steel unit

Calculation Sheet Cotents

Introduction For The Project

The structure is steel frame to cover area of 20x48 m2 of an industrial building.

*** The clear height is 6 m .**

*** The roof is inaccessible .**

*** The strcture is located in Egypt .**

*** The code used for loads is ECP 2008 .**

*** The code used for design is ECP 205 - 2001 .**

*** Steel used for design is Steel 37 .**

*** concrete strength for foundation (Fcu) = 300 Kg / cm2 .**

*** Bearing Capacity = 1.2 Kg / cm2 = 12 t / m2**

*** Use HSB Ø 22 Grade 10.9 For Frame Connections**

*** Use HSB Ø 16 Grade 10.9 For Other Connections**

*** Use steel 36 / 52 For Hinged Base Connection**

*** Crane loads capacity is 8 ton**

From Tables

max reaction = 6.3 t @ 2 m

min reaction = 1.8 t @ 2 m

*** Wind Load**

$$q = 0.5\,\rho V^2\, C_t\, C_s$$

q = (.5* 1.25 * 30^2 * 1 * 1) / 9.81 = 57.34 kg / m2

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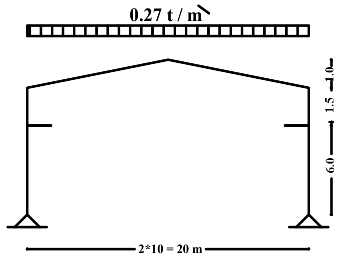
* Live loads

Il.l = 60- 66.6667*tan æ = 60 - 66.6667 * 0.1 = 53.33 kg / m2

Frame Loads

Dead loads



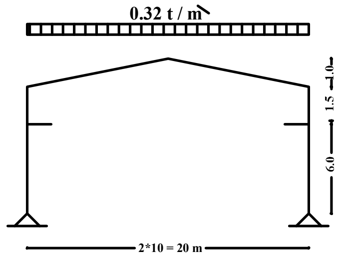
I o.w = 30 KG / m 2 **I cover = 15 kg / m 2**

W o.w = Io.w * S = (30 / 1000) * 6 = 0.18 t / m`

W cover = Icover * (S/cos æ) = $(\frac{15}{1000})$ * 6 = .09 t / m`

W d.l = W o.w + W cover = .18+.09 = .27 t / m`

Live loads



Il.l = 60- 66.6667*tan æ = 60 - 66.6667 * 0.1 = 53.33 kg / m2

W l.l = Il.l * S = (53.33 / 1000) * 6 = 0.32 t / m`

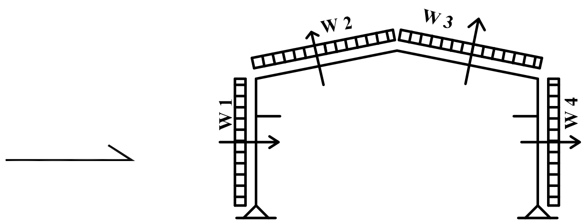
Wind loads

q = 57.34 kg / m2 **k = 1** **tan æ = 0.1**

Ww = Ce * k * q * s = Ce * 1 * (57.34/1000) * 6 = 0.344 Ce t / m`

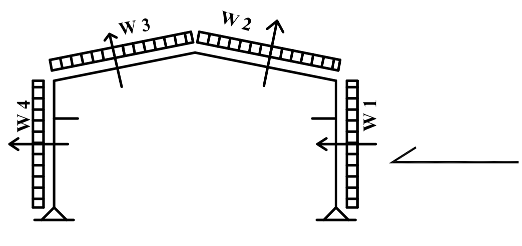
Wind Left

W 1 = 0.344 * 0.8 = 0.275 t / m`
W 2 = 0.344 * 0.8 = 0.275 t / m`
W 3 = 0.344* 0.5 = 0.172 t / m`
W 4 = 0.344* 0.5 = 0.172 t / m`



Wind Right

W 1 = 0.344 * 0.8 = 0.275 t / m`
W 2 = 0.344 * 0.8 = 0.275 t / m`
W 3 = 0.344* 0.5 = 0.172 t / m`
W 4 = 0.344* 0.5 = 0.172t / m`

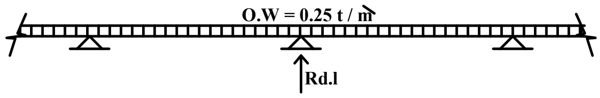


Crane loads

max reaction = 6.3 t
min reaction = 1.8 t

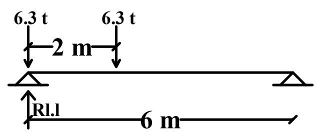
assume Io.w for C.T.G = 250 kg / m2

Rd.l = .25 *6 = 1.5 t



Vmax = Rd.l + 1.25 Rl.l

Rl.l = 6.3 + 6.3 * (4 / 6) = 10.5 t

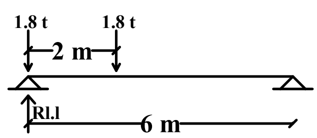


Vmax = Rd.l + 1.25 Rl.l = 1.5 + 1.25 * 10. 5 = 14.625 t

Hmax = .1 R l.l max = 1.05 t

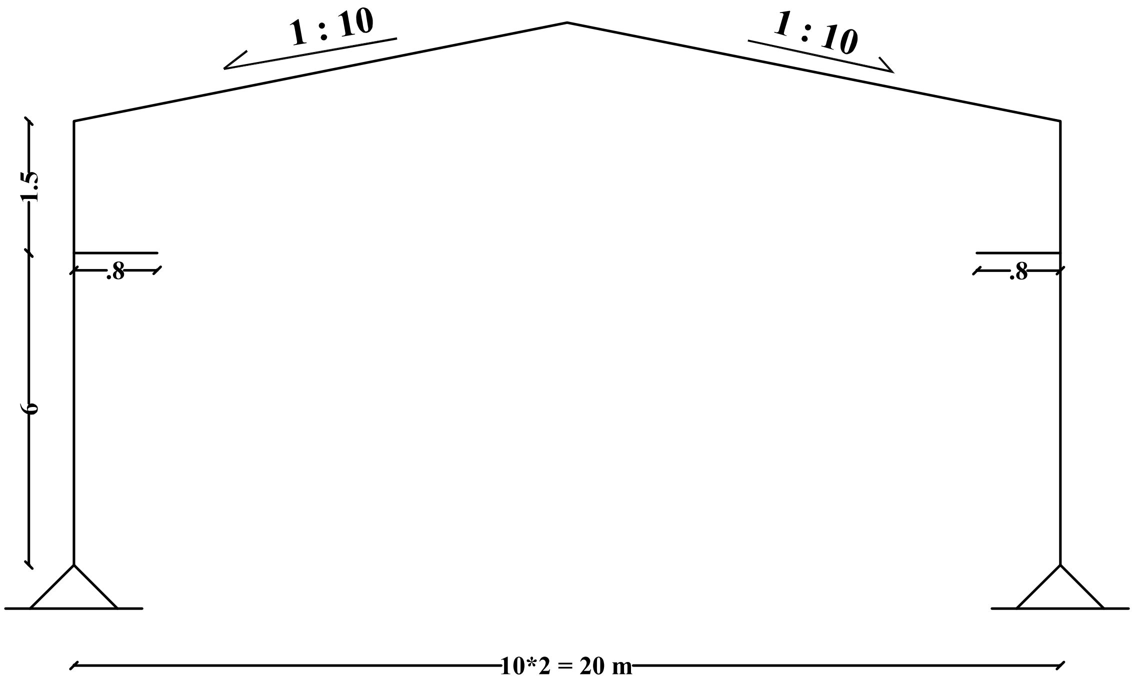
Vmin = Rd.l + 1.25 Rl.l

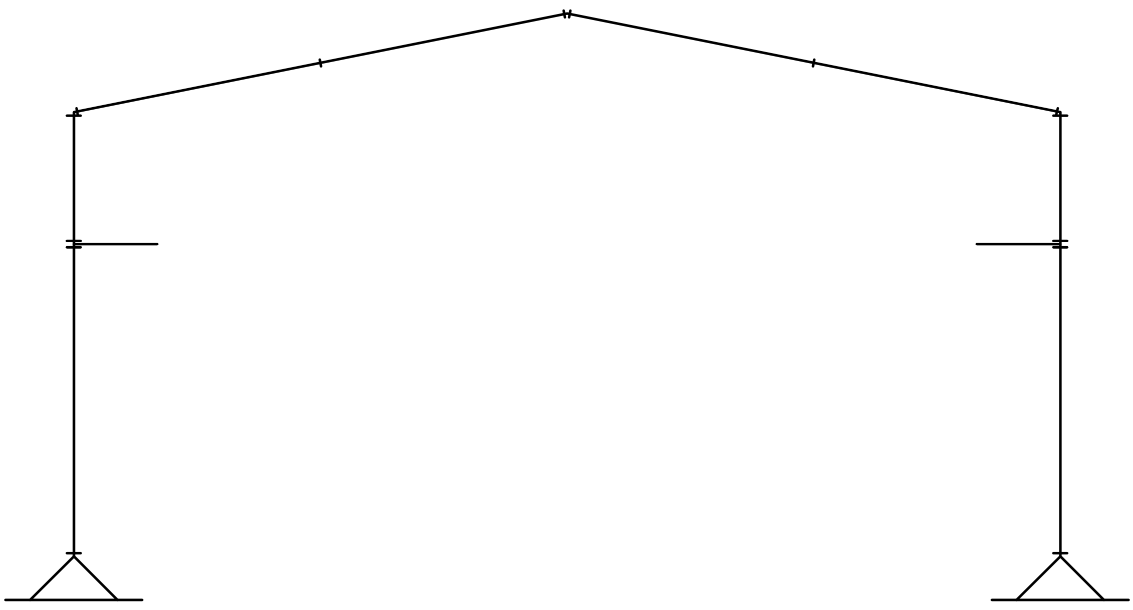
Rd.l = 1.5 t
Rl.l =1.8 + 1.8 * (4 / 6) = 3 t



Vmin = Rd.l + 1.25 Rl.l = 1.5 + 1.25 * 3 = 5.25 t

Hmin = .1 R l.l min = .3 t

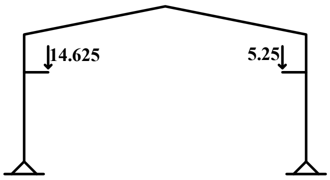

1 : 10
1 : 10
1.5
.8
.8
6
10*2 = 20 m



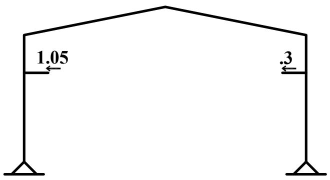
Crane states

Crane left

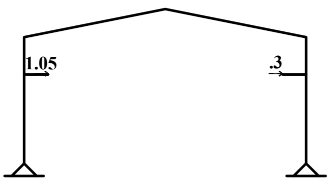
C.L 1



C.L 3

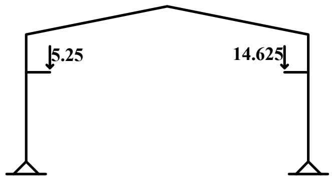


C.L 5

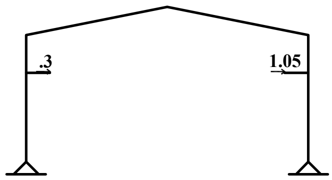


Crane Right

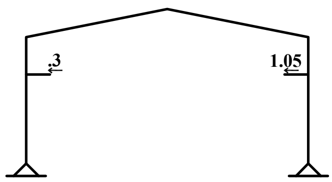
C.L 2



C.L 4



C.L 6



Purlins (continuous)

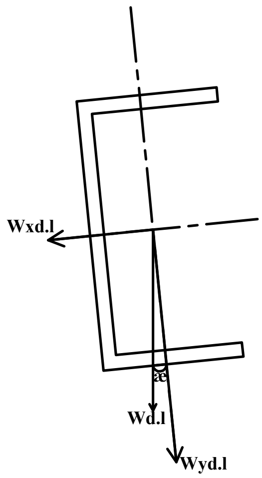
Dead loads

assume o.w. of purlins = 50 kg / m\` **I cover = 15 kg / m 2**

W o.w = (50 / 1000) = .05 t / m\`
W cover = Icover * (a/cos æ) = (15/1000) * (2 / cos 5.71) = .03 t / m\`
W d.l = W o.w + W cover = .05+.03 = .08 t / m\`

Wxd.l = Wd.l * sin æ
Wxd.l = .08 * sin 5.71 = .008 t / m\`

Wyd.l = Wd.l * cos æ
Wyd.l = .08 * cos 5.71 = .08 t / m\`



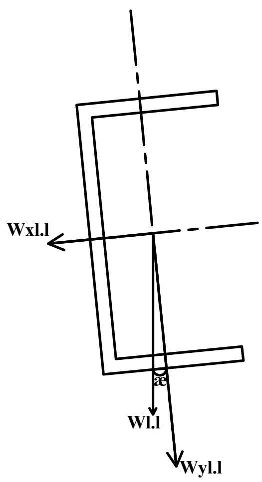
Live loads

Case 1

I l.l = 60- 66.6667*tan æ = 60 - 66.6667 * 0.1 = 53.33 kg / m 2
W l.l = Il.l * a = (53.33 / 1000) * 2 = 0.11 t / m

Wxl.l = Wl.l * sin æ
Wxl.l = .11 * sin 5.71 = .011 t / m\`

Wyl.l = Wl.l * cos æ
Wyl.l = .11 * cos 5.71 = .11 t / m\`



Case 2

100kg concetrated load

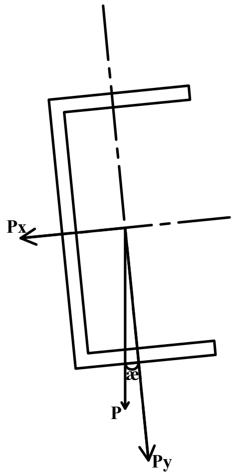
P = 100 kg

P x = p * sin æ

P x = (100/1000) * sin 5.71 = .01 t

P y = p * cos æ

P y = (100/1000) * cos 5.71 = .1 t



Wind loads

q = 57.34 kg / m2   **k = 1**   **tan æ = 0.1**

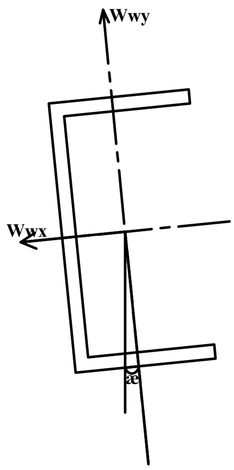
wind load will be sucction . study this state which can reverse the moment

Ww = Ce * k * q * (a/cosæ)

Ww = .8 * 1 * (57.34/1000) * (2/cos5.71) = .092 t / m`

Wwy = .092 t / m`

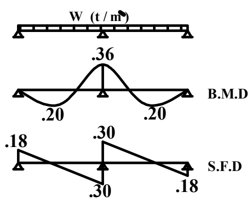
Wwx = 0



Calculation of Straining action on Purlins

Dead loads

*** Mx**

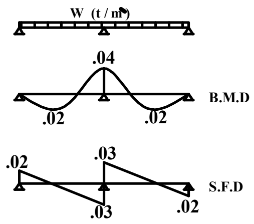


Wyd.l = Wd.l * cos æ

Wyd.l = .08 * cos 5.71 = .08 t / m

* **My**

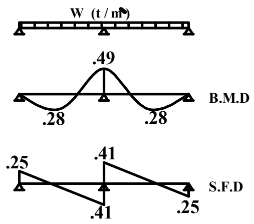
Wxd.l = Wd.l * sin æ
Wxd.l = .08 * sin 5.71 = .008 t / m



Live loads **Case (1)**

* **Mx**

Wyl.l = Wl.l * cos æ
Wyl.l = .11 * cos 5.71 = .11 t / m



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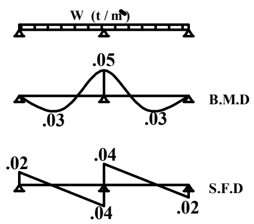
* **My**

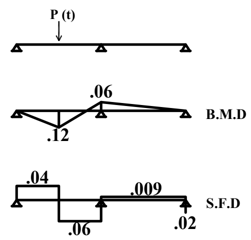
Wxl.l = Wl.l * sin æ
Wxl.l = .11 * sin 5.71 = .011 t / m



Case (2)

* **Mx**

P y = p * cos æ
P y = (100/1000) * cos 5.71 = .1 t



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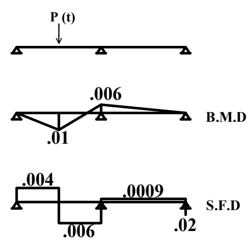
* **My**

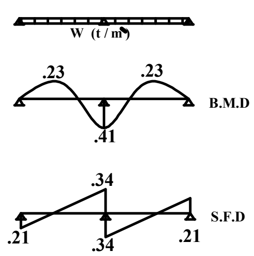
P x = p * sin æ
P x = (100/1000) * sin 5.71 = .01 t



Wind loads

* **Mx**

Wwy = .092 t / m`



Straining action on Purlins

		Mx	My	Qx	Qy
	D.L	36	4	.03	.30
	L.L.1	49	5	.04	.41
	L.L.2	12	1	.006	.06
	W.l	-41	0	0	-.34
Total	+ve	85	9	.07	.71
	-ve	-5	——	——	-.04

Design of Purlins

1 - Without tie rod

a- use I . P . E

Mx = 85 t.cm   My = 9 t.cm   Qx = .07 t   Qy = .71 t

*** Perliminary Design**

assume Zx = 7 Zy

Mx / Zx + My / Zy < Fb   assume Fb = .58 Fy = 1.4 t / cm2

85 / Zx + 9 / (Zx /7) < 1.4   Zx >105.71 cm3

Try I . P . E $\neq$ 160

*** Checks**

1 - check of stresses

c/tf=.5*(bf-tw-2tf)/tf = .5*(8.2-.5-2*.74) / .74 = 4.21 < (16.9/$\sqrt{Fy}$) = 10.91

d / tw = (h - 4 tf) / tw = (16 - 4 * .74) / .5 = 26.08 < 127 / $\sqrt{Fy}$ = 81.98

Section regarding to local buckling is compact

Lu act = max of $\left[\begin{array}{l}\text{spacing between bolts assume 120 cm spacing} \\ .2 * L = .2 * 600 = 120 \text{ cm}\end{array}\right.$

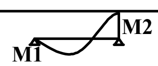
Lu act = 120 cm

$$Lu = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 8.2}{\sqrt{2.4}} = 105.87 \text{ cm} < \text{Lu act}$$

$$Fltb1 = \frac{800}{Lu * d / Af} * Cb = \frac{800*8.2*.74*1.75}{120 * 16} = 4.425 \text{ t / cm2} \not> .58 * Fy$$

Fbx = .58 * Fy = .58 * 2.4 = 1.392 t / cm2

$\boxed{Cb = 1.75 + 1.05 * (M1 / M2) + .3 * (M1 / M2)\text{^}2 = 1.75}$



$Fby = .72 * Fy = .72 * 2.4 = 1.728\ t / cm2$

$(Mx/Zx)/Fbx+(My/Zy)/Fby =(85/109)/1.392+(9/16.7)/1.728= .87\ t/cm2 <1.00$

Safe O.K .

2 - check of shear

$qy = Qy / (d * tw) = .71 / (16 * .5) = .089\ t / cm2 < .35 * Fy = .84\ t/ cm2$

$qx =(3/2)*(Qx /2*Af)=(3/2)*.07/(2*8.2*.74) =.0087\ t/cm2 <.35*Fy=.84t/cm2$

Safe O.K .

