STeel سلاسل

mid-Term

A/۱۲ 11

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11

MIDTERM REVISION

EXAMPLES

&

IDEAS

(2012-2013)

طريقة المراجعة

١ - قراء ملازم الشرح بسرعة جدا مع حل مسألة من كل ملزمة باليد .

٢ - حل مسألة صفحة ٩٢ فى ملزمة ال *Tension Members*

٣ - حل مسألة صفحة ٣٥ فى ملزمة ال *Quizzes*

٤ - حفظ خطوات تصميم ال *Tension and Compression members* و خطوات تصميم ال *Connections* من الورق ال *A3*

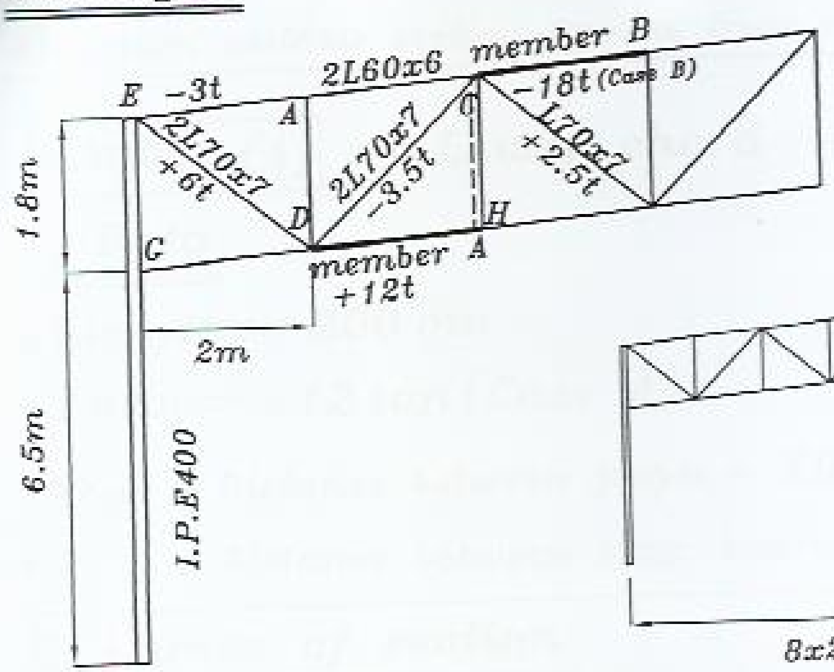
٥ - حل اللى نقدر عليه من ملازم المراجعة و قراء الباقى و ملازم المراجعة هم ثلاثة ملازم 12 & 11 & 7 .

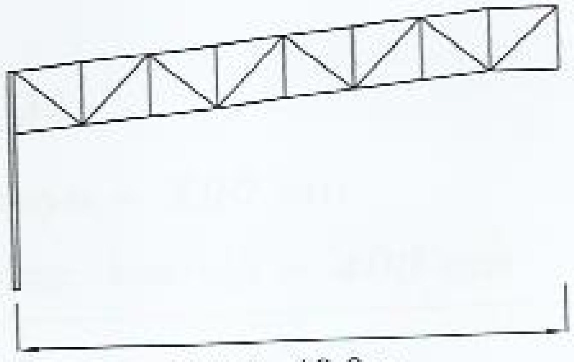
٦ - عند تحليل القوى يفضل الحل بال *Graphical Solution* لان الدكتور يستخدمها .

فى الامتحان

- لابد من أخذ جداول القطاعات
- احتياطيا الصفحات التى تحتاجها من الكود و لكن الدكتور سيعطيها فى الامتحان

Example





The figure show the typical truss of an industrial building:

<u>It is required to :</u>

a) Design suitable section for the truss members A&B.

b) Design Connections C&D using 16mm. diameter ordinary bolts.

Solution

a) Design suitable section for the truss members A&B.

Member (A) ⇒ **Lower chord** ⇒ ⅃L

1) Data

* Length = 200 cm
* Force = + 12 ton (Case A)
* $l_{b\,in}$ = Distance between joints = 200 cm
* $l_{b\,out}$ = Distance between long. bracing = 400 cm

2) Choice of section

a – From Stress Condition

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

Bolted

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

من الجدول ⟹ $\boxed{\text{Choose } \rfloor L\ 60x60x6}$ $a_1 = 6.0$ cm

b – From Slenderness Condition

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

$$\therefore 300 = \frac{l_{b\,in}}{r_{x_{\rfloor L}}} = \frac{200}{0.30a} \Rightarrow a = 2.22^{cm}$$

$$\therefore 300 = \frac{l_{b\,out}}{r_{y_{\rfloor L}}} = \frac{400}{0.45a} \Rightarrow a = 2.96^{cm}$$

نأخذ الاكبر $a_2 = 2.96^{cm}$

C – From Construction Condition

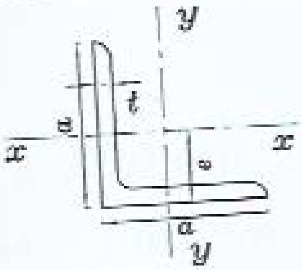
minimum angle $a_{min} = 1.1 * 3\ \phi = 1.1\ \ 3 * 1.6 = 5.28$ cm

$$a_3 = 5.28\ cm$$

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

$\boxed{\text{Choose } ⅃L\ 60x60x6}$

3) Checks



L 60*60*6
$A = 6.91\ Cm^2$
$e = 1.69\ Cm$
$r_x = r_y = 1.82\ Cm$

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

$$= 2\,[\,6.91 - (1.6 + 0.2\,cm) * 0.6\,] = 9.50\ cm^2$$

a) Stress $_{Tension}$

$$* \ f_t = \frac{Force}{A_{net}} = \frac{12}{9.50} = 1.26\ t \backslash cm^2 \leq F_t = 1.40\ t \backslash cm^2\ (Safe)$$

مساحة ال angles التى تم حسابها

b) Slenderness

$r_{x_{⅃L}} = r_{x_L}$ من الجدول $= 1.82\ cm$

assume $t_{Gp} = 1\ cm$

$$r_{y_{⅃L}} = \sqrt{r_{y_L}^2 + (e + \frac{t_{Gp}}{2})^2} = \sqrt{1.82^2 + (1.69 + \frac{1.0}{2})^2} = 2.85\ cm$$

$$* \ \lambda_{in} = \frac{l_{b\,in}}{r_{x_{⅃L}}} = \frac{200}{1.82} = 109 < 300 \Rightarrow (Safe)$$

$$* \ \lambda_{out} = \frac{l_{b\,out}}{r_{x_{⅃L}}} = \frac{400}{2.85} = 140.3 < 300 \Rightarrow (Safe)$$

C) Length to depth ratio (Deflection)

$$* \ \frac{L}{d} = \frac{200\ cm}{a} = \frac{200\ cm}{6.0} = 33.3 \leq 60 \Rightarrow (Safe)$$

Member (B) $\Rightarrow$ Upper chord $\Rightarrow$ ⊤⊤

1) Data

* Length $= 200$ cm
* Force $= -18$ ton (Case B)
* $l_{b\,in}$ = Distance between joints $= 200$ cm
* $l_{b\,out}$ = Distance between Purlins $= 200$ cm

2) Choice of section

From stresses

* assume $F_C = 0.75 * 1.20\ t \backslash cm^2$

$$\therefore A_{g\,\lrcorner\llcorner} = \frac{force}{F_C} = \frac{18}{0.75 * 1.20} = 20.0\ cm^2$$

$$\therefore A_{g\,\llcorner} = \frac{A_{g\,\lrcorner\llcorner}}{2} = \frac{20.0}{2} = 10.0\ cm^2$$

Choose $\xrightarrow{tables}$ $\llcorner\ 80*80*8$

From buckling

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

$$\therefore 100 = \frac{l_{b\,in}}{r_x} = \frac{200}{0.30 a_2}$$

$$\Rightarrow a_2 = \boxed{6.6\ cm}$$

$$\therefore 100 = \frac{l_{b\,out}}{r_y} = \frac{200}{0.45 a_3}$$

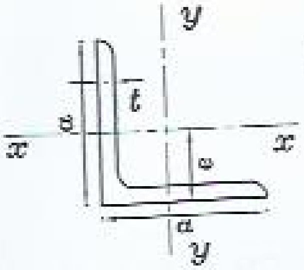
$$\Rightarrow a_3 = 4.44\ cm$$

$$a_{av} = \frac{a_1 + (\overset{\text{الاكبر}}{a_2\ Or\ a_3})}{2} = \frac{8.0+6.6}{2} = 7.3\ cm$$

$\boxed{\text{Choose } \lrcorner\llcorner\ 75*75*7}$

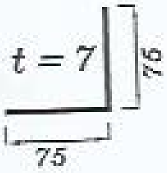
$>$ minimum angle $a_{min} = 1.1*3\,\phi = 1.1*3*1.6 = 5.28\ cm$

3) Checks



$\llcorner 75*75*7$
$A = 10.1\ Cm^2$
$e = 2.09\ Cm$
$r_x = r_y = 2.28\ Cm$
$r_v = 1.37\ Cm$

a) Class. of section



$* \frac{b}{t} = \frac{75}{7} = 10.7 < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$

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

b) Buckling

$r_{x_{\lrcorner\llcorner}} = r_{x_{\llcorner}}$ من الجدول $= 2.28\ cm$

assume $t_{Cp} = 1\ \ cm$

$$r_{y_{\lrcorner\llcorner}} = \sqrt{r^2_{y\llcorner} + (e + \frac{t_{Cp}}{2})^2} = \sqrt{2.28^2 + (2.09 + \frac{1.0}{2})^2} = 3.45\ cm$$

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x_{\lrcorner\llcorner}}} = \frac{200}{2.28} = 87.72 < 180 \Longrightarrow \text{(Safe)}$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{y_{\lrcorner\llcorner}}} = \frac{200}{3.45} = 57.97 < 180 \Longrightarrow \text{(Safe)}$$

c) Stress

$\lambda_{max.} = 87.72 \leqslant 100$

$$* F_C = 1.2 * [1.4 - 6.5 * 10^{-5} \lambda^2_{max.}] = 1.2 * [1.4 - 6.5 * 10^{-5} (87.72)^2]$$
$$= 1.08\ \ t \setminus cm^2$$

$$* f_C = \text{actual stress} = \frac{force}{2 * A_{g\llcorner}} = \frac{18}{2 * 10.1} = 0.89\ \ t \setminus cm^2$$

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

Design of tie plate

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

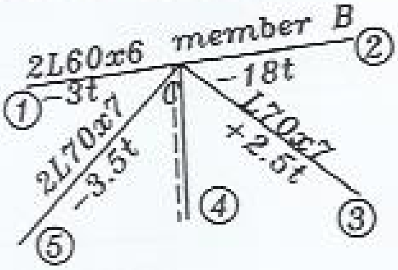
$$\frac{l'}{r_{vL}} = \frac{l'}{1.45} \leqslant 87.72 \Rightarrow l' \leqslant 1.45 * 87.72 = 127\,cm$$

$$l' \leqslant 127\,cm > \frac{l}{2} \Rightarrow$$ *Use one tie plate at the middle of member*

b) Design Connections C&D using 16mm. diameter ordinary bolts.

M16 ordinary bolts grade (4.6)

Connection (C)



$$\boxed{R_{Shear} = q_b * A_S * n}$$

$$\boxed{R_b = F_b * d * t_{min}}$$

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

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

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

$* \text{ Take } e > 2\phi = 3.2\ cm \Longrightarrow \alpha = 0.8$

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

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

$* R_b = (\alpha * F_u) * d * t_{min} = 0.8 * 3.6 * 1.6 * t_{min} = 4.6\ t_{min}$

Member (1)

$\daleth\!\!\ulcorner\ 60 * 60 * 6$

$* t_{min} = \overset{t_{G.p}}{1\ cm}$ or $\overset{t_{\llcorner}}{2 * 0.6\ cm} = 1.2\ cm \Longrightarrow \boxed{t_{min} = 1\ cm}$

$* R_b = 4.6\ t_{min} = 4.6 * 1 = \boxed{4.60\ ton}$

$* R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.60 \end{cases}$ $\quad R_{Least} = \boxed{4.02\ ton}$

$* n_1 = \dfrac{\text{Force}}{R_{Least}} = \dfrac{3.0}{4.02} = 0.75 \Longrightarrow \boxed{2\ \text{Bolts}}$

Compression force $\Longrightarrow$ No need to check block shear rupture

$\boxed{\text{Member (2)}}$ $\;\;⅂⌐\; 75*75*7$

$* \; t_{min} = \overset{t_{G.p}}{1\,cm} \quad \text{or} \quad \overset{t_{\llcorner}}{2*0.7}\ cm = 1.4\,cm \Longrightarrow \boxed{t_{min} = 1\,cm}$

$* \; R_b = 4.6\ t_{min} = 4.6 * 1 = \boxed{4.60\ ton}$

$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.60 \end{cases} \qquad R_{Least} = \boxed{4.02\ ton}$

$* \; n_2 = \dfrac{Force}{R_{Least} * 1.2} = \dfrac{18.0}{4.02*1.2} = 3.73 \Longrightarrow \boxed{4\ Bolts}$

Compression force $\Longrightarrow$ No need to check block shear rupture

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$\boxed{\text{Member (3)}}$ $\;\; \llcorner 70*70*7$

$* \; t_{min} = \overset{t_{G.p}}{1\,cm} \quad \text{or} \quad \overset{t_{\llcorner}}{0.7}\ cm \Longrightarrow \boxed{t_{min} = 0.7\ cm}$

$* \; R_b = 4.6\ t_{min} = 4.6 * 0.7 = \boxed{3.02\ ton}$

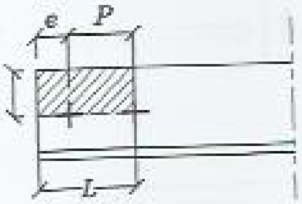
$* \; R_{Least} \rightarrow \begin{cases} R_{s.s} = \boxed{2.01} \\ R_b = 3.02 \end{cases} \qquad R_{Least} = \boxed{2.01\ ton}$

$* \; n_3 = \dfrac{Force}{R_{Least}} = \dfrac{2.50}{2.01} = 1.20 \Longrightarrow \boxed{2\ Bolts}$

Check block shear rupture

$*$ Take $e = 3.5\ cm$

$P = 5.0\ cm > P_{min.} = 3\ \phi$



$* \; m = \dfrac{a-t}{2} = \dfrac{7.5-0.7}{2} = 3.40\ cm$

$* \; L = e + 1*P = 3.5 + 1*5 = 8.5\,cm$

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

$$= [\ 8.5 \ - (\ 2 - 0.5\ )(\ 1.6 + 0.2\ )\,] * 0.7 = 4.06\,cm^2$$

$$* \; A_{net\ Tension} = [\,m - 0.5*(\phi + 0.2)\,] * t_{\llcorner}$$

$$= [\ 3.40 - 0.5 * (\ 1.6 + 0.2\ )\,] * 0.7 = 1.75\,cm^2$$

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

$= 0.4 * 2.4 * 4.06\ \ + 0.725 * 2.4 * 1.75 = 6.94\ ton$

* Check $\Longrightarrow$ $P = 6.94 ton > Tension\ force = 3.0 ton$

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

Member (5) $\neg\!\!\Gamma\ 70*70*7$

* $t_{min} = \overset{t_{G.p}}{1 cm}$ or $\overset{t_{\llcorner}}{2*0.7\ cm} = 1.4\ cm \Longrightarrow \boxed{t_{min} = 1\ cm}$

* $R_b = 4.6\ t_{min} = 4.6 * 1 = \boxed{4.60\ ton}$

* $R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.60 \end{cases}$ $\quad R_{Least} = \boxed{4.02\ ton}$

* $n_5 = \dfrac{Force}{R_{Least}} = \dfrac{3.50}{4.02} = 0.87 \Longrightarrow \boxed{2\ Bolts}$

Compression force $\Longrightarrow$ No need to check block shear rupture

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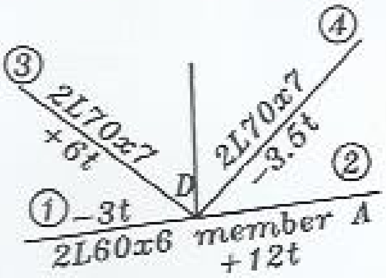
Connection (D)



Members (1,2) $\lrcorner\!\!\llcorner\ 60*60*6$

Design as continuous member

* $t_{min} = \overset{t_{G.p}}{1 cm}$ or $\overset{t_{\llcorner}}{2*0.6\ cm} = 1.2\ cm \Longrightarrow \boxed{t_{min} = 1\ cm}$

* $R_b = 4.6\ t_{min} = 4.6 * 1 = \boxed{4.60\ ton}$

* $R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.60 \end{cases}$ $\quad R_{Least} = \boxed{4.02\ ton}$

$F_{des.} \Longrightarrow$ **Design force** $\begin{cases} F_2 - F_1 \\ 12 - (-3) = \boxed{15} \quad \text{الاكبر} \\ \\ F_3 \cos\alpha - F_4 \cos\alpha \\ 6\ \cos 42 - (-3.5 \cos 42) = 7.1 \end{cases}$

$$* n_{1-2} = \frac{Force}{R_{Least}} = \frac{15}{4.02} = 3.70 \Longrightarrow \boxed{4\ Bolts}$$

Continuous Joint $\Longrightarrow$ No need to check block shear rupture

$\boxed{\textbf{Member (3)}}$ $\rfloor\llcorner$ 70*70*7

$$* t_{min} = \overset{t_{G.P}}{1\ cm} \quad or \quad \overset{t_{\llcorner}}{2*0.7\ cm} = 1.4\ cm \Longrightarrow \boxed{t_{min} = 1\ cm}$$

$$* R_b = 4.6\ t_{min} = 4.6 * 1 = \boxed{4.60\ ton}$$

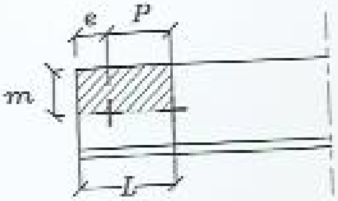
$$* R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.60 \end{cases} \qquad R_{Least} = \boxed{4.02\ ton}$$

$$* n_3 = \frac{Force}{R_{Least}} = \frac{6.0}{4.02} = 1.50 \Longrightarrow \boxed{2\ Bolts}$$

Check block shear rupture

* Take $e = 3.5\ cm$

$P = 5.0\ cm > P_{min} = 3\ \phi$



$$* m = \frac{a-t}{2} = \frac{7.0-0.7}{2} = 3.15\ cm$$

$$* L = e + 1*P = 3.5 + 1*5 = 8.5\ cm$$

$$* A_{net\ Shear} = [\ L - (n - 0.5)(\phi + 0.2)] * t_{\llcorner}$$
$$= [\ 8.5 - (2 - 0.5)(1.6 + 0.2)] * 0.7 = 4.06\ cm^2$$

$$* A_{net\ Tension} = [m - 0.5 * (\phi + 0.2)] * t_{\llcorner}$$
$$= [\ 3.15 - 0.5 * (1.6 + 0.2)] * 0.7 = 1.58\ cm^2$$

$$* \ P = \mathbf{0.4 F_y\ A_{net\ Shear} + 0.725\ F_y A_{net\ Tension}}$$
$$= 0.4 * 2.4 * 4.06 + 0.725 * 2.4 * 1.58 = 6.64\ ton$$

* Check $\Longrightarrow$ $2P = 13.3\ ton >$ Tension force $= 6.0\ ton$

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

$\boxed{\textbf{Member (4)}}$ $\rfloor\!\lfloor 70*70*7$

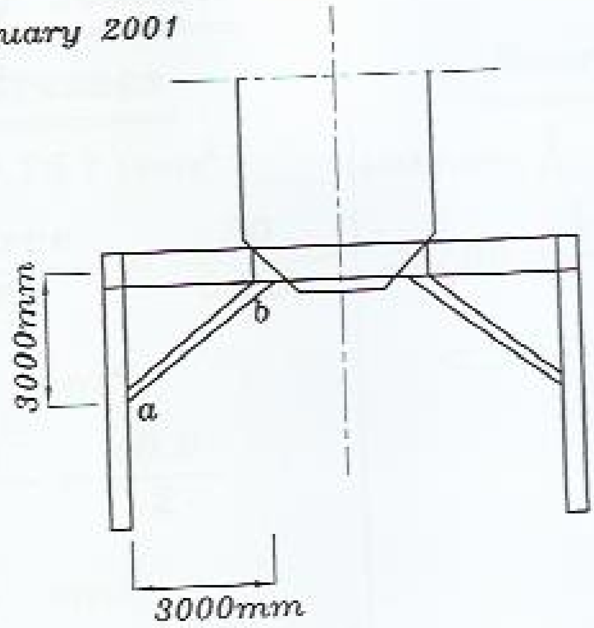
$* \; t_{min} = \overset{t_{G.P}}{1\,cm} \quad or \quad 2*\overset{t_{L}}{0.7}\ cm = 1.4\,cm \Longrightarrow \boxed{t_{min} = 1\,cm}$

$* \; R_b = 4.6\ t_{min} = 4.6 * 1 = \boxed{4.60\ ton}$

$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.60 \end{cases} \qquad R_{Least} = \boxed{4.02\ ton}$

$* \; n_4 = \dfrac{\textbf{Force}}{R_{Least}} = \dfrac{3.5}{4.02} = 0.87 \Longrightarrow \boxed{2\ \text{Bolts}}$

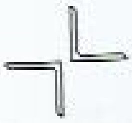
Example January 2001

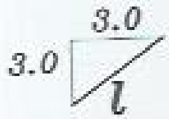


For the shown figure, it is required to design member (a–b) for compression force −30 ton. (Case A) using M20 pretensioned bolts. Then design the bolted connection at point (a) using M20 pretensioned bolts (grade 10.9) with connecting angles.

Solution

Member (ab) ⇨





1) Data

* $Length = \sqrt{300^2 + 300^2} = 424\ cm$
* $Force = -30\ ton\ (Case\ A)$
* $l_{b\,in}$ = Distance between joints = 190 cm
* $l_{b\,out}$ = Distance between joints = 190 cm

لان الطول كبير نسبيا لذلك يفضل استخدام Star shaped و ذلك لان الـ r المقاومة للBuckling تكون تقريبا 0.38a فى حين عند استخدام angles back to back تكون الـ r الاصغر هى 0.3a

و حيث أن الـ Buckling length متساوى In&out لذلك يفضل استخدام Star shaped

2) Choice of section

From stresses

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

$$\therefore A_{g\,\neg\llcorner} = \frac{force}{F_C} = \frac{30}{0.75}$$

$$= 40.0 \quad cm^2$$

$$\therefore A_{g\,\llcorner} = \frac{A_{g\,\neg\llcorner}}{2} = \frac{40.0}{2}$$

$$= 20.0 \quad cm^2$$

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

From buckling

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

$$\therefore 100 = \frac{l_{b\,out}}{r_u} = \frac{424}{0.38 a_2}$$

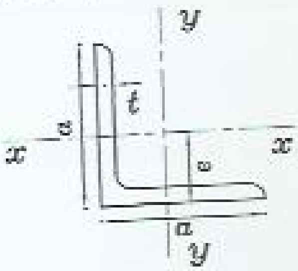
$$\Longrightarrow a_2 = \boxed{11.15\ cm}$$

$$a_{av} = \frac{a_1 + a_2}{2} = \frac{11 + 11.15}{2} = 11.07 cm$$

$\boxed{\text{Choose } \lrcorner\llcorner\ 110*110*10}$

$>$ minimum angle $a_{min} = 1.1*3\,\phi = 1.1*3*2.0 = 6.6\ cm$

3) Checks



$\llcorner 110*110*10$

$A = 21.2 Cm^2$

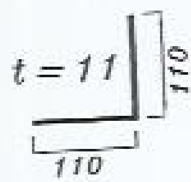
$e = 3.07\ Cm$

$r_u = 4.23\ Cm$

$r_v = 2.16\ Cm$

$r_{u\,\neg\llcorner} = r_{u\,\llcorner}$ من الجدول $= 4.23\ cm$

a) Class. of section



$$* \ \frac{b}{t} = \frac{75}{7} = 10.7 < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$$

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

b) Buckling

$$* \ \lambda_{out} = \frac{l_{b\,out}}{r_{u\,\neg\llcorner}} = \frac{424}{4.23} = 100.2 < 180 \Longrightarrow \text{(Safe)}$$

c) Stress

$\lambda_{max.} = 100.2 > 100$

$$*F_C = \frac{7500}{\lambda^2_{max.}} = \frac{7500}{100.2^2} = 0.74 \ t \backslash cm$$

$$*f_C = \text{actual stress} = \frac{force}{2 * A_{g\llcorner}} = \frac{30}{2 * 21.2} = 0.707 \ t \backslash cm^2$$

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

Design of tie plate

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

$$\frac{l'}{r_{v\llcorner}} = \frac{l'}{2.16} \leqslant 100.2 \Longrightarrow l' \leqslant 2.16 * 100.2 = 216.4cm$$

$$l' \leqslant 216.4 \ cm > \frac{l}{2} \Longrightarrow$$ Use one tie plate at the middle of member

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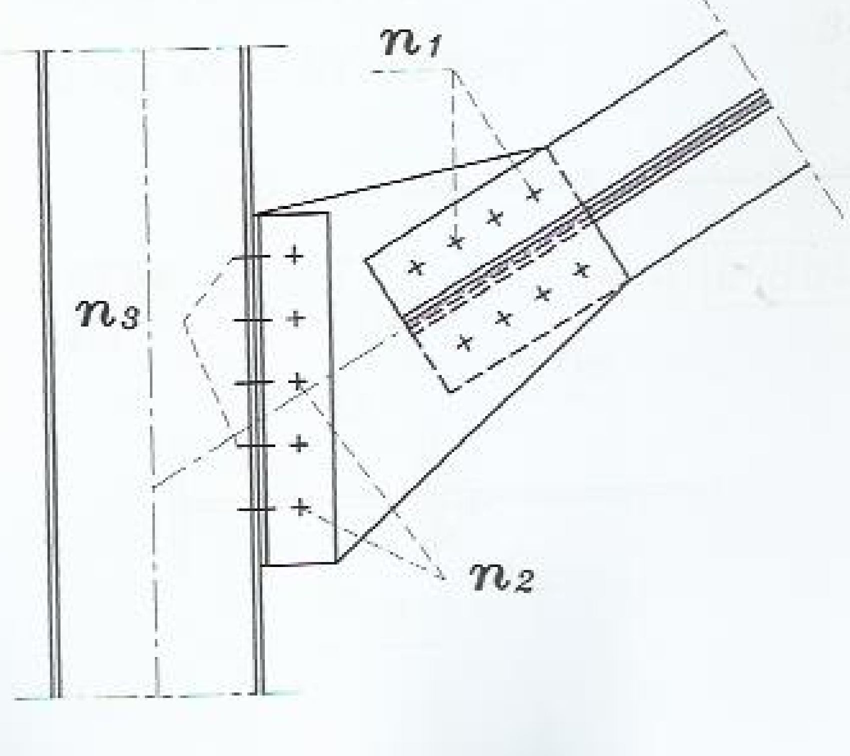
Design of connection (A)

هى وصلة تشبه وصلات الـ Truss members مع العمود و لتسهيل الحل نرسمها كروكي أولا



From code $\xrightarrow[\text{M16(Grade 10.9)}]{\text{Page 106}}$ $P_S = 4.93\ t$

$\boxed{\text{No Block shear rupture for slip-critical connection}}$

$\boxed{\text{For } n_1}$ $\Longrightarrow$ **Shear**

Connecting ┐└ 110 * 110 * 10 with gusset plate

$$*\,n_1 = \frac{\text{Force}}{P_S} = \frac{30}{4.93} = 6.08 = \boxed{8 \text{ Bolts}}$$

4 bolts for each angle of the star shaped

$\boxed{\text{For } n_2}$ $\Longrightarrow$ **Shear**

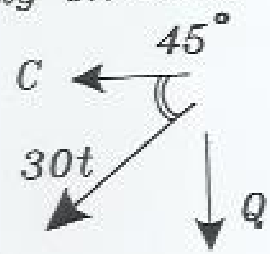
Connecting gusset plate with connecting angles

$$*\,n_2 = \frac{\text{Force}}{2 * P_S} = \frac{30}{2 * 4.93} = 3.04 = \boxed{4 \text{ Bolts}}$$

$\boxed{\text{For } n_3}$ $\Longrightarrow$ **Shear + Compression**

Connecting the column with connecting angles

Neglect the compression force and design only on shear



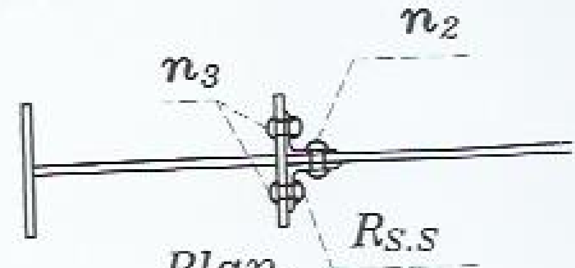
$*\ Q = 30 \cos 45 = 21.12$ ton

$*\ C = 30 \sin 45 = 21.12$ ton

دائما يتم اهمال الضغط على المسامير عند تصميمها حيث أنه غير مؤثر .

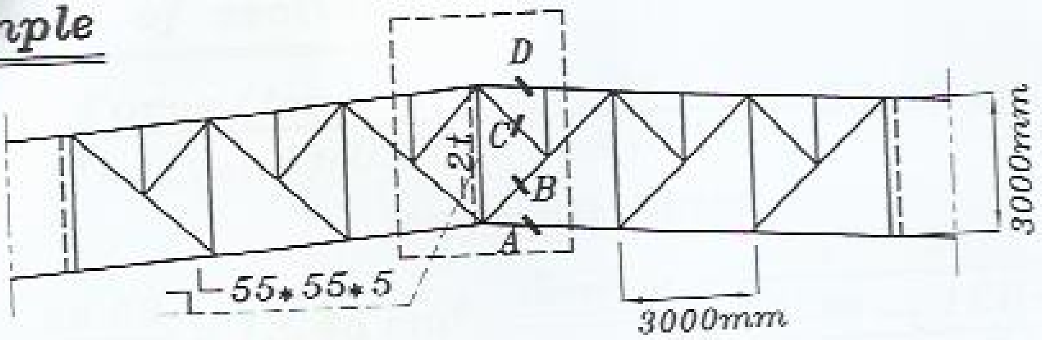
$$*\,n_3 = \frac{\text{Force}}{P_S} = \frac{21.12}{4.93} = 4.30 = \boxed{6 \text{ Bolts}}$$

3 bolts for each side of column



Plan

Example



For the truss shown, it is required to :

1) Design economic section for member (A) F=+32t, and for member (B) F=−1.5t.

__Note__ Use Unequal angles for member (A)

2) Design the joint in the enclosed dotted rectangle using M16 ordinary bolts (Grade 4.6), then draw it to scale 1:10

*Member (C) ⟹ └55*55*5 F = +0.7 t*

*Member (D) ⟹ ┘└ 90*90*9 F = −35 t*

$\boxed{\text{Member (A)}}$ ⟹ Lower chord ⟹ ┘└

1) Data

* Length = **300** cm
* Force = + **32** ton (Case A)
* $l_{b\,in}$ = Distance between joints = **300** cm
* $l_{b\,out}$ = Distance between long. bracing = **900** cm

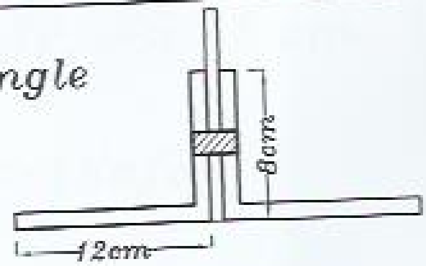
$$l_{b\,out} >>>> l_{b\,in} \xrightarrow{\text{For economic section}} \text{Use unequal angles}$$

2) Choice of section

$$A_{g_{\rfloor\lfloor}} = \frac{\textbf{Force (ton)}}{F_t\ (t\backslash cm^2)*\mathbf{0.85}} = \frac{32}{1.4*0.85} = 26.89\ cm^2$$

symmetric bolted

$$A_{g_{\lfloor}} = \frac{26.89}{2} = 13.45\ cm^2 \xrightarrow{\text{من الجدول}} \boxed{\text{Choose } \lfloor\ 120*80*8}$$



$a = 80\ mm = 8.0cm\ >\ minimum\ angle$

$a_{min} = 1.1*3\,\phi = 1.1*3*1.6 = 5.28\,cm$

المسمار يكون فى الضلع اللى طوله 8.0cm

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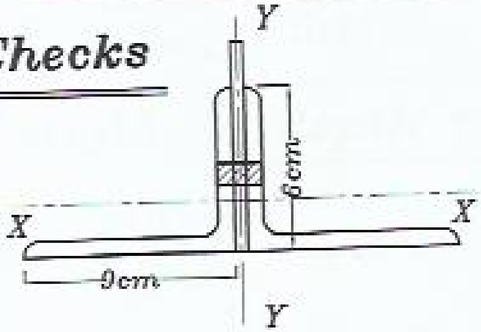
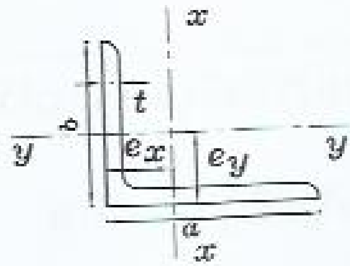
3) Checks





$\lfloor\ 120*80*8$
$A = 15.5\,Cm^2$
$e_x = 3.83\ Cm$
$r_x = 3.82\ Cm$
$r_y = 2.29\ Cm$

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

$$= 2\,[\,\mathbf{15.5} - (1.6 + 0.2\,cm) * 0.8\,] = \mathbf{28.12}\ cm^2$$

a) Stress

$$*\ f_t = \frac{Force}{A_{net}} = \frac{32}{28.12} = 1.14\ \ t\backslash cm^2$$

مساحة الـ angles التى تم حسابها

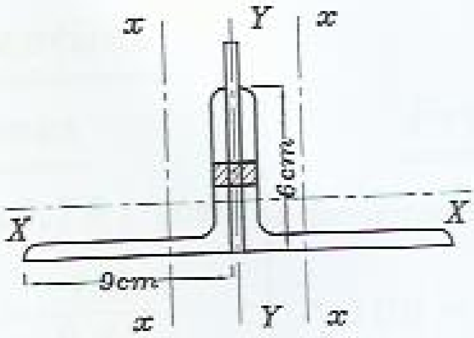
$$\leqslant F_t = 1.40\ t\backslash cm^2\ \ \textbf{(Safe)}$$

b) Slenderness

خد بالك

محاور الـــ Unequal angles معكوسة فى الجدول

$r_{x_{\rfloor\lfloor}} = r_{y_{\lfloor}}$ من الجدول $= 2.29\ cm$



$t_{Gp} = 1\ cm$

$$r_{y_{\lrcorner\llcorner}} = \sqrt{r_{x_{\llcorner}}^2 + (e_x + \frac{t_{Gp}}{2})^2} = \sqrt{3.82^2 + (3.83 + \frac{1.0}{2})^2} = 5.77\ cm$$

$$*\ \lambda_{in} = \frac{l_{b\,in}}{r_{x_{\lrcorner\llcorner}}} = \frac{300}{900} = 131 \quad < 300 \Longrightarrow (Safe)$$

$$*\ \lambda_{out} = \frac{l_{b\,out}}{r_{y_{\lrcorner\llcorner}}} = \frac{900}{5.77} = 155.72 < 300 \Longrightarrow (Safe)$$

C) Length to depth ratio (Deflection)

$$*\ \frac{L}{d} = \frac{300\ cm}{8\ cm} = 37.5 \leqslant 60 \Longrightarrow (Safe)$$

البعد الرأسى للقطاع هو الذى يقاوم ال Deflection

$\Longrightarrow$ $\boxed{Use \lrcorner\llcorner 120*80*8}$

Member (B) ⇨ Diagonal ⇨ └

1) Data

* $Length = \sqrt{150^2 + 150^2} = 212\ cm$
* $Force = -1.5\ ton\ (Case\ A)$
* $l_{b\,in}$ = Distance between joints = 212 cm
* $l_{b\,out} = 424\ cm$

2) Choice of section

From stresses

* assume $F_C = 0.60 * 0.75\ t \backslash cm^2$

$$\therefore A_{g\,\llcorner} = \frac{force}{F_C} = \frac{1.5}{0.45}$$

$$= 3.33\ cm^2$$

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

From buckling

* assume $\lambda_{out} = 100$

$$\therefore 100 = \frac{l_{bout}}{r_v} = \frac{424}{0.20 a_2}$$

$$\Rightarrow a_2 = \boxed{21.2\ cm}$$

$$a_{av} = \frac{a_1 + a_2}{2} = \frac{4.5 + 21.2}{2} = 12.85\,cm$$

$$\boxed{\llcorner\ 130 * 130 * 12}$$

هذه الـ angle كبيرة جدا لذلك يفضل استخدام 2 angles back to back

2) Choice of section

From stresses

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

$$\therefore A_{g\,\lrcorner\llcorner} = \frac{force}{F_C} = \frac{1.5}{0.75}$$

$$= 2.0\ cm^2$$

$$\therefore A_{g\,\llcorner} = \frac{A_{g\,\lrcorner\llcorner}}{2} = \frac{2.0}{2}$$

$$= 1.0\ cm^2$$

Choose $\xrightarrow{tables}$ ∟30 * 30 * 3

From buckling

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

$$\therefore 100 = \frac{l_{bin}}{r_x} = \frac{212}{0.30 a_2}$$

$$\Rightarrow a_2 = 7.06\,cm$$

$$\therefore 100 = \frac{l_{bout}}{r_y} = \frac{424}{0.45 a_3}$$

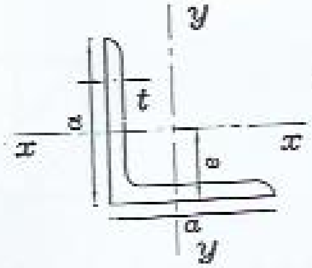
$$\Rightarrow a_3 = \boxed{9.42\,cm}$$

$$a_{av} = \frac{a_1 + (\overset{\text{الاكبر}}{a_2\ Or\ a_3})}{2} = \frac{3 + 9.42}{2} = 6.21\,cm$$

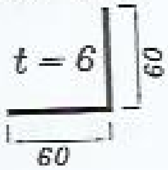
$$\boxed{\text{Choose } \lrcorner\llcorner\ 60*60*6}$$

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

3) Checks



$\llcorner\, 60*60*6$
$A = 6.91\ Cm^2$
$e = 1.69\ Cm$
$r_x = r_y = 1.82\ Cm$
$r_v = 1.17\ Cm$



a) Class. of section

$* \ \frac{b}{t} = \frac{60}{6} = 10 \quad < \frac{23}{\sqrt{F_y}} = \frac{23}{\sqrt{2.4}} = 14.84$

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

b) Buckling

$r_{x_{\lrcorner\llcorner}} = r_{x_{\llcorner}}$ من الجدول $= 1.82\ cm$

assume $t_{Gp} = 1\ cm$

$$r_{y_{\lrcorner\llcorner}} = \sqrt{r^2_{y\llcorner} + (e + \frac{t_{Gp}}{2})^2} = \sqrt{1.82^2 + (1.69 + \frac{1.0}{2})^2} = 2.85\ cm$$

$$* \ \lambda_{in} = \frac{l_{b\,in}}{r_{x_{\lrcorner\llcorner}}} = \frac{212}{1.82} = 74.38 < 180 \Longrightarrow \textbf{(Safe)}$$

$$* \ \lambda_{out} = \frac{l_{b\,out}}{r_{y_{\lrcorner\llcorner}}} = \frac{424}{2.85} = 148.7 < 180 \Longrightarrow \textbf{(Safe)}$$

c) Stress

$\lambda_{max.} = 148.7 \leqslant 100$

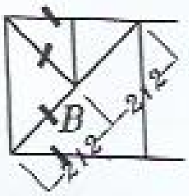
$$* F_C = \frac{7500}{\lambda^2_{max.}} = \frac{7500}{148.7} = 0.34\ t \setminus cm^2$$

$$* f_C = \text{actual stress} = \frac{force}{2 * A_{g\llcorner}} = \frac{1.5}{2 * 6.91} = 0.11\ t \setminus cm^2$$

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

You can try using $\lrcorner\llcorner\, 55*55*5$

Design of tie plate

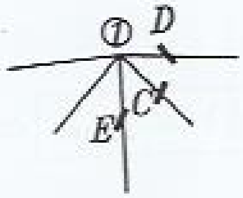


$\lambda_v \leqslant \lambda_{max.}$

$\frac{l'}{r_{vL}} = \frac{l'}{1.17} \leqslant 148.7 \Rightarrow l' \leqslant 1.17 * 148.7 = 174\ cm$

$l' \leqslant 174\, cm > \frac{l}{2} = \frac{212}{2} \Rightarrow$ *Use one tie plate at the middle of member*

Design of connection (1)



Member (C) $\Longrightarrow$ $\llcorner 55*55*5$ $\Longrightarrow$ $F = +0.7\ t$

Member (D) $\Longrightarrow$ $\lrcorner\llcorner 90*90*9$ $\Longrightarrow$ $F = -35\ t$

Member (E) $\Longrightarrow$ $\urcorner\llcorner 55*55*5$ $\Longrightarrow$ $F = -2\ t$

M 16 , Grade 4.6

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

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

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

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

$*$ Take $e > 2\phi = 3.2\ cm \Longrightarrow \alpha = 0.8$

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

$= 2.01\ ton$

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

$* R_b = (\alpha * F_u) * d * t_{min} = 0.8 * 3.6 * 1.6 * t_{min} = 4.6\ t_{min}$

Member (C) $\llcorner 55*55*5$

$* t_{min} = \overset{t_{G.p}}{1\ cm}$ or $\overset{t_{\llcorner}}{0.5\ cm}$ $\Longrightarrow \boxed{t_{min} = 0.5\ cm}$

$* R_b = 4.6\ t_{min} = 4.6 * 0.5 = \boxed{2.30\ ton}$

$* R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{2.01} \\ R_b = 2.30 \end{cases}$ $R_{Least} = \boxed{2.01\ ton}$

$* n_C = \dfrac{\text{Force}}{R_{Least}} = \dfrac{0.70}{2.01} = 0.38 \Longrightarrow \boxed{2\ \text{Bolts}}$ Minimum

$\boxed{\Longrightarrow \text{Check B.S.R}}$

Member (D) $\neg\!\Gamma\ 90*90*9$

$* \ t_{min} = \overset{t_{G.p}}{1\,cm} \quad or \quad \overset{t_{\llcorner}}{2*0.9}\ cm = 1.8\,cm \Longrightarrow \boxed{t_{min} = 1\,cm}$

$* \ R_b = 4.6\ t_{min} = 4.6*1 = \boxed{4.60\ ton}$

$* \ R_{Least} \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.60 \end{cases} \qquad R_{Least} = \boxed{4.02\ ton}$

$* \ n_1 = \dfrac{Force}{R_{Least}} = \dfrac{35}{4.02} = 10.1 > 6\ Bolts$

$\Longrightarrow$ Long joint $\Longrightarrow$ Upper chord

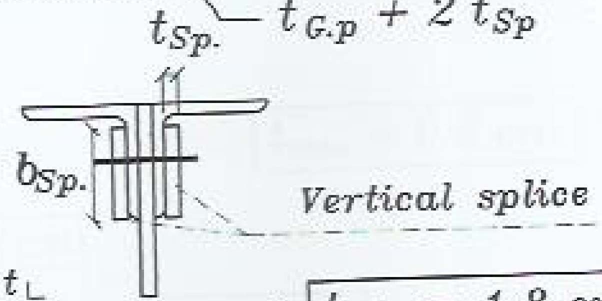
$\Longrightarrow$ we <u>have to</u> use Vl. splice for members 1&2

For n_{sp}

لان الشكل متماثل و بالتالى القوة فى ال 2 upper chords متساوية

$\Longrightarrow$ assume $F_{Sp} = F_{min} = 35\ ton$

$* \ n_{Sp} = \dfrac{Force}{R_{Least}} \begin{cases} R_{4\ S} = 4 * R_{S.S} \\ R_b = \alpha * F_u * d * t_{min} \begin{cases} 2\ t_{\llcorner} \\ t_{G.p} + 2\ t_{Sp} \end{cases} \end{cases}$



$* \ R_{4\ S} = 4 * R_{S.S} = \boxed{8.04\ ton}$

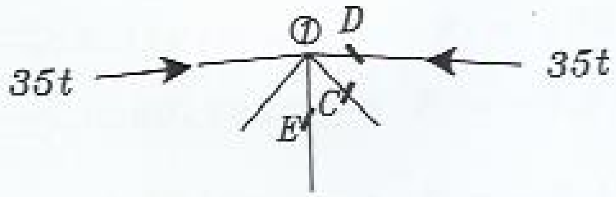
$* \ t_{min} = \overset{t_{G.p}}{1\,cm} + \overset{2*t_{Sp}}{2*1\,cm} \quad or \quad \overset{2*t_{\llcorner}}{2*0.9}\ cm \Longrightarrow \boxed{t_{min} = 1.8\ cm}$

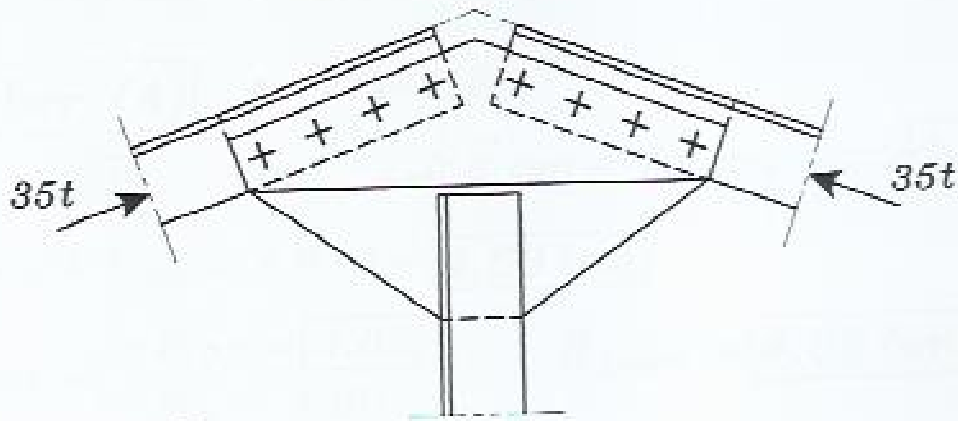
$* \ R_b = 4.6\ t_{min} = 4.60 * 1.8 = \boxed{8.28\ ton}$

$* \ n_{Sp} = \dfrac{Force}{R_{Least}} = \dfrac{35}{8.04} = 4.35 = \boxed{5\ Bolts}$

لان الشكل متماثل و بالتالى القوة فى ال 2 upper chords متساوية

For $n_{diff.}$





$$* \; n_{diff} = \frac{F - F_{Sp}}{R_l} = \frac{35 - 35}{R_l} = \textbf{Zero}$$

لان ال n_{diff} تنتج فى حالة وجود فرق فى القوى بين ال two members المربوطين بال Splice .

