ cm}}$

govern

$$* n_3 = \frac{Force}{R_{Least}} = \frac{5}{2.01} = \boxed{4 \text{ Bolts}} \quad 2 \text{ each side}$$

Design of Joint J2

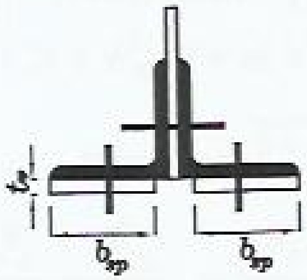


Member **6** 2L90*90*9

* $R_{D.S} = 4.02\ ton$ * $R_b = 6.90\ ton$

* $n_6 = \dfrac{Force}{R_{Least}} = \dfrac{29}{4.02} =$ **8 Bolts** $>$ 6 Bolts

So we have to use HZ. splice



$t_{sp} = 10\ mm$

* $b_{sp} = 0.8a = 0.8x8 = 6.40\ cm$

* $F_{sp} = \dfrac{F_{min.}}{2} = \dfrac{26}{2} = 13\ t$

* $R_{S.S} = (0.25 F_{ub}) * \dfrac{\pi d^2}{4} * 1 = (0.25 * 4) * \dfrac{\pi (1.6)^2}{4} * 1$
$= 2.01\ ton$

* $R_b = (\alpha * F_u) * d * \Sigma t_{min} = 1.2 * 3.6 * 1.6 * 0.80 = 5.529\ ton$

Σt_{min}: Min. of $t_L = 0.75\ cm$ OR $t_{g.p} = 1.00\ cm$

* $n_{sp} = \dfrac{F_{sp}}{R_{Least}} = \dfrac{13t}{2.01} =$ **8 Bolts** **4 each angle**

For n_6 **member (6)** $\rfloor\llcorner$ 80*80*8

$* R_b = (\alpha * F_u) * d * \Sigma t_{min} = 1.2 * 3.6 * 1.6 * 1.0 = 6.91$ ton

Σt_{min}: Min. of $2t_L = 1.60$ cm OR $t_{g.p} = 1.00$ cm

$$* n_6 = \frac{F_6 - F_{Sp}}{R_l} = \frac{F_6 - F_{Sp}}{R_{D.S}} = \frac{29 - 13}{4.02} = 3.98$$

$= $ **4 Bolts**

For n_5 **member (5)** $\rfloor\llcorner$ 75*75*7

$* R_b = (\alpha * F_u) * d * \Sigma t_{min} = 1.2 * 3.6 * 1.6 * 1.0 = 6.91$ ton

Σt_{min}: Min. of $2t_L = 1.50$ cm OR $t_{g.p} = 1.00$ cm

$$* n_5 = \frac{F_5 - F_{Sp}}{R_l} = \frac{F_5 - F_{Sp}}{R_{D.S}} = \frac{26 - 13}{4.02} = 3.24$$

$= $ **4 Bolts**

Member (7) $\llcorner$ 70*70*7

$* t_{min} = \overset{t_{G.p}}{1\ cm}$ or $\overset{t_L}{0.7\ cm}$ $\Longrightarrow$ $t_{min} = 0.7$ cm