3 - check of deflection

$\triangle_{all.} = L / 300 = 600 / 300 = 2\ cm$

$\triangle_{act.} \longrightarrow Ml.l = 49\ t.\ cm \longrightarrow Ml.l = (W * L^2) / 8$

$W = .00109\ t / cm = .109\ t / m$

$\triangle_{act.} = (5 * .109 * 600^4) / (384 * 2100 * 869 * 100) = 1.008\ cm$

Safe O.K .

$\boxed{\text{Use I . P . E} \neq 160}$

b - use U . P . N (channel section)

$Mx = 85\ t.cm \qquad My = 9\ t.cm \qquad Qx = .07\ t \qquad Qy = .71\ t$

*** Perliminary Design**

assume $Zx = 7\ Zy$

$Mx / Zx + My / Zy < Fb$ assume $Fb = .58\ Fy = 1.4\ t / cm2$

$85 / Zx + 9 / (Zx /7) < 1.4$ $Zx >105.71\ cm3$

Try U . P. N$\neq$160

*** Checks**

1 - check of stresses

$c / tf = bf / tf = 6.5 / 1.05 = 6.19 < (23 / \sqrt{Fy}) = 14.85$

$d / tw = (h - 4\ tf) / tw = (16 - 4 * 1.05) / .75 = 15.73 < 190 / \sqrt{Fy} = 122.64$

Section regarding to local buckling is noncompact

$$Lu\ act = \max\ of \begin{cases} \text{spacing between bolts assume 120 cm spacing} \\ .2 * L = .2 * 600 = 120\ cm \end{cases}$$

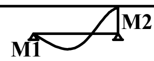
$Lu\ act = 120\ cm$

$$Lu = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 6.5}{\sqrt{2.4}} = 83.91 \text{ cm} < Lu \text{ act}$$

$$Fltb1 = \frac{800}{Lu * d / Af} * Cb = \frac{800*6.5*1.05*1.75}{120 * 16} = 4.98 \text{ t / cm2} \not> .58 * Fy$$

$$Fbx = .58 * Fy = .58 * 2.4 = 1.392 \text{ t / cm2}$$

$$\boxed{Cb = 1.75 + 1.05 * (M1 / M2) + .3 * (M1 / M2)^2 = 1.75}$$



$$Fby = .72 * Fy = .72 * 2.4 = 1.728 \text{ t / cm2}$$

$$(Mx/Zx)/Fbx+(My/Zy)/Fby =(85/116)/1.392+(9/18.3)/1.728= .811 \text{ t/cm2} <1.00$$

Safe O.K .

2 - check of shear

$$qy = Qy / (d * tw) = .71 / (16 * .75) = .059 \text{ t / cm2} < .35 * Fy = .84 \text{ t/ cm2}$$

$$qx =(3/2)*(Qx/2*Af)=(3/2)*.07/(2*6.5*1.05) =.0077 \text{ t/cm2} <.35*Fy=.84 \text{ t/cm2}$$

Safe O.K .

3 - check of deflection

$$\triangle_{all.} = L / 300 = 600 / 300 = 2 \text{ cm}$$

$$\triangle act. \longrightarrow Ml.l = 49 \text{ t. cm} \longrightarrow Ml.l = (W * L^2) / 8$$

$$W = .00109 \text{ t / cm} = .109 \text{ t / m}$$

$$\triangle act. = (5 * .109 * 600^4) / (384 * 2100 * 869 * 100) = 1.008 \text{ cm}$$

Safe O.K .

Use U . P . N $\neq$ 160

I . P . E	U . P . N
$\neq$160	$\neq$160
15.8 Kg / m	18.8 Kg / m

The I.P.E section is the most economic section

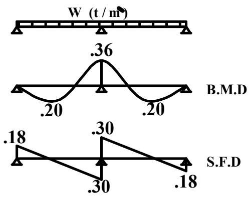
2 - With tie rod

*** Purlins are to be supported at thier one - third points between frames**

Calculation of Straining action on Purlins

Dead loads

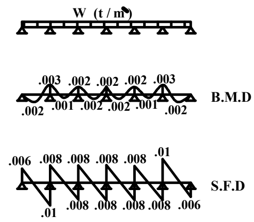
*** Mx**



Wyd.l = Wd.l * cos æ

Wyd.l = .08 * cos 5.71 = .08 t / m

*** My**

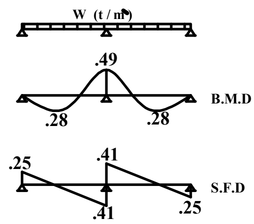


Wxd.l = Wd.l * sin æ

Wxd.l = .08 * sin 5.71 = .008 t / m

Live loads **Case (1)**

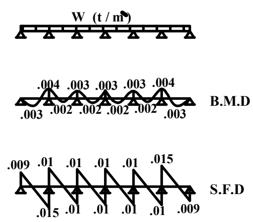
*** Mx**



Wyl.l = Wl.l * cos æ

Wyl.l = .11 * cos 5.71 = .11 t / m

*** My**

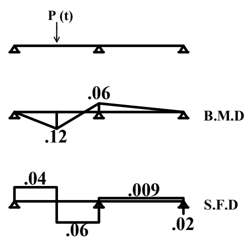


Wxl.l = Wl.l * sin æ

Wxl.l = .11 * sin 5.71 = .011 t / m

Case (2)

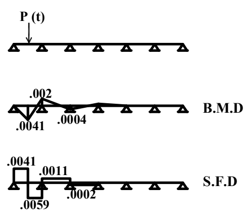
*** Mx**



P y = p * cos æ

P y = (100/1000) * cos 5.71 = .1 t

*** My**



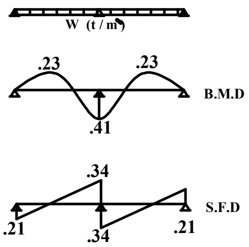
P x = p * sin æ

P x = (100/1000) * sin 5.71 = .01 t

Wind loads

*** Mx**

Wwy = .092 t / m`



Straining action on Purlins

		Mx	My	Qx	Qy
	D.L	36	.3	.01	.30
	L.L.1	49	.4	.015	.41
	L.L.2	12	.4	.006	.06
	W.l	-41	0	0	-.34
Total	+ve	85	.7	.025	.71
	-ve	-5	——	——	-.04

Design of Purlins

2 - With tie rod

a- use I . P . E

Mx = 85 t.cm My = .7 t.cm Qx = .025 t Qy = .71 t

*** Perliminary Design**

assume Zx = 7 Zy

Mx / Zx + My / Zy < Fb **assume Fb = .58 Fy = 1.4 t / cm2**

85 / Zx + 9 / (Zx /7) < 1.4 **Zx > 64.214 cm3**

Try I . P . E ≠140

*** Checks**

1 - check of stresses

c/tf = .5*(bf - tw -2tf)/tf=.5*(7.3-.47-2*.69)/.69=3.95 < (16.9/ $\sqrt{Fy}$)=10.91

d / tw = (h - 4 tf) / tw = (14 - 4 * .69) / .47 = 23.91 < 127 / $\sqrt{Fy}$ = 81.98

Section regarding to local buckling is compact

Lu act = max of ⎡ **spacing between bolts assume 120 cm spacing**
⎣ **.2 * L = .2 * 600 = 120 cm**

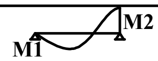
$L_{u\,act} = 120\ cm$

$$L_u = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 7.3}{\sqrt{2.4}} = 94.24\ cm < L_{u\,act}$$

$$F_{ltb1} = \frac{800}{L_u * d / A_f} * C_b = \frac{800 * 7.3 * .69 * 1.75}{120 * 14} = 4.198\ t / cm2 \not> .58 * Fy$$

$$F_{bx} = .58 * Fy = .58 * 2.4 = 1.392\ t / cm2$$

$$\boxed{C_b = 1.75 + 1.05 * (M1 / M2) + .3 * (M1 / M2)^2 = 1.75}$$



$$F_{by} = .72 * Fy = .72 * 2.4 = 1.728\ t / cm2$$

$$(Mx/Zx)/Fbx+(My/Zy)/Fby=(85/77.3)/1.392+(.7/12.3)/1.728=.823\ t/cm2<1.00$$

Safe O.K .

2 - check of shear

$$q_y = Q_y / (d * t_w) = .71 / (14 * .47) = .108\ t / cm2 < .35 * Fy = .84\ t/ cm2$$

$$q_x =(3/2)*(Q_x/2 A_f)=(3/2)*.025/(2*7.3*.69)=.00372\ t/cm2<.35*Fy=.84 t/cm2$$

Safe O.K .

3 - check of deflection

$$\triangle_{all.} = L / 300 = 600 / 300 = 2\ cm$$

$$\triangle_{act.} \longrightarrow M_{l.l} = 49\ t.\ cm \longrightarrow M_{l.l} = (W * L^2) / 8$$

$$W = .00109\ t / cm = .109\ t / m`$$

$$\triangle_{act.} = (5 * .109 * 600^4) / (384 * 2100 * 541 * 100) = 1.62\ cm$$

Safe O.K .

$\boxed{\text{Use I . P . E} \neq 140}$

*** If we use U . P . N**

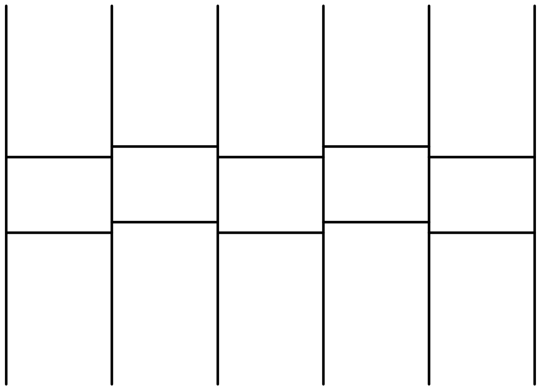
U . P . N ≠ 140 is safe . which have the weight 16 kg / m

I . P . E ≠140 have the weight 12.9 kg / m

*** The I.P.E section is the most economic section**

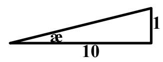
Design of Tie rod

Purlins = (12*12.9) / 20 = 7.74 kg / m2
Cover = 15 / cos æ = 15.07 kg / m2
Live load = 53.33 kg / m2
Wtotal =7.74+15.07+53.33= 76.14 kg/m2



Component of load parallel to roof=Wtotal*sinæ=76.14*sin5.71=7.58 kg/m2
Load on rod = 7.58 * (6 / 3) * (20 / 2) = 151.6 kg
Use 2.8 ton as minimum
Area req. = 2.8 / 1.4 = 2 cm2

Use 16 mm tie rod (area = 2.01 cm2)



cos æ = .995
sin æ = .0995

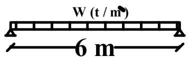
Force in tie between ridge purlins = 151.6 / cos æ = 152.36 kg
Use 2.8 ton as minimum
Area req. = 2.8 / 1.4 = 2 cm2

Use 16 mm tie rod (area = 2.01 cm2)

Design of Crane Track Girder

Dead loads

assume o.w. of C.T.G = 250 kg / m`
W o.w = (50 / 1000) = .025 t / m`
W d.l = .025 t / m`



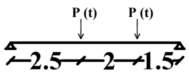
Md.l = (w * l^2) / 8 = 1.125 t.m

Qd.l = w * l / 2 = .75 t

Live loads

Max reaction of crane = 6.3 t
assume impact = 25 %

Case 1 (max moment)

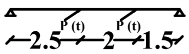


P = 6.3 t

Ml.l = 13.13 t.m

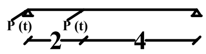
Ml.l+I = 1.25*13.13 = 16.413 t.m

*** Lateral shock**

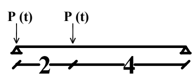


P1 = .1 * P = .63 t
My = .1 Ml.l = 1.313 t.m

Qx = .1 * 10.5 = 1.05 t



Case 2 (max shear)



Ql.l = 6.3 + 6.3 * (4 / 6) = 10.5 t

Ql.l+I = 1.25*10.5 = 13.13 t

Design values

Mx = Md.l + Ml.l+I = 1.125+16.413 = 17.538 t.m

My = 1.313 t.m

Qy = Qd.l + Ql.l+I = .75 + 13.13 = 13.88 t

Qx = 1.05 t

* Perliminary Design

$*$ Use H E A

assume $Zx = 3\ Zy$

$Mx / Zx + My / Zy < Fb$ assume $Fb = .58\ Fy = 1.4\ t / cm2$

$(17.54*100) / Zx + (1.313*100) / (Zx /3) < 1.4$ $Zx > 1543.03\ cm3$

Try H E A $\neq$340

* Checks

1 - check of stresses

$c/tf = .5*(bf-tw-2tf)/tf = .5*(30-.95-2*1.65)/1.65 =7.80< (16.9/\sqrt{Fy})=10.91$

$d / tw = (h - 4\ tf) / tw = (33 - 4 * 1.65) / .95 = 27.79 < 127 /\sqrt{Fy} = 81.98$

Section regarding to local buckling is compact

Lu act = 600 cm

$$Lu = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 30}{\sqrt{2.4}} = 387.3\ cm < Lu\ act$$

$$Fltb1 = \frac{800}{Lu * d / Af} * Cb = \frac{800*30*1.65*1.75}{600*33} = 3.5\ t / cm2 \ngtr .58 * Fy$$

$$Fbx = .58 * Fy = .58 * 2.4 = 1.392\ t / cm2$$

$$\boxed{Cb = 1.75 + 1.05 * (M1 / M2) + .3 * (M1 / M2)^2 = 1.75}$$

$$Fby = .72 * Fy = .72 * 2.4 = 1.728\ t / cm2$$

$$fact =(Mx/Zx)/Fbx+(My/Zy)/Fby$$

$$fact =(17.538*100/1678)/1.392+(1.313*100/496)/1.728= .904\ t/cm2 <1.00$$

Safe O.K .

2 - check of shear

$qy = Qy / (d * tw) = 13.88 / (33 * .95) = .44\ t / cm2 < .35 * Fy = .84\ t/ cm2$

$qx =(3/2)*(Qx /2*Af)=(3/2)*1.05/(2*30*1.65)=.0159\ t/cm2<.35*Fy=.84t/cm2$

Safe O.K .

3 - check of deflection

$\triangle_{all.} = L / 800 = 600 / 800 = .75\ cm$

$\triangle_{act.} \longrightarrow Ml.l+I = 16.413\ t.\ cm \longrightarrow Ml.l+I = (W * L^2) / 8$

$W = .0365\ t / cm$`

$\triangle_{act.} = (5 * .0365* 600^4) / (384 * 2100 * 27690) = 1.06\ cm$

unsafe

Try H E A $\neq$360

$\triangle$ act. = (5 * .0365* 600^4) / (384 * 2100 * 33090) = .86 cm

unsafe

Try H E A $\neq$400

$\triangle$ act. = (5 * .0365* 600^4) / (384 * 2100 * 45070) = .65 cm

Safe O.K .

$\boxed{\text{Use H E A} \neq 400}$

sec		D.L	L.L	CRANE LEFT			CRANE RIGHT			WIND		Mmax +ve N cor. Q cor.	case	Mmax -ve N cor. Q cor.	case	Mcor. N max+ve Q cor.	case	Mcor. N max-ve Q cor.	case
				C.L.1	C.L.3	C.L.5	C.L.2	C.L.4	C.L.6	left	right								
1	M	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	---		---		---		---	
	N	-2.71	-3.22	-14.25	-0.41	0.41	-5.63	0.41	-0.41	3.08	1.39	---		---		0.37	II	-20.18	I
	Q	-0.94	-1.11	-0.92	-0.76	0.76	-0.92	0.59	-0.59	2.59	-0.66	---		---		1.65	II	-2.97	I
2	M	5.64	6.69	5.50	4.54	-4.54	5.50	-3.56	3.56	-10.59	0.86	22.37	II	-4.95	II	-4.95	II	17.83	I
	N	-2.71	-3.22	-14.25	-0.41	0.41	-5.63	0.41	-0.41	3.08	1.39	-20.59	II	0.37	II	0.37	II	-20.18	I
	Q	-0.94	-1.11	-0.92	-0.76	0.76	-0.92	0.59	-0.59	0.94	0.37	-3.73	II	0.00	II	0.00	II	-2.97	I
3	M	0.00	0.00	-11.70	0.00	0.00	-4.20	0.00	0.00	0.00	0.00	0.00	II	-11.70	I	-11.70	II	-11.70	II
	N	0.00	0.00	0.00	-1.05	1.05	0.00	0.30	-0.30	0.00	0.00	-1.05	II	-1.05	I	1.05	II	-1.05	II
	Q	0.00	0.00	-14.63	0.00	0.00	-5.25	0.00	0.00	0.00	0.00	-14.63	II	-14.63	I	-14.63	II	-14.63	II
4	M	5.64	6.69	-6.20	4.54	-4.54	1.30	-3.56	3.56	-10.59	0.86	17.19	II	-15.69	II	-11.15	II	13.63	I
	N	-2.71	-3.22	0.38	-0.41	0.41	-0.38	0.41	-0.41	3.08	1.39	-6.72	II	1.16	II	0.75	II	-6.31	I
	Q	-0.94	-1.11	-0.92	0.29	-0.29	-0.92	0.29	-0.29	0.94	0.37	-3.38	II	-1.21	II	-0.92	II	-2.97	I
5	M	7.05	8.36	-4.83	4.10	-4.10	2.67	-4.00	4.00	-11.69	0.11	22.08	II	-13.57	II	-9.47	II	18.08	I
	N	-2.71	-3.22	0.38	-0.41	0.41	-0.38	0.41	-0.41	3.08	1.39	-6.72	II	1.16	II	0.75	II	-6.31	I
	Q	-0.94	-1.11	-0.92	0.29	-0.29	-0.92	0.29	-0.29	0.53	0.63	-3.26	II	-1.62	II	-1.33	II	-2.97	I
6	M	-7.05	-8.36	4.83	-4.10	4.10	-2.67	4.00	-4.00	11.69	-0.11	13.57	II	-22.08	II	---		-18.08	I
	N	-1.21	-1.43	-0.87	0.25	-0.25	-0.95	0.33	-0.33	0.83	0.77	-1.50	II	-3.92	II	---		-3.59	I
	Q	-2.61	-3.09	0.46	-0.43	0.43	-0.28	0.37	-0.37	3.01	1.32	1.29	II	-6.35	II	---		-5.98	I
7	M	2.65	3.14	2.50	-1.92	1.92	-1.25	2.13	-2.13	0.03	-4.57	10.21	II	-5.30	II	---		4.54	I
	N	-1.07	-1.27	-0.87	0.25	-0.25	-0.95	0.33	-0.33	0.83	0.77	-3.46	II	-1.58	II	---		-3.29	I
	Q	-1.26	-1.49	0.46	-0.43	0.43	-0.28	0.37	-0.37	1.63	0.46	-1.86	II	-1.45	II	---		-3.03	I
8	M	5.57	6.61	0.16	0.25	-0.25	0.16	0.25	-0.25	-4.69	-4.69	12.34	I	---	II	---		12.34	I
	N	-0.94	-1.11	-0.87	0.25	-0.25	-0.95	0.33	-0.33	0.83	0.77	-2.92	I	---	II	---		-3.00	I
	Q	0.09	0.11	0.46	-0.43	0.43	-0.28	0.37	-0.37	0.25	-0.41	0.66	I	---	II	---		-0.08	I

Design of Bracket

Max straining actions From table

M = 11.7 t.m **Q = 14.63 t**

*** neglect normal force and design as beam**

* Perliminary Design

Use I . P . E

Mx / Zx < Fb **1170 / Zx < 1.3**

Zx > 900 cm3

Try I . P . E ≠ 360

* Checks

1 - check of stresses

c/tf =.5*(bf-tw-2tf)/tf= .5* (17-.8-2*1.27)/1.27= 5.38 <(16.9/$\sqrt{Fy}$)=10.91

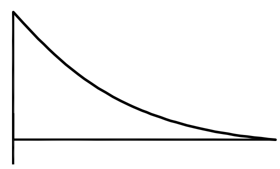
d / tw = (h - 4 tf) / tw = (36 - 4 * 1.27) / .8 = 38.65 < 127 / $\sqrt{Fy}$ = 81.98

Section regarding to local buckling is compact

Lu act = 80 cm

$$Lu1 = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 17}{\sqrt{2.4}} = 219.47 \text{ cm} > \text{Lu act}$$

$$Lu2 = \frac{1380\ Af}{d * Fy} * Cb$$



Cb = 1.75 **as æ = 0**

$$Lu2 = \frac{1380 * 17 * 1.27}{33.46 * 2.4} * 1.75 = 649.28 \text{ cm} > \text{Lu act}$$

Luact < Lu1 , Luact < Lu2

Fbx = .64 * Fy = 1.536 t / cm2

$$\frac{Mx / Zx}{Fb} = \frac{1170 / 904}{1.536} = .843 \text{ t / cm2} < 1.0$$

Safe O.K .