No need to check block shear rupture in case of splices

Member (E) $\lrcorner\llcorner 55*55*5$

$* \; t_{min} = \overset{t_{G.p}}{1\,cm}$ or $\overset{t_{\llcorner}}{0.5\ cm}$ $\Longrightarrow$ $\boxed{t_{min} = 0.5\ cm}$

$* \; R_b = 4.6\ t_{min} = 4.6 * 0.5 = \boxed{2.30\ ton}$

$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{2.01} \\ R_b = 2.30 \end{cases}$ $\quad R_{Least} = \boxed{2.01\ ton}$

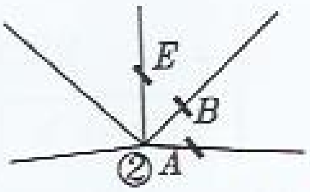
$* \; n_E = \frac{Force}{R_{Least}} = \frac{2.0}{2.01} = 1.0 \Longrightarrow \overset{Minimum}{\boxed{4\ Bolts}}$ 2 bolts for each angle of the star shaped

Desin of connection (1)

Member (A) $\Longrightarrow \rfloor\!\lfloor 120*80*8 \Longrightarrow F = +32\ t$

Member (B) $\Longrightarrow \rfloor\!\lfloor 60*60*6 \Longrightarrow F = -1.5\ t$

Member (E) $\Longrightarrow \urcorner\!\llcorner 55*55*5 \Longrightarrow F = -2\ t$



Member (A) $\rfloor\!\lfloor 120*80*8$

$* \; t_{min} = \overset{t_{G.p}}{1\,cm} \quad or \quad \overset{t_{\llcorner}}{2*0.8}\ cm = 1.6\,cm \Longrightarrow \boxed{t_{min} = 1\,cm}$

$* \; R_b = 4.6\ t_{min} = 4.6*1 = \boxed{4.60\ ton}$

$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.60 \end{cases} \qquad R_{Least} = \boxed{4.02\ ton}$

$$* \; n_A = \frac{\textbf{Force}}{R_{Least}} = \frac{32}{4.02} = 7.96 = 8\ Bolts \; > 6\ Bolts$$

$\Longrightarrow$ Long joint $\Longrightarrow$ Lower chord $\Longrightarrow$ Hz. splice

For n_{sp}

$$\Longrightarrow assume\ F_{Sp} = \frac{F_{min}}{2} = \frac{32}{2} = 16\ ton$$

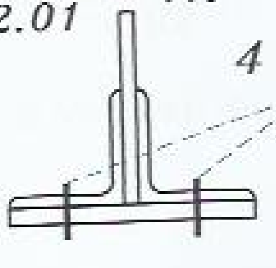
$* \; R_{S.S} = 2.01\ ton$

$* \; t_{min} = \overset{t_{G.p}}{1\,cm} \quad or \quad \overset{t_{\llcorner}}{0.8}\ cm \Longrightarrow \boxed{t_{min} = 0.8\ cm}$

$* \; R_b = 4.6\ t_{min} = 4.60*0.8 = \boxed{3.68\ ton}$

$$* \; n_{Sp} = \frac{\textbf{Force}}{R_{Least}} = \frac{16}{2.01} = 7.96 = \boxed{8\ Bolts}$$

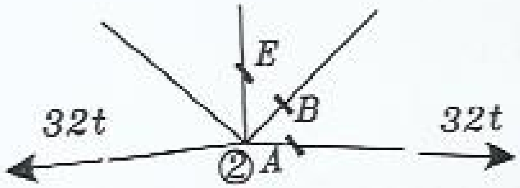
4 Bolts at each side

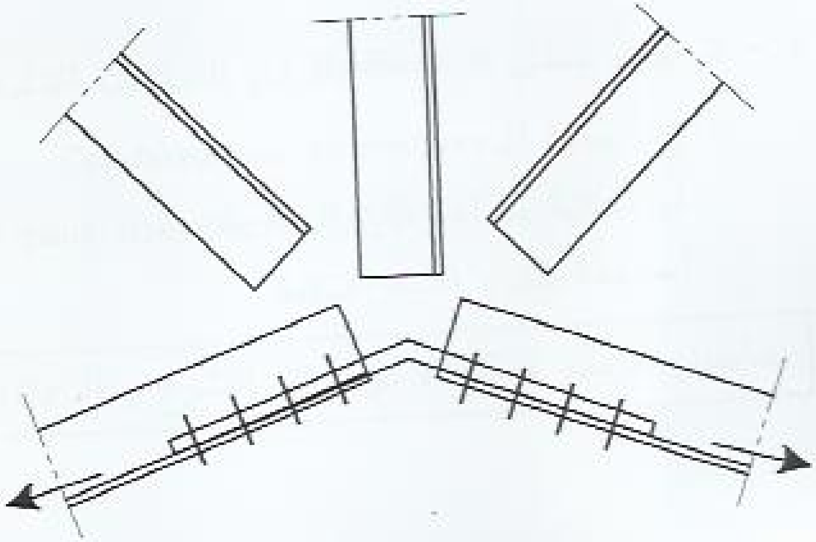


For n_A

$$* \; n_A = \frac{F_1 - F_{Sp}}{R_l} = \frac{F_1 - F_{Sp}}{R_{D.S}} = \frac{32 - 16}{4.02} = 3.90 = \boxed{4 \text{ Bolts}}$$

لان الشكل متماثل و بالتالى القوة فى الـ 2 lower chords متساوية لذلك لا نحتاج الى حساب المسامير فى الـ lower chord فى الناحية الاخرى .





Member (B) ⅃L 60*60*6

$$* \; t_{min} = \overset{t_{G.p}}{1 \, cm} \quad \text{or} \quad 2 * \overset{t_L}{0.6} \, cm = 1.2 \, cm \Longrightarrow \boxed{t_{min} = 1 \, cm}$$

$$* \; R_b = 4.6 \, t_{min} = 4.6 * 1 = \boxed{4.60 \text{ ton}}$$

$$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.60 \end{cases} \qquad R_{Least} = \boxed{4.02 \text{ ton}}$$

$$* \; n_2 = \frac{Force}{R_{Least}} = \frac{1.5}{4.02} = 0.38 \Longrightarrow \overset{Minimum}{\boxed{2 \text{ Bolts}}}$$

Compression force ⟹ No need to check block shear rupture

Member (E) $\lnot^{L}$ 55*55*5

تم تصميمه فى Connection (1) و بالتالى لا نحتاج الى تصميمه مرة أخرى .

Notes

١ - يجب عند الرسم اظهار كل تفاصيل التصميم بمعنى رسم الـ Hz. splice و الـ Vl. splice .

٢ - لا نقوم بعمل الـ Splices فى الحالات الاتية :

- فى حالة الـ Continuous members
- فى حالة ما اذا كان الـ two members لهما اشارات مختلفة أى أن أحدهما شد و الاخر ضغط.

تفاصيل الرسم موجودة فى ورق الشرح و الـ Details

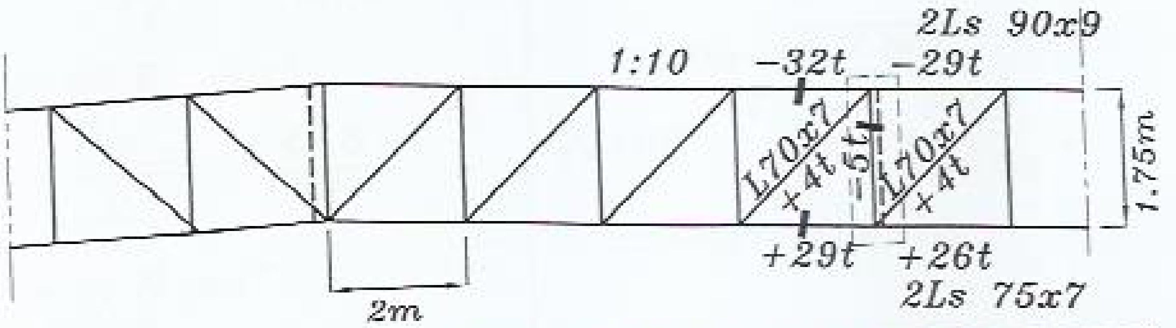
Example January 2002

For the shown truss :

a) Design the marked members.

b) Design the connection enclosed by dotted rectangle, using 16mm diameter non-pretensioned (ordinary) bolts grade (4.6) Consider the end distance equals 3 times the bolts diameter It is not allowed to use number of bolts in any member more 6 bolts.

C) Draw the part enclosed by dotted to scale 1:10.



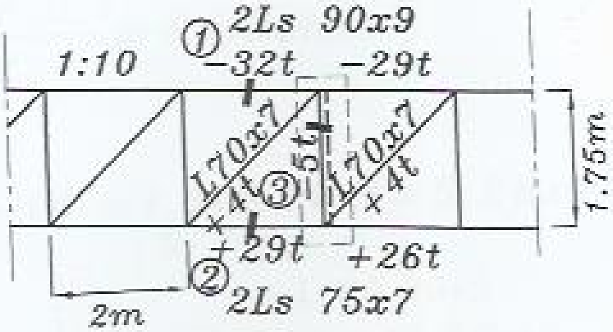
Solution

a) Design the marked members.

Member (1) ⇨ F = −32 t

Member (2) ⇨ F = +29 t

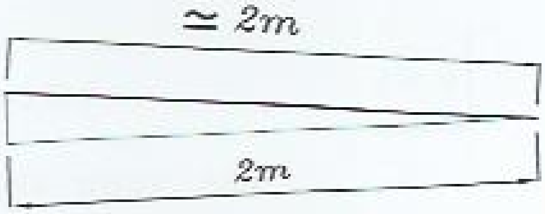
Member (3) ⇨ F = −5 t



Member (1) ⇨ **Upper chord** ⇨ 

1) Data

* Length = **200** cm



يمكن اخذ الطول المائل مساوى للطول

للطول الافقى فى حالة الميول الصغيرة

ولكن لا يمكن فى حالة الميول الكبيرة

مثل ال Fink Truss .

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

2) Choice of section

From stresses

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

$$\therefore A_{g\,\lrcorner\llcorner} = \frac{force}{F_C} = \frac{32}{0.75} = 42.6\ cm^2$$

$$\therefore A_{g\,\llcorner} = \frac{A_{g\,\lrcorner\llcorner}}{2} = \frac{42.6}{2} = 21.3\ cm^2$$

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

From buckling

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

$$\therefore 100 = \frac{l_{b\,in}}{r_x} = \frac{200}{0.30 a_2}$$

$$\Rightarrow a_2 = \boxed{6.7\ cm}$$

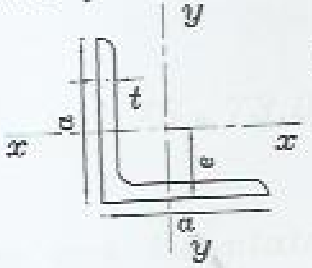
$$\therefore 100 = \frac{l_{b\,out}}{r_y} = \frac{200}{0.45 a_3}$$

$$\Rightarrow a_3 = 4.4\ cm$$

$$a_{av} = \frac{a_1 + (\overset{\text{الاكبر}}{a_2\ Or\ a_3})}{2} = \frac{11 + 6.7}{2} = 8.85\ cm$$

$\boxed{\text{Choose } \lrcorner\llcorner\ 90*90*9}$

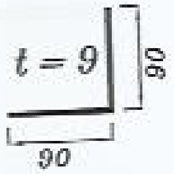
> minimum angle $a_{min} = 1.1*3\,\phi = 1.1*3*1.6 = 5.28\ cm$



∟ 90*90*9
$A = 15.5\ Cm^2$
$e = 2.54\ Cm$
$r_x = r_y = 2.74\ Cm$
$r_v = 1.76\ Cm$

3) Checks

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

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

assume $t_{Gp} = 1\ cm$

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

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x_{\rfloor\lfloor}}} = \frac{200}{2.74} = 73.0 < 180 \Longrightarrow (Safe)$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{y_{\rfloor\lfloor}}} = \frac{200}{4.09} = 49.0 < 180 \Longrightarrow (Safe)$$

c) Stress

$$\lambda_{max.} = 73.0 \leqslant 100$$

$$* F_C = 1.4 - 6.5 * 10^{-5} \lambda^2_{max.} = 1.4 - 6.5 * 10^{-5} (73.0)^2$$

$$= 1.05\ t \backslash cm^2$$

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

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

Design of tie plate

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

$$\frac{l'}{r_{v\llcorner}} = \frac{l'}{1.76} \leqslant 92.2 \Longrightarrow l' \leqslant 1.76 * 73.0 = 129\,cm$$

$$l' \leqslant 129\ cm > \frac{l}{2}$$ $\Longrightarrow$ Use one tie plate at the middle of member

Member (2) ⇨ Lower chord ⇨ $\top\!\top$

1) Data

* Length = 200 cm
* Force = +29 ton (Case A)
* $l_{b\,in}$ = Distance between joints = 200 cm
* $l_{b\,out}$ = Distance between long. bracing = 800 cm

2) Choice of section

a – From Stress Condition

$$A_{g_{\text{JL Bolted}}} = \frac{\text{Force ton}}{0.85 * F_t\ (t \backslash cm^2)} = \frac{29}{0.85*1.4} = 24.36\ cm^2$$

$$A_{g_{\text{L}}} = \frac{12.18}{2} = 12.18\ cm^2$$

$\xrightarrow{\text{من الجدول}}$ $\boxed{\text{Choose JL } 80x80x8}$ $a_1 = 8.0$ cm

b – From Slenderness Condition

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

$$\therefore 300 = \frac{l_{b\,in}}{r_{x_{JL}}} = \frac{200}{0.30a} \Rightarrow a = 2.22\,cm$$

$$\therefore 300 = \frac{l_{b\,out}}{r_{y_{JL}}} = \frac{800}{0.45a} \Rightarrow a = 5.92\,cm$$

نأخذ الاكبر
$a_2 = 5.92\,cm$

C – From Construction Condition

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

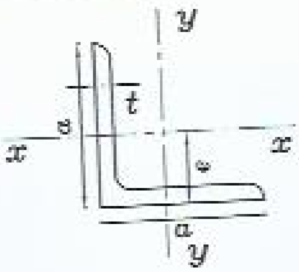
$$a_3 = 5.28\ cm$$

عرفنا انها Bolted Connection لانه مطلوب فى المسألة تصميم Bolted Conn.

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

$\boxed{Choose \; \lrcorner\llcorner \, 80x80x8}$

3) Checks



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

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

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

a) Stress $_{Tension}$

$$* \; f_t = \frac{Force}{A_{net}} = \frac{29}{21.72} = 1.34 \quad t \backslash cm^2$$

مساحة ال $angles$ التي تم حسابها

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

b) Slenderness

$r_{x_{\lrcorner\llcorner}} = r_{x_{\llcorner}}$ من الجدول $= 2.42 \; cm$

assume $t_{Gp} = 1 \; cm$

$$r_{y_{\lrcorner\llcorner}} = \sqrt{r^2_{y\llcorner} + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.42^2 + (2.26 + \frac{1.0}{2})^2} = 3.67 \; cm$$

$$* \; \lambda_{in} = \frac{l_{b\,in}}{r_{x_{\lrcorner\llcorner}}} = \frac{200}{2.42} = 82.6 \; < 300 \Longrightarrow \textbf{(Safe)}$$

$$* \; \lambda_{out} = \frac{l_{b\,out}}{r_{x_{\lrcorner\llcorner}}} = \frac{800}{3.67} = 218 < 300 \Longrightarrow \textbf{(Safe)}$$

c) Length to depth ratio (Deflection)

$$* \; \frac{L}{d} = \frac{200 \; cm}{a} = \frac{200 \; cm}{8.0} = 25 \leqslant 60 \Longrightarrow \textbf{(Safe)}$$

Member (3) $\Rightarrow$ Vertical member at Long. Bracing $\Rightarrow$ ┐└

1) Data

* Length = 175 cm
* Force = −5 ton (Case A)
* $l_{b\,in}$ = Distance between joints = 175 cm
* $l_{b\,out}$ = 175 cm

2) Choice of section

From stresses

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

$$\therefore A_{g\,┐└} = \frac{force}{F_C} = \frac{5}{0.75} = 6.67\ cm^2$$

$$\therefore A_{g\,└} = \frac{A_{g\,┐└}}{2} = \frac{6.67}{2} = 3.33\ cm^2$$

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

From buckling

* assume $\lambda_{out} = 100$

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

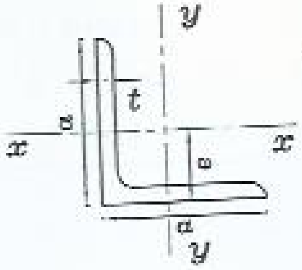
$$\Rightarrow a_2 = \boxed{4.6\ cm}$$

$$a_{av} = \frac{a_1 + a_2}{2} = \frac{4.5 + 4.6}{2} = 4.55\ cm$$

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

$\boxed{\text{Choose } \rfloor\!\llcorner\ 90*90*9}$

3) Checks



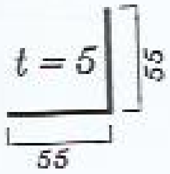
$\llcorner\ 55*55*5$

$A = 5.32\ Cm^2$

$r_u = 2.09\ Cm$

$r_v = 1.07\ Cm$

a) Class. of section



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

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

b) Buckling

$r_{u_{\rceil\llcorner}} = r_{u_{\llcorner}}$ من الجدول $= 2.09$ cm

$$* \lambda_{in} = \frac{l_{b\,out}}{r_{u_{\rceil\llcorner}}} = \frac{175}{2.09} = 84 < 180 \Longrightarrow \text{(Safe)}$$

C) Stress

$$\lambda_{max.} = 84.0 \leqslant 100$$

$$* F_C = 1.4 - 6.5 * 10^{-5} \lambda^2_{max.} = 1.4 - 6.5 * 10^{-5} (84.0)^2$$

$$= 0.94\ t \backslash cm^2$$

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

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

Design of tie plate

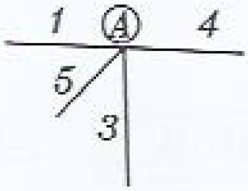
$$\lambda_v \leqslant \lambda_{max.}$$

$$\frac{l'}{r_{v\llcorner}} = \frac{l'}{1.76} \leqslant 84.0 \Longrightarrow l' \leqslant 1.07 * 84.0 = 90\ cm$$

$l' \leqslant 90$ cm $> \frac{l}{2}$ $\Longrightarrow$ Use one tie plate at the middle of member

Design the connection enclosed by dotted rectangle, using 16mm diameter non-pretensioned (ordinary) bolts grade (4.6) Consider the end distance equals 3 times the bolts diameter It is not allowed to use number of bolts in any member more 6 bolts.

Desin of connection (A)



Member (1) $\Longrightarrow$ ⊤⊤ $90*90*9$ $\Longrightarrow$ $F = -32\ t$

Member (4) $\Longrightarrow$ ⊤⊤ $90*90*9$ $\Longrightarrow$ $F = -29\ t$

Member (3) $\Longrightarrow$ ⌐∟ $55*55*5$ $\Longrightarrow$ $F = -5\ t$

Member (5) $\Longrightarrow$ ∟ $70*70*7$ $\Longrightarrow$ $F = +4\ t$

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

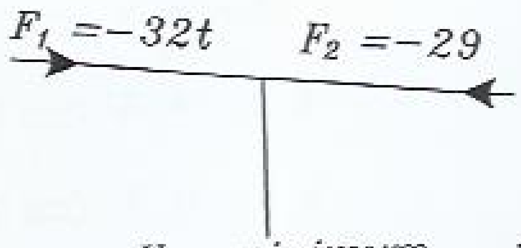
* $F_{ub} = 4\ t \backslash cm^2$ * $\phi = 1.6\ cm$
* $F_u \xrightarrow{for\ st.37} = 3.6\ t \backslash cm^2$
* $q_b = 0.25\, F_{ub}$ * $A_S = \dfrac{\pi\, d^2}{4}$
* Take $e \geqslant 3\phi = 4.8\ cm \Longrightarrow \alpha = 1.2$
* $R_{S.S} = (0.25\, F_{ub}) * \dfrac{\pi\, d^2}{4} * 1 = (0.25 * 4) * \dfrac{\pi (1.6)^2}{4}$
  $= 2.01\ ton$
* $R_{D.S} = (0.25\, F_{ub}) * \dfrac{\pi\, d^2}{4} * 2 = 2 * R_{S.S} = 4.02\ ton$
* $R_b = (\alpha * F_u) * d * t_{min} = 1.2 * 3.6 * 1.6 * t_{min} = 6.9\ t_{min}$

Member (1,4) ⊤⊤ $90*90*9$ $\Longrightarrow$ Continuous member

* $t_{min} = 1 \overset{tG.p}{cm}$ or $2 * \overset{t_{\llcorner}}{0.9}\ cm = 1.8\ cm \Longrightarrow \boxed{t_{min} = 1\ cm}$
* $R_b = 6.9\ t_{min} = 6.9 * 1 = \boxed{6.9\ ton}$
* $R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 6.90 \end{cases}$ $R_{Least} = \boxed{4.02\ ton}$

حيث أن الـ 2 members الموجودين بالـ Upper chord لهما نفس القطاع و على نفس الاستقامة لذلك يتم تصميمهما على أنهما Continuous member .

$$* \; n_{1-4} = \frac{F_1 - F_4}{R_{Least}}$$



$$= \frac{32-29}{4.02} = \frac{3}{4.02} = 0.75 \xrightarrow{\text{Use minimum}} \boxed{2 \text{ Bolts}}$$

$\Longrightarrow$ For continuous member

$\Longrightarrow$ No B.S.R Check

Member (3) $\ \rceil\!\llcorner 55*55*5$

$* \; t_{min} = \overset{t_{G.p}}{1 \text{ cm}}$ or $\overset{t_{\llcorner}}{0.5 \text{ cm}}$ $\Longrightarrow \boxed{t_{min} = 0.5 \text{ cm}}$

$* \; R_b = 4.6 \; t_{min} = 6.9 * 0.5 = \boxed{3.45 \text{ ton}}$

$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{2.01} \\ R_b = 3.45 \end{cases}$ $\quad R_{Least} = \boxed{2.01 \text{ ton}}$

$$* \; n_3 = \frac{Force}{R_{Least}} = \frac{5.0}{2.01} = 2.48 \Longrightarrow \overset{\text{Minimum}}{\boxed{4 \text{ Bolts}}}$$

2 bolts for each angle of the star shaped

Member (5) $\ \llcorner 70*70*7$

$* \; t_{min} = \overset{t_{G.p}}{1 \text{ cm}}$ or $\overset{t_{\llcorner}}{0.7 \text{ cm}}$ $\Longrightarrow \boxed{t_{min} = 0.7 \text{ cm}}$

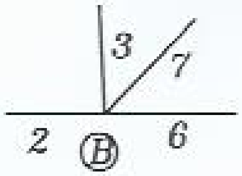
$* \; R_b = 4.6 \; t_{min} = 6.9 * 0.7 = \boxed{4.83 \text{ ton}}$

$* \; R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{2.01} \\ R_b = 4.83 \end{cases}$ $\quad R_{Least} = \boxed{2.01 \text{ ton}}$

$$* \; n_C = \frac{Force}{R_{Least}} = \frac{4.0}{2.01} = 2.0 \Longrightarrow \boxed{2 \text{ Bolts}} \text{ Minimum}$$

$\Longrightarrow$ Check B.S.R

Desin of connection (B)



Member (2) $\Rightarrow$ ⅃L $80*80*8$ $\Rightarrow$ $F = +29\ t$

Member (6) $\Rightarrow$ ⅃L $75*75*7$ $\Rightarrow$ $F = +26\ t$

Member (3) $\Rightarrow$ ˥L $55*55*5$ $\Rightarrow$ $F = -5\ t$

Member (7) $\Rightarrow$ L $70*70*7$ $\Rightarrow$ $F = +4\ t$

$\boxed{\textbf{Member (2)}}$ ⅃L $80*80*8$

$* \ t_{min} = \overset{t_{G.P}}{1\,cm}$ or $2* \overset{t_L}{0.8}\,cm = 1.6\,cm \Longrightarrow \boxed{t_{min} = 1\,cm}$

$* \ R_b = 6.9\ t_{min} = 6.9 * 1 = \boxed{6.9\ ton}$

$* \ R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 6.90 \end{cases} \qquad R_{Least} = \boxed{4.02\ ton}$

$$* \ n_3 = \frac{\text{Force}}{R_{Least}} = \frac{29.0}{4.02} = 7.20 \Longrightarrow 8 \text{ Bolts} > 6 \text{ Bolts}$$

$\Longrightarrow$ Long joint $\Longrightarrow$ Lower chord $\Longrightarrow$ Hz. splice

$\boxed{\textbf{For } n_{sp}}$

$$\Longrightarrow \text{assume } F_{Sp} = \frac{F_{min}}{2} = \frac{26}{2} = 13\ ton$$

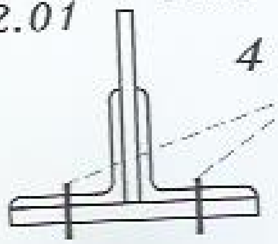
$* \ R_{S.S} = 2.01\ ton$

$* \ t_{min} = \overset{t_{G.P}}{1\,cm}$ or $\overset{t_L}{0.8}\ cm \Longrightarrow \boxed{t_{min} = 0.8\ cm}$

$* \ R_b = 4.6\ t_{min} = 6.9 \ * 0.8 = \boxed{5.52\ ton}$

$$* \ n_{Sp} = \frac{\text{Force}}{R_{Least}} = \frac{13.0}{2.01} = 6.50 = \boxed{8 \text{ Bolts}}$$

4 Bolts at each side



For n_A

$$* \; n_A = \frac{F_2 - F_{Sp}}{R_l} = \frac{F_1 - F_{Sp}}{R_{D.S}} = \frac{29 - 13}{4.02} = 3.0$$

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

Members (3)

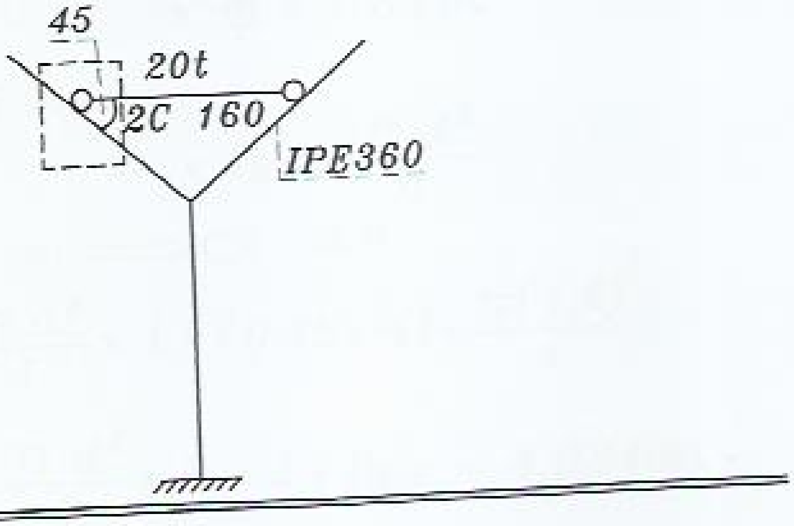
تم تصميمه فى Connection (A) و بالتالى لا نحتاج الى تصميمه مرة أخرى .

Members (7)

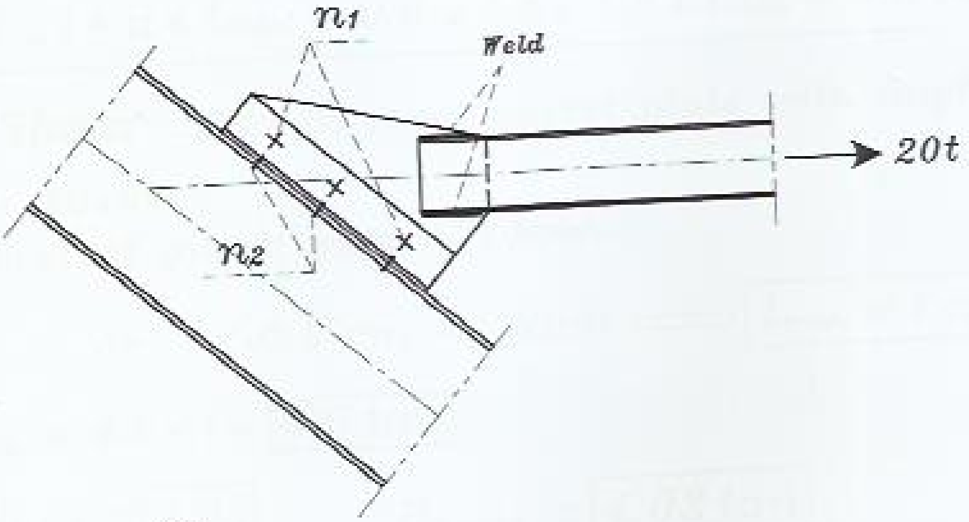
تم تصميمه فى Connection (A) و بالتالى لا نحتاج الى تصميمه مرة أخرى .

Example

Design the shown connection knowing that the member is welded to the plate while the plate is bolted to the column using M16 grade (4.6) bearing type. (excluded threads)



Solution



For Weld

* Force on weld $= \dfrac{20}{2} = 10$ ton ------ TWO CHANNELS

* Assume size of weld = 5 mm

* Stress on weld $= \dfrac{Force}{n * s * l} = \dfrac{10}{2 * 0.5 * l} = 0.72$ t\cm²

$l = 13.88$ mm

* Actual Length $= l + 2s = 13.88 + 2 * 0.5 = 14.88$ mm

* Take the actual length = 15 mm

Desin of connection

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

* $F_{ub} = 4\ t \backslash cm^2$
* $\phi = 1.6\ cm$
* $F_u \xrightarrow{\text{for st.37}} = 3.6\ t \backslash cm^2$
* $q_b = 0.25 F_{ub}$
* $A_S = \dfrac{\pi d^2}{4}$
* Take $e \geqslant 2\phi = 3.2\ cm \Longrightarrow \alpha = 0.8$
* $R_{S.S} = (0.25 F_{ub}) * \dfrac{\pi d^2}{4} * 1 = (0.25 * 4) * \dfrac{\pi (1.6)^2}{4}$

  $= 2.01\ ton$
* $R_{D.S} = (0.25 F_{ub}) * \dfrac{\pi d^2}{4} * 2 = 2 * R_{S.S} = 4.02\ ton$
* $R_b = (\alpha * F_u) * d * t_{min} = 0.8 * 3.6 * 1.6 * t_{min} = 4.6\ t_{min}$

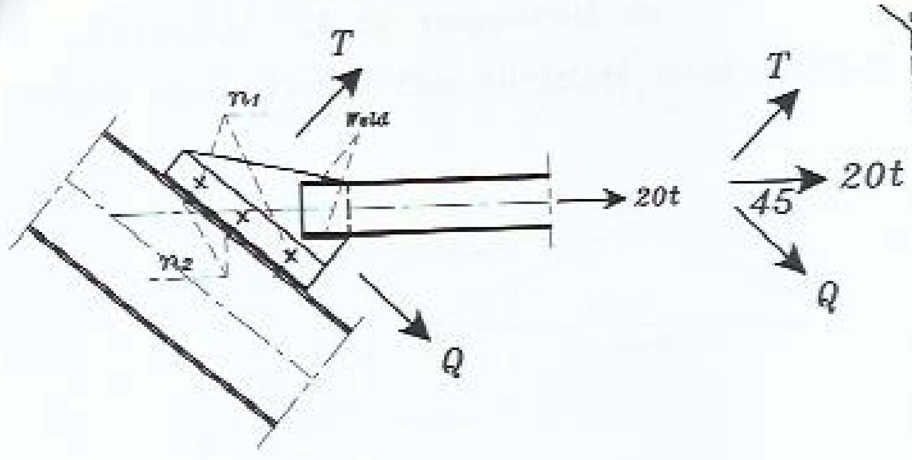
$\boxed{\text{For } n_1} \Longrightarrow$ **Shear** $\Longrightarrow$ Connecting gusset plate with angles

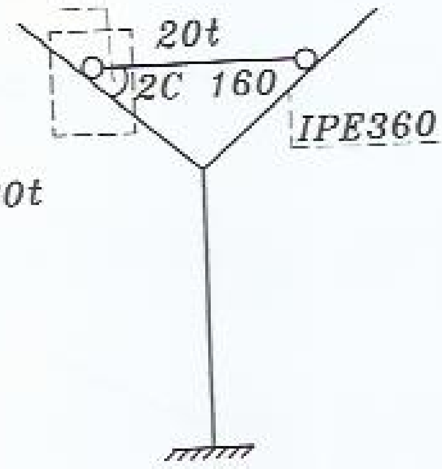
assume angles 80*80*8
assume thickness of gusset pate = 10mm

* $t_{min} = \overset{t_{G.p}}{1\ cm}$ or $\overset{t_L}{2 * 0.8}\ cm = 1.6\ cm \Longrightarrow \boxed{t_{min} = 1\ cm}$
* $R_b = 4.6\ t_{min} = 4.6 * 1 = \boxed{4.6\ ton}$
* $R_{Least} \rightarrow \begin{cases} R_{D.S} = \boxed{4.02} \\ R_b = 4.6 \end{cases}$ $R_{Least} = \boxed{4.02\ ton}$
* $n_1 = \dfrac{\textbf{Force}}{\textbf{R}_{Least}} = \dfrac{20}{4.02} = 4.99 \Longrightarrow \boxed{5\ \text{Bolts}}$

For n_2 $\Longrightarrow$ ***Shear+ Tension***

$\Longrightarrow$ *Connecting angles with column*





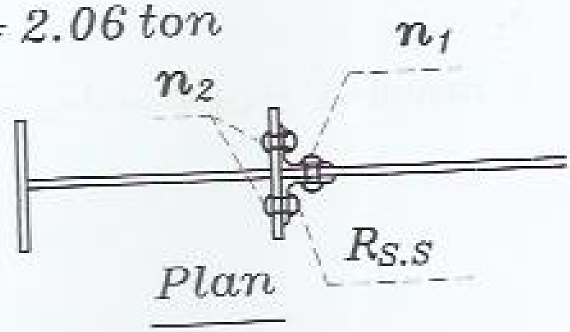
$* \; Q = 20 \cos 45 = 14.14 \text{ ton}$

$* \; T = 20 \sin 45 = 14.14 \text{ ton}$

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

$$R_t = 0.78 * \frac{\pi (1.6)^2}{4} * 0.33 * 4 = 2.06 \text{ ton}$$

$\Longrightarrow$ assume $n_4 = 2n_3 = 10$ Bolts
لوجودها فى الجمعتين



Interaction equation

$$\left(\frac{Q \setminus n}{R_{sh_{allowable}}}\right)^2 + \left(\frac{T \setminus n}{R_{t_{allowable}}}\right)^2 \leqslant 1$$

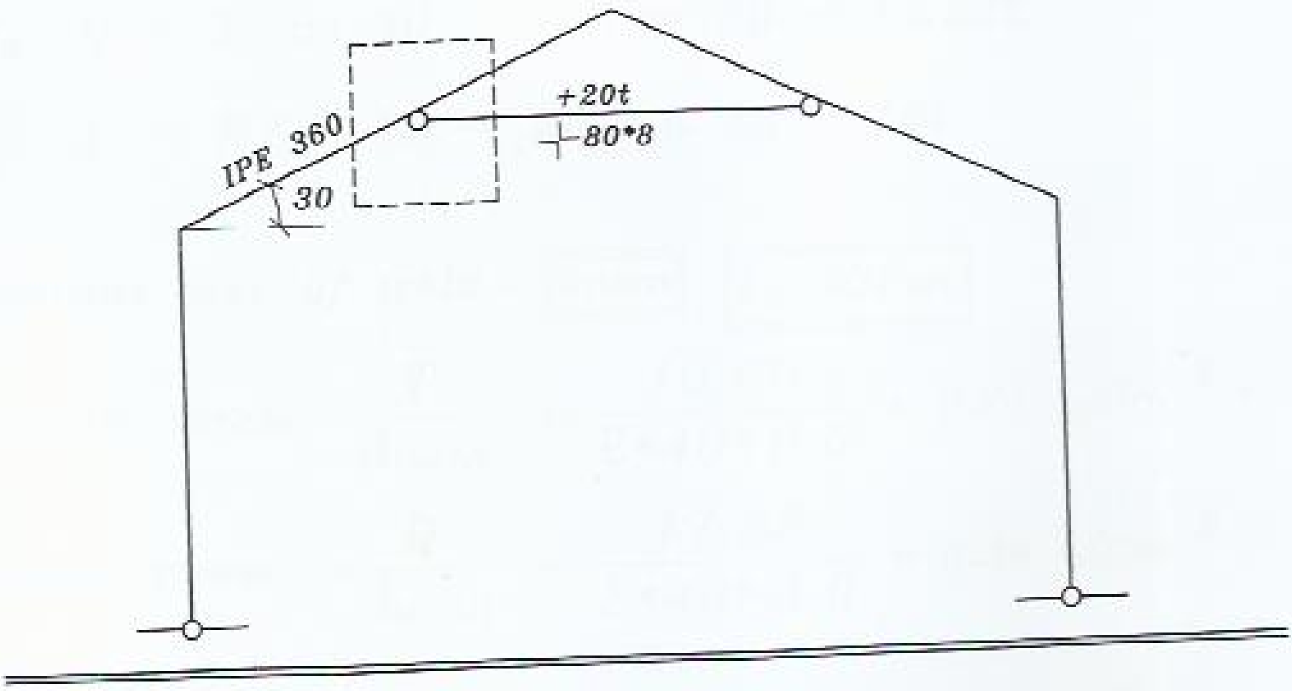
$$\left(\frac{\frac{14.14}{10}}{2.01}\right)^2 + \left(\frac{\frac{14.14}{10}}{2.06}\right)^2 = 0.96 \leqslant 1$$