$* R_b = (\alpha * F_u) * d * \Sigma t_{min} = 1.2 * 3.6 * 1.6 * 0.7 = 4.84$ ton

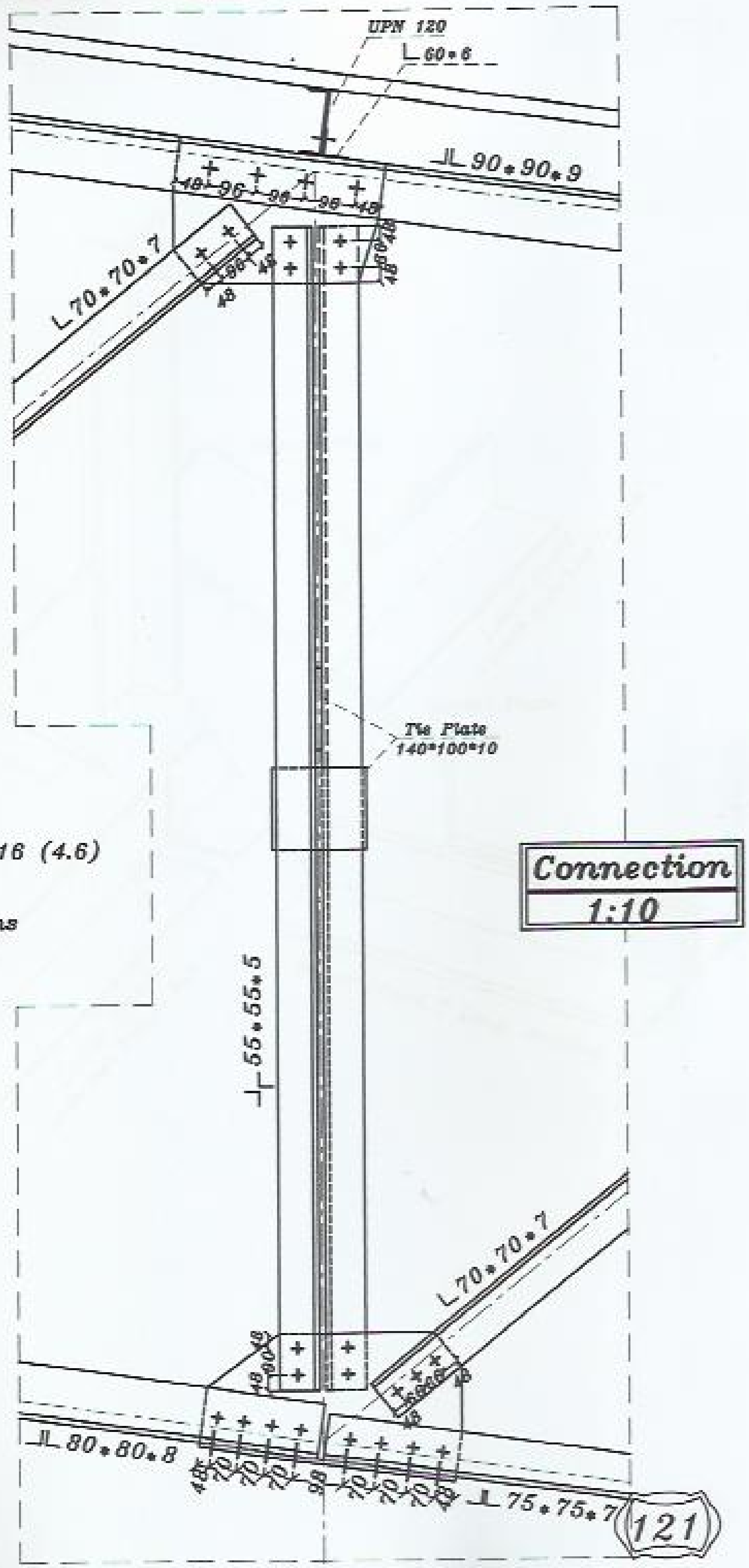
$* R_{Least} \rightarrow$ $R_{S.S} = 2.01$, $R_b = 4.84$ $\quad R_{Least} = 2.01$ ton

$$* n_3 = \frac{Force}{R_{Least}} = \frac{4}{2.01} = 1.98 = \text{2 Bolts}$$

3-Compare between the types of splices you can use to reduse the required number of bolts in chord members : The vertical splice and Horizontal splice

	Horizontal Splice	Vertical Splice
Use	Lower chord Only	Upper & Lower chord
Shear	Bolts Work in single and double shear	Bolts Work in 4 shear surfaces
number of bolts	Increase total number of bolts in the connection (number of bolts in one row is reduced)	Decrease total number of bolts in the connection