2 - check of shear

qy = Qy / (d * tw) = 14.63 / (36 * .8) = .51 t / cm2 < .35 * Fy = .84 t/ cm2

Safe O.K .

3 - check of deflection

$\triangle_{all.} = L / 800 = 80 / 800 = .1$ cm

$$\triangle_{act.} = \frac{pl^3}{3EI} + \frac{ql^4}{8EI}$$

q = o . w = 57.1 Kg / m = .0571 t / m   p = 14.63 t

Ix = 16270 cm4   E = 2100 t / cm2

$$\triangle_{act.} = \frac{14.63 * 80^3}{3 * 2100 * 16270} + \frac{.0571 * 80^4}{8 * 2100 * 16270} = .082 \text{ cm}$$

$\triangle_{all.} > \triangle_{act.}$

Safe O.K .

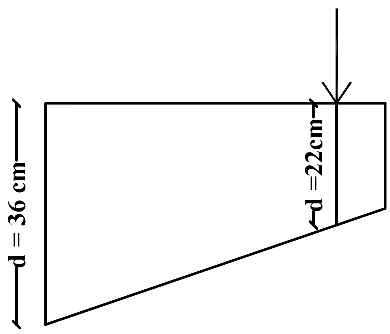
$\boxed{\text{Use I . P . E} \neq 360}$

---

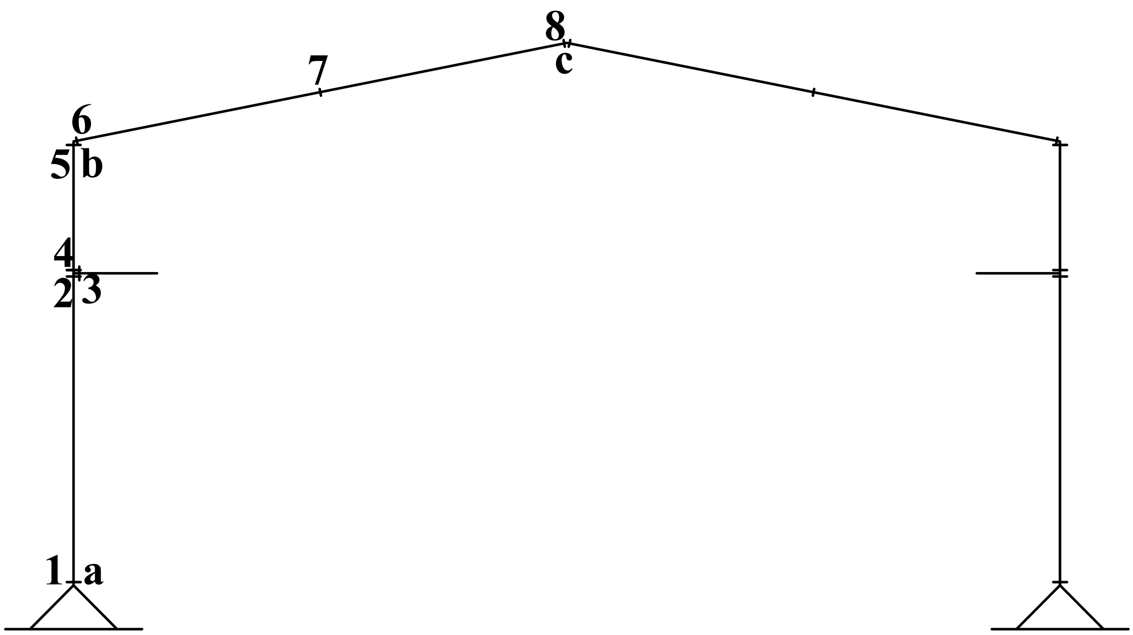
* If we use reduction in web depth

$$q_{all.} = \frac{Q}{d * tw} \qquad q_{all.} = .35 * Fy$$

$$.35 * 2.4 = \frac{14.63}{d * .8} \qquad d = 21.8 \text{ cm} \cong 22 \text{ cm}$$



Design of Frame



Design of Column ab

design from max straining action from table and then check other states

Mmax = 22.37 t. m (II)   Nmax = -20.59 t (II)   Q = 3.73 t

design from max straining action from table and then check other states

* Perliminary Design

*** Use H E A**

Mx / Zx < Fb   assume Fb = 1.3 t / cm2

(22.37*100) / Zx < 1.3   Zx > 1720.77cm3

Try H E A $\neq$360

A = 143 cm2   h = 35 cm   tw = 1 cm   bf = 30 cm   tf = 1.75 cm

Zx = 1891 cm3   ix = 15.2 cm   iy = 7.43 cm

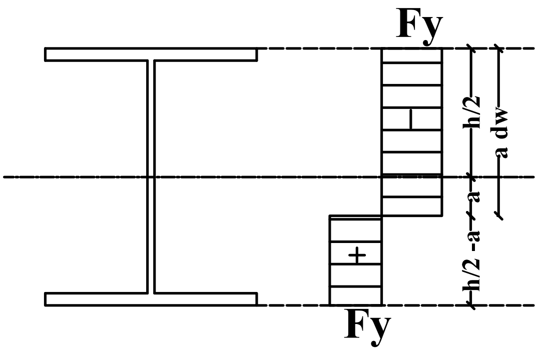
* Checks

check of stresses

c / tf = .5*(bf - tw) / tf = .5* (30-1) / 1.75 = 8.29 < (16.9/   Fy) = 10.91

web capacity = Fy * (hw -2tf) * tw =2.4*(35 - 2*1.75)*1= 75.6 t > 20.59 t

The plastic neutral axis is inside the web



N = ((h/2)+a)-((h/2)-a) * tw * Fy = 2a * tw * Fy

20.59 = 2a * 1 * 2.4 ⟶ a = 4.29 cm

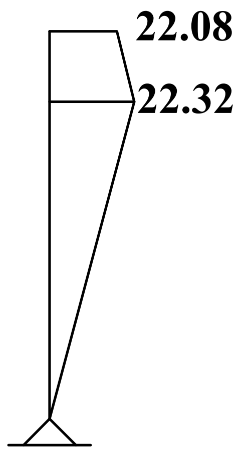
ædw = h/2 + a - 2tf ⟶ æ = .653 > .5

dw = h - 4tf = 35 - 4 * 1.75 = 28 cm

dw / tw = 28 / 1 = 28 < (699 / (Fy^.5)) /13 æ -1 = 60.24

Section regarding to local buckling is compact

Check of lateral torsional buckling



<u>* region 1</u>

æ = 0 Lu act = 600 cm

cb1 = 1.75

<u>* region 2</u>

æ = -22.08/22.37 = -.987 Lu act = 150 cm

cb1 = 1.75 + 1.05 æ +.3æ^2 = 1.01

*** Lu act = 150 cm**

$$Lu1 = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 30}{\sqrt{2.4}} = 387.3 \text{ cm} > \text{Lu act}$$

$$Lu2 = \frac{1380 * Af}{d * Fy} * Cb = \frac{1380*30*1.75*1.01}{35 * 2.4} = 871.13 \text{ cm} > \text{Lu act}$$

*** Lu act = 600 cm**

$$Lu1 = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 30}{\sqrt{2.4}} = 387.3 \text{ cm} < \text{Lu act}$$

Calculate Fltb

$$Fltb1 = \frac{800}{Lu * d / Af} * Cb = \frac{800 * 30 * 1.75 * 1.75}{600 * 35} = 3.50 \text{ t / cm2} \not> .58 * Fy$$

$$Fbx = .58 * Fy = .58 * 2.4 = 1.392 \text{ t / cm2}$$

*** to get Fc**

$$Ga = 10 \qquad Gb = \frac{\sum Ic / Lc}{\sum Ig / Lg} \qquad \text{assume } Ib = 2\, Ic$$

$$Gb = \frac{Ic / 7.5}{2Ic / 10.05} = .67$$

from chart Kx = 1.8 Ky = 1

$$\lambda x = \frac{Kx\, Lx}{ix} = \frac{1.8 * 750}{15.2} = 88.82$$

$$\lambda y = \frac{Ky\, Ly}{iy} = \frac{1 * 200}{7.43} = 26.92$$

$$\lambda max = 88.82$$

$$Fc = 1.7 - .000065\, \lambda^2 = .887 \text{ t / cm2}$$

$$fca = \frac{N}{A} = \frac{20.59}{143} = .144 \text{ t / cm2}$$

<u>* check of yielding   sec 1</u>

A1 = 1   Fc = 1.4 t / cm2   Fbcx = 1.392 t / cm2

N / A + A1 * ((Mx / Zx) / Fbcx) < 1.0

.144 / 1.4 + 1 * (0 / 1.536) = .103 < 1

Safe O.K .

<u>* check of instability   sec 4</u>

$$Fex = \frac{7500}{\lambda x ^ 2} = .951$$

$$A1 = \frac{Cmx}{1- fca / Fex} = 1.002$$

N / A + A1 * ((Mx / Zx) / Fbcx) < 1.0

.144 / .887 + 1.002 * ((2237/.85*1891) / 1.392) = 1.16 t / cm2 < 1 *1.2

Safe O.K .

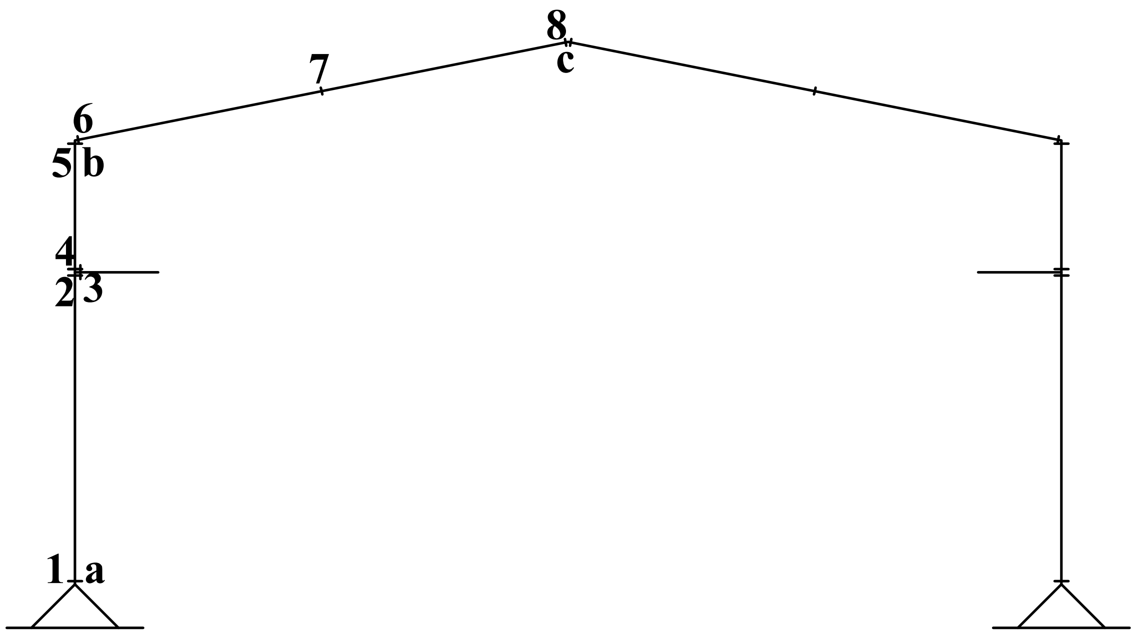
<u>* check of shear</u>

qy = Qy / (d * tw) = 3.73/ (35 * 1) = .107 t / cm2 < .35 * Fy = .84 t/ cm2

Safe O.K .

$\boxed{\text{Use H . E . A} \neq 360}$

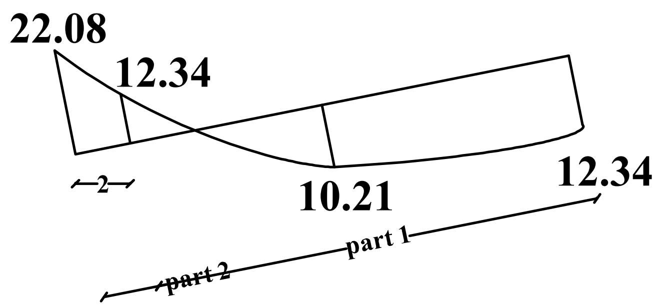
Design of Frame



Design of Rafter bc

design from max straining action from table and then check other states

*** neglect normal force and design as beam**



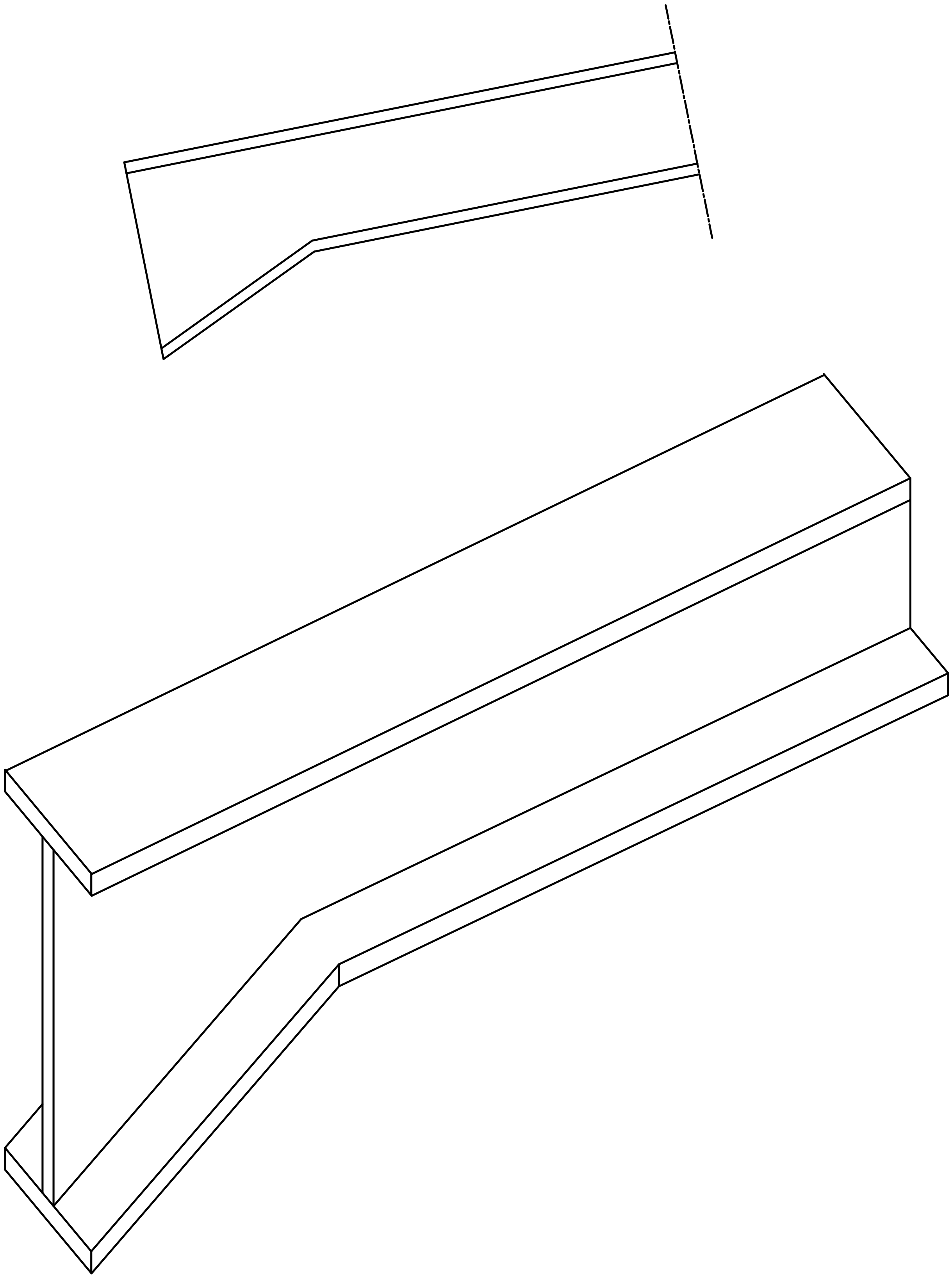
*** Design as built up section**

* Design of part 1

M max = 12.34 t.cm (II) **Q = 3.03 ton**

$$h > \frac{L}{24}$$

take h = 30 cm **tw = 1cm**



isometry for rafter beside column

Check of buckling due to shear

$æ = 0.0$

$$kq = 4 + \frac{5.34}{æ^2} = 4$$

$$\lambda q = \frac{d / tw}{57} * \sqrt{\frac{Fy}{kq}} = .75$$

$qall = .35\ Fy = .84\ t / cm2$

$$qact = \frac{Qt}{d * tw} = \frac{6.35}{55 * 1} = .115\ t / cm2 \quad < qall$$

Safe O.K .

$$Af = \frac{M}{.97*h*Fbc} = \frac{12.34 * 100}{.97 * 30 * 1.3} = 32.62\ cm2$$

Use $bf = 20cm$ $\qquad tf = \frac{32.62}{20} = 1.63\ cm$

take $tf = 1.8\ cm$

---

check of stresses

$c / tf = .5*(bf - tw) / tf = .5* (20-1-2*1) / 1.8 = 4.72 < 15.3 / \sqrt{Fy} = 9.88$

$d / tw = (30-2*1.8-2*1) / 1 = 24.4 < 127 / \sqrt{Fy} = 81.98$

Section regarding to local buckling is compact

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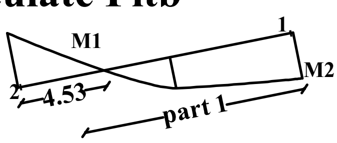
* Sec 1

Lu act1 = 201 cm

Calculate Fltb



$$Lu = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 20}{\sqrt{2.4}} = 258.2\ cm > Lu\ act$$

$$æ = \frac{M1}{M2} = \frac{-10.72}{12.34} = -.87$$

$Cb = 1.75 + 1.05 * (M1 / M2) + .3 * (M1 / M2)^2 = 1.06$

$cb = 1.06$

$$Lu2 = \frac{1380 * Af}{d * Fy} * Cb = \frac{1380*20*1.8*1.06}{30 * 2.4} = 731.4\ cm > Lu\ act$$

* Sec 2

Lu act1 (from sap) = 453 cm

$$Lu = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 20}{\sqrt{2.4}} = 258.2 \text{ cm} < \text{Lu act}$$

$$æ = \frac{M1}{M2} = 0$$

Cb = 1.75 + 1.05 * (M1 / M2) + .3 * (M1 / M2)^2 $\ngtr$ 2.3

cb = 1.75

$$Fltb1 = \frac{800}{Lu * d / Af} * Cb = \frac{800*20*1.8*1.75}{453 * 50} = 2.23 \text{ t / cm2} \leqslant .58 \text{ Fy}$$

Fbx = .58* Fy = .58 * 2.4 = 1.392 t / cm2

$$Ix = \frac{1 * 26.4^3}{12} + 2 * 20 * 1.8 * (28.2/2)^2 = 15847.6 \text{ cm4}$$

$$y = \frac{htotal}{2} = \frac{dw + 2tf}{2} = 15 \text{ cm}$$

Zx = Ix / y = 15847.6/ 15 = 1056.51 cm3

Mx / Zx < Fb $\longrightarrow$ 12.34*100 / (.85*1056.51) = 1.374 t /cm2

< 1.392*1.2=1.6704 t/cm2

safe o.k.