$R_{s.s}$

$\Longrightarrow$ Safe

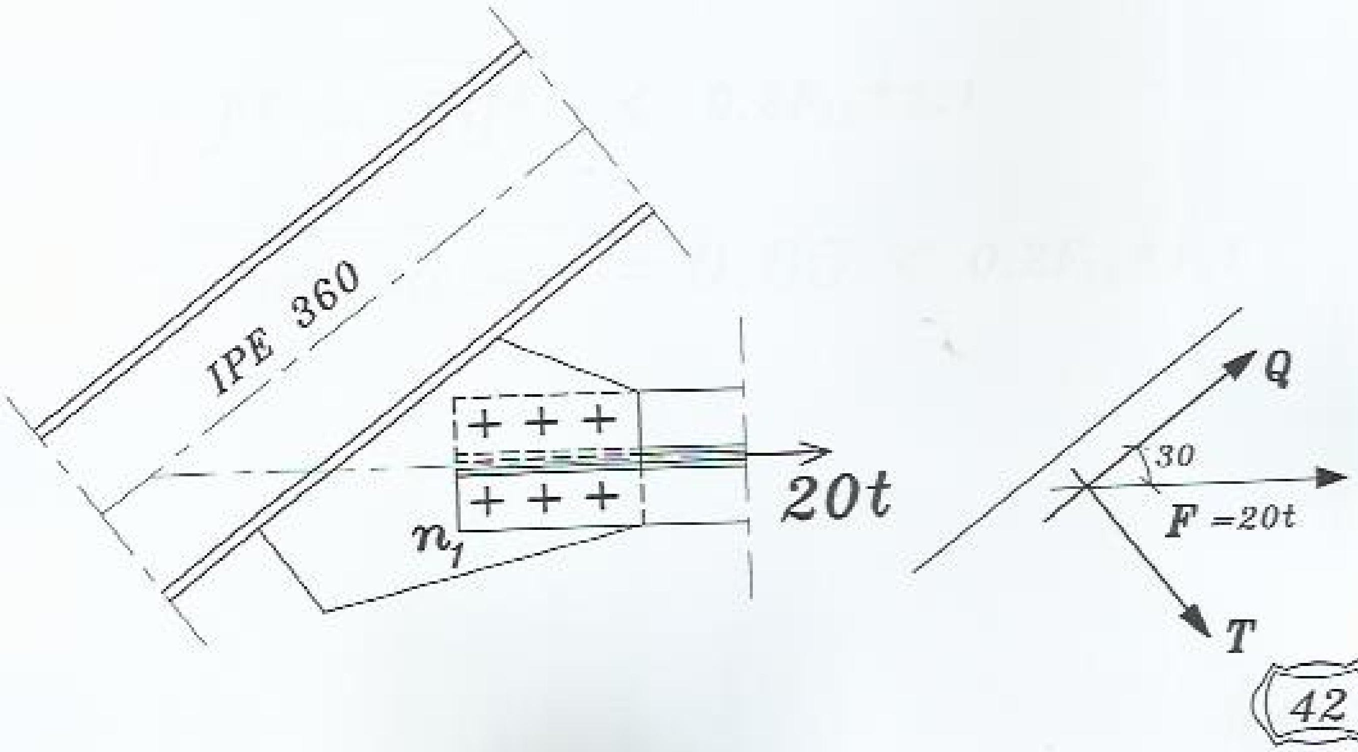
Example

*using steel 37, 10mms thick gusset plate and **M20** bolts (grade10.9), **it is required to** design and draw the welded and bolted shown*



ملحوم فى **ct plate**

ال ***I-beam*** ومربوط بمسامير مع ال ***member***



$P_s = 4.93t$

$$\boxed{n_1 = \frac{F_1 = 20}{2\ P_s\ =\ 4.93} = \underline{6\ \text{bolts}}}$$ 3each side

$\therefore\ Q = F\ Cos\ 30 = 20\ Cos\ 30 = 17.32t$

$T = F\ Sin\ 30 = 20\ Sin\ 30 = 10t$

assume size of weld = $\boxed{6mm}$ $\boxed{l = 40Cm}$

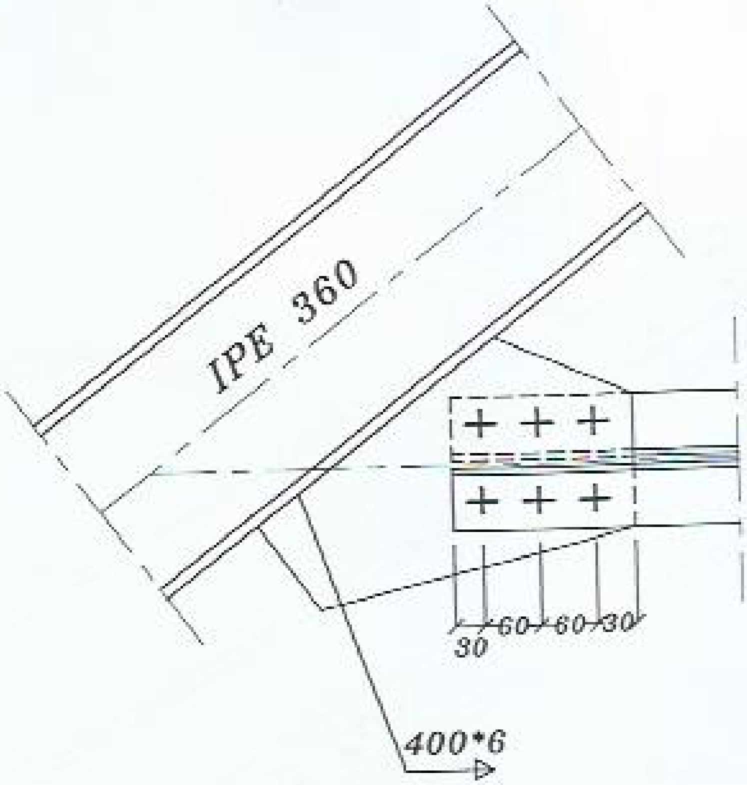
$$\text{tensile Stress} = \frac{T}{A_{weld}} = \frac{10.00}{2*40*0.6} = 0.21\ t/Cm^2$$

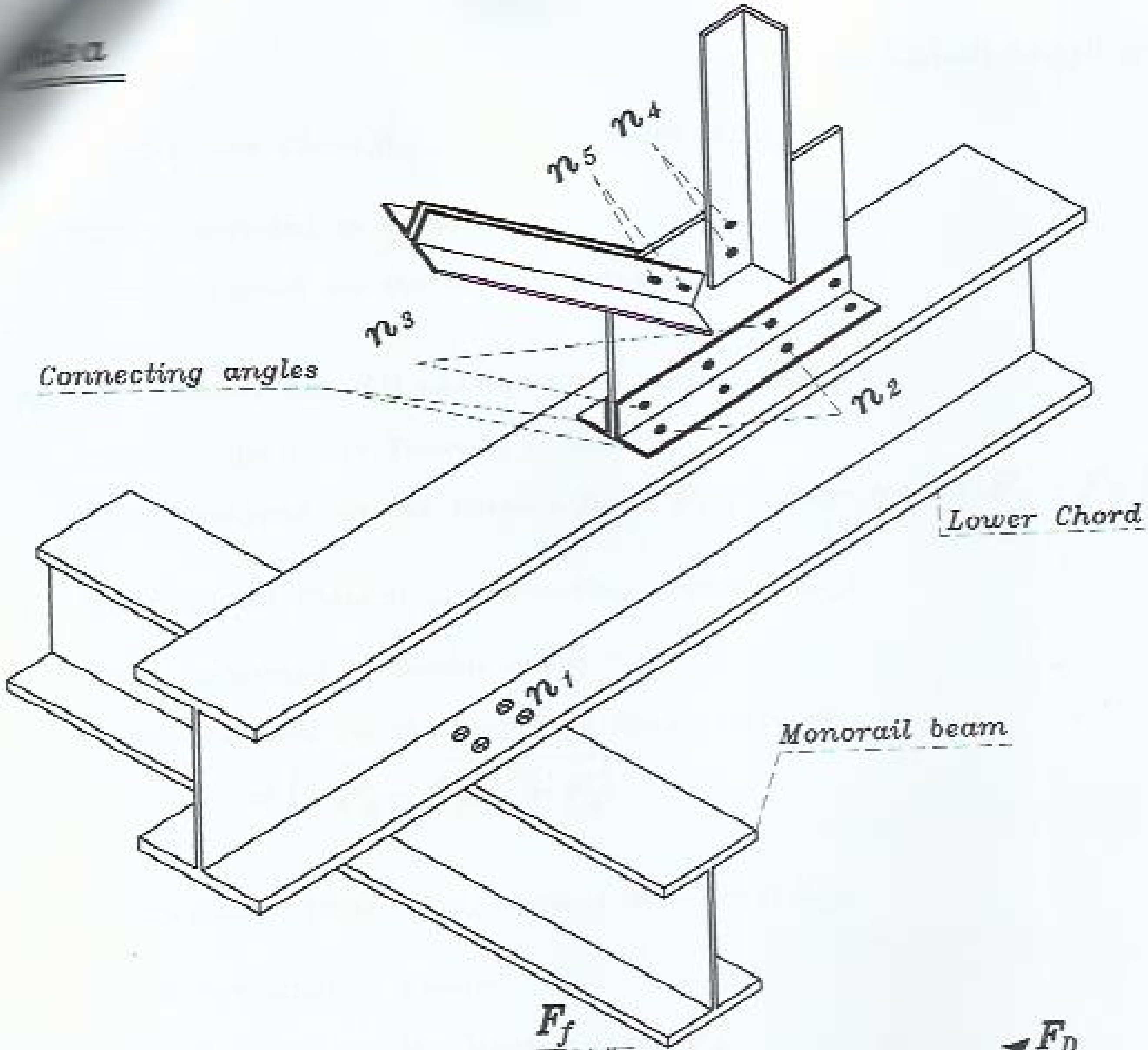
$$\text{shear Stress} = \frac{Q}{A_{weld}} = \frac{17.32}{2*40*0.6} = 0.36\ t/Cm^2$$

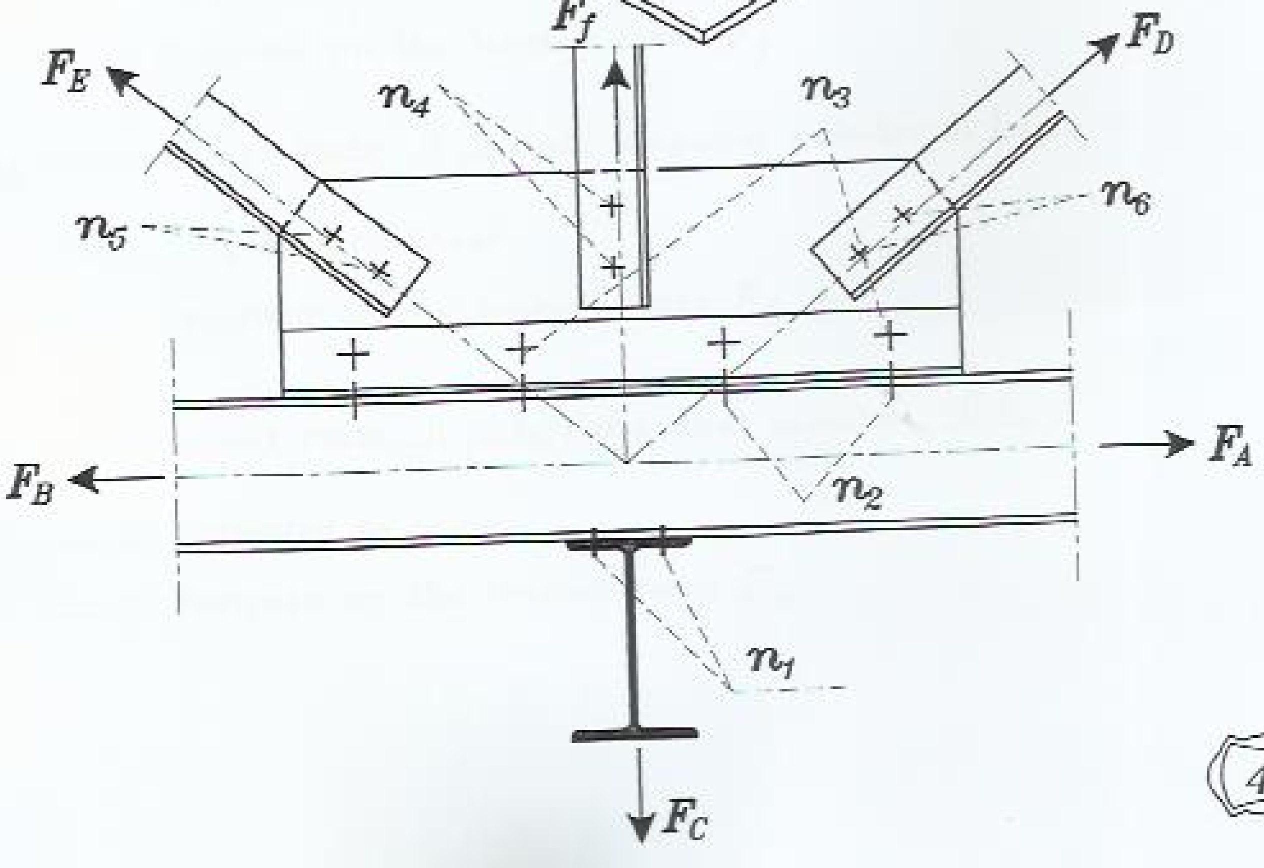
$\boxed{\text{Check using Interaction Equation}}$

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

$$\sqrt{0.210^2 + 3(0.360)^2} = 0.65 < 0.2F_u*1.1$$


IPE 360
30
60
60
30
400*6


n_5
n_4
n_3
Connecting angles
n_2
Lower Chord
n_1
Monorail beam


F_f
F_E
F_D
n_4
n_3
n_5
n_6
F_A
F_B
n_2
n_1
F_C

في الوصلة السابقة نجد أن

$n_1 \Longrightarrow$ *Lower Chord* مع ال *Monorail beam* تربط ال

$\Longrightarrow$ *Subjected to Tension*

$\Longrightarrow$ *Designed on the tension force* F_C

$n_2 \Longrightarrow$ *Lower Chord* مع ال *Connecting angles* تربط ال

$\Longrightarrow$ *Subjected to Tension + Shear*

$\Longrightarrow$ *Designed on the tension force* F_C + *Shear force* $[F_A - F_B]$

$n_3 \Longrightarrow$ *Gusset Plate* مع ال *Connecting angles* تربط ال

$\Longrightarrow$ *Subjected to Double shear*

$\Longrightarrow$ *Designed on the resultant shear force of*

$$R = \sqrt{[F_A - F_B]^2 + F_C^2}$$

$n_4 \Longrightarrow$ *Gusset Plate* مع ال *Vertical member* تربط ال

$\Longrightarrow$ *Subjected to Shear*

$\Longrightarrow$ *Designed on the tension force* F_f

$n_5 \Longrightarrow$ *Gusset Plate* مع ال *right diagonal member* تربط ال

$\Longrightarrow$ *Subjected to Shear*

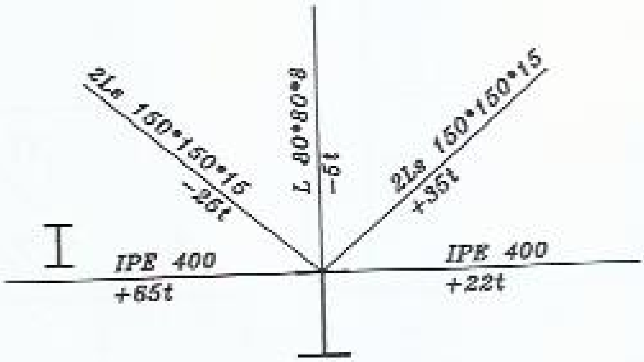
$\Longrightarrow$ *Designed on the tension force* F_E

$n_5 \Longrightarrow$ *Gusset Plate* مع ال *left diagonal member* تربط ال

$\Longrightarrow$ *Subjected to Shear*

$\Longrightarrow$ *Designed on the tension force* F_D

Example



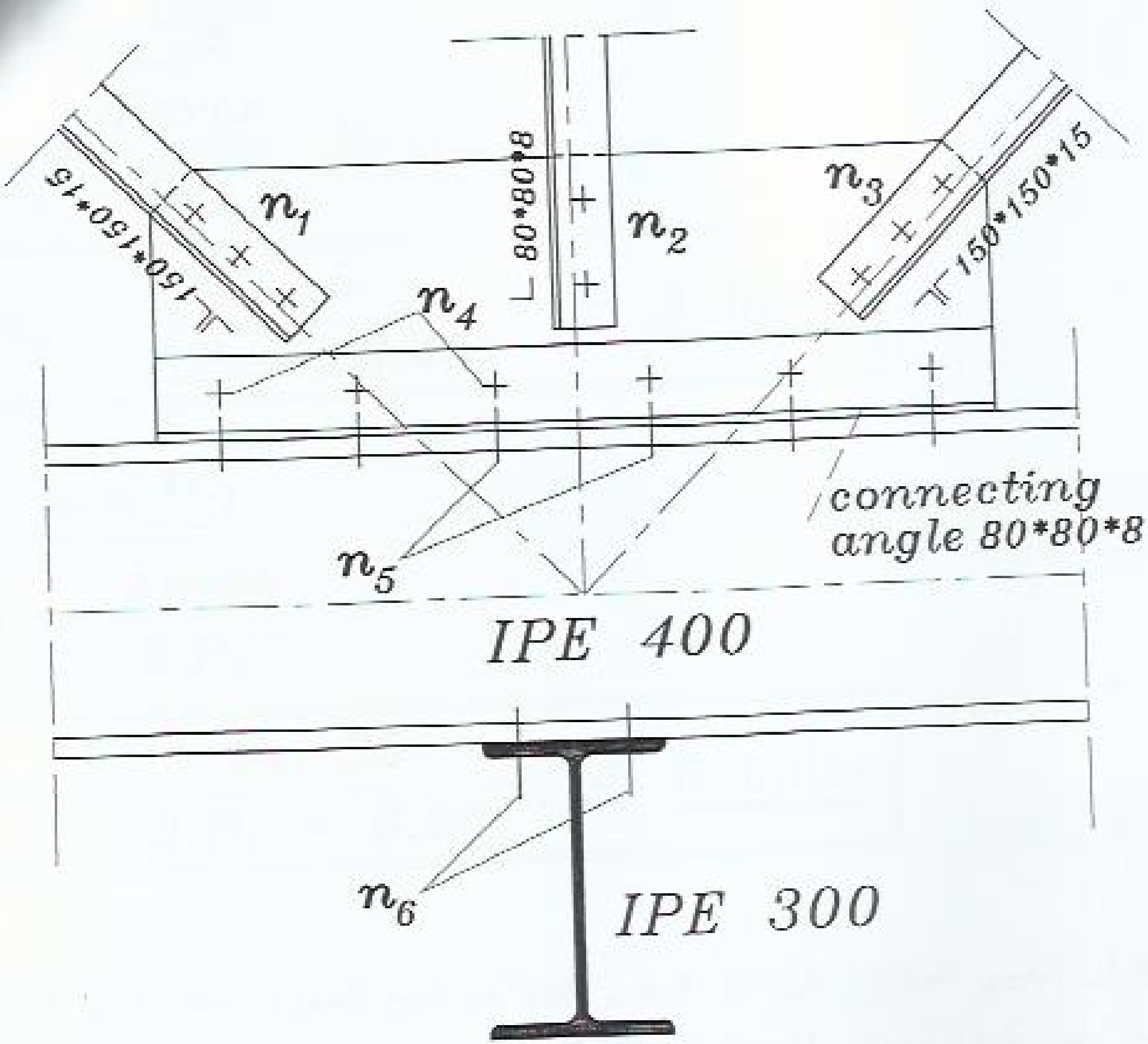
For the shown Connection and using Steel 37, bolt M20 grade8.8 and **15mm** thick gusset plate, **it is required to**

a)Design the slip-critical **bolted Connection**

b)Draw the Connection to scale 1:10

The mono Rail Capacity = 15ton

يجب الملاحظة انه في هذه المسائل قطاع ال **lower chord** هو عبارة عن **IPE 400** ومستمر خلال طول ال **lower chord** كما هو موضح بالرسم



$$P_s = 0.7 X\ 4.93t = \boxed{3.451t}$$

For bolts n_1

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

$$\boxed{n_1 = \frac{F_1 = 25}{2\ P_s\ =\ 6.90} = \underline{4\ \text{bolts}}}$$

For bolts n_2

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

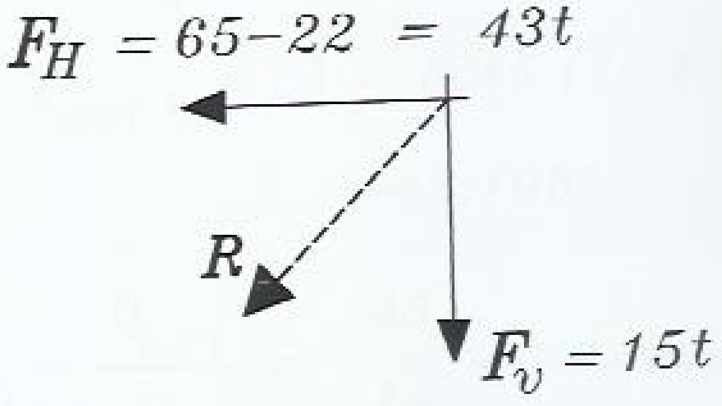
$$\boxed{n_2 = \frac{F_1 = 5}{P_s\ =\ 3.451} = \underline{2\ \ bolts}}$$

For bolts n_3

$$n_3 = \frac{Force}{2\ P_s} = \ .....$$

$$\boxed{n_3 = \frac{F_3 = 35}{2\ P_s\ =\ 6.90} = \underline{6\ \ bolts}}$$

لاحظ ان القوى المؤثرة على ال **n3 , n4** هو فرق القوى داخل ال **IPE 400** وقيمة حمل كمرة الونش **mono rail**



For bolts n_4

$$R = \sqrt{43^2 + (15)^2} = 45.5t$$

For bolts n_4

$$n_4 = \frac{R}{2\ P_s} = \ .....$$

$$\boxed{n_4 = \frac{F_4 = 45.5}{2\ P_s = 6.90} = \underline{7\ \text{bolts}}}$$

For bolts n_5

$Q = 65 - 22 = 43t$

$T = 15t$

$$\boxed{\text{assume } n_5 = 2\,n_4}$$

$$T_{ext,b} = \frac{T_{ext}}{n_5} = \frac{15}{14} = 1.07t < 0.6*0.7*T = 6.48t$$

$$P_s\,(1 - T_{ext,b} / T) = 3.45\ (1 - 1.07/10.8)$$

$$= 3.108t \qquad 0.7*T$$

$$Q_{ext,b} = \frac{Q_{ext}}{n} = \frac{43}{14} = 3.07t < 3.108t$$

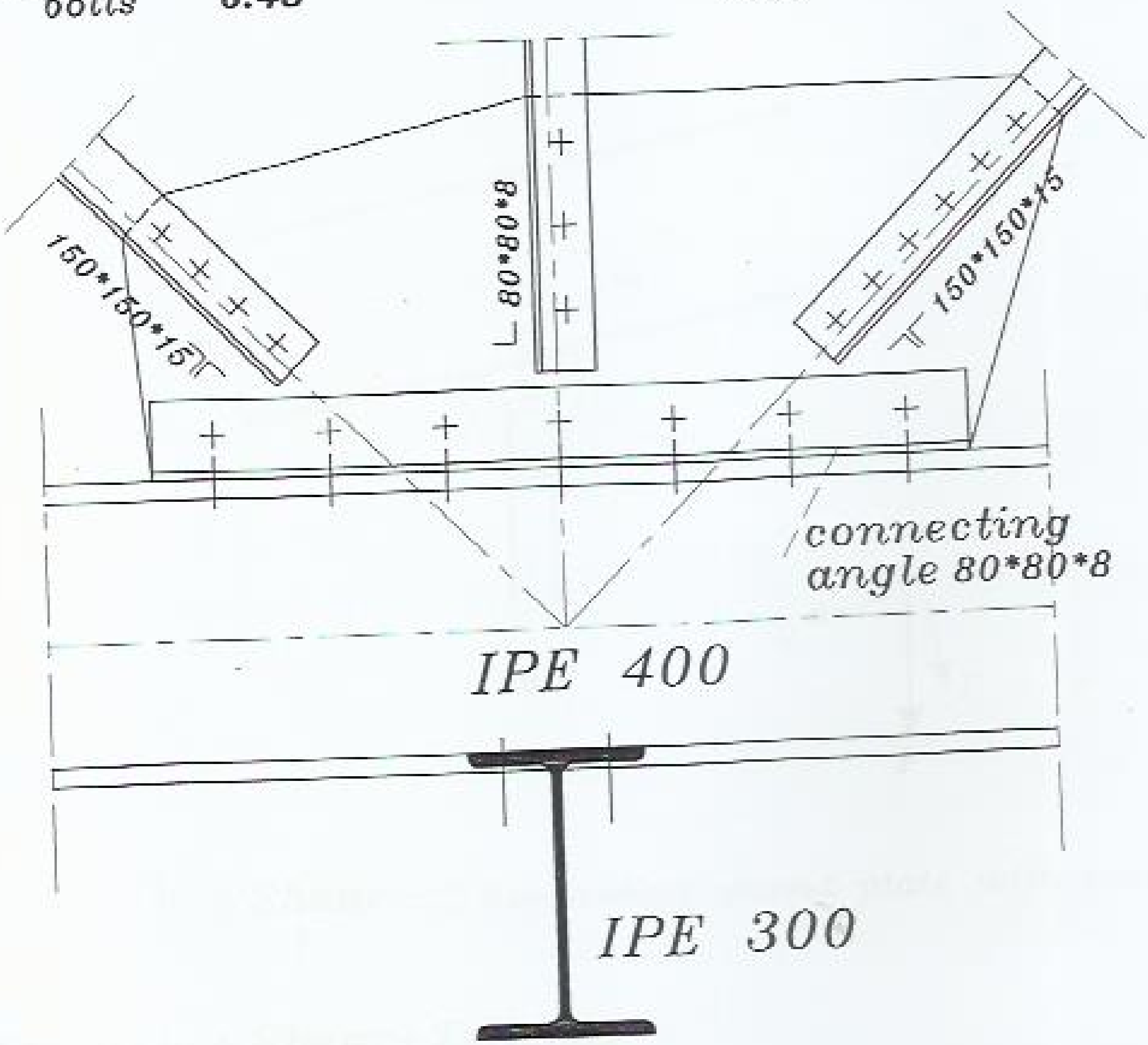
SAFE

For bolts n_6

$$\boxed{n_{bolts} = \frac{Force}{R_t}}$$

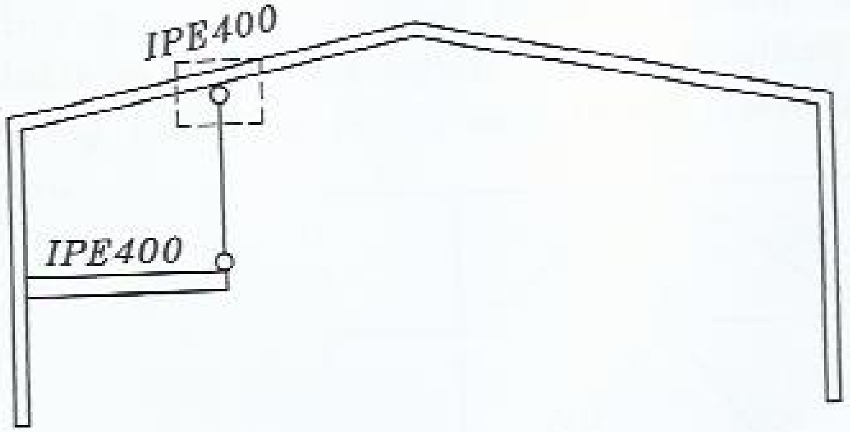
$$\boxed{\bullet\ R_t = 6.48t}$$

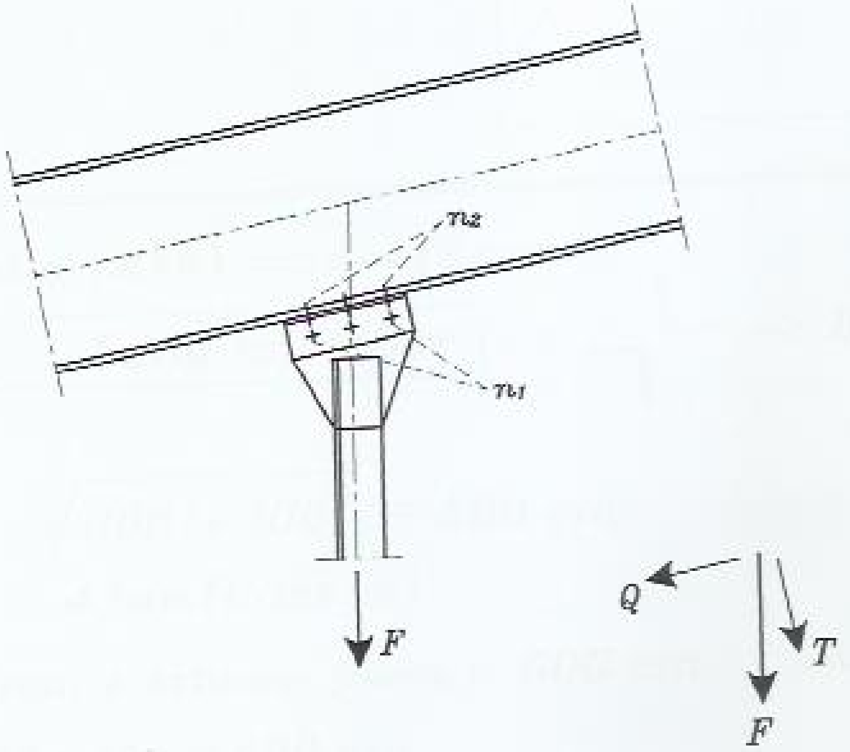
$n_{bolts} = \frac{15t}{6.48} = 2.31$ use $n_{bolts} = 4bolts$



لاحظ انه من الممكن رص المسامير فى الزاوية على **2 gauge line** وذلك لتقليل مساحة ال **gusset plate**

Idea





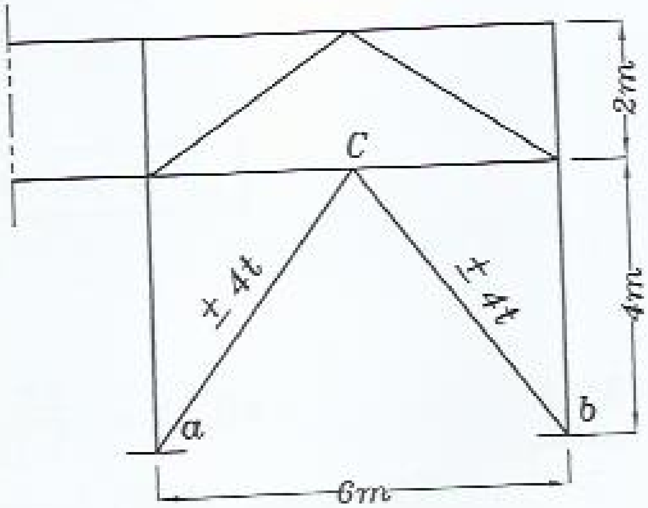
$\boxed{\text{For } n_1}$ ⇒ **Shear** ⇒ *Connecting gusset plate with angles*
F

$\boxed{\text{For } n_2}$ ⇒ **Shear+ Tension**
Q T

⇒ *Connecting angles with column*

Question EXAM (2003)

Figure (3) shows a part of vertical bracing system of a steel building. Available sections are equal angles 80x80x8 OR LESS Design the bracing members (a–C) and (b–C) from the available sections.



Assume Bolts (M16) $\Longrightarrow \phi = 16\ mm$

$\boxed{L > 4m \Longrightarrow \text{Long member}}$ $\Rightarrow$ ┘└ $\Rightarrow$ **Bolted**

1) Data

* Length $= \sqrt{300^2 + 400^2} = \mathbf{500\ cm}$
* Force $= \pm\ \mathbf{4}\ ton$ (Case A)
* $l_{b\,in}$ = Distance between joints = **500** cm   لا نحتاج الى حسابها
* $l_{b\,out} = 1.20 * 500 = \mathbf{600}\ cm$

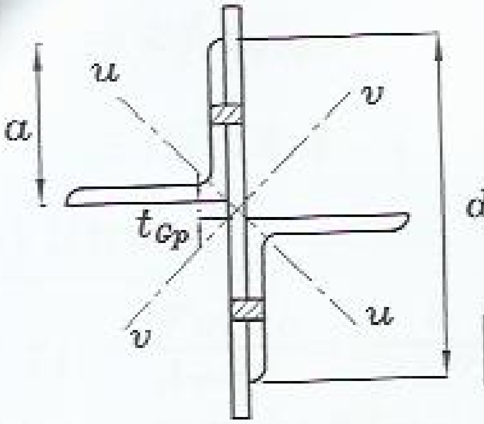
2) Choice of section

توفيرا للوقت لن نقوم بعمل Choice of section حيث أن القطاعات المتاحة هى 80x8 ∟
و بالتالى سنجرب 80x8 ∟ أولا و اذا كانت Waste من الممكن أن نختار أقل منها .

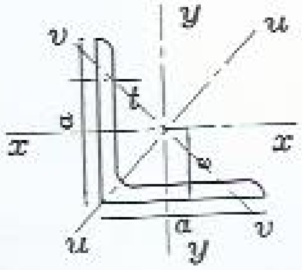
$\Longrightarrow$ $\boxed{\text{Use} \ \lrcorner\!\llcorner\ 80*80*8}$

نقوم أولا بعمل Checks على الـ Compression ثم الـ Tension حيث أنهما متساويان

Checks



$$\boxed{d = 2a + t_{Gp}}$$

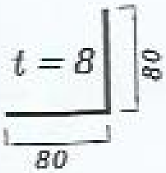


$\llcorner 80*80*8$

$A = 12.3\ Cm^2$

$r_u = 3.06\ Cm$

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) Slenderness (Buckling)

Bracing

Compare by 180 not 200

$r_{u_{\neg\llcorner}} = r_{u_{\llcorner}}$ من الجدول $= 2.47\ cm$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{u_{\neg\llcorner}}} = \frac{600}{3.06} = 196 < 200 \Longrightarrow (Safe)$$

c) Stress Compression

$$\lambda_{max.} = 196 > 100$$

$$* F_C = \frac{7500}{\lambda^2_{max.}} = \frac{7500}{196.0^2} = 0.195\ t \setminus cm^2$$

$$* f_C = \text{actual stress} = \frac{force}{2*A_{g\llcorner}} = \frac{4.0}{2*12.3} = 0.162\ t \setminus cm^2$$

$$\leqslant F_C \Longrightarrow (Safe)$$

$$* \frac{f_C}{F_C} = \frac{0.162}{0.195} = 0.83 \Longrightarrow (\text{Safe but waste})$$

Checks as tension member

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

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

a) Stress

$$* \ f_t = \frac{Force}{A_{net}} = \frac{4}{21.72} = 0.184\ t \backslash cm^2 \leqslant F_t = 1.40\ t \backslash cm^2 \ (Safe)$$

مساحة الـ angles التى تم حسابها

b) Slenderness (Stiffness)

لا نحتاج الى حسابه لاننا حسبناه فى الـ Compression و بسماحية أقل .

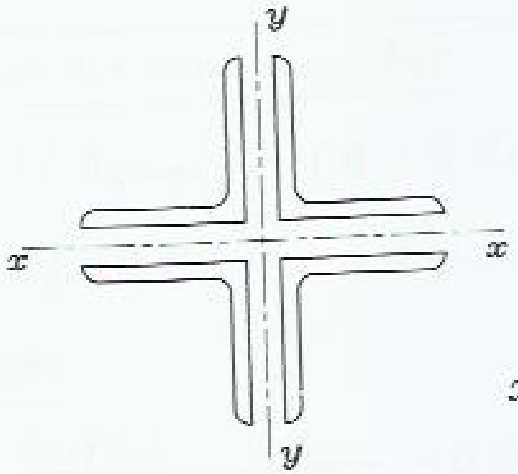
C) Length to depth ratio. (Deflection)

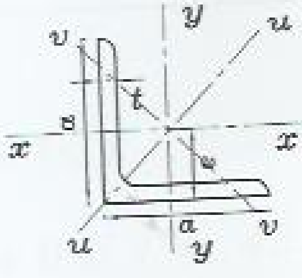
$$* \ \frac{L}{d} = \frac{500\ cm}{2a + t_{Gp}} = \frac{500\ cm}{2*8.0+1} = 29.4 \leqslant 60 \Longrightarrow (Safe)$$

$$\Longrightarrow \boxed{Use \ \neg\!\vdash 80*80*8}$$

فى حالة اذا كان استخدام ┤├ 80*80*8 كان Unsafe و نحن لا يمكننا هنا استخدام Angles اكبر من الممكن استخدام الشكل التالى ╬ 80*80*8

وفى هذه الحالة تكون الـ Checks كالتالى





$L\ 80*80*8$

$A = 12.3\ Cm^2$

$r_x = 2.42\ Cm$

$r_y = 2.42\ Cm$

$e = 2.26\ Cm$

نلاحظ أنه حدث نقل محاور لـ x & y

و بالتالى نحسب الـ r_x & r_y

Assume $t_{Gp} = 1.0$ cm

$$r_{y_{JL}} = \sqrt{r_{yL}^2 + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.42^2 + (2.26 + \frac{1.0}{2})^2} = 3.67 \text{ cm}$$

$$r_{x_{JL}} = \sqrt{r_{xL}^2 + (e + \frac{t_{Gp}}{2})^2} = \sqrt{2.42^2 + (2.26 + \frac{1.0}{2})^2} = 3.67 \text{ cm}$$

b) Slenderness (Buckling)

$$* \lambda_{in} = \frac{l_{b\,in}}{r_{x_{JL}}} = \frac{500}{3.67} = 136.2 < 180 \Rightarrow \text{(Safe)}$$

$$* \lambda_{out} = \frac{l_{b\,out}}{r_{y_{JL}}} = \frac{600}{3.67} = 163.4 < 180 \Rightarrow \text{(Safe)}$$

$\lambda_{max} = 163.4$

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

c) Stress Compression

$$\lambda_{max.} = 163.4 > 100$$

$$* F_C = \frac{7500}{\lambda^2_{max.}} = \frac{7500}{163.4^2} = 0.281 \ t \backslash cm^2$$

$$* f_C = \text{actual stress} = \frac{force}{4 * A_{gL}} = \frac{4.0}{4 * 12.3} = 0.08 \ t \backslash cm^2$$

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

$$* \frac{f_C}{F_C} = \frac{0.08}{0.281} = 0.29 \Rightarrow \text{(Safe but waste)}$$

Checks as tension member

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

$$= 4 [12.3 - (1.6 + 0.2 \text{cm}) * 0.8] = 43.44 \text{ cm}^2$$

a) Stress

$$* f_t = \frac{Force}{A_{net}} = \frac{4}{43.44} = 0.09 \ t \backslash cm^2$$

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

مساحة الـ angles التى تم حسابها

b) *Slenderness (Stiffness)*

لا نحتاج الى حسابه لاننا حسبناه فى ال *Compression* و بسماحية أقل .

C) ***Length to depth ratio. (Deflection)***

$$* \frac{L}{d} = \frac{500\ cm}{2a + t_{Gp}} = \frac{500\ cm}{2 * 8.0 + 1} = 29.4 \leqslant 60 \Longrightarrow \textbf{(Safe)}$$

Question EXAM (2003)

Discuss briefly the function of the longitudinal bracing.

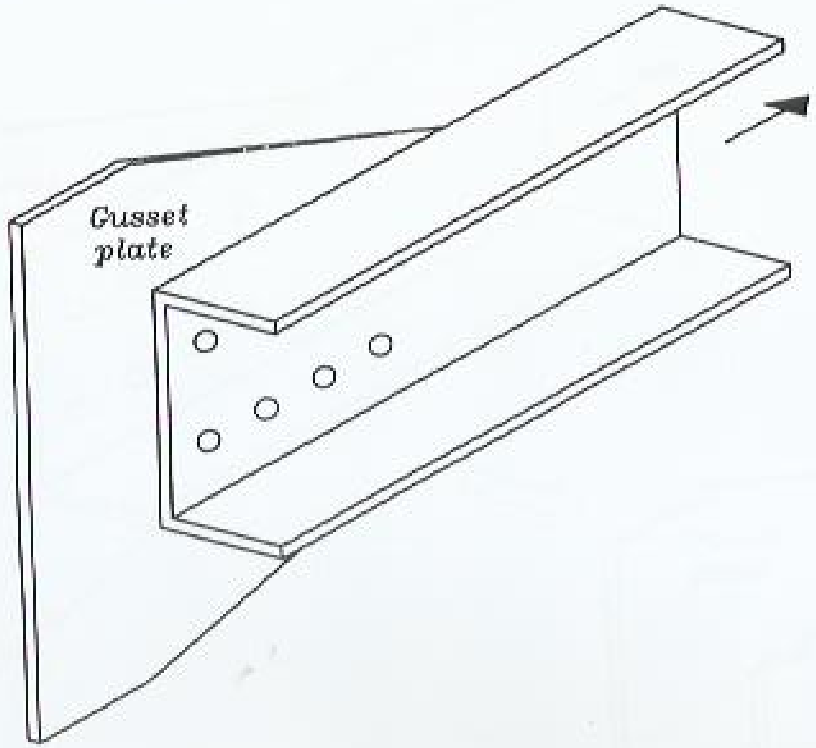
1- Brace the whole structure in the longitudinal direction which results in acting all trusses as a space structure to reduce the deflection.

2- Reduce the buckling length of the chord members.

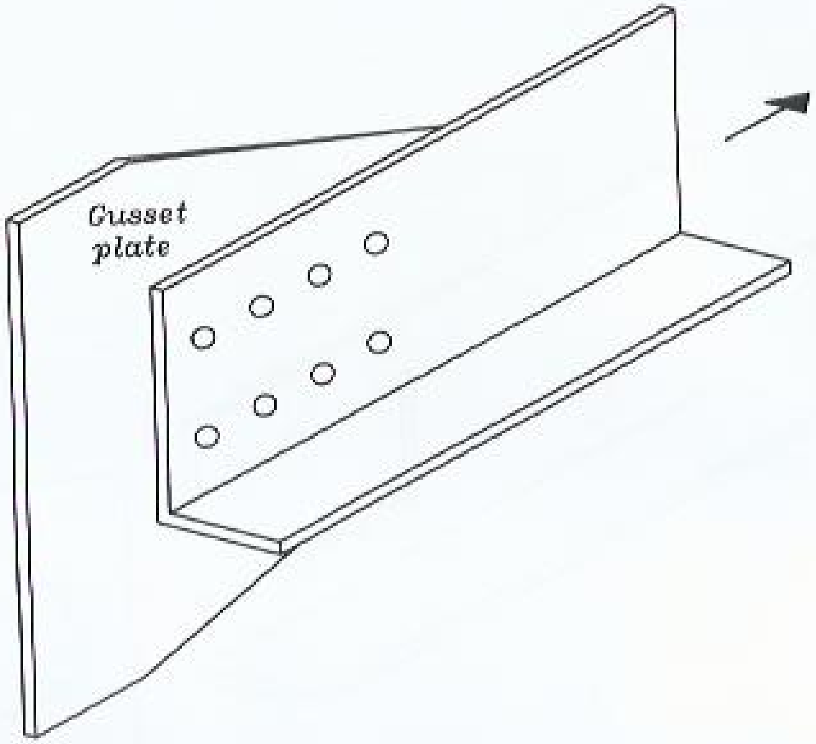
3- Brace the lower chord joints in the longitudinal direction.