- Draw the part encolsed by dotted rectangle to scale 1:10



Notes

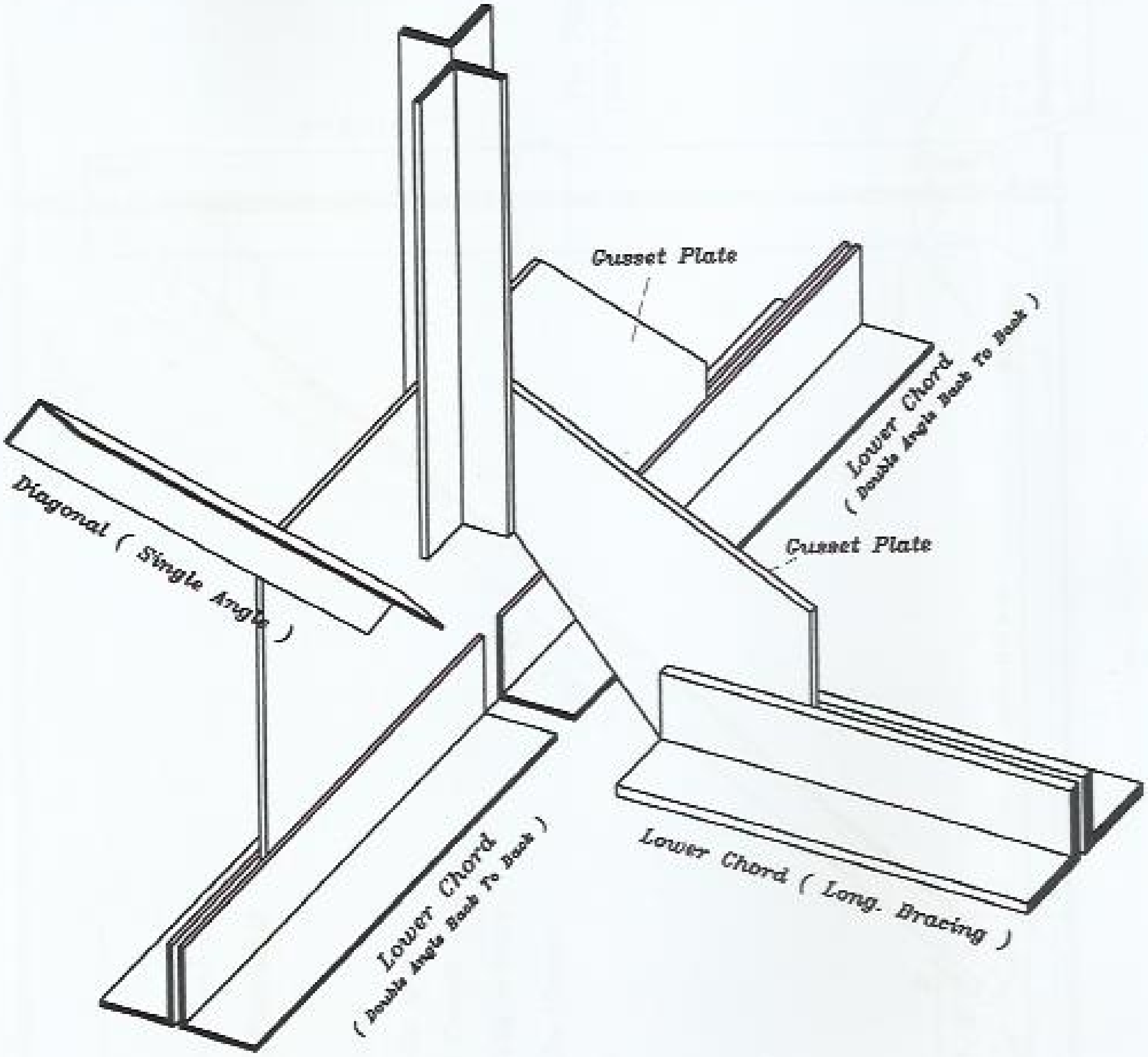
1-St. used is st.37

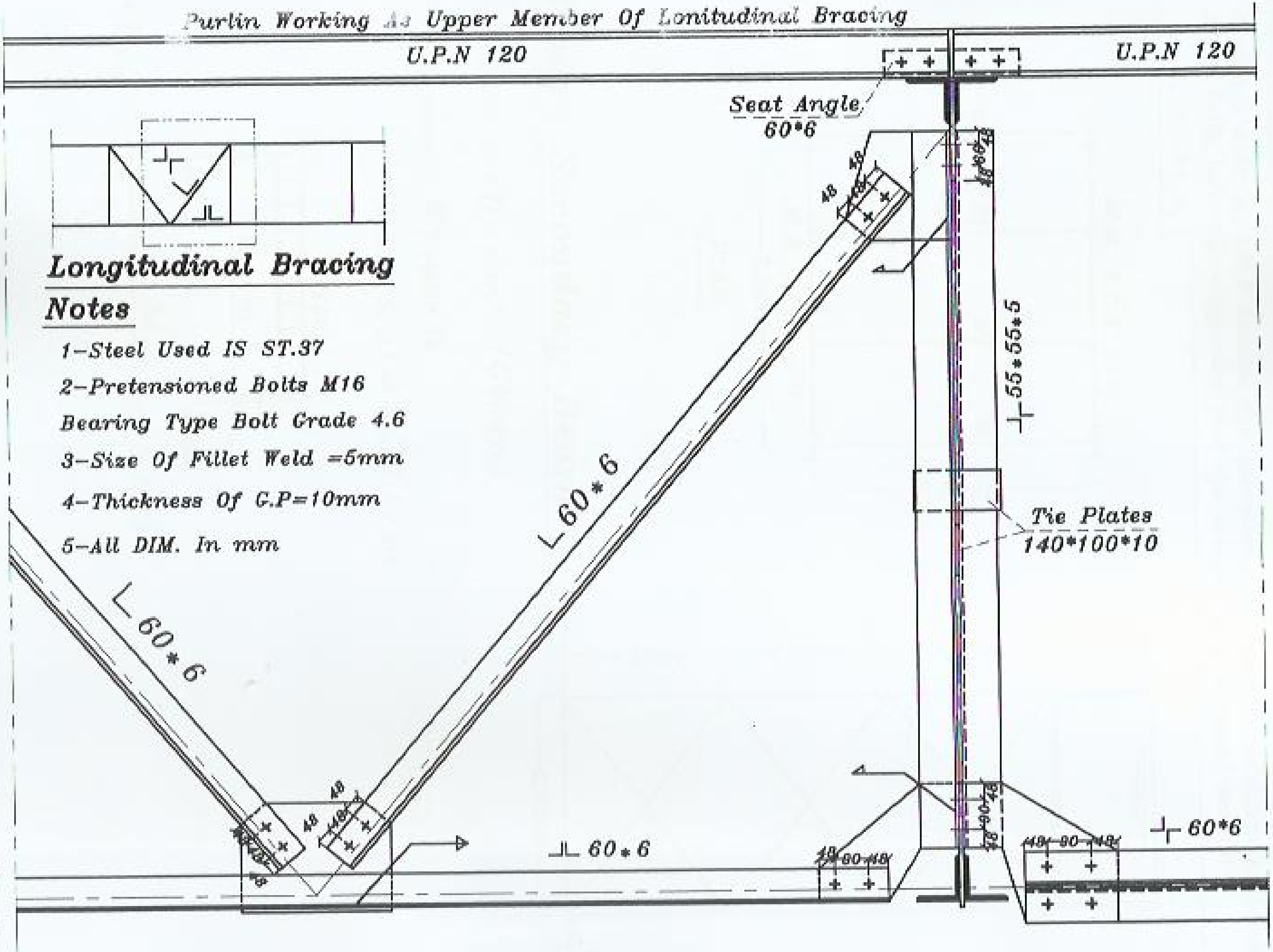
2-Non-pretensioned bolts M16 (4.6)