* Design of part 2

M max = 22.08 t.cm (II)   Q = 6.35 ton

$$Af = \frac{M}{.97*h*Fbc} = \frac{22.08 * 100}{.97 * h * 1.3} \longrightarrow h = 50 \text{ cm}$$

h = 50 cm   tw = 1 cm   bf = 20 cm   tf = 1.8 cm

Check of buckling due to shear

æ = 0.0

$$kq = 4 + \frac{5.34}{æ \string^ 2} = 4$$

$$\lambda q = \frac{d / tw}{57} * \sqrt{\frac{Fy}{kq}} = .68$$

qall = .35 Fy = .84 t / cm2

$$qact = \frac{Qt}{d * tw} = \frac{6.35}{50 * 1} = .127 \text{ t / cm2} < qall$$

Safe O.K .

Check of stresses

$$Ix = \frac{1 * 46.4 \string^ 3}{12} + 2 * 20 * 1.8 * (48.2/2) \string^ 2 = 50143.1 \text{cm4}$$

$$y = \frac{htotal}{2} = \frac{dw + 2tf}{2} = 25 \text{ cm}$$

Zx = Ix / y = 50143.1 / 25 = 2005.72 cm3

Mx / Zx < Fb ⟶ 22.08*100/(.85*2005.72)=1.295 t/cm2

<1.392 *1.2=1.67 t/cm2

safe o.k.

Design details

Presist > Papplied

$$2 * Sw * qw * 1.0 = \frac{Qt * Sf}{Ix}$$

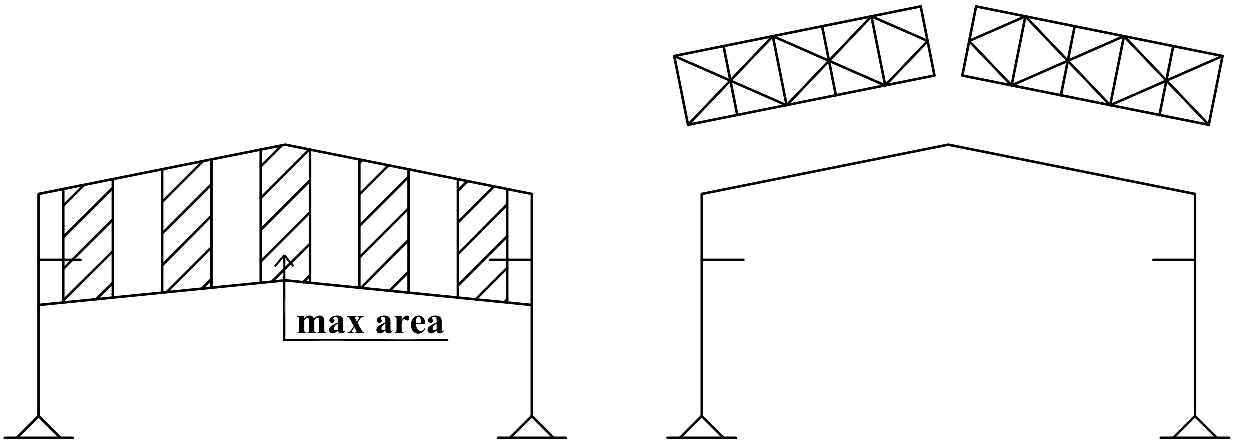
$$2 * Sw * .2 * 3.6 * 1.0 = \frac{6.35 * 20 * 1.8 * 14.1}{15847.6}$$

Sw = .14 cm **take Sw = Smin = 4 mm**

Use Built Up Section with
h = 30 / 50 cm tw = 1 cm bf = 20 cm tf = 1.8 cm
thickness of weld bet. flange and web (S) = 4 mm

Design of Wind Bracing

1 - HL (upper) wind bracing

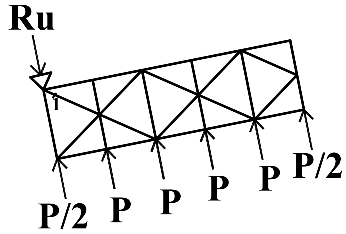


q = 57.34 kg / m2 k = 1

P = Ce * k * q * area served = Ce * 1 * (57.34/1000) * 2 * 4.25 = 0.39 t

Ru = 5 * P = 5 * .39 = 1.95 t

D.F1 = From sap = 1.4 ton



$L = L_x = \sqrt{2^2 + 3^2} = 3.61$ m

$L_y = 2 * .6 * L = 3.33$ m

* Perliminary Design

Use 2Ls BTB

* $\frac{L_x}{i_x} < 200$, $i_x = .3\,a \longrightarrow a > 6.017$ cm
* $\frac{L}{d} < 60$, $d = a \longrightarrow a > 6.017$ cm
* $A_{req.}\ 1L = \frac{D.F}{2 * .85 * F_{all.}} = \frac{1.40}{2 * 1.4 * .85} = .59$ cm2
* $a - t > 3\,Ø$, $Ø = 16$ mm $\longrightarrow a > 5.33$ cm

* Try 2 Ls 70 * 7

Checks

1 - stiffness condition

$$\frac{L}{d} = \frac{361}{7} = 51.57 < 60 \qquad \text{safe o.k.}$$

$$\frac{Lx}{ix} < 200 \qquad ix = \sqrt{\frac{Ix}{A}} = \sqrt{\frac{2 * 42.4}{2 * 9.4}} = 2.12$$

$$\frac{361}{2.12} = 169.98 < 200 \qquad \text{safe o.k.}$$

2 - Construction condition

a - t = 7 - .7 = 6.93 cm > 3Ø = 48 cm   safe o.k.

3 - check of stresses

Anet = 2 (A1l - (Ø+ .2) * t) = 2 * (9.40 - 2.2 * .7) = 15.72 cm2

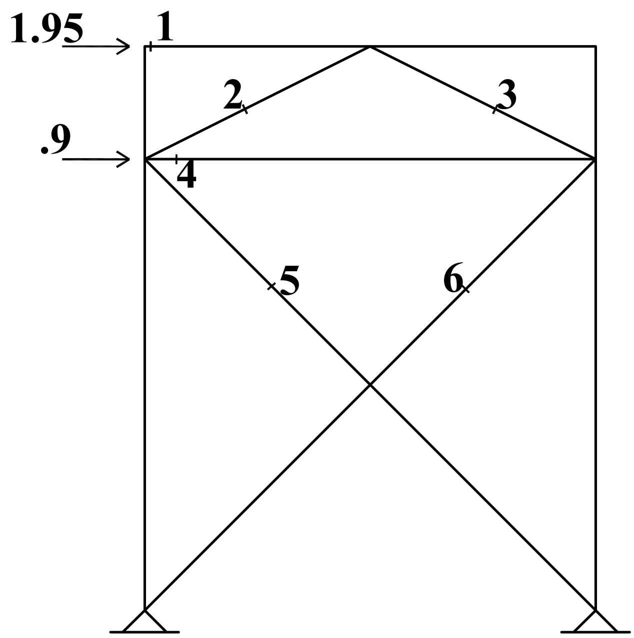
$$\text{Fact.} = \frac{D . F}{Anet} = \frac{1.4}{15.72} = .09 \text{ t / cm2} < \text{Fall.} = 1.4 \text{ t / cm2}$$

* note that the stiffness and construction requirements control the design

$\boxed{\text{Use 2 Ls 70 * 7}}$

Design of Wind Bracing

1 - VL (transversal) wind bracing



member 1

D.F1 = From sap = 1.95 ton

L = Lx = 3 m

Ly = 2 * .75 * L = 4.5 m

* Perliminary Design

Use 2Ls BTB

* $\frac{Lx}{ix} < 200$, ix = .3 a ⟶ a > 5 cm
* $\frac{L}{d} < 60$, d = a ⟶ a > 5 cm
* Areq. 1L = $\frac{D.F}{2 * .85 * Fall.} = \frac{1.95}{2 * 1.4 * .85}$ = .82 cm2
* a - t > 3 Ø , Ø = 16 mm ⟶ a > 5.33 cm

* Try 2 Ls 60 * 6

Checks

1 - stiffness condition

$$\frac{L}{d} = \frac{300}{6} = 50 < 60 \qquad \text{safe o.k.}$$

$$\frac{Lx}{ix} < 200 \qquad ix = \sqrt{\frac{Ix}{A}} = \sqrt{\frac{2 * 22.8}{2 * 6.91}} = 1.82$$

$$\frac{300}{1.82} = 164.84 < 200 \qquad \text{safe o.k.}$$

$$Iy = 2 * (22.8 + 6.91 * (1.69 + .5)^2) = 111.88 \text{ cm4}$$

$$iy = \sqrt{\frac{Iy}{A}} = \sqrt{\frac{111.88}{2 * 6.91}} = 2.85$$

$$\frac{450}{2.85} = 157.89 < 200 \qquad \text{safe o.k.}$$

2 - Construction condition

a - t = 6 - .6 = 5.4 cm > 3 Ø= 4.8 cm safe o.k.

3 - check of stresses

Anet = 2 (A1l - (Ø+ .2) * t) = 2 * (6.91 - 1.8 * .6) = 11.66 cm2

$$Fact. = \frac{D . F}{Anet} = \frac{1.95}{11.66} = .17 \text{ t / cm2} < Fall. = 1.4 \text{ t / cm2}$$

* note that the stiffness and construction requirements control the design

Use 2 Ls 60 * 6

members 2 & 3

D.F1 = From sap = 1.09 ton

$L = Lx = \sqrt{1.5^2 + 3^2} = 3.35$ m

$Ly = 2 * .6 * L = 4.02$ m

*** Perliminary Design**

Use 2Ls BTB

* $\frac{Lx}{ix} < 200$, ix = .3 a $\longrightarrow$ a > 5.58 cm

* $\frac{L}{d} < 60$, d = a $\longrightarrow$ a > 5.58 cm

* $\text{Areq. 1L} = \frac{D.F}{2 * .85 * Fall.} = \frac{1.09}{2 * 1.4 * .85} = .46 \text{ cm2}$

* a - t > 3 Ø , Ø = 16 mm $\longrightarrow$ a > 5.33 cm

* Try 2 Ls 60 * 6

Checks

1 - stiffness condition

$\frac{L}{d} = \frac{335}{6} = 55.83 < 60$ safe o.k.

$\frac{Lx}{ix} < 200$ $ix = \sqrt{\frac{Ix}{A}} = \sqrt{\frac{2 * 22.8}{2 * 6.91}}$

$\frac{335}{1.82} = 184.42 < 200$ safe o.k.

$Iy = 2 * (22.8 + 6.91 * (1.69 + .5)^2) = 111.88$ cm4

$iy = \sqrt{\frac{Iy}{A}} = \sqrt{\frac{111.88}{2 * 6.91}} = 2.85$

$\frac{402}{2.85} = 141.05 < 200$ safe o.k.

2 - Construction condition

a - t = 6 - .6 = 5.4 cm > 3 Ø= 4.8 cm

3 - check of stresses

Anet = 2 (A1l - (Ø+ .2) * t) = 2 * (6.91 - 1.8 * .6) = 11.66 cm2

$$\text{Fact.} = \frac{D\ .\ F}{\text{Anet}} = \frac{1.09}{11.66} = .09 \ t / cm2 < \text{Fall.} = 1.4 \ t / cm2$$

*** note that the stiffness and construction requirements control the design**

$$\boxed{\text{Use 2 Ls 60 * 6}}$$

member 4

D.F1 = From sap = .4 ton

L = Lx = 6 m

Ly = 6 m

Try 2 Cs No 12

Checks

1 - stiffness condition

$$\frac{L}{d} = \frac{600}{12} = 50 < 60$$ safe o.k.

$$Iy = 2 * (43.2 + 17 * (1.6 + .5)^2) = 236.34 \text{ cm4}$$

$$iy = \sqrt{\frac{Iy}{A}} = \sqrt{\frac{236.34}{2 * 17}} = 2.64$$

$$\frac{600}{2.64} = 227.6 > 200$$ not safe

Try 2 Cs No 16

Checks

1 - stiffness condition

$$\frac{L}{d} = \frac{600}{16} = 37.5 < 60$$ safe o.k.

$$Iy = 2 * (85.3 + 24 * (1.84 + .5)^2) = 433.43 \text{ cm4}$$

$$iy = \sqrt{\frac{Iy}{A}} = \sqrt{\frac{433.43}{2 * 24}} = 3.005$$

$$\frac{600}{3.01} = 199.67 < 200$$ safe o.k.

* No need to check stresses as D.F. is very small and stifness condition control the design

$\boxed{\text{Use 2 Cs No 16}}$

members 5 & 6

D.F1 = From sap = 2.09 ton

$L = Lx = \sqrt{3^2 + 3^2} = 4.24$ m

$Ly = 2 * .6 * L = 5.09$ m

*** Perliminary Design**

Use 2Ls BTB

* $\frac{Lx}{ix} < 200$, $ix = .3\,a \longrightarrow a > 7.07$ cm
* $\frac{L}{d} < 60$, $d = a \longrightarrow a > 7.07$ cm
* $Areq.\ 1L = \frac{D.F}{2 * .85 * Fall.} = \frac{2.09}{2 * 1.4 * .85} = .88$ cm2
* $a - t > 3\,Ø$, $Ø = 16$ mm $\longrightarrow a > 5.33$ cm

* Try 2 Ls 80 * 8

Checks

1 - stiffness condition

$\frac{L}{d} = \frac{424}{8} = 53 < 60$ safe o.k.

$\frac{Lx}{ix} < 200$ $ix = \sqrt{\frac{Ix}{A}} = \sqrt{\frac{2 * 72.3}{2 * 12.3}} = 2.42$

$\frac{424}{2.42} = 174.88 < 200$ safe o.k.

2 - Construction condition

$a - t = 8 - .8 = 7.2$ cm $> 3\,Ø = 4.8$ cm

3 - check of stresses

Anet = 2 (A1l - (Ø+ .2) * t) = 2 * (12.3 - 1.8 * .8) = 21.72 cm2

$$\text{Fact.} = \frac{D\,.\,F}{\text{Anet}} = \frac{2.09}{21.72} = .096\ t / cm2 < \text{Fall.} = 1.4\ t / cm2$$

safe o.k.

*** note that the stiffness and construction requirements control the design**

$\boxed{\text{Use 2 Ls 80 * 8}}$

Design of Side Girt

own weight = 20 Kg / m\`
steel cover = 8 Kg / m\`

q = 57.34 kg / m2 k = 1 ce = .8

Wx = Wwind = ce * k * q * a = .8 * 1 * 57.34 *2 /1000 = .092 t / m\`

$$Mx = \frac{w\, l^2}{8} = .41 \text{ t.m}$$

Wy due to dead load only

Wy = wc * a + o.w = (8/1000)*2 + 20/1000 = .036 t / m\`

$$My = \frac{w\, l^2}{8} = .162 \text{ t.m}$$

Use U . P . N

*** Perliminary Design**

assume Zx = 7 Zy

Mx / Zx + My / Zy < Fb assume Fb = 1.3 t / cm2

41 / Zx + 16.2 / (Zx /7) < 1.3 Zx > 118.77 cm3

Try U . P . N $\neq$ 160

*** Checks**

1 - check of stresses

c/tf = .5*(bf - tw -2tf)/tf=.5*(6.5-.75-2*.1.05)/.75=14.85 < (23/$\sqrt{Fy}$)=14.85
d / tw = (h - 4 tf) / tw = (16 - 4 * 1.05) / .75 = 15.73 < 190 / $\sqrt{Fy}$ = 122.64
Section regarding to local buckling is non compact

Lu act = 600 cm

$$Lu = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 6.5}{\sqrt{2.4}} = 83.91 \text{ cm} < Lu \text{ act} \qquad \text{Calculate Fltb}$$

cb = 1.13 from table

$$Fltb1 = \frac{800}{Lu * d / Af} * Cb = \frac{800*6.5*1.05*1.13}{600*11.8} = .87 t / cm2 < .58 Fy$$

*** Calculate Fltb2**

$$84 * \sqrt{\frac{Cb}{Fy}} = 84 * \sqrt{\frac{1.13}{2.4}} = 57.64$$

$$188 * \sqrt{\frac{Cb}{Fy}} = 188 * \sqrt{\frac{1.13}{2.4}} = 129$$

$$rt = \sqrt{\frac{Iy}{A}}$$

$$Iy = \frac{tf * bf ^3}{12} + \frac{(hw / 6) * tw ^3}{12} = 24.11 \text{ cm4}$$

$$A = bf\ tf + (hw\ tw)/6 = 8.56 \text{ cm2}$$

$$rt = \sqrt{\frac{Iy}{A}} = \sqrt{\frac{24.11}{8.56}} = 1.678$$

$$\frac{lu}{rt} = \frac{600}{1.678} = 357.57$$

$$\frac{lu}{rt} = 357.57 > 188 * \sqrt{\frac{Cb}{Fy}}$$

$$Fltb2 = \frac{12000 * cb}{(Lu / rt)^2} = .107 \text{ t / cm2}$$

$$Fltb = \sqrt{Fltb1^2 + Fltb2^2} = .88 \text{ t /cm2}$$

$$Fbx = Fltb = .88 \text{ t / cm2}$$

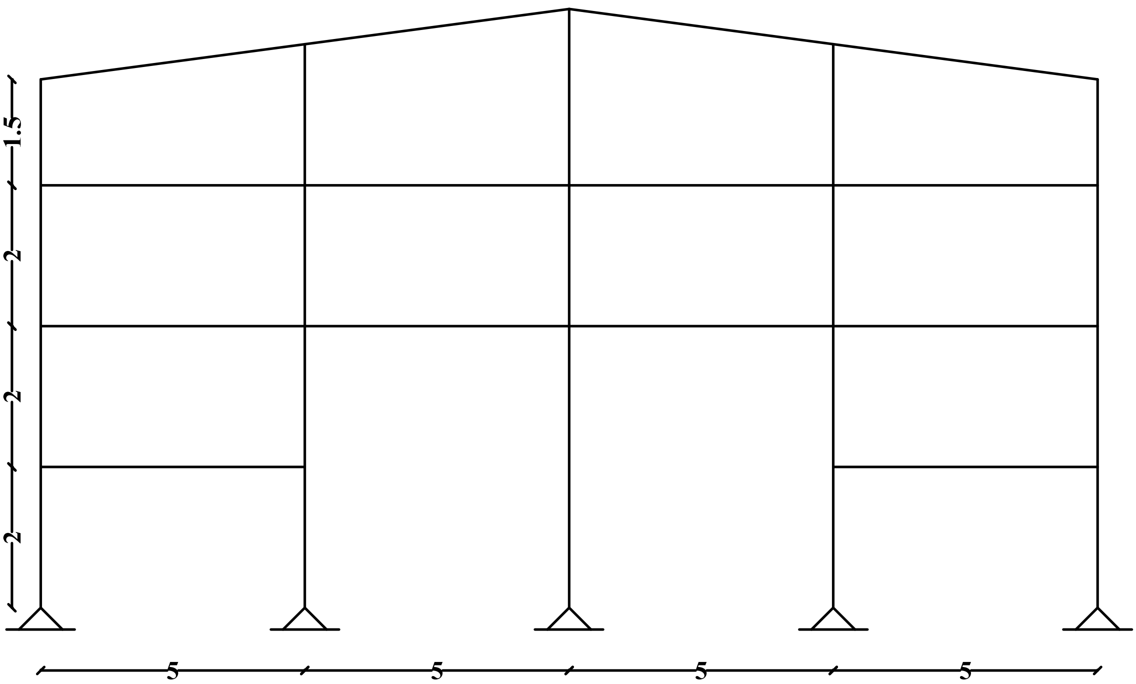
$$Fby = .58 * Fy = .58 * 2.4 = 1.392 \text{ t / cm2}$$

$$(Mx/Zx)/Fbx+(My/Zy)/Fby=(41/116)/.88+(16.2/18.3)/1.392 =1 \text{ t/cm2}<1.00$$

Safe O.K .