Shear rupture strength of the member Code page (112)

In case of two gauge lines



CHANNEL

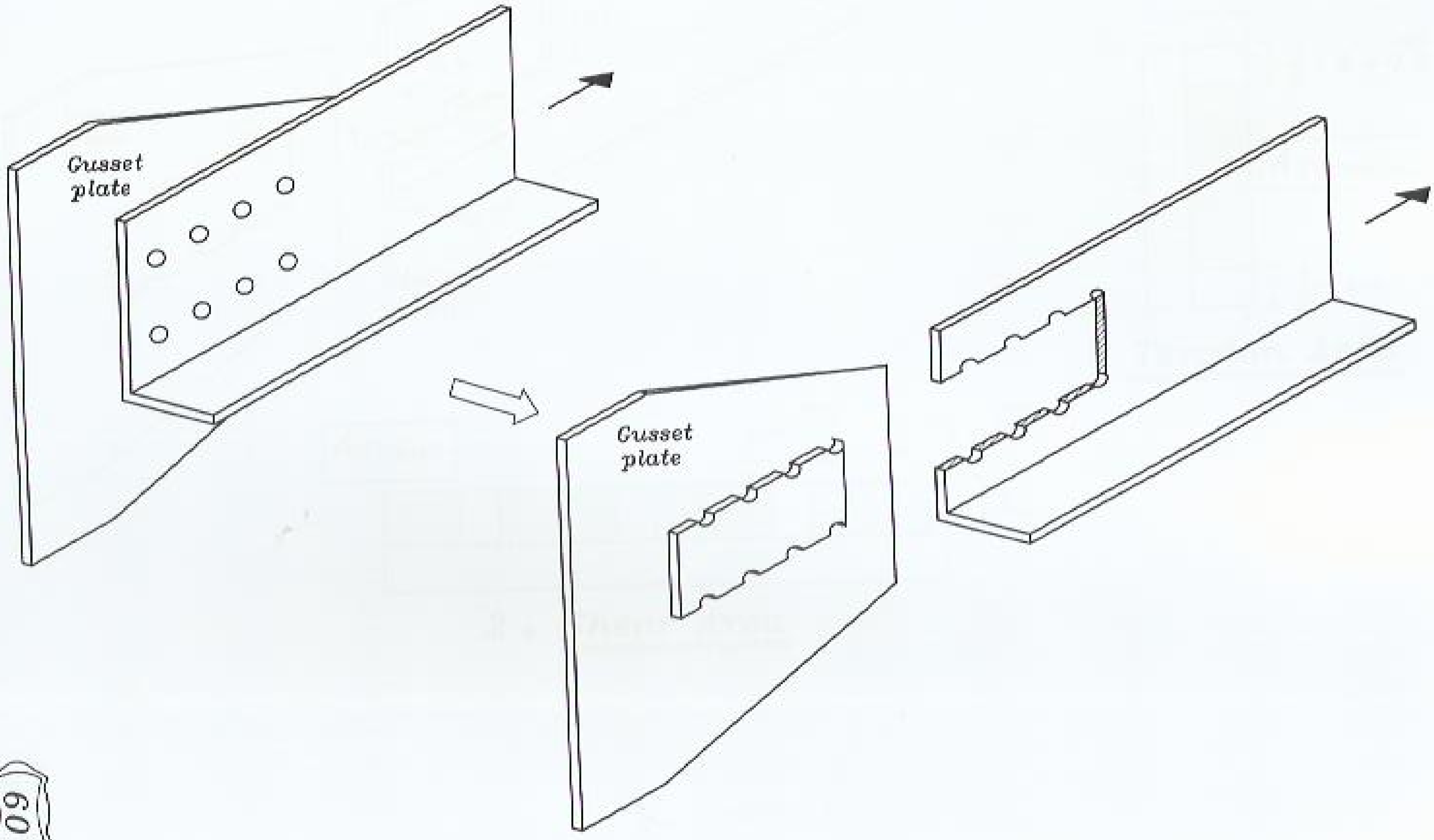


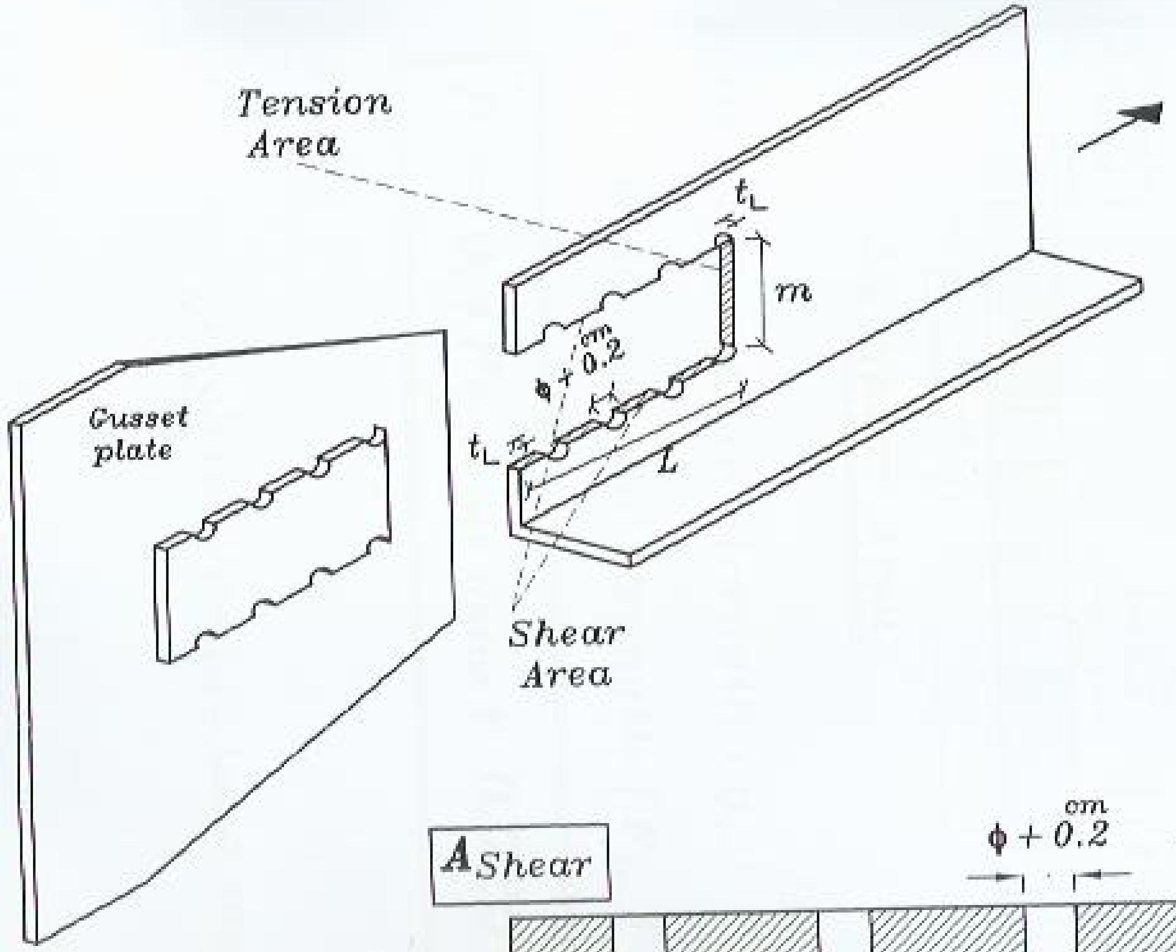
ANGLE

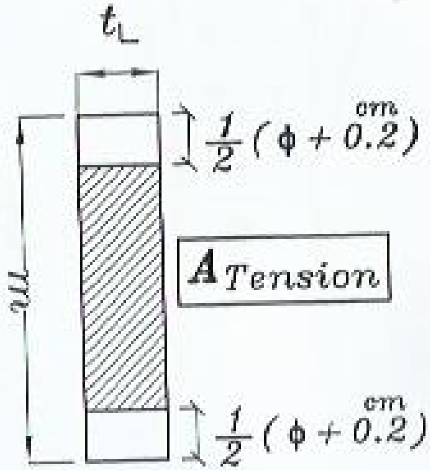
Shear rupture strength of the member Code page (112)

In case of two gauge lines

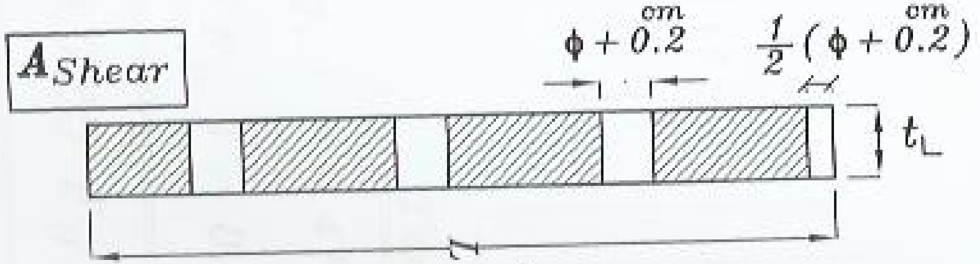
هو احتمال للـ Failure من الممكن حدوثه و هو عبارة عن Combination بين الـ Shear و الـ Tension فى الـ angles .



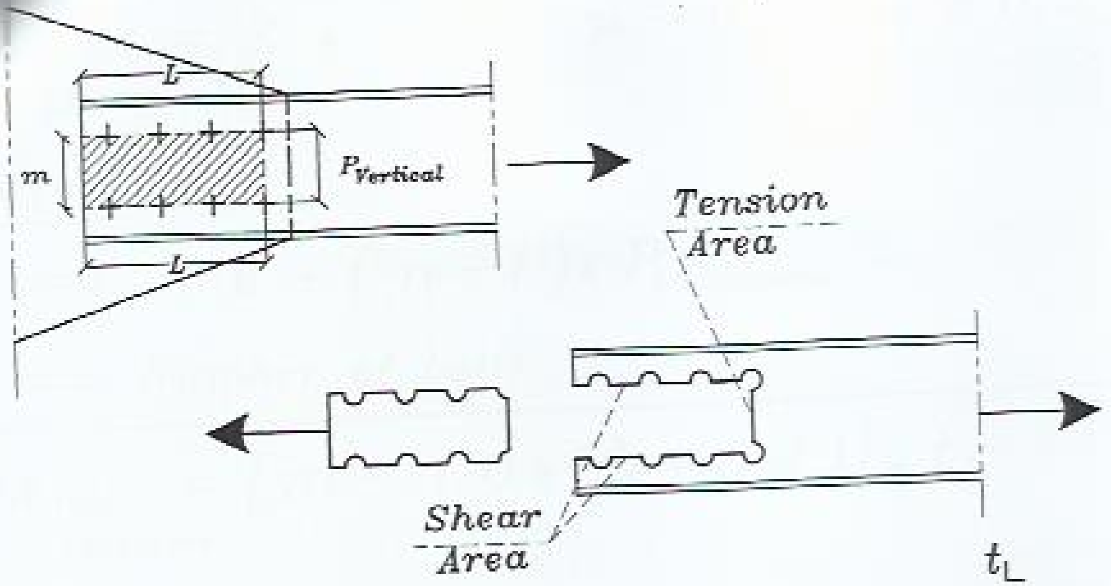


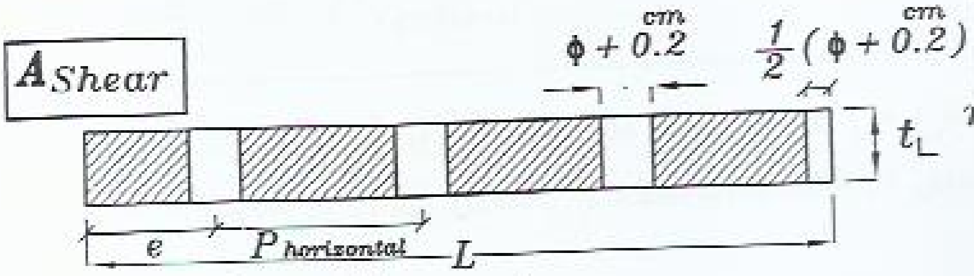


Tension Area

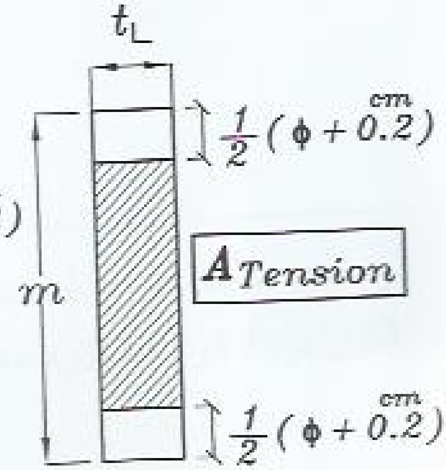


2 * Shear Area





2 * **Shear Area**



Tension Area

\# Allowable shear strength (q_r) = $0.4F_y$

\# Allowable tensile strength (F_{tr}) = $0.725\ F_y$

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

Where

$P \Longrightarrow$ أكبر Force يمكن لل angle تحملها بدون حدوث هذا النوع من ال Failure .

$$\Longrightarrow A_{\substack{net \\ Shear}} = 2 * [L - (n - 0.5)(\phi + 0.2)] * t_L$$

Where

$L \Longrightarrow = e + (n-1) * P_{horizontal}$

$n \Longrightarrow$ Number of bolts

$$\Longrightarrow A_{\substack{net \\ Tension}} = [m - 1.0 * (\phi + 0.2)] * t_L$$

Where

$m \Longrightarrow = P_{Vertical}$

و حتى نتأكد أن هذا النوع من الـ Failure لن يحدث لابد أن تكون الـ P أكبر من القوة المؤثرة على الـ angle الواحدة و بالتالى يكون الـ Check كالتالى :

Check

$P >$ Force on member $\Longrightarrow$ (Single angle) (Single Channel)

$2P >$ Force on member $\Longrightarrow$ (Double angle) (Double Channel)

ملحوظة هامة

الـ Shear rupture check يتم عمله فقط لـ Tension member
و لا يتم عمل هذا الـ check أيضا لـ Continuous member

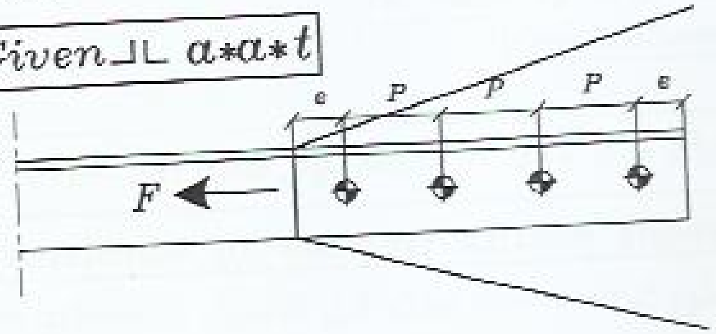
Maximum Capacity

الفكرة هنا أننا نريد أن نحسب لـ Connection موجودة بالفعل قدرة تحملها أي أكبر حمل يمكنها تحمله . و بمعنى أخر أننا نعلم كل شئ عن الـ Connection أي أبعاد الـ members و عدد المسامير و طول اللحام و سمكه لانها وصلة موجودة بالفعل و نريد أن نحسب الحمل التى يمكنها تحمله .

الفكرة هنا أننا سنقوم بحساب الحمل الذى يستطيع الـ member تحمله و الحمل الذى تستطيع المسامير تحمله و الحمل الذى يستطيع اللحام تحمله و هكذا ثم نختار الحمل الاقل و يكون هو قدرة تحمل الـ Connection .

فمثلا نريد أن نوجد الـ Maximum Capacity لهذه الـ Connection .

Given لـ $a*a*t$



أولا F_1

نوجد الحمل الذى يستطيع الـ member تحمله و الـ member عبارة عن Double angle .

a) Calculate the maximum force carried by member

In case of tension

Allowable tensile stress

$$Stress = \frac{Force}{Area} \qquad F_t = \frac{F}{A_{net}} = 0.58F_y$$

$$\boxed{F_1 = F_t * A_{net} = 0.58F_y * A_{net} = \checkmark \ ton}$$

In case of Compression

Allowable compressive stress

$$Stress = \frac{Force}{Area} \qquad F_C = \frac{F}{A_{gross}}$$

$$\boxed{F_1 = F_C * A_{gross} = \checkmark \ ton}$$

ثانيا F_2 & F_3

نوجد الحمل الذى تستطيع المسامير تحمله .

b) Calculate the maximum force carried by the number of bolts

نوجد أولا الحمل الذى تستطيع المسامير تحمله قبل انهيار المسامير نفسها .

b-1) From the bolts resistance $\boxed{\text{For Tension Force only}}$

$$n = \frac{F_2}{R_{Least}} \rightarrow \begin{cases} R_{Shear} \\ R_b \end{cases}$$

In case of bearing type Connection

$$\boxed{F_2 = \text{Maximum force carried by bolts} = n * R_{Least} = \checkmark \text{ ton}}$$

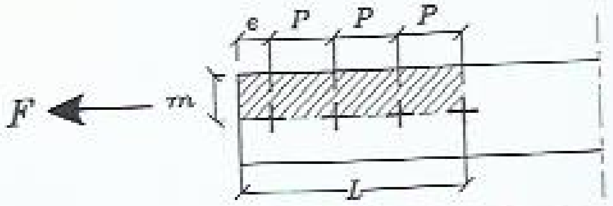
$$n = \frac{F_2}{2 * P_S}$$

In case of slip-critical Connection

$$\boxed{F_2 = \text{Maximum force carried by bolts} = n * 2 * P_S = \checkmark \text{ ton}}$$

b-2) From the resistance of member (Block shear rupture)

نوجد الحمل الذى يستطيع الـ member تحمله قبل الانهيار بالـ Block shear rupture



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

و الـ P هى مقاومة الزاوية الواحدة و لكن فى هذا المثال يوجد زاويتان و بالتالى تكون الـ F كالتالى

$$\boxed{F_3 = 2P}$$

من المفترض فى هذه المسائل أن تكون الـ e&P معطاه فى المسألة و لكن فى حالة عدم وجودهم نقوم بفرضهم على الـ minimum dimensions .

F_3 = Maximum force without block shear rupture

$= P$ ton $\Rightarrow$ For single angle

$= 2P$ ton $\Rightarrow$ For Double angles

ملحوظة هامة

لا نقوم بحساب تحمل الـ *Member* من الـ *Block shear rupture* فى الحالات الاتية

1- *Compression Forces*

2- *Slip Critical connection*

3- *Continuous Joint*

4- *When using splices*

و تكون الـ *Maximum Capacity* لهذه الـ *Connection* هو الحمل الاقل من $F_1 \& F_2 \& F_3$

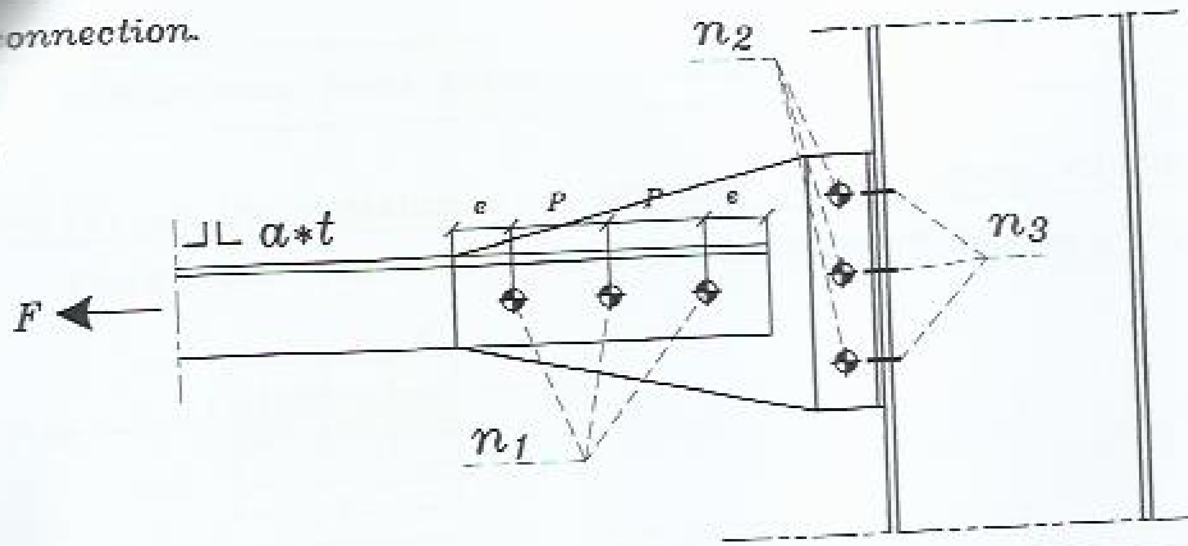
و سنقوم بتطبيق الفكرة السابقة على أشكال و أنواع مختلفة من الوصلات و المسامير كما درسنا سابقا أى أن ما ذكرناه سابقا سوف يختلف حسب نوع الوصلة و نوع المسامير .

ملحوظة هامة

من المفترض حساب قوة تحمل الـ *Gusset Plate* و لكن من الممكن اهمال هذه الخطوة .

Example

Find the maximum capacity (F) can be carried by the shown connection.



F_1 أولا

نوجد الحمل الذى يستطيع الـ member تحمله و الـ member عبارة عن Double angle .

a) Calculate the maximum force carried by member

In case of tension

$$Stress = \frac{Force}{Area} \qquad \underbrace{F_t}_{\text{Allowable tensile stress}} = \frac{F}{A_{net}} = 0.58F_y$$

$$\boxed{F_1 = F_t * A_{net} = 0.58F_y * A_{net} = \checkmark \ \text{ton}}$$

F_2 & F_3 ثانيا

نوجد الحمل الذى تستطيع المسامير n_1 تحمله .

هذه المسامير معرضة لـ Shear .

b) Calculate the maximum force carried by n_1

b-1) From the bolts resistance

$$n_1 = \frac{F_2}{R_{Least}} \quad R_{Least} \rightarrow \begin{cases} R_{Shear} \\ R_b \end{cases}$$

In case of bearing type Connection

$$\boxed{F_2 = \text{Maximum force carried by bolts} = n_1 * R_{Least} = \checkmark \ \text{ton}}$$

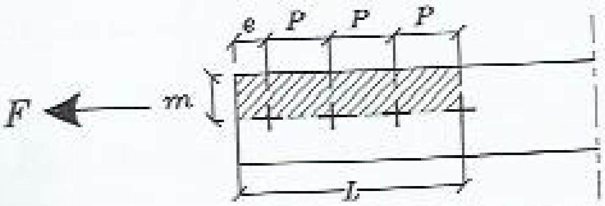
In case of slip-critical Connection

$$n_1 = \frac{F_2}{2 * P_S}$$

$$\boxed{F_2 = \text{Maximum force carried by bolts} = n_1 * 2 * P_S = \checkmark \text{ ton}}$$

b-2) From the resistance of member (Block shear rupture)

نوجد الحمل الذى يستطيع الـ member تحمله قبل الانهيار بالـ Block shear rupture



$$\boxed{P = 0.4 F_y \, A_{net\ Shear} + 0.725 \, F_y \, A_{net\ Tension}}$$

و الـ P هى مقاومة الزاوية الواحدة و لكن فى هذا المثال يوجد زاويتان و بالتالى تكون الـ F كالتالى

$$\boxed{F_3 = 2P}$$

ثالثا F_4

نوجد الحمل الذى تستطيع المسامير n_2 تحمله .

هذه المسامير معرضة لـ Shear .

C) Calculate the maximum force carried by n_2

C-1) From the bolts resistance

In case of bearing type Connection

$$n_2 = \frac{F_4}{R_{Least}} \quad R_{Least} \rightarrow \begin{cases} R_{Shear} \\ R_b \end{cases}$$

$$\boxed{F_4 = \text{Maximum force carried by bolts} = n_2 * R_{Least} = \checkmark \text{ ton}}$$

In case of slip-critical Connection

$$n_2 = \frac{F_4}{2 * P_S}$$

$$\boxed{F_4 = \text{Maximum force carried by bolts} = n_2 * 2 * P_S = \checkmark \text{ ton}}$$

رابعا F_5

نوجد الحمل الذى تستطيع المسامير n_3 تحمله .

هذه المسامير معرضة لـ Tension .

D) Calculate the maximum force carried by n_3

D-1)From the bolts resistance

In case of non-pretensioned Bolts

$$n_3 = \frac{F_5}{R_t}$$

$$\boxed{F_5 = \text{Maximum force carried by bolts} = n_3 * R_t = \checkmark \text{ ton}}$$

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

In case of pretensioned Bolts

$$n_3 = \frac{F_5}{0.6\ T}$$

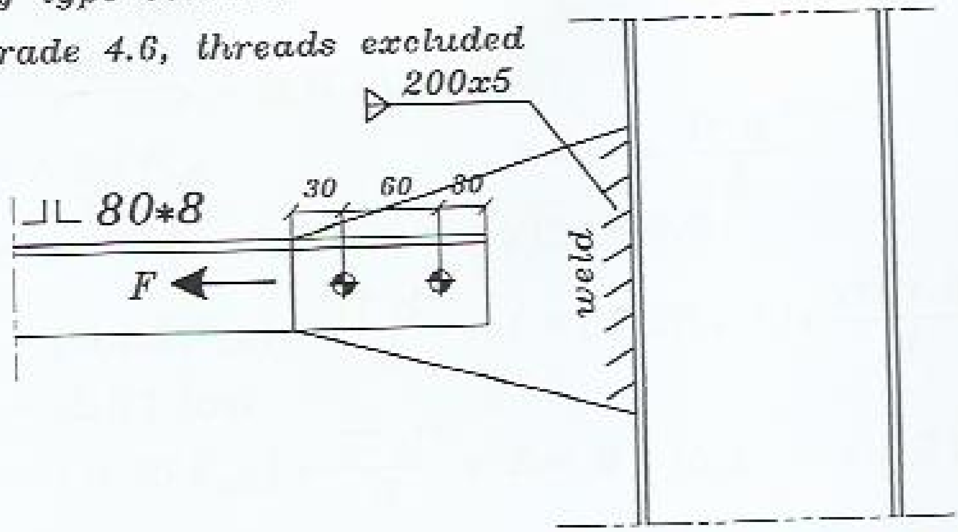
$$\boxed{F_5 = \text{Maximum force carried by bolts} = n_3 * 0.6\ T = \checkmark \text{ ton}}$$

و تكون الـ Maximum Capacity لهذه الـ Connection هو الحمل الاقل من $F_1 \& F_2 \& F_3$
$F_4 \& F_5$

Example

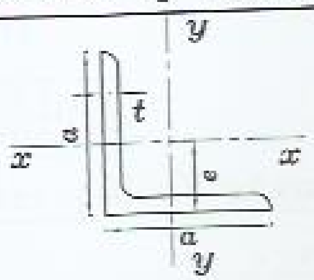
Find the maximum capacity (F) can be carried by the shown bearing type connection.

M16 grade 4.6, threads excluded



For the member

a) Calculate the maximum force carried by member



L 80*80*8

$A = 12.3 Cm^2$

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

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

$$* F_1 = F_t * A_{net} = 0.58 F_y * A_{net}$$

$$= 0.58 * 2.4 * 21.74 = \boxed{30.4\ ton}$$

b) Calculate the maximum force carried by the number of bolts

b-1) From the bolts resistance

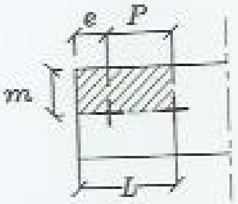
$$n = \frac{F_2}{R_{Least}} \rightarrow \begin{cases} R_{Shear} \\ R_b \end{cases}$$

In case of bearing type Connection

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

* $F_{ub} = 4\ t \backslash cm^2$ * $\phi = 1.6\ cm$
* $F_u \xrightarrow{for\ st.37} = 3.6\ t \backslash cm^2$
* $q_b = 0.25 F_{ub}$ * $A_S = \frac{\pi d^2}{4}$
* Take $e > 2\phi = 3.2\ cm \Longrightarrow \alpha = 0.8$
* $R_{S.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 1 = (0.25 * 4) * \frac{\pi (1.6)^2}{4}$
  $= 2.01\ ton$
* $R_{D.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 2 = 2 * R_{S.S} = 4.02\ ton$
* $R_b = (\alpha * F_u) * d * t_{min} = 0.8 * 3.6 * 1.6 * t_{min} = 4.6\ t_{min}$
* $t_{min} = \overset{tG.p}{1\ cm}$ or $\overset{t_L}{2 * 0.8}\ cm = 1.6\ cm \Longrightarrow t_{min} = 1\ cm$
* $R_b = 4.6\ t_{min} = 4.6 * 1 = 4.60\ ton$
* $F_2 = n * R_{Least} = 2 * 4.02 = \boxed{8.04\ ton}$

b-2) From the resistance of member (Block shear rupture)



* Given $e = 3.0\ cm$
  $P = 6.0\ cm$
* $m = \frac{a - t}{2} = \frac{8 - 0.8}{2} = 3.60\ cm$
* $L = e + 1 * P = 3.0 + 1 * 6 = 9.0\ cm$
* $A_{net\ Shear} = [L - (n - 0.5)(\phi + 0.2)] * t_L$
  $= [9.0 - (2 - 0.5)(1.6 + 0.2)] * 0.8 = 5.04\ cm^2$
* $A_{net\ Tension} = [m - 0.5 * (\phi + 0.2)] * t_L$
  $= [3.60 - 0.5 * (1.6 + 0.2)] * 0.8 = 2.16\ cm^2$
* $P = \mathbf{0.4 F_y\ A_{net\ Shear} + 0.725\ F_y A_{net\ Tension}}$
  $= 0.4 * 2.4 * 5.04 + 0.725 * 2.4 * 2.16 = 8.60\ ton$

Double angle $\Longrightarrow 2P = 17.2\,ton$

* $F_3 = \boxed{17.2\,ton}$

C) Calculate the maximum force carried by weld

* $F_4 = 0.2\,F_u * 2\,l * S_w$

لن يدخل فى امتحان ال miderm هذا العام

$= 0.2 * 3.6 * 2 * 20 * 0.5 = \boxed{14.4\,ton}$

نختار أقل Force

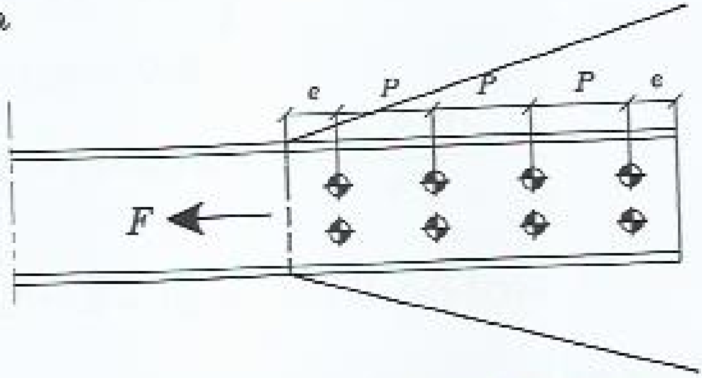
* $F_{maximum}$ = Maximum force carried by connection = 8.04 ton

Example

Find the maximum capacity (F) can be carried by the shown connection of a lower chord tension member with double UPN 160.

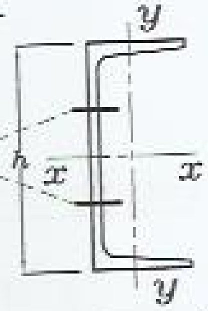
Bolts is grade M20 (8.8) Bearing type non-pretenisoned

* Gusset plate thickness = 12mm
* $l_{b\,in} = 2m$
* $l_{b\,out} = 6m$
* $L = 2m$



For the member

a) Calculate the maximum force carried by member



$\boxed{[\ 160}$ $\quad A = 24.0\ Cm^2$

$t_w = 0.75 Cm$

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

$$= 2[\,\mathbf{24.0} - (2.0 + 0.2_{cm}) * 0.75] = \mathbf{41.4}\ cm^2$$

$$* F_1 = F_t * A_{net} = 0.58 F_y * A_{net}$$

$$= 0.58 * 2.4 * 41.4 = \boxed{\mathbf{57.9}\ ton}$$

b) Calculate the maximum force carried by the number of bolts

b-1) From the bolts resistance

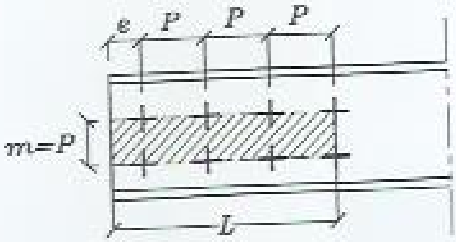
$$n = \frac{F_2}{R_{Least}} \rightarrow \begin{cases} R_{Shear} \\ R_b \end{cases}$$

In case of bearing type Connection

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

* $F_{ub} = 8\ t\backslash cm^2$ * $\phi = 2.0\ cm$
* $F_u \xrightarrow{\text{for st.37}} = 3.6\ t\backslash cm^2$
* $q_b = 0.25 F_{ub}$ * $A_S = \frac{\pi d^2}{4}$
* Take $e > 2\phi = 3.2\ cm \Longrightarrow \alpha = 0.8$
* $R_{S.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 1 = (0.25 * 8) * \frac{\pi (2.0)^2}{4}$
  $= 6.28\ ton$
* $R_{D.S} = (0.25 F_{ub}) * \frac{\pi d^2}{4} * 2 = 2 * R_{S.S} = 12.56\ ton$
* $R_b = (\alpha * F_u) * d * t_{min} = 0.8 * 3.6 * 2.0 * t_{min} = 5.76\ t_{min}$
* $t_{min} = \overset{tG.p}{1.2\ cm}$ or $2 * \overset{t[}{0.75}\ cm = 1.5\ cm \Longrightarrow t_{min} = 1.2\ cm$
* $R_b = 4.6\ t_{min} = 5.76 * 1.2 = 6.91\ ton$
* $F_2 = n * R_{Least} = 8 * 6.91 = \boxed{55.28 ton}$

<u>b-2) From the resistance of member (Block shear rupture)</u>



* Take $e = 4.0\ cm$
  $P = 6.0\ cm \geq P_{min.} = 3\phi$
* $m = P = 6.0\ cm$
* $L = e + 3 * P = 4.0 + 3 * 6 = 22.0\ cm$
* $A_{net\ Shear} = [L - (n - 0.5)(\phi + 0.2)] * t[* \overset{\text{Two sides}}{2}$
  $= 2 * [22.0 - (\underbrace{4 - 0.5}_{\text{Two Holes}})(2.0 + 0.2)] * 0.75 = 21.45\ cm^2$
* $A_{net\ Tension} = [m - 0.5 * 2 * (\phi + 0.2)] * t[$
  $= [6.0 - 0.5 * 2 * (2.0 + 0.2)] * 0.75 = 2.85\ cm^2$
* $P = 0.4 F_y\ A_{net\ Shear} + 0.725\ F_y A_{net\ Tension}$
  $= 0.4 * 2.4 * 21.45 + 0.725 * 2.4 * 2.85 = 25.55\ ton$

Double angle $\Longrightarrow 2P = 51.5\,ton$

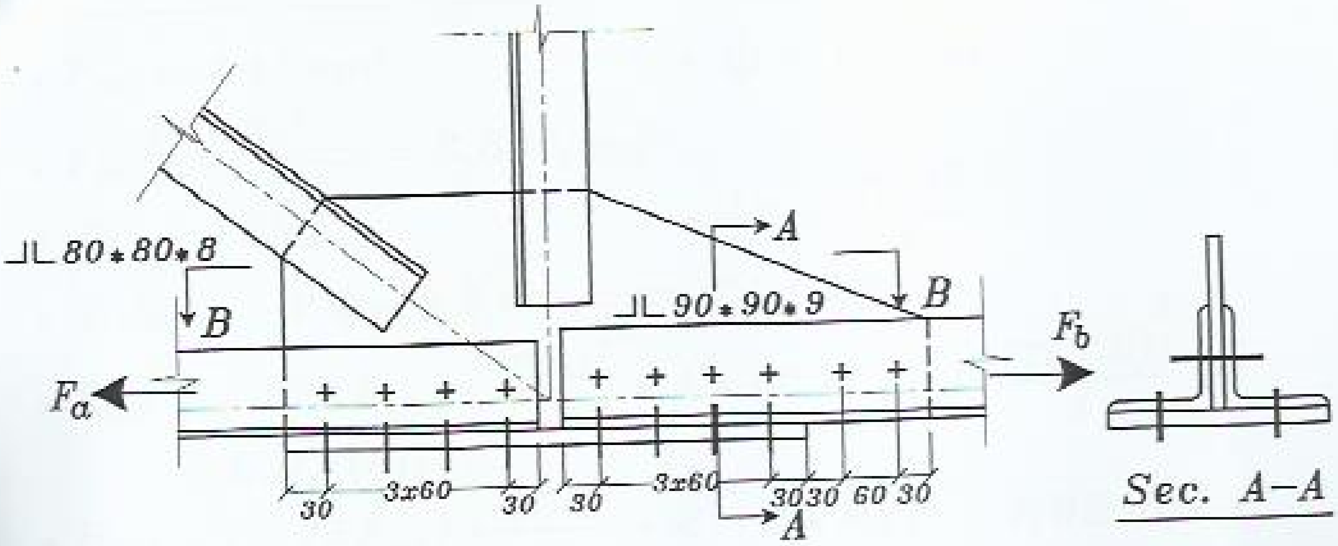
* $F_3 = \boxed{51.5\,ton}$

نختار أقل Force

* $\boxed{F_{maximum} = Maximum\ force\ carried\ by\ connection = 51.5\,ton}$

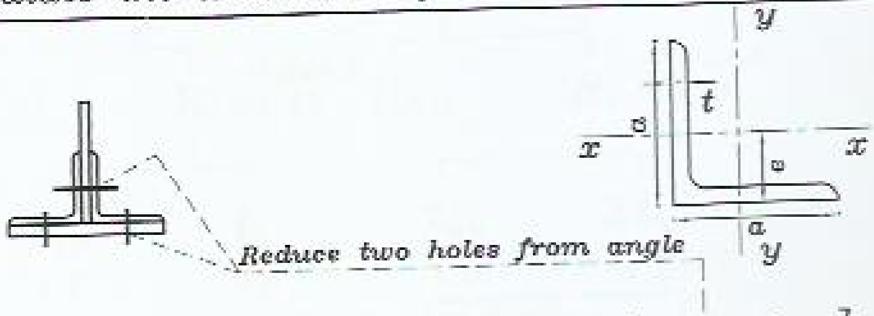
Example

Find the maximum capacity (Fa&Fb) can be carried by the shown connection.



For Capacity Fa

a) Calculate the maximum force carried by member



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

$$= 2[12.3 - (1.6 + 0.2\,cm) * 2 * 0.8] = \mathbf{18.90}\ cm^2$$

$$*F_1 = F_t * A_{net} = 0.58F_y * A_{net}$$

$$= 0.58 * 2.4 * 18.90 = \boxed{26.46\ ton}$$

Calculate the maximum force carried by the number of bolts

b-1) From the bolts resistance

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

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

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

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

$* \;$ Take $e > 2\phi = 3.2 \; cm \Longrightarrow \alpha = 0.8$

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

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

$$* \; R_b = (\alpha * F_u) * d * t_{min} = 0.8 * 3.6 * 1.6 * t_{min} = 4.6 \; t_{min}$$

$$* \; F_2 = n_{Sp} * R_{Least} + n_1 * R_{Least}$$

R_{Least} (first): $R_{S.S}$ or $R_b = 0.8 * 4.6 = 3.68$ (t_{sp} = 1.0, t_L = 0.8)

R_{Least} (second): $R_{D.S}$ or $R_b = 1.0 * 4.6 = 4.60$ (t_{gp} = 1.0, $2t_L$ = 1.6)

$$F_2 = 8 * 2.01 + 4 * 4.02 = \boxed{32.16 \; ton}$$

b-2) From the resistance of member (Block shear rupture)

Can be neglected as it will be very large.

نختار أقل Force

$$\boxed{* \; F_a = \text{Maximum force carried by connection} = 26.46 \; ton}$$

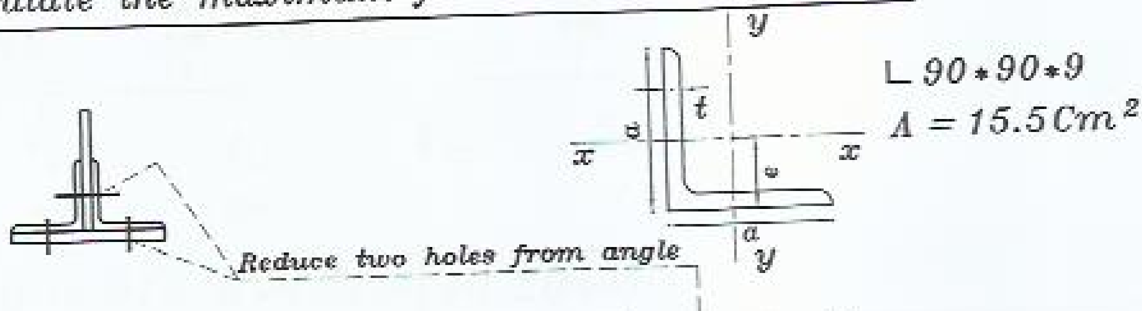
ملحوظة هامة

فى حالة ما اذا ذكر أن الوصلة Slip critical

$$* \; \boxed{F_2 = n_{Sp} * P_S + n_1 * 2P_S}$$

... Capacity F_b

a) Calculate the maximum force carried by member



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

$$= 2[15.5 - (1.6 + 0.2_{cm}) * 2 * 0.9] = 24.52\ cm^2$$

$$*F_1 = F_t * A_{net} = 0.58 F_y * A_{net}$$

$$= 0.58 * 2.4 * 24,52 = \boxed{34.1\ ton}$$

b) Calculate the maximum force carried by the number of bolts

b-1) From the bolts resistance

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

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

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

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

$* \text{Take } e > 2\phi = 3.2\ cm \Longrightarrow \alpha = 0.8$

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

$$= 2.01\ ton$$

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

$$* R_b = (\alpha * F_u) * d * t_{min} = 0.8 * 3.6 * 1.6 * t_{min} = 4.6\ t_{min}$$

$F_2 = n_{Sp} * R_{Least} + n_1 * R_{Least}$

R_{Least} (first term) → $R_{S.S}$; $R_b = 0.8 * 4.6 = 3.68$ → $t_{sp} = 1.0$; $t_{L} = 0.8$

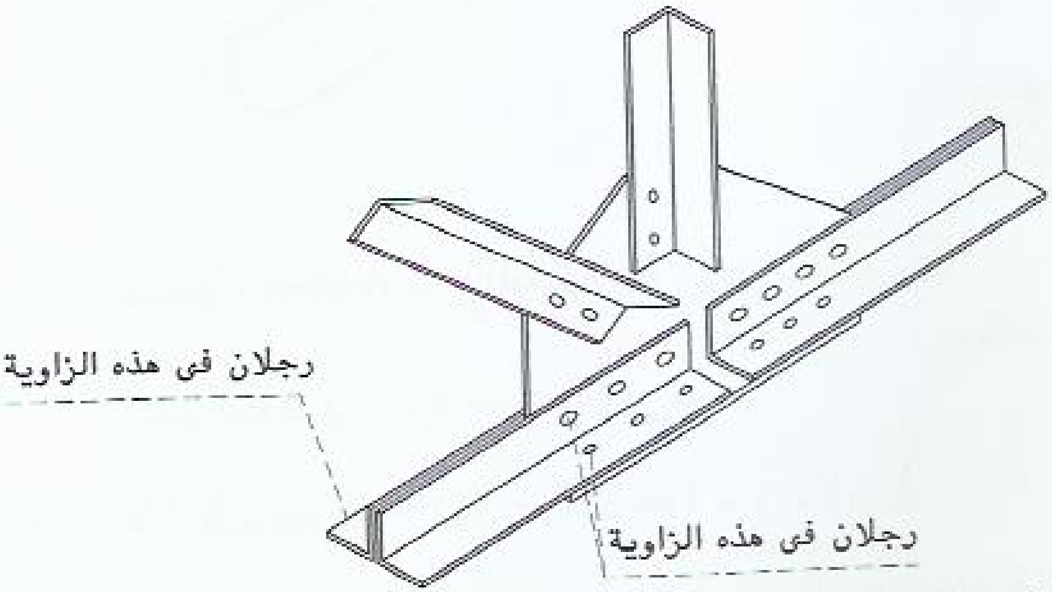
R_{Least} (second term) → $R_{D.S}$; $R_b = 1.0 * 4.6 = 4.60$ → $t_{gp} = 1.0$; $2t_{L} = 1.6$

$F_2 = 8 * 2.01 + 6 * 4.02 = \boxed{40.20\ ton}$

b-2) From the resistance of member (Block shear rupture)

Can be neglected as it will be very large.