3-Thickness Of G.P=10mm

4-All dimensions are in mms

5-Draw the section X-X to scale 1:10





Longitudinal Bracing

Notes

1-Steel Used IS ST.37

2-Pretensioned Bolts M16

Bearing Type Bolt Grade 4.6

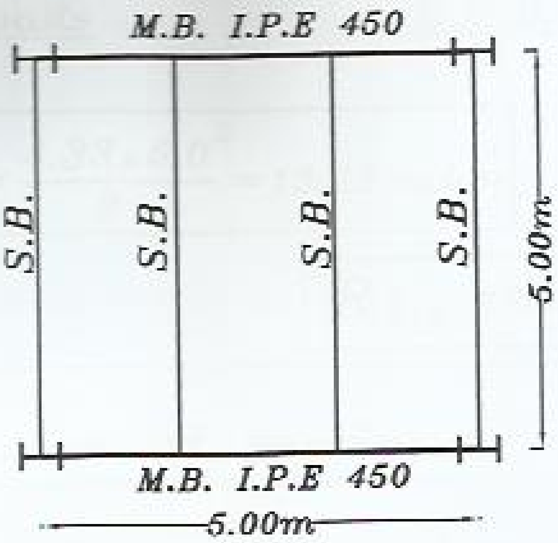
3-Size Of Fillet Weld =5mm

4-Thickness Of G.P=10mm

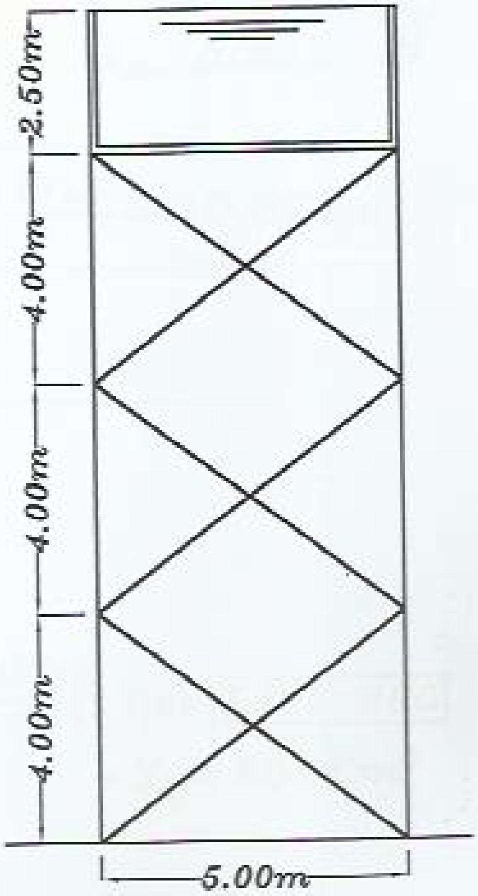
5-All DIM. In mm

Question (3)

1- Design a suitable I.P.E Section for the secondary beams (consider the beams to be fully laterally supported and the section is a compact section i.e. $F_b = 0.64\ F_y$