$\boxed{\text{Use U . P . N} \neq 160}$

Design of End gable

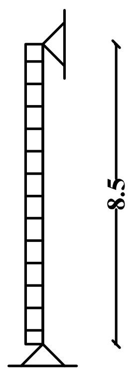


q = 57.34 kg / m2   k = 1   ce = .8

W= Wwind = ce * k * q * s * h = .8 * 1 * 57.34 * 5 * 8.5 /1000 = .23 t / m

M = wl^2 / 8 = 2.1 m.t

N = wc * s * h + o.w*h + w.t of girts * 4 * s = 1.17 t



* Perliminary Design

* Use I . P . E

Mx / Zx < Fb   assume Fb = 1.3 t / cm2

(2.1 * 100) / Zx < 1.3   Zx > 161.54 cm3

Try I P E $\neq$200

A = 28.5 cm2   h = 20 cm   tw = .56 cm   bf = 10 cm   tf = .85 cm

Zx = 194 cm3   ix = 8.25 cm   iy = 2.23 cm

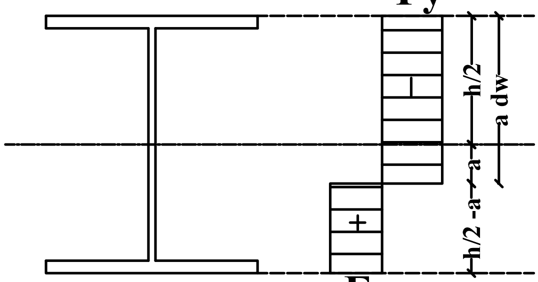
* Checks

check of stresses

c/tf=.5*(bf-tw-2tf)/tf=.5*(10-.56-2*.85)/.85=4.55< (16.9/ $\sqrt{Fy}$)=10.91

web capacity = Fy * (hw -2tf) * tw =2.4*(20 - 2*.85)*.56=24.6 t > 1.17 t

The plastic neutral axis is inside the web



N = ((h/2)+a)-((h/2)-a) * tw * Fy = 2a * tw * Fy

1.17 = 2a * .56 * 2.4 $\longrightarrow$ a = .44 cm

ædw = h/2 + a - 2tf $\longrightarrow$ æ = .53 > .5

dw = h - 4tf = 20 - 4 * .85 = 16.6 cm

dw / tw = 16.6 / .56 = 29.64 < (699 / (Fy^.5)) /13æ -1 = 76.6

Section regarding to local buckling is compact

Lx = 8.5 m Ly = 4 m

$$\lambda x = \frac{Kx\ Lx}{ix} = \frac{1 * 850}{8.25} = 103.03$$

$$\lambda y = \frac{Ky\ Ly}{iy} = \frac{1 * 400}{2.23} = 179.37$$

$$\lambda max = 179.37$$

Fc = 7500 / λ^2 = .233 t / cm2

$$fca = \frac{N}{A} = \frac{1.17}{28.5} = .04 \text{ t / cm2}$$

*** Lu act = 400 cm**

$$Lu1 = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 10}{\sqrt{2.4}} = 129.1 \text{ cm} < \text{Lu act}$$

$$Fltb1 = \frac{800 * Af}{d * Lu} * Cb = \frac{800*10*.85*1.75}{400 * 16.6} = 1.79 \text{ t / cm2} \ngtr .58\ Fy$$

Fbx = .64 * Fy = .58 * 2.4 =1.392 t / cm2

$$Fex = \frac{7500}{\lambda x \text{ ^ } 2} = .233$$

$$A1 = \frac{Cmx}{1 - fca / Fex} = 1.03$$

$$N / A + A1 * ((Mx / Zx) / Fbcx) < 1.0$$

$$.04 / .233 + 1.03 * ((210/194) / 1.392) = .84 \ t / cm2 < 1$$

Safe O.K .

$\boxed{\text{Use I . P . E} \neq 200}$

Design of Girt

own weight = 20 Kg / m`
steel cover = 8 Kg / m`

q = 57.34 kg / m2 **k = 1** **ce = .8**

Wx = Wwind = ce * k * q * a = .8 * 1 * 57.34 *3 /1000 = .138 t / m`

$$Mx = \frac{w\, l^2}{8} = .43 \text{ t.m}$$

Wy due to dead load only

Wy = wc * a + o.w = (8/1000)*3 + 20/1000 = .044 t / m`

$$My = \frac{w\, l^2}{8} = .138 \text{ t.m}$$

Use U . P . N

*** Perliminary Design**

assume Zx = 7 Zy

Mx / Zx + My / Zy < Fb **assume Fb = 1.3 t / cm2**

43 / Zx + 11.3 / (Zx /7) < 1.3 **Zx > 93.92 cm3**

Try U . P . N $\neq$ 160

*** Checks**

1 - check of stresses

$c/tf = .5*(bf - tw - 2tf)/tf = .5*(6.5-.75-2*.1.05)/.75 = 14.85 < (23/\sqrt{Fy}) = 14.85$

$d / tw = (h - 4\, tf) / tw = (16 - 4 * 1.05) / .75 = 15.73 < 190 / \sqrt{Fy} = 122.64$

Section regarding to local buckling is non compact

Lu act = 500 cm

$$Lu = \frac{20 * bf}{\sqrt{Fy}} = \frac{20 * 6.5}{\sqrt{2.4}} = 83.91 \text{ cm} < Lu\ act$$ **Calculate Fltb**

cb = 1.13 from table

$$Fltb1 = \frac{800}{Lu * d / Af} * Cb = \frac{800*6.5*1.05*1.13}{500*11.8} = 1.05 \text{ t/cm2} < .58\ Fy$$

*** Calculate Fltb2**

$$84 * \sqrt{\frac{Cb}{Fy}} = 84 * \sqrt{\frac{1.13}{2.4}} = 57.64$$

$$188 * \sqrt{\frac{Cb}{Fy}} = 188 * \sqrt{\frac{1.13}{2.4}} = 129$$

$$rt = \sqrt{\frac{Iy}{A}}$$

$$Iy = \frac{tf * bf ^3}{12} + \frac{(hw / 6) * tw ^3}{12} = 24.11 \text{ cm4}$$

$$A = bf\ tf + (hw\ tw)/6 = 8.56 \text{ cm2}$$

$$rt = \sqrt{\frac{Iy}{A}} = \sqrt{\frac{24.11}{8.56}} = 1.678$$

$$\frac{lu}{rt} = \frac{500}{1.678} = 297.97$$

$$\frac{lu}{rt} = 297.97 > 188 * \sqrt{\frac{Cb}{Fy}}$$

$$Fltb2 = \frac{12000 * cb}{(Lu / rt)^2} = .153 \text{ t / cm2}$$

$$Fltb = \sqrt{Fltb1^2 + Fltb2^2} = 1.06 \text{ t /cm2}$$

$$Fbx = Fltb = 1.06 \text{ t / cm2}$$

$$Fby = .58 * Fy = .58 * 2.4 = 1.392 \text{ t / cm2}$$

$$(Mx/Zx)/Fbx+(My/Zy)/Fby=(43/116)/1.06+(13.8/18.3)/1.392=.89 \text{ t/cm2}<1.00$$

Safe O.K .

$$\boxed{\text{Use U . P . N} \neq 160}$$

Design of Hinged base connection

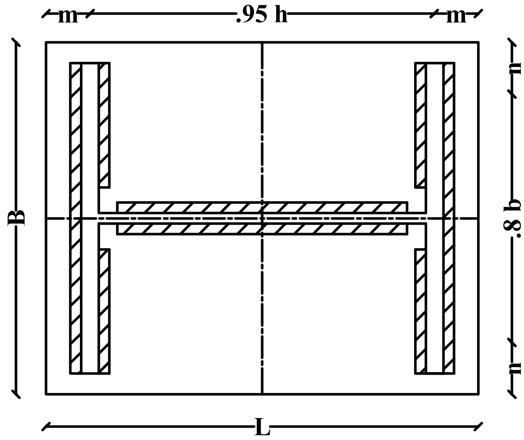
Max straining actions From table

$Nc = -20.18$ t $Xc = 2.97$ t

$Nc = .37$ t $Xc = 2.97$ t

Column section H . E . A $\neq$360

$h = 35$ cm $b = 30$ cm



$L = 35 + 4 = 39$ cm

$B = 30 + 4 = 34$ cm

1 - Check of concrete material

$$Wb = \frac{N}{B * L} = \frac{20.18 * 1000}{39 * 34} = 15.22 \text{ Kg /cm2} < \text{Fall}$$

Fall = 70 kg / cm2 for Fcu = 300 kg / cm2

2 - Design of plate

$$n = \frac{B - .8b}{2} = 5 \text{ cm}$$

$$m = \frac{L - .95h}{2} = 2.875 \text{ cm}$$

Mmax = Wb * (n^2 / 2) = 190.234 Kg.cm/cm = .19 t.cm/cm

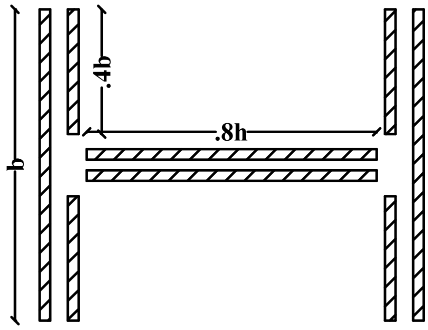
$\sigma = MY / I$ $Y = tp / 2$ $I = (1 * tp^3) / 12$

$\sigma = .72$ Fy = 1.728 t / cm2

$$tp = \sqrt{\frac{6 M}{\sigma}} = \sqrt{\frac{6 * .19}{1.728}} = .82 \text{ cm}$$

Use tpmin = 2 cm

3 - design of weld



assume Sw = 6 mm

Ahl = 2 * .8 * 35 * .6 = 33.6 cm2

Avl = 4 * .4 * 30 * .6 + 2 * 30 * .6 = 64.8 cm2

Atot. = Avl + Ahl = 98.4 cm2

$$\text{Fweld} = \frac{.6\ N}{\text{Atot.}} = \frac{.6 * 20.18}{98.4} = .123\ \text{t/cm2} < .2\text{Fu}$$

$$\text{qweld} = \frac{Q}{\text{Avl}} = \frac{2.97}{33.6} = .09\ \text{t/cm2} < .2\text{Fu}$$

$$F = \sqrt{\text{Fweld}^2 + 3 * \text{qweld}^2}$$

$$= \sqrt{.123^2 + 3 * .09^2} = .199\ \text{t / cm2} < 1.1 * .2\text{Fu}$$

$$\text{Fweld} = \frac{N}{\text{Atot.}} = \frac{.37}{98.4} = .123\ \text{t/cm2} < .2\text{Fu}$$

$$\text{qweld} = \frac{Q}{\text{Avl}} = \frac{2.97}{33.6} = .09\ \text{t/cm2} < .2\text{Fu}$$

$$F = \sqrt{\text{Fweld}^2 + 3 * \text{qweld}^2}$$

$$= \sqrt{.004^2 + 3 * .09^2}$$

$$= .156\ \text{t / cm2} < 1.1 * .2\text{Fu}$$

---

4 - design of bolts

assume we use two bolts Steel 36 / 52

$$Q = 2 * \frac{\pi Ø^2}{\text{Atot.}} * .25\ \text{Fu}$$

$$Ø = \sqrt{\frac{2.97}{2 * (\pi / 4) * .25 * 5.2}} = 1.21\ \text{cm}$$

Use 2 Ø 16 grade 36 / 52

---

5 - bolt length

Compression

L = 20 Ø = 32 cm

Tension

$$L = \frac{T}{2 * \pi * Ø * qb}$$

$$qb = .95\sqrt{\frac{Fcu}{\gamma c}} = .95\sqrt{\frac{300}{1.5}} = .0134 \text{ t / cm2}$$

$$L = \frac{.37}{2 * 3.14 * 1.6 * .0134} = 2.74 \text{ cm}$$

Use Lmax = 32 cm

Design Of Hinged Base of end gable

project		ENG /	

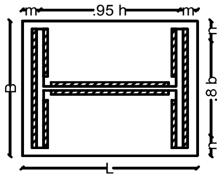
Fy =	2.40	t/cm^2	
Fcu =	300	Kg/cm^2	
Nc =	1.17	ton	case1
Nt =	0.00	ton	case2

Use	st 37	
bolt grade	st36/52	
Q =	0.98	ton
Q =	0.98	ton

column section properties

h =	20.00	cm
b =	10.00	cm

t_w =	0.56	cm
t_f =	0.85	cm



L =	24.00	cm
B =	14.00	cm

Check of concrete

compression

Wb =	3.482	Kg/cm^2
Fall. =	70	Kg/cm^2

safe

tention

Wb =	0	Kg/cm^2
Fall. =	20	Kg/cm^2

safe

Design of plate

n =	3	cm
m =	2.5	cm
Mmax =	0.016	t.cm/cm

tp =	2	cm

Design of weld

Sw =	0.5	cm

Ahl weld =	16	cm^2
Avl weld =	18	cm^2
Atotal weld =	34	cm^2

compression

Fweld =	0.021	t / cm^2
qweld =	0.061	t / cm^2

F =	0.108	t / cm^2

check of weld safe

tension

Fweld =	0	t / cm^2
qweld =	0.0613	t / cm^2

F =	0.1061	t / cm^2

check of weld safe

Design of bolt

req. dia(Ø) =	0.693	cm
selected dia=	16	mm

use 2Ø16 grade st36/52

Bolt length

compression

L =	32	cm

tension

L =	0	cm

Use 2Ø16 grade st36/52 with length 32 cm

Design Of Column

a - Frame Column

N = 20.18 t

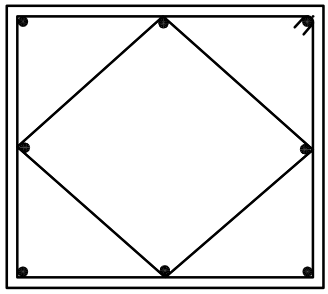
Pu = .35 * Fcu * Ac + .67 * Fy * As

20.18 * 10^3 = .35 * 300 * 40 * 45 + .67 * 3600 * As

As = -ve

Use As min = .8 % * Ac = (.8 / 100) * 40 *45 = 14.4 cm2

Use 8 Ø 16



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b - End Gable Column

N = 1.17 t

Pu = .35 * Fcu * Ac + .67 * Fy * As

1.17 * 10^3 = .35 * 300 * 40 * 45 + .67 * 3600 * As

As = -ve

Use As min = .8 % * Ac = (.8 / 100) * 20 *30 = 4.8 cm2

Use 4 Ø 16

Design Of Foundation

a - Frame Column

qall = 12 t / m2 Foundation Level = 1.5 m

Use Rc Column With Hight = 1.5 + .25 = 1.75 m

N = 20.18 t , X = 2.97 t

Column Section = (34+6)* (39+6) = 45 * 40

Plain Concrete

$$Area = A * B = \frac{1.5\ Pg.s}{qa} = \frac{1.5 * 20.18}{12} = 2.53\ m2$$

$$A = \sqrt{Area} = 1.6\ m \cong 2\ m$$

$$Mf.l = x * df = 2.97 * 1.5 = 4.46\ t.m \qquad Pf.l = 22.08\ t$$

$$e = \frac{Mf.l}{Pf.l} = \frac{4.46}{23.21} = .192\ m < A/6 = 2/6 = .33\ m$$

$$qmax = qa = \frac{Pf.l}{A * B}\left[1 + \frac{6\ e}{A}\right]$$

$$qa = \frac{1.15*20.18}{2 * B}\left[1 + \frac{6*.192}{2}\right] \longrightarrow B = 1.6\ m$$

$$q1 = \frac{1.15*20.18}{2 * 1.6}\left[1 + \frac{6*.192}{2}\right] = 12\ t / m2$$

$$q2 = \frac{1.15*20.18}{2 * 1.6}\left[1 - \frac{6*.192}{2}\right] = 3.075\ t / m2$$

Check : (Fmax / Fmin) = (12 / 3.075) = 3.9 < 4 For Crane Footing

take tp.c = .3 m

$\boxed{\text{Dim Of P.C} = 2 * 1.6 * .3}$

Reinforced Concrete

x = t = .3 m

A1 = A - 2x = 1.4 m

B1 = B - 2x = 1 m

M1 = x * (df - tp.c) = 2.97 * 1.2 = 3.56 t.m

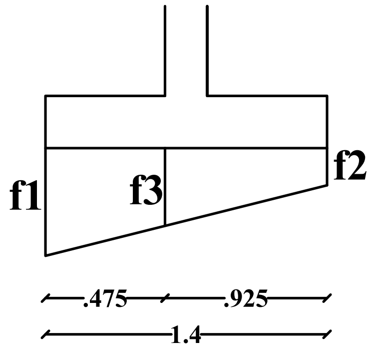
Pg.s = 20.18 t

$$e1 = \frac{M1}{Pg.s} = \frac{3.56}{20.18} = .176\ m$$

$$f1 = \frac{Pg.s}{A1*B1}\left[1 + \frac{6\ e1}{A}\right] = \frac{1.5 * 20.18}{1.4 * 1}\left[1 + \frac{6*.176}{1.4}\right] = 37.93\ t / m2$$

$$f2 = \frac{Pg.s}{A1*B1}\left[1 - \frac{6\,e1}{A}\right] = \frac{1.5 * 20.18}{1.4 * 1}\left[1 - \frac{6*.176}{1.4}\right] = 5.313\ t / m2$$

$$f3 = 5.313 + \frac{.925}{1.4}\left[37.93 - 5.313\right] = 26.86\ t / cm2$$



$$M_{I} = 26.86 * \left[\frac{1.4 - .45}{1.4}\right]^2 + (37.93 - 26.86) * \left[\frac{1.4 - .45}{1.4}\right]^2 * \frac{2}{3} = 7.73\ t.m/m`$$

$$M_{II} = \left[\frac{37.93 + 5.313}{2}\right] * \left[\frac{1 - .4}{2}\right]^2 = 1.95\ t.m/m`$$

$$d = c1\sqrt{\frac{M_{I}}{Fcu * b}} = 5 * \sqrt{\frac{7.73 * 10^5}{300 * 100}} = 35.38\ cm$$

d = 40 cm t = 45 cm

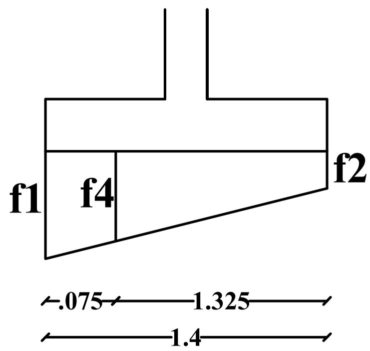
$$\boxed{\text{Dim Of R.C} = 1.4 * 1 * .45}$$

Check of shear

$$f4 = 5.313 + \frac{1.325}{1.4}\left[37.93 - 5.313\right] = 36.183\ t / cm2$$

$$Qs_{I} = \left[\frac{37.93 + 36.18}{2}\right] * .075 = 2.78\ t$$

$$Qs_{II} = \left[\frac{37.93 + 5.313}{2}\right] * \left[\frac{1 - .4}{2} - .4\right] = -ve$$



$$qcu = .75*\sqrt{\frac{Fcu}{1.5}} = 10.61\ Kg / cm2$$

$$qsu = \frac{Qs_{I}}{b\,d} = \frac{2.78 * 10^3}{100 * 40} = .695\ Kg / cm2 < 10.61$$

Safe O.K .