لان الذى يقاوم هنا أربع أرجل و ليس رجلان فقط لان المسامير موجودة فى رجلى كل زاوية .



و بالتالى اذا اردنا حساب القوة التى تتحملها ال members بدون B.S.R

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

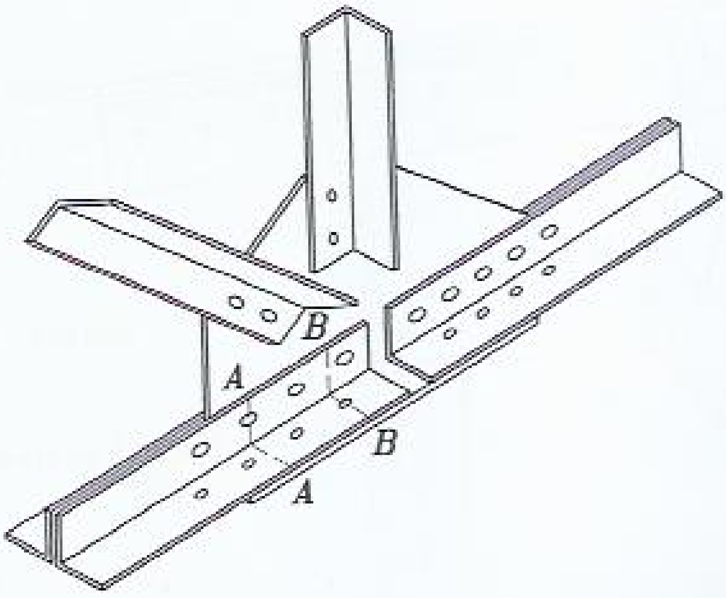
$F = 4P$

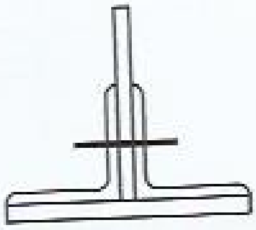
نختار أقل Force

$\boxed{* F_b = \text{Maximum force carried by connection} = 34.1\ ton}$

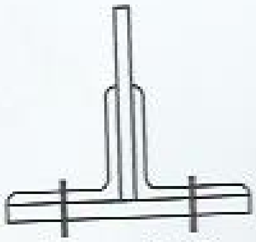
ملحوظة هامة

فى حالة ما اذا ذكر أن المسامير فى الـ *Splice* مرصوصة *Staggered* أى أن المسامير فى الرجل الرأسية ليست فى نفس مكان المسامير في الرجل الافقية .





Sec. A-A



Sec. B-B

و فى هذه الحالة عند حساب القوة التى يستطيع الـ *member* تحملها تكون الـ A_{net} كالتالى

سواء قطاع A أو B نطرح من الزاوية الواحدة فتحة واحدة

$$*\, A_{net} = 2\,[\, A_{gross\,\llcorner} - (\phi + 0.2\,cm) * 1 * t_{\llcorner} \,]$$

و لكن فى حالة عدم رص المسامير *Staggered* تكون الـ A_{net} كالتالى

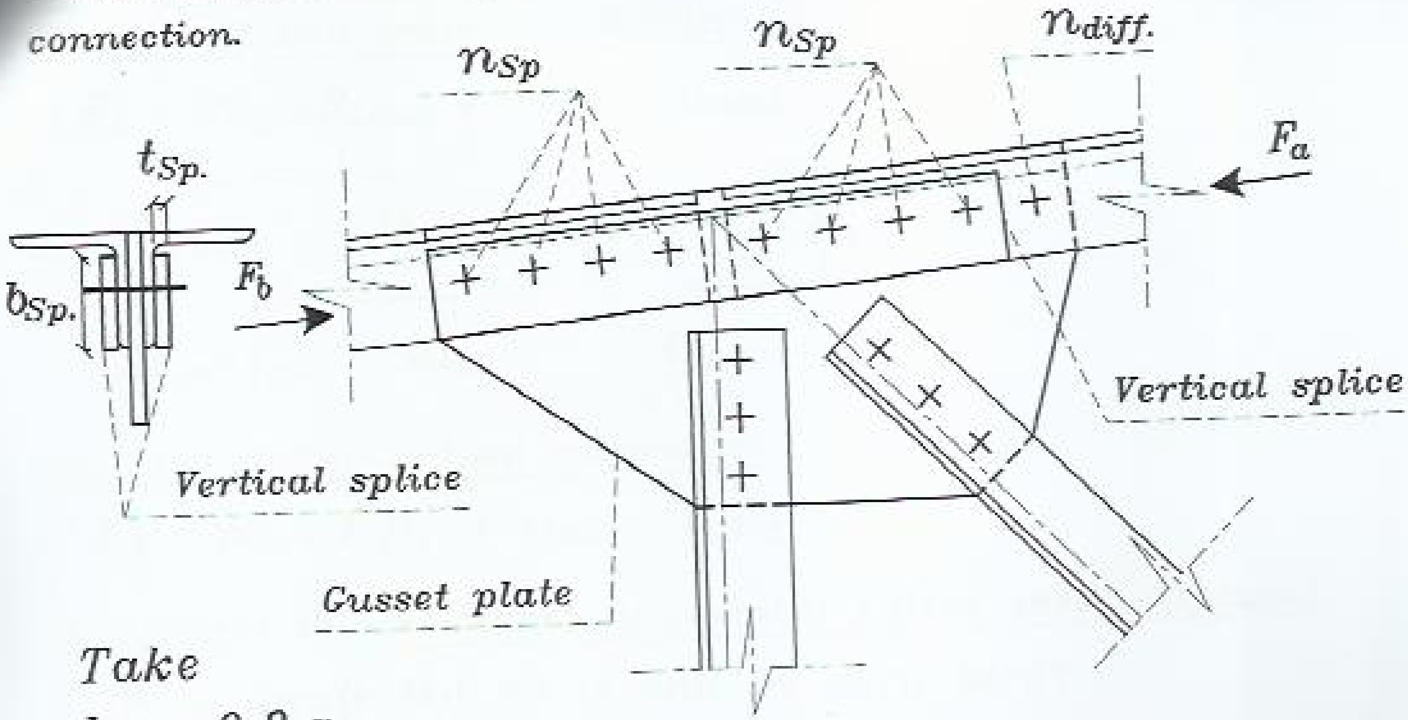
نطرح من الزاوية الواحدة فتحتان لانهما في نفس القطاع

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

ورص المسامير *Staggered* يعتبر أحد الطرق التى نتغلب بها على صغر الـ A_{net} و بالتالى نسمح لـ *member* بتحمل قوة أعلى .

Example

Find the maximum capacity (Fa&Fb) can be carried by the shown connection.



Take

$b_{Sp.} = 0.8\ a$

$t_{Sp.} = 10\ mm$

Solution

For Capacity Fa

a) Calculate the maximum force carried by member

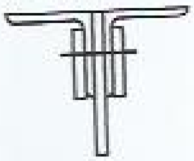
In case of compression

Calculate $\lambda_{max} \xrightarrow{Then}$ Calculate F_C

$* F_1 = F_C * A_{Gross}$

In case of Tension

Reduce two holes from angle



$* A_{net} = 2[A_{gross\,L} - (\phi + 0.2\,cm) * 1 * t_L]$

$* F_1 = F_t * A_{net} = 0.58 F_y * A_{net}$

Calculate the maximum force carried by the number of bolts

b-1) From the bolts resistance

In case of non-pretensioned Bolts

$$* F_2 = n_{Sp} * R_{Least} + n_{diff.} * R_{Least}$$

R_{Least} (first term): R_{4S} , R_b ($2t_{sp} + t_{gp}$, $2t_L$)

R_{Least} (second term): $R_{D.S}$, R_b (t_{gp} , $2t_L$)

In case of Slip critical connection

$$* F_2 = n_{Sp} * 4\ P_S + n_{diff.} * 2\ P_S$$

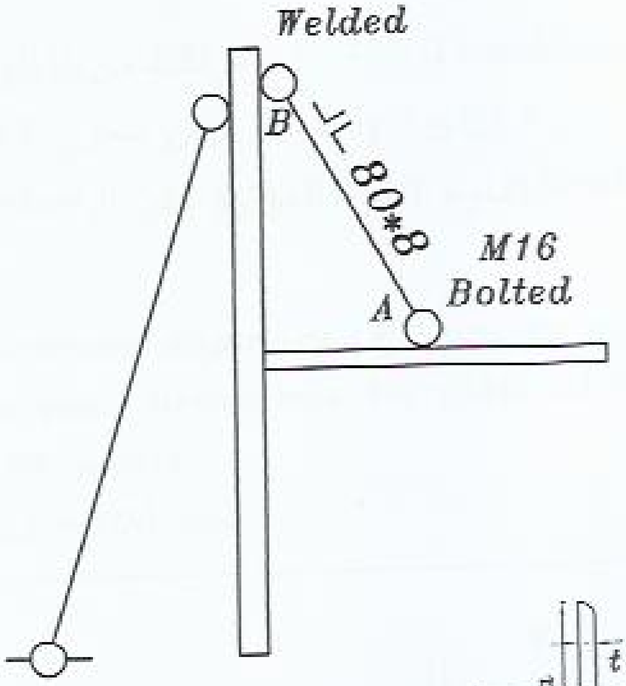
b-2) From the resistance of member (Block shear rupture)

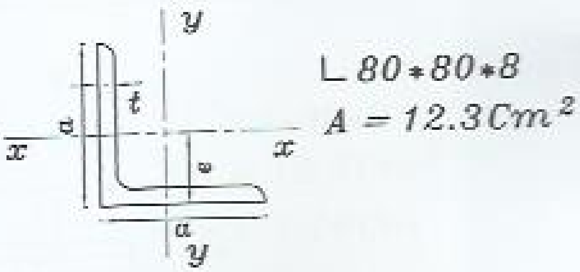
Can be neglected as it will be very large.

و تكون ال Maximum Capacity لهذه ال Connection هو الحمل الاقل من $F_1 \& F_2$

فكرة مهمة

عند وجود *member* له وصلة *Bolted* ووصلة أخرى *Welded* فعند حساب القوة التى يستطيع الـ *member* تحملها ناخذ الـ A_{net} وليس الـ A_{gross} حيث أن الـ A_{net} تعطى *Capacity* أقل .





For member AB

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

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

$$* F_{Max} = F_t * A_{net} = 0.58 F_y * A_{net}$$

$$= 0.58 * 2.4 * 21.74 = \boxed{30.4\ ton}$$

فكرة مهمة

فى حالة وجود Compression member تم تصميمه على أساس وجود Tie Plate واحد فى منتصف الـmember ثم طلب منا أن نحسب قوة تحمل الـ member فى حالة استخدام نفس الـ Double angles و لكن مع عدم وضع الـ Tie Plate .

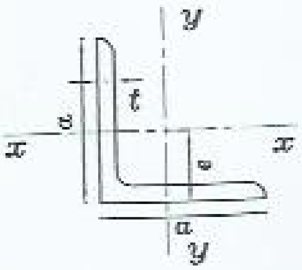
فى هذه الحالة سيحدث الـ Buckling لزاوية واحدة فقط من الزاويتين و بالتالى سيكون الـ Buckling حول محور V لزاوية واحدة و ليس محور X&Y للزاويتين . و لكن ستظل الـ Double angles تقاوم القوة المؤثرة على الـmember .

Example

Find the maximum capacity (F) can be carried by ⅃L 90∗90∗9 If it was designed using one tie plate at the middle, and now there is no tie plates.

$* \ l_{b\,in} = l_{b\,out} = 200 \ cm$

Solution



L 90∗90∗9

$A = 15.5 Cm^2$

$r_v = 1.76 Cm$

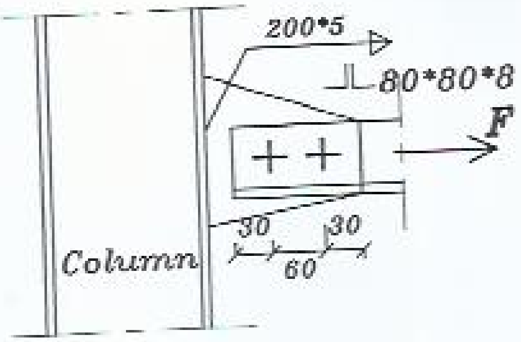
$$* \lambda_{out} = \frac{l_{b\,out}}{r_{v_L}} = \frac{200}{1.76} = 113.6 < 180 \Longrightarrow (Safe)$$

$$* F_C = \frac{7500}{\lambda^2_{max.}} = \frac{7500}{113.6} = 0.58 \ t \setminus cm^2$$

$$* F_{max.} = F_C * A_{gross} = 0.58 * 2 * 15.5 = 17.98 \ ton$$

Example

For the shown connection **it is required to find the** maximum Force (F)



use M16 bolt grade 4.6 **bearing type**, threads are **excluded** from plane of shear

---

لكى يتم حساب اقصى قوى **F** يتم اتباع الخطوات الاتية :-

١ - يتم حساب اقصى قوى تستطيع الـ **angle** تحملها
مع ملاحظة ان الـ **angle** معرضة الى قوى شد

a) For ⊥⊥80*80*8

- $f_{act} = \dfrac{force}{A_{net}} \leqslant 1.4 \ t/cm^2$

where:

من الجدول

$A_{net_{\perp\perp}} = 2[A_{g_{\llcorner}} - (\phi + 0.2^{cm}) x t_{\llcorner}]$

$= 2[12.3-(1.6+0.2)x0.8] = 21.72 \ cm^2 \quad \therefore 1.4 = \dfrac{force}{21.72}$

$F_1 = 21.72 * 1.4 = 30.408t$

$$\boxed{F_1 = \underline{30.408t}}$$

٢- يتم حساب اقصى قوى تستطيع وصلة المسامير تحملها

b) For bolts

$$n = \frac{F_2}{R_{least}}$$

$$R_{double\ shear}(R_{DS}) = q_b * A_s * 2$$

$$R_{D.S} = (0.25F_{ub} = 0.25*4)\left(\frac{\pi*1.6^2}{4}\right)(2) = 4.02t$$

$$R_b = F_b * d * \Sigma t_{min.} = \ldots\ ton$$

$$R_b = (0.8*3.6)(1.6)(1.0) = 4.60t$$

$\Sigma t_{min.}$: min. of $\boxed{2t_L = 0.8*2 = 1.6Cm}$ or $\boxed{t_{g.p} = 1Cm}$ govern

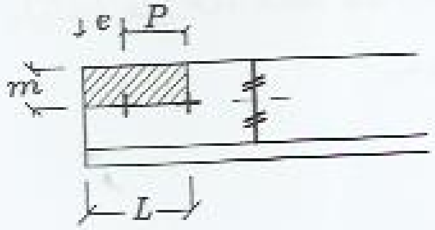
$$2 = \frac{F_2}{4.02} \qquad \boxed{F_2 = \underline{8.04t}}$$

c) block shear rupture

٣- يتم حساب اقصى مقاومه لل **block shear rupture**

e = 3.0Cm given

P = 6.0Cm given



$$m = \frac{a-t}{2} = \frac{8-0.8}{2} = 3.60Cm$$

$$L = 3.0+(2-1)*6 = 9Cm$$

$$A_{\substack{net\\shear}} = [L-(n-0.5)(\emptyset+0.2)]t_L$$

$$A_{\substack{net\\shear}} = [9.00-(1.5)(1.8)]*0.8 = \mathbf{5.04}\ Cm^2$$

$$A_{\substack{net\\tension}} = [m-0.5(\emptyset+0.2)]\,t_L$$

$$A_{\substack{net\\tension}} = [3.60-0.5(1.8)]*0.8 = \mathbf{2.16}Cm^2$$

$$P = 0.4F_y\ A_{\substack{net\\shear}} + 0.725F_y\ A_{\substack{net\\tension}}$$

$$P = 0.4*2.4*5.04+0.725*2.4*2.16 = \mathbf{8.6}\ ton$$

$$F_3 = 2P = 17.2\ ton \qquad \boxed{F_3 = \underline{17.2t}}$$

d)For Welded Connection

٤- يتم حساب اقصى مقاومه للحام الذى يربط لوح التجميع بالعمود

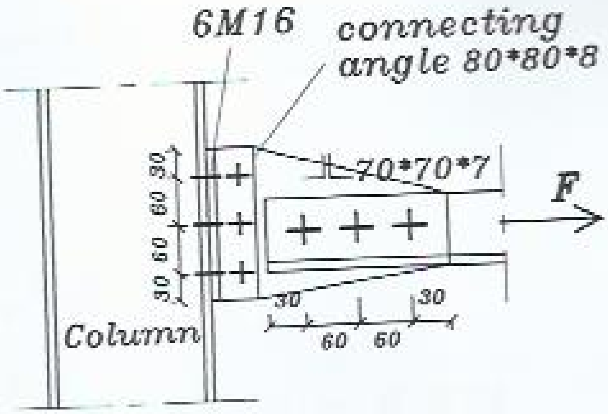
$$0.72\ t/Cm^2 = \frac{F}{A_{weld}} = \frac{F}{2*l*S} = \frac{F_4}{2*20*0.5}$$

$$\boxed{F_4 = \underline{14.4t}}$$

ويجب الملاحظة انه يتم اختيار اصغر قوى والتى تعمل على جعل جميع الحالات امنه وهى

$$\therefore \boxed{F_{max} = \underline{8.04t}}$$

Example



For the shown connection **it is required to** find the maximum Force (F)

use M16 bolt grade 10.9 **bearing type**, threads are **included** in plane of shear, **pretensioned** bolt

---

a) For ⫫80*80*8

- $f_{act} = \dfrac{force}{A_{net}} \leqslant 1.4\ t/cm^2$

where:

$A_{net\,⫫} = 2[A_{g_{L}} - (\phi + 0.2^{cm}) \times t_{L}]$

$= 2[9.4 - (1.6+0.2)x0.7] = 16.28\ cm^2 \quad \therefore 1.4 = \dfrac{force}{16.28}$

$F_1 = 16.28 * 1.4 = 22.792t$

$$\boxed{F_1 = \underline{22.792t}}$$

b) For bolts connecting lower chord to gusset plate

$$n = \frac{F_2}{R_{least}}$$

$$R_{double\ shear}(R_{DS}) = q_b * A_s * 2$$

$$R_{D.S} = (0.20F_{ub} = 0.20*10)\left(\frac{\pi*1.6^2}{4}\right)(0.78)(2) = 6.27t$$

$$R_b = F_b * d * \Sigma t_{min.} = \ldots\ ton$$

$$R_b = (0.8*3.6)(1.6)(1.0) = \mathbf{4.60t}$$

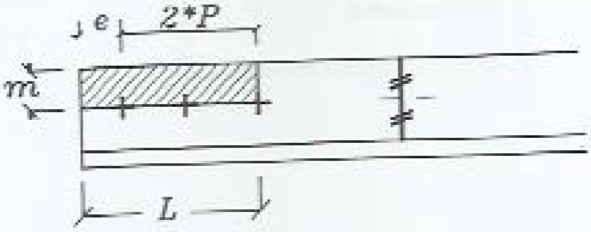
$\Sigma t_{min.}$: min. of $\boxed{2t_L = 0.7*2 = 1.4Cm}$ or $\boxed{t_{g.p} = 1Cm}$ **govern**

$$3 = \frac{F_2}{4.60} \qquad \boxed{F_2 = \underline{13.8t}}$$

c) block shear rupture

e = 3.0Cm **given**

P = 6.0Cm **given**



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

$$L = 3.0 + (3-1)*6 = 15Cm$$

$$A_{\substack{net\\shear}} = [L-(n-0.5)(\emptyset+0.2)]t_L$$

$$A_{\substack{net\\shear}} = [15.0-(2.5)(1.8)]*0.7 = 7.35 \ Cm^2$$

$$A_{\substack{net\\tension}} = [m-0.5(\emptyset+0.2)]\,t_L$$

$$A_{\substack{net\\tension}} = [3.15-0.5(1.8)]*0.7 = 1.575 Cm^2$$

$$P = 0.4F_y\, A_{\substack{net\\shear}} + 0.725F_y\, A_{\substack{net\\tension}}$$

$$P = 0.4*2.4*7.35+0.725*2.4*1.575 = 9.79 \ ton$$

$$F_3 = 2P = 19.6 \ ton \qquad \boxed{F_3 = \underline{19.6t}}$$

d) **For bolts connecting gusset plate to the connecting angle**

$$R_{D.S} = 6.27t$$

$$R_b = 4.60t$$

$$n = \frac{F_4}{R_{least}} \qquad 3 = \frac{F_4}{4.60t} \qquad \boxed{F_4 = \underline{13.8t}}$$

For bolts connecting the connecting angle to the column

subjected to tension force

$T = 9.89$ [from tables]

- $\boxed{R_t = 0.6 * T = 0.6 * 9.89 = 5.93t}$

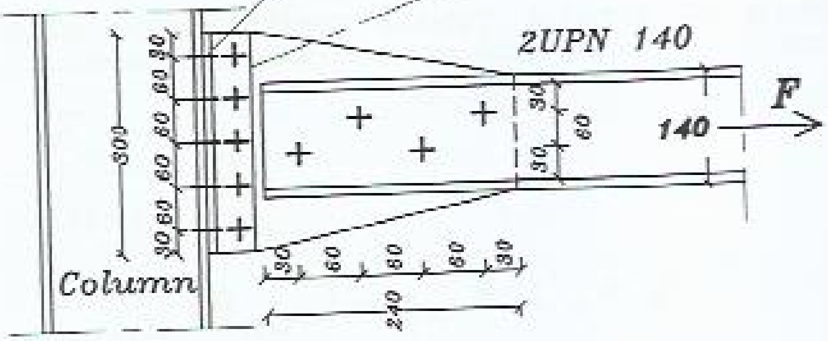
- $R_t = 5.93t$ $\boxed{n_{bolts} = \frac{F_5}{R_t}}$

$6 = \frac{F_5}{5.93}$ $\boxed{F_5 = \underline{35.6t}}$

$\therefore \boxed{F_{max} = \underline{13.8t}}$

Example

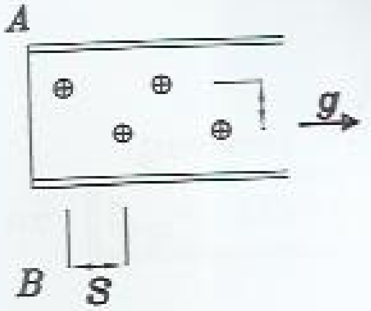
For the shown connection **it is required to find** *the maximum Force (F)*



use M16 bolt grade 4.6 **bearing type**, *threads are* **excluded** *from plane of shear*

---

a) For 2UPN 140



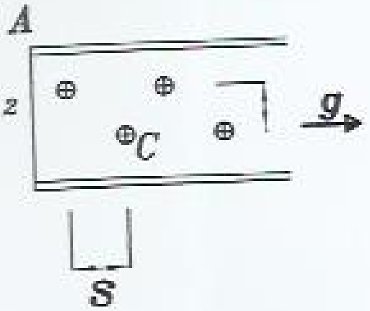
- $f_{act} = \dfrac{force}{A_{net}} \leq 1.4 \ t/cm^2$

where:

Sec AB → 1 bolt → راسى

$$A_{net(pathAB)} = 2\left[A_C - (\phi + 2)*t_w\right]$$

$$=2[20.4-(1.6+0.2)x0.7] = \mathbf{38.28} \ cm^2$$



Sec AC → 2 bolt → مائل

$$A_{net(pathAC)} = 2\left[A_C - 2(\phi + 2)*t + \frac{S^2 * t_w}{4g}\right]$$

$$=2[20.4-2(1.6+0.2)x0.7+ \frac{6^2*0.7}{4*6}] = \boxed{37.86 \ cm^2}$$ **govern**

$$F_1 = 37.86 * 1.4 = 53.004t$$

$$\boxed{F_1 = \underline{53.004t}}$$

b) For bolts connecting lower chord to gusset plate

$$n = \frac{F_2}{R_{least}}$$

$$R_{double\ shear}(R_{DS}) = q_b * A_s * 2$$

$$R_{D.S} = (0.25F_{ub} = 0.25*4)\left(\frac{\pi * 1.6^2}{4}\right)(2) = 4.02t$$

$$R_b = F_b * d * \Sigma t_{min.} = \ldots.. \ ton$$

$$R_b = (0.8*3.6)(1.6)(1.0) = 4.60t$$

$$\Sigma t_{min.} : \text{min. of } \boxed{2t_w = 0.7*2 = 1.4Cm} \quad \text{or} \quad \underset{}{\overset{\text{govern}}{\boxed{t_{g.p} = 1Cm}}}$$

$$4 = \frac{F_2}{4.02} \qquad \boxed{F_2 = \underline{16.08t}}$$

For bolts connecting gusset plate to the connecting angle

$R_{D.S} = 4.02t$

$R_b = 4.60t$

$$n = \frac{F_3}{R_{least}} \qquad 5 = \frac{F_3}{4.02t} \qquad \boxed{F_3 = \underline{20.1t}}$$

d) For bolts connecting the connecting angle to the column

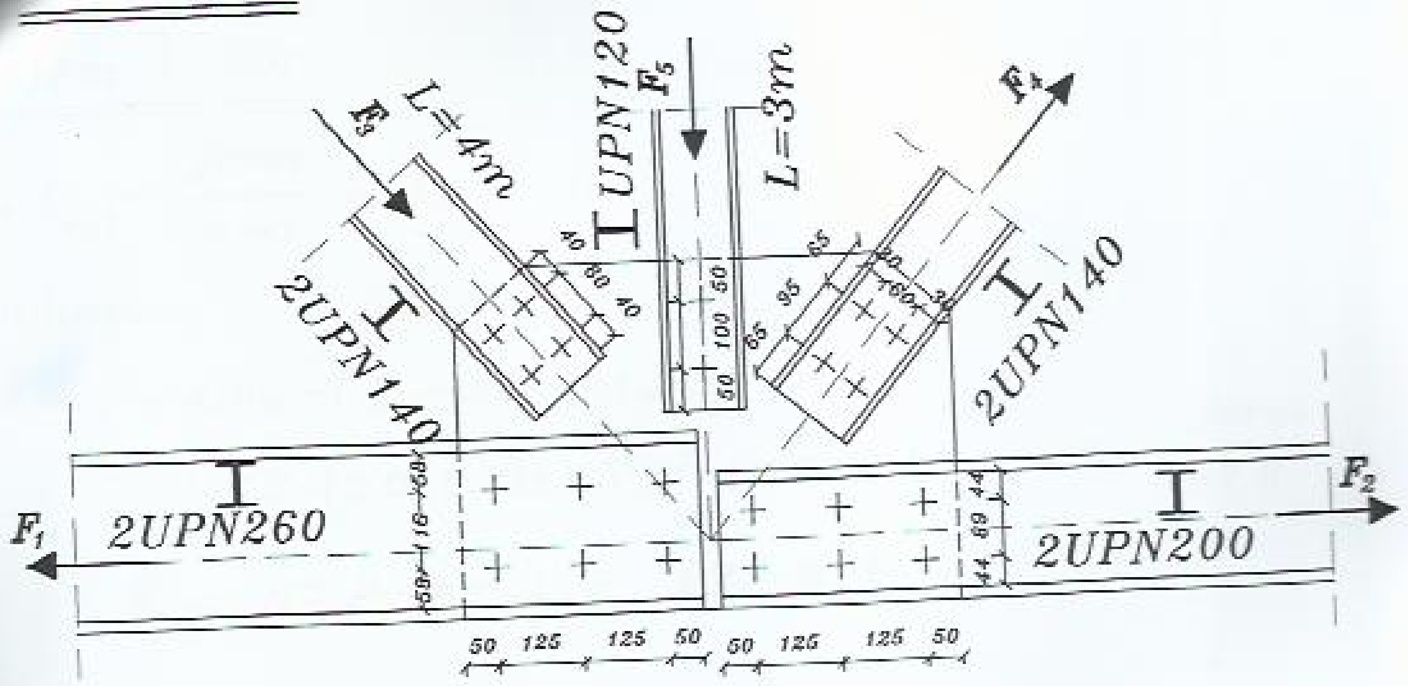
subjected to tension force

- $\boxed{R_t = A_s * F_{tb} = 0.78\left(\frac{\pi d^2}{4}\right)(0.33F_{ub})}$
- $R_t = 0.78\left(\frac{\pi 1.6^2}{4}\right) 0.33*4.00 = 2.07t$
- $R_t = 2.07t$ $\qquad \boxed{n_{bolts} = \frac{F_4}{R_t}}$

$$10 = \frac{F_4}{2.07} \qquad \boxed{F_4 = \underline{20.7t}}$$

$$\therefore \boxed{F_{max} = \underline{16.08t}}$$

Example



For the shown connection **it is required to** find the maximum Force (F_1, F_2, F_3, F_4, F_5)

use M20 bolt grade 8.8 **bearing type**, threads are **excluded** from plane of shear
gusset plate 15mm thick

يجب الملاحظة انه فى هذا المثال اعطى نوع القوى المطلوبة (شد ام ضغط) من خلال اتجاه القوى فى العنصر

$F_3 = -ve$ [compression]

$F_5 = -ve$ [compression]

ونلاحظ انه اعطى طول العنصر فى هذه الحالة وذلك لحساب مقاومة الضغط

For member 1

a) For $][$ 260

- $f_{act} = \dfrac{force}{A_{net}} \leqslant 1.4 \ t/cm^2$

where:

من الجدول

$A_{net\perp} = 2[A_{g[} - (\phi + 0.2^{cm}) x t_w \times 2]$

$= 2[48.3 - (2.0+0.2)x1.0*2] = 87.8 \ cm^2 \quad \therefore 1.4 = \dfrac{force}{87.8}$

$$F_1 = 87.8 * 1.4 = 122.92t$$

$$\boxed{F_1 = 122.92t}$$

b) For bolts

$$n = \frac{F_1}{R_{least}}$$

$R_{double\ shear}(R_{DS}) = q_b * A_s * 2$

$R_{D.S} = (0.25F_{ub} = 0.25*8)\left(\frac{\pi * 2.0^2}{4}\right)(2) = 12.56t$

$R_b = F_b * d * \Sigma t_{min.} = \ldots.. \ ton$

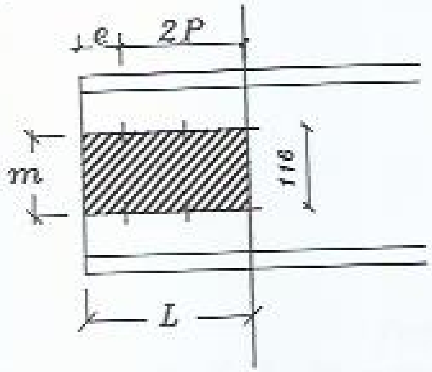
$R_b = (0.8*3.6)(2.0)(1.5) = 8.64t$

$\Sigma t_{min.}$: min. of $\boxed{2t_C = 1.0*2 = 2.0Cm}$ or $\overset{govern}{\boxed{t_{g.p} = 1.5Cm}}$

$6 = \dfrac{F_1}{8.64} \qquad \boxed{F_1 = 51.8t}$

3) block shear rupture

٣- يتم حساب اقصى مقاومه لل **block shear rupture**



$e = 5.0Cm$ **given**

$P_{Hoizontal} = 12.5Cm$ **given**

$m = P_{Vertical} = 11.6Cm$

$L = 5.0+(3-1)*12.5 = 30Cm$

$$A_{\substack{net\\shear}} = 2\left[L-(n-0.5)(\emptyset+0.2)\right]t_w$$

$$A_{\substack{net\\shear}} = 2\left[30.0-(2.5)(2.2)\right]*1.0 = \mathbf{49}\ Cm^2$$

$$A_{\substack{net\\tension}} = \left[m-1.0(\emptyset+0.2)\right]t_w$$

$$A_{\substack{net\\tension}} = \left[11.6-1.0(2.2)\right]*1.0 = \mathbf{9.40}Cm^2$$

$$P = 0.4F_y\, A_{\substack{net\\shear}} + 0.725F_y\, A_{\substack{net\\tension}}$$

$$P = 0.4*2.4*49+0.725*2.4*9.40 = \mathbf{63.39}\ ton$$

$F_1 = 2P = \mathbf{126.8}\ ton$ $\boxed{F_1 = \underline{126.8}}$

$\therefore \boxed{F_{1max} = \underline{51.8t}}$

~~or~~ member 2

a) For][200

- $f_{act} = \frac{force}{A_{net}} \leqslant 1.4\ t/cm^2$

where:

من الجداول

$A_{net_{\perp}} = 2[A_{g_{[}} - (\phi + 0.2^{cm}) x t_w \times 2]$

$= 2[32.2-(2.0+0.2)x0.85*2] = 56.9\ cm^2 \therefore 1.4 = \frac{force}{56.9}$

$F_2 = 56.9 * 1.4 = 79.68t$

$$\boxed{F_2 = \underline{79.68t}}$$

b) For bolts

$n = \frac{F_1}{R_{least}}$

$R_{double\ shear}(R_{DS}) = q_b * A_s * 2$

$R_{D.S} = (0.25F_{ub} = 0.25*8)\left(\frac{\pi * 2.0^2}{4}\right)(2) = 12.56t$

$R_b = F_b * d * \Sigma t_{min.} = ...\ ton$

$R_b = (0.8*3.6)(2.0)(1.5) = 8.64t$

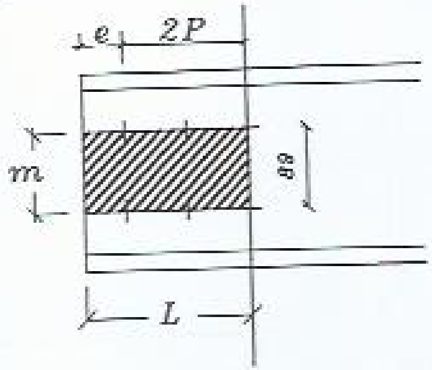
$\Sigma t_{min.}$: min. of $\boxed{2t_C = 0.85*2 = 1.70Cm}$ or $\boxed{\overset{govern}{t_{g.p} = 1.5Cm}}$

$6 = \frac{F_2}{8.64}$

$$\boxed{F_2 = \underline{51.8t}}$$

c) block shear rupture

٣- يتم حساب اقصى مقاومه لل **block shear rupture**



$e = 5.0Cm$ **given**

$P = 12.5Cm$ **given**

$m = P = 8.9Cm$

$L = 5.0+(3-1)*12.5 = 30Cm$

$$A_{\substack{net\\shear}} = 2\left[L-(n-0.5)(\emptyset+0.2)\right]t_w$$

$$A_{\substack{net\\shear}} = 2\left[30.0-(2.5)(2.2)\right]*1.0 = 49\ Cm^2$$

$$A_{\substack{net\\tension}} = \left[m-1.0(\emptyset+0.2)\right]t_w$$

$$A_{\substack{net\\tension}} = \left[8.90-1.0(2.2)\right]*1.0 = 6.70Cm^2$$

$$P = 0.4F_y\ A_{\substack{net\\shear}} + 0.725F_y\ A_{\substack{net\\tension}}$$

$$P = 0.4*2.4*49+0.725*2.4*6.70 = 58.69\ ton$$

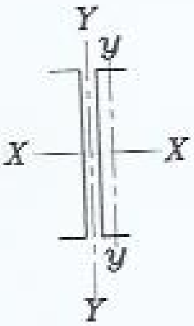
$$F_2 = 2P = 117.4\ ton \qquad \boxed{F_2 = \underline{117.4}}$$

$$\therefore \boxed{F_{2max} = \underline{51.8t}}$$

For member 3

a) For][140

- $l_{bin} = l_{bout} = 400 \ cm$



$$r_{x\,][} = 5.45 \ cm$$

$$r_{y\,][} = \sqrt{r_{x}^{2} + (0.75+e)^{2}} = \sqrt{5.45^{2} + (0.75+1.75)^{2}} = 5.99m$$

$$\lambda_{in} = \frac{400}{5.45} = 73.39 < 180 \ \text{———} \ \lambda_{max} = 73.39$$

$$\lambda_{out} = \frac{400}{5.99} = 66.77 < 180$$

$$F_C = [1.4 - 6.5X10^{-5} (73.39)^{2}] = 1.05 \ t/cm^{2}$$

- $F_C = \dfrac{force}{2 A_g} = 1.05 \ t/Cm^{2}$

$$F_3 = 2*20.40*1.05 = 42.84t$$

$$\boxed{F_3 = \underline{42.84t}}$$

) **For bolts** $n = \dfrac{F_3}{R_{least}}$

$$R_{double\ shear}(R_{DS}) = q_b * A_s * 2$$

$$R_{D.S} = (0.25F_{ub} = 0.25*8)\left(\frac{\pi * 2.0^2}{4}\right)(2) = 12.56t$$

$$R_b = F_b * d * \Sigma t_{min.} = \ldots\ldots \ ton$$

$$R_b = (0.8*3.6)(2.0)(1.4) = \underset{govern}{8.064t}$$

$\Sigma t_{min.}$: min. of $\boxed{2t_C = 0.7*2 = 1.4Cm}$ or $\boxed{t_{g.p} = 1.5Cm}$

$$4 = \frac{F_3}{8.064} \qquad \boxed{F_3 = \underline{32.2t}}$$

$$\therefore \boxed{F_{3max} = \underline{32.2t}}$$

No $\boxed{B.S.R}$ For Compression member

For member 4

a) For][140

- $f_{act} = \dfrac{force}{A_{net}} \leqslant 1.4\ t/cm^2$

where:

من الجدول

$A_{net_{][}} = 2[A_{g_{[}} - (\phi + 0.2^{cm}) x t_w \times 2]$

$= 2[20.4 - (2.0+0.2) x 0.70 * 2] = 34.6\ cm^2 \therefore 1.4 = \dfrac{force}{34.6}$

$F_4 = 34.6 * 1.4 = 48.44t$

$\boxed{F_4 = \underline{48.44t}}$

b) For bolts

$n = \dfrac{F_4}{R_{least}}$

$R_{double\ shear} (R_{DS}) = q_b * A_s * 2$

$R_{D.S} = (0.25 F_{ub} = 0.25 * 8) \left(\dfrac{\pi * 2.0^2}{4}\right) (2) = 12.56t$

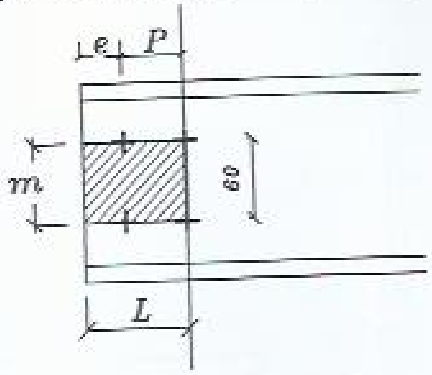
$R_b = F_b * d * \Sigma t_{min.} = \ldots\ldots\ ton$

$R_b = (0.8 * 3.6)(2.0)(1.4) = 8.064t$

$\Sigma t_{min.}$: min. of $\underset{govern}{\boxed{2t_C = 0.7*2 = 1.4Cm}}$ or $\boxed{t_{g.p} = 1.5Cm}$