PLAN

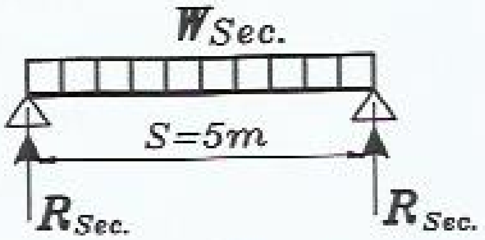


Design Secondary Beam

* $W_{Total} = 2.60\ t/m^2$ (Given)

* $W_{Sec.\ beam} = W_{Total} * a$

$$= 2.60 * (5/3) = 4.33\ t/m'$$



* $R_{Sec.} = \dfrac{w_{Sec.} * S}{2} = \dfrac{4.33 * 5}{2} = 10.82\,t$

1- Straining actions

$$M_X = \frac{4.33 * 5.0^2}{8} = 13.53 \text{ m.ton}$$

$$Q = \frac{4.33 * 5}{2} = 10.82 \text{ ton}$$

$$R_{Sec.} = 10.8 \text{ ton}$$

2- Choice of section

assume $F_b = 0.64 F_y = 1.536 \ t \backslash cm^2$

$$S_X = \frac{M_X}{1.536} = \frac{13.53 * 100}{1.536} = 880.85 \ Cm^3 \xrightarrow{\text{Tables}} \text{Use } \boxed{I.P.E\ 360}$$

$$* S_X = 904 \ Cm^3$$

3- Checks

a- Compactness (Local buckling)

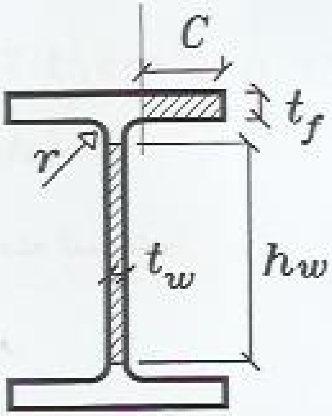
$h_w = 29.8 \ cm$ جداول

$t_w = 0.80 \ cm$

$b_f = 17.0 \ cm$

$t_f = 1.27 \ cm$

$r = 1.80 \ cm$



$$\frac{C}{t_f} = \frac{\frac{1}{2}(b_f - t_w - 2r)}{t_f} = \frac{\frac{1}{2}(17.0 - 0.80 - 2 * 1.80)}{1.27} = 4.96$$

$$\therefore \frac{C}{t_f} = 4.96 < \frac{16.9}{\sqrt{f_y}} = 10.9 \Longrightarrow \text{Compact Flange}$$

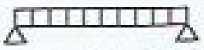
$$\frac{hw}{t_w} = \frac{29.8}{0.80} = 37.2 < \frac{127}{\sqrt{f_y}} = 82 \Longrightarrow \text{Compact Web}$$

∴ The section is compact

ملحوظة

من الممكن أخذ الـ $C = 0.4\, b_f$ مباشرة بدلا من حسابها

b-Lateral Torsional Buckling



$C_b = 1.13$

$L_{U\,act.} = \text{Zero}$ (Fully laterally supported) (Given)

$\Longrightarrow$ no L.T.B

Compact Section $\Longrightarrow F_b = \boxed{1.536\ t\backslash cm^2}$

3a-Check bending stresses

$$f_{act.} = \frac{M_X}{S_X} = \frac{13.53 * 100}{904} = 1.49\ t\backslash cm^2 < F_b = 1.536$$

904 From Tables (Safe)

3b-Check Shear stresses

$$q_{act.} = \frac{Q_{max.}}{A_{Web}} = \frac{Q_{max.}}{h * t_w} = \frac{10.82}{36 * 0.80} = 0.37\ t\backslash cm^2 < 0.35\ F_y = 0.84$$

(Safe)

3C-Check deflection

استخدمنا هذه القيمة لاننا لا نستطيع حساب الـ live load

$$\triangle_{act.} = \frac{5}{384} * \frac{w_{L.L} * S^4}{E * I_X}$$

($w_{L.L}$: t\cm, S: cm)

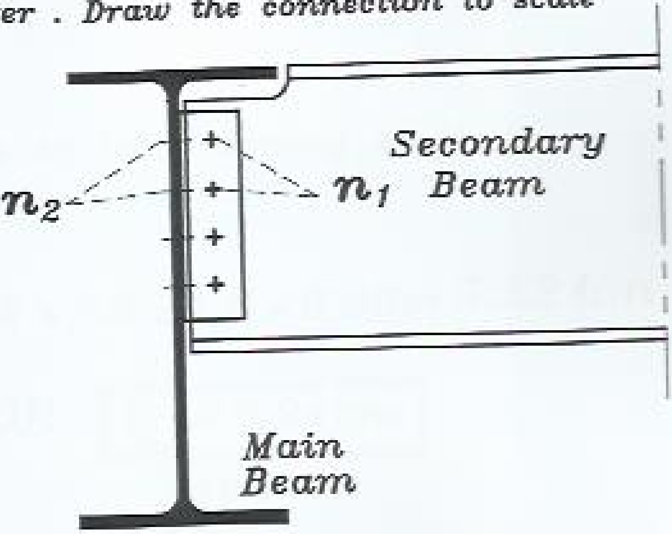
$$= \frac{5}{384} * \frac{(2.60\backslash 100) * (500)^4}{2100 * 16270} = 0.62\,cm < \frac{Span}{300}$$