Check of punching

$$Qp = 1.5 * 20.18 - \left[\frac{37.93 + 5.31}{2}\right] * ((.45+.4)*(.4+.4)) = 15.57t$$

$$Ap = .4 * 2 * ((.4+.4)+(.45+.4)) = 1.32 \text{ m2}$$

$$qp = \frac{Qp}{Ap} = \frac{15.57 * 10^3}{1.32 * 10^4} = 1.18 \text{ Kg / cm2}$$

$$qcup = (.5 + a/b)*\sqrt{\frac{Fcu}{1.5}} = 1.39 *\sqrt{\frac{Fcu}{1.5}} \ngtr \sqrt{\frac{Fcu}{1.5}}$$

$$qcup = \sqrt{\frac{Fcu}{1.5}} = 14.14 \text{ kg/ cm2}$$

$qp < qcup$

Safe O.K .

Footing Reinforcement

$$As_{I} = \frac{7.73 * 10^5}{3600 * .826 * 40} = 6.50 \text{ cm2 / m`}$$

$$As_{II} = \frac{1.95 * 10^5}{3600 * .826 * 40} = 1.64 \text{ cm2 / m`}$$

As min = .15 % * b * t = (.15 / 100) * 100 * 45 = 6.75 cm2 / m`

$\boxed{\text{Use 6 Ø 12 / m` in both direction}}$

Design Of Foundation

b - End Gable Column

qall = 12 t / m2 Foundation Level = 1.5 m

Use Rc Column With Hight = 1.5 + .25 = 1.75 m

N = 1.17 t , X = .98 t

Column Section = (24+6)* (14+6) = 30 * 20

Plain Concrete

$$\text{Area} = A * B = \frac{1.5\ Pg.s}{qa} = \frac{1.5 * 1.17}{12} = .15\ m2$$

take A = 1.6 m , B = 1.6 m , t = .3 m

Reinforced Concrete

x = t = .3 m

A1 = 1 m , B = 1 m , t = .3 m

Footing Reinforcement

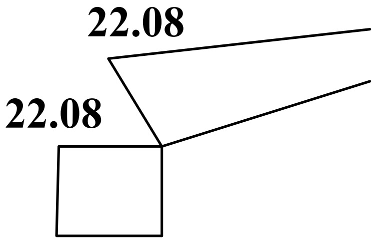
As min = .15 % * b * t = (.15 / 100) * 100 * 45 = 6.75 cm2 / m

Use 6 Ø 12 / m` in both direction

Connections

1 - connection of frame

a - beam to column connection

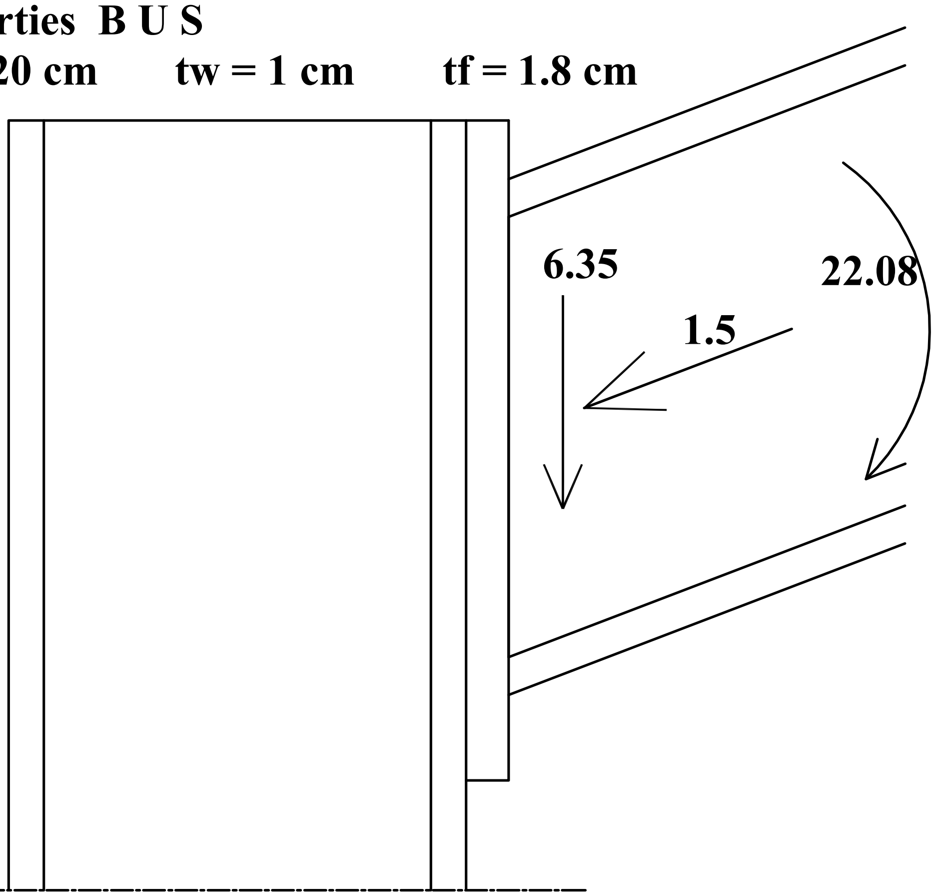


*** Column section properties H E A≠360**

h = 35 cm b = 30 cm tw = 1 cm tf = 1.75 cm

*** Beam section properties B U S**

h = 46.4 cm b = 20 cm tw = 1 cm tf = 1.8 cm

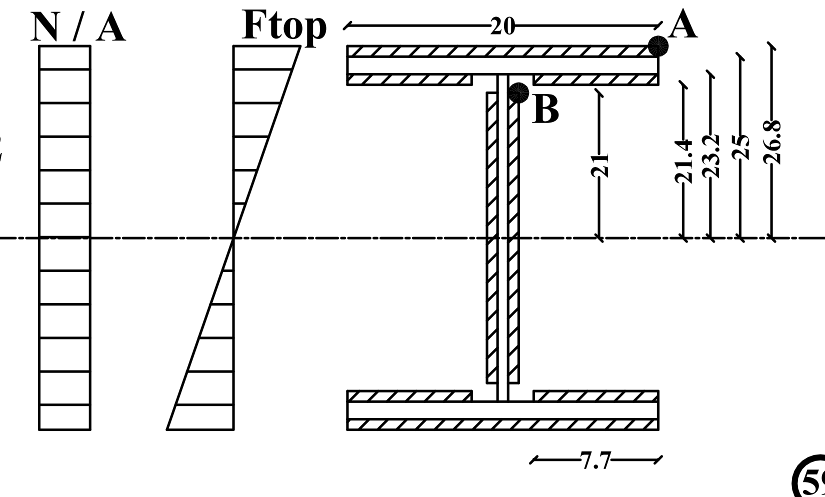


Use Sweb weld= 1cm

Use Sfl. weld= 1.8cm

take Sw = 1cm

Aweld = 2*20*1.8+4*7.7*1.8 + 2*42*1 = 211.44 cm2



Iweld=2*20*1.8*25.9^2+4*7.7*1.8*22.3^2+2*(1*42^3/12)=88216.08cm4

Check of weld at point A

$$\text{Fweld} = \frac{-N}{A} + \frac{Mx}{Ix} * y$$

$$\text{Fweld} = \frac{-1.5}{211.44} + \frac{22.08 * 100}{88216.08} * 26.8 \quad = -.0078+.671=.664 t/cm2$$

<.2Fu=.72t/cm2

Safe O.K .

Check of weld at point B

$$\text{Fweld} = \frac{-N}{A} + \frac{Mx}{Ix} * y$$

$$\text{Fweld} = \frac{-1.5}{211.44} + \frac{22.08 * 100}{88216.08} * 21 \quad = -.0078+.526=.519 t/cm2$$

<.2Fu=.72t/cm2

Safe O.K .

Aweb weld = 2 * 42 * 1 = 84 cm2

$$\text{qweld} = \frac{6.35}{84} = .076 \text{ t / cm2}$$

$$\text{Feq} = \sqrt{\text{Fweld}^2 + \text{qweld}^2} \quad = .525 \text{ t / cm2} \quad <1.1 * .2Fu = .792 \text{ t/cm2}$$

Safe O.K .

<u>Check of weld of flange</u>

$$T = \frac{M}{d} + \frac{N}{2} = \frac{2208}{48.2} + \frac{-1.5}{2} \quad = 45.06 \text{ t}$$

Aflange weld = 20*1.8 + 2 * 7.7 * 1.8 = 63.72 cm2

$$\text{Fweld} = \frac{45.06}{63.72} = .71 \text{ t/ cm2} < .2 \text{ Fu} = .72 \text{ t / cm2}$$

Safe O.K .

*** Check of weld for case that M = 13.57 m.t in the opposite direction**

$$T = \frac{M}{d} + \frac{N}{2} = \frac{1357}{48.2} + \frac{-1.5}{2} = 27.4 \text{ t}$$

Check of weld of flange

Aflange weld = 20*1.8 + 2 * 7.7 * 1.8 = 63.72 cm2

$$F_{weld} = \frac{27.4}{63.72} = .43 \text{ t/ cm2} < .2 \text{ Fu} = .72 \text{ t / cm2}$$

Safe O.K .

*** other checks will be safe**

Design of high strength bolts

*** Use M22 Grade 10.9**

$$T = \frac{M}{d} + \frac{N}{2} = \frac{2208}{48.2} + \frac{-1.5}{2} = 45.06\ t$$

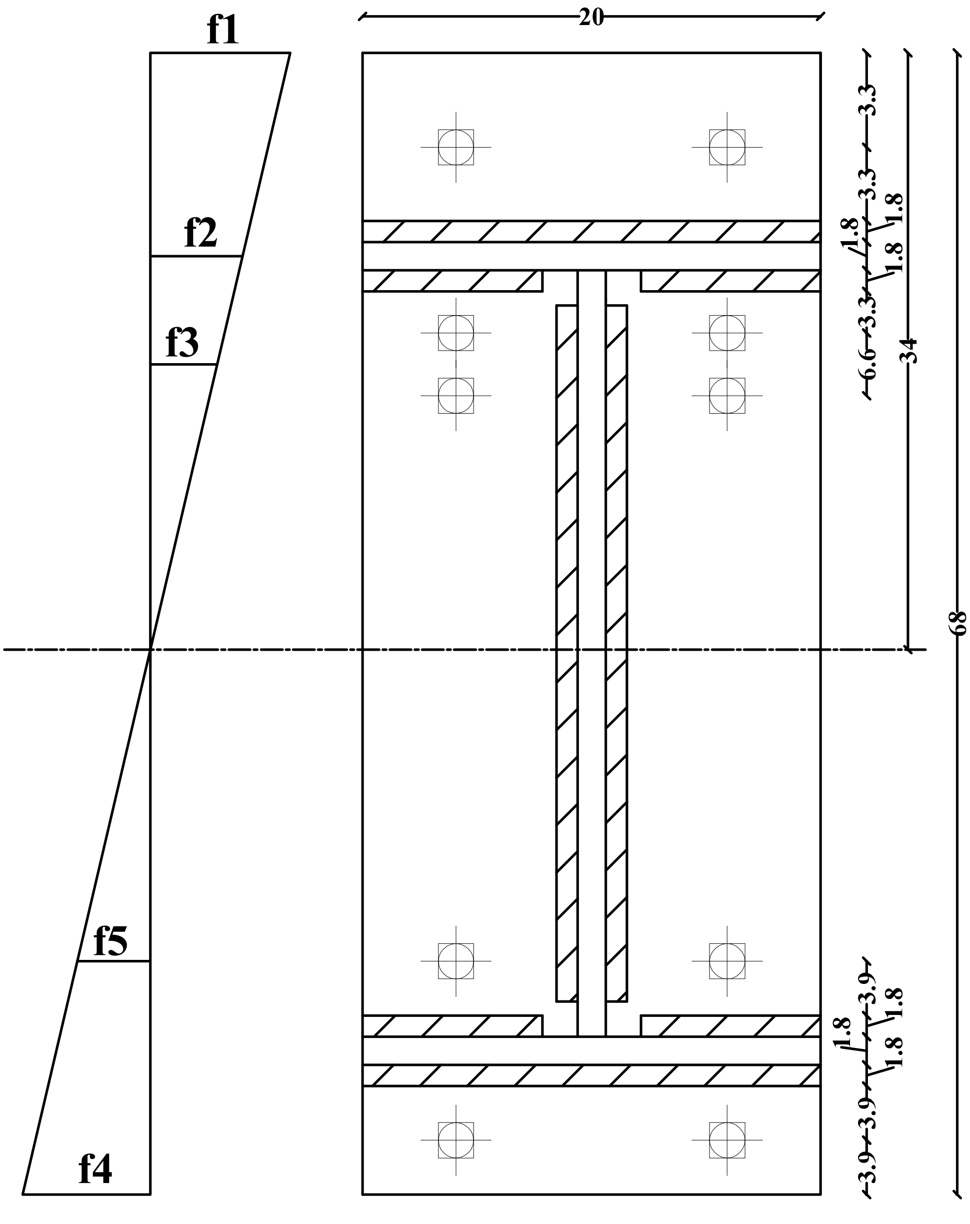
From Table $\quad T = 19.08\ t \quad ps = 7.27\ t$

$$T_{ext,b,m} + T_{ext,b} + P < .8\ T$$

$$T_{ext,b,m} + T_{ext,b} < .7\ T$$

$$n \geqslant \frac{45.06}{.7 * 19.08} \geqslant 3.37\ \text{bolt}$$

Use 4 M 22 (2 bolts * 2 rows)



$$Ix = \frac{20*68^3}{12} = 524053.33 \text{ cm4}$$

$$f1 = \frac{22.08 * 100 * 34}{524053.33} = .143 \text{ t / cm2}$$

$$f2 = \frac{22.08 * 100 * 24.7}{524053.33} = .104 \text{ t / cm2}$$

$$Fn = \frac{N}{A} = \frac{3.92}{20 * 68} = .0029 \text{ t / cm2}$$

$$Text,b,m = \frac{f1 + f2}{2} * 9.3 * 20 * \frac{1}{2} = 11.49 \text{ t}$$

$$Text,b = \frac{1.5}{10} = -.15 \text{ t / cm2}$$

Text,b,m + Text,b + P < .8 T

11.49 - .15 + P < .8 T= .8 * 19.08 = 15.26

safe under prying foce P < 3.92 t

---

* row 2

$$f3 = \frac{22.08 * 100 * 15.4}{524053.33} = .065 \text{ t / cm2}$$

$$Text,b,m = \frac{f2 + f3}{2} * 9.3 * 20 * \frac{1}{2} = 7.86 \text{ t } < Text,b,m \text{ of row 1}$$

---

* row 3

$$Text,b,m = \frac{f3}{2} * 15.4 * 20 * \frac{1}{2} = 5.01 \text{ t } < Text,b,m \text{ of row 1}$$

Check of shear

Qb < Ps (1 - Tex,b / T) $\longrightarrow$ N = -ve $\longrightarrow$ Qb < Ps

$$Qb = \frac{6.35}{10} = .64 \text{ t} < 7.27 \text{ t}$$

Safe O.K .

Design of End plate

L = 3.3 + 3*1.8 + 3.3 = 12 cm   T = 45.06 t

$$M = \frac{T\,L}{8} = \frac{45.06 * 12}{8} = 68.56 \text{ t.cm}$$

$$tep = \sqrt{\frac{6\,M}{b\,Fall}} = \sqrt{\frac{6 * 67.56}{20 * 1.4}} = 3.804 \text{ cm}$$

Use tep = 3.8 cm

Check of prying force

tep = 3.8 cm , w = 10 cm , a = 3.3 cm

b = 3.3 cm , As = 3.03 cm2

$$Text,b,m = \frac{(22.08 * 100) / 48.2}{6} = 7.63 \text{ t}$$

$$P = \left[\frac{\frac{1}{2} - \left(\frac{w\ tp\ \hat{}4}{30\ a\ As\ b\hat{}2}\right)}{\left(\frac{4a}{3b}\right)\left(\frac{a}{4b} + 1\right) + \left(\frac{w\ tp\ \hat{}4}{30\ a\ As\ b\hat{}2}\right)} \right] * Text,b,m$$

P = -.08 * 7.63 = -ve t

Safe O.K .

$\boxed{\text{Use Plate 680 * 200 * 38}}$

Check of crippling of column

L = tfb + 2 tep + 5K   where K = 2 tfc

L = 1.8 + 2 * 3.8 + 5 * 2 * 1.75 = 26.9 cm

Risisting area = L * twc = 26.9 * 1 = 26.9 cm2

Applied area = bfb * tfb = 20 * 1.8 = 36 cm2

Risisting area < Applied area   not safe

Use Stiffners

bst = .5 * (bfc - twc) = 14.5 cm

2bst tst = Applied area - Risisting area **tst = .31 cm**

check of local buckling bst / tst = 14.5 / .32 = 45.3 > 25 / $\sqrt{Fy}$ = 16.13

not safe

take tst = 1.2 cm

bst / tst = 14.5 / 1.2 = 12.08 > 25 / $\sqrt{Fy}$ = 16.13

Safe O.K .

<u>Check of column flange</u>

tfc = 1.75 cm < .4 $\sqrt{bb^{*}\ tb}$ = 2.4 cm

not safe

Use two Stiffners as before

<u>Check of distorsion</u>

$$twc = 1\ cm < \frac{M / db}{.35 * Fy * hc} = 1.6\ cm$$

Use doupler plate

$$t\ of\ one\ plate = \frac{\dfrac{M / db}{.35 * Fy * hc} - twc}{2}$$

t of one plate = .3 cm

*** Check of bolts for case that M = 13.57 m.t in the opposite direction**

$$f4 = \frac{13.57 * 100 * 34}{524053.33} = .088 \ t / cm2$$

$$f5 = \frac{13.57 * 100 * 23.5}{524053.33} = .061 \ t / cm2$$

$$Fn = \frac{N}{A} = \frac{3.92}{20 * 68} = .0029 \ t / cm2$$

$$Text,b,m = \frac{f4 + f5}{2} * 10.5 * 20 * \frac{1}{2} = 7.8 \ t$$

*** row 2**

$$Text,b,m = \frac{f5}{2} * 23.5 * 20 * \frac{1}{2} = 7.17 \ t$$

$$Text,b = \frac{1.5}{10} = -.15 \ t / cm2$$

Text,b,m + Text,b + P < .8 T

7.8 - .15 + P < .8 T= .8 * 19.08 = 15.26

safe under prying foce P < 8.24 t

Check of prying force

tep = 3.8 cm , w = 10 cm , a = 3.9cm

b = 3.9cm , As = 3.03 cm2

$$Text,b,m = \frac{(13.57 * 100) / 48.2}{4} = 7.04 \ t$$

$$P = \left[\frac{\frac{1}{2} - \left(\frac{w \ tp \ \hat{} 4}{30 \ a \ As \ b \hat{} 2}\right)}{\left(\frac{4a}{3b}\right)\left(\frac{a}{4b} + 1\right) + \left(\frac{w \ tp \ \hat{} 4}{30 \ a \ As \ b \hat{} 2}\right)} \right] * Text,b,m$$

P = .086 * 7.04 = .6 t < 8.24 t

Safe O.K .