$4 = \dfrac{F_3}{8.064}$ $\boxed{F_3 = \underline{32.2t}}$

٣- يتم حساب اقصى مقاومه لل **block shear rupture**



$e = 6.5Cm$ **given**

$P = 9.5Cm$ **given**

$m = P = 6.0Cm$

$L = 6.5+(2-1)*9.5 = 16Cm$

$$A_{\substack{net\\shear}} = 2\left[L-(n-0.5)(\emptyset+0.2)\right]t_w$$

$$A_{\substack{net\\shear}} = 2\left[16.0-(1.5)(2.2)\right]*0.70 = 17.8\ Cm^2$$

$$A_{\substack{net\\tension}} = \left[m-1.0(\emptyset+0.2)\right]t_w$$

$$A_{\substack{net\\tension}} = \left[6.00-1.0(2.2)\right]*0.70 = 2.66Cm^2$$

$$P = 0.4F_y\ A_{\substack{net\\shear}} + 0.725F_y\ A_{\substack{net\\tension}}$$

$$P = 0.4*2.4*17.8+0.725*2.4*2.66 = 21.71\ ton$$

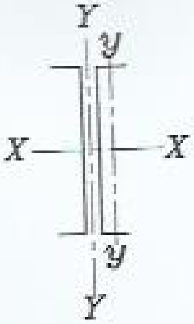
$$F_4 = 2P = 43.43\ ton$$

$$\boxed{F_4 = \underline{43.43}}$$

$$\therefore \boxed{F_{4max} = \underline{32.2t}}$$

member 5

a) For][120



- $l_{bin} = l_{bout} = 300 \ cm$

$$r_{x][} = 4.62 \ cm$$

$$r_{y][} = \sqrt{r_{x\lrcorner}^2 + (0.75+e)^2} = \sqrt{4.62^2 + (0.75+1.60)^2} = 5.18m$$

$$\lambda_{in} = \frac{300}{4.62} = 64.93 < 180 \text{ ——— } \lambda_{max} = 64.93$$

$$\lambda_{out} = \frac{300}{5.18} = 57.91 < 180$$

$$F_C = [1.4 - 6.5X10^{-5}(64.93)^2] = 1.12 \ t/cm^2$$

- $F_C = \dfrac{force}{2A_g} = 1.12 \ t/Cm^2$

$$F_5 = 2*17.00*1.12 = 38.08t$$

$$\boxed{F_5 = \underline{38.08t}}$$

5) For bolts $n = \frac{F_5}{R_{least}}$

$$R_{double\ shear}(R_{DS}) = q_b * A_s * 2$$

$$R_{D.S} = (0.25F_{ub} = 0.25*8)\left(\frac{\pi * 2.0^2}{4}\right)(2) = 12.56t$$

$$R_b = F_b * d * \Sigma t_{min.} = \ldots\ ton$$

$$R_b = (0.8*3.6)(2.0)(1.4) = 8.064t$$

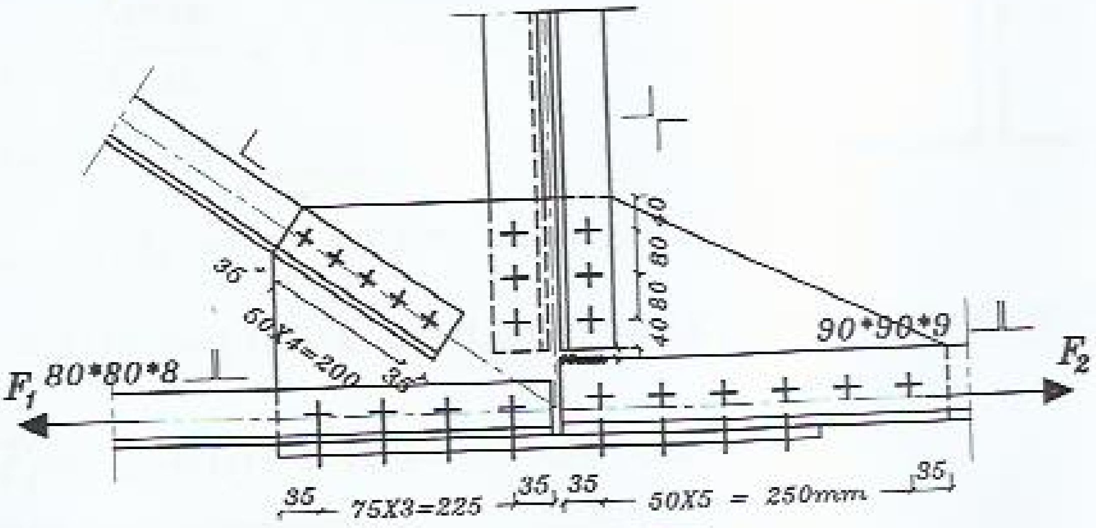
$$\Sigma t_{min.} : min.\ of \ \underbrace{\boxed{2t_C = 0.7*2 = 1.4Cm}}^{govern} \ or \ \boxed{t_{g.p} = 1.5Cm}$$

$$4 = \frac{F_3}{8.064} \qquad \boxed{F_3 = \underline{32.2t}}$$

$$\therefore \boxed{F_{5max} = \underline{32.2t}}$$

No $\boxed{B.S.R}$ For Compression member

Example



Elevation

For the shown Connection, **it is required to:-**

1-Calculate the maximum force F1, F2 in the lower chord

using high strength pretensioned bolts M16 grade 10.9

a) For $\rfloor\lfloor$ 90*90*9

- $f_{act} = \dfrac{force}{A_{net}} \leqslant 1.4 \ t/cm^2$

where:

من الجدول

$$A_{net_{\rfloor\lfloor}} = 2[A_{g_{\lfloor}} - (\phi + 0.2^{cm}) x t_{\lfloor} \times 2]$$

$$= 2[15.5 - (1.6+0.2) x 0.9 x 2] = 24.6 cm^2 \quad \therefore 1.4 = \frac{force}{24.66}$$

$$F_1 = 24.66 * 1.4 = 34.524t$$

$$\boxed{F_1 = \underline{34.524t}}$$

b) For bolts $\quad F_1 = P_S * n_{sp} + 2P_S * n_{chord1}$

$\boxed{P_S = \underline{3.16t}}$ From tables

$$F_1 = 3.16 * 8 + 2 * 3.16 * 6 = 63.2t$$

$$\boxed{F_1 = \underline{63.2t}}$$

$$\therefore \boxed{F_{1max} = \underline{34.5t}}$$

a) For $\rfloor\lfloor 80*80*8$

- $f_{act} = \dfrac{force}{A_{net}} \leqslant 1.4 \ t/cm^2$

where:

من الجدول

$$A_{net_{\rfloor\lfloor}} = 2[A_{g_{\lfloor}} - (\phi + 0.2^{cm}) x t_{\lfloor} \times 2]$$

$$= 2[12.3 - (1.6 + 0.2) x 0.8 x 2] = 18.8 cm^2 \quad \therefore 1.4 = \frac{force}{18.84}$$

$$F_2 = 18.84 * 1.4 = 26.376t$$

$$\boxed{F_2 = \underline{26.376t}}$$

b) For bolts $\quad F_2 = P_S * n_{sp} + 2P_S * n_{chord2}$

$$\boxed{P_S = \underline{3.16t}}$$ *From tables*

$$F_2 = 3.16 * 8 + 2 * 3.16 * 4 = 50.56t$$

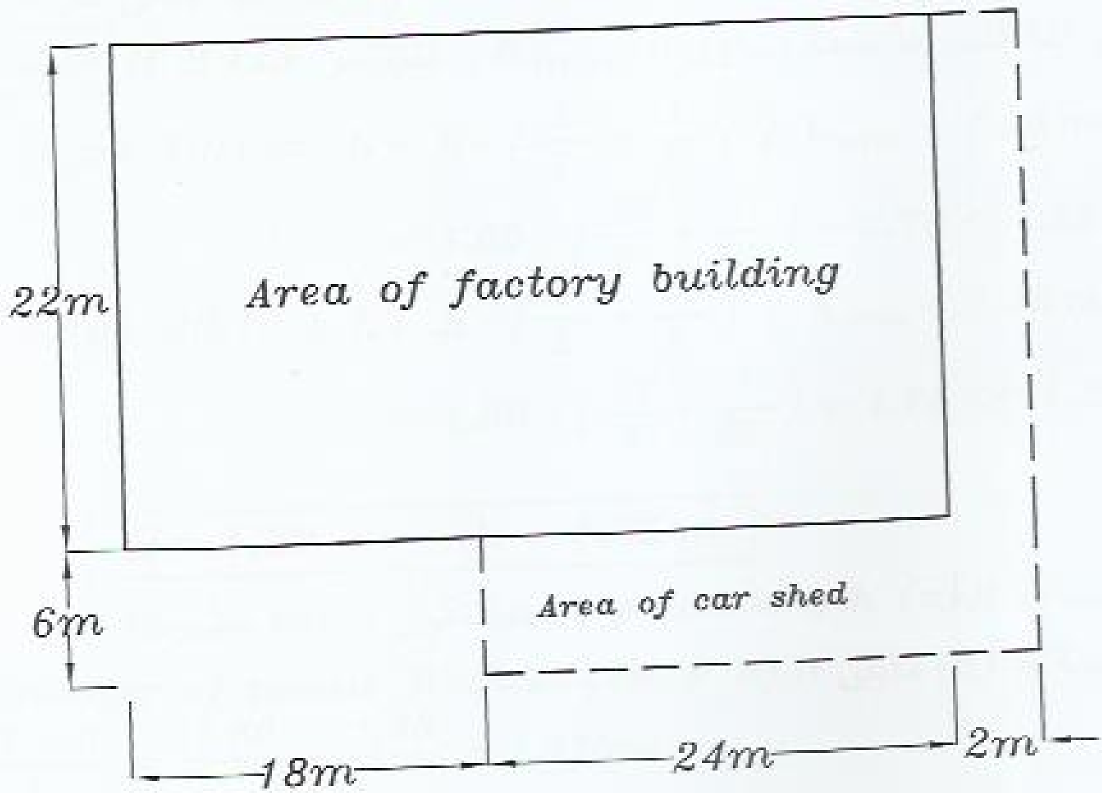
$$\boxed{F_2 = \underline{50.56t}}$$

$$\therefore \boxed{F_{2max} = \underline{26.3t}}$$

Example (3)

A factory building is to be constructed over a rectangular area (22m x 42m), and a car shed of area.
The main system is made up of steel trusses. Steel columns are provided along solid lines only, clear height = 6m, the covering material used is galvanized steel sheets.

It is required to draw to scale 1:100 or 1:200 a complete general layout showing all structural components.
(Main trusses, purlins and all bracing systems).



خطوات رسم الـ Layout

خد بالك

الـ span هى المسافة بين الاعمدة أى أنها هنا تساوى 22 m

١- تحديد نوع الـ Truss .

نبدأ بتجربة الـ N-Truss or K-Truss

٢- نحسب الـ depth للـ Truss .

$$H = \frac{Span\ (B)}{12 \Rightarrow 16} = \frac{22}{12 \Rightarrow 16} = 1.38 \Rightarrow 1.83 m$$

$$\boxed{Take\ \ \mathbf{H = 1.80\ \ m}}$$

٣- نحسب الـ h الموجودة فى نهاية الـ Truss ونتأكد أنها لاتقل عن الـ $h_{min.}$ و اذا قلت نأخذها تساوى الـ $h_{min.}$ و نغير من قيمة الـ H $\boxed{h_{min.} = 1.25 m}$

$$* \ Take\ \ slope\ \ 10:1 \Rightarrow \mathbf{h} = \mathbf{H} - [\frac{B}{2} * \frac{1}{Z}] \nless \mathbf{h}_{min.} = 1.25 m$$

$$= \mathbf{1.80} - [\frac{22}{2} * \frac{1}{10}] = 0.70 < 1.25 m$$

$$* \ Take\ \ slope\ \ 20:1 \Rightarrow \mathbf{h} = \mathbf{H} - [\frac{21}{2} * \frac{1}{Z}] \nless \mathbf{h}_{min.} = 1.25 m$$

$$= \mathbf{1.80} - [\frac{22}{2} * \frac{1}{20}] = 1.25 \nless 1.25 m$$

$$\boxed{\mathbf{H = 1.80\ \ m \qquad h = 1.25\ m}}$$

٤- حساب الـ panel length (a) و حتى تكون الزوايا مظبوطة يفضل ان تكون قيمة الـ (a) مابين الـ (H & h) و حساب الـ Number of panels .

$$\mathbf{a} \simeq \frac{H + h}{2} = \frac{1.80 \ \ + 1.25}{2} = 1.525 m$$

$$Number\ \ of\ \ panels = \frac{22}{1.525} = 14.4 \qquad \boxed{Take\ \mathbf{12}\ panels}$$ رقم زوجى

$$\mathbf{a}_{\ actual} = \frac{22}{12} = 1.833 m \qquad \boxed{\mathbf{a}_{\ actual} = \mathbf{1.833}\ m}$$

$$\Rightarrow Check\ \ tan^{-1}\frac{h}{a} = \frac{1.25}{1.83} = \mathbf{34.3^{\circ}}$$

$$\&\ \ \ tan^{-1}\frac{H}{a} = \frac{1.80}{1.83} = \mathbf{44.5^{\circ}}$$

و بالتالى من الممكن هنا استخدام الـ N-Truss or K-Truss

٥- تحديد عدد الـ *longitudinal bracing* و أماكنهم بحيث لا تزيد المسافة بينهم عن ٨م و يفضل أن تكون أماكن الـ *end gable columns* هى نفسها أماكن الـ *longitudinal bracing* .

$16 < span = 22 < 24 \Rightarrow$ $\boxed{\textbf{Use 4 longitudinal bracing}}$

و من الممكن استخدام اثنان فقط.

٦- تحديد عدد الـ *spacings* فى الاتجاه الطويل .

$$Number\ of\ spacings = \frac{36}{5 \Rightarrow 8} = 7.2 \Rightarrow 4.5$$

$$take\ 6\ spacings \Rightarrow Spacing = \frac{36}{6} = 6\ m$$

$$\boxed{\textbf{Number of spacings} = 6\ \&\ \textbf{Spacing} = 6\ m}$$

٧- اذا كان الـ *clear height* أكبر من *6m* سنحتاج الى اضافة *member* أفقى زيادة للعمود فى الـ *vertical bracing* لتقليل الـ *Buckling* .

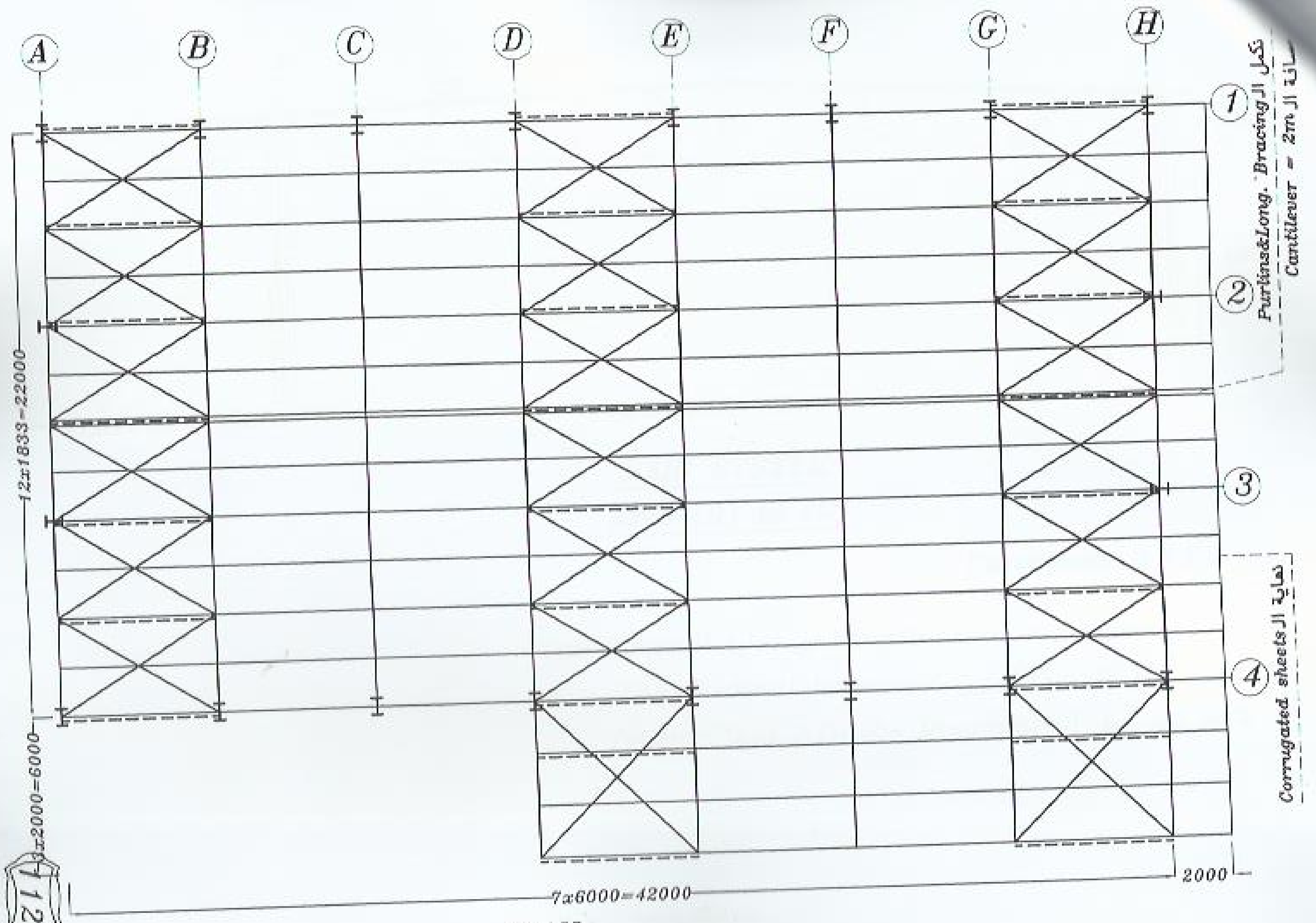
$Clear\ height = 6m$

$\Rightarrow$ $\boxed{\textbf{No need for add. member in vertical bracing}}$

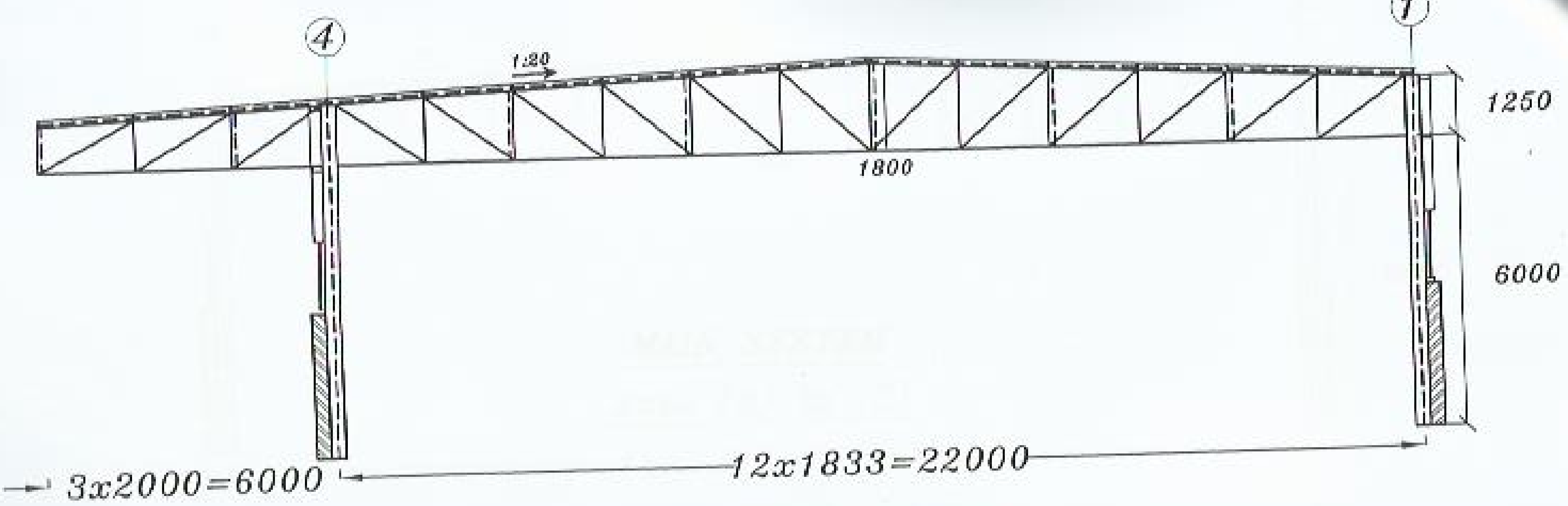
٨- اذا كانت المسافة بين أول و أخر *Horizontal bracing* أكبر من $25 \Rightarrow 30m$ نضيف *Horizontal bracing* بينهم .

$Distance\ between\ Horizontal\ bracings = 24m$

$\Rightarrow$ $\boxed{\textbf{No need for additional bracing}}$


A
B
C
D
E
F
G
H
1
2
3
4
Purlins&Long. Bracing ال كل
Cantilever = 2m ال ان
Corrugated sheets ال نهاية
12x1833=22000
3x2000=6000
7x6000=42000
2000

PLAN



MAIN SYSTEM

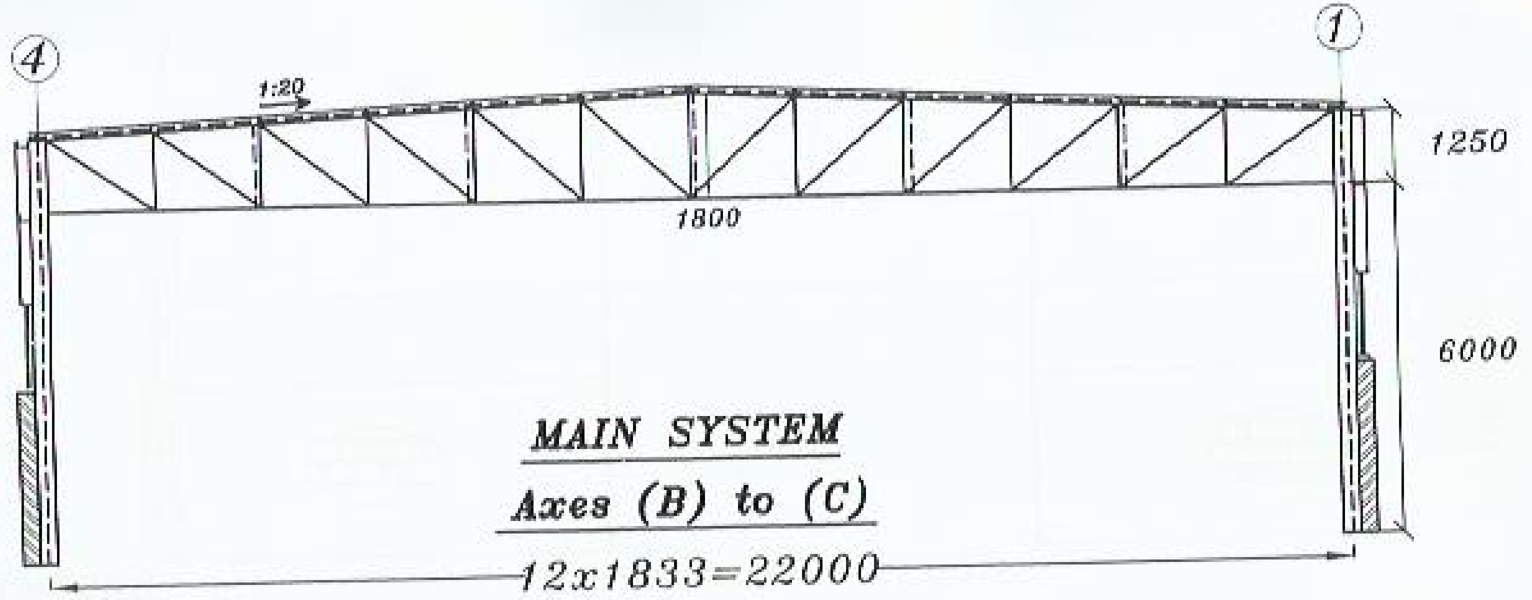
Axes (D) to (G)

فى حالة ال Cantilever

١- يجب وضع Longitudinal bracing فى نهاية ال Cantilever .

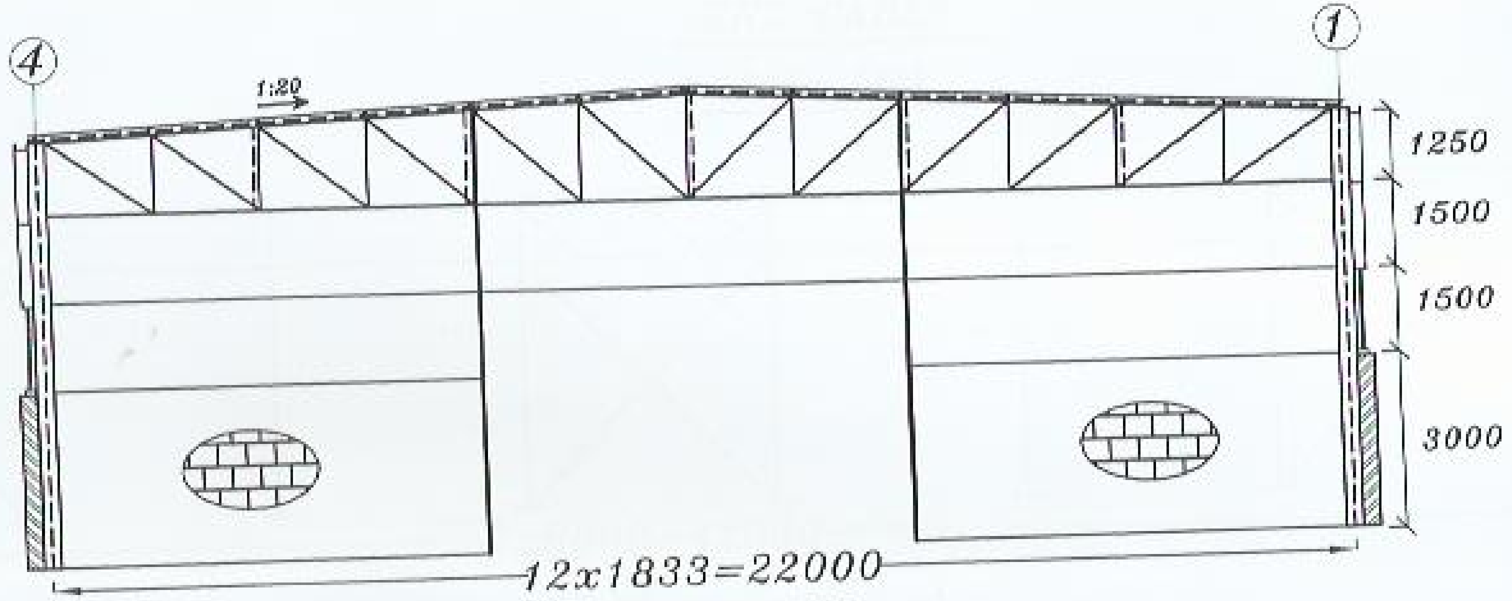
٢- يفضل تقليل المسافات بين ال Longitudinal bracing .

٣- لاحظ تغير ميل ال Diagonals حتى يكون عليها Tension .



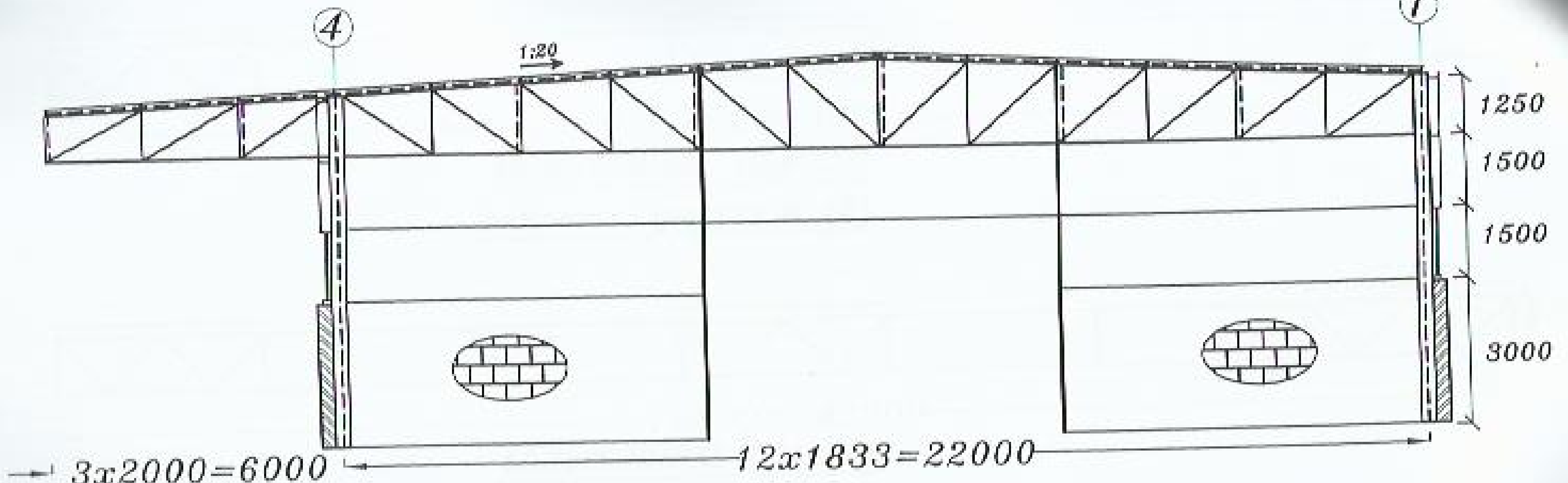
MAIN SYSTEM

Axes (B) to (C)



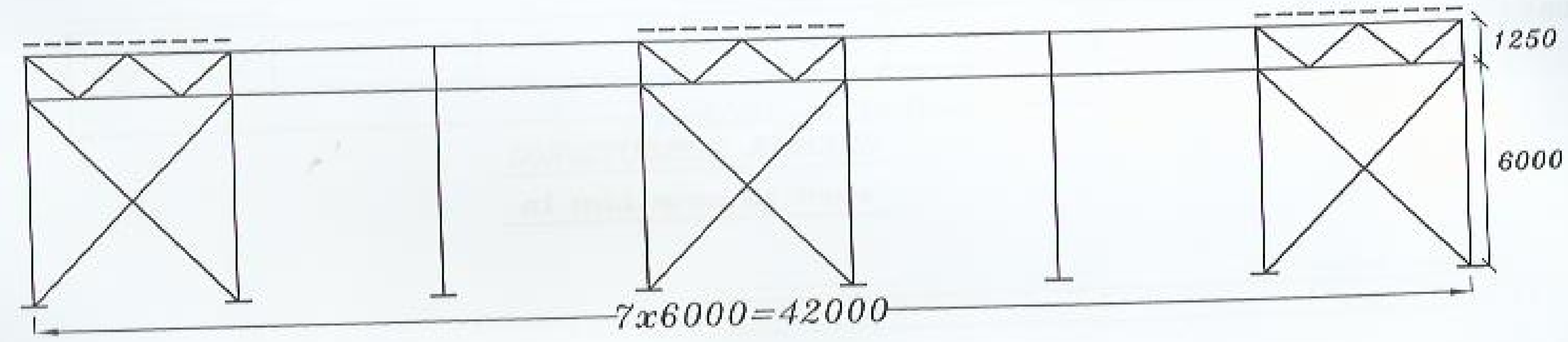
END GABLE

Axis (A)



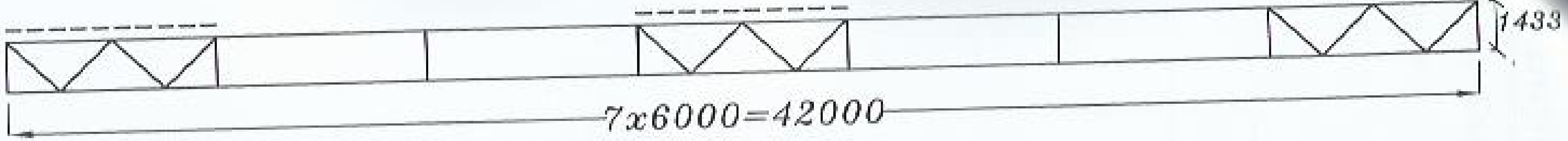
END GABLE

Axis (H)



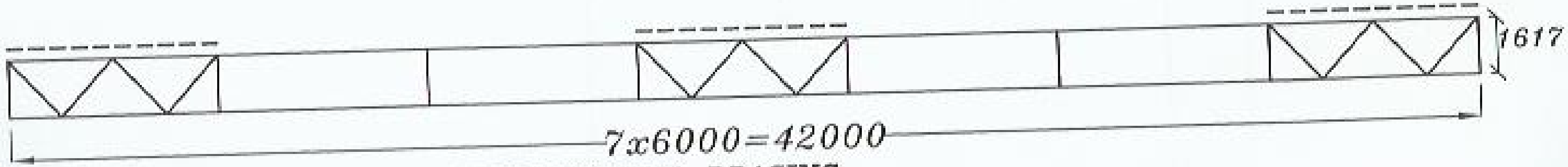
VERTICAL BRACING

Axes (1) & (4)



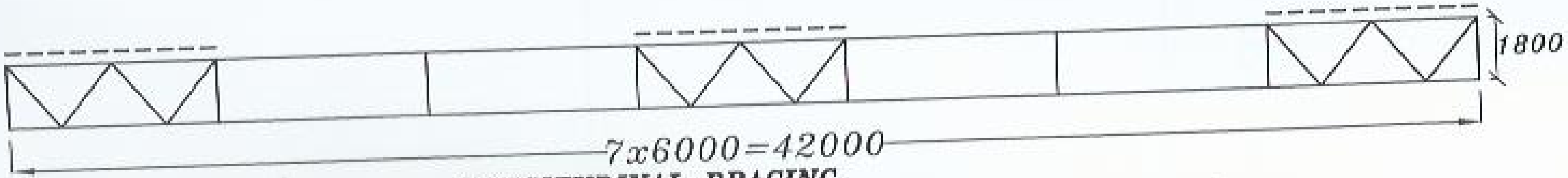
LONGITUDINAL BRACING

between Axes (1) & (2)



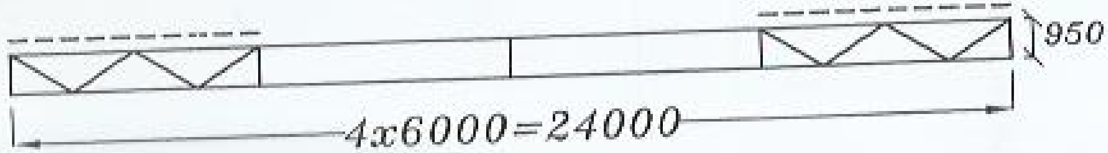
LONGITUDINAL BRACING

Axes (2) & (3)



LONGITUDINAL BRACING

at mid span of truss

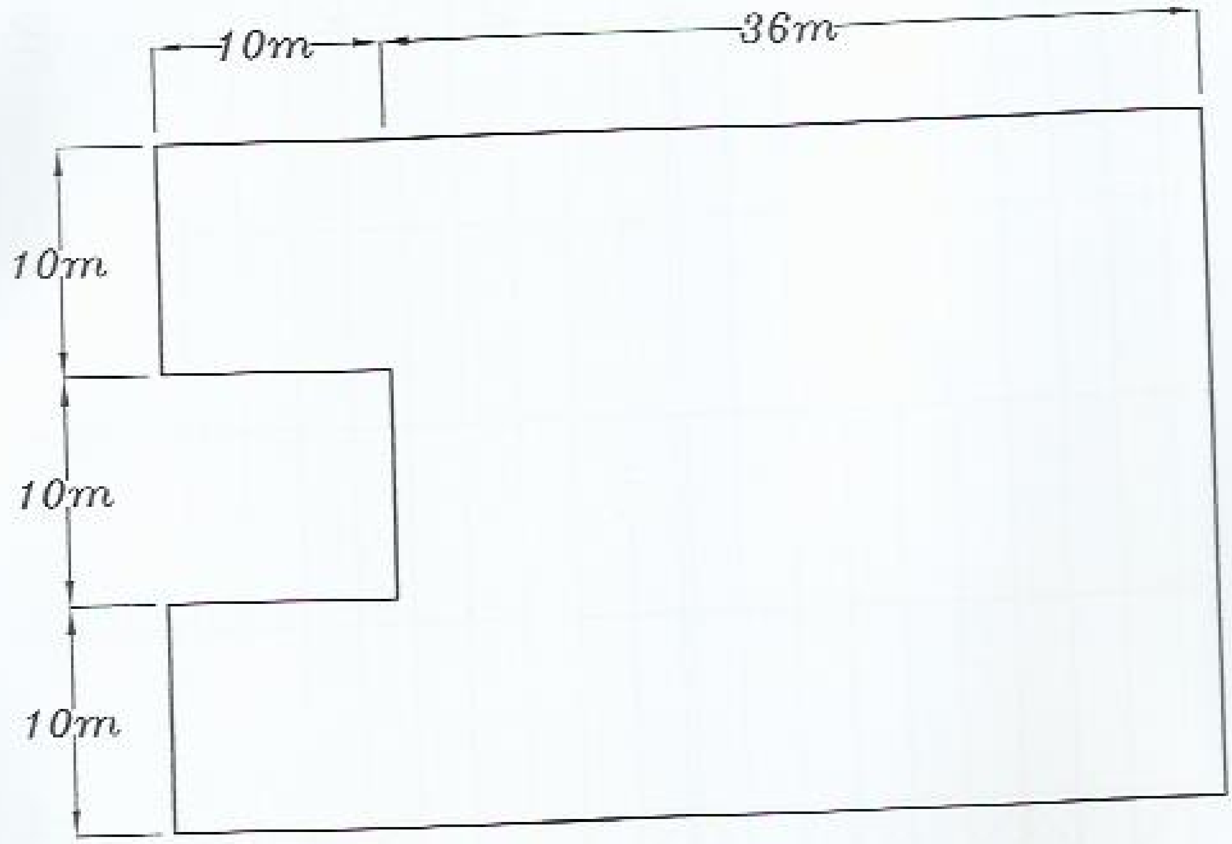


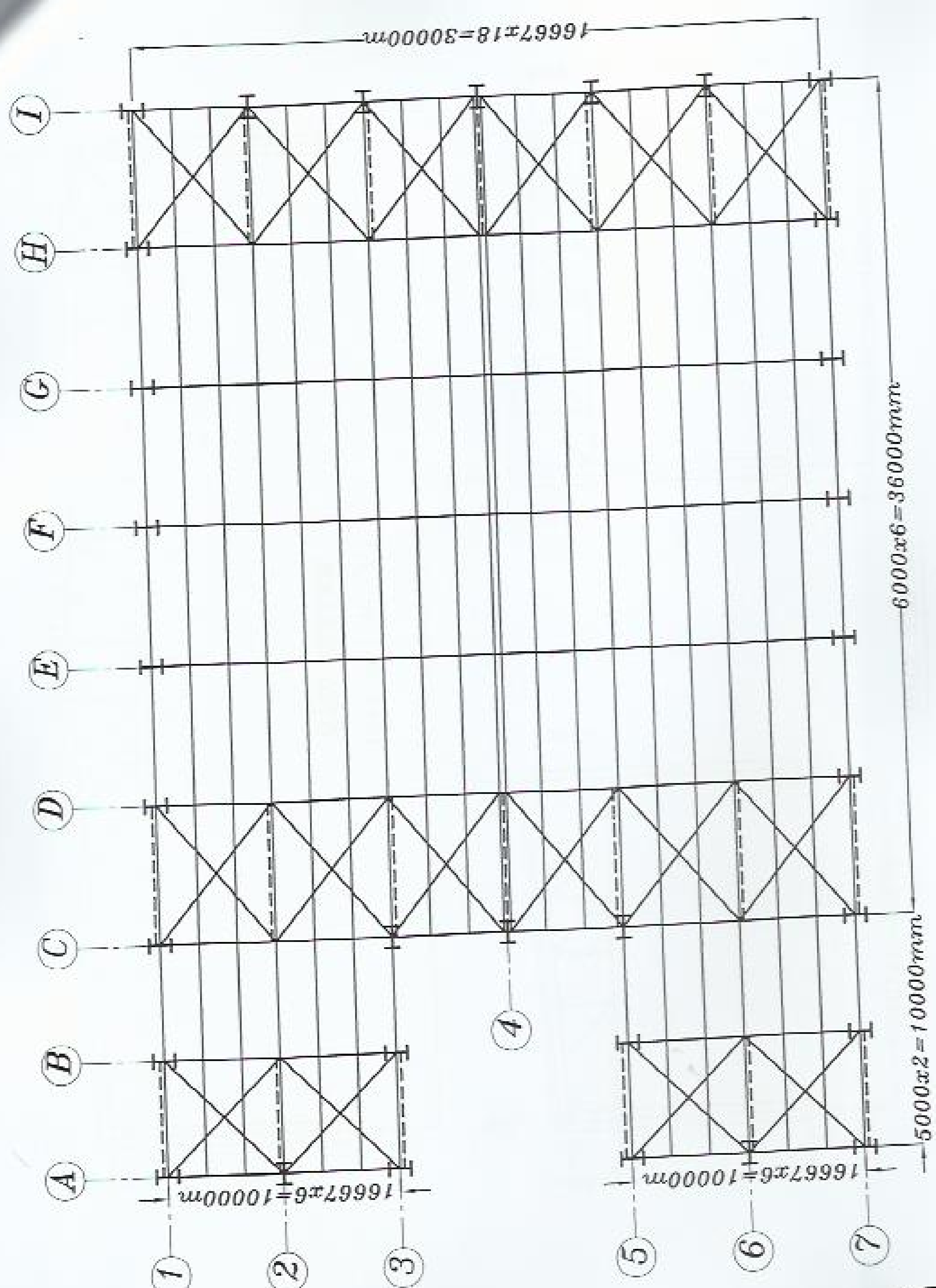
LONGITUDINAL BRACING

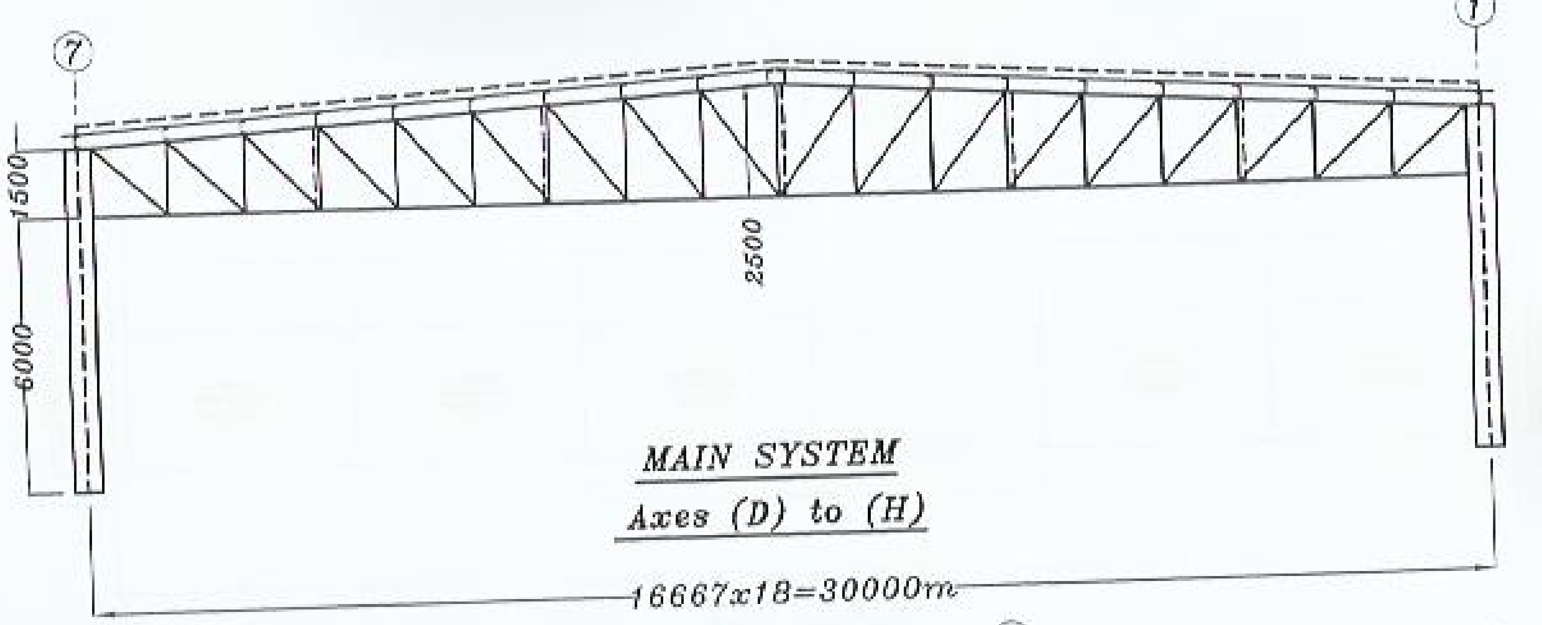
at the end of cantilever

Example (3)

Draw General Layout for the shown land with clear height equals 6m.