(E: t\cm², I_X: cm⁴)

$$< \frac{500}{300} = 1.67\,cm$$

(Safe)

2-Design the connection between the secondary beam and the main beam using non-pretensioned bolts (bearing type) bolts grade 8.80 consider the end distance = 3 times the bolt diameter . Draw the connection to scale 1:10 in different views .



Design of connection

BOLTS M16 Grade 8.8

Threads are excluded

Design of connection

1-For n_1

(Bolts connecting the sec. beam to the framing angles)

$$R_{Shear} = q_b * A_S * n \qquad R_b = F_b * d * t_{min}$$

$* F_{ub} = 8 \ t \backslash cm^2$ $\qquad * \ \phi = 1.6$ cm

$* F_u \xrightarrow{\text{for st.37}} = 3.6 \ t \backslash cm^2$

$* q_b = 0.25 F_{ub}$ $\qquad * A_S = \frac{\pi d^2}{4}$

سوف يتم التعويض عن α بقيمة تساوى ١,٢٠ بدلا من ٠,٨، وذلك لانه كان قد اعطى ان ال Edge dist.=3ϕ

$\Longrightarrow \alpha = 1.20$

$$* R_{S.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 1 = (0.25 * 8) * \frac{\pi (1.6)^2}{4} * 1$$
$$= 4.02 \text{ ton}$$

$$* R_{D.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 2 = 2 * R_{S.S} = 8.04 \text{ ton}$$

$$* R_b = (\alpha * F_u) * d * \Sigma t_{min} = 1.2 * 3.6 * 1.6 * 0.80 = 5.52 \text{ ton}$$

Σt_{min}: Min. of $\boxed{2t_L = 0.8*2 = 1.60 \text{ cm}}$ OR $\boxed{t_{w.s.b} = 0.80 \text{ cm}}$ govern

$$* n_1 = \frac{R_{sec.beam}}{R_{Least}} = \frac{10.80}{5.52} = 2.00 = \boxed{2 \text{ Bolts}}$$

Design of connection

2-For n_2

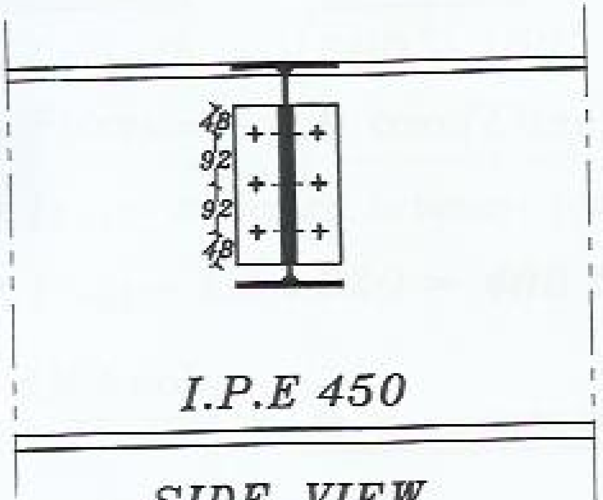
(Bolts connecting the main beam to the framing angles)

$* R_{S.S} = 4.02$ ton

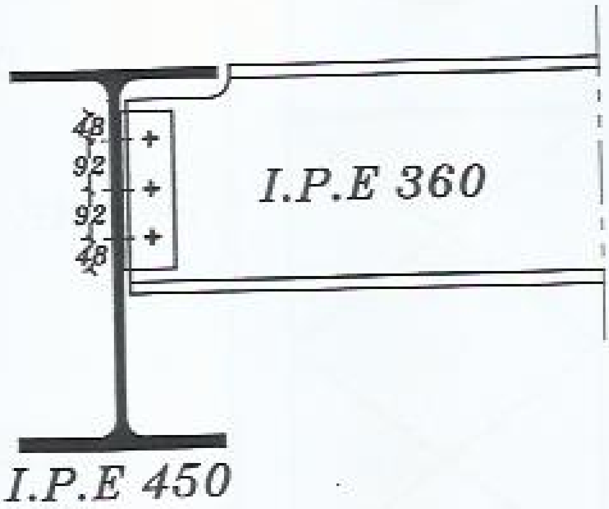
$* R_b = (\alpha * F_u) * d * \Sigma t_{min} = 1.2 * 3.6 * 1.6 * 0.80 = 5.52$ ton