Check of crippling of column

L = tfb + 2 tep + 5K   where K = 2 tfc

L = 1.8 + 2 * 3.8 + 5 * 2 * 1.75 = 26.9 cm

Risisting area = L * twc = 26.9 * 1 = 26.9 cm2

Applied area = bfb * tfb = 20 * 1.8 = 36 cm2

Risisting area < Applied area   not safe

Use Stiffners

bst = .5 * (bfc - twc) = 14.5 cm

2bst tst = Applied area - Risisting area   tst = .31 cm

check of local buckling bst / tst = 14.5 / .32 = 45.3 > 25 / $\sqrt{Fy}$ = 16.13

not safe

take tst = 1.2 cm

bst / tst = 14.5 / 1.2 = 12.08 > 25 / $\sqrt{Fy}$ = 16.13

Safe O.K .

Check of column flange

tfc = 1.75 cm < .4 $\sqrt{bb * tb}$ = 2.4 cm

not safe

Use two Stiffners as before

Check of distorsion

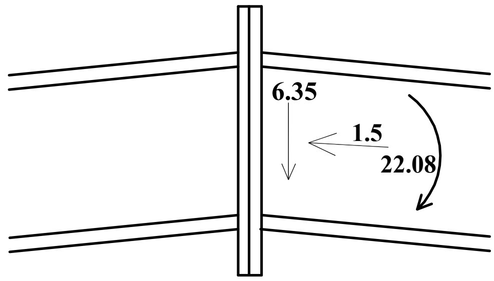
$$twc = 1\ cm > \frac{M / db}{.35 * Fy * hc} = .96\ cm$$

Safe O.K .

b - beam to beam connection

*** Beam section properties B U S**

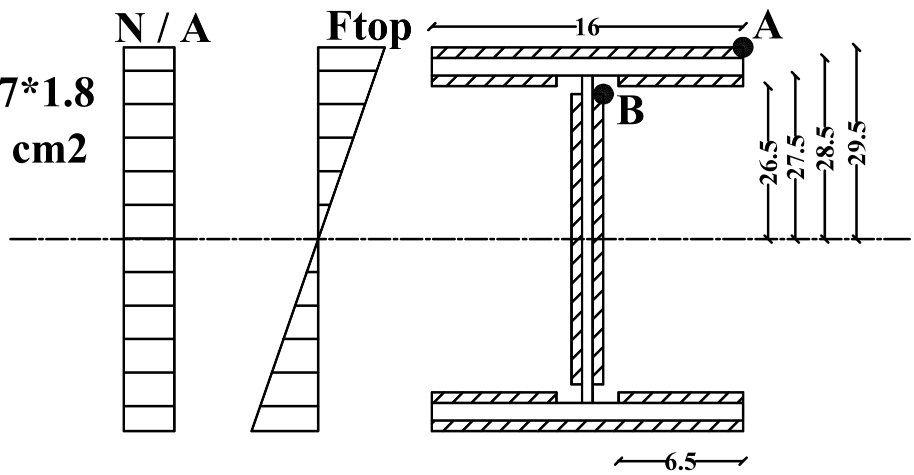
h = 55 cm b = 16 cm tw = 1 cm tf = 1 cm



Use Sweb weld= 1cm

Use Sfl. weld= 1.8cm

Aweld = 2*20*1.8 + 4*7.7*1.8 + 2*22*1 = 171.44 cm2



Iweld = 2*20*1.8*15.9^2+4*7.7*1.8*12.3^2 + 2*(1*22^3/12) =28364.5cm4

Check of weld at point A

$$Fweld = \frac{-N}{A} + \frac{Mx}{Ix} * y$$

$$Fweld = \frac{-2.92}{171.44} + \frac{12.34 * 100}{28364.5} * 16.8 = -.017+..73=.714 \text{ t/cm2}$$

<.2Fu=.72t/cm2

Safe O.K .

Check of weld at point B

$$Fweld = \frac{-N}{A} + \frac{Mx}{Ix} * y$$

$$Fweld = \frac{-2.92}{171.44} + \frac{12.34 * 100}{28364.5} * 11 = -.017+..479=.46 \text{ t/cm2}$$

<.2Fu=.72t/cm2

Safe O.K .

Aweb weld = 2 * 22 * 1 = 44 cm2

qweld $= \dfrac{.66}{44} = .015 \text{ t / cm2}$

Feq $= \sqrt{\text{Fweld}^2 + \text{qweld}^2} = .46 \text{ t / cm2} < 1.1 * .2\text{Fu} = .792 \text{ t/cm2}$

Safe O.K .

<u>Check of weld of flange</u>

$$T = \frac{M}{d} + \frac{N}{2} = \frac{1234}{28.2} + \frac{-2.92}{2} = 42.3 \text{ t}$$

Aflange weld = 20*1.8 + 2 * 7.7 *1.8 = 63.72 cm2

Fweld $= \dfrac{42.3}{63.72} = .664 \text{ t/ cm2} < .2 \text{ Fu} = .72 \text{ t / cm2}$

Safe O.K .

Design of high strength bolts

*** Use M22 Grade 10.9**

$$T = \frac{M}{d} + \frac{N}{2} = \frac{1234}{28.2} + \frac{-2.92}{2} = 42.3\ t$$

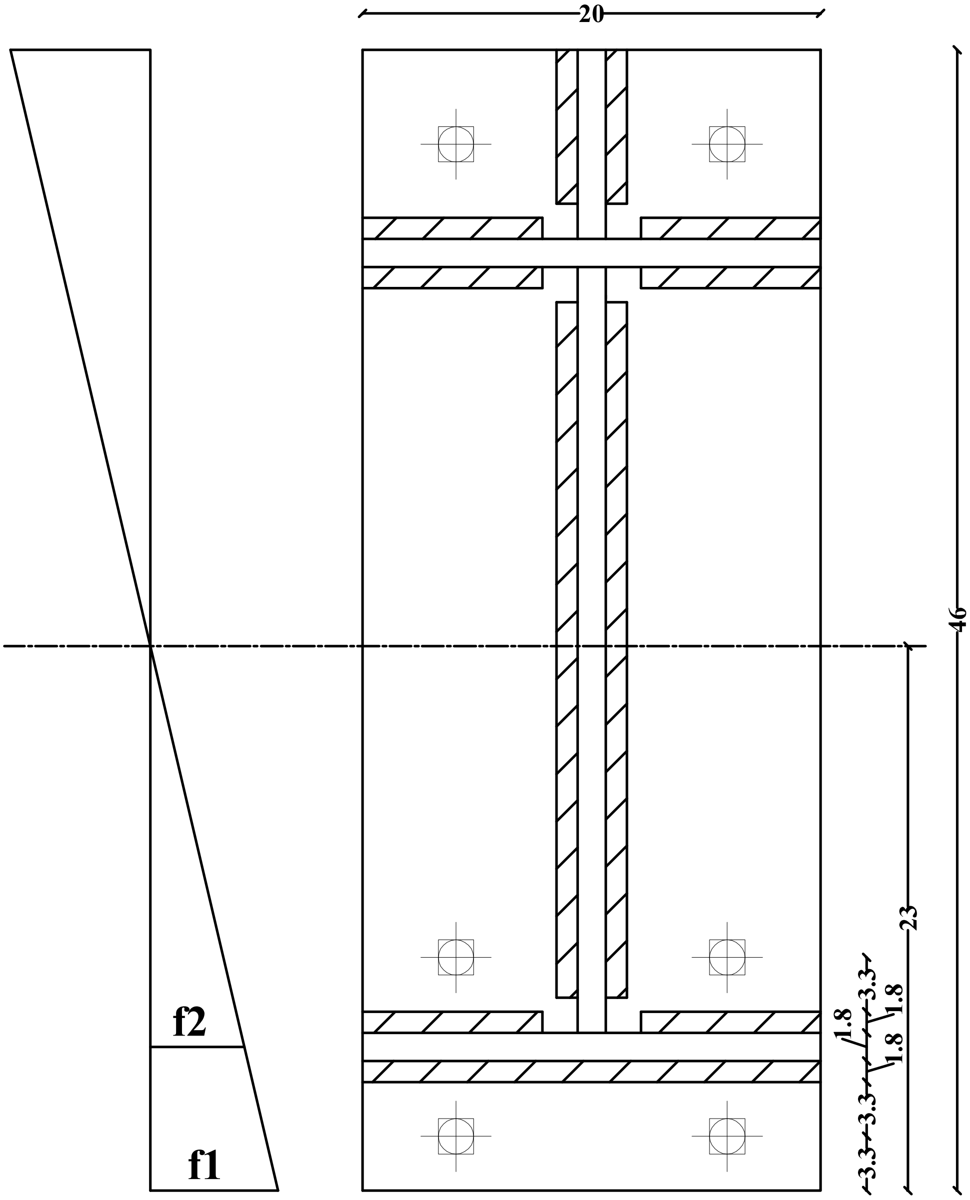
From Table $T = 19.08\ t$ $ps = 7.27\ t$

Text,b,m + Text,b + P < .8 T

Text,b,m + Text,b < .7 T

$$n \geqslant \frac{42.3}{.7 * 19.08} \geqslant 3.2 \text{ bolt}$$

Use 4 M 22 (2 bolts * 2 rows)



$$Ix = \frac{20 * 46^3}{12} = 162226.67 \text{ cm4}$$

$$f1 = \frac{12.34 * 100 * 23}{162226.67} = .175 \text{ t / cm2}$$

$$f2 = \frac{12.34 * 100 * 13.7}{162226.67} = .104 \text{ t / cm2}$$

$$Text,b,m = \frac{f1 + f2}{2} * 9.3 * 20 * \frac{1}{2} = 12.97 \text{ t}$$

*** row 2**

$$Text,b,m = \frac{f2}{2} * 13.7 * 20 * \frac{1}{2} = 7.124 \text{ t} < Text,b,m \text{ of row 1}$$

$$Fn = \frac{N}{A} = \frac{3}{16 * 70} = .0027 \text{ t / cm2}$$

$$Text,b = \frac{2.92}{6} = -.487 \text{ t / cm2}$$

$Text,b,m + Text,b + P < .8\ T$

$12.97 - .487 + P < .8\ T = .8 * 19.08 = 15.264$

safe under prying foce P < 2.78 t

Check of shear

$Qb < Ps\ (1 - Tex,b / T) \longrightarrow N = -ve \longrightarrow Qb < Ps$

$$Qb = \frac{.66}{6} = .11 \text{ t} < 7.27 \text{ t}$$

Safe O.K .

Design of End plate

$L = 3.3 + 3*1.8 + 3.3 = 12$ cm $\qquad T = 42.3t$

$$M = \frac{T\ L}{8} = \frac{42.3 * 12}{8} = 63.45 \text{ t.cm}$$

$$tep = \sqrt{\frac{6\ M}{b\ Fall}} = \sqrt{\frac{6 * 63.45}{20 * 1.4}} = 3.68 \text{ cm}$$

Use tep = 3.8 cm

Check of prying force

tep = 3.8 m , w = 10 cm , a = 3.3 cm

b = 3.3 cm , As = 3.03 cm2

$$\text{Text,b,m} = \frac{(12.34 * 100) / 28.2}{4} = 10.94 \text{ t}$$

$$P = \left[\frac{\frac{1}{2} - \left(\frac{w\ tp\ \hat{}4}{30\ a\ As\ b\hat{}2}\right)}{\left(\frac{4a}{3b}\right)\left(\frac{a}{4b} + 1\right) + \left(\frac{w\ tp\ \hat{}4}{30\ a\ As\ b\hat{}2}\right)} \right] * \text{Text,b,m}$$

P = -.088 * 10.94 = -ve

Safe O.K .

$\boxed{\textbf{Use Plate 460 * 200 * 38}}$

2 - Design Of Bracket - Column Connection

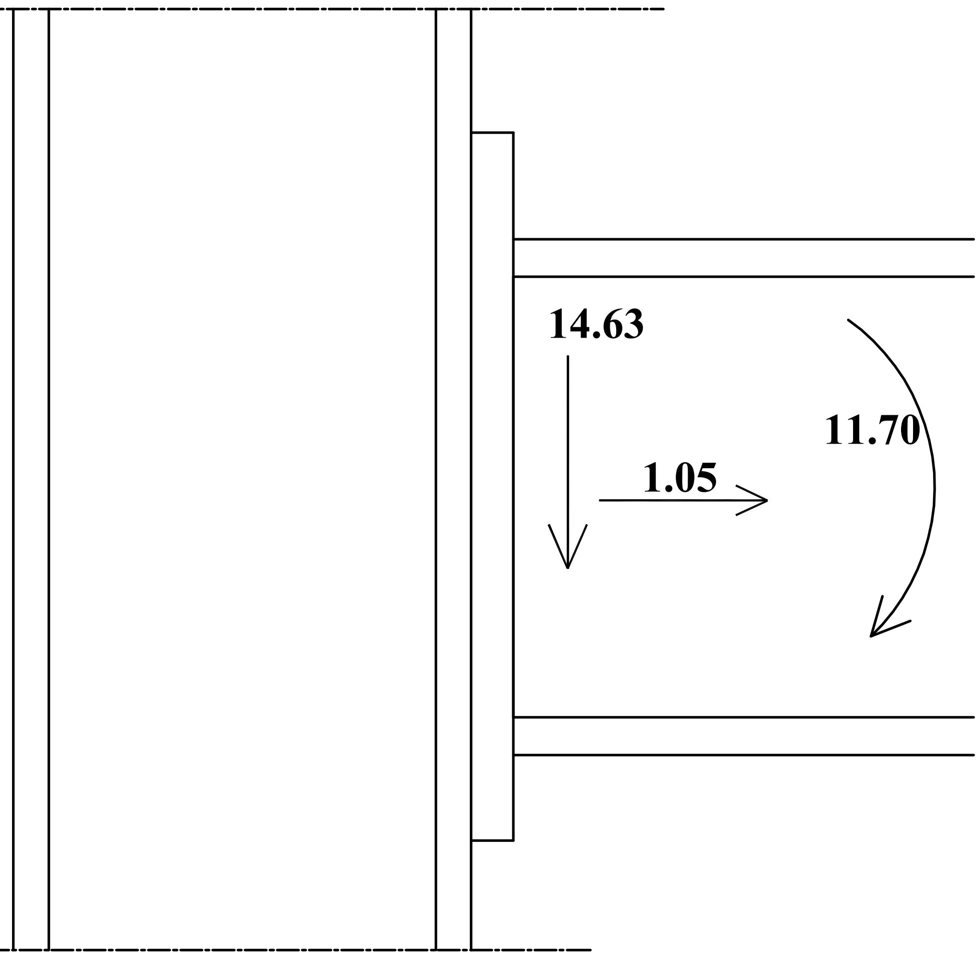
*** Column section properties H E A≠360**

h = 35 cm   b = 30 cm   tw = 1 cm   tf = 1.75 cm

*** Beam section properties I P E≠360**

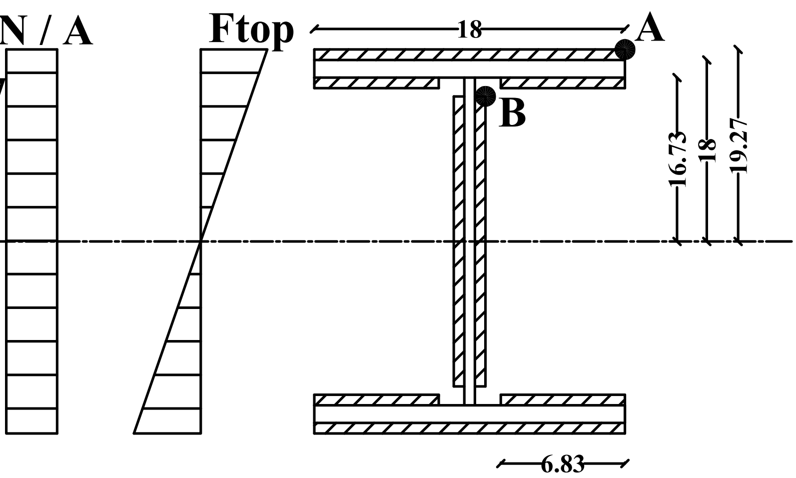
h = 36 cm   b = 17 cm   tw = .8 cm   tf = 1.27 cm

Use Sw fl = 1.27cm
Use Sw web=.86cm



take Sw = 1cm

Aweld = 2*17*1.27+4*6.83*1.27
+ 2*30.92*.86 = 127.35 cm2



Iweld = 2*17*1.27*18.64^2 + 4*6.83*1.27*16.1^2 + 2*(.86*30.92^3/12)
=27937.99cm4

Check of weld at point A

$$Fweld = \frac{N}{A} + \frac{Mx}{Ix} * y$$

$$Fweld = \frac{1.05}{127.35} + \frac{11.70 * 100}{27937.99} * 19.27 = .0085+..807=.816 t/cm2$$

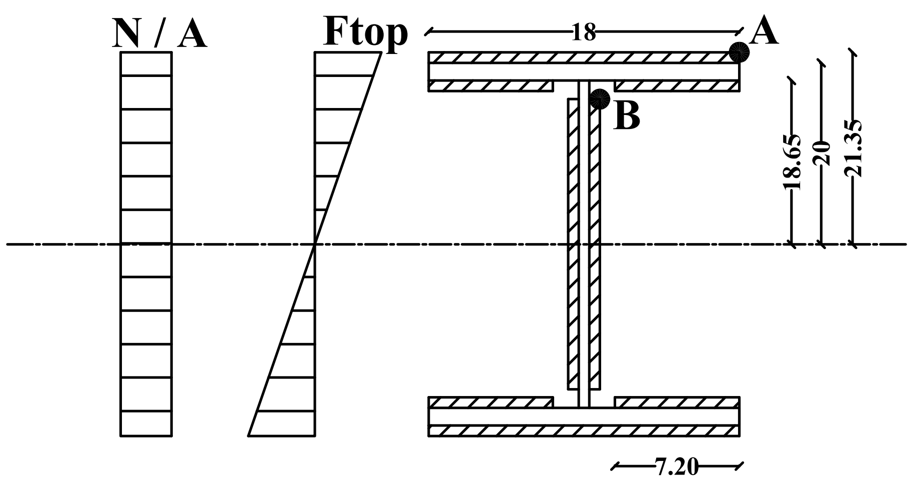
>.2Fu=.72t/cm2
not safe

Use I P E≠ 400 For Bracket

* Beam section properties I P E≠400

h = 40 cm b = 18 cm tw = .86 cm tf = 1.35 cm

Use Sw fl = 1.35cm
Use Sw web=.86cm



Aweld = 2*18*1.35+4*7.2*1.35+ 2*34.6*.86 = 146.992 cm2

Iweld = 2*18*1.35*20.68^2 + 4*7.2*1.35*17.98^2 + 2*(.86*34.6^3/12)
= 39273.61 cm4

Check of weld at point A

$$Fweld = \frac{N}{A} + \frac{Mx}{Ix} * y$$

$$Fweld = \frac{1.05}{146.9} + \frac{11.70 * 100}{39273.61} * 31.25 = .0071+.6359=.643 t/cm2$$

<.2Fu=.72t/cm2

Safe O.K .

<u>Check of weld at point B</u>

$$Fweld = \frac{N}{A} + \frac{Mx}{Ix} * y$$

$$Fweld = \frac{1.05}{146.9} + \frac{11.70 * 100}{39273.61} * 17.3 = .0071 + .515 = .522 \ t/cm2$$

$< .2Fu = .72 t/cm2$

Safe O.K .

Aweb weld = 2 * 34.6 * .86 = 59.512 cm2

$$qweld = \frac{14.63}{59.512} = .2458 \ t / cm2$$

$$Feq = \sqrt{Fweld^2 + qweld^2} = .577 \ t / cm2 \quad < 1.1 * .2Fu = .792 \ t/cm2$$

Safe O.K .