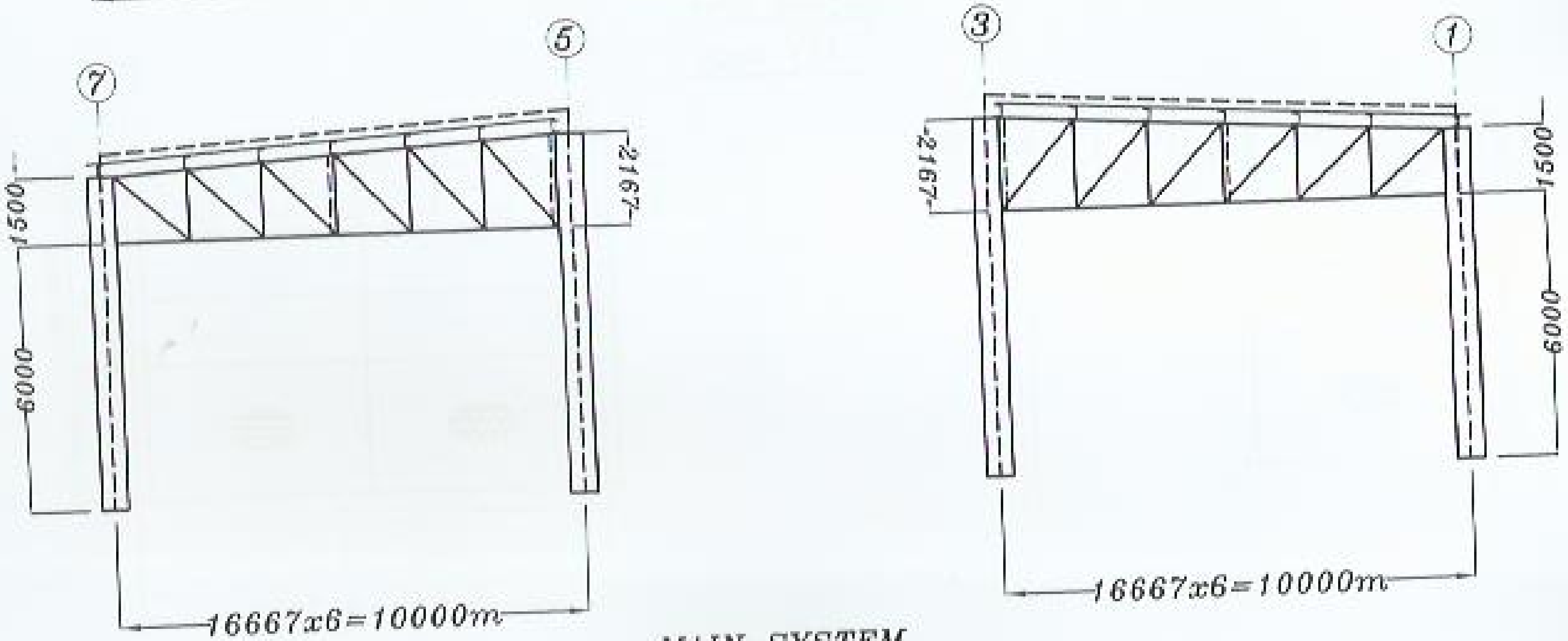



16667x18=30000mm
6000x6=36000mm
5000x2=10000mm
16667x6=10000mm
16667x6=10000mm
A
B
C
D
E
F
G
H
I
1
2
3
4
5
6
7



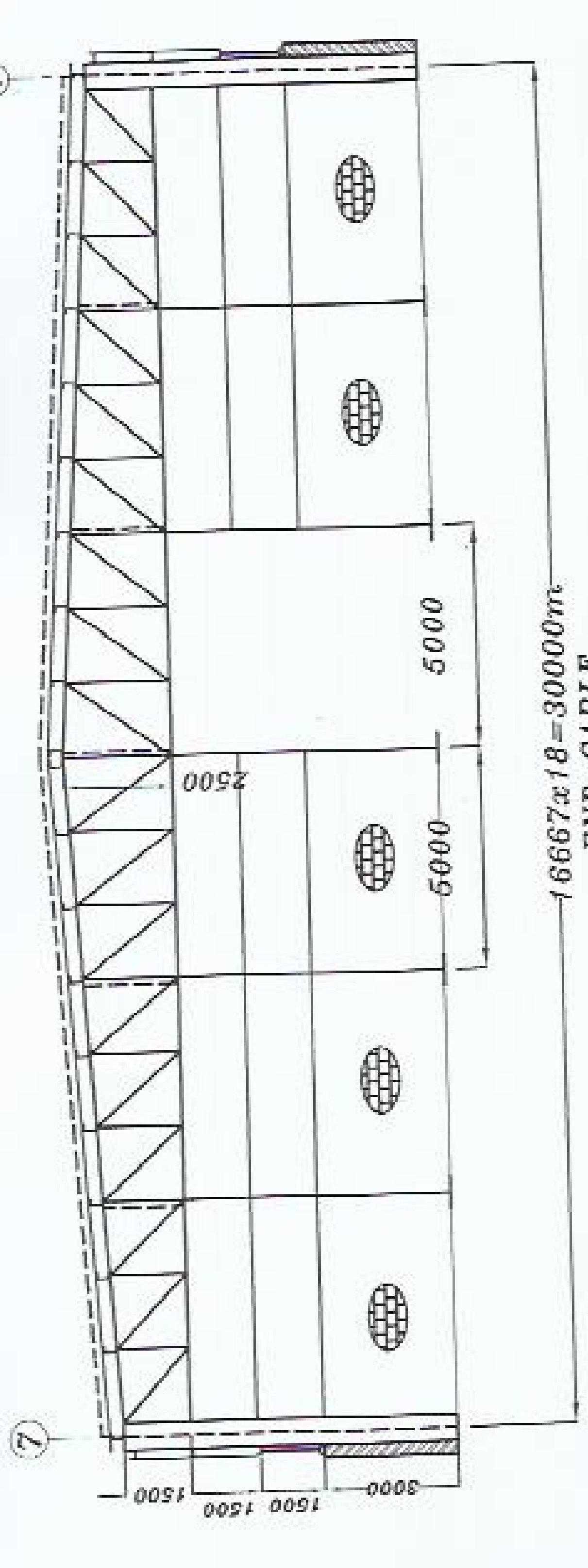
MAIN SYSTEM

Axes (D) to (H)



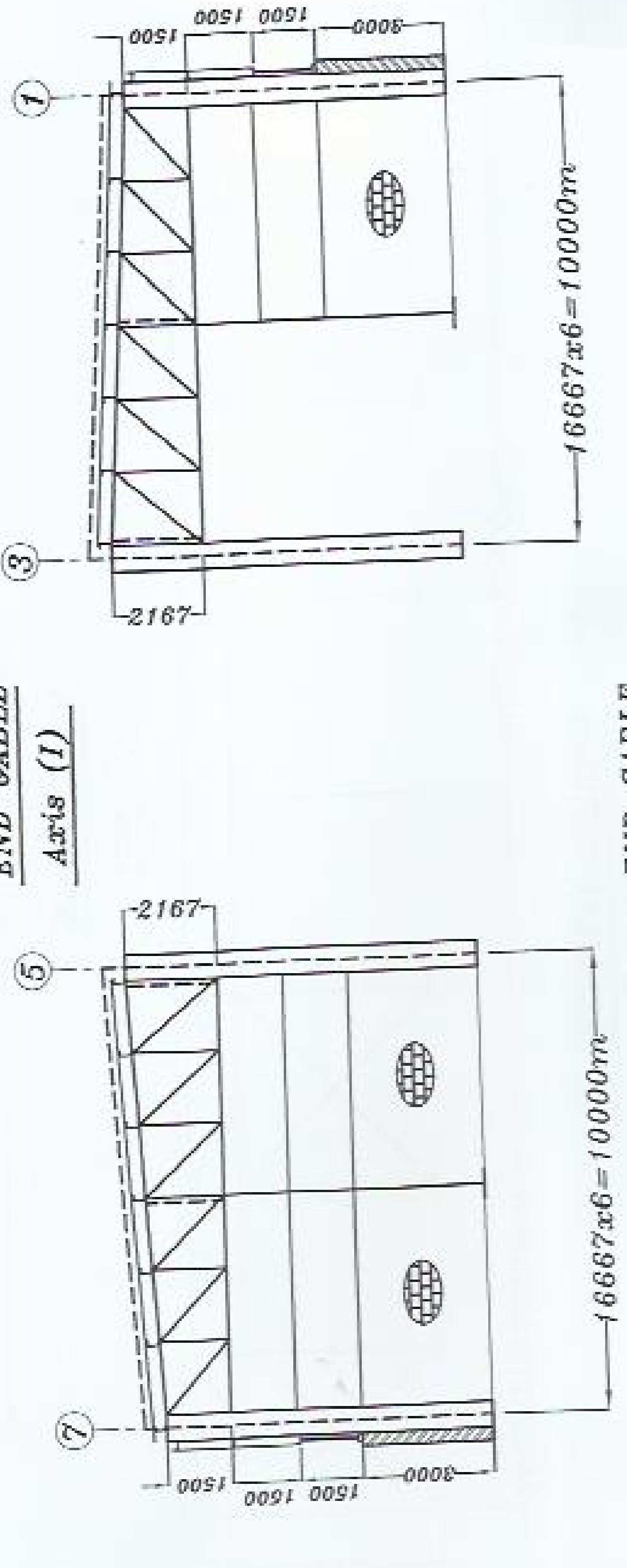
MAIN SYSTEM

Axis (B)



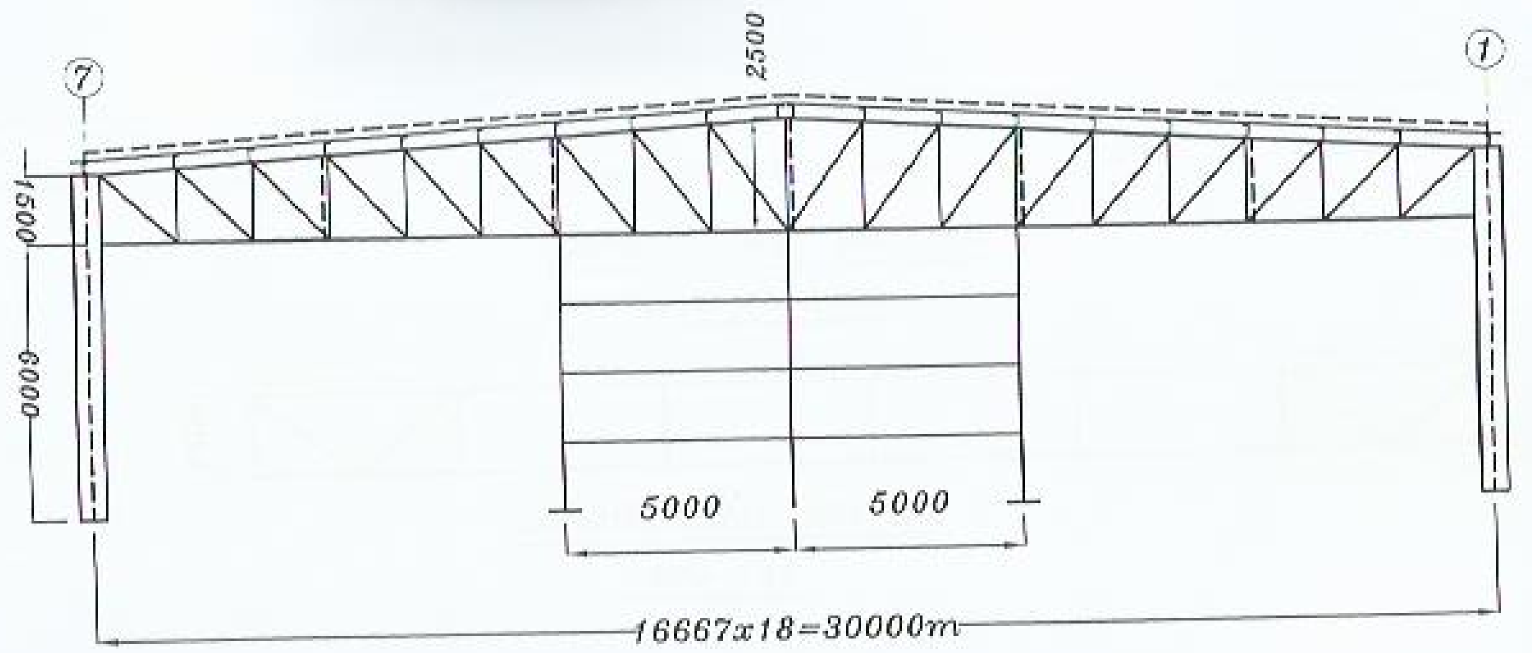
END GABLE

Axis (1)



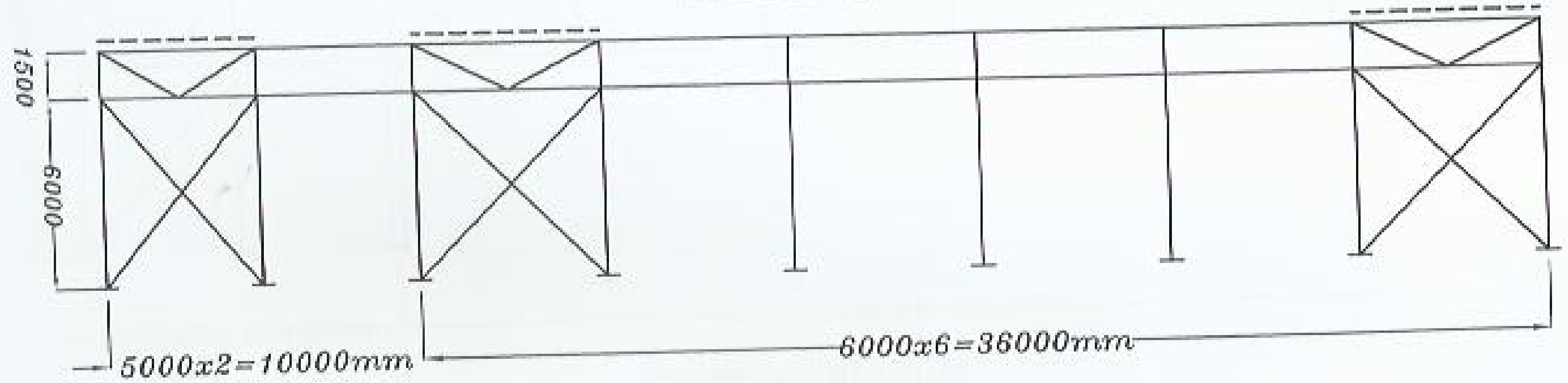
END GABLE

Axis (A)



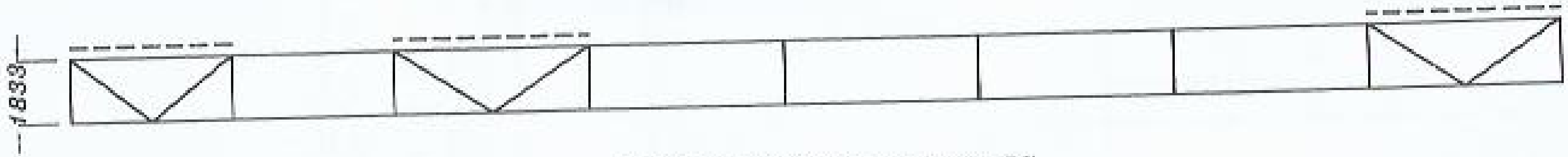
MAIN SYSTEM

Axis (C)



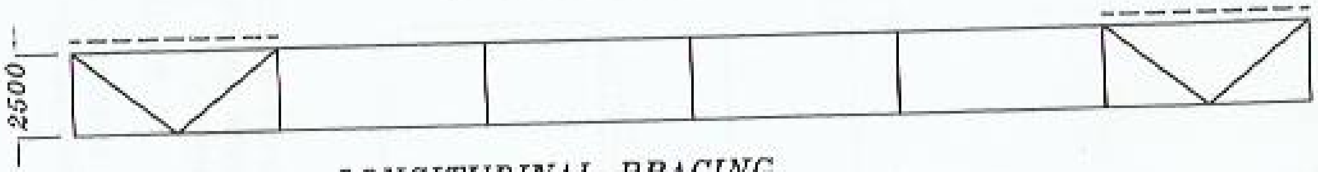
VERTICAL BRACING

Axis (1) & (7)



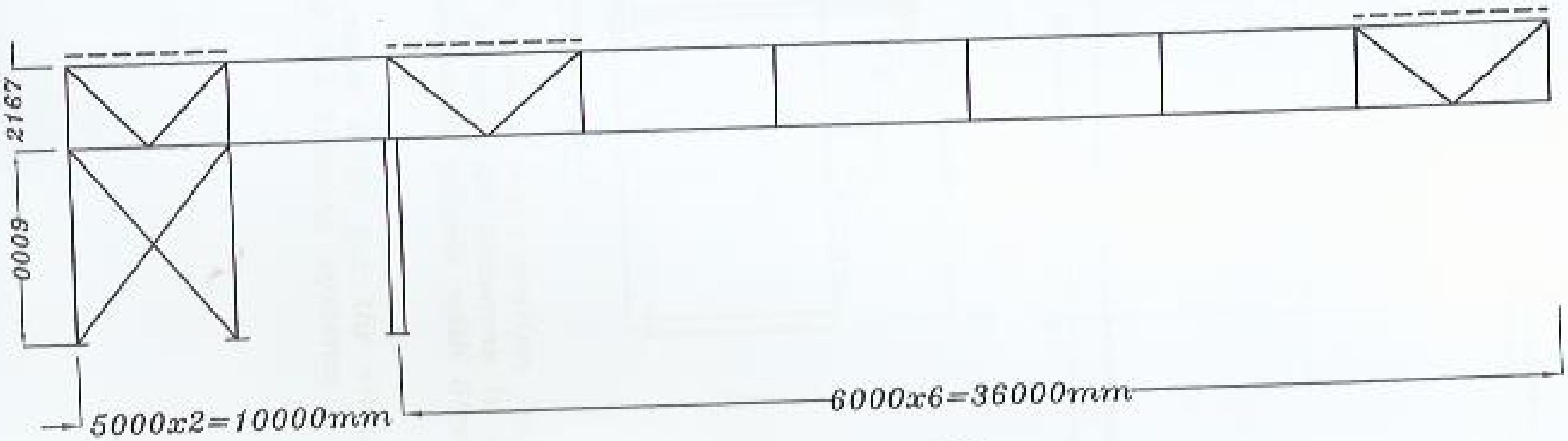
LONGITUDINAL BRACING

Axis (2)&(6)



LONGITUDINAL BRACING

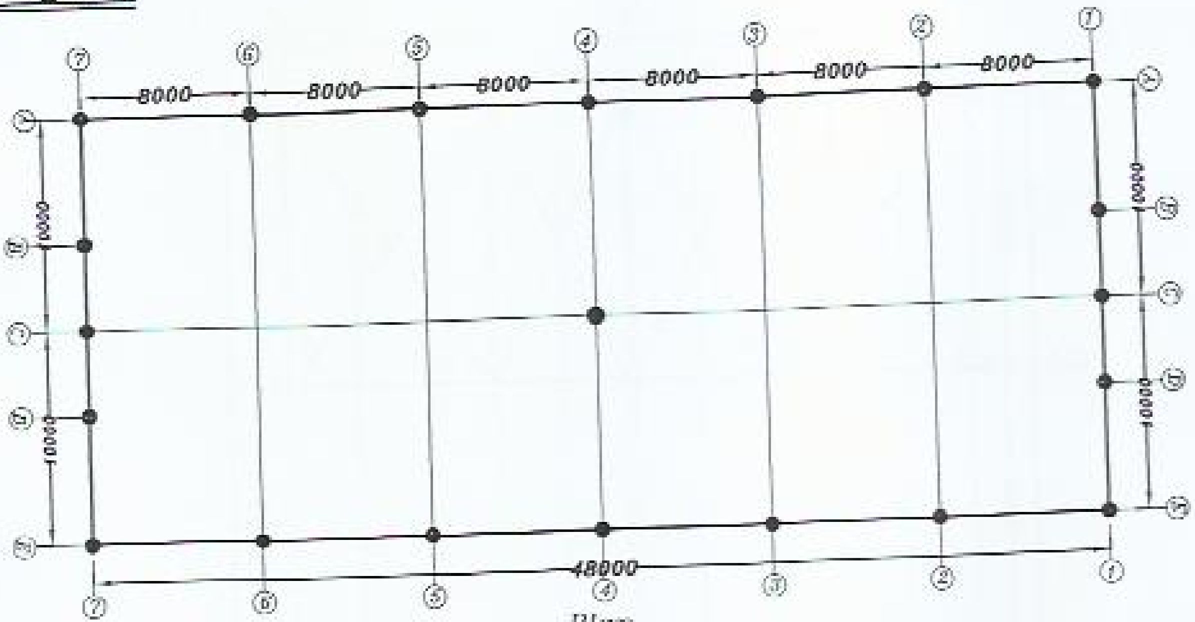
Axis (4)



VERTICAL BRACING

Axis (3)&(5)

Example



Plan

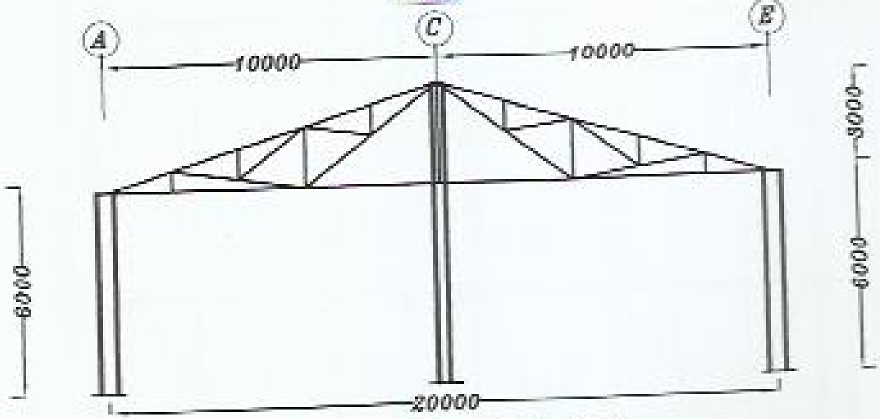
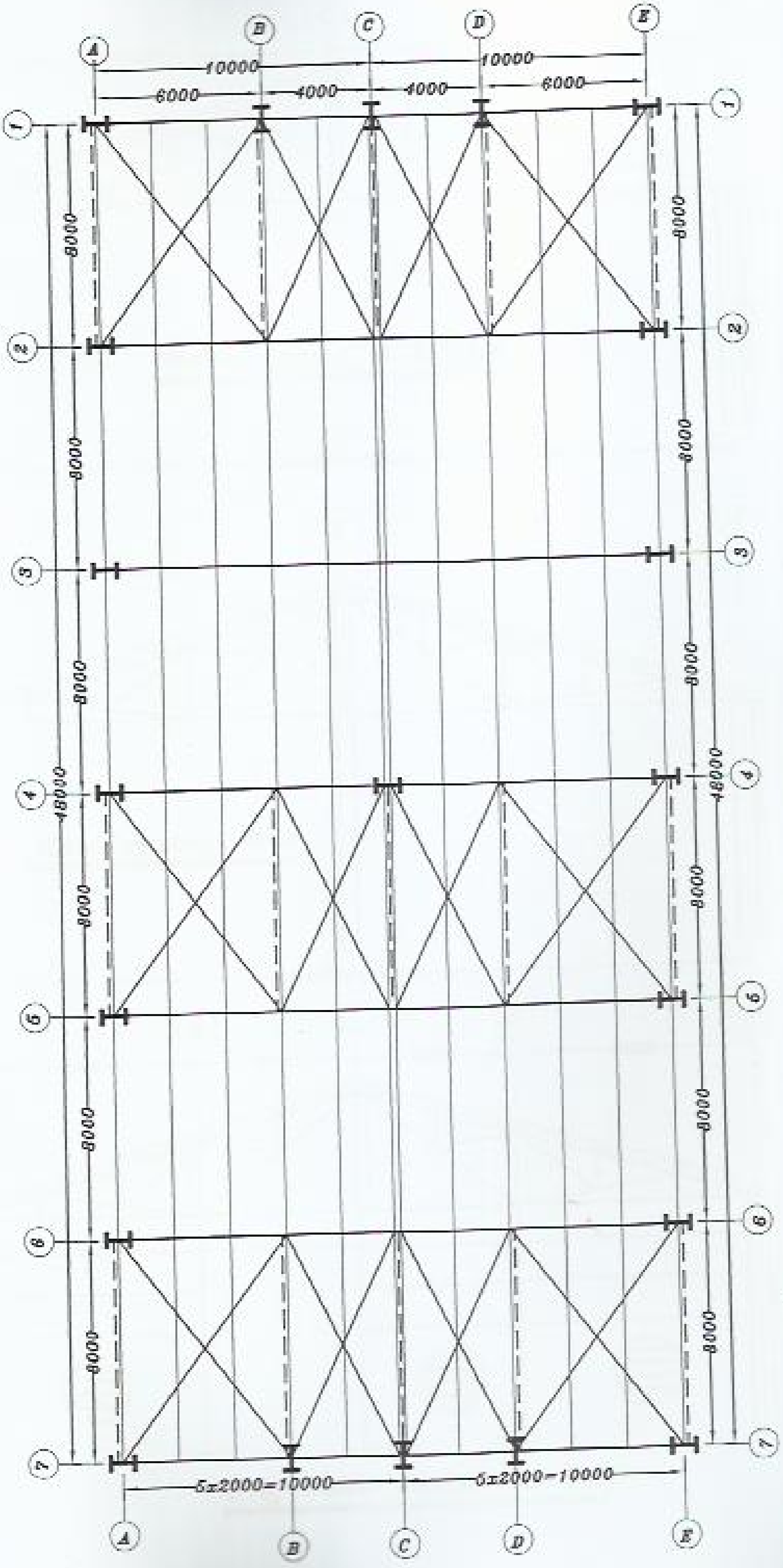


Fig.(1)

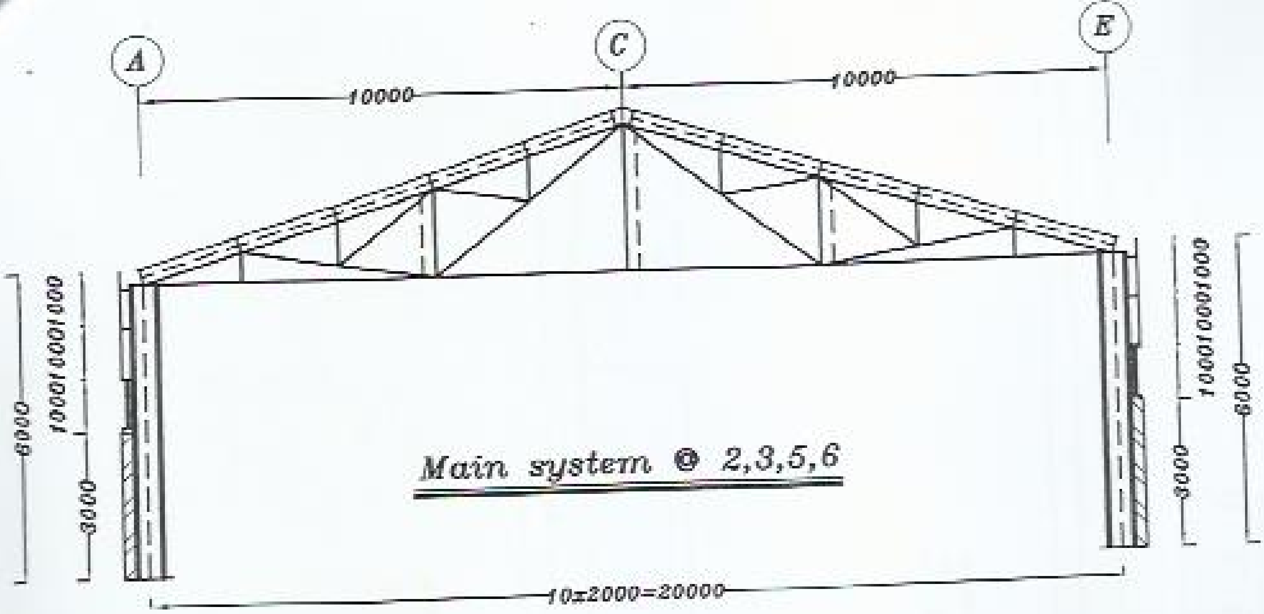
Main system @ Axis 4

Figure 1 shows the column's layout and a cross section for a structure composed of series of steel trusses. Assume all columns to be IPE400. & the roof cladding is composed of sandwich panels with 50mm rib height, it is required to :

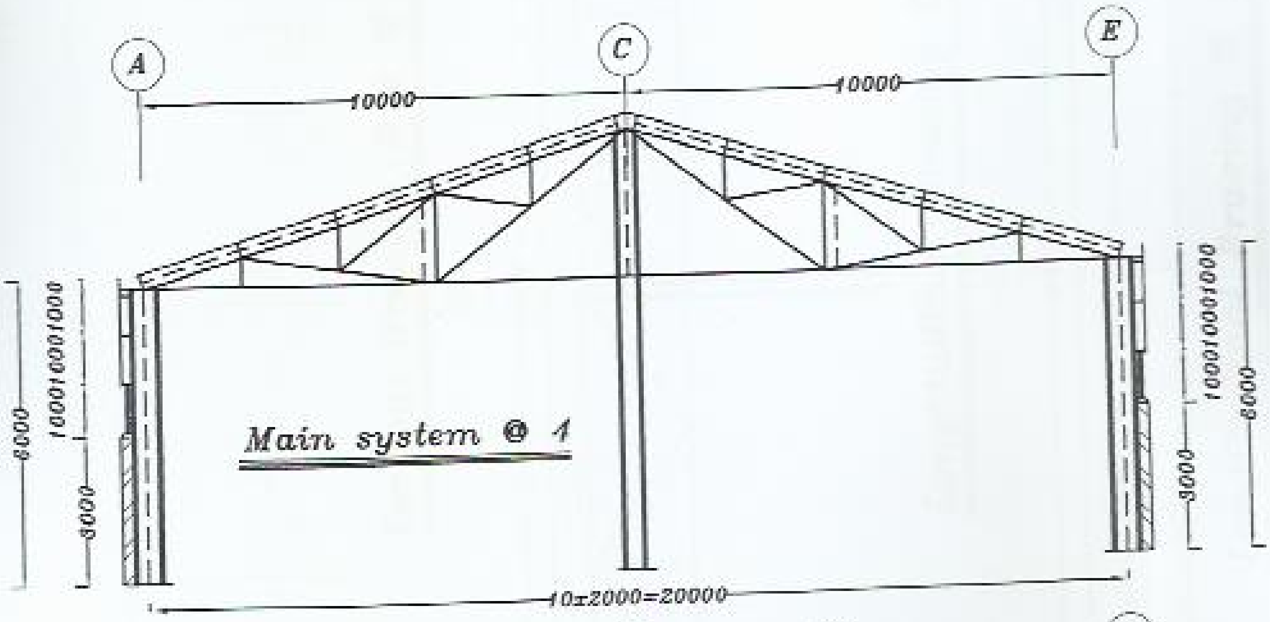
Draw a complete general layout to scale 1:100 for the structure showing all structural elements and the bracing systems.


A
B
C
D
E
10000
10000
6000
4000
4000
6000
1
2
3
4
5
6
7
8000
8000
8000
8000
8000
8000
48000
48000
5x2000=10000
5x2000=10000

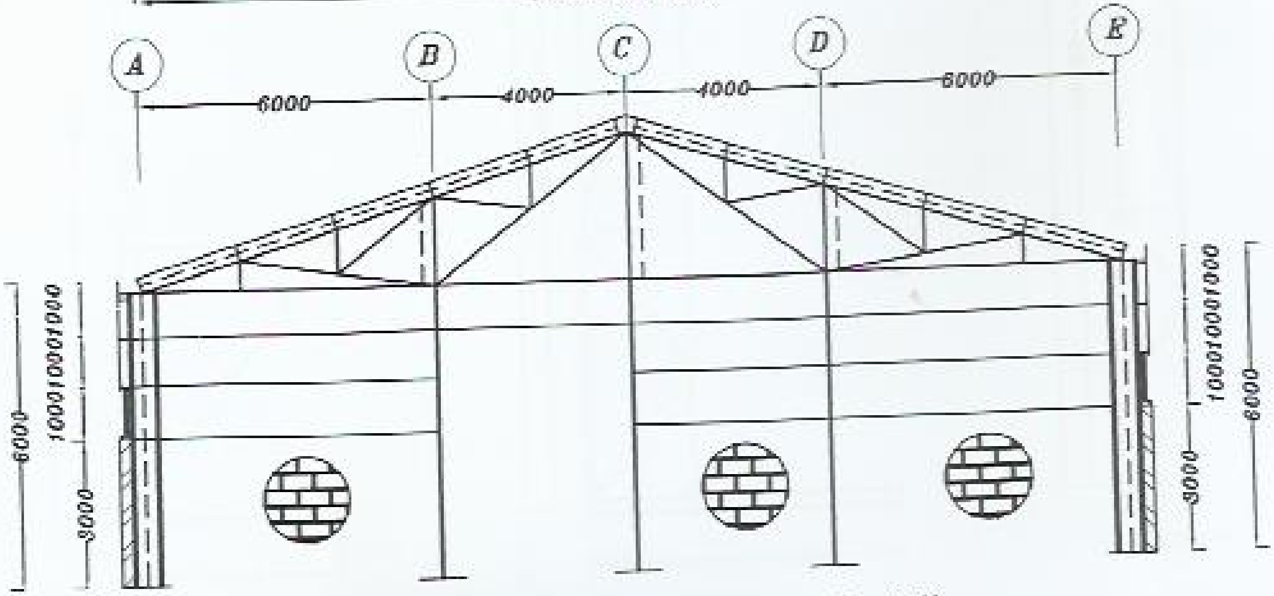
Plan



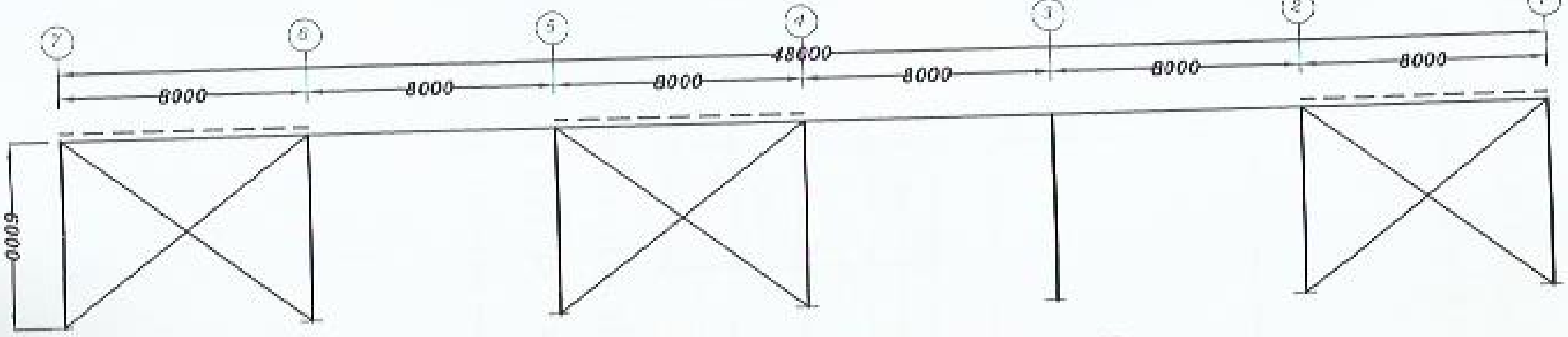
Main system @ 2,3,5,6



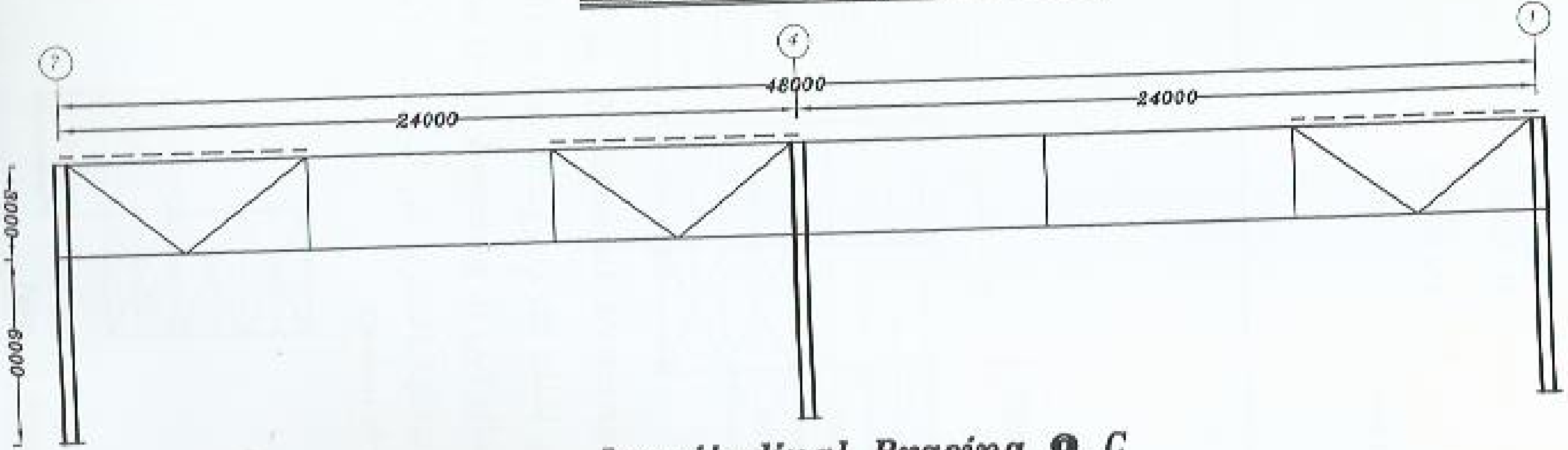
Main system @ 1



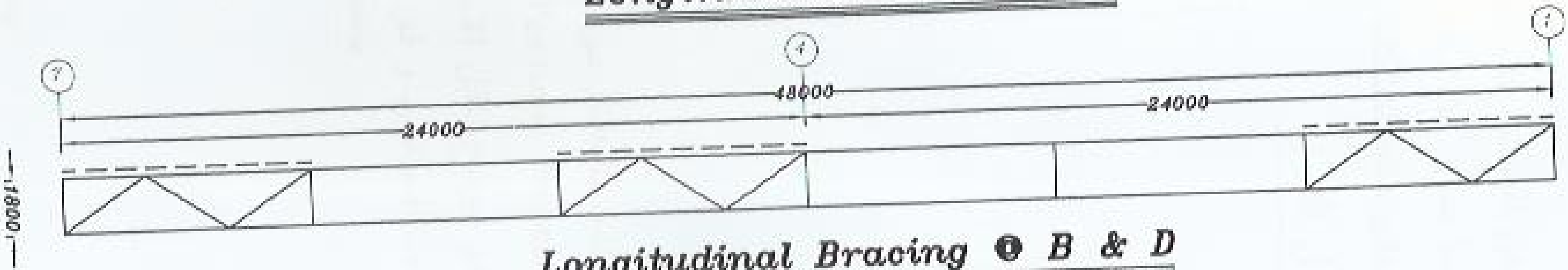
Eng Gable Section @ 1,7



Vertical Bracing @ A @ E



Longitudinal Bracing @ C

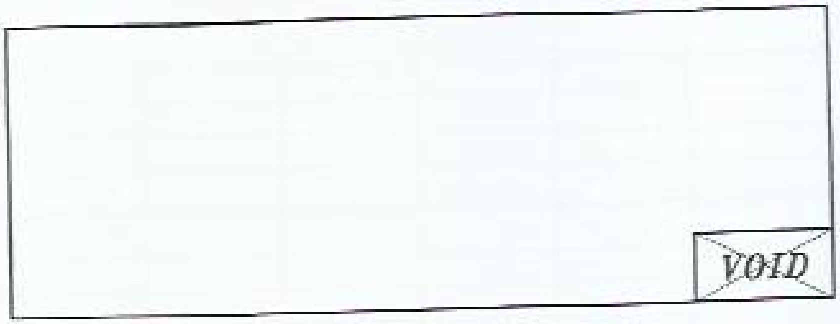


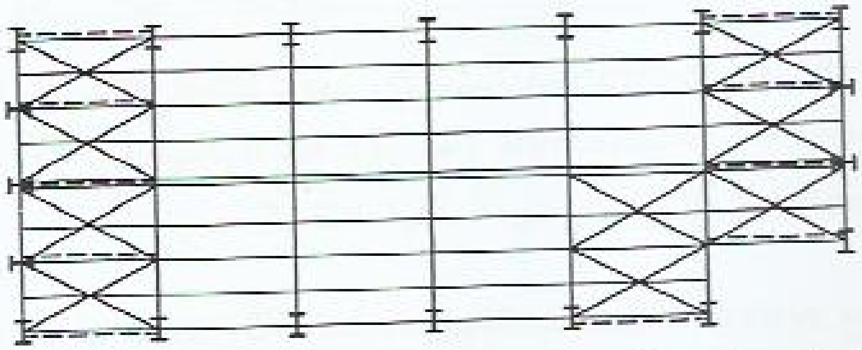
Longitudinal Bracing @ B & D

فكرة

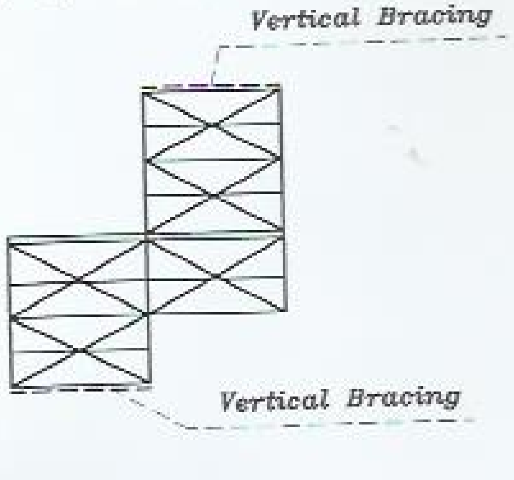
من الممكن عدم عمل *Horizontal Bracing* فى أول باكية فى حالة وجود ما يعوق هذا مثل فاتحة فى أول باكية و فى هذه الحالة يوجد حلان للتغلب على هذه المشكلة و هما :

الحل الاول

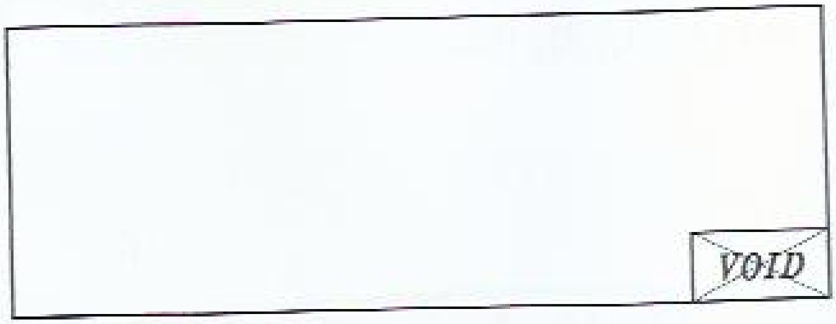


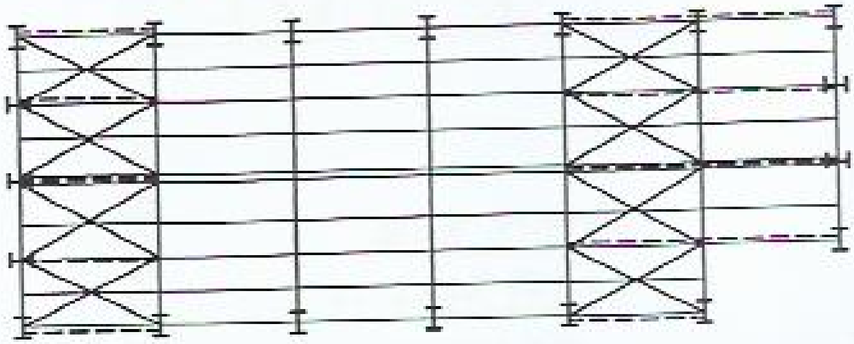


كما نعلم فالهدف من ال *Horizontal Bracing* هو نقل الاحمال الافقية الى ال *Vertical Bracing* و لذلك نقوم بربط ال *Hz. Bracing* فى أول باكيتين كما هو موضح لنقل الاحمال الى ال *Vertical Bracing* الموجود فى اول باكية فى احد الجانبين و فى الباكية الاخرى فى الجانب الاخر .