Σt_{min}: Min. of $t_L = 0.8$ OR $t_{w_{m.b}} = 0.94\,cm$
govern

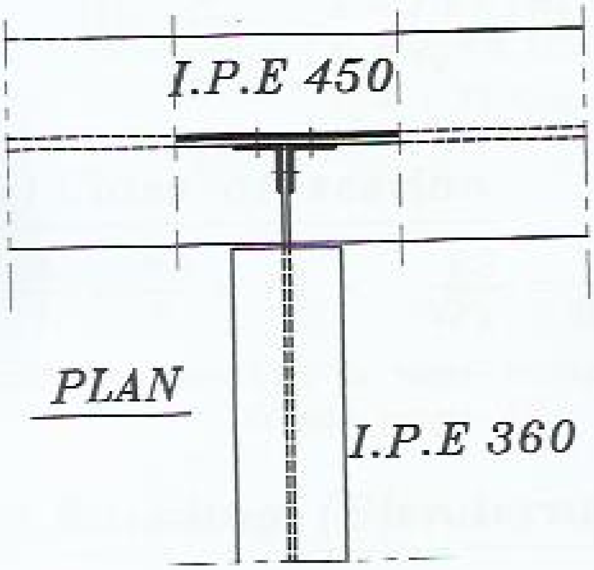
$* n_2 = \frac{R_{sec.beam}}{R_{Least}} = \frac{10.80}{4.02} =$ 4 Bolts 2 each side



SIDE VIEW



ELV.

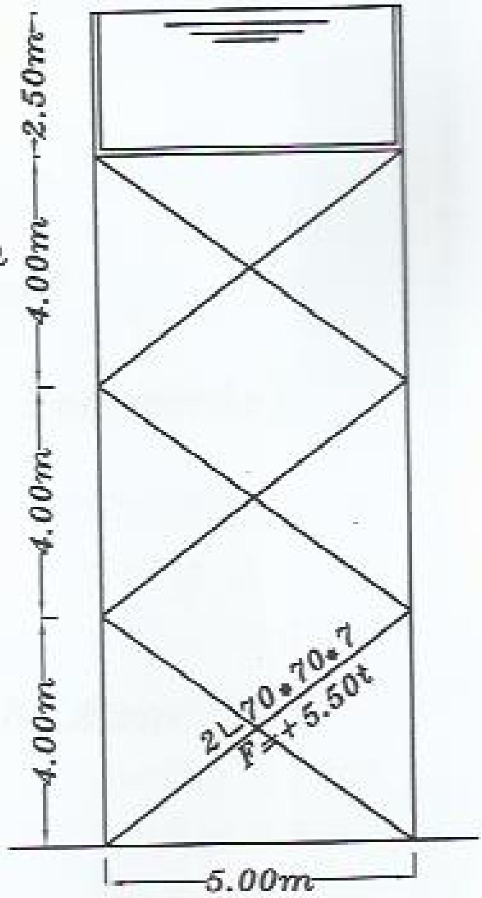


PLAN

NOTES

1- Steel used is ST.37
2- Non-pretensioned bolts M16
3- Framing angle used is 80X8
4- All. Dim. In mm.s

3- Check the stresses in the marked member shown in Figure (3)

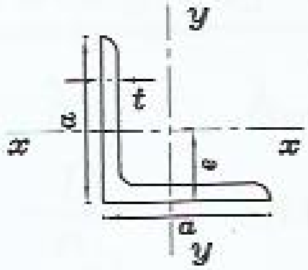


1) Data

* $Length = \sqrt{500^2 + 400^2} = 640$
* $Force = -3.0 \ ton \ (Case \ B)$
* $l_{b\,in}$ = Distance between joints = 320 cm
* $l_{b\,out} = 1.5 * 320 = 480$ cm

2) Checks

As a compression member



L 70*70*7

$A = 9.4 \ Cm^2$

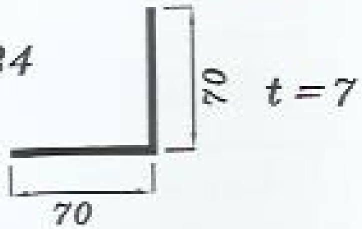
$e = 1.97 \ Cm$

$r_x = r_y = 2.12 \ Cm$

$r_v = 1.37 \ Cm$

a) Class of section

* $\frac{b}{t} = \frac{70}{7} = 10 < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$



⟹ The section is non-compact (Code page 12)

b) Buckling (Slenderness)