<u>Check of weld of flange</u>

$$T = \frac{M}{d} + \frac{N}{2} = \frac{1170}{38.65} + \frac{1.05}{2} = 30.797 \ t$$

Aflange weld = 18*1.35 + 2 * 7.2 * 1.35 = 43.74 cm2

$$Fweld = \frac{30.797}{43.74} = .704 \ t/ cm2 < .2 \ Fu = .72 \ t / cm2$$

Safe O.K .

Design of high strength bolts

*** Use M22 Grade 10.9**

$$T = \frac{M}{d} + \frac{N}{2} = \frac{1170}{38.65} + \frac{1.05}{2} = 30.797 \text{ t}$$

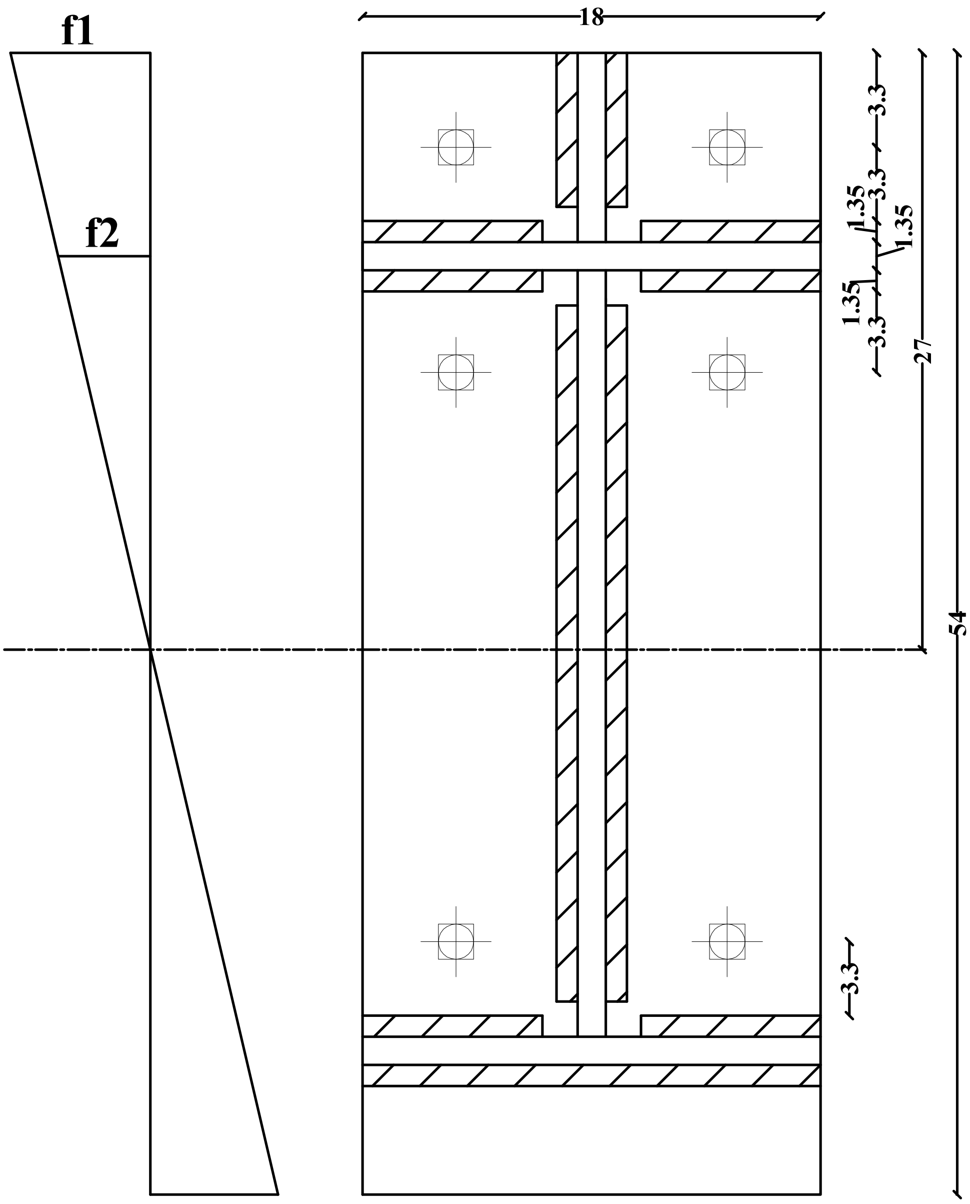
From Table $T = 19.08$ t $\quad ps = 7.27$ t

Text,b,m + Text,b + P < .8 T

Text,b,m + Text,b < .7 T

$$n \geqslant \frac{30.797}{.7 * 19.08} \geqslant 2.31 \text{ bolt}$$

Use 4 M 22 (2 bolts * 2 rows)



$$Ix = \frac{18 * 54^3}{12} = 236196 \text{ cm4}$$

$$f1 = \frac{11.70 * 100 * 27}{236196} = .134 \text{ t / cm2}$$

$$f2 = \frac{11.70 * 100 * 18.37}{236196} = .082 \text{ t / cm2}$$

$$Text,b,m = \frac{f1 + f2}{2} * 8.625 * 16 * \frac{1}{2} = 8.38 \text{ t}$$

$$Fn = \frac{N}{A} = \frac{1.05}{18 * 54} = .0011 \text{ t / cm2}$$

$$Text,b = \frac{1.05}{6} = .175 \text{ t / cm2}$$

Text,b,m + Text,b + P < .8 T

8.38 + .175 + P < .8 T= .8 * 19.08 = 15.26

safe under prying foce P < 6.705 t

*** row 2**

$$Text,b,m = \frac{f2}{2} * 18.375 * 18 * \frac{1}{2} = 6.78 \text{ t} < Text,b,m \text{ of row 1}$$

Safe O.K .

Check of shear

Qb < Ps (1 - Tex,b / T) ⟶ Tex,b = 1.05 / 6 = .175 t

$$Qb = \frac{14.63}{6} = 2.44 \text{ t} < 7.27 * (1 - (.175 / 19.08)) = 7.20 \text{ t}$$

Safe O.K .

Design of End plate

L =2*3.3 + 3 * 1.35 = 10.65 cm **T = 30.797 t**

$$M = \frac{T\ L}{8} = \frac{30.797 * 10.65}{8} = 40.999 \text{t.cm}$$

$$tep = \sqrt{\frac{6\ M}{b\ Fall}} = \sqrt{\frac{6 * 40.999}{18 * 1.4}} = 3.12 \text{ cm}$$

Use tep = 3.2 cm

Check of prying force

tep = 3.2 m , w = 9 cm , a = 3.3 cm

b = 3.3 cm , As = 3.03 cm2

$$\text{Text,b,m} = \frac{(11.70 * 100) / 38.65}{4} = 7.57\ \text{t}$$

$$P = \left[\frac{\frac{1}{2} - \left(\frac{w\ tp\ \text{^}4}{30\ a\ As\ b\text{^}2}\right)}{\left(\frac{4a}{3b}\right)\left(\frac{a}{4b} + 1\right) + \left(\frac{w\ tp\ \text{^}4}{30\ a\ As\ b\text{^}2}\right)} \right] * \text{Text,b,m}$$

P = .172 * 7.57 = 1.303 t < 6.705 t

Safe O.K .

$\boxed{\text{Use Plate 540 * 180 * 32}}$

Check of crippling of column

L = tfb + 2 tep + 5K where K = 2 tfc

L = 1.35 + 2 * 3.2 + 5 * 2 * 1.75 = 25.25 cm

Risisting area = L * twc = 25.25 * 1 = 25.25 cm2

Applied area = bfb * tfb = 18 * 1.35 = 24.3 cm2

Risisting area > Applied area Safe O.K .

no need to use stiffners

Check of column flange

$\text{tfb} = 1.75\ \text{cm} < .4\sqrt{\text{bb* tb}} = 1.97\ \text{cm}$

not safe

Use minimum stifnners bst = 14.5 cm tst = 1.2cm

Check of distorsion

twc = 1 cm <

$$\frac{M / db}{.35 * Fy * hc} \cong 1.0\ \text{cm}$$

Safe O.K .

3 - Design Of Connection Of Bracing

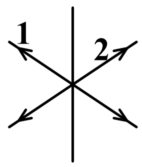
a - HL (Upper) Bracing

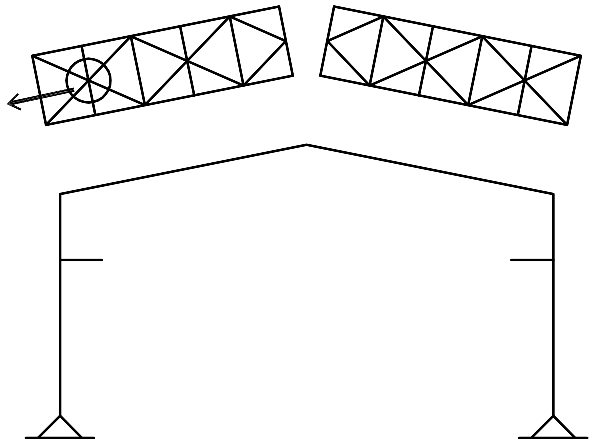
Use M 16 Grade 4.6

Fub = 4 t / cm2 , thickness of plate (t) = 10 mm

*** member 1 D.F = 1.40 t 2Ls 70 * 7 BTB continuous**

*** member 2 D.F = 1.40 t 2Ls 70 * 7 BTB separated**





*** Rsh = 2 *π / 4 *Ø^2 * qb**

Rsh = 2 *π / 4 *1.6^2 * .25 * 4 = 4.02 t

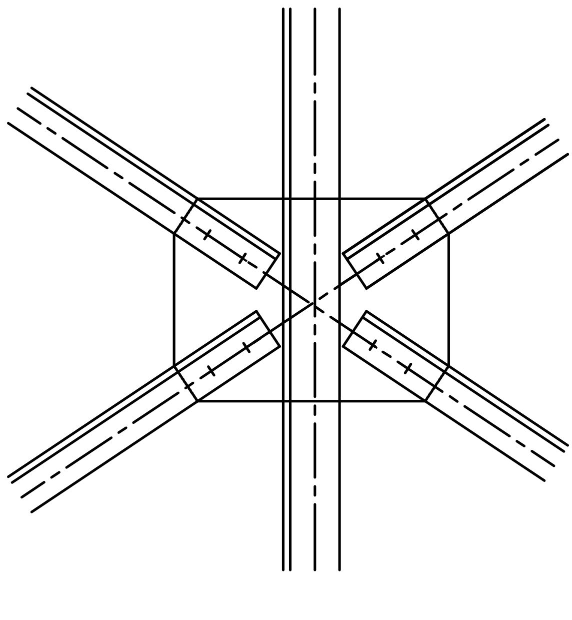
*** Rb = Ø * tmin * Fb Where Fb = æ Fu æ = .6 as e1 = 1.5 Ø**

Rb = 1.6 * .7 * .6 * 3.6 = 2.42 t

Rmin = 2.42 t

$$n\ req. > \frac{D.F}{Rmin} = \frac{1.40}{2.42} = .58\ bolt$$

Use minimum bolts = 2 bolts



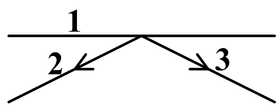
b - VL (Transeversal) Bracing

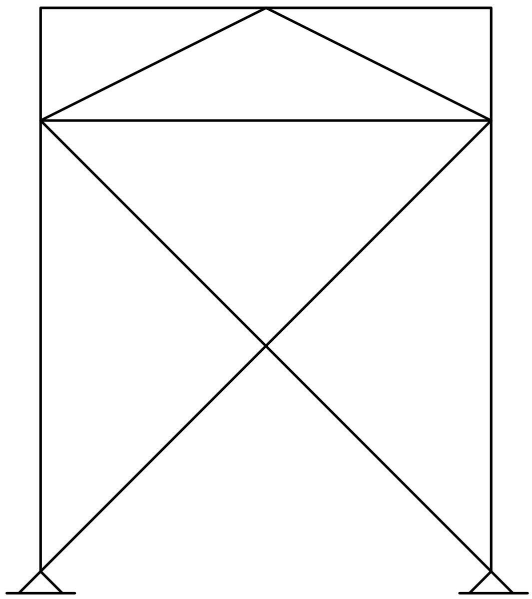
Connection 1

Use M 16 Grade 4.6

Fub = 4 t / cm2 , thickness of plate (t) = 10 mm

* member 1	D.F = 1.95 t	2Ls 60 * 6 BTB	continuous
* member 2	D.F = 1.09 t	2Ls 60 * 6 BTB	separated
* member 3	D.F = 1.09 t	2Ls 60 * 6 BTB	separated





* $R_{sh} = 2 * \pi / 4 * Ø^2 * q_b$

  $R_{sh} = 2 * \pi / 4 * 1.6^2 * .25 * 4 = 4.02$ t

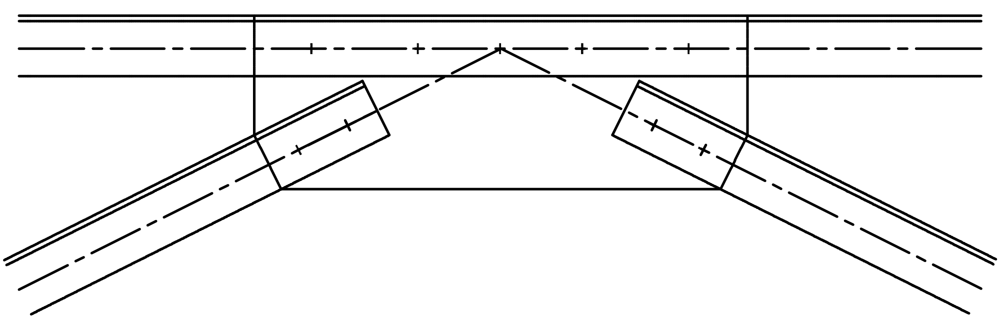
* $R_b = Ø * t_{min} * F_b$ Where $F_b = æ F_u$ æ = .6 as e1 = 1.5

  $R_b = 1.6 * .6 * .6 * 3.6 = 2.07$ t

  $R_{min} = 2.07$ t

$$n\ req. > \frac{D.F}{R_{min}} = \frac{1.95}{2.07} = .94 \text{ bolt}$$

Use minimum bolts = 2 bolts



Connection 2

Use M 16 Grade 4.6

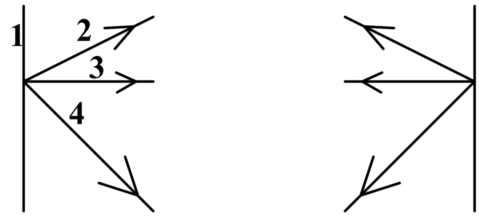
Fub = 4 t / cm2 , thickness of plate (t) = 10 mm

*** member 1 is the column**

*** member 2 D.F = 1.09 t 2Ls 60 * 6 BTB separeted**

*** member 3 D.F = .4 t 2 Cs No 16 BTB separated**

*** member 4 D.F = 2.09 t 2Ls 80 * 8 BTB separated**



*** Rsh = 2 * π / 4 * Ø^2 * qb**

Rsh = 2 * π / 4 * 1.6^2 * .25 * 4 = 4.02 t

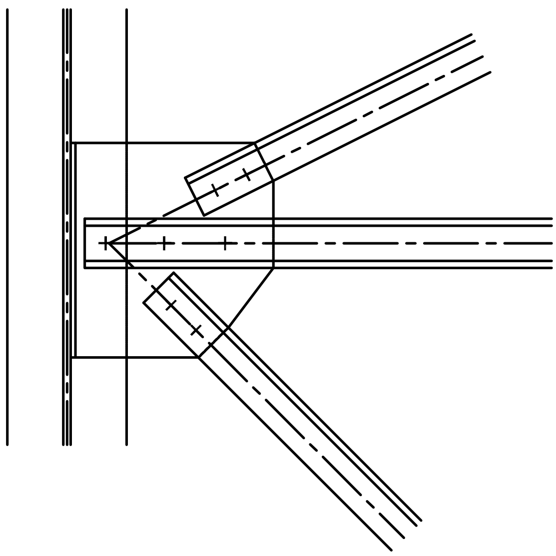
*** Rb = Ø * tmin * Fb Where Fb = æ Fu æ = .6 as e1 = 1.5**

Rb = 1.6 * .6 * .6 * 3.6 = 2.07 t

Rmin = 2.07 t

$$n\ req. > \frac{D.F}{Rmin} = \frac{2.09}{2.07} = 1.01\ bolt$$

Use minimum bolts = 2 bolts



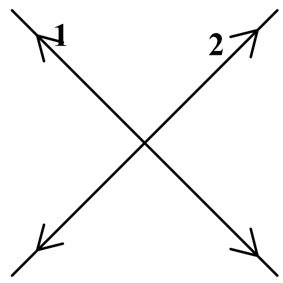
Connection 4

Use M 16 Grade 4.6

Fub = 4 t / cm2 , thickness of plate (t) = 10 mm

*** member 1 D.F = 0.0 t 2Ls 60 * 6 BTB continuous**

*** member 2 D.F = 2.09 t 2Ls 80 * 8 BTB separeted**



*** Rsh = 2 *π / 4 *Ø^2 * qb**

Rsh = 2 *π / 4 *1.6^2 * .25 * 4 = 4.02 t

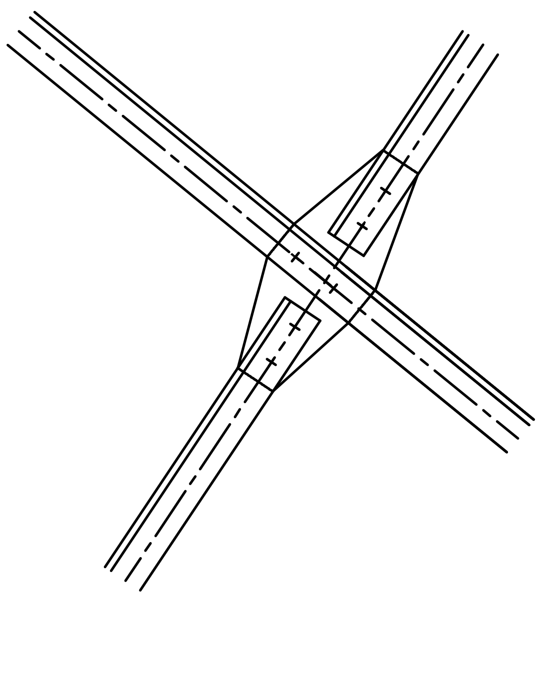
*** Rb = Ø * tmin * Fb Where Fb = æ Fu æ = .6 as e1 = 1.5**

Rb = 1.6 * .6 * .6 * 3.6 = 2.07 t

Rmin = 2.07 t

$$\frac{D\,.\,F}{Rmin} = \frac{2.09}{2.07} = 1.01 \text{ bolt}$$

Use minimum bolts = 2 bolts



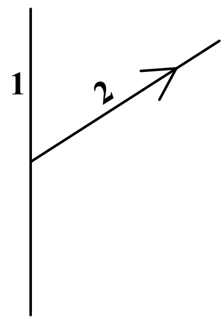
Connection 5

Use M 16 Grade 4.6

Fub = 4 t / cm2 , thickness of plate (t) = 10 mm

*** member 1 is the column**

*** member 2 D.F = 2.09 t 2Ls 80 * 8 BTB separeted**



*** Rsh = 2 *π / 4 *Ø^2 * qb**

Rsh = 2 *π / 4 *1.6^2 * .25 * 4 = 4.02 t

*** Rb = Ø * tmin * Fb Where Fb = æ Fu æ = .6 as e1 = 1.5**

Rb = 1.6 * .8 * .6 * 3.6 = 2.76 t

Rmin = 2.76 t

$$\frac{D.F}{Rmin} = \frac{2.09}{2.76} = .76 \text{ bolt}$$

Use minimum bolts = 2 bolts