الحل الثاني

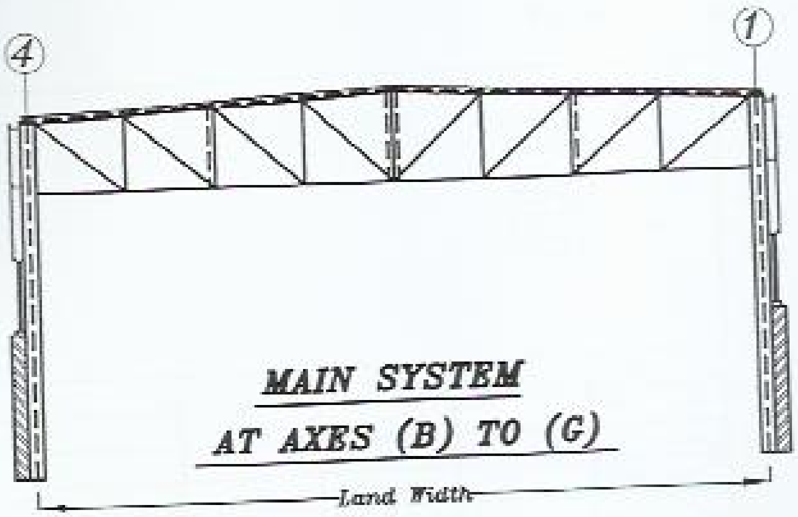


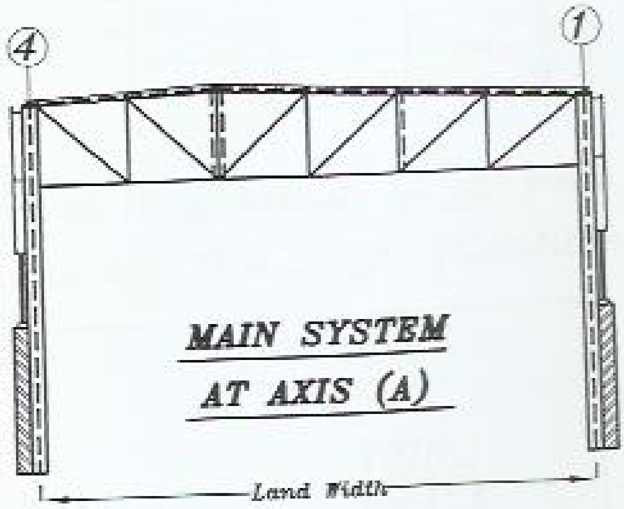


نقوم بوضع *Longitudinal Bracing* أمام كل عمود لتأخذ الاحمال الافقية و توصلها الى ال *Horizontal Bracing* الموجود فى الباكية الثانية و منها الى ال *Vertical Bracing* الموجود فى الباكية الثانية .

وفى حالة عدم وضع *Longitudinal Bracing* أمام كل عمود فان الذى يقوم بوظيفتها هى ال *Purlins* و لكن فى هذه الحالة لا تصمم ال *Purlins* على *moment* فقط و انما نصممها على *normal force & moment* .

يجب الملاحظة ان وجود فاتحه فى هذا الجزء من الارض ادى الى عدم انتظام شكل ال ***Main system*** فى كل المحاور



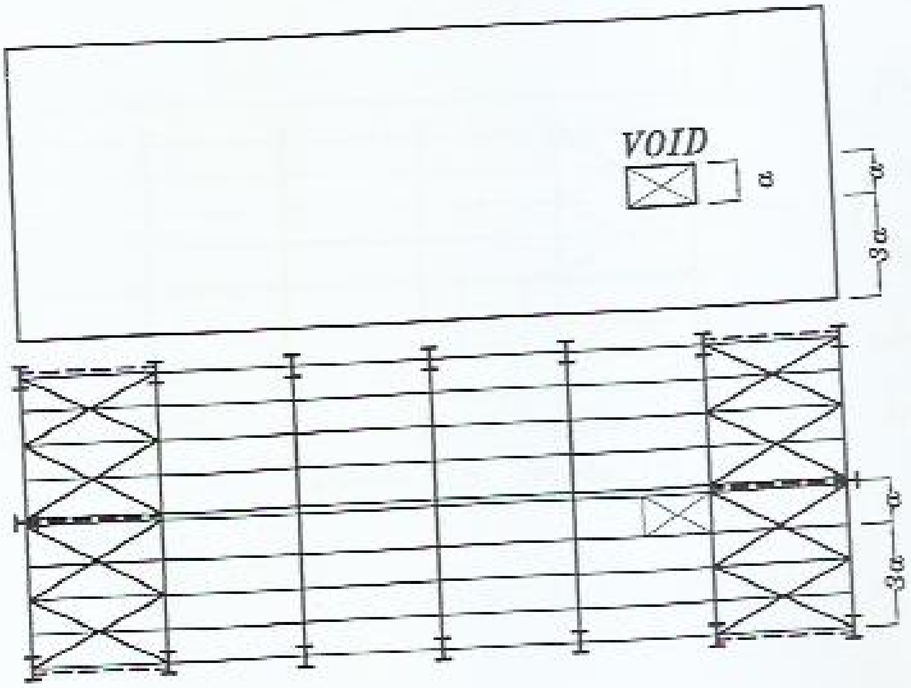


فكرة

فى حالة وجود Void فى منتصف الارض .

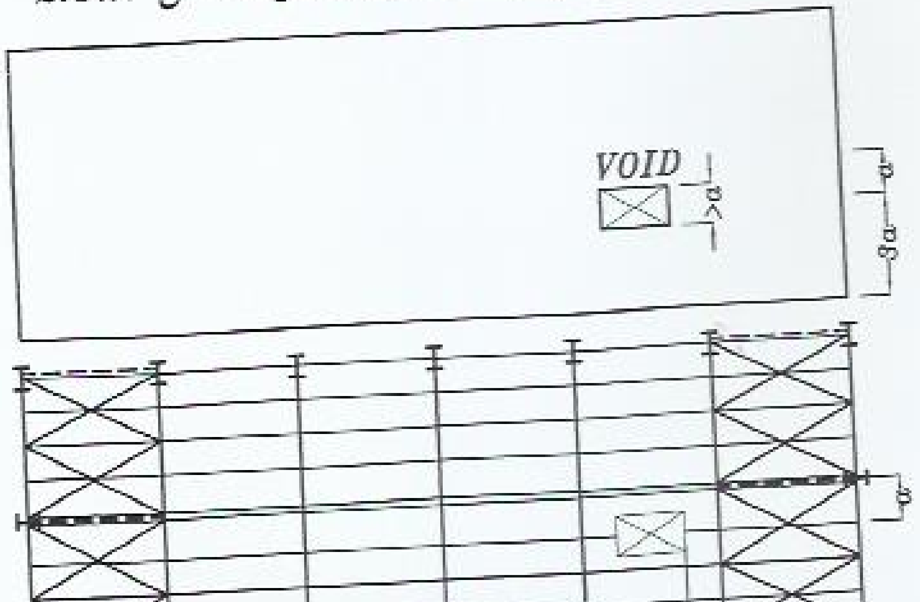
أولا

اذا كان عرض ال Void هو المسافة بين ال Purlins فانه لن تكون مشكلة لان ال Purlins ستمر بجوار ال Void

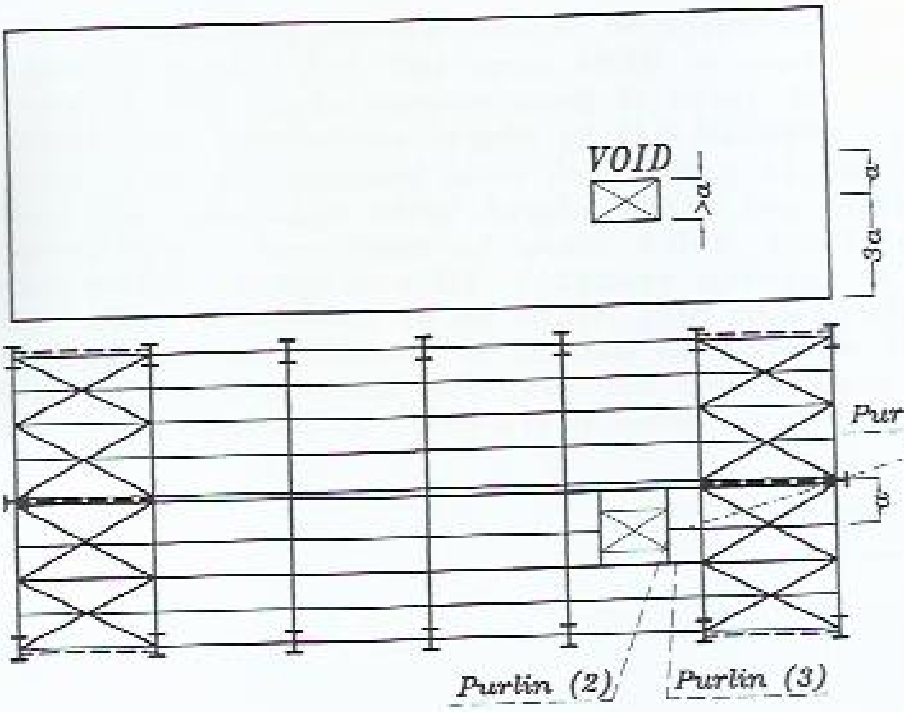


ثانيا

اذا كان عرض ال Void أكبر من المسافة بين ال Purlins فنمد الPurlins التى تقطعها الفتحة Cantilever و لكن بشرط الا يزيد طولها عن 2.5m



اما اذا زاد طول الجزء الـCantilever عن 2.5m فنحمل Purlins عرضية على الـPurlins الطولية لتحمل الـ corrugated sheets



Purlin (1)

↓ متشالة على

Purlin (2)

↓ متشالة على

Purlin (3)

Example

A dairy products factory has to be constructed over the area BCFE shown in figure (1). The area ABED is used for loading and unloading products. The main system used to cover the factory area is steel trusses. The maximum height of the building top level shall not exceed 9.00m from the ground level (according to the construction laws), while the minimum clear height inside the factory is 6.00m from the ground level. Two doors of width 6.00m must be taken into consideration along line BE. A square opening is the roof cover of dimensions 3*3m has to be taken into consideration for a chimney The opening of chimeny is located at distance 2m from line CF and at distance 2m from line EF. Columns are allowed only at lines BC,CF,FE, EB. it is required to Complete general layout for the building

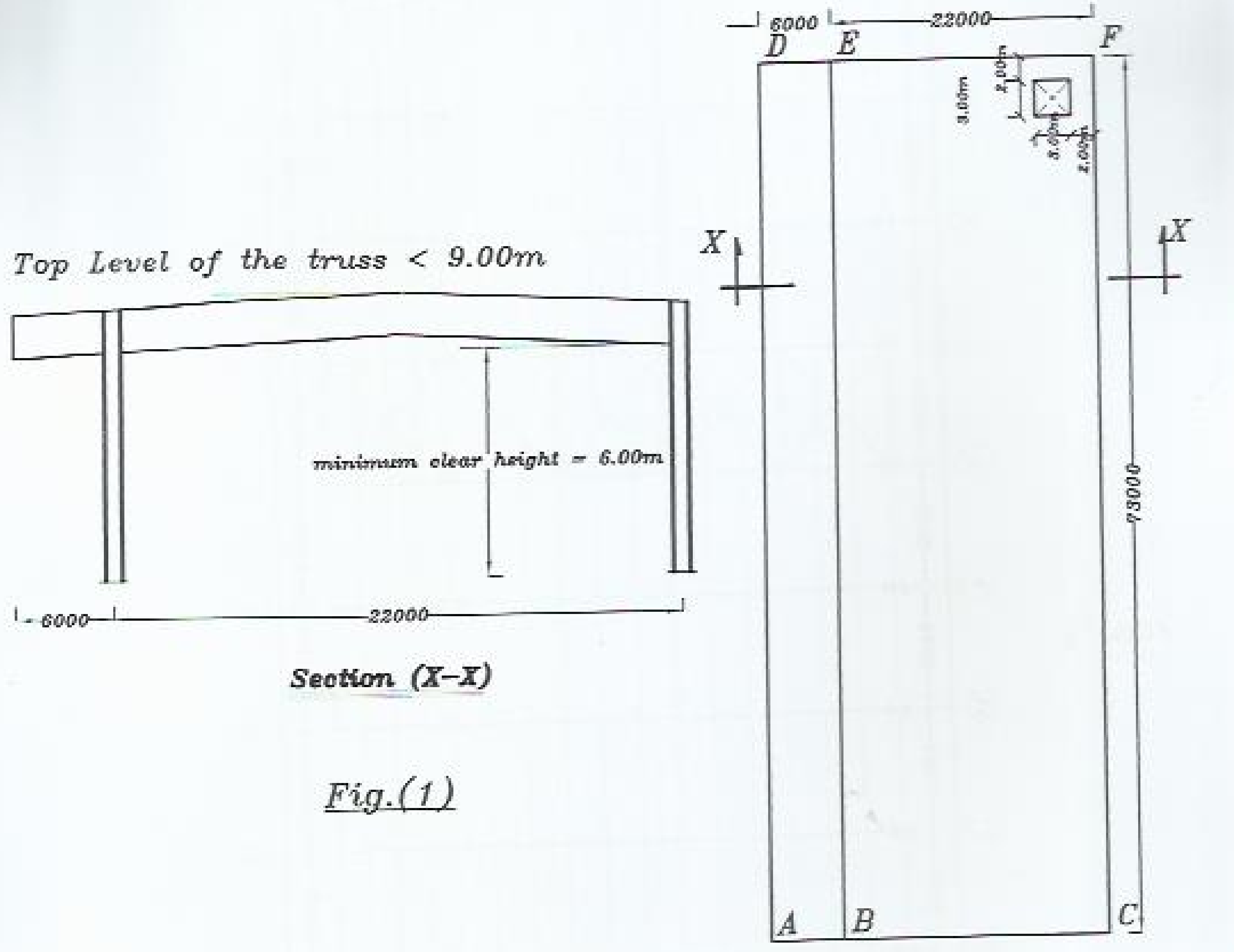
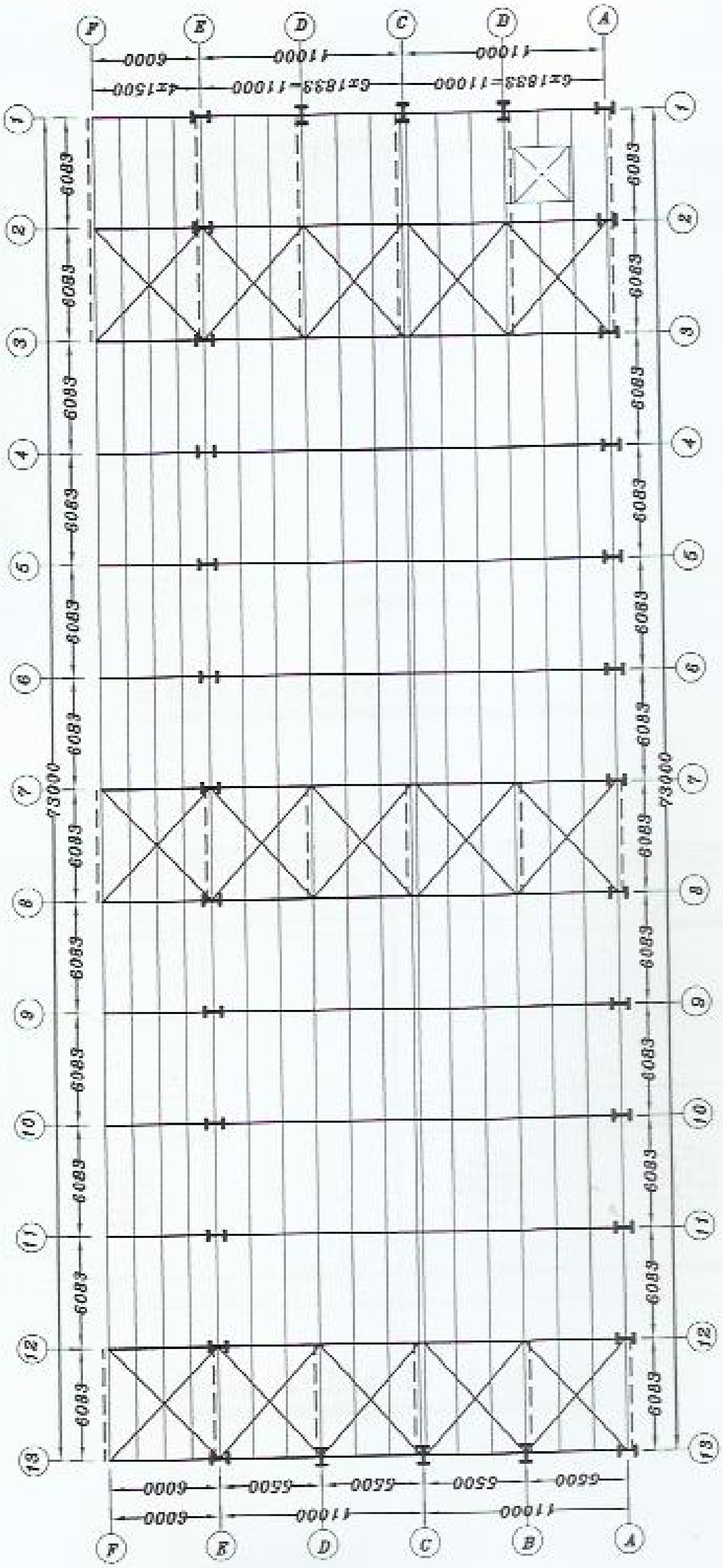
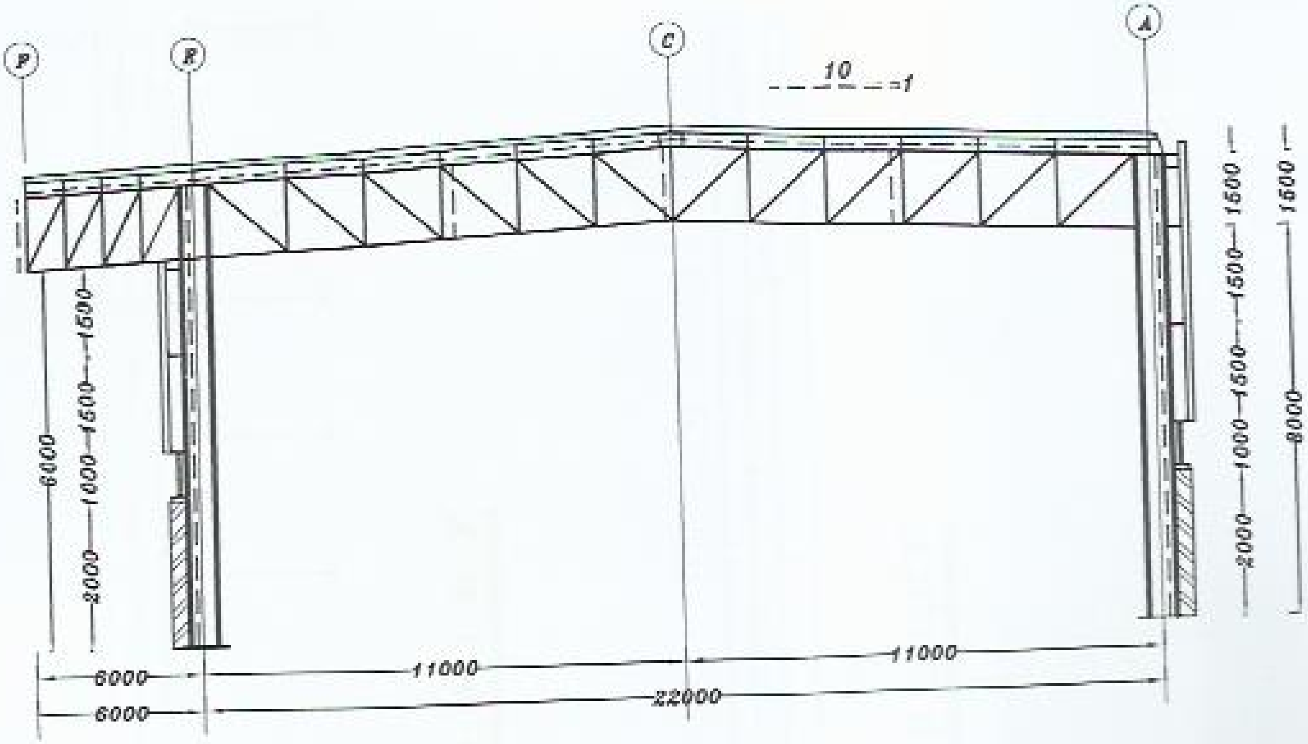


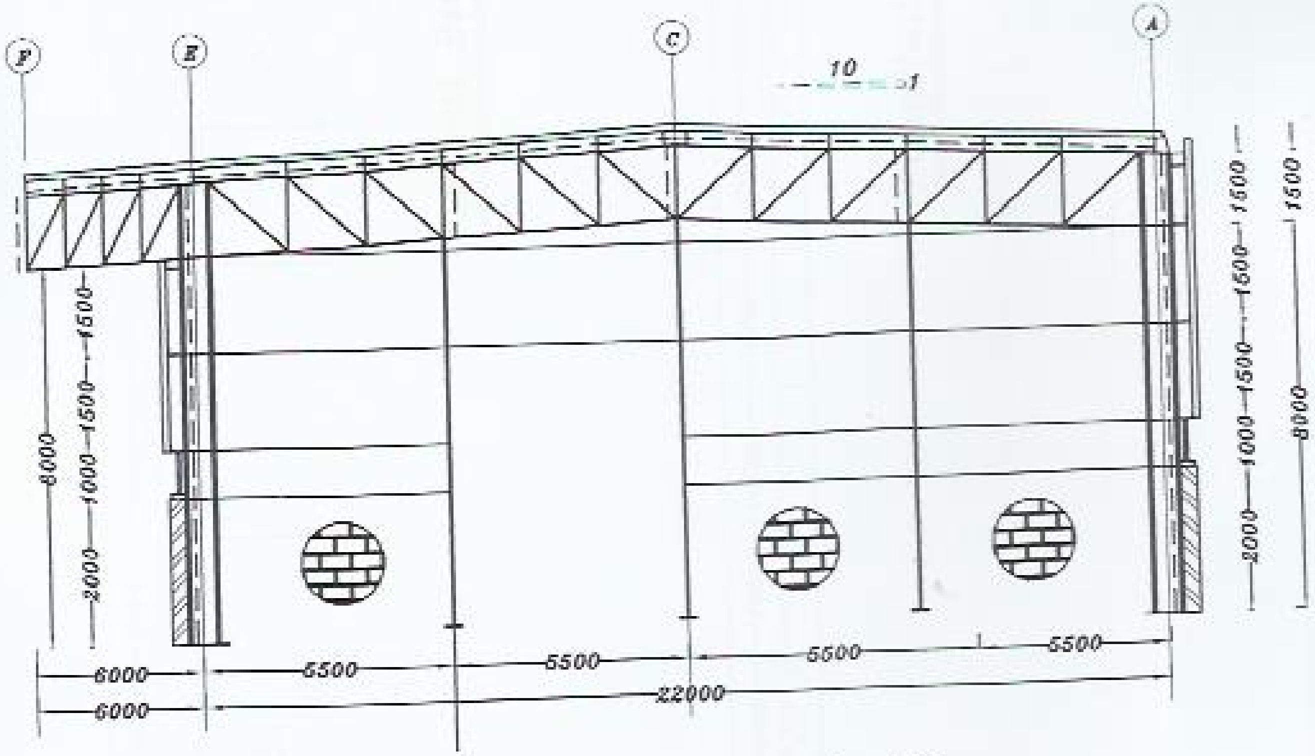
Fig.(1)


F
E
D
C
B
A
6000
11000
4x1500
6x1833=11000
1
2
3
4
5
6
7
8
9
10
11
12
13
6083
73000
6500

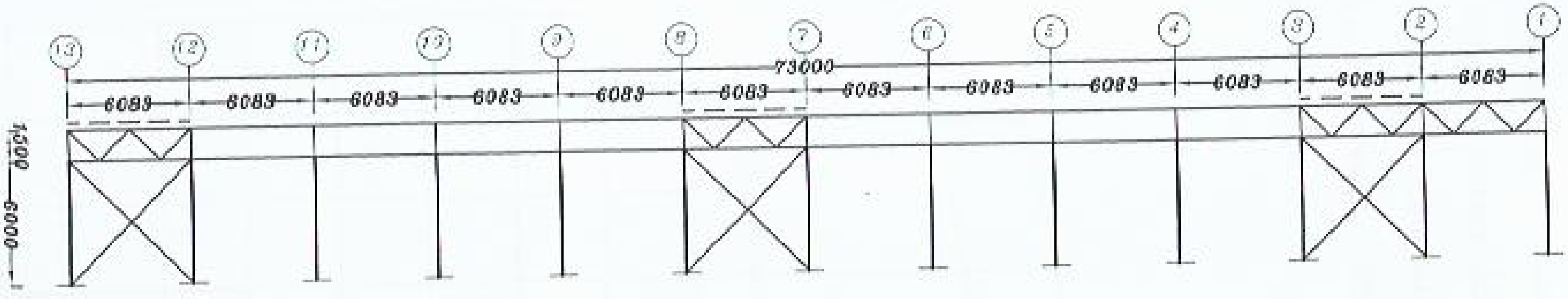
Plan



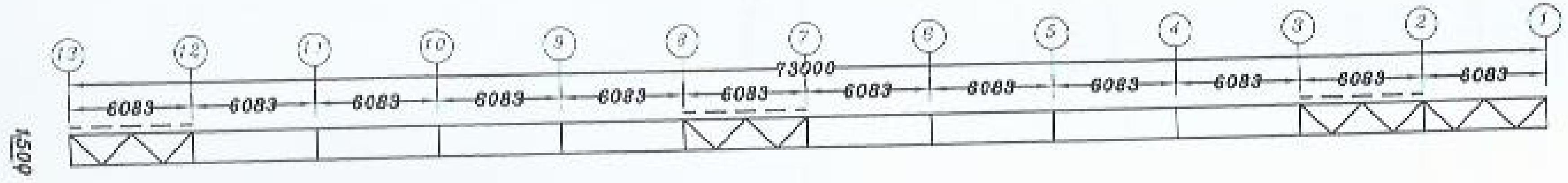
Main system ❶ 2 to 12



End Gable ❶ 1 & 13



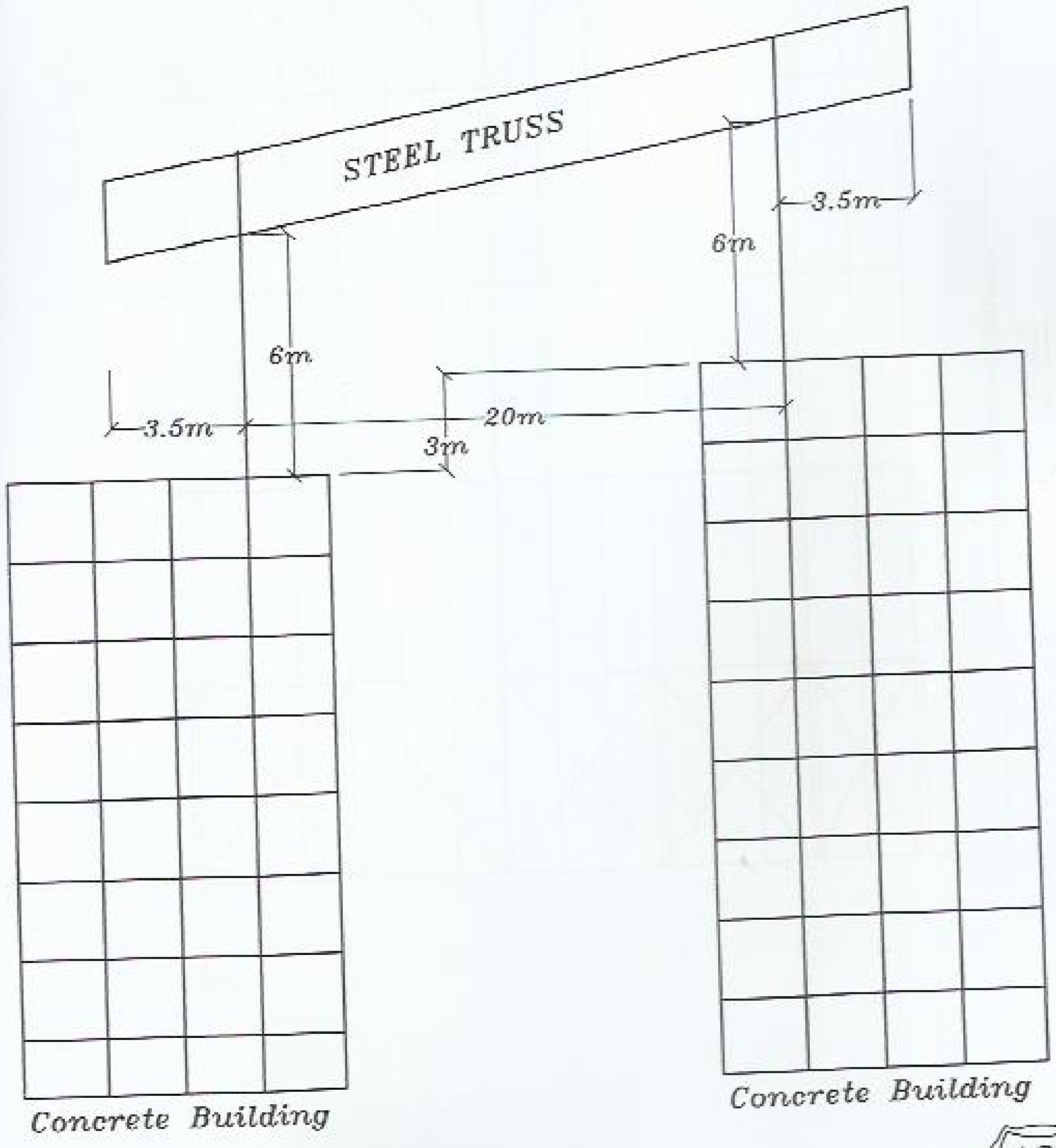
Vertical Bracing ❶ A ❷ E

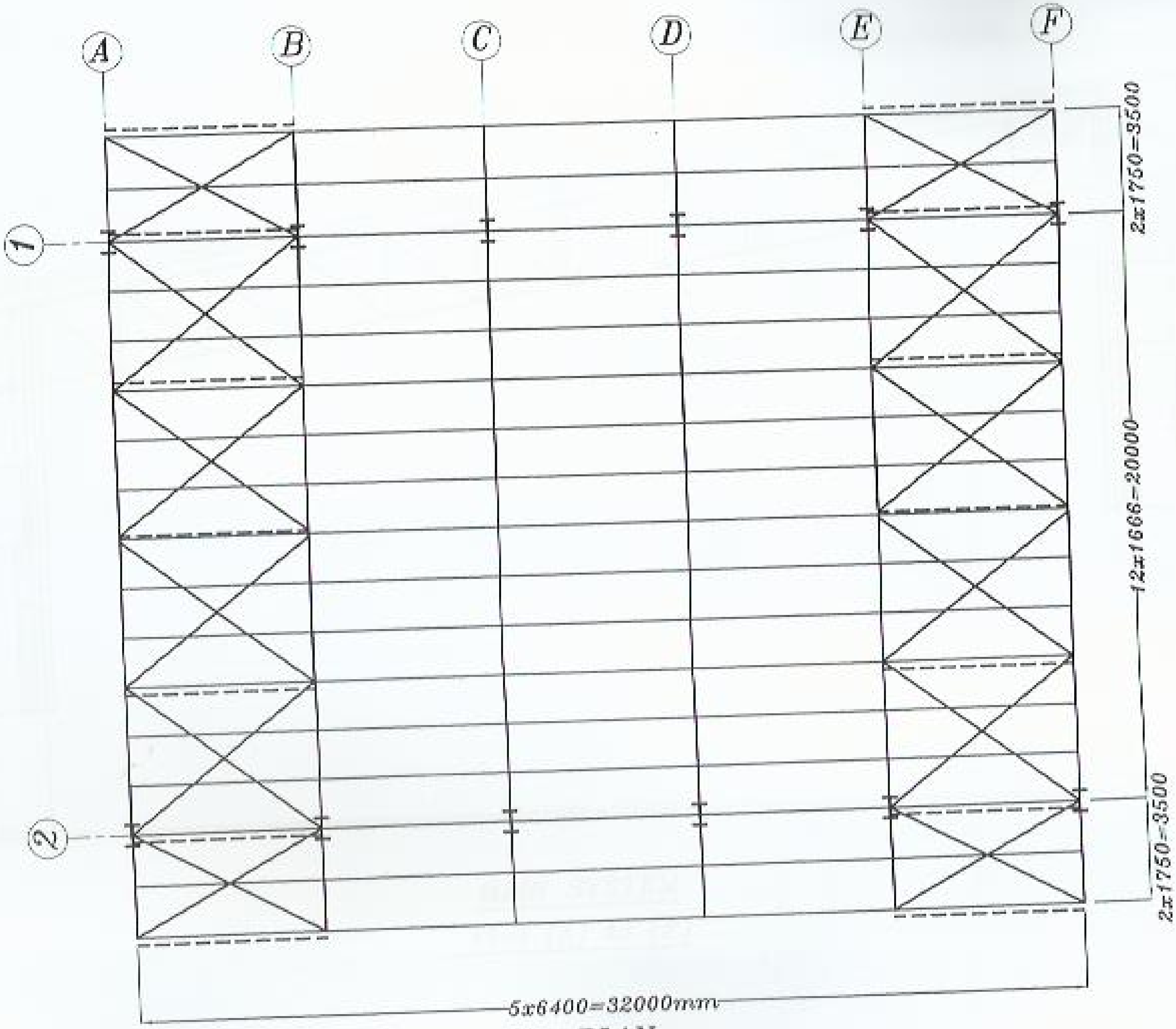


Longitudinal ❶ B,C,D,F

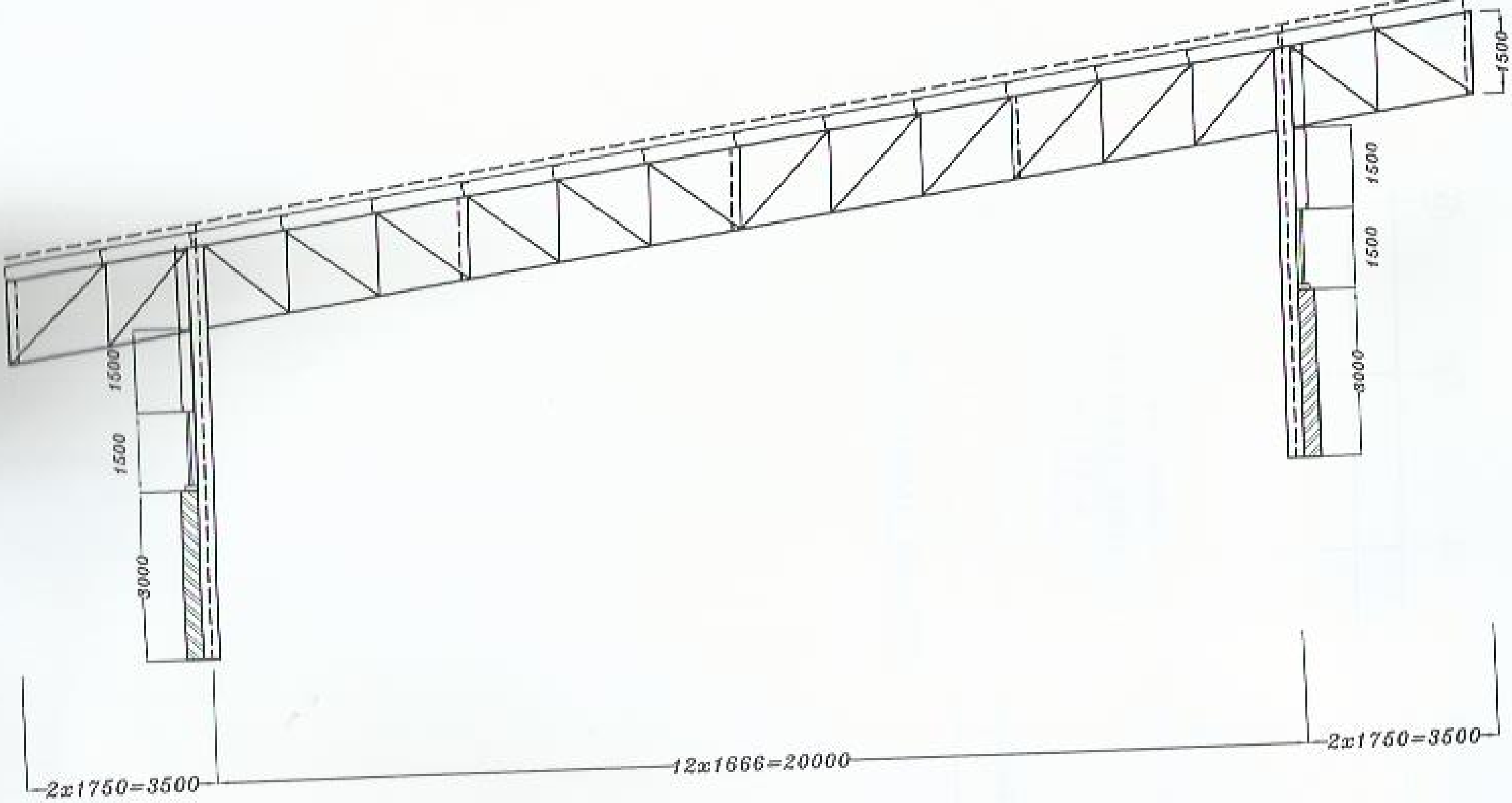
Example

It is required to construct a group of steel trusses to cover an area between two existing concrete buildings. The distance between the two columns is 20.00m, while the required length to be covered is 32.00m. Draw a complete general layout for the shown truss.



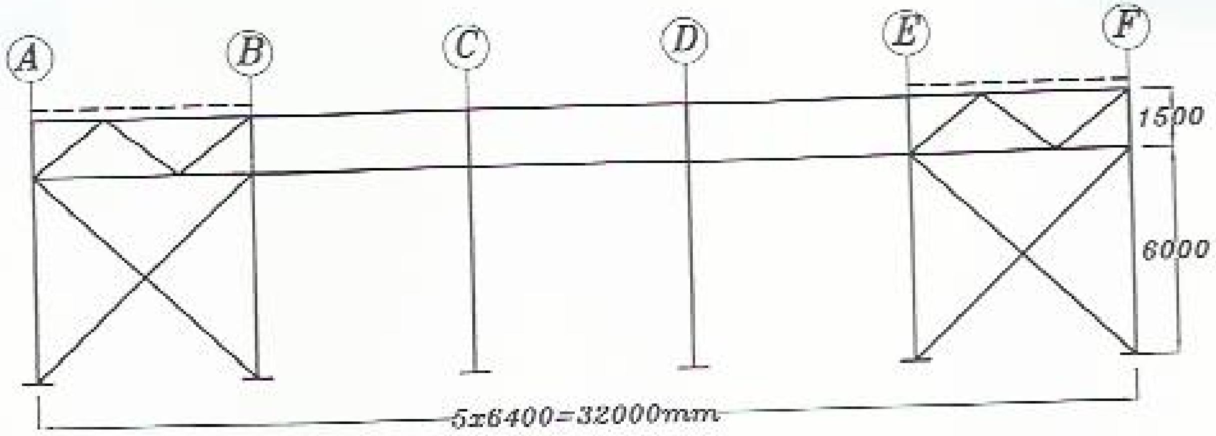

A
B
C
D
E
F
1
2
2x1750=3500
12x1666=20000
2x1750=3500
5x6400=32000mm

PLAN

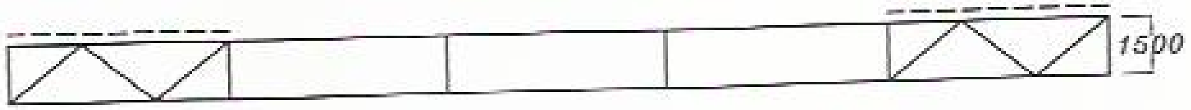


<u>*MAIN SYSTEM*</u>

<u>*Axes (A) to (F)*</u>



VERTICAL BRACING
Axes (1) & (2)



LONGITUDINAL BRACING