$r_{x\perp} = r_{x\,L}$ من الجدول $= 2.12$ cm

$$r_{y\perp} = \sqrt{r_{y\,L}^2 + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.12^2 + (1.97 + \frac{1.0}{2})^2} = 3.25 \ cm$$

$$\lambda_{in} = \frac{320}{2.12} = 150.94 < 180 \Longrightarrow (Safe)$$

$$* \ \lambda_{out} = \frac{480}{3.25} = 147.6 < 180 \Longrightarrow (Safe)$$

$$\boxed{\lambda_{max.} = 151.0}$$

c) **Stress**

$\lambda_{max.} = 151.0 > 100$

$$* F_C = \frac{7500}{\lambda^2_{max.}} = 0.328\ t \backslash cm^2$$

$$* f_C = \text{actual stress} = \frac{5.50}{2 * 9.40} = 0.29\ t \backslash cm^2$$

$\leqslant F_C \Rightarrow$ **(Safe & Economic)**

d) **Design of tie plate**

$\lambda_v \leqslant \lambda_{max.}$

$$\frac{l'}{r_{v\llcorner}} \leqslant 151 \Rightarrow l' \leqslant 1.37 * 151 = 206.8\ cm$$

$$l' > \frac{l}{2} \Rightarrow$$ Use one tie plate at the middle of member

Checks

As a Tension member

a) **Stress**

$$A_{net_{\lrcorner\llcorner}} = 2 [A_{gross\llcorner} - (\phi + 0.2\ cm) * t_{w\llcorner}]$$

$$= 2 [9.40 - (1.6 + 0.2\ cm) * 0.7] = 16.28\ cm^2$$

$$* f_{act.} = \frac{Force}{A_{net}} = \frac{5.50}{16.28} = 0.337\ t \backslash cm^2$$

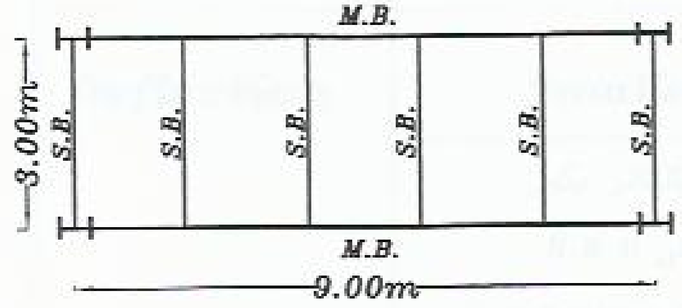
$\leqslant F_t = 1.40\ t \backslash cm^2$ **(Safe)**

b) **Deflection**

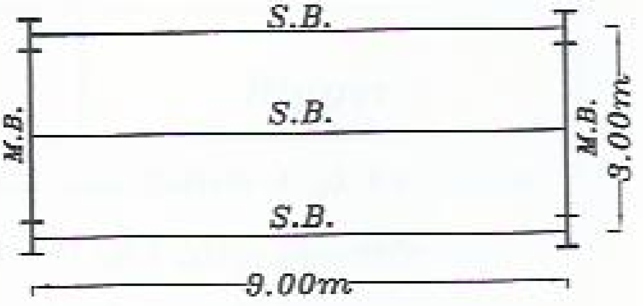
$$* \frac{L}{d} = \frac{320\ cm}{a} = \frac{320\ cm}{7} = 45.7 \leqslant 60 \Rightarrow \textbf{(Safe)}$$

Choose ⅃L 75x75x7

Question (4)



SYSTEM A



SYSTEM B

	SYSTEM A	SYSTEM B
Connections	16 Connections ٤ لل M.B مع العمود ٤ لل S.B مع العمود ٨ لل S.B مع الـ M.B	10 Connections ٤ لل M.B مع العمود ٤ لل S.B مع العمود ٢ لل S.B مع الـ M.B
Steel Weight	Heavier	Lighter
	من المتوقع أن تكون قطاعات الـ M.B فى SYSTEM A أكبر من قطاعات الـ S.B فى SYSTEM B بالاضافة الى أن مجموع أطوال كمرات SYSTEM A أكثر من SYSTEM B	
Manufacturing	Take more time عدد الكمرات أكثر و مجموع أطوالها أكبر	Take less time عدد الكمرات أقل و مجموع أطوالها أقل
Erection	Harder عدد الوصلات أكبر	Easier عدد الوصلات أقل

	SYSTEM A	SYSTEM B
Deflection	Smaller	Bigger
	الـ Span الـ S.B في System A صغير جدا و بالتالي يكون الـ deflection صغير و لكن تركيز الاحمال على الـ M.B يكون كبير و مع ذلك ليس من المتوقع حدوث deflection كبير حيث أن الـ M.B سيكون قطاعها كبير و بالتالي تستطيع مقاومة الـ deflection .	
L.T.B	Zero (Conc. slab)	Zero (Conc. slab)